Technical Analysis Of Hydrologic Issues For Dadin Kowa Dam Safety Evaluation

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Abstract: A technical analysis of hydrologic issues of the safety evaluation of existing dam (SEED) was performed on Dadin Kowa dam, involving review of initial design basis, flood frequency analysis of new data and flood routing studies with approved probable maximum flood (PMF) through the spillway and outlet works under large discharges. A comparison of the design and new PMF based on data from the gauging station and reservoir inflow data showed the latter to be 2.4 times the former at 2841 and 6879 m^3/s respectively. This obviated a review of the hydrologic bases of the design of the spillway and routing of the new PMF through the reservoir. It was found that the reservoir has the capacity to attenuate the new PMF to achieve a peak outflow of 1556 m^3/s compared to the design spillway discharge of 1100 m^3/s. Operating the spillway, penstock and irrigation release facility simultaneously during the new PMF has no advantage in reducing the peak outflow discharge. A laboratory model of the spillway and chute channel was earlier tested for a maximum outflow of 2000 m^3/s and found to be capable. It was therefore concluded that the safety of Dadin Kowa dam with regard to hydrologic issues is satisfactory.

Index Terms: Dadin Kowa dam, hydrologic issues, spillway, safety evaluation, flood routing, attenuate, probable maximum flood, flood frequency analysis, reservoir, penstock, inflow, outflow.

1. INTRODUCTION

THE first hydrologic studies for Dadin Kowa dam were performed in 1974 and 1976 based on 19 years of streamflow record between 1955 and 1976. These studies were based on annual maximal series of flood peaks. A peer review found the studies to have neglected the cumulative effects of a number of closely spaced flood peaks, which is a frequent occurrence in the project catchment area. Sequel to this observation, further review was performed. The hydrographs developed in the work were based on the extreme frequency analyses of recorded flow volumes as well as the flood peaks. In addition, the hydrograph time base was increased from 12 to 28 days. The maximum reservoir levels were determined by routing of the inflow design flood hydrograph through the reservoir. The predicted maximum water level of 250.8 m was obtained, which was higher than the previously predicted level of 249.0 m. The 1.8 m difference was found to be significant. The freeboard was also reviewed. An initial estimate of the freeboard under the maximum flood conditions was reduced from 3.0 m to 1.70 m. In the intervening years after completion of the dam, more data were acquired during the daily monitoring of the operation of the dam. This called for reassessment of the hydrologic issues for SEED for the dam. The original flood study was prepared in 1974. It was noted at the time that there were insufficient meteorological data to allow the derivation of the probable maximum flood (PMF). Given the required meteorological data, it was concluded that the diverse hydrological conditions of the catchment will make such an effort herculean tasks, with attendant assumptions and simplifications that may introduce unwarranted errors in the derived PMF.

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It was thus decided to estimate the design floods based on the statistical analysis of recorded flows in the Gongola River at Dadin Kowa. Gumbel extreme value frequency analysis was performed on the annual peak daily flows to estimate flood peaks for various return periods. The shape of the inflow design hydrograph was derived by analysing the largest recorded flood events. This was in 1956. This flood event was separated into base flow and surface runoff components. The major storm was then separated out of the complex surface runoff hydrographs. This event was scaled up to the value of the peak daily discharge estimated from the extreme value analysis. Other storm components of the hydrograph were not modified. This method of analysis was based on the assumption that the extreme flood event would be the result of a single large storm event which would produce a large surface runoff event superimposed on the baseflow and other lesser surface runoff events [1]. During the design stage in 1976, the results of the hydrologic studies were reviewed and used as the basis for determining the spillway capacity, maximum reservoir level, dam crest elevation, cofferdam elevation, and diversion conduit dimensions. During construction, the Contractor, Stirling Civil Engineering Ltd, (SCE) reviewed the initial hydrologic data and studies in order to evaluate the adequacy of the proposed facilities. It was found that the Consultant - ShawMont Nigeria Ltd (SMN), ignored the sequential nature of the flood events in the project area, which was found in the historical flood observations. These sequential events indicated that the extreme flood events may occur as the result of a series of closely – spaced relatively minor storm events. On the basis of this observations, SMN performed additional studies on the volumetric frequency in the observed annual hydrographs, to derive new design flood hydrographs, which were compared to the works of SCE. The main differences related to the shape of the hydrograph, rather than the volume of flow under the hydrograph or the peak discharge. It was generally agreed that the solution was sensitive to the assumed hydrograph shape. The final analysis was based on the historical records of average daily streamflows for the period of 1955 – 1976. Review of the stream flow data after 1976 found them to be of very poor quality, containing data gaps, which made them unsuitable. It was found from the inspection of the part of the annual hydrograph representing the extreme volume for the larger duration that the date of the annual peak followed no consistent pattern. The timing of the peak varied widely but on the average occurred approximately half way through the extreme volume event. This observation was the basis for the choice of a symmetrical bell shape design hydrograph. Analyses of the computed results indicated that the Gumbel extreme value distribution was superior to the alternatives of lognormal and log Pearson distributions in selecting the appropriate volume – duration relationship for the design flood. However, it was noted that the other distributions took better account of the skewness observed in the historical data. Thus the Gumbel extreme distribution was adopted as the basis for the design of Dadin Kowa Dam.

2 MATERIALS AND METHODS

2.1 Project design flood routing

The project inflow design flood was routed through the reservoir and discharged through the spillways to determine the maximum reservoir level. It was assumed that the reservoir will be filled to the full supply level at the start of the flood, and that there will be no discharges through the power house or irrigation release facilities, since these may be out of service under severe flood event. The routing based on the best estimate of the project design flood resulted in a maximum reservoir elevation of 250.0 m, while based on the Gumbel extreme value distribution, the maximum level was 250.8 m. The results are shown in Table 1 for four cases: (1) Gumbel extreme value distribution as base case (2) Best estimate distribution (3) Gumbel distribution, skewed left (4) Gumbel distribution, skewed right.

2.2 Freeboard

The original designs provided 5.0 m - freeboard at full supply level 247.0 m and 3.0 m at maximum flood level of 249.0 m. This was reviewed downward after the final hydrologic studies. The fetch for wave generation was determined using the US Corps of Engineer method and found to be 8 km and 56 km for the wind setup. The freeboard at full supply level and maximum flood level for a sustained wind speed of 10 km/hr during the wet season were found respectively to be 4.0 m above 247.0 m and 1.7 m above the maximum flood elevation. On the basis of these analyses, the dam crest elevation was selected as 252.5 m

2.3 Analysis of recent data

The annual maximal series were obtained for the daily flows for the period of available record. The peak flows for different return periods were obtained by graphical and statistical methods. In view of the findings during the design and construction of the dam, Gumbel extreme value Type I and Pearson Type III were used. This is to permit comparison of the result of the current work with the earlier hydrologic studies of SMN and SCE.
Most frequency functions can be generalized to the form [2]:

$$X = \bar{X} + K\sigma_x \quad (1)$$

where X is the flood of specified probability, $\bar{X}$ and σ_x are the mean and standard deviation of the flood series, K is the frequency factor defined by a specific distribution, as function of the probability level of X. The probability model for Gumbel is given in (2):

TABLE 1: *SUMMARY OF DESIGN FLOOD ROUTING ANALYSIS*

Probability Model	Peak inflows (m^3/s)	Peak outflow (m^3/s)	Max. Elevation (m)
Gumbel, Base case	3170	1557	250.8
Gumbel, skewed left	3170	1507	250.6
Gumbel, skewed right	3170	1611	251.1
Best Estimate	2925	1404	250.0
HRU, UniLag	3050	1515	250.6

$$X = \bar{X} + (0.7797y - 0.45)\sigma_x \quad (2)$$

$\bar{X}$ is the mean of the data series, σ_x is the standard deviation. The frequency factor, K, is

$$K = 0.7797y - 0.45 \quad (3)$$

The variate was determined using (4)

$$y = -In(-In(1-p)) \quad (4)$$

where p is the probability of exceedance for any return period T, given as:

$$p = \frac{1}{T} \quad (5)$$

Daily records of reservoir operation of Dadin Kowa dam were collected from Upper Benue River Basin Development Authority, Yola along with general dam and reservoir data.

2.4 Reservoir routing

Sizing of the spillway requires the peak outflow discharge arising from the passage of the inflow design flood through the reservoir. Usually the inflow peak discharge is attenuated; while the time to peak is lengthened in the outflow hydrograph. The computation requires detailed information on the storage capacity of the reservoir site at different levels. The problem is formulated by this equation:

$$I = O + \frac{dS}{dt} \quad (6)$$

I is the inflow to the reservoir, O is the outflow from the reservoir and S is the storage in the reservoir. For the time interval, Δt, the equation is approximated by (7):

$$\frac{I_1+I_2}{2}\Delta t - \frac{O_1+O_2}{2}\Delta t = S_2 - S_1 \quad (7)$$

where subscripts 1 and 2 denote values at beginning and end respectively of the time Δt. The routing time Δt should be shorter than the time of travel of the flood wave crest through the reservoir [3]. For routing of the inflow design hydrograph through the reservoir, the equation is rearranged such that all known terms are on one side as in (8):

$$\frac{1}{2}(I_2 + I_2)\Delta t + \left(S_1 - \frac{1}{2}O_1\Delta t\right) = \left(S_2 + \frac{1}{2}O_2\Delta t\right) \quad (8)$$

The routing process involves substituting the known values to obtain $S_2 + 0.5O_2\Delta t$ and using the known relationship between the storage and discharge to determine the outflow discharge. The equation can be applied to reservoir receiving inflow at one end and discharging at the other end through spillway and other outlet works.

3 RESULTS AND DISCUSSION

3.1 Flood frequency analysis

There were 19 years of annual maximal series of stream flows that formed the basis for the hydrologic studies in 1976. These were obtained from the record of Dadin Kowa gauging station on River Gongola between 1955 and 1974. Effort to improve accuracy by adding the peak floods between 1975 – 1981 at the time of construction and final review of the hydrologic studies was impossible because of the poor quality of data in the intervening years. We also have 24 years of annual maximum series of maximum daily discharges, between 1988 and 2011, which were combined with earlier records of stream flows between 1955 and 1972 for the flood frequency analysis. These were analysed using Gumbel extreme value type I distribution to obtain the peak discharges for various return periods of 2, 50, 100, 500, 1000, 5000 and 10000 years flood events. The peak discharges were superimposed on the dimensionless unit hydrograph of the United States Soil Conservation Service (USCS) [4] to arrive at the design flood hydrographs for the dam and reservoir. To account for the observed effects of the minor peaks and baseflow, the IDF was increased in line with the earlier works of SMN (1982) [5]. It was noted that earlier works of SMN and SCE were based on smaller data size of 19 years and formed the basis for the design and construction of the dam. For purposes of comparison with the previous works, we adopted this same method for flood frequency analysis and reservoir routing for two cases.

- Case I: Annual maximum series of data between 1987 and 2011, with sample size of 25 years
- Case II: Annual maximum series of available data between 1955 and 2011, with sample size of 44 years.

A summary of the selected events for 1955 – 1973 and 1987 – 2011 is given in Table 2.

***TABLE 2:** ANNUAL PEAK DAILY FLOWS AT DADIN KOWA GAUGING STATION, 1955 – 1973 AND DAM, 1987 - 2011*

Year	Date	Peak Daily Flow (m^3/s)	Year	Date	Peak Daily Flow (m^3/s)
1955	28-Sep	671	1990	16-June	463
1956	31-Aug	1437	1991	15-Aug	810
1957	22-Aug	836	1992	28-June	926
1958	23-Aug	1096	1993	13-July	1389
1959	28-Aug	1003	1994	14-Aug	926
1960	02-Sep	736	1995	06-July	579
1961	29-Aug	1077	1996	13-Aug	926
1962	25-Aug	1267	1997	08-Aug	810
1963	01-Sep	690	1998	17-Aug	1505
1964	24-Aug	1365	1999	05-May	1620
1965	05-Sep	1046	2000	09-Aug	2315
1966	10-Sep	542	2001	09-Aug	2315
1967	06-Sep	1213	2002	20-Sept	926
1968	07-Jul	651	2003	27-Jun	2342
1969	13-Sep	701	2004	23-Aug	926
1970	03-Sep	1171	2005	18-Aug	2894
1971	31-Aug	1175	2006	15-Aug	1389
1972	02-Sep	550	2007	15-Aug	2778
1973	21-Aug	964	2008	28-Aug	1792
1987	03-Sept	463	2009	05-Aug	2199
1988	11-Aug.	694	2010	13-Sept	3472
1989	30-Aug.	810	2011	30-Aug.	2549

The highest flow occurred in 1956 for the earlier record with a maximum daily stream flow of 1437 m^3/s. However highest flow after impoundment was 3472 m^3/s and was 2.41 times higher than the maximum recorded flow between 1955 and 1973. Indeed, several of the peak flows in the recent data series were higher than the highest value in the earlier record. This new record obviated the review of the hydrological bases for the sizing of the spillways and selection of the reservoir levels. The statistics of the peak flows are given in Table 3 for the three cases considered. The mean values for 1955 – 73, 1987 – 2011 and the combined data from 1955 – 2011 are respectively 957, 1513 and 1273 m^3/s. This is not unexpected because of the different periods in which the data were collected. In the earlier periods, the records were based on the rating curves on the Dadin Kowa gauging station, which changed seasonally depending on the morphology of the river bed. This was likely to cause errors in measurement. In the more recent period between 1988 and 2011 after impoundment of the reservoir, a more accurate data measurement was expected. This may partly explain the significant differences in the statistics of the data from the two epochs. The peak flows vary widely with a coefficient of variation of 29 % and 57 % for 1955 – 1973 and 1988 – 2011 respectively, with the later years showing more variability than the earlier data. Indeed, the value of the skewness coefficient of 0.0265 for the 1955 – 1972 data seems to be an aberration, rather than the rule. This is premised on the fact that peak streamflows are known to be positively skewed rather than being symmetrical. The recent record showed the data have a skewness coefficient of 0.67 and 1.33 for the combined data from 1955 – 2011. Thus the shape of the probability distribution is positively skewed to the right, as is common for peak streamflow data. It is therefore justified to fit Gumbel extreme value type I or Pearson Type 3 to the data. This punctured the assumption of a bell – shaped distribution that was the basis of the design flood hydrograph used by (SMN, 1976) for the flood routing. It may however be noted that the maximum reservoir level has earlier been found to be insensitive to the location of the peak of the inflow design hydrograph [5]. The predicted peak flows at different return periods are given in Table 4. Earlier record between 1955 and 1972 gave the peak discharge for the 10,000 – year flood event as 2841 m^3/s compared with a value of 7332 m^3/s for the recent data between 1988 and 2011. The latter is three times higher than the former. Clear differences exist in the other return periods. This is a strong premise for the review of the hydrological basis for the choice of spillway capacity and embankment crest levels.

3.2 Inflow design hydrograph
In the absence of detailed hourly records of inflow records at the spillway, determination of the shape of the inflow design flood becomes impossible using the unit hydrograph method. SMN (1976) [5] got around this problem by using the shape of the peak flood in 1956.

TABLE 3: *THE STATISTICS OF THE PEAK FLOWS*

Record	Case III: 1955 - 1973, 1987 – 2011	Case II: 1987 – 2011	Case I: 1955 - 1972
Mean	1,273	1,513	957
Std Dev	726	864	280
Skew	1.326	0.667	0.0265
CV	0.5704	0.5715	0.2922

The inflow design flood was then determined by increasing the ordinates in line with values obtained for the flood frequency analyses using the Gumbel extreme value type I distribution. To account for the complex nature of the flood peaks arising from several storms being superimposed upon one another, as was the case at Dadin Kowa, a secondary runoff was determined and added to the ordinates. The process was completed by determining and adding the baseflow, which was assumed to be constant for all the different storm events for different return periods. It was ensured in the process of determining the inflow design flood that the peak daily total runoff equals the corresponding peak daily flow obtained from the flood frequency analysis. The estimated flood hydrograph is shown in Fig 1 and 2 for 1955 – 1972 and 1955 – 2011 respectively.

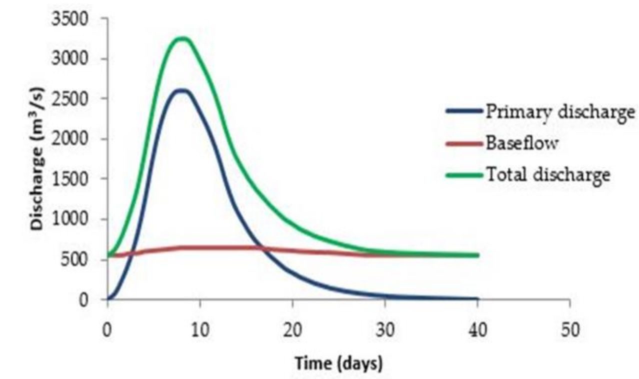

Fig. 1. *Inflow design flood showing excess runoff, baseflow and inflow design hydrograph for 10,000 – year flood*

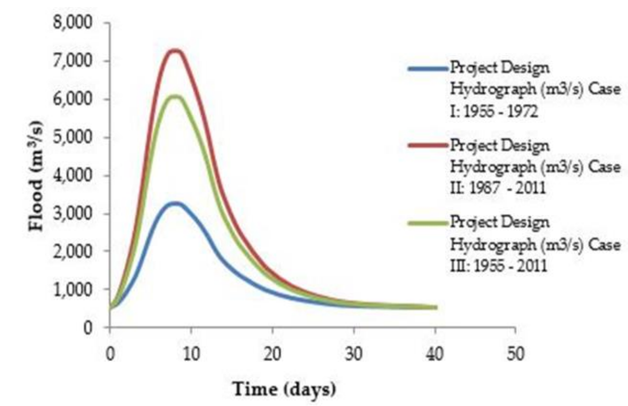

Fig. 2. *New inflow design flood for different cases for 10,000 – year event*

3.3 Reservoir routing
The routing of the inflow design hydrographs was performed using the following data:

- Reservoir elevation versus reservoir surface area
- Reservoir elevation versus reservoir capacity curve
- Reservoir elevation versus outlet work discharge relationship
- Reservoir elevation versus spillway discharge relationship
- Reservoir elevation versus outflow discharges for spillway and outlet works

Reservoir elevation versus S + 0.5OΔt and S – 0.5OΔt. For preparing these data, a storage unit was used. Each storage unit = routing period x 1 m^3/s. For the 19.2 hr interval, each storage unit = 19.2 x 3600 x 1 = 69.12 x 10^3 m^3 = 0.800 m^3/s day. It was assumed that the reservoir is at full supply level of 239 m asl at the commencement of inflow design flood. The routing considered the following scenarios:

- Outflows from the 3 No spillway gates only
- Outflows from the spillway and the outlet works (penstocks and irrigation release facility).

The summary of results of the reservoir routing are presented in Fig. 3 and Fig. 4 .

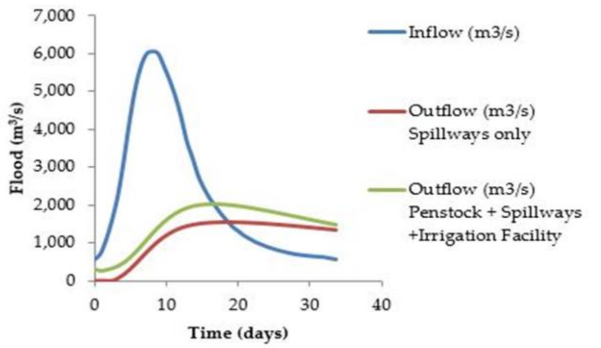

Fig. 3. *Inflow and outflow design flood for 10,000 – year flood event*

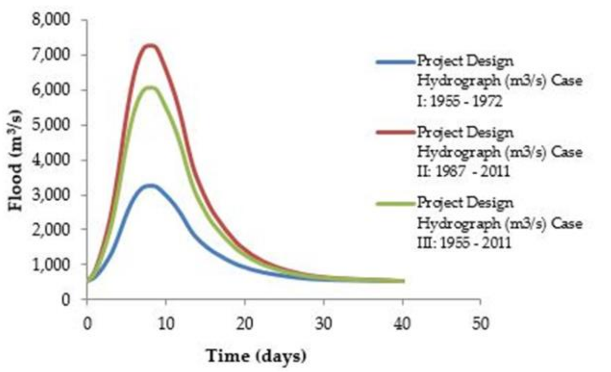

Fig. 4. *Reservoir surface level levels for 10,000 – year flood event for 0.8-day unit duration*

The results showed that the most critical condition occurs for a flood unit duration of 19.2 hours or 0.80 day. This obtains for an inflow design hydrograph with a base with of 40 days, the results showed that we attain a maximum reservoir elevation of 250.81m, having a minimum freeboard of 1.19m when the spillway is used. There was no improvement in the freeboard when all the outlet works were used for routing the inflow design flood. The spillway has a design discharge of 1100 m3/s. However, the new outflow discharge based on the routing of the new IDF was 2022 m^3/s, which was almost double the spillway design discharge. This excessive outflow condition was tested in a laboratory model of the spillway, chute channel and plunge pool [7]. It was found that the spillway and chute channel can cope with such a scenario in a laboratory model [7], [8]. It is however noted that the operation of the spillway and the outlet works simultaneously during the PMF has no advantage in that the combined operation reduced the time to peak with attendant increase in the peak outflow discharge. The use of only the spillway has better attenuating effect on the IDF, which generated a lower peak outflow discharge of 1556 m^3/s with a longer time to peak. It can be concluded that the reservoir has high capacity to route exceptional flood events.

4 Conclusion

Economic design of the spillway is based on the 100 – year flood event. Where special protection against failure is a matter of government policy, as it is in this case of Dadin Kowa dam, the spillway design was based on the 10,000 – year flood event or the probable maximum flood with minimum freeboard of 1.19 m. This is less than the prescribed value of 1.50 m. But it must be noted that the routed flood is exceptional being more than double the annual runoff volume. At other routing periods, the minimum freeboards are far higher than the recommended minimum values. It can be concluded from the routing studies that the dam is safe against failure by overtopping.

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