Flood Routing in Resevoir using Modified Puls Method

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Abstract: ***In the design of hydraulic structures it is not practical from economic considerations to provide for the safety of the structure and the system for the maximum-probable flood in the catchments. Small structures such as culverts and storm drainages can be designed for less severe floods because the consequences of a higher than-design flood may not be serious. On the other hand, storage structures such as dams demand greater attention to the magnitude of floods used in design. The failure of these structures causes a huge loss of life and property damage on the downstream of the structure. From this it is apparent that the type, importance of the structure and economic development of the surrounding area dictate the design criteria for choosing the flood magnitude. This project describes the methodology for estimating the design flood and routing the flood for overflow gravity structure for Kol Dam site in Himachal Pradesh and Kakkadavu dam site in Kerala.***

Keywords: Maximum-probable floods, design flood, flood Routing, Koldam. Kakkadavu dam

1. Introduction

Kol Dam

The Kol Dam is a 163 metre high rock fill dam on the Sutlej river near Kol village, 6 km upstream of Dehar Power house of BSN Project. A surface power house is located at the toe of dam with installed capacity of 800 MW (4 x 200MW) utilizing the head of 126m .Besides giving direct benefits, the project will serve as a support dam for Bhakra and likely to increase the life of the reservoir by at least 18 years.

Kakkadavu dam

The irrigation Department, Government of Kerala had proposed to construct a dam of 56 m height at Kakkadavu across Kariangode River to provide irrigation to 12196 Hectare lying b/w Kapum and Neeleswer River and 1770 hectare in Hosdrug Taluk lying on the Neeleswer basin. The dam will also provide water supply to Chennai and adjoining villages in Kariangode, Kannur district and also to naval academy, Government of India.

But due to steep resistance from project affected people the dam height was reduced to 35m above the bed level with crest gate installed. The storage created by the dam is sufficient to cater the demand for water supply to Chennai and adjoining Panchayats and Naval Academy. The requirement of water for water supply is estimated by KWA as 260 MLD for 2050 AD, out of which 20 MLD water is estimated requirement Naval Academy. Investigation, planning and designing proposed dam has been entrusted to WAPCOS. The present report covers hydrology of the Kakkadavu Dam Project in respect of water availability, flood estimation and sedimentation and reservoir based on the data collected by KWA.

2. Study Area

(a) Salient features of kakkadavu dam site

Project	Kakkadavu Dam Project, Kerala
Client	Kerala Water Authority
River	Kariangode
Location	Kakkadavu in kasaragod district (28 km from payannur)
Purpose	Water supply to cheemeni and adjoining pachayants & water supply to naval academy
Water demands	260 mld projected for 2050 a.d. (20 mld for cheemeni
mwl of dam	28 m (i phase) with scope for raising in ii phase 35 m
length of dam	100 m (approx.)
Type of dam	concrete gravity dam
Catchment area	272.5 km^2
Average bed level at	1.44 m above msl

Design flood

Hydrometeorlogical	by two bells system (as 3147.42 cumec as cwc approach recommendation)	
Flood frequency app.	by gumbel method	
	against t_r 1000 years	2590 cumec
	by log pearson type iii	
	against t_r 1000 years	3833 cumec

(b) Salient features of kol dam site

Project	Kol Dam, Himachal Pradesh
Client	NTPC
River	Satluj River
Location	Kol Dam on the Satluj River near Kol village, 6 km upstream Dehar power house
Purpose	to serve as a check dam for Bhakra dam and increase the life of the reservoir by at least 18 years, review of hydrology, power potential studies and the cost estimation
Installed capacity	800 mw (4 x 200 mw)
Height of dam	163 m
Type of dam	rock-fill dam
Catchment area	36900 km^2 (in Tibet) 16870 km^2 (in India)
Flood frequency app.	(a) by gumbel method against t_r 10000 yrs. 6117 cumec (b) by log pearson type III against t_r 10000 yrs 6832 cumec

3. Design for Flood

In the design of hydraulic structures it is not practical from economic considerations to provide for the safety of the structure and the system at the maximum-possible flood in the catchments. Small structures such as culverts and storm drainage can be designed for less severe floods as the consequences of a higher-than design flood may not be very serious for such structures. On the other hand, storage structures such as dams demand greater attention to the magnitude of floods used in the design. The failure of these structures causes large loss of life and great property damage on the downstream of the structure. Therefore, it is clear that the type, importance of the structure and economic development of the surrounding area dictate the design criteria for choosing the flood magnitude. This section highlights the procedures adopted in selecting the flood magnitude of the design of some hydraulic structures.
The following definitions are given as follows:-

3.1 Design Flood

Design flood is defined as the instantaneous peak discharge adopted for the design of a river headwork or control structure after accounting for the economic and hydrological factors. It is a flood that the project can sustain without any substantial damage, either to the objects which it protects or its own structures.

3.2 Spillway Design Flood

The design flood used for the specific purpose of designing the spillway of a storage structure is called spillway design flood. This term is frequently used to denote the maximum discharge that can be passed over a spillway without any damage or serious threat to the stability of the structure.

3.3 Standard Project Flood

The standard project flood (SPF) is the flood that would result from a severe combination of meteorological and hydrological factors reasonably applicable to the region. Extremely rare combinations of factors are excluded in computing the SPF.

3.4 Probable Maximum Flood

That probable maximum flood (PMF) is the extreme flood that is physically possible in a region as a result of most severe combinations including rare combinations of meteorological and hydrological factors. The PMF is used in situations where a failure of the structure would result in loss of life and catastrophic damage and as such complete security from potential floods is sought. On the other hand, SPF is often used where the failure of a structure is likely to cause less severe damages. Typically, the SPF is about 40-60 % of the PMF for the same drainage basins.

The criteria used for selecting the design flood for various hydraulic structures vary from one country to another.

4. Methods of Estimation of Design Flood

4.1 Hydrometer logical approach

This approach is practiced by all engineers in the estimation of design flood of a specific return period for fixing the water way vis a vis the design highest flood level (HFL) and foundation depth of bridge, culverts and cross drainage structures depending on their life and importance to ensure safety as well as economy.

This is accomplished by estimation of design storm along with time distribution over the project catchments and derivation of unit hydrograph. The design storm are worked out by frequency concept as well as standard project storm(SPS) or probable maximum storm concept involving detail analysis of historic storm events in and around the project catchment with due consideration to the storm transposition and moisture maximization.

The following physiographic parameters are involved to determine the unit hydrograph.

Physiographic parameters
1) Location of catchment area to be identified form survey of India topo sheet and measure the catchment areas (A).
2) Measure the length of the longest stream in km(L)
3) Length of the stream from a point opposite to C.G. of catchment to point of study in km (Ls).
4) Compute equivalent slope in m/km (Seq).

4.2 Flood Frequency Approach

Hydrologic process such, as floods are exceedingly complex natural events. They are resultants of no. of component parameters and are therefore very difficult to model analytically. For example, the floods in catchments depend upon the characteristic of the catchments, rainfall and antecedents' condition, each one of these factors in turns depends upon a host of constituent's parameters. This makes the estimation of the flood peak a very complex problem.

A frequency study interprets a past record of events to predict the future probabilities of occurrence with their return periods. The analysis should be based on the adequate and occurrence data. If the sample of data is too small then the prediction may not be accurate record for a period shorter then 20 years should not be used in the frequency analysis. Data should be homogeneous. For frequency analysis, the flood data may either consists of an annual series or partial duration series. In annual series the maximum value occurring in a particular year is considered. Where as a partial duration series 2 to 3 maximum values considered. Generally annual duration series is considered in analysis. The observed data is arranged in decreasing order of magnitude and return period of each event is calculated by expression given below

$T_r = (N + 1)/ m \frac{N+1}{m}$

$P = m / (N + 1)$

Where,
T_r = Return period of an event
P = Probability expressed in percentage
N = total number of events
m = Order or rank of a particular event

4.3 Gumbel's Extreme Value Distribution Function

This is the most widely used distribution function to predict a function event. Gumbel's proposed the formula for flood estimation as
$Y_t = a (x – x_f)$

Where,
Y_t = -0.834032-2.3 log log (Tr/Tr-1)
a and x_f are the parameters of the distribution which can be obtained by the following expression.
$a = 1.28255/S_x \frac{1.28255}{Sx}$
X_f = mean of X – 0.45005 S_x
Tr = Return period

Where,
Mean of X = mean of the given data series
S_x = Standard deviation of the given data series

4.4 log Pearson type III distribution function

Although this distribution as little theoretical basis, but it is being widely used as a tool to predict the feature food event by several western agencies. The following formulas are uses in determining the flood peak value for a given T_r
$Y_i = \log_{10} (X_i)$
Mean of Y = $(1/n).\sum^{n}_{i=1} Y_i$
S_y = √ (∑ (Y_i – mean of Y) ^{2})/ (n-1)
g = (n∑ (Y_i – mean of Y) ^{3})/ ((n-1) (n-2)
Sy2) $\frac{n\sum(Yi-\text{mean of } Y)^3}{(n-1)(n-2)Sy^3}$
Y = mean of Y + K S_y
Where, mean of Y = mean of the converted data series
S_y = Standard deviation of the converted data series
K = Factor to be determine from table
X = antilog (Y)

5. Determination of Peak Flood Discharge

5.1 Regional formula

Many formulas have been devised for the purpose of the estimating flood flows. They can be safely applied to the areas for which they were specifically derived, but some of them have general existence. But these formulas must be used with great prudence, and must never be used unless their origin has been investigated. No particular formula will give precise results for all the places. This is because of the fact that magnitude of the flood of a given frequency depends upon 15 to 20 factors. And no formula involves all these variables. Hence a formula involving only two or three variables in place of 20 variables cannot be expected to give generalized precise results. Some of these important formulae are given below.

5.2 Dickens formula

This formula is generally used for the catchment of north India, and states that
$Q_P = CA^{3/4}$
Where, Q_p = High flood discharge in cumecs
A = Catchment area in Sq. km.
C = a constant

5.3 Ryve's formula

This formula is applicable to the basin of south India
$Q_p = C.M^{2/3}$
Where, M = Catchment area (sq. km)
C = coefficient

5.4 Inglis formula

This formula applicable to the basin to the fan shaped catchments in old Bombay state
Q_p = 124 A/ (√ (A + 10.4))

Where, Q_p = Peak discharge (cumecs)
A = catchment area (sq. km.)

This formula was evolved by Creager from the study of flood peaks in USA and states that
$Q_P = C.A_1^{0.894A_1 \wedge -0.048}$
Where, C = 40 to 130
A_1 = area in square miles

6. Flood Routing

Flood Routing is the technique of determining the flood hydrograph at a section of river by utilizing the data of flood

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flow at one or more upstream sections. The hydrologic analysis of problems such as flood forecasting, flood protection, reservoir design and spillway design invariably include flood routing. In these applications two broad categories of routing can be recognised.

1. Reservoir Routing
2. Channel Routing.

6.1 Reservoir Routing

In Reservoir routing the effect of a flood wave entering a reservoir is studied. Knowing the Volume-Elevation characteristics of the reservoir and the outflow-elevation relationship for the spillway and other out-let structures in the reservoir, the effect of a flood wave entering the reservoir is studied to predict the variations of reservoir elevation and outflow discharge with time. This form of reservoir routing is essential

1) In the design of the capacity of spillways and other reservoir outlet structures ,and
2) In the location and sizing of the capacity of reservoir to meet specific requirements.

6.2. Channel Routing

In Channel routing the change in the shape of a hydrograph as it travels down a channel is studied. By considering a channel reach and an input hydrograph at the upstream end, this form of routing aims to predict the flood hydrograph at various sections of the reach. Information on the flood-peak attenuation and the duration of high- water levels obtained by the channel routing is of utmost importance in flood – forecasting operations and flood-protection works.

6.3 Basic Equations of Flood Routing

The passage of a flood hydrograph through a reservoir or a channel reach is an unsteady-flow phenomenon. It is classified in open- channel hydraulics as gradually varied unsteady flow. The equation of continuity used in all hydrologic routing as the primary equation states that the difference between the inflow & outflow rate is equal to the rate of change of storage, i.e.

$I - Q = ds/dt$

Where,

I= Inflow rate,

Q=out flow rate

S=storage

Alternatively, in a small time interval (delta t) the difference between the total inflow volume and total outflow volume in a reach is equal to the change in storage in that reach.

$$\bar{I}\,\Delta t - \bar{Q}\,\Delta t = \Delta S \qquad (1)$$

Where,

$\bar{I}$ = average inflow in time Δt

$\bar{Q}$ = average outflow in time Δt

ΔS= change in storage.

By taking

$\bar{I} = (I_1+I_2)/2$,

$\bar{Q} = (Q_1+Q_2)/2$ &

$\Delta S = S_2 - S_1$, with suffixes 1 & 2 to denote the beginning & end of time interval Δt, equation 1 is written as

$$\frac{(I_1+I_2)}{2}\Delta t - \frac{(Q_1+Q_2)}{2}\Delta t = S_2 - S_1 \qquad (2)$$

The time interval Δt should be sufficiently short so that the inflow & outflow hydrographs can be assumed to be straight line in that time interval. Further it must be shorter than the time of transit of the flood wave through the reach.

6.4 Modified Puls Method

Equation 2 is rearranged as

$$\frac{(I_1+I_2)}{2}\Delta t + \left(S_1 - \frac{Q_1\,\Delta t}{2}\right) = \left(S_2 + \frac{Q_2\Delta t}{2}\right) \qquad (3)$$

At the starting of flood routing the initial storage and out flow discharge are known .In equation 3 all the terms in the left hand side are known at the beginning of a time step Δt. Hence the value of the function $(S_2 + Q_2\Delta t/2)$ at the end of the time step is calculated by equation 3. Since the relation S = S (h) and Q = Q (h) are known, $(S+Q\ \Delta t/2)_2$ will enable one to determine the reservoir elevation and hence the discharge at the end of the time step. The procedure is repeated to cover the full inflow hydrograph.

For practical use in hand computations, the following semi graphical method is very convenient:

1) From the known storage- elevation and discharge-elevation data, prepare a curve of (S+Q $\Delta t/2$) vs elevation. Hence Δt is any chosen interval, approximately 20 to 40% of the time of rise of the inflow hydrograph.
2) On the same plot prepare a curve of outflow discharge vs elevation.
3) The storage, elevation and outflow discharge at the starting of routing are known. For the first time interval Δt, $\underline{(I_1+I_2)/2}\ \Delta t$ and $(S_1+ \underline{Q_1\ \Delta t/2})$ are known and hence by equation (3), $(S_2 + \underline{Q_2\Delta t/2})$ is determined.
4) The water- surface elevation corresponding to $(S_2 + \underline{Q_2\Delta t/2})$ is found by using the plot of step-1.
5) The outflow discharge Q_2 at the end of the time step Δt is found from plot of step-2
6) Deducting $Q_2\ \Delta t$ from $(S_2 + \underline{Q_2\Delta t/2})$ gives $(S - \underline{Q\ \Delta/2t})_1$ for the beginning of the next time step.
7) The procedure is repeated till the entire inflow hydrograph is routed.

6.5 Hydrologic channel routing

In Reservoir routing, the storage was a unique function of the outflow discharge, S= f (Q). However in channel routing the storage is a function of both outflow and inflow discharges and hence, a different routing method is needed. The flow in a river during a flood belongs to the category of gradually varied unsteady flow. The water surface in a channel reach is not only parallel to the channel bottom but also varies with time. Considering a channel reach having a flood flow, the total volume in storage can be considered under two categories as

1. Prism Storage
2. Wedge Storage

6.5.1. Prism Storage

It is the volume that would exist if the uniform flow occurred at the downstream depth, i.e. the volume formed by an imaginary plane parallel to the channel bottom drawn at the outflow section water surface.

6.5.2 Wedge Storage

It is the wedge - like volume formed between the actual water surface profile and the top surface of the prism storage. At a fixed depth at a downstream section of a river reach, the prism storage is constant while the wedge storage changes from a positive value at an advancing flood to a negative value during a receding flood. The prism storage S_p is similar to a reservoir and can be expressed as a function of the outflow discharge, $S_p = f(Q)$. The wedge storage can be accounted for by expressing it as $S_w = f(I)$. The total storage in the channel reach can then be expressed as

$$S = K[x I^m + (1-x) Q^m] \qquad (1)$$

Where K & x are coefficients and m=a constant exponent. It has been found that the value of m varies from 0.60 for rectangular channels to a value of about 1.00 for natural channels.

6.5.3 Muskingum Equation

Using m= 1.00, equation (6) reduces to a linear relationship for S in terms of I and Q as

$$S = K[x I + (1-x) Q] \qquad (2)$$

and this relationship is known as the Muskinghum Equation. In this the parameter x is known as weighing factor and takes a value between 0 and 0.50. When x= 0, obviously the storage is a function of discharge only and equation (7) reduces to

$$S = KQ \qquad (3)$$

Such storage is known as linear storage or linear reservoir. When x = 0.50 both the inflow and outflow are equally important in determining the storage.

The coefficient K is known as storage- time constant and has the dimensions of time. It is approximately equal to the time of travel of a flood wave through the channel reach.

Normally, for natural channels, the value of x lies between 0 to 0.30. For a given reach, the values of x and K are assumed to be constant.

For a given channel reach by selecting a routing interval Δt and using the Muskinghum equation, the change in storage is

$$S2 - S1 = K[x (I_2-I_1) + (1-x) (Q_2-Q_1)] \qquad (4)$$

Where suffixes 1 and 2 refer to the conditions before and after the time interval Δt. The continuity equation for the reach is

$$S_2-S_1 = \frac{(I_2+I_1)}{2}\Delta t - \frac{(Q_2+Q_1)}{2}\Delta t \qquad (5)$$

From equations (9) and (10), Q_2 is evaluated as

$$Q_2 = C_0 I_2 + C_1 I_1 + C_2 Q_1 \qquad (6)$$

$$\text{Where} \quad C_0 = \frac{-Kx+0.50\,\Delta t}{K - Kx+0.50\,\Delta t} \qquad (7a)$$

$$C_1 = \frac{Kx+0.50\,\Delta t}{K - Kx+0.50\,\Delta t} \qquad (7b)$$

$$C_2 = \frac{K-Kx-0.50\,\Delta t}{K - Kx+0.50\,\Delta t} \qquad (7c)$$

Note that $C_0+C_1+C_2 = 1.00$, Equation (11) can be written in a general form for the n^{th} time step as

$$Q_n = C_0 I_n + C_1 I_{n-1} + C_2 Q_{n-1} \qquad (8)$$

Equation (8) is known as Muskinghum Routing Equation and provides a simple linear equation for channel routing. It has been found that for best results the routing interval Δt should be so chosen that K > Δt > 2Kx, the coefficient C_0 will be negative. Generally, negative values of coefficients are avoided by choosing appropriate values of Δt.

To use the Muskinghum equation to route a given inflow hydrograph through a reach, the values of K and x for the reach and the value of the outflow Q_1, from the reach at the start are needed. The procedure is as

a) Knowing K and x, select an appropriate value of Δt.
b) Calculate C_0, C_1 and C_2.
c) Starting from the initial conditions I_1, Q_1 and known I_2 at the end of the first time step Δt calculate Q_2 by equation (11)
d) The outflow calculated in step (c) becomes the known initial outflow for the next time step. Repeat the calculations for the entire inflow hydrograph.

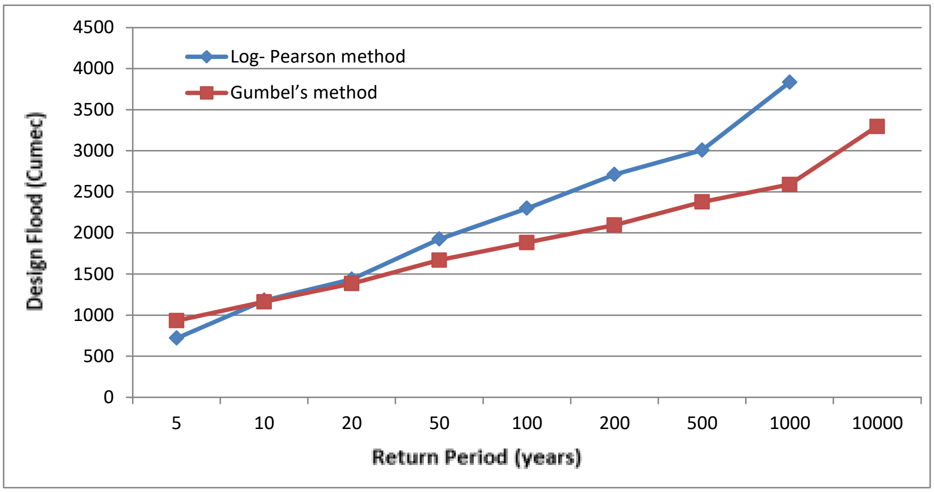

Figure 1: Comparison of design flood at Kakkadavu dam by different methods

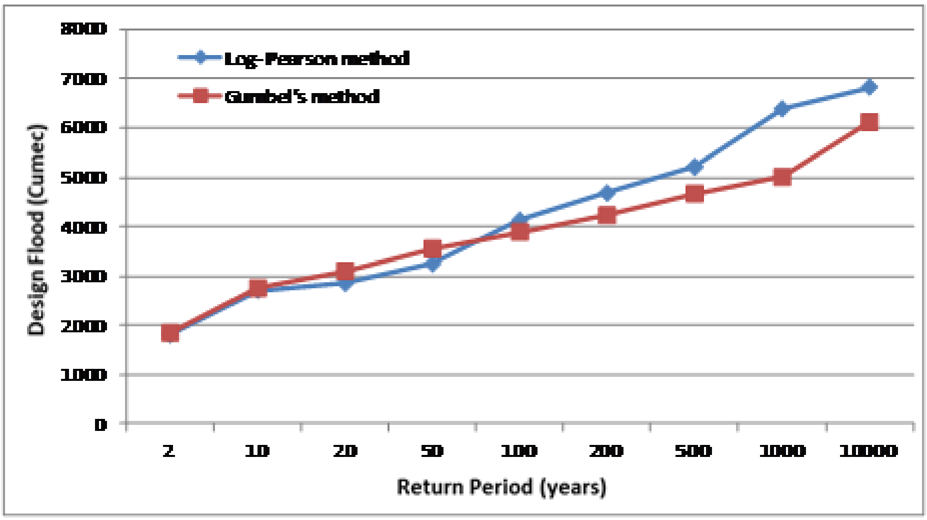

Figure 2: Comparison of design flood at Kol dam by different methods

Flood Routing at Kol dam site

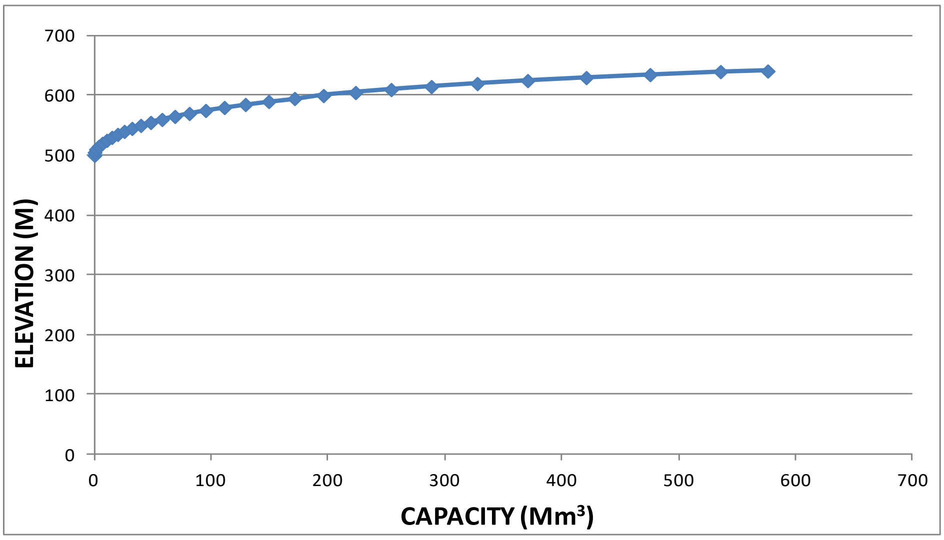

Figure 3: Elevation versus Capacity

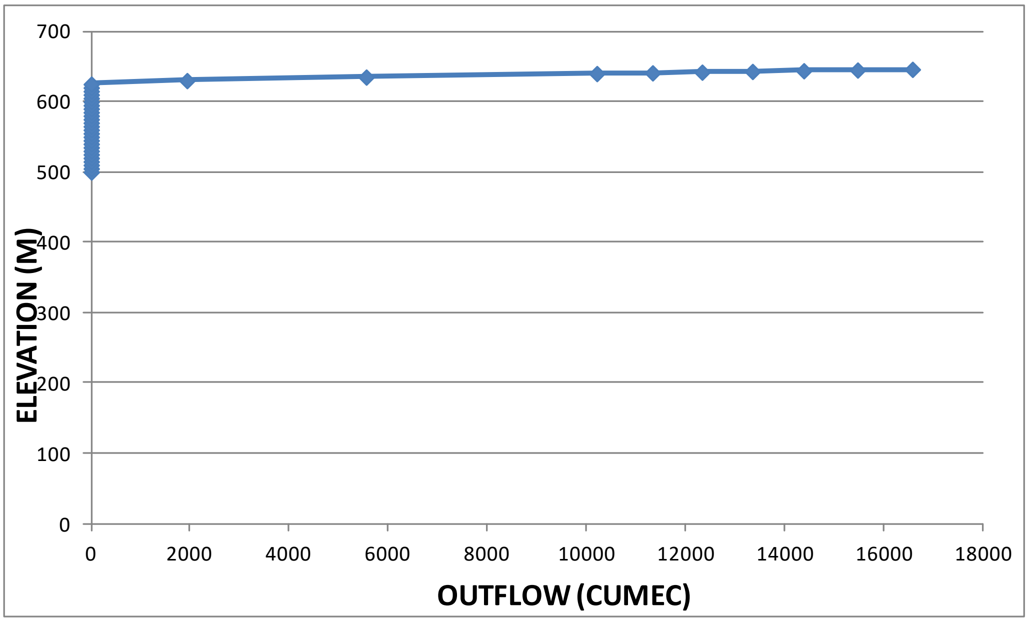

Figure 4: Elevations versus Outflow

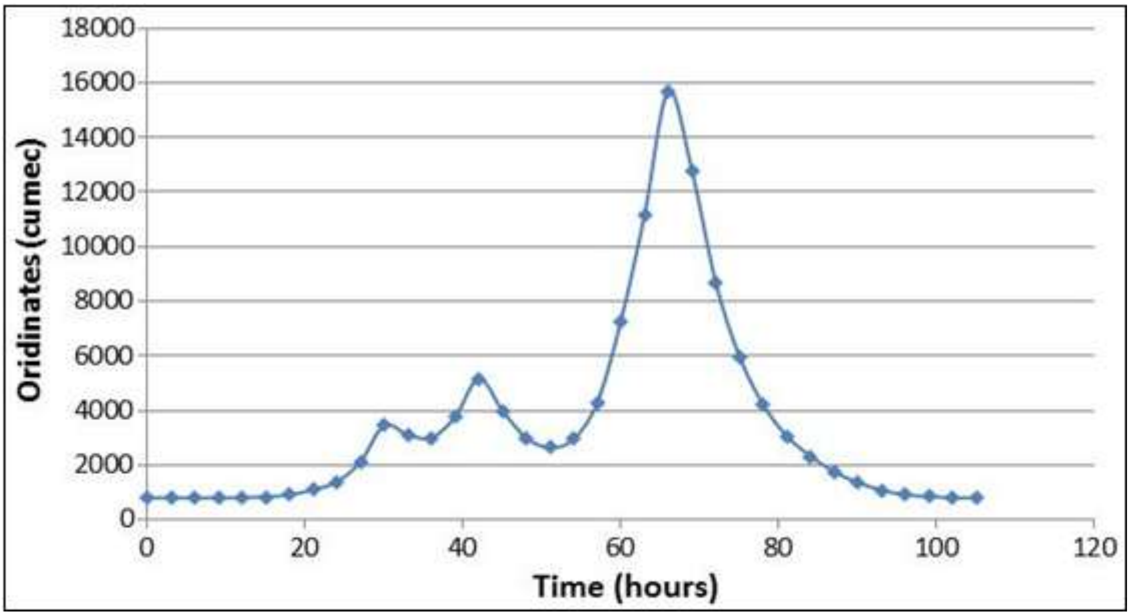

Figure 5: Inflow hydrograph

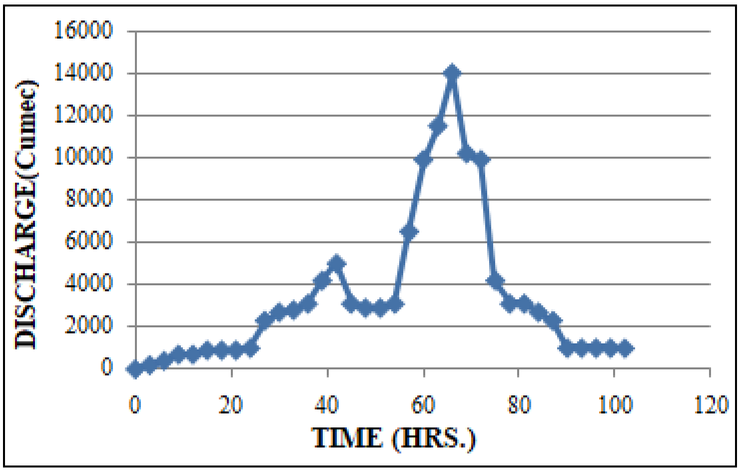

Figure 6: Outflow hydrograph

7. Conclusion

The estimation of design flood has been carried out for two dam sites, namely Kol Dam site in Himachal Pradesh and Kakkadavu Dam Site in Kerala. For the Kol Dam, peak flood discharge is available for the period 1964-1999. For the Kakkadavu dam site, the flood peak discharge data was available for period 1966-1994. The length of the available data was found satisfactory for the frequency analysis. The missing data for both the sites were filled-in using the mean values. Once the data set is completed, the analysis of data has been carried out and results described in the previous chapters. The rainfall and runoff data is available for both the basins. For the Kakkadavu dam site, the catchment area was calculated using different topographical sheets obtained from the Survey of India. Computation of catchment area is a prerequisite for applying the hydro meteorological approach.

Two approaches have been used for the estimation of design flood, namely hydro meteorological approach and frequency approach. For the Kol Dam, frequency analysis has been used to estimate the design flood. It was not possible to estimate the design flood for Kol Dam Site using hydro meteorological approach because the contribution to precipitation is mainly from snowmelt. For the Kakkadavu dam, both the frequency analysis and hydro meteorological approach have been used as there is no contribution of snowmelt. Using Hydro metrological approach for Kol Dam was estimated to be 15705 cumec. For the Kakkadavu dam site, the design flood using hydro meteorological approach was found to be 3148 cumec. Using the frequency approach the design flood was estimated at 3280 cumec. It can be seen that the value of design flood is approximately same for both the approaches. However, for the design purposes the higher value of 3280 cumec is recommended. The routed flood at koldam site is 14000 cumec which recommended for design discharge

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