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Support of the Water Reconciliation Strategy for the Algoa Water Supply System

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Document prepared by:

Aurecon South Africa (Pty) Ltd

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Aurecon Centre
1 Century City Drive
Waterford Precinct
Century City
Cape Town
7441

PO Box 494
Cape Town
8000
South Africa

T +27 21 526 9400
F +27 21 526 9500
E capetown@aurecongroup.com
W aurecongroup.com

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Author signature		Approver signature	
Name	O Human	Name	Erik van der Berg
Title	Engineer (Civil)	Title	Technical Director

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Directorates: National Water Resource Planning and Options Analysis

Support of the Water Reconciliation Strategy for the Algoa Water Supply System

OPTIONS ANALYSIS

Final: March 2019

Prepared by: Aurecon South Africa (Pty) Ltd
P O Box 494
Cape Town, 8000
South Africa

Tel: 021 5265790
Fax: 086 5269500
E-mail: erik.vanderberg@aurecongroup.com

Prepared for: Director: National Water Resource Planning,
Director: Options Analysis
Department of Water and Sanitation
Private Bag X313
Pretoria 0001
South Africa

Mr Tendayi Makombe (PE: NWRP, Central)
Tel: 012 336 8503
E-mail: makombet@dws.gov.za

Mr Menard Mugumo (CE: Options Analysis)
Tel: 012 336 6838
E-mail: mugumom@dws.gov.za

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SUPPORT OF THE WATER RECONCILIATION STRATEGY FOR THE ALGOA WATER SUPPLY SYSTEM

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STUDY TEAM

Approved for Aurecon South Africa (Pty) Ltd:

....
E VAN DER BERG
Technical Director

DEPARTMENT OF WATER AND SANITATION
Directorates National Water Resource Planning and Options Analysis

Approved for Department of Water and Sanitation:

....	....29/10/19
T MAKOMBE	***P MLILO***
PE: NWRP	***DIRECTOR: NWRP***

....
AJ MOORE
CE: Options Analysis

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Bold type indicates this Report.

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Executive Summary

Introduction

The objective of the Feasibility Component of the *Support of the Water Reconciliation Strategy for the Algoa Water Supply System* study is to:

- limit risks of shortfall in supply to the Nelson Mandela Bay Municipality (NMBM) and the Lower Sundays River Government Water Scheme (LSRGWS),
- remove potential operating system constraints for the sustainable delivery of bulk Orange River water supply to the LSRGWS and NMBM, for water requirements up to 2040, and
- limit operational risks to acceptable levels.

The existing Scheepersvlakte Balancing Dam is a balancing facility for water supply to the Lower Sundays River Water User Association (LSRWUA) and the Nelson Mandela Bay Municipality (NMBM), and for emergency supply. The Balancing Dam has been identified by NMBM officials and the DWS as a growing, high, operational risk to the bulk water supply of the NMBM system, with part of the supply area even running dry from time to time. The dam had an initial storage capacity of 820 000 m^3, but this has been reduced through siltation and is further constrained by operational limitations and problems.

The focus of the investigation is on providing additional balancing storage in addition to the existing Scheepersvlakte Balancing Dam.

This report describes the Options Analysis activity of the Feasibility Study.

Preliminary evaluation of dam sites

Several initial options were identified for improving the assurance of supply that is provided by the Scheepersvlakte Balancing Dam to the Nooitgedagt Water Treatment Works (WTW). Additional future balancing capacity of 210 Mℓ /day for 21 days (4.5 million m^3) should be provided. Following evaluation of seven initial options, it was recommended that the following options be evaluated further:

a) A larger dam near the present Scheepersvlakte Balancing Dam site, to be integrated with the existing gravity pipeline to the Nooitgedagt WTW.
b) A large balancing dam on the right bank near the Nooitgedagt WTW.

Preliminary dam sites evaluated

The engineering and environmental aspects of sub-options associated with the selected two balancing dam sites near Scheepersvlakte Dam and Nooitgedagt WTW respectively, as shown in **Figure E1**, were then identified and assessed.

The four balancing dam options identified near Scheepersvlakte Dam are:

1. Raising of the existing Scheepersvlakte Dam, which was found not to be feasible.
2. Upper Scheepersvlakte Dam, which would be situated immediately upstream of the existing Scheepersvlakte Dam and would require pumping.
3. Lower Coerney Dam, situated upstream of the Coerney Siphon. This is the only option near Scheepersvlakte Dam that would not require pumping.
4. Upper Coerney Dam, which would require pumping.

Four possible sites for a balancing dam near the Nooitgedagt WTW were evaluated, namely:

1. Nooitgedagt North Option 1 site
2. Nooitgedagt North Option 2 site
3. Nooitgedagt North Option 3 site
4. Nooitgedagt South site.

The main advantages of the Nooitgedagt sites are the following:

- The balancing dam would be located very close to the Nooitgedagt WTW and therefore could be easily managed by the operating staff at the WTW.
- The supply would not be vulnerable to a failure of the Scheepersvlakte to Nooitgedagt pipeline.

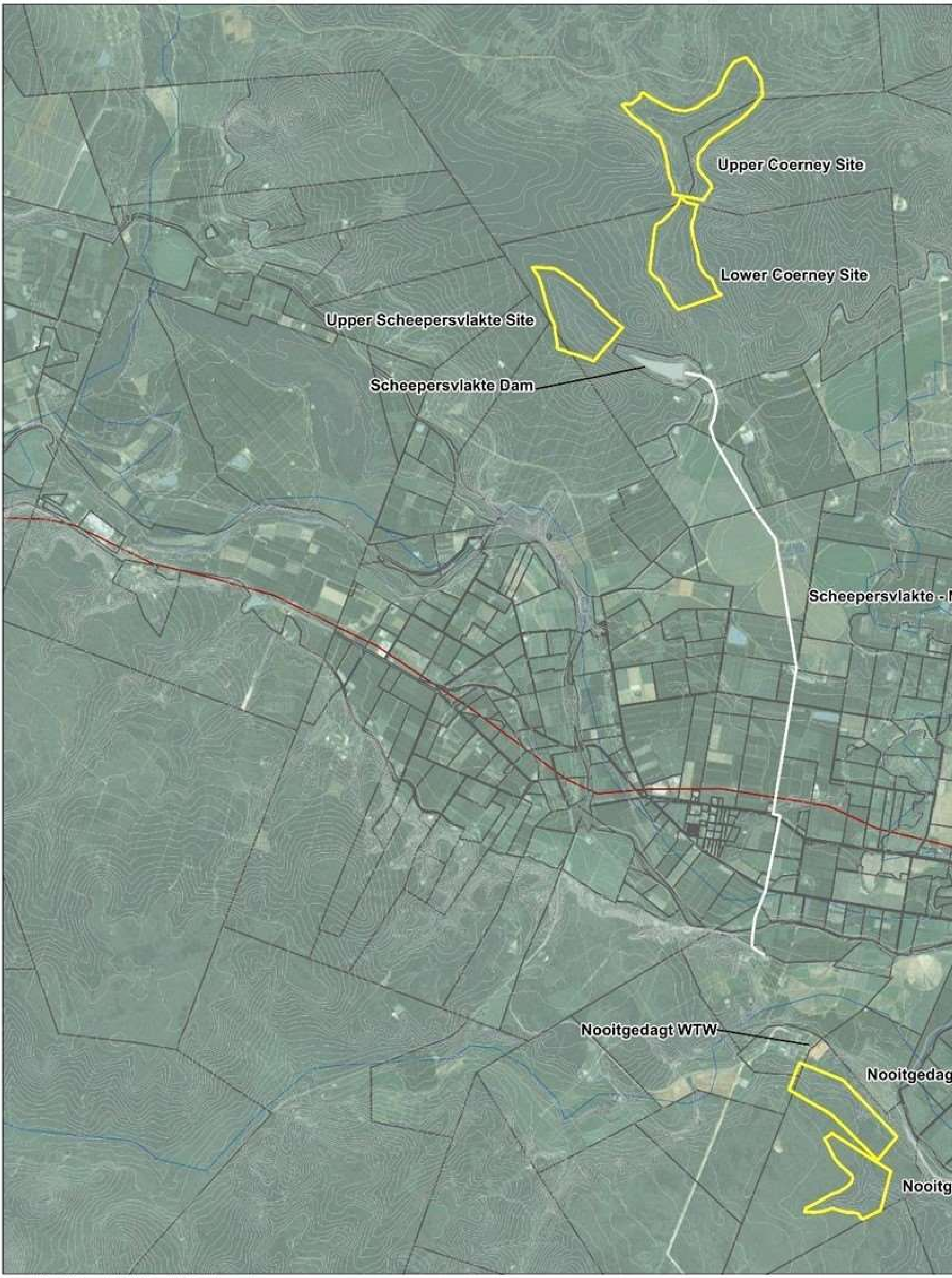

Figure E1: Options for Balancing Dams in the vicinty c

Operational considerations

The balancing dam would not be operated in the same way as normal water resource infrastructure as the water in the dam would only be abstracted in an emergency to supply the Nooitgedagt WTW. The dam would be filled over a certain filling period and would be topped up from time to time to make up evaporation and seepage losses, and possibly also operated to address water quality considerations. Because of this operation, the capital cost is more appropriate for comparing schemes rather than the unit reference value (URV).

Based on the capital cost comparison as well as other considerations, the Nooitgedagt Dam sites should not be evaluated further, because of their significantly higher costs and land owner objections. Never the less, the Nooitgedagt sites would provide a strategic advantage when compared with the Upper Scheepersvlakte and Coerney dam sites due to their proximity to the Nooitgedagt WTW.

On the other hand, the main risk of failure of the Upper Scheepersvlakte and Coerney dam options would be mitigated by providing an additional siphon through the Sundays River, as well as managing the process to enable quick replacement of damaged pipes should this be required.

Environmental constraints analysis

The purpose of the Environmental Constraints Analysis was to provide a desktop overview and analysis of the environmental sensitivity of the short-listed sites for a new balancing dam, highlighting potential issues and constraints and outlining the requisite environmental legal compliance requirements for each option. This provided high-level input regarding the environmental issues/constraints and legal requirements of the five short-listed sub-options.

From a terrestrial ecology perspective, the Upper Scheepersvlakte and Coerney sites are considered slightly more environmentally sensitive when compared to the Nooitgedagt sites, mostly due to an overlap with an Endangered Ecosystem. From an aquatic ecology perspective, the Coerney sites have a greater aquatic sensitivity due to the drainage line within which they are located. No fatal flaws were identified from a heritage and palaeontology as well as land use perspective.

From a purely environmental sensitivity perspective the Nooitgedagt sites are thus slightly preferred to the Upper Scheepersvlakte and Coerney sites. The aforementioned do, however, not qualify as “fatal flaws”, but merely something to take note of when evaluating the overall feasibility of the sites.

Comparison of options

A comparison of the balancing dam options is shown in **Table E1**.

Table E1: Comparison of options

	Potential dam sites				
EVALUATION FACTOR	**Upper Scheepers-vlakte**	**Lower Coerney**	**Upper Coerney**	**Nooitgedagt North - Option 1**	**Nooitgedagt South**
Capital cost (R million)	**R349**	**R237**	**R375**	**R457**	**R654**
Capital cost (cost of pumps reduced by 50%) (R million)	**R282**	**R231**	**R309**	**R403**	**R600**
Cost	2 - 2nd lowest	1 - Lowest	3 - 3rd lowest	4 - High	5 - Very High
Pumping required	X		X	X	X
Operational complexity	X	X			
Strategic location near WTW				X	X
Ecological considerations (Reserve)		X but likely easy to address	X but likely easy to address		
Consideration of floods		X	X		
Environmental & Social impacts	Limited differentiation	Limited differentiation	Limited differentiation	Limited differentiation	Limited differentiation

Preliminary recommendations

The following preliminary recommendations were made:

- The Nooitgedagt sites (North Option 1 and South) should be ruled out and not investigated further. Although these sites are strategically located near the Nooitgedagt WTW, the comparative cost of these options is nearly double that of the lowest cost option (Lower Coerney site).
- The Lower Coerney site is the preferred site, followed by the Upper Scheepersvlakte site and the Upper Coerney site. The main advantage of the Lower Coerney site, besides having the lowest comparative cost, is that water could be supplied by gravity from the canal to the dam. The risk of failure of these options could mostly be mitigated

by providing an additional siphon through the Sundays River, as well as managing the process for quick replacement of damaged pipes should this be required.

- It was recommended that the Lower Coerney and Upper Scheepersvlakte Dam sites be evaluated further. The topographical survey and geotechnical evaluation of these sites should proceed, to ensure that detailed information for the evaluation of these alternative options is available. Further evaluation of the Upper Coerney site is not recommended as it offers no additional advantage over the other two sites and the comparative cost is the highest of these three options.

Geotechnical survey

Geotechnical investigations were conducted at both recommended sites. These investigations included geophysical surveys, test pitting, sampling and laboratory testing, and rotary core drilling.

The general geology comprises thin grey sandstones, siltstones and mudrocks of the Sundays River Formation of the Uitenhage Group. The seismic hazard of the area is very low.

The geological profile at the respective sites is characterised by soil cover of variable origin and thickness, overlying weak rocks characterised by extensive and pervasive weathering.

The flat topography and foundation geology of weak rocks dictate that an earthfill embankment is the favoured dam type. The cut-off trench should be taken to the base of the soils, to the bedrock. Although the rock mass is generally tight, Lugeon testing recorded occasional water losses.

Limited investigation of embankment material sources shows wide scatter in material properties; some materials are suitable, but others are non-compliant.

Design-level investigations of the favoured site will require further geotechnical investigations, with a focus on including further confirmation of the geological profile / founding conditions for the cut-off trench, and also the intake – outlet conduit and end of the spillway. The availability of suitable embankment construction materials within the basin needs detailed confirmation. These investigations would include further rotary core drilling, test pitting and possibly trenching. A comprehensive laboratory testing programme must compliment these investigations.

Topographical survey

A topographical survey was completed by Survey Services: Southern Operations (National Water Resource Infrastructure) of the DWS for the Lower Coerney and Upper Scheepersvlakte dam sites, as well as immediate surrounding infrastructure.

Because more than 75% of the dam basin is covered in dense bush it was not possible to use ground-based survey methods to do a topographical survey by foot.

Contours from existing 1 m contour plans from 1977 and 1984, that was compiled from aerial photography for the design of the Lower Sundays River Government Water Scheme, were regenerated. Two test sections were surveyed in the field for the Upper Scheepersvlakte site and nine for the Lower Coerney site, to compare and verify the digitised data to the actual ground data, which resulted in a good match. A portion of the Upper Scheepersvlakte site was surveyed with GPS-RTK systems. Datasets were combined and final contours with 1 m intervals were generated.

Design flood analysis

The design flood peaks for various recurrence intervals were estimated for the Lower Coerney and Upper Scheepersvlakte Dam sites. Based on the height, storage capacity and expected hazard potential downstream of the dams, it is recommended that the Recommended Design Flood be equal to the 1:200 year flood and the Safety Evaluation Flood equal to the Probable Maximum Flood, as indicated in **Table E2**.

Table E2: Recommended design flood peaks

Recurrence Interval (y)	Design flood according to SANCOLD recommendations	Recommended Design Flood Peaks (m^3/s)	
		Lower Coerney	Upper Scheepersvlakte
1:200	Recommended Design Flood (RDF)	143	23
PMF	Safety Evaluation Flood (SEF)	835	141

Groundwater evaluation

The core drilling at the Lower Coerney Dam centreline indicated the occurrence of a gravel layer – paleo-channel at the Lower Coerney site, which created a need to understand the direction of flow of the groundwater and particulars about the expected rate of flow, to take its influence on the planned dam wall into account.

A brief assessment of the groundwater situation was undertaken, which confirmed that:

- The groundwater flow direction is downstream, with low flow rates, because of the low permeability of the saturated rocks, even with the steep hydraulic gradients.
- The gravel layer will become saturated but should not have a large impact on groundwater flow.
- The current groundwater movement is below the gravel layer and will not be affected by the proposed dam wall.

The core trench for the dam needs to be founded on the material below the gravel layer, which will intercept groundwater flow through the gravels. This will not impact on the current groundwater movement, which is below the gravel layer. The dam wall can be founded on the gravel layer as this will be considered during the stability analysis.

Refined dam designs and costs

The Lower Coerney and Upper Scheepersvlakte Dam designs and construction cost estimates were further refined, incorporating additional information from the topographical survey, geotechnical investigations and determination of the design flood peaks.

The topographical survey, which produced 1 m contour intervals, was in general very similar to the 5 m contour intervals and did not have a major impact on the design of either of the dams. The design flood peaks for both the sites reduced slightly, but which had only a minor impact on the spillway design. No flood hydrographs were produced in the study and flood attenuation was thus not considered. The geotechnical investigations showed that the sites were more challenging than originally envisaged, with deep founding levels for the spillway, outlet structure and core trench. Both sites assumed imported filter material and importation of the majority of the impervious clay core material.

The final estimated capital cost for the Lower Coerney Dam and associated works is R 252 million, excl. VAT and for the Upper Scheepersvlakte Dam and associated works is R 354 million, excl. VAT.

Recommendations

The Lower Coerney site is the preferred site and is recommended for feasibility design. The main advantages of the Lower Coerney site are:

- The lowest capital and operational cost.

- Water could be supplied by gravity from the canal to the dam and from the dam to the Nooitgedagt WTW, while pumping is required for all the other options.
- Moderate and mitigable environmental impacts.

The Lower Coerney Dam site falls on land being planned for new irrigation development by the Scheepersvlakte 98 Citrus Development Trust. The joint use of water from the dam by the Municipality and the private developer would need careful planning

The DWS should undertake an EWR determination study for non-perennial systems for the small ephemeral tributary of the Coerney River, in which the Lower Coerney Dam will be situated.

Further geotechnical investigations would be required for detail design purposes at the favoured site.

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Abbreviations

ADD	Average Day Demand
ARFs	Areal Reduction Factors
CBA	Critical biodiversity area
DEM	Digital elevation model
DN	Nominal diameter
DWA	(Previous) Department of Water Affairs
DWAF	(Previous) Department of Water and Forestry
DWS	Department of Water and Sanitation
EIA	Environmental Impact Assessment
EWR	Ecological water requirements
FSL	Full supply level
GN	Government notice
GWS	Government Water Scheme
ha	hectares
HDI	Historically Disadvantaged Individual
ID	Internal diameter
kWh	Kilowatt hours
Kv	kiloVolt
LFRGWS	Lower Fish River Government Water Scheme
LIDAR	Light detection and ranging
LSRWGS	Lower Sundays River Government Water Scheme
LSRWUA	Lower Sundays River Water User Association
Mℓ	Megalitre
mamsl	Metres above mean sea level
NERSA	National Energy Regulator of South Africa
NMBM	Nelson Mandela Bay Municipality
NOC	Non-overspill crest
OFS	Orange-Fish-Sundays
ORP	Orange River Project
PEM	Port Elizabeth Municipality
PMF	Probable maximum flood
RDF	Recommended design flood
RI	Recurrence interval
RL	Related level
PMF	Probable maximum flood
RPF	Resource-poor farmers

SAWB	South African Weather Bureau
SCS	(United States) Soil Conservation Services
SEF	Safety evaluation flood
SOTERSAF	Soil and Terrain Database for Southern Africa
SRTM	Shuttle Radar Topographic Mission
T_c	Time of concentration
TDS	Total dissolved solids
VAT	Value added tax
V:H	Vertical to horizontal
WRC	Water Research Commission
WTW	Water Treatment Works
WUA	Water User Association
WULA	Water use licence application

1 Introduction and Background

1.1 Study Objective

The objective of the Feasibility Component of the *Support of the Water Reconciliation Strategy for the Algoa Water Supply System* study is to:

- limit risks of shortfall in supply to the Nelson Mandela Bay Municipality (NMBM) and the Lower Sundays River Government Water Scheme (LSRGWS),
- remove potential operating system constraints for the sustainable delivery of bulk Orange River water supply to the LSRGWS and NMBM, for water requirements up to 2040, and
- limit operational risks to acceptable levels.

The focus of the investigation is on providing additional balancing storage in addition to the existing Scheepersvlakte Balancing Dam.

1.2 Purpose of this report

The purpose of this report is to describe the Options Analysis activity of the Feasibility Study, inclusive of background, dam sizing, options identification and evaluation, supporting investigations, and the recommended dam site for feasibility investigation.

1.3 Background

Bulk water supply provision to the Port Elizabeth region from the Orange River system was included in the planning and development phases of the Orange River Project (ORP) as far back as 1965. With the construction and completion of the De Mistkraal Weir on the Little Fish River in 1987, the infrastructure to convey water from the Gariep Dam to the Port Elizabeth region over a distance of some 400 km, was finally put in place. The Orange-Fish-Sundays Transfer Scheme is shown in **Figure 1.1**.

Based on the yield of existing resources for the region and the growth in water requirements over the period 1970 to 1985, it was anticipated that the then Port Elizabeth Municipality (PEM) would only require an ORP supply by about 2000 to 2002.

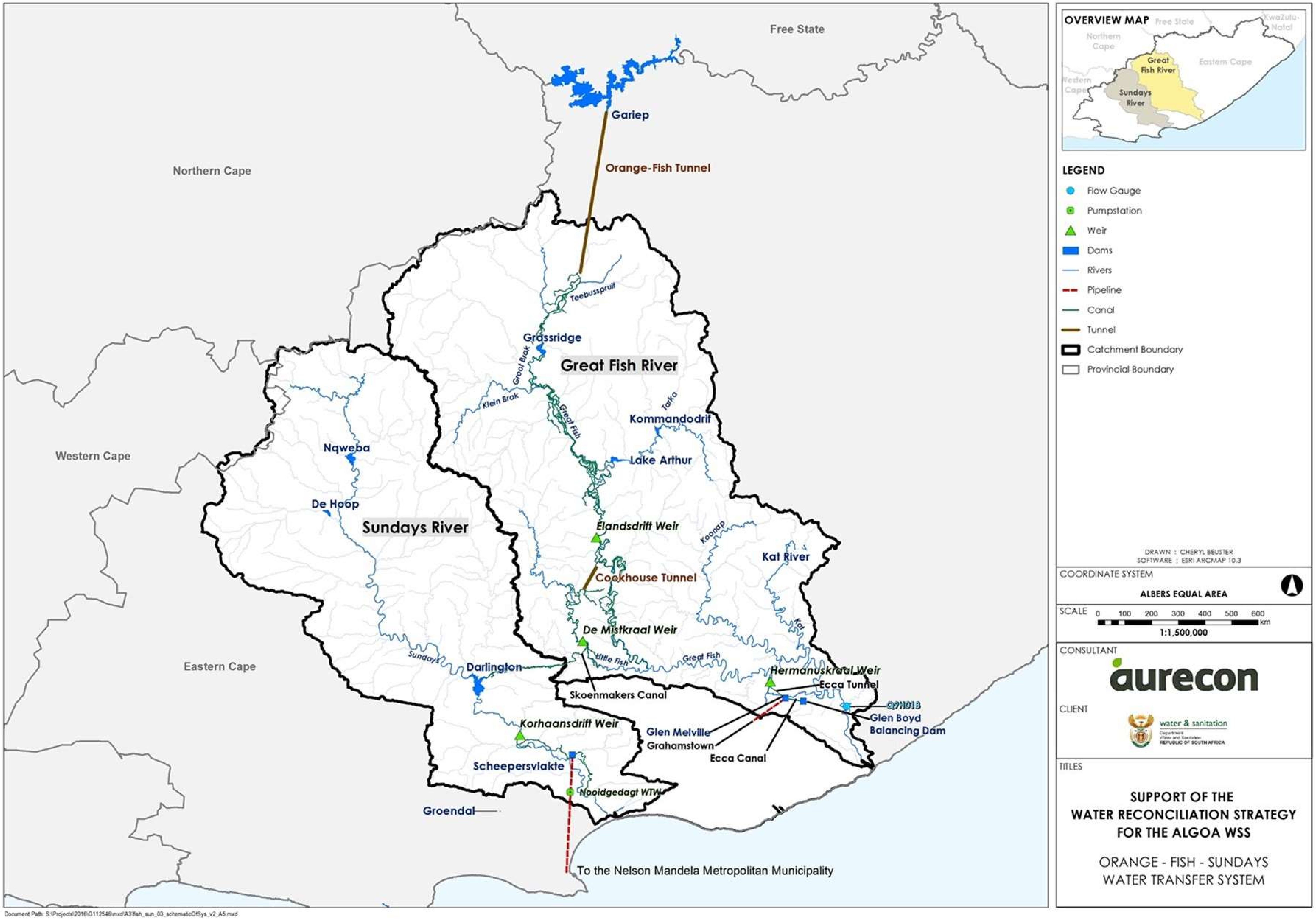

Figure 1.1: The Orange-Fish-Sundays Transfer Scheme

The extreme drought of 1987 to 1992, however, necessitated the then Department of Water Affairs (DWA) to assist the then PEM with the design and implementation of an emergency scheme, within record time frames.

The newly completed Scheepersvlakte Balancing Dam, which was designed and sized only to operate as a balancing facility for the Sundays River Irrigation Board (now the Lower Sundays River Water User Association (LSRWUA)), was selected as the only suitable point of abstraction available for such an emergency supply. The gravity supply pipeline from the Scheepersvlakte Balancing Dam was sized for long term flow requirements of both the NMBM and the right bank (Sundays River) irrigators.

In 1993 the Nooitgedagt Water Treatment Works (Nooitgedagt WTW) was completed and the ORP allocation then became a permanent water source for the region.

Following recommendations made by the Algoa Water Reconciliation Strategy Study Team in 2009, DWS increased the NMBM water licence for abstraction from the ORP in 2010 to 58.3 million m^3/a (160 Mℓ/day).

Following the expected completion of the Nooitgedagt WTW Phase 3 in 2021, the WTW will have a maximum capacity of 210 Mℓ/day. This has been designed to cater for peak/back-up supplies from the Nooitgedagt WTW at times when the older infrastructure, from sources to the west of Port Elizabeth, will be requiring maintenance or emergency repairs.

1.4 Problem Statement

The Scheepersvlakte Balancing Dam has been identified by NMBM officials and the DWS as a growing operational risk to the bulk water supply of the NMBM system. The Scheepersvlakte Dam Remedial Works project (Naidu Consulting, 2016) confirmed this by concluding that "*Scheepersvlakte Balancing Dam poses high operational risks to NMBM with part of the supply area running dry from time to time".*

The problem is therefore the reliability of the Scheepersvlakte Balancing Dam to supply water to the Nooitgedagt WTW. The key factors impacting on this reliability are described below.

1.4.1 Limited balancing capacity

The Scheepersvlakte Balancing Dam was designed and sized for the purpose of balancing irrigation supplies into the Lower Coerney canal only. The dam had an initial storage capacity of 820 000 m^3. The available balancing storage has been reduced by a number of factors:

- Silting has reduced the storage capacity to 769 320 m^3 (2014). Effective storage has thus been reduced to 93.8% of design capacity.
- To control algae growth, the canal system is drained over weekends by closing down the releases from Darlington Dam. This operational requirement then leaves only the water stored in the dam to supply the WTW.
- A further operational requirement, to avoid spillages and thereby curtail unrecoverable losses, is that the LSRWUA operates the balancing dam at some 550 000 m^3 (71% capacity and 14 m on gauge plate) to leave some balancing buffer capacity.
- The 1.424 mm ID steel gravity pipeline from the Scheepersvlakte Balancing Dam to the Nooitgedagt WTW, is supplied from the dam by 2 x 120 m long DN1000 stainless steel pipes. A critical point in the pipeline profile, that determines pipeline capacity under gravity flow, is the high point at chainage 400 m (pipe invert 87.40 m), where there is an air valve and the cross connection to the Scheepersvlakte siphon. With the total flow divided equally between the two outlet pipes in operation, the dam water level can be drawn down to 92.1 m. However, with only one outlet pipe in operation, serving both the Nooitgedagt WTW pipeline and the Lower Coerney Canal, the lowest draw-down level is 98.7 m, due to increased velocities and energy losses.

  This situation leaves a “dead storage volume” of some 200 000 m^3 and therefore an “actual balancing capacity” of some 350 000 m^3. This equates to 2.5 days balancing for Phase 2 treatment works capacity (140 Mℓ/day) and 1.67 days balancing storage for Phase 3 capacity (210 Mℓ/day) at the Nooitgedagt WTW.

Irrigation water releases from the Scheepersvlakte Balancing Dam into the Coerney Canal receives priority on a Monday morning, whether the dam level at that point in time permits sufficient flow to the Nooitgedagt WTW or not.

Limitations on draw-down levels (limited balancing capacity) will limit the peak capacity available to NMBM when the supply source to the west of Port Elizabeth has a breakdown. This limitation of balancing capacity is a high risk to the continuity of bulk water supply to the Nooitgedagt WTW.

1.4.2 Operational Limitations

The LSRWUA controls water releases from Darlington Dam, which is situated some 50 km upstream of the Korhaansdrift Weir. The LSRWUA must be notified in advance by all irrigators as well as the NMBM on what their water requirements for the following week will be. In the case of the NMBM, operations could change within hours, as a major pipe burst on bulk supplies from the western sources could happen over weekends or as a worst-case scenario, on a Monday when the Scheepersvlakte Balancing Dam is down to a minimum level.

This will require the Nooitgedagt WTW to increase output over a period of days, which then upsets the operation at the LSRWUA and impacts on the balance of water available for irrigators.

The Scheepersvlakte Balancing Dam, being an irrigation balancing dam, has a bottom outlet (intake to gravity pipeline). This bottom intake and bottom orientation of the offtakes to the gravity pipeline (emergency scheme modifications) result in sediment and debris from the dam being drawn into the Nooitgedagt pipeline. This is worsened when the dam level is low and at times when draining of the dam is required. Fish and trash are then drawn into the gravity supply to the WTW.

The outlet works, that conveys dam water to the Coerney Lower Canal and the Nooitgedagt pipeline, is prone to mechanical failures, which generally require a 3-day complete shut-down to remove or re-install a faulty valve. This operational problem transfers major risks onto the NMBM water supply system.

The 1200 mm diameter cross connection, between the Scheepersvlakte siphon and the 1400 mm diameter Nooitgedagt gravity pipeline, flows back through the outlet works into the dam when opened. This prevents maintenance work at the outlet works and thus requires the dam to be drained each time maintenance work must be done. Once the recommended improvements of the Scheepersvlakte Rehabilitation Report have been completed to this cross connection, the operational risk during emptying of the Scheepersvlakte Balancing Dam will be reduced.

1.4.3 Maintenance Risks

During winter dry periods, water supply is operated on the basis of three days on and two days off. This requires a major effort by the LSRWUA to ensure that Scheepersvlakte Balancing Dam, with such small balancing capacity, is operated with sufficient water in storage to meet NMBM's water requirements.

The present manner of accommodating the dry period maintenance programme appears to work well for the present. The infrastructure is, however, ageing, and it is doubtful whether the same methodology will remain applicable to maintain the canal system for another 25 to 30 years. The limited balancing capacity will then become a more serious risk to the NMBM.

The status of supply from the Scheepersvlakte Balancing Dam therefore poses a high risk to reliability of water supply to the NMBM.

1.5 Recent failure of the LSRWUA main canal

On 17 May 2017, the LSRWUA's main irrigation canal failed due to a land slide (see **Figure** 1.2). The failure was located downstream of Kirkwood's offtake, and the supply to Kirkwood was therefore unaffected. The supply to the Nooitgedagt WTW was affected by the failure; however, the repair of the failure was carried out timeously by DWS Construction South, with the assistance of the Citrus Growers Association. This canal failure highlights the need to have sufficient balancing storage, and as close as possible to the Nooitgedagt WTW site, to limit the risk of failure of supply to the NMBM.

Figure 1.2: Failure of the LSRWUA main irrigation canal on 17 May 2017

1.6 Content of this Report

Chapter 1: Introduction and Background (this Chapter): provides an introduction and background to the report and describes the current operational challenges and need for additional balancing storage.

Chapter 2: Infrastructure Capacity: describes the salient features and transfer capacities of components of the existing OFS Transfer Scheme.

Chapter 3: Design Water Requirements and Capacity: describes the water requirements of NMBM and the LSRWUA, and the determination of the design capacity of the balancing dam.

Chapter 4: Preliminary Identification and Screening of Options: describes the options analysis approach and the preliminary identification and evaluation of options.

Chapter 5: Identification of Sub-Options to Evaluate: describes the identification and evaluation of sub-options of the selected options, as well as remedial works and improvements at Scheepersvlakte Dam.

Chapter 6: Considerations for Balancing Storage and its Operation: describes factors to be considered in the evaluation of options.

Chapter 7: Options for Balancing Storage in the Vicinity of Scheepersvlakte Dam: describes the evaluation of the refined options for balancing storage in the vicinity of Scheepersvlakte Dam.

Chapter 8: Alternative Nooitgedagt Dam Sites: describes the evaluation of the refined options for balancing storage near the Nooitgedagt WTW.

Chapter 9: Comparison of Options and Recommendations: provides a comparison of the evaluated Scheepersvlakte and selected Nooitgedagt WTW dam sites. It also provides recommendations on the preferred site and the topographical survey and geotechnical evaluation to be undertaken.

Chapter 10: Environmental Constraints Analysis: provides a desktop overview and analysis of the environmental sensitivity of the five short-listed balancing dam sites.

Chapter 11: Geotechnical Survey: describes the geotechnical survey and materials investigations undertaken at the Lower Coerney and Upper Scheepersvlakte dam sites.

Chapter 12: Topographical Survey: describes the topographical survey undertaken at the Lower Coerney and Upper Scheepersvlakte Dam sites.

Chapter 13: Design Flood Analysis: describes the design flood peaks determined for various recurrence intervals for the Lower Coerney and Upper Scheepersvlakte Dam sites.

Chapter 14: Groundwater Evaluation: describes an evaluation of the potential groundwater impacts on the design of the Lower Coerney Dam.

Chapter 15: Refined Dam Designs and Costs: provides updated characteristics and costing of the Lower Coerney and Upper Scheepersvlakte Dam, considering updated information from the geotechnical and topographical surveys undertaken at these dam sites.

Chapter 16: Recommendations: provides recommendations regarding the preferred dam site and considerations for the feasibility design.

2 Infrastructure Capacity

2.1 Orange-Fish-Sundays Transfer Scheme

In the Orange-Fish-Sundays (OFS) Transfer Scheme, water is transferred from the Orange River to the Great Fish River, and then further to the lower Sundays River, to supplement local water supply for irrigation, to meet some urban use requirements of small towns, and to transfer water to the NMBM via this system. The existing transfer capacities of the OFS Transfer Scheme are as shown in **Table 2.1**.

Table 2.1: Capacity of major infrastructure in the Orange-Fish-Sundays Transfer Scheme

Transfer Element	Capacity (m^3/s)		Observed flow peak (m^3/s)	Available capacity (m^3/s)	Observed daily record used
Orange-Fish Tunnel	51		51	0	Q1H014
Grassridge Dam Outlet	60		54.9	5.1	Q1H022
Cookhouse Canal-Tunnel	42.6		42.6*	0	Q5H006
De Mistkraal Weir Outlet	23.8		23.8	0	Q8H013
Skoenmakers Canal	26.5		23.8	2.7	Q8H013

* The Cookhouse Canal transfers water from the Elandsdrift Weir to the Cookhouse Tunnel (see **Figure 1.1**). The capacity of the Cookhouse Tunnel is the constraint for this transfer, given as 42.6 m^3/s. The observed flow peak is higher than this capacity, at 49.3 m^3/s, as this is located at the Elandsdrift Weir outlet to the Cookhouse Canal. There is irrigation and canal losses between these two locations.

Notes:

- The observed flow peak is the maximum instantaneous flow measured to date.
- The original capacity of the Orange-Fish Tunnel was 54 m^3/s, however, the ISP Report (previous Department of Water and Forestry (DWAF), 2003) reports that this has reduced by about 10%, and the observed flow peak was therefore used as the capacity.

	• Although flows of up to 23.8 m^3/s can be released from De Mistkraal Weir, when the releases approach about 22 m^3/s, the Allemanskraal siphon (located on the Schoenmakers Canal approximately 16 km downstream of De Mistkraal Weir) begins to drown.

2.2 Darlington Dam

The maximum operating capacity of the Darlington Dam is currently maintained at 43% (78 million m^3) of its full storage capacity (181 million m^3) due to dam safety constraints, and the release gates are leaking badly and require refurbishment. Because of the significant storage capacity in the Darlington Dam, the capacity of elements of the OFS Transfer Scheme upstream of the Darlington Dam is not regarded as a constraint to delivering adequate flows to the LSRGWS and NMBM, now or in the foreseeable future.

2.3 Lower Sundays River Government Water Scheme

The LSRGWS primarily provides water for irrigation in the lower Sundays River valley, and for municipal supply to the NMBM and a few smaller towns in the Sundays River Local Municipality. Water from the Darlington Dam is released along the lower Sundays River, flowing down to the Korhaansdrift Weir, where water is diverted into the Lower Sundays main irrigation canal. Most of the water is used for irrigation, but some water flows to the Scheepersvlakte Balancing Dam from where water is transported to the Nooitgedagt WTW on the right bank of the Sundays River by gravity pipeline. The LSRWUA has a contract to supply urban water to the towns in the Sundays River Valley Local Municipality.

2.4 Korhaansdrift Weir

The Korhaansdrift Weir is located on the main stem of the Sundays River near Kirkwood. The weir provides the diversion structure for the lower Sundays River Irrigation Scheme, to abstract water that was released from the Darlington Dam. Every week, the LSRWUA receives the daily requirements in advance from the individual irrigators and plans the operation of the sluice gates at the Korhaandrift Weir accordingly. The NMBM is similarly expected to place orders for water to be released to the Nooitgedagt WTW, to enable the operator to balance the flows in the system.

The Korhaansdrift Weir was initially constructed well before 1900 and it was, since its initial construction, strengthened and improved on several occasions. In order to improve balancing capacity and operations for the then Sundays River Irrigation Board, the dam overflow height was increased by some 900 mm some years after 1970 (discussions with Manager LSRWUA).

Little is known about the actual construction history and no information on the river bed founding conditions could be sourced. This weir must therefore, due to its age and limited knowledge available, be considered as a real risk to the water supply (both to the irrigators and the NMBM) under flood conditions, and supports the reasoning for increased balancing capacity required for the NMBM water supply system.

2.5 Lower Sundays River Canal System

Water is diverted at the Korhaansdrift Weir and transported via the main canal (known as the Kirkwood primary canal) to the Scheepersvlakte Balancing Dam. The design capacity of this canal at the Korhaansdrift Weir offtake is 22.7 m^3/s. There are three secondary canals which offtake from the Kirkwood canal to supply the various areas of the scheme, namely Wesbank, Mistkraal and Tregeron/Selborne canals.

2.6 Scheepersvlakte Balancing Dam

The Scheepersvlakte Balancing Dam (**Figure 2.1**) was sized for irrigation water balancing only, with an original volume of 0.82 million m^3. The dam has since partly silted up, and by 2014, the dam had a capacity of 0.769 million m^3. The balancing dam has the effect of reducing the time it takes to deliver water to downstream water users while balancing any irrigation spills from upstream water users, and collecting and storing the water for supply to NMBM. The dam still provides significant operational flexibility to the LSRWUA, depending on the distribution of users requesting water for the week downstream of the balancing dam, by reducing the delivery time for the downstream irrigators.

The capacity of the balancing dam is not sufficient to meet the dual purpose of supplying the Nooitgedagt WTW as well as reducing the time of delivery to downstream irrigators. The lack of sufficient balancing capacity also negatively affects the maintenance of the canals. The Scheepersvlakte Dam Remedial Works Project (Naidu Consulting, 2016) concluded that the current operation of the Scheepersvlakte Dam leads to high operational risks for water supply to the NMBM. The report made recommendations to alleviate some risks, but this cannot solve the main problem because of the limited storage.

Figure 2.1: Scheepersvlakte balancing dam and side channel spillway

2.7 Nooitgedagt Water Treatment Works

Water is conveyed from the Scheepersvlakte Balancing Dam to the Nooitgedagt WTW by a 9 km long DN 1400 mm diameter steel gravity main, designed to convey 280 Mℓ/day. Phase 3 of the Nooitgedagt WTW upgrade is currently under construction. The original capacity was increased to a 125 Mℓ/day average and 140 Mℓ/day peak (Phase 2) supply, and the final capacity of 160 Mℓ/day average and 210 Mℓ/day peak is expected to be available by 2022 (Phase 3). From the Nooitgedagt WTW, water is pumped to the NMBM either via the High-Level Scheme or the Low-Level Scheme.

Figure 2.2 presents the layout plan of the high and low-level schemes.

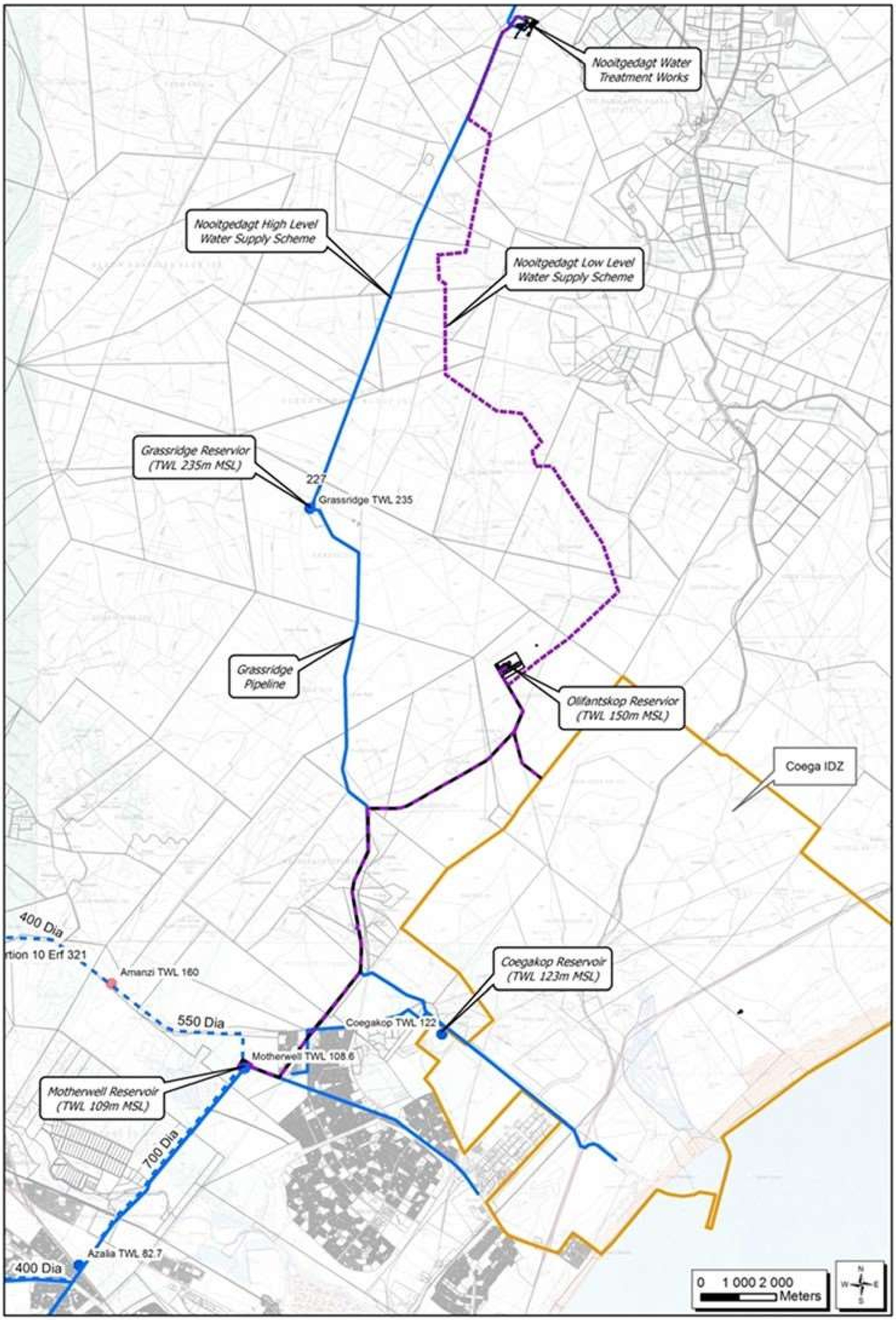

Figure 2.2: **Layout Plan of the Nooitgedagt High-level and Low-level schemes**

3 Design Water Requirements and Capacity

3.1 Water Requirements

3.1.1 Irrigation in the LSRWUA area of jurisdiction

The LSRWUA manages a scheduled irrigated area of 18 845 ha, with a total current scheduled quota of approximately 170 million m^3/a. As an initiative to promote socio-economic development for historically disadvantaged individuals (HDI), 3 000 ha were made available to HDI/emerging farmers in the Sundays River Valley, which will all be taken up in the foreseeable future. The future expected total irrigation allocation for the LSRWUA, including reserved water to be allocated at some future date for the proposed expansion of the irrigation area to serve more resource-poor farmers (RPFs), is 190 million m^3/a.

3.1.2 Nelson Mandela Bay Municipality

The current approved allocation of transferred Orange River water for the NMBM from the LSRGWS is 58.4 million m^3/a (160 Mℓ/day), with a design peak of 210 Mℓ/day.

3.2 Design Capacity

3.2.1 New balancing dam

A balancing storage of 21 days average daily demand (ADD) is recommended to limit the risk of shortfall in supply to the NMBM. This is based on the risk of failure of the old canal systems, as highlighted by the recent failure of a 100 m long section of the main canal on 17 May 2017, as well as the age of the Korhaansdrift Weir structure. For the design water requirement for NMBM of 76.6 million m^3/a (210 Mℓ/day), this equates to a balancing storage of 4. 41 million m^3.

Additional balancing storage is not required by the LSRWUA to supply the irrigators, as the balancing storage currently provided by the Scheepersvlakte Balancing Dam is only necessary for supply to the NMBM (as confirmed by the LSRWUA at the Study Management Meeting held on 30 November 2016).

Any changes to the current system operation may affect the design water requirements. The future system operation is dependent on the type and location of the additional balancing storage option chosen.

3.2.2 Gravity Pipeline

Ignoring the interim shortcomings of the Scheepersvlakte Balancing Dam offtake (limited balancing capacities and limited drawdown levels), the gravity pipeline from the Scheepersvlakte Balancing Dam to the Nooitgedagt WTW was sized for a 280 Mℓ/day transfer capacity, with additional allowance for right bank irrigation water.

3.2.3 Nooitgedagt WTW

The Nooitgedagt WTW was designed in 4 x 70 Mℓ/day modular treatment units, of which the first module was completed in 1993. Phase 3, currently under construction, will complete the third 70 Mℓ/day module.

3.2.4 Canal from Korhaansdrift Weir

The LSRWUA system operates on a five-day irrigation week, with two days of no irrigation (weekends). The allocation therefore equates to 3.46 mm per operational day. The canals were designed for 6.5 mm per day (DWAF, 2007), which under present operating conditions, equates to a summer peak week factor of 1.9.

Table 3.1 gives the capacities of the various sections of the main LSRWUA irrigation canal.

Table 3.1: LSRWUA irrigation canal capacities

Canal Section	Description	Max Capacity (m^3/s)
1	Korhaansdrift to Uierivier	22.7
2	Uierivier to Hesse’s Corner	18.0
3	Hesse’s Corner to Heatlieskrantz	14.5
4	Heatlieskrantz to Scheepersvlakte Dam	13.0

The peak flow required for future irrigation, with additional allocations to RFFs allocations included, would be 11.45 m^3/s at a summer 1.9 peak factor.

Once the NMBM abstracts their full allocation of 58.3 million m^3/a (160 Mℓ/day) from the OFS system, this would equate to a peak diversion of 210 Mℓ/day over 5 days, assuming a peak factor of just over 30%, amounting to a flow rate of 2.43 m^3/s via the canal system.

The future estimated peak flow required to supply water to the small towns receiving water from the canal, amounts to 0.25 m^3/s, at a peak factor of 1.3.

Allowing for 15% canal losses, the required future peak flow, without any additional Orange River water allocations to the NMBM, would amount to 16.25 m^3/s, considering the peak supply for irrigators, small towns and the NMBM. The spare "unused" capacity in Section 1 of the canal system would then amount to 6.46 m^3/s (157 million m^3/a). It is evident that such a large additional allocation cannot be made to the NMBM, indicating that the canal system (at Section 1) will not become a constraining factor in the conveyance of water diverted at the Korhaansdrift Weir to the Scheepersvlakte Dam, should this remain the preferred conveyance route.

The LSRWUA has also indicated that the canal system from the Korhaansdrift Weir to the Scheepersvlakte Balancing Dam will have sufficient capacity to supply the existing and additional future water requirements recommended.

4 Preliminary Identification and Screening of Options

4.1 Options Analysis Approach

The options identification and evaluation approach followed. is as follows:

- Study of relevant reports and identification of operational issues for both the NMBM and the LSRWUA.
- Conducting a site visit /field trip to all the relevant infrastructure components under discussion and options under consideration for an informed view of the scale of the problems.
- Determine the design water requirements and design capacities.
- Storage and supply options were conceptualised and briefly described to inform on their features.
- A high-level assessment of seven preliminary options (weighing the options in terms of advantages, disadvantages and red flags) was performed and two options were recommended for further evaluation.
- Holding a meeting with potentially affected land owners and conducting a site visit /field trip to the Lower Coerney and Upper Scheepersvlakte Dam sites.
- An environmental constraints analysis was conducted for all options and sub-options.
- Conceptualisation and evaluation of sub-options of the recommended options, with two sub-options recommended for further evaluation.
- Topographical and Geotechnical surveys were conducted for the Lower Coerney and Upper Scheepersvlakte Dam sites.
- The reconnaissance-level designs and costing of the Lower Coerney and Upper Scheepersvlakte Dam sites were refined, based on updated information from the topographical and geotechnical surveys.
- A dam site was recommended for feasibility-level design.

These options have also been described in more detail in the *Identification of Options for Balancing Storage sub-Report* of this study.

4.2 Preliminary identification and evaluation of options

Several options for improving the assurance of supply above that provided by the Scheepersvlakte Balancing Dam to the Nooitgedagt Water Treatment Works (WTW) were identified. The key factors which determine the reliability of the supply to Nooitgedagt are as follows:

- There is limited balancing capacity in Scheepersvlakte Dam, which is operated at a capacity of 550 000 m^3 to avoid spillages, although the dam has a total capacity of 820 000 m^3.
- There is a risk of failure of the aging upstream canal, siphon and weir infrastructure, such as the May 2017 failure of the main canal.

Additional future balancing capacity should be provided to supply 210 Mℓ /day for 21 days (4.1 million m^3). The following options were identified for providing improved assurance of supply to the WTW by various means, including balancing storage:

1. Balancing storage on the right bank (near the Nooitgedagt Water Treatment Works (Nooitgedagt WTW)) in combination with a raised Scheepersvlakte Balancing Dam wall.
2. Diverting water from the existing Korhaansdrift Weir via a right bank pipeline to Nooitgedagt WTW for additional delivery of the NMBM's water allocation.
3. Increased balancing capacity at the Korhaansdrift Weir and diverting water via a right bank pipeline to Nooitgedagt WTW for full delivery of the NMBM's water allocation.
4. Releasing water from the existing Korhaansdrift Weir and diverting it closer to the Nooitgedagt WTW via a new pump station for full delivery of the NMBM's water allocation.
5. Increased balancing capacity at the Korhaansdrift Weir, with water releases to a new pump station downstream in the Sundays River, close to the Nooitgedagt WTW.
6. Constructing a larger dam near the present Scheepersvlakte Balancing Dam site and integrate this dam with the existing gravity pipeline to the Nooitgedagt WTW.
7. Constructing a large balancing dam on the right bank near the Nooitgedagt WTW.

A two-day field trip was undertaken by DWS officials and study team members to observe first-hand what the options entail and the scale of the different options. The trip took place between 28 November 2016 and 29 November 2016.

The various options are shown graphically in **Figure 4.1** and briefly described in the following paragraphs.

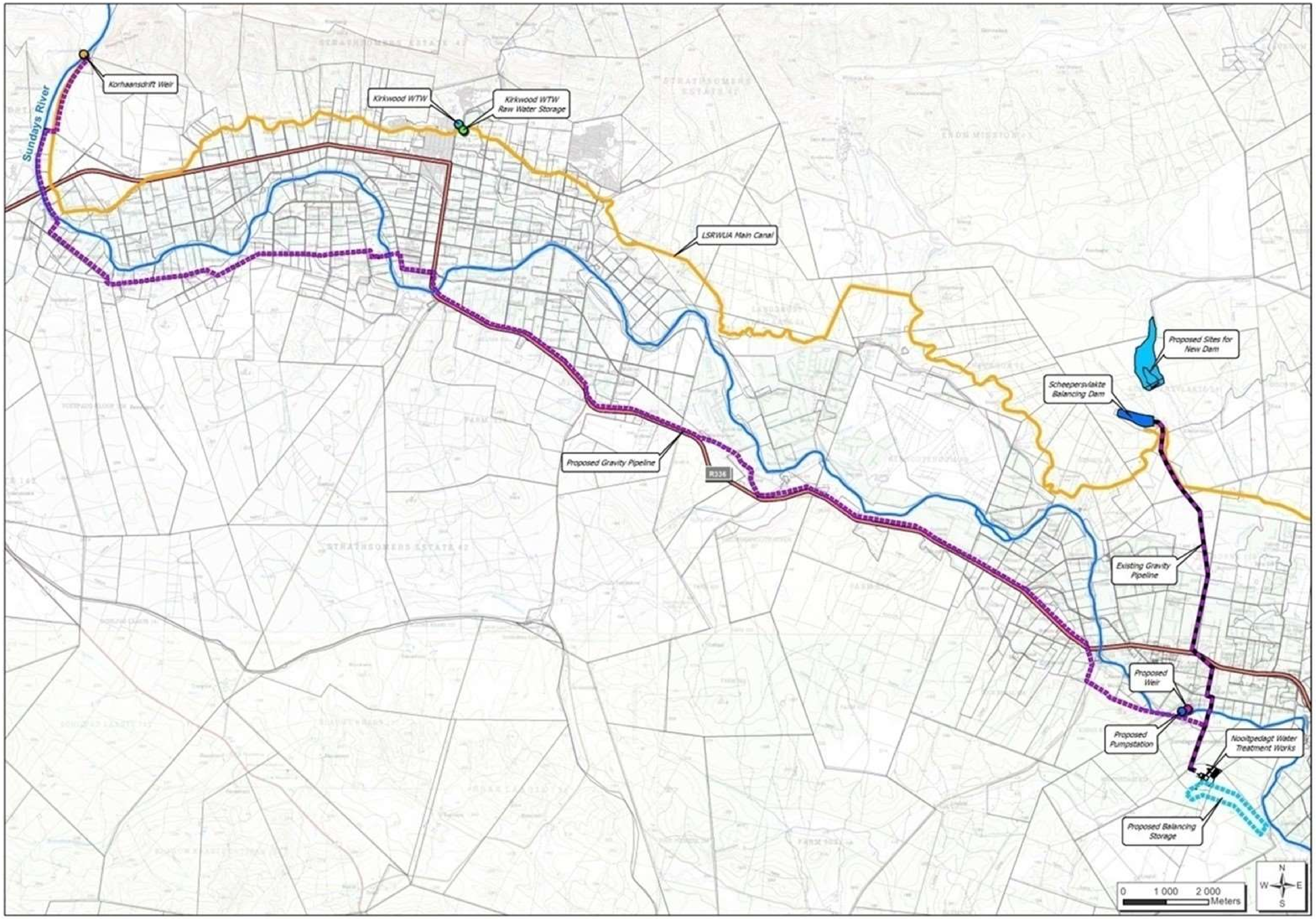

Figure 4.1: Layout of identified potential options to improve assurance of supply to NMBM

4.2.1 Option 1: Balancing storage on the right bank of the Sundays River near Nooitgedagt WTW in combination with a raised Scheepersvlakte Balancing Dam wall

This option consists of off-channel balancing storage consisting of a small dam in the valley to the north-west of the Nooitgedagt WTW in combination with an on-site storage facility, which could fit inside the present Nooitgedagt WTW site boundaries. The total storage available is limited due to the lack of available land on the Nooitgedagt WTW site (about 150 Mℓ storage in a cut-to-fill dam) and a possible 250 to 300 Mℓ in the valley surrounded by developed irrigation farm land.

Raising of the Scheepersvlakte Balancing Dam by 1.0 to 1.5 m could add some 160 Mℓ storage to achieve some 850 Mℓ storage. The maximum combined effective storage is estimated at 1000 to 1100 Mℓ, which may offer some 6 to 7 x ADD storage. This option thus cannot meet the required balancing storage.

4.2.2 Option 2: Diverting water from the existing Korhaansdrift Weir via a right-bank pipeline to Nooitgedagt WTW

The balancing capacity of the Korhaansdrift Weir is roughly estimated at 100 to 120 Mℓ, of which 80% would be utilised for this option. The proposed pipe route will initially start on the left bank (due to steep rocky slopes on the right bank) and then cross over to the right bank at 1.5 km downstream. The new pipeline will be 36 km long and will tie into the existing 1.5 m diameter pipeline from the Scheepersvlakte Balancing Dam to the Nooitgedagt WTW.

4.2.3 Option 3: Increased balancing capacity at Korhaansdrift Weir and diverting the water via a right-bank pipeline to Nooitgedagt WTW

The operation of this option is similar to Option 2, but additional balancing capacity will be created at Korhaansdrift Weir to accommodate the variability in the NMBM's water requirements and to minimise possible spillages under the LSRWUA long distance releases from Darlington Dam. This option requires that the Korhaansdrift Weir be raised by 4.5 m to create additional balancing capacity of 1 050 Mℓ. Given the age and history of the existing wall, raising the wall will require a new structure with the existing wall at best being used as a "shutter" to part of the new wall structure. The gravity pipeline will be 36 km long with a 1.5 m diameter.

4.2.4 Option 4: Releasing water from the existing Korhaansdrift Weir into the river and diverting closer to the Nooitgedagt WTW via a new pump station

This option is based on operating the existing Korhaansdrift Weir at present capacity, but to install a new outlet valve(s) in the present structure to allow for immediate releases on a short-term basis. At a distance some 44 km downstream of Korhaansdrift Weir, a large “hippo pool” was identified in the Sundays River as a good point of abstraction for a proposed right bank raw water pump station. From the proposed pump station, a 1.4 m diameter pipeline will be tied into the existing 1.4 m diameter gravity pipeline from the Scheepersvlakte Balancing Dam. The pump station will require an in-stream structure to maintain the present operating levels in the pool. Any permanent structure constructed above present “dry season” levels, will pose a flooding risk to adjacent irrigation land. The proposed structure would therefore be mass concrete or gabions, but not extend above the present water level. Total dissolved solids (TDS) levels at the proposed pump station could vary between 1 190 and 1 600 mg/ℓ.

4.2.5 Option 5: Increased balancing capacity at the Korhaansdrift Weir with releases to a new Pump Station downstream in the Sundays River

The operation of this option is similar to Option 4, but additional balancing capacity will be created at Korhaansdrift Weir to accommodate the variability in the water demands of NMBM and to minimise possible spillages under the LSRWUA long-distance releases from Darlington Dam. This option requires (as for Option 3) that the Korhaansdrift Weir be raised by 4.5 m in order to create additional balancing capacity of 1 050 Mℓ. Given the age and history of the existing wall, raising of the wall will require a new structure with the existing wall at best being used as a “shutter” to part of the new wall structure. As per Option 4, a downstream pump station will be required to abstract raw water from the “hippo pool” with level protection in the form of a low weir structure. From the proposed pump station, a 1.4 m diameter pipeline will be tied into the existing 1.4 m diameter gravity pipeline from the Scheepersvlakte Balancing Dam. The deterioration of water quality due to irrigation return seepage/flows between the Korhaansdrift Weir and the proposed pump station, and the high risk of water losses over the abstraction weir, are real concerns for this option as well.

4.2.6 Option 6: A larger dam near the present Scheepersvlakte Balancing Dam to be integrated with the existing gravity pipeline to Nooitgedagt WTW

This option is based on the construction of a dam in the valley north-east of the existing Scheepersvlakte Balancing Dam, as shown in **Figure 4.2**. Water will be abstracted just upstream of the last long weir in the main canal, but downstream of the Coerney siphon offtake. The supply pipeline between this main canal abstraction point and the proposed dam will be a 1.4 m diameter x 880 m long steel pipe. The gravity supply, between the dam and the existing gravity pipeline to the Nooitgedagt WTW, will be a 1.4 m diameter x 730 m long steel pipeline.

The site falls on land being planned for development by the Scheepersvlakte 98 Citrus Development Trust (Scheepersvlakte Farms (Pty) Ltd (SVPL)), hereafter referred to as "the Developer" (see **Figure 4.2**). The Developer plans to construct a small dam on the same site as the identified balancing dam. A meeting was held with the Trustees and Engineers of the Trust in December 2016. The Trust agreed to co-operate with the DWS evaluation and possible future works, should this new dam option be pursued further. The Developer has received a water use authorisation from the DWS for the abstraction of a maximum of 5 850 000 m^3/a, for the development of 650 ha of citrus. Their environmental impact assessment for the development is underway.

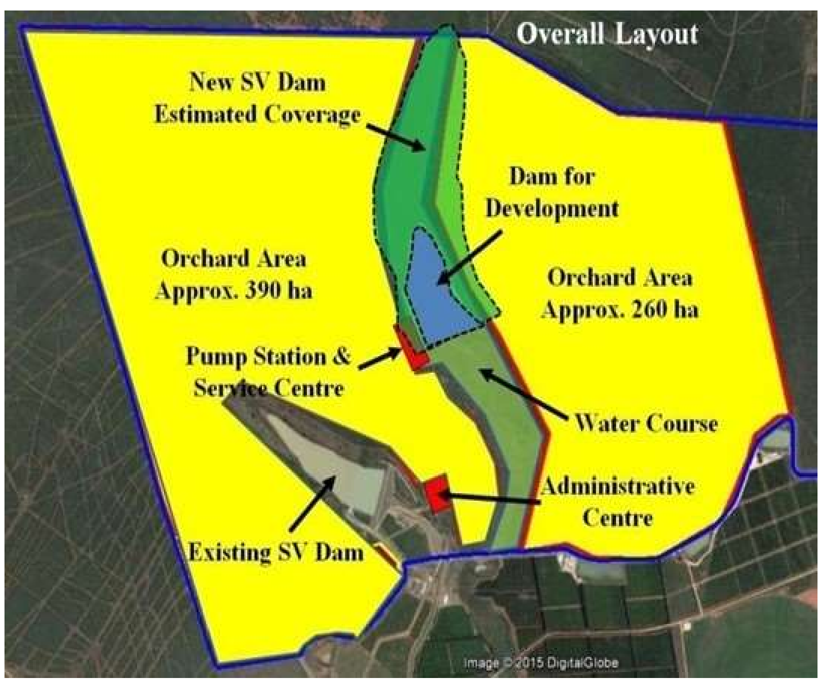

Figure 4.2: Scheepersvlakte 98 Citrus Development

The Developer is prepared to delay start of construction of their dam until year three of their development programme to allow DWS to finalise its decisions on options and preliminary design. This is subject to the Developer being allowed to pump for five years from the present canal.

The dam for this option shall be sized for the combined NMBM balancing capacity of 4 410 Mℓ (21 days balancing storage) plus the Developer's required irrigation capacity. Construction costs will be shared on a pro rata basis.

4.2.7 Option 7: A large balancing dam on the right bank near the Nooitgedagt WTW

This option considers the possibility of providing additional storage near the Nooitgedagt WTW, which would have the following advantages:

- The storage would enable the works to continue to operate for a reasonable period while maintenance or repairs are done on the damaged components of the upstream sections of the supply system (all components are upstream).
- The proposed dam would supply the Nooitgedagt WTW by gravity, although it may be necessary to pump water into the dam.
- All future peak demands on the Nooitgedagt WTW could be supplied by gravity.

The position of the proposed dam at the Nooitgedagt WTW is shown in **Figure 4.3**.

The existing 1.4 m diameter steel pipeline delivers water to a balancing tank located above the works at about related level (RL) 85 m. It may be possible to fill the dam by gravity when the Scheepersvlakte Balancing Dam is at or near full capacity. With the proposed full supply level (FSL) at RL 88 m and the Scheepersvlakte Balancing Dam at lower levels, a booster pump station will be required near the northern boundary of the Nooitgedagt WTW site.

It is proposed that water should be supplied at one end of the proposed dam and abstracted from the other end to provide circulation and minimise the risk of algal growth. On the other hand, wind and wave action is likely to cause circulation within the water body and therefore it seems unlikely that there would be any significant benefit in separating the inlet and outlet. However, it would probably be desirable to provide a multi-level abstraction tower. A very small spillway would suffice.

The electricity transmission line serving farms to the south-east of the Nooitgedagt WTW would have to be relocated.

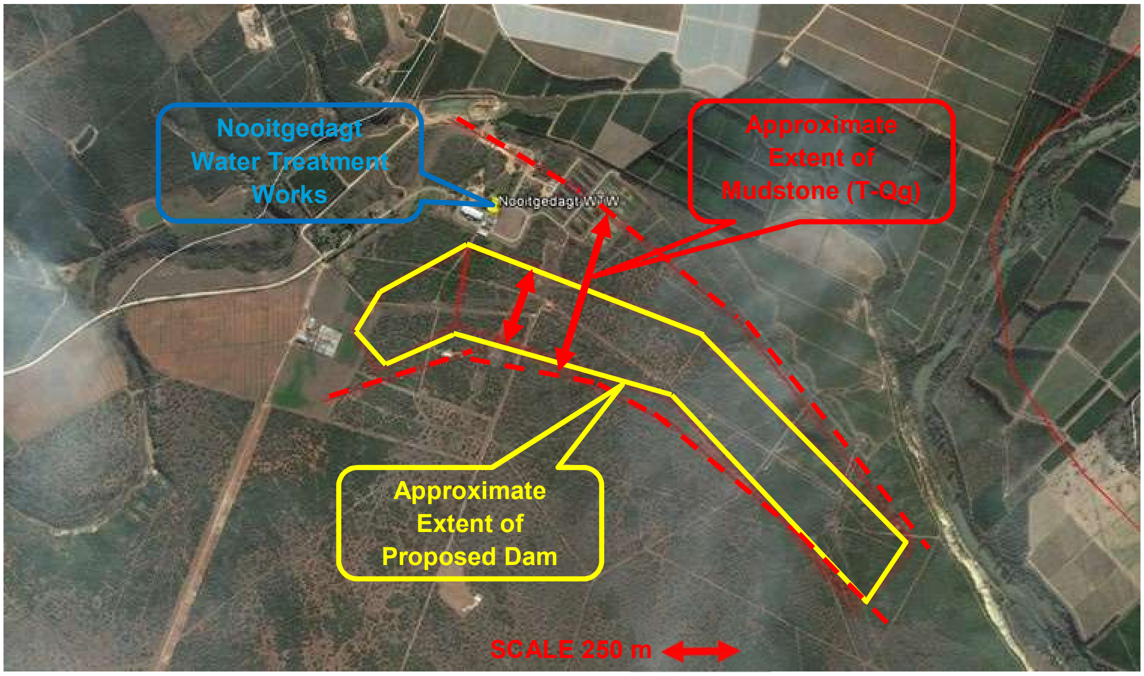

Figure 4.3: Possible site for proposed balancing *dam close* to the Nooitgedagt WTW

4.3 Preliminary Screening of options and Recommendations

4.3.1 Preliminary Screening

Table 4.1 is a Risk Matrix compiled for the seven options under consideration and is based on the discussions under **Section 4.2**.

Options 1 to 5 have high risks for the continuity of water supply, either during construction or during operation. The direct and indirect costs associated with the risk of interruptions in water supply for both urban and agricultural water users, have ruled these five options out for more detailed investigations and evaluation.

Table 4.1: Risk Assessment Matrix for Options under consideration

Criteria applied	Option 1	Option 2	Option 3	Option 4	Option 5	Option 6	Option 7
Risk to supply during Construction	Low	Medium	**High**	Medium	**High**	Low	Low
Risk to NMBM during Operation	**High**	**High**	Medium	**High**	Medium	Low	Low
Operational risks for LSRWUA	**High**	**High**	Medium	**High**	Medium	Low	Low
Capital and Operational costs	Low	**High**	**High**	Low	Low	**High**	Medium

Criteria applied	Option 1	Option 2	Option 3	Option 4	Option 5	Option 6	Option 7
Environmental risk issues	Low	Medium	Medium	Low	Low	Medium	Medium
Water quality deterioration in operation	Low	Low	Low	**High**	**High**	Low	Low

4.3.2 Preliminary Screening Recommendations

The following recommendations were made and approved by all role players following the preliminary screening of options:

1. Based on the assessments made on the various options, it is recommended that two options (including sub-options) be evaluated further, namely:
    - Construct a larger dam near the existing Scheepersvlakte Balancing Dam site and integrate it with the existing gravity pipeline to the Nooitgedagt WTW.
    - Construct a large balancing dam on the right-bank near the Nooitgedagt WTW in combination with a raw water booster pump station.
2. A detailed geotechnical study must be undertaken for both proposed dam sites to assess the suitability of both sites for the proposed dam structures.
3. A topographical site survey of both the proposed dam sites must be done to facilitate the preliminary design and costing of the structures.
4. The Korhaansdrift Weir has been identified as the structure in the present bulk raw water supply system with the highest risk in water supply for both the NMBM and the LSRWUA. A dam safety inspection should be carried out on the structure. The DWS Dam Safety regional representative was advised in this regard.
5. A preliminary environmental inspection of both dam sites should be carried out to determine whether any concerns or fatal flaws in terms of endangered flora and fauna exists, and what mitigation steps, if applicable, can be identified.
6. The request by the Developer, Scheepersvlakte 98 Citrus Development Trust, to abstract water for an interim period of five years from the Scheepersvlakte Balancing Dam outlet for five days per week (no discharges available over weekends) must be addressed as a matter of urgency.

All these recommendations were implemented.

5 Identification of Sub-Options to Evaluate

5.1 Identification of Sub-options

For the two short-listed options as recommended from the preliminary screening exercise, the following sub-options were identified in the vicinity of Scheepersvlakte Dam and close to Nooitgedagt WTW, as shown in **Figure 5.1**:

- Options in the vicinity of Scheepersvlakte Dam:
  - Measures to reduce the risk of a failure of the 1 420 mm pipeline from Scheepersvlakte to Nooitgedagt, which would be required together with each of the 'Scheepersvlakte Dam' options below.
  - Raising of Scheepersvlakte Dam.
  - Upper Scheepersvlakte Dam, which would be sited upstream of the existing Scheepersvlakte Dam and would impact on the planned Scheepersvlakte Farms private irrigation development.
  - Lower Coerney Dam, which would be shared with the Scheepersvlakte Farms private irrigation development.
  - Upper Coerney Dam.
- Four alternative sites for a balancing dam on the right bank near the Nooitgedagt WTW.

Preliminary visits to these sites by the study team (engineers and an engineering geologist) were undertaken on 9 and 10 October 2017, together with representatives of the land owners.

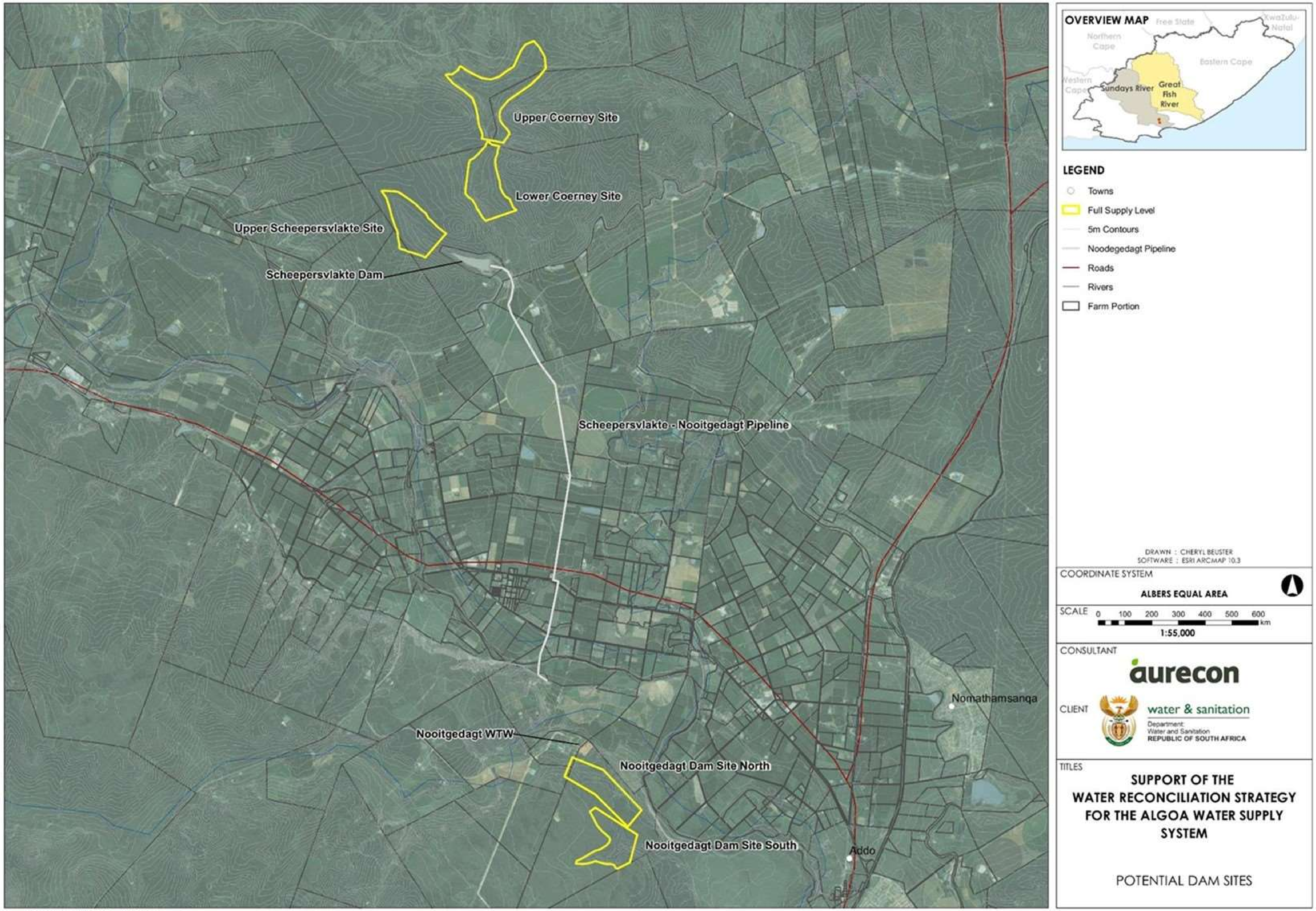

Figure 5.1: Options for Balancing Dams near Scheepersvlakte Dam and Nooitgedagt WTW

5.2 Previous Proposals for Remedial Works and Improvements at Scheepersvlakte Dam

The Second Draft Report on the *Reliability of Scheepersvlakte Dam for Domestic Water Supply* of DWS, prepared by Naidu Consulting and dated 6 January 2016, identified a number of additional problems with the existing infrastructure at Scheepersvlakte Dam, as well as possible solutions, as follows:

- The lack of usable capacity in the dam, which is determined by the top operating level, and the hydraulic characteristics of the dam's outlet works and of the Nooitgedagt pipeline.
- The reduced top operating level being set by the LSRWUA operations staff to avoid spillage from the dam.
- The draining of the canal system over weekends that leaves only the water stored in the dam to supply the WTW.
- The outlet works that convey dam water to the Coerney Lower Canal and the Nooitgedagt pipeline are prone to mechanical failures, which generally require a 3-day complete shut-down to remove or re-install a faulty valve.
- The Scheepersvlakte siphon cross connection to the Nooitgedagt pipeline causes backflow through the outlet works into the dam when opened, preventing maintenance work at the outlet works.
- The bottom intake in the dam and bottom orientation of the offtakes to the pipeline, results in sediment and debris from the dam being drawn into the Nooitgedagt pipeline. The sediment deposits from the canal are evident in **Figure 5.2**.

Figure 5.2: Sediment in Scheepersvlakte Dam

The Naidu Consulting report, prepared for the DWS, also recommends that the following betterment works should be undertaken:

- Install an isolating valve and a non-return valve on the 1 420 mm Nooitgedagt pipeline to prevent backflow from the cross-connection to the Scheepersvlakte siphon.
- Modify the dam's outlet works, to enable future maintenance and repairs to be undertaken without requiring 3-day shut downs and draining of the dam.
- Construct a direct connection between the Nooitgedagt pipeline and the main canal to replace the existing siphon, and separate the operation of the Nooitgedagt pipeline from the operation of the Upper Coerney Canal.
- Raise the Scheepersvlakte Dam wall and spillway to increase the dam's capacity to at least 3 days of NMBMM's water requirement.
- Attend to outstanding recommendations from the Dam Safety Inspection Reports.

6 Considerations for Balancing Storage and its Operation

6.1 Introduction

In view of the concerns described above, each of the options to provide 21 days of balancing storage for NMBM should also take the following into account:

- Sufficient storage should be provided to supply 210 Mℓ /day to NMBM for 21 days plus treatment losses of about 3% i.e. 4 542 Mℓ (4.54 million m^3).
- The canal is normally shut down for about 2 days per week because of labour considerations.
- If possible, the storage reservoir should be located so that it can supply water by gravity to the WTW.
- The scheme should preferably minimise reliance on Scheepersvlakte Dam.
- If pumping is required, then this should be as little as possible to maintain the storage.
- Sedimentation from the natural catchment area and from any on-going filling from the canal.
- The Dam Safety requirements concerning floods and freeboard.
- The risk of failure of the options should be similar, and therefore, to reduce the risk of failure of the 'Scheepersvlakte Options', it has been assumed that an additional siphon would be provided for the 1 400 mm pipeline under the Sundays River.

6.2 Sedimentation

6.2.1 Sediment survey

The reservoir basin surveys of Scheepersvlakte Dam undertaken by DWS in 1992 and 2014, as reported by DWS, indicate that the deposition of sediment has reduced the capacity by about 51 000 m^3 from 820 300 m^3 to 769 300 m^3 representing a loss of capacity of about 2 320 m^3/annum.

6.2.2 Sediment from catchment

Scheepersvlakte Dam has a catchment area of approximately 4.9 km^2 and is situated within quaternary catchment 40C. The Water Research Commission publication *Surface Water Resources of South Africa 1990* indicates that the annual sediment yield of this 580 km^2 quaternary is 12 000 tons/annum. Assuming that the density of the sediment in Scheepersvlakte Dam is 1.35 tons/m^3 (based on the typical 50-year density) and that the reservoir traps all sediment from the catchment, then the loss of capacity due to sediment transport from the catchment would be about 1650 m^3 or about 75 m^3/annum with the balance of about 44 500 m^3 having been transported into the dam by the canal. Therefore about 2 240 m^3/annum of the total sediment deposition of 2 320 m^3/annum in Scheepersvlakte Dam arises from the canal inflows.

6.2.3 Sediment from canal flows

The DWS *Reliability of Scheepersvlakte Dam for Domestic Water Supply* Report provides the following information concerning canal flows into Scheepersvlakte Dam:

The design capacity of the canal to Scheepersvlakte Dam is 13.0 m^3/s. A flow of 6.5 m^3/s can be abstracted upstream of the dam through the Scheepersvlakte Siphon to the Upper Coerney Canal. The balance flows into the dam and is currently allocated as follows:

- A flow of up to 1.448 m^3/s to the Nooitgedagt WTW;
- A flow of 0.581 m^3/s to 521 ha of irrigation on the right bank, and
- A flow of up to 3.573 m^3/s to the lower (old) Coerney canal.

The flow balance shows a surplus of 0.898 m^3/s flowing into the dam.

The canal is currently operated for about 4.5 days per week and is dried out for 2.5 days per week. Assuming that the inflow into Scheepersvlakte Dam is 5.6 m^3/s (6.5 m^3/s minus 0.9 m^3/s) for 4.5 days of operation during the week then the inflow into the dam is approximately 114 million m^3/annum. Therefore, the average sediment load of the canal inflows into Scheepersvlakte Dam is about 22 250 m^3/annum or 0.002% of the canal inflow.

6.2.4 Conclusion

In accordance with the above the sediment loads for assessing alternative balancing storage options have been based on the following:

- Sedimentation from catchment areas: 15 m^3/km^2/annum.
- Sedimentation from canal inflows into reservoir: 0.002 % of inflows.

Although an inflow of water from the Scheepersvlakte Canal of 220 Mℓ /day through the Scheepersvlakte Dam would only deposit about 64 000 m^3 of sediment in the reservoir over a 50-year period, it would probably be preferable to bypass as much of the supply directly to the Nooitgedagt WTW, to minimise sediment deposition in Scheepersvlakte Dam, as this is impacting on the operation of the outlet works as discussed below. It is therefore proposed that for 4.5 days per week water should be supplied directly from the canal to the Nooitgedagt WTW and water would only be delivered via Scheepersvlakte Dam for 2.5 days per week. Therefore, over a 50-year period the volume of sediment contained in these inflows that would be deposited in the reservoir would only be about 23 000 m^3.

Figure 6.1 shows that much of the sediment that is transported into Scheepersvlakte Dam from the canal is deposited between the canal inlet and the intake for the outlet works. This arrangement may result in occasional high silt loads due to sediment from the deposits being washed into the inlet of the outlet works. This could perhaps be mitigated by relocating the cascade inlet of the Scheepersvlakte Canal so that the point of discharge of the canal is located further upstream.

Figure 6.1: Sediment deposition from canal flows into Scheepersvlakte Dam

6.3 Dam safety in relation to floods

According to the Regulations Regarding the Safety of Dams as published under Government Notice R139 in Government Gazette 35062 of 24 February 2012 (in terms of Section 123(1) of the National Water Act, 1998) a dam with a wall height of more than 5 m and storage capacity of more than 50 000 m^3 must be registered as a *dam with a safety risk*. This is the case for all dams considered in this study. Registered dams are then classified into one of three classes (Category 1, 2 or 3) according to a combination of their *Size* and *Hazard Rating* as defined in **Table 6.1**, as reproduced from the regulations.

Table 6.1: Category classification of dams with a safety risk

Size class	Hazard potential rating		
	Low	Significant	High
Small	Category I	Category II	Category II
Medium	Category II	Category II	Category III
Large	Category III	Category III	Category III

The first step of the classification considers the *Size*, or maximum wall height of the dam according to the table in the regulations, reproduced in **Table 6.2**. All the dams considered have a maximum wall height of more than 12 m and less than 30 m and are thus in the *Medium* size class.

Table 6.2: SANCOLD Guideline: Size Classification

Size class	Maximum wall height (m)
Small	More than 5 and less than 12
Medium	Equal to or more than 12 but less than 30
Large	Equal to or more than 30

Secondly the dam's *Hazard Rating* is defined based on three factors in the case of a failure of the dam, namely potential loss of life, potential economic loss and potential adverse impact on resource quality. The *Hazard Rating* is considered in light of these three variables and is deemed to be *Severe*. Thus, consulting **Table 6.3** and **Table 6.1** from the regulations classifies the dam as *Category 3* hazard rating.

Table 6.3: Hazard potential classification

Hazard potential rating	Potential loss of life	Potential economic loss	Potential adverse impact on resource quality
Low	None	Minimal	Low
Significant	Not more than ten ...	Significant	Significant
High	More than ten	Great	Severe

The *SANCOLD Guidelines on Safety in Relation to Floods* prescribe the design flood magnitudes, based on the category classification of the dam. The Safety Evaluation Floods for the very preliminary designs of the spillways for the various dam options (as part of the initial options screening), described below, have been based on the "Regional Maximum Flood" as was DWS's 2016 Safety Evaluation Report for Scheepersvlakte Dam.

The then Department of Water Affairs' Technical Report TR 137 entitled *Regional Maximum Flood (RMF) Peaks in South Africa*, shows that the catchment areas of the proposed dams would be situated in Region 5.2. DWS' Dam Safety Report for Scheepersvlakte Dam was based on Region 5.2. However, in view of the history of flooding in the area, the preliminary designs of the spillways described in this report have been based on Region 5.4.

6.4 Filling and maintaining water quality in proposed balancing storage dam

6.4.1 Maintenance of water quality

To limit the increase in the salinity of water in a balancing dam, because of evaporation, it will be necessary to supply water from the dam on a weekly basis, and to supplement additional required volumes of water, likely in winter, to ensure that an acceptable quality can be maintained in the balancing dam. It has been assumed, for evaluation purposes of balancing dam options where pumping is required, that water would be pumped at a rate of between 80 and 110 Mℓ /day, and that pumping would take place continuously, as required. As filling or refilling of the dam could take place at any time of the year, pumping costs have been based on the average cost of electricity as described below. This aspect needs to be more critically evaluated during feasibility design.

6.4.2 Electricity costs

For those balancing dam options for which pumping would be required for filling and to maintain water quality, the cost of electricity has been based on Eskom's average 2017 Megaflex Tariff for non-local authorities, plus 5.23% for the increase for 2018 that was approved by NERSA in December 2017. This average tariff is estimated to be R0.80/kWh, as shown in **Appendix B**.

The available capacity in Scheepersvlakte Dam may limit withdrawals during weekends when electricity tariffs are lowest. Therefore, it has been assumed that arrangements will be made to provide sufficient releases into the canal to operate pumps throughout the week to fill, refill or provide water for dilution at those proposed balancing dams for which pumping would be required.

6.4.3 Pumping costs

For those options for which pumping would be required to fill the proposed balancing dams, it has been assumed that this would take place throughout the week and that pumps would be provided to deliver between 80 and 110 Mℓ /day, which would fill a 4 540 Mℓ balancing dam in 57 days and 41 days respectively.

7 Options for Balancing Storage in the Vicinity of Scheepersvlakte Dam

7.1 Introduction

Four possible dam sites in the vicinity of Scheepersvlakte Dam have been identified, which could provide balancing storage of 4 540 Mℓ for an emergency supply of 210 Mℓ /day for 21 days to the Nooitgedagt WTW, including 3% for treatment losses, in the event of a failure of the Scheepersvlakte Canal, such as the failure in May 2017 of the Main Canal. Although this significant failure of the Canal was repaired within eight (8) days, this necessitated exceptional effort and arrangements, and therefore it seems prudent to plan for a 21-day outage.

The following considerations and options are discussed in this section of the report:

- Improvements to Scheepersvlakte Dam as proposed by DWS.
- Concerns about a possible failure of the 1 420 mm pipeline from Scheepersvlakte to Nooitgedagt WTW and suggested improvements to the pipeline.
- Options for providing 21 days of balancing storage as shown in **Figure 7.1**:
    - Raising of the existing Scheepersvlakte Dam, which would not be feasible, as discussed below.
    - Upper Scheepersvlakte Dam, which would be situated immediately upstream of the existing Scheepersvlakte Dam and would require pumping.
    - Lower Coerney Dam, situated upstream of the Coerney Siphon. This is the only option near Scheepersvlakte Dam that would not require pumping.
    - Upper Coerney Dam, which would require pumping.

Each of these options is described and discussed, also taking account the existing operating problems at Scheepersvlakte Dam and the proposals for improvements.

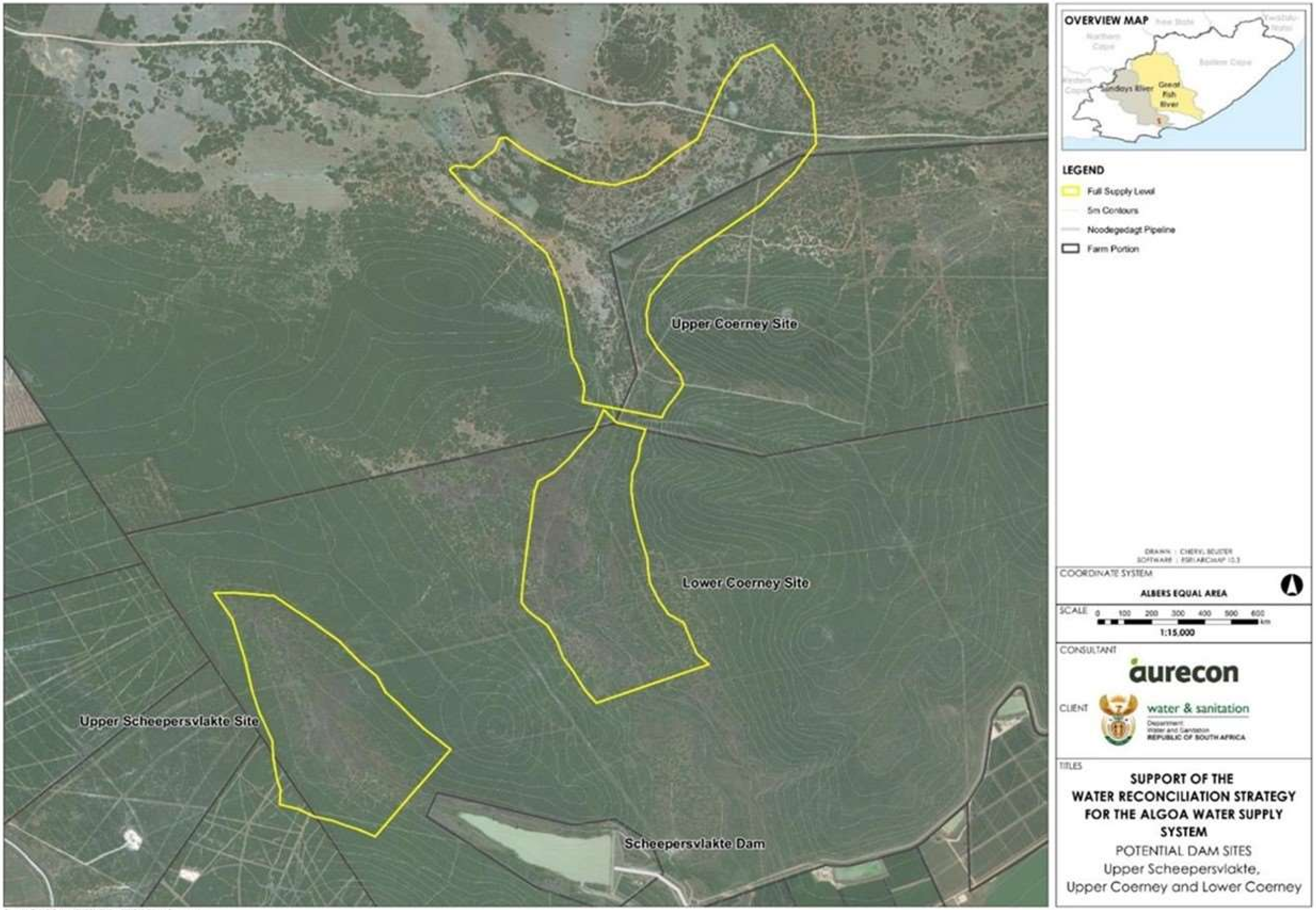

Figure 7.1: Possible sites for a balancing dam near Scheepersvlakte balancing dam

7.2 Improvements at Scheepersvlakte Dam

The DWS study undertaken by Naidu Consulting identified the following improvements that should be made at Scheepersvlakte Dam:

- Install an isolating valve and a non-return valve on the 1420 mm Nooitgedagt pipeline to prevent backflow from the cross-connection to the Scheepersvlakte siphon.
- Modify the dam's outlet works, to enable future maintenance and repairs to be undertaken without requiring 3-day shut downs and draining of the dam.
- Construct a direct connection between the Nooitgedagt pipeline and the main canal, to replace the existing siphon and separate the operation of the Nooitgedagt pipeline from the operation of the Upper Coerney Canal.

The first option above would probably require that Scheepersvlakte Dam is taken out of service for a few days and the second option would require a considerable time.

The last option could probably be undertaken by taking the Scheepersvlakte Dam out of service for a relatively short period of time. After implementation, this option would enable the Nooitgedagt WTW to be supplied directly from the canal for up to 4.5 days per week, while Scheepersvlakte Dam is taken out of service, or for longer if the canal is not emptied each weekend during the period that maintenance work is undertaken on Scheepersvlakte Dam.

The provision of a direct offtake from the canal would provide the additional benefit that the deposition of silt in Scheepersvlakte Dam would probably be considerably reduced as the volume of sediment laden water that would flow through the reservoir would be reduced.

7.3 Operational constraints and concerns for future planning

Discussions were held with officials of the LSRWUA concerning the following constraints and concerns regarding the future operation of the Canals and of the Scheepersvlakte Dam:

- The main reason for the current operation of the canals for 4.5 days per week is because the canals have sufficient capacity to supply the full current allocation during this period and because of the additional costs and potential staffing problems that would arise if the canals were to be operated for 7 days per week.
- The filamentous algae which occur are reduced by drying out the canals for two days per week and only occasionally occur. These algae result in increased maintenance (cleaning) of the canal and could affect the operation of the Nooitgedagt WTW. However, there is provision for dealing with algae at the WTW.

- The provision of an offtake for the 1 420 mm pipeline to Nooitgedagt from immediately upstream of the long weir in the Canal, as shown in **Figure 7.2**, would be feasible and would present no operational problems. The weir is situated immediately upstream of Scheepersvlakte Dam and has a crest level of RL 105.8 m. This would maximise the gravity flow to the Nooitgedagt WTW.
- The storage in Scheepersvlakte Dam only serves the Nooitgedagt WTW and one or two irrigators. The dam is filled at the end of the week to supply Nooitgedagt WTW during the weekend. It is assumed that the irrigators are not supplied during the weekend.

For this report it has been assumed that in future water will be supplied directly from upstream of the long weir in the canal shown in **Figure 7.2**, via additional infrastructure to be constructed, for 4.5 days per week and from the existing Scheepersvlakte Dam for 2.5 days per week.

Figure 7.2: Long weir upstream of Scheepersvlakte Dam - Crest Level: 105.8m

7.4 Scheepersvlakte-Nooitgedagt pipeline

7.4.1 Measures to improve pipeline security

It has been suggested by DWS that if the balancing storage is sited near Scheepersvlakte Dam, then it would also be necessary to reduce the risk of failure of the pipeline from there to the Nooitgedagt WTW (1 400 mm x 9 300 m long steel pipeline).

Duplication of the existing pipeline would probably rule out all options for balancing dams in the vicinity of Scheepersvlakte Dam because of the very high additional cost of approximately R240 million. Therefore, consideration has been given to the following:

- Duplicate the 180 m long siphon crossing of the Sundays River, which is potentially the most vulnerable section of the pipeline and would cost approximately R16 million.
- Provide and re-stock a small stockpile of pipes, which could be utilised to repair any pipes that may be damaged.
- Ensure that NMBM's pipeline maintenance unit is equipped to rapidly repair this pipeline, as well as the other major pipelines that serve the Metropolitan area.
- Plan other short-term emergency options, such as the utilisation of the spare capacity at the Loerie WTW (western supply system).

7.4.2 Pipeline capacity and utilisation

It has been assumed that the 1 420 mm cement mortar lined steel pipeline from Scheepersvlakte Dam to Nooitgedagt WTW would supply the following:

- 210 Mℓ /day to the Nooitgedagt WTW plus 3% losses, i.e. a total 24-hour demand of about 216 Mℓ /day.
- 600 mm/annum to 521 ha on the right bank of the Sundays River. If this irrigation water requirement is delivered over a period of 6 months during 4.5 days per week and 24 hours per day this would correspond to a water requirement of about 0.56 m^3/s. This is similar to the irrigation requirement of 0.581 m^3/s, which is mentioned in the DWS (Naidu Consulting) report and is discussed in Section 5.2.
- Therefore, for this preliminary investigation it has been assumed that the future water requirements to be supplied each week for an extended period, by the existing 1 420 mm pipeline, would be as follows:
  - 1 514 Mℓ /week (210 Mℓ /day 7 days per week) to the Nooitgedagt WTW including 3% for losses.
  - 0.581 m^3/s to irrigators for 12 hours per day for 4.5 days per week for a period of 6 months per year.

This worst-case flow pattern for the supply to the Nooitgedagt WTW was assessed for each of the options described in the following sections of this report.

7.4.3 Future supply from canal and Scheepersvlakte Dam

Supply to Nooitgedagt Water Treatment Works

For all options it is assumed that the proposed pipeline from the long weir to the existing 1 400 mm pipeline from Scheepersvlakte Dam would be constructed. This comprises a 180 m long 1 400 mm diameter pipeline from upstream of the long weir to connect to the existing Nooitgedagt pipeline, as well as associated pipework and valves. It is assumed that this system would normally be operated as follows:

- For 4.5 days per week the 180 m long 1 400 mm pipeline would supply water directly from the long weir in the canal to the Nooitgedagt WTW and the irrigators supplied from the pipeline.
- For 2.5 days per week water would be supplied from Scheepersvlakte Dam to the Nooitgedagt WTW.

Filling of Proposed Balancing Storage

For the rate of filling of the proposed balancing dam it is assumed that the water would be pumped or would gravitate (Lower Coerney site only) from the long weir in the canal, via the proposed 180 m long 1 400 mm pipeline. The dam would be sized to supply the full water requirement of 210 Mℓ /day plus 3% losses for 21 days.

The following alternatives for the rates of filling the proposed 21-day emergency storage dam are evaluated for each option:

- Filling in 90 days
- Filling in 180 days.

7.5 Raising of Scheepersvlakte Dam

Scheepersvlakte Dam is shown in **Figure 7.1**. If Scheepersvlakte Dam would be raised sufficiently to provide 21 days of balancing storage, then the dam wall would have to be raised by about 12 m to provide about 4.6 million m^3 of balancing storage for an emergency supply of about 220 Mℓ /day (210 Mℓ /day plus 3% losses). Therefore, the full supply level would have to be raised from 104.6 m to about 117 m. The raising of Scheepersvlakte Dam is, however, not feasible as the site is not suitable for raising the dam and spillway by the required 12 m.

If the site was suitable, the raised dam would require that most of the stored water would have to be pumped due to the lower level of the long weir in the canal that supplies Scheepersvlakte Dam (shown in **Figure 7.2**). The crest level of this weir is at RL 105.8 m, which is 11 m below the raised full supply level.

7.6 Upper Scheepersvlakte Dam

7.6.1 Introduction

The proposed Upper Scheepersvlakte Dam would be sited immediately upstream of the existing Scheepersvlakte Dam (refer to Figure 7.3) on Scheepersvlakte 98 Portion Number 7, as shown in **Figure 7.1**. This property is currently owned by Scheepersvlakte Farms, which proposes to develop the irrigation scheme that is discussed in **Section 7.7** below. The developer plans to establish approximately 60 ha orchards in the area that would be occupied by the proposed dam wall and would be inundated by the reservoir basin.

Figure 7.3: Upper Scheepersvlakte Dam site

The dam would be situated close to Scheepersvlakte Dam. Therefore, it is likely that the material available for construction and geotechnical conditions would be similar to those described in the then Department of Water Affairs 1992 Completion Report.

7.6.2 Description of dam

The main features of the proposed dam are described below and summarised in **Table 7.1**.

- The full supply level of the proposed dam would be at RL 128 m to provide a capacity of 4.6 million m^3. The lowest drawdown level would be at about RL 115 m.
- The storage in the dam would only be utilised in an emergency and therefore over 50 years only about 4 000 m^3 of sediment from the catchment would be deposited in the dam.
- The reservoir footprint would be about 60 ha.
- The dam would have a catchment area of 3.5 km^2 and although the safety evaluation flood would be about 220 m^3/s, this could be accommodated by a relatively small 10 m wide side channel spillway, with 2.5 m of freeboard that would provide significant flood attenuation.
- As the existing nearby Scheepersvlakte Dam is an embankment dam it is likely that suitable earthfill materials would be available in the vicinity to construct a zoned earthfill embankment dam with 1 in 3 upstream slope and 1 in 2 downstream slope, with cobblecrete upstream slope protection.

Table 7.1: Summary of characteristics of Upper Scheepersvlakte Dam

Characteristic	Upper Scheepersvlakte Dam
Type of dam	Zoned Earthfill Embankment
non-overspill crest level NOC (m amsl)	130.3
FSL (m amsl)	127.8
Freeboard (m)	2.5
Crest width (m)	5.0
DS slope (1V:H)	2.0
US slope (1V:H)	3.0
Embankment fill volume (m^3)	373,740
Core trench volume (m^3)	36,488
Crest length (m)	524
Total gross dam capacity (m^3)	4,600,000
Surface area at FSL (ha)	58.9
Maximum wall height (m)	25.3
Catchment area (km^2)	3.5

Characteristic	Upper Scheepersvlakte Dam
Unrouted safety evaluation flood (SEF) (m^3/s)	220
Spillway configuration description	Concrete-lined, 10m wide, side channel spillway located on the left abutment. (Note: spillway position dependant on geotechnical conditions)
Outlet works description	Dry well tower (25 m high) with inside dimensions of 4x4m. Three offtake levels controlled by valves.
Access road length (km)	2.0

The proposed layout of the Upper Scheepersvlakte Dam is given in **Figure 7.4**.

Figure 7.4: Proposed layout of the Upper Scheepersvlakte Dam and related bulk infrastructure

A typical cross-section of the proposed dam wall is given in **Figure 7.5**.

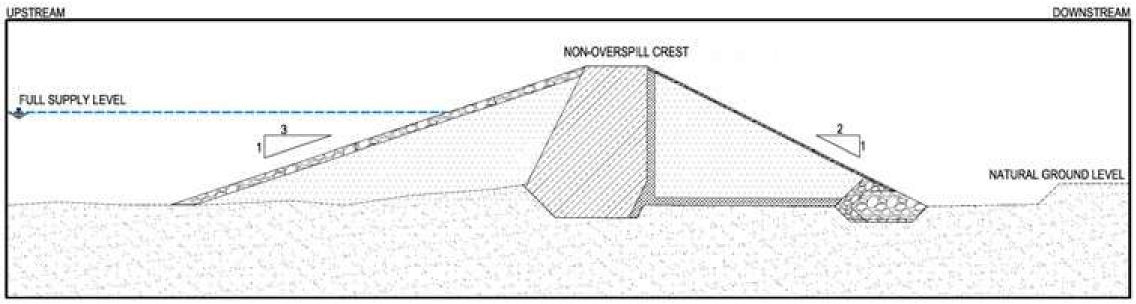

Figure 7.5: Typical cross-section of dam wall

7.6.3 Pipeline and pumping requirements

The proposed pipeline and pumping requirements to fill and release water from the proposed Upper Scheepersvlakte Dam would be as follows:

- A 1 580 m long 1 300 mm diameter pipeline would be required to supply the emergency releases of 210 Mℓ /day plus 3% losses from the proposed Upper Scheepersvlakte Dam to the Nooitgedagt WTW.
- The Upper Scheepersvlakte Dam would be filled by pumping as follows:
  - Water would be delivered by the 1 400 mm pipeline from the long weir in the canal to a pump station located below Scheepersvlakte Dam.
  - The pump station would deliver water to Upper Scheepersvlakte Dam via the 1 580 m long 1 300 mm pipeline.

7.6.4 Estimated capital costs

The estimated comparative (capital) costs (February 2018 prices, excluding VAT) of the proposed scheme are summarised below.

Upper Scheepersvlakte Dam	R129 million
Pipelines	R60 million
Pump Station	R116 million
TOTAL	**R305 million**

7.6.5 Advantages and disadvantages

The main advantages of the scheme would be as follows:

- The dam would be situated very close to the existing Scheepersvlakte Dam and associated conveyance infrastructure.

- The catchment area of the dam is small (3.5 km^2) and therefore a smaller spillway and less freeboard will be required.

The disadvantages of the scheme would be as follows:

- All the water stored in the dam would have to be pumped from the canal, which would be an additional operational cost.
- The dam would be situated on private property to be developed as orchards by Scheepersvlakte Farms, as indicated in Section 7.7.
- The developer may wish to share the use of the dam, which might complicate its operation.
- The pump station would be remote from the Nooitgedagt WTW and would have to be operated and maintained.
- The existing pipeline from Scheepersvlakte Dam to Nooitgedagt WTW may be vulnerable to damage by a major flood, as discussed in **Section 7.4**, although the risk would be significantly reduced by the proposed provision of a second siphon crossing, as included in the estimate of cost.

7.7 Lower Coerney Dam

7.7.1 Introduction

The proposed Lower Coerney Dam (**Figure 7.6**) would be sited upstream of the Coerney Siphon on Scheepersvlakte 98 Portion Number 7 of Scheepersvlakte Farms Pty Ltd in the vicinity of the site proposed by Scheepersvlakte Farms for a balancing dam, as indicated in **Figure 7.1**. The main advantage of the scheme is that it would provide a gravity supply to the WTW via the existing 1 400 mm Nooitgedagt pipeline and it would also be filled by gravity flow via the proposed pipeline from the canal.

7.7.2 Description of proposed dam

The dam would be situated in the valley adjacent to Scheepersvlakte Dam (refer to Figure 7.6). The Inconsult Engineers report on their *Geotechnical Investigation for the Proposed Irrigation Scheme Dam Near Port Elizabeth in the Eastern Cape*, dated 22 July 2016, recommended that a homogenous embankment dam, with upstream and downstream slopes of 1 in 3 and 1 in 2 respectively, be constructed at the site because of the very limited availability of impervious and semi-pervious material in the vicinity of the proposed site. Nevertheless, for this preliminary investigation it has been assumed that the proposed dam would comprise a zoned embankment with cobblecrete slope protection, as utilised for Scheepersvlakte Dam, which was constructed by DWS in the adjacent valley.

Figure 7.6: Lower Coerney Dam site

The main features of the dam are described below and summarised in **Table 7.2**.

- The full supply level of the proposed dam would be at about RL 99 m and the lowest drawdown level at about RL 86 m to provide a capacity of 4.8 million m^3, which was estimated as follows:
    - 4.6 million m^3 of storage for 21 days emergency supply to the Nooitgedagt WTW, as discussed in Section 7.5.
    - It was also assumed that 180 000 m^3 of storage would be provided for one (1) week of storage for the irrigation of 750 ha (360 ha plus 390 ha) of orchards on Scheepersvlakte Farms, assuming that irrigation of the 600 mm/annum allocation would take place over a period of 6 months.
    - The proposed dam would have a catchment area of 34 km^2. Assuming a sediment load of 15 m^3/km^2/annum, as discussed in Section 6.2.2, then about

26 000 m^3 of sediment from the catchment would be deposited over a 50-year period and another 4 000 m^3 from irrigation inflows over 50 years.
    - Indications are that, when the Coerney River flows, water quality of low flows can be poor, and measures may need to be provided to ameliorate the impact that this could have on the quality of water in the balancing dam. This may require a small weir with a pipeline routing poor quality low flows around the dam. This has not yet been investigated nor costed.
- The safety evaluation flood for the 34 km^2 catchment area would be approximately 890 m^3/s, however, attenuation would reduce this to approximately 800 m^3/s. As there does not appear to be any rock at the site, it has been assumed that a concrete lined side channel spillway with a crest width of 36 m and 5 m of freeboard would be provided.

Table 7.2: Summary of characteristics of Lower Coerney Dam

Characteristic	Lower Coerney Dam
Type of dam	Zoned Earthfill Embankment
NOC (m amsl)	103.8
FSL (m amsl)	98.8
Freeboard (m)	5.0
Crest width (m)	5.0
DS slope (1V:H)	2.0
US slope (1V:H)	3.0
Embankment fill volume (m^3)	355,993
Core trench volume (m^3)	46,798
Crest length (m)	623
Total gross dam capacity (m^3)	4,600,000
Surface area at FSL (ha)	59.7
Maximum wall height (m)	19.0
Catchment area (km^2)	34
Unrouted SEF (m^3/s)	890
Spillway configuration description	Concrete-lined, 36 m wide, side channel spillway located on the left abutment. (Note: spillway position dependant on geotechnical conditions) with downstream concrete outlet chamber, 4x4x3m, with 2 valves for the two pipes.
Outlet works description	Dry well tower (19 m high) with inside dimensions of 4x4m. Three offtake levels controlled by valves.
Access road length (km)	1.0

The proposed layout of the Lower Coerney Dam is given in **Figure 7.7**. The cross-section of the dam was assumed to be similar to that of the proposed Upper Scheepersvlakte Dam (**Section 7.6**), shown in **Figure 7.5**.

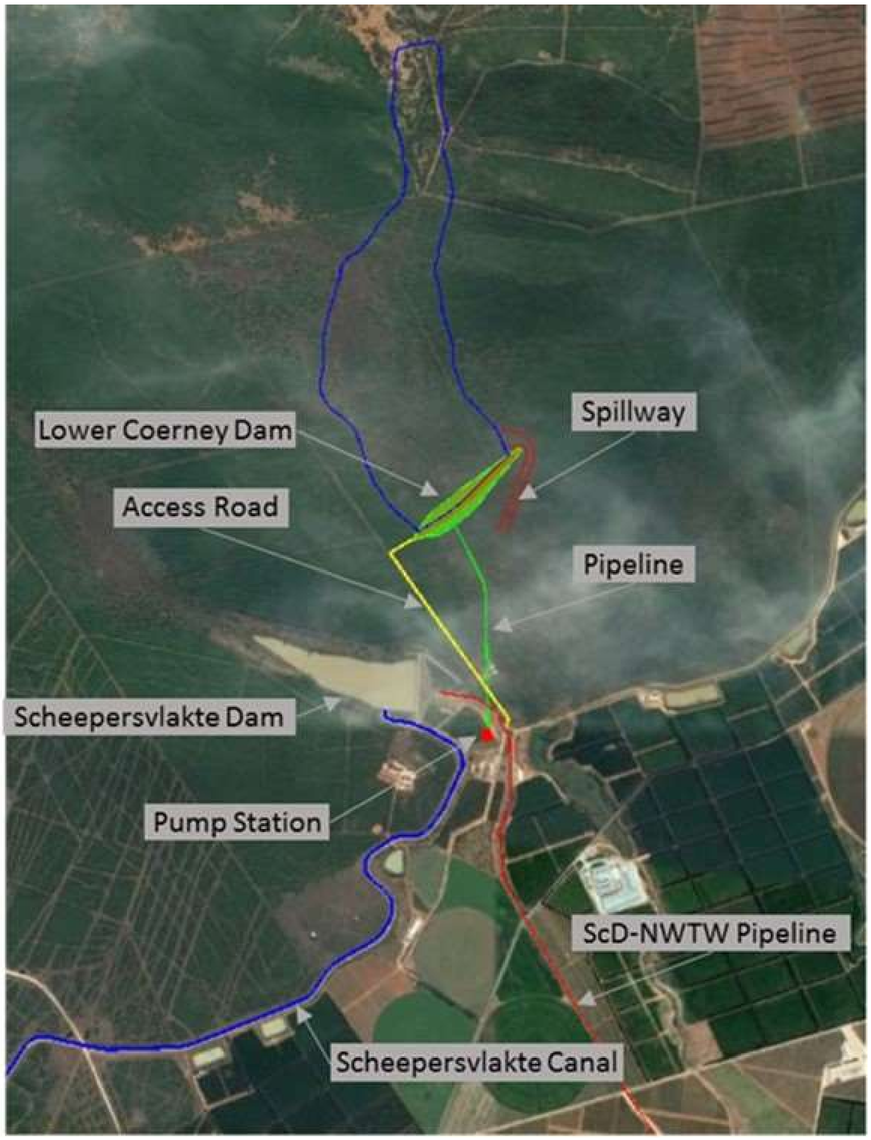

Figure 7.7: Proposed layout of the Lower Coerney Dam and related bulk infrastructure

7.7.3 Pipeline requirements and operation

The pipeline and operation of the proposed Lower Coerney Dam would be as follows:

- A 1 400 mm diameter 200 m long connector pipe would deliver water from the long weir in the canal to the 1 400 mm pipeline to Nooitgedagt WTW and to the proposed 940 m long 1 500 mm diameter pipeline to the Lower Coerney Dam.
- The 940 m long 1 500 mm gravity pipeline would deliver water to fill the dam and the pipeline would also be used to supply the Nooitgedagt WTW in the event of a failure of the canal.
- Modifications to the inlet to the Nooitgedagt WTW would also be required to make up for head loss and to increase the flow in the existing 1 400 mm Nooitgedagt pipeline at times when the storage in the proposed Lower Coerney Dam would be drawn down.

7.7.4 Estimated capital costs

The estimated comparative (capital) costs (February 2018 prices, excluding VAT) of the proposed scheme are summarised below.

Lower Coerney Dam	R152 million
Pipelines	R49 million
TOTAL	**R201 million**

No pumping would be required.

7.7.5 Operation for salinity

The irrigation usage from the proposed dam would be about 4.5 million m^3/annum and would probably provide sufficient dilution to limit the increase in salinity in the reservoir.

As no pumping would be required there would be no electricity costs.

7.7.6 Advantages and disadvantages

The main advantages of the scheme would be as follows:

- The dam would be situated close to Scheepersvlakte Dam and associated conveyance infrastructure.
- The scheme would be a gravity supply to fill the dam and to deliver water to Nooitgedagt WTW (no pumping required).
- The comparative capital cost as well as the cost of operation for this option is the lowest of the five options investigated.
- The irrigation water that passes through the dam would probably be sufficient to maintain acceptable salinity for urban consumption and may need to be managed to

ensure that the quality would be acceptable for citrus. No electricity costs would be incurred if water must be abstracted and replaced to maintain acceptable salinity levels.

The possible disadvantages of the scheme would be as follows:

- The dam would be situated at the outlet of a relatively large catchment area (34 km^2) and a major flood could cause damage downstream of the spillway as there is no evidence of rock at the site.
- The reserve storage and infrastructure would be remote from Nooitgedagt WTW and an additional siphon under the Sundays River would be required to reduce the risk of wash away of the existing 1 400 mm siphon.
- The potential joint use of the dam's water by the Municipality and the private developer would need careful planning.

7.8 Upper Coerney Site

7.8.1 Introduction

The proposed Upper Coerney dam site is situated about 1.5 km upstream of the Lower Coerney Dam site and approximately 2.3 km upstream of the Coerney siphon. The dam and its reservoir basin would extend across two privately owned properties: Enon Mission Station 40-0, which is owned by Enon Mission, and Uitenhage Road 713-0, which is owned by the Venter Wildlife Trust.

7.8.2 Description of dam

The proposed dam wall would be located where the valley narrows but widens upstream to provide a suitable storage basin. The main features of the dam are described below and summarised in **Table 7.3**.

- There is no geotechnical information available concerning materials in the reservoir basin, and therefore for this very preliminary assessment it has been assumed that the dam wall would comprise a zoned earth embankment, as suggested for the costing of the Lower Coerney Dam.
- The full supply level of the proposed dam would be at about RL 109.1 m and the lowest drawdown level at about RL 95.0 m to provide a capacity of 4.6 million m^3 for 21 days emergency supply. Pumping would be required to fill the dam.
- The proposed dam would have a catchment area of 30 km^2. Assuming a sediment load of 15 m^3/km^2/annum, as discussed in Section 6.2.2, then about 23 000 m^3 of sediment from the catchment area would be deposited over a 50-year period.

- The safety evaluation flood for the 30 km^2 catchment area of approximately 820 m^3/s, would be attenuated to about 700 m^3/s by the reservoir. As there does not appear to be any rock at the site, it has been assumed that a concrete lined side channel spillway with a 32 m crest width and 5 m of freeboard would be provided.
- Indications are that, when the Coerney River flows, water quality of low flows can be poor, and measures may need to be provided to ameliorate the impact that this could have on the quality of water in the balancing dam. This may require a small weir with a pipeline routing poor quality low flows around the dam. This has not been investigated further nor costed.

Table 7.3: Summary of characteristics of Upper Coerney Dam

Characteristic	Upper Coerney Dam
Type of dam	Zoned Earthfill Embankment
NOC (m amsl)	114.1
FSL (m amsl)	109.1
Freeboard (m)	5.0
Crest width (m)	5.0
DS slope (1V:H)	2.0
US slope (1V:H)	3.0
Embankment fill volume (m^3)	246,363
Core trench volume (m^3)	21,507
Crest length (m)	357
Total gross dam capacity (m^3)	4,600,000
Surface area at FSL (ha)	74.5
Maximum wall height (m)	19.3
Catchment area (km^2)	30
Unrouted SEF (m^3/s)	824
Spillway configuration description	Concrete-lined, 32 m wide, side channel spillway located on the right abutment. (Note: spillway position dependant on geotechnical conditions)
Outlet works description	Dry well tower (19 m high) with inside dimensions of 4x4m. Three offtake levels controlled by valves with downstream concrete outlet chamber, 4x4x3m, with 2 valves for the two pipes.
Access road length (km)	5.3

The proposed layout of the Upper Coerney Dam is given in **Figure 7.8**. The cross-section of the proposed dam was assumed to be similar to that of the proposed Upper Scheepersvlakte Dam (**Section 7.6**), as shown in **Figure 7.5**.

Figure 7.8: Proposed layout of the Upper Coerney Dam and related bulk infrastructure

7.8.3 Pipeline requirements and operation

The proposed dam would be filled by pumping water from the Scheepersvlakte Canal. The additional pipelines and other measures that would be required and their operation would be similar to those for the Lower Coerney Dam option, as described below, except that water would have to be pumped into the dam:

- A 1 400 mm diameter 200 m long connector pipe would deliver water from the long weir in the canal to the 1 400 mm pipeline to Nooitgedagt WTW, and to the proposed 2 460 m long 1 400 mm diameter pipeline to the dam, via the Lower Coerney Dam site.
- In the event of a failure of the canal, water would be released from the dam via the 2 460 m long 1 400 mm pipeline to supply the Nooitgedagt WTW.

7.8.4 Estimated capital costs

The estimated comparative (capital) costs (February 2018 prices, excluding VAT) of the proposed scheme are summarised below.

Upper Coerney Dam	R129 million
Pipelines	R80 million
Pump Station	R116 million
TOTAL	**R325 million**

7.8.5 Advantages and disadvantages

The main advantages of the scheme would be as follows:

- The dam would provide a gravity supply to deliver water in an emergency to Nooitgedagt WTW.
- The dam would be situated relatively close to the Scheepersvlakte Dam and associated conveyance infrastructure.

The possible disadvantages of the scheme would be as follows:

- The dam would be situated at the outlet of a relatively large catchment area (30 km^2) and a major flood could cause damage downstream of the spillway, as there is no evidence of rock at the site.
- Water would have to be pumped into the dam.
- The reserve storage and infrastructure would be remote from Nooitgedagt WTW and an additional siphon under the Sundays River would be required to reduce the risk of failure of the system.
- The dam and reservoir basin would extend across two properties.
- This option has the highest comparative capital cost of the three options investigated in the vicinity of Scheepersvlakte Dam. The Upper Coerney Dam does not offer any real advantage over the other two options.

8 Alternative Nooitgedagt Dam Sites

8.1 Introduction

Aurecon's Report entitled *Identification of Options for Balancing Storage* identified four possible sites for a balancing dam near the Nooitgedagt WTW. The main advantages of these sites would be as follows:

- The balancing dam would be located very close to the Nooitgedagt WTW and therefore could be easily managed by the operating staff at the Works.
- The supply would not be vulnerable to a failure of the Scheepersvlakte to Nooitgedagt pipeline.

Four possible sites for a balancing dam, to provide 21 days of storage, were assessed. All the sites would be situated on Erf 119 Portion 1, which is owned by Rolust Sondagsrivierplase CC, according to Windeed (but may currently be owned by Wicklow Trust). This property is currently utilised as a game reserve; however, the owners have indicated that they are planning to develop some of the area for irrigation. They will be requesting the LSRWUA to approve the relocation of the point of abstraction of their existing water allocation, from the 1 420 mm pipeline, to the vicinity of the Nooitgedagt WTW. Wicklow Trust has also advised in their letter dated 12 October 2017 that the construction of a dam at the Nooitgedagt North Option 1 site would not be acceptable and that only the Nooitgedagt South site would be acceptable as indicated.

8.2 Nooitgedagt North Option 1

8.2.1 Introduction

The location of the proposed Nooitgedagt North Option 1 dam is shown in **Figure 8.1**. The dam would be located close to three 11 kV/ 22 kV transmission lines and close to the main 400 kV transmission line, which supplies power to NMBM.

8.2.2 Description of Dam

The limited geotechnical inspection of the site indicated that suitable material would probably be available for the construction of a cut to fill dam. The dam would have virtually no catchment area, other than the reservoir basin, and therefore only a nominal overflow channel, which would discharge into the adjacent valley, would be provided. A summary of the characteristics of the dam is given in **Table 8.1**.

Table 8.1: Summary of characteristics of Nooigedagt North Option 1

Characteristic	Nooitgedagt North Option 1
Type of dam	Zoned Earthfill Embankment
NOC (m amsl)	103.0
FSL (m amsl)	101.0
Freeboard (m)	2.0
Crest width (m)	5.0
DS slope (1V:H)	2.0
US slope (1V:H)	3.0
Embankment fill volume (m^3)	1,053,106
Core trench volume (m^3)	98,621
Crest length (m)	2,739
Total gross dam capacity (m^3)	4,804,000
Surface area at FSL (ha)	78.0
Maximum wall height (m)	16.0
Catchment area (km^2)	N/A
Unrouted SEF (m^3/s)	N/A
Spillway configuration description	Unlined 5 to 10 m wide, side channel spillway located on the left abutment. (Note: spillway position dependant on geotechnical conditions)
Outlet works description	Dry well tower (19 m high) with inside dimensions of 4x4m. Three offtake levels controlled by valves with downstream concrete outlet chamber, 4x4x3m, with 2 valves for the two pipes.
Access road length (km)	2.8

The proposed layout of the Nooitgedagt North Option 1 Dam is given in **Figure 8.1**. The cross-section of the dam was assumed to be similar to that of the proposed Upper Scheepersvlakte Dam (**Section 7.6**), as shown in **Figure 7.5**.

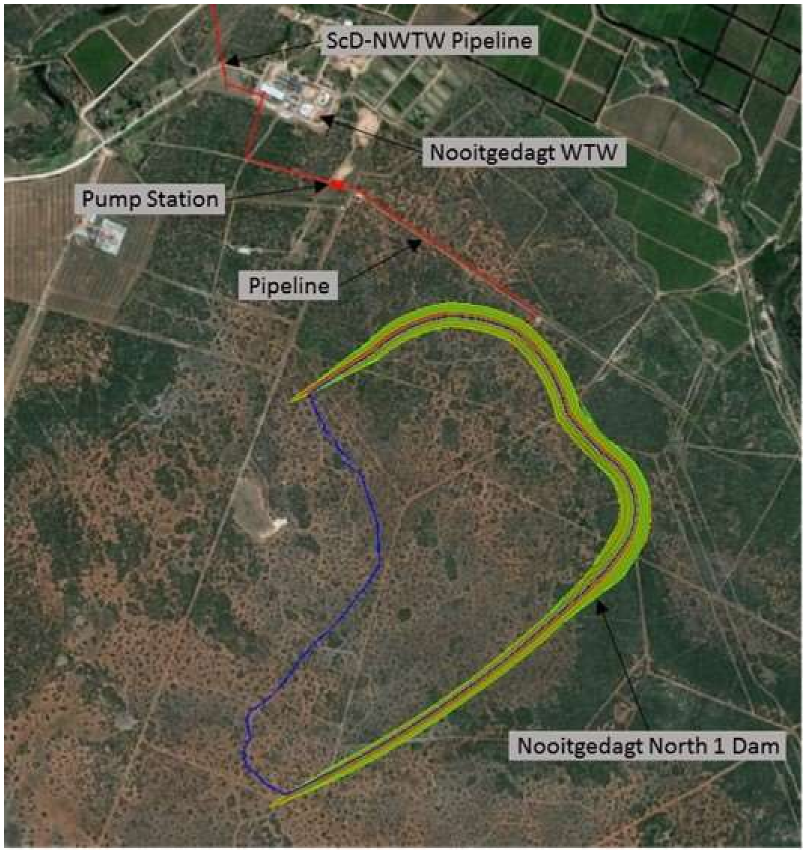

Figure 8.1: Proposed layout of the Nooitgedagt North Option 1 Dam and related bulk infrastructure

8.2.3 Pump and Pipeline Requirements

A pump station located on the site of the Nooitgedagt WTW would deliver water to the dam via a short 240 m long pipeline. The flow would be reversed in this pipeline when it is necessary to utilise water stored in the dam to supply the Nooitgedagt WTW.

8.2.4 Estimated Capital Costs

The estimated comparative (capital) costs (February 2018 prices, excluding VAT) of the proposed scheme are summarised below.

Nooitgedagt North Option 1 Dam	R297 million
Pipelines	R23 million
Pump Station	R94 million
TOTAL	**R414 million**

8.2.5 Advantages and Disadvantages

The main advantages of the scheme would be as follows:

- The dam would provide a gravity supply to deliver water in an emergency to Nooitgedagt WTW.
- The dam would be situated very close to the Nooitgedagt WTW and the pump station would be situated at the WTW site, which would facilitate maintenance and operation.
- This scheme has a lower risk of failure than those in the vicinity of Scheepersvlakte Dam as water is not supplied via a long pipeline and siphon.
- The dam would have virtually no catchment area and therefore only a small unlined spillway channel and limited freeboard would be required.

The possible disadvantages of the scheme would be as follows:

- The embankment volume to capacity ratio is relatively high and accounts for the relatively high cost, which is more than the most expensive option in the vicinity of Scheepersvlakte Dam. This cost could, however, potentially be slightly reduced. The 2 m of freeboard provided is conservative and other refinements may be possible.
- Water would have to be pumped into the dam.
- The property owner has advised that this proposed site for the dam is not acceptable due to possible seepage water affecting downstream orchards. Lining of the dam may therefore be required, depending on the soil permeability, which will further increase the capital cost.

8.3 Nooitgedagt North Option 2

8.3.1 Introduction

The location of the proposed Nooitgedagt North Option 2 dam is shown in **Figure 8.2**. The dam would require the relocation of three 11 kV/ 22 kV transmission lines and probably also the main 400 kV transmission line, which supplies power to NMBM.

8.3.2 Description of Dam, Pump and Pipeline Requirements

The main features of the dam would be similar to those for the Nooitgedagt North Option 1, as described in **Section 8.2.2**.

A summary of the characteristics of the dam is given in **Table 8.2**.

Table 8.2: Summary of characteristics of Nooigedagt North Option 2

Characteristic	Nooitgedagt North Option 2
Type of dam	Zoned Earthfill Embankment
NOC (mamsl)	95.0
FSL (mamsl)	93.0
Freeboard (m)	2.0
Crest width (m)	5.0
DS slope (1V:H)	2.0
US slope (1V:H)	3.0
Embankment fill volume (m^3)	1,341,678
Core trench volume (m^3)	113,319
Crest length (m)	2,960
Total gross dam capacity (m^3)	4,621,000
Surface area at FSL (ha)	72.0
Maximum wall height (m)	17
Catchment area (km^2)	N/A
Unrouted SEF (m^3/s)	N/A
Spillway configuration description	Unlined 5 to 10 m wide, side channel spillway located on the left abutment. (Note: spillway position dependant on geotechnical conditions)
Outlet works description	Dry well tower (19 m high) with inside dimensions of 4x4m. Three offtake levels controlled by valves with downstream concrete outlet chamber, 4x4x3m, with 2 valves for the two pipes.
Access road length (km)	2.9

The proposed layout of the Nooitgedagt North Option 2 Dam is given in **Figure 8.2**.

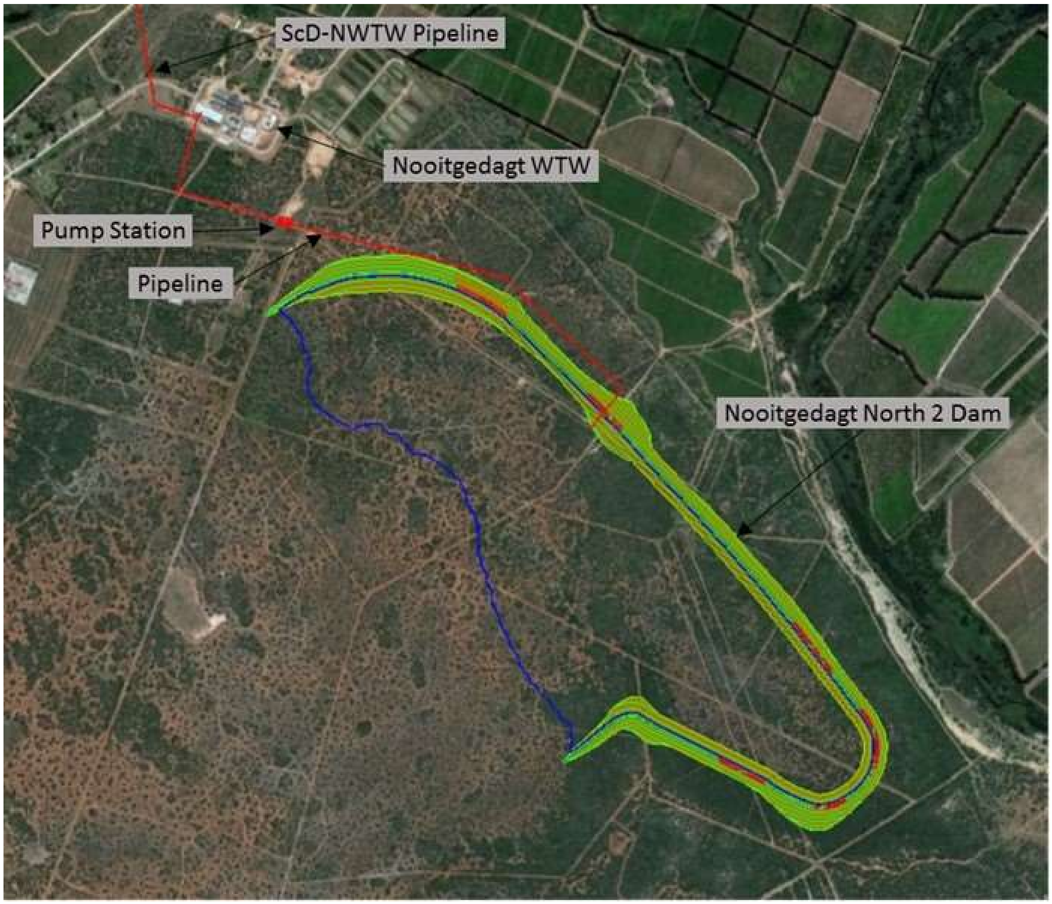

Figure 8.2: Proposed layout of the Nooitgedagt North Option 2 Dam and related bulk infrastructure

8.3.3 Estimated Capital Costs

The estimated comparative (capital) costs (February 2018 prices, excluding VAT) of the proposed scheme are summarised below.

Nooitgedagt North Option 2 Dam	R368 million
Pipelines	R23 million
Pump Station	R94 million
TOTAL	**R485 million**

8.3.4 Advantages and Disadvantages

The advantages and disadvantages of Nooitgedagt North Option 2 would be similar to those for Nooitgedagt North Option 1, as described in **Section 8.2.5**, but the dam would also have the following additional disadvantages:

- The 11/22 kV transmission lines and possibly also the 400 kV transmission line would have to be relocated.
- The capital cost would be significantly higher than that for Nooitgedagt North Option 1.

Nooitgedagt North Option 2 should therefore be eliminated from further consideration.

8.4 Nooitgedagt North Option 3

8.4.1 Description of Dam, Pump and Pipeline Requirements

The main features of the dam would be similar to those for Nooitgedagt North Option 1 as described in **Section 8.2.2**.

A summary of the characteristics of the dam is given in **Table 8.3**.

Table 8.3: Summary of characteristics of Nooigedagt North Option 3

Characteristic	Nooitgedagt North Option 3
Type of dam	Zoned Earthfill Embankment
NOC (m amsl)	104.0
FSL (m amsl)	102.0
Freeboard (m)	2.0
Crest width (m)	5.0
DS slope (1V:H)	2.0
US slope (1V:H)	3.0
Embankment fill volume (m^3)	922,688
Core trench volume (m^3)	89,000
Crest length (m)	2,159
Total gross dam capacity (m^3)	5,087,000
Surface area at FSL (ha)	68.0
Maximum wall height (m)	13
Catchment area (km^2)	N/A
Unrouted SEF (m^3/s)	N/A

Characteristic	Nooitgedagt North Option 3
Spillway configuration description	Unlined 5 to 10 m wide, side channel spillway located on the left abutment. (Note: spillway position dependant on geotechnical conditions)
Outlet works description	Dry well tower (19 m high) with inside dimensions of 4x4m. Three offtake levels controlled by valves with downstream concrete outlet chamber, 4x4x3m, with 2 valves for the two pipes.
Access road length (km)	2.7

The proposed layout of the Nooitgedagt North Option 3 Dam is given in **Figure 8.3**.

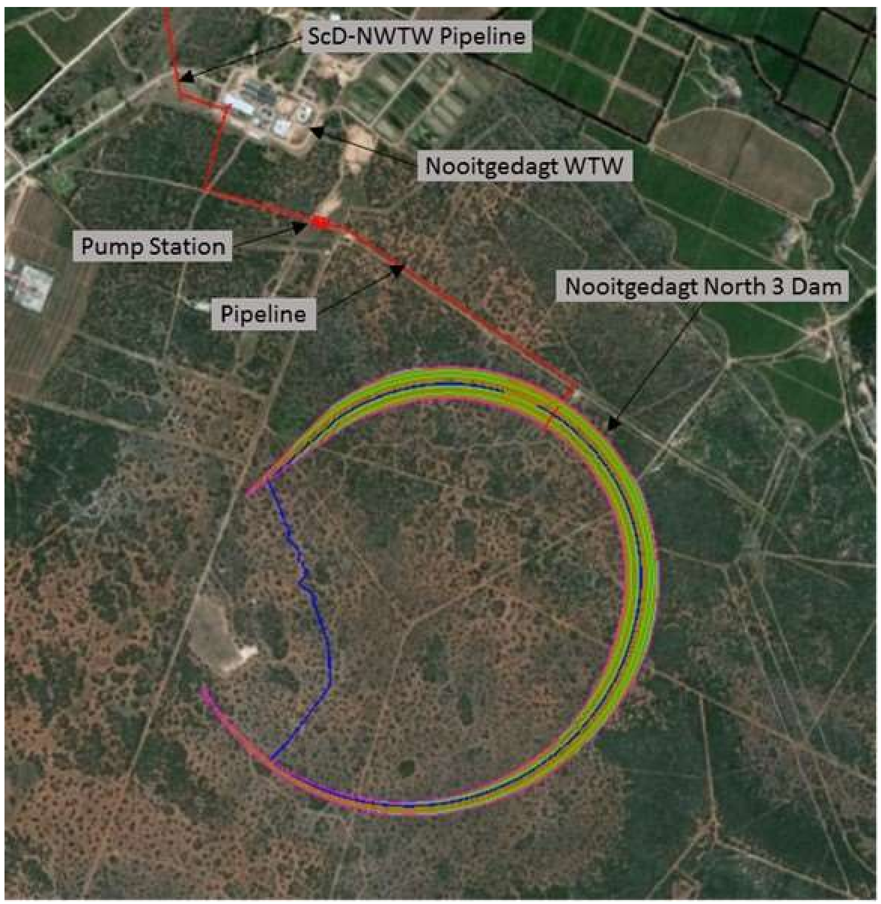

Figure 8.3: Proposed layout of the Nooitgedagt North Option 3 Dam and related bulk infrastructure

8.4.2 Estimated Capital Costs

The estimated comparative (capital) costs (February 2018 prices, excluding VAT) of the proposed scheme are summarised below.

Nooitgedagt North Option 3 Dam	R435 million
Pipelines	R23 million
Pump Station	R94 million
TOTAL	**R552 million**

8.4.3 Advantages and Disadvantages

The advantages and disadvantages of Nooitgedagt North Option 3 would be similar to those for Nooitgedagt North Option 1, as described in **Section 8.2**, but the dam would also have the following additional disadvantage:

- The capital cost would be significantly higher than that for Nooitgedagt North Option 1.

Nooitgedagt North Option 3 should therefore also be eliminated from further consideration.

8.5 Nooitgedagt South

8.5.1 Introduction

The site of the proposed Nooitgedagt South Dam is shown in **Figure 8.4**.

8.5.2 Description of Dam, Pump and Pipeline Requirements

The embankment dam would be located upstream of the 400 kV transmission line so that the line would not be impacted on by the dam. This site is not optimal for the dam, as indicated by the high construction cost, which is shown in **Section 8.5.3** below.

A summary of the characteristics of the dam is given in **Table 8.4**.

Table 8.4: Summary of characteristics of Nooigedagt South

Characteristic	Nooitgedagt South
Type of dam	Zoned Earthfill Embankment
NOC (m amsl)	89.9
FSL (m amsl)	87.9
Freeboard (m)	2.0
Crest width (m)	7.0
DS slope (1V:H)	2.0
US slope (1V:H)	3.0
Embankment fill volume (m^3)	1,755,715
Core trench volume (m^3)	102,849

Characteristic	Nooitgedagt South
Crest length (m)	1,970
Total gross dam capacity (m^3)	4,600,000
Surface area at FSL (ha)	46.8
Maximum wall height (m)	36.9
Catchment area (km^2)	N/A
Unrouted SEF (m^3/s)	N/A
Spillway configuration description	Unlined 5 to 10 m wide, side channel spillway located on the right abutment. (Note: spillway position dependant on geotechnical conditions)
Outlet works description	Dry well tower (19 m high) with inside dimensions of 4x4m. Three offtake levels controlled by valves with downstream concrete outlet chamber, 4x4x3m, with 2 valves for the two pipes.
Access road length (km)	2.9

The proposed layout of the Nooitgedagt South Dam is given in **Figure 8.4**. The cross-section of the dam was assumed to be similar to that of the proposed Upper Scheepersvlakte Dam (**Section 7.6**), as shown in **Figure 7.5**.

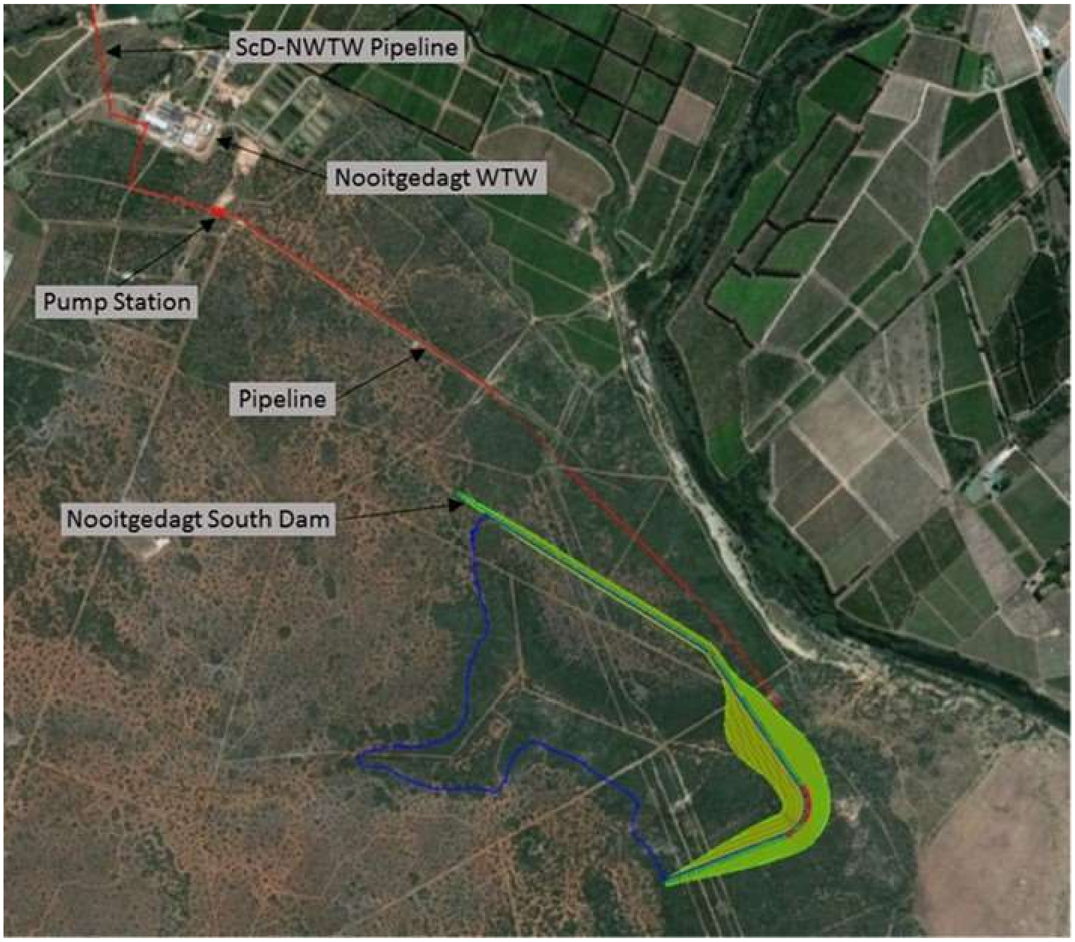

Figure 8.4: Proposed layout of the Nooitgedagt South Dam and related bulk infrastructure

8.5.3 Estimated Capital Costs

The estimated comparative (capital) costs (February 2018 prices, excluding VAT) of the proposed scheme are summarised below.

Nooitgedagt South Dam	R490 million
Pipelines	R34 million
Pump Station	R95 million
TOTAL	**R619 million**

8.5.4 Advantages and Disadvantages

The main advantages of the dam would be that water could gravitate into the dam, and it is the favoured site for the land owner, the Wicklow Trust.

The main disadvantages would be as follows:

- Water would have to be pumped to the Nooitgedagt WTW.
- The capital cost of the dam would be high. This in part arises from the siting of the dam so that construction would not take place below the 400 kV transmission line and the need for a relatively high dam wall, which would provide a relatively small reservoir basin.

9 Comparison of Options and Recommendations

9.1 Comparison of Sub-options

A comparison of the various balancing dam options as presented in this report is provided in **Table 9.1**. The capital costs include estimates for land acquisition, realignment of power lines (where applicable) as well as VAT.

Table 9.1: Comparison of options

	Potential dam sites				
EVALUATION FACTOR	**Upper Scheepers-vlakte**	**Lower Coerney**	**Upper Coerney**	**Nooitgedagt North - Option 1**	**Nooitgedagt South**
Comparative (Capital) cost (R million)	**R349**	**R237**	**R375**	**R457**	**R654**
Capital cost (pumps cost reduced by 50%) (R million)	**R282**	**R231**	**R309**	**R403**	**R600**
Cost	2 - 2nd lowest	1 - Lowest	3 - 3rd lowest	4 - High	5 - Very High
Pumping required	X		X	X	X
Operational complexity	X	X			
Strategic location near WTW				X	X
Ecological considerations (Reserve)		X but likely easy to address	X but likely easy to address		
Consideration of floods		X	X		
Environmental & Social impacts	Limited differentiation	Limited differentiation	Limited differentiation	Limited differentiation	Limited differentiation

When comparing the five options investigated, it must be noted that the balancing dam would not be operated in the same way as normal water resource infrastructure as the water in the dam would only be abstracted in an emergency to supply the Nooitgedagt WTW. The dam would be filled over a certain filling period and would be topped up from time to time to make up evaporation and seepage losses, and possibly also operated to address water quality considerations. Because of this operation, the comparative (capital) cost is more appropriate for comparing schemes, rather than the unit reference value (URV).

9.2 Recommendations

1) Based on the investigation and cost comparison of alternative balancing dam sites presented in this report, the Nooitgedagt sites (North Option 1 and South) should be ruled out and not investigated further. Although these sites are strategically located near the Nooitgedagt WTW, the comparative cost of these options is nearly double that of the lowest cost option (Lower Coerney site). The development of the balancing dam options near the Nooitgedagt WTW can therefore not be justified from a cost point of view.
2) The Lower Coerney site is the preferred site based on the evaluation presented in this report, followed by the Upper Scheepersvlakte site and the Upper Coerney site. The main advantage of the Lower Coerney site, besides having the lowest comparative cost, is that water could be supplied by gravity from the canal to the dam.
3) As stated above, the risk of failure of these options could mostly be mitigated by providing an additional siphon through the Sundays River, as well as managing the process for quick replacement of damaged pipes should this be required.
4) It is therefore recommended that the Lower Coerney and Upper Scheepersvlakte Dam sites be evaluated further in more detail.
5) The topographical survey and geotechnical evaluation of these sites should proceed, to ensure that detailed information for the evaluation of these alternative options is available.
6) Further evaluation of the Upper Coerney site is not recommended as it offers no additional advantage over the other two sites and the comparative cost is the highest of these three options.

10 Environmental Constraints Analysis

10.1 Purpose

The purpose of the Environmental Constraints Analysis was to provide a desktop overview and analysis of the environmental sensitivity of the five short-listed sites for a new balancing dam, highlighting potential issues and constraints and outlining the requisite environmental legal compliance requirements for each option. This provided high-level input regarding the environmental issues/constraints and legal requirements of the five short-listed sub-options.

The detailed analysis has been documented in the *Environmental Constraints Analysis Report* of this study.

10.2 Environmental sensitivity and fatal flaws

From a terrestrial ecology perspective, the Upper Scheepersvlakte and Coerney sites are considered slightly more environmentally sensitive when compared to the Nooitgedagt sites, mostly due to an overlap with an Endangered Ecosystem associated with the Albany Alluvial vegetation group. The vegetation cover associated with the Upper Scheepersvlakte and Coerney sites are also significantly more intact than that of the Nooitgedagt sites. The Coerney sites are further located along a well-defined riparian habitat which is usually associated with higher terrestrial biodiversity as well. No Red List species are known to occur at any of the sites (based on the *International Union for Conservation of Nature* spatial database).

From an aquatic ecology perspective, the Nooitgedagt sites, being located within an Aquatic CBA2 catchment, are technically more sensitive in terms of land use impacts than the Upper Scheepersvlakte and Coerney sites. The critical biodiversity area (CBA)2 classification is, however, linked to the Sundays River estuary and the off-stream balancing dams will have no impact on water quality or quantity supplied to the estuary. There will also be no impoundment or restriction of movement of instream freshwater species. Given the aforementioned, the Coerney sites are in fact considered to have a greater aquatic sensitivity due to the drainage lines within which they are located and thus the potential impact on a functional riparian habitat and sub-catchment hydrology. This is, however, not considered a fatal flaw or notable issue and is merely highlighting the fact that when comparing the proposed sites, the Coerney sites are ranked slightly higher in aquatic sensitivity than the other sites.

No fatal flaws were identified from a heritage and palaeontology as well as land use perspective.

From a purely environmental sensitivity perspective the Nooitgedagt sites are thus slightly preferred to the Upper Scheepersvlakte and Coerney sites. The aforementioned do however, not qualify as “fatal flaws”, but merely something to take note of when evaluating the overall feasibility of the sites.

10.3 Legal compliance and requirements

All sites will require similar authorisations in terms of environmental legislation with the period to complete all applications and processes estimated to take between 300 and 350 days. It should also be noted that application for a Waste Licence (National Environmental Management: Waste Act (No. 59 of 2008, as amended) as well as mining permit (Mineral and Petroleum Resources Development Act No. 28 of 2002, as amended) might be necessary. The exact legal compliance requirements will, however, only be clear once a location and scope of works have been defined in more detail. All the applications can be run concurrently within the 300 to 350-day timeframe mentioned above.

Note that the water use licence application (WULA) and appeals regulations (Government Notice (GN) R267 of 2017) has recently been promulgated, with the published timeframe for a WULA process adding to 300 cumulative days. Both the EIA process and WULA process timeframes also only refer to the regulated timeframes, i.e. once the application has been submitted and does thus not include report writing, undertaking of specialist studies and so forth. It is thus recommended that at least 18 months be allowed in total for environmental processes to be initiated and completed.

10.4 Other factors for consideration

The following is also worth mentioning when considering the feasibility and risks associated with each site.

10.4.1 Coerney sites’ catchment and irrigation

The Coerney sites do have a small catchment of which a notable portion will be transformed to orchards in the near future. This means that the Coerney sites could be subject to irrigation return flows high in nutrients, herbicides and pesticides. Allowance for sufficient buffer distances should thus be considered in order to mitigate potential impacts on water quality.

10.4.2 Scheepersvlakte existing authorisation for smaller dam

Scheepersvlakte 98 Citrus Development Trust has applied for a smaller dam in the same location as the proposed Coerney sites. From an administrative point of view, the Scheepersvlakte 98 Citrus Development Trust will be required to withdraw or surrender the authorisation for the smaller dam in order for the larger dam's EIA to proceed. This will expose the Scheepersvlakte 98 Citrus Development Trust to a certain level of risk as they will lose the security of a smaller dam, which has already been approved.

11 Geotechnical Survey

Following on the decision that the Lower Coerney and Upper Scheepersvlakte Dam sites were favoured, and should be investigated further, the geotechnical investigations were scoped in detail and specialist sub-contractors appointed.

11.1 Methodology

These geotechnical investigations at the respective sites included the following elements:

- Geophysical (resistivity) surveys, by specialist geophysicists; Engineering & Exploration Geophysical Surveys cc (EEGS). The thick bush necessitated clearing of cut-lines before these geophysical traverses could be conducted. A service provider; BK Bush Clearing, was appointed for this task.
- Test pitting, using a light tractor loader-backhoe.
- Rotary core drilling, by a specialist geotechnical drilling contractor; RWBE Geotechnical Drilling.
- Field testing including SPTs and packer (Lugeon) testing, conducted as part of the drilling contract.
- Laboratory testing on representative samples conducted by Tosca Lab in Port Elizabeth.

Borehole positions were surveyed on completion by DWS Survey Services.

11.2 Regional Geology

The general geology comprises thin grey sandstones, siltstones and mudrocks of the Sundays River Formation of the Uitenhage Group, part of a collection of sedimentary strata within the structurally controlled Algoa Basin. The seismic hazard of the area is considered to be very low and the Peak Ground Acceleration (PGA) values are less than 0.02g, with a 10% probability of being exceeded in a 50-year period.

11.3 Dam Sites' Geology

Both dam sites are characterised by gentle slopes, including the slopes defining the respective basins. The geological profiles for the respective sites are summarised below (**Table 11.1**). Both sites are characterised by soil cover of variable origin and thickness, overlying weak rocks that are characterised by extensive and pervasive weathering.

Table 11.1: Geological profile summary

Reference area	Lower Coerney	Upper Scheepersvlakte
Left flank	Soils to 7.2m, including horizon of gravelly soils between 4m and 7,2m; very soft rock mudstone, subordinate sandstone from 7.2m.	Soil horizons to depth 0.8m; thereafter very soft rock sandstone / dense residual soils to 5 m; very soft to soft rock sandstone / interbedded mudstone from 5 m to 11.2m; from 11.2 m medium hard rock sandstone
River section / central section	At the heel; sandy soil to 2.65m; gravelly soils to 7.7m; soft to very soft rock (occasionally weathered to clay) mudstone from 7.7m; medium hard to hard rock interbedded mudstone / sandstone from 9.8m. At the toe; sandy soils to 1.3m; gravel-sand horizon to 4m; very soft to soft rock sandstone from 4m; soft to medium hard rock sandstone, interbedded mudstone from 4.6m; hard rock sandstone from 12m.	At the heel; topsoil to 0.35m; sandy soils with some gravels to 11.1m; soft rock sandstone from 11.1m; medium hard rock sandstone from 11.5m At the toe; soils to 7.7m, including some gravels in places; soft to very soft rock (to clay in places) alternating sandstone / mudstone from 7.7m, becoming soft rock / medium hard rock from 10.1m.
Right flank	Topsoil to 0.8m; gravelly horizon to 2.7m; highly weathered, medium hard to soft rock from 2.7 m. Interbedded sandstones, mudstones.	Soils to 3.5m; with a gravel horizon to 5 m; bedrock from 5m, including very soft rock mudstone to stiff clay to 5.65m, very soft to soft rock interbedded mudstone from 5.65m
Spillway alignment	Upper spillway (near ogee / sill); soils to 4 m; gravelly soil horizon to 7.2 m; very soft / soft rock (mainly mudstone, subordinate sandstone) from 7.2 m. Lower (actually mid-) spillway; soils to 5.45 m; gravelly soils to 6.7 m; very soft rock sandstone	Upper spillway (near ogee / sill); topsoil to 0.35m; very soft rock / medium hard rock at 1.2 m. Sandstones and interbedded mudstones. Lower spillway; Soils to 3.35 m; very soft rock from 3.35 m. Sandstone with interbedded

Reference area	Lower Coerney	Upper Scheepersvlakte
	(sand in places) from 6.7 m; interbedded sandstone / mudstone from 8 m.	mudstone. Conditions are termination of spillway are still to be confirmed.
Reservoir basin	Soil cover attains thicknesses in excess of 2m. Soil strata include topsoil, colluvium (with or without pedogenic influence) as well as reworked terrace gravels comprising gravels / cobbles in a sand matrix.	Soils as on the dam footprint, comprising topsoil, colluvium (with or without pedogenic influence), overlying weathered bedrock. Soil thicknesses expected between 1m and 2.4m.

11.4 Geotechnical Considerations

11.4.1 Suitable dam type

Considering the site topography, as well as the founding geology, both sites are only suitable for an embankment structure; specifically, an earthfill embankment. A rockfill embankment would not be feasible considering the absence of local sources of suitable rock.

Current layouts allow for a bywash spillway on the left flanks. The soil cover as well as the underlying weathered, weak bedrock will be erodible, and it will be necessary to allow for full concrete lining of the spillway chute.

11.4.2 Excavation depths

Expected excavation depths for the embankment cut-off are summarised below (**Table 11.2**) for the respective dam sites. These depths are based on the general principle of excavating the cut-off to the base of the gravel-sand soils; i.e. to bedrock. Some excavation of the upper, weaker bedrock horizons is allowed for in some cases.

Table 11.2: Summarised excavation depths for the cut-off trench

Reference section	Lower Coerney	Upper Scheepersvlakte
Left flank	8m	5m (could be as shallow as 1.5m but for sake of consistency and uniformity the depths should be considered as 5m)
Central portion / river section	5.5m	8m, but in places the bedrock is up to 11.5m deep, and deeper foundation treatment (to 11.5m) might be necessary
Right flank	3.5m	5m (could be 3.5m)

11.4.3 Foundation permeability

The presence of the buried gravel–sand horizon at depth is a prime consideration in terms of foundation permeability and potential for seepage. The approach of ensuring the cut-off trench is taken to the base of this stratum is considered an appropriate strategy for dealing with this risk.

Water pressure (packer or Lugeon) tests, within the rock strata, showed that the rock mass is generally impermeable. A number of tests (at both sites) yielded significant losses. In addition, these water losses are associated with ‘wash out’ of the weaker, weathered material. It is likely that this is indicative of the potential for erosion damage to the weak bedrock under conditions of seepage and high hydraulic gradients.

11.4.4 Construction materials

It is understood that a zoned earthfill embankment is favoured at this stage. Limited investigations were conducted in the respective dam basins to characterise these local materials and assess their suitability for use in the embankment. The findings ae summarised below (**Table 11.3**). Some materials would not be available locally and will have to be sourced from further afield.

Table 11.3: Earthfill embankment construction material availability

Material type	Lower Coerney	Upper Scheepersvlakte
Impervious core material	The range of material types present show wide scatter; there is some broad compliance in parts, but also non-compliance with typical specifications	Local colluvial soils, with or without pedogenic origin, are generally compliant with typical specifications
Outer shell zones (semi-pervious materials)	The range of material types present show wide scatter; there is some broad compliance in parts, but also non-compliance with typical specifications	The same materials as above show some compliance in terms of grading, but generally do not comply with typical specifications
Concrete aggregates, and rock for rip-rap	Not available locally. Would have to be obtained from commercial sources.	
Filter sands	Not available locally. Would have to be sourced from commercial sources further afield. Other options like blending may be considered.	

11.4.5 Way forward

Current geotechnical investigations have been conducted at feasibility level specifically to provide inputs into preliminary design, and aid site selection. Investigations to date were constrained by limited access due to the dense bush, and environmental restrictions on bush-clearing.

Further geotechnical investigations would be required for detail design purposes at the favoured site. Specific aspects to be addressed would include;

- Further confirmation of the geological profile, in particular along the cut-off trench and the intake / outlet conduit. More detailed knowledge of the lateral and vertical continuity of the various soil and rock strata would allow better definition of the cut-off requirements.
- Confirmation of the founding conditions at the end of the spillway to aid appropriate design of the terminal structure.
- The availability, preferably within the basin area, of materials suitable for embankment construction; in particular, impervious core material and semi-pervious material for the outer shell zones. This must be a prime focus of further investigations and could have a large impact on final embankment design.

Such investigations on the dam and spillway footprint would include additional rotary core drilling, and test pitting. Deep trenching might be considered.

Within the basin and surrounds, an intensive test pitting programme using a TLB is required to delineate borrow areas. The fieldwork must be complimented by a comprehensive laboratory testing programme which must also allow for control testing by 3rd party laboratories.

12 Topographical Survey

12.1 Introduction

A topographical survey was completed by Survey Services: Southern Operations (National Water Resource Infrastructure) of the DWS for Lower Coerney and Upper Scheepersvlakte Dams in May 2018. The results are reported in the survey reports; Upper Scheepersvlakte Dam, Contour Survey (EC004/2018) and Coerney Dam Contour Survey (EC 003/2018).

12.2 Approach

From the report for the Upper Scheepersvlakte Dam; Contours from existing 1 m contour plans from 1977 and 1984, which were compiled from aerial photography for the design of the Lower Sundays River Government Water Scheme, was regenerated up to the 125 m contour value. For the Lower Coerney Dam contours were similarly generated from existing 1 m contour plans from 1977 up to the 110 m contour value.

The reports note that more than 75% of the dam basin is covered in dense bush, which made it impossible to use ground-based survey methods to do a topographical survey by foot. The method of digitising contours from historic 1 m contour maps was the only alternative option available at the time. Alternatively, light detection and ranging (LIDAR) was another option, but with no guarantee that it will provide any better result, due to the vegetation, and at a greater cost.

Briefly, the method used was to georeference the survey images, which are then converted to PDF, imported to modelling/CAD software and digitised. The contours were regenerated by digitising points on full contours and creating a triangle model to regenerate contours.

Two test sections were surveyed in the field for the Upper Scheepersvlakte site and nine for the Lower Coerney site, to compare and verify the digitised data to the actual ground data, which resulted in a good match. The rest of the Upper Scheepersvlakte site, up to the 132 m contour, was surveyed with GPS-RTK systems. Datasets were combined and final contours with 1 m intervals were generated.

Then in August 2018 the survey was updated and expanded to include the immediate surrounding infrastructure, which is reported on in the Scheepersvlakte Contour and Detail Survey Report (EC026/2018).

Figure 12.1 shows an excerpt of the combined surveys of the two dam sites that were selected for surveying.

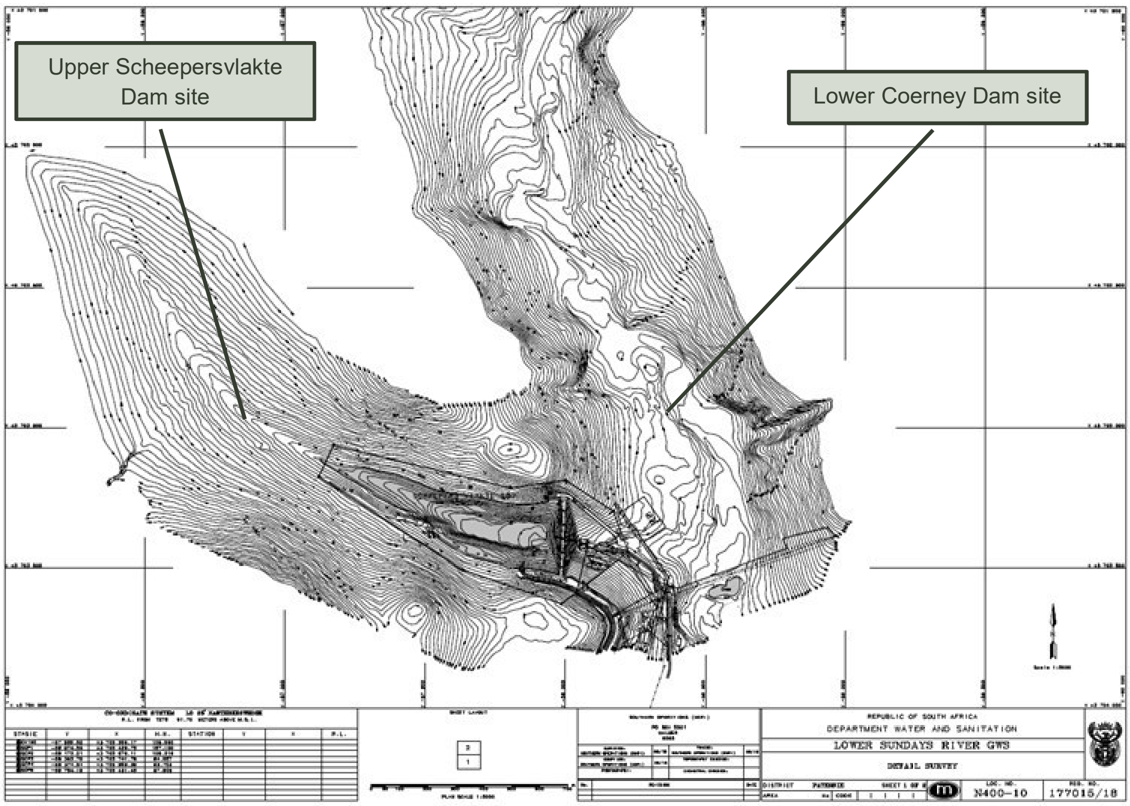

Figure 12.1: Excerpt of the combined surveys of the two dam sites completed in August 2018

13 Design Flood Analysis

13.1 Introduction

The design flood peaks for various recurrence intervals were estimated for the Lower Coerney and Upper Scheepersvlakte Dam sites. **Figure 13.1** shows the two sites and their catchment areas in relation to the existing Scheepersvlakte Dam.

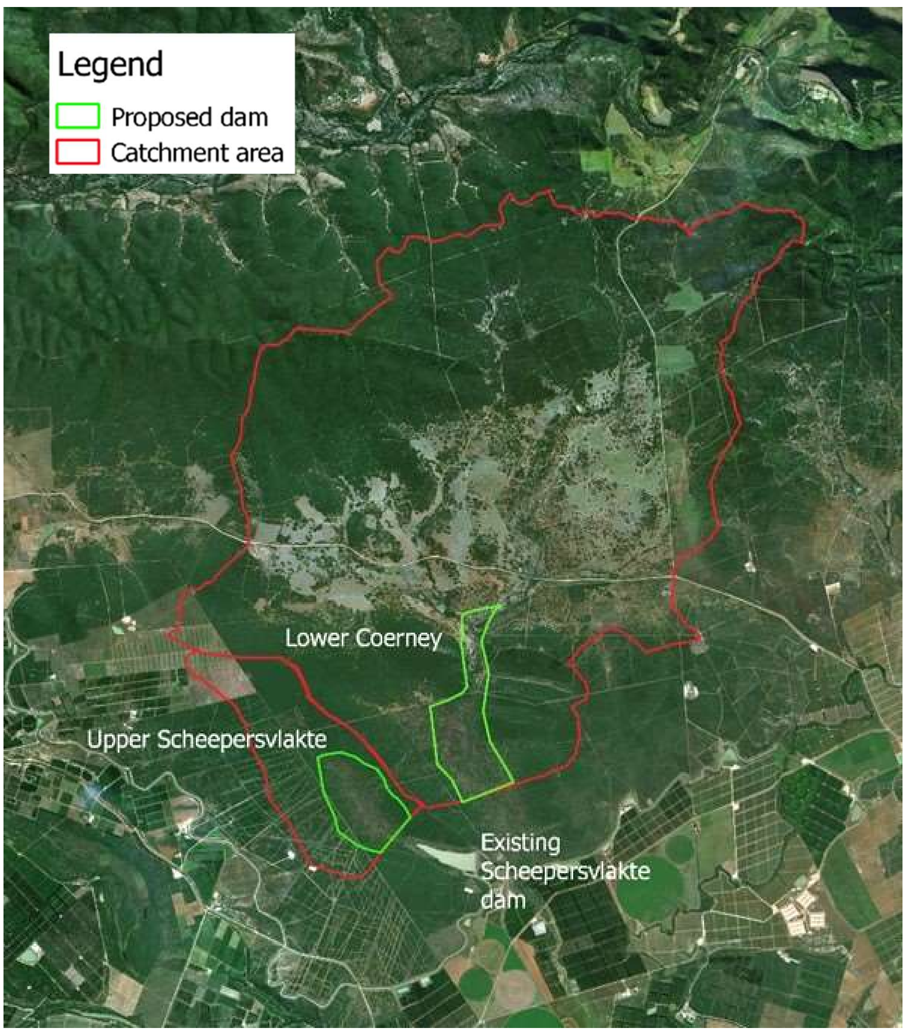

Figure 13.1: Proposed Lower Coerney and Upper Scheepersvlakte Dam sites

13.2 Scope and Objectives

The scope of work for this component of the Study encompassed the following sequential phases:

- Determining design rainfall for the Study catchments
- Determining physiographic catchment characteristics
- Undertaking appropriate deterministic design flood analyses for determination of design flood peaks for recurrence intervals (RIs) of 1:2, 1:5, 1:10, 1:20, 1:50 1:100 and 1:200 years, as well as the Probable Maximum Flood (PMF).

13.3 Approach and Methodology

Based on the size of the Study catchments and the lack of streamflow records in the Study catchments, it was decided to follow only a deterministic approach for the determination of the design floods.

Conventionally, under the deterministic category, three alternatives for design flood analyses are followed, namely the Unit Hydrograph, US Soil Conservation Services (SCS) and Rational Method-approaches. The non-availability of sub-daily rainfall records, as well as the non-availability of sub-daily streamflow records in or near the catchments negated the derivation of Unit Hydrographs. Hence, the SCS and Rational Method-approaches for design flood determination were employed.

13.4 Design Rainfall

Deterministic design flood methods require daily or sub-daily design rainfall as input. The Water Research Commission (WRC) Report by Smithers and Schulze (2000) was thus used. The station nearest to the two Study catchments is South African Weather Bureau (SAWB) number 0055655 W (Twembani). The location of the station relative to the catchments is shown in **Figure 13.2**.

Areal Reduction Factors (ARFs) of 96.7% and 100% were applied to the Lower Coerney and Upper Scheepersvlakte Study catchments respectively, based on the ARF relationships for South Africa developed by Alexander (1990).

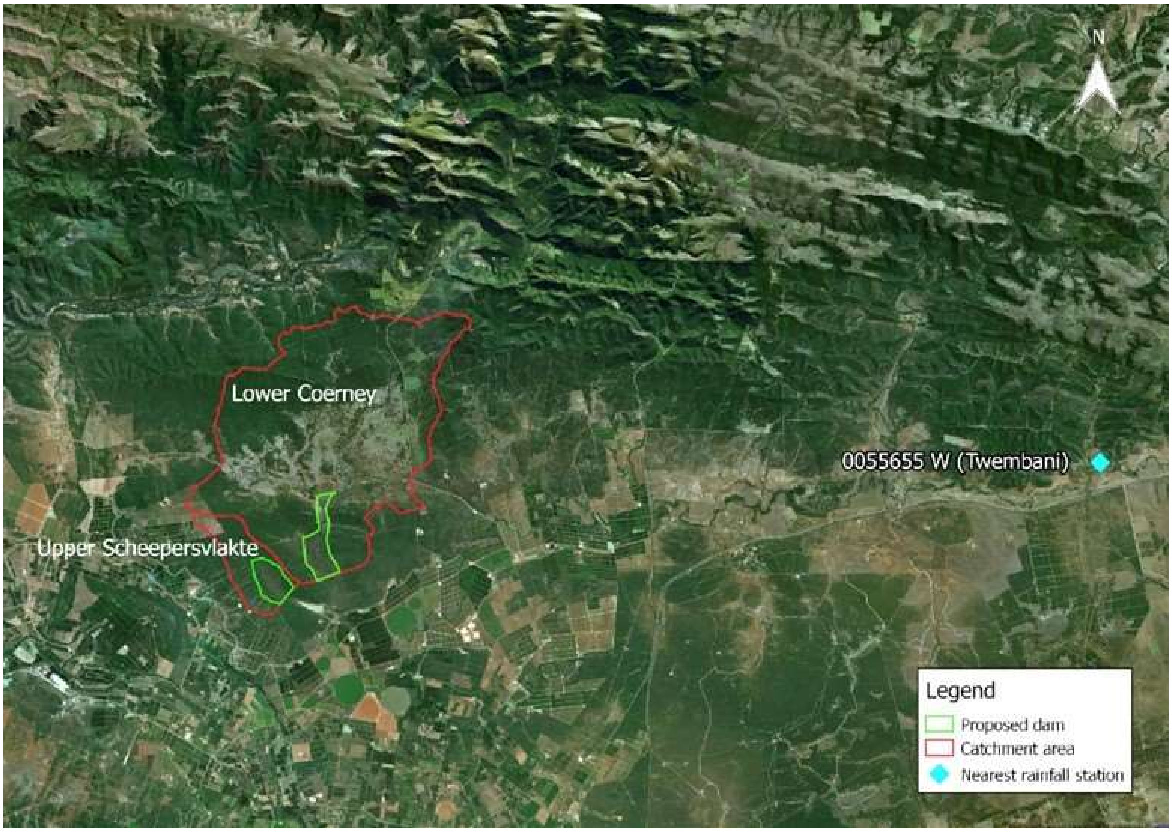

Figure 13.2: Nearest rainfall station to Study catchments

13.5 Deterministic Design Flood Determination

13.5.1 Catchment Characteristics

Relevant parameters describing the physiographic catchment characteristics that are required for design flood calculations were derived from the 30 m x 30 m NASA Shuttle Radar Topographic Mission (SRTM) digital elevation model (DEM) for the Study area. Key catchment parameters for the catchment upstream of the dam sites are presented in **Table** 13.1. The characteristics of the soil were based on the Soil and Terrain Database for Southern Africa (SOTERSAF, 2003).

Table 13.1: Catchment characteristics upstream of the dam sites

Characteristic	Quantum	
	Lower Coerney	Upper Scheepersvlakte
Area	33.6 km^2	3.4 km^2
Length of longest watercourse	9.83 km	3.67 km
Equal-Area Slope of longest watercourse	0.0148 m/m	0.0222 m/m
Average catchment slope	6.55 %	3.27 %

13.5.2 Rational Method

The Rational Method yields a design flood peak only (i.e. no flood hydrograph). The results of the Rational Method calculations at the Lower Coerney and Upper Scheepersvlakte Dam sites are summarised in **Table 13.2**.

Table 13.2: Rational Method – design flood peaks

Recurrence Interval (y)	Flood Peak (m^3/s)	
	Lower Coerney	Upper Scheepersvlakte
1:2	16	3
1:5	26	4
1:10	35	6
1:20	48	8
1:50	74	13
1:100	105	18
1:200	124	21
PMF	869	167

13.5.3 SCS Method

The SCS method estimates peak discharges and flood volumes based on the catchment soil retention (S), lag time (T_L), hydrological soil group and the Curve Number (CN).

The physical catchment characteristics were used in the SCS utility included in the HEC-HMS modelling package (U.S. Army Corps of Engineers, 2015) to calculate the RI flood peaks. The resultant SCS flood peak estimates are presented in **Table 13.3**.

Table 13.3: SCS method design flood peaks

Recurrence Interval (y)	Design Flood Peak (m^3/s)	
	Lower Coerney	Upper Scheepersvlakte
1:2	10	1
1:5	27	4
1:10	43	6
1:20	63	9
1:50	95	14
1:100	125	18
1:200	161	24
PMF	801	115

13.6 Recommended Design Flood Peaks

The foregoing Rational Method-based and SCS Method-based RI-flood peak estimates are subjected to similar types of uncertainties. It is therefore, recommend that the design flood peaks for the Lower Coerney and Upper Scheepersvlakte Dam sites be based on the averages of the above two sets of estimates, as presented in **Table 13.4**.

Table 13.4: Recommended design flood peaks at the dam sites

Recurrence Interval (y)	Design flood according to SANCOLD recommendations	Recommended Design Flood Peaks (m^3/s)	
		Lower Coerney	Upper Scheepersvlakte
1:2		13	2
1:5		27	4
1:10		39	6
1:20		56	9
1:50		85	14
1:100		115	18
1:200	**Recommended Design Flood (RDF)**	**143**	**23**
PMF	**Safety Evaluation Flood (SEF)**	**835**	**141**

Based on the height, storage capacity and expected hazard potential downstream of the dams the expected classification for both dams is Category 3. By that standard, the SANCOLD Guidelines in Relation to Floods (SANCOLD, 1991) recommend that the Recommended Design Flood (RDF) be equal to the 1:200 year flood (annual exceedance probability of 0.5%)

and the Safety Evaluation Flood (SEF) equal to the Probable Maximum Flood (PMF) as highlighted in **Table 13.4**.

14 Groundwater evaluation

14.1 Background/Brief

The core drilling at the Lower Coerney Dam centreline indicated the occurrence of a gravel layer – paleo-channel at this site. The mapping of standing water levels in the boreholes also raised some concerns on potential groundwater impacts on the design of the dam.

The small tributary of the Coerney River, in which the Lower Coerney Dam will be situated, is an ephemeral river, with no obvious river channel and it is currently pretty much overgrown. The geology indicates that the groundwater ecological water requirement (EWR) (also referred to as the Reserve) component may be important to consider. While the undertaking of an ecological Reserve is outside the scope of this study, there is a need to consider the most important aspects to be taken into consideration in the feasibility design. It will be preferable that an EWR for non-perennial systems be determined to take into account in the feasibility-level design. The assumption is in any case that any flows generated in the catchment will be passed through the dam.

A cut-off trench will be constructed to restrict seepage from the Lower Coerney Dam, but there is a need to have a better understanding of how the groundwater flows, for any design implications.

In particular, because of the occurrence of the paleo-channel, there is a need to understand the direction of flow of the groundwater and particulars about the expected rate of flow, to take its influence on the planned dam wall into account.

Dr Ricky Murray of Groundwater Africa was asked to evaluate the situation and to provide a professional opinion of the groundwater situation. This did not include a site visit.

14.2 Groundwater concerns in the Lower Coerney Dam site area

Dr Murray thus undertook a brief assessment of the groundwater situation in the Lower Coerney Dam site area, to address the following concerns about the shallow alluvial gravels that were encountered during core drilling at the proposed dam site.

The issues raised are:

- Groundwater flow direction,
- Groundwater flow rate, and
- Potential groundwater effect on the planned dam.

The discussion below presents what can be deduced from the available data supplied. This consists of the locations of the core boreholes (**Figure 14.1**), the geological logs of the core holes and groundwater levels (**Table 14.1**).

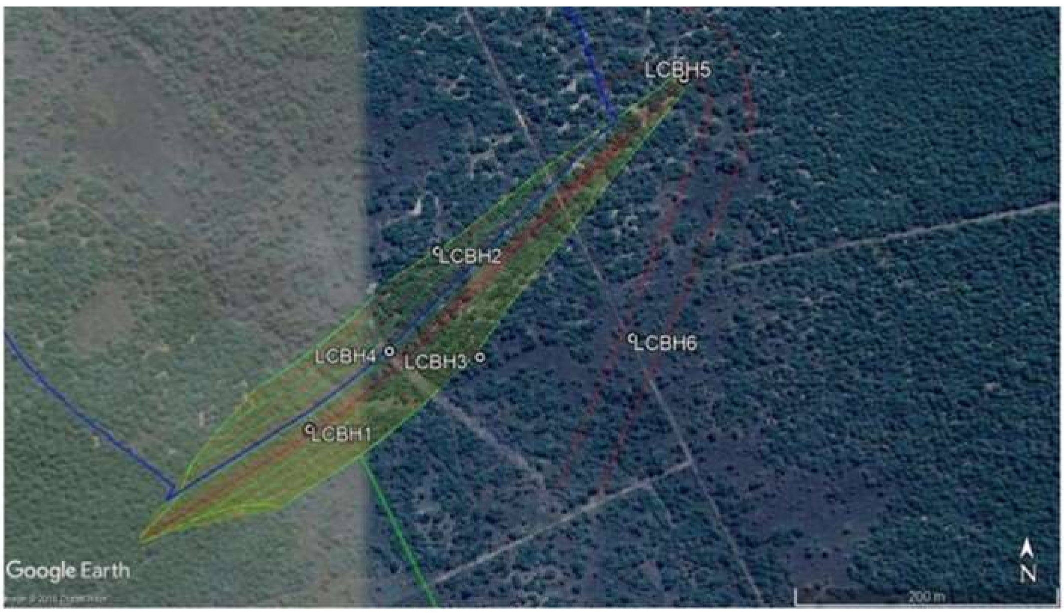

Figure 14.1: Borehole locations

14.3 Findings

Natural groundwater levels appear to mirror topography to produce a groundwater flow direction downstream in a roughly southerly direction (**Figure 14.2**). The hydraulic gradient is steep, around 0.03 – 0.05 (**Table 14.1**) which shows that the permeability of the saturated rocks is very low, as one would expect from the Kirkwood Formation mudstones, siltstones and sandstones. Even with the steep hydraulic gradients, the flow rates will be very low.

Table 14.1: Rest water levels (RWL) and approximate hydraulic gradients

BH #	Collar elevation (mamsl)	RWL (mbgl)	RWL (mamsl)	Gradient BHs	Appr. Distance (m)	Difference in RWL (m)	Hydraulic gradient
LC2	83.36	13.75	69.61				
LC2	89.15	9.6	69.55				

BH #	Collar elevation (mamsl)	RWL (mbgl)	RWL (mamsl)	Gradient BHs	Appr. Distance (m)	Difference in RWL (m)	Hydraulic gradient
LC3	84.30	18.1	66.20	LC2-LC3	100	3.35	0.034
LC4	81.82	12.7	69.12				
LC5	102.01	9.2	92.81				
LC6	89.98	8.8	81.18	LC5-LC6	220	11.63	0.053

Figure 14.2: Rough groundwater flow direction

14.4 Perched groundwater flow rate after dam construction

The groundwater table lies below the alluvial gravels. However, after constructing the dam, water can be expected to leak through the upper, near-surface layers and saturate the gravel layer. The leakage may be slow due to the presence of clayey material in places, and with time it may reduce, as additional clayey and silty material accumulates on the bottom of the dam. The hydraulic gradient, however, will be high and if the gravels are highly permeable, water will be able to flow relatively rapidly in this layer. The flow rate through the gravels, however, may not be a function of the permeability of the gravels, but rather of the leakage rate through the base of the dam, as this latter flow rate may be less than that of the gravels themselves. This is obviously unknown.

The maximum flow rate, i.e. the potential flow through the gravels, can be estimated once the hydraulic conductivity or transmissivity of the gravels are known. This can be obtained by conducting injection tests on the core boreholes if they are still sufficiently open (they were not back-filled but may be blocked with debris), or alternatively new boreholes can be drilled for testing purposes. The results of the permeability tests done on the bedrock are obviously not suitable to be used to estimate the gravels' permeability.

It is likely that leakage via the base of the dam and through the gravels will not daylight as new springs downstream of the dam wall as it appears as if the vegetation is sufficiently dense to opportunistically utilise this shallow water – water that would naturally be in this zone during heavy rainfall periods. A botanist should be consulted to comment on this.

14.5 Potential effect on natural groundwater flow

The leakage to the gravels and the underlying hard-rock geology would only produce a very limited impact on the hydrogeology of the area. The underlying hard-rock's permeability is probably too low to receive much water, and therefore the effect of the dam will likely be localised and small. The gravels have been discussed above, but the net effect on these will likely also be small because they are unlikely to be continuous for a great distance, and even if they are, it is unlikely that they will be highly permeable throughout their length. This, however, is not known, but 2D resistivity surveys can assist in mapping the gravel layer.

14.6 Potential groundwater effect on the planned dam

As stated above, the gravels will likely become permanently saturated below the dam and below the dam wall.

14.7 Findings

The evaluation undertaken:

- Confirms that groundwater flow direction is downstream.
- The gravel layer will become saturated but should not have a large impact on groundwater flow.

- The current groundwater movement is below the gravel layer and will not be affected by the proposed dam wall.

The core trench for the dam needs to be founded on the material below the gravel layer, which will intercept groundwater flow through the gravels. This will not impact on the current groundwater movement, which is below the gravel layer.

The dam wall can be founded on the gravel layer as this will be taken into account during the stability analysis. Although gravel has no cohesion, it has a higher internal angle of friction than clay.

15 Refined dam designs and costs

Following the completion of the topographical survey and geotechnical evaluation at the Lower Coerney and Upper Scheepersvlakte Dam sites, as well as the updated flood hydrology for these sites, the designs for these dams and the associated costs have been updated, as described below.

15.1 Refined Design and Costing for Lower Coerney Dam

15.1.1 Impacts of the updated topographical survey

The updated topographic data of the site has shown that the 5 m contours used in the costing for the sub-report tiled *Desktop Assessment of Short Listed Options Report*, dated 19 April 2018, are comparable to the more accurate 1 m contours used in the current evaluation. The FSL and NOC are within 1 m. However, the minimum basin level differs by a more substantial margin of 3.3 m.

The updated characteristics of the proposed dam, comparing levels and dimensions between the initial screening and current studies can be seen in **Table 15.1**.

15.1.2 Impacts of the updated geotechnical information

The geotechnical investigations, as discussed in **Section 11**, affected the proposed initial design of the dam. Most notably are the deep foundations, both for founding of the cut-off core trench, as well as the spillway and outlet structures. It was also shown that clay (which would be required for an impermeable core zone) is generally absent from the site. Lastly, the extensive and pervasive weathering of the foundation rock necessitates a protected spillway and discharge channel.

The foundation depth of the core trench varies between 3.5 m and 8.0 m, moving from the right to the left abutments respectively, while the founding depth in the central river section is approximately 5.5 m. Thereafter the foundation depth increases up the left abutment to the maximum depth at the location of the proposed spillway where it is 8.0 m deep.

Currently, geotechnical data and investigations for the spillway are targeted at the proposed site on the left abutment. The deep foundations found at this site suggests that further investigations into spillway founding conditions on the right abutment should be undertaken to determine the best site for the spillway. The lack of durable hard rock found in any of the boreholes, along with the magnitudes of the floods, necessitates a reinforced concrete lined spillway, regardless of the final spillway location.

The embankment is currently designed as a zoned earthfill embankment with a central core zone, general fill shells upstream and downstream of this. In general, the materials found at the site are not ideal for the impervious core zone, due to the practically absent clay fraction and low plasticity indices (PI's) of the soils tested. The permeability tests on the samples show mostly semi-pervious soils, with single results showing semi- to impervious characteristics. Provision has thus been made in the construction cost estimate for importing 70% of the core material, within a 5 km radius outside the basin. Suitable sources for these embankment zones must still be determined.

The selected test samples show at least an intermediate level of dispersivity. A filter system was therefore allowed for, consisting of a chimney drain downstream of the core zone, which connects to sand finger drains and finally a gravel and rock toe. A full blanket drain was not used due to the apparent lack of suitable quality and quantity of filter sand. Provision is made in the construction cost estimate for sourcing the sand and gravel filter zones from commercial sources.

To accommodate the design floods, safety evaluation flood (SEF) and recommended design flood (RDF) (see **Table 15.1**), in the topographical constraints on site, a side channel ogee shaped overflow weir is proposed. The use of a side channel arrangement will provide sufficient overflow length and freeboard while limiting the lining and excavation requirements of the discharge channel. The proposed overflow is a 7 m high 35 m wide concrete gravity overflow structure, which flows to a discharge channel down the left abutment back to the river channel. The spillway is designed to accommodate the SEF (i.e. unrouted incoming flow peak) of 835 m^3/s, using the full 4.8 m freeboard provided.

The depth and extent of weathering of the foundation rock on the left abutment further impacts on the spillway design. Deep excavations to suitable foundations and erosion protection/lining of the spillway and discharge channel are envisaged. The trapezoidal discharge channel is 5 m wide at the base with side slopes of 1V:0.5H. The channel is founded at an approximate average depth of 9 m, of which 4.5 m is lined with reinforced concrete, which can accommodate the SEF plus freeboard. The channel will terminate in an energy dissipating structure at the river channel as the soils are expected to be prone to erosion, particularly considering the water energy, which will need to be dissipated. This structure has not yet been designed, however, the cost estimate allows for a simple stilling basin.

15.1.3 Impacts of the updated flood hydrology

The SEF peak flow has slightly decreased in magnitude from 890 m^3/s to 835 m^3/s (6%) (the RDF was not calculated during the preliminary options identification phase).

The updated flood peaks can be compared with the peaks used in the previous evaluation in **Table 15.1**. The design of the spillway is thus not drastically impacted on by the flood magnitude. Rather, as discussed above, the depths and lining requirements of the spillway are the major refinements.

The effect of flood routing was not taken into account for the current level of study. Due to the size of the basin in relation to the catchment, routing could reduce the size of the spillway required and it is proposed that this be quantified in further investigations.

15.1.4 Summary of the refined Lower Coerney Dam characteristics

The dam characteristics produced during the preliminary options identification and screening can be compared to the current revised and refined options comparison in **Table 15.1** below.

Table 15.1: Comparison of dam characteristics for Lower Coerney Dam

Characteristic	Initial options design	Refined design
Type of dam	Zoned Earthfill Embankment	Zoned Earthfill Embankment
NOC (m amsl)	103.8	102.9
FSL (m amsl)	98.8	98.1
Freeboard (m)	5.0	4.8
Crest width (m)	5.0	5.0
DS slope (1V:H)	2.0	2.0
US slope (1V:H)	3.0	3.0
Embankment fill volume (m^3)	355,993	378,600
Core trench volume (m^3)	46,798	52,200
Max core trench depth	7.0	8.0
Min core trench depth	4.0	3.5
Average core trench depth	5.0	5.7

Characteristic	Initial options design	Refined design
Crest length (m)	623	420
Total gross dam capacity (m^3)	4,600,000	4,600,000
Surface area at FSL (ha)	59.7	71.1
Minimum basin level	84.8	81.5
Maximum wall height (m)	19.0	21.4
Catchment area (km^2)	34	34
Unrouted RDF (m^3/s)	-	143
Unrouted SEF (m^3/s)	890	835
Spillway configuration description	36 m wide side channel spillway on left abutment, with 2.5 m high ogee overflow structure. Trapezoidal discharge channel with 15 m base width, 6.8 m deep and 1V:0.5H side slopes. Lined with reinforced concrete 190 m long 6.8 m deep with a flip bucket/energy dissipater at end.	35 m wide side channel spillway on left abutment, with 7 m high ogee overflow structure. Trapezoidal discharge channel with 5 m base width, 9 m deep and 1V:0.5H side slopes. Lined with reinforced concrete 300 m long 4.5 m deep with a flip bucket/energy dissipater at end.
Outlet works description	Dry well tower (19 m high) with inside dimensions of 4x4m. Three offtake levels controlled by valves.	Dry well tower (21.4 m high) with inside dimensions of 4x4m. Three offtake levels controlled by valves.
Access road length (km)	1.0	1.0

15.1.5 Updated capital cost estimate

The estimated comparative (capital) costs (February 2018 prices, excluding VAT) of the proposed scheme are summarised below. A more detailed costing is provided in **Appendix A**.

Lower Coerney Dam R 174.6 million

Land acquisition R 18.8 million

Connecting pipework R 48.6 million

Power supply R 9.7 million

TOTAL **R 251.7 million**

No pumping would be required.

15.2 Refined Design and Costing for Upper Scheepersvlakte Dam

15.2.1 Impacts of the updated topographical survey

The updated topographic data of the site has shown that the 5 m contours used in the costing for the sub-report titled Desktop Assessment of Short Listed Options Report, dated 19 April 2018. are comparable to the more accurate 1 m contours used in the current evaluation. The FSL and minimum basin levels are within 0.5 m, which shows a good match of contours.

The updated characteristics of the proposed dam comparing levels and dimensions between the initial screening and current studies can be seen in **Table 15.2.**

15.2.2 Impacts of the updated geotechnical information

The geotechnical investigations as discussed in **Section 11** affected the proposed initial design of the dam. The Upper Scheepersvlakte Dam is affected much the same as the Lower Coerney Dam by the geotechnical conditions found on site.

The core trench foundation depth varies between 3.5 m and 8.0 m, moving from the right to the left abutments respectively, while the founding depth in the central river section is approximately 8.0 m. The foundation depth continues at this depth up the left abutment.

The embankment is currently designed as a zoned earthfill embankment with a central core zone, general fill shells upstream and downstream of this. In general, the materials found at the Upper Scheepersvlakte site are also not ideal for the impervious core zone, due to the practically absent clay fraction and low PIs of the soils tested. However, they appear to be generally more compliant to standard specifications. Provision has thus been made in the construction cost estimate for importing 70% of the core material, within a 5 km radius outside the basin. Suitable sources for these embankment zones must still be determined.

The selected test samples show at least an intermediate level of dispersivity. A filter system was therefore allowed for, consisting of a chimney drain downstream of the core zone, which connects to sand finger drains and finally a gravel and rock toe. A full blanket drain was not used due to the apparent lack of suitable quality and quantity filter sand. Provision is made in the construction cost estimate for sourcing the sand and gravel filter zones from commercial sources.

To accommodate the design floods, SEF and RDF (see **Table 15.1**), in the topographical constraints on site, a side channel ogee shaped overflow weir is proposed. The use of a side channel arrangement will provide sufficient overflow length and freeboard while limiting the lining and excavation requirements of the discharge channel. The proposed overflow is a 5.5 m high 10 m wide concrete gravity overflow structure, which flows to a discharge channel down the left abutment back to the river channel. The spillway is designed to accommodate the SEF (i.e. unrouted incoming flow peak) of 141 m^3/s, using the full 3.5 m freeboard provided.

The trapezoidal discharge channel is 3 m wide at the base with side slopes of 1V:0.5H. The channel is founded at an approximate average depth of 4 m, of which 1.5 m is lined with reinforced concrete, which can accommodate the SEF plus freeboard. The channel will terminate in an energy dissipating structure at the river channel as the soils are expected to be prone to erosion particularly considering the water energy, which will need to be dissipated. This structure has not yet been designed, however, the cost estimate allows for a simple stilling basin.

15.2.3 Impacts of the updated flood hydrology

The SEF peak flow has considerably decreased in magnitude from 220 m^3/s to 141 m^3/s (40%) (the RDF was not calculated during the preliminary options identification phase).

The updated flood peaks can be compared with the peaks used in the previous evaluation in **Table 15.2**.

The effect of flood routing was not taken into account for the current level of study. The catchment is small and there could be optimisation, which can be done with regards to the spillway design and attenuation of the incoming flood peak. This should be considered in further investigations.

15.2.4 Summary of the refined Upper Scheepersvlakte Dam characteristics

The dam characteristics produced during the preliminary options identification and screening can be compared to the current revised and refined options comparison in **Table 15.2** below.

Table 15.2: Comparison of dam characteristics for Upper Scheepersvlakte Dam

Characteristic	Initial options design	Refined design
Type of dam	Zoned Earthfill Embankment	Zoned Earthfill Embankment
NOC (m amsl)	130.3	131.8
FSL (m amsl)	127.8	128.3
Freeboard (m)	2.5	3.5

Characteristic	Initial options design	Refined design
Crest width (m)	5.0	5.5
DS slope (1V:H)	2.0	2.0
US slope (1V:H)	3.0	3.0
Embankment fill volume (m^3)	373,740	463,600
Core trench volume (m^3)	36,490	51,950
Max core trench depth	7.0	8.0
Min core trench depth	4.0	5.0
Average core trench depth	5.0	5.6
Crest length (m)	524	550
Total gross dam capacity (m^3)	4,600,000	4,600,000
Surface area at FSL (ha)	58.9	63.2
Minimum basin level	105	104.4
Maximum wall height (m)	25.3	27.4
Catchment area (km^2)	3.5	3.4
Unrouted RDF (m^3/s)	-	23
Unrouted SEF (m^3/s)	220	141
Spillway configuration description	10m wide side channel spillway on the left abutment with 1.5 m high ogee overflow weir. Trapezoidal discharge channel with 3.5 m base width, 4 m deep and 1V:0.5H side slopes. Lined with reinforced concrete 300 m long 3.5 m deep with a flip bucket/energy dissipater at end.	10 m wide side channel spillway on left abutment, with 5.5 m high ogee overflow weir. Trapezoidal discharge channel with 3 m base width, 4 m deep and 1V:0.5H side slopes. Lined with reinforced concrete 300 m long 1.5 m deep with a flip bucket/energy dissipater at end.
Outlet works description	Dry well tower (25 m high) with inside dimensions of 4x4m. Three offtake levels on each of the two 1 m diameter stacks, controlled by gate valves.	Dry well tower (27.4 m high) with inside dimensions of 4x4m. Three offtake levels on each of the two 1 m diameter stacks, controlled by gate valves.
Access road length (km)	2.0	2.0

15.2.5 Updated capital cost estimate

The estimated comparative (capital) costs (February 2018 prices, excluding VAT) of the proposed scheme are summarised below. A more detailed costing is provided in **Appendix A**.

Upper Scheepersvlakte Dam	R 155.4 million
Land acquisition	R 21.6 million
Connecting pipework	R 60.3 million
Pump Station and power supply	R 116.4 million
TOTAL	**R 353.7 million**

Detailed cost estimates of the updated dam designs have been included in **Appendix C**.

16 Recommendations

16.1 Screening of Options

The following potential options for improving the assurance of supply that is provided by the Scheepersvlakte Balancing Dam to the Nooitgedagt WTW was identified, evaluated and discarded, following the preliminary evaluation of options:

1. Balancing storage on the right bank of the lower Sundays River (near the Nooitgedagt WTW) in combination with a raised Scheepersvlakte Balancing Dam wall.
2. Diverting water from the existing Korhaansdrift Weir via a right bank pipeline to Nooitgedagt WTW for additional delivery of the NMBM's water allocation.
3. Increased balancing capacity at the Korhaansdrift Weir and diverting water via a right bank pipeline to Nooitgedagt WTW for full delivery of the NMBM's water allocation.
4. Releasing water from the existing Korhaansdrift Weir and diverting it closer to the Nooitgedagt WTW via a new pump station for full delivery of the NMBM's water allocation.
5. Increased balancing capacity at the Korhaansdrift Weir, with water releases to a new pump station downstream in the Sundays River close to the Nooitgedagt WTW.

Following preliminary screening, as well as the identification of sub-options, the following two preliminary options were further evaluated:

6. Constructing a larger dam near the present Scheepersvlakte Balancing Dam site and integrate this dam with the existing gravity pipeline to the Nooitgedagt WTW.
7. Constructing a large balancing dam on the right bank near the Nooitgedagt WTW.

The Nooitgedagt sites (North Option 1 and South) have been ruled out based on the investigation and cost comparison of alternative balancing dam sites. Although these sites are strategically located near the Nooitgedagt WTW, the comparative cost of these options is nearly double that of the lowest cost option (Lower Coerney site). The development of the balancing dam options near the Nooitgedagt WTW can therefore not be justified from a cost point of view.

The Upper Coerney site is not recommended as it offers no additional advantage over the other two Scheepersvlakte balancing dam sites and the comparative cost is the highest of these three options.

16.2 Recommended Dam Site

The Lower Coerney site is the preferred site, followed by the Upper Scheepersvlakte site and the Upper Coerney site. The main advantages of the Lower Coerney site are:

- The lowest capital and operational cost.
- Water could be supplied by gravity from the canal to the dam and from the dam to the Nooitgedagt WTW, while pumping is required for all the other options.
- Moderate environmental impacts that can be mitigated.

The Lower Coerney Balancing Dam will be a zoned earthfill embankment dam, with a side channel ogee shaped overflow weir for flood control. The dam will have a total gross dam capacity of 4.6 million m^3/a, with a crest length of 550 m and a maximum height of 27.4 m.

A 1 400 mm diameter 200 m long connector pipe would deliver water from the long weir in the canal to the 1 400 mm diameter pipeline to Nooitgedagt WTW, and to the proposed 940 m long 1 500 mm diameter pipeline to the Lower Coerney Dam.

The 940 m long 1 500 mm gravity pipeline would deliver water to fill the dam and the pipeline would also be used to supply the Nooitgedagt WTW in the event of a failure of the canal.

Modifications to the inlet to the Nooitgedagt WTW would also be required to make up for head loss and to increase the flow in the existing 1 400 mm Nooitgedagt pipeline at times when the storage in the proposed Lower Coerney Dam would be drawn down.

Because the reserve storage and infrastructure would be remote from Nooitgedagt WTW, an additional siphon under the Sundays River would be required to reduce the risk of wash away of the existing 1 400 mm siphon. A stockpile of pipes should be readily available and a process for the quick replacement of damaged pipes should be in place, should this be required.

16.3 Joint Use of Water

The Lower Coerney Dam site falls on land being planned for 650 ha of irrigation development by the Scheepersvlakte 98 Citrus Development Trust (Scheepersvlakte Farms (Pty) Ltd). The joint use of water from the dam by the Municipality and the private developer would need careful planning and has the following implications:

- A volume of 100 000 m^3 has been included in the volume of the proposed dam, for use by the private developer, in lieu of the development and use of their own balancing dam for irrigation. This volume should be confirmed before the design proceeds.
- A water use agreement with the Trust should be arranged. An interim offtake, until the dam has been completed, has been provided by the LSRWUA.
- While the irrigation water that passes through the dam would probably be sufficient to maintain acceptable salinity for urban use and may need to be managed to ensure that the quality would be acceptable for the irrigation of citrus.

16.4 Environmental

None of the sites are fatally flawed. The higher ecological sensitivity of the Coerney sites (i.e. for the recommended Lower Coerney site) will, however, require a terrestrial and aquatic ecology study and there might be recommendations emanating from such studies, which will have a cost implication. The need for a mining permit (to source material for the dam basin) as well as a waste licence to dispose on-site of inert waste (spoil) will only be clear once the exact scope of the project has been confirmed.

Based on the desktop and site analysis of the Coerney and Scheepersvlakte sites, it is envisaged that the following specialist studies will be required:

- Aquatic ecology assessment (inclusive of a wetland assessment),
- Terrestrial ecology assessment,
- Phase 1 Heritage, paleontological and cultural assessment,
- Ecological Water Requirements Determination Study (see comments under the next section).

Should a borrow/mining area be required, it is likely that the abovementioned studies will have to be done for that site as well (depending on location, size and ecological integrity).

16.5 Ecological Water Requirements

The DWS should undertake an EWR determination study for non-perennial systems for the small ephemeral tributary of the Coerney River, in which the Lower Coerney Dam will be situated. The Coerney River flows into the lower Sundays River near the Nooitgedagt WTW. The dam will have a natural catchment area of 34 km^2, and natural flows could potentially be routed through the dam, although this will have design and cost implications. While the undertaking of an EWR is outside the scope of this study, there is a need to identify the most important aspects to be take into consideration in the feasibility design.

16.6 Further Geotechnical Investigations

Further geotechnical investigations would be required for detail design purposes at the Lower Coerney dam site. Specific aspects to be addressed would include;

- Further confirmation of the geological profile, in particular along the cut-off trench and the intake / outlet conduit. More detailed knowledge of the lateral and vertical continuity of the various soil and rock strata would allow better definition of the cut-off requirements.
- Confirmation of the founding conditions at the end of the spillway to aid appropriate design of the terminal structure.
- The availability, preferably within the basin area, of materials suitable for embankment construction; in particular, impervious core material and semi-pervious material for the outer shell zones. This must be a prime focus of further investigations and could have a large impact on final embankment design.

Such investigations on the dam and spillway footprint would include additional rotary core drilling, and test pitting. Deep trenching might be considered.

Within the basin and surrounds, an intensive test pitting programme using a TLB is required to delineate borrow areas. The fieldwork must be complimented by a comprehensive laboratory testing programme, which must also allow for control testing by third party laboratories.

16.7 Other Design Considerations

Further considerations for the feasibility-level design of the Lower Coerney Dam are:

- A water quality evaluation should be undertaken, to determine if the water quality of natural runoff due to the geology will negatively impact on the water quality of the balancing dam.
- Further geotechnical investigations (as above) in concert with design development to determine:
  - Material requirements for the embankment and their sources (including identification of possible commercial sources for filter material).
  - Spillway and outlet works locations
- Consideration of the effect of attenuation of the design floods by determining flood hydrographs and development of the spillway arrangement.
- Refining the outlet works design requirements, including operational constraints and EWR.

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Appendix A

Comparative Costing of Dams

The comparative costing of the dam options and variations thereof is shown in the following table. The design elements and costing of the Upper Scheepersvlakte and Lower Coerney dam options have been refined, taking into account findings from the topographical and geotechnical surveys undertaken for these sites, as well as the design flood analysis.

Item	Unit	Upper Scheepersvlakte (refined design)	Lower Coerney (refined design)	Upper Coerney	Nooitgedagt North Option 1	Nooitgedagt North Option 2	Nooitgedagt North Option 4c	Nooitgedagt South
Latitude		33°27'7.88"S	33°26'54.12"S	33°26'5.91"S	33°32'14.52"S	33°32'14.52"S	33°32'14.52"S	33°32'45.84"S
Longitude		25°36'47.42"E	25°37'33.03"E	25°37'25.52"E	25°38'49.52"E	25°38'49.52"E	25°38'49.52"E	25°39'21.39"E
Embankment								
Full Supply Level (FSL)	masl	128.3	98.79	109.11	101	93	102	87.86
Non Overspill Crest level (NOC)	masl	131.8	103.79	114.11	103	95	104	89.86
Minimum basin level	masl	104.39	84.8	94.8	87	78	91	53
Maximum height	m	27.4	18.99	19.32	16	17	13	36.86
Freeboard	m	3.5	5.0	5.0	2.0	2.0	2.0	2.0
Capacity at FSL	m^3	4,600,000	4,600,000	4,600,000	4,804,000	4,621,000	5,087,000	4,600,000
Surface area at FSL	km^2	632,000	594,317	744,502	780,000	720,000	680,000	468,251
Crest length	m	550	623	357	2739	2960	2159	1970
Crest width	m	5.5	5	5	5	5	5	7.1
Upstream slope		1V:3H	1V:3H	1V:3H	1V:3H	1V:3H	1V:3H	1V:3H
Downstream slope		1V:2H	1V:2H	1V:2H	1V:2H	1V:2H	1V:2H	1V:2H
Maximum core trench depth	m	8	7	6	3.5	3.5	3.5	6
Minimum core trench depth	m	5	4	4	3.5	3.5	3.5	3.5
Average core trench depth	m	5.6	5	4.4	3.5	3.5	3.5	4
Total embankment volume	m^3	463,584	355,993	246,363	1,053,106	1,341,678	922,688	1,755,715
Core trench volume	m^3	51,950	46,798	21,507	98,621	113,319	89,000	102,849
Imported and overhaul material		Sand, gravel and rock and 70% of clay imported.	Sand imported. All clay found in basin	Sand imported. All clay found in basin	Sand imported. All clay found in basin	Sand imported. All clay found in basin	Sand imported. All clay found in basin	Sand imported. All clay found in basin
Filters		Chimney filter (0.8m thick), finger drains and rock toe	Chimney filter (0.5m thick), finger drains and rock toe	Chimney filter (0.5m thick), finger drains and rock toe	Chimney filter (0.5m thick), finger drains and rock toe	Chimney filter (0.5m thick), finger drains and rock toe	Chimney filter (0.5m thick), finger drains and rock toe	Chimney filter (0.5m thick), finger drains and rock toe
Spoil material from basin (@R45/m³)	m^3	-	-	-	-	-	1,000,000	-
Water to wall ratio		9.9	12.9	18.7	4.6	3.4	5.5	2.6
Core trench % of embankment	%	10%	13%	9%	9%	8%	10%	6%
Floods								
Regional Maximum Flood for zone 5.4	m^3/s	217	890	824	-	-	-	-
Recommended Design Flood (RDF) (unrouted)	m^3/s	22.5	143	824	-	-	-	-
Safety Evaluation Flood (SEF) (unrouted)	m^3/s	141	835	-	-	-	-	-
Catchment area	km^2	3.5	34	30	-	-	-	-
Reservoir area at FSL	km^2	0.632	0.59432	0.74450	-	-	-	-

Item	Unit	Upper Scheepersvlakte (refined design)	Lower Coerney (refined design)	Upper Coerney	Nooitgedagt North Option 1	Nooitgedagt North Option 2	Nooitgedagt North Option 4c	Nooitgedagt South
Spillway								
Spillway description		Side channel spillway with concrete lined discharge channel with flip bucket/energy dissipater at end.	Side channel spillway with concrete lined discharge channel with flip bucket/energy dissipater at end.	Side channel spillway with concrete lined discharge channel with flip bucket/energy dissipater at end.	Unlined bywash spillway on flank	Unlined bywash spillway on flank	Unlined bywash spillway on flank	Unlined bywash spillway on flank
Spillway overflow crest width (m)	m	10	36	32	5	5	5	5
Discharge coefficient		2.23 (at extreme flood)	2.23 (at extreme flood)	2	1.45	1.45	1.45	1.45
Maximum spillway discharge (m3/s)	m^3/s	146	805	716	34	34	34	34
Spillway channel side slopes		1V:0.5H	1V:2H	1V:2H	1V:2H	1V:2H	1V:2H	1V:2H
Spillway sill height	m	5.5	2.5	3.0	0.5	0.5	0.5	0.5
Discharge channel width	m	3.0	15.0	15.0	5.0	5.0	5.0	5.0
Discharge channel length	m	300	190	185	50	50	50	500
Discharge channel lining		Concrete	Concrete	Concrete	Unlined	Unlined	Unlined	Unlined
Outlet Works								
Intake structure description		Dry well tower with inside dimensions of 4x4m. Three offtake levels controlled by valves.	Dry well tower with inside dimensions of 4x4m. Three offtake levels controlled by valves.	Dry well tower with inside dimensions of 4x4m. Three offtake levels controlled by valves.	Dry well tower with inside dimensions of 2x2m. Three offtake levels controlled by valves.	Dry well tower with inside dimensions of 2x2m. Three offtake levels controlled by valves.	Dry well tower with inside dimensions of 2x2m. Three offtake levels controlled by valves.	Dry well tower with inside dimensions of 2x2m. Three offtake levels controlled by valves.
Tower height	m	27.4	19	19.32	15	15	15	36.9
Outlet pipe diameters	m	1	1	1	1	1	1	1
No of outlet pipes	No	2	2	2	2	2	2	2
D/S outlet chamber		Concrete outlet chamber, 10x10x5m, with 2 valves for the two pipes	Concrete outlet chamber, 4x4x3m, with 2 valves for the two pipes	Concrete outlet chamber, 4x4x3m, with 2 valves for the two pipes	Concrete outlet chamber, 4x4x3m, with 2 valves for the two pipes	Concrete outlet chamber, 4x4x3m, with 2 valves for the two pipes	Concrete outlet chamber, 4x4x3m, with 2 valves for the two pipes	Concrete outlet chamber, 4x4x3m, with 2 valves for the two pipes
No of offtake levels (incl. bottom scour)	No	3	3	3	3	3	3	3
Other		Provision made for access bridge to tower and hoists/cranes	Provision made for access bridge to tower and hoists/cranes	Provision made for access bridge to tower and hoists/cranes	Provision made for access bridge to tower and hoists/cranes	Provision made for access bridge to tower and hoists/cranes	Provision made for access bridge to tower and hoists/cranes	Provision made for access bridge to tower and hoists/cranes
DAM COST ESTIMATE								
Construction		R 86,940,092.00	R 97,663,314.00	R 63,158,159.19	R 145,726,174.84	R 180,550,582.22	R 213,584,528.07	R 240,309,566.49
P&G @ 30%		R 26,082,027.60	R 29,298,994.20	R 18,947,447.76	R 43,717,852.45	R 54,165,174.67	R 64,075,358.42	R 72,092,869.95
Contingencies @ 25%		R 28,255,529.90	R 31,740,577.05	R 20,526,401.74	R 47,361,006.82	R 58,678,939.22	R 69,414,971.62	R 78,100,609.11
Engineering fees @ 10%		R 14,127,764.95	R 15,870,288.53	R 10,263,200.87	R 23,680,503.41	R 29,339,469.61	R 34,707,485.81	R 39,050,304.55
Subtotal excl VAT		R 155,405,414.45	R 174,573,173.78	R 112,895,209.55	R 260,485,537.53	R 322,734,165.72	R 381,782,343.92	R 429,553,350.10
VAT @ 15%		R 23,310,812.17	R 26,185,976.07	R 16,934,281.43	R 39,072,830.63	R 48,410,124.86	R 57,267,351.59	R 64,433,002.51
Subtotal		**R 178,716,226.62**	**R 200,759,149.84**	**R 129,829,490.98**	**R 299,558,368.16**	**R 371,144,290.58**	**R 439,049,695.51**	**R 493,986,352.61**
Realignment of power lines		-	-	-	-	R 26,000,000.00	-	-
Land acquisition		R 21,598,816.76	R 18,788,735.00	R 19,355,593.86	R 23,272,998.69	R 22,591,020.92	R 20,321,909.78	R 15,361,734.67
GRAND TOTAL		**R 200,315,043.38**	**R 219,547,884.84**	**R 149,185 084.84**	**R 322,831,366.85**	**R 419,735,311.50**	**R 459,371,605.29**	**R 509,348,087.28**

Item	Unit	Upper Scheepersvlakte (refined design)	Lower Coerney (refined design)	Upper Coerney	Nooitgedagt North Option 1	Nooitgedagt North Option 2	Nooitgedagt North Option 4c	Nooitgedagt South
CONNECTING PIPEWORK								
Pipe infrastructure description		**200m** x 1.4m dia connector pipe (main canal->exist 1.4m dia to Nooitg), **1540m** x 1.3m dia pump/ gravity main to dam inlet/outlet; low lift pump st.; **180m** x 1.4m dia parallel S-River crossing; pipejack sleeve pipes under Main Canal	**200m** x 1.4m dia connector pipe (main canal -> exist 1.4m gravity to Nooitg); **940m** x 1.5m dia inlet/ outlet gravity pipe to dam inlet/ outlet; **180m** x 1.4m dia parallel S-River crossing; Pipejack sleeve pipes under Coerney Canal	**200m** x 1.4m dia connector pipe (main canal->exist 1.4m gravity to Nooitg); **2.46km** x 1.4m dia inlet/outlet pump/gravity to dam inlet/outlet; **180m** x 1.4m dia parallel S-River crossing; Pipejack sleeve pipes under Coerney Canal	**200m** x 1.4m dia connector pipe (main canal->exist 1.4m dia gravity to Nooitg); **240m** x1.4m dia inlet/outlet pump/gravity to dam inlet/outlet; **180m** x 1.4m dia parallel S-River crossing.	**200m** x 1.4m dia connector pipe (main canal->exist 1.4m dia gravity to Nooitg); **240m** x1.4m dia inlet/outlet pump/gravity to dam inlet/outlet; **180m** x 1.4m dia parallel S-River crossing.	**200m** x 1.4m dia connector pipe (main canal->exist 1.4m dia gravity to Nooitg); **240m** x1.4m dia inlet/outlet pump/gravity to dam inlet/outlet; **180m** x 1.4m dia parallel S-River crossing.	**200m** x1.4m dia connector pipe (main canal->exist 1.4m dia gravity to Nooitg); **660m** x 1.4m dia inlet/outlet pump/gravity to dam inlet/outlet; **180m** x1.4m dia parallel S-River crossing
Earthworks & Trenching		R 5,360,000	R 4,310,000	R 7,055,200	R 3,202,800	R 3,202,800	R 3,202,800	R 3,869,000
Pipework & Valves		R 18,912,000	R 14,102,000	R 27,974,000	R 6,151,800	R 6,151,800	R 6,151,800	R 11,342,500
Concrete works & Sleeve Jacking		R 9,500,000	R 8,750,000	R 9,934,000	R 3,585,000	R 3,585,000	R 3,585,000	R 3,817,500
Construction Cost		**R 33,772,000**	**R 27,162,000**	**R 44,963,200**	**R 12,939,600**	**R 12,939,600**	**R 12,939,600**	**R 19,029,000**
P&G @ 30%		R 10,131,600	R 8,148,600	R 13,488,960	R 3,881,880	R 3,881,880	R 3,881,880	R 5,708,700
Contingencies @ 25%		R 10,975,900	R 8,827,650	R 14,613,040	R 4,205,370	R 4,205,370	R 4,205,370	R 6,184,425
Engineering fees @ 10%		R 5,487,950	R 4,413,825	R 7,306,520	R 2,102,685	R 2,102,685	R 2,102,685	R 3,092,213
TOTAL (Excl VAT)		**R 60,367,450**	**R 48,552,075**	**R 80,371,720**	**R 23,129,535**	**R 23,129,535**	**R 23,129,535**	**R 34,014,338**
PUMP STATION & POWER SUPPLY								
Pump Station Design Requirements		3 x Low Lift Pumps (two duty + 1 stand-bye) total output 80-110 Mℓ /day with VSDs	Gravity supply	3 x Low Lift Pumps (two duty + 1 stand-bye) total output 80-110 Mℓ /day with VSDs	3 x Low Lift Pumps (two duty + 1 stand-bye) total output 80-110 Mℓ /day with VSDs	3 x Low Lift Pumps (two duty + 1 stand-bye) total output 80-110 Mℓ /day with VSDs	3 x Low Lift Pumps (two duty + 1 stand-bye) total output 80-110 Mℓ /day with VSDs	3 x Low Lift Pumps (two duty + 1 stand-bye) total output 80-110 Mℓ /day with VSDs
Modifications @ WTW Inlet to reduce head losses			R 5,400,000					
Civil Works		R 19,400,000		R 19,400,000	R 19,400,000	R 19,400,000	R 19,400,000	R 19,400,000
Mechanical & Electrical Works		R 28,800,000		R 28,800,000	R 28,800,000	R 28,800,000	R 28,800,000	R 28,800,000
Power supply		R 16,900,000	R -	R 16,900,000	R 4,300,000	R 4,300,000	R 4,300,000	R 5,000,000
Construction Cost		**R 65,100,000**	**R 5,400,000**	**R 65,100,000**	**R 52,500,000**	**R 52,500,000**	**R 52,500,000**	**R 53,200,000**
P&G @ 20%		R 19,530,000	R 1,620,000	R 19,530,000	R 15,750,000	R 15,750,000	R 15,750,000	R 15,960,000
Contingencies @ 15%		R 21,157,500	R 1,755,000	R 21,157,500	R 17,062,500	R 17,062,500	R 17,062,500	R 17,290,000
Engineering fees @ 7%		R 10,578,750	R 877,500	R 10,578,750	R 8,531,250	R 8,531,250	R 8,531,250	R 8,645,000
TOTAL (Excl VAT)		**R 116,366,250**	**R 9,652,500**	**R 116,366,250**	**R 93,843,750**	**R 93,843,750**	**R 93,843,750**	**R 95,095,000**
TOTALS								
DAM		R 155,405,414.45	R 174,573,173.78	R 112,895,209.55	R 260,485,537.53	R 322,734,165.72	R 381,782,343.92	R 429,553,350.10
REALIGNMENT OF HV POWER LINES		R -	R -	R -	R -	R 26,000,000.00	R -	R -
LAND ACQUISITION		R 21,598,816.76	R 18,788,735.00	R 19,355,593.86	R 23,272,998.69	R 22,591,020.92	R 20,321,909.78	R 15,361,734.67
CONNECTING PIPEWORK		R 60,367,450	R 48,552,075	R 80,371,720	R 23,129,535	R 23,129,535	R 23,129,535	R 34,014,338
PUMP STATION AND POWER SUPPLY		R 116,366,250	R 9,652,500	R 116,366,250	R 93,843,750	R 93,843,750	R 93,843,750	R 95,095,000
TOTAL		**R 353,737,931.21**	**R 251,566,483.77**	**R 328,988,773.41**	**R 400,731,821.22**	**R 488,298,471.64**	**R 519,077,538.70**	**R 574,024,422.27**
15% VAT		R 53,060,689.68	R 37,734,972.57	R 49,348,316.01	R 60,109,773.18	R 73,244,770.75	R 77,861,630.81	R 86,103,663.34
GRAND TOTAL		**R 406,798,620.89**	**R 289,301,456.34**	**R 378,337,089.42**	**R 460,841,594.40**	**R 561,543,242.38**	**R 596,939,169.51**	**R 660,128,085.61**
RANK		3	1	2	4	5	6	7

Appendix B
Eskom Tariff 2017/2018

Electricity costs were determined for Eskom's current 2017/2018 Megaflex Tariffs for Non-local Authorities taking account of the Time of Use Charges and of the Low Demand and High Demand Seasons to be as follows as indicated in the Table below:

- R0.67/kWh for the Low Demand Season (September to May)
- R1.06/kWh for the High Demand Season (June to August).

In December 2017 NERSA approved a 5.23% increase in Eskom's tariffs for 2018/2019. Assuming that the Megaflex Tariffs would also be increased by this percentage the tariffs for 2017/2018 would be approximately as follows:

- R0.70/kWh for the Low Demand Season (September to May)
- R1.11/kWh for the High Demand Season (June to August).

Therefore the average tariff for 2017/2018 would be approximately R0.81/kWh.

However, if pumping is only undertaken on weekdays as proposed then the average cost of electricity would be about R1.45/kWh.

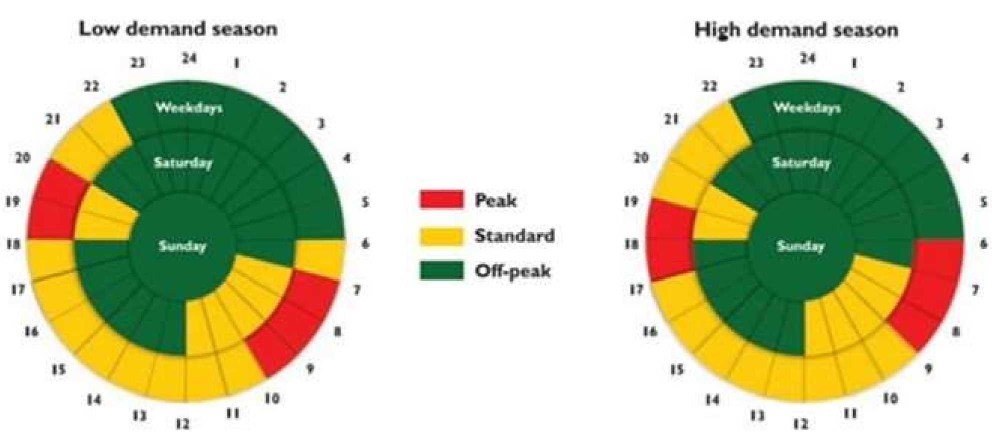

Figure B.1: Eskom Time of Use Periods for 2017-2018 – Low Demand Season

MEGAFLEX - Non-local authority rates

Transmission zone	Voltage	Active energy charge (c/kWh)												Transmission Network charge (R/kVA/m)	
		High demand season (Jun-Aug)						Low demand season (Sep-May)							
		Peak	VAT incl	Standard	VAT incl	Off Peak	VAT incl	Peak	VAT incl	Standard	VAT incl	Off Peak	VAT incl		VAT incl
≤300km	< 500V	278.33	**317.30**	84.68	**96.54**	46.24	**52.71**	91.14	**103.90**	62.89	**71.69**	40.09	**45.70**	7.96	**9.07**
	≥ 500V & < 66kV	273.96	**312.31**	83.00	**94.62**	45.07	**51.38**	89.36	**101.87**	61.51	**70.12**	39.02	**44.48**	7.28	**8.30**
	≥ 66kV & ≤ 132kV	265.29	**302.43**	80.36	**91.61**	43.65	**49.76**	86.55	**98.67**	59.56	**67.90**	37.79	**43.08**	7.09	**8.08**
	> 132kV*	250.03	**285.03**	75.74	**86.34**	41.14	**46.90**	81.58	**93.00**	56.13	**63.99**	35.62	**40.61**	8.95	**10.20**
> 300km & ≤ 600km	< 500V	280.60	**319.88**	85.02	**96.92**	46.16	**52.62**	91.54	**104.36**	63.02	**71.84**	39.98	**45.58**	8.02	**9.14**
	≥ 500V & < 66kV	276.70	**315.44**	83.82	**95.55**	45.52	**51.89**	90.27	**102.91**	62.12	**70.82**	39.41	**44.93**	7.35	**8.38**
	≥ 66kV & ≤ 132kV	267.90	**305.41**	81.15	**92.51**	44.06	**50.23**	87.39	**99.62**	60.14	**68.56**	38.15	**43.49**	7.14	**8.14**
	> 132kV*	252.53	**287.88**	76.51	**87.22**	41.52	**47.33**	82.36	**93.89**	56.69	**64.63**	35.95	**40.98**	9.04	**10.31**
> 600km & ≤ 900km	< 500V	283.40	**323.08**	85.85	**97.87**	46.60	**53.12**	92.45	**105.39**	63.63	**72.54**	40.35	**46.00**	8.12	**9.26**
	≥ 500V & < 66kV	279.48	**318.61**	84.67	**96.52**	45.98	**52.42**	91.16	**103.92**	62.75	**71.54**	39.81	**45.38**	7.41	**8.45**
	≥ 66kV & ≤ 132kV	270.63	**308.52**	81.98	**93.46**	44.51	**50.74**	88.27	**100.63**	60.76	**69.27**	38.54	**43.94**	7.18	**8.19**
	> 132kV*	255.08	**290.79**	77.26	**88.08**	41.97	**47.85**	83.21	**94.86**	57.26	**65.28**	36.34	**41.43**	9.17	**10.45**
> 900km	< 500V	286.25	**326.33**	86.74	**98.88**	47.09	**53.68**	93.39	**106.46**	64.26	**73.26**	40.79	**46.50**	8.16	**9.30**
	≥ 500V & < 66kV	282.26	**321.78**	85.50	**97.47**	46.41	**52.91**	92.06	**104.95**	63.35	**72.22**	40.20	**45.83**	7.50	**8.55**
	≥ 66kV & ≤ 132kV	273.34	**311.61**	82.80	**94.39**	44.96	**51.25**	89.16	**101.64**	61.37	**69.96**	38.93	**44.38**	7.25	**8.27**
	> 132kV*	257.56	**293.62**	78.06	**88.99**	42.41	**48.35**	84.07	**95.84**	57.88	**65.98**	36.74	**41.88**	9.24	**10.53**

* 132kV all Transmission connected

Distribution network charges						
Voltage	Network capacity charge (R/kVA/m)	VAT incl	Network demand charge (R/kVA/m)	VAT incl	Urban low voltage subsidy charge (R/kVA/m)	VAT incl
< 500V	15.82	**18.03**	30.00	**34.20**	0.00	0.00
≥ 500V & < 66kV	14.51	**16.54**	27.52	**31.37**	0.00	0.00
≥ 66kV & ≤ 132kV	5.18	**5.91**	9.60	**10.94**	12.78	**14.57**
> 132kV / Transmission connected	0.00	0.00	0.00	0.00	12.78	**14.57**

Customer categories	Service charge (R/account/day)	VAT incl	Administration charge (R/POD/day)	VAT incl
> 1 MVA	181.66	**207.09**	81.87	**93.33**
Key customer	3 559.79	**4 058.16**	113.69	**129.61**

Voltage	Ancillary service charge (c/kWh)	VAT incl
< 500V	0.37	**0.42**
≥ 500V & < 66kV	0.36	**0.41**
≥ 66kV & < 132kV	0.34	**0.39**
≥ 132kV*	0.32	**0.36**

* 132kV all Transmission connected

Reactive energy charge (c/kVArh)	
High season	VAT incl
12.80	**14.59**
Low season	VAT incl
0.00	0.00

Figure B.2: Eskom Megaflex Tariffs for 2017-2018

Table B.1: Estimated Eskom Low and High Demand Season Electricity Tariffs for 2017/2018 for Non-local Authorities

	LOW SEASON	HIGH SEASON
Transmission Network Charge R/kVA/month	*R 7.50*	*R 7.50*
Service Charge/day >1 MVA R/day	*R 181.66*	*R 181.66*
Admin Charge/day > 1 MVA R/day	*R 81.87*	*R 81.87*
Network Capacity Charge R/kVA/month	R 14.51	R 14.51
Network Demand Charge R/kVA/month	R 27.52	R 27.52
Reactive Energy Charge R/kVAr/month	R 12.78	R 12.78
Ancillary Service Charge R/kWh	R 0.0036	R 0.0036
During peak TOU R/kWh	R 0.9206	R 2.8226
During standard TOU R/kWh	R 0.6335	R 0.8550
During off peak TOU R/kWh	R 0.4020	R 0.4641
Power (kW)	500.00	500.00
Overall power factor (assumed)	0.95	0.95
Total Power demand (kW)	500.00	500.00
Estimated Maximum Demand (kVA)	526.32	526.32
Peak TOU hours/month	108.0	108.0
Standard TOU hours/month	269.0	269.0
Off peak TOU hours per month	351.0	351.0
Total hours per month	728.0	728.0
	Per Month	
Transmission Network Charge	*R 3,947.37*	*R 3,947.37*
Service Charge	*R 5,449.80*	*R 5,449.80*
Admin Charge	*R 2,456.10*	*R 2,456.10*
Network Capacity Charge	*R 7,636.84*	*R 7,636.84*
Network Demand Charge	*R 14,484.21*	*R 14,484.21*
Reactive Energy Charge	*R 6,726.32*	*R 6,726.32*
During peak TOU	*R 49,712.40*	*R 152,420.40*
During standard TOU	*R 85,205.75*	*R 114,997.50*
During off peak TOU	*R 70,551.00*	*R 81,449.55*
Estimated Total Cost per Month (excl Voltage surcharge)	**R 242,222.42**	**R 385,620.72**
Estimated Cost /kWh	**R 0.67**	**R 1.06**
NERSA December 2017 plus 5.3% Increase	**R 0.70**	**R 1.11**
Average Tariff 2018	**R 0.80**	

Appendix C

Detailed cost estimates of updated dam designs

- Updated construction costs for Upper Scheepersvlakte Dam.
- Updated construction costs for Lower Coerney Dam.

EARTHFILL DAM COST MODEL					
(Note that not all items in the cost model are applicable)				Max wall height (m)=	27.4
	19-Feb-19	Crest width (m)	5.5	NOCL=	RL 131.8 m
	Proposed Upper Scheepersvlakte : 4.6 Mm3 gross storage capacity			FSL=	RL 128.3 m
	Upper Scheepersvlakte Dam Earthfill - Costing v8.xlsx				
No	DESCRIPTION	UNIT	RATE Mar 16 Rand	QUANTITY	AMOUNT (Excl VAT) Rand
1	**Clearing**				
	(a) sparse	ha	25,000	0.0	0
	(b) bush	ha	35,000	2.1	75,185
	(c) trees	ha	55,000	2.1	118,148
2	**River diversion**	Sum	500,000		500,000
3	**Excavation**				
	(a) Bulk				
	(i) all materials	m³	80	59,750	4,779,993
	(ii) extra over for rock	m³	400	2,987	1,194,998
	(b) Confined				
	(i) all materials	m³	200	7,667	1,533,314
	(ii) extra over for rock	m³	600	383	229,997
	(c) Preparation of solum				
	(i) all materials	m³	35	42,063	1,472,206
	(ii) extra over for rock	m³	150	2,103	315,473
4	**Drilling & Grouting**				
	(a) Curtain grouting	m drill	1,500	598	896,588
	(b) Consolidation grouting	m drill	1,500	20	30,000
	(c) Slurry trench - fill	m3	1,000	0	0
5	**Embankment**				
	(a) Earthfill	m³	80	439,933	35,194,646
	(b) Filters	m³	600	8,271	4,962,842
	(c) Rip rap	m³	600	10,330	6,198,018
	(d) Overhaul beyond 1km (one way)	m³km	10	247,245	2,472,447
	(e) Toe drain	m³	600	2,751	1,650,411
	(f) Spillway channel protection with reno	m³	3,000	102	306,000
6	**Concrete Works**				
	(a) Formwork				
	(i) gang formed	m²	675	1,906	1,286,706
	(ii) intricate	m²	850	2,695	2,290,990
	(b) Concrete				
	(i) mass	m³	2,500	1,395	3,487,968
	(ii) structural	m³	2,800	547	1,531,198
	(c) Reinforcing	t	16,500	119	1,963,111
7	**Mechanical Items**				
	(a) Valves & gates	No	250,000	10	2,500,000
	(b) Cranes & hoists	Sum	2,000,000		2,000,000
	(c) Structural steelwork	Sum	1,500,000		1,500,000
	(d) Outlet pipe (SS316)	m	10,000	353	3,528,716
	SUB-TOTAL				82,018,955

No	DESCRIPTION	UNIT	RATE Mar 16 Rand	QUANTITY	AMOUNT (Excl VAT) Rand
8	**Fencing**	km	Incl in 10		
9	**Landscaping** (% of 1-9)	%	1		820,190
10	**Miscellaneous** (% of 1-9)	%	5		4,100,948
	SUB TOTAL A				86,940,092
11	**Preliminary & General** (% of sub-total A)	%	30		26,082,028
	SUB TOTAL B				113,022,120
12	**Contingencies** (% of sub total B)	%	25		28,255,530
	SUB TOTAL C				141,277,650
13	**Professional fees** (% of sub total C)	%	10		14,127,765
	SUB TOTAL D				155,405,415
	VAT 15%				23,310,812
	TOTAL CONSTRUCTION COST				**178,716,227**
14	**Cost of relocations**	Sum	0	0	0
15	**Cost of land acquisition**				
	(a) Irrigated	ha	1,000,000	0.0	0
	(b) Dryland farming	ha	600,000	13.5	8,099,556
	(c) Undeveloped	ha	250,000	54.0	13,499,260
	(d) Homesteads	No	0		0
	TOTAL PROJECT COST (as at Mar 2016)				**200,315,044**
	(Rounded to nearest R 100 000)				200,400,000

EARTHFILL DAM COST MODEL					
(Note that not all items in the cost model are applicable)				Max wall height (m)=	21.4
	19-Feb-19	Crest width (m)	5.0	NOCL=	RL 102.9 m
	Proposed Lower Coerney : 4.6 Mm3 gross storage capacity			FSL=	RL 98.1 m
	Lower Coerney Dam Earthfill - Costing v15.xlsx				
No	DESCRIPTION	UNIT	RATE Dec 18 Rand	QUANTITY	AMOUNT (Excl VAT) Rand
1	**Clearing**				
	(a) sparse	ha	25,000	0.0	0
	(b) bush	ha	35,000	2.0	70,961
	(c) trees	ha	55,000	2.0	111,511
2	**River diversion**	Sum	500,000		500,000
3	**Excavation**				
	(a) Bulk				
	(i) all materials	m³	80	77,856	6,228,484
	(ii) extra over for rock	m³	400	7,786	3,114,242
	(b) Confined				
	(i) all materials	m³	200	12,842	2,568,455
	(ii) extra over for rock	m³	600	1,284	770,536
	(c) Preparation of solum				
	(i) all materials	m²	35	39,049	1,366,729
	(ii) extra over for rock	m²	150	1,952	292,870
4	**Drilling & Grouting**				
	(a) Curtain grouting	m drill	1,500	1,094	1,640,732
	(b) Consolidation grouting	m drill	1,500	54	81,000
	(c) Slurry trench - fill	m3	1,000	583	583,000
5	**Embankment**				
	(a) Earthfill	m³	80	332,184	26,574,688
	(b) Filters	m³	600	7,748	4,648,837
	(c) Rip rap	m³	600	10,910	6,546,029
	(d) Overhaul beyond 1km (one way)	m³km	10	198,760	1,987,600
	(e) Toe drain	m³	600	4,197	2,518,487
	(f) Spillway channel protection with reno	m³	3,000	268	802,800
6	**Concrete Works**				
	(a) Formwork				
	(i) gang formed	m²	675	4,519	3,050,117
	(ii) intricate	m²	850	3,276	2,784,522
	(b) Concrete				
	(i) mass	m³	2,500	4,558	11,394,690
	(ii) structural	m³	2,800	459	1,285,572
	(c) Reinforcing	t	16,500	269	4,442,060
7	**Mechanical Items**				
	(a) Valves & gates	No	250,000	10	2,500,000
	(b) Cranes & hoists	Sum	2,000,000		2,000,000
	(c) Structural steelwork	Sum	1,500,000		1,500,000
	(d) Outlet pipe (SS316)	m	10,000	277	2,771,280
	SUB-TOTAL				92,135,202

No	DESCRIPTION	UNIT	RATE Dec 18 Rand	QUANTITY	AMOUNT (Excl VAT) Rand
8	**Fencing**	km	Incl in 10		
9	**Landscaping** (% of 1-9)	%	1		921 352
10	**Miscellaneous** (% of 1-9)	%	5		4 606 760
	SUB TOTAL A				97 663 314
11	**Preliminary & General** (% of sub-total A)	%	30		29 298 994
	SUB TOTAL B				126 962 308
12	**Contingencies** (% of sub total B)	%	25		31 740 577
	SUB TOTAL C				158 702 885
13	**Professional fees** (% of sub total C)	%	10		15 870 289
	SUB TOTAL D				174 573 174
	VAT 15%				26 185 976
	TOTAL CONSTRUCTION COST				**200 759 150**
14	**Cost of relocations**	Sum	0		0
15	**Cost of land acquisition**				
	(a) Irrigated	ha	1 000 000	0.0	0
	(b) Dryland farming	ha	600 000	0.0	0
	(c) Undeveloped	ha	250 000	75.2	18 788 735
	(d) Homesteads	No	0		0
	TOTAL PROJECT COST (as at Dec 2018)				**219 547 885**
	(Rounded to nearest R 100 000)				219 600 000

Aurecon South Africa (Pty) Ltd
1977/003711/07
Aurecon Centre
1 Century City Drive
Waterford Precinct
Century City
Cape Town
7441
PO Box 494
Cape Town
8000
South Africa
T +27 21 526 9400
F +27 21 526 9500
E capetown@aurecongroup.com
W aurecongroup.com