SECTION 2 : HYDROLOGY

2.1 INTRODUCTION

This section deals with the derivation of flow duration relationships, the estimation of flood peaks and the computation of water surface profiles along the Okavango River.

The flow duration relationships provide information on the percentage of time that any given flow is equalled or exceeded. This information is required for sizing and optimisation of hydro power generation.

Flood peaks are required for the sizing and design of the outlets and spillways at the identified weir sites.

The water surface profiles are necessary to establish tailwater levels at the weir sites, as required for the preliminary design of the weirs, for positioning the turbines and also to establish the headwater levels (or inundation) upstream of the weirs.

2.2 FLOW DURATION RELATIONSHIPS

2.2.1 OBJECTIVES

The objective of establishing flow duration relationships is to provide information for the hydro power analysis. These relationships have been derived for each month, representing the percentage of time that any given flow is equalled or exceeded. This is required in the hydro power analysis to optimise the number of turbines and their capacities. Relationships are prepared for each month because there is typically a seasonal variation in flow available for hydro power generation.

2.2.2 AVAILABLE DATA

There are two flow gauging stations on the Okavango River that record daily flows upstream of the proposed Popa Falls Hydro Power Project. These are at Rundu and Mukwe. The Cuito contribution to the flow in the Okavango River downstream of the confluence of the Cuito River, is the difference between the Mukwe and the Rundu flow data.
The latter is very close to the proposed site and can be used without adjustment. Records were provided by the Department of Water Affairs (**DWA**) of the Ministry of Agriculture, Water and Rural Development (**MAWRD**) for both stations for the periods of 1 November 1945 to 30 November 2002 and between 1 October 1949 and 30 September 2002 respectively. For each station the record was provided in two formats. One was in a tabular format grouped in calendar years and giving daily flows in 12 columns representing each month. The other format is a single date entry per line for each day. This record contains an evaluation code for each data point and was useful in scrutinising the data for relevance and accuracy. Due to the extent of the data it is not included in the report, but can be made available electronically. For the Rundu record the last five years were rejected in total due to incomplete observations; the last four years of the Mukwe record were not used in the analysis as some records still need

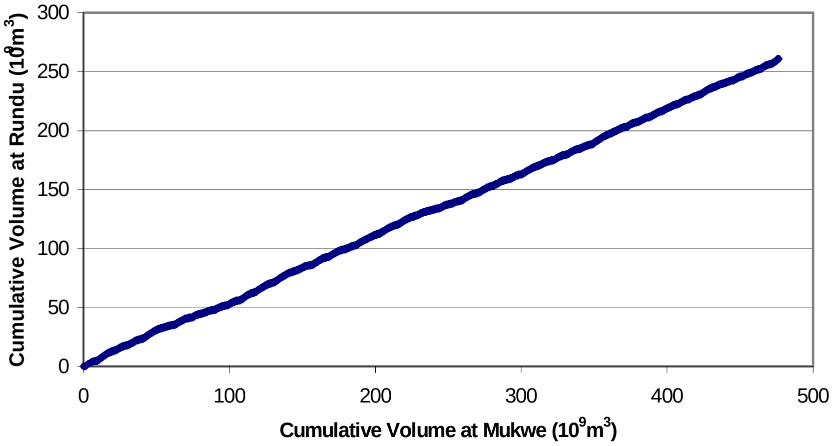
gaps filled in. The addition of these years would in any event not have changed the long term statistics of the used flow series.

2.2.3 METHOD OF ANALYSIS

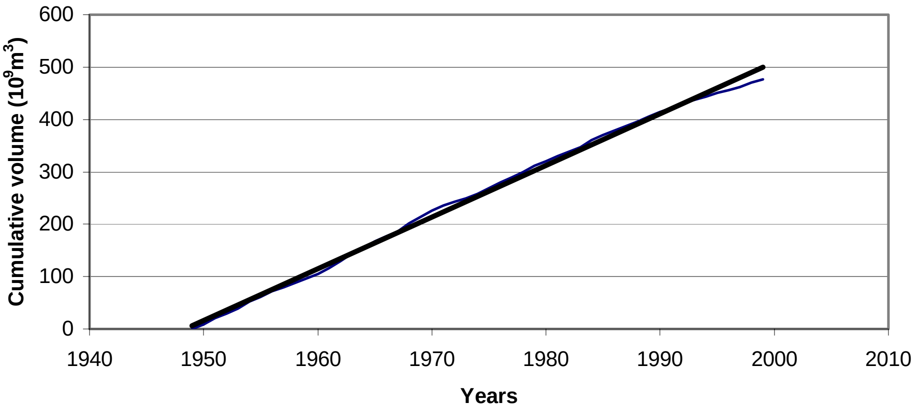
Both records were analysed for consistency in terms of normal flow and flood peaks. They were also compared to each other by means of a double mass plot, as shown in **Figure 2-1**. There was a good correlation between the two records, confirming the absence of major deviations. Single mass plots for Mukwe and Rundu are shown in **Figures 2-2** and **2-3** respectively. Both of these graphs show a slight deviation from the straight line from about 1995. Such deviations can be indicative of changed conditions in the catchment (which is very unlikely), of changes in the river cross-section at the gauging station, or of a revised stage-discharge formula. Data for at least another 5 years will be required to ensure that the deviations are not just normal fluctuations.

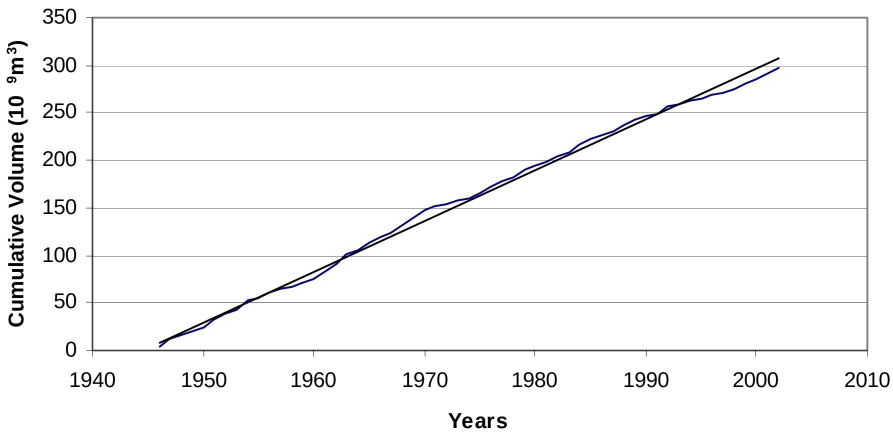
After the above initial evaluations the Mukwe record was screened and all values with unclear origins or obvious inconsistencies were removed. These included estimated values with low reliability and flagged as such in the record as well as values that were either uncharacteristically low or high when compared to adjoining values.

The remaining values were separated for individual months by means of a specially developed small utility computer program. These 12 monthly series and the full annual series were then ranked from highest flow to lowest flow. For each of these sequences the relevant flow exceedence values were calculated in 0% to 100% increments.


Cumulative Volume at Rundu (10^9m^3)
300
250
200
150
100
50
0
0
100
200
300
400
500
Cumulative Volume at Mukwe (10^9m^3)

Fi **Iukwe**


Cumulative volume (10^9m^3)
600
500
400
300
200
100
0
1940
1950
1960
1970
1980
1990
2000
2010
Years


Cumulative Volume (10^9m^3)
350
300
250
200
150
100
50
0
1940
1950
1960
1970
1980
1990
2000
2010
Years

2.2.4 RESULTS

The results obtained from the various sequences are provided in **Table 2-1**, while graphic presentations of the flow duration relationships, or curves, are shown in **Figures 2-4** and **2-5** for the different months and in **Figure 2-6** for the annual flows.

Table 2-1 : Flow Duration Relationships for Mukwe

% Exceedence	River Flow (m^3/s)												
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
0	1086	1469	1313	1292	1017	596	489	442	341	233	277	621	1468
5	457	692	828	997	750	484	362	292	237	189	205	313	670
10	392	586	762	851	665	441	328	265	219	178	185	270	554
15	357	534	672	757	617	414	309	251	209	172	173	251	476
20	343	495	627	721	575	393	297	240	202	166	166	236	424
25	325	451	600	693	550	375	288	233	195	161	161	226	383
30	307	423	569	654	529	357	280	226	189	157	157	216	349
35	289	405	544	617	504	345	270	222	185	154	152	208	320
40	277	384	505	585	485	336	262	215	179	151	149	201	293
45	269	368	467	555	470	324	254	205	175	147	148	195	269
50	261	353	447	529	451	312	247	199	170	143	145	188	247
55	251	337	432	499	433	300	233	195	166	140	140	179	229
60	243	328	411	471	410	289	223	188	163	137	137	170	210
65	232	316	393	455	388	277	215	185	160	134	132	166	195
70	219	305	375	438	367	265	205	180	157	131	128	157	182
75	205	292	354	423	349	254	199	176	152	128	126	151	170
80	189	277	340	400	332	243	192	172	148	125	123	146	160
85	173	257	325	383	316	230	185	164	140	120	120	140	149
90	163	230	309	367	297	215	176	154	134	112	105	134	139
95	151	205	277	345	266	189	158	145	126	102	95	123	126
100	126	140	216	254	182	149	142	126	104	85	80	95	81
Mean	280	396	361	573	467	320	250	208	175	145	146	197	292

The results show that there is a significant variation in flow for the different months and that this will have a definite influence on the availability of flow to generate power throughout the year. Otherwise, the curves are fairly flat and the ends, despite their going up and down at the 0 and 100% exceedence values, indicate limited extremes, which is typical of large catchments.

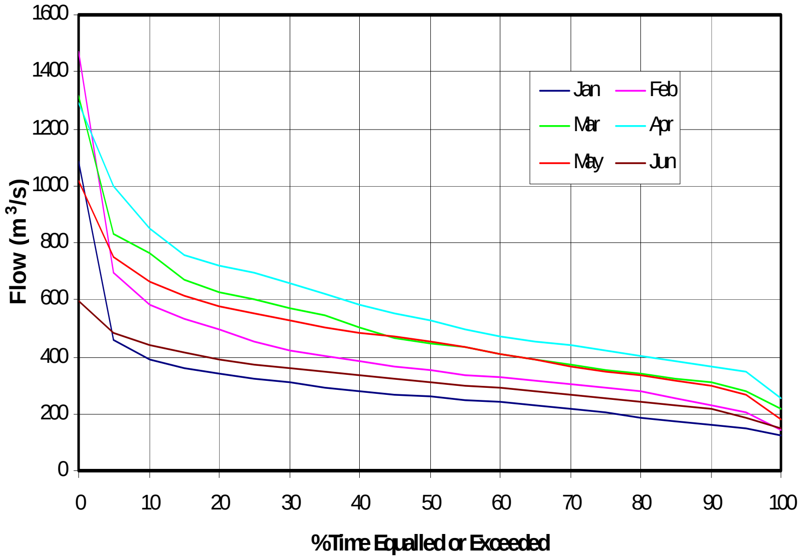
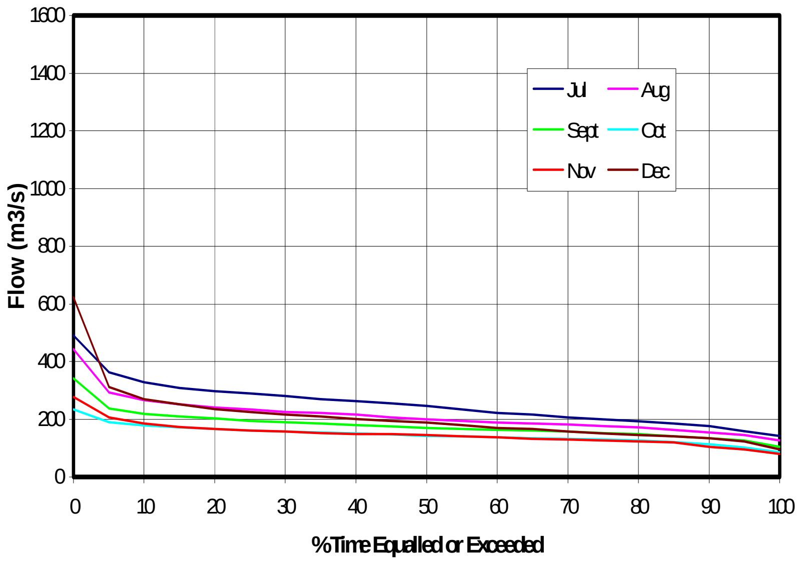


Figure 2-4 : Flow Duration Curves : January to June

Figure 2-5 : Flow Duration Curves : July to December



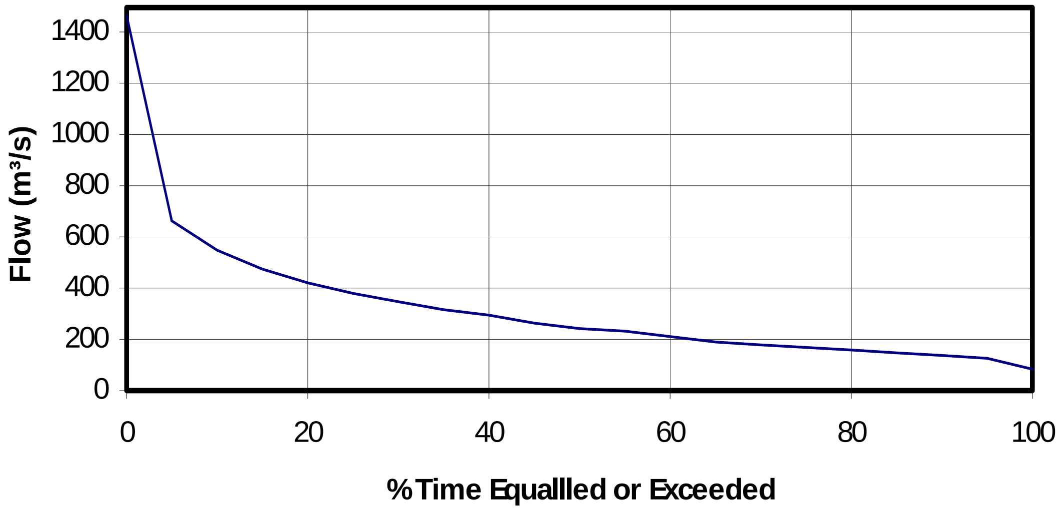


Figure 2-6 : Flow Duration Curves : Annual

2-3 FLOOD PEAKS

2.3.1 OBJECTIVES

Flood peak estimation for various risk levels (return periods) is required for the evaluation and design of the outlet works and spillway configurations, as well as for river diversion works during construction.

Flood peaks are also relevant in evaluating sluicing options, which may pose a potential problem.

2.3.2 AVAILABLE DATA

The available data and initial screening is discussed in **Section 2.2.2**. Further consistency tests were performed by plotting annual flood peaks in chronological order. This indicated a general trend after 1985 of slightly decreasing flood peaks in both the Rundu and Mukwe records, as shown in **Figures 2-7** and **2-8**. This normally indicates some form of storage development in the catchment, but it is known that no additional storage was created or provided during this period. More data will be required to establish whether the gradual decrease can be attributed to a normal long-term cycle of smaller floods or as a result of cyclic changes in the rainfall in southern Africa

The flow records, provided in the form of annual tables, also include the maximum instantaneous peak for each year. These were used provided they were not highlighted as having serious uncertainties.

The Mean Annual Runofff (**MAR**) was determined in the 1969 study to be 11 500 Mil m³. The long term flow records of the Mukwe gauging station, made available by the Department of Water affairs, show that the MAR is now reduced to 9 585 mil. m³

2.3.3 STATISTICAL ANALYSES

The instantaneous flood peaks referred to above were used in a statistical analysis. The Log Pearson Type 3 distribution was selected to estimate flood peaks for different recurrence intervals. The Log Pearson Type 3 distribution is most commonly used in hydrological analyses in southern Africa and it fits most sets of hydrological data.

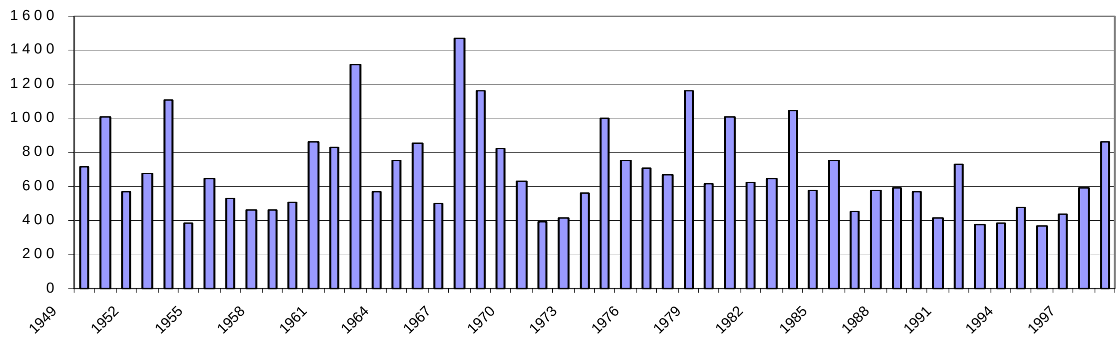
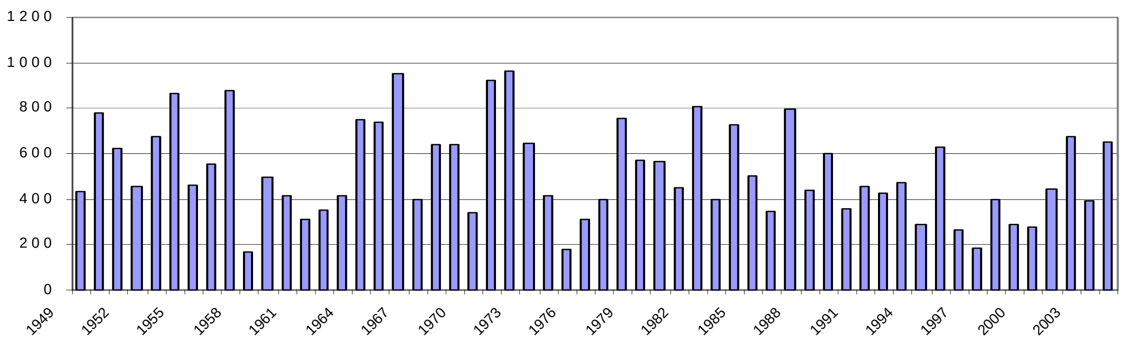


Figure 2-7 : Annual Recorded Flood Peaks at Mukwe

Figure 2-8 : Annual Record Flood Peaks at Rundu



The statistical parameters are shown in **Table 2-2**. The low value of the skew coefficient indicates a limited number of extreme values.

Table 2-2 : Log Pearson Type 3 Analysis : Statistical Parameters

Parameter	Natural	Logs
Mean	692,07	6,477
Standard Deviation	259,39	0,352
Skew Coefficient	1,057	0,340

In addition to the estimated values, upper and lower confidence limits were also calculated for a confidence range of 90%. The results are given in **Table 2-3** and are represented graphically in **Figure 2-9**. Due to problems with electronic formats, the return period is plotted on a logarithmic scale rather than one of the conventional systems such as the Gumbel system. The recorded flood peaks are plotted in **Figure 2-9** according to the general purpose Cunane plotting positions, expressed as:

$$\mathbf{T = (N + 0{,}4) / (M – 0{,}4)}$$

in which : T = the return period (yrs)
N = is the total number of observations, and
M = the rank of the recorded flood peak in descending order of magnitude.

Table 2-3 : Estimated Flood Peaks by Log Pearson Type 3

Return Period (years)	Estimate (m^3/s)	90% Confidence Limit	
		Upper Limit (m^3/s)	Lower Limit (m^3/s)
2	639	649	628
5	869	965	796
10	1031	1173	932
20	1194	1399	1064
50	1417	1708	1231
100	1593	1958	1368
200	1779	2242	1514
500	2039	2622	1698
1000	2249	2955	1844
10000(PMF)	3037	4263	2373

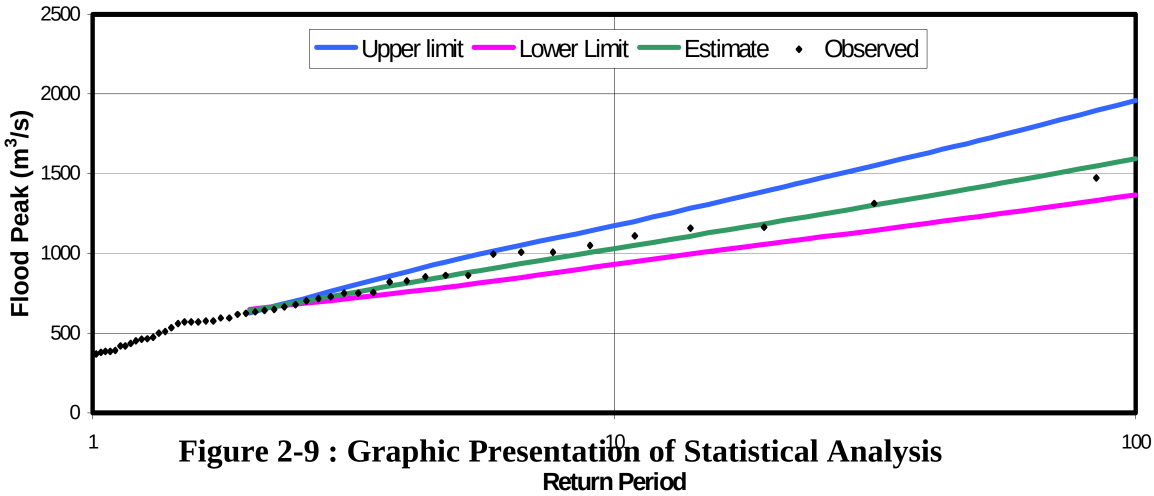


Figure 2-9 : Graphic Presentation of Statistical Analysis

2.3.4 EMPIRICAL ANALYSIS

The empirical analysis is based on the assessment of the Regional Maximum Flood (**RMF**), which is an empirically-established upper limit of flood peaks that can be reasonably expected along the Okavango River in this case. The procedure is described in Technical Report TR137 entitled Regional Flood Peaks in Southern Africa of the South African Ministry of Water Affairs.

Southern Africa is delineated in different flood peak regions designated by a regional coefficient or K-value.

The region in which the Okavango River falls is designated with a K<2.8. The RMF is calculated with the following formula:

$$\mathbf{RMF = 10^6(A/10^8)^{(1-0,1K)}}$$

In which: RMF = the regional maximum flood
A = the effective catchment area. and
K = the regional coefficient

Applying this formula to a gross catchment area of 203 000 km^2 with a K-value of 2,8 results in a regional maximum flood of 11 500 m^3/s. Ratios in terms of the RMF are used to estimate flood peaks with return periods of 50, 100 and 200 years.

The results are shown in **Table 2-4**.

Table 2-4 : Flood Peaks Based on the RMF Method

`

Return Period	Estimated Peak
1:50	8 100
1:100	8 900
1:200	9 600
RMF	11 500

Reducing the K-value to 2,0, for example, decreases the Regional Maximum Flood to 7 000 m^3/s. In addition, it is not known at this stage whether the area of 203 000 km^2 is representative of the effective catchment area. This empirical method is not considered suitable and it is recommended that the results be discarded.

2.3.5 CUNENE RIVER FLOOD PEAKS

As an additional check, it was thought wise to transpose the updated Cunene River flood peaks used for the Epupa Hydro Power Project study. However, upon enquiry it was established that the Epupa flood peaks were generated using the long term flow records of the Okavango River at Mukwe. Using the Epupa data would consequently not benefit the Popa Falls study. . It was therefore decided to transpose the estimated flood peaks for Calueque Dam to Popa Falls using the simple equation:

$$\mathbf{Q_P / Q_C = (A_P / A_C)^{0,5}}$$

In which: Q_P = the flood peak at Popa Falls
Q_C = the previously estimated flood peak at Calueque Dam
A_P = the catchment area at Popa Falls (203 000 km^2 gross), and
A_C = the catchment area at Calueque Dam (81 500 km^2 effective)

The flood peaks for Calueque Dam on the Cunene River have been extracted from Calueque Dam Developed Design Report No. 4, which deals with spillway and crest level proposals prepared by Hydroconsults in September 1970 on behalf of SWAWEK. These flood peaks were based on the consideration of limited recorded flood peaks at Gove, Matala and Ruacana, which proved to be inconclusive. Finally, estimates of extreme flood peaks on the basis of a Creager rating of 30 were used.

The transposition of these flood peaks to the Popa Falls on the Okavango River is provided in **Table 2-5** below. The reliability of the Calueque Dam flood peaks is questionable and therefore may not be suitable for transposition to the Okavango River.

Table 2-5 : Transposition of Flood Peaks from Calueque Dam

Return Period	Flood Peaks (m^3/s)	
	Calueque Dam (Cunene River)	Popa Falls (Okavango River)
1:50	3 000	4 700

1:100	3 600	5 700
1:500	5 000	7 900
Probable Maximum Flood	8 000	12 600

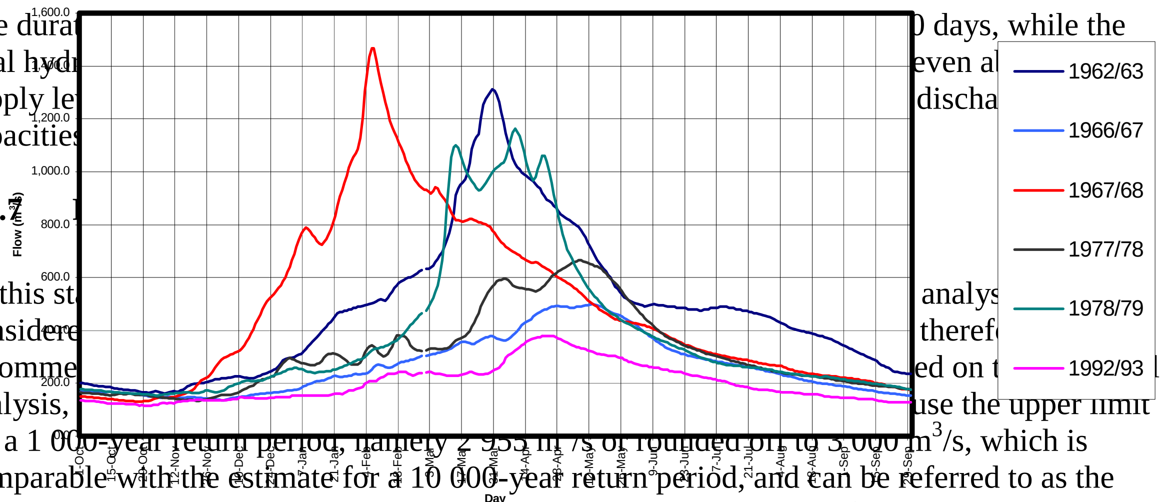
The remark regarding the effective catchment area made in the previous section is equally applicable here. It should also be noted that the bed slope of the Cunene River is much steeper than that of the Okavango, which has an effect on the direct transposition of flood peaks.

2.3.6 FLOOD HYDROGRAPHS

To obtain typical flood hydrographs, the three highest observed peaks were selected, two typically average and one which is the lowest recorded to date. These observed flood hydrographs are shown in **Figure 2-10** below and represent the recorded flood hydrographs of the years given in the legend.

Figure 2-10 : Selected Flood Hydrographs

These floods have extremely long durations and up to 20 days either side of the peak still yielded flows of some 50% to 70% of the peak flow. This shows that retardation and attenuation significantly affect the magnitude of the flood peaks.



The durat… 0 days, while the total hydr… even al… supply le… discha… capacities…

2.3.7 …

At this st… analys… considere… theref… recomme… ed on t… analysis, … use the upper limit for a 1 000-year return period, namely 2 955 m^3/s or rounded off to 3 000 m^3/s, which is comparable with the estimate for a 10 000-year return period, and can be referred to as the PMF (Probable Maximum Flood). **Table 2-6** lists the recommended flood peaks.

Table 2-6 : Recommended Flood Peaks at Popa Falls on the Okavango River

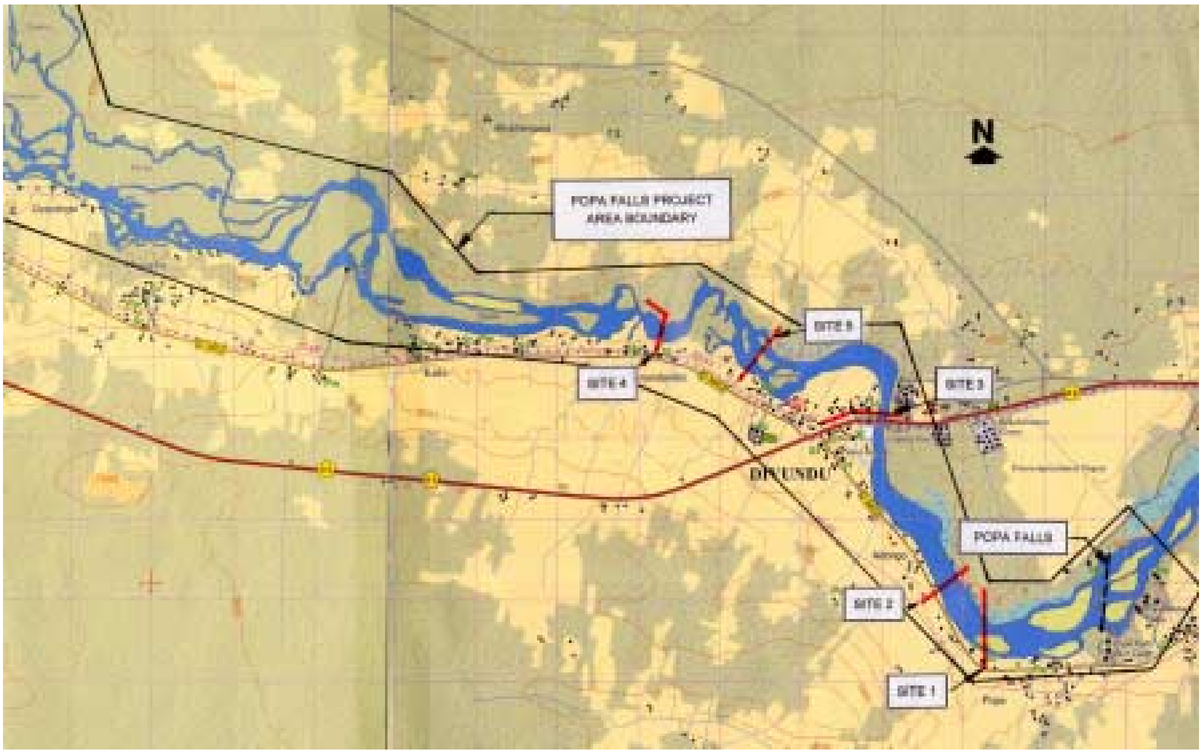
Return Period (years)	Flood Peak (m^3/s)
2	640
5	870
10	1030
20	1195
50	1420
100	1595
200	1780
500	2040
Probable Maximum Flood	3000

The calculated water surface levels at Mukwe Gauging Station show that the stage-discharge formula being used by NamWater may under-estimate the larger floods (refer to **Section 2.4.5** and **Figure B2-1** of **Appendix B** of **Volume 2**). Obviously an under-estimation of recorded flood peaks will affect the statistical analysis and it is recommended that this aspect be further investigated in follow-on studies.

2.4 WATER SURFACE PROFILES

2.4.1 OBJECTIVES

Four weir sites have been identified and are referred to as Sites 2, 3, 4 and 5. Site 2 is about 2 km upstream of the Popa Falls, Site 3 is immediately upstream of the bridge at Divundu across the Okavango River, Site 5 is some 4 km upstream of the Divundu bridge and Site 4 about 1km upstream of Site 5. **Figure 2-11** below shows the location of the different weir sites.



The storage volumes created by these weirs are relatively small, and the purpose of these weirs is therefore not to provide storage, but to provide additional height as required for hydro power generation.

The objectives of calculating water surface profiles along the Okavango River are to:

- Establish tailwater levels at the four weir sites and immediately downstream of the Popa Falls for different discharges or flood peaks. These are required to determine the positioning levels of the turbines, the discharge characteristics of bottom scour outlets and the stability of the weir structures. These tailwater levels represent the existing natural conditions, and to
- determine the upstream flooding or inundation caused by the weirs and to establish whether the weirs would affect the water surface levels in Angolan territory.

2.4.2 AVAILABLE DATA AND INFORMATION

The analyses have been based on the following data and information:

- Topographical survey with a 0,5 m contour based on an airborne laser survey carried out in mid-January 2003.
- A discharge of 215 m^3/s and a gauge plate reading of 2,8 m at Mukwe Gauging Station on the day of the airborne laser survey together with the following stage-discharge formula for the gauging station:

$$\mathbf{Q = 188{,}5077\ (H - 1{,}72)^{1{,}7222}}$$

in which: Q = the discharge (m^3/s)
H = the gauge plate reading (m)

In accordance with the formula, a gauge plate reading of 1,72 would represent zero flow. This implies that the average water depth across the river for a discharge of 215 m^3/s would be 1,08 m.

- Flood peaks estimated for this study (see Section 2.3.7).
- Discharge characteristics at Site 2, 4 and 5 for different weir heights and different flood peaks as listed in **Table 2-7** below

Table 2-7 : Headwater Levels

Flood Peak m^3/s	Site 2 Headwater levels (mamsl) for Full supply levels of			Sites 4 and 5 Headwater levels for a full supply level of 1 010,1 mamsl
	1 005,02 mamsl	1 007,52 mamsl	1 010,02 mamsl	
640	1005.50	1007.95	1010.37	1010.54
870	1005.56	1007.98	1010.39	1010.60
1030	1005.61	1008.00	1010.40	1010.64
1195	1005.66	1008.02	1010.42	1010.67
1420	1005.72	1008.06	1010.44	1010.73
1595	1005.75	1008.08	1010.45	1010.77
1780	1005.85	1008.11	1010.47	1010.81
3000	1006.26	1008.28	1010.58	1011.10

2.4.3 METHODOLOGY

Flow in an open channel is classified as **steady** if the flow depth can be assumed to be constant during the time interval under consideration. The flow is **unsteady** if the depth changes with time, which is typically associated with a flood hydrograph. The water surface profiles (or floodlines) represent the maximum water surface profile that can be reached by a given flood peak. Therefore, only considering the flood peak (or a very short time interval) the maximum water levels (or floodlines) can be computed on the basis of **steady** flow, which minimises the calculation effort. Furthermore, the flow depth changes along the length of channel, which is classified as **varied** flow, as opposed to **uniform** when the flow depth remains the same along the channel length. **Varied** flow is classified as either **rapidly** varied (where the depth changes abruptly over a short distance, e.g. over the Popa Falls) or **gradually** varied (where the depth changes gradually over a long distance, e.g. channel flow). The calculation of the maximum water surface profiles (or floodlines) has been based on **steady gradually varied** flow for the river reaches upstream and downstream of the Popa Falls and the other rapids, and upstream of the weirs.

The basic calculation procedure is based on an iterative solution of the energy equation. Given the water surface elevation at one cross-section for a given flow, the water surface elevation is calculated at the adjacent cross-section. For sub-critical flow, the calculations begin at the downstream boundary and proceed upstream, as is the case along the Okavango River. At the Popa Falls and the other rapids, the critical flow depth is calculated and used as a (new) starting water surface level to solve the energy equation in an upstream direction.

The procedure can be summarised as follows (assuming subcritical flow):

- Assume a water surface elevation at a given cross-section based on uniform flow conditions;
- Determine the area, hydraulic radius, and velocity at the given cross-section based on the cross-sectional profile;
- Calculate the associated conveyance and velocity head values;

- Calculate friction slope, friction loss, and contraction / expansion loss;
- Solve the energy equation for the water surface elevation at the adjacent cross-section.

2.4.4 COMPUTER SOFTWARE

The first step was to extract the longitudinal section and cross-sections for the Okavango River from the laser survey data. The programs described below were used to manipulate the contour and point data to obtain the longitudinal section and cross-sections at pre-defined points along the river.

ACAD 2000I was used together with Surf Mate and Pipe Mate. Surf Mate is a Digital Terrain Model (DTM) program written for the southern hemisphere making use of the "QuickSurf" engine, an American DTM program. The contour and point data are extracted and a 3-D terrain model is generated either by a Triangulated Irregular Network (TIN) or a GRID. The sections are then projected onto the model to extract the elevations at all the section/triangle crossing points.

The data, distance and elevation have been used to extract the cross-sections at locations shown in **Figure B2-6** (Sheets 1 -4) of **Appendix B** in **Volume 2**.

Pipe Mate is a design program used for gravity pipes but can also be used to draw a longitudinal section. It should be noted that the longitudinal bed profile shown in **Figure B2-7** of **Appendix B** in **Volume 2 – Appendices and Annexures**, represents the average river bed levels at the selected cross-sections.

Water surface level computations were performed by means of the HEC-RAS River Analysis System (Version 3.0.1, March 2001 developed by the US Army Corps of Engineers).

2.4.5 CALIBRATION OF THE HEC-RAS MODEL

The flow of 215 m^3/s with a depth varying from 1 m to 2 m along the Okavango River on the day of the laser survey limits the profile details of the river channel to the water surface levels surveyed. However, the longitudinal profile of the water surface levels provided an indication of the actual hydraulic gradients for a flow of 215 m^3/s. Using these hydraulic gradients and the surveyed flow width at each cross-section it was possible to calibrate the model through an iterative process by varying the average water depth at each cross-section until the water levels as surveyed were achieved. The cross-sections as extracted from the laser survey were then adapted to represent these average water depths in the river channel at each section. This calibration process was in fact a means to determine a longitudinal bed profile for the Okavango River. The results are shown in **Figure B2-7** of **Appendix B** in **Volume 2**.

The stage-discharge formula provided for the Mukwe Gauging Station was also used as a further verification of the calculated water surface levels for different flows at Mukwe. **Figure B2-1** of **Appendix B** in **Volume 2** shows a comparison of the water surface elevations at Mukwe for different flows based on the stage-discharge formula and as calculated with the HEC-RAS Model. In accordance with the stage-discharge formula for Mukwe Gauging Station, an average flow depth of 1,08 m would be applicable to a flow of 215 m^3/s. With reference to **Figure B2-1** of **Appendix B** in **Volume 2**, the difference in average water depth between the formula and the HEC-RAS modelling increases gradually to 0,16 m for a flow of 1 030 m^3/s (10 years), 0,29 m for a flow of 1 420 m^3/s (50 years) and as high as 0,8 m for the maximum flood of 3 000 m^3/s. The formula may be under-estimating flood peaks, which may affect the recorded flood peaks used for the statistical analysis. This discrepancy can only be cleared by detailed surveys below the water surface along the river channel, which must obviously follow as part of the feasibility study and detailed design.

Friction losses have been calculated on the basis of the following values for Manning's roughness coefficient:

Single river channel	:	0,025
Braided river channels	:	0,030
River banks	:	0,045

2.4.6 RESULTS

The calculated tailwater levels (natural flow levels) at Mukwe and immediately downstream of Popa Falls, and at Sites 2, 4 and 5 are listed in **Table 2-8** and graphically presented in **Figures B2-1**, to **B2-5** of **Appendix B** in **Volume 2**

Table A2-1 of **Appendix A** in **Volume 2** lists the water surface levels for different flows or floods under natural conditions along the entire reach of the Okavango River under consideration. The locations (chainages) of the cross-sections are shown in **Figure 2-6** of **Appendix B** in **Volume 2**

The headwater levels upstream of the different weir sites are listed in the following tables in **Appendix A**. in **Volume 2**:

Table A2-2	:	Water surface levels for weir at Site 2 - FSL 1 005,00 m.a.m.s.l.
Table A2-3	:	Water surface levels for weir at Site 2 - FSL 1 007,50 m.a.m.s.l.
Table A2-4	:	Water surface levels for weir at Site 2 - FSL 1 010,00 m.a.m.s.l.
Table A2-5	:	Water surface levels for weir at Site 4 - FSL 1 010,00 m.a.m.s.l.
Table A2-6	:	Water surface levels for weir at Site 5 - FSL 1 010,00 m.a.m.s.l.

The blank spaces in these tables represent the cross-sections where the headwater levels coincide with the water surface levels under natural conditions listed in **Table A2-1** of **Appendix A**. in **Volume 2**.

It is understood that the sizing of the spillway gates should be such that an average velocity of 1,5 m/s can be achieved through the weir reservoir under the 5-year flood condition (870 m^3/s peak).

Table A2-7 of **Appendix A** in **Volume 2** lists all the hydraulic details including flow velocities for the 5-year flood under natural conditions. It is to be noted that the above velocity criterion is not met at certain cross-sections under natural flow conditions.

The hydraulic details including flow velocities upstream of the different weir sites for the 5-year flood condition are listed in the following tables in **Appendix A** in **Volume 2**:

Table A2-8 : 5-year flood (870 m^3/s) flow data for Weir at Site 2 with a FSL at 1 005,00 m.a.m.s.l.

Table A2-9 : 5-year flood (870 m^3/s) flow data for Weir at Site 2 with a FSL at 1 007,50 m.a.m.s.l.

Table A2-10 : 5-year flood (870 m^3/s) flow data for Weir at Site 2 with a FSL at 1 010,00 m.a.m.s.l.

Table A2-11 : 5-year flood (870 m^3/s) flow data for Weir at Site 4 with a FSL at 1 010,00 m.a.m.s.l.

Table A2-12 : 5-year flood (870 m^3/s) flow data for Weir at Site 5 with a FSL at 1 010,00 m.a.m.s.l.

The water surface profiles along the Okavango River, under normal conditions and with the weirs in place, are graphically shown in **Figure B2-8** for the 5-year flood, **Figure B2-9** for the 20-year flood, **Figure B2-10** for the 100-year flood and **Figure B2-11** for the PMF, all of **Appendix B** of **Volume 2.**

The backwaters or inundation created by the weirs will not extend across the Namibia/Angola border upstream of Mukwe Gauging Station.

Table 2-8 : Tailwater Levels (Existing Natural Conditions)

Return Period (years)	Flood Discharge (m^3/s)	Tailwater Levels (mamsl) at					Water levels at Mukwe (mamsl)	
		Downstream of Popa Falls	Site 2 Weir	Site 3 Weir	Site 4 Weir	Site 5 Weir	Calculated	Theoretical (Rating Curve)
Average river bed level	0	995.66	1000.02	1000.24	1001.09	1 000,46	1015.37	1015.37
Low Flow	80	996.34	1000.78	1001.22	1001.91	1 001,51	1015.95	1015.98
Low Flow	150	996.69	1001.07	1001.59	1002.32	1 001,94	1016.23	1016.25
Flow on 15/01/03	215	996.96	1001.28	1001.86	1002.62	1 002,25	1016.44	1016.45
1:2 years	640	998.28	1002.17	1002.98	1003.96	1 003,56	1017.35	1017.4
1:5 years	870	998.74	1002.49	1003.4	1004.46	1 004,05	1017.68	1017.8
1:10 years	1030	999.02	1002.68	1003.65	1004.75	1 004,35	1017.89	1018.05
1:20 years	1195	999.27	1002.86	1003.88	1005.02	1 004,64	1018.08	1018.29
1:50 years	1420	999.59	1003.08	1004.16	1005.35	1 005,00	1018.31	1018.6

1:100 years	1595	999.81	1003.24	1004.37	1005.6	1 005,26	1018.49	1018.83
1:200 years	1780	1000.02	1003.39	1004.58	1005.84	1 005,52	1018.65	1019.05
Max. Flood	3000	1001.15	1004.19	1005.79	1007.14	1 006,94	1019.56	1020.36

Water surface profiles for the probable maximum flood with the weir in place are given in **Figure B2-12** of **Appendix B** of **Volume 2** in the case of Site 2 and **Figure B2-13** of **Appendix B** of **Volume 2** in the case of Sites 4 and 5.