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Greater Mangaung Water Augmentation Project

Main Feasibility Study Report

Xhariep Pipeline Feasibility Study

ZUTARI
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Main Feasibility Study Report

APPROVAL

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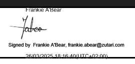
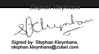
Author signature		Approver signature	
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Report Structure

This report forms part of the following suite for the study:

REPORT INDEX	REPORT NAME	DWS NUMBER
1	Inception Report	P WMA 06/D00/00/3423/1
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3	Stakeholder Management Report	P WMA 06/D00/00/3423/3
4	Data Analysis and Collection Report	P WMA 06/D00/00/3423/4
5	Pre-feasibility Study Report	P WMA 06/D00/00/3423/5
6	**Main Feasibility Study Report**	**P WMA 06/D00/00/3423/6**
7	Geological and Materials Investigations Report	P WMA 06/D00/00/3423/7
8	Topographical Survey and Mapping Report	P WMA 06/D00/00/3423/8
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17	Water Use License Application Summary Report	P WMA 06/D00/00/3423/17
18	Integrated Water and Waste Management Plan	P WMA 06/D00/00/3423/18
19	Water Resources Analysis Report	P WMA 06/D00/00/3423/19

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DWS Report Number: P WMA 06/D00/00/3423/6

Prepared by Zutari (Pty) Ltd

Executive Summary

The conclusions contain a detail description of the detailed feasibility design for the Xhariep Pipeline Project are as such repeated below as the Executive Summary.

The pre-feasibility study concluded that Scheme 1B, as shown in Figure E1, was the optimum configuration to address the water shortages experienced within the Greater Bloemfontein Water Supply System (GBWSS), which includes Bloemfontein, Botshabelo and Thaba Nchu.

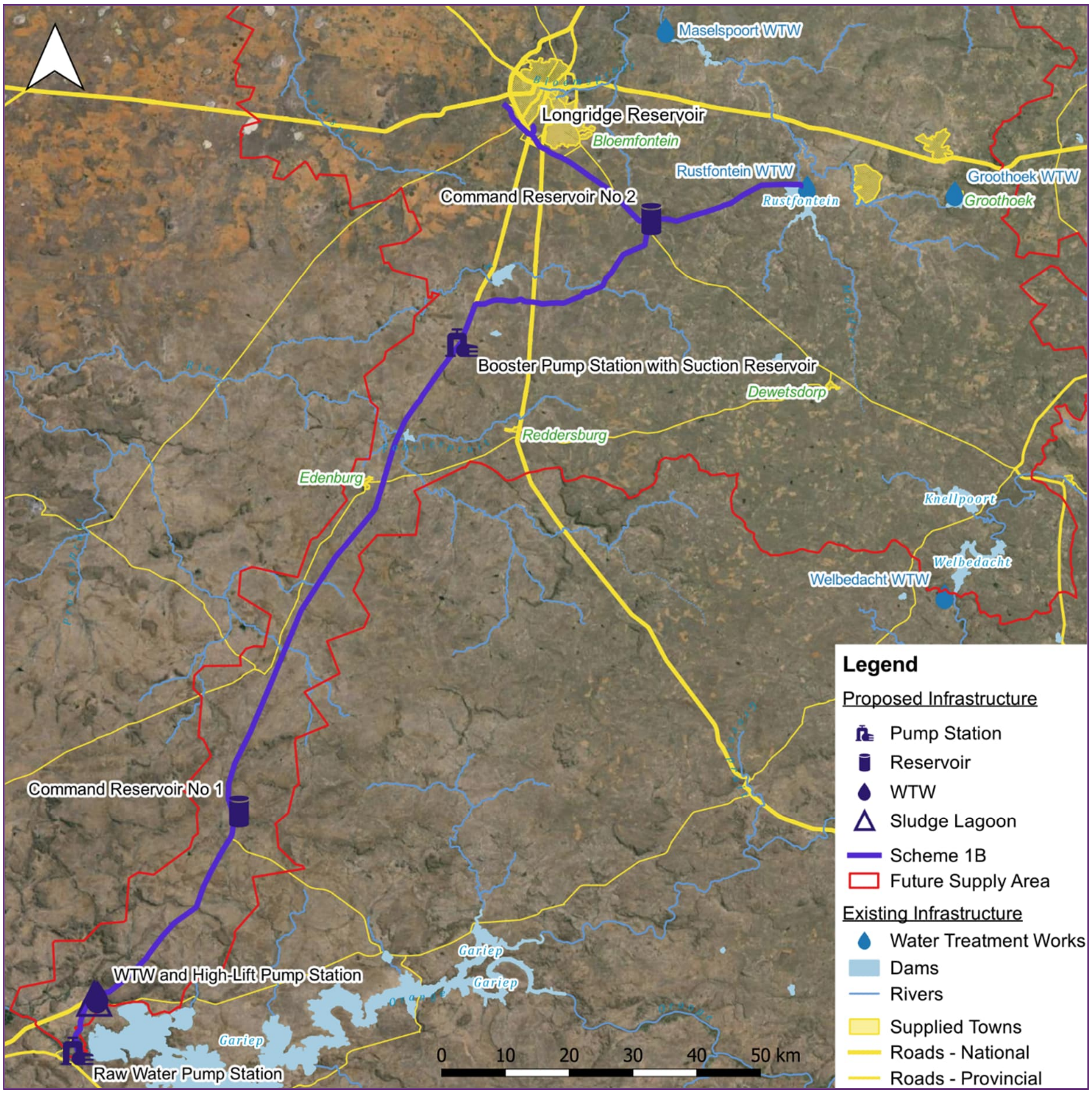

Figure E1: Main infrastructure components of Scheme 1B

A long-term stochastic analysis confirmed that the proposed potable transfer scheme at a capacity of 101 million m³/a is capable of meeting the GBWSS demands at the required assurance of supply until at least the year 2050.

It was determined, from the design flow calculations, that the raw water infrastructure had to be sized for a peak week flow of 3.797 m³/s (329 Mℓ/d), whereas the potable infrastructure, including the water treatment plant (WTP), had to be sized for a peak week flow of 3.616 m³/s (312 Mℓ/d). The two command reservoirs were sized for 6 hours storage at the peak week flow rate of 3.616 m³/s (312 Mℓ/d), equating to a storage capacity of 80 Mℓ per reservoir.

A Light Detection and Ranging (LiDAR) survey was undertaken for the overall study area to undertake the detailed feasibility design and to provide the required topographical data for the detailed design phase of the project. As part of the survey, control points and benchmarks were installed, and digital colour images of the project area were obtained.

A geotechnical field investigation was undertaken for the overall study area. The fieldwork investigation included the excavation of 410 test pits, 106 in-situ Dynamic Cone Penetrometer (DCP) tests, 120 Dynamic Probe Super Heavy (DPSH) tests, rotary core drilling of 44 boreholes, electrical resistivity testing at 200 m intervals along the pipeline routes, as well as the associated laboratory testing.

At the time of undertaking the topographic survey and geotechnical investigation, access to certain privately owned properties was not available and wayleaves from MMM were not received and had to be excluded. The topographical survey and geotechnical investigation of these areas need to be concluded as part of the detailed design phase of the project.

A water quality testing programme, consisting of 12 samples taken over a period from March 2024 to August 2024 (6 months) was undertaken to supplement the raw water quality data available from DWS and VCWB.

The main infrastructure components of Scheme 1B, as shown in Figure E1, include the following:

- Tie-in at the existing DN2100 pipeline downstream of Gariep Dam Wall,
- A pipeline from Gariep Dam to the Raw Water Pump Station (± 2 km long),
- The Raw Water Pump Station,
- A pipeline from the Raw Water Pump Station to a break pressure tank (± 2 km long),
- A pipeline from the break pressure tank to the Xhariep water treatment works (WTP), ± 9 km long,
- The Xhariep WTP, which is designed for a capacity of 312 Mℓ/d of which 208 Mℓ/d will be constructed as Phase 1, with a future 104 Mℓ/d to be constructed later. The site will, however, be planned for an ultimate capacity of 416 Mℓ/d,
- The High Lift Pump Station located at the WTP site, which will pump water to Command Reservoir No 1,
- The pipeline from the High Lift Pump Station to Command Reservoir No 1 (± 43 km long),
- Command Reservoir No 1 (80 Mℓ storage),
- A pipeline from Command Reservoir No 1 to the Booster Pump Station (± 95 km long),
- A Booster Pump Station with Suction Reservoir (10 Mℓ storage),
- A pipeline from the Booster Pump Station to Command Reservoir No 2 (± 44 km long),
- Command Reservoir No 2 (80 Mℓ storage),
- A pipeline from Command Reservoir No 2 to the existing Rustfontein WTP (± 25 km long), and
- A pipeline from Command Reservoir No 2 to the existing Longridge Reservoirs (± 28 km long).

The pipeline diameters of the pumping mains were optimised based on net present value (NPV) calculations that considered capital, maintenance and operational costs. Various sensitivity analyses were undertaken that considered different discount rates, different growth patterns in water demand, different inflation rates for energy costs, etc. The recommended optimum diameters for the pumping mains are:

- Pipeline from the Raw Water Pump Station to a break pressure tank = DN 1800,
- Pipeline from the High Lift Pump Station to Command Reservoir No 1 = DN 1800, and,
- Pipeline from the Booster Pump Station to Command Reservoir No 2 = DN 1800.

The pipeline diameters for the gravity pipelines were determined based on the available head and the design flow rates. The recommended optimum diameters for the gravity pipelines are:

- Pipeline from Gariep Dam to the Raw Water Pump Station = DN 1800,
- Pipeline from the break pressure tank to the Xhariep water treatment works = DN 2000,
- Pipeline from Command Reservoir No 1 to the Booster Pump Station = DN 1800,
- Pipeline from Command Reservoir No 2 to the existing Rustfontein WTP = DN 1400, and,
- Pipeline from Command Reservoir No 2 to the existing Longridge Reservoirs = DN 2000.

The duty points for the three pump stations (i.e. raw water pump station, high-lift pump station and booster pump station) were calculated based on the optimised pipe diameters. The pump types available to achieve the required duty points were evaluated, concluding that horizontal split-casing and vertical turbine pumps were the only pump options that could deliver the required flows and heads. The horizontal split-casing pumps were, however, preferred as they are more economical and easier to operate and maintain.

All three pump stations were designed with a three duty, one standby, pump configuration. Critical aspects such as operating speed, hydraulic efficiency, net positive suction head required, and head rise to shut-off head were evaluated for each pump selection. Details of the selected pumps are summarised in Table E1.

Table E1: Pump selection details

Description	Raw water	High Lift	Booster
Pump duty	3.797 m³/s @ 73m	3.616 m³/s @ 320m	3.616 m³/s @ 127m
Pump Model	SMD 500-750 A	HPDM-450-1000	SMD 600-1250 B
Configuration (duty/standby)	3 duty, 1 standby	3 duty, 1 standby	3 duty, 1 standby
Maximum rated speed (rpm)	990	990	740
Variable speed or fixed speed	Variable	Fixed	Fixed
Hydraulic efficiency at duty point (%)	89.9	83.1	83.7
Net Positive Suction Head (NPSH) required at duty point (m)	5.7	6.2	3.5
Head rise to shut-off head (%)	26	14	18
Hydraulic power per pump at duty point (kW)	1,005	4,547	1,790
Maximum power per pump in operating range (kW)	1,060	5,000	2,036
Recommended motor size (kW)	1,200	5,780	2,400

It is evident from Table E1 that the raw water pump station pump sets will be fitted with variable speed drives (VSDs), whereas the other two pump stations will operate at fixed speed. The VSDs are required due to the large fluctuation in water levels within Gariep Dam and to ensure that the raw water flow matches the flow to be treated at the proposed Xhariep WTP.

A hydraulic and waterhammer analysis was undertaken to determine the maximum working and surge pressures. In order to mitigate excessive surge pressures during a pump trip event, non-return valves were recommended at the following locations:

- Pipeline from raw water pump station to break pressure tank = at chainage 4100 m, approximately 100 m upstream of the break pressure tank,
- Pipeline from high-lift pump station to Command Reservoir No 1 = at chainage 38 500 m, and
- Pipeline from booster pump station to Command Reservoir No 2 = at chainage 43 000 m.

The maximum design and field test pressure for each pipeline was determined in accordance with DWS1110, which states that “Test pressures will generally be 1.25 times the pipeline design pressure for design pressures up to and including 3.2 MPa and 1.1 times the design pressure for higher pressures.” Table E2 summarises the maximum design and field test pressures for the various pipeline sections.

Table E2: Maximum design and field test pressures

Pipe section	Pipe diameter (mm)	Maximum design pressure (m)	Maximum field test pressure (m)	Maximum pressure rating of valves, specials, etc. (m)
Gariep Dam to Xhariep WTP	1800	110	138	160

Pipe section	Pipe diameter (mm)	Maximum design pressure (m)	Maximum field test pressure (m)	Maximum pressure rating of valves, specials, etc. (m)
	2000	110	138	160
High-lift pump station to Command Reservoir No 1	1800	377	415	400
Command Reservoir No 1 to suction reservoir at booster pump station	1800	276	345	400
Booster pump station to Command Reservoir No 2	1800	195	244	250
Command Reservoir No 2 to Rustfontein WTP	1400	203	254	250

The pump station layouts were based on the sizes of the mechanical and electrical equipment required. Provision was made for storage rooms, offices, loading bays and control rooms at each pump station.

The analysis of the raw water quality indicates the following treatment requirements after evaluation and consideration of the errant data:

- Turbidity levels are moderately high and require treatment intervention.
- Total aluminium levels are high. The dissolved aluminium levels are very low and do not require treatment. The non-dissolved fraction will be removed along with general turbidity if care is taken not to re-dissolve the aluminium through extensive pH manipulation.
- The microbiological indicators will be removed adequatly through normal disinfection protocols. No positive results were noted for chlorine resistant cysts or oocysts in any of the water qualty data sets.
- A small number of datapoints reflected very high dissolved organic carbon (DOC) levels. All the high values were however reported prior to 2003. Subsequent reports all indicated DOC levels below the national standard. No specific treatment regimes are included to address organic carbon removal.
- Both the Ryznar index as well as CCPP indicate that the water is aggressive and will require stabilisation.
- The additional sampling of the water source and analysis for a significantly expanded set of parameters indicated little to no risk associated with chlorophyll-a or contaminants of emerging concern at the proposed treatment plant.
- Some historical data sets indicate a number of determinands are present at levels of concern. The high initial values are likely the result of laboratory detection limits exceeding the specified water quality targets.The latest data sets indicate that these values, when appropriately analysed, are below levels of concern.

Based on the available water quality data the water can be described as of very good quality. Turbidity, microbiology, and stability are the only determinants requiring particular attention. Conventional flocculation, settling and filtration is proposed.

The key WTP design aspects are:

- Laboratory tests indicate that the preferred flocculant for treatment of the source water is a poli-aluminium chloride flocculant.
- An options analysis indicates that hydrated lime at around 5 to 10 mg/ℓ is the most economical approach to stabilisation.
- The treatment technology proposed for the WTP can be described as conventional:
  - Flocculation and clarification will take place in a pulsator clarifier. A total of 12 separate pulsators will be required to deliver 312 Mℓ/d.
  - The rapid gravity sand filters are designed for deep penetration of floc into the filter bed. A total of 30 filters, with a surface area of 84.7 m2 each, are required to deliver 312 Mℓ/d.

The filter loading rate is 6 m/hr with all 30 filters operational and 6.67 m/hr if three filters have been removed from operation for maintenance purposes.

- A chlorine demand study was undertaken that indicated a single chlorine dose at the treatment plant would not be sufficient to sustain chlorine levels in the transfer system. Chloramination cannot be considered as Mangaung Metropolitan Municipality (MMM) does not presently receive chloraminated water from its other sources and the mixing of chloraminated and non-chloraminated water cannot be permitted. The system will therefore be chlorinated with a booster injection of chlorine at the Booster Pump Station at which time the water will have been in the transfer system for around 48 hours. A comparison of chlorination systems indicated that chlorine gas systems are more economical than on-site chlorine generation systems.
- Main disinfection will take place in the on-site storage reservoir.
- The storage reservoir make provision for another 30 minutes of storage for high lift pump balancing purposes.
- Treatment residuals will be thickened in sludge ponds. An options analysis was undertaken to compare various pond construction approaches. A Hyson cell lined pond was found to be most cost effective.

Based on the pipe diameters and operating pressures, steel was considered the only feasible pipe material for the project. Grade X52 steel, with a yield strength of 358 MPa, is recommended. The pipeline structural design was based on AWWA M11 guidelines, but using the factors of safety recommended by DWS, i.e. a factor of safety of 1.67 for both the working and surge pressures. It was calculated that wall thicknesses will vary from 8 mm on the DN 1400 pipelines to up to 22 mm on the DN 1800 pipeline, immediately downstream of the high-lift pump station.

Various options are available for the pipe lining (e.g. cement mortar, epoxy) and coating (e.g. polymer modified bitumen, fusion bonded medium density polyethylene, trilaminate polyethylene, rigid polyurethane, etc.). The preferred lining and coating need to be selected during the detailed design phase in consultation with the entity responsible for the operation and maintenance of the pipelines.

Other pipeline aspects considered, included the sizing of air valves and scour valves, the installation of inline isolation valves, the provision of off-takes to end-users from the bulk pipelines, river and stream crossings, road crossings and dealing with existing services.

Three types of reservoirs were considered for command reservoirs with a storage capacity of 80 Mℓ, namely (a) conventional above ground post-tensioned circular reinforced concrete reservoirs, (b) conventional above ground circular or rectangular reinforced concrete reservoirs, and (c) earth-fill embankment type reinforced concrete lined reservoirs. It was established that earth-fill embankment type reinforced concrete lined reservoirs will be the most economical of the three reservoir types.

Table E3 provides a summary of the capital cost estimate for the Xhariep Pipeline Project.

Table E3: Estimated Capital Expenditure for the Xhariep Pipeline Project

Description	Estimated CAPEX (ZAR)
Preliminary and General	3,939,328,274
Raw Water Pump Station	162,443,508
High-Lift Pump Station	359,650,870
Command Reservoir No 1	137,008,420
Booster Pump Station and Suction Reservoir	292,708,069
Command Reservoir No 2	304,276,170
Pipelines	14,385,273,022
Water Treatment Works (Phase 1)	2,248,730,000
Subtotal Capital Cost (Excl. VAT)	**21,829,418,333**
Contract Price Adjustment (CPA) @ 7% p.a.	4,912,557,790

Description	Estimated CAPEX (ZAR)
Allowance for Foreign Exchange Variation @ 5% p.a.	434,230,000
Contingency @ 15%	4,076,430,918
Project Cost (Excl. VAT)	**31,252,637,041**
Engineering Design Fees @ 8%	2,500,210,963
Disbursements and Recoverable Costs	124,960,856
Professional Fees (Excl. VAT)	**2,625,171,819**
Total Project Cost (Excl. VAT)	**33,877,808,861**
Total Project Cost (Incl. VAT)	**38,959,480,190**

Notes:

1 Cost Estimate Base Date – November 2024.

2 Construction Commencement Date – November 2028

The estimated operation and maintenance budget required for the first year of operation is summarised in Table E4, showing an estimated minimum O&M budget requirement.

Table E4: Estimated Annual Operation and Maintenance Budget for the Xhariep Pipeline Project

Description	Estimated Annual O&M Budget (ZAR)	
	Complete Phase 312 Mℓ/d [1]	Phase 1 208 Mℓ/d [1]
Maintenance	200,782,452	184,932,452
Labour	24,180,000	21,160,000
Energy	82,533,048	53,752,032
Chemicals	71,590,000	47,730,000
Sludge Disposal	13,940,000	9,300,000
Total OPEX (Excl. VAT)	**393,025,500**	**316,874,484**
Plant Cost (Amortised @ 20 years)	633,040,000	475,680,000
Total Annual Cost of Ownership (Excl. VAT)	**1,026,065,500**	**792,554,484**

Notes:

1 Estimated other operational cost required for first year of plant operation based on 2024 Costs

The socio-economic assessment reiterated that the implementation of the Xhariep Pipeline is essential to ensure long-term water security and economic stability in the GBWSS region.

Five sources of financing are available for a public infrastructure project of this nature, i.e., (a) grants from central government, (b) the public delivery agency’s own resources, (c) equity, (d) commercial debt, and (e) concessionary debt. If funding options are considered, the option of a 75% capital grant and a loan for the balance of the capital costs at a low interest rate is probably the first of the different funding options that will ensure that the project is affordable to households.

From an economic perspective, the project is expected to generate substantial socio-economic benefits through direct, indirect, and induced impacts. This includes job creation during the construction and operational phases and an improvement in water security, which is critical for supporting regional economic activities, particularly in the agriculture and manufacturing sectors. The financial viability of the project is achievable if tariff structures are well managed, ensuring affordability for households while maintaining financial sustainability.

The assessment highlights that, while various institutional options exist, the financial sustainability of the pipeline project will be dependent on the chosen implementing entity's capacity to manage operations and maintenance effectively. The report findings underscores that the project’s success hinges on collaboration between national, provincial and local governments, and a clear delineation of responsibilities among entities. Based on this, it is recommended that DWS should initiate the establishment of and lead a **Working Group** that involves all the relevant stakeholders, with representation at an executive and strategic level, so that agreement can be reached on:

- Responsibilities with respect to the implementation, operation and maintenance of the scheme, e.g. MMM (as they should take a leading role) can request VCWB (legally) or DWS (administratively) (including TCTA or the NWRIA) to implement the project on their behalf,
- Financing options, taking consideration that at least a 75% capital grant and a loan for the balance of the capital costs at a low interest rate would be required to result in affordable bulk water tariff increases. The creditworthiness of each institution must be considered as part of the financing options to minimise the cost impact on the end-users, and,
- Development of an implementation timeframe.

It is estimated that, if a professional service provider for the detailed design phase can be appointed towards the end of 2025, construction could commence towards the end of 2028 with commissioning taking place at end 2032.

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Abbreviations

Acronyms

Acronym	Description
AADD	Average Annual Daily Demand
AWWA	American Water Works Association
BW	Bloem Water
CBD	Central business district
DB	Distribution Board
DCP	Dynamic Cone Penetrometer
DEM	Digital Elevation Model
DOL	Direct On Line
DPSH	Deep Probe Super Heavy
DN	Nominal Diameter
DWS	Department of Water & Sanitation
EFR	Environmental flow requirements
EIA	Environmental Impact Assessment
FSL	Full Supply Level
GAADD	Gross Average Annual Daily Demand
GBWSS	Greater Bloemfontein Water Supply System
GIS	Geographical information systems
IAP	Interested and Affected Parties
IDP	Integrated Development Plan
ISP	Internal Strategic Perspective
IWULA	Integrated Water Use License Application
LiDAR	Light Detection and Ranging
LV	Low Voltage
MMM	Mangaung Metropolitan Municipality
MV	Medium Voltage
NPSH	Net Positive Suction Head
NPV	Net Present Value
NWRP	DWA Directorate: National Water Resource Planning
ORS	Orange River System
P/s	Pump station
PFC	Power Factor Correction
PPP	Private Public Partnerships
PPPr	Public Participation Process
PRV	Pressure reducing valve
RDP	Reconstruction and Development Programme
RPM	Revolutions Per Minute
RO	Reverse osmosis

Acronym	Description
RPST	Reconciliation Planning Support Tool
SDF	Spatial Development Plan
SMC	Study Management Committee
SPT	Standard Penetration Test
SWL	Safe Working Load
ToR	Terms of Reference
UAW	Unaccounted for Water
URV	Unit Reference Values
VCWB	Vaal Central Water Board
VFC	Variable Frequency Converter
VSD	Variable Speed Drive
WARMS	Water Authorisation and Registration Management System
WC/WDM	Water Conservation and Water Demand Management
WMA	Water Management Area
WRYM	Water Resource Yield Model
WSDP	Water Service Development Plan
WTP/WTW	Water Treatment Plant / Water Treatment Works

Measurement Units

Symbol	Description
Ha	Hectares
km	Kilometres
m	Meters
m^3/a	Cubic meters per annum
m^3/d	Cubic meters per day
m^3/s	Cubic meters per second
million m^3	Million cubic meters
mm/a	Millimetres per annum
million m^3/a	Million cubic meters per annum

1 Introduction

1.1 Background

The Water Reconciliation Strategy Study for the Larger Bulk Water Supply Systems: Greater Bloemfontein Area (DWS, 2012) (henceforth referred to as the "2012 Reconciliation Strategy") identified that the Greater Bloemfontein Water Supply System (GBWSS) would need to secure a sustainable water supply for the future water demands in the area. The 2012 Reconciliation Strategy recommended that the development of a major surface water augmentation scheme should be given consideration as a possible option in conjunction with the implementation of various other interventions.

Following the 2012 Reconciliation Strategy, the area experienced water shortages and the major surface water augmentation scheme option, now called the Greater Mangaung Water Augmentation Project – Xhariep Pipeline, was accelerated. Vaal Central Water Board (VCWB), previously known as Bloem Water, and Mangaung Metropolitan Municipality (MMM) independently investigated the same three route options from Gariep Dam to tie-in points within the GBWSS area (see Figure 1-1). Each institution reached a different conclusion as to which of the three was the best route/scheme.

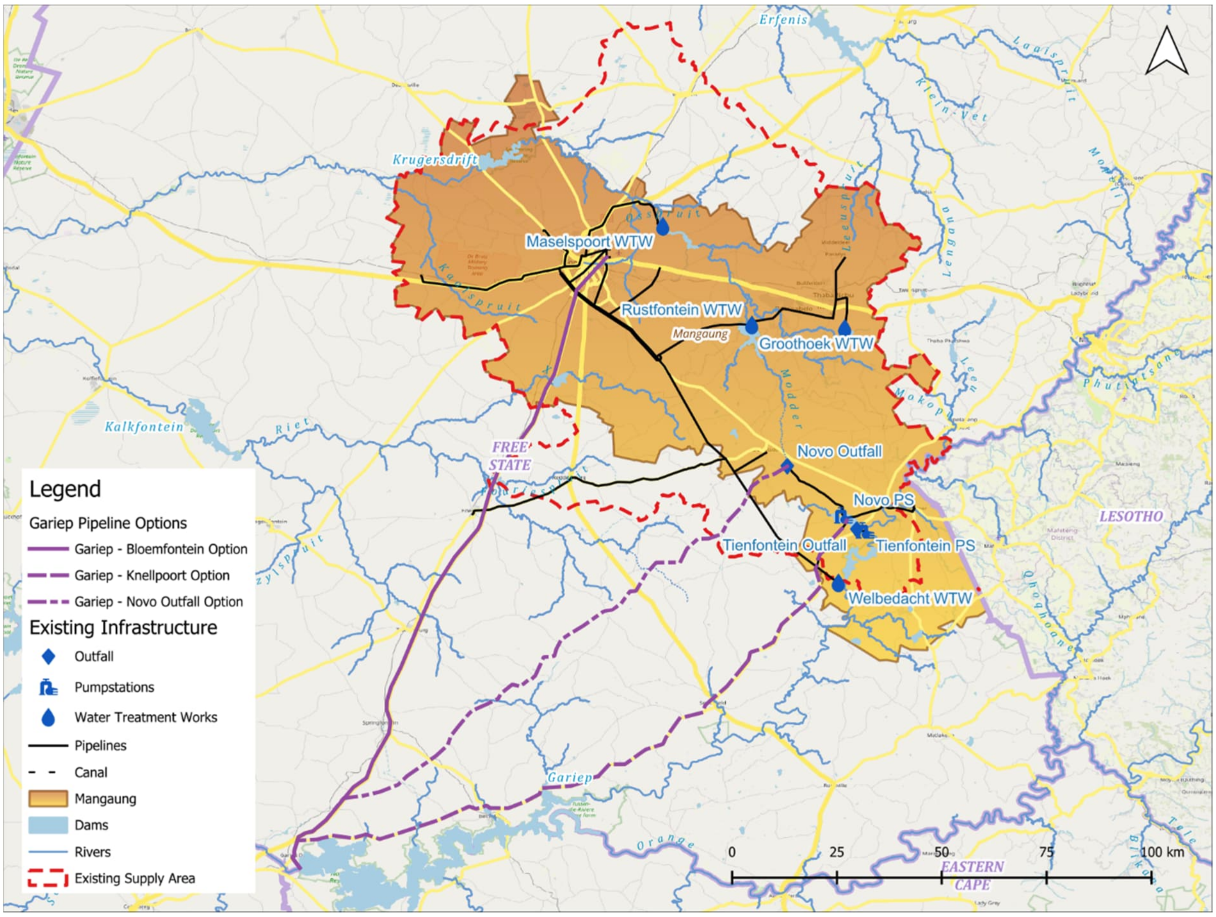


Figure 1-1: Previously studied route options from Gariep Dam to the GBWSS

The Xhariep Pipeline project was and remains of critical importance to address growing water demands on a regional basis; thus, the Department of Water and Sanitation (DWS, the Client) appointed Zutari to complete the pre-feasibility study, which included reviewing all previous studies, and recommending the optimal scheme from a national and regional perspective. This included determining routing and sizing to be taken forward to a detailed feasibility stage. Upon completion of the pre-feasibility stage, DWS in consultation with the Project Steering Committee (PSC) approved the preferred option, whereafter Zutari carried out the detailed feasibility study, the water use license application and the environmental authorisation process. The detailed design of the Xhariep Pipeline project, and the preparation of the procurement documentation, will be the subject of a future appointment and does not form part of this project.

1.2 Study Objectives

The pre-feasibility study conducted an independent investigation that built on the information collected and analysed in previous work. The objective of this study was to:

- Evaluate options for the Greater Mangaung Water Augmentation Project with Gariep Dam as the source,
- Conduct additional pre-feasibility level investigations, and,
- Select the optimal size, phasing, and configuration of the best water conveyance infrastructure option.

After DWS approval in consultation with the PSC of the selected option identified during the pre-feasibility stage, the detailed feasibility stage proceeded where the objectives were to:

- Assess the technical, financial, economic, and environmental aspects at detailed feasibility level,
- Assess the risks and redundancy of the proposed bulk infrastructure system when operated in conjunction with the existing bulk infrastructure,
- Assess the impact of the project on existing systems including the Orange River System (ORS),
- Integration and utilisation of the available capacities in the existing infrastructure, and,
- Conduct stakeholder engagement workshops.

As this study is complex in nature, the detailed feasibility stage of the project also considered:

- Institutional arrangements for ownership and operation,
- Financing options,
- Affordability and bankability in line with the National Treasury guidelines, and,
- Opportunities for phased implementation.

This pre-feasibility and detailed feasibility study is a continuation of the 2012 Reconciliation Strategy and focus specifically on the recommendation that a major surface water augmentation scheme will be required, in addition to the implementation of various other interventions, to ensure a sustainable water supply to the GBWSS until at least 2050.

This study does not address the other interventions identified in the 2012 Reconciliation Strategy (e.g. construction of a bi-directional pipeline between Knellpoort and Welbedacht dams, increasing the Tienfontein pump station's pumping capacity, implementation of a re-use of treated effluent scheme, etc.). It is, however, important to note that these other interventions are still required in addition to the Xhariep Pipeline project to satisfy the projected 2050 water demands.

1.3 Report Structure

The purpose of this Main Feasibility Study Report is to present the findings from the detailed feasibility designs undertaken for the preferred option identified during the Pre-Feasibility Phase of the project. The Main Feasibility Study Report is structured as follows:

Chapter 1 presents the background and objectives of the study.

Chapter 2 summarises the main findings of the pre-feasibility phase of the study, including the design flows and proposed scheme infrastructure.

Chapter 3 presents a summary of the site investigations undertaken.

Chapter 4 details the optimisation of the various scheme components.

Chapter 5 evaluates the various pump types available and the pump type selection for the respective pump stations.

Chapter 6 describes the hydraulic and waterhammer analysis undertaken.

Chapter 7 presents the detailed feasibility design of the pump stations.

Chapter 8 presents the detailed design of the water treatment plant.

Chapter 9 presents the detailed feasibility design of the pipelines.

Chapter 10 presents the detailed feasibility design of the reservoirs.

Chapter 11 describes the site access to the various major infrastructure components.

Chapter 12 summarises design aspects that will require special consideration during the detailed design phase of the project.

Chapter 13 provides a summary of the authorisation processes for the project.

Chapter 14 presents the updated construction and project cost estimates.

Chapter 15 summarises the findings of the socio-economic impact assessment & the legal, institutional and financing arrangements that could be considered for this project.

Chapter 16 contains a possible project implementation programme.

Chapter 17 contains the conclusions and the recommendations on the way forward.

The Main Feasibility Study Report is a collation of the detailed technical information presented in various study reports. If more technical detail is required, the reader is referred to the respective feasibility design reports.

2 Findings of Pre-Feasibility Study

2.1 Water Resources Analysis

The Water Resource Yield Model (WRYM) was used to determine the potential increase in the yield of the GBWSS due to abstraction of water from the Gariep Dam. Various scenarios were analysed and operating rules adjusted to determine the minimum volume that can be transferred from the Gariep Dam that will ensure that the 2050 water demands can be satisfied.

It was determined that, for the preferred scheme selected at the end of the pre-feasibility phase, a transfer capacity of 101 million m³/a was required from Gariep Dam. This transfer capacity is based on the following interventions being implemented:

- The bi-directional pipelines between Knellpoort and Welbedacht dams;
- Upgrading of the Tienfontein pump station capacity to 7 m³/s;
- A re-use scheme with a yield of 12.9 million m³/a; and
- Raising the full supply levels of Knellpoort and Rustfontein dams by 2m.

The raising of Knellpoort and Rustfontein dams was not considered as part of the 2012 Reconciliation Strategy but was identified as an intervention during this study to improve the operational flexibility of the overall system and to marginally reduce the volumes to be transferred from Gariep Dam. It was recommended in the Water Resource Analysis Report that the operating rules be further refined and that the impact of operational recommendations from earlier Annual Operating Analyses be evaluated, which would inform the need to implement the raising of the two dams and if so, the timing thereof.

A long-term stochastic analysis confirmed that the proposed potable transfer scheme at a capacity of 101 million m³/a can meet the GBWSS demands at the required assurance of supply.

2.2 Design Flow Rates

2.2.1 Peak Design Factors

The WRYM determines the monthly and annual flows to be transferred by the various infrastructure components (e.g. water treatment works, pump stations, pipelines, etc.) within the GBWSS to satisfy the water demands. The maximum monthly flow for each infrastructure component represents the peak monthly flow rate.

The DWS developed design and planning criteria in a document entitled “Technical Guidelines for the Development of Water and Sanitation Infrastructure”. The relevant criteria were used to determine the respective peak factors and to calculate the design capacities of the various infrastructure components.

Table J.9 in the Neighbourhood Planning and Design Guide (Red Book) recommends a peak week factor of 1.30 for large residential areas, business areas and inner city (CBD). This corresponds to the 1.31 factor for potable pipelines determined based on the DWS guidelines. In terms of the Xhariep Pipeline project, a peak week factor of 1.30 was adopted for the potable infrastructure, whereas a 5% loss factor was included for the raw water infrastructure.

2.2.2 Average and Peak Monthly Flow Rates

The average monthly flow rates, as well as the peak monthly flow rates, were determined in the WRYM for the 2035 and 2050 water demand scenarios.

Figure 2-1 and Figure 2-2 show the average and peak monthly flow rates for 2035 and 2050, respectively. The values shown in blue represent the average monthly flows (i.e. the annual flows divided by 12 months), whereas the values shown in red represent the peak monthly flows.

The average flows are used to determine operating costs (e.g. electricity, chemicals, etc.), whereas the peak flows are used for the sizing of the infrastructure components.

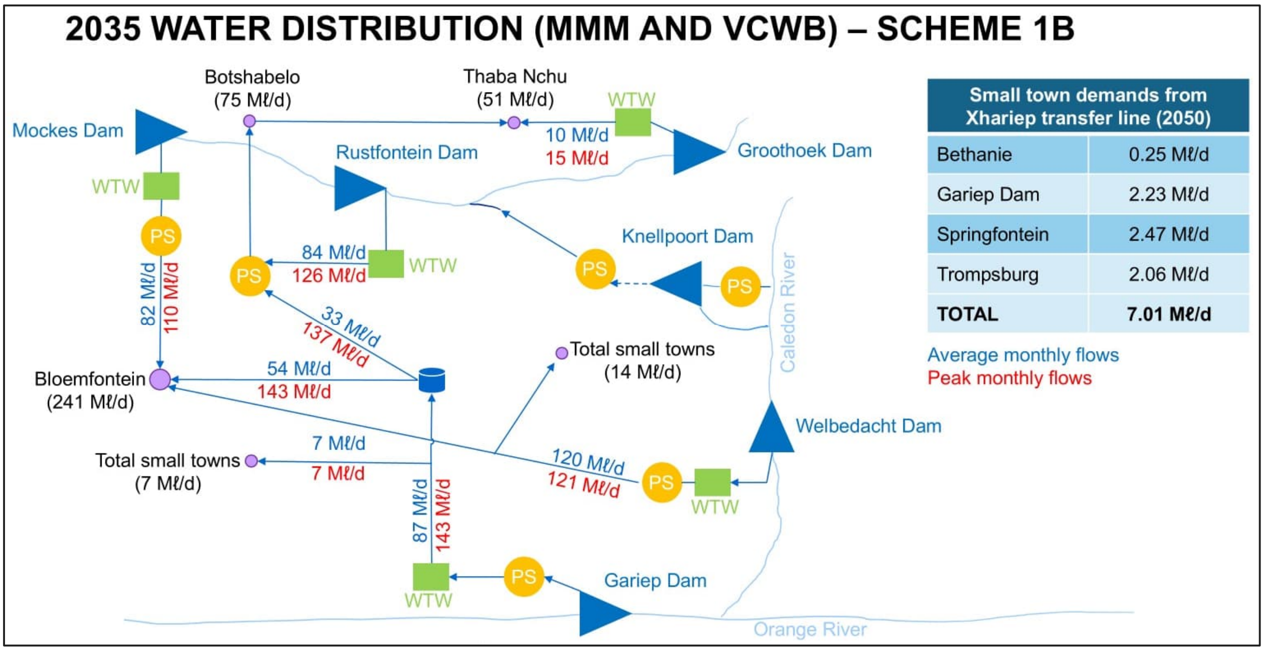


Figure 2-1: Average and peak monthly flow rates (2035)

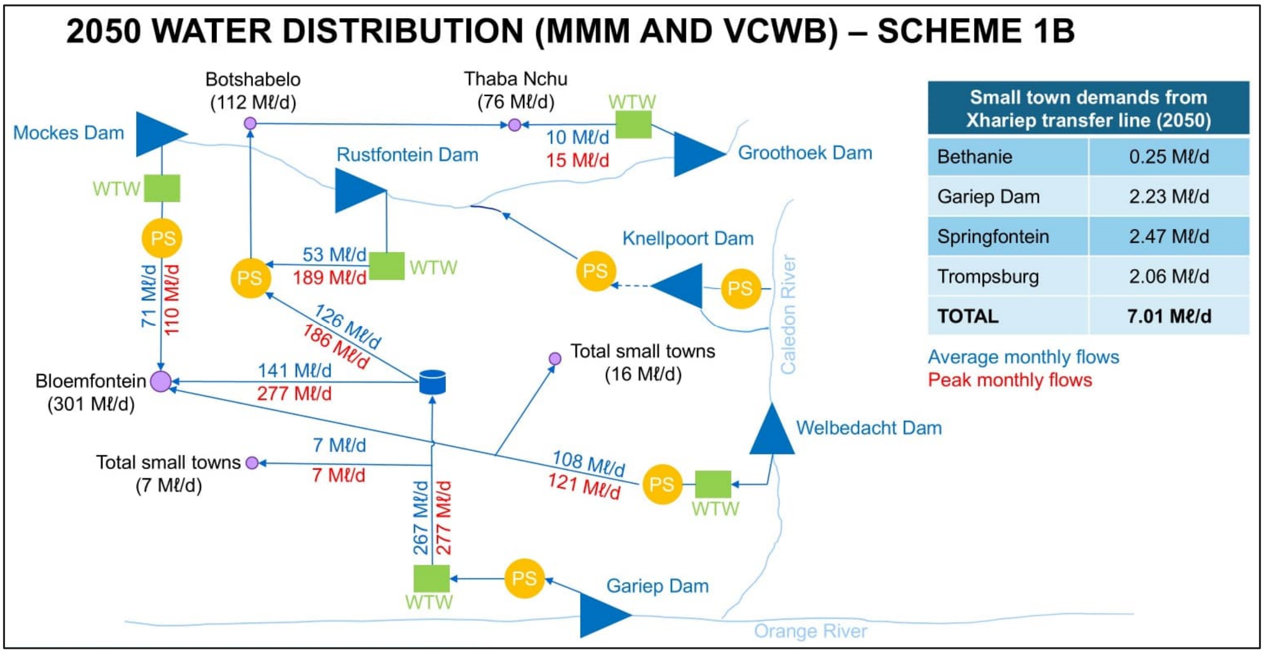


Figure 2-2: Average and peak monthly flow rates (2050)

2.2.3 Peak Week Flow Rates

The potable bulk water infrastructure is sized based on the peak week flows, using a peak week demand factor of 1.30. The peak monthly flows shown in Figure 2-2 already include a peak monthly factor of 1.15, meaning that these flows need to be increased by a factor of 1.13 to determine the peak week flows. A further loss factor of 5% needs to be added for the raw water infrastructure.

Table 2-1 provides a summary of the peak week flows of the various infrastructure components.

Table 2-1: Summary of peak week flow rates

Infrastructure component	Peak month flow rate (Mℓ/d)	Peak factor	Peak week flow rate (Mℓ/d)	Peak week flow rate (m³/s)
Raw water pipeline and pump station	277	1.13 x1.05	329	3.797
Water treatment works (WTP)	277	1.13	312	3.616
Potable water pump stations and pipelines (WTP to Command Reservoir No 2)	277	1.13	312	3.616
Command Reservoir No 2 to Longridge Reservoir	277	1.13	312	3.616
Command Reservoir No 2 to Rustfontein WTP	186	1.13	210	2.428

2.3 Proposed Bulk Water Infrastructure for Scheme 1B

Scheme 1B was the option approved by DWS in consultation with the PSC at the end of the pre-feasibility study for which the detailed feasibility design was undertaken. The main infrastructure components of Scheme 1B are shown in Figure 2-3 and summarised in Table 2-2.

Table 2-2: Summary of main infrastructure components of Scheme 1B

Infrastructure component	Capacity / Size
Low-lift pump station	3,797 m³/s
Water treatment works	312 Mℓ/d
High-lift pump station	3,616 m³/s
1st command reservoir	80 Mℓ (1)
Booster pump station	3,616 m³/s
2nd command reservoir	80 Mℓ (1)

(1) 6 hours x pump rate of 312 Mℓ/d = 78 Mℓ storage. Reservoir sizes designed for 80 Mℓ storage.

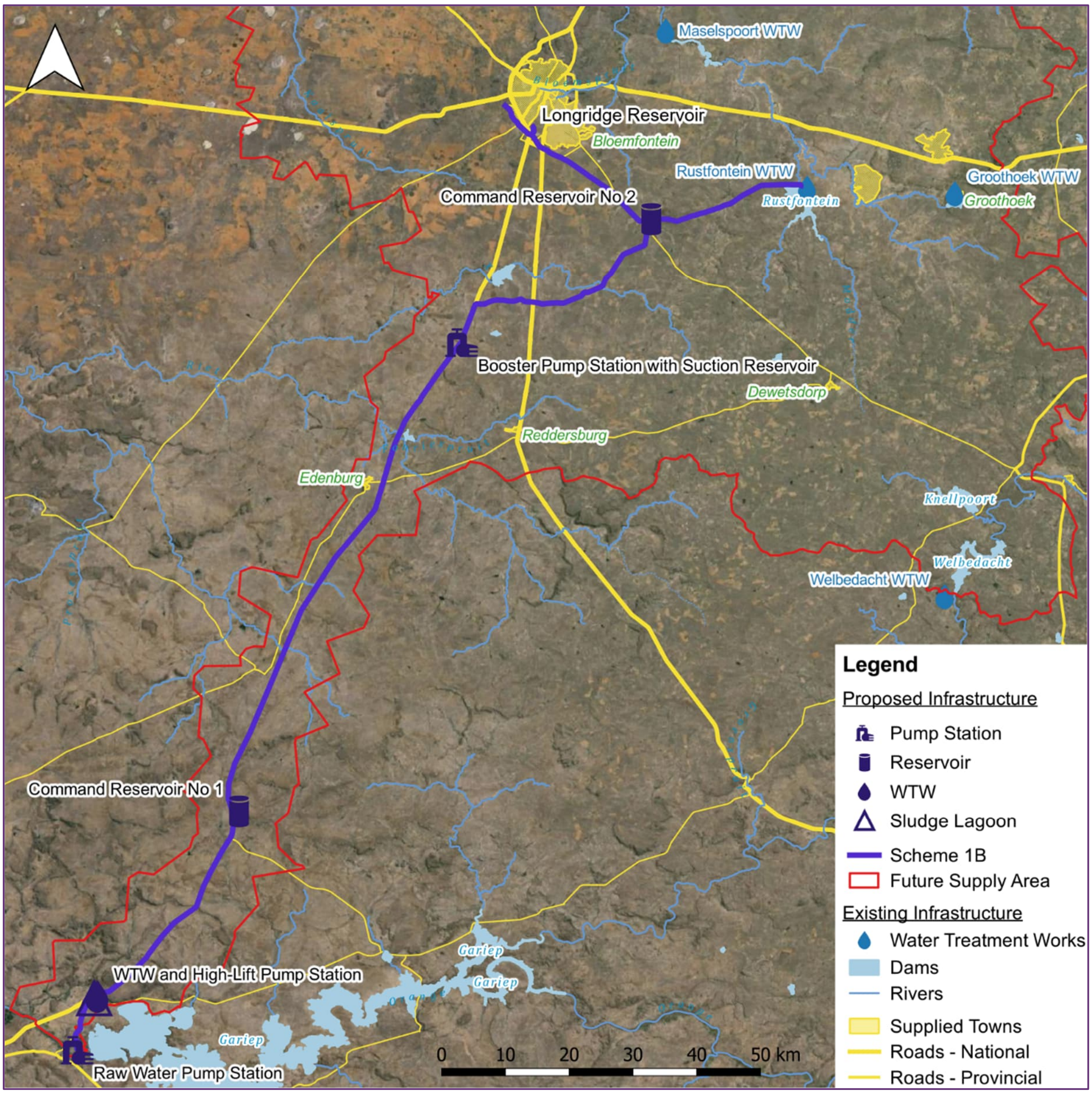


Figure 2-3: Main infrastructure components of Scheme 1B

3 Site Investigations

3.1 Topographical Survey

Details of the topographical survey undertaken for the project are contained in the Topographical Survey and Mapping Report (Ref No.: P WMA 06/D00/00/3423/8). A summary of the salient aspects from the report is provided below.

The topographical survey comprised:

- A Light Detection and Ranging (LiDAR) survey complete with installation of ground control points;
- Digital colour images of the project area;
- Installation and surveying of control points; and
- Field topographical in-fill surveys at streams, rivers and culverts.

Access to certain sections of the project was not granted by the respective landowners and/or authorities and had to be excluded from the scope. This includes the area where Command Reservoir No 2 is to be located and where the proposed pipeline infrastructure needs to tie in at Longridge and Brandkop Reservoirs in Bloemfontein. It is important that access to these areas be resolved and that the survey for these sections be completed during the detailed design phase.

The following is a summary of the topograhical survey deliverables produced under this project:

- CAD design files in Microstation DGN, DWG and DXF format showing:
    - Orthophoto tiles and LiDAR point block layout
    - The surveyed project area with boundaries
    - Contours at 0.5m, 1m and 2m intervals (Note – these contours have been smoothed and are merely an aesthetic representation of the ground shape)
- Ortho-rectified aerial images in GEOTIFF and ECW format with an 10cm pixel resolution.
- Composite Image in ECW format at 0.5m
- 1m Raster DEM
- 1m Elevation Grid
- Google Earth Overlay in KMZ format at 0.5m
- Full LiDAR points in LAS1.4 format

3.2 Geotechnical Field Investigations

Details of the geotechnical field investigations undertaken for the project are contained in the Geological and Materials Investigation Report (Ref No.: P WMA 06/D00/00/3423/7). A summary of the salient aspects from the report is provided below.

The geotechnical fieldwork included the excavation and profiling of 410 No. test pits, 44 No. boreholes, 106 No. Dynamic Cone Penetrometer (DCP) tests and 120 No. Dynamic Probe Super Heavy (DPSH) tests undertaken along the pipeline alignment and at the locations of the proposed major structures (i.e. the pump stations, reservoirs and water treatment plant). Soil electrical resistivity testing was also conducted along the pipeline at 200 m intervals.

At the time of the field investigation, access to certain privately owned farmland was not granted and wayleaves from Mangaung Metropolitan Municipality were not received. Thus, there are sections along the proposed pipeline route and at Command Reservoir No 2 that were excluded from the investigation. The field investigation for these areas needs to be undertaken as part of the detailed design phase of the project.

The general area is characterized by a gentle topography with local hills. Most of the proposed route alignment is adjacent to road reserves, with the northern section spanning over farmland. With the exception of cultivated land, the surface is mainly covered by grass and bushes, with occasional trees. The bush and tree cover are mainly found over the local hills and along drainage courses. Additionally, within the developed portion towards the Rustfontein WTP, heaps of building rubble were noted.

The 1: 250 000 scale geological maps of the area indicates that the pipeline is largely underlain by interbedded sandstone, shale and mudstone belonging to the Beaufort Group of the Karoo Supergroup, with dolerite intrusions throughout the site.

The general ground conditions found through the geotechnical investigation are summarised below.

3.2.1 Pipeline

With the investigation spanning over approximately 250 km, the ground profile naturally varied along the pipeline. However, most of the route is underlain by relatively shallow bedrock (0.3 m – 3.5 m below surface) that is covered by a combination of silty clay to silty sand (colluvium and alluvium), silty to sandy clay with occasional gravel, cobbles and boulders (residual dolerite and mudstone), silty sand (residual sandstone) and clayey silt (residual siltstone) that is partly covered by angular gravel with occasional dolerite cobbles and boulders in a silty sand matrix (fill within the road reserve).

Calcrete typically formed towards the base of the transported materials (colluvium and alluvium) at depths ranging between 0.6 m to 2.8 m, with local hardpan calcrete lenses along the pipe route. The DPSH test results indicated occasionally deep bedrock levels that ranged from 5.7 m to 12.6 m, typically at river crossings.

Ground water seepage and water rest levels were noted between 0.2 m to 7.9 m, with the shallow occurrences likely related to local peached groundwater tables.

The electrical resistivity analysis indicates that approximately 83% of the surveyed sections along the pipeline route traverse soils which range from mildly corrosive to extremely corrosive to buried steel. The pH and conductivity test results also indicated that the material encountered on site are generally extremely corrosive. It is recommended that protection of the steel pipelines will be required in the form of protective coatings and/or cathodic protection.

Soil samples taken were from test pits to determine the material quality to be considered for use during construction. The results showed that the excavated material over the area is generally not suitable for use as bedding or engineered fill. However, there are very few occasions where the colluvium, residual dolerite and residual sandstone tested as G7 - G6, that could be suitable as engineered fill. The residual soil also generally classifies as SM, with a PI less than 15 that could be suitable as selected fill blanket. However, it should be noted that the test results are variable, and not all the samples passed the required specifications. Good quality control would thus be required should this material be considered.

Based on the geological maps of the study area and the findings of the geotechnical investigation, no feasible locations for borrow pits, which could yield suitable material for use as pipe bedding, could be identified. Enquiries have also been made with consultants and contractors that were involved on construction projects in the study area over the past decade to determine whether they used material from commercial sources or from borrow pits. All of them indicated that they used material imported from commercial sources.

With the pipe invert level between 3.0 – 4.5 m below surface, most of the pipeline will be situated within bedrock (hard excavation). A combination of mudstone and sandstone underly the majority of the pipeline, with dolerite encountered in only 11% of the test pits. Blasting may be required for the dolerite intrusions. Additionally, there are local zones, along approximately 7% of the proposed alignment, with a thick soil cover that is generally related to the wetlands, rivers and streams.

No sidewall collapse was encountered within the test pits and based on this assessment excavations are considered stable. Should deep excavations be required (beyond 3.0 m), shoring or battering of excavation faces must be considered to ensure safety, which will need to be assessed by a geo-professional. It is recommended that the excavated face be inspected by a competent engineering geologist or geotechnical engineer to confirm the stability of excavations during construction.

It is also important to note that the mudstone from the Beaufort Group undergoes rapid disintegration after exposure to air if it is not protected or backfilled immediately after excavation.

3.2.2 Raw Water Pump Station

The raw water pump station area is underlain by shallow bedrock from surface to a depth of 1.1 m. Most of the area is underlain by mudstone that is covered by a hard rock dolerite sill towards the north-west. Groundwater was encountered from 2.3 m.

"Soft to intermediate" excavation to 0.8 m over most of the site, becoming "hard" thereafter, with "hard excavation" from surface over the dolerite sill is anticipated. An evaluation of seismicity found that the site classifies as ground type 1 (after SANS 10160-4:2017).

Foundations should be placed at a depth of 1 m on bedrock, with an allowable bearing pressure (ABP) of 400 kPa. Allowance for local blasting should be made for the dolerite sill.

3.2.3 Xhariep WTP and High-Lift Pump Station

The Xhariep water treatment plant (WTP) and high-lift pump station zone is underlain by shallow bedrock from 0.1 to 2.0 m below surface. Bedrock depth is generally within 0.5 m from surface but is locally deeper along the southeastern boundary. Most of the area is underlain by mudstone, with a thin dolerite dyke intrusion that traverses the site in a northwest to southeast direction. The water rest level is variable (between 1.6 - 5.7 m).

"Soft to intermediate" excavation to 0.5 m, with "hard" excavation below, over most of the site is anticipated. However, "hard" excavation is anticipated from 0.2 m over the dolerite sill. An evaluation of seismicity found that the site classifies as ground type 1 (after SANS 10160-4:2017).

Foundations should be placed at a depth of 1.1 m on bedrock, with an allowable bearing pressure (ABP) of 400 kPa. Should the foundation footprint extend over the weathered zone (where bedrock is typically deeper), the foundation should be undercut to 1 m below foundation level or bedrock (whichever is the shallowest) and replaced with a G6 material, or better, that is compacted to 95% MOD AASHTO at ± 2% optimum moisture content in 300mm thick layers. The ABP should be limited to 200 kPa. Allowance should also be made for dewatering of excavations, especially after periods of heavy and / or continuous rain. Local blasting may be required at the dolerite dyke.

3.2.4 Command Reservoir No 1

The Command Reservoir No 1 site is underlain by interbedded sandstone and mudstone with the competent bedrock between 1.3 m becoming deeper towards the south (to 2.5 m). A local hard rock dolerite sill was intersected in one of the test pits within the central eastern portion of the site, with bedrock from 1.2 m. Groundwater is generally encountered from 3.8 m.

The foundation should extend through the colluvium and be placed on a combination of bedrock and dense residual sandstone. Allowance for hard rock excavation should be made where an excavation deeper than 1.2 m is required. An evaluation of seismicity found that the site classifies as ground type 1 (after SANS 10160-4:2017).

The foundation should extend through the colluvium and placed at a depth of 1.3 m on a combination of bedrock and dense residual sandstone, with an allowable bearing pressure of 200 kPa. Allowance for local hard rock excavation should be made for the dolerite intrusion.

3.2.5 Booster Pump Station and Suction Reservoir

The booster pump station and suction reservoir area are underlain by shallow interbedded mudstone and sandstone bedrock from 0.2 m to 0.5 m below surface. Measured groundwater rest levels varied between 1.2 – 5.9 m.

“Soft to intermediate” excavation to 0.5 m, with intermediate to hard excavation beyond 0.5 m depth, is anticipated. An evaluation of seismicity found that the site classifies as ground type 1 (after SANS 10160-4:2017).

Foundations should be placed at a depth of 0.5 m on bedrock, with an allowable bearing pressure (ABP) of 400 kPa.

It should be noted the recommended allowable bearing pressures at each of the structure positions are included to guide the conceptional and detailed designs. Geotechnical design, with verification of these estimates, and associated settlement calculations will need to be included at detailed design phase.

3.3 Water Quality Testing

The objective of a water quality assessment is to characterise the raw water quality that will be abstracted at Gariep Dam.

Historical raw water quality data for the Gariep Dam is available for two periods, 2004 - 2017 and 2018 - 2022. The data was sourced from the DWS and VCWB respectively. The DWS dataset consists of 564 samples drawn in intervals varying from hours to months. The VCWB dataset consists of 75 samples drawn typically at monthly intervals. Each sample was analysed for several determinands. This resulted in each significant determinand being measured in the order of 70 to 220 times over the period.

The determinands covered in the DWS and VCWB dataset are operations and compliance focussed and do not cover all determinands required to determine all the specific design needs of a new WTP.

The absence of certain parameters from the available dataset was a concern. No algal bloom related data (e.g., chlorophyll-a, taste and odour, algal toxins, etc.) or data related to Contaminants of Emerging Concern (CECs), was available for the water abstracted into the existing Gariep WTP. A study (Venter, 2000) reported that the average chlorophyll-a concentration ranged between 0.4 - 1084 µg/l, with an average concentration of 10.8 µg/l. The high value was measured in February 1999 but was reported by the author as an exceptionally high value. The box plots in Figure 3-1 summarise the chlorophyll-a concentrations at various points along the upper Orange River catchment (up to the year 2000). The data from the Gariep Dam was of particular interest to this study. The peak levels are of concern and may require the addition of flotation to the treatment process.

Some DWS chlorophyll-a data was available for water sampled near the dam wall. The measured levels were sufficiently high at times to justify a dedicated treatment step. It was however uncertain how much of the algal load will be drawn into the WTP.

There was also no indication of the status of Contaminants of Emerging Concern (CECs) in the dam from the available data.

An additional and expanded set of water quality data was therefore generated through a dedicated sampling and analysis run that spanned 12 samples over the period spanning March 2024 to August 2024 (6 months).

An analysis was performed on the data focusing on a comparison of the data to the South African drinking water quality specification (SANS241-1, 2015) as well as WHO guidelines (World Health

Organisation, 2017). The analysis aimed to highlight determinands that do not meet the drinking water quality standard and that justify treatment interventions.

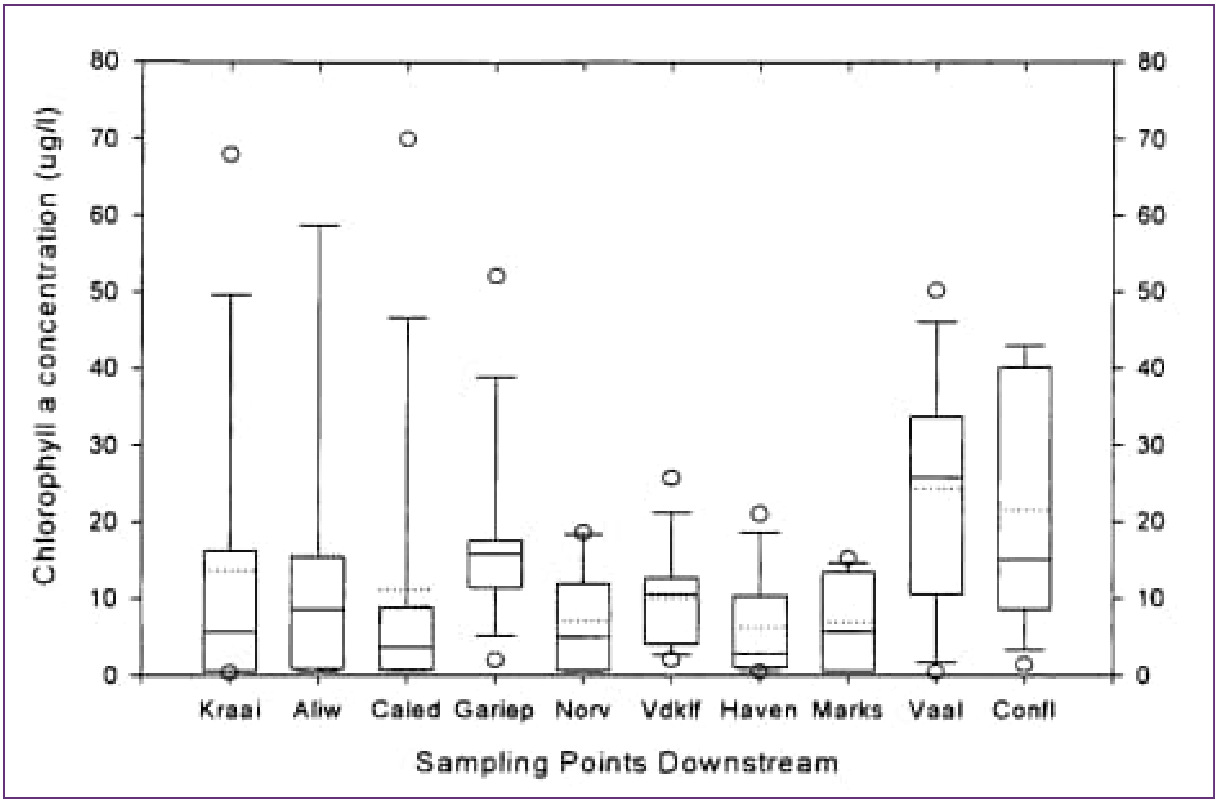


Figure 3-1: Box plots of the chlorophyll-a concentrations at different sampling sites in the Orange River system for an extended period (not specified) (Venter, 2000)

The analysis of the raw water quality indicates the following treatment requirements after evaluation and consideration of the errant data:

- Turbidity levels are moderately high and require treatment intervention.
- Aluminium levels are high. The data reflects the total aluminium and therefore includes the non-dissolved fraction which will likely be associated with suspended particles. Removal processes focussing on suspended particles will then also address the aluminium concerns. The dissolved aluminium levels are very low and do not require treatment.
- The data reflects the quality of untreated water, and the microbiological results are therefore not a significant concern. These numbers are expected to reduce significantly when the water is subjected to standard treatment protocols.
- A small number of datapoints reflected very high dissolved organic carbon (DOC) levels. All the high values were however reported prior to 2003. Subsequent reports all indicated DOC levels below the national standard. No specific treatment regimes are included to address organic carbon removal.
- Both the Ryznar index as well as CCPP indicate that the water is aggressive and will require stabilisation.

Based on the available water quality data the water can be described as of very good quality. Turbidity, microbiology, and stability are the only determinands requiring particular attention.

The additional sampling of the water source and analysis for a significantly expanded set of parameters indicated little to no risk associated with chlorophyll-a or contaminants of emerging concern at the proposed treatment plant. Several other determinands which previously showed elevated levels were now confirmed as not being of concern. The high initial values are likely the result of laboratory detection limits exceeding the specified water quality targets.

4 Scheme Optimisation

The optimum diameters for the rising mains needed to be determined for the peak week flows. The optimisation was performed by developing a costing model to calculate the net present values (NPVs) for various pipeline diameters. The NPV takes into account capital, operating and maintenance costs. It should be noted that the NPV does not represent the construction cost of the project.

For each rising main, the NPVs were calculated for a range of pipeline diameters (i.e. typically from DN 1400 to DN 2200). The diameter with the lowest NPV represented the optimum pipeline diameter. In addition to this, various sensitivity analyses were undertaken, which included:

- Two options for demand growth – an option where the full demand was used over the entire project life (i.e. assuming delays with the implementation of other 2012 Reconciliation Strategy interventions), and an option where a linear increase in demand happens over time with the full demand only needed by 2050.
- The pipe wall thicknesses were calculated using two methods – the one method only considered internal pressures or hoop stress, the other method determined the minimum wall thickness required to satisfy the external loads and hoop stress requirements.
- Different discount rates were considered.
- The impact of energy costs increasing at rates higher than Consumer Price Index (CPI) was tested, assuming that energy costs could increase annually by up to 4% more than CPI.
- It was found that the optimum pipeline diameters did not change when these sensitivity analyses were undertaken. Based on the pipeline optimisation, the following pipeline diameters were recommended:
    - Raw water pump station to break pressure tank = DN 1800
    - High-lift pump station to Command Reservoir No 1 = DN 1800
    - Booster pump station to Command Reservoir No 2 = DN 1800

The diameters of the pipelines operating under gravity are dictated by the available head (pressure) and corresponding design flows. The hydraulic analyses of these gravity pipeline sections are discussed in Chapter 6 of this report.

5 Pump Type Selection

5.1 Pump Duties

Based on the pipeline diameters recommended as part of the scheme optimisation, the pump duties for each of the pump stations can be determined. Characteristic system curves (referred to henceforth as "system curves") are developed for each pump station, which enable pumps to be selected that can operate over the full spectrum of anticipated operating conditions.

5.1.1 Raw Water Pump Station

The following levels were used to determine the system curves for the raw water pump station:

- Minimum recorded level in Gariep Dam = 1232 masl
- Average operating water level in Gariep Dam = 1254 masl
- Maximum recorded water level in Gariep Dam = 1262 masl
- Level of discharge tank on the rising main = 1315 masl

The water levels provided for Gariep Dam are based on historical water level data, recorded weekly, from 1971 to 2023 as shown in Figure 5-1.

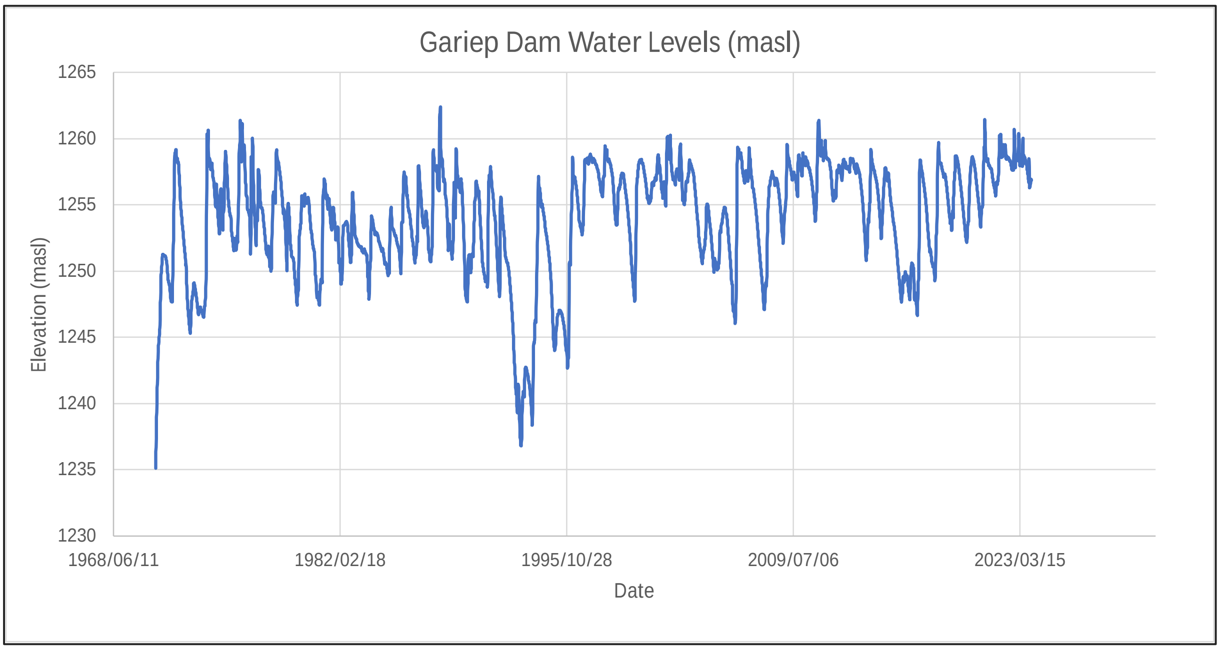


Figure 5-1: Weekly water levels of Gariep Dam (1971 to 2023)

Due to the short length of the pumping main, only a pipe friction coefficient of 0.6mm was used in calculating the system curves, which are shown in Figure 5-2. The duty point of 3.797 m³/s @ 73m head was determined based on a water level of 1245 masl in Gariep Dam, as the dam level only dropped below this level on two occasions over a period of more than 40 years.

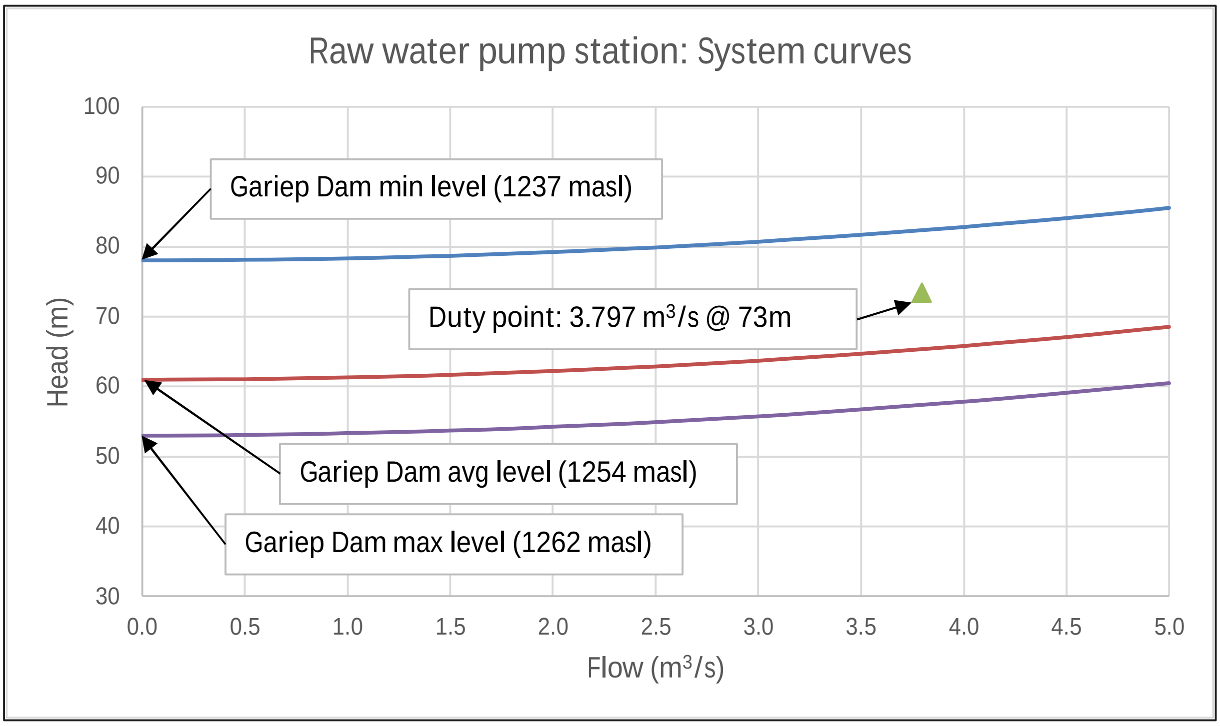


Figure 5-2: Raw water pump station system curves

5.1.2 High-Lift Pump Station

The following levels were used to determine the system curves for the high-lift pump station:

- Average operating water level in suction reservoir = 1285 masl
- Full supply level of Command Reservoir No 1 = 1565 masl

Given the length of the rising main, system curves were developed using pipe friction coefficients of 0.015mm (representative of a newly installed pipeline) and 0.60mm (representative of an aged pipeline), respectively. The system curves for the high-lift pump station are shown in Figure 5-3. The duty point of the high-lift pump station is 3.616 m³/s @ 320m head.

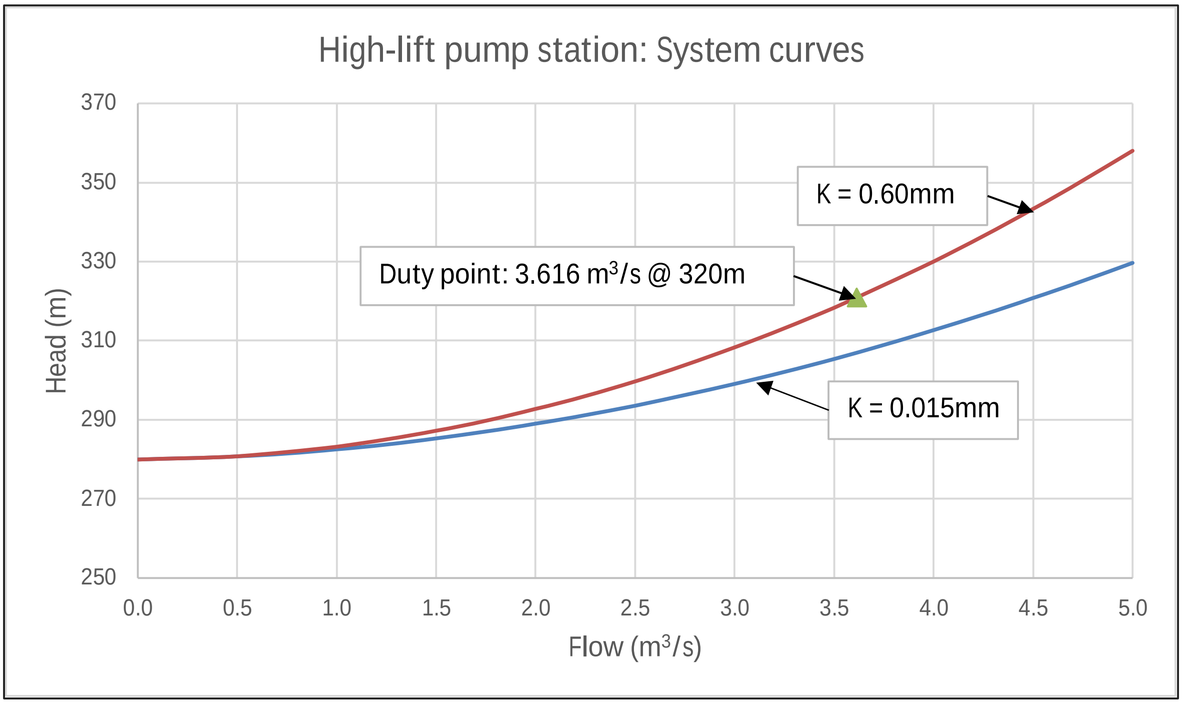


Figure 5-3: High-lift pump station system curves

5.1.3 Booster Pump Station

The following levels were used to determine the system curves for the booster pump station:

- Average operating level in suction reservoir = 1445 masl
- Full supply level of Command Reservoir No 2 = 1530 masl

Given the length of the rising main, system curves were developed using pipe friction coefficients of 0.015mm (representative of a newly installed pipeline) and 0.60mm (representative of an aged pipeline), respectively. The system curves for the booster pump station are shown in Figure 5-4. The duty point of the booster pump station is 3.616 m³/s @ 127m head.

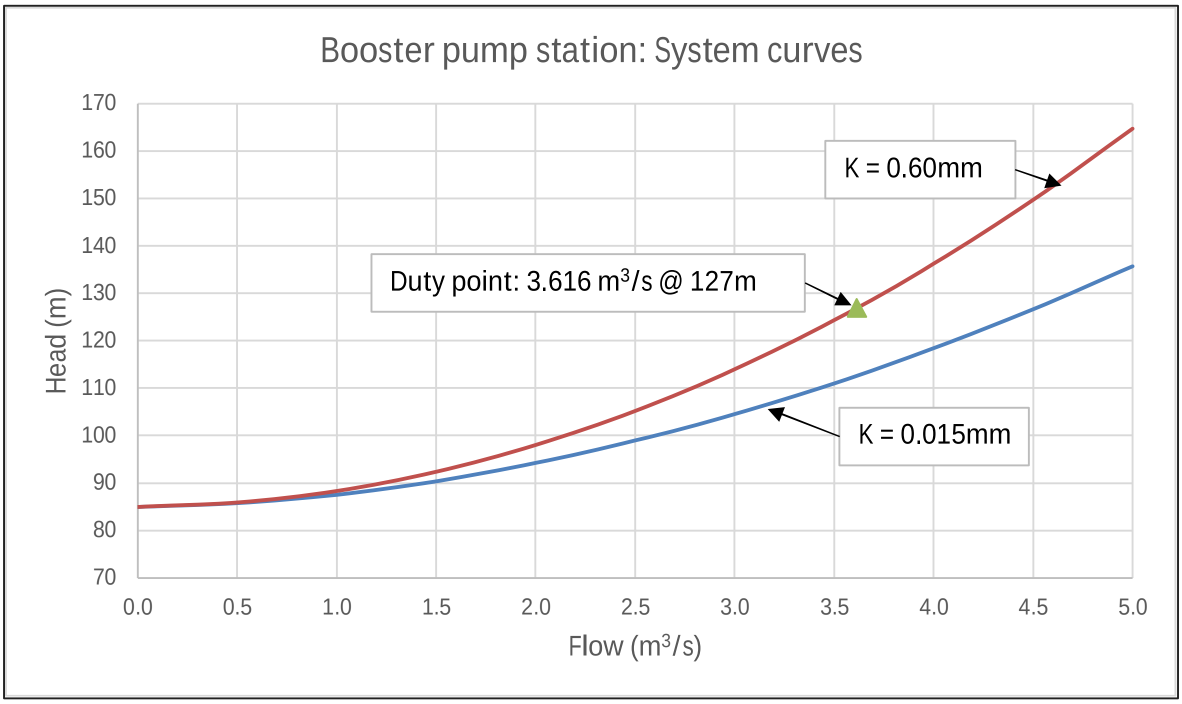


Figure 5-4: Booster pump station system curves

5.2 Available Pump Types

Table 5-1 summarises the basic pump types available for pumping raw and treated potable water, including information on their flows and pressure ranges and applicability to the proposed pump stations. Variations of the basic pump types are available, e.g. the horizontal split casing pumps can be a single-stage pump or be fitted with two or three stages. In applications with high flows, the split casing pumps can also be fitted with a double suction inlet (i.e. two inlet pipes). The variations are not discussed in detail in Table 5-1.

Table 5-1: Summary of available pump types for raw and potable water applications

Pump type	Flow range (per pump)	Pressure range	Applicability to this project		
			Raw water PS	High-lift PS	Booster PS
Vertical turbine	25 ℓ/s to > 2 000 ℓ/s	15 m to > 300 m	Yes	Yes	Yes
End-suction centrifugal	1 ℓ/s to 500 ℓ/s	5 m to 70 m	No (1)	No (1)	No (1)
Multi-stage centrifugal	1 ℓ/s to 125 ℓ/s	5 m to 270 m	No (2)	No (2)	No (2)
Horizontal split casing	25 ℓ/s to > 2 500 ℓ/s	7 m to 140 m	Yes	Yes (3)	Yes

(1) Flow and pressure range of pumps not suitable for this project

(2) Flow range of pumps not suitable for this project
(3) Horizontal split casing pumps can be provided with multiple stages, similar to a multi-stage pump

5.3 Proposed Pump Options

Based on the available pump types discussed in Section 5.2, pump selection software available from reputable pump manufacturers were used to identify suitable pump options for each of the pump stations. The proposed pump options were also verified with pump manufacturers to ensure that these pump selections are appropriate and optimised solutions.

The proposed pump options referenced in this report are based on pumps manufactured by Sulzer. In the majority of cases, Sulzer was able to offer more than one pump type for each of the pump stations – the report only discusses the most efficient option. Where possible, it was also verified that the proposed pump option could be offered by multiple pump manufacturers.

5.3.1 Raw Water Pump Station

The recommended pump selected for the raw water pump station, to achieve the duty point of 3.797 m^3/s @ 73m, was a Sulzer SMD 500-750A pump with three pumps operating in parallel to deliver the total flow. Table 5-2 summarises the relevant pump details for the raw water pump station.

Table 5-2: Raw water pump selection details

Description	Details
Pump duty	3.797 m^3/s @ 73m
Pump Model	SMD 500-750 A
Configuration (duty/standby)	3 duty, 1 standby
Maximum rated speed (rpm)	990
Variable speed or fixed speed	Variable
Hydraulic efficiency at duty point (%)	89.9
Net Positive Suction Head (NPSH) required at duty point (m)	5.7
Shut-off head (m)	92
Head rise to shut-off head (%)	26
Hydraulic power per pump at duty point (kW)	1,005
Maximum power per pump in operating range (kW)	1,060
Recommended motor size (kW)	1,200

In terms of alternative pump models, options were available that will result in a two duty, one standby configuration (pumps operating at 990 rpm) or a four duty, one standby configuration (pumps operating at 1485 rpm).

With respect to the information presented in Table 5-2, the following should be noted regarding the proposed pump selected:

- The pumped media will be raw water that could contain suspended particles/solids during times of floods. These suspended particles/solids cause wear and tear on mechanical equipment. In order to mitigate the risk of wear and tear, it is preferable to reduce the operating speed and to provide protective coatings on the internal parts of the pump. A pump speed of 990 rpm is preferred to a pump speed of 1485 rpm.
- A three duty, one standby pump offers more flexibility than a two duty, one standby pump, especially as the flow will increase over time.
- The motors will be fitted with variable speed drives (VSDs) so that the pump rate can match the actual flows to be treated at the WTP.

- Hydraulic efficiencies above 80% is typically considered as very good and will result in the lowest energy costs.
- The NPSH required for the pump needs to be lower than the NPSH available. The lower the operating speed of the pump, the lower the NPSH required.
- It is good engineering practice to select pumps where the shut-off head is at least 10% to 15% higher than the head at the duty point. For the proposed pump, the shut-off head of 92 m is 26% higher than the head at duty point, which is 73 m.
- Motor sizes are typically selected to have a 10% to 15% safety margin above the highest power demand along the pump curve when the pump operates at maximum speed. A 1,200 kW motor will therefore provide a safety margin of 13% above the maximum power of 1,060 kW.

Given the fluctuation in water levels of Gariep Dam, variable speed drives (VSDs) are required to change the motor speed to ensure that the pump curves intersect the system curves. Figure 5-5 shows the system and pump curves for the raw water pump station.

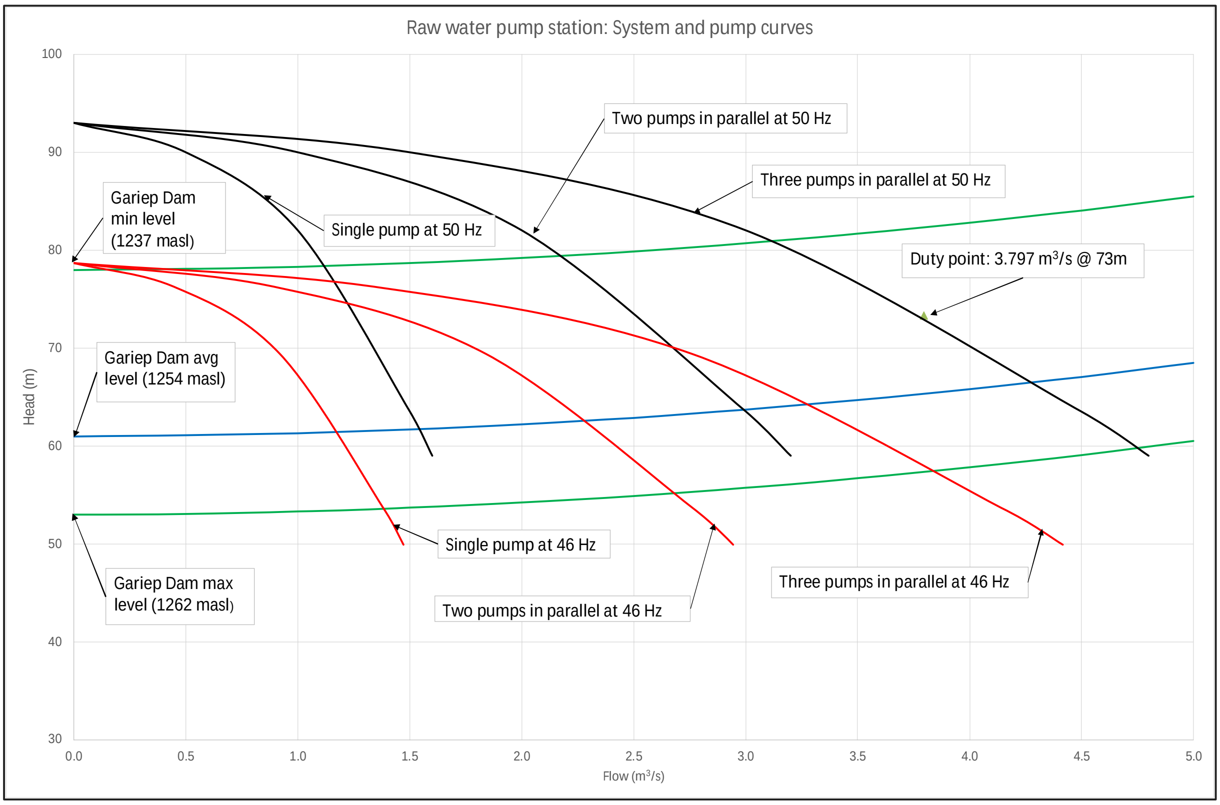


Figure 5-5: Raw water pump station – system and pump curves

The following information is evident from Figure 5-5:

- The pump speed of a single pump must be reduced to 46 Hz (910 rpm) to intersect the system curve when Gariep Dam is at its maximum level of 1262 masl.
- A single pump operating at 50 Hz, and Gariep Dam being at its average water level of 1254 masl, will deliver a flow of approximately 1.6 m³/s. The pump station pipework needs to be designed to handle this as the maximum flow per pump.
- The pump speed needs to be increased to 50 Hz (990 rpm) and three pumps must operate in parallel to deliver a flow of 3.797 m³/s.
- With three pumps operating in parallel at 50 Hz, and Gariep Dam being at its average water level of 1254 masl, it will be possible to deliver a total flow of approximately 4.3 m³/s. The pump station manifolds, the rising main and gravity main need to be designed to handle this as the maximum flow delivered by the pump station.

5.3.2 High-Lift Pump Station

The recommended pump selected for the high-lift pump station, to achieve the duty point of 3.616 m^3/s @ 320m, was a Sulzer HPDM-450-1000-d+2-26 pump with three pumps operating in parallel to deliver the total flow. Table 5-3 summarises the relevant pump details for the high-lift pump station.

Table 5-3: High-lift pump selection details

Description	Details
Pump duty	3.616 m^3/s @ 320m
Pump Model	HPDM-450-1000
Configuration (duty/standby)	3 duty, 1 standby
Maximum rated speed (rpm)	990
Variable speed or fixed speed	Fixed
Hydraulic efficiency at duty point (%)	83.1
Net Positive Suction Head (NPSH) required at duty point (m)	6.2
Shut-off head (m)	365
Head rise to shut-off head (%)	14
Hydraulic power per pump at duty point (kW)	4,547
Maximum power per pump in operating range (kW)	5,000
Recommended motor size (kW)	5,780

In terms of alternative pump models, options were available that will result in a two duty, one standby configuration (pumps operating at 990 rpm) or a four duty, one standby configuration (pumps operating at 990 rpm).

With respect to the information presented in Table 5-3, the following should be noted regarding the proposed pump selected:

- The operating speed of all three pump options is 990 rpm (i.e. 6-pole motors).
- A three duty, one standby pump offers more flexibility than a two duty, one standby pump, especially as the flow will increase over time. The pump offered for the four duty, one standby, arrangement had a lower hydraulic efficiency and was therefore not further considered.
- The motors will be fixed speed motors as pumping will be to a command reservoir where the balancing of flow will take place.
- Hydraulic efficiencies above 80% is typically considered as very good for high-pressure applications and will result in the lowest energy costs.
- The high-lift pumps will draw water from the clearwell at the WTP. The clearwell is located approximately 3 m higher than the pump centreline, meaning that the NPSH available is approximately 13 m. The maximum NPSH required for the pump is approximately 10 m (i.e. when a single pump is in operation), which is lower than the NPSH available. With three pumps in operation, the NPSH required is only 6.2 m.
- It is good engineering practice to select pumps where the shut-off head is at least 10% to 15% higher than the head at the duty point. For the proposed pump, the shut-off head of 365 m is 14% higher than the head at duty point, which is 320 m.
- Motor sizes are typically selected to have a 10% to 15% safety margin above the highest power demand along the pump curve when the pump operates at maximum speed. A 5,780 kW motor will therefore provide a safety margin of 16% above the maximum power of 5,000 kW.

Figure 5-6 shows the system and pump curves for the high-lift water pump station.

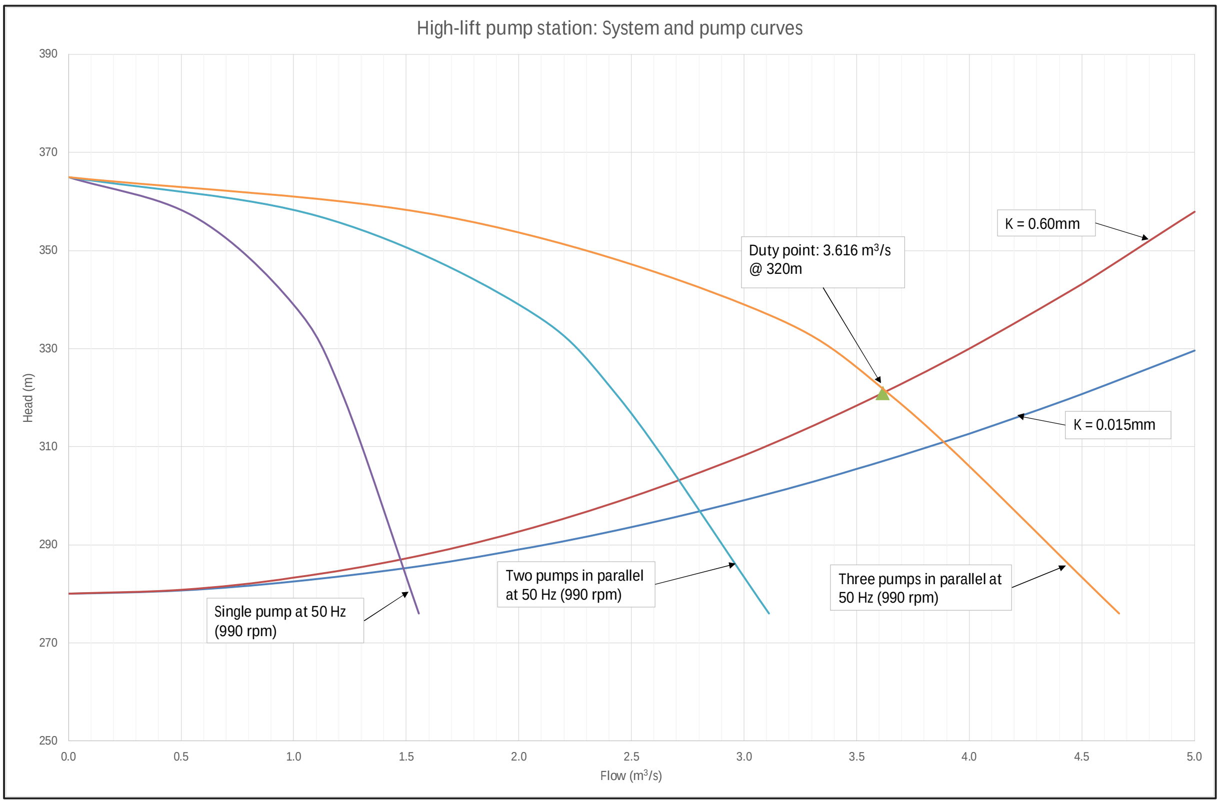


Figure 5-6: High-lift pump station – system and pump curves

The following information is evident from Figure 5-6:

- A single pump operating at 50 Hz, and based on the system curve of a newly installed pipeline (i.e. k = 0.015 mm), will deliver a flow of approximately 1.5 m³/s. The pump station pipework needs to be designed to handle this as the maximum flow per pump.
- With three pumps operating in parallel at 50 Hz, and based on the system curve of a newly installed pipeline (i.e. k = 0.015 mm), it will be possible to deliver a total flow of approximately 3.9 m³/s. The pump station manifolds and the rising main need to be designed to handle this as the maximum flow delivered by the pump station.

5.3.3 Booster Pump Station

The recommended pump selected for the booster pump station, to achieve the duty point of 3.616 m³/s @ 127m, was a Sulzer SMD 600-1250 B pump with three pumps operating in parallel to deliver the total flow. Table 5-4 summarises the relevant pump details for the booster pump station.

Table 5-4: Booster pump selection details

Description	Details
Pump duty	3.616 m³/s @ 127m
Pump Model	SMD 600-1250 B
Configuration (duty/standby)	3 duty, 1 standby
Maximum rated speed (rpm)	740
Variable speed or fixed speed	Fixed
Hydraulic efficiency at duty point (%)	83.7
Net Positive Suction Head (NPSH) required at duty point (m)	3.5
Shut-off head (m)	149
Head rise to shut-off head (%)	18

Description	Details
Hydraulic power per pump at duty point (kW)	1,790
Maximum power per pump in operating range (kW)	2,036
Recommended motor size (kW)	2,400

In terms of alternative pump models, options were available that will result in a two duty, one standby configuration (pumps operating at 740 rpm) or a four duty, one standby configuration (pumps operating at 740 rpm).

With respect to the information presented in Table 5-4, the following should be noted regarding the proposed pump selected:

- All the pump options will operate at a fixed speed of 740 rpm (i.e. 8-pole motors). The hydraulic efficiency of the three duty, one standby configuration was the highest of the various pump models and therefore the preferred pump option.
- The motors will be fixed speed motors as pumping will be to a command reservoir where the balancing of flow will take place.
- Hydraulic efficiencies above 80% is typically considered as very good and will result in the lowest energy costs.
- The booster pumps will draw water from the suction reservoir at the pump station site. The reservoir is located approximately 2 m higher than the pump centreline, meaning that the NPSH available is approximately 12 m. The maximum NPSH required for the pump is approximately 8 m (i.e. when a single pump is in operation), which is lower than the NPSH available. With three pumps in operation, the NPSH required is only 3.5 m.
- It is good engineering practice to select pumps where the shut-off head is at least 10% to 15% higher than the head at the duty point. For the proposed pump, the shut-off head of 149 m is 18% higher than the head at duty point, which is 127 m.
- Motor sizes are typically selected to have a 10% to 15% safety margin above the highest power demand along the pump curve when the pump operates at maximum speed. A 2,400 kW motor will therefore provide a safety margin of 18% above the maximum power of 2,036 kW.

Figure 5-7 shows the system and pump curves for the booster pump station.

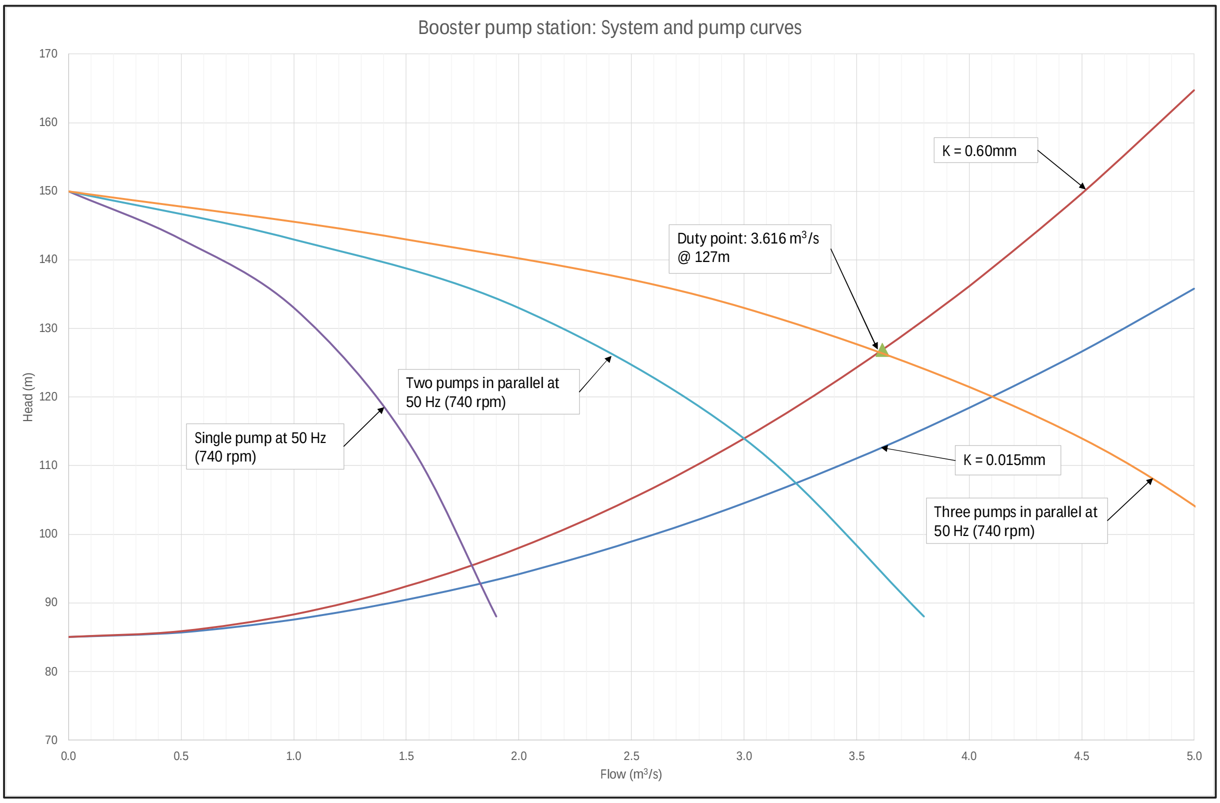


Figure 5-7: Booster pump station – system and pump curves

The following information is evident from Figure 5-7:

- A single pump operating at 50 Hz, and based on the system curve of a newly installed pipeline (i.e. k = 0.015 mm), will deliver a flow of approximately 1.85 m^3/s. The pump station pipework needs to be designed to handle this as the maximum flow per pump.
- With three pumps operating in parallel at 50 Hz, and based on the system curve of a newly installed pipeline (i.e. k = 0.015 mm), it will be possible to deliver a total flow of approximately 4.1 m^3/s. The pump station manifolds and the rising main need to be designed to handle this as the maximum flow delivered by the pump station.

6 Hydraulic and Waterhammer Analysis

6.1 Hydraulic Analysis

The Darcy-Weisbach equation was used to calculate the frictional losses in the pipeline. The Darcy-Weisbach equation can be expressed as:

$$h_f = \frac{fLv^2}{2gD}$$

Where:

h_f= friction head loss (m)
f = friction coefficient (Colebrook-White friction factor as a function of pipe roughness, k)
L = length of pipe (m)
v = velocity (m/s)
g = gravitational acceleration (9.81 m/s²)
D = internal diameter of pipe (m)

The typical pipe roughness values (k) for new and aged cement mortar lined steel pipes are 0.15 mm and 0.60 mm, respectively. In comparison, the pipe roughness (k) for a new and aged epoxy lined steel pipe is 0.03 mm and 0.15 mm, respectively. Although the proposed pipelines would be newly installed for this project, a more conservative pipe roughness of 0.60 mm was selected to calculate the maximum anticipated working pressures for calculating the pipe wall thickness. The decision on the preferred pipe lining (i.e. epoxy or cement mortar) will be taken during the detailed design stage of the project, but a 15mm thick cement mortar lining was assumed as a conservative approach to determining the working pressures.

The overall project was divided into smaller sections to reflect the respective hydraulic controls as discussed below.

6.1.1 Gariep Dam to Water Treatment Works

Based on the pipeline optimisation undertaken in Section 4, a DN1800 pipeline is required from Gariep Dam to the high point where the break pressure tank is located.

It was noted in Section 5.3.1 that the rising and gravity mains need to be designed for a maximum flow rate of 4.3 m³/s (i.e. three pumps operating at 50 Hz with Gariep Dam at its average water level) even though the flow at the designed duty point is only 3.797 m³/s. A DN2000 pipeline is proposed for the gravity section from the break pressure tank to the Xhariep water treatment plant (WTP).

Figure 6-1 shows the hydraulic gradeline from Gariep Dam to the Xhariep WTP for a flow of 3.797 m³/s, as well as the PN10 (i.e. 100m) working pressure line.

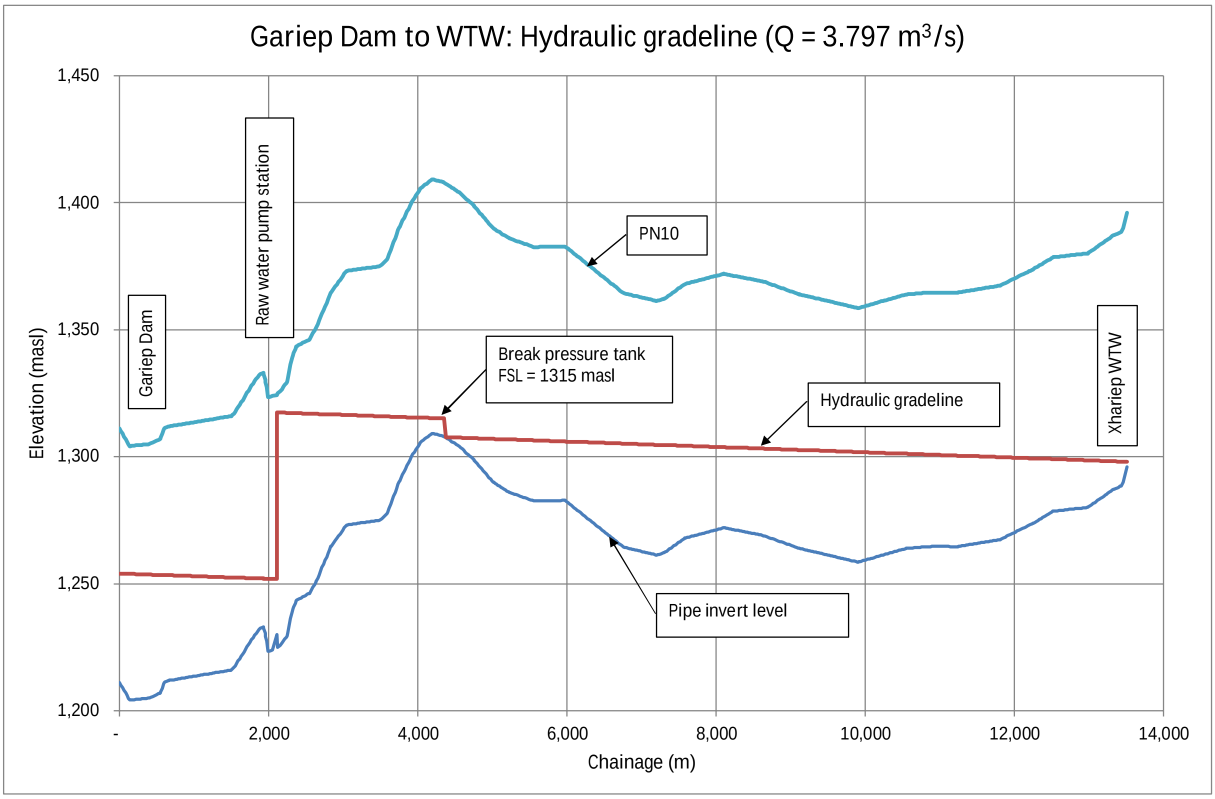


Figure 6-1: Gariep Dam to WTP – Hydraulic Gradeline

It is evident from Figure 6-1 that the high point located at chainage 4,354 m is higher than the inlet to the Xhariep WTP. This means that the section of pipeline immediately downstream of the high point will drain each time pumping is stopped. The uncontrolled filling of this drained section when the pumps re-start could result in excessive surge pressures, hence the need for a break pressure tank with an estimated full supply level of 1,315 masl. The break pressure tank will be designed to slowly fill the downstream pipeline until all air is expelled, which will mitigate the risk of excessive surge pressures during pump start and pump stop events.

An inlet level of 1,302 masl was assumed at the Xhariep WTP. At a flow of 3.797 m³/s, the hydraulic gradeline at the outlet of the break pressure tank will be at 1,307.6 masl. The hydraulic gradeline will increase to 1,309.1 masl if the flow increases to 4.3 m³/s. Even at a flow of 5.0 m³/s, the hydraulic gradeline will only be at 1,311.6 masl, which is still lower than the full supply level of the break pressure tank. The hydraulic gradelines were also calculated should a DN1800 pipeline be installed along the gravity section of the pipeline. At a flow of 4.3 m³/s, the hydraulic gradeline will increase to 1,314.4 masl in the DN1800 pipeline, which is almost at the full supply level of the break pressure tank. This will require the elevation of the break pressure tank to be increased, which in turn will increase the pumping costs. The DN2000 pipeline diameter is therefore the optimal diameter for the gravity section of the pipeline from the break pressure tank to the WTP.

It is further evident from Figure 6-1 that the working pressure in the entire pipeline is below PN10. The maximum working pressure along the entire pipeline is 92m, which will be experienced at the discharge side of the raw water pump station.

6.1.2 High-Lift Pump Station to Command Reservoir No 2

The infrastructure from the high-lift pump station, located at the Xhariep WTP, to Command Reservoir No 2 comprises the following sub-components or sub-systems:

- High-lift pump station to Command Reservoir No 1;
- Command Reservoir No 1 to the suction reservoir at the booster pump station; and
- Booster pump station to Command Reservoir No 2.

The hydraulics of these sub-systems are interdependent, e.g. if the full supply level of Command Reservoir No 1 is increased, it will result in an increased pumping head at the high-lift pump station and could result in a reduced pipeline diameter between Command Reservoir No 1 and the suction reservoir at the booster pump station.

Based on the optimisation discussed in Section 4, it was concluded that a DN1800 pipeline will be the optimum pipeline diameter from the high-lift pump station to Command Reservoir No 2 for a design flow of 3.616 m³/s. Figure 6-2 shows the hydraulic gradeline using a pipe roughness of 0.60 mm, as well as the static pressures, from the high-lift pump station to Command Reservoir No 2. It also shows the PN16 (160 m), PN25 (250 m) and PN40 (400 m) pressure lines.

Table 6-1 summarises the pressure ratings based on the hydraulic gradeline and static pressures presented in Figure 6-2.

Table 6-1: Pressure ratings of pipeline sections (high-lift pump station to Command Reservoir No 2)

Start chainage (m)	End chainage (m)	Maximum working pressure (m)	Pressure rating of valves, specials, etc. (m)
0	11,000	320	400
11,000	21,500	238	250
21,500	42,650	149	160
42,650	Command Reservoir No 1 with full supply level = 1565 masl		
42,650	69,000	151	160
69,000	81,000	189	250
81,000	97,000	151	160
97,000	138,100	246	250
138,100	Suction Reservoir (FSL = 1445 masl) & Booster Pump Station		
138,100	155,000	196	250
155,000	181,500	153	160
181,500	Command Reservoir No 2 with full supply level = 1530 masl		

6.1.3 Command Reservoir No 2 to Longridge Reservoir

Command Reservoir No 2 will supply Longridge Reservoir, with a full supply level of 1,475 masl, under gravity. The peak design flow that needs to be conveyed in this pipeline is 3.616 m³/s.

Based on a friction coefficient of k = 0.6 mm and a DN1800 pipeline, the hydraulic gradeline at Command Reservoir No 2 will be 1,504 masl, which is much lower than the proposed full supply level of Command Reservoir No 2, i.e. 1,530 masl. In the event of installing a DN2000 pipeline, the hydraulic gradeline at Command Reservoir No 2 will be 1,491 masl.

In future, MMM may prefer to supply from Command Reservoir No 2 to Brandkop Reservoir, with a full supply level of 1,493 masl and located approximately 6 km further than Longridge Reservoir. The hydraulic gradeline at Command Reservoir No 2 for a DN1800 and DN2000 pipeline to Brandkop Reservoir will be 1,529 masl and 1513 masl, respectively.

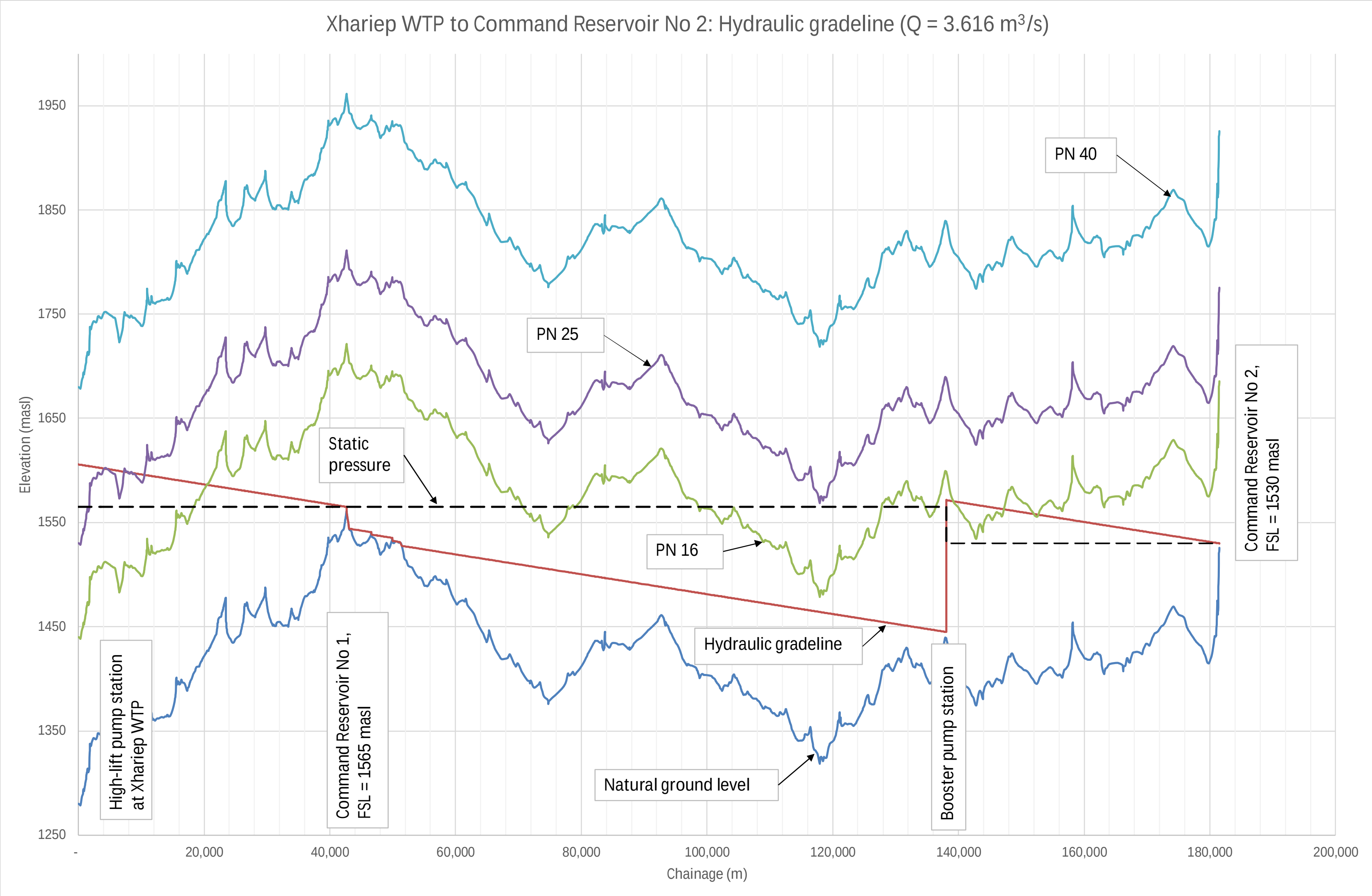


Figure 6-2: High-lift pump station to Command Reservoir No 2 – Hydraulic Gradeline

The minimum operating level of Command Reservoir No 2 is 1,520 masl. A DN2000 pipeline will therefore provide operational flexibility to supply directly to Brandkop Reservoir in future, should this be required.

Figure 6-3 shows the hydraulic gradeline from Command Reservoir No 2 to Longridge Reservoir for a flow of 3.616 m³/s conveyed in a DN2000 pipeline. The PN16 (160 m) working pressure line is also shown.

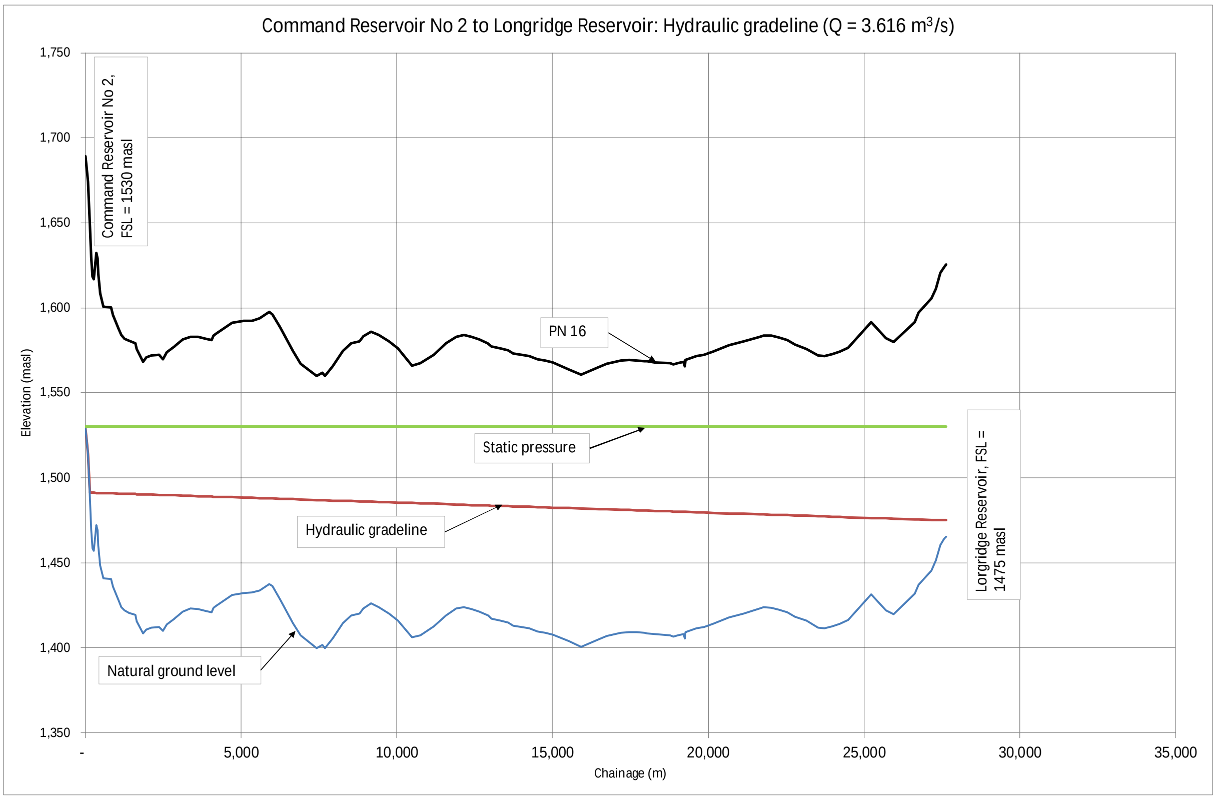


Figure 6-3: Command Reservoir No 2 to Longridge Reservoir – Hydraulic Gradeline

The maximum working that will be experienced along the pipeline is 121 m, which occurs under static conditions.

6.1.4 Command Reservoir No 2 to Rustfontein Water Treatment Plant

Command Reservoir No 2 will supply the existing clearwell reservoir at the Rustfontein WTP, with a full supply level of 1,368 masl, under gravity. The peak design flow that needs to be conveyed in this pipeline is 2.428 m³/s.

Based on a friction coefficient of k = 0.6 mm and a DN1400 pipeline, the hydraulic gradeline at Command Reservoir No 2 will be approximately 1,490 masl, which is much lower than the proposed full supply level of Command Reservoir No 2, i.e. 1,530 masl. In the event of installing a DN1200 pipeline, the hydraulic gradeline at Command Reservoir No 2 will be 1,587 masl, which is higher than the full supply level, i.e. the DN1200 pipeline is too small to convey the required flow rate. A DN1400 pipeline is therefore the correct pipeline size for the pipeline section from Command Reservoir No 2 to Rustfontein WTP.

Figure 6-4 shows the hydraulic gradeline from Command Reservoir No 2 to Rustfontein WTP for a flow of 2.428 m³/s conveyed in a DN1400 pipeline. The PN16 (160 m) working pressure line is also shown.

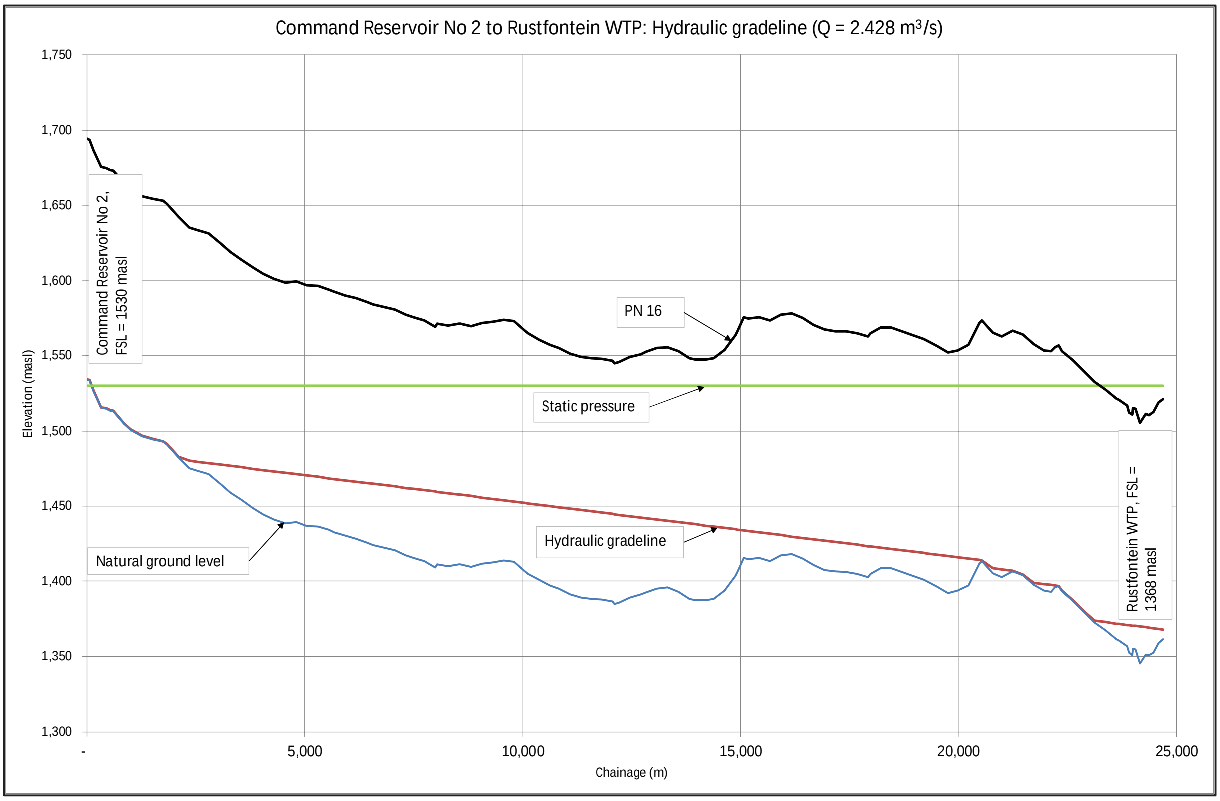


Figure 6-4: Command Reservoir No 2 to Rustfontein WTP – Hydraulic Gradeline

The maximum working that will be experienced along the pipeline is 185 m, which occurs under static conditions at the Rustfontein WTP. The pipeline can be PN16 rated from Chainage 0 to Chainage 22,500, with the last section from Chainage 22,500 to 24,700 to be PN25 rated.

6.2 Waterhammer Analysis

A preliminary waterhammer (i.e. surge) analysis was undertaken for the various pipeline components that could experience surge pressures caused by the stopping and starting of pumps, or the opening and closure of valves at the inlets to reservoirs that are gravity fed. Details of the waterhammer analysis, as well as the maximum and minimum surge pressures expected in each pipeline component, are described below. It is important to note that the waterhammer analysis would need to be repeated during the detailed design phase based on the final horizontal and vertical pipeline alignments.

6.2.1 Gariep Dam to Break Pressure Tank

A pump start time of 30 seconds was used as the pump sets are fitted with variable speed drives (VSDs), whereas an instantaneous pump trip (e.g. due to failure in power supply) was assumed.

In order to mitigate excessive waterhammer pressures during a pump trip event, it is recommended to install a non-return valve at chainage 4.1 km, or approximately 100 m upstream of the break pressure tank. With the non-return valve, the maximum waterhammer pressures are expected to reduce to 110 m. Vacuum pressures up to -10 m are still anticipated for the last ± 500 m which must be considered for the pipeline's structural design.

6.2.2 High-Lift Pump Station to Command Reservoir No 1

A pump start time of 2 seconds was used as the motors will be started direct-on-line (DOL), whereas an instantaneous pump trip (e.g. due to failure in power supply) was assumed.

To mitigate the excessive waterhammer pressures during a pump trip scenario, it is recommended to install a non-return valve at chainage 38.5 km on the pipeline. With the non-return valve, the maximum waterhammer pressures are expected to reduce to 343 m. Vacuum pressures of up to – 10 m are still anticipated from chainage 20 km, which must be considered for the pipeline’s structural design.

6.2.3 Command Reservoir No 1 to Booster Pump Station

An analysis was conducted to determine the waterhammer pressures that would be generated when a valve is closed at the inlet of the booster pump station’s suction reservoir. This analysis assumed the installation of a DN1400 butterfly valve at the reservoir's inlet. It was established that the valve closure time for the last 15% of the valve should not be less than 10 minutes. This will result in maximum waterhammer pressures of up to 276 m.

6.2.4 Booster Pump Station to Command Reservoir No 2

A pump start time of 2 seconds was used as the motors will be started direct-on-line (DOL), whereas an instantaneous pump trip (e.g. due to failure in power supply) was assumed.

To mitigate the excessive waterhammer pressures during a pump trip event, it is recommended to install a non-return valve at chainage 43 km on the pipeline. With the non-return valve, the maximum waterhammer pressures are expected to reduce to 187 m. Vacuum pressures of up to – 10 m are still anticipated for sections of the pipeline, which must be considered for the pipeline’s structural design.

6.2.5 Command Reservoir No 2 to Longridge Reservoir

This analysis determined the waterhammer pressures that would be generated when a valve is closed at the inlet of the Longridge reservoir. This analysis assumed the installation of a DN1600 butterfly at the reservoir's inlet. It was established that the valve closure time for the last 15% of the valve should be 5 minutes or longer. This will result in maximum waterhammer pressures of up to 140 m.

6.2.6 Command Reservoir No 2 to Rustfontein Water Treatment Plant

This analysis determined the waterhammer pressures that would be generated when a valve is closed at the inlet of the Rustfontein Water Treatment Plant. This analysis assumed the installation of a DN1000 butterfly at the WTP inlet. It was established that the valve closure time for the last 15% of the valve should be 10 minutes or longer. This will result in maximum waterhammer pressures of up to 190 m.

6.3 Design and Field Test Pressures

6.3.1 Design Pressures

DWS recommends that the design pressure should be taken as the maximum pressure to which the pipeline would be subjected under working, static and surge conditions.

Figure 6-5 shows the maximum working and surge pressures for the pipeline section from Gariep Dam to the proposed Xhariep WTP. It is evident from Figure 6-5 that the surge pressures will dictate the maximum design pressure. The maximum design pressure along this section of the pipeline is 110 m, which occurs immediately downstream of the raw water pump station.

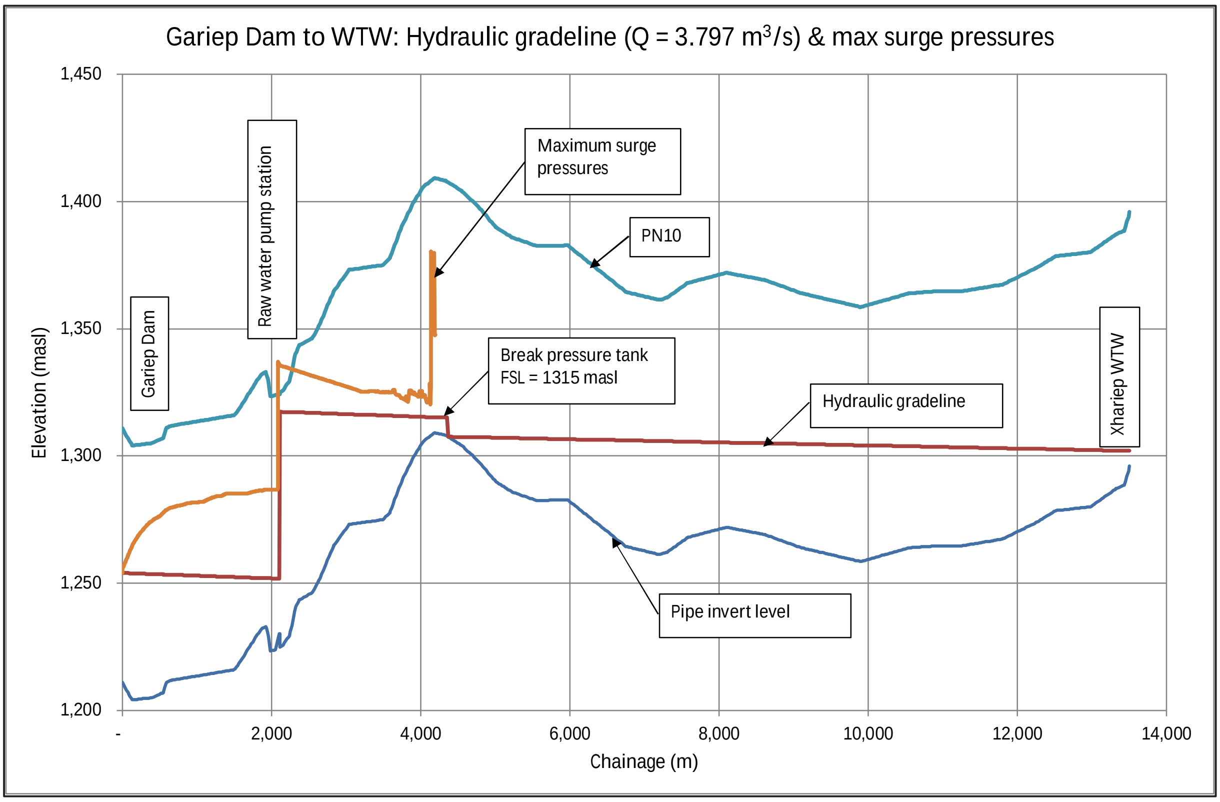


Figure 6-5: Gariep Dam to Xhariep WTP – Maximum working and surge pressures

Figure 6-6 shows the maximum working, static and surge pressures for the pipeline section from the high-lift pump station to Command Reservoir No 2. It is evident from Figure 6-6 that the surge pressures dictate the maximum design pressure. The maximum design pressures along the sub-sections of the pipeline are as follow:

- High-lift pump station to Command Reservoir No 1 = 377 m immediately downstream of the high-lift pump station;
- Command Reservoir No 1 to suction reservoir at booster pump station = 276 m, approximately 76.4 km downstream of Command Reservoir No 1; and
- Booster pump station to Command Reservoir No 2 = 195 m, immediately downstream of the booster pump station.

Figure 6-7 shows the maximum working, static and surge pressures for the pipeline section from Command Reservoir No 2 to Rustfontein WTP. It is evident from Figure 6-7 that the surge pressures will dictate the maximum design pressure. The maximum design pressure along this section of the pipeline is 203 m, which occurs just upstream of Rustfontein WTP.

Figure 6-8 shows the maximum working, static and surge pressures for the pipeline section from Command Reservoir No 2 to Longridge Reservoir. It is evident from Figure 6-8 that the surge pressures will dictate the maximum design pressure. The maximum design pressure along this section of the pipeline is 140 m, which occurs approximately 16 km downstream of Command Reservoir No 2.

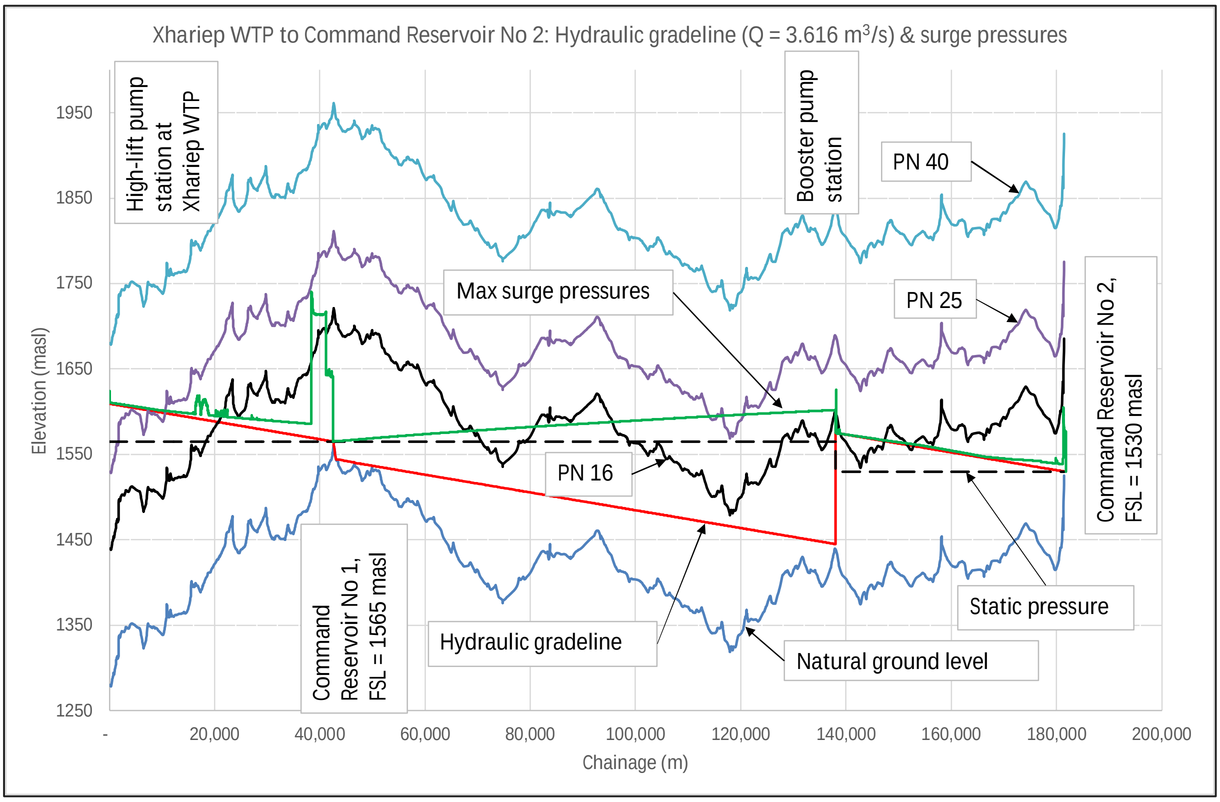


Figure 6-6: High-lift pump station to Command Reservoir No 2 – Maximum working and surge pressures

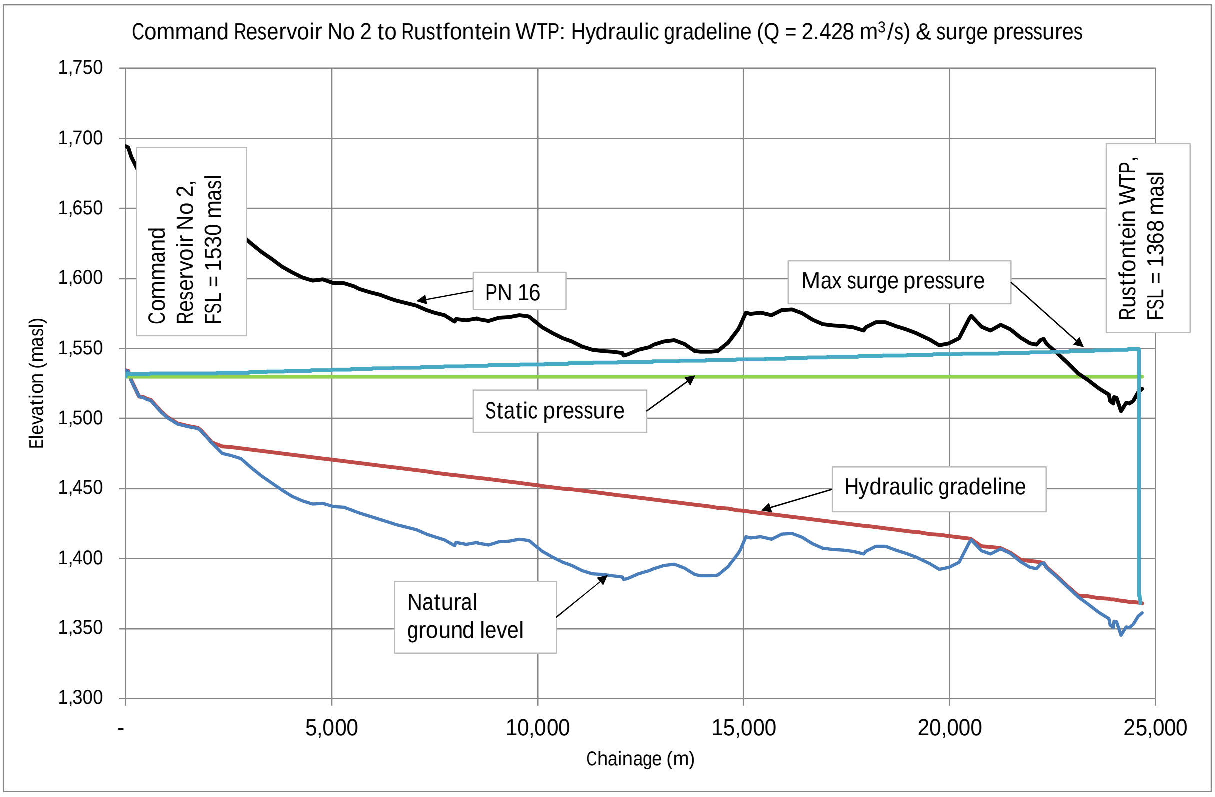


Figure 6-7: Command Reservoir No 2 to Rustfontein WTP – Maximum working and surge pressures

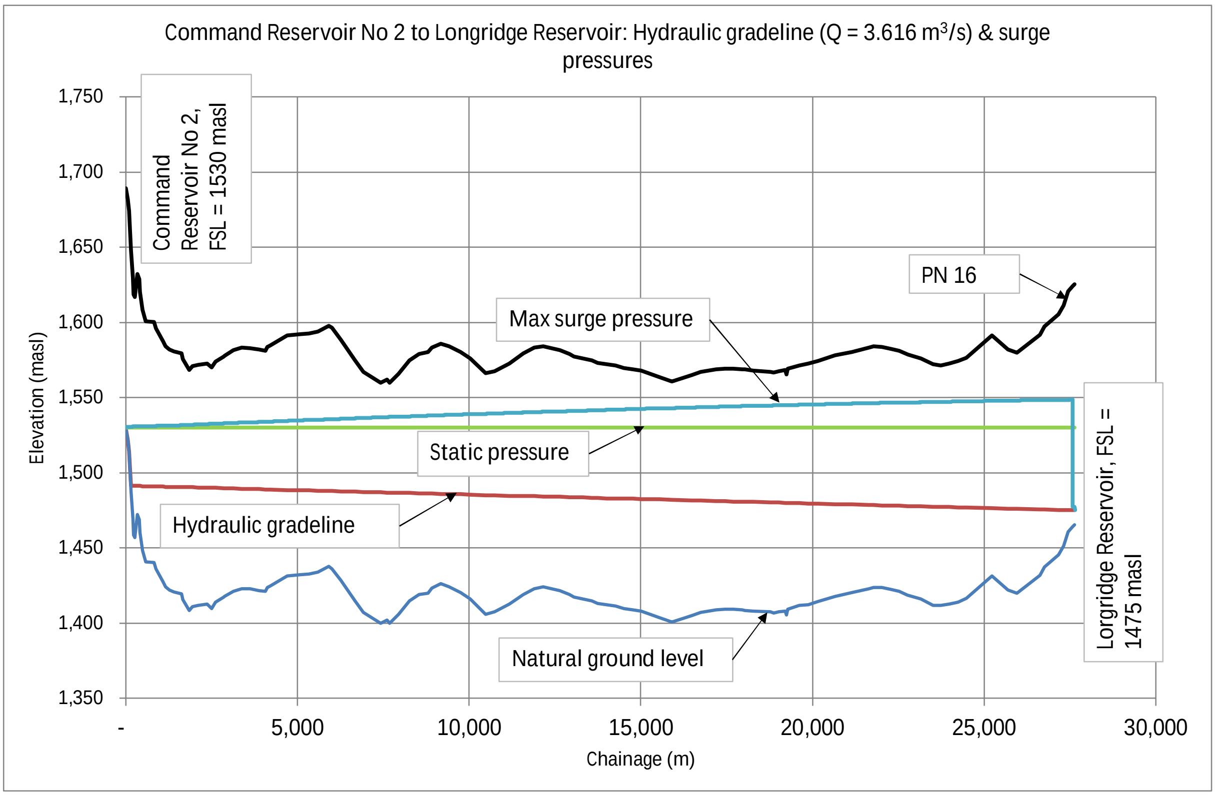


Figure 6-8: Command Reservoir No 2 to Longridge Reservoir – Maximum working and surge pressures

6.3.2 Field Test Pressures

DWS1110 states that "Test pressures will generally be 1.25 times the pipeline design pressure for design pressures up to and including 3.2 MPa and 1.1 times the design pressure for higher pressures."

The DWS further recommends that the pressure rating of the valves be determined such that the field test pressure does not exceed the valve's rating (e.g. a PN 40 valve must not be subjected to a test pressure exceeding 400 m).

The above criteria were adopted in determining the maximum field test pressures in each pipeline section as shown in Table 6-2.

Table 6-2: Maximum design and field test pressures

Pipe section	Pipe diameter (mm)	Maximum design pressure (m)	Maximum field test pressure (m)	Maximum pressure rating of valves, specials, etc. (m)
Gariep Dam to Xhariep WTP	1800	110	138	160
	2000	110	138	160
High-lift pump station to Command Reservoir No 1	1800	377	415(1)	400
Command Reservoir No 1 to suction reservoir at booster pump station	1800	276	345	400
Booster pump station to Command Reservoir No 2	1800	195	244	250
Command Reservoir No 2 to Rustfontein WTP	1400	203	254	250
Command Reservoir No 2 to Longridge Reservoir	2000	140	175	250

(1) It is proposed that measures be evaluated during the detailed design phase to reduce the maximum field test pressure to not exceed 400 m in order to install PN40 fittings.

7 Pump Station Design

7.1 Building Design

Three main pump stations will be constructed as part of the Xhariep Project, namely:

- Raw water pump station – supplying raw water from Gariep Dam to the proposed Xhariep WTP
- High-lift pump station – supplying potable water from the Xhariep WTP to Command Reservoir No 1
- Booster pump station – supplying potable water from Command Reservoir No 1 to Command Reservoir No 2

Figure 7-1, Figure 7-2 and Figure 7-3 shows a 3D view, a sectional view and a plan view of the raw water pump station.

Figure 7-4, Figure 7-5 and Figure 7-6 shows a 3D view, a sectional view and a plan view of the high-lift pump station.

Figure 7-7, Figure 7-8 and Figure 7-9 shows a 3D view, a sectional view and a plan view of the booster pump station.

The detailed drawings for the pump station and pipework layouts are included in Appendix B.

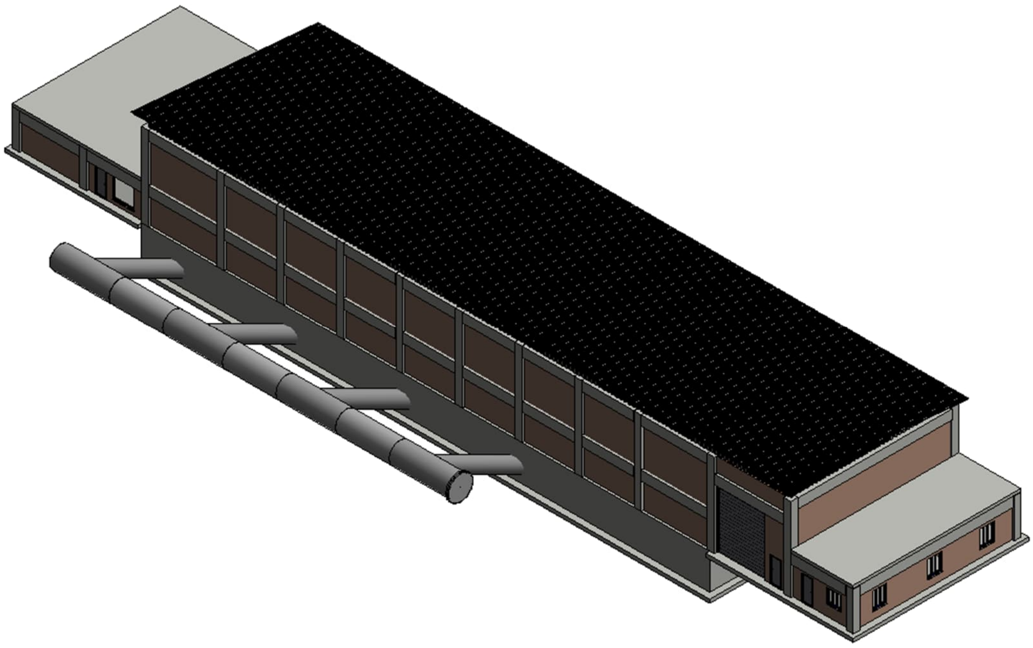


Figure 7-1: Raw water pump station – 3D view

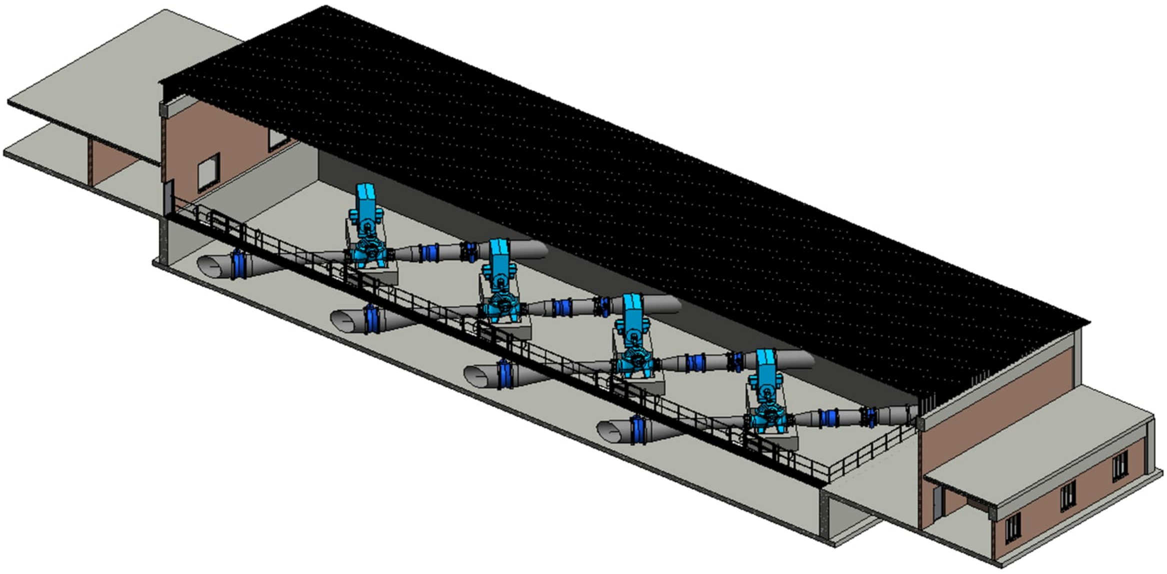


Figure 7-2: Raw water pump station – Sectional view

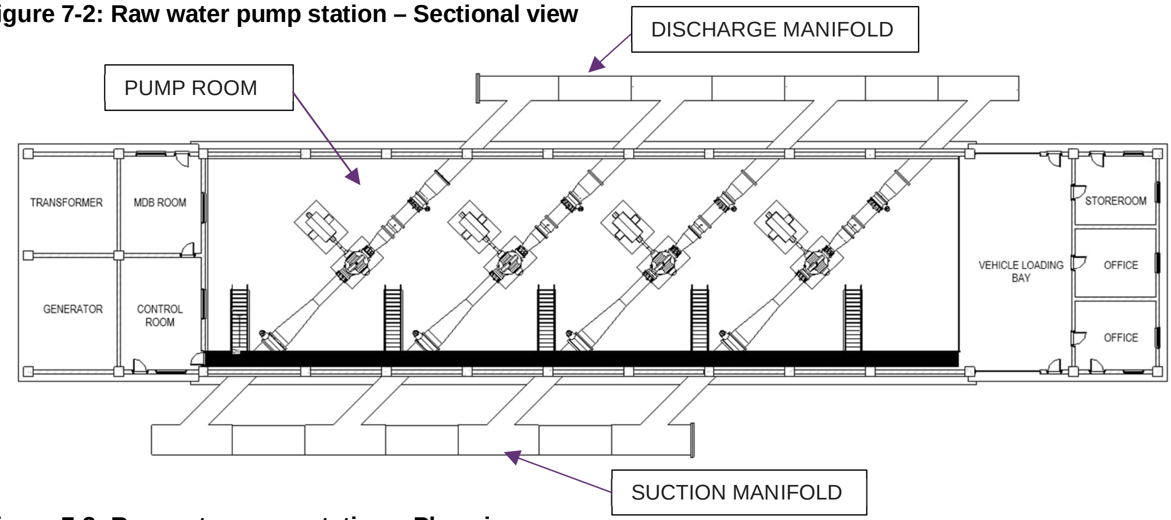


Figure 7-3: Raw water pump station – Plan view

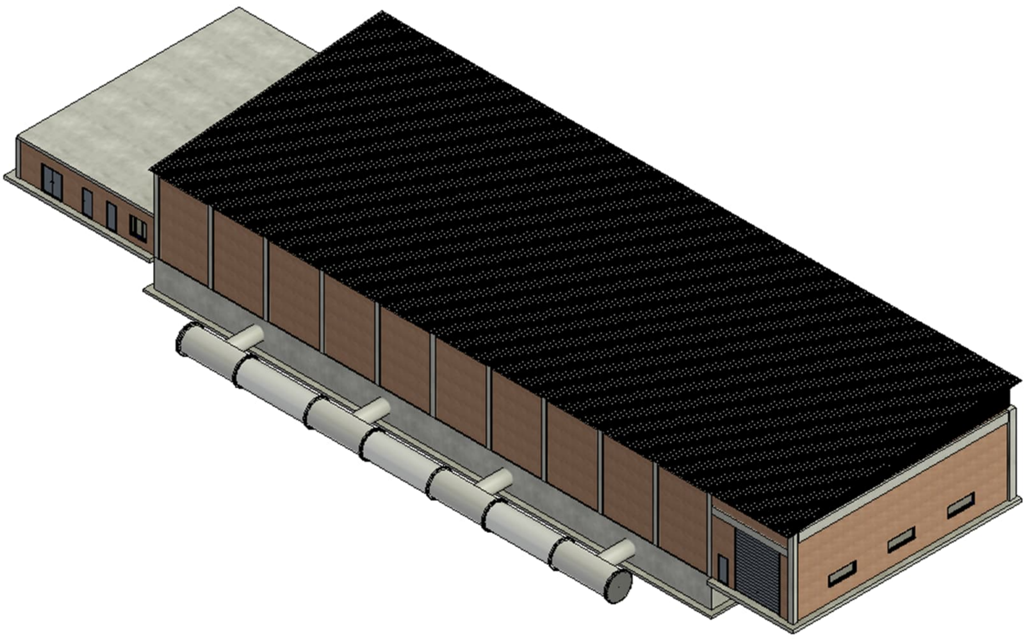


Figure 7-4: High-lift pump station – 3D view

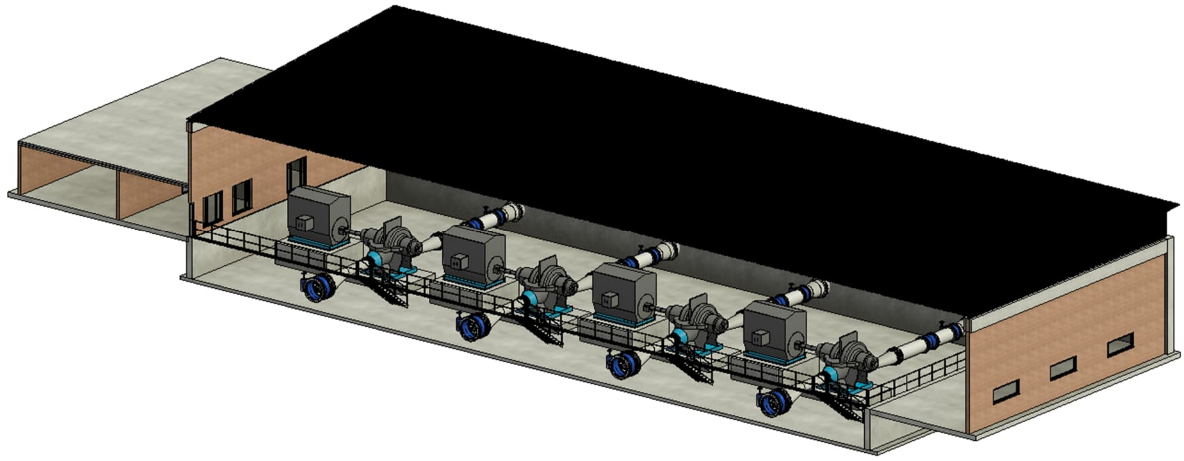


Figure 7-5: High-lift pump station – Sectional view

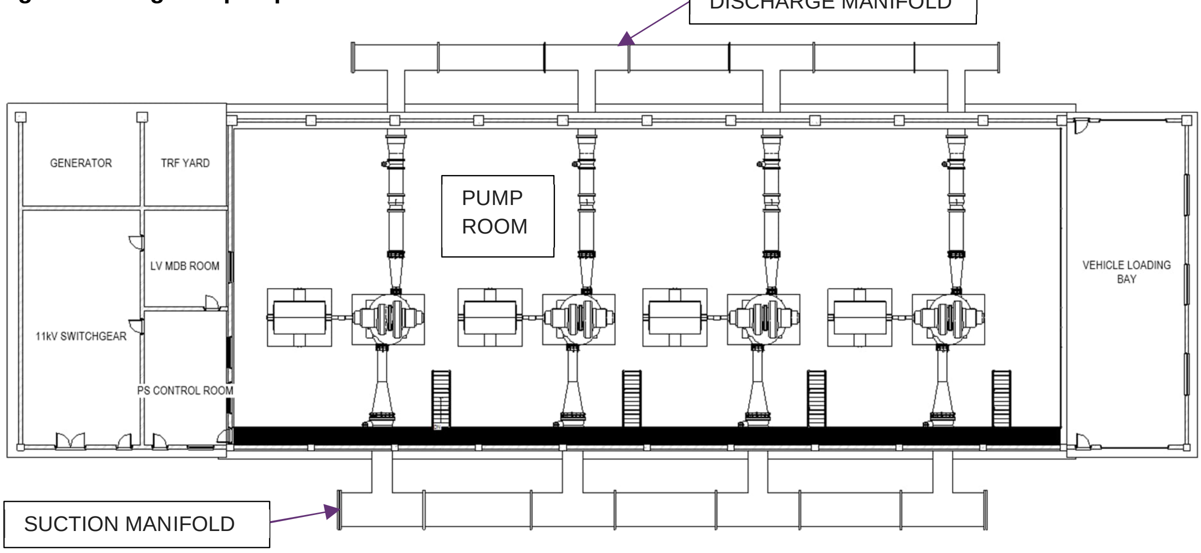


Figure 7-6: High-lift pump station – Plan view

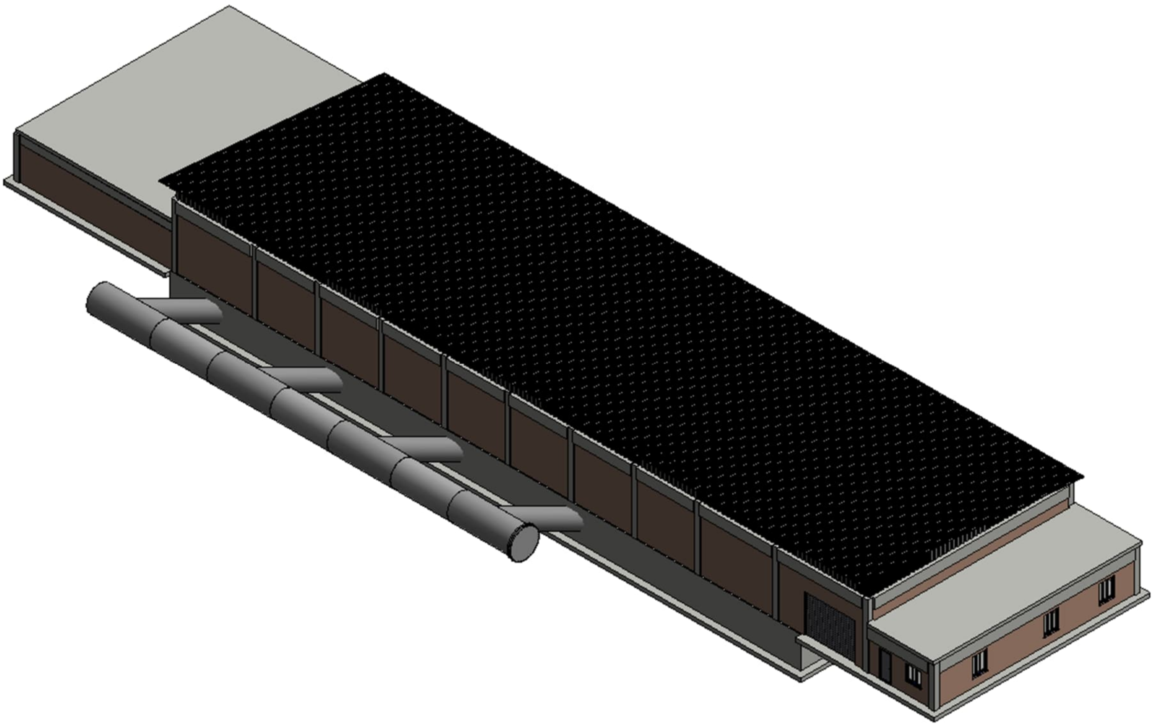


Figure 7-7: Booster pump station – 3D view

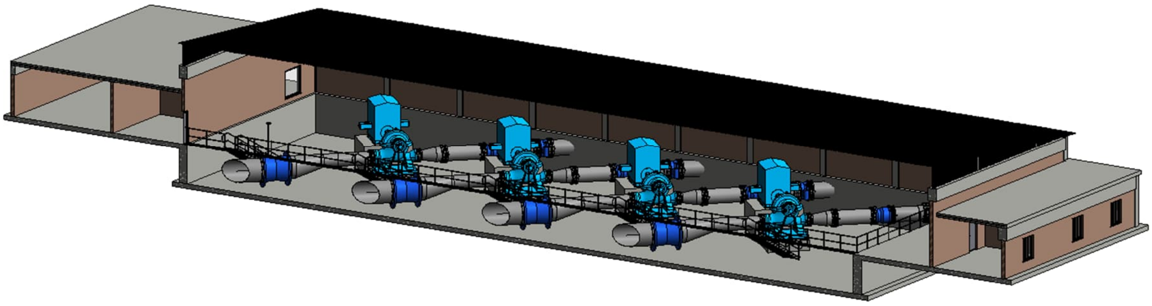


Figure 7-8: Booster pump station – Sectional view

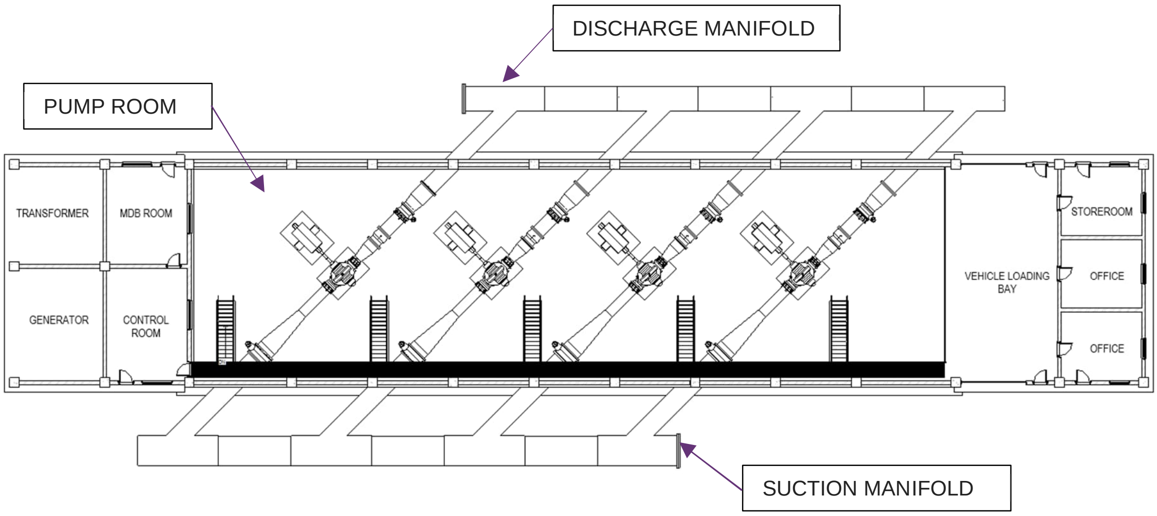


Figure 7-9: Booster pump station – Plan view

Each of the pump stations consist of a pump room, loading bay for vehicle access, storage room, offices, control room, medium voltage (MV) distribution room, transformer room and a generator room. No provision was made for storage rooms or offices at the high-lift pump station as staff will be based at the water treatment plant.

The pump room contains the pumping assembly with the suction and discharge manifolds located outside the pump station building. The pump floor is sloped to a drainage canal which naturally drains to the outside of the pump station at the raw water pump station, whereas a drainage pump is required at the high-lift and booster pump stations.

The loading bay provides vehicle access into the pump station with a roller shutter door on both sides of the loading bay. The vehicle drives in through one door and exits the pump station through the second door.

The storage room allows for the storage of spare parts and tools.

The offices provide space for administrative work to be performed although the pump stations will not be permanently manned.

The control room contains the SCADA, control desk and PLCs. The pump station is controlled from the control room and can also be remotely controlled from the WTP.

Access to the steel walkway inside the pump station is provided from a single door next to the storage room or from the loading bay. The single door is the main entry door for operator access into the pump station.

Access to the pump room or pump bay is provided by a steel walkway extending from the loading bay to the opposite end of the pump station. The walkway also provides access to the electrical rooms.

Access to the MV Switchgear room is also provided from the outside of the pump station through a double door to allow for installation of the MV Switchgear and LV distribution panels. A double door between the MV Switchgear room and LV distribution room allows for installation of the LV distribution panels.

The pump station comprises a concrete structure (floor, columns and beams) with brick infill. The pump station has a steel roof structure with steel roof sheeting, whereas the electrical rooms have a concrete roof structure. The option to install translucent roof sheeting for natural light into the pump room should be evaluated during the detailed design phase. Due to security reasons, no windows are provided.

Ventilation equipment will be located on the concrete roof structure of the electrical rooms to provide forced ventilation into the pump station building. Access to the equipment will be provided by means of an external cat ladder, with access restricted by a steel security gate or similar at the ladder.

The height of the pump station roof is determined by the required height of the overhead travelling crane above the loading bay area.

Access to the overhead travelling crane is provided by a maintenance platform at the loading bay.

7.2 Mechanical Design

7.2.1 Pump Duties and Pump Selection

The pump duties and pump selections for the pump stations are discussed in Sections 5.1 and 5.3.

7.2.2 Pipework Layouts

Figure 7-10 and Figure 7-11 show the overall pipework arrangement for the raw water pump station, as well as the layout of pipework layout of a single pump leg.

Figure 7-12 and Figure 7-13 show the overall pipework arrangement for the high-lift pump station, as well as the layout of pipework layout of a single pump leg.

Figure 7-14 and Figure 7-15 show the overall pipework arrangement for the booster pump station, as well as the layout of pipework layout of a single pump leg.

The sizing of the pipework is discussed in Section 7.2.3.

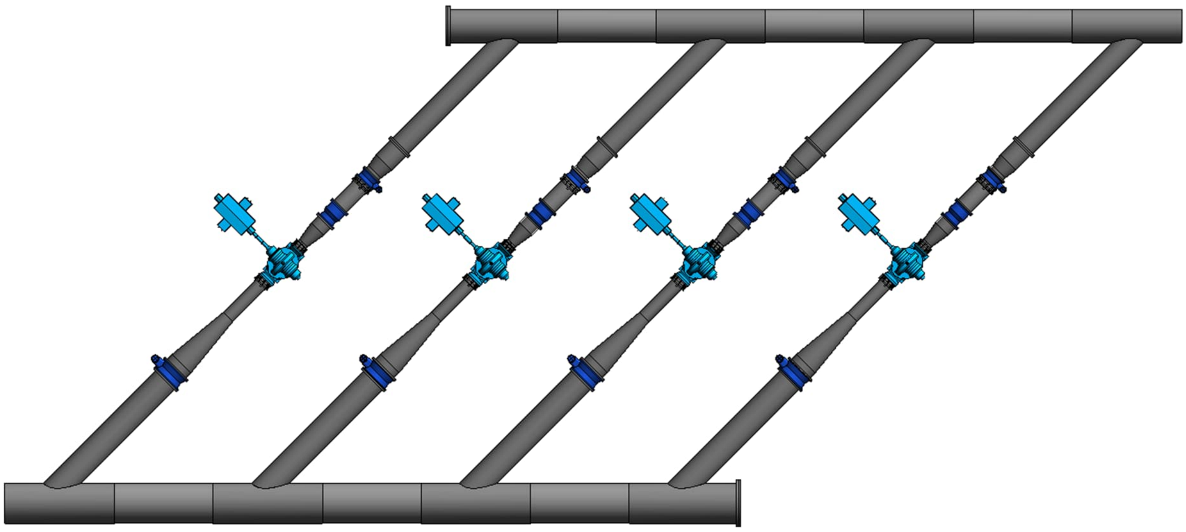


Figure 7-10: Raw water pump station – overall pipework layout

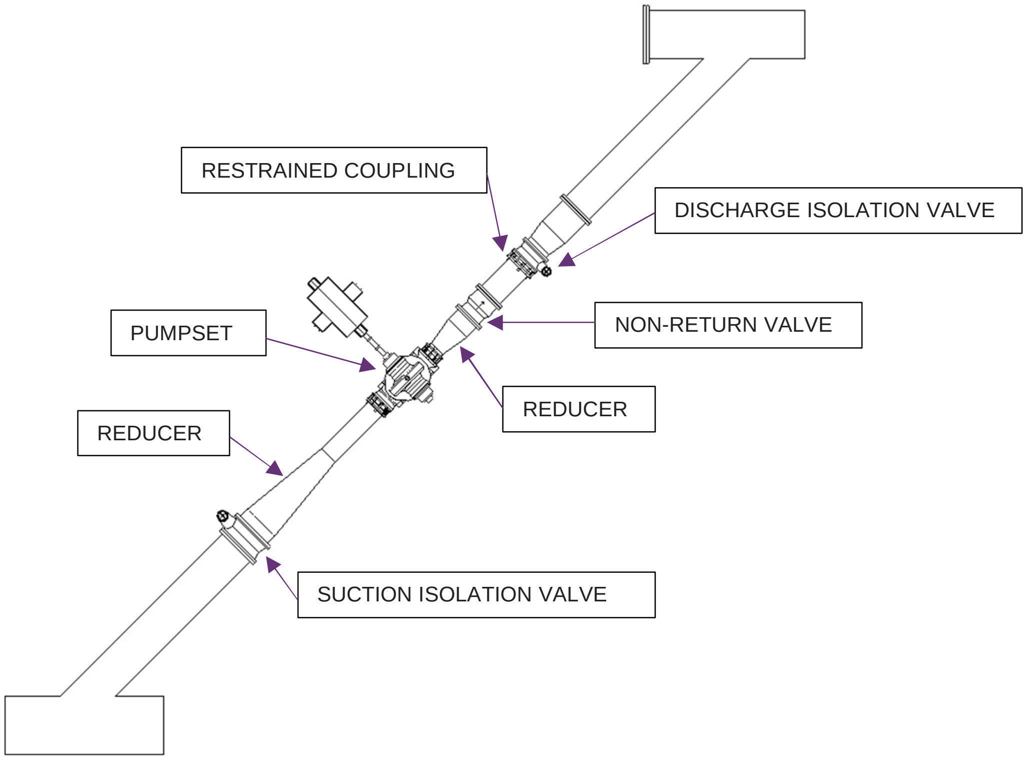


Figure 7-11: Raw water pump station – single pump leg plan view

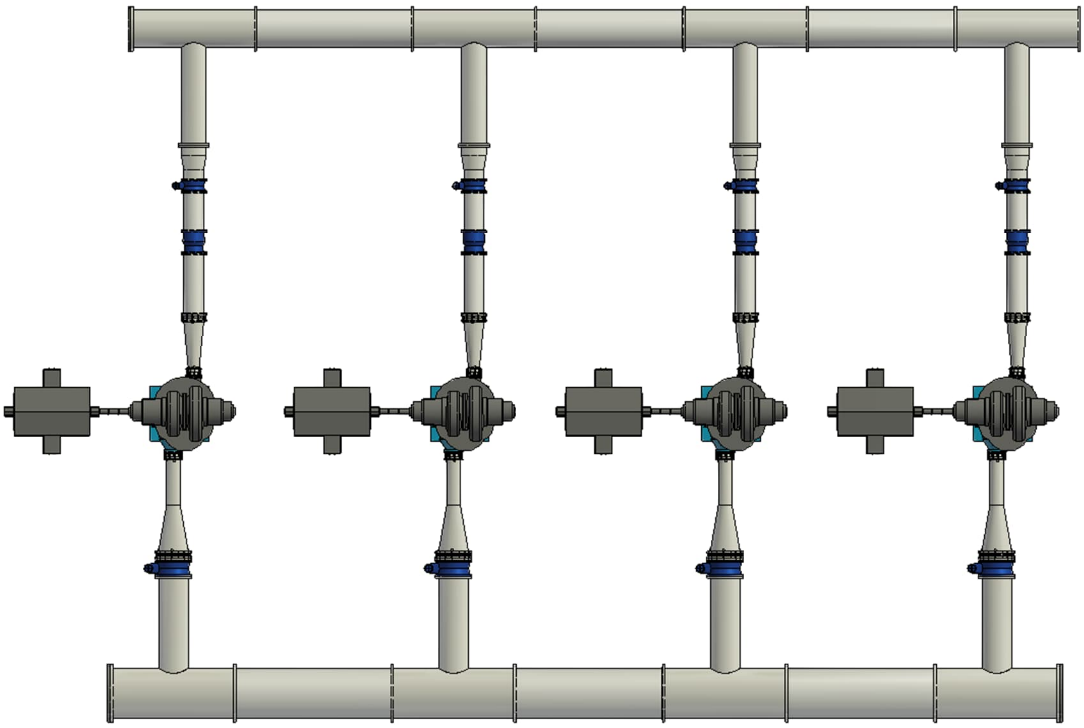


Figure 7-12: High-lift pump station – overall pipework layout

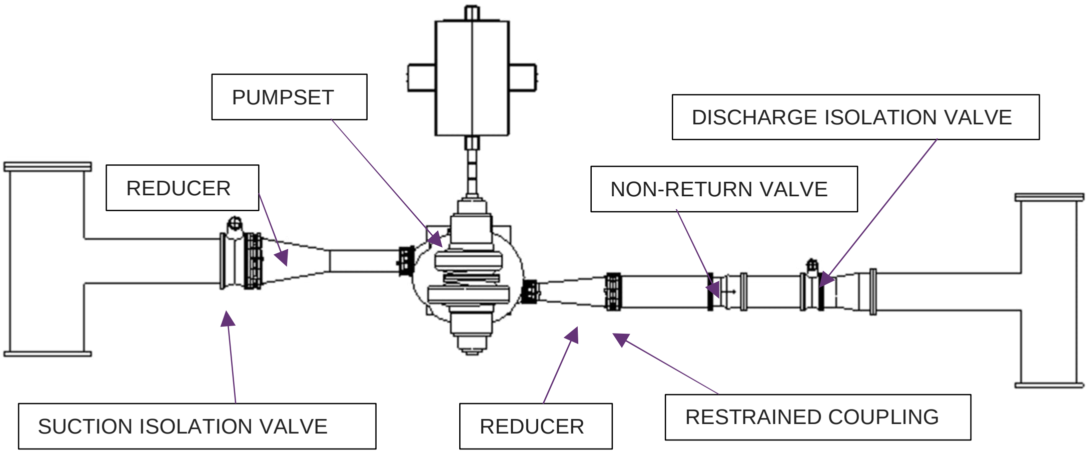


Figure 7-13: High-lift pump station – single pump leg plan view

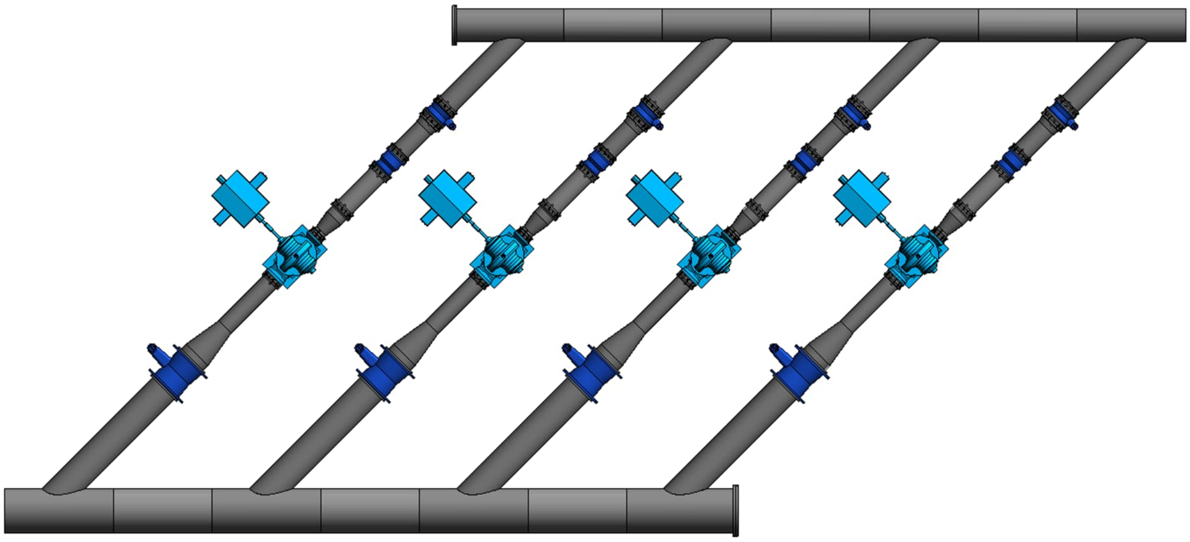


Figure 7-14: Booster pump station – overall pipework layout

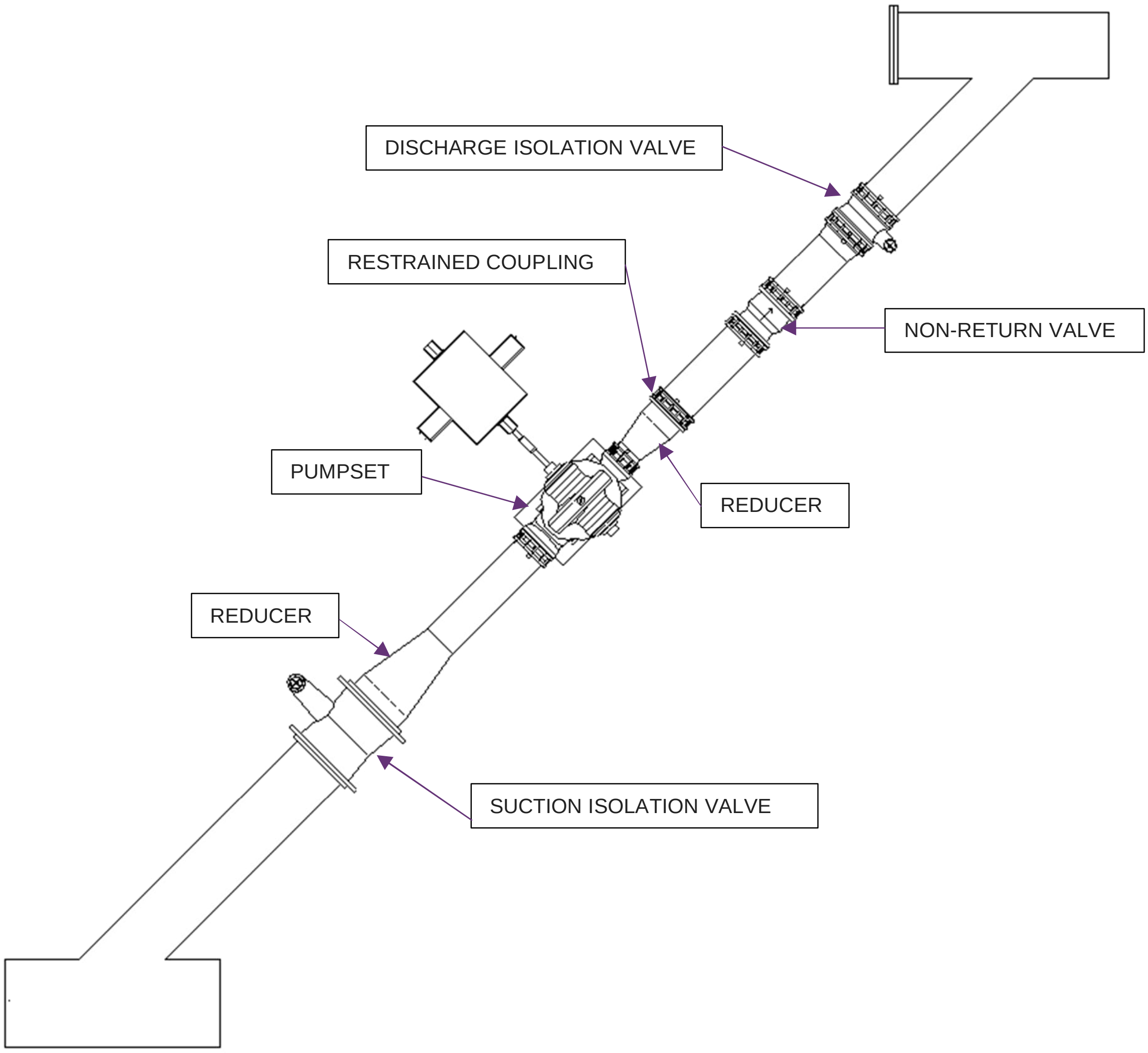


Figure 7-15: Raw water pump station – single pump leg plan view

7.2.3 Pipework Sizing

The pipework layouts and sizing are based on the American National Standard for Rotodynamic Pumps for Pump Piping (ANSI/HI 9.6.6-2009) and the American National Standard for Rotodynamic Pumps for Pump Intake Design (ANSI/HI 9.8-2018).

The pipe diameters within the pump stations are designed for suction and discharge velocities of approximately 1 – 1.5 m/s and 1.5 – 2.5 m/s, respectively. The relevant design flows, calculated pipe diameters and design pressures for each of the pump stations are shown in Table 7-1.

Table 7-1: Pipe diameters and pressure ratings at pump stations

Design parameter	Raw water pump station	High-lift pump station	Booster pump station
Maximum pump station flow (m^3/s)	4.3	3.9	4.1
Maximum flow per pump (m^3/s)	1.6	1.5	1.85
Minimum flow per pump (m^3/s)	1.0	1.2	1.2
Suction manifold diameter (mm)	2000	1900	1900
Suction manifold velocity (m/s)	1.37	1.38	1.45

Design parameter	Raw water pump station	High-lift pump station	Booster pump station
Suction pipework diameter (mm)	1200	1200	1300
Suction pipework velocity (m/s)	1.41	1.33	1.39
Suction pipework pressure rating	PN10/PN16	PN10/PN16	PN10/PN16
Discharge pipework diameter (mm)	1000	900	1000
Discharge pipework velocity (m/s)	2.04	2.36	2.36
Discharge manifold diameter (mm)	1500	1500	1500
Discharge manifold velocity (m/s)	2.43	2.21	2.32
Discharge pipework pressure rating	PN10/PN16	PN40	PN16
Minimum spacing for parallel pumps between suction lines (mm)	4000	3800	3800

Two of the pump stations, the high-lift pump station and the booster pump station, draw water from suction reservoirs. The operating levels in the reservoirs must be such to prevent the formation of vortices, which could result in air being introduced that causes cavitation and damage to the pumps.

The suction and discharge reducers have been designed based on the guidelines provided in the Pump Handbook, 3rd edition (Karrassik, et al.).

Structural reinforcing of pipe specials (e.g. wrappers, collars and crotch plates) should be designed based on the provisions of American Water Works Association (AWWA) M11: Steel Pipe – A Guide for Design and Installation (5th edition).

The pipework will be flanged, with flanges that will comply with SANS 1123 or BS EN 1092.

Flange adaptors, with restraints where necessary, will be provided in strategic positions to cater for misalignment and provide flexibility for removal of items for future maintenance.

Pipework will be horizontally mounted, and all hydraulically created thrust forces will be restrained by steel supports anchored to concrete plinths integral to the floor slab. The location of the pipe supports and concrete plinths needs to be finalised as part of the detailed design phase.

7.2.4 Pipework Materials

Pipework within the pump stations will be of Grade S355JR mild steel and be epoxy coated internally and externally. The wall thickness of pipe specials within the pump stations should not exceed a pipe diameter to wall thickness ratio of 100.

Flanges and pipe supports will be of the same material as the pipe.

7.2.5 Isolation Valves

Isolation butterfly valves, of the double eccentric type, are provided on the suction and discharge side of each pump. The valves will be isolated to remove the pump or non-return valve if maintenance is required.

The pressure ratings of the isolation butterfly valves are shown in Table 7-1. The pressure rating of the isolation butterfly valves on the discharge side of the pumps should be able to withstand the full shutoff head of the pump.

The isolation valves will be open under normal operating conditions. These valves will not be operated on a regular basis and will generally be used for maintenance purposes. Therefore, these valves will be manually operated by handwheel.

7.2.6 Non-Return Valves

Fast-closing non-return valves (e.g. Noreva or similar) are proposed at all the pump stations to mitigate the risk of excessive surge pressures during a pump stop or pump trip event. These valves are designed with a short travel and spring-loaded to prevent the occurrence of reverse flow.

Typically, a minimum of 4 x pipe diameter is required upstream of the non-return valve and 2 x pipe diameter downstream of the non-return valve to ensure optimal functioning of the non-return valve. The springs are also designed based on the expected velocity range at the valve.

The design was based on a velocity range of 2 – 3 m/s, which allows the upstream and downstream straight lengths to be reduced to 3 x the pipe diameter and 1.5 x pipe diameter, respectively. These requirements need to be verified during the detailed design with the valve manufacturers, especially should the valve type be changed.

7.2.7 Ventilation

Heat will be generated in each of the pump stations by the operating pump motors. An approximate 3% loss in motor efficiency is generally applied to compute the heat dissipated into the pump room. The air flow is calculated to limiting the temperature rise to 5 degrees.

The pump station will be force-ventilated by one fan. The supply air will be provided at a low level and the heated air in the pump room will exit via the roof ventilators or ventilators constructed in the walls just below the roof. The fan will be provided with standard upstream and downstream attenuators in order to limit noise generation.

The fan and attenuators will be installed outside the building on top of the electrical room’s concrete roof and be situated at a high level. This high level installation will minimise the amount of dust drawn into the pump station. Dust filters will, however, be provided as air from the pump room will be used to ventilate the control and switch rooms. Regular maintenance of these filters will be required.

The fan will operate continuously in order to provide positive air pressure within the pump station and this will prevent the windblown ingress of dust or foreign particles through the weather louvres. Should it be decided to only operate the fan when the pumps are operating, smaller fans would need to be installed to ventilate the control and switch rooms.

7.2.8 Lifting Equipment

An overhead travelling crane will be provided at each of the pump stations, travelling the full pump station footprint. The crane and hoist will be electrically powered.

The safe working load (SWL) will be 1.3 times the weight of the pump set, i.e. the total weight of the pump, motor, base frame and couplings as one unit.

A fixed steel ladder will be provided at a suitable position within the building for accessing the platform on the crane.

After installation and testing, load testing must be conducted at intervals not exceeding 12 months in accordance with the Occupational Health and Safety Act and Regulations.

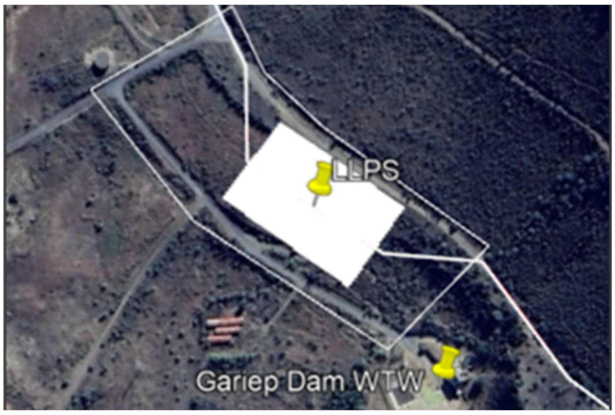
7.3 Electrical Design

7.3.1 Bulk Electrical Supply

The Eskom Free State and Eskom Northern Cape offices were contacted to assess the availability of power for each of the infrastructure sites, and to determine the upgrades/infrastructure required to provide power at each site.

Table 7-2 provides a summary of the medium and high-voltage infrastructure required to supply each of the sites.

Table 7-2: Summary of HV and MV supply arrangement to each site

Site	Image and Coordinates	Medium and high-voltage scope of works
Low-lift Pump Station (LLPS)	 Latitude = 30°37'29.76"S Longitude = 25°29'6.97"E	• Closest Substation from Eskom is Oranjekrag where Eskom can add two new 22kV feeder bays. • DWS to install two 800 meter overhead powerlines or buried cables for the new LLPS. • At LLPS, install two incomer feeder bays, busbar section and two 22/11kV 5MVA transformers bays with a 11kV Indoor Circuit Breaker Building.
Water Treatment Works and High-lift Pump Station (HLPS)	 Latitude = 30°32'30.95"S Longitude = 25°30'58.52"E	• New 132kV Loop-In Loop-Out from the Ruigtevallei-Valley Dora 132kV Line with Eskom Switching Station. • Two 132kV incomer feeders and two 132kV out-going feeder bays including busbar section. • Firm supply 30MVA 132/11kV • Eskom 11kV Indoor Switchgear building substation, complete with metering protection, earth grid and 11kV Circuit Breaker Room for supply to the WTP and HLPS.

Site	Image and Coordinates	Medium and high-voltage scope of works
Command Reservoir 1	 Latitude = 30°15'2.05"S Longitude = 25°44'17.71"E	• Build 800m 11kV overhead Line from Pole SPEL 5-24-3 on Elstow 11kV Line to reservoir position. • Install 25kVA 11/0.4kV Pole mounted Transformer with metering kiosk.
Booster Pump Station	 Latitude = 29°30'53.25"S Longitude = 26° 5'14.73"E	• Two 66kV feeder bays and possible busbar reconfiguration at Reddersburg Substation. • Build new 66kV Double Circuit Overhead Line, 18km long. • Add new 66/11kV 10 MVA substation complete with MV and LV switchgear.
Command Reservoir 2	 Latitude = 29°19'17.44"S Longitude = 26°23'4.76"E	• Build 500m 22kV Paradys overhead line (OHL) from Lovedale 22kV OHL to reservoir position. • Install 25kVA 11/0.4kV pole mounted transformer with metering kiosk.

The applications for the power supply to the various infrastructure sites must be submitted to Eskom during the detailed design phase of the project, whereafter Eskom must undertake the design and construction of the electrical infrastructure. Only then will the supply arrangement to each site be confirmed and the relevant on-site Eskom yard and substation details be finalised by the Eskom design team.

All bulk electrical supply infrastructure is anticipated to become property of Eskom and will be designed based on Eskom specifications and standards.

7.3.2 11 kV Motor Control Switchboard

An 11 kV motor control switchboard comprising two incomer circuit-breaker panels, bus-section breaker and four motor feeder/starter panels will be provided in a dedicated room at each pump station. Two feeder panels will also be provided for the supply of the local pump station / WTP 11/0.4 kV transformers to supply LV power.

For the Raw Water Pump Station, the pump feeder panels will be feeder units to the medium voltage (MV) variable frequency converter (VFC) panels (housed in a different room), while for the High-Lift Pump Station and Booster Pump Station, these panels will be fitted with direct-on-line (DOL) motor starters.

The switchboard will comprise conventional metal-enclosed switchgear and will be specified for compliance with:

- SANS 1885: AC metal-enclosed switchgear and controlgear for rated voltages above 1kV and up to and including 36kV
- (DWS standard specification for medium voltage equipment

The following specific requirements will also be specified:

- Insulation medium : Air
- Incomer CB type : Vacuum
- Incomer protection : Overcurrent, earth fault and arc detection
- Motor starter type : Contactor
- Starter isolation : Switch-disconnector/withdrawable contactor
- Starter protection : HRC fuses and motor protection relay
- Short-time current rating : 31.5kA kA for 3 seconds
- Insulation levels : 95kV (BIL) and 28kV (power frequency)
- Internal arc classification : AFLR 25kA 1 second
- Cable terminations : Bolted with heat-shrink insulation
- Control of starters : Local/remote and auto/manual

7.3.3 Power Factor Correction and Capacitors

Either Capacitor banks or dynamic power factor correction (DPFC) will be provided at the high-lift pump station and booster pump station MV busbar to assist with voltage stabilisation during startup of the DOL motors.

Power factor correction (PFC) will be provided to reduce electricity demand charges and losses upstream of the PFC. The simplest and most economical form of PFC for pump stations with few large loads (e.g. main pumps) is distributed PFC by way of capacitors directly connected to motor feeder circuits.

Each motor feeder circuit will therefore be equipped with capacitors located at either the motors or the motor starters. For the 11 kV main pump motors, the capacitors will be located at the motors. For low voltage (LV) motors, the capacitors will be directly connected to the starter contactor.

To avoid self-excitation of motors, PFC capacitors will be sized to provide 90% of the motor no-load reactive power draw.

7.3.4 MV VFCs

The MV VFCs will either be of the active front end (AFE) or multi-level H-bridge type, to limit harmonics generated from the pump station to the Eskom network. The harmonic filter design will be developed to meet the harmonic network requirements provided by Eskom during detail design.

VFC panels will be in a separate room in the raw water pump station and use air for cooling. Forced ventilation will be provided into the room via fans and ducted out the room by ducting from the top of the VFC panels. Mesh filters will be provided to the room fans and VFC panel doors based on the dust in the area.

A MV contactor panel and busbar rising panel will be provided adjacent to each VFC panel to ensure the VFC can be isolated when not in use of when a pump E-stop is pressed.

7.3.5 LV Main Distribution Board

A 400V main distribution board (MDB) will be provided to house the switchgear and control gear for all LV motor loads and to provide power to the building electrical services DB.

The MDB will be specified for compliance with SANS 1973 Low-voltage switchgear and control gear assemblies.

The MDB (or 'MCC' at the WTP) will be designed and specified to meet the following main specific requirements:

- Type : Floor-standing
- Access : Front and back
- Cable entry : Bottom
- IP ratings : IP 44 (doors closed)
  : IP 2 x (components inside with doors open)
  : IP 3 x (between compartments)
- Form of separation : Form 3b
- Material : Mild steel / Aluzinc
- Corrosion protection : Epoxy-coated

The MDB will be installed in the LV MDB / MCC room at each pump station or WTP.

7.3.6 MV and LV Cables

MV cables will be 3-core copper conductor XLPE cables with steel wire armouring and a voltage rating to suit the system voltage (i.e. 22 kV or 11 kV). Cable size will be selected to suit the load, installation conditions and method, and supply system fault level. Cable terminations will be of the heat-shrink type.

LV cables will be multicore copper conductor, PVC-insulated cables with steel wire armouring. Cable sizes will be selected to suit the load, installation conditions and methods, voltage drop limits and installation fault levels.

Electrical cables will be sized for current carrying capacity maximum allowable volt drop, installation method and derating factors in accordance with SANS 10142. The cable sizes will be restricted within the range of 4C x 2.5 mm² to 4C x 185 mm² to allow for the most practical installation.

Single core, aluminium wire armoured, cable will be used where the use of 4C x 185 mm² cable is not practicable to achieve the required current handling capabilities.

Cable support systems will be hot-dipped galvanised cable ladder and wire-mesh tray for large and small cables respectively. Cables installed in cable trenches will be secured to cable ladders.

7.3.7 Backup Power

Backup power will be provided to each pump station and WTP by means of a diesel-powered generator set, which will be an indoor set sized for the powering of all essential loads. Essential loads will include small power, lighting, building security systems, plant control systems, instrumentation, electrically actuated valves and sluice gates, solenoid valves, lifting equipment and sump pumps. In essence, the plant can be safely shut down during a power outage, but water cannot be treated or conveyed. The anticipated generator size for essential loads at each plant will be about 200kVA.

The diesel generator set will be equipped with a fuel tank sized with the capacity to supply 24-hours of continual operation of essential equipment. In the case of a power failure there will be an automatic changeover to generator power.

No allowance has been made for renewable power sources or inverter-based battery backup systems.

7.3.8 Earthing and Lightning Protection

An integrated earthing system will be provided for the safety of equipment and persons, including:

- Systems earthing i.e. earthing of transformer neutrals
- Equipment earthing i.e. earthing of exposed conductive part of MV and LV equipment
- Lightning protection i.e. earthing of building LP down conductors
- Bonding of exposed and extraneous conductive parts

The earthing of the transformer neutrals will be done as follows:

- MV Supply Voltage / 11 kV transformers: resistively-earthed with NERs to limit earth fault current to 300A
- 22 kV/420 V transformers: solidly-earthed

A single building foundation earth matt will be provided at each pump station and shall be extended if required to provide an earth resistance of maximum 1 Ω to allow combining of MV and LV earthing. This earth electrode will then also meet the requirements for lightning protection Type B earthing and will be designed in accordance with SANS 1019: Design and Installation of Earth Electrodes.

It is considered prudent to provide lightning protection for each pump station and WTP for the following reasons:

- The lightning ground flash density is relatively high in these areas.
- Loss of infrastructure will cause loss of an essential service and economic loss
- The plants will contain electronic equipment which is susceptible to lightning damage.

Lightning protection will be designed in accordance with the following parts of SANS 62305: Protection against lightning:

- Part 3: Physical damage to structure and life hazard
- Part 4: Electrical and electronic systems within structures

For buildings having a metal roof, the lightning protection system will only comprise adequate down conductors to bond the roof and its metal supporting structure to the foundation earth electrode. For the part of the building having a concrete slab roof, an air termination rod will be provided to connect to the down conductors.

7.3.9 Building Electrical Services

A distribution board (DB) will be provided at each pump station and WTP to house the switchgear and control gear for electrical building services.

Interior and exterior lighting will be provided in the form of wall- or ceiling-mounted, corrosion protected luminaires with LED lamps. A floodlight will be wall-mounted opposite each pump set in the pump station buildings to provide additional illumination when required for maintenance purposes. Emergency (battery back-up) lighting will be provided at exits.

Motion sensors will be used to ensure interior lights are not kept on unnecessarily.

Single-phase switched socket outlets will be provided in all rooms and the loading bays as required. Three-phase 32 A switched socket outlets for welding machines will be provided in each pump room and the loading bays. Each will be connected to the supply DB using house wire sized to meet the current and voltage drop requirements.

Conduits that are installed in concrete or brickwork will be PVC. Surface conduits (if required) will be hot dipped galvanized steel with PVC end caps.

7.3.10 Area and Road Lighting

Pole-mounted streetlight luminaires will be provided along the access roads at the pump station, WTP and reservoir sites to serve as road and area lighting. Day/night switches will be utilised to ensure automatic switching during the nighttime.

The luminaires will be fitted with LED lamps and mounted on 5m high glass fibre reinforced polyester poles with hinged galvanised steel baseplates mounted on concrete bases.

7.4 Electronic Design

7.4.1 Overall Scheme Control

Each pump station and WTP will be controlled locally by programmable logic controller (PLC) based control systems. A local control room will be included at each pump station and the WTP for local SCADA based monitoring and control.

These plants and various reservoirs will communicate via a 4G based information sharing VPN protected network on a site-to-site basis.

Information can simultaneously be shared to a central control station SCADA room for remote monitoring.

Plant and pump station automation systems will be set up in according with each plant's control philosophy requirements.

The contractor will provide a detailed control system functional design specification (FDS). This document will detail the control architecture, control philosophy, HMI and SCADA mimics, equipment etc. that will make up the automation system.

7.4.2 Pump Control and Protection

Where duty/standby pumps are allowed for, duty rotation will be done by the PLC based on run hours. It will not be possible to operate the duty and standby pumps simultaneously.

The pumps will operate automatically under normal operating conditions.

It will be possible to operate each pump manually.

Each pump will be provided with a pump control panel in the pump station. An HMI and manual/off/automatic selector switch will be provided at each respective motor starter panel.

The control system will have password protected access at various levels ensuring that changes made to the system are by authorized personnel only.

In automatic mode, the pumps will start and stop automatically according to the levels in the reservoirs or as described in the scheme control philosophy.

In manual mode, the pumps will be started and stopped by local control pushbuttons.

In off mode, the pumps will not be capable of operating.

Switching from automatic to manual for any pump will cause the pump to switch off.

Field E-stop/start stations will be installed within arm's reach of equipment as far as possible. The field E-stops will be equipped a start push button and emergency stop push button.

Normal equipment protections will be active during both automatic operation and manual control.

Each pump / motor set will typically be protected by providing the following functions:

- Dry running: low level protection in suction reservoir according level transmitter, with backup level switch
- No flow: flow according to proximity switch on counterweight of non-return valves or flow-switches
- No flow: flow according to proximity switches on suction and delivery valve open and closed positions.
- Motor overload: high motor windings temperature
- High pump bearing temperature (drive end and non-drive end)
- High motor bearing temperature (drive end and non-drive end)
- Excessive pumpset vibration

7.4.3 PLCs and HMIs

Programmable Logic Controllers (PLCs) will be provided for the automatic control of the pump stations and WTP. The PLCs will also serve to provide process interlocking and protection functionality, and to serve as interfaces with SCADA systems. PLC panels will be incorporated into the MDB / MCC panels.

Associated with each PLC will be an HMI which will be provided to serve as an operator interface with the PLC for the following:

- Monitoring of equipment status / conditions
- Monitoring of process variables (reservoirs levels and pump station flow rates)
- Monitoring of alarm and trip events
- Logging and trending of selected information
- Adjustment of process control, alarm and protection setpoints

The HMIs will have graphics capability and will be set up for display mimic diagrams for monitoring and control purposes.

On plant communication will be a combination of:

- Serial, to minimise the number of cables and ensure transmitter data (like flow counters) can be directly accessed
- Hardwired (4-20mA for analogue, and digital) to protection systems

No onsite wireless communication will be used.

Equipment and materials which will be installed on the plant must be able to operate in the environment.

Equipment must be IP rated to levels suitable to the area of installation. Electrical equipment installed outside will have a minimum IP rating of 65.

Due to procurement being done in the public sector realm, specific OEMs for control equipment will be selected by the Contractor during tender.

7.4.4 SCADA

Provision will be made for a Remote View Node SCADA system at each pump station. The SCADA information of the scheme (pump stations, reservoirs and WTP) will be shared through the cloud on a secured gateway to ensure it can be viewed by each part of the scheme's control rooms.

Each SCADA station for monitoring and controlling of the WTP / pump station will be installed in the local control room. The SCADA will come complete with a rack mounted server (in a server room), historian server, UPS, a printer, one desktop computer, two 50” wall mounted screens, two 23” monitors, a mouse and keyboard. The SCADA will provide the following functionalities:

- Report generation. The SCADA will be equipped with a reporting feature and will provide the following functionalities in the form a screen display, downloadable file, as well as printable hard copies:
  - Daily reports
  - Monthly reports
  - Yearly reports
  - Alarm and disturbance reports
  - Maintenance reports
  - On demand query reports
- Process visualizations. The process visualization feature will provide the following functionality:
  - Dynamic process symbols (i.e., images and mimics). The mimics will be based on the works and the piping and instrumentation diagrams (P&IDs).
  - Display of trend curves from historic data.
  - Display of trend curves on real time basis.
  - Display of operator alerts, messages, alarms, and events.
- Trending (historic and on-line). This function will allow live and historic trends of analogue or calculated values. Each stored value will be the instantaneous or average values of a number of samples, depending on the desired resolution.
- Operator command interface. Process commands, set points and parameter changes will be allowed via SCADA faceplates and will include:
  - Control system set points.
  - Switching drives on/off, opening and closing of actuated valves etc.
  - Acknowledgement of error and alarm messages.
- Operator access security. The system will have multi-level password protection whereby different levels of operators will be granted different operational authorizations

7.4.5 Field Instrumentation

Field instrumentation will be provided to provide the required monitoring and control described functional requirements.

Conventional 4-20 mA current loops will be used for continuous measurement, and 24V DC or potential-free contacts for discrete measurement.

The following instrumentation will be provided:

- Hydrostatic Level transmitters for all pump station and destination reservoirs
- Level switches for back-up protection at the pump station suction reservoir
- Clamp-on ultrasonic or inline magnetic flow meters at pump stations
- Proximity switches on pump delivery non-return valves (where feasible)
- Suction and delivery pressures transmitters and gauges at all pumps.
- Temperature sensors (RTDs) in main pump motor windings and on pump- and motor bearings

7.4.6 Control, Instrumentation and Data Cables

Control cables will be provided as required for control circuitry of all controlled equipment, and for remote controlling devices.

Control cables will be 600/1000V multicore 1,5mm² minimum copper conductor, PVC-insulated cables with galvanised steel wire armouring and PVC serving.

Instrumentation cables will be provided as required for all instrumentation signals where the instrumentation is not equipped with an integral cable long enough to reach the termination point.

Instrumentation cables will be a minimum 1.5 mm^2 twisted-pair, copper conductor, individually and overall screened, PVC-insulated with galvanised steel wire armouring and PVC serving.

Data cables for linking the HMIs, PLCs UPSs and pump motor starters will be industrial Cat5/6e UTP Ethernet cable or of a type suitable for the equipment supplied by the contractor.

Fibre optic cables will be multimode.

7.4.7 Earthing and Surge Protection

All electronic equipment will be connected to a comprehensive earthing system consisting of separate instrument earths and protective earths. Functional and protective earthing will be provided as required to all electronic equipment.

All cable trays carrying cables for electronic equipment will be bonded to earth.

Surge protection devices will be specified for all power and signal circuits of electronic equipment.

7.4.8 UPSs

Uninterruptible power supplies (UPSs) will be provided to condition the power supply and provided back-up power to the PLCs, HMIs and instrumentation.

The UPSs will be specified to be of the single-phase type with an isolation transformer and a bypass to mains. Rated stored energy time will be standard in the 15-30 minutes range. Batteries will be of the sealed, maintenance-free lithium iron type.

7.4.9 Building Electronic and Security Systems

Buildings will be provided with fire detection sensors and alarm panels as per the requirement identified in the Fire Plan.

CCTV systems will be provided at each pump station with viewing screens in the local control room. Motion detection will be incorporated into the CCTV software, thus removing the need for separate intruder detections systems.

Access control to the pump station will be by card readers.

8 Xhariep Water Treatment Plant

8.1 WTP Capacity

The capacity of the plant is also dictated by the requirements of the scheme as defined in the Pre-feasibility Study Report (Ref No.: P WMA 06/D00/00/3423/5). The required plant capacity is 312 Mℓ/d. The site planning however provides for an additional 104 Mℓ/d to facilitate future plant expansion. The total planned site capacity is therefore 416 Mℓ/d. The sizing horizon for the WTP is 30 years.

The design of the WTP is based on the establishment of two clusters of treatment infrastructure, each consisting of two semi-independent trains of 104 Mℓ/d each. Each cluster then has a capacity of 208 Mℓ/d. It is anticipated that the site will be developed to deliver 208 Mℓ/d at first and then to be augmented with a further 104 Mℓ/d around 10 years later at which time the capacity of the WTP will be 312 Mℓ/d. The further provision of 104 Mℓ/d is not scheduled at present but will allow for consideration of delivery to the south of the proposed WTP as well as further distribution into the southern Free State Province. The plant design will reach the target daily production capacity in 22 hours per day.

The proposed treatment site is located approximately 5 km north of the Xhariep Town and immediately east of the N1 highway as shown in Figure 8-1. Access to the site will be by exiting to the north from the R701. The site is not currently being utilized for any intensive agricultural or other activities. The site is located between 1280 and 1300 m above mean sea level (AMSL).

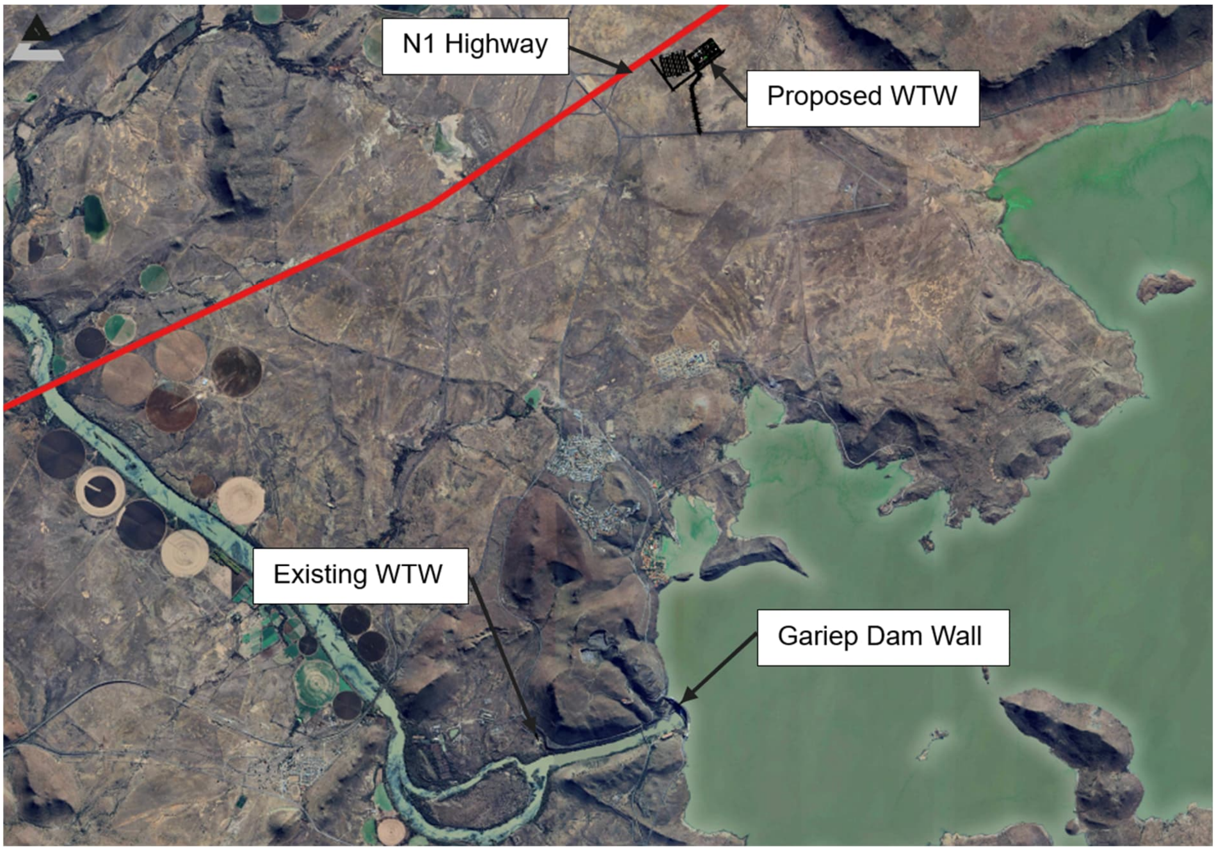


Figure 8-1: General location of the proposed Xhariep WTP

The geotechnical investigation found that the site is underlain by rock at shallow depth. Most trail pits encountered rock at less than 1m depth with an average of around 0.5m. There is therefore not sufficient soft material to allow for the excavation of pipe trenches for the main process pipelines. The design therefore requires that most main structures, treatment reactors and pipelines be constructed and installed above ground to reduce the cost of excavation and to avoid the requirement for excessive pumping between process units.

8.2 Raw Water Quality Assessment

Several sources of data were consulted to define the quality of raw water to be treated at the proposed WTP. These included historical data for the Gariep Dam from the archives of the Department of Water and Sanitation (DWS), historical data for the existing Gariep Dam WTP which was obtained from the Vaal Central Water Board (VCWB) (previously Bloem Water), and an in-depth monitoring programme which was undertaken as part of this study. The latter consisted of 12 samples over the period spanning March 2024 to August 2024 (6 months) and which focussed on gaps identified in the DWS and VCWB data.

The analysis of the raw water quality indicates the following treatment requirements after evaluation and consideration of the errant data:

- Turbidity levels are moderately high and require treatment intervention.
- Total aluminium levels are high. The dissolved aluminium levels are very low and do not require treatment. The non-dissolved fraction will be removed along with general turbidity if care is taken not to re-dissolve the aluminium through extensive pH manipulation.
- The microbiological indicators will be removed adequatly through normal disinfection protocols. No positive results were noted for chlorine resistant cysts or oocysts in any of the water qualty data sets.
- A small number of datapoints reflected very high dissolved organic carbon (DOC) levels. All the high values were however reported prior to 2003. Subsequent reports all indicated DOC levels below the national standard. No specific treatment regimes are included to address organic carbon removal.
- Both the Ryznar index as well as CCPP indicate that the water is aggressive and will require stabilisation.
- The additional sampling of the water source and analysis for a significantly expanded set of parameters indicated little to no risk associated with chlorophyll-a or contaminants of emerging concern at the proposed treatment plant.
- Some historical data sets indicate a number of determinands are present at levels of concern. The high initial values are likely the result of laboratory detection limits exceeding the specified water quality targets.The latest data sets indicate that these values, when appropriately analysed, are below levels of concern.

Based on the available water quality data the water can be described as of very good quality. Turbidity, microbiology, and stability are the only determinants requiring particular attention. Conventional flocculation, settling and filtration is proposed. A comparable process is currently successfully employed at the Gariep Dam WTP. The new Xhariep WTP will additionally focus on stabilisation and a suitable disinfection strategy.

Laboratory tests indicate that the preferred flocculant for treatment of the source water is a poli-aluminium chloride flocculant. The average design dosage of 12 mg/ℓ and maximum of 35 mg/ℓ are informed by laboratory tests as well as current operational experience at the existing Gariep Dam WTP.

Water quality data and laboratory tests indicate that stabilisation is required to achieve the required CCPP target of 2 to 5 mg/ℓ. An options analysis indicates that hydrated lime at around 5 to 10 mg/ℓ is the most economical approach to stabilisation.

A chlorine demand study was undertaken that indicated a single chlorine dose at the treatment plant would not be sufficient to sustain chlorine levels in the transfer system. Chloramination cannot be considered as Mangaung Metropolitan Municipality (MMM) does not presently receive chloraminated water from its other sources and the mixing of chloraminated and non-chloraminated water cannot be permitted. The system will therefore be chlorinated with a booster injection of chlorine at the Booster Pump Station at which time the water will have been in the transfer system for around 48 hours. Chlorine decay models indicate that this approach will result in a chlorine residual being present in the water when it reaches the receiving reservoirs in Mangaung. A comparison of chlorination systems indicated that chlorine gas systems are more economical than on-site chlorine generation systems. This report is based on the use of 1 tonne chlorine gas drums. More economical gas delivery approaches may be available, but this can only be confirmed following direct negotiation with chlorine gas suppliers.

Main disinfection will take place in the on-site storage reservoir. The reservoir will be partitioned to allow for 30 minutes of contact time. At a residual level of 4 mg/ℓ chlorine the CT-requirement for Giardia, Cryptosporidium as well as viruses will be satisfied. The stated chlorine level is required to ensure a residual of chlorine remains present when the water reaches the chlorine booster station. The total chlorine dose at the WTP is estimated to be in the order of 6.7 mg/ℓ on average and 7.9 mg/ℓ at maximum. The booster dosage requirement is estimated to be 3 mg/ℓ.

The chlorine dosing requirements are high and suggest that trihalomethane production may be a risk. Laboratory tests indicate a marginal risk for this albeit at higher chlorine dosages and on untreated water. Trihalomethanes will have to be monitored as a risk determinand once the WTP is in operation.

The storage reservoir make provision for another 30 minutes of storage for high lift pump balancing purposes. This is anticipated to be sufficient to allow for smooth operation of the high lift pumps.

8.3 Treatment Technology

The treatment technology proposed for the WTP can be described as conventional:

- Flocculation and clarification will take place in a pulsator clarifier. Experience at other treatment sites along the Orange River indicate that pulsators operate acceptably if routine maintenance is in place. The pulsators also typically operate at higher rates than horizontal settlers and radial flow clarifiers resulting in a smaller footprint and lower costs. The proposed design is based on a loading rate of 2.5 m/h which is conservative for pulsators but will be appropriate for the occasional turbidity peaks that may exceed 200 NTU in extreme cases. A total of 12 separate pulsators will be required to deliver 312 Mℓ/d. The modular design will allow for diversion of flow to the remaining units if a unit is to be removed from operation for maintenance purposes.
- The rapid gravity sand filters are designed for deep penetration of floc into the filter bed. The filter beds are 1 m deep and the effective media size (d_{10}) is 0.9 mm. The filter will be cleaned using a combined air and water scour to facilitate collapse pulsing which will be followed by a water rinse. A total of 30 filters, with a surface area of 84.7 m^2 each, are required to deliver 312 Mℓ/d. The filter loading rate is 6 m/hr with all 30 filters operational and 6.67 m/hr if three filters have been removed from operation for maintenance purposes.

Treatment residuals will be thickened in sludge ponds. Mechanical thickening was not considered as an alternative due to the additional operational complexity that this brings to a WTP as well as the fact that sufficient land is available to allow for consideration of the low cost and effort alternative of ponds. Pond supernatant will be returned to the inlet works to ensure efficient use of the water resource. An options analysis was undertaken to compare various pond construction approaches. A Hyson cell lined pond was found to be most cost effective. The lining will allow for mechanical cleaning of the pond at a higher frequency than for clay lined ponds, hence a smaller pond area can be specified, and at a lower cost than comparably sized concrete lined ponds.

8.4 Support Buildings

The following support buildings have been included in the feasibility design:

- Administration and control building with a laboratory, meeting rooms, control offices and staff facilities;
- A maintance workshop with workshop floor area supported by overhead cranes, store areas, offices, meeting rooms and facilities for maintenance personnel; and

- Chemical storage areas for bulk chemical storage and housing of dosing equipment. Chlorine is stored and controlled from separate facilities that can be developed in a phased manner.

All structures, services and installed plant shall be designed and constructed to meet DWS standards as well as the WTP owner's standards where these exceed the requirement of DWS.

8.5 Mechanical and Electronic Works

The mechanical equipment selections, power ratings and capacities are preliminary, and will be finalised during detailed design. The estimated rated power consumptions required for Phase 1 (208 Mℓ/d) and Phase 1 and 2 (312 Mℓ/d) is 664 kW and 1152 kW respectively. This excludes the High Lift Pump Station.

The design approach for the electronic works shall achieve the following:

- Centralised control of the WTW allowing for easy and convenient operation.
- The installation can be controlled and monitored remotely from the administration building's SCADA, the Xhariep scheme's remote command centre or in the field at the Human Machine Interfaces (HMI) at various possible locations, like filters and dosing systems.

All drawings related to the proposed Xhariep WTP are contained in the Book of Drawings, technical details can be found in the Feasibility Design Report – Water Treatment Works (P WMA 06/D00/00/3423/11).

9 Pipeline Design

9.1 Horizontal and Vertical Alignment

The horizontal pipeline alignments were determined during the pre-feasibility phase of the project and is detailed in the Pre-Feasibility Phase Report. These horizontal pipeline alignments were included as part of the basic assessment process for authorisation. The Department of Forestry, Fisheries and the Environment (DFFE) authorised the project, taking into account the following:

- A 100m wide corridor was assessed for the proposed pipeline route;
- The construction corridor will be 40m wide, unless restricted by existing infrastructure or other constraints; an
- The final servitude width will be 15m.

Any changes to the pipeline alignment during the detailed design phase need to take cognisance of the above and should take place within the 100m wide corridor that was assessed. The preliminary construction corridors and servitude widths are shown on the horizontal and vertical alignment drawings included in the Book of Drawings.

9.2 Pipe Material Selection

Based on the required pipeline diameters, anticipated operating pressures, economic considerations, ease of operation and maintenance, mild steel was considered the preferred pipe material for the Xhariep Pipeline Project. This has also been the material of choice for most of the large bulk water infrastructure projects undertaken in South Africa over the past two decades, e.g. Berg Water Project, Vaal River Eastern Subsystem Augmentation Project, Mokolo Crocodile Water Augmentation Project, etc.

9.3 Pipeline Structural Design

The pipeline's structural design was based on the AWWA M11 guidelines as well as the design information contained in Stephenson (*Pipeline Design for Water Engineers*). The traffic loads were calculated based on the equations contained in AWWA M45, as the AWWA M11 equations do not cover pipes deeper than 1.2 m, meaning that AWWA M11 overstates the vehicle loads on deeper pipes.

The AWWA M11 recommends 2.00 and 1.33 as the factors of safety for working and surge pressures, respectively, whereas DWS uses a factor of safety of 1.67 for both the working and surge pressures. The design was based on DWS's factor of safety.

Based on the geotechnical investigation, the minimum E-value of the native material ranges from 1 MPa to 15 MPa in the layers encountered above the bedrock level with 9 MPa being the average E-value. It was, however, reported that 93% of the pipeline will be located in areas with shallow bedrock with E-values ranging from 5 GPa to 30 GPa. In terms of the soil classification guidelines provided in AWWA M45, an E-value of 15 MPa is adopted for the wall thickness calculations – this E-value corresponds to a DCP value, measured as mm/blow, of less than 20. Imported bedding material would be required along the majority of the pipeline route and is expected to have a minimum E-value of at least 9 MPa.

The design of the pipe was based on Grade X52 steel with a Yield Modulus of 358 MPa.

Based on the design parameters, the required wall thicknesses can be calculated for different pressure classes and pipe diameters.

Table 9-1 summarises the required pipe wall thicknesses based on pipe diameters and pressure classes.

Table 9-1: Pipe wall thicknesses for different diameters and pressure classes

Pipe diameter (mm)	Design pressure up to (m)	Required wall thickness (mm)
1400	160	8
1400	200	8
1400	250	12
1400	254 (1)	12
1800	160	10
1800	200	11
1800	250	14
1800	320	20
1800	400	22
1800	415 (1)	22
2000	160	12
2000	175 (1)	12

(1) Maximum field test pressure per pipe diameter

9.4 Pipeline Lining and Coating

9.4.1 Lining

Although the hydraulics were based on the assumption of using cement mortar lining, it is likely that the pipelines will be internally lined with epoxy as (a) easy access into the pipe is available based on the large diameters to undertake joint repairs, (b) epoxy linings are generally more economical, and (c) larger pipe deflections can be handled by epoxy lined pipes (i.e. up to 5% deflection) compared to cement mortar lined pipes (i.e. up to 2% deflection).

Field joints shall be repaired with the same lining to the same thickness as the main pipeline.

9.4.2 Coating

The following coatings are generally considered for mild steel pipes:

- Polymer Modified Bitumen (PMB)
- Fusion Bonded Medium Density Polyethylene (FBMDPE)
- Trilaminate Polyethylene Coating (3LPE)
- Rigid Polyurethane (RPU)

Table 9-2 summarises the field joint coatings, as well as the coatings of buried specials, recommended for each of the four coatings.

Table 9-2: Recommended coatings for field joints and buried specials

Coating	Field joints	Buried specials
PMB	PMB	PMB
FBMDP	Cold applied tape	PU, PMB, or liquid epoxy with cold applied tape
3LPE	Liquid epoxy with cold applied tape	Liquid epoxy with PMB or reinforced visco-elastic
PU	PU	PU

It is recommended that a coating and lining selection report be compiled during the detailed design phase, which should take cognisance of factors such as the soil resistivity results, the presence of sulphate reducing bacteria, location relative to electrified railway lines and the surrounding high-voltage overhead powerlines.

9.5 Air Valve Design

9.5.1 General

The positioning of the air valves has been determined primarily on design criteria such as high points along the route, slope changes (i.e. both positive and negative slopes), long straight sections (i.e. for the release of air under normal operating conditions), as well as the guidelines contained in the WRC Report: Quantifying the Influence of Air on the Capacity of Large Diameter Water Pipelines and Developing Provisional Guidelines for Effective De-Aeration, Volume 1 (WRC Report No 1177/01/04) and Volume 2 (WRC Report No 1177/02/04).

Sizing and positioning of air valves was based on the rate at which air will be introduced or expelled from the pipeline, taking account of the following:

- Filling conditions
- Scour conditions
- Pipe rupture
- Normal operating conditions

9.5.2 Filling of the Pipeline

It is recommended that the maximum filling velocity be limited to 0.5 m/s in the main pipelines.

9.5.3 Scouring of the Pipeline

The critical air valves for the scour scenario are the high lying air valves that would be activated first when draining the pipeline. A sufficient number of air valves must be installed to ensure that the air intake capacity of the air valves is equal to or more than the discharge rate of the scours. The design of the scour outlets therefore limits the maximum scouring velocity to 0.5 m/s within the pipeline.

9.5.4 Pipe Burst Conditions

The critical air valves for the pipe burst scenario are the high lying air valves that would be activated first when a pipe failure occurs. A sufficient number of air valves must be installed to ensure that the air intake capacity of the air valves is equal to or more than the discharge rate.

The discharge rate is calculated based on the following formula:

$$Q = C_d \times A \times (2 \times g \times h)^{0.5}$$

with,

Q = Flow (m³/s)
A = Area of the scour (m²)
C_d = Discharge coefficient
h = Water pressure (m)
g = Gravitational acceleration (m/s²).

The area of burst/rupture is assumed to be 10% of the cross-sectional area of the main pipeline and C_d is assumed to be 0.6.

The differential pressure across air valves during air intake will be limited to 3.5 m for both the scour and pipe burst scenarios.

9.5.5 Normal Operating Considerations

Allowance is generally made to install additional air valves every 500 m to 600 m, should no air valves be required based on the filling, scouring or burst conditions. This is done for the effective de-aeration of the pipeline and to provide access to the pipeline at convenient intervals for maintenance purposes.

The minimum operating pressure at any air valve must be 5 m as to ensure adequate sealing to prevent the valves from leaking. Where the operating pressure is less than 5 m at particular air valves, these air valves would need to be fitted with additional bias mechanisms to ensure proper sealing.

9.5.6 Air Valve Sizes

The utility programmes developed by VENT-O-MAT and ARI were used as a secondary aid to confirm the initial sizing. Based on the analysis, various air valve sizes were recommended at different locations with the majority of air valves being DN200. It is recommended that the air valve sizes be standardise as DN200 to simply maintenance and to limit the spares required.

The size, positions and numbers of air valves need to be finalised during the detailed design phase based on the final vertical pipeline alignment and surge analysis.

9.5.7 Air Valve Chamber Details

Drawings with details of the air valve chambers are included in the Book of Drawings.

9.6 Scour Valve Design

9.6.1 General

The scour valve outlets were sized on the following guidelines:

- Limiting the velocity in all pipelines to 0.5 m/s to prevent secondary surge pressures when closing the scour valve.
- Limiting the velocity in the scour outlet pipe to less than 6.0 m/s to prevent damage to the isolating valve and outlet pipework due to excessive velocities, vibrations and cavitation.

9.6.2 Location of Scour Valves

Scour installations is provided at all low points along the pipeline vertical profile. An important component of the design philosophy is to locate the scour installations in such a manner that the pipeline will be completely drained during scouring operations, thereby minimizing the need for pumping the remaining water using mobile pumping equipment which has limitations concerning pumping rates.

Provision have been made for secondary type scours which are effectively vertical riser pipes with blank flanges at the top, which could be used to access the remaining water by lowering a pump down the riser pipes.

The applicable scour types will be provided along the pipeline route are based on the static pressure acting on the scour installation. The selection criteria for each scour type are shown below:

- Type 1: Static pressures over 60 m (one isolating valve together with a sleeve valve).

- Type 2: Static pressures over 10 m and up to 60 m (two isolating valves, one being a sacrificial valve).
- Type 3: Static pressures up to 10 m (only vertical riser in pipe with a blank flange).

9.6.3 Scour Valve Chamber Details

Drawings with details of the different types of scour valve chambers are included in the Book of Drawings.

9.7 Inline Isolating Valve Design

The need for inline isolating valves must be determined during the detailed design phase based on the operational requirements of the implementing agent and the organisation that will be responsible for the operation and maintenance of the pipeline.

Inline isolation valves are generally provided at certain points along the pipeline to enable sections of the pipeline to be isolated to scour each section in approximately 6 to 8 hours when attending to a pipe breakage or performing routine maintenance. This will typically enable repairs to be completed and the pipeline to be refilled within a 24-hour period.

An example of inline isolating valve chambers is included in the Book of Drawings. The following should be noted with respect to the details:

- Two air valve tees are provided upstream and downstream of the inline butterfly valve to:
  - Introduce air through the air valves when draining the pipeline in either direction, and,
  - Provide an access point into the pipeline.
- A bypass pipeline, fitted with orifice plates, is provided around the butterfly valve in order to fill the pipeline once remedial work or maintenance has been completed. This is required due to the high differential head across the butterfly valve and the risk of damaging the valve and/or the pipe lining if no bypass is provided. The orifice plates need to be designed to limit the filling velocity to 0.5 m/s in the main pipeline. The bypass pipeline is also equipped with wedge gate valves to facilitate the replacement of the orifice plates if required.

9.8 Off-Takes

Off-take chambers are located along the pipeline route to supply future end-users or towns. Each chamber is fitted with an isolating valve with provision inside the chamber for a strainer and flow meter, as well as a downstream isolation valve.

The layouts of the off-take chambers are shown on drawings included in the Book of Drawings.

It should also be noted that the air valve chambers can be converted to include off-takes. The DN200 riser pipe in the air valve chamber is fitted with an isolation valve with the air valve located on top of it. It is possible to install an equal tee on top of the isolation valve to provide an off-take from the branch with the air valve still located at the top. A separate chamber needs to be constructed adjacent to the air valve chamber that will house the isolation valves, strainer and flow meter.

9.9 Cathodic Protection

The cathodic protection design for the steel pipelines will be done during detailed design. A specialist sub-contractor will be employed to design the temporary and permanent cathodic protection system.

Preliminary soil resistivity results have indicated that the corrosiveness of the soil ranges from mild to severe over the pipeline length, meaning that cathodic protection will be required.

10 Reservoir Design

10.1 Sizing of Reservoirs

Three reservoirs are required as part of the project – two command reservoirs and a suction reservoir located at the booster pump station. The sizing of these reservoirs is discussed in Section 2.2.1 and is based on the DWS Technical Guidelines for the Development of Water and Sanitation Infrastructure (2nd edition, 2004).

The command reservoirs are each sized for a capacity of 80 Mℓ (i.e. 80,000 m³), whereas the suction reservoir at the booster pump station is sized for a capacity of 10 Mℓ (i.e. 10,000 m³).

10.2 Reservoir Types

Three types of reservoirs are typically considered for reservoirs with a storage capacity of more than 50 Mℓ, namely (a) conventional above ground post-tensioned circular reinforced concrete reservoirs, (b) conventional above ground circular or rectangular reinforced concrete reservoirs, and (c) earth-fill embankment type reinforced concrete lined reservoirs.

In the case of smaller reservoirs like the suction reservoir, only types (a) and (b) are considered.

A comparison between the various reservoir types for 80 Mℓ storage is provided below.

10.2.1 Type 1: Conventional Above Ground Post-Tensioned Circular Reinforced Concrete Reservoir

The approximate dimensions of a conventional above ground post-tensioned circular reinforced concrete reservoir suitable for all three sites would most likely be in the order of:

- Diameter = 98 m
- Wall height at full supply level (FSL) = 10.60 m
- Footprint area = 7 545 m²
- Reinforced concrete quantity = 8 350 m³

Figure 10-1 illustrates a cut-out image of how the geometry of such a circular reservoir could look.

10.2.2 Type 2: Conventional Above Ground Rectangular Reinforced Concrete Reservoir

The approximate dimensions of a conventional above ground rectangular reinforced concrete reservoir suitable for all three preferred sites would most likely be in the order of:

- Length = 156 m
- Width = 78 m
- Cantilever wall height at FSL = 6.7 m
- Footprint area = 12 170 m²
- Reinforced concrete quantity = 7 590 m³

Figure 10-2 illustrates a cut-out image of how the geometry of such a rectangular reservoir could look.

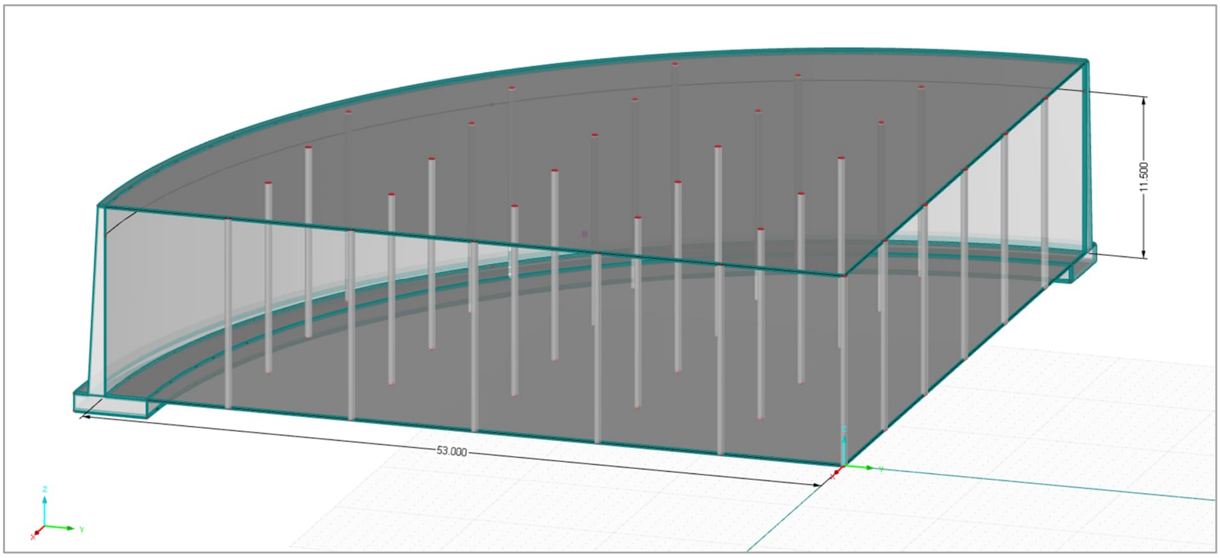


Figure 10-1: Type1 - Conventional above ground post-tensioned circular reinforced concrete reservoir (m)

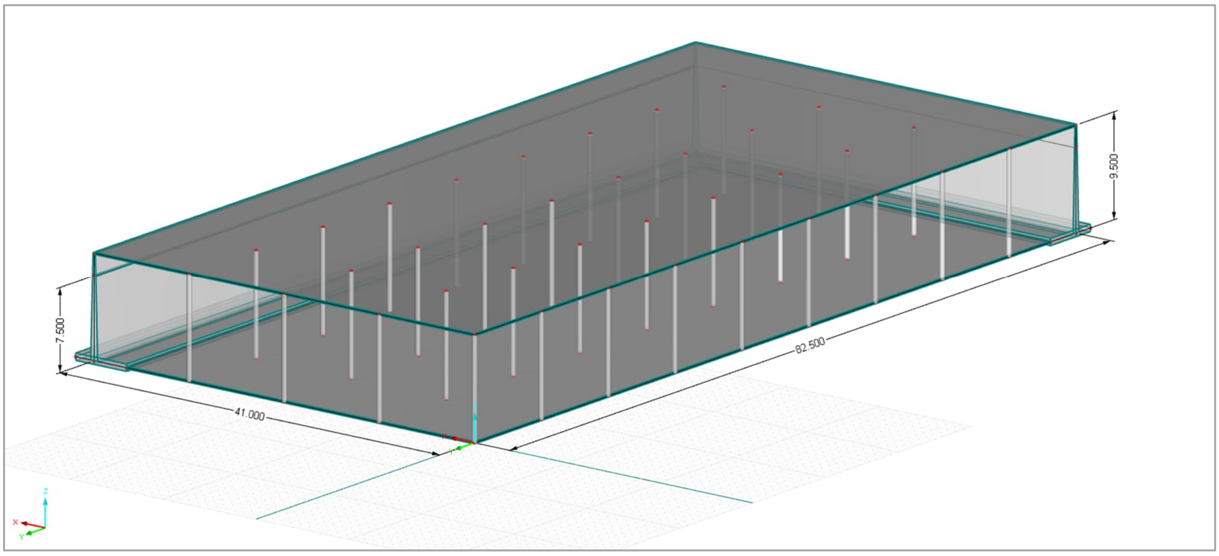


Figure 10-2: Type 2 - Conventional above ground rectangular reinforced concrete reservoir (m)

10.2.3 Type 3: Earth-Fill Embankment Type Reinforced Concrete Lined Reservoir

The dimensions of a preliminary designed earth-fill embankment type or hopper bottom reinforced concrete lined reservoir suitable for all three preferred sites are:

- Length = 138 m
- Width = 70 m
- Cantilever wall height at FSL = 4.5 m
- Water height from hopper bottom Invert Level (IL) at FSL = 10.0 m
- Footprint 9 660 m^2
- Reinforced concrete quantity = 6 185 m^3

Figure 10-3 illustrates a cut-out image of how the geometry of such an earth-fill embankment type reservoir could look.

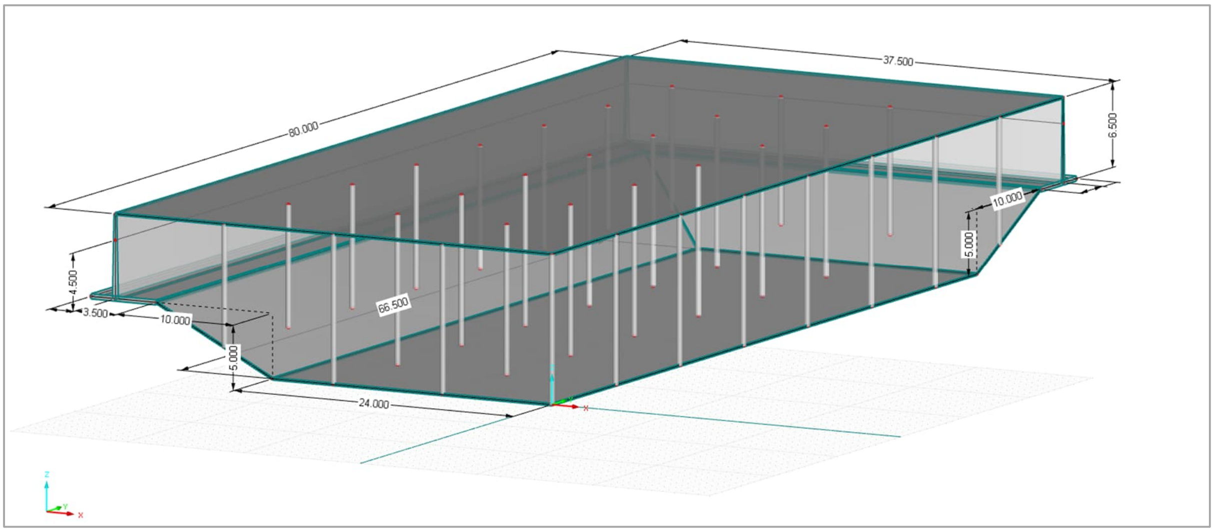


Figure 10-3: Type 3 - Example of Earth fill embankment type or hopper bottom reservoir (m)

10.2.4 Comparison Between the Different Reservoir Types

A high-level comparison of the reinforced concrete quantities for the three types of reservoirs revealed the following:

- The bulk earthwork quantities and therefor the cost for each type of reservoir would be similar.
- The reinforced concrete component cost for a Type 3 - Earth-fill embankment type reservoir is by far the cheapest.
- The main reason is that 40% of the water is stored in the reinforced concrete lined earth embankment portion or hopper and 60% in upper part formed by the reinforced concrete cantilever retaining walls.
- This configuration results in a significant saving in concrete volumes compared with Type 1 and Type 2.
- The reinforced concrete component cost for a Type 1 - Post tensioned circular reservoir is approximately 35% higher than that of a Type 3 because of the higher concrete volumes and the high post-tensioning cost.
- The reinforced concrete component cost for a Type 2 - Rectangular reservoir is approximately 23% higher than that for a Type 3 because of the higher concrete volumes.

Apart from financial considerations, the following factors need to be considered in the selection of the preferred reservoir type:

- **Visual impact** – embankment type reservoirs have a significantly reduced visual impact compared to a post-tensioned circular reservoir. Due to the post-tensioning, no backfilling is allowed against the reservoir, meaning that it would protrude 10.6m above the finished ground levels. In comparison, an embankment reservoir will protrude only 1.5m above the finished ground levels.
- **Maintenance** – the sloped corners and sides of the embankment reservoirs allow for vehicular access into the reservoirs, which shortens maintenance times and provides safer access compared to other types of reservoirs.

From the high-level cost comparison, and considering other factors as well, the Type 3, Earth-fill embankment type reservoir seems to be the most viable reservoir type for the command reservoirs.

Post-tensioned reservoirs are typically only considered for reservoir sizes from 15 Mℓ and larger. As such, the suction reservoir will be a conventional above ground reinforced concrete reservoir.

10.3 Reservoir Layouts

Layout drawings for Command Reservoir No 1 and Command Reservoir No 2 are included in the Book of Drawings.

10.4 Operating Levels

The operating levels in the various reservoirs are summarised in Table 10-1.

Table 10-1: Operating levels in reservoirs

Description	Command Reservoir No 1	Suction Reservoir	Command Reservoir No 2
Top water level (masl)	1565.00	1445.00	1530.00
Overflow weir/pipe level (masl)	1565.10	1445.10	1530.10
Top of floor level (masl)	1555.00	1440.00	1520.00
Minimum operating level (masl)	1557.50	1441.00	1522.50

11 Site access

11.1 Technical requirements/widths

The main site access roads are designed to include two 3.4m wide lanes to accommodate bi-directional traffic with 0.9m wide shoulders along either side of the road, resulting in a total blacktop width of 8.6m. An additional 0.6m is allowed along either side of the surfaced road to make up the full width of the road prism. A road servitude width of 12 m is proposed.

The stormwater drainage for all main site access roads is proposed as trapezoidal earth channels along the cut side of the road to accommodate stormwater runoff, both from the roadway itself and as a cut-off drain for runoff from the existing terrain towards the roadway. The channel is to be designed to include concrete lined sections along its entire length for ease of maintenance and to accommodate supercritical flows at steep longitudinal grades. Culvert crossings need to be allowed for where the access road crosses natural watercourses or valleys in the terrain.

The fencing proposed along the main site access roads is a permeable wildlife fence along either side of the length of the road. A farm style double 3.4m wide swing gate can be allowed at the pump station and reservoir sites. The access to the water treatment plant will be formalised and be gate controlled from the site's guardhouse.

The pavement layers need to be determined during the detailed design phase based on the estimated traffic loads and the design vehicles. It is envisaged that the pavement layers could comprise the following:

- 40mm asphalt surfacing;
- 150mm G2 Crushed stone base;
- 250mm C4 Cement stabilized subbase;
- 200mm to 400mm G7 Selected subgrade; and
- 150mm in-situ rip and compact to 90% of MAMDD

Details of the access roads to the respective infrastructure components (e.g. raw water pump station, Xhariep WTP, Command Reservoir No 1, booster pump station and Command Reservoir No 2) are included in the Book of Drawings.

12 Special Design Considerations

12.1 Control Narrative

A Block Flow Diagram (BFD) is shown on Drawing 1002533-0010 (refer to Book of Drawings) of the overall scheme. The following is a description of the overall control narrative of the Xhariep Pipeline Project.

Command Reservoir No 2 will supply the Rustfontein WTP's clearwell and Longridge Reservoir under gravity. The level control valves at Rustfontein WTP and Longridge Reservoir will open when the levels in these reservoirs drop and water will be supplied from Command Reservoir No 2 until these reservoirs reach their full supply level, whereafter the level control valves will close.

When the water level in Command Reservoir No 2 drops to a preset level (say 85%), a signal will be transmitted to the booster pump station to start one pump. In the event that the level in Command Reservoir No 2 drops further than a 2nd preset level (say 75%), a second pump will start, followed by a third pump when the reservoir reaches a 3rd present level (say 65%). The booster pump station will continue to deliver water to Command Reservoir No 2 until the command reservoir reaches its full supply level.

Once the booster pump station starts to deliver water to Command Reservoir No 2, the water level in Command Reservoir No 1 will start to decrease. When the water level in Command Reservoir No 1 drops to a preset level (say 85%), a signal will be transmitted to the high-lift pump station to start one pump. In the event that the level in Command Reservoir No 1 drops further than a 2nd preset level (say 75%), a second pump will start, followed by a third pump when the reservoir reaches a 3rd present level (say 65%). The high-lift pump station will continue to deliver water to Command Reservoir No 1 until the command reservoir reaches its full supply level.

The operator at the Xhariep water treatment plant (WTP) will adjust the treatment capacity to match that of the high-lift pump station and/or to ensure that the WTP's clearwell reservoir remains above a minimum level.

The raw water pumps are fitted with variable speed drives to match the flow being treated at the Xhariep WTP. Once the operator at the WTP adjusts the flow at the WTP, a signal will be transmitted to the raw water pump station and the pump speed will be varied automatically to match the flow of the WTP.

12.2 Connection to Longridge Reservoirs

No information could be obtained from Mangaung Metropolitan Municipality (MMM) regarding the as-built pipework at the Longridge Reservoirs or any information on proposed upgrades to the Longridge Reservoir complex.

It is proposed that MMM be engaged during the detailed design phase of the project to finalise the connection details to Longridge Reservoirs and to make provision for the future extension of the pipeline to Brandkop Reservoir, should this be required.

12.3 River and stream crossings

The proposed pipelines will be crossing numerous rivers and streams, as well as dry dongas and stormwater discharge channels where water flow is concentrated for short periods during rains.

The river crossings will be constructed by means of open trench excavation with the pipe being encased with concrete through the river. The geotechnical investigation has shown that shallow bedrock can be expected at most of the river crossings, meaning that the pipe will be founded on rock. The pipeline will also be wrapped with Denso Ultraflex or similar where encased in concrete to reduce the risk of damage to the coating during the installation process.

For the river crossings only, the wall thickness is to be increased by 20% as per DWS standards.

At all river crossings and larger stormwater crossings, allowance need to be made for rip-rap construction to prevent erosion. A typical river crossing detail is included in the Book of Drawings.

12.4 Road crossings

There are several road crossings along the proposed pipeline route.

The major road crossings will be undertaken by installing DN2400 Class 100D concrete pipe sleeves by means of pipe jacking. The section of steel pipe that is to be sleeved through the jacked concrete pipe will have an increase wall thickness of 20% relative to the connecting pipeline as per DWS standards.

Minor gravel road crossings will be performed by means of open cut trench excavation, with no increase in wall thickness required.

Access for communities and affected road users will be maintained at all times.

Details of the major road crossings are shown on the drawings included in the Book of Drawings.

12.5 Servitudes

The proposed servitude and working widths are shown on the drawings included in the Book of Drawings. The working width was based on 40 m, which is regarded as an acceptable working width for pipelines with diameters up to DN2000.

It is recommended that a 15 m wide servitude be registered along the proposed pipeline.

The servitude areas required at the proposed structures (i.e. water treatment plant, reservoirs and pump stations) are shown on the respective layout drawings, which are included in Appendix C.

The registration of servitudes and the associated compensation is handled by the Sub Directorate: Land Rights Administration (LRA), which is the unit within Department of Water and Sanitation (DWS) responsible for all land related matters for the DWS, country-wide. The LRA needs to be engaged as soon as the detailed design phase commences so that communication with affected landowners can commence.

12.6 Dealing with existing services

The following existing services are either known to exist, or are highly likely to exist, in the vicinity of the planned infrastructure:

- Bulk and reticulation water pipelines;
- Bulk and reticulation sewerage pipelines;
- Bulk electricity power transmission lines both overhead and buried;
- Distribution electrical power lines both overhead and buried;
- Railway lines and electrical cables and infrastructure associated with railway lines;
- Water pipelines associated with the railway lines;
- Roads, both municipal, provincial and national;
- Stormwater infrastructure (pipeline and culverts); and
- Telecommunication infrastructure both overhead and buried.

Information on the existence and locality of the abovementioned existing services was requested from the various service providers responsible for each or several of the expected existing services. Unfortunately, very limited to no information had been received, mainly as the water board, district municipalities and local municipalities did not have accurate as-built information. It is proposed that a ground penetration radar survey be undertaken near build-up areas to locate existing services as part of the detailed design phase.

12.7 Wayleave applications

Wayleave applications for the geotechnical fieldwork investigation was submitted to the following authorities:

- Eskom;
- Free State Provincial Roads;
- Kopanong Local Municipality;
- Liquid Fibre;
- Mangaung Metropolitan Municipality;
- Openserve / Telkom;
- Sanral;
- Transnet; and
- Vaal Central Water Board.

It is proposed that route approvals be submitted to these authorities as soon as the pipeline routes have been finalised during the detailed design phase, which will enable the contractor(s) to obtain the necessary wayleaves when construction commences.

12.8 Other detailed design phase considerations

The normal services and deliverables (where applicable) required to be provided in terms of the detailed design phase, construction monitoring and supervision phase, and close-out phase are described in the Guideline Professional Fees (Scope of Services and Tariff of Fees for Persons Registered in terms of the Engineering Profession Act, 46 of 2000) as published by the Engineering Council of South Africa (ECSA). Additional services, which are required over and above the normal services, are defined separately. Below is a description of additional services that will be required by the Service Provider during the next phases of the project.

12.8.1 Preferential Procurement and Targeted Participation

The Service Provider shall provide all services related to preferential procurement and targeted participation in respect of the construction contracts, including but not limited to:

(a) the incorporation into the contract documentation of:

  i. preferential procurement requirements in respect of B-BBEE and, if applicable, local production and content, and
  ii. targeted participation goals in respect of targeted labour and/or resources.

(b) the monitoring and verification of compliance with (i) and (ii) in (a) above during the construction contracts (including the receiving and collation of documentary evidence submitted by the Contractor in this regard).

12.8.2 Act as Leader of the Professional Team

Where other disciplines have sub-contracted their services to the Service Provider, the Service Provider shall assume leadership of the professional team and be responsible for the overall administration, co-ordination, programming of design and financial control of all works included in the services.

12.8.3 Act as the Employer's agent in terms of the Occupational Health and Safety Act

The Service Provider, in submitting a tender for the professional services contract, shall be deemed to have acknowledged acceptance of the appointment as the client's agent in terms of the Occupational Health and Safety Act, 85 of 1993 and the Construction Regulations, 2014, should the Employer accept the tender. The Service Provider shall, as such, execute all of the duties of the client as contemplated in the Construction Regulations.

The Service Provider's attention is also drawn to the responsibilities of the designer(s) in terms of the Construction Regulations and shall comply with all requirements in this regard.

The Service Provider shall, apart from conducting his own activities in compliance with the Occupational Health and Safety Act, 85 of 1993 and Construction Regulations, 2014, ensure that any sub-consultants/sub-contractors employed by the Service Provider also comply with the requirements of the Act and Regulations. The Service Provider shall enter into an agreement with the Employer in this regard before the commencement of any work related to this contract.

12.8.4 Environmental Control Officer

The Service Provider shall provide from within its own organisation or, if necessary, appoint as a sub-consultant, an experienced Environmental Control Officer (ECO) who shall be full-time on site for the duration of the construction contract. The ECO shall undertake all duties required to ensure full compliance with the Environmental Authorisation.

12.8.5 Stormwater Management Plan

The Service Provider shall be responsible for the preparation of the Stormwater Management Plans (SWMP) required for the project and at all the respective infrastructure sites (e.g. water treatment works, pump stations, reservoirs and along the pipeline).

The Service Provider shall be responsible for obtaining approval of the SWMPs from the respective authorities, whereafter it shall be incorporated as part of the WULA submission.

12.8.6 Geotechnical Field Investigations and Laboratory Testing

At the time of the detailed feasibility design of the project, access was not available to sections of the project for the geotechnical field investigations and associated laboratory testing. The scope that was omitted is detailed in the Geological and Materials Investigations Report.

The Service Provider shall be responsible for completing the remaining field investigations and laboratory testing and shall prepare a tender to appoint a sub-contractor for this scope (i.e. test pitting, drilling of boreholes, laboratory testing, etc.). The Service Provider shall manage the appointed sub-contractor and be responsible for logging of all test pits, boreholes and compiling a supplementary Geotechnical and Materials Investigations Report.

Obtaining all wayleaves and permissions for the investigation shall form part of the Service Provider's obligations.

12.8.7 Topographical Survey

At the time of the detailed feasibility design of the project, access was not available to sections of the project for the topographical survey and installation of benchmarks. The scope that was omitted is detailed in the Topographical Survey and Mapping Report.

The Service Provider shall be responsible for completing the remaining topographical surveys and shall prepare a tender to appoint a sub-contractor for this scope (i.e. topographical surveys, installation of benchmarks, etc.). The Service Provider shall manage the appointed sub-contractor and be responsible for compiling a supplementary Topographical Survey and Mapping Report.

Obtaining all wayleaves and permissions for the topographical surveys shall form part of the Service Provider's obligations.

12.8.8 Eskom Power Supply Liaison

The power supply to the respective sites shall be designed by Eskom. The Service Provider shall be responsible for all applications to Eskom, including confirmation of design loads, as well as all liaison required with Eskom to ensure the timeous construction of the power supplies to the respective sites.

12.8.9 Preparation of Servitude Diagrams

The registration of servitudes and the associated compensation is handled by the Sub Directorate: Land Rights Administration (LRA), which is the unit within Department of Water and Sanitation (DWS) responsible for all land related matters for the DWS.

The Service Provider shall be responsible for appointing a suitably qualified land surveyor to prepare the draft servitude diagrams for review by LRA, as well as the lodging of the final servitude diagrams at the Surveyor General.

12.8.10 Finalising Environmental Management Programme (EMPr)

The Environmental Authorisation stipulates that the Environmental Management Programme (EMPr) needs to be updated during the detailed design phase, circulated for comment to registered Interested and Affected Parties and then submitted to DFFE for approval prior to the commence of construction.

The Service Provider shall be responsible for updating the EMPr, all public engagements, incorporation of comments and submission to DFFE.

12.8.11 Environmental Specialist Studies as per Environmental Authorisation

The Environmental Authorisation (EA) requires certain inputs from environmental specialists at the commencement of construction and during the construction phase.

The Service Provider shall be responsible for appointing and managing the environmental specialists to ensure compliance with the EA requirements.

13 Authorisation Processes

13.1 Environmental Authorisation

The Basic Assessment Report (BAR) submission to the Department of Forestry, Fisheries and the Environment (DFFE) is contained in Report No P WMA 06/D00/00/3423/15.

The environmental authorisation was issued on 26 September 2024 (DFFE Reference Number 14/12/16/3/3/1/2996) and is valid for a period of 10 years.

The key findings and specific conditions from the Environmental Impact Assessment are summarised below.

- A copy of the final site layout maps must be made available (during the detailed design phase) for comments to the registered Interested and Affected Parties and the holder of this environmental authorisation must consider such comments. Once amended, the final development layout map must be submitted to the Department for written approval prior to commencement of the activity.
- The Environmental Management Programme (EMPr) must be amended to include measures as dictated by the final site layout map. The EMPr must be made available for comments by registered Interested and Affected Parties and the holder of this environmental authorisation must consider such comments. Once amended, the final EMPr must be submitted to the Department for written approval prior to the commencement of the activities.
- A protected tree permit must be first obtained before the removal of the protected wild olive trees on the site earmarked for the booster pump station with suction reservoir.
- The packed stone kraal (J001) which is on the alignment of the pipeline must be avoided and not subject to impacts arising from the project. A buffer of 10m is recommended around this site.
- A pre-construction archaeological walkover survey of those portions of the pipeline route which cross dolerite ridges and river valleys, and those infrastructure areas that could not be accessed during the TerraMare Archaeological survey, must be undertaken.
- A walkthrough of the final layout must be undertaken by an Ecologist before construction commence.
- Installation of the pipeline through wetlands and watercourses should preferably be undertaken during the winter months (July to September) when baseflow will be at its lowest level.
- A permit must be obtained from the relevant authorities for the removal or disturbance of any TOPs, Red Data listed or provincially protected species prior to construction.
- No exotic plants must be used for rehabilitation purposes. Only indigenous plants of the area must be utilised.
- Should any archaeological sites, artefacts, paleontological fossils, or graves be exposed during construction work, work in the immediate vicinity of the find must be stopped, the South African Heritage Resources Agency (SAHRA) must be informed, and the services of an accredited heritage professional obtained for an assessment of the heritage resources.
- Hazardous substances must be stored in abunded and designated area to avoid accidental leakage into the environment.
- An integrated waste management approach must be implemented that is based on waste minimisation and must incorporate reduction, recycling, re-use and disposal where appropriate. Any solid waste must be disposed of at a landfill licensed in terms of Section 20 (b) of the National Environmental Management Waste Act, 2008 (Act No. 59 of 2008).

13.2 Water Use Licence Application

The Water Use Licence Application (WULA) can only be submitted once the implementing agent has been finalised. Under this project, the following reports have been compiled:

- Water Use Licence Summary Report
- Integrated Water and Waste Management Plan
- Comment and Response Report (based on public participation process)

These reports need to be finalised during the detailed design phase of the project and submitted to DWS for the Water Use Licence.

14 Financial Considerations

14.1 Capital Cost Estimate

The capital expenditure (CAPEX) estimate was prepared based on historical data collected from projects of a similar nature and complexity and is summarised in Table 14-1.

The assumptions used for the capital costing were as follows:

- Rates and unit costs (civil, mechanical, electrical, and electronic) were adopted from projects of a similar nature in terms of size and capacity.
- Capital cost estimates for civil works, mechanical equipment, electrical and electronic equiment based on dated quotes were escalated at 8% per annum (p.a.) to allow for market related inflation.
- Civil works preliminary and general expenses were based on 25%, while the mechanical, electrical and electronic work preliminary and general expenses were based on 30%.
- Contract Price adjustment of 7% per annum (p.a.) were assumed for the construction period.
- Foreign exchange adjustment was calculated for specific mechanical and electrical WTP components that may be imported. Exchange rates at the time of tender and at the time of order were considered and the resultant foreign exchange adjustment was determined.
- Allowance for foreign exchange variation of 5% per annum (p.a.) were assumed for the construction period.
- Contingencies of 15% were assumed to account for unforeseen items.
- Professional Fees were calculated based on ECSA 2021 Fee Scales gazetted on 26 March 2021 for each scenario based on the CAPEX and engineering discipline.
- Site Supervision was not included as part of the CAPEX
- Cost estimates can be assumed to be at an accuracy of 25 – 30% as part of detailed feasibility studies, which should improve to 10 – 15% at the end of the detailed design phase.

Table 14-1 provides a summary of the capital cost estimate for the Xhariep Pipeline Project.

Table 14-1: Estimated Capital Expenditure for the Xhariep Pipeline Project

Description	Estimated CAPEX (ZAR)
Preliminary and General	3,939,328,274
Raw Water Pump Station	162,443,508
High-Lift Pump Station	359,650,870
Command Reservoir No 1	137,008,420
Booster Pump Station and Suction Reservoir	292,708,069
Command Reservoir No 2	304,276,170
Pipelines	14,385,273,022
Water Treatment Works (Phase 1)	2,248,730,000
Subtotal Capital Cost (Excl. VAT)	**21,829,418,333**
Contract Price Adjustment (CPA) @ 7% p.a.	4,912,557,790
Allowance for Foreign Exchange Variation @ 5% p.a.	434,230,000
Contingency @ 15%	4,076,430,918
Project Cost (Excl. VAT)	**31,252,637,041**
Engineering Design Fees @ 8%	2,500,210,963
Disbursements and Recoverable Costs	124,960,856
Professional Fees (Excl. VAT)	**2,625,171,819**

Description	Estimated CAPEX (ZAR)
Total Project Cost (Excl. VAT)	**33,877,808,861**
Total Project Cost (Incl. VAT)	**38,959,480,190**

Notes:
1 Cost Estimate Base Date – November 2024.
2 Construction Commencement Date – November 2028

The estimated operation and maintenance budget required for the first year of operation is summarised in Table 14-2, showing an estimated minimum O&M budget requirement.

Table 14-2: Estimated Annual Operation and Maintenance Budget for the Xhariep Pipeline Project

Description	Estimated Annual O&M Budget (ZAR)	
	Complete Phase	**Phase 1**
	312 Mℓ/d	**208 Mℓ/d**
Maintenance	200,782,452	184,932,452
Labour	24,180,000	21,160,000
Energy	82,533,048	53,752,032
Chemicals	71,590,000	47,730,000
Sludge Disposal	13,940,000	9,300,000
Total OPEX (excl. VAT)	**393,025,500**	**316,874,484**
Plant Cost (Amortised @ 20 years)	633,040,000	475,680,000
Total Annual Cost of Ownership (excl. VAT)	**1,026,065,500**	**792,554,484**

Notes:
1 Estimated other operational cost required for first year of plant operation based on 2024 Costs

15 Socio-Economic Impact Assessment & Legal, Institutional and Financing Arrangements

The following is a summary of the work undertaken as part of this Socio-Economic Impact Assessment & Legal, Institution and Financing Report (Ref. No P WMA 06/D00/00/3423/10).

15.1 Socio-Economic Assessment

The socio-economic assessment reiterated that the implementation of the Xhariep Pipeline is essential to ensure long-term water security and economic stability in the GBWSS region.

From the socio-economic impact assessment, the following are key takeaways:

- The implementation of the Xhariep Pipeline Project would allow economic activities to continue at both the 2035 and 2050 horizons.
- The projected socio-economic impacts by 2050 (in 2023 prices) of the Xhariep Pipeline are:
  - Total Gross Domestic Product (GDP) = R416,665 million,
  - Capital generated = R892,574 million,
  - Employment Opportunities Maintained = 948,040,
  - Annual Household Income Generated = R158,013 million, with
    - R23,892 million to low-income households, and,
  - Additional taxes paid to the different authorities = R107,236 million.
- The Xhariep Pipeline Scheme 1B was found to be economically viable as it would contribute to the socio-economic circumstances in the GBWSS and would improve security of supply of water of the system.
- The affordability analysis concluded that the total capital and interest repayment over a 30-year periods is not affordable for the paying households or the business and industrial sectors.
  - The paying households cannot afford the additional R28/m³.
- If alternative funding options are considered, the option of a 75% capital grant and a loan for the balance of the capital costs at a low interest rate is probably the first of the different funding options that will ensure that the project is affordable to households.
- A concern raised through the socio-economic impact assessment was the financial management of Mangaung Metropolitan Municipality (MMM).

15.2 Financing Arrangements

The financing of the Xhariep Pipeline project is critical to its long-term success and sustainability. The financing model must balance the capital expenditure, operational costs and the affordability for the end-users. As there is limited opportunity for implementation phasing, the project requires substantial upfront investment, which will require a mix of public funding, concessional loans and potential private sector involvement.

The key takeaways from the financing arrangements assessment are:

- Various implementation scenarios were analysed. The project was split into three components and several implementation combinations were explored – with each entity implement one, two or all three components. The components were namely:
  - Component A (encompassing the infrastructure from the Gariep Dam to the 2nd Command Reservoir),

- Component B (encompassing the infrastructure from the 2nd Command Reservoir to Longridge Reservoir) and,
- Component C (encompassing the infrastructure from the 2nd Command Reservoir to Rustfontein water treatment works).

- ► The financial information required to accurately assess the implementation capacity of the entities was not available, however, financial information about previous projects implemented in the past 10 years by the entities was assessed as a proxy indicator.
- ► Five sources of financing are available for a public infrastructure project of this nature, i.e., (a) grants from central government, (b) the public delivery agency’s own resources, (c) equity, (d) commercial debt, and (e) concessionary debt. The share of the financing sources assumed to be available to the implementation entities were presented in the report.
- ► The cost of capital from each financing source was assessed for each implementing entity. It should be noted that MMM and Vaal Central Water Board (VCWB) are expected to have to pay a premium above market rate for loans, given their vulnerable financial positions.
- ► For each implementation scenario considered, the projected bulk tariff was calculated taking into account the direct costs, long-term operational expenses and debt repayment obligations.
- ► Options 1C (where the project is implemented entirely by a DWS entity) and Option 3B (where components A and B are implemented by DWS and component C by VCWB) were found to have similar bulk tariff implications, R16.57/kl and R16.44/kl, respectively.
- ► Introducing a public-private partnership (PPP) in Option 3B did not result in any savings, as the private sector efficiencies were not sufficient to overcome the public entities’ access to lower financing cost.
- ► An affordable tariff structure, particularly for the lower income households is crucial to ensuring that the socio-economic benefits of the project are shared across the population.
- ► The project can achieve financial sustainability, provided that cost recovery is effectively implemented thorugh a carefully structured bulk water tariff system.

15.3 Legal and Institutional Arrangements

The legal and institutional arrangements analysis explained that the chosen institutional arrangement should be based on a cooperative framework that involves all relevant institutions with a mandate and responsibility to provide water.

Key takeaways from the legal and institutional arrangements include:

- ► The same implementation scenarios were assessed from a legal and institutional mandate perspective as were assessed from a financing perspective.
- ► The analysis described the mandates of the three spheres of government involved in the Xhariep Pipeline Project implementation process, the national (DWS), water board (VCWB) and the water service authority (MMM and the local municipalities), highlighting that neither sphere has hierarchy over another but that there is interdependency and interrelation between them.
- ► From the Constitution, the Municipal Systems Act, the Municipal Structures Act and the Water Services Act, the following has been deduced:
  - There is a duty on all spheres of government to ensure that water supply services are provided in a manner that is efficient, equitable and sustainable.
  - The water service authorities, i.e., MMM and the local municipalities, have the mandate to ensure provision of water supply services within their area of jurisdiction.
- ► Therefore, the municipalities can decide how best to fulfill their consitutional mandate of providing access to water supply services and should decide on who should be the implementing entity for the Xhariep Pipeline. In doing so, these municipalities must take into account, among others:
  - Alternative ways of providing access to the services,

- The need for regional efficiency,
- The need to achieve benefit of scale,
- The need for low costs,
- The requirements of equity,
- The availability of resources from neighbouring authorities,
- Institutional capacity,
- Financial capacity,
- Technical competency,
- Manpower, etc.

► If necessary, the municipalities may request VCWB to be the implementing entity of the scheme and VCWB may only refuse the request if, for sound technical and financial reasons, they would not be able to provide those services.

► If the municipalities fail to take up their responsibilities, the organs of State in the provincial and national sphere of Government may intervene to resolve the matter.

► It should be noted that there is potentially an inability by MMM to take up this responsibility. Their current administration and infrastructure management are poor, but they are also constrained by inadequate support from the other spheres of government and a critical shortage of funds. The other local municipalities may face similar challenges though these were not specifically examined as they would use about 3% of the water produced by the scheme with MMM using 97%.

This assessment highlights that, while various institutional options exist, the financial sustainability of the pipeline project will be dependent on the chosen implementing entity's capacity to manage operations and maintenance effectively. The report findings underscores that the project's success hinges on collaboration between national, provincial and local governments, and a clear delineation of responsibilities among entities.

From an economic perspective, the project is expected to generate substantial socio-economic benefits through direct, indirect, and induced impacts. This includes job creation during the construction and operational phases and an improvement in water security, which is critical for supporting regional economic activities, particularly in the agriculture and manufacturing sectors. The financial viability of the project is achievable if tariff structures are well managed, ensuring affordability for households while maintaining financial sustainability.

Recommendations

Based on the finding of the assessments the following recommendations are made:

► The findings should be presented and discussed at a Working Group Committee meeting to all the relevant stakeholders, which should be initiated by DWS and attendees must include MMM, Kopanong Local Municipality, Mantsopa Local Municipality and VCWB;

► The project success hinges on collaboration and cooperation between national, provincial and local governments. DWS should initiate the establishment of and lead a **Working Group** that involves all the relevant stakeholders, with representation at an executive and strategic level, so that agreement can be reached on:

  - Responsibilities with respect to the implementation, operation and maintenance of the scheme, e.g. MMM (as they should take a leading role) can request VCWB (legally) or DWS (administratively) (including TCTA or the NWRIA) to implement the project on their behalf,
  - Financing options, taking consideration that at least a 75% capital grant and a loan for the balance of the capital costs at a low interest rate would be required to result in affordable bulk water tariff increases. The creditworthiness of each institution must be considered as part of the financing options to minimise the cost impact on the end-users, and,
  - Development of an implementation timeframe.

16 Project Programme

Table 16-1 shows an indicative programme for the implementation of Phase 1 of the Xhariep Pipeline Infrastructure project. Figure 16-1 shows the programme in Gantt chart form.

Table 16-1: Xhariep Pipeline Infrastructure implementation programme (Phase 1)

Task description	Start date	Duration (days)	End date
End of Feasibility Study	2025/02/28	1	2025/03/01
Finalisation of Institutional Arrangements	2025/03/01	150	2025/07/29
Funding approval	2025/07/29	120	2025/11/26
Procure PSP* for Detailed Design	2025/04/30	210	2025/11/26
Detailed Design & Tender Documentation	2025/11/26	540	2027/05/20
Procure Contractor	2027/05/20	300	2028/03/15
Section 33 process	2028/03/15	360	2029/03/10
Pipeline construction (250 km)	2029/06/08	1260	2032/11/19
Water Treatment Works construction	2029/03/10	1440	2033/02/17
Pump stations (x 3)	2029/03/10	900	2031/08/27
Command Reservoirs (x2)	2029/03/10	720	2031/02/28
Testing and commissioning	2033/02/17	120	2033/06/17
Commencement of Defects Liability Period	2033/06/17	1	2033/06/18

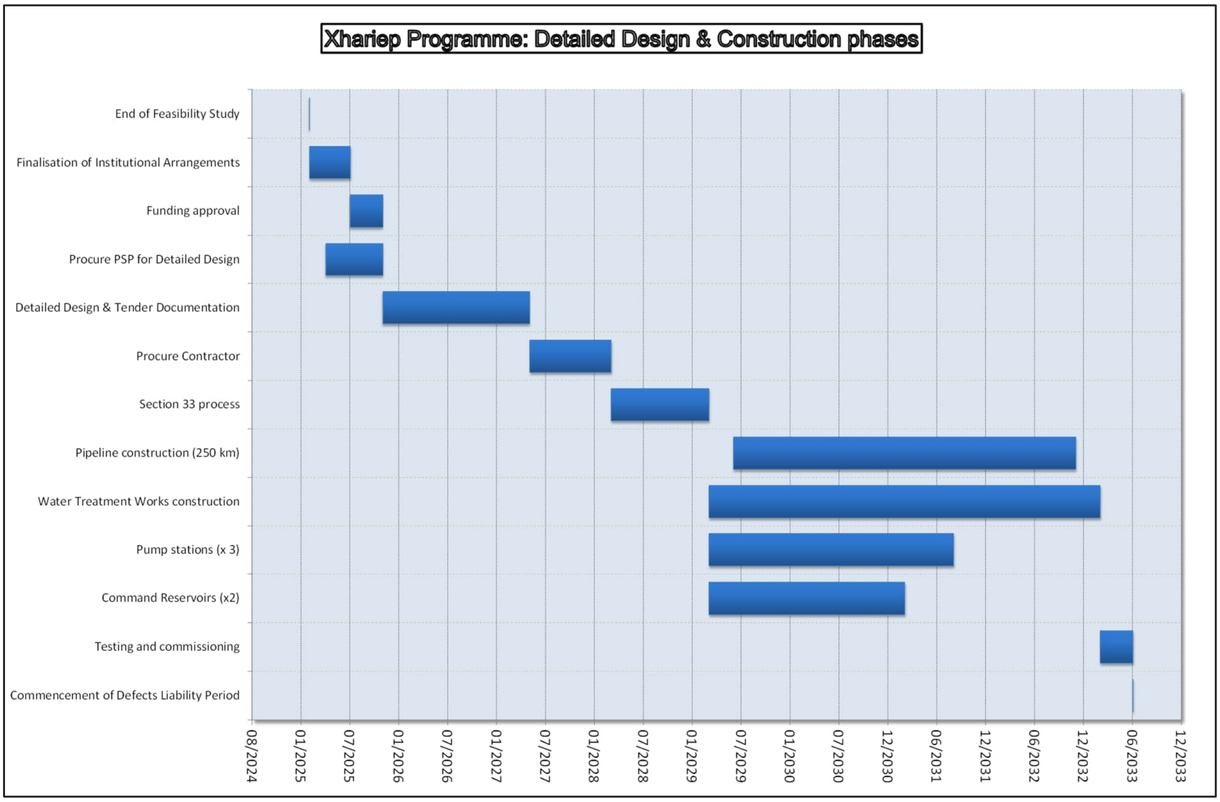


Figure 16-1: Xhariep Pipeline Infrastructure implementation programme (Phase 1)

Based on the indicative programme, the detailed design needs to commence towards the end of 2025 for construction to commence towards the end of 2028. The commissioning should be completed towards the end of 2032 with the scheme being fully operational by early 2033.

17 Conclusions and Recommendations

The pre-feasibility study concluded that Scheme 1B, as shown in Figure 17-1, was the optimum configuration to address the water shortages experienced within the Greater Bloemfontein Water Supply System (GBWSS), which includes Bloemfontein, Botshabelo and Thaba Nchu.

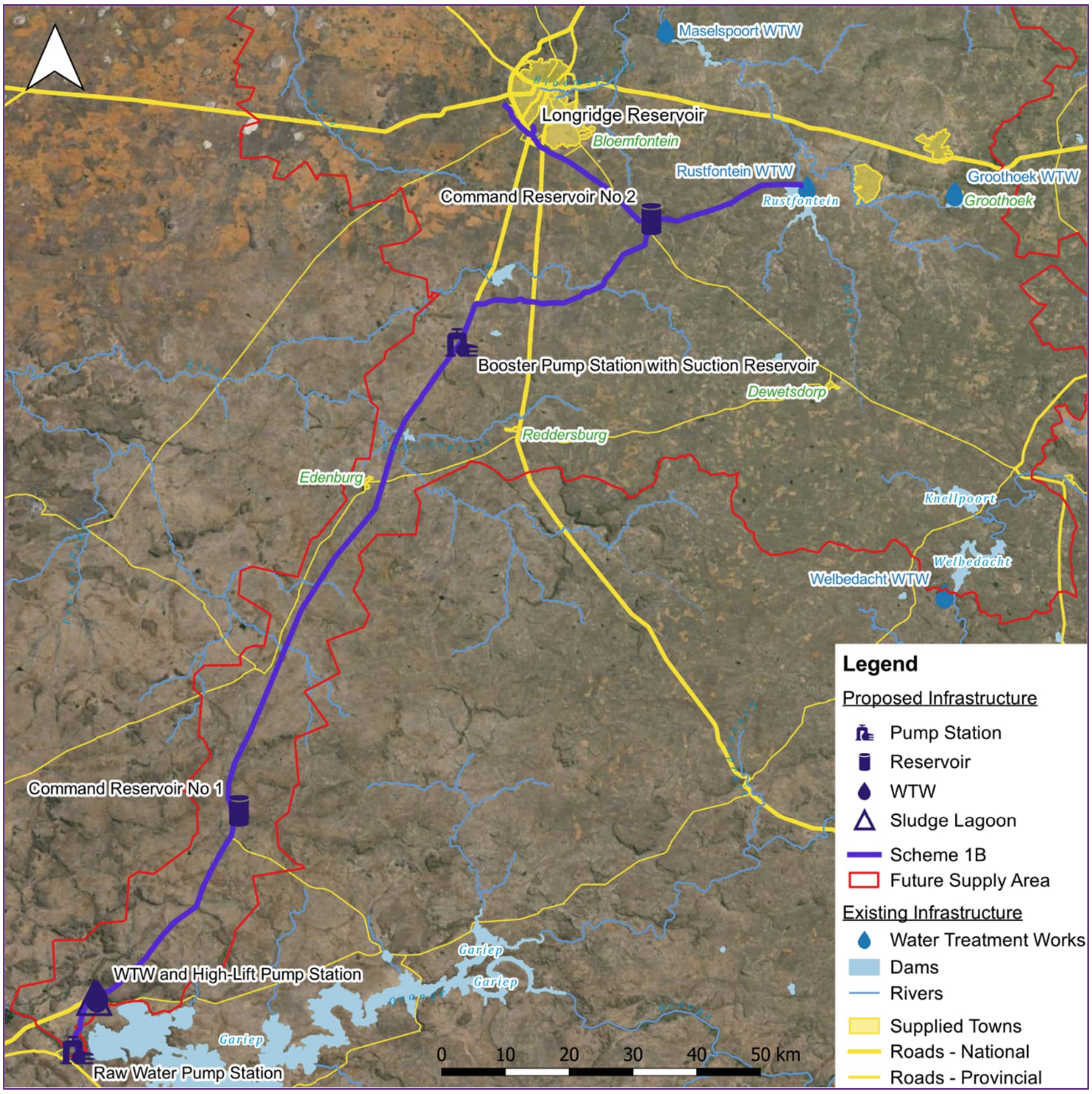


Figure 17-1: Main infrastructure components of Scheme 1B

A long-term stochastic analysis confirmed that the proposed potable transfer scheme at a capacity of 101 million m³/a is capable of meeting the GBWSS demands at the required assurance of supply until at least the year 2050.

It was determined, from the design flow calculations, that the raw water infrastructure had to be sized for a peak week flow of 3.797 m³/s (329 Mℓ/d), whereas the potable infrastructure, including the water treatment plant (WTP), had to be sized for a peak week flow of 3.616 m³/s (312 Mℓ/d). The two command reservoirs were sized for 6 hours storage at the peak week flow rate of 3.616 m³/s (312 Mℓ/d), equating to a storage capacity of 80 Mℓ per reservoir.

A Light Detection and Ranging (LiDAR) survey was undertaken for the overall study area in order to undertake the detailed feasibility design and to provide the required topographical data for the detailed design phase of the project. As part of the survey, control points and benchmarks were installed and digital colour images of the project area were obtained.

A geotechnical field investigation was undertaken for the overall study area. The fieldwork investigation included the excavation of 410 test pits, 106 in-situ Dynamic Cone Penetrometer (DCP) tests, 120 Dynamic Probe Super Heavy (DPSH) tests, rotary core drilling of 44 boreholes, electrical resistivity testing at 200 m intervals along the pipeline routes, as well as the associated laboratory testing.

At the time of undertaking the topographic survey and geotechnical investigation, access to certain privately owned properties was not available and wayleaves from MMM were not received and had to be excluded. The topographical survey and geotechnical investigation of these areas need to be concluded as part of the detailed design phase of the project.

A water quality testing programme, consisting of 12 samples taken over a period from March 2024 to August 2024 (6 months) was undertaken to supplement the raw water quality data available from DWS and VCWB.

The main infrastructure components of Scheme 1B, as shown in Figure 17-1, include the following:

- Tie-in at the existing DN2100 pipeline downstream of Gariep Dam Wall,
- A pipeline from Gariep Dam to the Raw Water Pump Station (± 2 km long),
- The Raw Water Pump Station,
- A pipeline from the Raw Water Pump Station to a break pressure tank (± 2 km long),
- A pipeline from the break pressure tank to the Xhariep water treatment works (WTP), ± 9 km long,
- The Xhariep WTP, which is designed for a capacity of 312 Mℓ/d of which 208 Mℓ/d will be constructed as Phase 1, with a future 104 Mℓ/d to be constructed later. The site will, however, be planned for an ultimate capacity of 416 Mℓ/d,
- The High Lift Pump Station located at the WTP site, which will pump water to Command Reservoir No 1,
- The pipeline from the High Lift Pump Station to Command Reservoir No 1 (± 43 km long),
- Command Reservoir No 1 (80 Mℓ storage),
- A pipeline from Command Reservoir No 1 to the Booster Pump Station (± 95 km long),
- A Booster Pump Station with Suction Reservoir (10 Mℓ storage),
- A pipeline from the Booster Pump Station to Command Reservoir No 2 (± 44 km long),
- Command Reservoir No 2 (80 Mℓ storage),
- A pipeline from Command Reservoir No 2 to the existing Rustfontein WTP (± 25 km long), and
- A pipeline from Command Reservoir No 2 to the existing Longridge Reservoirs (± 28 km long).

The pipeline diameters of the pumping mains were optimised based on net present value (NPV) calculations that considered capital, maintenance and operational costs. Various sensitivity analyses were undertaken that considered different discount rates, different growth patterns in water demand, different inflation rates for energy costs, etc. The recommended optimum diameters for the pumping mains are:

- Pipeline from the Raw Water Pump Station to a break pressure tank = DN 1800,
- Pipeline from the High Lift Pump Station to Command Reservoir No 1 = DN 1800, and
- Pipeline from the Booster Pump Station to Command Reservoir No 2 = DN 1800.

The pipeline diameters for the gravity pipelines were determined based on the available head and the design flow rates. The recommended optimum diameters for the gravity pipelines are:

- Pipeline from Gariep Dam to the Raw Water Pump Station = DN 1800,
- Pipeline from the break pressure tank to the Xhariep water treatment works = DN 2000,
- Pipeline from Command Reservoir No 1 to the Booster Pump Station = DN 1800,
- Pipeline from Command Reservoir No 2 to the existing Rustfontein WTP = DN 1400, and
- Pipeline from Command Reservoir No 2 to the existing Longridge Reservoirs = DN 2000.

The duty points for the three pump stations (i.e. raw water pump station, high-lift pump station and booster pump station) were calculated based on the optimised pipe diameters. The pump types available to achieve the required duty points were evaluated, concluding that horizontal split-casing and vertical turbine pumps were the only pump options that could deliver the required flows and heads. The horizontal split-casing pumps were, however, preferred as they are more economical and easier to operate and maintain.

All three pump stations were designed with a three duty, one standby, pump configuration. Critical aspects such as operating speed, hydraulic efficiency, net positive suction head required, and head rise to shut-off head were evaluated for each pump selection. Details of the selected pumps are summarised in Table 17-1.

Table 17-1: Pump selection details

Description	Raw water	High Lift	Booster
Pump duty	3.797 m³/s @ 73m	3.616 m³/s @ 320m	3.616 m³/s @ 127m
Pump Model	SMD 500-750 A	HPDM-450-1000	SMD 600-1250 B
Configuration (duty/standby)	3 duty, 1 standby	3 duty, 1 standby	3 duty, 1 standby
Maximum rated speed (rpm)	990	990	740
Variable speed or fixed speed	Variable	Fixed	Fixed
Hydraulic efficiency at duty point (%)	89.9	83.1	83.7
Net Positive Suction Head (NPSH) required at duty point (m)	5.7	6.2	3.5
Head rise to shut-off head (%)	26	14	18
Hydraulic power per pump at duty point (kW)	1,005	4,547	1,790
Maximum power per pump in operating range (kW)	1,060	5,000	2,036
Recommended motor size (kW)	1,200	5,780	2,400

It is evident from Table 17-1 that the raw water pump station pump sets will be fitted with variable speed drives (VSDs), whereas the other two pump stations will operate at fixed speed. The VSDs are required due to the large fluctuation in water levels within Gariep Dam and to ensure that the raw water flow matches the flow to be treated at the proposed Xhariep WTP.

A hydraulic and waterhammer analysis was undertaken to determine the maximum working and surge pressures. To mitigate excessive surge pressures during a pump trip event, non-return valves were recommended at the following locations:

- Pipeline from raw water pump station to break pressure tank = at chainage 4100 m, approximately 100 m upstream of the break pressure tank,
- Pipeline from high-lift pump station to Command Reservoir No 1 = at chainage 38 500 m, and
- Pipeline from booster pump station to Command Reservoir No 2 = at chainage 43 000 m.

The maximum design and field test pressure for each pipeline was determined in accordance with DWS1110, which states that "Test pressures will generally be 1.25 times the pipeline design pressure for design pressures up to and including 3.2 MPa and 1.1 times the design pressure for higher pressures". Table 17-2 summarises the maximum design and field test pressures for the various pipeline sections.

Table 17-2: Maximum design and field test pressures

Pipe section	Pipe diameter (mm)	Maximum design pressure (m)	Maximum field test pressure (m)	Maximum pressure rating of valves, specials, etc. (m)
Gariep Dam to Xhariep WTP	1800	110	138	160
	2000	110	138	160
High-lift pump station to Command Reservoir No 1	1800	377	415(1)	400
Command Reservoir No 1 to suction reservoir at booster pump station	1800	276	345	400
Booster pump station to Command Reservoir No 2	1800	195	244	250
Command Reservoir No 2 to Rustfontein WTP	1400	203	254	250

The pump station layouts were based on the sizes of the mechanical and electrical equipment required. Provision was made for storage rooms, offices, loading bays and control rooms at each pump station.

The analysis of the raw water quality indicates the following treatment requirements after evaluation and consideration of the errant data:

- Turbidity levels are moderately high and require treatment intervention.
- Total aluminium levels are high. The dissolved aluminium levels are very low and do not require treatment. The non-dissolved fraction will be removed along with general turbidity if care is taken not to re-dissolve the aluminium through extensive pH manipulation.
- The microbiological indicators will be removed adequatly through normal disinfection protocols. No positive results were noted for chlorine resistant cysts or oocysts in any of the water qualty data sets.
- A small number of datapoints reflected very high dissolved organic carbon (DOC) levels. All the high values were however reported prior to 2003. Subsequent reports all indicated DOC levels below the national standard. No specific treatment regimes are included to address organic carbon removal.
- Both the Ryznar index as well as CCPP indicate that the water is aggressive and will require stabilisation.
- The additional sampling of the water source and analysis for a significantly expanded set of parameters indicated little to no risk associated with chlorophyll-a or contaminants of emerging concern at the proposed treatment plant.
- Some historical data sets indicate a number of determinands are present at levels of concern. The high initial values are likely the result of laboratory detection limits exceeding the specified water quality targets.The latest data sets indicate that these values, when appropriately analysed, are below levels of concern.

Based on the available water quality data the water can be described as of very good quality. Turbidity, microbiology, and stability are the only determinants requiring particular attention. Conventional flocculation, settling and filtration is proposed.

The key WTP design aspects are:

- Laboratory tests indicate that the preferred flocculant for treatment of the source water is a poli-aluminium chloride flocculant.
- An options analysis indicates that hydrated lime at around 5 to 10 mg/ℓ is the most economical approach to stabilisation.
- The treatment technology proposed for the WTP can be described as conventional:
    - Flocculation and clarification will take place in a pulsator clarifier. A total of 12 separate pulsators will be required to deliver 312 Mℓ/d.

- The rapid gravity sand filters are designed for deep penetration of floc into the filter bed. A total of 30 filters, with a surface area of 84.7 m2 each, are required to deliver 312 Mℓ/d. The filter loading rate is 6 m/hr with all 30 filters operational and 6.67 m/hr if three filters have been removed from operation for maintenance purposes.

- A chlorine demand study was undertaken that indicated a single chlorine dose at the treatment plant would not be sufficient to sustain chlorine levels in the transfer system. Chloramination cannot be considered as Mangaung Metropolitan Municipality (MMM) does not presently receive chloraminated water from its other sources and the mixing of chloraminated and non-chloraminated water cannot be permitted. The system will therefore be chlorinated with a booster injection of chlorine at the Booster Pump Station at which time the water will have been in the transfer system for around 48 hours. A comparison of chlorination systems indicated that chlorine gas systems are more economical than on-site chlorine generation systems.
- Main disinfection will take place in the on-site storage reservoir.
- The storage reservoir make provision for another 30 minutes of storage for high lift pump balancing purposes.
- Treatment residuals will be thickened in sludge ponds. An options analysis was undertaken to compare various pond construction approaches. A Hyson cell lined pond was found to be most cost effective.

Based on the pipe diameters and operating pressures, steel was considered the only feasible pipe material for the project. Grade X52 steel, with a yield strength of 358 MPa, is recommended. The pipeline structural design was based on AWWA M11 guidelines, but using the factors of safety recommended by DWS, i.e. a factor of safety of 1.67 for both the working and surge pressures. It was calculated that wall thicknesses will vary from 8 mm on the DN 1400 pipelines to up to 22 mm on the DN 1800 pipeline, immediately downstream of the high-lift pump station.

Various options are available for the pipe lining (e.g. cement mortar, epoxy) and coating (e.g. polymer modified bitumen, fusion bonded medium density polyethylene, trilaminate polyethylene, rigid polyurethane, etc.). The preferred lining and coating need to be selected during the detailed design phase in consultation with the entity responsible for the operation and maintenance of the pipelines.

Other pipeline aspects considered, included the sizing of air valves and scour valves, the installation of inline isolation valves, the provision of off-takes to end-users from the bulk pipelines, river and stream crossings, road crossings and dealing with existing services.

Three types of reservoirs were considered for command reservoirs with a storage capacity of 80 Mℓ, namely (a) conventional above ground post-tensioned circular reinforced concrete reservoirs, (b) conventional above ground circular or rectangular reinforced concrete reservoirs, and (c) earth-fill embankment type reinforced concrete lined reservoirs. It was established that earth-fill embankment type reinforced concrete lined reservoirs will be the most economical of the three reservoir types.

Table 17-3 provides a summary of the capital cost estimate for the Xhariep Pipeline Project.

Table 17-3: Estimated Capital Expenditure for the Xhariep Pipeline Project

Description	Estimated CAPEX (ZAR)
Preliminary and General	3,939,328,274
Raw Water Pump Station	162,443,508
High-Lift Pump Station	359,650,870
Command Reservoir No 1	137,008,420
Booster Pump Station and Suction Reservoir	292,708,069
Command Reservoir No 2	304,276,170
Pipelines	14,385,273,022
Water Treatment Works (Phase 1)	2,248,730,000

Description	Estimated CAPEX (ZAR)
Subtotal Capital Cost (Excl. VAT)	**21,829,418,333**
Contract Price Adjustment (CPA) @ 7% p.a.	4,912,557,790
Allowance for Foreign Exchange Variation @ 5% p.a.	434,230,000
Contingency @ 15%	4,076,430,918
Project Cost (Excl. VAT)	**31,252,637,041**
Engineering Design Fees @ 8%	2,500,210,963
Disbursements and Recoverable Costs	124,960,856
Professional Fees (Excl. VAT)	**2,625,171,819**
Total Project Cost (Excl. VAT)	**33,877,808,861**
Total Project Cost (Incl. VAT)	**38,959,480,190**

Notes:
1 Cost Estimate Base Date – November 2024.
2 Construction Commencement Date – November 2028

The estimated operation and maintenance budget required for the first year of operation is summarised in Table 17-4, showing an estimated minimum O&M budget requirement.

Table 17-4: Estimated Annual Operation and Maintenance Budget for the Xhariep Pipeline Project

Description	Estimated Annual O&M Budget (ZAR)	
	Complete Phase 312 Mℓ/d [1]	Phase 1 208 Mℓ/d [1]
Maintenance	200,782,452	184,932,452
Labour	24,180,000	21,160,000
Energy	82,533,048	53,752,032
Chemicals	71,590,000	47,730,000
Sludge Disposal	13,940,000	9,300,000
Total OPEX (Excl. VAT)	**393,025,500**	**316,874,484**
Plant Cost (Amortised @ 20 years)	633,040,000	475,680,000
Total Annual Cost of Ownership (Excl. VAT)	**1,026,065,500**	**792,554,484**

Notes:
1 Estimated other operational cost required for first year of plant operation based on 2024 Costs

The socio-economic assessment reiterated that the implementation of the Xhariep Pipeline is essential to ensure long-term water security and economic stability in the Greater Bloemfontein Water Supply System (GBWSS) region.

Five sources of financing are available for a public infrastructure project of this nature, i.e., (a) grants from central government, (b) the public delivery agency's own resources, (c) equity, (d) commercial debt, and (e) concessionary debt. If funding options are considered, the option of a 75% capital grant and a loan for the balance of the capital costs at a low interest rate is probably the first of the different funding options that will ensure that the project is affordable to households.

From an economic perspective, the project is expected to generate substantial socio-economic benefits through direct, indirect, and induced impacts. This includes job creation during the construction and operational phases and an improvement in water security, which is critical for supporting regional economic activities, particularly in the agriculture and manufacturing sectors. The financial viability of the project is achievable if tariff structures are well managed, ensuring affordability for households while maintaining financial sustainability.

The assessment highlights that, while various institutional options exist, the financial sustainability of the pipeline project will be dependent on the chosen implementing entity's capacity to manage operations and maintenance effectively. The report findings underscores that the project's success hinges on collaboration between national, provincial and local governments, and a clear delineation of responsibilities among entities. Based on this, it is recommended that DWS should initiate the establishment of and lead a **Working Group** that involves all the relevant stakeholders, with representation at an executive and strategic level, so that agreement can be reached on:

- Responsibilities with respect to the implementation, operation and maintenance of the scheme, e.g. MMM (as they should take a leading role) can request VCWB (legally) or DWS (administratively) (including TCTA or the NWRIA) to implement the project on their behalf,
- Financing options, taking consideration that at least a 75% capital grant and a loan for the balance of the capital costs at a low interest rate would be required to result in affordable bulk water tariff increases. The creditworthiness of each institution must be considered as part of the financing options to minimise the cost impact on the end-users, and,
- Development of an implementation timeframe.

It is estimated that, if a professional service provider for the detailed design phase can be appointed towards the end of 2025, construction could commence towards the end of 2028 with commissioning taking place at end 2032.

Appendix A

Book of Drawings (separate volume)