REPORT NUMBER: P RSA A000/00/9109

Department: Water Affairs

Chief Directorate: Integrated Water Resource Planning
Directorate: Options Analysis

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

Project No. WP9528

PRE-FEASIBILITY STAGE: REPORT 4:
DAMS, ABSTRACTION WEIRS AND RIVER WORKS

Lead Consultant: Africon

In association with: KV3, VELAVKE

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
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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:

Department of Water Affairs, South Africa, 2008. Mokolo Crocodile (West) Water Augmentation Project (MCWAP) Pre-Feasibility Stage Report 4: Dam, Abstraction Weirs and River Works. Prepared by Africon in association with Kwezi V3 Engineers, Vela VKE and specialists.

DWA Report No. P RSA A000/00/9109

REPORT DETAILS PAGE

Project name:	***Mokolo and Crocodile River (West) Water Augmentation Project (MCWAP)***
Report Title:	**Pre-Feasibility Study Report 4 - Dams, Abstraction Weirs and River Works: Pre-Feasibility Stage Report**
Author:	***BC Viljoen (Vela VKE)***
DWA report reference no.:	***P RSA A000/00/9109***
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PSP

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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.

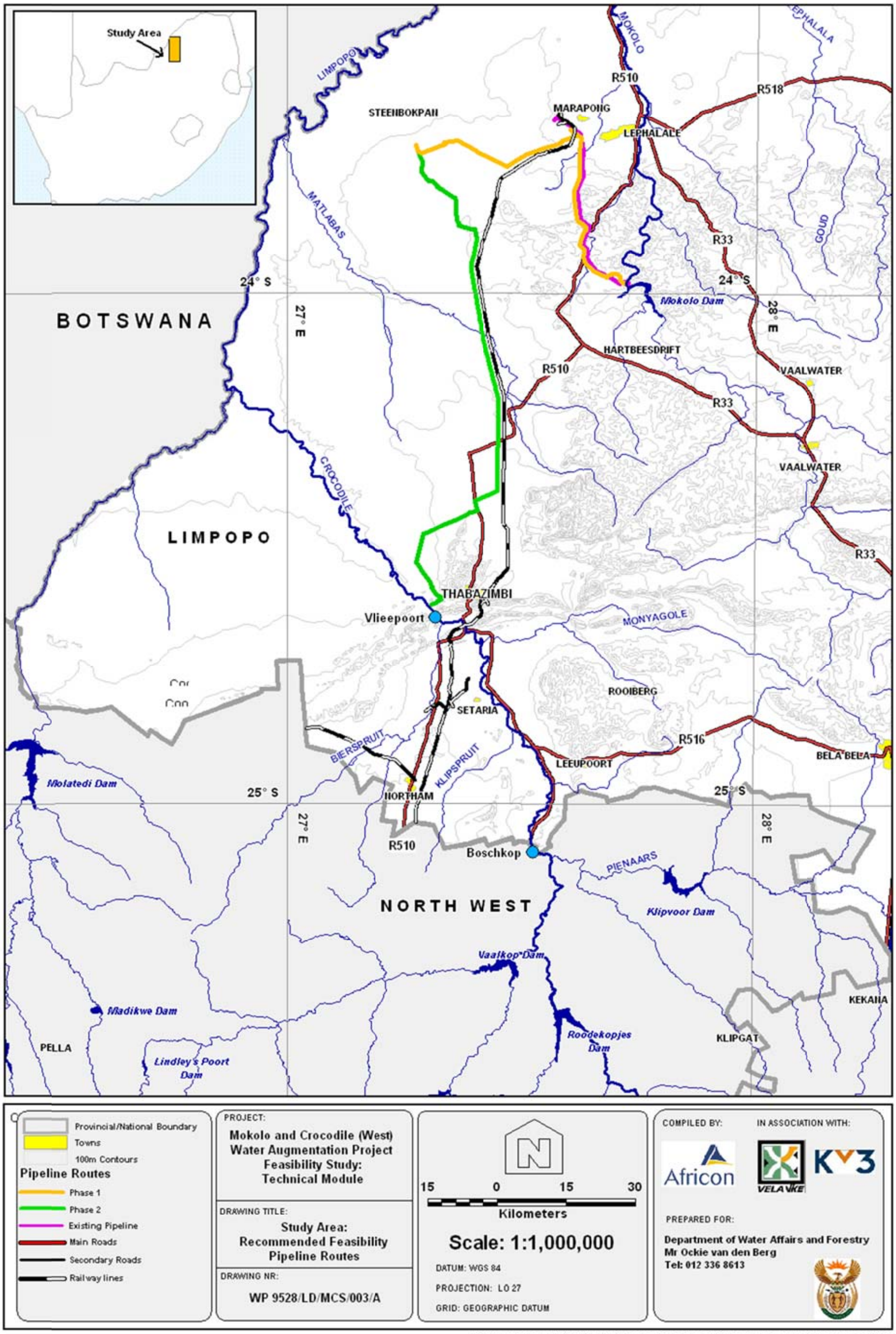
The Department of Water Affairs (DWA) commissioned the Mokolo and Crocodile River (West) Water Augmentation Project (MCWAP) to analyse the options for transferring water from the Crocodile River (West). In April 2008, the Technical Module of this study was awarded to Africon in association with Kwezi 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. The solution must over the long term, optimally utilise the full yield from Mokolo Dam.

- Phase 2: Transfer water from the Crocodile River (West) to the Lephalale area. Options to phase the capacity of the transfer pipeline (Phase 2A 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. 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 report during October 2009 to the MCWAP Main Report[8] in November 2009.

This report (Report 4 – Pre-Feasibility Stage Report: Dam, Abstruction Weirs and River Works, P RSA A000/00/9109) cover the different options and recommend the preferred development options, which will be followed by a Feasibility level investigation of the preferred water schemes.



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EXECUTIVE SUMMARY

1. Introduction

Report 4: Dams, Abstraction Weirs and River Works [(1)] cover all the work that was done during the investigation and pre-feasibility stages of this study. The scheme components that are dealt with in this report include:

- *Abstraction Weirs. Five sites along the Crocodile River and five sites along the Mokolo River were investigated for appropriateness. Two sites along the Crocodile River (Boschkop and Vlieëpoort) were selected and taken to Pre-feasibility investigation level. One site along the Mokolo River (Site 3 at the end of the Mokolo River gorge) was selected and taken to conceptual level only. Components associated with the abstraction weirs included:*
    - *Low-Lift Pump Stations*
    - De-siltation Structures
    - High-Lift Pump Station Balancing Dams.
- *Assessment of River Losses along Crocodile (West) and Mokolo Rivers.*
- *Terminal dams, reservoirs and client balancing reservoir options (to conceptual level only). As the study developed it became clear that the water usage centre of gravity had moved towards Steenbokpan to the west of Lephalale and that the original concept of a high terminal dam above Lephalale had become redundant. A single terminal reservoir at the centre of gravity was initially favoured, but was later replaced with the concept of the users providing their own water receiving and storage facilities and the concept of Client Balancing or Terminal reservoirs was finally adopted.*
- *Raising of Mokolo Dam (to conceptual level only). Yield analyses of Mokolo Dam (refer to Report 2: Water Resources[(2)]) indicated that no benefit would be gained in the short term from the raising of the dam and consequently further work on this was terminated at the end of the conceptual (investigation) stage.*

2. Design Flows and Capacities

Design capacity parameters were generated from data obtained from the Water Resources Report and Water Requirements Report(1) sections of the study (Reports 1 and 2) and are summarised in Table 1.

Table 1: Terminal Dam/Reservoir and Abstraction Weir Design Flow and Capacity Parameters

Item No.	Design Data	SCENARIO 4		SCENARIO 8	
		Design	Peak Flows	Design	Peak Flows
1.	**Water Requirements**	**Million m^3/a**	**Million m^3/a**	**Million m^3/a**	**Million m^3/a**
1.1	Phase 1A Transfer requirements (maximum average).	28,7	28,7	50,4	50,4
1.2	Exxaro pipeline contribution.	13,5	0	13,5	0

Item No.	*Design Data*	*SCENARIO 4*		*SCENARIO 8*	
		Design	*Peak Flows*	*Design*	*Peak Flows*
1.3	*Phase 1A Transfer requirements at Weir 3 (maximum average)*	*15,2*	*42,2*	*36,9*	*63,9*
1.4	*Phase 2 Crocodile River (West) Transfer requirements (maximum average), including system losses (2%) along Phase 1A and Phase 2 pipelines and reservoirs.*	*98,9*	*127,6 (2)*	*195,6*	*195,6*
1.5	*Incremental Losses in Crocodile River (due to additional release) for weir at Boschkop*	*22*	*25*	*30*	*30*
1.6	*Incremental Losses in Crocodile River (due to additional release) for weir at Vlieëpoort*	*51*	*59*	*70*	*70*
1.7	*Irrigation requirements up to Boschkop*	*42,9*	*42,9*	*42,9*	*42,9*
1.8	*Irrigation requirements up to Vlieëpoort*	*120,0*	*120,0*	*120,0*	*120,0*
1.9	*Total Releases from Dams to provide for Phase 2 – Boschkop Option*	*163,9*	*195,5*	*268,5*	*268,5*
1.10	*Total Releases from Dams to provide for Phase 2 – Vlieëpoort Option*	*269,9*	*306,6*	*385,6*	*385,6*
1.11	*Total Flow Releases from Dams to provide for Phase 2 - Boschkop Option*	*5,2 m^3/s*	*6,2 m^3/s*	*8,5 m^3/s*	*8,5 m^3/s*
1.12	*Total Flow Releases from Dams to provide for Phase 2 - Vlieëpoort Option*	*8,6 m^3/s*	*9,7 m^3/s*	*12,2 m^3/s*	*12,2 m^3/s*
2.	***Mokolo Weir (for reporting purposes only)***				
2.1	*Peak flow allowance (assumed to be available through short-term over-utilisation of Mokolo Dam.*	*9%*	*0%*	*9%*	*0%*
2.2	*Recovery Period allowance (assumed to be available through short-term over-utilisation of Mokolo Dam.*	*0%*	*20%*	*0%*	*0%*
2.3	*Design Flow*	*0,53 m^3/s*	*1,09 m^3/s*	*1,28 m^3/s*	*1,60 m^3/s*
2.4	*Number of Low Lift Pump Station bays and pumps sets (3).*	*2 No.*	*3 No.*	*4 No.*	*4 No.*
2.5	*Number of de-silting channels in De-silting Works (3).*	*2 No.*	*3 No.*	*4 No.*	*4 No.*
2.6	*Total capacity of high-lift pump station balancing dam provided.*	*20 300m^3*	*20 300m^3*	*30 900m^3*	*30 900m^3*
2.7	*Live storage capacity of high-lift pump station balancing dam provided.*	*17 000m^3*	*17 000m^3*	*25 900m^3*	*25 900m^3*

Item No.	Design Data	SCENARIO 4		SCENARIO 8	
		Design	Peak Flows	Design	Peak Flows
3.	***Boschkop/Vlieëpoort Abstraction Weir***				
3.1	*Peak flow allowance*	*9%*	*0%*	*9%*	*0%*
3.2	*Recovery Period allowance*	*0%*	*0%* (2)	*0%*	*20%*
3.3	*Design Flow Boschkop/Vlieëpoort Weir*	*3,4* m^3/s	*4,0* m^3/s	*6,8* m^3/s	*7,4* m^3/s
3.4	*Number of Low Lift Pump Station bays and pump sets.*	*5 No.*	*5 No.*	*8 No.*	*8 No.*
3.5	*Number of de-silting channels in De-silting Works.*	*5 No.*	*5 No.*	*8 No.*	*8 No.*
3.6	*Total capacity of high-lift pump station balancing reservoir provided.*	*70 300*m^3	*70 300*m^3	*136 700*m^3	*136 700*m^3
3.7	*Live storage capacity of high-lift pump station balancing reservoir provided.*	*57 400*m^3	*57 400*m^3	*111 700*m^3	*111 700*m^3
4.	***Terminal dams/Reservoirs***	***Million*** m^3	***Million*** m^3	***Million*** m^3	***Million*** m^3
4.1	*Emergency storage provision based on 5% down time per annum allowance (days).*	*18,3*	*18,3*	*18,3*	*18,3*
4.2	*Total Live Storage Provision (single Terminal reservoir or Terminal dam).*	*5,4*	*5,4*	*10,7*	*10,7*

Notes:

1. *Total Phase 2 water requirements less the Phase 1A contribution plus allowance for seasonal peaks.*
2. *The worst case emergency scenario for Phase 2 Works occurs when the Phase 1A Scheme (Mokolo Delivery) makes no contribution to transfer scheme (the Phase 2 Crocodile Works therefore transfers the full water requirement), OR, 20% allowance for recovery period after downtime, whichever is the largest.*

   The worst case emergency scenario for Phase 1A (Mokolo Dam supply) occurs when the Exxaro pipeline is out of operation.
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. In the case of the Mokolo weir the number of bays is based on 0.6* m^3/s *submersible pumps. Data for suitable pumps were obtained from pump suppliers.*
4. *Only evaluated at conceptual level for Scenario 8 as the provision of the user terminal reservoirs is the responsibility of the bulk consumers who will also operate and maintain the reservoirs and is therefore not a MCWAP responsibility. Also see Supporting Reports 6*(5) *and 10*(7) *for further details.*

5. *Ultimate Mokolo Dam supply after commissioning of Crocodile River (West) Transfer System (28.7 x 10^6 m^3/annum, including any losses). Maximum short-term supply from Mokolo Dam during interim period (50.4 x 10^6 m^3/annum, including any losses). Also refer to Supporting Report 1 for details.*
6. *Sized for maximum average transfer plus 9% average seasonal peaks.*

3. River Losses

The assessment of river losses proved to be a formidable task in view of the large number of variables that had to be dealt with and the relative paucity of relevant data. The data presented in the design flow capacity tables in this report are based on work that was done up to November 2008. This data was also used in the calculation of unit reference values for the various scheme options that were investigated and presented in Report 5(4) (Phase 1A Scheme Options) and Report 6(5) (Phase 2 Scheme Options).

Further work is continuing and the latest assessments and report is included in Section 9 of this Report. The data presented in Section 9 will be taken through to the feasibility stage of the Study. The latest data, which indicate slightly lower losses than those calculated and used to date, will not change the outcomes of the Pre-feasibility Stage, but are presented in this report as it will form the point of departure for the Feasibility Stage.

4. Sizing

Sizing criteria was prepared and the structures sized accordingly. Pertinent sizing data for the structures investigated are summarised in Table 2.

Table 2: Abstraction Weir Design Sizing Data

Item No.	DESIGN DATA	Option 1	Option 2
1.	**Mokolo Weir**		
1.1	Design Flood (Recommended Design Discharge (RDD)) (1:100 year flood)	5 427 m^3/s	N/A
1.2	Safety Evaluation Flood (SEF) (Probable Maximum Flood (PMF))	10 769 m^3/s	N/A
1.3	Riverbed Level	818,8 masl	N/A
1.4	Lowest Overspill Crest (OC) Level.	821,0 masl	N/A
1.5	Non-overspill Crest (NOC) Level (PMF plus 0,5m Freeboard).	828,6 masl	N/A
1.6	OC Length	286 m	N/A
1.7	Total Length of Structure	240 m	N/A
2.	**Boschkop Weir**		
2.1	Design Flood (RDD) (1:200 year flood)	4 779 m^3/s	4 779 m^3/s
2.2	Safety Evaluation Flood (SEF) (PMF)	9 558 m^3/s	9 558 m^3/s
2.3	1:20 year Return Period Flood	2 390 m^3/s	2 390 m^3/s

Item No.	*DESIGN DATA*	*Option 1*	*Option 2*
2.4	*1:50 year Return Period Flood*	*3 380 m^3/s*	*3 380 m^3/s*
2.5	*Riverbed Level*	*929,0 masl*	*929,0 masl*
2.6	*Lowest OC Level.*	*932,2 masl*	*932,2 masl*
2.7	*NOC Level (PMF plus 0,5m Freeboard).*	*951,9 masl*	*941,8 masl*
2.8	*OC Length*	*221 m*	*72 m*
2.9	*Total Length of Structure*	*295 m*	*90 m*
3.	***Vlieëpoort Weir***		
3.1	*Design Flood (RDD) (1:100 year flood)*	*5 741 m^3/s*	*5 741 m^3/s*
3.2	*Safety Evaluation Flood (SEF) (PMF)*	*11 184 m^3/s*	*11 184 m^3/s*
3.3	*1:20 year Return Period Flood*	*2 870 m^3/s*	*2 870 m^3/s*
3.4	*1:50 year Return Period Flood*	*4 020 m^3/s*	*4 020 m^3/s*
3.5	*Riverbed Level*	*890,0 masl*	*890,0 masl*
3.6	*Lowest OC Level.*	*893,2 masl.*	*893,2 masl.*
3.7	*NOC Level (PMF plus 0,5 m Freeboard).*	*912,8 masl*	*901,2 masl*
3.8	*OC Length*	*153 m*	*101 m*
3.9	*Total Length of Structure*	*308 m*	*122 m*

Notes:

1. *Design values based on normal design approach.*
2. *Design values based on submerged design approach (1:20 return period flood levels). Outflanking measures up to PMF level was provided in the form of jet grouted cut-offs and heavy rock groynes.*

5. Description of Components

(a) Abstraction Works General

The abstraction weir arrangement consists of:

- *Mass concrete gravity type Diversion Weir with ogee and roller bucket spillway.*
- *Gravel Traps 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 Non-overspill Crest (NOC) flank of the weir and partly into the riverbank.*
- *Low pressure pipeline to the de-silting works.*
- *De-silting Works with flushing facility located near the low-lift pump station, but above the Probable Maximum Flood level (PMF).*

- *A gravity pipeline between the De-silting Works and a Balancing Reservoir.*
- *A Balancing Reservoir equipped with submersible pumps to supply the adjacent high-lift pump station. The Balancing Reservoir will also be equipped with a silt flushing facility although only infrequent use, perhaps once every 10 years, is expected.*

(b) Mokolo Abstraction Works

- *The mass concrete gravity weir OC will be 2.0 m above river bed level and will be 5 m wide. The first Non-overspill Crest (NOC) will be 0.3 m above the OC level and 193 m wide, it then increases in height in steps to follow the river bank levels to a level above the Recommended Design Discharge (RDD).*
- *The gravel trap will be approximately 9 m long with three (3) channels leading to the two pump wells.*
- *The low pressure pipeline will consist of two 750 mm diameter steel pipes approximately 50 m long. Each pipe will have a gate valve in a valve chamber adjacent to the de-silting works.*
- *The de-silting works will consist of three 80 m long channels, 2.5 m wide and depth varying from 3.8 m to 4.8 m.*
- *Each de-silting channel will have an outlet in the form of a 750 mm diameter steel pipeline gravitating to the Balancing Reservoir inlet.*
- *The Balancing Reservoir will have top dimensions of 150 m x 50 m. The depth varies from 6.65 m at the inlet side to 4.65 m at the outlet side providing 0.5 m of freeboard above the Full Supply Level (FSL).*

(c) Boschkop and Vlieëpoort Abstraction Works

- *Two weir layout options were considered. Option 1 places the low-lift pump station controls and access above the Probable Maximum Flood (PMF) level above the pump wells. The weir is also extended in steps to a level above the Probable Maximum Flood (PMF) level. Option 2 places the pump controls away from the weir on the right river bank above the Probable Maximum Flood (PMF) level. The structure is therefore much lower with the top of the structure corresponding with the level of the right bank.*
- *The gravel trap will be approximately 33 m long with eight channels leading to the eight pump wells.*
- *The low pressure pipeline will consist of a 2 100 mm diameter steel pipeline approximately 245 m long. It will then be split with a manifold into eight 750 mm diameter pipes leading to the de-silting works inlets. Each pipe will have a gate valve in a valve chamber adjacent to the de-silting works.*
- *The de-silting works will consist of nine 80 m long channels, 2.5 m wide and depth varying from 3.8 m to 4.8 m.*
- *Each de-silting channel will have an outlet in the form of a 750 mm diameter steel pipeline gravitating to the Balancing Reservoir inlet.*

- *The Balancing Reservoir will have top dimensions of 300 m x 100 m. The depth varies from 6.65 m at the inlet side to 4,65m at the outlet side providing 0.5 m of freeboard above the Full Supply Level (FSL).*

(d) Terminal Dam/Reservoir and Client Terminal Reservoirs

- *The Client Terminal reservoirs will be artificial dams using a waterproofed earth fill embankment, similar to the abstraction weir balancing reservoir.*
- *These reservoirs are sized to provide 18 days of average annual demand at each of the delivery nodes. Freeboard of 1m was allowed above the Full Supply Level (FSL). Each dam will be subdivided into compartments with a width of between 75 m and 105 m.*

  One additional compartment over and above those provided for 18 days storage will also be provided. One compartment will be operational at a time and will be emptied before switching to the next one. This will prevent stagnant areas from forming that would otherwise occur in a single large reservoir whilst at the same time insuring that 18 days of storage is always available.
- *The inlet to each bay will be by means of a manifold coming off the main delivery pipe. The inflow will then be spread across the width of the bay by using a baffled weir type intake structure. The even spread of inflow will assist with the prevention of the forming of stagnant water zones in the reservoir.*

6. Capital Costs

Capital cost estimates were undertaken using the cost models presented in Supporting Report 3 (3) as basis. Cost models for each of the structures considered are included in Appendix A.

Unit rates were based on an April 2008 base date. Further details on the derivation of the unit rates can be found in Report 3(3): Guidelines for Peliminary Sizing, Costing and Economic Evaluation of Development Options.

Quantities were calculated, using the Pre-feasibility stage drawings listed in Section 12 of this Report and included in Appendix B.

A summary of the estimated capital costs associated with each of the components that were studied are included in Table 3.

Table 3: Estimated Capital Costs

Item No.	DESCRIPTION OF COMPONENT	Scenario 4 Rand	Scenario 8 Rand
1.	**Mokolo Works**		
1.1	Abstraction Weir and Low-Lift Pump Station Civil Works	153 462 000	153 462 000
1.2	Low-Lift Pump Station M&E Works (2),(3)	Refer Supporting Report 5	Refer Supporting Report 5
1.3	De-silting Works	20 984 000	20 984 000
1.4	High-Lift Pump Station Balancing Dam	28 191 000	28 191 000

Item No.	DESCRIPTION OF COMPONENT	Scenario 4 Rand	Scenario 8 Rand
1.5	**Total Cost**	**202 637 000**	**202 637 000**
2.	**Boschkop Works**		
2.1	Abstraction Weir and Low-Lift Pump Station Civil Works	173 894 000	173 894 000
2.2	Low-Lift Pump Station M&E Works (2),(3)	Refer Supporting Report 6(5)	Refer Supporting Report 6(5)
2.3	De-silting Works	32 332 000	51 731 000
2.4	High-Lift Pump Station Balancing Dam	56 392 000	90 226 000
2.5	**Total Cost**	**262 618 000**	**315 851 000**
3.	**Vlieëpoort Works**		
3.1	Abstraction Weir and Low-Lift Pump Station Civil Works	155 555 000	155 555 000
3.2	Low-Lift Pump Station M&E Works (2),(3)	Refer Supporting Report 6(5)	Refer Supporting Report 6(5)
3.3	De-silting Works	29 788 000	47 660 000
3.4	High-Lift Pump Station Balancing Reservoir	36 571 000	58 512 000
3.5	**Total Cost**	**221 914 000**	**261 727 000**
4.	**Terminal dams (total Storage Capacity = 11 Million m^3)**		
4.1	Site 1		281 751 000
4.2	Site 2		215 781 000
4.3	Site 3		342 364 000
4.4	Site 4		345 273 000
5.	**Client Terminal reservoirs (4)**	**Net Volume (m^3)**	**Scenario 8 Rand**
5.1	Client Terminal reservoir - Zealand	624 100	148 309 000
5.2	Client Terminal reservoir – Exxaro Lephalale	1 090 800	259 214 000
5.3	Client Terminal reservoir – Eskom Lephalale	880 000	209 120 000
5.4	Client Terminal reservoir – Steenbokpan	1 396 400	331 835 000
5.5	Client Terminal reservoir – Exxaro Steenbokpan	306 000	72 717 000
5.6	Client Terminal reservoir – Sasol Steenbokpan	3 700 000	879 254 000
5.7	Client Terminal reservoir – Eskom Steenbokpan	2 250 000	534 681 000
5.8	**Totals for all Client Terminal Reservoirs**	**10 247 300**	**2 435 130 000**
5.9	**Total Single Storage Provision (MCWAP Terminal Reservoir).**	**10 050 000**	**1 582 502 000**

Notes:

1. *The cost estimates for Scenario 4 was not calculated to the same level of detail employed for the Scenario 8 estimates. As the weir presents 60% of the cost of the structure and will remain essentially unchanged for the Scenario 4 design, savings amounting to only approximately 15% of the Scenario 8 estimates have been allowed for should Scenario 4 materialise in the Pre-feasibility stage. The Terminal dams and Reservoirs were not investigated for Scenario 4.*
2. *The costs of pipework, valves, screens and cranage have been included in the civil works portions of the cost estimate.*
3. *The costs of the pumps and any M&E control equipment required are not included. For the purposes of the Pre-feasibility stage these costs have been included with the pump station costs in Supporting Report 6*[(5)]*.*
4. *Only evaluated at conceptual level for Scenario 8 as the provision of the user terminal reservoirs will be the responsibility of the bulk consumers who will also operate and maintain the reservoirs and is therefore not a MCWAP responsibility. Also see Supporting Reports 6*[(5)] *and 10*[(7)] *for further details.*
5. *Final sizing requirements subsequently determined to be 10.7 Million m^3.*

7. *Drawings*

The Pre-feasibility Stage drawings are listed in Section 12 of this Report and are included in Appendix B.

8. *Pertinent Issues Proposed to be dealt with in the Feasibility Stage*

For the Pre-feasibility stage it was assumed that all the water requirements would be available in the river at the Abstraction Works. A system that was capable of immediate responses to changes in water requirements, changes in river loss patterns and accruals were therefore assumed. On the ground the situation would be quite different because 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 between three and five 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 scenarios for operation of the Abstraction Works are possible:

(i) *Only abstract what is required and let the rest of the flow pass (and ensure that enough water is released so as not to run out at the Abstraction Works).*

(ii) *Design enough capacity into the Abstraction Works (Weir, Low-lift pump station and High-lift pump station Balancing Reservoir to allow for abstraction of surplus water, when available, to storage 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. Note that the implementation of Scenario (ii) may well remain a key component necessary for the successful implementation of Scenario (iii).*

In view of the shortage of water in the Crocodile River (West) catchment, it is recommended that Scenarios (ii) and (iii) be investigated further.

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

TECHNICAL MODULE

REPORT 4: DAMS, ABSTRACTION WEIRS AND RIVER WORKS

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DT	Discharge Table
DWA	Department of Water Affairs
API	Aerial Photography Interpretation
CFRD	Concrete Faced Rockfill Dam
EMP	Environmental Management Plan
ER	Ecological Reserve
EW	East-West
FSL	Full Supply Level
IFR	Instream Flow Requirements
MOL	Minimum Operating Level
MCWAP	Mokolo and Crocodile River (West) Water Augmentation Project
NE	North-East
NKP	National Key Point
NOC	Non-overspill Crest
NW	North-West
OC	Overspill Crest
PMF	Probable Maximum Flood
PV	Present Value
RDD	Recommended Design Discharge
RDF	Recommended Design Flood
RMF	Regional Maximum Flood
ROD	Record of Decisions
SED	Safety Evaluation Discharge
SEF	Safety Evaluation Flood
URV	Unit Reference Value
VAPS	Vaal Augmentation Planning Study
WMA	Water Management Area
WTW	Water Treatment Works

1. PURPOSE OF THE REPORT

The Dams, Abstraction Weirs and River Works Report encapsulate all the work that was done during the Pre-feasibility Stage of the Study on the following:

- The water requirements and storage requirements that will be used for the sizing and costing of abstraction works, terminal dams and reservoir options by taking account of operational, reliability and redundancy requirements.
- Potential abstraction weir and dam sites.
- Layouts for the recommended structures for sizing and costing purposes.
- River losses along the Crocodile and Mokolo Rivers.

The overall layout of the Mokolo and Crocodile River (West) Water Augmentation Project (MCWAP) study area is given in Figure 1-1 overleaf.

The report provides pre-feasibility level capital cost estimates for each of the options which were carried forward to the scheme engineering economics analyses.

The abstraction weir and pumping station investigations were done based on the specific transfer options for each of the two weir abstraction sites identified along the Crocodile River (Boschkop and Vlieëpoort), as well as the various sub-options (including two-phase implementation) from these weirs. Options were generated at pre-feasibility level, one of which will be developed further to feasibility level, depending on the final selected transfer route.

The objectives of options development were to identify efficient, workable and reliable options, as this abstraction weir and pump station systems will in turn form part of the system development options that will be compared, and the option selected for development to feasibility level.

The purpose of this report is to examine the options, to provide appropriate cost estimates for use in the subsequent two options reports and to identify other issues that cannot be mitigated and that will have residual impacts or benefits. The report also enumerates the processes that were adopted to make the appropriate contributions to the complete Pre- and Feasibility Study stages.

All facets of the abstraction works and pump station pre-feasibility study, layout and sizing were conducted by the specialist disciplines involved on an integrated basis.

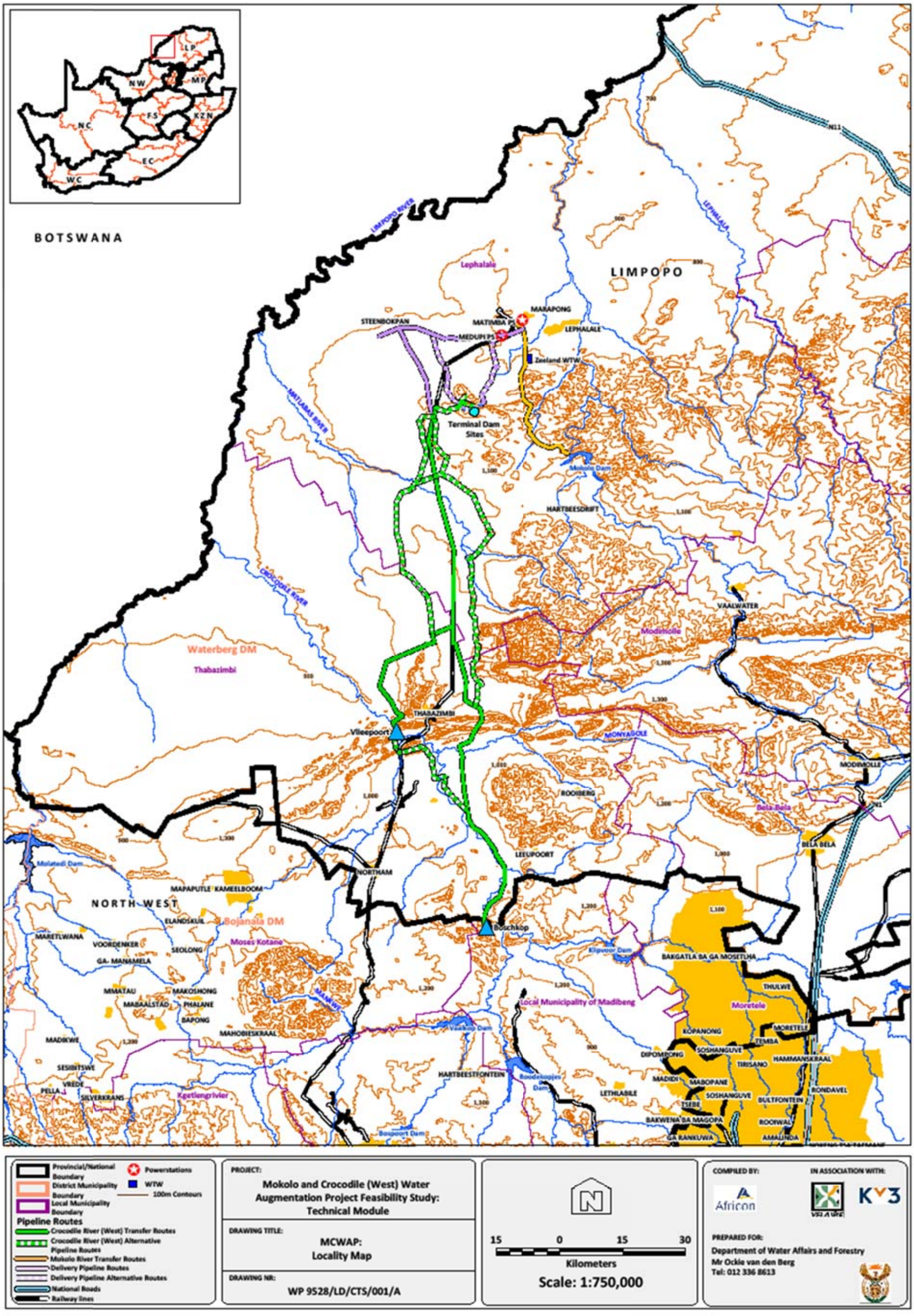

Figure 1-1: Overall Layout of MCWAP Scheme

2. DEMANDS AND DESIGN FLOWS

2.1. Reliability and Redundancy

The following general criteria were applied during the pre-feasibility design:

- The abstraction works and terminal dams 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.
- Dams and weirs will be designed according to the SANCOLD, (1991)(9) Guidelines on Safety in Relation to Floods for Category III dams with significant high hazard rating to cater for the strategic importance of the Scheme.
- Design floods will have a recurrence interval of 200 years and extreme safety evaluation floods will be based on the probable maximum flood.
- Duplicate screened inlet and outlet works will apply for dams design.
- Abstraction (low-lift) pump station sites will have the switchgear and control instrumentation located above the 1:100 flood level plus 0.5 m freeboard. Two options for the siting of the switchgear and control instrumentation were investigated:
  - Option 1: In the superstructure of the weir above the pump bays in the weir; and
  - Option 2: In a separate structure on the riverbank to reduce the impact of the weir on the river cross-section profile.
- High lift and booster pump stations other than the low lift abstraction pump stations will be positioned above the Probable Maximum Flood (PMF) and designed such that they will always be free-draining in the event of flooding due to failure of internal pipework
- High-lift pump stations will be designed with a minimum of three duty pump units and 25% standby units' capacity per duty rating.
- Low-lift pump stations will be provided with an additional pumping bay for use during 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 demand plus all the downstream system losses supplied to on-site balancing dams.
- A terminal dam or terminal reservoir/s will be provided to cater for less than 100% system availability and to reduce the risk of failure of water supply to the strategic industries.

- Three options of providing strategic storage were considered:
  - (i) A terminal dam at an altitude to supply all consumers by gravity;
  - (ii) A terminal reservoir at the centroid of the users;c and
  - (iii) On-site client terminal reservoirs. In the case of on-site/client terminal reservoirs live storage will be limited to a volume of 9 days consumption for gravity feed supply systems and the full 18 days for pumped supply systems.
- The capacity and location of the terminal dam to take into account the reliability and redundancy provided from the Mokolo Dam.
- Downtime for scheduled preventative maintenance will be taken into account.
- Sufficient additional water will be made available by the Department of Water Affairs (DWA) in the Crocodile River (West) (Phase 2) to supply the full requirement via the Crocodile River (West) Transfer Scheme during emergency situations.
- Reserve storage above the lowest operating level in the Mokolo Dam will be considered to allow for emergency situations.

Allowing for a scheme to be inoperative 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 due to flooding;
- Pipeline repairs; and
- The time required to restore power supplies after major interruptions such as bushfires, lightning strikes, flooding, etc.

2.2. Design Capacities

Various combinations covering user water requirements, seasonal variations of requests, and water transfer options for the Phase 1A (Mokolo Dam supply) and Phase 2 (Crocodile River (West) Transfer) Schemes were investigated and design capacities for the various components determined. These combinations are summarised in the table below. The combinations considered were:

(1) Combination 1A: Phase 2 water requirements and deducting Phase 1A contribution and assuming seasonal peaks and terminal dam/reservoirs and client balancing reservoir recovery periods do not coincide. Phase 1A with pipeline only solution.

(2) Combination 1B: Phase 2 water requirements and deducting Phase 1A contribution and assuming seasonal peaks and terminal dam/reservoirs and client balancing reservoir recovery periods coincide. Phase 1A with pipeline only solution.

(3) Combination 2A: Phase 2 water requirements and deducting Phase 1A contribution and assuming seasonal peaks and terminal dam/reservoirs and client balancing reservoir recovery periods do not coincide. Phase 1A with pipeline and abstraction weir solution.

(4) Combination 2B: Phase 2 water requirements and deducting Phase 1A contribution and assuming seasonal peaks and terminal dam/reservoirs and client balancing reservoir recovery periods coincide. Phase 1A with pipeline and abstraction weir solution.

(5) Combination 3A: Phase 2 water requirements and disregarding Phase 1A contribution and assuming seasonal peaks and terminal dam/reservoirs and the client balancing reservoir recovery periods do not coincide. Use the larger of the Phase 1A contribution or the recovery period flows without added provision for seasonal peaks.

(6) Combination 3B: Phase 2 water requirements and disregarding Phase 1A contribution and assuming seasonal peaks and the terminal dam/reservoirs and client balancing reservoir recovery periods coincide. Use the larger of the Phase 1A contribution or the recovery period flows and with added provision for seasonal peaks.

(7) Combination 4A: Phase 2 water requirements and deducting Phase 1A contribution and assuming seasonal peaks and terminal dam/reservoirs and client balancing reservoir recovery periods do not coincide. Phase 1A with pipeline only solution and fully raised Mokolo Dam at maximum yield.

(8) Combination 4B: Phase 2 water requirements and deducting Phase 1A contribution and assuming seasonal peaks and terminal dam/reservoirs and client balancing reservoir recovery periods coincide. Phase 1A with pipeline only solution and fully raised Mokolo Dam at maximum yield.

Derivation of the base data can be found in Supporting Report 1: Water Requirements[1] Water requirement Scenarios 4 and 8 were analysed for the combinations listed above. During the course of the study the following combinations of possible solutions were discounted:

(a) Combination 2A and 2B: Mokolo Weir (Site 3 Weir or Rivers Bend Weir) as alternative to the doubling of the existing Exxaro Pipeline was discarded on the basis that the yield of Mokolo Dam was insufficient to afford river losses associated with a river conveyance solution. Refer to Supporting Report 5[4]: Mokolo River Development Options and Section 8 below for further details.

(b) Combination 4A and 4B: The raising of Mokolo Dam was discarded as a solution because the system yield analysis (Report 2: Water Resources[2]) showed that any increase in yield resulting from raising of the Dam would be discounted by increased Instream Flow Requirement (IFR) releases.

Combinations 1A & 1B and 3A & 3B were taken further to the Pre-feasibility stage. The Combination 1A was selected as the combination that would represent the normal working design case best. Combination 3B was considered to represent the most rational worst case scenario. Refer to Table 2-1 for further details on the application of the various allowances and factors in the design case combinations.

Table 2-1: Allowances and Factors used in Design Scenarios

Item No.	Allowance and Factors Applied	Design Case	Peak Design Case
1.	Allowance for water requirement peaks (average annual allowance) [1], [4]	9%	0%
2.	System Losses. Phase 1A (Mokolo Dam supply) added to Phase 2 Crocodile River (West) transfer system losses	2%	2%
3.	Allowance for 90 day Recovery Period after	0%	20%

Item No.	Allowance and Factors Applied	Design Case	Peak Design Case
	maximum 18 day system outage (3)		
4.	95% Reliability factor (4)	5%	0%
5.	Allowance for variations in river flow (5)	0%	0%
6.	Failure of Phase 1A Mokolo Dam supply (due to over-usage, etc.) (6)	Nil	28,7 Million m^3

Notes:

1. Refer to Supporting Report No 1 for details.
2. The % allowances factor 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 for the system loss factor) to avoid compounding of related allowances.
4. The greater of the peak flow factor and reliability flow factor was used.
5. For the Pre-feasibility stage it was assumed that all the water requirements would be available in the river at the abstraction works. This aspect will be further addressed in the Feasibility stage. Also refer to discussion in Section 2.3 below.
6. Only used if greater than 20% allowance for Recovery Period.

The design values were based on data obtained for Combination 1A (where the Phase 1A Scheme (Mokolo Dam Supply) is fully operational and the peak design check values were based on the greater of:

- Recovery period allowance of 20%;
- The emergency condition where Phase 1A Scheme (Mokolo Dam Supply) is in-operational and consequently makes no contribution to transfer scheme (Combination 3B); or
- In the case of the Phase 1A Mokolo Weir, where the Exxaro pipeline is out of commission.

The pertinent design data is summarised in Table 2-2.

Table 2-2: Pertinent Abstraction Works Design Data

Item No.	DESIGN DATA	SCENARIO 4		SCENARIO 8	
		Design	Peak Flows	Design	Peak Flows
1.	**Water Requirements**	**Million m^3/a**	**Million m^3/a**	**Million m^3/a**	**Million m^3/a**
1.1	Phase 1A Transfer requirements (maximum average)	28,7	28,7	50,4	50,4
1.2	Exxaro pipeline contribution.	13,5	0	13,5	0
1.3	Phase 1A Transfer requirements at Weir 3 (maximum average)	15,2	28,7	36,9	50,4

Item No.	DESIGN DATA	SCENARIO 4		SCENARIO 8	
		Design	Peak Flows	Design	Peak Flows
1.4	Phase 2 Crocodile River (West) Transfer requirements (maximum average), including system losses (2%) along Phase 1A and Phase 2 pipelines and reservoirs.	98,9	127,6 (2)	195,6	195,6
1.5	Incremental Losses in Crocodile River (due to additional release) for weir at Boschkop	22	25	30	30
1.6	Incremental Losses in Crocodile River (due to additional release) for weir at Vlieëpoort	51	59	70	70
1.7	Irrigation requirements up to Boschkop	42,9	42,9	42,9	42,9
1.8	Irrigation requirements up to Vlieëpoort	120,0	120,0	120,0	120,0
1.9	Total Releases from Dams to provide for Phase 2 – Boschkop Option	163,9	195,5	268,5	268,5
1.10	Total Releases from Dams to provide for Phase 2 – Vlieëpoort Option	269,9	306,6	385,6	385,6
1.11	Total Flow Releases from Dams to provide for Phase 2 - Boschkop Option	5,2 m^3/s	6,2 m^3/s	8,5 m^3/s	8,5 m^3/s
1.12	Total Flow Releases from Dams to provide for Phase 2 - Vlieëpoort Option	8,6 m^3/s	9,7 m^3/s	12,2 m^3/s	12,2 m^3/s
2.	**Mokolo Weir (for reporting purposes only)**				
2.1	Peak flow allowance (assumed to be available through short-term over-utilisation of Mokolo Dam	9%	0%	9%	0%
2.2	Recovery Period allowance (assumed to be available through short-term over-utilisation of Mokolo Dam	0%	20%	0%	0%
2.3	Design Flow	0,53 m^3/s	1,09 m^3/s	1,28 m^3/s	1,60 m^3/s
2.4	Number of Low Lift Pump Station bays and pump sets	2 No.	3 No.	4 No.	4 No.
2.5	Number of de-silting channels in De-silting Works	2 No.	3 No.	4 No.	4 No.
2.6	Total capacity of high-lift pump station balancing dam provided	20 300m^3	20 300m^3	30 900m^3	30 900m^3
2.7	Live storage capacity of high-lift pump station balancing dam provided	17 000m^3	17 000m^3	25 900m^3	25 900m^3

Item No.	DESIGN DATA	SCENARIO 4		SCENARIO 8	
		Design	Peak Flows	Design	Peak Flows
3.	**Boschkop/Vlieëpoort Abstraction Weir**				
3.1	Peak flow allowance	9%	0%	9%	0%
3.2	Recovery Period allowance	0%	0% (2)	0%	20%
3.3	Design Flow Boschkop/Vlieëpoort Weir	3,4 m^3/s	4,0 m^3/s	6,8 m^3/s	7,4 m^3/s
3.4	Number of Low Lift Pump Station bays and pump sets	5 No.	5 No.	8 No.	8 No.
3.5	Number of de-silting channels in De-silting Works	5 No.	5 No.	8 No.	8 No.
3.6	Total capacity of high-lift pump station balancing reservoir provided	70 300m^3	70 300m^3	136 700m^3	136 700m^3
3.7	Live storage capacity of high-lift pump station balancing reservoir provided	57 400m^3	57 400m^3	111 700m^3	111 700m^3
4.	**Terminal dams/Reservoirs**	**Million m^3**	**Million m^3**	**Million m^3**	**Million m^3**
4.1	Emergency storage provision based on 5% down time per annum allowance (days)	18,3	18,3	18,3	18,3
4.2	Total Live Storage Provision (single Terminal reservoir or Terminal dam)	5,4	5,4	10,7	10,7

Notes:

1. Design values based on data obtained for Combination 1A and application of allowance factors detailed in Table 2-1.
2. Design check values based on data obtained for Combination 3B: The emergency condition where Phase 1A Scheme (Mokolo Dam Supply) makes no contribution to the MCWAP, OR when the recovery period to refill Terminal dam/Reservoirs occurs (20% additional allowance), whichever is the largest.

   For Phase 1A (Mokolo Dam supply) the worst case scenario occurs when the Exxaro pipeline is out of operation.
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. In the case of the Mokolo weir the number of bays is based on 0.6 m^3/s submersible pumps. Data for suitable pumps were obtained from pump suppliers.
4. Ultimate Mokolo Dam supply after commissioning of Crocodile River (West) transfer system (28.7×10^6 m^3/a, including any losses). Maximum short-term supply from Mokolo Dam during interim period (50.4×10^6 m^3/a, including any losses). Also refer to Supporting Report 1(1) for details.

5. Sized for maximum average transfer plus 9% average seasonal peaks.

2.3. Pertinent Issues to be dealt with in Feasibility Stage

For the Pre-feasibility stage it was assumed that all the water requirements would be available in the river at the Abstraction Works. A system that was capable of immediate responses to changes in water requirements, changes in river loss patterns and accruals were therefore assumed. On the ground the situation would be quite different because 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 between three and five 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 scenarios for operation of the Abstraction Works are possible:

(i) Only abstract what is required and let the rest of the flow pass (and ensure that enough water is released so as not to run out at the Abstraction Works).

(ii) Design enough capacity into the Abstraction Works (Weir, Low-lift pump station and High-lift pump station Balancing Reservoir to allow for abstraction of surplus water, when available, to storage 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. Note that the implementation of Scenario (ii) may well remain a key aspect necessary for the successful implementation of Scenario (iii).

In view of the shortage of water in the Crocodile River (West) catchment, it is recommended that Scenarios (ii) and (iii) be investigated further.

3. DESCRIPTION OF COMPONENTS AND OPTIONS CONSIDERED

3.1. Options Investigated

3.1.1. Abstraction Weir Sites Investigated

Site Evaluation Criteria

(1) Weir to be located downstream of main supply dams in Crocodile River (West) being Vaalkop, Roodekopjes and Klipvoor Dams. Consequently, only the weir sites downstream of Pienaars River confluence will meet with 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 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 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.

(7) Founding conditions. Bed rock to be present to avoid costly foundation treatment and 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 (Crocodile River (West) options). In the case of the Phase 1A of the Scheme the criterion is the shortest distance to Zeeland Works to reduce pipeline costs (Mokolo River options).

(10) Weir to be as close as possible to sources of water (dams listed in criterion (1) and Mokolo Dam for the Mokolo River) 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 from 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.

Sites Identified

Several possible weir sites along the Crocodile and Mokolo Rivers were identified from aerial photography and the first site visit on 11 to 13 June 2008. Each site was 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 each of the Crocodile River (West) and Mokolo Rivers.

A common feature along both rivers was the deep alluvial sands and silts that filled the river valleys with depths of 10 to 20m reported. Rock exposures along both rivers were a rarity. Foundations for river crossings typically consisted of compacted dumped rock. As the foundations settled into the riverbed (or was washed away be floods) more rock was dumped and the crossing 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 foul 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 weir, would require extensive (and expensive) protection works to ensure the required longevity of the structure. Consequently all potential weir sites located in the wide flood plain were discounted very early in the process.

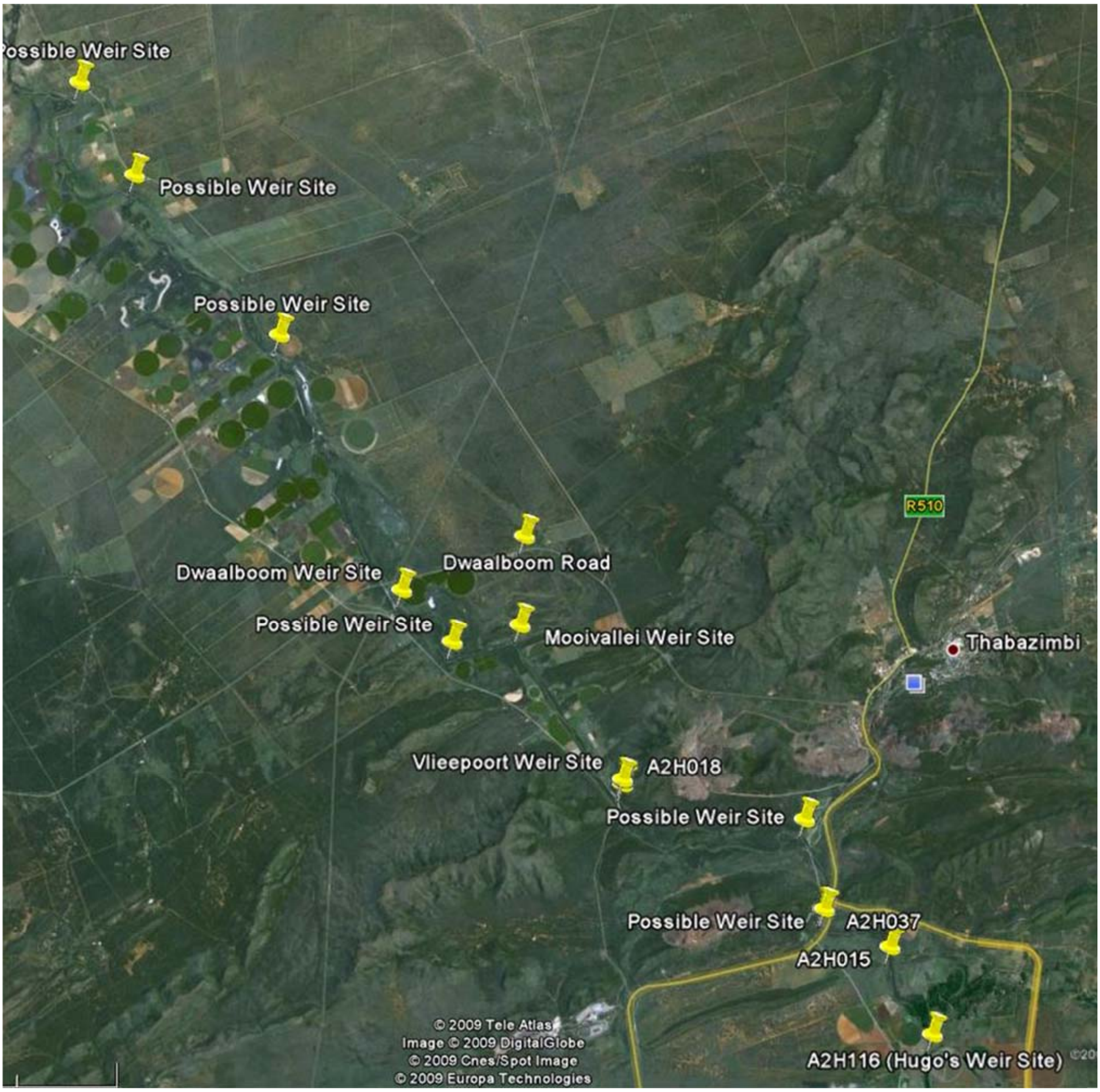

Figure 3-1: Crocodile River (West) – reach between Hugo's Weir and Makoppa Farms

The sites between Boschkop and Vlieëpoort and those downstream of Mooivallei (Makoppa reach) were discounted after the first round of evaluations. At that time the Dwaalboom site was considered typical of the potential weir sites along the Makoppa reach, but for the purposes of this discussion a separate assessment of the Faure Site has been prepared. Note that the Faure Weir Site is the location of the present DWA gauging weir A2H128.

CROCODILE RIVER (WEST)

An aerial view of the Crocodile River (West) reaches between the Makoppa irrigation farms in the north and Roodekopjes Dam in the south is given in

Figure 3-1 to Figure 3-4 below. All the possible weir sites that were considered are indicated on the images.

Reach between Hugo's Weir and Makoppa Farms. As can be seen in Figure 3-1, a total of ten (10) sites were initially identified and tested against the site evaluation criteria detailed above. With the exception of the Vlieëpoort, Mooivallei and Dwaalboom sites, all the other sites were discarded on the basis of evaluation criteria (4), (5), (7), (12) and (13).

A summary of the findings for the more promising sites, indicating advantages and disadvantages are as follows.

Vlieëpoort Weir Site

- ✓ Located on a suitable bend in the river.
- ✓ Narrow floodplain between two hills.
- ✓ Good sites for the de-silting works and high lift pump station balancing dam available, but some distance away from the weir site.
- ✓ Requires 30 km shorter pipeline than Boschkop option.
- ✓ Nature reserve immediately upstream won't 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 75 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 aren't taken.

Table 3-1: 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	Narrow river valley
(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	River channel deep.
(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, manageable
(13)	Potential for flood damage	Low, but manageable

This site was listed as potentially useful, but with question marks against criteria (7) and (10).

Faure Weir Site

The Faure Site is approximately 50.7 km downstream of the Vlieëpoort Weir site (river channel distance) and is located on the wide flood plain forming the bottom reach of the lower Crocodile River (West).

- ✓ Narrow river channel, approximately 30 m wide.
- ✓ Required pipeline length approximately 10 km short than for the Vlieëpoort option.
- ✓ Good access roads and power lines located close to the site.
- × Not located on a suitable bend in the river.
- × Not situated on the same side of river as pipeline route – will require expensive river crossing.
- × Very wide, open floodplain. A 20 m deep flood would flow about 9 km wide.
- × High risk of outflanking.
- × No information available on founding conditions.
- × Weir basin expected to be very large, resulting in high evaporation losses.
- × Very long river conveyance, leading to more losses in river.
- × Upstream infrastructure (irrigated farmlands and a road bridge) might be affected by higher flood levels.
- × High risk for flood damage in the flood plain.

Table 3-2: Site Evaluation Summary for Faure Weir

Criterion No.	Description	Comments
(1)	Downstream of Pienaars River Confluence	Yes
(2)	Abstraction works on outside of river bend	No
(3)	Abstraction works on same side of river as pipeline	No
(4)	Narrow river valley or flood plain	Very wide, open floodplain
(5)	Potential for outflanking to be manageable	High risk of outflanking
(6)	Narrow river channel	Yes, approximately 30m wide
(7)	Good founding conditions	No information available
(8)	Small weir basin	Hard to gauge depth of the channel
(9)	Pipeline length to users as short as possible	Yes, shorter pipeline than from Vlieëpoort
(10)	Upstream river length as short as possible to curtail losses	No, longer than to Vlieëpoort.
(11)	Proximity of access roads, power lines etc.	Yes
(12)	Upstream infrastructure affected by higher flood levels	Yes
(13)	Potential for flood damage	Yes

Due to non-compliance with evaluation criteria (2), (3), (4), (5), (10) and (13) this site is not suitable.

For the purpose of the discussion the following aspects that were covered for the preferred sites are also noted.

Mooivallei Weir Site

- ✓ Located on a suitable bend in the river.
- ✓ Possibly situated on quartzite or Ventersdorp lava, certainly higher up on right bank.
- ✓ Good sites for the de-silting works and high lift pump station balancing dam available.
- ✓ Requires 35 km shorter pipeline than Boschkop option.
- × Side stream from right bank entering river near preferred location.
- × Apart from right bank no visible rock outcrops, founding probably on deep sands.
- × Low left bank with wide floodplain and clear evidence of river course changes in recent time.
- × Approximately 80 km longer river conveyance than Boschkop option thereby increasing potential water losses.

Table 3-3: Evaluation Criteria Assessment for Mooivallei 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	No, right bank promising
(5)	Potential for outflanking to be manageable	No, left bank problematic
(6)	Narrow river channel	Reasonable
(7)	Good founding conditions (rock)	No, deep alluvium. RB good
(8)	Small weir basin	No, left bank a concern
(9)	Pipeline length to users as short as possible	Yes
(10)	Upstream river length as short as possible to curtail losses	No, 2nd furthest site
(11)	Proximity of access roads, power lines, etc.	Yes
(12)	Upstream infrastructure affected by higher flood levels	Possible, manageable
(13)	Potential for flood damage	Moderate, but manageable

Due to non-compliance with evaluation criteria (4), (5), (8), (10) and (13) this site was not recommended.

Dwaalboom Weir Site

- ✓ Requires 30 km shorter pipeline than Boschkop option.
- ✓ Close to Eskom power lines.
- ✓ Will result in the shortest possible pipeline route to the users.
- × Very flat and wide floodplain and extensive outflanking protection will be required.
- × Various irrigated lands and structures on the upstream floodplain which will be affected by raised water levels.
- × Old river course present along right bank.
- × During site inspection old erosion channels were observed along left bank.
- × Existing causeway has failed, indicating possibly deep founding conditions.
- × Located on a straight section of river, which is not suitable for abstraction works.
- × Approximately 85 km longer river conveyance than Boschkop option thereby increasing potential water losses.

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

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

Due to non-compliance with evaluation criteria (2), (4), (5), (7), (8), (10) and (13) this site was discarded.

<u>Reach between Koedoeskop and Hugo's Weir</u>. As can be seen in Figure 3-2 below a total of six weir sites were initially identified and tested against the site evaluation criteria detailed above. Without exception all the sites were discarded on the basis of evaluation criteria (4), (5), (7), (12) and (13). In spite of this the Hugo's Weir site was retained for future use as a gauging weir facility. Hugo's weir is located at the bottom end of the Crocodile River Irrigation Board's area of responsibility and is therefore a logical location for a gauging weir.

Hugo's Weir (Existing Farmer Abstraction Weir)

- ✓ Existing structure could be used with some modification.
- ✓ Evidence of bed rock in the riverbed was observed.
- × Flat and very wide floodplain. Ancillary facilities will need to be located very far from the weir to ensure they are above flood levels. Rip-rap has been used along the right bank to counter outflanking. Evidence of the onset of outflanking along both riverbanks is visible.
- × River course changes possible. High flood secondary channel noted along right bank.
- × Existing structure damaged and will require repairs if to be used as gauging weir.
- × Various irrigated lands on the upstream and adjacent floodplain which will be affected by raised water levels.

Table 3-5: Evaluation Criteria Assessment for Hugo's 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	No
(4)	Narrow river valley or flood plain	No, very wide
(5)	Potential for outflanking to be manageable	No, both banks problematic
(6)	Narrow river channel	Reasonable
(7)	Good founding conditions (rock)	Some outcrops noted
(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, 4th furthest site
(11)	Proximity of access roads, power lines, etc.	Yes
(12)	Upstream infrastructure affected by higher flood levels	Possible, manageable
(13)	Potential for flood damage	High, in middle of flood plain

Due to non-compliance with evaluation criteria (3), (4), (5) and (13) this site was discarded.

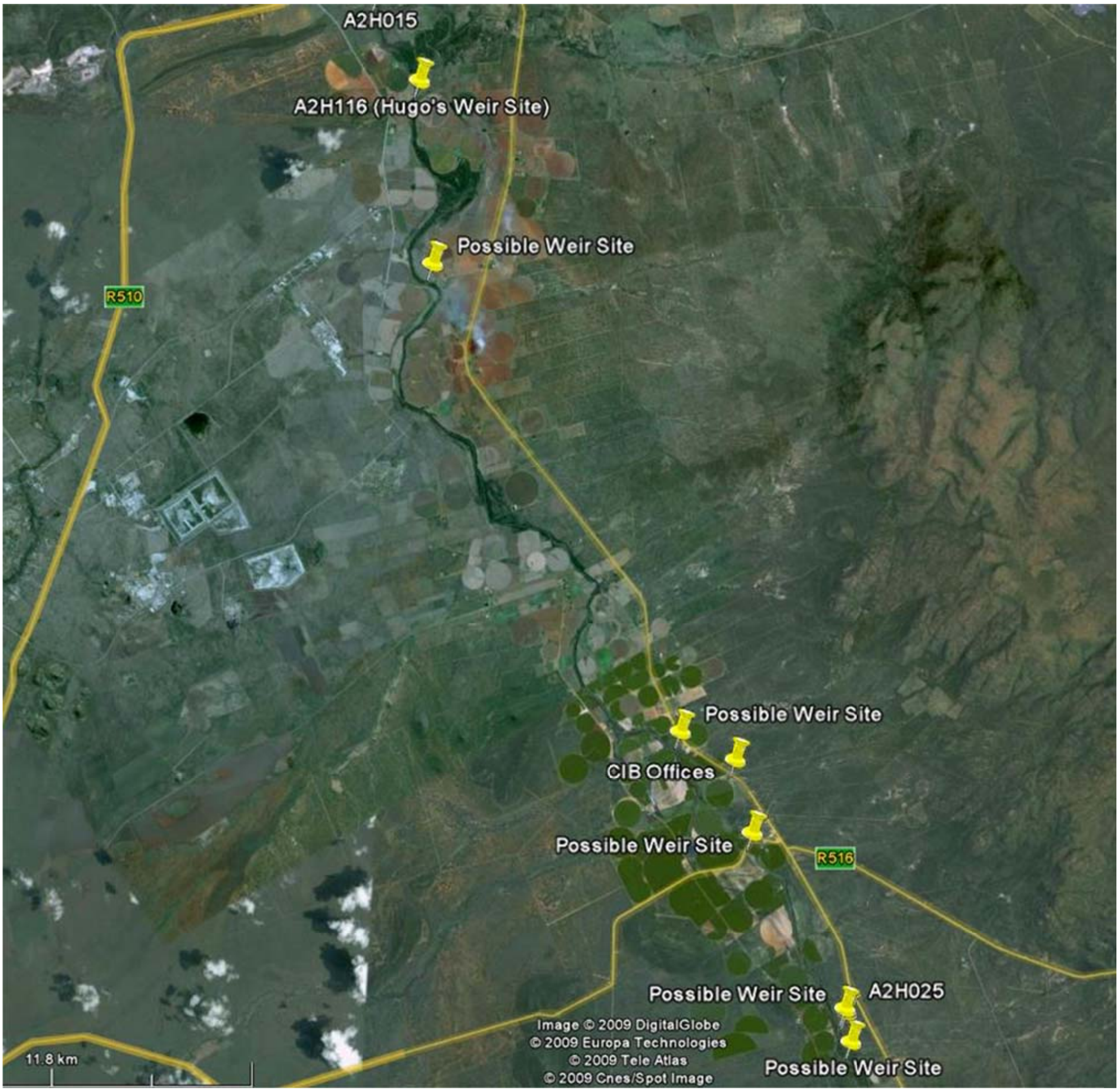

Figure 3-2: Crocodile River (West) - reach between Koedoeskop and Hugo's Weir

Reach between Atlanta Gauging Weir and Koedoeskop. As can be seen in Figure 3-3 below a total of nine weir sites were initially identified and tested against the site evaluation criteria detailed above. All the sites but the Boschkop site were discarded on the basis of evaluation criteria (1) for the first 5 sites and (4), (5), (7), (12) and (13) for the others. Along the Boschkop reach only the original dam site met with the key evaluation criteria. The idea of converting the existing Nooitgedacht gauging weir into an abstraction facility was also considered because of the excellent founding conditions (gauging weir founded on rock). This site was the only promising site with good bed rock visible across the entire cross section.

Boschkop Site (Original Dam Site)

- ✓ Located on a suitable bend in the river.
- ✓ Deep (approximately 12m) 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 de-silting works and high lift pump station balancing dam conveniently close to the weir site.
- ✓ Rock outcrops (dolomite) in right riverbank indicating possible good founding conditions for low-lift pump station.
- ✓ Deep channel and steep floodplains allow for close placement of sedimentation pond and high lift pump station above the flood lines.
- ✓ 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 3-6: 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 in RB
(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.	Ye.
(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), (7) and (9).

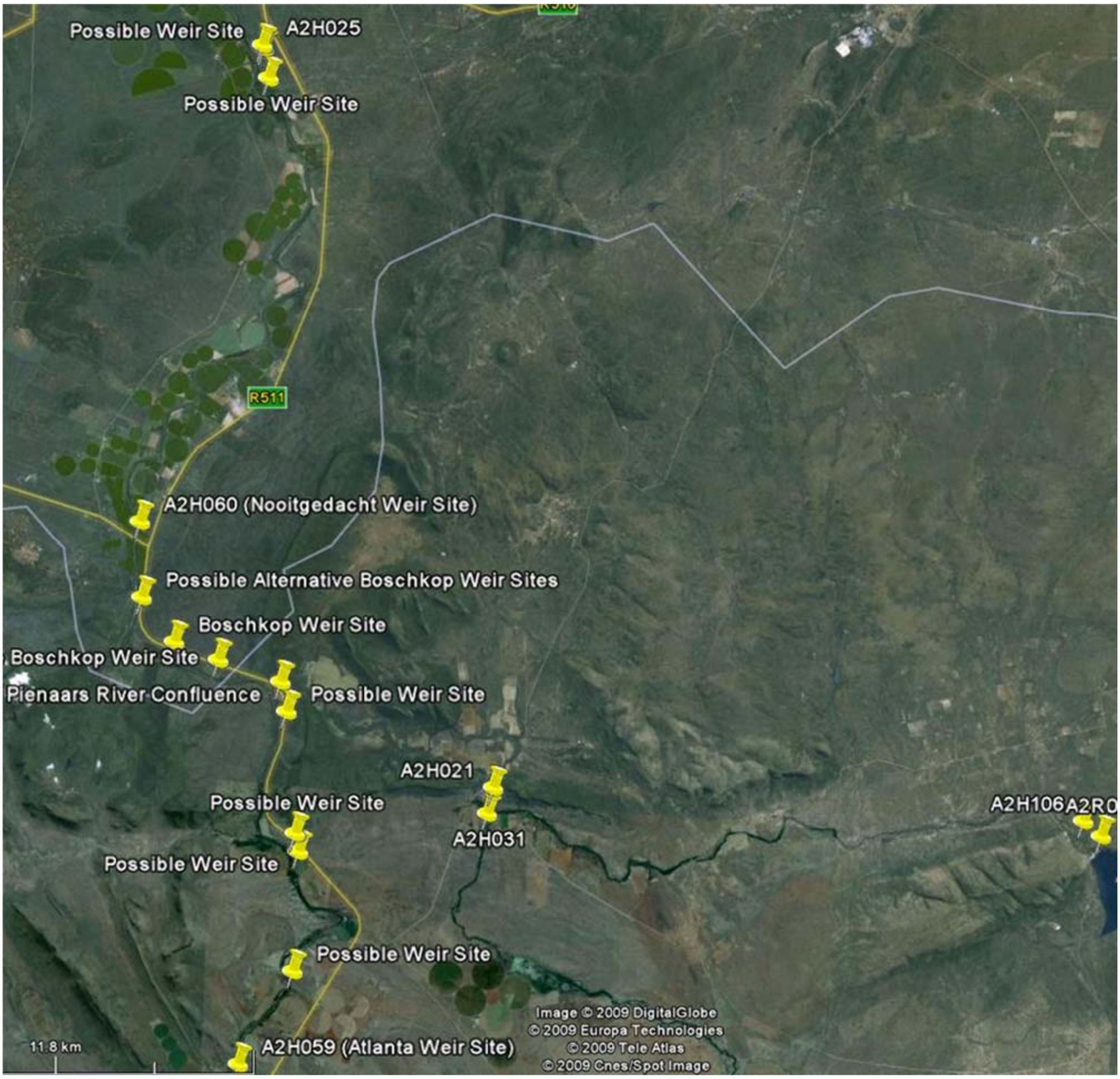

Figure 3-3: Crocodile River (West) – reach between Atlanta Weir and Koedoeskop

Nooitgedacht DWA Gauging Weir

- ✓ Existing structure could be used with some modification.
- × Existing provincial road bridge may need additional erosion protection.
- × Flat and wide floodplain. Ancillary facilities will need to be located some distance from the weir to ensure they are above flood levels (may well be on the other side of the R510 provincial road).
- × The ancillary facilities cannot be placed on the same side of the river as the abstraction works, and will require additional pipework across the river.
- × Various irrigated lands on the upstream floodplain which will be affected by raised water levels.

A site downstream of the provincial road bridge also offered possibilities, but a wider river channel and insufficient space between the bridge and the end of the rock shelf counted against this site option. This site would also have forced the abstraction works onto the wrong side of the river. Although the riverbed sloped fairly steeply through the bridge, there was a distinct possibility that the weir would have had a detrimental effect on the performance of the gauging weir.

Table 3-7: Evaluation Criteria Assessment for Nooitgedacht Weir Site

Criterion No.	Description	Comments
(1)	Downstream of Pienaars River confluence	Yes
(2)	Abstraction works on outside of river bend	No, unless d/s site is selected
(3)	Abstraction works on same side of river as pipeline	Yes
(4)	Narrow river valley or flood plain	No, wide
(5)	Potential for outflanking to be manageable	No, both banks problematic
(6)	Narrow river channel	Reasonable
(7)	Good founding conditions (rock)	Good bed rock noted
(8)	Small weir basin	Yes, in deep river channel
(9)	Pipeline length to users as short as possible	No, 2nd longest pipeline
(10)	Upstream river length as short as possible to curtail losses	Yes, close to upstream dams
(11)	Proximity of access roads, power lines, etc.	Yes
(12)	Upstream infrastructure affected by higher flood levels	Possible, manageable
(13)	Potential for flood damage	Moderate, in flood plain

Due to non-compliance with evaluation criteria (2), (4), (5) and (13) this site was not recommended.

Reach between Roodekopjes Dam and Atlanta Gauging Weir. As can be seen in Figure 3-4 below a number of possible weir sites were initially identified and tested against the site evaluation criteria detailed above. All the sites were discarded on the basis of evaluation criteria (1).

Summary

All the weir sites along the Crocodile River (West) are located in an area of moderate seismicity.

Based on the initial scoping and visits to the respective sites, the following two abstraction locations were identified as viable for further consideration during the pre-feasibility stage of the project:

- Boschkop Site: Least potential river water losses.
- Vlieëpoort Site: Shortest transfer pipeline route.

Figure 3-4: Crocodile River (West) – reach between Roodekopjes Dam and Atlanta Weir

MOKOLO RIVER

Reach between Mokolo Dam and Lephalale. The main advantage of using the Mokolo River as a conveyance was that the abstraction works could be located close to the Zeeland Water Treatment Works (WTWs). A saving of more than 50% of the Mokolo Dam – Zeeland pipeline option (following the Exxaro pipeline route) would have been possible. River losses would however count against the use of the river as a conveyance as is discussed in Section 8.2 and in Supporting Report 5[4]: Mokolo River Development Options.

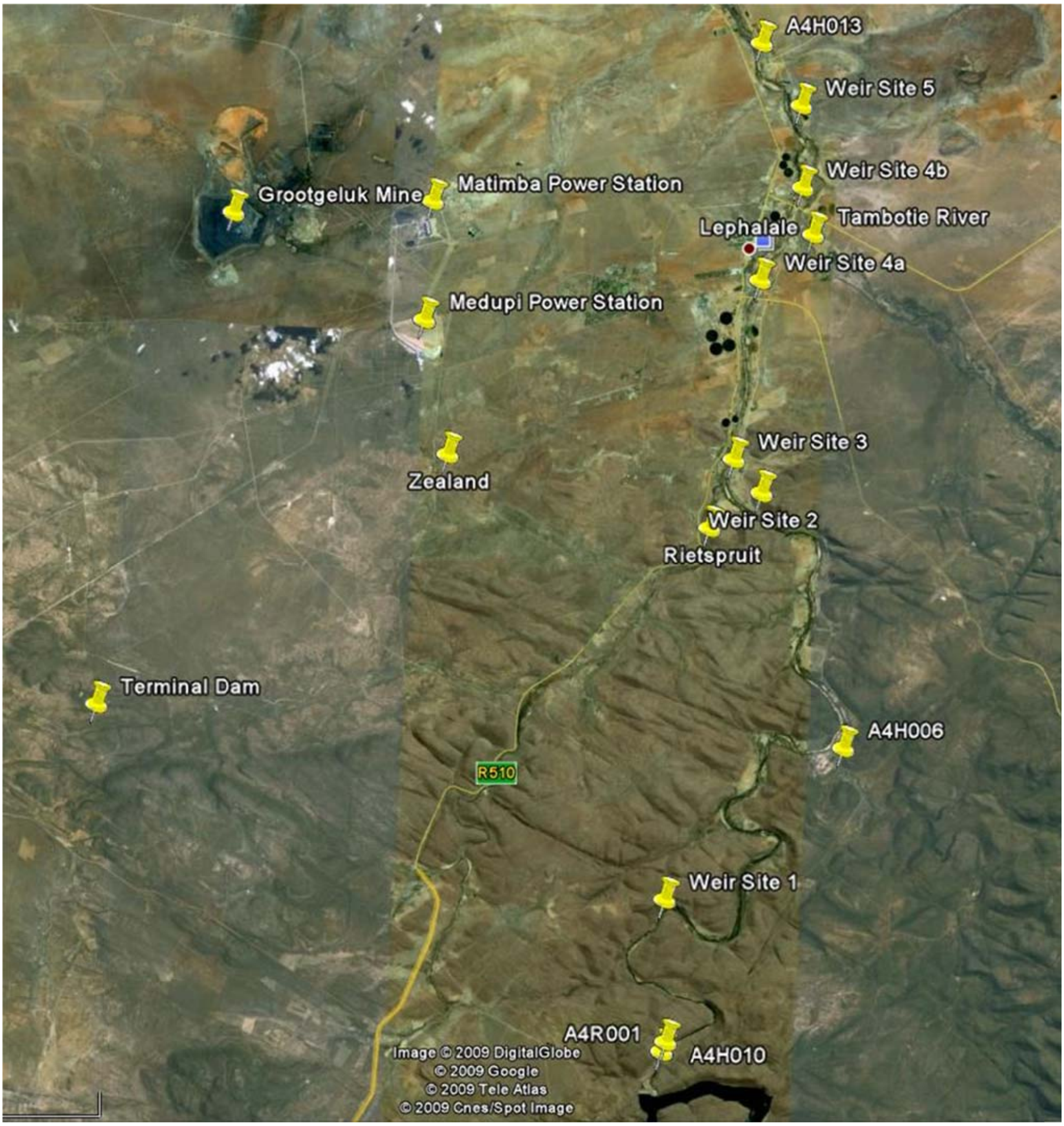

Figure 3-5: Possible Weir Sites identified along the Mokolo River

As can be seen in Figure 3-5, a total of six weir sites were initially identified and tested against the site evaluation criteria detailed above. All the sites around Lephalale (Sites 4a, 4b and 5) were discarded on the basis of evaluation criteria (4), (5), (6), (7), (12) and (13). The first site, Site 1, in the Mokolo gorge failed criterion (9), leaving only Sites 2 and 3 as potentially useful sites.

Briefly each of the weir sites can be described as follows:

- **Weir Site 1:**
  Approximately 10 km downstream of Mokolo Dam, this site will require a roughly 110 m long structure, with apparently relatively good founding conditions, but the resulting long length of pipeline, when compared with Sites 2 and 3, being the fatal flaw. The total length of pipeline required for the Site 1 solution is similar to the length required for the Mokolo Dam – Zeeland solution.

- **Weir Site 2:**
  This site, located approximately 42 km downstream of Mokolo Dam, will require a roughly 130 m long structure with possibly poor founding conditions and high risk of outflanking as evidenced by a secondary river channel between the river proper and the road running along the right bank. Access to the site and the pipe line route will be across the river floodplain and an all-weather crossing of the Rietspruit would be required for both the access road and pipeline. Because of space restrictions the de-silting works, balancing dam and high-lift pump station will have to be located remotely on the Zeeland side of the R510.

- **Weir Site 3:**
  The site, 2.3 km further downstream from Site 2 (and 250 m downstream of the Rietspruit confluence), is located at an existing district road crossing linking the R510 and R33. Although the structure would be some 170 m long the risk of outflanking is much reduced as higher ground is gained relatively quickly on both river banks. This location would result in a saving of approximately 2 km of pipeline when compared to the Site 2 arrangement. The district road crossing would be indicative of possibly better founding conditions. This is the preferred site.

  Note that this weir option is discussed in more detail in Supporting Report 5[(4)] (named the Rivers Bend Weir in that report).

- **Weir Sites 4a and 4b:**
  Possible sites at the R518 and R33 road crossings near Lephalale will require a 200 m wide structure on deep alluvial sand beds in both cases. These weirs would have a negative impact on river flood levels due to the shallow river channel and flatness of the surrounding terrain, with the R33 position somewhat better, being upstream of the town. The increased length of pipeline to Zeeland counted against these locations.

- **Weir Site 5:**
  A site immediately downstream of the Tambotie River confluence and some 3 km downstream of the R518 crossing which could utilise the additional yield form the Tambotie River. A narrower structure, some 120 m long would be possible, but again the increased length of pipeline to Zeeland counts against this location.

Site 3 is recommended on the basis of the present available information, striking a balance the delivery pipeline length and less than ideal site conditions.

All the weir sites are located in an area of low seismicity.

3.1.2. Weirs and Abstraction Works

As discussed above, a number of river abstraction works options were considered before agreeing on the proposed arrangements depicted in Section 12. These options included a diversion weir and:

- Floating platform intake;
- Fixed platform intake above the riverbank;
- Off-channel reservoir with a channel connection to the river; and
- Fixed intake facility in the river.

Sediment bed load, design flood levels and the nature of the riverbanks dictated against any of the above options and an arrangement that have worked well in similar conditions encountered along the Berg and Olifants Rivers in the Western Cape and Limpopo respectively was adopted. The arrangement adopted consists of:

a) Mass concrete gravity type Diversion Weir with ogee and roller bucket spillway, with Recommended Design Discharge (RDD) the 1:100 year flood and Safety Evaluation Flood (SEF) the RMF.

b) Gravel Traps 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 riverbank.

c) Low pressure pipeline to the de-silting works.

d) De-silting Works with flushing facility located near the low-lift pump station, but above the PMF level.

e) A gravity pipeline between the De-silting Works and a balancing dam.

f) Balancing reservoir or forebay to supply the adjacent high-lift pump station. The balancing dam will also be equipped with a sediment flushing facility although only infrequent use, perhaps once every 10 years, is expected.

The diversion weir design will require careful attention as no rock outcrops in the riverbed were observed at any of the preferred abstraction sites. Founding will in all likelihood have to be achieved on deep sands overlying the bedrock. A design based on foundation pre-treatment with jet-grouting is envisaged.

Weir types that were considered prior to adoption of the mass gravity type included buttress weirs, rock weirs and Ambughler type weirs (pre-cast planks on concrete buttress sections on flexible foundations). The mass gravity concrete weir was selected on the basis of superior stability characteristics under deep submerged flood conditions. Overspill Crest (OC) configurations considered included ogee, broad crested and crump shapes. For the moment the ogee type OC has been selected because of its slightly superior discharge capacity in relation to a crump shape.

The abstraction weir will be located on a river bend with the gravel trap and low-lift pump station on the outside of the bend. The weir itself will be orientated at right angles to the river.

3.1.3. River Abstraction Pump Stations

The following criteria were applied in order to achieve the required level of reliability in the design and operation of pumps:

- Pump stations were sited such that the pump station building and all associated ancillary structures would experience no risk from natural flood waters. In the case of river abstraction pumping stations, similarly all electrical switchgear will be located above the PMF level.
- The external power supply to a pump station site must be reliable and the risk to the power lines from natural flooding, bush fires and lightning must be minimised.
- De-silting structures were provided to remove all sediment particles up to 0.07 mm diameter upstream of the high lift pump stations transferring water from the Crocodile (West) and Mokolo Rivers.
- Security: Pump station installations will be secured and hardened to National Key Point (NKP) requirements.

3.1.4. Terminal dam Sites Evaluation Criteria

A comparison of the following characteristics was made to select the preferred site from the available options:

a) Location of site, river bed elevation and maximum practical FSL, with reference to required hydraulics of the transfer system.

b) Water depth to surface area and gross storage capacity relationships for the sites within the possible required live storage range of 8 to 12 Million m^3.

c) Spillway sizing as determined by the RDD and the SED generated by the dam's catchment and the corresponding freeboard requirements; options include side channel spillways cut in rock, use of natural saddles depending on height and location of embankment, central overflow and drop-inlet/siphon type spillway designs.

d) Availability of suitable construction materials: Local soils are listed as sandy with clay content <15%. The rock type is predominantly Waterberg series sandstone and quartzite displaying a high degree of jointing/fracturing.

e) Geology and founding conditions.

f) Potential basin seepage losses: This needs to be investigated due to the apparent fractured nature of the bed rock and sandy nature of the in-situ soils at all sites.

g) Type of embankment dam suited to the site: From the available GIS data and the site visit, the availability of hard rock and sandy soils, but lack of suitable clay deposits, in the area suggests that the dam type options are concrete faced rock-fill, asphalt faced rock-fill and rollcrete. For the purpose of comparing the sites, a typical concrete faced rockfill dam design was used.

h) Embankment quantities to height relationships based on rockfill embankment with 1:1.75 U/S slope and 1:1.5 D/S slope and 7m crest width since the dam sites are located in an area of low seismicity. Quantities included rockfill, upstream lining and foundation works as the main cost variables per site.

i) Cost of construction to height of embankment curves was used to correlate cost of construction with capacity.

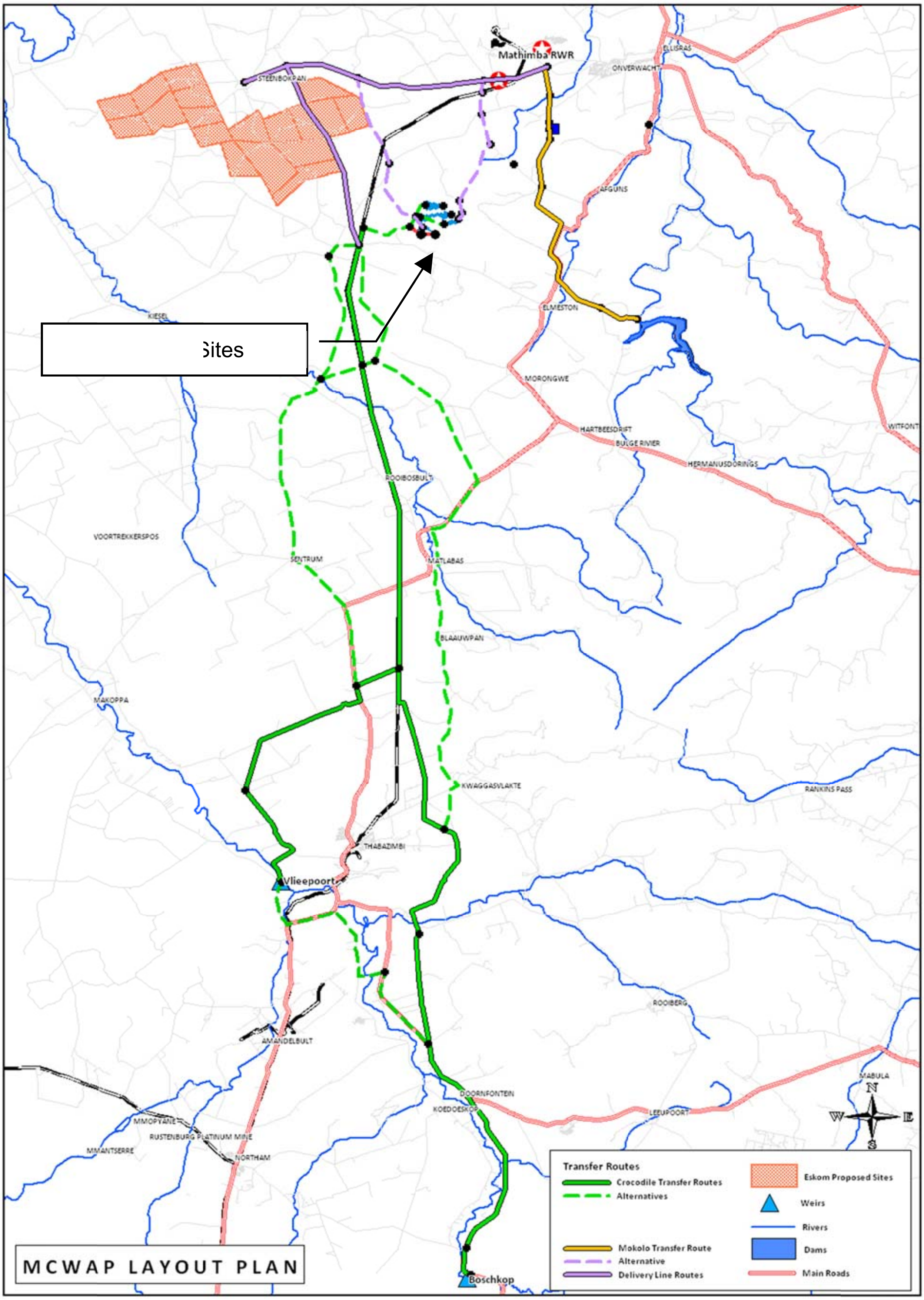

Figure 3-6: MCWAP Layout Plan Showing Terminal Dam Sites

j) Cost of construction to dam capacity curves for the sites was compared to determine the most cost efficient site.

k) Environmental considerations and relocation of red data listed flora may be a factor in determining basin clearing costs and obtaining a positive Record of Decisions (ROD).

l) Site access and pipeline routes vary between sites and were consequently also considered in the site selection process.

3.1.5. Terminal Dam/Reservoir Options Investigated

After consideration of the water requirements together with the geographic locality of the major bulk consumers, it appeared that the centre of gravity of supply had moved west to Steenbokpan. This can be seen from the water requirements tables where the Lephalale and Steenbokpan demand centre figures are:

	Total Annual Demand
Lephalale Demand Centre	: 55 x 10^6 m^3/ a
Steenbokpan Demand Centre	: 173 x 10^6 m^3/ a
Total	: 228 x 10^6 m^3/ a

If the Mokolo Dam available yield of approximately 28.7 x 10^6 m^3/a (excluding irrigation) is deducted from the Lephalale demand centre, the consumption ratio between the two centres is:

$$\frac{26 \times 10^6\ m^3/a}{173 \times 10^6\ m^3/a} = 15\%$$

Only 15% of the Crocodile River (West) transferred water needs to be supplied to the Lephalale area and 85% to the Steenbokpan area. This means that the Crocodile River (West) rising main close to the terminal dam area could move westwards, thus reducing the total length of the rising main significantly.

A number of alternatives to the terminal dam were investigated during the course of the study. The following options were considered, namely:

Option 1: Terminal Dam

The terminal dam options have the following advantages:

- Water can gravitate from here to all the consumers, i.e. saving on pumping costs. May require some boosting when dam level is low.
- The terminal dam option will be considerably cheaper than the on-site terminal reservoirs option (see Section 10: Capital Costs).
- The pumping system from the Crocodile River (West) will be very easy to manage and operate.

The locations of the terminal dam options investigated are shown in Figures 3-6 and 3-7.

Schematic drawings showing typical details of the terminal dams are included in **Appendix B**.

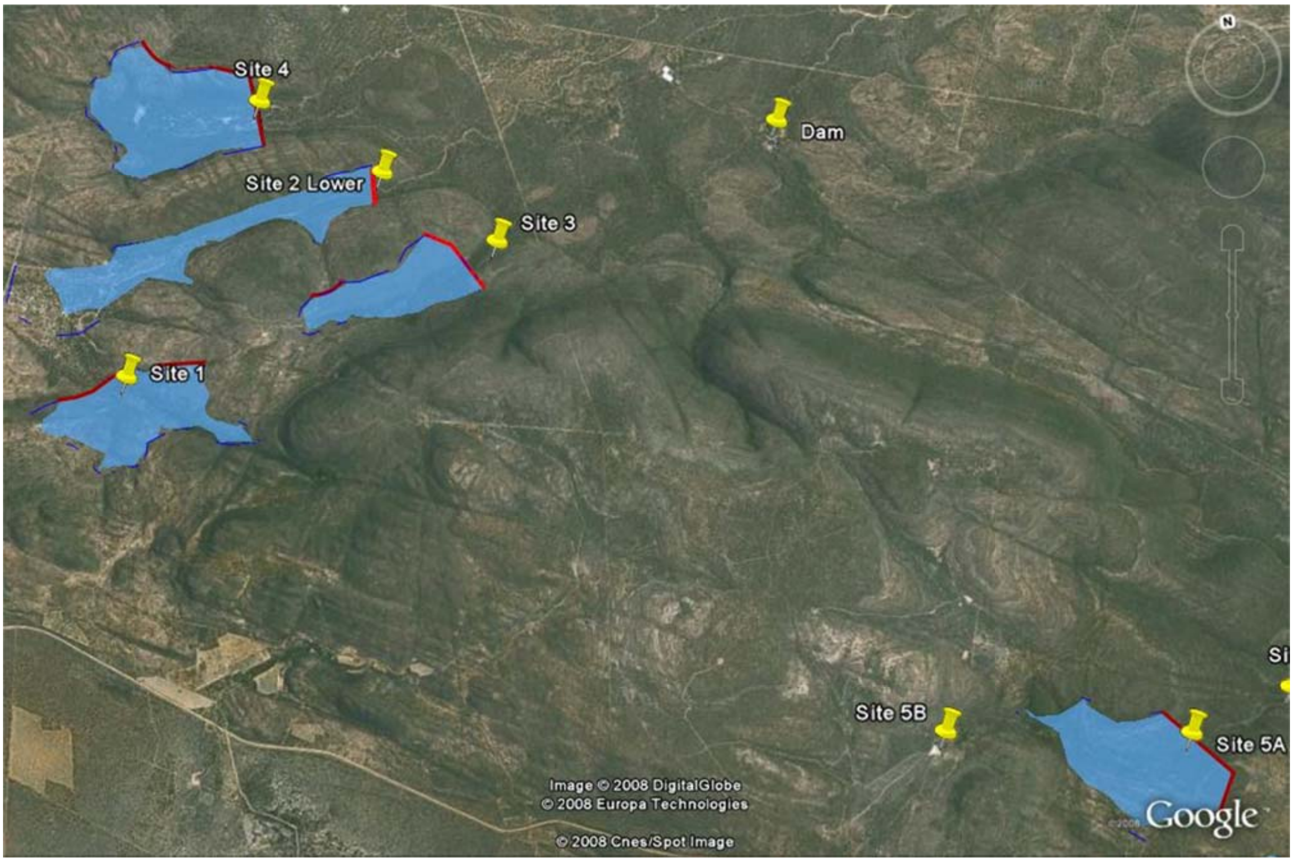

Figure 3-7: Terminal Dam Sites that were investigated

<u>Option 2: Multiple Terminal Reservoirs</u>

This option comprises the Crocodile River (West) transfer pipeline feeding into an operational reservoir (approximately 12 km due west of the present terminal dam sites) from where a gravity pipeline will feed multiple user reservoirs, one at each consumer end. These can either be owned and operated by DWA or owned and operated the by consumers.

The advantages and disadvantages of this option are listed below:

Advantages:

- The system retains the simplicity of operation of the terminal dam option.
- The overall pipeline lengths and costs could be shorter and cheaper than via the Terminal dam option.
- The water can gravitate from the operational reservoir (assume 24 hours storage) to the on-site consumer terminal reservoirs.
- The overall impact on the environment will be less than for the terminal dam option, and will be concentrated closer to the mining and other industrial areas.
- With approximately 18 days reserve storage in the consumers on-site terminal reservoirs, the provision of 9 days storage at the end of the delivery gravity pipelines system from the terminal dam will not be required. This option does reduce the overall storage required on the scheme by 9 days.

- It is possible that there could be a saving in total pumping energy costs, i.e. pumping from the Crocodile River (West) into the operational reservoir instead of into the terminal dam. Refer to Supporting Report 6(5): Water Transfer Scheme Options for further details.

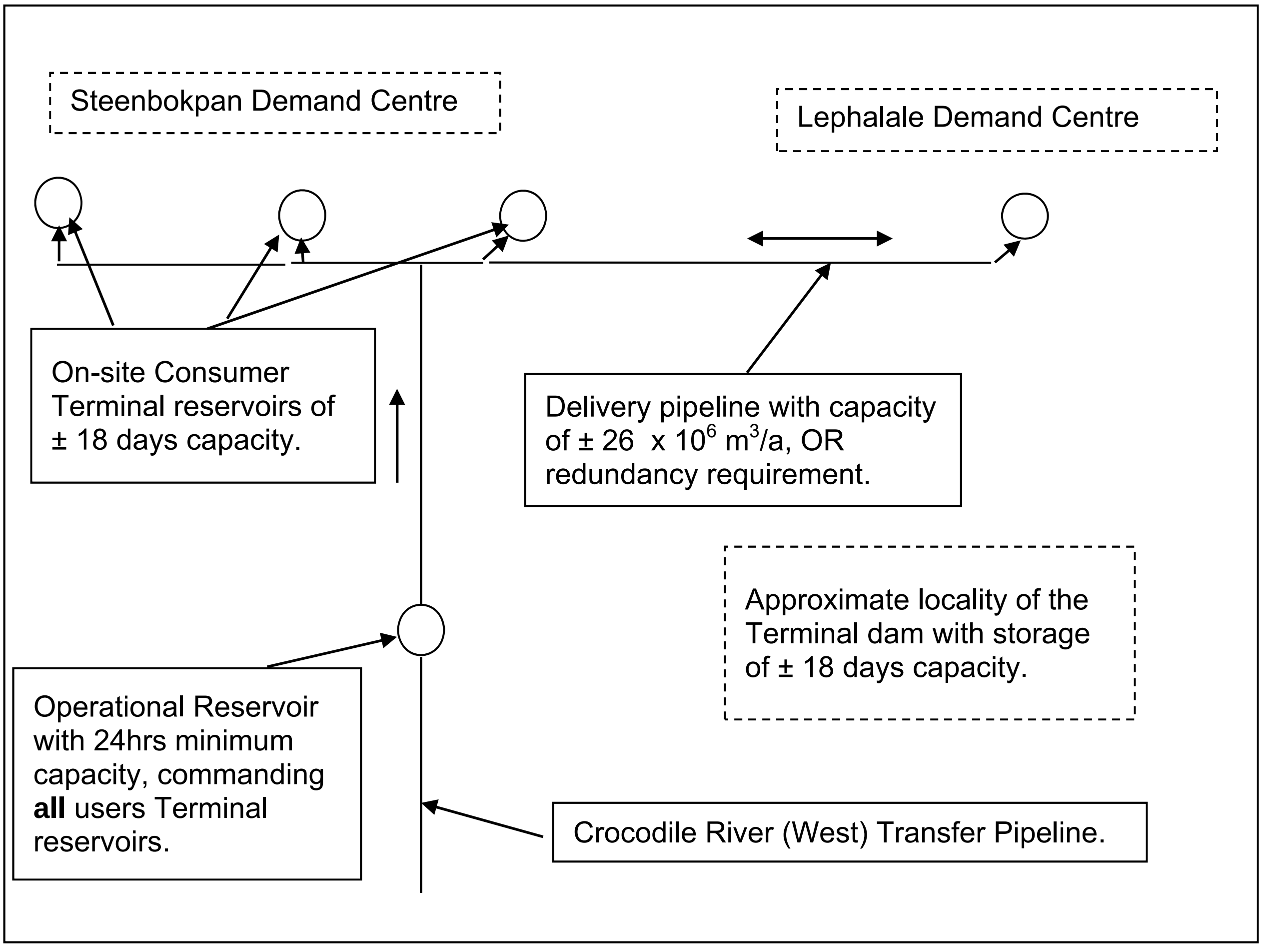

Figure 3-8: Option 2 System Operation Schematic

- Pumping from the Crocodile River (West) into the terminal reservoirs will be a bit more complex than into the terminal dam, but manageable by controlling the flow into these reservoirs.
- It might be necessary to pump the water that must be provided to the Lephalale area (i.e. Zeeland treatment plant) if it cannot be gravitated from the in-line balancing reservoir to the treatment plant at Zeeland.

<u>Option 3: Large Terminal reservoir at Steenbokpan Demand Centre:</u>

Advantages:

- It avoids the pipeline construction difficulties and additional costs associated with the Terminal dam option.
- It avoids the negative environmental impact of the Terminal dam option.
- The operation is as simple as the Terminal dam option.

Disadvantages:

- Pumping will be required to command the consumers' reservoirs.
- Additional storage (approximately 18 days) will now be required at each consumer off-take to comply with the reliability and redundancy requirements.

Further discussion of Options

To provide further clarification of the alternatives investigated the advantages and disadvantages of each particular alternative are further elaborated on in the tables below.

Table 3-8: Evaluation of Terminal Dam Alternatives. Terminal Dam on the Farm Witvogelfontein as was specified in the RFP and Inception Report

Option 1: TERMINAL DAM	
For	**Against**
1. Cost of preferred terminal dam at Southern Site 1 will be R332 Million for 18 days storage.	1. Cost of cheapest terminal dam (at central Site 2 in front of farmer's lodge) will be R208 Million for 18 days storage, but this site is not judged to be acceptable due to the impact on the game lodge and surroundings (subjective).
2. Will allow for gravity delivery line to consumers. May require some boosting when dam level is low.	2. Sites could present environmental issues, especially Site 2 where relocation of the game lodge could result in significant additional costs.
3. The Site 3 terminal dam has the smallest surface area resulting in a saving of approximately R 1,0 Million per annum in the cost of water. At R 416 Million the cost of the Site 3 dam is however the highest.	3. Contamination of ground water and bad smells from the dam could present negative environmental impacts. Additional costs associated with dealing with this could be as high as R 20 Million as a first estimate.
4. On gravity feed delivery pipeline the split to different users is fairly easy to control with flow control/pressure sustaining valves.	4. The surface areas of terminal dams at Sites 1, 2 and 4 are somewhat smaller than that of the terminal reservoir (100 000 m^2 or 10% smaller).
5. Each user must supply its own on site storage for peaks, redundancy and reliability for possibly 9 days (Zeeland and Grootegeluk may be exceptions).	5. Water quality management will become a larger task as the surface area increases. Only Site 3 has a distinct advantage over the terminal reservoir option.
6. The terminal dam sites are located in a mountainous area implying that leakage from the dam basin would be limited to leakage at geological features that can be dealt with fairly cost-effectively.	6. Pumping head to terminal dam is higher than some other options (± 60 m).
	7. Pipeline routes to and from dams will be costly as will access arrangements. Farm access roads will also need to be relocated.

Table 3-9: A Single Terminal Reservoir with the same capacity as the Terminal Dam, but located at the Centroid of the Users near Steenbokpan

Option 2: TERMINAL RESERVOIR AT CENTROID OF END USERS	
For	**Against**
1. Will place terminal storage at the end of the pipeline and close to the consumers.	1. Cost of terminal reservoir is approximately R1 140 million for 18 days storage.
2. Has a slightly larger surface area when compared to the preferred Terminal dam at Site 1 and will cost in the order of R800 000 per annum more in evaporation losses.	2. Depending on geological conditions making the dam watertight could cost somewhat more, as much as R100 million if entire dam needs to be lined.
3. Very easy pipeline access to and from dam.	3. Potential overflow from the reservoir in case of undetected system operation failure could be as high as 8 m^3/s. Not an insignificant flow to deal with when receiving rivers are not close by. Depending on conditions on site this could mean that an overflow management facility may also need to be provided.
	4. Dam will rise approximately 12 m above surrounding plain. Environmental issues regarding height and footprint of approximately 1500 x 750 m could result.
	5. Water quality management will become a much larger task because of the large surface area. A non-ideal dam shape (from cut-fill point of view) in the form of a pointed ellipse could be required.
	6. The terminal reservoir will have surface area of around 1 1000 000 m^2. This is approximately double the surface area of the Site 3 Terminal dam resulting in additional evaporation losses amounting to as much as R1,6 Million per annum.

Table 3-10: On-Site Terminal Reservoirs at each User

Option 3: ON-SITE TERMINAL RESERVOIRS	
For	**Against**
1. Will place terminal reservoirs close to the main consumers.	1. Each user will be directly responsible for the capital, operation and maintenance costs of their terminal reservoirs, leading to inefficiencies.
2. No dams are required in ecologically sensitive areas.	2. The control over the splitting of the water supply to the different users will be complex due to the fact that it is directly linked to the pumps in the pump station at the operational reservoir. The duty point of the pumps and the number of pumps running will vary depending on the number of users

Option 3: ON-SITE TERMINAL RESERVOIRS	
For	**Against**
	requiring water at any specific time. The control of pumps and the operating of valves will require complex control systems. On a gravity system the system will be downstream controlled, which will be much simpler to manage.
3. Each user will supply his own on site storage for peaks, redundancy and reliability for the full period of 18 days (Zeeland and Grootegeluk may be exceptions).	3. The combined surface area of the on-site terminal reservoirs (1 700 000 m^2) will be larger than the single terminal reservoir (1 100 000 m^2) and the terminal dam (920 000 m^2) resulting in an increased amount of evaporation losses. In terms of cost of water losses the on-site terminal reservoirs will cost approximately R 3,6 Million per annum more than the terminal dam.
4. 9 Days storage is saved by providing on-site terminal reservoirs with 18 days storage instead of 18 days for the terminal dam plus 9 days for the on-site user storage.	

Conclusion

With the move of the user requirements centre of gravity towards the west the need for the terminal dam has fallen away and a terminal/balancing reservoir is favoured. As the users will be supplying their own 9 day on-site storage facilities anyway the need for an expensive single large terminal reservoir is obviated by each user upgrading his terminal storage facility capacity from 9 days to 18 days storage. The users have agreed to this arrangement.

The on-site terminal reservoirs option will therefore result in a saving of 9 days of storage in the system with concomitant savings in capital costs. The on-site terminal reservoirs option (Option 3) is therefore recommended.

3.1.6. Mokolo Dam Options

The present systems analysis undertaken by WRP has indicated that little or no benefit would be gained by the raising of Mokolo Dam; partly because of the required changes to the IFR requirements should the dam be raised. Only a basic assessment of the dam raising options has therefore been done to assist with the evaluation of project reliability and redundancy options.

The Dam Safety Office of DWA has classified the Mokolo Dam as a Category III dam with a high hazard rating. In addition to this it is proposed that the damage caused under extreme flood conditions should also not cause the supply of water from the dam to be interrupted for any prolonged period, because of the strategic importance of most of the water requirements to be supplied by Mokolo Dam. The proposal has particular reference to the erosion donga in the spillway channel and the potential impacts on the operation of the Mokolo Pump Station. On this basis repair of the spillway channel would be recommended regardless of whether any other work on the dam itself is undertaken.

The preliminary engineering geological evaluation of the spillway return channel and the donga will provide the basis for deciding on the remedial measures that may be required. As may be applicable use of mass concrete, rock anchors/bolts, mesh and shotcrete should be made in the proposed remedial measures. The remedial measures were not investigated beyond the concept stage as DWA had advised that the remedial works will be undertaken by them as part of their responsibilities in terms the Dam Safety Regulations.

The dam raising options that were assessed are:

(1) Raising of FSL without raising the dam embankment. On the basis of preliminary analyses the present total freeboard of 10m is considerably more than what is required. Therefore it is possible to raise the existing FSL to some extent without having to raise the crest of the rockfill embankment. This will avoid the likely problem of not finding sufficient quantities of suitable soil for the clay core within economical haul distances.

(2) Raise the embankment crest by 12.0 m to RL 934.00 corresponding to the deck level of the intake tower.

For the purpose of the Pre-Feasibility Stage the raising of the rockfill embankment will be sized according to the details shown on the original drawings prepared by DWA for the raised embankment. These were based on preliminary designs performed at the time.

For the two raising options two spillway options were also assessed:

(i) A straight uncontrolled concrete ogee type spillway; and

(ii) A reinforced concrete labyrinth spillway. Because of the better discharge characteristics of a labyrinth spillway an approximately 4.5 m to 4.8 m increase in FSL can be achieved depending on the Safety Evaluation Discharge (SED) without raising the embankment crest.

The various spillway options considered included the more classic uncontrolled straight ogee overspill section, labyrinth weir, fuse gates or breaching sections. A gated spillway was not investigated because of the reservations that DWA have about the reliability and use of spillway gates. Furthermore, the strategic nature of the bulk of the water requirements means that a high level of security of supply must be maintained. This is specifically not the case with fuse gates or breaching sections since large volumes of stored water can be lost after high flood conditions. This therefore limits the options to only uncontrolled straight or labyrinth ogee overspill sections.

The straight uncontrolled concrete spillways were sized as for concrete gravity dams. This was in accordance with the Vaal Augmentation Planning Study (VAPS) [(11)] Sizing Guidelines (DWA Directorate Water Resources Planning Report No. PC 000/00/14394).

The walls of the concrete labyrinth spillways have a vertical upstream face and a batter of 1:10 on the downstream side. The heights and preliminary configuration and top width of the walls were according to the hydraulic design guidelines for labyrinth spillways, fitted into the available present spillway width of 200 m and also to comply with the freeboard criteria.

A 15m wide concrete apron was provided across the width of the spillway to prevent erosion of the rock due to the increased energy of the water that will be released over the raised spillway. The apron dimensions were determined in accordance with the VAPS Sizing Guidelines.

The pre-feasibility level sizing of the spillways and associated freeboard for raising Mokolo Dam was based on the SANCOLD, (1991)[9] Guidelines on Safety in Relation to Floods for Category III dams with significant or high hazard rating. The SEF was based on the PMF.

The main purpose of the basic assessments was to develop a parametric relationship between the raised FSL and the cost of raising. The parametric costs together with the storage capacity and yield characteristics of the dam provide the basis for determining the incremental and marginal costs of any additional water secured through raising the FSL.

3.2. Description of Components

3.2.1. Abstraction Works

The locations of the proposed Phase 2 weirs and abstraction works along the Crocodile River (West) are shown in Figure 3-9 and Figure 3-10. Figure 3-11 shows the location of the proposed Phase 1A Works along the Mokolo River. The sites are not ideal, but the most suitable along the river for the reasons discussed in Section 3.1 above.

Figure 3-9: Location of Boschkop Abstraction Works

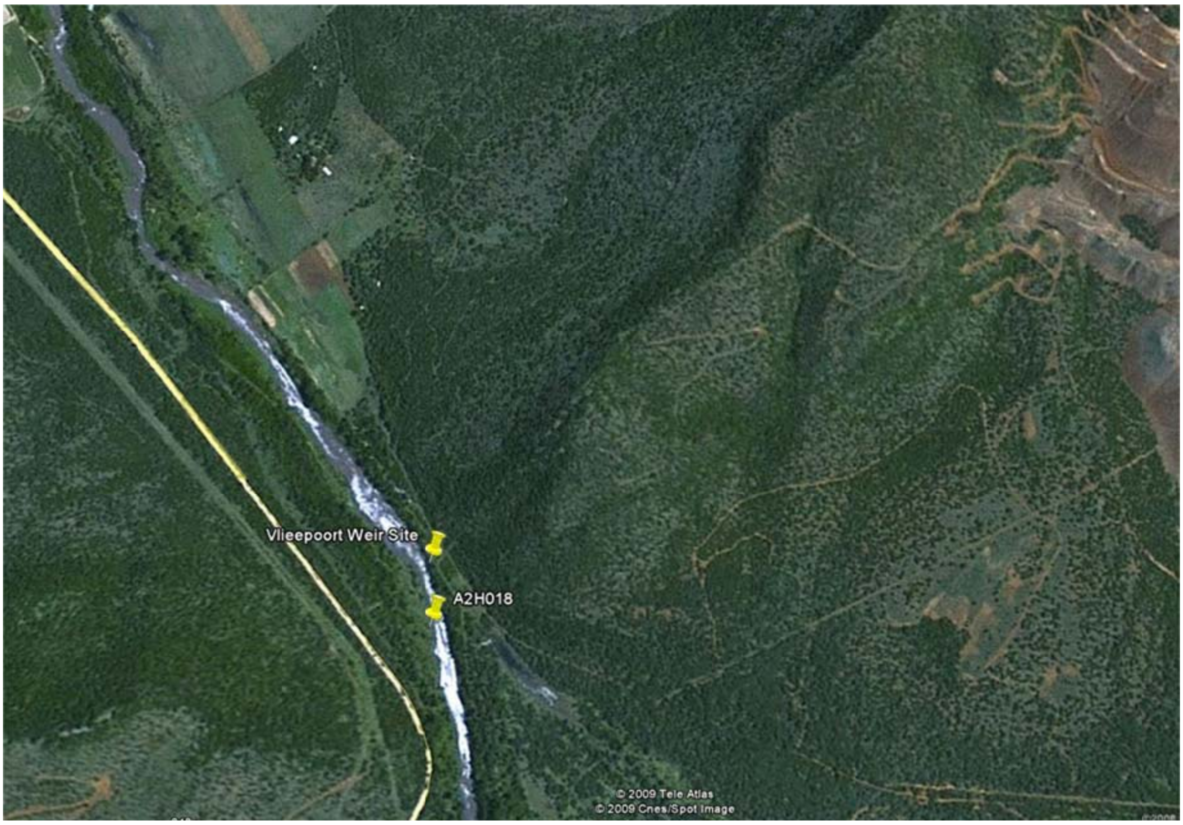

Figure 3-10: Location of Vlieëpoort Abstraction Works

Figure 3-11: Location of Mokolo Abstraction Works (Site 3)

The layouts of the proposed abstraction works at Boschkop, Vlieëpoort and Mokolo River sites are included in **Appendix B**. The abstraction works will consist of the following components:

- Weir across the river to create head for flushing of sediment from the abstraction works. The weir would be about 3 m high, depending on the number of pump bays. The weir has a low notch near the intakes. The weir is not designed for storage and it is assumed it will silt up. Sedimentation will however not affect the abstraction works.
- Gravel trap upstream of the intakes to the pumps to remove coarse sediment. The gravel trap can be flushed by opening the downstream radial gate. The gravel trap is hydraulically steep
- Trash racks upstream of the pump intake canals. The trash racks can be raised for cleaning. The trash racks would be under water.
- The trash rack intake wall is orientated in the flow to create secondary flow currents which would divert coarse sediment (sand and gravel) away from the intakes during floods.
- Pump canals to allow uniform flow conditions at the pumps at an approach flow velocity of less than 0.3 m/s.
- Fine screens will be placed upstream of the pumps.
- Flushing of pump canals will be done by opening vertical gates downstream of the pumps. The pumps will be raised during flushing.
- The pumps must be robust submersible pumps with special impellers that could handle coarse sediment of say 100 mm diameter in case of damage to the screens.

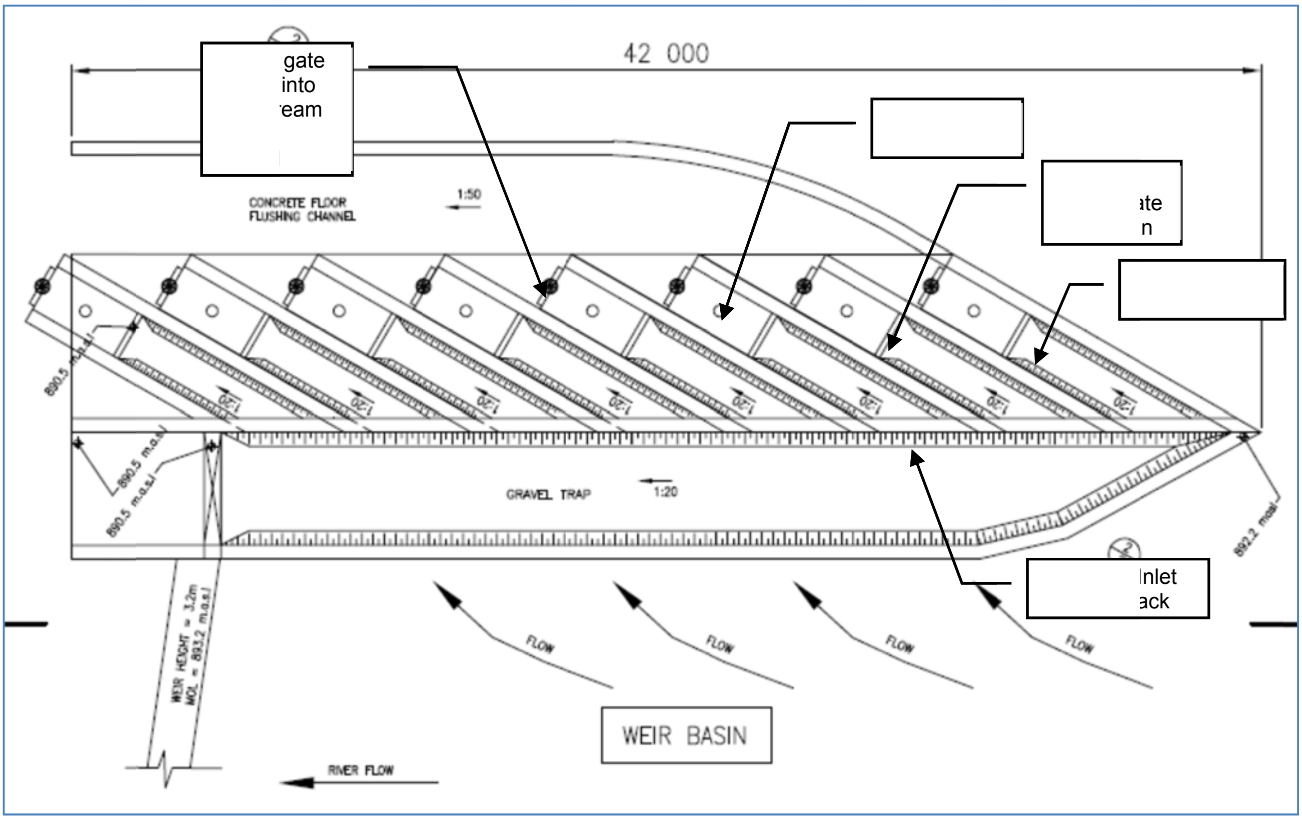

Figure 3-12: Typical Layout Plan of Low-lift Pump Station

- The pumps can be raised for maintenance.
- Extra pump and spares to be stored on site. An extra pumping bay over and above the design requirements will also be provided.
- Energy dissipation at the weir will be by solid roller bucket and riprap downstream.
- A fish way could be added to the weir if required.
- Downstream flows will depend on inflows, abstraction for the MCWAP and IFR requirements. IFR control will be carried out by low level outlets in the weir, the weir low notch, the fish way and the gravel trap operation.
- The pump controls and switchgear should preferably be above the SED water level, or the PMF because of the strategic importance of the MCWAP.
- The top of the concrete at the abstraction works should be at the level of the RDD including freeboard other than surcharge, according to SANCOLD, 1991(9), Guidelines on Safety in Relation to Floods. The structure will be able to withstand the SED or PMF without major damage, due to the strategic importance of the MCWAP.

3.2.2. River Abstraction Pumps (Low-Lift Pump Station)

Abstraction pump station options have been identified for the pre-feasibility stage in conjunction with the most suitable weir and de-silting structure configurations and conditions. Sites were considered based on the following parameters:

- 1st Stage de-gritting is done in the river gravel trap (Refer to Figure 3-12 above for further details).
- 2nd Stage de-silting to a maximum particle size of 0.07 mm will be done in De-Silting Channels which will be located next to the balancing dam (Refer to layout drawings in **Appendix B** for further details).
- The pump controls and electrical switchgear should be above the PMF level. Two layout options were considered; Option 1 in control room above the pump bays and Option 2 in separate facility on the right bank as shown on the drawings. A separate electrical switch yard is also provided for.

Regular flush cleaning of the 2nd stage De-Silting Channels will be required on a regular basis with the intervals dependent on the silt load in the river. Flushing pipelines will return the silt to the river. It may be necessary to provide a sedimentation pond from where the silt can be collected and disposed of, if so required by the Environmental Management Plan (EMP).

Further factors and site considerations that affected the selection of options included:

- Delivery heads and absorbed energy of the abstraction low-lift pump station. Maximum delivery head of pumps will affect the location of the 2nd Stage De-Silting Channels and Balancing Dam.
- Primarily two types of pumps are suitable for this application. These together with their maximum delivery heads are:
  - Submersible pumps : up to 50 m for 1 m^3/sec units
  - Vertical spindle drive pumps : up to approximately 175 m
- The approximate site configuration, with respect to pumping heads, is given below:

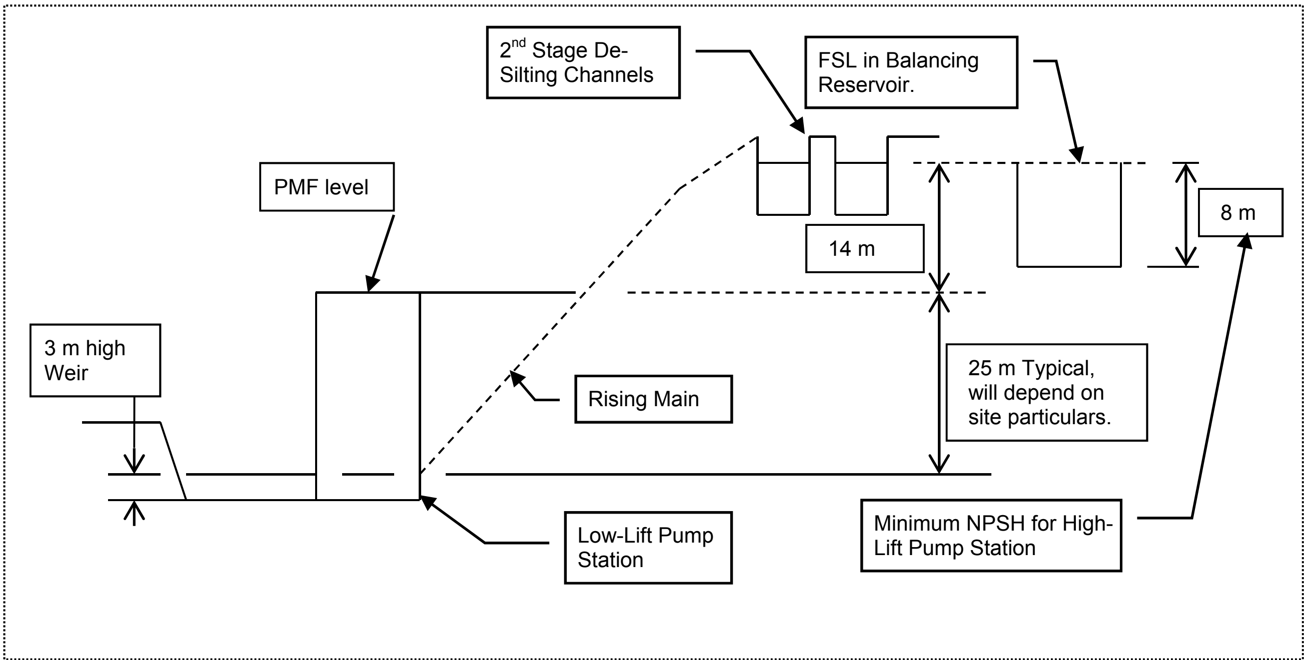

Figure 3-13: Schematic of Pump Station Arrangement

Only submersible pumps were considered for this Study based on the following:

- They are more suitable, particularly for the considerations of reliability.
- With the high anticipated flood levels above the weir crest level, vertical spindle pumps will have unacceptably long pump shaft lengths. This has very specific disadvantages.
- If the same criteria of a minimum of 20% standby capacity are adopted, two additional fully equipped 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 their quick replacement time. Nevertheless, a spare pump bay is provided for anyway in the proposed layouts.

The approximate maximum delivery head for both the Boschkop and Vlieëpoort sites (conclusion after site visit), will be in the order of:

Maximum delivery head: 8 m NPSH for high-lift pumps

plus 6 m from balancing reservoir MOL (minimum operating level) to FSL

plus 5 m allowance for head losses

plus 25 m typical allowance for high flood levels

= ± 44 m

The maximum delivery head for submersible pump = 50 m, therefore feasible.

The estimated absorbed energy requirement for the low-lift abstraction pump station is thus approximately:

P = $1/\eta$ Q g H with Q = maximum Phase 2 flow requirement

= 1/0,85 x 7,4 m^3/s x 9,81 m/s^2 x 50 m

= ± 4,3 MW

3.2.3. Terminal Reservoirs

The client terminal reservoirs will be artificial dams using a waterproofed earthfill embankment, similar to the abstraction weir balancing dam. In the absence of detailed geotechnical data (and the precise locations of these reservoirs) is has been conservatively assumed that no sources of suitably impermeable material would be available to allow for the cheaper zoned embankments solution.

These dams are sized to provide 18 days of average annual water requirement at each of the delivery nodes. Freeboard of 0.5 m was allowed above the FSL. Each terminal reservoir will be subdivided into bays with a width of between 75 m and 105 m. One additional bay over and above the provided 18 days storage will also be provided to cater for one bay to be operational at a time and to be emptied before switching to the next one. The additional bay will also be able to cater for maintenance, since water can be drawn off and replenished on a continuous basis from one bay at a time for as long as it takes to replace all the stored water. This will prevent stagnant areas that could otherwise occur in a single large dam/reservoir and will increase the average retention time slightly, depending on the number of bays.

The inlet to each bay will be by means of a manifold coming off the main delivery pipe. The inflow will then be spread across the width of the bay by using a long weir type structure. The even spread of inflow will once again improve water quality. Multiple outlet pipes will be provided for at each bay.

Refer to the drawings in **Appendix B** for typical details of the proposed client balancing reservoirs.

3.2.4. Raising of Mokolo Dam

Figure 3-14: Crump Weir at Mokolo Dam Spillway

Spillway Channel

Mokolo Dam has a spillway crest consisting of a Crump weir (stepped) (200 m long). The spillway chute is unlined rock excavation and a large flood in 1996 has deepened the scour channel across the chute that developed shortly after the first filling of the dam. This channel is about 20 to 30 m wide and 20 to 30 m deep and could cut back upstream in future floods. The spillway crest is shown in Figure 3-14 and the eroded channel in the chute in Figure 3-15.

The FSL is at 912 masl, while the dam NOC is at 922 masl, which allows 10 m freeboard. In this Study the possible raising of the FSL was considered by raising the spillway crest without raising the embankment.

Figure 3-15: Deep Scour in Spillway Chute.

Spillway Discharge Capacity

The RDD and the SED were considered. The RDD with freeboard was calculated, as well as the SED without freeboard. The dam was classified as follows:

a) Large dam (> 30 m): maximum height 51.22 m

b) Hazard rating: High

Based on the SANCOLD, (1991)[9] Guidelines on Safety in Relation to Floods for Category III dams, the RDD should be the 1:200 year flood, which was obtained from the DWA (2003) study: Mokolo Dam frequency analysis: Estimated flood peaks for required probability. The 1:200 year flood is 2 085 m^3/s.

SANCOLD (1990)[10] on Interim guidelines of Freeboard for dams specifies the SED as the RMF in a higher region. In region K value of 5,2, the SED is therefore 8 060 m^3/s, for a catchment area of 4319 km^2 at the dam.

The spillway has a theoretical Crump weir calculated discharge table (DT) by DWA with a maximum discharge of 2 253 m^3/s at 3.5 m head. The RDD will have damming of 3,34 m based on the DT which converts to a water level in the reservoir of 915,34 masl. A two dimensional mathematical model was, however, set up of the spillway and this model indicated a damming of 3,63 m, i.e. 0,29 m more than the value of the current DT02 of DWA.

The freeboard was calculated based on the SANCOLD (1990)[10] Interim Guidelines of Freeboard for dams and is shown in Table 3-2 for different freeboard combinations. The required freeboard is therefore 5.48 m above FSL. It would therefore be possible to raise the spillway by 4.52 m based on the RDD with freeboard if a Crump weir is used, but the SED also has to be considered. The fetch length is 1,6 km and the design 1:100 year wind speed considered over water 23,7 m/s, which is based on the Milford (1987)(12) map in SANCOLD (1990)[10] on Interim Guidelines of Freeboard for dams.

Table 3-11: Mokolo Dam Freeboard based on SANCOLD (1990)[10]

Combi-nation number	RDD Sur-charge	20-Year flood surcharge (m)	Wind wave and Run-up (m)		Wind setup (m)	Flood surges and seiches (m)	Earth-quake wave (m)	Free-board (m)
			25-year	100-year				
1			0.838		0.011			0.85
2	3.63		0.838		0.011	1		5.48
3		1.91		0.904	0.012	1		3.83
4							2	2.00

Note: *No Landslide wave investigated

The 2D hydrodynamic model was used to simulate damming for the SED. A water level of 920,17 masl was found in the reservoir, which is only 1.83 m below the NOC. It should therefore be possible to increase the FSL by 1.83 m, without raising the dam, if a ogee weir is considered. Higher raising of the spillway crest is however possible if a labyrinth spillway is used.

Possible Raising of the FSL

Raising of the Mokolo Dam FSL by 1.83 m to 913,83 masl by raising the existing Crump weir, will increase the FSC from 145,9 million m^3 (1999 survey) to 161,7 million m^3. This represents an increase of 11 % in storage capacity.

To increase the FSC and discharge capacity further a labyrinth spillway could be used. Typical labyrinth spillway dimensions are indicated in Figure 3-16.

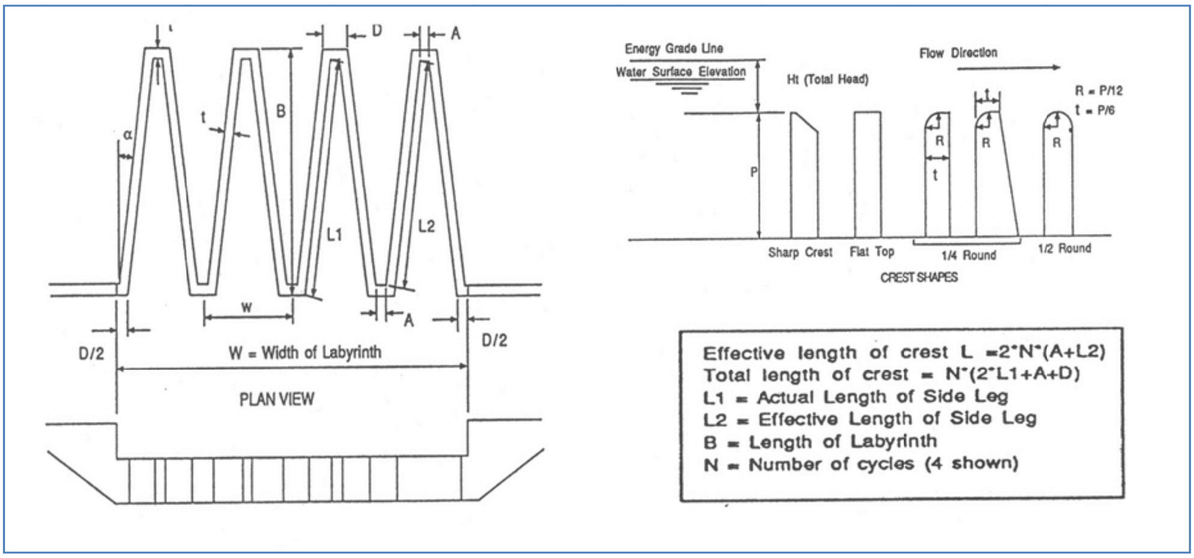

Figure 3-16: Typical Labyrinth Spillway Dimensions

The discharge of the spillway is calculated with the following equation:

$Q = (^2/_3)\ C_D\ L\ (2g)^{0.5}\ H_0^{\ 1.5}$

Table 3-3 indicates the design parameters of a labyrinth spillway which could be used to raise the Mokolo Dam FSL without raising the dam crest (NOC). The current FSL could be raised by 4.8 m and will accommodate the SED and RDD with freeboard. The new FSL would be 916,8 masl, which would give a reservoir storage capacity of 189 Million m^3, compared to the current storage capacity of 145,9 Million m^3. This is an increase in storage capacity of 43 Million m^3 or 30%.

Table 3-12: Labyrinth Spillway Parameters for the SED at Mokolo Dam for a Raised Spillway

Parameter	Values (FSL= 916,8 masl)
α(degrees)	12
D (m)	4,4
A (m)	2
L1 (m)	28,3
L2 (m)	27,13
W (m)	18,18
t (m)	1,2
Actual Crump length (m)	200
N	11
Raise spillway by (m)	4,8
FSL (masl)	916,80
Effective crest length L (m)	640,94
P (m)	6,3
H	5,2

Parameter	Values (FSL= 916,8 masl)
H/P	0,712329
Cd	0,36
Discharge (m^3/s) (SED = 8 060 m^3/s)	8 079
Dam Wall NOCL (masl)	922,0

Further to the above configurations a number of more detailed analyses were undertaken on labyrinth and ogee type spillways. As shown on the drawings in **Appendix B**, three labyrinth options were considered and, in addition two straight uncontrolled concrete ogee type spillway designs were analysed and cost estimates prepared (presented in Section 10 and in Appendix A). The options considered were:

- Option 1: Labyrinth spillway, FSL 916,50 masl, NOCL = 922,0 masl.
- Option 2: Labyrinth spillway, FSL 929,30 masl, NOCL = 934,0 masl.
- Option 3: Labyrinth spillway, FSL 916,80 masl, NOCL = 922,0 masl (similar to the labyrinth configuration discussed above.
- Option 4: Ogee type spillway, FSL 913,83 masl, NOCL = 922,0 masl.
- Option 5: Ogee type spillway, FSL 925,80 masl, NOCL = 934,0 masl.

Higher raising of the spillway and dam is possible, but will be expensive. An analysis was also done for the case of the SEF being equal to the PMF since the flood attenuation is likely to be small, particularly with a labyrinth spillway. For PMF of 10 000 m^3/s it was found that the flood rise would be 5,5 m and therefore the FSL level could still be raised by 4,5 m.

Raising of the spillway crest would increase the risk of retrogressive scour of the spillway chute, which will have to be investigated in more detail for the current and any possible future scenarios if raising is considered.

The storage capacity curve for Mokolo Dam showing the various raising options considered is given in Figure 3-17 below.

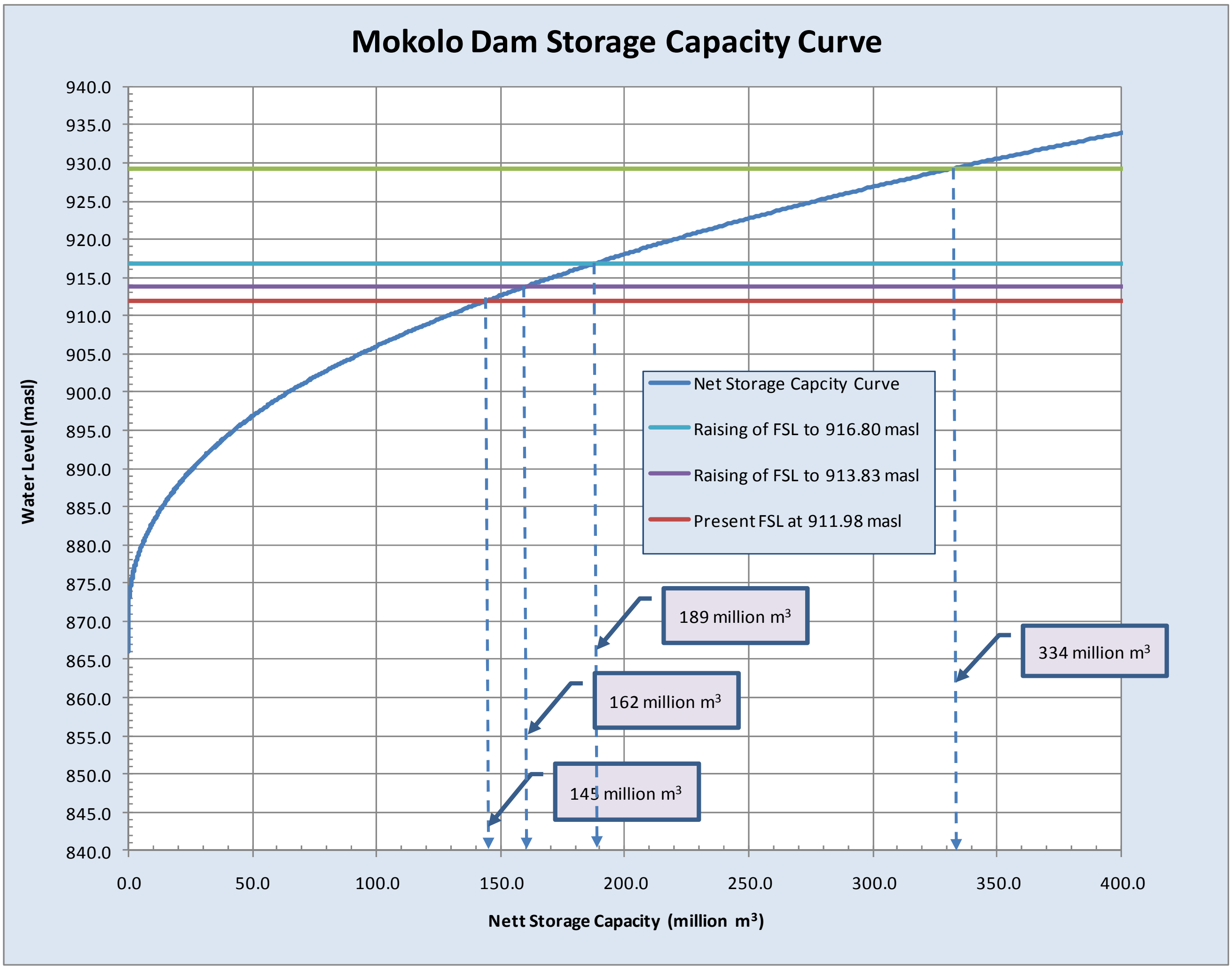

Figure 3-17: Storage Capacity Curve for Mokolo Dam

4. FLOODS AND FLOOD LEVELS

4.1. Design Approach

The determination of design floods and levels was based on a twofold approach covering firstly the operational requirements and, secondly, the structural integrity of the Works. Two sets of criteria were applied in the design of the Abstraction Works:

(1) Reliability Criterion. The reliability criterion was applied to all components of the works that could not tolerate inundation without jeopardising the operational readiness of the works under any circumstances. All these components were located above the PMF level. These components included:

- Mechanical and electrical control equipment of the valves, sluice gates and screens.
- Electrical switch gear and sub-stations.
- Control rooms for the weir, low lift and high lift pump stations.
- Prevention of overtopping of the de-silting works and high lift pump station balancing reservoirs and low-lift pump stations by placing the tops of the structures above the PMF level plus 0.5 m freeboard.

(2) Structural Design Criterion. The structural design criteria applied to the weir structure proper were in accordance with the SANCOLD, 1991 (Floods)(9) recommendations..

(3) Because of the strategic nature of the project a further criterion that must be applied during the Feasibility and Detailed Design stages is to ensure that during the passage of the extreme flood the structural integrity must be retained. Damage must not result in failure of the structure or its functionality by outflanking for example. The PMF will be used as basis for this evaluation.

4.2. Flood Peaks

Flood peaks at the proposed abstraction works sites were estimated by using the Regional Maximum Flood (RMF) method of Kovacs (1987)(13), which would give conservatively high estimates of flood peaks (Table 4-1).

Table 4-1: RMF Method Estimated Flood Peaks

Weir Location	Upstream Catchment Area (km^2)	Q_{100} (m^3/s)	Q_{200} (m^3/s)	RMF (m^3/s)
Boschkop	21 783	4 142	4 779	6 372
Vlieëpoort	28 303	4 995	5 741	7 456
Mokolo Site 2	5 153	4 307	5 025	7 179
Mokolo Site 3	5 693	5 427	5 282	7 545

4.3. Flood Levels at Abstraction Works Sites

Flood levels at the proposed sites were determined on the Crocodile and Mokolo Rivers, with and without the abstraction works. The hydraulic roughness was assumed as Manning's "n" of 0.045 in the main channel and 0.06 on the floodplains. Normal flow depth was assumed far downstream of the weirs. The weirs were assumed to be constructed in steps up the left bank, while the high pump station on the right bank (Crocodile sites) was made high enough according to the Interim guidelines of SANCOLD (1990)[10] on Freeboard for dams design guidelines for a recommended design discharge with freeboard (see Section 4.3). A larger flood (SED) therefore flows across the road and top of the structure on the right bank (Crocodile sites) during the RMF.

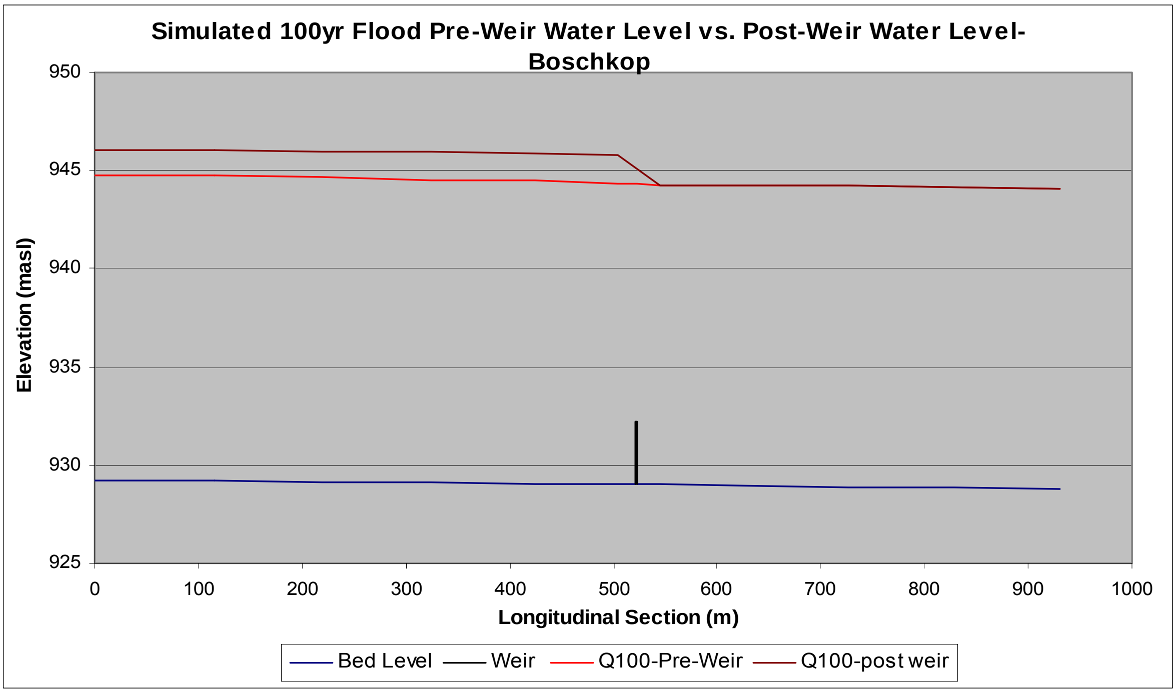

Figure 4-1: 1:100 Year Flood Levels at the Boschkop Site on the Crocodile River (West)

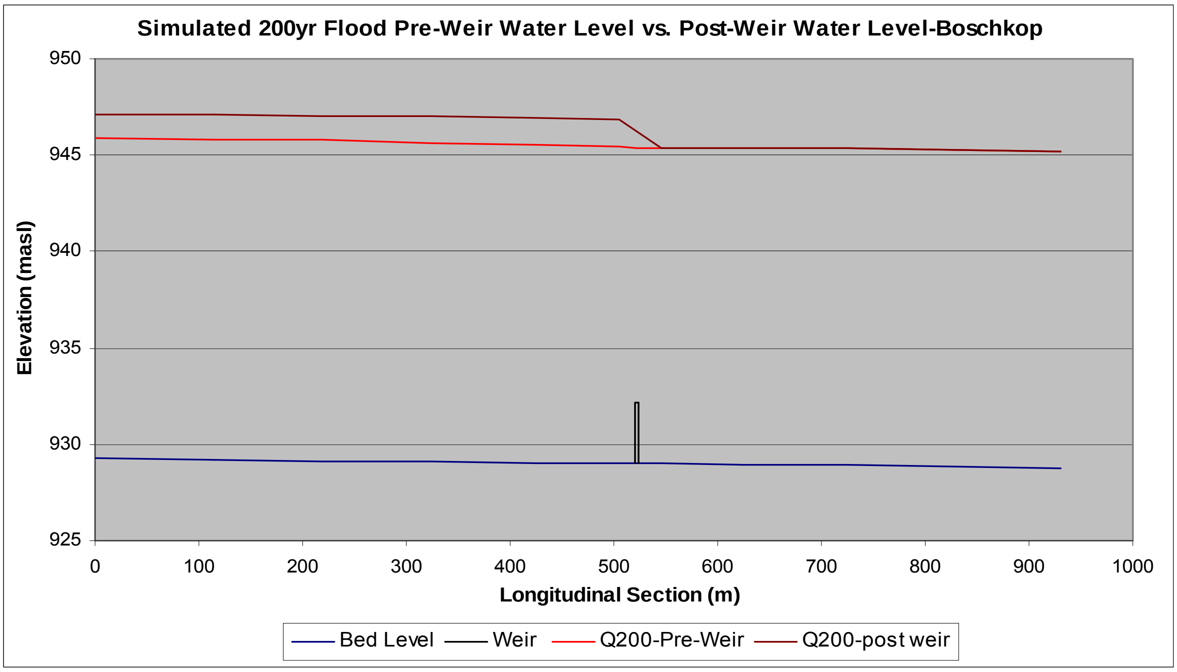

Figure 4-2: 1:200 Year Flood Levels at the Boschkop site on the Crocodile River (West)

During the 1:100 year flood the flow depth at the Boschkop weir site under present conditions (before construction of the weir) is about 15 m (Figure 4-1) above riverbed, but increased by 2 m to 17 m as a result of damming created by the weir. For the 1:200 year flood the flow depth under present conditions is about 16 m and the damming created after construction of the weir is about 1,5 m (Figure 4-2) resulting in an increased flow depth of 17.5 m. During the RMF the damming created by the weir is about 1.3 m upstream of the weir. The flow depth during the RMF upstream of the weir increases from 18.7 m under present conditions to 20 m above riverbed level after construction of the weir (Figure 4-3).

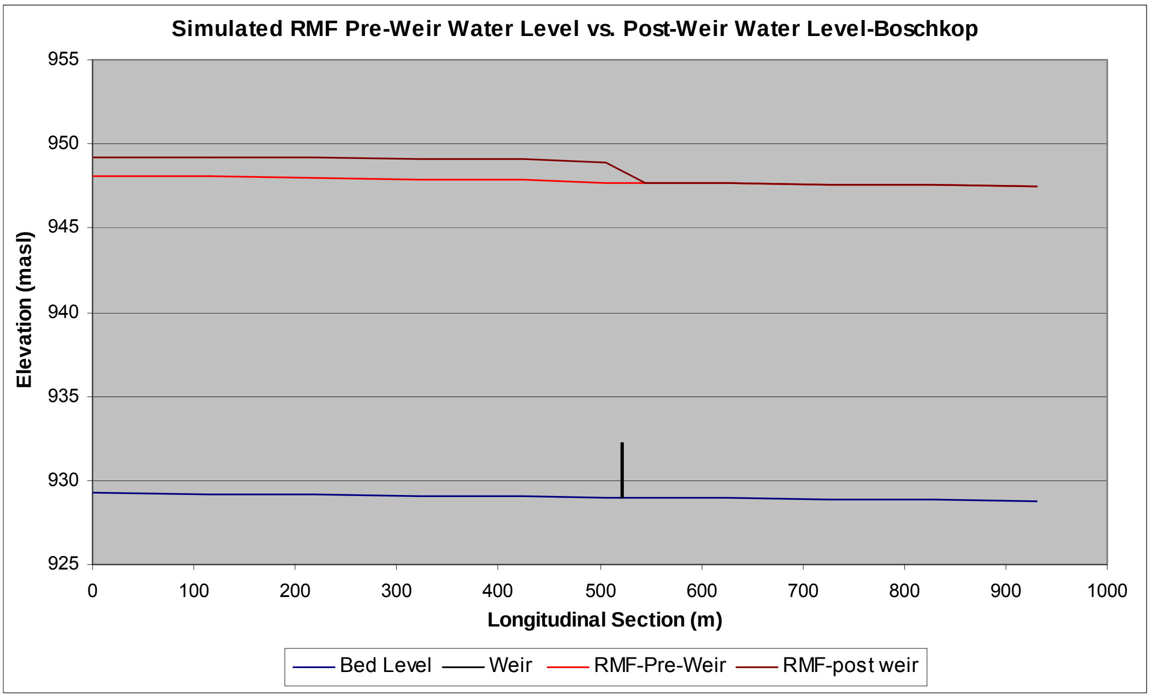

Figure 4-3: RMF Flood Levels at the Boschkop Site on the Crocodile River (West)

The floods at Vlieëpoort is larger than at Boschkop (refer to Table 4-1). During the 1:100 year flood the flow depth is presently (before construction of the weir) 13.5 m (Figure 4-4) above riverbed, and the weir creates damming of approximately 1.5 m, thereby increasing the flow depth to 15 m (which is similar to the case at Boschkop). For the 1:200 year flood the flow depth before construction of the weir increases from 14 m due to damming created by the weir to about 16 m (Figure 4.5). The flow depth during the RMF increases from 16 m under present conditions to 18m with the weir in place, which is 2 m less than at the Boschkop site (Figure 4-5).

In all cases a nominal weir height of 3 m was assumed for both of the Boschkop and Vlieëpoort weir options.

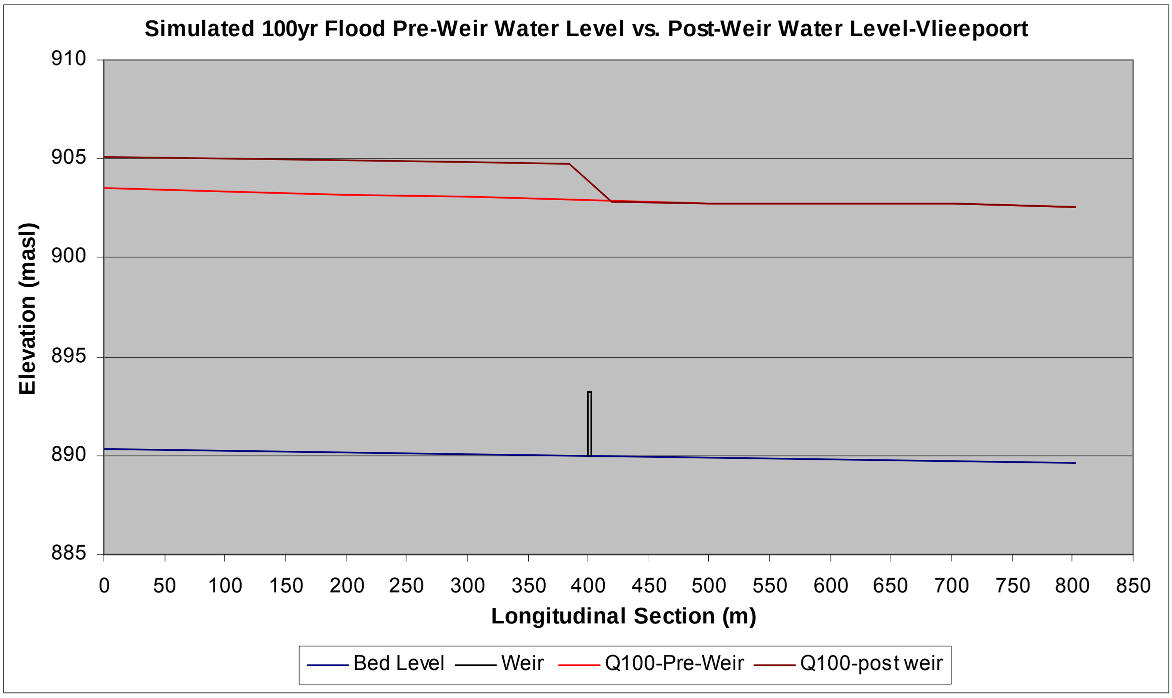

Figure 4-4: 1:100 Year Flood Levels at the Vlieëpoort Site on the Crocodile River (West)

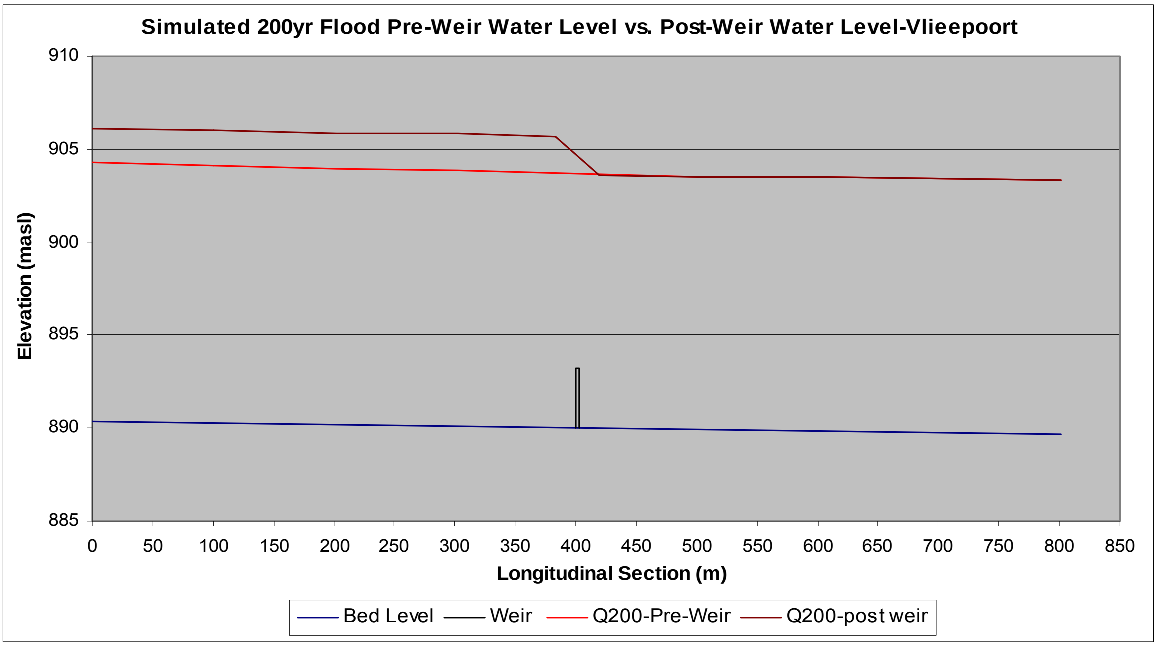

Figure 4-5: 1:200 Year Flood Levels at the Vlieëpoort Site on the Crocodile River (West)

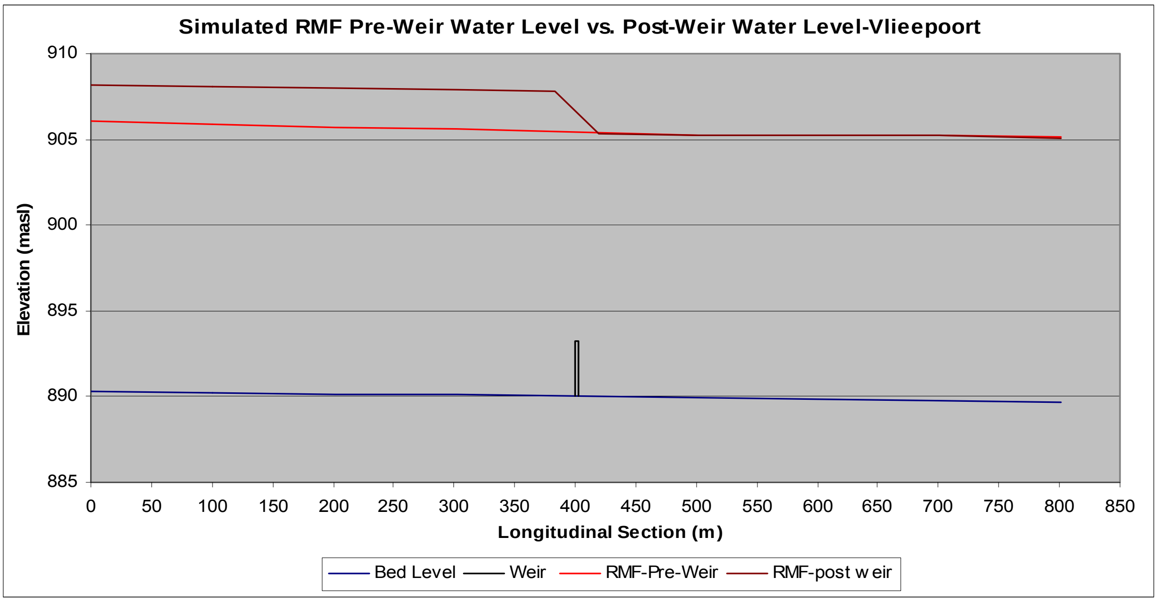

Figure 4-6: RMF Flood Levels at the Vlieëpoort site on the Crocodile River (West)

It should be noted that all these flood levels are based on ortho-photo maps with 5 m contours. Detailed surveys of the sites and relevant river reaches are required to obtain reliable flood levels.

The proposed weir at Site 3 on the Mokolo River was analysed in a similar manner to the weirs on the Crocodile River (West). A nominal weir height of 3 m was also assumed for this weir.

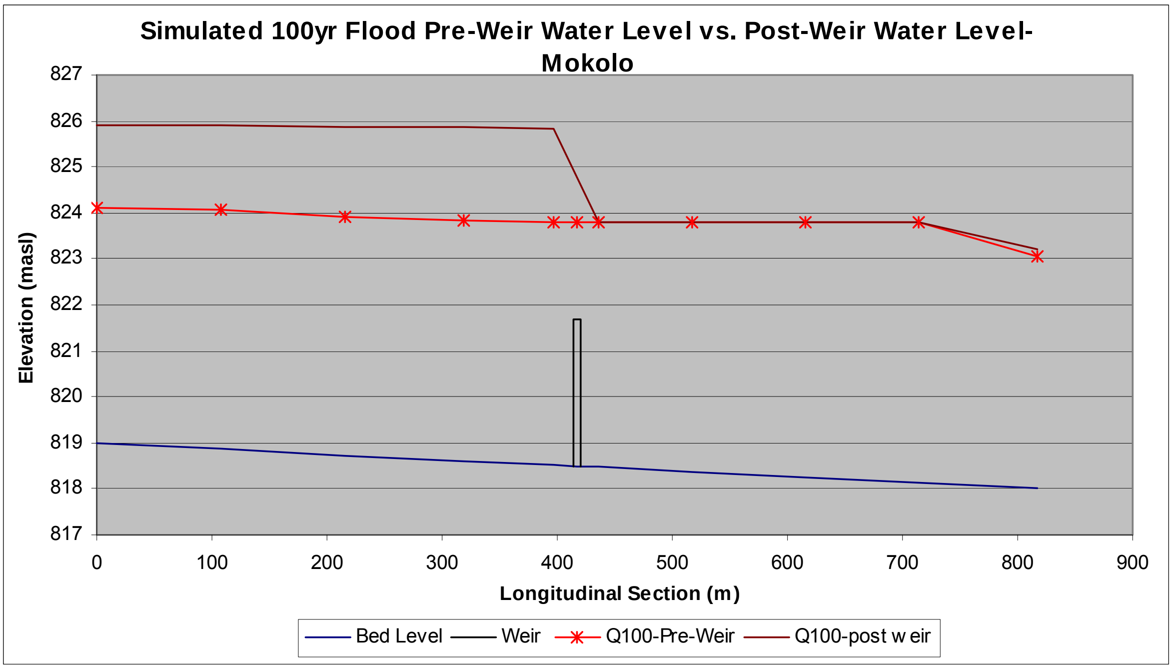

Figure 4-7: 1:100 Year Flood Levels at the Mokolo River Site 3

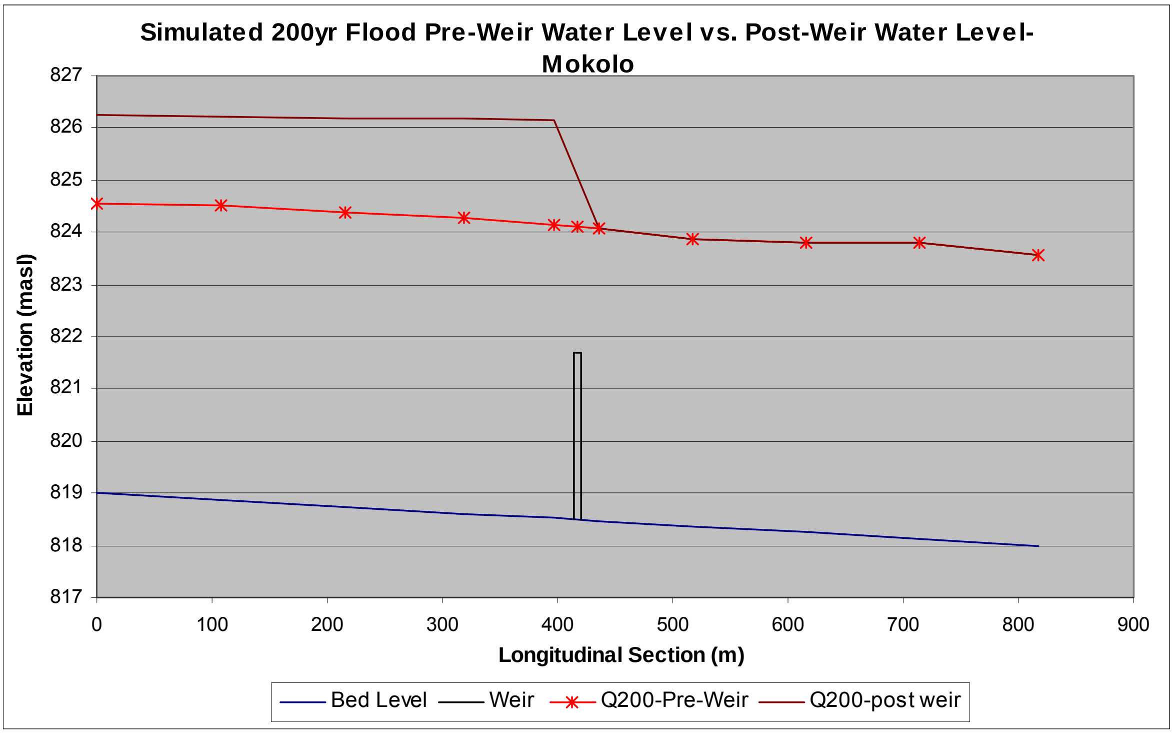

Figure 4-8: 1:200 Year Flood Levels at the Mokolo River Site 3

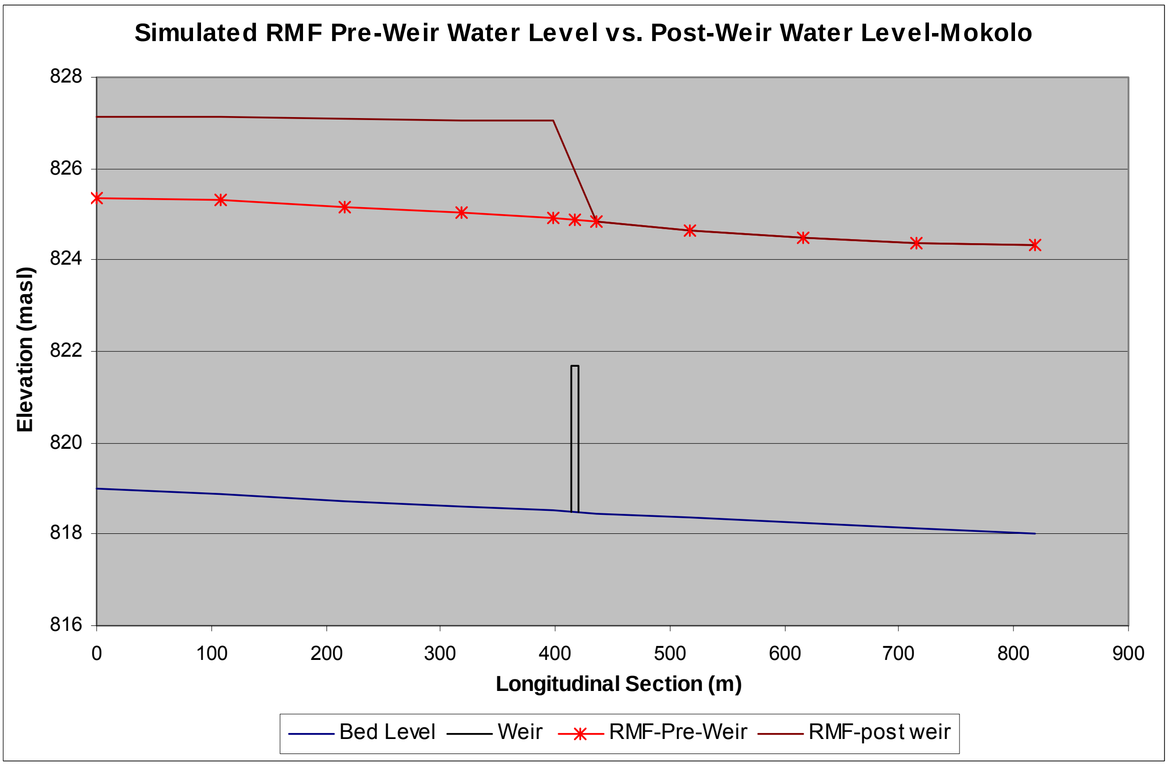

Figure 4-9: RMF Flood Levels at the Mokolo River Site 3

Table 4-2 gives a summary of calculated flood levels at the proposed weirs. Water levels are taken 20 m upstream of the weirs.

Table 4-2: Summary of Simulated Recurrence Interval Flood Levels.

Weir Location	1:100yr flood level (masl)		1:200yr flood level (masl)		RMF K value	RMF flood level (masl)	
	Pre-Weir	Post-Weir	Pre-Weir	Post-Weir		Pre-Weir	Post-Weir
Boschkop	944,29	945,77	945,41	946,86	4,00	947,67	948,93
Vlieëpoort	903,07	904,76	903,84	905,71	4,00	905,57	907,79
Mokolo Site 3	823,69	825,83	824,14	826,15	5,00	824,92	827,03

4.4. Design Floods and Levels

It is proposed that the weirs are designed for floods indicated in the SANCOLD, 1991(9), Guidelines on Safety in Relation to Floods. The height of the dam and the possible loss of lives or economical loss determine the hazard rating of a dam. At both the weirs on the Crocodile River (West), the 1:100 year flood depths exceed 12 m and the weirs will be categorized as of medium height. The weirs are however low (FSL) and do not store much water. Therefore sunny day failures should not have a major impact on the river downstream. The loss of lives is likely to be less than 10 and the economic loss downstream is likely to be minimal to significant at most. The economic loss to consumers supplied by the MCWAP will, however, be major. These weirs would therefore normally have had a significant hazard rating and could have been classified as Category II structures based on the SANCOLD, 1991(9), Guidelines on Safety in Relation to Floods. This means that the design floods for the Crocodile River weirs would have been as indicated in Table 4-3 based on the SANCOLD, (1991)(9) Guidelines on Safety in Relation to Floods.

Table 4-3: Crocodile River (West) Weirs Design and Safety Discharge to SANCOLD, 1991(9), Guidelines on Safety in Relation to Floods

Weir location	Recommended design discharge (RDD) recurrence interval (1:yr)	Recommended design discharge (RDD) (m^3/s)	Safety Evaluation Discharge (SED) recurrence interval	Safety Evaluation Discharge (SED) (m^3/s)
Boschkop	1:100	4 142	RMF	6 372
Vlieëpoort	1:100	4 995	RMF	7 456

However, due to the potentially high economic loss to the users supplied by the MCWAP, the hazard rating is high and should be classified as Category III structures. Under those circumstances the RDD and SED should be the 1:200 year flood and the RMF flood respectively. As stated previously, it is also proposed that for the Feasibility and Detailed design stages the SEF be taken as equal to the PMF, with a condition that overtopping is permissible provided that the structure retains its functionality. The incremental cost to the MCWAP will be relatively small. A further consideration would, however, also be whether temporary repairs can be done in a short period in order to re-commission the structure. The recommended design values are shown in Table 4-4.

Table 4-4: Crocodile River (West) Weirs Recommended Design and Safety Discharge

Weir location	Recommended design discharge (RDD) recurrence interval (1:yr)	Recommended design discharge (RDD) (m^3/s)	Safety Evaluation Discharge (SED) recurrence interval	Safety Evaluation Discharge (SED) (m^3/s)
Boschkop	1:200	4 779	PMF	9 558
Vlieëpoort	1:200	5 741	PMF	11 184

Freeboard components for wind waves, setup, run-up, searches, etc. have to be added to the RDD surcharge water level in different risk combinations at the weirs, based on the Interim guidelines of SANCOLD (1990)[(10)] on Freeboard for dams. The freeboard components were calculated for each weir and the data is indicated in Table 4-5. Note that the required top of the structures for the two abstraction works are slightly above the 1:200 year flood levels (Table 4-2).

Table 4-5: Freeboard Components at Weirs

Description	Boschkop weir	Vlieëpoort weir
Fetch length (km)	3	2.5
Design wind speed (m/s) (1:50 yr)	23.9	23.55
Significant wave height Hs (m) (1:25 yr)	0.93	0.85
Wave run up (1:3 slope riprap) (m)	0.93	0.85
Wind setup (m) (1:25 yr)	0.05	0.057
Flood surges and seiches	0.5	0.5
Surcharge (1:100 year flood) (m)	13.57	11.56
Total freeboard (m)	**15.05**	**12.97**
Required top structure for RDD (masl)	**947.25**	**906.17**
1:200 year post-weir flood levels (masl)	946.86	905.71

On the Mokolo River the flood depth at Site 3 for the 1:100 year flood is less than 12 m. This weir is therefore small, but because of the expected significant hazard rating it would therefore have a Category II classification and the required design floods are indicated in Table 4-6.

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Table 4-6: Mokolo River Weirs Recommended Design and Safety Discharge

Weir location	Recommended design discharge (RDD) recurrence interval (1:yr)	Recommended design discharge (RDD) (m^3/s)	Safety Evaluation Discharge (SED) recurrence interval	Safety Evaluation Discharge (SED) (m^3/s)
Mokolo Site 3	1:100	5427	PMF	7 545

The abstraction works will be designed with the top of the pump station above the RDD with freeboard to allow access during floods. At the Mokolo weir the top of the abstraction works concrete would be above the 1:200 year flood elevation, which is based on the RDD (1:100 year flood) plus other freeboard components. Flood levels for these floods are indicated in Table 4-2. The pump station would be able to operate during extreme flood conditions (SEF), with the switchgear located on the bank above the flood level.

4.5. 1:20 and 1:50 Floods and Levels

One of the design criteria that were proposed for the design of the weirs was that the impact of the weirs under flood conditions should be as small as possible when compared to the present condition without the weirs. The average bed slope along the river is in the order of 1:3 000 and any significant change in flood levels will consequently have an impact over an extended section of the river. Purchase boundaries are normally based on the 1:100 year return period flood lines plus 1m height or 20m horizontal clearance line, whichever is the further from the river.

Raising of flood levels at the weir sites by 2 m could therefore influence flood levels as far as 6 km upstream of the weirs, impacting on farming activities and other infrastructure such as roads, especially in the vicinity of the Boschkop and lower Mokolo Weir sites. Detailed flood line and flood impact analyses, including purchase boundary assessments, will be undertaken during the Feasibility stage of the study once comprehensive survey data becomes available.

In order to reduce these potential impacts the possibility of designing weirs for submerged conditions was investigated. The objective was to design a structure for the 1:20 or 1:50 year return period flood and to provide counter outflanking measures for all the larger floods. The reliability and safety criteria (PMF) for all flood sensitive components would still remain in place. Layouts depicting such submerged designs are provided in **Appendix B** as Option 2 layouts.

The 1:20 and 1:50 year returns periods have been selected since these floods will recede below these levels fairly soon after major flood event. Access to the abstraction works will therefore not be interrupted for any long period.

A significant reduction in the weir profile presented to the river could be achieved, but due to the poor survey data that was available for the pre-feasibility stage of the study, a detailed analysis of the benefits of this type of design will only be undertaken during the feasibility stage when detailed survey data would be available.

The applicable flood design data is provided in Table 4-7 and Figure 4-10 to Figure 4-13 below.

Table 4-7: Crocodile River Weirs – 1:20 and 1:50 Return Period Floods and Levels

Weir Location	1:20 yr flood peak (m^3/s)	1:20yr flood level-Post Weir (masl)	1:50 yr flood peak (m^3/s)	1:50yr flood level-Post Weir (masl)
Boschkop	2 390	941.83	3 380	944.23
Vlieëpoort	2 870	901.17	4 020	903.12

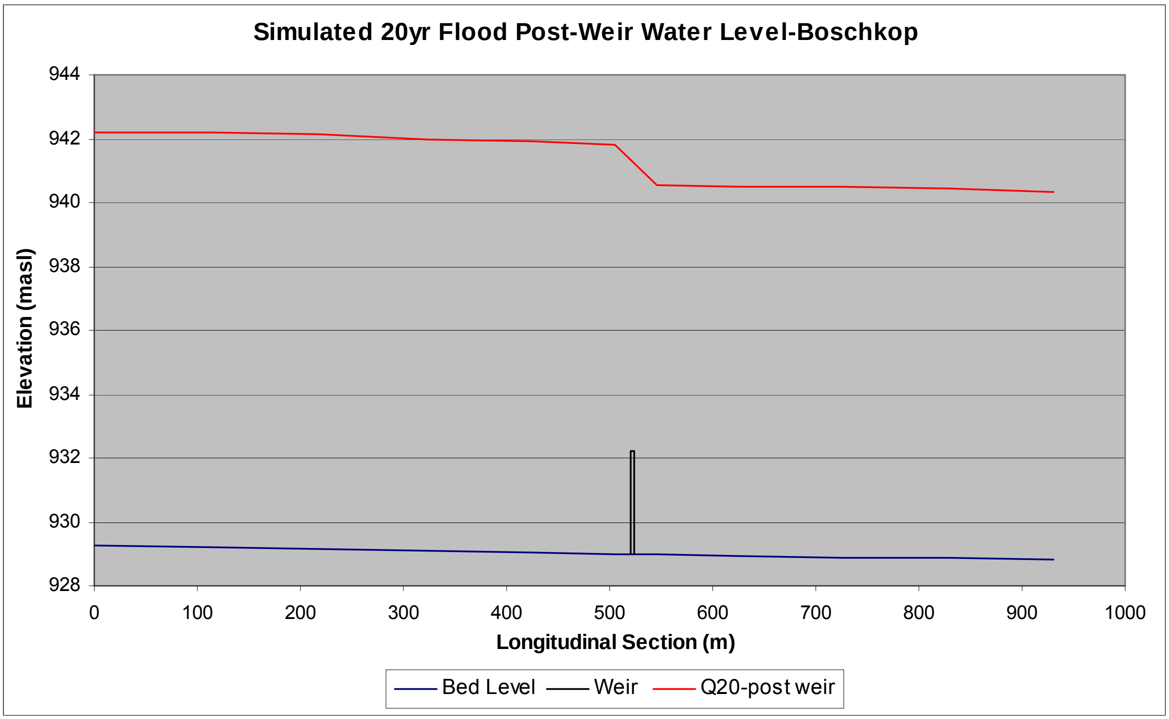

Figure 4-10: 1:20 Year Flood Levels at Boschkop Weir

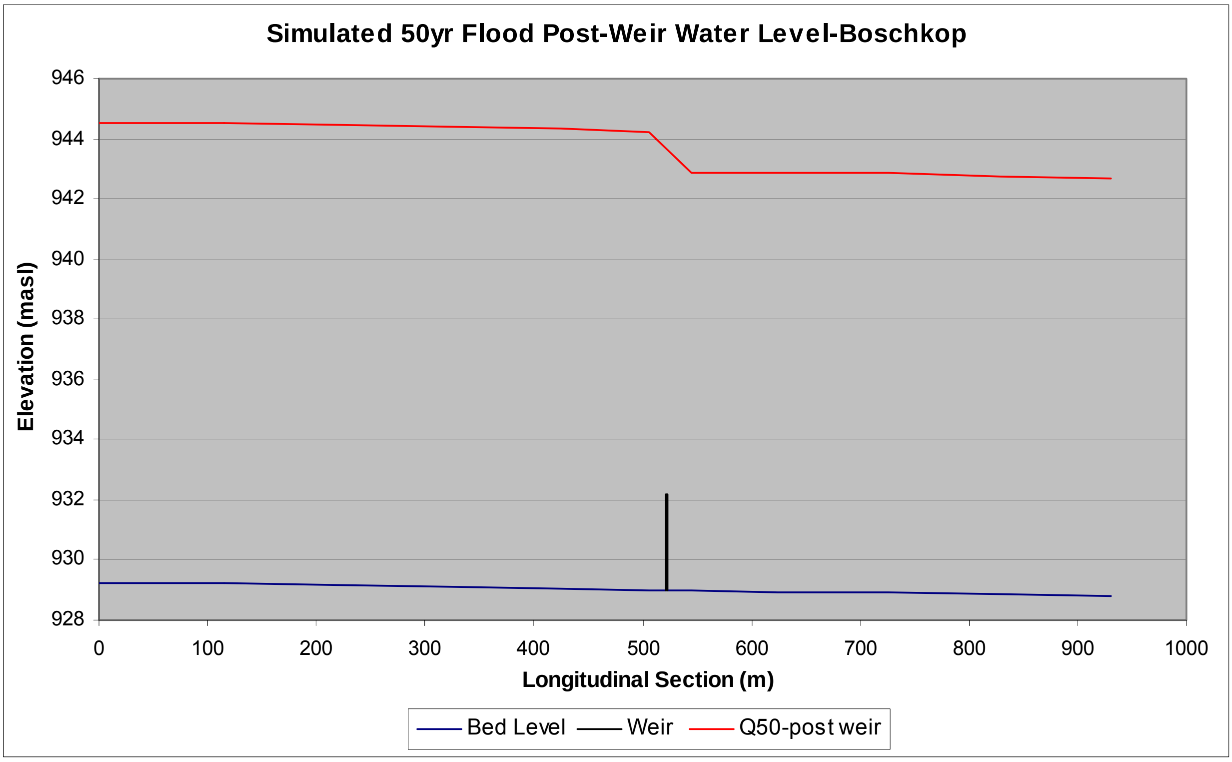

Figure 4-11: 1:50 Year Flood Levels at Boschkop Weir

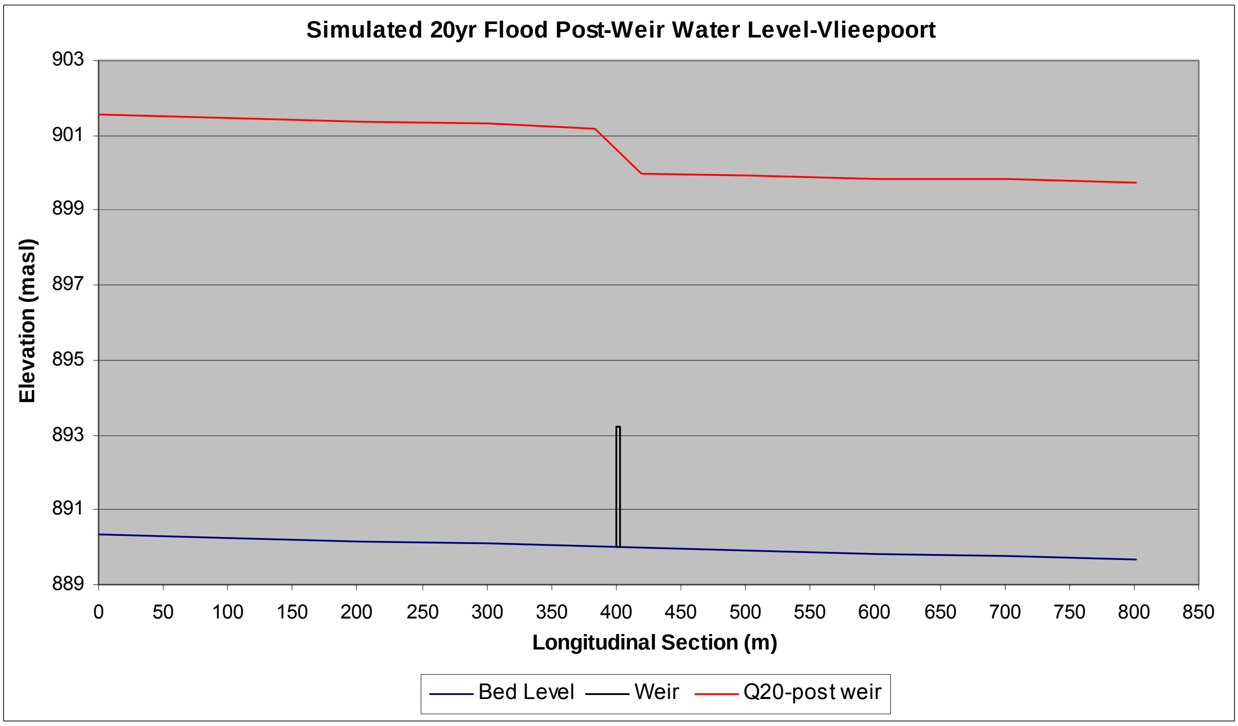

Figure 4-12: 1:20 Year Flood Levels at Vlieëpoort Weir

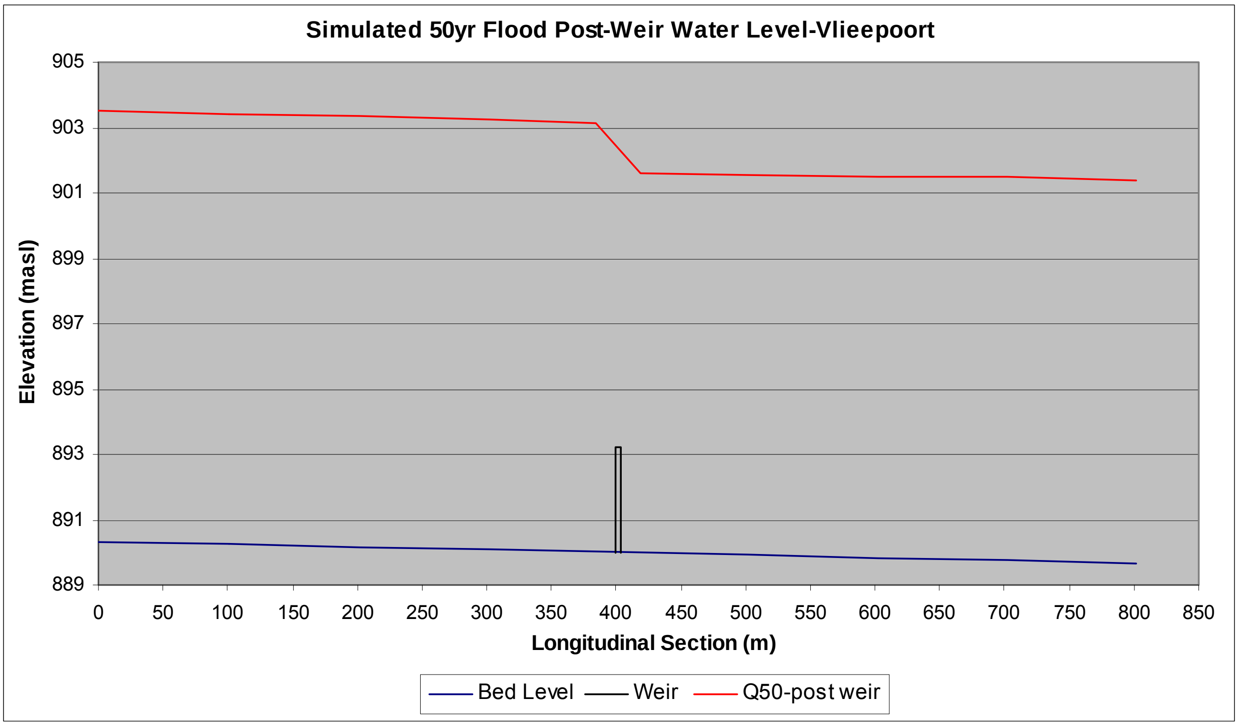

Figure 4-13: 1:50 Year Flood Levels at Vlieëpoort Weir

The analyses were not done for the Mokolo River, but the same principles would apply.

5. ENVIRONMENTAL AND SOCIAL SCREENING

The screening of the probable social and environmental impact of the envisaged works were conducted and is reported in Report 7[(6)] – Environmental and Social Screening Report (P RSA A000/00/9409).

6. GEOTECHNICAL SCREENING

The following dam / weir sites were considered for screening, namely:

- Boschkop
- Vlieëpoort
- Terminal dam sites (No's 1 and 3)
- Mokolo Weir site No 3

All the above sites, and many other options, were visited, but the visits were very brief and detailed verification of the founding conditions was not possible. Also note that because the locations of the terminal reservoirs were not known (the users considered their locations as sensitive information) no geotechnical screening of these sites were possible.

The following summary of the assumed geological conditions at the respective sites is based on a study of available information; including previous reports where available, published geological maps (Council for Geoscience), published ortho-photos (Chief Directorate: Surveys and Mapping), and images from Google Earth, as well as observations during the brief site visits.

Coordinates listed below were obtained from a hand-held GPS, and the usual allowances for accuracy should be made. Coordinates are in accordance with the WGS84 system, using the South African grid (Lo 27).

6.1. Boschkop

Approximate coordinates for the favoured Boschkop site, located on the farm Boschkop 138 JQ, are Lo 27 Y -53 203, X 2 776 664.

The site is located along a reasonably straight section of the Crocodile River (West) (Figure 6-1), where it skirts the northern boundary of the hill 'Boschkop'

It should be noted that previous investigations were conducted at a time when construction of a large dam was likely being considered for this site. Currently, the envisaged structure comprises a relatively low, abstraction/diversion weir, with associated appurtenant works comprising a de-silting works and pumping station. Structure dimensions were not available at the time of writing, but such a structure will likely not exceed 5 m in height (above current river level).

It should further be noted that previous studies concluded the site was not suited to construction of a concrete dam, but it should be borne in mind that the currently-envisaged structure is a small diversion weir where storage is not required and which will not elevate the water level significantly beyond the natural river channel.

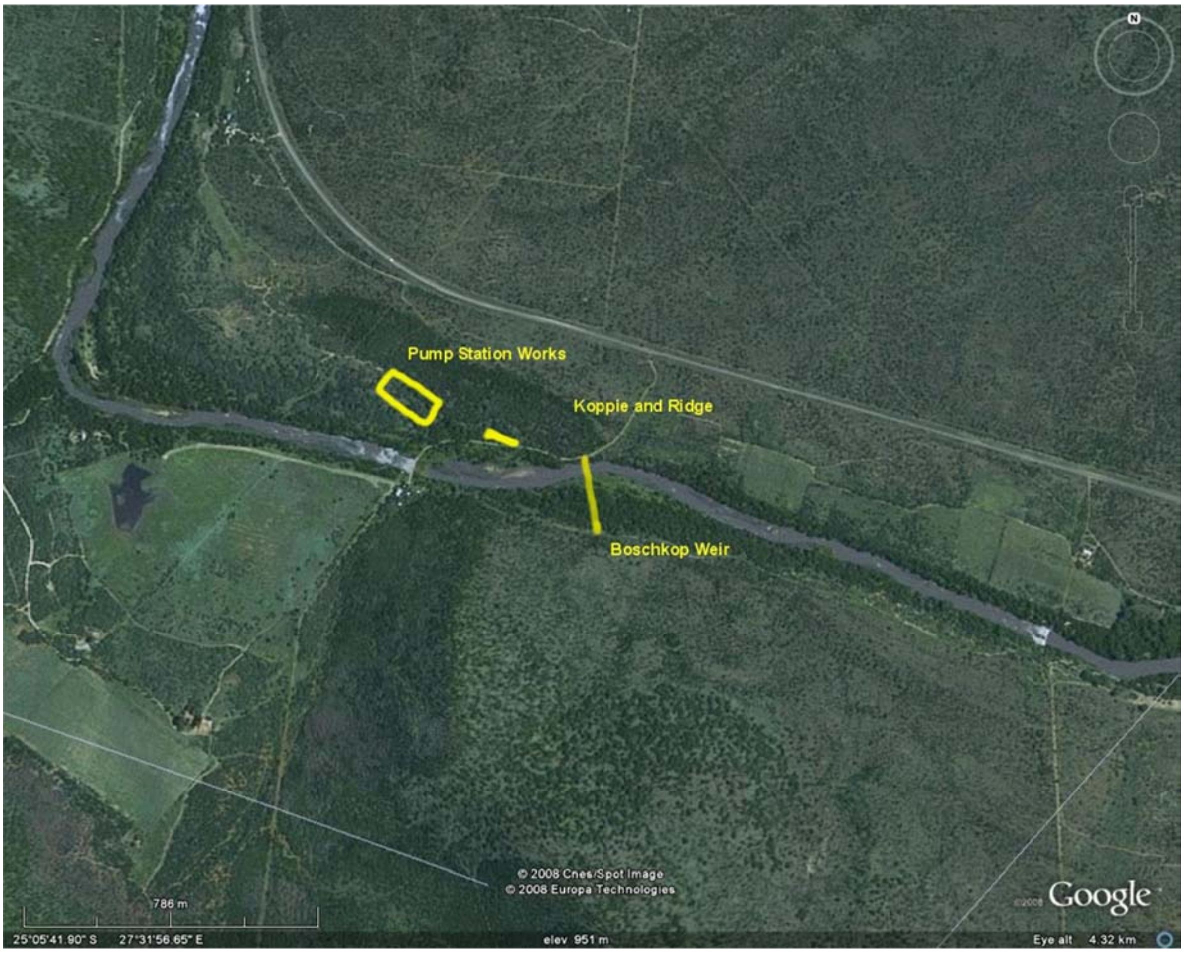

Figure 6-1: General Plan of the Boschkop Weir Site

6.1.1. Geological Setting

The Boschkop site is located on rocks of the Pretoria Group, of the Transvaal Supergroup, where these strata are surrounded by intrusive Bushveld rocks; within the so-called Crocodile River Inlier.

The rocks in the greater valley floor comprise dolomite of the Chuniespoort Group, with shales and minor quartzites of the Pretoria Group occurring on the higher-lying valley slopes. Pyroclastic and metacarbonatite rocks of the volcanic Kruidfontein Complex occur to the south of the proposed site.

The 1:250 000 geological map shows a relatively minor, inferred fault which may be traced along the river, intersecting the centre-line of the proposed site.

6.1.2. Previous Investigations

As summarized in the Inception Report, a number of separate geological investigations have been conducted previously at this site, in the period between 1937 and 1958. These reports are listed in the Inception Report.

Geological mapping has been conducted, and geophysical surveys carried out including magnetometer readings and a gravity survey. In addition, in excess of fifty boreholes have been drilled.

It should be noted, however that although the previous reports could be located, none of the factual data (geological plans, borehole logs, geophysical survey results) could be found.

6.1.3. Site Description

The site is located at a slight constriction and minor bend in the river, where the river skirts the northern slopes of the 'Boschkop' hill, i.e. which constitute the left flank of the weir. A low koppie and ridge are located on the opposite, i.e. right bank, but this elevated area is slightly offset in a downstream direction.

6.1.4. Site Geology

The river section is covered by alluvial sand deposits (Figure 6-2). Horizons of boulders might be present within the alluvial deposits. No bedrock outcrop is visible. Previous drilling indicated these sands were at least 20 m in thickness. The river channel is defined by river banks which rise an estimated 2 m to 5 m above river level. The fault is likely present along the river section. Although initial drilling did not intersect this fault, subsequent drilling intersected a number of fault zones which proved to be cemented.

The bedrock profile underlying the alluvial deposits is likely to be highly irregular.

Scattered outcrop of bedrock is noted on the lower flank areas, comprising banded ironstones as well as dolomite. Previous reports mention the fact that these rocks are deeply weathered.

Previous concerns were expressed regarding interconnected cavities and the potential for significant leakage, but some studies concluded there was no interconnection between boreholes.

6.1.5. Envisaged Founding Conditions and Foundation Treatment

Because the respective elevated flanks are slightly offset with respect to each other, it follows that the weir centre-line would likely be optimally aligned slightly obliquely with respect to the river, in order to best utilize the topography and lower the risk of outflanking during flood events.

A small diversion weir located in the Crocodile River would have to be a concrete structure to be able to successfully pass the expected flood events. It follows that the structure would require non-erodible foundations, of sufficiently high strength.

Suitable foundations would conventionally comprise sound bedrock; moderately weathered or better. Because of the expected depth of alluvium (15 m to 20 m), an alternative might be to construct a cut-off and foundation by means of jet-grouted columns or similar.

It is likely that bedrock beneath the alluvium would be suitable for founding of a low concrete structure, although an upper horizon of unsuitable rock might be present. In such case, foundation preparation would require removal of the alluvium to a depth of 15 m to 20 m. Such excavation would likely be ‘soft’ although the possibility of boulder horizons cannot be excluded. There should be some allowance for minor ‘hard’ excavation for removal of an upper horizon of unsuitable rock; say to a depth of 1 m to 2 m.

Figure 6-2: A View of the Proposed Boschkop Weir Site

The photo provided in Figure 6-2 was taken from a position slightly downstream of the centre-line, looking towards the low koppie and ridge on the right bank. River flow is therefore from right to left.

These deep excavations in soft alluvial deposits would need to be cut back significantly and / or shored to ensure stability of the temporary slopes. As these excavations would be below river level, major seepage into the foundations is likely, and some form of cut-off will be required.

If the required founding solution requires excavation to bedrock then difficulties are to be expected within these saturated alluvial sands. Significant seepage problems should be expected due to excavations being below river level, and the highly pervious nature of the alluvial deposits. In addition, temporary excavation slopes in the alluvial sands would need to be shored, or cut back, to ensure worker safety. The impracticalities of conventional excavations within this alluvial environment dictate the need for other approaches; either by utilising other means of cut-off, for example by installing sheet piles, or by other construction techniques such as slurry trench or by using jet-grouted columns.

The previous reports reflect earlier concerns regarding potential seepage problems and water leakage. As the structure is a diversion weir and is not intended as a storage dam, the water losses themselves are not a major concern; but rather the potential for erosion of the founding materials, for example the weathered ironstone and dolomite which was associated with very high core losses, or where the structure is founded above the bedrock. There should therefore be allowance for grouting of the foundation; the purpose of which would not be the ‘sealing’ of the foundation by means of a grout curtain, but rather a programme of compaction grouting to fill any cavities which might be present. If the fault is not completely re-cemented then curtain grouting of this feature would be required, where the primary aim would be to prevent internal erosion of possible weak materials in this presumed fault zone. Similar attention will have to be given to the foundations if the structure is founded above the bedrock.

6.1.6. Construction Materials

A concrete weir structure would require both coarse and fine aggregate. Coarse aggregate volumes are not likely to be sufficient to justify opening of a dedicated quarry, and this might favour purchase from commercial sources.

Fine aggregate (sand) would likely be sourced locally. A test pitting exercise would be required to prove a suitable source. A total of twenty test pits is assumed at this stage. Testing will be required to confirm the materials conform to SABS specifications for fine aggregate. A total of 20 samples are assumed.

The same approach would be followed to test and source materials for the weir flank embankment and balancing dam embankment fills, filters and rip-rap.

6.1.7. Recommendations

Weir:

If the possibility of jet grouted columns is to be considered, then the composition of the alluvial deposits will have to be investigated, specifically whether boulders are present and the diameter of these boulders.

A geophysical survey is recommended prior to drilling; with the aim of identifying overburden thicknesses including anomalous areas, and confirming the location of faults. A gravity survey should be included in order to detect whether cavities underlie the weir footprint.

The bedrock depths as well as the bedrock condition would need to be confirmed. Exploratory drilling is therefore necessary. For these feasibility-level investigations a total of four boreholes would be required, drilled 5 m into bedrock (total 4 No, length 120 m); comprising two each on the respective river banks. At least one should be angled beneath the existing river channel, or to intersect the fault – if the position could be confirmed during the geophysical survey.

If possible, bearing in mind the materials at river level are likely to comprise saturated sands, at least two test pits would be required on the respective river banks (total 4 No). Representative samples would be submitted for laboratory testing which would comprise:

- Foundation indicators, including Proctor compaction (10 No)

- Double hydrometers for dispersivity determination (5 No)
- Chemical testing to determine potential corrosivity (4 No)
- Grading analyses for fill and filter materials and fine aggregates for concrete.

Appurtenant Works:

Sites for the appurtenant works comprising de-silting works, balancing reservoir and high-lift pump station and have been identified.

Because the sites may be underlain by dolomite, it is recommended that gravity surveys be conducted prior to drilling to confirm whether or not potential cavities underlie the site. These sites would also require the drilling of two boreholes at each site (total four boreholes), drilled 5 m into bedrock, with SPT testing at 1.5 m grid spacing in the soft overburden. At this stage it is assumed that a total drilling length of 60 m will be required (4 BH's).

Test pitting is required at the sites of the de-silting works, balancing reservoir and high-lift pump station. At least two test pits are required at each site (total No 4), to be excavated by means of excavator. Representative samples would be submitted for laboratory testing which would comprise:

- Foundation indicators, including Proctor compaction (10 No)
- Double hydrometers for dispersivity determination (5 No)
- Chemical testing to determine potential corrosivity (4 No)
- Grading analyses for fill and filter materials and fine aggregates for concrete.

6.2. Vlieëpoort Weir Site

Approximate coordinates for the Vlieëpoort site, which is located on the farm Hanover 341KQ, are Lo 27 Y -31 979, X 2 725 486. The proposed site is located at a narrowing of the valley where the Crocodile River (West) cuts through the Vlieëpoortberge (Figure 6-3).

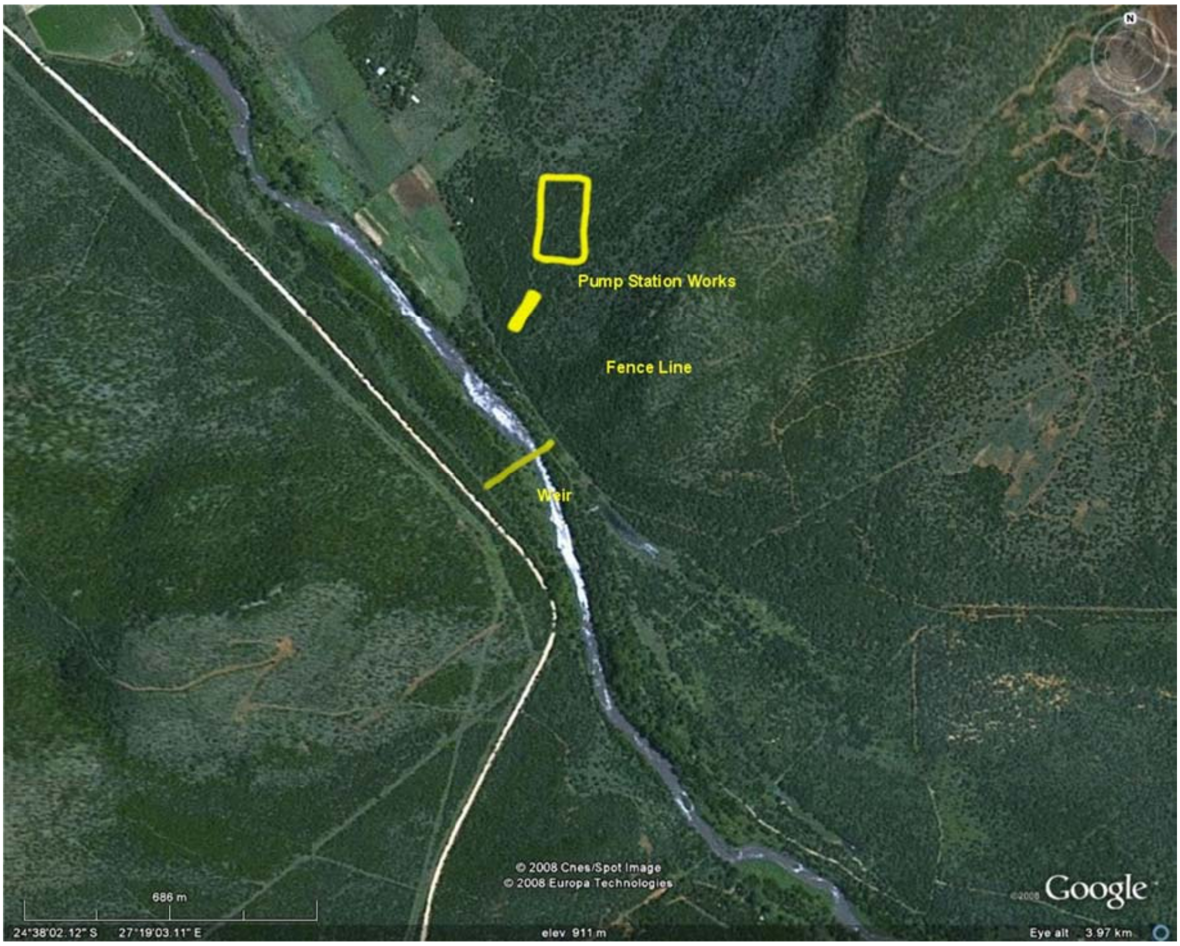

Figure 6-3: General Site Plan of the Vlieëpoort Weir Site

6.2.1. Geological Setting

The published 1:250 000 geological map (Thabazimbi Sheet 2426, Council for Geoscience) indicates the Vlieëpoort mountains are aligned with the banded ironstone formations of the Penge Formation, and the dolomites, cherts and subordinate shales of the Malmani Subgroup; all of the Chuniespoort Group, Transvaal Supergroup. The strata strike roughly in a north-easterly direction, and dip at angles between 20° and 30° in a south-easterly, i.e. upstream, direction.

At the position indicated in Figure 6-3, the weir is probably located on the dolomitic rocks, while the younger banded ironstone formations are slightly upstream, in the area of the bend in the road.

Upstream of the mountain range the wider valley area is underlain by the sedimentary strata of the Timeball Hill Formation of the Pretoria Group, comprising shales and sandstones.

The geological map indicates a minor fault immediately downstream of the indicated weir site, aligned roughly parallel to the proposed centre-line.

6.2.2. Previous Investigations

Although recognized as a potential dam site for many decades, with a previous geological report dated 1938, there is no record of detailed investigations ever being conducted.

As with the Boschkop site, this Vlieëpoort site was previously considered for construction of a large dam. The currently-envisaged structure would comprise a low diversion weir with appurtenant works comprising a pumping station and de-silting works located a short distance downstream of the weir site and the narrow gorge, at a point where the valley widens.

6.2.3. Site Description

The Vlieëpoortberge which are bisected by the Crocodile River (West) rise to elevations in excess of 1 400 masl on either side of the river, where the elevation of the river bed is less than 900 masl.

The site is characterised by a relatively wide river section. A gravel road is located on the left bank of the river.

6.2.4. Site Geology

The envisaged weir structure is likely only a low structure, which will be confined to the greater river section and flanks rising to higher ground.

The prominent mountains which rise to a significant height comprise banded ironstones and dolomite at shallow depths; even outcropping in places. These shallow bedrock conditions do not extend through the river channel, but are indicative of the broader geology which might be expected to underlie the alluvial cover.

At the foot of these slopes accumulations of sands and gravels are present. These colluvial (talus) materials will become finer grained towards the river, and will grade into the alluvial deposits which occur within the river section. In places there will be some mixing of these colluvial and alluvial materials.

A number of minor terraces may be identified with the valley section. An upper terrace is present at the foot of the steep slopes and mainly comprises the coarser gravelly colluvium. An intermediate terrace is recognized within the greater river section where the alluvial sand deposits occur at an elevation approximately 5 m above the level of the present river channel(Figure 6-4) but lower than the upper ‘talus’ terrace.

Figure 6-4: Vlieëpoort Weir Site

The photo provided in Figure 6-4 provides a view of the approximate position of the Vlieëpoort Weir Site. The view is directed downstream, with extensive alluvial sand deposits evident.

No bedrock outcrop is evident within the river section. Extensive deposits of alluvial sand cover the river section, with estimated thickness of at least 15 m to 25 m. Exposures within the river banks suggest the alluvium comprises sand, but the possibility of boulder horizons at depth cannot be discounted.

The underlying bedrock is expected to comprise dolomite, and a highly irregular bedrock profile is to be expected. An upstream centre-line shift would move towards the underlying banded ironstones, but the required shift is likely to be so far as to imply a significantly longer centre-line and will not meet the hydraulic requirements of the abstraction works. The condition of the underlying dolomites is unknown at this stage, and the possibility of interconnected solution channels and cavities cannot be excluded.

6.2.5. Envisaged Founding Conditions and Foundation Treatment

A low diversion structure located at this constriction in the valley would undoubtedly be subjected to regular flooding and therefore needs to withstand regular overtopping in the river section. For this reason a mass concrete structure would appear to be the logical choice.

Conventionally, a concrete structure would be founded on sound bedrock. Because of the expected depths of at least 15 m to 25 m, foundation excavations would be significant.

Such excavations in the alluvium would mainly be classed as soft excavation, but some intermediate or even hard excavation cannot be excluded if an upper bedrock horizon is encountered that would also require removal.

Excavation slopes in the alluvial sands would need to be shored, or cut back, to ensure worker safety. Significant seepage problems should be expected due to excavations being below river level, and the highly pervious nature of the alluvial deposits, and some form of cut-off would be required.

If the required founding solution requires excavation to bedrock then difficulties are to be expected within these saturated alluvial sands. Significant seepage problems should be expected due to excavations being below river level, and the highly pervious nature of the alluvial deposits. In addition, temporary excavation slopes in the alluvial sands would need to be shored, or cut back, to ensure worker safety. The impracticalities of conventional excavations within this alluvial environment dictate the need for other approaches; either by utilising other means of cut-off, for example by installing sheet piles, or by other construction techniques such as slurry trench or by using jet-grouted columns.

An alternative might be to utilize the alluvial sands and construct a jet-grouted cut-off which would then comprise the foundations for a concrete structure. The characteristics of the alluvial sands would have to be confirmed to confirm whether this is a viable alternative; the presence of large boulders would be undesirable and cannot be excluded.

If cavities are present then these will have to be filled by a programme of compaction grouting. Curtain grouting to form an impervious cut-off would not be required, unless weak, erodible materials are present which would be susceptible to internal erosion, or where the structure is founded above the bedrock.

6.2.6. Construction Materials

A concrete weir structure would require both coarse and fine aggregate. Coarse aggregate volumes are not likely to be sufficient to justify opening of a dedicated quarry, and this might favour purchase from commercial sources. Nearby sources which would warrant further investigation are the various dumps of waste rock from the mines in close proximity to the weir site.

Fine aggregate (sand) would likely be sourced locally. A test pitting exercise would be required to prove a suitable source. A total of twenty test pits is assumed at this stage. Testing will be required to confirm the materials conform to SABS specifications for fine aggregate. A total of 20 samples are assumed.

The same approach would be followed to test and source materials for the weir flank embankment and balancing dam embankment fills, filters and rip-rap.

6.2.7. Recommendations

Weir:

If the possibility of jet grouted columns is to be considered, then the composition of the alluvial deposits will have to be investigated, specifically whether boulders are present and the diameter of these boulders.

A geophysical survey is recommended prior to drilling; with the aim of identifying overburden thicknesses including anomalous areas, and confirming the location of discontinuities. Because the site is located on dolomite, these geophysical surveys should include a gravity survey in order to detect potential cavities in the footprint area.

The bedrock depths as well as the bedrock condition would need to be confirmed. Exploratory drilling is therefore necessary. A total of four boreholes would be required for these feasibility-level investigations, drilled at least 5 m into bedrock (total 4 No, length 160 m); comprising two each on the respective river banks, where at least one is angled beneath the river channel, or targeting anomalies if identified during the geophysical survey.

If possible, at least two test pits would be required on the respective river banks (total 4 No). Representative samples would be submitted for laboratory testing which would comprise;

- Foundation indicators, including Proctor compaction (10 No)
- Double hydrometers for dispersivity determination (5 No)
- Chemical testing to determine potential corrosivity (4 No)
- Grading analyses for fill and filter materials and fine aggregates for concrete.

<u>Appurtenant Works</u>:

Sites for the appurtenant works comprising de-silting works, balancing reservoir and high-lift pump station have been identified.

Because the sites may be underlain by dolomite, it is recommended that gravity surveys be conducted prior to drilling to confirm whether or not potential cavities underlie to sites.

These sites would require the drilling of two boreholes at each site (total four boreholes), drilled 5 m into bedrock, with SPT testing at 1,5 m grid pattern in the soft overburden. At this stage it is assumed that a total drilling length of 60 m will be required (4 BH's).

Test pitting is required at the sites of the de-silting works, balancing reservoir and high-lift pump station. At least two test pits are required at each site (total No 4), to be excavated by means of excavator. Representative samples would be submitted for laboratory testing which would comprise;

- Foundation indicators, including Proctor compaction (10 No)
- Double hydrometers for dispersivity determination (5 No)
- Chemical testing to determine potential corrosivity (4 No)
- Grading analyses for fill and filter materials and fine aggregates for concrete.

6.3. Terminal Dam Sites

Four sites were identified previously as possible sites for construction of a terminal dam. Two of these alternatives are favoured as potential sites; namely Sites No 1 and 3, where the sites are numbered as per Figure 6-5. This summary only includes discussion on these two sites. As was discussed in Section 3.1.5 Site 1 was the favoured site because of practical considerations and Site 2 was discarded for environmental reasons. This left Site 4 that was abandoned on technical grounds leaving Site 3 as perhaps the only alternative. The advantage expanding the geotechnical screening to include Site 3 was that it also expanded the database on the geology of the area surrounding Site 1 by finding, for example, geological features that may have been missed during the screening of Site 1, being in an adjacent valley.

All the sites are on the farm Witvogelfontein 362 LQ; approximate coordinates for Site 1 are Lo 27 Y -49 260, X 2 641 465.

6.3.1. General Geology

The published geological map (Sheet 2326 Ellisras, Council for Geoscience) indicates all four possible sites are located in an area underlain by coarse-grained, purplish brown sandstone of the Mogolakwena Formation of the Kranskop Sub-group, Waterberg Group. These sedimentary strata are traversed by diabase dykes and numerous linear features which might represent minor faults or additional diabase dykes. A prominent, inferred fault with a north-westerly strike passes relatively close to the proposed dam sites. The sedimentary strata dip at angles which vary between 10° and 30° in a south south-easterly to south south-westerly direction.

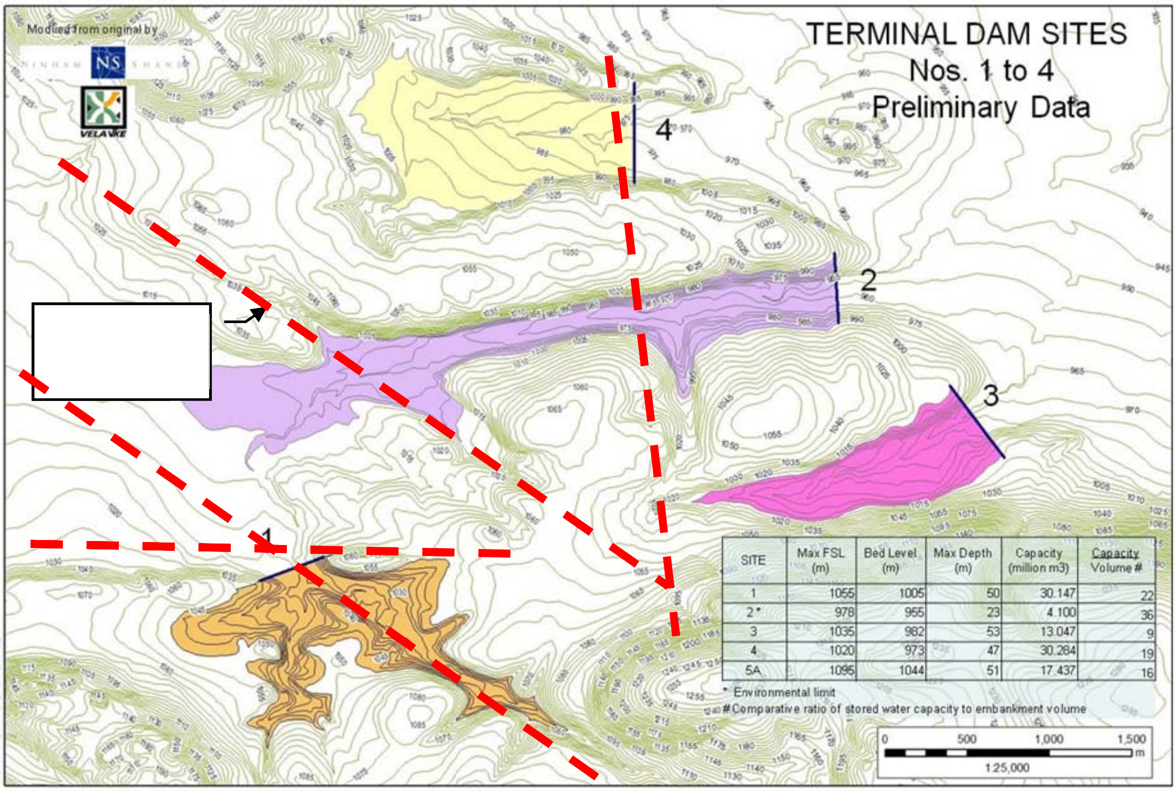

Figure 6-5: A General Plan indicating the Four Possible Terminal Dam Sites

6.3.2. Previous Investigations

There is no record of any previous investigations of potential dam sites being conducted on the farm Witvogelfontein.

6.3.3. Site Description

The identified potential dam sites are located as positions where the respective river valleys provide a storage basin, and a narrowing of the valley suggests the possibility of constructing a dam wall.

It might be noted that the terminal dam is essentially an off-channel storage dam which will be filled with water diverted from the Crocodile River; as such dam sites are not dependent on the expected run-off characteristics.

6.3.4. Site Geology

6.3.4.1. Site 1

A general view of Terminal dam Site 1 is included as Figure 6-6.

Figure 6-6: A View of the Terminal Dam Site 1, from the Left Flank towards the Right Flank

The site is characterised by moderately steep flanks and a relatively wide river section. At the time of writing no detailed site surveys had been compiled and the respective gradients and site dimensions are uncertain.

A number of lineaments are recognized in the area of the possible dam. These include the following; an EW striking lineament which passes immediately downstream of the proposed centre-line, a prominent NW-striking lineament which follows the main river valley and therefore intersects the proposed centre-line, and at least three relatively minor

NE- and NNE-striking lineaments which traverse the proposed basin. Also refer to Figure 6-5 for further details.

These lineaments might represent possible faults, preferentially weathered diabase dykes or zones of closely spaced joints. As such, these lineaments are recognized as potential weakness zones or seepage paths, and verification of actual conditions is essential at a later detail phase. These lineaments are not considered to represent fatal flaws at this stage.

The respective flanks are covered by loose angular cobbles and boulders with thin, poorly developed soils. In places, outcrop of the sandstone bedrock is noted; in other areas the underlying bedrock is beneath this cover of colluvial cobbles and boulders.

No detailed observation of bedrock outcrop was conducted, but it is expected that the sandstone bedrock on the respective flanks comprises moderately weathered, closely to medium jointed, hard rock sandstone. No information on the jointing is available at this stage, but it might be expected that the main joint sets mirror the orientations of the above-mentioned lineaments. The key orientation would be a set aligned with the main valley, i.e. joints which might represent potential seepage paths. Overall, jointing of the rock mass is expected to be well-developed.

Within the river section bedrock is covered beneath alluvial clayey sands of uncertain thickness, but possibly in the order of 5 m. The condition of the bedrock within this river section is also not known.

6.3.4.2. Site 3

Site 3 is slightly asymmetrical and is characterised by a left flank which is steeper than the right. The river section is relatively wide (Figure 6-7).

Several lineaments are recognized in the vicinity of the proposed site. A prominent NE-striking lineament may be traced along the river valley. Other lineaments are noted which traverse the potential basin and are aligned roughly parallel to the proposed centre-line. The major, inferred fault mentioned previously coincides with the break in slope, i.e. opening of the valley, immediately downstream of the proposed centre-line.

No bedrock outcrop is evident within the river section. The thickness of clayey soils is uncertain but is expected to be substantial (estimated 10 m to 20 m). The condition of the sandstone bedrock beneath the soil cover is uncertain.

The flanks are characterised by shallow overburden comprising poorly developed sandy to gravelly soils and loose, angular cobbles or boulders of weathered sandstone.

Figure 6-7: Terminal Site 3

The photo provided in Figure 6-7 is a view into the basin of proposed Terminal Site 3, from a position roughly coinciding with the centre-line.

The thickness of the unconsolidated overburden on the respective flanks is not expected to exceed 1 m to 2 m. Thicker accumulations may be present at the toe of the respective flanks where talus deposits have collected.

Bedrock underlying the flanks is expected to be deeply weathered; with the rock mass likely comprising moderately to highly weathered, closely to widely jointed, hard rock sandstone. Jointing is likely well-developed, and joint sets are expected to mirror the orientation of the observed lineaments. Joints which are sub-parallel to the NE-striking feature will represent potential seepage paths. Also refer to Figure 6-5 for further details.

6.3.5. Envisaged Founding Conditions and Foundation Treatment

Founding conditions at the respective Site 1 and Site 3 options are broadly similar.

The rock mass underlying the proposed centre-lines is expected to comprise weathered, well jointed sandstone. Stricter founding criteria for a mass concrete dam, as opposed to a rockfill embankment, would favour construction of the latter. In addition, spillway requirements would either be unnecessary or minimal, as these dams do not have large catchments.

Typical foundation treatment for such rockfill structures would require the removal of unconsolidated overburden as well as very poor rock mass conditions only in the area of the impervious cut-off (for a conventional clay core) or the plinth area (for a concrete-faced rockfill option).

The thickness of alluvial deposits in the river section at Site 3 is expected to be substantially greater than at Site 1. For Site 1, expected excavation depths are likely to vary between 1 m and 2 m on the flanks, and up to 5 m within the river section. For Site 3, expected excavation depths are similarly likely to vary between 1 m and 2 m on the flanks, and between 10 m and 20 m within the river section.

The well-jointed, bedded sandstone rock mass is likely to be highly pervious. A programme of foundation grouting is expected to be necessary. Consideration will also have to be given to the water tightness of the respective basins.

6.3.6. Construction Materials

Abundant rock suitable for use as rockfill is available in the immediate environs of the respective dams. No potential quarry sites have been identified at this stage.

The choice of rockfill dam will be largely influenced by the availability of impervious core material. Abundant clayey soils are not expected in this geological environment, favouring construction of a concrete-faced rockfill dam (CFRD). Sand for use as fine aggregate in concrete is also expected to be sourced locally, in part as crusher run during the processing of the coarse aggregate, rockfill and filters.

6.3.7. Recommendations

Follow-up geotechnical investigations would be required at the favoured dam sites, or at the two alternative sites in order to assist with site selection.

A geophysical survey is recommended prior to drilling; with the aim of identifying overburden thicknesses and anomalous areas such as major discontinuities and possible faults.

Actual foundation conditions would need to be verified; a minimum of four boreholes (total length 120 m) would be required at the centre-line. Water pressure tests (Lugeon tests) must be conducted to verify the permeability of the rock mass, and there should be allowance for at least two additional boreholes (total length 80 m) to investigate the basin geology and water tightness.

A potential quarry site for rip-rap, rockfill, coarse aggregate and filters would have to be identified and drilling conducted in order to prove that sufficient volumes of suitable material occur. Depending on the required material volumes, at least six boreholes would be required (total length 200 m).

A laboratory testing programme would be essential, including:

- Determination of the strength and deformation characteristics of the rock material (UCS / point load tests)
- Compliance with the different specifications for coarse aggregate, rockfill, rip-rap and filter specifications as applicable.

6.4. Mokolo Weir Site

A possible site for a diversion weir on the Mokolo River was identified at a position downstream of the gorge where an existing drift crosses the river. Approximate coordinates are Lo 27 Y -75 327, X 2 628 324, on the farm Wonderboomhoek 550 LQ (Figure 6-8).

6.4.1. General Geology

The published 1:250 000 geological map (Sheet 2326 Ellisras, Council for Geoscience) indicates the area is underlain by coarse-grained, purplish brown sandstones of the Mogolakwena Formation of the Kranskop Sub-group, Waterberg Group.

The low-lying areas are covered by Quaternary sandy soils, while the river courses are filled with alluvium.

No major faults are indicated on the geological map, but lineaments striking in a rough north-easterly direction are present, with a prominent south-west striking lineament evident downstream of the proposed weir site (Figure 6-8). In some places diabase dykes have been mapped and it is possible that the lineaments correspond to these dykes, or even minor faults.

6.4.2. Previous Investigations

There is no record of any previous geological investigations conducted at this site.

6.4.3. Site Description

The proposed site is located at the position of an existing drift across the Mokolo River, downstream of the confluence between the Mokolo River and the Rietspruit. Upstream of the confluence the topography is quite rugged, flattening significantly in the area of the confluence and extending northwards.

At the proposed site, the river banks are slightly elevated above the level of the river (estimated 2 m to 4 m); with the respective flanks beyond the river comprising gentle slopes (Figure 6-9).

The existing drift is constructed of dumped boulders and builders rubble and is not indicative of shallow bedrock.

The envisaged structure would likely only be a couple of metres in height and would largely be confined to the present river channel.

6.4.4. Site Geology

There is no evidence of bedrock in the vicinity of the proposed centre-line.

The entire river section is covered with alluvial sands of indeterminate thickness. Estimated thicknesses would be no more than a gross estimate at this point, say 10 m to

20 m, or even more. There appears to be no evidence of boulder beds but these might be present at depth.

The condition of the underlying sandstone bedrock is not known at this stage. Deep weathering is a possibility.

The aerial map provided in Figure 6-8 shows the location of the proposed Mokolo weir Site no 3. It is located downstream of the confluence between the Mokolo River and the Rietspruit, at an existing drift. The drift is shown in the photo provided in Figure 6-9.

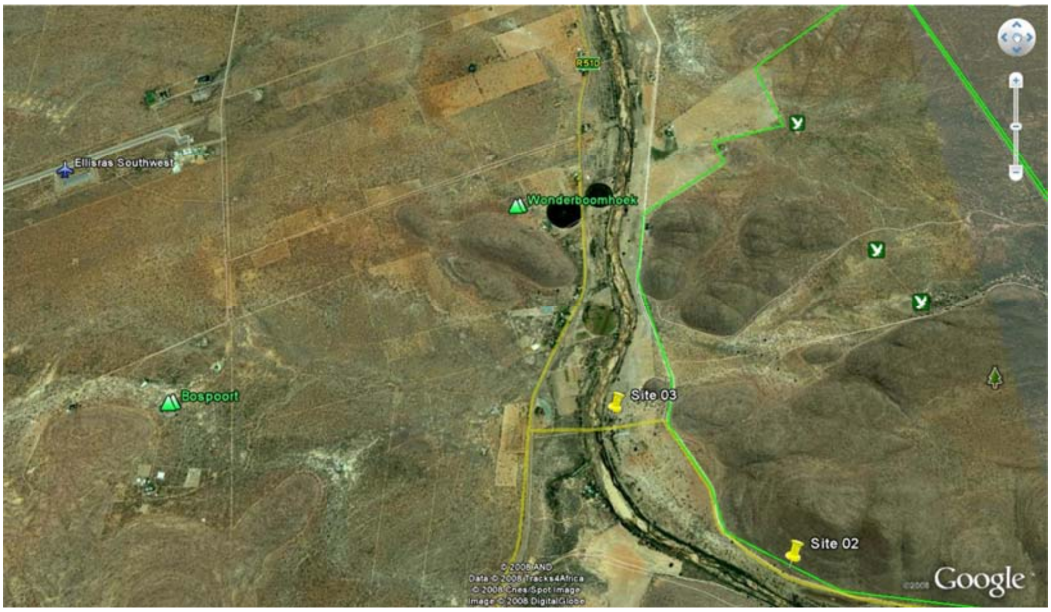

Figure 6-8: Mokolo Site 3 (Map)

Figure 6-9: The proposed Mokolo Site 3 (Photo)

6.4.5. Envisaged Founding Conditions and Foundation Treatment

As described above, the site is characterised by expected thick deposits of alluvial sands.

The envisaged low diversion structure would undoubtedly be subjected to regular flooding and therefore needs to withstand regular overtopping. For this reason a mass concrete structure would appear to be the logical choice.

Conventionally, a concrete structure would be founded on sound bedrock. Because of the expected depths of at least 15 m to 25 m, foundation excavations would be significant. Such excavations in the alluvium would mainly be classed as soft excavation, but some intermediate or even hard excavation cannot be excluded if an upper bedrock horizon is encountered that would also require removal.

If the required founding solution requires excavation to bedrock then difficulties are to be expected within these saturated alluvial sands. Significant seepage problems should be expected due to excavations being below river level, and the highly pervious nature of the alluvial deposits. In addition, temporary excavation slopes in the alluvial sands would need to be shored, or cut back, to ensure worker safety. The impracticalities of conventional excavations within this alluvial environment dictate the need for other approaches; either by utilising other means of cut-off, for example by installing sheet piles, or by other construction techniques such as slurry trench or by using jet-grouted columns.

The actual bedrock condition beneath the alluvial covering materials would dictate the need for additional foundation treatment. An intact, sound rock mass would only require cleaning. Foundation grouting would not need to achieve a ‘sealing’ of the foundation, unless the rock mass proved susceptible to internal erosion. Depending on bedrock condition, a programme of shallow consolidation grouting might be beneficial in improving the integrity of the founding rock mass.

6.4.6. Construction Materials

A concrete weir structure would require both coarse and fine aggregate. Coarse aggregate volumes are not likely to be sufficient to justify opening of a dedicated quarry, and this might favour purchase from commercial sources. At this stage no further work has been conducted in locating possible sources of coarse aggregate.

Fine aggregate (sand) would likely be sourced locally. A test pitting exercise would be required to prove a suitable source. A total of twenty test pits is assumed at this stage. Testing will be required to confirm the materials conform to SABS specifications for fine aggregate. A total of 20 samples are assumed.

The same approach would be followed to test and source materials for the weir flank embankment and balancing dam embankment fills, filters and rip-rap.

6.4.7. Recommendations

Weir:

If the possibility of jet grouted columns is to be considered, then the composition of the alluvial deposits will have to be investigated, specifically whether boulders are present and the diameter of these boulders.

A geophysical survey is recommended prior to drilling; with the aim of identifying overburden thicknesses including anomalous areas, and confirming the location of major discontinuities, such as potential faults, and identifying target areas for limited exploratory drilling.

The bedrock depths as well as the bedrock condition would need to be confirmed. Exploratory drilling is therefore necessary. A total of four boreholes would be required for these feasibility - (total 4 No, length 160 m); comprising two each on the respective river banks, where at least one is angled beneath the river channel, or targeting anomalies if identified during the geophysical survey.

If possible, at least two test pits would be required on the respective river banks (total 4 No). Representative samples would be submitted for laboratory testing which would comprise;

- Foundation indicators, including Proctor compaction (10 No)
- Double hydrometers for dispersivity determination (5 No)
- Chemical testing to determine potential corrosivity (4 No)
- Grading analyses for fill and filter materials and fine aggregates for concrete.

Appurtenant Works:

Sites for the appurtenant works comprising de-silting works, balancing reservoir and high-lift pump station have been identified.

These sites would require the drilling of two boreholes at each site (total four boreholes), drilled 5 m into bedrock, with SPT testing at 5 m grid pattern in the soft overburden. At this stage it is assumed that a total drilling length of 60 m will be required (4 boreholes).

Test pitting is required at the sites of the de-silting works, balancing reservoir and high-lift pump station. At least two test pits are required at each site (total No 4), to be excavated by means of excavator. Representative samples would be submitted for laboratory testing which would comprise;

- Foundation indicators, including Proctor compaction (10 No)
- Double hydrometers for dispersivity determination (5 No)
- Chemical testing to determine potential corrosivity (4 No)
- Grading analyses for fill and filter materials and fine aggregates for concrete.

7. BULK POWER SUPPLY

Bulk power requirements and the investigation for supply to the abstraction weirs are reported on in the Pre-Feasibility Main Report[(8)] (P RSA A000/00/8109).

Note that the cost models presented in Section 10 do not provide for permanent bulk power supply to the abstraction works. Allowances have been made for construction power supply only.

8. RIVER LOSSES

8.1. Crocodile River

8.1.1. Methodology

In this study river losses between the three dams which support irrigation on the lower Crocodile River and the proposed sites at Boschkop and Vlieëpoort were determined by:

a) Determining the irrigation areas from aerial photography.

b) Scaling the total irrigation area to obtain a total area of 15 000 ha as was reported at meetings with Schoeman (2008) and others during this study.

c) Calculation of irrigation requirements based on a total allocation of 8 000 m^3/ha/a and a monthly distribution based on the Schoeman report.

d) Determination of riparian vegetation areas from aerial photography.

e) Use of WR90 to calculate riparian vegetation evapo-transpiration.

f) Setting up a hydrodynamic model of the river to simulate observed base flow releases from the dams to Vlieëpoort, with irrigation and evapo-transpiration added as abstractions on the river reaches, and surface evaporation calculated by the model. The simulated flows were compared with the observed flows recorded at gauging stations near Boschkop and Vlieëpoort. The net difference between the observed and simulated flows is the river losses (or gains) due to seepage, tributary inflows, return flows, and possible illegal water use. It is assumed that these losses will remain the same in future with possible increased river flows.

It was assumed that the following were unaffected by additional releases from the dams:

- Run-off accruals;
- Return flows;
- Seepage losses; and
- Evapo-transpiration of the riparian vegetation.

Aerial photography of the Lower Crocodile River (West) showing the three dams and two abstraction sites are shown in Figure 3-1 to Figure 3-4 in Section3.1.1. Figure 3-9 and Figure 3-10 in Section 3.2.1 show the proposed abstraction works sites on the Crocodile River (West).

8.1.2. Irrigation Areas

The irrigation areas determined from satellite images were calculated with ACAD as shown in Table 8.1 (second column), with a total area of 17 487 ha. This area was scaled to obtain a total of 15 000 ha (last column). 36% of the irrigation area is upstream of Boschkop and 64% between Boschkop and Vlieëpoort, based on the satellite images. The main channel river surface area and riparian vegetation areas are also indicated in Table 8-1.

Table 8-1: Irrigation, Evapo-transpiration and Riparian Vegetation Areas.

Location	Google measured Irrigation area (ha)	Area from DWA report-Schoeman report 2008 (ha)*	Riparian vegetati on area (ha)	River Main channel area (ha)	Schoeman meeting 11 Sep 2008 (ha)	Scaled irrigation area (ha)
From downstream of 3 Dams to Boschkop	6 256	170,5	997	269	3 000	5 366
From Boschkop to Vlieëpoort	11 232	2 062,6	1 527	221	12 000	9 634
Total	**17 487**	**2 233**	**2 524**	**490**	**15 000**	**15 000**

Note: * surface and borehole water.

8.1.3. Simulation of Current Losses

Table 8-2 shows the simulated and observed flows along the river, with river losses calculated. The river losses are met and represent tributary inflows, return flows and seepage losses. From the simulations the results indicate negative losses of 21,8 Million m^3/a at Boschkop and 7,6 Million m^3/a at Vlieëpoort. This means there is a net inflow after evapo-transpiration and evaporation losses were considered. The analyses have been performed on the basis of a so-called first-come-first-served-abstraction of irrigation water.

Table 8-2: Current Condition River Losses in Addition to Evaporation and Evapo-transpiration

Year	Irrigation releases from dams	Simulated flow at Boschkop (m^3)	Simulated flow at Vlieëpoort (m^3)	Observed flow at Boschkop (m^3)	River losses at Boschkop (m^3)	Observed flow at Vlieëpoort (m^3)	River losses at Vlieëpoort (m^3)
1986 - 1987	98 874 199	60 218 033	27 493 684	71 647 193	-11 429 159	28 879 338	-1 385 654
1987 - 1988	87 184 680	38 575 839	11 215 201	68 891 916	-30 316 076	21 906 191	-10 690 990
1988 - 1989	107 276 793	58 502 946	24 398 789	80 264 379	-21 761 433	36 862 811	-12 464 021
1989 - 1990	115 320 893	65 237 082	23 432 606	90 376 889	-25 139 808	45 306 673	-21 874 068
1990 - 1991	122 605 902	71 940 986	25 612 744	105 929 824	-33 988 838	53 388 241	-27 775 497
1991 - 1992	116 784 295	65 616 916	32 823 733	76 828 140	-11 211 225	22 910 055	9 913 678
1992 - 1993	78 653 334	40 621 171	16 491 248	54 247 917	-13 626 747	13 473 583	3 017 665
1993 - 1994	85 724 767	42 612 718	11 177 468	68 639 725	-26 027 007	21 234 681	-10 057 213
1994 - 1995	104 542 625	53 807 203	19 242 837	76 680 471	-22 873 268	15 952 466	3 290 370
Average:	**101 885 276**	**55 236 988**	**21 320 923**	**77 056 273**	**-21 819 284**	**28 879 338**	**-7 558 415**

Notes:
1. Excluding net river losses.

8.1.4. River Losses at Boschkop and Vlieëpoort with Future Increased Flows

The river flows available at Boschkop are indicated inTable 8-3, and for Vlieëpoort in Table 8-4 on the basis of a so-called first-come-first-served-abstraction of irrigation water.

Table 8-3: River Flows at Boschkop Site

Description	Current (Million m^3/a)	Current + 7.5 m^3/s dam releases (Million m^3/a)	Current + 10 m^3/s dam releases (Million m^3/a)
Dam releases	101.9	338.6	417.5
Simulated river flow at site	55.2	283.7	362.2
Adjustment (loss/gain)	-21.8 *	-21.8 *	-21.8 *
Available flow at site **	77.0	305.5	384.0
Unmet irrigation demands	5.9	0	0
Irrigation demand downstream of site	106	106	106
Net flow at site available for abstraction	0	199.5	278
% Water available for transfer	0%	59%	67%

Note: * Return flow, tributary inflow, possible illegal or reduced irrigation water use and seepage, based on “river losses” in Table 8-2.

** Available flow includes for downstream observed flow requirements: 77 Million m^3/a to Vlieëpoort.

Table 8-4: River Flows at Vlieëpoort Site

Description	Current (Million m^3/a)	Current + 7.5 m^3/s dam releases (Million m^3/a)	Current + 10 m^3/s dam releases (Million m^3/a)
Dam releases	101.9	338.6	417.5
Simulated river flow at site	21.3	191.2	267.9
Adjustment (loss/gain)	-7.6	-7.6	-7.6
Available flow at site *	28.9	198.8	275.5
Unmet irrigation demands	64.1	1.84	0.44
Irrigation demand downstream of site	28.9	28.9	28.9
Net flow at site available for abstraction	0	168.1 **	246.2 **
% Water available for transfer	0%	50%	59%

Note: * Available flow includes observed flow requirements downstream of Vlieëpoort: 4 459 ha x 8 000 m^3/ha/a = 35,7 Million m^3/a, or 14,5 Million m^3/a if only the surface water use is considered based on the Schoeman report (2008). Say 28.9 Million m^3/a is required for downstream irrigation.

** Unmet irrigation demands in future scenarios were assumed would be met by improved release patterns and/or change in the irrigation demand pattern.

From Table 8.3 and Table 8.4 it is clear that under current conditions the irrigation requirements exceeded the actual historical releases from the dams upstream and therefore the requirements could not be met in the hydrodynamic model. In the future scenarios some irrigation failures occur but these are limited and occur downstream of Boschkop. The consequences of the unmet irrigation requirements under current conditions is that in future increased flow scenarios one could expect more irrigation from the river, and therefore the "losses" in future scenarios would be relatively high, unless the irrigation water use is managed or controlled to existing use.

Figure 8-1 shows the results graphically for uncontrolled irrigation abstraction.

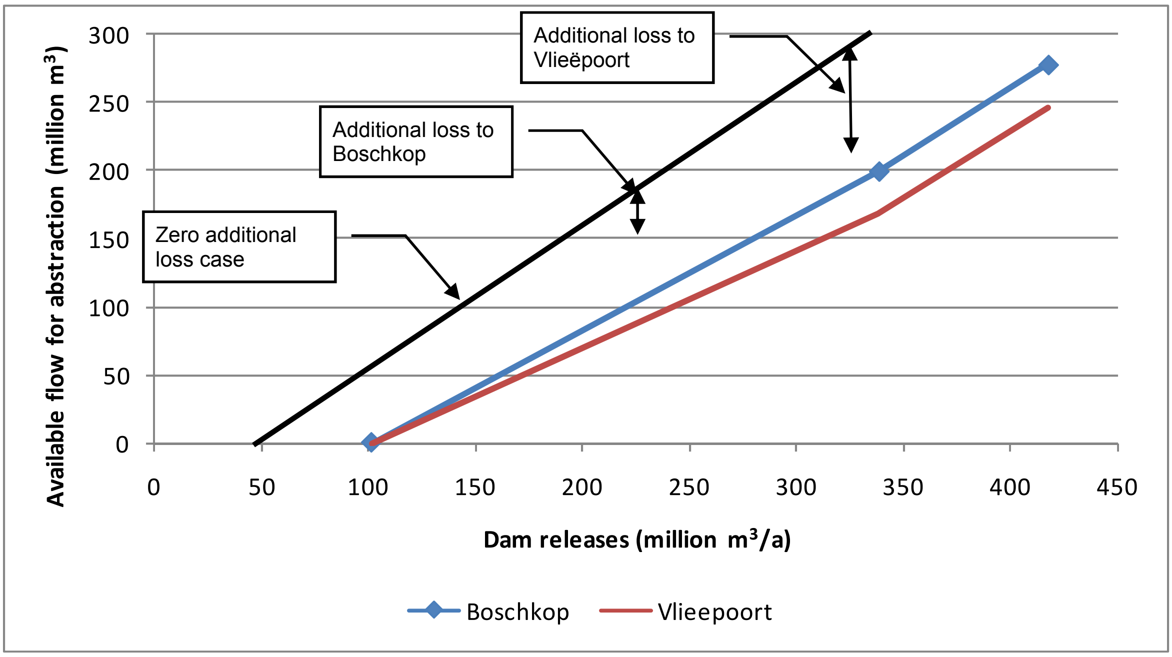

Figure 8-1: Available Flow for MCWAP Abstraction at Boschkop and Vlieëpoort Sites

Figure 8-2 to Figure 8-4 show the requested irrigation and simulated supplied irrigation for current and future scenarios for irrigation between Boschkop and Vlieëpoort along the Crocodile River (West). Note that abstraction from the river is treated as negative flow in the model.

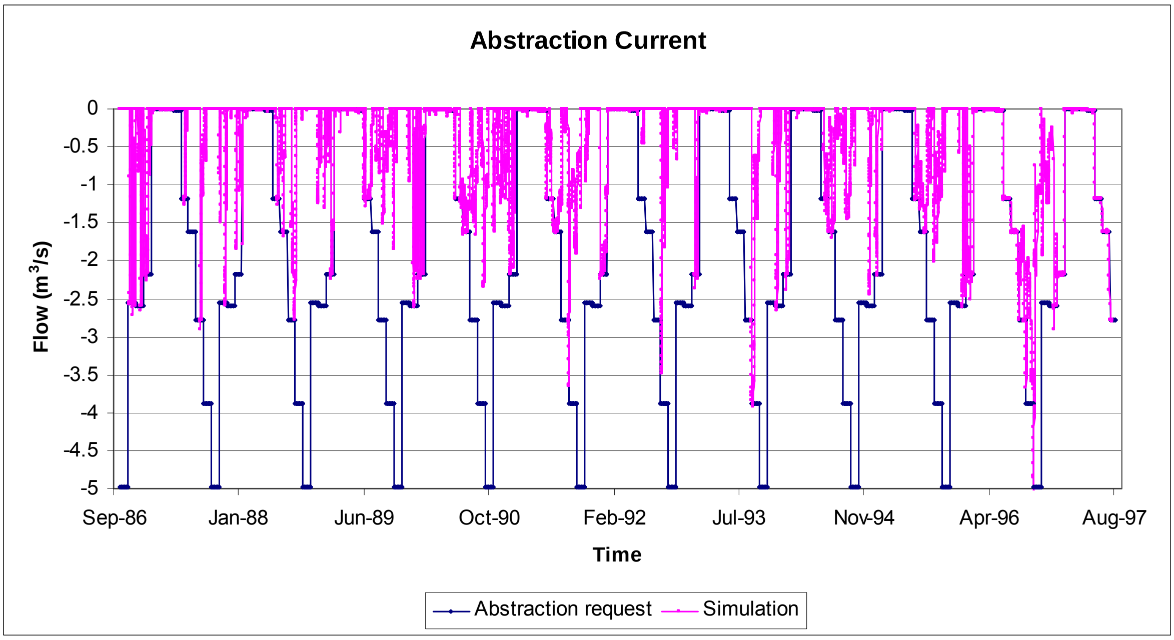

Figure 8-2: Current Scenario Irrigation Supply between Boschkop and Vlieëpoort

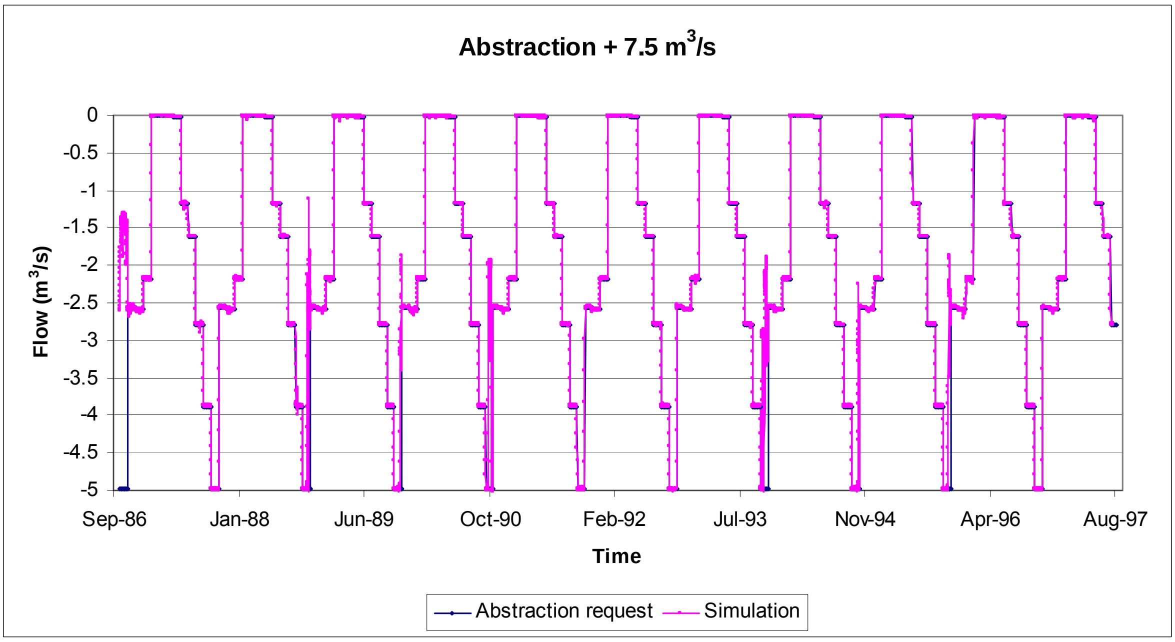

Figure 8-3: Future Scenario Irrigation Supply between Boschkop and Vlieëpoort

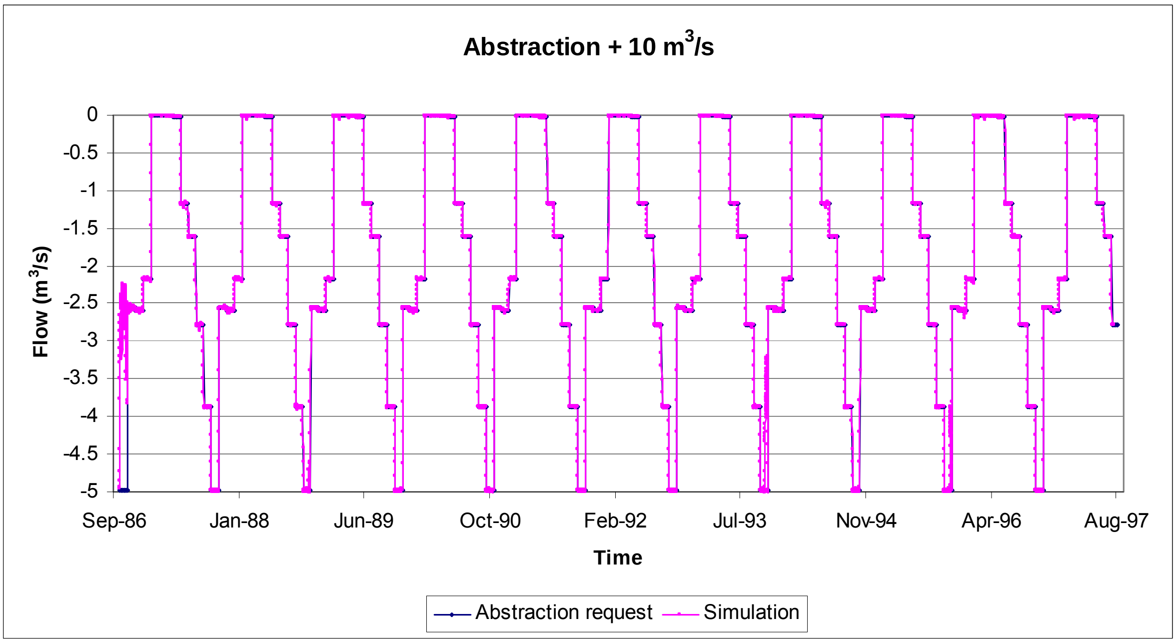

Figure 8-4: Future Scenario Irrigation Supply between Boschkop and Vlieëpoort

The simulated flows at Boschkop and Vlieëpoort for different scenarios are shown in **Appendix C**. Note that for the calculations of flows in this report, the current scenario flows were cut off at 6.59 m^3/s at Boschkop and Vlieëpoort (based on the discharge table limit of one of the flow gauging stations; in future scenarios the cut-off was 6.59+7 m^3/s or 6.59+10 m^3/s).

It is clear that the releases for the MCWAP will not be constant, but will have to be varied to suit downstream conditions.

A further analysis of the above results has made it possible to estimate the additional water losses by evaporation from the water surface and the potential additional water uses by the irrigators if the releases from the dams were increased to supply the MCWAP and if the water uses by the irrigators (current and future) were in proportion to the irrigated areas upstream and downstream from Boschkop respectively. The findings are given in Table 8-5.

Table 8-5: Additional River Losses and Irrigation Water Use with Increased Water Releases from the Dams

Description	**Dams to Boschkop**		**Dams to Vlieëpoort**	
Additional dam releases (m^3/s)	7,5	10,0	7,5	10,0
Additional water surface evaporation (million m^3/a)	2,1	2,5	4,3	5,1
Additional irrigation use (million m^3/a)	22,9	22,9	64,1	64.1
TOTAL ADDITIONAL LOSSES (million m^3/a)	**25,0**	**25,4**	**68,4**	**69,2**

Increased releases from the dams to supply the MCWAP would therefore mean that the potential loss of water released from the dams is about 25 Million m^3/a and 69 Million m^3/a at Boschkop and Vlieëpoort, respectively. It is unlikely that the irrigation abstractions can be fully controlled and therefore the above losses at Boschkop and Vlieëpoort can only be reduced by some proportion of the potential losses of 23 Million m^3/a and 64 Million m^3/a.

Given the high cost of the water made available in the Crocodile River (West) for use by the MCWAP it is essential to manage the river and the abstractions and incur the necessary costs. The benefits that the irrigators are likely to derive from unauthorised abstractions must also be addressed.

8.2. Mokolo River

8.2.1. Methodology

In this study river losses between Mokolo Dam which supports irrigation and mine water use requirements on the Mokolo River, and the proposed abstraction site downstream of the dam were determined by:

a) Determining the irrigation area between the dam and the abstraction site from aerial photography.
b) Calculation of irrigation requirements based on a total allocation of 8 000 $m^3/ha/a$.
c) Determination of riparian vegetation areas from aerial photography.
d) Use of WR90 to calculate riparian vegetation evapo-transpiration.
e) Setting up a hydrodynamic model of the river to simulate observed base flow releases from the dam, with irrigation and evapo-transpiration added as abstractions on the river reaches, and surface evaporation calculated by the model. The simulated flows could not be compared with observed flows since there is no flow gauging station downstream of the dam. No tributary inflows, return flows or possible unauthorised water use was considered.
f) It was assumed that the evapo-transpiration losses and irrigation requirements would remain the same in future scenarios

Figure 8-5 and Figure 3-11 in Section 3.2.1 show the possible abstraction works at Site 3 on the Mokolo River.

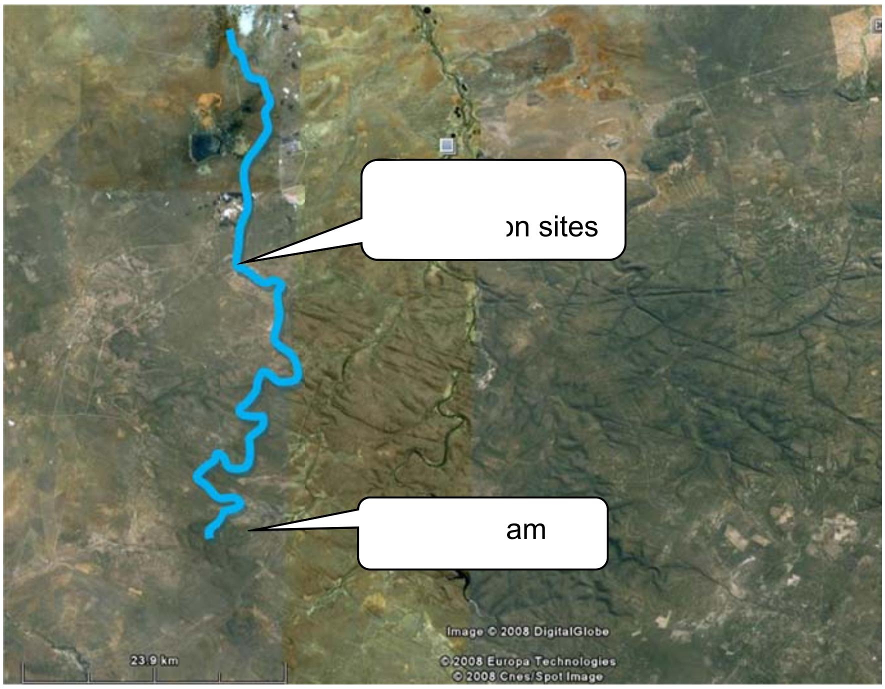

Figure 8-5: Aerial Photography of Mokolo River Study Area

8.2.2. Mokolo River Irrigation Requirements

The current total irrigation area along the river is about 1 800 ha according to the Irrigation Board, but this is downstream of the possible abstraction site. Schoeman and Joubert (2007)[14] carried out a study for DWA to quantify the irrigation areas and irrigation losses. However, in this study the irrigation is downstream of the possible abstraction works and was therefore not considered in the hydrodynamic model.

8.2.3. Simulation of Current Losses

The possible abstraction site (39 km downstream of the dam) is upstream of the current irrigation area of about 1800 ha. No downstream flow gauging station exists.

For the current scenario (Table 8-6) the base flow release at the dam observed historically was on average 33 Million m^3/a, but quite variable. The simulated losses from the river between the dam and the abstraction site are 6 Million m^3/a net.

Based on information received during this study, Exxaro releases about 29 Million m^3/a, of which 16 Million m^3/a goes to the farmers along the entire river. Of the 29 Million m^3/a Exxaro uses less than 10 Million m^3/a and the estimated river loss was 6 Million m^3/a before this study was carried out. This is in agreement with the simulated loss value found in this study.

Table 8-6 shows the simulated flows along the river, based on observed base flow releases at Mokolo Dam.

Table 8-6: Current Conditions Observed Dam Releases and Simulated Flow at Abstraction Works Site (including losses)

Year	Observed low flow dam releases (m^3/a)	Simulated flow at site (m^3/a)
1987 -1988	11,196,477	7,542,737
1988 -1989	22,895,696	16,223,065
1989 -1990	24,456,568	16,104,267
1990 -1991	42,866,334	33,701,744
1991 -1992	22,620,620	16,137,020
1992 -1993	8,947,265	6,473,686
1993 -1994	22,544,491	17,591,194
1994 -1995	7,959,101	5,245,522
1995 -1996	73,215,263	65,019,888
1996 -1997	77,062,946	68,392,285
1997 -1998	19,311,983	14,541,142
1998 -1999	31,337,604	24,711,057
1999 - 2000	78,833,300	71,109,771
2000 - 2001	48,471,677	41,296,442
2001 - 2002	42,197,639	33,613,944
2002 - 2003	11,369,895	6,939,302
2003 - 2004	46,277,024	41,164,411
2004 - 2005	12,882,050	8,652,569
2005 - 2006	47,898,316	43,038,791
2006 - 2007	15,435,884	10,924,044
Average	**33,389,007**	**27,421,144**

8.2.4. River Losses with Future Increased Dam Releases

The river flows available at the abstraction site are indicated in Table 8-7. If the downstream irrigation remains 1 800 ha and requires 8 000 m^3/ha/a and Exxaro uses 10 Million m^3/a from the river, under current conditions about 9% of the flow released from the dam (3 Million m^3/a) is available on average for abstraction. This value is small as expected. In future increased dam releases selected as 1 m^3/s and 2 m^3/s added to the observed dam release record, 30.9 and 61.7 Million m^3/a would be available at the abstraction site for transfer respectively.

Table 8-7: River Flows at Abstraction Site

Description	Current (Million m^3/a)	Current +1 m^3/s dam release (Million m^3/a)	Current +2 m^3/s dam release (Million m^3/a)
Mokolo Dam release	33.4	64.9	96.5
Simulated river flow at site	27.4	55.3	86.1
Unmet Irrigation demands*	0	0	0
Irrigation demand downstream of site	14.4	14.4	14.4
Exxaro	10	10	10
Net flow at site available for abstraction	3.0	30.9	61.7
% Water available for transfer	9%	48%	64%
Losses (Million m^3/a)	6,0	9,6	10,4

Note: *Irrigation demand upstream of site 191 ha x 8000 m^3/ha/a + 50 Houses x 6people @ 100 ℓ/person/day = Upstream water requirement of 1.54 Million m^3/a

Figure 8-6 shows the simulation results graphically.

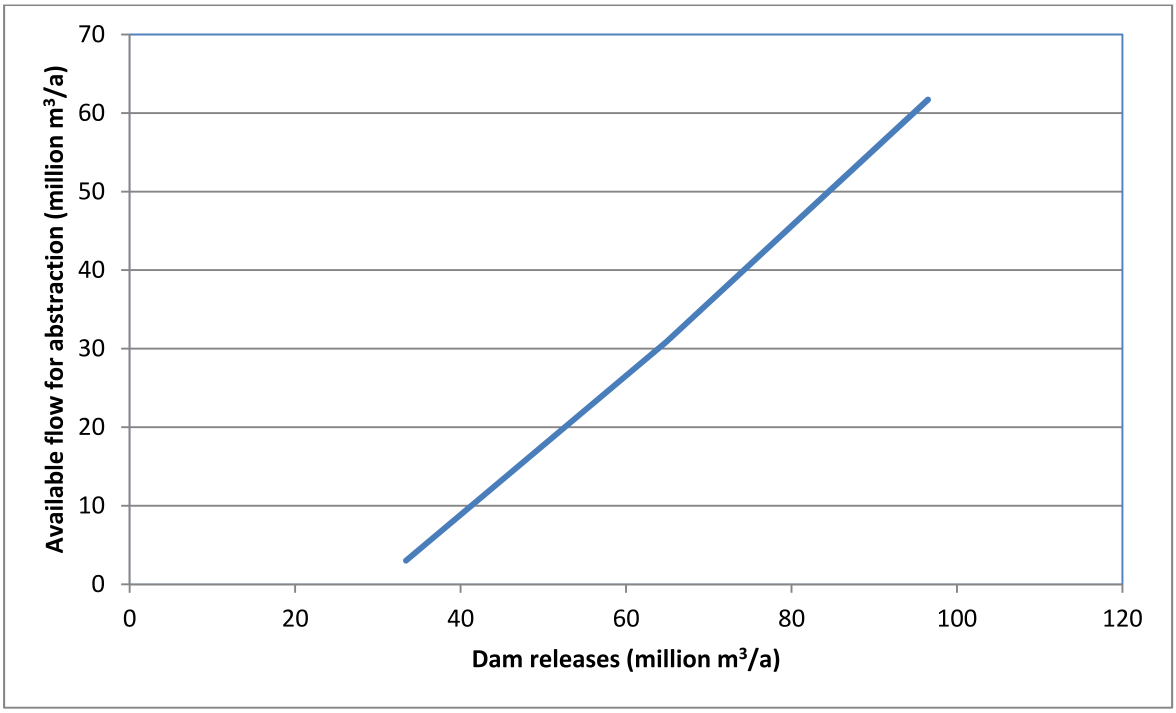

Figure 8-6: Available Flow for Transfer at Abstraction Sites

The simulated flows at the abstraction site for different scenarios are shown in **Appendix C**. Note that for the calculations of flows in this report, the current scenario flows were cut off at 3.5 m^3/s at Mokolo Dam and the abstraction site, while in future scenarios the cut-off was 3.5+1 m^3/s or 3.5+2 m^3/s).

9. COMPONENT SIZES

9.1. Weirs and Abstraction Works

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

(1) Weir OC and furthest gravel trap inlet at the same level with 1:20 slope from gravel trap inlet to radial gate at the outlet end of the trap that will be incorporated into the weir. Weir height 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 5 m radius roller bucket for energy dissipation.

(4) The weir NOC will be located at the DFL plus 1 m freeboard.

(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.

(6) The weir flank cut-off walls (tongue walls) will intrude 5 x maximum differential head into the riverbanks.

(7) Maximum flow velocity in low-lift pump station lead-in channels not to exceed 0.9 m/s.

(8) Low-lift pump station working level above PMF plus 0.5 m freeboard. Working level will also have vehicular access.

(9) Low pressure pipeline to be protected against flood damage.

(10) De-silting works will have a channel for each low lift pump, with one standby unit. Maximum flow velocity in de-silting channels not to exceed 0.3 m/s. Freeboard was selected at 0.5 m.

(11) The de-silting works outlet arrangement will allow staged re-commissioning of the works as de-silting and maintenance operations on the channels are completed to minimise system down time.

(12) Balancing reservoir will have a live storage capacity of 4 hours to allow for de-silting and maintenance of the de-silting works. This will typically result in a 100 x 300 m plan area dam. The MOL of the balancing dam will be 8m above the high-lift pumps in the adjacent high-lift pump station. No separate allowance for maintenance of the balancing reservoir was made (for example by providing an additional compartment in the reservoir) and such requirements were assumed to be included in the planned maintenance provision for the overall scheme.

(13) The balancing dam outlet will be sized to drain and de-silt the dam within 1 hour. This operation was anticipated to take place in 10-year cycles.

9.2. Terminal Dam/Reservoir Sizing

The following criteria were used for sizing the terminal dam and reservoirs (as applicable). Note that the work on the Terminal dams and Reservoirs were only done to concept level. Future tense is therefore for work that would have been done at Pre-feasibility and Feasibility level.

(1) Live storage based on 18 days of maximum average annual demand, based on overall system availability criteria. The live storage capacity currently being

considered are 10.7 Million m^3 and correspond to the maximum average required transfer plus the 9% allowance for seasonal peaks (7,4 m^3/s for 5% downtime in 365 days).

(2) Gross Basin Capacity is determined as Live Storage plus 5% nominal plus allowances for a) Sedimentation and erosion; b) Evaporation losses, and c) Seepage losses.

(3) A sedimentation rate of 49 tons/km^2 per annum for Sediment Region 1 has been adopted for the calculations for the Terminal dam. The erosion factor F is given in the table below. For 80% assurance, the average rate was multiplied by a factor of 3 and applied over 50 years. An average SG of 1,35 was used to convert sediment mass to cubic meters.

(4) Erosion in the catchments were considered and the multiplication factor of 1,23 was determined.

(5) The total evaporation from the dams' surface will be based on Mokolo Dam's S-Pan mean annual evaporation of 2 031 mm and Rainfall Zone A4E with a MAP of 550 mm. The Terminal dam may receive some water from the catchment, but is has been accepted that all the natural catchment runoff will be released for the Ecological Reserve (ER). The evaporation allowance included in (2) above has been based on a full dam and transferring water at the average annual net evaporation rate and providing balancing storage to cater for the seasonal variation in the net rate of evaporation.

(6) Seepage losses should be assessed from a geological study of the dam basin and hydraulic permeability test results. Based on impressions from the initial site visit and a desk study of contours and aerial photographs, Site 2 probably has the least permeable basin, followed by Site 1. Seepage losses should enter the groundwater and recovery of seepage (and other) losses by pumping directly from groundwater may be a cost effective alternative to pumping make-up water from the Crocodile River (West).

(7) Spillway sizing for the RDD and the SED has assumed the spillway crest level and the FSL, or maximum operating level of the Terminal dam to be the same. The pre-feasibility sizing of the spillway and associated freeboard for the Terminal dam will be based on the SANCOLD, (1991) Guidelines on Safety in Relation to Floods for Category III dams with significant hazard rating. The total freeboard requirements will be finalised during the feasibility stage. The RDF and SEF should be based on the SANCOLD, 1991[(9)], Guidelines on Safety in Relation to Floods.

(8) The design of the inlet/outlet works will depend, to an extent, on the elevation of the dam and operating water levels in relation to the preferred hydraulic gradient of the transfer system and the route selected for the pipelines to and from the Terminal dam. The inlet-outlet works can be combined into one structure, or separated depending on the required connections with the transfer system. If a bottom inlet arrangement is adopted to conserve energy, two individual inlet pipelines should be provided and sized to convey the peak flow rates specified for the Delivery Pipeline within the specified head loss limits.

(9) The river outlet works will be sized to pass the ER independently of the outlet to the delivery pipeline. The outlet works to the delivery system will be bifurcated. Subject to water quality requirements. A multi-level intake tower should be considered to deal with water quality.

(10) Engineering interpretation of the available geotechnical information for foundation assessments, grouting, proportions of hard and soft excavation and the like would

have been done during the Pre-feasibility investigation and later stages and not during the conceptual level investigation at the end of which the work on the Terminal dams and Reservoirs were brought to a close by DWA. The same applies to points (6) to (8) above.

9.3. M&E Associated with the Low-Lift and High-Lift Pump Stations

These aspects are dealt with in Supporting Report 5[(4)] and Supporting Report 6[(5)] as applicable. This report only dealt with the civil works associated with the pump stations as these structures formed an integral part of the abstraction works.

10. CAPITAL COSTS

10.1. Pre-Feasibility Level Costing

Capital cost estimates were undertaken using the cost models described in Supporting Report 3(3) as basis. The standard dams' model was modified to create models for the abstraction weirs, de-silting works and balancing dams. Cost models for each of the structures considered are included in **Appendix A**.

A summary of the estimated capital costs associated with each of the components that were studied at pre-feasibility investigation level are included in Table 10-1.

Table 10-1: Estimated Capital Costs for Abstraction Works (excluding M&E)

Item No.	DESCRIPTION OF COMPONENT	Scenario 4 Rand	Scenario 8 Rand
1.	**Mokolo Works**		
1.1	Abstraction Weir and Low-Lift Pump Station Civil Works	153 462 000	153 462 000
1.2	Low-Lift Pump Station M&E Works	Refer Supporting Report 5	Refer Supporting Report 5
1.3	De-silting Works	20 984 000	20 984 000
1.4	High-Lift Pump Station Balancing Dam	28 191 000	28 191 000
1.5	**Total Cost**	**202 637 000**	**202 637 000**
2.	**Boschkop Works**		
2.1	Abstraction Weir and Low-Lift Pump Station Civil Works	173 894 000	173 894 000
2.2	Low-Lift Pump Station M&E Works	Refer Supporting Report 6(5)	Refer Supporting Report 6(5)
2.3	De-silting Works	32 332 000	51 731 000
2.4	High-Lift Pump Station Balancing Dam	56 392 000	90 226 000
2.5	**Total Cost**	**262 618 000**	**315 851 000**
3.	**Vlieëpoort Works**		
3.1	Abstraction Weir and Low-Lift Pump Station Civil Works	155 555 000	155 555 000
3.2	Low-Lift Pump Station M&E Works	Refer Supporting Report 6(5)	Refer Supporting Report 6(5)
3.3	De-silting Works	29 788 000	47 660 000
3.4	High-Lift Pump Station Balancing Reservoir	36 571 000	58 512 000
3.5	**Total Cost**	**221 914 000**	**261 727 000**

Notes:

1. The cost estimates for Scenario 4 was not calculated to the same level of detail employed for the Scenario 8 estimates. As the weir presents 60% of the cost of the structure and will remain essentially unchanged for the Scenario 4 design, savings amounting to only

approximately 15% of the Scenario 8 estimates have been allowed for should Scenario 4 materialise in the Pre-feasibility stage.

2. The costs of pipework, valves, screens and cranage have been included in the civil works portions of the cost estimate.
3. The costs of the pumps and any M&E control equipment required are not included. For the purposes of the Pre-feasibility stage these costs have been included with the pump station costs in Supporting Report 6[(5)].

Unit rates were based on an April 2008 base date. Further details on the derivation of the unit rates can be found in Report 3[(3)]: Guidelines for Preliminary Sizing, Costing and Economic Evaluation of Development Options.

Because of a lack of survey and geotechnical data the contingency provision was set relatively high at 20%.

Quantities were calculated using the Pre-feasibility stage sizing drawings included in **Appendix B** of this Report.

10.2. Conceptual Level Costing

10.2.1. Terminal Dams

The high terminal dams on the farm Witvogelfontein and the raising of Mokolo Dam was only investigated at conceptual level (for the reasons discussion elsewhere in the report). Cost functions were prepared for these structures to provide the required costing information during the early stages of the Study. The cost functions for the Terminal dams options investigated are included as Figure 10-1.

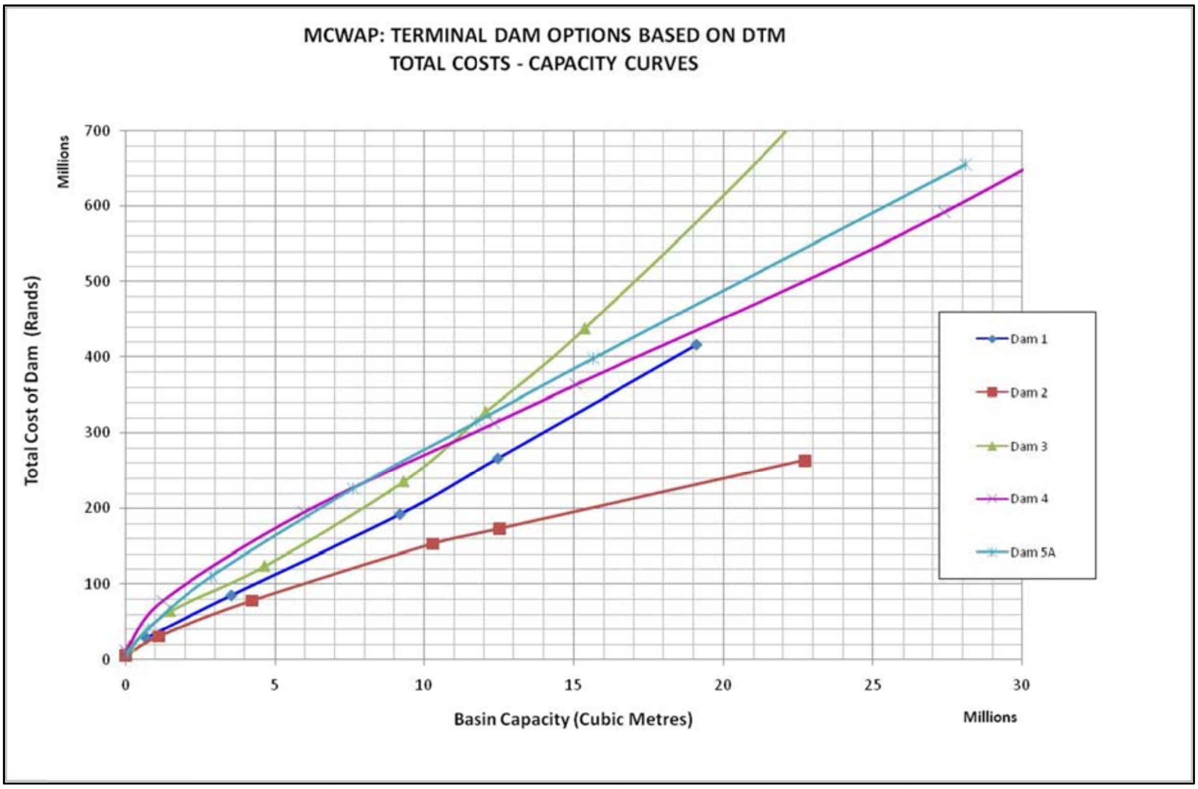

Figure 10-1: Cost Functions for the Terminal dam Options Investigated

Also refer to Section 3.1.5 above for a description of the Terminal Dam Options that were investigated.

The costing for the Terminal dam options were converted to cost model format before the work was finally terminated. Additional provision for contingencies were made in the cost models and explains the difference between the costs derived from Figure 10-1 and those listed in Table 10-2. Details of the cost models are provided in **Appendix A**.

Table 10-2: Estimated Capital Costs of Terminal Dams

Item No.	Description of Component	Cost Rand
1.	**Terminal dams (Total Storage Capacity = 11,2 Million m^3) – Scenario 8**	
1.1	Site 1	281 751 000
1.2	Site 2	215 781 000
1.3	Site 3	342 364 000
1.4	Site 4	345 273 000

The estimated storage capacity of the Terminal dam at the time that the work was concluded was 11,2 Million m^3. Since then the final assessment has been refined to 10,7 Million m^3. Schematic drawings used for the costing of the Terminal dams are provided in **Appendix B**.

10.2.2. Raising of Mokolo Dam

As shown on the drawings in **Appendix B**, three labyrinth options were considered. In addition two straight uncontrolled concrete ogee type spillway designs were investigated and conceptual level cost estimates prepared (presented in Section 10 and in **Appendix A**). The options considered were:

- Option 1: Labyrinth spillway, FSL 916,50 masl, NOCL = 922,0 masl
- Option 2: Labyrinth spillway, FSL 929,30 masl, NOCL = 934,0 masl
- Option 3: Labyrinth spillway, FSL 916,80 masl, NOCL = 922,0 masl
- Option 4: Ogee type spillway, FSL 913,83 masl, NOCL = 922,0 masl
- Option 5: Ogee type spillway, FSL 925,80 masl, NOCL = 934,0 masl

The options were costed and the results are presented in Figure 10.2 below.

Also refer to Sections 3.1.6 and 3.2.4 above for a description of the Mokolo Dam Raising Options that were investigated.

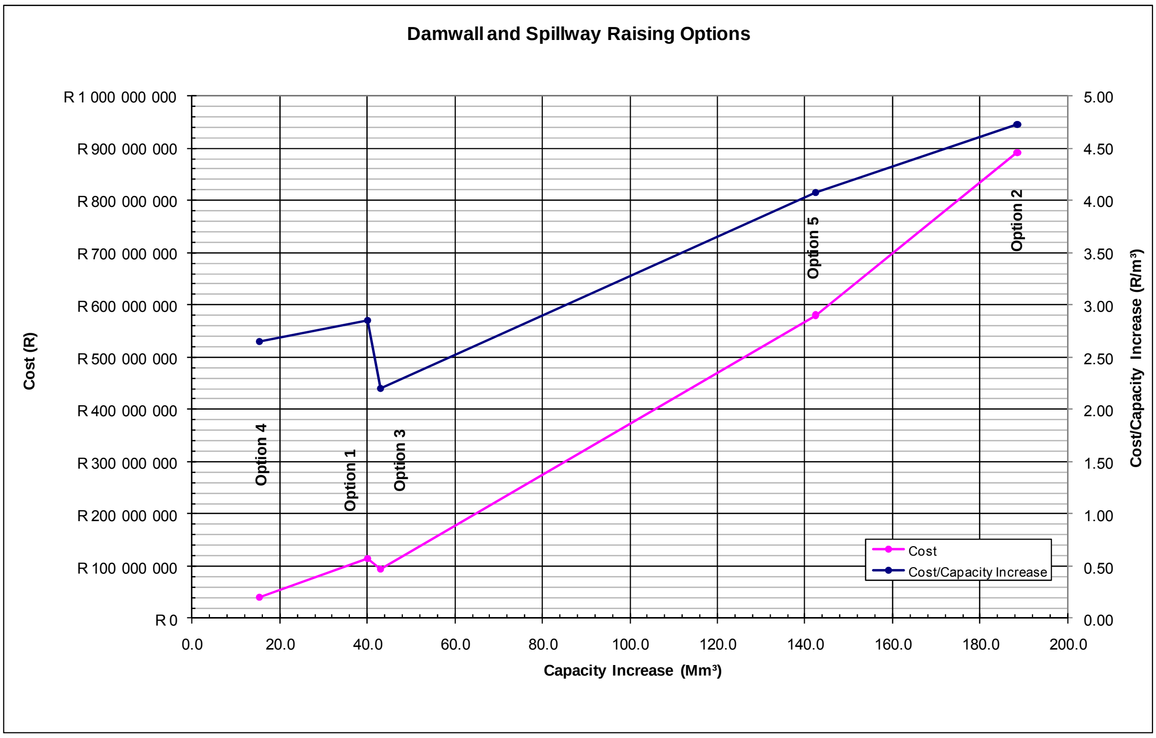

Figure 10-2: Cost Functions for the Raising of Mokolo Dam Options Investigated

10.2.3. Terminal Reservoirs

A summary of the estimated capital costs associated with each of the terminal reserviors that were investigated and sized at conceptual level are included in Table 10-3.

Table 10-3: Estimated Capital Costs of Terminal Reservoirs

Item No.	Description of component	Net Volume (m^3)	Cost Rand
1.	**Terminal reservoir/Balancing Dams – Scenario 8**		
1.1	User Terminal reservoir - Zealand	624 100	148 309 000
1.2	User Terminal reservoir – Exxaro Lephalale	1 090 800	259 214 000
1.3	User Terminal reservoir – Eskom Lephalale	880 000	209 120 000
1.4	User Terminal reservoir – Steenbokpan	1 396 400	331 835 000
1.5	User Terminal reservoir – Exxaro Steenbokpan	306 000	72 717 000
1.6	User Terminal reservoir – Sasol Steenbokpan	3 700 000	879 254 000
1.7	User Terminal reservoir – Eskom Steenbokpan	2 250 000	534 681 000
1.8	**Total for all User Terminal reservoirs**	**10 247 300**	**2 435 130 000**
1.9	Total Storage Provision (single Terminal reservoir) (2).	10 050 000	1 582 502 000

Notes:

1. Only evaluated at conceptual level for Scenario 8 as the provision of the user terminal reservoirs will be the responsibility of the bulk consumers who will also operate and maintain the reservoirs and is therefore not a MCWAP responsibility. Also see Supporting Reports 6(5) and 10(7) for further details.
2. Final sizing requirements subsequently determined to be 10,7 Million m^3.
3. As the locations of these structures are unknown (the users considered the information as confidential) the same assumptions that were made in the other cost models for land acquisition and cost of relocations were applied.

The ownership of the terminal reservoirs was confirmed to be that of the users and no further work past the conceptual stage was done. Also refer to **Appendix B** for details of a typical layout of a terminal reservoir.

10.3. General Notes on Costing Models

The following general notes apply to all the models included in **Appendix A**.

(1) Items with zero quantities. These items are either not expected to be used for the proposed solution or there may not be enough information available to make a reasonable estimate at this stage. Provision for these unknowns is made in the contingency allowance. For the Pre-feasibility stage this has been fixed at 20%.

(2) Preliminary Works. First order estimates of the costs of basic infrastructure requirements before construction can commence.

(3) For ease of reference to Supporting Report 3(3), item numbers and payment reference numbers have been kept the same for all the models. This has unfortunately resulted in, for example, item 2 (river diversion works) not appearing in the models for the de-silting works and balancing dams.

(4) The following allowances were made:

- Railhead costs. An allowance of 1.5% of the quantity proportional cost of the works was allowed.
- Costs of relocation and land acquisition costs. A total overall and all inclusive cost of R50 000 per hectare to be purchased was assumed. For the purposes of the Pre-feasibility cost estimate the sum allowed for the cost of relocations was taken to be the same as the amount allowed for land acquisition. The present layouts will not affect people living on the land, but the indirect costs (for example sterilisation of portions of the land) would be difficult to quantify at this stage.

11. UNIT REFERENCE VALUES

No independent unit reference values were calculated for the scheme components discussed in this report. Capital cost estimates were carried forward to Report 5[(4)]: Mokolo River Development Options and Report 6[(5)]: Water Transfer Scheme Options where URV calculations for the various scheme options were done and reported on.

12. LAYOUT DRAWINGS

A drawing register of the Pre-feasibility Stage drawings that was prepared for the Study is presented in Table 12-1.

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 weir site layouts.
- 2003 Aerial photography. Photographs were available for the Mokolo River, but none were available for the section of the Crocodile River (West) 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.

It is recommended that the Feasibility stage be based on the detailed survey drawings that should be forthcoming from the proposed aerial survey and mapping contract to be let at the end of the Pre-feasibility stage of the study.

Table 12-1: Drawing Register

Project:	Series:	Component:	Drawing Number:	Title:
Boschkop Abstraction Works				
WP 9528	LD	BKW	001	Boschkop Site Layout
WP 9528	DD	BKW	001	Boschkop Weir Elevation - Option 1
WP 9528	DD	BKW	002	Boschkop Weir Elevation - Option 2
WP 9528	DD	BKW	003	Boschkop Weir Section Details
WP 9528	DD	BKW	004	Boschkop Silt trap Cross Section
WP 9528	LS	BKW	001	Boschkop Silt trap and Balancing Dam Long Section
Vlieëpoort Abstraction Works				
WP 9528	LD	VPW	001	Vlieëpoort Site Layout
WP 9528	DD	VPW	001	Vlieëpoort Weir Elevation - Option 1
WP 9528	DD	VPW	002	Vlieëpoort Weir Elevation - Option 2
WP 9528	DD	VPW	003	Vlieëpoort Weir Section Details
WP 9528	DD	VPW	004	Vlieëpoort Silt trap Cross Section
WP 9528	LS	VPW	001	Vlieëpoort Silt trap and Balancing Dam Long Section
Mokolo Abstraction Works				
WP 9528	LD	MD	001	Mokolo Dam General Layout and Sections
WP 9528	LD	MD	002	Mokolo Dam Labyrinth Spillway Options
WP 9528	LD	MD	003	Mokolo Weir Site Layout
Client Balancing Dams				

Project:	Series:	Component:	Drawing Number:	Title:
WP 9528	LD	TR	001	User Terminal reservoirs

A3 sized versions of the drawings are included in **Appendix B** of the Report, as well as schematics showing the typical details of the Terminal dams.

13. REFERENCES

(1) Mokolo and Crocodile River Water Augmentation Project (MCWAP): Pre-Feasibility Stage: Report 1 Water Requirements

(2) Mokolo and Crocodile River Water Augmentation Project (MCWAP): Pre-Feasibility Stage: Report 2 Water Resources

(3) Mokolo and Crocodile River Water Augmentation Project (MCWAP): Pre-Feasibility Stage: Report 3 Guidelines for preliminary Sizing, Costing and Economic Evaluation of Development Options

(4) Mokolo and Crocodile River Water Augmentation Project (MCWAP): Pre-Feasibility Stage: Report 5 Mokolo River Development Options

(5) Mokolo and Crocodile River Water Augmentation Project (MCWAP): Pre-Feasibility Stage: Report 6 Water Transfer Scheme Options

(6) Mokolo and Crocodile River Water Augmentation Project (MCWAP): Pre-Feasibility Stage: Report 7 Social and Environmental Screening

(7) Mokolo and Crocodile River Water Augmentation Project (MCWAP): Pre-Feasibility Stage: Report 10 Institutional Management of Operational Scheme

(8) Mokolo and Crocodile River Water Augmentation Project (MCWAP): Feasibility Study Technical Module Summary Report : Report 13

(9) SANCOLD, Safety Evaluation of Dams, Report 4, Guideline on Safety in Relations to Floods, December 1991

(10) SANCOLD, Safety Evaluation of Dams, Report 3, Interim Guideline on Freeboard for Dams, September1990

(11) VAPS

(12) Milford (1987)

(13) Kovacs (1987)

(14) Schoeman & Joubert (2007) for DWA

APPENDIX A

COST MODELS

APPENDIX B

DRAWINGS

REPORT DETAILS PAGE

Project name:	***Mokolo and Crocodile River (West) Water Augmentation Project (MCWAP)***
Report Title:	**Pre-Feasibility Study Report 4 - Dams, Abstraction Weirs and River Works: Pre-Feasibility Stage Report**
Author:	***BC Viljoen (Vela VKE)***
DWA report reference no.:	***P RSA A000/00/9109***
PSP project reference no.:	***WP 9528***
Status of report:	***Final***
First issue:	***December 2008***
Final issue:	***October 2009***

PSP

Approved for PSP by:

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