“POWER POTENTIAL AND HYDRLOGICAL ANALYSIS OF LUHRI HEP (STAGE-1)”

A PROJECT

Submitted in partial fulfillment of the requirements for the award of the degree of

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IN

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Under the supervision of :

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By

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to

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CERTIFICATE

This is to certify that the work which is being presented in the project title **"Power Potential and Hydrological Analysis of Luhri HEP stage-1 "** in partial fulfillment of the requirements for the award of the degree of Bachelor of technology and submitted in Civil Engineering Department, **Jaypee University of Information Technology, Waknaghat** is an authentic record of work carried out by Aayushi Mahajan (121639) during a period from **August 2014 to May 2015** under the supervision of **Mr Lav Singh** Assistant Professor, Civil Engineering Department, Jaypee University of Information Technology, Waknaghat.

The above statement made is correct to the best of my knowledge.

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Aayushi Mahajan (121639)

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREVIATIONS

1	PDF	Probability Density Function
2.	CDF	Cumulative Distribution Function
3.	MW	Mega Watt
4.	NTPC	National Thermal Power Corporation
5.	FRL	Full Reservoir Level
6.	EL	Elevation
7.	MDDL	Minimum Draw Down Level
8.	CWC	Central Water Commission
9.	SJVN	Satluj Jal Vidyut Nigam Limited
10.	DPR	Detailed Project Report
11.	HEP	Hydro Electric Project
12..	ROR	Run Of River
13.	CEA	Central Electricity Authority
14.	FDC	Flow Duration Curves
15.	EF	Estimated Flood
16.	PFR	Preliminary Flow Reconnaissance

ABSTRACT

This project involves analysis of discharge data collected over a period of almost forty years to determine various parameters that are essential for determining the viability of the project as well as values that are required in the design stage . The parameters that were calculated were : 90% dependable year , Annual Generation and Installed Capacity Optimization was also done using the first two . 90% dependability gives those discharges which will be available 90% of the time during the operation of the project and hence using these discharge values the energy that we compute will be generated 90% of the time .

Flood frequency studies were done using the peak flows in each year and the maximum likely flood that can occur in a certain return period was computed . this value will be used in design phase where the dam, sluices and other support structures have to be designed to hold and pass a certain amount of flood . Flood frequency analysis was done using Gumbels Method . The probability distribution adopted here was Log Pearson type-3 distribution . For example, the study of peak flows uses just the largest flow recorded each year at a gaging station out of the many thousands of values recorded.

The analysis also involves the determination of probable maximum flood , which is the single greatest amount of flood that the structure will have to withstand occurring due to the worst possible storm that can occur in a given catchment . First the unit hydrograph is estimated using empirical formulae later the convolution of storm with the unit hydrograph after the addition of base flow leads to the probable maximum flood which is basically a discharge verses time graph **.** Flood routing studies determine the amonuts of flood that can be passed safely and the way they are to be assed using various input graphs that will be plotted

Keywords : 90% dependable year ,Installed capacity ,flood frequency , Probable Maximum Flood, Flood Routing

CHAPTER 1: INTRODUCTION

1.1 The Project

The Government of India and the State of Himachal Pradesh have identified the Satluj River as one of the main promising source of hydroelectric power. Major development of the Satluj River was started by the Bhakra Nangal Project. Since then, further major hydroelectric projects have been initiated along the length of the Satluj and its tributaries. Luhri Hydroelectric Project Stage-l (219 MW) is located in Kullu and Shimla district of Himachal Pradesh on river Satluj, downstream of Rampur Hydroelectric project and upstream of Kol Dam project of NTPC Ltd.

This project envisages construction of a concrete gravity dam at Nirath (catchment area 51600 km^2) around 86 m high above deepest foundation level with integral spillway and dam toe power house with 3 nos. turbine on right bank with installed capacity of 200 MW and 2 nos. turbine on left bank with installed capacity of 19 MW. FRL is kept at EL 862.90m and MDDL is fixed at EL 860 m.

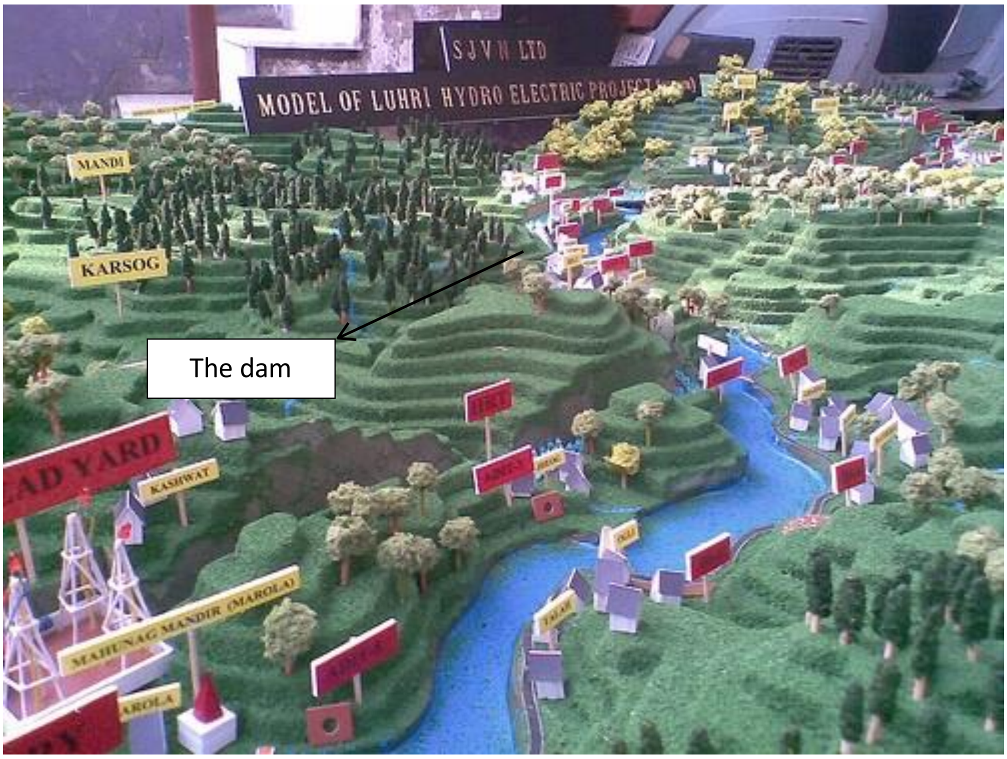


Fig. 1 A model of the project

1.2 Satluj Catchment Characteristics

The geographical limits of the Satluj basin upto Bhakra Dam lie between latitude 30°N and 33°N and longitude 76°E and 83°E covering area from Nan Khorsam province of Tibet (China) to Himachal Pradesh State of India. The climatic conditions of the Satluj River basin are strongly influenced by orthographic effects.

The boundary between areas receiving mostly precipitation in the form of rain and those receiving mostly snow is at an elevation of (approximately) 1525m. The catchment covers approximately 51,600 km2 and 75% of this is largely snow bound. The catchment receives precipitation due to the South-West monsoon as well as the western disturbances that pass over the north-west part of the country during winter.

The South-West monsoon generally lasts from June to September, but may occasionally extend up to early October. During this period rainfall is generally not heavy but at times snowmelt contributes significantly to flood runoff, with maximum flows occurring between June and August**.** The winter precipitation falls either as rain or snow depending upon altitude and other meteorological conditions and may be very heavy on occasions but does not usually contribute directly to river discharge significantly and mostly goes to feed the snow glacier bound areas of the catchment**.**

1.3 Physiographic Parameters

Sr. No.	Parameters	Value
1.	Total Catchment area (A)	51,600 km^2
2.	Total catchment area in Indian Territory	14,700 km^2
3.	Total catchment area in Tibetan(China)Territory	36,900 km^2
4.	River Length in Tibetan territory	320 km
5.	Permanent Snow Line Elevation	4200 m
6.	Total Snow fed area (As)	38827 km^2 (75.25% of A)

Table 1

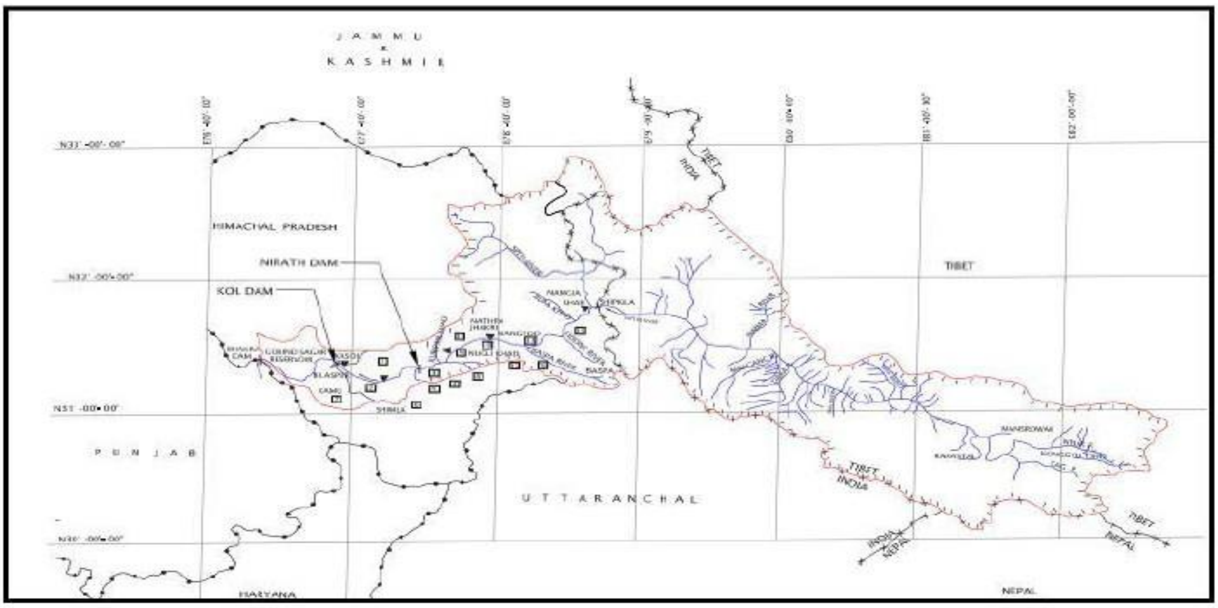


Fig. 2 Satluj Catchment

A catchment area is a hydrological unit. Each drop of precipitation that falls into a catchment area eventually ends up in the same river going to the sea if it doesn't evaporate. However, it can take a very long time. Catchment areas are separated from each other by watersheds

1.4 River Flows

There are seven river gauge and discharge sites on the main stem of the Satluj River up to Bhakra dam: their details (catchment area and period of data availability) are shown in Table 1-2. The longest period of record is from 1909 at Onlinda (Bhakra) (no.7). However, the discharge observations from 1960 onwards are made from Gobind Sagar level fluctuations and release through the power house unit and through spillway/irrigation outlets. A gauging station was established in 1966 at Kasol (no.6), upstream of Bhakra. Initially, the observations were made by float, but current meter observations commenced in 1984.

The gauging site at Rampur , maintained by BBMB, was established in June 1963. The stream flow data were initially measured using a current meter and subsequently using floats. No Central Water Commission discharge observation site is available in the catchment above the Rampur project site.

Location	Catchment Area (km^2)			Availability of Data
	Tibet	India	Total	
1. Satluj at Khab	34505	150	34655	1972 to 2014
2. Spiti at Khab	2395	7085	9480	1972 to 2002
3. Wangtoo/ Nathpa Dam site	36900	12920	49820	1966 to 2004
4. Rampur	36900	13980	50880	1963 to 2008
5. Nirath Dam Site	36900	14700	51600	2005 to 2014
6. Sunni	36900	16015	52915	1963 to 2006
7. Kasol	36900	16870	53700	1963 to 2006
8. Onlinda / Bhakra Dam	36900	19975	56875	1909 to 1965

Table 2 Guage and Discharge sites .

Since 1970 state Government staff have regularly measured the discharge of Shoulding Khad, which has a catchment area of 86 km^2 and joins the Satluj River between Nathpa and Jhakri power house on the left bank. However, since 1987 CWC have disregarded discharge data from this point, as they were inconsistent and inaccurate.

A gauge was established at the Nirath site by SJVN in March 2005 when that site was identified for the project. Whilst there are no long term flow records for Nirath there are four gauging stations on the Satluj River with catchments of a similar size (within 5%) to Nirath (see Table 1-3). The closest, at Rampur, has a catchment which is 1.4% smaller.

Gauging Station	Catchment Area	Ratio to Catchment Area at Nirath
Nathpa	49820 km^2	0.966
Rampur	50880 km^2	0.986
Nirath	51600 km^2	1
Sunni	52915 km^2	1.025
Kasol	53700 km^2	1.041
Kol	53770 km^2	1.042

Table 3 Catchment Areas

1.5 Water Availability for Generation

During examination of DPR of Luhri HEP (775MW), hydrology chapter were modified as per the observations of CWC and water availability series was developed from year 1972 to 2008. The observations of CWC and reply thereof are appended as Appendix-A and the approval of water availability series is appended as Annexure-B. However, the wafer availability series has now been updated upfo year 2014. The methodology adopted for development of water availability series are given below: The flow series at Nirath was computed using following approach:

1. The intermediate catchment contribution between Rampur and Khab was estimated by subtracting 10-daily flow available downstream of confluence of Spiti and Satluj at Khab (1972-2008) from the Rampur observed 10-daily flow.
2. The Inflow series at Nirath was estimated by transferring intermediate catchment flow between Rampur and Khab to Nirath dam in catchment area proportion (7465/6745) and Khab 10-daily flows were added.
3. In few cases the computed intermediate flow were found negative and there were few gaps also in the Khab data, in such cases, the 10-daily flow at Rampur were directly transferred to Nirath dam site on catchment area proportion.

CHAPTER 2 : LITERATURE REVIEW

2.1 90% Dependability Studies and Installed Capacity

From Journals :

- *Assessment of environmental flow requirements for hydropower projects in India (Sharad K. Jain), Current Science, Vol. 108, No. 10, 25 May 2015*

Appraisal of the hydrology of the river and flow regime which is best described by the time series of daily discharge30. The flow regime can be summarized as a FDC, which is the graphical representation of discharge versus the exceedance probability. A great advantage of using FDCs is that they can be readily transferred between sites in the same basin and thus are useful in estimating flows at ungauged locations.

FDCs corresponding to different dependability years help understand the response of the basin in different hydrologic scenarios. In India, 90% dependable flows are used for hydropower planning31 . (b) A number of scenarios of EF are constructed using, for example, FDC for the 90% dependable year. Note that any other dependability may as well be chosen, but the 90% dependability is convenient from implementation point of view since there is very low probability that the actual flow in the river will be less than these flows.

- *MOWR, Guidelines for preparation of detailed project report of irrigation and multipurpose projects. Ministry of Water Resources, Government of India, 2010*

Planning of HE Project is carried out based on 90 per cent dependability criteria. For determination of 90% dependable year, the total energy generation in all the years for which hydrological data is available (say N year) is arranged in descending order and the (N+1) x 0.9 th year would represent the 90 per cent dependable year.

The 90 per cent dependable year is thus, termed as the year in which the annual generation has the probability of being equal to or exceed 90 per cent of the time on annual basis during the expected period of operation of the scheme. For example, if inflow data is available for a period of 20 years (N=20), then,

90% Dependable year = (20+1)*0.9= 18.9 = 19^{th} yr

Power Potential Studies :

Power Potential studies are carried out for assessment of available Power Potential of a river/basin based on a set of inflows and available head conditions under various operating policies. These studies play an important role in the optimisation and design of new hydro facilities. They are used for examination of various configurations and their integration into existing networks. The studies are carried out for optimization of project parameters and for evaluation of Energy and Power benefits.

- *CEA Guidelines for DPRs (appendix 1)*

The first step is to compute 90% dependable year :

- Obtain 10-daily hydrological inflow series in m3/sec for all hydrological years, year-wise.
- Calculate unrestricted energy generation in MUs.
- Arrange unrestricted annual energy generation in descending order.
- 0.9(n+1)th year is the 90% dependable year, where n is the number of years for which hydrological inflows data is available.

Fixating the installed capacity :

- Calculate firm power available based on average power generation during the lean months flows in a 90% dependable year.
- Consider a number of alternatives of installed capacities in suitable steps say 5%, for load factors say about 40% down to about 15%.
- Compute incremental energy generation (ΔKWH) for every incremental MW (ΔMW) and plot result on a graph.
- Installed capacity is fixed at a value where the fall in the graph is sharp. B/C ratio and incremental benefit cost ratio (ΔB/ΔC) is also considered for fixing the installed capacity.
- An alternative for installed capacities which provides maximum net benefit (B-C) and ensures incremental (ΔB/ΔC) higher than unity is considered optimum .

PFR studies of Devsari H.E. Project :

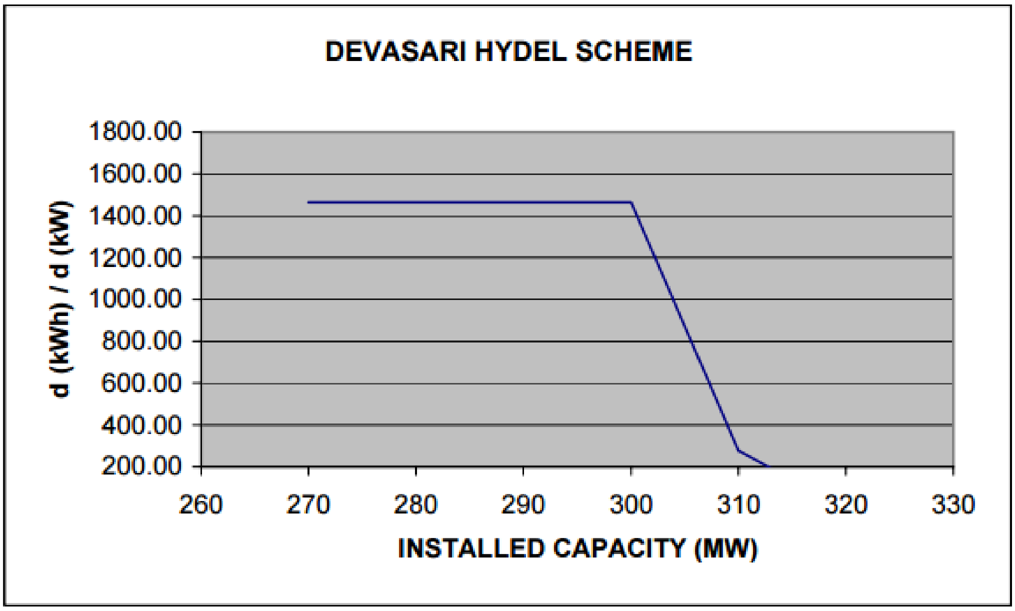


Fig. 3 Installed capacity optimization curve for proposed Devsari project in Uttrakhand .

Computing Design energy : 10-daily unrestricted energy generation in 90% dependable year is restricted to 95% of the installed capacity of the power house. The total of these 10-daily restricted energies for the year gives the annual design energy generation

2.2 Flood Frequency Studies and Gumbel Method

From books :

- *Varshney R.S (1986) Engineering Hydrology. Nem Chand and Bros. Roorkee . Third edition*

Dams are important hydraulic structures which are constructed to serve a variety of purpose. Most dams have a capacity to store substantial amount of water in the reservoir, and a portion of the inflow flood gets stored and the excess overflows through the spillways. According to Bureau of Indian Standard Guidelines IS: 11223-1985, "Guidelines for fixing spillway capacity".

Flood frequency analysis studies interpret past record of events to predict the future probabilities of occurrence and estimate the magnitude of an event corresponding to a specific return period. For the estimation of flood flows of large return periods, it is often necessary to extrapolate the magnitude outside the observed range of data.

Gumbel distribution is a member of family of Extreme Value distributions with the value of parameter k = 0. It is a two parameter distribution and is widely used in hydrology.

The PDF is given as :

$$f(x) = \frac{1}{\alpha} \exp\left[-\frac{x-\mu}{\alpha} - \exp\left\{ -\frac{x-\mu}{\alpha} \right\} \right]$$

And CDF is given as :

$$F(x) = \exp\left[-\exp\left\{ -\frac{x-\mu}{\alpha} \right\} \right]$$

From Journals :

- *Flood frequency Modeling using Gumbel's and Powell's method for Dudhkumar river (June 30 , 2013)*

Flood frequency analysis (FFA) is the estimation of how often a specified event will occur. Before the estimation can be done, analyzing the stream flows data are important in order to obtain the probability distribution of flood (Ahmad et. al., 2010). One of the greatest challenges facing the Hydrology is to gain a better understanding of flood regimes.

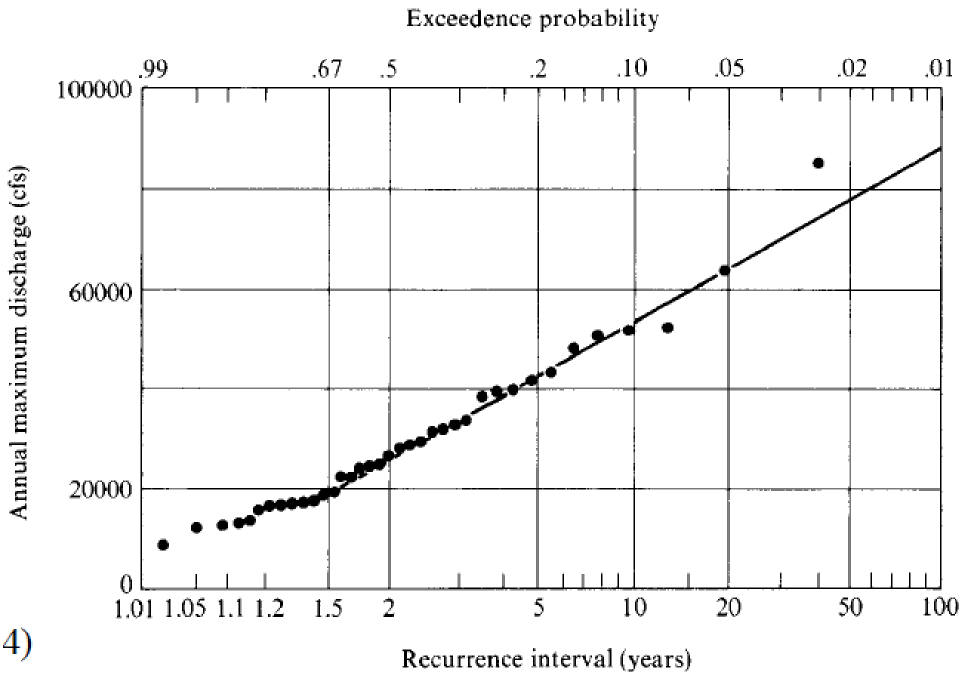
To do this, flood frequency analysis (FFA) is most commonly used by engineers and hydrologists worldwide and basically consists of estimating flood peak quantities for a set of non-exceedance probabilities. The validity of the results in the application of FFA is theoretically subject to the hypothesis that the series are independent and identically distributed (Stedinger et al., 1993; Khaliq et al., 2006). Nevertheless, to determine flood flows at different recurrence intervals for a site or group of sites is a common challenge in hydrology .

- *Evolution of Methods £or Evaluating the Occurrence of Floods . Manuel A. Benson. Geological Survey Water-Supply Paper 1580-A*

The frequency curve, or cumulative distribution curve, of flood data from a given site is used to determine the floods of specific recurrence intervals or probabilities, such as the 25-year flood or its equivalent, the 0.04-probability flood.

The most recent work by Gumbel describes three basic "asymptotic distributions of extremes," of which the first two, and possibly the third, may be used for floods, depending on the distribution of the data in the sample. The free choice between the three, on the basis of best fit, emphasizes that this statistical method is merely an empirical process of curve fitting. In spite of this deficiency, the Gumbel method of fitting flood data is useful because in some ranges, particularly the low range, the frequency curve tends toward a straight line.

Flood analysis . Dunne and Leopold (1978) :



4)

Fig. 4 Ideal Food Frequency distribution chart

2.3 Flood Frequency Analysis using Log Pearson Type-3 distribution

From journals :

- *Flood Frequency Analysis of Upper Krishna River Basin catchment area using Log Pearson*

Type III Distribution (B. K. Sathe , M. V. Khire , R. N. Sankhua) ISSN: 2250-3021 Volume 2, Issue 8 (August 2012) .

As much of the hydraulic data like flow rate (discharge) and rainfall are statistical in nature, statistical methods are most frequently needed to be used often with the goal of fitting a statistical distribution to the data . Design flood is the discharge adopted for the design of a hydraulic structure and it is obviously very costly to design any hydraulic structure so as to make it safe against the maximum flood possible in the catchment.

The procedure for estimating the frequency of occurrence (return period) of a hydrological event such as flood is known as (flood) frequency analysis. Though the nature of most hydrological events (such as rainfall) is erratic and varies with time and space, it is commonly possible to predict return periods using various probability distributions . Flood frequency analysis was developed as a statistical tool to help engineers, hydrologists, and watershed managers to deal with this uncertainty

The Log-Pearson Type III distribution is a statistical technique for fitting frequency distribution data to predict the design flood for a river at some site. Once the statistical information is calculated for the river site, a frequency distribution can be constructed. The probabilities of floods of various sizes can be extracted from the curve. The advantage of this particular technique is that extrapolation can be made of the values for events with return periods well beyond the observed flood events. This technique is the standard technique used by Federal Agencies in the United States.

The Log-Pearson Type III distribution is calculated using the general equation

$$X=\bar{X}+K\sigma \qquad (1)$$

The model parameters , standard deviation and the skew coefficient (g) are computed from n observations X, with the

following formula :

$$\bar{X} = \frac{1}{n}\sum_{i=1}^{n} Xi \qquad (2)$$

$$\sigma = \left[\frac{1}{(n-1)}\sum(X-\bar{X})^2\right]^{1/2} \qquad (3)$$

$$g = \frac{\frac{1}{(n-1)}\sum(X-\bar{X})^2}{(n-1)(n-2)\sigma^2} \qquad (4)$$

However, the Log Pearson Type III distribution of X which has been widely adopted to reduce skewness is equivalent to applying Pearson Type III to the transformed variable log X and it is represented in the literature (e.g. HannC.T.(1977) Das and Saikia (2009); Jagadesh and Jayaram (2009); Wurbs and James, 2009) as:

$$\mathrm{log}X = \overline{logX} + \mathrm{K}\boldsymbol{\sigma}_{\mathrm{logX}} \qquad (5)$$

The frequency factor K is a function of skewness coefficient and return period and can be read from published tables (Table 5) developed by integrating the appropriate probability density function.The flood magnitude for various return periods are found by solving the general equation. The mean,standard deviation of the data and skewness coefficient can be calculated using the following formula :

$$\overline{logX} = \frac{\sum \log X}{n} \qquad (6)$$

$$\boldsymbol{\varphi}_{\boldsymbol{logX}} = \left[\frac{\sum(\boldsymbol{logXi} - \boldsymbol{logX})^2}{(\boldsymbol{n}-\boldsymbol{1})}\right]^{1/2} \qquad (7)$$

$$g = \frac{\sum(\boldsymbol{logXi} - \overline{\boldsymbol{logX}})^2}{(\boldsymbol{n}-\boldsymbol{1})(\boldsymbol{n}-\boldsymbol{2})\boldsymbol{\sigma}^3_{\boldsymbol{logx}}} \qquad (8)$$

From books :

- *Applied Hydrology (Ven Te Chow , David R. Maidment, Larry W. Mays) . 1988*

The study of extreme hydrologic events involves the selection of a sequence of the largest or smallest observations from sets of data. For example, the study of peak flows uses just the largest flow recorded each year at a gaging station out of the many thousands of values recorded. In fact, water level is usually recorded every 15 minutes, so there are 4 x 24 = 96 values recorded each day, Annual exceedence

Annual maximum Rank of values Original data. Base for annual exceedence values Magnitude Magnitude and 365 x 96 = 35,040 values recorded each year; so the annual maximum flow event used for flood flow frequency analysis is the largest of more than 35,000 observations during that year.

And this exercise is carried out for each year of historical data. Since these observations are located in the extreme tail of the probability distribution of all observations from which they are drawn (the parent population), it is not surprising that their probability distribution is different from that of the parent population.

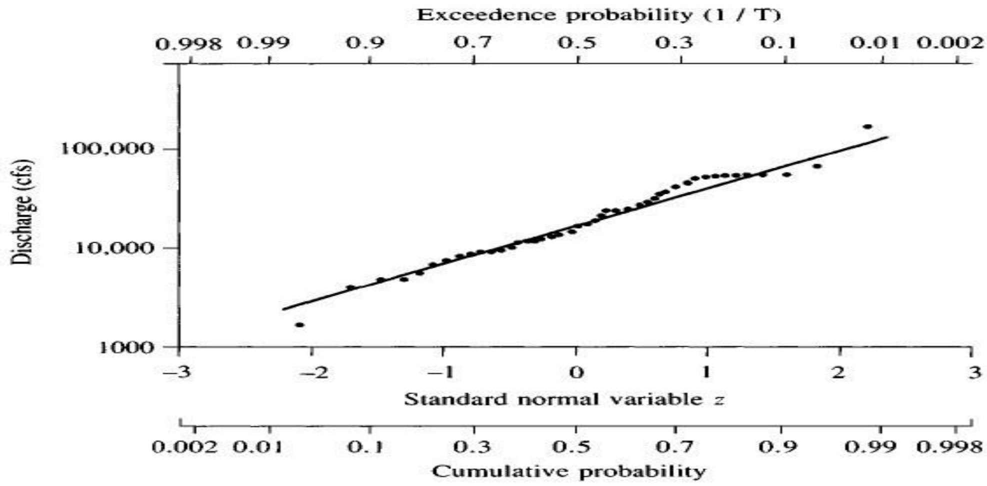


Fig.5 standard flood frequency curve

Log-Pearson Type 3 Distribution. For this distribution, the first step is to take the logarithms of the hydrologic data, $y = \log x$. Usually logarithms to base 10 are used. The mean *y,* standard deviation *sy,* and coefficient of skewness Cs are calculated for the logarithms of the data. The frequency factor depends on the return period *T* and the coefficient of skewness C5. When C5 = 0, the frequency factor is equal to the standard normal variable z.

2.4 Probable Maximum Flood Using Snyder unit hydrograph .

- *Engineering guidelines to determine the probable maximum flood . (U.S. Army Corps of Engineers, 1959)*

Synthetic Unit Hydrograph :

The unit hydrograph developed from rainfall and streamflow data on a watershed applies only for that watershed and for the point on the stream where the Deviation *en* between observed and estimated direct runoff hydrographs is the sum of a positive deviation *Sn* and a negative deviation */3n* for Time solution by linear programming. Estimated DRH Observed DRH Direct runoff streamflow data were measured. Synthetic unit hydrograph procedures are used to develop unit hydrographs for other locations on the stream in the same watershed or for nearby watersheds of a similar character.

There are three types of synthetic unit hydrographs: (1) those relating hydrograph characteristics (peak flow rate, base time, etc.) to watershed characteristics (Snyder, 1938; Gray, 1961), (2) those based on a dimensionless unit hydrograph (Soil Conservation Service, 1972), and (3) those based on models of watershed storage (Clark, 1943) In a study of watersheds located mainly in the Appalachian highlands of the United States, and varying in size from about 10 to 10,000 mi2 (30 to 30,000 km2), Snyder (1938) found synthetic relations for some characteristics of a *standard unit hydrograph* . Additional such relations were found later (U.S. Army Corps of Engineers, 1959). These relations, in modified form are given below.

From the relations, five characteristics of a *required unit hydrograph* for a given excess rainfall duration may be calculated: the peak discharge per unit of watershed area, *qPR,* the basin lag *tPR* (time difference between the centroid of the excess rainfall hyetograph and the

unit hydrograph peak), the base time fy, and the widths *W* (in time units) of the unit hydrograph at 50 and 75 percent of the peak discharge. Using these characteristics the required unit hydrograph may be drawn.

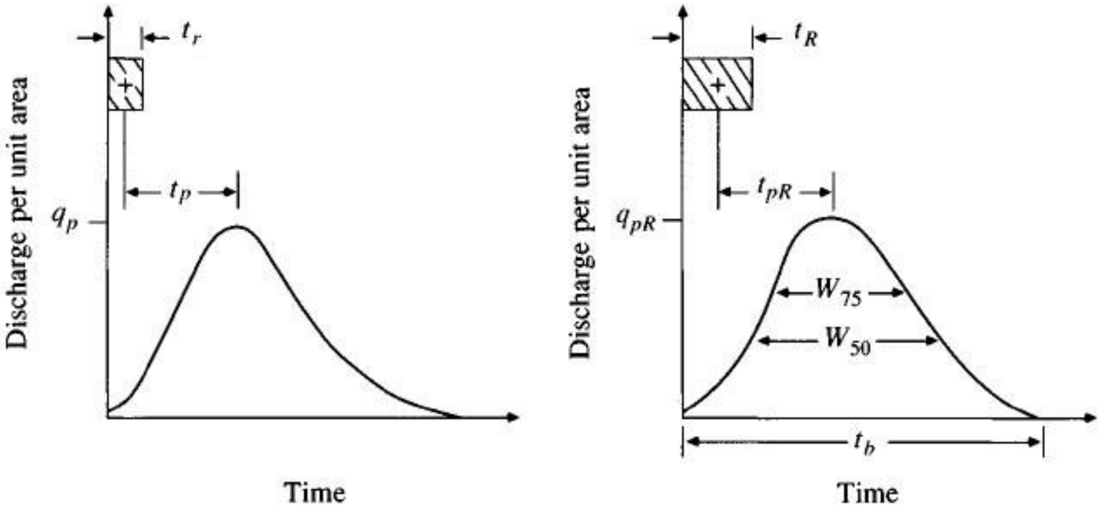


Fig. 6 standard Snyder unit hydrograph

1.) The basin lag is :

$$t_p = C_1 C_t (L L_c)^{0.3}$$

where *tp* is in hours, *L* is the length of the main stream in kilometers (or miles) from the outlet to the upstream divide, *Lc* is the distance in kilometers (miles) from the outlet to a point on the stream nearest the centroid of the watershed area, *C* = 0.75 (1.0 for the English system), and *Ct* is a coefficient derived
from gaged watersheds in the same region.

2. The peak discharge per unit drainage area in m3/s-km2 (cfs/mi2) of the standard unit hydrograph is

$$q_p = \frac{C_2 C_p}{t_p}$$

where *C2* = 2.75 (640 for the English system) and *Cp* is a coefficient derived from gaged watersheds in the same region. To compute *Ct* and *Cp* for a gaged watershed, the values of L and *Lc* are measured from the basin map. From a *derived unit hydrograph* of the watershed are obtained values of its effective duration *tR* in hours, its basin lag *tpR* in hours, and its peak discharge per unit drainage area, *qpRy* in m3/s*km2-cm (cfs/mi2-in for the English system). If *tpR = 5.5tR,* then *tR = trJpR = tp,* and *qpR = qp,* and *Ct* and *Cp* are computed by and If *tpR* is quite different from 5.5^, the standard basin lag is

$$t_p = t_{pR} + \frac{t_r - t_R}{4}$$

and Eqs. are solved simultaneously for *tr* and *tp.* The values of *Ct* and *Cp* are then computed from and with *qpR = qp* and *tpR = tp.* When an ungaged watershed appears to be similar to a gaged watershed, the coefficients *Ct* and *Cp* for the gaged watershed can be used in the above equations to derive the required synthetic unit hydrograph for the ungagged watershed.

3. The relationship between *qp* and the peak discharge per unit drainage area *qpR* of the required unit hydrograph is

$$q_{pR} = \frac{q_p t_p}{t_{pR}}$$

4. The base time *t^* in hours of the unit hydrograph can be determined using the fact that the area under the unit hydrograph is equivalent to a direct runoff of 1 cm (1 inch in the English system). Assuming a triangular shape for the unit hydrograph, the base time may be estimated by :

$$t_b = \frac{C_3}{q_{pR}}$$

where C_3 = 5.56 (1290 for the English system).

5. The width in hours of a unit hydrograph at a discharge equal to a certain percent of the peak discharge q_{PR} is given by :

$$W = C_w q_{pR}^{-1.08}$$

where *Cw* = 1.22 (440 for English system) for the 75-percent width and (770, English system) for the 50-percent width. Usually one-third of this width is distributed before the unit hydrograph peak time and two-thirds after the peak .

2.5 Flood Routing

From journals :

- *Hydrological Flood Routing in Rivers .V. Fasahat, A. Honarbakhsh, H. Samadi, S.J. Sadatinejad . Shahrekord, Iran (2013)*

flood routing is an important part of flood management. Although hydraulic models are commonly employed in the routing studies, hydrological models offer more effective and suitable methods in this matter Flood is a natural phenomenon that human societies have accepted it as an inevitable event. Flood is defined as a condition in which stream flow unexpectedly is increased so as to cause financial and fatal damage (Abbasi, 2005). Hydrological issues, particularly those concerning the prevention and control of floods, have been discussed for years in the world which indicates how important it is. Nevertheless, considering this issue may focus more on application of logical and improper principles to better control of this phenomenon.

The main concept of flood routing is that if there are hydrographical specifications in a point of a river, how one might estimate the hydrograph in another point in the downstream. This subject is important specially where agricultural lands located in downstream. Certainly, hydrographs of these two points may are not identical; because the specifications of the rout water is passing or is flowing in can change the shape of hydrograph. Basically routing is based on nonlinear correlation between reservoir and flow

- *Extended Muskingum method for flood routing . D. Nagesh Kumar , Falguni Baliarsingh , K. Srinivasa Raju . Department of Civil Engineering, Indian Institute of Science, Bangalore, India 12 August 2010.*

Flood routing is an important aspect in reservoir operation for flood control. This requires suitable flood routing relationship explicitly in the formulation of the policy. The releases from reservoir during floods should be so controlled that the total flow at a downstream station is within the safe limit. The downstream station at which the specified maximum flow is to be restricted is herein after referred as flood control station. The factors causing floods at flood control station are the release for power and spill from reservoir, measured inflow to the river from tributaries between the reservoir and the flood control station and unmeasured lateral flow from the intermediate catchment. The extended Muskingum method is examined in this study for its applicability as flood routing method for the case study of Hirakud reservoir, Mahanadi river basin, India. Nine floods from 1992e1995 are analyzed for this purpose.

- *Practice Manual For Small Dams, Pans and Other Water Conservation Structures in Kenya Ministry of water environment and natural resources .2011.*

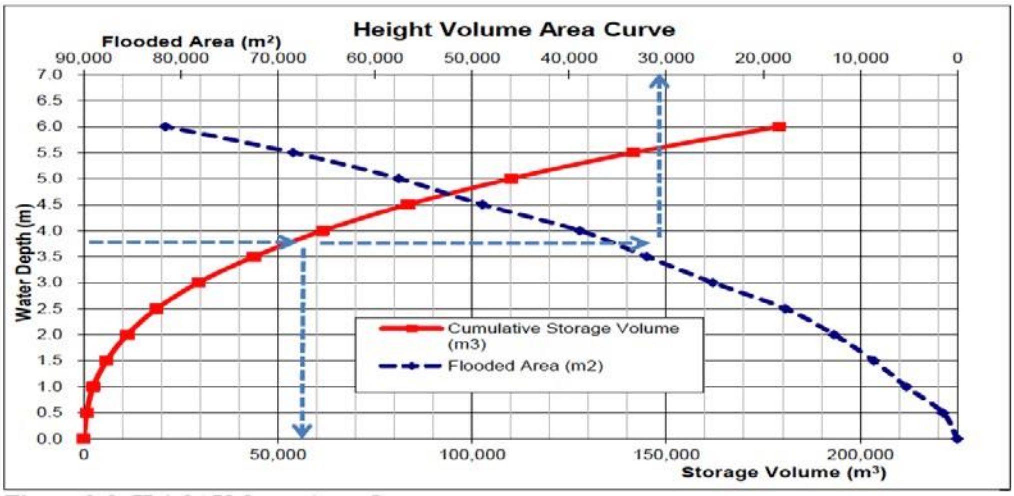


Fig. 7 standard area capacity curve

- *Hydraulic design manual .chapter 6 Channel Analysis Methods. Online manuals.txdot.gov*

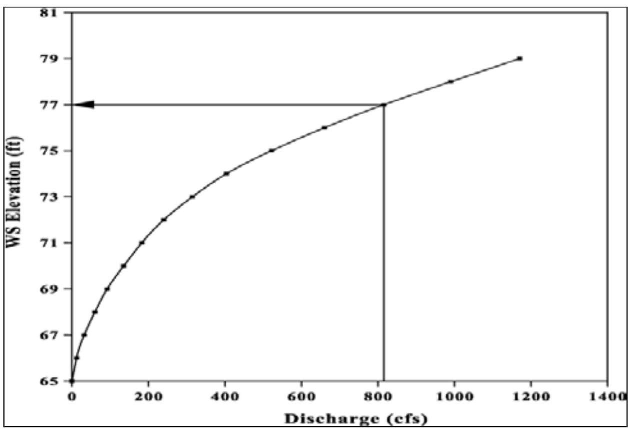


Fig. 8 A general sluice rating curve

- **Data Disclaimer :**

  The hydrological data like ten daily discharge data ,the annual peak flow data , rainfall data , physiograhic parametres were taken from SJVN Ltd who have been assigned this project by Govt. of Himachal Pradesh .

CHAPTER 3 : PROJECT OBJECTIVES AND SCOPE

Objectives and scope for Semester-7 :

- Learning about the catchment area of the river under consideration (Satluj) , the method by which **Water Availibility Series** was obtained for the project . Various aspects associated with the catchment like physiographic parametres, the river flow will be studied .
- Analysis of hydrological data for calculation of **90% dependable year** by using Weibull Probability approach . The '90% Dependable year' is defined as :If the total energy generation in the years for which hydrological data is available (say N years) is arranged in descending order, **the (N+1) x 0.9th year** would represent the 90% dependable year.
- Computing **the energy generation in 90% dependable year** , hence computing the Plant Load Factor (PLF) . The ***Plant Load*** **Factor (PLF)** is the ratio between the actual energy. generated by the *plant* to the maximum possible energy that can be. generated with the *plant* working at its rated power and for a duration
- Performing **Installed Capacity Optimization** by using Incremental Analysis. Installed capacity optimization involves varying the installed capacity and observing the variation of power generated with it. Installed capacity with maximum value of power generation is taken as the installed capacity for the project .
- Learning about the importance of **Flood Frequency Studies** and their suitability in different regions .
- Performing Flood Frequency Analysis using **Gumbel Method** .

Objectives and scope for Semester-8 :

- Performing **Flood Frequency Analysis** by using **Log Pearson Type-3 probability** distribution using the peak flow data as attached in annexure .
- Computation of **Probable Maximum Flood** using unit hydrograph approach . . this will require the use of rainfall and physiographic data as attached in annexure . **Probable Maximum Flood** (PMF) is the theoretically largest flood resulting from a combination of the most severe meteorological and hydrologic conditions that could conceivably occur in a given area.
- **Flood Routing Studies** (a method of operation of sluices) of the reservoir . the peak flow data , the inflow data from guage and discharge sites is utilized here . In hydrology, **routing** is a technique used to predict the changes in shape of a hydrograph as water moves through a river channel or a reservoir

CHAPTER 4 : METHODOLOGY

4.1 **Flow Chart**

The following flow chart illustrates the step by step process that was put followed while performing various computations to compile the project .

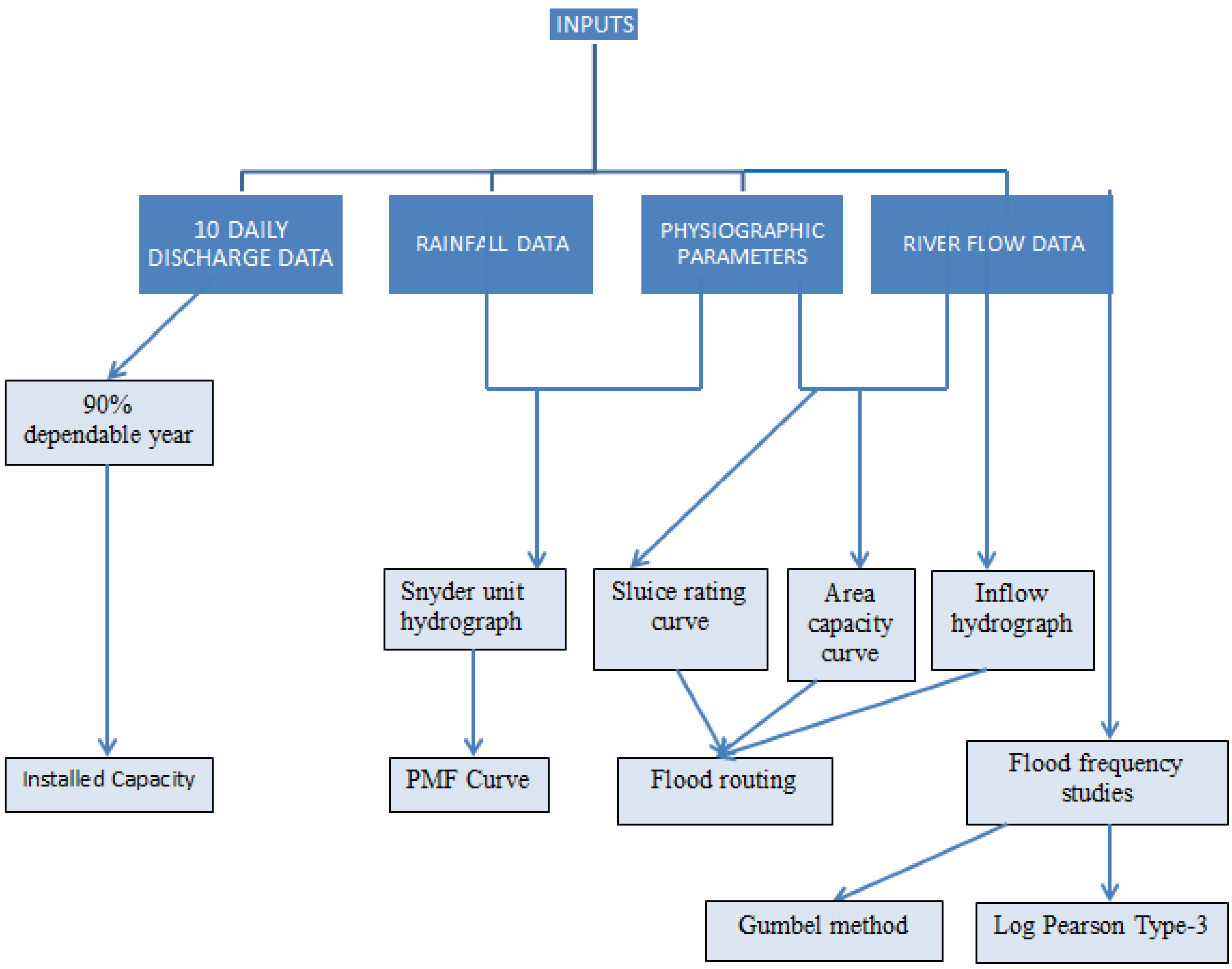


Fig 9 Methodology Flow Chart

The various types of input data that was required was the discharge data , the rainfall data , the physiographic parametres and the river flow data was taken from guage and discharge sites , which was provided upon request by the authorities at SJVN Ltd , Shimla . The 10 Daily dischage data was used to find the 90% dependable year which tells us about the dependability of the project and the revenue its capable of generating and hence the financial viability . the installed capacity was found out using the 90% dependable year , as the increase in energy generation of each year was plotted against the range of installed capacities . Both parametres together give us the viability of the project and are used to find the revenue generation .

The rainfall data is recorded by IMD and is shared by them with the organization , from this data coupled with the physiographic parametres like length of catchment area , the elevation at various levels , the area of catchment , the height of dam and sluice gates , all these were used as they were to be put in certain formulas which were essential in computing the project parametres. The snyder unit hydrograph was developed from the collective of rainfall and physiographic parametres . The ordinates of the unit hydrograph were derived using the formulas given by Snyder . these values were then smoothened out and more values were inserted in between using interpolation techniques and a final graph was then plotted which gave the value of the Probable Maximum Flood for which the project is to be designed .

The flood data was also used for the flood frequency studies which determine the worst flood that can occur in the river for a given return period as that flood will influence the structure so that value is required to be known for design purpose . The analysis was done using two methods : Gumbel Method and Log Pearson Type 3 Method . Then the physiographic parametres with the flood data were used to plot the three grapghs which act as inputs for flood routing table which is constructed to determine the amount of flood which can be passed at a given time and the opening it will require depending on how much oressure will act accirding to the amount of water .

CHAPTER 5 : POWER POTENTIAL STUDIES

5.1 Calculation of 90% Dependable year and installed capacity :

Weibull Method for 90% dependable year :

For water availability studies for a SHP the FDC is drawn for **90% dependable years.** The 90% and 75% dependable year is generally calculated by arranging data in descending order, the annual runoff of all the years for which observed or extrapolated / extended discharge data is available and using **Weilbuls' formula:**

$$P = \frac{m}{N_{+1}} \text{ x } 100$$

P is dependability percent, m is the rank of runoff of the desired dependability, N is the number of data. If P is 90% N = 19, m works out as x (19 1) 18 100 90 + = . Thus 90% dependable flow year will correspond to the runoff which is at rank 18 from the top. For working out the FDC for 90% dependable year, the 10-day discharge series of that year is considered.

5.2 Hydrological data

A sample of the data has been given below :

MONTHS	PERIOD	No. OF DAYS	1972-73	1973-74	1974-75	1975-76	1976-77	1977-78	1978-79	1979-80
JUN	I	10	481	786	343	708	844	392	820	267
	II	10	616	1610	521	1040	652	260	649	708
	III	10	820	1542	433	1186	463	716	1179	1722
JUL	I	10	932	1242	503	779	846	1362	1419	1256
	II	10	851	1292	992	1372	1228	1458	1171	1497
	III	11	951	1045	976	1134	1439	1119	1162	1201
AUG	I	10	865	924	960	1258	875	1484	1492	1097
	II	10	845	909	928	1366	732	878	1278	1053
	III	11	651	921	639	1019	587	656	820	801

Table 4 10 daily discharge data .

The data is arranged according to the hydrological year which is from June to May . Discharge readings are taken every hour during the day and mean of those values is noted .

The mean discharge values of ten days is taken, that is called the first **ten daily discharge value** of the month **.** Hence three ten daily discharges are obtained for each month . These values were taken from 1972 to 2014 .

5.3 Power Potential Studies

Power Potential studies are carried out for assessment of available Power Potential of a river/basin based on a set of inflows and available head conditions under various operating policies. These studies play an important role in the optimisation and design of new hydro facilities. They are used for examination of various configurations and their integration into existing networks. The studies are carried out for optimization of project parameters and for evaluation of Energy and Power benefits.

ROR Schemes with Pondage : In case of ROR schemes with pondage, the flow data is arranged hydrological year wise i.e. June to May etc. and Unrestricted Energy Benefits are worked out for all the years. Based on the above, 90% dependable year is selected. To determine the optimum installed capacity, a number of alternatives of installed capacities are considered and energy generation during the 90% dependable year is computed for each of the alternative installed capacity scenario based on average 10-daily inflows**.**

Installed capacity is selected after carrying out Incremental Analysis for the most attractive alternative. The unit size is selected by considering pattern of generation in various periods, transportation constraints, if any, and system considerations etc , load factor is also considered and studies for cost of alternative Thermal/ Gas and Hydro sometimes carried out in order to work out optimum installed capacity. Hourly operation of the pondage is carried out to work out

pondage requirements for peaking. The unit size is selected by considering, transportation constraints, if any, and system considerations etc.

5.4 Energy Generation[3]

Associated terms:

1.) **Availability**:- in relation to a project it means the capacity of a project , including the generating units , to generate power on availability of water . the annual availabilities of a project shall be determined as per the following formula :-
Percentage annual availability =
(H1U1+H2U2+…………….HnUn)*100/(U1+U2+…………+Un)

Where U1 ,U2 . . . . Un are the capacities in mega watts of different units and H1, H2 . . .
. . . ..Hn are the hours for which the respective units were available for operation during the year.
2.) **Design energy:-** it is the quantum of energy which could be generated in a 90 percent dependable year with 95 percent availability of installed capacity of the station.
3.) **90% and 50% dependable flows:-** these flows are worked out by considering the annual inflows for the hydrological year from june to may for a given number of years and by using Weibull's plotting formula i.e.

$$p=n/(m+1)$$

where p, is the probability of exceedence , n is the rank of occurrence ,and m is the total number of occurrences. If the total energy generation in the years for which hydrological data is available (say N years) is arranged in descending order , the $(N+1)*0.9^{th}$ year would represent the 90 per cent dependable year. The 90 per cent dependable year is the year in which the annual energy generation has the probability of being equal to or more than 90 per cent of the of the expected period of operation of the scheme.

***selection of 90 per cent dependable year**

Planning of HE Project is carried out based on 90 per cent dependability criteria. For determination of 90% dependable year, the total energy generation in all the years for which hydrological data is available (say N year) is arranged in descending order and the (N+1) x 0.9 th year would represent the 90 per cent dependable year. The 90 per cent dependable year is thus, termed as the year in which the annual generation has the probability of being equal to or exceed 90 per cent of the time on annual basis during the expected period of operation of the scheme. For example, if inflow data is available for a period of 20 years (N=20), then,

90% Dependable year = (20+1)*0.9= 18.9 = 19th yr

Power generation is then found out using the formula ;

$$P=9.81*\eta*Q*H/1000 \qquad \text{(in MW)}$$

Here , η= combined efficiency , Q= discharge in cumecs , H= net head available

4.) **Installed capacity**:- it is the summation of name plate capacity of the generating units in the station or the capacity as decided in consultation with the authority from time to time considering the uprating and derating as may be applicable.

Primary energy: it is the quantum of energy generated upto design energy on an annual basis in the station.

5.) **Project** : it includes the complete hydro power generating facility covering all components such as dam , intake, water conducter systems , power station , generating units of the scheme as apportioned to power generation and as decided by authority

6.) **Secondary energy** : it is the quantum of energy generated in excess of the design energy on an annual basis in the station.

7.) **Station :** it is a hydro power generating station having an installation of one or more hydro generating units including reversible units

According to the calculations that were performed the 90% dependable year was obtained and have been attached in the annexure .The discharges obtained in the 39th year which is actually the 2001-2002 period are the discharges that are to be used in the computation of power generated . This is the 90% dependable year . Formula for power generated :

$$P=9.81*\eta*Q*H/1000 \qquad \text{(in MW)}$$

5.5 CEA Guidelines for determination of installed capacity[2]

For selection of installed capacity, benefits from the project with different possible installed capacities are evaluated. Optimum installed capacity is selected after carrying out Incremental analysis for the most attractive alternative and also considering the system Plant Load Factor

(PLF).

FOR ROR SCHEMES : The first step is to compute 90% dependable year :

- Obtain 10-daily hydrological inflow series in m3 /sec for all hydrological years, year-wise.
- Calculate unrestricted energy generation in MUs.
- Arrange unrestricted annual energy generation in descending order. 0.9(n+1)th year is the 90% dependable year, where n is the number of years for which hydrological inflows data is available.

Fixing of the installed capacity is known as installed capacity optimization , the results of which are shown as below :

SNo.	INST. CAP. (MW)	ANNUAL ENERGY GEN (GWh)	KWh/KW	d(KWh/dKW)
1	180	760	4222.22	
2	190	779	4100.00	1900
3	200	798	3990.00	1900
4	210	817	3890.48	1900
5	220	836	3800.00	1900
6	230	854	3713.04	1800
7	240	865	3604.17	1100
8	250	865	3460.00	0

Table 5 Installed capacity Optimization

The graph obtained was as under :

Sharp fall in the graph is observed from **219 MW hence this is the installed capacity .**

Col(2) The installed capacity is taken in range of of increments of 5-10% greater and less than what is specified on the machines .
Col(3) For these installed capacities the annual power generation is worked by the formulas for power generation .
Col(4) the annual energy upon installed capacity in Kwh/Kw is computed .
Col(5) then the incremental change is calculated for i.e for increasing installed capacity what is the corresponding change in power generation .
A graph of Col(5) v/s Col(2) gives us a curve in which we have to notice the point of sharp Decline which will give us the installed capacity .

This capacity obtained in the curve is to be taken for economic viability calculations as this is the level at which various units at the power house will operate when the project is in operation .

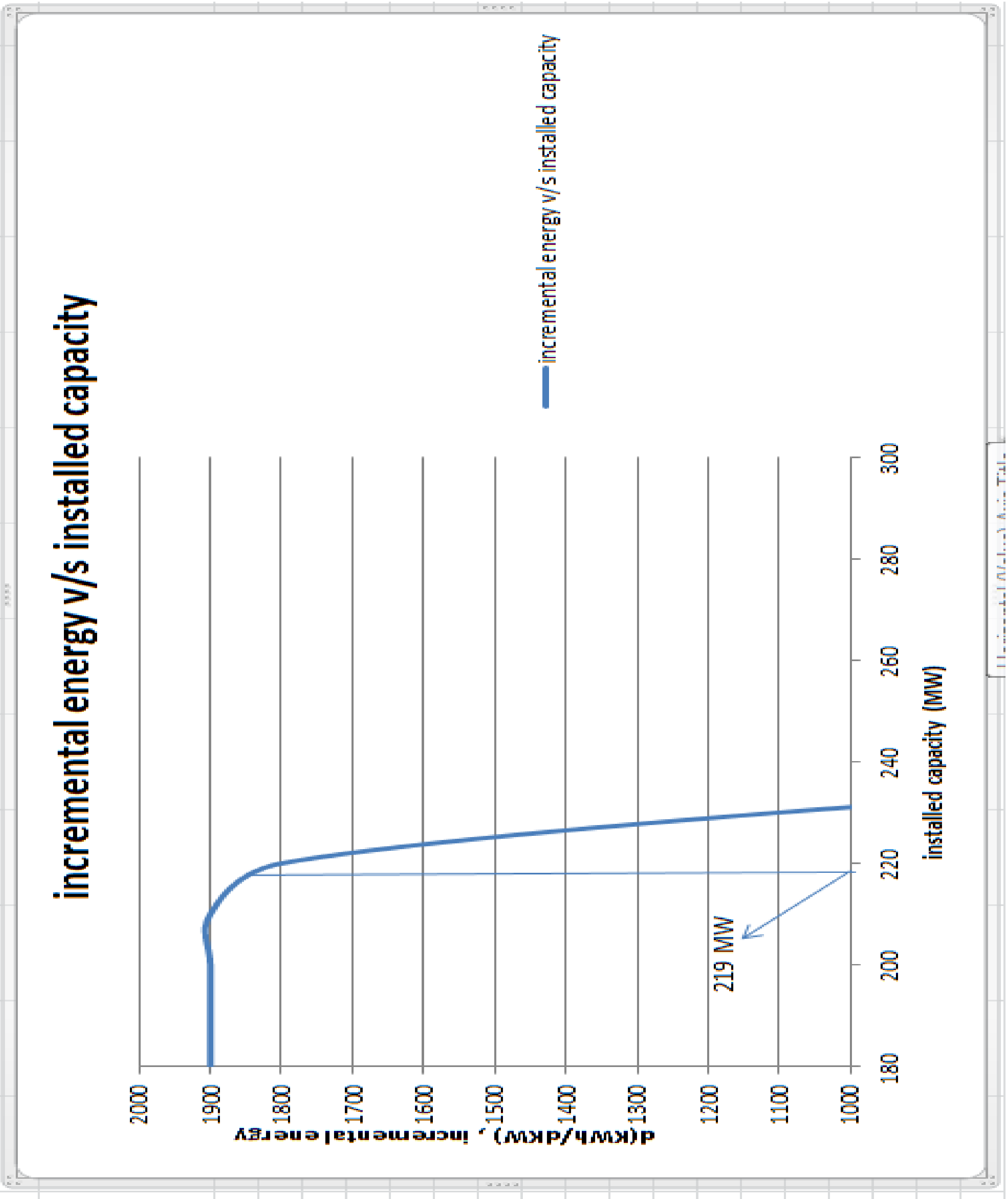


Fig. 10 Iinstalled Capacity Optimization

CHAPTER 6 : FLOOD FREQUENCY ANALUSIS AND PMF

6.1 Return Period Floods[5]

Flood frequency analysis has been carried out to determine the return period floods, and this approach has been used for other projects in the Satluj River including Nathpa-Jhakri, Karcham Wangtoo and Rampur. Empirical formulae such as Dickens, Ryves and Inglis do not take into account the components of storm rainfall and other physiographical and hydrological factors which vary from catchment to catchment and hence have limited application.

The flood frequency analysis will also take into account the floods created by landslides and in this respect it is understood that whilst the maximum flow in the data series, which occurred in 2000, was due to a major rainfall event there is circumstantial evidence that it might have been due to the failure of a natural dam in Tibet. Ideally the frequency analysis requires a long term annual instantaneous peak flow series. In this case the most appropriate data series available is limited to 43 years of peak annual flows for Rampur.

These data, after pro-rata adjustment for the marginally greater catchment area and further 10% increase to account for the readings being taken at a predefined time of the day rather than at peak flood levels, have been analyzed using a Gumbel Distribution and the results of these analyses are shown graphically later in the report .

For a hydraulic structure planned within the river (like a dam or a barrage) or on an adjoining area (like flood control embankments), due consideration should be given to the design of the structure so as to prevent it from collapsing and causing further damage by the force of water released from behind the structure. Hence an estimate of extreme flood flow is required for the design of hydraulic structures, though the magnitude of such flood may be estimated in accordance with the importance of the structure.

6.2Gumbel method for flood frequency analysis[6]

Various steps involved in frequency analysis by the Gumbel method are as follows:

- List and arrange annual floods Q in descending order of magnitude.
- Assign rank ‘m’, m = 1 for highest value and so on.
- Values of return period together with respective flood magnitude give plotting positions.
- Now calculate mean Q’; the deviation Q’-Q; $(Q'-Q)^2$ and hence standard deviation S.

The detailed calculations have been attached in the annexure .

The results were obtained as :

Mean discharge Q' = 1945.957198 Cumecs

Standard deviation Sigma = Sum((Q'-Q)^2/(n-1))^0.5

= 1203.436969

Coeff. Of variation Cv = Sigma/Q'

= 0.6184

Next the return period of the flood value is taken as T . then log of T/T-1 gives the factor XT shown in the table . from this value of X_T the reduced variate y= -0.834042-2.302585X_T for Gumbels method is calculated .

The value of maximum flood is found out by , Q=Q'(1+CV(y-yn)/Sn) . The values of yn and Sn are taken Gumbels table of expected means and standard deviations of reduced extremes .

The calculations have been attached in the annexure .

Associated terms :

Return period : This is the recurrence interval of a flood . eg if the return period is 1000 years then it means that a flood of certain magnitude will come once in 1000 years .

Probability of Exceedence : Probability that an event selected at random will exceed a specified magnitude .

Maximum flood : The value of discharge in cumecs that the river might carry due to flooding in a certain return period .

Estimates of flood frequency quantities are important in planning and design of water reource structures, Hence there is a need to seek for the most appropriate design estimator that would meet both safety and economic considerations of such structures. Flood frequency analysis is a tool used to estimate the frequencies of likely an occurrence of future floods

The result of this analysis was obtained as a graph which can be viewed in the next page . It is evident from the graph that as the return period goes on increasing the value of flood or discharge that the river might have goes on increasing . For a return period of 10000 years the maximum flooding that can occur in the river is 10973 cumecs .

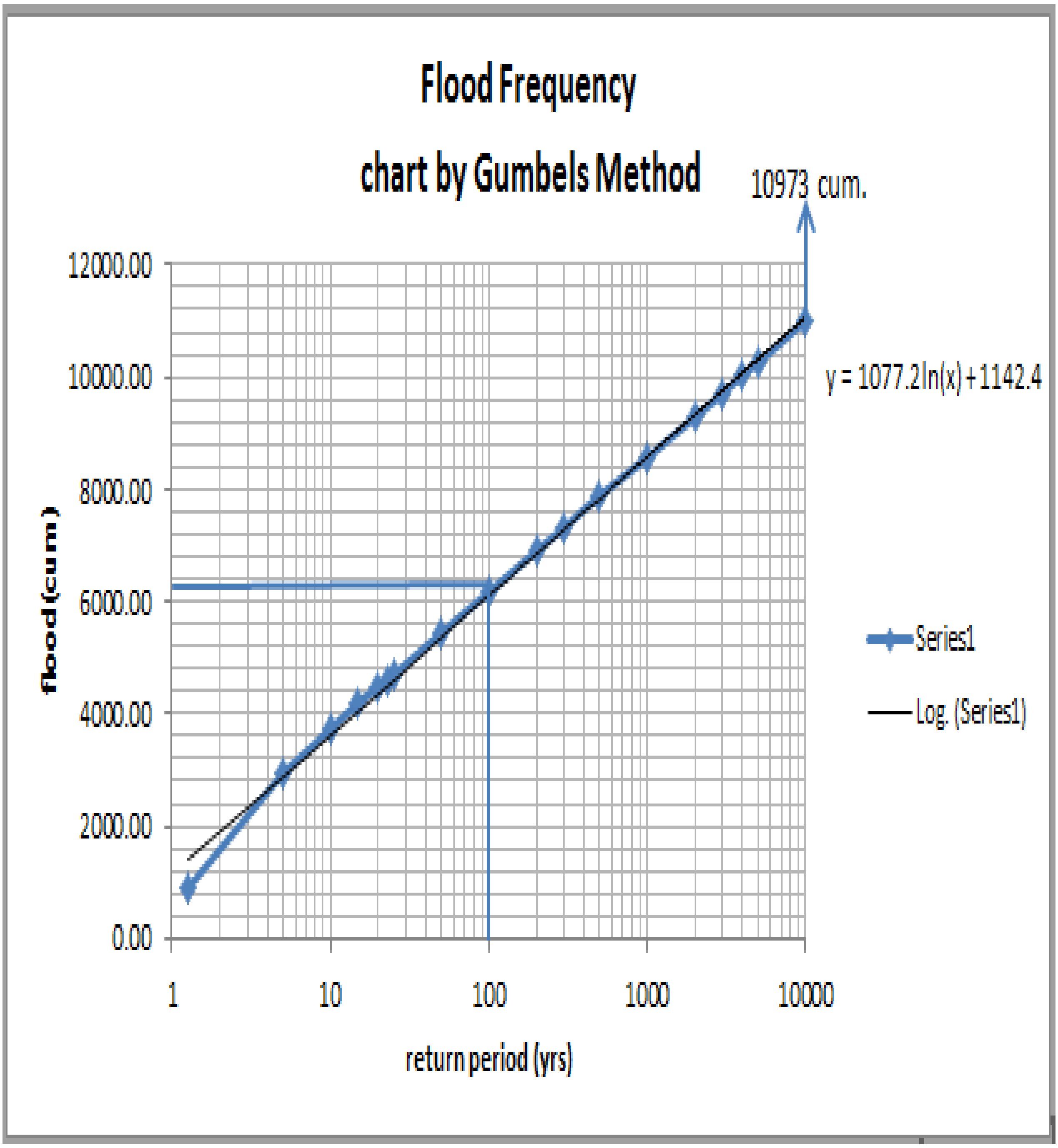


Fig 11. Flood Frequency by Gumbel method

6.3 Log Pearson Type -3 for flood frequency analysis[9]

Rank (m)	Year	Max. Flood Discharge	Max. Flood Discharge,in descending order Q	Log Q	Log Q-Avg.(LogQ)^2	Log Q-Avg.(LogQ)^3	Return Period T_r=(n+1)/m	Exceedance Probability (1/T_r)
1	1963	1657.1	3458.0	3.54	0.0800	0.0226	44.00	0.0227
2	1964	1547.8	3266.1	3.51	0.0666	0.0172	22.00	0.0455
3	1965	3266.1	2715.6	3.43	0.0317	0.0056	14.67	0.0682
4	1966	2131.0	2517.4	3.40	0.0210	0.0030	11.00	0.0909
5	1967	1596.6	2326.8	3.37	0.0123	0.0014	8.80	0.1136
6	1968	1692.1	2308.4	3.36	0.0115	0.0012	7.33	0.1364

Table 6 Log Pearson Type-3 calculations

n is the number of years of record and m is the rank obtained by arranging the annual flood series in descending order of magnitude with the maximum being assigned the rank of 1. In carrying out the flood frequency analysis using the log-Pearson Type III distribution, the following steps were adopted:

(i) The annual flood series were assembled
(ii) The logarithms of the annual flood series were calculated as yi = log Xi
(iii) The mean *y*, the standard deviation *y* and skew coefficient Cs of the logarithm yi were calculated. (iv) The logarithms of the flood discharge i.e. log Qi for each of the several chosen probability level Pj were calculated

The following formulas were used and these results were obtained :

Variance = Sum(logQ- avg(logQ)^2/(n-1) =	0.014
Standard deviation =sigma logQ = √ Variance	0.120
Shew Coefficient = sum(log Q-avglogQ)^3/(n-1)(n-2)(sigmalogQ)^3 =	-0.214

Table 7 Resulting Values

Now using the tables relating the value of skewness coefficient and probability of occurrence the value of deviate K was determined .:

Probability (p)	Return Period =1/p (Years)	Deviate K	Flood = Avg.LogQ +Ksigma(logQ)	Flood = 10^Column 4
0.0001	10000	3.27	3.65	4444.69
0.0005	2000	2.99	3.61	4115.01
0.001	1000	2.83	3.59	3931.72
0.002	500	2.65	3.57	3747.68
0.005	200	2.40	3.54	3495.76
0.01	100	2.17	3.52	3278.21
0.02	50	1.94	3.49	3075.41
0.04	25	1.67	3.46	2860.98
0.05	20	1.58	3.45	2789.02
0.1	10	1.26	3.41	2549.40
0.2	5	0.85	3.36	2278.72

Table 8 Resulting flood values

The graph of flood frequency was plotted putting column two in the X-axis and column four in the Y-axis :

6.4 Findings from graph :

- The Log Pearson Type-3 analysis gave the volume of flood as 4444.68 cumecs of water .
- This is the amount of water for which the dam , the sluice gate openings should be designed for .
- Estimating return period floods is important as they are needed to determine the maximum discharge and maximum potential and estimate cost and size of structure.
- Estimates of flood frequency quantities are important in planning and design of water

  reource structures, Hence there is a need to seek for the most appropriate design estimator that would meet both safety and economic considerations of such structures. Flood frequency analysis is a tool used to estimate the frequencies of likely an occurrence of future floods
- It is evident from the graph that as the return period goes on increasing the value of flood or discharge that the river might have goes on increasing

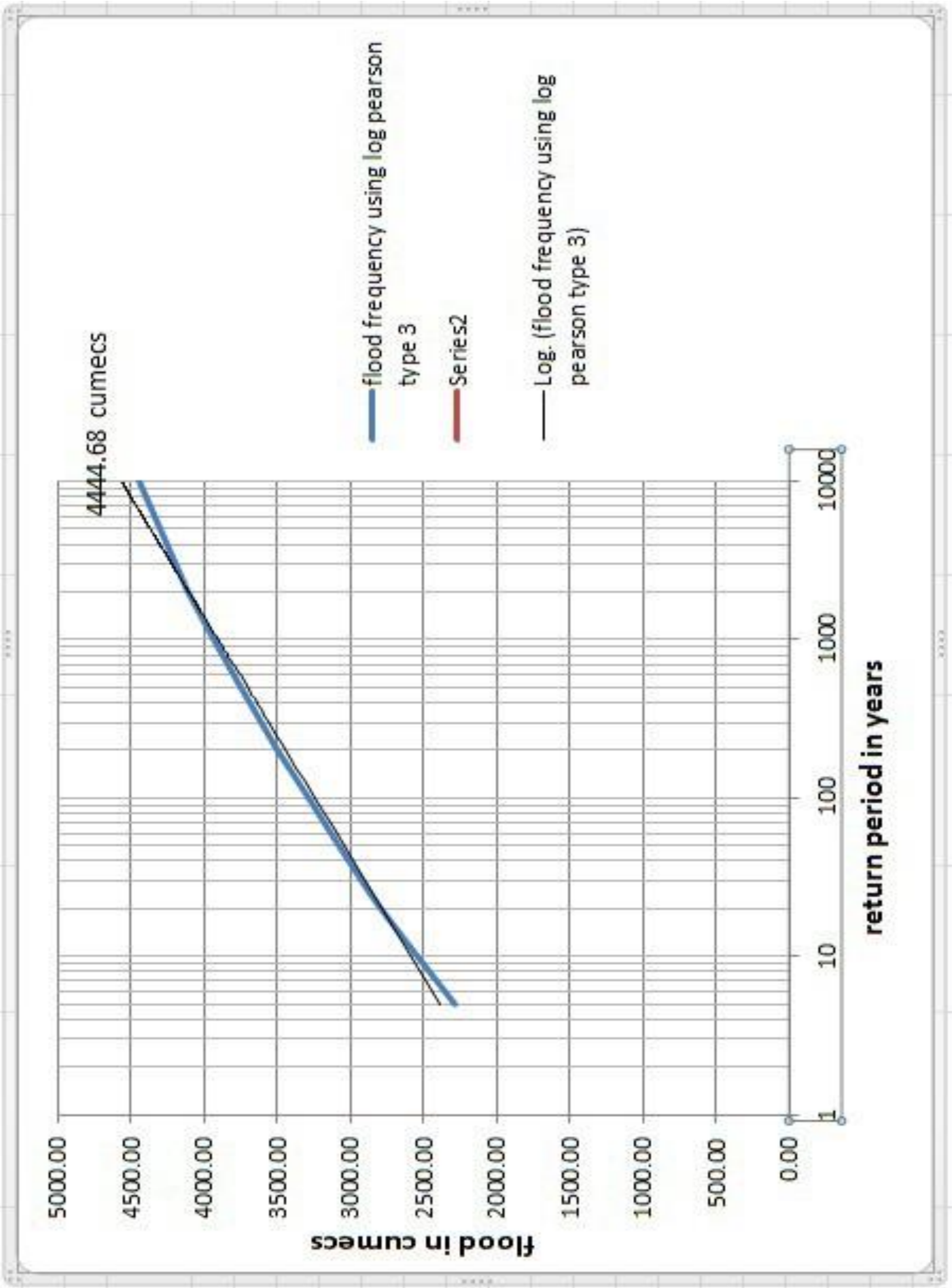


Fig. 12 Flood frequency by log pearson type 3 distribution

6.5 **Unit Hydrograph (Snyders Unit Hydrograph)**[11]

A unit hydrograph is defined as the hydrograph of runoff produced by excess rainfall of 1cm occurring uniformly over the entire drainage basin at a uniform rate over the entire specified duration. Once the UH is established for a basin, hydrographs resulting from any amounts of runoff may be computed from the UH.

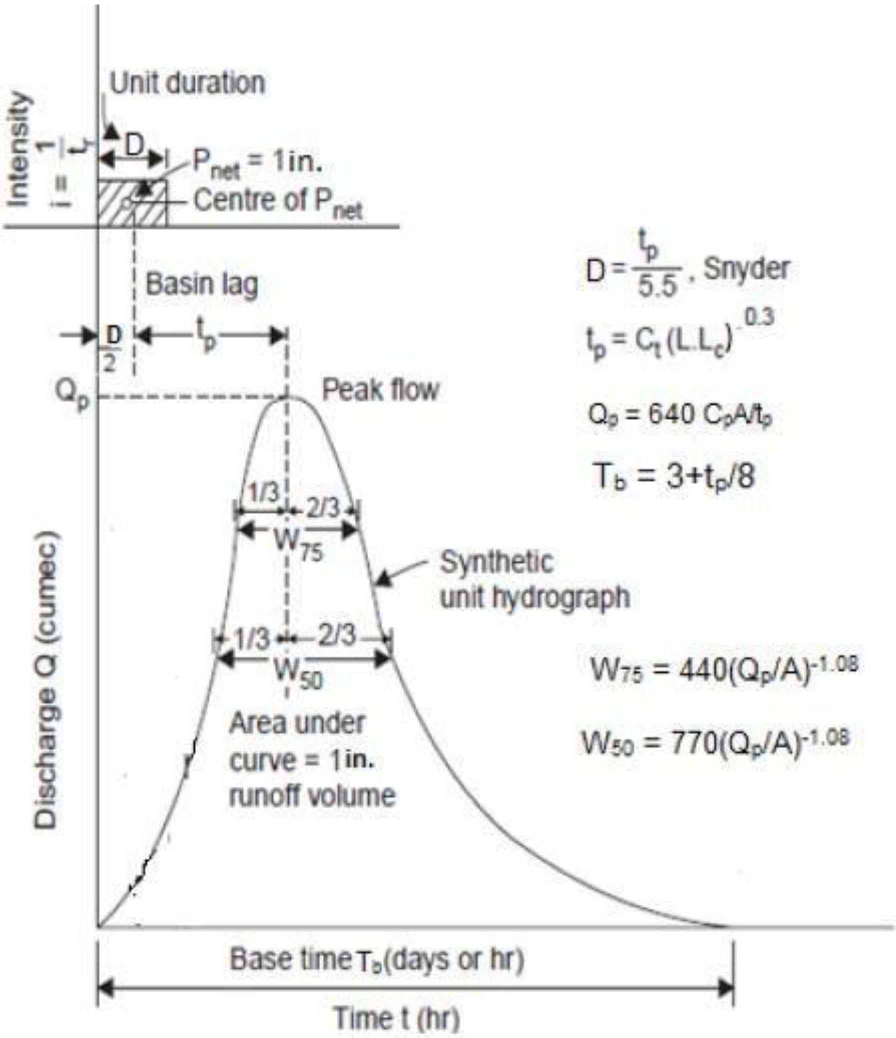


Fig. 13 Snyder Unit Hydrograph parametres

6.6 Process involved :

First step is the computation of equivalent slope :

The equivalent slope and rain fed catchment area has been determined by digital elevation model .

Equivalent Slope $\mathbf{S = \sum L_i \ (D_{i-1}) + D_i)/L^2}$ $\quad$ **=9.52 m/km**

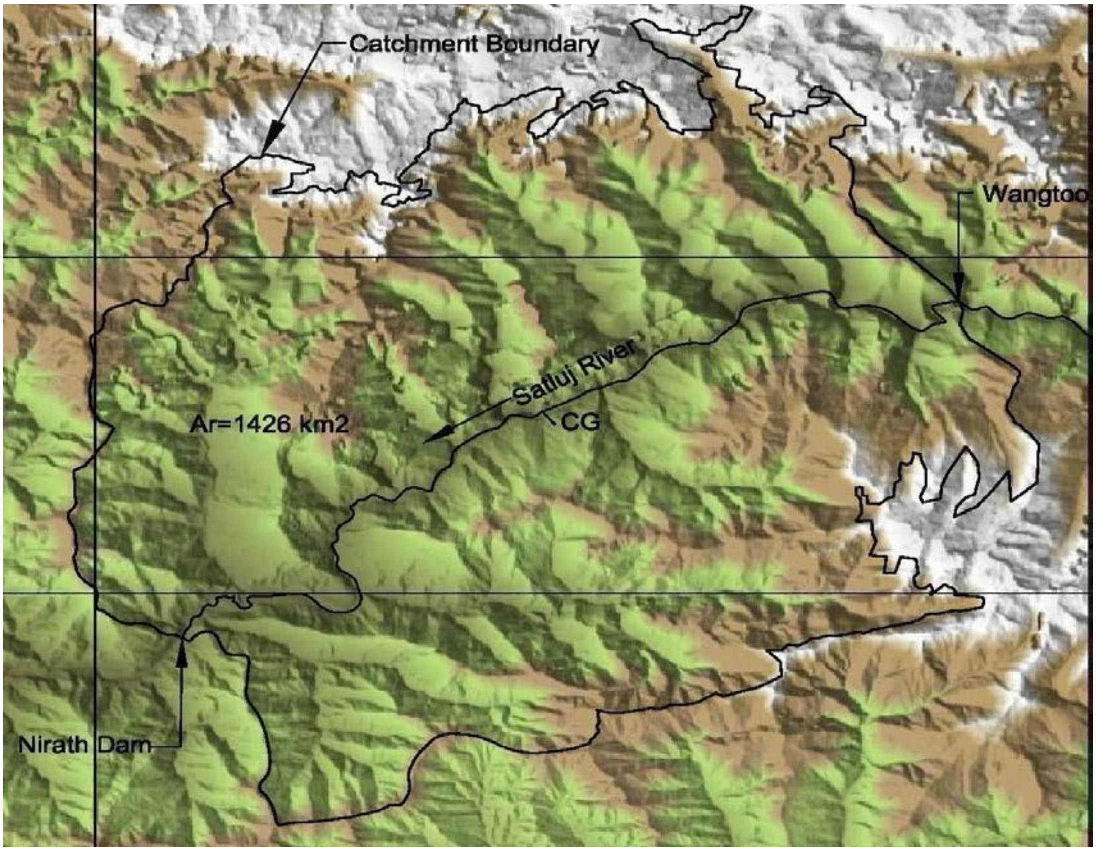


Fig 14 Total rain fed catchment

Parametres given :

Parameters	Value
A	51600 km^2
Ar	1426 km^2
L	60.75 km
Lc	32.70 km
S	9.52 m/km

Table 9 Values for Snyder unit hydrograph

The parameters computed were :

- A = Catchment are in Km^2
- As = Snow fed Catchment area in Km^2

- Ar = Rain fed Catchment area in Km^2
- L = Length of longest stream along the river course in km.
- Lc = Length of longest main stream from a point opposite to centroid of the
- catchment area to point of study.
- S = Equivalent stream slope in m/km.
- tr = Unit duration in hrs.
- tp = Time from the centre of effective rain fall duration of the unit hydrograph (U.G.) peak (hrs).

- tm = Time from start of rise to the peak of U.G. (hrs).
- TB = Base width of U.G. (hrs).
- qp = Peak discharge in $m^3/s/km^2$
- Qp = Peak discharge of unit hydrograph (m^3/s)
- W50 = Width of U.G. measured at 50% of Peak Discharge Ordinate (hrs)
- W75 = Width of U.G. measured at 75% of Peak Discharge Ordinate (hrs)
- WR50 = Width of the rising limb of U.G. measured at 50% of Peak Discharge Ordinate (hrs).
- WR75 = Width of the rising limb of U.G. measured at 75% of Peak Dischare ordinate (hr)

Sr. No.	Parameters	Formula	Value	Unit
1	tp =	2.498*(L*Lc/S)^0.156	5.74	hrs
2	qp =	1.048*(tp)^-0.178	0.75	cumecs
3	W50 =	1.954*(L*Lc/S)^0.099	3.3	hrs
4	W75 =	0.972*(L*Lc/S)^0.124	1.88	hrs
5	WR50 =	0.189*(W50)^1.769	1.58	hrs
6	WR75 =	0.419*(W75)^1.246	0.92	hrs
7	TB =	7.845*tp)^0.453	17.32	hrs
8	Qp =	qp*Ar	1069.50	Cumecs
9	Tm =	tp+0.5	6.25	Hrs
10	∑Qi=	d*Ar/(tr*0.36)	3961.11	Cumecs

Table 10 Calculated values for unit hydrograph

Using these values the ordinates of the unit hydrograph were determined . the first hydrograph plotted was :

Time	UG coordinates (cumecs)
0	0
4.7	534.75
5.3	802.13
6.2	1069.50
7.2	802.13
8	534.75
17	0

Table 11. ordinates of unit hydrograph

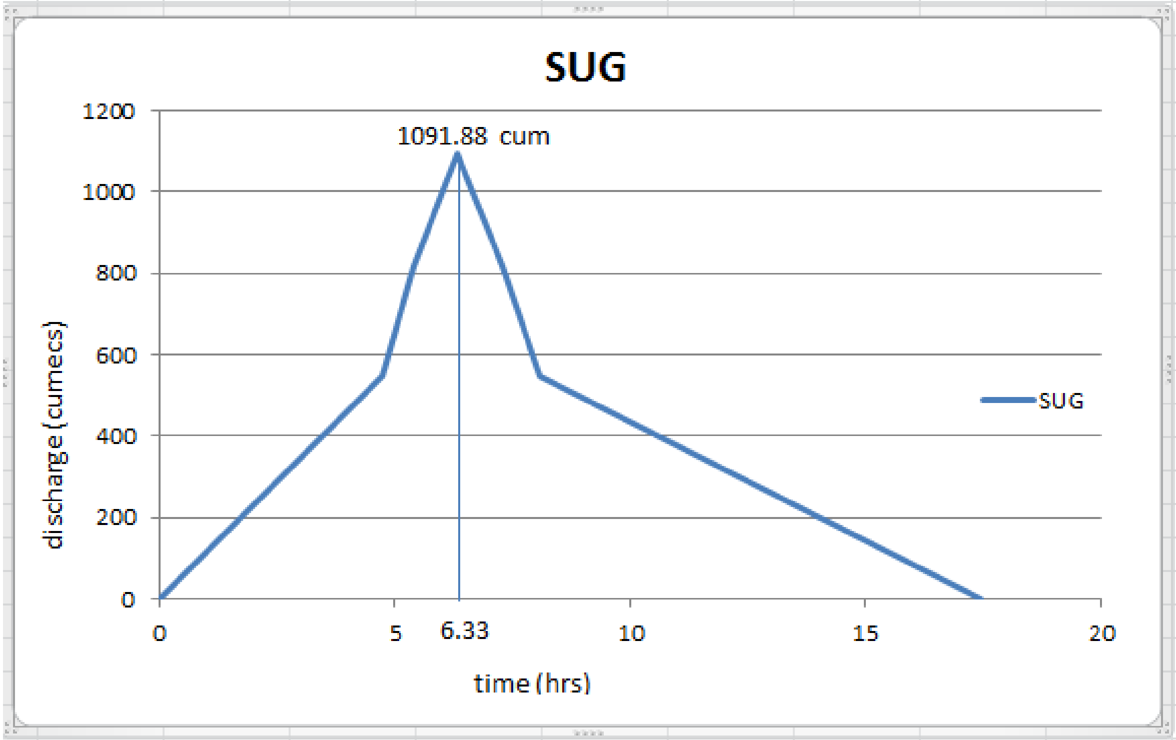


Fig. 15 Snyder unit hydrograph

Next the coordinates were smoothened out and values were inserted in between through trial and error method :

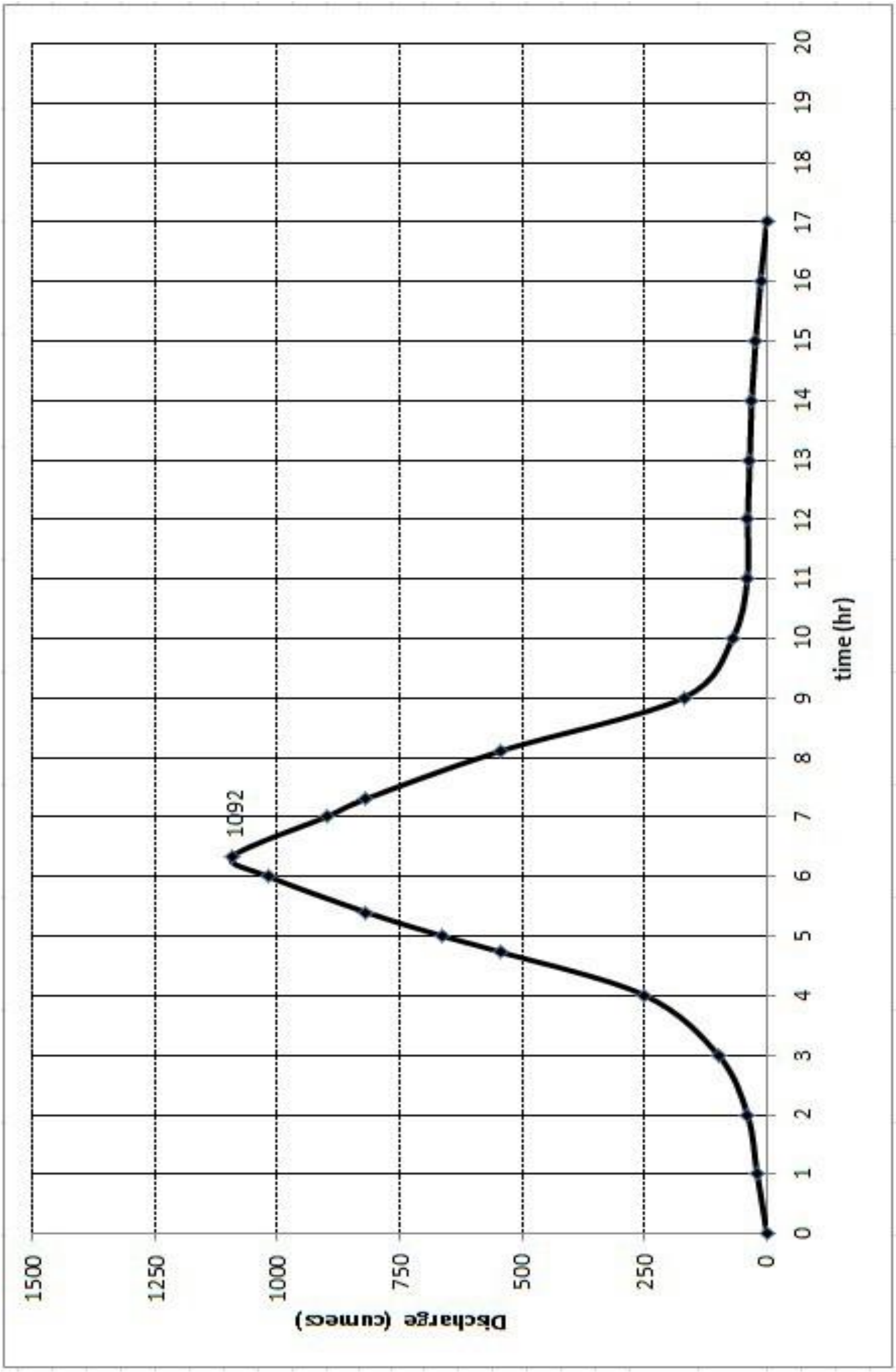


Fig. 16 Adopted final unit hydrograph

Now , convolution of unit hydrograph with worst possible catchment storm is done and base flow is added to it to obtain the final PMF graph from which the value of probable maximum flood will be obtained .

Effective rainfall values obtained above are applied to 1 Hr Unit Hydrograph ordinates. The effective rainfall ordinates are arranged against the ordinate of UG in such a way that maximum value of rainfall is placed against the peak value of the UG, the next lower with the next lower value of UG and so on . As per CWC “Zone-7 Flood Estimation Report” baseflow is **0.05 cumecs/km^2**. Rain catchment area between Wangtu and Nirath is 1554 km^2, therefore the base flow above Wangtu comes out to be **77.7 cumecs**.

The adopted PMF graph is as follows :

6.7 Findings from curve :

- The probable maximum flood (PMF) or the standard project flood (SPF) is estimated using the hydro-meteorological approach.
- For the PMF calculations the worst possible maximum storm (PMS) pattern is obtained from the CWC & IMD .
- This is then applied to the unit hydrograph of the catchment to obtain the PMF.

  The PMF obtained is 9260 cumecs for the worst possible storm

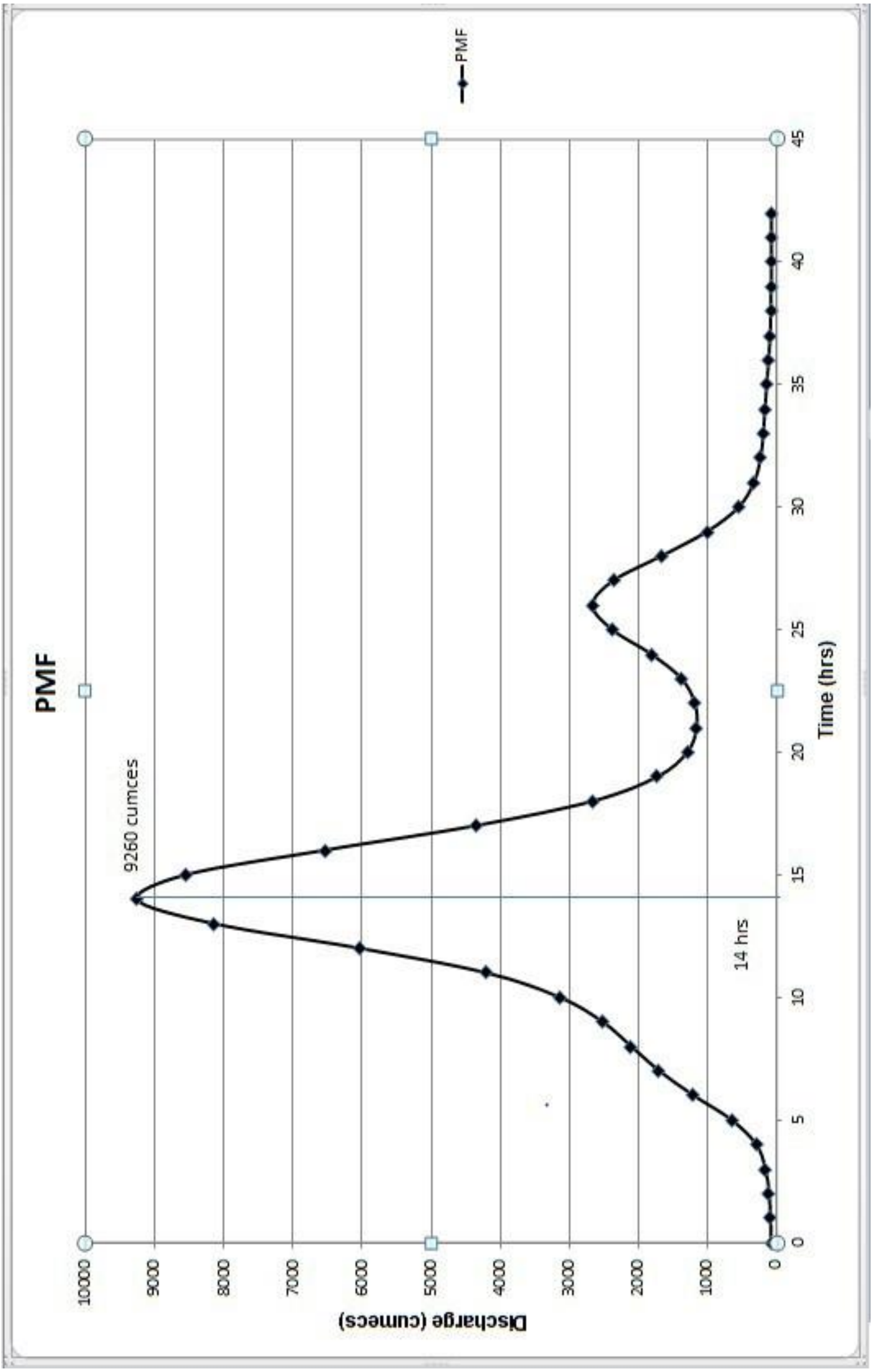


Fig. 17. Adopted PMF curve .

CHAPTER 7 : FLOOD ROUTING

7.1 Meaning of flood routing[10]

“Flood routing is a technique of determining the flood hydrograph at a section of a river by utilizing the data of flood 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. The main concept of flood routing is that if there are hydrographical specifications in a point of a river, how one might estimate the hydrograph in another point in the downstream. This subject is important specially where agricultural lands located in downstream.

7.2 Inputs required[18]

1.) **Inflow hydrograph :** this is a discharge versus time curve from which values are taken up to determine the amount of inflow coming into the reservoir . The values in the hydrograph are taken from the guage and discharge sites data and the amount of flood coming in at a unit time is note and is plotted . the inflow hydrograph used for the computations has been attached in the next page. The table of values is as below :

time (hrs)	discharge (cumces)
0	1702
1	2105
2	2521
3	3141
4	4207
5	6038
6	8141
7	**9260**
8	8548
9	6541
10	4358
11	2664
12	1725

Table 12. Inflow hydrograph

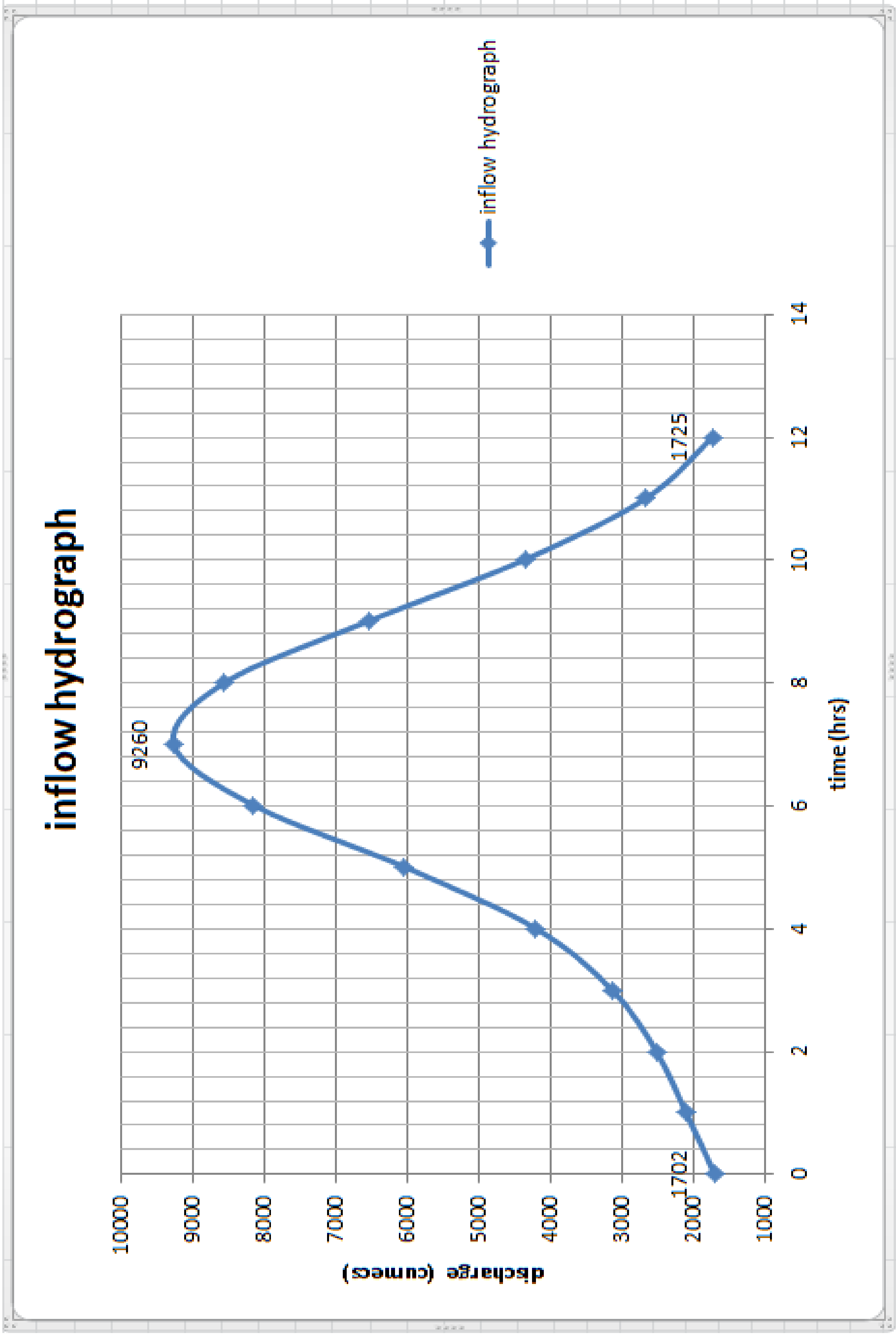


Figure 18. Plot of inflow hydrograph

2.) Reservoir storage versus elevation curve (the area capacity table)

This curve tells us the volume of water stored in the reservoir corresponding to a particular elevation . The cumulative capacity of the reservoir is shown . this is also a given data and just the curve needs to be plotted while for some values of elevation the capacity of the reservoir needs to be worked out using interpolation techniques. The a sample of the table obtained to be used for flood routing is as follows :

S.No.	Contour Elevation (m)	Cumm. Capacity (ham)
1	820	0.000
2	833	0.44
3	825	0.876
4	833	0.98
5	830	1.79
6	833	3.19
7	835	4.13
8	840	6.22
9	841	7.04
10	848	11.96
11	841	7.04
12	845	10.00
13	848	12.19
14	848	12.71
15	848	12.54

Table 13 area capacity curve table

The area capacity curve is as follows :
The curve gives us the amount of water that rises with the increase in elevation of the reservoir .the values on the curve will act as inputs for the flood routing table .

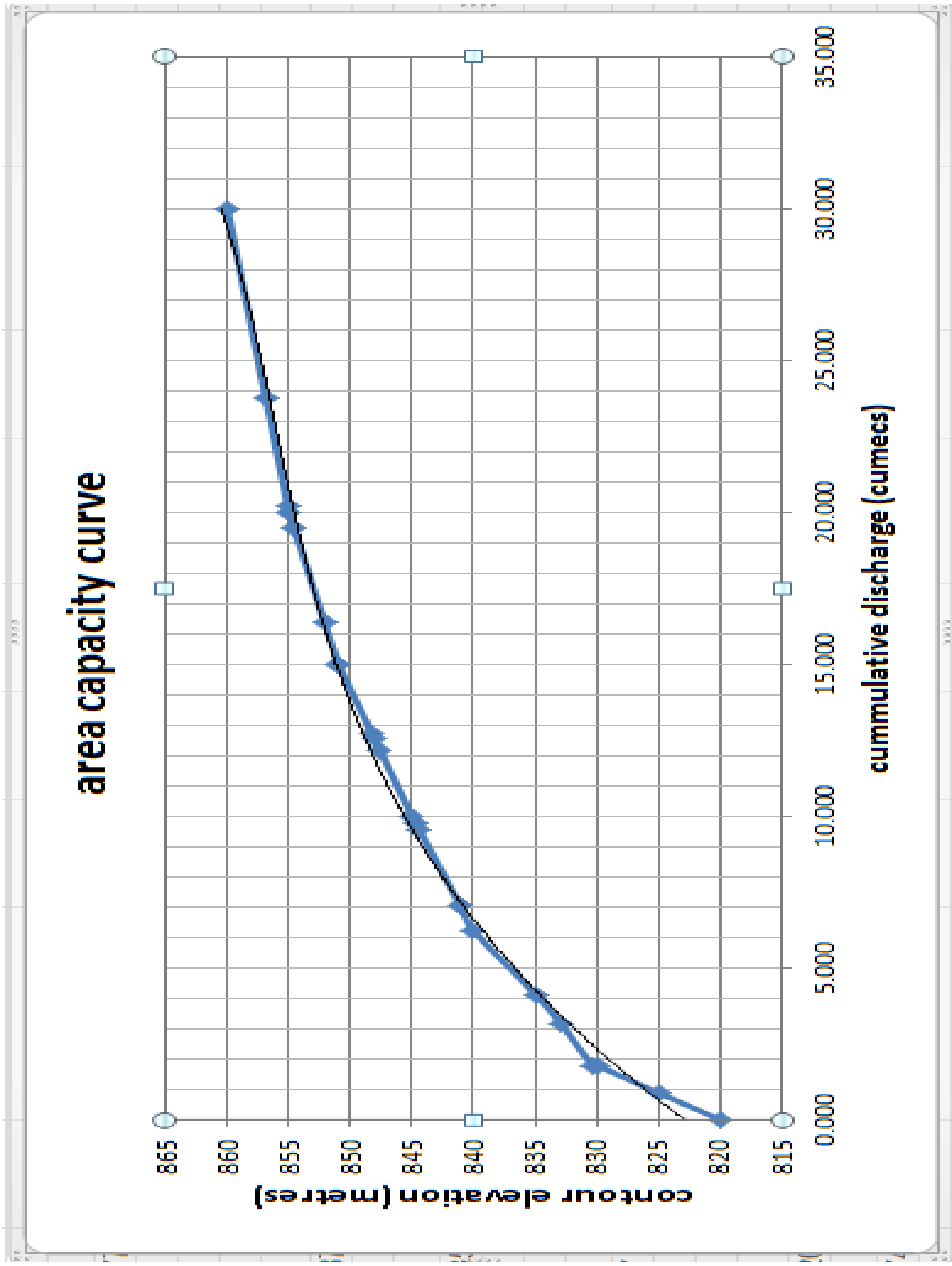


Figure 19. Area Capacity Curve

3.) Sluice rating curve (outflow curve)

Here the discharge capacity of the sluices is worked out according to the head of water available above the sluice crest or the centreline of sluice according to available head. Calculations are started from the sluice crest and then go up ultimately to the FRL (full reservoir level) and the flood that can be passed through is worked out using various formulae . the inputs used for the formulae are :

Crest El. Of sluice =	822	m
Full reservoir level =	857	m
Sluice width =	8.5	m
Sluice height =	15	m
El. Of C/L of sluice =	829.5	m

Table 14 Given parametres

Discharge for weir flow condition has been taken as per equation

Q= 2/3 Cb(2g)^0.5H^3/2

value of coeff. Cb varies from 0.57 to 0.63 & value of Cb has been taken as 0.62 .

As per equation : **Q= Cd A(2gH)^0.5** ,

where Cd is coeff. Of discharge for pressurised flow, value taken as 0.85 , A =area of sluice opening & H= head upto Centre line of sluice opening

A sample of the table for sluice rating curve is as shown below :

S. No.	Reservoir Elevation	Discharge Through Sluice Opening	Remarks
	(m)	(Cumecs)	
1.	820	0	
2.	822	44.02	Weir Flow
3.	824	124.50	Weir Flow
4	826	228.72	Weir Flow
5	828	352.13	Weir Flow
6	830	492.12	Weir Flow
7	832	646.91	Weir Flow
8	834	815.19	Weir Flow
9	836	900.00	Transition Stage
10	838	1100.00	Transition Stage
11	840	1300.00	Transition Stage
12	842	1500.00	Transition Stage
13	844	1650.00	Transition Stage
14	846	1850.00	Transition Stage
15	848	2064.74	Pressurised Flow

Table 15.. Calculations for sluice rating curve

Now the sluice rating curve or outflow curve is drawn according to these values . the values of the transition stage are obtained from graph plotted using the weir flow and pressurized phases . The graph is as shown below

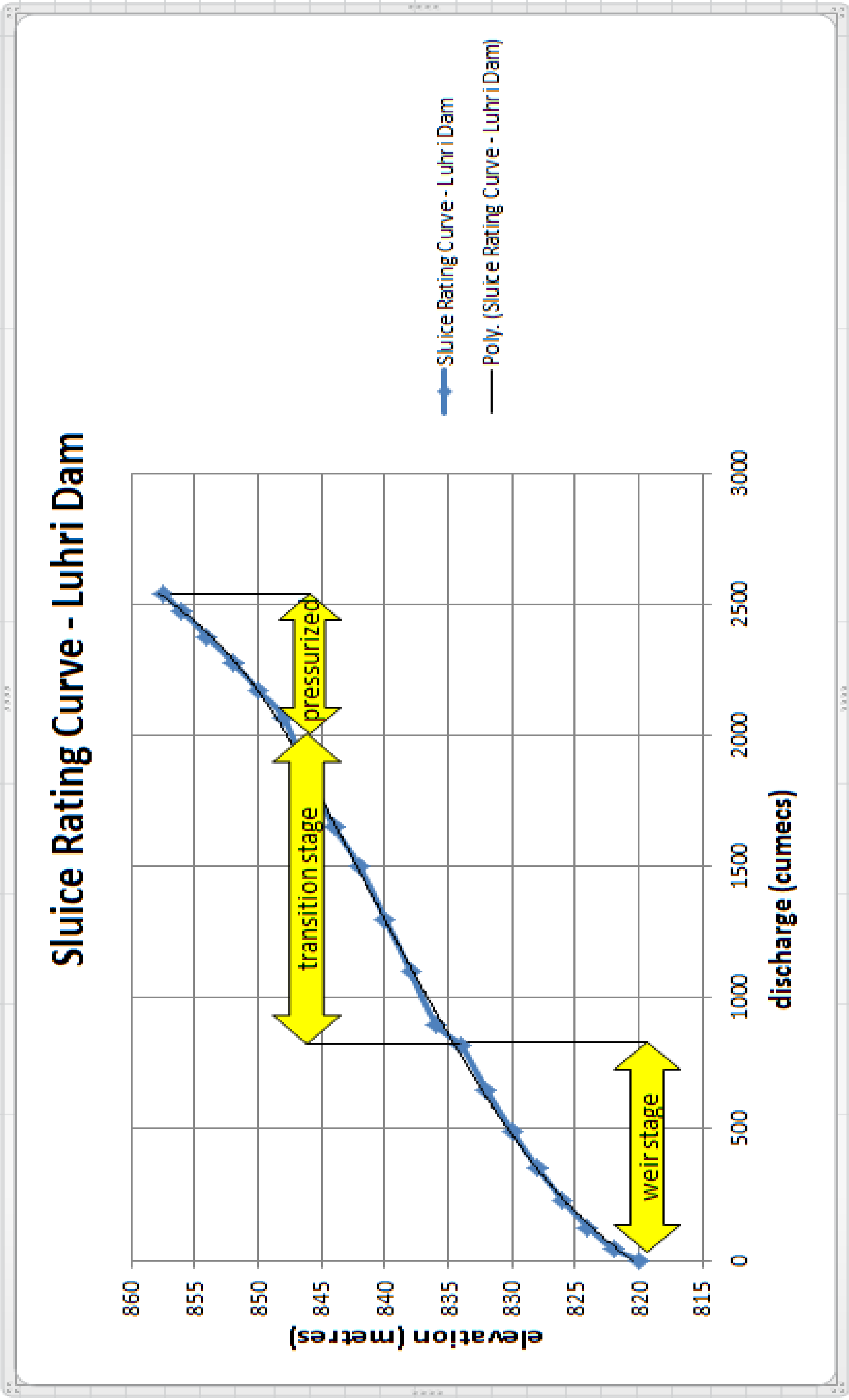


Fig. 20 Sluice Rating Curve

7.3 Steps involved[18] :

1.) First the above three inputs with their graphs are estimated.
2.) Next a table is prepared using the values from all three inputs . The first column will show time in hours . the unit time taken is according to the unit time of the inflow hydrograph . (one hour units)
3.) Now the inflow values is taken from inflow hydrograph for first two hours , the average of the two is taken ahead for the average rate of flow for the third column .
4.) The average rate of inflow is then converted into million cubic metre from cumecs .
5.) In the next column a trial storage elevation is assumed . the trial elevation so assumed lies a bit below the MDDL (minimum draw down level) of the reservoir .
6.) Corresponding to this level on outflow from the sluice rating curve was estimated which is then written there .
7.) The outflow is then converted into million cubic metre.
8.) The change in storage ΔS is calculated . **(ΔS = Qinflow - Qoutflow)**
9.) If the change in storage is negative then the reservoir is depleting hence an elevation lower than the previous trial elevation is taken otherwise if the change is positive then a higher elevation is taken .
10.) The change is added with sign to the previous storage as shown by area capacity curve .
11.) The level of reservoir at the end of the time interval Δt should be same as calculated from area capacity table then the calculations are correct .
12.) The same above steps are to be continued for the entire inflow hydrograph to determine how a flood will be passed , to keep the reservoir elevation same how many sluices will have to be opened , depending on the change in storage .

The calculations have been attached in the annexure .

7.4 Significance of analysis[21] :

The result of the above analysis is a table which has been attached in the annexure . the importance of this analysis is :

1.) To finalize the sequence of operation of sluice gates for passing the peak flood value of the unit hydrograph .
2.) Determining the initial level of reservoir behind Luhri dam to be kept on receiving advanced warning of the occurrence of flood from upstream guasge and discharge sites .
3.) To determine the sluice gate opening size and height to be maintained for assing the different values of flood .
4.) Stop dam over toppling by advance and proper operation of sluice gates maintaining proper free board above the FRL .
5.) To convey to the downstream the amount of water to be released through the sluice gates so that the same is passed by the structure below , also to safeguard human lives and property .

CHATER 8 : CONCLUSIONS & DISCUSSION

- Working on this project was a great learning experience . Application of various concepts of hydrology were involved in this project like : recording of river discharges , flood frequency studies especially Gumbel method , also energy generation , economic viability of a hydropower project and calculation of installed capacity, how it is optimized so that maximum benefit can be attained , provided some insight into the prerequisites for the construction of a hydroelectric project .

- These computations are very important since Techno-economic studies are carried out based on the energy generated from 90% dependable year flows as per guidelines of Central Electricity Authority(CEA), Ministry of Power, Govt. of India. **90% dependable year flows** : The flows of **the 39th year** (which is actually the 2001-2002 hydrological year) are used to determine the annual energy generation from the hydro power project. This energy generated is used to calculate the tariff of energy to be sold to the beneficiaries and to know the Cost/ Benefit Ratio of the project Optimization of Installed capacity of a hydro project is very important aspect to arrive at the most productive Installed Capacity of the generating machines.

- Using generating machine of capacities higher than the optimum **Installed capacity (219 MW)** would result in higher costs of the project components and of generating machines and lower Benefit/Cost ratio. If machines of installed capacities lower than the optimum installed capacities are used, energy generation benefits would become lower than what it should be and Benefit/ Cost ratio would be lower again . The graphs were in accordance with the ones put up in the literature review .

- The work done is this project resulted in the attainment of the values of flood that are very important in the design phase of the project .flood frequency studies are more important in the pre-feasibility studies of the project while the probable maximum flood is important while preparing the detailed project report . It was learnt that a flood is commonly considered to be an unusually high stage of a river..

- For a hydraulic structure planned within the river (like a dam or a barrage) or on an adjoining area (like flood control embankments), due consideration should be given to the design of the structure so as to prevent it from collapsing and causing further damage by the force of water released from behind the structure. Hence an estimate of extreme flood flow is required for the design of hydraulic structures, though the magnitude of such flood may be estimated in accordance with the importance of the structure. The 10,000 year return period flood was computed as 10,973 cumecs , using Gumbel Method and the graph obtained was according to the standard graph given in literature review .

- Hence, the water resource structure such as dam, spillways, diversion works, bridge etc should be designed according to this value of flood .

- Application of various concepts of hydrology were involved in this project like : recording of river discharges , flood frequency studies especially Log pearson type-3 distribution . The Log Pearson Type-3 analysis gave the volume of flood as 4444.68 cumecs of water This is the amount of water for which the dam , the sluice gate openings should be designed for . Estimating return period floods is important as they are needed to determine the maximum discharge and maximum potential and estimate cost and size of structure.
- The probable maximum flood (PMF) or the standard project flood (SPF) is estimated using the hydro- meteorological approach. For the PMF calculations the worst possible maximum storm (PMS) pattern is obtained from the CWC & IMD . This is then applied to the unit hydrograph of the catchment to obtain the PMF. The PMF obtained is 9260 cumecs for the worst possible storm .
- A brief study of the river basin and catchment area characteristics was also involved Understanding of how floods affect engineering structures as how the structure must be sound to withstand the calculated floods as large areas may be endangered by the failure of these structures . Flood routing studies are essential to determine the sequence and method of operation of sluices to discharge the maximum flood coming into the catchment . “Flood routing is a technique of determining the flood hydrograph at a section of a river by utilizing the data of flood flow at one or more upstream sections.” Flood Routing studies are used to finalize the sequence of operation of sluice gates for passing the peak flood value of the unit hydrograph .Determining the initial level of reservoir behind Luhri dam to be kept on receiving advanced warning of the occurrence of flood from upstream guasge and discharge sites . To determine the sluice gate opening size and height to be maintained for assing the different values of flood . Stop dam over toppling by advance and proper operation of sluice gates maintaining proper free board above the FRL . To convey to the downstream the amount of water to be released through the sluice gates so that the same is passed by the structure below , also to safeguard human lives and property .

SCOPE FOR FUTURE WORK

There is a lot of scope for future work in this field which is as follows :

- Flood frequency analysis can be done by using other methods as well as judging the suitability of different methods by the chi square fit approach which will provide us more clarity as to which method is suitable where .

- The hydrological data can be checked for discrepancies and corrected by us methods are available for that as well .

- The next step after computations is the model studies where a model of the dam can be made and the same conditions of flood are simulated to check whether the sluices or dam are operational and stable for that amount of flood .

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ANNEXURE

Luhri Stage -I HEP					
Studies to find out 90% Dependable year & 50%Mean year					
For River Satluj at Neerath					
(Data w.e.f. 1963 to 2014)					
S.No.	Year	Annual Energy From June to May (MU)	Annual Energy From June to May In Descending order (MU)	Probability of Exceedance	
1	1972-73	1360.87	1607.64	0.023	
2	1973-74	1537.49	1577.72	0.047	
3	1974-75	1136.68	1543.87	0.070	
4	1975-76	1543.87	1537.49	0.093	
5	1976-77	1154.42	1515.51	0.116	
6	1977-78	1386.02	1493.70	0.140	
7	1978-79	1577.72	1486.74	0.163	
8	1979-80	1401.70	1478.66	0.186	
9	1980-81	1365.97	1466.18	0.209	
10	1981-82	1252.06	1424.82	0.233	
11	1982-83	1466.18	1409.91	0.256	
12	1983-84	1478.66	1401.70	0.279	
13	1984-85	1125.86	1395.85	0.302	
14	1985-86	1163.76	1386.02	0.326	
15	1986-87	1364.60	1382.20	0.349	
16	1987-88	1382.20	1368.26	0.372	
17	1988-89	1316.60	1365.97	0.395	
18	1989-90	1354.91	1364.60	0.419	
19	1990-91	1409.91	1360.87	0.442	
20	1991-92	1424.82	1354.91	0.465	
21	1992-93	1231.19	1332.92	0.488	
22	1993-94	1066.03	1316.60	0.512	50% Dependable Year
23	1994-95	1493.70	1257.81	0.535	
24	1995-96	1257.81	1252.06	0.558	
25	1996-97	1332.92	1231.19	0.581	
26	1997-98	1045.83	1208.79	0.605	
27	1998-99	1515.51	1199.23	0.628	
28	1999-00	1113.43	1183.46	0.651	
29	2000-01	970.70	1163.76	0.674	
30	2001-02	1001.69	1154.42	0.698	
31	2002-03	1208.79	1136.68	0.721	
32	2003-04	1096.81	1125.86	0.744	
33	2004-05	759.39	1113.43	0.767	
34	2005-06	1183.46	1096.81	0.791	
35	2006-07	1063.13	1066.03	0.814	
36	2007-08	814.87	1063.13	0.837	
37	2008-09	1039.86	1045.83	0.860	
38	2009-10	1199.23	1039.86	0.884	
39	2010-11	1607.64	1001.69	0.907	90% Dependable Year
40	2011-12	1395.85	970.70	0.930	
41	2012-13	1368.26	814.87	0.953	
42	2013-14	1486.74	759.39	0.977	

90% dependable Year =0.9*(n+1)th year	38.7	th year
say	39	th year

50% dependable Year =0.5*(n+1)th year	21.5	th year
say	22	th year

Unrestricted Energy Generation for Luhri Stage-I Project

Input Parameters		
FRL	862.9	m
MDDL	860	m
Weighted Average Water Level	861.9333	m
Normal Tail Water Level with three units	817	m
Head Losses	1.128	m
Gross Head	44.933	m
Net Head	43.805	m
Design Discharge	565	Cumecs
Combined Efficiency of Units	0.9259	

MONTHS	PERIOD	No. OF DAYS	1972-73	1973-74	1974-75	1975-76	1976-77	1977-78	1978-79	1979-80	1980-81	1981-82	1982-83	1983-84	1984-85	1985-86	1986-87	1987-88	1988-89	1989-90	1990-91
JUN	I	10	45.932	75.057	32.754	67.609	80.596	37.433	78.304	25.497	59.015	39.152	58.346	75.248	90.718	54.717	30.462	77.063	57.200	86.517	73.721
	II	10	58.824	153.744	49.752	99.313	62.261	24.828	61.975	67.609	79.068	37.624	116.024	63.503	76.394	61.115	77.540	68.564	57.391	66.941	62.548
	III	10	78.304	147.250	41.348	113.255	44.213	68.373	112.586	164.439	108.003	84.989	85.275	88.045	69.805	70.092	131.876	76.394	96.734	62.166	126.051
JUL	I	10	88.999	118.602	48.033	74.389	80.787	130.061	135.505	119.939	105.520	73.912	121.181	99.408	78.973	70.092	93.488	107.239	95.779	70.856	108.289
	II	10	81.265	123.377	94.729	131.016	117.265	139.229	111.822	142.953	122.804	99.790	120.512	65.508	54.717	78.591	108.671	96.639	99.313	101.127	92.246
	III	11	99.895	109.769	102.521	119.118	151.156	117.542	122.059	126.156	123.950	134.139	140.231	127.836	84.139	90.546	147.689	124.160	121.744	128.152	90.757
AUG	I	10	82.601	88.235	91.673	120.130	83.556	141.712	142.475	104.756	120.226	127.006	112.586	124.141	72.193	71.429	110.581	89.859	103.801	73.625	91.769
	II	10	80.692	86.803	88.617	130.443	69.901	83.843	122.040	100.554	74.580	108.385	110.676	103.228	79.164	81.933	97.689	80.692	85.562	68.086	84.893
	III	11	68.383	96.744	67.122	107.038	61.660	68.908	86.135	84.139	64.286	76.996	77.311	117.962	81.093	80.672	82.248	83.929	70.588	82.248	82.038
SEP	I	10	63.503	83.461	46.410	62.357	51.089	61.211	88.140	60.352	50.420	47.937	48.797	82.792	61.020	59.015	59.874	64.362	51.089	54.049	63.216
	II	10	44.595	58.251	31.417	46.792	33.232	48.510	60.829	37.051	38.293	33.900	41.157	61.593	39.630	40.203	34.950	43.545	38.961	36.383	50.420
	III	10	26.929	43.640	24.924	32.850	26.738	32.563	38.484	22.536	27.025	27.597	28.457	40.871	23.491	31.035	27.502	33.805	47.746	30.749	37.911
OCT	I	10	21.104	25.974	17.380	25.592	21.772	23.300	29.125	19.958	22.154	20.435	19.767	27.788	17.666	23.014	20.626	20.913	27.311	22.154	25.306
	II	10	18.335	20.149	16.807	21.772	17.953	18.717	22.632	17.093	18.239	16.616	17.666	22.536	15.374	26.356	18.048	16.616	20.626	18.717	21.008
	III	11	16.071	20.693	15.021	22.269	18.592	18.172	20.483	16.702	17.542	16.492	17.122	20.903	15.126	20.063	17.017	16.282	18.172	18.908	19.643
NOV	I	10	13.369	16.616	12.414	18.335	14.801	15.661	17.189	14.419	13.465	14.419	14.133	16.616	11.937	15.088	14.515	13.655	15.088	16.043	15.279
	II	10	12.128	14.515	10.791	16.998	13.465	14.419	15.947	12.701	11.937	11.650	12.892	14.037	10.122	13.083	13.751	12.510	14.133	14.419	14.897
	III	10	10.886	12.128	8.976	14.515	12.223	13.178	14.992	12.223	10.504	10.027	11.173	11.364	9.358	11.555	12.987	12.223	13.560	13.465	14.324
DEC	I	10	10.027	10.886	9.645	12.510	10.218	12.223	13.942	10.313	9.454	9.072	9.836	11.268	8.881	9.931	12.701	11.555	12.892	12.701	13.560
	II	10	10.027	9.645	9.072	11.841	9.454	11.268	12.701	9.167	8.785	8.594	9.358	10.027	8.690	9.072	12.510	11.173	12.510	11.746	13.083
	III	11	10.714	10.714	8.929	12.290	10.504	11.450	12.710	10.084	9.454	8.929	9.769	10.399	9.034	9.454	13.340	11.975	13.866	12.815	12.710
JAN	I	10	9.263	9.454	7.926	10.504	9.358	8.785	10.409	8.976	8.117	8.021	8.499	9.167	7.926	7.926	11.459	10.695	12.510	10.313	9.072
	II	10	9.167	9.072	8.403	10.027	8.976	8.690	10.122	8.976	7.639	7.830	8.881	8.594	7.448	7.544	11.173	10.600	11.937	10.027	8.785
	III	11	10.084	10.084	10.189	10.504	10.084	9.664	10.924	9.769	8.508	8.403	9.559	9.034	8.088	8.088	11.975	11.134	11.975	11.029	9.454
FEB	I	10	8.881	8.690	10.695	9.454	8.785	8.785	9.645	8.785	7.639	7.830	8.785	8.021	7.162	7.830	10.695	8.403	10.695	10.313	8.785
	II	10	8.976	8.212	11.268	9.836	8.021	8.690	9.167	8.881	7.544	8.021	8.785	8.021	6.971	7.926	10.695	7.066	10.504	10.313	8.976
	III	8	7.487	6.494	8.862	9.626	7.105	6.875	7.181	7.563	5.882	6.341	7.028	7.735	5.730	6.494	8.709	6.532	8.251	8.021	7.334
MAR	I	10	9.358	9.072	12.414	10.982	9.454	8.881	9.740	8.594	8.117	8.403	8.785	9.072	8.021	9.072	10.886	7.448	10.886	10.409	9.645
	II	10	9.931	9.167	13.655	10.791	9.358	10.409	11.937	6.589	8.403	8.212	9.358	10.982	8.594	10.409	11.173	8.308	11.459	11.077	10.504
	III	11	15.651	13.025	15.756	14.076	9.244	12.290	14.496	9.979	10.609	10.714	11.134	14.916	9.874	11.660	13.340	9.874	13.655	14.916	15.126
APR	I	10	17.953	13.465	19.099	13.369	10.122	11.459	18.717	11.555	10.504	17.189	13.369	13.846	8.976	11.841	13.846	9.645	12.510	13.560	20.435
	II	10	22.154	15.756	19.385	17.093	10.027	20.245	24.160	14.419	20.054	18.430	14.992	15.852	11.173	18.144	13.655	25.401	13.751	16.998	18.526
	III	10	55.004	20.531	27.025	33.041	11.268	20.722	25.210	20.722	25.401	24.446	20.054	24.446	14.610	23.969	19.003	31.417	13.560	23.587	22.250
MAY	I	10	106.284	27.502	32.659	30.176	11.841	38.293	35.332	29.221	41.730	36.765	33.805	29.698	19.576	28.075	23.014	46.887	16.138	33.041	35.237
	II	10	55.195	27.502	56.914	39.343	14.133	54.717	31.608	31.322	42.590	32.086	57.009	46.028	22.632	48.319	21.199	54.717	29.030	86.039	48.033
	III	11	72.899	23.214	54.097	54.622	25.210	64.916	28.992	37.710	58.509	41.702	63.866	68.172	61.555	29.412	35.714	60.925	55.672	83.404	64.076

Total Annual Generation (MU)	1360.870	1537.494	1136.681	1543.873	1154.424	1386.023	1577.715	1401.703	1365.969	1252.056	1466.18	1478.661	1125.862	1163.763	1364.604	1382.203	1316.6	1354.911	1409.906

Unrestricted Energy Generation for Luhri Stage-I Project

Sheet 2 of 2

Input Parameters		
FRL	862.9	m
MDDL	860	m
Weighted Average	861.9333	m
Normal Tail Water Level	817	m
Head Losses	1.128	m
Gross Head	44.933	m
Net Head	43.805	m
Design Discharge	565	Cumecs
Combined Efficiency of Units	0.9259	

MONTHS	PERIOD	No. OF DAYS	1993-94	1994-95	1995-96	1996-97	1997-98	1998-99	1999-00	2000-01	2001-02	2002-03	2003-04	2004-05	2005-06	2006-07	2007-08	2008-09	2009-10	2010-11	2011-12	2012-13	2013-14
			49.179	71.047	67.800	57.391	31.226	66.941	26.165	34.377	30.749	70.283	100.936	25.497	26.643	55.481	25.019	45.932	48.701	45.264	43.449	43.354	114.019
JUN	I	10	67.514	63.025	90.145	102.941	44.213	59.779	52.044	44.977	42.876	87.185	89.095	40.012	44.977	36.956	57.105	98.644	41.253	40.394	63.980	47.937	97.880
	II	10	57.296	119.653	53.190	147.346	64.267	109.435	62.452	53.954	39.821	75.535	76.490	39.057	96.925	41.062	49.465	72.479	78.782	83.747	98.835	77.158	126.051
	III	10	66.845	126.242	75.248	78.400	61.784	146.200	73.243	46.028	54.049	79.641	79.355	51.089	102.750	76.967	62.070	74.198	64.076	85.180	84.034	83.747	140.184
JUL	I	10	90.241	105.329	82.792	94.061	75.535	130.825	72.002	63.694	62.166	71.429	71.333	47.937	105.615	70.665	46.601	70.187	85.466	90.241	74.866	70.760	110.294
	II	10	69.118	143.068	95.168	104.622	84.139	104.202	80.042	103.362	75.000	71.639	82.983	51.891	99.265	108.509	64.496	70.063	88.551	139.811	115.967	91.912	111.975
	III	11	65.795	124.427	81.742	94.729	73.721	92.915	97.116	92.819	62.357	68.850	76.872	52.235	84.893	127.388	58.060	61.593	78.018	158.327	89.095	104.278	100.363
AUG	I	10	56.914	94.061	73.816	107.525	79.355	97.116	69.614	59.397	73.339	80.787	63.216	52.712	64.840	86.230	63.503	64.553	75.057	141.903	98.167	93.774	109.244
	II	10	58.088	98.109	73.425	72.164	65.651	73.109	65.967	59.769	62.185	74.580	61.030	45.273	55.567	76.576	59.979	56.828	57.773	112.920	84.559	101.261	97.479
	III	11	47.078	76.585	61.879	56.054	40.871	44.213	43.258	51.757	43.163	51.184	49.943	28.170	44.882	32.754	42.590	30.080	46.505	82.124	68.564	76.490	56.436
SEP	I	10	42.208	45.741	39.343	45.264	35.428	41.444	36.287	33.041	27.502	42.494	36.860	29.125	33.423	31.990	24.446	23.109	51.662	61.784	55.290	63.694	43.163
	II	10	26.834	25.592	27.311	30.940	20.913	47.174	30.080	23.969	19.194	26.261	28.839	18.812	25.401	21.295	20.531	33.232	34.950	43.067	32.277	35.905	35.332
	III	10	19.863	22.823	22.345	24.733	13.178	29.889	22.154	16.998	15.183	19.194	17.093	15.947	16.234	16.998	14.037	17.857	24.351	32.181	25.306	25.879	25.688
OCT	I	10	16.711	19.767	19.194	19.385	9.167	27.597	18.717	15.852	14.037	16.902	14.801	14.419	13.465	14.610	10.886	14.228	20.626	21.486	19.576	18.908	22.059
	II	10	15.966	17.857	17.647	17.962	7.983	27.521	19.328	16.177	14.496	17.227	15.651	14.076	12.815	13.235	12.185	12.185	19.328	26.366	17.227	21.849	18.908
	III	11	13.178	14.801	13.942	14.897	6.875	20.340	16.616	14.133	13.178	14.324	13.655	12.128	10.600	11.364	8.976	10.695	18.239	19.576	16.234	19.290	15.183
NOV	I	10	12.892	13.942	13.465	13.655	8.021	18.717	15.565	13.465	12.223	13.560	12.892	9.836	10.122	11.173	8.499	10.027	17.571	20.340	16.138	16.138	17.953
	II	10	12.319	13.083	12.892	12.987	12.892	17.666	14.324	12.510	10.504	12.987	10.695	8.403	10.027	10.600	8.021	9.167	17.380	17.666	17.666	15.279	20.435
	III	10	11.937	12.510	12.414	12.605	11.746	16.329	13.560	11.841	9.358	12.510	9.836	7.639	9.454	9.931	7.257	8.690	19.099	18.812	13.560	12.701	12.796
DEC	I	10	10.600	12.128	11.746	11.841	11.077	15.470	12.987	10.313	8.785	11.746	9.263	7.162	8.499	8.881	7.353	9.072	17.762	17.475	12.319	11.841	10.982
	II	10	11.029	12.920	12.815	12.290	11.240	16.387	13.235	11.240	9.349	10.714	9.874	7.773	9.349	8.298	7.353	10.084	20.483	20.273	14.916	11.660	10.609
	III	11	10.122	11.364	11.364	10.791	9.454	14.419	11.746	9.836	8.403	6.780	8.785	6.875	8.212	7.448	6.875	13.369	14.515	17.380	17.475	10.122	8.212
JAN	I	10	10.218	10.886	10.886	10.600	9.072	14.037	11.650	9.645	7.926	5.921	8.785	6.875	7.926	7.735	6.875	14.133	12.892	15.852	16.329	10.027	7.544
	II	10	10.819	11.555	11.660	11.345	9.664	14.916	12.710	10.399	8.613	6.618	7.248	6.933	9.139	8.719	7.668	15.546	14.181	16.702	17.017	9.769	8.719
	III	11	9.549	10.218	10.313	10.313	8.499	13.369	11.268	8.499	7.735	6.971	6.685	6.685	8.021	8.117	6.875	12.701	12.605	12.892	13.178	10.218	6.875
FEB	I	10	9.645	10.504	10.218	10.313	8.212	12.223	11.459	8.403	8.117	7.066	7.735	7.257	8.212	7.926	6.780	12.319	12.987	14.133	14.419	10.600	6.016
	II	10	7.945	8.709	9.368	8.174	6.646	9.855	10.141	6.875	6.341	5.806	7.133	5.806	6.570	7.133	6.102	10.485	11.516	11.860	12.204	10.227	4.899
	III	8	9.836	10.600	10.791	10.218	8.881	12.319	11.268	8.499	9.358	8.021	8.021	8.403	8.594	8.594	6.685	11.841	12.987	14.515	14.801	12.223	5.921
MAR	I	10	11.268	10.027	12.414	11.077	10.027	12.796	11.650	8.308	10.313	7.639	8.308	9.454	8.499	10.122	7.353	11.841	13.178	15.565	15.852	12.510	6.398
	II	10	13.550	13.445	14.601	11.765	11.450	13.971	13.445	10.399	12.815	11.240	10.189	11.660	9.349	8.508	7.983	12.815	17.017	17.857	17.962	15.966	11.450
	III	11	13.274	11.937	13.942	11.268	12.510	14.706	14.037	10.313	12.605	11.077	9.836	10.122	9.167	11.650	7.639	11.077	11.268	14.801	14.801	16.234	13.369
APR	I	10	12.510	13.274	22.059	13.178	13.178	19.385	15.565	11.077	14.897	18.239	10.218	11.173	10.313	12.987	9.836	10.027	16.425	15.852	16.043	19.481	13.083
	II	10	13.465	16.043	35.046	16.807	24.828	30.844	17.189	12.223	21.390	23.205	11.746	16.902	18.239	16.807	9.740	12.796	19.481	21.772	22.059	27.979	14.515
	III	10	23.014	21.390	36.383	22.441	32.850	33.805	20.435	17.093	30.176	26.356	11.173	26.547	41.635	21.868	12.032	17.284	36.383	31.131	31.895	28.743	29.125
MAY	I	10	27.025	62.070	34.568	15.947	33.232	39.821	40.203	25.401	78.782	55.195	19.672	25.019	59.492	26.929	17.857	25.974	27.406	46.314	47.078	46.314	23.014
	II	10	62.185	38.971	55.882	26.891	78.046	55.777	45.903	24.055	64.706	79.622	30.567	20.483	84.349	27.626	36.135	44.748	40.756	62.080	62.710	84.034	30.567
	III	11																					
Total Annual Generation (MU)			1066.026	1493.701	1257.81	1332.92	1045.83	1515.511	1113.429	970.6954	1001.692	1208.788	1096.813	759.3886	1183.463	1063.133	814.87	1039.861	1199.229	1607.643	1395.849	1368.261	1486.739

Luhri Stage -I (Neerath Dam) HYDROELECTRIC PROJECT (219 MW)

POWER GENERATION IN A 90 % DEPENDABLE YEAR

(Hydrology as per river Pinder data w.e.f. 1977 to 2007

Power & Energy values 95% of actual values as per notification dated 30th March, 1992

of Ministry of Power. Govt. of India.

FRL		862.9	m
Combined Efficiency		0.9259	
Net Head		43.805	m
Installed Capacity		219.00	MW
Design Discharge		565.00	Cumecs
Station Availability		95.00	%
Annual Energy Generation		884	GWH
Water release d/s of dam for environmental purpose 20% lean period discharge		0.00	Cumecs

PERIOD		INFLOWS IN CUMECS		POWER/ENERGY GENERATION			
				UNRESTRICTED		RESTRICTED TO 95% OF 219 MW	
		Satluj at Neerath	Discharge after min.water relase downstream of dam	POWER	Energy	Power	Energy
		(Cumecs)	(Cumecs)	MW	GWH	MW	GWH
JUNE	I	322.00	322.00	128.12	30.75	128.12	30.75
	II	449.00	449.00	178.65	42.88	178.65	42.88
	III	417.00	417.00	165.92	39.82	165.92	39.82
JULY	I	566.00	565.00	224.81	53.95	208.05	49.93
	II	651.00	565.00	224.81	53.95	208.05	49.93
	III	714.00	565.00	224.81	59.35	208.05	54.93
AUGUST	I	653.00	565.00	224.81	53.95	208.05	49.93
	II	768.00	565.00	224.81	53.95	208.05	49.93
	III	592.00	565.00	224.81	59.35	208.05	54.93
SEPTEMBER	I	452.00	452.00	179.85	43.16	179.85	43.16
	II	288.00	288.00	114.59	27.50	114.59	27.50
	III	201.00	201.00	79.98	19.19	79.98	19.19
OCTOBER	I	159.00	159.00	63.26	15.18	63.26	15.18
	II	147.00	147.00	58.49	14.04	58.49	14.04
	III	138.00	138.00	54.91	14.50	54.91	14.50
NOVEMBER	I	138.00	138.00	54.91	13.18	54.91	13.18
	II	128.00	128.00	50.93	12.22	50.93	12.22
	III	110.00	110.00	43.77	10.50	43.77	10.50
DECEMBER	I	98.00	98.00	38.99	9.36	38.99	9.36
	II	92.00	92.00	36.61	8.79	36.61	8.79
	III	89.00	89.00	35.41	9.35	35.41	9.35
JANUARY	I	88.00	88.00	35.01	8.40	35.01	8.40
	II	83.00	83.00	33.02	7.93	33.02	7.93
	III	82.00	82.00	32.63	7.83	32.63	8.61
FEBRUARY	I	81.00	81.00	32.23	7.73	32.23	7.73
	II	85.00	85.00	33.82	8.12	33.82	8.12
	III	83.00	83.00	33.02	8.72	33.02	6.34
MARCH	I	98.00	98.00	38.99	9.36	38.99	9.36
	II	108.00	108.00	42.97	10.31	42.97	10.31
	III	122.00	122.00	48.54	12.82	48.54	12.82
APRIL	I	132.00	132.00	52.52	12.61	52.52	12.61
	II	156.00	156.00	62.07	14.90	62.07	14.90
	III	224.00	224.00	89.13	21.39	89.13	21.39
MAY	I	316.00	316.00	125.73	30.18	125.73	30.18
	II	825.00	565.00	224.81	53.95	208.05	49.93
	III	616.00	565.00	224.81	59.35	208.05	54.93
				TOTAL	918.52		883.54

	N	O	P	Q	R	S	T	U	V	W	X	Y	Z	AA
1	**Evaluation of Flood Frequencies of River Satluj At Nirath By Gumble's Method**													
2			**(Flood Peak Data w.e.f. 1968 to 1995)**											
3				**Reduced variate y = -0.834032-2.302585*Xt**	**Maximum flood* =**	**Probability of exceedence each year**								
4														
5	**Return Period T**	**log (T/T-1)**	**Xt= loglog(T/T-1)**		**1375.93+1042.05y**									
6														
7	**(Years)**				**(Cumecs)**	**(%)**								
8														
9														
10	10000	4.34316E-05	-4.362193973	9.210290409	10973.46	0.0001								
11	5000	8.68676E-05	-4.06114226	8.517093251	10251.12	0.0002								
12	4000	0.000108587	-3.964221388	8.293924704	10018.57	0.0003								
13	3000	0.000144789	-3.839264551	8.006200966	9718.75	0.0003								
14	2000	0.000217202	-3.663137088	7.600652512	9296.15	0.0005								
15	1000	0.000434512	-3.361998451	6.907255203	8573.60	0.0010								
16	500	0.000869459	-3.060751036	6.213607425	7850.79	0.0020								
17	300	0.001450066	-2.838612112	5.70211367	7317.79	0.0033								
18	200	0.002176919	-2.662157679	5.29581234	6894.40	0.0050								
19	100	0.004364805	-2.360035114	4.600149453	6169.49	0.0100								
20	50	0.008773924	-2.056806117	3.901938912	5441.92	0.0200								
21	25	0.017728767	-1.751321469	3.198534544	4708.94	0.0400								
22	22.7	0.019566123	-1.708495213	3.09992345	4606.19	0.0441								
23	20	0.022276395	-1.652155096	2.970195541	4471.00	0.0500								
24	15	0.029963223	-1.523411468	2.673752395	4162.10	0.0667								
25	10	0.045757491	-1.339537801	2.250367648	3720.91	0.1000								
26	5	0.096910013	-1.013631348	1.499940338	2938.93	0.2000								
27	1.25	0.698970004	-0.155541461	-0.475884565	880.03	0.8000								
28														
29														
30														
31	Q' =	1945.957198		*Maximum flood	Q'(1+Cv*(y-yn)/sigma n)									
32	Cv =	0.6184		=	1375.93	1042.05	y							
33	yn =	0.54703		=	1375.93+1042.05y									
34	sigma n =	1.15488												
35														
36														
37														
38														
39	No. of Years (occurences)		N	10	15	20	25	30	40	50	60	70	80	90
40	Reduced Mean y_n			0.4952	0.5128	0.5236	0.5309	0.5362	0.5436	0.5485	0.5521	0.5548	0.5569	0.5586
41	Reduced Standard Deviation S_n			0.9457	1.206	1.0628	1.0915	1.1124	1.1413	1.1607	1.1747	1.1854	1.1938	1.2007
42														
43														

	A	B	C	D	E	F	G	H	I	J	K
1	**Calculation of Standard Deviation & Coeff. Of Variation For Design flood at luhri stage-1 dam .**										
2		**(Flood peak data w.e.f. 1963 to 1995 at Rampur/Jhakri for lean period i.e. Oct. to March)**									
3											
4	**S.No.**	**Year**	**Flood Peak**		**Flood peak**		**S.No.**	**Q**	**Q'-Q**	**(Q'-Q)^2**	
5				**Instantaneous Max. Discharge**	**in descending order**						
6			**(Cumecs)**	**(Cumecs)**	**(Cumecs)**			**(Cumecs)**	**(Cumecs)**	**(Cumecs)^2**	
7											
8	1	1963	1325.7	1546.1	8280		1	8280.127	-6334.169	40121700.753	
9	2	1964	1238.2	1444.0	5248		2	5247.968	-3302.010	10903272.038	
10	3	1965	2612.9	3047.2	3047		3	3047.203	-1101.246	1212742.700	
11	4	1966	1704.8	1988.2	2534		4	2533.602	-587.645	345326.517	
12	5	1967	1277.3	1489.6	2348.640389		5	2348.640	-402.683	162153.752	
13	6	1968	1353.7	1578.7	2170.792601		6	2170.793	-224.835	50550.959	
14	7	1969	1724.4	2011.0	2153.649241		7	2153.649	-207.692	43135.985	
15	8	1970	1061.6	1238.1	2113		8	2112.832	-166.875	27847.105	
16	9	1971	1431.2	1669.1	2044.141652		9	2044.142	-98.184	9640.187	
17	10	1972	1159.2	1351.9	2041.809222		10	2041.809	-95.852	9187.611	
18	11	1973	1861.4	2170.8	2011		11	2011.021	-65.064	4233.317	
19	12	1974	1565.5	1825.7	1988.163332		12	1988.163	-42.206	1781.358	
20	13	1975	1661.5	1937.7	1965		13	1964.722	-18.765	352.133	
21	14	1976	1752.8	2044.1	1937.666223		14	1937.666	8.291	68.740	
22	15	1977	2172.5	2533.6	1934.517442		15	1934.517	11.440	130.868	
23	16	1978	1846.7	2153.6	1880		16	1880.055	65.902	4343.073	
24	17	1979	2013.9	2348.6	1871.541832		17	1871.542	74.415	5537.647	
25	18	1980	1582.5	1845.5	1845.535238		18	1845.535	100.422	10084.570	
26	19	1981	1612.1	1880.1	1839.47092		19	1839.471	106.486	11339.327	
27	20	1982	1464.5	1707.9	1825.709583		20	1825.710	120.248	14459.489	
28	21	1983	1528	1782.0	1782		21	1781.977	163.981	26889.663	
29	22	1984	880	1026.3	1756.902898		22	1756.903	189.054	35741.528	
30	23	1985	1022.5	1192.5	1708		23	1707.922	238.035	56660.818	
31	24	1986	1811.7	2112.8	1670.952852		24	1670.953	275.004	75627.390	
32	25	1987	1388.9	1619.8	1669		25	1669.087	276.870	76657.157	
33	26	1988	1506.5	1756.9	1619.756014		26	1619.756	326.201	106407.212	
34	27	1989	1658.8	1934.5	1578.705246		27	1578.705	367.252	134873.996	
35	28	1990	1432.8	1671.0	1546		28	1546.051	399.906	159924.786	
36	29	1991	1577.3	1839.5	1543.368931		29	1543.369	402.588	162077.312	
37	30	1992	1271	1482.3	1490		30	1489.606	456.351	208256.033	
38	31	1993	1252.6	1460.8	1482		31	1482.259	463.698	215015.773	
39	32	1994	1750.8	2041.8	1461		32	1460.801	485.156	235376.624	
40	33	1995	1027	1197.7	1444.007413		33	1444.007	501.950	251953.586	
41	34	1996	1604.8	1871.5	1418.11744		34	1418.117	527.840	278614.810	
42	35	1997	1323.4	1543.4	1405.872183		35	1405.872	540.085	291691.823	
43	36	1998	1684.7	1964.7	1352		36	1351.876	594.081	352931.961	
44	37	1999	1205.5	1405.9	1238		37	1238.054	707.903	501127.158	
45	38	2000	7100	8280.1	1220		38	1220.444	725.513	526369.403	
46	39	2001	856.2	998.5	1197.702805		39	1197.703	748.254	559884.636	
47	40	2002	1046.5	1220.4	1192.454838		40	1192.455	753.502	567765.807	
48	41	2003	1216	1418.1	1026.2692		41	1026.269	919.688	845826.013	
49	42	2004	682.8	796.3	998.513283		42	998.513	947.444	897649.971	
50	43	2005	4500	5248.0	796.291602		43	796.292	1149.666	1321730.982	
51										60826942.570	
52							Sum Q =	83676.159	Sum (Q'-Q)^2 =		
53							Mean discharge Q' =	1945.957198	Cumecs		
54							Standard deviation Sigma =	Sum((Q'-Q)^2/(n-1))^0.5			
55							=	1203.436969			
56							Coeff. Of variation Cv =	Sigma/Q'			
57							=	0.6184			

Design Flood Determination for a Hydro Electric Project By Log Pearson Type III Distribution

Rank (m)	Year	Max. Flood Discharge	Max. Flood Discharge,in descending order Q	Log Q	Log Q-Avg.(LogQ)^2	Log Q-Avg.(LogQ)^3	Return Period T_r=(n+1)/m	Exceedance Probability (1/T_r)
1	**1963**	**1657.1**	**3458.0**	3.538824989	0.08003	0.02264	44.00	0.0227
2	**1964**	**1547.8**	**3266.1**	3.514032802	0.06661	0.01719	22.00	0.0455
3	**1965**	**3266.1**	**2715.6**	3.433869798	0.03166	0.00563	14.67	0.0682
4	**1966**	**2131.0**	**2517.4**	3.400947915	0.02103	0.00305	11.00	0.0909
5	**1967**	**1596.6**	**2326.8**	3.366749723	0.01228	0.00136	8.80	0.1136
6	**1968**	**1692.1**	**2308.4**	3.363306362	0.01153	0.00124	7.33	0.1364
7	**1969**	**2155.5**	**2264.6**	3.354996297	0.00981	0.00097	6.29	0.1591
8	**1970**	**1327.0**	**2191.0**	3.340642378	0.00717	0.00061	5.50	0.1818
9	**1971**	**1789.0**	**2188.5**	3.340146551	0.00709	0.00060	4.89	0.2045
10	**1972**	**1449.0**	**2155.5**	3.333548027	0.00602	0.00047	4.40	0.2273
11	**1973**	**2326.8**	**2131.0**	3.32858345	0.00528	0.00038	4.00	0.2500
12	**1974**	**1956.9**	**2105.9**	3.323432589	0.00456	0.00031	3.67	0.2727
13	**1975**	**2076.9**	**2076.9**	3.317410359	0.00378	0.00023	3.38	0.2955
14	**1976**	**2191.0**	**2073.5**	3.31670404	0.00369	0.00022	3.14	0.3182
15	**1977**	**2715.6**	**2015.1**	3.304301991	0.00234	0.00011	2.93	0.3409
16	**1978**	**2308.4**	**2006.0**	3.302330929	0.00215	0.00010	2.75	0.3636
17	**1979**	**2517.4**	**1978.1**	3.296253732	0.00163	0.00007	2.59	0.3864
18	**1980**	**1978.1**	**1971.6**	3.294824316	0.00151	0.00006	2.44	0.4091
19	**1981**	**2015.1**	**1956.9**	3.291563085	0.00127	0.00005	2.32	0.4318
20	**1982**	**1830.6**	**1910.0**	3.281033367	0.00063	0.00002	2.20	0.4545
21	**1983**	**1910.0**	**1883.1**	3.274879149	0.00036	0.00001	2.10	0.4773
22	**1984**	**1100.0**	**1830.6**	3.262599389	0.00004	0.0000003	2.00	0.5000
23	**1985**	**1278.1**	**1791.0**	3.253095586	0.00001	0.0000000	1.91	0.5227
24	**1986**	**2264.6**	**1789.0**	3.252610341	0.00001	0.0000000	1.83	0.5455
25	**1987**	**1736.1**	**1763.4**	3.246344679	0.00009	-0.0000009	1.76	0.5682
26	**1988**	**1883.1**	**1736.1**	3.239580991	0.00027	-0.0000044	1.69	0.5909
27	**1989**	**2073.5**	**1692.1**	3.228432442	0.00076	-0.00002	1.63	0.6136
28	**1990**	**1791.0**	**1657.1**	3.219355269	0.00134	-0.00005	1.57	0.6364
29	**1991**	**1971.6**	**1654.3**	3.218601143	0.00139	-0.00005	1.52	0.6591
30	**1992**	**1588.8**	**1596.6**	3.203202925	0.00278	-0.00015	1.47	0.6818
31	**1993**	**1565.8**	**1588.8**	3.201055564	0.00301	-0.00017	1.42	0.7045
32	**1994**	**2188.5**	**1565.8**	3.19472242	0.00375	-0.00023	1.38	0.7273
33	**1995**	**1283.8**	**1547.8**	3.189700813	0.00439	-0.00029	1.33	0.7500
34	**1996**	**2006.0**	**1520.0**	3.181843588	0.00549	-0.00041	1.29	0.7727
35	**1997**	**1654.3**	**1506.9**	3.178077228	0.00606	-0.00047	1.26	0.7955
36	**1998**	**2105.9**	**1449.0**	3.161068385	0.00900	-0.00085	1.22	0.8182
37	**1999**	**1506.9**	**1327.0**	3.122870923	0.01771	-0.00236	1.19	0.8409
38	**2000**	**3458.0**	**1308.1**	3.116649246	0.01940	-0.00270	1.16	0.8636
39	**2001**	**1070.3**	**1283.8**	3.108480457	0.02174	-0.00321	1.13	0.8864
40	**2002**	**1308.1**	**1278.1**	3.10657333	0.02231	-0.00333	1.10	0.9091
41	**2003**	**1520.0**	**1100.0**	3.041392685	0.04603	-0.00988	1.07	0.9318
42	**2004**	**853.5**	**1070.3**	3.029485237	0.05128	-0.01161	1.05	0.9545
43	**2005**	**1763.4**	**853.5**	2.931203525	0.10545	-0.03424	1.02	0.9773
					Sum =	**Sum =**		
			Avg. log Q =	**3.255937861**	0.60274	-0.01471		
n =	43							
						0.063171319		

Derivation of UG for area below Wangtoo

L	65.16	km
Lc	33.47	km
Ar	1426	km^2

S.No.	Elevation (m)	Length (km)	Length of each segment. km	Height above Datum, m (Di)	Di+Di-1	Li(Di+Di-1)
1	834	0.00	0.00	0.00	0.00	0.00
2	862	5.00	5.00	28.00	28.00	140.00
3	890	10.00	5.00	56.00	84.00	420.00
4	923	15.00	5.00	89.00	145.00	725.00
5	963	20.00	5.00	129.00	218.00	1090.00
6	1021	25.00	5.00	187.00	316.00	1580.00
7	1093	30.00	5.00	259.00	446.00	2230.00
8	1138	35.00	5.00	304.00	563.00	2815.00
9	1220	40.50	5.50	386.00	690.00	3795.00
10	1289	45.00	4.50	455.00	841.00	3784.50
11	1365	50.00	5.00	531.00	986.00	4930.00
12	1424	55.00	5.00	590.00	1121.00	5605.00
13	1489	60.00	5.00	655.00	1245.00	6225.00
14	1550	65.16	5.16	716.00	1371.00	7074.36
			65.16			40413.86

Equivalent Slope S= **9.52**

1	tp =	2.498*(L*Lc/S)^0.156	5.83	hrs	
2	qp =	1.048*(tp)^-0.178	0.77	cumecs	0.765694353
3	W50 =	1.954*(L*Lc/S)^0.099	3.35	hrs	
4	W75 =	0.972*(L*Lc/S)^0.124	1.91	hrs	
5	WR50 =	0.189*(W50)^1.769	1.60	hrs	
6	WR75 =	0.419*(W75)^1.246	0.94	hrs	
7	TB =	7.845*tp)^0.453	17.44	hrs	
8	Qp =	qp*Ar	1091.88	cumecs	
9	Tm =	tp+0.5	6.33	hrs	
	∑Qi=	(Arxd)/(trX0.36)	3961.11	cumecs	

Time	UG cordinates
0	0
4.7	545.94
5.4	818.91
6.3	1091.88
7.3	818.91
8.1	545.94
17.4	0

times	cordinates derived from UG			
0	0	0	0	
1	20	1	20	
2	40	2	40	
3	100	3	100	
4	250	4	250	
4.7	546	5	665	
5	665	6.0	1020	
5.4	819	7	900	
6	1020	8	545	
6.3	1092	9	170	
7	900	10	70	
7.3	818.91	11	40	
8.1	546	12	39	
9	170	13	35	
10	70	14	31	
11	40	15	**23**	
12	39	16	13	
13	35	17	0	
14	31			
15	23	**Total Vol Eq to 1 cm**	3961.0	Cumec
16	13	Total=∑Qixtrx0.36 (Ar)	**1.000**	cm
17	0			
Eq to	**3961.94**			
	3961.11			

CONVOLUTION OF EXCESS RAINFALL AT NIRATH FOR CATCHMENT AREA BETWEEN NIRATH AND WANGTU

Time (hour)		Contribution Due to Access rain fall (cm) 1st Bell												Contribution Due to Access rain fall (cm) IInd Bell													Baseflow	Flood Hydrograph
	UG ordinates	0.53	0.53	0.53	0.53	0.78	0.78	1.51	2.97	3.46	2.48	1.02	0.78	0.15	0.15	0.15	0.15	0.13	0.21	0.37	0.86	1.02	0.69	0.13	0.13	DSRO		Ordinate
0	0	0.00																								0.00	77.70	78
1	20	10.63	0.00																							10.63	77.70	88
2	40	21.26	10.63	0.00																						31.88	77.70	110
3	100	53.14	21.26	10.63	0.00																					85.02	77.70	163
4	250	132.85	53.14	21.26	10.63	0.00																				217.87	77.70	296
5	665	353.38	132.85	53.14	21.26	15.50	0.00																			576.13	77.70	654
6	1020	542.03	353.38	132.85	53.14	31.01	15.50	0.00																		1127.91	77.70	1206
7	860	457.00	542.03	353.38	132.85	77.52	31.01	30.13	0.00																	1623.92	77.70	1702
8	535	284.30	457.00	542.03	353.38	193.80	77.52	60.26	59.39	0.00																2027.68	77.70	2105
9	210	111.59	284.30	457.00	542.03	515.51	193.80	150.66	118.78	69.14	0.00															2442.81	77.70	2521
10	80	42.51	111.59	284.30	457.00	790.70	515.51	376.65	296.94	138.28	49.64	0.00														3063.13	77.70	3141
11	45	23.91	42.51	111.59	284.30	666.67	790.70	1001.89	742.35	345.70	99.27	20.38	0.00													4129.29	77.70	4207
12	39	20.72	23.91	42.51	111.59	414.73	666.67	1536.73	1974.65	864.25	248.18	40.76	15.50	0.00												5960.22	77.70	6038
13	35	18.60	20.72	23.91	42.51	162.79	414.73	1295.68	3028.79	2298.91	620.45	101.90	31.01	3.00	0.00											8063.00	77.70	8141
14	31	16.47	18.60	20.72	23.91	62.02	162.79	806.03	2553.68	3526.14	1650.40	254.75	77.52	6.00	3.00	0.00										9182.04	77.70	**9260**
15	23	12.22	16.47	18.60	20.72	34.88	62.02	316.39	1588.63	2973.02	2531.44	677.64	193.80	15.00	6.00	3.00	0.00									8469.83	77.70	8548
16	13	6.91	12.22	16.47	18.60	30.23	34.88	120.53	623.57	1849.50	2134.35	1039.38	515.51	37.50	15.00	6.00	3.00	0.00								6463.65	77.70	6541
17	0	0.00	6.91	12.22	16.47	27.13	30.23	67.80	237.55	725.97	1327.76	876.34	790.70	99.75	37.50	15.00	6.00	2.50	0.00							4279.85	77.70	4358
18		0.00	0.00	6.91	12.22	24.03	27.13	58.76	133.62	276.56	521.18	545.17	666.67	153.00	99.75	37.50	15.00	5.00	4.13	0.00						2586.63	77.70	2664
19		0.00	0.00	0.00	6.91	17.83	24.03	52.73	115.81	155.57	198.54	213.99	414.73	129.00	153.00	99.75	37.50	12.51	8.25	7.38	0.00					1647.52	77.70	1725
20			0.00	0.00	0.00	10.08	17.83	46.70	103.93	134.82	111.68	81.52	162.79	80.25	129.00	153.00	99.75	31.27	20.63	14.75	17.13	0.00				1215.14	77.70	1293
21				0.00	0.00	0.00	10.08	34.65	92.05	121.00	96.79	45.86	62.02	31.50	80.25	129.00	153.00	83.17	51.58	36.89	34.26	20.38	0.00			1082.47	77.70	1160
22					0.00	0.00	0.00	19.59	68.30	107.17	86.86	39.74	34.88	12.00	31.50	80.25	129.00	127.57	137.21	92.22	85.65	40.76	13.88	0.00		1106.57	77.70	1184
23						0.00	0.00	0.00	38.60	79.51	76.94	35.67	30.23	6.75	12.00	31.50	80.25	107.56	210.46	245.30	214.12	101.90	27.76	2.50	0.00	1301.04	77.70	1379
24							0.00	0.00	0.00	44.94	57.08	31.59	27.13	5.85	6.75	12.00	31.50	66.91	177.45	376.24	569.55	254.75	69.39	5.00	2.50	1738.64	77.70	1816
25								0.00	0.00	0.00	32.26	23.44	24.03	5.25	5.85	6.75	12.00	26.26	110.39	317.23	873.60	677.64	173.48	12.51	5.00	2305.68	77.70	2383
26									0.00	0.00	0.00	13.25	17.83	4.65	5.25	5.85	6.75	10.01	43.33	197.34	736.56	1039.38	461.47	31.27	12.51	2585.44	77.70	2663
27										0.00	0.00	0.00	10.08	3.45	4.65	5.25	5.85	5.63	16.51	77.46	458.21	876.34	707.81	83.17	31.27	2285.67	77.70	2363
28											0.00	0.00	0.00	1.95	3.45	4.65	5.25	4.88	9.29	29.51	179.86	545.17	596.78	127.57	83.17	1591.51	77.70	1669
29												0.00	0.00	0.00	1.95	3.45	4.65	4.38	8.05	16.60	68.52	213.99	371.25	107.56	127.57	927.96	77.70	1006
30													0.00	0.00	0.00	1.95	3.45	3.88	7.22	14.39	38.54	81.52	145.73	66.91	107.56	471.14	77.70	549
31														0.00	0.00	0.00	1.95	2.88	6.40	12.91	33.40	45.86	55.51	26.26	66.91	252.08	77.70	330
32															0.00	0.00	0.00	1.63	4.75	11.43	29.98	39.74	31.23	10.01	26.26	155.02	77.70	233
33																0.00	0.00	0.00	2.68	8.48	26.55	35.67	27.06	5.63	10.01	116.08	77.70	194
34																	0.00	0.00	0.00	4.80	19.70	31.59	24.29	4.88	5.63	90.88	77.70	169
35																		0.00	0.00	0.00	11.13	23.44	21.51	4.38	4.88	65.34	77.70	143
36																			0.00	0.00	0.00	13.25	15.96	3.88	4.38	37.46	77.70	115
37																				0.00	0.00	0.00	9.02	2.88	3.88	15.77	77.70	93
38																					0.00	0.00	0.00	1.63	2.88	4.50	77.70	82
39																						0.00	0.00	0.00	1.63	1.63	77.70	79
40																							0.00	0.00	0.00	0.00	77.70	78
41																								0.00	0.00	0.00	77.70	78
42																									0.00	0.00	77.70	78

	R	S	T
1	**LHEP - Area Capacity Curve Table**		
2			
3			
4	**S.No.**	**Contour**	**Cumm.**
5		**Elevation**	**Capacity**
6		**(m)**	**(ham)**
7			
8	**1**	**820.000**	**0.000**
9	**2**	832.905	0.438
10	**3**	**824.999**	**0.876**
11	**4**	**832.905**	0.976
12	**5**	**830.000**	**1.786**
13	**6**	**833.000**	**3.194**
14	**7**	**835.000**	**4.133**
15	**8**	**840.000**	**6.218**
16	**9**	**841.091**	**7.043**
17	**10**	**847.595**	**11.963**
18	**11**	**841.091**	**7.043**
19	**12**	**845.000**	**10.000**
20	**13**	**847.626**	**12.188**
21	**14**	**848.255**	12.713
22	**15**	**848.048**	**12.540**
23	**16**	**844.467**	**9.556**
24	**17**	851.000	15.000
25	**18**	**851.000**	**15.000**
26	**19**	**852.125**	**16.407**
27	**20**	**854.600**	**19.500**
28	**21**	**855.000**	**20.000**
29	**22**	**855.000**	**20.000**
30	**23**	**855.122**	**20.245**
31	**24**	**856.887**	**23.775**
32	**25**	**860.000**	**30.000**
33	**26**	**863.951**	**33.000**

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1		**Flood Routing Computations For Luhri Dam Reservoir**												
2				**(Taking four sluices as operative)**										
3	**Time**	**Delta t**	**Cummulative**	**Inflow at**	**Average**	**Inflow**	**Trial**		**Average**	**Outflow**	**Incremental**	**Total**	**Reservoir**	
4	**t**	**(hrs./minutes)**	**Time**	**time t**	**rate of inflow**	**during**	**reservoir**	**Outflow**	**rate of**	**during**	**storage**	**storage**	**elevation**	
5			**(Minutes)**		**Qi for Delta t**	**time**	**storage**	**rate at**	**outflow**	**time**	**Delta s**		**at end of**	
6						**Delta t**	**elevation at**	**time t**	**Qo for Delta t**	**Delta t**			**Delta t**	
7				**(Cumecs)**	**(Cumecs)**	**(mcm)**	**time t**	**(Cumecs)**	**(Cumecs)**	**(mcm)**	**(mcm)**	**(mcm)**	**(m)**	**Remarks**
8														
9	0			1702			847.5949847	1903.56				10.000		
10		60.000			1903.5035	6.852613	847.5949847	1903.5035	1903.532	6.853	0.000	10.000	847.5949847	O.K.
11														
12			60.000											
13														
14														
15	1			2105										
16														
17		50.882			2312.9465	7.061187	847.6256808	1288.95	1596.228	4.873	2.188	12.18797	847.6256808	O.K.
18														
19			50.882											
20														
21	2			2521										
22		4.938			2830.668	0.838704	848.048	4134.83	2711.889	0.804	0.035	12.223	848.048	O.K.
23			5.000											
24														
25	3			3141										
26		60.775			3673.906	13.39689	848.255	3118.38	3626.600	13.224	0.172	12.396	848.255	O.K.
27														
28														
29			61.000											
30														
31	4			4207										
32		60.000			5122.4548	18.44084	851.00	4451.720	3785.048	13.626	4.815	17.210	851	O.K.
33			60.000											
34														
35														
36	5			6038										
37		59.716			7089.3121	25.40064	854.60	7215.007	5833.363	20.901	4.500	21.710	854.6	O.K.
38			60.000											
39	6			8141										
40		53.292			8700.2198	27.819	855.12	9719.598	8467.302	27.074	0.745	22.455	855.1223773	O.K.
41														
42														
43														
44	7			**9260**										
45		60.00			8903.6326	32.05301	856.89	10048.793	9884.196	35.583	3.530	25.985	856.8873874	O.K.
46														
47	8			8548										
48		60.000			7544.4389	27.15998	852.13	9133.477	9591.135	34.528	-7.368	18.617	852.1253356	O.K.
49														
50														
51	9			6541										
52		60.000			5449.449	19.618	844.467	5571.439	7352.458	26.469	-6.851	11.766	844.467	O.K.
53														
54														
55	10			4358										
56		74.30			3510.937133	15.65095	841.09	2577.905	4074.672	18.164	-2.513	9.253	841.0906203	O.K.
57	11			2664										
58		60.000			2194.776633	7.901238	832.90	3607.073	4029.396	14.506	-6.605	2.648	832.9045847	O.K.

	W	X	Y	Z	AA
1		Luhri Dam -Sluice Rating Curve			
2	Coeff. Of discharge (for pressurised flow)		0.85		
3	Width of sluice		8.5	m	
4	Height of sluice		15	m	
5	El. Upto C/L of sluice		829.5	m	
6	Sluice crest El.		820	m	
7					
8			**Discharge**		
9			**Through**		
10	**S. No.**	**Reservoir Elevation**	**Sluice**		
11		**(m)**	**(Cumecs)**		
12					
13	**1.**	**820**	**0**		
14	**2.**	**822**	**44.02**	**Weir Flow**	
15	**3.**	**824**	**124.50**	**Weir Flow**	
16	**4**	**826**	**228.72**	**Weir Flow**	
17	**5**	**828**	**352.13**	**Weir Flow**	
18	**6**	**832.9045847**	**721.41**	**Weir Flow**	
19	**7**	**830**	**492.12**	**Weir flow**	
20	**8**	**834.9045847**	**895.47**	**Weir Flow**	
21	**9**	**835.9045847**	**987.08**	**Weir Flow**	
22	**10**	**837.9045847**	**900.00**	**Transition Stage**	
23	**11**	**839.9045847**	**1100.00**	**Transition Stage**	
24	**12**	**841.9045847**	**1300.00**	**Transition Stage**	
25	**13**	**843.9045847**	**1500.00**	**Transition Stage**	
26	**14**	**841.0906203**	**1288.95**	**Transition Stage**	
27	**15**	**845.9045847**	**1650.00**	**Transition Stage**	
28	**16**	**847.5949847**	**1903.56**	**Transition Stage**	
29	**17**	**847.9045847**	**1950.00**	**Transition Stage**	
30	**18**	**847.6256808**	**2043.74**	**Pressurised Flow**	
31	**19**	**848.048**	**2067.41**	**Pressurised Flow**	
32	**20**	**850.048**	**2176.02**	**Pressurised Flow**	
33	**21**	**851**	**2225.86**	**Pressurised Flow**	
34	**22**	**852.5**	**2302.20**	**Pressurised Flow**	
35	**23**	**852.1253356**	**2283.37**	**Pressurised Flow**	
36	**24**	**848.26**	2078.917	**Pressurised Flow**	
37	**25**	**851.00**	2225.860	**Pressurised Flow**	
38	**26**	**854.60**	2405.002	**Pressurised Flow**	
39	**27**	**855.12**	2429.900	**Pressurised Flow**	
40	**28**	**856.89**	2512.198	**Pressurised Flow**	
41	**29**	**844.4670075**	1857.146	**Pressurised Flow**	