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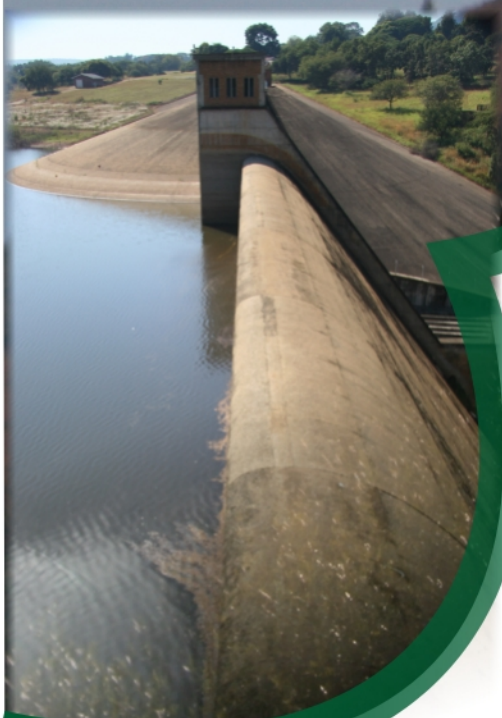
GROOT LETABA RIVER WATER DEVELOPMENT PROJECT (GLeWaP)

TECHNICAL STUDY MODULE:

Preliminary Design of Nwamitwa Dam



VOLUME 6

Annexure 1: Appendices

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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
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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
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APPENDICES

Aurecon Electronic Reference: I:\HYDRO\401775 Groot Letaba\Reports\Vol 6 Annex 1 Apps excl B and H\Vol 6 Annexure 1 Nwamitwa Dam Preliminary Design -Appendices.doc

APPENDIX A : Hydrology

APPENDIX A.1 : Flood Hydrology

(The Flood Hydrology Report refers to initial dam sizes of 0.5, 1.0 and 1.5 MAR respectively. These sizes were selected based on the estimated MAR at the start of this study. During the course of the study, the MAR value at the dam site was adjusted to 160.9 million m^3. This resulted in the above values being adjusted to 0.41, 0.85 and 1.16 MAR respectively).

Nwamitwa Dam

Design Flood Analysis

Prepared by

Ninham Shand (Pty) Ltd

March 2008

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1. INTRODUCTION

The proposed Nwamitwa Dam in the Limpopo Province will be situated on the Groot Letaba River, immediately downstream of its confluence with the Nwanedzi River, approximately 40 km east of the town of Tzaneen. The total catchment area of Nwamitwa Dam is 1944 km^2 (see Figure 1).

The Nwamitwa Dam is classified as a Category III dam in terms of the Dam Safety Regulations. Consequently, in accordance with Sub-Clause 3.4.2 of the SANCOLD Guidelines (SANCOLD, 1991), it is "necessary to perform hydrological calculations appropriate to the site" as part of the spillway design flood analysis. As such, the SANCOLD Guidelines recommend that:

- the Recommended Design Flood (RDF) should be the 1 in 200 year recurrence interval (RI) flood (Sub-Clause 5.2.1)
- the Safety Evaluation Flood (SEF) should be the Probable Maximum Flood (PMF) (Sub-Clause 5.2.2)

A further recommendation which supports the use of the PMF as the SEF can 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 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."

This report presents the results of the design flood analysis for Nwamitwa Dam and provides estimates of the 100 year recurrence interval (RI) flood, the 200 year RI flood and the Probable Maximum Flood. The 1 in 100 year RI flood was estimated in order to allow expropriation levels in the dam basin to be determined. For all of these flood events, the critical inflow hydrograph to the dam is provided along with the outflow (routed) hydrograph for both a 200m and 400m spillway length and for a range of dam sizes. In addition to the above design floods, the Regional Maximum Flood (RMF) as well as a series of 1 in 10 year, 1 in 20 year and 1 in 50 year inflow hydrographs were also determined. The 10, 20 and 50 year RI floods are required for the design of diversion works during dam construction.

2. 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 1. 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 2-1.

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 2-1: Catchment Characteristics

Sub-catchment	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[(1)]	33.8
Average channel slope (m/m)	0.0066[(1)]	0.003
Catchment Index	34795	44433
Basin lag (h)	8.2	9.0
Unitgraph peak (m^3/s)	29.1	53.0

(1): refer to **Appendix A** for further notes on Tzaneen Dam catchment characteristics.

3. 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.

Appendix B presents a summary of the design rainfall for the range of RIs and storm durations that were considered.

4. SPILLWAY DESIGN FLOOD ANALYSIS

In order to size the spillway of Nwamitwa Dam, the peak outflows associated with the RDF and the SEF were determined. Essentially, this entailed two key tasks viz.:

- the determination of inflow hydrographs to Nwamitwa Dam for a range of storm durations
- the routing of these hydrographs through Nwamitwa Dam in order to determine the critical storm duration, i.e. the duration which results in the maximum outflow rate

4.1 Design flood estimates

Three methods were used to determine the inflow flood peaks and/or hydrographs to Nwamitwa Dam for the range of RIs that were considered. These include:

- Unit hydrograph techniques using dimensionless regional unit hydrographs (HRU, 1972).

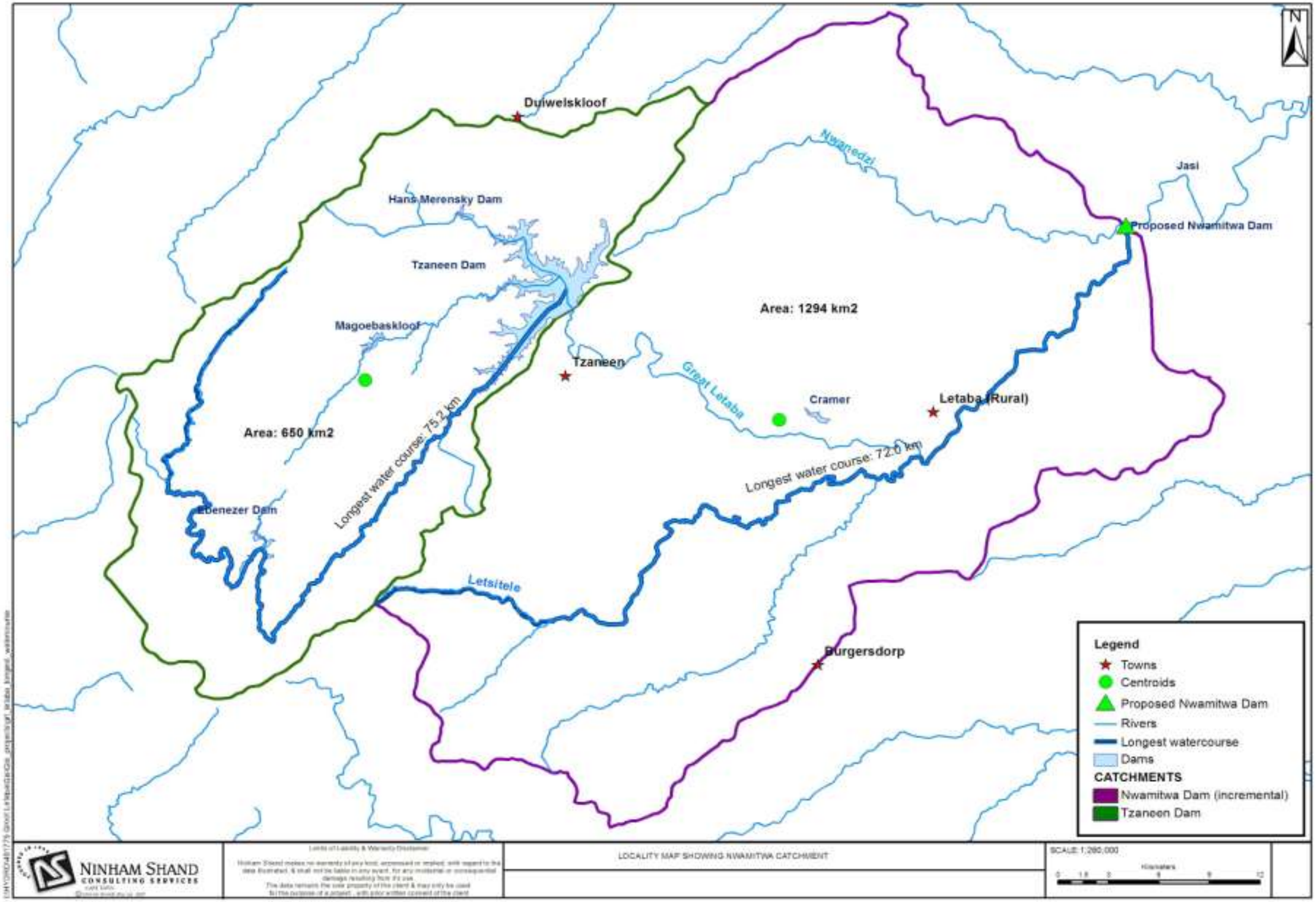
- Probabilistic (flood frequency) techniques using a range of probability distributions. The closest streamflow gauge to the proposed Nwamitwa Dam is Gauge B8H009 on the Groot Letaba River approximately 20 km upstream of the proposed dam site.

- Empirical flood estimation techniques in the form of the Francou-Rodier approach, as used by Kovacs to develop the Regional Maximum Flood (RMF) peak (Kovacs, 1988).

Whereas the unit hydrograph method provided full flood hydrographs, the probabilistic and empirical methods only provided estimates of flood peaks.

4.1.1 Regional Unit Hydrograph Technique

The synthetic regionalised unit hydrographs, which were developed for South Africa by the Hydrological Research Unit (HRU 3/69, 1969), were used to derive 100 year and 200 year RI and PMF flood hydrographs for both the Tzaneen catchment and the Nwamitwa incremental catchment for a range of storm durations. The standard *HDYP01* software was used for this purpose. As *HDYP01* only accommodates a limited number of time intervals for defining the time distribution of storm rainfall, a spreadsheet, which accommodates an hourly distribution of storm rainfall irrespective of the storm duration was used to mimic the *HDYP01* algorithms. This allowed more accurate hydrographs to be simulated in the case of long storm durations.


Duiwelskloof
Hans Merensky Dam
Tzaneen Dam
Magoebaskloof
Area: 650 km2
Longest water course: 75.2 km
Ebenezer Dam
Tzaneen
Nwanedzi
Area: 1294 km2
Great Letaba
Cramer
Letaba (Rural)
Longest water course: 72.0 km
Letsitele
Burgersdorp
Jasi
Proposed Nwamitwa Dam
Legend
Towns
Centroids
Proposed Nwamitwa Dam
Rivers
Longest watercourse
Dams
CATCHMENTS
Nwamitwa Dam (incremental)
Tzaneen Dam
NINHAM SHAND
CONSULTING SERVICES
LOCALITY MAP SHOWING NWAMITWA CATCHMENT
SCALE: 1:280,000

In order to account for translation and attenuation effects as the flood hydrograph progresses down the Groot Letaba River, the Tzaneen subcatchment hydrographs were routed through Tzaneen Dam, after which the outflow hydrograph was routed along the 62 km stretch of the Groot Letaba River down to Nwamitwa Dam. At this point, the routed Tzaneen subcatchment hydrograph was superimposed onto the incremental Nwamitwa subcatchment hydrograph to provide a combined inflow hydrograph to Nwamitwa Dam. This routing sequence, which was simulated in HEC-1, is displayed schematically in Figure 2. (Note that it was assumed that Tzaneen Dam is at FSL at the start of the routing calculation.)

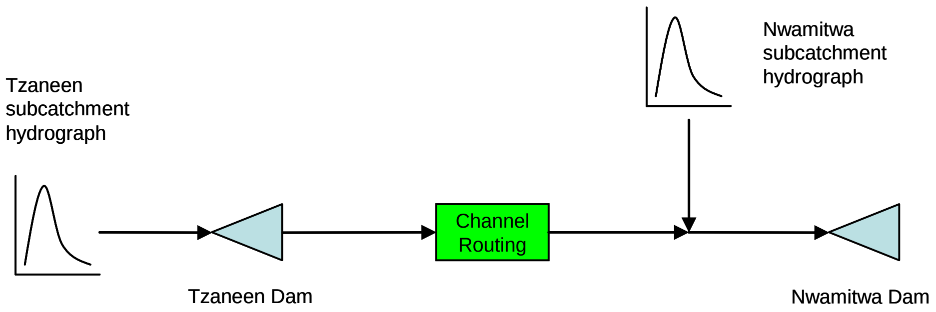


Figure 2: HEC-1 flood routing configuration

The standard level pool routing technique was used for routing calculations through Tzaneen Dam, while the Muskingum equation was used for channel routing. The Muskingum coefficients were calibrated based on observed flood hydrographs at flow gauging stations B8H002 and B8H009, which are both situated on the Groot Letaba River downstream of Tzaneen Dam. Coefficients of 7.65 h (K) and 0.49 (x) were eventually determined for the stretch of river between Tzaneen Dam and Nwamitwa Dam. The results of the routing calculations indicated that, although the river channel between Tzaneen and Nwamitwa dams causes a lag of about 7.5 h, there is negligible attenuation of the flood peak along this stretch of river. Furthermore, whereas the Tzaneen subcatchment hydrograph contributes significantly to the total volume of the inflow hydrograph to Nwamitwa Dam, it has little influence on the peak of this hydrograph. (**Appendix C** provides information on the area-capacity and spillway discharge relationships for Tzaneen Dam as well as on the flood event that was used to calibrate the Muskingum coefficients. A typical example of a routing calculation is also included.)

The range of 100 year RI, 200 year RI and PMF inflow peaks to Nwamitwa Dam, for the different storm durations that were considered, are indicated in Table 4-1. Storm durations of between 4h and 36h were considered.

Table 4-1: Results of Regional Unit Hydrograph Analysis

Design Flood	Nwamitwa Dam Inflow Peak (m^3/s)
100 Year RI	2498 - 3032
200 Year RI	2917 - 3580
PMF	11802 - 16864

4.1.2 Regional Maximum Flood

In addition to the unit hydrograph approach, inflow flood peaks to Nwamitwa Dam were also calculated by means of the empirical, area-based RMF technique outlined in the DWAF Technical Report TR 137 (Kovacs, 1988). The Nwamitwa catchment is situated within K region 5.2.

Table 4-2 presents the results of the RMF calculations and, based on typical ratios of different RI flood peaks to RMF (Kovacs, 1988), also provides estimates of 100 year RI and 200 year RI flood peaks. The calculations were performed for K values of 5.2 and 5.4 respectively. The latter represents the K region which is numerically one step higher than 5.2. (It is important to note that the RMF flood peaks do not take any flood attenuation into account.)

Table 4-2: Results of RMF calculations

K		Flood Peak (m^3/s)
5.2	RMF	5495
	1 in 100 year RI	3143
	1 in 200 year RI	3775
5.4	$RMF_{+\Delta}$	6807
	1 in 100 year RI	3894
	1 in 200 year RI	4676

4.1.3 Flood Frequency Analysis

A third approach towards the estimation of design inflow flood peaks at Nwamitwa Dam involved a probabilistic analysis of observed flood peaks. The closest streamflow gauge to the proposed Nwamitwa Dam is Gauge B8H009, which has a catchment area of 851 km^2 and which is located on the Groot Letaba River immediately upstream of its confluence with the Letsitele River, approximately 20 km upstream of the proposed dam site. Although annual flood peak data at this gauge is available from 1959, the flood frequency analysis only considered the period from 1977 to 2006. Observed flood peaks during this period are affected by the attenuation effect of Tzaneen Dam which is situated about 30 km upstream of the gauge and which was constructed in 1976. The implication of using this data set is that design flood peaks estimated by Flood Frequency Analysis could be expected to be lower than would be the case for an un-dammed river.

Since 1977, the rating limit at Gauge B8H009 has been exceeded twice during high flow events. For one of these events (1995), an estimate of the flood peak was obtained based on the primary data for the event and an extension of the rating curve. As the other event (1999), coincided with missing data, no estimate of the flood peak was possible. To account for the difference in catchment area at Nwamitwa Dam compared to the catchment area at Gauge B8H009, estimates of RI flood peaks at Gauge B8H009 were adjusted by the ratio of the square roots of the respective catchment areas i.e. $\sqrt{1944}/\sqrt{851}$.

Table 4-3 summarises the results of the annual flood frequency analysis. (Note that the Log-Normal distribution produced the best fit of the observed annual flood peaks.) **Appendix D** lists the annual flood peak data at Gauge B8H009.

Table 4-3: Results of Annual Flood Frequency Analysis

Recurrence Interval (years)	10	20	50	100	200	1 000
Flood Peak[(1)] (m^3/s)	606	1080	2090	3279	4900	11125

(1): Represents area-adjusted peak flows at the location of Nwamitwa Dam.

4.1.4 Comparison and Discussion of Flood Peaks

Table 4-4 compares the inflow flood peaks to Nwamitwa Dam as calculated with the various flood estimation techniques.

Table 4-4: Comparison of inflow flood peaks (m^3/s)

Design Flood	Regional Unit Hydrograph Technique(2)	Flood Frequency Analysis	RMF(1) Approach
100 year RI	3032	3279	3143
200 year RI	3580	4900	3775
RMF	N.A.	N.A.	5495
$RMF_{+\Delta}$	N.A.	N.A.	6807
PMF	16864	N.A.	N.A.

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

It follows from Table 4-4 that, although the 100 year RI flood frequency peak is similar to the unit hydrograph and RMF peaks, the 200 year peak is significantly larger. However, it should be borne in mind that the area adjustment factor which was applied to the B8H009 flood peaks might not fully account for the differences in the catchment response times of the various subcatchments that feed into Nwamitwa Dam. The flood frequency estimates are furthermore somewhat problematic because of the problems of the underlying data set. If the RMF-based flood peaks are compared with the 100 year and 200 year RI unit hydrograph peaks, the RMF peaks are slighty higher, which is expected as the nature of the RMF-based approach is such that it leads to estimates that tend to be on the high side.

Also evident from Table 4-4 is that the PMF is significantly higher than the RMF and the $RMF_{+\Delta}$, with the PMF in the order of 3 times as high as the RMF. This relatively high PMF/RMF ratio confirms the results of Görgens *et al.* (2006), who, as part of a recent Water Research Commission (WRC) Study on Extreme Design Floods, investigated PMF/RMF ratios at 109 flow gauging station 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. A map indicating the spatial distribution of PMF/RMF ratios was also developed as part of the WRC Study and is included as Figure 3. The map shows that Nwamitwa Dam is situated within a PMF/RMF ratio region of 2 to 4. It is also interesting to note that the equivalent return period of the PMF peak of 16864 m^3/s equals approximately 6000 years.

As part of the Görgens *et al.* (2006) study, it was found that observed extreme storm rainfall in the northern part of South Africa seem to approach the HRU 1/72 PMP envelope curve, which implies that the HRU 1/72 PMP values that were used in the unit hydrograph analysis for this study are acceptable. For the estimation of the PMF, HRU 1/72 promotes the use of minimum storm losses in order to convert storm rainfall to runoff and provides two approaches towards the estimation of these losses - an envelope of recorded floods and estimates based on maximum runoff efficiency. For the above PMF analysis, in accordance with the HRU 1/72 recommendation, the minimum losses were based on maximum runoff efficiency. As a sensitivity analysis, in light of the high PMF peak, the PMF was recalculated for the scenario where storm losses are based on the envelope of recorded floods. This increased the storm loss from 11 mm to 68 mm, but only resulted in a slightly lower PMF peak of 14627 m^3/s as opposed to the previously calculated PMF peak of 16864 m^3/s.

An additional check on the order of magnitude of the PMF peak entailed the determination of equivalent K-values for the PMF peak. K-values of 6.2 and 6.1 respectively were calculated for the PMF peaks of 16864 m^3/s and 14627 m^3/s (which are less than the K-value of 6.5 for the envelope of world record peaks). Again this confirms the findings of Görgens *et al.* (2006), who showed that, at 10 out of 19 gauging stations and dam sites in the HRU Veld Type Zone Region C, to which the Groot Letaba catchment belongs, the PMF peak represented a K-value of 6.0 or more.

Görgens *et al.* (2006) concluded that, despite a number of conceptual and technical shortcomings of the RMF methodology as per TR 137, the RMF was spatially relatively consistent, in contrast with the HRU 1/72 methodology for PMF determination, which appeared to suffer from notable spatial inconsistencies. A further finding of the WRC Study was that, anomalously, the critical storm durations of HRU 1/72-based PMFs for the non-dam case were generally some 20% to 60% shorter than those of the conventional Recurrence Interval floods for the non-dam case. This is contrary to empirical historical evidence that suggests that large-scale extreme floods in the summer rainfall region are usually the result of longer storm durations, i.e. 24 – 96 hours, with concomitantly larger volumes of runoff. On the other hand, in cases where the critical duration was determined in the presence of attenuation through significant dam storage above FSL, an anomaly of this variety was mostly not an issue. Of equal concern is the implication that critical HRU 1/72-based PMFs in the non-dam case have notably smaller volumes than those determined in the presence of dam attenuation. The WRC Study therefore recommended in-depth research that would modernise the

HRU 1/72 methodology and resolve the above (and other) inconsistencies and anomalies.

The SANCOLD (1991) Guidelines stipulate that, in case of the RMF being used as the Safety Evaluation Discharge, it should be expressed as a level pool (unrouted) discharge, whereas, in the case of the PMF being used as the SEF, the PMF hydrograph has to be routed through the dam. The following section presents the results of the PMF routing calculations and demonstrates that the routed PMF peaks are in fact comparable to the RMF (unrouted) peak.

4.2 Spillway design discharges

The range of 100 year RI, 200 year RI and PMF inflow flood hydrographs to Nwamitwa Dam, as calculated with the regional unit hydrograph approach, were routed through the dam in order to determine design spillway discharges. For each RI, the critical storm duration was determined as that duration for which the peak outflow rate is a maximum. (It is important to note that the storm duration which produces the highest spillway discharge is not necessarily equivalent to the storm duration which results in the highest peak inflow.) Two spillway lengths (200m and 400m respectively) and three dam sizes of 0.5 MAR (FSL 473.5 masl), 1.0 MAR (FSL 477.5 masl) and 1.5 MAR (FSL 479.5 masl) respectively were considered. It was assumed that Nwamitwa Dam is at FSL at the start of the routing calculation. (**Appendix C** provides information on the area-capacity and spillway discharge relationships for Nwamitwa Dam).

The HEC-1 model was initially used for the routing calculations until the critical hydrograph was identified, at which point the routing calculation was repeated for the critical duration by means of a spreadsheet analysis which provided more accurate results. (The spreadsheet used an exponential equation to describe the dam's stage-capacity curve whereas the HEC-1 model makes use of linear interpolation between discreet points on this curve.) The results of the Nwamitwa Dam routing exercise are summarised in Table 4-5 and Table 4-6 for 200m and 400m wide spillways respectively. The table also indicates the percentage attenuation in each case.

The ordinates of the simulated inflow and outflow hydrographs for Nwamitwa Dam, along with the variation in stage level, for the range of RIs, spillway lengths and dam sizes that were considered are presented in **Appendix E**.

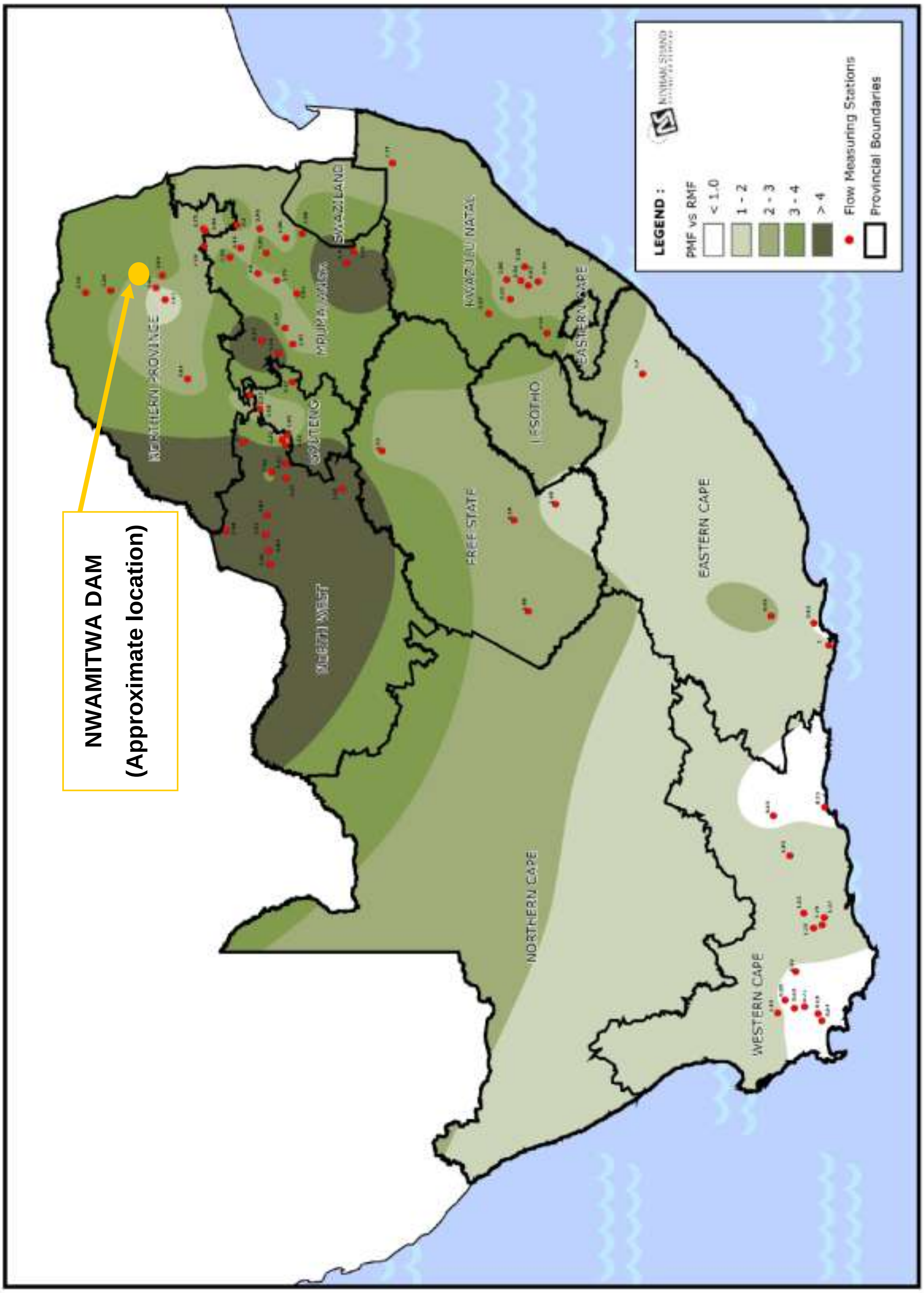


Figure 3: Map indicating the spatial distribution of PMF/RMF ratios for SA (Görgens et al, 2006)

Table 4-5: Results of Nwamitwa Dam Flood routing with 200 m spillway

RI	0.5 MAR Nwamitwa Dam - FSL - 473.5 m - 200m spillway				
	Critical Storm Duration (h)	Q_{in} (m^3/s)	Q_{out} (m^3/s)	Max Stage (m)	Height above FSL (m)
1in100	26	2823	2048 (27%)	476.305	2.805
1in200	26	3312	2447 (26%)	476.658	3.158
PMF	20	13179	10727 (19%)	481.959	8.459
RI	1.0 MAR Nwamitwa Dam - FSL – 477.5 m - 200m spillway				
	Critical Storm Duration (h)	Q_{in} (m^3/s)	Q_{out} (m^3/s)	Max Stage (m)	Height above FSL (m)
1in100	34	2498	1701 (32%)	479.978	2.478
1in200	34	2917	2035 (30%)	480.293	2.793
PMF	26	12030	9861 (18%)	485.498	7.998
RI	1.5 MAR Nwamitwa Dam - FSL - 479.5 m - 200m spillway				
	Critical Storm Duration (h)	Q_{in} (m^3/s)	Q_{out} (m^3/s)	Max Stage (m)	Height above FSL (m)
1in100	34	2498	1551 (38%)	481.831	2.331
1in200	34	2917	1863 (36%)	482.133	2.633
PMF	26	12030	9415 (22%)	487.254	7.754

Table 4-6: Results of Nwamitwa Flood routing with 400 m spillway

RI	0.5 MAR Nwamitwa Dam - FSL - 473.5 m - 400m spillway				
	Critical Storm Duration (h)	Q_{in} (m^3/s)	Q_{out} (m^3/s)	Max Stage (m)	Height above FSL (m)
1in100	26	2823	2447 (13%)	475.490	1.990
1in200	22	3572	2952 (17%)	475.754	2.254
PMF	8	16185	12802 (21%)	479.496	5.996
RI	1.0 MAR Nwamitwa Dam – FSL - 477.5 m - 400m spillway				
	Critical Storm Duration (h)	Q_{in} (m^3/s)	Q_{out} (m^3/s)	Max Stage (m)	Height above FSL (m)
1in100	26	2823	2114 (25%)	479.304	1.804
1in200	26	3312	2535 (23%)	479.537	2.037
PMF	20	13179	11347 (14%)	483.032	5.532
RI	1.5 MAR Nwamitwa Dam – FSL - 479.5 m - 400m spillway				
	Critical Storm Duration (h)	Q_{in} (m^3/s)	Q_{out} (m^3/s)	Max Stage (m)	Height above FSL (m)
1in100	30	2641	1938 (27%)	481.203	1.703
1in200	30	3108	2340 (25%)	481.431	1.931
PMF	20	13179	10964 (17%)	484.907	5.407

5. RIVER DIVERSION FLOODS

In addition to the estimation of design discharges for Nwamitwa Dam spillway, 10 year, 20 year and 50 year RI flood hydrographs for a range of durations were also estimated at the proposed Nwamitwa Dam site. These hydrographs will be used for the design of diversion works during dam construction. The hydrographs were derived in the same manner as the spillway design inflow hydrographs to Nwamitwa Dam, i.e. the regional unit hydrograph technique was used to simulate separate hydrographs for the Tzaneen Dam catchment and the incremental Nwamitwa Dam catchment, after which the Tzaneen catchment hydrograph was routed through Tzaneen Dam and along the Groot Letaba River down to Nwamitwa Dam. At this point the Tzaneen subcatchment hydrograph was superimposed onto the incremental Nwamitwa subcatchment hydrograph to provide a combined hydrograph at the location of Nwamitwa Dam. (Note that these hydrographs were not routed through Nwamitwa Dam, i.e. only the inflow hydrographs to Nwamitwa Dam were determined.)

The flood hydrographs for a range of durations were calculated and are presented in **Appendix F**. Table 5-1 provides a summary of the range of flood peaks and flood volumes that were calculated for the various RIs and also shows the flood peaks as determined from the flood frequency analysis at Gauge B8H009 (refer to Section 4.1.3). The table shows that the unit hydrograph flood peaks are significantly higher than the peaks as calculated from the flood frequency analysis, specifically in the case of the 10 year and 20 year RI floods. This might be attributed to the fact that the unit hydrograph and routing approach assumes that Tzaneen Dam is at FSL at the start of each storm event. However, in reality, specifically during the minor RI events, Tzaneen Dam will absorb a significant proportion of the flood volume, which will be reflected in the peak flow record at Gauge B8H009. Such an under representation of flood peaks at Gauge B8H009 for the lower RI floods will be exacerbated when the area adjustment factor is used to upscale the B8H009 flood peaks, as this approach implicitly assumes that the same degree of attenuation applies to the rest of the catchment.

Table 5-1: River Diversion Floods

	Unit Hydrograph Technique			Flood Frequency Analysis
	Duration			
	8h	20h	34h	
	1 in 10 Year RI			
Flood Peak (m^3/s)	1172	1504	1239	606
Flood Volume (Mm^3)	17.2	28.2	32.2	
	1 in 20 Year RI			
Flood Peak (m^3/s)	1522	1903	1561	1080
Flood Volume (Mm^3)	23.0	36.8	42.0	
	1 in 50 Year RI			
Flood Peak (m^3/s)	2200	2516	2080	2090
Flood Volume (Mm^3)	32.6	49.5	55.8	

6. CONCLUSIONS

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

As it was not possible, under this Feasibility Study, to do any fresh research on extreme rainfall-versus-flood patterns in the region of the Groot Letaba catchment, a lead was taken from the SANCOLD Guidelines, which specifies the use of a Safety Evaluation Discharge (SED) for safety assessments on existing dams. According to the Guidelines the dam spillway must be capable of discharging the SED so that, although there may be extensive damage to the structure, it will not fail. For the "Large Dam/Significant to High Hazard" category (in which Nwamitwa Dam falls), the SED is set as the $RMF_{+\Delta}$, i.e. the RMF for the region one step higher numerically than that in which the study catchment lies; in this case for K = 5.4. It is therefore recommended that the unrouted $RMF_{+\Delta}$ value of 6800 m^3/s be used as an alternative SEF to the outgoing flood peak of an HRU 1/72-based PMF for the preliminary spillway design for Nwamitwa Dam. This implies an inflowing flood peak, before attenuation, of about 8500 m^3/s for a 1.0 MAR dam and a spillway length of 200 m. The K-value of such a flood peak is 5.6 and its ratio over RMF is 1.55.

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

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

Table 6-1: Recommended Spillway Design Floods

Design flood	100 year RI			RDF (200 year RI)			SEF ($RMF_{+\Delta}$)		
Dam capacity	**0.5 MAR**	**1.0 MAR**	**1.5 MAR**	**0.5 MAR**	**1.0 MAR**	**1.5 MAR**	**0.5 MAR**	**1.0 MAR**	**1.5 MAR**
Spillway length (m)	200								
Peak Outflow (m^3/s)	2048	1701	1551	2447	2035	1863	6800	6800	6800
Spillway length (m)	400								
Peak Outflow (m^3/s)	2447	2114	1938	2952	2535	2340	6800	6800	6800

In addition to the spillway design floods, 10 year, 20 year and 50 year RI flood hydrographs for a range of durations were estimated at the proposed Nwamitwa Dam site. Table 6-2 provides a summary of the range of flood peaks and flood volumes that were calculated.

Table 6-2: Calculated River Diversion Floods

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

7. REFERENCES

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Appendix A

Further Notes on Tzaneen Dam Catchment Characteristics

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 A1) 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) results in an unrealistic estimate (10.0 km) for L_c, 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 A1. 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 L_c.

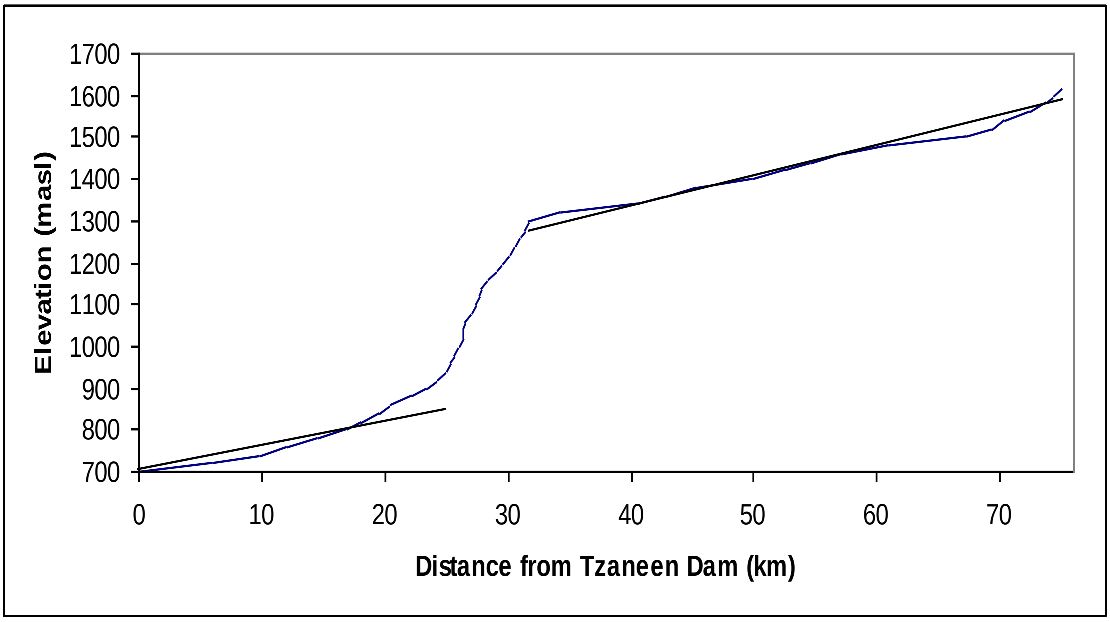

Figure A1: Longitudinal Profile of Groot Letaba River in Tzaneen Catchment

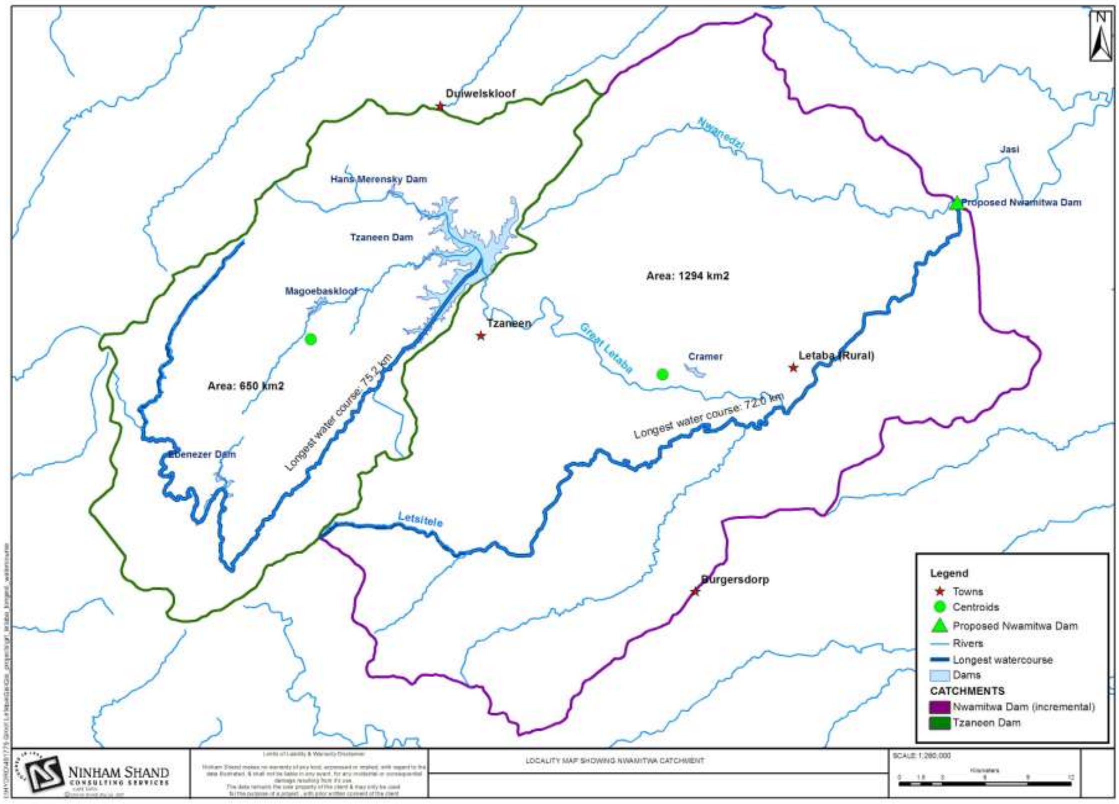

Figure A2: Tzaneen Dam: Catchment Centroid in Relation to Longest Watercourse

Appendix B

Storm Rainfall

Catchment characteristics	NWAMITWA (Incremental)				
Catchment area	129	km^2			
Veld zone	8	-	(HRU 1/72 Fig. F1)		
Extreme rainfall zone	1 &	-	(HRU 1/72 Fig. C3)		
Length of longest watercourse (L)	71.9	km		Point Rainfall	Smithers and Shulze, 2002
Length from centroid to outlet (Lc)	33.8	km		ARF	Alexander, 1990
Height 0.85L	635	masl		Storm loss factor	HRU 1/72 Figure G1 & G2
Height 0.10L	472	masl			
Average channel slope (Savg)	0.00	-			
Catchment Index	444	-			
Basin Lag	8.94	h	(HRU 1/72 Fig. F2)		

Design Rainfall

Return Period	10	year												
Duration (h)	8	10	12	16	18	20	22	24	26	28	30	32	34	36
Point rainfall (mm)	134.	141.	148.	159.	163.	167.	171.	175.	176.	176.	177.	178.	179.	179.
ARF	0.78	0.79	0.80	0.82	0.82	0.83	0.84	0.84	0.84	0.84	0.85	0.85	0.85	0.85
Catchment rainfall	105	112	118.	130	135	140	144	148	149	150	151	152	153	154
Storm loss factor	0.76	0.75	0.75	0.73	0.72	0.72	0.71	0.71	0.71	0.71	0.70	0.70	0.70	0.70
Storm loss (mm)	80	85	89	96	98	101	103	105	106	107	107	108	108	109
Storm rainfall (mm)	25	27	29	35	37	39	41	43	43	44	44	44	45	45

Return Period	20	year												
Duration (h)	8	10	12	16	18	20	22	24	26	28	30	32	34	36
Point rainfall (mm)	160.	169.	176.	189.	195.	200.	204.	209.	210.	211.	211.	212.	213.	214.
ARF	0.78	0.79	0.80	0.82	0.82	0.83	0.84	0.84	0.84	0.84	0.85	0.85	0.85	0.85
Catchment rainfall	125	134	141.	156	161	167	172	177	178	179	180	182	183	184
Storm loss factor	0.74	0.72	0.71	0.70	0.69	0.69	0.69	0.68	0.68	0.68	0.68	0.68	0.68	0.68
Storm loss (mm)	93	98	102	110	113	116	119	122	122	123	123	124	125	125
Storm rainfall (mm)	32	36	40	46	49	51	53	55	56	56	57	57	58	59

Return Period	100	year												
Duration (h)	8	10	12	16	18	20	22	24	26	28	30	32	34	36
Point rainfall (mm)	227.	240.	250.	269.	276.	283.	290.	296.	297.	299.	300.	301.	302.	304.
ARF	0.78	0.79	0.80	0.82	0.82	0.83	0.84	0.84	0.84	0.84	0.85	0.85	0.85	0.85
Catchment rainfall	177	190	200.	221	229	237	244	251	252	254	256	257	259	261
Storm loss factor	0.68	0.67	0.66	0.65	0.64	0.64	0.63	0.63	0.63	0.63	0.63	0.63	0.63	0.63
Storm loss (mm)	122	128	133	144	148	152	156	159	160	161	162	163	163	164
Storm rainfall (mm)	56	62	67	76	80	85	88	92	92	93	94	95	96	97

Return Period	200	year												
Duration (h)	8	10	12	16	18	20	22	24	26	28	30	32	34	36
Point rainfall (mm)	259.	274.	286.	307.	315.	324.	331.	339.	340.	341.	343.	344.	346.	347.
ARF	0.78	0.79	0.80	0.82	0.82	0.83	0.84	0.84	0.84	0.84	0.85	0.85	0.85	0.85
Catchment rainfall	203	217	229.	252	261	271	279	286	288	290	292	294	296	298
Storm loss factor	0.66	0.65	0.64	0.63	0.62	0.62	0.62	0.62	0.62	0.61	0.61	0.61	0.61	0.61
Storm loss (mm)	135	142	149	160	165	169	173	178	179	180	181	182	183	185
Storm rainfall (mm)	68	74	81	92	97	102	106	109	110	110	111	112	113	114

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.	540.	560.	600.	610.	620.	625.	630.	637.	645.	652.	660.	667.	675.
ARF	0.78	0.79	0.80	0.82	0.82	0.83	0.84	0.84	0.84	0.84	0.85	0.85	0.85	0.85
Catchment rainfall	406	427	448.	492	505	518	525	532	540	548	555	563	571	579
Storm loss factor	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03
Storm loss (mm)	12	13	13	15	15	16	16	16	16	16	17	17	17	17
Storm rainfall (mm)	393	414	435	477	490	502	509	516	524	531	539	546	554	561

Catchment characteristics	**TZANEEN**				
Catchment area	650.	km^2			
Veld zone	8	-	(HRU 1/72 Fig. F1)		
Extreme rainfall zone	1 &	-	(HRU 1/72 Fig. C3)		
Length of longest watercourse (L)	75.1	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)	0.00	-			
Catchment Index	347	-			
Basin Lag	8.19	h	(HRU 1/72 Fig. F2)		

Design Rainfall

Return Period	10	year												
Duration (h)	8	10	12	16	18	20	22	24	26	28	30	32	34	36
Point rainfall (mm)	151.	160.	167.	180.	185.	191.	195.	200.	201.	202.	203.	204.	205.	206.
ARF	0.78	0.79	0.80	0.82	0.82	0.83	0.84	0.84	0.84	0.84	0.85	0.85	0.85	0.85
Catchment rainfall	118	127	134.	148	154	159	164	169	171	172	173	174	176	177
Storm loss factor	0.74	0.73	0.72	0.71	0.70	0.70	0.69	0.69	0.68	0.68	0.68	0.68	0.68	0.68
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	year												
Duration (h)	8	10	12	16	18	20	22	24	26	28	30	32	34	36
Point rainfall (mm)	180.	191.	200.	215.	221.	227.	233.	239.	240.	241.	242.	243.	244.	245.
ARF	0.78	0.79	0.80	0.82	0.82	0.83	0.84	0.84	0.84	0.84	0.85	0.85	0.85	0.85
Catchment rainfall	141	151	160.	177	183	190	196	202	204	205	206	208	209	211
Storm loss factor	0.71	0.70	0.70	0.68	0.67	0.66	0.65	0.64	0.64	0.64	0.64	0.64	0.64	0.64
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	year												
Duration (h)	8	10	12	16	18	20	22	24	26	28	30	32	34	36
Point rainfall (mm)	256.	271.	284.	305.	314.	323.	331.	339.	340.	342.	343.	345.	347.	348.
ARF	0.78	0.79	0.80	0.82	0.82	0.83	0.84	0.84	0.84	0.84	0.85	0.85	0.85	0.85
Catchment rainfall	200	214	227.	250	260	270	278	287	289	291	293	295	297	299
Storm loss factor	0.65	0.64	0.63	0.62	0.62	0.62	0.61	0.61	0.61	0.61	0.61	0.61	0.61	0.61
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	year												
Duration (h)	8	10	12	16	18	20	22	24	26	28	30	32	34	36
Point rainfall (mm)	293.	309.	324.	348.	359.	369.	378.	387.	389.	391.	392.	394.	396.	398.
ARF	0.78	0.79	0.80	0.82	0.82	0.83	0.84	0.84	0.84	0.84	0.85	0.85	0.85	0.85
Catchment rainfall	229	245	259.	286	297	308	318	327	330	332	334	337	339	342
Storm loss factor	0.63	0.62	0.62	0.61	0.61	0.61	0.61	0.60	0.60	0.60	0.60	0.60	0.60	0.60
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.	540.	560.	600.	610.	620.	625.	630.	637.	645.	652.	660.	667.	675.
ARF	0.78	0.79	0.80	0.82	0.82	0.83	0.84	0.84	0.84	0.84	0.85	0.85	0.85	0.85
Catchment rainfall	406	427	448.	492	505	518	525	532	540	548	555	563	571	579
Storm loss factor	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
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

Appendix C

Flood Routing Information

Tzaneen Dam Flood Routing

Storage-Area-Elevation relationship (First Dam Safety Inspection of Tzaneen Dam. BKS, July 1999)

Elevation (masl)	Area (ha)	Volume (Mm^3)
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

Table C 1: Relationship between Stage, Area and Storage for Tzaneen Dam

Spillway characteristics (Great Letaba Main Irrigation Board: Tzaneen Dam Raising. BKS, 1998)

FSL: 723.9 masl

Spillway length: 91.44 m

Spillway discharge coefficient: 2.30

Nwamitwa Dam Flood Routing

Storage-Area-Elevation relationship

The storage-area-elevation relationship for Nwamitwa Dam was determined from a combination of 2m and 5m interval contours within the dam basin and is presented in **Table C 2**. The basin capacity relationship is represented by the following equation:

$$Volume = 0.0038y^{3.42}$$

where *y* is the water depth (m)

Elevation (masl)	Area (ha)	Volume (Mm^3)
456	13.1	0.0
458	23.5	0.4
460	55.1	1.2
462	85.9	2.6
464	167.7	5.1
466	310.5	9.9
468	485.6	17.8
470	693.8	29.6
472	1091.7	47.5
474	1478.2	73.2
476	1863.7	106.6
478	2291.6	148.2
480	2835.3	199.4

Table C 2: Relationship between Stage, Area and Storage for Nwamitwa Dam

Spillway characteristics

FSL:	0.5 MAR (FSL 473.5 masl)
	1.0 MAR (FSL 477.5 masl)
	1.5 MAR (FSL 479.5 masl)
Spillway length:	200 m and 400m
Spillway discharge coefficient:	2.18

Channel Routing (Tzaneen Dam to Nwamitwa Dam)

In order to calibrate the Muskingum coefficients for the reach of the Groot Letaba River between Tzaneen and Nwamitwa dams, flow records at gauging stations B8H002 and B8H009 were obtained. Station B8H002 is situated approximately 2 km downstream of Tzaneen Dam, while station B8H009 is located a further 31.5 km downstream. There are no significant tributaries which join the Groot Letaba River between these stations. The flood event of 20/12/1960 was selected as the calibration event. The corresponding flood hydrograph peaked at 565 m^3/s (station B8H002) and took 3 hours and 40 minutes to travel between the stations, with less than 1% attenuation. Based on this event, the Muskingum coefficients for the 62 km reach of the Groot Letaba River between Tzaneen and Nwamitwa dams were determined as 7.65 and 0.49 respectively.

Example

A typical example of a flood calculation is presented below. The selected storm is the 1 in 200 year event with a 34 hour duration. This storm results in the maximum outflow at Nwamitwa Dam with a 200 m spillway and a capacity equal to 1 MAR. It is clear from the data that the contribution made by the Tzaneen catchment in terms of the flow peak of the input hydrograph to Nwamitwa Dam is small. It does however add a large volume to the hydrograph tail as evident from Figures C 1 to C 3.

	Peak Flow (m^3/s)	Time of peak (h)	Maximum stage (m)
Inflow Hydrograph at Tzaneen Dam	1864	20.00	
Outflow Hydrograph at Tzaneen Dam	1239 (34%)	25.58	727.2
Attenuation to Nwamitwa Dam	1238	33.17	
Nwamitwa subcatchment hydrograph	2938	21.00	
Inflow Hydrograph at Nwamitwa Dam	2917	21.00	
Outflow Hydrograph at Nwamitwa Dam	2035 (30%)	29.17	480.3

Table C 3: System response to 1 in 200 year, 34h storm event

(Nwamitwa Dam = 1 MAR; spillway length = 200m)

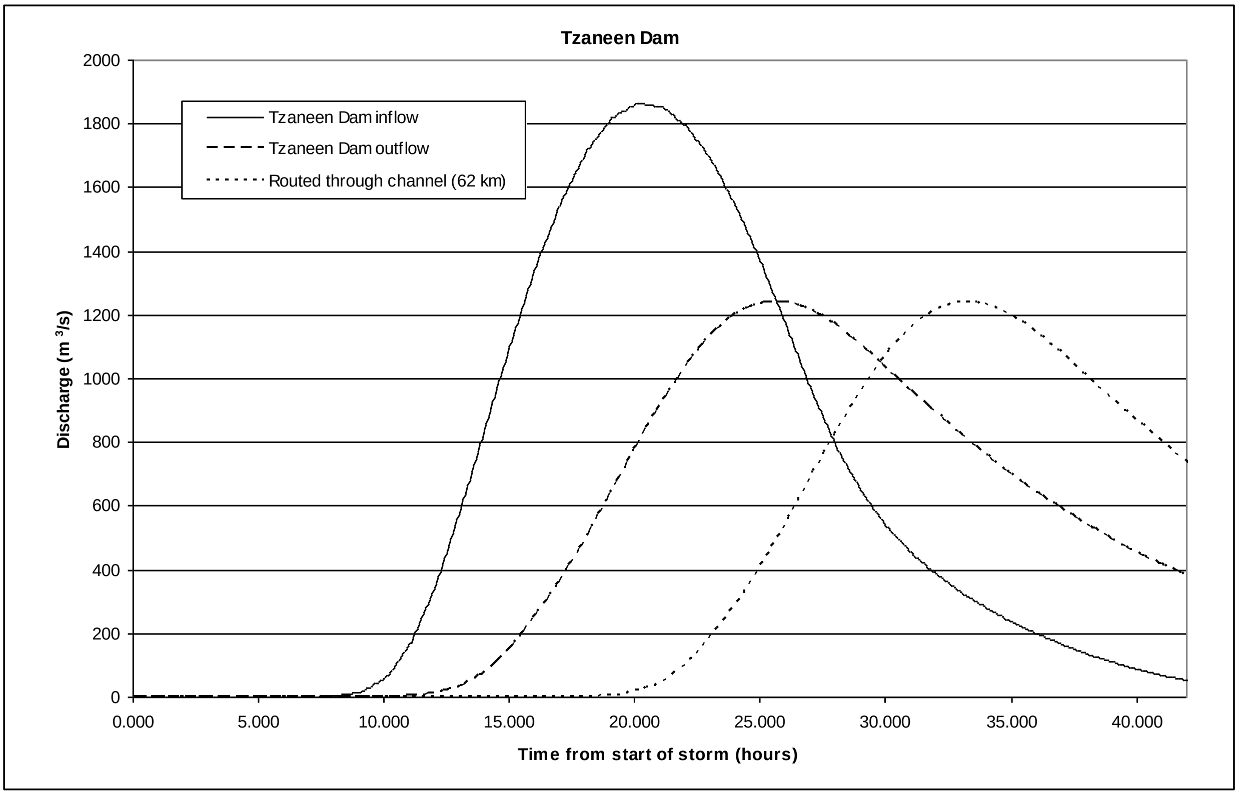

Figure C 1: Inflow and outflow hydrographs for Tzaneen Dam

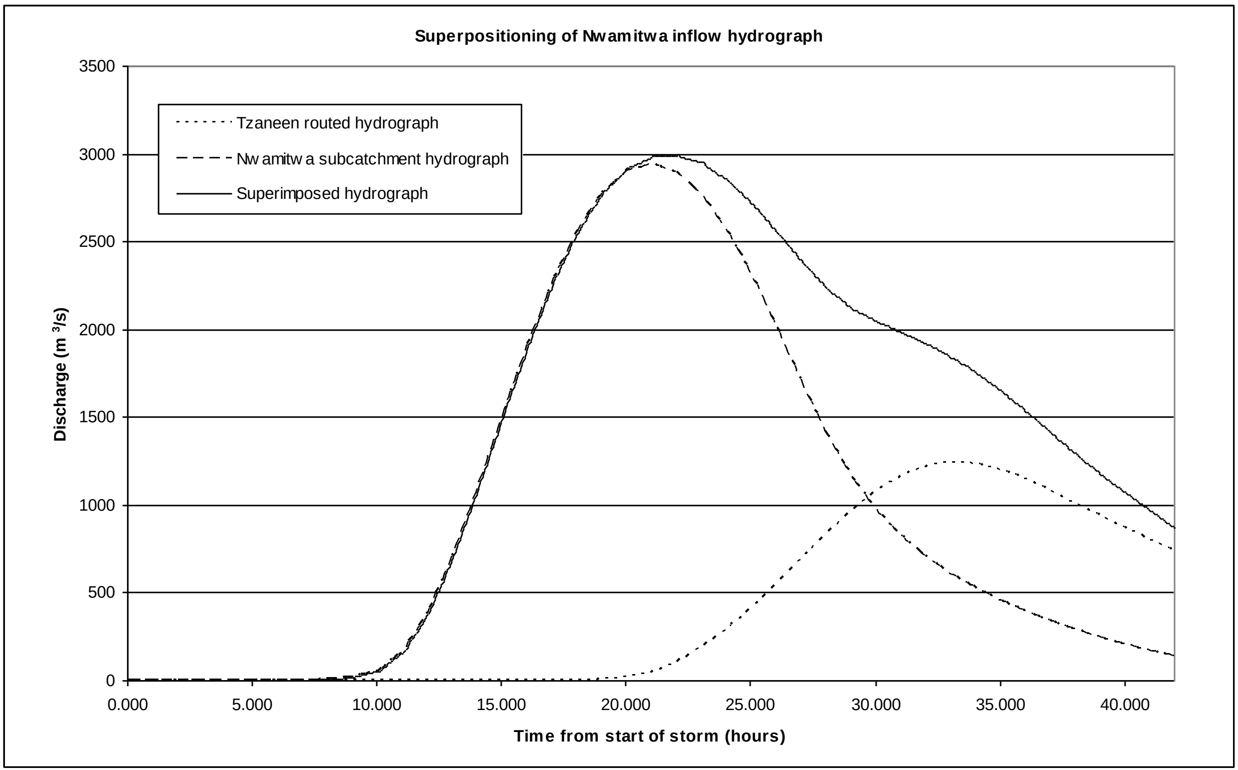

Figure C 2: Superposition of Nwamitwa Dam inflow hydrograph

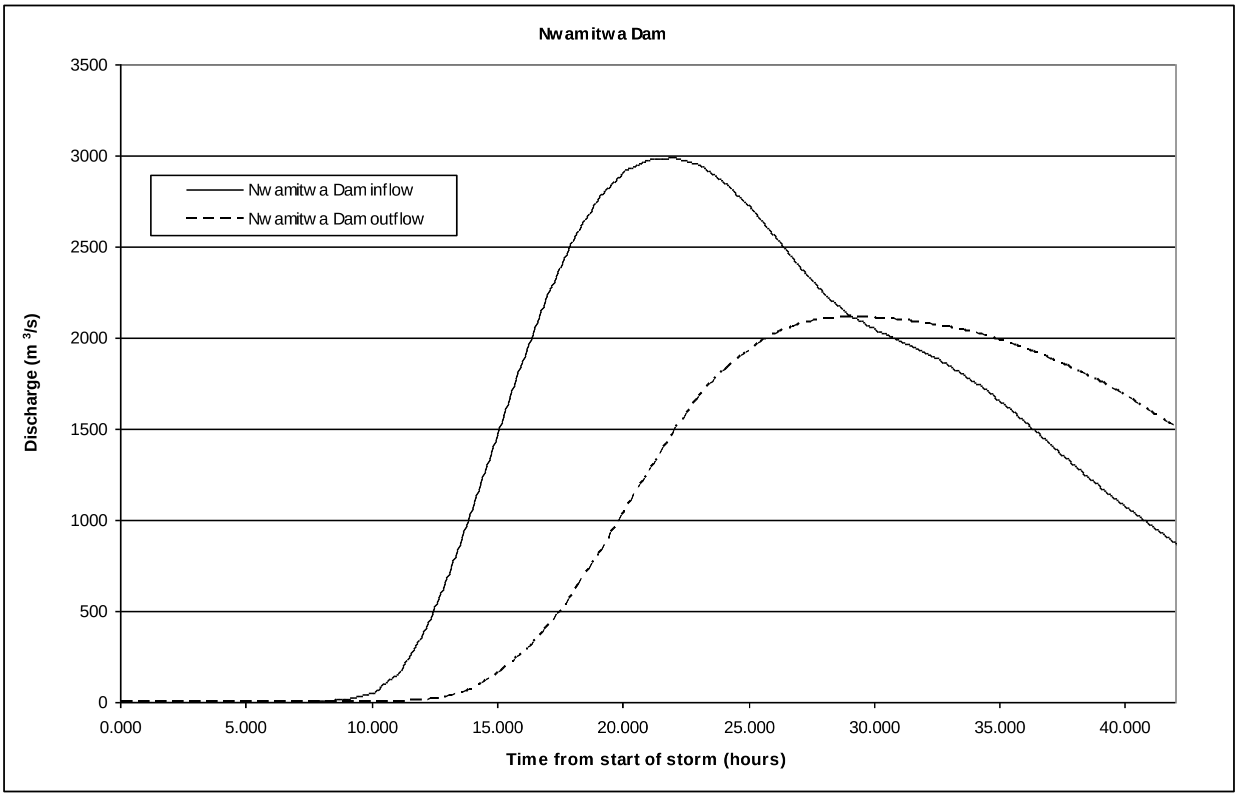

Figure C 3: Inflow and outflow hydrographs for Nwamitwa Dam

Appendix D

B8H009: Annual Maximum Flood Peaks

Department of Water Affairs and Forestry SAFPEAK Output 2007/10/08 11:25:24

B8H009 Great-Letaba River @ The Junction

Date	Time of Peak	Level at Peak (m)	Peak Flow (cumec)
1959/1960	02-09 01:00	1.018	39.491 M
1960/1961	12-20 06:00	4.443	560.354
1961/1962	01-20 00:42	0.863	29.556
1962/1963	12-12 11:48	1.170	49.239 M
1963/1964	02-10 01:48	3.380	333.451
1964/1965	12-13 08:42	2.890	247.762 M
1965/1966	01-26 22:24	1.375	62.400 M
1966/1967	02-10 00:42	1.423	65.481 M
1967/1968	03-09 15:00	0.733	21.233 M
1968/1969	03-17 01:12	1.251	54.437
1969/1970	12-10 04:48	0.610	13.369 M
1970/1971	01-22 18:30	0.919	33.145
1971/1972	02-24 09:54	3.823	421.236 M
1972/1973	09-29 20:18	0.728	20.913 M
1973/1974	02-08 08:12	4.351	538.522 M
1974/1975	02-27 17:06	1.760	96.860 M
1975/1976	01-31 15:00	4.460	564.401 M
1976/1977	02-09 22:24	0.988	37.567
1977/1978	01-04 12:06	3.410	339.096
1978/1979	08-25 05:06	0.525	9.001
1979/1980	02-25 23:54	2.020	125.702
1980/1981	02-05 08:37	4.000	459.030
1981/1982	12-31 09:12	1.060	42.184
1982/1983	03-23 08:24	0.659	16.500
1983/1984	11-12 11:18	0.495	7.872
1984/1985	02-08 19:06	2.530	192.534 M
1985/1986	02-02 11:06	1.501	71.693 M
1986/1987	12-29 11:18	2.295	160.075 M
1987/1988	02-26 07:18	1.962	118.957
1988/1989	02-28 22:24	0.935	34.169
1989/1990	12-09 02:06	0.958	35.644 M
1990/1991	02-08 11:36	1.505	72.057
1991/1992	03-15 00:18	0.547	9.891
1992/1993	12-31 05:18	0.528	9.120
1993/1994	02-01 10:00	1.714	92.130
1994/1995	10-15 23:00	0.081	0.360
1995/1996	02-13 03:24	6.571	566.810 A
1996/1997	03-06 05:30	3.833	423.338 M

1997/1998	01-30 17:36	1.196	50.909 M
1998/1999	02-05 11:48	1.455	67.628 M
1999/2000	02-25 03:24	7.421	566.810 M A
2000/2001	03-01 09:20	1.216	52.190
2001/2002	12-02 14:48	1.056	41.927
2002/2003	02-06 06:24	0.849	28.660 Q
2003/2004	04-06 12:48	1.083	43.661 Q
2004/2005	03-05 02:48	0.487	7.588 Q
2005/2006	03-03 06:11	2.589	201.136 Q
2006/2007	12-29 07:24	1.577	78.735 M

Department of Water Affairs and Forestry SAFPEAK Output 2007/10/08 11:25:24

B8H009 Great-Letaba River @ The Junction

Date	Time of Peak	Level at Peak (m)	Peak Flow (cumec)

Explanation of codes:

! - Data not yet checked

> - Minimum Value

A - Above Rating

M - Permanent Gap, Temporary Gap

Q - Good edited unaudited

T - Rating missing

Appendix E

Nwamitwa Dam Spillway: Routed Flood Hydrographs

200 m spillway									
0.5 MAR Nwamitwa Dam									
Hours	1 in 100 yr			1 in 200 yr			PMF		
	Q_{in} (m^3/s)	Q_{out} (m^3/s	Stage (masl)	Q_{in} (m^3/s	Q_{out} (m^3/s	Stage (masl)	Q_{in} (m^3/s	Q_{out} (m^3/s	Stage (masl)
0.0	0.0	0.0	473.5	0.0	0.0	473.5	0.0	0.0	473.5
1.0	0.0	0.0	473.5	0.0	0.0	473.5	3.9	0.0	473.5
2.0	0.0	0.0	473.5	0.0	0.0	473.5	13.7	0.0	473.5
3.0	0.0	0.0	473.5	0.0	0.0	473.5	39.4	0.4	473.5
4.0	0.0	0.0	473.5	0.0	0.0	473.5	106.7	2.1	473.5
5.0	0.0	0.0	473.5	0.0	0.0	473.5	361.8	11.9	473.6
6.0	2.6	0.0	473.5	3.2	0.0	473.5	699.5	46.6	473.7
7.0	12.0	0.0	473.5	14.6	0.0	473.5	1266.7	135.2	474.0
8.0	35.5	0.3	473.5	43.0	0.4	473.5	1971.0	316.6	474.3
9.0	95.6	1.8	473.5	115.5	2.4	473.5	3315.4	661.0	474.8
10.0	297.1	9.4	473.6	360.4	12.5	473.6	4818.4	1237.3	475.5
11.0	650.4	38.5	473.7	780.1	50.6	473.7	6896.7	2089.4	476.3
12.0	1112.4	114.3	473.9	1322.5	147.7	474.0	8814.6	3200.0	477.3
13.0	1606.9	259.1	474.2	1898.4	329.1	474.3	10399.8	4452.2	478.2
14.0	2064.7	477.3	474.6	2429.1	597.0	474.7	11649.2	5733.0	479.1
15.0	2444.0	753.4	474.9	2869.6	930.1	475.2	12535.4	6957.5	479.8
16.0	2703.0	1058.0	475.3	3170.1	1292.5	475.6	13178.9	8077.1	480.5
17.0	2823.4	1357.2	475.6	3312.0	1644.5	475.9	12916.9	9003.5	481.0
18.0	2808.7	1620.6	475.9	3300.3	1951.8	476.2	12517.1	9685.7	481.4
19.0	2678.5	1826.6	476.1	3157.5	2190.7	476.4	11994.1	10151.9	481.7
20.0	2471.3	1964.7	476.2	2929.6	2350.6	476.6	11674.6	10454.5	481.8
21.0	2224.3	2035.1	476.3	2645.7	2432.5	476.6	11272.2	10637.3	481.9
22.0	1963.3	2045.0	476.3	2347.8	2443.3	476.7	10894.7	10717.3	482.0
23.0	1774.7	2011.2	476.3	2121.4	2402.4	476.6	10583.3	10721.6	482.0
24.0	1661.3	1955.9	476.2	1990.5	2336.1	476.6	10281.6	10670.1	481.9
25.0	1611.8	1896.2	476.2	1927.0	2264.5	476.5	9777.9	10554.9	481.9
26.0	1590.8	1841.7	476.1	1898.8	2198.2	476.4	9188.3	10360.9	481.8
27.0	1569.4	1793.6	476.1	1874.5	2139.9	476.4	8614.4	10095.4	481.6
28.0	1541.1	1750.0	476.0	1838.3	2087.1	476.3	8054.2	9773.2	481.5
29.0	1490.8	1707.2	476.0	1775.0	2034.9	476.3	7506.4	9406.6	481.2
30.0	1419.4	1661.1	475.9	1686.0	1978.1	476.2	6930.6	9000.9	481.0
31.0	1334.4	1609.1	475.9	1579.8	1913.7	476.2	6389.0	8563.9	480.8
32.0	1241.7	1550.4	475.8	1461.9	1840.5	476.1	5850.9	8104.7	480.5
33.0	1146.3	1485.6	475.8	1342.8	1759.0	476.0	5304.2	7626.6	480.2
34.0	1052.7	1415.8	475.7	1229.7	1671.6	475.9	4787.8	7135.0	479.9
35.0	960.1	1342.3	475.6	1121.6	1580.6	475.9	4275.7	6636.1	479.6
36.0	873.9	1266.5	475.5	1019.7	1487.7	475.8	3782.4	6133.2	479.3

200 m spillway									
1.0 MAR Nwamitwa Dam									
Hours	1 in 100 yr			1 in 200 yr			PMF		
	Q_{in} (m^3/s)	Q_{out} (m^3/s	Stage (masl)	Q_{in} (m^3/s	Q_{out} (m^3/s	Stage (masl)	Q_{in} (m^3/s	Q_{out} (m^3/s	Stage (masl)
0.0	0.0	0.0	477.5	0.0	0.0	477.5	0.0	0.0	477.5
1.0	0.0	0.0	477.5	0.0	0.0	477.5	5.6	0.0	477.5
2.0	0.0	0.0	477.5	0.0	0.0	477.5	16.6	0.0	477.5
3.0	0.0	0.0	477.5	0.0	0.0	477.5	40.9	0.2	477.5
4.0	0.0	0.0	477.5	0.0	0.0	477.5	112.8	1.2	477.5
5.0	0.0	0.0	477.5	0.0	0.0	477.5	402.6	6.6	477.6
6.0	0.0	0.0	477.5	0.0	0.0	477.5	635.8	23.7	477.6
7.0	0.8	0.0	477.5	1.1	0.0	477.5	961.9	60.0	477.8
8.0	4.7	0.0	477.5	5.9	0.0	477.5	1504.4	130.6	477.9
9.0	15.4	0.0	477.5	19.0	0.0	477.5	2372.3	265.4	478.2
10.0	42.6	0.2	477.5	52.4	0.3	477.5	3569.8	507.1	478.6
11.0	127.8	1.3	477.5	158.6	1.8	477.5	4945.1	894.1	479.1
12.0	310.5	5.7	477.6	377.0	7.8	477.6	6424.9	1444.5	479.7
13.0	581.3	19.7	477.6	695.3	26.3	477.7	7875.6	2150.0	480.4
14.0	909.4	52.0	477.7	1076.5	67.9	477.8	9185.3	2977.2	481.1
15.0	1256.2	111.3	477.9	1477.4	143.0	478.0	10277.3	3877.0	481.8
16.0	1593.8	203.0	478.1	1867.5	257.5	478.2	11110.4	4797.7	482.4
17.0	1897.3	327.9	478.3	2217.8	411.2	478.5	11658.3	5691.8	483.0
18.0	2146.2	481.2	478.6	2505.3	597.6	478.7	11949.2	6521.2	483.6
19.0	2329.4	654.0	478.8	2718.1	805.5	479.0	12030.1	7260.9	484.0
20.0	2444.9	835.4	479.0	2852.0	1021.8	479.3	11961.1	7898.8	484.4
21.0	2498.1	1014.5	479.3	2913.9	1233.9	479.5	11794.8	8433.2	484.7
22.0	2492.4	1182.2	479.4	2916.6	1431.2	479.7	11585.1	8868.1	485.0
23.0	2444.9	1331.6	479.6	2867.6	1606.5	479.9	11344.0	9214.9	485.1
24.0	2356.8	1458.0	479.7	2766.6	1754.3	480.0	11047.1	9478.8	485.3
25.0	2228.5	1558.1	479.8	2625.9	1870.4	480.1	10739.1	9667.1	485.4
26.0	2078.3	1629.9	479.9	2458.2	1953.8	480.2	10422.4	9788.8	485.5
27.0	1918.0	1674.6	480.0	2279.5	2005.4	480.3	10087.7	9851.1	485.5
28.0	1782.1	1695.8	480.0	2124.3	2030.0	480.3	9686.1	9856.3	485.5
29.0	1686.1	1700.5	480.0	2005.9	2034.3	480.3	9125.2	9796.8	485.5
30.0	1619.4	1694.7	480.0	1923.7	2025.7	480.3	8444.1	9662.5	485.4
31.0	1565.6	1682.4	480.0	1859.7	2009.0	480.3	7753.5	9454.4	485.3
32.0	1511.9	1665.0	479.9	1795.5	1986.5	480.2	7111.0	9184.6	485.1
33.0	1453.3	1642.9	479.9	1723.1	1958.2	480.2	6524.7	8867.8	485.0
34.0	1384.7	1615.8	479.9	1637.5	1923.7	480.2	5973.7	8516.7	484.8
35.0	1305.1	1583.0	479.9	1538.8	1882.0	480.2	5456.2	8140.2	484.5
36.0	1217.5	1544.1	479.8	1428.1	1832.5	480.1	4979.6	7745.9	484.3

200 m spillway									
1.5 MAR Nwamitwa Dam									
Hours	1 in 100 yr			1 in 200 yr			PMF		
	Q_{in} (m^3/s)	Q_{out} (m^3/s	Stage (masl)	Q_{in} (m^3/s	Q_{out} (m^3/s	Stage (masl)	Q_{in} (m^3/s	Q_{out} (m^3/s	Stage (masl)
0.0	0.0	0.0	479.5	0.0	0.0	479.5	0.0	0.0	479.5
1.0	0.0	0.0	479.5	0.0	0.0	479.5	5.6	0.0	479.5
2.0	0.0	0.0	479.5	0.0	0.0	479.5	16.6	0.0	479.5
3.0	0.0	0.0	479.5	0.0	0.0	479.5	40.9	0.2	479.5
4.0	0.0	0.0	479.5	0.0	0.0	479.5	112.8	0.8	479.5
5.0	0.0	0.0	479.5	0.0	0.0	479.5	402.6	4.8	479.5
6.0	0.0	0.0	479.5	0.0	0.0	479.5	635.8	17.4	479.6
7.0	0.8	0.0	479.5	1.1	0.0	479.5	961.9	44.2	479.7
8.0	4.7	0.0	479.5	5.9	0.0	479.5	1504.4	97.5	479.9
9.0	15.4	0.0	479.5	19.0	0.0	479.5	2372.3	200.6	480.1
10.0	42.6	0.2	479.5	52.4	0.2	479.5	3569.8	389.1	480.4
11.0	127.8	0.9	479.5	158.6	1.3	479.5	4945.1	698.3	480.9
12.0	310.5	4.2	479.5	377.0	5.7	479.6	6424.9	1149.8	481.4
13.0	581.3	14.4	479.6	695.3	19.2	479.6	7875.6	1744.9	482.0
14.0	909.4	38.3	479.7	1076.5	50.2	479.7	9185.3	2461.7	482.7
15.0	1256.2	82.9	479.8	1477.4	106.9	479.9	10277.3	3261.6	483.3
16.0	1593.8	153.1	480.0	1867.5	195.0	480.1	11110.4	4100.5	484.0
17.0	1897.3	250.8	480.2	2217.8	316.1	480.3	11658.3	4934.5	484.5
18.0	2146.2	373.4	480.4	2505.3	466.5	480.5	11949.2	5726.3	485.1
19.0	2329.4	515.2	480.6	2718.1	638.9	480.8	12030.1	6450.2	485.5
20.0	2444.9	668.3	480.8	2852.0	823.2	481.0	11961.1	7091.2	485.9
21.0	2498.1	823.8	481.0	2913.9	1009.4	481.3	11794.8	7644.3	486.2
22.0	2492.4	974.1	481.2	2916.6	1188.2	481.5	11585.1	8110.5	486.5
23.0	2444.9	1112.9	481.4	2867.6	1352.7	481.6	11344.0	8497.5	486.7
24.0	2356.8	1235.3	481.5	2766.6	1497.4	481.8	11047.1	8808.6	486.9
25.0	2228.5	1337.7	481.6	2625.9	1617.5	481.9	10739.1	9048.6	487.1
26.0	2078.3	1417.6	481.7	2458.2	1711.3	482.0	10422.4	9224.9	487.1
27.0	1918.0	1475.0	481.8	2279.5	1778.4	482.1	10087.7	9343.5	487.2
28.0	1782.1	1512.3	481.8	2124.3	1821.9	482.1	9686.1	9406.3	487.2
29.0	1686.1	1534.5	481.8	2005.9	1847.1	482.1	9125.2	9406.5	487.2
30.0	1619.4	1546.4	481.8	1923.7	1859.3	482.1	8444.1	9335.3	487.2
31.0	1565.6	1551.2	481.8	1859.7	1862.8	482.1	7753.5	9193.4	487.1
32.0	1511.9	1550.1	481.8	1795.5	1859.4	482.1	7111.0	8990.9	487.0
33.0	1453.3	1543.5	481.8	1723.1	1849.2	482.1	6524.7	8740.9	486.9
34.0	1384.7	1531.1	481.8	1637.5	1831.9	482.1	5973.7	8453.9	486.7
35.0	1305.1	1512.5	481.8	1538.8	1806.8	482.1	5456.2	8137.9	486.5
36.0	1217.5	1487.5	481.8	1428.1	1773.6	482.0	4979.6	7800.2	486.3

400 m spillway									
0.5 MAR Nwamitwa Dam									
Hours	1 in 100 yr			1 in 200 yr			PMF		
	Q_{in} (m^3/s)	Q_{out} (m^3/s	Stage (masl)	Q_{in} (m^3/s	Q_{out} (m^3/s	Stage (masl)	Q_{in} (m^3/s	Q_{out} (m^3/s	Stage (masl)
0.0	0.0	0.0	473.5	0.0	0.0	473.5	0.0	0.0	473.5
1.0	0.0	0.0	473.5	0.0	0.0	473.5	12.9	0.0	473.5
2.0	0.0	0.0	473.5	0.0	0.0	473.5	49.3	0.8	473.5
3.0	0.0	0.0	473.5	0.0	0.0	473.5	125.5	5.2	473.5
4.0	0.0	0.0	473.5	0.0	0.0	473.5	340.2	24.4	473.6
5.0	0.0	0.0	473.5	2.8	0.0	473.5	1165.1	126.0	473.8
6.0	2.6	0.0	473.5	10.1	0.0	473.5	2282.3	454.4	474.1
7.0	12.0	0.1	473.5	35.4	0.6	473.5	3473.1	1089.9	474.7
8.0	35.5	0.6	473.5	101.3	3.5	473.5	5488.1	2124.4	475.3
9.0	95.6	3.5	473.5	312.4	19.3	473.6	8789.8	3806.8	476.2
10.0	297.1	18.2	473.6	624.0	73.9	473.7	12655.6	6241.8	477.2
11.0	650.4	73.2	473.7	1320.7	229.2	473.9	15567.3	9043.5	478.3
12.0	1112.4	210.2	473.9	2022.4	550.8	474.2	16185.1	11447.4	479.1
13.0	1606.9	456.6	474.1	2630.9	1017.8	474.6	13975.8	12684.4	479.5
14.0	2064.7	801.9	474.4	3082.7	1552.2	475.0	11327.8	12638.2	479.4
15.0	2444.0	1204.1	474.7	3382.1	2070.1	475.3	9043.3	11750.7	479.2
16.0	2703.0	1608.1	475.0	3571.5	2519.3	475.5	7623.3	10531.8	478.8
17.0	2823.4	1962.2	475.2	3363.0	2824.6	475.7	7003.9	9381.3	478.4
18.0	2808.7	2229.3	475.4	3057.4	2946.9	475.8	6943.1	8528.1	478.1
19.0	2678.5	2390.9	475.5	2611.3	2903.5	475.7	7321.0	8045.5	477.9
20.0	2471.3	2446.7	475.5	2258.9	2745.9	475.6	7605.0	7854.3	477.8
21.0	2224.3	2411.6	475.5	2009.4	2544.6	475.5	7614.5	7779.9	477.8
22.0	1963.3	2307.1	475.4	1857.6	2346.3	475.4	7342.9	7679.9	477.8
23.0	1774.7	2166.2	475.3	1824.6	2185.8	475.3	6879.7	7479.7	477.7
24.0	1661.3	2024.4	475.3	1812.4	2070.8	475.3	6337.5	7169.4	477.6
25.0	1611.8	1903.7	475.2	1822.5	1992.9	475.2	5765.0	6771.5	477.4
26.0	1590.8	1811.0	475.1	1805.3	1938.5	475.2	5206.0	6313.4	477.2
27.0	1569.4	1741.0	475.1	1752.6	1889.8	475.2	4682.6	5829.3	477.0
28.0	1541.1	1685.1	475.1	1670.0	1834.9	475.1	4177.6	5336.2	476.8
29.0	1490.8	1634.4	475.0	1568.3	1768.5	475.1	3709.4	4848.5	476.6
30.0	1419.4	1580.5	475.0	1453.9	1689.5	475.1	3275.5	4376.8	476.4
31.0	1334.4	1519.4	474.9	1338.9	1600.1	475.0	2891.3	3931.4	476.2
32.0	1241.7	1450.3	474.9	1228.3	1504.6	474.9	2540.7	3517.3	476.0
33.0	1146.3	1374.3	474.9	1122.2	1406.4	474.9	2230.0	3136.0	475.8
34.0	1052.7	1293.7	474.8	1021.6	1308.1	474.8	1955.8	2790.1	475.7
35.0	960.1	1210.5	474.7	925.4	1211.1	474.7	1707.0	2476.8	475.5
36.0	873.9	1126.5	474.7	837.2	1116.9	474.7	1483.7	2193.3	475.3

400 m spillway									
1.0 MAR Nwamitwa Dam									
Hours	1 in 100 yr			1 in 200 yr			PMF		
	Q_{in} (m^3/s)	Q_{out} (m^3/s	Stage (masl)	Q_{in} (m^3/s	Q_{out} (m^3/s	Stage (masl)	Q_{in} (m^3/s	Q_{out} (m^3/s	Stage (masl)
0.0	0.0	0.0	477.5	0.0	0.0	477.5	0.0	0.0	477.5
1.0	0.0	0.0	477.5	0.0	0.0	477.5	3.9	0.0	477.5
2.0	0.0	0.0	477.5	0.0	0.0	477.5	13.7	0.0	477.5
3.0	0.0	0.0	477.5	0.0	0.0	477.5	39.4	0.4	477.5
4.0	0.0	0.0	477.5	0.0	0.0	477.5	106.7	2.0	477.5
5.0	0.0	0.0	477.5	0.0	0.0	477.5	361.8	11.3	477.6
6.0	2.6	0.0	477.5	3.2	0.0	477.5	699.5	44.6	477.6
7.0	12.0	0.0	477.5	14.6	0.0	477.5	1266.7	131.7	477.8
8.0	35.5	0.3	477.5	43.0	0.4	477.5	1971.0	313.6	478.0
9.0	95.6	1.7	477.5	115.5	2.3	477.5	3315.4	669.7	478.3
10.0	297.1	8.9	477.5	360.4	11.9	477.6	4818.4	1286.2	478.8
11.0	650.4	36.9	477.6	780.1	48.6	477.6	6896.7	2230.4	479.4
12.0	1112.4	111.0	477.8	1322.5	144.0	477.8	8814.6	3497.9	480.0
13.0	1606.9	255.4	477.9	1898.4	326.4	478.0	10399.8	4952.0	480.7
14.0	2064.7	478.4	478.2	2429.1	602.6	478.3	11649.2	6444.2	481.3
15.0	2444.0	766.6	478.4	2869.6	953.9	478.6	12535.4	7855.7	481.8
16.0	2703.0	1089.8	478.7	3170.1	1342.2	478.8	13178.9	9116.6	482.3
17.0	2823.4	1409.8	478.9	3312.0	1721.6	479.1	12916.9	10106.4	482.6
18.0	2808.7	1690.7	479.1	3300.3	2050.9	479.3	12517.1	10761.6	482.8
19.0	2678.5	1906.8	479.2	3157.5	2301.4	479.4	11994.1	11133.7	483.0
20.0	2471.3	2045.5	479.3	2929.6	2460.5	479.5	11674.6	11306.0	483.0
21.0	2224.3	2107.9	479.3	2645.7	2530.4	479.5	11272.2	11345.2	483.0
22.0	1963.3	2103.5	479.3	2347.8	2521.1	479.5	10894.7	11277.4	483.0
23.0	1774.7	2052.8	479.3	2121.4	2457.1	479.5	10583.3	11140.9	483.0
24.0	1661.3	1981.7	479.2	1990.5	2369.1	479.4	10281.6	10962.0	482.9
25.0	1611.8	1909.4	479.2	1927.0	2280.4	479.4	9777.9	10726.0	482.8
26.0	1590.8	1845.7	479.1	1898.8	2201.7	479.4	9188.3	10410.3	482.7
27.0	1569.4	1791.3	479.1	1874.5	2135.0	479.3	8614.4	10027.5	482.6
28.0	1541.1	1743.3	479.1	1838.3	2076.5	479.3	8054.2	9598.3	482.4
29.0	1490.8	1697.2	479.1	1775.0	2019.9	479.3	7506.4	9138.1	482.3
30.0	1419.4	1648.2	479.0	1686.0	1959.4	479.2	6930.6	8652.1	482.1
31.0	1334.4	1593.4	479.0	1579.8	1891.1	479.2	6389.0	8148.9	481.9
32.0	1241.7	1532.1	479.0	1461.9	1814.2	479.1	5850.9	7638.3	481.7
33.0	1146.3	1464.7	478.9	1342.8	1729.1	479.1	5304.2	7121.3	481.6
34.0	1052.7	1392.6	478.9	1229.7	1638.6	479.0	4787.8	6602.8	481.4
35.0	960.1	1317.2	478.8	1121.6	1545.1	479.0	4275.7	6088.8	481.2
36.0	873.9	1240.0	478.8	1019.7	1450.6	478.9	3782.4	5581.0	480.9

400 m spillway									
1.5 MAR Nwamitwa Dam									
Hours	1 in 100 yr			1 in 200 yr			PMF		
	Q_{in} (m^3/s)	Q_{out} (m^3/s	Stage (masl)	Q_{in} (m^3/s	Q_{out} (m^3/s	Stage (masl)	Q_{in} (m^3/s	Q_{out} (m^3/s	Stage (masl)
0.0	0.0	0.0	479.5	0.0	0.0	479.5	0.0	0.0	479.5
1.0	0.0	0.0	479.5	0.0	0.0	479.5	3.9	0.0	479.5
2.0	0.0	0.0	479.5	0.0	0.0	479.5	13.7	0.0	479.5
3.0	0.0	0.0	479.5	0.0	0.0	479.5	39.4	0.3	479.5
4.0	0.0	0.0	479.5	0.0	0.0	479.5	106.7	1.5	479.5
5.0	0.0	0.0	479.5	0.0	0.0	479.5	361.8	8.3	479.5
6.0	0.4	0.0	479.5	0.7	0.0	479.5	699.5	32.9	479.6
7.0	3.6	0.0	479.5	5.0	0.0	479.5	1266.7	98.3	479.7
8.0	13.5	0.0	479.5	17.7	0.0	479.5	1971.0	238.4	479.9
9.0	38.4	0.3	479.5	49.7	0.4	479.5	3315.4	519.5	480.2
10.0	109.2	1.5	479.5	144.4	2.2	479.5	4818.4	1021.1	480.6
11.0	301.0	7.2	479.5	378.5	10.5	479.6	6896.7	1814.4	481.1
12.0	608.0	27.2	479.6	742.9	38.0	479.6	8814.6	2916.0	481.7
13.0	994.2	76.7	479.7	1194.1	103.8	479.7	10399.8	4226.1	482.4
14.0	1405.3	171.3	479.8	1671.0	225.6	479.9	11649.2	5618.5	483.0
15.0	1796.9	318.9	480.0	2124.7	411.9	480.1	12535.4	6980.4	483.5
16.0	2139.2	517.2	480.2	2520.5	657.6	480.3	13178.9	8237.3	484.0
17.0	2401.2	753.0	480.4	2824.0	945.5	480.6	12916.9	9271.5	484.3
18.0	2569.5	1006.1	480.6	3020.3	1250.3	480.8	12517.1	10013.9	484.6
19.0	2641.3	1254.3	480.8	3108.2	1545.7	481.0	11994.1	10495.6	484.8
20.0	2627.2	1477.8	480.9	3094.9	1809.0	481.1	11674.6	10782.1	484.8
21.0	2543.8	1662.2	481.0	3006.5	2024.7	481.3	11272.2	10930.1	484.9
22.0	2410.4	1800.6	481.1	2861.2	2185.5	481.3	10894.7	10962.4	484.9
23.0	2236.9	1890.5	481.2	2662.2	2288.1	481.4	10583.3	10914.0	484.9
24.0	2038.8	1932.7	481.2	2440.6	2334.9	481.4	10281.6	10809.8	484.9
25.0	1860.0	1935.1	481.2	2228.8	2333.9	481.4	9777.9	10639.9	484.8
26.0	1733.2	1910.5	481.2	2080.5	2301.1	481.4	9188.3	10388.1	484.7
27.0	1658.6	1873.2	481.2	1984.6	2252.6	481.4	8614.4	10064.9	484.6
28.0	1614.2	1832.4	481.1	1928.1	2199.5	481.4	8054.2	9689.0	484.5
29.0	1575.7	1791.7	481.1	1882.8	2147.3	481.3	7506.4	9275.3	484.3
30.0	1533.9	1751.4	481.1	1830.1	2095.9	481.3	6930.6	8829.6	484.2
31.0	1480.7	1710.0	481.1	1763.4	2043.3	481.3	6389.0	8360.9	484.0
32.0	1410.9	1665.3	481.0	1675.6	1986.5	481.2	5850.9	7878.8	483.8
33.0	1328.2	1615.5	481.0	1571.5	1923.0	481.2	5304.2	7385.7	483.7
34.0	1237.3	1559.6	481.0	1455.4	1851.8	481.2	4787.8	6886.9	483.5
35.0	1142.8	1498.1	480.9	1337.3	1773.0	481.1	4275.7	6388.2	483.3
36.0	1049.7	1431.8	480.9	1224.6	1688.8	481.1	3782.4	5892.1	483.1

Appendix F

River Diversion Floods

	1:10 year Hydrographs for various Storm Durations (m^3/s)												
	Duration (h)												
Time (h)	**8**	**10**	**12**	**16**	**18**	**20**	**22**	**24**	**26**	**28**	**30**	**32**	**34**
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2.0	11.4	3.8	4.0	4.0	-0.1	1.3	0.9	0.8	0.4	0.0	0.0	0.0	0.0
2.5	51.7	27.2	22.8	18.7	6.2	10.3	8.5	7.4	4.2	1.8	0.9	0.3	0.0
3.0	259.6	120.8	119.9	109.1	38.6	61.3	51.5	40.4	25.6	14.3	8.7	5.2	2.6
3.5	837.6	586.1	442.8	319.4	144.0	197.9	172.2	182.6	111.6	58.2	35.7	22.7	12.6
4.0	1171.5	1174.5	968.9	713.4	683.0	662.5	633.4	538.4	386.2	265.9	179.4	118.8	70.0
4.5	962.0	1254.5	1301.6	1152.7	1148.5	1101.9	1075.5	959.7	760.2	591.4	442.9	331.1	232.2
5.0	665.9	915.2	1131.2	1368.6	1354.7	1369.2	1348.3	1285.9	1107.5	938.1	765.1	626.2	492.6
5.5	469.0	623.8	809.9	1244.9	1414.5	1503.9	1500.0	1451.6	1346.2	1218.9	1055.8	915.3	767.8
6.0	360.0	459.1	580.1	918.6	1087.5	1218.2	1296.5	1383.5	1393.5	1352.2	1242.3	1135.8	1003.9
6.5	354.6	397.3	459.4	679.5	777.7	898.0	1002.8	1168.3	1293.3	1352.1	1317.9	1267.4	1172.0
7.0	374.7	419.3	447.1	566.3	633.6	716.6	789.1	910.2	1068.4	1212.8	1259.7	1279.7	1238.7
7.5	376.1	434.9	469.4	551.3	582.9	636.1	682.0	755.4	855.6	1015.8	1125.2	1212.5	1229.2
8.0	360.3	424.4	470.1	563.5	594.7	637.9	668.1	708.3	752.4	847.6	945.7	1074.4	1141.8
8.5	333.7	398.4	448.5	556.2	595.2	638.6	664.7	693.0	708.8	756.1	809.2	915.2	1018.0
9.0	303.5	365.8	416.7	528.9	572.2	619.7	649.8	684.9	695.4	722.4	741.8	804.2	884.0
9.5	273.2	331.1	380.3	488.2	531.3	579.8	612.3	655.0	675.9	700.8	708.1	740.5	782.1
10.0	244.5	297.4	344.4	443.4	484.8	531.4	563.7	609.4	640.5	675.3	686.0	710.2	731.6
10.5	216.2	264.6	308.8	397.8	436.4	479.5	510.1	555.2	590.9	632.9	653.1	681.0	696.4
11.0	191.2	233.0	273.5	355.8	389.2	428.9	457.0	499.8	536.3	580.8	607.7	644.2	665.2
11.5	169.3	203.8	240.1	316.7	345.8	379.7	405.6	445.1	480.4	524.2	553.5	594.8	623.1
12.0	148.3	179.3	209.3	278.8	307.2	336.2	358.3	394.0	427.2	468.6	497.9	540.1	572.5
12.5	127.7	155.6	181.6	242.2	268.3	296.0	315.2	345.9	376.2	414.9	442.9	483.7	517.0
13.0	110.2	133.9	157.1	208.5	231.3	256.8	275.4	303.4	330.1	364.3	391.2	429.4	461.9
13.5	96.8	116.3	136.0	178.3	197.5	220.3	236.7	263.4	288.9	318.8	342.6	377.5	408.4
14.0	86.3	103.2	119.5	154.7	169.8	188.1	202.9	226.5	250.2	278.9	300.4	330.6	358.2
14.5	78.0	91.1	105.8	135.5	148.0	162.4	173.7	193.8	214.8	240.6	261.2	289.7	313.7
15.0	70.5	81.6	93.2	119.7	130.4	142.8	152.0	167.4	184.1	206.9	225.1	251.6	274.9
15.5	63.6	73.6	83.0	105.7	115.2	126.0	133.9	146.8	159.8	177.3	193.0	216.5	237.4
16.0	57.4	66.4	74.8	92.7	101.5	111.0	118.1	129.4	140.6	154.8	166.8	185.7	204.5
16.5	51.8	60.0	67.5	82.5	89.0	97.5	103.9	114.0	123.8	136.1	146.1	161.1	175.6
17.0	46.8	54.1	61.0	74.3	79.7	85.9	90.9	100.2	109.0	119.9	128.6	141.5	153.4
17.5	42.3	48.9	55.0	67.1	71.9	77.2	81.2	87.9	95.6	105.5	113.2	124.5	134.7
18.0	38.0	44.1	49.7	60.6	64.9	69.7	73.2	78.8	84.5	92.3	99.5	109.5	118.6
18.5	34.9	39.8	44.8	54.7	58.6	62.9	66.1	71.1	76.1	82.2	87.4	96.1	104.3
19.0	32.4	36.1	40.5	49.4	52.9	56.8	59.6	64.2	68.7	74.1	78.5	84.9	91.4
19.5	30.1	33.4	36.6	44.5	47.7	51.3	53.8	57.9	62.0	66.9	70.8	76.4	81.5
20.0	27.9	31.0	33.8	40.2	43.1	46.3	48.6	52.3	55.9	60.4	63.9	69.0	73.5
20.5	26.0	28.8	31.4	36.4	38.8	41.8	43.8	47.2	50.5	54.5	57.7	62.3	66.3
21.0	24.1	26.8	29.2	33.7	35.4	37.6	39.5	42.6	45.6	49.2	52.1	56.2	59.9
21.5	22.4	24.9	27.1	31.3	32.8	34.6	35.9	38.4	41.2	44.4	47.0	50.7	54.1
22.0	20.8	23.1	25.2	29.0	30.5	32.1	33.3	35.1	37.1	40.1	42.5	45.8	48.8

22.5	19.3	21.5	23.4	27.0	28.3	29.8	30.9	32.6	34.2	36.3	38.2	41.4	44.0
23.0	18.0	20.0	21.7	25.1	26.3	27.7	28.7	30.3	31.8	33.6	35.0	37.3	39.7
23.5	16.7	18.5	20.2	23.3	24.5	25.8	26.7	28.1	29.5	31.2	32.5	34.3	36.1
24.0	15.5	17.2	18.8	21.6	22.7	23.9	24.8	26.1	27.4	29.0	30.2	31.9	33.4
24.5	14.4	16.0	17.4	20.1	21.1	22.2	23.0	24.3	25.5	26.9	28.0	29.6	31.0
25.0	13.4	14.9	16.2	18.7	19.6	20.7	21.4	22.6	23.7	25.0	26.0	27.5	28.8
25.5	12.4	13.8	15.0	17.3	18.2	19.2	19.9	21.0	22.0	23.2	24.2	25.6	26.8
26.0	11.5	12.8	14.0	16.1	16.9	17.8	18.5	19.5	20.4	21.6	22.5	23.8	24.9
26.5	10.8	11.9	13.0	15.0	15.7	16.6	17.2	18.1	19.0	20.0	20.9	22.1	23.1
27.0	10.2	11.0	12.1	13.9	14.6	15.4	15.9	16.8	17.6	18.6	19.4	20.5	21.5
27.5	9.8	10.4	11.2	12.9	13.6	14.3	14.8	15.6	16.4	17.3	18.0	19.0	19.9
28.0	9.3	10.0	10.5	12.0	12.6	13.3	13.7	14.5	15.2	16.1	16.7	17.7	18.5
28.5	8.9	9.5	10.0	11.1	11.7	12.3	12.8	13.5	14.1	14.9	15.6	16.4	17.2
29.0	8.5	9.1	9.6	10.5	10.9	11.5	11.9	12.5	13.1	13.9	14.5	15.3	16.0
29.5	8.2	8.7	9.2	10.0	10.3	10.7	11.0	11.6	12.2	12.9	13.4	14.2	14.8
30.0	7.8	8.3	8.8	9.6	9.9	10.2	10.4	10.8	11.3	12.0	12.5	13.2	13.8
30.5	7.5	8.0	8.4	9.1	9.4	9.7	9.9	10.3	10.6	11.1	11.6	12.2	12.8
31.0	7.1	7.6	8.0	8.7	9.0	9.3	9.5	9.8	10.1	10.5	10.8	11.4	11.9
31.5	6.8	7.3	7.7	8.4	8.6	8.9	9.1	9.4	9.7	10.0	10.3	10.6	11.0
32.0	6.5	6.9	7.3	8.0	8.2	8.5	8.7	9.0	9.2	9.6	9.8	10.1	10.4
32.5	6.2	6.6	7.0	7.6	7.9	8.1	8.3	8.6	8.8	9.1	9.4	9.7	10.0
33.0	5.9	6.3	6.7	7.3	7.5	7.8	7.9	8.2	8.4	8.7	9.0	9.3	9.5
33.5	5.7	6.1	6.4	7.0	7.2	7.4	7.6	7.8	8.1	8.3	8.6	8.9	9.1
34.0	5.4	5.8	6.1	6.7	6.9	7.1	7.2	7.5	7.7	8.0	8.2	8.5	8.7
34.5	5.2	5.5	5.8	6.4	6.6	6.8	6.9	7.2	7.4	7.6	7.8	8.1	8.3
35.0	5.0	5.3	5.6	6.1	6.3	6.5	6.6	6.8	7.0	7.3	7.5	7.7	7.9
35.5	4.7	5.1	5.3	5.8	6.0	6.2	6.3	6.5	6.7	7.0	7.1	7.4	7.6
36.0	4.5	4.8	5.1	5.6	5.7	5.9	6.0	6.2	6.4	6.7	6.8	7.1	7.3
36.5	4.3	4.6	4.9	5.3	5.5	5.7	5.8	6.0	6.2	6.4	6.5	6.7	6.9
37.0	4.1	4.4	4.7	5.1	5.2	5.4	5.5	5.7	5.9	6.1	6.2	6.5	6.6
37.5	4.0	4.2	4.5	4.9	5.0	5.2	5.3	5.5	5.6	5.8	6.0	6.2	6.3
38.0	3.8	4.0	4.3	4.6	4.8	4.9	5.0	5.2	5.4	5.6	5.7	5.9	6.1
38.5	3.6	3.9	4.1	4.4	4.6	4.7	4.8	5.0	5.1	5.3	5.4	5.6	5.8
39.0	3.5	3.7	3.9	4.2	4.4	4.5	4.6	4.8	4.9	5.1	5.2	5.4	5.5
39.5	3.3	3.5	3.7	4.1	4.2	4.3	4.4	4.6	4.7	4.9	5.0	5.1	5.3
40.0	3.2	3.4	3.6	3.9	4.0	4.1	4.2	4.4	4.5	4.6	4.8	4.9	5.1
40.5	3.0	3.2	3.4	3.7	3.8	3.9	4.0	4.2	4.3	4.4	4.5	4.7	4.8
41.0	2.9	3.1	3.2	3.5	3.7	3.8	3.9	4.0	4.1	4.2	4.3	4.5	4.6
41.5	2.8	2.9	3.1	3.4	3.5	3.6	3.7	3.8	3.9	4.0	4.2	4.3	4.4
42.0	2.6	2.8	3.0	3.2	3.3	3.4	3.5	3.6	3.7	3.9	4.0	4.1	4.2
42.5	2.5	2.7	2.8	3.1	3.2	3.3	3.4	3.5	3.6	3.7	3.8	3.9	4.0
43.0	2.4	2.6	2.7	3.0	3.0	3.1	3.2	3.3	3.4	3.5	3.6	3.8	3.9
43.5	2.3	2.5	2.6	2.8	2.9	3.0	3.1	3.2	3.3	3.4	3.5	3.6	3.7
44.0	2.2	2.3	2.5	2.7	2.8	2.9	2.9	3.0	3.1	3.2	3.3	3.4	3.5
44.5	2.1	2.2	2.4	2.6	2.7	2.7	2.8	2.9	3.0	3.1	3.2	3.3	3.4
45.0	2.0	2.1	2.3	2.5	2.5	2.6	2.7	2.8	2.9	3.0	3.0	3.1	3.2
45.5	1.9	2.0	2.2	2.4	2.4	2.5	2.6	2.6	2.7	2.8	2.9	3.0	3.1
46.0	1.8	2.0	2.1	2.3	2.3	2.4	2.5	2.5	2.6	2.7	2.8	2.9	2.9
46.5	1.8	1.9	2.0	2.2	2.2	2.3	2.3	2.4	2.5	2.6	2.6	2.7	2.8
47.0	1.7	1.8	1.9	2.1	2.1	2.2	2.2	2.3	2.4	2.5	2.5	2.6	2.7
47.5	1.6	1.7	1.8	2.0	2.0	2.1	2.1	2.2	2.3	2.4	2.4	2.5	2.6
48.0	1.5	1.6	1.7	1.9	1.9	2.0	2.0	2.1	2.2	2.3	2.3	2.4	2.5
Flood Volume (Mm^3)	17.18	19.37	21.03	24.93	26.30	28.21	29.30	30.60	30.93	31.62	31.54	32.16	32.22

1:20 year Hydrographs for various Storm Durations (m^3/s)													
	Duration (h)												
Time (h)	**8**	**10**	**12**	**16**	**18**	**20**	**22**	**24**	**26**	**28**	**30**	**32**	**34**
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2.0	15.1	5.9	6.0	5.8	0.9	2.4	1.8	1.1	0.6	0.2	0.0	0.0	0.0
2.5	67.3	38.1	32.2	26.6	11.5	15.4	13.3	10.0	6.2	3.1	1.3	0.6	0.1
3.0	341.7	175.9	171.5	154.1	69.0	90.7	79.1	53.6	36.1	21.4	12.0	7.7	4.0
3.5	1084.7	784.8	605.7	446.1	234.3	283.8	253.3	243.9	160.5	90.6	48.8	32.7	18.4
4.0	1522.3	1529.6	1274.3	951.0	911.0	863.7	829.1	697.2	518.6	361.7	243.1	167.6	100.6
4.5	1263.3	1645.6	1697.1	1497.9	1488.6	1405.2	1373.7	1222.2	988.3	768.3	580.2	440.9	314.2
5.0	879.6	1209.5	1508.2	1769.5	1749.0	1736.2	1711.6	1626.2	1417.8	1193.7	979.6	806.1	640.5
5.5	618.4	825.8	1102.4	1627.3	1822.9	1903.9	1899.2	1833.7	1714.6	1536.8	1338.2	1161.0	979.8
6.0	484.1	612.1	793.1	1209.5	1406.1	1569.0	1662.1	1758.4	1779.5	1701.8	1568.3	1430.1	1268.7
6.5	492.9	537.6	631.6	900.1	1013.9	1179.3	1305.0	1509.1	1668.9	1709.7	1664.8	1593.2	1475.0
7.0	528.5	573.4	620.0	764.1	841.4	955.0	1043.1	1195.1	1412.6	1555.0	1605.0	1616.1	1561.3
7.5	529.6	588.4	639.6	739.8	779.2	852.6	911.8	1004.7	1163.4	1332.9	1458.6	1548.2	1561.6
8.0	507.1	573.2	640.0	755.8	795.6	853.0	894.3	941.7	1023.4	1128.6	1255.9	1397.9	1472.6
8.5	469.8	537.0	610.5	743.9	796.2	856.9	895.9	923.4	961.3	1007.4	1079.2	1221.3	1344.0
9.0	427.6	491.3	565.6	705.5	766.1	836.5	882.5	921.1	948.0	963.6	990.1	1076.6	1181.1
9.5	385.3	443.1	513.7	649.9	709.5	783.2	832.7	887.2	928.0	941.5	950.2	989.1	1048.8
10.0	346.7	397.4	462.3	589.4	643.8	715.3	764.0	826.9	883.2	913.3	927.5	952.8	980.9
10.5	309.6	354.5	412.3	528.4	578.4	640.8	687.5	751.3	814.3	858.1	889.0	917.6	939.0
11.0	272.7	314.7	365.0	470.3	515.9	571.4	610.8	672.5	735.6	785.9	830.0	871.2	900.4
11.5	238.6	276.6	322.4	415.3	456.4	506.5	541.1	594.3	654.7	705.6	755.3	804.6	845.8
12.0	206.2	240.5	283.6	363.5	401.8	446.8	478.0	525.5	577.6	625.6	676.4	727.8	776.7
12.5	175.8	205.6	244.8	317.3	349.5	390.0	418.0	461.6	508.9	551.3	597.1	647.8	698.5
13.0	151.9	174.4	208.7	276.3	303.5	337.5	362.6	402.3	445.7	485.0	526.6	571.1	619.3
13.5	133.5	151.0	177.2	236.6	262.5	292.3	312.6	347.4	387.0	423.0	461.7	502.3	545.3
14.0	118.6	133.8	154.8	202.2	224.9	253.0	271.8	300.4	334.2	367.0	402.4	439.7	479.3
14.5	104.8	118.5	136.8	173.5	192.5	217.0	234.1	261.3	289.4	316.6	348.2	382.1	418.2
15.0	91.8	104.5	120.8	152.5	166.7	186.6	201.6	225.5	251.8	275.8	301.8	330.6	363.5
15.5	81.8	91.4	106.3	134.5	146.9	162.5	173.7	194.1	216.8	238.8	263.2	287.3	314.3
16.0	73.8	81.5	93.1	118.3	129.4	143.0	152.3	168.0	186.5	205.6	227.3	250.0	274.4
16.5	66.6	73.5	82.7	103.9	113.6	125.7	133.9	147.5	162.0	176.7	195.5	215.2	237.6
17.0	60.1	66.3	74.5	90.9	99.6	110.2	117.6	129.6	142.4	154.5	168.8	184.8	204.6
17.5	54.3	59.9	67.3	81.1	87.5	96.6	103.2	113.7	125.1	135.9	148.2	160.6	175.7
18.0	49.0	54.1	60.7	73.1	78.5	85.2	90.3	99.8	109.7	119.4	130.3	141.2	153.6
18.5	44.2	48.8	54.8	66.0	70.8	76.7	80.7	87.6	96.1	104.8	114.4	124.1	135.1
19.0	39.9	44.0	49.5	59.6	63.9	69.2	72.8	78.6	84.9	91.6	100.4	108.9	118.8
19.5	36.2	39.7	44.6	53.8	57.7	62.5	65.7	70.9	76.4	81.6	88.0	95.4	104.3
20.0	33.5	36.0	40.3	48.5	52.1	56.4	59.3	64.0	68.9	73.6	78.9	84.4	91.2
20.5	31.1	33.4	36.5	43.8	47.0	50.9	53.5	57.8	62.2	66.4	71.2	76.0	81.3
21.0	28.9	31.0	33.7	39.5	42.5	46.0	48.3	52.1	56.2	60.0	64.3	68.6	73.4
21.5	26.8	28.8	31.3	35.9	38.2	41.5	43.6	47.1	50.7	54.1	58.0	61.9	66.2
22.0	24.9	26.8	29.1	33.3	35.0	37.4	39.3	42.5	45.8	48.9	52.4	55.9	59.8
22.5	23.2	24.9	27.0	30.9	32.5	34.4	35.8	38.3	41.4	44.1	47.3	50.4	53.9
23.0	21.5	23.1	25.1	28.7	30.2	32.0	33.1	35.0	37.3	39.8	42.7	45.5	48.7
23.5	20.0	21.5	23.3	26.7	28.0	29.7	30.8	32.5	34.3	36.1	38.4	41.1	43.9
24.0	18.6	19.9	21.7	24.8	26.1	27.6	28.6	30.2	31.9	33.4	35.2	37.1	39.6
24.5	17.2	18.5	20.1	23.0	24.2	25.6	26.6	28.1	29.6	31.0	32.6	34.2	36.0
25.0	16.0	17.2	18.7	21.4	22.5	23.8	24.7	26.1	27.5	28.8	30.3	31.7	33.3
25.5	14.9	16.0	17.4	19.9	20.9	22.1	22.9	24.2	25.6	26.8	28.2	29.5	31.0

26.0	13.8	14.8	16.1	18.5	19.4	20.5	21.3	22.5	23.7	24.9	26.2	27.4	28.8
26.5	12.9	13.8	15.0	17.1	18.0	19.1	19.8	20.9	22.1	23.1	24.3	25.5	26.7
27.0	11.9	12.8	13.9	15.9	16.8	17.7	18.4	19.4	20.5	21.5	22.6	23.6	24.8
27.5	11.1	11.9	12.9	14.8	15.6	16.5	17.1	18.0	19.0	20.0	21.0	22.0	23.1
28.0	10.5	11.0	12.0	13.7	14.5	15.3	15.9	16.8	17.7	18.5	19.5	20.4	21.4
28.5	10.0	10.4	11.1	12.8	13.4	14.2	14.7	15.6	16.4	17.2	18.1	19.0	19.9
29.0	9.5	10.0	10.5	11.9	12.5	13.2	13.7	14.5	15.3	16.0	16.8	17.6	18.5
29.5	9.1	9.5	10.0	11.0	11.6	12.3	12.7	13.4	14.2	14.9	15.6	16.4	17.2
30.0	8.7	9.1	9.6	10.4	10.8	11.4	11.8	12.5	13.2	13.8	14.5	15.2	16.0
30.5	8.3	8.7	9.2	9.9	10.3	10.7	11.0	11.6	12.2	12.8	13.5	14.1	14.8
31.0	8.0	8.3	8.8	9.5	9.8	10.2	10.4	10.8	11.4	11.9	12.5	13.1	13.8
31.5	7.6	7.9	8.4	9.1	9.4	9.7	9.9	10.3	10.6	11.0	11.6	12.2	12.8
32.0	7.3	7.6	8.0	8.7	9.0	9.3	9.5	9.8	10.1	10.4	10.8	11.3	11.9
32.5	7.0	7.3	7.6	8.3	8.6	8.9	9.1	9.4	9.7	10.0	10.3	10.6	11.0
33.0	6.6	6.9	7.3	7.9	8.2	8.5	8.7	9.0	9.3	9.5	9.8	10.1	10.4
33.5	6.4	6.6	7.0	7.6	7.8	8.1	8.3	8.6	8.9	9.1	9.4	9.7	10.0
34.0	6.1	6.3	6.7	7.2	7.5	7.7	7.9	8.2	8.5	8.7	9.0	9.2	9.5
34.5	5.8	6.1	6.4	6.9	7.1	7.4	7.6	7.8	8.1	8.3	8.6	8.8	9.1
35.0	5.5	5.8	6.1	6.6	6.8	7.1	7.2	7.5	7.7	8.0	8.2	8.4	8.7
35.5	5.3	5.5	5.8	6.3	6.5	6.8	6.9	7.1	7.4	7.6	7.8	8.1	8.3
36.0	5.1	5.3	5.6	6.0	6.2	6.5	6.6	6.8	7.1	7.3	7.5	7.7	7.9
36.5	4.8	5.1	5.3	5.8	6.0	6.2	6.3	6.5	6.7	6.9	7.2	7.4	7.6
37.0	4.6	4.8	5.1	5.5	5.7	5.9	6.0	6.2	6.4	6.6	6.8	7.0	7.3
37.5	4.4	4.6	4.9	5.3	5.4	5.6	5.8	6.0	6.2	6.3	6.5	6.7	6.9
38.0	4.2	4.4	4.7	5.0	5.2	5.4	5.5	5.7	5.9	6.1	6.3	6.4	6.6
38.5	4.0	4.2	4.4	4.8	5.0	5.2	5.3	5.4	5.6	5.8	6.0	6.1	6.3
39.0	3.9	4.0	4.2	4.6	4.8	4.9	5.0	5.2	5.4	5.5	5.7	5.9	6.1
39.5	3.7	3.9	4.1	4.4	4.5	4.7	4.8	5.0	5.1	5.3	5.5	5.6	5.8
40.0	3.5	3.7	3.9	4.2	4.3	4.5	4.6	4.8	4.9	5.1	5.2	5.4	5.5
40.5	3.4	3.5	3.7	4.0	4.2	4.3	4.4	4.5	4.7	4.8	5.0	5.1	5.3
41.0	3.2	3.4	3.5	3.8	4.0	4.1	4.2	4.3	4.5	4.6	4.8	4.9	5.1
41.5	3.1	3.2	3.4	3.7	3.8	3.9	4.0	4.2	4.3	4.4	4.6	4.7	4.8
42.0	2.9	3.1	3.2	3.5	3.6	3.8	3.8	4.0	4.1	4.2	4.4	4.5	4.6
42.5	2.8	2.9	3.1	3.4	3.5	3.6	3.7	3.8	3.9	4.0	4.2	4.3	4.4
43.0	2.7	2.8	3.0	3.2	3.3	3.4	3.5	3.6	3.7	3.9	4.0	4.1	4.2
43.5	2.6	2.7	2.8	3.1	3.2	3.3	3.4	3.5	3.6	3.7	3.8	3.9	4.0
44.0	2.5	2.6	2.7	2.9	3.0	3.1	3.2	3.3	3.4	3.5	3.6	3.7	3.9
44.5	2.4	2.5	2.6	2.8	2.9	3.0	3.1	3.2	3.3	3.4	3.5	3.6	3.7
45.0	2.2	2.3	2.5	2.7	2.8	2.9	2.9	3.0	3.1	3.2	3.3	3.4	3.5
45.5	2.1	2.2	2.4	2.6	2.6	2.7	2.8	2.9	3.0	3.1	3.2	3.3	3.4
46.0	2.1	2.1	2.3	2.4	2.5	2.6	2.7	2.8	2.9	2.9	3.0	3.1	3.2
46.5	2.0	2.0	2.2	2.3	2.4	2.5	2.6	2.6	2.7	2.8	2.9	3.0	3.1
47.0	1.9	2.0	2.1	2.2	2.3	2.4	2.4	2.5	2.6	2.7	2.8	2.9	2.9
47.5	1.8	1.9	2.0	2.1	2.2	2.3	2.3	2.4	2.5	2.6	2.7	2.7	2.8
48.0	1.7	1.8	1.9	2.0	2.1	2.2	2.2	2.3	2.4	2.5	2.5	2.6	2.7
Flood Volume (Mm^3)	23.02	25.48	27.96	32.64	34.31	36.78	38.26	39.82	40.85	41.16	41.24	41.83	42.04

1:50 year Hydrographs for various storm durations (m^3/s)													
Time	Duration (h)												
(h)	8	10	12	16	18	20	22	24	26	28	30	32	34
0.0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.5	0	0	0	0	0	0	0	0	0	0	0	0	0
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0
2.0	22	9	9	9	3	4	3	2	1	0	0	0	0
2.5	97	55	45	38	20	24	21	16	9	5	2	1	0
3.0	500	261	242	220	116	139	125	84	52	33	18	12	7
3.5	1548	1085	829	630	372	424	387	356	233	144	74	49	32
4.0	2184	2061	1697	1295	1250	1178	1135	952	712	514	352	247	165
4.5	1846	2233	2246	1997	1990	1871	1832	1625	1320	1042	803	614	460
5.0	1292	1658	2026	2350	2329	2297	2266	2141	1868	1583	1318	1084	882
5.5	906	1134	1506	2178	2429	2516	2510	2410	2247	2018	1776	1536	1316
6.0	716	846	1091	1632	1896	2109	2223	2327	2336	2231	2070	1877	1681
6.5	725	760	888	1229	1391	1616	1767	2029	2212	2249	2201	2085	1944
7.0	768	811	875	1062	1172	1323	1427	1646	1911	2070	2141	2122	2060
7.5	764	836	907	1031	1084	1177	1242	1384	1593	1813	1979	2052	2072
8.0	727	819	919	1065	1115	1182	1223	1287	1392	1539	1738	1885	1979
8.5	670	767	882	1057	1123	1190	1227	1259	1308	1369	1501	1670	1828
9.0	607	699	818	1003	1084	1162	1209	1252	1286	1308	1372	1470	1620
9.5	545	627	740	921	1002	1085	1139	1204	1257	1274	1305	1349	1440
10.0	487	562	662	830	907	983	1039	1116	1193	1234	1266	1291	1338
10.5	430	499	586	738	810	878	928	1007	1094	1156	1208	1239	1273
11.0	377	440	519	651	716	778	823	895	980	1053	1122	1173	1215
11.5	330	385	456	569	627	683	723	789	867	938	1013	1078	1137
12.0	288	335	399	498	547	594	631	691	763	829	900	968	1039
12.5	246	290	344	433	476	515	546	599	665	727	793	857	927
13.0	208	248	296	374	411	446	472	517	574	632	693	753	820
13.5	179	212	255	320	353	383	406	446	495	544	600	656	718
14.0	157	183	221	276	302	329	349	384	428	470	517	566	624
14.5	139	160	191	240	263	283	300	330	368	405	447	489	537
15.0	122	141	166	208	228	247	261	285	316	349	386	423	465
15.5	107	124	146	179	197	214	227	248	274	300	332	365	402
16.0	94	109	128	156	170	184	196	215	239	262	287	314	347
16.5	83	95	112	137	149	160	169	185	206	227	250	273	299
17.0	75	84	98	120	131	140	147	160	177	195	216	237	261
17.5	68	76	86	105	115	123	129	140	154	168	185	204	225
18.0	61	68	78	92	100	108	113	123	135	147	161	175	194
18.5	55	62	70	82	88	94	99	108	119	129	141	153	167
19.0	50	56	63	74	79	84	87	95	104	113	124	134	146
19.5	45	50	57	67	71	75	78	84	91	100	108	118	129
20.0	41	45	52	60	64	68	71	75	81	87	95	103	113
20.5	37	41	47	54	58	61	64	68	73	78	84	90	99
21.0	34	37	42	49	52	55	58	61	66	71	76	81	87
21.5	31	34	38	44	47	50	52	55	60	64	68	73	78
22.0	29	32	35	40	43	45	47	50	54	58	62	66	70
22.5	27	29	32	36	38	41	42	45	49	52	56	59	64
23.0	25	27	30	34	35	37	38	41	44	47	50	54	57
23.5	23	25	28	31	33	34	35	37	40	42	45	48	52
24.0	22	24	26	29	30	32	32	34	36	38	41	44	47

1:50 year Hydrographs for various storm durations (m^3/s)													
24.5	20	22	24	27	28	29	30	32	33	35	37	39	42
25.0	19	20	22	25	26	27	28	29	31	32	34	36	38
25.5	17	19	21	23	24	25	26	27	29	30	32	33	35
26.0	16	18	19	22	23	23	24	25	27	28	29	31	32
26.5	15	16	18	20	21	22	22	24	25	26	27	29	30
27.0	14	15	17	19	19	20	21	22	23	24	25	27	28
27.5	13	14	15	17	18	19	19	20	21	22	24	25	26
28.0	12	13	14	16	17	18	18	19	20	21	22	23	24
28.5	11	12	13	15	16	16	17	18	18	19	20	21	22
29.0	11	11	12	14	15	15	16	16	17	18	19	20	21
29.5	10	11	11	13	13	14	14	15	16	17	18	18	19
30.0	10	10	11	12	13	13	13	14	15	16	16	17	18
30.5	9	10	10	11	12	12	12	13	14	14	15	16	17
31.0	9	9	10	10	11	11	12	12	13	13	14	15	15
31.5	8	9	9	10	10	11	11	11	12	12	13	14	14
32.0	8	8	9	10	10	10	10	11	11	12	12	13	13
32.5	8	8	9	9	9	10	10	10	10	11	11	12	12
33.0	7	8	8	9	9	9	9	10	10	10	11	11	12
33.5	7	7	8	8	9	9	9	9	10	10	10	10	11
34.0	7	7	7	8	8	8	9	9	9	9	10	10	10
34.5	6	7	7	8	8	8	8	8	9	9	9	9	10
35.0	6	6	7	7	7	8	8	8	8	9	9	9	9
35.5	6	6	6	7	7	7	7	8	8	8	8	9	9
36.0	6	6	6	7	7	7	7	7	8	8	8	8	9
36.5	5	6	6	6	7	7	7	7	7	7	8	8	8
37.0	5	5	6	6	6	6	7	7	7	7	7	8	8
37.5	5	5	5	6	6	6	6	6	7	7	7	7	7
38.0	5	5	5	6	6	6	6	6	6	7	7	7	7
38.5	4	5	5	5	5	6	6	6	6	6	6	7	7
39.0	4	4	5	5	5	5	5	6	6	6	6	6	7
39.5	4	4	5	5	5	5	5	5	6	6	6	6	6
40.0	4	4	4	5	5	5	5	5	5	5	6	6	6
40.5	4	4	4	4	5	5	5	5	5	5	5	6	6
41.0	4	4	4	4	4	4	5	5	5	5	5	5	5
41.5	3	4	4	4	4	4	4	4	5	5	5	5	5
42.0	3	3	4	4	4	4	4	4	4	5	5	5	5
42.5	3	3	3	4	4	4	4	4	4	4	4	5	5
43.0	3	3	3	4	4	4	4	4	4	4	4	4	5
43.5	3	3	3	3	3	4	4	4	4	4	4	4	4
44.0	3	3	3	3	3	3	3	4	4	4	4	4	4
44.5	3	3	3	3	3	3	3	3	4	4	4	4	4
45.0	2	3	3	3	3	3	3	3	3	3	4	4	4
45.5	2	2	3	3	3	3	3	3	3	3	3	4	4
46.0	2	2	3	3	3	3	3	3	3	3	3	3	3
46.5	2	2	2	3	3	3	3	3	3	3	3	3	3
47.0	2	2	2	2	3	3	3	3	3	3	3	3	3
47.5	2	2	2	2	2	2	3	3	3	3	3	3	3
48.0	2	2	2	2	2	2	2	2	3	3	3	3	3
Flood Volume (Mm3)	32.6	35.1	38.4	44.4	46.7	49.5	51.1	52.8	54.0	54.4	54.9	55.3	55.8

APPENDIX A.2 : Sedimentation

Groot Letaba River Water Resources Development Project
Bridging Studies
Reservoir Sedimentation of the proposed Nwamitwa Dam

1. INTRODUCTION

This study investigated the reservoir sedimentation rates expected at a proposed Nwamitwa Dam to be built on the Groot Letaba River, just downstream of the confluence with the Nwanedzi River.

2. METHODOLOGY

The following methodology was followed in this study to assess the sediment yield of the proposed Nwamitwa Dam:

a) Review of findings of previous studies.

b) Analysis of sediment yields of existing dams on the Groot Letaba and other rivers in the region.

c) Analysis of sediment yields based on suspended sediment data observed on the Groot Letaba River.

3. SEDIMENT YIELD DETERMINATION

3.1 Previous studies

Rooseboom (1990) in the Letaba Basin Study proposed the following maximum sediment yields:

Nwanedzi River	:	320 t/km^2.year (220 km^2)
Thabina River	:	350 t/km^2.year (150 km^2)
Letsitele River	:	360 t/km^2.year (170 km^2)

The proposed sediment yields were based on observed sedimentation rates of existing reservoirs in the region.

In the Letaba Water Resource Development: Pre-Feasibility Study of 1994, Rooseboom reviewed his 1990 sediment yields, based on a regional method developed for the SA Water Research Commission (Rooseboom, 1992). Based on this method the predicted average sediment yield for the Nwamitwa Dam site was 280 t/km^2.a, for a 1 352 km^2 effective catchment area (measured downstream of Tzaneen Dam) and reservoir storage capacities that ranged from 58.7 to 192 million m^3. This sediment yield estimation was based on observed sedimentation rates of existing reservoirs in the region.

3.2 Sediment yields of existing dams

The latest reservoir basin survey data were obtained from DWAF for this study. Observed sediment yield data of dams (Figure 3.2-1) near the proposed dam site are shown in Table 3.2-1.

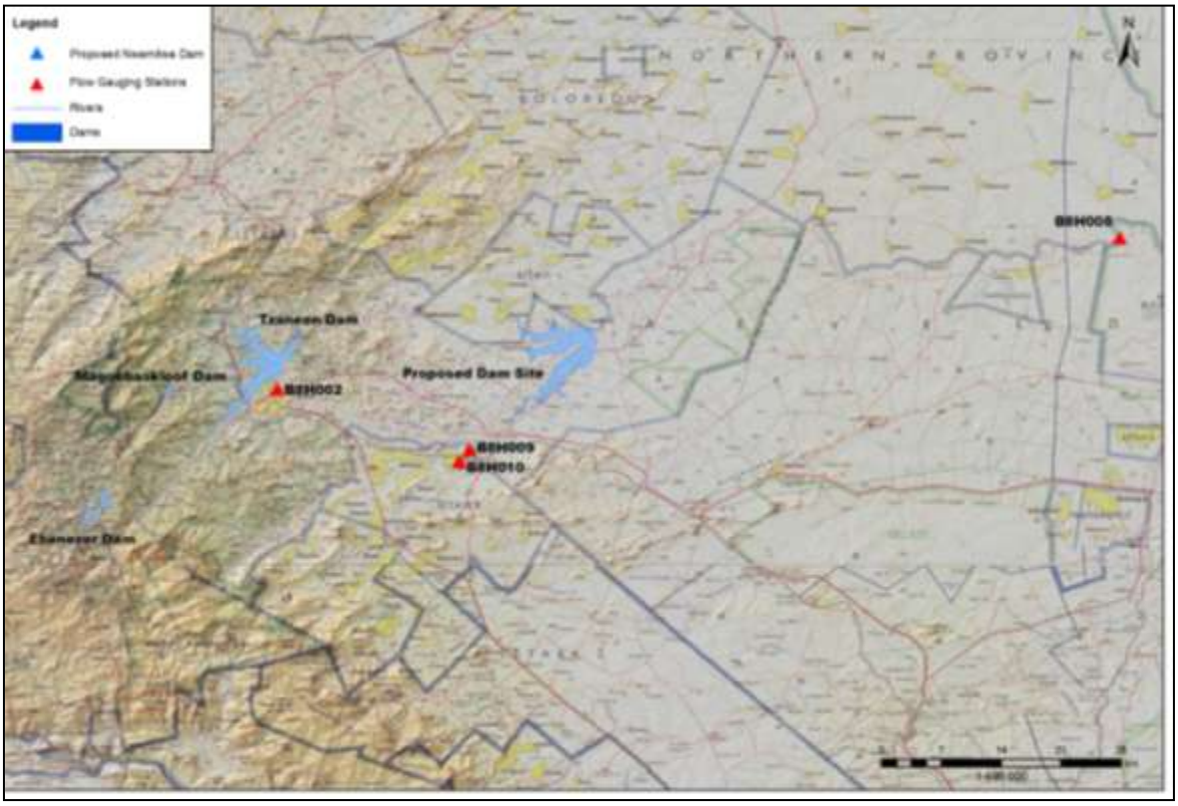

Figure 3.2-1 Dams and gauging stations located in the region of Nwamitwa dam site

Table 3.2-1 Observed sediment yields based on reservoir surveys

Dam	River	Effective catchment area (km^2)	First survey	Last survey	Sediment yield (t/km^2.a)*
Ebenezer	Groot Letaba	156	1959	1986	155
Magoebaskloof	Politsi	64	1970	2000	93**
Dap Naude	Broederstroom	14	1961	1987	357***
Tzaneen	Groot Letaba	419	1976	1990	285
Massingir	Olifants	41480	-	-	245****
Middel Letaba	Middel Letaba	1799	1986	2001	293

Notes:
* A 100 % sediment trapping efficiency was assumed in the reservoirs.
** The sediment yield of Magoebaskloof Dam is not reliable due to the small storage capacity – mean annual runoff ratio at the dam of only 0.13, which makes it difficult to estimate the sediment trapping efficiency of the reservoir.
*** The Dap Naude Dam sediment yield was found to be the highest, but the dam has a very small effective catchment area of only 14 km^2. In larger catchments the sediment delivery ratio is usually reduced due to more sediment deposition occurring.
**** Massingir Dam in Mozambique was included since it is located downstream the proposed Nwamitwa Dam site. Basson (2002) determined the sediment yield of Massingir Dam based on suspended sediment data and reservoir basin surveys. The catchment area of Massingir Dam is very large compared to the 1352 km^2 of Nwamitwa Dam, and covers a large catchment area to the south of the Nwamitwa Dam site.

From Table 3.2-1 the data of Tzaneen Dam, Middel Letaba Dam and Massingir Dam are probably most applicable to the proposed Nwamitwa Dam. Ebenezer Dam has a relatively small catchment area and is located upstream of Tzaneen Dam. The latter dam has a much higher sediment yield than Ebenezer Dam.

3.3 Sediment yield based on suspended sediment data

Suspended sediment grab samples are taken at some DWAF flow gauging stations in South Africa. Data were obtained at the gauging stations listed in Table 3.3-1.

Table 3.3-1 Suspended sediment data at flow gauging stations

Station	Location	Total catchment Area (km^2)	Sampling period	Max Q (m^3/s)	Max concentration (mg/l)
B8H008	Letaba Ranch on Groot Letaba	4710	1981-1982; 1998-1999	149	2072
B8H009	Junction on Groot Letaba	851	1981; 1999	55	123
B8H010	Letsitele River	477	1981-1982; 1998	9	2172

Figure 3.3-1 shows the data of these three stations. From Table 3.3-1 it is clear that data were only obtained for relatively short periods in the past, and that the data sets are very small. Only the data of B8H008 could be used since it had a relatively large recorded discharge in the sediment load-discharge relationship. The sediment load-discharge relationship was integrated with the observed flow record of B8H008 to obtain a sediment yield for the period 1966 to 2002. The sediment load-discharge relationship represents a “high probable” curve in order to obtain a conservatively high sediment yield. Figure 3.3-2 shows the recorded flow data. The station’s discharge table limit is about 1000 m^3/s and reached this limit 5 times during the 36 year historical period. Actual sediment loads predicted with this method could therefore be higher. The sediment yield calculated at B8H008 is 278 $t.km^2.a$, and takes into account bedload and non-uniformity in suspended sediment transport which was added by adjusting the suspended sediment concentration data by a factor of 1.25. The sediment yield obtained by this method is in agreement with the data obtained with reservoir basin surveys, but it is based on very limited suspended sediment data, obtained at relatively small flows and floods.

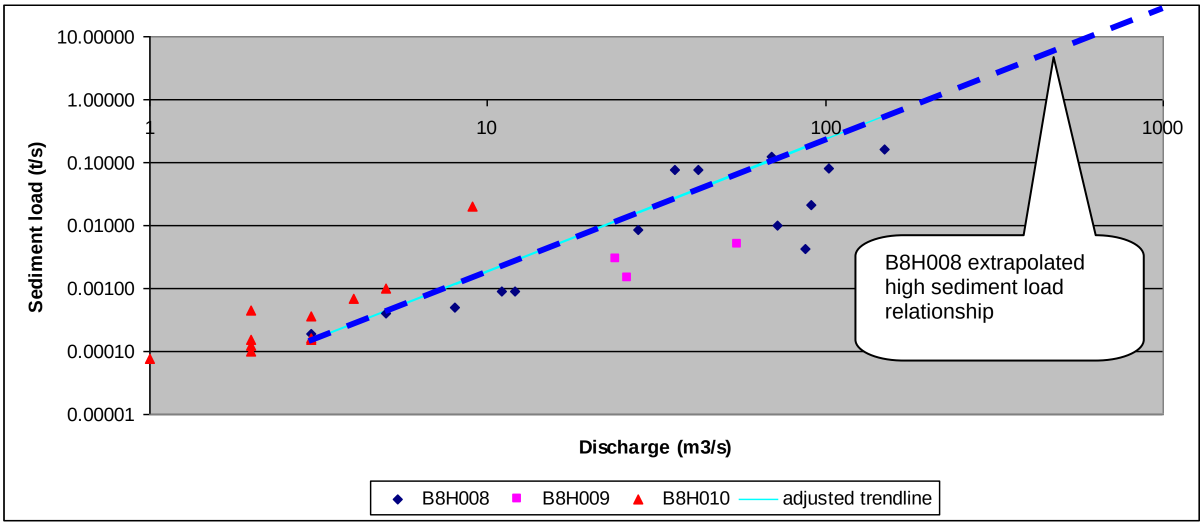

Figure 3.3-1 Sediment load-discharge relationships

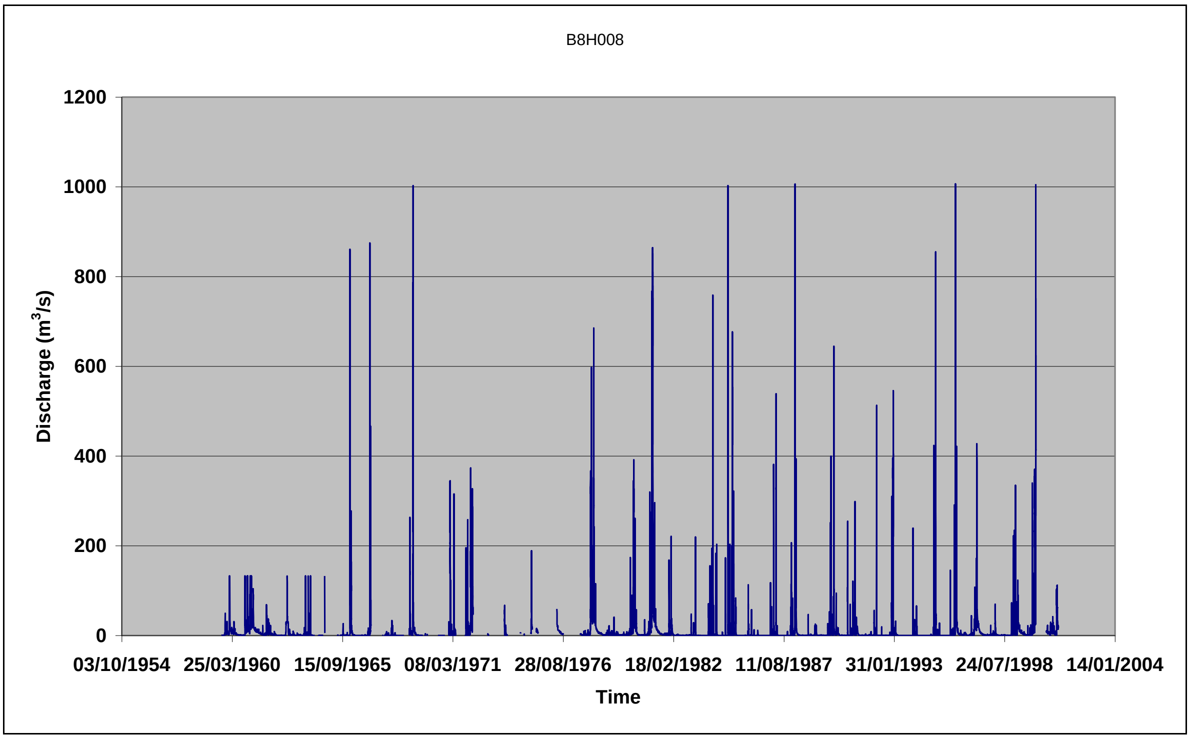

Figure 3.3-2 Recorded flow data (instantaneous) at B8H008

3.4 Proposed sediment yield

The methods described above yielded the following sediment yields:

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

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

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

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

The future land use could affect the sediment yield. The current land use consists mainly of forestry, irrigated commercial farming, urban areas and subsistence farming (Figure 3.2-1). The catchment area of the Nwamitwa Dam falls in the high and medium soil erosivity regions of the Rooseboom (1992) method. If due to future land degradation the medium region changes to high erosivity, the maximum possible sediment yield would be 350 t/km^2.a based on a 95 percentile assurance. Possible maximum sediment yield values in the order of 350 t/km^2.a were also proposed in the 1990 study by Rooseboom (see Section 3.1).

Due to possible future land degradation and the effect of climate change, it is recommended that a sediment yield of 350 $t/km^2.a$ is used for the design of Nwamitwa Dam.

4. ESTIMATED RESERVOIR SEDIMENT DEPOSITION IN NWAMITWA RESERVOIR

Based on the Brune (1953) sediment trapping efficiency relationship, it was assumed the proposed reservoir would trap 100 % of the incoming sediment load. The sediment density of deposited sediment was assumed to be 1.35 t/m^3 after a 50 year period. An effective catchment area of 1352 km^2 was used for Nwamitwa Dam. Table 4-1 shows the sediment volumes expected as deposited sediment in Nwamitwa Reservoir.

Table 4-1 Estimated Nwamitwa Reservoir sedimentation

Sediment yield ($t/km^2.a$)	Effective catchment area (km^2)	Estimated sediment volumes (m^3)		
		After 10 years	After 20 years	After 50 years
350	1352*	6.92	11.49	17.53

Note: * From the Rooseboom (1994) study

5. REFERENCES

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Basson, G.R. (2002). Sedimentation Study: *Corumana, Macaretane, Massingir and Pequenos Libombos Reservoirs*. Republic of Mozambique. First National Water Development Project.

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APPENDIX A.3 : Backwater Analysis

1. BACKWATER ANALYSIS

1.1 Introduction

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

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

1.2 Analysis

The 1:100 year flood line baseline information, which is needed for developing the expropriation line, was determined through backwater analysis of the system of rivers flowing into Nwamitwa Dam with the aid of an unsteady HEC-RAS (version 4.0) model. The HEC-RAS model computes water surface profiles based on river geometry and structures crossing the channel.

Input to the model primarily includes cross-section profiles, Mannings 'n' coefficients, expansion and contraction coefficients, ineffective flow area, geometry of the hydraulic structures crossing the channel and flow data.

The cross-sectional profiles were extracted with the aid of HEC-GeoRAS v3.1 extension for ArcView v3.2 (USACE, 2002) using as input, the TIN (triangular irregular network) topographic representation of the ground surface developed from the 2 m contour basin survey. The Manning's 'n' values which relate to surface friction caused by absolute roughness were estimated based on the nature of the channel and floodplain surface observed through the inspection of the imagery. In many areas, the channels consisted of either cobbles with small boulders or plain sand and gravel. The floodplains were vegetated with light to medium brush in many places and dense trees in some places. Chow (1964), which has pictorial illustrations of calibrated channel and floodplain Manning's 'n' values was used to estimate the Manning's 'n' applicable to this study and a value of 0.04 was assumed for both the channel and the floodplain.

Expansion and contraction coefficients were determined according to the ratio of effective flow area occurring at stream cross-sections and road crossings. Table 1 lists typical coefficients used in this study.

Ineffective flow areas were determined using cross-section profiles and contour information. Examples of ineffective flow area include:

- Hydraulically blocked floodplain areas due to obstruction or irregularities in the floodplain;
- Hydraulically blocked road encroachment due to contraction and expansion of flow through bridges or culverts; and
- Floodplain areas significantly below the top of bank not hydraulically connected to the channel downstream.

Table 1 Expansion and Contraction Coefficients (from HEC-RAS Hydraulic Manual, USACE (2008))

TRANSITION TYPE	EXPANSION COEFFICIENT	CONTRACTION COEFFICIENT
Gradual	0.3	0.1
Road Crossing	0.5	0.3
Abrupt	0.8	0.6

At minimum, an unsteady HEC-RAS model requires an inflow hydrograph to be established at the top of each reach as shown in Figure 1. The inflow hydrographs for the reaches modelled are shown in Figure 2. These hydrographs were derived from the catchment hydrology outlined in the feasibility study report entitled “The Groot Letaba Water Resource Development” Volume 8 compiled by BKS and Consultburo in 1998.

The delineation of the expropriation line was based on the pre- and post-dam construction 1:100 year floodlines. The first criterion for determining the expropriation line as per (DWAF, 2001) guidelines requires that the 1:100 year floodline be raised by 1.5 m. This was achieved by adding 1.5 m to the water surface elevations in the HEC-RAS export file prior to post-processing with HEC-GeoRAS. According to the second criterion above, the expropriation should be 15 m horizontally outside the 1:100 year flood line. GIS techniques were used to achieve this. In order to determine the greater of the two, which is essentially the final expropriation line, the two flood lines (i.e. vertically raised floodline and the horizontally offset floodline) were merged and the resulting floodline represents the greater of the two. The extent of the expropriation line from the dam wall is marked by point D in Figure 3. Point A, which is the intersection of pre-and post-dam construction floodlines, marks the end of influence of the dam on the water surface levels. The water level at point B is equal to the water level at point A raised by 1.5 m. Point C is the ground level equal to the water level at B and its distance from the dam wall is represented by point D.

1.3 Results

The final expropriation line is presented on drawing 401775 CT 280. Details of the water levels and the expropriation line levels are contained in the table which is included on drawing 401775 CT 280.

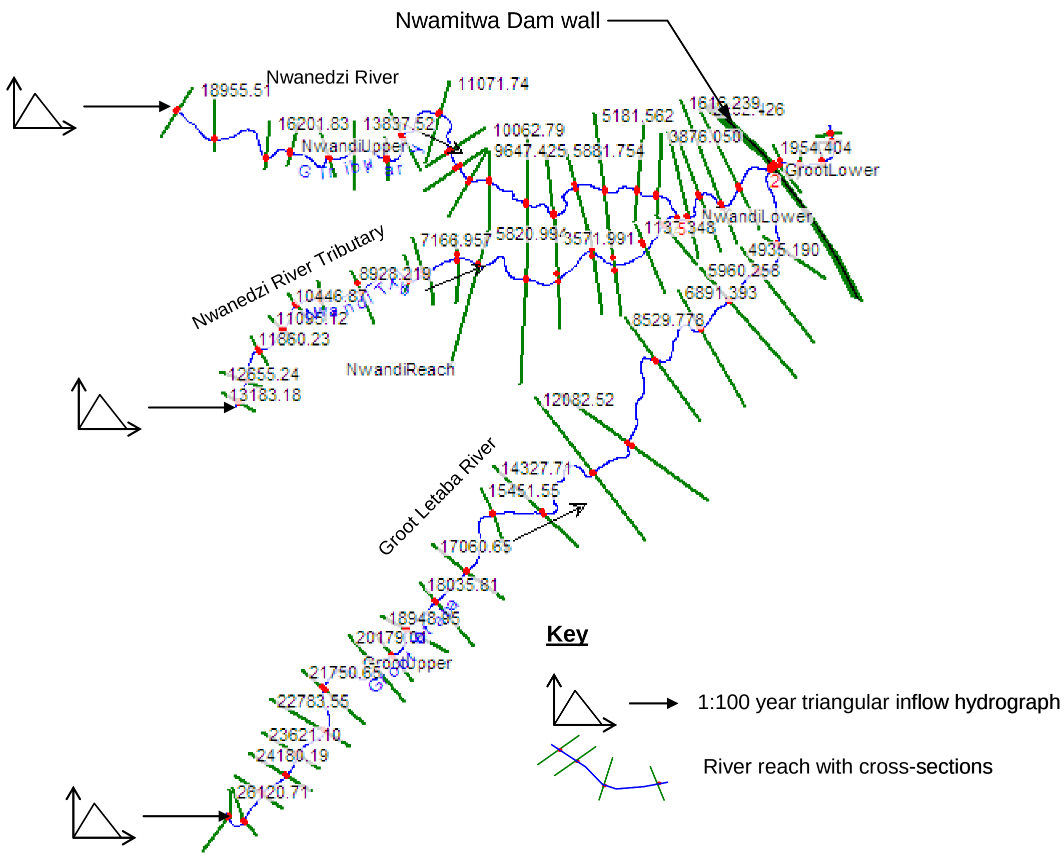

Figure 1 Nwamwitwa Dam HEC-RAS model configuration

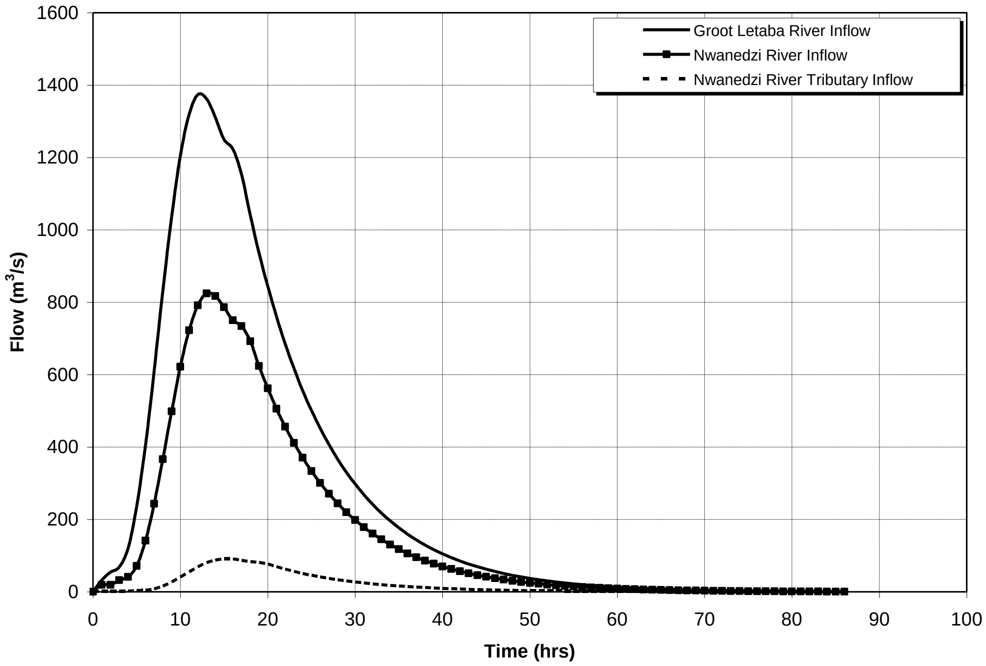

Figure 2 1:100 year inflow hydrographs

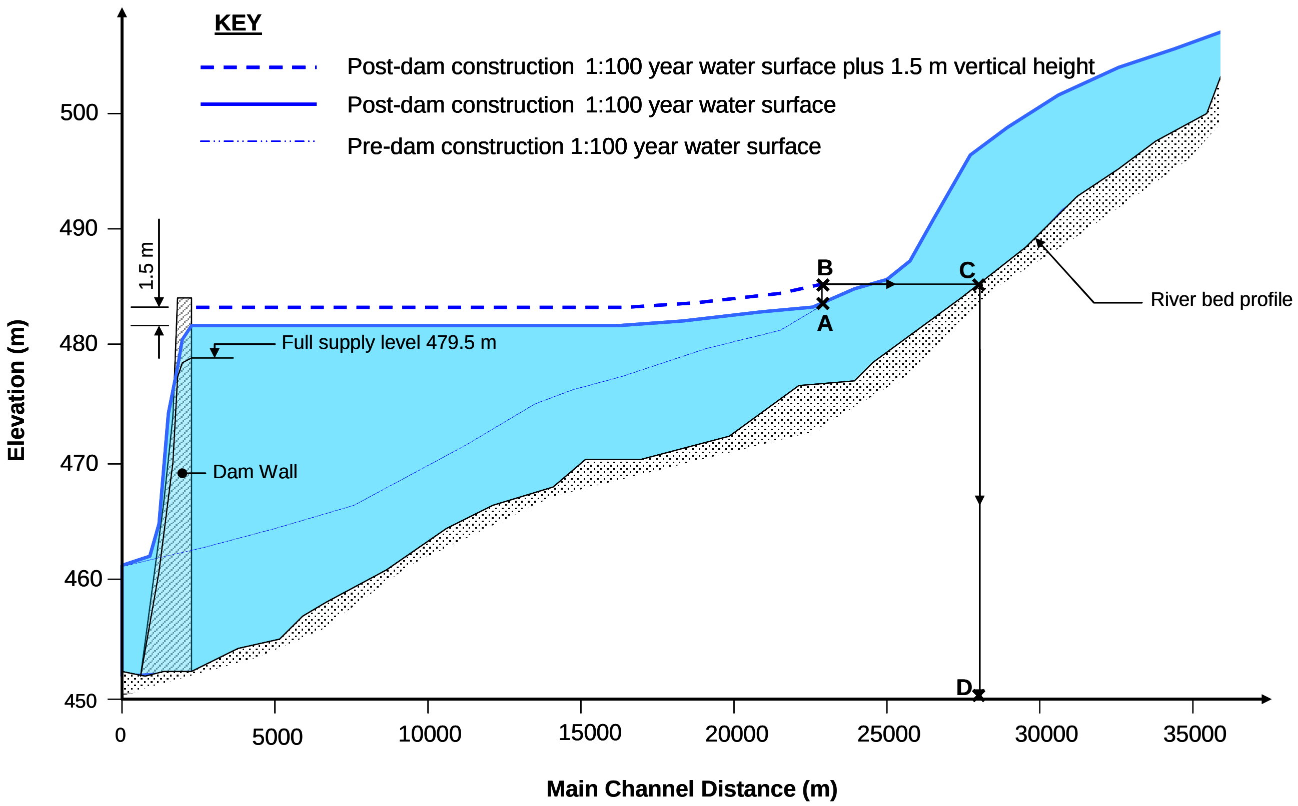

Figure 3 Backwater curve analysis concept for expropriation line determination (Groot Letaba River Reach)

1.4 References

Chow (1964) Chow, V T. 1964. *Handbook of applied hydrology.* New York, McGraw-Hill, Section 4-III.

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USACE (2008) U.S. Army Corps of Engineers (USACE). 2008. *HEC-RAS: River Analysis System User's Manual (CPD-68),* Hydrologic Engineering Centre (HEC), Davis, California

APPENDIX B : Geotechnical Investigations

(see separate volumes)

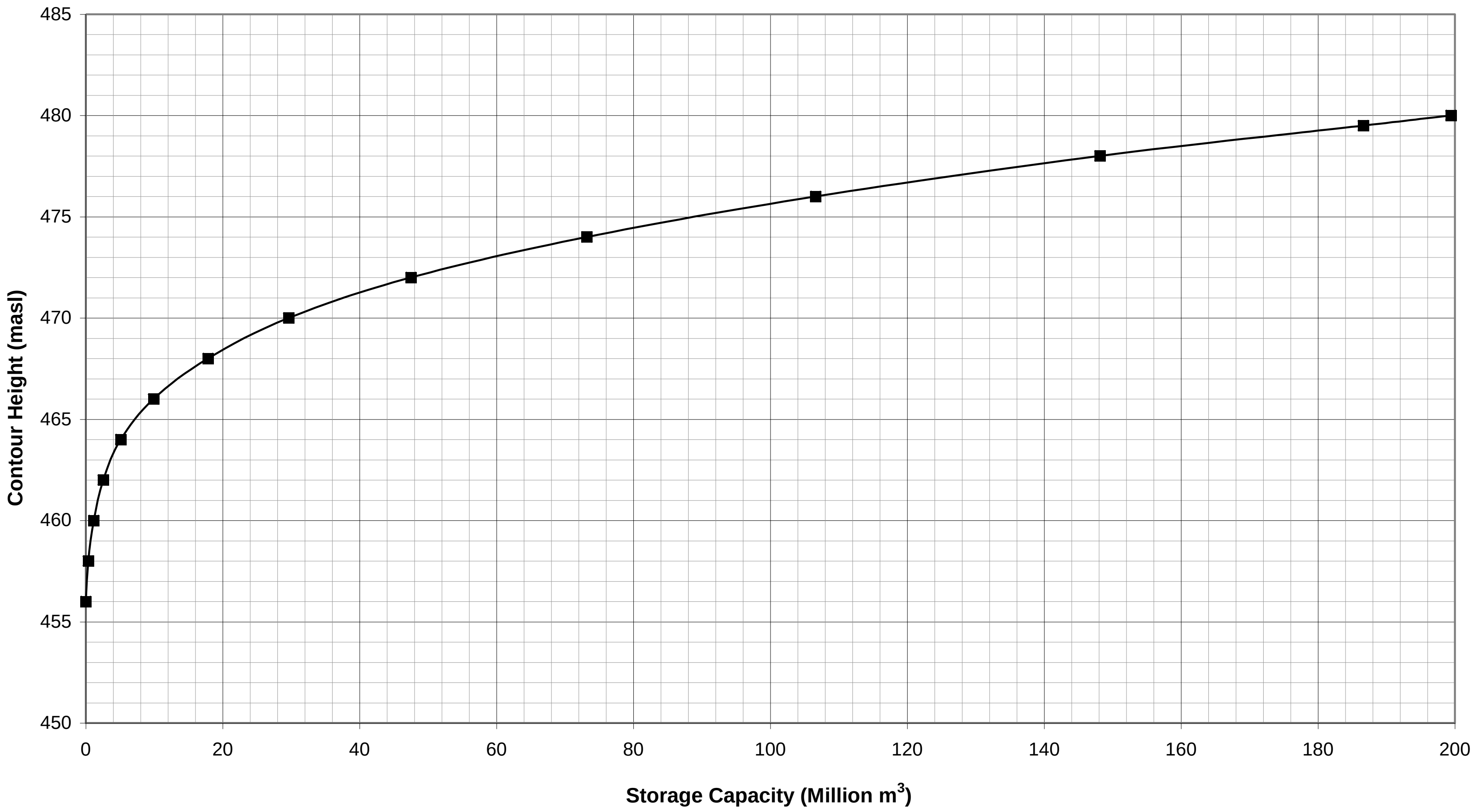
Volume 6 – Annexure 2: Appendix B (Part 1): Geotechnical Investigation (Text)

Volume 6 – Annexure 3: Appendix B (Part 2): Geotechnical Investigation (Appendices)

APPENDIX C : Embankment

APPENDIX C.1 : Stage Capacity Curve

APPENDIX C.1 STAGE CAPACITY CURVE

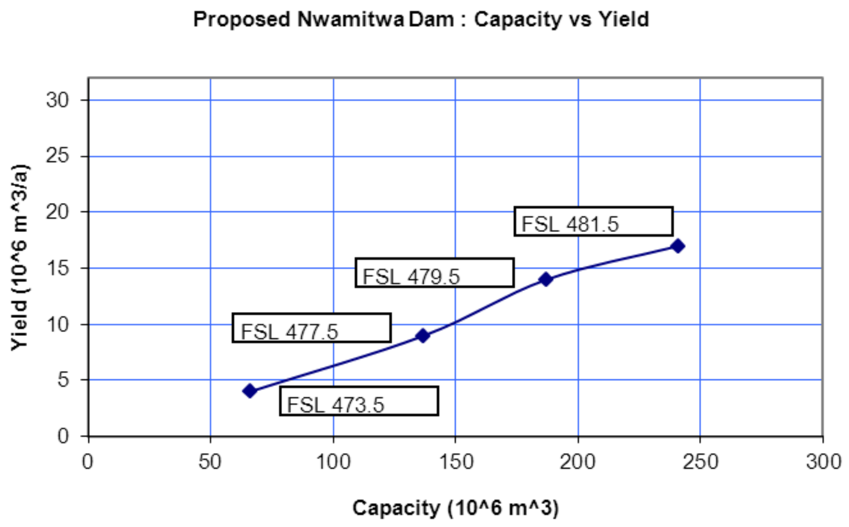


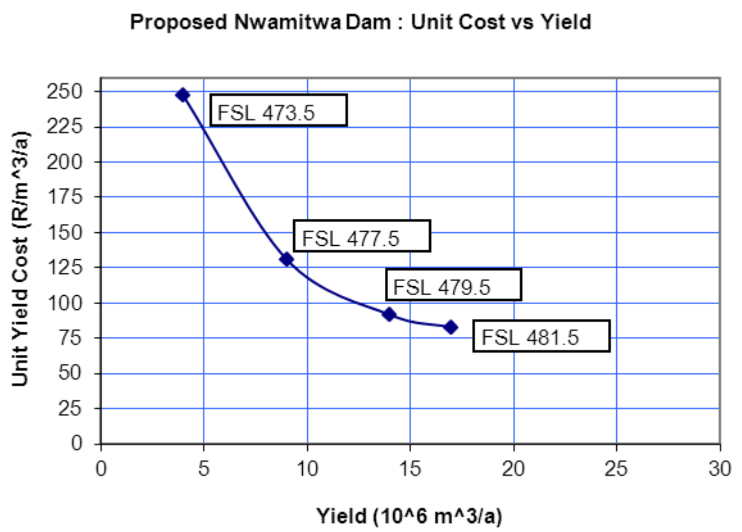
APPENDIX C.2 : Optimisation of Dam Size

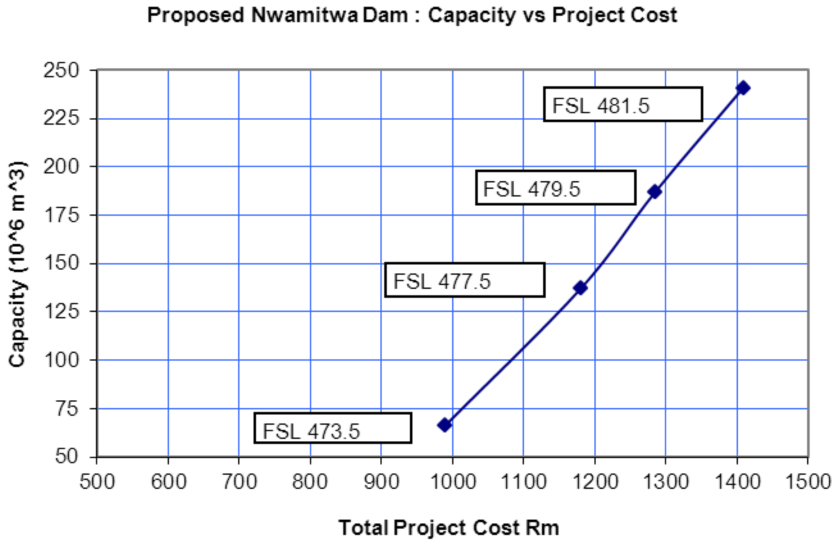
APPENDIX C.2 NWAMITWA DAM OPTIMISATION OF DAM SIZE

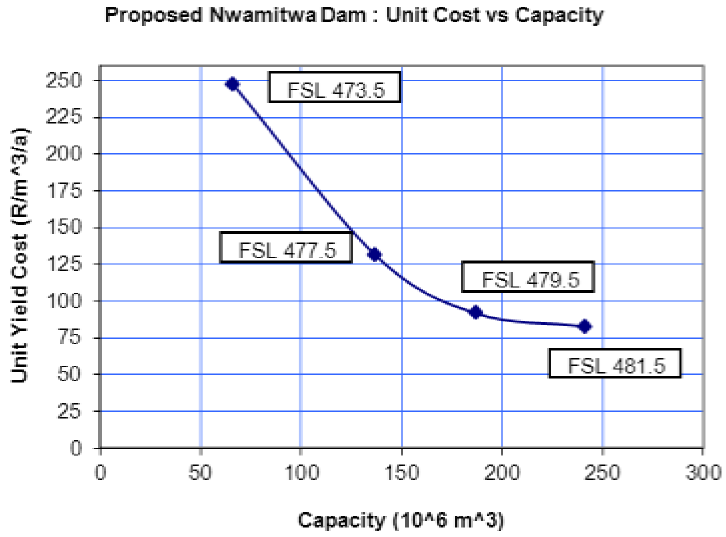
Dam size (Factor of MAR*)	Dam capacity (million m^3)	Full Supply Level (masl)	Total Project Cost (R million) Excluding VAT	Yield Mm^3/a	Unit Cost R/m3/a Dam construction and land costs
0.41	66	473.50	989	4	247.25
0.85	137	477.50	1180	9	131.11
1.16	187	479.50	1285	14	91.79
1.50	241	481.50	1409	17	82.88

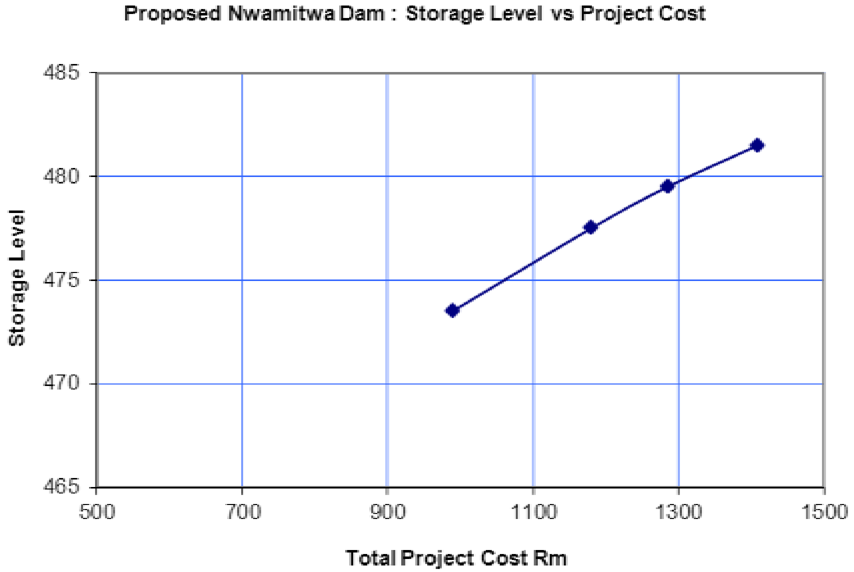
* Natural incremental MAR between Tzaneen and Nwamitwa Dams = 160.9 Mm3/a



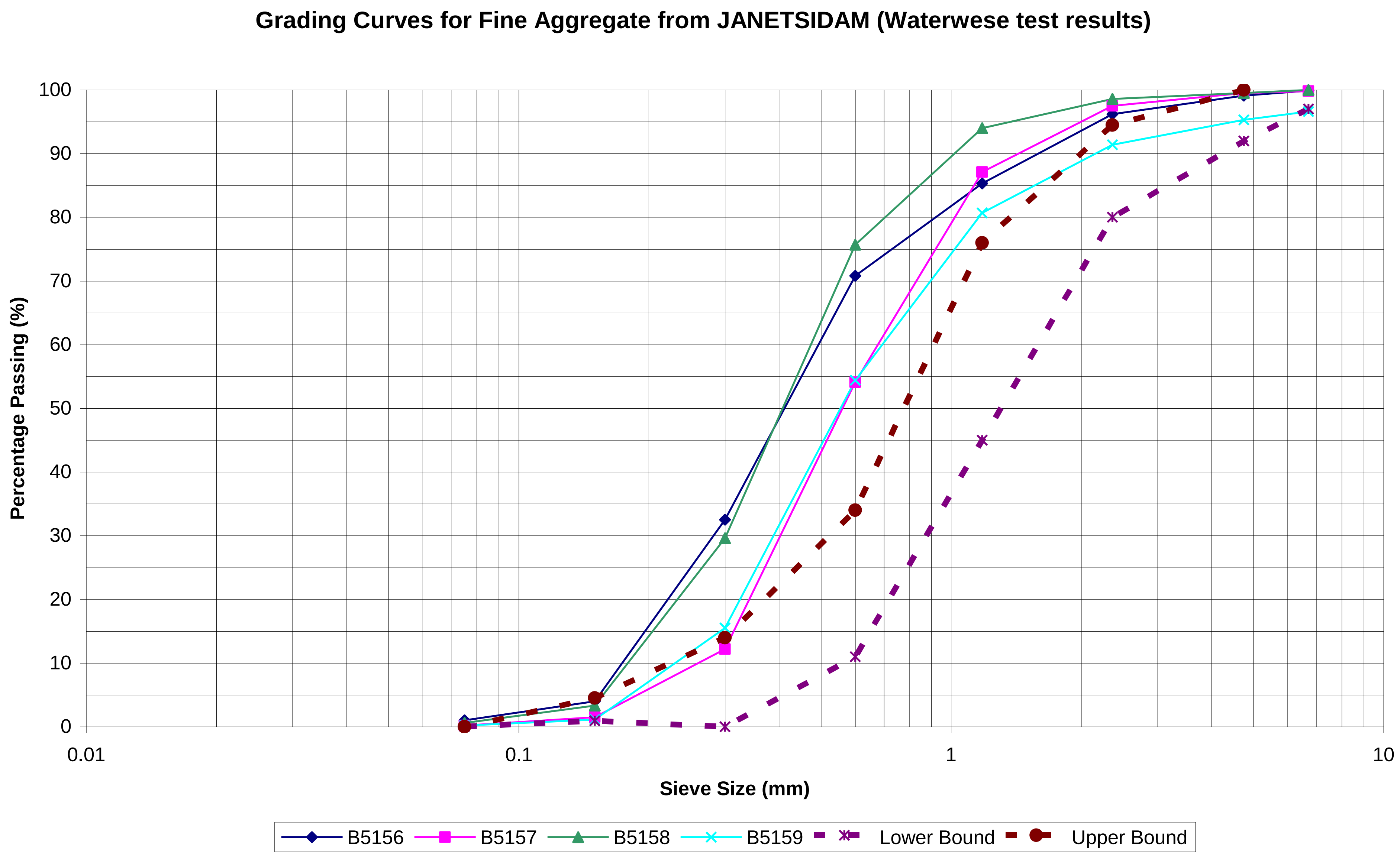



Proposed Nwamitwa Dam : Capacity vs Project Cost
Capacity (10^6 m^3)
Total Project Cost Rm
FSL 481.5
FSL 479.5
FSL 477.5
FSL 473.5
250
225
200
175
150
125
100
75
50
500
600
700
800
900
1000
1100
1200
1300
1400
1500

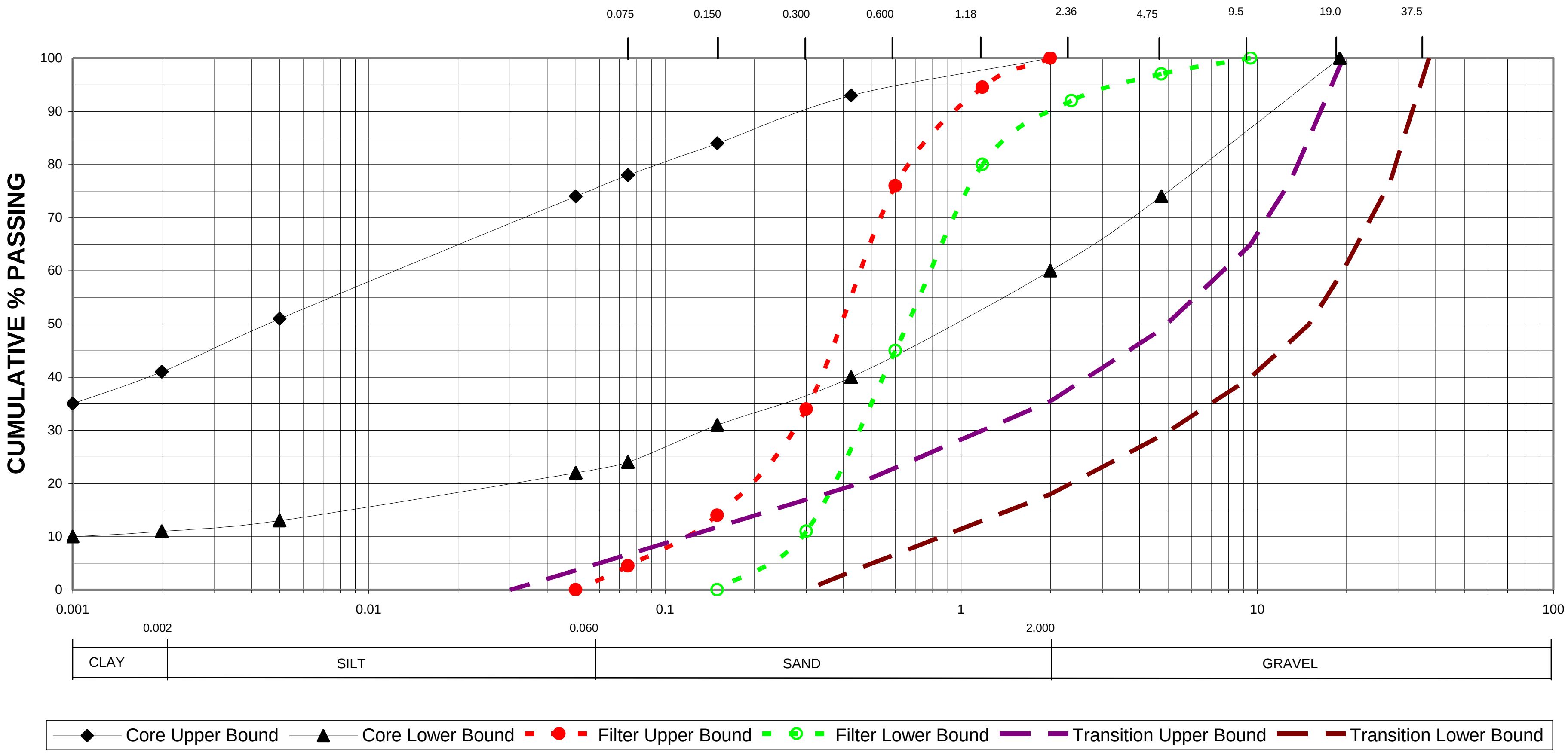

Proposed Nwamitwa Dam : Unit Cost vs Capacity
Unit Yield Cost (R/m^3/a)
Capacity (10^6 m^3)
FSL 473.5
FSL 477.5
FSL 479.5
FSL 481.5
250
225
200
175
150
125
100
75
50
25
0
0
50
100
150
200
250
300


Proposed Nwamitwa Dam : Storage Level vs Project Cost
Storage Level
Total Project Cost Rm
485
480
475
470
465
500
700
900
1100
1300
1500

APPENDIX C.3 : Grading Envelopes

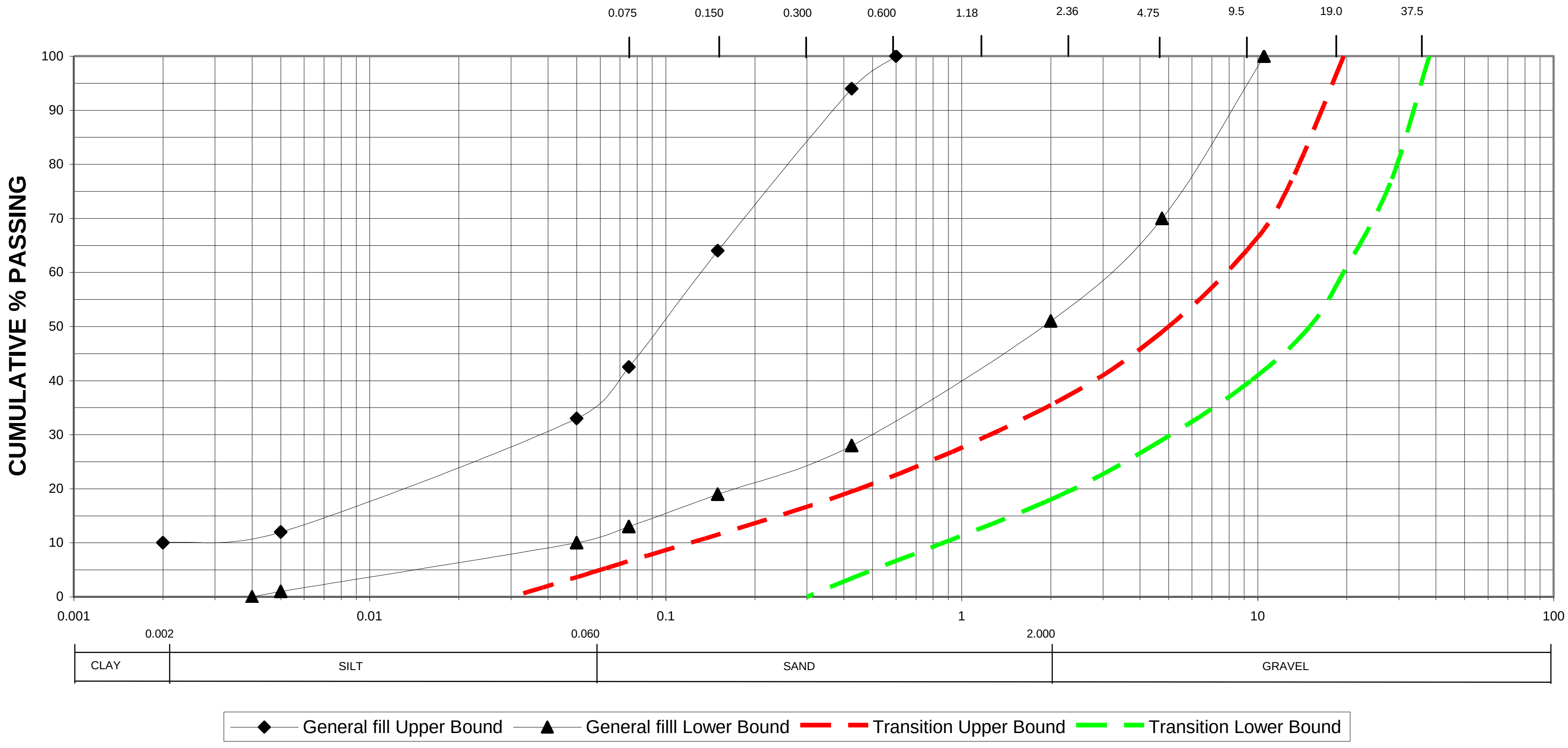

Grading Curves for Fine Aggregate from JANETSIDAM (Waterwese test results)
Percentage Passing (%)
100
90
80
70
60
50
40
30
20
10
0
0.01
0.1
1
10
Sieve Size (mm)
B5156
B5157
B5158
B5159
Lower Bound
Upper Bound

APPENDIX C.3.1 NWAMITWA DAM CORE, CHIMNEY & TRANSITION MATERIAL GRADING ENVELOPES



<u>NOTE</u> : CORE AND CHIMNEY GRADING ENVELOPES FROM DWAF MATERIALS INVESTIGATION (1996)

APPENDIX C.3.2 NWAMITWA DAM GENERAL FILL AND TRANSITION ZONE ENVELOPES



NOTE : GENERAL FILL GRADING ENVELOPE FROM DWAF MATERIALS INVESTIGATION (1996)

Determination of Filter Criteria for Nwamitwa Dam

Waterwese grond toetse
Kernmateriaal

Sieve	% Passing
Maks	
2	100
0.425	93
0.15	84
0.075	78
0.05	74
0.005	51
0.002	41
0.001	35
MIN	
19	100
4.75	74
2	60
0.425	40
0.15	31
0.075	24
0.05	22
0.005	13
0.002	11
0.001	10

Skilmateriaal

Sieve	% Passing
Maks	
0.6	100
0.425	94
0.15	64
0.075	42.5
0.05	33
0.005	12
0.002	10
0.001	9
MIN	
10.5	100
4.75	70
2	51
0.425	28
0.15	19
0.075	13
0.05	10
0.005	1
0.004	0

Grading envelop for core material done by HS.

Waterwese
Results of tests on Fine Aggregate

Information for Grading of Chimney material obtained from Nwamitwa Dam - DCM Report
Information presented below

Grain Size Distribution (% passing sieve size)	B5156 Phatle G1	B5157 Phatle G2	B5158 Marekome G1	B5159 Marekome G2	LB Grain Size	Lower Bound	UB Grain Size	Upper Bound
					9.5	100		
6.7	99.9	99.8	100	96.6	4.75	97		
4.75	99.1	99.5	99.5	95.3	2.36	92	2	100
2.36	96.2	97.5	98.6	91.4	1.18	80	1.18	94.5
1.18	85.3	87.1	94	80.7	0.6	45	0.6	76
0.6	70.8	54.1	75.7	54.4	0.3	11	0.3	34
0.3	32.5	12.2	29.6	15.5	0.15	0	0.15	14
0.15	4	1.5	3.3	1.1		0.95	0.075	4.5
0.075	1	0.2	0.6	0.2		0.05	0.05	0

Upper Bound and Lower Bound Curves obtained by entering values that were above highest and below lowest values respectively

Grain Size Distribution (% passing sieve size) mm	Transition Lower Bound	mm	Transition Upper Bound
38	100		
28	76.5		
19.5	60	19.5	100
15	50	13	77
9.5	40	9.5	65
4.75	29	4.75	49
2	18	2	35.5
0.45	4	0.45	20
0.3	0	0.06	5
		0.03	0

Upper and Lower bound values obtained from DWG 401639 CEN 02 which is included in DCM report
Envelope represents grading of Transition gravel for Richmond Dam

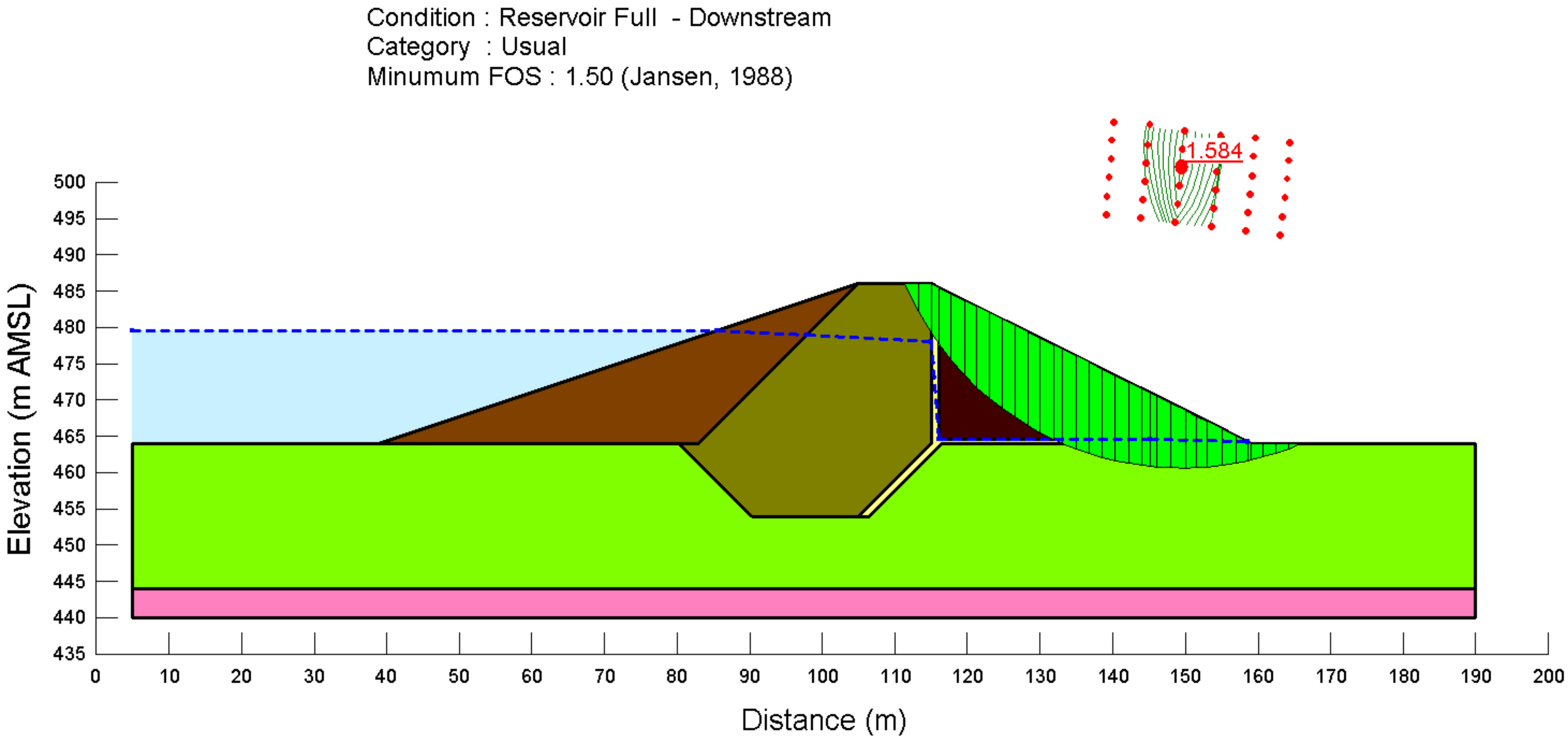
Parameter	mm		mm
D_{85} Core	0.17	to	8.5
D_{50} Core	0.0045	to	0.95
D_{15} Core	0.001	to	0.009
D_{85} Chimney	0.8	to	1.35
D_{50} Chimney	0.4	to	0.65
D_{15} Chimney	0.15	to	0.33
D_{50} Transition	5	to	16
D_{15} Transition	0.24	to	1.7
D_{85} Blanket	0.8	to	1.35
D_{50} Blanket	0.4	to	0.65
D_{15} Blanket	0.15	to	0.33
D_{85} General Fill	0.305	to	7.3
D_{50} General Fill	0.097	to	2
D_{15} General Fill	0.0072	to	0.095

Values obtained by scaling of graph with Envelopes.
Some points were interpolated

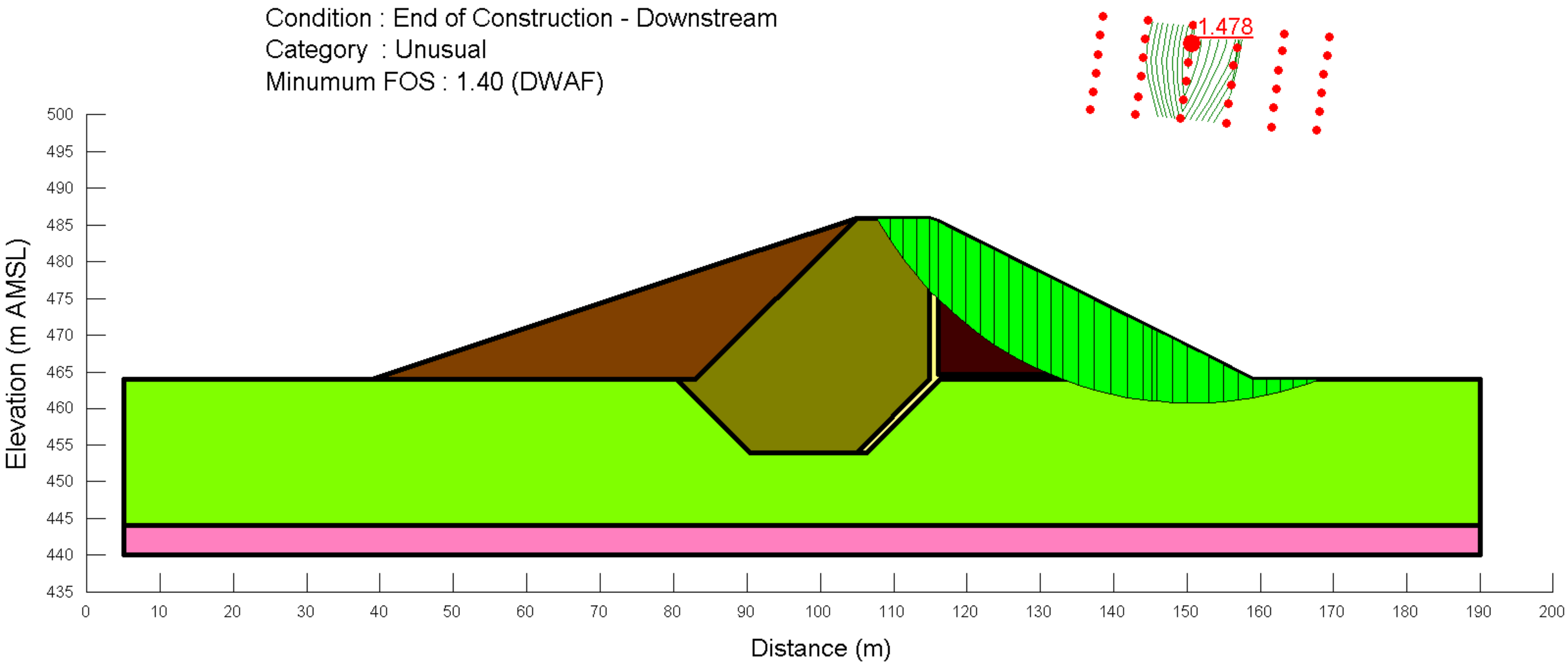
Interface	Criteria	Actual Factor, f		Recommended Factor	Comments
		From	To		
Core / Chimney	D15 of Chimney / D85 of Core	0.02	1.9	f < 5	OK
	D50 of Chimney / D50 of Core	0.42	144.4	f < 25	Complies Partially
	D15 of Chimney / D15 of Core	16.67	330.0	5 < f < 40	Complies Partially
Blanket / Transition	D15 of Transition / D85 of Blanket	0.18	2.1	f < 5	OK
	D50 of Transition / D50 of Blanket	7.69	40.0	f < 25	Complies Partially
	D15 of Transition / D15 of Blanket	0.73	11.3	5 < f < 40	Complies Partially
General Fill / Transition	D15 of Transition / D85 of gen fill	0.03	5.6	f < 5	Complies Partially
	D50 of Transition / D50 of gen fill	2.50	164.9	f < 25	Complies Partially
	D15 of Transition / D15 of gen fill	2.53	236.1	5 < f < 40	Does not comply

APPENDIX C.4 : Slope Stability Analysis

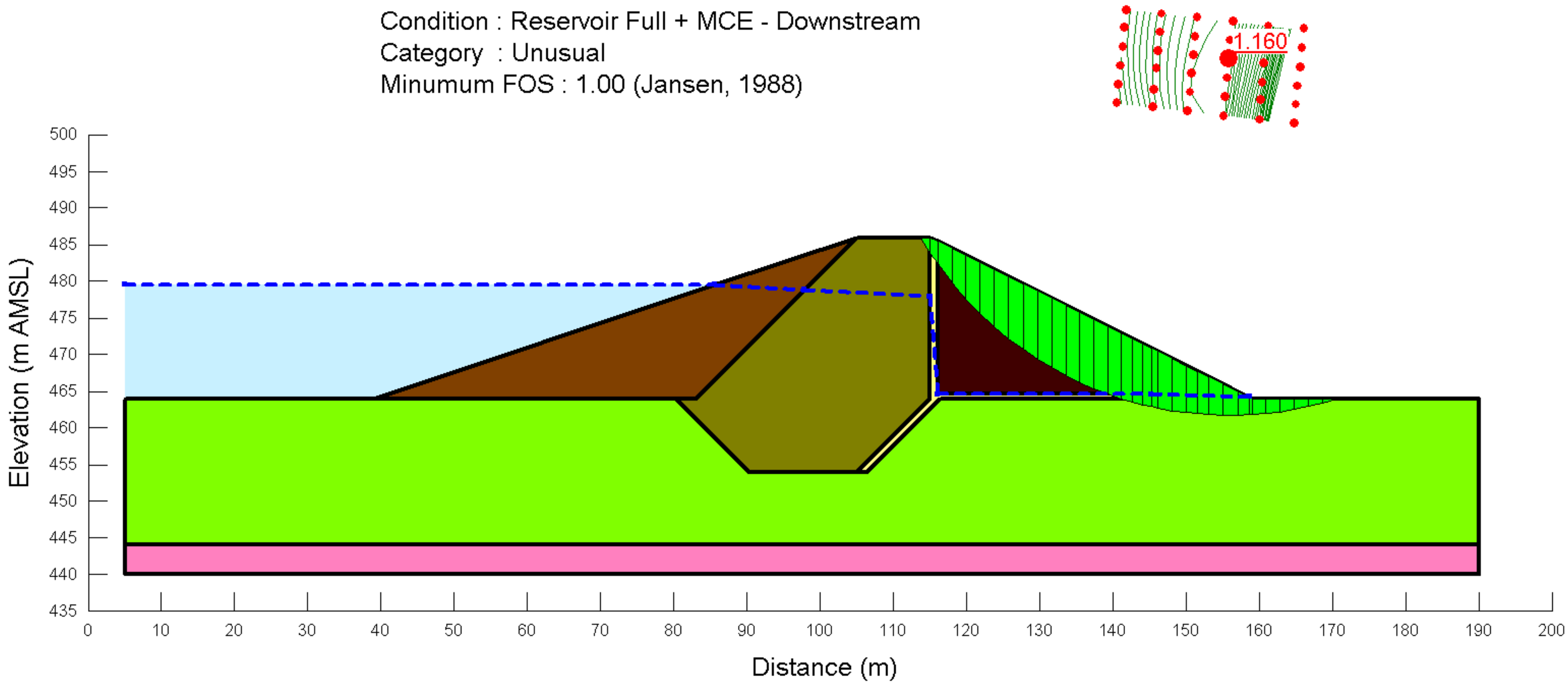
Appendix C.4.1 – DOWNSTREAM SLOPE – RESERVOIR FULL



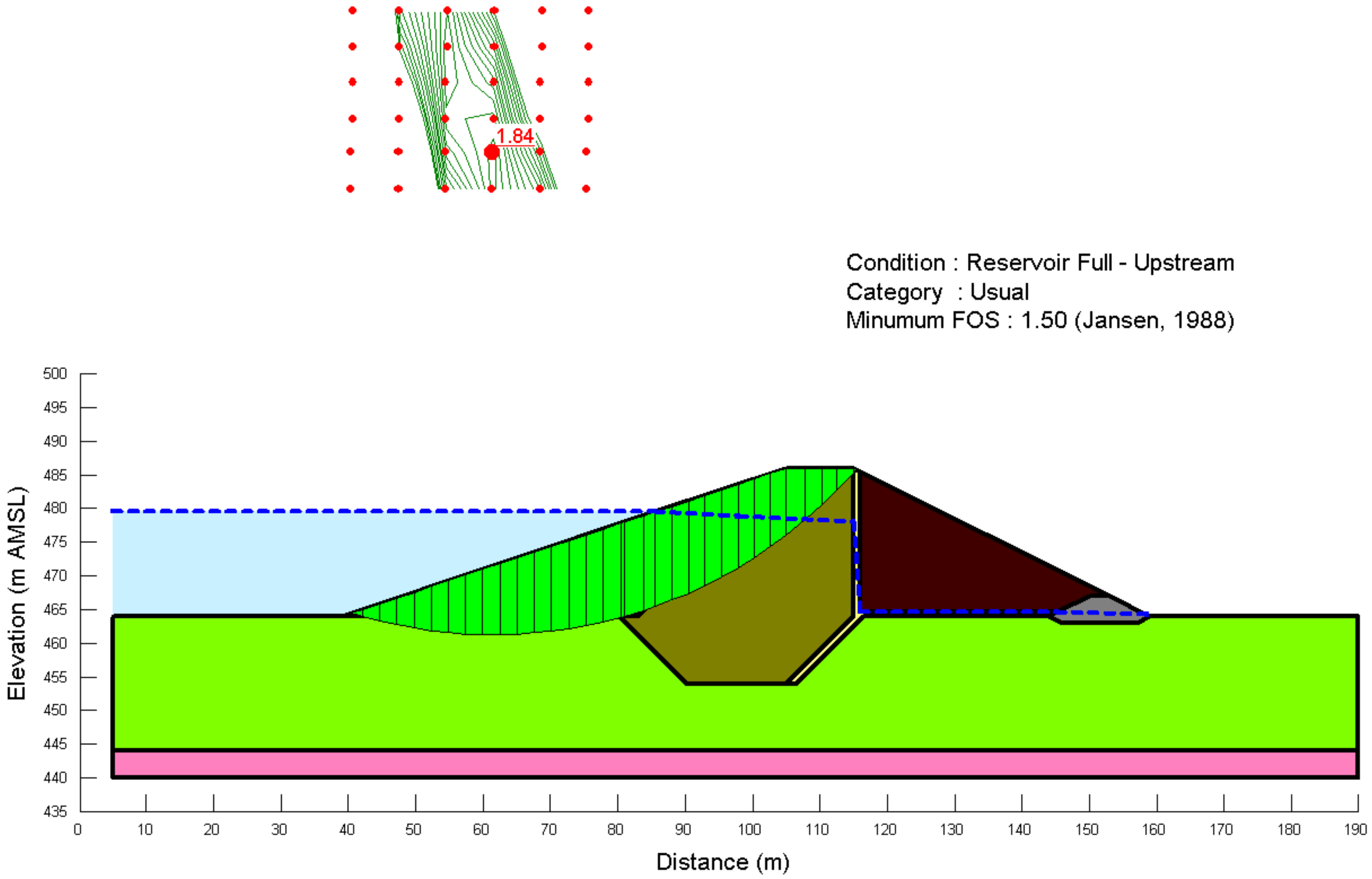
Appendix C.4.2 – DOWNSTREAM SLOPE – END OF CONSTRUCTION



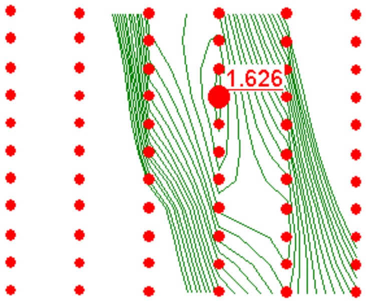
Appendix C.4.3 – DOWNSTREAM SLOPE – RESERVOIR FULL + MCE



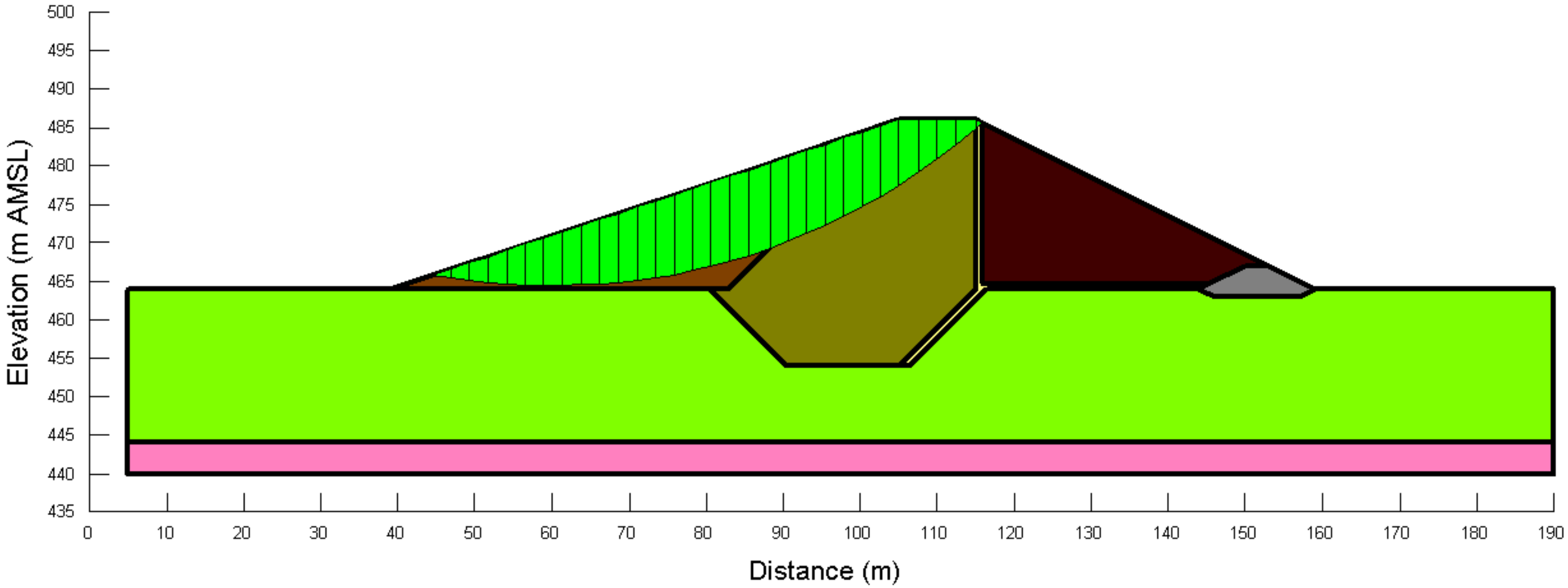
Appendix C.4.4 – UPSTREAM SLOPE – RESERVOIR FULL



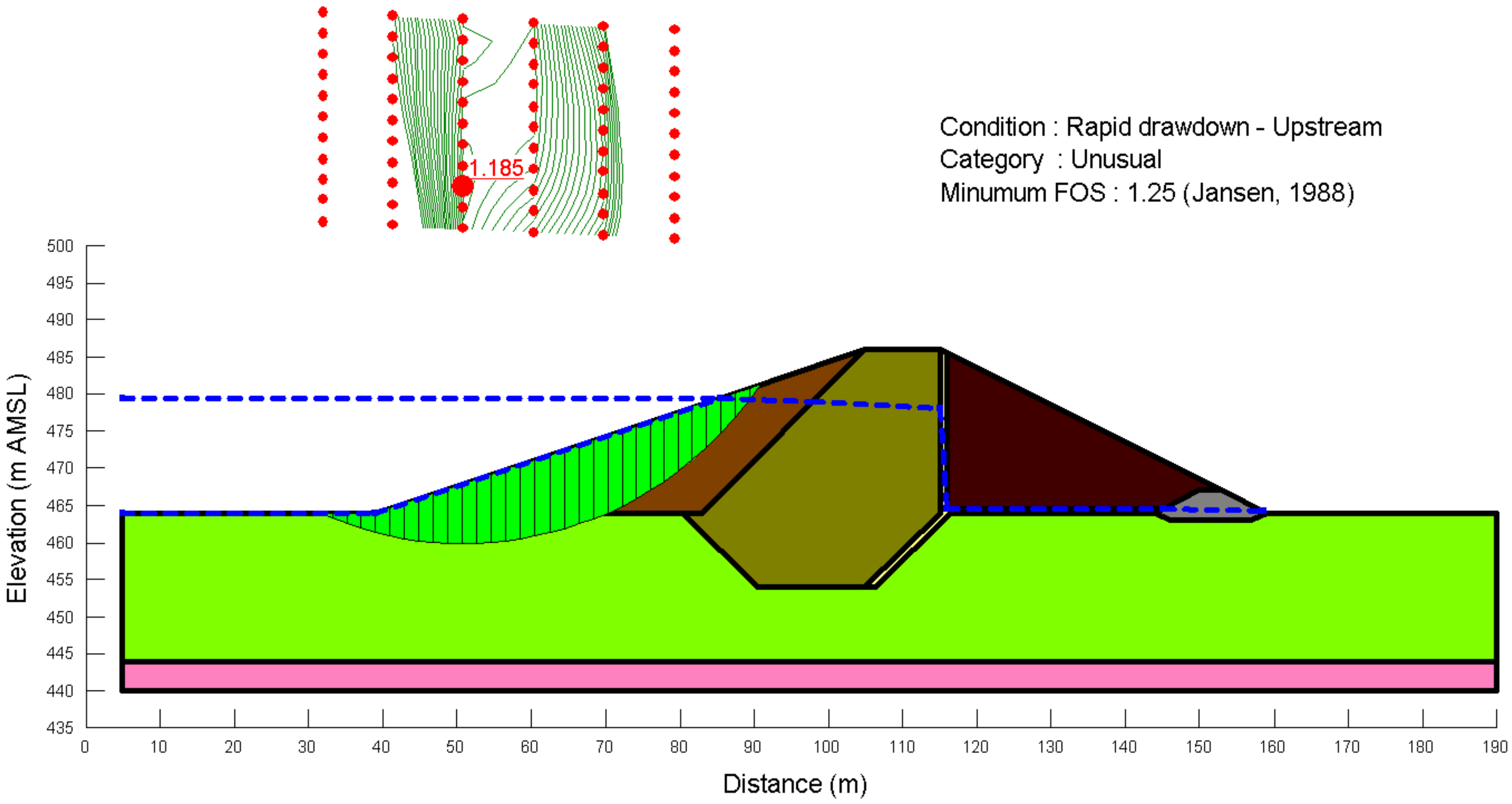
Appendix C.4.5 – UPSTREAM SLOPE – END OF CONSTRUCTION



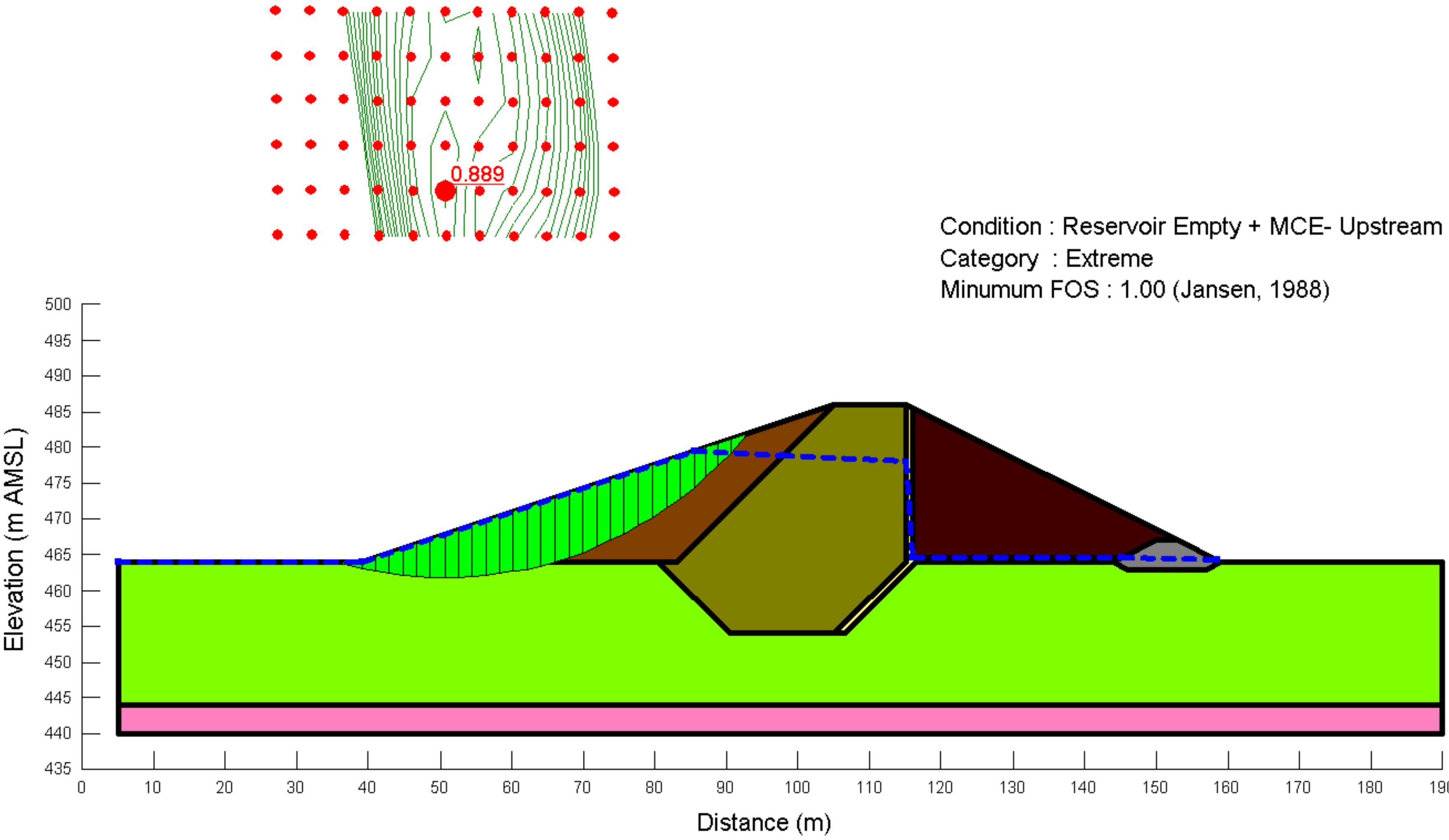
Condition : End of Construction - Upstream
Category : Unusual
Minumum FOS : 1.40 (DWAF)



Appendix C.4.6 – UPSTREAM SLOPE – RAPID DRAWDOWN



Appendix C.4.7 – UPSTREAM SLOPE – RESERVOIR EMPTY + MCE



APPENDIX C.5 : Freeboard Calculations

APPENDIX C.5 NWAMITWA DAM FREEBOARD

Based on SANCOLD publication Interim Guidelines on Freeboard for Dams, Report No 3, August 1990

Wave height for 1:25 year recurrence interval

Symbol Unit Calc Comments

Effective Fetch

Angle from central Radial (a)	Cos a	Length of radial Xi (km)		Sum ((Cos a) * (Xi))
42	0.743	5.267	543.5	3.91
36	0.809	5.81	634.2	4.70
30	0.866	3.369	721.4	2.92
24	0.914	3.238	780.9	2.96
18	0.951	6.679	887.3	6.35
12	0.978	4.66	978.4	4.56
6	0.995	3.807	1958.3	3.79
0	1.000	2.083	2584.8	2.08
6	0.995	1.684	1889.9	1.68
12	0.978	7.053	1434.8	6.90
18	0.951	7.507	1247.6	7.14
24	0.914	5.238	512.4	4.79
30	0.866	3.678	385.9	3.19
36	0.809	4.241	309.7	3.43
42	0.743	4.032	279.8	3.00
TOTALS	13.512			61.38

Effective fetch m 4543 (Sum Xi*Cos a)/(Sum Cos a)

Significant wave height

v land m/s 20.3 Recorded highest hourly wind speed for Polokwane

F m 4543 Effective fetch

Effective fetch (km)	1	2	4	6	8+	
Wind speed ratio		1.1	1.16	1.23	1.28	1.3

Ratio = Over water / Over land ex Saville

Wind speed ratio 1.244

v water m/s 25.3

Hs m 1.24 Significant wave height = 0.0026*(v^2/g)*(g*F/v^2)^0.47 (Saville)

Design wave height

Wave period Tsec		3.8	Figure A3 in Sancold Report No 3
Wave length L	m	23	L = 1.56*T^2
Storage capacity		10^6 m^3	187.0
Surface area	hectare		2700
Ave water depth		m	7
Water depth/L		0.31	

Shallow water = water depth < 1/3 to 1/2 wave length

Result : Dealing with shallow water

Hd / Hs 1.1 Design wave height in terms of significant wave height for Earthfill Dam with road on crest

Hd 1.37

Wave run-up

Emb slope 3

Surface : Minimum - thick permeable riprap

Run-up ratio 1.0 Wave run-up / Design wave height ex Figure A5 in Sancold Report No 3

Run-up m 1.37

APPENDIX D : Spillway

D1 Spillway Type Selection Report

D2 Spillway Stage Discharge Curve

D3 Spillway Energy Dissipation and Stilling Basin Calculations

D4 Tailwater Curve

D5 River Diversion Water Profile Calculations

APPENDIX D.1 : Spillway Type Selection Report

(Note: Electronic MS Word version excludes this Report. Only available in pdf format.)

water & forestry

Department:
Water Affairs and Forestry
REPUBLIC OF SOUTH AFRICA

REPORT NO.:

GROOT LETABA RIVER WATER DEVELOPMENT PROJECT : BRIDGING STUDIES – TECHNICAL STUDY (GLEWAP)

NWAMITWA DAM

SPILLWAY TYPE SELECTION REPORT

MAY 2008

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Schoeman & Partners

Prepared for: Department of Water Affairs and Forestry

DRAFT

REPORT DETAILS PAGE

Project name:	***Groot Letaba River Water Development Project – Bridging Study***
Report Title:	***Nwamitwa Dam – Spillway Type Selection Report***
Authors:	***Dawid van Wyk, Herman Smit***
DWAF report reference no.:	***[will be provided]***
Status of report:	***Draft***
First issue:	***May 2008***
Final issue:	

PSP
Approved for PSP by:

-- --------------------

SC Vogel *Date*
Study Leader

PROJECT CO-ORDINATION AND MANAGEMENT TEAM
BKS (Pty) Ltd - *Approved for the Project Coordinator by**:***

-- --------------------

RA Pullen *Date*
Project Coordinator & Manager

DEPARTMENT WATER AFFAIRS & FORESTRY (DWAF)
*Approved for DWAF by**:***

OJS van den Berg	*Date*	*LS Mabuda*	*Date*
Chief Engineer: Options Analysis North		*Director: Options Analysis*	

ACRONYMS AND ABBREVIATIONS

FSL	Full Supply Level
H	Height (m)
MAR	Mean Annual Runoff
NOC	Non Overspill Crest
PMF	Probable Maximum Flood
RDF	Recommended Design Flood
RI	Recurrence Interval
RMF	Regional Maximum Flood
SEF	Safety Evaluation Flood

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Page No.

APPENDICES

1 INTRODUCTION

The original brief for the feasibility study of the Nwamitwa Dam called for a Dam Type Selection Report. The valley shape factor for the Nwamitwa Dam site is in excess of 50, which is a clear indication that the most appropriate dam type would be an embankment type dam. The Dam Type Selection Report was therefore replaced by a Spillway Type Selection Report as the spillway was the only component of the dam that could feasibly be investigated for alternative layouts / types.

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

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

The comparison was made for a 1.2 MAR (Mean Annual Runoff) dam size with a FSL (Full Supply Level) at 479.5 masl. The general layout of the dam is shown on Drawing 401775 CEN 10 in Appendix C.1.

2 SPILLWAY FLOODS

2.1 Flood peaks

The following flood peaks have been selected to size the spillway:

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

2.2 Freeboard

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

3 DISCHARGE CAPACITY

3.1 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

In order to size the ogee section of the spillway, a design head (H_d) of 4.5 m was selected. This will allow for an increased discharge capacity over the full range of overflow depths, eg C_d = 2.30 at the maximum head of 6.5 m. In order to retain some conservatism in this comparative exercise, the design head C_d of 2.18 was used for all ogee sections.

3.2 Labyrinth Spillway

The design procedure for the labyrinth spillway was adopted from "Design of Labyrinth Spillways", Journal of Hydraulic Engineering, Volume 121, No 3, March 1995 by J.P Tullis, N Amanian and D Waldron. The procedure provides a design calculation presented in a spreadsheet format, as shown in Table 3-1.

Table 3-1 : Labyrinth Dimensions

NWAMITWA DAM TYPICAL CALCULATION OF LABYRINTH DIMENSIONS				
Parameter	**Symbol**	**Quantity**	**Units**	**Comment**
Maximum flow	Qmax	6800	m^3/s	Input = SEF (Q = 6800)
Maximum reservoir elevation	res	486	m	Input
Approach channel elevation		450.0	m	Input
Crest elevation	el	479.5	m	Input
Total head	Ht	6.5	m	Ht = res - crest - loss
Estimated inlet loss at Qmax	Loss	0	m	Estimated
Number of cycles	N	12	-	
Crest height	P	9.1	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	1.5	m	Select between t and 2t
Outside width of apex	D	3.34	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	334.22	m	1.5*Qmax/[(Cd*Ht^1.5)*(2*g)^0.5]
Length of apron (parallel to flow)	B	14.09	m	[L/(2*N)+t*tan(45-alpha/2)-A]*cos(alpha)+t
Actual length of side leg	L1	13.35	m	(B-t)/@cos(alpha)
Effective length of side leg	L2	12.43	m	L1-t*tan(45-alpha/2)
Total length of walls	L3	378.42	m	N*(2*L1+D+A)
Distance between cycles	w	11.75	m	2*L1*sin(alpha)+A+D
Width of labyrinth (normal to flow)	W	141.00	m	N*w
Length of linear weir for same flow		182.84	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	1.29	-	

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 full supply level (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 = 12 was chosen for practical construction.

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

4 COST ESTIMATES

The cost estimate has been based on the analysis of the construction costs of the following dam projects as part of the Lesotho Highlands Further Phases (LHFP) Feasibility Study:

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

Subsequent construction tenders indicated that reinforced concrete related rates had increased well above the escalation indices. The following adjustments have therefore been made to the LHFP rates:

- Reinforced concrete LHFP rate x 2.5
- Formwork LHFP rate x 2.0
- Reinforcement LHFP rate x 1.5

5 STRAIGHT OGEE SPILLWAY

5.1 Description

The spillway would consist of a mass gravity RCC section 190 m long with tongue walls on either side with a total length of 200 m to accommodate the outlet works and to tie into the earth embankments. The proposed layout and cross sections are shown on Drawings 401775 CEN 11 and 12 in Appendix C.2.

The specific discharge during the RDF would be 10 m^3/s.m which would require very little in terms of an energy dissipating structure at the toe of the spillway as most of the energy dissipation would take place on the steps. However, due to the significant difference between the RDF and the SEF, it is recommended that the energy dissipating structure be sized for (say) half of the SEF, which is approximately 3 500 m^3/s. The specific discharge of 18 m^3/s.m would result in skimming flow over the RCC steps with reduced energy dissipating capacity. A 35 m long concrete lined stilling basin was therefore added to the spillway. The length of the stilling basin was based on the assumption that some 20% energy dissipation would still take place on the RCC steps. A conjugate depth of about 8.4 m (456.4 masl) would be required to generate the hydraulic jump in the stilling basin.

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

The tail water level at the dam during a flood peak of 3 500 m^3/s is estimated at level 461 masl, which would be well above the required conjugate depth level stated above. The hydraulic jump would therefore be forced upstream towards the toe of the spillway and might even become partly submerged.

5.2 Outlet Works

The outlet works would be incorporated in a reinforced concrete block immediately to the left of the spillway.

5.3 River diversion

The first phase of the river diversion would comprise upstream and downstream coffer dams and a diversion channel excavated from the Nwamitwa River through the left flank of the dam. An interconnecting channel would be excavated between the Groot Letaba and Nwamitwa Rivers.

The second phase of the river diversion would be achieved by leaving appropriately sized openings in the RCC section.

5.4 Cost estimate

The cost estimate for an Nwamitwa Dam with a FSL at 479.5 masl and a straight ogee spillway is **R771 million**. The cost estimate includes the relocation of the two provincial roads located on the north-western and south-eastern sides of the dam basin (Road R529 and Road P43/3 respectively). It also includes planning, design and supervision costs, but excludes VAT and land costs.

Details of the cost estimate are shown in Appendix A.1.

6 TROUGH SPILLWAY

6.1 Description

A trough spillway would consist of a curved mass gravity RCC section protruding upstream of the embankments. It would be situated in the same position as the straight ogee spillway, discharging directly into the river in the direction of the natural flow. The tongue walls on either side would be replaced by mass gravity sections against which the embankments would abut. The proposed layout and sections are shown on Drawings 401775 CEN 13 and 14 in Appendix C.3.

In order to allow for some reduction in the discharge capacity of the curved section due to the converging flow lines, the overall length of the spillway was increased by 8% to 205 m. The trough was sized by using the programme CFP. In the event of the SEF, the water level at the upstream end of the trough would be at the FSL. There would therefore be no risk of submergence of the ogee crest.

The invert level of the trough would be raised to accommodate a diversion conduit below as was done at Inyaka Dam. The trough would terminate in a 30 m long stilling basin.

6.2 Outlet Works

The outlet works would consist of a free standing intake tower 40 m upstream of the spillway. The outlet pipes would be encased in the left hand side wall of the trough spillway.

6.3 River diversion

The first phase of the river diversion would be similar to that proposed for the straight ogee spillway.

The second phase of the river diversion would be achieved by constructing a diversion conduit from the intake tower through the bottom of the trough spillway.

6.4 Cost estimate

The cost estimate for an Nwamitwa Dam with a FSL at 479.5 masl and a trough spillway is **R1 107 million**. Details of the cost estimate are shown in Appendix A.2.

7 LABYRINTH SPILLWAY

7.1 Description

In order to reduce the length of the straight ogee spillway, a labyrinth spillway was investigated. The proposed layout and a longitudinal section are shown on Drawings 401775 CEN 15 and 16 in Appendix C.4.

The layout would be similar to that of the straight ogee spillway with tongue walls on either side to accommodate the outlet works and to tie into the earth embankments. However, due to the labyrinth arrangement the spillway section could be shortened to 140 m.

As for the straight ogee spillway, a 35 m long stilling basin would be required, but with a reduced width of 140 m.

The discharge curves for a labyrinth spillway were developed with a relatively flat discharge channel. In the case of this proposal, some increase in the discharge capacity of the labyrinth could be expected due to the free fall nature of the RCC section below the labyrinth. This could, however, only be verified through a hydraulic model study.

7.2 Outlet Works

The outlet works would be incorporated in a reinforced concrete block immediately to the left of the spillway as for the straight ogee spillway.

7.3 River diversion

The river diversion would also be similar to that proposed for the straight ogee spillway.

7.4 Cost estimate

The cost estimate for an Nwamitwa Dam with a FSL at 479.5 masl and a labyrinth spillway is **R857 million**. Details of the cost estimate are shown in Appendix A.3.

8 SIDE CHANNEL SPILLWAY

8.1 Description

Favourable founding conditions on the left flank led to the investigation of a side channel spillway. The proposed layout are shown on Drawing 401775 CEN 17 in Appendix C.5.

As the freeboard and ogee crest arrangements would be the same as for the straight ogee spillway, a 190 m long side channel would be required to pass the SEF.

The side channel was sized by using the programme CFP. The bottom width would vary from 20 m at the upstream end to 50 m at the downstream end. The channel depth would vary from 12.5 m below the FSL at the upstream end to 16.5 m at the downstream end of the ogee. The side slopes would be 1V:0.5H. The maximum water level at the upstream end of the side channel was limited to 3 m above the FSL to prevent submergence of the ogee crest during the SEF.

A side channel spillway normally ends in a deflector bucket and plunge pool arrangement, which is a cost effective energy dissipating structure. Due the required depth of the discharge channel and the high tail water levels downstream of the dam, the deflector bucket would be completely drowned during low flood peaks. It would therefore not be effective as an energy dissipating structure.

An alternative energy dissipating structure would be a concrete lined stilling basin. The invert of the stilling basin would have to be at least 5 m below the existing river bed level to be effective during low flows. The side walls would have to be at least above the 3 500 m^3/s tail water level, which would require a structure 15 m deep.

8.2 Outlet Works

The outlet works would consist of a free standing intake tower. The outlet pipes would be housed in a reinforced concrete conduit underneath the embankment.

8.3 River diversion

The first phase of the river diversion would be similar to that proposed for the straight ogee spillway.

The second phase of the river diversion would be achieved by constructing a reinforced concrete diversion conduit from the intake tower underneath the embankment.

8.4 Conclusion

Based on the above technical constraints, a side channel spillway was discarded as a viable option.

9 RECOMMENDATIONS

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

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

The estimated capital costs of the first three types are as follows:

• Straight ogee spillway	R 771 million
• Trough spillway	R1 107 million
• Labyrinth spillway	R 857 million

The cost estimates include the relocation of the two provincial roads located on the north-western and south-eastern sides of the dam basin (Road R529 and Road P43/3 respectively). It also includes planning, design and supervision costs, but excludes VAT and land costs.

The side channel spillway was discarded as a viable option due to technical constraints.

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

APPENDIX A

COST ESTIMATES

APPENDIX A.1

COST ESTIMATE FOR STRAIGHT OGEE SPILLWAY

NWAMITWA DAM WITH STRAIGHT OGEE SPILLWAY

FSL = 479.5 masl

NOC = 486.0 masl

No	DESCRIPTION	UNIT	RATE May 08 Rand	QUANTITY	AMOUNT Rand
1	**Clearing**				
	(a) sparse	ha	5 227	1.61	8 416
	(b) bush	ha	10 454	7.25	75 742
	(c) trees	ha	20 909	7.25	151 484
2	**River diversion**	Sum			20 000 000
3	**Excavation**				
	(a) Bulk				
	(i) all materials	m^3	30	508 902	15 114 388
	(ii) extra over for rock	m^3	55	100 000	5 500 000
	(b) Preparation of solum				
	(i) all materials	m^2	14	41 681	596 038
	(II) extra over for rock	m^2	14	16 729	239 225
4	**Drilling & Grouting**				
	(a) Curtain grouting	m drill	770	11 790	9 078 300
	(b) Consolidation grouting	m drill	770	7 020	5 405 400
5	**Embankment**				
	(a) Earthfill	m^3	34	1 069 418	36 467 150
	(b) Filters & transition	m^3	124	67 095	8 339 909
	(c) Rip-rap & rock toe	m^3	75	90 666	6 781 817
	(d) Overhaul beyond 3km	m^3km	4	3 155 220	13 882 968
6	**Concrete Works**				
	(a) Formwork				
	(i) gang formed	m^2	238	37 632	8 956 416
	(ii) intricate	m^2	498	2 233	1 112 034
	(b) Concrete				
	(i) RCC	m^3	513	220 700	113 108 750
	(ii) mass	m^3	1 338	6 195	8 288 910
	(iii) structural	m^3	1 693	9 217	15 599 773
	(c) Reinforcing	t	11 762	830	9 756 517
7	**Mechanical Items**				
	(a) Valves & gates	Sum			11 000 000
	(b) Cranes & hoists	Sum			5 500 000
	(c) Structural steelwork	t	19 360	10	193 600
	SUB-TOTAL				295 156 836

No	DESCRIPTION	UNIT	RATE May 08 RAND	QUANTITY	AMOUNT
8	**Landscaping** (% of 1-7)	%	5	295 156 836	14 757 842
9	**Miscellaneous** (% of 1-7)	%	10	295 156 836	29 515 684
	SUB TOTAL A				339 430 361
10	**Preliminary & General** (% of sub-total A)	%	40	339 430 361	135 772 144
11	**Relocation of roads**				
	(a) D1292	Sum			98 000 000
	(b) P43-3	Sum			36 000 000
	SUB TOTAL B				609 202 505
12	**Contingencies** (% of sub total B)	%	10	609 202 505	60 920 251
	SUB TOTAL C				670 122 756
13	**Planning design & supervision** (% of sub total C)	%	15	670 122 756	100 518 413
	TOTAL COST (excl. VAT)				**770 641 169**

APPENDIX A.2

COST ESTIMATE FOR TROUGH SPILLWAY

NWAMITWA DAM WITH TROUGH SPILLWAY

FSL = 479.5 masl

NOC = 486.0 masl

No	DESCRIPTION	UNIT	RATE May 08 Rand	QUANTITY	AMOUNT Rand
1	**Clearing**				
	(a) sparse	ha	5 227	2.02	10 559
	(b) bush	ha	10 454	9.09	95 030
	(c) trees	ha	20 909	9.09	190 061
2	**River diversion**	Sum			20 000 000
3	**Excavation**				
	(a) Bulk				
	(i) all materials	m^3	30	421 333	12 513 590
	(ii) extra over for rock	m^3	55	100 000	5 500 000
	(b) Preparation of solum				
	(i) all materials	m^2	14	43 346	619 848
	(II) extra over for rock	m^2	14	15 890	227 227
4	**Drilling & Grouting**				
	(a) Curtain grouting	m drill	770	11 790	9 078 300
	(b) Consolidation grouting	m drill	770	7 020	5 405 400
5	**Embankment**				
	(a) Earthfill	m^3	34	1 573 851	53 668 312
	(b) Filters & transition	m^3	124	83 729	10 407 477
	(c) Rip-rap & rock toe	m^3	75	107 025	8 005 433
	(d) Overhaul beyond 3km	m^3km	4	3 815 064	16 786 282
6	**Concrete Works**				
	(a) Formwork				
	(i) gang formed	m^2	238	25 819	6 144 922
	(ii) intricate	m^2	498	3 713	1 849 074
	(b) Concrete				
	(i) RCC	m^3	513	116 530	59 721 625
	(ii) mass	m^3	1 338	160 930	215 324 340
	(iii) structural	m^3	1 693	6 500	11 001 250
	(c) Reinforcing	t	11 762	585	6 880 478
7	**Mechanical Items**				
	(a) Valves & gates	Sum			11 000 000
	(b) Cranes & hoists	Sum			5 500 000
	(c) Structural steelwork	t	19 360	10	193 600
	SUB-TOTAL				460 122 808

No	DESCRIPTION	UNIT	RATE May 08 RAND	QUANTITY	AMOUNT
8	**Landscaping** (% of 1-7)	%	5	460 122 808	23 006 140
9	**Miscellaneous** (% of 1-7)	%	10	460 122 808	46 012 281
	SUB TOTAL A				529 141 229
10	**Preliminary & General** (% of sub-total A)	%	40	529 141 229	211 656 492
11	**Relocation of roads**				
	(a) D1292	Sum			98 000 000
	(b) P43-3	Sum			36 000 000
	SUB TOTAL B				874 797 720
12	**Contingencies** (% of sub total B)	%	10	874 797 720	87 479 772
	SUB TOTAL C				962 277 492
13	**Planning design & supervision** (% of sub total C)	%	15	962 277 492	144 341 624
	TOTAL COST (excl. VAT)				**1 106 619 116**

APPENDIX A.3

COST ESTIMATE FOR LABYRINTH SPILLWAY

NWAMITWA DAM WITH LABYRINTH SPILLWAY

FSL = 479.5 masl

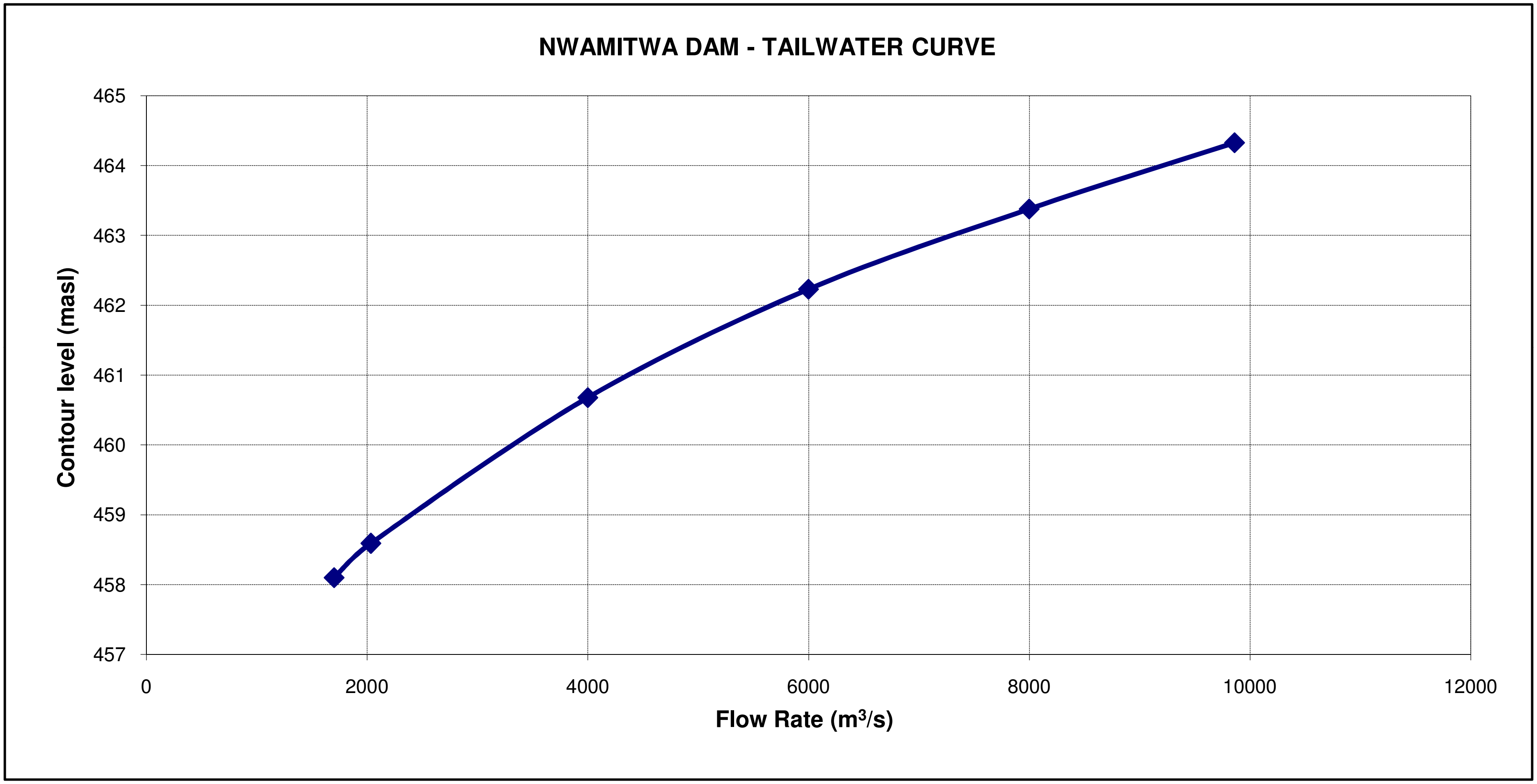
NOC = 486.0 masl

No	DESCRIPTION	UNIT	RATE May 08 Rand	QUANTITY	AMOUNT Rand
1	**Clearing**				
	(a) sparse	ha	5 227	1.72	8 991
	(b) bush	ha	10 454	7.74	80 917
	(c) trees	ha	20 909	7.74	161 834
2	**River diversion**	Sum			20 000 000
3	**Excavation**				
	(a) Bulk				
	(i) all materials	m^3	30	444 060	13 188 582
	(ii) extra over for rock	m^3	55	100 000	5 500 000
	(b) Preparation of solum				
	(i) all materials	m^2	14	42 965	614 400
	(II) extra over for rock	m^2	14	17 552	250 994
4	**Drilling & Grouting**				
	(a) Curtain grouting	m drill	770	11 790	9 078 300
	(b) Consolidation grouting	m drill	770	7 020	5 405 400
5	**Embankment**				
	(a) Earthfill	m^3	34	1 141 938	38 940 086
	(b) Filters & transition	m^3	124	70 038	8 705 723
	(c) Rip-rap & rock toe	m^3	75	93 597	7 001 056
	(d) Overhaul beyond 3km	m^3km	4	3 272 700	14 399 880
6	**Concrete Works**				
	(a) Formwork				
	(i) gang formed	m^2	238	29 234	6 957 702
	(ii) intricate	m^2	498	9 483	4 722 534
	(b) Concrete				
	(i) RCC	m^3	513	248 181	127 192 763
	(ii) mass	m^3	1 338	6 195	8 288 910
	(iii) structural incl labyrinth walls	m^3	1 693	17 095	28 933 288
	(c) Reinforcing	t	11 762	1 836	21 591 644
7	**Mechanical Items**				
	(a) Valves & gates	Sum			11 000 000
	(b) Cranes & hoists	Sum			5 500 000
	(c) Structural steelwork	t	19 360	10	193 600
	SUB-TOTAL				337 716 601

No	DESCRIPTION	UNIT	RATE May 08 RAND	QUANTITY	AMOUNT
8	**Landscaping** (% of 1-7)	%	5	337 716 601	16 885 830
9	**Miscellaneous** (% of 1-7)	%	10	337 716 601	33 771 660
	SUB TOTAL A				388 374 092
10	**Preliminary & General** (% of sub-total A)	%	40	388 374 092	155 349 637
11	**Relocation of roads**				
	(a) D1292	Sum			98 000 000
	(b) P43-3	Sum			36 000 000
	SUB TOTAL B				677 723 728
12	**Contingencies** (% of sub total B)	%	10	677 723 728	67 772 373
	SUB TOTAL C				745 496 101
13	**Planning design & supervision** (% of sub total C)	%	15	745 496 101	111 824 415
	TOTAL COST (excl. VAT)				**857 320 516**

APPENDIX B

TAILWATER CURVE

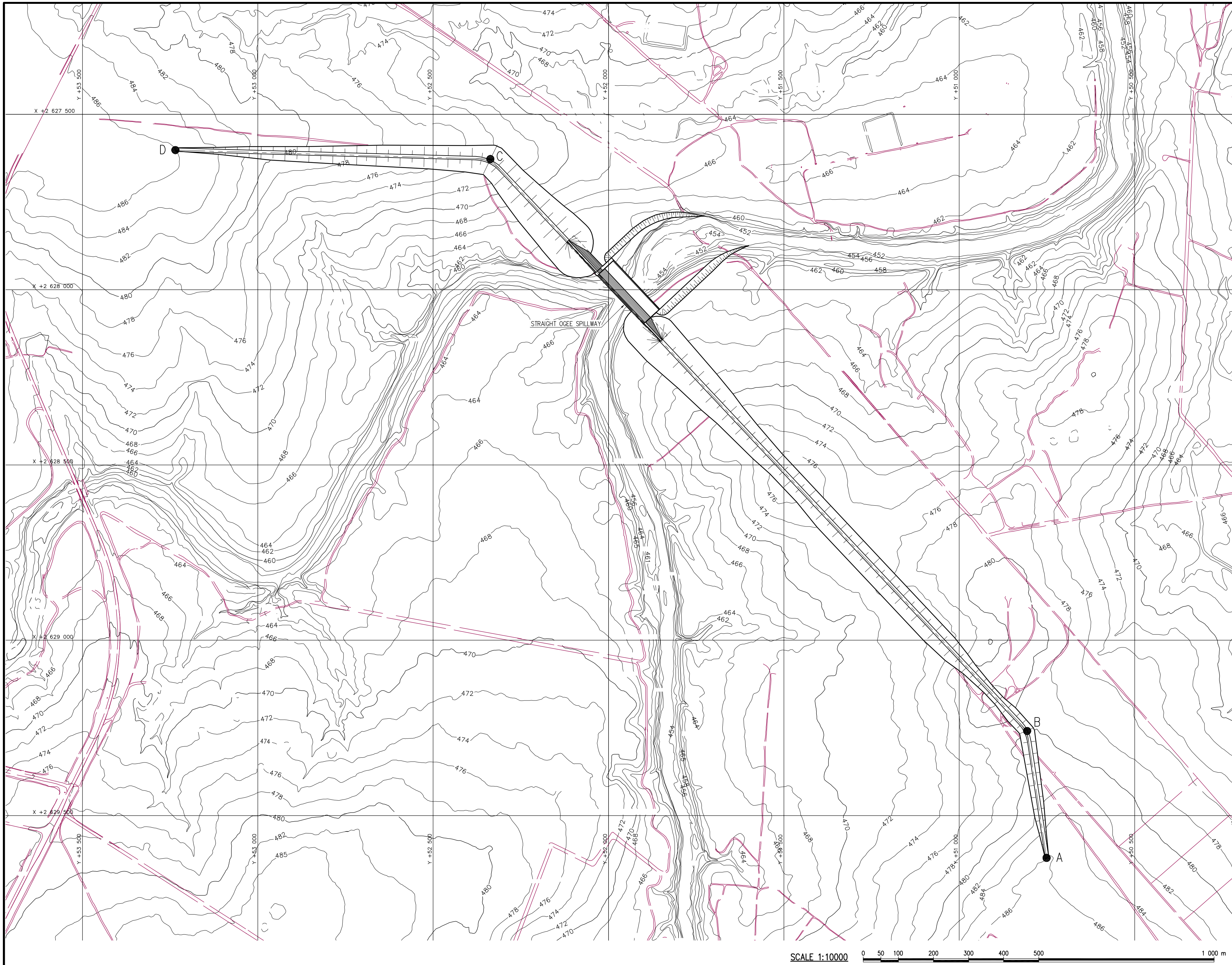

NWAMITWA DAM - TAILWATER CURVE
Contour level (masl)
465
464
463
462
461
460
459
458
457
0
2000
4000
6000
8000
10000
12000
Flow Rate (m³/s)

APPENDIX C

DRAWINGS

APPENDIX C.1

GENERAL LAYOUT OF DAM



CO-ORDINATES
LO 31 (CAPE DATUM)

POINT No	Y	X
A	50 751.10	2 629 620.50
B	50 806.44	2 629 258.81
C	52 336.94	2 627 627.54
D	53 235.00	2 627 602.01

NOTES:

1. PRINCIPAL PARAMETERS OF LAYOUT:
   FSL 479.5 masl
   NOC 486.0 masl

No	AMENDMENTS	APPR.	DATE

DESIGNED	HS
DESIGN CHECKED	DIvW
DRAWN	RB
DRAWING CHECKED	DIvW

PROJECT MANAGER
DATE

Pr Eng
PROJECT DIRECTOR
DATE

NINHAM SHAND
FOUNDED IN 1932
CONSULTING SERVICES
CENTURION

GROOT LETABA RIVER
DEVELOPMENT PROJECT
BRIDGING STUDIES

NWAMITWA DAM

PROPOSED LAYOUT

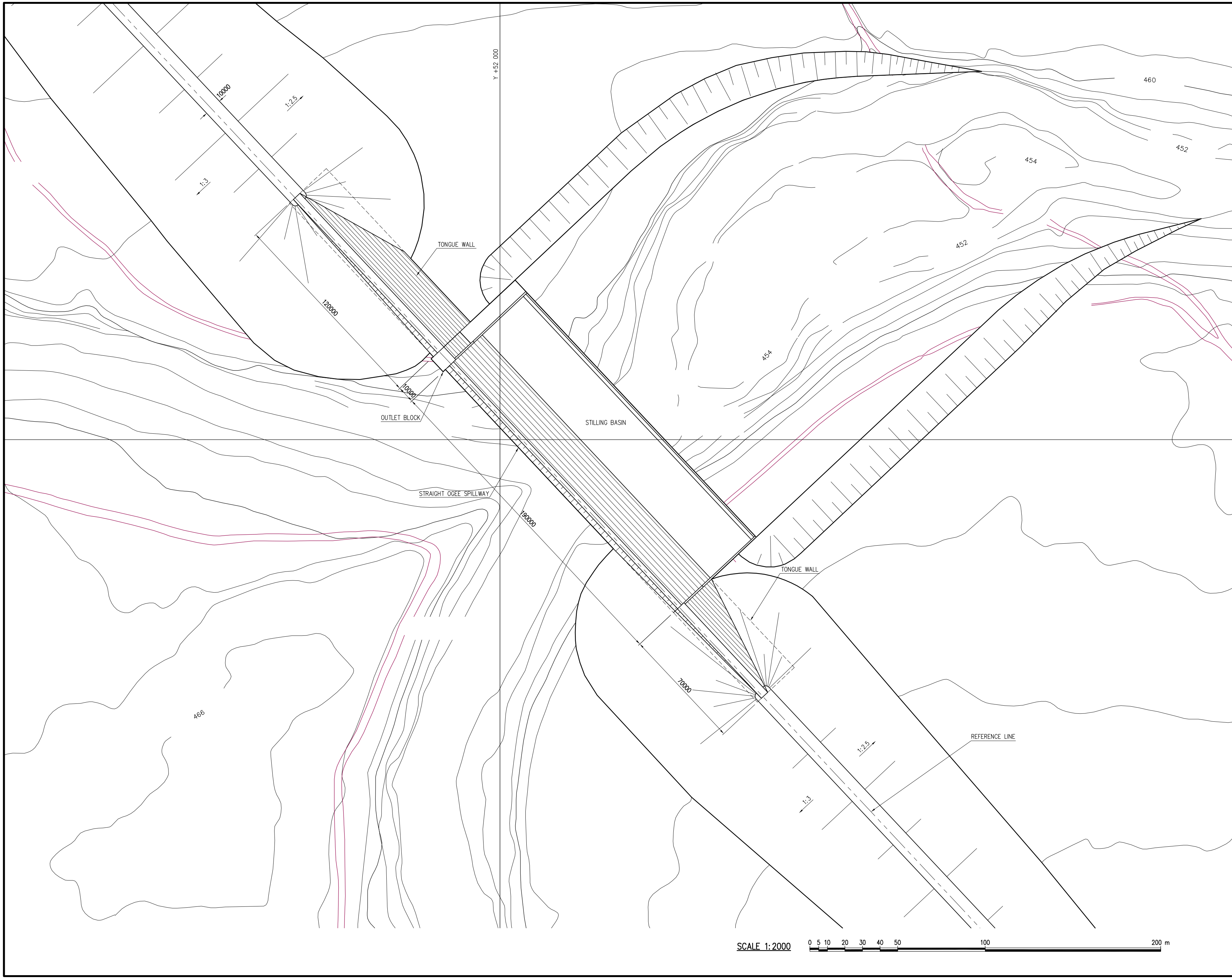
SCALE AS SHOWN

SHEET 1 OF 1

DRAWING No
401775 CEN 10

APPENDIX C.2

STRAIGHT OGEE SPILLWAY



LEGEND:

OGL = ORIGINAL GROUND LEVEL
ARL = ASSUMED ROCK LEVEL
NOC = NON OVERSPILL CREST
FSL = FULL SUPPLY LEVEL
TWL = TAIL WATER LEVEL
RDF = RECOMMENDED DESIGN FLOOD

No	AMENDMENTS	APPR.	DATE

DESIGNED	HS
DESIGN CHECKED	DivW
DRAWN	RB
DRAWING CHECKED	DivW

PROJECT MANAGER
DATE

Pr Eng
PROJECT DIRECTOR
DATE

NINHAM SHAND
FOUNDED IN 1932
CONSULTING SERVICES
CENTURION

GROOT LETABA RIVER
DEVELOPMENT PROJECT
BRIDGING STUDIES

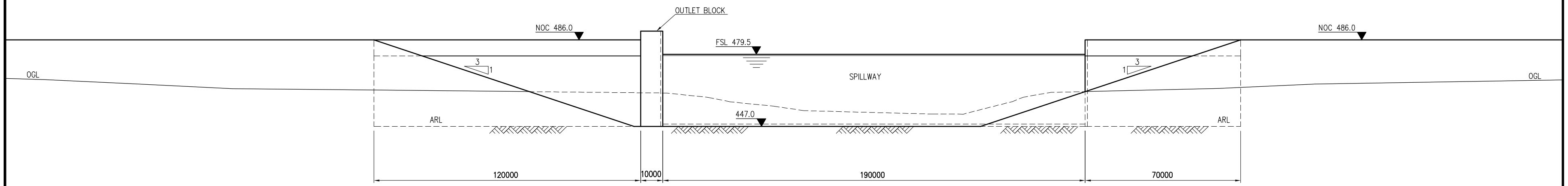
NWAMITWA DAM

STRAIGHT OGEE SPILLWAY
LAYOUT

SCALE
AS SHOWN

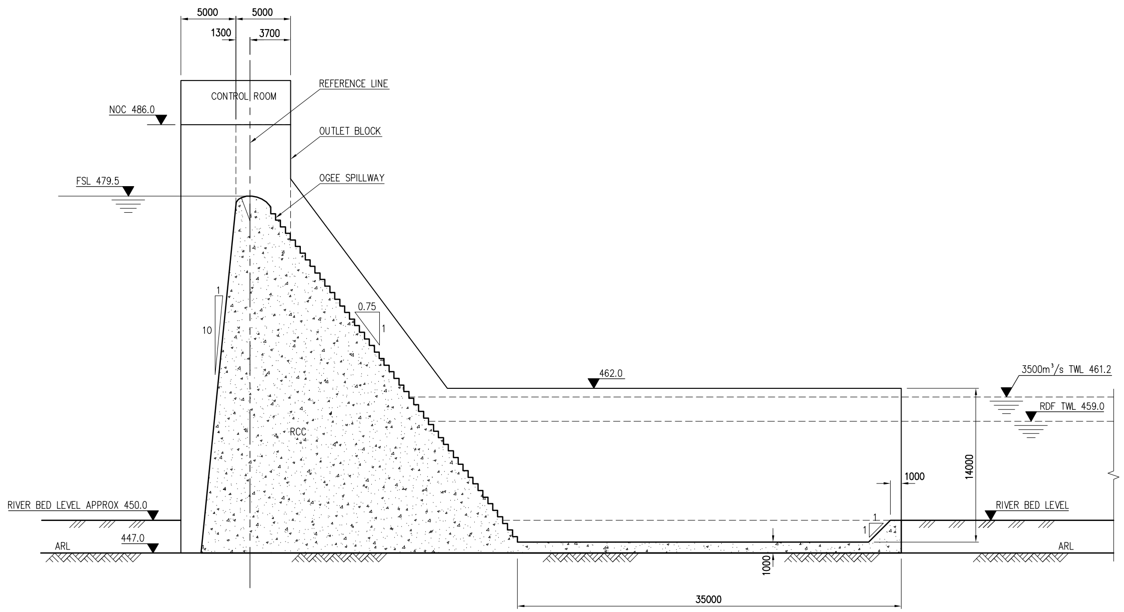
SHEET 1 OF 1

DRAWING No
401775 CEN 11



UPSTREAM ELEVATION

SCALE 1:2000



SECTION THROUGH SPILLWAY

SCALE 1:500

SCALE 1:500 0 5 10 15 20 25 50 m

SCALE 1:2000 0 5 10 20 30 40 50 100 200 m

LEGEND:

OGL = ORIGINAL GROUND LEVEL
ARL = ASSUMED ROCK LEVEL
NOC = NON OVERSPILL CREST
FSL = FULL SUPPLY LEVEL
TWL = TAIL WATER LEVEL
RDF = RECOMMENDED DESIGN FLOOD

No	AMENDMENTS	APPR.	DATE

DESIGNED	HS
DESIGN CHECKED	DivW
DRAWN	RB
DRAWING CHECKED	DivW

PROJECT MANAGER
DATE

Pr Eng
PROJECT DIRECTOR
DATE

NINHAM SHAND
FOUNDED IN 1932
CONSULTING SERVICES
CENTURION

GROOT LETABA RIVER DEVELOPMENT PROJECT BRIDGING STUDIES

NWAMITWA DAM

STRAIGHT OGEE SPILLWAY SECTIONS

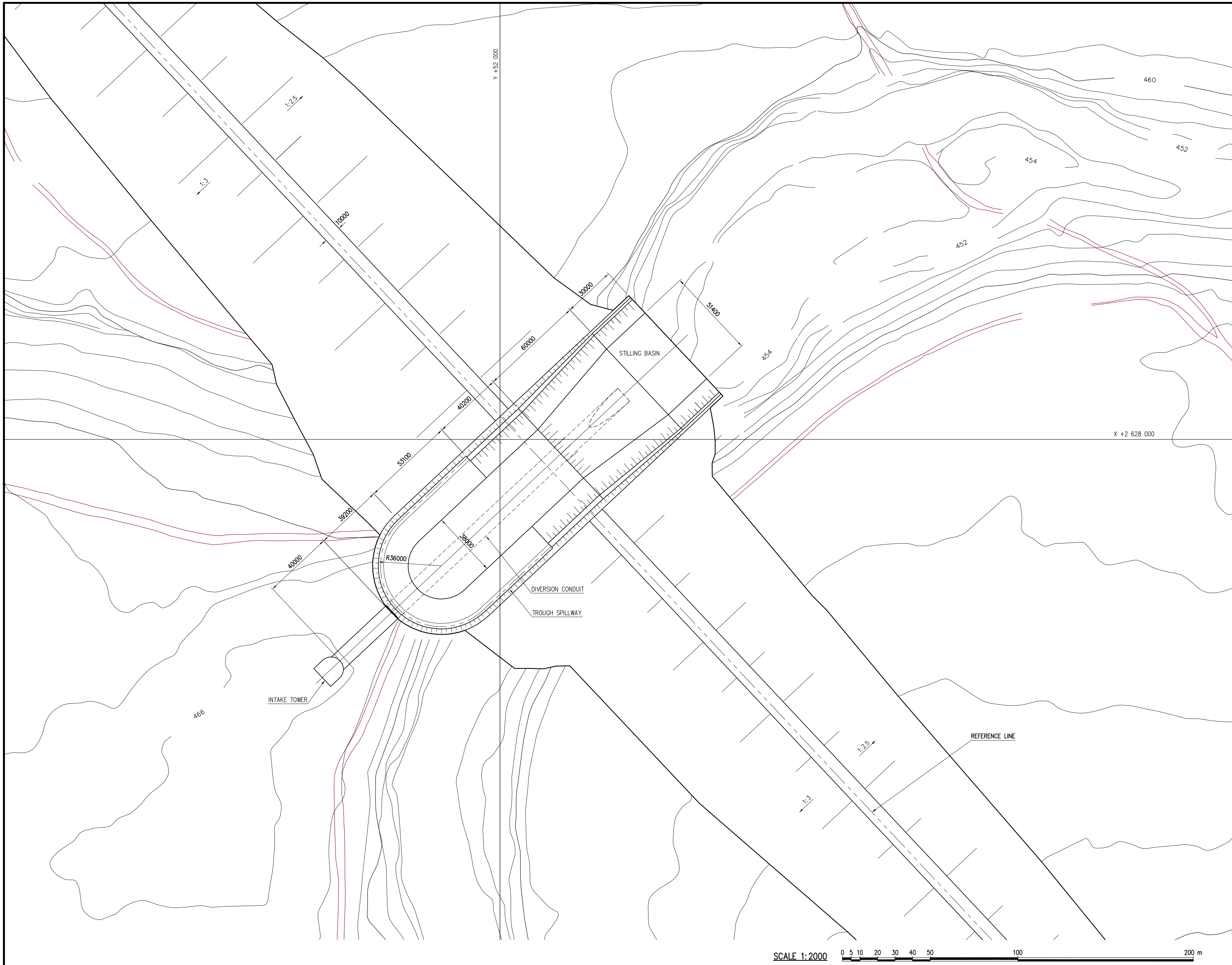
SCALE AS SHOWN

SHEET 1 OF 1

DRAWING No
401775 CEN 12

APPENDIX C.3

TROUGH SPILLWAY



LEGEND:

OGL = ORIGINAL GROUND LEVEL
ARL = ASSUMED ROCK LEVEL
NOC = NON OVERSPILL CREST
FSL = FULL SUPPLY LEVEL
TWL = TAIL WATER LEVEL
RDF = RECOMMENDED DESIGN FLOOD

No	AMENDMENTS	APPR.	DATE

DESIGNED	HS
DESIGN CHECKED	DivW
DRAWN	RB
DRAWING CHECKED	DivW

PROJECT MANAGER
DATE

Pr Eng
PROJECT DIRECTOR
DATE

NINHAM SHAND
FOUNDED IN 1932
CONSULTING SERVICES
CENTURION

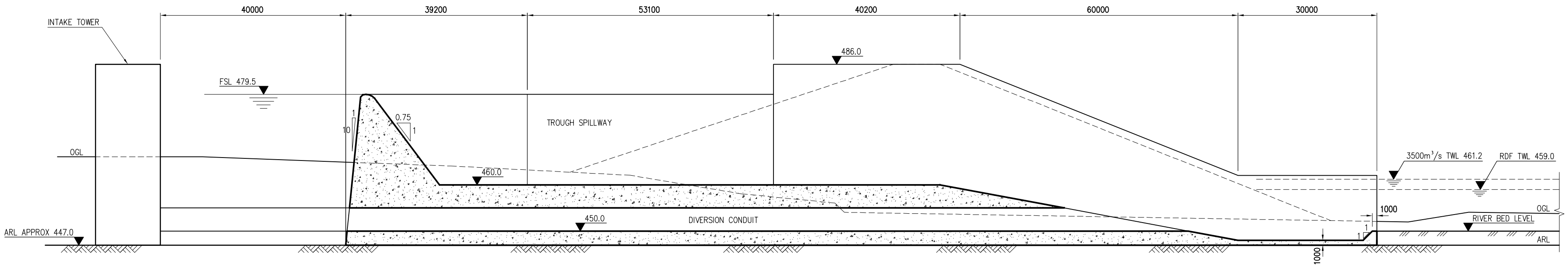
GROOT LETABA RIVER DEVELOPMENT PROJECT BRIDGING STUDIES

NWAMITWA DAM

TROUGH SPILLWAY LAYOUT

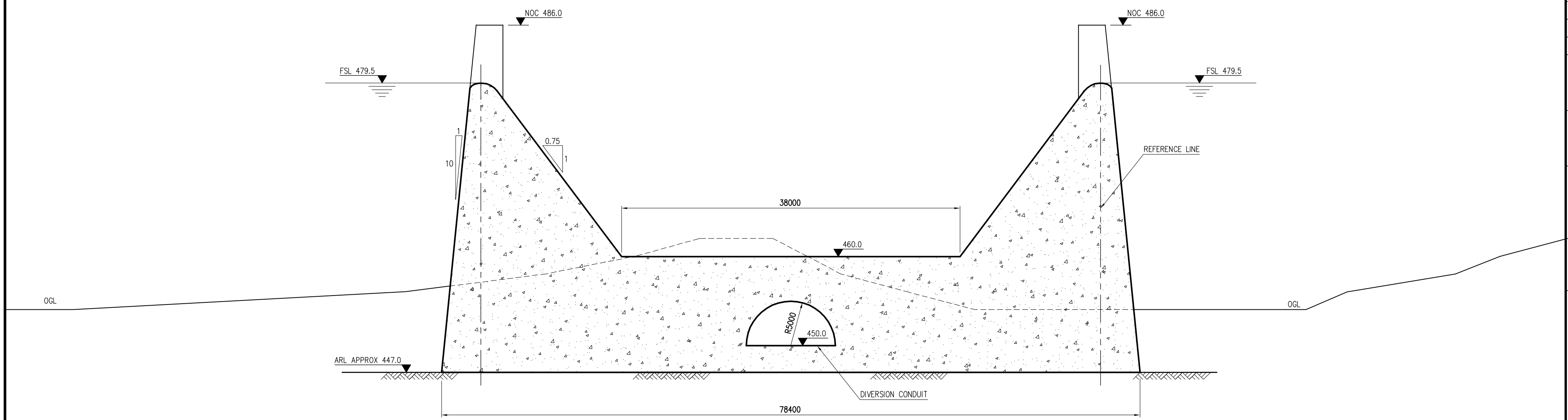
SCALE AS SHOWN
SHEET 1 OF 1

DRAWING No
401775 CEN 13



TROUGH SPILLWAY LONG SECTION

SCALE 1:1000



SECTION THROUGH SPILLWAY

SCALE 1:500

SCALE 1:500 — 0, 5, 10, 15, 20, 25, 50 m

SCALE 1:1000 — 0, 5, 10, 20, 30, 40, 50, 100 m

LEGEND:

OGL = ORIGINAL GROUND LEVEL
ARL = ASSUMED ROCK LEVEL
NOC = NON OVERSPILL CREST
FSL = FULL SUPPLY LEVEL
TWL = TAIL WATER LEVEL
RDF = RECOMMENDED DESIGN FLOOD

No	AMENDMENTS	APPR.	DATE

DESIGNED	HS
DESIGN CHECKED	DIvW
DRAWN	RB
DRAWING CHECKED	DIvW

PROJECT MANAGER
DATE

Pr Eng
PROJECT DIRECTOR
DATE

NINHAM SHAND
FOUNDED IN 1932
CONSULTING SERVICES
CENTURION

GROOT LETABA RIVER DEVELOPMENT PROJECT BRIDGING STUDIES

NWAMITWA DAM

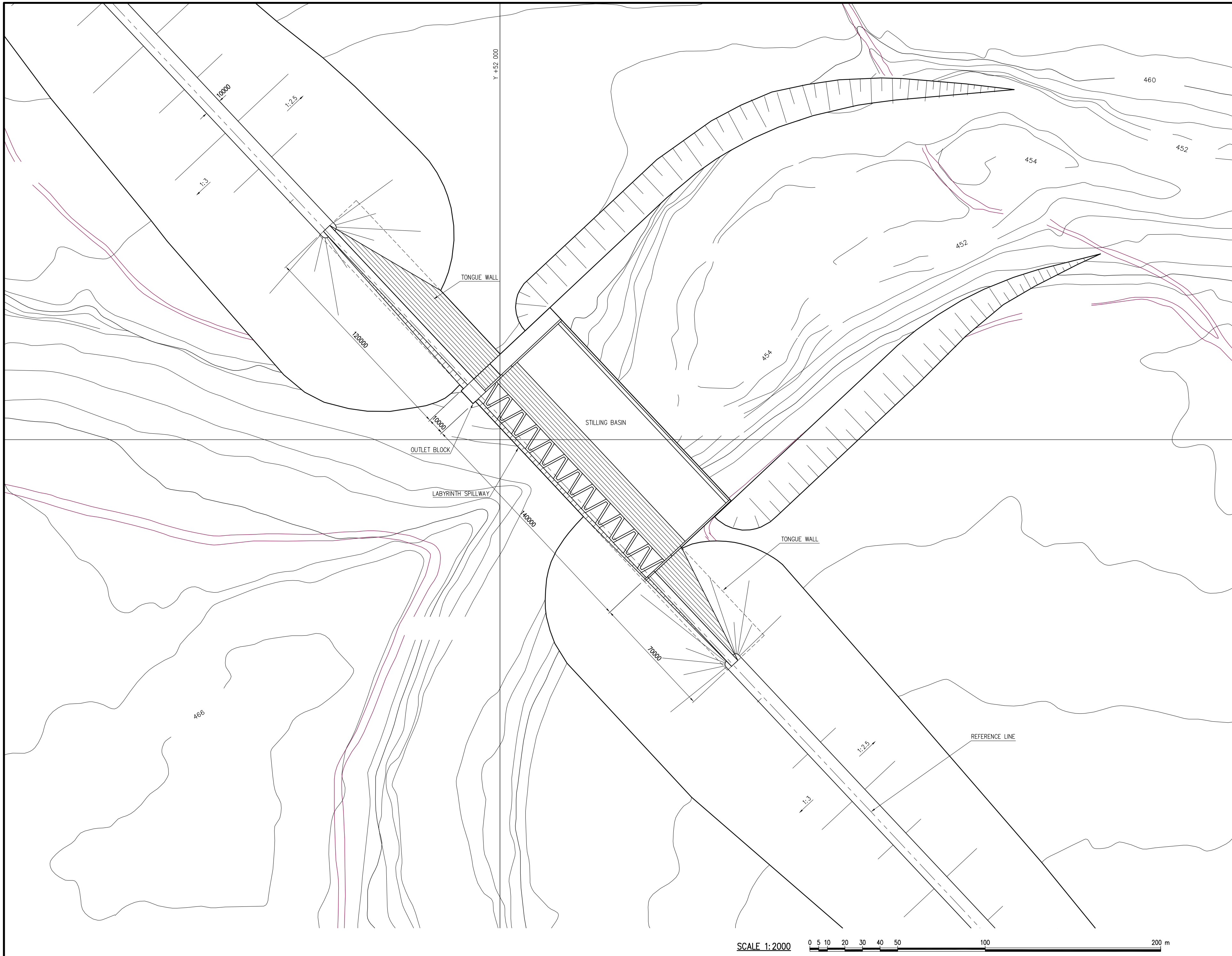
TROUGH SPILLWAY SECTIONS

SCALE AS SHOWN | SHEET 1 OF 1

DRAWING No
401775 CEN 14

APPENDIX C.4

LABYRINTH SPILLWAY



LEGEND:

OGL = ORIGINAL GROUND LEVEL
ARL = ASSUMED ROCK LEVEL
NOC = NON OVERSPILL CREST
FSL = FULL SUPPLY LEVEL
TWL = TAIL WATER LEVEL
RDF = RECOMMENDED DESIGN FLOOD

No	AMENDMENTS	APPR.	DATE

DESIGNED	HS
DESIGN CHECKED	DivW
DRAWN	RB
DRAWING CHECKED	DivW

PROJECT MANAGER
DATE

Pr Eng
PROJECT DIRECTOR
DATE

NINHAM SHAND
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CONSULTING SERVICES
CENTURION

GROOT LETABA RIVER
DEVELOPMENT PROJECT
BRIDGING STUDIES

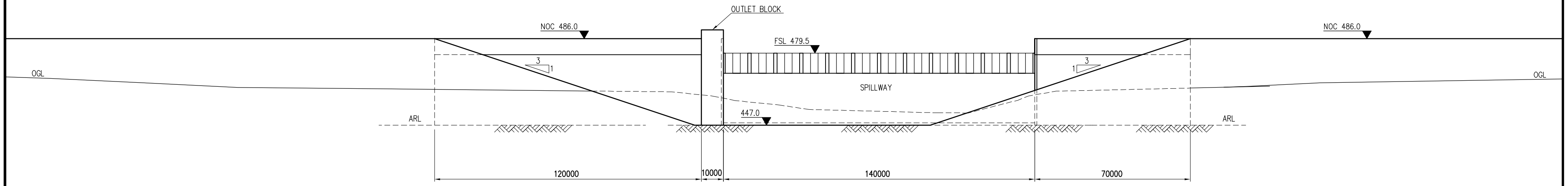
NWAMITWA DAM

LABYRINTH SPILLWAY
LAYOUT

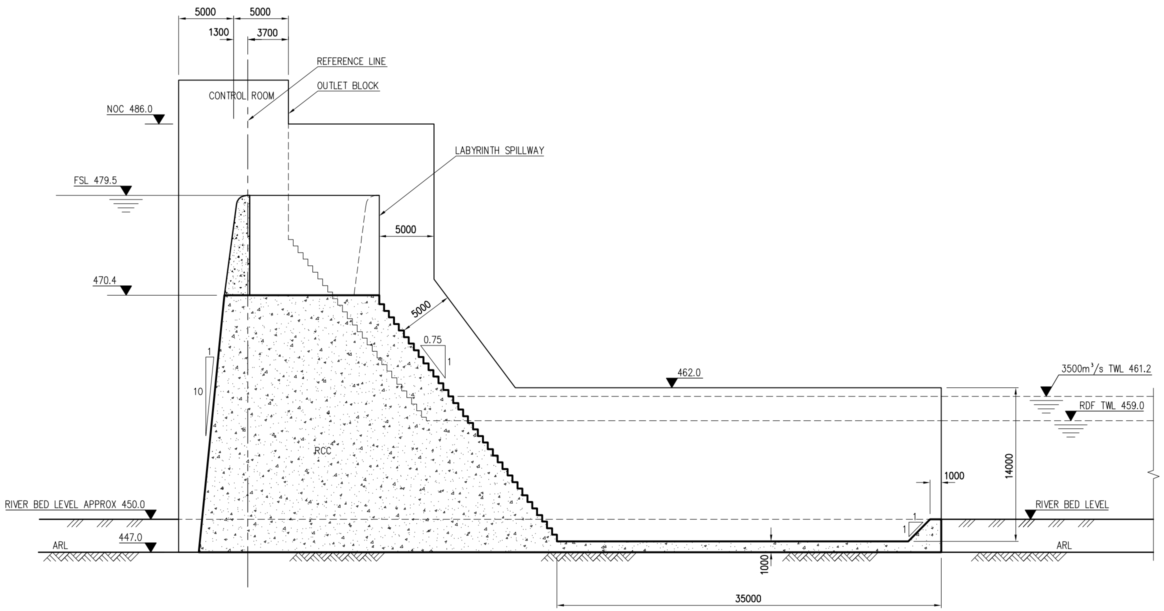
SCALE
AS SHOWN

SHEET 1 OF 1

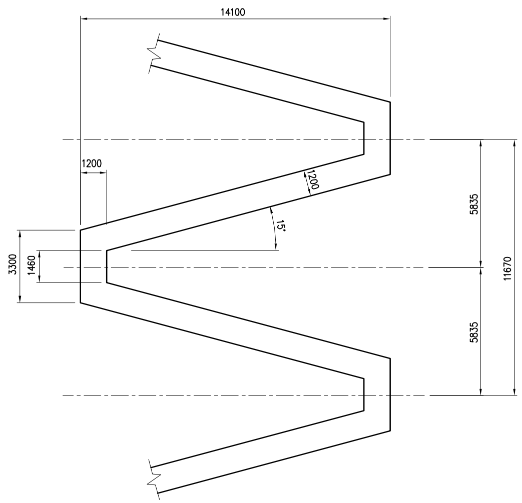
DRAWING No
401775 CEN 15



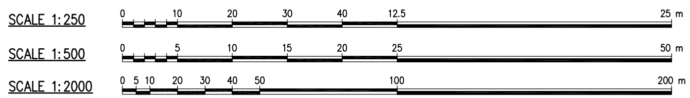
UPSTREAM ELEVATION
SCALE 1:2000



SECTION THROUGH SPILLWAY
SCALE 1:500



DETAIL OF LABYRINTH
SCALE 1:250



LEGEND:

- OGL = ORIGINAL GROUND LEVEL
- ARL = ASSUMED ROCK LEVEL
- NOC = NON OVERSPILL CREST
- FSL = FULL SUPPLY LEVEL
- TWL = TAIL WATER LEVEL
- RDF = RECOMMENDED DESIGN FLOOD

No	AMENDMENTS	APPR.	DATE

DESIGNED	HS
DESIGN CHECKED	DivW
DRAWN	RB
DRAWING CHECKED	DivW

PROJECT MANAGER
DATE

Pr Eng
PROJECT DIRECTOR
DATE

NINHAM SHAND
FOUNDED IN 1932
CONSULTING SERVICES
CENTURION

GROOT LETABA RIVER
DEVELOPMENT PROJECT
BRIDGING STUDIES

NWAMITWA DAM

LABYRINTH SPILLWAY
SECTIONS

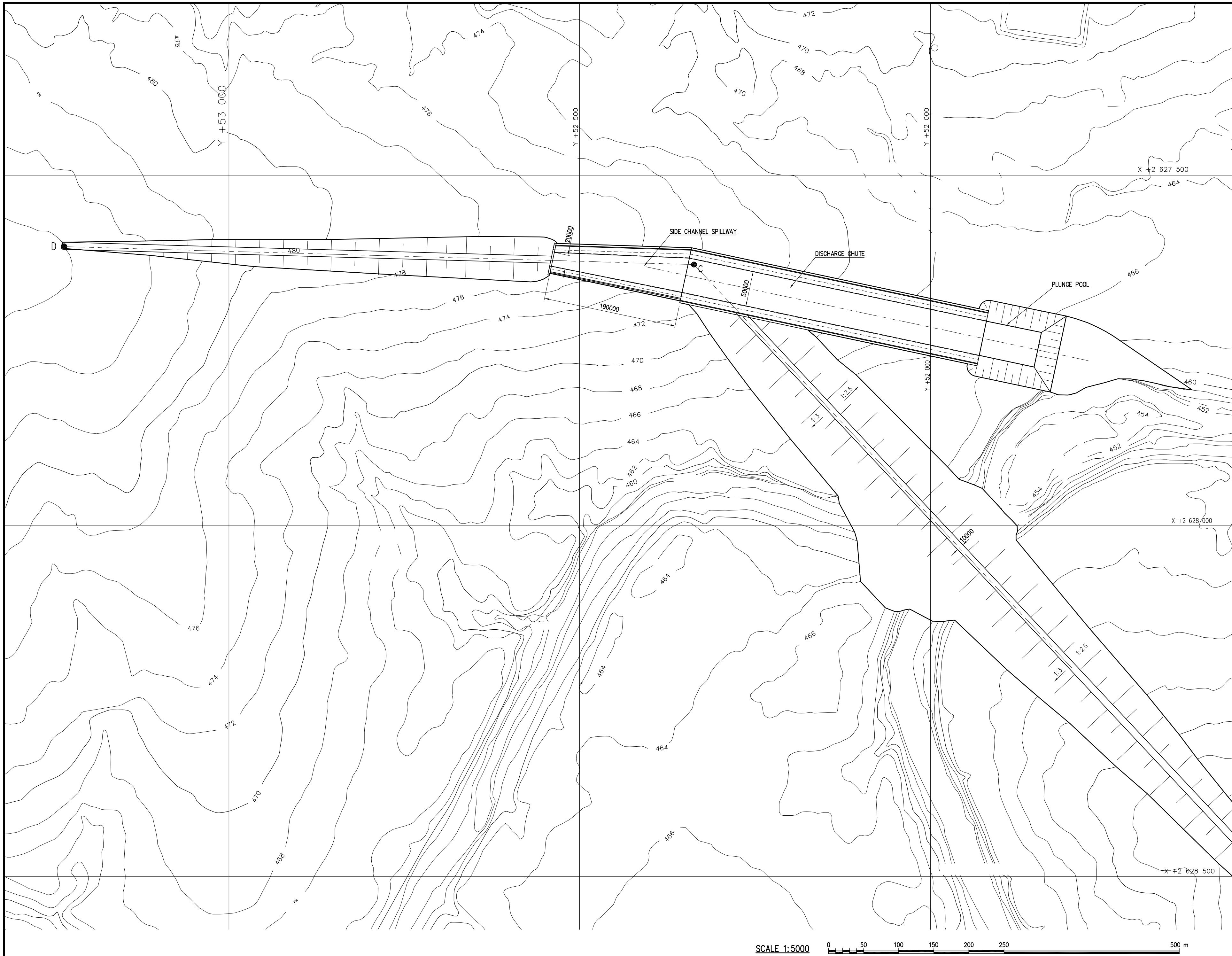
SCALE
AS SHOWN

SHEET 1 OF 1

DRAWING No
401775 CEN 16

APPENDIX C.5

SIDE CHANNEL SPILLWAY



LEGEND:

OGL = ORIGINAL GROUND LEVEL
ARL = ASSUMED ROCK LEVEL
NOC = NON OVERSPILL CREST
FSL = FULL SUPPLY LEVEL
TWL = TAIL WATER LEVEL
RDF = RECOMMENDED DESIGN FLOOD

No	AMENDMENTS	APPR.	DATE

DESIGNED	HS
DESIGN CHECKED	DivW
DRAWN	RB
DRAWING CHECKED	DivW

PROJECT MANAGER
DATE

Pr Eng
PROJECT DIRECTOR
DATE

NINHAM SHAND
FOUNDED IN 1932
CONSULTING SERVICES
CENTURION

GROOT LETABA RIVER
DEVELOPMENT PROJECT
BRIDGING STUDIES

NWAMITWA DAM

SIDE CHANNEL SPILLWAY
LAYOUT

SCALE
AS SHOWN

SHEET 1 OF 1

DRAWING No
401775 CEN 17

APPENDIX D.2 : Spillway Stage Discharge Curve

Spillway Stage Discharge Curve

H Selected	**Elevation**	**C**	**Effective Channel Width (L)**	**Flow Q**
		Assumed to be constant	190 + 40%*(0.25*H)	Q = C*L*(Hd)^1.5
0.00	479.50	2.18	190.00	0.00
0.20	479.70	2.18	190.00	37.05
0.40	479.90	2.18	190.00	104.79
0.60	480.10	2.18	190.00	192.50
0.80	480.30	2.18	190.00	296.38
1.00	480.50	2.18	190.00	414.20
1.20	480.70	2.18	190.00	544.48
1.40	480.90	2.18	190.00	686.12
1.60	481.10	2.18	190.00	838.28
1.80	481.30	2.18	190.00	1000.27
2.00	481.50	2.18	190.00	1171.53
2.20	481.70	2.18	190.00	1351.59
2.40	481.90	2.18	190.00	1540.02
2.60	482.10	2.18	190.00	1736.48
2.80	482.30	2.18	190.00	1940.65
3.00	482.50	2.18	190.00	2152.25
3.20	482.70	2.18	190.00	2371.02
3.40	482.90	2.18	190.00	2596.74
3.60	483.10	2.18	190.00	2829.20
3.80	483.30	2.18	190.00	3068.21
4.00	483.50	2.18	190.00	3313.60
4.20	483.70	2.18	190.00	3565.20
4.40	483.90	2.18	190.00	3822.87
4.60	484.10	2.18	190.00	4086.46
4.80	484.30	2.18	190.00	4355.84
5.00	484.50	2.18	190.00	4630.90
5.20	484.70	2.18	190.00	4911.51
5.40	484.90	2.18	190.00	5197.57
5.60	485.10	2.18	190.00	5488.99
5.80	485.30	2.18	190.00	5785.65
6.00	485.50	2.18	190.00	6087.47
6.20	485.70	2.18	190.00	6394.37
6.40	485.90	2.18	190.00	6706.25
6.46	485.96	2.18	190.00	6800.00

APPENDIX D.3 : Spillway Energy Dissipation and Stilling Basin Calculations

APPENDIX D.3.1 : NWAMITWA DAM STEPPED SPILLWAY ENERGY DISSIPATION

Reference Comparison of energy dissipation between nappe and skimmimg flow regimes on stepped chutes"
H Chanson, Journal of Hydraulic Research, Vol 32, 1994, No 2

Symbol	Unit	Calc	Comments
Q	m^3/s	1860	Discharge over 190m long spillway
H_{dam}	m	29.5	Spillway height above downstream toe
H	m	2.72	Water depth over spillway
H_{max}	m	32.22	Total head
d_c	m	1.81	
f		1.3	Friction factor - mean value based on experimental data
α	degrees	53	Spillway slope for 1V:0.75H
α	radians	0.93	
First term		0.35415641	(f/(8*SIN α)^0.333)*COS α
Second term		1.445329405	0.5/(f/(8*SIN α))^(2/3)
Third term		16.92399052	$2/3+H_{dam}/d_c$
∆H		28.80	
Remaining head		3.43	

TABLE OF FLOW VELOCITIES AND DEPTHS FOR DIFFERENT Q's

Q m^3/s	**Remaining H** m	v_1 m/s	d_1 m
200	0.75	3.84	0.27
400	1.19	4.83	0.44
600	1.57	5.55	0.57
800	1.91	7.29	0.58
1000	2.23	6.61	0.80
1500	2.95	7.61	1.04
1860	3.43	8.20	1.19
2000	3.60	8.40	1.25

APPENDIX D.3.2 : NWAMITWA DAM STILLING BASIN

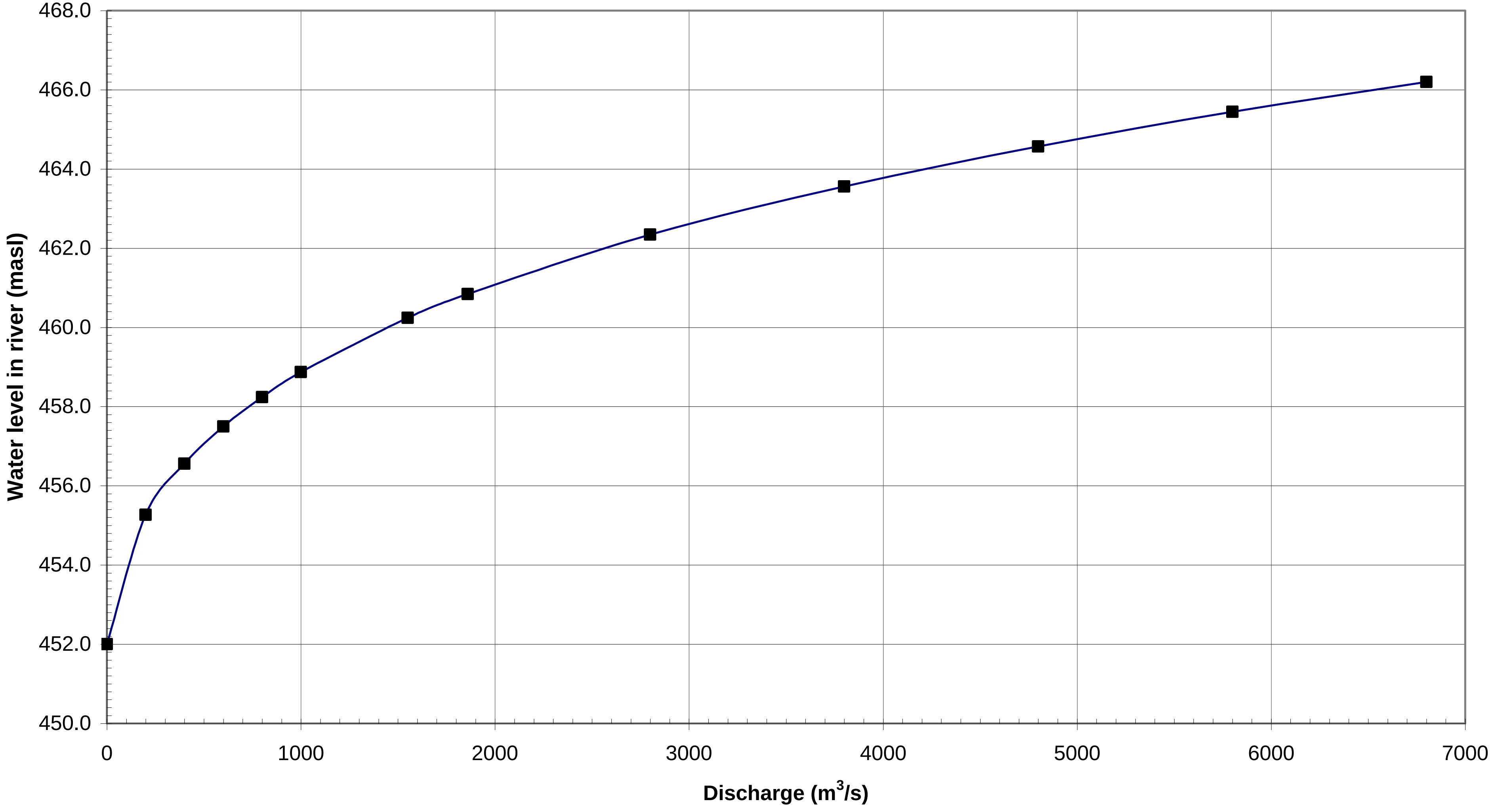
Calculation of stilling basin dimensions for low flows

Stepped spillway			
Q	**m^3/s**	**1860**	**RDF**
Froude number			
v	m/s	8.20	Flow velocity from energy dissipation calcs ex Chanson
y1	m	1.19	Flow depth before stilling basin (Q=V*A)
Fr		2.40	Fr = v/(g*y1)^0.5
Stilling basin dimensions			
Conjugate depth y2		**3.49**	y2 = 0.5*(((8*Fr^2 +1)^0.5-1)*y1)
Basin length L/y2		5.30	Figure 12 Eng Monograph No 25 for Type 1 stilling basins (natural jump)
Basin length L		**18.5**	
Specific discharge	$m^3/s.m$	16.17	

Q m^3/s	**y2** m	**L** m	**TWL** m
200	0.78	4.0	3.00
400	1.24	6.3	4.00
600	1.47	8.5	5.00
800	2.23	12.0	5.70
1000	2.29	11.9	6.40
1500	3.01	16.0	7.60
1860	3.49	18.5	8.30
2000	3.67	19.4	8.50

APPENDIX D.4 : Tailwater Curve

APPENDIX D.4 TAILWATER CURVE



APPENDIX D.5 : River Diversion Water Profile Calculations

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CPF version 3.01f                 N              Ninham Shand                        Page : 1
Nwamitwa                                         401775                              Date : 23/04/2010
River Diversion                                  IAB                                 Time : 09:52:50 AM

Run Information

Data File ....................... : I:\WEDG\401775-3\T-TECH-1\04-TAS-1\047007-1.6-R\CFP\NWAMRDIV.OUT
Run label ....................... : Run Title
Calculation interval dRV ........ :    20.000
Upstream Depth .................. :     0.000
Downstream Depth ................ :     0.000
Mult. Factors - Discharge ....... :  1250.000
              - Manning's N ..... :     1.000
Hydraulic jump calculations ..... :     Area * Centroid
Transition loss coef. applied ... :     No
No. of cross sections ........... :    11
Conveyance calculation type ..... : Individual calc - adjacent bed segments(Type 1)

Long Section Data:

 Section     Stake      Unit        Transition          Section Title
   No.       Value      Flow     Loss Div.  Coef.Conv.
              [m]     [cumecs]     [-]        [-]
     1       0.000     0.670      0.00       0.00      Grt Letaba-500m US of CL
     2     255.000     0.670      0.00       0.00      Start of Channel
     3     355.000     0.670      0.00       0.00      Inside Channel
     4     604.000     0.670      0.00       0.00      US of Nwanedzi confluence
     5     605.000     1.000      0.00       0.00      Confluence
     6     850.000     1.000      0.00       0.00      Embankment CL
     7    1200.000     1.000      0.00       0.00      Inside Channel
     8    1285.000     1.000      0.00       0.00      End of Channel
     9    1525.000     1.000      0.00       0.00      Grt Letaba-500m DS of CL
    10    2025.000     1.000      0.00       0.00      Grt Letaba-1000m DS of CL
    11    2500.000     1.000      0.00       0.00      Grt Letaba-1495m DS of CL

Cross Section Data:

Section No. :   1  Stake Value :     0.000   Title : Grt Letaba-500m US of CL
  Chainage     Level   Mannings n
     0.000   464.000     0.035
    15.000   462.000     0.035
    30.000   460.000     0.035
    40.000   458.000     0.035
    50.000   456.000     0.035
    51.000   454.000     0.035
    75.000   454.100     0.035
    93.000   454.000     0.035
    95.000   456.000     0.035
   100.000   458.000     0.035
   105.000   460.000     0.035
   110.000   462.000     0.035
   146.000   464.000     0.035

Section No. :   2  Stake Value :   255.000   Title : Start of Channel
  Chainage     Level   Mannings n
     0.000   464.000     0.035
    17.900   462.570     0.035
    35.000   454.000     0.025
   100.000   454.000     0.035
   120.000   464.000     0.035

Section No. :   3  Stake Value :   355.000   Title : Inside Channel
  Chainage     Level   Mannings n
     0.000   466.000     0.025
    24.400   453.800     0.025
    54.400   453.800     0.025
    78.800   466.000     0.025

Section No. :   4  Stake Value :   604.000   Title : US of Nwanedzi confluence
  Chainage     Level   Mannings n
     0.000   466.000     0.025
    25.400   453.322     0.025
    55.400   453.322     0.025
    80.720   466.000     0.025

Section No. :   5  Stake Value :   605.000   Title : Confluence
  Chainage     Level   Mannings n
     0.000   466.000     0.025
    25.400   453.320     0.025
    55.400   453.320     0.025
    80.720   466.000     0.025

Section No. :   6  Stake Value :   850.000   Title : Embankment CL
  Chainage     Level   Mannings n
     0.000   466.000     0.025
    35.000   464.000     0.025
    42.200   463.600     0.025
    65.000   452.840     0.025
    95.000   452.840     0.025
   111.000   460.180     0.025
   115.000   466.000     0.025

Section No. :   7  Stake Value :  1200.000   Title : Inside Channel
  Chainage     Level   Mannings n
     0.000   466.000     0.025
    35.000   464.000     0.025
    42.200   463.600     0.025
    65.000   452.200     0.025
    95.000   452.200     0.025
   111.000   460.180     0.025
   115.000   466.000     0.025

Section No. :   8  Stake Value :  1285.000   Title : End of Channel
  Chainage     Level   Mannings n
     0.000   466.000     0.035
    65.000   464.000     0.035
    95.700   462.600     0.035
   117.000   452.000     0.025
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Nwamitwa 401775 Date : 21/04/2010
River Diversion LAB Time : 09:52:58 AM

Cross Section Data:

165.000	452.000	0.035
175.000	454.000	0.025
235.000	454.000	0.035
240.000	452.000	0.035
255.000	454.000	0.035
260.000	456.000	0.035
270.000	458.000	0.035
275.000	460.000	0.035
280.000	462.000	0.035

Section No. : 9 Stake Value : 1525.000 Title : Grt Letaba-500m DS of CL

Chainage	Level	Mannings n
0.000	466.000	0.035
50.000	464.000	0.035
100.000	462.000	0.035
120.000	460.000	0.035
130.000	458.000	0.035
135.000	456.000	0.035
140.000	454.000	0.035
148.000	452.000	0.035
159.000	451.160	0.035
170.000	452.000	0.035
185.000	454.000	0.035
195.000	456.000	0.035
210.000	458.000	0.035
225.000	458.000	0.035
230.000	458.000	0.035
240.000	460.000	0.035
245.000	462.000	0.035
265.000	462.000	0.035
295.000	462.000	0.035
380.000	464.000	0.035
445.000	466.000	0.035

Section No. : 10 Stake Value : 2025.000 Title : Grt Letaba-1000m DS of CL

Chainage	Level	Mannings n
0.000	464.000	0.035
109.000	462.000	0.035
145.000	460.000	0.035
158.000	458.000	0.035
163.000	456.000	0.035
175.000	454.000	0.035
177.000	452.000	0.035
200.000	450.930	0.035
220.000	452.000	0.035
225.000	454.000	0.035
228.000	456.000	0.035
240.000	458.000	0.035
250.000	460.000	0.035
260.000	462.000	0.035
315.000	464.000	0.035

Section No. : 11 Stake Value : 2500.000 Title : Grt Letaba-1495m DS of CL

Chainage	Level	Mannings n
0.000	462.000	0.035
65.000	461.000	0.035
90.000	461.000	0.035
100.000	460.000	0.035
103.000	458.000	0.035
120.000	456.000	0.035
132.000	454.000	0.035
140.000	452.000	0.035
155.000	450.780	0.035
175.000	452.000	0.035
185.000	454.000	0.035
190.000	456.000	0.035
195.000	458.000	0.035
198.000	460.000	0.035
208.000	462.000	0.035

Final Profiles :

Point	Stake Value [m]	Bed Level [m]	Depth [m]	Surface level [m]	Surface Width [m]	Area [m^2]	Discharge [m^3/s]	Velocity [m/s]	Energy level [m]	Froude No. [-]	Comments :
1	0.000	454.000	7.14	461.14	86.40	417.17	837.50	2.01	461.35	0.29	Grt Letaba-500m US of CL
2	21.250	454.000	7.14	461.14	86.99	429.52	837.50	1.95	461.33	0.28	
3	42.500	454.000	7.14	461.14	87.59	441.99	837.50	1.89	461.32	0.27	
4	63.750	454.000	7.14	461.14	88.20	454.53	837.50	1.84	461.31	0.26	
5	85.000	454.000	7.14	461.14	88.80	467.11	837.50	1.79	461.31	0.25	
6	106.250	454.000	7.15	461.15	89.41	479.72	837.50	1.75	461.30	0.24	
7	127.500	454.000	7.15	461.15	90.02	492.36	837.50	1.70	461.30	0.23	
8	148.750	454.000	7.15	461.15	90.63	505.00	837.50	1.66	461.29	0.22	
9	170.000	454.000	7.15	461.15	91.25	517.65	837.50	1.62	461.29	0.22	
10	191.250	454.000	7.16	461.16	91.83	530.30	837.50	1.58	461.28	0.21	
11	212.500	454.000	7.16	461.16	92.43	542.95	837.50	1.54	461.28	0.20	
12	233.750	454.000	7.16	461.16	93.03	555.60	837.50	1.51	461.28	0.20	
13	255.000	454.000	7.16	461.16	93.62	568.24	837.50	1.47	461.27	0.19	Start of Channel
14	275.000	453.960	7.18	461.14	86.69	519.47	837.50	1.61	461.27	0.21	
15	295.000	453.920	7.19	461.11	79.70	469.81	837.50	1.78	461.27	0.23	
16	315.000	453.880	7.18	461.06	72.71	419.11	837.50	2.00	461.26	0.27	
17	335.000	453.840	7.15	460.99	65.80	367.07	837.50	2.28	461.26	0.31	
18	355.000	453.800	7.08	460.88	56.34	313.11	837.50	2.67	461.25	0.37	Inside Channel
19	375.750	453.760	7.12	460.88	56.47	315.07	837.50	2.66	461.24	0.37	
20	396.500	453.720	7.15	460.87	58.61	317.05	837.50	2.64	461.23	0.36	
21	417.250	453.680	7.19	460.87	58.75	319.04	837.50	2.63	461.22	0.36	
22	438.000	453.641	7.22	460.86	58.89	321.05	837.50	2.61	461.21	0.36	
23	458.750	453.601	7.26	460.86	59.02	323.07	837.50	2.59	461.20	0.35	
24	479.500	453.561	7.29	460.85	59.16	325.10	837.50	2.58	461.19	0.35	
25	500.250	453.521	7.32	460.85	59.30	327.16	837.50	2.56	461.18	0.35	
26	521.000	453.481	7.36	460.84	59.44	329.24	837.50	2.54	461.17	0.35	
27	541.750	453.442	7.39	460.84	59.58	331.33	837.50	2.53	461.16	0.34	
28	562.500	453.402	7.43	460.83	59.72	333.41	837.50	2.51	461.15	0.34	
29	583.250	453.362	7.46	460.83	59.86	335.49	837.50	2.50	461.14	0.34	

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Nwamitwa 403775 Date : 21/04/2010
River Diversion IAB Time : 09:52:50 AM

Final Profiles :

Point	Stake Value (m)	Bed Level (m)	Depth (m)	Surface level (m)	Surface Width (m)	Area (m^2)	Discharge (m^3/s)	Velocity (m/s)	Energy level (m)	Froude No. (-)	Comments :
30	604.000	453.322	7.50	460.82	60.00	337.57	827.50	2.46	461.13	0.33	US of Nwanedzi confluence
31	605.000	453.320	6.17	459.49	54.68	261.39	1250.00	4.78	460.66	0.70	Confluence
32	625.417	453.280	6.18	459.46	54.88	262.47	1250.00	4.76	460.62	0.70	
33	645.833	453.240	6.19	459.43	55.08	263.58	1250.00	4.74	460.58	0.69	
34	666.250	453.200	6.20	459.40	55.28	264.73	1250.00	4.72	460.54	0.69	
35	686.667	453.160	6.22	459.38	55.49	265.91	1250.00	4.70	460.50	0.69	
36	707.083	453.120	6.23	459.35	55.70	267.13	1250.00	4.68	460.47	0.68	
37	727.500	453.080	6.24	459.32	55.91	268.39	1250.00	4.66	460.43	0.68	
38	747.917	453.040	6.26	459.30	56.13	269.69	1250.00	4.63	460.39	0.68	
39	768.333	453.000	6.27	459.27	56.34	271.02	1250.00	4.61	460.36	0.67	
40	788.750	452.960	6.29	459.25	56.57	272.41	1250.00	4.59	460.32	0.67	
41	809.167	452.920	6.31	459.23	56.79	273.83	1250.00	4.56	460.29	0.66	
42	829.583	452.880	6.32	459.20	57.02	275.28	1250.00	4.54	460.25	0.66	
43	850.000	452.840	6.34	459.18	57.25	276.78	1250.00	4.52	460.22	0.66	Embankment CL
44	870.588	452.802	6.34	459.14	57.15	276.51	1250.00	4.52	460.18	0.66	
45	891.176	452.765	6.34	459.11	57.04	276.24	1250.00	4.53	460.15	0.66	
46	911.765	452.727	6.34	459.07	56.94	275.96	1250.00	4.53	460.12	0.66	
47	932.353	452.689	6.34	459.03	56.84	275.67	1250.00	4.53	460.08	0.66	
48	952.941	452.652	6.35	459.00	56.73	275.38	1250.00	4.54	460.05	0.66	
49	973.529	452.614	6.35	458.96	56.63	275.08	1250.00	4.54	460.01	0.66	
50	994.118	452.576	6.35	458.92	56.52	274.78	1250.00	4.55	459.98	0.66	
51	1014.706	452.539	6.35	458.89	56.41	274.47	1250.00	4.55	459.94	0.66	
52	1035.294	452.501	6.35	458.85	56.31	274.16	1250.00	4.56	459.91	0.66	
53	1055.882	452.464	6.35	458.81	56.20	273.83	1250.00	4.56	459.88	0.66	
54	1076.471	452.426	6.35	458.78	56.09	273.50	1250.00	4.57	459.84	0.66	
55	1097.059	452.388	6.35	458.74	55.98	273.16	1250.00	4.58	459.81	0.66	
56	1117.647	452.351	6.35	458.70	55.87	272.82	1250.00	4.58	459.77	0.66	
57	1138.235	452.313	6.35	458.66	55.76	272.46	1250.00	4.59	459.74	0.66	
58	1158.824	452.275	6.35	458.63	55.65	272.10	1250.00	4.59	459.70	0.66	
59	1179.412	452.238	6.35	458.59	55.54	271.73	1250.00	4.60	459.67	0.66	
60	1200.000	452.200	6.35	458.55	55.43	271.35	1250.00	4.61	459.63	0.66	Inside Channel
61	1221.750	452.150	7.18	459.35	86.31	468.09	1250.00	2.68	459.61	0.37	
62	1243.500	452.100	7.31	459.41	115.13	643.55	1250.00	1.96	459.60	0.26	
63	1263.750	452.050	7.43	459.48	143.57	818.45	1250.00	1.53	459.60	0.20	
64	1285.000	452.000	7.52	459.52	171.90	994.71	1250.00	1.26	459.60	0.17	End of Channel
65	1305.000	451.930	7.58	459.51	166.77	953.79	1250.00	1.31	459.59	0.17	
66	1325.000	451.860	7.64	459.50	161.69	912.00	1250.00	1.37	459.59	0.18	
67	1345.000	451.790	7.69	459.48	156.66	869.31	1250.00	1.44	459.59	0.19	
68	1365.000	451.720	7.75	459.47	151.66	825.68	1250.00	1.51	459.58	0.21	
69	1385.000	451.650	7.80	459.45	146.70	781.06	1250.00	1.60	459.58	0.22	
70	1405.000	451.580	7.85	459.43	141.76	735.36	1250.00	1.70	459.58	0.24	
71	1425.000	451.510	7.90	459.41	136.82	688.47	1250.00	1.82	459.57	0.26	
72	1445.000	451.440	7.93	459.37	131.87	640.23	1250.00	1.95	459.57	0.28	
73	1465.000	451.370	7.96	459.33	126.87	590.37	1250.00	2.12	459.56	0.31	
74	1485.000	451.300	7.97	459.27	121.76	538.41	1250.00	2.32	459.55	0.35	
75	1505.000	451.230	7.96	459.19	116.43	483.43	1250.00	2.59	459.53	0.41	
76	1525.000	451.160	7.90	459.06	110.58	423.29	1250.00	2.95	459.50	0.48	Grt Letaba-500m DS of CL
77	1545.000	451.151	7.87	459.02	109.52	421.88	1250.00	2.96	459.47	0.48	
78	1565.000	451.142	7.84	458.98	108.45	420.55	1250.00	2.97	459.43	0.48	
79	1585.000	451.132	7.81	458.94	107.39	419.32	1250.00	2.98	459.39	0.48	
80	1605.000	451.123	7.78	458.90	106.32	418.18	1250.00	2.99	459.36	0.48	
81	1625.000	451.114	7.75	458.87	105.26	417.15	1250.00	3.00	459.32	0.48	
82	1645.000	451.105	7.72	458.83	104.20	416.21	1250.00	3.00	459.29	0.48	
83	1665.000	451.096	7.70	458.79	103.14	415.38	1250.00	3.01	459.25	0.48	
84	1685.000	451.086	7.67	458.76	102.08	414.65	1250.00	3.01	459.22	0.48	
85	1705.000	451.077	7.64	458.72	101.03	414.02	1250.00	3.02	459.18	0.48	
86	1725.000	451.068	7.62	458.68	99.97	413.50	1250.00	3.02	459.15	0.47	
87	1745.000	451.059	7.59	458.65	98.92	413.09	1250.00	3.03	459.12	0.47	
88	1765.000	451.050	7.57	458.62	97.88	412.78	1250.00	3.03	459.08	0.47	
89	1785.000	451.040	7.54	458.58	96.84	412.57	1250.00	3.03	459.05	0.47	
90	1805.000	451.031	7.52	458.55	95.80	412.47	1250.00	3.03	459.02	0.47	
91	1825.000	451.022	7.50	458.52	94.77	412.48	1250.00	3.03	458.99	0.46	
92	1845.000	451.013	7.47	458.49	93.74	412.59	1250.00	3.03	458.95	0.46	
93	1865.000	451.004	7.45	458.46	92.71	412.80	1250.00	3.03	458.92	0.46	
94	1885.000	450.994	7.42	458.43	91.69	413.11	1250.00	3.03	458.89	0.46	
95	1905.000	450.985	7.41	458.40	90.63	413.54	1250.00	3.02	458.86	0.45	
96	1925.000	450.976	7.39	458.37	89.55	414.10	1250.00	3.02	458.83	0.45	
97	1945.000	450.967	7.37	458.34	88.47	414.73	1250.00	3.01	458.80	0.44	
98	1965.000	450.958	7.36	458.31	87.46	415.44	1250.00	3.01	458.77	0.44	
99	1985.000	450.948	7.34	458.29	86.50	416.22	1250.00	3.00	458.75	0.44	
100	2005.000	450.939	7.32	458.26	85.58	417.07	1250.00	3.00	458.72	0.43	
101	2025.000	450.930	7.31	458.24	84.71	417.98	1250.00	2.99	458.69	0.43	Grt Letaba-1000m DS of CL
102	2045.652	450.920	7.28	458.20	84.76	414.56	1250.00	3.02	458.66	0.44	
103	2066.304	450.910	7.25	458.16	84.79	411.03	1250.00	3.04	458.63	0.44	
104	2086.957	450.900	7.23	458.13	84.82	407.38	1250.00	3.07	458.60	0.45	
105	2107.609	450.890	7.19	458.08	84.83	403.59	1250.00	3.10	458.57	0.45	
106	2128.261	450.880	7.16	458.04	84.55	399.78	1250.00	3.13	458.54	0.46	
107	2148.913	450.870	7.13	458.00	85.08	395.82	1250.00	3.16	458.51	0.47	
108	2169.565	450.860	7.09	457.95	85.18	391.67	1250.00	3.19	458.47	0.48	
109	2190.217	450.850	7.06	457.91	85.25	387.32	1250.00	3.23	458.44	0.48	
110	2210.870	450.840	7.03	457.86	85.28	382.73	1250.00	3.27	458.40	0.49	
111	2231.522	450.830	6.98	457.81	85.27	377.89	1250.00	3.31	458.36	0.50	
112	2252.174	450.820	6.93	457.75	85.22	372.75	1250.00	3.35	458.32	0.51	
113	2272.826	450.810	6.88	457.69	85.11	367.27	1250.00	3.40	458.28	0.52	
114	2293.478	450.800	6.83	457.63	84.95	361.41	1250.00	3.46	458.24	0.54	
115	2314.130	450.790	6.77	457.56	84.71	355.12	1250.00	3.52	458.19	0.55	
116	2334.783	450.780	6.71	457.49	84.37	348.54	1250.00	3.59	458.14	0.56	
117	2355.435	450.770	6.64	457.41	83.83	341.41	1250.00	3.66	458.09	0.58	
118	2376.087	450.760	6.56	457.32	83.38	333.60	1250.00	3.75	458.04	0.60	
119	2396.739	450.750	6.47	457.22	82.70	324.94	1250.00	3.85	457.98	0.62	
120	2417.391	450.740	6.37	457.11	81.82	315.11	1250.00	3.97	457.91	0.65	
121	2438.043	450.730	6.25	456.98	80.67	303.63	1250.00	4.12	457.84	0.68	
122	2458.696	450.720	6.10	456.82	79.10	290.21	1250.00	4.31	457.76	0.72	
123	2479.348	450.710	5.89	456.60	76.82	272.79	1250.00	4.58	457.67	0.78	
124	2500.000	450.700	5.48	456.18	72.21	241.17	1250.00	5.18	457.55	0.91	Grt Letaba-1495m DS of ClCtrl

APPENDIX E – Outlet Works

E1 Water Quality Report

APPENDIX E.1 : Water Quality Report

GROOT LETABA WATER DEVELOPMENT PROJECT : BRIDGING STUDIES – TECHNICAL STUDY (GLEWAP)

WATER QUALITY REPORT

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1. WATER QUALITY MODELLING

1.1 INTRODUCTION

The Groot Letaba River and its tributaries, the Klein Letaba, Middle Letaba, Letsitele and Molototsi Rivers, drain an area of approximately 13 400 km^2 in the Limpopo Province. After the confluence with the Middle and Klein Letaba River, the Groot Letaba flows through Kruger National Park (KNP) where it meets up with the Olifants River close to the Mozambican border. There are over 20 major dams along the watercourse and the system is therefore highly regulated (DWAF, 2006).

The land uses taking place and their probable impacts on water quality have been outlined in the Reserve Determination Study that was carried for the Department of Water Affairs and Forestry (DWAF) in 2006. The main land uses identified in the catchment are intensive irrigated commercial agriculture, particularly of sub-tropical fruits on the banks of the Groot Letaba and afforestation in the mountainous upper parts. The instream dams presently supply the intensive irrigation taking place. Water quality problems associated with the irrigation agriculture are salinisation and release of biocides into the environment, while the impacts associated with the forestry sector are increased turbidity due to erosion and sedimentation.

The industrial activity taking place in the catchment is not expected to impact considerably on water quality as most of the effluent is recycled or used for irrigation. There is also increasing domestic demand being placed on the resource especially with increasing living standards in the area. Furthermore, the area has dense settlements and informal settlements which give rise to the discharge of sewage effluent into rivers which is likely to cause eutrophication problems (DWAF, 2006).

The KNP served by the Letaba in the downstream reaches, is a prominent conservation area and is a contributor to the tourism industry. For this reason the flow regimes need to be sustained by releases from the numerous dams for the protection of aquatic biota, riparian vegetation and terrestrial life. Additionally, there is an international obligation to release water of a reasonable quality standard to Mozambique.

According to the Inception Report for this study, the competition for water in the Letaba Catchment by the various sectors has led to a scarcity in the resource and prevalence of severe water restrictions over the past 25 years. Additionally, a considerable portion of the population does not have access to basic services and agricultural development has been put on hold.

A Feasibility Study of the development and management options for the Groot Letaba River (1998) proposed the construction of a dam at Nwamitwa and the possible raising of Tzaneem Dam as options for augmenting water supply from the Groot Letaba River. The DWAF now has to re-assess these proposals.

One of the tasks that have emerged and is the purpose of this study is a water quality analysis of the proposed Nwamitwa Dam to inform the design of the outlet structure of the dam, as well as the mitigating effects of installing a multi-level outlet structure. The Feasibility Study states that it is expected that the water impounded on the Groot Letaba River will stratify during the summer months resulting in an anaerobic hypolimnion and aerobic epilimnion. The water quality task will assess water quality impacts and will inform the preliminary design and operation of the outlet structures. The hydrodynamic and water quality model CE-QUAL-W2 will be configured and used to inform the above study objectives.

The model outputs will include:

- In-lake temperature profiles for the hypolimnion and epilimnion
- Temperatures of releases from various outlet arrangements

The Instream Flow Requirements (IRF) outlined in the Reserve Determination Study will inform the water quality task of the ecological temperature requirements downstream of the dam wall. In terms of the "optimised scenario" that was selected to guide the regulation of flows to meet the IRF, water quality is not expected to change considerably. Therefore the outlet structure should be designed so as not to impact considerably on the present temperature conditions of the system.

The layout of this report includes a background of the CE-QUAL-W2 model, the application to the Groot Letaba Catchment, followed by the results of the modelling and discussion, as well as conclusions and recommendations.

1.2 BACKGROUND TO CE-QUAL-W2 MODEL

CE-QUAL-W2 (Cole and Wells, 2001) is a two-dimensional (2-D), laterally averaged, hydrodynamic and water quality simulation model. The model is based on the assumption that the water body shows maximum variation in water quality along its length and depth. Therefore, the model is suited to relatively long and narrow water bodies that show water quality gradients in the longitudinal and vertical directions. The two-dimensional model simulates the vertical and longitudinal distributions of thermal energy (water temperature) and selected biological and chemical constituents in a water body with time.

Inputs to the model include the following:

- *Bathymetric Data* - data representing the layout and volumetric dimensions of the water body.
- *Initial Conditions* - data representing the starting conditions within the reservoir in terms of temperature and reactant distribution.
- *Meteorological Data* - this data includes the site specific values for air temperature, wind speed, wind direction, dew point temperature and cloud cover.
- *Upstream Boundary Conditions* - this data includes the flow rates of the incoming streams as well as the time varying concentrations of the reactants being modelled.
- *Flow Rates of Releases* - this includes the data describing the predicted (or measured) release pattern from the reservoir and is essential for volume balance calculations.

In this study, version 3.11 of the CE-QUAL-W2 Model was used.

1.3 APPLICATION OF THE MODEL TO THE PROPOSED NWAMITWA DAM

To obtain a realistic prototype of the Dam it was necessary to represent the physical constraints as accurately as possible. As mentioned in **Chapter 0**, these include bathymetric data, initial conditions, meteorological data and upstream/downstream boundary conditions. These will be discussed in more detail in the ensuing chapters.

1.3.1 Input data

Meteorological data
Meteorological data was obtained from the Agricultural Research Council (ARC). Hourly data was available at the meteorological station in the Tzaneen area and the variables measured at each are listed below:

- Rainfall (mm)
- Dry bulb temperature (°C)
- Leaf wetness
- Relative humidity (%)
- Evaporation (mm)
- Short wave solar radiation (W/m^2)
- Wind speed (m/s)
- Wind direction (degrees from north)

Bathymetric data
The bathymetric description of the Dam is probably the most fundamental data required to construct a numerical grid which is used in the model. The numerical grid is a simplified mathematical description of the volume and shape of the Dam. It is absolutely essential to construct an accurate description of the Dam, as this will determine how well the water level in the Dam is modelled. The water level in the Dam is closely linked to water quality modelling and if the initial hydraulic calibration is not achieved, then water quality calibration will be difficult, if not impossible.

Data for construction of the numerical grid was obtained from the DWAF and was available as cross-sections through the Dam basin. The original data was imported into the Civil Designer programme where break-lines (joining high and low points) were generated. This surface was sed to calculate volume in the Dam at 0.5 m intervals. To expedite the process of constructing the bathymetry file, it was decided to allow the model segment boundaries to coincide with the cross-sections for determining sedimentation. The orientation of a segment was obtained by connecting the midpoint of the cross-section (between banks) to the midpoint of the following cross-section with the angle being measured relative to north, in a clockwise direction. The procedure used is outlined below:

i.) Discretise the reservoir into segments.
ii.) Draw a line from the midpoint of the downstream segment boundary to the midpoint of the upstream segment boundary.
iii.) At the outlet of the segment draw in a North-South line.
iv.) The angle between the two lines (defined in 2 and 3 above) in a clockwise direction from north was then measured and taken to be the segment orientation.

A plan view depicting the segment layout of the Dam is shown in **Figure 1.**

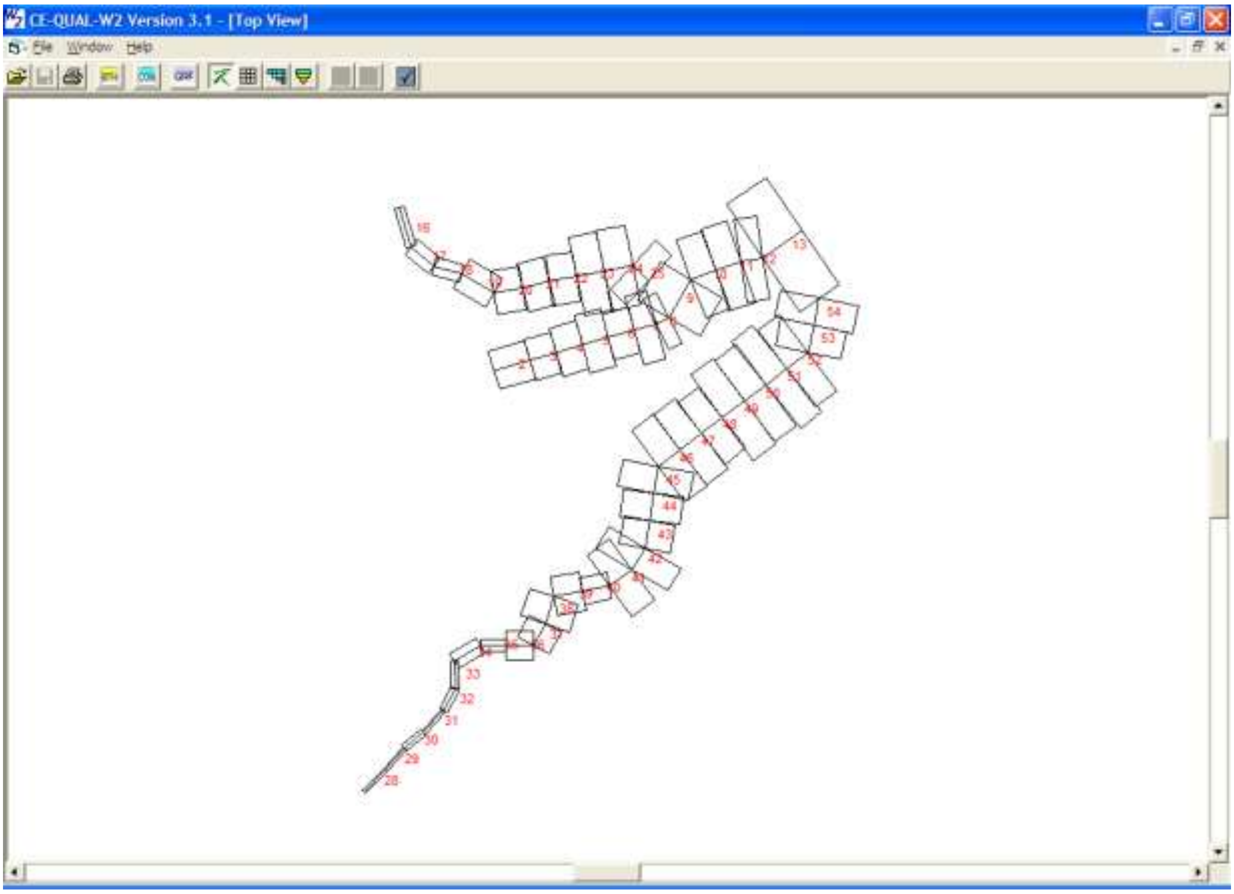

Figure 1 Layout of Segments Used for the proposed Nwamitwa Dam

Each segment was divided into a number of layers, 0.5 m in thickness, extending from the FSL to the bottom of the Dam. The width of each cell was then calculated using the formula below:

s

$$\frac{Volume\_in\_cell}{length\_of\_Segment\_x\_height\_of\_Segment}$$

Using this method, the calculated volume of each cell in the grid was preserved. The entire grid for the proposed Nwamitwa Dam was made up of 55 segments and 56 layers with segments 1, 14, 15, 26, 27 and 55 representing boundary segments while layers 1 and 56 represented boundary layers that have zero width. These cells, however, need to be specified to enable the model to function. A visual representation of the grid is depicted in **Figure 2**. It should be noted that the branch receiving most of the inflow, branch 3, has 27 active segments, while branches 1 and 2 have 12 and 10 active segments, respectively.

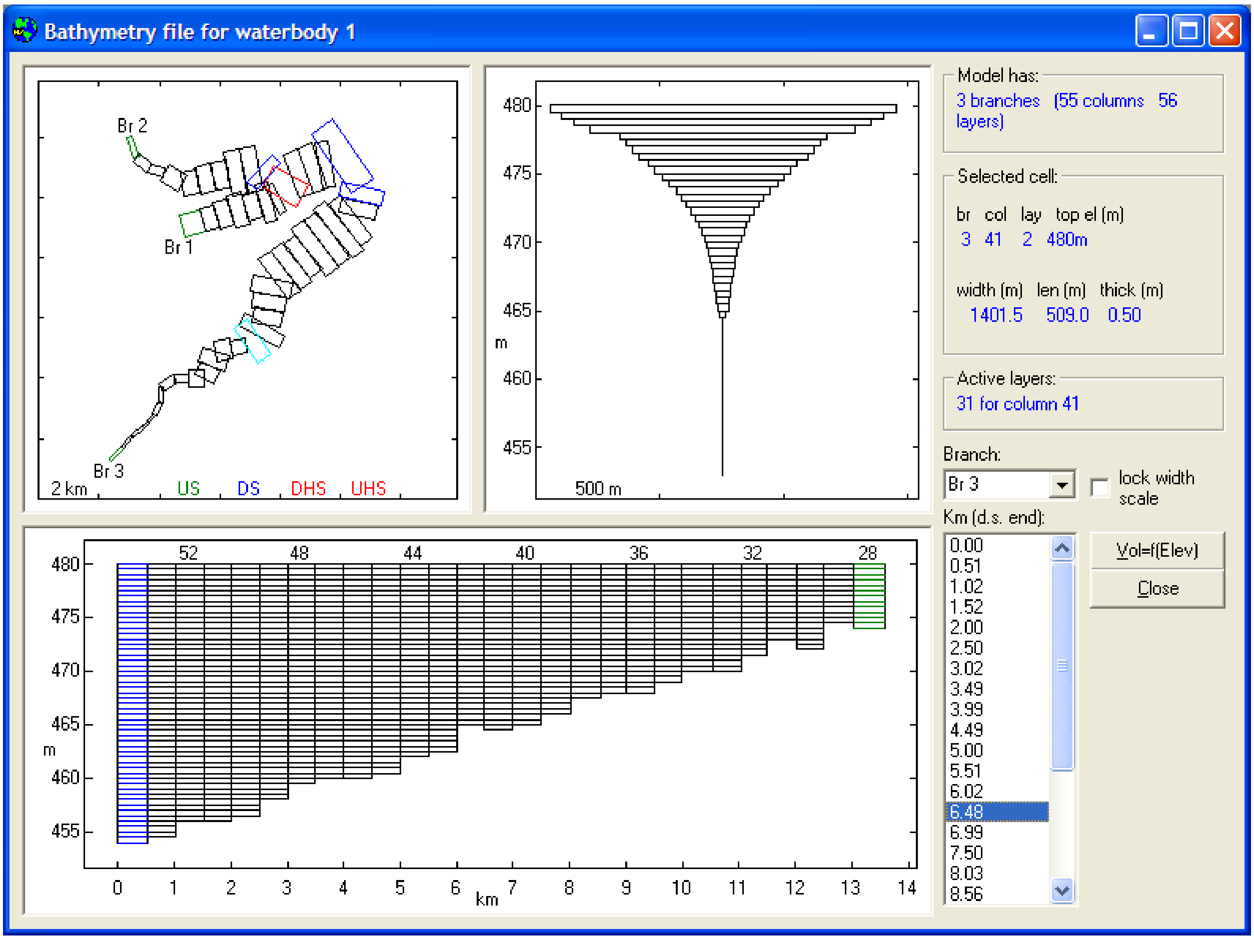

Figure 2 Proposed Nwamitwa Dam Bathymetry

The mathematical grid is only a representation of reality and should be compared with measured data to ensure that the grid is realistic. Ideally the calculated volume-height relationship should be compared with the volume-height relationships determined from the first sediment survey of the Dam.

The calculated volume-height relationship is shown in **Figure 3.**

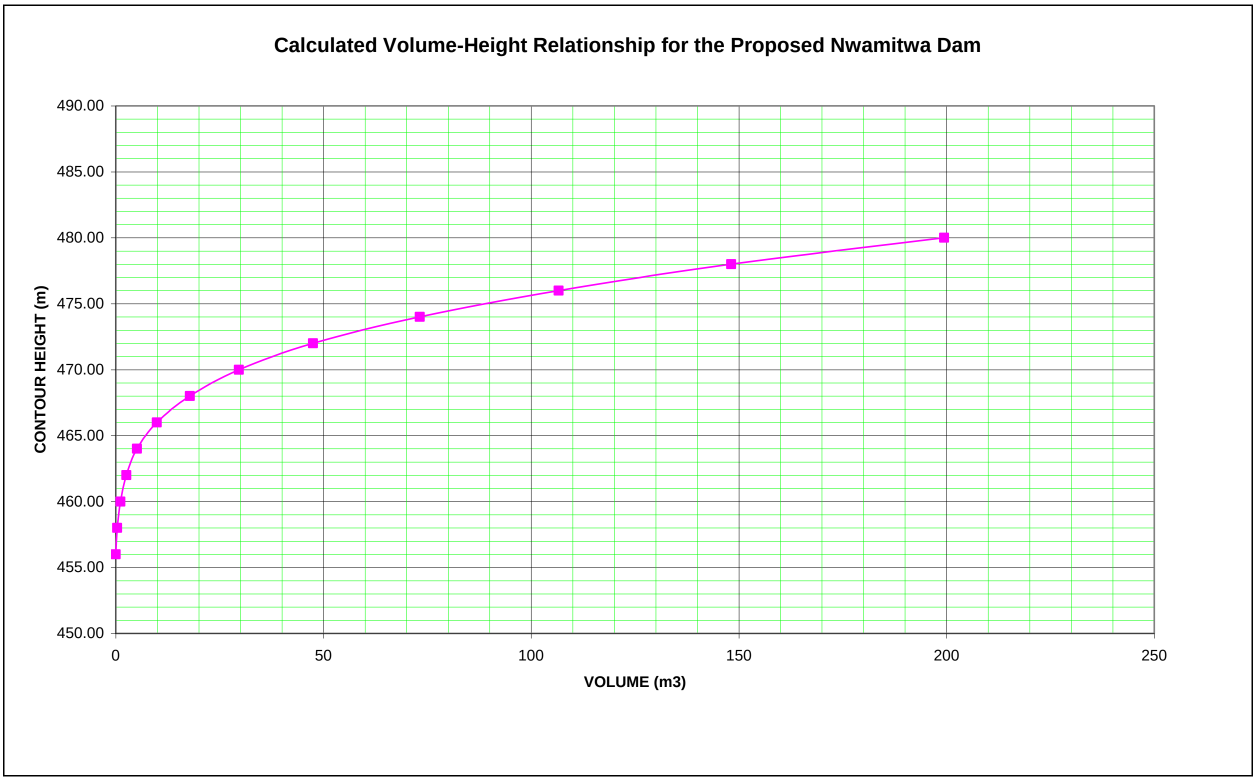

Figure 3 Calculated volume-height relationships for the proposed Nwamitwa Dam

Upstream boundary conditions (inflows)

Inflows

As mentioned previously, the proposed Nwamitwa Dam will be situated on the Groot Letaba River, downstream of Tzaneen Dam. Flow into the proposed Dam would be measured at gauging station B8H009. The aim was to find a period where a minimum number of gaps existed in the inflow record and which also overlapped with the meteorological data which was available. Examination of the inflow record revealed that the period 17-08-2006 to 31-10-2007 was reasonably free of gaps and overlapped with the available meteorological data. Two ungauged tributaries, make up the remainder of the inflows to the Dam and was assumed to contribute 10% of the inflow contributed by the Groot Letaba River.

Temperature

No inflow temperature data was collected for the Groot Letaba River and it was necessary to estimate this information from the daily temperature measurements collected at the Tzaneen meteorological station. The daily inflow temperature was estimated based on the method of Pligrim MP, Fang X & Stefan HG (1998) but was modified based on the fact that releases from Tzaneen dam were bottom releases and would of necessity be colder than the “natural” stream flow temperature. The original and modified equations of Pilgrim *et al*. are shown below:

$$T_w = 4.4 + 0.81T_a$$

$$T_w = 4.4 + 0.81T_a - 2$$

Where

T_w = Average daily temperature in degrees Celsius and
T_a = Average daily streamflow temperature in degrees Celsius

Downstream boundary conditions (outflows)

For the purposes of this study it has been assumed that 50% of the inflows to the dam would be released downstream to meet the agricultural and ecological demands.

1.3.2 Simulation scenarios considered

Since the Dam has not reached the final design stage no information was available on the hydraulic parameters for the dam wall and it was assumed that spills could be calculated using the following relationship, as required by the CE-QUAL-W2 model.

$$Q = \alpha \times (\Delta h)^{\beta}$$

Where,

Q = flow in m^3/s
α = constant = 76.069
β = constant = 1.626
Δh = height above the spillway

In developing this equation it was assumed that the length of the spillway was 40m. The arrangement for the outlet works are discussed in the ensuing sections.

Bottom release
For this scenario it was assumed that the Dam has a FSL of 480 mamsl and that only a bottom outlet located at 462.75 mamsl

Multi-level outlet
For this scenario it was assumed that the Dam has a FSL of 480 mamsl and that releases could be made from outlets located at 462.75, 467.75 and 472.75 mamsl.

The original daily volumes released from the bottom outlet structure were split amongst the various off-takes in the multi-level outlet structure to ensure that not only cold bottom waters were released and in an attempt to reduce the depth of stratification that may be experienced.

2. RESULTS

The results of the various modelling runs are presented in the ensuing sections.

2.1 TEMPERATURE SIMULATIONS

The temperatures of the Dam releases, using the bottom and multi-level outlet structures respectively, are depicted in **Figure 4.** The inflow temperatures to the dam were use to represent the “target” temperature downstream of the Dam.

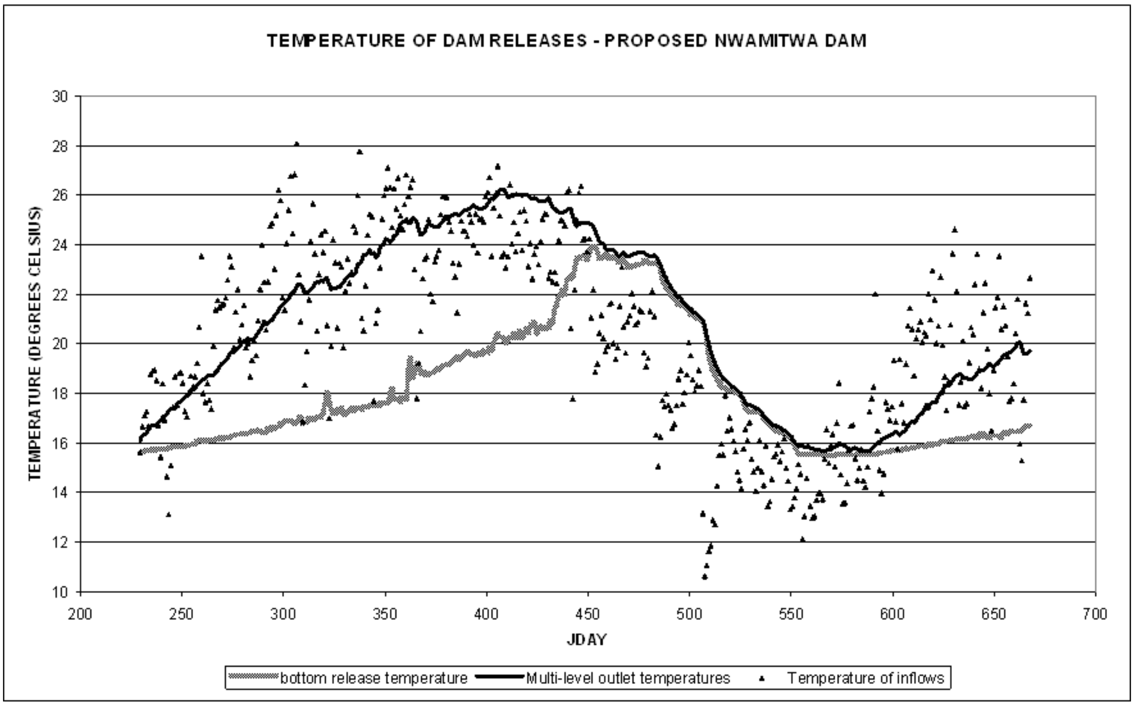

Figure 4 Temperature of dam releases made from the bottom outlet and from the multi-level outlet structure (FSL = 480 mamsl) (Julian day 228 = 17 August 2006)

It can be seen that the difference in temperature of releases from the Dam is significant and although there is some uncertainty about the initial temperature profile in the Dam, it should be pointed out that both simulations had the same initial conditions and that the only difference was the multi-level outlet structure.

Figure shows that the temperature of the releases from the multi-level outlet structure matches the target temperature more closely, especially during the summer months when the bottom releases are naturally much cooler as a result of the thermal stratification.

Time-depth plots showing the in-lake temperature for the bottom outlet and multi-level outlet scenarios are shown in **Figure 5** and **Figure 6**, respectively. These figures suggest that the stratification that would occur, using a multi-level outlet structure, would be fairly similar to just using a bottom outlet structure. This can be seen more clearly in **Figure 7** which shows the comparison of the temperature profiles which would exist when using only the bottom outlet structure compared to using the multi-level outlet. The hypolimnion temperatures of the latter scenario being almost equal to that of the scenario using only a bottom release. The in-lake temperature conditions were a by-product of the simulation to determine the temperatures of releases made from the Dam. This should be analysed from an ecological perspective for acceptability. In this way, the model could (if other water quality variables are also modelled) provide early warning of unacceptable in-lake conditions that may exist as a result of the chosen release patterns to meet the downstream temperature requirements.

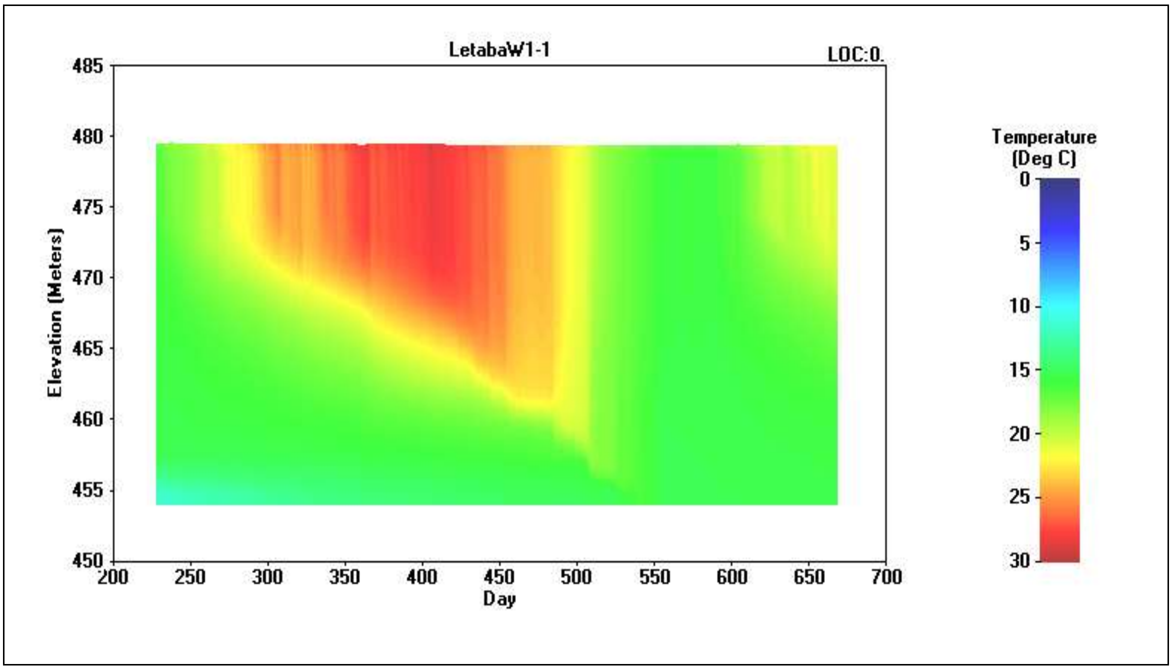

Figure 5 In-lake temperature conditions for the bottom-release scenario (FSL = 480 mamsl) (Julian day 228 = 17 August 2006)

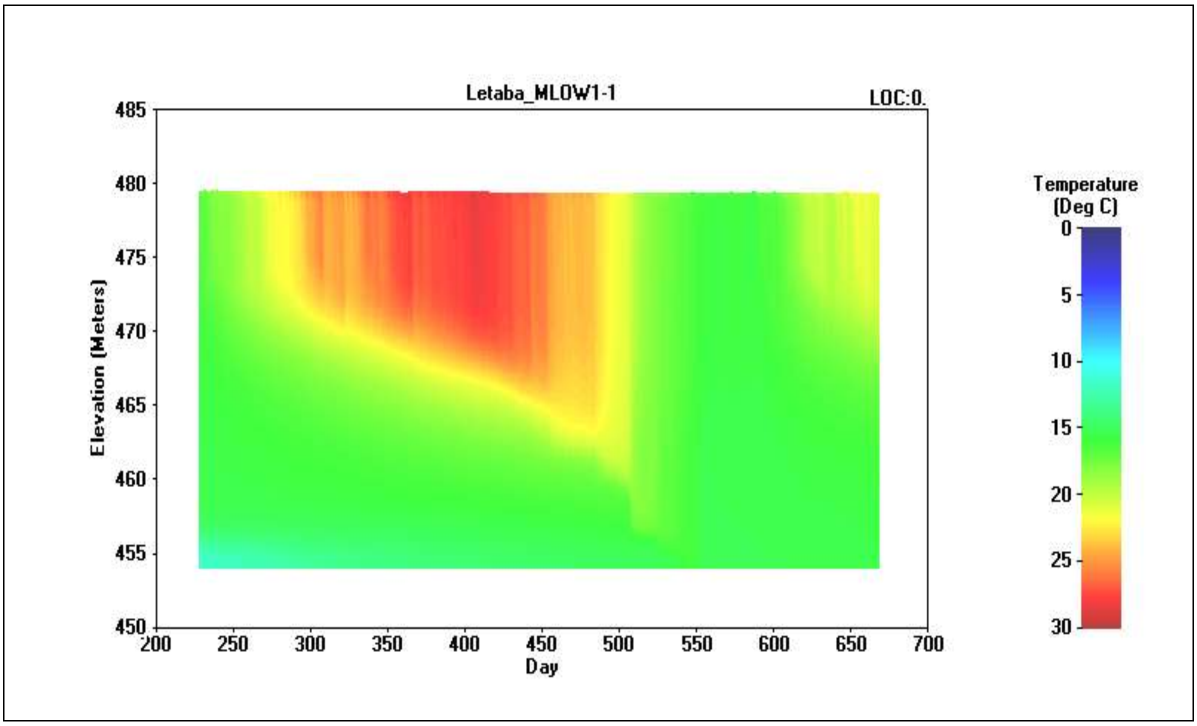

Figure 6 In-lake temperature conditions for multi-level outlet scenario (FSL = 480 mamsl) (Julian day 228 = 17 August 2006)

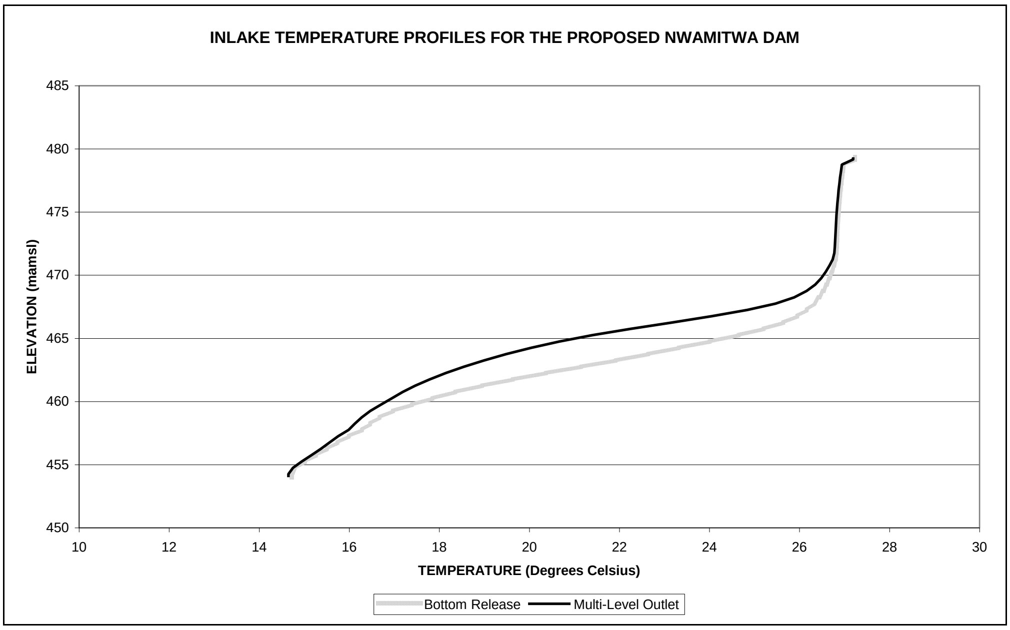

Figure 7 Inlake Temperature Profiles, (FSL = 480 mamsl, Julian day 434.5 = 11 March 2007)

2.2 DISSOLVED OXYGEN SIMULATIONS

Two scenarios were considered for these simulations, *viz*.

Optimistic scenario This scenario does not account for the effect that algal respiration and organic matter would have on the rate of oxygen depletion and as such presents an optimistic scenario in term of the oxygen depletion rate

Pessimistic scenario This scenario incorporates a hypothetical algal species exerting additional oxygen demand on the in-lake oxygen concentration. The oxygen demand exerted by decaying organic material is still not included.

The pathways for oxygen addition and/or removal from the reservoir water column are depicted in **Figure 8**.

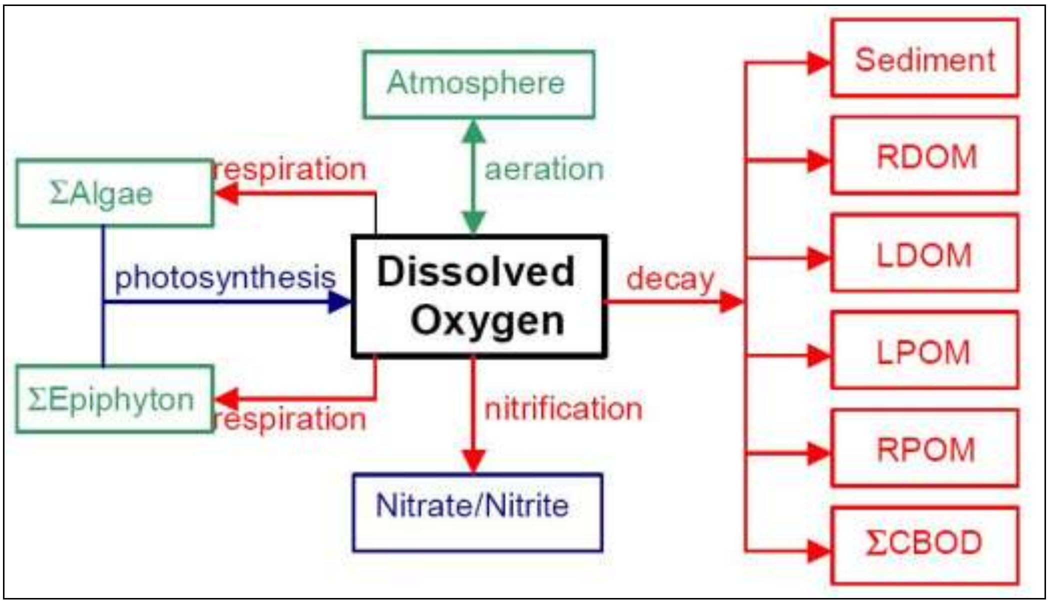

Figure 8 Internal flux between dissolved oxygen and other constituents

2.2.1 Optimistic scenario

For this simulation scenario, it was assumed that only a zero-order sediment oxygen demand of 0.5 $g.O_2.m^{-2}.day^{-1}$ and the nitrification demand (oxygen depletion for the conversion of ammonium to nitrate and/or nitrite) was exerted in each segment of the Dam. It was also assumed that dissolved oxygen was not depleted by respiration, decay of organic matter and could only be replenished by re-aeration.

The dissolved oxygen concentration of the releases made from the bottom outlet and from the multilevel outlet is depicted in **Figure 9** which shows that the multilevel outlet structure has a significant effect on the on the dissolved oxygen concentration of the dam releases. It is expected, however, that re-aeration would occur immediately after these releases are made.

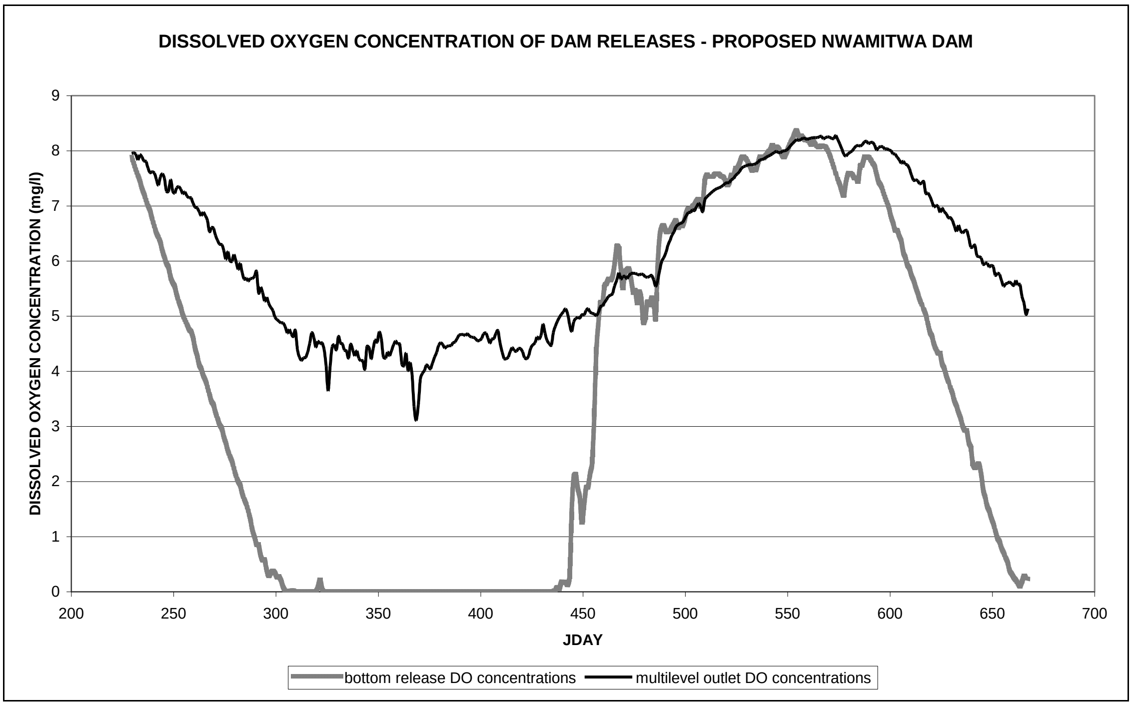

Figure 9 Dissolved oxygen concentration of dam releases made from the bottom outlet and from the multi-level outlet structure – Algal respiration not included(FSL = 480 mamsl) (Julian day 228 = 17 August 2006)

Time-depth plots of the inlake dissolved oxygen concentrations for the bottom outlet and multilevel outlet scenario is depicted in **Figure 10** and **Figure 11**, respectively. These figures show that, for both the bottom and multilevlel outlet scenarios, the Dam could be de-oxygenated between its lowest level and 470 mamsl for most of the year. The oxygen is probably replenished during the colder months (early May to end of July) as a result of oxygen-rich, cooler inflows that plunge deeper into the dam to mix with the deoxygenated water.

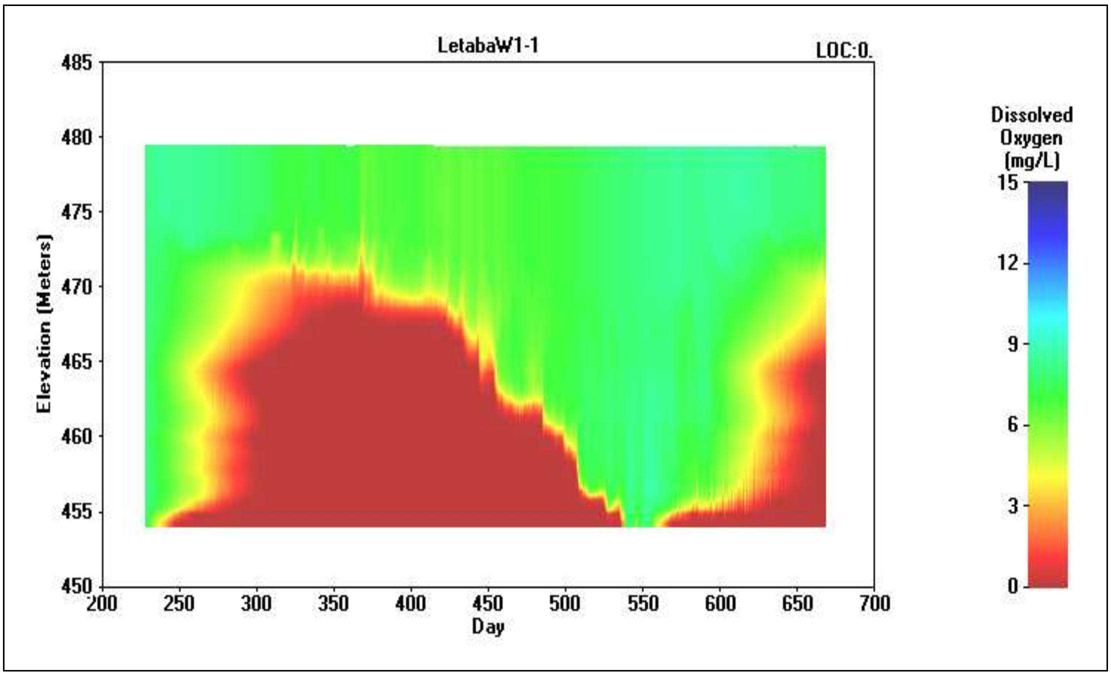

Figure 10 In-lake concentrations of Dissolved Oxygen for the bottom outlet scenario – algal respiration not included (FSL = 480 mamsl) (Julian day 228 = 17 August 2006)

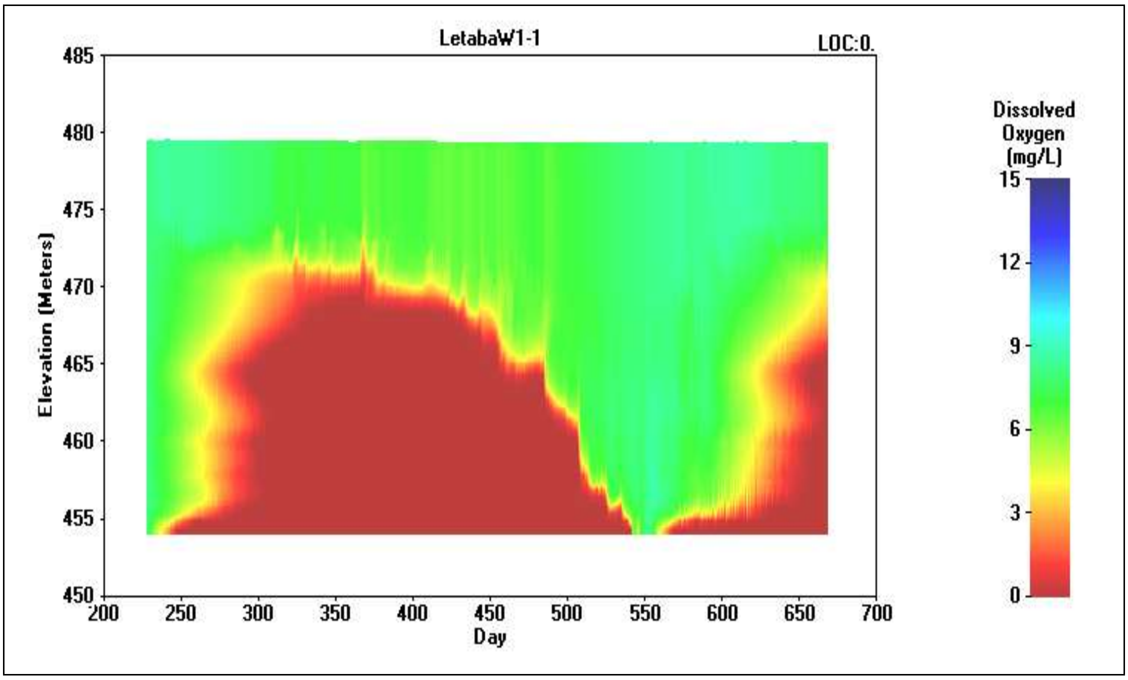

Figure 11 In-lake concentrations of Dissolved Oxygen for multilevel outlet scenario – algal respiration not included (FSL = 480 mamsl) (Julian day 228 = 17 August 2006)

2.2.2 Pessimistic scenario

For this simulation scenario, it was assumed that in addition to the a zero-order sediment oxygen demand of 0.5 $g.O_2.m^{-2}.day^{-1}$ and the nitrification demand (oxygen depletion for the conversion of ammonium to nitrate and/or nitrite), an algal respiration oxygen demand was also present.

The dissolved oxygen concentration of the releases made from the bottom outlet and from the multilevel outlet is depicted in **Figure 9** which shows that the multilevel outlet structure has a significant effect on the on the dissolved oxygen concentration of the dam releases. It is expected, however, that re-aeration would occur immediately after these releases are made. The dissolved oxygen concentration of the releases based on the pessimistic scenario does not differ significantly from those calculated for the optimistic scenario, indicating that the algal respiration demand is probably lower than was initially surmised.

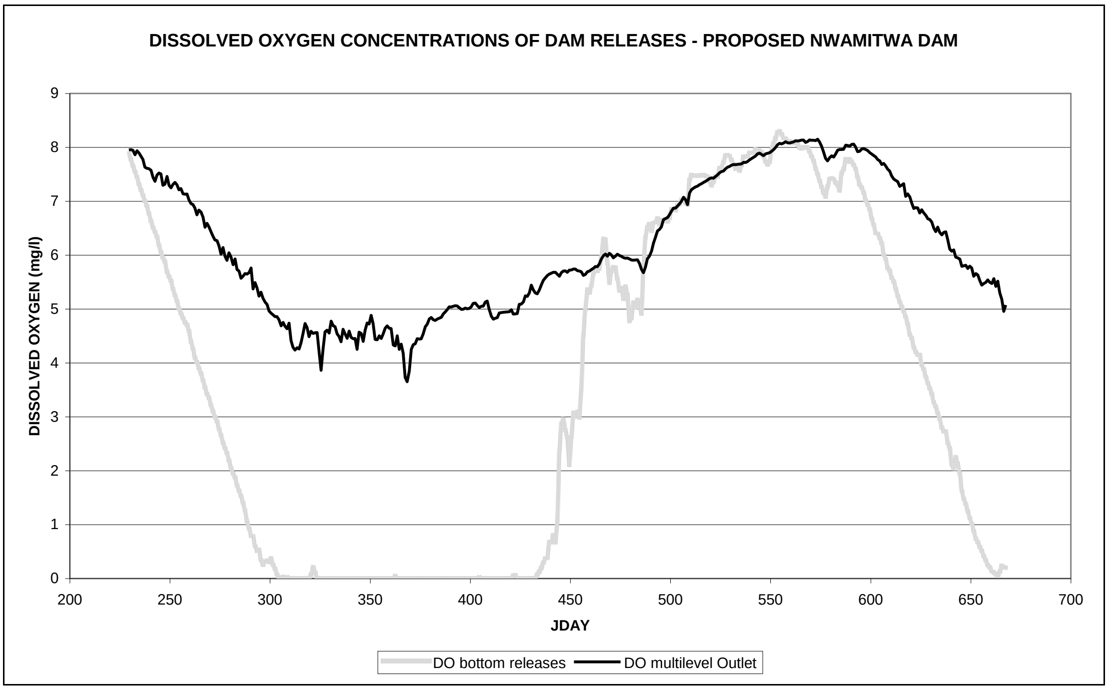

Figure 12 Dissolved oxygen concentration of dam releases made from the bottom outlet and from the multi-level outlet structure – Algal respiration included (FSL = 480 mamsl) (Julian day 228 = 17 August 2006)

Time-depth plots of the inlake dissolved oxygen concentrations for the bottom outlet and multilevel outlet scenario is depicted in **Figure 13** and **Figure 14** respectively. As before, these figures show that, for both the bottom and multilevlel outlet scenarios, the Dam could be de-oxygenated between its lowest level and 470 mamsl for most of the year. The oxygen is probably replenished during the colder months (early May to end of July) as a result of oxygen-rich, cooler inflows that plunge deeper into the dam to mix with the deoxygenated water.

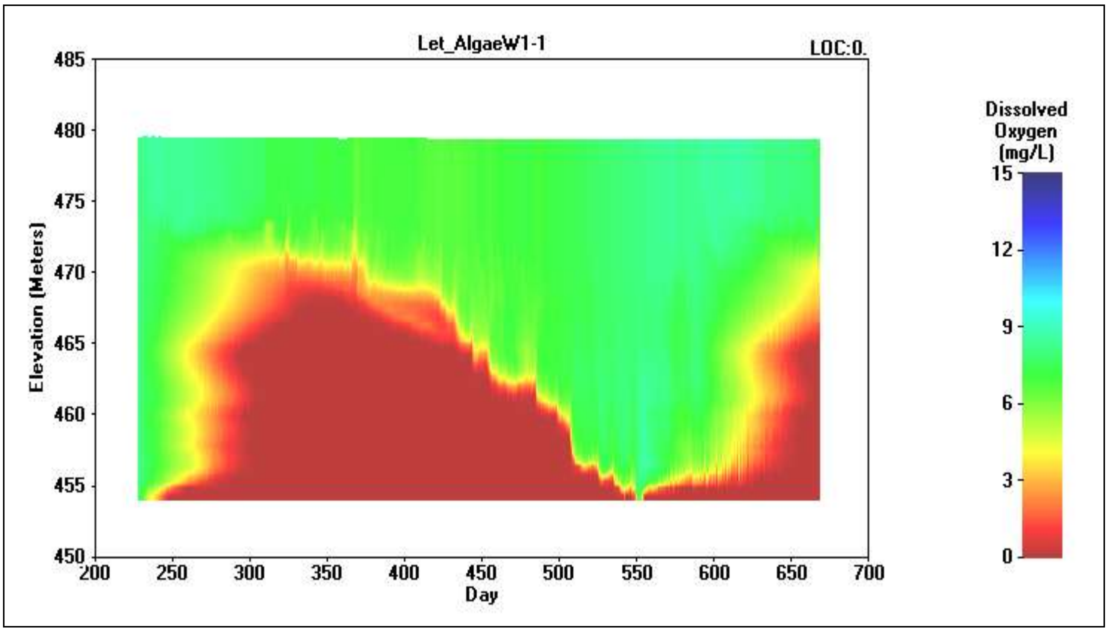

Figure 13 In-lake concentrations of Dissolved Oxygen for the bottom outlet scenario – algal respiration included (FSL = 480 mamsl) (Julian day 228 = 17 August 2006)

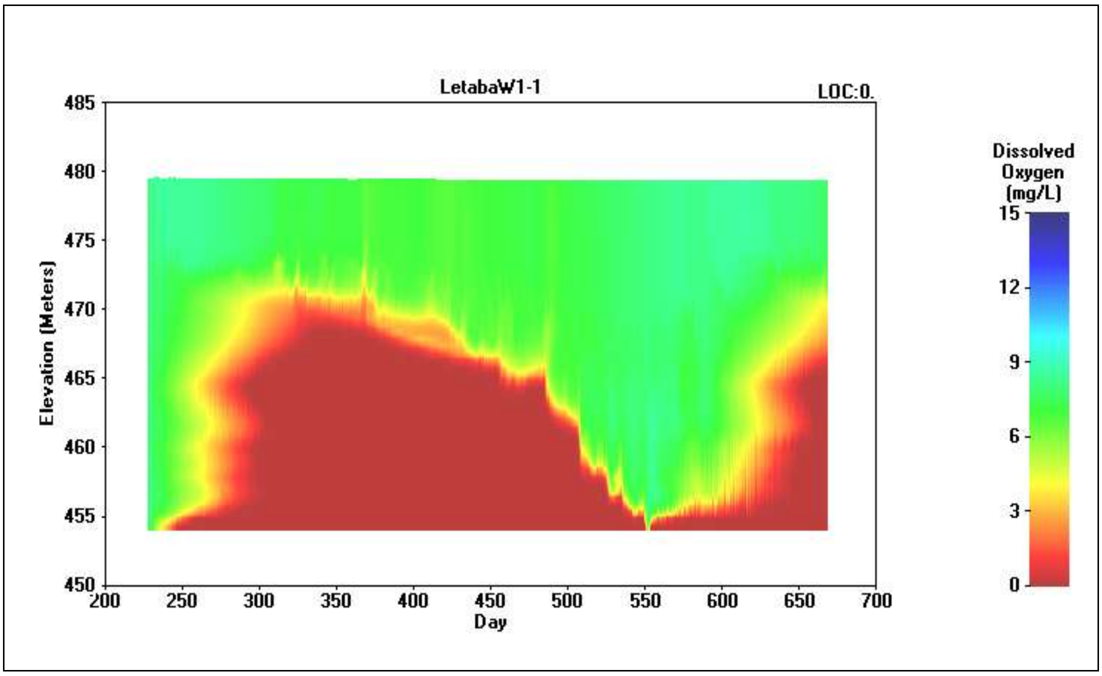

Figure 14 In-lake concentrations of Dissolved Oxygen for multilevel outlet scenario – algal respiration included (FSL = 480 mamsl) (Julian day 228 = 17 August 2006)

3. DISCUSSION

3.1 DATA REQUIREMENTS FOR AN INSTALLED MODEL

During the modelling exercise it was recognised that the monitoring of inflow and outflow rates from the Dam would be important, if it is intended to use mathematical models for informing the operating philosophy of the Dam. This is particularly crucial when a hydrodynamic and water quality model (such as CE-QUAL-W2) is to be employed, because the in-lake temperature regimes are largely determined by the hydraulics of the system, which in turn is influenced by the inflow and outflow patterns from the Dam. Similarly, it is important that temperature be measured at the inlet to the Dam. Although dissolved oxygen was not considered in this study it could possibly become important, when considered with the other non-conservative water quality constituents. Several additional water quality parameters would have to be measured at the inflow, should modelling of dissolved oxygen be required.

Meteorology is also an important driving force in the model and needs to be determined as accurately as possible. In this study, meteorological data was obtained from a weather station situated at Tzaneen which is approximately 36 km west of the proposed Nwamitwa Dam site. Since the model is sensitive to the local weather data it would have been preferable to have a weather station at the Dam site.

3.2 OPTIMISATION OF RELEASES FOR TEMPERATURE

Hanna (1999) developed an optimisation programme that could be used with CE-QUAL-W2 to obtain the downstream temperature targets. With this programme it would be possible to determine what the required volume of flow through each level in the off-take structure should be, assuming that water from a maximum of two different levels could be mixed. This algorithm has recently been updated to be more rigorous in terms of the thermodynamic characteristics of the system (TCTA, 2007). In the latter approach, the ***Target Temperature*** (or more correctly, the enthalpy) of the outflow stream, as determined by the ecologist, is used as the target with which the simulated temperatures are compared, thus providing a more direct approach for determining the relevant outlet structures required to meet the downstream target temperature.

4. CONCLUSIONS

Based on the results and discussion in preceding sections, the following can be concluded:

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

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

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

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

5. RECOMMENDATIONS

Based on the results, discussions and conclusions, the following recommendations can be made:

- A monitoring programme for the systematic monitoring of the pertinent data for assessing or modelling water quality in the reservoir should be instituted as soon as possible. This programme should include:

a. Hourly meteorological data (air temperature, dew point temperature, wind speed, wind direction, and percentage sunshine).
b. Inflow rates.
c. Inflow and in-lake water quality (WQ) (see Appendix A for a complete list of WQ variables).
d. Release rates.

- Since a simplistic approach for making the ecological releases was used in this study, it is recommended that a more representative release pattern be created, based on the operating rules of the Dam as well as on realistic ecological requirements and irrigation demands downstream of the Dam. This, in addition to more representative meteorological and inflow data, will provide a more realistic representation of the temperature profile most likely to exist in the Dam and the ability to match the required temperature downstream of the Dam.

- A multi-level outlet structure should be considered for further investigation since it provides more flexibility in mixing water from different levels in the dam, providing an increased probability of meeting the downstream water quality requirements.

- For the proposed outlet heights of the multi-level outlet structure, the following approach should be adopted to determine the level of confidence that can be attached to the results presented in this report. This would provide an indication of the probability of making releases from the highest outlet levels during the summer months when higher target temperatures will be set:

  a. Re-run the Dam trajectories with realistic ecological requirements imposed, to determine the most probable dam level at the beginning of the simulation.
  b. Re-run the hydrodynamic and water quality model, using the most probable starting level at the beginning of the simulation period and realistic releases to determine the probability of meeting the downstream temperature requirement during the summer months.
  c. Decide, in consultation with the ecologist whether the determined probability to meet the downstream temperature requirement is acceptable.

- The zero order sediment demand is an estimated value and should be varied in additional model runs to establish the sensitivity of oxygen depletion rate to changes in this parameter.

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APPENDIX A

DATA REQUIREMENTS FOR WATER QUALITY MODELLING

(a) APPENDIX A : Data requirements for water quality modelling

Water Quality Constituents To Be Measured At The Inflow/Outflow Boundaries

Boundary Conditions		
Minimum Parameters	**Additional Parameters**	**Frequency**
Inflow temperature conductivity	Dissolved oxygen pH Total Dissolved Salts[1]	Daily or continuous
Total organic carbon	Dissolved Organic Carbon	Weekly grab sampling Storm sampling, if required
Soluble reactive phosphate Total phosphorous		Weekly grab sampling. Storm sampling, if required
Nitrate-nitrite nitrogen Ammonium nitrogen Total Kjeldahl nitrogen		Weekly grab sampling Storm sampling, if required
	Total suspended solids Inorganic suspended solids	Weekly grab sampling Storm sampling, if required
	Chlorophyll-*a* Dissolved silica Alkalinity	Weekly grab sampling Storm sampling, if required

[1] *Enough samples to allow for the determination of the relationship between EC and TDS*

In-Lake Water Quality Constituents To Be Measured

Minimum Parameters	**Additional Parameters**	**Frequency**
Temperature[2] Dissolved Oxygen[3] pH[3] TDS[3] or EC		Monthly grab sampling
Chlorophyll-*a*[4] Algal biomass and type		Monthly grab sampling
Total Organic Carbon[4]	Dissolved Organic Carbon	Monthly grab sampling
Soluble reactive phosphate[4] Total phosphorous[4]		Monthly grab sampling
Nitrate + nitrite nitrogen[4]		Monthly grab sampling

Ammonia nitrogen[4] Total Kjeldahl nitrogen		
Alkalinity		Monthly grab sampling
	Total suspended solids[4] Inorganic suspended solids[4]	Monthly grab sampling
Secchi depth		Monthly grab sampling
	Dissolved/total iron[5] Dissolved/total manganese[5] Dissolved/total silica[5] Total dissolved sulphide[5] Sulphate[5] Iron sulphide[5]	Monthly grab sampling

[2] *Initially, this could be done on a bi-weekly basis to supplement the automatic temperature vertical profiling at the Dam wall and to provide an alternative data set for calibration. Vertical profiling at 1m intervals using field instruments.*

[3] *Vertical profiling at 1m intervals, on a bi-weekly basis using field instruments.*

[4] *Vertical profiling at 3m intervals using field instruments.*

[5] *When concerned with the release from the sediment during anoxic conditions these parameters should be measured.*

APPENDIX F - Cost Estimates

APPENDIX F.1 – Cost Estimate of Preferred Dam Size

NWAMITWA DAM

1.16 MAR - FSL 479.5 - NOC 486 - 190m spillway

No	DESCRIPTION	UNIT	RATE Apr 09 Rand	QUANTITY	AMOUNT Rand
1	**Clearing**				
	(a) sparse	ha	6,700	1.61	10,787
	(b) bush	ha	11,700	7.25	84,767
	(c) trees	ha	23,400	7.25	169,533
2	**River diversion**	Sum			20,000,000
3	**Excavation**				
	(a) Bulk				
	(i) all materials	m^3	35	883,000	30,905,000
	(ii) extra over for rock	m^3	60	164,190	9,851,400
	(b) Preparation of solum				
	(i) all materials	m^2	16	36,200	579,200
	(II) extra over for rock	m^2	16	14,160	226,560
4	**Drilling & Grouting**				
	(a) Curtain grouting	m drill	930	13,423	12,483,390
	(b) Consolidation grouting	m drill	930	11,050	10,276,500
5	**Embankment**				
	(a) Earthfill	m^3	40	1,490,000	59,600,000
	(b) Filters & transition	m^3	140	127,210	17,809,400
	(c) Rip-rap & rock toe	m^3	90	169,500	15,255,000
	(d) Overhaul beyond 3km	m^3km	4.4	5,934,200	26,110,480
6	**Concrete Works**				
	(a) Formwork				
	(i) gang formed	m^2	220	31,840	7,004,800
	(ii) intricate	m^2	305	3050	930,250
	(b) Concrete				
	(i) RCC	m^3	455	248,530	113,081,150
	(ii) Mass	m^3	530	7,520	3,985,600
	(ii) structural	m^3	840	6,700	5,628,000
	(c) Reinforcing	t	9,600	603	5,788,800
7	**Mechanical Items**				
	(a) Valves & gates	Sum			11,000,000
	(b) Cranes & hoists	Sum			5,500,000
	(c) Structural steelwork	t	23,500	10	235,000
	SUB-TOTAL				356,515,617

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No	DESCRIPTION	UNIT	RATE Apr 09 RAND	QUANTITY	AMOUNT
8	**Landscaping** (% of 1-7)	%	5	356,515,617	17,825,781
9	**Miscellaneous** (% of 1-7)	%	10	356,515,617	35,651,562
	SUB TOTAL A				409,992,959
10	**Preliminary & General** (% of sub-total A)	%	40	409,992,959	163,997,184
11	**Relocation of roads**				
	(a) D1292	Sum			166,864,982
	(b) P43-3	Sum			61,297,340
	(c) Bridges	Sum			51,046,150
12	**Relocation of Services**				
	(a) ESKOM	Sum			10,583,146
	(b) TELKOM	Sum		allow	10,000,000
	SUB TOTAL B				873,781,761
13	**Contingencies** (% of sub total B)	%	10	873,781,761	87,378,176
	SUB TOTAL C				961,159,937
14	**Planning design & supervision** (% of sub total C)	%	15	961,159,937	144,173,991
	SUB TOTAL D				1,105,333,927
15	**Expropriation costs**	Sum			179,756,716
	TOTAL COST (excl. VAT)				**1,285,090,643**

APPENDIX F.2 – Cost Estimates of Other Dam Sizes

NWAMITWA DAM

0.41 MAR - FSL 473.5 - NOC 480.0 - 190m spillway

No	DESCRIPTION	UNIT	RATE Apr 09 Rand	QUANTITY	AMOUNT Rand
1	**Clearing**				
	(a) sparse	ha	6,700	1.61	10,787
	(b) bush	ha	11,700	7.25	84,767
	(c) trees	ha	23,400	7.25	169,533
2	**River diversion**	Sum			20,000,000
3	**Excavation**				
	(a) Bulk				
	(i) all materials	m^3	35	756,150	26,465,250
	(ii) extra over for rock	m^3	60	144,850	8,691,000
	(b) Preparation of solum				
	(i) all materials	m^2	16	26,100	417,600
	(II) extra over for rock	m^2	16	13,100	209,600
4	**Drilling & Grouting**				
	(a) Curtain grouting	m drill	930	10,423	9,693,390
	(b) Consolidation grouting	m drill	930	8,600	7,998,000
5	**Embankment**				
	(a) Earthfill	m^3	40	649,000	25,960,000
	(b) Filters & transition	m^3	140	61,500	8,610,000
	(c) Rip-rap & rock toe	m^3	90	78,200	7,038,000
	(d) Overhaul beyond 3km	m^3km	4.4	2,794,000	12,293,600
6	**Concrete Works**				
	(a) Formwork				
	(i) gang formed	m^2	220	27,240	5,992,800
	(ii) intricate	m^2	305	2,755	840,275
	(b) Concrete				
	(i) RCC	m^3	455	187,365	85,251,075
	(ii) Mass	m^3	530	6,812	3,610,360
	(iii) structural	m^3	840	5,673	4,765,320
	(c) Reinforcing	t	9,600	511	4,901,472
7	**Mechanical Items**				
	(a) Valves & gates	Sum			11,000,000
	(b) Cranes & hoists	Sum			5,500,000
	(c) Structural steelwork	t	23,500	10	235,000
	SUB-TOTAL				249,737,829

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No	DESCRIPTION	UNIT	RATE Apr 09 RAND	QUANTITY	AMOUNT
8	**Landscaping** (% of 1-7)	%	5	249,737,829	12,486,891
9	**Miscellaneous** (% of 1-7)	%	10	249,737,829	24,973,783
	SUB TOTAL A				287,198,503
10	**Preliminary & General** (% of sub-total A)	%	40	287,198,503	114,879,401
11	**Relocation of roads**				
	(a) D1292	Sum			166,864,982
	(b) P43-3	Sum			61,297,340
	(c) Bridges	Sum			51,046,150
12	**Relocation of Services**				
	(a) ESKOM	Sum			10,583,146
	(b) TELKOM	Sum		allow	10,000,000
	SUB TOTAL B				701,869,522
13	**Contingencies** (% of sub total B)	%	10	701,869,522	70,186,952
	SUB TOTAL C				772,056,474
14	**Planning design & supervision** (% of sub total C)	%	15	772,056,474	115,808,471
	SUB TOTAL D				887,864,945
15	**Expropriation costs**	Sum			101,280,260
	TOTAL COST (excl. VAT)				**989,145,205**

NWAMITWA DAM

0.85 MAR - FSL 477.5 - NOC 484 - 190m spillway

No	DESCRIPTION	UNIT	RATE Apr 09 Rand	QUANTITY	AMOUNT Rand
1	**Clearing**				
	(a) sparse	ha	6,700	1.61	10,787
	(b) bush	ha	11,700	7.25	84,767
	(c) trees	ha	23,400	7.25	169,533
2	**River diversion**	Sum			20,000,000
3	**Excavation**				
	(a) Bulk				
	(i) all materials	m^3	35	843,280	29,514,800
	(ii) extra over for rock	m^3	60	161,800	9,708,000
	(b) Preparation of solum				
	(i) all materials	m^2	16	33,100	529,600
	(II) extra over for rock	m^2	16	13,900	222,400
4	**Drilling & Grouting**				
	(a) Curtain grouting	m drill	930	12,823	11,925,390
	(b) Consolidation grouting	m drill	930	10,570	9,830,100
5	**Embankment**				
	(a) Earthfill	m^3	40	1,148,500	45,940,000
	(b) Filters & transition	m^3	140	104,300	14,602,000
	(c) Rip-rap & rock toe	m^3	90	136,900	12,321,000
	(d) Overhaul beyond 3km	m^3km	4.4	4,824,000	21,225,600
6	**Concrete Works**				
	(a) Formwork				
	(i) gang formed	m^2	220	30,592	6,730,240
	(ii) intricate	m^2	305	2,967	904,935
	(b) Concrete				
	(i) RCC	m^3	455	231,411	105,292,005
	(ii) Mass	m^3	530	7,520	3,985,600
	(ii) structural	m^3	840	5,760	4,838,400
	(c) Reinforcing	t	9,600	518	4,976,640
7	**Mechanical Items**				
	(a) Valves & gates	Sum			11,000,000
	(b) Cranes & hoists	Sum			5,500,000
	(c) Structural steelwork	t	23,500	10	235,000
	SUB-TOTAL				319,546,797

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No	DESCRIPTION	UNIT	RATE Apr 09 RAND	QUANTITY	AMOUNT
8	**Landscaping** (% of 1-7)	%	5	319,546,797	15,977,340
9	**Miscellaneous** (% of 1-7)	%	10	319,546,797	31,954,680
	SUB TOTAL A				367,478,816
10	**Preliminary & General** (% of sub-total A)	%	40	367,478,816	146,991,526
11	**Relocation of roads**				
	(a) D1292	Sum			166,864,982
	(b) P43-3	Sum			61,297,340
	(c) Bridges	Sum			51,046,150
12	**Relocation of Services**				
	(a) ESKOM	Sum			10,583,146
	(b) TELKOM	Sum		allow	10,000,000
	SUB TOTAL B				814,261,961
13	**Contingencies** (% of sub total B)	%	10	814,261,961	81,426,196
	SUB TOTAL C				895,688,157
14	**Planning design & supervision** (% of sub total C)	%	15	895,688,157	134,353,223
	SUB TOTAL D				1,030,041,380
15	**Expropriation costs**	Sum			150,032,064
	TOTAL COST (excl. VAT)				**1,180,073,444**

NWAMITWA DAM

1.50 MAR - FSL 481.5 - NOC 488 - 190m spillway

No	DESCRIPTION	UNIT	RATE Apr 09 Rand	QUANTITY	AMOUNT Rand
1	**Clearing**				
	(a) sparse	ha	6,700	1.61	10,787
	(b) bush	ha	11,700	7.25	84,767
	(c) trees	ha	23,400	7.25	169,533
2	**River diversion**	Sum			20,000,000
3	**Excavation**				
	(a) Bulk				
	(i) all materials	m^3	35	928,800	32,508,000
	(ii) extra over for rock	m^3	60	167,228	10,033,680
	(b) Preparation of solum				
	(i) all materials	m^2	16	39,300	628,800
	(II) extra over for rock	m^2	16	14,530	232,480
4	**Drilling & Grouting**				
	(a) Curtain grouting	m drill	930	14,523	13,506,390
	(b) Consolidation grouting	m drill	930	11,790	10,964,700
5	**Embankment**				
	(a) Earthfill	m^3	40	1,906,100	76,244,000
	(b) Filters & transition	m^3	140	149,300	20,902,000
	(c) Rip-rap & rock toe	m^3	90	191,110	17,199,900
	(d) Overhaul beyond 3km	m^3km	4	6,808,200	29,956,080
6	**Concrete Works**				
	(a) Formwork				
	(i) gang formed	m^2	220	33,814	7,439,080
	(ii) intricate	m^2	305	3,053	931,165
	(b) Concrete				
	(i) RCC	m^3	455	260,114	118,351,870
	(ii) Mass	m^3	530	8,555	4,534,150
	(ii) structural	m^3	840	6,700	5,628,000
	(c) Reinforcing	t	9,600	603	5,788,800
7	**Mechanical Items**				
	(a) Valves & gates	Sum			11,000,000
	(b) Cranes & hoists	Sum			5,500,000
	(c) Structural steelwork	t	23,500	10	235,000
	SUB-TOTAL				391,849,182

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No	DESCRIPTION	UNIT	RATE Apr 09 RAND	QUANTITY	AMOUNT
8	**Landscaping** (% of 1-7)	%	5	391,849,182	19,592,459
9	**Miscellaneous** (% of 1-7)	%	10	391,849,182	39,184,918
	SUB TOTAL A				450,626,559
10	**Preliminary & General** (% of sub-total A)	%	40	450,626,559	180,250,623
11	**Relocation of roads**				
	(a) D1292	Sum			166,864,982
	(b) P43-3	Sum			61,297,340
	(c) Bridges	Sum			51,046,150
12	**Relocation of Services**				
	(a) ESKOM	Sum			10,583,146
	(b) TELKOM	Sum		allow	10,000,000
	SUB TOTAL B				930,668,800
13	**Contingencies** (% of sub total B)	%	10	930,668,800	93,066,880
	SUB TOTAL C				1,023,735,680
14	**Planning design & supervision** (% of sub total C)	%	15	1,023,735,680	153,560,352
	SUB TOTAL D				1,177,296,032
15	**Expropriation costs**	Sum			232,669,796
	TOTAL COST (excl. VAT)				**1,409,965,828**

APPENDIX F.3 – Cost Estimate of Road Relocations

Cost estimate included in Dam costing sheets

Please refer to Item 11: Relocation of roads in Appendix F.1 and F.2

APPENDIX F.4 – Cost Estimate of Expropriation Costs

(Note: Electronic MS Word version excludes this file. Only available in pdf format.)

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April 2009

NWAMITWA DAM

Determination of initial expropriation costs of land and other structures within the proposed expropriation area

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1. INTRODUCTION

Schoeman & Vennote was appointed to conduct a desktop study to determine the expropriation costs of the land and other structures that will be inundated when the proposed Nwamitwa dam is built. Initially three dam sizes were investigated namely 0.5, 1.0 and 1.5 MAR and the preliminary findings were presented in September 2007. The latest expropriation boundaries (which closely matches the 1.5 MAR) were received during March 2009 and the values contained in this report are based on these boundaries. The current expropriation line is still curved and the results will change (increase) when the expropriation boundaries are straightened.

2. SCOPE

The scope of the assignment was the determination of preliminary expropriation costs of agricultural land and structures such as farm houses and sheds situated within the expropriation boundaries. The findings must be presented on a property basis and the cost estimations must be based on values provided by a professional valuer. The team was also requested to identify agricultural land on which it is now uneconomical to farm as a result of the envisaged expropriation.

3. METHODOLODGY

The following processes and data sources were used and/or consulted to classify the land use and identify structures;

- SPOT satellite image acquired on 6 May 2006. The satellite image was overlaid with the property boundaries and the identified land uses were manually digitised and captured in a GIS to determine the extent thereof.
- Topo-cadastral maps 2330CB and 2330CD.
- Title Deed information obtained from the Registrar of Title Deeds.
- Land use information based on field surveys that were conducted between August 1994 and November 2001 in the Great Letaba River Catchment Area for the Department of Water Affairs and Forestry: Sub Directorate : Abstraction and Storage.

3.1 LAND USE

In terms of land use the following categories were used to classify the fields situated on each property:

- Orchards

  All identifiable orchards were digitised and classified based on visible evidence obtained from the SPOT satellite images and previous survey information. No attempt was made to differentiate between the different types of orchards (citrus, avocado's, etc.)

- Irrigated fields

  This typically includes cash crops under irrigation where the satellite image indicated fields with a high biomass value and includes *inter alia,* irrigation with centre pivots.

- Grazing/Veld

  Grazing and veld were not digitised from the satellite image and the values were calculated by subtracting the identified orchards and irrigated fields from the property extent situated within the expropriation boundaries.

3.2 STRUCTURES

The following categories were used to classify the different structures on each property:

- Farm houses and dwellings

  All identifiable farm houses and dwellings were digitised from the SPOT satellite image and their areas calculated.

- Sheds and outbuildings

  Sheds and outbuildings were identified and digitised from the SPOT satellite image and their areas calculated.

- Labour housing

  Labour housing was identified through visual inspection of the satellite image. No attempt was made to calculate the extent of every individual house and the area of each was accepted as 30 m^2.

3.3 VALUES OF AGRICULTURAL LAND AND STRUCTURES

Initial determination of compensation for land and improvements payable to owners affected by the project was restricted to a desktop study based on general land values applicable to the varying land use categories found in the area and on depreciated replacement cost for improvements.

The following land use and improvement categories were established for the affected area together with guideline values provided by a professional valuer.

Category	**Notes**	**Guideline values**
Grazing/Veld	Generally small pieces of uncultivated land but including grazing for livestock and game farming	R4,000 – R8,000/ha
Irrigation	Land equipped with infrastructure for irrigation purposes, e.g. mother lines, etc. but excluding surface irrigation systems, e.g. pivots. "Water rights" are included.	R30,000 to R50,000/ha
Orchards (irrigated)	Mostly comprises citrus orchards equipped with micro/drip irrigation. Compensation includes surface irrigation equipment and dams solely used for water storage as well as the "water right".	R30,000 to R120,000/ha
Improvements	Generally farm related improvements including: Dwellings Sheds and packhouses (equipment excluded) Labour housing Compensation will depend on degree of depreciation and application of the *Held* principle.	Replacement cost / m^2 R3,000 to R5,000 R500 – R2,000 ±R2,000

A copy of the report provided by the valuer is included in **Appendix A**.

4. RESULTS

By using the information contained in the GIS, the total area for each category on every property affected by the expropriation was calculated.

The **upper** values provided by the professional valuer for each of the categories were used to do an initial estimation of compensation payable. A summary is provided in the following table.

Category	Extent	Upper category value	Total compensation payable
Orchards (ha)	1 019	R 120 000	R 122 259 468
Irrigated fields (ha)	42	R 50 000	R 2 092 675
Grazing/Veld (ha)	2 947	R 8 000	R 23 575 823
Farm houses/Dwellings (m2)	1 184	R 5 000	R 5 922 950
Labour housing (m2)	2 310	R 2 000	R 4 620 000
Sheds/Outbuildings (m2)	10 643	R 2 000	R 21 285 800
Total			**R 179 756 716**

A detailed breakdown per property showing the calculated areas per category and compensation payable is included in **Appendix B.**

A map indicating all the identified fields, property boundaries and expropriation line is included in **Appendix C**.

During the course of the project a request was made to indicate properties where it is not possible to continue with viable farming practices following the expropriation of certain areas on individual properties. When such a study is to be undertaken accurately, it is necessary to evaluate various criteria such as gradients, soil types, accessibility to water, etc. These factors were however not taken into account when it was decided whether the continuation of farming practices are viable on a property or not. The main criterion was whether the remaining extent of a property outside the expropriated area, is large enough to re-establish the orchards and/or irrigation that will be lost due to expropriation. The accessibility to the property was also taken into account since there are cases where a property is divided into two sections due to the expropriation.

A map showing the properties where it is believed that viable farming are no longer possible is included in **Appendix D**.

F Joubert

APPENDIX A

Professional Valuer Report

Griffiths Valuations
Professional Valuers | Professionele Waardeerders

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The Project Leader
Groot Letaba Study

PRELIMINARY VALUATION METHODOLOGY – NWAMITWA DAM SITE

1. INSTRUCTION

The first phase of the determination of compensation for land and improvements payable to owners affected by the Nwamitwa Dam Project is restricted to a desktop study based on general land values applicable to the varying land use categories found in the area and on depreciated replacement cost for improvements. At this stage it is impossible to determine whether compensation for businesses or running concerns are affected and what the compensation thereof would entail, nor is it possible to determine the extent of depreciation for severance.

2. PRINCIPLES

The valuation principle applicable for the determination of compensation on expropriation is market value. The 1975 Expropriation Act, Act 63 of 1975, stipulates that compensation should not exceed the market value of the land plus an amount for actual financial loss caused by the expropriation.

The concept of Market Value reflects the collective perceptions and actions of a market and is the basis for valuing most resources in market-based economies.

Market Value is defined (precise definitions vary) as:

> The estimated amount for which a property should exchange on the date of valuation between a willing buyer and a willing seller in an arm's-length transaction after proper marketing wherein the parties had each acted knowledgeable, prudently, and without compulsion.

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3. VALUATION METHODOLOGY

Market Value is estimated through application of valuation methods and procedures that reflect the nature of property and the circumstances under which given property would most likely trade in the market.

There are generally three valuation methods, namely Comparable Sales, Income- and Cost. All Market Value measurement methods, techniques and procedures will if applicable and if appropriately and correctly applied, lead to a common expression of Market Value when based on market-derived criteria.

3.1 Comparable Sales

Sales comparisons or other market comparisons should evolve from market observations. The underlying principle is that a purchaser in the open market would not pay more for a specific property than what he would for an alternative property with similar features. In appropriate circumstances this method can be considered the best method and it is generally the method preferred by our courts, *Minister of Water Affairs v Von During,* 1971 (1) SA 858 (A) on 871A.

3.2 Income

Income capitalization should be based on market determined observations.

This method is applied mostly in relation to investment properties.

3.3 Costs

Construction costs and depreciation should be determined by reference to an analysis of market-based estimates of costs and accumulated depreciation.

This method is used in conjunction with the comparable sales method to determine compensation for improvements. In specific it applies to the calculation of compensation of actual financial loss in the case of partial expropriation or severance which results in the owner of land having to replace affected improvements on the remainder of the property in accordance to the *Held* principle, *Held v Administrateur-Generaal vir die Gebied van Suidwes-Afrika,* 1988 (2), SA 218 (SWA).

4. INPUTS

4.1 Land use categories and Improvements

Affected farms are broken up into the applicable land use categories found on the property, e.g. grazing, irrigation and orchards. These categories were observed in the field and are calculated preliminary by use of GIS data. The extent of improvements are also calculated by use of GIS data.

4.2 Guideline Values

The guideline values for the determination of a preliminary value for land and improvements affected by the proposed expropriation were determined by market research.

5. VALUATION CATEGORIES & GUIDELINE VALUES

The following land use and improvement categories were established for the affected area together with guideline values:

Grazing	Generally small pieces of uncultivated land but including grazing for livestock and game farming	R4,000 – R8,000/ha
Dry land	Generally absent due to unfavourable soil and climatic conditions	R4,000 – R8,000/ha
Irrigation	Land equipped with infrastructure for irrigation purposes, e.g. mother lines, etc. but excluding surface irrigation systems, e.g. pivots. Water rights are included.	R30,000 to R50,000/ha
Orchards (dry land)	Dry land orchards are generally absent due to unfavourable soil and climatic conditions.	R10,000 – R30,000/ha
Orchards (irrigated)	Mostly comprises citrus orchards equipped with micro/drip irrigation. Compensation includes surface irrigation equipment and dams solely used for water storage as well as the water right.	R30,000 to R120,000/ha
Lifestyle properties	Generally absent. Includes small holdings and farms for rural bushveld living. Depending on size, location and aesthetics.	R400,000 to R1,000,000 per property.
Improvements	Generally farm related improvements including: Dwellings Sheds and packhouses (equipment excluded) Labour housing Compensation will depend on degree of depreciation and application of the *Held* principle.	Replacement cost / m^2 R3,000 to R5,000 R500 – R2,000 ±R2,000

I trust that meets with your satisfaction.

Regards

DERRICK GRIFFITHS

Professional Property Valuer.

APPENDIX B

Detail breakdown per property

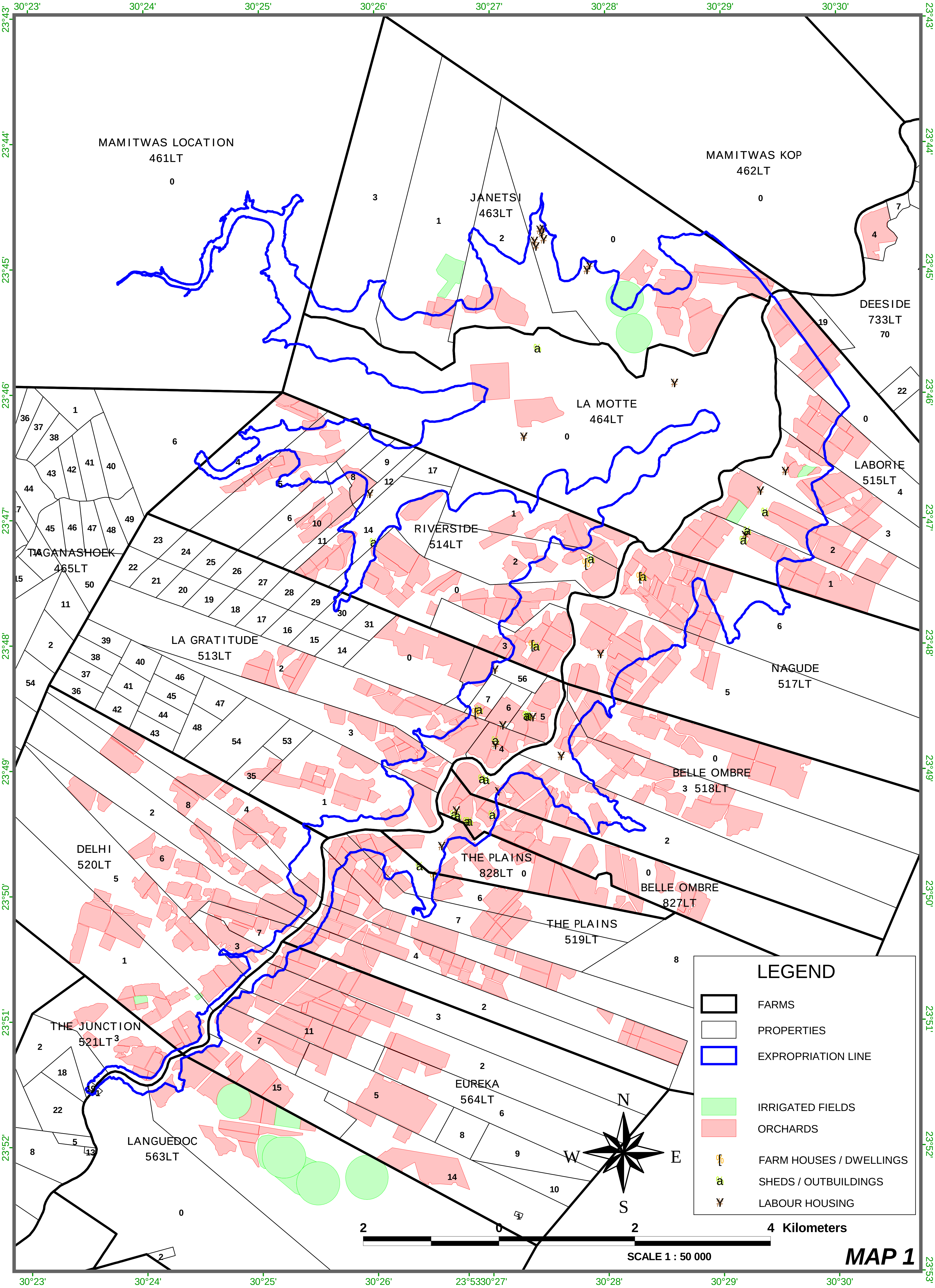
Property	Title Deed Extent (ha)	Property extent within expropriation area (ha)	Areas within expropriation borders						Expropriation costs						
			Orchards (ha)	Irrigation (ha)	Grazing (ha)	Labour housing (m^2)	Sheds (m^2)	Farm houses (m^2)	Orchards (R)	Irrigation (R)	Grazing (R)	Labour housing (R)	Sheds (R)	Farm houses (R)	Total cost (R)
461LT/0	6 848.8	95.3	0	0	95.3	0	0	0	0	0	762 287	0	0	0	762 287
462LT/0	2 705.1	1.0	0	0	1.0	0	0	0	0	0	7 602	0	0	0	7 602
463LT/0	755.3	330.6	78.5	34.2	217.9	240	0	0	9 422 604	1 710 115	1 742 920	480 000	0	0	13 355 639
463LT/1	449.3	70.4	8.5	0	61.9	0	0	0	1 020 720	0	495 172	0	0	0	1 515 892
463LT/2	257.0	142.0	22.7	0	119.3	0	0	0	2 724 384	0	954 711	0	0	0	3 679 095
463LT/3	449.3	38.1	0	0	38.1	0	0	0	0	0	304 612	0	0	0	304 612
464LT/0	1 460.7	1 163.3	46.3	0	1 116.9	120	631	0	5 559 600	0	8 935 533	240 000	1 263 940	0	15 999 073
465LT/6	257.0	2.3	0	0	2.3	0	0	0	0	0	18 363	0	0	0	18 363
513LT/1	257.0	60.1	16.5	0	43.6	0	0	0	1 985 412	0	348 706	0	0	0	2 334 118
513LT/2	386.6	33.3	20.4	0	12.9	0	0	0	2 446 320	0	103 526	0	0	0	2 549 846
513LT/3	171.3	31.2	17.0	0	14.2	0	0	0	2 040 852	0	113 262	0	0	0	2 154 114
513LT/4	41.7	41.7	30.0	0	11.7	90	228	0	3 596 352	0	93 618	180 000	457 860	0	4 327 830
513LT/5	47.1	47.1	31.6	0	15.5	90	613	0	3 791 376	0	124 116	180 000	1 226 400	0	5 321 892
513LT/6	21.4	21.4	12.0	0	9.4	180	0	0	1 438 800	0	75 386	360 000	0	0	1 874 186
513LT/7	21.4	20.8	2.8	0	18.0	0	465	167	331 152	0	144 098	0	930 520	838 800	2 244 570
513LT/29	21.4	0.4	0	0	0.4	0	0	0	0	0	2 953	0	0	0	2 953
513LT/30	21.4	2.6	0	0	2.6	0	0	0	0	0	21 046	0	0	0	21 046
513LT/35	28.4	0.7	0	0	0.7	0	0	0	0	0	5 418	0	0	0	5 418
513LT/56	53.5	34.2	1.1	0	33.0	30	0	0	135 600	0	264 233	60 000	0	0	459 833
514LT/0	244.8	59.1	23.1	0	36.0	0	0	0	2 772 924	0	288 182	0	0	0	3 061 106
514LT/1	171.3	63.8	18.0	0	45.7	0	0	0	2 164 080	0	365 978	0	0	0	2 530 058
514LT/2	171.3	63.5	24.9	0	38.6	0	285	256	2 982 660	0	308 990	0	571 600	1 284 950	5 148 200
514LT/3	128.5	73.4	49.9	0	23.6	0	117	473	5 983 368	0	188 584	0	234 940	2 366 900	8 773 792
514LT/4	171.3	37.6	0.2	0	37.4	0	0	0	24 264	0	299 444	0	0	0	323 708
514LT/5	171.3	64.0	18.8	0	45.3	0	0	0	2 253 348	0	362 141	0	0	0	2 615 489
514LT/6	51.7	2.1	0	0	2.1	0	0	0	0	0	16 926	0	0	0	16 926
514LT/8	38.5	24.7	4.4	0	20.3	0	0	0	529 212	0	162 383	0	0	0	691 595
514LT/9	21.7	19.6	0	0	19.6	0	0	0	0	0	156 542	0	0	0	156 542

Property	Title Deed Extent (ha)	Property extent within expropriation area (ha)	Areas within expropriation borders						Expropriation costs						
			Orchards (ha)	Irrigation (ha)	Grazing (ha)	Labour housing (m^2)	Sheds (m^2)	Farm houses (m^2)	Orchards (R)	Irrigation (R)	Grazing (R)	Labour housing (R)	Sheds (R)	Farm houses (R)	Total cost (R)
514LT/10	59.3	5.0	0	0	5.0	0	0	0	0	0	39 745	0	0	0	39 745
514LT/11	21.4	0.9	0	0	0.9	0	0	0	0	0	7 563	0	0	0	7 563
514LT/12	21.4	19.8	0	0	19.8	30	0	0	0	0	158 253	60 000	0	0	218 253
514LT/14	146.3	62.1	0.9	0	61.2	0	381	0	107 808	0	489 468	0	763 600	0	1 360 876
514LT/17	25.0	25.0	0	0	25.0	0	0	0	0	0	200 082	0	0	0	200 082
515LT/0	271.7	132.8	66.6	0	66.2	0	0	0	7 989 852	0	529 380	0	0	0	8 519 232
515LT/1	271.7	112.5	53.3	0	59.2	150	1 024	0	6 399 804	0	473 497	300 000	2 048 240	0	9 221 541
515LT/2	271.7	109.0	47.5	5.3	56.3	180	100	0	5 699 880	262 880	450 233	360 000	201 740	0	6 974 733
515LT/3	271.7	73.9	22.1	2.4	49.4	330	0	0	2 654 484	119 680	395 451	660 000	0	0	3 829 615
515LT/4	271.7	72.1	32.2	0	39.9	0	0	0	3 864 012	0	319 481	0	0	0	4 183 493
517LT/5	720.3	153.4	87.4	0	66.0	90	0	0	10 484 340	0	528 004	180 000	0	0	11 192 344
517LT/6	709.5	185.8	93.9	0	91.9	0	275	142	11 269 092	0	735 133	0	550 820	711 900	13 266 945
518LT/0	425.1	19.3	10.4	0	8.9	0	0	0	1 248 240	0	70 820	0	0	0	1 319 060
518LT/2	425.1	103.7	42.7	0	61.0	0	4 154	0	5 126 652	0	487 790	0	8 309 440	0	13 923 882
518LT/3	428.3	43.8	12.3	0	31.5	30	0	0	1 480 452	0	252 029	60 000	0	0	1 792 481
519LT/2	225.9	10.3	7.0	0	3.3	0	0	0	840 636	0	26 199	0	0	0	866 835
519LT/3	553.2	13.1	5.6	0	7.5	0	0	0	670 056	0	59 782	0	0	0	729 838
519LT/4	214.1	6.4	2.2	0	4.2	0	0	0	260 772	0	33 826	0	0	0	294 598
519LT/6	189.7	44.6	17.5	0	27.0	0	448	144	2 103 552	0	216 292	0	896 480	720 400	3 936 724
519LT/7	192.8	19.8	11.2	0	8.6	0	0	0	1 349 580	0	68 540	0	0	0	1 418 120
520LT/1	445.8	9.4	1.4	0	8.0	0	0	0	167 796	0	63 635	0	0	0	231 431
520LT/2	201.8	2.5	1.0	0	1.4	0	0	0	124 464	0	11 486	0	0	0	135 950
520LT/3	42.8	15.8	5.0	0	10.7	0	0	0	605 916	0	85 654	0	0	0	691 570
520LT/4	171.3	30.7	4.4	0	26.3	0	0	0	523 404	0	210 602	0	0	0	734 006
520LT/5	342.6	6.4	0.0	0	6.4	0	0	0	1 584	0	50 866	0	0	0	52 450
520LT/6	179.6	2.3	0.7	0	1.6	0	0	0	78 708	0	13 160	0	0	0	91 868
520LT/7	42.8	7.0	0.9	0	6.1	0	0	0	106 956	0	49 128	0	0	0	156 084
520LT/8	171.3	11.5	6.8	0	4.7	0	0	0	818 760	0	37 772	0	0	0	856 532
521LT/2	74.5	2.7	0	0	2.7	0	0	0	0	0	21 602	0	0	0	21 602
521LT/3	154.3	11.9	1.4	0	10.5	0	0	0	163 116	0	84 293	0	0	0	247 409
521LT/19	0.8	0.8	0	0	0.8	0	0	0	0	0	6 499	0	0	0	6 499

Property	Title Deed Extent (ha)	Property extent within expropriation area (ha)	Areas within expropriation borders						Expropriation costs						
			Orchards (ha)	Irrigation (ha)	Grazing (ha)	Labour housing (m^2)	Sheds (m^2)	Farm houses (m^2)	Orchards (R)	Irrigation (R)	Grazing (R)	Labour housing (R)	Sheds (R)	Farm houses (R)	Total cost (R)
563LT/0	641.9	7.8	0	0	7.8	0	0	0	0	0	62 393	0	0	0	62 393
563LT/1	1.3	0.3	0	0	0.3	0	0	0	0	0	2 301	0	0	0	2 301
563LT/3	1 239.1	13.2	3.7	0	9.4	0	0	0	447 192	0	75 534	0	0	0	522 726
564LT/2	306.1	10.0	2.4	0	7.6	0	0	0	285 852	0	60 555	0	0	0	346 407
564LT/3	257.0	8.1	5.2	0	2.9	0	0	0	622 404	0	23 301	0	0	0	645 705
564LT/7	72.8	5.8	0.5	0	5.3	0	0	0	58 140	0	42 278	0	0	0	100 418
564LT/11	85.7	2.7	0.3	0	2.4	0	0	0	41 556	0	19 030	0	0	0	60 586
564LT/15	119.5	5.4	0.5	0	4.9	0	0	0	60 312	0	38 866	0	0	0	99 178
827LT/0	428.3	47.1	31.5	0	15.6	210	1 915	0	3 781 896	0	124 733	420 000	3 830 220	0	8 156 849
828LT/0	204.2	49.0	13.5	0	35.5	540	0	0	1 619 172	0	283 835	1 080 000	0	0	2 983 007
TOTAL	**26 324**	**4 008**	**1 019**	**42**	**2 947**	**2 310**	**10 643**	**1 185**	**122 259 468**	**2 092 675**	**23 575 823**	**4 620 000**	**21 285 800**	**5 922 950**	**179 756 716**

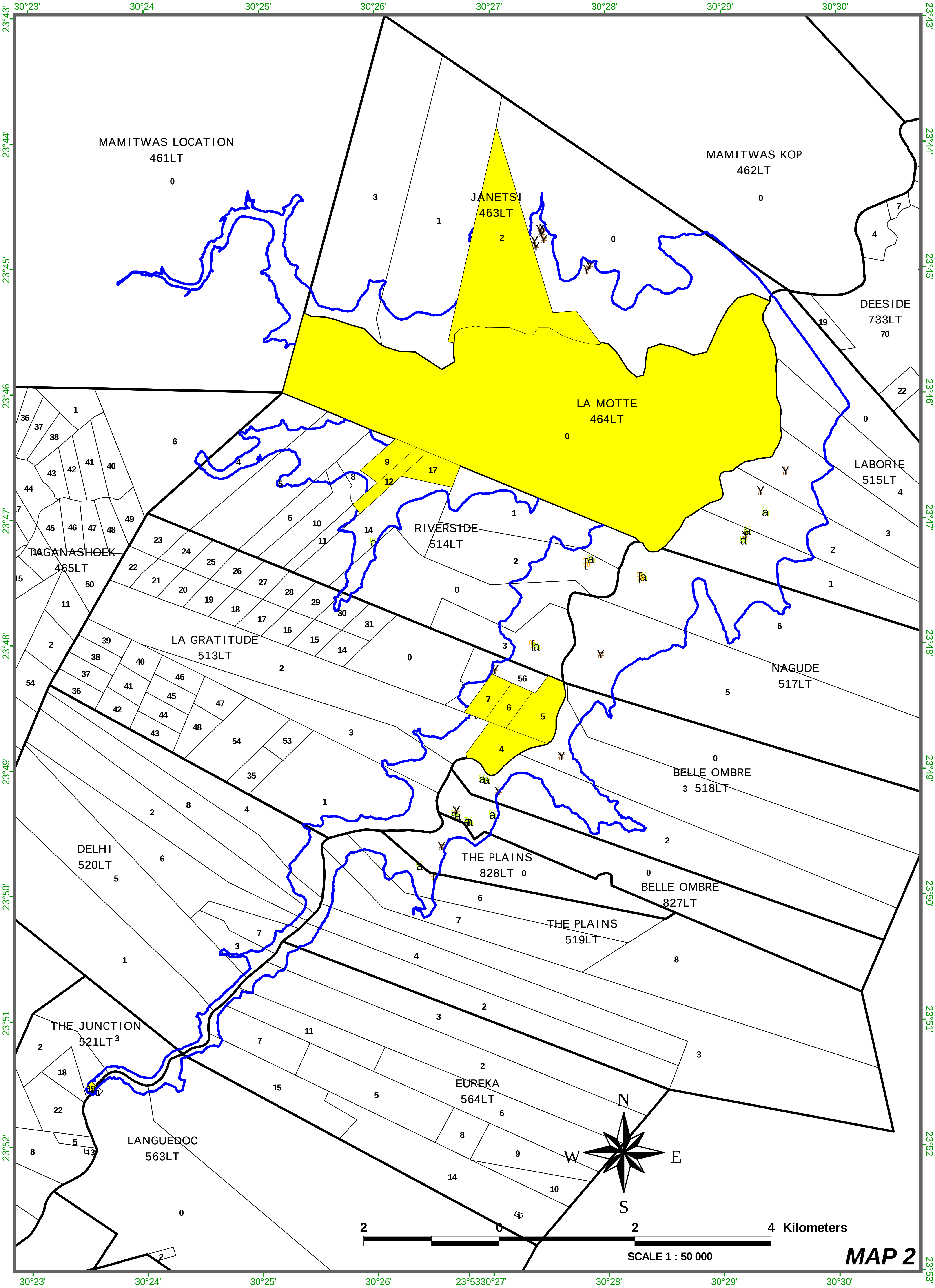
APPENDIX C

Map of expropriation area


MAMITWAS LOCATION
461LT
MAMITWAS KOP
462LT
JANETSI
463LT
LA MOTTE
464LT
DEESIDE
733LT
LABORIE
515LT
RIVERSIDE
514LT
TAGANASHOEK
465LT
LA GRATITUDE
513LT
NAGUDE
517LT
BELLE OMBRE
518LT
DELHI
520LT
THE PLAINS
828LT
BELLE OMBRE
827LT
THE PLAINS
519LT
THE JUNCTION
521LT
EUREKA
564LT
LANGUEDOC
563LT
LEGEND
FARMS
PROPERTIES
EXPROPRIATION LINE
IRRIGATED FIELDS
ORCHARDS
FARM HOUSES / DWELLINGS
SHEDS / OUTBUILDINGS
LABOUR HOUSING
N
S
E
W
2
0
2
4 Kilometers
SCALE 1 : 50 000
MAP 1
30°23'
30°24'
30°25'
30°26'
30°27'
30°28'
30°29'
30°30'
23°43'
23°44'
23°45'
23°46'
23°47'
23°48'
23°49'
23°50'
23°51'
23°52'
23°53'

APPENDIX D

Map of properties where farming is not viable

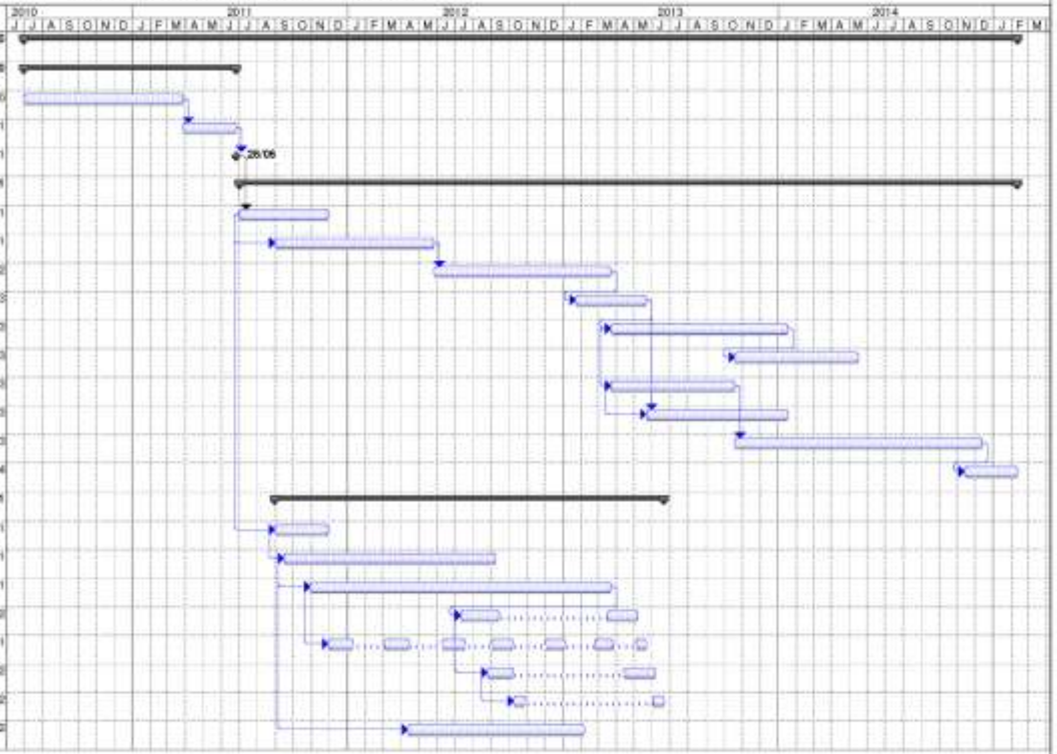

MAMITWAS LOCATION
461LT
MAMITWAS KOP
462LT
JANETSI
463LT
LA MOTTE
464LT
DEESIDE
733LT
LABORIE
515LT
TAGANASHOEK
465LT
RIVERSIDE
514LT
LA GRATITUDE
513LT
NAGUDE
517LT
BELLE OMBRE
518LT
THE PLAINS
828LT
BELLE OMBRE
827LT
DELHI
520LT
THE PLAINS
519LT
THE JUNCTION
521LT
EUREKA
564LT
LANGUEDOC
563LT
N
W
E
S
2 0 2 4 Kilometers
SCALE 1 : 50 000
MAP 2

APPENDIX G - Construction Programme

GROOT LETABA RIVER WATER DEVELOPMENT PROJECT

NWAMITWA DAM IMPLEMENTATION PROGRAMME

ID	Task Name	Duration	Start
1	**Nwamitwa Dam Tender & Construction Programme**	**1204 days**	**Thu 01/07/10**
2	**Design & Tender Phase**	**257 days**	**Thu 01/07/10**
3	Tender Design	9 emons	Thu 01/07/10
4	Tender Period	3 emons	Mon 28/03/11
5	Tender Award	0 days	Sun 26/06/11
6	**Dam Construction Phase**	**942 days**	**Fri 01/07/11**
7	Mobilisation	5 emons	Fri 01/07/11
8	River Diversion	9 emons	Tue 30/08/11
9	Spillway Excavation	10 emons	Sat 26/05/12
10	Spillway Grouting	4 emons	Mon 21/01/13
11	Outlet Block Concreting	10 emons	Fri 22/03/13
12	RCC	7 emons	Fri 18/10/13
13	Cut-off Trench Excavation	7 emons	Fri 22/03/13
14	Embankment Grouting	8 emons	Tue 21/05/13
15	Embankment Fill	14 emons	Fri 18/10/13
16	Demobilisation	3 emons	Wed 12/11/14
17	**Roadworks Construction Phase**	**472 days**	**Tue 30/08/11**
18	Roadbed	3 emons	Tue 30/08/11
19	Mass Earthworks	12 emons	Wed 14/09/11
20	Layerworks	17 emons	Sat 29/10/11
21	Surfacing	10 emons	Tue 10/07/12
22	Structures (small)	18 emons	Mon 28/11/11
23	Road Marking	9.5 emons	Fri 24/08/12
24	Finishing	8.5 emons	Mon 08/10/12
25	Bridges	10 emons	Wed 11/04/12



Project: Construction Programme Rev
Date: Tue 04/05/10

Task | Progress | Summary | External Tasks | Deadline
Split | Milestone | Project Summary | External Milestone

Page 1

APPENDIX H - Drawings (See separate Volume 6 – Annexure 4: Appendix H: Drawings)

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

APPENDIX I – Comments

I1	Comments by DWAF Directorate : Civil Engineering
I2	Comments by Knight Piesold (Pty) Ltd
I3	Comments by Prof A van Schalkwyk
I4	Aurecon's response to comments

APPENDIX I.1 : Comments by DWAF 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.

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

- ***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 I.2 : Comments by Knight Piesold (Pty) Ltd

Knight Piésold
CONSULTING

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Your Ref:

Our Ref: 3030018301 CJA Rvw 01-1

Contact: CJ Abrahamson

4th January 2010

BKS Consulting Engineers
Block D, Hatfield Gardens
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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 m³ it would increase the system yield by about 47 million m³ per year. (By comparison, the capacity of Tzaneen Dam is 157,5 million m³).

It was also proposed to increase the capacity of Tzaneen Dam to approximately 203 million m³ by raising the dam wall. This could increase the firm yield of the dam by about 6% from 60 million m³/a to 64 million m³/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

- 4^{th} 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 5^{th}, 6^{th} & 7^{th} 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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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 I.3 : Comments by Prof A van Schalkwyk

(PTY) LTD

ENGINEERING AND MANAGEMENT

REG. NO. 1966/006628/07

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23 October 2009

Aurecon SA (Pty) Ltd
81 Church Street
CAPE TOWN
8001

Attention: Mike Killick

Dear Mike

GROOT LETABA RIVER WATER DEVELOPMENT PROJECT (GLeWaP) DRAFT MAIN REPORT ON THE TECHNICAL STUDY MODULE: PCMT COMMENTS

Herewith the Draft Report P02/B810/00/0608 – Volume 1 dated June 2009 with my comments and suggestions for finalization. Please nominate a day on which this can be discussed in Pretoria as a means for expediting finalization.

Also enclosed is a copy of comment by Monty van Schalkwyk on your draft report on the Geotechnical Investigations for the Bulk Water Infrastructure and for Nwamitwa Dam.

Yours sincerely,

RA Pullen PrEng
Project Coordinator

P:\H5057 - Groot Letaba Bridging Studies\H5057A Project Administration\5 Correspondence\Letter Mike Killick Technical Study Module Oct 2009.doc

GROOT LETABA RIVER DEVELOPMENT PROJECT: NWAMITWA DAM REVIEW OF GEOTECHNICAL INVESTIGATIONS FOR PRELIMINARY DESIGN

BY

A VAN SCHALKWYK

AUGUST 2009

1. INTRODUCTION

BKS (Pty) Ltd requested the author to review a report by Africon (Pty) Ltd on the geotechnical investigations for preliminary design of the above dam. The section of the report dealing with the re-alignment of roads is not included in the review.

The proposed Nwamitwa Dam is a composite structure with maximum height above river bed level of about 32m and crest length of about 3 200m. It will comprise two long embankments on the flanks and a 190m wide ogee-type concrete gravity overspill located across the river channel and on the lower left flank. The total length of the concrete structure, including the tongue walls is about 350m.

2. GEOTECHNICAL INVESTIGATIONS

The Council for Geoscience conducted reconnaissance level geotechnical investigations during 1994, followed by feasibility level investigations during 1996. Africon (Pty) Ltd conducted a post-feasibility bridging investigation for preliminary design during 2008. The results of all the geotechnical investigations are contained in Appendix B: "Geotechnical Investigations" of the Preliminary Design report by Africon (Pty) Ltd, dated June 2009.

2.1 DAM SITE

Investigations at the dam site comprised a desk study of available information, field geological mapping using aerial photograph interpretation (API), geophysical surveys, test pitting, core drilling, Lugeon water pressure tests, laboratory tests and a seismic hazard assessment.

2.1.1 Desk study

The report refers to information from published and unpublished geological maps. However, no regional geological maps are included in the report to show the distribution of rock types, dykes and other geological structures in the dam basin and surrounding area.

2.1.2 Geological mapping

There are no rock outcrops along the proposed dam centre line, and the distribution of surface soils and lineaments as identified on airphoto's, is presented as a strip map along the centre line.

2.1.3 Geophysical surveys

Seismic refraction, apparent conductivity and magnetic surveys were conducted along lines A and B on the left flank and lines C, F and G along the right flank. The surveys were not extended across the river. The positions of two lines, D and E along the river banks are shown on the plan, but no results are provided.

The absence of data across the river section means that only about half of the founding area for the proposed concrete spillway section was surveyed. Seismic basement velocities were generally around 3 000m/s. These velocities are considered low for granite and gneiss, and indicate a degree of jointing and perhaps weathering that might extend to considerable depths.

Except for BH 1213 that was drilled to investigate a seismic anomaly on the left flank, there is no indication that the results of the geophysical surveys have been taken into account for the positioning of boreholes, the compilation of the geological map or the assessment of founding levels.

2.1.4 Test pitting

A total of 11 test pits were dug along the centre line to depths of between 1,2m and 3,2m using a Bell 315 TLB. In the river section, the TLB could not reach bedrock, while along the flanks it refused on soft to medium hard rock. For this investigation, the use of a heavy tracked excavator would have been more appropriate.

2.1.5 Core drilling

A total of 19 cored boreholes, totalling about 569m in length, were drilled by DWAF along the proposed upper centre line. Core recovery in the upper layers of weathered and fractured rock was generally poor and this negatively affects the reliability of predicted founding conditions.

2.1.6 Lugeon water pressure testing

Water pressure tests were only done in one of the boreholes drilled during 1996 and in 8 of the borehole drilled during 2008. In the upper parts of the holes (typically 6m – 10m), the packers could not seal, and there is a gap in the information on permeability between these levels and the bottoms of test pits (average depth 2,5m).

2.1.7 Laboratory testing

Disturbed samples from the test pits were subjected to Grading, Atterberg, Compaction, Shear Strength and Permeability testing, while the Moisture Contents and Dry Densities of a few undisturbed samples were determined.

A large number of Point Load tests were conducted on core samples, while six core samples were subjected to tests for Uniaxial Compressive Strength, Modulus and Poisson's Ratio.

2.1.9 Seismic hazard analysis

A well-known expert in the field, did the seismic hazard analysis, and the results appear to be correct.

2.1.10 Summary

The scope of the geotechnical investigation is summarised in **Table 1**. Based on the guidelines for dam site investigations (Van Schalkwyk, 1983), it is evident that only the right hand side of the spillway section had been reasonably well investigated for feasibility (and preliminary design) purposes.

Acceptable levels of completeness could have been achieved by the following additional investigations:

- Five cored boreholes in the left side of the spillway section.
- Six excavator test pits on the left flank.

- Ten excavator test pits along the right flank.
- Four shallow boreholes and infiltration tests on left flank
- Six shallow boreholes with infiltration tests on right flank.
- Ten DCP tests on the alluvium along the lower flanks.

Table 1. Summary of geotechnical investigations

	LEFT FLANK	SPILLWAY SECTION		RIGHT FLANK
		LEFT SIDE	RIGHT SIDE	
LENGTH (m)	1220	200	150	1850
MAX. HEIGHT (m)	23	32	32	22
GEOPHYSICS (m)	930	170	100	1800
TEST PITS (no)	3	0	3	5
TP SPACING (m)	310		30	360
TP DEPTH (m)	6,9	0	9,3	8,8
BOREHOLES (no)	6	1	6	6
BH SPACING (m)	200	200	25	310
BH LENGTH (m)	135,4	19,5	282,5	131,7
SCOPE OF INVESTIGATION (% of guideline)*	51	17	86	48

* Van Schalkwyk, 1983.

2.2 ROCK QUARRIES

Two potential quarry sites (A and B) were located on high-lying ground at distances of 15km and 19km respectively from the dam site. Seismic refraction, conductivity and magnetic surveys were conducted at both sites, and three vertical cored boreholes were drilled at each. Core samples were submitted for Petrographic, Point Load Strength and Crushing tests.

At both sites there appear to be dolerite intrusions, while the boreholes and seismic surveys indicate overburden thickness varying between about 5m to about 12m.

2.3 SOIL BORROW AREAS

The Materials Laboratory of DWAF conducted investigations for sources of impervious, semi-pervious and fine aggregate materials during 1996. A potential borrow area for embankment materials was identified below FSL of the dam on the right bank of the Groot Letaba River and investigated by means of 450mm diameter auger holes, drilled on a 200m grid. Extensive laboratory testing was conducted and it was proved that 952 000m^3 of impervious material and 935 000m^3 of semi-pervious material were available.

Potential deposits of fine aggregate were identified in the Phatle/Lervatlou Rivers and the Marekome River, located respectively about 11km and 20km from the dam site. About 162 000m^3 of sand was proved, while there appears to be other potential sources in the area.

4 INTERPRETATION OF RESULTS

4.1 DAM SITE

The flanks are underlain by shallow (generally less than 2m) of alluvial, colluvial and residual soils above variably weathered granite. The upper horizon of bedrock is described in the test pits as highly weathered, grey brown, very soft (friable) to medium hard granite. Sound bedrock levels vary between about 6m and 20m on the left flank and typically over 20m along the right flank. Geophysical anomalies indicate the possible presence of dolerite dykes and fracture zones that intersect the centre line and could represent zones of even deeper weathering. The report also states that troughs and depressions in the bedrock profile on the left flank might represent buried alluvial palaeo-channels.

Alluvial deposits varying in thickness between about 9m and 16m underlie the river section. Beneath the alluvium, there is a layer of weathered granite that varies in thickness between 0m and 6m with local zones of weathering extending to more than 40m. The lineaments intersecting the river section might represent a fault or faults, indications of which were found in BH1205.

Africon have derived minimum founding criteria by back-calculating Rock Mass Ratings (RMR) from the assumed required foundation deformation modulus as summarised in Table 2 (from Table 7.3 in the Africon report).

For concrete gravity dams, the problem with the above criteria is that the joint conditions cannot be assessed from borehole cores. By specifying moderately weathered rock and a RQD of 80%, it is still possible that 20% of the core had been lost during drilling and that the lost parts represent soft joint fill material. The reviewer recommends that a parameter for core recovery (>98%) be added, and that only slightly weathered or unweathered rock be permitted. The use of RMR as founding criterion is not recommended. The reviewer also does not agree with the use of straight lines between sound rock levels found in boreholes to portray excavation lines for design purposes. In predicting excavation lines, the one-dimensional nature of a borehole must be considered in relation to the rock mass, and the results of surface and/or geophysical investigations and experience from other similar sites must be taken into account. The reviewer is of the opinion that the recommended excavation line for the concrete section could lie between 4m and 6m deeper than the depths indicated by individual boreholes.

Table 2. Minimum founding criteria (after Africon, June 2009)

Foundation criteria	Envisaged dam components		
	Mass concrete, gravity	Core	Rockfill shells
Modulus (GPa)	10	2	1
RMR or GSI	50	30	20
Degree of weathering	Moderate to unweathered	Moderate to slight	High to moderate
Rock material strength (MPa)	100	50	25
RQD (%)	>80	>40	>20
Joint spacing (mm)	>200	60-200	<20
Joint condition	Slightly rough surfaces Separation <1mm Med hard walls	Open continuous Or Infil<5mm Or slickensided	Continuous Or open >5mm Or soft infill >5mm

It was noted that the foundation design was based on a minimum RMR of 40. This is not in accordance with the geotechnical report and is considered by the reviewer as over optimistic. The preliminary design report also does not give the values for cohesion and friction angle from which the FOS against sliding was calculated.

For the core of an embankment dam, the use of a modulus value as foundation criterion is not appropriate. The main criterion should be either watertightness or groutability. Unfortunately, the results of the investigation do not provide permeability information on the critical zone of weathered rock between the recommended core founding depth (about 4m) and the depth at which water pressure tests were started (about 6m to 10m). The reviewer is of the opinion that deep test pits and infiltration tests in shallow boreholes should have been used to assess the properties and permeability of this zone. It was noted that the recommended core founding level on the right flank does not meet the above founding criterion at any of the borehole positions.

Founding criteria for rockfill shells are probably not relevant at this site, since rock will have to be transported from distant quarries that had not been proved. For earth shells, only nominal excavation will be required along the major parts of both flanks. Only close to the river section where there are thick alluvial deposits, deeper excavations may be required. The alluvium was described as medium dense, while dry density tests on a few samples taken a shallow depths showed in situ densities of between 80% and 90% of the Standard Proctor density. For present costing purposes (until further DCP and possibly consolidation testing is done), it is recommended that this material be removed.

4.2 ROCK QUARRIES

Laboratory test results show the unweathered granite to be generally suitable for use as concrete aggregate, but that the aggregate might be prone to Alkali Aggregate Reaction (AAR).

The volume of rock required for construction is not mentioned in the geotechnical report, and no attempt was made to determine the available volumes of suitable material or the volume of overburden that will have to be stripped. From the design report it appears that about 200 000m^3 of rock for concrete aggregate and 90 600m^3 for riprap will be required. That means a source of at least 450 000m^3 has to be proved.

Suitable quarry sites for rock material are not easy to find in areas with low Weinert N-values and underlain by granitic rocks. At Tzaneen and Magoebaskloof Dams, no quarry sites could be located, while at Injaka Dam several sites were unsuitable, and the one finally used proved to be problematic. Much more investigation work, particularly core drilling will have to be conducted in order to prove sufficient material for concrete aggregate and rip-rap.

Other problems associated with the proposed quarry sites are environmental sensitivity (visible locations and proximity to other developments) and distance from the dam site.

The reviewer is of the opinion that insufficient investigations have been conducted to determine the volume of overburden and to prove the required quantity and quality of material for concrete aggregate and rip-rap.

4.3 SOIL BORROW AREAS

According to the design report, 1 640 000m^3 of impervious material and 1 340 000m^3 of semi-pervious material has to be proved. If material from necessary excavations can be used as fill, there would be sufficient volumes of semi-pervious material. The DWAF Materials Report mentions that additional sources of pervious materials could possibly be found on the right bank of the river. This will have to be investigated.

Nearby rivers had deposited good quality sand for use as fine aggregate and filter materials, and about 162 000m^3 of sand were proved. These materials will have to be transported over distances of between 15km and 20km. According to the design report, about 200 000m^3 of material for filters and fine aggregate would be needed, and additional investigations will therefore have to be conducted.

5. CONCLUSIONS

5.1 Investigations for rock quarries were not sufficient to determine the volume of overburden and to prove the required quantity and quality of material for concrete aggregate and rip-rap. There is a major risk that the proposed quarry sites will be very difficult to work due to variable weathering conditions, and that excessive volumes of overburden will have to be removed.

5.2 Investigations for the concrete spillway section (particularly the left side) were not sufficient to make a reliable assessment of founding depths, and the interpretation of the available results is considered to be over optimistic. There is a large risk that excavations for the concrete section will be on average between 4m and 6m deeper than shown on the design drawings.

5.3 Along the flanks, there is a gap in information on permeability and groutability of the zone between the bottoms of test pits (about 2,5m) and the depths where water pressure tests were done (6m - 10m). There is a moderate risk that the material below the proposed excavation for the core trench will be moderately permeable, possibly prone to piping, and not groutable.

5.4 Although insufficient volumes of impervious fill and sand had been proved, the risk that other sources cannot be found is considered small.

A v Schalkwyk

A van Schalkwyk

31 August 2009

APPENDIX I.4 : Aurecon's response to comments

1. COMMENTS RECEIVED

1.1 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
- Prof A van Schalkwyk

The comments, as well as Ninham Shands' response, are attached to the report as Appendix J. 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 13 of report

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

1.2 DWAF DIRECTORATE : CIVIL ENGINEERING

A) : <u>GENERAL COMMENT</u>

1) The official title of the study is "Groot Letaba Water Development Study"

2) Noted

3) Added executive summary

4) Noted

B) : <u>NWAMITWA DAM : PRELIMINARY DESIGN REPORT</u>

- Par 1.1 *Inserted reference on page 1*
- Par 1.1 *Inserted reference on page 2*
- Table 2.1 *Added spillway height*
- Par 3.1 *Detailed design*
- Par 4.3.4 *Inserted drawing number*
- Par 4.3.6 *Appendix B submitted as separate volume*

- Par 5.4 *Semantics – not incorporated in report*
- Par 5.5 *Detailed design*
- Par 7.3.4 *Reference to previous version of drawings*
- Par 9.1.2 *Changed reference to "the owner"*
- Par 11.5 *Inserted reference*
- Par 12 *Included construction programme*
- Conclusions / Recommendations *Added*

C) : <u>NWAMITWA DAM : APPENDICES</u>

- Appendix C.2 of Appendix D.1 *Part of Selection Study – superseded by drawings in Appendix H*
- Appendix C.5 *Wind speed ratios correct as shown*

D) : <u>NWAMITWA DAM : APPENDIX H</u>

- Drg No 401775 CEN 210 *Survey as received from DWAF*
- Drg No 401775 CEN 212 *Detailed design*
- Drg No 401775 CEN 216 *Sloped u/s slope will ensure positive compaction of core material against concrete face – ogee cap to be constructed with conventional concrete – apron length part of detailed design – alignment of relocated roads shown in Figures 9.1 and 9.2.*
- Drg 401775 CEN 271 *Detailed design*

1.3 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.

6.5.1 Spillway Design Floods and Freeboard

- Justification of 6.5 m freeboard *Retained text as is*

6.5.2 River Diversion

- Second paragraph *Re-evaluate during detailed design*

6.5.4 Outlet Works

- Second paragraph *Amended text*
- Third paragraph *DN = Nominal diameter of pipe*

  Added to Abbreviations

6.5.6 Sedimentation

- Table *t/km^2.a is international unit*
- Last paragraph *Covered in Preliminary Design Report*

6.6 Re-alignment of Roads

- Various comments *Amended text*

6.6.1 Affected Roads

- Various comments *Amended text*

6.6.2 Bridges

- Various comments *Amended text*

1.4 KNIGHT PIESOLD (PTY) LTD

1 INTRODUCTION

No comment

2 GENERAL OVERVIEW

Noted – no further response

3 NWAMITWA DAM

3.1 Items not available for review

Noted

3.2 Specific comments

3.2.1 Executive Summary *Included in report*

3.2.2 Section 1 – Introduction

• Bridging study	*Retained text as is*
• Cross references	*Amended report*

3.2.3 Section 2 – Principal Details of Proposed Nwamitwa Dam

• Firm yield	*Corrected unit*
• 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 3.1*
• Dam site	*Selected downstream of confluence – acknowledged as poor dam site*
• Max base width	*Max embankment height 23 m and not 34 m*
• NOC level	*Corrected*
• Spillway discharge	*Retained text as is*
• Spillway elevation	*Retained text as is*

3.2.4 Section 3 – Hydrology

• Designation of RI	*Corrected text*
• 3.1 Spillway Floods	*Noted*
• 3.2 and 3.5 Diversion	*Retained text as is*
• 3.5 Diversion Strategy	*Detailed design*
• 3.5.1 Diversion Floods	*Corrected catchment areas and floods*
• 3.5.2 First Stage	*Noted*
• 3.5.3 Second Stage	*Stated bed level is approximate*

3.2.5 Section 4 – Geology and Geotechnics

• 4.3.4 Rotary Drilling	*Inserted drawing number*

3.2.6 Section 5 – Materials

• 5.3 Semi-pervious Material	*Detailed design*
• 5.5 Available Volumes	*Detailed design*

3.2.7 Section 6 – Embankment

• 6.4.3 Cut-off Trench	*Detailed design*
• 6.4.7 D/s slope protection	*DWAF preference – detailed design*
• 6.5 Filter criteria	*Detailed design*
• 6.6.1 Stability Analysis	*Detailed design*
• 6.6.2 Stability Results	*Too conservative*
• 6.7 Grouting	*Detailed design*

3.2.8 Section 7 – Spillway

• 7.4.2 Structural Loadings	*Amended text*
• Table 7.2 Results	*Noted*

3.2.9 Section 8 – Outlet Works

• 8.3 Description of Works	*Detailed design*

3.2.10 Section 9 – Relocation of Roads

• 9.1.2 Missing Name	*Changed reference to "the owner"*

3.2.11 Section 11 – Cost Estimates

• 11.1.1 Introduction	*Added LHWC to List of Abbreviations*
• 11.1.2 Payment items	*Clearing refers to footprint of embankment and appurtenant works – refer to quantities* *Number of secondary holes very dependent on geology – minor impact on total cost*

3.2.12 Section 13 – References

• Numbering	*Retained text as is – reference via name of author*

1.5 PROF A VAN SCHALKWYK

2 GEOTECHNICAL INVESTIGATIONS

2.1.1 Desk study

- Regional geological maps — *Although no regional geological map is included in the Preliminary Design report, the available maps were studied and reference to the published geological map is included in the report. This regional information was included in the previous geological reports for the proposed dam.*

2.1.2 Geological mapping

- Rock outcrops — *While it is true that no rock outcrop occurs on the dam footprint, outcrop is recorded a short distance downstream of the centre-line and this is indicated on the engineering geological site plan.*

2.1.3 Geophysical surveys

- Data in river section — *The results of the seismic surveys on the respective river banks are available. Investigation of geophysical anomalies and verification of actual geological conditions is not limited to BH 1203. A number of other anomalies are in fact investigated by boreholes (for example BH1201 drilled at the position of anomalies in the apparent conductivity, and BH 1211 investigates an anomaly in the seismic refraction results).*

2.1.4 Test pitting

- Use of tracked excavator — *Plant provide by DWAF – full access not*
    - *available in citrus orchards.*

2.1.10 Summary

- Scope of investigation — *Recommendations noted – further work part of detailed design.*

4 INTERPRETATION OF RESULTS

4.1 Dam site

The approach of defining minimum founding criteria by back-calculating the Rock Mass Ratings has been adopted for many recent large dams (Berg River Dam, De Hoop Dam etc), and has found general acceptance from the various design teams and the expert review panels. Certainly cognizance must be made of material losses in the assessment and the reviewer's recommendation for inclusion of a minimum core recovery of 98% is noted.

Straight lines used to extrapolate information between boreholes is certainly not representative of actual conditions and it is for this very reason that such lines are incorporate, i.e. to reflect an 'unnatural' oversimplified model. Ideally such information would be presented as a contoured surface, but with the limited number of data points available the end result would then have been a further misrepresentation of the accuracy of the information in hand.

The point that the main founding criteria for the impervious core should be watertightness or groutability is noted. For reasons mentioned above, deeper investigations (i.e. requiring large trenches) to characterize the upper weathered zone of bedrock could not be conducted for these preliminary design investigations will have to be conducted during detail design investigations.

4.2 Quarries

The point that expanded investigation of proposed hard rock quarry sites is required is noted. Budgetary and time constraints did not allow such comprehensive investigations at the time.

4.3 Soil borrow areas

The point that further investigations are required is noted. At the time of the investigations, the Study Team made the decision to accept the previous DWAF report at face value and not to conduct further investigations of these materials at that stage.

5 CONCLUSIONS

There is agreement with the comment that additional investigations of the proposed hard rock quarry sites are required.

While additional investigations will be required at detail design level, the statement that investigations of the spillway section were insufficient is at least partly based on an incorrect tally of the investigations conducted.

There is agreement that additional investigation of the upper, weathered rock horizon on the flanks is required. Such investigations could not be conducted at the time of the preliminary design investigations.

There is agreement that insufficient volumes of embankment materials have been proved, and further investigations are required in this regard.