REPORT NUMBER: P RSA D000/00/8309

Department:
Water Affairs and Forestry

Chief Directorate: Integrated Water Resource Planning
Directorate: Options Analysis

MOKOLO AND CROCODILE (WEST) WATER AUGMENTATION PROJECT (MCWAP) FEASIBILITY STUDY: TECHNICAL MODULE

Project No. WP9528

SUPPORTING REPORT NO 12
TECHNICAL MODULE FEASIBILITY STAGE
PHASE 2

Lead Consultant: Africon

In association with: KV3 VELA VKE

LIST OF REPORTS

REPORT NO	DESCRIPTION	REPORT NAME
FEASIBILITY STAGE		
P RSA A000/00/8109	Main Report	MCWAP FEASIBILITY STUDY TECHNICAL MODULE SUMMARY REPORT
P RSA A000/00/8409	Supporting Report 8A	GEOTECHNICAL INVESTIGATIONS PHASE 1
P RSA A000/00/8709	Supporting Report 8B	GEOTECHNICAL INVESTIGATIONS PHASE 2
P RSA A000/008509	Supporting Report 9	TOPOGRAPHICAL SURVEYS
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P RSA A000/00/9409	Supporting Report 7	SOCIAL AND ENVIRONMENTAL SCREENING
INCEPTION STAGE		
P RSA A000/00/9609	Inception	INCEPTION REPORT

REFERENCE

This report is to be referred to in bibliographies as:
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REPORT DETAILS PAGE

Project name:	***Mokolo and Crocodile River (West) Water Augmentation Project (MCWAP)***
Report Title:	**Report 12 – Phase 2 Feasibility Stage**
Author:	***BC Viljoen (VELA VKE) & JF Prinsloo (Africon)***
DWA report reference no.:	***P RSA A000/00/8309***
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PSP

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Project Coordinator & Manager

DEPARTMENT OF WATER AFFAIRS (DWA)
Approved for Chief Directorate: Integrated Water Resources Planning by:

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Preface

The Mokolo (Mogol) River catchment is part of the Limpopo Water Management Area (WMA). The Mokolo River originates close to Modimolle (Nylstroom) and then drains to the north into the Limpopo River. The Mokolo Dam (formerly known as the Hans Strijdom Dam) is the largest dam in the catchment. The dam was constructed in the late 1970s and completed in July 1980, to supply water to Matimba Power Station, Grootegeluk Mine, Lephalale (Ellisras) Municipality and for irrigation downstream of the dam. Based on the water infrastructure, the current water availability and water use allows only limited spare yield existing for future allocations for the anticipated surge in economic development in the area.

There are a number of planned and anticipated consequential developments in the Lephalale area associated with the rich coal reserves in the Waterberg coal field for which additional water will be required. These developments include inter alia the development of further power stations by Eskom, the potential development of coal to liquid fuel facilities by Sasol and the associated growth in mining activities and residential development.

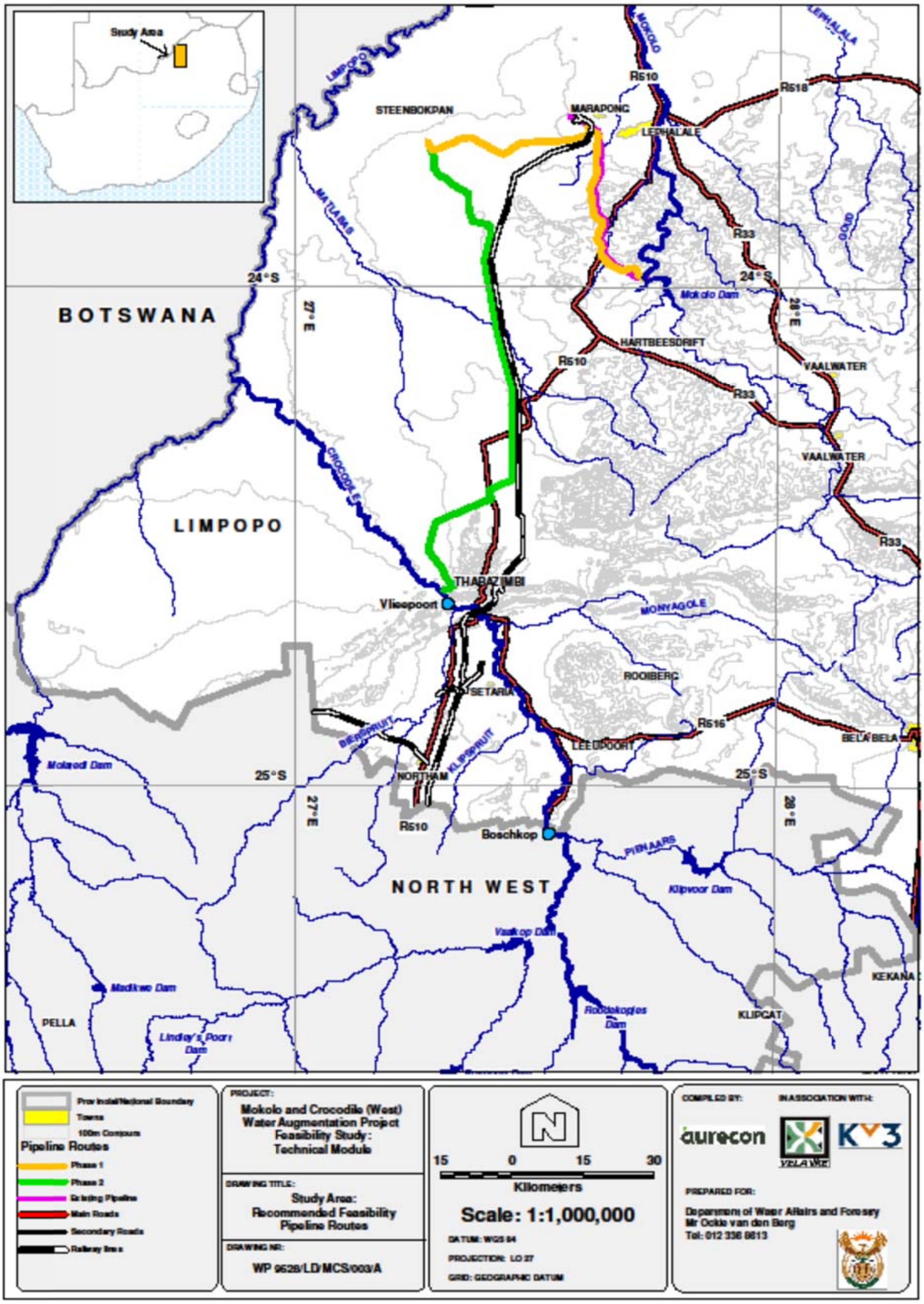
The development of new power stations is of high strategic importance with tight timeframes. Commissioning of the first generation unit will start in September 2010 and additional water needs to be available by mid-2011 according to the expected water requirements. A solution addressing the water needs of the Lephalale area must be pursued. The options to augment existing water supplies include transferring surplus effluent return flows from the Crocodile River (West) / Marico WMA to Lephalale and the area around Steenbokpan shown on the map indicating the study area on the following page.

The Department of Water Affairs (DWA) commissioned the Mokolo and Crocodile River (West) Water Augmentation Project (MCWAP) to analyse the options for transferring water from the Crocodile River (West) (CRW). In April 2008, the Technical Module of this study was awarded to Africon in association with Kwezi V3, Vela VKE and specialists. The focus of the Technical Module is to investigate the feasibility of options to:

- Phase 1: Augment the supply from Mokolo Dam to supply in the growing water requirement for the interim period until a transfer pipeline from the Crocodile River (West) can be implemented. Thereafter the scheme must continue to supply water to optimally utilise the full yield of Mokolo Dam.
- Phase 2: Transfer water from the Crocodile River (West) to the Steenbokpan and Lephalale area. Options to phase the capacity of the transfer pipeline (Sub-Phases 2 and 2B) must be investigated.

The Technical Module has been programmed to be executed at a Pre-feasibility level of investigation to identify different options and recommend the preferred schemes, which was followed by a Feasibility level investigation of the preferred water schemes. Recommendation on the preferred options for Phase 1 and Phase 2 Schemes were presented to DWA during October 2008 and draft reports were submitted during December 2008. The Feasibility Stage of the project commenced in January 2009 and considered numerous water requirement scenarios, project phasing and optimisation of pipeline routes. The study team submitted draft Feasibility reports during October 2009 and the draft MCWAP Main Report in November 2009.

This report (Report 12 – Phase 2 Feasibility Stage, P RSA A000/00/8309) covers the Feasibility Investigation for the water transfer scheme from the Crocodile River (West) to the Steenbokpan and Lephalale areas, including the integration of the Crocodile River (West) and Mokolo Schemes as the combined Mokolo and Crocodile River (West) Water Augmentation Project.


Study Area
BOTSWANA
LIMPOPO
NORTH WEST
STEENBOKPAN
MARAPONG
LEPHALALE
HARTBEESDRIFT
VAALWATER
THABAZIMBI
Vlieepoort
ROOIBERG
SETARIA
LEEUPOORT
NORTHAM
BELA BELA
Boschkop
KEKANA
KLIPGAT
PELLA
Mokolo Dam
Molatedi Dam
Klipvoor Dam
Vaalkop Dam
Madikwe Dam
Roodekopjes Dam
Lindley's Poort Dam
R510
R518
R33
R516
24°S
25°S
27°E
28°E
Prov incial/National Boundary
Towns
100m Contours
Pipeline Routes
Phase 1
Phase 2
Existing Pipeline
Main Roads
Secondary Roads
Railway lines
PROJECT:
Mokolo and Crocodile (West) Water Augmentation Project Feasibility Study: Technical Module
DRAWING TITLE:
Study Area: Recommended Feasibility Pipeline Routes
DRAWING NR:
WP 9528/LD/MCS/003/A
15 0 15 30
Kilometers
Scale: 1:1,000,000
DATUM: WGS 84
PROJECTION: LO 27
GRID: GEOGRAPHIC DATUM
COMPILED BY:
IN ASSOCIATION WITH:
aurecon
PREPARED FOR:
Department of Water Affairs and Forestry
Mr Ockie van den Berg
Tel: 012 336 8813

MOKOLO AND CROCODILE RIVER (WEST) WATER AUGMENTATION PROJECT FEASIBILITY STUDY:

WATER TRANSFER SCHEME

PHASE 2 FEASIBILITY STAGE

EXECUTIVE SUMMARY

The Executive Summary is structured as follows:

1. *Background*
2. *Water Requirements and Design Flows*
3. *River Abstraction Works*
4. *High Lift Pump Station and Crocodile River (West) Transfer Scheme*
5. *MCWAP Infrastructure Summary*
6. *MCWAP Cost Estimate*

BACKGROUND

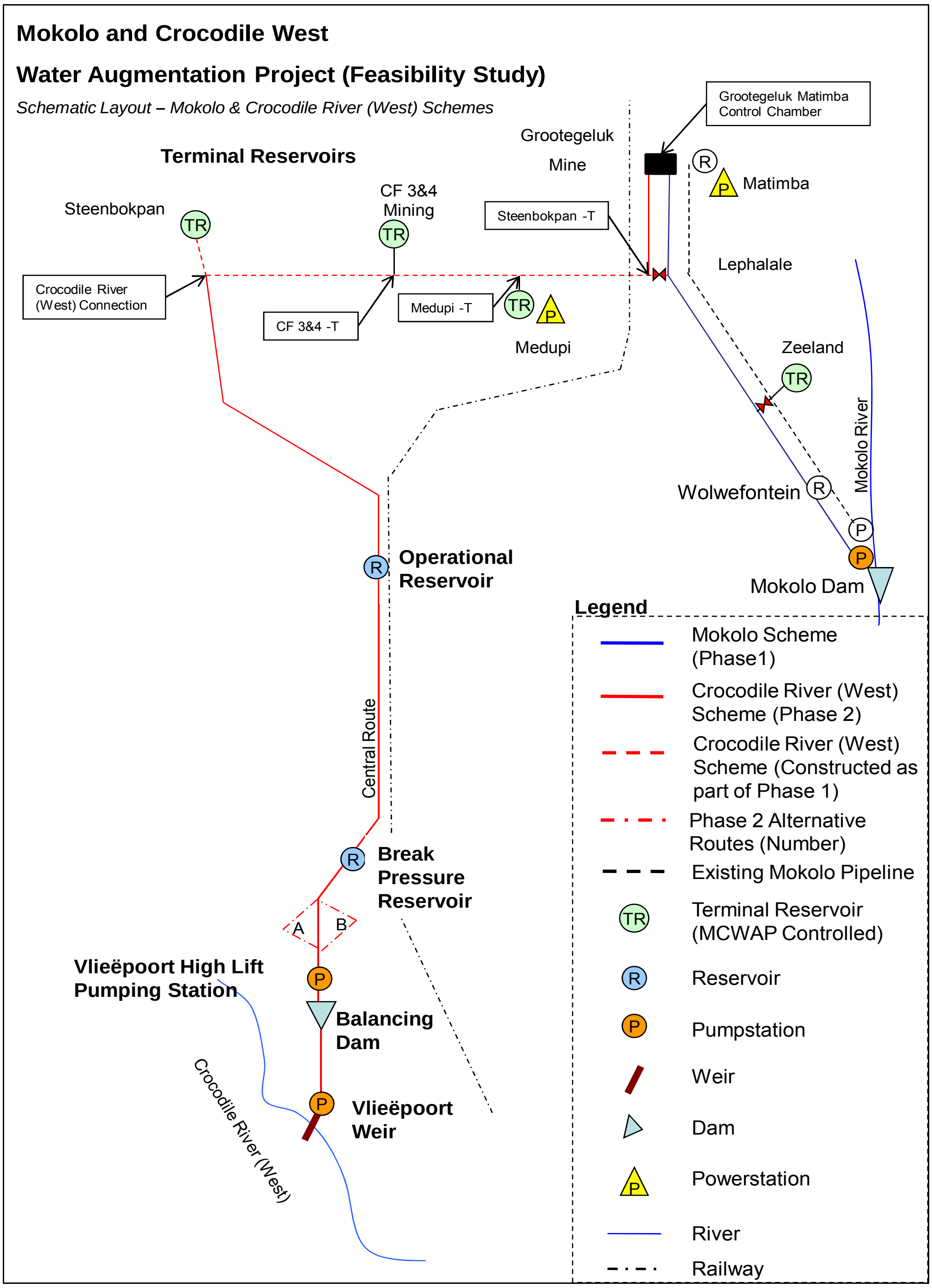
The primary purpose of the Mokolo and Crocodile River (West) Water Augmentation Project (MCWAP) is to develop the options to transfer water from the Mokolo and Crocodile River (West) to the Lephalale area to supply the primary and industrial users in this fast developing area.

Various options have been identified to convey water to the end users. These include the Mokolo Scheme, as well as the Crocodile River (West) Transfer Scheme to be operated in combination as the Mokolo and Crocodile River (West) Water Augmentation Project. The Mokolo Scheme is intended to supply the interim water requirements for a period until the Crocodile River (West) Transfer Scheme has been constructed and to support the reliability and redundancy requirements once the Crocodile River (West) Transfer Scheme is operational.

The combined Mokolo and Crocodile River (West) Water Augmentation Project is illustrated schematically below. The project will be implemented in phases as follows:

- *Phase 1: Augment the supply from Mokolo Dam to supply in the growing water requirement for the interim period until a transfer pipeline from the Crocodile River (West) can be implemented. The solution must over the long term, optimally utilise the full yield from Mokolo Dam.*
- *Phase 2A: Transfer system from the Crocodile River (West) to the Lephalale and Steenbokpan area where the pipeline will link to the infrastructure constructed as part of Phase 1.*

The findings of the Phase 1 Feasibility Investigation are documented in Supporting Report No. 11 (P RSA A000/00/8209). The objective of this report (Supporting Report No. 12), is to refine the work done at Pre-Feasibility stage for the Crocodile River (West) Transfer Scheme that will be implemented as Phase 2A of the Mokolo and Crocodile River (West) Water Augmentation Project.



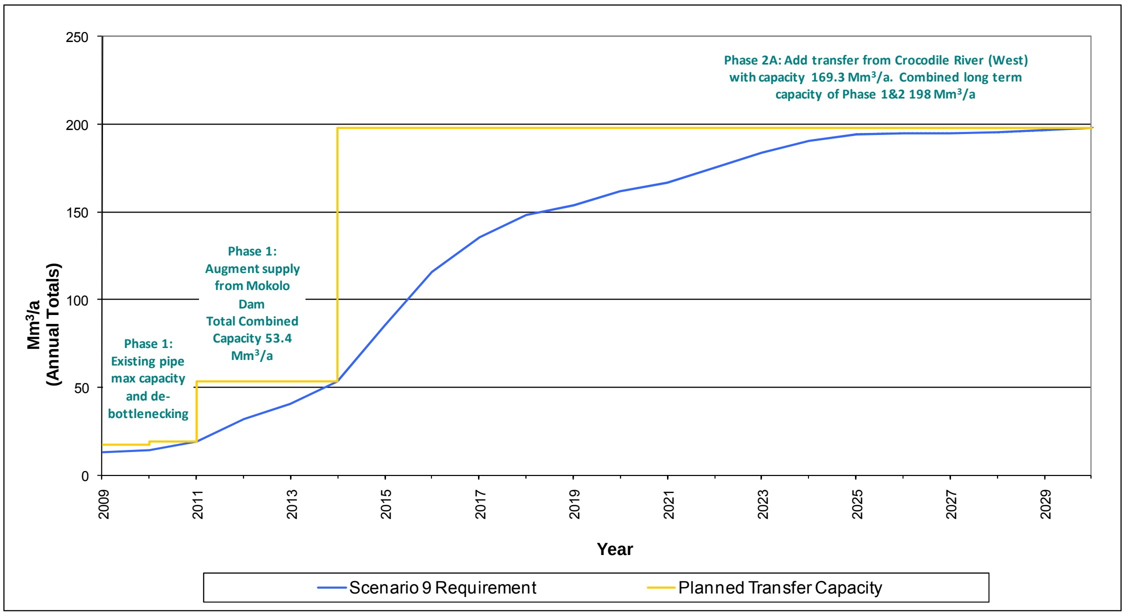
WATER REQUIREMENTS AND DESIGN FLOWS

The Mokolo and Crocodile River (West) Water Augmentation Project was sized to transfer the Scenario 9 water requirements, incorporating the following:

- ***Eskom**: Matimba Fluidised Bed Combustion (FBC), Medupi plus four additional coal fired power stations (the Flue Gas Desulphurisation (FGD) retrofit for Medupi was scheduled for the first major shutdown)*
- ***Independent Power Producers (IPP's)**: Equivalent of (one) (1) Eskom power station (starting in July 2010)*
- ***Exxaro**: Matimba coal supply, as well as implementation of projects A to K (new coal mines)*
- ***Coal mining**: Allowance for four (4) additional coal mines each supplying a power station*
- ***Sasol**: Mafutha one (1) Coal to Liquid Fuel (CTL) plant and associated coal mine. (starting in July 2011)*
- ***Lephalale and Steenbokpan**: Estimate based on projected growth in households for construction and permanent workforce*

All infrastructures, apart from the abstraction works, will be sized for the 2025 requirement, but taking into account the growth associated with projects not fully commissioned by 2025. Further projects after 2025 will require expansion of the Phase 2A infrastructure. The Scenario 9 water requirements are therefore reported up to 2030 and the system capacity is also based on the 2030 requirement. The abstraction works will be sized for the planned ultimate water requirement in 2050

The combined net water requirement and planned transfer capacity of the project is illustrated below.



MCWAP Water Requirements and Transfer Capacity

The strategic importance of the users that will account for the bulk of the water consumption requires that the risk of failure in the supply of water be kept to a minimum. Specific reliability and redundancy criteria were therefore allowed for in the combined Mokolo and Crocodile River (West) Water Augmentation Project (MCWAP).

RELIABILITY AND REDUNDANCY

General Criteria

The schemes shall be sized for 95% reliability, implying that water shall continue to be supplied without interruption even if the scheme is inoperative for up to 18 days of any one year, and the scheme capacity adjusted to allow the full annual requirements to be supplied in 347 days. Eighteen days storage capacity will be designed into the system to ensure that strategic customers will not be exposed to an unduly high risk of supply failure. The storage facilities for 18 days of water use must be provided by the end users and are therefore excluded from the project cost estimate. The operation of the facilities will however be under the control of the Mokolo and Crocodile River (West) Water Augmentation Project (MCWAP).

Limited redundancy will be provided by interconnecting the Mokolo and Crocodile River (West) Schemes. No redundancy will however, be available during the interim period (Phase 1) before the Crocodile River (West) Transfer Scheme is operational.

Reliability Criteria

The following sizing criteria were incorporated into the planning and costing of components to ensure reliability of supply:

- *Terminal reservoirs must be provided at all end user delivery points to provide on-site storage with a minimum storage capacity of 18 days.*
- *System losses were assumed to be 2% of the average annual water requirement.*
- *The diameter optimisation and economic evaluation was based on 105% of the gross annual average water requirement (including system losses) to account for the annual 18 days downtime of the scheme (Design Flow Rate).*
- *Pumping stations were sized and pipe pressure rating (wall thickness) determined to enable a transfer rate of 120% (Recovery Peak Flow Rate) of the gross annual water requirement (at sub-optimal pumping rates) in order to refill the Terminal Reservoirs over a 90 day period, following 18 days of continuous downtime.*
- *The worst case emergency scenario for the Crocodile River (West) Transfer Scheme occurs when the Phase 1 Scheme (Mokolo Delivery) makes no contribution to the project. The Crocodile River (West) Transfer Scheme (Phase 2) must therefore be able to transfer the full water requirement in the short term to those water users that can accept the lower quality Crocodile River (West) water. The flow under these circumstances was found to be less than the 120% recovery peak flow and no additional allowance was made for this scenario in the sizing of the scheme components.*

- *The annual peak of 9% (Peak Flow Rate) was not applied simultaneously with the design or recovery peak factors in sizing the components. Should recovery be required under seasonal peak flow conditions, it is recommended that the full transfer capacity from the Mokolo Scheme installed as part of Phase 1 (38.7 x 1.02 x 1.2 = 47.4 – 28.7 = 18.7 Million m^3/annum surplus supply) be utilised in the short term through the redundancy connection between the Mokolo and Crocodile River (West) Schemes to reduce the recovery period on the Crocodile River (West) Transfer Scheme to less than 90 days.*
- *Switchgear and instrumentation at abstraction sites will be located in the superstructure of the abstraction weir, or on the river bank next to the weir, but in both cases the equipment will be located above the Probable Maximum Flood (PMF) level. Other components forming part of the abstraction and desilting process (i.e. secondary desilting bays, balancing dam, etc.) will also be located above the PMF level.*
- *High-lift and booster pump stations will be positioned above the PMF and designed such that they will always be free-draining in the event of flooding due to failure of internal pipework.*
- *High-lift and booster pump stations will be designed with a minimum of one standby pump unit per station ensuring a minimum standby capacity of 25%. The maximum motor size will be limited to 10 MW per unit.*
- *Abstraction pump stations will consist of multiple abstraction bays housing submersible pumps capable of pumping a maximum of 1 m^3/s per unit. In the case of Vlieëpoort, one additional fully equipped standby bay plus one full spare pump including Mechanical and Engineering (M&E), valves and screens will be provided.*
- *All electrical equipment will be located above the PMF level.*
- *Strategic spares and equipment will be kept for medium voltage (MV) and low voltage (LV) electrical equipment and other critical components.*
- *100% Duplication of the power supply from the switch yards to the pump stations will be provided and a duplicate power supply (firm) will be provided by Eskom.*
- *Gravity pipelines downstream of the Break Pressure Reservoir will also have a capacity of 120% of the gross average annual demand, as determined by the rising main capacity.*

Redundancy Criteria

The following criteria were incorporated into the planning, sizing and costing of components to ensure redundancy of supply:

- *The existing pipeline from the Mokolo Dam will be refurbished and operated in parallel with the new pipeline to eventually provide redundancy for this scheme. The location of inter connections between the pipelines must be optimised as part of the detail design. The ultimate combined peak transfer capacity of the Mokolo Scheme after decommissioning of the existing pump station is 47.4 Million m^3/annum [(Q_{AADD} (53.4 – 14.7 = 38.7 Million m^3/annum) + losses (2%)) x 1.20]. The long-term available yield form* *the dam is 28.7 Million m^3/annum resulting in an 18.7 Million m^3/annum surplus supply capacity being available to provide redundancy backup in case of an emergency in the Crocodile River (West) Transfer Scheme.*

- *Redundancy will further be provided by an interconnection between the Crocodile River (West) and the Mokolo Schemes for those users that can accept the lower quality Crocodile River (West) water, so that either system can be augmented from the other.*
- *It should be noted that the Crocodile River (West) Transfer Scheme will not provide redundancy to Zeeland Water Treatment Works (WTW) due to the difference in water quality that cannot be accommodated by the treatment plant. Due care must therefore be taken with regards to the difference in water quality supplied by the Mokolo and Crocodile River (West) Schemes when designing for the redundancy connections between the two schemes.*
- *It should be considered to amend the water license agreement to allow the transfer of additional Mokolo Dam water (better quality and less expensive) to more of the end users during times of excess water availability (i.e. when the dam is spilling and there is surplus water flowing into the Limpopo River from the Mokolo River), by utilising the surplus transfer capacity on the Mokolo Scheme.*

Pump failures will be dealt with by having spare pumps and bays. If the power fails, three (3) possible solutions are possible:

- *Water flows past the weir for the duration – note that the civil works are designed for a 1:200 year flood recurrence interval event; the electrical supply should be similar.*
- *Provide standby diesel generators to keep part or the entire Abstraction Works going. The same would then have to apply to the rest of the MCWAP.*
- *Provide storage in the weir basin to cater for this situation. This resurrects the problem of upstream flood levels. Also raises the question of the duration of the "design power failure".*

DESIGN CAPACITY

The reliability and redundancy criteria adopted for this project resulted in the definition of the three design cases as summarised below.

Allowances and Factors used in Design Scenarios

Item No.	Allowance and Factors Applied[(2)]	Basic Design Case	Peak Design Case	Recovery Peak Design Case
1.	95% Reliability factor [(4)].	5%	0%	0%
2.	Allowance for water requirement peaks (average annual allowance) [(1), (4)].	0%	9%	0%
3.	Allowance for 90 day Recovery Period after maximum 18 day system outage [(3)].	0%	0%	20%
4.	System Losses. Phase 1A (Mokolo Dam supply) added to Phase 2A Crocodile River (West) transfer system losses.	2%	2%	2%
5.	Allowance for variations in river flow [(5)].	0%	0%	0%
6.	Failure of Phase 1A Mokolo Dam supply (due to over-usage, etc.) [(6)].	Nil	Nil	28.7 million m^3/a

Notes:

1. *Refer to Section 3.2 for details.*
2. *The % allowances factors was applied in the form: Flow x (1 + %).*
3. *The allowance for the 90-day Recovery Period was used independent of the other factors (apart from the system loss factor) to avoid compounding of related allowances. Pump selection and the pressure rating of pipelines as well as gravity flow sections of the system were designed to be able to transfer the recovery peak flow case.*
4. *The economic optimisation of infrastructure components that are part of a pumping system was based on the design flow incorporating the reliability factor (1.05). The reliability and peak factors were not applied simultaneously.*
5. *It was assumed that all the water requirements would be available in the river at the abstraction works.*
6. *Only used if greater than 20% allowance for Recovery Peak.*

River and Abstraction Works

Design capacity parameters for the Feasibility stage were generated from data obtained from the Water Resources and Water Requirements reports and are summarised in the following table.

Abstraction Works: Design Flow and Capacity Parameters

Item No.	Design Data	SCENARIO 9			SCENARIO 9 IN 2050		
		Design Flow [(1)]	Peak Flow [(2)]	Recovery Peak Flow	Design Flow [(1)]	Peak Flow [(2)]	Recovery Peak Flow
1.	**Water Requirements**	**million m^3/a**	**million m^3/a**	**million m^3/a**	**million m^3/a**	**million m^3/a**	**million m^3/a**
1.1	Phase 1A Transfer requirements (maximum long-term average) [(4)].	28.7	28.7	28.7	28.7	28.7	28.7
1.2	Exxaro pipeline contribution.	14.7	14.7	0	14.7	14.7	0
1.3	Phase 1A Transfer requirements (maximum long-term average)	14.0	14.0	43.4	14.0	14.0	43.4
1.4	Phase 2A Crocodile River (West) Transfer requirements (maximum average), including system losses (2%) along Phase 1A and Phase 2A pipelines and reservoirs.	173.3	173.3	173.3	410.9	410.9	410.9
1.5	Additional Losses in Crocodile River (due to additional release) for weir at Vlieëpoort. See section on river losses below.	28.7	28.7	28.7	30.5	30.5	30.5
1.6	Total irrigation water requirements upstream of Vlieëpoort	120.0	120.0	120.0	120.0	120.0	120.0
1.7	Present water requirements downstream of Vlieëpoort	28.9	28.9	28.9	28.9	28.9	28.9
1.8	Total Releases from Dams to provide for Phase 2A – Vlieëpoort Option	302.7	302.7	302.7	562.1	562.1	562.1
1.9	Total Flow Releases from Dams to provide for Phase 2A - Vlieëpoort Option	9.6 m^3/s	9.6 m^3/s	9.6 m^3/s	17.2 m^3/s	17.2 m^3/s	17.2 m^3/s

Item No.	Design Data	SCENARIO 9			SCENARIO 9 IN 2050		
		Design Flow [(1)]	Peak Flow [(2)]	Recovery Peak Flow	Design Flow [(1)]	Peak Flow [(2)]	Recovery Peak Flow
2.	**Vlieëpoort Abstraction Weir**						
2.1	Design flow allowance	5%	0%	0%	5%	0%	0%
2.2	Peak flow allowance	0%	9%	0%	0%	9%	0%
2.3	Recovery Period allowance	0%	0%	20%	0%	0%	20%
2.4	Design Flow Vlieëpoort Weir	5.8 m^3/s	6.0 m^3/s	6.6 m^3/s	13.7 m^3/s	14.2 m^3/s	15.6 m^3/s
2.5	Number of Low Lift Pump Station bays[(3)].	4 No.	4 No.	4 No.	8 No.	8 No.	9 No.
2.6	Number of pump sets[(3)].	7 No	7 No	8 No	15 No	16 No	17 No
2.7	Number of desilting channels in Desilting Works [(3)].	7 No.	7 No.	8 No.	15 No.	16 No.	17 No.
3.	**Net Storage Capacity Options of High-lift Pump Station Balancing Dam**	m^3	m^3	m^3	m^3	m^3	m^3
3.1	Capacity based on Operational Storage Criterion of 6 hours.	129 600	129600	151 200	324 000	324 000	367 200
3.2	Capacity based on Nominal Storage Criterion of 3 days.	1 425 000	1 425 000	1 425 000	3 543 000	3 543 000	3 543 000
3.3	Capacity based on River Hydrograph Storage Criterion adopted for Feasibility Stage.	1 300 000	1 300 000	1 300 000	1 820 000	1 820 000	1 820 000

Notes:

1. *Total Phase 2A water requirements less the Phase 1 contribution plus allowance for seasonal peaks.*
2. *The worst case peak emergency scenario for the Phase 2 Works occurs when the Phase 1 Scheme (Mokolo Delivery) makes no contribution to the MCWAP (the Phase 2A Crocodile Works therefore transfers the full water requirement), OR, 20% allowance for recovery period after downtime, whichever is the largest.*
3. *One additional fully equipped standby bay plus one full spare pump including M&E, valves, screens for the design case. For the Crocodile weirs this is based on submersible pump with 1 m^3/s rated capacity. Data for suitable pumps were obtained from pump suppliers. Nine double pump bays were provided to cater for the projected long-term requirements.*
4. *Ultimate Mokolo Dam supply after commissioning of Crocodile River (West) Transfer Scheme (28,7 Million m^3/a, including any losses).*

RIVER LOSSES

The Pre-feasibility analyses were based on a model where all irrigation water requirements were assumed to be abstracted directly from the river and the assumption that the evapo-transpiration losses from the vegetation in the riparian strip and the seepage outflows from the river would not be significantly affected by increased releases from Klipvoor, Roodekoppies and Vaalkop Dams. The additional water losses that could occur were deemed to be due to increased evaporation losses from the river water surface and increased abstraction by the irrigators. The water abstracted from the boreholes in the alluvial aquifers underlying the floodplains in the river valley for irrigation and the unregulated runoff contributions from the catchments downstream of the Klipvoor, Roodekoppies and Vaalkop Dams were not included in the analyses, but would in fact be available to the system to supply the irrigators during periods with simulated shortfalls.

By considering the mass balance relationship at Vlieëpoort, a re-arrangement of the data presented in Report No. 4 – Technical Module: Dams, Abstraction Weirs and River Works, was possible. Consequently, when additional water is released from the upstream dams for the MCWAP, there will be a mean diffuse net seepage water loss of 23,5 Million m^3/annum to the alluvial aquifers connected to the river.

The analyses have been done on the basis of medium–term mean annual flows and the actual average daily river flows can vary significantly from these mean flows. Active management of water releases from the upstream dams will be required to take maximum advantage of downstream inflows. Since the aquifers will be full most of the time once the water is released from the upstream dams, there will be less induced recharge of the aquifer during the high flow months and therefore more water is likely to flow past Vlieëpoort during the high flow season. This would constitute an additional loss from the system that can only be quantified by means of river flow measurements, but the loss is expected to be less than the above 23,5 Million m^3/annum.

<u>Crocodile River (West) Transfer Scheme</u>

The three design cases were applied as follows to size components of the Crocodile River (West) Transfer Scheme (Vlieëpoort Weir high lift pump station to the Steenbokpan connection with the Steenbokpan-Lephalale pipeline):

Crocodile River (West) Transfer Scheme Design Capacity

Item No.	***Design Data***	***SCENARIO 9***		
		Design Flow [(1)]	***Peak Flow [(2)]***	***Recovery Peak Flow [(3)]***
1.	***Water Requirements***	***million m^3/a***	***million m^3/a***	***million m^3/a***
1.1	*Phase 1A Transfer requirements (maximum average) [(5)].*	*28.7*	*28.7*	*28.7*
1.2	*Exxaro pipeline contribution.*	*14.7*	*14.7*	*0*
1.3	*Phase 1A Transfer requirements (maximum average)*	*14.0*	*14.0*	*43.4*
1.4	*Phase 2 Crocodile River (West) Transfer requirements (maximum average), including system losses (2%) along Phase 1A and Phase 2A pipelines and reservoirs.*	*173.3*	*173.3*	*173.3*
2.	***Transfer Scheme***			
2.1	*Design flow allowance*	*5%*	*0%*	*0%*
2.2	*Peak flow allowance*	*0%*	*9%*	*0%*
2.3	*Recovery Period allowance [(3)].*	*0%*	*0%*	*20%*
2.4	*Design Flow Transfer Scheme*	*5.8 m^3/s*	*6.0 m^3/s*	*6.6 m^3/s*
2.5	*Number of high lift pump sets [(4)].*	*5 No.*	*5 No.*	*5 No.*
3.	***Net Storage Capacity of Break Pressure and Operational Reservoirs***	***m^3***	***m^3***	***m^3***
3.1	*Capacity based on Operational Storage Criterion of 8 hours.*	*167 000*	*173 000*	*190 000*

Notes:

1. *Total Phase 2A water requirements less the Phase 1 contribution plus 5 % allowance for reliability peak.*
2. *Total Phase 2A water requirements less the Phase 1 contribution plus 9% allowance for seasonal peaks.*
3. *The worst case peak emergency scenario for Phase 2A Works occurs when the Phase 1A Scheme (Mokolo Delivery) makes no contribution to the MCWAP (the Phase 2A Crocodile Works therefore transfers the full water requirement), OR, 20% allowance for recovery period after downtime, whichever is the largest.*
4. *Based on 4 duty, 1 standby pump configuration for the 2030 Scenario 9 requirement. Future upgrading will require additional pumping units.*
5. *Ultimate Mokolo Dam supply after commissioning of Crocodile River (West) transfer system (28.7 Million m^3/a, including any losses).*

Refer to Report No. 11 - Technical Module: Phase 1 Feasibility Stage, for the design capacities of the Phase 1 infrastructure components.

WATER REQUIREMENTS AND DESIGN FLOW: KEY ISSUES TO BE ADDRESSED DURING DETAIL DESIGN

The Pre-Feasibility and Feasibility Stages of the Mokolo and Crocodile River (West) Water Augmentation Project (MCWAP) took place within a very dynamic planning environment. As result, further variations to the water requirements and design capacities are to be expected and must be incorporated as part of the detail design process. In this regard, the following need to be performed:

- *Confirm and implement the latest, approved water requirement scenario for the project.*
- *Re-confirm the reliability and redundancy criteria that must be applied in sizing the Phase 2A infrastructure components. Aspects that might impact on these criteria include the following:*
  - *The conditions of the final end user agreements;*
  - *The final system operating philosophy; and*
  - *Risk assessment.*
- *Quantify evaporation and system transmission losses more accurately (a value of 2% was assumed at Feasibility stage).*
- *Perform a water balance incorporating latest demands, system losses, peak and recovery factors and a statistical assessment of storage requirements and system reliability.*

RIVER ABSTRACTION WORKS

The details of the River Abstraction Works are given in Report No. 11 – Technical Module: Phase 1 Feasibility Stage and are summarised below.

During the Feasibility Stage, the following aspects of the River Abstraction Works component of the MCWAP Study were developed further:

- *The Vlieëpoort Abstraction Weir, Gravel Trap, Low-lift Pump Station, Desilting Works and High-lift Pump Station Balancing Dam;*
- *River Management and in particular issues such as possible functional arrangements, river flow management, gauging weir requirement assessments and operation and maintenance philosophies;*
- *Assessment of dam options at Vlieëpoort; and*
- *Stakeholder and bulk user liaison.*

SIZING

Sizing criteria were prepared during the course of the Pre-feasibility stage and the structures were sized accordingly for costing purposes. Pertinent sizing data for the Vlieëpoort River Abstraction Works are summarised below.

Vlieëpoort Abstraction Weir Design and Sizing Data

Item No.	*DESIGN DATA*	*VALUE*
1.	*Design Flood (RDF) (1:200 year Recurrence Interval Flood)*	*5 740 m^3/s*
2.	*Safety Evaluation Flood (SEF) (PMF)*	*11 180 m^3/s*
3.	*1:20 year Recurrence Interval Flood*	*2 870 m^3/s*
4.	*1:50 year Recurrence Interval Flood*	*4 020 m^3/s*
5.	*River bed Level*	*890.0 masl*
6.	*Lowest OC Level.*	*893.2 masl.*
7.	*NOC Level (PMF plus 0.5m Freeboard).*	*912.8 masl*
8.	*OC Length*	*153 m*
9.	*Total Length of Structure*	*308*

DESCRIPTION OF COMPONENTS

The Abstraction Weir arrangement consists of:

- *Mass concrete gravity type Diversion Weir, 3.2 m high with ogee and roller bucket spillway. This will be built to the final requirements for Phase 2.*
- *Gravel Trap in weir basin with flushing facility and trash rack with concrete channels leading from gravel trap to each pump-well in the low-lift pump station that is incorporated partly into the non-overspill crest (NOC) flank of the weir and partly into the river bank. Nine pump bays, each capable of accommodating two fully equipped 1.0 m^3/s capacity submersible grit pumps were provided for. This will be built to the final requirements for Phase 2, but only be equipped for Phase 2A initially.*
- *Low pressure rising main to the Desilting Works for Phase 2A, which will be duplicated for Phase 2B.*
- *The rising main for Phase 2A will consist of a 2 100 mm diameter steel pipeline approximately 5 km long. It will then be split with a manifold into nine 750 mm diameter pipes leading to the Desilting Works inlets. Each pipe will have a gate valve in a valve chamber adjacent to the Desilting Works.*
- *Desilting Works with flushing facility and sediment storage dump located near the low-lift pump station, but above the Probable Maximum Flood level (PMF) to be built for Phase 2A and upgraded for Phase 2B.*
- *The Desilting Works for Phase 2A will consist of eight 80 m long channels, 2.5 m wide and depth varying from 3.8 m to 4.8 m.*
- *A gravity pipeline between the Desilting Works and Balancing Dam inlets.*
- *Each desilting channel will have a 750 mm diameter steel pipe outlet.*

- *A multi-compartment Balancing Dam sized to provide balancing storage to cater for unplanned changes in river flows and for differences in inflows from the Desilting Works and outflows to the High Lift Pump Station. The Balancing Dam will also be equipped with a silt flushing facility although only infrequent use, perhaps once every 10 years, is expected.*
- *The Balancing Dam has top dimensions of 600 m x 370 m, five compartments and a total live storage capacity of 1 300 000 m^3. The depth varies from 10.5 m at the inlet side to 13.2 m at the outlet side. A freeboard provision of 0.5 m above the spillway crest, which is 0.5 m above the full spillway crest (FSC), was made.*

ABSTRACTION WORKS: KEY ISSUES TO BE ADDRESSED DURING DETAIL DESIGN

The following issues were identified during the course of the Feasibility stage and would require further investigation to ensure fit for purpose designs:

1. *Depth of scour at Vlieëpoort during high floods. Scour potential at the weir must be modelled to confirm the depth of founding of the weir structure. The present Feasibility stage layout assumes that the proposed jet grouting foundation treatment will provide adequate founding conditions and that together with the roller bucket spillway design and extensive downstream heavy riprap protection will protect the structure against scour.*
2. *Foundation Design. Deep jet grouted foundations have been successfully used in the past to improve hydraulic structure founding conditions. Once the results of a detailed materials investigation are available, the Feasibility layouts should be reviewed and developed further.*
3. *Alluvial aquifer flows at Vlieëpoort. The Feasibility stage layouts show that the entire river bed section below the weir will be jet grouted, thereby effectively blocking the flow in the aquifer. Whilst this arrangement is intended to prevent piping foundation failure, greater loads could be imposed on the weir and its foundations if the water table downstream of the weir is lowered. This can be counteracted if the flow past Vlieëpoort is regulated sufficiently to maintain a continuous flow over the weir. The water table level downstream of the weir should nevertheless be monitored continuously to alert the operators of any potentially dangerous situation.*
4. *Liquefaction potential. The nature of the underlying alluvial sands and silts at Vlieëpoort must be investigated to determine the potential for liquefaction during a natural or induced seismic event.*
5. *Sizing and configuration of Desilting Channels. Feedback received on the operation of the Lebalelo Abstraction Works in the Olifants River in Limpopo Province indicated that the very fine fraction of the suspended silt in the Olifants River, when in flood, failed to completely settle out in the de-silting channels. This fraction requires longer retention times to settle out and therefore only settled in the balancing dams where it affected the operational availability of the system and was also difficult and time-consuming to*

remove, primarily because the balancing dams were not designed to be maintained at frequent intervals. In the case of the Crocodile River (West) Transfer Scheme the problem is accentuated by the relatively large storage capacity and retention times of the Balancing Dam.

6. *Location of High Lift Pump Station Balancing Dam. The Feasibility layouts identified two potential sites for the dam. The site closest to the Abstraction Weir has since been confirmed to be located on dolomite and should therefore be avoided if possible. The preferred site is some 5km downstream of the Abstraction Weir and on much more favourable founding conditions (residual Ventersdorp lava), but further planning is required to refine the layout and assess the socio-economic impacts.*
7. *Sizing of the High-lift Pump Station Balancing Dam. The present approach is based on river flow management with a 3 – 4 day river flow response time from the upstream dams to Vlieëpoort. With improved control over flows in the river and shorter actual response times it is anticipated that the required capacity of the Balancing Dam should reduce accordingly. A storage capacity in the order of 200 000 m^3 less may be possible.*
8. *Hydraulic computer modelling of the river is recommended once the detail survey becomes available. This model will allow for better computation of flood levels applicable to the base conditions and post-construction conditions and allow better assessments of the impact of the Abstraction Works on affected landowners and existing infrastructure.*
9. *The hydraulic model will also provide flood levels downstream of the weir that are required for the placement of the Desilting Works, Balancing Dam, High Lift Pump Station and switchyards and might also influence the choice of the site for these components.*
10. *A prototype or Computational Fluid Dynamics (CFD) model of the Abstraction Weir, Gravel Trap and Low Lift Pump Station is recommended in order to optimise the placement, layout and size of these structures.*
11. *During flushing of the Desilting works and desilting of the Balancing Dam, high amounts of silt need to be handled which cannot be discharged into the river. Further investigation is required to confirm environmental requirements and to identify appropriate silt separation facilities and storage and/or disposal thereof.*
12. *Flows passing the Abstraction Weir must be measured. A downstream flow gauging structure will be required to measure surface flows since flows over the weir may not be uniform enough.*

HIGH LIFT PUMP STATION AND TRANSFER SCHEME

PIPELINES

An option analysis for the Phase 2A pipeline found that a rising main to an Operational (and also Break Pressure) Reservoir at Ch 26700 and gravity flow to a possible future

Operational Reservoir and further to the connection with the Lephalale-Steenbokpan pipeline were the most feasible.

The transfer pipeline starts at the Vlieëpoort High-lift Pump Station and continues north along the Thabazimbi-Dwaalboom Road (D1649). The pipeline crosses the farm Paarl 124 KQ parallel to an existing high voltage power line before turning east towards the R510. The proposed site for the Operational (and also Break Pressure) Reservoir is located on the farm Zondagskuil 130 KQ. From the Operational Reservoir, the route continues north along the R510 for a short distance before turning east on the boundary between Tarantaalpan and Diepkuil. The route then heads north along the western boundary of the railway line servitude to the site of the possible future Operational Reservoir located on the Farm Rooipan 357 LQ, crossing the Matlabas River en route. From the possible future Operational Reservoir, a gravity pipeline continues north-west towards Steenbokpan, where it links to the pipeline from Lephalale constructed as part of Phase 1 of the MCWAP. Refer to Drawings 9528/LD/CS/001 and 002 included in Appendix C for the locality and layout of the Phase 2A Abstraction Works.

Design Considerations

Coating and lining

The following generic coating and lining systems are recommended for the Crocodile River (West) Transfer Scheme pipelines:

Pipe Coating Options

Product/Method		***Field Joint Repair Method***
External Coating	*Preferred:* *Trilaminate Polyethylene (3LPE)* *or* *Polyurethane* *Alternative:* *Polymer modified bitumen/Glass Fibre (Bituguard)*	 *Liquid or powder epoxy plus cold tape wrap* *Polyurethane* *Bituguard hot applied tape*
Internal Lining	*Preferred:* *Epoxy* *or* *Cement Mortar*	 *Epoxy* *Cement Mortar*

<u>Long Term Roughness</u>

The recommended long term roughness parameters, as well as the influence of biofilm are summarised below.

Long Term Roughness

Parameter	***Cement Mortar Lining***		***Epoxy Lining***	
	Suggested	***Maximum***	***Suggested***	***Maximum***
Long term absolute roughness (mm)	*1.1*	*1.5*	*0.5*	*0.7*
Influence of biofilm	*Reduction in diameter of 5-8 mm*			

Structural Design and Optimisation

The design of flexible buried pipes involves consideration of the interaction between the steel shell and the surrounding backfill and the deflection limits appropriate to the lining and coating system. Flexible pipes obtain a large portion of their load carrying capacity from the surrounding backfill, and therefore, the incorporation of this interaction is important. The design method should take into account the strength of the pipe-soil system as a whole, without relying solely on the strength of the individual components. A detailed structural analysis and optimisation was not performed during the Feasibility stage investigation and must be carried out during the detail design stage.

Minimum Pipe Wall Thickness

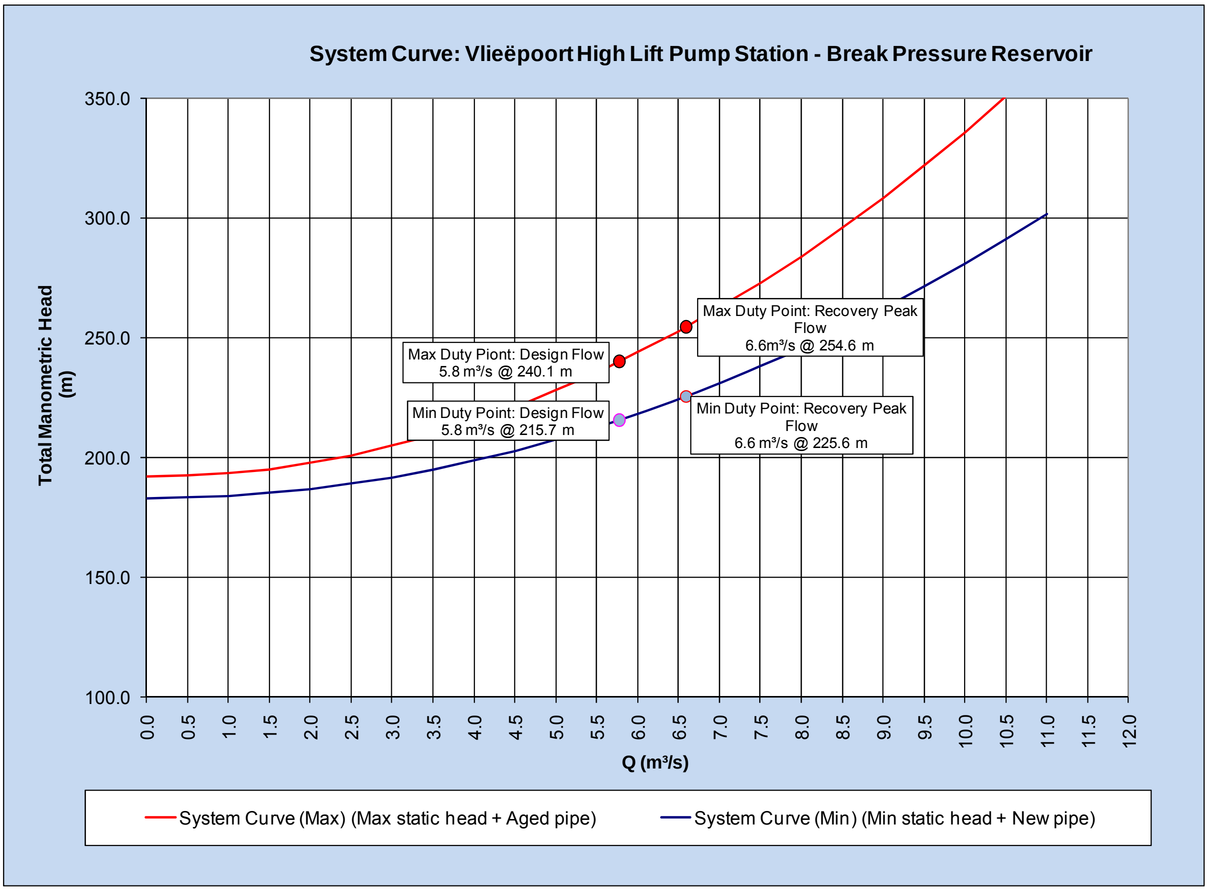
The wall thickness is a function not only of the internal pressure (i.e. operating pressure and the surge over-pressure), but also of the external pressure exerted on the pipe (i.e. soil overburden, external fluid pressure, vehicular and vacuum loading etc.). On large diameter steel pipelines practical requirements such as pipe handling and installation requirements may in fact control the minimum pipe wall thickness. Therefore, the selection of the minimum pipe wall thickness is a function of:

- *Internal Design Pressure (i.e. maximum design pressure and vacuum);*
- *External Design Pressure (i.e. soil overburden plus vacuum and vehicular loading);*
- *Pipe Handling Requirements (i.e. pipe manufacture, transportation, laying and backfilling requirements); and*
- *Pipe Buckling Capacity (Considering good quality soils, poor quality soils, as well as hydrostatic or unsupported soil conditions).*

Optimum Diameter Selection

The optimum pipe diameter for the rising main was determined by performing an economic analysis over a 45-year period for a number of different pipe diameters and resultant D/t ratios. The analysis found that a 1 900 mm ND was the optimum pipe diameter.

The respective system duty points are illustrated below.



Vlieëpoort Pump Station System Curve

System Hydraulics

Based on a hydraulic assessment of the scheme, the pipe sections required for the Crocodile River (West) Transfer Scheme are summarised below.

System Hydraulic Assessment

Pipe Section	***Diameter (mm)***	***Length (km)***	***D/t & wt***	***Velocity (m/s)***	
				Design Flow	***Recovery Peak Flow***
Vlieëpoort High Lift Pump Station to Operational and Break Pressure Reservoir (Rising Main)	*1900*	*26.7*	*110 17.5 mm*	*2.05*	*2.34*
Operational and Break Pressure Reservoir to possible future Operational Reservoir (Gravity Main)	*2200*	*62.7*	*130 17.2 mm*	*1.52*	*1.73*
Operational Reservoir to Crocodile River (West) Connection (Gravity Main)	*2300*	*28.2*	*140 16.7 mm*	*1.39*	*1.58*

Pipe Section	***Diameter (mm)***	***Length (km)***	***D/t & wt***	***Velocity (m/s)***	
				Design Flow	***Recovery Peak Flow***
Crocodile River (West) Connection to Steenbokpan (Constructed as part of Phase 1) (Gravity Main)	*1900*	*1.4*	*160 12.1 mm*	*1.58*	*1.80*
Crocodile River (West) Connection to CF 3&4 Mining T-off (Constructed as part of Phase 1) (Gravity Main)	*1100*	*27.2*	*160 7.0 mm*	*1.35*	*1.54*
CF 3&4 Mining to Medupi T-off (Constructed as part of Phase 1) (Gravity Main)	*900*	*3.6*	*160 5.7 mm*	*1.48*	*1.69*
Medupi T-off to Steenbokpan T-off (Constructed as part of Phase 1) (Gravity Main)	*900*	*8.2*	*160 5.7 mm*	*1.23*	*1.41*
Steenbokpan T-off to Grootegeluk / Matimba Control Chamber (Gravity Main)	*800*	*1.9*	*160 5.1 mm*	*1.56*	*1.78*

The final position and number of end user Terminal Reservoirs has not been confirmed as most of the end users are still in various stages of planning. The position of the system off-takes and Terminal Reservoirs and the ultimate water requirements of the end users will have an influence on the pipe sizes and system operation along the Lephalale-Steenbokpan link and must be confirmed as part of the detail design stage. The operation of the Lephahale-Steenbokpan link, built under Phase 1 will be reversed for the ultimate system operation to transfer water from west to east to provide water to users in the Lephalale area. This will require careful consideration of the valve selection and positioning, as well as the accommodation of potential surge pressures. Consideration must also be given to the water quality requirements of the various users and the effects on the valve positions.

A detailed hydraulic analysis for the positioning of the air valves as well as isolating, reflux and control valves must therefore be performed as part of the detail design for each of the Phase 1 and Phase 2 Schemes.

Cathodic Protection and AC Mitigation

The proposed pipeline routes run parallel to and cross a number of existing and proposed future high voltage power line routes, most notably, the planned new Eskom corridors that will be constructed as part of the Mmamabula-Medupi Transmission Integration Project. These corridors will contain six 765 kV overhead high voltage alternating electrical current (AC) power lines.

The pipeline also runs parallel to the railway line for a significant distance. The railway line is currently not electrified, but if electrified in future, it is expected to be with AC power.

Stray current interference is expected on the pipeline and cathodic protection (CP) and AC mitigation measures will be required to protect the proposed pipeline.

HIGH-LIFT PUMP STATION

The pump investigations showed that the required delivery capacity can be achieved by using four duty pumps with one standby unit. Each pump set will comprise an in-line booster pump and a main high pressure pump (no valve in between). The minimum static suction head required for the booster pumps, based on the site conditions and the likely Net Positive Suction Head (NPSH) of the booster pumps, was estimated to be 8 m. To reduce excavation depth careful design of the inlet conditions in the Balancing Dam and the suction manifold will be required. It is presently envisaged that Variable Speed Drive (VSD) units be installed to enable continuous, more economical pumping and improve the flexibility of the pumping scheme. In addition, VSD drives would greatly reduce starting currents and reduce pressure surges in the system.

The estimated absorbed power (MVA) at the design flow and recovery peak flow duty ratings are summarised below for the minimum and maximum operating head.

Estimated Absorbed Power

	Design Flow	***Recovery Peak Flow***
Minimum System Head (Min static head and 0.05 mm absolute roughness)		
Duty	*5.8 m^3/s @ 216 m*	*6.6 m^3/s @ 226 m*
Absorbed power	*14.8 MVA*	*17.7 MVA*
Maximum System Head (Max static head and 0.5 mm absolute roughness)		
Duty	*5.8 m^3/s @ 240 m*	*6.6 m^3/s @ 255 m*
Absorbed power	*16.5 MVA*	*19.9 MVA*

Bulk Electricity Connection

The Vlieëpoort site will be fed from the Thabazimbi Munic (Thaba Combined) and Thabazimbi Rural (Thabatshipi) 132kV substations. High voltage transmission lines will be built from each substation to the Vlieëpoort site, in order to ensure redundancy. (Loop in – Loop out system)

RESERVOIRS

The Crocodile River (West) Transfer Scheme includes an Operational and Break Pressure Reservoir located on the farm Zondagskuil 130 KQ, as well as a possible future Operational Reservoir located on the farm Zoutpan 367 LQ.

The scheme will supply water into Terminal Reservoirs located on the sites of the end consumers. The Terminal Reservoirs must provide a minimum of 18 days reliability storage and will be built by the respective end users, but will be operated and controlled by the MCWAP.

The Reservoirs will generally be in the form of an artificial dam formed by shallow excavation and surrounding earthfill embankments. The final depth and size of the reservoirs will be determine by the site topography (cut and fill balance) with the aim of minimising surface area to reduce evaporation and maximum flow through to prevent stagnation of the water.

Reservoirs will have to be lined with an appropriate waterproof lining system (HDPE or similar material) and suitable sub-surface drainage must be provided.

As part of Phase 2B it is expected that the Operational and Break Pressure Reservoir can be converted to a Surge Reservoir after converting the first portion of the gravity pipeline to a rising main to a new Operational Reservoir (at Node 15) to increase the capacity of the Crocodile River (West) Transfer Scheme.

The Terminal Reservoirs, will typically be compartmentalised and have a minimum storage of 18 days of the consumers' average annual water requirement (reliability requirement), which will be reserved for purposes of the MCWAP operation and maintenance only, plus storage to be determined by consumers for their own internal peak balancing and operational requirements

OPERATION AND MAINTENANCE

The control and operation of all sites forming part of the MCWAP will be monitored and managed by means of a System Control and Data Acquisition (SCADA) system from a central control room manned on a 24hr/day basis. The monitoring system must provide adequate planning, operational and costing reports to effectively manage, operate and maintain the system.

In addition, the maintenance philosophy must address mechanical, electrical and civil engineering aspects, categorised as follows:

- *Routine planned maintenance;*
- *Major breakdown repairs; and*
- *Minor breakdown repairs.*

Repairs on pipe and check valves will have to take place during planned system maintenance.

ENVIRONMENTAL AND SOCIAL IMPACTS

The pipeline traverses some sensitive areas where particular care should be taken. These will be pinpointed during a detailed environmental investigation. Rocky areas are most sensitive due to the presence of aloe species as well as the distinct habitat it provides for animal species. The construction of the river Abstraction Works (and Balancing Dam) and High Lift Pump Station at Vlieëpoort will have an impact that must be mitigated. To minimise this impact the sites must be identified in conjunction with faunal and floral specialists where not dictated by physical features.

The location of the pipeline adjacent to existing linear infrastructure together with adequate mitigation measures will ensure that the construction of the pipeline will have a minimal lasting effect on the surrounding area.

The detailed investigations envisaged for the design stage will be the responsibility of the consultant responsible for the Environmental Impact Assessment. The Pre-feasibility and Feasibility stages only consisted of a desktop investigation and a brief site visit to identify major fatal flaws, if any should exist. During the Design Phase detailed fauna and flora investigations will have to be conducted to identify specific sensitive plant communities that are sensitive as well as sensitive habitats that will be affected by the MCWAP. The investigation also needs to indicate how well such communities are represented in the vicinity and elsewhere.

The most significant socio-economic impacts of the proposed pipelines are:

- *Negative impacts:*
  - *Loss of agricultural land;*
  - *Foreign work force and inflow and outflow of workers;*
  - *Workers' camps and effect on communities in vicinity;*
  - *Possible disruption of daily living;*
  - *Safety and security;*
  - *Impact on property values; and*
  - *Aesthetic impacts.*

- *Positive impacts:*
  - *Increased government income and stimulation of local economy;*
  - *Employment and decrease in local unemployment levels;*
  - *An increase in new businesses and in sales;*
  - *Increased standards of living; and*
  - *Transfers of skills.*

Management procedures need to be put in place and implemented so that the negative impacts can be reduced and the positive impacts enhanced. The construction site should be isolated by the erection of temporary fencing in order to avoid stock and game losses. Fencing used during construction must still enable farmers to have access to their land and dwellings. Game fencing taken down during construction must be re-instated to the approval of the individual property owners. Workers' camps need to be planned for well in advance to ensure that various negative social impacts are curbed.

CROCODILE RIVER (WEST) TRANSFER SCHEME: KEY ISSUES TO BE ADDRESSED DURING DETAIL DESIGN

The following issues were identified during the course of the Feasibility stage and would require further investigation to ensure fit for purpose designs:

1. *It should be noted that since April 2008 there have been a number a changes to the parameters that could influence the capacity, location and design of the MCWAP and the Crocodile River (West) Transfer Scheme in particular. The pipe systems can also be optimised further when final design capacities, and more detailed survey and geotechnical information becomes available. It is therefore recommended that a more comprehensive evaluation and optimisation be performed during the detail design stage to verify the Feasibility findings.*

2. *Route planning and coordination. The following is required:*
   - *Detailed coordination and a commitment to the MCWAP by the bulk consumers along the Lephalale-Steenbokpan corridor in order to ensure integrated planning of infrastructure and water requirements. Eskom is planning to construct a number of high-voltage power lines through the region. A number of these will be located in a corridor routed from north to south that could affect the routing and design of the Crocodile River (West) Transfer Scheme pipelines.*
   - *Agree on the permanent servitude requirements to allow for future expansion.*
   - *Source detailed cadastral and existing services information along the final pipeline route alignments.*
   - *Facilitate and support the land acquisition and servitude registration process taking cognisance of issues raised by interested and affected parties during the public participation process.*
   - *Confirm the location of farmer off-takes and requirements.*
   - *Services coordination/way-leave approvals to be performed with:*
     - *Eskom – Capital projects planning, Transmission and Distribution;*
     - *Spoornet – Apply for permission to use railway line access road during construction and for future maintenance access to the pipeline and confirm future upgrade/electrification planning for the railway line;*
     - *South African National Roads Agency Limited (SANRAL) – Apply for a concession to use the road reserves as temporary construction servitudes where*

pipelines are located nearby. Also apply for access point to pipeline servitude from road reserve;

- *Limpopo Provincial Roads Department - Apply for a concession to use the road reserves as temporary construction servitudes where pipelines are located nearby. Also apply for access point to pipeline servitude from road reserve;*
- *Thabazimbi Local Municipality – Future township establishment that might affect pipeline routes;*
- *Lephalale Local Municipality – Future township establishment that might affect pipeline routes;*
- *Telkom – Confirm the location of services and apply for way-leaves to cross the services;*
- *Neotel – Confirm the location of services and apply for way-leaves to cross the services;*
- *Department of Minerals and Energy – Inform them of the planned pipeline route in order to update their database;*
- *Coordination with Department of Water Affairs (DWA) – Confirm the need to apply for water use licenses for river and stream crossings and obtain the necessary permission if required; and*
- *Local farmers – Take forward the land acquisition and servitude registration process to ensure that servitudes for the pipeline and borrow areas are agreed timeously to prevent delays during construction.*

3. *Abstraction Works: The following issues were identified during the course of the Feasibility stage and would require further investigation to ensure fit for purpose designs:*
    - *Depth of scour at Vlieëpoort during high floods. Scour potential at the weir must be modelled to confirm the depth of founding of the weir structure. The present Feasibility stage layout assumes that the proposed jet grouting foundation treatment will provide adequate founding conditions and that together with the roller bucket spillway design and extensive downstream heavy riprap protection will protect the structure against scour.*
    - *Foundation Design. Deep jet grouted foundations have been successfully used in the past to improve hydraulic structure founding conditions. Once the results of a detailed materials investigation are available, the layouts need to be reviewed and refined.*
    - *Alluvial aquifer flows at Vlieëpoort. The Feasibility stage layouts show that the entire river bed section below the weir will be jet grouted, thereby effectively blocking the flow in the aquifer. Whilst this arrangement is intended to prevent piping foundation failure, greater loads could be imposed on the weir foundations if the water table downstream of the weir is lowered. This can be counteracted if the flow past Vlieëpoort is regulated sufficiently to maintain a continuous flow over the*

weir. The water table level downstream of the weir should nevertheless be monitored continuously to alert the operators of any potentially dangerous situation.

- *Liquefaction potential. The nature of the underlying alluvial sands and silts at Vlieëpoort must be investigated to determine the potential for liquefaction during a natural or induced seismic event.*
- *Sizing and configuration of Desilting Channels. Feedback received on the operation of the Lebalelo Abstraction Works in the Olifants River in Limpopo Province indicated that the very fine fraction of the suspended silt in the Olifants River, when in flood, failed to completely settle out in the de-silting channels. This fraction requires longer retention times to settle out and therefore only settled in the balancing dams where it affected the operational availability of the system and was also difficult and time-consuming to remove, primarily because the balancing dams were not designed to be maintained at frequent intervals. In the case of the Crocodile River (West) Transfer Scheme the problem is accentuated by the relatively large storage capacity and retention times of the Balancing Dam.*
- *Location of High-Lift Pump Station Balancing Dam. The Feasibility layouts identified two potential sites for the dam. The site closest to the Abstraction Weir has since been confirmed to be located on dolomite and should therefore be avoided if possible. The preferred site is some 5 km downstream of the Abstraction Weir and on much more favourable founding conditions (residual Ventersdorp lava), but further planning is required to refine the layout and assess the socio-economic impacts.*
- *Sizing of the High-lift Pump Station Balancing Dam. The present approach is based on river flow management with a 3 to 4 day river flow response time from the upstream dams to Vlieëpoort. With improved control over flows in the river and shorter actual response times it is anticipated that the required capacity of the Balancing Dam should reduce accordingly. A storage capacity in the order of 200 000 m^3 less may be possible.*
- *Hydraulic computer modelling of the river is recommended once the detail survey becomes available. This model will allow for better computation of flood levels applicable to the base conditions and post-construction conditions and allow better assessments of the impact of the Abstraction Works on affected land owners and existing infrastructure.*
- *The hydraulic model will also provide flood levels downstream of the weir that are required for the placement of the Desilting Works, Balancing Dam, High-Lift Pump Station and switchyards and might also influence the choice of the site for these components.*
- *A prototype or CFD model of the Abstraction Weir, Gravel Trap and Low Lift Pump Station is recommended in order to optimise the placement, layout and size of these structures.*

- *During flushing of the Desilting works and desilting of the Balancing Dam, high amounts of silt need to be handled which cannot be discharged into the river. Further investigation is required to confirm environmental requirements and to identify appropriate silt separation facilities and storage and/or disposal thereof.*
- *Flows passing the Abstraction Weir must be measured. A downstream flow gauging structure will be required to measure surface flows since flows over the weir may not be uniform enough.*

4. *The following detail design and optimisation actions must also be performed:*
   - *Confirmation of the systems operating and control philosophy.*
   - *Review of the pump selection philosophy with specific reference to the option of implementing Variable Speed Drives and the associated implications it has on the operational control, power supply, etc.*
   - *A detailed pipeline design (optimum diameters and wall thickness). Consider both the interim (rising main/Operational and Break Pressure Reservoir/gravity mains) and ultimate (rising main directly to a new operational reservoir with the initial operational and break pressure reservoir converted to a surge reservoir) scenarios and perform the detailed surge analyses.*
   - *A detailed hydraulic analysis to determine the optimum positioning of the air (type and size), isolating, reflux, drainage and control valves. Pipeline dewatering will require careful consideration due to:*
     - *Potential poor water quality and fears of contamination; and*
     - *Very flat topography – management of scour water will be problematic. Scour time of the pipeline must be considered.*
   - *Optimum sizing of the Operational and Break Pressure Reservoirs to take cognizance of final operating philosophy and risk assessment. The detailed design of Operational and Break Pressure Reservoirs must consider operational storage requirements, storage time, and water quality management to prevent 'dead zones' in the reservoirs. The initial Operational and Break Pressure Reservoir must be configured to allow conversion to a surge tank during later phases of the development.*
   - *River and stream crossings – Matlabas River crossing will require careful consideration of geotechnical conditions at the site, environmental considerations and rehabilitation. The stream and river crossings of the pipelines should preferably coincide with the crossing of the railway line.*

5. *Pipeline coatings and linings: New pipeline coating and lining processes are becoming available on the market and must be considered. A detailed corrosion protection design will be required.*

6. *Detailed AC mitigation design:*

- *Cognisance of possible future infrastructure that might affect the design.*
- *A detailed soil resistivity survey at 100 to 500 m centres, depending on soil conditions.*
- *Soil sampling and analysis to confirm the aggressiveness of the soil and the possible presence of Sulphate Reducing Bacteria (SRB) that could affect the coating selection.*
- *Detailed AC modelling to confirm the extent of AC mitigation required.*

7. *Detailed Geotechnical Investigation:*
   - *Geological mapping - Delineation and description of outcrop areas including discontinuity survey, geological structures, etc.*
   - *Test pitting with an excavator at selected spots at an average of about 200 m centres – The maximum depth of the proposed pipeline is generally more than 4 m, deeper than the reach of a tractor loader backhoe (TLB). The soil profile must be described according to the standard method of Jennings et. al. with reference to shallow water table conditions, excavatibility, etc.*
   - *Core drilling – to investigate pipe jacking and reservoir sites.*
   - *In situ testing - For the determination of soil parameters for pipeline design (the empirical E' value (bulk modulus of horizontal soil reaction), limited plate load tests must be conducted at selected representative positions.*
   - *Sampling and Laboratory testing – Disturbed and undisturbed samples of selected representative soil horizons must be collected and tested at an SABS approved laboratory to determine the soil characteristics such as grading, expansiveness, collapse, potential use for backfill, indicators, etc.*
   - *The corrosiveness of the material must be determined by analysing the pH and electrical conductivity of selected samples.*
   - *Identification and proving of potential borrow sites – Borrow sites to be identified to ensure that haul distances are kept to a minimum. The volume of borrow material to be proven by a dense grid survey and adequate laboratory testing, providing at least twice the volume required at each site.*
   - *Field electric resistivity survey - A field survey must be conducted to determine the in situ electrical resistivity along the entire route in collaboration with the Cathodic Protection analysis and design.*

MCWAP INFRASTRUCTURE SUMMARY

A summary of the infrastructure components comprising the combined MCWAP is given in the following table.

Mokolo and Crocodile River (West) Project – Summary of Infrastructure Components

Component	***Description***
Mokolo Dam Scheme (Phase 1)	
Phase 1	*New pumping station and additional pipeline from Mokolo Dam to end-users located from Lephalale in the east to Steenbokpan in the west. The Lephalale-Steenbokpan link will be built as part of Phase 1 but will ultimately form part of the Crocodile River (West) Transfer Scheme to transport Crocodile River (West) water to Medupi and the Grootegeluk/Matimba control chamber.*
Lephalale-Steenbokpan Link *Crocodile River (West) Connection to Steenbokpan* *Crocodile River (West) Connection to CF 3&4 Mining T-off* *CF 3&4 Mining to Medupi T-off* *Medupi T-off to Steenbokpan T-off*	*Diameter : 1900 mm ND* *Length : 1.4 km* *Diameter : 1100 mm ND* *Length : 27.1 km* *Diameter : 900 mm ND* *Length : 3.6 km* *Diameter : 900 mm ND* *Length : 8.2 km*
Crocodile River (West) Transfer Scheme (Phase 2A)	
Vlieëpoort Abstraction Works	*Concrete weir, gravel trap and pump intake structure– civil structures sized for ultimate project water requirements (431 Million m^3/a)* *1 x fully equipped standby bay plus 1 standby pump unit (stored on site)* *8 x 1.0 m^3/s submersible pumps* *Maximum duty point: 6.6 m^3/s @ 49.5 m* *Absorbed Power: 4.7 MVA* *1 300 000 m^3 active balancing storage*
High-lift pump station	*Static head : 183-192 m* *Design peak flow (DPF) : 5.8 m^3/s* *Min manometric head at DPF : 216 m* *Recovery peak flow (RPF) : 6.6 m^3/s* *Max manometric head at RPF : 255 m* *Power consumption DPF/RPF : 16/19 MW*
Pipelines *Rising main – High-lift pump station to Operational and Break Pressure Reservoir (Node 10)* *Break Pressure and Operational*	*Diameter : 1900 mm ND* *Length : 26.7 km* *8 hrs storage of recovery peak flow : 190 000 m^3*

Component	*Description*		
Reservoir	*rate*		
Operational and Break Pressure	*Diameter*	*:*	*2200 mm ND*
Reservoir to Node 15	*Length*	*:*	*62.7 km*
to Crocodile River (West) Transfer	*Diameter*	*:*	*2300 mm ND*
Scheme Connection (Phase 2A)	*Length*	*:*	*28.2 km*
Steenbokpan T-off to	*Diameter*	*:*	*800 mm ND*
Grootegeluk/Matimba Control Chamber	*Length*	*:*	*1.9 km*

MCWAP COST ESTIMATE

The cost estimates considered the following:

- *Capital costs;*
- *Energy costs;*
- *Operations and maintenance costs; and*
- *Raw water costs.*

The total capital cost for the Mokolo and Crocodile River (West) Water Augmentation Project is summarised below. The capital cost estimate includes the costs of Phases 1 and 2. The cost includes infrastructure, preliminary and general (P&G's), contingencies and design fees and excludes Value Added Tax (VAT). The base date for the cost estimate is April 2008.

Mokolo and Crocodile River (West) Project Capital Cost Estimate

Component		*Total (R)*
Mokolo Dam Scheme – Phase 1		
1.1	*Pump Station (Maximum duty 1.5 m^3/s @ 263 m)*	
	- Civil Works	*64 805 000*
	- Mechanical & Electrical Work	*70 770 000*
1.2	*Rising Main*	
	- 900 mm diameter (5 700 m)	*86 540 000*
1.3	*Gravity Mains(2)*	
	- 1 900 mm diameter (1 400 m)	*51 243 000*
	- 1 100 mm diameter (42 950 m)	*692 875 000*
	- 1 000 mm diameter (19 970 m)	*308 070 000*
	- 900 mm diameter (11 770 m)	*152 910 000*
	- 800mm diameter (1 940 m)	*27 544 000*
1.4	*Eskom Electricity to Site*	*76 430 000*
1.5	*Compensation*	*2 170 000*

Component		Total (R)
1.6	*Environmental and Socio-economic*	*1 000 000*
	Sub Total	***1 534 357 000***
Crocodile River (West) Transfer Scheme - Phase 2		
2.1	*Abstraction Weir and Low Lift Pump Station Civil Works*[(3)]	*247 983 890*
2.2	*Low-Lift Pump Station M&E Works* [(4)]	*74 073 110*
2.3	*Rising Main to De-silting Works*	*171 573 000*
2.4	*Desilting Works*	*86 148 000*
2.5	*High-Lift Pump Station Balancing Dam*	*318 909 000*
2.6	*High-Lift Pump Station (Maximum duty 6.6 m^3/s @ 255 m)*	*350 544 000*
2.7	*Rising Main*[(5)] *- 1 900 mm diameter (26 700 m)*	 *1 263 545 000*
2.8	*Gravity Mains (not constructed under Phase 1)*[(6)] *- 2 200 mm diameter (62 700 m)* *- 2 300 mm diameter (28 200 m)* *- 800 mm diameter (1 940 m)*	 *3 464 072 000* *1 440 550 000* *28 110 000*
2.9	*Operational and Break Pressure Reservoir*	*118 964 000*
2.10	*Eskom electricity to Vlieëpoort site*[(7)]	*156 564 000*
	Sub Total	***7 721 036 000***
	TOTAL COMBINED CAPITAL COST – MOKOLO AND CROCODILE RIVER (WEST) WATER AUGMENTATION PROJECT (Phases 1 and 2A)	***9 255 393 000***

Notes:

1. *The residual value of the existing pump station at Mokolo Dam, as well as the existing pipeline between Mokolo Dam and Matimba was calculated as R8 million and R33 million, respectively. These costs were added to the project capital cost in the engineering economic analysis.*
2. *Includes the Lephalale-Steenbokpan link sized for the ultimate scheme requirements*
3. *The costs of pipework, valves, screens and craneage have been included in the civil works portions of the cost estimate.*
4. *Only includes for the costs of the pumps and any M&E control equipment required as well as any pipework and valve items directly associated with the pump installations.*
5. *At the 2050 Scenario 9 flows, the retention time in the Balancing Dam compartments is reduced to 5 hours. Under these circumstances retention times can be increased by adding two more compartments and running the compartments in tandem.*
6. *Rising main from the High Lift Pump Station to the Operational and Break Pressure Reservoir*
7. *Includes the gravity pipeline sections from the Operational and Break Pressure Reservoir to the Crocodile River (West) Connection near Steenbokpan, as well as the connection from the Steenbokpan tee-off to the Matimba control chamber required to prevent mixing Crocodile River (West) and Mokolo Dam water. The remainder of the Lephalale-Steenbokpan link will be built under Phase 1.*
8. *Includes for the bulk electrical supply to High Lift Pump Station and the Low Lift Pump Station.*

The following table summarise the annual operation and maintenance costs, when the scheme is operating at maximum capacity (2030), excluding overhaul costs of pump stations and VAT.

Mokolo and Crocodile River (West) Project Annual Operations and Maintenance Costs

	Component	Total (R)/a
	Mokolo Dam Scheme – Phase 1	
	New Phase 1 Works	
1.1	*Pump Station*	
	- Civil Works	*141 000*
	- Mechanical & Electrical	*2 462 000*
	- Electricity	*14 131 000*
1.2	*Rising Main*	*376 000*
1.3	*Gravity Mains*	*5 359 000*
	Existing Exxaro Works	
2.1	*- Civil*	*6 000*
	- Mechanical & Electrical	*223 000*
2.2	*Pipeline*	*165 000*
3.1	*Raw Water Costs* (1)	*58 571 000*
	Sub Total	***81 434 000***
	Crocodile River (West) Scheme - Phase 2	
4.1	*Abstraction Weir, Low-Lift Pump Station, De-silting Works and Balancing Dam*	
	- Civil	*995 000*
	- Mechanical & Electrical	*2 035 000*
	- Electricity	*17 336 000*
4.2	*High-Lift Pump Station (Maximum duty 6.6 m^3/s @ 255 m)*	
	- Civil	*87 000*
	- Mechanical & Electrical	*4 797 000*
	- Electricity	*76 866 000*
4.3	*Rising Main*	*3 018 000*
4.4	*Gravity Mains (not constructed under Phase 1)*	*11 848 000*
4.5	*Operational and Break pressure Reservoirs*	*308 000*
4.6	*Raw water costs*	*1 142 408 000*
	Sub Total	***1 317 098 000***

Component		Total (R)/a
5	***Annual River Management Cost***	*4 500 000*
	TOTAL COMBINED ANNUAL O&M COST (2030) – MOKOLO AND CROCODILE RIVER (WEST) WATER AUGMENTATION PROJECT	***1 345 632 000***

(1) Raw water priced at R4,50/m^3

MOKOLO AND CROCODILE RIVER (WEST) WATER AUGMENTATION PROJECT FEASIBILITY STUDY:

WATER TRANSFER SCHEME

FEASIBILITY STAGE PHASE 2

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DWA	Department of Water Affairs
AC	Alternating Electrical Current
BPR	Break Pressure Reservoir
CFD	Computational Fluid Dynamics
CFRD	Concrete Faced Rockfill Dam
CL	Centre Line
CML	Cement Mortar Lining
CP	Cathodic Protection
CRW	Crocodile River (West)
CTL	Coal to Liquid Fuel
DCVG	Direct Current Voltage Gradient
DFL	Design Flood Level
DFT	Dry Film Thickness
DPF	Design Peak Flow
DRA	Definition Release Approval
EFR	Environmental Flow Requirement
EIA	Environmental Impact Assessment
Em	Efficiency of Motor
Ep	Efficiency of Pump
FBC	Fluidised Bed Combustion
FBE	Fusion Bonded Epoxy
FGD	Flue Gas Desulphurisation
FSL	Full Supply Level
GAAR	Gross Average Annual (water) Requirement
IFR	Instream Flow Requirements
IPP	Independent Power Producer
LV	Low Voltage
MAR	Mean Annual Runoff
M&E	Mechanical and Engineering
MOL	Minimum Operating Level
MCWAP	Mokolo and Crocodile (West) Water Augmentation Project
MV	Medium Voltage
MVA	Mega Volt Amperes
ND	Nominal Diameter
NDU	Natural Drain Unit
NOC	Non-overspill Crest
NPSH	Net Positive Suction Head
OC	Overspill Crest
O &M	Operation and Maintenance
OPC	Ordinary Portland Cement

PE	Polyethylene
Pf	Power Factor
P&G	Preliminary and General
PMF	Probable Maximum Flood
PSP	Professional Service Provider
PVC	Polyvinyl Chloride
RDF	Recommended Design Flood
RI	Recurrence Interval
ROD	Record of Decisions
RPF	Recovery Peak Flow
SANCOLD	South African National Committee on Large Dams
SANRAL	South African National Roads Agency Limited
SCADA	System Control and Data Acquisition
SEF	Safety Evaluation Flood
SRB	Sulphate Reducing Bacteria
STD	Sexually Transmitted Disease
TLB	Tractor Loader Backhoe
TRU	Transformer Rectifier Unit
TWL	Top Water Level
URV	Unit Reference Value
VAT	Value Added Tax
VSD	Variable Speed Drive
WMA	Water Management Area
WTW	Water Treatment Works

1. INTRODUCTION AND BACKGROUND

Development from Lephalale westwards towards Steenbokpan and the Botswana border is driven by large coal deposits. Potential large users (Eskom, Exxaro and Sasol) are planning significant development in the area and have provided estimates of their expected water consumption for the interim to long-term industrial, commercial and domestic use. The Mokolo Dam (formerly known as the Hans Strijdom Dam) was completed in July 1980, to supply water to Matimba Power Station, Grootegeluk Mine, Lephalale (Ellisras) Municipality and for irrigation downstream of the dam. Based on the water infrastructure, the current water availability and water use, only limited spare yield exists for future allocations for the anticipated surge in economic development in the area.

The primary purpose of the Mokolo and Crocodile River (West) Water Augmentation Project (MCWAP) is therefore to develop the options to transfer water from the Mokolo and Crocodile River (West) to the Lephalale area to supply the domestic, commercial and industrial users in this fast developing area.

Various options have been identified to convey water to the end users. These include the Mokolo Scheme, as well as the Crocodile River (West) (CRW) Transfer Scheme to be operated in combination as the MCWAP Project. The Mokolo Scheme is intended to supply the interim water requirements for a period until the CRW Transfer Scheme has been constructed and to improve the reliability and provide redundancy once the Crocodile River (West) Transfer Scheme is operational.

The combined MCWAP Project is illustrated by **Figure 1-1**, showing the terminology adopted for different components forming part of the combined project. The infrastructure components associated with the different schemes are described in more detail later in this report, as well as in other supporting reports listed in the front of this document. A locality plan is included in **Appendix A** of the report (DWG No WP 9528/LD/CTS/001).

The project will be implemented in phases as follows:

- Phase 1 (Mokolo Scheme): Augment the supply from Mokolo Dam to supply in the growing water requirement for the interim period until a transfer pipeline from the CRW can be implemented. The solution must over the long term, optimally utilise the full yield from Mokolo Dam.
- Phase 2A (CRW Scheme): Transfer system from the CRW to the Lephalale and Steenbokpan area where the pipeline will link to the infrastructure constructed as part of Phase 1. Phase 2 will also be implemented in stages as follows:
    - Phase 2A - Infrastructure as defined in this report and which will be implemented now, catering for a design horizon up to 2030; and
    - Phase 2B - Later upgrading of the CRW Transfer Scheme to cater for developments after 2030 (booster pump station(s) and/or parallel

pipeline(s)). The extent of future phases will be determined by the future water requirements.

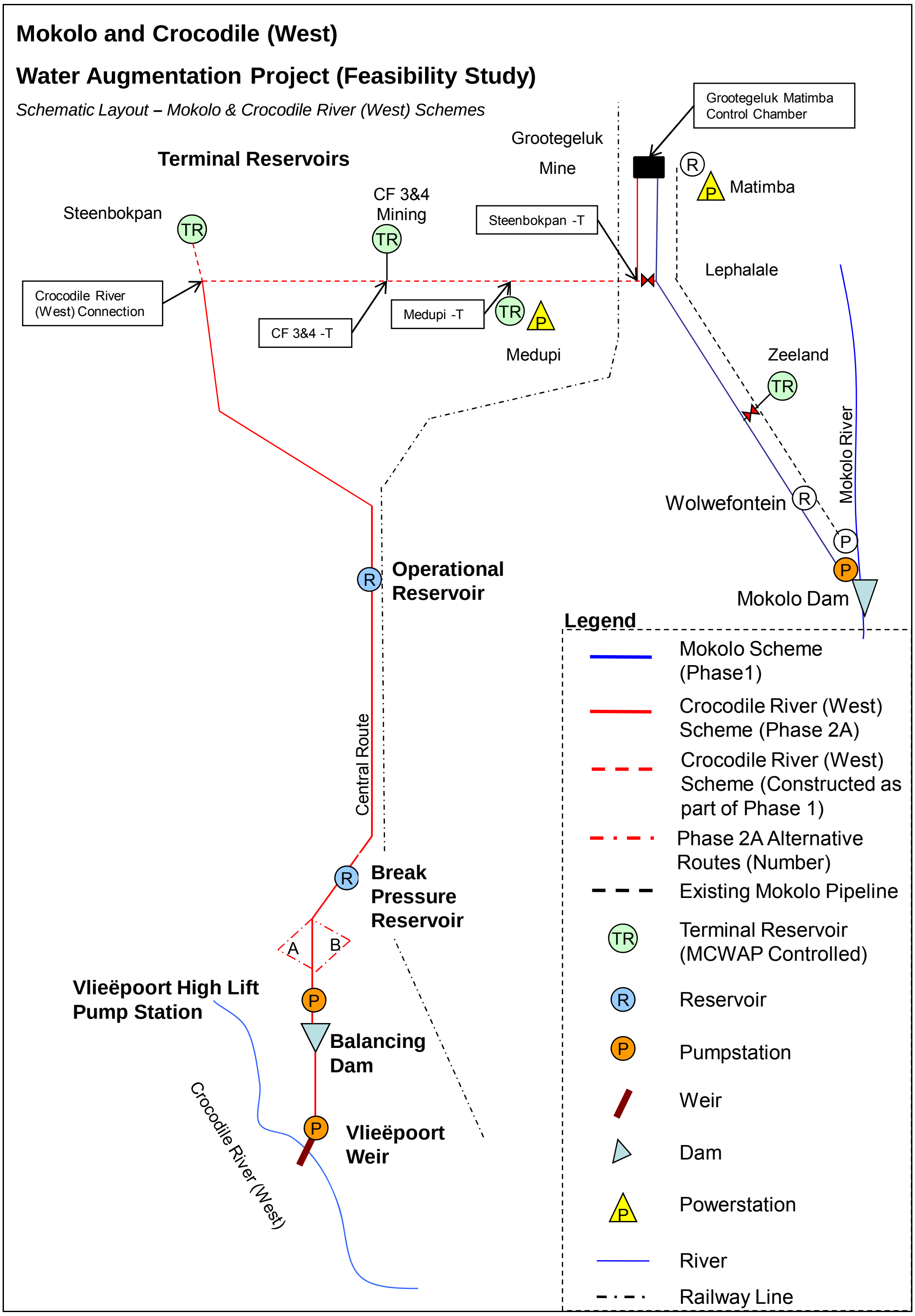


Figure 1-1: Schematic Layout of the MCWAP

A summary of the infrastructure components comprising the combined MCWAP is given in **Table 1-1**. More detailed descriptions of the various components are provided later in the report.

Table 1-1: MCWAP – Summary of Infrastructure Components

Component	**Description**
Mokolo Dam Scheme (Phase 1)	
Phase 1	New pumping station and additional pipeline from Mokolo Dam to end-users located from Lephalale in the east to Steenbokpan in the west. The Lephalale-Steenbokpan link will be built as part of Phase 1, but will ultimately form part of the CRW Transfer Scheme to transport CRW water to Medupi and the Grootegeluk/Matimba control chamber.
Lephalale-Steenbokpan Link CRW Connection to Steenbokpan CRW Connection to CF 3&4 Mining T-off CF 3&4 Mining to Medupi T-off Medupi T-off to Steenbokpan T-off	 Diameter : 1 900 mm ND Length : 1.4 km Diameter : 1 100 mm ND Length : 27.1 km Diameter : 900 mm ND Length : 3.6 km Diameter : 900 mm ND Length : 8.2 km
Crocodile River (West) Transfer Scheme (Phase 2A)	
Vlieëpoort Abstraction Works	Concrete weir, gravel trap and pump intake structure– civil structures sized for ultimate project water requirements (431 Million m^3/a) 1 x fully equipped standby bay plus 1 standby pump unit (stored on site) 8 x 1.0 m^3/s submersible pumps Maximum duty point: 6.6 m^3/s @ 49.5 m Absorbed Power: 4.7 MVA 1 300 000 m^3 active balancing storage
High-Lift Pump Station	Static head : 183-192 m Design peak flow (DPF) : 5.8 m^3/s Min manometric head at DPF : 216 m Recovery peak flow (RPF) : 6.6 m^3/s Max manometric head at RPF : 255 m Power consumption DPF/RPF : 16/19 MW
Pipelines Rising main – High-Lift Pump Station to Operational and Break Pressure Reservoir (Node 10) *Break Pressure and Operational Reservoir*	 Diameter : 1 900 mm ND Length : 26.7 km 8 Hours storage of recovery peak flow rate : 190 000 m^3

Component	Description		
Operational and Break Pressure Reservoir to Node 15	Diameter Length	: :	2 200 mm ND 62.7 km
to CRW Transfer Scheme Connection (Phase 2A)	Diameter Length	: :	2300 mm ND 28.2 km
Steenbokpan T-off to Grootegeluk/Matimba Control Chamber	Diameter Length	: :	800 mm ND 1.9 km

During the Pre-feasibility stage, a number of options were identified and investigated to implement Phases 1 and 2A. The findings of the pre-feasibility investigations are described fully in Report No. 5 - Technical Module: Mokolo River Development Options and Report No. 6 – Technical Module: Water Transfer Scheme Options. A brief summary of the findings is provided in Section 2.

The objective of this report is to refine the work done in the Pre-Feasibility stage and to develop this option to a sufficient level of detail to confirm the technical feasibility and environmental acceptability, as well as compiling a revised project cost estimate.

2. PRE-FEASIBILITY STAGE FINDINGS

The MCWAP will be implemented in phases. Phase 1 is intended to augment the supply to Lephalale and Steenbokpan from Mokolo Dam, for the interim period until the CRW Transfer Scheme is operational. Phase 2A is a transfer scheme from the CRW to the Lephalale and Steenbokpan area where the pipeline will link to the infrastructure constructed as part of Phase 1. Phase 2A is planned to meet the water requirements up to 2030 (refer to Section 3). Further augmentation from the CRW to the Lephalale and Steenbokpan regions will be determined by the future water requirements and further augmentation possibilities in the CRW and will be implemented as future phases (Phase 2B).

Numerous options were evaluated during the Pre-feasibility stage to implement Phases 1 and 2A. The options and findings are summarised below.

2.1. Phase 1: Augmentation Scheme from Mokolo Dam

During the Pre-Feasibility stage, the following options of augmenting the existing supply from the Mokolo Dam to the consumers were identified and investigated:

- Construct a pump station and new pipeline from Mokolo Dam to Zeeland Water Treatment Works (WTW), Matimba and Medupi Power Stations, as well as Steenbokpan (to supply the development of further Eskom power stations, Sasol, and coal mining activities). This pipeline will be constructed parallel (or close) to the existing pipeline for much of the route up to the Matimba terminating chamber.
- Construct a weir in the Mokolo River downstream of the Mokolo Dam with a pipeline to Zeeland WTW, Matimba and Medupi Power Stations and Steenbokpan.

The Pre-Feasibility investigation concluded that the weir option be discarded and that the option to construct a pipeline from Mokolo Dam be investigated further at Feasibility level. Refer to Supporting Report No. 5 (P RSA A000/00/9209) for the Mokolo River Development Options Pre-Feasibility Investigation.

2.2. Phase 2A: Transfer Scheme from the Crocodile River (West)

During the Pre-Feasibility stage, the following options for abstracting and transferring water from the Crocodile River (West) to the demand area were investigated:

- A number of different weir and abstraction sites along the CRW.
- Three potential pipeline routes, i.e. East, Central and West.
- Terminal and/or on-site storage options with adequate storage capacity to provide balancing capacity and emergency storage. These included:
  - Terminal Dam options combined with Terminal Reservoirs at the end user sites; or

- Alternatively, an Operational Reservoir (located on the farm Rooipan 357 LQ), providing short-term balancing storage along the gravity supply main to the end user Terminal Reservoirs.

A weir and abstraction works at Vlieëpoort and supply to the end users via the central pipeline route was identified during the Pre-Feasibility stage as the most viable option and will be investigated further in this report. The option of pumping directly to the Operational Reservoir or pumping to a Break Pressure Reservoir (BPR) located near Thabazimbi with gravity supply to the Operational Reservoir will also be analysed further as part of the Feasibility investigation. Refer to Report No. 6 – Technical Module: Water Transfer Scheme Options for the Pre-Feasibility Investigation of the Water Transfer Scheme Options and Report No. 4 – Technical Module: Dams, Abstraction Weirs and River Works for the Pre-Feasibility investigation of the Dams, Abstraction Weirs and River Works.

3. WATER REQUIREMENTS AND DESIGN FLOW

The water requirements, design parameters and principles for sizing of MCWAP infrastructure are described in this section under the following topics:

- Projected water requirements for the project;
- Reliability and redundancy; and
- Design horizon and derivation of design flow.

3.1. Projected Water Requirements

Development from Lephalale westwards towards Steenbokpan and the Botswana border is driven by large coal deposits. Potential large users (Eskom, Exxaro and Sasol) have provided estimates of their expected water consumption for the immediate to long-term industrial, commercial and domestic use. The Department of Water Affairs' (DWA) Regional Office in Polokwane also commissioned a study that quantified the expected water use of the Lephalale Local Municipality as a result of the expected growth in the area.

Two water requirement scenarios were analysed at Pre-Feasibility stage for the period up to 2030:

- Scenario 4 – Matimba Power Station equipped with existing fluidised bed combustion (FBC) technology, Medupi Power Station equipped with flue gas desulphurisation (FGD) technology, three (3) new additional power stations (FGD), coal supply to five (5) power stations, Exxaro projects, the associated construction activities and the associated growth in Lephalale and Steenbokpan; and
- Scenario 8 – Scenario 4: Sasol development of two coal to liquid fuel (CTL) plants and the associated mine and construction activities, as well as the associated population growth in Steenbokpan.

In February 2009, updated water requirements were released and Scenario 8 was superseded by Scenario 9 which was subsequently used for the Feasibility stage investigation. The detailed water requirement calculation sheet is included in **Appendix B**.

Scenario 9 incorporates the following water requirements:

- **Eskom**: Matimba (FBC), Medupi and four additional coal fired power stations (the FGD retrofit for Medupi was scheduled for the first major shutdown);
- **Independent Power Producers (IPP's)**: Equivalent of one (1) Eskom power station (starting in July 2010);
- **Exxaro**: Matimba and Medupi coal supply, as well as implementation of projects 'A to K' (new coal mines);

- **Coal mining**: Allowance for four (4) additional coal mines (to the ones already operation) to supply planned future power stations;
- **Sasol**: Mafutha one (1) CTL plant and associated coal mine (starting in July 2011); and
- **Lephalale and Steenbokpan**: Estimate based on projected growth in households for construction and permanent workforce.

The comparison between the Scenario 8 and 9 water requirements are illustrated by **Figure 3-1** and the relative contributions by the respective users for Scenario 9 are summarised in **Table 3-1** and illustrated by **Figure 3-2**.

All infrastructures, apart from the abstraction works, will be sized for the 2025 development, but taking into account the growth associated with projects not fully commissioned by 2025. Further projects after 2025 will require expansion of the Phase 2A infrastructure capacity. The Scenario 9 water requirements are therefore reported up to 2030 and the system capacity is also based on the 2030 requirement. The abstraction works will be sized for the planned ultimate water requirement in 2050 as illustrated by **Figure 3-5**.

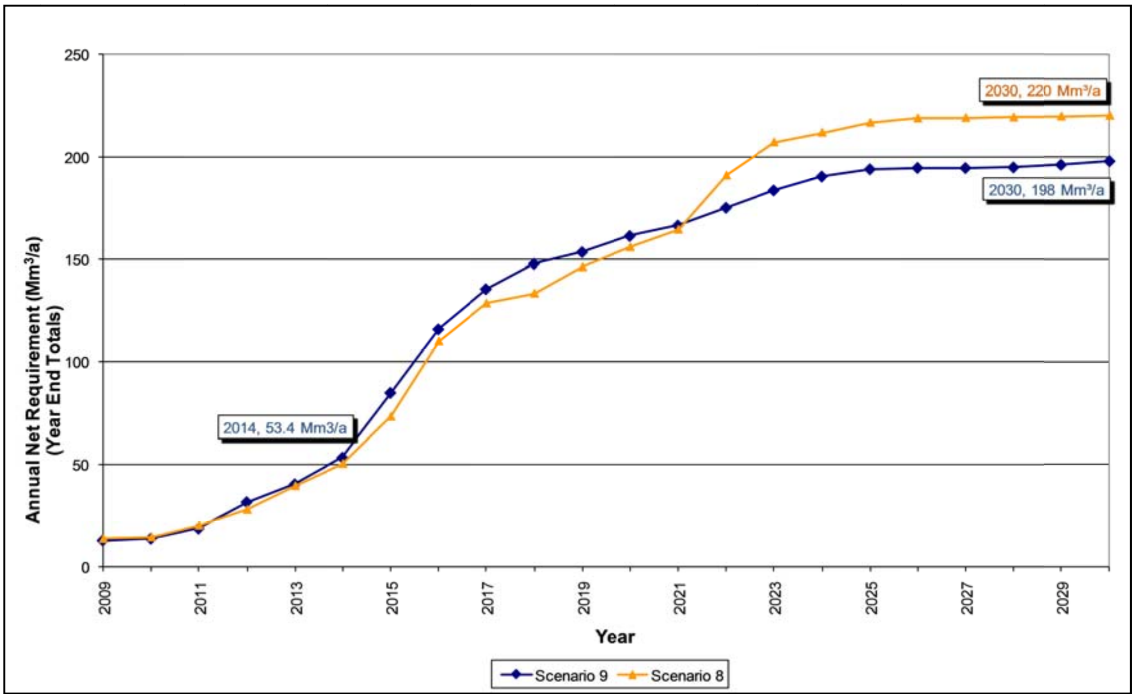


Figure 3-1: Comparison between Scenario 8 and Scenario 9 Water Requirements (excluding Irrigation)

Table 3-1: Scenario 9: Water Requirement Projection per Major User Group

	Annual Water Requirement (Million m³/a)									
Year	**2009**	**2010**	**2011**	**2012**	**2013**	**2014**	**2015**	**2020**	**2025**	**2030**
Eskom	4.3	4.3	4.9	6.8	9.3	10.9	14.3	50.9	77.6	77.6
IPPs	0.0	0.4	0.9	0.9	1.5	4.4	13.2	15.6	15.6	15.6
Coal Mining (Power Generation)	0.0	0.0	1.1	2.7	4.4	5.3	6.8	14.1	20.0	20.0
Exxaro Projects	3.0	3.2	3.7	4.7	6.6	9.2	10.8	16.9	16.2	19.2
Sasol (Mafutha 1)	0.0	0.0	0.4	6.1	6.6	9.9	25.2	43.5	43.5	44.0
Municipality	5.6	5.9	7.7	10.4	12.0	13.6	14.5	20.4	21.2	21.6
Total	**12.9**	**13.8**	**18.7**	**31.7**	**40.4**	**53.4**	**84.8**	**161.4**	**194.1**	**198.0**
Irrigation	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4
Total + Irrigation	**23.3**	**24.2**	**29.1**	**42.1**	**50.8**	**63.8**	**95.2**	**171.8**	**204.5**	**208.4**

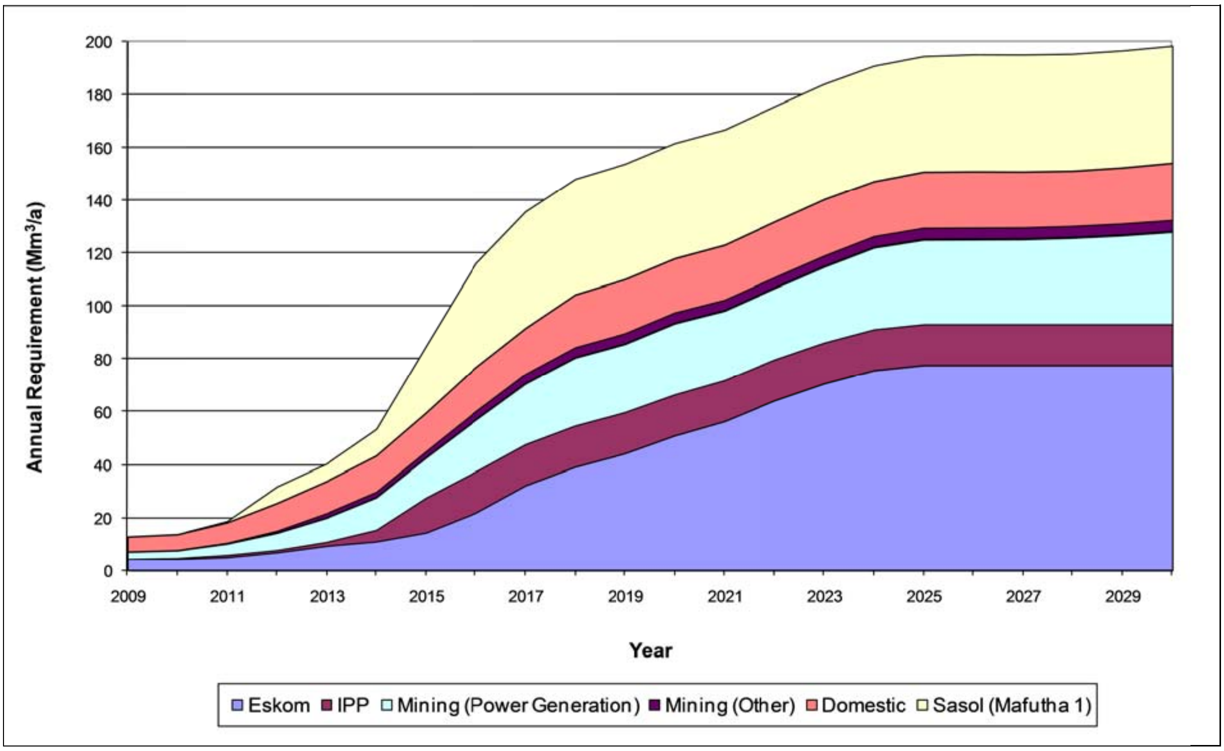


Figure 3-2: Scenario 9: Water Requirement Projection per Major User Group (excluding Irrigation)

The expected annual peak requirements on the system are illustrated by **Figure 3-3**. Peak flow requirements have been applied on the monthly water requirement for Eskom and the Lephalale Municipality. The peak factor included for Eskom is based on historic measurements at Matimba Power Station, which indicates that a 25% peak is experienced annually from August to October. Monthly peaks included for Lephalale Municipality are based on historic flow measurements taken at Zeeland WTW.

The resultant monthly peak flow requirement, based on the annual average daily demand for the total scheme, is 9%.

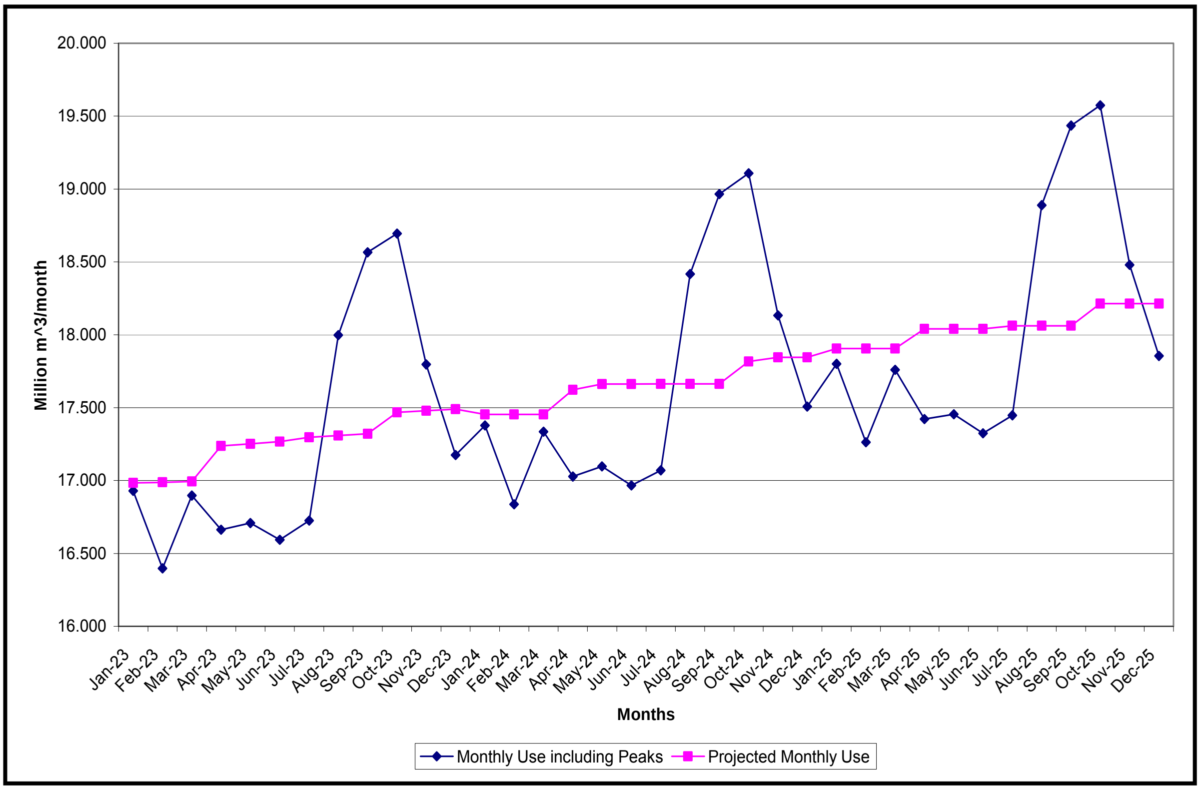


Figure 3-3: Annual Peak Water Requirements

3.1.1. Infrastructure Planning and Sizing Philosophy

The infrastructure sizing philosophy implemented for the project took cognisance of the following aspects:

- Growth in water requirements and the need to deliver water to key capital projects within a specified time frame (i.e. commissioning of Eskom Medupi Power Station units);
- Minimum implementation timeframe for the Phase 2A infrastructure;
- Yield analysis of the Mokolo Dam. The water resources, long-term sustainable yield and various scenarios of over-utilising the Mokolo Dam for an interim period until Phase 2A can be commissioned, resulting in the drawdown of the dam level and even a possible impact on irrigation downstream of the dam is described fully in Supporting Report No. 2 (P RSA A000/00/8909). The system modelling is continuously updated and revised to feed the latest information into the planning processes;
- Reliability and redundancy of the scheme as a whole; and
- Practical considerations such as delivery location and water quality.

The planning and sizing of the Phases 1 and 2A infrastructure components followed an integrated approach whereby the interim water requirements of the region must be met by the infrastructure implemented as part of Phase 1 and the ultimate water requirements by Phases 1 and 2A, operating as a combined system. Based on the above, the net water requirements for the respective phases of the project were calculated as follows.

Phase 1:

- Net water requirement at the time of commissioning Phase 2A (2014) = 53.4 Million m^3/a.
- Peak capacity of the existing Exxaro pipeline = 17.98 Million m^3/a.
- Average capacity of the existing Exxaro pipeline (incorporating 20% reliability allowance and 2% losses) = 14.7 Million m^3/a.
- Average capacity of the Mokolo System following improvement of the first 9.0 km (incorporating 20% reliability allowance and 2% losses) = 18.8 Million m^3/a.
- Maximum interim water requirement from Mokolo Dam (2014) to be supplied via new pipeline: 53.4-14.7 = 38.7 Million m^3/a.
- Long term yield of the Mokolo Dam = 39.1 Million m^3/a.
- Irrigation allocation = 10.4 Million m^3/a.
- Long term supply from the Mokolo Dam: 39.1-10.4 = 28.7 Million m^3/a (at the dam, including losses).
- Losses on the Mokolo Scheme will result in a slight reduction in the volume of water ultimately delivered to end users. These losses are allowed for in the transfer capacity of the CRW Transfer Scheme to ensure that the total net water requirements of all end users are supplied by the combined project.

As stated in Supporting Report No. 11 (P RSA A000/00/8209), no additional allowance was made for seasonal peak factors for the calculation of the design flows on Phase 1 since these peaks will be absorbed by the reliability peak factor. The highest summer peak factor calculated for 2014 is approximately 19%, which is less than the 20% reliability peak accepted by the major stakeholders. It should be noted that the pipelines from Mokolo Dam will only operate at its design flow capacity for the last few months of 2014. After this date, the Phase 2A pipelines will be operational, resulting in the water requirement from the Mokolo Dam reducing to 28.7 Million m^3/a. The additional capacity built into the Mokolo Scheme will ensure that the total yield of the Dam can be transferred via the new pipeline, while the existing pipeline is refurbished. In addition, it provides additional redundancy to the greater MCWAP in an emergency situation when the dam can again be over-utilised for a limited time period.

Phase 2A

- 2030 net water requirement = 198 Million m^3/a.
- Long term augmentation from the Mokolo Dam = 28.7 Million m^3/a.
- Ultimate annual Phase 2A transfer capacity: 198 – 28.7 = 169.3 Million m^3/a (excluding system losses and reliability and redundancy requirements).

- The supply from the CRW must make allowance for losses on both the Mokolo and Crocodile River (West) Schemes resulting in a slight increase in the required transfer capacity of the CRW Scheme.

The combined net water requirement and planned transfer capacity of the project is illustrated by **Figure 3-4**.

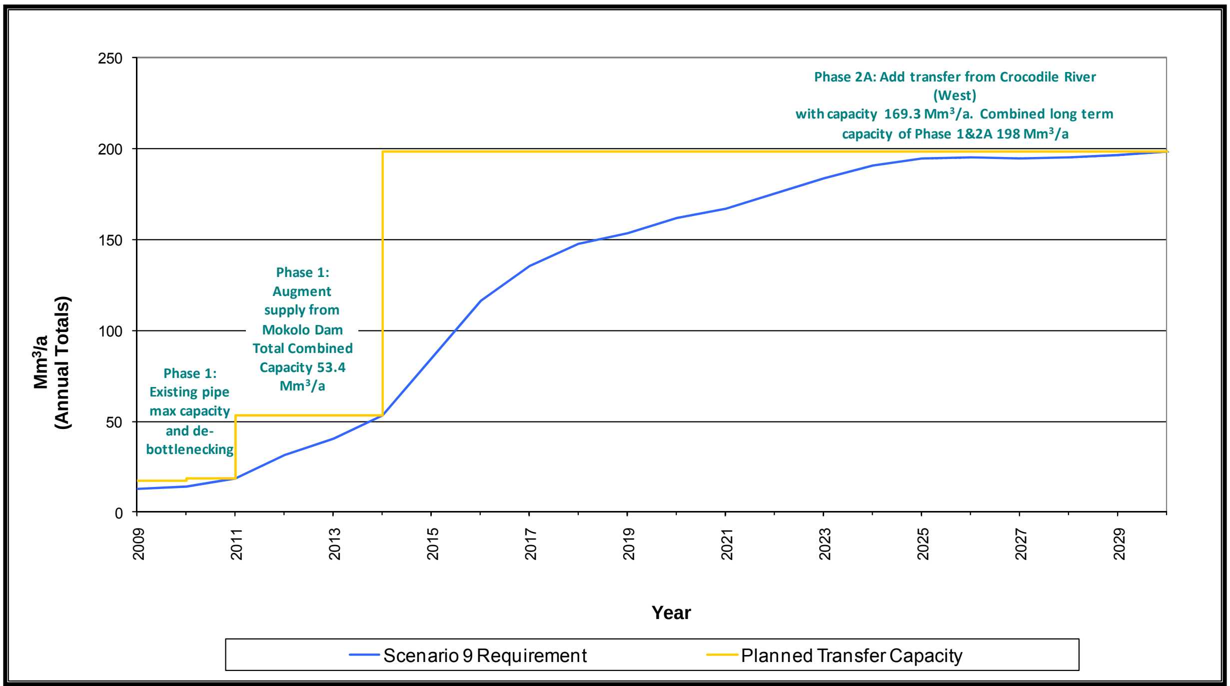


Figure 3-4: Mokolo and Crocodile River (West) Combined Net Water Requirement and Planned Project Transfer Capacity

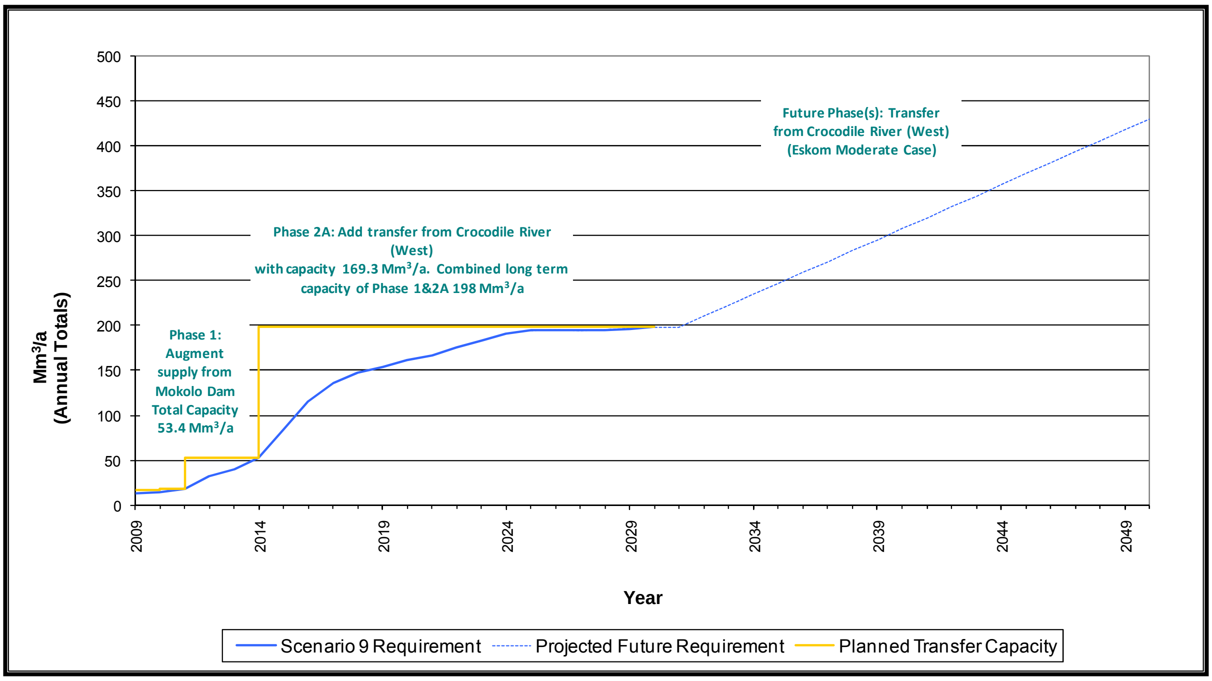


Figure 3-5: Mokolo and Crocodile River (West) Scenario 9 Ultimate Water Requirement

At the time of compiling this report, the water requirements were under review again. It is recommended that the detail design Professional Service Provider (PSP) review the above water requirements to take into account any changes made by the users since February 2009 when Scenario 9 was published.

Return flows (estimated to be approximately 10.8 Million m^3/a in 2030) were not included as a potential source in the estimated transfer capacities given above.

The expected annual peak requirements on the system were analysed as part of the Pre-Feasibility investigation. Peak flow requirements have been applied to the monthly water requirement for Eskom and Lephalale Municipality. The peak factor included for Eskom is based on historic measurements at Matimba Power Station, which indicates that a 25% consumption peak is experienced annually from August to October. Monthly peaks included for Lephalale Municipality are based on historic flow measurements taken at Zeeland WTW.

The resultant monthly peak flow requirement based on the annual average daily requirement for the combined scheme is 9%. The potential impact of peak factors on the design and operation of the scheme is discussed in more detail under Section 3.2.2.

The design capacity of the Phases 1 and 2A infrastructure components must incorporate certain criteria to ensure reliability and redundancy of supply. These criteria and the calculation of the required design flows are described in the following section.

3.2. Reliability and Redundancy

The strategic importance of the users that will account for the bulk of the water consumption requires that the risk of failure in the supply of water be kept to a minimum. Sufficient reliability and redundancy must therefore be provided in the combined MCWAP.

3.2.1. General Criteria

It is not feasible or possible to provide absolute reliability or 100% system availability, i.e. no risk of an interruption in the delivery of water from a scheme. It is, however, possible to reduce the risks to of the project to acceptably low levels, to allow for the strategic importance of most of the water that will be supplied by the project. The risk can further be reduced by providing redundancy between schemes.

In this regard, the transfer schemes shall be sized for 95% reliability, implying that water shall continue to be supplied without interruption even if the scheme is inoperative for up to 18 days of any one year, and the scheme capacity adjusted to allow the full annual requirements to be supplied in 347 days. Eighteen days storage capacity will be designed into the system to ensure that strategic customers will not be exposed to an unduly high risk of supply failure. The storage facilities must be provided by the end users and are therefore excluded from the project cost estimate. The operation of the facilities will, however, be under the control of the MCWAP.

Allowing for a scheme to be inoperative continuously for 5% of the time during any one year (18 days) will be sufficient to cater for the following situations:

- Pump station failures if there had been severe damage such as flooding of the electrical equipment, etc.;
- Constructing temporary by-passes to repair pipeline linings and joints; and
- The time required to restore power supplies after major interruptions such as bushfires, flooding, lightning, etc.

Limited redundancy will be provided by interconnecting the Mokolo and Crocodile River (West) Schemes. No redundancy will, however, be available during the interim period (Phase 1) before the CRW Transfer Scheme is operational.

3.2.2. Reliability Criteria

The following sizing criteria were incorporated into the planning and costing of components to improve the reliability of supply:

- Terminal reservoirs must be provided at all end user delivery points to provide on-site storage with a minimum storage capacity of 18 days.
- System losses were assumed to be 2% of the average annual water requirement.
- The diameter optimisation and economic evaluation was based on 105% of the gross annual average water requirement (including system losses) to account for the annual 18 days downtime of the scheme (Design Flow Rate).
- Pumping stations were sized and pipe pressure rating (wall thickness) determined to enable a transfer rate of 120% (Recovery Peak Flow Rate) of the gross annual water requirement (at sub-optimal pumping rates) in order to refill the Terminal Reservoirs over a 90 day period, following 18 days of continuous downtime.
- The worst case emergency scenario for the CRW Transfer Scheme occurs when the Phase 1 Scheme (Mokolo Delivery) makes no contribution to the project. The CRW Transfer Scheme (Phase 2) must therefore be able to transfer the full water requirement in the short term to those water users that can accept the lower quality CRW water. The flow under these circumstances was found to be less than the 120% recovery peak flow and no additional allowance was made for this scenario in the sizing of the scheme components.
- The annual peak of 9% (Peak Flow Rate) (refer to Section 3.1) was not applied simultaneously with the design or recovery peak factors in sizing the components. In the interim period (lower demand), the system will have sufficient capacity under

normal operating conditions to accommodate the expected annual peak requirements. The normal reliability capacity (Design Flow Rate) will be able to supply the monthly peak until approximately the end of 2024. Should an 18 day continuous system failure occur on the Crocodile River (West) Transfer Scheme during a period of seasonal peak flow in 2030, the maximum recovery period could be a much as 164 days. This risk can be mitigated by:

(i) Providing additional storage at the terminal reservoir sites (approximately 8 days additional storage would be required at each site);

(ii) Increasing the capacity of the CRW Transfer Scheme to cater for the anticipated peak of 9%; and

(iii) Temporary utilisation of the full transfer capacity from the Mokolo Scheme installed as part of Phase 1 (38.7 x 1.02 x 1.2 = 47.4 – 28.7 = 18.7 Million m^3/a surplus supply). Thus, will result in a reduction in the recovery period on the CRW Transfer Scheme to 83 days.

Option (iii) is recommended as it would not involve further capital expenditure to increase the size of infrastructure components and it is in line with the redundancy approach adopted for the project (see below).

- Switchgear and instrumentation at abstraction sites will be located in the superstructure of the abstraction weir, or on the river bank next to the weir, but in both cases the equipment will be located above the Probable Maximum Flood (PMF) level. Other components forming part of the abstraction and desilting process (i.e. secondary desilting bays, balancing dam, etc.) will also be located above the PMF level.
- High-lift and booster pump stations will be positioned above the PMF and designed such that they will always be free-draining in the event of flooding due to failure of internal pipe work.
- High-lift and booster pump stations will be designed with a minimum of one standby pump unit per station ensuring a minimum standby capacity of 25%. The maximum motor size will be limited to 10 MW per unit. A 4 duty-1 standby configuration is preferred.
- Abstraction pump stations will consist of multiple abstraction bays housing submersible pumps capable of pumping a maximum of 1 m^3/s per unit. In the case of Vlieëpoort, one additional fully equipped standby bay plus one full spare pump including Mechanical and Engineering (M&E), valves and screens will be provided.
- All electrical equipment will be located above the PMF level.
- Strategic spares and equipment will be kept for medium voltage (MV) and low voltage (LV) electrical equipment and other critical components.
- 100% Duplication of the power supply from the switch yards to the pump stations will be provided and a duplicate power supply (firm) will be provided by Eskom.

- Gravity pipelines downstream of the Break Pressure Reservoir will also have a capacity of 120% of the gross average annual requirement, as determined by the rising main capacity.

3.2.3. Redundancy Criteria

The following criteria were incorporated into the planning, sizing and costing of components to ensure redundancy of supply:

- The existing pipeline from the Mokolo Dam will be refurbished and operated in parallel with the new pipeline to eventually provide redundancy for this scheme. The location of inter connections between the pipelines must be optimised as part of the detail design. The ultimate combined peak transfer capacity of the Mokolo Scheme after decommissioning of the existing pump station is 47.4 Million m^3/a [(AAR (53.4 – 14.7 = 38.7 Million m^3/a) + losses (2%)) x 1.20]. The long term available yield from the dam is 28.7 Million m^3/a resulting in a 18.7 Million m^3/a surplus supply capacity being available to provide redundancy backup in case of an emergency on the CRW Transfer Scheme.
- Redundancy will further be provided for those users that can accept the lower quality CRW water by an interconnection between the Crocodile River (West) and the Mokolo Schemes for those users that can accept the lower quality CRW water, so that either system can be augmented from the other.
- The CRW Transfer Scheme will not provide redundancy to Zeeland WTW due to the difference in water quality that cannot be accommodated by the treatment plant. Due care must therefore be taken with regards to the difference in water quality supplied by the Mokolo and Crocodile River (West) Schemes when designing for the redundancy connections between the two schemes.
- It can be considered to amend the water license to allow the transfer of additional Mokolo Dam water (better quality and less expensive) to more of the end users during times of excess water availability (i.e. when the dam is spilling and there is surplus water flowing into the Limpopo River from the Mokolo River), by utilising the surplus transfer capacity on the Mokolo Scheme.

Pump failures will be dealt with by having spare pumps and bays. If the power fails, three (3) possible solutions are possible:

- Water flows past the weir for the duration – note that the civil works are designed for a 1:200 year flood recurrence interval event; the electrical supply should be similar;
- Provide standby diesel generators to keep part or the entire Abstraction Works going. The same would then have to apply to the rest of the MCWAP; and

- Provide storage in the weir basin to cater for this situation. This resurrects the problem of upstream flood levels. Also raises the question of the duration of the "design power failure".

3.3. Design Flow and Capacity

Details of the application of the various allowances and factors described in Section 3.2 above to arrive at the three design case combinations that were considered are given in **Table 3-2**.

Table 3-2: Allowances and Factors used in Design Scenarios

Item No.	Allowance and Factors Applied[2]	Basic Design Case	Peak Design Case	Recovery Peak Design Case
1.	95% Reliability factor [4].	5%	0%	0%
2.	Allowance for water requirement peaks (average annual allowance) [1], [4].	0%	9%	0%
3.	Allowance for 90 day Recovery Period after maximum 18 day system outage [3].	0%	0%	20%
4.	System Losses [Phase 1 (Mokolo Dam Scheme) losses added to Phase 2A CRW Transfer Scheme losses]	2%	2%	2%
5.	Allowance for variations in river flow [5].	0%	0%	0%
6.	Recovery Peak on the CRW Transfer Scheme due to failure of Phase 1 Mokolo Dam supply (due to over-usage, etc.) [6].	Nil	Nil	Greater of 20% GAAR or 19.5 million m^3/a

Notes:

1. Refer to Section 3.2 for details.
2. The % allowances factors were applied in the form: Flow x (1 + %/100).
3. The allowance for the 90 day Recovery Period was used independent of the other factors (apart from the system loss factor) to avoid compounding of related allowances. Pump selection and the pressure rating of pipelines, as well as gravity flow sections of the system were designed to be able to transfer the recovery peak flow case.
4. The economic optimisation of infrastructure components that are part of a pumping system was based on the design flow incorporating the reliability factor (1.05). The reliability and peak factors were not applied simultaneously.

5. It was assumed that all the water requirements would be available in the river at the abstraction works. This aspect is further addressed in Section 3.3.1.
6. Based on the greater of 20% of gross average annual (water) requirement Gross Average Annual (water) Requirement (GAAR) on the CRW Transfer Scheme or the water requirements on the Mokolo Scheme, excluding the supply to Zeeland WTW (9.2 Million m^3/a in 2025) which cannot be supplied from the CRW due to the variation in water quality.

3.3.1. Abstraction Works

The sizing of the abstraction works considered both the 2030 and 2050 requirements for Scenario 9. The three design cases were applied as follows to size components of the Abstraction Works, other than the Weir itself, which is discussed in the next section.

- Basic and Peak Design Case:
  - River flows and losses.

- Recovery Peak Design Case:
  - Number of pumps and pump bays required in Low-lift Pump Station (provision for one additional pumping bay then added);
  - Number of channels to be provided in the Desilting Works (provision for one additional channel then added);
  - The 6-hour operational storage requirements at the Balancing Dam; and
  - Sizing of rising main and other pipework between Low-Lift Pump Station and High-lift Pump Station.

Design capacity parameters for the Feasibility stage layouts were generated from data obtained from the Water Requirements analyses described in Sections 3.1 and 3.2 above and application of the allowances detailed in **Table 3-2** and are summarised in **Table 3-3** below.

Table 3-3: Vlieëpoort Abstraction Works Design Flow and Capacity Parameters

Item No.	Design Data	Unit	Scenario 9	Scenario 9 in 2050
1.	**Water Requirements**			
1.1	Phase 1: Mokolo total combined transfer capacity (long-term GAAR) (1).	Million m^3/a	28.7	28.7
1.2	Phase 2A: Crocodile River (West) Transfer requirements (long-term GAAR), including system losses (2%) along Phase 1 and Phase 2A pipelines and reservoirs. (2)	Million m^3/a	173.3	410.9
1.3	Additional Losses in Crocodile River (West) (due to additional release) for weir at Vlieëpoort. See section on river losses below.	Million m^3/a	27.5	29.3
1.4	Total irrigation water requirements upstream of Vlieëpoort.	Million m^3/a	120.0	120.0
1.5	Present water requirements downstream of Vlieëpoort.	Million m^3/a	28.9	28.9
1.6	Total Releases from Dams to provide for Phase 2A – Vlieëpoort Option.	Million m^3/a	302.7	562.1
1.7	Total Flow Releases from Dams to provide for Phase 2A - Vlieëpoort Option.	m^3/s	9.6	17.8

Item No.	Design Data	Unit	Scenario 9			Scenario 9 in 2050		
2.	**Vlieëpoort Abstraction Weir**		**Design Flow (3)**	**Peak Flow (4)**	**Recovery Peak Flow (5)**	**Design Flow (3)**	**Peak Flow (4)**	**Recovery Peak Flow (5)**
2.1	Design flow allowance	%	5	0	0	5	0	0
2.2	Peak flow allowance	%	0	9	0	0	9	0
2.3	Recovery Period allowance (5).	%	0	0	20	0	0	20
2.4	Design Flow Vlieëpoort Weir(6)	m^3/s	5.8	6.0	6.6	13.7	14.2	15.6
2.5	Number of Low-lift Pump Station bays(7).	No.	4	4	4	8	8	9
2.6	Number of Low-lift Pump Station pump sets (7).	No.	7	7	8	15	16	17
2.7	Number of desilting channels in Desilting Works (7).	No.	7	7	8	15	16	17
3.	**Net Storage Capacity Options of High-lift Pump Station Balancing Dam**							
3.1	Capacity based on Operational Storage Criterion of 6 hours.(8)	m^3	125 300	129 600	142 600	296 000	307 000	337 000
3.2	Capacity based on Nominal Storage Criterion of 3 days.	m^3	1 503 400	1 555 200	1 710 800	3 551 100	3 680 800	4 043 700
3.3	Capacity based on River Hydrograph Storage Criterion adopted for Feasibility Stage.	m^3	1 173 000	1 173 000	1 173 000	1 321 000	1 321 000	1 321 000

Notes:

1. Ultimate Mokolo Dam supply after commissioning of CRW Transfer Scheme [Long-term yield of Mokolo Dam (39.1 Million m^3/a) – Irrigation allocation (10.4 Million m^3/a) = 28.7 Million m^3/a].
2. 2030: Phase 2A CRW supply [2030 combined project water requirements (198 Million m^3/a) – Ultimate Mokolo Dam supply capacity (28.7 Million m^3/a) = 169.3 Million m^3/a + losses on the Mokolo Scheme (2% of 28.7 Million m^3/a = 0.57 Million m^3/a) + losses on the CRW Transfer Scheme (2% of 169.3 Million m^3/a = 3.39 Million m^3/a) = 173.3 Million m^3/a].

   2050: Phase 2 future phases CRW supply [2050 combined project water requirements (431 Million m^3/a) – Ultimate Mokolo Dam supply capacity (28.7 Million m^3/a) = 402.3 Million m^3/a + losses on the Mokolo Scheme (2% of 28.7 Million m^3/a = 0.57 Million m^3/a) + losses on the CRW Transfer Scheme (2% of 402.3 Million m^3/a = 8.05 Million m^3/a) = 410.9 Million m^3/a].
3. Total Phase 2A water requirements plus allowance for reliability peak.
4. Total Phase 2A water requirements plus allowance for seasonal peaks.
5. The worst case peak emergency scenario for Phase 2A works occurs when the Phase 1 Scheme (Mokolo Delivery) makes no contribution to the transfer scheme. The Phase 2A CRW Transfer Scheme therefore transfers the full water requirement (198 Million m^3/a), OR, 20% allowance for recovery period after downtime (1.2 x 173.3 Million m^3/a=208 Million m^3/a), whichever is the largest.
6. Based on GAAR x design, peak and recovery factors as listed in lines 2.1, 2.2 and 2.3, respectively for the different design cases.
7. One additional fully equipped standby bay plus one full spare pump including M&E, valves, screens for the design case. For the Crocodile weir this is based on submersible pump with 1 m^3/s rated capacity. Data for suitable pumps were obtained from pump suppliers. Nine double pump bays were provided to cater for the projected long-term requirements.
8. Based on actual design flow required at Vlieëpoort Weir (line 2.4)

3.3.2. River Losses

a) Background

The assessment of river losses for the Pre-feasibility stage (Supporting Report 4) was based on a deterministic approach on the likely additional water losses that can be expected in the CRW as a result of larger releases from the Roodekopjes Dam (and possibly also the Klipvoor and Vaalkop Dams). The analyses were based on a model where all irrigation water requirements were assumed to be abstracted directly from the river and the assumption that the evapo-transpiration losses from the vegetation in the riparian strip and the seepage outflows from the river would not be significantly affected by increased releases from the above dams.

The additional water losses that could occur downstream of the dams as a result of the increased releases from the dams were deemed to be due to increased evaporation losses from the river water surface and increased abstraction by the irrigators in those cases where the current releases from the upstream dams were insufficient to supply all the irrigation requirements directly from the river. The water abstracted from the boreholes in the alluvial aquifers underlying the floodplains in the river valley for irrigation and the unregulated runoff contributions from the catchments downstream of the Klipvoor, Roodekopjes and Vaalkop Dams were not included in the analyses, but would in fact be available to the system to supply the irrigators during periods with simulated shortfalls. This hypothesis was essentially confirmed by the irrigators at a meeting in January 2009, attended by the Client and the PSP, and where it was stated that water shortages were experienced during only one of the previous nine years.

Downstream of the Klipvoor, Roodekopjes and Vaalkop Dams, the CRW is characterised by a very flat slope and a number of prominent meanders in flat alluvial plains. Many irrigators are abstracting water from boreholes in the alluvium, on the basis that it was groundwater. It was therefore decided to perform a preliminary desktop study to establish the nature and extent of the aquifers underlying these alluvial plains. These investigations have led to the conclusions that the alluvial plains are indeed underlain by relatively coarse lenticular alluvial deposits that are hydraulically connected to the CRW and that have created alluvial aquifers that are recharged by rainfall and from the river. Details of the preliminary desktop study that was undertaken are included in Section 4.4 below.

b) Basis of Analyses

The previous analyses had examined the current situation. The average values for the simulation period are summarised in the **Table 3-4** below for the entire river reach between the Klipvoor, Roodekopjes and Vaalkop Dams and Vlieëpoort abstraction site. Hydrologic data was based on that at Paul Hugo Weir (A2116), which was used as a proxy for Vlieëpoort.

Table 3-4: River Losses between Dams and Vlieëpoort

Item No.	Description	Scenario 9	Scenario 9 2050
		Million m^3/a	Million m^3/a
1.	Observed historic dam releases.	101.9	101.9
2.	Observed historic flow past Vlieëpoort site.	28.9	28.9
3.	Total Area under Irrigation between Dams and Vlieëpoort (ha).	15 000	15 000
4.	Unit irrigation allowance (m^3/ha/a).	8 000	8 000
5.	Net irrigation water requirements from Dams [(3)].	120.0	120.0
6.	Evaporation and evapo-transpiration losses (calculated).	24.7	24.7
7.	MAR in sub-catchment (WR90) to Paul Hugo Weir (A2116).	48.2	48.2

Item No.	Description	Scenario 9	Scenario 9 2050
		Million m^3/a	Million m^3/a
8.	Accruals Balance = Net inflow from runoff downstream of the dams plus other diffuse inflows minus diffuse outflows (Pre-feasibility Report Tables 8-3 and 8-4).	7.6	7.6
9.	Diffuse Outflows - diffuse inflows other than natural runoff (Items 7 – 8).	40.6	40.6
10.	Dam releases minus net irrigation and evaporation plus accruals balance (Items 1 – (5 + 6 + 8)).	-35.2	-35.2
11.	Irrigation requirements not supplied by dams (4) (Items 2 – 10).	64.1	64.1
12.	River Losses other than evaporation losses (mean irrigation shortfall) (Items 11 – 9).	23.5	23.5
13.	Total River Losses (evapo-transpiration and evaporation losses plus shortfall) (Items 6 + 12).	48.2	48.2
14.	MCWAP Phase 2 Water Requirements (maximum average) including system losses	173.3	410.9
15.	Additional evaporation and evapo-transpiration losses (due to additional releases) (calculated)	4.0	5.8
16.	Additional releases to cater for losses resulting from additional releases for MCWAP water requirements (Items 6 + 15).	28.7	30.5
17.	Total Dam Releases Required (Items 2+5+6-7+14+15)).	302.7	562.2

Notes:

1. Average net evaporation = 1 200 mm/a.
2. Total river length U/S of Boschkop = 93 km and total river length U/S of Vlieëpoort = 176 km.
3. Net requirements from the river i.e. after allowing for distribution losses and irrigation return flows or enhanced runoff.
4. Derived from the mass balance relationship given below.

The mass balance relationship is expressed as follows for a particular river reach under consideration, i.e. between the upstream dams and Vlieëpoort abstraction site as the case may be:

[Outflow] = [Releases from the dams] *PLUS*
[Runoff from the catchment between the dams and the site + diffuse other inflows – diffuse outflows] *LESS*

[Irrigation use] *LESS*
[Evaporation loss from water surface + evapo-transpiration use from riparian strip]

The natural mean annual runoff (MAR) from the catchments between the Klipvoor, Roodekopjes and Vaalkop Dams and Vlieëpoort abstraction site was obtained from WR90 and were approximately 48.2 Million m^3.

c) Findings

Using the previously modelled mean annual results for the current situation, the estimated natural runoffs from the catchments between the Klipvoor, Roodekopjes and Vaalkop Dams and Vlieëpoort abstraction site and the mass balance relationship for the respective river reaches, the following was concluded:

Diffuse outflows – diffuse inflows other than natural runoff = 40.6 Million m^3/a (**Table 3-4** - Item 9).

This mean net outflow would recharge the alluvial aquifers, but it is less than the simulated mean irrigation shortfall of 64.1 Million m^3/a (**Table 3-4** - Item 11). The irrigators would therefore be drawing down the water table in the alluvial aquifers to make up for the mean shortfall of 23.5 Million m^3/a and / or experience water shortages (**Table 3-4** - Item 12).

On the basis of the January 2009 meeting with the irrigators, it is reasonable to assume that the long-term average water shortages are small since the potential shortfalls of 23.5 Million m^3/a are largely made up from the aquifers, which are then recharged as soon as the river flows increase. Also see **Figure 4-1** in this regard.

It can therefore be expected that when additional water is released from the Roodekopjes Dam for the MCWAP, there will be a mean diffuse net seepage water loss of 23.5 Million m^3/a to the alluvial aquifers connected to the river. This loss can only be reduced if the water use by the irrigators is curtailed during extreme drought conditions. The curtailments would, however, have to be limited so that the irrigators are no worse off than at present, i.e. to ensure that the existing water balance is not disturbed. The mean reduction in outflows to the aquifers will however be small.

For planning purposes, it would therefore be prudent to assume a water loss of about 27.5 Million m^3/a to also allow for the additional evaporation losses from the water surface of the river due to the higher flows when water is released for use by the MCWAP (**Table 3-4** - Item 16).

The analyses have been done on the basis of medium–term mean annual flows. The actual average daily river flows can vary significantly from these mean flows and therefore active management of water releases from the dams will be required to take maximum advantage of downstream inflows. Since the aquifers will be full most of the time when once the water is released from Roodekopjes Dam for the MCWAP there will be less induced recharge of the aquifer during the high flow months and therefore more water is likely to flow past Vlieëpoort during the high flow season. This would constitute an additional loss from the system that can only be quantified by means of river flow measurements, but is expected to be less than the above 23.5 Million m^3/a.

Much of the present surface flow past Vlieëpoort probably disappears below the surface flow and recharges the downstream alluvial aquifers. Therefore, some of the low flow that passes Vlieëpoort at present can possibly be reduced as long as it is compensated for on an annual basis by increased flows during the high flow months to ensure that the existing water balance is maintained in the aquifers. An additional allowance to the above 27.5 Million m^3/a could therefore be made for a reduction in the utilisable runoff downstream of the Klipvoor, Roodekopjes and Vaalkop Dams.

3.3.3. Crocodile River (West) Transfer Scheme

The three design cases were applied as follows to size components of the CRW Transfer Scheme (Vlieëpoort Weir High-lift pump station from the Balancing Dam to the Steenbokpan connection with the Steenbokpan-Lephalale pipeline).

- Basic and Peak Design Case:
  - Economic optimisation of rising main diameters (basic design case only); and
  - Calculation of system recovery period under peak flow conditions.

- Recovery Design Case:
  - Peak transfer capacity of High-lift Pump Station and rising main from the High-lift Pump Station to the Break Pressure Reservoir;
  - Selection of appropriate pipeline wall thickness and pressure rating;
  - Pump selection at the High-lift Pump Station;
  - The 8 hour storage requirements at the Break Pressure and Operational Reservoirs; and
  - Sizing of the gravity mains downstream of the Break Pressure and Operational Reservoirs.

Design capacity parameters for the Feasibility stage layouts were generated from data obtained from the Water Requirements analyses described in Sections 3.1 and 3.2 above and application of the allowances detailed in **Table 3-2** and are summarised in **Table 3-5** below for the transfer scheme. The sizing of the transfer scheme components only considered the 2030 case for Phase 2A.

Table 3-5: Crocodile River (West) Transfer Scheme Design Capacity

Item No.	Design Data	Unit	SCENARIO 9		
			Design Flow (2)	**Peak Flow (3)**	**Recovery Peak Flow (4)**
1.	**Water Requirements**				
1.1	Phase 2A: Crocodile River (West) Transfer Scheme requirements (long-term GAAR), including system losses (2%) along Phase 1 and Phase 2A pipelines and reservoirs (1)	Million m^3/a	173.3		
2.	**Transfer Scheme**				
2.1	Design flow allowance	%	5	0	0
2.2	Peak flow allowance	%	0	9	0
2.3	Recovery Period allowance (4).	%	0	0	20
2.4	Design Flow Rate	m^3/s	5.8	6.0	6.6
2.5	Number of High-lift pump sets (5).	No	5	5	5
3.	**Net Storage Capacity of Break Pressure and Operational Reservoirs**				
3.1	Capacity based on Operational Storage Criterion of 8 hours at the Recovery Peak Flow rate	m^3	167 000	173 000	190 000

Notes:

1. 2030: Phase 2A Crocodile River (West) supply [2030 combined project water requirements (198 Million m^3/a) – Ultimate Mokolo Dam supply capacity (28.7 Million m^3/a) = 169.3 million m^3/a + losses on the Mokolo Scheme (2% of 28.7 Million m^3/a = 0.57 Million m^3/a) + losses on the CRW Transfer Scheme (2% of 169.3 Million m^3/a = 3.39 Million m^3/a) = 173.3 Million m^3/a].
2. Total Phase 2A water requirements plus 5% allowance for reliability peak.
3. Total Phase 2A water requirements plus 9% allowance for seasonal peaks.
4. The worst case peak emergency scenario for the Phase 2 Works occurs when the Phase 1 Scheme (Mokolo Delivery) makes no contribution to the water supply. The Phase 2A CRW Transfer Scheme therefore transfers the full water requirement (198 Million m^3/a), OR, 20% allowance for recovery period after downtime (1.2 x 173.3 Million m^3/a=208 Million m^3/a), whichever is the largest.
5. Based on 4 duties, 1 standby pumps configuration for the 2030 Scenario 9 requirement. Future upgrading will require additional pumping units.

Refer to Report No. 11 – Technical Module: Phase 1 Feasibility Stage for the design capacities of the Phase 1 infrastructure components.

3.4. Conclusion: Water Requirements and Design Flow

The water requirements for the Feasibility Investigation were based on the Scenario 9 projections for infrastructure projects and associated domestic water requirements up to 2030. Reliability and redundancy criteria were applied to arrive at the recommended design capacity of the respective scheme components.

The Pre-Feasibility and Feasibility stages of the MCWAP took place within a very dynamic planning environment. As a result, further variations to the water requirements and design capacities are to be expected and must be incorporated as part of the detail design process. In this regard the following needs to be done:

- Confirm and implement the latest, approved water requirement scenario for the project.
- Re-confirm the reliability and redundancy criteria that must be applied in sizing the Phase 2 infrastructure components. Aspects that might impact on these criteria include the following:
  - The conditions of the final end user agreements;
  - The final system operating philosophy; and
  - Risk assessment.

- Quantify evaporation and scheme transmission losses from the points of abstraction to and including the losses from the Terminal Reservoirs more accurately (a value of 2% was assumed at feasibility stage).
- Re-confirm the water requirements and design flows incorporating latest water requirement figures, system losses, peak and recovery factors and a statistical assessment of storage requirements and system reliability.

4. RIVER ABSTRACTION WORKS

The details of the River Abstraction Works are given in Report 11 – Technical Module: Phase 1 Feasibility Stage and are summarised below.

During the Feasibility stage, the following aspects of the River Abstraction Works component of the MCWAP Study were developed further:

- The Vlieëpoort Abstraction Weir, Gravel Trap, Low-lift Pump Station, Desilting Works and High-lift Pump Station Balancing Dam;
- River Management and in particular, issues such as possible functional arrangements, river flow management, gauging weir requirement assessments and operation and maintenance philosophies; and
- Assessment of dam options at Vlieëpoort.

Sizing criteria were prepared during the course of the Pre-feasibility stage and the structures were sized accordingly for costing purposes. Pertinent sizing data for the Vlieëpoort River Abstraction Works are summarised below.

4.1. General Design Criteria and Assumptions

The following general criteria were applied for the Feasibility Stage layouts:

- The Abstraction Works were designed for 95% reliability or system availability in any one year, implying that the scheme may be inoperative for up to 18 days of any one year, and the scheme capacity was adjusted to allow the full annual requirements to be supplied in 347 days.
- The Abstraction Weir will be designed according to the South African National Committee on Large Dams (SANCOLD) Guidelines for Category III dams with high hazard rating to cater for the strategic importance of the Scheme.
- Design floods will have a recurrence interval of 200 years and safety evaluation floods will be based on the PMF.
- The Low-lift Pump Station will have the switchgear and control instrumentation located above the PMF flood level plus 0.5 m freeboard.
- The High-lift Pump Station will be positioned above the PMF and designed such that the pump station will always be free-draining in the event of flooding due to failure of internal pipe work.
- The Low-lift Pump Station will be provided with an additional pumping bay for use during routine maintenance and emergencies.
- Strategic spares and equipment will be provided for all components.
- 100% spares will be maintained for all MV and LV switchboards.

- 100% duplication of the power supply from the switch yards to the pump stations will be provided and a duplicate power supply will be considered.
- Delivery pipelines (rising mains) will have a capacity of 120% of the average annual water requirement plus all the downstream system losses supplied to on-site balancing dams.
- Downtime for scheduled preventative maintenance will be taken into account.
- Sufficient additional water will be made available by DWA in the CRW to supply the full water requirement via the CRW Transfer Scheme during emergency situations.

4.2. Design Floods

Flood sizing criteria were prepared during the course of the Pre-feasibility stage and the structures were sized accordingly. Pertinent sizing data for the Vlieëpoort River Abstraction Works are summarised in **Table 4-1**. Further refinements to flood level determinations should be done as soon as detail survey and mapping of the study area becomes available.

Table 4-1: Vlieëpoort Abstraction Weir Design and Sizing Data

Item No.	Design Data	Value
1.	Gross Catchment Area	28 300 km^2
2.	Design Flood (RDF) (1:200 year flood)	5 740 m^3/s
3.	Safety Evaluation Flood (SEF) (PMF)	11 180 m^3/s
4.	1:20 year Return Period Flood	2 870 m^3/s
5.	1:50 year Return Period Flood	4 020 m^3/s
6.	River bed Level	890.0 masl
7.	Lowest OC Level	893.2 masl.
8.	NOC Level (PMF plus 0.5 m Freeboard)*	914.43 masl
9.	OC Length	153 m
10.	Total Length of Structure	308 m

* The Non Overspill Crest (NOC) level refers to the top of river training wall level along the left bank, the control room floor level in the Low-lift Pump Station and the embankment/access bridge level on the right bank.

4.3. Component Sizing Criteria

The components of the Abstraction Works were sized according to the following guidelines:

1. Weir lowest Overspill Crest (OC) level is 0.5 m above the gravel trap OC level with the gravel trap invert at the upstream end of the trap 1 m below the gravel trap OC. The gravel trap invert slopes down towards radial gate at the outlet end of the trap at a 1:20

slope. The radial gate will be incorporated in the weir structure. The pump bay inlet OC levels are 0.5 m below the gravel trap OC. The height of the Weir is consequently dependant on the number of pumps to be used.

2. Weir OC length will be sized to minimise upstream impacts during a flood condition.
3. The weir overflow will end in a 1.6 m radius roller bucket for energy dissipation.
4. The weir NOC will be located at the Design Flood Level (DFL) plus 1 m freeboard or PMF level plus 0.5 m, whichever is the highest.
5. In addition flank embankments will be provided to further reduce the risk of outflanking and to assist with the direction of overbank flood return flows back into the river. Riverbank erosion protection works will be provided at the re-entry points. These points cannot be shown on the longitudinal sections, but will be shown on the plan layouts.
6. Maximum flow velocity in the Low-lift Pump Station lead-in channels not to exceed 0.9 m/s.
7. Low-lift Pump Station working level above PMF and 0.5 m freeboard. Working level will also have vehicular access.
8. Low pressure pipeline to be protected against flood damage.
9. Desilting Works will have a channel for each low lift pump. Maximum flow velocity in desilting channels not to exceed 0.3 m/s. Freeboard was selected at 0.5 m.
10. The Desilting Works outlet arrangement will allow staged recommissioning of the works as desilting and maintenance operations on the channels are completed to minimise system down time.
11. Balancing Dam will have a minimum live storage capacity of 6 hours to allow for operation and maintenance, as well as for river management requirements. The minimum operating level (MOL) of the Balancing Dam will be 8 m above the High-lift pumps in the adjacent High-lift Pump Station.
12. A multi-compartment Balancing Dam sized to cater for unplanned changes in river flows. The Balancing Dam will also be equipped with a silt flushing facility although only infrequent use, perhaps once every 10 years, is expected. The capacity of the rest of the Abstraction Works must also be sized to deal with variable river flows, starting with the low lift pump station, which does not have the capacity to deal with this as sized.
13. The Balancing Dam has top dimensions of 600 m x 370 m, five compartments and a total live storage capacity of 1 300 000 m^3. The depth varies from 10.5 m at the inlet side to 13.2 m at the outlet side. A freeboard provision of 0.5 m was made. The large capacity of the Balancing Dam is justified in the light that the topography of the preferred site makes it possible to fit the dam on one farm if it is preferred by the land owners. This possibility was discussed with the affected land owners.

4.4. Geomorphology of the Crocodile River (West)

The Crocodile River (West) downstream of the Klipvoor, Roodekopjes and Vaalkop Dams is characterised by a very flat slope (between 1:2000 and 1:3000) and a number of prominent meanders in flat alluvial plains. Because of the increasing importance of understanding the nature, extent and functioning of the alluvial aquifers and their potential impacts on the Abstraction Works design a preliminary desktop study and interviews with experts familiar with the groundwater regime in the area was undertaken.

(a) Water Geosciences Consulting has been doing studies for Anglo Platinum and the following is a summary of their findings:

- The alluvial aquifer system is in hydraulic connection with the CRW. The hydraulic interaction depends on the prevailing gradient between the river and the alluvial aquifer, as well as on the occurrence and properties of clogging/colmation layers in the streambed. The alluvial system represents the most significant aquifer in the Crocodile River (West) and Marico Water Management Area (WMA), supporting groundwater abstraction for a multitude of farms along the CRW.
- Groundwater investigations in the Amandelbult area, focusing particularly on the alluvial sediments of the CRW, were completed 20 to 30 years ago. Under the old Water Act, the CRW was proclaimed as a Government Water Control Area (Proclamation 111 of 10th May 1968), effecting control over the abstraction and usage of surface water. However, groundwater was still regarded as a private resource and abstracted in significant volumes from the alluvial sediments, hence, inducing surface water inflow into the alluvial aquifer and boreholes. A groundwater or subterranean water control area was then considered to 'control' groundwater abstraction within the alluvial aquifer along the river.

(b) Previous Geo-Hydrological Investigations. Previous geo-hydrological investigations of the study area were carried out by Bredenkamp and Porsasz (1967), BRGM (1976) Hobbs, P. J. (1982), Hobbs, P. J. (1987). The results of these studies are very similar and summarised below:

- The alluvial deposits of the CRW constitute the primary aquifer that exhibits a fairly uniform character although varying greatly in areal extent and thickness. The alluvial deposits (average thickness of 17 m) reach their maximum lateral extent in the extreme north of the study area (south of Thabazimbi) narrowing southwards due to the presence of outcrops of bedrock closer to river.
- Groundwater levels in the alluvial aquifer are controlled by river stage fluctuations and the abstraction of groundwater for irrigation.
- An impermeable clay layer is present at depths varying from 2 to 8 m below the river bed, overlain by unconsolidated and saturated coarse quartzite sands and gravels.
- Major recharge of the alluvial aquifer is due to, and follows, extensive flooding in the valley, i.e. recharge is primarily from the CRW. Reservoir influenced flow in

the river serves to maintain groundwater levels in the alluvial aquifer in close proximity to the river.

- The CRW changes from effluent (i.e. gaining) to influent (i.e. losing) conditions along the river course (with respect to the alluvial aquifer).
- The semi-pervious river banks, together with an impermeable clay layer at varying depths below the river bed, limits recharge during low-flow conditions.
- Minor recharge of the alluvial aquifer comes from precipitation and interaction with the bedrock aquifer.
- The alluvial aquifer changes in the study area from semi-confined to unconfined conditions.
- The river water and groundwater in the alluvial aquifer (i.e. close to the river) show similar chemical compositions. Groundwater samples from bedrock aquifers show a different chemical composition.
- Deeper boreholes tapping the weathered bedrock aquifer show smaller yields compared to shallower boreholes tapping the alluvial aquifer.
- The most prominent geological feature is a major normal fault striking NNW – SSE that generally determines the course of the CRW to the south. The projected position of the fault to the north does not correspond with the present course of the CRW. The fault displays a down-throw to the east, placing granites on the east against the gabbros/norites on the west. The alluvium, on average 17 m thick, effectively conceals the position of the fault.

(c) Conclusions

The flow of groundwater, mostly irrigation return flows, from the shallow, weathered Bushveld aquifer system to the alluvial aquifer system seems to be insignificant. The irrigation return flows derive from groundwater abstracted from the alluvial aquifer and/or abstraction of water directly from the CRW.

The deeper fractured Bushveld aquifer system is generally detached from the shallow aquifer systems (i.e. the alluvial and the weathered aquifer systems), as well as the CRW. This is evident from the extensive dewatering of especially the deeper underground mines, which do not affect water levels in the shallow aquifers. The deeper mine fissure inflows are generally highly variable in their chemical character, pointing to varied sources for these inflows. The latter may not be applicable to shallow open cast mining within the weathered Bushveld rocks or in close proximity to the CRW, for example sand mining.

As a result of the above, a generic approach to determine the (ground) water balance for the Crocodile River (West) and Marico WMA may not be applicable. Such a generic approach of recharge in higher-lying regions with discharge to the river, minus exchanges with the atmosphere or abstractions must be revised / adjusted to account for the different aquifer systems.

Instead the measurement and monitoring of groundwater levels in the alluvial aquifer as a result of varying river stages to quantify the interaction is recommended for a future study. The interaction between the river and the alluvial aquifer, as well as the hydraulic characteristics of the alluvium and the rate of water abstraction from it will strongly influence the rate at which surface water releases will move through the system and experience transmission losses along its flow path.

Ages did a groundwater model of the CRW in addition to the investigations by Water Geoscience Consulting, but did not take the losses into account. They point out that Johan Rossouw of BKS did such a study and found the losses to be in the order of 30%. Their report was entitled: AGES. (2007), The Assessment of Water Availability in the CRW Catchment by Means of Water Resource Related Models in Support of the Planned Future Licensing Process. (AGES Report No. AS-R-2007-01-31).

These investigations have led to the conclusions that the alluvial plains are indeed underlain by relatively coarse ventricular alluvial deposits that are hydraulically connected to the CRW and that have created alluvial aquifers that are recharged by rainfall and from the river. This forms a major source of water for the irrigators along the CRW upstream of Vlieëpoort.

4.5. Geology

4.5.1. Abstraction Works

An exploratory geotechnical investigation was undertaken during the course of the Feasibility stage and is reported on in Report No 8B – Technical Module: Geotechnical Investigations Phase 2. Pertinent findings of the exploratory work were:

- The Vlieëpoort Abstraction Weir site that has been selected appears to be situated on dolomite upstream of the Dolomite/Ironstone contact known to be present in the poort. This will reduce the complexity of the foundation designs to be undertaken.
- The depth of the alluvium appears to be significantly thicker than originally thought and in the central section of the river bedrock was only found 40 m below surface. The extent of the foundation treatment would therefore effectively be double of what was originally anticipated. This has consequently resulted in a significant increase in the estimated cost of the Abstraction Works.
- The Balancing Dam site closest to the Abstraction Works was confirmed to be located on dolomite. The second site, located on residual Ventersdorp lava some 5 km

downstream of the Abstraction Works, is now favoured as placement of a dam on dolomite should be avoided if alternate locations are available.

A full geotechnical investigation during the future design phases will be required to:

1. Refine the location of the Weir in order to reduce founding costs and costs of seepage control;
2. Confirm the materials properties of the alluvium (grading, permeability, clay content, etc.);
3. Investigate foundation improvement alternatives;
4. Low-lift rising main centreline investigation; and
5. Undertake a detailed investigation of the two proposed Balancing Dam and High-lift Pump Station sites.

4.5.2. Balancing Dam, Silt Trap and High-Lift Pump Station

The exploratory geotechnical investigation revealed that the proposed site is underlain by dolomitic rocks, cherts and sub-ordinate shales of the Malmani Sub-group, Chuniespoort Group and Transvaal Super-group.

The proposed site is located on the gentle topography to the north-west of the Vlieëpoort Mountains, which are aligned with the banded ironstone formations of the Penge Formation. The strata strike roughly in a north-easterly direction, and dip at angles between 20° and 30° in a south-easterly direction.

The proposed site is covered by colluvial materials and no geological structure is indicated on the geological map. It is likely, however, that the dolomitic strata reflect a similar attitude, i.e. dipping at moderate angles in a south-easterly direction.

The geological map indicates a minor fault aligned roughly parallel to the Vlieëpoort Mountains, a short distance downstream of the indicated weir site. The geological map gives no indication of faulting at the proposed site, although this may be masked by the cover of colluvial materials.

The rotary core boreholes revealed the following horizons:

- Colluvium;
- Dolomite residuum; and
- Dolomite bedrock.

No water tables were recorded in any of the four boreholes drilled on the footprint and it may be assumed that the water table occurs at depths greater than 10 m.

4.5.3. Pipeline

The pipeline route commences in the south at Vlieëpoort, where it is underlain by rocks of the Transvaal Supergroup (mostly dolomite, chert, arkoses and andesite), before crossing onto Archaean Granite. After crossing back onto Transvaal Super-group rocks, it then traverses mainly Waterberg sediments (sandstone and some mudstone), with patches of granite and diabase. In the north (from about 10 km south of the point where the pipeline splits from the Spoornet line), the pipeline is on Quaternary sands (with calcrete and ferricrete), which overlie Waterberg Group sandstone.

The various geological units encountered along the centreline of the pipeline are given sequentially (from south to north on **Table 4-2**) and their extent is shown.

Table 4-2: Geology Summary for Phase 2A Route

km	Anticipated Geology
0 - 5	Dolomite
5 - 7.7	Pretoria Group and dolomite
7.7 - 15.7	Granite
15.7 - 34.2	Alluvium
34.2 - 37.2	Dolomite
37.2 - 42.6	Waterberg sandstone
42.6 - 57.6	Alluvium
57.6 - 86.6	Waterberg sandstone, diabase
86.6 - 98.8	Alluvium
98.8 - 125	Quaternary sand, diabase

Test pits were excavated at a nominal spacing of 5 km along the route of the pipeline. No seepage was encountered in any of the test pits dug along the route and it appears that this is unlikely, except in the vicinity of streams (and particularly on the south bank of the Matlabas River).

No investigations for bedding and soft backfill were carried out. Notwithstanding this, bedding material and soft backfill should be freely available north of about km 55 and an average haul distance of 5 km seems to be indicated. In this area, it is probable that much of the soil excavated from the pipe trench will be re-usable for bedding and soft backfill.

Between km 0 and 55 borrow sources would seem to be difficult to locate and haul distances of at least 10 km are likely.

4.6. Site Selection, Layout and Description

4.6.1. Site Selection Evaluation Criteria

During the Conceptual and Pre-feasibility stages, a number of potential weir sites were identified, using aerial photography and recommendations received from local landowners. The following selection criteria were used to identify promising locations and to eliminate the poor locations:

1. Weir to be located downstream of main supply dams in CRW being Vaalkop, Roodekopjes and Klipvoor Dams. Consequently, only the weir sites downstream of Pienaars River confluence will meet this criterion.
2. Weir to be located at a bend in the river with the abstraction works on the outside of the bend. The river bend helps the generation of secondary flow patterns to facilitate coarse sediment diversion past the pump station intakes.
3. Abstraction Works to be located preferably on the same side of the river as the main pipeline route to avoid an expensive river crossing of the pipeline.
4. River valley to be as narrow as possible to simplify flood management and to make the footprint of the works in the flood plain as small as possible. Nearby high ground to locate balancing dam and High-lift pumps above the PMF level is essential.
5. Potential for outflanking by the river changing course to be manageable or not present.
6. River channel to be narrow as possible to minimise the cost of the weir. The narrow river channel means: (i) that river diversion arrangements during the construction phase should be simpler, (ii) implies stable river banks, (iii) indicates a smaller downstream gauging weir, and (iv) allows for shorter low OC lengths (for flow measurement at the weir) without impacting too much on the overall flow regime in the natural river channel.
7. Founding conditions: Bed rock preferably to be present at shallow depth to avoid costly foundation treatment to ensure structural integrity during flood conditions.
8. Weir basin to be as small as possible to reduce evaporation losses and minimise impacts on upstream landowners.
9. The location of the weir to result in the shortest possible length of pipeline to the users (CRW options). In the case of the Phase 1 of the Scheme (Mokolo River options) the criterion was the shortest distance to Zeeland WTW to reduce pipeline costs.
10. Weir to be as close as possible to sources of water (dams listed in criterion (1)) to curtail river losses.
11. Proximity (positive) of existing infrastructure such as access roads, power lines, etc., resulting in potential cost savings in the extent of additional infrastructure to be provided.

12. Presence (negative) of existing infrastructure such as other structures in the river, provincial roads, power lines, mining activities, etc., to be avoided as far as possible in the upstream reach of influence of the abstraction weir.
13. Lowest potential for flood damage: Damage at the Abstraction Works under extreme flood conditions should not cause the supply of water to be interrupted for any prolonged periods, because of the strategic importance of the water requirements to be supplied. The forms of flood damage that would fall into this category include loss of structural integrity, clogging of the Works by debris, outflanking, isolation of the Works due to loss of access and interruption of power supply to the Works.

4.6.2. Site Selection

A common feature along the CRW was the deep alluvial sands and silts that filled the river valleys with depths of between 10 and 20 m reported. Rock exposures along the river were a rarity. Foundations for river crossings typically consisted of compacted dumped rock. As the foundations settled into the riverbed (or were washed away by floods), more rock was dumped and the crossings rehabilitated.

Another common feature along the river reaches in the wide river valleys and flood plains were changes in river course (older channel parallel to present channel) and oxbow features. Even small river structures ran afoul of this characteristic with a number of outflanked DWA gauging weirs being easy to identify in this regard. Clearly a larger structure, such as the proposed Abstraction Works, would require extensive (and expensive) protection works to ensure the required integrity of the structure. Consequently all potential weir sites located in the wide flood plain were excluded very early in the process.

The promising weir sites along the CRW were inspected during a site visit on 11 to 13 June 2008. Each site was re-evaluated for appropriateness using the criteria listed above. After a second site inspection on 22 July 2008, all but two sites were eliminated as options along the CRW.

The advantages and disadvantages of each of these two sites, as well as the evaluation criteria assessments are listed below:

Vlieëpoort Weir Site:

- ✓ Located on a suitable bend in the river.
- ✓ Narrow floodplain between two hills.
- ✓ Good sites for the desilting works and High-lift pump station balancing dam available, but some distance away from the weir site.
- ✓ Requires a 50 km shorter pipeline than the Boschkop option via the Central route.

- ✓ Nature reserve immediately upstream will not be as badly affected by raised water levels as irrigated or occupied land.
- × Possibly situated on dolomites.
- × No visible rock outcrops, founding probably on deep sands.
- × Very high observed flood levels will require ancillary facilities to be located some distance away from the weir.
- × Approximately 83 km longer river conveyance than Boschkop option thereby increasing potential water losses.
- × Unstable left bank indicating that the river alignment may change if preventative measures are not taken.

Table 4-3: Evaluation Criteria Assessment for Vlieëpoort Weir Site

Criterion No.	Description	Comments
1.	Downstream of Pienaars River confluence.	Yes.
2.	Abstraction Works on outside of river bend.	Yes.
3.	Abstraction Works on same side of river as pipeline.	Yes.
4.	Narrow river valley or flood plain.	Yes.
5.	Potential for outflanking to be manageable.	Yes. In narrow poort.
6.	Narrow river channel.	Reasonable.
7.	Good founding conditions (rock).	No, deep alluvium.
8.	Small weir basin.	Yes. Deep river channel.
9.	Pipeline length to users as short as possible.	Yes.
10.	Upstream river length as short as possible to curtail losses.	No. 3rd furthest site.
11.	Proximity of access roads, power lines, etc.	Yes.
12.	Upstream infrastructure affected by higher flood levels.	Possible, but manageable.
13.	Potential for flood damage.	Low and manageable.

This site was listed as potentially useful, but with question marks against criteria (6) and (9).

Boschkop Site (Original Dam Site)

- ✓ Located on a suitable bend in the river.
- ✓ Deep (approximately 12 m) channel.
- ✓ Narrow floodplain between two hills. The hill on the left bank is not directly in line with the weir site and will require more extensive protection Works to prevent outflanking.

- ✓ Favourable sites for the Desilting Works and High-lift pump station balancing dam conveniently close to the weir site above the flood lines due to deep channel and steep flood plains.
- ✓ Rock outcrops (dolomite) in right river bank indicating possible good founding conditions for Low-lift pump station.
- ✓ Raised water levels won't affect many structures/irrigated lands, etc. upstream.
- ✓ This is the furthest upstream site, minimum potential water losses due to river conveyance.
- × Situated on dolomites, which should normally be avoided.
- × This is the furthest upstream site, requiring the longest pipeline and possibly higher cost.

Table 4-4: Evaluation Criteria Assessment for Boschkop Weir Site

Criterion No.	Description	Comments
1.	Downstream of Pienaars River confluence.	Yes.
2.	Abstraction Works on outside of river bend.	Yes.
3.	Abstraction Works on same side of river as pipeline.	Yes.
4.	Narrow river valley or flood plain.	Yes. Left bank not ideal.
5.	Potential for outflanking to be manageable.	Yes. Left bank not ideal.
6.	Narrow river channel.	Reasonable
7.	Good founding conditions (rock).	Some outcrops noted on right bank.
8.	Small weir basin.	Yes. Deep river channel.
9.	Pipeline length to users as short as possible.	No. Longest pipeline.
10.	Upstream river length as short as possible to curtail losses.	Yes. Closest to u/s dams.
11.	Proximity of access roads, power lines, etc.	Yes.
12.	Upstream infrastructure affected by higher flood levels.	Possible, but manageable.
13.	Potential for flood damage.	Low. In narrow poort.

This site was listed as potentially useful, but with question marks against criteria (4), (5), (6) and (8).

Finally, the main consideration used for evaluating the two sites was the cost of the additional water losses to the Vlieëpoort site versus the cost and potential delays associated with the additional pipeline length to the Boschkop site. The Pre-feasibility stage found that the additional water losses to the Vlieëpoort site were not so high as to justify the major additional expense of the longer pipeline. The pipeline construction is on the critical path of the project programme and any additional length added to the pipeline will increase the construction time required and most probably delay the project completion. The Vlieëpoort site was consequently selected for further investigation during the Feasibility stage.

4.6.3. Site Layout and Description

The following components form part of the Vlieëpoort Abstraction Works:

- Abstraction Weir;
- Low-lift Pump Station;
- Low Pressure Pipeline between Low-lift Pump Station and Desilting Works;
- Desilting Works; and
- Balancing Dam.

The Abstraction Weir is located in the Vlieëpoort where the poort is; its narrowest across the CRW, in line with the boundary fence between Ben Alberts Nature Reserve and the Mooivallei farms. The Low-lift Pump Station was placed on the right bank (looking downstream) on the outside of the bend in the river. The rising main follows existing access roads and boundary fences on the Mooivallei farms towards the Desilting Works and Balancing Dam downstream of the Weir (towards the North-west). The Desilting Works is located adjacent to the South-eastern edge of the Balancing Dam. Two sites for the Balancing Dam were identified.

The site with the preferred foundation conditions (on residual Ventersdorp lava) is located on portions 1 and 2 of Mooivallei, some 5 km downstream of the Abstraction Weir, and Alternative Site 1 (on dolomite) is 3 km downstream of the Abstraction Weir on portions 6 and 7 of Mooivallei. Also see Section 4.5 and drawing WP9528-LD-VPW-001 (Sheet 4).

The High-lift Pump Station is located on the Eastern corner of the Balancing Dam. The Rising Main from the High-lift Pump Station will follow existing farm boundaries and access roads towards the Thabazimbi-Dwaalboom Road (D1649). See drawings WP9528-LD-VPW-001 and WP9528-LD-VPW-004, included in **Appendix C** for further details.

4.6.4. Changes from Pre-feasibility Stage Layouts

4.6.4.1. Abstraction Weir

- The left bank arrangement was modified to include for a river training to improve the outflanking protection along that bank. As a consequence of the raised water levels the vertical alignment of the adjacent district road was also raised;
- Weir crest was changed from a stepped crest to a sloped crest;
- The lowest weir height was increased from 3.2 to 4.3 m to raise the lowest crest level to provide sufficient head room for the extended capacity low-lift pump station; and
- The idea of reducing the flood profile of the Abstraction Works by placing the control room and overhead equipment on the river bank (Pre-feasibility Stage Weir Option 2) was abandoned because of the operational complexities arising from separating the pumps from their associated controls and overhead equipment. The Pre-feasibility Stage Weir Option 1 (fully equipped Low-lift Pump Station) was therefore adopted and further developed in the Feasibility stage.

4.6.4.2. Low-Lift Pump Station

- The capacity of the Abstraction Works was increased to cater for the transfers that would be required at the end of the design life of the Works in 2050. Presently, the ultimate water transfer requirements is estimated at 431 Million m^3/a. The additional capacity was created by increasing the size of the pump bays to accommodate two 1 m^3/s submersible pumps each and by adding another pump bay, thereby increasing the total number of bays provided from 8 to 9. The pump bays were widened from 1.75 to 2.5 m to accommodate the additional pump.
- The relocation of the Desilting Works, Balancing Dam and High-lift Pump Station to the site with the preferred foundation conditions will require an additional 5 km of low-lift rising main.

4.6.4.3. Desilting Works

- Following a site visit to the Lebelelo Weir on the Olifants River (of similar design) and assessment of the operational problems being experienced, the length of the desilting channels was increased from 80 to 120 m and the depth of the channels was increased by 1m to improve settling out of the finer fraction (< 0.3 mm) of silt.
- Two additional transverse walls were added to create a revised inlet arrangement to improve the early development of stable flow patterns thereby increasing the effective length of the channels.
- A silt settling pond was provided for in the waste water return channel to allow for settling out of the heavily silt-laden water before the water is returned to the river.

Further consideration of options to deal with the silt that has accumulated in the settling pond will be required. Possible options include selling of the silt to farmers, controlled dumping on approved waste sites or, ensuring that the pond is large enough to permanently accommodate the silt.

4.6.4.4. Balancing Dam at High-Lift Pump Station

- The capacity of the Balancing Dam capacity was increased from 111,700 m³ (4 hours operational capacity) to 1,300,000 m³ to cater for flow variations in the river at the Abstraction Works. The minimum operational storage requirement for the Balancing Dam was increased from 4 to 6 hours.
- Improved operation capabilities during planned maintenance. The reservoir was divided into compartments (similar to the Terminal Reservoir layouts proposed in the Pre-feasibility design). The compartments would operate independently thereby allowing off-line maintenance and, as a secondary benefit, prevent stagnation of water in the dam.
- Although harmful sediment must be removed in the Desilting Works, additional dead storage was provided in the Balancing Dam to allow for unplanned sedimentation.
- Each compartment was equipped with high flow capacity inlet, outlet and dewatering structures. Each compartment is also provided with overflow protection in the form of concrete spillways. A collector canal in turn channels the overflows to the Desilting Works waste water canal leading back to the river.
- Pipework and valve arrangements were planned to allow for gravity flow through the system from the Desilting Works to the High-lift Pump Station.

4.6.5. Description of the Works

4.6.5.1. Abstraction Weir

The weir is required to be located on a bend in the river. This allows the intakes to the Low-lift Pump Station to be placed on the outside of the bend in order to minimise sedimentation at the intakes. The weir is not designed for storage and it is assumed that it will silt up (and, because of its low height, should be scoured clear during most large flood events). The particular design that was adopted will, however, minimise the effects of sedimentation on the operation of the Works. Details of this design are provided below.

The weir was classified as a Category III (high hazard) structure in terms of the dam safety regulations because of the strategic importance of MCWAP. This approach is discussed and motivated in the section on design floods in Report No 4 – Technical Module: Dams, Abstraction Weirs and River Works. The Recommended Design Flood (RDF) for the weir is the 1:200 year return period flood and the Safety Evaluation Flood (SEF) the unrouted

PMF, because of the small reservoir area. All control and electrical equipment and access to the Works were located above the PMF level.

The weir is designed as a gravity mass concrete structure. The height of the weir is determined by the length of the Gravel Trap (see section on Gravel Trap below) which forms part of the Low-lift Pump Station. The broader goal is to minimise the height of the weir in order to minimise the cost and any upstream impacts. The lowest OC is about 4.3 m above riverbed and is based on a Low-lift Pump Station arrangement with nine pump bays. It is recommended that further work should be done in the future design stages to reduce the height of the weir. The lowest OC section is located nearest to the Low-lift Pump Station. 95% of the average daily flows measured at the Paul Hugo Weir upstream of the Vlieëpoort site were below 3.6 m³/s. This flow was used as the low-flow design flow to size the lowest OC of the weir. The lowest OC is 5 m wide, resulting in an overflow depth of 0.5 m at the low-flow design discharge. This provides for additional storage in the weir basin to allow pumps to extract 120% of requirements without having switch on and off at short intervals. The weir OC level gradually increases towards the left bank (looking downstream) following the original ground level and terminating in the NOC structures above the PMF flood level (also refer to **Table 4-1** above), consisting of the river training wall along the left bank and the Low-lift Pump Station and access embankment along the right bank, in order to prevent outflanking. See drawing WP9528-DD-VPW-001 for an elevation of the weir.

The lowest OC level, Gravel Trap long weir level and Low-lift Pump Station operating levels were selected such that surplus water in the river, when available, could be abstracted and stored in the Balancing Dam (also refer to discussions in the sections below). The Weir beyond the low-flow section will therefore only overflow under higher river flow conditions. The Abstraction Works operating rules and MCWAP upstream dam releases will ensure compliance with planned instream flow requirements (IFRs), but short-term lows in the river flows may make it necessary to release water from the Weir to augment short-term downstream water requirements (during this period the surpluses in the Balancing Dam will be used to support the MCWAP water requirements). For this purpose, low level outlets (with flow measurement) should also be provided in the Weir to facilitate these releases.

An ogee crest profile was selected in order to maximise the discharge capacity of the weir. A discharge coefficient of 1.66 was used for the Feasibility study, but this will depend on the degree of submergence. On upstream soil contact faces of the weir, a 10:1 slope was used to ensure that the active soil pressure on this face is attained. On water contact surfaces the upstream face can be vertical. A concrete roller bucket energy dissipation structure with downstream rip-rap protection is proposed just downstream of the weir. Further analysis is required to optimise the sizing of the downstream energy dissipation structures. See drawing WP9528-DD-VPW-003 for typical sections through the weir.

Following the exploratory geotechnical investigation it was concluded that the weir will be constructed on deep sands (maximum depth of approximately 40 m). Conventional founding methods will not be suitable and foundation treatment consisting of three rows of jet-grouted piles and curtain grouting along the entire length of the weir will be implemented. This cut-off will extend down to at least 20 m below the surface or to bedrock to control seepage and prevent piping underneath the weir. Using a Lane's constant (refer to the US Bureau of Reclamation Manual for Design of Small Dams) of 8 for coarse sandy material, a vertical leakage path length of at least 40 m is required with a differential head of 5 m, thus requiring a cut-off depth of at least 20 m deep.

The jet grouted piles were conservatively placed at 1.5 m intervals in each row to ensure that adequate provision was made for foundation treatment. Depending on the effectiveness of the jet grouting to improve founding conditions, additional concrete piling may be required at the Low-lift Pump Station (dependant on further geotechnical investigations and calculation of foundation loadings). The scope definition of a future detail geotechnical investigation (see Section 4.5) should take cognisance of the following weir design requirements:

- Bearing capacity of friction piles under submerged conditions;
- Foundation bearing capacity (weight, over-turning moments, uplift, etc.);
- Shear key capacity (protection against sliding);
- Foundation drainage between pile rows (uplift);
- Protection against undercutting (outward facing inclined piles may be necessary);
- Permeability assessments (seepage below or between piles); and
- Overall stability (weight of downstream toe, liquefaction, seismicity).

The areas immediately upstream and downstream of the weir will be cleared and suitable erosion protection measures such as grassing and heavy rip-rap will be applied. The existing gravel road (D727) on the left bank will need to be raised locally at the weir. See drawing WP9528-LD-VPW-002 for a layout of the weir structure. Toe protection of the road embankment is required. Whether this is done by heavy riprap protection (original design) or by the training wall (advised by reviewers) is a matter for the designers.

A gauging weir and boreholes to monitor water table levels (and also the health of the aquifer) will also be required downstream of the abstraction weir to monitor flows downstream of the Abstraction Works. This system can be linked to the pipeline control centre.

4.6.5.2. Low-lift Pump Station

The Low-lift Pump Station building will be in reinforced concrete, about 25 m high and will be located on the right bank of the river and be integral with the weir. The structure will be approximately 70 m long, sub-parallel to the right river bank, and will extend up to approximately 25 m into the right bank (extents to be finalised once detail geotechnical information is available). See drawings WP9528-SHT-06-DD-VPW-004 and WP9528-SHT-07-DD-VPW-005 for details on the Low-lift Pump station.

Capacity and Pumping:

Constant speed submersible pumps and motors capable of handling suspended solids are recommended in the Low-lift Pump Station for the following reasons:

- They are more robust and have large open impellers made of more erosion resistant materials and are more tolerant of larger grit particles (do not require clarified water for bearings).
- With the PMF being approximately 25 m above the weir crest level, vertical spindle pumps will have unacceptably long pump shaft lengths. This has very specific disadvantages.
- Constant speed submersible pumps are more tolerant of larger grit particles and do not require clean water for cooling of bearings.
- If the criterion of a minimum of 25% standby capacity is adopted, two additional pump bays with installed pumps will be required for vertical spindle pumps. In the case of submersible pumps, the standby units can be stored on site because of quick replacement time.
- Variable speed drive pumps have the advantage of being capable of abstracting flows as they occur in the river thereby avoiding intermittent turning of pumps on and off to follow the river hydrograph. This advantage was not considered to be significant enough at this stage to warrant a change from the proposed constant speed pumps since the system has enough storage and contains enough pumps to allow for automatic starting and stopping pumps according to the river flow.

The pump bays were sized assuming pumps with a fixed discharge capacity of 1.0 m^3/s and maximum discharge head of 50 m. The maximum Stage 1 flow rate of 6.6 m^3/s (refer to **Table 3-3**) would require 7 bays plus one spare bay, totalling 8 bays thereby providing a total capacity of 8.0 m^3/s.

The bays also allow for a second pump to be installed in series which can provide the additional requirements during Stage 2 (Scenario 2050) of the project. The Stage 2 maximum pumping capacity requirements of 15.6 m^3/s (refer to **Table 3-3**) forced another extra bay resulting in the 9 double pump bays shown on the drawings. Two of the additional bays will be fitted with a 0.5 m^3/s and a 0.25 m^3/s submersible pump, respectively. These two additional pumps will provide flexibility and finer control over the pumping rate, allowing adjustment of the discharge rate in 0.25 m^3/s intervals. The additional provision of a river return flow outlet with a control valve will allow for more precise abstraction, if required. The additional bays also improve the reliability of the system and provides for the additional capacity required to allow for the abstraction of surplus flows from the river for storage in the Balancing Dam. The hydrograph for Paul Hugo Weir (A2H116) shown in **Figure 4-1** would represent a good example of what can be expected at the Vlieëpoort Abstraction Works if a constant flow were released from upstream. Gauge plates and water level sensing / recording equipment at the weir must also be installed to provide flow information to use for operating the pumps to suit the incoming / outgoing flow.

The pump characteristics used to calculate the power absorption are as follows:

- Static head 38 m
- Rising main length 5 000 m
- Rising main diameter 2 100 mm
- Maximum pipe roughness ($k_{smax)}$ 0.5 mm
- Efficiency of pump (Ep) 75%
- Efficiency of motor (Em) 97%
- MW/MVA ratio (power factor Pf) 0.96

The estimated absorbed power (Mega Volt Amperes (MVA)) at the two duty ratings are summarised in **Table 4-5** for the expected maximum operating head.

Table 4-5: Vlieëpoort Low-Lift Pump Station Duty Point and Absorbed Power

	Design Flow	Recovery Peak Flow
Maximum System Head (Max static head and 0.5 mm absolute roughness)		
Duty	5.8 m^3/s @ 48 m	6.6 m^3/s @ 49.5 m
Absorbed power	4.0 MVA	4.7 MVA

The estimated peak power consumption will be approximately 4 MVA for the Phase 2A Transfer Scheme. Future upgrading of the pump station has not been evaluated.

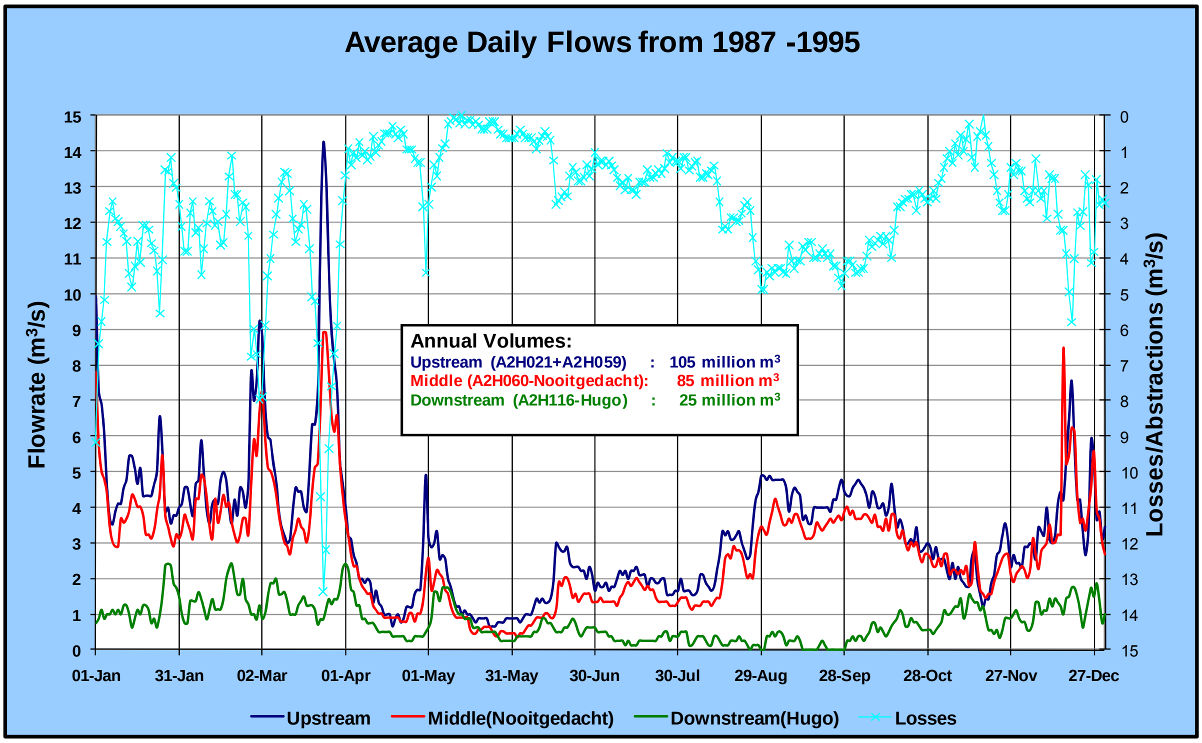


Figure 4-1: Daily Average Flows in the Crocodile River (West)

Gravel Trap:

A gravel trap, which is effectively a long weir, with a downstream setting compartment will be constructed in front of the pump bay intakes. This gravel trap will allow coarse gravel particles to be deposited before water reaches the low-lift pumps. A radial gate is installed at the downstream end of the trap. The trap will be flushed from time to time by opening the radial gate. The invert of the gravel trap will be sloped upstream at a grade of 1:20 to allow for flushing under gravity. As the gravel trap spans the entire length of the Low-lift Pump Station, the 1:20 slope and the length of the structure (width and number of pump bays) determines the height of the weir. The total length of the gravel trap crest is approximately 60 m. The ultimate design flow of 16.4 m³/s will require a flow depth over the gravel trap crest of 0.32 m. The crest was placed 0.5 m below the weir lowest OC to ensure that enough head was available. On the furthest upstream end of the gravel trap, the gravel trap floor was placed 0.5 m below the pumping bay intake to allow for gravel build-up in the trap. The depth of the gravel trap increases to 3.8 m on the furthest downstream end at the radial gate.

Pumping Bays:

The low-lift pump station is divided into nine separate pumping bays. During Stage 1 of the project each bay will have 1 Low-lift pump installed. Provision was made for an additional pump in each bay to allow for future expansion during phases after Phase 2A. The width of the bays and spacing of the pumps were determined in accordance with the following guidelines from Prosser for two pumps in series:

- Bellmouth Diameter (D) = 1.75 x Pipe Diameter (D);
- Width (W) = 3 x D; and
- Pump Spacing (L) = 8 x D.

Assuming a pipe diameter of 500 mm:
- D= 875 mm;
- W = 2.625 m; and
- L = 7.0 m.

The intake openings from the gravel trap to the pumping bays will be covered by trash racks to prevent debris from entering the pumps. A trash rack cleaning mechanism will be provided as cleaning is anticipated to be required on a regular basis. Larger debris, such as tree stumps, is anticipated to mostly flow over the weir structure. The design flow for the intake openings is 2.05 m^3/s each. An allowance of 50% blockage in the trash racks were allowed for. This results in a required opening height of 0.62 m for weir flow. Openings of 1.0 m high were provided. The tops of intake openings were placed at the same level as the lowest weir OC. A screen, as additional debris protection, and a sluice gate and stop log gate were provided in each pump bay to allow for maintenance.

Some silt and sand build-up is anticipated in the pumping bays. Each bay will be provided with a sluice gate on the downstream end to allow for manual cleaning and flushing when required. The sluice gate discharges into a flushing channel on the downstream side of the Low-lift Pump Station, which will direct the flushing water and silt back into the river. Regular flushing of the pump bays should ensure that the silt concentration in the flushed water is low enough not have a major impact on the silt load in the river. Flushing should ideally be done during minor flood events when the silt load in the river is already high.

Operating Levels, Switchgear, etc.:

The operating levels and switchgear were located above the PMF flood level. Crawl beams and cranes will be installed at the operating level which will be used to install, maintain or replace pumps, pipes and any other equipment in the Low-lift Pump Station.

Pump Station Access Embankment:

An earthfill embankment with a crest level above the PMF level provides access to the pump station from the right bank and prevents outflanking of the structure during large floods. Appropriate erosion and flood protection measures such as riprap on the slopes of the embankment were provided. Crane rails for a mobile crane can be installed on the embankment to allow for the movement of heavy equipment to and from the Low-lift Pump Station. Alternatively, a road surface can be provided to allow truck access to the Low-lift Pump Station.

This structure might have a significant impact on the upstream flood lines (which should be further investigated once detail survey and mapping becomes available), because of the large profile presented to the river. As one of the weir design criteria is to minimise the upstream impacts, the embankment could be replaced with a bridge structure if the impact on the upstream water levels is sufficiently improved by such a change and the Abstraction Works and river bank can be protected from scour.

Return Flow Outlet:

The disadvantage of constant speed submersible pumps (or on/off pumps) is that only a stepped and not an infinitely variable rate of abstraction is possible. Because river flow rates will be different to installed capacity and will vary due to changes in upstream use and loss patterns, some alternative means of controlling abstraction rates other than switching pumps on or off should also be considered. A return flow outlet on the delivery main can provide the required flexibility. This outlet will allow finer adjustments of the flow delivered to the Balancing Dam by allowing a variable flow to return to the river upstream of the weir, depending on the incoming flow in the river.

4.6.5.3. Low-Lift Pump Station Rising Main

The rising main from the Low-lift Pump Station will consist of a 2 100 mm diameter steel pipeline of approximately 5 000 m long to the site of the Preferred Balancing Dam (2 800 m to the Alternative 1 Balancing Dam). The pipeline will be installed underground with at least 1 m of cover above the pipe. The pipeline will be laid without any reverse grades to avoid low points that may accumulate sediment should the velocities fall too low. Access and valve chambers will be located at approximately 500 m intervals along the route. These will be concrete structures protruding slightly above the natural ground level. The pipeline will be split with a manifold into nine 900 mm diameter steel pipes leading to the Desilting Works inlets. Each of these pipes will have a gate valve in a valve chamber adjacent to the Desilting Works. The construction servitude will typically be 40 – 50 m wide. The permanent servitude will typically be 20 – 30 m wide and will depend on future upgrading requirements. A second pipeline will be required during Stage 2 of the project which will also be located within this servitude. Permanent access along the pipeline servitude will be required after construction. An access road parallel to the pipeline will be provided within this servitude. Pipeline markers (concrete posts) will be installed at changes in direction and at regular intervals along the route. Farming activities (stock and crop farming) can continue within the servitude area after construction, taking cognisance of the need for permanent access to the pipeline servitude. The alignment will follow existing access roads and farm boundaries where possible.

The following aspects will need further consideration during the detail design phase of the project:

- Minimum allowable flow velocities that must be achieved to effectively transport the expected silt loading of the water. Higher minimum flow velocities during the early stages of the project when the system requirement is low might necessitate the construction of smaller diameter pipelines. This would also improve the redundancy of the system; and
- Selection of an appropriate pipe lining given the possibility of higher silt loadings that might be abrasive.

4.6.5.4. Desilting Works

The Desilting Works with flushing facility will be located adjacent to the Balancing Dam within the earthfill embankment of the dam. For Stage 1, the Desilting Works will consist of nine (9) 120 m long concrete channels, 2.5 m wide with a depth varying from 5.5 m at the inlet end to 4.0 m at the outlet end. The channels were sized to limit the velocity to 0.1 m/s to allow most of the silt and sand particles to settle out before the water is transferred (by gravity) to the Balancing Dam. A duplicate set of channels will be required for Stage 2. Freeboard of 0.5 m was allowed above the maximum water level in the channels.

The Low-lift Pump Station rising main supplies the nine channels through a manifold. Each manifold branch discharges into a baffle-type stilling basin. This dissipates the energy of the incoming flow into the channel. A second baffle wall in the channel calm the flow further to ensure the flow in the de silting channel is as evenly spread as possible to allow for maximum silt deposition.

At the outlet of each channel essentially clear water overflows into a channel, feeding the 2.1 m diameter steel gravity pipe to the Balancing Dam inlet works. A head of at least 2.3 m is required between the Desilting Works and the Balancing Dam to allow for a gravity-based system.

The desilting channel floors are sloped at 1:80 to allow for greater ease of cleaning under gravity. The flushing facility will flush to a sediment settlement pond before the water is returned to the river. The pond will allow the sediment in the flushing water to settle out, and clear water will leave the pond and return to the river via a suitable river return conduit and outlet structure with energy dissipation and erosion protection works. This return conduit will be combined with the Balancing Dam spillway collector. The figures below explain the operation of the desilting channels.

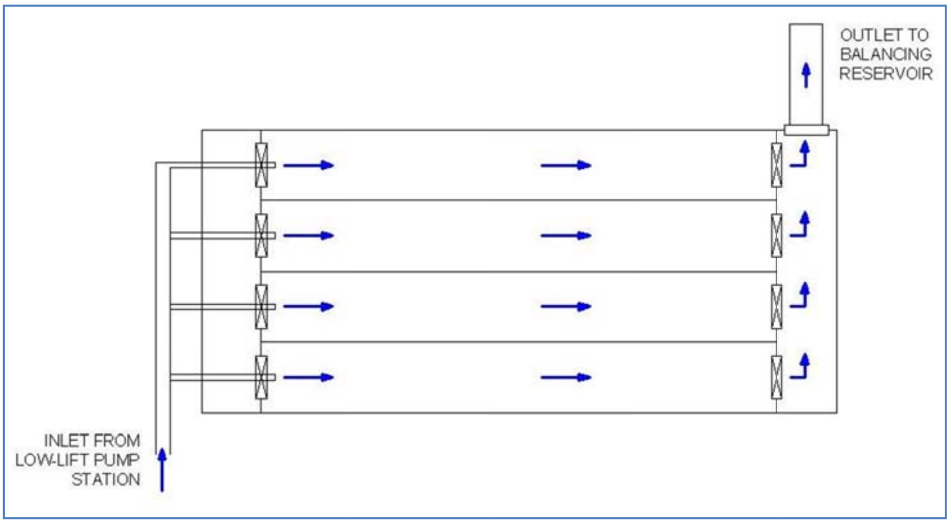


Figure 4-2: Desilting Operation

(Note: Only 4 channels shown)

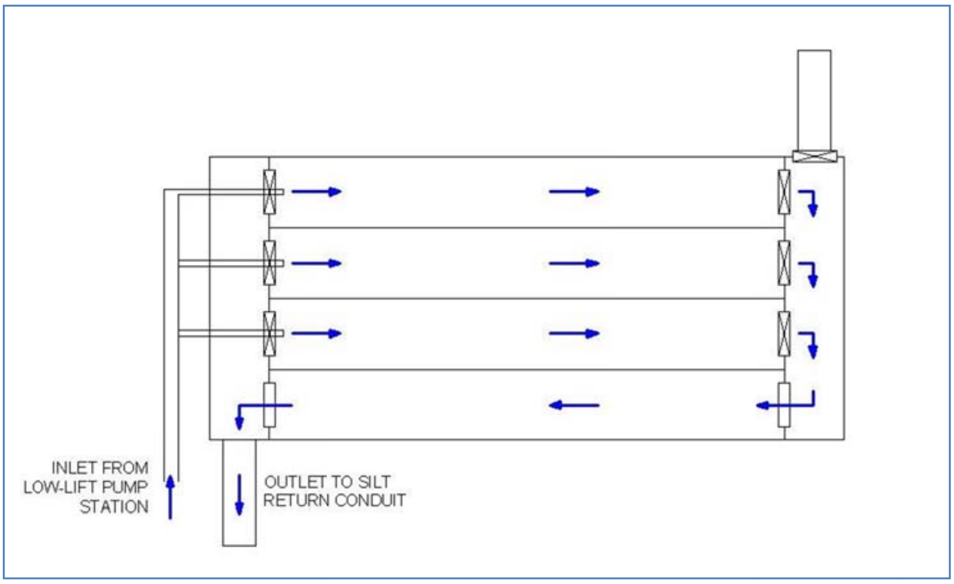


Figure 4-3: Flushing Operation

(Note: Only 4 channels shown as an illustration)

4.6.5.5. Balancing Dam

Site Selection and Alternatives:

Two viable sites were identified downstream of the Vlieëpoort Abstraction Weir. Placement of the Balancing Dam is determined by the PMF level and the maximum Low-lift Pump Station available head. The originally preferred site (now Alternative Site 1) on Mooivallei Farm portions 6 and 7 was found to be underlain by dolomites and an alternative site on portions 1 and 2 further downstream was identified, which is now the Preferred Site. See drawing WP9528-SHT-08-LD-VPW-003. No drilling investigation was done at this site, but a desktop study suggests that the site is underlain by Ventersdorp Lavas.

The Preferred Site has some disadvantages over Alternative Site 1 including:

- It is further away from the abstraction works, which decreases the available head from the Low-lift Pumping Station and would also require a separate access road and electricity supply.
- The farm portion on which it is situated seems to be the most economically viable portions on the Mooivallei Farms. Portions 6 and 7, on which Alternative Site 1 is located, are much smaller and further subdivided.

However, the risks and additional costs associated with a reservoir on dolomite outweigh the abovementioned disadvantages; therefore the Preferred Site is recommended. The Preferred Site also seems less steep and once detailed survey information becomes available, further refinement of the site layout is recommended to minimise the impact on the land owners. It might be possible by changing the shape of the reservoir to only affect one of the two farm portions.

Sizing and Operation:

Various sizing criteria were considered. The first part of the required balancing storage allows for routine maintenance on any component of the Abstraction Works. A total of 6 hours' storage at the peak design flow will be provided as per **Table 4-5**. To compensate for possible fluctuations in the river flow, additional storage will be required. This storage requirement was estimated in two ways:

i) *Estimates based on evapo-transpiration and evaporation losses along the CRW downstream of the Roodekoppies Dam.*

Using December as the worst month regarding variability, based on warmer spells and cooler spells around the mean, a three day response time from the upstream dams and warmer and cooler spells lasting for at least three days continuously, a total storage requirement of 550 000 m³ for the eventual layout was calculated. The monthly distribution of evaporation presented in **Figure 4-4** is also of interest.

ii) *Estimates based on Variability of Observed Flow at Gauging Station A2H116.*

Average daily flow records at the Paul Hugo Weir (A2H116), some 15 km upstream of the proposed Vlieëpoort Weir, were available between 1987 and 1995 (refer to **Figure 4-1** for details). A number of statistical analyses of the daily data were done to estimate how the flows, the running flow averages and the flow duration curves would look for various lag times as an indication of the storage requirements.

The daily flow records at A2H116 were analysed on the following basis:

- Assume that one would adjust the releases from the dams on the basis of the observed flow at the time of making the adjustment;
- Assume that there are various lag or response times between the dam release and the time it arrives at A2H116. Lags of 1, 2, 3 and 4 days were considered;
- These lags would mean different flow fluctuations. Storage was then provided so as to remove the fluctuations and then the changes in storage, shortfalls and surpluses were determined for various storage capacities; and
- Maximum average abstraction from the river set at 8.0 m^3/s for Stage 1. This could imply interruptions in filling the Terminal Reservoirs if they are in a state of drawdown. The effects on total system storage will however be small and it will mean better utilisation of the spare pumping capacity.

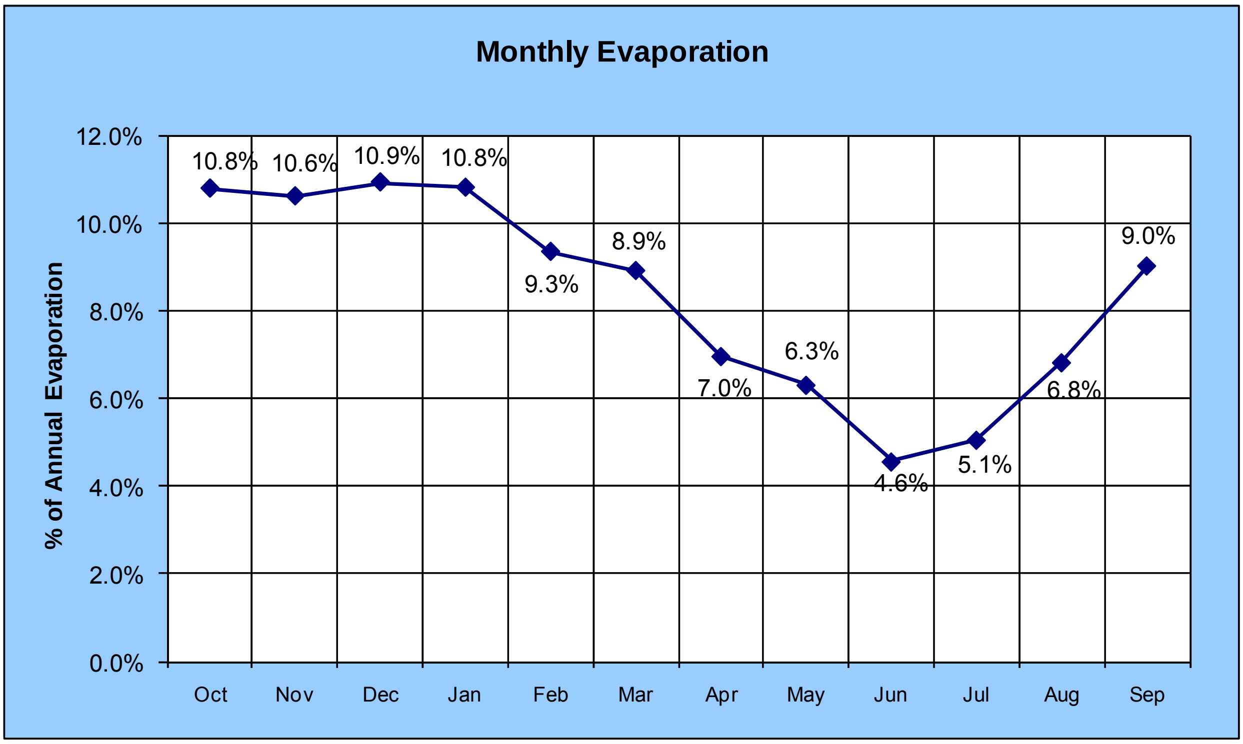


Figure 4-4: Monthly Distribution of Evaporation in the Study Area

(Note: Net MAE = 1 200 mm)

It then became apparent that a further condition was needed. The dam was assumed to be always full and additional releases were then also made immediately to make up the daily drawdown in the dam capacity. This was found to reduce the shortfalls, but increase the spills. However, the spills were not considered to be a problem because water has to be allowed to flow downstream. The rate of compensation for dam drawdown should therefore possibly be increased to cut back on the shortfalls. The shortfalls occur over very short periods and coincide with the flow recession curve. The maximum cumulative shortfall within any short period was estimated at about 0.8 Million m³ for a 4-day lag and 1.0 Million m³ storage capacity. For a 3-day lag and 0.75 Million m³ storage, the maximum shortfall is about 0.7 Million m³. In practice, the shortfalls can be regarded as one of the supply interruption events catered for in the Terminal Reservoir storage capacity.

A balancing dam storage volume somewhere between 0.5 and 1.0 Million m³ seems to be adequate, and if one accepts that the river response time does not exceed 3 days then somewhere between 0.5 and 0.75 Million m³ should be sufficient. This can be refined, together with the operating rules, which should include operating the dam below its full supply capacity in order to utilise high river flows better.

Further to the 6 hours storage for maintenance and the differences between inflow and outflow pumping rates and possible between 0.5 and 0.75 Million m³ required to manage river fluctuations, an additional reservoir compartment is required to allow for maintenance and desilting operations without affecting the transfer of water. Other issues to be considered in the design of the reservoir include prevention of algae blooms/growths, stratification and stagnation of the water in the reservoir. This will be achieved by providing a number of independent compartments in the reservoir that are filled and drained in succession to minimise water retention time. One compartment will be operational at a time and will be emptied before switching to the next one. Compartments were sized to ensure that it takes 12 hours or less to be drawn down in order to minimise retention time.

Table 4-6: Balancing Dam Compartment Sizing

Item No.	Component	Unit	Scenario 9	Scenario 9 in 2050
1.	6 Hours Maintenance Storage	Million m³	0.142	0.354
2.	River Fluctuation Storage	Million m³	0.750	0.750
3.	**Total Storage Required**	Million m³	**0.892**	**1.104**
4.	Maximum Emptying / Filling time per Compartment	hrs	12	5
5.	Number of Compartments Required	No.	3	4
6.	Compartment Capacity	Million m³	0.293	0.269
7.	Additional Compartment	No.	1	1
8.	Total Compartments	No.	4	5
9.	**Total Balancing Dam Volume**	Million m³	**1.173**	**1.343**

The full Scenario 9 (2050) would only require one (1) additional compartment compared to the Scenario 9 Stage 1 arrangement, and as the compartment sizes for Stages 1 and 2 are similar, constructing the reservoir to fulfil Stage 2 requirements during Stage 1 seems feasible. Depending on how well the river can be managed during Stage 1, adding additional compartments as required during Stage 2 remains an option. During Stage 2, two compartments can be operated simultaneously, increasing emptying and filling the time from 5 to 10 hours. This could obviate night-time operation of inlets and outlets. A draw-down rate of 1 m per hour should be adhered to as a general principle.

An analysis was also done to optimise the depth of the dam and the number of compartments from a cost perspective for a total operating storage capacity of 1 300 000 m^3. Only the principal direct costs were considered (earthworks and waterproof lining). The analysis showed that an 8 m deep four (4) compartment dam was perhaps the optimum solution, but also that there was not much difference in cost between a 3, 4 or 5 compartment dam (refer to **Figure 4-5**). Operation and maintenance issues therefore determined the final arrangement (**Table 4-5**). The requirement that the additional compartment provided for maintenance purposes had to have the same size as the operational compartments counted heavily against the two (2) compartment option.

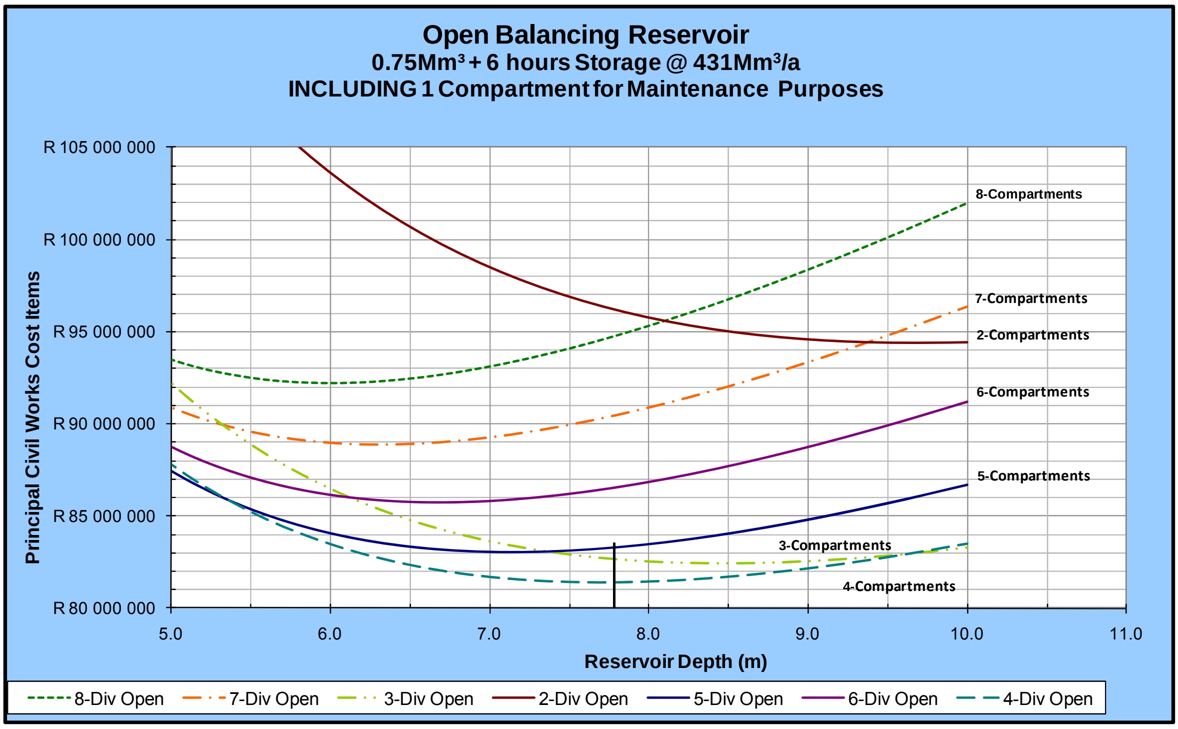


Figure 4-5: Assessment of Balancing Dam Depth and Number of Compartments

The Balancing Dam will be in the form of an artificial dam formed by shallow excavation and surrounding earthfill embankments. The footprint area of the dam including the Desilting Works is approximately 620 m x 440 m. Each of the compartments has an active storage capacity of 260 000 m^3/s and has top dimensions of approximately 400 m x 100 m. Placement of the dam is determined by the PMF level and the maximum Low-lift Pump Station head available. The High-lift Pump Station Floor was placed 1 m above the PMF level and the MOL of the dam needs to be 10 m above the High-lift pumps. . The difference between these levels governs the maximum depth of the reservoir. On both the preferred and alternative sites, the depth varied from 13.0 m at the inlet side to 10.5 m at the outlet side providing 1 m of freeboard above the Full Supply Level (FSL). The MOL is 9 m below the FSL.

When accurate survey and accurate flood levels become available, the depth of the reservoir can be optimised. Due to limited space available as well as evaporation considerations and not for pumps reservoir might be preferable.

Lining:

The dam will be lined with an appropriate waterproof lining system (HDPE or similar material) and suitable sub-surface drainage provided. The alternative location for the Balancing Dam is situated on dolomite. If this option is pursued, additional measures to prevent leakage, such as a double waterproof liner with a leakage detection system, may be required. See Details 01 and 02 on drawing WP9528-SHT-12-DD-VPW-007.

The option of providing a cover to eliminate evaporation losses was also investigated. The capital cost of a HDPE cover was R47 million and the present value of the R3.3 million annual cost of evaporation losses was R32.6 million (design life of 50 years at an interest rate of 10% per annum). This, together with the O&M difficulties associated with a cover, led to the abandonment of this option.

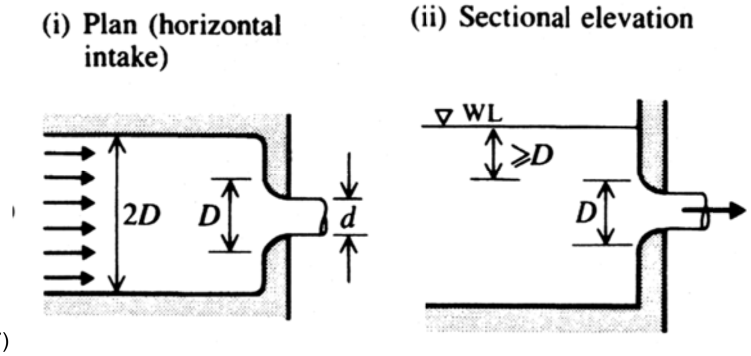
Inlets:

A manifold off the Desilting Works outlet distributes flow to the reservoir compartment inlets. Each inlet is fitted with a valve which is opened when a compartment needs to be filled. The diameter of the inlet pipe from the manifold is 2 100 mm, which at the design flow limits the velocity to 2.2 m/s. The inlet pipe discharges at the top of each compartment. An energy dissipation structure, such as a stepped spillway that was provided for the feasibility stage layouts, will be required at every inlet. Further investigation is required into a suitable top inlet structure with energy dissipation. A bottom inlet solution can also be considered.

Outlets:

An outlet structure from each compartment connects to the intake manifold of the High-lift Pump Station suction line. This manifold will have a diameter of approximately 3 m and is located just towards the south-western side of the reservoir.

The outlet structure is based on typical sump designs as recommended by Prosser (SeeFigure 4-6). A Bellmouth diameter (D) of 1.8 x Pipe diameter was used. The diameter of the outlet pipe to the manifold is 2 100 mm, which at the design flow limits the velocity to 2.2 m/s, resulting in a Bellmouth diameter of 3 780 mm.



(after Prosser, 1977)

Figure 4-6: Typical Sump Designs

While the water from the desilting channels may contain some suspended matter, it should be small enough and the water clear enough to be pumped without further concern. Any potential problem must be sorted out at the desilting channels. A minimum approach length of 5 x D is recommended. See drawing WP9528-SHT-11-LS-VPW-006.

Each compartment will also require a 25 m wide concrete spillway which discharges into collector which will return any spilled water to the river. The capacity of the spillway at an overflow depth of 0.5 m corresponds to the maximum system flow rate of 6.6 m^3/s. An erosion protected outlet structure will be provided where the water is discharged into the river.

De-silting:

The balancing dam will also be equipped with a silt flushing facility although only infrequent use, perhaps once every 10 years, is expected. A sump is provided on the inlet side of each compartment of the dam from where the floor slopes up towards the outlet to assist the maintenance teams with the cleaning of the compartments. The silt settling pond provided as part of the Desilting Works will also be used to separate the silt and the water flushed from the reservoir, to ensure that only reasonable clear water is discharged back to the river.

4.7. Upstream Impacts

In deciding on the sizing and layouts of the abstraction weir, the planning team was very sensitive to the impacts of the Works on existing infrastructure upstream of the weir. The existing railway line was considered in the present analyses and should not be impacted upon by the Works at Vlieëpoort. The planning team is also aware of other infrastructure such as low level river crossings, mine haul roads, storage areas and recreational facilities that may be affected, but the level of accuracy of the available survey data would exclude a definitive response on the impacts of the weir on high flood levels at this stage. No infrastructure will be affected with the water at the FSL of the weir and significant additional submergence may only start to occur for floods larger than the 1:50 year flood. The present flood line analyses are based on 20 m contour mapping and the results would at best be indicative only. Once detailed and accurate survey data becomes available comprehensive analyses of the river flood levels will be undertaken to confirm these findings. Follow-up discussions with all the interested and affected parties will then be arranged to facilitate transparency and acceptance by all.

Also see Section 4.10 for further details on the upstream infrastructure.

4.8. Bulk Electricity Supply

New bulk electricity lines will be needed from Eskom to the Low-lift Pump Station and the High-lift Pump Station. Eskom is planning a new 132 kV route in the Vlieëpoort vicinity. During the detail design phase liaison with them will be important to ensure that their routes are suitable. An area of 100 x 100 m was allowed for a switch yard next to the Balancing Dam and High-lift Pump Station. Both the Low-lift and High-lift Pump Stations will require 11 kV electricity supply from this switchyard. Double power supply will be need at each component of the Works according to the reliability/redundancy criteria. The bulk electricity supplies to Vlieëpoort are discussed in more detail in Section 5.4.5.

4.9. Access

Access will be required to the Abstraction Weir, Low-lift Pump Station, Desilting Works, High-lift Pump Station and the Balancing Dam. The existing access road to the Mooivallei farms from the Thabazimbi-Dwaalboom Road (D1649) on the right bank of the river can be upgraded and extended to the Abstraction Works. This access road will also provide access to the Alternative Balancing Dam Location on Portions 6 and 7 of Mooivallei. The current access Road on Portion 1 of Mooivallei can be upgraded to provide access to the preferred Balancing Dam location.

4.10. Vlieëpoort Dam Options

4.10.1. Introduction and Background

Water will be released down the CRW through the outlets of the Roodekopjes Dam (and possibly also the Klipvoor and Vaalkop Dams) to supply most of the water required for MCWAP.

The CRW catchment is part of the Crocodile River (West) and Marico River Water Management Area (WMA). The CRW originates in the northern suburbs of Johannesburg and drains to the northwest into the Limpopo River. The Hartebeespoort Dam (in the catchment area of the Roodekopjes Dam) and the Klipvoor, Roodekopjes and Vaalkop Dams are all significant dams upstream of the weir site at Vlieëpoort on the CRW from which water will be abstracted to augment the supplies to the Lephalale area. The natural runoff from the catchments of these dams is already fully utilised, either by direct abstraction from the dams and their catchments and/or to regulate the natural runoff downstream of the dams, to supply existing users in the catchment. Some of the effluent flows from the Johannesburg and Tshwane area are also being used to augment the supplies to these users. In addition, the CRW forms part of the Limpopo Watercourse that is shared by South Africa, Botswana, Mozambique and Zimbabwe and water is also required for the riverine ecosystems downstream of the dams.

This section sets out the basis that has been adopted to obtain a first order estimate of the additional water that will be available from the system if additional storage is provided at Vlieëpoort. A map showing the Vlieëpoort area is presented in **Figure 4-14**.

This first order estimate will provide a basis on which to decide whether significant additional utilisable water can possibly be secured by a dam at Vlieëpoort and whether more detailed studies should be performed. Any interpretation of the results of this analysis must, however, also consider the fact that the proposed dam must not cause significant harm to the downstream users and riverine ecosystems. With this in mind, compensatory water releases may therefore be required.

The WR90 reports (Midgley *et al*, 1994) have formed the basis for this first order estimate of the additional water that can be secured by means of storage at Vlieëpoort.

4.10.2. Description of Catchment and Water Resources

The gross catchment area of the CRW at Vlieëpoort is approximately 26 830 km^2. The net catchment area is about 26 170 km^2 as a result of some endoergic areas. The net catchment area between the Klipvoor, Roodekopjes and Vaalkop Dams and Vlieëpoort is approximately 7 940 km^2.

Downstream of the Klipvoor, Roodekopjes and Vaalkop Dams, the CRW is characterised by a very flat slope and a number of prominent meanders in flat alluvial plains. Preliminary desktop investigations have led to the conclusions that the alluvial plains are underlain by relatively coarse ventricular alluvial deposits that are hydraulically connected to the CRW. These have created alluvial aquifers, equivalent to reservoirs that are recharged by rainfall and from the river and from which water is also abstracted by the irrigators.

The average MAR over the catchment between the Klipvoor, Roodekopjes and Vaalkop Dams and Vlieëpoort is about 602 mm, which is the same as the rainfall at Vlieëpoort. The gross mean annual Symon's pan evaporation at Vlieëpoort is about 1 750 mm, resulting in a net mean annual lake evaporation of about 1 200 mm during the dry period critical for the yield of the dam.

The net natural MAR at Vlieëpoort is about 575 Million m^3. Between the Klipvoor, Roodekopjes and Vaalkop Dams and Vlieëpoort site, the net natural MAR is about 134 Million m^3.

The total (net) storage capacity of the existing dams upstream of Vlieëpoort is about 550 Million m^3, which is equivalent to about 96% of the net natural MAR.

The natural runoff from the catchments of these dams is already fully utilised to supply existing users in and downstream of the catchments of the existing dams. Use is also being made of some of the effluent flows from the Johannesburg and Tshwane areas to augment the supplies to the existing users. The releases from the Roodekopjes Dam (and to a lesser extent also from the Klipvoor and Vaalkop Dams) are varied to augment and make the most use of the natural runoff in the CRW downstream of the dam. Water is also abstracted from the alluvial aquifers underlying the flood plains when the surface flow in the river is insufficient to supply the users. These aquifers are then naturally recharged during periods of higher flow. The situation downstream of Vlieëpoort is similar since the users downstream of Vlieëpoort also depend mostly on river flows past Vlieëpoort.

4.10.3. Sedimentation

The sedimentation estimates have been based on the work of Rooseboom *et al*, (1992) and Brune (1953).

The catchment area between the Klipvoor, Roodekopjes and Vaalkop Dams and Vlieëpoort is in Sediment Yield Region 1 with a medium sediment yield potential. The corresponding standardised average sediment yield for this region is 49 t/a/km^2. For an 80% probability of not being exceeded the average sediment yield amounts to about 800 000 t/a. After 50 years, this would translate to a volume of about 30 Million m^3 for 100% trap efficiency, assuming that the trap efficiency of the Klipvoor, Roodekopjes and Vaalkop Dams is 100%.

However, for storage capacities between 15 Million m^3 and 100 Million m^3 the trap efficiencies will be between about 80% and 95%, respectively. This is in line with the rule of thumb that a low dam or weir with a height somewhere in excess of 5 m built in a large river is at a high risk of trapping large volumes of sediment. The volumes lost to sedimentation could therefore be about 24 Million m^3 to 28 Million m^3 after 50 years. At a probability of only 50% of not being exceeded, the aforementioned volumes lost to sedimentation would reduce to about 12 Million m^3 to 15 Million m^3.

The sediment volume in a reservoir is not a linear function of time because the sediment gradually compacts. After 20 years, the volume of stored sediment would already amount to about 65% of the volume after 50 years. Significant increases in water levels in the upstream delta region of the dam are also likely.

4.10.4. Method Adopted for the Hydrological Analysis

The main purpose of these analyses has been to estimate the incremental yield benefit of additional storage at Vlieëpoort, and not to estimate the total yield of the system.

On the basis of the catchment characteristics described above, it has therefore been accepted that it would be reasonable to work with a system configuration where all the existing storage in the catchment upstream of Vlieëpoort (550 Million m^3) has been transposed to Vlieëpoort to provide the baseline to estimate the additional yield that can be obtained with storage at Vlieëpoort. The additional yield available from the natural runoff from the catchment would therefore be equal to the difference between the yield of a dam at Vlieëpoort having a net storage capacity equal to 550 Million m^3 plus the additional stored volume at Vlieëpoort minus the yield of a dam having a net storage capacity of 550 Million m^3.

Table 4-7: Gross System Yield

NET STORAGE CAPACITY (million m^3)		GROSS SYSTEM YIELD (million m^3/a)			
		1:10 Year RI		1:20 Year RI	
Total Storage	Storage Increase	Total Yield	Increase in Yield	Total Yield	Increase in Yield
550	0	429	0	358	0
575	25	440	11	366	8
590	40	446	17	370	12
600	50	450	21	372	14
625	75	460	31	379	21
650	100	468	39	385	27

The analyses were therefore done using the net natural MAR of the entire CRW catchment at Vlieëpoort and the weighted average regionalised storage-gross draft characteristics of the individual quaternary catchments upstream of Vlieëpoort as obtained from WR90 (Midgley *et al*, 1994). The yields were determined for the existing system and the existing system plus the additional storage for recurrence intervals of 1:10 years and 1:20 years, which are roughly equivalent to risks of emptying the dams of 10% and 5%, respectively.

The results of the estimates of the corresponding gross yields are summarised in the **Table 4-7** above. The results of the analyses are also presented in **Figures 4-7** and **4-8**.

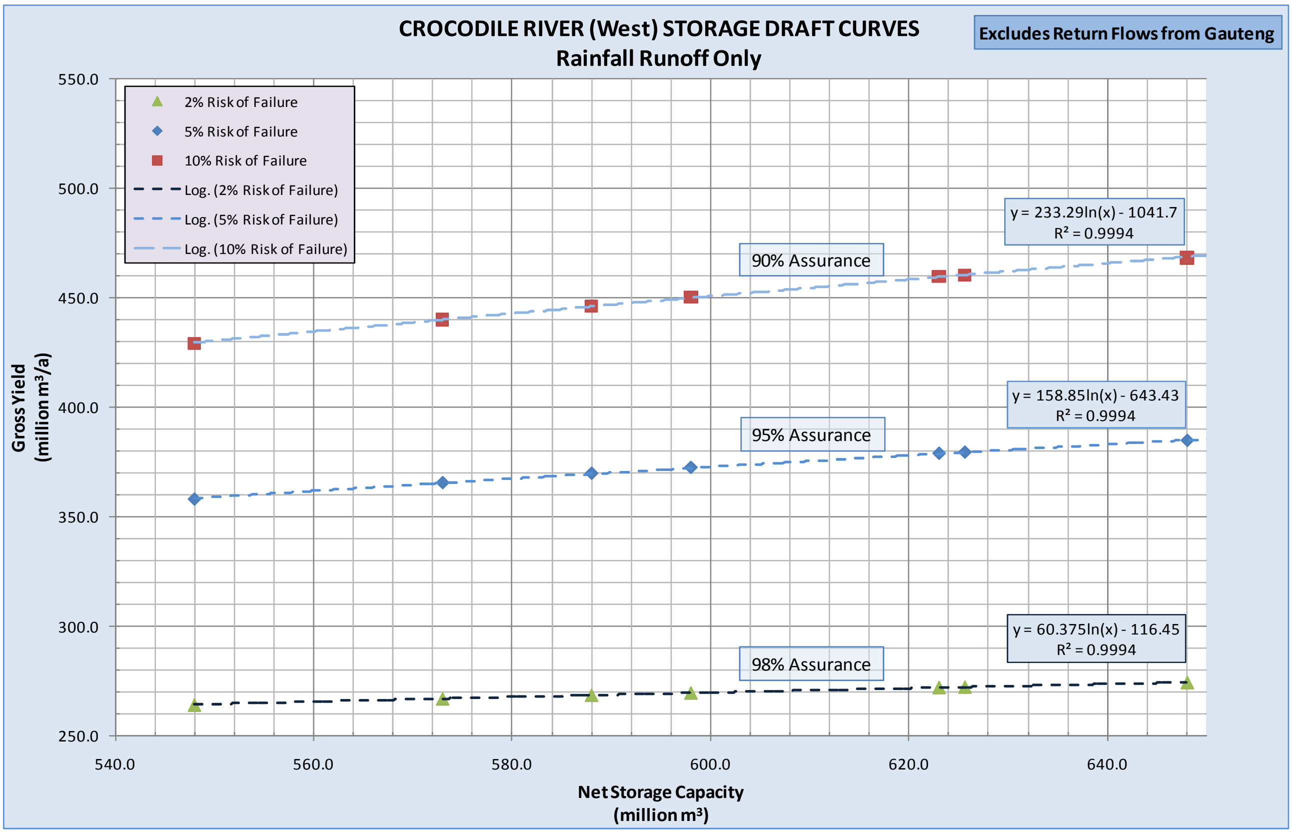


Figure 4-7: Storage Draft Curves for Crocodile River (West)

Building a dam at Vlieëpoort will reduce the downstream long-term MAR by an amount equal to the increase in the gross yield that can be expected from the dam. Compensation releases will therefore have to be made to offset any negative effects of this on the downstream water users in South Africa, on the riverine ecosystems in the CRW and the Limpopo River and the flow that is required by the downstream Watercourse States.

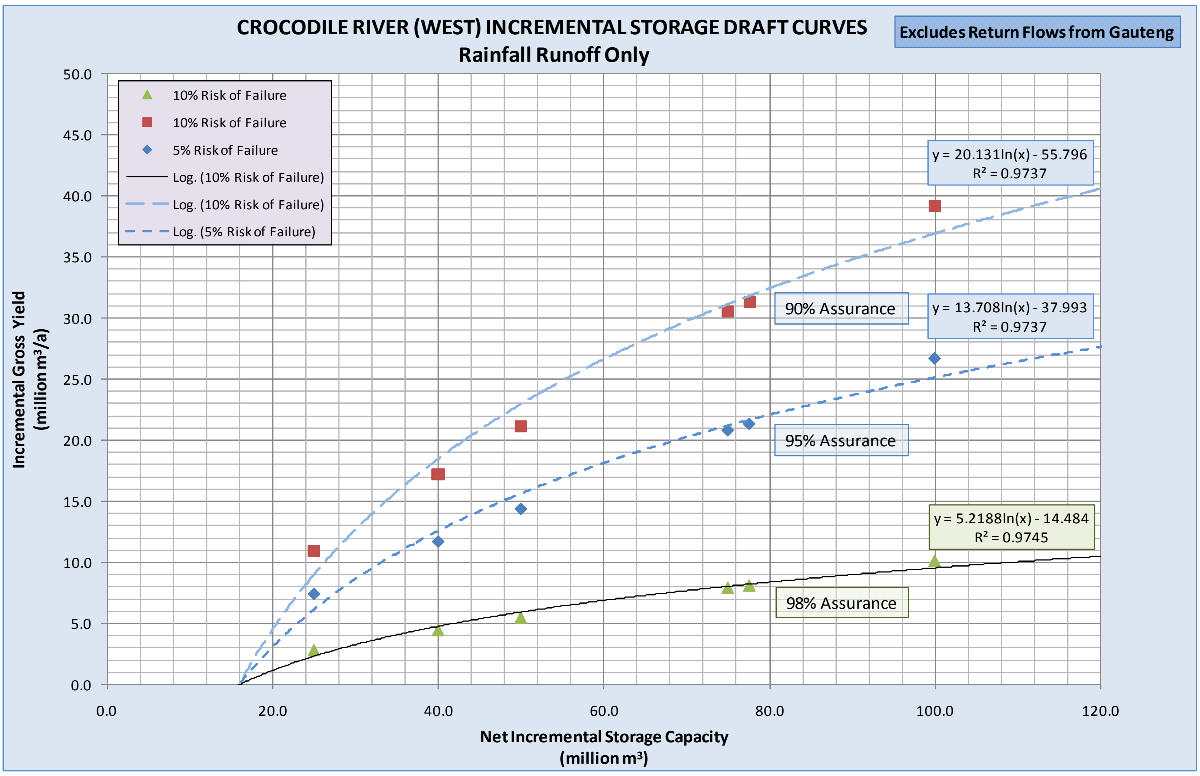


Figure 4-8: Incremental Storage Draft Curves

In order to obtain the actual net yield benefits provision must be made for both the loss of storage capacity due to sedimentation and the evaporation losses from the surface of the reservoir at Vlieëpoort. The evaporation losses and therefore the reduction in gross yield will be about equal to the average water surface area of the reservoir during the driest period with a net unit evaporation loss of 1 200 mm/a. The typical net yields that can be expected from a dam at Vlieëpoort are given in the **Table 4-8** and **Figure 4-9** below.

The sedimentation effects and therefore the net reservoir capacities have been determined on the assumption that the sediment will all be deposited below the FSL of the dam. This is, however, unlikely to be the case, but no attempt has been made in the estimate of the net storage capacity for the volume of sediment that will be deposited above the FSL.

From **Table 4.8**, it is therefore clear that the effects of both sedimentation and evaporation losses will eventually reduce the net yield of a significant dam at Vlieëpoort to an insignificant amount.

Table 4-8: Additional Yield

STORAGE AT VLIEËPOORT (Million m^3)		FIRST ORDER ESTIMATE OF ADDITIONAL YIELD (Million m^3/a)			
		GROSS		NET**	
GROSS	NET*	1:10 year RI	1:20 year RI	1:10 year RI	1:20 year RI
25	5	3	2	0	0
50	30	13	9	0	0
75	55	23	16	3	0
100	80	32	22	5	0

Notes:

* Based on an average sediment yield of 20 Million m^3 after 50 years between the Klipvoor, Roodekopjes and Vaalkop Dams and Vlieëpoort.

** Allowing for both sedimentation and evaporation losses.

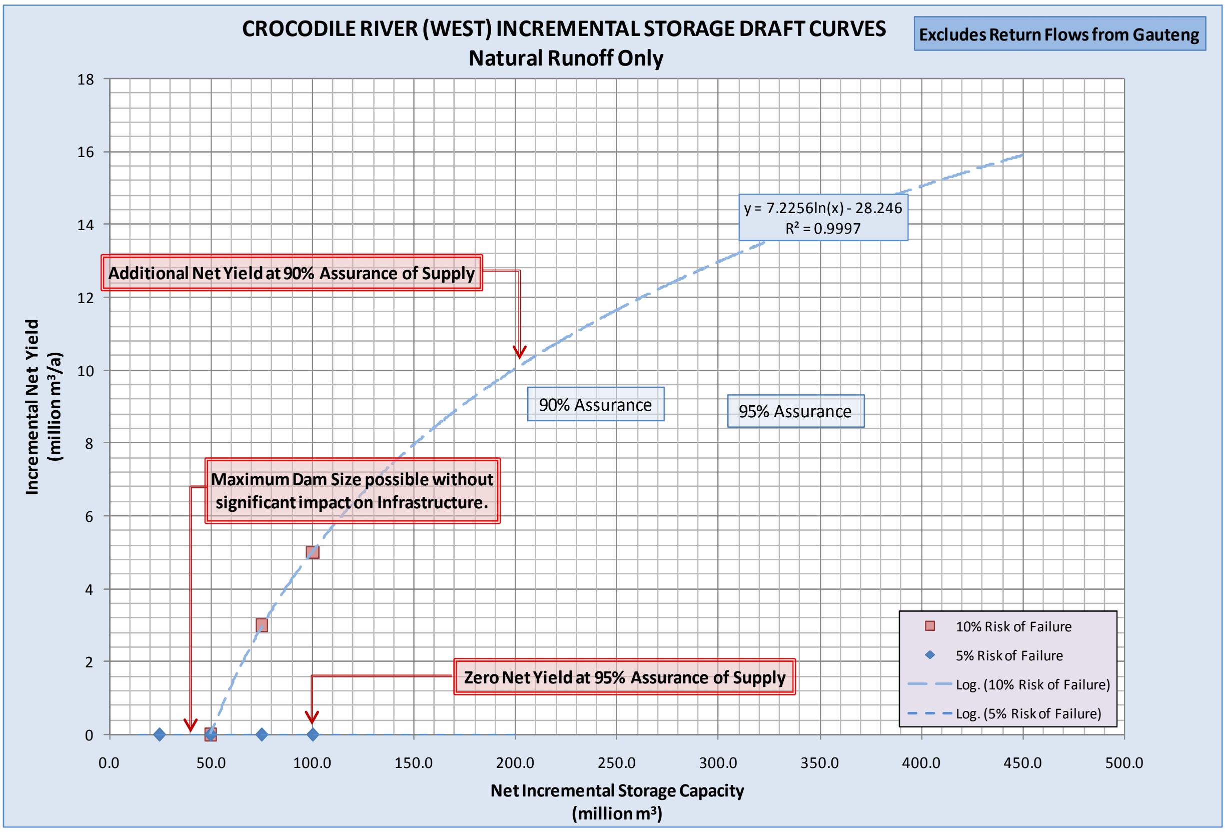


Figure 4-9: Net Incremental Storage Draft Curves

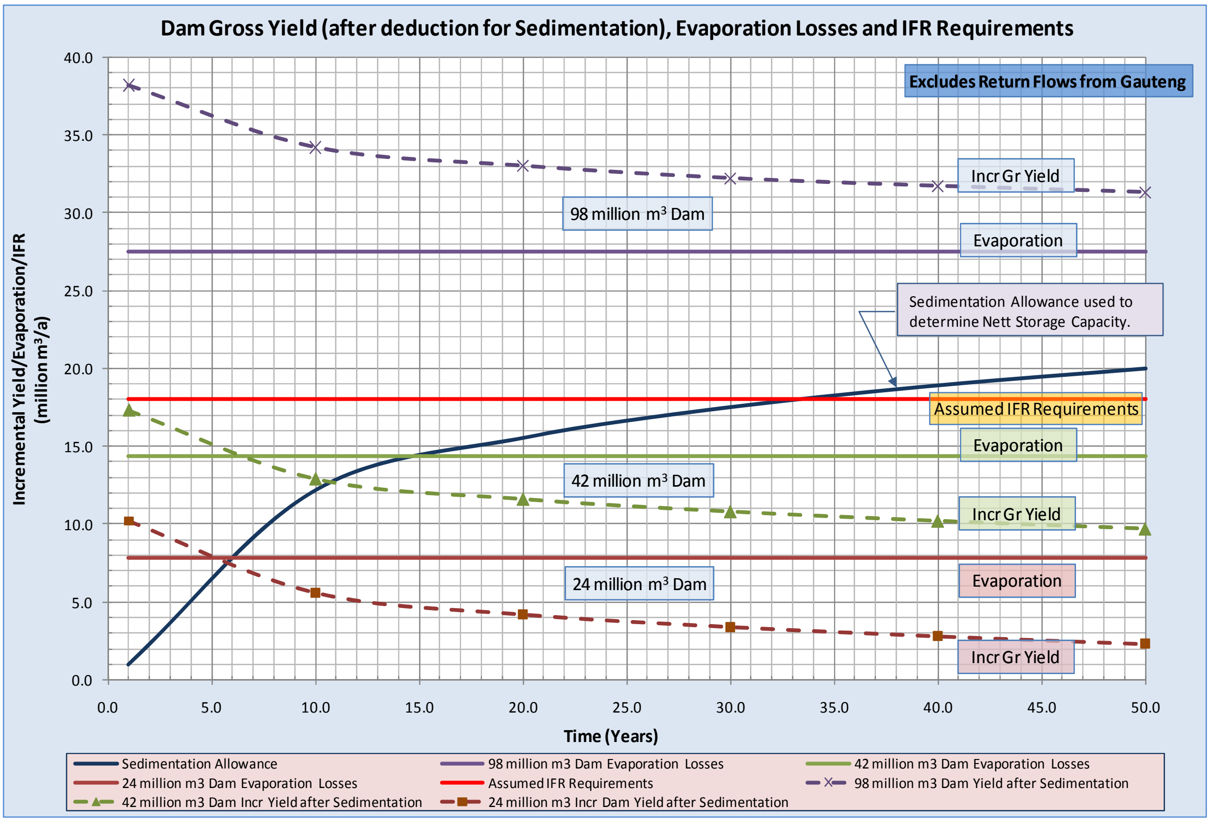


Figure 4-10: Contribution of Evaporation in reducing Gross Incremental Yield

- Some additional yield from a dam at Vlieëpoort is therefore possible, but referring to **Figure 4-9**, the required dam gross storage capacity will have to be well in excess of 100 Million m^3 before any additional exploitable yield could be possible;
- Reduction in gross yield due to the impacts of sedimentation, evaporation and Environmental Flow Requirements (EFRs) (the main components affecting net incremental yield are shown in **Figure 4-10**) will not be easily made up by increasing dam size; and
- The MCWAP water requirements will not be met by a dam at Vlieëpoort alone.

4.10.5. Dam Capacity Analysis

Existing survey data, consisting of 5 m interpolated DEM data, was used to generate DTM models from which a storage capacity curve for the Vlieëpoort Dam was prepared. 1:50 000 Mapping and aerial photography were used to identify infrastructure that would be affected and to estimate associated levels at the infrastructure. The storage capacities of the dam options considered and details of affected infrastructure in the upstream dam basins are shown in **Tables 4-9** and **4-10**.

Table 4-9: Key Data for Dam Options that were considered

Item No.	Description	Gross Storage Capacity (1),(3) (million m^3)	Additional Net Yield (2) (million m^3/a)	FSL (5) (masl)
1.	Abstraction Weir	0.3	0	893.0
2.	Dam Option 1	25	0	900.0
3.	Dam Option 2 (4)	50	0	902.5
4.	Dam Option 3	75	3	904.0
5.	Dam Option 4	100	5	905.0
6.	Dam Option 5	500	17	914.0

Notes

1. Storage capacities and flood levels determined from existing satellite DEM and 1:50 000 scale mapping, which was considered acceptable for first order evaluations.
2. Based on 1:10 year recurrence interval (RI) first order determination. Also see **Figure 4-9**.
3. Indicative only. When detail terrain survey becomes available the assessments that were done for gross storage capacity and river flood levels should be recalculated.
4. For dam options larger than Option 2 major relocation of infrastructure on the flood plain upstream of the Kumba mines will be required.
5. Average bed slope = 1:3 000 along most of the river. Add 12 m to estimate 1:100 year RI flood levels and 20 m to estimate PMF flood levels.

Table 4-10: Inferred Key Infrastructure Levels Upstream of Dam

Item No.	Description	Level (2) (masl)
1.	Riverbed level at Weir/Dam	890.0
2.	Left Bank/Right Bank level	893.0 / 896.0
3.	Kumba Low Level River Crossings	898.0
4.	Thabazimbi Golf Course	902.0
5.	Kumba Haul Roads on Flood Plain	902.0
6.	R510 level in poort approaching Thabazimbi	908.0
7.	Hugo’s Weir riverbank level	914.0
8.	Koedoeskop riverbank level	929.0
9.	Boschkop riverbank level	940.0

Notes:

1. Levels were determined from existing satellite DEM and 1:50 000 mapping.
2. Indicative only. When detail terrain survey becomes available the level assessments that were done should be determined accurately and checked in the field.

The storage capacity and FSL surface area curves are presented in **Figure 4-11**. Comparing Figures 4-9, 4-10 and 4-11, it is clear that:

- A dam solution to increase the amount of water available in the CRW for use by MCWAP is not feasible.
- Using a dam at Vlieëpoort to store water for use by the MCWAP to mitigate shortcomings with water requirements management (dam releases, irrigation, unauthorised use, surplus water passing Abstraction Works, etc.) is unlikely to be successful as losses due to high levels of evaporation from the dam will negate any of the benefits gained by storage.

4.10.6. Meeting with the Crocodile Farmers Forum on 14 May 2009

4.10.6.1. Discussion

The following points were highlighted:

(i) The analyses only considered natural run-off in the CRW catchment. Inter-basin transfers, such as the present transfers from the water care facilities serving the metropolitan areas of Tshwane and Johannesburg (and any future augmentation thereof), were not taken into account. The majority of the water emanating from these facilities has the Vaal River System as source. These return flows amount to perhaps 40 to 60 Million m^3/a (presentation by Seef Rademeyer).

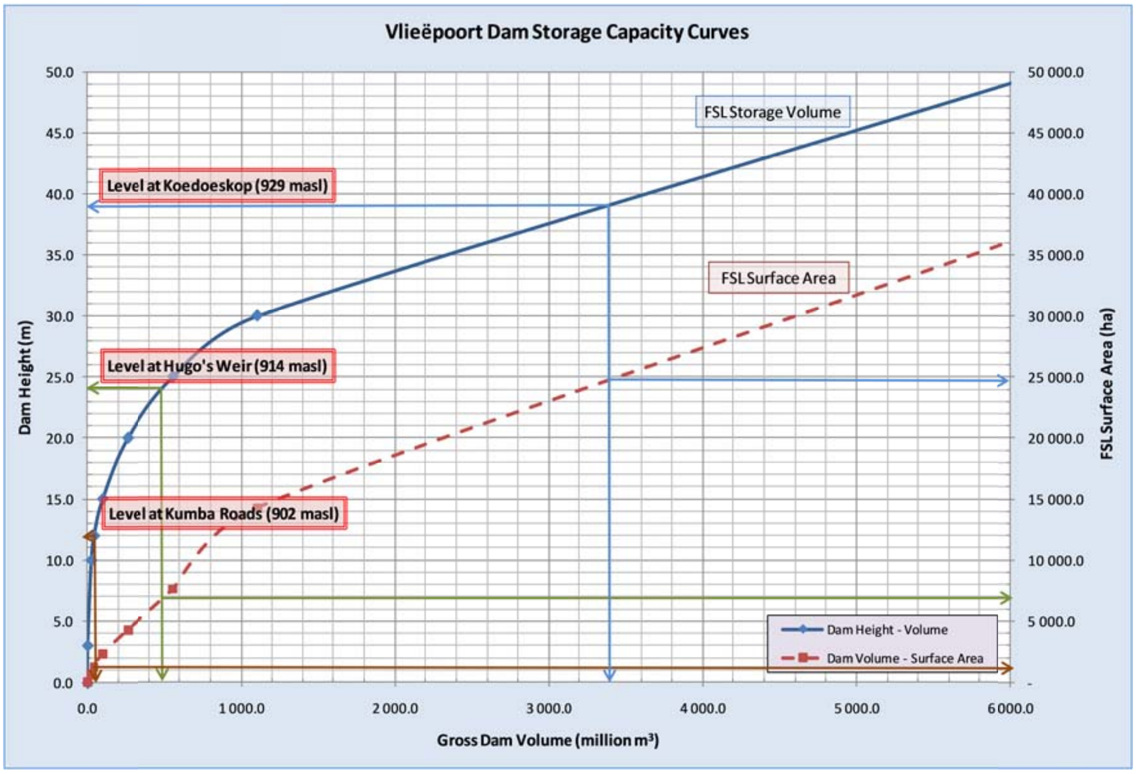


Figure 4-11: Vlieëpoort Dam Storage Capacity and Surface Area Curves

(ii) The impacts of these flows can be seen on the increased average dam levels recorded over the past decades in the main dams of the CRW catchment, and in particular that of Hartebeespoort, Roodekopjes and Klipvoor Dams.

(iii) Due to the nature of these flows a fairly constant flow hydrograph results which makes the selection of a small run of river abstraction works ideal.

(iv) The total storage capacity of the upstream dams amounts to 96% of the MAR of the CRW at Vlieëpoort. The observed floods that pass through the system at Vlieëpoort (and the floods from the Bierspruit and Sandspruit tributaries) are required to produce the required downstream environmental flows. The argument for retaining the floods and letting them pass slowly fails because of the high evaporation losses from such a storage facility at Vlieëpoort.

(v) Sedimentation will be a problem and estimated sedimentation volumes will form a significant percentage of the storage capacity of a dam of realistic size at Vlieëpoort. The realistic size of a dam at Vlieëpoort is considered to be the size that would not affect the area under control of the CRW Irrigation Board and have a detrimental impact on the surrounding mines and towns. This criterion would limit the size of such a dam to 100 Million m^3 maximum.

(vi) Additional water, over and above the normal releases for irrigation, will be released from the upstream dams (Kilpvoor, Roodekopjes, Hartebeespoort and Vaalkop) to provide for the requirements of the MCWAP. The unintended side effect of these releases will be that the upward trend on average dam levels, as noted in (ii), will be turned around and these levels will start returning to the levels that existed before the Tshwane-Johannesburg return flows became significant in the system. This will in turn result in the extra storage capacity in the existing dams that would have been created by a dam at Vlieëpoort.

4.10.6.2. Slides Presented

Further to the figures discussed above, the following slides were also presented and discussed at the meeting.

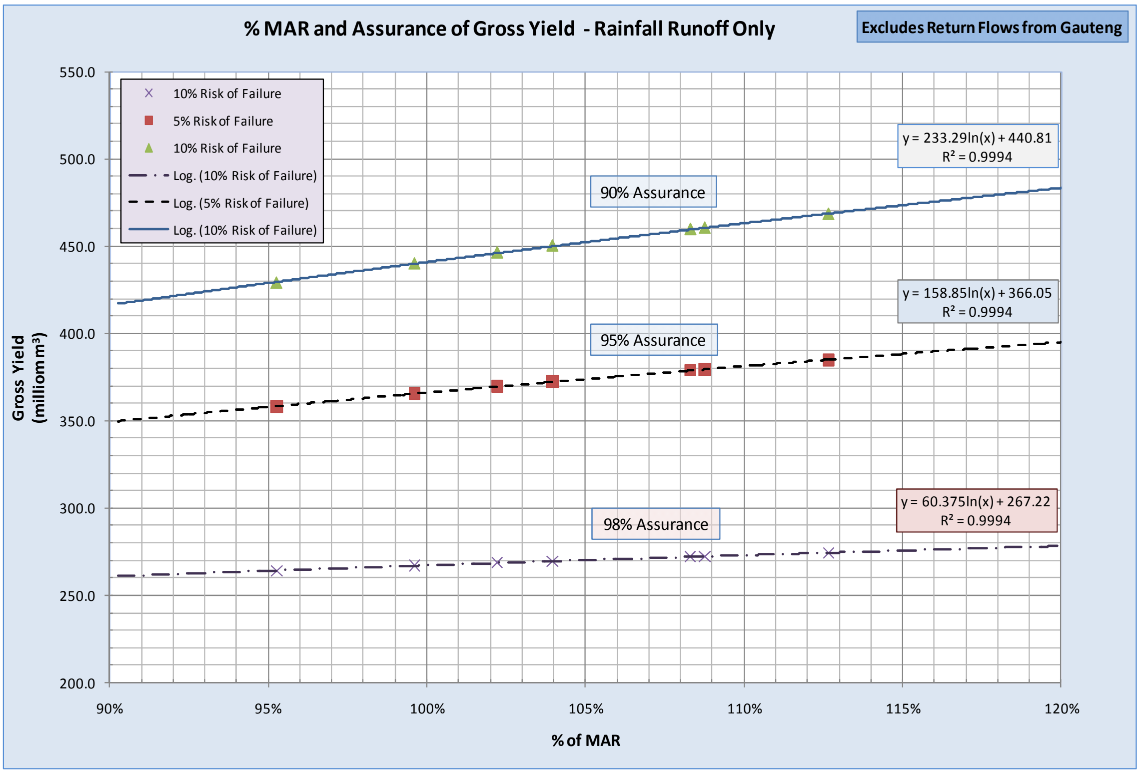


Figure 4-12: Assurance of Supply and Gross Yield

The total storage capacity of the dams in the catchment account for 96% of the MAR of the CRW catchment at Vlieëpoort.

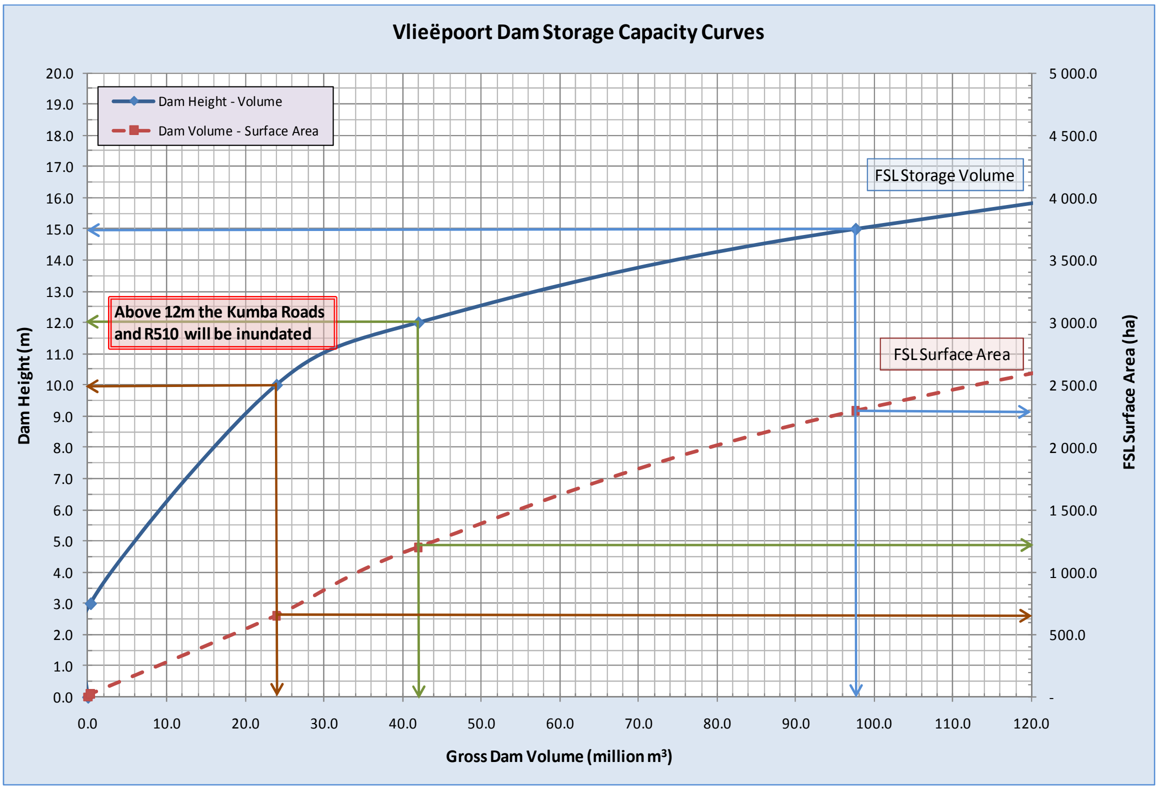


Figure 4-13: Gross Storage Capacity Curves for Vlieëpoort Dam

A map of the areas that could be affected by the possible Vlieëpoort Dam is shown in **Figure 4-14**. The map is an extract of the existing 1:50 000 mapping of the area and only provides 20 m contour interval information plus spot levels at key locations. The shaded areas is therefore only an indication of the area that could be inundated and the map should be read in conjunction with the data presented in the tables and figures discussion in the preceding sections of the report.

A similar first order level analysis was also undertaken for a possible dam at the present Boschkop abstraction works site. The storage capacity curves for a dam at Boschkop is comparable with that of the possible Vlieëpoort Dam and the conclusions arrived at for the Vlieëpoort Dam options would apply equally to the Boschkop Dam options.

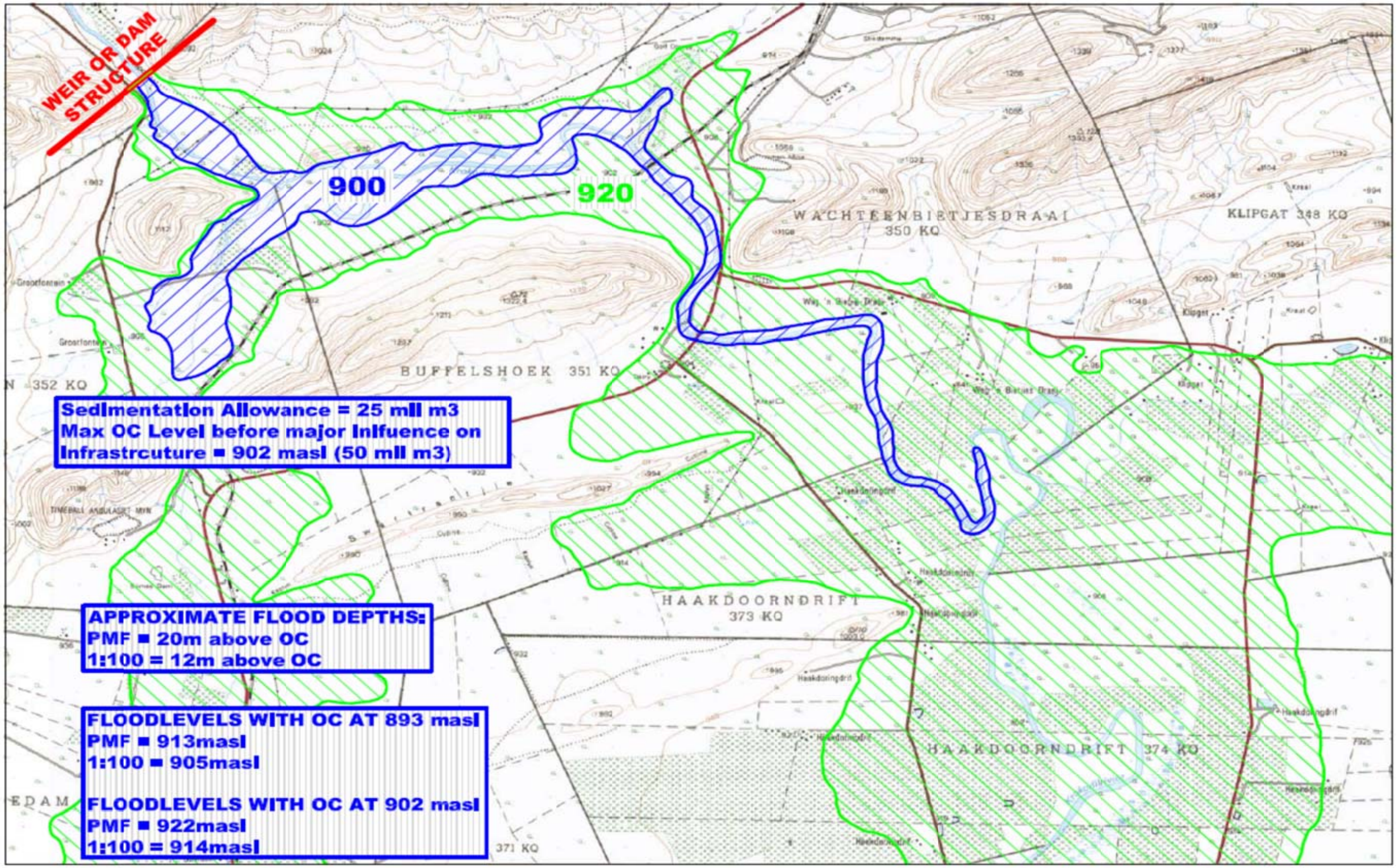


Figure 4-14: Indicative Plan of the Possible Vlieëpoort Dam Basin

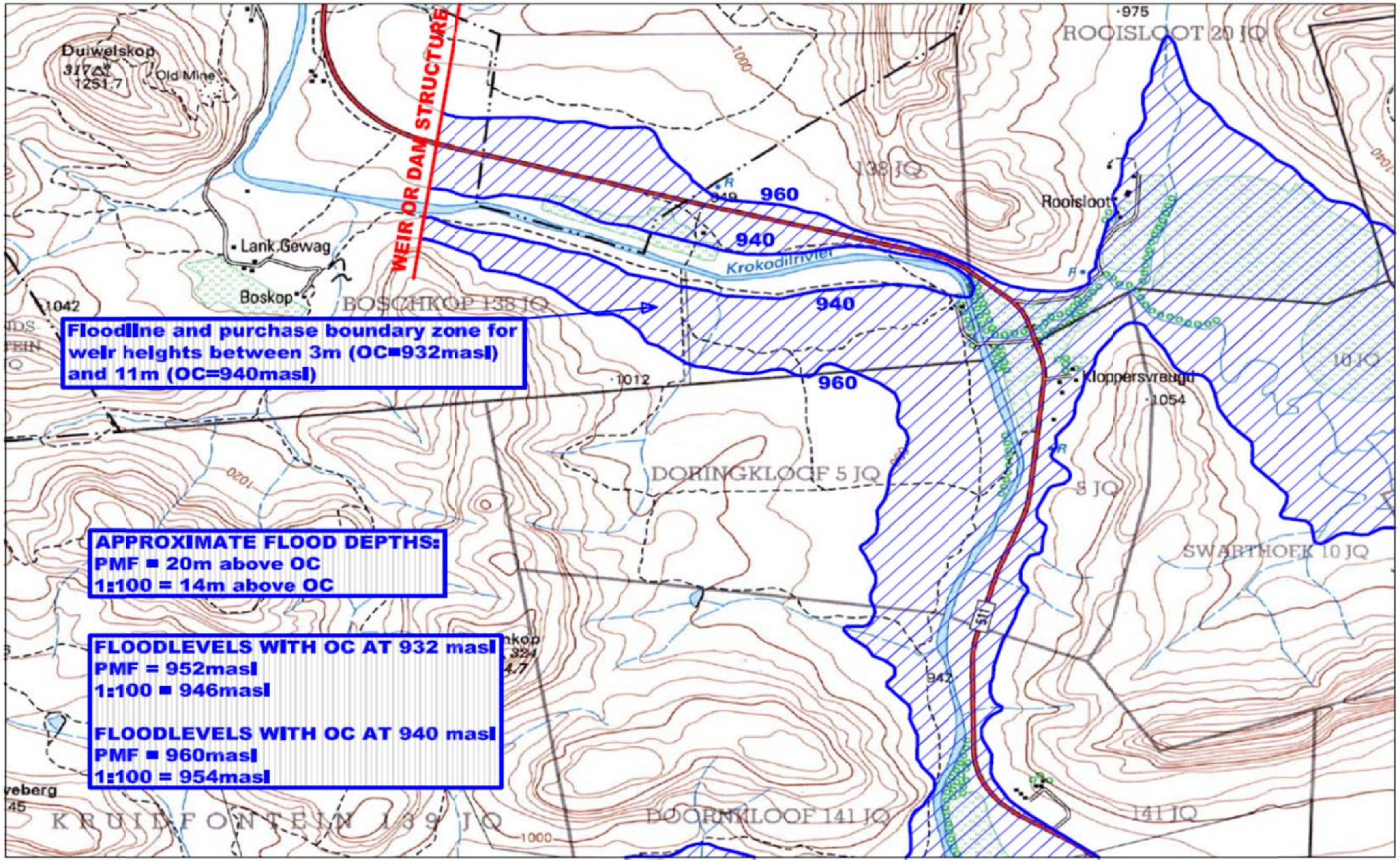


Figure 4-15: Indicative Plan of the Possible Boschkop Dam Basin

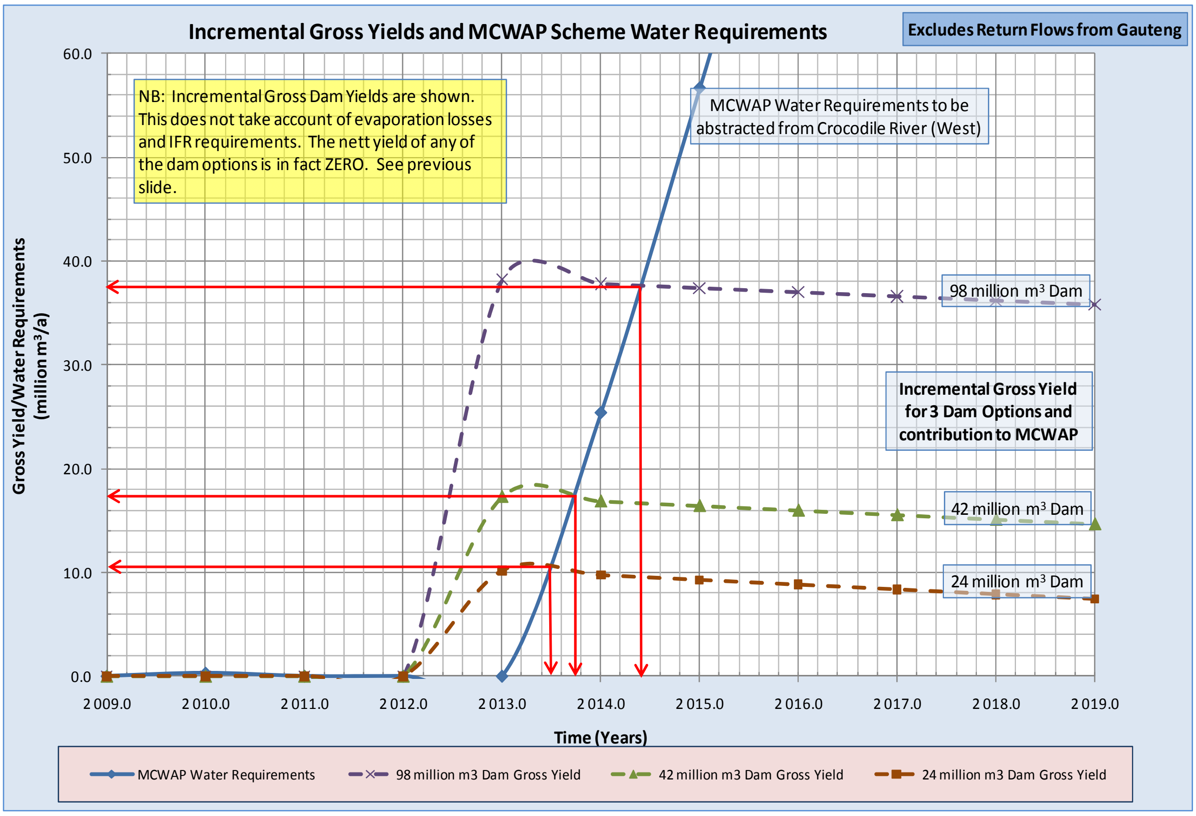


Figure 4-16: Dam Contribution to MCWAP Water Requirements

The possible dam would therefore at the very best and ignoring evaporation losses and downstream EFRs, provide for the needs of the MCWAP until mid-2014.

During the discussions, the Forum representatives enquired about the possibility of raising Klipvoor Dam to create additional storage capacity. Klipvoor Dam presently has a full storage capacity of 42.9 Million m^3, which is similar to Vlieëpoort Dam Option 2 (**Table 4-9**). Although Klipvoor Dam would have a much more efficient storage capacity curve than Vlieëpoort Dam and consequently have less evaporation losses, the inherent problems associated with the CRW catchment of available additional yield and onerous EFRs associated with new dam construction in a cross-border catchment remain. In fact, because Klipvoor Dam is located on the Pienaars River, which is a tributary of the CRW, the ratio of additional yield to additional storage, established for Vlieëpoort Dam is probably much worse. The raising of Klipvoor Dam is therefore expected not to be of benefit to the MCWAP Scheme.

4.10.7. Conclusions

Refinements of the work that was done will be possible once detailed mapping of the Vlieëpoort area is available and after a more detailed hydrological analysis of the catchment has been done.

The conclusion that additional yield that can be created by adding storage in the CRW catchment is not likely to make a significant contribution to supplementing the water requirements of the MCWAP is, however, unlikely to change.

4.10.8. References

1. Brune G.M., (1953). *Trap Efficiency of Reservoirs*. Am. Geophys. Union Trans. v. 34, No. 3.
2. Rooseboom A, Verster E, Zietsman H.L. and Lotriet H.H., (1992). *The development of the New Sediment Yield Map of Southern Africa*. Water Research Commission Report No 297/2/92.
3. Midgley D.C., Pitman W.V and Middleton B.J. (1994). *Surface Water Resources of South Africa 1990*. Water Research Commission Report Nos. 298/1.1/94 and 298/1.2/94.
4. AGES, (2007). *The Assessment of Water Availability in the Crocodile (West) River Catchment by Means of Water Resource Related Models in Support of the Planned Future Licensing Process*. AGES Report No. AS-R-2007-01-31.

4.11. Layout Drawings

A register of the Feasibility stage drawings that was prepared for the Study is presented in **Table 4-11**.

Sources of drawing data that were used include:

- Hard copies of 1:10 000 ortho-photo maps. The 5 m contours used on the layout drawings were digitised from these maps. The contours were used in the river losses computer models and for the Abstraction Works site layouts.
- 2003 Aerial photography. Photographs were available for the Mokolo River, but none were available for the section of the CRW that fell inside the Study area.
- Photographs taken during site visits were used to expand the low level of detail obtained from the 5 m digitised contours.
- 1:50 000 Maps.

Table 4-11: Drawing Register

DWA Project No.	Drawing Sheet No.	MCWAP NUMBER			Drawing Title
		Series	Com-ponent	Drawing No.	
Vlieëpoort Abstraction Works					
WP 9528	1	LD	VPW	001	Site Layout
WP 9528	2	LD	VPW	002	Weir Layout

DWA Project No.	Drawing Sheet No.	MCWAP NUMBER			Drawing Title
		Series	Com-ponent	Drawing No.	
WP 9528	3	DD	VPW	001	Weir Elevation
WP 9528	4	LD	VPW	001	Site Layout Alternatives
WP 9528	5	DD	VPW	003	Weir Section Details
WP 9528	6	DD	VPW	004	Low-lift Pump Station Layout and Elevation
WP 9528	7	DD	VPW	005	Low-lift Pump Station Sections
WP 9528	8	LD	VPW	003	Balancing Dam Layout
WP 9528	9	DD	VPW	006	Desilting Works Layout and Cross Section
WP 9528	10	LS	VPW	001	Desilting Works Long Section
WP 9528	11	LS	VPW	002	Balancing Dam Compartment Long Section
WP 9528	12	DD	VPW	007	High-lift Pump Station Layout and Section

A3 sized versions of the drawings are included in **Appendix C** of the Report.

5. HIGH-LIFT PUMP STATION AND TRANSFER SYSTEM

5.1. Introduction and Background

The following components form part of the Transfer System from Vlieëpoort Weir to Steenbokpan:

- High-lift Pump Station;
- Rising main and gravity pipe sections; and
- Break Pressure and Operational Reservoirs.

The infrastructure capacity is based on the water requirements and design cases defined under Sections 3.1 and 3.2.

A number of different route options were considered during the Pre-feasibility stage of the investigation. The respective routes were evaluated on the following grounds:

- Financial, engineering and economic evaluation to determine the lowest combined scheme cost (URV);
- Environmental and social impacts;
- Geotechnical screening;
- Access to the proposed pipeline route; and
- Technical, including CP requirements, constructability, hydraulic and practical considerations.

The Pre-feasibility investigation concluded the following:

- The abstraction works must be located at Vlieëpoort as described under Section 4.
- The pipeline from Vlieëpoort to an Operational Reservoir located on the Farm Rooipan 357 LQ must follow the Central route along the railway line. From the Operational Reservoir, a gravity pipeline will convey the water north-west towards Steenbokpan, where it will link to the pipeline from Lephalale constructed as part of Phase 1 of the MCWAP. Refer to Drawing 9528/LD/CTS/001 and 002 included in **Appendix A** for the locality and layout of Phase 2A of the MCWAP.
- The pipeline profile lends itself to the construction of a BPR closer to Thabazimbi with a gravity pipeline to the Operational Reservoir. This option is investigated further under Section 5.2.
- The Pre-feasibility economic analysis found in favour of an un-phased implementation of the Phase 2A Transfer Scheme to supply the 2030 water requirements. The difference between the phased and un-phased implementation approaches was,

however, found to be small and should be revisited as part of the detail design of the project when updated water requirements and growth patterns are known.

5.2. Phase 2A Route Options Analysis

Three alternatives to the central route were identified during the Feasibility stage to further investigate the impact of the local high point along the profile, approximately 30 km north of Vlieëpoort on the hydraulic performance of the system. The feasibility route (Central) and alternatives are shown on the attached layout plan (DWG 9528 LD/CTS/003) and the superimposed pipeline profiles of the respective routes from Vlieëpoort to Steenbokpan are illustrated below.

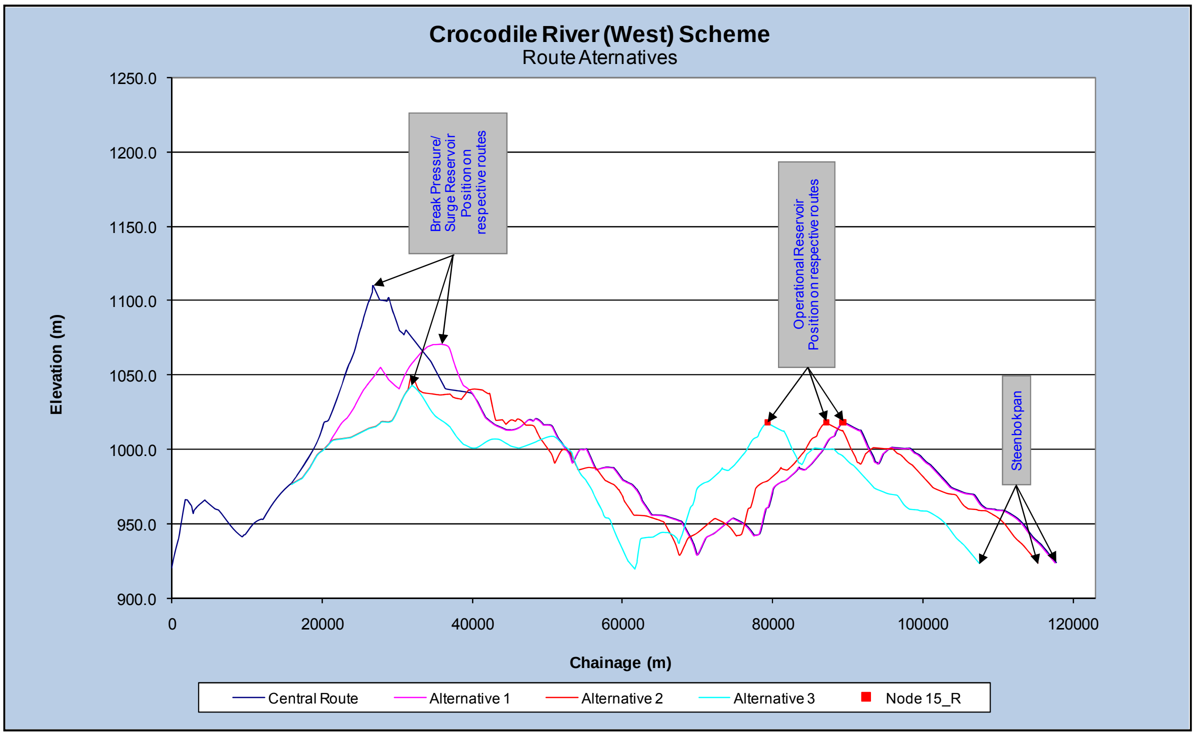


Figure 5-1: Phase 2A Alternative Route Pipeline Profiles

The planning team evaluated the alternative routes qualitatively and concluded as follows:

- Despite being the shortest, Alternative 3 is not regarded as viable due to its location along a high voltage power line and because the route fragments a number of farm properties which will result in an unacceptable social impact.
- Apart from a lower ground level in the vicinity of chainage 32 000 m, there is very little difference between Alternatives 1 and 2. Only Alternative 1 was therefore evaluated further.

The option to pump to the BPR and gravitate to the Operational Reservoir vs. pumping directly to the Operational Reservoir and providing a Surge Reservoir in the vicinity of chainage 26 000 m (PI48) was evaluated.

The following practical aspects need to be considered in deciding on the location of the Break Pressure and Operational Reservoirs and the pumping philosophy for the Scheme:

- Supplying directly to the Operational Reservoir will require pumping over a high ridge and then ‘downhill’ to the reservoir. This will cause operational difficulties, especially during the initial stages of the scheme’s operation, when flows are expected to be low. It will also require more complex control mechanisms at the inlet of the Operational Reservoir to prevent the pipeline from draining when the pumps have switched off and will require special consideration during pump start-up to ensure that surge pressures are controlled. Pumping directly to the Operational Reservoir during the period when the water requirements are initially lower will result in an increased risk from an operational point of view. This approach will become more feasible when the water requirements increases, requiring continuous pumping at increased pumping rates.
- It is preferred to locate the Operational Reservoir as close as possible to the end users from where gravity flow will still be possible. The variable end-user water requirements will then be balanced by the Operational Reservoir, ensuring more stable operating conditions further upstream in the pipeline between the Break Pressure and Operational Reservoirs. It will also reduce the risk of draining the pipeline during no flow conditions as the gravity pipeline will be controlled from the downstream side.
- Providing an Operational Reservoir on the farm Rooipan will reduce the static head on the infrastructure downstream of the Operational Reservoir by between 50 and 90 m depending on the position of the BPR (Central route vs. Alternative Route 1) compared to supplying the end users directly from the BPR location. This has definite cost and operational advantages for the downstream users.
- Providing a BPR en-route will ensure that the rising main delivers directly into a reservoir located at a higher elevation with no ‘downhill’ pumping. This improves and simplifies the operational control of the pump station, and reduces the risk of draining the pipe when the pumps are turned off. It also simplifies the overall control of the system.
- Sizing the pipeline between the Break Pressure and Operational Reservoirs for gravity flow now, will enable the later boosting of the gravity pipe section by bypassing the BPR and pumping directly to the Operational Reservoir during periods of high water requirement. This will improve the future upgrading capability of the scheme as a whole but will require careful consideration during the initial design of the system.

Apart from practical considerations listed above, the engineering economic evaluation of the different supply options was also evaluated and are summarised below. The evaluation comprised a hydraulic and economic analysis to confirm the most viable and practical long term solution.

5.2.1. Scenario Description

Four scenarios (as described below) along two alternative routes were investigated. Only the relevant infrastructure between Vlieëpoort Weir and the Operational Reservoir was included in the assessment. Infrastructure components which are common to all scenarios or those which would not have an influence on the analysis (i.e. downstream of the Operational Reservoir) were excluded.

5.2.1.1. Scenario 1A: Central Route – Rising Main to Break Pressure Reservoir – Gravity Supply to Operational Reservoir

Pump station and rising main via the Central Route, pumping from Vleëipoort Weir to a BPR at chainage 26 000 (PI 48). The BPR was sized for 8 hours storage (at peak recovery flow rate). From the BPR, the water will flow under gravity to the Operational Reservoir. The Operational Reservoir was also sized to provide 8 hours storage. Water is supplied from the Operational Reservoir under gravity to the end user Terminal Reservoirs. Relevant system criteria are as follows:

Vlieëpoort Weir Balancing Dam MOL	923.0 m
Vlieëpoort Weir High-lift Pump Station Pump Centre Line (CL) Level	913.5 m
Break Pressure Reservoir Top Water Level (TWL)	1 115.0 m
Break Pressure Reservoir MOL	1 110.0 m
Operational Reservoir TWL	1 023.1 m
Operational Reservoir MOL	1 018.1 m
Rising main: Vlieëpoort – BPR	26.7 km
Gravity main: Break Pressure Reservoir – Operational Reservoir	62.7 km

5.2.1.2. Scenario 1B: Central Route – Rising Main to Operational Reservoir

Pump station and rising main via the Central Route, pumping from Vlieëpoort Weir directly to the Operational Reservoir. A 20 Mℓ Surge Reservoir was allowed for at chainage 26 000 (PI 48). The Operational Reservoir was sized for 8 hours storage (at peak recovery flow rate). Water is supplied from the Operational Reservoir under gravity to the end user's Terminal Reservoirs. Relevant system criteria are as follows:

Vlieëpoort Weir Balancing Dam MOL	923.0 m
Vlieëpoort Weir High-lift Pump Station Pump CL level	913.5 m
Operational Reservoir TWL	1 023.1 m
Operational Reservoir MOL	1 018.1 m
Rising main: Vlieëpoort – Operational Reservoir	89.5 km

5.2.1.3. *Scenario 2A: Alternative Route 1 – Rising Main to Break Pressure Reservoir – Gravity Supply to Operational Reservoir*

This scenario is similar to Scenario 1A, however, the pipeline follows Alternative Route 1 further towards the west from Vlieëpoort Weir to a BPR at chainage 35 800 (PI 38). The BPR was sized for 8 hours storage (at peak recovery flow rate). From the BPR, the water will flow under gravity to the Operational Reservoir. The Operational Reservoir was also sized to provide 8 hours storage (at peak recovery flow rate). Water is supplied from the Operational Reservoir under gravity to the end user Terminal Reservoirs. Relevant system criteria are as follows:

Vlieëpoort Weir Balancing Dam MOL	923.0 m
Vlieëpoort Weir High Lift Pump Station Pump CL level	913.5 m
Break Pressure Reservoir TWL	1 076.0 m
Break Pressure Reservoir MOL	1 071.0 m
Operational Reservoir TWL	1 023.1 m
Operational Reservoir MOL	1 018.1 m
Rising main: Vlieëpoort – BPR	35.8 km
Gravity main: Break Pressure Reservoir – Operational Reservoir	53.3 km

5.2.1.4. *Scenario 2B: Alternative Route 1 – Rising Main to Operational Reservoir*

This scenario is similar to Scenario 1B, however, the pipeline follows Alternative Route 1 further towards the west from Vleëipoort Weir to the Operational Reservoir. A 20 Ml Surge Reservoir was allowed for at chainage 35 800 (PI 38) along Alternative Route 1. Water is supplied from the Operational reservoir under gravity to the end users Terminal Reservoirs. Relevant system criteria are as follows:

Vlieëpoort Weir Balancing Dam MOL	923.0 m
Vlieëpoort Weir High Lift Pump Station Pump CL level	913.5 m
Operational Reservoir TWL	1 023.1 m
Operational Reservoir MOL	1 018.1 m
Rising main: Vlieëpoort – Operational Reservoir	89.2 km

5.2.2. Hydraulic Assessment

A hydraulic analysis of the scenarios was done to determine the performance of the system at different flow conditions and with absolute roughness (k_s) values varying from 0.05 mm (for new pipes) to 0.5 mm (for aged pipes) in order to test the effect on the pumping head, hydraulic gradients and pipe rating.

Due to the lower elevation of the BPR along Alternative Route 1 (Scenario 2A), it is not viable to achieve gravity flow between the BPR and the Operational Reservoir as it requires a minimum pipe diameter of 2 500 mm. The results of Scenario 2A is therefore not shown in the tables below, or on the long sections included in **Appendix D**.

Scenario 1A: Central Route – Rising main to break pressure reservoir - Gravity supply to Operational Reservoir

Pipelines

	Diameter	Length
Rising Main	1 900 mm Ø	26.7 Km
Gravity Pipeline	2 200 mm Ø	62.7 Km

	K_s = 0.05mm			
Flow Condition [1]	**Q (m³/s)**	H_{ps}	H_{bpr}	H_{or}
Low	0.659	212.1	15.0[2]	80.9
Design	5.779	240.2	15.0[2]	50.5
Recovery	6.593	248.8	15.0[2]	41.2

	K_s = 0.5mm			
Flow Condition [1]	**Q (m³/s)**	H_{ps}	H_{bpr}	H_{or}
Low	0.659	212.3	15.0[2]	80.7
Design	5.779	255.6	15.0[2]	34.1
Recovery	6.593	268.8	15.0[2]	19.8

1. Low flow condition based on 10% of recovery flow. Design and Recovery flows as per **Table 3-5**.
2. Pumping head at Vlieëpoort governed by the min residual head of 15 m required at the BPR inlet to account for control valve inlet losses, etc.

Scenario 1B: Central Route – Rising main to Operational Reservoir

Pipelines

	Diameter	Length
Rising Main	2 100 mmØ	89.5 Km

	K_s = 0.05mm			
Flow Condition[1]	**Q (m³/s)**	H_{ps}	H_{sr}	H_{or}
Low	0.659	211.9	15.0[2]	90.7
Design	5.779	228.6	15.0[2]	51.4
Recovery	6.593	233.7	15.0[2]	39.4

	K_s = 0.5mm			
Flow Condition[1]	**Q (m³/s)**	H_{ps}	H_{sr}	H_{or}
Low	0.659	212.0	15.0[2]	90.4
Design	5.779	237.7	15.0[2]	30.3
Recovery	6.593	248.7	18.2	15.0[3]

1. Low flow condition based on 10% of recovery flow. Design and Recovery flows as per Table 3-5.
2. Pumping head at Vlieëpoort governed by the min residual head of 15 m required at the break pressure reservoir inlet to account for control valve inlet losses, etc.
3. Pumping head at Vlieëpoort governed by minimum residual head required at the Operational Reservoir. Higher head loss due to increased roughness moves the control to the Operational Reservoir.

Scenario 2B: Alternative Route 1 – Rising main to Operational Reservoir

Pipelines

	Diameter	Length
Rising Main	2 100 mmØ	89.2 Km

	K_s = 0.05mm			
Flow Condition [1]	**Q (m³/s)**	H_{ps}	H_{sr}	H_{or}
Low	0.659	175.2	15.0[2]	54.2
Design	5.779	197.6	15.0[2]	20.9
Recovery	6.593	208.7	19.3	15.0[3]

	K_s = 0.5mm			
Flow Condition [1]	**Q (m³/s)**	H_{ps}	H_{sr}	H_{or}
Low	0.659	175.4	15.0[2]	54.0
Design	5.779	221.8	27.1	15.0[3]
Recovery	6.593	247.9	42.7	15.0[3]

1. Low flow condition based on 10% of recovery flow. Design and Recovery flows as per **Table 3-5**.
2. Pumping head at Vlieëpoort governed by minimum residual head required at the surge reservoir location.
3. Pumping head at Vlieëpoort governed by minimum residual head required at the operational reservoir.

Legend

H_{ps}	Head at Pump Station
H_{sr}	Head at Surge Reservoir
H_{bpr}	Head at Break Pressure Reservoir
H_{or}	Head at Operational Reservoir

Discussion of the results:

- The recovery peak flow rate (6.6 m^3/s) can be transferred under gravity between the BPR and the Operational Reservoir along the Central Route (Scenario 1A) through a 2 200 mm dia pipeline at the long term absolute roughness of 0.5 mm and a flow velocity of 1.73 m/s.
- The differences in ground levels between the Central Route and Alternative 1 (Scenario 1B vs. 2B) do not significantly influence the pumping head at Vlieëpoort or the flow flexibility that can be achieved through the system. Under low flow conditions, the ground levels in the vicinity of the Surge Reservoir dictate the minimum required pumping head at Vlieëpoort, resulting in approximately 30 m higher pumping heads for Scenario 1B compared to 2B. Under recovery flow conditions and for aged pipelines, the high-ground in the vicinity of the Surge Reservoir do not govern pumping heads anymore. Careful flow and pressure control will be required at the inlet to the Operational Reservoir for lower flow conditions (initial stages of the project) to prevent the hydraulic gradeline from intersecting the ground profile in the vicinity of the Surge Reservoir.
- The increase in the absolute roughness coefficient from 0.05 to 0.5 will result in increased friction losses and pumping heads over time as shown by the results and the longitudinal section profiles.

- Due to the relatively small difference between the hydraulic performance and flow flexibility achievable between Scenarios 1B and 2B, and because for Scenario 2A gravity flow is not a viable option between the BPR and the Operational Reservoir via Alternative Route 1, Scenarios 2A and 2B were not considered further in the engineering economic evaluation.

5.2.3. Engineering Economic Evaluation

The comparative URVs for Scenarios 1A and 1B were calculated to determine the least cost option. The comparative URV calculations excluded all infrastructure components that are common to both scenarios. The base date for costs is April 2008.

5.2.3.1. Scenario 1A: Central Route – Rising Main to Break Pressure Reservoir – Gravity Supply to Operational Reservoir

Table 5-1: Scenario 1A: Central Route – Rising Main to BPR – Gravity Supply to Operational Reservoir

Component		Total (R)
Pipeline		
Vlieëpoort - Break Pressure Reservoir		1 286 513 590
Break Pressure Reservoir - Operational Res		2 946 779 639
Reservoirs		
Break Pressure Reservoir		59 482 000
Pump Station		
Vlieëpoort	(Pumping 255.5 m)	510 843 641
Scenario 1A: Capital Cost		**4 765 167 899**

Operations and Maintenance Cost

Component		Total (R)
Pipeline		
Vlieëpoort - Break Pressure Reservoir		3 069 369
Break Pressure Reservoir - Operational Res		8 118 438
Reservoirs		
Break Pressure Reservoir		51 363
Pump Stations		
Vlieëpoort	Civil	87 000
	Mechanical and Electrical	4 281 135
	Electricity	73 096 081
Scenario 1A: Annual O&M		**88 703 386**

Present Value of Cost

Discount Rate	Capital (R)	O & M (R)	Total (R)
6%	3 869 249 301	883 380 546	4 752 629 848
8%	3 629 100 925	608 440 729	4 237 541 655
10%	3 409 571 818	437 242 288	3 846 814 106

Unit Reference Value

Discount Rate	Discounted Present Value (R)	URV
6%	4 752 629 848	2.36
8%	4 237 541 655	3.02
10%	3 846 814 106	3.76

5.2.3.2. Scenario 1B: Central Route – Rising Main to Operational Reservoir

Table 5-2: Scenario 1B: Central Route – Rising Main to Operational Reservoir

Capital Cost

Component	Total (R)
Pipeline	
Vlieëpoort – Operational Reservoir	5 520 583 367
Reservoirs	
Surge Reservoir	59 482 000
Pump Station	
Vlieëpoort (Pumping 232 m)	471 878 992
Scenario 1B: Capital Cost	**6 013 493 388**

Operations and Maintenance Cost

Component		Total (R)
Pipeline		
Vlieëpoort – Operational Reservoir		13 257 345
Reservoirs		
Surge Reservoir		51 363
Pump Stations		
Vlieëpoort	Civil	87 000
	Mechanical and Electrical	3 567 398
	Electricity	66 418 467
Scenario 1B: Annual O&M		**83 381 573**

Present Value of Cost

Discount Rate	Capital (R)	O & M (R)	Total (R)
6%	4 890 743 255	834 915 961	5 725 659 216
8%	4 587 555 857	575 814 426	5 163 370 283
10%	4 310 265 506	414 341 707	4 724 607 214

Unit Reference Values

Discount Rate	Discounted Present Value (R)	URV
6%	5 725 659 216	2.84
8%	5 163 370 283	3.67
10%	4 724 607 214	4.61

5.2.3.3. Summary of Comparative URVs

Table 5-3 below summarise the calculated comparative URVs for each of the Scenarios evaluated.

Table 5-3: Phase 2A Options Analysis Comparative URVs

Scenario	Comparative URV (excluding VAT)		
	@ **6%**	@ **8%**	@**10%**
1A	2.36	3.02	3.76
1B	2.84	3.67	4.61

5.2.4. Steel Pipe Price Sensitivity

The impact of steel pipe price fluctuations were analysed to test the sensitivity of this parameter.

The fluctuation in the price of steel over the past 34 months is illustrated by **Figure 5-4**. Pipe prices for costing purposes were obtained from suppliers in April 2008. Due to rampant escalation at the time, some allowance for escalation was included in these prices, resulting in the effective price of steel pipes in the costing models being equivalent to the actual pipe prices in July 2008. The current steel price is approximately 38% less than the price upon which the estimates for this study were based. The steel price reached a minimum in May 2009 and has since shown a steady increase again. It should be noted that the price of steel makes up approximately 55% of the final pipe price.

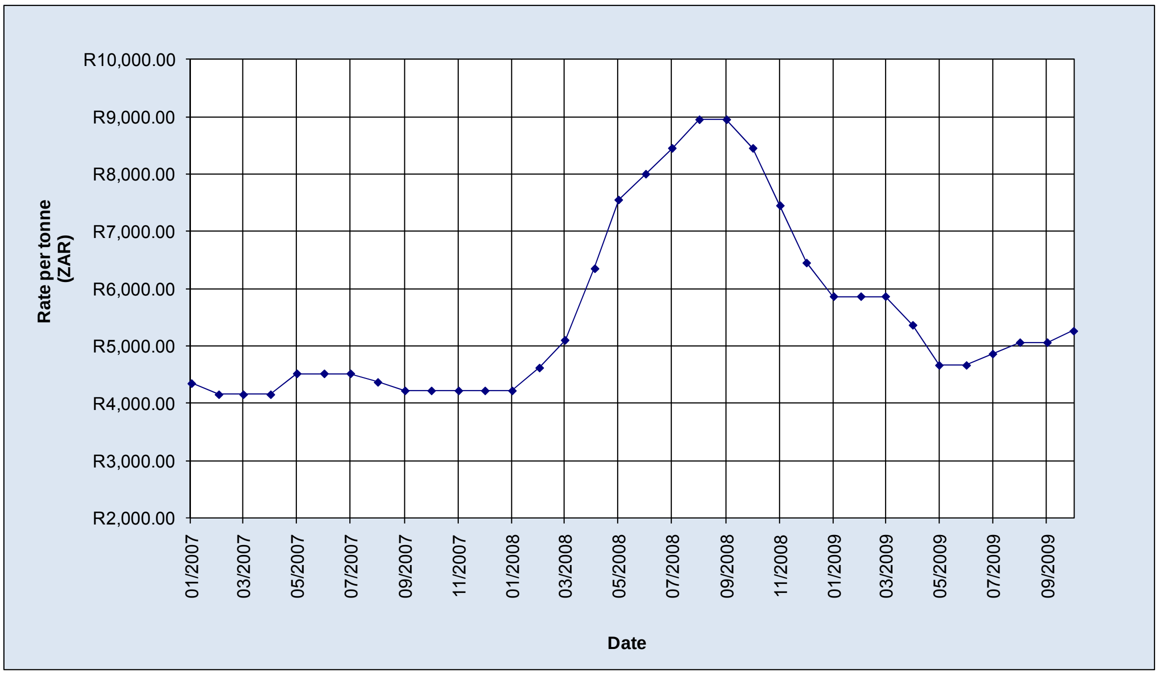


Figure 5-2: Steel Price Fluctuation

Table 5-4: URV Sensitivity to Steel Price Fluctuations

Scenario	URV (excluding VAT)		
	@ 6%	@ 8%	@10%
1A +15% Increase	2.57	3.30	4.19
1A – April 2008	***2.36***	***3.02***	***3.76***
1A – 15 % Decrease	2.14	2.73	3.39
1A – 30 % Decrease	1.93	2.45	3.03
1A – 45 % Decrease	1.72	2.17	2.67
1B +15% Increase	3.12	4.05	5.10
1B – April 2008	**2.84**	**3.67**	**4.61**
1B – 15 % Decrease	2.56	3.10	4.13
1B – 30 % Decrease	2.28	2.92	3.65
1B – 45 % Decrease	1.10	2.55	3.17

From **Table 5-4**, it is therefore clear that the relative values of the URV have not changed so much that the choice between Scenarios 1A and 1B will be affected. Based on practical considerations Scenario 1A is preferred.

5.2.5. Electricity Cost Sensitivity

The URV calculations allow for an annual increase of 20% (compounded) for the next five years and inflation bound increases thereafter. It was found that increases of more than 20% per annum will not affect the choice between Scenarios 1A and 1B.

5.2.6. System Upgrade Capability

Due to the uncertainty with regards to the ultimate water requirement of the end users, the upgrading potential of the scheme is an important consideration. As shown, implementing Scenario 1A (Rising main to BPR with gravity flow to the Operational Reservoir) will enable the transfer of the 2030 recovery peak flows based on the Scenario 9 water requirement projections. Further increases in the projected water requirements are, however, expected after 2030.

Implementing Scenario 1A will enable the later upgrading of the system to achieve a significant increase in the transfer capacity between Vlieëpoort and the Operational Reservoir. This can be achieved by installing another 1 100 mm dia pipeline between Vlieëpoort and the BPR (Chainage 26 700), bypassing the BPR and converting the gravity section between the BPR and the Operational Reservoir into a rising main. The BPR can then be reconfigured to act as a Surge Reservoir to protect the system. The transfer capacity that can be achieved under this configuration at various pumping heads at Vlieëpoort is summarised below (**Table 5-5**). The system limits were set at either a pumping head of 350 m or a flow velocity of 2.5 m/s to determine maximum possible flow rate.

Table 5-5: Scenario 1A System Upgrade Capacity

Flow Rate (m^3/s)	Flow Velocity - 2 200 mm dia (m/s)	Pumping Head at Vlieëpoort (m)	% Increase Compared to Initial Recovery Peak Flow Capacity
8	2.10	264	21%
9	2.36	299	36%
9.5	2.50	317	44%

5.2.7. Phase 2A Options Analysis Conclusion and Recommendation

Based on the evaluation of options, the following is concluded:

- The difference in comparative URVs between Scenarios 1A and 1B consistently favour Scenario 1A, but only amount to about 5% of the URVs given in Section 10.5 for the total MCWAP (Phase 1 and Phase 2A).
- Steel pipe prices have reduced considerably since July 2008. As shown, a reduction in the steel pipe price will reduce the difference in URVs between the two scenarios, but not enough to change the ranking.
- Due to the higher pumping head associated with Scenario 1A, recent increases in the energy cost compared to the Pre-feasibility assumptions will also reduce the difference in URVs, but not sufficiently to change the ranking.
- Implementing Scenario 1A will enable the peak capacity to be increased by 44% without the need to construct a parallel pipeline between the BPR and the Operational Reservoir. It will, however, require careful consideration during the design stage to

ensure that the gravity section between the BPR and the Operational Reservoir has the required pressure rating to allow it to be boosted in future.

- Scenario 1A will allow for easier operational control of the system during the initial stages of operation (low flow conditions) with a rising main to the BPR and gravity flow to the Operational Reservoir.

Based on the above, Scenario 1A (Rising main to the BPR at Ch 26700 and gravity flow to the Operational Reservoir and further to the connection with the Lephalale-Steenbokpan pipeline) are recommended for implementation. The options analysis was presented to the Technical Task Team who accepted the recommendation.

It should be noted that there have been a number a changes to the parameters that influence the engineering economic evaluation of options since April 2008. The pipe systems can also be optimised further when final design capacities, and more detailed survey and geotechnical information becomes available. It is therefore recommended that a more comprehensive evaluation and optimisation be performed during the detail design stage to re-confirm these findings.

5.3. Pipelines

5.3.1. Pipeline Design Considerations

5.3.1.1. Lining and Coating Selection

Although the selection of a pipeline coating does not affect the optimisation of a pipeline in terms of internal diameter and steel grade, the internal lining does affect the structural optimisation of the pipeline. For example, rigid linings such as cement mortar follow limiting strain design principles while flexible linings such as epoxy follow limiting stress principles.

Also, rigid linings are more vulnerable to pipe deformation and are thus typically limited to lower acceptable limits compared to flexible linings. As a result, better quality backfill material, more installation control and even thicker pipe walls may be required in some cases to prevent large pipe deformation from occurring that may damage the lining.

Preliminary field investigations indicate that soil conditions for the MCWAP pipelines comprise high resistivity sand, gravel and rock with pockets of black clay in low lying areas. Conditions are generally dry and some areas of significant mineralisation are present.

The pipeline routes could experience severe AC (alternating electrical current) interference from the present and future high voltage power lines which run parallel to and cross the proposed pipeline at several locations. The Phase 2A route also runs parallel to the existing railway line, which is understood will be electrified with 25 kV AC traction in the foreseeable future, for a considerable length. With reference to Section 5.3.5, the possible presence and impact of Sulphate Reducing Bacteria must also be considered.

Power supplies to CP Stations from 11 kV distribution lines are limited and the locations of these stations will have to be optimised, implying a high reliance on the integrity of the pipeline coating.

Coating System Comparison:
The predominant characteristics required for the pipeline coating for this project will be:

- Resistance to soil shrinkage stresses in clayey areas;
- Low current density for CP; and
- High electrical stress resistance for AC interference.

Available pipe coatings from the two major pipe manufacturers in South Africa are summarised in **Table 5-6**.

Table 5-6: Coating System Summary

Coating System	Characteristics/Practical Considerations
Trilaminate Polyethylene (3)(4)	• Low current demand and high mechanical damage resistance. • High technical specification required. • Field joint coatings require specialist skill and strict quality control (5,6,7) • Currently not available for pipe sizes in excess of 1 100 mm.
Liquid Coatings (1)	• Polyurethane or flake glass reinforced epoxy resin systems. • Low current demand and high mechanical damage resistance. • No restriction on diameter. • Field Joint Coating is done with the same material as the main pipeline.
Fusion Bonded Polyethylene ("Sintakote") (2)	• Sintakote (II) - not recommended due to poor adhesion and cathodic disbonding properties and difficulty of joint making-good and field repairs.
Bitumen/Glass Fibre	• Not recommended due to high current demand, high water absorption and inconsistent raw materials. • Long-term local raw material supply is uncertain
Polymer modified bitumen/Glass Fibre ("Bituguard")	• Bituguard overcomes issues associated with bitumen/fibreglass. • Field Joint Coating is done with the same material as the main pipeline. • Likely to be available in South Africa during 2010.

Notes

1. Liquid coatings for field joints require a high level of skill in the field, but if used with a liquid coating on the pipeline will provide a seamless system. A dedicated application crew with specialized equipment will be required.
2. The fusion bonded polyethylene systems suffer from relatively poor adhesion to the substrate and also poor cathodic disbonding resistance. In the event of coating damage, there is no secondary defence against corrosion as the polyethylene shields the underlying substrate from CP.
3. Polyethylene systems are difficult to repair in the field due to poor adhesion properties of other materials to the polyethylene.
4. The problems associated with three-layer coatings which have come to light in recent years are primarily related to the Fusion Bonded Epoxy (FBE) primer. Many of the causes of these problems have been identified and can be addressed in terms of the specification requirements. These enhancements may have some impact on procurement as not all pipe manufacturers may have the capability of implementing the enhanced specification. Pre-qualification will have to include long-term accelerated testing of applied coatings, such as the 28-day hot-water soak test.
5. Although stand-alone heat shrink sleeves have been used extensively in the past, application sensitivity and the lack of a suitable primer remain pertinent issues. The sleeves are difficult to apply to larger diameter pipelines and the final quality is entirely dependent on operator expertise. Site weather conditions at the time of application can significantly influence the final product. The long-term issues of adhesion, cracking and relaxation only become apparent after burial, and cannot be readily identified at the time of application. Recent poor experience in South Africa with the use of these sleeves mitigates against their use.
6. Tape wrapping systems selected for the joints may incorporate polyethylene (PE) or polyvinyl chloride (PVC) backing. It is essential that the joint making- good material matches the pipeline in terms of electrical characteristics, i.e. the dielectric strength and resistance to permeability by water vapour should be the same.
7. Liquid or fusion bonded epoxy systems for making-good of field joints in 3LPE coatings are preferred, together with cold applied tapes.

All high insulation-value coatings will require effective temporary CP and AC mitigation during construction, in addition to effective permanent CP and AC mitigation systems. There have been many cases in South Africa where significant metal loss has occurred on pipelines before commissioning, due to poor or ineffective temporary CP during construction.

Lining System Comparison:

The two lining systems generally available and used for water pipelines in South Africa are Cement Mortar Lining (CML) and solvent free epoxy.

CML may be applied either in the factory or as an in-situ lining once the pipe is laid.

Factory applied CML has the advantage of smoother linings, and possibly better control of lining thickness and quality. Repair of joints is however critical in factory CML pipelines, as uncoated areas represent high corrosion rate locations due to galvanic cell formation.

In general cement mortar linings are less forgiving when subject to conditions of poor curing, poor workmanship, pressure surges and also vibration due to blasting in close proximity to the pipeline. A further aspect requiring careful consideration is the possibility of aggressive water causing leaching of the cement. In general, inland water sources in South Africa are not soft although they may be aggressive to concrete for other reasons. The use of water based epoxy sealers on CML has shown promising results in preventing shrinkage cracking and reducing the leaching of lime from the cement mortar in aggressive waters. Careful analysis of water quality will be required to ensure that CML is suitable.

Epoxy linings are factory applied using solvent free materials applied by hot plural component airless spray. Field making-good of these coatings requires blast cleaning of the exposed steel inside the pipe. This may be carried out with portable suction guns and fine grit. The performance of the epoxy lining field joints is critically dependant on application, and is therefore more difficult to achieve in smaller diameter pipes.

The minimum recommended pipe diameter for successful repair of internal field joints requiring worker-entry is 800 mm.

Based on current availability, both CML and solvent free epoxy linings could be considered for this project. The basic requirements for each lining system are listed below.

<u>Epoxy Linings</u>

The following basic requirements for epoxy linings should be specified:

- Minimum Dry Film Thickness (DFT) of 400 micron shall be applied (unless a thicker application is required by the epoxy supplier).
- All epoxy linings shall be solvent free.
- All epoxy linings shall have potable water certification.
- All epoxy linings shall be compatible with site repair (weld repair) technologies.
- All linings (and coatings) applied to pipes shall withstand a maximum pipe deformation of 5% of the diameter with no short- or long-term consequences to the integrity of the lining (or coating).
- Useful life span requirement of the epoxy linings shall be a minimum of 45 years.

<u>Cement Mortar Linings</u>

The following basic requirements for CML linings should be specified:

- Cement mortar linings to be either shop-applied or in-situ applied in accordance with the latest editions of AWWA C205 and C602 respectively, with relevant modifications where necessary.
- DWEA Specification DWS1131 shall also be applicable.
- The suggested lining thicknesses and tolerances are:
    - Pipe O.D. 325 mm to 610 mm: 11 mm +4 mm -0 mm;
    - Pipe O.D. 611 mm to 1 220 mm: 12 mm +5 mm -0 mm; and
    - Pipe O.D. > 1 220 mm: 13 mm +6 mm -0 mm.

- Cement shall be OPC CEM I (Ordinary Portland Cement or Rapid Hardening Portland Cement or a combination of these cements) which shall comply with the latest requirements of SABS ENV 197-1 : 1992.
- Cement-mortar linings (CML) shall withstand a maximum pipe deformation of 2% of the diameter with no short- or long-term consequences to the integrity of the lining.

Solvent free epoxy linings are considered as the primary lining for this project and are preferred to CML on pipelines that are big enough to allow worker- entry for internal field joint repair.

Coating and Lining Selection Recommendation:

Based on the above information, the following generic coating and lining systems are recommended for the MCWAP Pipeline:

Table 5-7: Recommended Coating and Lining Systems

Product/Method		Field Joint Repair Method
External Coating	Preferred: Trilaminate Polyethylene (3LPE) or Polyurethane Alternative: Polymer modified bitumen/Glass Fibre (Bituguard)	 Liquid or powder epoxy plus cold tape wrap Polyurethane Bituguard hot applied tape
Internal Lining	Preferred: Epoxy Alternative: Cement Mortar	 Epoxy Cement Mortar

The following should be noted:

- Field joint coatings should be undertaken by a pre-qualified specialist sub-contractor.
- Irrespective of which coating system is finally chosen, it is vital to ensure that adequate independent third party inspection is undertaken during all the stages of the coating application. This will require inspection at the coating mill (all stages of application); during handling, storage and transportation; and during installation, joint coating and backfill.
- Inspection of field joint coatings should be on a 100% witness and approval hold point basis.
- A coating survey using the direct current voltage gradient (DCVG) or similar technique should be undertaken after the pipeline has been backfilled.

- The specifications prepared for the pipeline construction must include relevant clauses for the DCVG survey, such as when it will be done, how it will be done, the ranking system that will be used to determine post-installation excavation/repair requirements, specifying what cut-off value will be used for repair of defects. The specification should make the contractor responsible for all post-installation excavation and repairs according to these criteria, at the contractor's cost.

5.3.1.2. Long-Term Roughness Parameters

For the selection of pumps and the economic analysis, the energy loss in a bulk supply pipeline plays a significant role and hence has to be determined as accurately as possible. A number of different formulae can be used to calculate the energy losses through a pipeline system. The Darcy Weisbach–Colebrook White relationship is generally accepted and widely applied under all flow conditions and was used to calculate the friction losses in the pipeline system.

The following must be considered when doing a hydraulic assessment of a pipe system:

- The appropriate friction or roughness factors for the pipeline design;
- The ageing of the pipeline and increase of the roughness over time; and
- The influence of biofilm on the hydraulic capacity of the pipeline.

The above aspects were reviewed by Professor Fanie van Vuuren of the University of Pretoria with reference to recent research on the subject and confirmed by field investigations to calibrate the findings. The recommended long-term roughness parameters, as well as the influence of biofilm is summarised in **Table 5-8**.

Table 5-8: Suggested Long-Term Roughness Parameters and the Influence of Biofilm

Parameter	Cement Mortar Lining		Epoxy Lining	
	Suggested	Maximum	Suggested	Maximum
Long-term absolute roughness (mm)	1.1	1.5	0.5	0.7
Influence of biofilm	Reduction in diameter of 5 - 8 mm			

For hydraulic optimisation and modelling purposes, a long-term roughness value of 0.5 mm was used. The effect of a reduction in the internal pipe diameter of 5 mm due to biofilm formation was found to have a minor impact on the hydraulic performance of the pipe system over the range of diameters being considered. The application of these criteria must, however, be considered at the time of performing the final detail design.

5.3.1.3. Structural Design and Optimisation

The design of flexible buried pipes involves consideration of the interaction between the steel shell and the surrounding backfill and the deflection limits appropriate to the lining and coating system. Flexible pipes obtain a large portion of their load carrying capacity from the surrounding backfill, and therefore, the incorporation of this interaction is important. The design method should take into account the strength of the pipe-soil system as a whole, without relying solely on the strength of the individual components. A detailed structural analysis and optimisation was not performed during the feasibility investigation. The principles of the process that must be followed during the detail design phase are, however, stated below.

Structural Design of Steel Pipe:

Steel pipelines are generally designed in accordance with flexible pipe principles. Flexible pipes are defined as those pipes whose properties are such that the first limit state reached is either excessive deformation or buckling collapse. However, what is not always appreciated is that although steel pipes are considered flexible in bending, they are rigid in hoop (ring) compression. This is the reason why the external pressure on metal conduits in general is more than the soil prism loads (i.e. negative arching) because not all of the friction strength in the soil is mobilised.

Minimum Pipe Wall Thickness:

The wall thickness is a function not only of the internal pressure (i.e. operating pressure and the surge over-pressure), but also of the external pressure exerted on the pipe (i.e. soil overburden, external fluid pressure, vehicular and vacuum loading, etc.). On large diameter steel pipelines, practical requirements such as pipe handling and installation requirements may in fact control the minimum pipe wall thickness. Therefore, the selection of the minimum pipe wall thickness is a function of:

- Internal Design Pressure (i.e. maximum design pressure and vacuum);
- External Design Pressure (i.e. soil overburden plus vacuum and vehicular loading);
- Pipe Handling Requirements (i.e. pipe manufacture, transportation, laying and backfilling requirements); and
- Pipe Buckling Capacity (Considering good quality soils, poor quality soils as well hydrostatic or unsupported soil conditions).

When the wall thickness is not controlled by internal pressure requirements, the minimum thickness is typically determined from the pipe handling and installation requirements. For large diameter thin-walled steel pipelines this is generally determined as a function of the diameter to wall thickness ratio (D/t). This ratio is a good indication of the flexibility of the pipe and increases with increasing pipe size and/or decreasing wall thickness. For large D/t ratios it becomes very difficult to transport, handle, install and backfill these pipes without causing shape distortions (which may damage internal/external linings and coatings) due to increased flexibility. As a result, limits are placed on the maximum allowable D/t or procedures are specified to limit the pipe deformation.

a) Design for Internal Pressure

Minimum pipe wall thicknesses based on internal pressure requirements have been determined for X42 grade steel utilizing a design stress equal to 50 % of the minimum yield strength to allow for surge pressures since a surge analyses was not done as part of the feasibility study. Higher grade steel was not considered due to the relatively low design pressures. The well-known Barlow equation was used to calculate the minimum pipe wall thickness as a function of the maximum allowable design stress, pipe outside diameter and maximum design pressure.

b) Pipe Buckling Capacity for No Soil Support

Buckling of unsupported pipe and even buried pipe is very important when optimizing large diameter pipe with regards to minimum wall thickness. Pipe buried in soil may not necessarily have proper soil support when for example the pipe is buried with shallow soil cover and when trenches are excavated adjacent to it. Also, where pipelines traversing through very compressible material, such as in “vlei” areas, the soil support may be compromised. In cases where soil support cannot be guaranteed the following formulations may be used to calculate the critical external unsupported hydrostatic pipe buckling pressures:

i. Von Mises / Timoshenko Hydrostatic Buckling Approach;
ii. Stewart Hydrostatic Buckling Approach;
iii. BS EN1295-1: 1998 Hydrostatic Buckling Approach; and
iv. WRC 1988 Hydrostatic Buckling Approach.

c) Pipe Handling Requirements

Minimum pipe stiffness is required for practical handling and installation without undue care or bracing. Pipe stiffness (PSB) is computed from EI/D^3 where D is the pipe diameter, I is the moment of inertia and E represents the modulus of elasticity of the pipe material. Various approaches are available to calculate the minimum pipe stiffness. For practical purposes, a lower limit of 2.0 kN/m/m may be used which translates into a maximum D/t of 205. Pipe stiffness is further affected by the quality of the backfill material and should be carefully evaluated. For example, with decreasing backfill material quality, increased compaction energy is required to achieve the design soil stiffness and strength gauged by soil compaction density. The problem is compounded by the fact that fine-grained soils produce significantly more pipe distortion during compaction than course grained soils due to the lower friction angles and less interlocking of soil particles. Another factor to consider when determining the minimum pipe stiffness is the allowable longitudinal and transverse pipe self-weight deflection (which may affect welding of field joints).

d) External Design Pressure

The external loading on the pipe should consider both the soil overburden load and vehicular loading where traffic crosses over the pipeline. The minimum allowable soil cover affects the soil overburden and vehicular loading differently. The shallower the soil cover, the larger the live vehicular load pressure. A minimum design soil cover of 1.0 m was used. Road crossings may be evaluated later using non-linear finite element analyses. Live load (vehicle loading) pressures are expected to be smaller than the internal vacuum pressure of 85 kPa at the minimum suggested soil cover of 1.0 m and therefore, are not critical. Also, the probability that the pipe will be experiencing full vacuum pressure during an actual pipe crossing by live load is very small. The only possible concern with live loading may be when farmers cross over the pipeline with very heavy farming equipment under reduced soil cover situations resulting from soil settlement, planting operations and erosion.

e) Pipe Buckling Capacity with Soil Support

The buckling capacity of a pipe buried in good quality backfill material is significantly larger than a pipe without soil support. The following buckling design approaches should be considered.

i. AWWA M11 Buried Pipe Buckling Approach (Soil Support);
ii. BS EN 1295-1: 1998 Buried Pipe Buckling Approach (Soil Support);
iii. WRc Buried Pipe Buckling Approach (Soil Support); and
iv. CIRIA Report No. 78 Buried Pipe Buckling Approach (Soil Support).

f) Design for Undermined/Unstable Areas

In general, most pipelines undergo little or no vertical movement in service and therefore the longitudinal stress in a pipeline seldom approaches the limiting design value. Pipeline serviceability is therefore seldom if ever of concern. In contrast, localized areas may exist along a pipeline where soils and/or slopes are unstable or where subsidence or differential longitudinal settlement can occur.

In these areas, longitudinal stresses may become severe enough to cause a failure, particularly buckling. The pipe cross-sectional bending moment is directly proportional to the pipe curvature and therefore, the maximum predicted subsidence or settlement is critical. The areas of potential instability should be identified during the detail design phase to ensure that it is incorporated into the design of the pipeline.

5.3.1.4. Pipe Design Parameters Adopted for the Feasibility Stage Investigation

The following parameters were adopted for the feasibility design and costing of the Phase 2A pipelines:

- Pipeline material: Grade X42 Steel with yield stress of 290 MPa. Optimisation considering anticipated surge pressures, soil conditions, operational parameters, the pipeline profile and higher grades of steel must be performed during the detail design phase.
- Maximum allowable design stress: 50% of yield stress of steel.
- Maximum allowable D/t: Based on maximum internal pipeline pressure only.
- Corrosion protection: Sintakote external coating and epoxy internal lining. The final selection of the preferred pipe coating and lining should be performed during the detail design considering the water quality, soil conditions, CP design, pipeline structural design, pipeline industry capability and constructability.
- Initial absolute roughness coefficient: 0.05 mm.
- Long-term roughness coefficient: 0.5 mm.
- Maximum allowable flow velocity: 2.5 m/s.
- Maximum allowable pumping head: 350 m.
- Minimum dynamic pressure at peak flow: 15 m at any point in the system.
- Minimum head at T-off points to end users: 25 m (15 m for hydraulic loss through valves and 10 m for change in elevation and head loss in pipeline to Terminal Reservoir), as agreed with end users.

5.3.2. Pipeline Route Description

The transfer pipeline starts at the Vlieëpoort High-lift Pump station and continues north along the Thabazimbi-Dwaalboom Road (D1649). The pipeline crosses the farm Paarl 124 KQ parallel to an existing high voltage power line before turning east towards the R510. The proposed site for the BPR is located on the farm Zondagskuil 130 KQ. From the BPR, the route continues north along the R510 for a short distance before turning east on the boundary between Tarantaalpan and Diepkuil. The route then heads north along the western boundary of the railway servitude to the site of the Operational Reservoir located on the Farm Rooipan 357 LQ, crossing the Matlabas River en route. From the Operational Reservoir, the pipeline continues north-west towards Steenbokpan, where it links to up with the pipeline from Lephalale constructed as part of Phase 1 of the MCWAP. Refer to Drawing 9528/LD/CTS/001 and 9528/LD/CTS/002 included in **Appendix A** for the locality and layout of Phase 2A of the MCWAP.

It became apparent during the Public Participation process that some farmers along the route would be interested to have off-takes from the pipeline. The exact locations and numbers of these off-takes have not been confirmed and further liaison with farmers will be required to confirm the locations and to inform the farmers on the possible impacts of poor water quality and the expected cost of the water.

5.3.3. Optimum Pipeline Diameter Selection

The optimum pipe diameter for the rising main from Vlieëpoort to the BPR was determined by performing an economic analysis over a 45-year period for a number of different pipe diameters and resultant D/t ratios. The system parameters used in the calculation were as follows:

Vlieëpoort Weir Balancing Dam FSL	932.0 m
Vlieëpoort Weir Balancing Dam MOL	923.0 m
Vlieëpoort Weir High Lift Pump Station Pump CL level	913.5 m
Break Pressure Reservoir TWL – assume top inlet	1 115.0 m
Rising main: Vlieëpoort – BPR	26.7 km
Maximum static head	192 m
Minimum static head	183 m
Minimum pipe roughness (k_{smin})	0.05 mm
Maximum pipe roughness (k_{smax})	0.5 mm

A URV was calculated for the sub-scheme (pumping station and rising main only) based on the latest Design Flow Rate in accordance with the Scenario 9 water requirements to determine the combination that yielded the lowest URV value at April 2008 prices. The results are summarised in **Table 5-9** and illustrated by **Figure 5-3**.

Table 5-9: Vlieëpoort - Break Pressure Reservoir Optimum Pipe Diameter Selection

Options	Diameter (mm)	D/t	Velocity at the Design Flow Rate (m/s)	Manometric Pumping Head at the Design Flow Rate and Long-term Roughness (m)	URV @ 8% Discount Rate (R/m^3)
1	2300	130	1.39	228	1.40
2	2200	120	1.52	232	1.39
3	2100	120	1.67	237	1.34
4	2000	110	1.85	245	1.34
5	1900	110	2.05	255	1.29
6	1800	100	2.29	270	1.31
7	1700	90	2.58	291	1.33

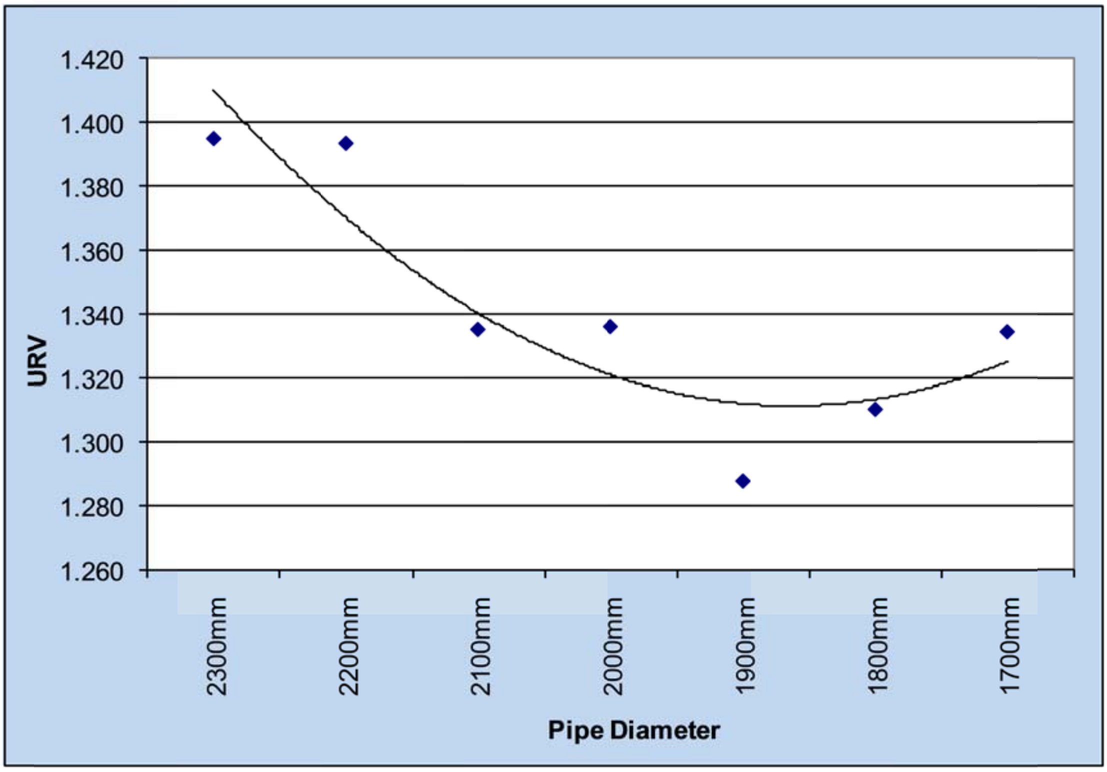


Figure 5-3: Vlieëpoort Break Pressure Reservoir Optimum Pipe Diameter Selection

The results of the analysis showed that the most economical pipe diameter for the rising main between Vlieëpoort High-lift Pump Station and the BPR is 1 900 mm. A sensitivity analysis using discount rates of 6% and 10% was carried out and showed the same order of economical preference as shown in **Table 5-9**.

Based on the selected pipe diameter and considering the maximum and minimum operating levels in the Balancing Dam, the initial and long term pipeline roughness, as well as the different design and peak flow rates, the system curve for the Vlieëpoort to BPR rising main is as illustrated by **Figure 5-4**.

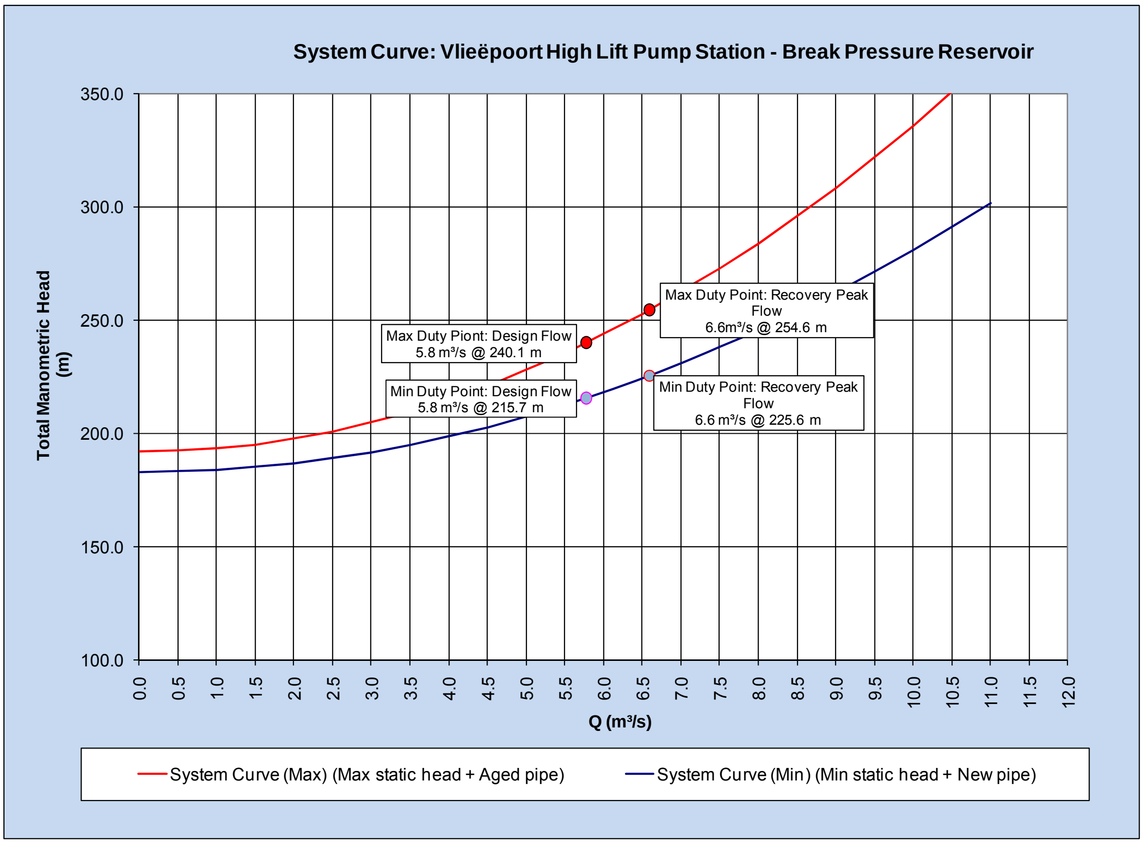


Figure 5-4: Vlieëpoort to Break Pressure Reservoir Rising Main System Curve

The respective system duty points are summarised in **Table 5-10** below.

Table 5-10: Vlieëpoort to Break Pressure Reservoir System Duty Point

Design Case	Flow (m^3/s)	Velocity (m/s)	Total H_L high (m)	Total H_L low (m)	System-H Min. (m)	System-H Max. (m)
Design Flow	5.8	2.0	48.1	32.7	215.7	240.1
Recovery Peak Flow	6.6	2.3	62.6	42.6	225.6	254.6

A pump selection has not been performed as part of the Feasibility investigation. The final selection will have to take cognisance of the minimum, design and recovery peak flow rates and consider the use of variable speed drives.

5.3.4. Crocodile River (West) Transfer Scheme Hydraulic Assessment

The CRW Transfer Scheme will ultimately supply water up to Medupi and Matimba via the Lephalale-Steenbokpan link, built as part of Phase 1 of the project. The sizing and optimisation of infrastructure provided during Phase 1 of the MCWAP have to consider the ultimate system operational requirements. Refer to supporting report No 11 (P RSA A000/00/8209) for a detailed description of the infrastructure to be provided as part of Phase 1 of the MCWAP. The schematic layout of the MCWAP is illustrated below for ease of reference. A schematic summary of the ultimate system requirements is also included in **Appendix E**.

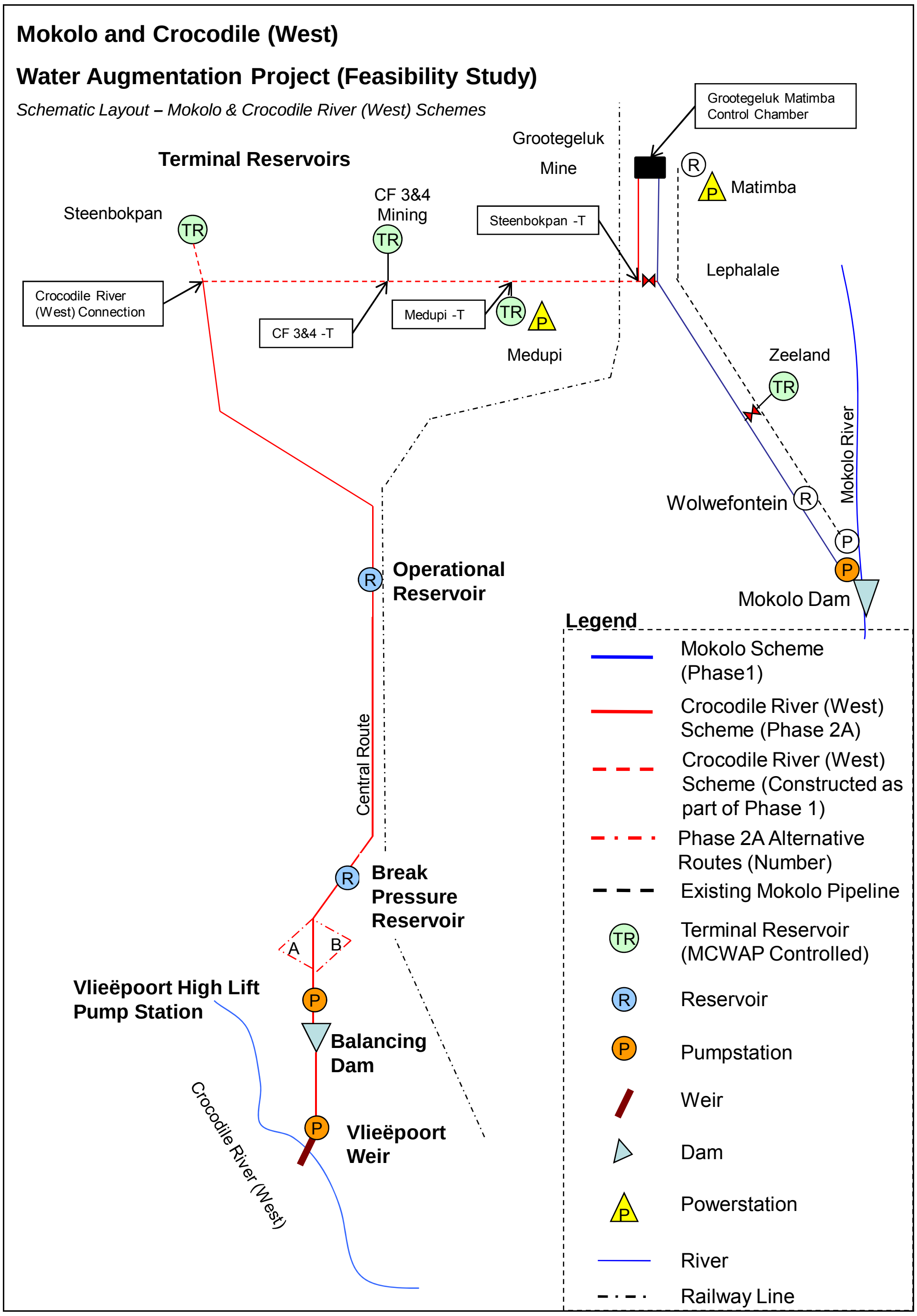


Figure 5-5: MCWAP Schematic Layout

The expected long-term hydraulic performance of the CRW Transfer Scheme is illustrated by **Figure 5-6**. The pipe sections comprising the CRW Transfer Scheme are summarised in **Table 5-11**.

Table 5-11: Crocodile River (West) Transfer Scheme Pipe Sections

Pipe Section	Diameter (mm)	Length (km)	D/t & wt	Velocity (m/s)	
				Design Flow	Recovery Peak Flow
Vlieëpoort High-lift Pump Station to Break Pressure Reservoir (Rising Main	1900	26.7	110 17.5 mm	2.05	2.34
Break Pressure Reservoir to Operational Reservoir (Gravity Main)	2200	62.7	130 17.2 mm	1.52	1.73
Operational Reservoir to Crocodile River (West) Connection to Phase 1 (Gravity Main)	2300	28.2	140 16.7 mm	1.39	1.58
Crocodile River (West) Connection to Steenbokpan (Constructed as part of Phase 1) (Gravity Main)	1900	1.4	160 12.1 mm	1.58	1.80
Crocodile River (West) Connection to CF 3&4 Mining T-off (Constructed as part of Phase 1) (Gravity Main)	1100	27.2	160 7.0 mm	1.35	1.54
CF 3&4 Mining to Medupi T-off (Constructed as part of Phase 1) (Gravity Main)	900	3.6	160 5.7 mm	1.48	1.69
Medupi T-off to Steenbokpan T-off (Constructed as part of Phase 1) (Gravity Main)	900	8.2	160 5.7 mm	1.23	1.41
Steenbokpan T-off to Grootegeluk/Matimba Control Chamber (Gravity Main)	800	1.9	160 5.1 mm	1.56	1.78

The final position and number of end user Terminal Reservoirs has not been confirmed as most of the end users are still in various stages of planning. The position of the system off-takes and Terminal Reservoirs and the ultimate water requirements of the end users will have an influence on the pipe sizes and system operation along the Lephalale-Steenbokpan link and must be confirmed as part of the detail design for Phase 1. The operation of the Lephahale-Steenbokpan link, built under Phase1 will be reversed over its full length for the ultimate system operation to transfer water from west to east to provide water to users in the Lephalale area. This will require careful consideration of the valve selection and positioning as well as the potential accommodation of surge pressures.

A detail hydraulic analysis for the positioning of the air valves, as well as isolating, reflux and control valves must therefore be performed as part of the detail design. A detailed surge analysis has not been performed as part of the feasibility investigation. The pipe wall thickness selection was, however, based on a conservative maximum allowable design stress of 50% of the steel yield strength.

The pump rate from Vlieëpoort will be determined by the level of the BPR in order to maintain a set reservoir level. The flow rate from the BPR to the Operational Reservoir will in turn be controlled from the Operational Reservoir (downstream control) to maintain a set reservoir level at the Operational Reservoir. A similar operational control methodology will be used downstream of the Operational Reservoir, supplying the end user Terminal Reservoirs. The flow rate to the respective Terminal Reservoirs will also have to be controlled to ensure an even distribution of flow to all consumers and prevent the excessive draw down of the hydraulic grade line downstream of the Operational Reservoir.

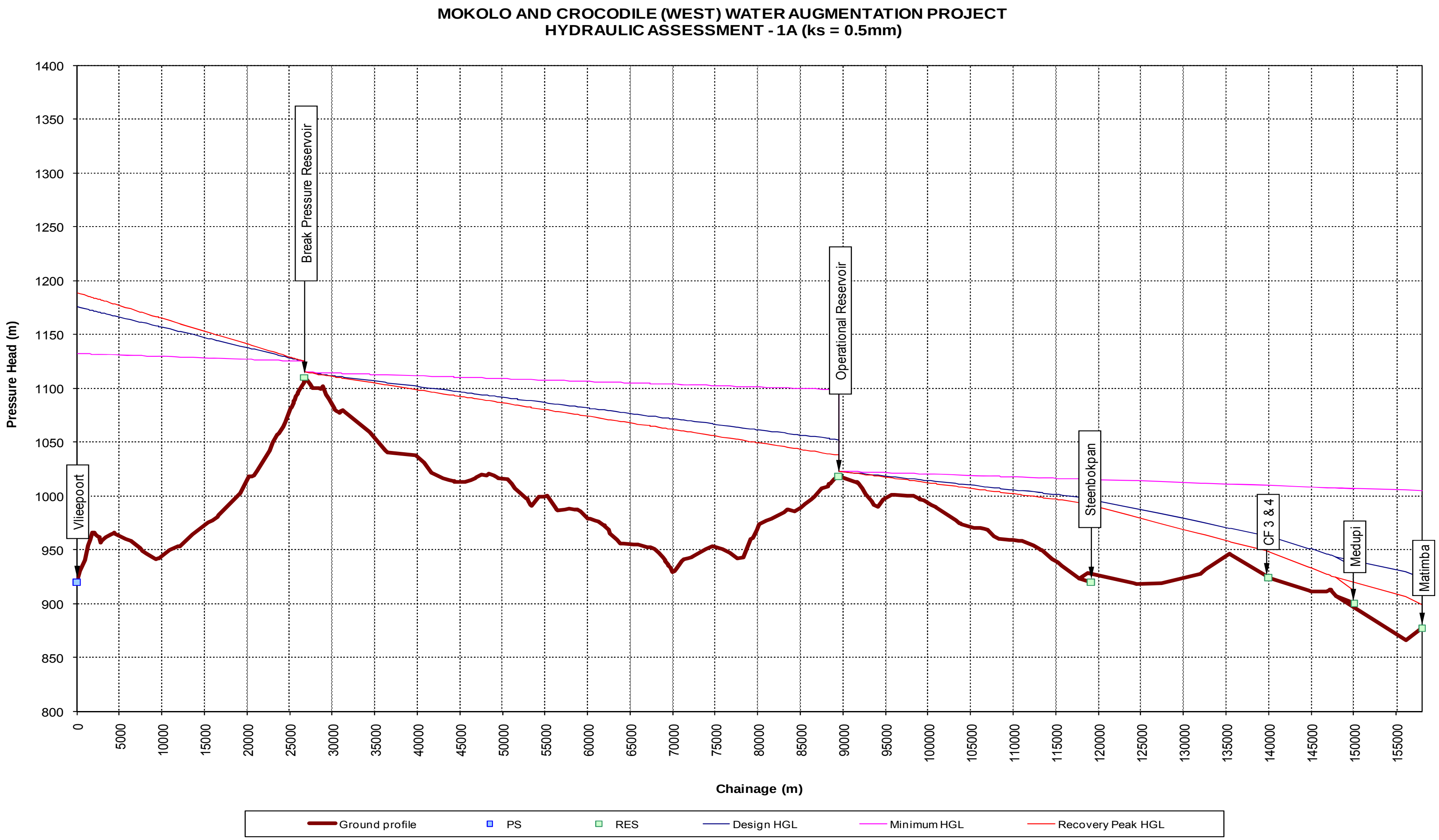


Figure 5-6: Crocodile River (West) Transfer Scheme Hydraulic Performance – Ultimate System Performance

5.3.5. Cathodic Protection and AC Mitigation

The proposed pipeline routes run parallel to and cross a number of existing and proposed future high voltage power line routes, most notably, the planned new Eskom corridors that will be constructed as part of the Mmamabula-Medupi Transmission Integration Project. These corridors will contain six 765 kV overhead high voltage AC power lines.

The pipeline also runs parallel to the railway line for a significant distance. The railway line is currently not electrified, but if electrified in future, it is expected to be with AC power.

Stray current interference is expected on the pipeline and an assessment was therefore done to determine the required CP and AC mitigation measures that will be required to protect the proposed pipeline. The assessment was based on the following conditions and assumptions:

- Preliminary site investigations of the routes of the proposed pipelines have confirmed the presence of significant corrosive soil conditions in places.
- The quantities are based on the route lengths and pipeline diameters as analysed for the feasibility study. Wall thicknesses were based on an average D/t ratio of 100.
- Isolation joints at pump stations and reservoirs have not been allowed for at this stage.
- The provision of AC power for the Transformer Rectifier Units (TRU) is based on running overhead cables at a cost of R220 000 per kilometre. It was assumed that TRUs can be positioned within 2 km of distribution lines (11 kV network).
- Allowance was made for temporary CP during construction. This is essential for construction of pipelines with high integrity coatings in AC interference situations.
- The CP system is based on using Sintakote™ pipeline coating and tape wrap at the field joints.
- CP test posts have been allowed at 500 m centres.
- Cross-bonding facilities between pipelines have been allowed for.
- Both AC power lines and railway lines that run parallel to and within a 2 km corridor of the pipeline have been taken into consideration.
- The investigation has assumed that all proposed power line corridors shown on the layout drawings will be populated. Sections with pipe/power line parallelism will require AC mitigation. This comprises the use of two zinc wires in the trench, next to the pipeline and earth mats installed at each test post.
- The railway lines in the area are not electrified at this time and it is envisaged that, when electrified, it is expected to be with AC power as stated earlier. This means that Natural Drainage Units (NDU) will probably not be required.

Soil Resistivity:

The significance of soil resistivity in relation to CP requirements is that it provides an indication of the ability of the soil to conduct electric current. In high resistivity soils, corrosion currents are small or negligible; hence corrosion is not a major problem. Low resistivity soils not only allow higher corrosion currents, but they are also associated with high moisture contents and dissolved salts, which may be corrosive in themselves.

The most commonly used classification for soil resistivity and the requirement for CP is given in **Table 5-12**.

Table 5-12: Classification of Soil Resistivity

Resistivity (Ohm.m)	Classification	Cathodic Protection Requirements
< 20Ωm	Extremely corrosive	Definitely required
> 20Ωm but < 50Ωm	Corrosive	Definitely required
> 50Ωm but < 100Ωm	Mildly corrosive	Usually required
> 100Ωm	Not generally corrosive	Not generally required

A first order soil resistivity assessment along the Central route found that the route is characterised by lower resistivity, with values of less than 50 Ωm and isolated areas dropping to less than 10 Ωm. The resistivity readings along the Central route are illustrated by **Figure 5-7**.

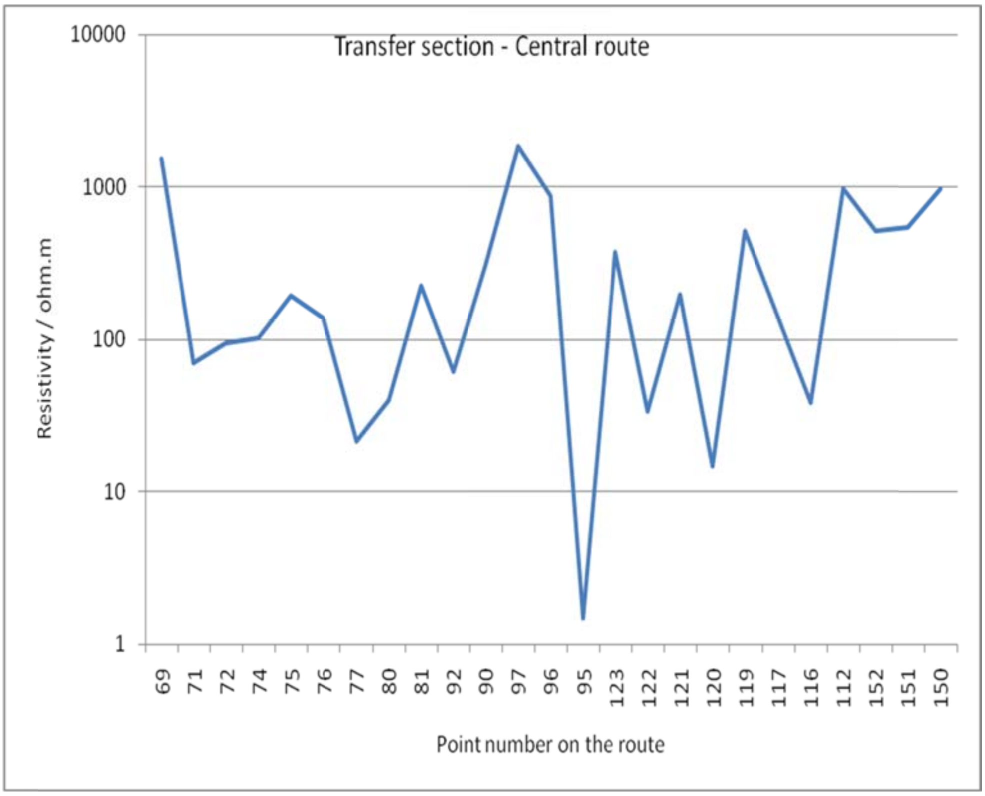


Figure 5-7: Resistivity along Central Route

Should a corridor not be used for transmission lines, it would result in a cost saving on the AC mitigation requirements for a parallel pipeline. It was, however, found that the cost saving that could be achieved, was not significant compared to the overall cost of the CP and AC mitigation. The total cost of CP and AC mitigation, assuming all transmission line corridors to be populated, was thus used in the cost estimate of the scheme. The presence of high voltage transmission lines parallel to a pipeline would result in a bigger maintenance burden on the operator of the scheme to ensure that the CP and AC mitigation system is diligently operated and maintained. It would also imply a higher health and safety risk associated with the maintenance of the system.

The following actions need to be performed as part of the detail design of the CP and AC mitigation design:

- A detailed soil resistivity survey at 100 to 500 m centres, depending on soil conditions.
- Soil sampling and analysis to confirm the aggressiveness of the soil and the possible presence of Sulphate Reducing Bacteria (SRB) that could affect the coating selection.
- Detailed AC modelling to confirm the extent of AC mitigation required

5.3.6. Geotechnical Overview

An exploratory geotechnical investigation was undertaken during the course of the Feasibility Stage and is reported on in Supporting Report 8B (P RSA A000/00/8709). Test pits were excavated at a nominal spacing of 5 km along the route of the pipeline. No laboratory testing was undertaken, nor were potential borrow areas investigated in detail. The purpose of the investigation was to give a general impression of likely geotechnical conditions along the pipeline route. Due to the wide spacing of the pits, only indicative conclusions and summaries can be made regarding the expected ground conditions. Pertinent findings of the exploratory work are summarised in this section.

The geology along the pipeline route varies considerably as shown in **Table 5-13** and on DWG W9528/GEO/CS/002 included in **Appendix A**.

Table 5-13: Phase 2A Geology

Rock Types	Formation	Group
Sand, ferricrete, calcrete		Quaternary
Diabase		Post-Waterberg intrusive
Sandstone	Mogalakwena Formation	Waterberg Group
Sandstone, mudstone	Makgabeng Formation	
Dolomite, chert	Malmani Subgroup	Transvaal Supergroup
Quartzite, shale	Black Reef Formation	
Arkose, greywacke, andesite	Buffelsfontein Group	
Granite		Lebowa Granite Suite
Granite, gneiss		Archaean Complex

The pipeline route commences in the south at Vlieëpoort, where it is underlain by rocks of the Transvaal Supergroup (mostly dolomite, chert, arkose and andesite), before crossing onto Archaean Granite. After crossing back onto Transvaal Supergroup rocks, it then traverses mainly Waterberg Group sediments (sandstone and some mudstone), with patches of granite and diabase. In the north (from about 10 km south of the point where the pipeline turns away from the railway line) the pipeline is on Quaternary sands (with calcrete and ferricrete), which overlie Waterberg Group sandstone.

The various geological units encountered along the centreline of the pipeline are given sequentially (from south to north) in **Table 5-14** and their extent is shown.

Table 5-14: Phase 2A - Geology Summary

Approx Chainage Distance from Vlieëpoort (km)	Anticipated Geology
0 – 5	Dolomite
5 - 7.7	Pretoria Group and dolomite
7.7 - 15.7	Granite
15.7 - 34.2	Alluvium
34.2 - 37.2	Dolomite
37.2 - 42.6	Sandstone
42.6 - 57.6	Alluvium
57.6 - 86.6	Sandstone, diabase
86.6 - 98.8	Alluvium
98.8 – 125	Quaternary sand, diabase

In general, the Phase 2A route is characterised by varied geology. The variation is less prevalent within the Waterberg Group to the north, which is similar to the geology encountered along the western half of the Phase 1 pipeline alignment. Along the southern half of the Phase 2A route, extensive outcropping of rock was observed within the rail cuttings. Significant alluvium deposits are also anticipated along the Phase 2A route, which may include expansive soils. At the southern end of the pipeline route Transvaal Supergroup rocks will be encountered.

The profile summary indicates generally thinner soils over the southern half of the route. The test pits and documented geology indicate that borrow pits are likely to be easily identified over the northern half of the route. The thinner, more gravelly soils over the southern half will make the location of borrow pits more difficult, and significant haulage distances may be necessary. Transportation of the borrow material will not be significantly hampered by adverse topography, as the majority of the pipeline lies alongside existing roads and adjacent to the railway reserve. Where the pipeline route lies within farm boundaries, the land is also generally flat.

The geotechnical conditions anticipated along the pipeline are summarised below:

a) Excavatability

The average depth to refusal was predicted to be as follows:

Table 5-15: Phase 2A Average Depth to Refusal

Approx Chainage Distance from Vlieëpoort (km)	Average Refusal Depth Achieved with TLB (m)
0-37	2.2
37-54	2.5
54-56	0.0
56-59	2.0
59-71	1.5
71-75	0.0
75-78	2.5
78-92	3.0

b) Suitability for bedding and backfill

The predicted thickness of material suitable for use as bedding or soft backfill is as follows. It should be noted that no laboratory testing was performed to confirm the engineering properties of the excavated material. These figures should therefore be regarded as indicative only.

Table 5-16: Phase 2A Average Thickness of Suitable Material

Approx Chainage Distance from Vlieëpoort (km)	Average Thickness of Suitable Material (m)
0-6	0.0
6-15	1.5
15-37	0.0

Approx Chainage Distance from Vlieëpoort (km)	Average Thickness of Suitable Material (m)
37-50	1.5
50-56	0.0
56-58	2.0
58-61	1.0
61-75	0.0
75-82	2.0
82-88	3.0
88-101	1.5
101-106	2.0
106-120	1.0

c) Groundwater

No seepage was encountered in any of the test pits dug along the route. Test pits were, however, only at a nominal spacing of 5 km and might not be indicative of the conditions along the whole route.

d) Borrow Sources

No investigations for bedding and soft backfill were carried out. Notwithstanding this, bedding material and soft backfill should be freely available north of about km 55 and an average haul distance not exceeding 5 km should be attainable. In this area, it is probable that much of the soil excavated from the pipe trench will be re-usable for bedding and soft backfill. Between km 0 and 55 borrow sources would seem to be difficult to locate and haul distances of more than 5 km are likely. Possible locations of borrow sources are indicated on Drawing WP 9528/GEO/CTS/015 included in **Appendix A**.

A detailed geotechnical investigation must be performed as part of the detail design appointment. The investigation should address the following as a minimum requirement:

- Geological mapping - Delineation and description of outcrop areas including discontinuity survey, geological structures, etc.
- Test pitting with an excavator at selected spots at an average of about 200 m centres – The maximum depth of the proposed pipeline is generally more than 4 m, deeper than the reach of a tractor loader backhoe (TLB). The soil profile must be described

according to the standard method of Jennings *et al* with reference to shallow water table conditions, excavatibility, etc.

- Core drilling – to investigate pipe jacking and reservoir sites.
- In situ testing - For the determination of soil parameters for pipeline design (the empirical E' value (bulk modulus of horizontal soil reaction), limited plate load tests must be conducted at selected representative positions.
- Sampling and Laboratory testing – Disturbed and undisturbed samples of selected representative soil horizons must be collected and tested at an SABS approved laboratory to determine the soil characteristics such as grading, expansiveness, collapse, potential use for backfill, indicators, etc.
- The corrosiveness of the material must be determined by analysing the pH and electrical conductivity of selected samples.
- Identification and proving of potential borrow sites – Borrow sites to be identified to ensure that haul distances are kept to a minimum. The volume of borrow material to be proven by a dense grid survey and adequate laboratory testing, providing at least twice the volume required at each site.
- Field electric resistivity survey - A field survey must be conducted to determine the in situ electrical resistivity along the entire route in collaboration with the CP analysis and design.

5.4. High-Lift Pump Station

5.4.1. Pump Station Design Parameters

The High-lift pump station will be required to deliver water via a rising main of 26.7 km to a BPR from where it will flow to an Operational Reservoir and Terminal Reservoirs under gravity (refer to Section 5.2). The design capacity is based on the water requirements and design criteria listed under Section 3 and summarised in **Table 3-5**.

The pump investigations showed that the design flow rate can be achieved by using four active pumping lines with one standby line. Each pumping line will comprise an in-line booster pump and a main high pressure pump (no valve in between). The minimum static suction head required for the booster pumps was estimated to be 8 m based on the site conditions and the likely Net Positive Suction Head (NPSH) of the booster pumps. To reduce excavation depth careful design of the inlet conditions in the reservoir and the suction manifold will be required. It is presently envisaged that Variable Speed Drive (VSD) pump sets be installed to enable continuous, more economical pumping and improve the flexibility of the pumping scheme. In addition, VSD drives would greatly reduce starting currents and reduce pressure surges in the system.

With a maximum monometric pumping head in the order of 255 m (Refer Section 5.3.3), a single high lift pump station is considered the most economical solution.

Standard provision will be made with respect to aspects such as:

- The pump station will be free draining (i.e. be above the PMF level).
- Drive through access through the pump station under the station craneage.
- Valving inside and outside the pump station.
- Craneage.
- Local operational control room, offices and meeting room.
- Local operational control consoles.
- MV switchgear.
- Store rooms.
- Ablution facilities.
- Provision for potable water and sewage disposal.
- Parking facilities.
- Flow measurement in the rising main directly downstream of the High-lift pump station.
- Strategic spares (See Supporting Report No 10: Requirements for the Sustainable Delivery of Water, Appendix E).
- Provision for future extension of the pump station.
- Security provision.

5.4.2. Preliminary Layout

As stated in Section 4.6.5, submersible pumps are the most suitable for the abstraction of water from the CRW. Based on the pump criteria (max pumping head of 50 m), as well as the requirements for the desilting channels, the Balancing Dam, the static head required at the pump suction (i.e. 8 m below the minimum operating level in the Balancing Dam) and the need for the High-lift pump well to be free draining (i.e. pump well, being approximately 1.0 m above the PMF level of the river at the pump station site), the layout of the desilting channels, Balancing Dam and High-lift pump station was configured. This is shown on drawings WP 9528 LD/CTS/001 and DD/CTS/001 included in **Appendix C**.

The position of the Eskom switchyard is also shown on the layout together with the access roads and provision for future extension of the pump station.

5.4.3. Geotechnical Considerations

An exploratory geotechnical investigation was undertaken during the course of the Feasibility stage and is reported on in Supporting Report 8B (P RSA A000/00/8709). Refer to summary of the Vlieëpoort site geotechnical considerations summarised under Section 4.5. It was confirmed that the site located closest to the abstraction weir is located on dolomite. This could have a significant impact on the location of the pump station and associated facilities and need to be investigated further as part of the detailed geotechnical investigation.

5.4.4. Pump Selection

The design capacity, based on the water requirements and design criteria listed under Section 3 and summarised in **Table 3-5** is as follows for the 2030 design horizon:

Design flow : 5.8 m^3/s
Recovery peak flow : 6.6 m^3/s

The optimum pipe diameter selection is described under Section 5.3.3 and the expected system duty points are summarised in **Table 5-10** and illustrated by **Figure 5-4**.

The pump characteristics used to calculate the power absorption are as follows:

- High-lift pump sets 4 duty + 1 standby
- Efficiency of pump (Ep) 90%
- Efficiency of motor (Em) 97%
- MW/MVA ratio (power factor Pf) 0.96

The estimated absorbed power (MVA) at the two duty ratings are summarised in **Table 5-17** for the minimum and maximum operating head.

Table 5-17: Vlieëpoort High-Lift Pump Station Duty Point and Absorbed Power

	Design Flow	**Recovery Peak Flow**
Minimum System Head (Min static head and 0.05 mm absolute roughness)		
Duty	5.8 m^3/s @ 216 m	6.6 m^3/s @ 226 m
Absorbed power	14.8 MVA	17.7 MVA
Maximum System Head (Max static head and 0.5 mm absolute roughness)		
Duty	5.8 m^3/s @ 240 m	6.6 m^3/s @ 255 m
Absorbed power	16.5 MVA	19.9 MVA

The estimated peak power consumption will be in the order of 20 MVA for the Phase 2A Transfer Scheme. Future upgrading of the pump station has not been evaluated and the final load requirements must be evaluated when the pump selections are done. The pump selection must be evaluated carefully to provide the required flow flexibility that the system requires.

5.4.5. Bulk Electricity Supply

The external power supply to the pump station site shall be firm and the risk of natural flooding, bush fires and lightning to the power lines must be minimised. The Eskom sub-station is positioned to the North-West of the pump station alongside the Balancing Dam as shown on DWG WP 9528 LD/CTS/001.

5.4.5.1. Vlieëpoort Bulk Connection

The Vlieëpoort site will be fed from the Thabazimbi Munic (Thaba Combined) and the planned Thabazimbi Rural (Thabatshipi) 132 kV sub-stations. High voltage transmission lines will be built from each substation to the Vlieëpoort site, in order to ensure redundancy. (Loop in – Loop out system).

Eskom is in the process of strengthening the 132 kV system in the Thabazimbi area. Transmission line routes must, however, still be finalised. The available loading capacity of these lines will exceed the required demand. The proposed transmission lines may be used by Eskom as load transfer lines between Thabazimbi Munic sub-station and Thabazimbi Rural sub-station in the future. The planning for these lines is currently in the Eskom Definition Release Approval (DRA) stage.

Figure 5-8 shows the general distribution areas and the proposed transmission line routes.

Figure 5-8: Bulk Electrical Connection to Vlieëpoort Site

The following must form part of Eskom's scope of services to provide bulk power to the site:

- Construction of new 132 kV transmission lines as part of the strengthening of the 132 kV system in the Thabazimbi area.
- Construction of a new sub-station at Vlieëoort site, approximately 6.3 km from the new Thabatshipi sub-station (or old Thabazimbi Rural sub-station).
- Installation of 1 x 40 MVA 132/11 kV transformer unit and all associated equipment.
- The sub-station will cater for two incoming 132 kV feeder bays and an 11 kV busbar.
- Adequate space must be reserved for two additional transformers should they be needed in future.
- Provide a Kingbird type T-off line on the new Thabatshipi-Thabazimbi combined line into Vlieëpoort sub-station.
- Energise the sub-station and provide power as required.
- The following premium equipment must be installed to ensure redundancy:
    - A second 1 x 40 MVA 132/11 kV transformer and all associated equipment;
    - Equip a second 132 kV feeder bay at Vlieëpoort sub-station; and
    - Convert the T-off on the Thabatshipi-Thabazimbi combined line into a loop-in loop- out section by building another Kingbird type connection to Vlieëpoort.

Medium voltage power will be delivered to the High-lift pump station by means of cables laid in ducts (to be determined in the design phase) to appropriate transformers outside the switch gear room of the pump station.

The proposed network diagram for the bulk electricity supply to the Vlieëpoort site is illustrated by **Figure 5-9**.

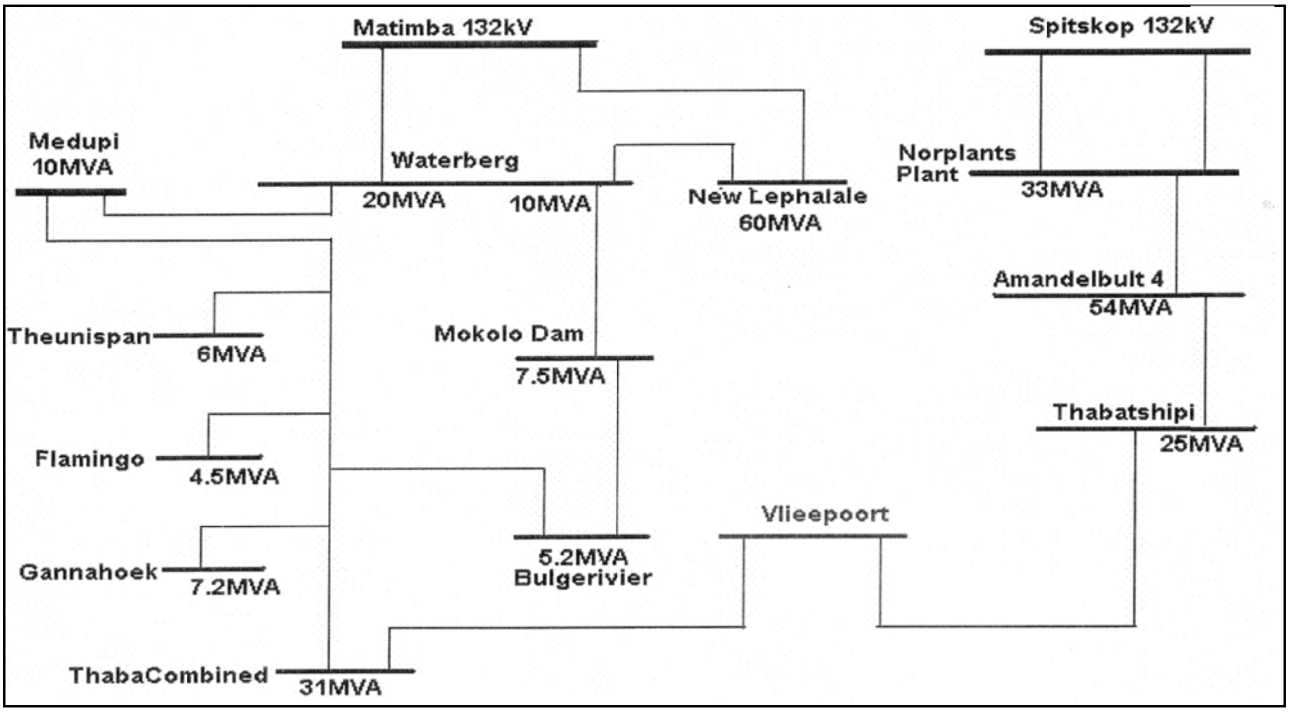


Figure 5-9: Bulk Electricity Supply Network Diagram

5.4.5.2. Vlieëpoort Abstraction Pump Station

The estimated peak power consumption of the Abstraction pump station will be 5 MVA for the Phase 2A peak flow.

It is proposed that the abstraction pump station be fed from the substation located at the High-lift pump station via two 11 kV lines (Hare) to a switching station located at the abstraction works above the PMF level. The connection to the Vlieëpoort site must have sufficient capacity to supply both pump stations.

5.4.5.3. Reservoir Sites

Electricity supply will also be required to the respective reservoir sites to power the telemetry and control equipment. The load will, however, be small and the supply can be provided via the existing Eskom rural distribution system. Applications for electricity connections to the reservoir sites have not been submitted to Eskom and must be done timeously as soon as the exact site layout and loads is known. Options to firm up the supply to the respective sites (loop-in loop-out), or standby power supply must also be considered.

5.5. Break Pressure, Operational and Terminal Reservoirs

The CRW Transfer Scheme includes a BPR located on the farm Zondagskuil 130 KQ, as well as an Operational Reservoir located on the farm Zoutpan 367 LQ.

The scheme will supply water into Terminal Reservoirs located on the sites of the end consumers. The Terminal Reservoirs must provide a minimum of 18 days storage and will be built by the respective end users, but will be operated and controlled by the MCWAP.

Although smaller in size, the Operational and Break Pressure reservoirs will be similar in design to the Balancing Dam at the Vlieëpoort site as illustrated on the drawings included in **Appendix C**.

5.5.1. General Criteria

The following general criteria will apply to all the reservoirs:

- The Reservoirs will be in the form of an artificial dam formed by shallow excavation and surrounding earthfill embankments. The final depth and size of the reservoirs will be determined by the site topography (cut and fill balance) with the aim of minimising surface area to reduce evaporation and maximum flow through to prevent stagnation in areas of dead water.
- Reservoirs will be lined with an appropriate waterproof lining system (HDPE or similar material) and suitable sub-surface drainage must be provided.
- Reservoirs will be compartmentalised to allow for normal operation, maintenance and cleaning (i.e. a minimum of two compartments), as well as the mitigating requirements relating to water quality that may be required.
- Reservoir inlets will be top-entry with an energy dissipation structure, such as a stepped spillway. Further investigation is required into a suitable inlet structure with energy dissipation especially considering possible high static head conditions during low flow conditions. A bottom inlet solution can also be considered.
- All reservoirs will have bottom outlets connected to an outlet manifold with suitable isolation and flow control. Automatic high flow protection valves must be considered downstream of the Break Pressure and Operational Reservoirs to protect the pipeline against draining in the event of a catastrophic pipe burst. Inlet control valves will be required to maintain pressure in the incoming pipeline and ensure full flow conditions at all times.
- During the detail design phase the effect and frequency of power failures and the emptying of the gravity sections (from the Break Pressure and Operational Reservoirs) must be investigated, and if necessary remedial design measures implemented.
- All reservoirs will have level indication linked to the remote control centre for normal operational purposes. These levels will also be monitored by the end consumer(s) in the case of the Terminal Reservoirs.
- Allowance must be made for future extensions to cater for the ultimate delivery phase. In the case of the Terminal Reservoirs, it will be the responsibility of the end users to make sufficient land available for this purpose.

- Adequate provision must be made at each reservoir site for site drainage as well as reservoir scour and overflow, taking cognisance of the potential impact of poor quality water on the surrounding farming activities, streams and rivers.

5.5.2. Break Pressure Reservoir

5.5.2.1. Site Selection

As indicated in Section 5.2, the pipeline route from the Vlieëpoort High-lift pump station crosses over high ground at chainage 26.7 km. The elevation in this area is such that a BPR can be located at this point to enable gravity flow onwards to the Operational Reservoir. Reducing the length of the rising main is beneficial from a pump operation point of view, especially during initial stages of the scheme operation when water requirements are low. The final positioning of the reservoir must be done once the detail survey is available and the geotechnical investigation is complete. Due consideration should also be given to minimising the environmental, social and visual impact of the reservoir structure and associated infrastructure.

5.5.2.2. Design Parameters

The function of this reservoir will be twofold:

i) To break the pressure at the end of the rising main and provide balancing storage between the rising main from the pump station and the gravity main to the Operational reservoir; and

ii) To feed water into the gravity pipeline linking the Break Pressure and Operational Reservoirs.

This reservoir will have a minimum total combined storage capacity of 8 hours of the recovery peak flow to provide effective balancing capacity for differences in outflow and inflow. A minimum of two compartments will be provided for normal operational and maintenance purposes.

An automatic altitude shut-off valve at the outlet of the rising main with pressure and/or low flow activated switches at the pumps is proposed to close off the inflow to prevent the reservoir from spilling in the event of a communication failure.

It is expected that the BPR will have to be converted to a surge reservoir when converting the gravity pipeline to a rising main to the Operational Reservoir to increase the transfer capacity of the scheme (refer to Section 5.2.6). The detail design process must consider the possible future conversion of the BPR in terms of the positioning and sizing of inlet and outlet pipework, valves and isolation, possibility of reducing the Surge Reservoir size by further compartmentalisation of the BPR and the possibly of covering the surge reservoir or ensuring constant circulation through the surge reservoir due to quality concerns with long term stagnant water.

5.5.3. Operational Reservoir

5.5.3.1. Site Selection

Water will flow from the BPR through a 62.7 km long, 2 200 mm ND gravity main to the Operational Reservoir located on the farm Zoutpan 367 LQ at an elevation from where all the end user Terminal Reservoirs can be commanded. The final positioning of the reservoir must be done once the detail survey is available and the geotechnical investigation is complete. Due consideration should also be given to minimising the environmental, social and visual impact of the reservoir structure and associated infrastructure.

5.5.3.2. Design Parameters

This reservoir will have a minimum total combined storage capacity of 8 hours of the recovery peak flow to allow for effective distribution and smoothing out of variations in the demand. The final sizing of the reservoir should take cognisance of the final end user water requirements. A minimum of two compartments must be provided for normal operational and maintenance (O&M) purposes. It must also be designed to ensure that the gravity inflow pipeline will be running full at all times down to the lowest operating level and over the full range of flows and pipe roughness.

To reduce the high level of automation on the project, consideration must be given to the use of multiple sleeve inlet control valves (i.e. three sleeve valves vertically aligned at approximately 0.3 m) for staged closing to full supply level.

5.5.4. Terminal Reservoirs

5.5.4.1. Site Selection

The Terminal Reservoirs will be located on sites made available by the different consumer authorities. The gravity feed pipelines from the Operational Reservoir were sized for costing purposes to provide the full consumer requirement at a minimum residual head of 25 m to account for uncertainty on the final location and levels of the reservoirs (15 m for hydraulic losses through valves and 10 m for any changes in elevation and head loss in the connecting pipelines to the Terminal Reservoirs).

5.5.4.2. Design Parameters

The Terminal Reservoirs will typically be compartmentalised and each will have a minimum storage capacity of 18 days of the average annual water requirement of the consumers supplied from it, which will be reserved for purposes of the MCWAP operation and maintenance only, plus additional storage to be determined by consumers for their own internal balancing and operational requirements. The latter shall include consumers internal peaking requirements, fire fighting needs, etc., and will have to be provided in full for each development phase (i.e. may not be staggered within a development phase) to ensure that the 18-day minimum required storage is always available and that other consumers are not put at risk when new developments take place.

These reservoirs, and possible associated consumers pumping stations, will be designed, constructed, operated and maintained by the consumers. Abstraction from any of these Terminal Reservoirs shall be managed and operated by the consumers, whether by gravity or by pumping. This will include all valves, pipe work and pump stations. Included in the operation of the operations of the reservoirs will be due consideration and allowance for addressing the water quality concerns associated with long term storage of CRW water.

The MCWAP will provide and control the following components in the gravity feed main directly upstream of the Terminal Reservoirs on the connection to the bulk supply pipeline:

- An in-line isolating valve with chamber;
- An in-line revenue flow meter with chamber; and
- An in-line flow volume control valve (with chamber) which will limit the incoming flow to the cumulative annual average flow allocation to be revised and adjusted regularly to allow for growth during the early stages of the project.

To limit the high level of automation on the project, consideration must be given to the use of multiple sleeve inlet control valves (i.e. three sleeve valves vertically aligned at approximately 0.3 m) for staged closing to full supply level.

5.6. Summary: MCWAP Infrastructure Components

A summary of the infrastructure components comprising the combined MCWAP is given in **Table 5-18**.

Table 5-18: MCWAP – Summary of Infrastructure Components

Component	Description		
Mokolo Dam Scheme (Phase 1)			
Phase 1	New pumping station and additional pipeline from Mokolo Dam to end-users located from Lephalale in the east to Steenbokpan in the west. The Lephalale-Steenbokpan link will be built as part of Phase 1, but will ultimately form part of the CRW Transfer Scheme to transport CRW water to Medupi and the Grootegeluk/Matimba control chamber.		
Lephalale-Steenbokpan Link			
Crocodile River (West) Connection to Steenbokpan	Diameter	:	1 900 mm ND
	Length	:	1.4 km
Crocodile River (West) Connection to CF 3&4 Mining T-off	Diameter	:	11 00 mm ND
	Length	:	27.1 km
CF 3&4 Mining to Medupi T-off	Diameter	:	900 mm ND

Component	Description		
Medupi T-off to Steenbokpan T-off	Length Diameter Length	: : :	3.6 km 900 mm ND 8.2 km
Crocodile River (West) Transfer Scheme (Phase 2A)			
Vlieëpoort Abstraction Works	Concrete weir, gravel trap and pump intake structure– civil structures sized for ultimate project water requirements (431 Million m^3/a) 1 x fully equipped standby bay plus 1 standby pump unit (stored on site) 8 x 1.0 m^3/s submersible pumps Maximum duty point: 6.6 m^3/s @ 49.5 m Absorbed Power: 4.7 MVA 1 300 000 m^3 active balancing storage		
High lift pump station	Static head Design peak flow (DPF) Min manometric head at DPF Recovery peak flow (RPF) Max manometric head at RPF Power consumption DPF/RPF	: : : : : :	183-192 m 5.8 m^3/s 216 m 6.6 m^3/s 255 m 16/19 MW
Pipelines Rising main – High-lift pump station to Operational and Break Pressure Reservoir (Node 10) *Break Pressure and Operational Reservoir* Operational and Break Pressure Reservoir to Node 15 to Crocodile River (West) Transfer Scheme Connection (Phase 2A)	Diameter Length 8 hrs storage of recovery peak flow rate Diameter Length Diameter Length	: : : : : : :	1 900 mm ND 26.7 km 190 000 m^3 2 200 mm ND 62.7 km 2 300 mm ND 28.2 km
Steenbokpan T-off to Grootegeluk/Matimba Control Chamber	Diameter Length	: :	800 mm ND 1.9 km

5.7. Conclusions

5.7.1. High-Lift Pump Station

All the requirements relating to the suction head required for the estimated NPSH of the booster pumps, a free draining pump station with no risk of flooding, etc. have been satisfied for the selected site configuration.

The pump configuration that has been adopted comprises four identical duty sets plus one standby set comprising an in-line booster pump and a main high pressure pump without a valve between the two pumps.

VSD pump sets are recommended to enable more economical continuous pumping with added flexibility and soft start features.

The calculated duty point and absorbed power requirements for the pumping station is summarised in **Table 5-17**.

5.7.2. Pipelines

A rising main to the BPR at Ch 26700 and gravity flow to the Operational Reservoir and further to the connection with the Lephalale-Steenbokpan pipeline were found to be the most viable and practical solution and are recommended for implementation. The optimum pipe diameter for the rising main from Vlieëpoort to the BPR was found to be 1 900 mm with the gravity pipeline from the BPR to the Operational Reservoir requiring a 2 200 mm ND and a 2 300 mm ND downstream of the Operational Reservoir to the CRW connection point to the infrastructure provided as part of Phase 1 with diameters ranging from 800 mm to 1 900 mm ND.

The pipeline route planning took cognisance of existing linear infrastructure and farm boundaries as far as possible and practical to limit the social and environmental impact. A permanent servitude width of 50 m was used for costing purposes. An exploratory geotechnical and CP investigation was performed as part of the Feasibility Investigation. No adverse conditions that would totally prohibit the construction of the pipeline were found to exist along the proposed routes. Further detailed coordination with services authorities and affected parties will be required to obtain the necessary way leaves and approvals.

Possible coating and lining options are recommended in **Table 5-7** and the recommended pipe roughness parameters to be used during the detailed hydraulic design are summarised in **Table 5-8**.

The structural design and optimisation of the pipeline must be performed as part of the detail design.

5.7.3. Reservoirs

Criteria for the location and design of the Break Pressure, Operational and Terminal Reservoirs were defined in line with the operating philosophy of the scheme. Allowance was made for 8 hours storage capacity in the Break Pressure and Operational Reservoirs. The sizing of the Break Pressure and Operational Reservoirs can be optimised further when the final system capacity and operating philosophy and the associated risk is agreed with the end-users. The Terminal Reservoirs was sized to provide 18 days storage in line with the reliability criteria for the project.

5.8. Key Aspects to be considered during the Next Stage of the Design

The following issues were identified during the course of the Feasibility stage and would require further investigation as part of the detail design:

1. Since April 2008, there have been a number of changes to the parameters (most notably the variations in the water requirements) that could influence the capacity, location and design of the MCWAP and the CRW Transfer Scheme in particular. The pipe systems can be optimised further when final design capacities, and more detailed survey and geotechnical information becomes available. It is therefore recommended that a more comprehensive evaluation and optimisation be performed during the detail design stage to verify the Feasibility findings before starting with the detail design of components.

2. Route planning and coordination. The following is required:
   - Detailed coordination and a commitment to the MCWAP by the bulk consumers along the Lephalale-Steenbokpan corridor is required in order to ensure integrated planning of infrastructure and water requirements.
   - Eskom is planning to construct a number of high-voltage power lines through the region. A number of these will be located in a corridor routed from north to south that could affect the routing and design of the MCWAP pipelines.
   - Agree on the permanent servitude requirements to allow for possible future expansion (parallel pipeline(s)).
   - Detailed cadastral and existing services information must be obtained along the final pipeline route alignments.
   - The land acquisition and servitude registration process must be facilitated and supported, taking cognisance of issues raised by interested and affected parties during the public participation process.
   - The locations of farmer off-takes and water requirements must be confirmed.
   - Further services coordination and way-leave approvals typically need to be obtained from the following parties:
     - <u>Eskom</u>: Capital Projects Planning, Transmission and Distribution.
     - <u>Spoornet</u>: Apply for permission to use railway line access road during construction and for future maintenance access to the pipeline and confirm future upgrade/electrification planning for the railway line. Apply for way-leaves at all railway line crossing sites.
     - <u>South African National Roads Agency Limited (SANRAL)</u>: Apply for a concession to use the road reserves as temporary construction servitudes where pipelines are located nearby a national road. Also apply for access points to pipeline servitude from road reserve and for way-leaves at all road crossings.
     - <u>Limpopo Provincial Roads Department</u>: Apply for a concession to use road reserve as temporary construction servitude where pipeline is located

parallel to a national road. Also apply for access to pipeline servitude from road reserve and for wayleaves at all road crossings.

- Thabazimbi Local Municipality: Obtain future township planning and establishments that might affect pipeline routes.
- Lephalale Local Municipality: Obtain future township planning and establishments that might affect pipeline routes.
- Telkom: Confirm the location of services and apply for way-leaves to cross the services.
- Neotel: Confirm the location of services and apply for way-leaves to cross the services.
- Department of Minerals and Energy: Inform them of the planned pipeline route in order to update their database.
- DWA: Confirm the need to apply for water use licenses for river and stream crossings and obtain the necessary permission, if required.
- Local farmers and land owners: Take forward the land acquisition and servitude registration process to ensure that servitudes for the pipeline and borrow areas are agreed timeously to prevent delays during construction.

3. Abstraction Works: The following issues were identified during the course of the Feasibility stage and would require further investigation to ensure fit for purpose designs:

- Depth of scour at Vlieëpoort during high floods. Scour potential at the weir must be modelled to confirm the depth of founding of the weir structure. The present Feasibility stage layout assumes that the proposed jet grouting foundation treatment will provide adequate founding conditions and that together with the roller bucket spillway design and extensive downstream heavy riprap protection will protect the structure against scour.
- Foundation Design. Deep jet grouted foundations have been successfully used in the past to improve hydraulic structure founding conditions. Once the results of a detailed materials investigation are available, the layouts need to be reviewed and refined.
- Alluvial aquifer flows at Vlieëpoort. The Feasibility stage layouts show that the entire river bed section below the weir will be jet grouted, thereby effectively blocking the flow in the aquifer. Whilst this arrangement is intended to prevent piping foundation failure, greater loads could be imposed on the weir foundations if the water table downstream of the weir is lowered. This can be counteracted if the flow past Vlieëpoort is regulated sufficiently to maintain a continuous flow over the weir. The water table level downstream of the weir should nevertheless be monitored continuously to alert the operators of any potentially dangerous situation.
- Liquefaction Potential. The nature of the underlying alluvial sands and silts at Vlieëpoort must be investigated to determine the potential for liquefaction during a natural or induced seismic event.

- Sizing and Configuration of Desilting Channels. Feedback received on the operation of the Lebalelo Abstraction Works in the Olifants River in Limpopo Province indicated that the very fine fraction of the suspended silt in the Olifants River, when in flood, failed to completely settle out in the de-silting channels. This fraction requires longer retention times to settle out and therefore only settled in the balancing dams where it affected the operational availability of the system and was also difficult and time-consuming to remove, primarily because the balancing dams were not designed to be maintained at frequent intervals. In the case of the CRWTransfer Scheme the problem is accentuated by the relatively large storage capacity and retention times of the Balancing Dam.
- Location of High-lift Pump Station Balancing Dam. The Feasibility layouts identified two potential sites for the dam. The site closest to the Abstraction Weir has since been confirmed to be located on dolomite and should therefore be avoided if possible. The preferred site is some 5 km downstream of the Abstraction Weir and on much more favourable founding conditions (residual Ventersdorp lava), but further planning is required to refine the layout and assess the socio-economic impacts.
- Sizing of the High-lift Pump Station Balancing Dam. The present approach is based on river flow management with a 3 to 4 day river flow response time from the upstream dams to Vlieëpoort. With improved control over flows in the river and shorter actual response times it is anticipated that the required capacity of the Balancing Dam should reduce accordingly. A storage capacity in the order of 200 000 m^3 less may be possible.
- Hydraulic computer modelling of the river is recommended once the detail survey becomes available. This model will allow for better computation of flood levels applicable to the base conditions and post-construction conditions and allow better assessments of the impact of the Abstraction Works on affected landowners and existing infrastructure.
- The hydraulic model will also provide flood levels downstream of the weir that are required for the placement of the Desilting Works, Balancing Dam, High-lift Pump Station and switchyards and might also influence the choice of the site for these components.
- A prototype or Computational Fluid Dynamics (CFD) model of the Abstraction Weir, Gravel Trap and Low-lift Pump Station is recommended in order to optimise the placement, layout and size of these structures.
- During flushing of the Desilting works and desilting of the Balancing Dam, high amounts of silt need to be handled which cannot be discharged into the river. Further investigation is required to confirm environmental requirements and to identify appropriate silt separation facilities and storage and/or disposal thereof.
- Flows passing the Abstraction Weir must be measured. A downstream flow gauging structure will be required to measure surface flows since flows over the weir may not be uniform enough.

4. The following detail design and optimisation actions must be performed:
   - Confirm and agree on the systems operating and control philosophy.
   - Review the pump selection philosophy with specific reference to the option of implementing VSD and the associated implications it has on the operational control, power supply, etc.
   - A detailed pipeline design (optimum diameters and wall thickness). Consider both the interim (rising main/operational and break pressure reservoir/gravity mains) and ultimate (rising main directly to a new operational reservoir with the initial operational and break pressure reservoir converted to a surge reservoir) scenarios and perform detailed surge analyses.
   - A detailed hydraulic analysis to determine the optimum positioning of the air valves (type and size), as well as isolating, reflux, drainage and control valves and acceptable system operating and control procedures. Pipeline dewatering and drainage will require careful consideration due to:
     - Potential poor water quality and fears of contamination; and
     - Very flat topography – management of flushing and drainage water will be problematic. Drainage time of the pipeline must be considered.
   - Optimum sizing of the Operational and Break Pressure Reservoirs to take cognizance of final operating philosophy and risk assessment. The detailed design of the Operational and Break Pressure Reservoirs must consider operational storage requirements, storage time, and water quality management to prevent ‘dead zones’ in the reservoirs. The initial Operational and Break Pressure Reservoir must be configured to allow conversion to a surge tank during later phases of the development if required.
   - River and stream crossings – Matlabas River crossing will require careful consideration of geotechnical conditions at the site, environmental considerations and rehabilitation, as also at all other river and stream crossings.

5. Pipeline Coatings and Linings: New pipeline coating and lining processes are becoming available on the market and must be considered.

6. Detailed AC mitigation design:
   - Cognisance of possible future infrastructure that might affect the design.
   - A detailed soil resistivity survey at 100 to 500 m centres, depending on soil conditions.
   - Soil sampling and analysis to confirm the aggressiveness of the soil and the possible presence of SRB that could affect the coating selection.
   - Detailed AC modelling to confirm the extent of AC mitigation required.

7. Detailed Geotechnical Investigation:
   - Geological mapping - Delineation and description of outcrop areas including discontinuity survey, geological structures, etc.

- Test pitting with an excavator at selected spots at an average of about 200 m centres – The maximum depth of the proposed pipeline is generally more than 4 m, deeper than the reach of a TLB. The soil profile must be described according to the standard method of Jennings *et al* with reference to shallow water table conditions, excavatibility, etc.
- Core drilling – to investigate pipe jacking and reservoir sites.
- In situ testing - For the determination of soil parameters for pipeline design (the empirical E' value (bulk modulus of horizontal soil reaction), limited plate load tests must be conducted at selected representative positions.
- Sampling and Laboratory testing – Disturbed and undisturbed samples of selected representative soil horizons must be collected and tested at an SABS approved laboratory to determine the soil characteristics such as grading, expansiveness, collapse, potential use for backfill, indicators, etc.
- The corrosiveness of the material must be determined by analysing the pH and electrical conductivity of selected samples.
- Identification and proving of potential borrow sites – Borrow sites to be identified to ensure that haul distances are kept to a minimum. The volume of borrow material to be proven by a dense grid survey and adequate laboratory testing, providing at least twice the volume required at each site.
- Field electric resistivity survey - A field survey must be conducted to determine the in situ electrical resistivity along the entire route in collaboration with the CP analysis and design.

6. RIVER FLOW MANAGEMENT AND INSTITUTIONAL ARRANGEMENTS

River flow management and associated institutional processes are fully described in Supporting Report 10 (P RSA A000/00/8609).

The Pre-feasibility investigation found that the additional cost of the water losses to the Vlieëpoort Abstraction Works could not justify the major additional expense of a longer pipeline from an abstraction site closer to the supply dams (Klipvoor, Roodekopjes and Vaalkop). The pipeline construction is on the critical path of the project programme and any additional length added to the pipeline will increase the construction time required and most probably delay the project completion.

At the start of the Pre-feasibility stage, it was assumed that all the water requirements would be available in the river at the Abstraction Works in accordance with a directive received from DWA. It is, however, known that the CRW experience very high river losses. A system of infrastructure that was capable of being operated to reduce transmission losses that occur in response to changes in water requirements, changes in river loss patterns and utilising accruals was envisaged, knowing that the source of the water to be transferred is far removed from the Abstraction Works. In the case of Vlieëpoort, the estimated travel time of water released from Roodekopjes Dam would be in the order of three days. As a result any changes in water requirements, irrigation use, weather patterns (evaporation and rainfall), unauthorised use and other accruals could have a profound effect on flow in the river at the Abstraction Works. In dealing with this problem, three possible approaches for operation of the Abstraction Works (including combinations of these) are possible:

i) Only abstract what is required and let the rest of the flow pass (and ensure that enough water is released from the Roodekopjes Dam so as not to run dry at the Abstraction Works). This approach could result in significant wastage of expensive water if constant continuous releases were to be made for MCWAP at the Roodekopjes Dam.
ii) Design enough capacity into the Abstraction Works (Weir, Low-lift Pump Station and Balancing Dam to allow for abstraction of surplus water, when available, to store for later use when flows in the river are below the required flows.
iii) Implement a River Management System to plan, monitor and control river flows to the best advantage of all users by varying the releases from the Roodekopjes Dam to reduce transmission losses. Note that the implementation of Approach (ii) may well remain a key aspect necessary for the successful implementation of Approach (iii).

The extent of the problem is depicted in **Figure 6-1** below, with the hydrograph for A2H116 (Hugo's Weir) of particular importance.

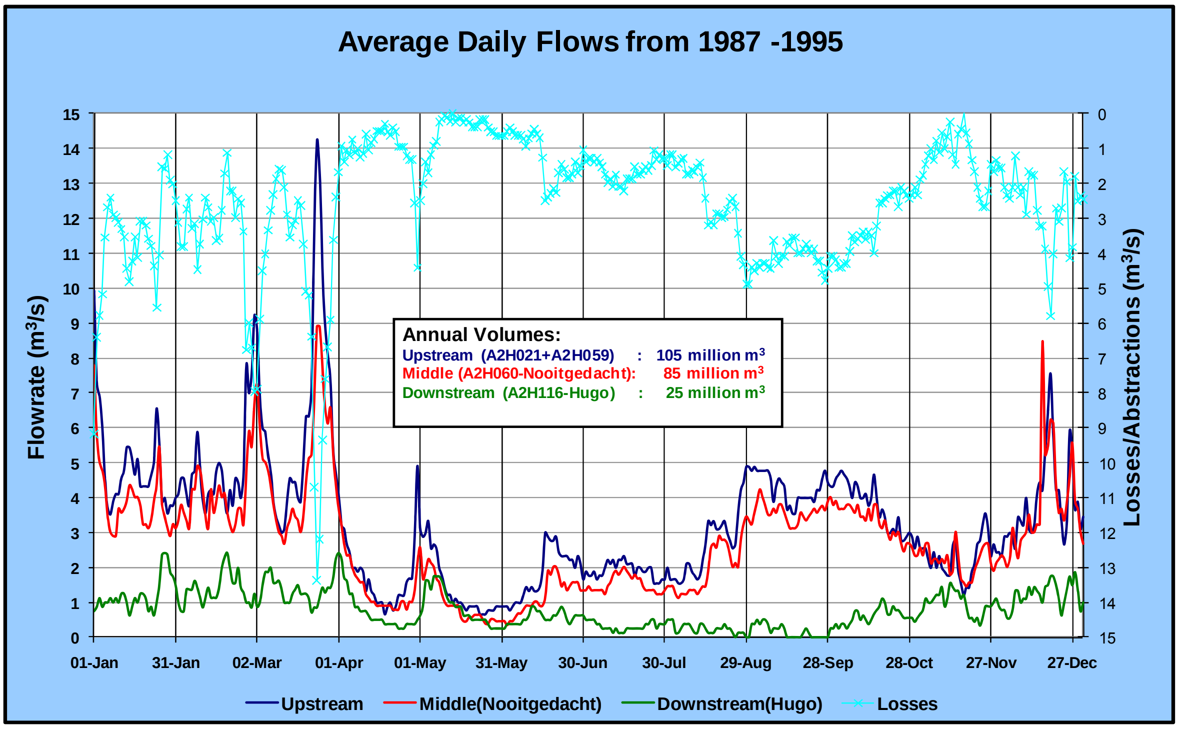


Figure 6-1: Daily Average Flows in the Crocodile River (West)

In view of the pressure on the water resources in the CRW catchment, it was recommended at the close of the Pre-feasibility stage that Approaches (ii) and (iii) be investigated further.

Supporting Report 10 deals with the institutional arrangements, river management systems and operating procedures to minimise transmission losses and unauthorised water use in order to ensure the reliable operation of the MCWAP.

7. OPERATING, MAINTENANCE AND CONTROL PHILOSOPHIES

The control and operation of all sites forming part of the MCWAP will be monitored by means of System Control and Data Acquisition (SCADA) and be managed from a central control room. Furthermore, all sites must be capable of local operation and have sufficient redundancy memory so that, in the event of communications or computer failure, the data can be restored automatically for completeness purposes after restoration of the communication or computer systems.

It is envisaged that the operational control centre will be manned on a 24 hour/day basis. The sites, together with the relevant monitoring functions, are listed below. All the daily, weekly, monthly and annual reports necessary for operational and revenue purposes, must be available for extraction from the data captured by the SCADA system.

For more detail on the operation, maintenance and control of the MCWAP, refer to Report No. 10 – Technical Module: Requirements for the Sustainable Delivery of Water.

7.1. Operational Control Centre

The SCADA will be programmed to give planning, operational and costing reports for a variety of purposes. The following functions will be performed:

- Full operational control of all sites including the abstraction works, pump stations, Balancing Dam, Break Pressure, Operational and end user Terminal Reservoirs and control valves and flow meters that will be provided as part of Phases 1 and 2A of the MCWAP. Also included will be the infrastructure provided to perform the river flow management.
- Monitor river releases and flows to perform the river flow management by the CRW Management Authority.
- The control of the abstraction of surplus river flows in the CRW at Vlieëpoort into off-channel storage into the Balancing Dam optimize water usage.

7.2. MCWAP Operational Monitoring and Control Requirements

The following operational control, monitoring and data capturing will be required at the respective sites on the CRW Transfer Scheme as a minimum. The system must be refined and developed further as part of the detail design.

Table 7-1: MCWAP SCADA Operational Monitoring and Control Requirements

Site	Operational Monitoring and Control Requirements
Vlieëpoort Abstraction Works and High-Lift Pump Station	• CRW water level in the weir at the Abstraction Works. • Discharge rate over the weir (downstream release). • Monitor starting and stopping of river abstraction pumps. • No of desilting channels in use. • Starting and stopping of high lift pumps. • Changes in pump speed of the VSDs on the high lift pumps. • Status of bulk electrical consumption and supply. • Electric power supply voltage in pump stations. • Electric power consumed in each pump station. • Suction and delivery water pressure and flows abstracted from the river and Balancing Dam (both flow rate and cumulative volume required). • Water quality and sediment content of water entering and leaving desilting channels and Balancing Dam. • Status of all manual and automatically actuated valves. • Balancing Dam water levels in each compartment at the abstraction works and which compartments are being drained and filled. • "General health" of all the M&E equipment. • Manual operation of the de-gritting and desilting channels. • Security and access. • Slushing of desilting channels and Balancing Dam. • Weather station at the Abstraction Works .
Mokolo Dam High-Lift Pump Station	• Mokolo Dam water level. • Starting and stopping of high lift pumps. • Changes in pump speed of the VSDs on the high lift pumps. • Status of bulk electrical consumption and supply. • Electric power supply voltage in pump stations. • Electric power consumed in each pump station. • Suction and delivery water pressure and flows (both flow rate and cumulative volume required). • Water quality. • Status of all manual and automatically actuated valves. • "General health" of all the M&E equipment. • Security and access. • Weather station at Mokolo Dam.
Rising Mains (Phases 1 and 2A)	• CP system (i.e. transformer rectifier installations).
Break Pressure and Operational Reservoirs	• Flow into the reservoir (rate and cumulative volume). • Flow out of the reservoir (rate and cumulative volume).

Site	Operational Monitoring and Control Requirements
(Phase 2A) Wolvenfontein Reservoir (Phase 1)	• The water level in both (or all of) the reservoir compartments. Adequate allowance to be made for level measurement redundancy. • Status and operation of all valves and flow meters. • Security and access.
Gravity pipelines feeding the Operational and Terminal Reservoirs (Phase 2A)	• Revenue water meters flow rate and cumulative volume. • CP system (i.e. transformer rectifier installations).
Terminal Reservoirs (Phases 1 and 2A)	• Flow control valve. • Inflow and outflow flow meters (flow rate and cumulative volume). • Inlet pressure. • Isolating valves. • Water levels in every reservoir compartment and whether compartments are being filled or emptied.

7.3. Maintenance Philosophy

Maintenance is generally divided into the three major engineering disciplines namely: mechanical, electrical and civil. For each of these disciplines maintenance will be categorised as follows:

- Routine planned maintenance;
- Major breakdown repairs; and
- Minor breakdown repairs.

7.3.1. Mechanical

7.3.1.1. Routine Planned Maintenance

A schedule of routine maintenance will be compiled to cover all mechanical components such as:

- Exchange of pump and motor unit(s);
- Bearing replacements;
- Water and oil seal adjustment and replacement;
- Servicing (lubrication, oil changing and or refilling);
- Inspection and repair of leaks;
- Painting of components such as valves, pipes and gates;
- Inspection and repair of valves and gate seals in the pump stations, weirs and the de-gritting and desilting channels at the abstraction works;
- Inspection and repair of any hydraulic piping;
- Inspection and repair of all gates, sluices, and valves; and
- Identify and rectify any gradual loss of efficiency on pump sets.

In certain instances, maintenance functions will be based on efficiency monitoring of pump sets and other mechanical components.

Routine maintenance will generally be done by any one or a combination of the following:

- Staff exchanging strategic spares units and taking old units in for refurbishment or replacement;
- Contractors doing maintenance repairs;
- Contractors doing SCADA maintenance on call out; and
- Pump contractors servicing/maintaining units on a regular basis.

7.3.1.2. Major Breakdown Repairs

These repairs will include the rectification of faults shown by SCADA, such as:

- Bearing faults;
- Power supply breakdowns; and
- Rectifying rapid loss of efficiency on pump sets.

7.3.1.3. Minor Breakdown Repairs

These repairs will cover mechanical components such as:

- Exchange of pump and motor unit(s);
- Bearing replacements;
- Water and oil seal adjustment and replacement;
- Repair of leaks;
- Repair of all gates, sluices, and valves; and
- Repair of any hydraulic piping.

7.3.2. Electrical

7.3.2.1. Routine Planned Maintenance

A schedule of routine maintenance will be compiled to cover all electrical components such as:

- Checking/servicing transformer oils;
- Switchgear components;
- Routine calibration of instruments; and
- Routine cleansing of equipment, depending on design.

In certain instances maintenance functions could be based on efficiency monitoring of electrical motors and components.

7.3.2.2. Major Breakdown Repairs

These repairs will cover the rectification of faults shown by SCADA, such as:

- Power supply breakdowns; and
- Motor faults.

7.3.2.3. Minor Breakdown Repairs

Breakdowns of the following nature can be done by staff or large/small contractors (i.e. electricians, etc.):

- Replacement of lights and bulbs;
- Repair of light and other switches;
- Faulty control units;
- Replacement of transducers and switches; and
- Repair of wiring faults.

7.3.3. Civil

7.3.3.1. Routine Planned Maintenance

A schedule of routine maintenance will be compiled to cover all components such as:

- Five-yearly dam safety inspections of river abstraction works and other qualifying reservoirs (if delegated);
- Regular inspection and repair of pipelines and chambers including fencing, gates, access roads, road crossings, etc.;
- Regular painting of valves and pipes in chambers;
- Inspection and repair of pipe linings at intervals (say 5 years);
- Inspection and repair of all reservoir embankments and structural and other concrete elements of all the principal components mentioned above. This will include checking for leaks and leakage rates from all reservoirs;
- Inspection and repair of erosion and flood damage caused at any of the principal components;
- Keeping the pipeline servitudes free of shrubs and trees;
- Painting of buildings; and
- Maintenance of building services.

7.3.3.2. Major Breakdown Repairs

These repairs will include aspects such as:

- Repair of leaks in reservoir linings;
- Structural repairs to the abstraction structures; and
- Repair of major erosion damage.

7.3.3.3. Minor Breakdown Repairs

These repairs will include aspects such repairs to buildings and structures (i.e. safety handrails, doors, roofs, windows, etc.).

7.4. Conclusions

The control and operation of all sites forming part of the MCWAP will be monitored and managed by means of a SCADA system from a central control room manned on a 24 hour/day basis. The monitoring system must provide adequate planning, operational and costing reports to effectively manage operate and maintain the system.

In addition, the maintenance philosophy must address mechanical, electrical and civil engineering aspects, categorised as follows:

- Routine planned maintenance;
- Major breakdown repairs; and
- Minor breakdown repairs.

8. ENVIRONMENTAL ASPECTS

8.1. Background

The development of new power stations is of high strategic importance and the construction of the first new power station, Medupi, is already underway. The first units will be commissioned by the end of 2010. The proposed Mokolo Dam Scheme will be implemented as a first phase because of the availability of water in the Mokolo Dam and the shorter time required commissioning it while the CRW Transfer Scheme is under construction.

The required Environmental Impact Assessments (EIAs) and obtaining of an Environmental Authorisation for both phases are currently underway as part of the Environmental Module of the MCWAP Feasibility Study. The focus of this process is similar to the implementation planning for the two phases of the project with Phase 1 receiving priority attention.

The Environmental Feasibility of the Phase 1 and Phase 2 routes was evaluated as part of the Pre-Feasibility investigation as an initial screening (Refer to Report No 7 – Technical Module: Social and Environmental Screening).

A screening process was conducted to evaluate each route option and identify potential fatal flaws that may eliminate a specific route or position of an infrastructure component. Both the biophysical and social environments were assessed and reported on. The above report also comments on the likely costs associated with mitigating the impacts.

8.2. Listed Activities

Activities identified in terms of Section 24(2) (a) and (d) of the National Environmental Management Act 1998 (Act 107 of 1998) (the Act) may not commence without environmental authorisation from the competent authority. These activities require the investigation, assessment and communication of potential impact of activities according to the procedure as described in Regulations 22 to 26 of the Environmental Impact Assessment Regulations, 2006, promulgated in terms of Section 24(5) of the Act, are listed below.

The construction of major civil Works and pipelines are listed activities in terms of the Act. The following listed activities are included under Regulation 386 indicating a basic assessment:

> 1(k) The bulk transportation of sewage and water, including storm water, in pipelines with -
>
> (i) an internal diameter of 0.36 metres or more; or
>
> (ii) a peak flow of 120 litres per second or more

4. The dredging, excavation, infilling, removal or moving of soil, sand or rock

exceeding 5 cubic metres in volume from a river, tidal lagoon, tidal river, lake, in-stream dam, floodplain or wetland.

Although indicated as a Basic Assessment, it is anticipated that several detailed specialist investigations such as fauna, flora and heritage assessments will have to be completed. The timing of the MCWAP is therefore significant as some of the studies can only be conducted during certain periods of the year. Due to the extent of the MCWAP, a full EIA must also be conducted.

The duration of a full Environmental Impact Assessment process can take anything from 18 to 24 months.

8.3. Potential Environmental Impacts of the Crocodile River (West) Transfer Scheme (Phase 2A)

The construction of a pipeline could have numerous environmental impacts, including the following:

- Destruction of vegetation;
- Faunal habitat loss;
- Soil erosion;
- Hydrocarbon pollution of soil, ground and surface water;
- Air pollution (dust during blasting and drilling); and
- Noise pollution.

In the southern regions, the proposed pipeline is located relatively close to some sensitive rocky outcrops in certain areas and particular care is required to minimise the disturbance of these areas. For the most part, the central route runs along the railway line from Thabazimbi to Lephalale and then along an existing gravel road to Steenbokpan. The railway line has a maintenance road adjacent to it.

The railway line and road have resulted in an existing linear impact along most of the proposed pipeline route. The vegetation types along the route consist mainly of Western Sandy Bushveld and Limpopo Sweet Bushveld. Both these vegetation types are listed as Least Threatened. Due to the pipeline being aligned parallel and near to the railway line, roads, farm boundaries and other existing linear infrastructure, it limits the impact on farm areas and it should not lead to significant further fragmentation.

The alignment of the central route crosses only one major hydrological feature (Matlabas River). The crossing of the river by the pipeline should preferably coincide with the crossing of the railway line. The area has already been disturbed and the impact of the river crossing for the pipeline should not be significant.

The Abstraction Works, Balancing Dam, High-lift Pump Station and switchyards are located on cultivated land in the Mooivallei area. This portion of the works will have a higher and more permanent impact on the area and would have to be mitigated. Similarly, the impact associated with the Break Pressure and Operational Reservoirs will be higher and more permanent and would also have to be mitigated. The construction of the Terminal Reservoirs is part of the end users responsibility and as such must be incorporated into EIA process of the developments that will be supplied from the Terminal Reservoirs.

Many of the potential impacts associated with the pipeline and barrow pits and quarries can be negated or minimised through proper construction management and diligent communication and consultation with affected land owners.

8.4. Conclusion

The pipeline does traverse some sensitive areas where particular care should be taken. These will be pinpointed during a detailed environmental investigation. Rocky areas are most sensitive due to the presence of aloe species, as well as the distinct habitat it provides for certain animal species. The construction of the river Abstraction Works, Balancing Dam and High-lift Pump Station at Vlieëpoort, as well as the Break Pressure and Operational Reservoirs will have an impact that must be mitigated. To minimise this impact, the site for these works must be identified in conjunction with faunal and floral specialists.

The location of the pipeline adjacent to existing linear infrastructure, together with adequate mitigation measures, will ensure that the construction of the pipeline will have a minimal lasting effect on the surrounding areas.

The detailed investigations envisaged for the design stage will be the responsibility of the consultant appointed for the EIA. The Pre-feasibility and Feasibility stages only consisted of a desktop investigation and a brief site visit to identify major fatal flaws, if any should exist. During the Design Phase detailed faunal and floral investigations will have to be conducted to identify specific plant communities that are sensitive, as well as sensitive habitats that will be affected by the scheme. The investigation also needs to indicate how well such communities are represented in the vicinity and elsewhere.

9. SOCIAL ASPECTS

The permanent servitude width for the CRW Transfer Scheme pipelines is 50 m in addition to the land required for the works at Vlieëpoort and the Operational and Break Pressure Reservoirs. The most significant socio-economic impacts of the proposed CRW Transfer Scheme are discussed below. The Public Participation process performed for the Pre-feasibility and Feasibility stages of the MCWAP was facilitated by the appointed EIA Consultants. The Technical Team took part in many of the public meetings to disseminate information about the MCWAP and familiarise themselves with the public perception of and attitude towards the project. The cadastral boundaries, farm ownership and perceived social opposition towards the project are illustrated on drawings W9528 LD/CTS/004 – 007 included in **Appendix C**.

The most significant socio-economic impacts of the proposed pipelines are:

- Negative impacts:
  - Loss of agricultural Land;
  - Foreign work force and inflow and outflow of workers;
  - Workers' camps and effect on communities in the vicinity;
  - Possible disruption of daily living;
  - Safety and security;
  - Impact on property values; and
  - Aesthetic impacts.

- Positive impacts:
  - Increased government income and stimulation of local economy;
  - Employment and decrease in local unemployment levels;
  - An increase in new businesses and in sales;
  - Increased standards of living; and
  - Transfers of skills.

Management procedures need to be put in place and implemented so that the negative impacts can be reduced and the positive impacts enhanced. The construction site should be isolated by the erection of temporary fencing in order to avoid stock and game losses. Fencing used during construction should still enable farmers to have access to their land and dwellings. Game fencing taken down during construction must be re-instated to the approval of the individual property owners. A workers' camp needs to be planned for well in advance to ensure that various negative social impacts are curbed.

9.1. Loss of Agricultural Land

The pipeline servitude will mainly run alongside various existing fences, roads, power lines and the railway line. Along such sections of the route, the socio-economic impacts of the pipeline route are expected to be minimal.

Most of the land that will be affected by the pipeline servitude is currently natural pasture (uncultivated land with bushes and shrubs).

During construction, the owners of the affected farms will experience a loss of either cultivated or pastural land. The permanent servitude will, however, not be fenced and the owners will be able to regain use of the land after construction. The inconvenience to the farmer will therefore mostly only be during the construction phase.

Farming activities and arable land might be negatively affected, especially during construction, for an area larger than the servitude width due to vehicle movements, dust, vibrations, etc. Provision must be made for fencing to be put up during the construction of the pipeline, with provision for to gain access to all areas of the farms.

During the operation phase, the land preparation and construction of the pipeline would have removed vegetation, causing increased surface run-off, erosion, etc. The necessary management practices and procedures need to be put in place and implemented so that the negative impacts can be minimised.

9.2. Foreign Work Force and Influx and Efflux of Workers

Local socio-economic impacts of large-scale development projects tend to be closely associated with the location (immigration) of temporary project workers and their families to communities near the project site.

The influx of people could be brought about by a number of factors. Through its positive economic impacts, the construction phase can attract unemployed persons in search of work (both directly and indirectly related to the construction). Squatter camps can develop and will have a number of environmental impacts, which include adverse health effects resulting from lack of sanitation facilities, domestic waste disposal facilities, poor ventilation, sexually transmitted diseases and a potential increase in crime.

The presence of a workforce from outside the project area could lead to conflict between them and the local residents due to factors such as differences in culture and values, competition for employment opportunities and a perception among local residents that services are being provided for outsiders while their own needs are not addressed, etc.

9.3. Workers' Camps and Effects on Communities in Proximity

It is proposed that workers' camps be provided for the construction of the CRW Scheme, since not all the workers will be locally recruited. Failure to provide for workers' camps may cause influx of squatters in search of work. This could result in the contractors not being able to adequately manage the workers which lead to negative impacts.

While relatively close proximity to the construction sites is attractive when determining suitable sites for the workers' camps, factors such as the availability of space for temporary housing, camps and adequate public and commercial services must also be considered.

The workers' camps should preferably be within close proximity to existing towns/settlements (in this case Steenbokpan/Thabazimbi) and within the vicinity of the pipeline route and sites of other related works.

The exact locations for the workers' camps need to be determined beforehand in consultation with Lephalale and Thabazimbi Municipalities. The positive impacts of availing land for workers' camps outweighs the negative factors as highlighted previously and this should be clearly indicated to the applicable Municipality.

The decision to allow employees to live in accommodation separate from the workers' camp will contribute towards curtailing an increase in the incidence in Sexually Transmitted Diseases (STDs), including HIV/AIDS. Furthermore, the accommodation of staff within single-sex workers' camps or as residents within neighbouring villages – in the absence of the family members – has the potential to result in an increased incidence of STDs.

Early efforts to provide workers' camps and support services during the Pre-construction stage should be initiated. The workers' camps need to conform to public health and safety regulations.

Other less tangible impacts that may occur in the areas as a result of the workers' camps include: reduction in social stability, loss of social support structure, decrease in safety and security, community conflicts and loss of sense of community.

9.4. Possible Disruption of Daily Living

Changes in the routine living, activities, movement patterns and infrastructure (to a lesser degree) of residents in the affected areas will be brought about by the alteration to the visual environment, noise, transportation route changes, etc. These impacts will be most significant during the Construction phase.

Numerous temporary gravel roads in the area will also affect the flow of pedestrians and vehicular traffic along routes in the area and cause more dust. Furthermore, construction vehicles on these roads will increase air pollution.

Some structures may be affected by the infrastructure to be provided as part of the scheme and some farmers may lose major portions of their farms in the Mooivallei area. Apart from the impact associated with the works at Vlieëpoort and at the Break Pressure and Operational Reservoir sites, the overall social impact of the pipeline is considered to be very modest.

Although temporary, the Construction stage will be responsible for the greatest amount of disruption caused during the entire implementation, operation and maintenance stages.

During operation, regular inspections will be undertaken and a certain amount of maintenance will need to be carried out periodically. This will include repair to the pipeline. Access will need to be granted to operating, inspection and maintenance teams to all components of the scheme where the servitude is not located close to the road, railway line or power line servitude, which may inconvenience farmers. Where pipeline problems occur below ground, excavation may need to be done in order to assess the problem. This will lead to further temporary disruption to address the problem. It is expected that pipeline sections alongside existing servitudes (roads, railway and power lines) will be relatively easy to access and therefore, to inspect and maintain.

9.5. Safety and Security

During the Construction phase and to a lesser degree during the Operation phase, safety and security problems are foreseen due to people having to gain access to private land. Individuals could sustain permanent physical harm during the construction period from accidents, noise, dust and stress sometimes causing long-term psychological problems. Since few dwellings are located near the pipeline route, most safety and security impacts for the construction workers could be more significant.

9.6. Impact on Property Values

The prices of farms in the impacted areas may be affected. The uncertainty of property owners and potential new property owners could have negative impacts on the value of land and surrounding farms affected by the pipeline although the potential of obtaining private off-takes from the scheme for small water supplies will be beneficial.

The farmers need to get compensated for any loss of value of land in the vicinity of the area on which Servitude is to be registered.

9.7. Aesthetic Impacts

Aesthetic impacts on the surrounding landscape will be most notable during the Construction phase, which is temporary. Factors such as the width of the servitude and size of the buried pipeline have a temporary influence on the visual quality of the landscape during the construction phase.

Other visual intrusions during the Construction phase include:

- Fencing erected in and around the construction servitude area;
- Workers' camps at the proposed locations;
- Prefabricated offices and vehicle storage sites at the Abstraction Works, Break Pressure and Operation Reservoir site and along the route; and
- The 50-metre wide servitude along the length of the pipeline.

Since the pipeline is below ground, except for the valve and access chambers, the visual impact is far less marked, than had it been situated above ground. During operation, the only visual impact will be gravel access roads and valve chambers along the pipeline. The visual impact during operation is thus relatively small.

The permanent infrastructure at the Abstraction Works, Break Pressure and Operational Reservoir sites is located above ground and covers vast areas. These facilities will result in a permanent aesthetic impact that will be difficult to mitigate.

9.8. Employment and Decrease in Local Unemployment Levels

The MCWAP will provide sufficient water to allow the mines and industries to grow at the pace desired, thus bringing an increase in employment and decreasing the local unemployment level.

9.9. Increase in New Business Sales

The expected increased employment will benefit the regional and local economy. Increased employment is associated with increased income and consequently with increased buying power in the area, thus leading to new business sales to accommodate the new demand for services and goods. The population will spend more money which will be injected into the economy.

9.10. Increased Government Income and Stimulation of Local Economy

The potential positive economic benefits such as increased financial spending, increased infrastructure investment, increased expenditure by employees, etc. are likely to result in increased markets for the sale of local goods for the new and future employment (permanent and temporary) that will be created by the scheme and the direct future employment by the mines and industries, such as Eskom and Sasol.

The supply of water can thus be seen as an economic injection to the area that would also lead to increased Government income, through an increased tax base, and increase the capacity of the Local Municipality to increase and/or improve social and service support actions and local spending.

9.11. Increased Standards of Living

The multiplier or spin-off effects from this economic activity will improve standards of living, decrease dependence on pensions, increase disposable income and ability to purchase additional goods and/or establish other business enterprises. Apart from having the potential to create occupational opportunities, the proposed development could also stimulate economic growth in the region by attracting other commercial activities. If this is the case, indirect local benefits may accrue in the form of job opportunities in other sectors and industries. The proposed development may also serve as a catalyst for the improvement of services and infrastructure in the longer term. A stimulation of the economy is also expected in the transport sector, as more public transport will have to be made available for workers and their families. An increase in trade, which includes retailers, wholesalers, restaurants and accommodation establishments, is also expected when large numbers of people enter an area.

9.12. Transfer of Skills

With an increase in employment, a definite improvement and transfer of skills will result. Skills development is a pre-requisite for human resource development, and will have a lasting impact on the economy.

9.13. Compensation Costs

The compensation cost for the land to be acquired is based on a 50 m wide permanent servitude for the pipeline and the physical area required at the Abstraction Works and Reservoir Sites. Apart from the physical land value, the additional cost associated with implied losses as a result of the impact of construction on game farming and hunting, accommodation and eco-tourism will have to be considered. To account for this, a land value for purposes of estimating the scheme cost was based on a conservatively high estimate of R24 000/ha.

10. COST ESTIMATE AND ENGINEERING ECONOMIC ANALYSIS

Capital cost estimates were undertaken using the cost models presented in Report No. 3 – Technical Module: Guidelines for Preliminary Sizing, Costing and Economic Evaluation of Development Options. The cost models that were generated are included in **Appendix F**. Unit rates were based on April 2008 as a base date for prices.

Quantities were calculated, using the Feasibility stage drawings and preliminary designs, as described earlier in the report.

Summaries of the estimated capital costs and operation maintenance costs associated with each of the components that were studied are included in Sections 10.2 and 10.3, respectively. In order to obtain a cost for the total project, the costs associated with the Phase 1 infrastructure is also summarised as obtained for Report No. 11 – Technical Module: Feasibility Stage Phase 1.

10.1. Input Values and Assumptions used in the Engineering Economic Evaluation

The approach to sizing and costing engineering infrastructure components is described in Report No 3 – Technical Module: Guidelines for Preliminary Costing and Economic Evaluation of Development Options.

10.1.1. Energy Costs

- Eskom MegaFlex Tariff structure;
- Allowance for peak and off-peak electricity (active and reactive energy) tariffs and further differentiation between summer and winter tariffs;
- Annual pumping hours calculated based on the annual average water requirements from the water requirement tables; and
- Fixed network, admin, etc. charges included.

10.1.2. Economic Parameters

- Discount rates of 6, 8 and 10%;
- Useful life (or discounting period) taken as 45 years; and
- Electricity cost escalated by 20% per annum (compounded) over the first five years.

10.1.3. Operation and Maintenance Costs

- 0.5% of pipeline capital cost per annum;
- 4% of the electrical and mechanical installation of a pump station per annum;
- 15% of the initial capital cost of pump and motors every 15 years for major overhaul;
- 0.25% of the capital cost of civil structures, including the civil portion of pump stations per annum; and

- Apart from the capital investment every 15 years on mechanical components, the costs of replacement infrastructure, land acquisition, design and supervision fees were excluded from the calculated O&M costs.

10.1.4. Net Water Requirements and Raw Water Costs

The net water requirement is based on the total annual water requirement (taking cognisance of installed transfer capacity) and has been used to determine the URV. Transfer, evaporation and leakage losses from the terminal reservoirs are estimated at 2%. These, and river losses estimated, as described under Section 3.3.1, are added to obtain the gross water requirement used to calculate the raw water costs for the combined project (Mokolo and Crocodile River (West) Schemes).

The raw water cost was based on a rate of R2.00/m^3 for the Mokolo Scheme. The CRW Transfer Scheme has a current allocation of 45 Million m^3/a at a cost of R2.00/m^3. Raw water requirements in excess of the 45 Million m^3/a were priced at R6.76/m^3 to compensate for the additional cost associated with transferring water from the Klip River in the Vaal River catchment in Gauteng to the CRW catchment. Raw water volumes were based on the gross water requirements, including the losses in the CRW Scheme.

10.2. Capital Costs

The total capital cost for the MCWAP is summarised in **Table 10-1**. The capital cost estimate includes the costs of Phases 1 and 2A. The cost includes infrastructure and rehabilitation costs, P&G items, contingencies and design fees and excludes VAT. The base date of prices for the cost estimate is April 2008.

Table 10-1: MCWAP Project Capital Cost

Component		Total (R)
Mokolo Dam Scheme – Phase 1 (1)		
1.1	Pump Station (Maximum duty 1.5 m^3/s @ 263 m)	
	- Civil Works	64 805 000
	- Mechanical & Electrical Work	70 770 000
1.2	Rising Main	
	- 900 mm diameter (5 700 m)	86 540 000
1.3	Gravity Mains(2)	
	- 1 900 mm diameter (1 400 m)	51 243 000
	- 1 100 mm diameter (42 950 m)	692 875 000
	- 1 000 mm diameter (19 970 m)	308 070 000
	- 900 mm diameter (11 770 m)	152 910 000
	- 800 mm diameter (1 940 m)	27 544 000
1.4	Eskom Electricity to Site	76 430 000
1.5	Compensation	2 170 000
1.6	Environmental and Socio-economic	1 000 000

Component		Total (R)
	Sub Total	**1 534 357 000**
Crocodile River (West) Transfer Scheme - Phase 2		
2.1	Abstraction Weir and Low-Lift Pump Station Civil Works(3)	247 983 890
2.2	Low-Lift Pump Station M&E Works (4)	74 073 110
2.3	Rising Main to Desilting Works	171 573 000
2.4	Desilting Works(3)	86 148 000
2.5	High-Lift Pump Station Balancing Dam	318 909 000
2.6	High-lift pump station (Maximum duty 6.6 m^3/s @ 255 m)	350 544 000
2.7	Rising Main(5) - 1 900 mm diameter (26 700 m)	1 263 545 000
2.8	Gravity Mains (not constructed under Phase 1)(6) - 2 200 mm diameter (62 700 m)	3 464 072 000
	- 2 300 mm diameter (28 200 m) - 800 mm diameter (1 940 m)	1 440 550 000 28 110 000
2.9	Operational and Break Pressure Reservoir(3)	118 964 000
2.10	Eskom electricity to Vlieëpoort site(7)	156 564 000
	Sub Total	**7 721 036 000**
	TOTAL COMBINED CAPITAL COST – MCWAP (Phases 1 and 2A)	**9 255 393 000**

Notes:

1. The residual value of the existing pump station at Mokolo Dam, as well as the existing pipeline between Mokolo Dam and Matimba was calculated as R8 million and R33 million, respectively. These costs were only added to the project capital cost in the engineering economic analysis.
2. Includes the Lephalale-Steenbokpan link sized for the ultimate scheme requirements.
3. The costs of pipe work, valves, screens and craneage have been included in the civil works portions of the cost estimate.
4. Only includes for the costs of the pumps and any M&E control equipment required as well as any pipe work and valve items directly associated with the pump installations.
5. Rising main from High-Lift Pump Station to the Operational and BPR.
6. Includes the gravity pipeline sections from the Operational and Break Pressure Reservoir to the CRW Connection near Steenbokpan, as well as the connection from the Steenbokpan tee-off to the Matimba control chamber required to prevent mixing CRW and Mokolo Dam water. The remainder of the Lephalale-Steenbokpan link will be built under Phase 1 and operated in reverse flow mode.
7. Includes for the bulk electrical supply to High-Lift Pump Station and the Low-Lift Pump station.

10.3. Operation and Maintenance Costs

Table 10-2 summarises the annual operation and maintenance costs at April 2008 prices, when the scheme is operating at maximum capacity (2030), excluding overhaul costs of pump stations and VAT.

Table 10-2: MCWAP Annual O&M Costs

	Component	Total (R)/a
Mokolo Dam Scheme – Phase 1		
New Phase 1 Works		
1.1	Pump Station	
	- Civil Works	141 000
	- Mechanical & Electrical	2 462 000
	- Electricity	14 131 000
1.2	Rising Main	376 000
1.3	Gravity Mains	5 359 000
	Existing Exxaro Works	
2.1	- Civil	6 000
	- Mechanical & Electrical	223 000
2.2	Pipeline	165 000
3.1	Raw Water Costs [(1)]	58 571 000
	Sub Total	**81 434 000**
Crocodile River (West) Scheme - Phase 2		
4.1	Abstraction Weir, Low-Lift Pump Station, De-silting Works and Balancing Dam	
	- Civil	995 000
	- Mechanical & Electrical	2 035 000
	- Electricity	17 336 000
4.2	High-lift pump station (Maximum duty 6.6 m^3/s @ 255 m)	
	- Civil	87 000
	- Mechanical & Electrical	4 797 000
	- Electricity	76 866 000
4.3	Rising Main	3 018 000

	Component	Total (R)/a
4.4	Gravity Mains (not constructed under Phase 1)	
	- 2 200 mm diameter (62 700 m)	8 304 000
	- 2 300 mm diameter (28 200 m)	3 478 000
	- 800 mm diameter (1940 m)	66 000
4.5	Operational and Break pressure Reservoirs	308 000
4.6	Raw water costs	1 142 408 000
	Sub Total	**1 317 098 000**
5	**Annual River Management Cost**	4 500 000
	TOTAL COMBINED ANNUAL O&M COST (2030) – MCWAP	**1 345 632 000**

(1) Raw water priced at R4,50/m^3

10.4. Present Value

The discounted present values for the total MCWAP are summarised in **Table 10-3** below. These figures, excluding VAT, are based on April 2008 prices and are discounted to 2008.

All discounting was done to 2008 and over a period of 45 years after completion of construction of Phase 2A. Residual values at the end of the period were excluded from the analyses.

Table 10-3: Summary of Discounted Present Value

Discount Rate	Capital (R)	O&M (R)	Total (R)
6%	7 728 200 000	12 733 903 000	20 462 103 000
8%	7 268 346 000	8 682 042 000	15 950 388 000
10%	6 847 206 000	6 181 959 000	13 029 165 000

10.5. Unit Reference Values

The unit reference value (URV) of water is not the tariff for the water, but the value attached to the net water requirement supplied to the consumers so that the discounted present value of the water is equal to the discounted present value of the cost.

The URV of water has been determined for a discount rate of 6%, 8% and 10% and is based on the net water transferred to the demand centres for a 45-year period. The URVs for the MCWAP are summarised in **Table 10-4**. These figures, excluding VAT, are based on April 2008 prices. All discounting was done to 2008 and over a period of 45 years after completion of construction of Phase 2A. Residual values at the end of the period were excluded from the analyses.

Table 10-4: Unit Reference Values

Discount Rate	Discounted Present Value of Net Water @ R1/m^3 (R)	Discounted Present Value (R)	Unit Reference Value (R/m^3)
6%	2 020 000 000	20 462 103 000	10.14
8%	1 410 000 000	15 950 388 000	11.35
10%	1 020 000 000	13 029 165 000	12.72

Refer to **Appendix G** for the economic analysis as well as the cost models of each component.

11. IMPLEMENTATION PROGRAMME

The prospective beneficiaries of the MCWAP were requested to provide key dates of their water requirement timeframes. A detailed project programme for the project was compiled taking into account these key dates. The project implementation is, however, taking place within a very dynamic planning environment and the project programme had to be revised on numerous occasions. Revisions to the key project dates up to 30 June 2009 are summarised below.

Table 11-1: Project Key Dates

Item No.	DESCRIPTION	Original Programme dated April 2008	Revised Programme dated June 2009	Revised Programme dated July 2009
1.	Topographical Survey		14 Aug 2009	28 Sep 2009
2.	Detail Geotechnical Investigations P1		28 Aug 2009	14 Aug 2009
3.	Detail Geotechnical Investigations P2A		15 Jan 2010	7 Jun 2010
4.	Environmental Module		20 Jul 2010	13 Sep 2010
5.	User Water Supply Agreements P1		09 Dec 2009	09 Dec 2009
6.	User Supply Agreements P2A			12 June 2011
7.	Procure Engineering Services		08 Jun 2009	31 July 2009
7.	Land Acquisition Phase 1		09 Jun 2010	6 Dec 2010
8.	Land Acquisition Phase 2A		12 Oct 2010	28 Jun 2011
9.	Award Contracts Phase 1		08 Oct 2010	6 Dec 2010
10.	Award Contracts Phase 2A		30 May 2011	9 Aug 2011
11.	Water Delivery Phase 1	Nov 2011	12 April 2012	3 Dec 2012
12.	Water Delivery Phase 2A	Jun 2014	16 Dec 2014	12 Aug 2015

The original target date for delivery of water to Medupi was September 2010 and for delivery to Steenbokpan, November 2011. The target date for commissioning of the Phase 2A infrastructure was originally June 2014.

As shown above, the key dates derived from the revised program dated 30 June 2009, indicate that the delivery date for water from Phase 1 has shifted to December 2012 and for Phase 2A to August 2015.

The engineering economic analysis discussed under Section 10 was based on the earlier implementation planning for the project of December 2011 for Phase 1 and December 2014 for Phase 2A.

The actual project implementation will be dictated by the finalisation of the User Supply Agreements which are expected to remain dynamic well into the detail design phase.

12. PHASE 2A ALTERNATIVE ROUTE ALIGNMENTS

After completion of the Pre-Feasibility stage, the MCWAP Technical Module PSP was invited to participate in the public participation process with affected land owners that were arranged by MCWAP Environmental Module PSP. Consultation also took place with bulk water consumers like Sasol, Eskom and Exxaro. The purpose of the discussions was to assimilate more information about the planned future developments that could affect the positioning of the pipelines to be constructed during Phases 1 and 2A. Due to the dynamic nature of the planning currently taking place, the positioning of infrastructure components will also be a dynamic process that will require close coordination during the detail design phase to ensure that other planning processes are considered in the final positioning of the pipelines and structures.

During the part of the consultation process that involved the land owners, further information came to hand that could affect the routing of the pipeline between Vlieëpoort and Steenbokpan. There is strong public opposition towards the CRW Transfer Scheme in the areas north-west of Vlieëpoort. Eskom also provided revised details of planned future power station development around Steenbokpan, which would affect the routes of the pipelines.

A number of alternative routes were therefore identified during this process that could potentially limit the impacts associated with the CRW Transfer Scheme in the south and bypass planned future infrastructure in the north. Due to their late addition, these routes were, however, not investigated in sufficient detail during the Feasibility stage to confirm their viability. It is therefore recommended that the process be taken forward as part of the detail design when more detailed information will also be available on the planned developments in the north. It must be noted that changes to the preferred route should be coordinated with the Environmental Consultant as a matter of urgency to ensure that any changes can be incorporated into the final EIA.

The alternative routes are illustrated on a layout map (DWG W9528/LD/CTS/002) included in **Appendix A** and summarised in the following table, together with a brief motivation for selecting the route and an indication if further investigation is required to confirm its feasibility.

Table 12-1: Phase 2 Route Alternatives

Alternative Route	Description	Motivation/Implication	Further Investigation Required (Y/N)
Phase 2 Alt A&B	Local deviations around the boundary of the Farm Paarl 124 KQ.	Strong opposition against the project from affected land owners. The deviation will reduce the impact of the project on Farm Paarl 124 KQ. Deviation will result in an impractical route alignment (additional length and sharp bends) and a significant extra cost to the project.	N – Negotiated settlement to be pursued
Phase 2 Alt C	From Vlieëpoort east towards Thabazimbi passing through a neck in the mountains and the Regologile informal settlement before heading north along the R510 and linking up with the original proposed route near PI48.	Route will avoid the area to the west of Vlieëpoort where local land owners are heavily opposed to the project and the proposed route through the area. Will require higher pumping head from Vlieëpoort, but will enable a reduction in the gravity pipe diameter from the BPR to the Operation Reservoir. Surge alleviation will be required due to problematic ground profile.	Y – proper Feasibility assessment required to compare Alt C to feasibility stage recommended route
Phase 2 Alt D	Continue along the R510 for a longer distance before turning north along the railway.	The farms Tarantaalpan and Diepkuil might become part of a planned conservation area. Land owners oppose development in the area. Route along the R510 will improve access to the pipeline.	Y – final decision will depend on negotiations with land owners and the National Roads Agency
Phase 2 Alt E	Earlier deviation to the east to join the railway alignment further south	Only an option if Alt C is viable. Will avoid a rocky outcrop on the Farm Leeuwbosch 129 KQ by deviating away from the R510 further to the south.	Y – subject to viability of Alt C

Alternative Route	Description	Motivation/Implication	Further Investigation Required (Y/N)
Phase 2 Alt F	Continue north along the Railway line from the Operational Reservoir and link to the Lephalale-Steenbokpan pipeline further to the east.	The route of the pipeline in the north will be affected by the position of the demand Centre along the Lephalale-Steenbokpan corridor. The position of the demand Centre will in turn be determined by the positioning of Eskom, Sasol, IPP and mining infrastructure in this area which is currently in varying stages of planning. The final positioning of the pipeline will require extensive further coordination with the future users to ensure that an integrated approach to locating the infrastructure is followed.	Y – will be dependent on the future infrastructure planning of Eskom, Sasol, IPP's and the mines along the Lephalale – Steenbokpan corridor
Phase 2 Alt G	Continue north from the Operational Reservoir along farm boundaries along a route that will avoid planned future Eskom power stations and ash-dumps.		

13. CONCLUSIONS AND RECOMMENDATIONS

13.1. Introduction

The primary purpose of the Feasibility Study for the MCWAP is to develop the options to transfer water from the Mokolo Dam and Crocodile River (West) to the Lephalale area to supply the primary and industrial users in this fast developing area.

Various options have been identified to convey water to the end users. These include the Mokolo Dam Scheme, as well as the CRW Transfer Scheme to be operated in combination as the MCWAP. The Mokolo Scheme is intended to supply the interim water requirements for a period until the CRW Transfer Scheme has been constructed and then continue to supply a reduced quantity of water and also to provide some redundancy once the CRW Transfer Scheme is operational.

The combined MCWAP is illustrated by **Figure 13-1**, showing the terminology adopted for different components, forming part of the combined project. A locality plan is included in **Appendix A** of the report (DWG No WP 9528/LD/CTS/001).

The project will be implemented in phases as follows:

- Phase 1 - Mokolo Dam Scheme: Augmentation of the Mokolo Dam system to supply in the growing water requirement for the interim period until a transfer pipeline from the CRW can be implemented. Thereafter the scheme must continue to supply water to optimally utilise the full yield of Mokolo Dam.
- Phase 2A - Crocodile River (West) Transfer Scheme: Transfer system from the CRW to the Lephalale and Steenbokpan area where the pipeline will link to the infrastructure constructed as part of Phase 1 to provide the balance of the water requirements that cannot be supplied by Phase1.

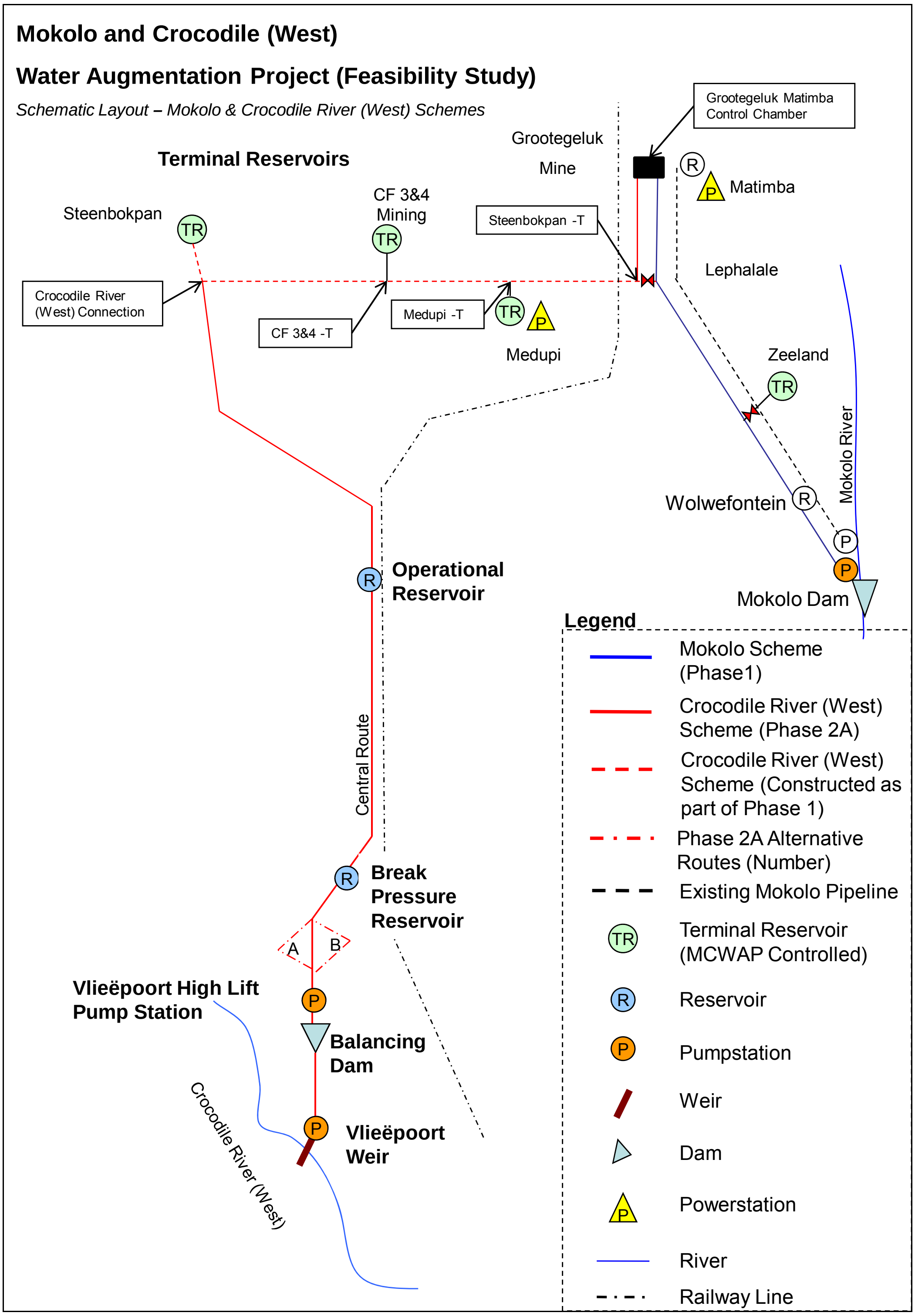


Figure 13-1: Mokolo and Crocodile (West) Water Augmentation Project

13.2. Water Requirements and Design Flow

The water requirements for the Feasibility Investigation were based on the Scenario 9 projections for infrastructure projects and associated domestic water requirements up to 2030. Reliability and redundancy criteria were applied to arrive at the recommended design capacity of the respective scheme components.

Details on the application of the various allowances and factors to arrive at the three design case combinations that were considered are summarised in **Table 13-1**.

Table 13-1: Allowances and Factors used in Design Scenarios

Item No.	Allowance and Factors Applied(2)	Basic Design Case	Peak Design Case	Recovery Peak Design Case
1.	95% Reliability factor (4).	5%	0%	0%
2.	Allowance for water requirement peaks (average annual allowance) (1), (4).	0%	9%	0%
3.	Allowance for 90-day Recovery Period after maximum 18-day system outage (3).	0%	0%	20%
4.	System Losses [Phase 1 (Mokolo Dam Scheme) losses added to Phase 2A CRW Transfer Scheme losses].	2%	2%	2%
5.	Allowance for variations in river flow (5).	0%	0%	0%
6.	Recovery Peak on the CRW Transfer Scheme due to failure of Phase 1 Mokolo Dam supply (due to over-usage, etc.) (6).	Nil	Nil	Greater of 20% GAAR or 19.5 Million m^3/a

Notes:

1. Refer to Section 3.2 for details.
2. The % allowances factors were applied in the form: Flow x (1 + %/100).
3. The allowance for the 90-day Recovery Period was used independent of the other factors (apart from the system loss factor) to avoid compounding of related allowances. Pump selection and the pressure rating of pipelines, as well as gravity flow sections of the system were designed to be able to transfer the recovery peak flow case.
4. The economic optimisation of infrastructure components that are part of a pumping system was based on the design flow incorporating the reliability factor (1.05). The reliability and peak factors were not applied simultaneously.
5. It was assumed that all the water requirements would be available in the river at the abstraction works. This aspect is further addressed in Section 3.3.1.
6. Based on the greater of 20% of GAAR on the CRW Transfer Scheme or the water requirements on the Mokolo Scheme, excluding the supply to Zeeland WTW (9.2 Million m^3/a in 2025) which cannot be supplied from the CRW due to the variation in water quality.

The application of the above criteria to arrive at the design flow for the Abstraction Works, and transfer system capacities are summarised in **Tables 3-3** and **3-5**.

13.3. Infrastructure Requirements

The infrastructure components that are recommended for implementation are summarised in **Table 13-4** consisting of the following main components as part of Phase 2A:

13.3.1. Weir and Abstraction Works

The abstraction weir and de-silting works consist of the following components:

- Mass concrete gravity type Diversion Weir, 4.3 m high with ogee and roller bucket spillway.
- Gravel Trap in weir basin with flushing facility and thrash rack with concrete channels leading from gravel trap to each pump-well in the Low-lift pump station that is incorporated partly into the NOC flank of the weir and partly into the river bank. Nine (9) pump bays, each capable of accommodating two (2) fully equipped 1 m^3/s capacity submersible pumps were provided.
- Low pressure rising main to the Desilting Works.
- The rising main will consist of a 2 100 mm diameter steel pipeline approximately 5 km long. It will then be split with a manifold into nine 750 mm diameter pipes leading to the Desilting Works inlets. Each pipe will have a gate valve in a valve chamber adjacent to the Desilting Works.
- Desilting Works with flushing facility located near the Balancing Dam, but above the PMF level.
- The Desilting Works will consist of nine 120 m long channels, 2.5 m wide and depth varying from 4.0 m to 5.3 m.
- A gravity pipeline between the Desilting Works and a Balancing Dam.
- Each desilting channel overflow into a collector canal with a 2 100 mm diameter steel outlet pipe gravitating to the Balancing Dam inlets through a manifold with isolating gate valves.
- A multi-compartment Balancing Dam sized to cater for unplanned changes in river flows. The Balancing Dam will also be equipped with a silt removal flushing facility although only infrequent use is expected.
- The Balancing Dam has top dimensions of 600 m x 370 m, five compartments and a total live storage capacity of 1 300 000 m^3. The depth varies from 10.5 m at the inlet side to 13.2 m at the outlet side. A freeboard provision of 0.5 m was made.

13.3.2. Transfer Scheme

13.3.2.1 Pump Station

All the requirements relating to the suction head required for the estimated NPSH of the booster pumps, a free draining pump station with no risk of flooding, etc. have been satisfied for the selected site configuration.

The pump configuration that has been adopted comprises four identical duty sets plus one standby set comprising an in-line booster pump and a main high pressure pump without a valve between the two pumps.

Variable Speed Drive (VSD) pump sets are recommended to enable more economical continuous pumping with added flexibility and soft start features.

The calculated duty point and absorbed power requirements for the pumping station are summarised in **Table 5-17**.

The cost estimate includes for a firm electricity supply to the Vlieëpoort site for both the Low-lift and High-lift pumping stations.

13.3.2.2 Pipelines

A rising main to the BPR at Ch 26700 and gravity flow to the Operational Reservoir and further to the connection with the Lephalale-Steenbokpan pipeline were found to be the most viable and practical solution and are recommended for implementation. The optimum pipe diameter for the rising main from Vlieëpoort to the BPR was found to be 1 900 mm with the gravity pipeline from the Break Pressure Reservoir to the Operational Reservoir requiring a 2 200 mm ND and a 2 300 mm ND downstream of the Operational Reservoir to the CRW connection point to the infrastructure provided as part of Phase 1 with diameters ranging from 800 mm to 1 900 mm ND.

The pipeline route planning took cognisance of existing linear infrastructure and farm boundaries as far as possible and practical to limit the social and environmental impact. A permanent servitude width of 50 m was used for costing purposes. An exploratory geotechnical and CP investigation was performed as part of the Feasibility Investigation. No adverse conditions, that would totally prohibit the construction of the pipeline, were found to exist along the proposed routes. Further detailed coordination with services authorities and affected parties will be required to obtain the necessary way-leaves and approvals.

The following generic coating and lining systems are recommended for the MCWAP Pipeline:

Table 13-2: Recommended Coating and Lining Systems

Product/Method		**Field Joint Repair Method**
External Coating	Preferred: Trilaminate Polyethylene (3LPE) or Polyurethane Alternative: Polymer modified bitumen/Glass Fibre (Bituguard)	Liquid or powder epoxy plus cold tape wrap Polyurethane Bituguard hot applied tape
Internal Lining	Preferred: Epoxy Alternative: Cement Mortar	Epoxy Cement Mortar

The recommended pipe roughness parameters to be used during the detailed hydraulic design are summarised below.

Table 13-3: Recommended Long-Term Roughness Parameter

Parameter	**Cement Mortar Lining**		**Epoxy Lining**	
	Suggested	**Maximum**	**Suggested**	**Maximum**
Long-term absolute roughness (mm)	1.1	1.5	0.5	0.7
Influence of biofilm	Reduction in diameter of 5-8 mm			

13.3.2.3 Reservoirs

The CRW Transfer Scheme includes a BPR located on the farm Zondagskuil 130 KQ, as well as an Operational Reservoir located on the farm Zoutpan 367 LQ. These reservoirs will have a minimum total combined storage capacity of 8 hours of the recovery peak flow to provide effective balancing capacity for differences in outflow and inflow. A minimum of two compartments will be provided for normal operational and maintenance purposes.

The scheme will supply water into Terminal Reservoirs located on the sites of the end consumers. The Terminal Reservoirs must provide a minimum of 18 days storage and will be built by the respective end users, but will be operated and controlled by the MCWAP.

Table 13-4: MCWAP – Summary of Infrastructure Components

Component	Description
Mokolo Dam Scheme (Phase 1)	
Phase 1	New pumping station and additional pipeline from Mokolo Dam to end-users located from Lephalale in the east to Steenbokpan in the west. The Lephalale-Steenbokpan link will be built as part of Phase 1, but will ultimately form part of the CRW Transfer Scheme to transport CRW water to Medupi and the Grootegeluk/Matimba control chamber.
Lephalale-Steenbokpan Link CRW Connection to Steenbokpan CRW Connection to CF 3&4 Mining T-off CF 3&4 Mining to Medupi T-off Medupi T-off to Steenbokpan T-off	 Diameter : 1 900 mm ND Length : 1.4 km Diameter : 1 100 mm ND Length : 27.1 km Diameter : 900 mm ND Length : 3.6 km Diameter : 900 mm ND Length : 8.2 km
Crocodile River (West) Transfer Scheme (Phase 2A)	
Vlieëpoort Abstraction Works	Concrete weir, gravel trap and pump intake structure– civil structures sized for ultimate project water requirements (431 Million m^3/a). 1 x fully equipped standby bay plus 1 standby pump unit (stored on site). 8 x 1.0 m^3/s submersible pumps. Maximum duty point: 6.6 m^3/s @ 49.5 m. Absorbed Power: 4.7 MVA. 1 300 000 m^3 active balancing storage.
High-lift pump station	Static head : 183-192 m Design peak flow (DPF) : 5.8 m^3/s Min manometric head at DPF : 216 m Recovery peak flow (RPF) : 6.6 m^3/s Max manometric head at RPF : 255 m Power consumption DPF/RPF : 16/19 MW
Pipelines Rising main – High-lift pump station to Operational and Break Pressure Reservoir (Node 10) *Break Pressure and Operational Reservoir*	 Diameter : 1 900 mm ND Length : 26.7 km 8 hours storage of recovery peak flow rate : 190 000 m^3

Component	Description		
Operational and Break Pressure Reservoir to Node 15	Diameter Length	: :	2 200 mm ND 62.7 km
to CRW Transfer Scheme Connection (Phase 2A)	Diameter Length	: :	2 300 mm ND 28.2 km
Steenbokpan T-off to Grootegeluk/Matimba Control Chamber	Diameter Length	: :	800 mm ND 1.9 km

13.4. Operation, Maintenance and Control Philosophies

The control and operation of all sites forming part of the MCWAP will be monitored and managed by means of a SCADA system from a central control room manned on a 24 hours/day basis. The monitoring system must provide adequate planning, operational and costing reports to effectively manage, operate and maintain the system. Repairs on pipe and check valves will have to take place during planned system maintenance.

In addition, the maintenance philosophy must address mechanical, electrical and civil fields, categorised as follows:

- Routine planned maintenance;
- Major breakdown repairs; and
- Minor breakdown repairs.

The management of water releases for the MCWAP from any of the Mokolo, Klipvoor, Roodekopjes or Vaalkop Dams or management of flows in the CRW or MCWAP are not included in the Operating, Maintenance and Control Philosophies for the MCWAP.

13.5. Environmental and Social Aspects

The pipeline and the Break Pressure and Operational Reservoir(s) traverse some sensitive areas where particular care should be taken. These will be pinpointed during a detailed environmental investigation. Rocky areas are most sensitive due to the presence of aloe species, as well as the distinct habitat they provide for animal species. The construction of the River Abstraction Works, Balancing Dam and High-lift Pump Station at Vlieëpoort will have an impact that must be mitigated. To minimise this impact the site for the pump station must be identified in conjunction with faunal and floral specialists.

The location of the pipeline adjacent to existing linear infrastructure together with adequate mitigation measures will ensure that the construction of the pipeline will have a minimal lasting effect on the surrounding area.

The detailed investigations envisaged for the design stage will be the responsibility of the consultant responsible for the EIA. The Pre-feasibility and Feasibility stages only consisted of a desktop investigation and a brief site visit to identify major fatal flaws, if any should exist. During the Design phase, detailed fauna and flora investigations will therefore have to be conducted to identify specific sensitive plant communities, as well as sensitive habitats that will be affected by the scheme. The investigation also needs to indicate how well such communities are represented in the vicinity and elsewhere.

The most significant socio-economic impacts of the proposed pipelines from Vlieëpoort to Steenbokpan are:

- Negative impacts:
  - Loss of agricultural land;
  - Foreign work force and inflow and outflow of workers;
  - Workers' camps and effect on communities in vicinity;
  - Possible disruption of daily living;
  - Safety and security;
  - Impact on property values; and
  - Aesthetic impacts.

- Positive impacts:
  - Increased government income and stimulation of local economy;
  - Employment and decrease in local unemployment levels;
  - An increase in new businesses and in sales;
  - Increased standards of living; and
  - Transfers of skills.

Management procedures need to be put in place and implemented so that the negative impacts can be reduced and the positive impacts enhanced. The construction site should be isolated by the erection of temporary fencing in order to avoid stock and game losses. Fencing used during construction should still enable farmers to have access to their land and dwellings. Game fencing taken down during construction must be re-instated to the approval of the individual property owners. A workers' camp needs to be planned for well in advance to ensure that various negative social impacts are curbed.

13.6. Implementation Programme

The original target date for delivery of water to Medupi was September 2010 and for delivery to Steenbokpan, November 2011. The target date for commissioning of the Phase 2 infrastructure was originally June 2014.

The actual project implementation will be dictated by the finalisation of the User Supply Agreements which is expected to remain dynamic well into the Detail Design phase.

13.7. Cost Estimate

The cost estimates considered the following:

- Capital costs;
- Energy costs;
- Operations and maintenance costs; and
- Raw water costs.

The total capital cost for the MCWAP is summarised in **Table 13-5**. The capital cost estimate includes the costs of Phases 1 and 2. The cost includes infrastructure, Preliminary and General (P&Gs), contingencies and design fees and excludes VAT. The base date for the cost estimate is April 2008.

Table 13-5: MCWAP Capital Cost Estimate

Component	Total (R)
Mokolo Scheme (Phase 1)	1 534 357 000
River Abstraction Works (Phase 2)	898 687 000
Crocodile River (West) Transfer Scheme (Phase 2)	6 822 349 000
TOTAL COMBINED CAPITAL COST – MCWAP	**9 255 393 000**

Table 13-6 summarises the annual operation and maintenance costs, when the scheme is operating at maximum capacity (2030), excluding overhaul costs of pump stations and VAT, but including the raw water cost.

Table 13-6: MCWAP Annual Operation and Maintenance Costs

Component	Total (R)
Mokolo Scheme (Phase 1)	81 434 000
River Abstraction Works (Phase 2A)	20 366 000
Crocodile River (West) Transfer Scheme (Phase 2A)	1 239 322 000
Annual River Management Cost	4 500 000
TOTAL COMBINED ANNUAL O&M COST (2030) – MCWAP	**1 345 632 000**

13.8. Recommendation for Further Actions during the Detail Design Phase

13.8.1. Water Requirements

The Pre-Feasibility and Feasibility studies of the MCWAP took place within a very variable planning environment. As a result, further variations to the water requirements and design capacities are to be expected and must be incorporated into the detail design process. In this regard, the following needs to be performed:

- Confirm and implement the latest, approved water requirement scenario for the MCWAP.
- Re-confirm the reliability (system availability) and redundancy criteria that must be applied to size the Phases 1 and 2A infrastructure components. Aspects that might impact on these criteria include the following:
  - The conditions in the final end user agreements;
  - The final system operating philosophy; and
  - Risk assessment.
- Quantify evaporation and system transmission losses more accurately (a value of 2% was assumed at Feasibility stage).
- Perform a water balance incorporating latest water requirements, system losses, peak and recovery factors and a statistical assessment of storage requirements and system reliability (or availability).

13.8.2. River Abstraction Works

Recommendations for further work and investigations required during the Detail Design phase of the project are discussed in this section.

1. Depth of scour at Vlieëpoort during high floods. Scour potential at the weir must be modelled to confirm the depth of founding of the weir structure. The present Feasibility stage layout assumes that the proposed jet grouting foundation treatment will provide adequate founding conditions and together with the roller bucket spillway design and extensive downstream heavy riprap protection, it will protect the structure.

2. Foundation Design. Deep jet grouted foundations have been successfully used in the past to improve hydraulic structure founding conditions. Once the results of a detailed materials investigation are available, the Feasibility layouts should be reviewed and developed further.

3. Alluvial aquifer flows at Vlieëpoort. The Feasibility stage layouts show that the entire river bed section below the weir will be jet grouted, thereby effectively blocking the flow in the aquifer. Whilst this arrangement will prevent piping in the foundation, large differential loading on the weir due to lowering of the water table downstream of the weir by the Makoppa irrigators must be prevented. A continuous and adequate flow of water should therefore be maintained over the weir of through the Abstraction Works at all times. Continuous water table level monitoring should also be undertaken to ensure that the downstream water level is maintained.

4. Liquefaction potential. The nature of the underlying alluvial sands and silts at Vlieëpoort must be investigated to determine the potential for liquefaction and corresponding high loads or loss of support during a natural or induced seismic event.

5. Sizing and configuration of Desilting Channels. Feedback received on the operation of the Lebelelo Abstraction Works indicated that the very fine fraction of the suspended silt in the Olifants River in Limpopo Province, when in flood, failed to completely settle out in the desilting channels. This fraction requires longer retention times to settle out and therefore only settled in the balancing dams where it affected the on operational availability of the system and was also difficult and time-consuming to remove, principally because the balancing dams were not designed to be maintained at frequent intervals. In the case of the CRW Transfer Scheme, the problem is accentuated by the relatively large storage capacity (and longer retention times) of the Balancing Dam.

6. Location of High-lift Pump Station Balancing Dam. The Feasibility layouts identified two potential sites for the dam. The site closest to the Abstraction Weir has since been confirmed to be located on dolomite and should therefore be avoided if possible. The preferred site is some 5 km downstream of the Abstraction Works and on much more favourable founding conditions (residual Ventersdorp lava), but further planning is required to refine the layout and assess the relocation implications.

7. Sizing of the High-lift Pump Station Balancing Dam. The present approach is based on river flow management with a 3 to 4 day river flow response time from the upstream dams to Vlieëpoort. With improved control over flows in the river and shorter actual response times, it is anticipated that the required storage capacity of the Balancing Dam should reduce accordingly. A saving in storage capacity in the order of 200 000 m^3 may be achieved which will result in a cost saving and a reduction in retention time. Modular implementation together with an observational approach should be considered.

8. Hydraulic computer flood modelling of the river is recommended once the detail survey becomes available. This model will allow the computation of flood levels applicable to the base conditions and post-construction conditions and allow better assessments of the impact of the Abstraction Works on affected landowners and existing infrastructure. The final Abstraction Works design should aim to limit the resulting impacts on upstream flood levels. As mentioned in Section 4.6.5.2, the option of replacing the right flank embankment with a bridge structure may be required and the model will be essential to confirm this.

9. The hydraulic flood model will also be used to estimate flood levels downstream of the weir that will be used to position the Desilting Works, Balancing Dam, High-lift Pump Station and switchyards and might also influence the choice of the site for these components as discussed in Section 4.6.5.5.

10. Both the inlet and outlet structures of the Balancing Dam need to be further optimised and other alternatives investigated, to ensure optimal operation.

11. A prototype or CFD model of the Abstraction Weir, Gravel Trap and Low-lift Pump Station is recommended in order to optimise the placement, layout and size of these structures.

12. During flushing of the Desilting Works and to lesser extent when desilting the Balancing Dam, high volumes of sediment laden water need to be disposed of; this cannot be discharged directly into the river. A solution was presented in Sections 4.6.5.4 and 4.6.5.5, but further investigation is required to confirm environmental constraints and to identify appropriate sediment removal facilities and disposal sites.

13. Flows passing the Abstraction Weir should be measured. A flow gauging structure that can possibly be incorporated into the weir will be required to measure flows released downstream of the weir. In addition, a downstream borehole water level monitoring system is recommended to ensure that the alluvial aquifer is not harmfully affected by the Works and to assist with the management of downstream releases. Other gauging weirs on the upstream CRW tributaries, the Bierspruit and the Sand River, may also be required as part of the River Management Plan. Also refer to Report No. 10 – Technical Module: Requirements for the Sustainable Delivery of Water, in this regard.

13.8.3. Crocodile River (West) Transfer Scheme

The following issues were identified during the course of the Feasibility Stage and would require further investigation as part of the detail design:

1. Since April 2008, there have been a number of changes (most notably the variations in the water requirements) to the parameters that could influence the capacity, location and design of the MCWAP and the CRW Transfer Scheme in particular. The pipe systems can be optimised further when final design capacities, and more detailed survey and geotechnical information becomes available. It is therefore recommended that a more comprehensive evaluation and optimisation be performed during the Detail Design stage to verify the Feasibility findings before starting with the detail design of components.

2. Route planning and coordination. The following is required:
   - Detailed coordination and a commitment to the MCWAP by the bulk water consumers along the Lephalale-Steenbokpan corridor is required in order to ensure integrated planning of infrastructure and water requirements.
   - Eskom is planning to construct a number of high-voltage power lines through the region. A number of these will be located in a corridor routed from north to south that could affect the routing and design of the MCWAP pipelines.

- Agree on the permanent servitude requirements to allow for possible future expansion (parallel pipeline(s)).
- Detailed cadastral and existing services information must be obtained along the final pipeline route alignments.
- The land acquisition and servitude registration process must be facilitated and supported, taking cognisance of issues raised by interested and affected parties during the Public Participation process.
- The locations of farmer off-takes and water requirements must be confirmed.
- Further services coordination and way-leave approvals typically need to be obtained from the following parties:
  - Eskom: Capital Projects Planning, Transmission and Distribution.
  - Spoornet: Apply for permission to use railway line access road during construction and for future maintenance access to the pipeline and confirm future upgrade/electrification planning for the railway line. Apply for way-leaves at all railway line crossing sites.
  - South African National Roads Agency Limited (SANRAL): Apply for a concession to use the road reserves as temporary construction servitudes where pipelines are located parallel to or near a national road. Also apply for access points to pipeline servitudes from road reserves and for way-leaves at all road crossings.
  - Limpopo Provincial Roads Department: Apply for a concession to use the road reserves as temporary construction servitudes where pipelines are located parallel to or near a national road. Also apply for access points to pipeline servitudes from road reserves and for wayleaves at all road crossings.
  - Thabazimbi Local Municipality: Obtain future Township planning and establishments that might affect pipeline routes.
  - Lephalale Local Municipality: Obtain future Township planning and establishments that might affect pipeline routes.
  - Telkom: Confirm the location of services and apply for way-leaves to cross the services.
  - Neotel: Confirm the location of services and apply for way-leaves to cross the services.
  - Department of Minerals and Energy: Inform them of the planned pipeline route in order to update their database.
  - DWA: Confirm the need to apply for water use licenses for river and stream crossings and obtain the necessary permission if required.
  - Local farmers and land owners: Take forward the land acquisition and servitude registration process to ensure that servitudes for the pipelines and borrow areas are agreed timeously to prevent delays during construction.

3. The following detail design and optimisation actions must be performed:
   - Confirm and agree on the systems operating and control philosophy.
   - Review the pump selection philosophy with specific reference to the option of implementing VSDs and the associated implications it has on the operational control, power supply, etc.
   - A detailed hydraulic analysis to determine the optimum positioning of the air valves (type and size), as well as isolating, reflux, drainage and control valves and acceptable system operating and control procedures. Pipeline flushing and drainage will require careful consideration due to:
     - Potential poor water quality and fears of contamination; and
     - Very flat topography – management of flushing and drainage water will be problematic. Drainage time of the pipeline must be considered.
   - A detailed pipeline design (optimum diameters and wall thickness). Consider both the interim (rising main/operational and BPR/gravity mains) and ultimate (rising main directly to a new operational reservoir with the initial operational and BPR converted to a surge reservoir) scenarios and perform detailed surge analyses.
   - Optimum sizing of the Operational and Break Pressure Reservoirs to take cognizance of final operating philosophy and risk assessment. The detailed design of the Operational and Break Pressure Reservoirs must consider operational storage requirements, storage time, and water quality management to prevent 'dead zones' in the reservoirs. The initial Operational and Break Pressure Reservoirs must be configured to allow conversion to a surge tank during later phases of the development if required.
   - River and stream crossings – Matlabas River crossing will require careful consideration of geotechnical conditions at the site, environmental considerations and rehabilitation, as also at all other river and stream crossings.

4. Pipeline coatings and linings: New pipeline coating processes are becoming available on the market and must be considered.

5. Detailed AC mitigation design:
   - Cognisance of possible future infrastructure that might affect the design.
   - A detailed soil resistivity survey at 100 to 500 m centres, depending on soil conditions.
   - Soil sampling and analysis to confirm the aggressiveness of the soil and the possible presence of SRB that could affect the coating selection.
   - Detailed AC modelling to confirm the extent of AC mitigation required.

6. Detailed Geotechnical Investigation:
    - Geological mapping - Delineation and description of outcrop areas, including discontinuity survey, geological structures, etc.
    - Test pitting with an excavator at selected spots at an average of about 200 m centres – The maximum depth of the proposed pipeline is generally more than 4 m, deeper than the reach of a TLB. The soil profile must be described according to the standard method of Jennings *et al* with reference to shallow water table conditions, excavatibility, etc.
    - Core drilling – to investigate pipe jacking and reservoir sites.
    - In-situ testing - For the determination of soil parameters for pipeline design (the empirical E' value (bulk modulus of horizontal soil reaction), limited plate load tests must be conducted at selected representative positions.
    - Sampling and Laboratory testing – Disturbed and undisturbed samples of selected representative soil horizons must be collected and tested at an SABS approved laboratory to determine the soil characteristics such as grading, expansiveness, collapse, potential use for backfill, indicators, etc.
    - The corrosiveness of the material must be determined by analysing the pH and electrical conductivity of selected samples.
    - Identification and proving of potential borrow sites – Borrow sites to be identified to ensure that haul distances are kept to a minimum. The volume of borrow material to be proven by a dense grid survey and adequate laboratory testing, providing at least twice the volume required at each site.
    - Field electric resistivity survey - A field survey must be conducted to determine the in situ electrical resistivity along the entire route in collaboration with the CP analysis and design.

APPENDIX A

MAPS

APPENDIX B

WATER REQUIREMENTS

APPENDIX C

DRAWINGS

APPENDIX D

PIPELINE PROFILES

APPENDIX E

PHASE 1 SCHEME DIAGRAM

APPENDIX F

INTERESTED AND AFFECTED PARTIES

CONTACT LIST

APPENDIX G

COST MODELS

REPORT DETAILS PAGE

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