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GROOT LETABA RIVER WATER DEVELOPMENT PROJECT (GLeWaP)

TECHNICAL STUDY MODULE:

Preliminary Design of the Raising of Tzaneen Dam

VOLUME 7

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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
P WMA 02/B810/00/0508/5	Project Coordination and Management Team
P WMA 02/B810/00/0608	**Technical Study Module**
P WMA 02/B810/00/0608/1	Technical Study Module: Main Report: Vol 1
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P WMA 02/B810/00/0608/3	Technical Study Module: Groundwater: Vol 3
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P WMA 02/B810/00/0608/6	Technical Study Module: Preliminary Design of Nwamitwa Dam: Vol 6
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
P WMA 02/B810/00/1110/3	Technical Study Module: Preliminary Design of Nwamitwa Dam: Vol 6 - Annexure 3: Appendix B (Part 2): Geotechnical Investigations
P WMA 02/B810/00/1110/4	Technical Study Module: Preliminary Design of Nwamitwa Dam: Vol 6 - Annexure 4: Appendix H: Drawings
P WMA 02/B810/00/0608/7	Technical Study Module: Preliminary Design of the Raising of Tzaneen Dam: Vol 7
P WMA 02/B810/00/0608/8	Technical Study Module: Bulk Water Distribution Infrastructure: Vol 8
P WMA 02/B810/00/1110/5	Technical Study Module: Bulk Water Distribution Infrastructure: Vol 8 - Annexure 1 : Appendices
P WMA 02/B810/00/0708	**Environmental Management Module**
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P WMA 02/B810/00/0708/2	Environmental Management Module: Environmental Impact Assessment Report: Vol 2
P WMA 02/B810/00/0708/3	Environmental Management Module: Environmental Management Programme for Borrow Area 1 on the Farm Laborie 515: Vol 3
P WMA 02/B810/00/0708/4	Environmental Management Module: Environmental Management Programme for Borrow Area 2 on the Farm La Parisa 729 (Gubits Farm): Vol 4
P WMA 02/B810/00/0708/5	Environmental Management Module: Environmental Management Programme for Borrow Area 3 on the Farm Letaba Drift 727: Vol 5
P WMA 02/B810/00/0708/6	Environmental Management Module
P WMA 02/B810/00/0708/7	Environmental Management Module
P WMA 02/B810/00/0708/8	Environmental Management Module
P WMA 02/B810/00/0708/9	Environmental Management Module
P WMA 02/B810/00/0708/10	Environmental Management Module
P WMA 02/B810/00/0808	**Public Involvement Program**
P WMA 02/B810/00/0808/1	Public Involvement Program: Main Report: Vol 1
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P WMA 02/B810/00/0808/3	Public Involvement Program
P WMA 02/B810/00/0808/4	Public Involvement Program
P WMA 02/B810/00/0808/5	Public Involvement Program

REPORT DETAILS PAGE

Project name:	***Groot Letaba River Water Development Project***
Report Title:	***Preliminary Design of the Raising of Tzaneen Dam : Volume 7***
Author:	***D I van Wyk, V Jonker, C V Seddon, H Smit, I Bey***
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EXECUTIVE SUMMARY

This report investigates with the different methods that could be used to raise the Full Supply Level (FSL) of the Tzaneen Dam.

The previous study on the raising of Tzaneen Dam identified the following options (BKS, 1998):

- Hydroplus Fusegates
- Fishbelly Flap Gates
- TOPS Gates

The latter two gates are both of an all-steel construction with a significant number of moving parts, which will require ongoing maintenance and inspections. With the present skills shortage in South Africa it is not recommended that such a system be implemented.

For the present study the options as listed below have been considered. The estimated capital and maintenance costs are as follows:

- Hydroplus fusegates R59 million
- Labyrinth spillway R42 million
- Side channel spillway R72 million

The comparison was made for a 3 m raising of the FSL to level 726.9 masl. The amount by which the FSL and Non Overspill Crest (NOC) can be raised is limited by the soffit levels of the Sybrand and Marietjie van Niekerk Bridges and the fact that additional land may have to be acquired for the dam basin and surrounding buffer strip.

The side channel option, whilst technically feasible, was discarded as an option based on cost. The two remaining options are both considered to be technically feasible. However, given the fact that the labyrinth spillway option is the most cost effective solution coupled with the fact that this option has no future maintenance costs, it is recommended that this method of raising be adopted.

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ABBREVIATIONS

CGS	Council for Geoscience
DBE	Design Basis Earthquake
DWAF	Department of Water Affairs and Forestry
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
LORMS	Lower Orange River Management Study
masl	meters above sea level
MCE	Maximum Credible Earthquake
NOC	Non Overspill Crest
OA	Options Analysis
OBE	Operation Basis Earthquake
PGA	Peak Ground Acceleration
PMF	Probable Maximum Flood
RDF	Recommended Design Flood
RI	Recurrence Interval
RMF	Regional Maximum Flood
SANCOLD	South African National Committee on Large Dams
SED	Safety Evaluation Discharge
SEF	Safety Evaluation Flood
SHA	Seismic Hazard Assessment
VAPS	Vaal Augmentation Planning Study
WRC	Water Research Commission

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 (then) Department of Water Affairs and Forestry (now 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 is shown in **Figure 1.1**. It consists of the catchment of the Letaba River, including the Groot, Middle and Klein Letaba Rivers, as well as the main Letaba River downstream to its entry into Mozambique. The catchment falls within the Mopane District Municipality, which is made up of six Local Municipalities. The Local Municipalities that lie within the catchment area are Greater Tzaneen, Greater Letaba, Greater Giyani and part of the Kruger National Park. The major town in the study area is Tzaneen, with the urban centre of Polokwane 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.

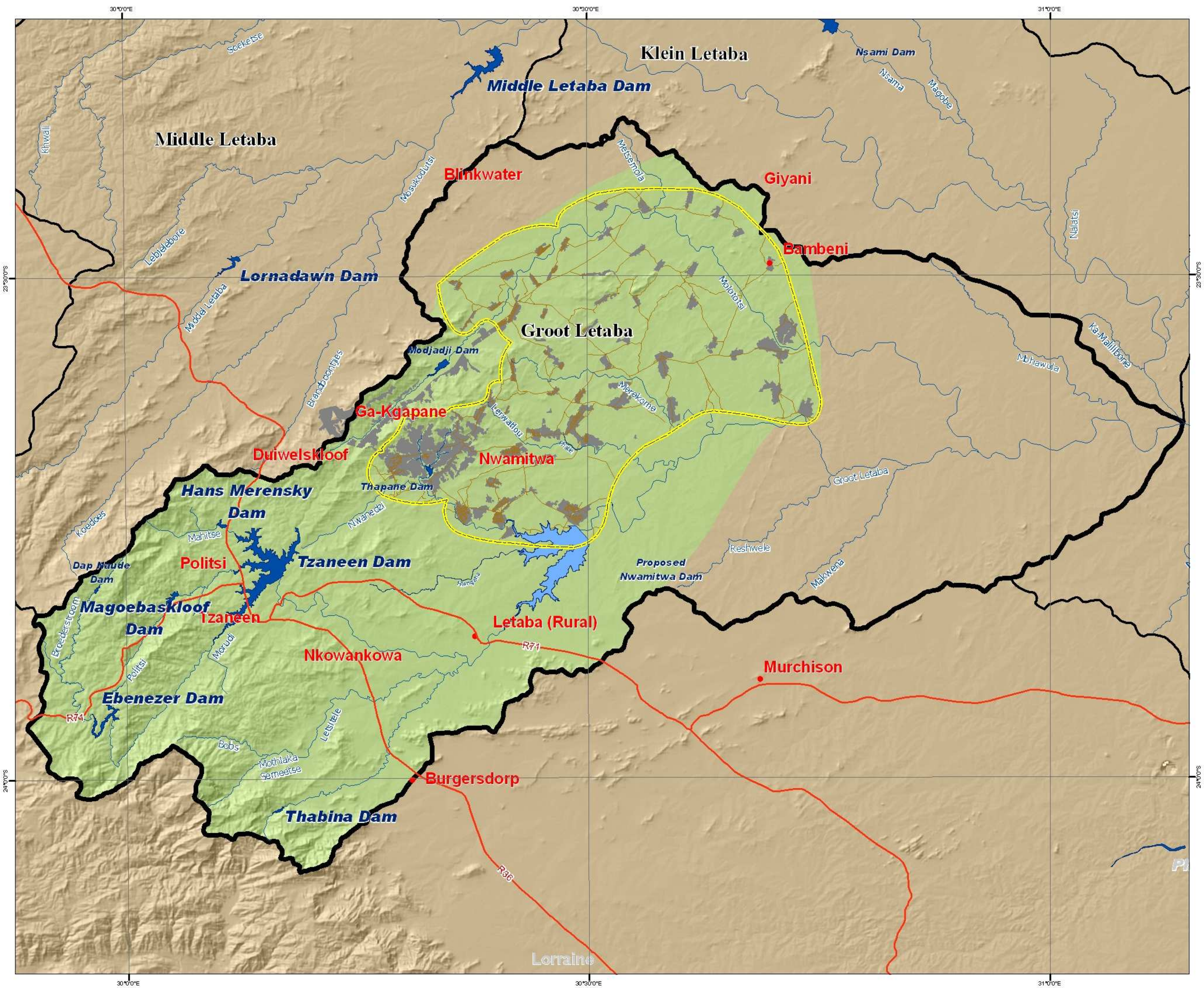
1.2 SCOPE AND ORGANISATION OF PROJECT

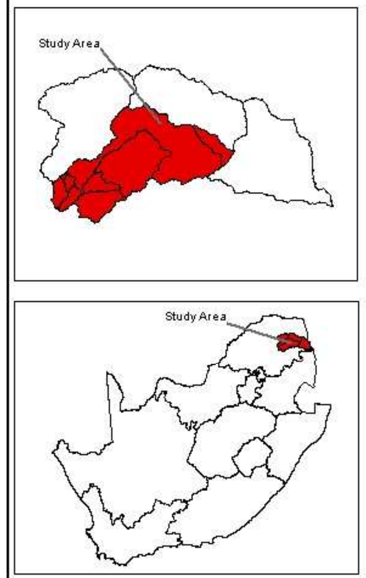
The Department's Directorate: Options Analysis (OA), appointed Ninham Shand in Association with a number of sub consultants (listed below) to undertake this study. The official title of the study is: "The Groot Letaba Water Development Project: Bridging Studies: Technical Study Module".

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.





LEGEND

- Towns
- National Roads
- Secondary Roads
- Rivers
- Dams
- Villages
- Study Area
- Proposed Supply Area from Nwamitwa Dam

PATH: s\projects\mxd\Reports\mxd\Tzaneen_Dam\fig11.mxd

SCALE: 0 5 10 15 km

1:380,000

PROJECTION INFORMATION :

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

CONSULTANT :

aurecon

water affairs
Department:
Water Affairs
REPUBLIC OF SOUTH AFRICA

TITLES :

GROOT LETABA RIVER WATER DEVELOPMENT PROJECT

PROJECT AREA

FIGURE NO. :

1.1

1.3 SCOPE OF THIS REPORT

This report investigates with the different methods that could be used to raise the Full Supply Level (FSL) of the Tzaneen Dam.

The previous study on the raising of Tzaneen Dam (BKS, 1998) identified the following options:

- Hydroplus Fusegates
- Fishbelly Flap Gates
- TOPS Gates

The latter two gates are both of an all-steel construction with a significant number of moving parts, which will require ongoing maintenance and inspections. With the present skills shortage in South Africa it is not recommended that such a system be implemented.

For the present study the following options have being considered:

- Hydroplus Fusegates
- Labyrinth Spillway
- Side Channel Spillway

The comparison was made for a 3 m raising of the FSL to level 726.9 masl. The amount by which the FSL and Non Overspill Crest (NOC) can be raised is limited by the soffit levels of the Sybrand and Marietjie van Niekerk Bridges and the fact that additional land may have to be acquired for the dam basin and surrounding buffer strip.

2. PRINCIPAL DETAILS OF TZANEEN DAM

Tzaneen Dam was completed in 1977. It comprises a mass concrete gravity spillway section flanked by earthfill embankments. The spillway is an uncontrolled ogee type 91.44 m long with a crest level of 723.90 masl. The NOC is 1 063.5 m long with a crest level of 730.60 masl. Both upstream and downstream faces of the earth embankments are protected by interlocking concrete blocks.

The gross storage capacity of the dam is 157.3 million m^3 (DWAF, 1999). This would be increased to 193 million m^3 with a 3 m raising of the FSL. The firm yield from the dam would be increased from 60 to 64 million m^3 per annum.

3. FLOOD HYDROLOGY

3.1 INTRODUCTION

The flood hydrology for the raised Tzaneen Dam was investigated as part of the Preliminary Design Report for the proposed Nwamitwa Dam. Relevant extracts from **Appendix A** of the latter report have been included in **Appendix A** of this report for ease of reference.

3.2 SPILLWAY FLOODS

The Tzaneen Dam is a large dam (>30 m high) with a high hazard potential (due to extensive downstream developments) and has been classified as a Category III dam in terms of the Dam Safety Regulations. As the proposed raising of the dam would constitute a new design, it was considered "necessary to perform hydrological calculations appropriate to the site" for a Category III dam in accordance with Sub-Clause 3.4.2 of the SANCOLD Guidelines (SANCOLD, 1991).

The recommended floods for the sizing of the raised 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).

- 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)	RMF Approach
1:100 year RI	1 741	1 750
1:200 year RI	1 935	2 150
RMF (Region 5.2)	n/a	3 240
$RMF_{+\Delta}$ (Region 5.4)	n/a	4 120
PMF	7 365	n/a

(1) Represents maximum inflow flood peak for critical storm duration.

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 2.3 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 large dams with a significant or high hazard rating, as in the case of Nwamitwa Dam. 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 equivalent K-value and the PMF/RMF ratio. In the case of Nwamitwa Dam, these upper limits are exceeded: The equivalent PMF K-value is 6,2, while the PMF/RMF ratio is in the order of 3,0. Therefore, taking cognisance of the HRU 1/72-based PMF-related concerns expressed in the findings of the Water Research Commission (WRC) Study on Extreme Design Floods, the use of a SEF lower than the PMF-routed values determined during this study, but higher than the RMF (unrouted), 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 Tzaneen 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 4 120 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 a raised Tzaneen Dam.

For the 1 in 100 year and 200 year RI floods at Tzaneen 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 through application of the empirical RMF technique. The simulated 1 in 100 year and 200 year RI flood hydrographs for a range of storm durations were routed through the raised Tzaneen 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 have been selected to size the raised spillway:

- 1 in 100 year RI flood 1 170 m^3/s
- Recommended Design Flood (RDF) (1:200 year RI) 1 360 m^3/s
- Safety Evaluation Flood (SEF) (RMF+Δ) (Region 5.4) 4 120 m^3/s

The 1 in 100 year and 1 in 200 year RI flood peaks were obtained by routing the respective hydrographs through the Tzaneen Dam reservoir with a 3 m raising of the FSL with a labyrinth spillway in place.

The SEF resembles an unrouted flood peak. In order to generate an incoming hydrograph for the Hydroplus option, the Probable Maximum Flood (PMF) hydrograph producing the highest outgoing flood peak of 4 700 m^3/s was scaled down in the ratio 4 120/4 700.

4. DISCHARGE CAPACITY OF RAISED SPILLWAY

4.1 OGEE SPILLWAY

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

The discharge capacity of the existing spillway was calculated to be 3 500 m^3/s.

4.2 HYDROPLUS FUSEGATES

The discharge capacity over the spillway sill (with all the fusegates having tipped) 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 = 1.86
- L = crest length in m
- H_t = total head on crest in m

The discharge capacity over the fusegates has been determined by model studies of similar layouts and will be confirmed by a hydraulic model study of the final layout if accepted.

4.3 LABYRINTH SPILLWAY

The design procedure for the labyrinth spillway was adopted from “Design of Labyrinth Spillways” by (Tullis et al, 1995). The procedure provides a design calculation presented in a spreadsheet format, as shown in **Table 4.1**.

Table 4.1 Labyrinth Dimensions

RAISING OF TZANEEN DAM CALCULATION OF LABYRINTH DIMENSIONS FOR SEF				
Parameter	**Symbol**	**Quantity**	**Units**	**Comment**
Maximum flow	Qmax	4120	m^3/s	Input = RMF + Δ
Maximum reservoir elevation	res	733	m	Input = 2.4m raising of NOC
Approach channel elevation		684	m	Input
Crest elevation	el	726.9	m	Input
Total head	Ht	6.1	m	Ht = res - crest - loss
Estimated inlet loss at Qmax	Loss	0	m	Estimated
Number of cycles	N	8	-	
Crest height	P	8.54	m	Set P approx = 1.4 Ht
Angle of side legs	alpha	15	deg	Normally 8 - 16 deg
Thickness of wall at top	t	1.2	m	Input
Inside width at apex	A	1.2	m	Select between t and 2t
Outside width of apex	D	3.04	m	D=A+2*t*tan(45-alpha/2)
Total head/crest height	Ht/P	0.71	-	-
Crest coefficient	Cd	0.416	-	Equation relevant to alpha (Equ 2 - 9)
Effective crest length	L	222.63	m	1.5*Qmax/[(Cd*Ht^1.5)*(2*g)^0.5]
Length of apron (parallel to flow)	B	14.63	m	[L/(2*N)+t*tan(45-alpha/2)-A]*cos(alpha)+t
Actual length of side leg	L1	13.90	m	(B-t)/@cos(alpha)
Effective length of side leg	L2	12.98	m	L1-t*tan(45-alpha/2)
Total length of walls	L3	256.37	m	N*(2*L1+D+A)
Distance between cycles	w	11.44	m	2*L1*sin(alpha)+A+D
Width of labyrinth (normal to flow)	W	91.50	m	N*w
Length of linear weir for same flow		121.79	m	1.5*Qmax/[(Cd*Ht^1.5)*(2*g)^0.5]: (Cd for linear weir = 0.76)
Distance between cycles/crest height	w/P	1.09	-	

The upper block lists typical input data that would come from the hydrological analysis of the system. This includes the maximum required spillway flow, the corresponding maximum reservoir elevation (NOC) and the FSL.

The second block contains assumed data. The number of cycles has a significant effect on the overall layout of the labyrinth. The value of N is varied to determine the most appropriate number of cycles that gives the least cost and a hydraulically effective layout. An increase in the value of N reduces concrete volumes. The value of N = 8 was chosen to fit the labyrinth within the existing length of ogee spillway.

The third block of data contains the detailed calculations identifying the geometry of the labyrinth. Such calculations are most efficiently done using a spreadsheet.

5. HYDROPLUS FUSEGATES

5.1 DESCRIPTION

The Hydroplus proposal comprises the installation of ten 6.3 m high fusegates on the spillway. The existing spillway crest level will be lowered by 3.3 m to form a platform which will carry the fusegates.

No tipping of any fusegate will occur up to the RDF, an event which has only a 0.5% probability of occurrence in any specific year.

The SEF will be passed over the spillway with the maximum water level at the present NOC level of 730.6 masl. All the fusegates would tip during the SEF.

Routine maintenance of the fusegate system will be restricted to visual inspections only, which could be undertaken by the DWA operating staff situated at the Tzaneen Dam. Major maintenance will be required every 20 years during a period of natural low water level in the dam, during which time the fusegates need to be jacked free of the base, the rubber seals replaced and the corrosion protection of the steel components attended to.

Further details are provided in the Hydroplus proposal in **Appendix B**.

5.2 IMPACT OF RAISED FSL

Properties around the dam basin of Tzaneen Dam had been expropriated up to the level of the NOC. A 3 m raising of the FSL would therefore not involve any additional expropriation.

5.3 IMPACT OF FUSEGATE ROTATION ON YIELD

A study has been carried out by WV Pitman, MD Watson and WD Hakin on behalf of DWA involving 30 river catchments in South Africa to assess the possible impact on the firm yield of a reservoir caused by the rotation of a fusegate. The results indicated that the risk of fusegate rotation impacting on the firm yield is extremely low, especially if the first tip is designed to occur for floods of 1 in 100 year recurrence interval or greater, and that the reinstatement period is only of a few month's duration. A report on the study is included in **Appendix C**.

5.4 COST ESTIMATE

Hydroplus provided a cost estimate for the conceptual design and the detail design and construction stages of the fusegates.

A ballpark rate for the demolition of the top section of the existing spillway has been included in the cost estimate.

The maintenance cost of the fusegates was assumed to amount to R500 000 every 20 years. The replacement cost of a fusegate was assumed to be double that of the initial construction cost (approx R40 million / 10 gates x 2 = R8 million).

The cost estimate for a 3 m raising of Tzaneen Dam with Hydroplus fusegates is **R59 million**. Details of the cost estimate are shown in **Appendix D.1**.

6. LABYRINTH SPILLWAY

6.1 DESCRIPTION

A maintenance free option to raise the FSL of the dam would be to modify the top of the existing overflow structure to accommodate a labyrinth spillway. The preliminary layout comprises 8 cycles with 15º wall angles. The top 7.5 m of the existing structure would have to be demolished to accommodate the 2 m thick labyrinth base and 8.5 m high labyrinth walls, thereby raising the FSL by 3 m. The upstream apexes of the labyrinth would be cantilevered 2 m upstream of the existing structure to reduce the amount of overhang on the downstream side. On the downstream side, the existing structure would have to be widened by 3.7 m by placing mass concrete on the downstream face to support the downstream apexes. Details are shown on Drawing 401775 CEN 20 Rev B in **Appendix E**. In order to accommodate the SEF a gravity wall 2.4 m high would have to be constructed over the full length of the NOC.

The RDF would be discharged over the labyrinth with the water level in the dam 1.6 m below the existing NOC.

The initial sizing of the labyrinth was based on research where water was discharged into a sub-critical downstream pool, thereby creating high downstream water levels and a consequent reduction in the discharge capacity of the labyrinth. If a labyrinth were to be constructed on top of the existing spillway structure, the outgoing water would be discharged freely. This could improve the overall discharge capacity of the labyrinth thereby reducing the height of the gravity wall. The potential higher discharge capacity would have to be confirmed by a hydraulic model study if the labyrinth option were to be selected.

6.2 IMPACT OF RAISED NOC

The raising of Tzaneen Dam with a labyrinth spillway would require a 2.4 m raising of the NOC. As stated in Section 5.2, properties around the dam basin had been expropriated up to the level of the NOC. The normal practice is to expropriate land up the 1 in 100 year RI flood level plus 1.5 m (DWAF, 2001). This could be accommodated below the existing NOC and no additional expropriation would be necessary.

On 9 April 2008 a site visit was conducted to gain a clearer perspective of what structures would be influenced by raising the FSL and what information would be required to fully assess the impact of the raising.

From the 1:50 000 topographical map 2330CC Tzaneen specific areas of interest were identified where a rise in water level could possibly influence and/or damage existing infrastructure. The areas visited are listed in **Table 6.1**.

Table 6.1 Road Infrastructure around Tzaneen Dam Basin

Road Reference No.	Description	Latitude (dd,mm,ss)	Longitude (dd,mm,ss)
R36/R71	Sybrand and Marietjie van Niekerk Bridges	23º49'34.9''	30º07'51.9''
R36	Culvert	23º48'52.2''	30º07'26.9''
R36	Culvert	23º47'39.0"	30º07'14.0''
Secondary road	Bridge No. 3080	23º45'49.2''	30º08'22.6''
Secondary road	Bridge No. 3081	23º45'20.2''	30º11'11.8''
	Railway Bridge	23º49'42.0''	30º07'43.2''

Subsequent measurements at the Sybrand and Marietjie van Niekerk Bridges indicated that the soffit of the bridge deck is at approximately level 731.0 masl. The bridge deck itself is at approximately level 734.0 masl. The raised NOC of the dam would be at level 733.0 masl. The integrity of the two bridges risk during the SEF therefore requires detailed study during the detailed design phase.

None of the culverts or the other bridges would require significant improvement.

The railway bridge runs parallel to the Van Niekerk bridges. Visual inspection indicated that the elevation of the railway bridge is much higher than the Van Niekerk bridges.

6.3 STRUCTURAL DESIGN

6.3.1 Introduction

The ogee spillway is a concrete gravity dam and the structural stability of the revised section was checked in accordance with the publications "Concrete Gravity Dams" (Design of Small Dams, 1987) and "Gravity Dam Structures" (Kroon, 1984).

6.3.2 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, excluding 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 and uplift (drains working)
- FSL water level, silt, DBE and uplift (drains working)

Abnormal load combinations:

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

Extreme load combinations:

- FSL water level with MCE and uplift

A seismic hazard assessment for the proposed Nwamitwa Dam was conducted by the Council for Geoscience (Kijko and Singh, 2008). As the Tzaneen Dam is in close proximity to the proposed Nwamitwa Dam, the same peak ground acceleration values were adopted, namely an OBE value of 0.024g and a MCE value of 0.14g.

The stability criteria in terms of the limitation of tensile stress at the upstream face are given in **Table 6.2**.

Table 6.2 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 6.3**.

Table 6.3 Stability Results for Raised Spillway

	Working Load Combinations		**Abnormal Load Combinations**		**Extreme Load Combination**
	RDF + silt + tailwater + uplift	**FSL + silt + DBE + uplift**	**RDF + silt + tailwater + uplift with drains blocked** -	**SEF + silt + tailwater + uplift**	**FSL + MCE + uplift**
Maximum stress at U/S face	+410 kPa	+290 kPa	0 kPa	+210 kPa	+380 kPa
Maximum stress at D/S face	+660 kPa	+760 kPa	+760 kPa	+860 kPa	+580 kPa
Safety factor against sliding (Q)	4.8	4.7	3.9	4.1	1.6

+ indicates compression: - indicates tension

It can be seen from the above results that the spillway would comply with all the required criteria.

6.4 COST ESTIMATE

During the execution of the Vaal Augmentation Planning Study (VAPS), the Project Planning Directorate of the 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, 1996).

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.

In order to obtain a realistic cost estimate of the demolition of the top section of the existing spillway, a feasibility level quote was obtained from Jet Demolition (Pty) Ltd, the same company that did the demolition of the top section of the Midmar Dam spillway. Details of the quote are provided in **Appendix D.2**.

The cost estimate for a 3 m raising of Tzaneen Dam with a labyrinth spillway is **R42 million**. Details of the cost estimates are shown in **Appendix D.2**.

7. SIDE CHANNEL SPILLWAY

7.1 DESCRIPTION

The topography of the left abutment of the dam lends itself to the construction of a side channel spillway. As an alternative to the labyrinth spillway, a 3 m high fixed raising of the existing spillway was investigated supplemented by a 45 m long side channel spillway. A gravity wall 2.4 m high would still have to be constructed over the full length of the NOC.

In order to continue discharging the smaller floods over the ogee spillway, the overflow crest level of the side channel spillway was set 1.1 m higher than the raised FSL of the dam. The raised ogee spillway would be able to discharge 3 000 m^3/s, whilst the remainder of the SEF of 1 120 m^3/s would be discharged by the side channel spillway.

A conceptual layout of the side channel spillway is shown on Drawing 401775 CEN 21 Rev A in **Appendix E**.

7.2 IMPACT OF RAISED NOC

The impact of the raising would be similar to that described in Section 6.2.

7.3 COST ESTIMATE

The cost estimate has been based on rates as described in Section 6.4.

The cost estimate for a 3 m raising of Tzaneen Dam with a side channel spillway is **R72 million**. Details of the cost estimates are shown in **Appendix D.3**.

8. COMMENTS RECEIVED

8.1 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

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

- Incorporated in the report as amendments
- Rejected as noted in response
- Listed for action during detailed design as shown below

8.2 ACTION POINTS FOR DETAILED DESIGN

- Stability analysis of embankment with raised FSL
- Checking of integrity of Sybrand and Marietjie van Niekerk Bridges during SEF
- Stability analysis of spillway with raised FSL

9. CONCLUSIONS AND RECOMMENDATIONS

The following three methods were investigated for the raising of Tzaneen Dam:

- Hydroplus fusegates
- Labyrinth spillway
- Side channel spillway

The estimated capital and maintenance costs are as follows:

- Hydroplus fusegates R59 million
- Labyrinth spillway R42 million
- Side channel spillway R72 million

The cost estimates include planning, design and supervision costs, but excludes VAT and land costs.

The side channel option, whilst technically feasible, was discarded as on option based on cost. Both the labyrinth spillway and fusegate options have a number of advantages and disadvantages as can be seen from **Table 9.1** below.

Table 9.1 Summary of Advantages and Disadvantages

Option	Advantages	Disadvantages
Labyrinth spillway	1) Potentially lowest cost solution 2) Minimum maintenance 3) Low risk	1) Potential impact on the integrity of the Sybrand and Marietjie van Niekerk bridges during very high flood conditions (flood peak in excess of 1:1 000 year RI) 2) NOC has to be raised by 2.4 m
Hydroplus fusegates	1) Least construction impact on the dam wall itself 2) No impact on Sybrand and Marietjie van Niekerk bridges 3) NOC of dam wall does not have to be raised	1) Loss in storage if fusegate topples (flood peak in excess of 1:200 year RI) 2) Potential loss in yield if a critical period follows directly after a 1: 200 year flood event - probability of occurrence regarded as very low, but nevertheless remains a risk 3) Replacement cost of the fusegates should they topple 4) Long term maintenance costs associated with 20 year major maintenance

Both the labyrinth spillway and the hydroplus fusegate options are considered to be technically feasible. However, given the fact that the labyrinth spillway option is the most cost effective solution coupled with the fact that this option has very low future maintenance costs, it is recommended that this method of raising be adopted. Should it be discovered during the detailed design phase, that the potential impact of the extreme flood events or the integrity of the Sybrand and Marietjie can Niekerk Bridges is considered unacceptable, then the hydroplus fusegate option would become the preferred option for the raising of Tzaneen Dam.

10. REFERENCES

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APPENDIX A

FLOOD HYDROLOGY

(Extract from Nwamitwa Dam Design Flood Analysis Report, Nwamitwa Dam Preliminary Design Report, Appendix A)

A1 INTRODUCTION

The flood hydrology for the raised Tzaneen Dam was investigated as part of the Preliminary Design Report for the proposed Nwamitwa Dam.

The Tzaneen Dam is a large dam (>30 m high) with a high hazard potential (due to extensive downstream developments) and has been classified as a Category III dam in terms of the Dam Safety Regulations. As the proposed raising of the dam would constitute a new design, it was considered "necessary to perform hydrological calculations appropriate to the site" for a Category III dam in accordance with Sub-Clause 3.4.2 of the SANCOLD Guidelines (SANCOLD, 1991).

The recommended floods for the sizing of the raised 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).
- Empirical flood techniques in the form of the Francou-Rodier approach, used by Kovacs to develop the Regional Maximum Flood (RMF) peak (Kovacs, 1988).

A2 CATCHMENT CHARACTERISTICS

In order to determine representative design floods at Nwamitwa Dam, the attenuation effect of Tzaneen Dam, which is located within the upper Nwamitwa Dam catchment, had to be accounted for. Consequently, the Nwamitwa catchment was split into two subcatchments as shown in **Figure A2.2**. The upper catchment, representing the Tzaneen Dam catchment, has an area of approximately 650 km^2, while the remaining incremental catchment has an area of 1294 km^2. Relevant catchment characteristics are presented in **Table A2.1**.

The attenuation effects of other, smaller dams within the Nwamitwa Dam catchment including the Ebenezer, Dap Naudé, Magoebaskloof, Hans Merensky and Vergelegen dams were not considered in this study.

Table A2.1 Catchment Characteristics

Subcatchment	Tzaneen Dam	Nwamitwa Dam (incremental)
Latitude	23°48' S	23°45' S
Longitude	30°10' E	30°29' E
Catchment area (km^2)	650	1294
Generalized veld type zone	8	8
Extreme point rainfall zone	1 & 2	1 & 2
Length of longest water course (km)	75.2	72.0
Distance to centroid (km)	37.6	33.8
Average channel slope (m/m)	0.0066	0.003
Catchment Index	34795	44433
Basin lag (h)	8.2	9.0
Unitgraph peak (m^3/s)	29.1	53.0

The Tzaneen Dam has a unique catchment in terms of topography and drainage pattern. The steep section of the Groot Letaba River along its middle reaches (**Figure A2.1**) results in an average watercourse slope of 0.013 as calculated by both the 10-85 and the equal-area methods. Similarly, the oxbow shape of the river in plan view (**Figure A2.2**) results in an unrealistic estimate (10.0 km) for Lc, which represents the length along the main watercourse to a point opposite the catchment centroid. Both of these estimates lead to the calculation of a short basin lag, which in turn results in very conservative (high) estimates of flood peaks.

In order to obtain a more realistic estimate of basin lag for the Tzaneen catchment, an alternative methodology for the calculation of the average watercourse slope was adopted in which the steep middle section of the longitudinal profile was disregarded and the average watercourse slope for the whole catchment equated to the average of the upper and lower reach slopes as shown in **Figure A2.1**. This resulted in an average watercourse slope of 0.0066. Similarly, a value of 37.6 km (half of the total river length) was accepted for Lc.

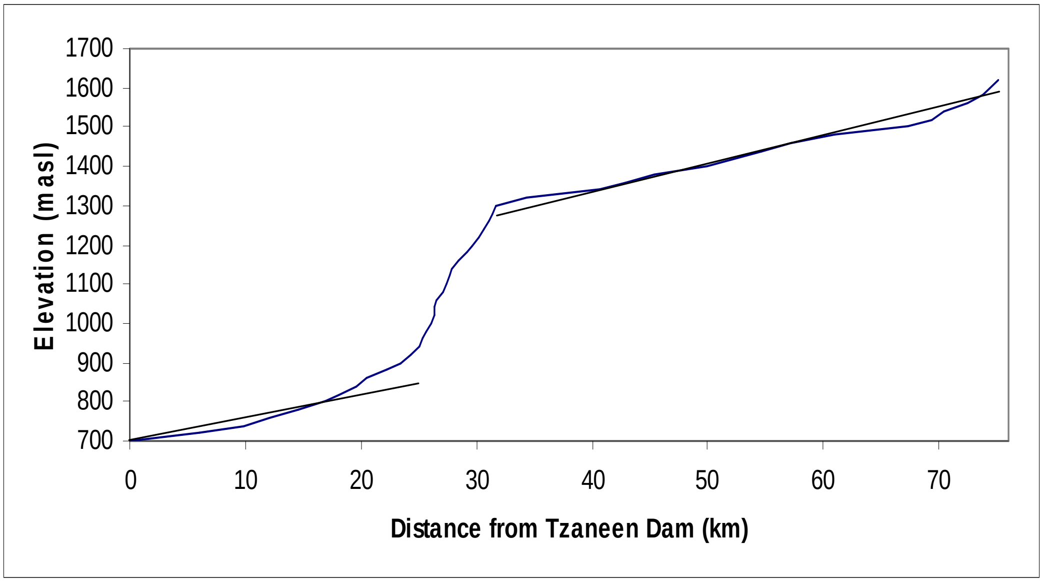

Figure A2.1 Longitudinal Profile of Groot Letaba River in Tzaneen Catchment

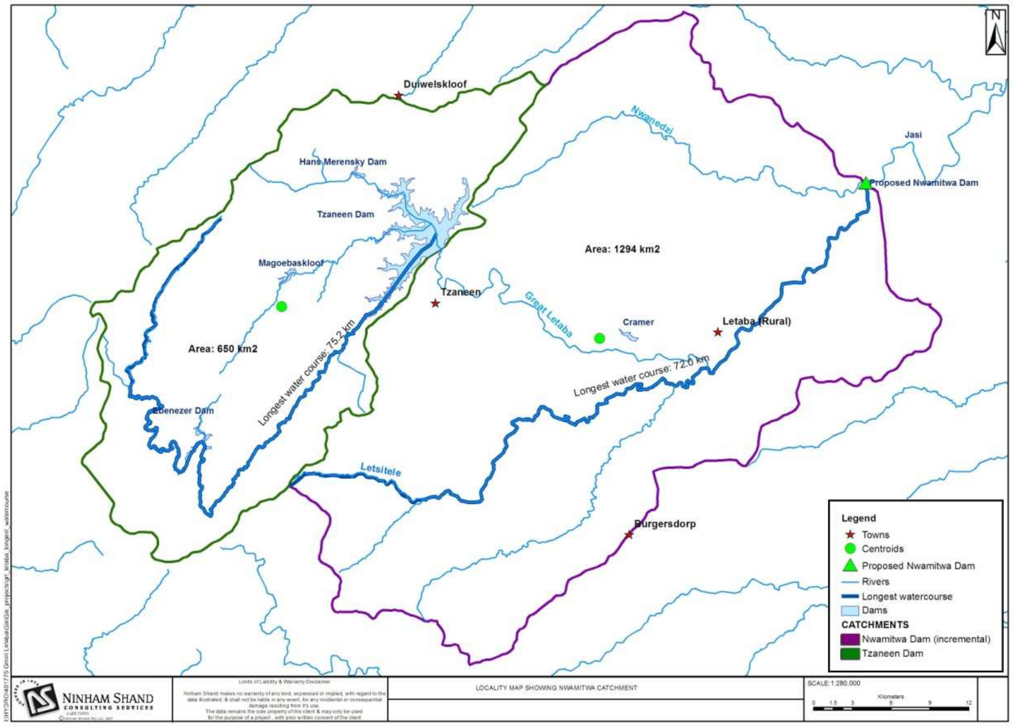

Figure A2.2 Tzaneen Dam: Catchment Centroid in Relation to Longest Watercourse

A3 DESIGN RAINFALL

Estimates of design rainfall for the range of recurrence intervals that were considered were based on the minute by minute design point rainfall grid as developed by Smithers and Schulze (2002). Estimates of Probable Maximum Precipitation (PMP) were based on envelope curves of maximum observed rainfall in South Africa as presented in HRU 1/72 (HRU, 1972).

In order to convert point rainfall to catchment storm rainfall, standard areal reduction factors (Alexander, 1990) and regional storm loss factors (HRU 1/72, 1972) were applied. The temporal distributions of storms were based on the HRU 1/72 distributions for medium-area storms.

Table A2.2 presents a summary of the design rainfall for the Tzaneen Dam catchment for the range of RIs and storm durations that were considered.

Table A2.2 Tzaneen Dam Design Rainfall

Catchment characteristics	**TZANEEN**				
Catchment area	650.5	km^2			
Veld zone	8	-	(HRU 1/72 Fig. F1)		
Extreme rainfall zone	1 & 2	-	(HRU 1/72 Fig. C3)		
Length of longest watercourse (L)	75.18	km		Point Rainfall	Smithers and Shulze, 2002
Length from centroid to outlet (Lc)	37.6	km		ARF	Alexander, 1990
Height 0.85L		masl		Storm loss factor	HRU 1/72 Figure G1 & G2
Height 0.10L		masl			
Average channel slope (Savg)	0.007	-			
Catchment Index	34795	-			
Basin Lag	8.198	h	(HRU 1/72 Fig. F2)		

Design Rainfall

Return Period	10	years											
Duration (h)	8	10	12	16	18	20	22	24	26	28	30	32	34	36
Point rainfall (mm)	151.6	160.4	167.9	180.5	185.8	191.0	195.8	200.5	201.4	202.4	203.3	204.2	205.2	206.1
ARF	0.780	0.790	0.800	0.820	0.828	0.835	0.840	0.845	0.847	0.849	0.851	0.853	0.855	0.858
Catchment rainfall	118	127	134.3	148	154	159	164	169	171	172	173	174	176	177
Storm loss factor	0.747	0.737	0.727	0.712	0.706	0.701	0.696	0.691	0.689	0.688	0.687	0.686	0.684	0.683
Storm loss (mm)	88	93	98	105	109	112	114	117	118	118	119	120	120	121
Storm rainfall (mm)	30	33	37	43	45	48	50	52	53	54	54	55	55	56

Return Period	20	years											
Duration (h)	8	10	12	16	18	20	22	24	26	28	30	32	34	36
Point rainfall (mm)	180.9	191.3	200.3	215.4	221.7	227.9	233.6	239.2	240.3	241.4	242.6	243.7	244.8	245.9
ARF	0.780	0.790	0.800	0.820	0.828	0.835	0.840	0.845	0.847	0.849	0.851	0.853	0.855	0.858
Catchment rainfall	141	151	160.2	177	183	190	196	202	204	205	206	208	209	211
Storm loss factor	0.719	0.709	0.700	0.683	0.675	0.665	0.656	0.649	0.648	0.647	0.647	0.646	0.645	0.645
Storm loss (mm)	101	107	112	121	124	126	129	131	132	133	134	134	135	136
Storm rainfall (mm)	40	44	48	56	60	64	68	71	72	72	73	74	74	75

Return Period	100	years											
Duration (h)	8	10	12	16	18	20	22	24	26	28	30	32	34	36
Point rainfall (mm)	256.4	271.2	284.0	305.4	314.2	323.1	331.1	339.1	340.7	342.3	343.9	345.4	347.0	348.6
ARF	0.780	0.790	0.800	0.820	0.828	0.835	0.840	0.845	0.847	0.849	0.851	0.853	0.855	0.858
Catchment rainfall	200	214	227.2	250	260	270	278	287	289	291	293	295	297	299
Storm loss factor	0.650	0.643	0.636	0.627	0.625	0.620	0.616	0.614	0.614	0.614	0.614	0.614	0.613	0.613
Storm loss (mm)	130	138	145	157	163	167	171	176	177	178	180	181	182	183
Storm rainfall (mm)	70	77	83	93	98	102	107	111	111	112	113	114	115	116

Return Period	200	years											
Duration (h)	8	10	12	16	18	20	22	24	26	28	30	32	34	36
Point rainfall (mm)	293.0	309.9	324.5	348.9	359.0	369.1	378.3	387.5	389.3	391.1	392.9	394.7	396.5	398.3
ARF	0.780	0.790	0.800	0.820	0.828	0.835	0.840	0.845	0.847	0.849	0.851	0.853	0.855	0.858
Catchment rainfall	229	245	259.6	286	297	308	318	327	330	332	334	337	339	342
Storm loss factor	0.636	0.629	0.625	0.614	0.613	0.612	0.610	0.609	0.609	0.609	0.609	0.608	0.608	0.608
Storm loss (mm)	145	154	162	176	182	189	194	199	201	202	204	205	206	208
Storm rainfall (mm)	83	91	97	110	115	120	124	128	129	130	131	132	133	134

Return Period	PMF	HRU 1/72 Figure C4											
Duration (h)	8	10	12	16	18	20	22	24	26	28	30	32	34	36
Point rainfall (mm)	520.0	540.0	560.0	600.0	610.0	620.0	625.0	630.0	637.5	645.0	652.5	660.0	667.5	675.0
ARF	0.780	0.790	0.800	0.820	0.828	0.835	0.840	0.845	0.847	0.849	0.851	0.853	0.855	0.858
Catchment rainfall	406	427	448.0	492	505	518	525	532	540	548	555	563	571	579
Storm loss factor	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Storm loss (mm)	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Storm rainfall (mm)	406	427	448	492	505	518	525	532	540	548	555	563	571	579

A4 SPILLWAY DESIGN FLOOD ANALYSIS

A4.1 DESIGN FLOOD ESTIMATES

The following calculation methods were used:

- Unitgraph techniques using dimensionless regional unitgraphs (HRU, 1972).
- 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 A4.1**.

Table A4.1 Comparison of Inflow Flood Peaks (m^3/s)

Flood	Regional Unit Hydrograph Technique (1)	RMF Approach
1:100 year RI	1 740	1 750
1:200 year RI	1 935	2 150
RMF (Region 5.2)	n/a	3 240
$RMF_{+\Delta}$ (Region 5.4)	n/a	4 120
PMF	7 365	n/a

(1) Represents maximum inflow flood peak for critical storm duration.

It is evident from **Table A4.1** that the PMF is significantly higher than the RMF and the $RMF_{+\Delta}$, with the PMF in the order of 2.3 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 mention the use of the PMF method in designing spillways for large dams with a significant or high hazard rating, as in the case of Nwamitwa Dam. 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 equivalent K-value and the PMF/RMF ratio. In the case of Nwamitwa Dam, these upper limits are exceeded: The equivalent

PMF K-value is 6,2, while the PMF/RMF ratio is in the order of 3,0. Therefore, taking cognisance of the HRU 1/72-based PMF-related concerns expressed in the findings of the Water Research Commission (WRC) Study on Extreme Design Floods, the use of a SEF lower than the PMF-routed values determined during this study, but higher than the RMF (unrouted), 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 Tzaneen 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 4 120 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 a raised Tzaneen Dam.

For the 100 year RI and 200 year RI floods at Tzaneen 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 through application of the empirical RMF technique.

A4.2 FLOOD ROUTING

The simulated 100 year RI and 200 year RI flood hydrographs for a range of storm durations were routed through the raised Tzaneen Dam in order to determine the effect of attenuation on the simulated flood peaks.

The storage-area-elevation relationship was obtained from the First Dam Safety Inspection Report of Tzaneen Dam (DWAF, July 1999) and is shown in **Table A4.2**.

Table A4 2 Relationship between Stage, Area and Storage for Tzaneen Dam

Elevation (masl)	Area (ha)	Volume (million m³)
685	0.0	0.0
690	7.4	0.1
695	50.0	1.0
700	200.0	6.9
705	340.0	20.0
710	514.7	42.3
715	716.2	72.6
720	951.5	116.1
725	1229.4	170.3
730	1544.1	237.2
735	1900.0	316.1

The stage-discharge curve for the labyrinth spillway is shown in **Figure A4.1**.

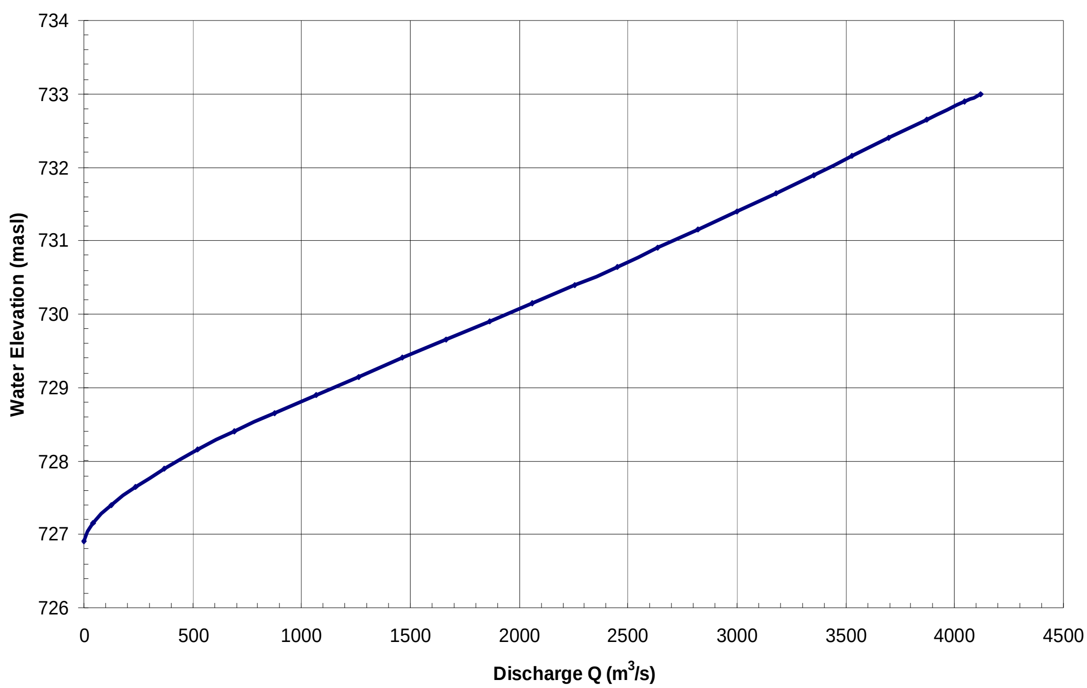

Figure A4.1 Stage Discharge Curve for Labyrinth Spillway

Based on the results of the above analyses, the following spillway floods have been selected to size the raised spillway:

Table A4.3 Flood Peaks for Labyrinth Spillway

Flood	Inflow Peak (m^3/s)	Outflow Peak (m^3/s)	Water Level (masl)
1:100 year RI	1 740	1 170	729.03
RDF (1:200 year RI)	1 935	1 360	729.27
SEF ($RMF_{+\Delta}$ - Region 5.4)	n/a	4 120	733.00

The SEF resembles an unrouted flood peak. In order to generate an incoming hydrograph for the Hydroplus option, the Probable Maximum Flood (PMF) hydrograph producing the highest outgoing flood peak of 4 700 m^3/s was scaled down in the ratio 4 120/4 700.

The routed hydrographs for the 1 in 100 year and 200 year RI floods are shown in **Figures A4.2** and **A4.3**.

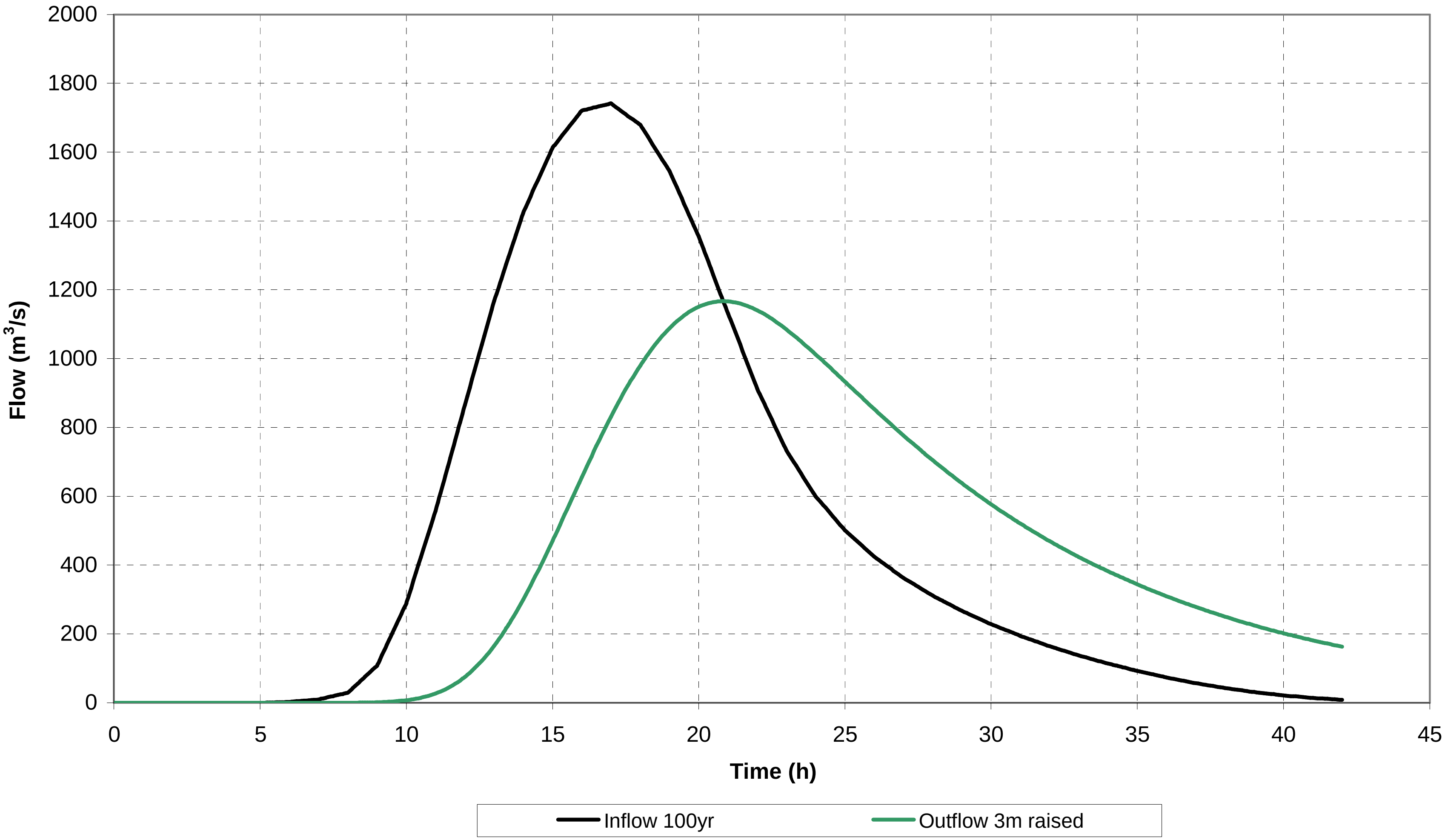

Figure A4.2 1 in 100 year RI Flood Hydrographs

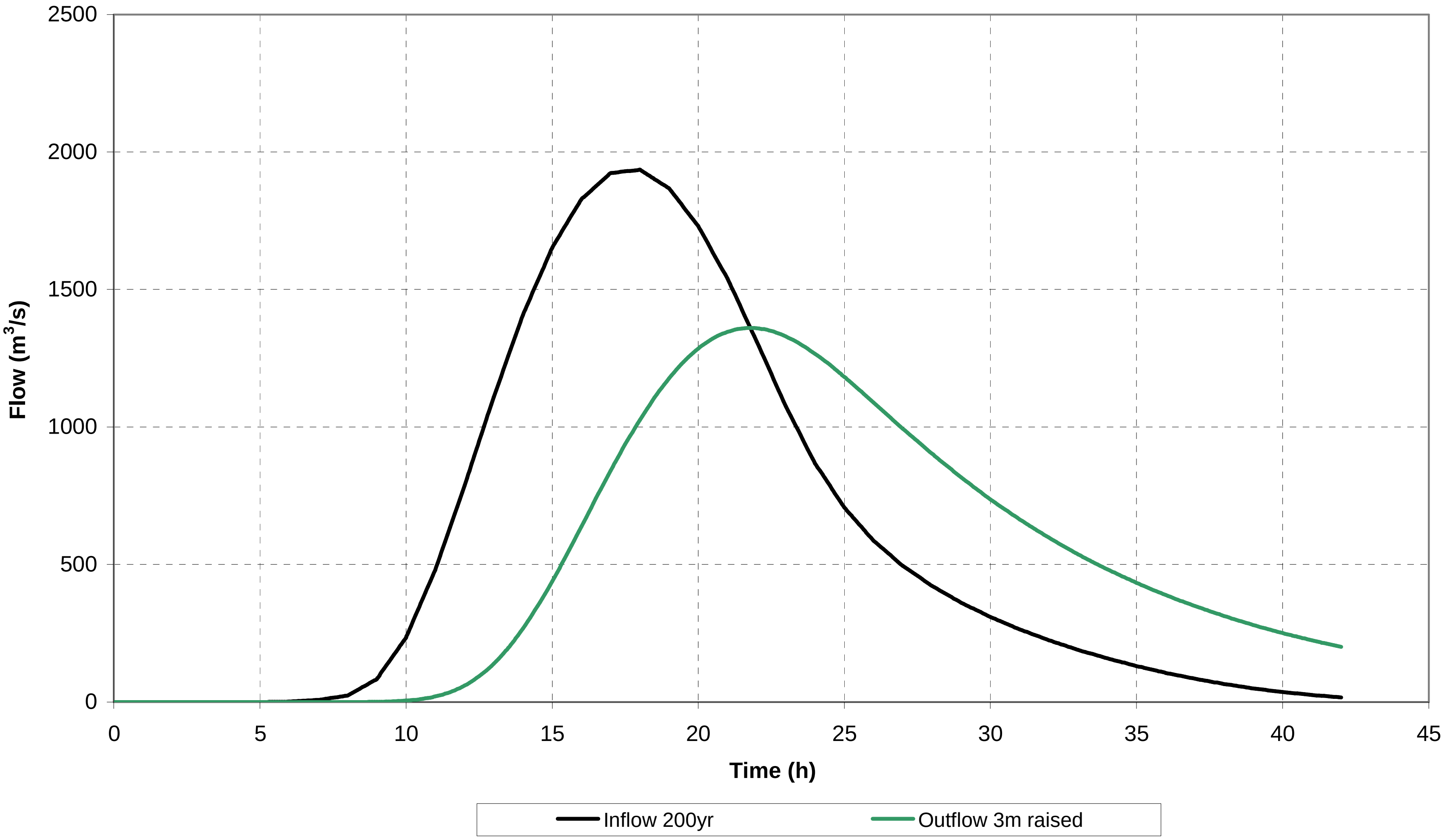

Figure A4.3 1 in 200 year RI Flood Hydrographs

APPENDIX B

HYDROPLUS PROPOSAL

TZANEEN DAM

USE OF THE FUSEGATE SYSTEM

REVISED PROPOSAL

KAMUZU II DAM (MALAWI) - 5.0m high labyrinth Fusegates

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May 2008

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I. PREAMBLE

Tzaneen dam, formerly known as the Fanie Botha Dam, consists of an earth embankment construction and a mass concrete gravity spillway. It is constructed across the Groot Letaba River, near the town of Tzaneen (Limpopo Province).

The possibility of raising the dam to meet the increasing demand for water using the Fusegate System was studied on several occasions since 1994. Upon the recent request of Ninham Shand, Hydroplus has been invited to update their last technical proposal dated October 2007 in order to reflect the following changes:

- raising of the FSL by 3.0m to RL726.90 m
- maintaining the NOC level at RL730.60 m
- taking into account the revised SEF (incoming peak 6170 m^3/s)
- taking into account the revised RDF (incoming peak 1935 m^3/s)

The Fusegate System consists of independent free standing blocks made in steel or concrete set on a flattened spillway weir and designed to tip-off during extreme flood events when the reservoir level reaches a predetermined elevation. The application of this system enables to have a progressive and controlled release of floods for exceptional flood conditions and ultimately prevents overtopping of the dam during the maximum design flood.

It appears that the above requirement could be met with use of 6.30m high labyrinth crested Fusegates. This proposal allows for increasing the storage capacity by 43.5 million m^3 while maintaining the maximum water level reached during a SEF event below the dam crest (which is not the case with the existing spillway design). In addition, the solution has been engineered so that no Fusegates tip-off before an RDF event. Such a low frequency ensures that there is a negligible effect on the firm yield of the raised reservoir (refer to the paper titled "Impact of Fusegate Rotation on a Reservoirs Firm Yield" by Hakin W.D., Pitman W.V., Watson M.D. which concludes that providing the first Fusegate doesn't tip before a 1 : 200 year flood event and water can be stored up to the raised Full Supply Level within approximately six month of the tip, (ie by installing stoplogs are constructing a replacement Fusegate) there is unlikely to be any reduction in the firm yield of the reservoir).

This proposal gives the main features of the proposed Fusegate arrangement.

II. BASIC DATA

The data used to perform the study are extracted from the documents supplied to HYDROPLUS and are summarized hereafter for ease of reference.

Tzaneen basin

II.1 DAM

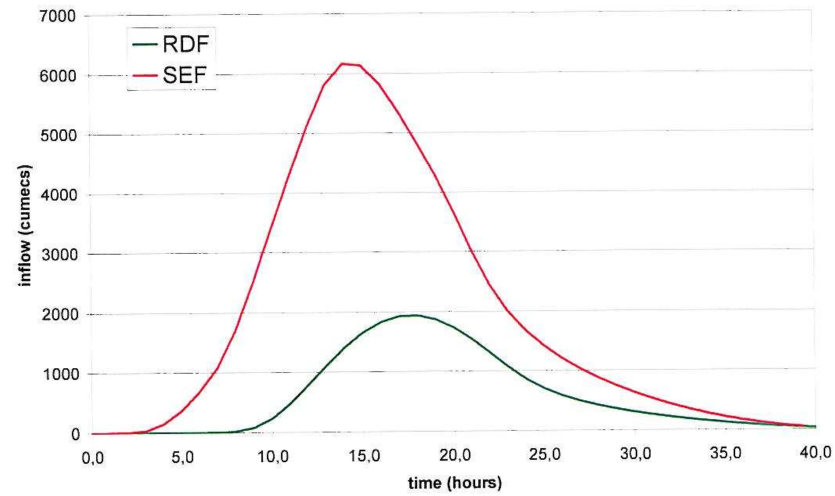
- Dam height above foundation: 50 m
- Non Over Spill Crest (NOSC) level : RL730.60 m
- Full Supply Level (FSL): RL723.90 m

II.2 SPILLWAY

- Spillway type: Free overflow (ogee crest)
- Spillway length : 91.44 m
- Spillway crest level: RL723.90 m
- Spillway discharge coefficient: 2.2

II.3 HYDROLOGY

- SEF incoming peak 6170 m^3/s
- RDF incoming peak 1935 m^3/s



II.4 RESERVOIR TOPOGRAPHIC DATA

Reservoir level (RLm)	Area (m^2)
712,47	7 100 000
721,04	11 700 000
727,70	16 000 000
731,52	20 000 000

II.5 PROJECT REQUIREMENTS

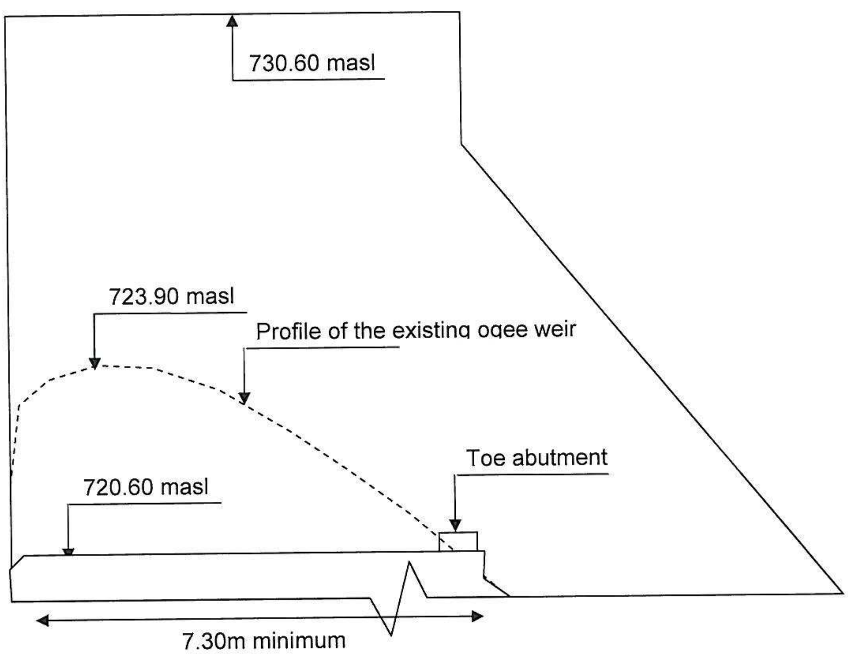
The proposal has been developed in such a way that:

- No modification to the dam embankment is required
- The Full Supply Level is raised by 3.0m to RL726.90 m
- No Fusegate tip-off before an RDF event
- The SEF is passed below the dam crest
- The release of floods is gradual and no artificial flood is created downstream.

III. SPILLWAY CONFIGURATION

III.1 MODIFICATION OF THE SPILLWAY SILL

The existing spillway sill would be lowered by 3.30m from RL723.90 m down to RL720.60m and flattened to provide a platform which would be equipped with Fusegates. The upstream edge of the sill will be rounded to enhance its hydraulic efficiency (R: 0.30 m). Small toe abutments blocks will be constructed to prevent the proposed Fusegates from sliding. Typical cross section of the modified spillway sill equipped is given hereafter:

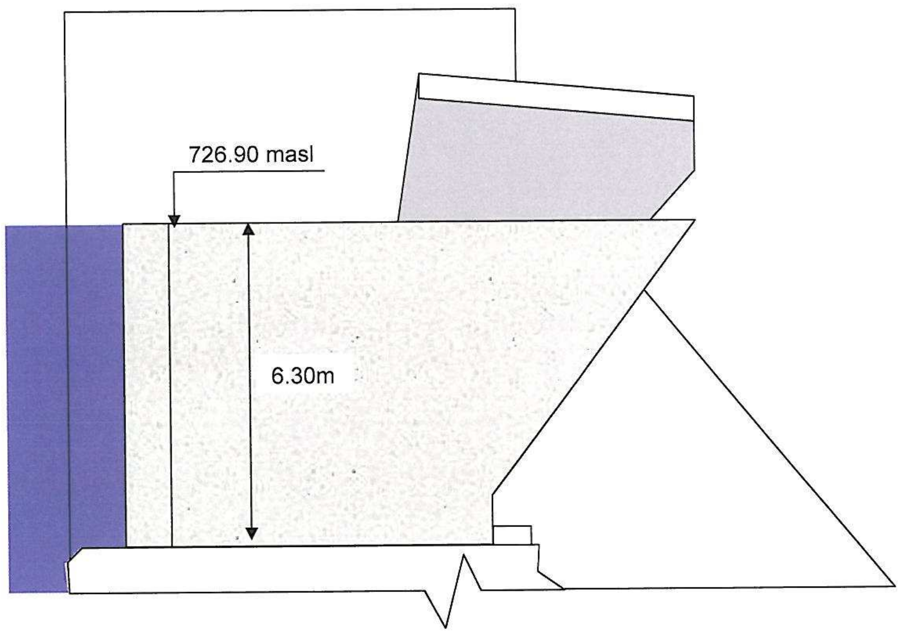


The main features of the modified spillway sill are given in the table which follows:

	Existing	Modified
Sill type	Ogee	Broad crested weir
Sill level	723.90 RLm	720.60 RLm
Minimum length (U/S to D/S)	N/A	7.3m
Spillway width	91.44m	91.44m

III.2 MODIFIED SPILLWAY CONFIGURATION

The storage is increased with the installation side by side of 6.30m high labyrinth crested Fusegates forming a watertight barrier. In such a configuration, the Full Supply Level is raised to the Fusegates crest elevation at RL726.90 m. Typical cross section of the modified spillway sill equipped with Fusegates is given hereafter:



The main features of the proposed Fusegates are given in the table which follows:

	Existing	Modified
Type of Fusegates	N/A	Labyrinth crested
Elevation of the Fusegates crest	N/A	RL726.90 m
Height / width of Fusegates	N/A	6,30 m / 9.14 m
Number of Fusegates	N/A	10
Number of tipping sequence	N/A	6
Lowest tipping level	N/A	RL729.10 m
Highest tipping level	N/A	RL730.10 m

III.3 MODIFIED SPILLWAY FUNCTIONING

The Fusegates would form a watertight barrier enabling the water to be stored up to the level of their overspilling crest. The medium to large floods are discharged over the Fusegates crest as shown in the following photo.

Photo of Saint Herbot Dam in France; 1.50m high labyrinth Fusegates

However, in case of exceptional floods (above the RDF at Tzaneen), the Fusegates would tip off progressively when the reservoir reaches predetermined elevations (see the following photo of live experiment performed to tip one Fusegate by an artificial flood).

All the Fusegates would have tipped off by SEF event, thus allowing:

- To safely pass the design flood,
- To have a progressive release of water during major flood events,
- To trigger only the tipping of the Fusegate required to safely passing a given flood.

III.4 FUSEGATES CONFIGURATION

It is proposed to use the following materials for Fusegates construction:

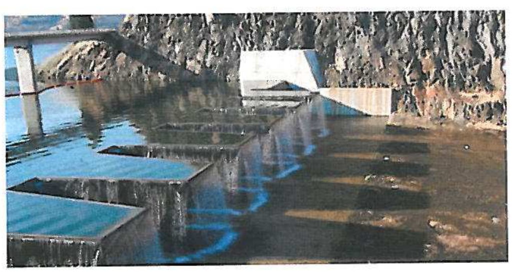
- Main structure: cast-in-situ reinforced concrete
- Seal gasket: EPDM
- All steel items: mild steel with corrosion protection[1]

The selection of such materials offers the following benefits:

- high life expectancy (minimum 100 years for the concrete structures),
- minimum maintenance cost,
- only annual visual examination required in addition to the twenty year inspection,
- little vulnerability to vandalism.

It is proposed to use the labyrinth type of Fusegates in a Wide, Moderate Head (WMH) Configuration, which offers a high hydraulic efficiency. Such type of Fusegates has been the subject of numerous model tests to assess its behaviour under all normal and abnormal working scenarios.

Various projects have been implemented considering similar Fusegates (in terms of either height or configuration). Some of our references are given here after:



Terminus dam, California

The 6.50m high labyrinth crested Fusegates are used to increase the reservoir storage capacity and flood protection potential

Kamuzu dam, Malawi

The 5.0m high labyrinth crested Fusegates enable to double the reservoir storage capacity.

[1] All components made of mild steel will be protected with an epoxy coating (sand blasting SA2.5, zinc primary coating 100 µm, secondary epoxy coating 70 µm, finish coating 60 µm).

III.5 MAINTENANCE OF THE FUSEGATE SYSTEM

Inspections of the Fusegate System are typically part of the routine inspections carried out on the dam. During each of the following inspections the operator in charge of the inspection should complete the relevant inspection form provided by HYDROPLUS. The different inspections are defined below with their maintenance actions and periodicity.

- *Routine Inspections*

  These are undertaken in order to derive information on the overall aspect of the system and on some crucial points of the Fusegates. These need visual assessment requiring no direct access to the Fusegates but observation from remote vantage points with the use of binoculars.

- *Annual Inspections*

  These are undertaken in order to derive information on specific areas such as the base chambers and wells and need close visual assessment requiring direct access to the Fusegates.

- *Twenty years Inspections*

  These are undertaken to check the integrity of the whole system in detail. They require the reservoir level to be below the bottom of the Fusegate during a period of natural low water level. The twenty years inspection is to be undertaken in three distinct stages as outlined below.

  - Preparation:
    1. Dismantling of all the vertical and horizontal seals,
    2. Jacking of each Fusegate to ensure no bonding has developed,
    3. Cleaning all parts of the Fusegates by means of a high pressure water jet.
  - Inspection:
    1. Visual inspection for corrosion or concrete attack of the Fusegates,
    2. Visual inspection of the concrete sill and side walls.
  - Maintenance action:
    1. Removal of floating debris,
    2. Replacement of damaged nuts and bolts,
    3. Repair of any damaged concrete,
    4. Repair to the corrosion protection,
    5. Removal of any blockages in the inlet well, shaft and base chamber.
    6. Replacement of rubber seals.

IV. HYDRAULIC PERFORMANCES

IV.1 BASIC ASSUMPTIONS

1. It is assumed for the flood routing studies that the reservoir level at the beginning of the flood is at the Fusegate crest level (RL 726.90m).
2. The discharge over the spillway sill (with all Fusegates having tipped) per unit length can be analyzed by the following formula:

$$Q = Cd.H^{3/2}$$

In which Q is the flow through the spillway in m^3/s, H is the upstream head (in m). C_d is the discharge coefficient of the spillway sill. At Tzaneen dam, it has been assumed that the discharge coefficient of the modified spillway is equal to 1.86. This coefficient is much lower than the one of the existing spillway (2.20) due to the platform required for accommodating the Fusegates which does not offer a high hydraulic efficiency.

IV.2 FLOOD ROUTING SIMULATION

The result of the flood routing simulations are given in the following table:

Proposed spillway equipped with Fusegates					
Flood Designation	% of SEF	Max Inflow (m^3/s)	Max outflow (m^3/s)	Max Water Level (RL m)	# of units having tipped
RDF	N/A	1935	1137	729,69	0/10
1st Tipping	31%	1940	1225	729,10	0/10
2nd Tipping	38%	2315	1680	729,30	1/10
3rd Tipping	45%	2750	2145	729,50	2/10
4th Tipping	52%	3230	2625	729,70	4/10
5th Tipping	64%	3955	3440	729,90	6/10
6th Tipping	77%	4775	4280	730,10	8/10
SEF	100%	6172	5385	730,60	10/10

It will be noted from the above table that:

- No Fusegate will tip-off before the RDF event.
- The SEF is passed just below the dam crest.

Flood routing simulations were also undertaken through the existing spillway for comparison purposes:

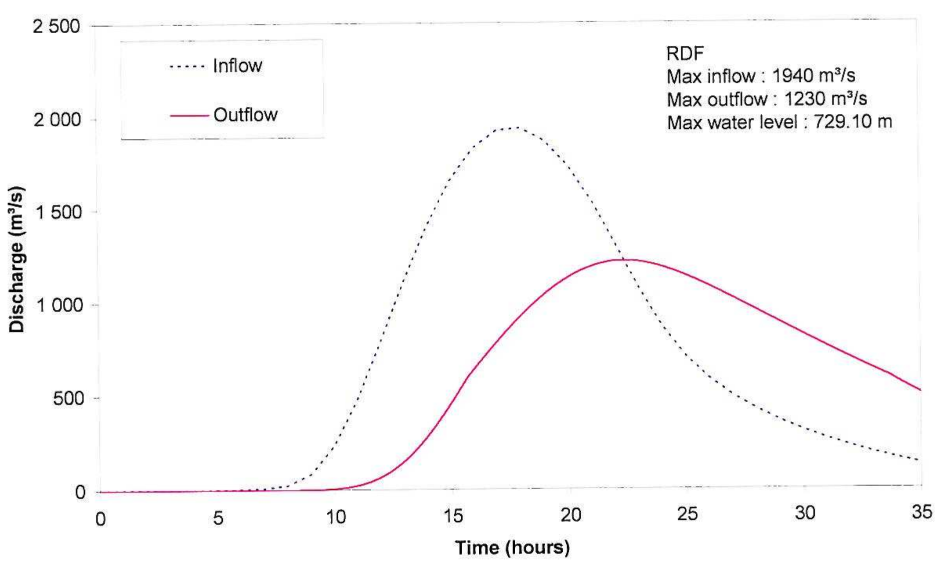
Existing spillway					
Flood Designation	% of SEF	Max Inflow (m^3/s)	Max outflow (m^3/s)	Max Water Level (RLm)	# of units having tipped
RDF	N/A	1935	1054	726.90	N/A
SEF	100%	6270	3970	731.15	N/A

It will be noted from the above table that:

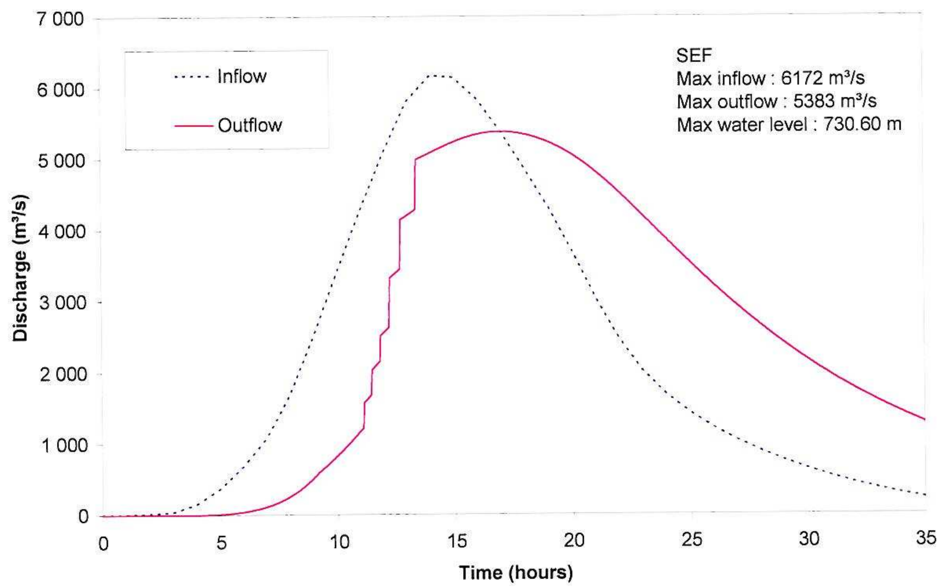
- Max outflow for RDF is only slightly increased because of the spillway modifications,
- The SEF is passed 0.55m above the dam crest

IV.3 FLOOD HYDROGRAPHS

Hydrographs for the RDF and SEF are presented on the following pages.



TZANEEN DAM
USE OF THE FUSEGATE SYSTEM
PRELIMINARY PROPOSAL



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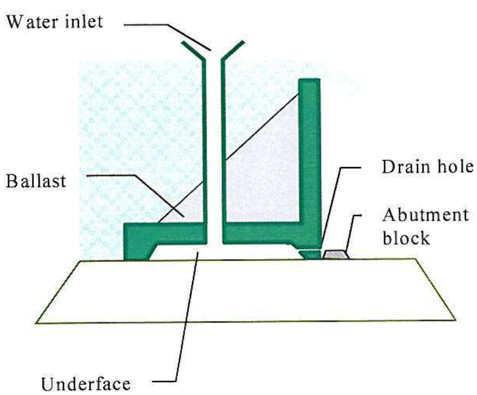
V. HYDROPLUS CONCEPT

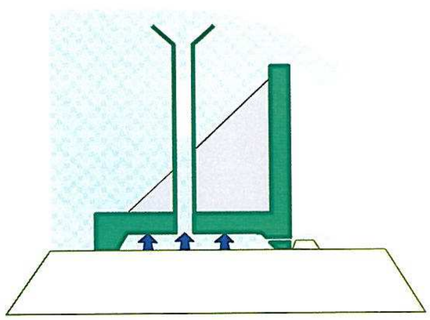
V.1 FUNCTIONING PRINCIPLE



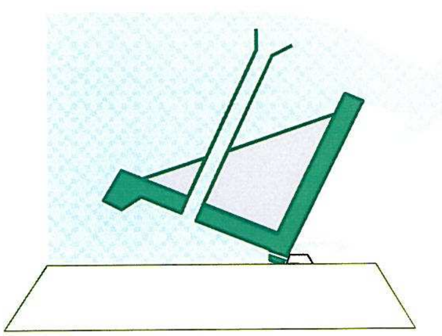
The Fusegate System is based on the following concept:

- Fusegates are free-standing units installed side-by-side on a spillway sill to form a watertight barrier.
- They bear against small abutment blocks set in the sill to prevent them from sliding before they are required to rotate (under extreme flood conditions).
  There is a chamber in the base of each Fusegate, with drain holes to discharge incidental inflow (due to leaking seals for example).

An inlet well on the upstream side of the Fusegate crest discharges water into the chamber when the headwater reaches a predetermined level (Well lips on individual Fusegates are actually set at different levels).



During very large floods, water entering the chamber over the inlet well causes an uplift pressure to develop in the chamber. The uplift pressure, combined with the hydrostatic pressure (acting from left to right on the adjacent diagram) is sufficient to overcome the restraining forces and the imbalance causes rotation of the unit off the spillway. The Fusegate is then washed away clear of the spillway by the flood.



If the water level continues to rise after the first breach more Fusegates can rotate, all according to pre-determined upstream water levels until eventually there are no more units remaining and the spillway is free to pass the original maximum design flood. Until rotation of the first Fusegate, (for floods of extremely low risk of occurrence), the user has the benefit of the additional storage. Each Fusegate has a different overturning level, precisely determined by the height of the water inlet and its own unique stability.

V.2 STABILITY

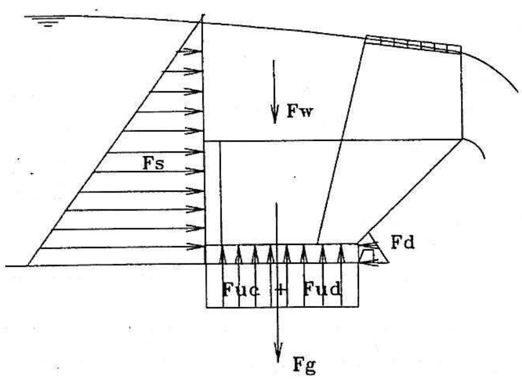
- ***Fusegate stability***

The stability of a Fusegate is dependent upon considerations of sliding and overturning.

- Sliding is prevented by the construction of toe pedestals in the spillway sill.
- The Fusegate rotates about the downstream edge when the overturning moment exceeds the stability moment:

The overturning moments are caused by the hydrostatic forces and the uplift forces (Uplift forces, which act on the Fusegate, are developed in the chamber during large floods feeding the well).

The stability moment is a function of the weight of the Fusegate and the water therein.



Fs	upstream hydrostatic force
Fg	dead load of the Fusegate
Fw	water load above the Fusegate
Fd	downstream hydrostatic force
Fuc	uplift force within the chamber
Fud	uplift forces under the beams

- ***Stability margin***

The stability margin of a Fusegate is defined as the difference between the stability and the overturning moment for any assumed upstream water level.

The calculations show that the high stability margins, which are achieved, allow the Fusegates for being not likely affected by the impacts of floating debris as well as for withstanding quite large earthquakes.

- ***Minimum tipping level***

During a very large flood, the well admits water into the chamber, causing the uplift pressure to rise significantly. The Fusegate starts to tilt about the toe pedestals, and the chamber is then open to the reservoir, causing a rapid rise in uplift pressure so that the

Fusegate finally overturns. When it first lifts off the sill, uplift in the chamber cannot exceed a maximum pressure, which ranges somewhere between the upstream head and the head along the sides of the units. The magnitude of this uplift has been determined by calculation and verified by model testing.

The maximum uplift curve allows for determining the *minimum tipping level* below which the Fusegate cannot overturn. It will be noticed that the alternative is engineered so that this level is obtained above the Full Supply Level.

As long as headwater remains below this *minimum tipping level*, the Fusegate will not overturn even if water enters the chamber through the well accidentally, or if the impact of an extremely heavy body causes the upstream edge to lift off the sill momentarily.

- ***Critical situation***

The safety analyses conducted to investigate the operational reliability of the Fusegates consider the following extreme critical situations:

- Upstream seal completely destroyed
- Drain holes completely blocked.

These studies highlight that these extremely unlikely cases of malfunctioning would not lead to a possible early tip-off. Furthermore, these are entirely theoretical situations, which could occur only through wilful damage or complete dereliction of the dam inspection and maintenance system.

V.3 RELIABILITY OF OPERATION

- ***Safety features***

The Fusegate System has valuable safety features inherent to the concept and not shared by other spillway control systems

- they overturn automatically, responding to those physical forces acting upon them.
- they are entirely self-operating and do not require any source of power to operate.
- Only minimal maintenance is required compared to other mechanical gates.

The minimum tipping level (defined earlier) constitutes an engineered safeguard which is of vital importance for the people living and working downstream of the dam since it removes the risk of a sudden downstream artificial flood.

Besides the normal operating condition, Fusegates are engineered in such a way that no malfunction (upstream seal completely destroyed or drain holes completely blocked) could lead to a tip-off before water is admitted through the well.

- ***Waves, floating debris and impacts***

The effect of waves and impacts have been the subject of specific research in reputable hydraulic laboratories (such as Utah State University, Tennessee Valley Authority), which have demonstrated their minimal incidence on the system. Floating debris are simply discharged over the Fusegates crest when sufficient spillage occurs and do not have any significant impact on the Fusegate stability

Example of impact of large floating debris (Tennessee Valley Authority)

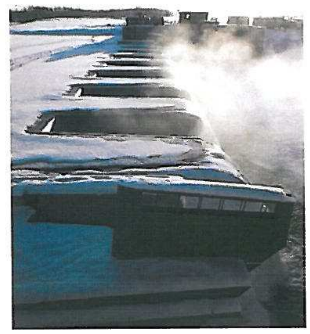
- ***Ice-affected environments***

The effect of ice is examined with reference to tests undertaken in the hydraulic laboratories of the National Research Council (NRC) in Newfoundland, Canada and of the Institute of Energy Structures in Moscow, Russia. Generally speaking, thermal expansion of ice and ice run off generally has very little influence on the Fusegates stability.

The behaviour of the Fusegates in ice-affected environments has been observed on Khorobrovskaïa scheme (Russia). The 4 off 1.80m high Fusegates have successfully withstood the pressure of the 0.6m thick ice sheet and to the subsequent ice run-off.

Model tests at the NRC

Khorobrovskaya in winter



- ***Earthquake***

Seismic effects are examined in each individual project using a pseudo-static approach or a finite element analysis if required. However, the stability of the Fusegates is usually sufficient to prevent problems induced by earthquakes.

The behaviour of the Fusegates during major a seism has been observed in Gujarat State in India that was hit early 2001 by a 7,6 magnitude (on the Richter scale) seism. None of the Fusegates installed on the four dams located within a 50 miles radius from the epicentre were affected.

- ***Advantages over other systems***

Unlike most spillway control systems, the construction of the Fusegates themselves can often be undertaken in the state where the project takes place and will thus benefit the local industry.

The Fusegate System does not involve any chemical product, which could later involve a pollution of the river stream (such as oil, paint...). In addition, the construction materials are chosen to mitigate the visual impact of the rehabilitation.

The Fusegate massive structure offers little vulnerability to vandalism and, considering their high stability margin, to terrorist threat.

The Fusegates will be mainly fabricated in concrete. The maintenance and life expectancy of the System are therefore optimal. In addition, the Fusegate units do not have any moving part and are therefore less subject to wear and tear and to aging.

VI. FINANCIAL PROPOSAL

VI.1 SCOPE OF WORKS

Hydroplus herein proposes two stages for Hydroplus's scope of work: the Concept Design and the Design/ Construction stage as outlined below:

- ***Conceptual report***

Conceptual design and production of a full report including stability analysis of Fusegates, flood routing, specifications and sketches and civil work technical requirements/ specifications. After the completion of the Concept Design stage it will be possible for HYDROPLUS to offer a fixed price for the design and construction stages.

- ***Design / engineering***

- Detailed design of the Fusegates including structural design, construction drawings for the Fusegates and inlet wells and seal fixing arrangement
- Spillway sill : general layout and minimum specifications
- Detailed design Report
- Maintenance manual

- ***Fabrication and installation***

- Fabrication of the steel components and corrosion protection
- Fabrication in situ of the Fusegates
- Supply of all the components required for the horizontal and vertical seals
- Installation of the inlet wells and seal fixing arrangement
- Setting of the concrete ballast

- ***Exclusion***

It would be noted that our commercial proposal does not include the following works:

- Preparation of access roads
- Modification/construction of the new spillway sill

VI.2 BUDGET ESTIMATE

Hydroplus proposes to perform the above Scope of Works for a lump price of:

- Conceptual design stage:	ZAR 0.9 million
- Design and Construction stage :	ZAR 41.0 million

The Conceptual design fee is a fixed fee not subject to change (i.e. no contingencies are required on this portion). The D & C prices are for budget estimate purposes and are accurate to 10%. The D & C estimates will be confirmed with fixed lump sum prices after the Concept Design stage.

The above budget prices do not include taxes and any specific insurances other than Hydroplus's standard Euro 1.5 million Professional Indemnity and Euro 10 million Public Liability.

APPENDIX C

IMPACT OF FUSEGATE ROTATION ON A RESERVOIR'S FIRM YIELD

1998

Impact of fusegate rotation on a reservoir's firm yield

W.V. Pitman, Stewart Scott, South Africa
M.D. Watson, Department of Water Affairs and Forestry (DWA & F), South Africa
W.D. Hakin, Hydroplus International, South Africa.

A study has been carried out on behalf of the DWA & F involving 30 river catchment areas in South Africa to assess the possible impact on the firm yield of a reservoir caused by the rotation of a fusegate. The results can be applied to any catchment in the country.

The installation of fusegates on the spillway crest of an existing dam can be a cost-effective way of increasing the storage capacity of a reservoir and hence its firm yield*. However one area of concern regarding the installation of fusegates is that, after the first tipping, some or all of the increased storage could be lost. If such a loss were to occur just before the critical period** there may be insufficient time for the water to return to the new storage capacity level. In such cases the new firm yield would be reduced. To address this concern, a theoretical analysis was carried out on 30 catchment areas throughout South Africa, and in addition on two existing dams, one in Malawi and one in South Africa. The analysis, undertaken on historical streamflow and on stochastically generated time series, comprised the following steps:

- Establish the critical period determining the firm yield for three different dam sizes, that is, 50 per cent, 100 per cent and 200 per cent of mean annual runoff (MAR).
- Select a plausible reinstatement period immediately before the critical period for reinstallation of the tipped fusegate(s).
- Identify floods (if any) with a return period of 10 years or greater, that occurred during the reinstatement period.
- Summarise the results and establish trends.

The results showed clearly how the risk of rotation impacting on firm yield increases with the size of reservoir in relation to the MAR. The stochastic analysis also showed how increasing the return period of the flood, designed to initiate rotation, reduces the risk of impact on firm yield, regardless of the dam size. The study also demonstrated the degree to which the risk of impact on firm yield can be minimized by reducing the reinstatement period. No regional trends emerged from the study and there was also no correlation with the coefficient of variability of the historical streamflow.

It can therefore be concluded that the results can be applied to any catchment area in South Africa (and probably elsewhere). These results indicate that the risk of fusegate rotation impacting on firm yield is extremely low, especially if the first tip is designed to occur for floods of 100 year return period or greater, and that the reinstatement period is only of a few months' duration.

The rotation of fusegates

To maintain the capability of the spillway to pass extreme floods, it is essential that the fusegates rotate according to a predetermined tipping sequence to provide adequate passage for the floodwaters. The design return period for each sequence of rotation depends on several factors, such as the original spillway configuration, the increase in full supply level and the given flood hydrology. The first tipping sequence is usually designed to occur in the event of a flood with a 100 year return period, or greater.

A possible concern regarding the installation of fusegates to increase storage capacity is that, after the first tipping, some or all of the increased storage will be lost and that this loss could cause a reduction in the firm yield of a dam. As the firm yield is determined by the water balance of the dam over the critical period, any incidence of fusegate tipping that affects this bal-

Table 1: Details of selected catchments

Catchment number	Catchment area (km²)	MAR (10^6m^3)	MAR (mm)	CV (std.dev./MAR)
A21	6336	209.5	33	0.785
A31	6684	94.5	14	1.156
A42	8395	312.3	37	0.882
A180	4203	113.2	27	1.190
B51	6170	46.6	7.5	0.821
B81	4952	381.0	77	0.797
C11	8791	548.1	62	0.689
C33	9843	37.1	3.8	2.364
C52	17366	185.6	11	1.374
C70	6656	192.3	29	0.851
D17	7179	1108.6	154	0.417
D35	5638	53.1	9.4	1.962
D61	13405	29.1	2.2	1.134
E40	2722	27.1	9.9	1.251
G10	8912	913.3	102	0.476
G40	3058	502.5	164	0.391
H20	832	99.2	119	0.760
J11	5646	37.9	6.7	1.500
K10	911	65.1	71	0.653
L82	2820	148.2	53	0.929
N30	1934	35.1	18	1.148
Q41	1292	24.7	19	1.228
R20	1286	108.5	84	0.908
S20	1607	65.7	41	0.739
T20	2600	392.2	151	0.512
U20	4439	739.9	167	0.560
V11	2635	915.9	348	0.393
W21	5274	464.4	88	0.843
W51	3894	570.5	147	0.538
X21	30912	507.9	164	0.416

*The firm yield is defined as the maximum supply that can be sustained throughout the historical period for which the hydrology is available.

**The critical period is defined as the period spanning the time from which a reservoir was last full to when it reached maximum drawdown. The length of the critical period can range between a few months and several years, depending on the capacity of the reservoir in relation to the average inflow.

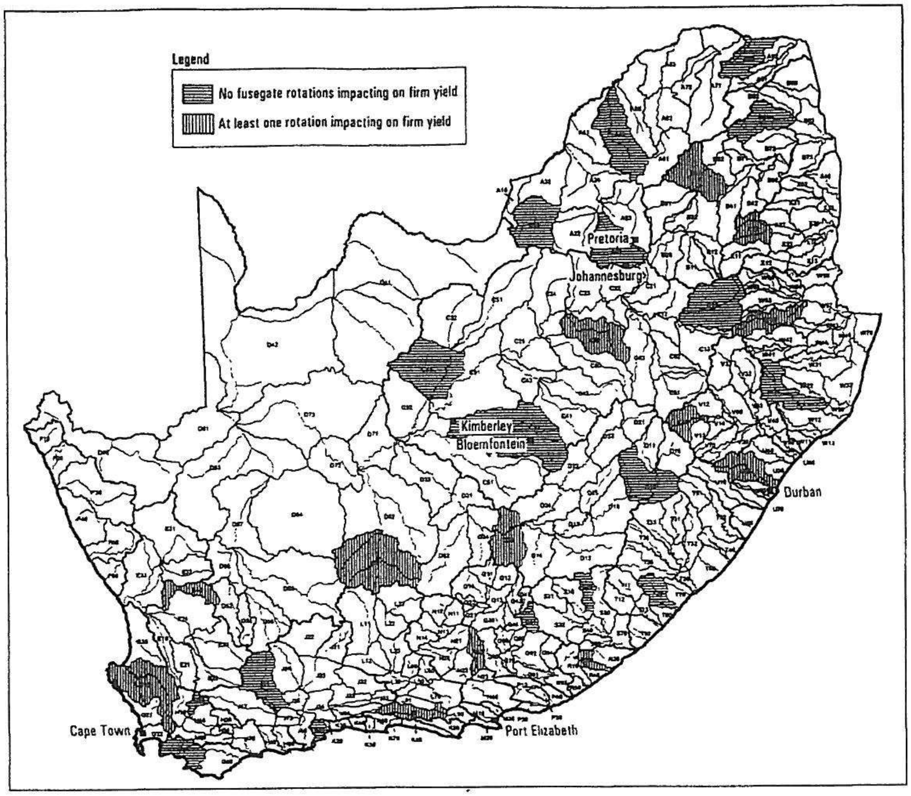


Fig. 1. Tertiary catchments selected for historical analysis.

ance will also affect the firm yield. In the light of this concern, a study was undertaken to evaluate the probability of fusegate rotation causing a reduction in the firm yield of a dam.

Methodology of the study

The purpose of the study was not to investigate specific reservoirs, but to undertake a theoretical exercise covering the full range of hydrology likely to be encountered across South Africa. To this end, 30 catchments were selected as shown in Fig.1. Seventy year (1920 to 1989 hydrological years) time series of monthly natural flows for each catchment were obtained from the Water Resources of South Africa 1990 database (Midgley, Pitman and Middleton, 1994[1]). Relevant hydrological characteristics of each time series are given in Table 1.

For each time series, the seven largest monthly flows were identified and assigned return periods ranging from 70 years (largest) to 10 years (7th largest). Although the largest event in 70 years would not necessarily have a return period of 70 years, the assumption is reasonable when one is analysing a large number of catchments. A further assumption is that a flood peak of return period R would occur in the month associated with a volume of return period R. Although this would not always be the case, it was considered to be a reasonable assumption for the purpose of this analysis. Furthermore, this assumption greatly simplified the stochastic analysis.

Long unbroken records that include all flood peaks can be analysed to verify this assumption. Unfortunately such records are sparse, since major floods often destroy the gauging equipment. As an example, Table 2 contains the results of an analysis of monthly flood peaks and volumes for a 35 year record on the Orange river. Closer examination of this Table shows that the two highest peaks occurred in the months with the highest volumes. For less severe events, the return periods of volume and peak do not always match but, since fusegates are usually designed to tip only during major floods, the assumption does indeed appear to be plausible.

The next step was to establish the firm yield sustainable over the full 70 years and to note the associated critical period. For this exercise, three theoretical dam sizes were analysed: 50 per cent MAR, 100 per cent MAR and 200 per cent MAR. The dam sizes are appropriate to the 'raised' situation, that is, with fusegates in place.

The critical periods span the time to maximum drawdown from when the dam was last full. It is extremely unlikely that the critical period would include a

Table 2: Analysis of flood peaks and volumes*

Ranked maximum monthly flows		Peak discharge in month	
Volume ($10^6 m^3$)	Return period (years)	Discharge (m^3/s)	Return period (years)
6493	36	7703	36
5825	18	6825	18
4710	12	3149	5.1
4037	9	3593	9
3557	7.2	5465	12

*Guage D3H003 on the Orange river

flood (that is a monthly volume) with a return period of 10 years or more. Even if this were to occur, the flood would be completely absorbed by the reservoir since, by definition, no spillage takes place during the critical period. However there is a distinct chance that a significant flood could occur a short time before the critical period begins. If this were to happen, it is possible that the fusegates (or stoplogs if provided) could not be reinstated and the loss in storage made good before the critical period began. To cater for such an eventuality, the critical period was extended backwards in time. If the fusegate rotated within the extended period before the critical period, it was assumed that insufficient time would be available for reinstallation or stoplogging, and storage recovery to the raised full supply level, hence the critical period would be extended with a consequent reduction in firm yield. (In this study no attempt has been made to quantify the reduction in firm yield.)

Two assumptions were made with regard to the extension of the critical period. A conservative period of five months was tested, in addition to a more realistic time scale of two months. The five-month period was selected to embrace the wet season preceding the onset of the critical period. With the use of stoplogs, however, it was assumed that storage could be reinstated within two months.

The position of each of the seven largest monthly flows was tested to ascertain whether overlap occurred with the extended critical periods. This check for overlap was carried out for the critical periods associated with each of the three dam sizes analysed.

Historical analysis

The procedure described was applied to each of the 30 time series, and note was taken of any occurrences of the seven largest monthly flows within the extended critical periods. Results of the historical analysis are summarised in Table 3 and shown in graphical form as Figs. 2a and 2b. Probabilities are expressed as annual probabilities, which indicate the risk of rotation impacting on firm yield in any given year. The reinstatement delay indicated in the diagrams is equal to the critical period extension.

Table 3: Historical analysis summary

Flood return period (years)	Rotations for dam sizes of (per cent MAR)			Total no of rotations
	50	100	200	
70	–	–	–	0
35	L82 (5)*	L82 (5) V11 (2)	L82 (4) U20 (3)	5
23.3	–	D35 (3) E40 (3)	D35 (3) E40 (3) W51 (2)	5
17.5	–	–	–	0
14	–	B51 (4) R20 (2)	B51 (2)	3
11.7	C70 (4)	G10 (5)	G10 (5) N30 (2)	4
10	–	–	D61 (2) X21 (4)	2
Total failures (five month extn.)	2	7	10	19
Total failures (two month extn.)	0	2	4	6

The code before the bracket refers to the catchment analysed (see Table 1) and the figure in brackets is the time elapsed (in months) between rotation and startof critical period.

Fig. 2a shows a definite trend of increasing probability of fusegate rotation having an impact on firm yield with increasing dam size. This trend can be attributed to the behaviour of dams of varying size (relative to the MAR) when operated on a firm yield basis. Small dams remain relatively full for most of the time, and a significant flood is not required to fill them. On the other hand, large dams fill on relatively few occasions, and it usually requires a large flood to fill them. Since a critical period begins when the dam is full, it is therefore more likely for a flood to occur just before the critical period in the case of a large dam.

The relationship between flood return period and the probability of the flood impacting on firm yield (that is to say, occurring within the extended critical period) appears to be fairly random (see Fig.2b). However there were no cases of 70 year floods overlapping with critical periods in any of the catchment areas analysed. The absence of a clear relationship between increasing

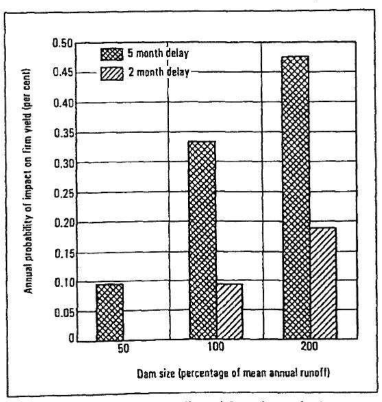


Fig. 2a. Historical analysis: effect of dam size and reinstatement delay on the probability of rotation impacting in the firm yield.

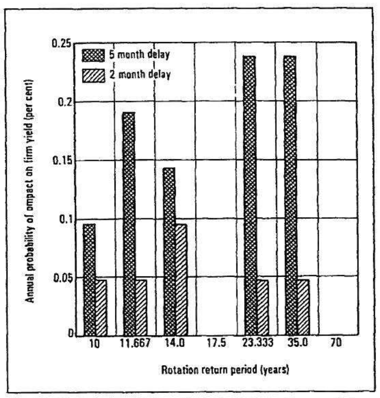


Fig. 2b. Historical analysis: effect of rotation return period and reinstatement delay on the probability of rotation impacting on the firm yield.

Table 4: Flood classification

Class	Range in return period	Expected number of (in 2870 years)
1	10-15	96
2	15-20	48
3	20-30	48
4	30-50	39
5	50-100	28
6	100-200	14
7	>200	14

flood return period and a decreasing probability of flood impact on firm yield may be attributed to the relatively small sample size of 30 historical sequences.

In 13 of the 30 catchments analysed at least one case of rotation occurred in the extended five month critical period. The distribution of these catchments (see Fig.1) appears to be completely random, which suggests that none of the regions can be seen as having an elevated risk of fusegate rotation close to the start of a critical period.

A shortening of the critical period extension (reinstatement delay) from five to two months reduces the total number of rotations by about two-thirds. The implication of this result is that the probability of rotation impacting on the yield can be reduced significantly by prompt action such as stoplogging.

Stochastic analysis

Since fusegates are usually designed to rotate for floods with return periods greater than 70 years, it was necessary to undertake a stochastic analysis to ascertain the risk of such floods impacting on the firm yield.

For the stochastic analysis, 41 seventy-year sequences were generated for each of the 30 catchments. The GENMAC suite of computer programmes was used for this purpose (McKenzie and van Rooyen, 1997[2]). The 41 sequences were then analysed in a similar fashion to the historical time series with the excep-

Table 5: Stochastic analysis summary (five month extension of critical period)

Return period period (years)	*Rotations for dam sizes of (per cent MAR)			Total no of rotations
	50	100	200	
10-15	39	47	73	159
15-20	19	32	47	98
20-30	17	19	34	70
30-50	6	12	30	48
50-100	4	10	21	35
100-200	1	5	18	24
>200	1	2	8	11
Total	87	127	231	445

*Of a total of 1230 (30×41) generated sequences

Table 6: Stochastic analysis summary (two month extension of critical period)

Return period period (years)	*Rotations for dam sizes of (per cent MAR)			Total no of rotations
	50	100	200	
10-15	16	15	29	60
15-20	7	12	25	44
20-30	9	11	14	34
30-50	2	4	9	15
50-100	2	5	8	15
100-200	0	2	8	10
>200	0	0	4	4
Total	36	49	97	182

*Of a total of 1230 (30×41) generated sequences

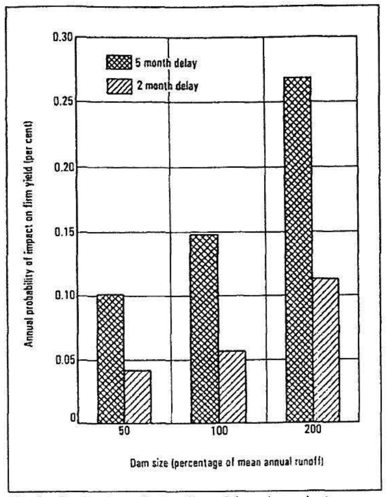


*Fig. 3a. Stochastic analysis: effect of **dam size** and reinstatement delay on the probability of rotation impacting on the firm yield.*

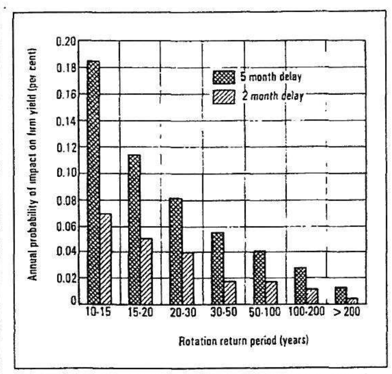


*Fig. 3b. Stochastic analysis: effect of **rotation return period** and reinstatement on the probability of rotation impacting on the firm yield.*

tion of the following step.

The seven largest floods in each sequence were assembled together to form a list of 287 (that is, 7 × 41) values. These were ranked in descending order and assigned appropriate return periods. For example, the return period of rank 1 is 2870 years (the total number of years generated) and that of rank 287 is 10 years (2870/287). The floods were then classified according to seven ranges, shown in Table 4.

Flood events falling within the five-month and two-month periods before the critical period were noted according to dam size and return period classification. Results of the stochastic analysis are summarized in Tables 5 and 6.

The data in Tables 5 and 6 indicate a clear trend of increasing likelihood of fusegate rotation with

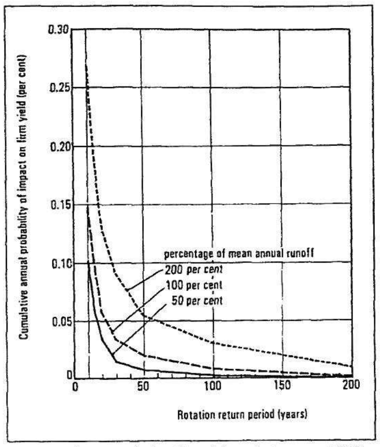


Fig. 4a. tochastic analysis: effect of dam size (per cent MAR) and rotation return period for a five month reinstatement delay.

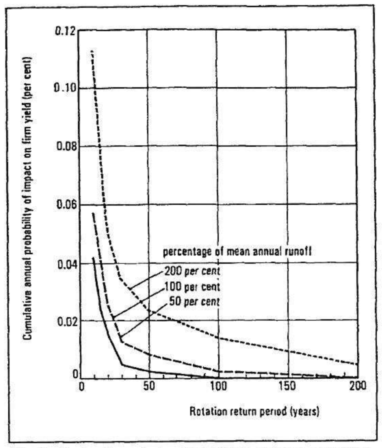


Fig. 4b. Stochastic analysis: effect of dam size (per cent MAR) and rotation return period for a two month reinstatement delay.

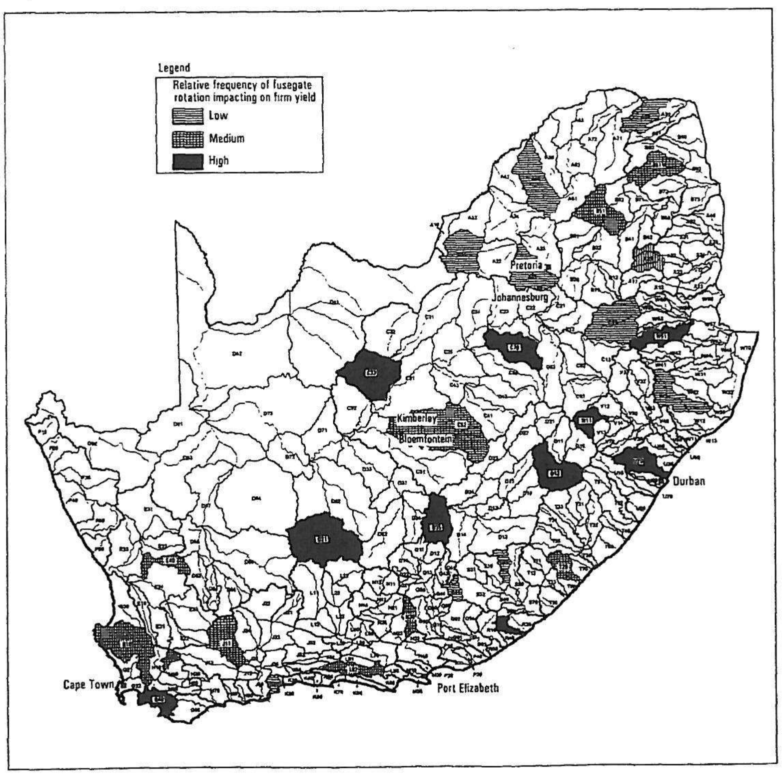


Fig. 5. Tertiary catchments: stochastic analysis.

increasing dam size (in relation to the MAR), supporting the results of the historical analysis. The Tables also reflect a decreasing likelihood of rotation with increasing return period. Such a result is not unexpected, but the historical sample size was too small to give any indication of such a trend.

The results in Tables 5 and 6 are shown in graphical form in Fig. 3a (effect of dam size) and Fig. 3b (effect of rotation return period), with occurrences expressed as probabilities. A comparison of Figs. 2a and 3a shows the probabilities derived from the stochastic analysis to be generally lower than those derived from historical data. Figs. 3a and 3b show the probabilities for the two-month reinstatement delay to be of the order of 40 per cent of those derived for the five-month period. It should be emphasized again that the probabilities given are annual probabilities, which indicate the risk of rotation impacting on firm yield in any given year.

Figs. 4a and 4b show the cumulative probability of rotation of a fusegate designed for a flood of given return period for dam sizes of 50 per cent, 100 per cent and 200 per cent MAR. (The cumulative probability is defined here as the cumulative probability of all events equalling or exceeding the return period in question). The diagrams show clearly how the risk of rotation just before a critical period increases with increasing dam size to MAR ratio, and decreasing return period. For example, Fig. 4a shows that a fusegate designed to tip in a 50-year flood would have an annual risk of rotation (within five months of a critical period) of slightly more than 0.05 per cent if the dam size were 200 per cent MAR. However for a dam size of 50 per cent MAR, the annual risk is less than 0.01 per cent. If the return period of rotation of the 50 per cent MAR were to be reduced to 15 years the annual risk would increase to about 0.05 per cent.

Fig. 4b indicates clearly the reduced risks associated with fusegate rotation within two months of the critical period. A fusegate designed to tip in a 50 year flood would have an annual risk of rotation of about 0.025 per cent if the dam size were 200 per cent MAR. For a dam size of 50 per cent MAR, the risk is less than 0.005 per cent. If the return period of rotation with the 50 per cent MAR dam were to be reduced to 15 years, the increased risk would still be less than 0.025 per cent on an annual basis.

The results presented in Table 4 and Figs. 3 and 4 are based on the stochastic analysis of the full sample of 30 catchments. To give a qualitative assessment of vulnerability to fusegate rotation impacting negatively on the firm yield, the catchments have been classified into three groups of ten, indicating low, medium and high (relative) probability of rotation. This simple classification is shown in Fig. 5. As was the case for the historical analyses, no regional pattern emerged. There is also no correlation with the variability (CV) of the historical time series. For example, the 'high' category embraces catchments of very low CV (for example, G40 and U20) and very high CV (for example, C33 and D61).

Case studies

Kamuzu II dam, Malawi

It is proposed to increase the height of this dam by 5 m with the installation of fusegates. This will increase the capacity from 8.9 to 19.8 × 10^6m^3. The first fusegate is designed to tip at a discharge of 784 m^3/s, which is approximately equal to the 200-year flood. A stochastic analysis yielded a minimum gap of 90 months between a flood of 200 year return period or greater, and the start of the critical period, thus implying a negligible risk of rotation within the permitted two month reinstatement period. Such a result was to be expected, as the increased capacity is only 14 per cent MAR.

Midmar dam, South Africa

There were originally two proposals for heightening this dam with fusegates. The existing capacity of 159.2 × 10^6m^3 can be increased with fusegates to 247.2 × 10^6m^3 by raising the height of the dam by 4 m or to 257.2 × 10^6m^3 by raising the height by 4.57 m. When increased, the capacity will be approximately 125 per cent MAR. For both options, the first rotation is designed to occur for an inflow of 1800 m^3/s, which is slightly larger than the 200-year peak. A stochastic analysis generated one case out of a total of 41 of a 36-year flood occurring within the two-month reinstatement period, but not a single case of a 200-year (or greater) flood occurring before a critical period. These results suggest a negligible risk of fusegate rotation impacting on firm yield, which is in accordance with the theoretical study.

Summary and conclusions

Generally the study indicates the extremely low risk of floods of sufficiently large magnitude to cause a fusegate to rotate actually occurring just before the critical period.

The results show the risk of reduction in firm yield increasing with the size of reservoir. As a rough approximation, doubling the reservoir size to MAR ratio will also double the risk. The stochastic analysis also showed how the probability of impacting on firm yield can be reduced by increasing the return period of rotation of the first fusegate, regardless of the dam size. The probability of impacting on yield can also be minimized by reducing the period required to reinstate storage after rotation has occurred.

The results of the study are summarized in Figs. 4a and 4b, which present the interrelationship between reservoir size, rotation return period and probability of rotation impacting on firm yield. Figs. 4a and 4b can be considered to apply to any catchment in South Africa, since no regional or other trends emerged from the study. Fig. 4a can be used to aid decision making by showing, for example, that there is an annual risk of about 0.01 per cent (equivalent to a less than 1 per cent chance in a period of 70 years) that the increased yield of the dam (with increased storage equivalent to one MAR or less) would be reduced by a tip with the first fusegate designed to rotate for floods in excess of the 100-year event. This is based on the conservative assumption that the increased storage capacity cannot be regained in the five months following the tip.

Fig. 4b clearly shows how the probability of fusegate rotation impacting on yield can be reduced if this five-month period can be shortened to two months. The 0.01 per cent annual risk associated with the five-month period (see previous paragraph) can be reduced to less than 0.005 per cent.

The main shortcoming of the study lies in the use of monthly data throughout. Floods are short-duration events and, for example, the 100-year flood would not necessarily coincide with the monthly inflow of similar return period, as has been assumed in the analysis. It should also be appreciated that no attempt has been made to assess the degree of impact on firm yield. In practice it may be possible to restore some of the additional storage originally created by the installation of fusegates. The degree of impact will be governed by how much of the additional storage can be restored before the onset of the critical period.

It is suggested that the techniques used in this study could be applied to other dams where the use of fusegates to increase storage capacity is under consideration. ◊

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1. **Midgley, D.C., Pitman, W.V., and Middleton, B.J.**. "Surface Water Resources of South Africa", Water Research Commission Report Nos. 298/1/94 to 298/6.2/94 (13 volumes); 1994.
2. **McKenzie, R.S., and Van Rooyen, P.G.**, "GENMAC Users Guide". BKS. Pretoria. South Africa; 1997.

Malcolm Watson is currently chief engineer for the Water Resources Planning Department for the Central region of South Africa (Orange and Vaal river basins). After graduating from the University of the Witwatersrand with a BSc(Eng) in 1979 and a Msc in 1981, he worked for a consulting company specialising in water engineering until 1994 when he joined the Department of Water Affairs and Forestry.

Dr W.V. Pitman is acknowledged as being one of the leading hydrologists in southern Africa. In addition to being the founder of the widely used Pitman catchment hydrological model and its successor, the WRSM90 model, he has played a prominent role in the development of southern African water resources and flood design manuals. He and his team have undertaken numerous hydrological and water resource studies. Dr. Pitman has been honoured by his peers with several prestigious awards for his contributions in the field of hydrology.

Bill Hakin is the Chief Executive of the South African branch of Hydroplus International. Since graduating from the University of Bradford in 1982 with a BSc (Eng), he has worked for various contracting and construction engineering firms.

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W.V. Pitman

M.D. Watson

W.D. Hakin

APPENDIX D

COST ESTIMATES

APPENDIX D.1

COST ESTIMATE FOR RAISING WITH HYDROPLUS FUSEGATES

RAISING OF TZANEEN DAM WITH HYDROPLUS FUSEGATES

FSL = 726.9 masl

NOC = 733.0 masl

No	DESCRIPTION	UNIT	RATE Apr 09 Rand	QUANTITY	AMOUNT Rand
1	**Demolition of top 3.3m of existing crest (including establishment)**	Sum	Escalated to April 2009		2 816 000
2	**Fusegates**				
	(a) Conceptual design stage	Sum	Escalated to April 2009		1 000 000
	(b) Design and Construction stage	Sum	Escalated to April 2009		45 210 000
3	**Miscellaneous** (% of 1-2)	%	5	49 026 000	2 451 300
	SUB TOTAL A				51 477 300
4	**Contingencies** (% of sub total B)	%	10	51 477 300	5 147 730
	SUB TOTAL B				56 625 030
5	**Planning design & supervision** (% of sub total B)	%	5	56 625 030	2 831 252
	SUB TOTAL C				59 456 282
6	**Net Present Value of Future Maintenance**				
	(a) Maintenance over 200 years Assume R500 000 every 20 years @ discount rate of 6%				227 000
	(b) Fusegate replacement Assume R8 000 000 every 200 years @ discount rate of 6%				666 000
	TOTAL COST (excl. VAT)				**59 456 282**

Raising of Tzaneen Dam PDR - Costing - Apr 09

APPENDIX D.2

COST ESTIMATE FOR RAISING WITH LABYRINTH SPILLWAY

RAISING OF TZANEEN DAM WITH LABYRINTH SPILLWAY

FSL = 726.9 masl

NOC = 733.0 masl

No	DESCRIPTION	UNIT	RATE Apr 09 Rand	QUANTITY	AMOUNT Rand
1	**Demolition of top 7.5m of existing crest (including establishment)**	Sum	Escalated to April 2009		7 130 000
2	**Concrete Works**				
	(a) Formwork				
	(i) gang formed	m^2	220	9 228	2 030 155
	(ii) intricate	m^2	305	1 718	523 862
	(b) Concrete				
	(i) mass	m^3	530	4 842	2 566 260
	(ii) structural	m^3	840	6 516	5 473 549
	(c) Reinforcing	t	9 600	586	5 629 936
3	**Miscellaneous** (% of 1-2)	%	10	23 353 762	2 335 376
	SUB TOTAL A				25 689 139
4	**Preliminary & General** (% of items 2 and 3)	%	40	18 559 139	7 423 655
	SUB TOTAL B				33 112 794
5	**Contingencies** (% of sub total B)	%	10	33 112 794	3 311 279
	SUB TOTAL C				36 424 073
6	**Planning design & supervision** (% of sub total C)	%	15	36 424 073	5 463 611
	TOTAL COST (excl. VAT)				**41 887 684**

Raising of Tzaneen Dam PDR - Costing - Apr 09

JET DEMOLITION (PTY) LTD

PO Box 35866
Northcliff 2115
South Africa

1332 Clubhouse Street
Maraisburg Ext 2
Johannesburg

National (011) Tel: 495-3800
International (27-11) Fax: 495-3838
Company Registration Number: 1994/006895/07
Directors: JR Brinkmann, EA Brinkmann
E-mail: demolish@mweb.co.za
Website: www.jetdemolition.co.za

Heavy Industrial & Commercial Demolition – Implosions – Dismantling – Controlled Explosive & Mechanical Methods
Site Reclamation & Environmental Rehabilitation – Earthworks – Decontamination of Radiological Hazards
Asbestos Stripping – Reinforced Concrete Breaking

Ninham Shand (Pty) Ltd
Private Bag X136
Centurion
0046

Attention: Mr Herman Smit
via e-mail

26 June 2008

DEMOLITION OF SPILLWAY CREST AT TZANEEN DAM

Dear Sir

Thank you for your invitation to submit a feasibility study quote for this work; herewith our proposal.

1) Background Information on Jet Demolition (Pty) Ltd

Our group of companies (Jet Demolition (Pty) Ltd; Jet Technologies (Pty) Ltd; Blasting and Geotechnologies (Pty) Ltd (Blastech) has been involved in the demolition of mine and industrial plants for the past 18 years and we are the foremost demolition company in South Africa. We use both advanced mechanical and explosive methods in demolition of all types of concrete and steel structures. Our company has developed and holds international patents for a range of specialized shaped charges for controlled demolition of large structures. We also employ a wide array of mechanical demolition methods using modern machinery.

Our Internet Web Site illustrates the use of mechanical and explosive demolition methods within close proximity of occupied structures and operating equipment. **We specialize in mining infrastructure demolition and rehabilitation, heavy industrial demolition, and large commercial demolition projects. At present we have over 60 items of mobile demolition plant and are the best equipped for demolition work of any company in South Africa. This equipment includes 23 excavators equipped with specialized demolition tools (hydraulic hammers, concrete crushers, concrete pulverizers, steel shears, grapples, buckets).** Our main demolition equipment is renewed on a 3 yearly basis. This policy, together with stringent maintenance and a fully equipped plant support department, contributes strongly to safe, reliable and efficient operation on our demolition projects.

At present we employ in excess of 250 personnel. We have extensive experience in partial demolition of large mining and industrial plants still in operation where access is restricted and vital plant services are present in close proximity. We have undertaken controlled blasting of large structures under the most demanding conditions (within 1,5 m of operating explosive plants) and carry out heavy mechanical demolition on top of, or in close proximity to, operating plant and services on a regular basis.

Our main areas of expertise include:

- **Heavy Industrial Demolition, Mining Infrastructure Demolition and Rehabilitation, and Large Commercial Demolition**
- **Fast Track Shutdowns and Demolition under 24/7 Conditions**
- **Turn-key Projects for Purchase, Demolition and Rehabilitation of Industrial and Mine Plants**
- **Controlled Explosive Demolition of Large Structures Under Demanding Conditions**
- **Dismantling of Equipment and Structures**
- **Asbestos Removal**
- **Decontamination of Radiological Hazards**
- **Site Reclamation and Environmental Rehabilitation**
- **Salvage and Recycling of Redundant Equipment and Scrap Metals**

Attached please find an e-brochure on our company. Also, please visit our Internet Web Site to view illustrations of recent projects and our state-of-the-art demolition equipment.

We have previously successfully carried out this type of project at the Midmar Dam.

2) Demolition of Spillway Crest at Tzaneen Dam

2.1) Scope of Work and Method of Demolition

- Demolish the spillway crest as given in the drawing supplied to enable construction of a new labyrinth spillway. The newly created horizontal surface to have a tolerance of about 25 mm to enable accurate placement of the rebar for the new construction.
- Preserve the upstream corner during demolition.
- Preserve the integrity (i.e. do not cause damage) to the remaining concrete of the dam.
- Spoil most of the concrete upstream of the dam wall during demolition.
- Demolished concrete which falls downstream is to be collected and taken to a tip area on site.
- The dam level will be lowered by 1-1,5 m below the final demolition line by DWAF prior to the start of demolition and maintained at this level during demolition.
- We will make use of controlled precision blasting, controlled mechanical demolition and diamond saw and diamond wire rope cutting to carry out this work.

2.2) Schedule of Work

The estimated time required to carry out the demolition works is 3 months.

2.3) Financial (base June 2008)

We offer to carry out this work at a fixed lump sum cost of R 6 481 000-00 (Excl VAT).

3) CIDB

Our current CIDB rating is 6SE and we expect this to be increased soon to 7SE upon the registration of a further Professional Engineer within our company.

4) Insurance

Our insurance brokers are Alexander Forbes (contact person Mr Hugo du Plessis, ph: +27-11-669-3009). We carry plant all risks, SASIRA and R 50 000 000-00 public liability insurance. Additional insurance can be arranged on request. We have a no claims history on our insurance policy.

5) Financial and Bank Details

Our bankers are Nedbank and our relationship manager is Mr L Vorster (ph: +27-11-214-3600) who can be contacted to discuss and verify our financial stability.

6) Safety

On any demolition job the greatest concern is the safety of persons. The largest safety risk in demolition is posed by gravity, with the two manifestations of this being the risks of persons falling from height and the risks of materials falling from height onto personnel. Hence, our approach in demolition work is to use advanced blasting techniques where appropriate, or alternatively controlled mechanical methods to put structures onto the ground, from where they are sectioned into smaller elements using large excavators equipped with mechanical demolition tools (grapples, hydraulic hammers, concrete crushers, concrete pulverisers, steel shears and buckets). Much effort has been spent by our company in the development of explosive and mechanical tools and methods to minimize the risks of demolition, and we have repeatedly demonstrated our abilities on difficult jobs. **Our safety system, our safety performance and our safety commitment are far in advance of any other company in South Africa that undertakes demolition.**

Our demolition workers are responsible and proud of their work. Our safety programme is used to provide all workers with the necessary training, motivation and back up to ensure a safe work environment during demolition works which are inherently dangerous. A full set of our safety and standards programme can be viewed on request.

Jet Demolition has invested heavily in modern equipment and the development of our advanced safety programme to achieve a workforce which delivers safe, rapid and cost effective demolition projects. **We are the only demolition company in South Africa with a NOSA Grading and OHSAS 18001 Accreditation.**

Our company places a great emphasis on safety related to all aspects of our demolition operations and our continued improvement in this area is summarised as follows:

2001:NOSA 4 Star Grading; DIFR equivalent to 5 Star

2002:NOSA 5 Star Grading;
NOSA Award for 2nd Place for Best Safety System for Contractors

2003:NOSA 5 Star Grading;
NOSA - NOSCAR Award;
NOSA Award for 1st Place for Best Safety System for Contractors, Category F

2004:NOSA 5 Star Grading;
NOSA - NOSCAR Award;
Winners of NOSA International Competition, Category F, Construction, Health & Safety System;

2005:OHSAS 18001 SYSTEM ACCREDITATION;
NOSA 5 Star Grading;
NOSA – NOSCAR Award;
Winners of NOSA Award for Best Occupational Health Practitioner, `Category 10

2006:OHSAS 18001 SYSTEM ACCREDITATION
NOSA 5 Star Grading;
NOSA – NOSCAR Award

2007:OHSAS 18001 SYSTEM ACCREDITATION
NOSA 5 Star Grading;
NOSA – NOSCAR Award

7) Closing Remarks

In closing I would like to add that our high level of service and our commitment to customer satisfaction have resulted in a high level of repeat business and we are very proud of our reputation in the industry:

*** Our company has never left a job uncompleted;**
*** We have never been kicked off a job;**
*** We have never made a claim for extra compensation on any of our jobs;**
*** We have never liquidated any of our companies; and,**
*** We have never entered into litigation with any of our customers.**

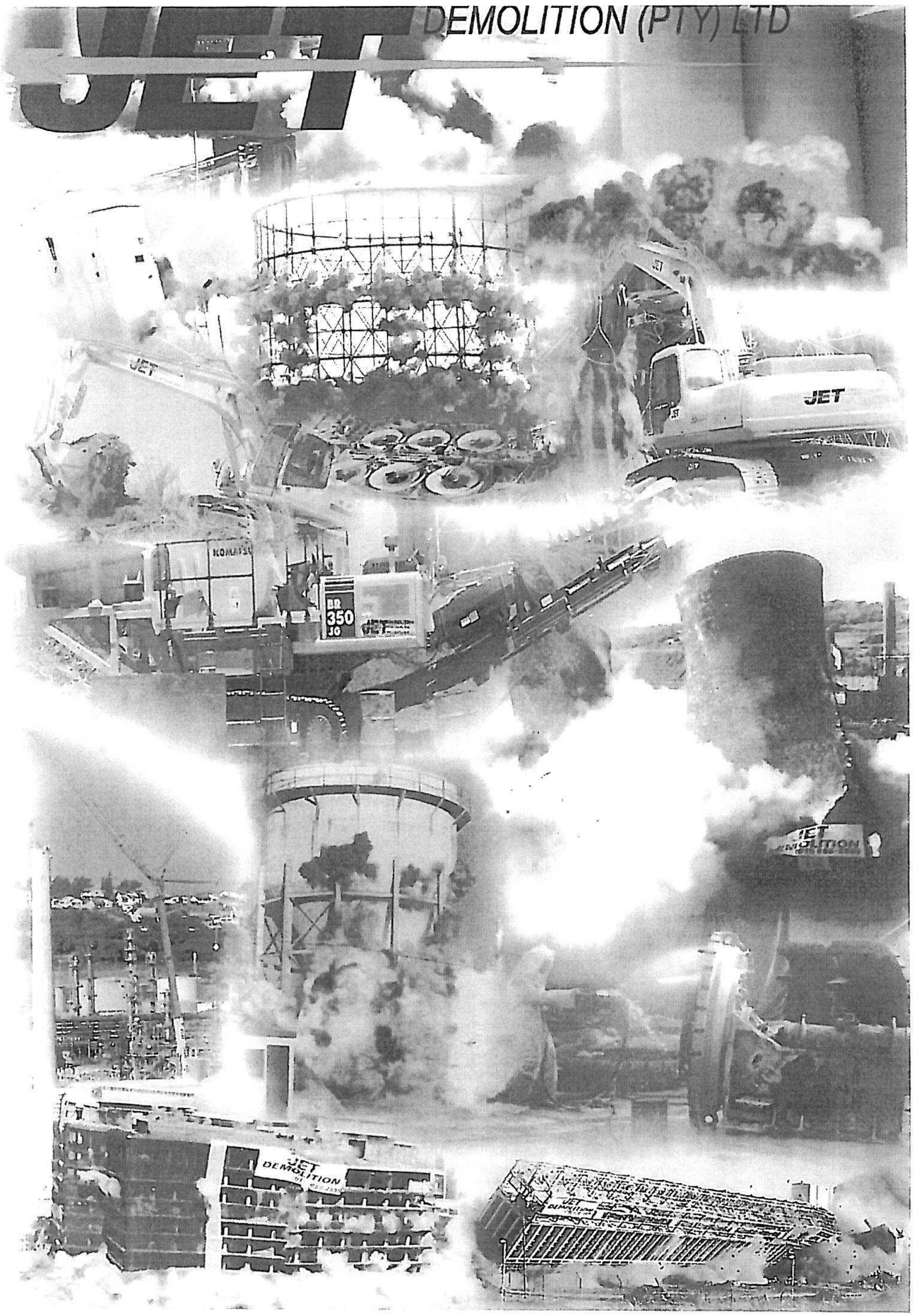
These attributes are not very common in the demolition industry.

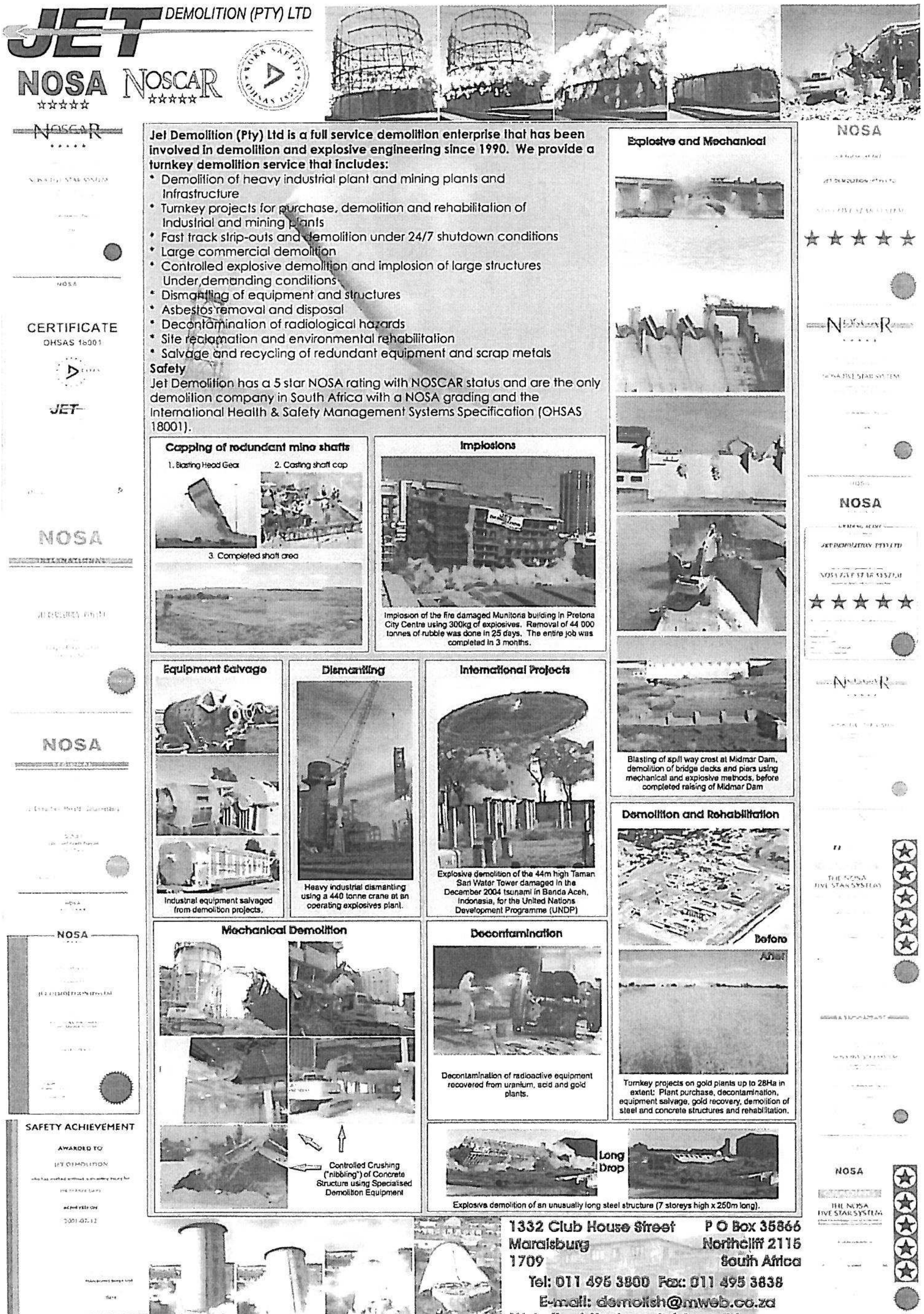
Please do not hesitate to contact me should you have any questions or require additional information.

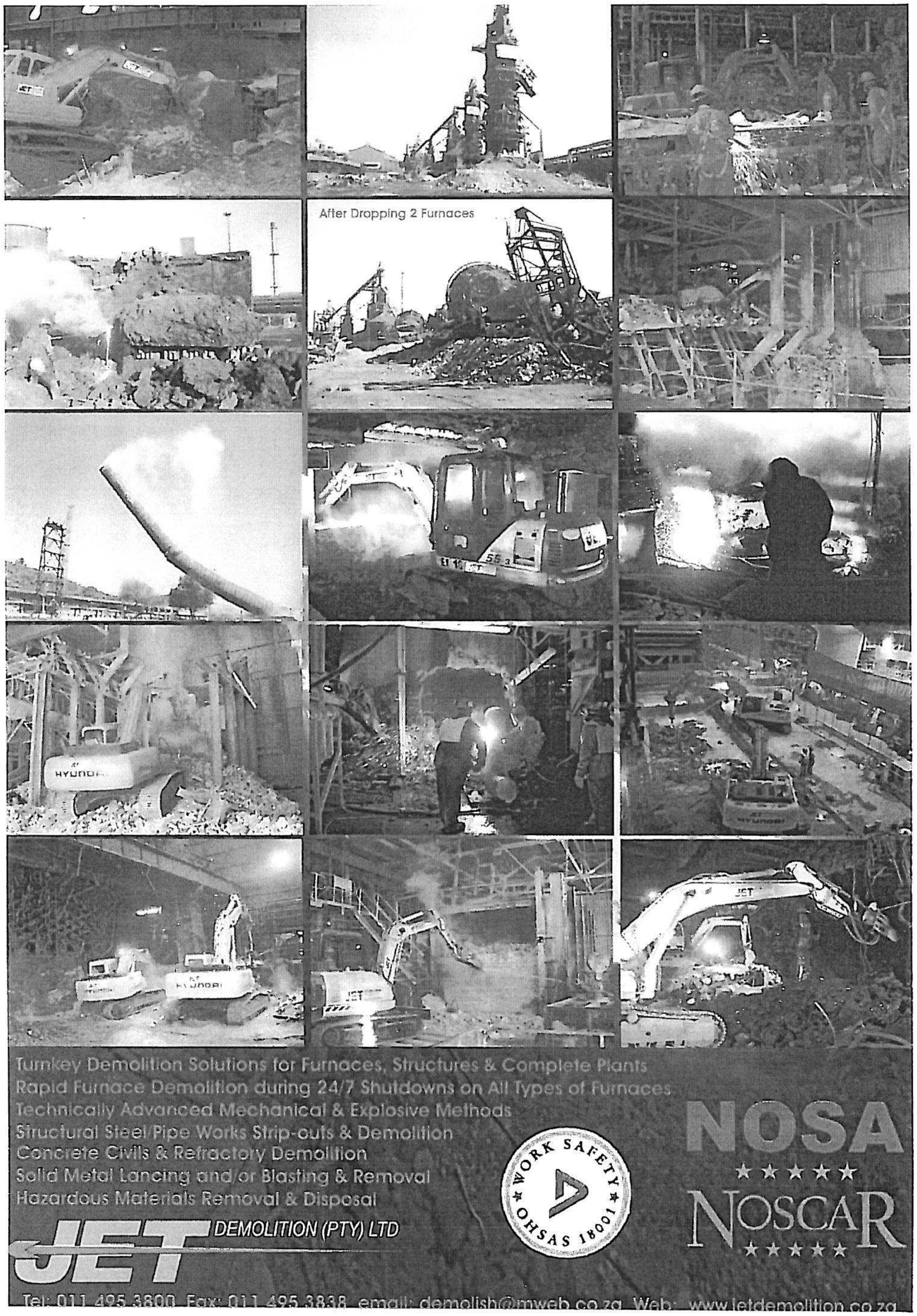
Thank you for your time and consideration and for the opportunity to submit this proposal

Yours sincerely

J R Brinkmann Pr Eng
Managing Director


JET DEMOLITION (PTY) LTD
JET
JET
350
JET DEMOLITION
JET DEMOLITION


JET DEMOLITION (PTY) LTD
NOSA
NOSCAR
Jet Demolition (Pty) Ltd is a full service demolition enterprise that has been involved in demolition and explosive engineering since 1990. We provide a turnkey demolition service that includes:
* Demolition of heavy industrial plant and mining plants and Infrastructure
* Turnkey projects for purchase, demolition and rehabilitation of Industrial and mining plants
* Fast track strip-outs and demolition under 24/7 shutdown conditions
* Large commercial demolition
* Controlled explosive demolition and implosion of large structures Under demanding conditions
* Dismantling of equipment and structures
* Asbestos removal and disposal
* Decontamination of radiological hazards
* Site reclamation and environmental rehabilitation
* Salvage and recycling of redundant equipment and scrap metals
Safety
Jet Demolition has a 5 star NOSA rating with NOSCAR status and are the only demolition company in South Africa with a NOSA grading and the International Health & Safety Management Systems Specification (OHSAS 18001).
Explosive and Mechanical
Blasting of spill way crest at Midmar Dam, demolition of bridge decks and piers using mechanical and explosive methods, before completed raising of Midmar Dam
Capping of redundant mine shafts
1. Blasting Head Gear
2. Casting shaft cap
3. Completed shaft area
Implosions
Implosion of the fire damaged Munitoria building in Pretoria City Centre using 300kg of explosives. Removal of 44 000 tonnes of rubble was done in 25 days. The entire job was completed in 3 months.
Equipment Salvage
Industrial equipment salvaged from demolition projects.
Dismantling
Heavy industrial dismantling using a 440 tonne crane at an operating explosives plant.
International Projects
Explosive demolition of the 44m high Taman Sari Water Tower damaged in the December 2004 tsunami in Banda Aceh, Indonesia, for the United Nations Development Programme (UNDP)
Demolition and Rehabilitation
Before
After
Turnkey projects on gold plants up to 28Ha in extent: Plant purchase, decontamination, equipment salvage, gold recovery, demolition of steel and concrete structures and rehabilitation.
Mechanical Demolition
Controlled Crushing ("nibbling") of Concrete Structure using Specialised Demolition Equipment
Decontamination
Decontamination of radioactive equipment recovered from uranium, acid and gold plants.
Long Drop
Explosive demolition of an unusually long steel structure (7 storeys high x 250m long).
CERTIFICATE
OHSAS 18001
SAFETY ACHIEVEMENT
AWARDED TO
JET DEMOLITION
THE NOSA FIVE STAR SYSTEM
1332 Club House Street
Maraisburg
1709
P O Box 35866
Northcliff 2115
South Africa
Tel: 011 495 3800 Fax: 011 495 3838
E-mail: demolish@mweb.co.za
Website: http:/www.jetdemolition.co.za


After Dropping 2 Furnaces
Turnkey Demolition Solutions for Furnaces, Structures & Complete Plants
Rapid Furnace Demolition during 24/7 Shutdowns on All Types of Furnaces
Technically Advanced Mechanical & Explosive Methods
Structural Steel/Pipe Works Strip-outs & Demolition
Concrete Civils & Refractory Demolition
Solid Metal Lancing and/or Blasting & Removal
Hazardous Materials Removal & Disposal
JET DEMOLITION (PTY) LTD
WORK SAFETY OHSAS 18001
NOSA
NOSCAR
Tel: 011 495 3800 Fax: 011 495 3838 email: demolish@mweb.co.za Web: www.jetdemolition.co.za

APPENDIX D.3

COST ESTIMATE FOR RAISING WITH SIDE CHANNEL SPILLWAY

RAISING OF TZANEEN DAM WITH SIDE CHANNEL SPILLWAY

FSL = 726.9 masl
NOC = 733.0 masl

No	DESCRIPTION	UNIT	RATE Apr 09 Rand	QUANTITY	AMOUNT Rand
1	**Excavation**				
	(a) Bulk				
	(i) all materials	m^3	35	160 000	5 600 000
	(ii) extra over for rock	m^3	60	100 000	6 000 000
	(b) Preparation of solum				
	(i) all materials	m^2	16	12 900	206 400
	(II) extra over for rock	m^2	16	10 600	169 600
2	**Concrete Works**				
	(a) Formwork				
	(i) gang formed	m^2	220	12 600	2 772 000
	(ii) intricate	m^2	305	1200	366 000
	(b) Concrete				
	(i) mass	m^3	530	6 300	3 339 000
	(ii) structural	m^3	840	6 450	5 418 000
	(c) Reinforcing	t	9 600	581	5 572 800
	(d) Spillway bridge	Sum			1 500 000
3	**Miscellaneous** (% of 1-2)	%	25	30 943 800	7 735 950
	SUB TOTAL A				38 679 750
4	**Preliminary & General** (% of sub-total A)	%	40	38 679 750	15 471 900
	SUB TOTAL B				54 151 650
5	**Contingencies** (% of sub total B)	%	15	54 151 650	8 122 748
	SUB TOTAL C				62 274 398
6	**Planning design & supervision** (% of sub total C)	%	15	62 274 398	9 341 160
	TOTAL COST (excl. VAT)				**R 71 615 557**

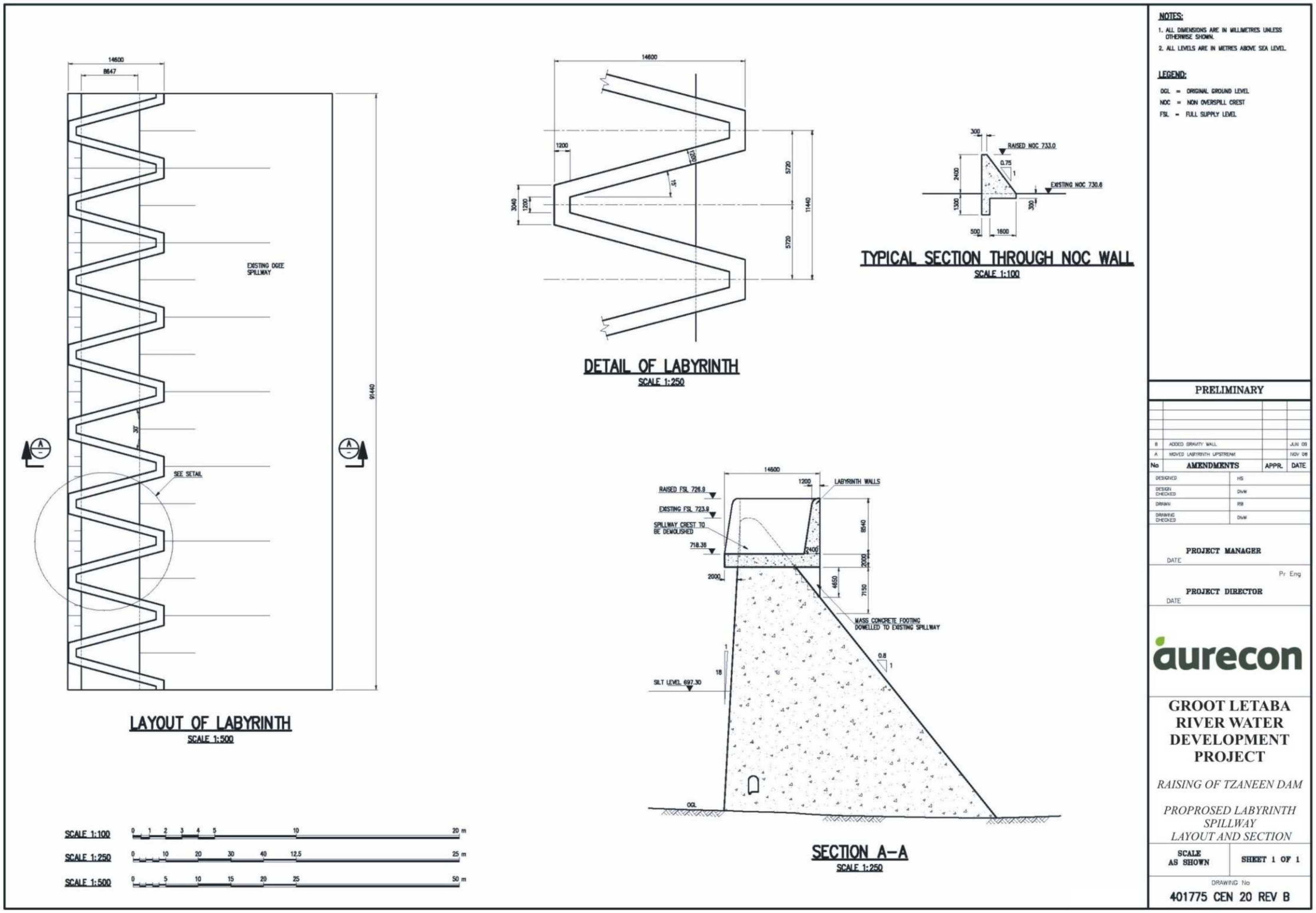
Raising of Tzaneen Dam PDR - Costing - Apr 09

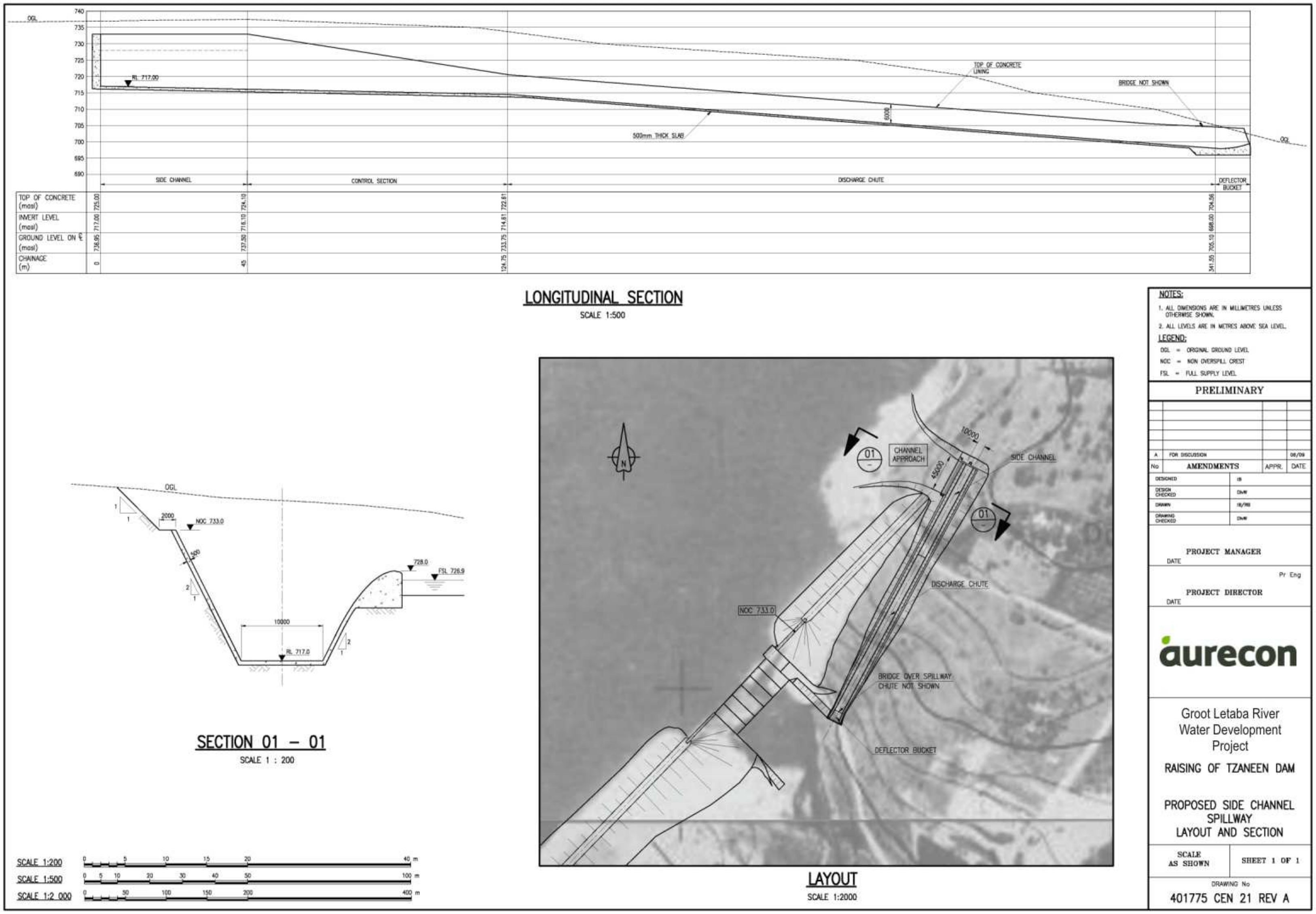
APPENDIX E

DRAWINGS

Drawing No 401775 CEN 20 Rev B :
Raising of Tzaneen Dam : Proposed Labyrinth Spillway Layout and Section

Drawing No 401775 CEN 21 Rev A :
Raising of Tzaneen Dam : Proposed Channel Spillway Layout and Section


NOTES:
1. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE SHOWN.
2. ALL LEVELS ARE IN METRES ABOVE SEA LEVEL.
LEGEND:
OGL = ORIGINAL GROUND LEVEL
NOC = NON OVERSPILL CREST
FSL = FULL SUPPLY LEVEL
DETAIL OF LABYRINTH
SCALE 1:250
TYPICAL SECTION THROUGH NOC WALL
SCALE 1:100
RAISED NOC 733.0
EXISTING NOC 730.6
EXISTING OGEE SPILLWAY
SEE DETAIL
LAYOUT OF LABYRINTH
SCALE 1:500
RAISED FSL 726.9
EXISTING FSL 723.9
SPILLWAY CREST TO BE DEMOLISHED
LABYRINTH WALLS
MASS CONCRETE FOOTING DOWELLED TO EXISTING SPILLWAY
SILT LEVEL 697.30
OGL
SECTION A–A
SCALE 1:250
SCALE 1:100
SCALE 1:250
SCALE 1:500
PRELIMINARY
B ADDED GRAVITY WALL JUN 09
A MOVED LABYRINTH UPSTREAM NOV 08
No AMENDMENTS APPR. DATE
DESIGNED HS
DESIGN CHECKED DvW
DRAWN RB
DRAWING CHECKED DvW
PROJECT MANAGER
DATE
Pr Eng
PROJECT DIRECTOR
DATE
aurecon
GROOT LETABA RIVER WATER DEVELOPMENT PROJECT
RAISING OF TZANEEN DAM
PROPROSED LABYRINTH SPILLWAY LAYOUT AND SECTION
SCALE AS SHOWN
SHEET 1 OF 1
DRAWING No
401775 CEN 20 REV B


LONGITUDINAL SECTION
SCALE 1:500
OGL
RL 717.00
TOP OF CONCRETE LINING
BRIDGE NOT SHOWN
500mm THICK SLAB
SIDE CHANNEL
CONTROL SECTION
DISCHARGE CHUTE
DEFLECTOR BUCKET
TOP OF CONCRETE (masl)
INVERT LEVEL (masl)
GROUND LEVEL ON ℄ (masl)
CHAINAGE (m)
SECTION 01 – 01
SCALE 1 : 200
NOC 733.0
FSL 726.9
728.0
10000
RL 717.0
2000
LAYOUT
SCALE 1:2000
CHANNEL APPROACH
SIDE CHANNEL
DISCHARGE CHUTE
BRIDGE OVER SPILLWAY CHUTE NOT SHOWN
DEFLECTOR BUCKET
SCALE 1:200
SCALE 1:500
SCALE 1:2 000
NOTES:
1. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE SHOWN.
2. ALL LEVELS ARE IN METRES ABOVE SEA LEVEL.
LEGEND:
OGL = ORIGINAL GROUND LEVEL
NOC = NON OVERSPILL CREST
FSL = FULL SUPPLY LEVEL
PRELIMINARY
A FOR DISCUSSION
No AMENDMENTS APPR. DATE
DESIGNED
DESIGN CHECKED
DRAWN
DRAWING CHECKED
PROJECT MANAGER
DATE
Pr Eng
PROJECT DIRECTOR
DATE
aurecon
Groot Letaba River Water Development Project
RAISING OF TZANEEN DAM
PROPOSED SIDE CHANNEL SPILLWAY LAYOUT AND SECTION
SCALE AS SHOWN
SHEET 1 OF 1
DRAWING No
401775 CEN 21 REV A

APPENDIX F

COMMENTS ON REPORT

F.1 Comments by DWA Directorate : Civil Engineering

F.2 Comments by Knight Piesold (Pty) Ltd

F.3 Aurecon's Response to Comments

APPENDIX F.1

COMMENTS BY DWA DIRECTORATE : CIVIL ENGINEERING

THE GROOT LETABA RIVER WATER RESOURCE DEVELOPMENT PROJECT

(GLeWaP)

COMMENT BY THE DIRECTORATE : CIVIL ENGINEERING ON THE :

Draft

“ Raising of Tzaneen Dam “

And

“ Nwamitwa Dam “

June 2009

PREPARED BY:

Ninham Shand
PO Box 1347
CAPE TOWN
8000

In association with others

8 March 2010

1

➢ **1) INTRODUCTION**

Reference to a request by the D : OA for comment on the draft set of documents forwarded to the D : CE in 2009, requesting comment on the suite of documents reporting on the Bridging Studies that were undertaken to re-assess recommendations in the previously done Feasibility Studies. These documents included the Bulk Water Distribution Infrastructure, which Sub-Directorates Dam Design and Open Conveyance Systems cannot comment on.

Comment was also previously given in a formal letter as well as during a meeting at BKS offices in Pretoria.

The following documents are commented on in this commentary document:

a) Nwamitwa Dam : Preliminary Design Report
b) Nwamitwa Dam : Preliminary Design Report : APPENDICES
c) Nwamitwa Dam : Preliminary Design Report : APPENDIX H
d) Raising of Tzaneen Dam : Draft Report

Comments will be given by **referring to the applicable paragraph or drawing number** in a chronological order and *quoting where necessary in italics*:

➢ **A) : *GENERAL COMMENT***

1) Some documents use the terminology " Groot Letaba River Development Project", whereas others talk about " Groot Letaba Water Development Project".

If the Olifants River Water Resources Development Project (ORWRDP) which was also initiated by the Directorate : Options Analysis (D:OA) can be used as an example, an abbreviation of : GLRWRDP, would have prevented this apparent confusion.

2) The majority of comment has reference to technical details, which may not necessarily change the recommended options, but will definitely need to be considered during the next detail design phase of the project, if it is to go ahead.

3) Executive summary to be completed for Nwamitwa Dam.

4) DWA : Civil Engineering commented previously on earlier editions of the reports (2008/08/14).

➢ **B) : *NWAMITWA DAM : PRELIMINARY DESIGN REPORT***

- ***Par 1.1 : Last paragraph on P.1:*** Reference not given.
- ***Par 1.1 : Last paragraph on P.2:*** Reference not given.
- ***Par 2.1 : Table 2.1:*** Add the maximum spillway height.
- ***Par 3.1 : Spillway Floods:*** An external review and/or independent report will be required of the Flood Magnitudes for Design Purposes before the detail design stage could commence. Refer Letter dated 6 February 2008 as well as 14 August 2008.
- ***Par 4.3.4 : Rotary Core Drilling*** : Drawing reference not given. Could not be found.
- ***Par 4.3.6 : Seismic Hazard Assessment*** :The assessment referred to in Appendix B could not be found.
- **Par 5.4 : Fine aggregate (sand) :** *"If required, additional sources for finer aggregate do occur." :* To be rephrased.
- ***Par 5.5*** **: Available Volumes of Material :** If the project is to go ahead, investigations to prove enough impervious material should be done sooner rather than later.
- ***Par 7.3.4 : Stilling Basin*** :A spillway apron length of 16 m is indicated, although the drawings in the "*APPENDICES*" show a length of about 35 m. This is contradictory. In addition : Modern RCC dams with stepped spillways which have been extensively tested with hydraulic models, are being built with apron lengths of less than 10 m.

3

- ***Par 9.1.2 : Discussions with affected parties*** : Surname missing.
- ***Par 11.5 Estimated Project Costs:*** Reference to cost estimates to be corrected.
- ***Par 12 Construction Programme*** : To be completed.
- ***Conclusions/Recommendations:*** Paragraph on Conclusions / Recommendations to be added.

➢ **C) *: NWAMITWA DAM – APPENDICES***

- ***Appendix C.2 : Straight Ogee Spillway :*** No gallery shown. Contradicting with other cross-sections shown.
- ***Appendix C.2 : Straight Ogee Spillway :*** Refer comment above with regard to spillway apron length.
- ***Appendix C.2 : Straight Ogee Spillway :*** Refer comment above with regard to spillway u/s slope.
- ***Appendix C.5 Nwamitwa Dam Freeboard:*** Check/recalculate wind speed ratio's:

➢ **D) *NWAMITWA DAM : APPENDIX H***

- ***Drg No : 401775 CEN 210:*** Contours are missing.
- ***Drg No : 401775 CEN 212:*** The d/s slope of 1:2 is too steep if a crushed gravel is used on the d/s slope. Kerbs to be used on both sides of the crest as well as a bituminous surface seal.
- ***Drg No : 401775 CEN 216:*** The reason for use of a sloped u/s slope for the spillway is unclear. The ogee cap is quite narrow when RCC is intended to be used for the construction of the spillway.

4

The length of apron of 16 m is quite uncommon taking into account the lengths of aprons used recently for RCC stepped spillways. Recent model studies show that the RCC steps dissipates energy very efficient, which in turn results in the use of a short apron/stilling basin.

It is noted that a hydraulic model study will have to be constructed to verify the configuration, but a unit discharge of approximately 36 $m^3/s/m$ (6 800 m^3/s divided by 190 m) can easy be accommodated on stepped spillways with a relatively short apron. The introduction of aeration (e.g. Robert's Splitters) could also be considered.

- Layout drawing required showing the dam reservoir and the realigned roads as well as bridge positions.

- ***Drg No : 401775 CEN 271:*** A deck level of 486,37 masl is too low, and need to be reviewed during the detail design stage (refer to "Raising of Tzaneen Dam", Paragraph 6.2, "The integrity of the two bridges could therefore be at risk during the SEF.")

➢ **E) *RAISING OF TZANEEN DAM : DRAFT REPORT***

- ***Paragraph 4.3 :*** Suggested that the labyrinth design sheet is moved to Praragraph 6, and that an abbreviated write-up of the labyrinth spillway hydraulic design and capacity is given in this section.

- ***Paragraph 6.1 :*** To remove the top 7,5 m of the existing ogee spillway in order to raise the dam by 3 m is quite excessive. This could severely affect the short term yield of the dam during construction and will place stress on the construction programme. This should be re-looked at during the detail design stage. A hydraulic model study will be required during the detail design stage.

- The use of MSE (Mechanically Stabilized Earthfill) would also need to be investigated at detail design stage in lieu of a concrete wall to raise the NOC of the embankment.

- The eight 11m wide labyrinth cycles do not coincide with the 50-feet joint spacing.

APPENDIX F.2

COMMENTS BY KNIGHT PIĒSOLD (PTY) LTD

Knight Piésold
CONSULTING

Knight Piésold (Pty) Ltd
Reg. No. 1995/012742/07
Established 1921

Head Office: Knight Pièsold House
4, De la Rey Road
Rivonia 2128

P O Box 221, Rivonia 2128, South Africa

Telephone: +27 (11) 806-7111
Facsimile: +27 (11) 806-7100
email:
cabrahamson@knightpiesold.com

Offices in Durban, Gaborone, Mbabane, Nelspruit, Phalaborwa, Polokwane, Pretoria and Windhoek

Your Ref:

Our Ref: *3030018301 CJA Rvw 01-1*

Contact: *CJ Abrahamson*

4th January 2010

BKS Consulting Engineers
Block D, Hatfield Gardens
333 Grosvenor St.
Hatfield
Pretoria
0083

Attention: Hermien S. Pieterse

Dear Hermein

I take this opportunity to extend our best wishes for a good 2010 all of you at BKS.

The following pages contain my review of the preliminary design reports of the Groot Letaba River Development Project Bridging studies for the proposed Nwamitwa Dam and the raising of Tzaneen Dam.

We trust that this will be useful in finalising these reports.

I will be available to discuss the reports with the design team, but wish to advise that I will be away from 8th February to 18th March this year.

Yours faithfully

CJ Abrahamson
For Knight Piésold Consulting (Pty) Ltd

Knight Piésold
CONSULTING

GROOT LETABA RIVER DEVELOPMENT PROJECT

REVIEW OF

PRELIMNIMINARY DESIGN REPORTS

FOR

NWAMITWA DAM AND RAISING OF TZANEEN DAM

1. INTRODUCTION

On 4th September 2009, confirmation of a verbal request for Mr CJ Abrahamson to review the preliminary design reports for the proposed Nwamitwa dam and the Raising of Tzaneen dam. The following files were received by email on 5th September 2009:

- **Nwamitwa Dam**

1. Nwamitwa Dam Preliminary Design Report Ver 0 7 LW MK small (this email)
2. Nwamitwa Dam Preliminary Design – Appendices MK V1 small
3. Appendix F.4 – Map 1
4. Appendix F.4 – Nwamitwa Dam Valuation Report
5. Appendix F.4 – Map 2
6. Appendix D.1 – Spillway Type Selection report
7. Appendix D.5 – River Diversion Water Profile calculations

8. Appendix H – Drawings – not received - therefore no comments.

- **Tzaneen Dam**

1. Raising of Tzaneen Dam Ver 0.1 small
2. Appendix B – Hydroplus Proposal
3. Appendix C – Impact of Fusegate Rotation
4. Appendix D.1 – Hydroplus Cost Estimate
5. Appendix D.3 – Side Channel Spillway Cost Estimate
6. Appendix D.2 – Labyrinth Cost Estimate
7. Drg 401775 CEN20B – Labyrinth Spillway
8. Drg 401775 CEN21A – Side Channel Spillway

2. GENERAL OVERVIEW

Both dam options were well researched with the conclusions being reasonably presented. However, recommendations into the way forward towards detailed design are missing and should be presented.

Only the Nwamitwa dam report gives a background leading up to the preliminary design reports. No background on the sizing of the dams is given. The reports are not clear whether a decision must be made to go ahead with the one project or the other or both.

It is noted that the Nwamitwa dam will yield considerably more water than the raising of Tzaneen dam but at about 9 times the unit cost. There should be some reason given for choosing to not to raise the

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Tzaneen dam by more than 3m – possibly unacceptable impacts on existing properties/infrastructure or a small increase in firm yield.

It is suggested that the following wording taken from the DWAF Groot Letaba website be inserted as follows:

The main component of the proposed project comprises a new major storage dam at a site in the Groot Letaba River referred to as the Nwamitwa site, downstream of the confluence of the Nwanedzi River. The proposed dam wall could be 36m high and comprise a concrete structure in the river section accommodating a spillway and outlet works, with earth embankments on both flanks. With a storage capacity of 144 million m^3 it would increase the system yield by about 47 million m^3 per year. (By comparison, the capacity of Tzaneen Dam is 157,5 million m^3).

It was also proposed to increase the capacity of Tzaneen Dam to approximately 203 million m^3 by raising the dam wall. This could increase the firm yield of the dam by about 6% from 60 million m^3/a to 64 million m^3/a, but more importantly, the dam could then be operated so as to minimize the frequency and intensity of restrictions on water allocations for the irrigation of permanent fruit orchards.

Some of the figures in the above may need to be corrected in line with these two reports. Additional notes relating to the construction time and other infrastructural requirements can be added.

Specific matters relating to each of the two reports follow:

3. NWAMITWA DAM

3.1 Items not Available for Review

The following items referred to in the report were not available for review although much of the information was contained in the report itself:

Appendix B: Geotechnical Investigations

Appendix C: Embankment - containing:

- C1 Stage Capacity Curve
- C2 Optimisation of Dam Size
- C3 Grading Envelopes
- C4 Slope Stability Analysis
- C5 Freeboard Calculations

Appendix G: Construction Programme

Appendix H: Drawings – containing:

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

3.2 Specific Comments

3.2.1 Executive Summary

It is noted that the executive summary is still to be completed.

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3.2.2 Section 1 – Introduction

1.1 - Background to Project

- 4th paragraph - "This bridging study...." should read "A bridging study....."
- There are a number of places where auto cross referencing has printed as "**Error! Reference source not found**" – eg 5th, 6th & 7th paragraphs as well as Section 11.5.

1.3 – Scope of this Report

Although the scope is well described, the report should end with conclusions and recommendations.

3.2.3 Section 2 - Principal Details of Proposed Nwamitwa Dam

Table 2.1 - Principal Details of Proposed Nwamitwa Dam

The table is a clear representation of the dam showing the main aspects at a glance. However, it should be stated that the table provides the principal details of **the recommended option** to be carried through into the final design.

The following comments should be addressed:

- Firm Yield .... unit - m^3/a, (not Mm^3/a).
- "Recommended Design Flood (RDF) = 1:200 year RI routed flood peak" should be "Recommended Design Discharge (RDD) = 200-year RI routed flood peak. (RDF refers to the whole hydrograph whereas RDD refers to the designed spillway discharge which, in this case, is the peak discharge of the routed RDF over the spillway).
- Likewise, "Safety Evaluation Flood (SEF) = Unrouted $RMF_{+\Delta}$" should read "Safety Evaluation Discharge (SED) = Unrouted $RMF_{+\Delta}$"
- The embankment crest length at 3.5km appears to be very long. Is this the best site from a topographical view point?
- Base width of embankment at maximum cross section – 126m. The stated u/s and d/s slopes (1V : 3H and 1V : 2H respectively) indicate that this should be at least 180m for a 34m high embankment with a 10m wide crest.
- Non Overspill Crest elevation should be 486 masl (not 986).
- Spillway "Design Discharge" should read "Maximum discharge capacity (zero freeboard)"
- "Elevation at design discharge" should read "Reservoir elevation at maximum discharge".

3.2.4 Section 3 Hydrology

Designation of the return period flood

The report uses various terms for the return period floods such as 1:100 RI flood, 1 in 100 year flood, 100 year RI flood, 1:10 yr etc. It would be more consistent to simply use one term such as 10-year flood, 100-year flood, etc which could be explained in a list of acronyms or definitions.

3.1 - Spillway Floods

The study on the flood hydrology is well researched with good logic applied in downsizing the SEF from the PMF. Setting the SED as the unrouted $RMF_{+\Delta}$ of 6 800 m^3/s implying an SEF peak of 8 900 m^3/s (equivalent Francou Rodier K = 5.6 or $RMF_{+2\Delta}$) may be conservative but fine for the purpose of the preliminary design report.

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3.2 and 3.5 – Diversion Floods and Diversion Strategy

Diversion Strategy and **Diversion Floods** both relate to river diversion during construction. Therefore it is suggested that the two sections should follow immediately after one another.

3.5 – Diversion Strategy

The diversion strategy must depend largely on the construction programming in relation to risks of flooding at any particular time. There should be a paragraph describing the diversion arrangements and how they fit into the construction programming of the various portions of work.

3.5.1 - Selection of river diversion floods

The second paragraph gives the ratio of the incremental catchment to total catchment as 1 739/2 917 km^2. In Section 3.3 the intervening effective catchment is quoted at 1 352 km^2 and Table 2.1 gives the total catchment as 1 944 km^2. Thus the incremental catchment is about 70% of the total rather than 60%. Therefore the scaled down river diversion floods would be adjusted to 1 000, 1 500 and 1 900 m^3/s for the 10-year, 20- year and 50-year floods respectively.

3.5.2 – First Stage

Drawing No's 401775 CEN 213 and 214 are not yet available to the reviewer. Drawings are needed to fully understand the diversion planning described here. It would seem that the embankment foundations and walls along the abutments could be constructed concurrently with the river bed excavations to reduce the amount of earthworks required later, thereby reducing the risk of delays and overtopping during construction.

3.5.3 - Second stage

It is noted that the diversion culvert will be located at 454 masl which calculates from the data given in Table 2.1 to be about 2m above river bed. As such, the statement in the last sentence of the 1st paragraph regarding the water level upstream of the works appears to be incorrect.

3.2.5 Section 4 – Geology and Geotechnics

4.3.4 - Rotary Core Drilling

The drawing number given in the last sentence of the 1st paragraph needs to be finalised.

3.2.6 Section 5 – Materials

5.3 - Semi-pervious Material

The coefficient of permeability for this material is assigned the same value as the impervious material. This should be checked, and if so, there could a case for combining the impervious and semi-pervious zones into one.

5.5 - Available Volumes of Material

If the two impervious and semi-pervious zones were combined as suggested above, there would be a smaller imbalance between available volume and volume to be proven. (See also Section 6.4.2 – Core Zone).

3.2.7 Section 6 – Embankment

6.4.3 - Cut-off Trench

If rock levels are deep, the RMR criterion would seem to be too stringent – especially in the upper parts of the embankments. Seepage path length should be considered as well.

Additional comment should be made on treatment of cut-off trench surfaces eg reverse slopes in excavated rock, slush grouting or shotcreting. Allowances for these should be made in cost estimates.

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3.2 and 3.5 – Diversion Floods and Diversion Strategy

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Additional comment should be made on treatment of cut-off trench surfaces eg reverse slopes in excavated rock, slush grouting or shotcreting. Allowances for these should be made in cost estimates.

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6.4.7 - Downstream slope protection

Grass should be considered as a more economical alternative to crushed stone – normal rainfall at this site should be sufficient to ensure good grass cover. However, maintenance will be a requirement but the cost thereof can be offset by the capital saving. The maintenance of the grassed surface will involve use of manual labour which should be encouraged.

6.5 – Filter Criteria

The filter design given in the Preliminary Design report should be considered as provisional. Not having a copy of the geotechnical report, it was not possible to check whether dispersivity tests had been conducted and whether the design took this into account. Dispersivity should be assessed using all laboratory methods – not just one.

It is suggested that the US Army Corps of Engineers publication ref EM 1110-2-1913 – Appendix D – July 2004 should be used in the final design. The method embraces Sherard & Dunnigan criteria and covers dispersive materials. Other recommendations by AL Melvill are also worth considering. Dispersivity should be checked by all laboratory methods.

6.6.1 – Stability Analysis - Shear strength parameters

It is noted that a cohesion value of 5 kPa has been assumed for the core and 3 kPa for general fill / foundation below the wall. The Dam Safety Office has previously commented on similar designs that it is now a well established fact that apparent cohesion for fine grained soil materials under saturated conditions approaches zero over the long term. The assumed shear strength values should be verified by further investigations and careful laboratory testing if necessary. Slow consolidated drained triaxial tests with high back pressure using de-aired water to ensure 100% saturation and with pore pressure measurement, are considered appropriate to obtain the true effective shear strength parameters of fine-grained soils. Shear strength results should also be plotted against axial deformation (up to 15% deformation) to determine strain softening characteristics. Apparent cohesion of sandy materials is lost after 0,5% to 3% of axial deformation and that of clayey materials after 3% to 15% axial deformation. The relevant shear deformations will probably be much smaller. Deformation beyond the threshold value can occur in dam walls and foundations due to the progressive failure mechanism as has been demonstrated by many case studies. These studies should be considered in the final design.

It is also noted in the paragraph below Table 6.4 that significant build-up of pore water pressures is not expected in these relatively sandy materials. Considering that the quoted permeability of 3×10^{-8} cm/s and the material compacted at OMC (up to 23%), construction pore water pressures could, indeed become significant and should be considered in the design.

6.6.2 – Stability Analysis – Results

Although the phreatic surface will not change significantly in the short duration of a flood, pore pressures below the phreatic surface do increase with increased reservoir levels and should be considered in these events.

6.7 – Grouting

The report makes no mention of the type of grouting envisaged – GIN grouting or conventional, upstage or downstage. These details will need to be determined in the final design for the preparation of the Specifications. The nature of the foundation geology should make it possible to determine the best grouting method to be adopted.

3.2.8 Section 7 – Spillway

7.4.2 – Structural Design – Loadings

5th bullet - Presumably the silt in the reservoir is assumed to build up to the design level over 100 years – not at the 100-year RI level.

Table 7.2 - Stability Results for Ogee Spillway

Without the sectional geometry of the ogee section (drawings not available to reviewer), the results couldn't be verified, but appear realistic.

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3.2.9 Section 8 – Outlet Works

8.3 - Description of Outlet Works

What is the invert level of the outlet sleeve valves? The discharge capacity (21 m^3/s) suggests that it is about 2m above river bed level. Is that sufficiently above normal flood levels?

The design of the pipework in an integral outlet block is noted. It would be better to contain it in an intake structure upstream of the gravity dam so that it will not interfere with RCC placement. It can be built independently of the RCC.

3.2.10 Section 9 - Relocation of Roads

9.1.2 - Discussions with affected parties

4th sentence – fill in missing name. If unknown, rephrase.

3.2.11 Section 11 – Cost Estimates

11.1.1 – Introduction

3rd paragraph - The meaning of LHWC should be added to the list of abbreviations.

11.1.2 – Descriptions of Payment Items

- **Clearing** - It is not clear whether the term "dam footprint" includes the reservoir basin.
- **Drilling and Grouting** – The number of secondary holes is more likely to be equal to the number of primary holes, because they are needed to verify the effectiveness of the primary holes.

3.2.12 Section 13 – References

These references should be numbered and referred to in the report text where they are mentioned.

4. RAISING OF TZANEEN DAM

3.3 Items not Available for Review

All items listed in the report contents were available for review.

3.4 Specific Comments

3.4.1 General

It is noted that the extent of work in the Raising of Tzaneen dam is limited to simply raising the spillway with the addition of a parapet wall on the embankment. Hence the report does not consider aspects of the dam such as those investigated in the preliminary design of the Nwamitwa dam. However, there should be some consideration given to the impact of increased water levels on the embankment stability.

It is also noted that the report does not investigate various raising heights. Presumably this had been covered in previous studies, in which case the details should be given in a summary.

3.4.2 Executive Summary

- The previous study mentioned in the executive summary should be referenced.
- Even though the use of automatic steel gates will need regular maintenance and inspections by skilled personnel who may not be available, this should not be a reason to completely reject them. Hydroplus gates also need maintenance and inspections. There are concerted efforts to develop and keep such skills in South Africa that should be encouraged. Pro's and cons plus

costs of these systems should be investigated. Reliability and safety of such gated systems under all conditions of operation should be the major consideration.

- The 1st sentence of the 3rd paragraph should be corrected to read "For the present study the options as listed below have **been** considered."

3.4.3 Section 2 - Principal Details of Tzaneen Dam

The principal details given do not fully describe the existing dam and proposed alterations. It is suggested that all details of the existing dam be provided in table form (as with Nwamitwa dam). For each option, there should be additional columns for those parts that are to be altered.

3.4.4 Section 3 – Flood Hydrology

Designation of the return period flood

As mentioned in 3.2.4 (a), it is suggested that only one term for the return period floods should be used, such as 10-year flood, 100-year flood, etc which could be explained in a list of acronyms or definitions.

Definition of RDF, SEF, RDD and SED

As mentioned in 3.2.3 above, there needs to be clear distinction between RDF / SEF and RDD / SED. RDF and SEF refer to the whole inflow hydrographs. RDD and SED refer to flood peak discharges from the dam. In this case, the RDD is the routed RDF peak (= 200-year routed flood peak) and the SED is equivalent to the unrouted $RDF_{+\Delta}$ taken as the discharge over the spillway. The statements in the last three paragraphs of the section need to be amended in this regard.

Section 3.2 – Spillway Floods

The outflow flood peaks would probably vary according to the type of spillway chosen. Presumably the figures given are for the recommended labyrinth spillway. If so, this should be stated.

The maximum reservoir level for these floods should be stated.

Appendix A4.2 – Flood Routing

- The 100-year flood should be 1170 m3/s, not 1070 m3/s as given in the bulleted figures after Figure A4.1.
- The inflow and outflow hydrographs for the 100-year and 200-year floods should also show the maximum reservoir level for those floods. These maximum levels should also be given in the report text.

3.4.5 Section 5 – Hydroplus Fusegates

5.1 - Description

3rd paragraph – the number of fusegates tipping in the SEF should be stated.

3.4.6 Section 6 - Labyrinth Spillway

6.2 - Impact of raised NOC

4th paragraph – The RDD maximum reservoir level will be about 730 masl which is 1m below bridge soffit level. Most bridges are only designed for the 50-year flood so a bridge clearing the RDD level shouldn't be a problem in terms of normal road designs. This also applies to the statement in Table 8.1.

6.3.2 – Loadings

5th bullet - Presumably the silt in the reservoir is assumed to build up to the design level over 100 years – not at the 100-year RI level.

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Table 6.3 - Stability Results for Raised Spillway

The maximum stresses and safety factors against sliding appear incorrect – the calculations should be checked and corrected if necessary. The reviewer's rough check for the abnormal case yielded maximum stress at U/S face = -339kPa & -127kPa and sliding SF's = 1.71 & 1.98 respectively for the two conditions in those columns.

3.4.7 Section 8 – Conclusions and Recommendations

This section is not fully conclusive until the options of installing gates on the spillway have been fully investigated. Such systems could well be considerably more economical even using some of the revenue saved for organised monitoring and maintenance.

APPENDIX F.3

AURECON'S RESPONSE TO COMMENTS

F3 COMMENTS RECEIVED

F3.1 INTRODUCTION

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

- DWAF Directorate : Civil Engineering
- BKS (Pty) Ltd
- Knight Piesold (Pty) Ltd

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

- Incorporated in the report as amendments
- Rejected as noted in response
- Listed for action during detailed design in Section 8 of report

The response follows the same numbering system as used in the comments.

F3.2 DWA DIRECTORATE : CIVIL ENGINEERING

E) : <u>RAISING OF TZANEEN DAM : DRAFT REPORT</u>

- Par 4.3 — *Retained text as is*
- Par 6.1 — *Detailed design*

F3.3 BKS (PTY) LTD

The BKS comments were made on the Technical Study Main Report. The response below addresses those comments that coincide with text in this report.

F3.3.1 Background

- Justification of 3 m raising — *Added note wrt expropriation levels*

F3.3.2 Flood Hydrology

- Second paragraph — *Retained text as is*

F3.3.3 Hydroplus Fusegates

- Second paragraph — *Amended text*

F3.3.4 Labyrinth Spillway

• Second paragraph	*Amended text*

F3.3.5 Impact of Raised NOC

• Fourth paragraph	*Amended text*
• Fifth paragraph	*Amended text*

F3.4 KNIGHT PIESOLD (PTY) LTD

F3.4.1 Specific Comments

F3.4.2 General

• Stability of embankment	*Detailed design*
• Raising heights	*Limited by bridges*

F3.4.3 Executive Summary

• Previous study	*Added reference*
• Steel gates	*Disagree – high maintenance and lack of skills*
• Third para	*Corrected text*

F3.4.4 Section 2 – Principal Details of Tzaneen Dam

• Principal details	*Retained text as is*

F3.4.5 Section 3 – Flood Hydrology

• Designation of RI	*Corrected text*
• RDD/RDF terminology	*Disagree – RDD refers to unrouted flood peaks RDF refers to routed flood hydrograph peaks*
• SED/SEF terminology	*Disagree – SED refers to unrouted flood peaks - SEF refers to routed flood hydrograph peaks – defined as compromise between PMF and RMF approaches – see Section 3.2*
• Appendix A3.2 - floods	*Added note re routing for labyrinth option*
• Appendix A4.2	*Corrected 1 in 100 year RI flood peak*
• Appendix A4.2	*Added water levels*

F3.4.6 Section 5 – Hydroplus Fusegates

- 5.1 Description *Added number of fusegates*

F3.4.7 Section 6 – Labyrinth Spillway

- 6.2 Impact of raised NOC *Detailed design*
- 6.3.2 Loadings *Amended text*
- Table 6.3 Stability analysis *To be checked during detailed design*

F3.4.8 Section 8 – Conclusions and Recommendations

- Steel gates *Disagree*