water & sanitation
Department:
Water and Sanitation
REPUBLIC OF SOUTH AFRICA

Report Number: P WMA 06/D00/00/3423/9

Directorate: Water Resource Development Planning
Department of Water & Sanitation
Private Bag X313
Pretoria
0001
South Africa

Greater Mangaung Water Augmentation Project

Feasibility Design Report – Pipelines, Pump Stations and Reservoirs

ZUTARI
IMPACT. ENGINEERED.

Submission date: 2025/02/28
Revision: B

water & sanitation

Department:
Water and Sanitation
REPUBLIC OF SOUTH AFRICA

Directorate Water Resource Development Planning

Feasibility Design Report – Pipelines, Pump Stations and Reservoirs

APPROVAL

Title	:	Feasibility Design Report – Pipelines, Pump Stations and Reservoirs
Project Name	:	Xhariep Pipeline Feasibility Study
DWS Report Number	:	P WMA 06/D00/00/3423/9
PSP Name	:	Zutari (Pty) Ltd
Report status	:	Final
Date	:	2025/02/28

CONSULTANT: ZUTARI (PTY) LTD
Approved for the Consultant:

Stephan Kleynhans

Signed by Stephan Kleynhans,
stephan.kleynhans@zutari.com

S KLEYNHANS
Design Director | Study Leader

DEPARTMENT OF WATER & SANITATION
Directorate: Water Resource Development Planning
Approved for Department of Water & Sanitation:

M MUGUMO
Chief Engineer: Water Resource Development Planning

C FOURIE
Director: Water Resource Development Planning

Document control record

Document prepared by:

Zutari (Pty) Ltd

Reg No 1977/003711/07

1 Century City Drive
Waterford Precinct
Century City
Cape Town

South Africa

PO Box 494
Cape Town
8000
Docex: DX 204

T +27 21 526 9400

E capetown@zutari.com

Document Control						
Project name		Feasibility Design Report – Pipelines, Pump Stations and Reservoirs				
Document number		P WMA 06/D00/00/3423/9	**Project number**			
Client		Department of Water and Sanitation				
Client contact		Ms. Sanet van Jaarsveld	**Client reference**		WP11405-142213	
Rev	**Date**	**Revision details/status**	**Author**	**Reviewer**	**Verifier** **(if required)**	**Approver**
A	2025/02/28	Issue for Client Review	I vd Merwe			S Kleynhans
B	2025/02/28	Final	I vd Merwe			S Kleynhans
Current revision		**B**				

Approval			
Author signature		Approver signature	
Name	Izak van der Merwe	Name	Stephan Kleynhans
Title	Associate	Title	Design Director

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
2	Site Visit Report	P WMA 06/D00/00/3423/2
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
9	**Feasibility Design Report – Pipelines, Pump Stations and Reservoirs**	**P WMA 06/D00/00/3423/9**
10	Socio-Economic Impact Assessment & Legal, Institutional and Financing Report	P WMA 06/D00/00/3423/10
11	Feasibility Design Report – Water Treatment Works	P WMA 06/D00/00/3423/11
12	Land Matters	P WMA 06/D00/00/3423/12
13	DFFE Application Submission	P WMA 06/D00/00/3423/13
14	Public Participation Report	P WMA 06/D00/00/3423/14
15	DFFE BAR Submission	P WMA 06/D00/00/3423/15
16	Summary Feasibility Study Report	P WMA 06/D00/00/3423/16
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 Resource Analysis Report	P WMA 06/D00/00/3423/19

Reference

This report is to be referred to in bibliographies as:

Department of Water and Sanitation, South Africa. 2025. *Greater Manguang Water Augmentation Project – Xhariep Pipeline Feasibility Study: Feasibility Design Report – Pipelines, Pump Stations and Reservoirs*.

DWS Report Number: P WMA 06/D00/00/3423/9

Prepared by Zutari (Pty) Ltd

Executive Summary

The conclusions contain a detail description of the detailed feasibility design of the pipelines, pump stations and reservoirs and 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.

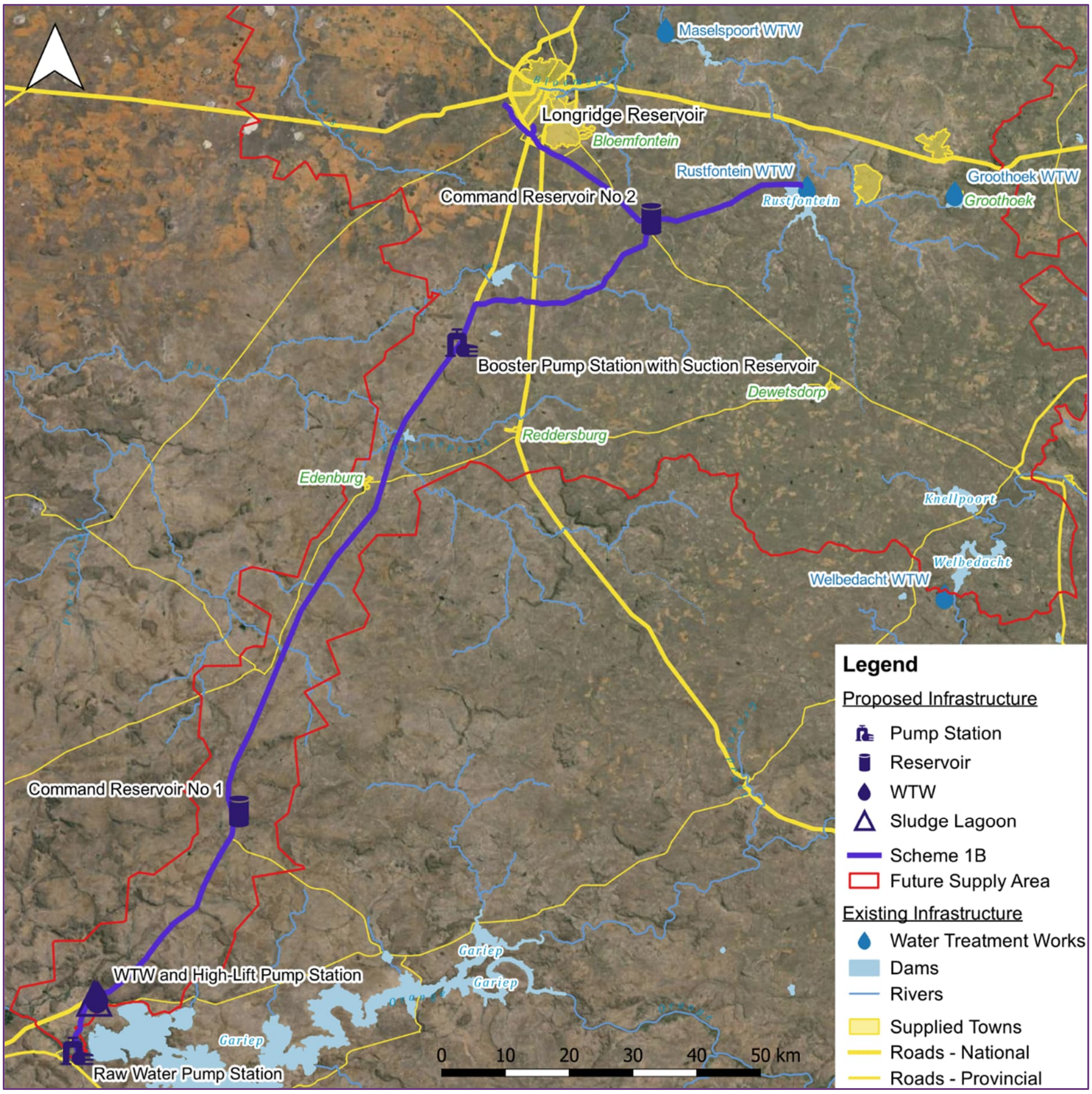


Figure E1: Main infrastructure components of Scheme 1B

A long-term stochastic analysis confirmed that the proposed potable transfer scheme at a capacity of 101 million m^3/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 m^3/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 m^3/s (312 Mℓ/d). The two command reservoirs were sized for 6 hours storage at the peak week flow rate of 3.616 m^3/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.

The main infrastructure components of Scheme 1B, as shown in Figure 15-1, 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 (WTW), ± 9 km long,
- The Xhariep WTW, 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 WTW 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 WTW (± 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 WTW = DN 1400, and
- Pipeline from Command Reservoir No 2 to the existing Longridge Reservoirs = DN 2000.

The duty points for the three pump station (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 m^3/s @ 73m	3.616 m^3/s @ 320m	3.616 m^3/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 pumpsets 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
	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.

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 Infrastructure, excluding the Xhariep WTP.

Table E3: Estimated Capital Expenditure for the Xhariep Pipeline Infrastructure

Description	**Estimated CAPEX Total cost (Rand)**
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
Subtotal capital cost (excl. VAT)	**19,580,688,333**
Contract Price Adjustment (CPA) @7% p.a.	4,406,496,844
Contingency @ 15%	3,598,077,777
Project cost (excl. VAT)	**27,585,262,954**
Professional fees (excl. VAT)	**2,317,162,088**
Engineering design fees @8%	2,206,821,036
Disbursements and recoverable costs	110,341,052
Total project cost (excl. VAT)	**29,902,425,042**

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 Infrastructure (excl. WTP)

Description	Estimated Annual Maintenance and Operating Budget (Rand)	
	Complete Phase 312 Mℓ/d [1]	**Phase 1 208 Mℓ/d** [1]
Maintenance	141,202,452	141,202,452
Labour	6,720,000	6,720,000
Energy	34,863,048	23,242,032
TOTAL OPEX (Excl. VAT)	**182,785,500**	**171,164,484**

Notes:

1 Estimated other operational cost required for first year of plant operation based on 2025 Costs

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.

Table of Contents

Appendices

Figures

Tables

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
RPST	Reconciliation Planning Support Tool

Acronym	Description
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.

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 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 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,
- Opportunities for phased implementation, and,
- Stakeholder preferences.

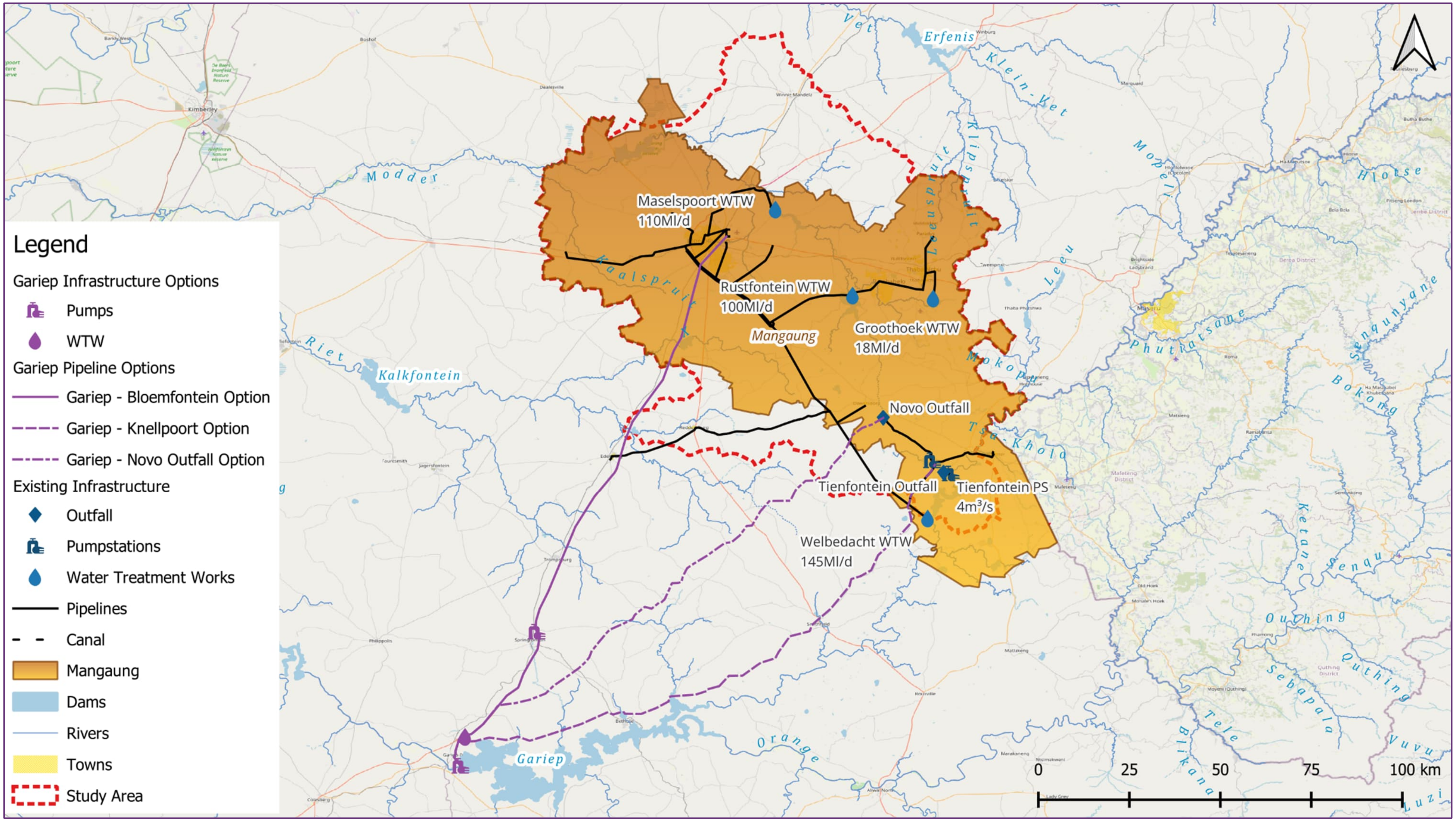


Figure 1-1: Previously studied route options from Gariep Dam to the GBWSS

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 Detailed Feasibility Study Report is present the findings from the detailed feasibility design undertaken for the preferred option identified during the Pre-Feasibility Phase of the project. The Detailed 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 brief 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 feasibility design of the pipelines.

Chapter 9 presents the detailed feasibility design of the reservoirs.

Chapter 10 describes the site access to the various major infrastructure components.

Chapter 11 summarises design aspects that will require special consideration during the detailed design phase of the project.

Chapter 12 provides a brief summary of the authorisation processes for the project.

Chapter 13 presents the updated construction and project cost estimates.

Chapter 14 contains the project implementation programme.

Chapter 15 contains the conclusions.

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 m^3/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 m^3/s;
- A re-use scheme with a yield of 12.9 million m^3/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 m^3/a is capable of meeting 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 Department of Water and Sanitation (DWS) developed design and planning criteria in a document entitled “Technical Guidelines for the Development of Water and Sanitation Infrastructure”. The relevant criteria that were used to calculate the design capacities of the various infrastructure components is summarised in Table 2-1.

Table 2-1: Design criteria for infrastructure planning

Description / Criteria	DWS recommendation	Applied to Xhariep Pipeline project
Water treatment works loss	10%	5% (1)
Conveyance loss	10%	0% (2)
Gross AADD	AADD * (1 + losses)	AADD * (1 + losses)
Summer peak factor	1.2 to 1.5	1.2
Pump duration factor	20 hours/day	22 hours/day (3)
Break pressure tanks / Command reservoirs	30 minutes * pumped inflow	6 hours * GAADD (4)

(1) Water treatment works designed with recovery system for backwash water, which will reduce losses to 5%.

(2) 10% conveyance losses already accounted for in the water demand projections

(3) A pump factor of 24/22 = 1.09 is recommended due to the additional storage provided at the command reservoirs.

(4) The DWS design guidelines recommend that break pressure tanks be sized for a minimum capacity of 30 minutes x the pumped inflow, but also states that reservoirs used for pump control should preferably be sized for a minimum of 4 hours x GAADD. The 6 hours x GAADD recommended for the Xhariep Pipeline project also considers the filling of pipelines due to maintenance requirements.

Based on Table 2-1, the following peak factors were applied:

- Raw water pipelines

Q_{raw} = AADD x losses(5% + 10%) x Summer Peak Factor(1.2) x Pump Duration Factor(1.09)
= 1.37 x AADD

- Potable water pipelines:

Q_{pot} = AADD x Summer Peak Factor (1.2) x Pump Duration Factor (1.09)
= 1.31 x AADD

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.

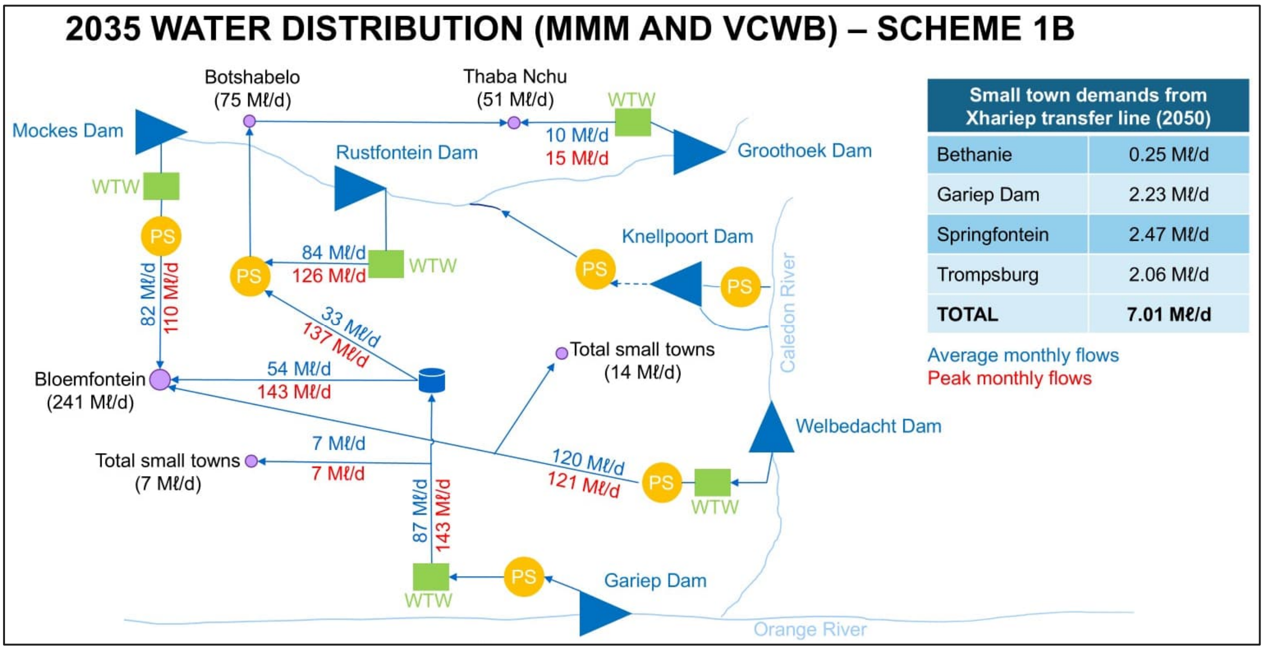


Figure 2-1: Average and peak monthly flow rates (2035)

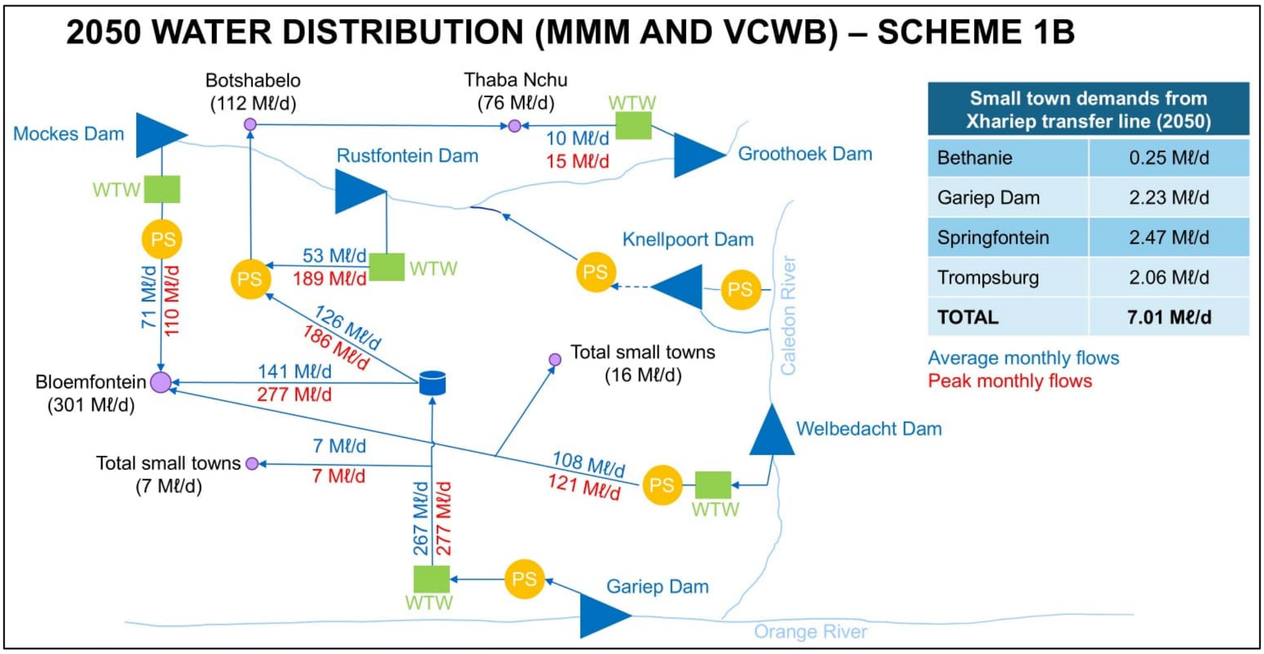


Figure 2-2: Average and peak monthly flow rates (2050)

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

Table 2-2: 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 (m^3/s)
Raw water pipeline and pump station	277	1.13 x1.05	329	3.797
Water treatment works (WTW)	277	1.13	312	3.616
Potable water pump stations and pipelines (WTW 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 WTW	186	1.13	210	2.428

2.3 Proposed Bulk Water Infrastructure for Scheme 1B

Scheme 1B was the option approved by DWS at the end of the pre-feasibility study for which the detailed feasibility design needs to be undertaken. The main infrastructure components of Scheme 1B are shown in Figure 2-3 and summarised in Table 2-3. The pipeline diameters and lengths are based on the preliminary optimisation undertaken during the pre-feasibility study and will be revised/updated as part of the detailed feasibility study.

Table 2-3: Summary of main infrastructure components of Scheme 1B

Infrastructure component	Capacity / Size	Length (km)
Low-lift pump station	3,797 m^3/s	-

Infrastructure component	Capacity / Size	Length (km)
Raw water pipeline	1800 mm	10.5 km
Water treatment works	312 Mℓ/d	-
High-lift pump station	3,616 m^3/s	-
1st command reservoir	80 Mℓ (1)	-
Booster pump station	3,616 m^3/s	-
2nd command reservoir	80 Mℓ (1)	-
Potable pipeline from high-lift pump station to 2nd command reservoir	1800 mm	176.4 km
Potable pipeline from 2nd command reservoir to Bloemfontein	2000 mm	31.4 km
Potable pipeline from 2nd command reservoir to Rustfontein WTW	1400 mm	24.5 km

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

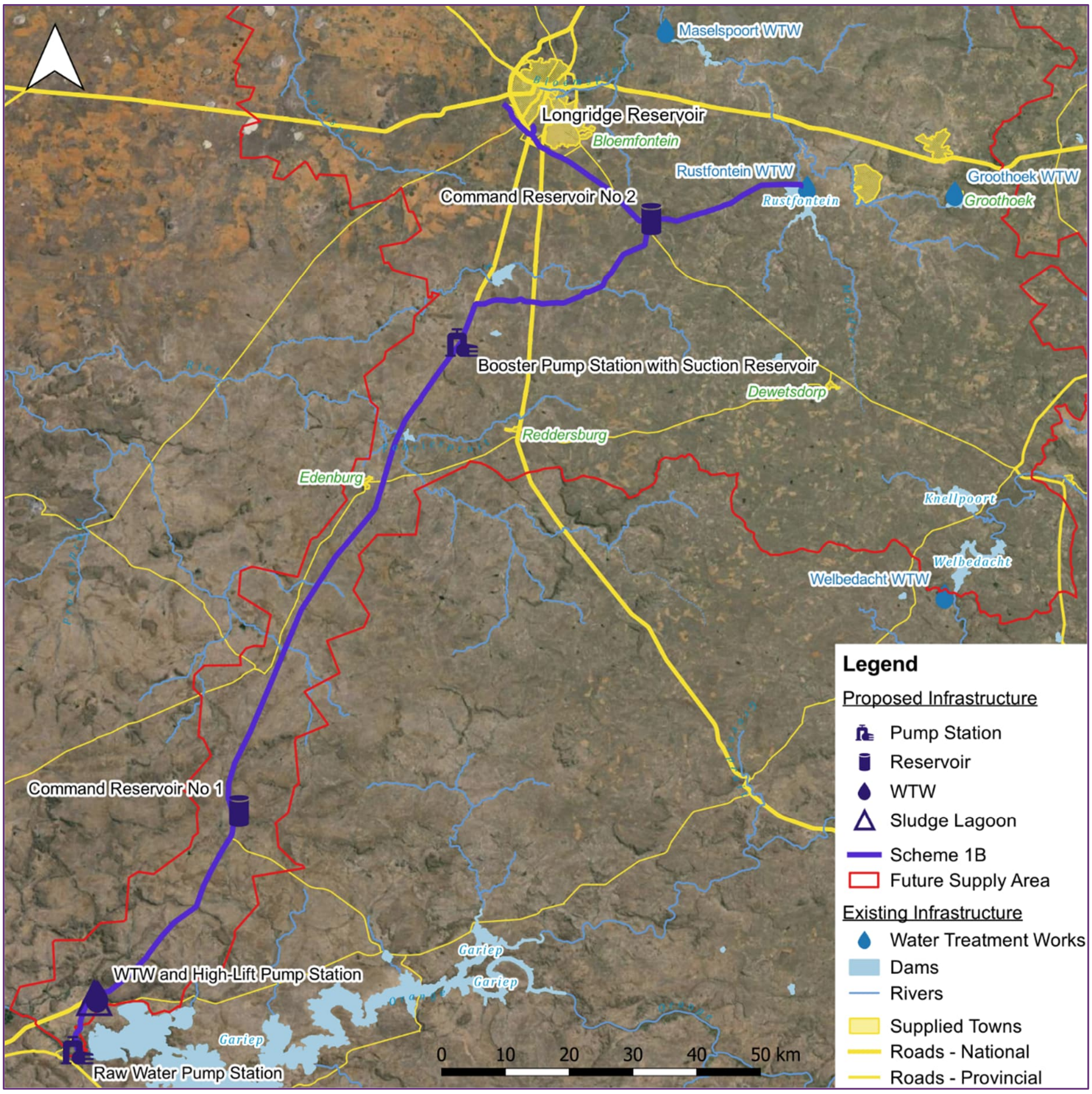


Figure 2-3: Main infrastructure components of Scheme 1B

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 (Report No. P WMA 06/D00/00/3423/8). A brief 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 include 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 (Report No. P WMA 06/D00/00/3423/7). A brief summary of the salient aspects from the report is provided below.

- The fieldwork investigation included:
- Test pits at 400 m intervals;
- In-situ Dynamic Cone Penetrometer (DCP) and Deep Probe Super Heavy (DPSH) tests;
- An electrical resistivity survey at 200 m intervals;
- Rotary core drilling, with in-situ Standard Pentration Testing (SPT) tests at 1.5 m intervals within the soil profile at the proposed structure positions, road / railway crossings and major river / stream crossings; and
- Laboratory testing.

As per the topographical survey, access was not available to certain sections of the project. The geotechnical field investigations need to be completed for these sections during the detailed design phase, especially for Command Reservoir No 2.

The investigation along the proposed pipeline route and at the major structures (i.e. pump stations, reservoirs and water treatment plant) found that:

- The ground conditions are generally characterized by shallow bedrock (0.1 m to 3.8 m below ground level) that is covered by a combination of silty sand, clayey silt and silty to sandy clay that has calcrete formation towards the bottom of the transported deposits. Occasional hardpan calcrete was also encountered. The bedrock mostly consists of interbedded sandstone, mudstone and shale from the Karoo Supergroup, that is locally intruded by dolerite.
- Deeper rock levels were noted along approximately 7% of the pipeline route which is generally related to wetlands, rivers and streams.
- Groundwater levels typically vary across the pipeline route, with seepage noted within 2 % of the test pits, ranging between 0.5 m to 3.8 m below surface. Here seepage is generally found within bedrock at intersecting stream and river crossings.
- The electrical resistivity survey found that approximately 83% of the surveyed sections along the pipeline route traverse soils which range from mildly corrosive to extremely corrosive to buried steel, which is confirmed by the pH and conductivity test results. Therefore, cathodic protection will be required. It is highly recommended that BRE/DIN chemical testing be done during the detailed design phase to be more accurate in determining the corrosivity of the soil towards steel and aggressiveness towards buried concrete.
- Samples taken were from test pits to determine the material quality to be considered for use during construction. The results showed that all of the excavatable material over the area generally classifies as G9 – >G9 according to COLTO specifications and is not suitable for use as bedding or engineered fill. Locally some of the colluvium, residual dolerite and residual sandstone tested as G7 - G6, but it is variable and good quality control would be required should this material be considered.
- With the pipe invert level between 3.0 – 4.5 m below surface, most of the pipeline will be situated within bedrock. Localised blasting is anticipated through hard rock dolerite.

4 Scheme Optimisation

4.1 Input parameters

The optimum diameters for especially the rising mains need 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.

Table 4-1 summarises some of the critical input parameters used in the optimisation model, as well as the values assigned to these parameters.

Table 4-1: Critical input parameters for optimisation

Input parameter	Assigned value(s)
Flow	Peak week demand
Growth in demand (1)	(a) Full demand over project life, and (b) Linear increase from 2023 to 2050
Pipeline roughness coefficient	0.6mm
Pipe material and diameter (2)	Grade X52 steel: 1600mm to 2400mm
Pipe wall thickness calculations (3)	(a) Hoop stress only (internal pressures only), and (b) Hoop stress and external loads
Pumping duration	365 days, 22 hours per day
Electricity costs	180 c/kWh
Costs functions	Steel pipe prices, excavation, hard rock excavation, bedding, pipeline structures and services, pump station costs (civil, mechanical, electrical), etc.
Mechanical and electrical replacement costs	Full mechanical and electrical replacement every 15 years
Discount rate	4% and 6%
Electrical inflation rate	0% and 4% above CPI inflation
Discount period	30 years
Maintenance costs	0.5% for civil infrastructure; 4.0% for mechanical and electrical infrastructure
Administrative costs (4)	1.0% of total construction cost

(1) Two options for demand growth were tested as part of the sensitivity checks – 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.

(2) The optimisation model is based on internal pipe diameters. The actual pipe diameter required would be a function of the wall thickness and internal lining thickness (for pipes with linings).

(3) 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.

(4) It was assumed that a 100% grant will be received, so no interest and redemption costs were allowed.

The optimisation will be done, commencing with a “base scenario”, using the following input parameters:

- Constant demand over project life;
- Wall thicknesses based only on hoop stress;
- Inflation in energy costs to be the same as the consumer price inflation; and
- A 4% discount rate.

Sensitivities of other scenarios will be test relative to the results of the “base scenario”, e.g. changing the discount rate to 6%.

4.2 Gariep Dam to Water Treatment Plant

The infrastructure included in the optimisation from Gariep Dam to the Water Treatment Plant (WTP) includes:

- The suction pipeline from Gariep Dam to the raw water pump station;
- The raw water pump station; and
- The rising main from the raw water pump station to the filling tank located at the high-point.

The pipeline section from the filling tank to the WTP will flow under gravity and is therefore not included in the optimisation, i.e. the diameter of this gravity section will be determined based on the maximum conveyance capacity. It was determined as part of the optimisation that the following factors did not significantly influence the calculated NPVs:

- Constant demand versus growing demand
- Wall thickness based on hoop stress only versus based on hoop stress and external loads (mainly due to reasonably low operating pressures)

As such, the above variables are not further discussed as part of the optimisation of the infrastructure from Gariep Dam to the WTP.

Table 4-2 summarises the NPV determined for the "base scenario" for all the infrastructure components from Gariep Dam to the WTP.

Table 4-2: Optimisation of Gariep Dam to WTP based on "base scenario"

Pipe Diameters	Total Capital Cost (Rand Million)	NPV of O&M Cost (Rand Million)	Total NPV (Rand Million)
DN1400	860	1,652	2,512
DN1600	864	1,612	2,476
DN1800	870	1,593	2,463
DN2000	879	1,585	2,464
DN2200	889	1,582	2,471

It is evident from Table 4-2 that a DN1800 pipeline is the most economical for the "base scenario".

Table 4-3 shows the calculated costs and NPV for the "base scenario" but escalating the energy costs at 4% above the rate of inflation.

Table 4-3: Optimisation of Gariep Dam to WTP based on "base scenario" energy at 4% above inflation

Pipe Diameters	Total Capital Cost (Rand Million)	NPV of O&M Cost (Rand Million)	Total NPV (Rand Million)
DN1400	860	2,067	2,927
DN1600	864	2,015	2,880
DN1800	870	1,990	2,860
DN2000	879	1,980	2,859
DN2200	889	1,975	2,864

It can be seen from Table 4-3 that the total NPV for the DN1800 and DN2000 pipeline diameters is similar.

Table 4-4 shows the calculated costs and NPV for the "base scenario" but using a 6% discount rate instead of 4%.

Table 4-4: Optimisation of Gariep Dam to WTP based on "base scenario" with 6% discount rate

Pipe Diameters	Total Capital Cost (Rand Million)	NPV of O&M Cost (Rand Million)	Total NPV (Rand Million)
DN1400	860	1,301	2,161

Pipe Diameters	Total Capital Cost (Rand Million)	NPV of O&M Cost (Rand Million)	Total NPV (Rand Million)
DN1600	864	1,270	2,134
DN1800	870	1,255	2,125
DN2000	879	1,249	2,128
DN2200	889	1,247	2,136

It is evident from Table 4-4 that a change in discount rate from 4% to 6% does not impact the optimum pipeline diameter. A DN1800 pipeline diameter remains the diameter with the lowest total NPV cost.

The calculated costs and NPV for the "base scenario" but allowing an inflation in energy cost of 4% higher than CPI, as well as a discount rate of 6%, are shown in Table 4-5.

Table 4-5: Optimisation of Gariep Dam to WTP based on "base scenario" with 4% additional inflation in energy cost and 6% discount rate

Pipe Diameters	Total Capital Cost (Rand Million)	NPV of O&M Cost (Rand Million)	Total NPV (Rand Million)
DN1400	860	1,583	2,443
DN1600	864	1,544	2,408
DN1800	870	1,525	2,395
DN2000	879	1,517	2,396
DN2200	889	1,513	2,402

It is evident from Table 4-5that an increase in energy tariffs, higher than annual inflation, as well as an increased discount rate of 6%, does not change the optimum pipeline diameter.

Based on all the sensitivity checks undertaken, the optimum pipeline diameter for each optimisation option/scenario tests remained a DN1800 pipeline.

4.3 High-Lift Pump Station to Command Reservoir No 2

The infrastructure included from the high-lift pump station (HLPS) to Command Reservoir No 2 was optimised as a single analysis. The infrastructure includes:

- The high-lift pump station at the water treatment works;
- The rising main from the high-lift pump station to Command Reservoir No 1;
- Command Reservoir No 1;
- The gravity pipeline from Command Reservoir No 1 to the booster pump station;
- The booster pump station;
- The rising main from the booster pump station to Command Reservoir No 2; and
- Command Reservoir No 2.

The reason for including all of the above infrastructure into a single optimisation is due to the interdependencies between the infrastructure components, e.g. if the full supply level of Command Reservoir No 1 is increased, the pumping head and energy cost of the high-lift pump station will increase, but it might also be possible to reduce the size of the gravity pipeline from Command Reservoir No 1 to the booster pump station.

Table 4-6 summarises the NPV determined for the "base scenario" for all the infrastructure components from the HLPS to Command Reservoir No 2.

Table 4-6: Optimisation of HLPS to Command Reservoir No 2 based on "base scenario"

Pipe Diameters	Total Capital Cost (Rand Million)	NPV of O&M Cost (Rand Million)	Total NPV (Rand Million)
DN1600	13,669	18,949	32,618

Pipe Diameters	Total Capital Cost (Rand Million)	NPV of O&M Cost (Rand Million)	Total NPV (Rand Million)
DN1800	13,902	17,732	31,634
DN2000	14,300	17,218	31,518
DN2200	14,862	17,027	31,889
DN2400	15,490	16,998	32,488

It is evident from Table 4-6 that a DN2000 pipeline will have the lowest total NPV, with a DN1800 pipeline having the second lowest total NPV.

Table 4-7 shows the calculated costs and NPV for the “base scenario” but determining the wall thicknesses based on hoop stress and external loads.

Table 4-7: Optimisation of HLPS to Command Reservoir No 2 based on “base scenario” with hoop stress and external loads

Pipe Diameters	Total Capital Cost (Rand Million)	NPV of O&M Cost (Rand Million)	Total NPV (Rand Million)
DN1600	15,999	18,428	34,427
DN1800	16,078	17,173	33,251
DN2000	16,284	16,613	32,897
DN2200	16,729	16,394	33,123
DN2400	17,356	16,364	33,720

It can be seen from Table 4-7 that the optimum pipeline diameter remains DN2000 when considering hoop stress and external loads as part of the pipeline wall thickness calculations.

Table 4-8 shows the calculated costs and NPV for the “base scenario” but using a 6% discount rate instead of 4%.

Table 4-8: Optimisation of HLPS to Command Reservoir No 2 based on “base scenario” with 6% discount rate

Pipe Diameters	Total Capital Cost (Rand Million)	NPV of O&M Cost (Rand Million)	Total NPV (Rand Million)
DN1600	13,669	17,864	31,533
DN1800	13,902	16,647	30,549
DN2000	14,300	16,133	30,433
DN2200	14,862	15,942	30,804
DN2400	15,490	15,913	31,403

It is evident from Table 4-8 that a change in discount rate from 4% to 6% does not impact the optimum pipeline diameter. A DN1800 pipeline diameter remains the second lowest total NPV cost.

The calculated costs and NPV for the “base scenario” but allowing an inflation in energy cost of 4% higher than CPI, are shown in Table 4-9.

Table 4-9: Optimisation of HLPS to Command Reservoir No 2 based on “base scenario” with 4% additional inflation in energy cost

Pipe Diameters	Total Capital Cost (Rand Million)	NPV of O&M Cost (Rand Million)	Total NPV (Rand Million)
DN1600	13,669	21,577	35,246
DN1800	13,902	20,030	33,933
DN2000	14,300	19,359	33,659
DN2200	14,862	19,083	33,945
DN2400	15,490	19,008	34,497

It is evident from Table 4-9 that an increase in energy tariffs, higher than annual inflation, does not change the optimum pipeline diameter.

Table 4-10 is the same scenario as shown in Table 4-9, except that the capital costs and NPV are based on a growing water demand pattern.

Table 4-10: Optimisation of HLPS to Command Reservoir No 2 based on "base scenario" with 4% additional inflation in energy cost and growing water demand pattern

Pipe Diameters	Total Capital Cost (Rand Million)	NPV of O&M Cost (Rand Million)	Total NPV (Rand Million)
DN1600	13,669	21,586	35,255
DN1800	13,902	19,981	33,884
DN2000	14,300	19,280	33,580
DN2200	14,862	18,989	33,851
DN2400	15,490	18,904	34,393

It is evident from Table 4-10 that the optimum pipeline diameter will remain a DN2000 pipeline.

Table 4-11 shows the capital costs and NPV when allowing energy tariffs to grow by 4% per annum faster than inflation, assuming a growing water demand pattern and applying a 6% discount rate.

Table 4-11: Optimisation of HLPS to Command Reservoir No 2 based on "base scenario" with 4% additional inflation in energy cost, growing water demand pattern and 6% discount rate

Pipe Diameters	Total Capital Cost (Rand Million)	NPV of O&M Cost (Rand Million)	Total NPV (Rand Million)
DN1600	13,669	18,714	32,383
DN1800	13,902	17,485	31,387
DN2000	14,300	16,963	31,263
DN2200	14,862	16,769	31,631
DN2400	15,490	16,737	32,227

It is evident from Table 4-11 that the optimum pipeline diameter remains a DN2000 pipeline.

Based on all the sensitivity checks undertaken, the optimum pipeline diameter for each optimisation option/scenario tests remained a DN2000 pipeline. In the majority of instances, a DN1800 pipeline had the second lowest total NPV.

The ultimate decision with respect to the optimum pipeline diameter should not only be based on the NPV, but cognisance should also be taken of the following considerations:

- ► The difference in NPV and capital costs between the most economical and second most economical pipeline diameters
  - The total NPV of a DN1800 pipeline was less than 1% of the total NPV of a DN2000 pipeline in most cases, i.e. the NPVs are very similar.
  - The capital cost of a scheme with DN1800 pipelines is typically R 200 million to R 400 million lower than a scheme with DN2000 pipelines.
  - Based on financial considerations, the costs associated with a DN1800 and DN2000 pipeline are almost identical.
- ► The actual operating velocities in the pipelines
  - At peak week flows, the velocities in a DN1800 and DN2000 pipelines are 1.42m/s and 1.15m/s, respectively. Both these velocities are within the acceptable range for potable pipelines.
  - During periods of low demand, the monthly average flows can be approximately 70% of the AADD. The corresponding veloties in the DN1800 and DN2000 pipelines will then reduce to 0.77m/s and 0.62m/s, respectively.

  - The higher velocities that will be experienced in the DN1800 pipeline is preferred to minimise the decrease in the residual chlorine concentrations before the water is delivered to the end-users.

- The volume of water stored in the pipelines
  - The DN1800 pipeline will store 449 Mℓ of water compared to the 554 Mℓ that will be stored in the DN2000 pipeline.
  - Based on operational and maintenance considerations, e.g. scouring of the pipeline, refilling the pipeline from the command reservoirs, etc., it should be more advantageous to install a DN1800 pipeline to reduce the volumes of water to be dealt with.

- Any other assumptions that could influence the NPV calculations
  - It was assumed in the NPV calculations that a 100% grant will be obtained for the implementaion of the Xhariep Pipeline project. In the event that a loan is required, which will attract interest and redemption payments, the scheme with the lower capital cost is preferred, i.e. the DN1800 pipeline option.
  - The Xhariep Pipeline project will result in an increase in the water tariff payable by the end-users. This could result in a slower uptake in demand over time, meaning that the full capacity of the scheme could only be required after 2050. It is highly unlikely that the demands will exceed the demand projections, meaning that a DN2000 pipeline might result in an over-conservative demand.

Based on the above considerations, it is recommended that DN1800 pipelines be installed from the HLPS to Command Reservoir No 2.

4.4 Command Reservoir No 2 to Longridge Reservoirs

This pipeline section operates under gravity. The pipeline diameter is dictated by the available head (pressure) and the corresponding hydraulic capacities of the respective pipeline diameters. The hydraulic analysis of this pipeline section is discussed further in Chapter 6 of this report.

4.5 Command Reservoir No 2 to Rustfontein Water Treatment Works

This pipeline section operates under gravity. The pipeline diameter is dictated by the available head (pressure) and the corresponding hydraulic capacities of the respective pipeline diameters. The hydraulic analysis of this pipeline section is discussed further 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.

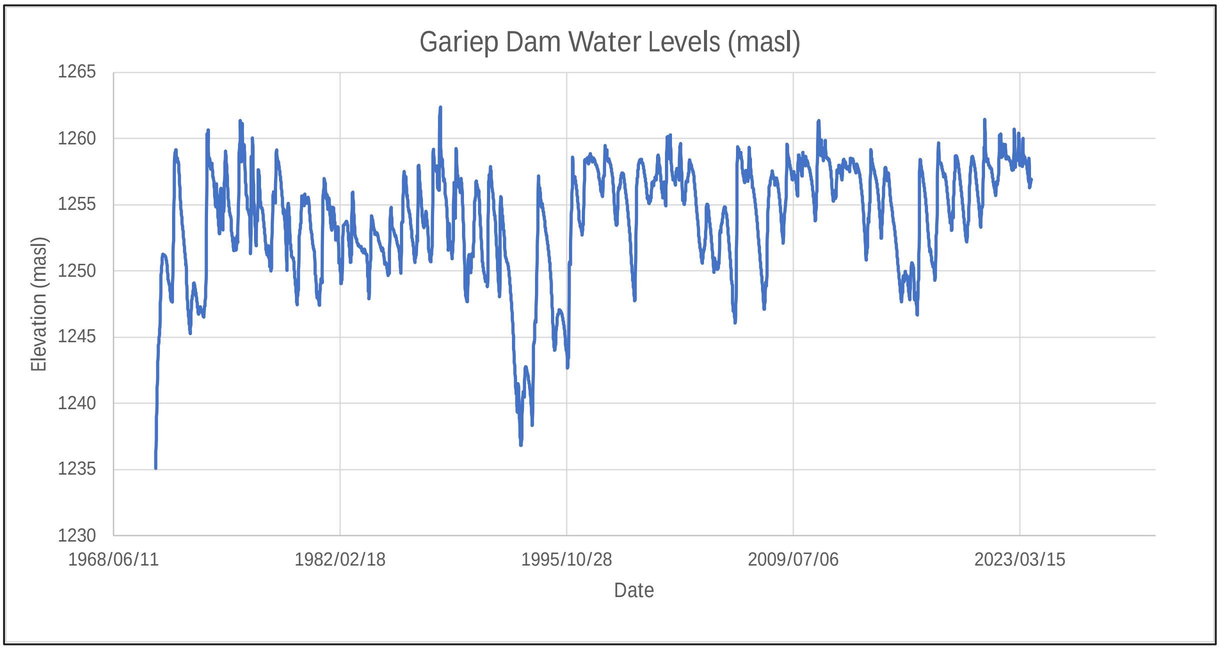


Figure 5-1: Weekly water levels of Gariep Dam (1971 to 2023)

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 m^3/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.

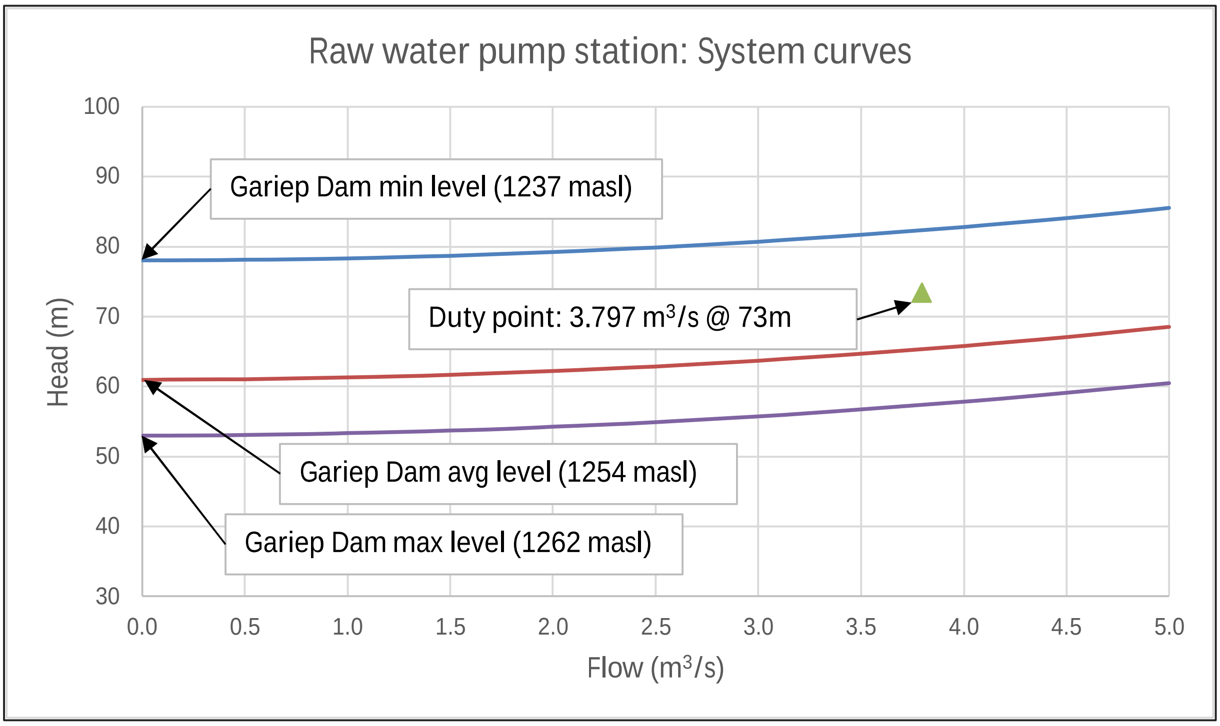


Figure 5-2: Raw water pump station system curves

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 m^3/s @ 320m head.

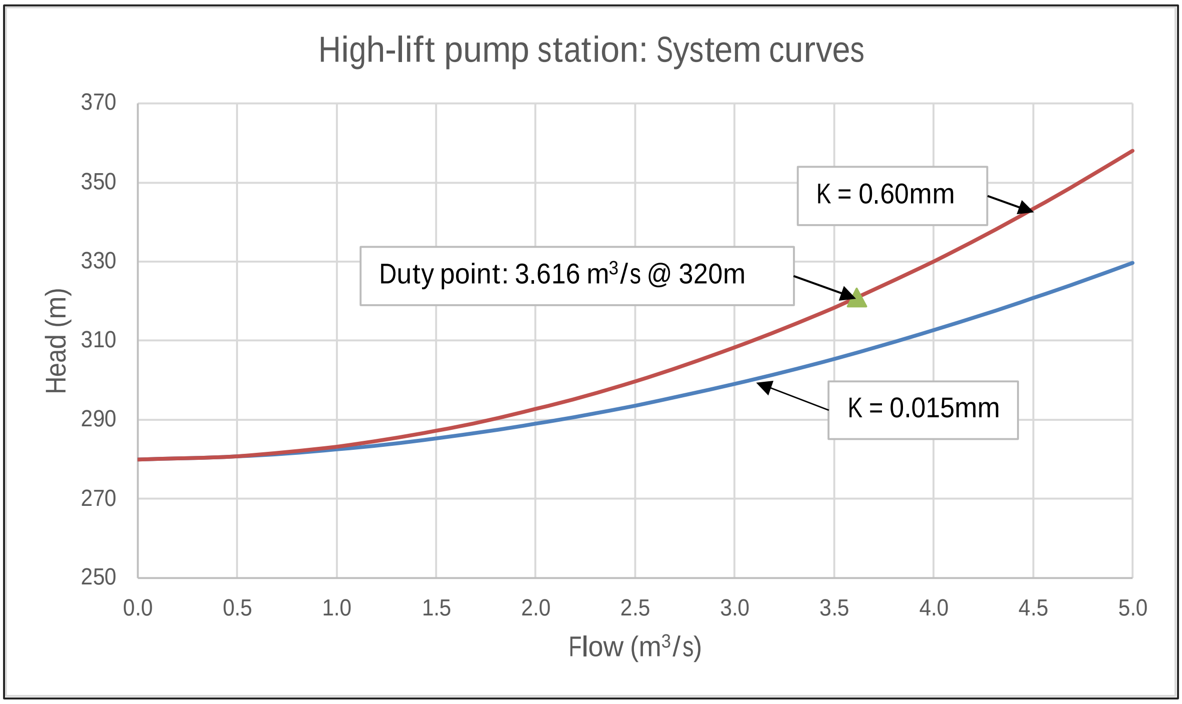


Figure 5-3: High-lift pump station system curves

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 m^3/s @ 127m head.

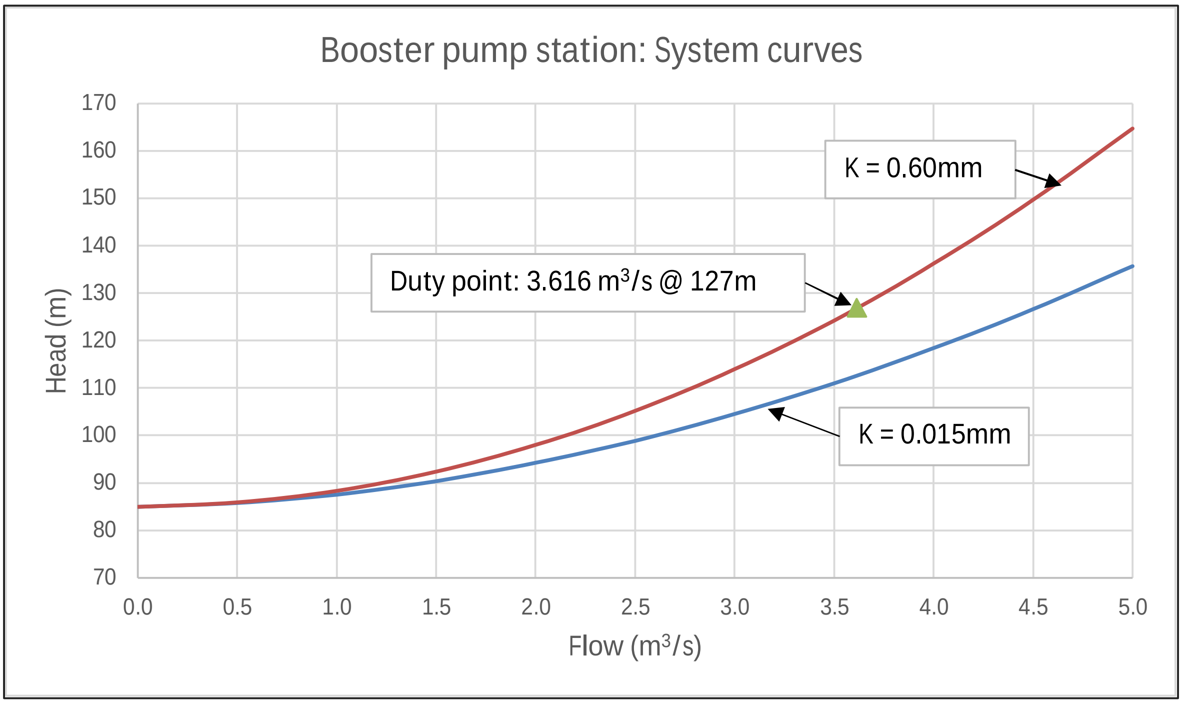


Figure 5-4: Booster pump station system curves

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. Further details on the pump curves and pump dimensions are included as part of Appendix A.

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
Impeller size at duty point (mm)	740
Full-size impeller (mm)	750
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
Suction size (mm)	600
Discharge size (mm)	500
Pump weight (kg)	5,193
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.
- It is preferable to select pumps where a smaller diameter impeller (e.g. 740 mm), compared to the full-size impeller (e.g. 750 mm), can be installed to achieve the required duty point. This provides flexibility in future in increase the impeller size, which will result in an increase in flow, provided that the motor size is adequate to handle the increased power requirements.
- 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.

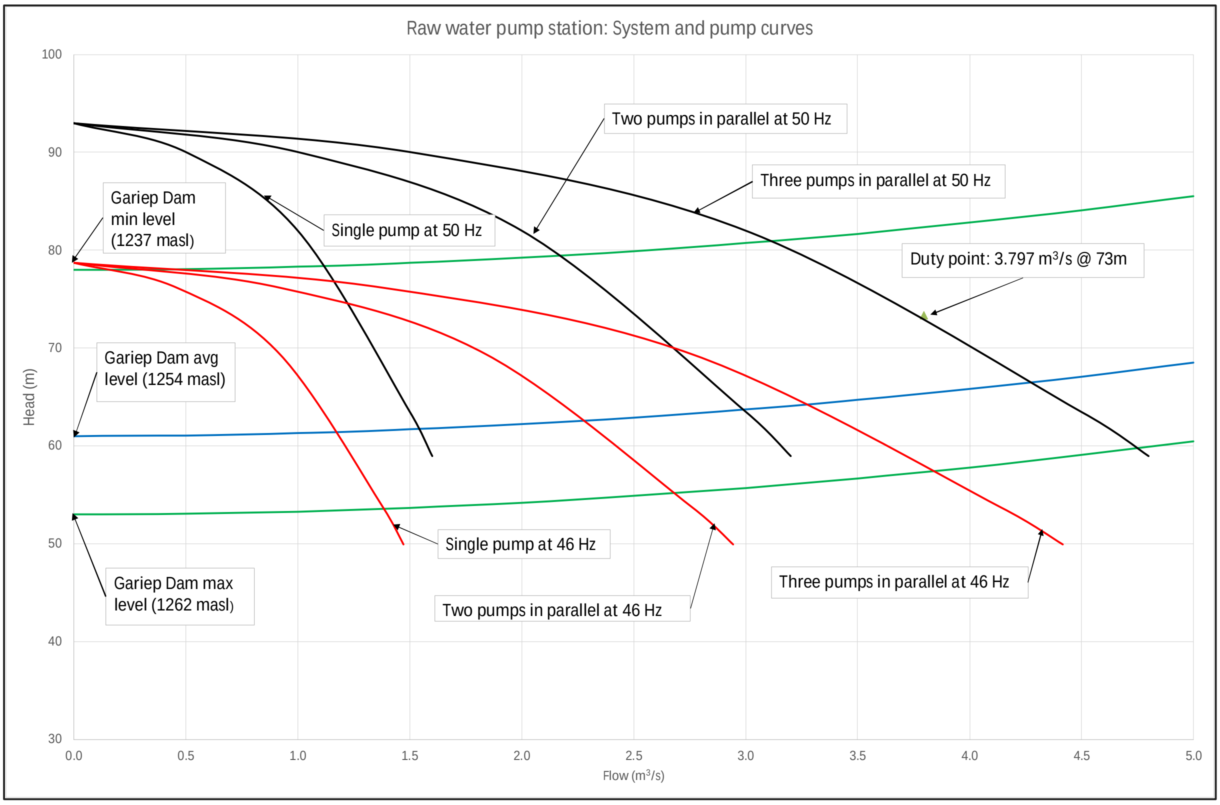


Figure 5-5: Raw water pump station – system and pump curves

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 m^3/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 m^3/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 m^3/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. Further details on the pump curves and pump dimensions are included as part of Appendix A.

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
Impeller size at duty point (mm)	1005
Full-size impeller (mm)	TBC
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
Suction size (mm)	550
Discharge size (mm)	450
Pump weight (kg)	15,450
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.

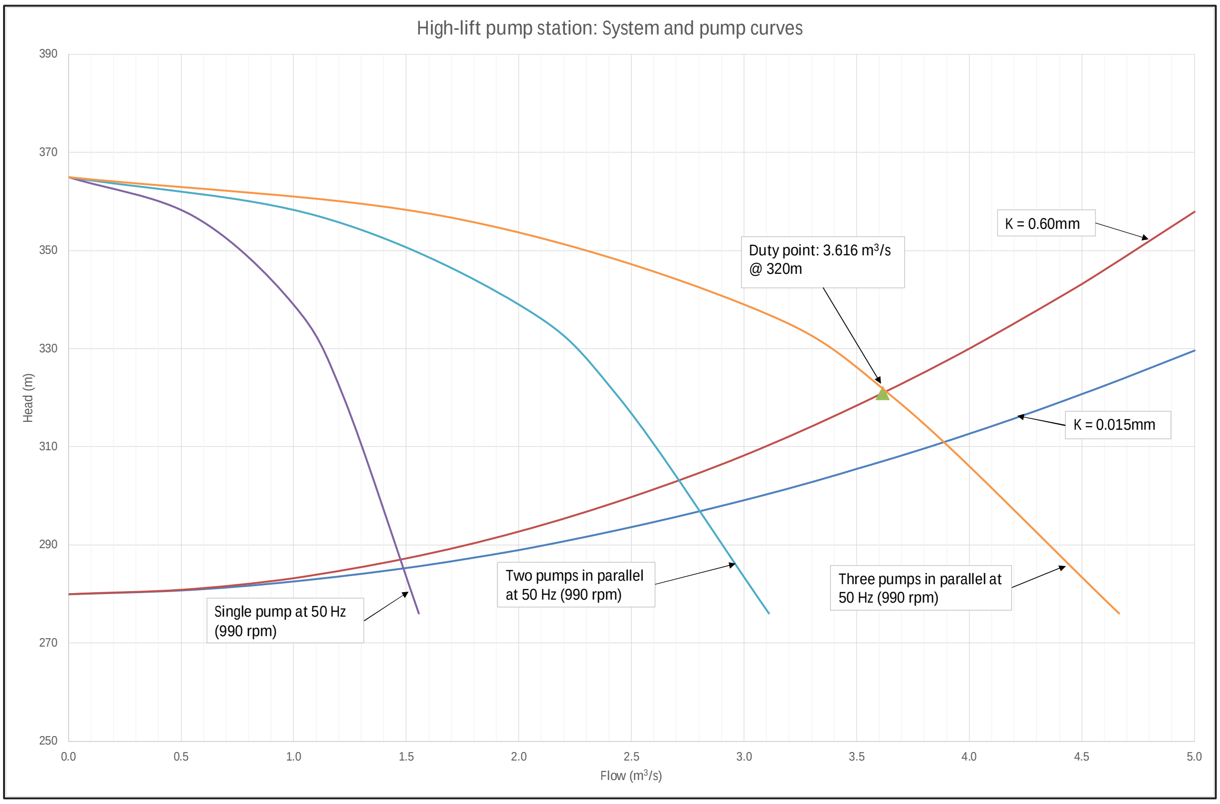


Figure 5-6: High-lift pump station – system and pump curves

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 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 3.9 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.

5.3.3 Booster Pump Station

The recommended pump selected for the booster pump station, to achieve the duty point of 3.616 m^3/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. Further details on the pump curves and pump dimensions are included as part of Appendix A.

Table 5-4: Booster pump selection details

Description	Details
Pump duty	3.616 m^3/s @ 127m

Description	Details
Pump Model	SMD 600-1250 B
Configuration (duty/standby)	3 duty, 1 standby
Impeller size at duty point (mm)	1221
Full-size impeller (mm)	1250
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
Suction size (mm)	800
Discharge size (mm)	600
Pump weight (kg)	11,776
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.
- It is preferable to select pumps where a smaller diameter impeller (e.g. 1221 mm), compared to the full-size impeller (e.g. 1250 mm), can be installed to achieve the required duty point. This provides flexibility in future in increase the impeller size, which will result in an increase in flow, provided that the motor size is adequate to handle the increased power requirements.
- 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.

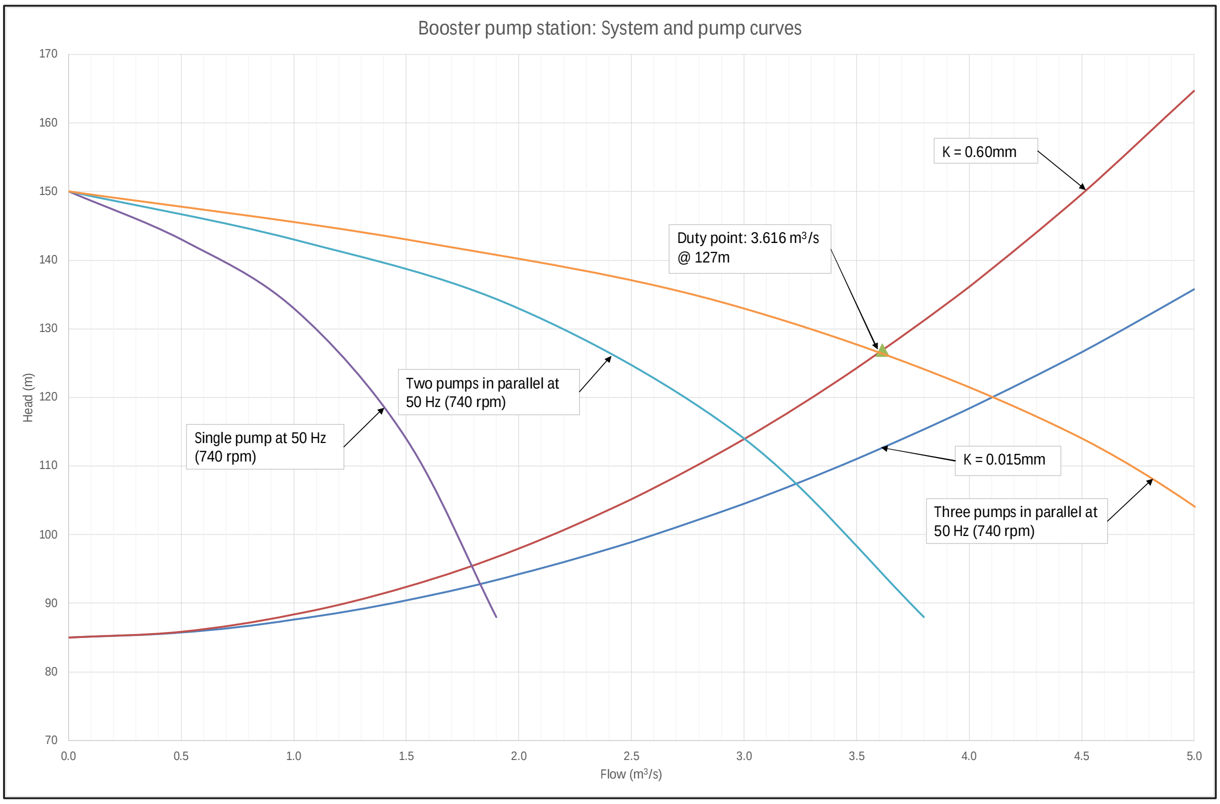


Figure 5-7: Booster pump station – system and pump curves

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 4.2, 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 m^3/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 m^3/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 m^3/s, as well as the PN10 (i.e. 100m) working pressure line.

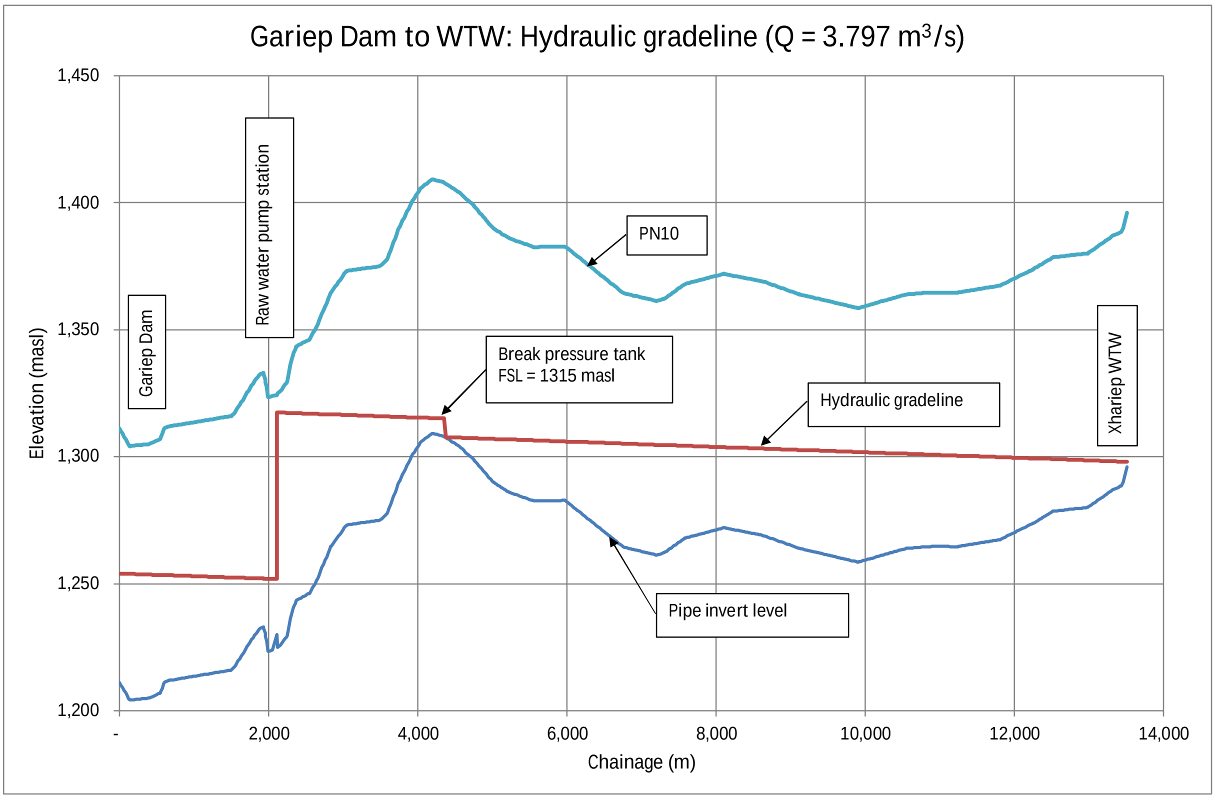


Figure 6-1: Gariep Dam to WTW – Hydraulic Gradeline

It is evident from Figure 6-1 that the high point located at chainage 4,354m is higher than the inlet to the Xhariep WTW. 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 1315 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 1302 masl was assumed at the Xhariep WTP. At a flow of 3.797 m^3/s, the hydraulic gradeline at the outlet of the break pressure tank will be at 1307.6 masl. The hydraulic gradeline will increase to 1309.1 masl if the flow increases to 4.3 m^3/s. Even at a flow of 5.0 m^3/s, the hydraulic gradeline will only be at 1311.6 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 m^3/s, the hydraulic gradeline will increase to 1314.4 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 4.3, 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 m^3/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 1475 masl, under gravity. The peak design flow that needs to be conveyed in this pipeline is 3.616 m^3/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 1504 masl, which is much lower than the proposed full supply level of Command Reservoir No 2, i.e. 1530 masl. In the event of installing a DN2000 pipeline, the hydraulic gradeline at Command Reservoir No 2 will be 1491 masl.

In future, Mangaung Metropolitan Municipality (MMM) may prefer to supply from Command Reservoir No 2 to Brandkop Reservoir, with a full supply level of 1493 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 1529 masl and 1513 masl, respectively.

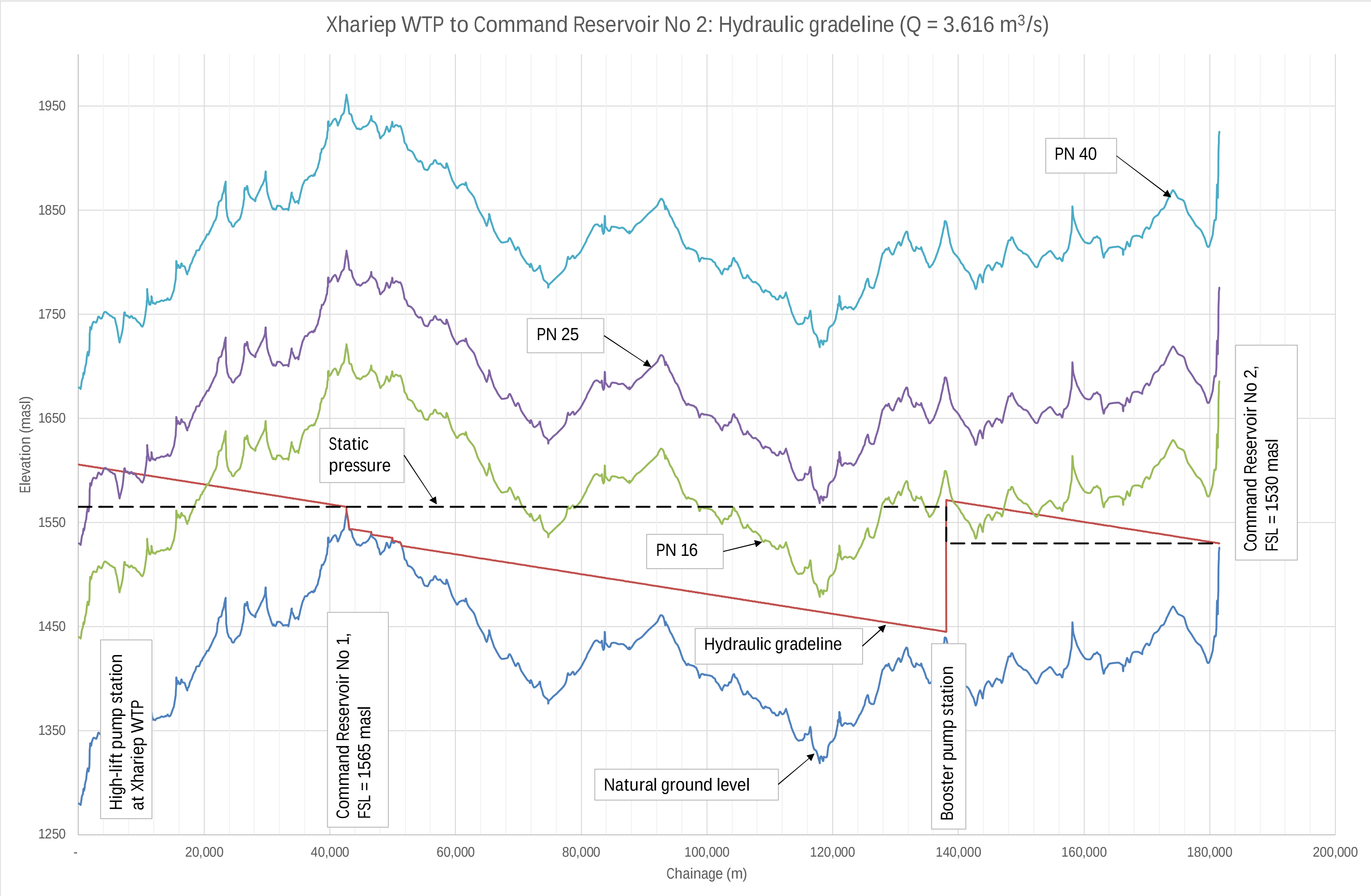


Figure 6-2: High-lift pump station to Command Reservoir No 2 – Hydraulic Gradeline

The minimum operating level of Command Reservoir No 2 is 1520 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 m^3/s conveyed in a DN2000 pipeline. The PN16 (160 m) working pressure line is also shown.

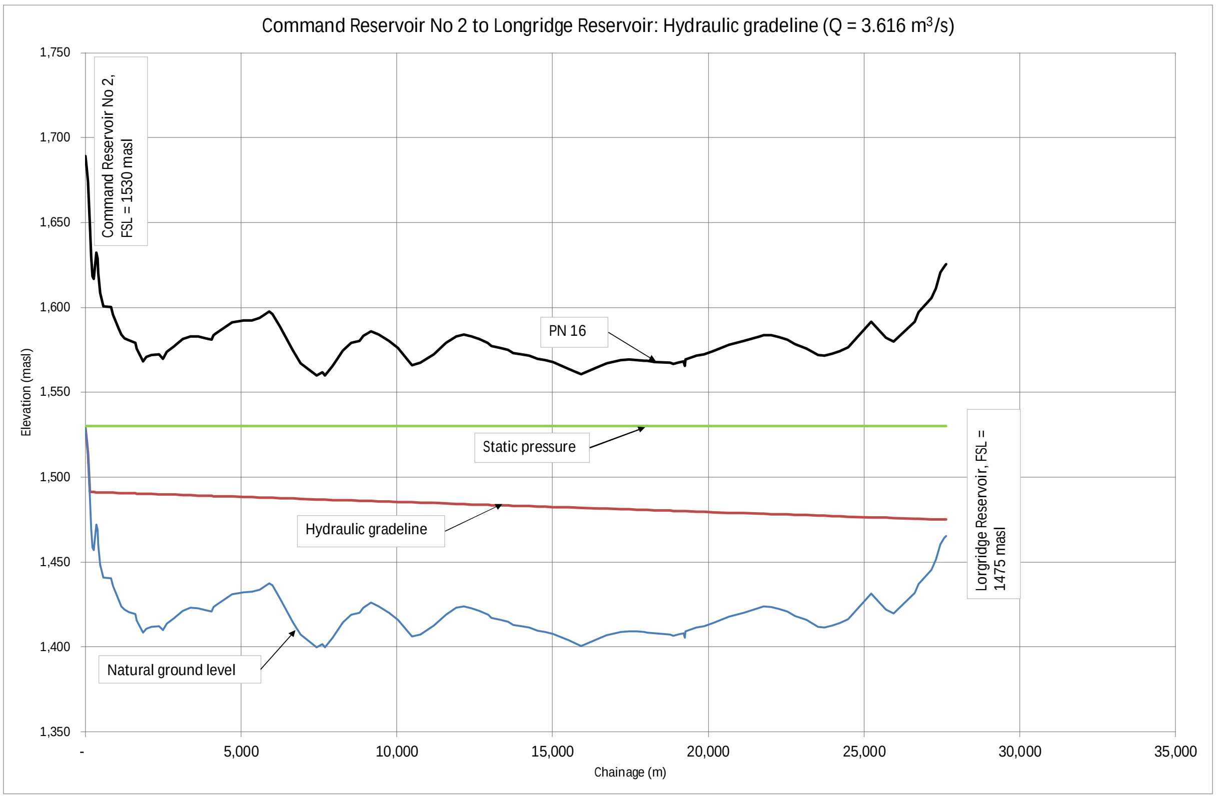


Figure 6-3: Command Reservoir No 2 to Longridge Reservoir – Hydraulic Gradeline

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 1368 masl, under gravity. The peak design flow that needs to be conveyed in this pipeline is 2.428 m^3/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 1490 masl, which is much lower than the proposed full supply level of Command Reservoir No 2, i.e. 1530 masl. In the event of installing a DN1200 pipeline, the hydraulic gradeline at Command Reservoir No 2 will be 1587 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 m^3/s conveyed in a DN1400 pipeline. The PN16 (160 m) working pressure line is also shown.

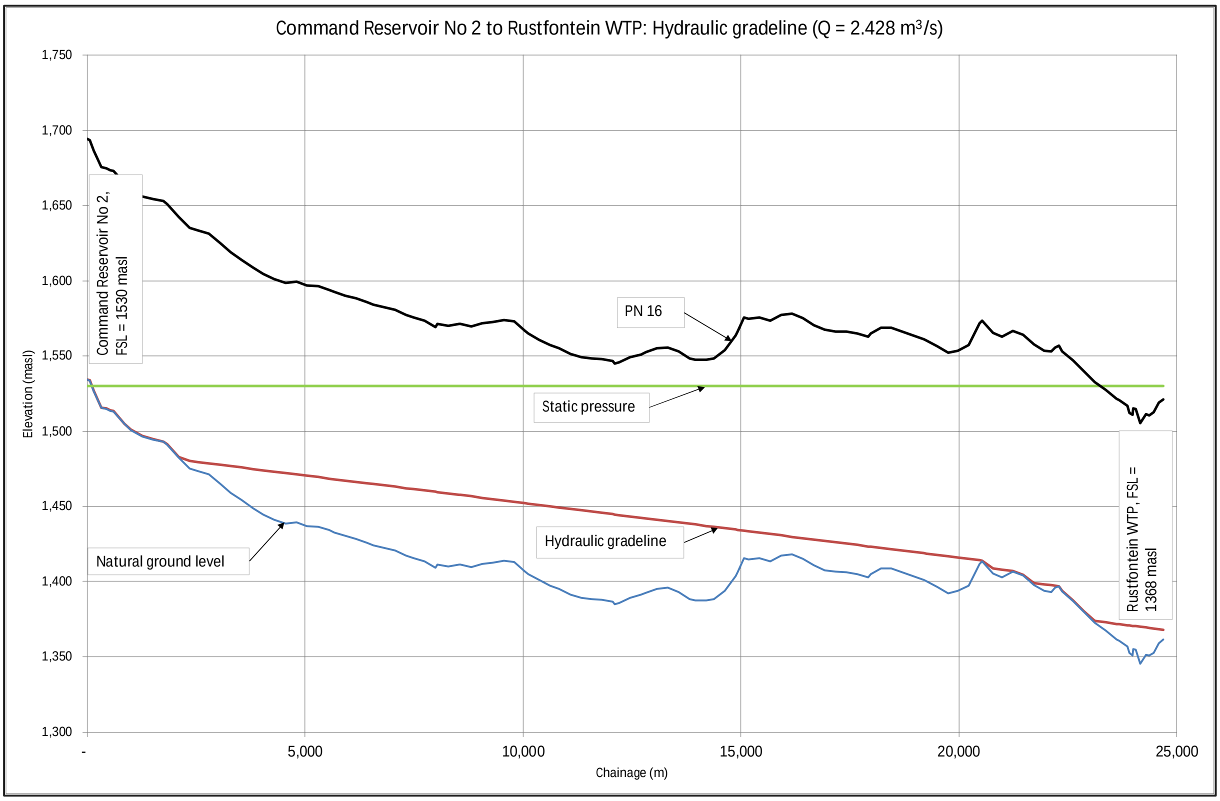


Figure 6-4: Command Reservoir No 2 to Rustfontein WTP – Hydraulic Gradeline

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 Input Data

6.2.1.1 Pipeline Data

The proposed bulk pipeline sizes for the schemes include DN1400, DN1800, and DN2000 pipelines. Additionally, there is an existing DN2100 outlet pipeline from the Gariep Dam that forms part of the scheme. Table 6-2 summarises the properties of the pipelines used for the waterhammer analysis.

Table 6-2: Pipeline Properties for Waterhammer Analysis

Nominal Diameter (mm)	1400	1800	2000	2100
Pipe material	Steel	Steel	Steel	Steel
Outer Diameter (mm)	1420	1820	2020	2120
Wall Thickness (mm) (1)	10	14	14	16

Nominal Diameter (mm)	1400	1800	2000	2100
Internal Diameter (mm) (2)	1400	1792	1992	2088
Wave celerity (m/s) (3)	1000	1050	1000	1050

(1) – Wall thickness is based on a D/t ratio of 150, rounded up to the nearest 2mm.
(2) – The internal diameter used for the surge analysis assumes a coated lining (epoxy or polyurethane)
(3) - The calculated wave celerities were rounded up to the nearest 50 m/s.

6.2.1.2 Pump Station Data

In addition to the data presented in Section 5.3 for the pumps, the inertia and specific speeds of the three pump types in the scheme were also calculated as shown in Table 6-3.

Table 6-3: Pump Properties for Waterhammer Analysis

Pump details	Raw water pump station	High-lift pump station	Booster pump station
Pump inertia (kg.m^2)	28.7	121.3	114.7
Motor Inertia (kg.m^2)	121.4	1130.1	438.1
Combined Inertia (kg.m^2)	150.1	1251.4	552.8
Specific Speed	44.6	14.4	21.5

The analyses also considered **fast-closing nozzle-type non-return valves** (see Section 7.2.6) on the discharge of each pump.

6.2.1.3 Air Valve Data

The air valve sizing conducted in Section 8.5 recommends standardizing all air valves in the scheme to DN200. The waterhammer analyses included DN200 air valves at each high point on the pipelines, i.e. the lower-lying air valves were omitted from the analysis as they are not activated during a pump start, pump trip or valve closing/opening event.

The waterhammer analysis was conducted for 2-stage air valves with inlet and outlet orifice sizes equal to the nominal diameter of the valves. This approach provides a conservative estimate of surge pressures, although three-stage anti-shock air valves will be specified for the project, i.e. the anti-shock orifice assists in dampening surge pressures.

6.2.1.4 Isolation Valve Data

The gravity systems were analyzed with an isolation/control valve closure at the downstream reservoir. The analysis assumed that butterfly isolation valves would be utilized in the scheme with a nominal diameter two sizes below the pipeline's nominal diameter (e.g. a DN1400 isolation valve on a DN1800 pipeline).

The fully open discharge coefficients used for the butterfly valves are summarized in Table 6-4.

Table 6-4 – Isolation Valve Discharge Coefficients

	DN1000	DN1400	DN1600
Discharge Coefficient (K_v) ($m^3/s/bar^{0.5}$) (1)	90 720	196585	247675
Discharge Coefficient (C_v) ($m^3/s/m_{H2O}{}^{0.5}$) (2)	8.05	17.444	21.977

(1) – As published for the Series 756 Butterfly Valve by AVK International
(2) – Converted for to the input units required for Bentley Hammer.

6.2.2 Gariep Dam to Break Pressure Tank

This analysis investigated the waterhammer pressures that can be expected during the start-up operation and during a pump trip event for the raw water pump station. The analysis includes the intake pipework from the Gariep Dam to the pump station, as well as the rising main from the pump station to the break pressure tank.

6.2.2.1 Pump Start Scenario

The pumps at the raw water pump station will be equipped with variable speed drives (VSDs). Figure 6-5 illustrates the expected waterhammer pressures when the VSDs are utilized to start the pumps in a controlled manner over a 30-second period. This controlled start restricts the maximum surge pressure to 110 meters.

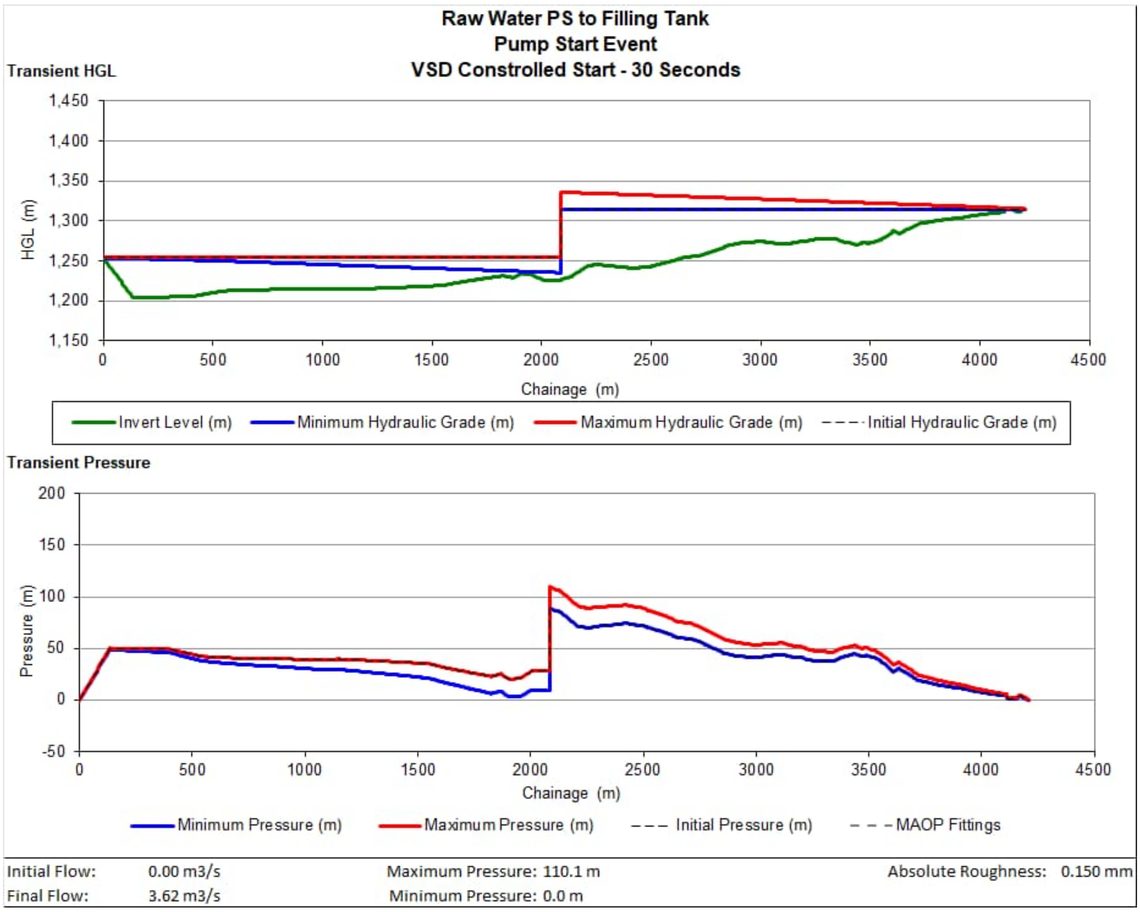


Figure 6-5: Raw water pump station - Controlled pump start (VSD)

6.2.2.2 Pump Trip Scenario

Figure 6-6 shows the anticipated waterhammer pressures for a pump trip event at the raw water pump system. Maximum waterhammer pressures could reach up to 165 m. Vacuum pressures up to -10 m are also anticipated for the last ±500 m of the pipeline up to the break pressure tank.

To mitigate these waterhammer pressures, it is recommended to install a non-return valve at chainage 4.1 km, or approximately 100 m upstream of the break pressure tank, as shown in Figure 6-7. With the non-return valve, the maximum waterhammer pressures are expected to reduce to 110 m as shown in

Figure 6-7. Vacuum pressures up to -10 m are still anticipated for the last ± 500 m which must be considered for the pipeline's structural design.

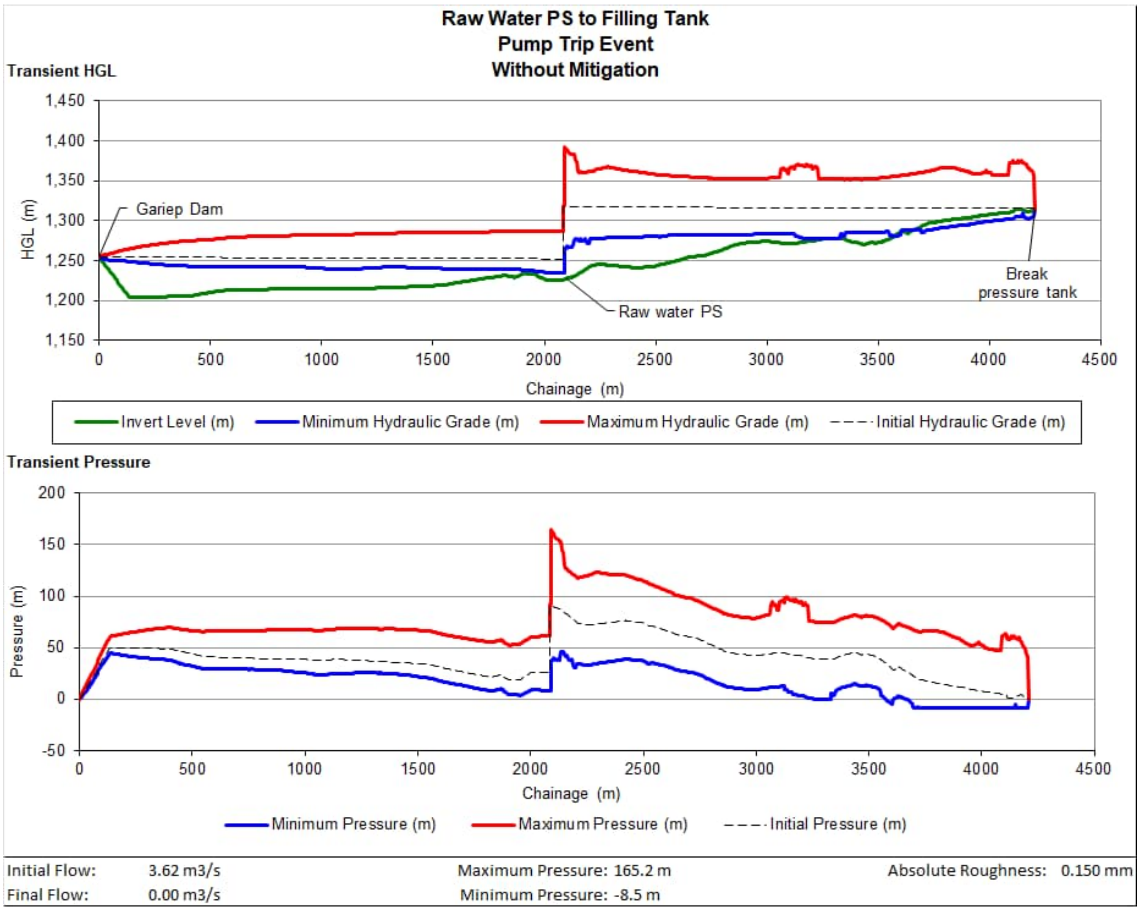


Figure 6-6: Raw water pump station – Pump trip without mitigation

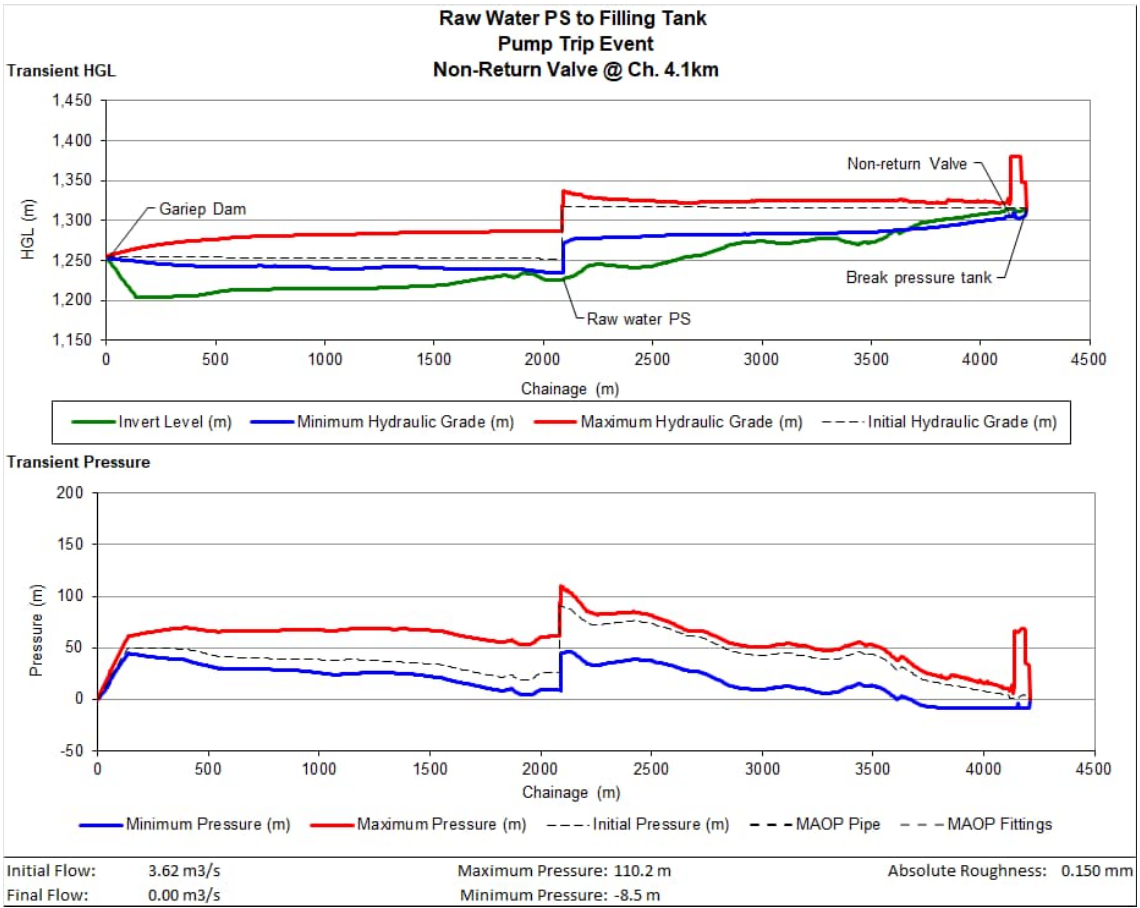


Figure 6-7: Raw water pump station – Pump trip with mitigation

6.2.3 High-Lift Pump Station to Command Reservoir No 1

This analysis investigated the waterhammer pressures that can be expected during the start-up operation and during a pump trip event for the high-lift pump station.

6.2.3.1 Pump Start Scenario

Figure 6-8 shows the anticipated waterhammer pressures for a rapid start (i.e. over a 2-second period), as is the case with a direct-on-line (DOL) installation, for the high-lift pumps. This rapid start causes a positive pressure surge up to 382 m.

Installing soft starters can reduce the positive pressure surge during startup. Given the high static pressure of the high-lift pump station, the pumps only begin to deliver flow at 85 to 90% of their operating speed. A recommended approach is to use a soft starter with dual ramp capabilities to quickly ramp the pumps up to 85% of the operating speed, then slowly increase to the full operating speed over 5 minutes. This controlled start limits surge pressure to 331m. The soft starters will also reduce startup current and thereby lowering electrical infrastructure needs. Figure 6-9 illustrates the anticipated waterhammer pressures for this scenario. Alternatively, if soft starters are not preferred, the pumps can also be started with a (suitable) closed isolation valve on the discharge side, e.g. a plunger valve, which is then slowly opened to reduce the startup waterhammer pressures.

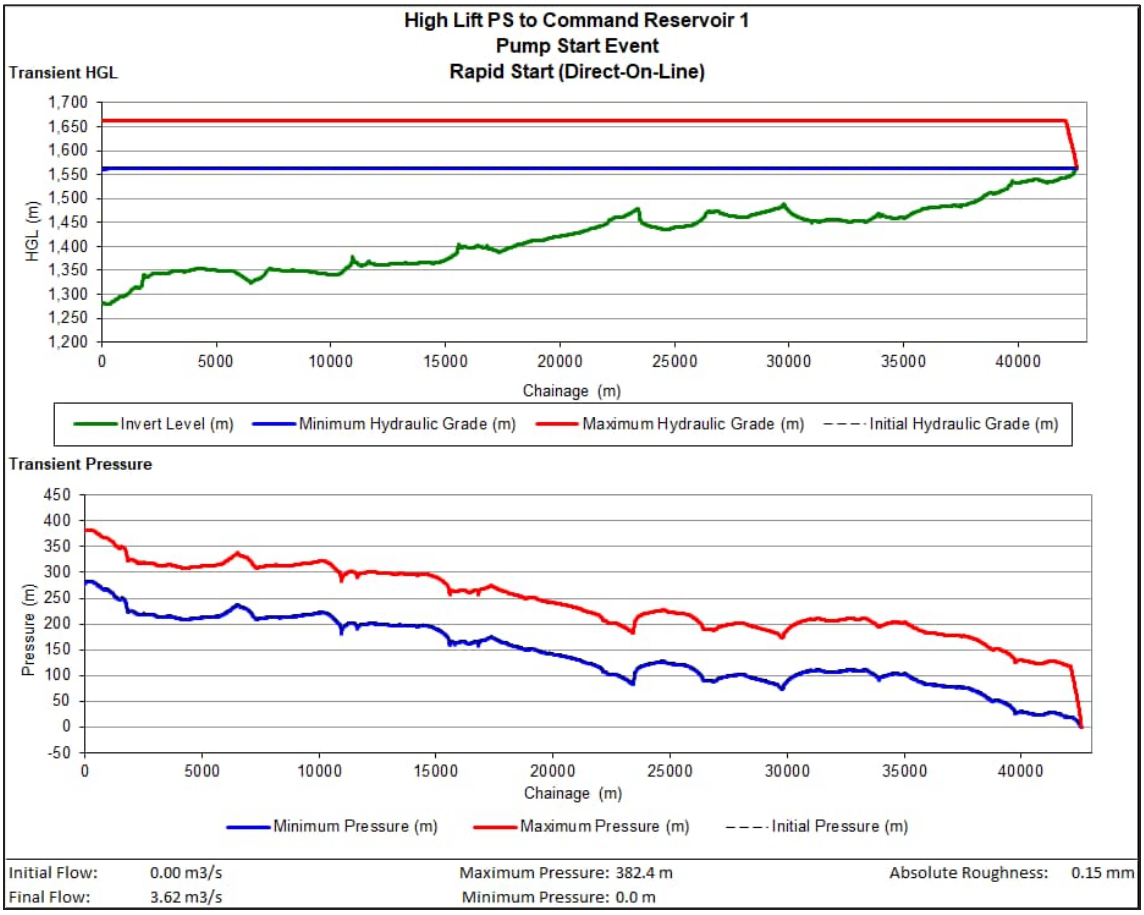


Figure 6-8: High-lift pump station - Rapid pump start (DoL)

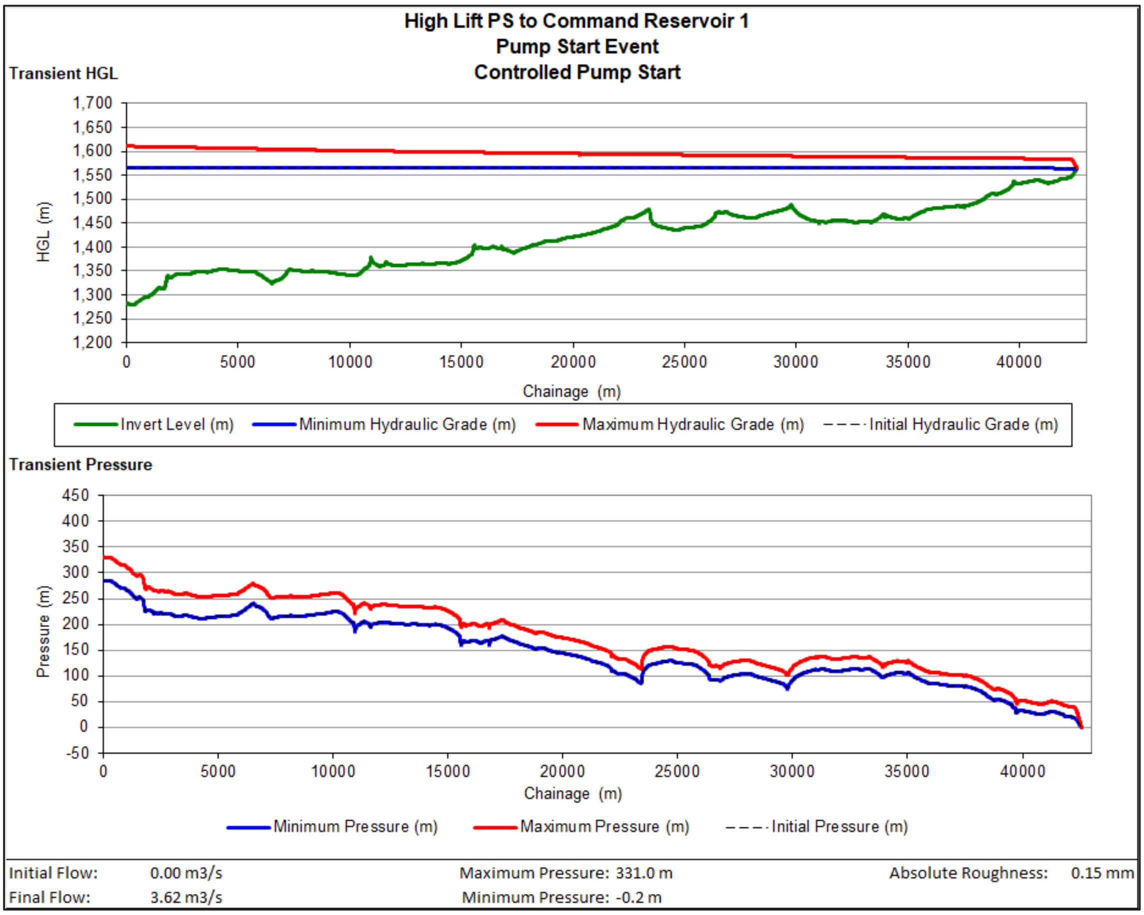


Figure 6-9: High-lift pump station - Controlled pump start (soft starters or against closed valve)

6.2.3.2 Pump Trip Scenario

Figure 6-10 shows the anticipated waterhammer pressures for a pump trip event at the high-lift pump station. Waterhammer pressures could reach 532 m. Vacuum pressures are also anticipated for the pipeline from chainage 20 km to Command Reservoir No 1.

To mitigate these waterhammer pressures, 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 as shown in Figure 6-11. Vacuum pressures of up to – 10 m are still anticipated from chainage 20 km, which must be considered for the pipeline's structural design.

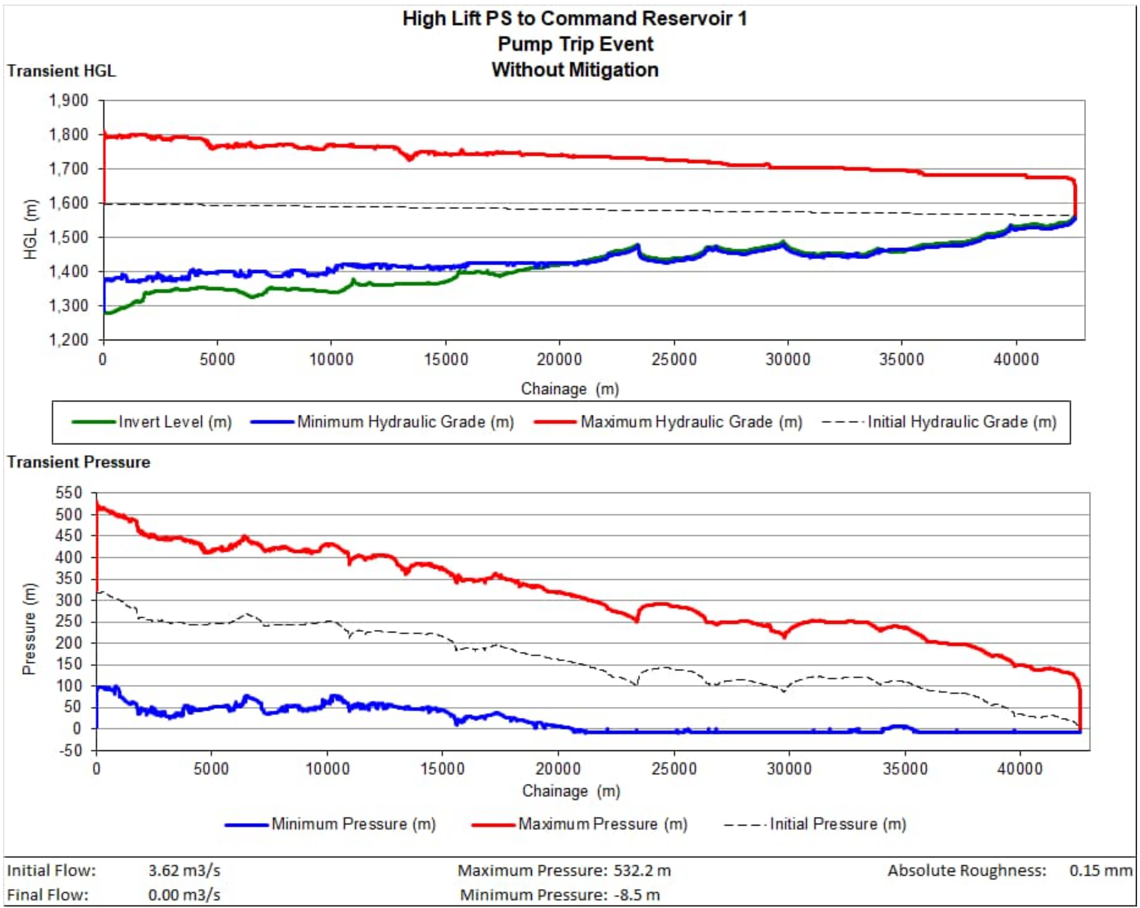


Figure 6-10: High-lift pump station – Pump trip without mitigation

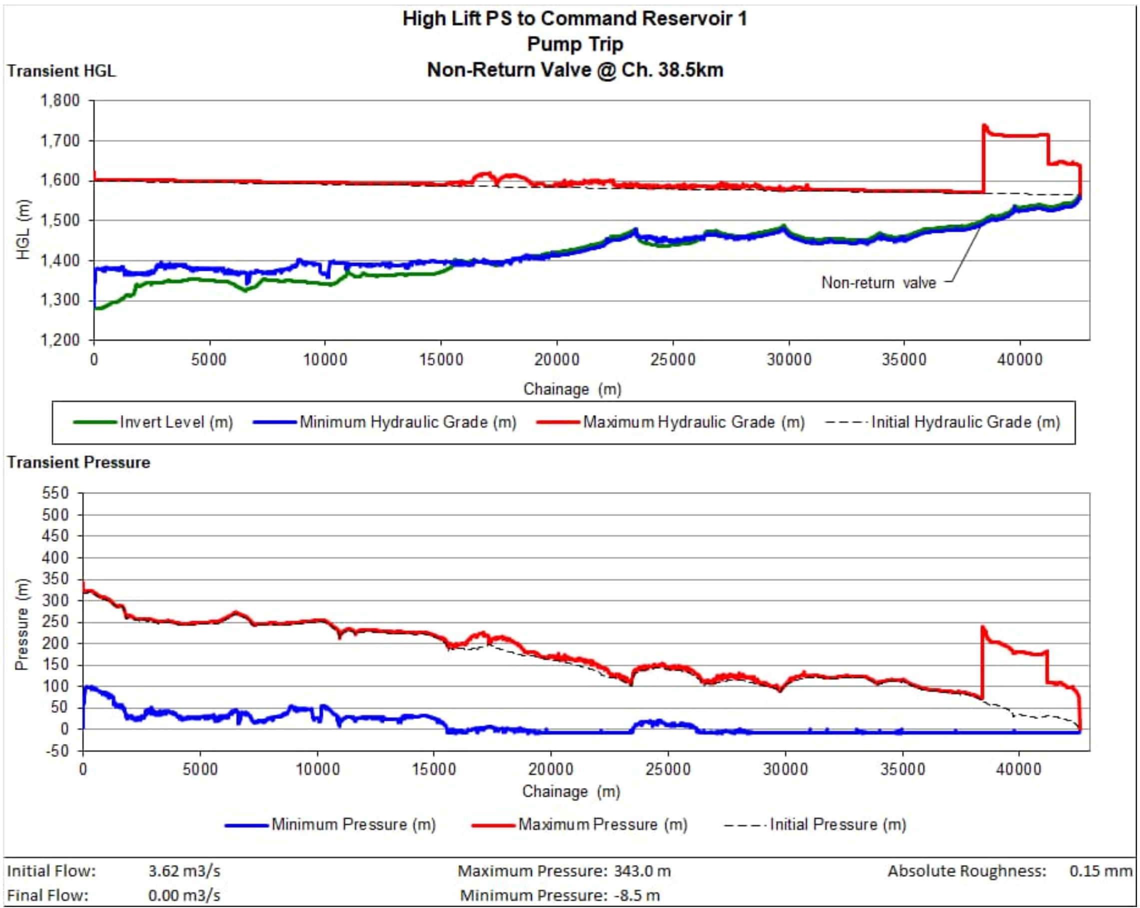


Figure 6-11: High-lift pump station – Pump trip with mitigation

6.2.4 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.

The analysis was conducted with the valve partially closed to limit the flow to the pipeline design flow. To mitigate waterhammer pressures, it is advised to close the last 15% of the valve over a duration of no less than 10 minutes. Figure 6-12 shows the anticipated water hammer pressures for closing the valve over this period. The maximum anticipated waterhammer pressures are 276 m.

If the valve is closed from fully open to fully closed at a constant speed, a total closure time of 67 minutes will be required. However, the initial ±60% closure has minimal impact on surge pressure and may be executed faster. However, the final 40% of closure should be performed over a period of approximately 30 minutes or longer.

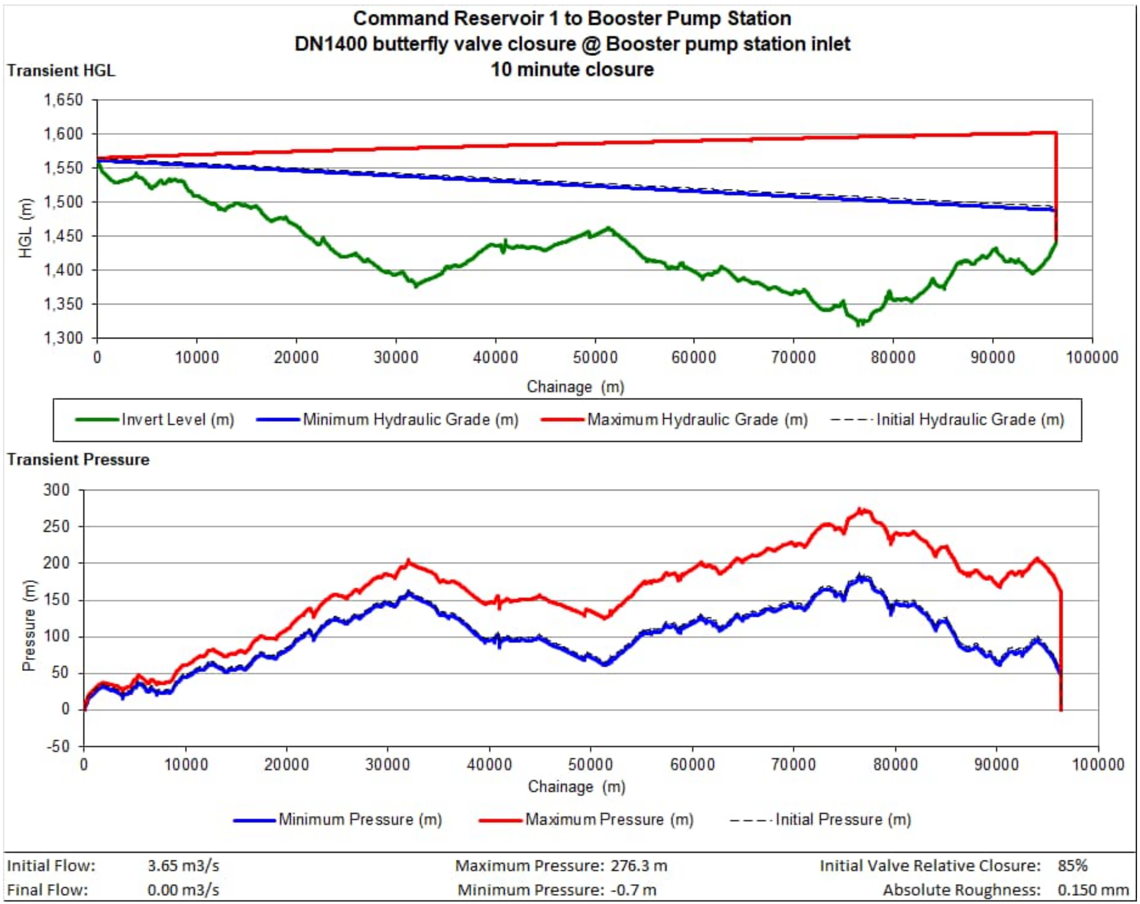


Figure 6-12: Command Reservoir No 1 to Booster Pump Station – Valve Closure

6.2.5 Booster Pump Station to Command Reservoir No 2

This analysis investigated the waterhammer pressures that can be expected during the start-up operation and during a pump trip event for the booster pump station.

6.2.5.1 Pump Start Scenario

Figure 6-13 shows the anticipated waterhammer pressures for a rapid start (i.e. over a 2-second period), as is the case with a direct-on-line (DOL) installation, for the booster pumps. This rapid start causes a positive pressure surge up to 219 m.

Installing soft starters can reduce the positive pressure surge during startup. Given the high static pressure of the booster pump station, the pumps only begin to deliver flow at 75 to 80% of their operating speed. A recommended approach is to use a soft starter with dual ramp capabilities to quickly ramp the pumps up to 75% of the operating speed, then slowly increase to the full operating speed over 3 minutes. This controlled start limits start-up surge pressure to 195m. The soft starters will also reduce startup current and thereby lowering electrical infrastructure needs. Figure 6-14 illustrates the anticipated waterhammer pressures for this scenario. Alternatively, if soft starters are not preferred, the pumps can also be started against a (suitable) closed isolation valve on the discharge side, e.g. a plunger valve, which is then slowly opened to reduce the startup waterhammer pressures.

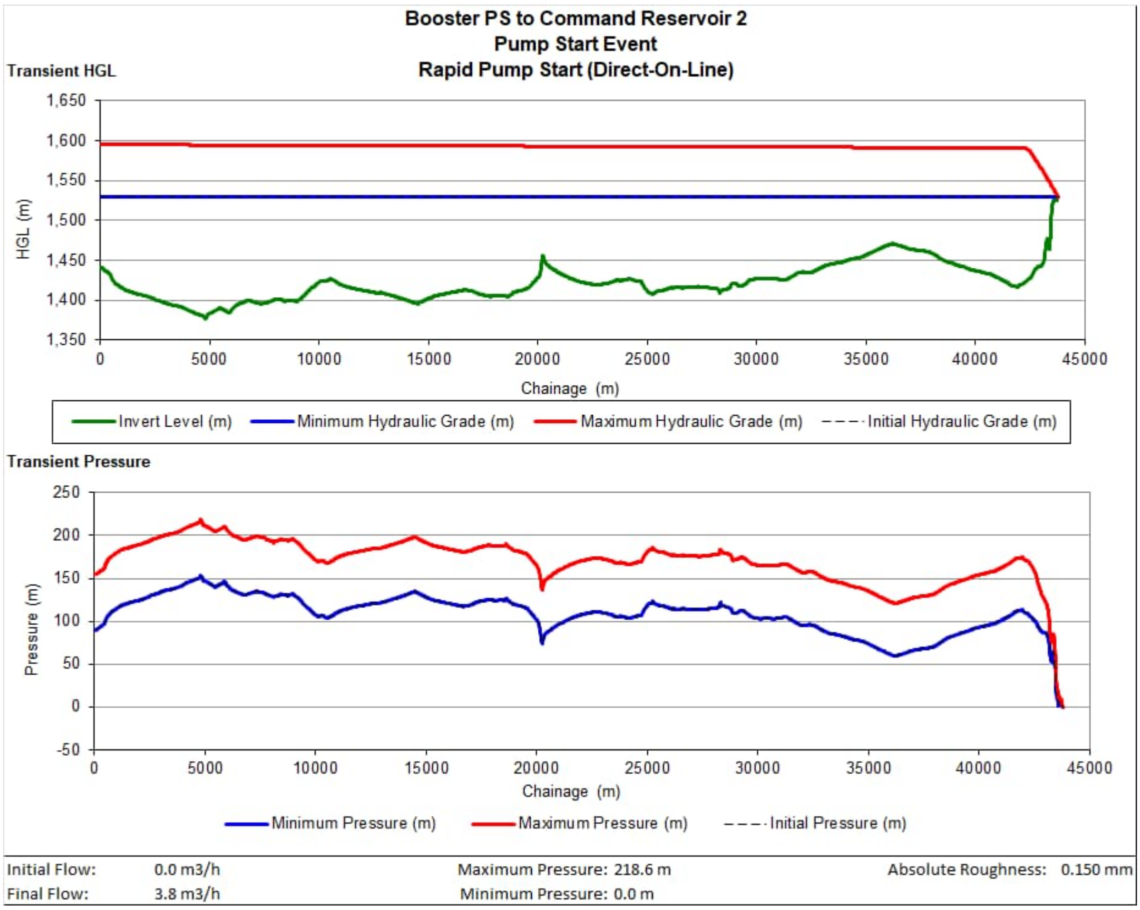


Figure 6-13: Booster pump station - Rapid pump start (DoL)

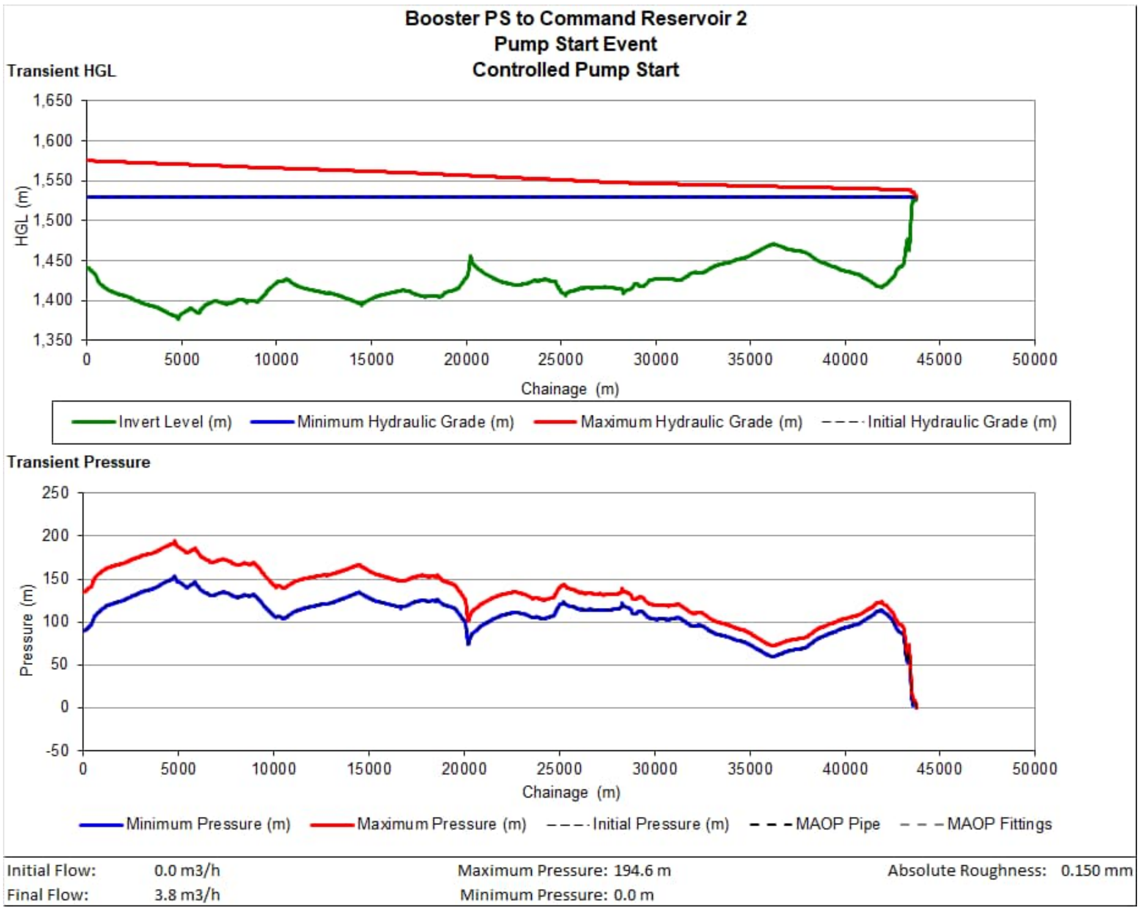


Figure 6-14: Booster pump station - Controlled pump start (soft starters or against a closed valve)

6.2.5.2 Pump Trip Scenario

Figure 6-15 shows the anticipated waterhammer pressures for a pump trip event at the booster pump station. Waterhammer pressures could reach up to 338 m. Vacuum pressures of up to – 10 m are also anticipated along large sections of the pipeline.

To mitigate these waterhammer pressures, 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 as shown in Figure 6-16. 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.

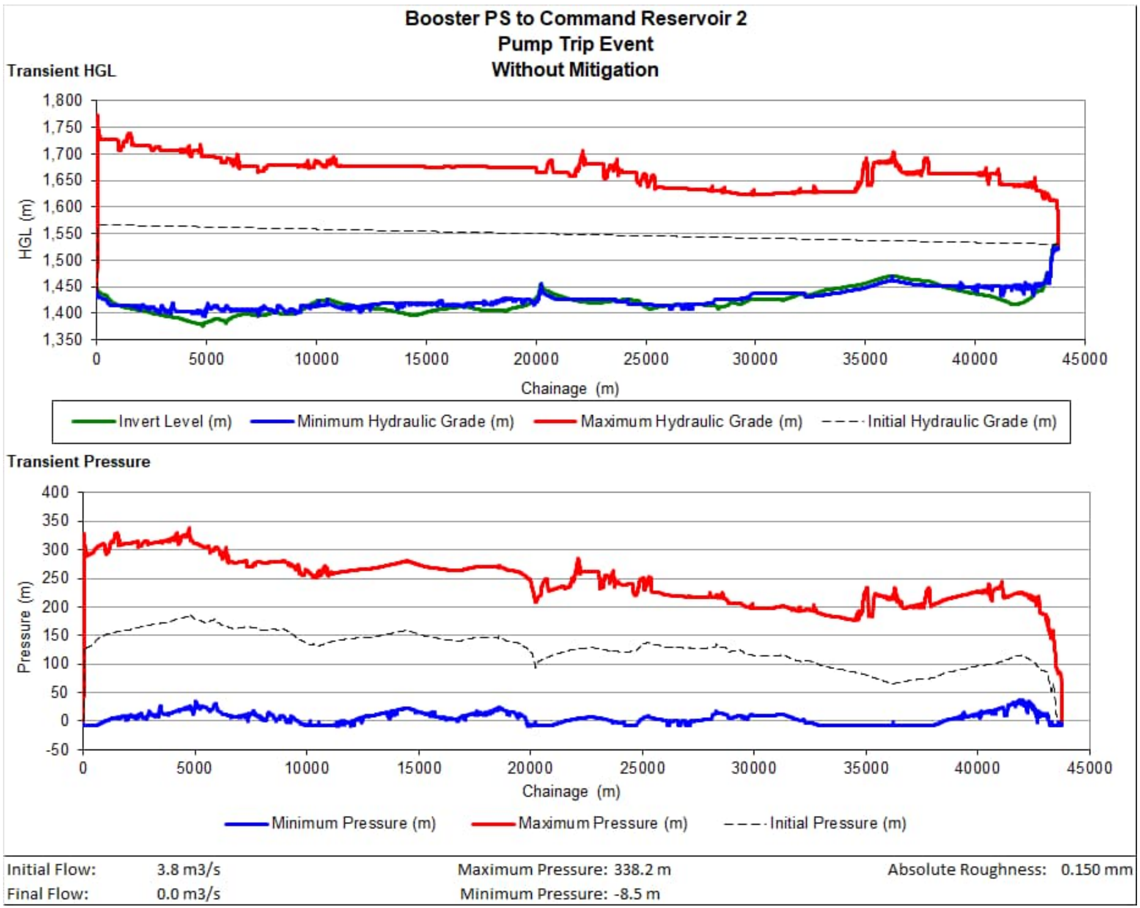


Figure 6-15: Booster pump station – Pump trip without mitigation

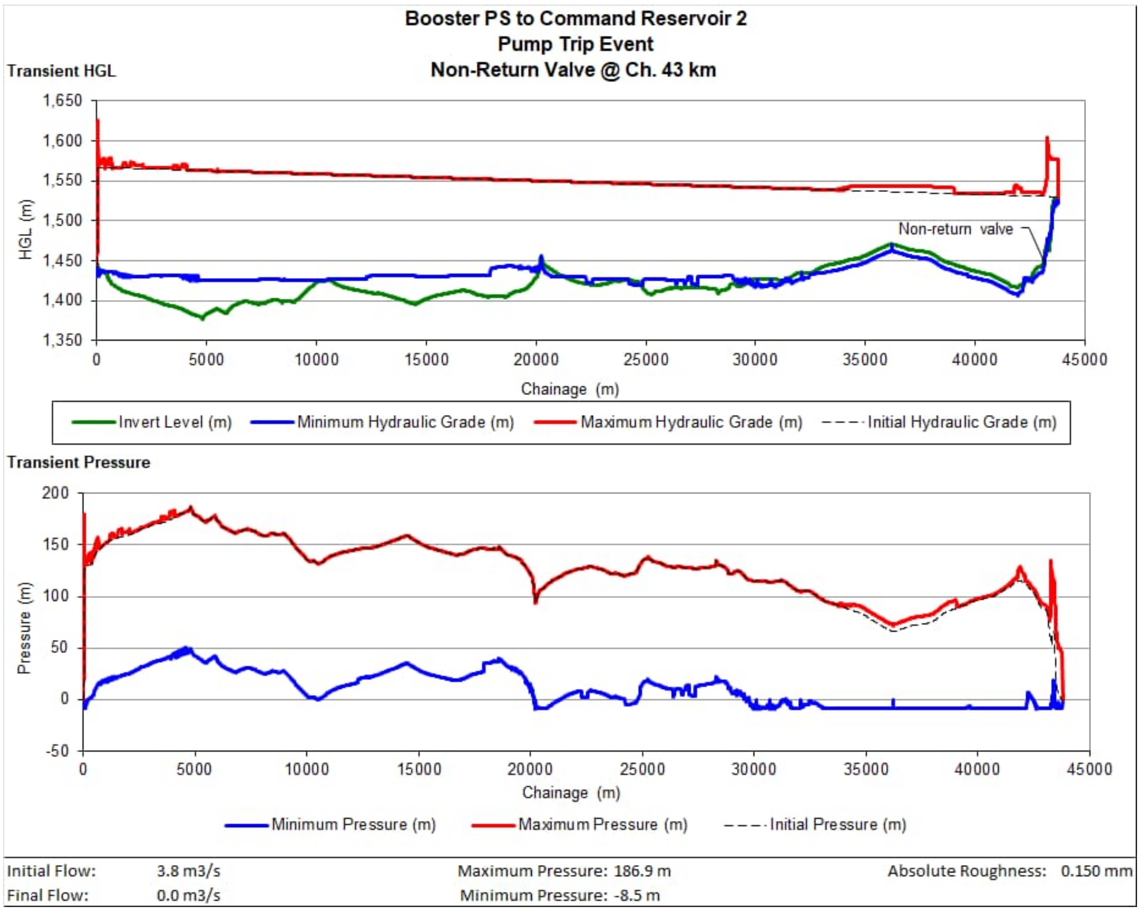


Figure 6-16: Booster pump station – Pump trip with mitigation

6.2.6 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.

To limit the flow to the pipeline's design flow, the inlet valve was set to have an initial closure of 86%. It is recommended that the remaining 14% is closed over a period of 5 minutes or longer. This equates to a total valve closure time of 36 minutes from fully open to fully closed when closed at a constant rate. This operation would limit the maximum waterhammer pressures to 140 m as shown in Figure 6-17.

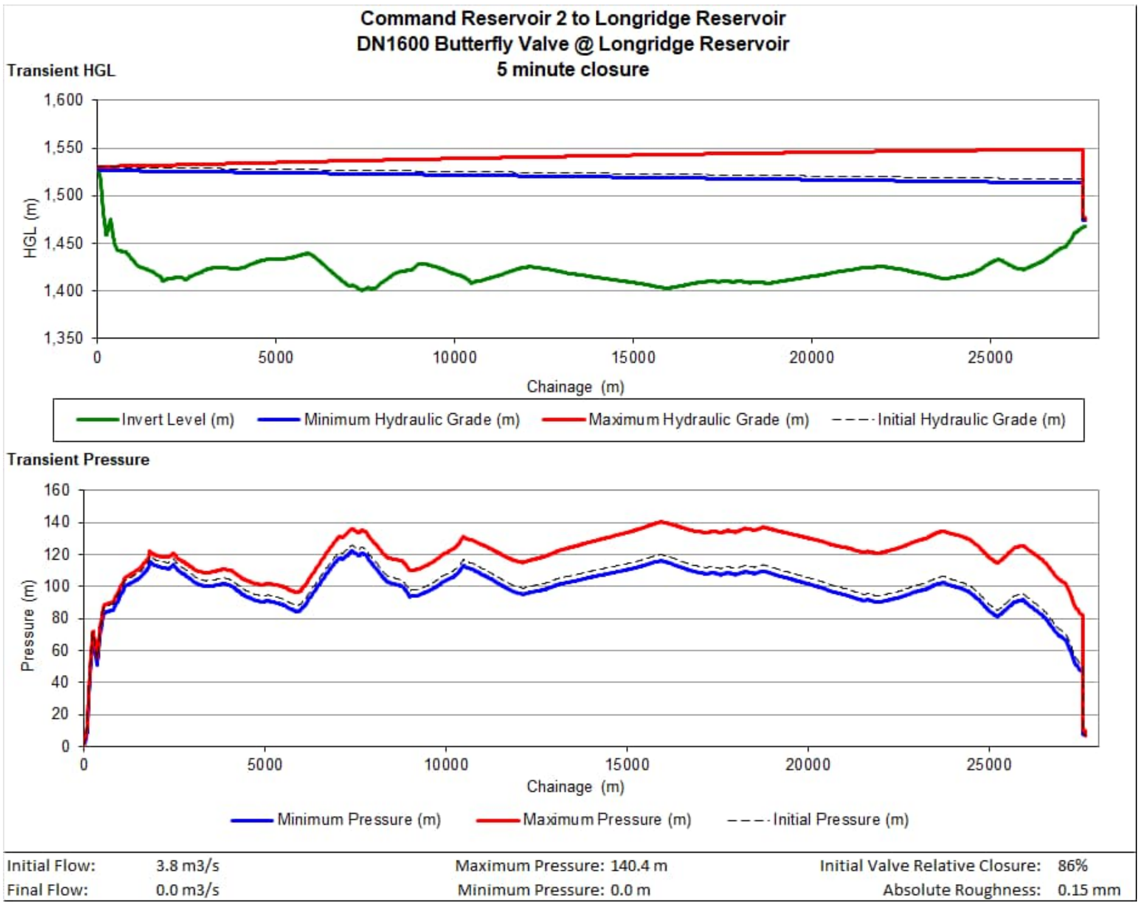


Figure 6-17: Command Reservoir No 2 to Longridge Reservoir – Valve Closure

6.2.7 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.

To limit the flow to the pipeline's design flow, the inlet valve was set to have an initial closure of 86%. It is recommended that the remaining 14% is closed over a period of 10 minutes or longer. This equates to a total valve closure time of 72 minutes from fully open to fully closed when closed at a constant rate. This operation would limit the maximum waterhammer pressures to 190 m as shown in Figure 6-18.

The initial ±50% closure has minimal impact on surge pressure and may be executed faster. However, the final 50% of closure should be performed over a period of approximately 35 minutes or longer.

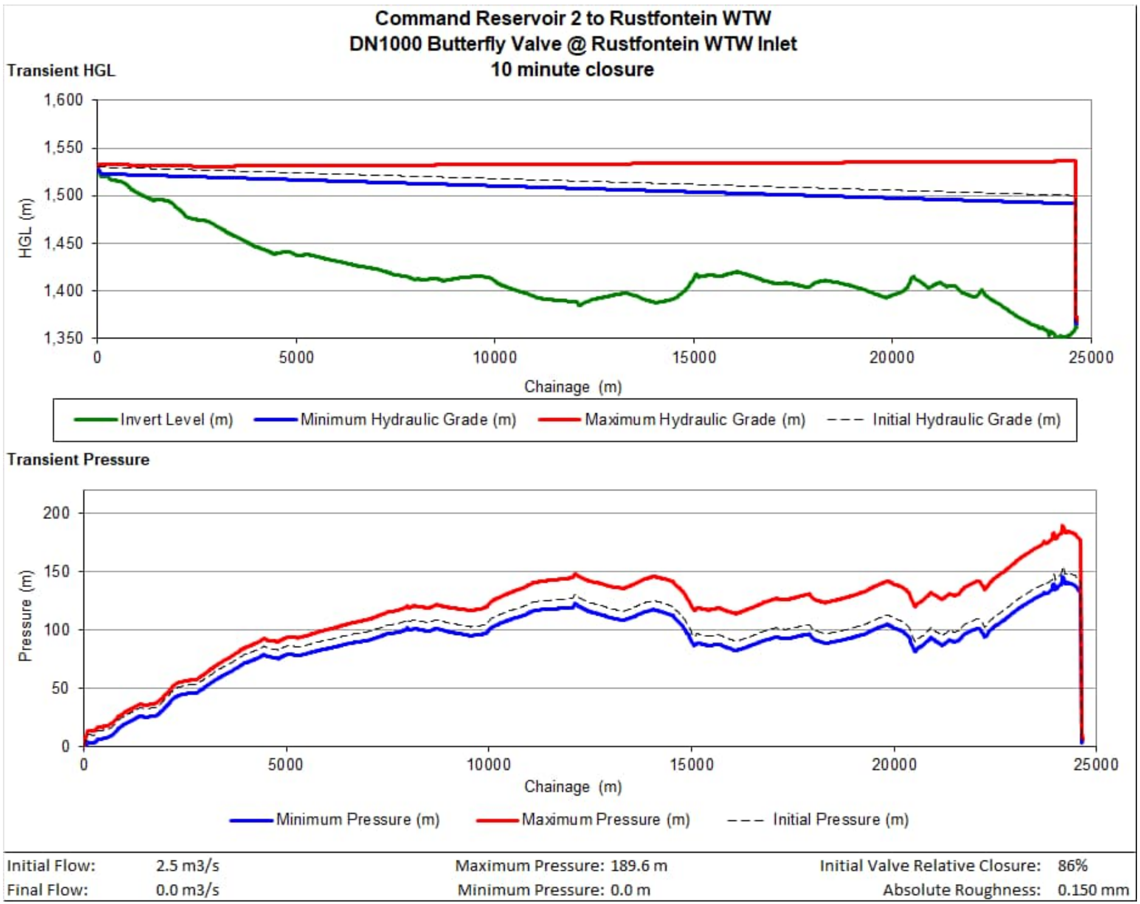


Figure 6-18: Command Reservoir No 2 to Rustfontein WTP – Valve Closure

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 6-19 shows the maximum working and surge pressures for the pipeline section from Gariep Dam to the proposed Xhariep WTP. It is evident from Figure 6-19 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 6-20 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 6-20 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.

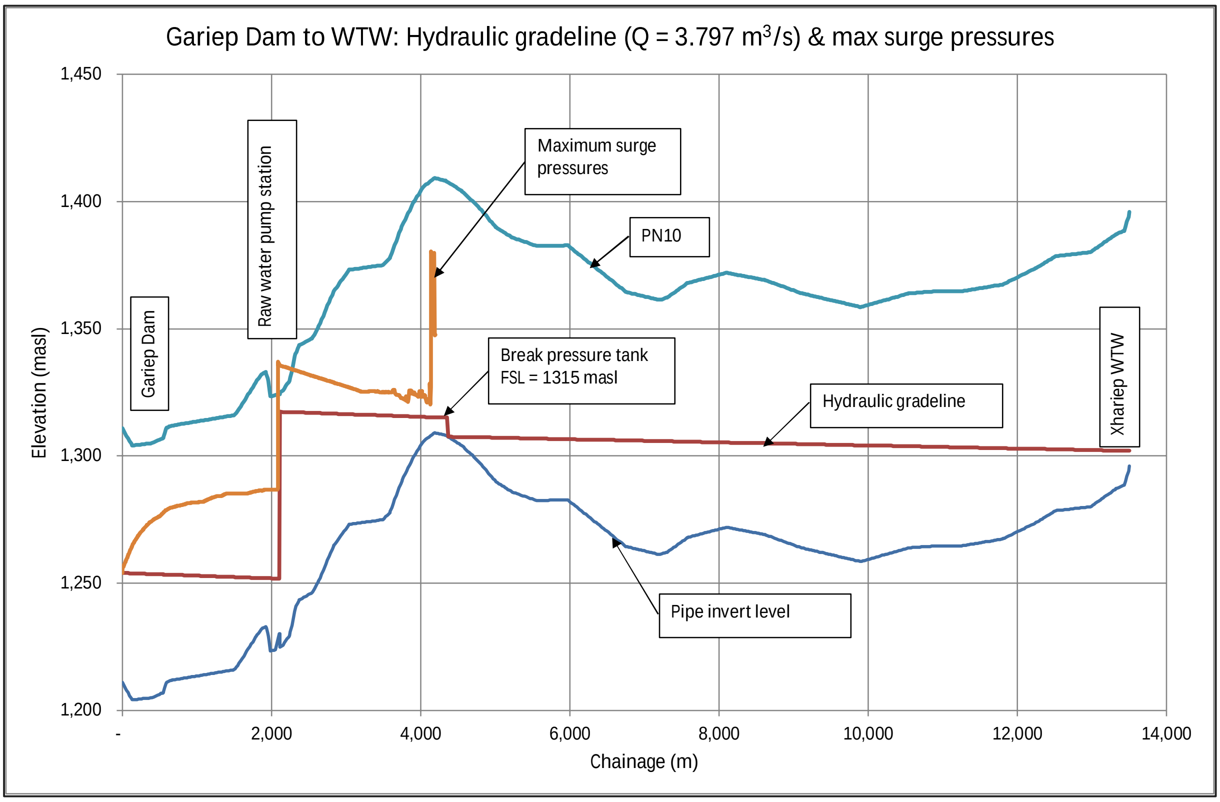


Figure 6-19: Gariep Dam to Xhariep WTW – Maximum working and surge pressures

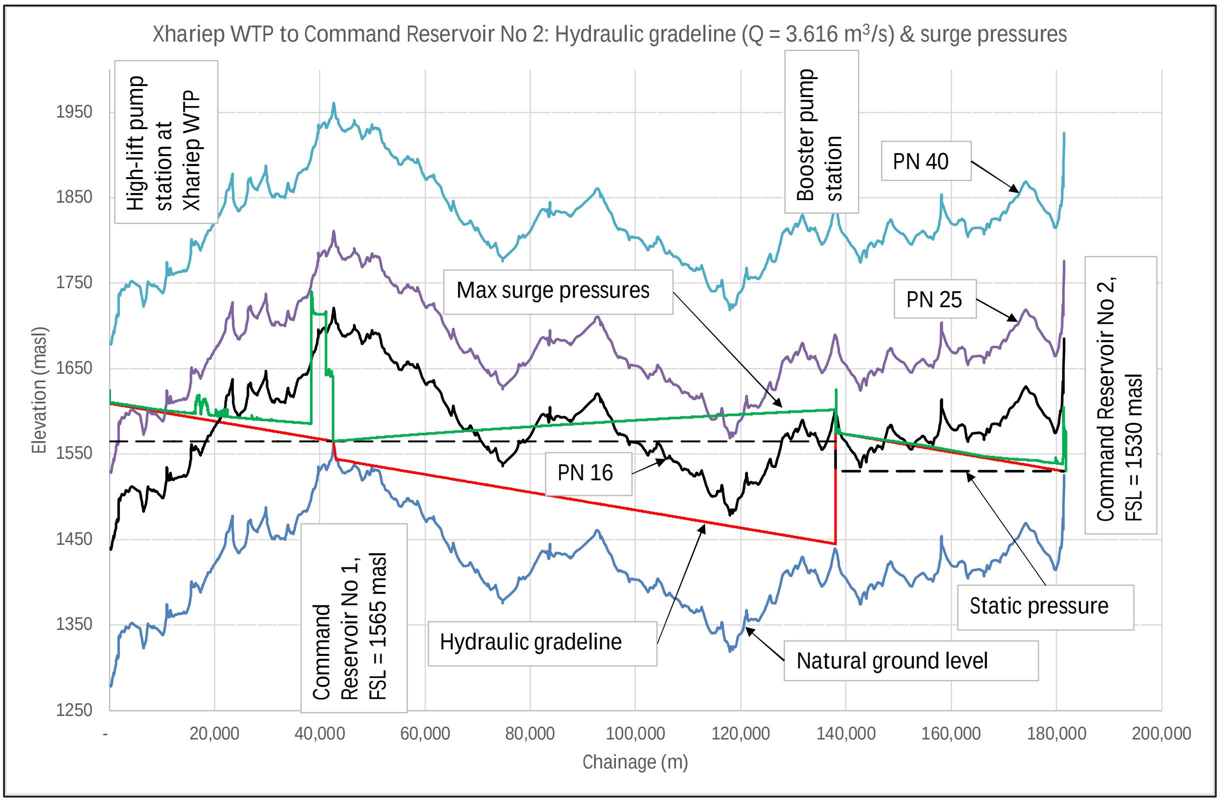


Figure 6-20: High-lift pump station to Command Reservoir No 2 – Maximum working and surge pressures

Figure 6-21 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 6-21 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.

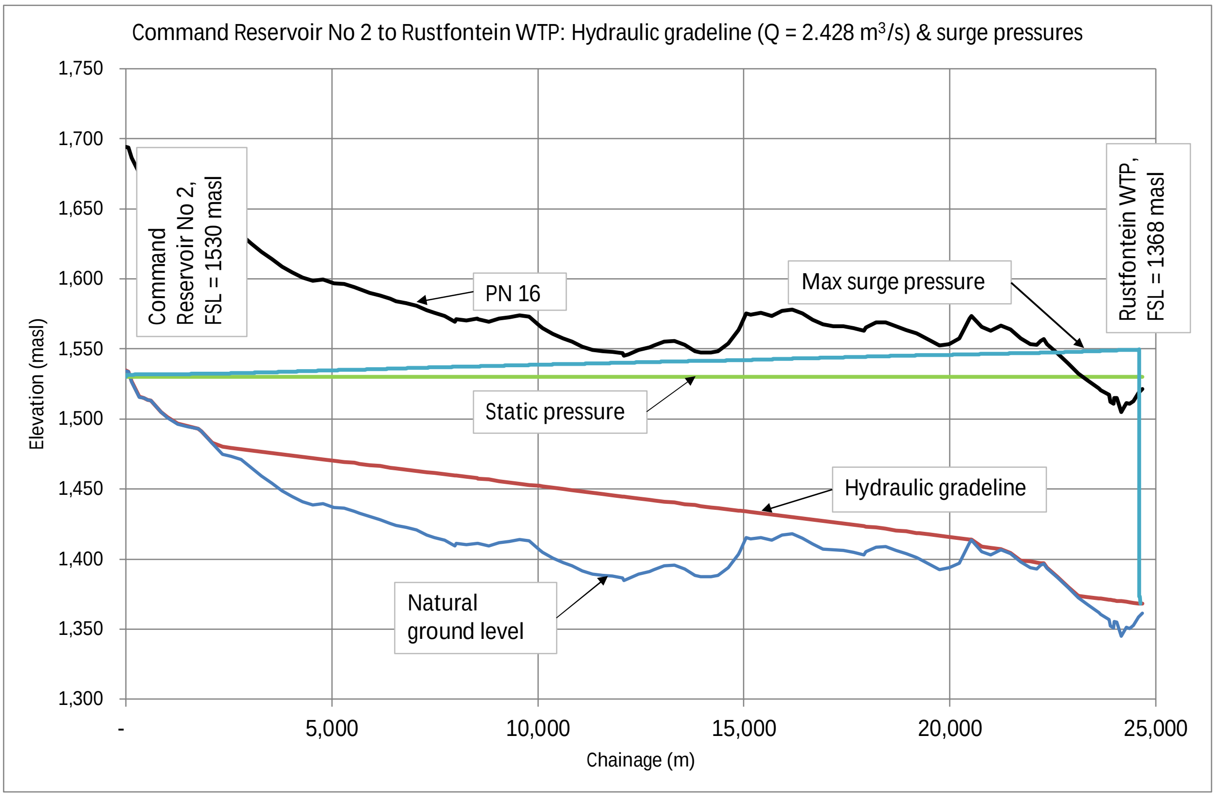


Figure 6-21: Command Reservoir No 2 to Rustfontein WTP – Maximum working and surge pressures

Figure 6-22 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 6-22 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.

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 6-5.

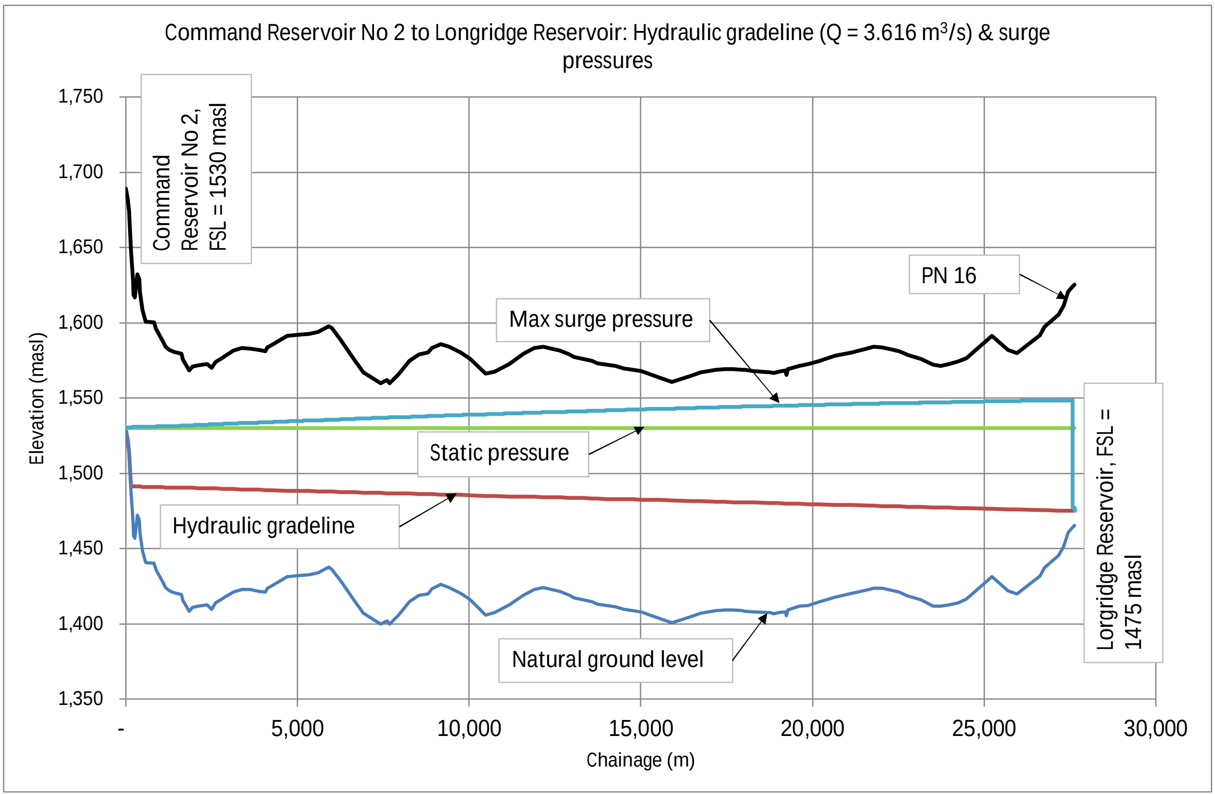


Figure 6-22: Command Reservoir No 2 to Longridge Reservoir – Maximum working and surge pressures

Table 6-5: 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.

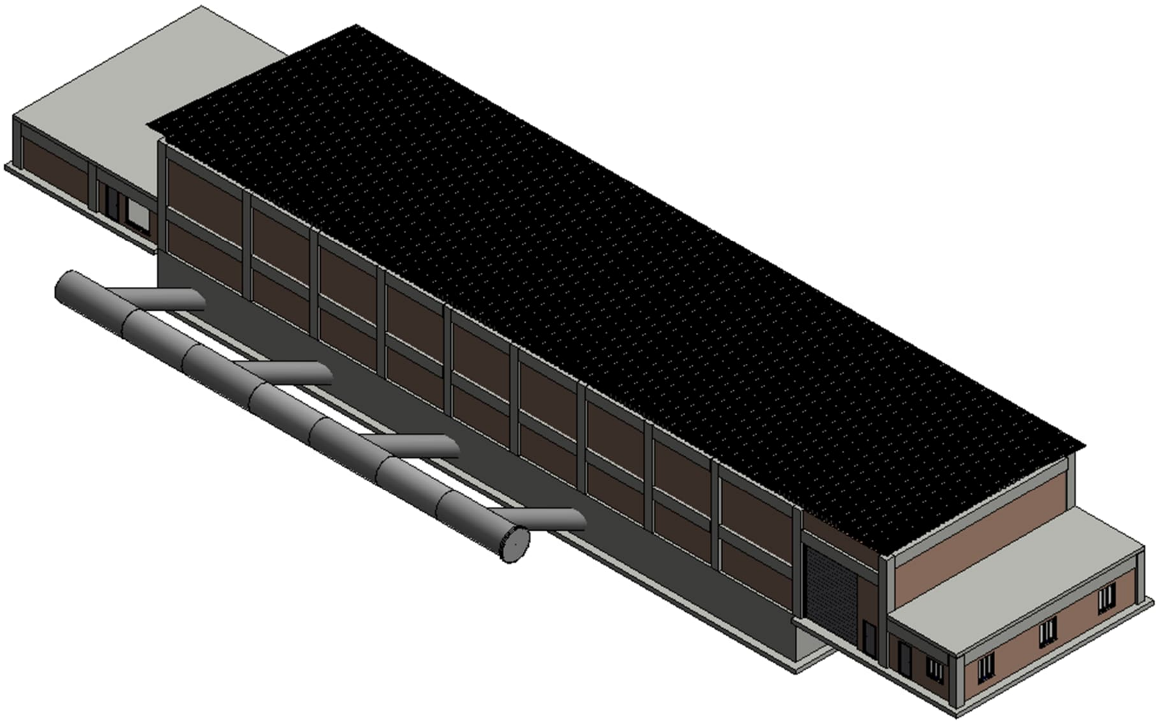


Figure 7-1: Raw water pump station – 3D view

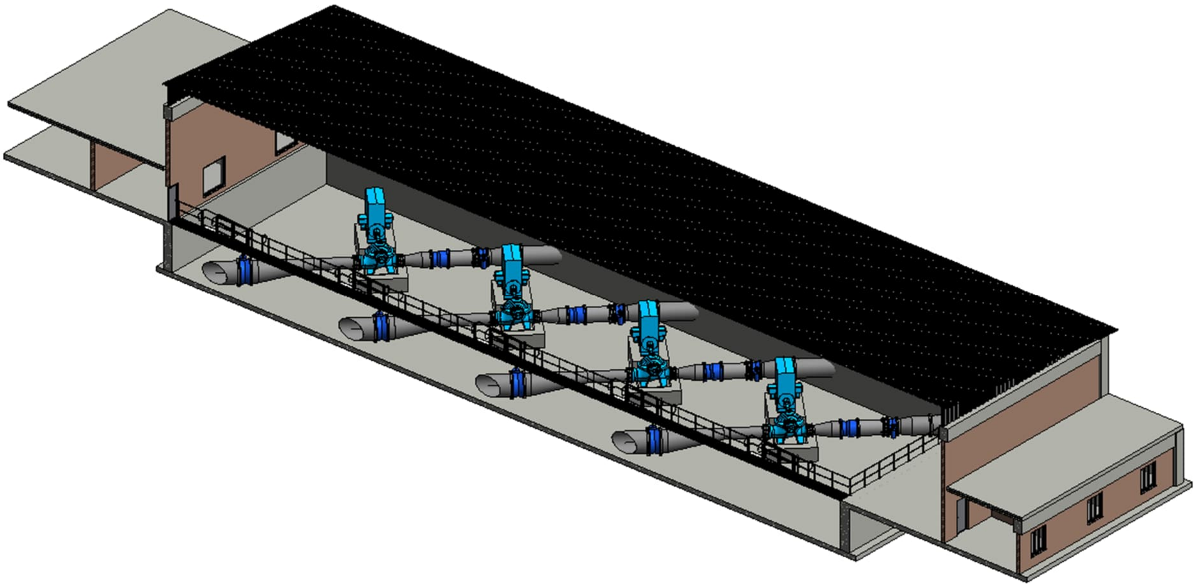


Figure 7-2: Raw water pump station – Sectional view

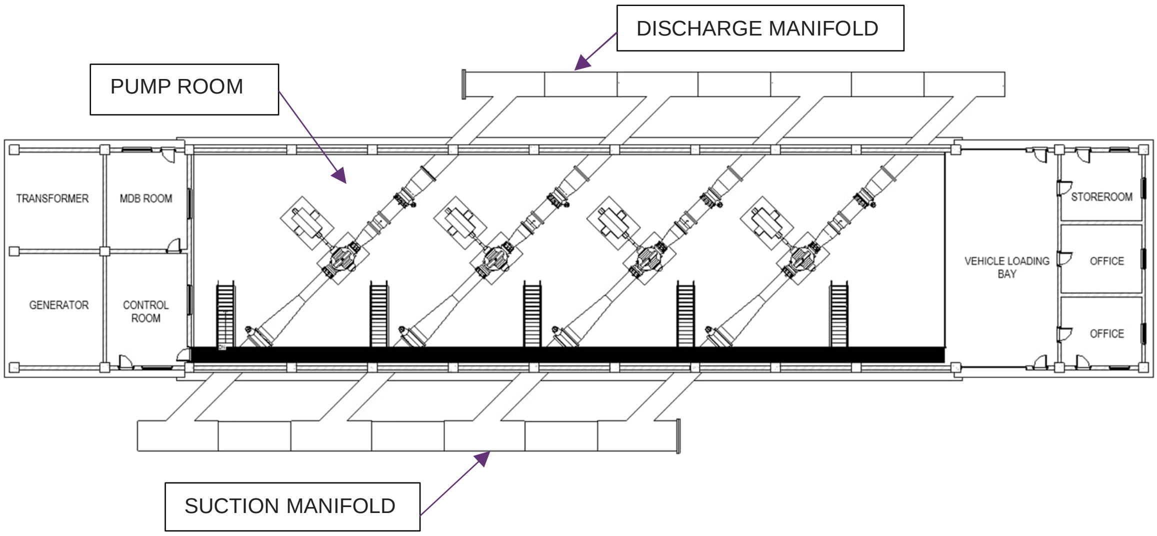


Figure 7-3: Raw water pump station – Plan view

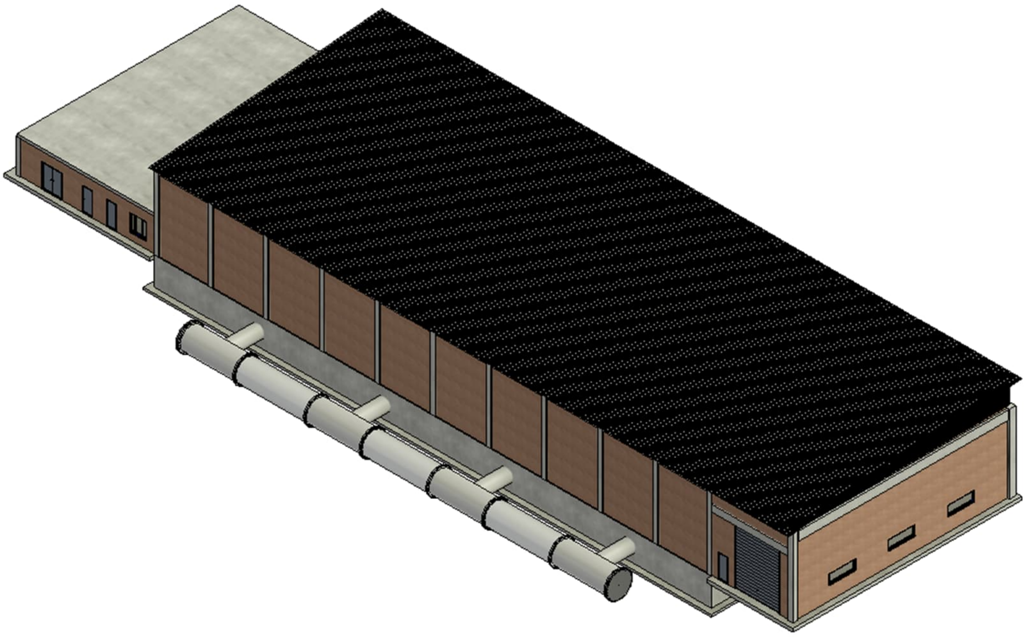


Figure 7-4: High-lift pump station – 3D view

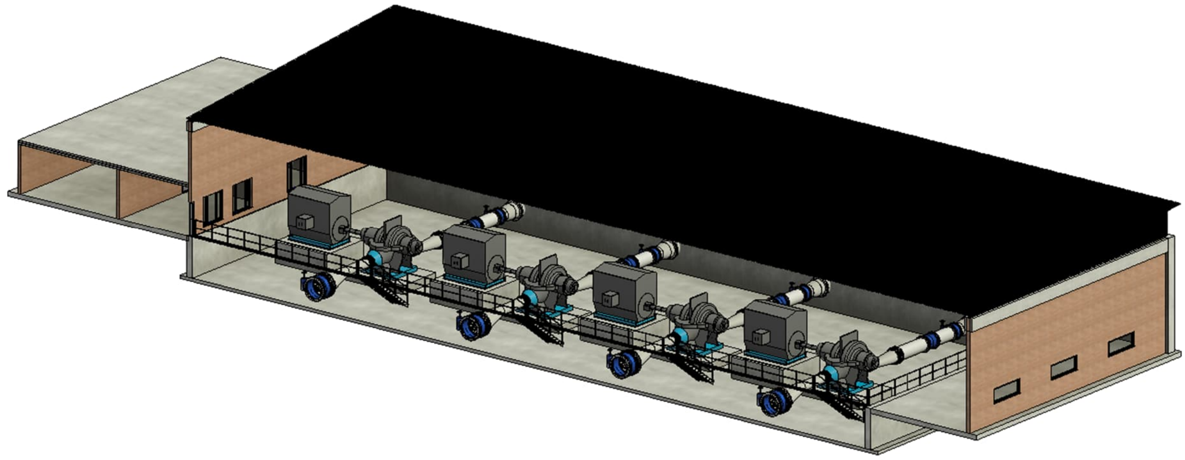


Figure 7-5: High-lift pump station – Sectional view

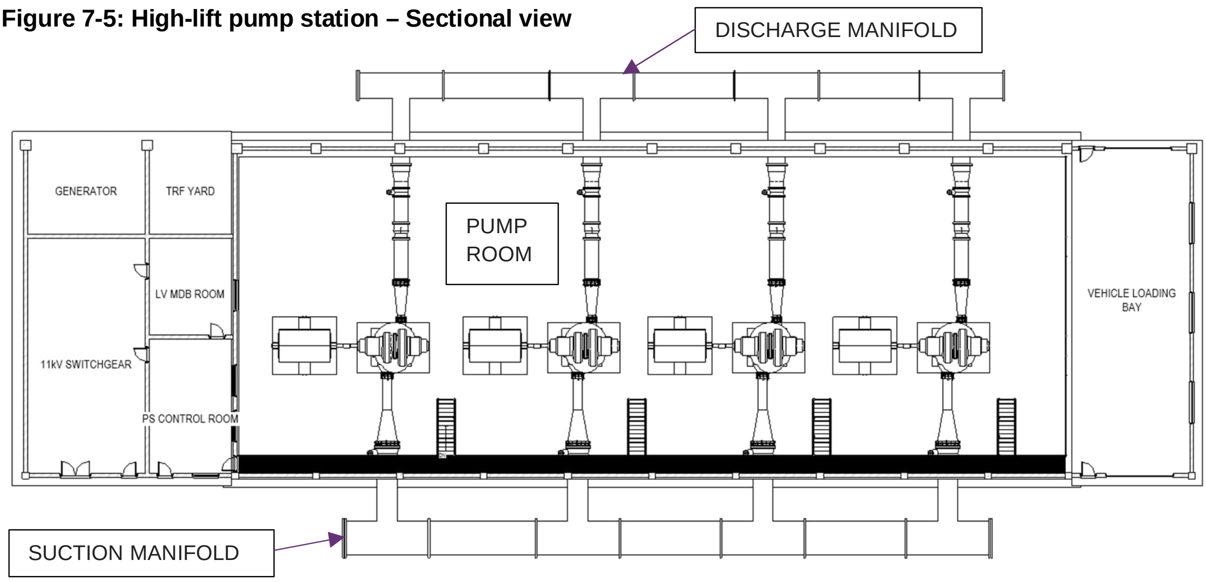


Figure 7-6: High-lift pump station – Plan view

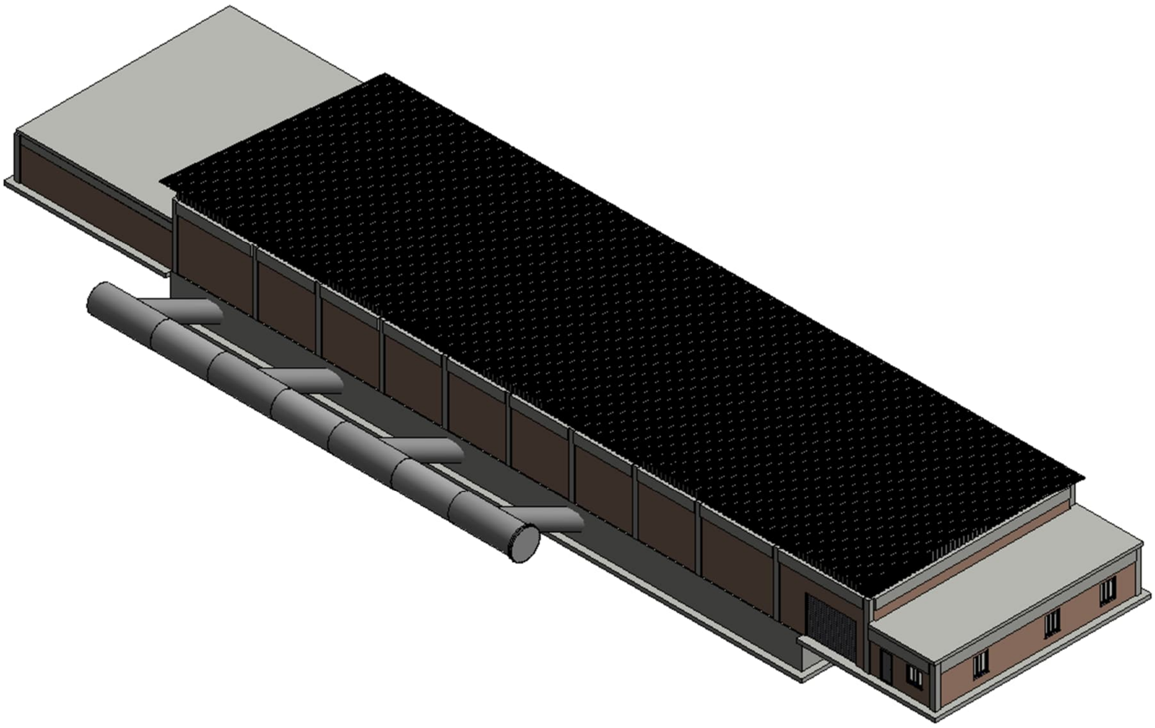


Figure 7-7: Booster pump station – 3D view

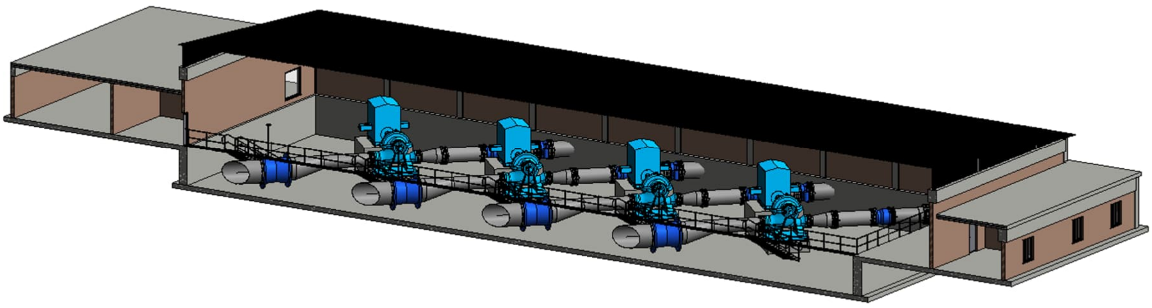


Figure 7-8: Booster pump station – Sectional view

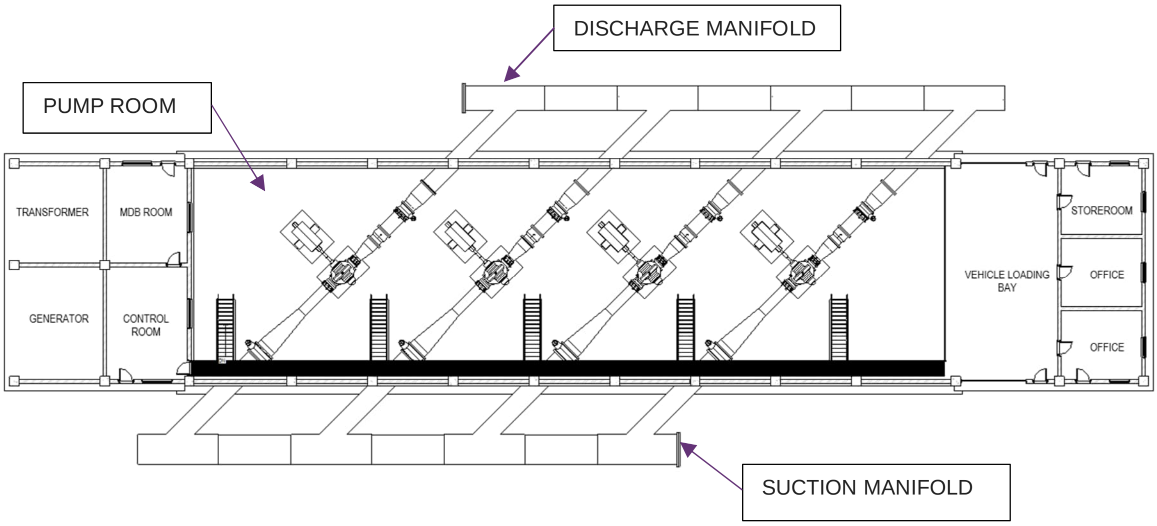


Figure 7-9: Booster pump station – Plan view

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 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.

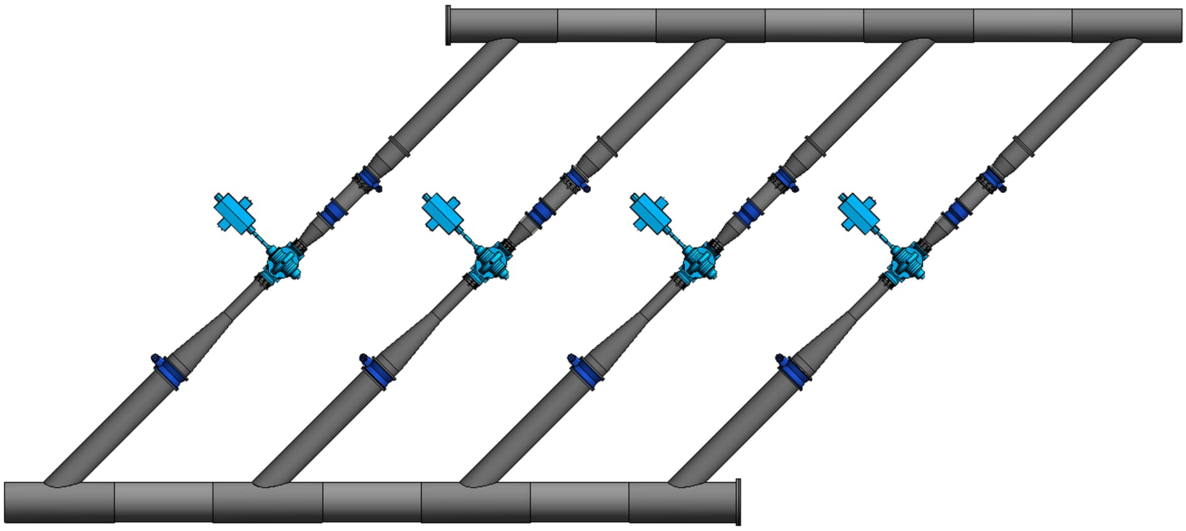


Figure 7-10: Raw water pump station – overall pipework layout

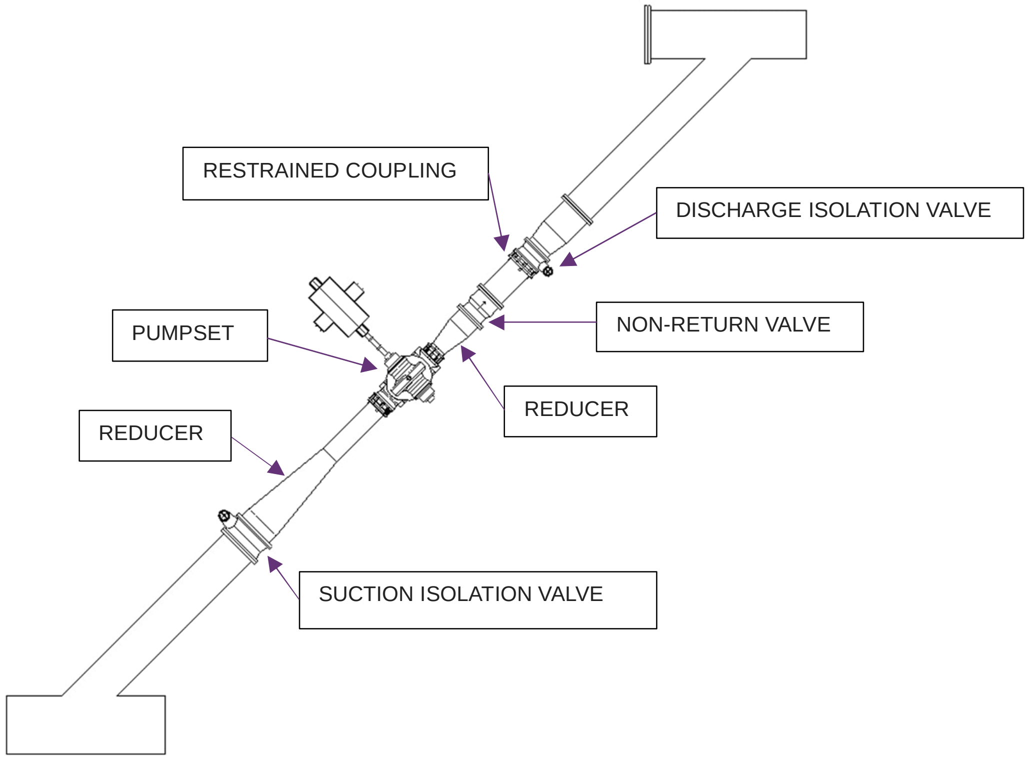


Figure 7-11: Raw water pump station – single pump leg plan view

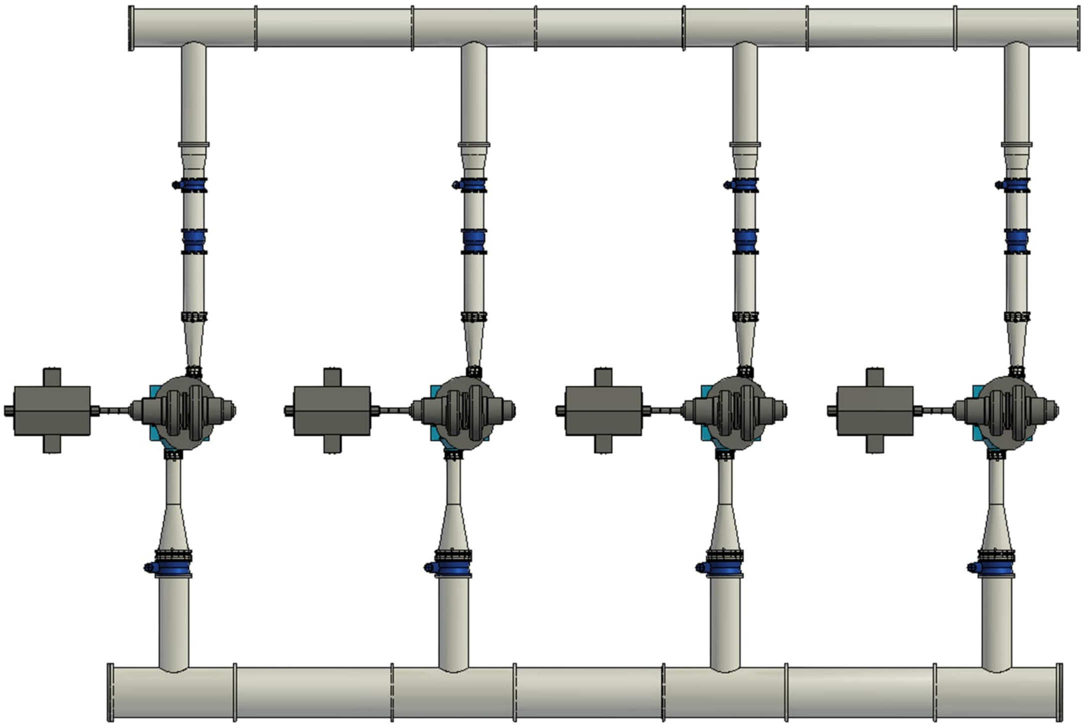


Figure 7-12: High-lift pump station – overall pipework layout

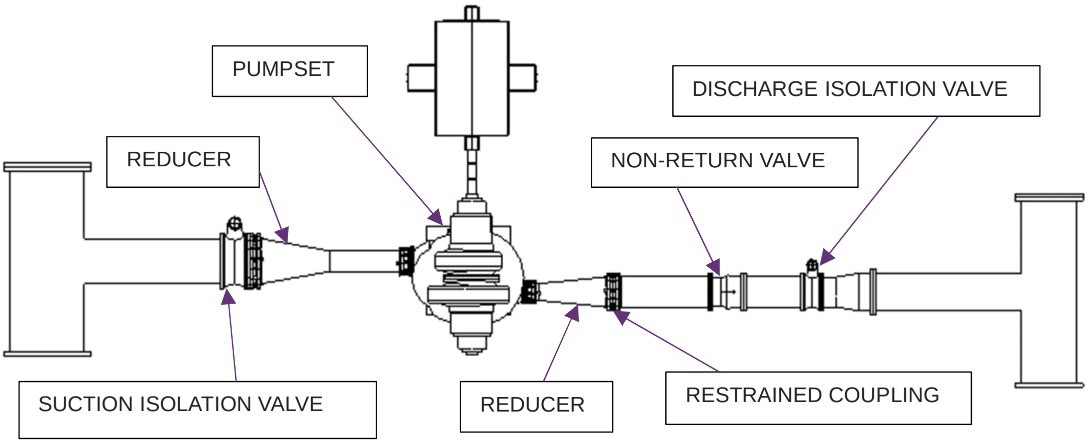


Figure 7-13: High-lift pump station – single pump leg plan view

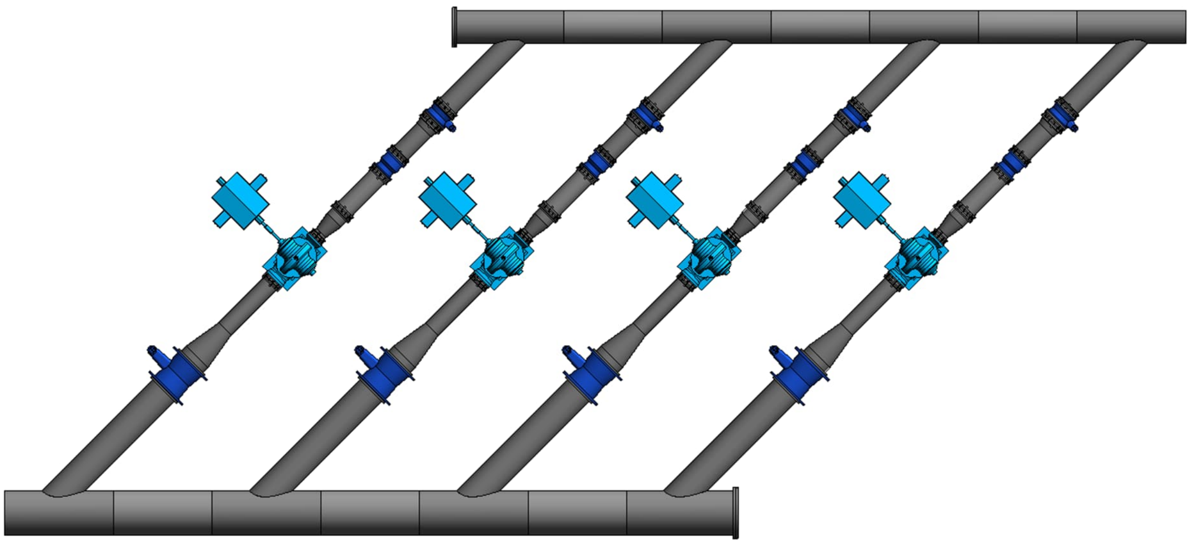


Figure 7-14: Booster pump station – overall pipework layout

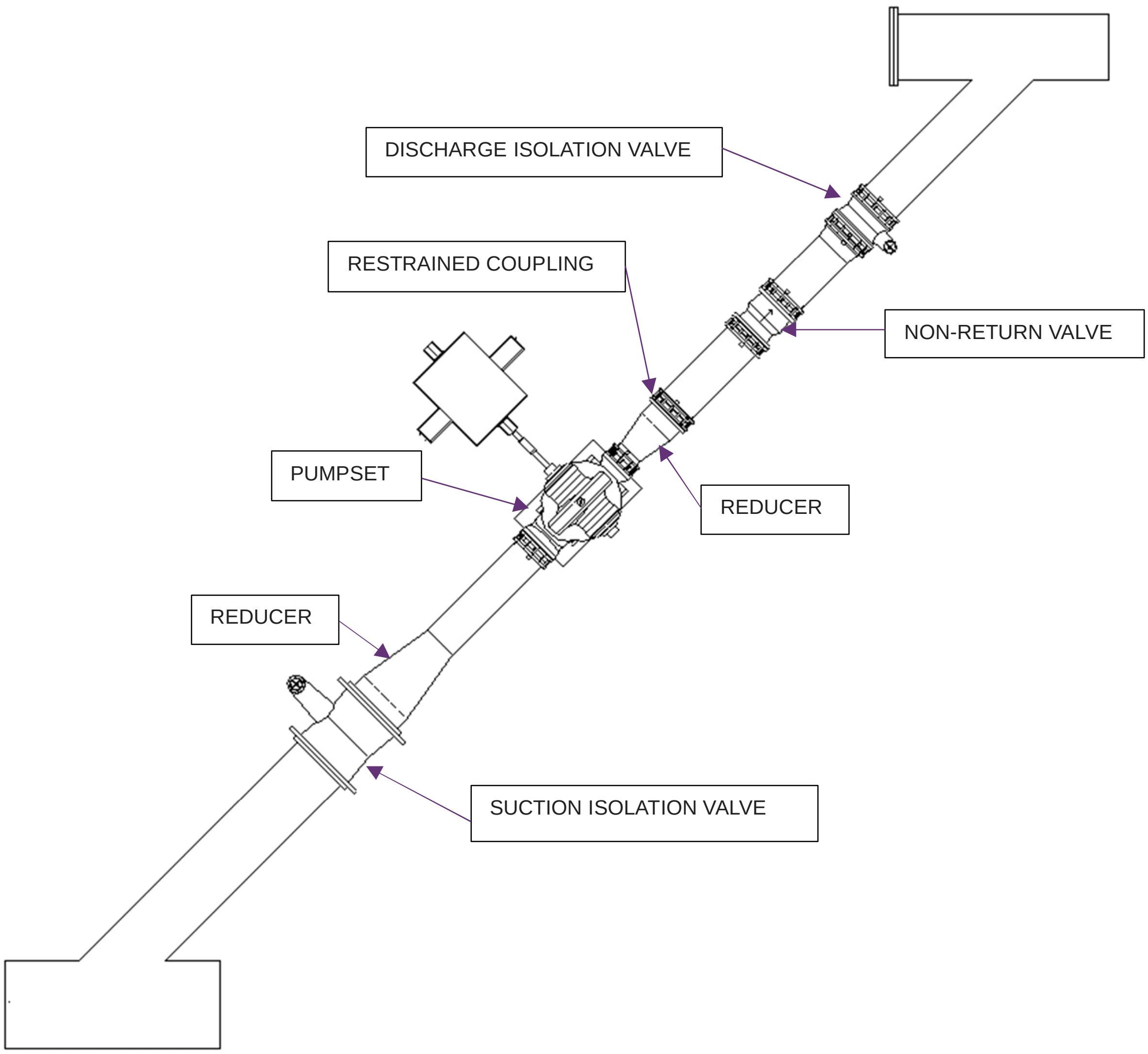


Figure 7-15: Raw water pump station – single pump leg plan view

7.2.3 Pipework

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
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. In order to mitigate the risk of vortices, the minimum submergence levels in the reservoir should be calculated as:

$$S = D(1 + 2.3F_r)$$

And

$$F_r = V/(gD)^{0.5}$$

With S = minimum submergence (m)

D = Outside diameter of bellmouth inlet (m)

Fr = Froude number (dimensionless)

V = Velocity at bellmouth inlet (m/s)

g = gravitational acceleration (m/s^2)

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 pumpset, i.e. the total weight of the pump, motor, baseframe 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.

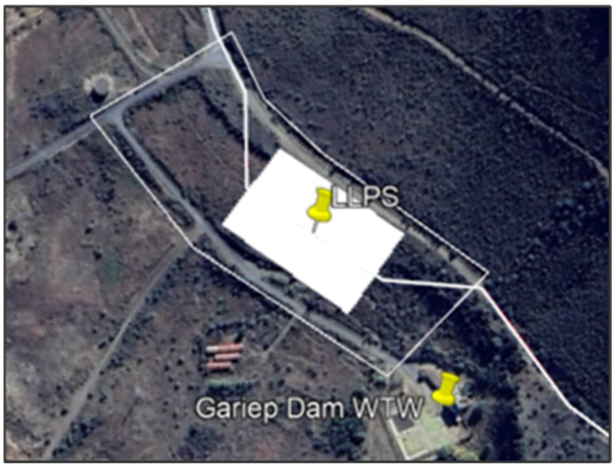
After installation and testing, load testing must be conducted at intervals not exceeding 12 months in accordance with the Occupational Health and Safety Act and Regulations.

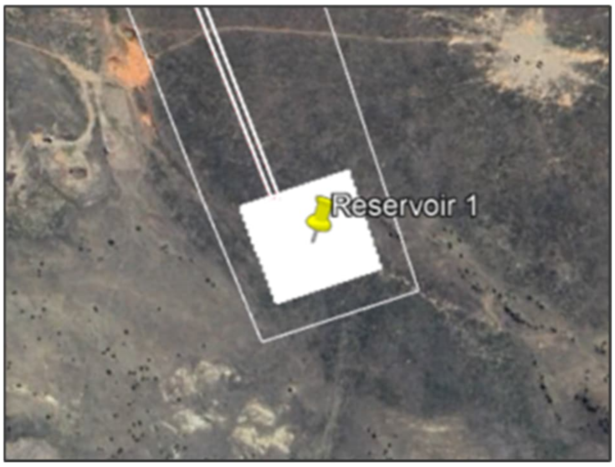
7.3 Electrical Design

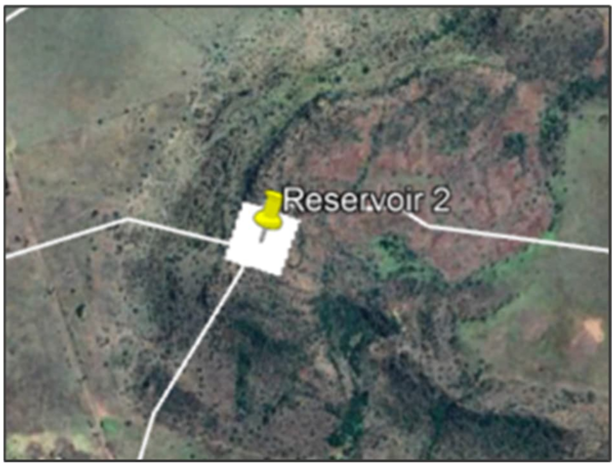
7.3.1 Bulk Electrical Supply

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.

Site	Image and coordinates	Medium and high-voltage scope of works
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 WTW and HLPS.
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.

Site	Image and coordinates	Medium and high-voltage scope of works
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
- Department of Water and Sanitation (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 located 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 controlgear 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 controlgear 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 mm² 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 Pipeline Design

8.1 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 Appendix C.

The pipeline vertical alignment follows the site topography to a large extent for reasons of economy, although smoothed out to create definite high and low points to minimize the number of air valve and scour valve installations and also to facilitate the proper scouring of the pipeline. The minimum cover over the pipe has been kept as close as possible to 1.25 m throughout since the large pipeline diameters already cause deep trench excavations. The geotechnical investigations also indicated the presence of hard rock excavations along the majority of the pipeline.

It is custom with long pipelines to include chainage breaks say every 5 km or 10 km, e.g. if each drawing contains the details of a 1 km long pipeline section, then after 9 km one drawing is left blank and a 1 km long chainage break is added. The next drawing then starts at chainage 10 km, but the actual pipeline length at this point is 9 km. If any changes in pipeline alignment are required between chainage 0 km – 9 km that will increase the actual length to say 9.4 km, then only these first 10 drawings are updated and the length of the chainage break is reduced to 600 m. Without chainage breaks, a few hundred drawings will need to be updated each time the alignment changes. Any reference to chainages in this report, refers to the actual pipe length. The chainages shown in the horizontal and vertical alignment drawings contained in Appendix C include the chainage breaks, meaning that the end chainage of the pipeline drawings will not correspond to the actual pipeline length.

8.2 Pipe Material Selection

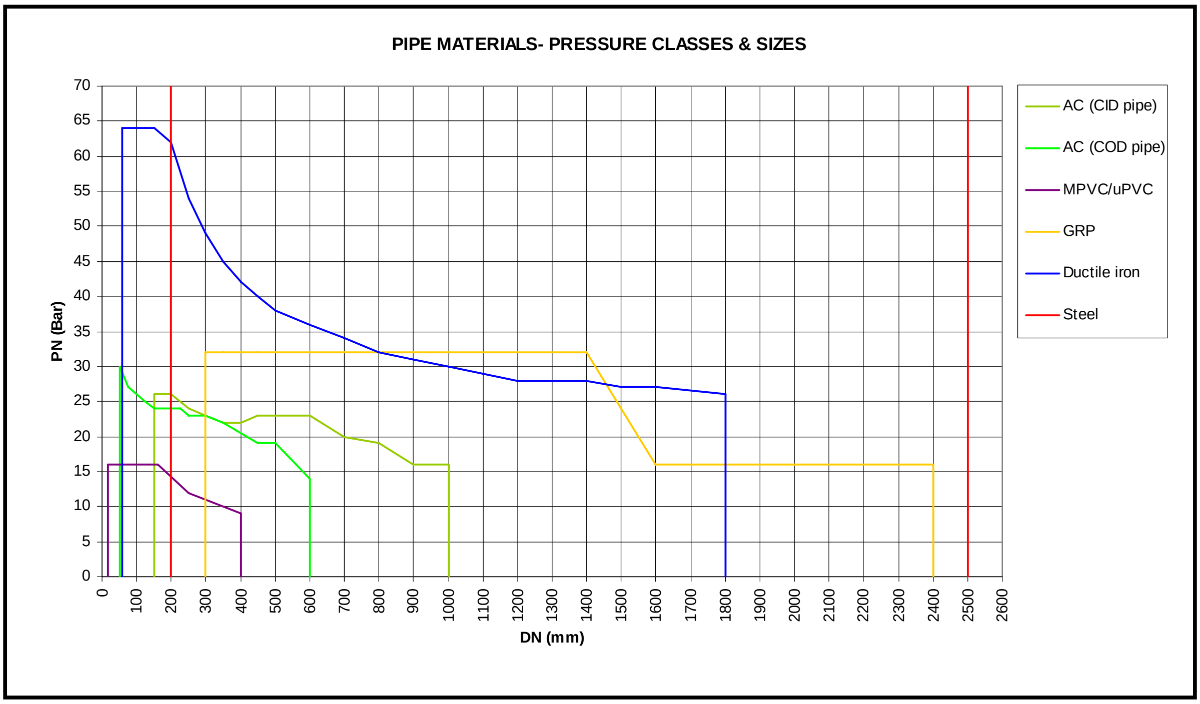
Figure 8-1 provides a graphical representation of the various pipe materials available, as well as the ranges with regard to available pressure classes and pipe diameters. As can be seen from Figure 8-1, the only pipe materials that can be considered for the proposed DN 1400 to DN 2000 pipelines are:

- Glass Reinforced Polyester (GRP) Pipes
- Mild Steel Pipes
- Ductile Iron Pipes

GRP pipes are manufactured in South Africa by Flowtite. Although the pipes have been successfully used on numerous bulk infrastructure projects, the pipes are susceptible to damage during transport and installation. The pipes also cannot handle differential settlement.

Ductile iron pipes are imported from France and China. The ductile iron pipes are generally more expensive than steel and GRP, resulting in very few ductile iron pipes being installed in South Africa over the past decade.

Based on the above, mild steel is the preferred pipe material for the Xhariep Project. This has 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.



Source: Pipeline Engineering and Hydraulics – One Day Course, Water Engineering, SAICE. Course Presented on 2 October 2002 – Pretoria. Presenters: Professor SJ van Vuuren & Mr. D van Bladeren

Figure 8-1: Pipe material diameters and pressure ratings

8.3 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 following factors needs to be accounted for during detail design

- A live wheel load of 80 kN
- Maximum water table at 500 mm below natural ground level

- Minimum soil cover on pipe = 1.25 m

- Maximum soil cover on pipe = 2.7 m (deeper depths occur at river and road crossings, but here the pipe is encased in concrete or installed in a sleeve, and the wall thickness is increased by 20% - refer to Sections 11.3 and 11.4)
- Maximum working pressures = as per Section 6.3 of the report for the respective pipelines
- Maximum surge pressures = as per Section 6.3 of the report for the respective pipelines
- Minimum surge pressures = - 10.33 m (vacuum pressure)
- Factor of safety for working pressures = 1.67
- Factor of safety for surge pressures = 1.67
- Factor of safety for combined stresses = 1.65
- Maximum deflection with to epoxy lining = 3%

The design of the pipe was based on Grade X52 steel with a Yield Modulus of 358 MPa. The applicability of a Grade X42 steel with a Yield Modulus of 300 MPa, was tested and showed that the pipe wall thickness would need to be substantially increased when opting for X42. Given the marginal increase of ± 2% in price for Grade X52, as quoted by the pipe manufacturers, the Grade X52 steel was used in the structural design of the pipe.

Based on the above design parameters, the required wall thicknesses can be calculated for different pressure classes and pipe diameters. Table 8-1 summarises the required pipe wall thicknesses based on pipe diameters and pressure classes.

Table 8-1: 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

8.4 Pipeline Lining and Coating

8.4.1 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.

8.4.2 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 8-2 summarises the field joint coatings, as well as the coatings of buried specials, recommended for each of the four coatings.

Table 8-2: 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.

8.5 Air Valve Design

8.5.1 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

8.5.2 Filling of the pipeline

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

8.5.3 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.

8.5.4 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.

8.5.5 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.

8.5.6 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.

8.5.7 Air valve chamber details

Drawings with details of the air valve chambers are included under Appendix C.

8.6 Scour Valve Design

8.6.1 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.

8.6.2 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).

8.6.3 Scour valve chamber details

Drawings with details of the different types of scour valve chambers are included under Appendix C.

8.7 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 in order 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 Appendix C. 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.

8.8 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 Appendix C.

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.

8.9 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.

9 Reservoir Design

9.1 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 m^3), whereas the suction reservoir at the booster pump station is sized for a capacity of 10 Mℓ (i.e. 10 000 m^3).

9.2 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.

9.2.1 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 m^2
- Reinforced concrete quantity = 8 350 m^3

Figure 9-1 illustrates a cut-out image of how the geometry of such a circular reservoir could look.

9.2.2 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 m^2
- Reinforced concrete quantity = 7 590 m^3

Figure 9-2 illustrates a cut-out image of how the geometry of such a rectangular reservoir could look.

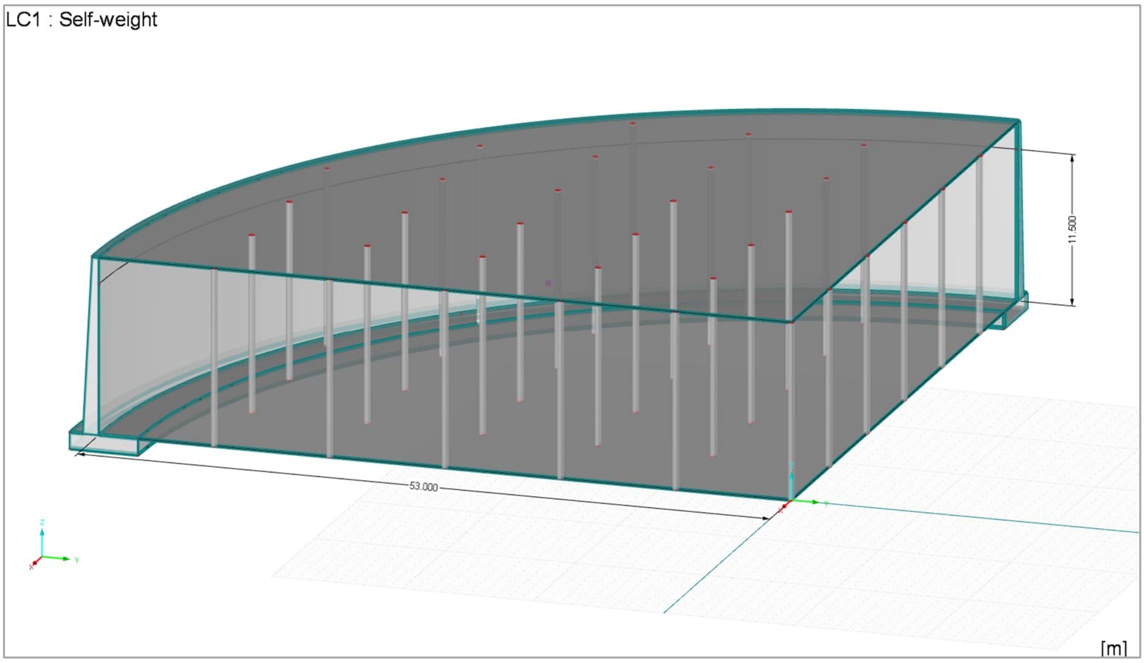


Figure 9-1: Type1 - Conventional above ground post-tensioned circular reinforced concrete reservoir

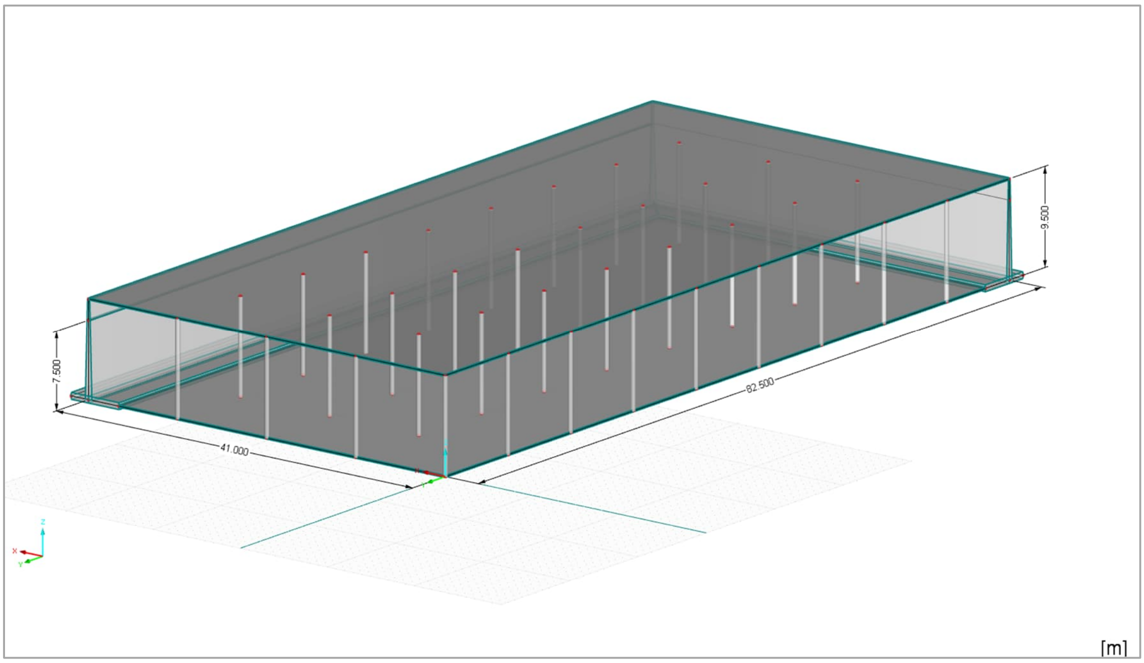


Figure 9-2: Type 2 - Conventional above ground rectangular reinforced concrete reservoir

9.2.3 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 9-3 illustrates a cut-out image of how the geometry of such an earth-fill embankment type reservoir could look.

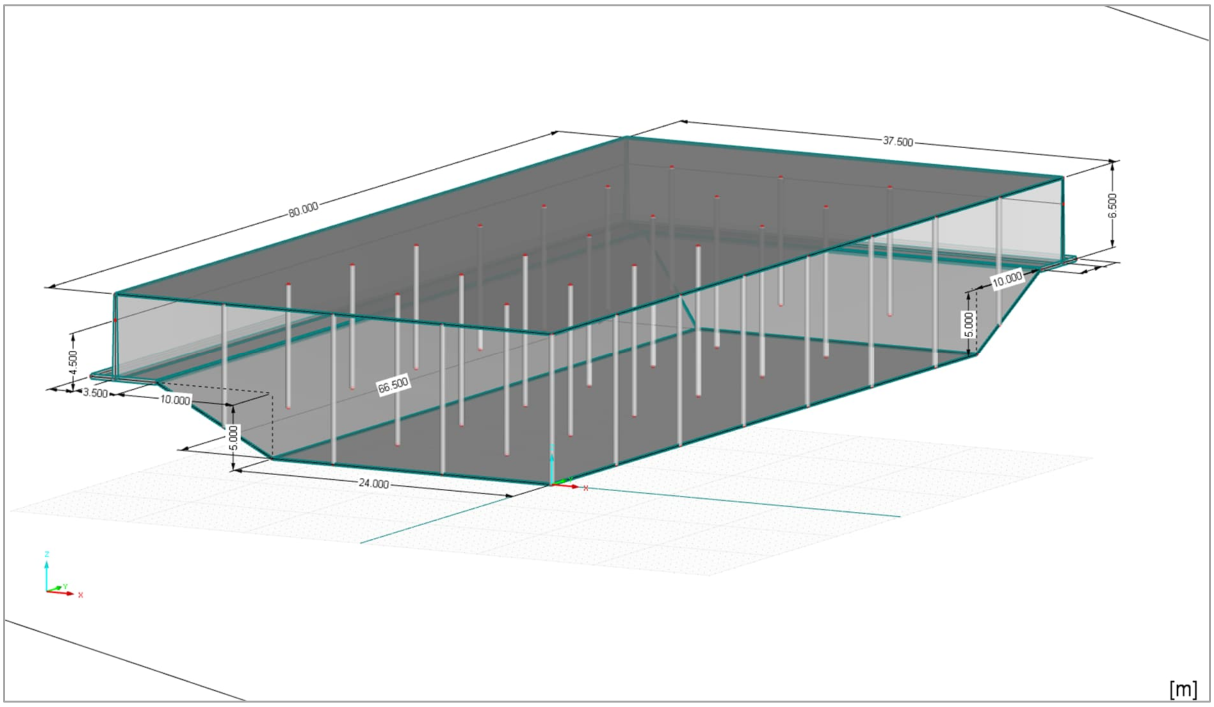


Figure 9-3: Type 3 - Example of Earth fill embankment type or hopper bottom reservoir

9.2.4 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.

9.3 Structural design

9.3.1 Design Standards

The following codes of practice were utilized to design the reservoir and need to be considered during the detailed design phase:

- BS8007 – 1987: Code of practice for design of concrete structures for retaining aqueous liquids.
- BS 8110-1997: Structural Use of Concrete-Code of Practice for Design and Construction
- SANS 10100 – 1 & 2: The structural use of Concrete Part I – Design and part II – Materials and execution of work.
- SANS 282 – 2004: Bending dimensions and scheduling of steel reinforcement for concrete.
- SANS 10160 suite of documents – Basis of Structural Design.

In addition to the above, water-retaining and water-excluding structures are designed in accordance with BS8007:1987 (incorporating BS8110-1: 1997 for both normally reinforced concrete and prestressed concrete elements design). Updated cement hydration temperatures and further guidance in crack width design shall be obtained from Ciria C766:2018 *Control of cracking caused by restrained deformation in concrete*. ACI 350.3-01 "*Seismic Design of Liquid-Containing Concrete Structures*" will also be used in conjunction with the above standards during detailed design.

The following provisions are applicable for the project:

- Durability, that the water:cementitious binder (w/b) ratio shall not exceed 0.5, with a minimum cementitious binder content of 300 kg kg/m^3 and a maximum cementitious binder content of 360kg/m^3;
- Placing, that in the case of continuous walls, these are to be cast in lifts of such heights that each lift can be poured uninterruptedly in one continuous operation over the entire perimeter of the wall – no vertical or inclined construction joint of any kind will be permitted; and
- The roof will be treated as a water retaining element, since the contractor will need to test the roof slab for such.

The specifications above (not necessarily limited to) address the foundation and durability issues for the structure.

The design loads used are:

- Walls: Hydrostatic pressure of 10kN/m^2 per m depth
- Roof: Imposed Live load of 0.5kN/m^2
- Imposed dead load of 75mm layer of course aggregate may be applied depending on whether the roof will be precast or not.

9.3.2 Concrete Design

Structural concrete elements are designed for ultimate limit state (ULS) and serviceability limit state (SLS) according to the applicable SANS design codes. Furthermore, the following will also be integrated into the detailed design:

- Water-retaining and water-excluding structures will be designed for serviceability limit state (SLS) to a maximum allowable crack width of 0.20mm for normal operating conditions, e.g. water-retaining structure full to maximum operating top water level. This includes Full Supply Level (FSL) for the reservoir structure.
  - BS 8007 allows a maximum stress in steel of 130MPa limiting the crack-width in the concrete. The reason for this is that at the crack it is assumed that the bond between steel and concrete is lost and the strain (and hence stress) in the steel is at its maximum. At the crack the strain in the concrete reduces to zero and as bond is re-established along the bar, the strain in the steel reduces to a level no greater than the strain capacity of the concrete at some distance from the crack.
  - Often plastic shrinkage cracks can be eliminated by re-vibrating the fresh concrete prior to the initial set (i.e. around 30-60 minutes after casting). This can help to re-mix and restructure the surface layer of the concrete which is susceptible to moisture-loss (and thus cracking) in the freshly placed state. It is also critical to immediately protect such slabs by use of plastic sheeting, moisture sprays etc.
- Water-retaining and water-excluding structures will be designed for the ultimate limit state (ULS) for faulted conditions, e.g. overspill in water-retaining structures.
- To ensure concrete durability, concrete mix design specifications (especially for watertight concrete) shall be guided by the need to minimise the permeability of the concrete, reduce the heat of hydration and thermal gradient of thick sections at early-age, and maximise the chemical resistance of the concrete to aggressive agents in the environment, if applicable.
- Concrete durability to be further ensured by specifying replacement of cement content with suitable cement extenders such as fly-ash or ground-granulated blastfurnace slag, limiting water-binder ratios, adequate concrete cover, etc.
- All steel reinforcement shall be in accordance with SANS 10144:2012 (Detailing of steel reinforcement for concrete). Reinforcement steel design strength (fy) used in design shall be fy = 450 MPa for high tensile reinforcement (Y), and fy = 250 MPa for mild steel reinforcement (R)

9.3.3 Finite element analysis of forces in structure

The Design software used for the finite element modelling may include (but is not be limited to):

- Prokon Structural Engineering Analysis and Design Software;
- CSI Etabs Building Analysis and Design; and
- Dlubal RFEM Structural Analysis and Design Software.

During the current design phases (concept/feasibility), key structural checks and sizing of key elements were carried out by the use of manual calculations and / or the use of the Prokon suite of software listed above. Once detailed design commences and other key design parameters are available e.g. geotechnical parameters, a Finite Element Model (FEM) for the entire reservoir structure will be created using PROKON and / or DLUBAL RFEM software.

Various load combinations, each with its specific set of load cases, will be considered and applied to the FEM. The resultant response of the structure to each load combination will then be analysed to determine various design parameters / outcomes, including but not limited to the following:

- Support reactions and ground / soil bearing pressures.
- Load responses e.g. bending moments and shear forces.
- Internal stresses of structural members e.g. walls, roof members, floor slabs and columns under permanent and transient loads.
- Stabilities of structural members under hydrostatic and hydrodynamic pressures (containment liquid), static and dynamic earth-retaining pressures (external backfill) and lateral seismic loads due to mass inertia of the elements.

9.4 Reservoir pipework

The section below provides brief background to the design and sizing of the various pipework components.

9.4.1 Freeboard

The reservoir was designed with a 1500 mm freeboard between the top water level (TWL) and the underside of the reservoir roof.

The top of the overflow weir is located 100mm above the TWL.

9.4.2 Inlet pipework

The invert level of the inlet pipework is located 300mm above the footing of the 4.5 m high concrete retaining wall. The diameter of the inlet pipework is the same as that of the main pipeline feeding the reservoir, i.e. DN1800 for Command Reservoir No 1 and Command Reservoir No 2. The top of the inlet pipework is therefore 2 900 mm below the TWL.

The high-level inlet also improves the pump selection by ensuring that the pumps do not have a significant variation in the operating heads of the suction and discharge reservoirs.

9.4.3 Outlet pipework

Two reservoir outlets will be provided, each with the same diameter as the main outlet pipeline from the reservoir, i.e. DN 1800.

The outlet pipe will protrude the concrete floor of the reservoir by 200mm to prevent large particles (e.g. stones) from entering the outlet pipeline. In order to minimise the submergence at the outlets, a bellmouth can be fitted to the outlet pipeline or a vortex breaker device can be installed at the outlet.

9.4.4 Overflow pipework

The overflow is designed as a weir located on the side of the reservoir. The overflow weir is sized for the maximum inflow into the reservoir, i.e. 3.616 m^3/s.

The overflow pipework will discharge into a stormwater pond with energy dissipation structure, before the pond overflows in a controlled manner into the downstream environment.

9.4.5 Scour pipework

Two scour pipes are provided, which run parallel to the two outlet pipelines. The scour pipework will each be fitted with an isolation valve, which will generally be in the closed position.

Each scour pipeline will discharge into the stormwater pond, which will also receive water from the reservoir overflow.

9.4.6 Pipe material

All pipework that will be encased in concrete and located inside the reservoir will be uncoated Grade 304 stainless steel.

All pipework that will be encased in concrete and located inside the reservoir will be uncoated Grade 316 stainless steel. All joints will be welded and no flanged joints to be cast in. The welded joints will reduce the risk of leakage due to flange bolts not being tightened properly.

9.5 Operating levels

The operating levels in the various reservoirs are summarised in Table 9-1.

Table 9-1: 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

10 Site access

10.1 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

10.2 Raw water pump station

Access to the raw water pump station will be from an existing surfaced road as shown on Drawing 1002533-1150.

10.3 Xhariep Water Treatment Plant

Access to the Xhariep Water Treatment Plant will be from the R701 leading to Bethulie as shown on Drawings 1002533-2501 and 1002533-2502.

10.4 Command Reservoir No 1

Access to Command Reservoir No 1 will be from an existing surfaced road located to the east of the N1 high-way as shown on Drawing 1002533-4150.

10.5 Booster Pump Station

Access to the Booster Pump Station will preferably be from the N1 high-way as shown on Drawing 1002533-5150. The exact location of the access onto the N1 high-way will need to be confirmed during the detailed design phase with SANRAL based on the expected access requirements and frequency of use. Alternatively, access will need to be provided from existing gravel roads.

10.6 Command Reservoir No 2

Access to Command Reservoir No 2 will be from the R702, using the landowner’s existing access to his property, as shown on Drawing 1002533-6150.

11 Special Design Considerations

11.1 Control Narrative

A Block Flow Diagram (BFD) is shown on Drawing 1002533-0010 (refer to Appendix C) 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.

11.2 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.

11.3 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 Appendix C.

11.4 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 Appendix C.

11.5 Servitudes

The proposed servitude and working widths are shown on the drawings included in Appendix C. 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.

11.6 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.

11.7 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 Authorisation Processes

12.1 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.

12.2 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 Wastewater 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.

13 Financial Considerations

13.1 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 13-1.

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.
- 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 13-1: Estimated Capital Expenditure for the Xhariep Pipeline Infrastructure

Description	Estimated CAPEX (Rand)
Preliminary and General	**3,939,328,274**
Civil Contractor P&G	3,765,398,720
Mechanical Contractor P&G	113,412,000
Electrical Contractor P&G	57,831,054
Electronic Contractor P&G	2,686,500
Raw water pump station	**162,443,508**
Building/structural	31,826,025
Earthworks	2,555,863
Internal roads	2,700,390
Access road	5,699,000
Site services	1,928,850
Miscellaneous (e.g. fencing, stormwater)	1,157,310
Pumpsets	28,564,800
Valves	8,923,200
Pipework	12,038,400
Crane and ventilation	3,273,600
MV switchgear	34,251,310
VSDs and cabling	22,235,200
LV switchgear	3,895,430
Control and instrumentation	2,987,560

Description	Estimated CAPEX (Rand)
Electrical building services	406,570
High-lift pump station	**359,650,870**
Building/structural	51,037,800
Earthworks	3,726,100
Internal roads	3,936,800
Access road	Included in WTP costs
Site services	2,812,000
Miscellaneous (e.g. fencing, stormwater)	1,687,200
Pumpsets	118,825,240
Valves	37,119,160
Pipework	50,077,920
Crane and ventilation	13,617,680
MV switchgear	61,703,810
VSDs and cabling	6,681,950
LV switchgear	5,519,880
Control and instrumentation	2,498,760
Electrical building services	406,570
Command Reservoir No 1	**137,008,420**
Building/Structural	63,200,000
Earthworks	10,452,400
Internal roads	6,963,300
Access road	45,592,250
Services	8,880,470
Miscellaneous (e.g. fencing, stormwater)	1,920,000
Booster Pump Station and Suction Reservoir	**292,708,069**
Building/structural	66,381,775
Earthworks	17,513,854
Internal roads	5,639,100
Access road	29,634,800
Site services	4,516,500
Miscellaneous (e.g. fencing, stormwater)	2,283,900
Pumpsets	57,129,600
Valves	17,846,400
Pipework	24,076,800
Crane and ventilation	6,547,200
MV switchgear	52,432,530
VSDs and cabling	2,154,430
LV switchgear	3,895,640
Control and instrumentation	2,248,970
Electrical building services	406,570
Command Reservoir No 2	**304,276,170**

Description	Estimated CAPEX (Rand)
Building/Structural	63,200,000
Earthworks	104,773,200
Internal roads	6,963,300
Access road	118,539,200
Services	8,880,470
Miscellaneous (e.g. fencing, stormwater)	1,920,000
Pipelines	**14,385,273,022**
Site clearance	180,121,973
Excavation	4,734,180,756
Pipelines	8,665,464,831
Chambers	670,569,297
Road crossings	86,566,704
Cathodic protection	48,369,460
Subtotal capital cost (excl. VAT)	**19,580,688,333**
Contract Price Adjustment (CPA) @7% p.a.	4,406,496,844
Contingency @ 15%	3,598,077,777
Project cost (excl. VAT)	**27,585,262,954**
Professional fees (excl. VAT)	**2,317,162,088**
Engineering design fees @8%	2,206,821,036
Disbursements and recoverable costs	110,341,052
Total project cost (excl. VAT)	**29,902,425,042**

Notes:

1 Cost Estimate Base Date – November 2024.

2 Construction Commencement Date – November 2028

13.2 Operation and Maintenance Cost

A design for a pumping scheme is highly dependent on the operation and maintenance requirements, which in turn are based on the operational complexity of the infrastructure installed. The requirements of the Client, technical skills of staff and ability of the Client to maintain the infrastructure are also key aspects to be considered.

In addition to comparing the capital costs, it is also worthwhile to compare the costs over the project lifecycle for the different options including the capital costs as well as the operation and maintenance costs (particularly electricity). The following assumptions were made for estimation of the operation and maintenance costs:

- Maintenance costs were based on the type of structure, mechanical or electrical component. This was done as a percentage of the CAPEX and/or capital replacement cost (CRC).
- Electricity costs were estimated by using a rate of electricity of R1.44/kWh and the estimated average energy consumption of the different options.
- Labour costs were estimated based on the estimated number of personnel required for the operation of the pump stations and reservoirs.

13.2.1 Estimated Maintenance Cost

The annual maintenance costs are estimated as a percentage of the CRC of the specific components as shown in Table 13-2.

Table 13-2: Estimated annual maintenance cost as a function of CRC

Description	Maintenance	Value (% of total)	Annual maintenance cost
Civil	0.75% of CRC	96.2%	141,202,452
Mechanical	2.25% of CRC	2.5%	11,057,670
Electrical	3.0% of CRC	1.3%	7,518,037
C&I	3.0% of CRC	0.1%	349,245
Total			**160,127,404**

Notes:
1 Estimated maintenance cost required for first year of scheme operation are based on 2025 Costs

13.2.2 Estimated Operational Cost

13.2.2.1 Human Resources Cost

The cost of staff employed for the operation and maintenance of the pump stations, reservoirs and pipelines was calculated based on the level of superintendent, foremen, technicians and labourers. The salary estimates were escalated accordingly and based on industry accepted monthly/annual salaries for suitably qualified personnel. Note that these are first order estimates only and have been derived from previous projects of a similar nature and size.

The costs documented in Table 13-3 represents a summary of the approximate annual salaries.

Table 13-3: Approximate annual staffing costs for the Xhariep Infrastructure (excl. WTP)

Description	R/Month	Number of staff	Annual staffing cost (Rand)
Superintendent	R 75 000	2	1,800,000
Mechanical foreman	R 40 000	2	960,000
Electrical foreman	R 40 000	2	960,000
Mechanical and electrical technician	R 25 000	4	1,200,000
General labourers	R 20 000	6	1,800,000
Total		**16**	**6,720,000**

13.2.2.2 Other Operational Costs

The other operational costs related to the pump stations include energy cost. The energy cost was estimated based on the estimated mechanical/loads of the three pump stations with a blended energy tariff of R 1.44/kWh. It should be noted that these costs will increase gradually as the flow to the scheme increases until the design horizon.

The other operational costs are shown in Table 13-4 for the first year of operation of the scheme.

Table 13-4: Estimated other Annual Operational Costs for the Xhariep Scheme (excl. WTP)

Description	Estimated Additional Annual Operating Cost (Rand)		
	Complete Phase 312 Mℓ/d [1]	Phase 1 208 Mℓ/d [2]	Phase 2 +104 Mℓ/d [3]
Energy	34,863,048	23,242,032	22,860,297
TOTAL	**34,863,048**	**23,242,032**	**22,860,297**

Notes:
1 Estimated other operational cost required for first year of plant operation based on 2025 Costs
2 Estimated other operational cost required for first year of plant operation based on 2025 Costs
3 Estimated other operational cost required for first year of plant operation based on 2035 Costs

13.2.2.3 Summary

The estimated operation and maintenance budget required for the first year of operation is summarised in Table 13-5, showing an estimated minimum O&M budget requirement.

Table 13-5: Estimated Annual Operation and Maintenance Budget for the Xhariep Infrastructure (excl. WTP)

Description	Estimated Annual Maintenance and Operating Budget (Rand)	
	Complete Phase 312 Mℓ/d [1]	Phase 1 208 Mℓ/d [1]
Maintenance	141,202,452	141,202,452
Labour	6,720,000	6,720,000
Energy	34,863,048	23,242,032
TOTAL OPEX (Excl. VAT)	**182,785,500**	**171,164,484**

Notes:
1 Estimated other operational cost required for first year of plant operation based on 2025 Costs

14 Project Programme

Table 14-1 shows an indicative programme for the implementation of Phase 1 of the Xhariep Pipeline Infrastructure project. Figure 14-1 shows the programme in Gantt chart form.

Table 14-1: 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[1] 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

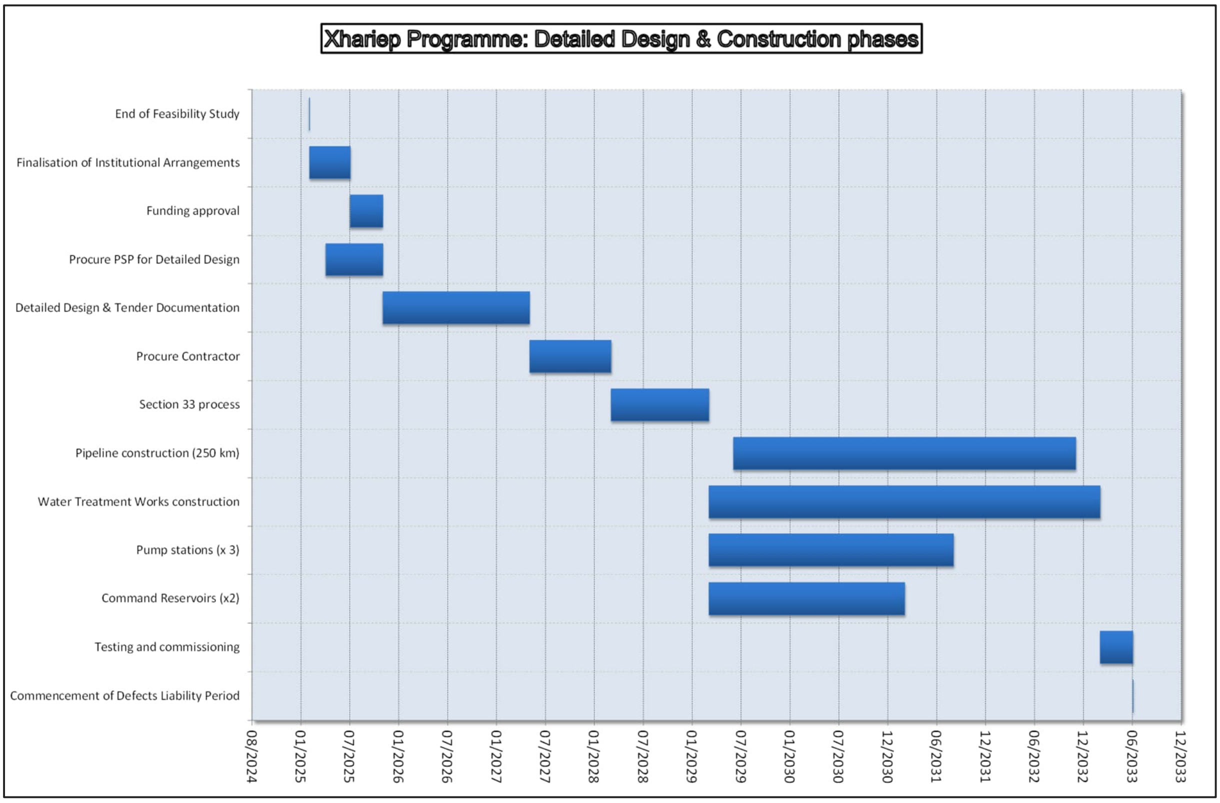


Figure 14-1: Xhariep Pipeline Infrastructure implementation programme (Phase 1)

Based on the indicative programme, the detailed design needs to commence towards the end of 2025 in order 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.

15 Conclusions and Recommendations

The pre-feasibility study concluded that Scheme 1B, as shown in Figure 15-1, 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.

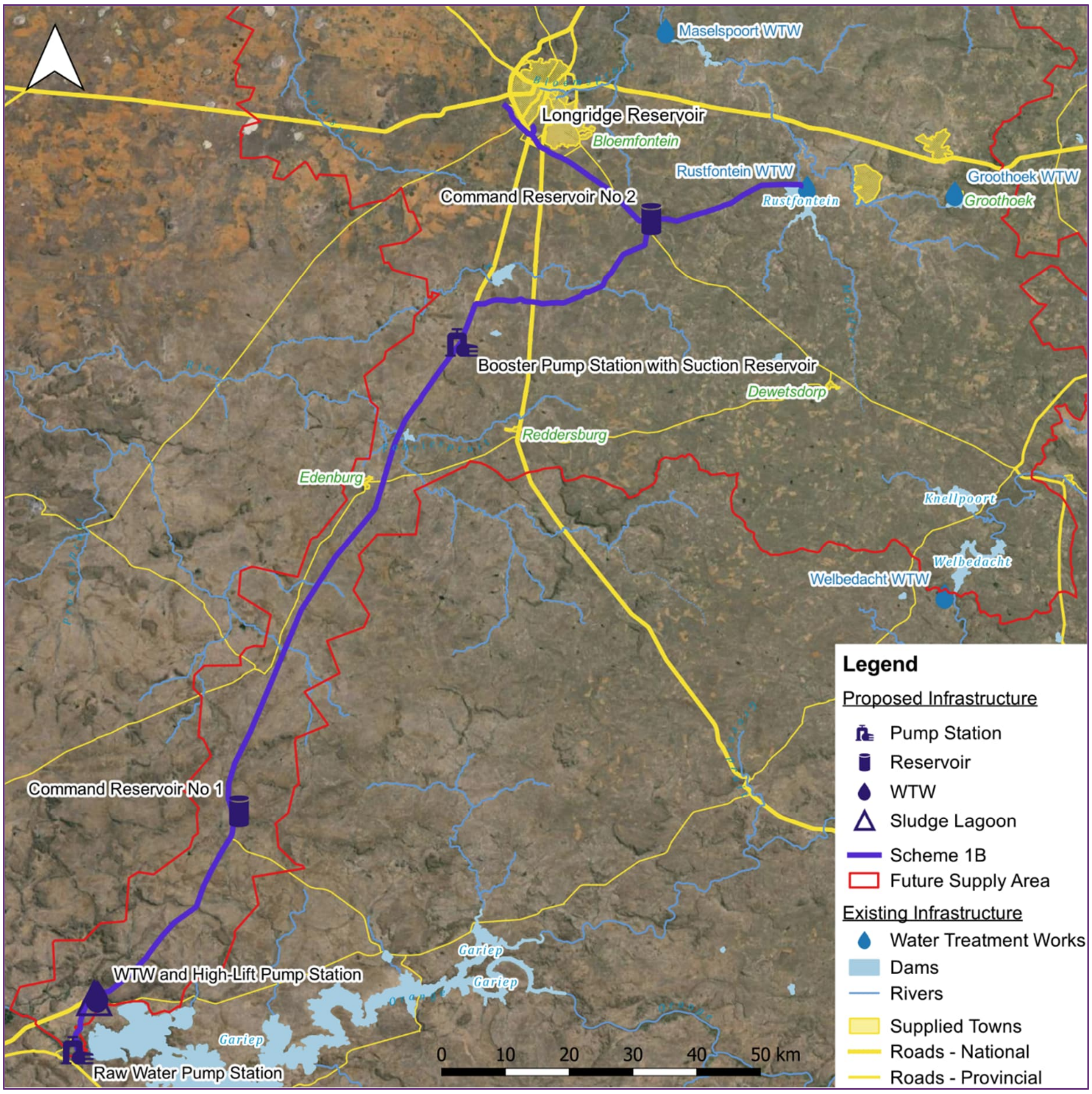


Figure 15-1: Main infrastructure components of Scheme 1B

A long-term stochastic analysis confirmed that the proposed potable transfer scheme at a capacity of 101 million m^3/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 m^3/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 m^3/s (312 Mℓ/d). The two command reservoirs were sized for 6 hours storage at the peak week flow rate of 3.616 m^3/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.

The main infrastructure components of Scheme 1B, as shown in Figure 15-1, 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 (WTW), ± 9 km long,
- The Xhariep WTW, 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 WTW 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 WTW (± 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 WTW = DN 1400, and
- Pipeline from Command Reservoir No 2 to the existing Longridge Reservoirs = DN 2000.

The duty points for the three pump station (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 15-1.

Table 15-1: Pump selection details

Description	Raw water	High-lift	Booster
Pump duty	3.797 m^3/s @ 73m	3.616 m^3/s @ 320m	3.616 m^3/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 15-1 that the raw water pump station pumpsets 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 15-2 summarises the maximum design and field test pressures for the various pipeline sections.

Table 15-2: 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

Pipe section	Pipe diameter (mm)	Maximum design pressure (m)	Maximum field test pressure (m)	Maximum pressure rating of valves, specials, etc. (m)
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.

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 15-3 provides a summary of the capital cost estimate for the Xhariep Pipeline Infrastructure, excluding the Xhariep WTP.

Table 15-3: Estimated Capital Expenditure for the Xhariep Pipeline Infrastructure

Description	Estimated CAPEX
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
Subtotal capital cost (excl. VAT)	**19,580,688,333**
Contract Price Adjustment (CPA) @7% p.a.	4,406,496,844
Contingency @ 15%	3,598,077,777
Project cost (excl. VAT)	**27,585,262,954**
Professional fees (excl. VAT)	**2,317,162,088**
Engineering design fees @8%	2,206,821,036

Description	Estimated CAPEX
Disbursements and recoverable costs	110,341,052
Total project cost (excl. VAT)	**29,902,425,042**

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 15-4, showing an estimated minimum O&M budget requirement.

Table 15-4: Estimated Annual Operation and Maintenance Budget for the Xhariep Infrastructure (excl. WTP)

Description	Estimated Annual Maintenance and Operating Budget (Rand)	
	Complete Phase 312 Mℓ/d [1]	Phase 1 208 Mℓ/d [1]
Maintenance	141,202,452	141,202,452
Labour	6,720,000	6,720,000
Energy	34,863,048	23,242,032
TOTAL OPEX (Excl. VAT)	**182,785,500**	**171,164,484**

Notes:
2 Estimated other operational cost required for first year of plant operation based on 2025 Costs

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

Pump curves and pump details

SULZER

Pump Performance Datasheet

Customer	:
Customer reference	:
Item number	: 001
Service	:
Quantity	: 1

Sulzer Reference ID	: 152066
Type / Size	: SMD 500-750 A
Stages	: 1
Based on curve number	: SMD-030.009-53-21-01
Date of Last Update	: 31 Oct 2024 6:34 AM

Operating Conditions

Flow, rated	: 1.27 m3/s
Differential head / pressure, rated (requested)	: 73.00 m
Suction pressure, rated / max	: 0.00 / 0.00 bar.g
NPSH available, rated	: Ample
Site Supply Frequency	: 50 Hz

Performance

Speed criteria	: Synchronous
Speed, rated	: 990 rpm
Impeller diameter, rated	: 740 mm
Impeller diameter, maximum	: 750 mm
Impeller diameter, minimum	: 600 mm
Efficiency	: 89.98 %
NPSH (3% head drop) / margin required	: 5.68 / 1.46 m
Ns (imp. eye flow) / Nss (imp. eye flow)	: 1,579 / 11,038 US Units
MCSF	: 0.63 m3/s
Head, maximum, rated diameter	: 92.02 m
Head rise to shutoff	: 26.06 %
Flow, best eff. point	: 1.25 m3/s
Flow ratio, rated / BEP	: 101.48 %
Diameter ratio (rated / max)	: 98.67 %
Head ratio (rated dia / max dia)	: 95.59 %
Cq/Ch/Ce/Cn [ANSI/HI 9.6.7-2010]	: 1.00 / 1.00 / 1.00 / 1.00
Selection status	: Acceptable

Liquid

Liquid type	: Water
Additional liquid description	:
Solids diameter, max	: 0.00 mm
Solids concentration, by volume	: 0.00 %
Temperature, rated / max	: 20.00 / 20.00 deg C
Fluid density, rated / max	: 0.998 / 0.998 kg/dm3
Viscosity, rated	: 1.00 cSt
Vapor pressure, rated	: 0.02 bar.a

Material

Material selected	: Ductile Iron

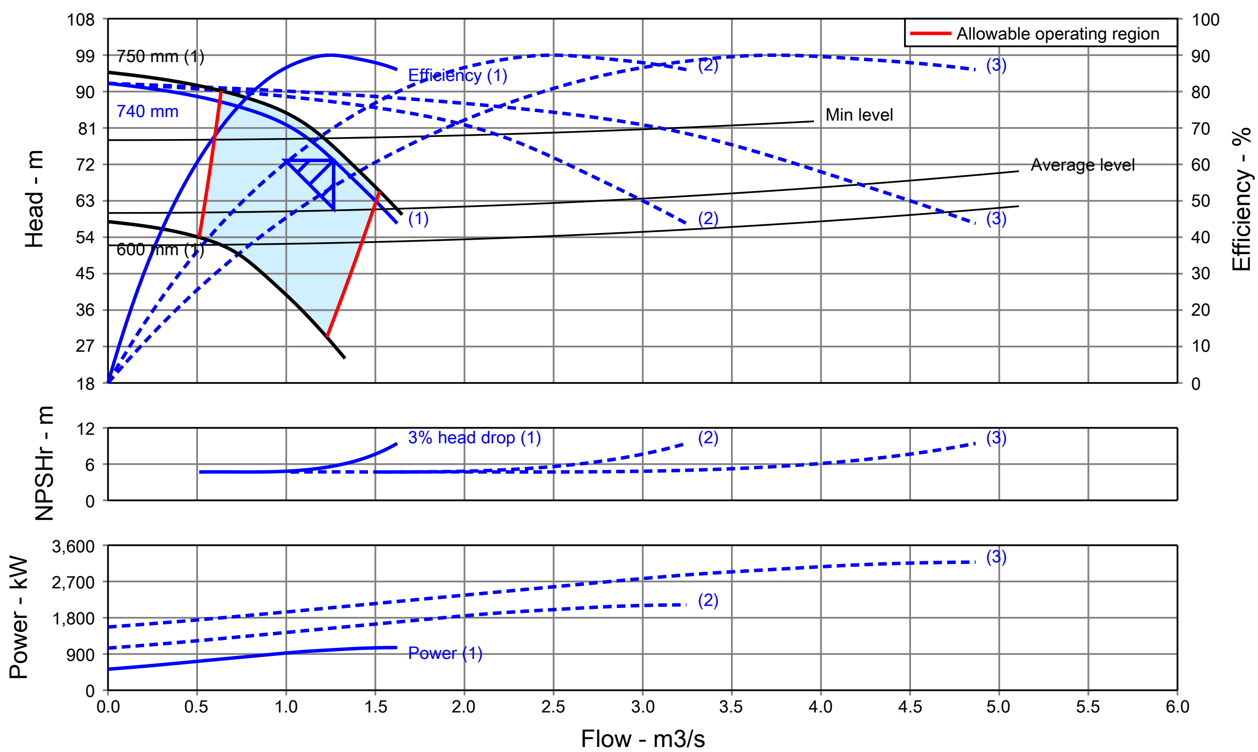
Pressure Data

Maximum casing/bowl working pressure	: 9.01 bar.g
Maximum allowable working pressure	: 16.00 bar.g
Maximum allowable suction pressure	: 2.00 bar.g
Hydrostatic test pressure	: 20.80 bar.g

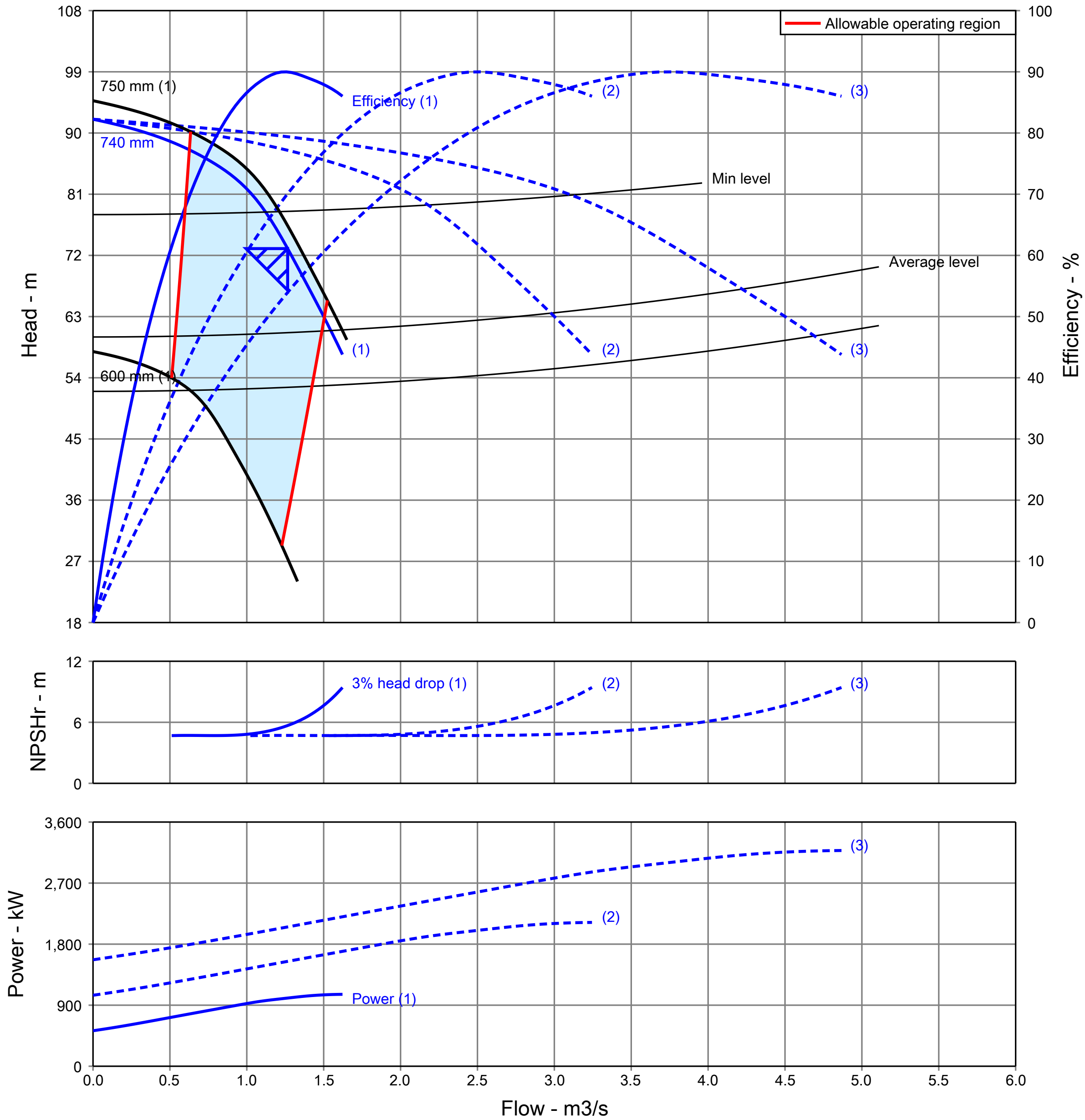
Driver & Power Data (@Max density)

Driver sizing specification	: Rated power
Margin over specification	: 0.00 %
Service factor	: 1.00
Power, hydraulic	: 904 kW
Power, rated	: 1,005 kW
Power, maximum, rated diameter	: 1,060 kW
Minimum recommended motor rating	: 1,120 kW / 1,502 hp

Hydraulic Performance Acceptance Test: ISO 9906:2012 / HI 14.6-2011 Grade 1B



Pump Performance Curve

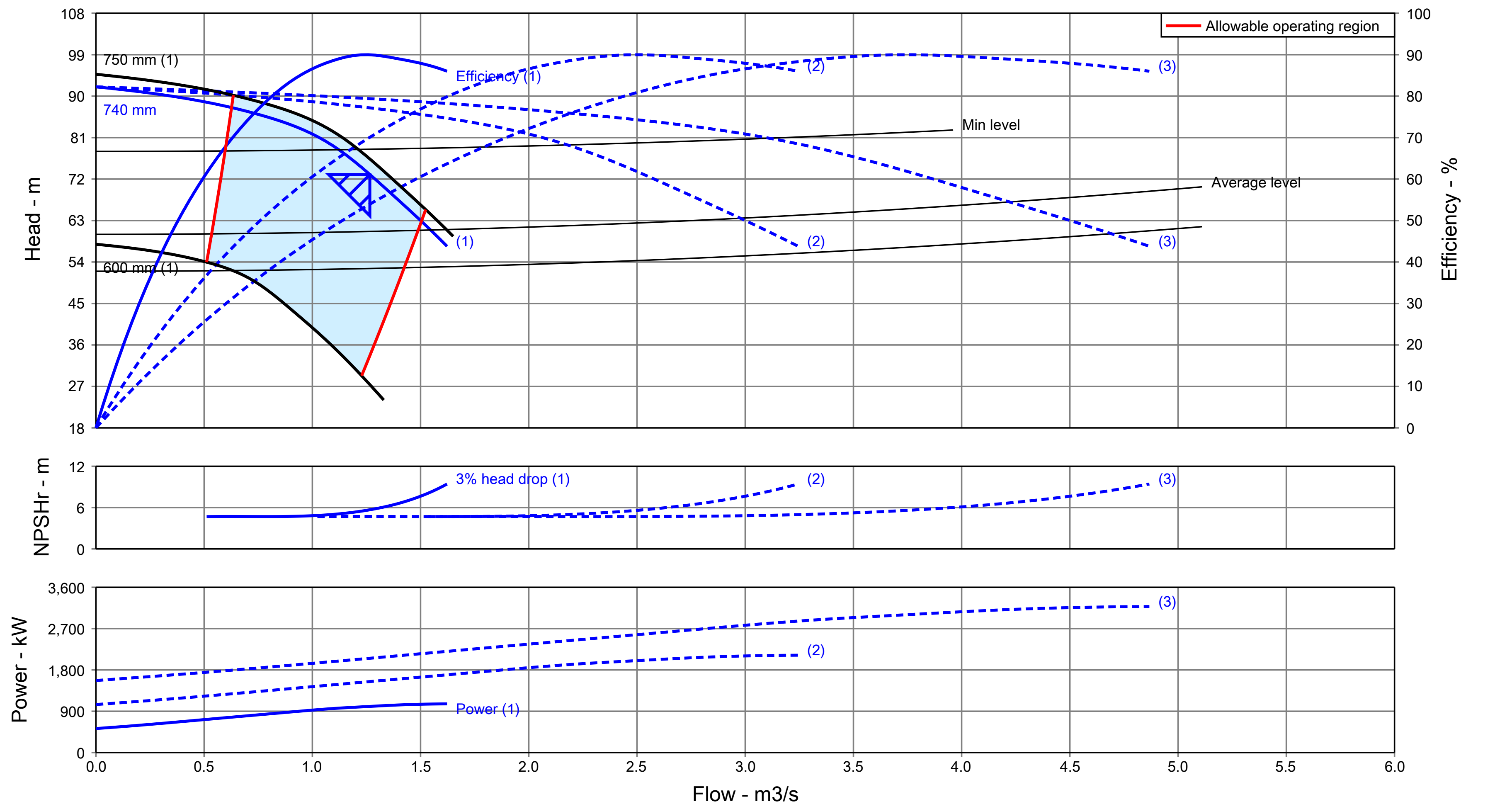


Hydraulic Performance Acceptance Test: ISO 9906:2012 / HI 14.6-2011 Grade 1B

Customer	:	Type / Size	: SMD 500-750 A
Customer reference	:	Stages	: 1
Item number	: 001	Speed, rated	: 990 rpm
Service	:	Based on curve number	: SMD-030.009-53-21-01
Quantity	: 1	Efficiency	: 89.98 %
Sulzer Reference ID	: 152066	Power, rated	: 1,005 kW
Date of Last Update	: 31 Oct 2024 6:34 AM	NPSH (3% head drop)	: 5.68 m
Flow, rated	: 1.27 m3/s	Temperature, rated / max	: 20.00 / 20.00 deg C
Differential head / pressure, rated	: 73.00 m	Fluid density, rated / max	: 0.998 / 0.998 kg/dm3
Liquid type	: Water	Viscosity, rated	: 1.00 cSt
Additional liquid description	:	Cq/Ch/Ce/Cn [ANSI/HI 9.6.7-2010]	: 1.00 / 1.00 / 1.00 / 1.00

SULZER

Pump Performance Curve



Hydraulic Performance Acceptance Test: ISO 9906:2012 / HI 14.6-2011 Grade 1B

Customer	:	Type / Size	: SMD 500-750 A	Flow, rated	: 1.27 m3/s
Customer reference	:	Stages	: 1	Differential head / pressure, rated	: 73.00 m
Item number	: 001	Speed, rated	: 990 rpm	Liquid type	: Water
Service	:	Based on curve number	: SMD-030.009-53-21-01	Additional liquid description	:
Quantity	: 1	Efficiency	: 89.98 %	Temperature, rated / max	: 20.00 / 20.00 deg C
Sulzer Reference ID	: 152066	Power, rated	: 1,005 kW	Fluid density, rated / max	: 0.998 / 0.998 kg/dm3
Date of Last Update	: 31 Oct 2024 6:34 AM	NPSH (3% head drop)	: 5.68 m	Viscosity, rated	: 1.00 cSt
				Cq/Ch/Ce/Cn [ANSI/HI 9.6.7-2010]	: 1.00 / 1.00 / 1.00 / 1.00

SULZER

Centrifugal Pump Datasheet

Customer:		Sulzer Reference ID:	152066
Project name:	Default	Inquiry Date:	
Customer reference:		Bid Submitted Date:	
Item number:	001	Date of Last Update:	31 Oct 2024 6:34 AM
Service:		Quantity:	1

Operating Conditions

Liquid type:	Water
Temperature, Rated / Max:	20.00 deg C / 20.00 deg C
Fluid density, rated / max:	0.998 kg/dm3 / 0.998 kg/dm3
Vapor pressure, rated:	0.02 bar.a
Viscosity, rated:	1.00 cSt
Consistency:	-
Air Content:	-
Discharge Flow, Rated:	1.27 m3/s
Differential Head, Rated / Actual:	73.00 m / 72.98 m
Suction pressure, rated / max:	0.00 bar.g / 0.00 bar.g
NPSH Available:	Ample

Performance

Performance Curve No.(s):	SMD-030.009-53-21-01
Pump speed:	990 rpm
Frequency:	50 Hz
Fixed / Variable Speed:	Constant Speed
Impeller diameter, rated:	740 mm
Impeller diameter, maximum:	750 mm
Impeller diameter, minimum:	600 mm
Efficiency:	89.98 %
NPSH (3% head drop):	5.68 m / 1.46 m
Ns / Nss:	1,579 US: Ns (imp. eye flow) / / 11,038 US: Ns (imp. eye flow)
Head, maximum, rated diameter:	92.02 m
Head rise to shutoff:	26.06 m
Flow, best eff. point:	1.25 m3/s
Diameter ratio (rated / max):	98.666667
Head ratio (rated dia / max dia):	95.58771
Viscous Coefficients (CQ / CH / CE):	1 / 1 / 1

Pressure Data

Maximum Working Pressure:	9.01 bar.g
Working Pressure Limit:	16.00 bar.g
Suction Pressure Limit:	2.00 bar.g
Hydrostatic Test Pressure (Suction/Discharge):	20.8
Suction pressure, rated / max:	0.00 bar.g / 0.00 bar.g
Discharge pressure, rated:	7.15 bar.g
Differential Pressure, Rated:	7.15 bar

Equipment Weights (Approximate)

Pump:	4,240.0 kg
Driver:	0.00 kg
Baseplate:	953.0 kg
Total Package	5,193.0 kg

Pump Design Data

Pump Type:	SMD, Horizontal double suction axially split single stage centri
Product Line:	SMD
Pump Size / No. of Stages:	SMD 500-750 A / 1
Rotation (viewed from drive end):	Clockwise
Impeller type:	Radial closed, double suction
Casing mounting:	Foot
Casing split:	Radial
Casing Type:	Double volute,between bearing

Nozzle	*Size*	*Rating*	*Face*	*Position*
Suction	DN600	PN16	RF	Side
Discharge	DN500	PN16	RF	Side

Lineshaft Diameter:	D150
Bearing Type, Radial:	-
Lineshaft Bearing / Bowl Bearing (Vert. pumps only):	-
Bearing Type, Thrust:	-
Bearing lubrication:	Grease
Baseplate type:	Separate baseplates under pump and motor

Materials

API Material Class:	-
Barrel / Can:	-
Case / Bowls:	Ductile iron (ASTM A395 Gr 60-40-18)
Column:	-
Discharge Head:	-
Impeller:	Duplex (ASTM A890 3A)
Case / Impeller Wear Rings:	Aluminium Bronze (SB 271) / Not included
Shaft:	Chromium steel (ASTM A276 Type 420)
Diffusers:	-

Shaft Sealing, Flush & Cooling Piping Plans

Seal Size / Type:	- / Mechanical seal, installed on shaft sleeve
Seal Code:	AES SAI-MAX (316L, SiC/ C, FKM)
Seal Manufacturer:	-
Seal Flush Piping, Primary:	Similar to Plan 11, Recirculation flushing fluid ; Includes 2 Flushing Plans per Pump
Seal Flush Piping, Secondary:	-
Cooling Water Piping:	N/A

Driver & Power Data

Driver Size:	1,120 kW
Volts/ Phase / Hz:	0 / 3 / 50 Hz
Service factor:	1
Power, rated:	1,005 kW
Power, maximum, rated diameter:	1,060 kW
Enclosure:	-

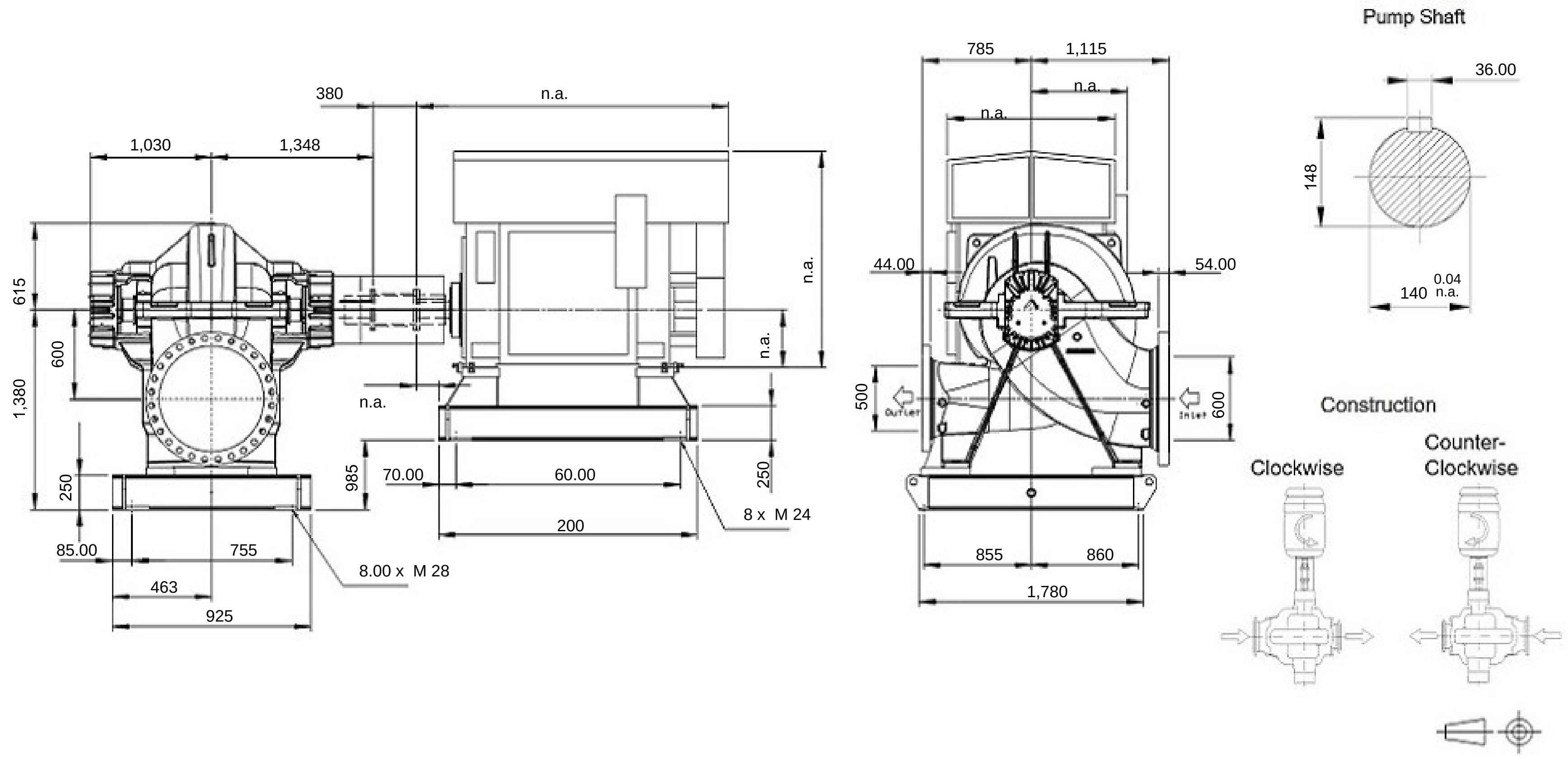
Accessories

Driver:	Special Driver - Type your text
Coupling:	1928

Comments

-

Dimensions in mm - NOTE: DO NOT USE FOR CONSTRUCTION UNLESS CERTIFIED



MOTOR INFORMATION

Power	1,120 kW
Frame size	1
Synchronous speed	1000 rpm
Voltage	0
Frequency	50 Hz
Enclosure	-

PUMP INFORMATION

Rated speed	990 rpm
Pump Size	SMD 500-750 A
No. of Stages	1

WEIGHTS

Pump	4,240.0 kg
Baseplate	953.0 kg
Motor	0.00 kg
TOTAL	5,193.0 kg

FLANGES

	Size	Drilling
Suction Flange	DN 600	EN PN16
Discharge Flange	DN 500	EN PN16

TENDER SUMMARY

Sulzer Reference ID	152066
Project name	Default
Customer	
Customer reference	
Item number	001
Date Last Saved	31 Oct 2024 6:34 AM
Service	

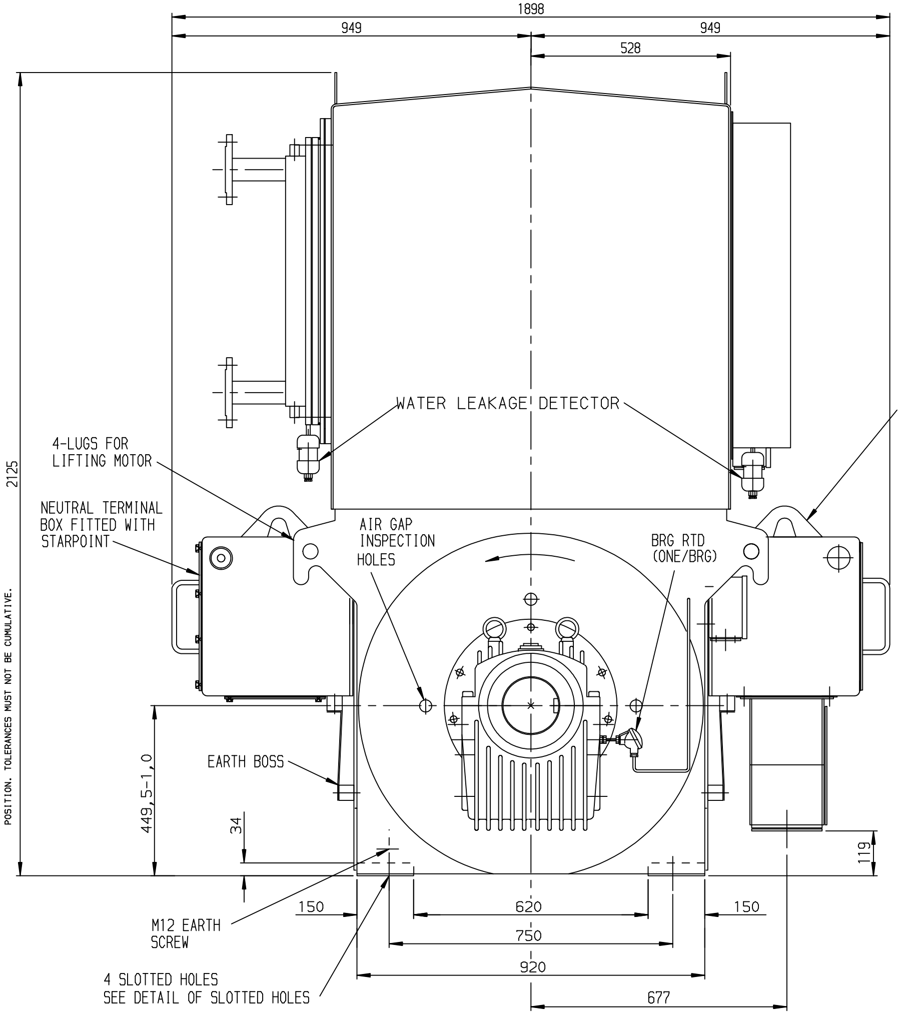
SULZER
General Arrangement Drawing

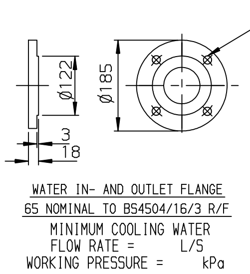
4502-2-614

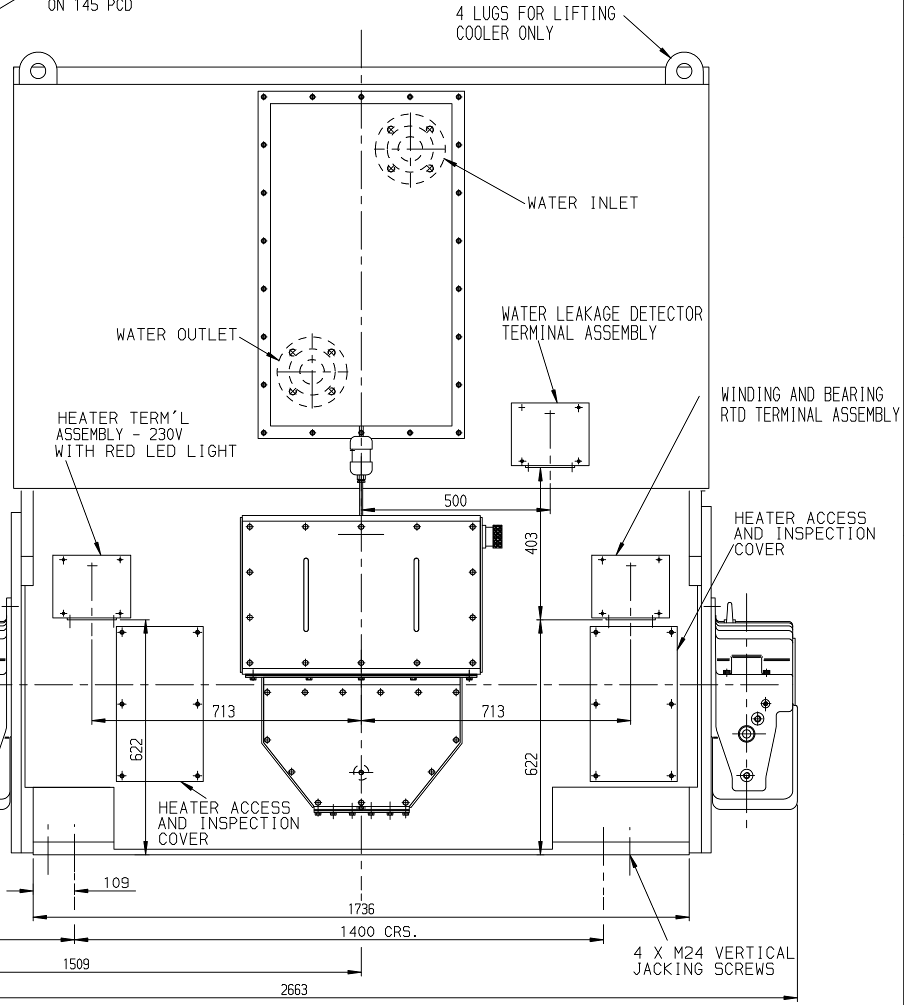
THIRD ANGLE PROJECTION. ALL DIMENSIONS IN MILLIMETRES(EXCEPT WHERE OTHERWISE STATED)
ALL WELD SYMBOLS ARE IN ACCORDANCE WITH SABS 044:PART 2-1963

IF IN DOUBT ASK

SURFACES MARKED ARE TO BE MACHINED AND ALLOWANCE MUST BE MADE FOR THIS MACHINING. SURFACE TEXTURE CLA VALUES SHOWN IN MICROMETERS THUS (MACHINED) (UNMACHINED). UNTOLERANCED MACHINING DIMENSIONS ARE TO BE WITHIN 0,1 OF TRUE SIZE. UNTOLERANCED FABRICATING DIMENSIONS ARE TO BE WITHIN ±3,0 OF TRUE SIZE. HOLE CENTRES ARE TO BE WITHIN ±0,4 OF TRUE POSITION. TOLERANCES MUST NOT BE CUMULATIVE.







kW	-	r/min	-	AMPS.	-	VOLTS	-	PH	3	Hz	50

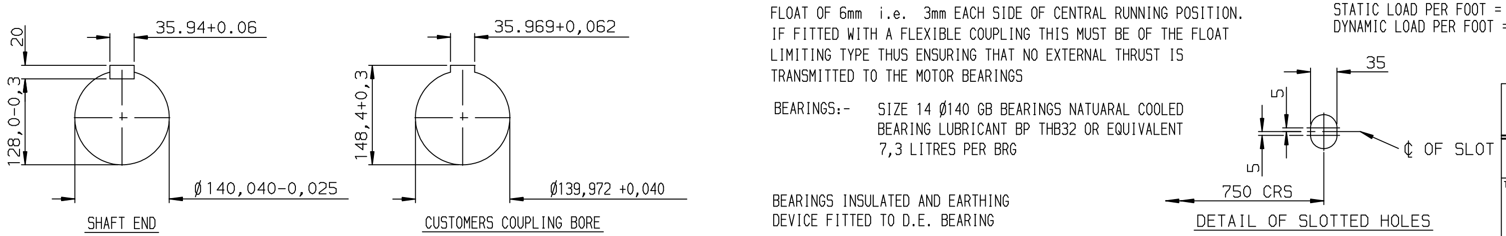
THIS MOTOR IS FITTED WITH BEARINGS ALLOWING A SELF-CONTAINED FLOAT OF 6mm i.e. 3mm EACH SIDE OF CENTRAL RUNNING POSITION. IF FITTED WITH A FLEXIBLE COUPLING THIS MUST BE OF THE FLOAT LIMITING TYPE THUS ENSURING THAT NO EXTERNAL THRUST IS TRANSMITTED TO THE MOTOR BEARINGS

BEARINGS:- SIZE 14 Ø140 GB BEARINGS NATUARAL COOLED
BEARING LUBRICANT BP THB32 OR EQUIVALENT
7,3 LITRES PER BRG

BEARINGS INSULATED AND EARTHING DEVICE FITTED TO D.E. BEARING

APPROX. MOTOR MASS = 6200 Kg
STATIC LOAD PER FOOT = 15206 N
DYNAMIC LOAD PER FOOT = 15206± N

ACTOM
LARGE MOTORS
This drawing is supplied
FOR TENDER PURPOSES ONLY



REV	DATE	CH'D	APP'D	DESCRIPTION
0 (ORIG ISSUE)	02-11-2011			
1	02-02-2012	OJN	PVW	VOLTS WAS 11000. SPACER BOX REMOVED. DIM'S 1278 & 2125 WAS 1228 & 2075. CN 31651
2	07-02-2012	OJN	MA / PVW	DIM'S 1278 & 2125 WAS 1228 & 2075. CN.31665
3	17-05-2012	PRM	OJN / PVW	LED LIGHT ADDED ON HEATER TERM'L ASSY. CN.31831.

PVW

ACTOM
LARGE MOTORS

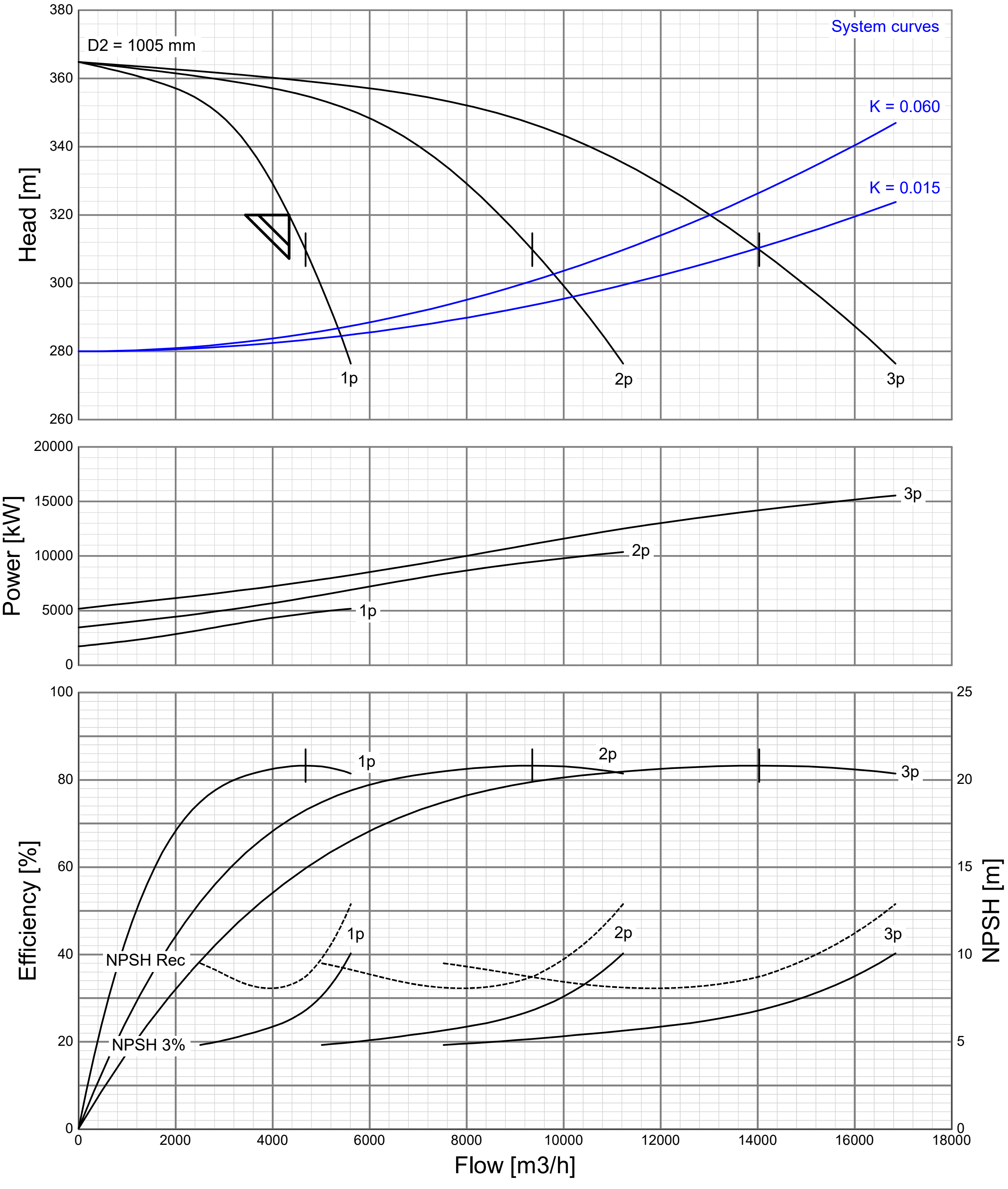
TITLE: OUTLINE FOR UL450/140

REFERENCE	DRAWING NUMBER	REV	DR'N	OJN
4502-2-420	4502-2-614	3	TR'D	
			CH'D	PVW
ORIGL SHEET A2	FIRST ORDER B005184	SCALE 1 : 10	APR'D	PVW

SULZER

Pump type and plant:	**HPDM 450-1000-d+s/26 Xhariep Pipeline Project - 3+1**

Pump Medium:	Water	Shaft speed:	990 rpm	Flow	= 4339.2 m3/h
Density:	998.2 kg/m3	Stage Number:	2	Head	= 320.0 m
Temperature:	20.0 °C	nq:	26	Efficiency	= 83.1 %
kinem. Viscosity:	1.003 mm2/s	Basis:	BPC_026_068	Power	= 4547 kW
				NPSH-3%	= 6.2 m

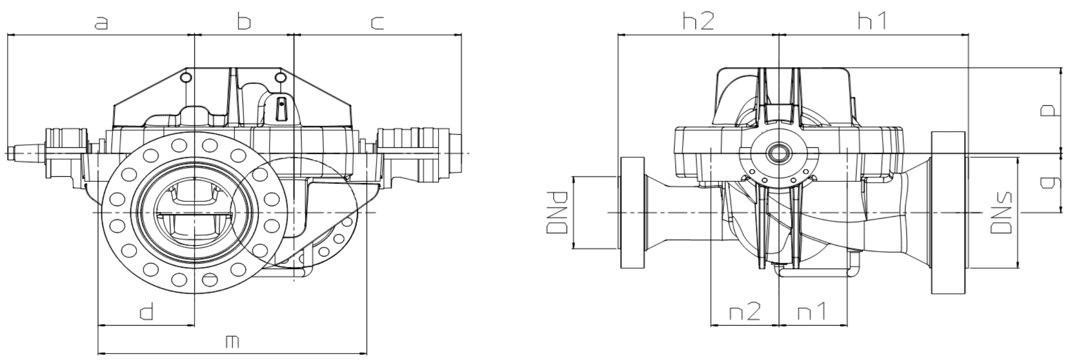


Revision:	-	Sulzer Reference ID	**Doc. No:**	**507271344EN**
Name/Date:	MuO 07.11.2024	ZAF.6087-CHN.24.8279-B0		

SULZER

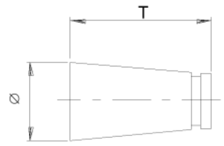
HPDM d+s
OUTLINE DRAWING & PUMP WEIGHT

Customer:	**Zutari**	Pump Type:	**HPDM-450-1000-d+s-26**
Project:	**Xhariep Pipeline Project**		
Sulzer Ref. No.:	**ZAF.6087-CHN.24.8279-B0**	Doc. / Rev. No.:	**507271347EN / -**
AEAR No.:	**AEAR-23617**	Name / Date:	**MuO / 13.11.2024**



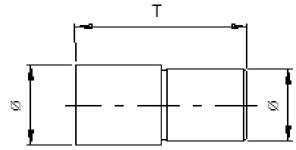
PUMP SHAFT END :

TAPERED
(SHRINK FIT)

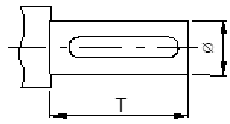


Taper: 1:20

CYLINDRICAL STEPPED
(SHRINK FIT)



CYLINDRICAL
PARALLEL FIT WITH
KEY



All dimensions are preliminary (mm) and are not to be used for site construction

Flanges according to DIN or ANSI		
Suction	DN_S	550
	Flange standard:	Class/PN
Delivery	DN_D	450
	Flange standard:	Class/PN
Rotor --> Mass moment of interia (with water)		[kgm^2]

Dimensions [mm]			
a	2'200	T	240
b	900	h1	1'600
c	1'600	h2	1'400
d	780	n1	900
m	2'200	n2	900
∅	168	g	880
		p	1'400

Bearing type

Preliminary bearing selection:	RL 6 / RAL 6-17

Weights of final machined parts

Casing weight	10'900	kg
Upper casing weight	3'500	kg
Lower casing weight	7'410	kg
Rotor weight	2'695	kg
Shaft weight	1'450	kg
Suction impeller weight	500	kg
Series impeller weight	500	kg
Bearing units	750	kg
Total pump weight	15'450	kg

Casing material

Option 1.1 - A 216 Gr - WCB

Design data

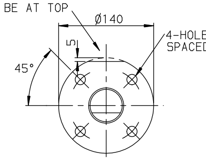
Design suction pressure	2.0	bar.g
Design discharge pressure	40.0	bar.g
Design test pressure	60.0	bar.g
Labyrinth clearance type	API	
Split flange sealing method	o-ring cord	

8002-1-213

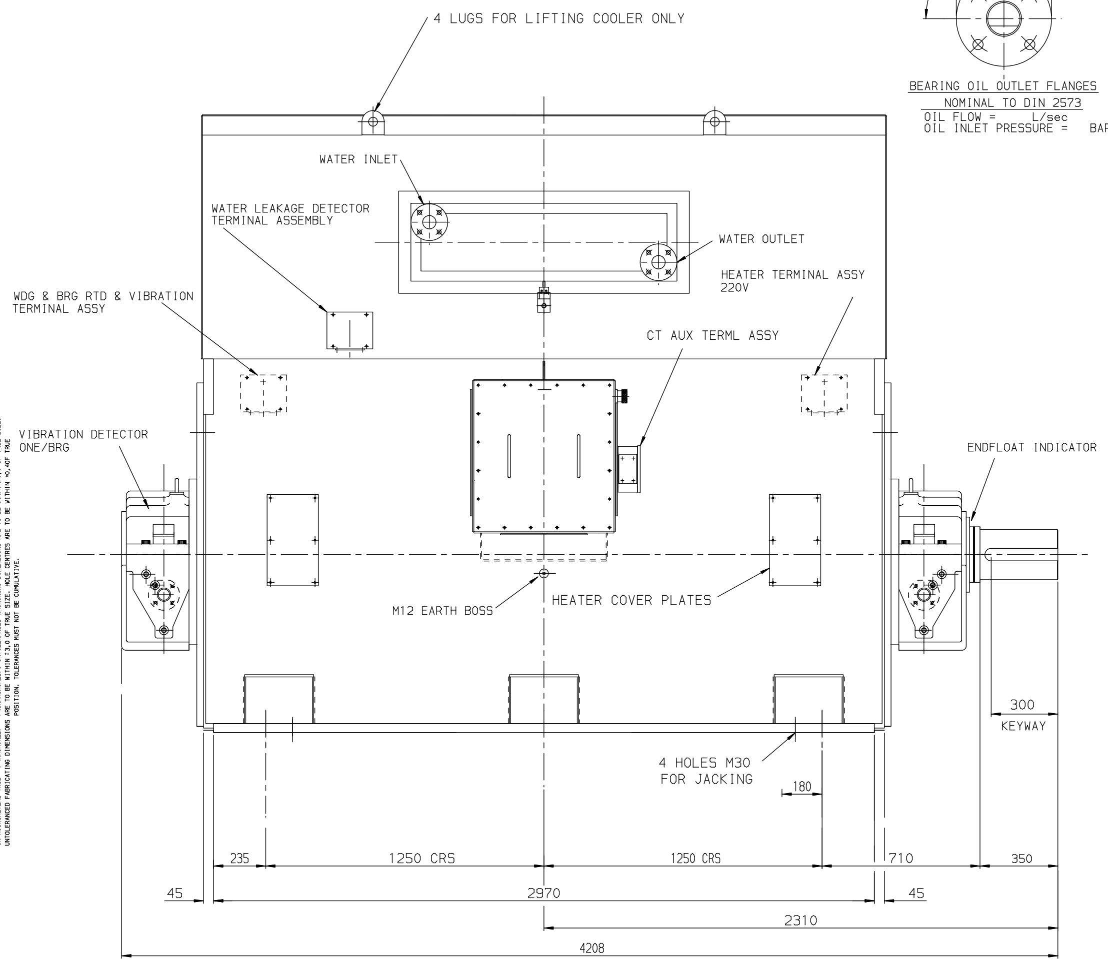
THIRD ANGLE PROJECTION. ALL DIMENSIONS IN MILLIMETRES (EXCEPT WHERE OTHERWISE STATED)
ALL WELD SYMBOLS ARE IN ACCORDANCE WITH SABS 044:PART 2-1963

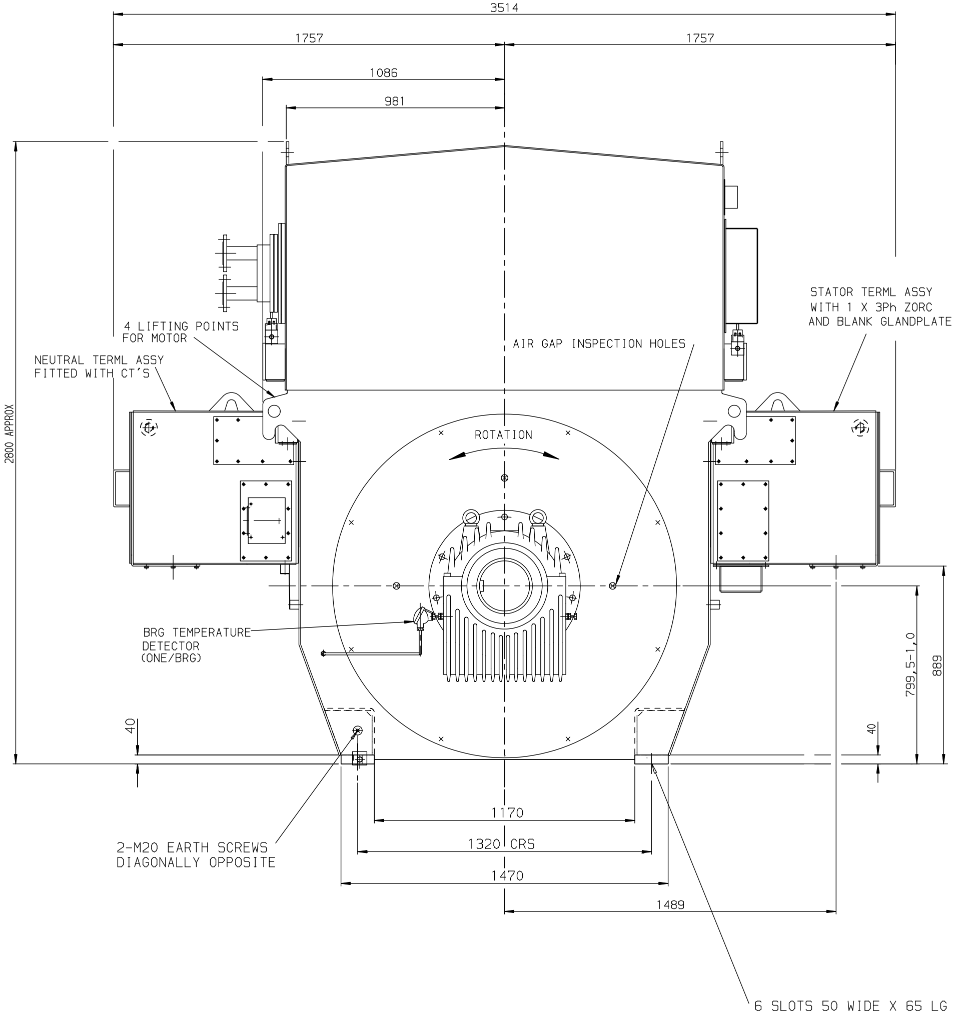
IF IN DOUBT ASK

NOTE:-
USE SPREADER BARS TO LIFT MOTOR OR REMOVE COOLER BEFORE LIFTING.

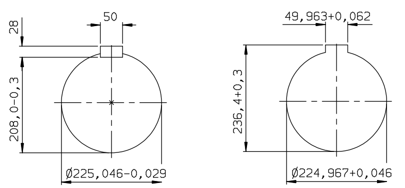


BEARING OIL OUTLET FLANGES
NOMINAL TO DIN 2573
OIL FLOW = L/sec
OIL INLET PRESSURE = BAR





SURFACES MARKED ✓ ARE TO BE MACHINED AND ALLOWANCE MUST BE MADE FOR THIS MACHINING. SURFACE TEXTURE CLA VALUES SHOWN IN MICROMETRES THUS ✓ (MACHINED). UNTOLERANCED MACHINING DIMENSIONS ARE TO BE WITHIN ±1,0 OF TRUE SIZE. UNTOLERANCED FABRICATING DIMENSIONS ARE TO BE WITHIN ±3,0 OF TRUE SIZE. HOLE CENTRES ARE TO BE WITHIN ±0,4 OF TRUE POSITION. TOLERANCES MUST NOT BE CUMULATIVE.



SHAFT EXTENSION

COUPLING BORE

MOTOR MASS = 20000 Kg APPROX.
STATIC LOAD AT EACH PAD= 32700 N
DYNAMIC LOAD AT EACH PAD= 32700 ± N

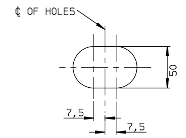
BEARINGS:- SIZE 22 Ø225 GB BEARINGS INSULATED WITH WATER COOLING
BEARING LUBRICANT BP THB32 OR EQUIVALENT
21 LITRES PER BRG

BEARINGS INSULATED AND EARTHING DEVICE FITTED TO D.E. BEARING

THIS MOTOR IS FITTED WITH BEARINGS ALLOWING A SELF-CONTAINED FLOAT OF 6 MM I.E. 3 MM EATHER SIDE OF CENTRAL RUNNING POSITION. IF FITTED WITH A FLEXIBLE COUPLING THIS MUST BE OF THE FLOAT-LIMITING TYPE, THUS ENSURING THAT NO EXTERNAL THRUST IS TRANSMITTED TO THE MOTOR BEARINGS.

RADIATOR WATER FLOW = L/sec

KW		VOLTS	11000	PH	3	HZ	50	RPM		AMPS	



DETAIL OF SLOTTED HOLES

ACTOM
LARGE MOTORS
This drawing is supplied
FOR TENDER PURPOSES ONLY

ACTOM
LARGE MOTORS

TITLE: OUTLINE FOR UL800/250

REFERENCE	DRAWING NUMBER	REV	DR'N	OJN
	8002-1-213	0	TR'D	
			CH'D	P>W
ORIGL SHEET A1	FIRST ORDER QD35572K	SCALE 1 : 12	APR'D	P>W

ORIG ISSUE 0
DATE 07-11-2024

Pump Performance Datasheet

Customer	:	Sulzer Reference ID	: 152066
Customer reference	:	Type / Size	: SMD 600-1250 B
Item number	: 001	Stages	: 1
Service	:	Based on curve number	: SMD-018.014-53-21-01
Quantity	: 1	Date of Last Update	: 05 Nov 2024 11:12 AM

Operating Conditions

Flow, rated	: 1.21 m3/s
Differential head / pressure, rated (requested)	: 127.0 m
Suction pressure, rated / max	: 0.00 / 0.00 bar.g
NPSH available, rated	: Ample
Site Supply Frequency	: 50 Hz

Performance

Speed criteria	: Synchronous
Speed, rated	: 743 rpm
Impeller diameter, rated	: 1,221 mm
Impeller diameter, maximum	: 1,250 mm
Impeller diameter, minimum	: 1,000 mm
Efficiency	: 83.70 %
NPSH (3% head drop) / margin required	: 3.50 / 1.00 m
NPSHr (0% head drop) required	: -
Ns (imp. eye flow) / Nss (imp. eye flow)	: 940 / 11,643 US Units
MCSF	: 0.76 m3/s
Head, maximum, rated diameter	: 149.7 m
Head rise to shutoff	: 17.90 %
Flow, best eff. point	: 1.50 m3/s
Flow ratio, rated / BEP	: 80.26 %
Diameter ratio (rated / max)	: 97.68 %
Head ratio (rated dia / max dia)	: 94.69 %
Cq/Ch/Ce/Cn [ANSI/HI 9.6.7-2010]	: 1.00 / 1.00 / 1.00 / 1.00
Selection status	: Acceptable

Liquid

Liquid type	: Water
Additional liquid description	:
Solids diameter, max	: 0.00 mm
Solids concentration, by volume	: 0.00 %
Temperature, rated / max	: 20.00 / 20.00 deg C
Fluid density, rated / max	: 0.998 / 0.998 kg/dm3
Viscosity, rated	: 1.00 cSt
Vapor pressure, rated	: 0.02 bar.a

Material

Material selected	: Ductile Iron

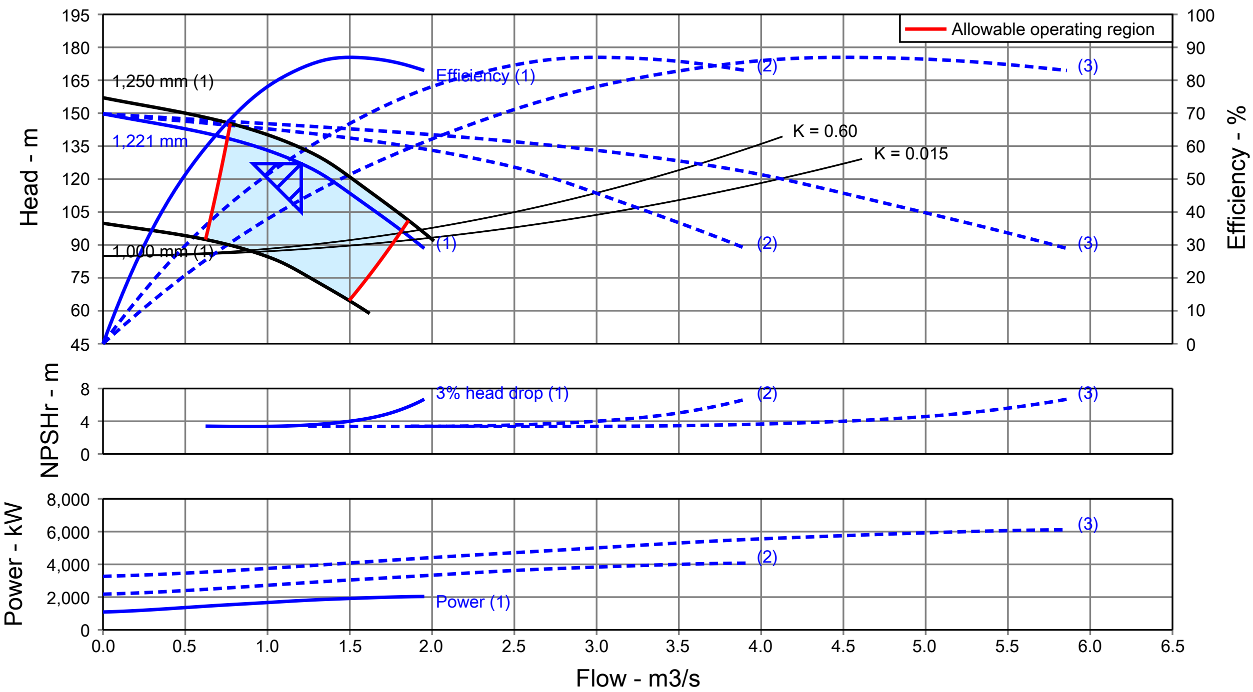
Pressure Data

Maximum casing/bowl working pressure	: 14.66 bar.g
Maximum allowable working pressure	: 31.00 bar.g
Maximum allowable suction pressure	: 2.00 bar.g
Hydrostatic test pressure	: 40.30 bar.g

Driver & Power Data (@Max density)

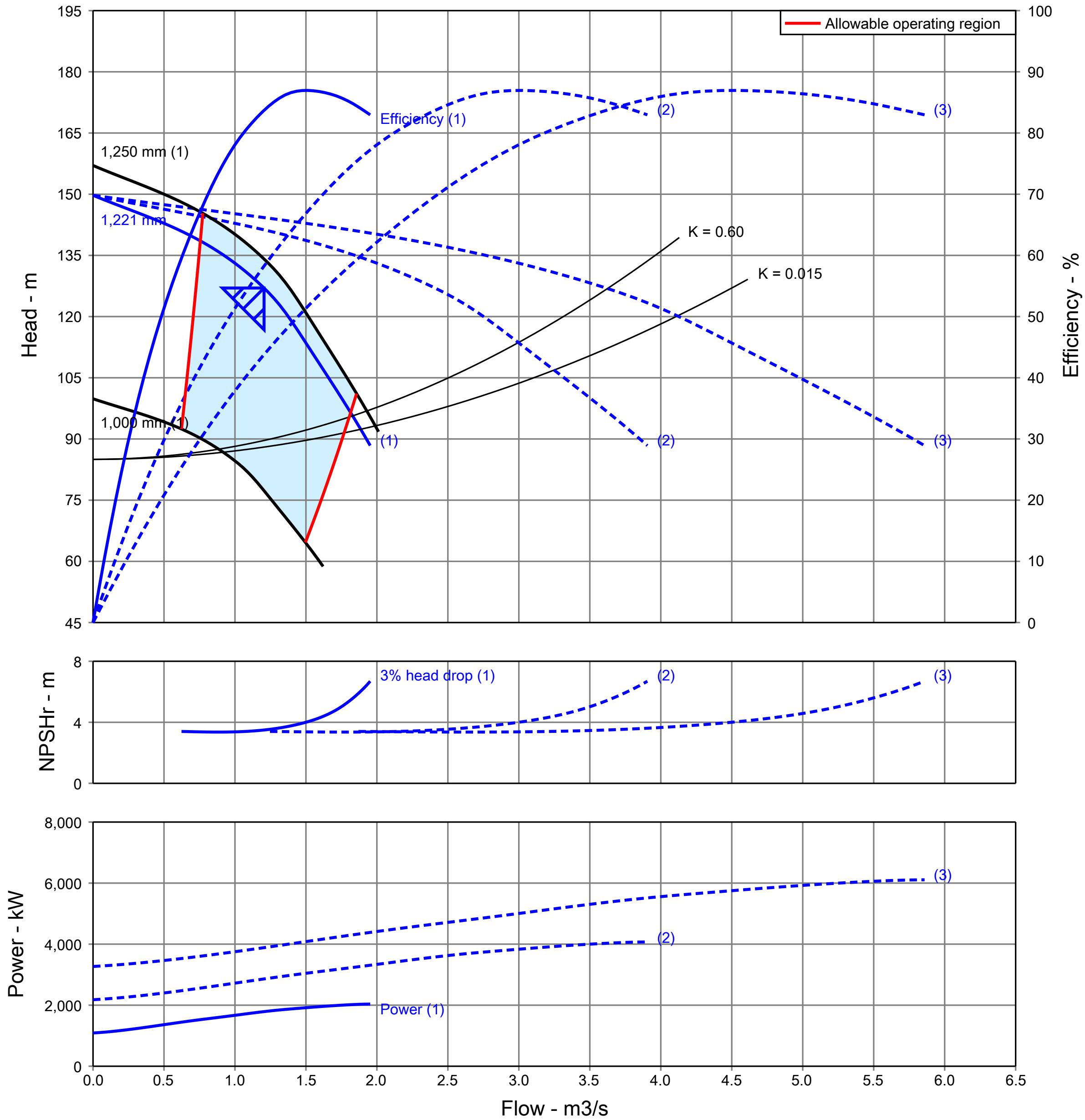
Driver sizing specification	: Rated power
Margin over specification	: 0.00 %
Service factor	: 1.00
Power, hydraulic	: 1,498 kW
Power, rated	: 1,790 kW
Power, maximum, rated diameter	: 2,036 kW
Minimum recommended motor rating	: 1,800 kW / 2,414 hp

Hydraulic Performance Acceptance Test: ISO 9906:2012 / HI 14.6-2011 Grade 1B



SULZER

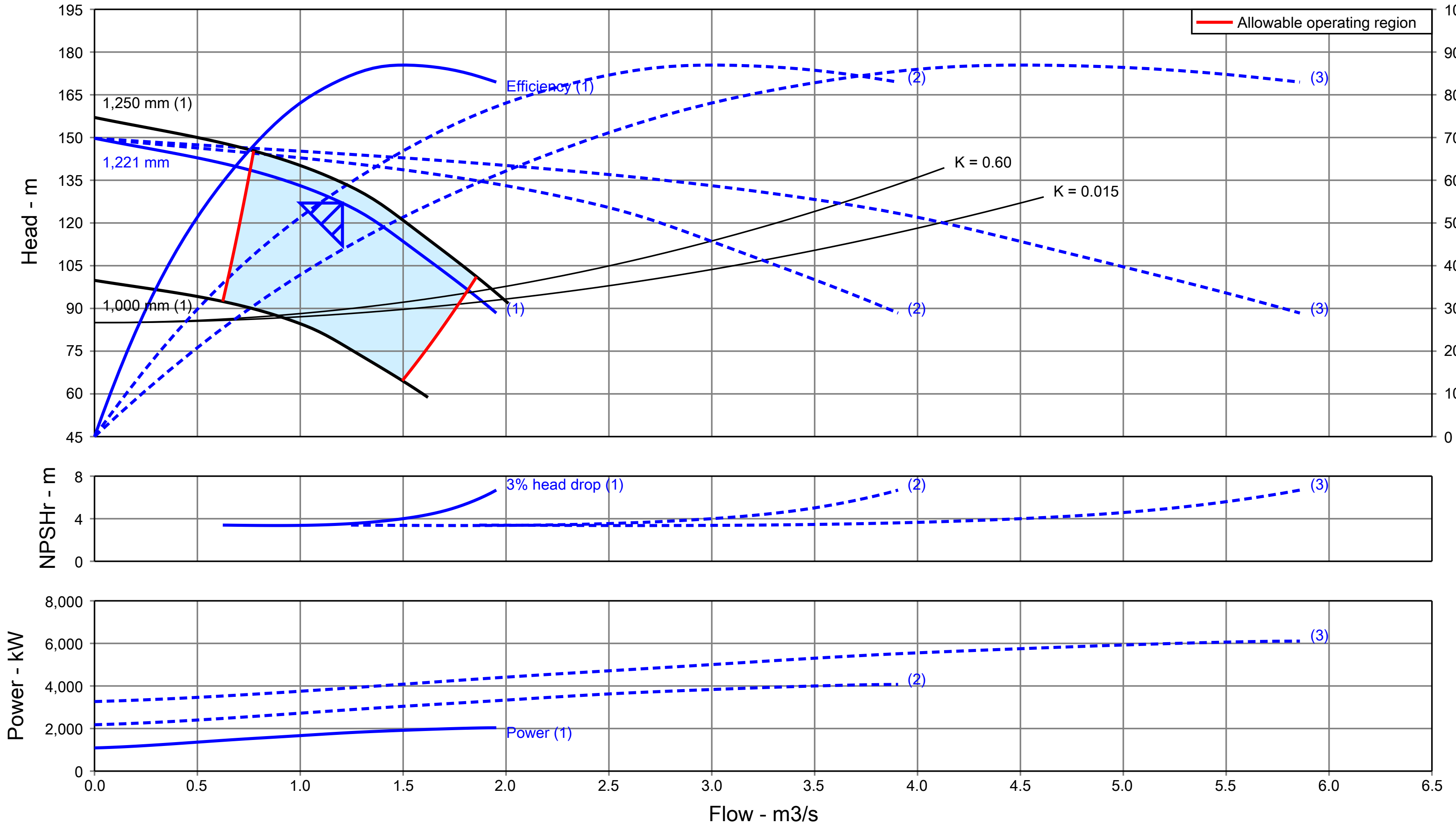
Pump Performance Curve



Hydraulic Performance Acceptance Test: ISO 9906:2012 / HI 14.6-2011 Grade 1B

Customer	:	Type / Size	: SMD 600-1250 B
Customer reference	:	Stages	: 1
Item number	: 001	Speed, rated	: 743 rpm
Service	:	Based on curve number	: SMD-018.014-53-21-01
Quantity	: 1	Efficiency	: 83.70 %
Sulzer Reference ID	: 152066	Power, rated	: 1,790 kW
Date of Last Update	: 05 Nov 2024 11:12 AM	NPSH (3% head drop)	: 3.50 m
Flow, rated	: 1.21 m3/s	Temperature, rated / max	: 20.00 / 20.00 deg C
Differential head / pressure, rated	: 127.0 m	Fluid density, rated / max	: 0.998 / 0.998 kg/dm3
Liquid type	: Water	Viscosity, rated	: 1.00 cSt
Additional liquid description	:	Cq/Ch/Ce/Cn [ANSI/HI 9.6.7-2010]	: 1.00 / 1.00 / 1.00 / 1.00

Pump Performance Curve

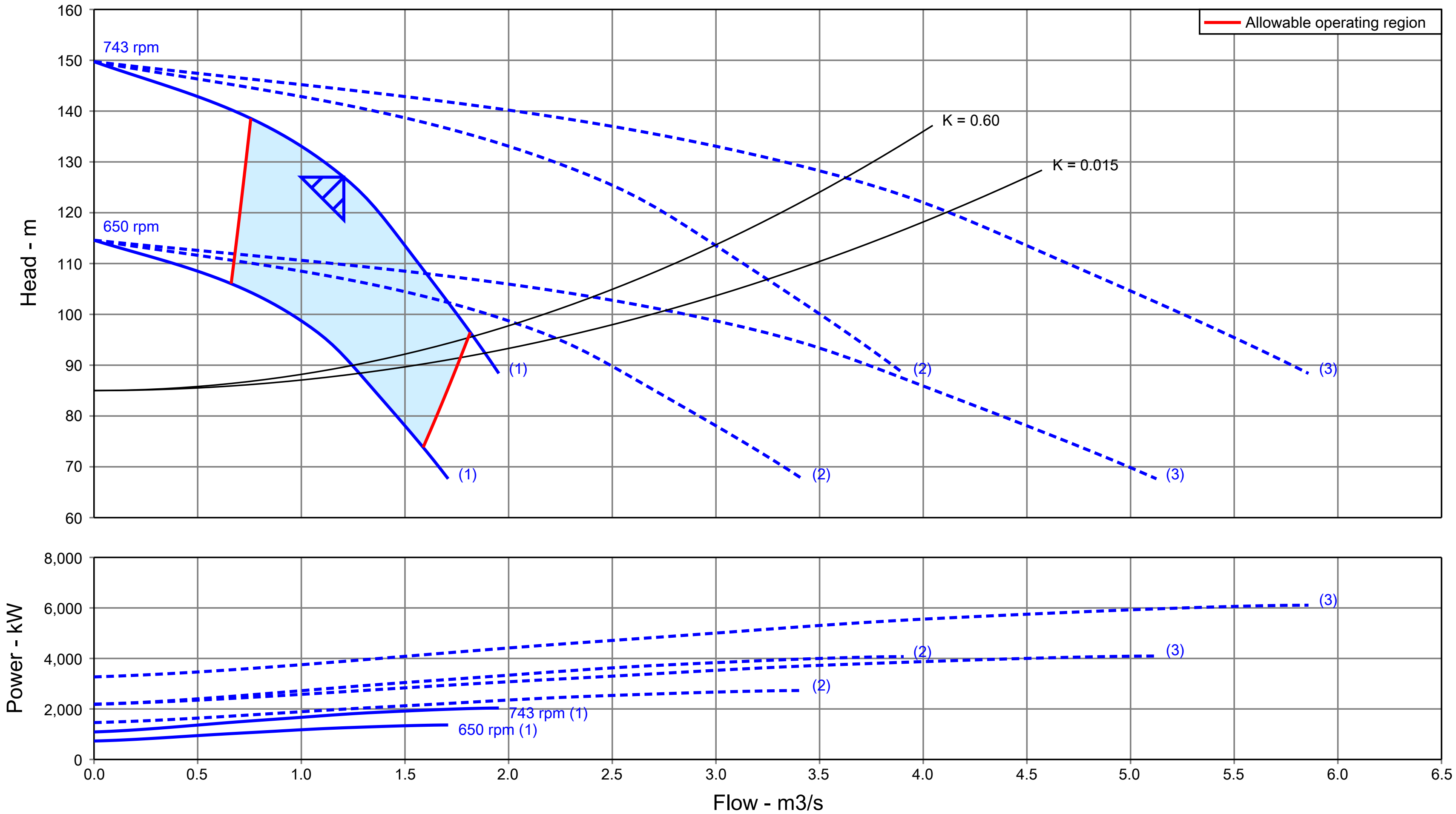


Hydraulic Performance Acceptance Test: ISO 9906:2012 / HI 14.6-2011 Grade 1B

Customer	:	Type / Size	: SMD 600-1250 B	Flow, rated	: 1.21 m3/s
Customer reference	:	Stages	: 1	Differential head / pressure, rated	: 127.0 m
Item number	: 001	Speed, rated	: 743 rpm	Liquid type	: Water
Service	:	Based on curve number	: SMD-018.014-53-21-01	Additional liquid description	:
Quantity	: 1	Efficiency	: 83.70 %	Temperature, rated / max	: 20.00 / 20.00 deg C
Sulzer Reference ID	: 152066	Power, rated	: 1,790 kW	Fluid density, rated / max	: 0.998 / 0.998 kg/dm3
Date of Last Update	: 05 Nov 2024 11:12 AM	NPSH (3% head drop)	: 3.50 m	Viscosity, rated	: 1.00 cSt
				Cq/Ch/Ce/Cn [ANSI/HI 9.6.7-2010]	: 1.00 / 1.00 / 1.00 / 1.00

SULZER

Multi-Speed Performance Curve



Customer	:	Stages	: 1	Nominal speed	: 745 rpm
Customer reference	:	Based on curve number	: SMD-018.014-53-21-01	Flow, rated	: 1.21 m3/s
Item number	: 001	Efficiency	: 83.70 %	Differential head / pressure, rated	: 127.0 m
Service	:	Power, rated	: 1,790 kW	Speed, rated	: 743 rpm
Quantity	: 1	NPSH (3% head drop)	: 3.50 m	Impeller diameter, rated	: 1,221 mm
Sulzer Reference ID	: 152066	Site Supply Frequency	: 50 Hz	Fluid density, rated / max	: 0.998 / 0.998 kg/dm3
Date of Last Update	: 05 Nov 2024 11:12 AM			Viscosity, rated	: 1.00 cSt
Type / Size	: SMD 600-1250 B			Cq/Ch/Ce/Cn [ANSI/HI 9.6.7-2010]	: 1.00 / 1.00 / 1.00 / 1.00

Centrifugal Pump Datasheet

Customer:		Sulzer Reference ID:	152066
Project name:	Default	Inquiry Date:	
Customer reference:		Bid Submitted Date:	
Item number:	001	Date of Last Update:	05 Nov 2024 11:12 AM
Service:		Quantity:	1

Operating Conditions		Pump Design Data	
Liquid type:	Water	Pump Type:	SMD, Horizontal double suction axially split single stage centri
Temperature, Rated / Max:	20.00 deg C / 20.00 deg C	Product Line:	SMD
Fluid density, rated / max:	0.998 kg/dm3 / 0.998 kg/dm3	Pump Size / No. of Stages:	SMD 600-1250 B / 1
Vapor pressure, rated:	0.02 bar.a	Rotation (viewed from drive end):	Clockwise
Viscosity, rated:	1.00 cSt	Impeller type:	Radial closed, double suction
Consistency:	-	Casing mounting:	Foot
Air Content:	-	Casing split:	Radial
Discharge Flow, Rated:	1.21 m3/s	Casing Type:	Double volute,between bearing
Differential Head, Rated / Actual:	127.0 m / 127.0 m		
Suction pressure, rated / max:	0.00 bar.g / 0.00 bar.g		
NPSH Available:	Ample		

Nozzle	*Size*	*Rating*	*Face*	*Position*
Suction	DN800	EN PN40	RF	Side
Discharge	DN600	EN PN40	RF	Side

Performance			
Performance Curve No.(s):	SMD-018.014-53-21-01	Lineshaft Diameter:	D180
Pump speed:	743 rpm	Bearing Type, Radial:	-
Frequency:	50 Hz	Lineshaft Bearing / Bowl Bearing (Vert. pumps only):	-
Fixed / Variable Speed:	Constant Speed	Bearing Type, Thrust:	-
Impeller diameter, rated:	1,221 mm	Bearing lubrication:	Grease
Impeller diameter, maximum:	1,250 mm	Baseplate type:	Separate baseplates under pump and motor
Impeller diameter, minimum:	1,000 mm		

		Materials	
Efficiency:	83.70 %	API Material Class:	-
NPSH (3% head drop):	3.50 m / 1.00 m	Barrel / Can:	-
Ns / Nss:	940 US: Ns (imp. eye flow) / N / 11,643 US: Ns (imp. eye flow)	Case / Bowls:	Ductile iron (ASTM A395 Gr 60-40-18)
Head, maximum, rated diameter:	149.7 m	Column:	-
Head rise to shutoff:	17.90 m	Discharge Head:	-
Flow, best eff. point:	1.50 m3/s	Impeller:	Duplex (ASTM A890 3A)
Diameter ratio (rated / max):	97.68	Case / Impeller Wear Rings:	Aluminium Bronze (SB 271) / Not included
Head ratio (rated dia / max dia):	94.687727	Shaft:	Chromium steel (ASTM A276 Type 420)
Viscous Coefficients (CQ / CH / CE):	1 / 1 / 1	Diffusers:	-

Pressure Data		Shaft Sealing, Flush & Cooling Piping Plans	
Maximum Working Pressure:	14.66 bar.g	Seal Size / Type:	- / Mechanical seal, installed on shaft sleeve
Working Pressure Limit:	31.00 bar.g	Seal Code:	Burgmann H75S2 (316Ti, SiC-Si / C, EPDM)
Suction Pressure Limit:	2.00 bar.g	Seal Manufacturer:	-
Hydrostatic Test Pressure (Suction/Discharge):	40.3	Seal Flush Piping, Primary:	Similar to Plan 11, Recirculation flushing fluid ; Includes 2 Flushing Plans per Pump
Suction pressure, rated / max:	0.00 bar.g / 0.00 bar.g	Seal Flush Piping, Secondary:	-
Discharge pressure, rated:	12.43 bar.g	Cooling Water Piping:	N/A
Differential Pressure, Rated:	12.43 bar		

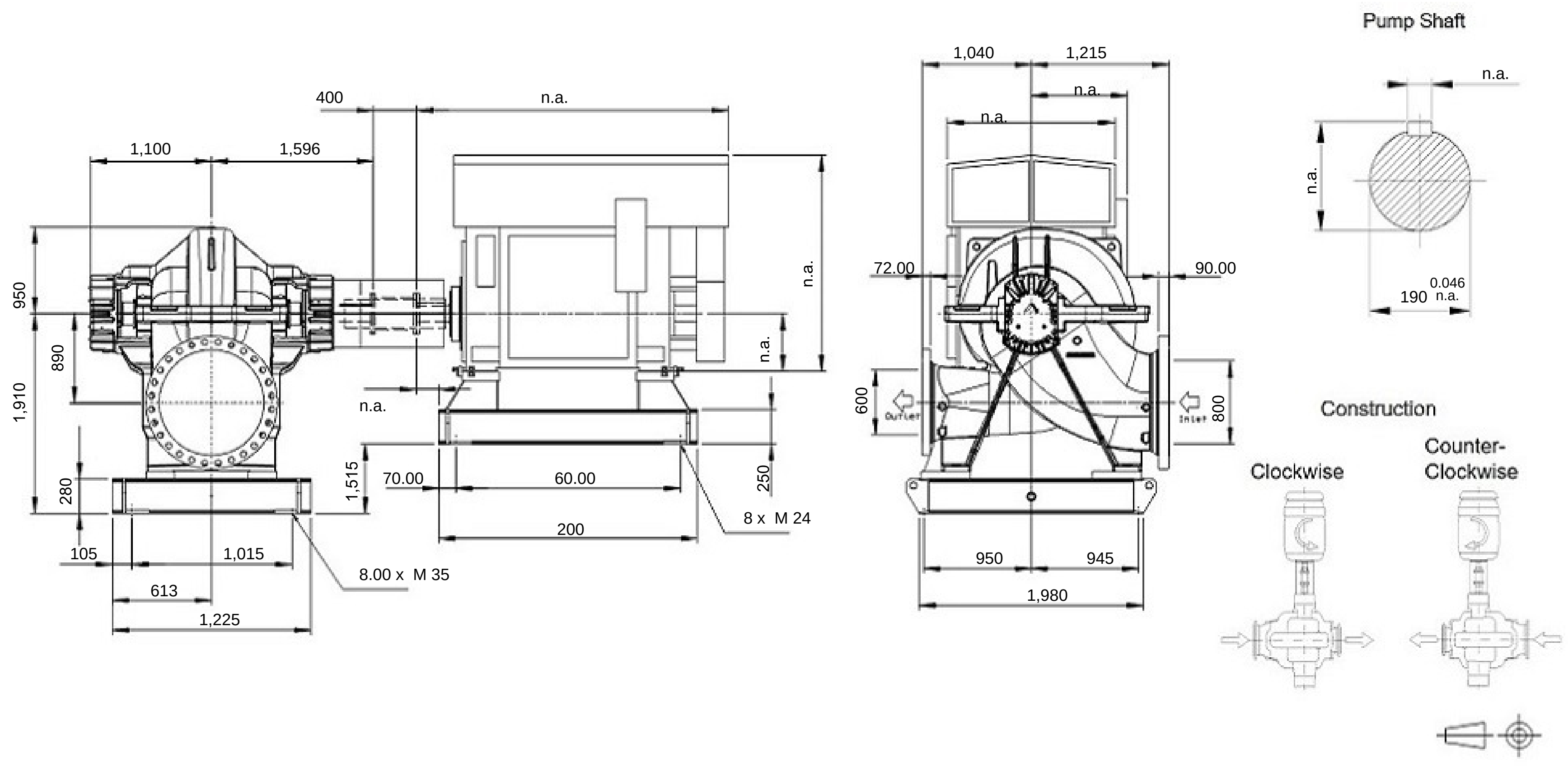
Equipment Weights (Approximate)		Driver & Power Data	
Pump:	10,400.0 kg	Driver Size:	1,800 kW
Driver:	0.00 kg	Volts/ Phase / Hz:	0 / 3 / 50 Hz
Baseplate:	1,376.0 kg	Service factor:	1
Total Package	11,776.0 kg	Power, rated:	1,790 kW
		Power, maximum, rated diameter:	2,036 kW
		Enclosure:	-

Accessories	
Driver:	Special Driver - Type your text
Coupling:	2468

Comments

-

Dimensions in mm - NOTE: DO NOT USE FOR CONSTRUCTION UNLESS CERTIFIED



MOTOR INFORMATION

Power	1,800 kW
Frame size	1
Synchronous speed	750 rpm
Voltage	0
Frequency	50 Hz
Enclosure	-

PUMP INFORMATION

Rated speed	743 rpm
Pump Size	SMD 600-1250 B
No. of Stages	1

WEIGHTS

Pump	10,400.0 kg
Baseplate	1,376.0 kg
Motor	0.00 kg
TOTAL	11,776.0 kg

FLANGES

	Size	Drilling
Suction Flange	DN 800	EN PN40
Discharge Flange	DN 600	EN PN40

TENDER SUMMARY

Sulzer Reference ID	152066
Project name	Default
Customer	
Customer reference	
Item number	001
Date Last Saved	05 Nov 2024 11:12 AM
Service	

SULZER
General Arrangement Drawing

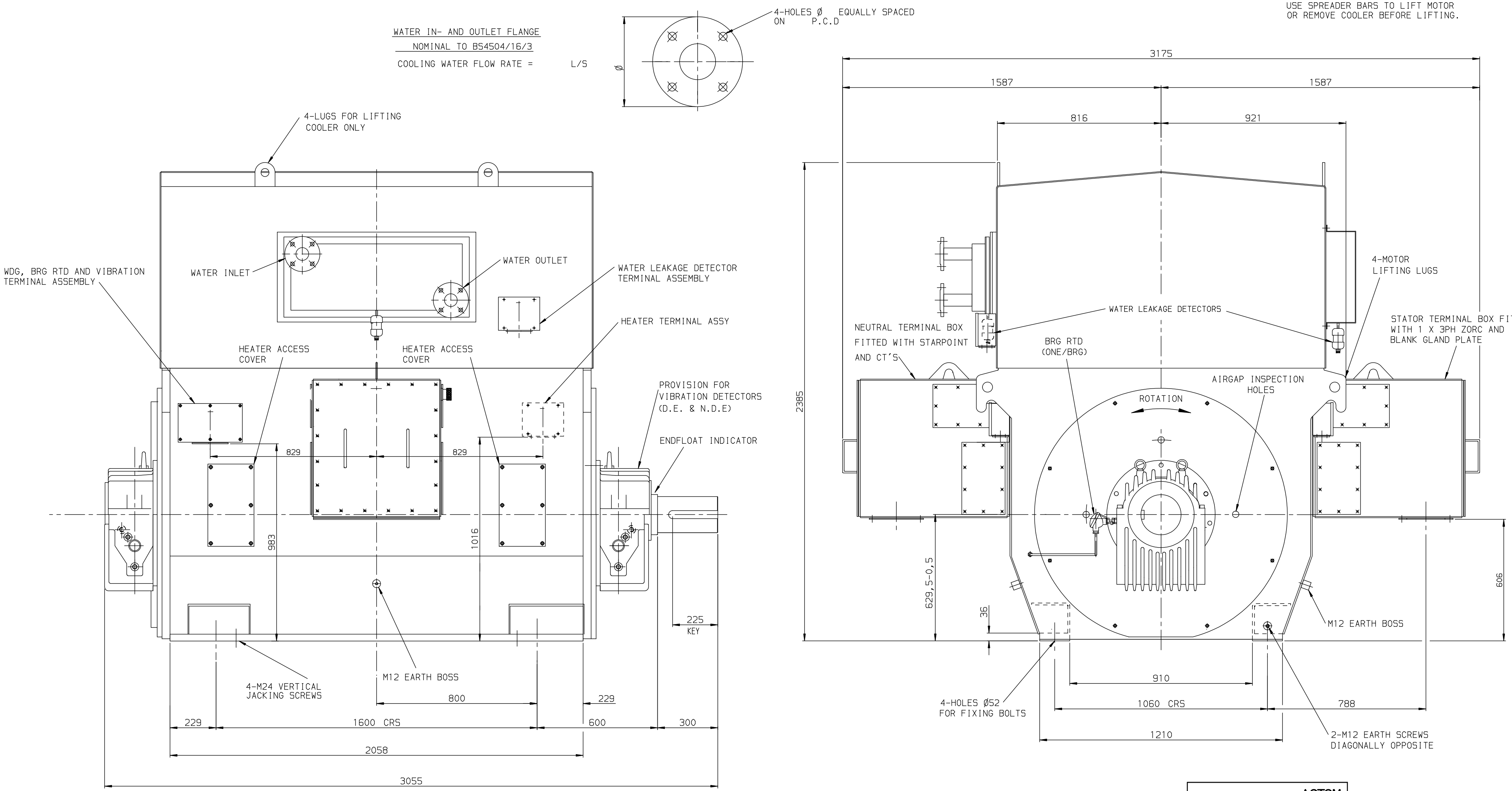
6302-1-378

THIRD ANGLE PROJECTION. ALL DIMENSIONS IN MILLIMETRES (EXCEPT WHERE OTHERWISE STATED)
ALL WELD SYMBOLS ARE IN ACCORDANCE WITH SABS 044;PART 2-1963

IF IN DOUBT ASK

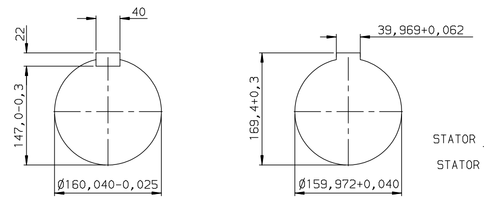
NOTE:-
USE SPREADER BARS TO LIFT MOTOR OR REMOVE COOLER BEFORE LIFTING.

WATER IN- AND OUTLET FLANGE
NOMINAL TO BS4504/16/3
COOLING WATER FLOW RATE = L/S



SURFACES MARKED ✓ ARE TO BE MACHINED AND ALLOWANCE MUST BE MADE FOR THIS MACHINING. SURFACE TEXTURE CLA VALUES SHOWN IN MICROMETERS THUS 1.6 (MACHINED) 1.6 (UNMACHINED). UNTOLERANCED MACHINING DIMENSIONS ARE TO BE WITHIN ±0,1 OF TRUE SIZE. UNTOLERANCED FABRICATING DIMENSIONS ARE TO BE WITHIN ±3,0 OF TRUE SIZE. HOLE CENTRES ARE TO BE WITHIN ±0,4 OF TRUE POSITION. TOLERANCES MUST NOT BE CUMULATIVE.

ACTOM
LARGE MOTORS
This drawing is supplied
FOR TENDER PURPOSES ONLY



SHAFT END

COUPLING BORE

kW	-
r/min	-
STATOR AMPS.	-
STATOR VOLTS	-
PH	3
HZ	50

BEARINGS:- SIZE 18 Ø180 GB BEARING
NATURALLY COOLED

BEARING LUBRICANT ISO VG 46 OR EQUIVALENT
11,7 LITRES PER BRG

BEARINGS INSULATED AND EARTHING DEVICE FITTED TO D.E. BEARING

BEARING OIL INLET PIPES FITTED WITH FLOW SENSOR

THIS MOTOR IS FITTED WITH BEARINGS ALLOWING A SELF-CONTAINED FLOAT OF 6mm i.e. 3mm EACH SIDE OF CENTRAL RUNNING POSITION. IF FITTED WITH A FLEXIBLE COUPLING THIS MUST BE OF THE FLOAT LIMITING TYPE THUS ENSURING THAT NO EXTERNAL THRUST IS TRANSMITTED TO THE MOTOR BEARINGS

MOTOR MASS = 13000Kg APPROX.
STATIC LOAD AT EACH PAD= N
DYNAMIC LOAD AT EACH PAD= ± N

ACTOM
LARGE MOTORS
TITLE: OUTLINE FOR UL630/160

REFERENCE	DRAWING NUMBER	REV		
6302-1-351	6302-1-378	0	DR'N	WJS
ORIGL SHEET A1	FIRST ORDER QD33019-94	SCALE 1:10	CH'D	MMA
			APP'D	MMA

ORIG ISSUE 0
DATE 26-11-2018

Appendix B

Pump Station Drawings

Refer to Book of Drawings included under Main Feasibility Study Report for drawings.

Appendix C

Pipeline Drawings

Refer to Book of Drawings included under Main Feasibility Study Report for drawings.

Appendix D

Reservoir Drawings

Refer to Book of Drawings included under Main Feasibility Study Report for drawings.