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GROOT LETABA RIVER WATER DEVELOPMENT PROJECT (GLeWaP)

TECHNICAL STUDY MODULE:

Preliminary Design of Nwamitwa Dam

VOLUME 6

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LIST OF STUDY REPORTS IN GROOT LETABA RIVER WATER DEVELOPMENT PROJECT (BRIDGING STUDIES)

This report forms part of the series of reports, done for the bridging studies phase of the GLeWaP. All reports for the GLeWaP are listed below.

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P WMA 02/B810/00/0508	**Project Coordination and Management Team**
P WMA 02/B810/00/0508/1	Project Coordination and Management Team: Executive Summary Report: Vol 1
P WMA 02/B810/00/0508/2	Project Coordination and Management Team: Main Report: Vol 2
P WMA 02/B810/00/0508/3	Project Coordination and Management Team: Register of Decisions: Vol 3
P WMA 02/B810/00/0508/4	Project Coordination and Management Team
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P WMA 02/B810/00/0608	**Technical Study Module**
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P WMA 02/B810/00/1110/1	Technical Study Module: Preliminary Design of Nwamitwa Dam: Vol 6 - Annexure 1: Appendices
P WMA 02/B810/00/1110/2	Technical Study Module: Preliminary Design of Nwamitwa Dam: Vol 6 - Annexure 2: Appendix B (Part 1): Geotechnical Investigations
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P WMA 02/B810/00/0808/5	Public Involvement Program

REPORT DETAILS PAGE

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EXECUTIVE SUMMARY

1.1 BACKGROUND

The feasibility study of the development and management options in the Groot Letaba River which was completed in 1998, proposed the construction of a dam at Nwamitwa and the possible raising of Tzaneen Dam as options for augmenting water supply from the Groot Letaba River. Approximately 10 years had passed since the Feasibility Study was completed. The Department of Water Affairs (DWA) decided to embark on Bridging Studies to update the information and to re-assess the recommendations contained in the Feasibility Study.

1.2 SCOPE OF THIS REPORT

This report deals with the preliminary design of the proposed Nwamitwa Dam. It addresses the determination of the design flood peaks used to size the spillway, the further geotechnical investigations that were undertaken to supplement the work done during previous studies, the optimisation of the dam size based on the latest annual yield figures and dam costs, the selection of the most economical spillway type and the preliminary design of the embankment, spillway and outlet works. It also addresses the costs associated with the expropriation of the required properties to be inundated by the dam as well as the preliminary design and costing of the roads in the dam basin to be relocated.

1.3 HYDROLOGY

1.3.1 Spillway Floods

The Nwamitwa Dam will be a large dam (>30 m high) with a high hazard potential (due to extensive downstream developments) and will be classified as a Category III dam in terms of the Dam Safety Regulations.

The following calculation methods were used to determine the spillway floods:

- *Unitgraph techniques using dimensionless regional unitgraphs.*
- *Probabilistic (flood frequency) techniques using a range of probability distributions.*
- *Empirical flood techniques in the form of the Francou-Rodier approach, used by Kovacs to develop the Regional Maximum Flood (RMF) peak.*

The following flood peaks were selected to size the spillway:

- *Recommended Design Flood (RDF) (1:200 year RI)* *1 860 m^3/s*
- *Safety Evaluation Flood (SEF) ($RMF_{+\Delta}$)* *6 800 m^3/s*

1.3.2 River Diversion Floods

In addition to the spillway design floods, 10 year, 20 year and 50 year return interval (RI) flood hydrographs for a range of durations were estimated at the proposed Nwamitwa Dam site. The river diversion floods were calculated to be between 1 500 and 2 500 m^3/s depending on the chosen recurrence interval. Due to the significant size of these floods, it is proposed that the floods be scaled down as described below.

As the Tzaneen Dam does not frequently overflow, it is considered reasonable to assume that floods with low recurrence intervals that are generated in the catchment area upstream of Tzaneen Dam would be partially absorbed by Tzaneen Dam.

This implies that low recurrence interval river diversion floods at Nwamitwa Dam would mainly be generated from the incremental catchment downstream of Tzaneen Dam. As this phenomenon would be reflected in the peak flow record at Gauge B8H009, the flood frequency analysis at this gauge implicitly already accounts for this absorption. Based on this assumption, the calculated 1:10 year flood peak at Gauge B8H009 was scaled up based on the ratio of the square root of the incremental catchment downstream of Tzaneen Dam (1 294 km^2) to the incremental gauge catchment downstream of Tzaneen Dam (201 km^2). For the 1:50 year recurrence intervals flood peaks, it was assumed that although some flood absorption might occur in Tzaneen Dam, some spilling might also occur. For this flood event, the calculated flood peak at Gauge B8H009 was adjusted based on the full catchment area at Nwamitwa Dam and at the gauge respectively, i.e. $\sqrt{1944}$ / $\sqrt{851}$. *For the 1:20 year peak, both methods were applied and the average value was then calculated.*

The scaled river diversion floods at Nwamitwa Dam therefore results in the following figures:

- *1:10 yr* *1 000 m^3/s*
- *1:20 yr* *1 450 m^3/s*
- *1:50 yr* *2 100 m^3/s*

1.3.3 River Diversion Strategy

The Design Criteria Memorandum (DCM) calls for river diversion floods with a 1 in 20 year RI for a composite dam consisting of both concrete and earth fill sections and with a 1 in 50 year RI for an earth fill embankment. The first stage river diversion must protect the spillway foundation construction works, and the diversion works were therefore sized to handle a 1 in 20 year flood of 1 450 m^3/s. The second stage river diversion is required to protect the earth embankment during construction and has been designed to accommodate the 1 in 50 year flood of 2 100 m^3/s. It is recommended that the second stage of the river diversion strategy commence at the onset of the dry season in order to facilitate the installation of a diversion culvert 4 m wide by 3 m high with its invent at the river bed level of 454 masl in the middle of the spillway section. On the right hand side of the culvert a 60 m wide section of the spillway must be kept 3 m lower than the rest of the spillway for the duration of the spillway construction. The discharge capacity of the low section will be approximately 500 m^3/s, which will allow for the passing of floods during the dry season. The diversion culvert will keep the upstream water level at approximately the river bed level during normal dry season flow. As the dam is raised, the flood absorption capacity of its basin will increase. The 1 in 20 year flood volume will be absorbed when the embankment is at level 469 masl and the 1 in 50 year flood volume when it is at level 470,4 masl.

1.3.4 Sedimentation

Based on studies by Prof A Rooseboom, a sediment yield of 350 t/km^2.a has been estimated for the Nwamitwa Dam. The anticipated sediment volume after 50 years is estimated to be 17 Mm^3.

1.4 Geotechnical investigations

A number of engineering geological studies were undertaken prior to the current Bridging Study. These are listed below:

- *A **reconnaissance-level appraisal** of the proposed dam site, then known as the Janetsi site was conducted initially.*
- ***Feasibility-level engineering geological investigations** were conducted in 1996 during which a total of seventeen rotary core boreholes were drilled, at two possible centre-lines. Investigations shifted to the upstream centre-line after initial boreholes at the downstream site revealed unfavourable conditions.*
- *A **materials investigation** was conducted by the DWA Materials Laboratory in 1996.*

As part of this study, the following further geotechnical studies were undertaken:

- *Desk study of available geological information,*
- *Field mapping,*
- *Geophysical surveys,*
- *Additional rotary core drilling,*
- *Test pitting,*
- *Water pressure (Lugeon) testing and measurement of the water table,*
- *Laboratory testing, and*
- *Seismic hazard assessment.*

The location of the proposed Nwamitwa Dam is underlain by Mesoaechean granitoid gneisses, specifically the Groot Letaba Gneiss (previously Goudplaats Gneiss) which has been intruded by younger diabase dykes. No major faults occur in the vicinity of the dam but a number of lineaments are present. The level of seismic hazard may be described as moderate. At the dam site, shallow colluvial soils cover the left flank while the right flank is partly covered by reworked alluvial gravels and the remainder by colluvial sands; underlain by thin residual soils. Thick alluvial deposits occur within the river section. The granite bedrock generally occurs at shallow depth on the left flank but is deeply and variably weathered. On the right flank the bedrock is generally moderately or highly weathered. Within the river section the underlying bedrock is generally unweathered.

A seismic hazard assessment (SHA) for the proposed Nwamitwa Dam was conducted by the Council for Geoscience (CGS). In accordance with the CGS report the results from the attenuation model by Toro et al were adopted, namely an OBE value of 0.024g and a MCE value of 0.14g.

1.5 MATERIALS

The materials investigation by the DWA Materials Laboratory identified acceptable earthfill material on the right bank of the Groot Letaba River immediately upstream of the dam site for the construction of the earthfill sections of the proposed dam. The impervious material is a weathered dolerite and the semi-pervious material is a weathered granite. Some 700 000 m^3 of impervious material still needs to be proven.

Sand deposits were identified at two separate borrow areas; the first 20 km downstream of the dam site in the Merekome River and the second 11 km downstream of the dam site in the Phatle/Lerwatlou River.

The samples comprise quartzitic river sand. All samples tested complied with the SANS specifications for fine aggregate.

Two potential quarry sites were identified. Quarry Site A is located to the north-west of the dam site, approximately 15 km by road, in the hills above the settlement of Babanana. Quarry Site B is located to the south-west of the dam site near Letsitele, approximately 19 km by road.

Both sites are underlain by granites. The laboratory test results indicated that the unweathered granite will be generally suitable for use as concrete aggregate, but that the aggregate might be prone to Alkali Aggregate Reaction.

1.6 EMBANKMENT

The valley shape factor for the proposed Nwamitwa Dam site is in excess of 50, which is a clear indication that the most appropriate dam type would be an embankment type dam.

The following dam sizes were investigated:

Table 1.6.1 Selected Dam Sizes

Dam size	*Capacity (Million m^3)*	*FSL (masl)*	*NOC (masl)*
*0.41 MAR**	*66*	*473.5*	*480.0*
*0.85 MAR**	*137*	*477.5*	*484.0*
*1.16 MAR**	*187*	*479.5*	*486.0*
*1.50 MAR**	*241*	*481.5*	*488.0*

** Based on natural incremental MAR between Tzaneen and Nwamitwa Dams = 160.9 Mm^3*

The embankment will be 34 m high with a total crest length (including the spillway) of 3 500 m. The embankment volume above the original ground level is 1 430 000 m^3. The cross-section consists of an impervious core zone, semi-pervious general fill zones, chimney and blanket drains, rip-rap protection on the upstream slope and aggregate protection on the downstream slope. The non-overspill crest (NOC) level will be at 486.0 masl.

1.7 SPILLWAY

1.7.1 Selection of Spillway Type

The following four types of spillways were investigated for the proposed Nwamitwa Dam:

- *Straight ogee spillway*
- *Trough spillway*
- *Labyrinth spillway*
- *Side channel spillway*

*Due to technical constraints, the side channel spillway option was discarded as a viable option. The length of the proposed spillway would be almost double that of the largest side channel spillway constructed to date. Preliminary designs and associated cost estimates were undertaken for the remaining three options. The costs of the proposed Nwamitwa Dam for the three alternative spillway arrangements are given in **Table 1.7.1**. Please note that these cost estimates are comparative cost estimates (May 2008), and were based on preliminary spillway sizes and road relocation costs. Updated cost estimates are contained in **Table 1.11.1** of this report.*

Table 1.7.1 Cost of alternative spillway arrangements

Type of Spillway	Cost of proposed Nwamitwa Dam
Straight ogee spillway	R 777 million
Trough spillway	R 1 160 million
Labyrinth spillway	R 857 million

It was therefore recommended that the straight ogee spillway be adopted in the preliminary design of Nwamitwa Dam.

1.7.2 Description

The spillway would be constructed across the river channel, which would require significant widening to accommodate the 190 m long mass gravity RCC section. Tongue walls with a total length of 161 m would be provided on either side to accommodate the outlet works and to tie into the earth embankments.

For the most part the granite gneiss underlying the river section comprises unweathered, very hard rock, although upper, weathered horizons are present. Excavation depths are expected to vary from 10 m in the river section increasing to 16 m on the left bank and 27 m on the right bank respectively.

The maximum discharge capacity of the spillway is 6 800 m^3/s. The total freeboard is 6.5 m.

1.8 OUTLET WORKS

1.8.1 Flow Requirements

The outlet works need to fulfil the following duties:

- *Release the active storage capacity in accordance with the demand curve*
- *Release the environmental water requirements (EWR)*
- *Empty the dam during emergency drawdown conditions*

1.8.2 Description

The outlet works will be housed in an integral outlet block on the left hand side of the spillway. It will be equipped with precast concrete trash racks, stainless steel fine screens and a maintenance gate to close off the intakes to the pipe stacks.

The outlet pipework was based on the EWR as set out in the Preliminary Reserve Determination Study Scenario 7 at Site 3 (DWAF, 2006). The recommended Class II flood release category at EWR Site 3 at Prieska calls for discharge capacities of 12 to 18 m^3/s. As it is common practice to allow for 100% redundancy in the outlet pipework for operational flow releases, two pipe stacks will be provided. Due to the infrequent nature and short duration of the EWR, it is considered prudent from a cost point of view to use both pipe stacks to release the EWR. Each pipe stack was therefore sized to discharge 9 m^3/s.

The pipework will consist of two DN 1 200 mm pipe stacks. The intakes to the pipe stacks will be staggered at 4.5 m intervals to allow for flexibility in selecting the most appropriate abstraction level. Two DN 1000 mm sleeve valves will be provided on the downstream side.

1.9 RELOCATION OF ROADS

1.9.1 Introduction

A number of existing provincial roads are affected by the proposed dam. Possible routes for the re-alignment of these roads were investigated and preliminary costs determined before a decision was made regarding the route alignments that were selected for the preliminary design stage.

1.9.2 Road R529

Road R529 follows a north/south alignment to the west of the Groot Letaba River. The road is affected over a length of approximately 6.1 km and five alternative re-alignments were investigated before the preliminary design commenced. The road crosses the proposed dam basin over a section of approximately 3.0 km. The road will be constructed on an earthfill embankment with rock protection (rip-rap) where it crosses the dam basin to protect the fill against wave action. Two bridges are planned where the road crosses the Hlangana and Nwanedzi Rivers. A short section of road D1292 (approximately 1.4 km long) will also have to be re-aligned to join into road R529.

1.9.3 Road R43-3

Road P43-3 follows a north/south alignment along the eastern side of the Groot Letaba River. The road is affected over a length of approximately 6.9 km where it crosses a number of small tributaries. The existing road has to be raised to the NOC level where it crosses these tributaries. A section of 1.1 km of the existing road must be raised over three sections along the current horizontal alignment. The road was re-aligned horizontally where it crosses a number of small farms in conjunction with the landowners to minimise the impact it will have on their farming operations. The re-aligned road follows a fairly flat topography with no major cuts or fills and crosses a number of small streams where culverts are necessary.

1.9.4 Bridges

The re-aligned Road R529 crosses the Hlangana and Nwanedzi Rivers at km 2.15 and km 3.76 respectively where major bridges are required. The total road width is 12.4 m, made up of 2x3.7 m lanes and 2x2.5 m shoulders. A total bridge width of 13.35 m is therefore required. The road embankment is between 18 and 25 m high at these crossings, and will have side batters of 1V:3H, with 2 m wide terraces at 5 m vertical intervals.

1.10 EXPROPRIATION COSTS

The expropriation costs up to the 1 in 100 year RI flood level were included in the overall cost of the dam. The expropriation line, which depicts the minimum land purchase requirements due to dam construction, was determined according to the "Policy and Guidelines for the Acquisition of Land Rights at Departmental Dams" (DWAF, 2001). The 1:100 year flood line baseline information, which is needed for developing the expropriation line, was determined through backwater analysis of the system of rivers flowing into Nwamitwa Dam.

The estimated total compensation costs payable for the preferred dam size of 1.16 MAR (FSL 479.5 masl) amounts to R180 million.

1.11 DAM COSTS

Construction cost estimates were prepared for the four dam sizes that were used for the dam size optimisation. The cost estimates allow for the relocation of services (roads, electricity and telecommunications), contingencies, planning, design and supervision costs and expropriations costs.

*The cost estimates are summarised in **Table 1.11.1**.*

Table 1.11.1 Optimisation of Dam Sizes

Dam size (factor of MAR*)	Dam capacity (Mm^3)	Full Supply Level (masl)	Total Project Cost (R million) exc. VAT	Yield (Mm^3/a)	Unit Cost ($R/m^3/a$)
					Dam construction and land costs
0.41	66	473.50	989	4	24
0.85	137	477.50	1 180	9	13
1.16	187	479.50	1 285	14	91
1.50	241	481.50	1 409	17	82

* *Natural incremental MAR between Tzaneen and Nwamitwa Dams = 160.9 Mm^3/a*

***Figure 1.11.1** shows the relationship between the capacity of alternative dam sizes and the increase in historical firm yield from the system derived from each dam capacity. Whilst the analysis did not point to "one" optimal size, a dam with a FSL of 479.5 m and a historical firm yield of 14 Mm^3/a was proposed as the preferred dam size. This size will ensure that sufficient yield is obtained in order to meet the anticipated future water requirements of the area surrounding Nwamitwa Dam, limit expropriation costs and limit the amount of evaporation from the proposed dam.*

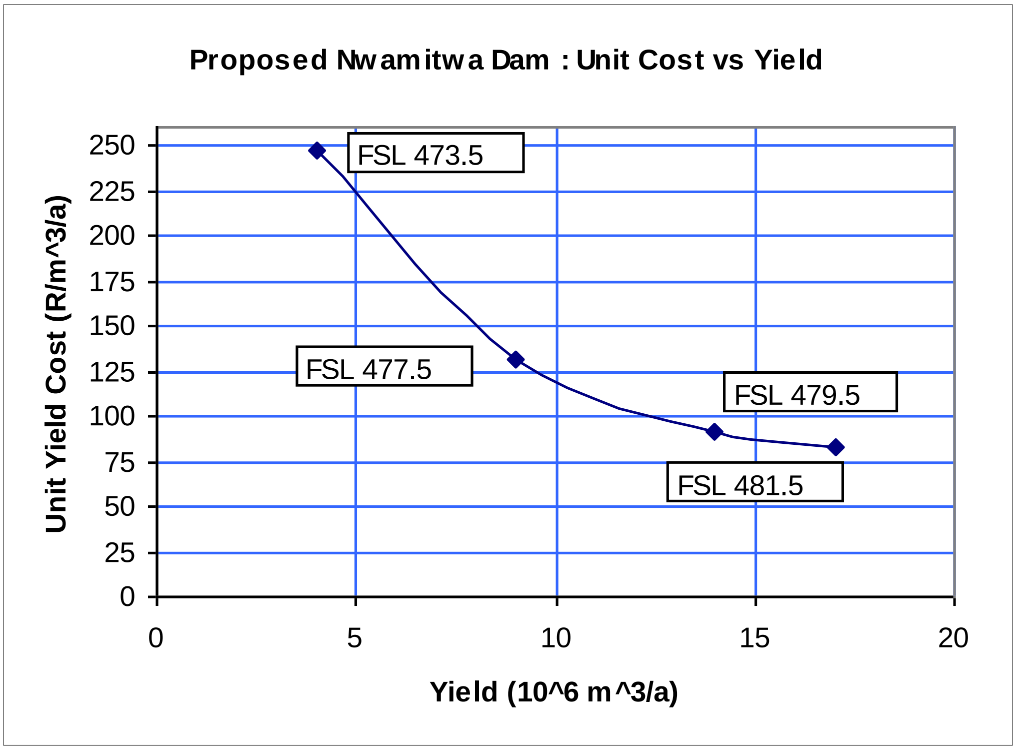

Figure 1.11.1 Nwamitwa Dam: Unit Cost vs Yield Curve

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(see separate Volume 6 Annexure 1 : Appendices)

A.1 Flood Hydrology Report
A.2 Sedimentation
A.3 Backwater Analysis

APPENDIX B : GEOTECHNICAL INVESTIGATIONS

(*see separate volumes : Volume 6 Annexure 2 : Appendix B (Part 1) : Geotechnical Investigations [text only] and Volume 6 Annexure 3 : Appendix B (Part 2) : Geotechnical Investigations [Appendices A-K])*

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(see separate Volume 6 Annexure 1 : Appendices)

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C4 Slope Stability Analysis
C5 Freeboard Calculations

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D2 Spillway Stage Discharge Curve
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(see separate Volume 6 Annexure 1 : Appendices)

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F4 Cost Estimate of Expropriation Costs

APPENDIX G : CONSTRUCTION PROGRAMME

(see separate Volume 6 Annexure 1 : Appendices)

APPENDIX H : DRAWINGS

(see separate Volume 6 Annexure 4 : Appendix H : Drawings)

H1 Preliminary Dam Design
H2 Preliminary Road Design
H3 Preliminary Bridge Design
H4 Expropriation Plan

APPENDIX I : COMMENTS

(see separate Volume 6 Annexure 1 : Appendices)

ABBREVIATIONS

API	Aerial Photograph Interpretation
CFP	Channel Flow Profiles
CGS	Council for Geoscience
DBE	Design Basis Earthquake
DCM	Design Criteria Memorandum
DN	Nominal diameter
d/s	downstream
DWA	Department of Water Affairs
EEGS	Engineering and Exploration Geophysical Services cc.
Emod	Deformation Modulus
EWR	Environmental Water Requirements
FSL	Full Supply Level
GLeWaP	Groot Letaba River Water Development Project
HRU	Hydrological Research Unit
ICOLD	International Committee on Large Dams
LHFP	Lesotho Highlands Further Phases
LHWC	Lesotho Highlands Water Commission
LORMS	Lower Orange River Management Study
MAR	Mean Annual Runoff
masl	meters above sea level
MCE	Maximum Credible Earthquake
NOC	Non Overspill Crest
OA	Option Analysis
OBE	Operation Basis Earthquake
OMC	Optimum Moisture Content
PCMT	Project Co-ordination and Management Team
PGA	Peak Ground Acceleration
PMF	Probable Maximum Flood
PSP	Professional Service Provider
RCC	Roller Compacted Concrete
RDF	Recommended Design Flood
RI	Recurrence Interval
RMF	Regional Maximum Flood
RMR	Rock Mass Rating
SANCOLD	South African National Committee on Large Dams

SANS	South African National Standards
SED	Safety Evaluation Discharge
SEF	Safety Evaluation Flood
SHA	Seismic Hazard Assessment
VAPS	Vaal Augmentation Planning Study
WRC	Water Research Commission
u/s	upstream

1. STUDY INTRODUCTION

1.1 BACKGROUND TO PROJECT

The catchment of the Groot Letaba River has many and varied land uses with their associated water requirements. These include significant use by agriculture in the form of irrigated crops, commercial afforestation, tourism (particularly linked to the Kruger National Park, which lies partially within the catchment), as well as primary demands by the population in the catchment. The water resources available in the catchment are limited, and considerable pressure has been put on these resources in the past, with periods of severe and protracted water restrictions occurring over the past 25 years. This situation has been investigated at various levels by the Department of Water Affairs (DWA).

The first major study undertaken for this area was the Letaba River Basin Study in 1985 (DWAF, 1990), which comprised the collection and analysis of all available data on water availability and use, as well as future water requirements and potential future water resource developments. This was followed by a Pre-feasibility Study (DWAF 1994), which was completed in 1994. The focus of the Pre-feasibility Study was the complete updating of the hydrology of the Basin. The next study undertaken was the Feasibility Study of the Development and Management Options (DWAF, 1998), which was completed in 1998.

The Feasibility Study proposed several options for augmenting water supply from the Groot Letaba River. These included some management interventions, as well as the construction of a dam at Nwamitwa and the possible raising of Tzaneen Dam. These options would enable additional water to be allocated to the primary water users, would allow the ecological Reserve to be implemented and could also improve the assurance of supply to the agricultural sector.

This Bridging Study was initiated by the DWA in order to re-assess the recommendations contained in the Feasibility Study in the light of developments that have taken place in the intervening 10 years. Other contributing factors to the DWA's decision to undertake Bridging Studies were the promulgation of the Water Services Act and the National Water Act in 1997 and 1998 respectively, and the recently completed Reserve Study on the Letaba River.

The study area, shown in **Figure 1.1**, consists of the catchment of the Groot Letaba River, upstream of its confluence with the Klein Letaba River. The catchment falls within the Mopani District Municipality, which is made up of six local municipalities. The four

Local Municipalities, parts or all of which are within the catchment area, are Greater Tzaneen, Greater Letaba, Ba Phalaborwa and Greater Giyane. The major town in the study area is Tzaneen, with Polokwane the provincial capital city of Limpopo located just outside of the catchment to the West.

The site of the proposed Nwamitwa Dam is also shown in **Figure 1.1**. The focus of the Feasibility Study was the Groot Letaba catchment, with the catchments of the other rivers being included to check that environmental flow requirements into the Kruger National Park were met, and international agreements regarding flow entering Moçambique were met. This focus was kept for the Bridging Study.

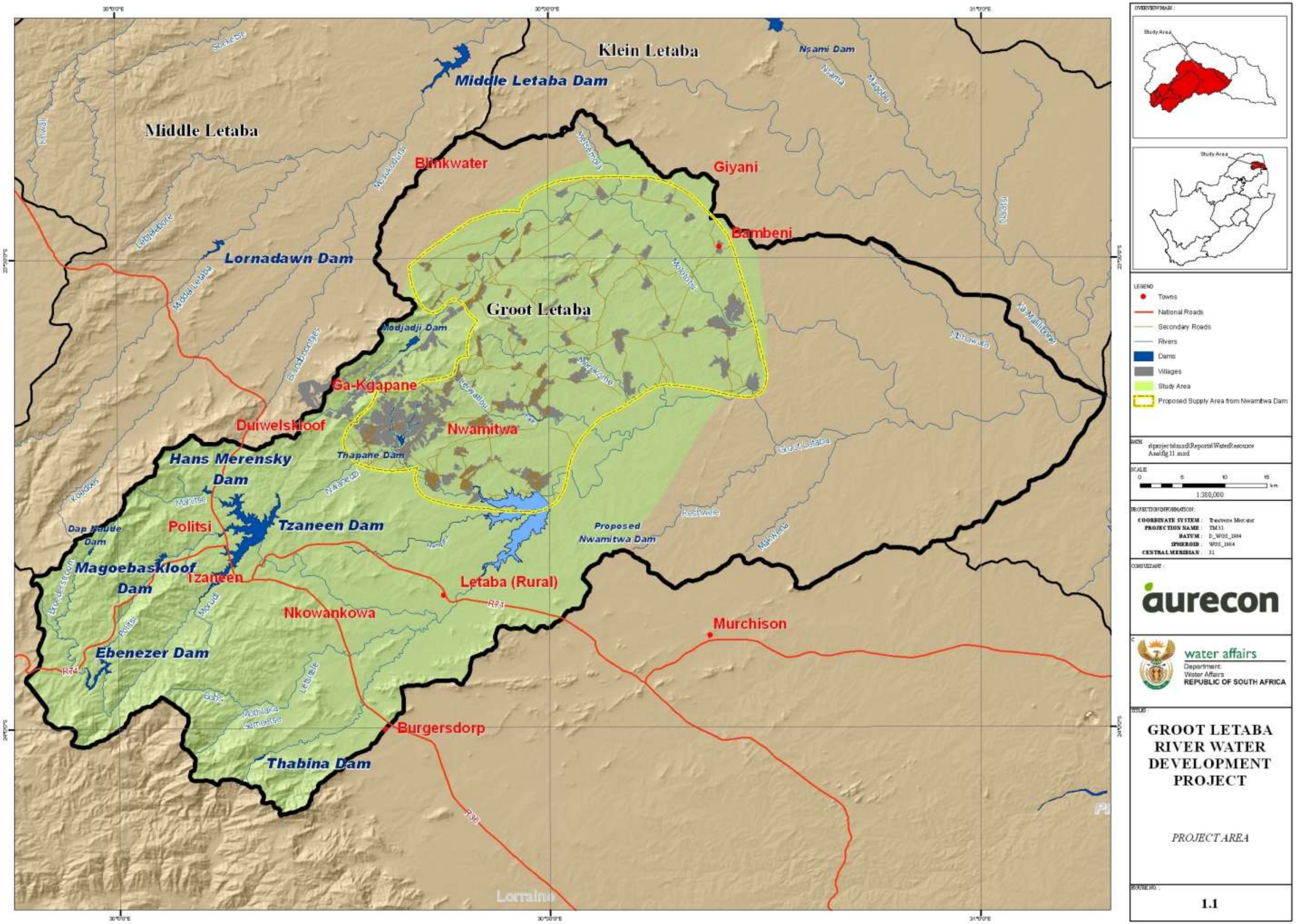
1.2 SCOPE AND ORGANISATION OF PROJECT

The Department's Directorate: Options Analysis (OA), appointed Ninham Shand in Association with a number of sub consultants (listed below) to undertake this study. The official title of the study is: "The Groot Letaba River Water Development Project: Bridging Studies: Technical Study Module".

An association exists between the following consultants for the purposes of this study:

- Aurecon (previously Ninham Shand)
- Semenya Furumele Consulting
- KLM Consulting Services
- Urban-Econ Developmental Economists
- Schoeman & Vennote

The Bridging Study comprises a number of modules, namely: an Environmental Management Module (EMM), a Public Involvement Programme (PIP), and a Technical Study Module (TSM). This Report focuses on part of the scope of work for the TSM.



Klein Letaba
Middle Letaba Dam
Nsami Dam
Middle Letaba
Blinkwater
Giyani
Bambeni
Lornadawn Dam
Groot Letaba
Modjadji Dam
Ga-Kgapane
Duiwelskloof
Nwamitwa
Thapane Dam
Hans Merensky Dam
Tzaneen Dam
Politsi
Dap Naude Dam
Magoebaskloof Dam
Tzaneen
Letaba (Rural)
Proposed Nwamitwa Dam
Nkowankowa
Murchison
Ebenezer Dam
Burgersdorp
Thabina Dam
R71
R71
R36
Lorraine
OVERVIEW MAP :
Study Area
Study Area
LEGEND
Towns
National Roads
Secondary Roads
Rivers
Dams
Villages
Study Area
Proposed Supply Area from Nwamitwa Dam
SCALE
0
5
10
15
km
1:380,000
PROJECTION INFORMATION:
COORDINATE SYSTEM : Transverse Mercator
PROJECTION NAME : TM 31
DATUM : D_WGS_1984
SPHEROID : WGS_1984
CENTRAL MERIDIAN : 31
CONSULTANT :
aurecon
water affairs
Department:
Water Affairs
REPUBLIC OF SOUTH AFRICA
TITLE :
GROOT LETABA RIVER WATER DEVELOPMENT PROJECT
PROJECT AREA
FIGURE NO. :
1.1

1.3 SCOPE OF THIS REPORT

This report deals with the preliminary design of the proposed Nwamitwa Dam. It addresses the determination of the design flood peaks used to size the spillway, the further geotechnical investigations that were undertaken to supplement the work done during previous studies, the optimisation of the dam size based on the latest annual yield figures and dam costs, the selection of the most economical spillway type and the preliminary design of the embankment, spillway and outlet works. It also addresses the costs associated with the expropriation of the required properties to be inundated by the dam as well as the preliminary design and costing of the roads in the dam basin to be relocated.

2. PRINCIPAL DETAILS OF PROPOSED NWAMITWA DAM

2.1 PRINCIPAL DETAILS

The principal details of the proposed Nwamitwa Dam are summarised in **Table 2.1**.

Table 2.1 Principal Details of Proposed Nwamitwa Dam

Classification		
Size	Large	
Hazard potential	High	
Classification	III	
Site		
Location (dam wall)	23° 45' 14.68" S 30° 29' 28.89" E	
Catchment area including Tzaneen Dam	1944	km^2
Mean Annual Runoff (MAR)	161×10^6	m^3
Probable Maximum Flood (PMF)	16 864	m^3/s
Regional Maximum Flood (RMF) (K value 5.2)	5 495	m^3/s
Regional Maximum Flood ($RMF_{+\Delta}$) (K value 5.4)	6 807	m^3/s
1:200 year RI peak inflow	3 580	m^3/s
1:100 year RI peak inflow	3 032	m^3/s
1:20 year RI peak inflow	1 903	m^3/s
Estimated average annual sediment load	350 000	m^3
Full supply level (FSL)	479.5	masl
Gross storage capacity at FSL	187×10^6	m^3
Surface area of water at FSL	2 700	ha
Firm yield	14×10^6	m^3/a
Recommended Design Flood (RDF) = 1:200 year RI routed flood peak	1 860	m^3/s
Safety Evaluation Flood (SEF) = Unrouted $RMF_{+\Delta}$	6 800	m^3/s
Dam Embankment		
Type of embankment	Earthfill	
Maximum height of embankment (above river bed level at d/s toe)	34	m
Embankment crest length (including spillway)	3 500	m
Base width of embankment at maximum cross section	126	m
Crest width of embankment	10	m
Non Overspill Crest elevation (excluding settlement allowance)	486.0	masl
Upstream slope	1V : 3H	m/m
Downstream slope	1V : 2H	m/m
Total embankment volume above original ground level	1 430 000	m^3
River Diversion		
Channel width	30	m
Discharge capacity	1 100	m^3/s

Spillway		
Type	Ogee	
Maximum height (above lowest foundation)	43.5	m
Crest length	190	m
Crest level	479.5	masl
Total freeboard	6.5	m
Design discharge	6 800	m^3/s
Elevation at design discharge	486.0	masl
Energy dissipation	Stilling basin 16m long	
Total concrete volume (including outlet works)	263 000	m^3
Outlet Works		
Pipe stacks	DN1200 x 2	mm
Upstream isolating valves	DN1200 butterfly x 5	mm
Downstream isolating valves	DN1200 butterfly x 2	mm

2.2 AREA / CAPACITY RELATIONSHIPS

The stage / area / capacity relationships are given in **Table 2.2**.

Table 2.2 Stage / Area / Capacity Relationship

Contour level (masl)	Area		Capacity million m^3
	m^2	ha	
456	131 205	13.12	0
458	234 503	23.45	0.366
460	550 894	55.09	1.151
462	859 286	85.93	2.561
464	1 677 199	167.72	5.098
466	3 104 784	310.48	9.880
468	4 855 880	485.59	17.840
470	6 938 401	693.84	29.635
472	10 917 277	1 091.73	47.490
474	14 781 747	1 478.17	73.189
476	18 636 529	1 863.65	106.608
478	22 915 524	2 291.55	148.160
480	28 353 036	2 835.30	199.428

The capacity data is represented in graphical format in **Appendix C.1**.

2.3 DISCHARGE RELATIONSHIPS

The spillway stage/discharge capacity relationship is given in **Appendix D.2**.

3. HYDROLOGY

3.1 SPILLWAY FLOODS

The Nwamitwa Dam will be a large dam (>30 m high) with a high hazard potential (due to extensive downstream developments) and will be classified as a Category III dam in terms of the Dam Safety Regulations. In accordance with Sub-Clause 3.4.2 of the SANCOLD Guidelines (SANCOLD, 1991), it was "necessary to perform hydrological calculations appropriate to the site" for a Category III dam.

The recommended floods for the sizing of the spillway have initially been selected in accordance with the SANCOLD Guidelines to be as follows:

- The Recommended Design Flood (RDF) was the 1 in 200 year recurrence interval (RI) flood.
- The Safety Evaluation Flood (SEF) was the Probable Maximum Flood (PMF) (Sub-Clause 5.2.2).

Further justification for the selection of the PMF as the SEF could be found in ICOLD Bulletin 59 (ICOLD, 1987). Sub-Clause 3.2.2 states that "All available hydrometric and pluviometric data should be taken into account when determining the design flood. Probabilistic and/or deterministic methods, such as the Probable Maximum Flood (PMF), may be used. The latter should derive from the combination of maximum precipitation with maximum runoff conditions and is to produce the design flood hydrograph."

Flood peaks were also determined for the 1 in 100 year RI flood to determine expropriation levels in the dam basin.

The following calculation methods were used:

- Unitgraph techniques using dimensionless regional unitgraphs (HRU, 1972).
- Probabilistic (flood frequency) techniques using a range of probability distributions. The closest streamflow gauge to the proposed Nwamitwa Dam is Gauge B8H009 on the Groot Letaba River approximately 20 km upstream of the proposed dam site.
- Empirical flood techniques in the form of the Francou-Rodier approach, used by Kovacs to develop the Regional Maximum Flood (RMF) peak (Kovacs, 1988).

The results of the flood analysis are shown in **Table 3.1**.

Table 3.1 Comparison of Inflow Flood Peaks (m^3/s)

Flood	Regional Unit Hydrograph Technique (1)	Flood Frequency Analysis	RMF(2) Approach
1:100 year RI	3 032	3 279	3 143
1:200 year RI	3 580	4 900	3 775
RMF (Region 5.2)	n/a	n/a	5 495
$RMF_{+\Delta}$ (Region 5.4)	n/a	n/a	6 807
PMF	16 864	n/a	n/a

(1) Represents maximum inflow flood peak for critical storm duration. Upstream attenuation accommodated.
(2) The RMF flood peaks do not take any upstream flood attenuation into account.

It is evident from **Table 3.1** that the PMF is significantly higher than the RMF and the $RMF_{+\Delta}$, with the PMF in the order of 3 times as high as the RMF. This relatively high PMF/RMF ratio confirms the results of Görgens *et al* (2006), who, as part of a Water Research Commission (WRC) Study on Extreme Design Floods, investigated PMF/RMF ratios at 109 flow gauging stations across South Africa and found that at 46 out of 51 gauging stations and dam sites in Limpopo, Gauteng, North-West, Mpumalanga and KwaZulu-Natal, the PMF/RMF ratio exceeds 2.0.

As stated above, both the SANCOLD Guidelines and the ICOLD Bulletin 59 specifically mentions the use of the PMF method in designing spillways for dams. However, in the case of the PMF approach being followed, the SANCOLD Guidelines also recommend upper limits of 6.0 and 2.0, respectively, to the PMF K-value and PMF/RMF ratio. Given that the PMF K-values and PMF/RMF ratios for the Nwamitwa Dam site, as determined during this study, are quite high in comparison with these upper limits, and taking cognisance of the HRU 1/72-based PMF-related concerns expressed in the findings of the abovementioned WRC Study, the use of a SEF lower than the PMF-routed values determined during this study, but higher than the RMF, is recommended as an alternative to the HRU 1/72-based PMF.

As it was not possible, under this Feasibility Study, to do any fresh research on extreme rainfall-versus-flood patterns in the region of the Groot Letaba catchment, a lead was taken from the SANCOLD Guidelines, which specifies the use of a Safety Evaluation Discharge (SED) for safety assessments on existing dams. According to the Guidelines the dam spillway must be capable of discharging the SED so that, although there may be extensive damage to the structure, it will not fail. For the

"Large Dam/Significant to High Hazard" category (in which Nwamitwa Dam falls), the SED is set as the $RMF_{+\Delta}$, i.e. the RMF for the region one step higher numerically than that in which the study catchment lies; in this case for K = 5.4. It is therefore recommended that the unrouted $RMF_{+\Delta}$ value of 6 800 m^3/s be used as an alternative SEF to the outgoing flood peak of an HRU 1/72-based PMF for the preliminary spillway design for Nwamitwa Dam. This implies an inflowing flood peak, before attenuation, of about 8 900 m^3/s for a dam with a FSL of 479.5 masl and a spillway length of 200 m. The K-value of such a flood peak is 5.6 and its ratio over RMF is 1.62.

For the 1 in 100 year RI and 200 year RI floods at Nwamitwa Dam, the floods as determined in accordance with the HRU 1/72 regional unit hydrograph method, are recommended. The order of magnitude of these design floods were broadly confirmed by means of a flood frequency analysis and through application of the empirical RMF technique. The simulated 1 in 100 year RI and 200 year RI flood hydrographs for a range of storm durations were routed through the proposed Nwamitwa Dam in order to determine the effect of attenuation on the simulated flood peaks.

Based on the results of the above analyses, the following spillway floods are proposed for the 100 year RI scenario, the RDF and the SEF:

Table 3.2 Recommended Spillway Floods

Design flood	**100 year RI**			**RDF (200 year RI)**			**SEF ($RMF_{+\Delta}$)**		
Dam capacity	**FSL (masl)**			**FSL (masl)**			**FSL (masl)**		
	473.5	**477.5**	**479.5**	**473.5**	**477.5**	**479.5**	**473.5**	**477.5**	**479.5**
Spillway length (m)	200								
Peak Outflow (m^3/s)	2048	1701	1551	2447	2035	1863	6800	6800	6800
Spillway length (m)	400								
Peak Outflow (m^3/s)	2447	2114	1938	2952	2535	2340	6800	6800	6800

3.2 DIVERSION FLOODS

In addition to the spillway design floods, 1 in 10 year, 20 year and 50 year RI flood hydrographs for a range of durations were estimated at the proposed Nwamitwa Dam site. **Table 3.3** provides a summary of the range of flood peaks and flood volumes that were calculated.

Table 3.3 River Diversion Floods

	Duration		
	8h	**20h**	**34h**
1 in 10 year RI			
Flood Peak (m^3/s)	1172	1504	1239
Flood Volume (Mm^3)	17.2	28.2	32.2
1 in 20 year RI			
Flood Peak (m^3/s)	1522	1903	1561
Flood Volume (Mm^3)	23.0	36.8	42.0
1 in 50 year RI			
Flood Peak (m^3/s)	2200	2516	2080
Flood Volume (Mm^3)	32.6	49.5	55.8

3.3 SEDIMENTATION

A review of the expected sedimentation rates at the proposed Nwamitwa Dam was undertaken by Prof Gerrit Basson from the Stellenbosch University. The full report is included in **Appendix A.2**, which includes a list of the references quoted below.

The following methodology was followed:

a) Review of findings of previous studies
b) Analysis of sediment yields of existing dams on the Groot Letaba and other rivers in the region
c) Analysis of sediment yields based on suspended sediment data observed on the Groot Letaba River

The three methods yielded the following sediment yields:

a) Rooseboom (1992) regional empirical method: 280 $t/km^2.a$ at proposed dam site

b) Reservoir basin surveys: 245 to 293 $t/km^2.a$

c) River suspended sediment samples: 278 $t/km^2.a$ at Letaba Ranch

The three methods resulted in very similar sediment yields. (The method in (a) is based on data of (b); method (c) had very limited suspended sediment data and the sediment load-discharge relationship had to be extrapolated for larger floods).

The future land use could affect the sediment yield. The current land use consists mainly of forestry, irrigated commercial farming, urban areas and subsistence farming. The catchment area of the Nwamitwa Dam falls in the high and medium soil erosivity regions of the Rooseboom (1992) method. If due to future land degradation the medium region changes to high erosivity, the maximum possible sediment yield would be 350 t/km^2/a based on a 95 percentile assurance. It is recommended that a sediment yield of 350 t/km^2.a is used for the design of Nwamitwa Dam.

Based on the Brune (1953) sediment trapping efficiency relationship, it was assumed the proposed reservoir would trap 100% of the incoming sediment load. The sediment density of deposited sediment was assumed to be 1,35 t/m^3 after a 50 year period. An effective catchment area of 1 352 km^2 downstream of the Tzaneen Dam was used for Nwamitwa Dam. Based on the abovementioned assumptions, the anticipated sediment volume after 50 years is estimated to be 17,53 million m^3.

Table 3.4 Estimated Nwamitwa Reservoir Sedimentation

Sediment yield (t/km^2.a)	Effective catchment area (km^2)	Estimated sediment volumes (million m^3)		
		After 10 years	After 20 years	After 50 years
350	1 352*	6.92	11.49	17.53

Note: * From the Rooseboom (1994) study.

3.4 BACKWATER CALCULATIONS

A backwater analysis was carried out to determine the expropriation line.

The expropriation line, which depicts the minimum land purchase requirements due to dam construction, was determined according to the *Policy and Guidelines for the Acquisition of Land Rights at Departmental Dams* (DWAF, 2001). This document defines the expropriation line as the minimum of 1.5 m vertically above the 1 in 100 year RI flood line or at least 15 m horizontally outside the 1 in 100 year RI flood line, whichever results in the greater horizontal distance.

The 1 in 100 RI year flood line baseline information, which is needed for developing the expropriation line, was determined through backwater analysis of the system of rivers flowing into Nwamitwa Dam with the aid of an unsteady HEC-RAS (version 4.0) model. Further details are provided in **Appendix A1.3**.

The expropriation line is shown on Drawing 401775 CEN 200 in **Appendix H.4**.

3.5 RIVER DIVERSION STRATEGY

3.5.1 Selection of river diversion floods

In Section 3.2 the river diversion floods were calculated to be between 1 500 and 2 500 m^3/s depending on the chosen recurrence interval. Due to the significant size of these floods, it is proposed that the floods be scaled down as described below.

In addition to the spillway design floods, 10 year, 20 year and 50 year return interval (RI) flood hydrographs for a range of durations were estimated at the proposed Nwamitwa Dam site. The river diversion floods were calculated to be between 1 500 and 2 500 m^3/s depending on the chosen recurrence interval. Due to the significant size of these floods, it is proposed that the floods be scaled down as described below.

As the Tzaneen Dam does not frequently overflow, it is considered reasonable to assume that floods with low recurrence intervals that are generated in the catchment area upstream of Tzaneen Dam would be partially absorbed by Tzaneen Dam.

This implies that low recurrence interval river diversion floods at Nwamitwa Dam would mainly be generated from the incremental catchment downstream of Tzaneen Dam. As this phenomenon would be reflected in the peak flow record at Gauge B8H009, the flood frequency analysis at this gauge implicitly already accounts for this absorption. Based on this assumption, the calculated 1:10 year flood peak at Gauge B8H009 was scaled up based on the ratio of the square root of the incremental catchment downstream of Tzaneen Dam (1 294 km^2) to the incremental gauge catchment downstream of Tzaneen Dam (201 km^2). For the 1:50 year recurrence intervals flood peaks, it was assumed that although some flood absorption might occur in Tzaneen Dam, some spilling might also occur. For this flood event, the calculated flood peak at Gauge B8H009 was adjusted based on the full catchment area at Nwamitwa Dam and at the gauge respectively, i.e. $\sqrt{1944}$ / $\sqrt{851}$. For the 1:20 year peak, both methods were applied and the average value was then calculated.

The scaled river diversion floods at Nwamitwa Dam therefore results in the following figures:

- 1:10 yr 1 000 m^3/s
- 1:20 yr 1 450 m^3/s
- 1:50 yr 2 100 m^3/s

The Design Criteria Memorandum (DCM) calls for river diversion floods with a 1 in 20 year RI for a composite dam consisting of both concrete and earth fill sections and with a 1 in 50 year RI for an earth fill embankment. The first stage river diversion must protect the spillway foundation construction works, and the diversion works were therefore sized to handle a 1 in 20 year flood of 1 450 m^3/s. The second stage river diversion is required to protect the earth embankment during construction and has been designed to accommodate the 1 in 50 year flood of 2 100 m^3/s. It is recommended that the second stage of the river diversion strategy commence at the onset of the dry season in order to facilitate the installation of a diversion culvert 4 m wide by 3 m high with its invent at the river bed level of 454 masl in the middle of the spillway section. On the right hand side of the culvert a 60 m wide section of the spillway must be kept 3 m lower than the rest of the spillway for the duration of the spillway construction. The discharge capacity of the low section will be approximately 500 m^3/s, which will allow for the passing of floods during the dry season. The diversion culvert will keep the upstream water level at approximately the river bed level during normal dry season flow. As the dam is raised, the flood absorption capacity of its basin will increase. The 1 in 20 year flood volume will be absorbed when the embankment is at level 469 masl, and the 1 in 50 year flood volume when it is at level 470,4 masl.

3.5.2 First stage

The proposed river diversion strategy is shown on Drawing No's 401775 CEN 213 and 214 in **Appendix H**.

A 30 m wide diversion channel will be excavated through the left abutment beyond the spillway section. The channel will be extended through the 'nose' between the two rivers with an assumed split between the Groot Letaba and Nwanedzi Rivers of approximately 2/3 : 1/3 based on the respective catchment areas. Two 8 m high coffer dams will be constructed in the two rivers.

A further coffer dam will be constructed downstream of the dam. The proposed alignment of the downstream coffer dam diagonally across the Groot Letaba River will allow for the excavation of most of the spillway return channel.

The proposed size of the diversion channel was confirmed by water profile calculations (see **Appendix D.5**). A total channel length of 2 500 m comprising 11 river and diversion channel sections were analysed. Manning's n-values of 0.035 were used for sections that cover the river reach and 0.025 for sections that cover the diversion channel. A sensitivity analysis was also carried out and it was found that if the coffer dams were to be raised to 10 m high, the diversion works would be able to discharge a flow of 2 300 m^3/s.

3.5.3 Second stage

The second stage of the river diversion strategy will commence at the onset of the dry season. A diversion culvert 4 m wide by 3 m high will be installed at level 454 masl in the middle of the spillway section. On the right hand side of the culvert a 60 m wide section of the spillway will be kept 3 m lower than the rest of the spillway. The discharge capacity of the low section will be approximately 500 m^3/s, which will allow for the passing of floods during the dry season. The diversion culvert will keep the upstream water level at approximately the river bed level.

The upstream coffer dam in the Groot Letaba River will be replaced by a coffer dam between the 'nose' between the two rivers and the spillway section. The downstream coffer dam will be removed. The diversion channel through the left abutment will be blocked off by two coffer dams at the upstream and downstream ends.

As the invert level of the low section of the spillway increases, the flood absorption capacity of the dam basin will increase. The flood volume of the 1 in 20 year RI flood of approximately 24 million m^3 (37 million m^3 - 34%) would be absorbed by the dam basin when the embankment is at level 469 masl. Similarly the flood volume of the 1 in 50 year RI flood of approximately 33 million m^3 (50 million m^3 - 34%) would be absorbed by the dam basin when the embankment is at level 470.4 masl.

4. GEOLOGY AND GEOTECHNICS

4.1 PREVIOUS GEOLOGICAL INVESTIGATIONS

A number of previous engineering geological studies for the proposed Nwamitwa Dam had been conducted. These are briefly summarized below, in chronological order.

A **reconnaissance-level appraisal** of the proposed dam site, then known as the Janetsi site was conducted initially (Council for Geoscience, 1984). The investigation comprised a desk study followed by a brief site visit during which an assessment of likely geological conditions along the proposed centre-line was made. Possible sources of construction materials were listed, as derived from an earlier broad study in the area.

Some years later this was followed by **feasibility-level engineering geological investigations** which were conducted in 1996 (Council of Geoscience, 1997) during which a total of seventeen rotary core boreholes were drilled (numbered BH1001 to BH1017), at two possible centre-lines. Investigations shifted to the upstream centre-line after initial boreholes at the downstream site revealed unfavourable conditions.

The report provided an assessment of expected excavation depths for a central concrete spillway section as well as typical excavation depths for the core trench on the embankments and included geological sections along the respective centre-lines. No investigations of potential sources of construction materials were conducted, although various sources of both fine (i.e. sand) and coarse aggregate were identified.

A **materials investigation** was conducted by the DWA Materials Laboratory in 1996 (DWAF, 1996a). Adequate earthfill material was identified on the right bank of the Groot Letaba River immediately upstream of the dam site for the construction of the earthfill sections of the proposed dam. The impervious material is a weathered dolerite and the semi-pervious material is a weathered granite.

Sand deposits were identified at two separate borrow areas; the first 20 km downstream of the dam site in the Merekome River and the second 11 km downstream of the dam site in the Phatle/Lerwatlou River.

No investigation was done to determine the availability of coarse concrete aggregate or rip-rap material.

4.2 GENERAL GEOLOGICAL SETTING

The area of interest is underlain by Mesoaechean granitoid gneisses, specifically the Groot Letaba Gneiss (previously Goudplaats Gneiss) which has been intruded by younger diabase dykes. No major faults occur in the vicinity of the dam but a number of lineaments are present. The level of seismic hazard may be described as moderate.

At the dam site, shallow colluvial soils cover the left flank while the right flank is partly covered by reworked alluvial gravels and the remainder by colluvial sands; underlain by thin residual soils. Thick alluvial deposits occur within the river section. The granite bedrock generally occurs at shallow depth on the left flank but is deeply and variably weathered. On the right flank the bedrock is generally moderately or highly weathered. Within the river section the underlying bedrock is generally unweathered.

4.3 FURTHER GEOTECHNICAL INVESTIGATIONS

4.3.1 Description

As part of this study, the following further geotechnical studies were undertaken:

- Desk study of available geological information,
- Field mapping,
- Geophysical surveys,
- Additional rotary core drilling,
- Test pitting,
- Water pressure (Lugeon) testing and measurement of the water table,
- Laboratory testing, and
- Seismic hazard assessment.

The results of the above investigations are presented in **Appendix B** as a separate volume to this report.

4.3.2 Desk study and field mapping

In addition to the available geological information, aerial photograph interpretation (API) was conducted to identify major structural features, such as faults or lineaments, etc, which could negatively impact on the founding of the dam. This information as well as field mapping of the general footprint of the proposed dam was used to produce an engineering geological site plan (Drawing No 103577-G1-003 in **Appendix B**).

4.3.3 Geophysical surveys

Geophysical surveys were conducted at the proposed dam site as well as the proposed quarry sites by Engineering and Exploration Geophysical Services cc (EEGS).

Three geophysical techniques were employed for this geophysical survey, namely surveys of seismic velocity, electro-magnetics as well as a magnetic survey. Instruments utilised included a Smartseis seismograph, a proton magnetometer and a Geonics EM34-3.

4.3.4 Rotary core drilling

A further twelve rotary core boreholes were drilled by the DWA Drilling Branch, in the period September 2007 to October 2008. Boreholes NBH 1201 to NBH 1213 were drilled along the dam centre-line (Drawing No 103577-G1-003 in **Appendix B**). Boreholes on the upper flank areas were drilled vertically, while boreholes on the respective river banks were angled into the river section. In addition to the boreholes drilled on the dam footprint, a further six rotary boreholes were drilled at two possible hard rock quarry sites (numbered NBH 3001 to NBH 3003, and NBH 3101 to NBH 3103) as possible sources of coarse aggregate / rip-rap material. These quarry sites are shown on Drawing No 103577-G1-001 in **Appendix B**).

The new boreholes were primarily drilled to fill in gaps from the previous investigations. Founding conditions for an envisaged side-channel spillway and stilling basin on the left flank were investigated by three boreholes (NBH 1202 to NBH 1204). An additional borehole (NBH 1213) was drilled to investigate founding conditions at the position of an anomaly identified during the geophysical survey.

For the dam foundations, expected founding depths for either the mass concrete spillway or the earthfill embankment, were assessed on the basis of empirically determined Deformation Modulus (Emod) values for the founding rock mass. Lugeon (packer) tests were used to assess the permeability of the founding rock mass which proved to mainly be impervious.

4.3.5 Test pitting

Representative samples were recovered from the test pits on the dam footprint and submitted to the DWA Materials Laboratory for testing. Testing of the disturbed samples comprised determination of foundation indicators (i.e. grading analyses including both sieve and hydrometer analyses, as well as determination of the Atterberg limits), and the determination of compaction properties, shear strengths and permeabilities.

Geological conditions along the proposed relocated sections of road were characterised using test pits and laboratory testing, and the suitability of the respective layers for use in the road construction was assessed.

4.3.6 Seismic hazard assessment

Published seismic hazard maps of Southern Africa (Kijko *et al*, 2003) indicated a peak ground acceleration (PGA) in the order of 0.11g to 0.13g for the area in which the proposed Nwamitwa Dam is located. This may be considered a moderate level of seismic hazard.

A seismic hazard assessment (SHA) for the proposed Nwamitwa Dam was conducted by the Council for Geoscience (CGS) (See **Appendix B**). The probabilistic seismic hazard assessment was performed using the classical Cornell-McGuire procedure; with earthquake recurrence parameters calculated using the procedure of Kijko and Sellevoll.

The PGA was calculated for the Operating Basis Earthquake (OBE) as well as the Maximum Credible Earthquake (MCE), where the OBE is defined as an earthquake having a 50% probability of exceedence in 100 years, i.e. a recurrence interval of 144 years (ICOLD, 2003). The recurrence interval of the MCE is considered to be 10 000 years. Four attenuation models were used with the resultant OBE values falling within the range of 0.01g to 0.03g. The MCE values fall in the range 0.09g to 0.18g.

In accordance with the CGS report the results from the attenuation model by Toro *et al* were adopted, namely an OBE value of 0.024g and a MCE value of 0.14g (see Section 6 of CGS report in **Appendix B**).

5. MATERIALS

5.1 PREVIOUS INVESTIGATIONS

A materials investigation was conducted by the DWA Materials Laboratory in 1996 (DWAF, 1996a). Adequate earthfill material for a dam with a FSL of 477.5 masl was identified on the right bank of the Groot Letaba River immediately upstream of the dam site for the construction of the earthfill sections of the proposed dam. The impervious material is a weathered dolerite and the semi-pervious material is a weathered granite. The report is included in **Appendix B** of this study.

5.2 IMPERVIOUS MATERIAL

The weathered dolerite to be used as impervious material in the core zone of the embankment yielded the following average parameters:

Liquid Limit	38.9%
Plasticity Index	18.6
Linear Shrinkage	9.0%
Maximum Dry Density (Standard Proctor Compaction)	1592 kg/m^3 to 1810 kg/m^3
Optimum moisture content	15.1% to 23.5%
Coefficient of Permeability	3.0 x 10^{-8} cm/sec
Internal Angle of Friction	24.6°

5.3 SEMI-PERVIOUS MATERIAL

The weathered granite to be used as semi-pervious material in the general fill zone of the embankment yielded the following average parameters:

Liquid Limit	30.9%
Plasticity Index	14.0
Linear Shrinkage	6.5%
Maximum Dry Density (Standard Proctor Compaction)	1931 kg/m^3 to 2060 kg/m^3
Optimum moisture content	10.2% to 12.6%
Coefficient of Permeability	3.0 x 10^{-8} cm/sec
Internal Angle of Friction	37.0°

5.4 FINE AGGREGATE (SAND)

Sand deposits were identified at two separate borrow areas; the first 20 km downstream of the dam site in the Merekome River and the second 11 km downstream of the dam site in the Phatle/Lerwatlou River.

The samples comprise quartzitic river sand. All samples tested complied with the SANS specifications for fine aggregate (SANS 1083:1994).

If required, additional sources for fine aggregate do occur. Only four samples were recovered from the two areas for testing; it is likely that additional sand reserves occur within the same rivers.

Extensive sand deposits were also noted to occur within the Molototsi River, located approximately 35 km from the dam site. No sampling of this source has been conducted to date. Other sand deposits have been noted along the route of the bulk water pipeline. These deposits were sampled to determine their suitability for use as backfill materials.

5.5 AVAILABLE VOLUMES OF MATERIAL

A comparison between the required and available volumes of the embankment materials are presented in **Table 5.1**.

Table 5.1 Available Volumes of Embankment Material

Embankment material	Volume to be proven* (m^3)	Available volume (m^3)
Impervious	1 640 000	952 000
Semi-pervious	1 340 000	1 366 000**
Filters	120 800	162 000

* Required volume x 2

** Borrow area 936 000 m^3 + essential excavations 430 000 m^3

From the above table it can be seen that a further 690 000 m^3 of impervious material needs to be proven.

6. EMBANKMENT

6.1 INTRODUCTION

The valley shape factor for the proposed Nwamitwa Dam site is in excess of 50, which is a clear indication that the most appropriate dam type would be an embankment type dam.

6.2 STORAGE CAPACITY

During the optimization of the proposed dam size, the following dam sizes were investigated:

Table 6.1 Selected Dam Sizes

Dam size	Capacity (Million m^3)	FSL (masl)	NOC (masl)
0.41 MAR*	66	473.5	480.0
0.85 MAR*	137	477.5	484.0
1.16 MAR*	187	479.5	486.0
1.50 MAR*	241	481.5	488.0

* Based on natural incremental MAR between Tzaneen and Nwamitwa Dams = 160.9 million m^3

The costing of the various dam sizes was based on an earthfill embankment dam with a 400 m long central ogee spillway, being sufficient to discharge the routed PMF (the spillway was subsequently reduced to 190 m long to discharge the RMF). The costing schedules are included in **Appendix C.1**.

The expropriation costs up to the 1 in 100 year RI flood level were determined by Schoeman & Vennote as shown in the letter dated 18 September 2007 in **Appendix F.4**.

The results of the optimisation exercise are shown in **Appendix C.1** and are summarised in **Table 6.2**.

Table 6.2 Optimisation of Dam Sizes

Dam size (Factor of MAR*)	Dam construction costs ** (R million)	Expropriation Area (ha)	Expropriation costs*** (R million)	Total Project Cost (R million)	Yield Mm^3/a	Unit Cost $R/m^3/a$
						Dam construction and land costs
0.41	524	800	92	616	4	153.91
0.85	624	1600	137	761	9	84.56
1.16	682	2250	164	846	14	60.43
1.50	753	2900	212	965	17	56.76

Figure 6.1 shows the relationship between the capacity of alternative dam sizes and the increase in historical firm yield from the system derived from each dam capacity. Whilst the analysis did not point to "one" optimal size, a dam with a FSL of 479.5 m and a historical firm yield of 14 Mm^3/a was proposed as the preferred dam size. This size will ensure that sufficient yield is obtained in order to meet the anticipated future water requirements of the area surrounding Nwamitwa Dam, limit expropriation costs and limit the amount of evaporation from the proposed dam.

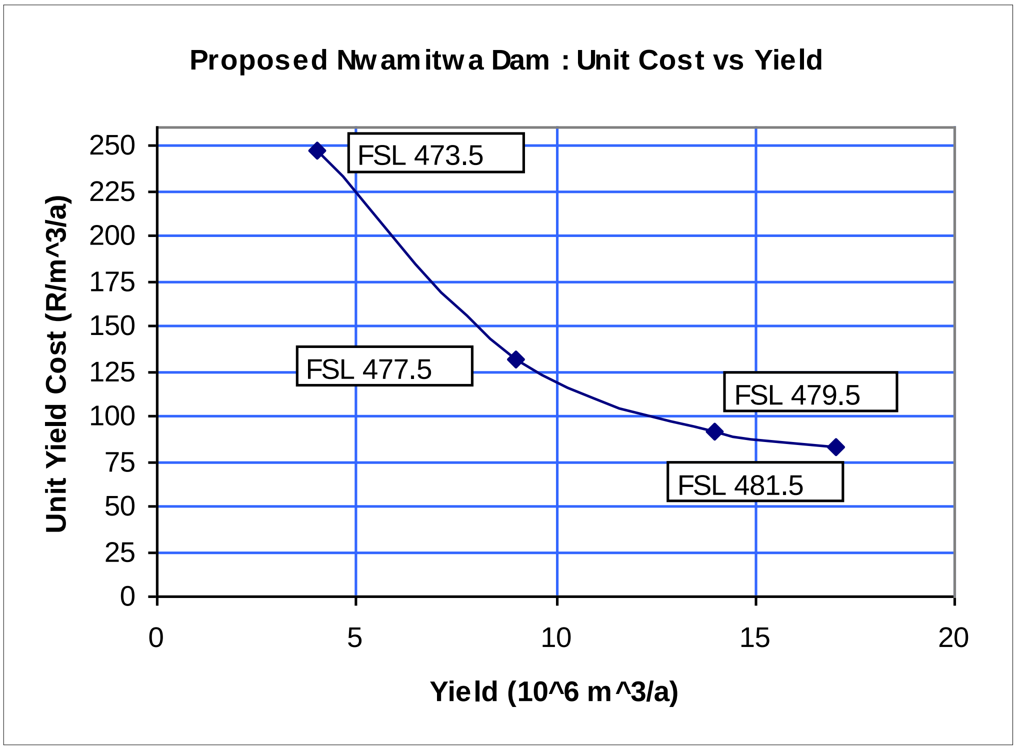


Figure 6. 1 Nwamitwa Dam: Unit Cost vs Yield Curve

6.3 HORIZONTAL ALIGNMENT

Two centre-lines were investigated during the Feasibility Study. The upstream centre-line has been retained for this study as it will prevent flooding of Nkambako water treatment works on the left flank and the foundation rock in the river section appears to be shallower. The alignment was extended on the left flank and re-routed on the right flank to accommodate the higher Non Overspill Crest (NOC) level of 486 masl compared to the NOC level of 480 masl used during the Feasibility Study.

6.4 CROSS SECTION

6.4.1 Non overspill crest

The width of the NOC depends on considerations such as the following (Design of Small Dams, 1987):

- the nature of the embankment materials and the minimum allowable percolation distance through the embankment at FSL,
- the height and importance of the structure,
- the possible roadway requirements, and
- the practicability of construction.

A further general guideline is to make the width equal to dam height H/5 (34/5 = 6.8 m). DWA expressed preference for a minimum width of 10 m to allow for easy movement of construction and maintenance plant.

6.4.2 Core zone

The impervious material for the core zone will be obtained from a weathered dolerite borrow area on the right bank immediately upstream of the dam site. Approximately 1.2 times the required volume had been proven during the DWA materials investigation in 1996 and some further investigations would be required to prove at least 2 times the required volume.

6.4.3 Cut-off trench

The minimum width of the cut-off trench below the embankment was selected to be equal to H/2 (Jansen, 1988). The cut-off trench will be founded on rock with a Rock Mass Rating (RMR) of 20 or higher.

The expected depths of the cut-off trench are as follows:

- Left abutment — Between 2 and 3 m on average up to 10 m near spillway
- Right abutment — Between 5 and 7 m on average up to 15 m near Ch 1900

6.4.4 General fill zone

The semi-pervious material for the general fill zone will be obtained from a weathered granite borrow area on the right bank immediately upstream of the dam site. Sufficient material will be available from the borrow area as well as essential excavations to comply with the required volume x 2 criterion.

6.4.5 Chimney and blanket drains

A 1 000 mm thick vertical chimney drain and a 600 mm thick horizontal blanket drain will be provided. The blanket drain will terminate in a rock toe with an integral toe drain. The chimney drain will also be provided against the downstream inclined face of the cut-off trench to intercept any seepage through the cut-off trench. Adequate sources of filter sand have been proven during the DWA materials investigation (see Section 5.5).

6.4.6 Upstream slope protection

The upstream slope will be protected from erosion due to wave action by placing a layer of rip-rap on the slope. According to **Appendix C.5** the significant wave height (Hs) is 1.24 m. The maximum wave height is 1.8 x Hs = 2.23 m. The recommended rip-rap specification for such a wave height is as follows (Thomas, 1962):

- D_{50} — 500 mm
- Maximum rock — 1 100 kg
- Layer thickness — 800 mm (1 000 mm provided)

In order to comply with filter criteria a 300 mm thick transition layer needs to be placed between the rip-rap and the underlying general fill. The transition layer will have to be imported from sources off site.

6.4.7 Downstream slope protection

The downstream slope will be protected from erosion due to rain by placing a 250 mm thick layer of continuously graded aggregate (6 to 75 mm) on the slope.

6.5 FILTER CRITERIA

Filter criteria have been applied to the following interfaces in accordance with the publication *Filters and Leakage Control in Embankment Dams* (Sherard and Dunnigan, 1985):

- Core zone to chimney drain
- Blanket drain to transition zone
- General fill to transition zone

The grading envelopes for the core and general fill zones, chimney and blanket drains and the transition zones are shown in **Appendix C.3**. The data for the first three zones was obtained from the DWA Materials Laboratory Report (**Appendix B**). The envelope for the transition zone is similar to typical G3 gravel.

The filter criteria are summarised in **Table 6.3**.

Table 6.3 Filter Criteria

Interface	Criteria	Actual Factor f	Recommended Factor f	Comments
Core / Chimney	D_{15} of chimney / D_{85} of core	0.02 to 1.9	f<5	-
	D_{50} of chimney / D_{50} of core	0.42 to 144	f<25	Complies partially
	D_{15} of chimney / D_{15} of core	17 to 330	5<f<40	Complies partially
Blanket / Transition	D_{15} of transition / D_{85} of blanket	0.18 to 2.1	F<5	
	D_{50} of transition / D_{50} of blanket	7.7 to 40	f<25	Complies partially
	D_{15} of transition / D_{15} of blanket	0.73 to 11	5<f<40	Complies partially
General fill / Transition	D_{15} of transition / D_{85} of gen fill	0.03 to 5.6	f<5	-
	D_{50} of transition / D_{50} of gen fill	2.5 to 165	f<25	Complies partially
	D_{15} of transition / D_{15} of gen fill	2.5 to 236	5<f<40	Complies partially

General compliance with the critical D_{15}/D_{85} criteria is achieved. The D_{50}/D_{50} criterion is a measure of the similarity between the respective grading curves and is not considered critical with regard to piping (Sherard, *et al,*1984). The partial compliance of the D_{15} of chimney / D_{15} of core criterion means that some of the D_{15} particles of the core may be washed out. This will occur next to the chimney drain where a transition zone will be created preventing any further loss of material. The partial compliance of the D_{15} of general fill / D_{15} of transition criterion means that some of the D_{15} particles of the general fill may be washed out. This will occur on the upstream slope of the embankment where a transition zone will be created preventing any further loss of material.

6.6 STABILITY ANALYSIS

6.6.1 Shear strength parameters

The different zones of the embankment are shown on Drawing No 401775 CEN 212 in **Appendix H**. The material properties as obtained from the DWA Materials Laboratory Report in **Appendix B** and precedent information are given in below.

Table 6.4 Properties of Embankment Construction Materials

Zone	Material	Field Density (kN/m^3)	Cohesion C (kPa)	Internal angle of friction ϕ (degrees)
Core (OMC)*	Weathered dolerite	19.2	5	23º
General fill (OMC)*	Weathered granite	19.5	3	33º
Chimney and blanket drain	Cohesionless sand	18.6	0	31º
Rock toe	Broken granite	22.6	0	40º
Foundation	Weathered granite	19.5	3	29º

* OMC = Optimum Moisture Content

Due to the relatively sandy nature of the fill materials and the moderate rate of placing for an embankment of this size, it is not expected that there will be a significant build-up of pore-water pressures. The following B-bar values will be used:

- Core zone 0.3 to 0.4
- General fill zone 0.2

The B-bar values give the water pressure at the base of a slice and are multiplied by the height of the slice to obtain the pore-water pressure.

6.6.2 Results

The stability analyses were carried out using a computer program called SLOPE/W (GEO-SLOPE, 2004a). The analyses were based on the Morgenstern-Price method, with pore pressure conditions being defined by the phreatic surface.

The embankment was analysed at the section with the maximum height, upstream and downstream.

The phreatic surface estimated for the analyses under steady - state seepage conditions was based on the reservoir water level at FSL, as it is not expected that full steady - state seepage would develop for the short durations that the water level would exceed FSL under flood conditions.

The pore-pressures allowed for in the rapid drawdown condition were based on an assumed drawdown of the phreatic surface from FSL to the upstream slope of the general fill zone below the rip-rap and transition layers.

Various conditions for both the upstream and downstream embankment slopes were analysed and the desired minimum factors of safety were selected based on the following three categories:

- Usual conditions
- Unusual conditions
- Extreme conditions

The results of the analysis are summarised in **Table 6.5** below and the critical slip circles are shown in **Appendices C.4.1** to **C.4.7**.

Table 6.5 Stability Analysis Results

Condition		Category	Minimum F.O.S.	
			Desired	Obtained
Downstream:	Reservoir full	Usual	1.50*	1.58
	End of construction	Unusual	1.40**	1.48
	Reservoir full + MCE	Unusual	1.00*	1.16
Upstream:	Reservoir full	Usual	1.50*	1.84
	End of construction	Unusual	1.40**	1.63
	Rapid drawdown	Unusual	1.25*	1.19
	Rapid drawdown + MCE	Extreme	1.00*	0.89

* (Jansen, 1988)
** DWAF

As can be seen from the above, the factors obtained are all satisfactory, except for the rapid drawdown and rapid drawdown + MCE conditions. The F.O.S. for the latter condition indicates that there is likely to be some damage to the upstream face of the dam, but a combination of these loading conditions is extremely unlikely. The reservoir would be empty at this stage so overall failure of the structure is unlikely and there would be little risk to downstream developments.

6.7 GROUTING

Curtain grouting will be taken to at least $^{2}/_{3}$ of the height of the dam at any particular point, with a minimum depth of 10 m. Primary holes will be spaced at 6 m intervals with allowance for secondary and tertiary holes over 50% of the area. Grouting will be done until a Lugeon value of less than 3 has been achieved.

Consolidation grouting will be low pressure grouting undertaken in the core trench to a depth of 3 m with holes drilled 1.5 m either side of the grout curtain in a staggered pattern at 3 m centres. Below the spillway the consolidation grouting will be done over the complete foundation at 3 m centres in both directions.

6.8 FREEBOARD

6.8.1 Introduction

The required freeboard above the FSL of the dam was determined in accordance with the publication, “Interim Guidelines on Freeboard for Dams” (SANCOLD, 1990).

6.8.2 Water levels in dam

The maximum water level in the dam for a particular recurrence interval flood was obtained by routing various storm duration hydrographs through the reservoir as described in Section 3.1.

6.8.3 Wave height and run-up

The wave heights were calculated in accordance with the publication ‘Freeboard allowances for waves in inland reservoirs’ (Saville *et al*, 1962). The highest hourly wind speeds were obtained from the station at Polokwane. The wind speeds were increased by a factor of 1.244 to cater for the higher speeds over water than over land. The run-up ratio (wave run-up / design wave height) was obtained from **Figure A5** in (SANCOLD, 1990).

The various parameters used are as follows:

• Effective fetch	4.54 km
• 1:25 year wind speed over land	20.3 m/s
• Wave run-up factor	1.37
• Spillway length	190 m

6.8.4 Required Freeboard

The following design combinations of freeboard conditions were analysed (the calculations are shown in **Appendix C.5**):

- **Combination 2***

  RDF water depth + 1:25 year wind wave and run-up + wind set-up + flood surges and seiches of 0.50 m.

  Total required freeboard .... 4.26 m

- **Combination 4***

  Combination 4 allows for an earthquake wave. As the Nwamitwa Dam is situated in an area of low to moderate seismic risk, this combination was not considered to be applicable.

- **Combination 5***

  Combination 5 allows for a landslide wave. This is considered unlikely to occur as all the slopes in the dam basin are flat.

- **SEF**

  SEF water depth

  Total required freeboard .... 6.50 m

  * Combination numbers refer to those of Table 1 in the SANCOLD publication

A freeboard of 6.5 m has been provided.

7. SPILLWAY

7.1 SPILLWAY TYPE

The following four types of spillways were investigated for the Nwamitwa Dam:

- Straight ogee spillway
- Trough spillway
- Labyrinth spillway
- Side channel spillway

The side channel spillway was discarded as a viable option due to technical constraints. Of the remaining three options the straight ogee spillway proved to be the most cost effective.

The Spillway Type Selection Report is included in **Appendix D.1** for ease of reference.

7.2 FLOODS

7.2.1 Flood peaks

The following flood peaks have been selected to size the spillway (see Section 3.1):

- Recommended Design Flood (RDF) (1:200 year RI) 1 860 m^3/s
- Safety Evaluation Flood (SEF) ($RMF_{+\Delta}$) 6 800 m^3/s

The required freeboard above the FSL of the dam was determined in accordance with the publication “Interim Guideline on Freeboard for Dams” (SANCOLD, 1990). In order to allow for the SEF to go over the spillway, a freeboard of 6.5 m is required.

7.2.2 Freeboard

The 1998 feasibility design of the Nwamitwa Dam allowed for a total freeboard of 3.4 m. For the SEF this would have resulted in a spillway length of 500 m. During the initial phases of this study a freeboard of 6.5 m was adopted with a resultant spillway length of 190 m for a straight ogee spillway. The higher freeboard will result in increased expropriation costs of some R40 million, compared to a reduction in the cost of the dam of approximately R85 million due to the replacement of 310 m of RCC spillway with earthfill embankment. It was therefore decided to retain the freeboard of 6.5 m.

7.3 DESCRIPTION OF SPILLWAY

7.3.1 Location

The spillway would be constructed across the river channel, which would require significant widening to accommodate the 190 m long mass gravity RCC section. Tongue walls with a total length of 161 m would be provided on either side to accommodate the outlet works and to tie into the earth embankments. The proposed layout and cross sections are shown on Drawings 401775 CEN 215 and 216 in **Appendix G**.

7.3.2 Founding conditions

For the most part the granite gneiss underlying the river section comprises unweathered, very hard rock, although upper, weathered horizons are present. The founding level below the spillway was based on the RMR 40 level of approximately 442.5 masl. This will involve excavation depths of 10 m in the river section increasing to 16 m on the left bank and 27 m on the right bank respectively.

7.3.3 Discharge capacity

The discharge capacity for an ogee spillway is given by the following relationship:

$$Q = C_d * L * H_t^{1.5}$$

Where Q = discharge in m^3/s

C_d = discharge coefficient *($1.587 + 0.593 (H_t/H_d)^{0.5} = 2.18$ at design head H_d)*

L = crest length in m

H_t = total head on crest in m

In order to size the ogee section of the spillway, a design head (H_d) of 4.5 m was selected. This will allow for an increased discharge capacity over the full range of overflow depths, e.g. $C_d = 2.30$ at the maximum head of 6.5 m. The design head C_d of 2.18 was retained for the preliminary design to be consistent with the spillway type selection exercise. During the detailed design stage consideration could be given to reduce the spillway length for the SEF to 180 m, based on a C_d of 2.30. The spillway stage discharge curve is shown in **Appendix D.2**.

7.3.4 Stilling basin

The energy dissipation on the spillway steps was determined in accordance with the research done by Hubert Chanson (Chanson, 1994). Reference was also made to the publication "Dam Hydraulics" by (Vischer and Hager, 1999).

The energy losses were calculated based on an equation addressing the following parameters:

- Dam head = spillway height above apron
- Critical depth of water on ogee
- Friction factor of steps
- Downstream slope of spillway

The remaining head was calculated for a range of discharges from 200 to 2000 m^3/s (see spreadsheet in **Appendix D.3**).

The remaining head at the toe of the spillway was transformed into a flow velocity and water depth, which was used to determine the hydraulic jump conjugate depth and the stilling basin length. The conjugate depths were found to be consistently lower than the tailwater depths and full energy dissipation of the remaining head will therefore take place at the toe of the spillway.

A nominal stilling basin / apron with a length of 16 m has been provided to protect the toe of the spillway as well as to provide a stilling basin for the river releases from the outlet works.

The stilling basin characteristics were calculated in accordance with Engineering Monograph No 25 (Peterka, 1974) for a Type I basin. The dimensions of the stilling basin were determined by the following equation:

$$y_2 = 0.5*(((8*F_r^2 +1)^{0.5}-1)*y_1)$$

Where y_2 = conjugate depth in m

F_r = Froude number of flow entering stilling basin

y_1 = depth of flow entering stilling basin in m

The calculations are shown in a spreadsheet in **Appendix D.3**.

7.3.5 Tailwater analysis

Tail water depths were calculated with the water surface profile programme Channel Flow Profiles (CFP). Sections were taken from the 1:10 000 mapping of the Groot Letaba River. A Manning's n value of 0.035 was used to simulate the thick riparian vegetation along the river banks. The tail water curve is presented in **Appendix D.4**.

7.4 STRUCTURAL DESIGN

7.4.1 Introduction

The ogee spillway is a concrete gravity dam and was designed as such in accordance with the publications "Concrete Gravity Dams" (Design of Small Dams, 1987) and "Gravity Dam Structures" (Kroon, 1984). The spillway will be founded on rock with a RMR of 40 or higher.

7.4.2 Loadings

The following loadings were considered:

- Reservoir water at FSL
- Reservoir water at RDF level
- Reservoir water at SEF level
- Hydrostatic uplift below the base, including the effects of tail water
- Silt in reservoir after 100 years
- Earthquake loading applicable to the DBE
- Earthquake loading applicable to the MCE

The load combinations were as follows:

Working load combinations:

- RDF water level, silt, tail water and uplift (drains working)
- FSL water level, silt, DBE and uplift (drains working)
- Reservoir empty with DBE.

Abnormal load combinations:

- RDF water level, silt, tail water and uplift (drains blocked)
- SEF water level, silt, tail water and uplift (drains working)

Extreme load combinations:

- FSL water level with MCE and uplift

The stability criteria in terms of the limitation of tensile stress at the upstream face are given in **Table 7.1**.

Table 7.1 Allowable Stresses and Factors of Safety

	Working Load Combinations	Abnormal Load Combinations	Extreme Load Combinations
Maximum allowable vertical tensile stress at upstream face	Zero	100 kPa	200 kPa
Maximum allowable compressive stress	0.25 x compressive crushing strength after 90 days		
Minimum FOS against sliding	3.0	2.0	1.5

The results of the stability analysis are given in **Table 7.2**.

Table 7.2 Stability Results for Ogee Spillway

	Working Load Combinations			Abnormal Load Combinations		Extreme Load Combination
	RDF + silt + tailwater + uplift	**FSL + silt + DBE + uplift**	**Reservoir empty + DBE**	**RDF + silt + tailwater + uplift with drains blocked** -	**SEF + silt + tailwater + uplift**	**FSL + MCE + uplift**
Maximum stress at U/S face	+40 kPa	+115 kPa	+749 kPa	-30 kPa	-104 kPa	-40 kPa
Maximum stress at D/S face	+598 kPa	+567 kPa	+178 kPa	+538 kPa	+694 kPa	+722 kPa
Safety factor against sliding (Q)	5.5	6.0	228	4.7	3.7	6.0

+ indicates compression: - indicates tension

It can be seen from the above results that the spillway would comply with all the required criteria other than the SEF + silt + tailwater + uplift load combination, where the maximum tensile stress at the upstream face is marginally higher than the criterion.

8. OUTLET WORKS

8.1 FLOW REQUIREMENTS

The outlet works need to fulfil the following duties:

- Release the active storage capacity in accordance with the demand curve
- Release the environmental water requirements (EWR)
- Empty the dam during emergency drawdown conditions

8.2 SELECTION OF OUTLET PIPEWORK

The outlet pipework was based on the EWR as set out in the Preliminary Reserve Determination Study Scenario 7 at Site 3 (DWAF, 2006). The recommended Class II flood release category at EWR Site 3 at Prieska calls for discharge capacities of 12 to 18 m^3/s. As it is common practice to allow for 100% redundancy in the outlet pipework for operational flow releases, two pipe stacks will be provided. Due to the infrequent nature and short duration of the EWR, it is considered prudent from a cost point of view to use both pipe stacks to release the EWR. Each pipe stack was therefore sized to discharge 9 m^3/s.

8.3 DESCRIPTION OF OUTLET WORKS

The outlet works will consist of two DN 1 200 mm pipe stacks. The intakes to the pipe stacks will be staggered at 4.5 m intervals to allow for flexibility in selecting the most appropriate abstraction level. The maximum flow velocity during drawdown conditions will be 9 m/s. The two DN 1000 mm sleeve valves on the downstream side will be able to discharge 21 m^3/s with the water level in the dam at FSL.

The outlet pipework will be housed in an integral outlet block on the left hand side of the spillway. It will be equipped with precast concrete trash racks, stainless steel fine screens and a maintenance gate to close off the intakes to the pipe stacks.

8.4 WATER QUALITY

A water quality analysis of the proposed Nwamitwa Dam was undertaken to inform the design of the outlet structure of the dam, as well as the mitigating effects of installing a multi-level outlet structure.

A detailed Water Quality Report is contained in **Appendix E.1**. Based on the analysis undertaken the following conclusions could be drawn:

- More realistic outputs in terms of the temperature distribution in the Dam and temperature of releases can only be obtained if the input data to the model is more reliable. This is particularly relevant for inputs which drive the temperature profile within the Dam, viz. meteorological data, inflow temperature and volumes as well as release rates.

- The temperature of the dam releases made from the multi-level outlet structure is more representative of the inflowing temperature. This is expected since the warmer water higher up in the dam profile can now be released through this structure.

- Oxygen depletion (to anoxic levels) of the hypolimnion can be expected during a large proportion of the year, only to be re-oxygenated by cooler, oxygen-rich inflows that can plunge into this zone.

- Limited mitigation of in-lake de-oxygenation is provided by the multi level outlet structure and this concern would have to be addressed in an alternative approach, possibly looking at other engineering solutions.

The Water Quality Report (Appendix E.1) recommended that a multi-level outlet structure should be considered for further investigation since it provides more flexibility in mixing water from different levels in the dam, providing an increased probability of meeting the downstream water quality requirements.

9. RELOCATION OF ROADS

9.1 ROADS

9.1.1 Introduction

A number of existing provincial roads are affected by the proposed dam. Possible routes for the re-alignment of these roads were investigated and preliminary costs determined before a decision was made regarding the route alignments that were selected for the preliminary design stage.

9.1.2 Discussions with affected parties

The proposed re-alignment of the various routes was discussed with the landowners and the Roads Agency Limpopo (RAL) before a preliminary design of the proposed re-alignments commenced. Some adjustments were made to the re-alignment of route P43-3 after further consultation with some of the affected land owners. The owner of Nagude 517-LT Ptn 5 / Nagude 517-LT REM Ptn 6 was not available for some of the discussions but was represented by a family member. The owner had some concerns with the final alignment that was discussed in detail with his representative. The concerns expressed by the land owner were submitted after the completion of the designs of this road and a decision was taken that this concern should be addressed during the detailed design phase of the project.

9.1.3 Discussion of the affected roads

Road R529

This road follows a north/south alignment to the west of the Groot Letaba River. The road is affected over a length of approximately 6.1 km and 5 (five) alternative re-alignments were investigated before the preliminary design commenced (see **Figure 9.1**). The alignment that follows the existing road as far as possible (alternative 4) was preferred by RAL as the most suited re-alignment (see **Figure 9.2**). This alignment is also the shortest option and would have the least impact in the long term on the road users in terms of travel time and running costs. The road crosses the proposed dam basin between km 1.6 and km 4.6, a section of approximately 3.0 km. The road will be constructed on an earthfill embankment with rock protection (rip-rap) where it crosses the dam basin to protect the fill against wave action. Two bridges are planned where the road crosses the Hlangana and Nwanedzi Rivers. A short section of road D1292 (approximately 1.4 km long) has to be re-aligned to join into road R529 at km 2.79.

The cross-section for this road consists of:

Crossing the dam

- 3.3 m shoulder (2.5 m surfaced)
- 2 x 3.7 m lanes
- Fill slopes 1V:3H

<u>Remaining section</u>

<u>Fill <3 m high</u>

- 3.0 m shoulder (0.3 m surfaced)
- 2 x 3.7 m lanes

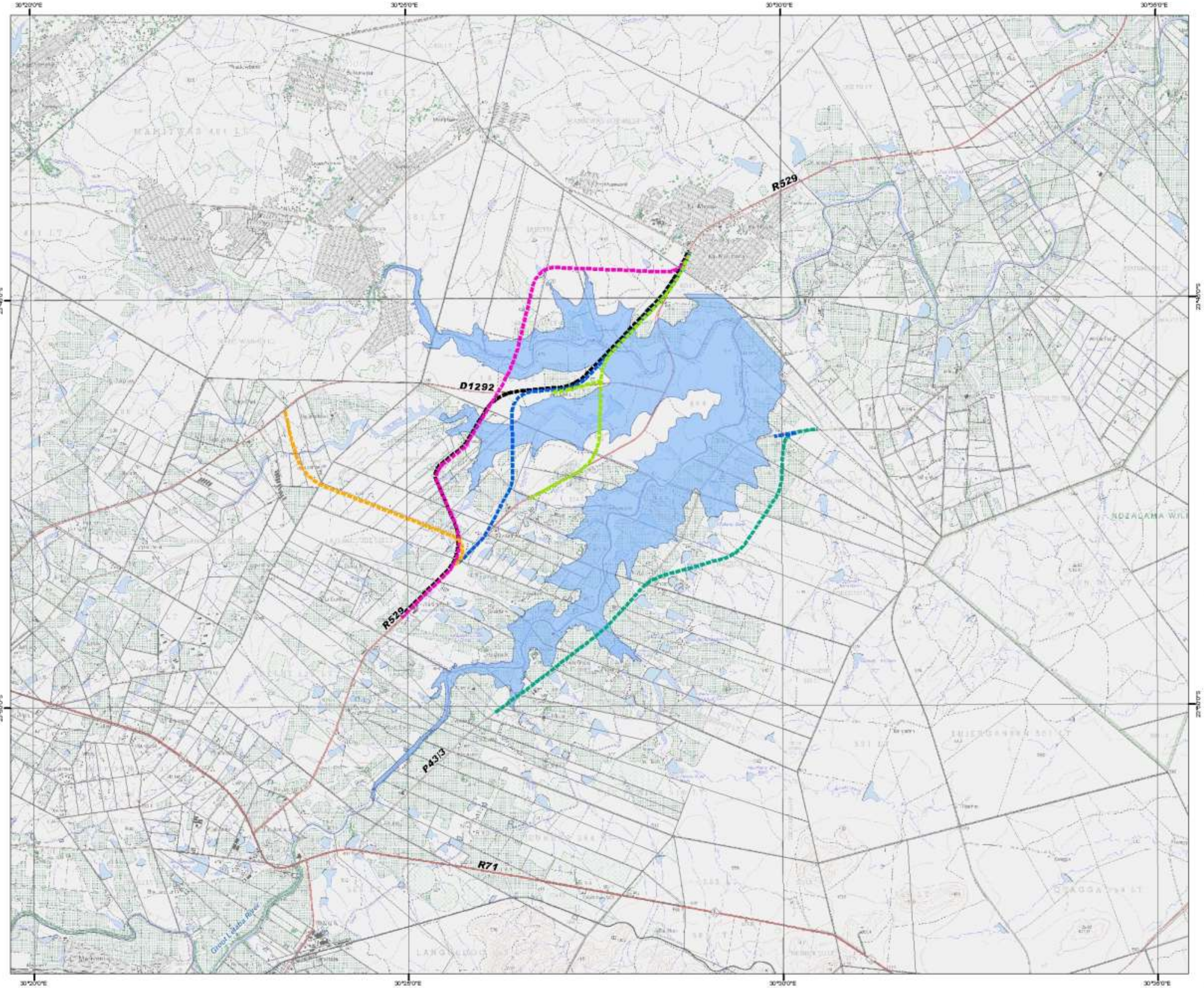
<u>Fill >3 m high</u>

- 3.3 m shoulder (0.3 m surfaced)
- 2 x 3.7 m lanes

<u>Cuts</u>

- 3.3 m shoulder (2.5 m surfaced)
- 2 x 3.7 m lanes

The cut slope is 1V:1.5H maximum (1:1 maximum in rock) and the fill slope 1V:1.5H maximum except for the section where the road crosses the dam where the fill slope is 1V:3H.



OVERVIEW MAP

LEGEND

Proposed Road Alternatives

- ROADS ALT 1
- ROADS ALT 2
- ROADS ALT 3
- ROADS ALT 4
- ROADS ALT 5
- ROADS P43-3
- Proposed Nwamitwa Dam
- Farm Boundaries

DATE

s:/projects/8401775/mxd/Reports/
Prelim_Design/fig91.mxd

SCALE

0 1 2 3 Km

1:80,000

PROJECTION INFORMATION :

COORDINATE SYSTEM :	Transverse Mercator
PROJECTION NAME :	TM31
DATUM :	D_WGS_1984
SPHEROID :	WGS_1984
CENTRAL MERIDIAN :	31

CONSULTANT :

aurecon

CLIENT :

water affairs
Department:
Water Affairs
REPUBLIC OF SOUTH AFRICA

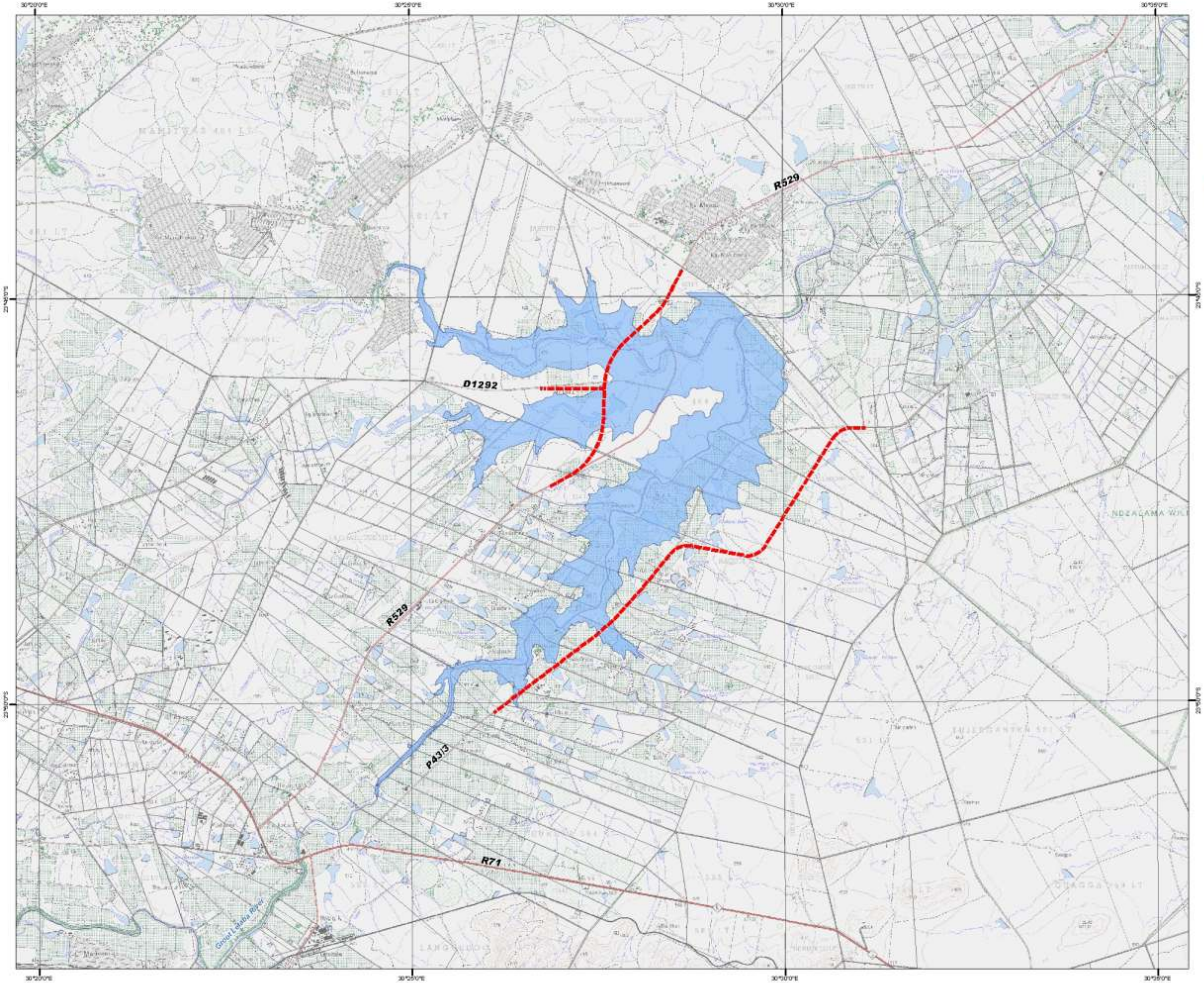
TITLE

GROOT LETABA RIVER WATER DEVELOPMENT PROJECT

ALTERNATIVE ROAD ROUTE INVESTIGATION

FIGURE NO

9.1



OVERVIEW MAP

LEGEND

Final Proposed Road Route

Proposed Nwamitwa Dam FSL 479.5 masl

Farm Boundaries

DATE: x:/projects/H401775/mxd/Reports/Prelim_Design/fig92.mxd

SCALE: 0 1 2 3 Km
1:80,000

PROJECTION INFORMATION

COORDINATE SYSTEM:	Transverse Mercator
PROJECTION NAME:	TM31
DATUM:	D_WGS_1984
SPHEROID:	WGS_1984
CENTRAL MERIDIAN:	31

CONSULTANT

aurecon

CLIENT

water affairs
Department:
Water Affairs
REPUBLIC OF SOUTH AFRICA

TITLE

GROOT LETABA RIVER WATER DEVELOPMENT PROJECT

FINAL PROPOSED ROAD ALTERNATIVES

FIGURE NO.

9.2

Road P43-3

Road P43-3 follows a north/south alignment along the eastern side of the Groot Letaba River. The road is affected by the proposed dam where it crosses a number of small tributaries. The existing road has to be raised to the non-overspill (NOC) level where it crosses these tributaries. A section of 1.1 km of the existing road must be raised over three sections along the current horizontal alignment.

The road was re-aligned horizontally from km 5.5 from where it crosses a number of small farms in conjunction with the landowners to minimize the impact it will have on their farming operations.

The re-aligned road follows a fairly flat topography with no major cuts or fills and crosses a number of small streams where culverts are necessary.

The positions of access to the various properties have been agreed with the landowners including positions for services ducts to install future pipes for the landowner's uses.

The cross-section for this road is similar to that of Road R529.

9.1.4 Geometric alignment standards

The major design parameters for the design of the roads affected are listed below :

	Road R529	Road P43-3
Design speed (km/hr)	100	80
Minimum horizontal curvature (m)	850	250
Minimum vertical curve length (m)	180	140
Minimum 'k' value – crest	62	33
Minimum 'k' value sag	37	26
Maximum super elevation (%)	8	6
Maximum gradient (%)	6	8
Shoulder sight distance (m)	300	250
Passing sight distance (m)	680	560

9.1.5 Pavement design

(a) Traffic counts

Traffic counts for 2007 indicate that the Annual Average Daily Traffic (AADT) amounts vary from 2 400 to 2 800 vehicles with heavy vehicle traffic accounting for approximately 12 to 14% of the traffic.

(b) Road classification

Based upon the analysis of the traffic counts applicable to the adjacent roads, and taking into consideration the levels of mobility and access provided by the road in conjunction with its role within the wider road network of the area, the road can be defined as a Category C Road.

(c) Structural design period

TRH4:1996 recommends that the structural design period for Category C pavements range from 10 – 20 years. For the analysis and design of the structural capacity of the roads under investigation a structural design period of 20 years will be utilised.

(d) Traffic growth rate

It is difficult to estimate the expected traffic growth rate, as no historic traffic data was available for the adjacent road network. Typically traffic growth rates on provincial roads are of the order of 1% to 3% per annum. A sensitivity analysis was therefore conducted on the influence of growth rate on the design E80's. The two growth rates analysed were:

- Low to medium traffic growth 2%, and
- Medium to high traffic growth 4%

(e) Equivalent 80 kN Standard Axle Load (ESALs)

As the E80s per heavy vehicle values were not measured in the field, a sensitivity analysis of the influence of E80s on the pavement design was conducted. Three scenarios were identified in order to estimate the average axial loading, namely:

- E80/Heavy Vehicle Factor (Light) 1,
- E80/Heavy Vehicle Factor (Medium) 2, and
- E80/Heavy Vehicle Factor (Heavy) 3

(f) Sensitivity analysis of traffic data

A sensitivity analysis was conducted as part of the traffic loading calculations in order to determine the sensitivity of the traffic loading to changes in growth rate defined as a conservative (4%) and realistic (2%) model.

The results of the sensitivity analysis are included in the figures below. It is evident that the predicted traffic over the 20 year design period varies between 5 million Equivalent Standard Axles (MESA) and 7.5 MESA dependent on the traffic counting location and the expected traffic growth.

g) Design traffic loading

Taking into consideration the variation in traffic loading prediction and the uncertainties related to the prediction of traffic growth on low volume rural roads, it is proposed that the traffic design be based on a 75th percentile of the maximum traffic loading expected. This constitutes design traffic of approximately 5 MESA.

h) Pavement analysis methodology

Although TRH4:1996 is the current specification utilised for the design of pavement structures in South Africa, certain of the limitations of this catalogue design method need to be considered. Due to the current industry norms concerning loading and tyre inflation pressure, pure catalogue design from TRH4:1996 can result in a significant over-estimation of the structural capacity of a pavement, particularly with regards to the upper pavement structure. Furthermore, the TRH4 Catalogue Design provides the user with a finite number of proposed pavement structures.

The design evaluated within the report is based on mechanistic analysis of the pavement design alternatives utilising the South African Mechanistic Design Method and taking cognisance of variation in the current trafficking regime particularly with regards to truck traffic.

i) Mechanistic pavement design

The pavement design was undertaken utilising MePADS design software, in conjunction with the design criteria relating to traffic, road classification and design period as defined in the preceding section of the report.

The pavement structure was evaluated under the following design parameters:

- Tyre inflation pressure 650 kPa
- Axle load 80 kN dual wheel load

The basic design parameters and materials characteristics utilised in the design are contained in **Figure 9.4**.

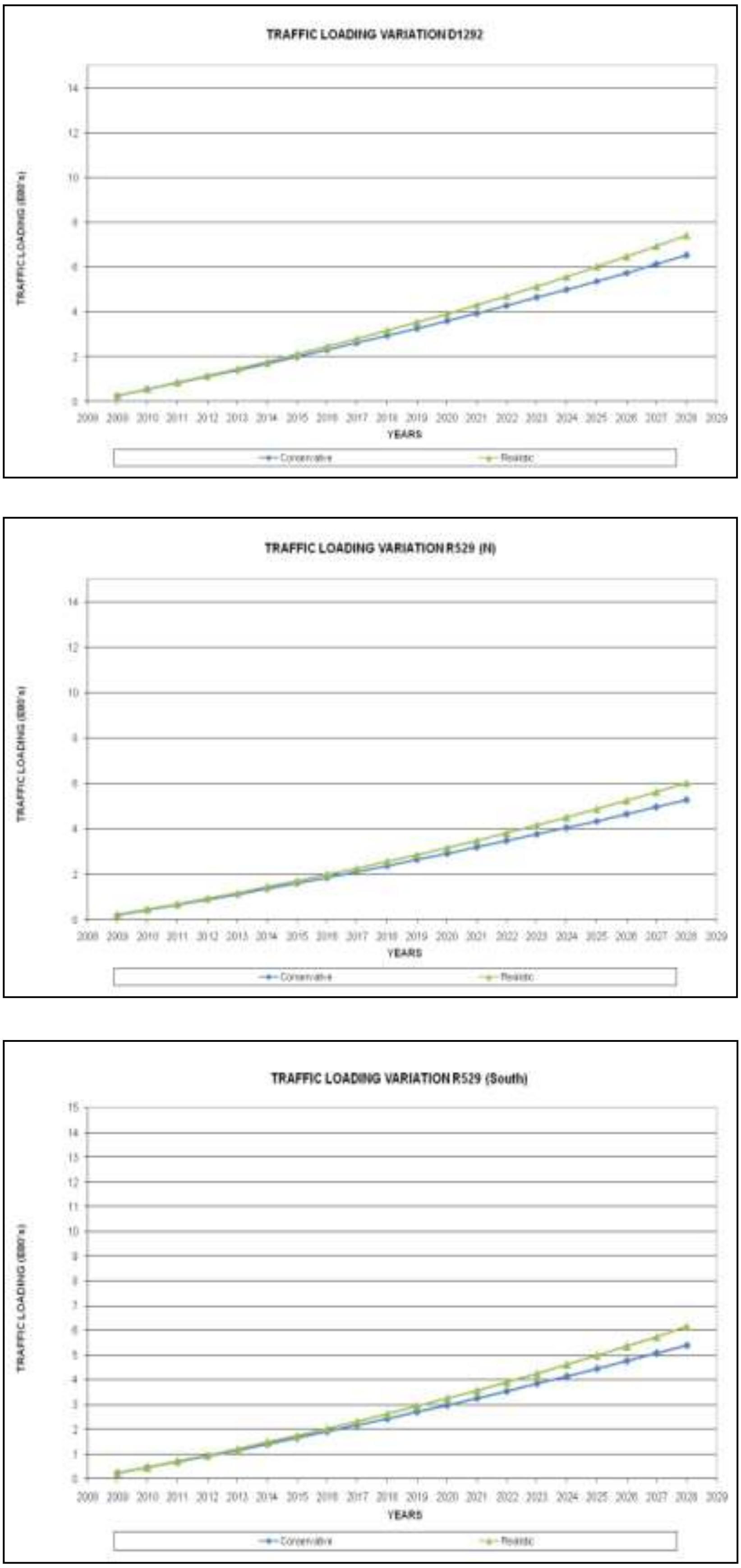


Figure 9:3 Design traffic loading

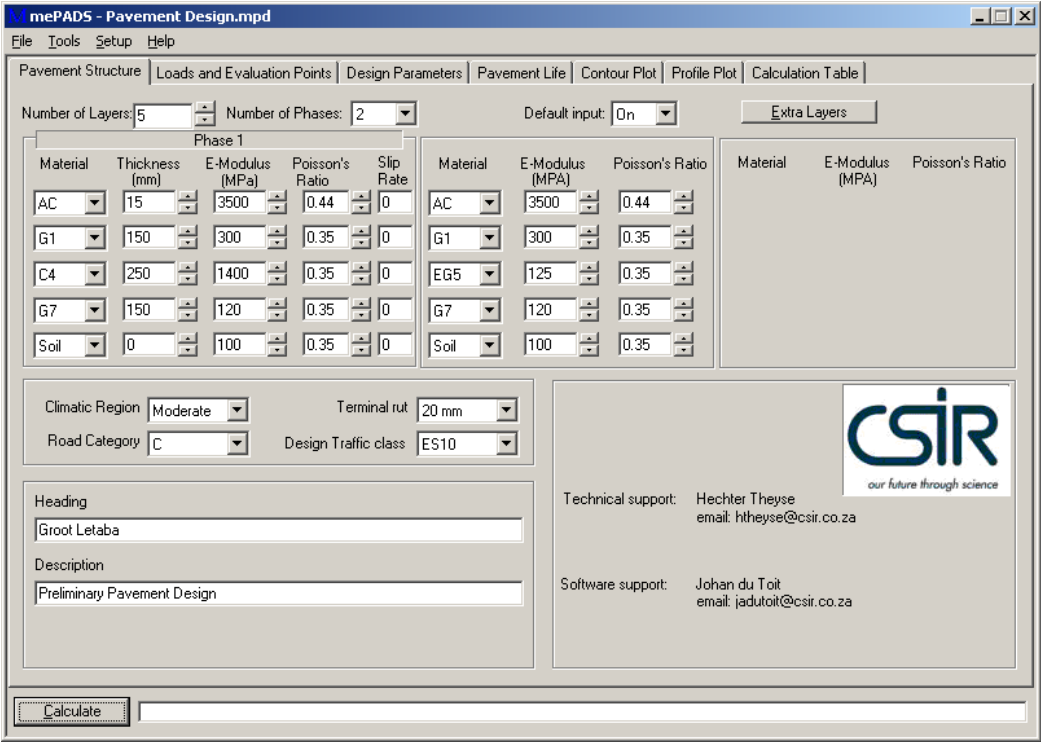


Figure 9:4 Pavement design parameters

Figure 9.5 provides a graphically representation of the expected pavement life in terms of equivalent standard 80kN axles. Based on the analysis the proposed pavement structure the predicted structural capacity of the pavement is 5.17 MESA indicated by the layer bearing capacity values.

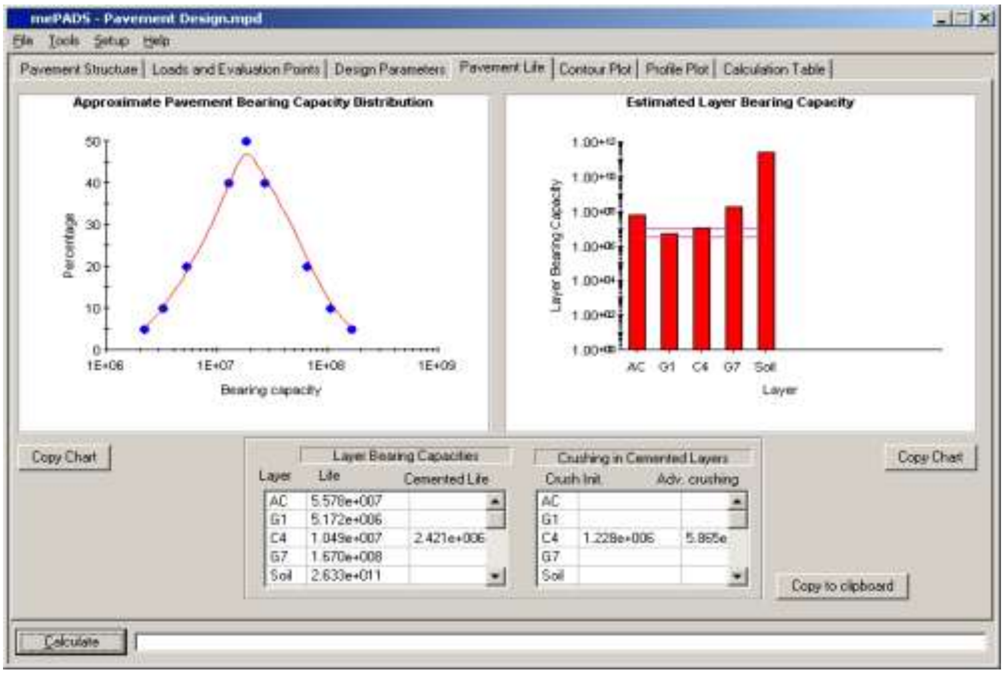


Figure 9:5 Pavement structural capacity

The G1 base and the cement stabilised subbase are the critical layers in the pavement structure. It is thus crucial that the road is sealed and maintained in order to prevent moisture ingress into the upper pavement layers and subsequent moisture accelerated distress of the base layer.

9.1.6 Materials for construction purposes

It is anticipated that materials for construction of the fills, riprap embankment protection and layerworks will be obtained from a weathered granite borrow pit located within the proposed dam basin area.

Depending on the suitability of this material the material for the construction of the G1 base may have to be obtained from other sources in the area.

9.1.7 Road reserve requirements

The existing road reserve along both these roads is approximately 30 m wide and it is not anticipated that this width will be exceeded except within the dam basin area.

9.1.8 Stormwater design

The drainage areas have been determined using available mapping and 1:10 000 maps of the area. The drainage areas will have to be confirmed during the detailed design phase due to the scale of the maps used for this purpose at this stage.

Flows have been determined using the Rational method and are considered realistic. Culvert configurations are however preliminary. Although the opening sizes will be similar, the actual culvert configurations (height, width, number of cells etc.) will be optimised during the detailed design.

Along Road P43-3 a number of existing culverts have to be lengthened. It is not anticipated at this stage that these culverts will have to be re-build or that the capacity of the existing culverts be increased.

The design parameters for the drainage design is summarised below :

Mean annual precipitation (MAP)	700 mm per annum
Return period	1 in 20 years
Runoff calculation	Rational Method
Rainfall Region	Summer

9.2 BRIDGES

9.2.1 Introduction

The re-aligned Road R529 crosses the Hlangana and Nwanedzi Rivers at km 2.15 and km 3.76 respectively where major bridges are required.

The total road width is 12.4 m, made up of 2x3.7 m lanes and 2x2.5 m shoulders. A total bridge width of 13.35 m is therefore required.

The road embankment is between 18 and 25 m high at these crossings, and will have side batters of 1V:3H, with 2 m wide terraces at 5 m vertical intervals. For road embankments of this order of height, and relative flatness of the side slopes, longer bridge structures with “spill-through” abutments are considerably more economical than the equivalent bridges with closed abutments. A spill-through type structure is proposed for both bridges as cost of the additional length of deck and piers is substantially less than the cost of the long and high walls that are required to retain the embankment.

Both structures have been configured to have an odd number of spans, in order that the main river channel is unobstructed by a central pier at low dam levels.

9.2.2 Founding conditions

A number of test pits were dug along the road re-alignment with a light backhoe. These all refused on very dense residual Gneiss. This material may be adequate for conventional spread footings, but as none of the trial pits were closer than 600 m from the bridge sites, and as competent founding material may be considerably deeper in the vicinity of the river channel, it has, at this stage, been conservatively assumed that piled foundations will be required for both bridges.

9.2.3 Hlangana River bridge

At the Hlangana River, the road crosses the river at an angle of approximately 62°, and the road level is approximately 18 m above the natural ground level.

The width of opening perpendicular to the flow, required to accommodate the 1V:3H embankment spill-through slopes and 2 m wide terraces, as well as a nominal channel of approximately 7 m width at ground level, is 105 m, requiring an approximate length of structure of 120 m. By increasing the slope of the top 10 m of the spill-through embankment, where flow velocities will be relatively low, to 1V:2H, the required length of structure can be reduced to 105 m.

For a structure of 105 m in length, and 18 m above ground, a bridge deck comprising precast concrete beams with a cast in-situ top slab will be the most economical, and 3 x 35 m spans with this deck type, is proposed for the Hlangana River bridge. The common alternative of incrementally launched cast in-situ deck construction is generally regarded as un-economical for bridges less than 200 m in length.

Suitable embankment protection will be required on the steeper slopes in order to prevent slumping due to rising and falling water levels. Embankment protection against flooding of the river at low dam levels is also proposed for the lowest slopes of the spill through embankment.

The piers can be conventional reinforced concrete walls, widened at the top to accommodate the precast beams.

In the absence of adequate founding information, it is at this stage proposed that the abutments be perched on piles driven through the road embankment. Some settlement of the high fills can therefore take place before the abutments are completed. To reduce the effect of further settlement of the embankments relative to the bridge abutments, 6 m long approach slabs are recommended on each side.

9.2.4 Nwanedzi River bridge

At the Nwanedzi River, the road locally crosses the river channel at an approximate angle of 52°, and the road level varies between 18 and 26 m above the natural ground level. The general direction of the river, upstream and downstream of the bridge, is at an approximate angle of 62° to the road, and as this will be the general direction of flow at all but low dam levels, the bridge has been aligned at this angle.

The width of opening required perpendicular to the flow to accommodate the 1V:3H embankment spill-through slopes with the 2 m wide terraces, as well as the approximately 15 m wide well defined river channel, is 160 m. This results in a required length of structure of approximately 185 m. As for the Hlangana River bridge, the slope of the top 10 m of the spill-through embankment has been increased to 1V:2H, thus shortening the length of bridge to 175 m, and increasing the effective width of channel at the bottom, by 8 m, to accommodate the localized increase in the angle of skew.

As for the Hlangana River bridge, a deck comprising precast concrete beams with a cast in-situ top slab is considered to be the most appropriate for this structure. The required length is made up of 5 x 35 m spans, thus standardising the precast beams for both structures. Piers and abutments will also be similar to those proposed for the Hlangana Bridge, as well as embankment protection at the lower levels and on the upper 1V:2H slopes of the spill-through embankments.

Should a detailed geotechnical investigation reveal that the bridge can be suitably founded at say 2 m below natural ground level, conventional spill-through abutments can be used instead of the perched abutments proposed.

10. EXPROPRIATION OF LAND

10.1 INTRODUCTION

The consulting firm of Schoeman & Vennote was contracted to conduct a desktop study to determine the expropriation costs of the land and other structures that will be inundated when the dam is built.

The study was done in two phases. The initial study was done in September 2007 and provided estimated expropriation costs for three dam sizes (see Section 6.2 and the letter dated 18 September 2007 in **Appendix F.4**). The expropriation line, which depicts the minimum land purchase requirements due to dam construction, was determined according to the *Policy and Guidelines for the Acquisition of Land Rights at Departmental Dams* (DWAF, 2001*)*. Further information is provided in Section 3.4.

The second phase of the study was done in April 2009 and provided an updated cost estimate for the preferred dam size of 1.16 MAR (FSL 479.5 masl). The report is included in **Appendix F.4**. The estimated total compensation costs payable amounts to R180 million.

11. COST ESTIMATES

11.1 NWAMITWA DAM

11.1.1 Introduction

During the execution of the Vaal Augmentation Planning Study (VAPS), the Project Planning Directorate of the South African DWA recognised that the standard methodology developed during the study for the sizing and costing of water resource project components and for the economic evaluation of water resource development options would be a valuable tool for subsequent planning exercises. It was accordingly decided to capture the guidelines in a single document which would be made available to planning professionals both within the Department and those consultants appointed by the Department to undertake specific assignments (DWAF, 1996b).

During the Lower Orange River Management Study (LORMS), the dam rates from VAPS were reviewed and updated to a base date of April 2004 (DWAF, 2005). The following additional sources of information were used:

- Maguga Dam
- Mohale Dam
- Inyaka Dam
- Matsoku Weir
- Paris Dam

During the Lesotho Highlands Further Phases Study (LHFP), the dam rates from LORMS were again reviewed and updated to January 2006 (LHWC, 2007). It was also compared with the Engineer's Estimate for the Berg River Dam.

For the Bridging Studies of the Groot Letaba River Water Development Project, the LHFP dam rates were further escalated to April 2009. Cognisance was also taken of rates for the De Hoop Dam.

11.1.2 Descriptions of Payment Items

(a) Clearing

The quantities only allow for the clearing of the dam footprint. The area was split 20% sparse, 40% bush and 40% trees.

(b) River diversion

The lump sum allows for the excavation of the diversion channel and the construction of the cofferdams.

(c) Excavation

The quantities allow for the excavation of the cut-off trench and the spillway including the return channel to the river. The rock excavation constitutes the volume between the RMR 20 and 40 levels underneath the spillway.

(d) Drilling and grouting

The quantities for the curtain grouting allow for primary holes spaced at 6 m intervals with allowance for secondary and tertiary holes over 50% of the area. The quantities for the consolidation grouting allow for grouting in the core trench to a depth of 3 m with holes drilled 1.5 m either side of the grout curtain in a staggered pattern at 3 m centres. Below the spillway the consolidation grouting will be done to a depth of 3 m over the complete foundation at 3 m centres in both directions.

(e) Embankment

The quantities for the earthfill allow for the impervious material placed in the cut-off trench and the core zone as well as the semi-pervious material placed in the general fill zone.

(f) Concrete works

The quantities for mass concrete allow for the outlet block below the level of the pipework, whilst the quantities for the structural concrete allow for the remainder of the outlet block.

The reinforcement quantities were based on a mass of 80 kg/m^3 of structural concrete.

11.2 ROAD RELOCATIONS

The cost estimates for the road relocations were based on a detailed Schedule of Quantities as shown in **Appendix F.3**.

11.3 RELOCATION OF ESKOM AND TELKOM INFRASTRUCTURE

The cost estimate for the relocation of the Eskom infrastructure is shown in **Appendix F.4**. The cost estimate for the relocation of the Telkom infrastructure is still outstanding.

11.4 EXPROPRIATION COSTS

As stated in Section 10, an updated cost estimate for the preferred dam size was prepared in April 2009. The report is included in **Appendix F.5**.

11.5 ESTIMATED PROJECT COSTS

Construction cost estimates were prepared for the four dam sizes that were used for the dam size optimisation. The cost estimates allow for the relocation of services (roads, electricity and telecommunications), contingencies, planning, design and supervision costs and expropriations costs.

The cost estimates are shown in **Appendices F.1** and **F.2** and summarised in **Table 11.1**.

Table 11.1 Estimated Project Costs

Dam size	Capacity (Million m^3)	FSL (masl)	Estimated Project costs (excl VAT) (million Rand)
0.41 MAR	66	473.5	989
0.85 MAR	137	477.5	1 180
1.16 MAR	**187**	**479.5**	**1 285**
1.50 MAR	241	481.5	1 410

A unit cost versus yield curve was drawn up for all four dam sizes. The results of this exercise are shown in **Appendix C.2**.

12. IMPLEMENTATION PROGRAMME

A proposed implementation programme has been compiled and is shown on the next page (**Figure 12.1**). The programme has been based on the following production rates:

- Soft excavation 40 000 m^3/month
- Hard excavation 25 000 m^3/month
- Grouting 2 500 m/month
- Reinforced concrete 1 000 m^3/month
- RCC 35 000 m^3/month
- Earthfill 100 000 m^3/month

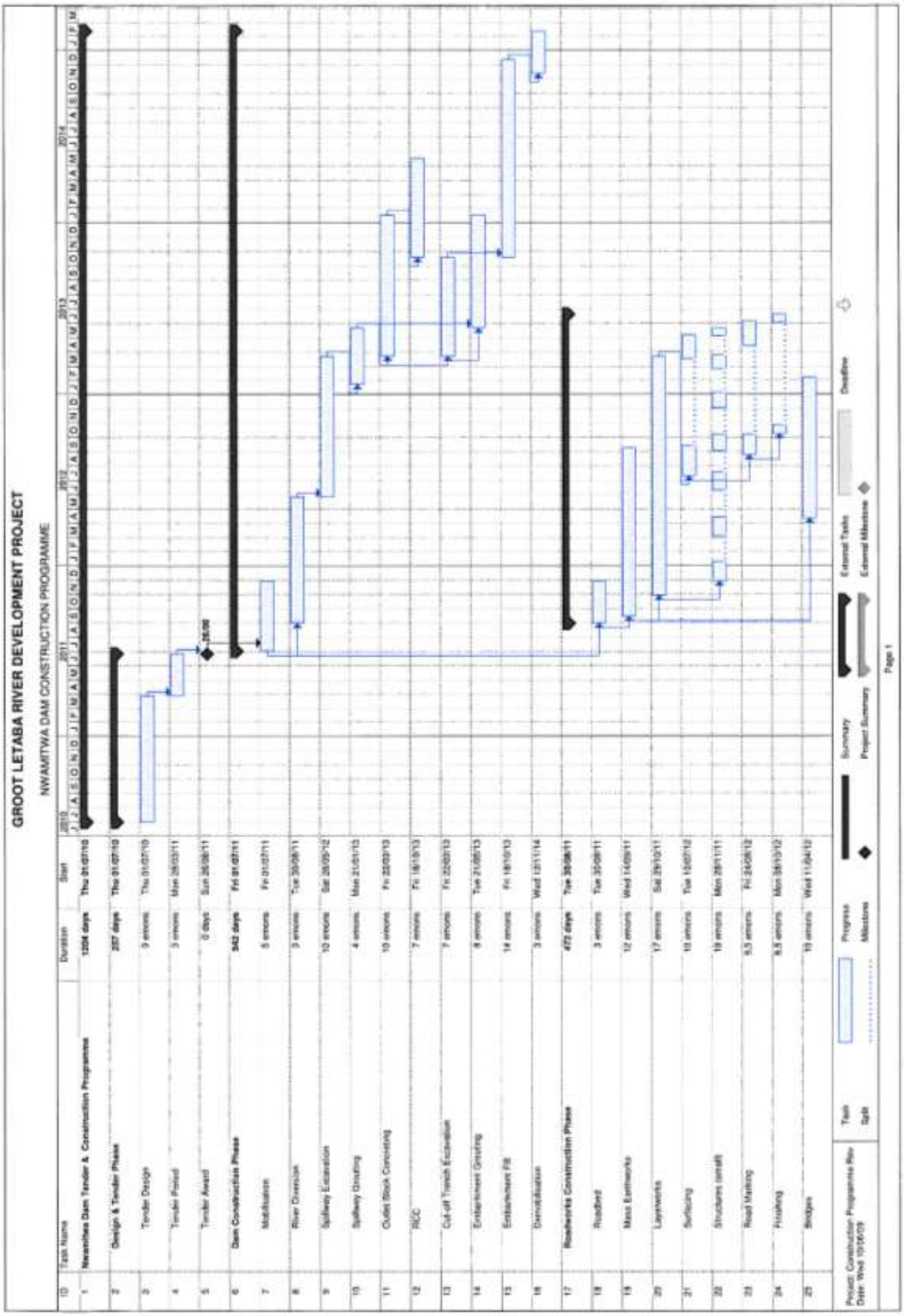


Figure 12:1 Implementation Programme

13. COMMENTS RECEIVED

13.1 INTRODUCTION

Comments on the draft Preliminary Design Report were received from the following sources:

- DWA Directorate : Civil Engineering
- BKS (Pty) Ltd
- Knight Piesold (Pty) Ltd
- Prof A van Schalkwyk

The comments, as well as Aurecon's response, are attached to this report as **Appendix I**. The response has been divided as follows:

- Incorporated in the report as amendments
- Not included as noted in response
- Listed for action during detailed design as shown below

13.2 ACTION POINTS FOR DETAILED DESIGN

- External review of flood hydrology
- Review of river diversion strategy
- Hydraulic model study of spillway with particular reference to the stilling basin design
- Further geotechnical investigations to confirm foundation conditions as well as to prove sufficient volumes of construction materials
- Review of embankment materials and cross-section
- Deck levels of bridges on relocated roads around dam basin

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