STANDARDS/MANUALS/ GUIDELINES FOR SMALL HYDRO DEVELOPMENT

1.3
General–
Project hydrology and installed capacity

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Alternate Hydro Energy Center
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Standards/ Manuals/Guidelines series for Small Hydropower Development

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1.1	Small hydropower definitions and glossary of terms, list and scope of different Indian and international standards/guidelines/manuals
1.2 Part I	Planning of the projects on existing dams, Barrages, Weirs
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SECTION-I: PROJECT HYDROLOGY

1.0 SCOPE

The scope of this publication is limited to hydrological data collection, estimation / assessment and data analysis to establish a reliable water flow with time variability and the peak flood discharge at the project site alongwith other hydrological inputs required for fixing the installed capacity and establishing energy generation. Accurate assessment of the hydrology is important as it plays a vital role in the planning of small hydropower schemes and the design of various hydrological structures. An over estimate of water availability may lead to higher installation and larger investment whereas lower estimate may result in non-utilization of potential optimally. Water availability at the project intake site decides the techno-economic feasibility of the project and flood discharge is important for safe design of the structures. The objectives of this guideline are as follows:

(1) To provide the knowledge of data requirement, source of data, evaluation and extension of data.
(2) To familiarize various methods of synthesizing data at an ungauged site.
(3) To provide an overview of various analysis techniques available to work out stream flows with time variability and peak flood discharge.
(4) To provide hydrologic considerations for inputs other than stream flow.

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3.0 DATA REQUIREMENT

The basic requirement of the data for hydropower development is the head and discharge available at the site. Generally observed long term stream flow data are not available for the small hydro project sites, hence discharge is to be started measuring at the first possible opportunity. The discharge data may also be assessed by use of indirect methods with the help of the following data:

(i) Catchment area with following characteristics:
- (a) Altitude
- (b) Raingauge locations and their long term rainfall data
- (c) River system
- (d) Shape
- (e) Land use pattern
- (f) Snow cover, if any
- (g) Soil types
- (h) Details of existing projects in the catchment, if any.

(ii) Stream flow and rainfall data of adjoining and/or similar catchments.
(iii) Water utilization data from existing projects/facilities.

3.1 Sources of Data

(i) Rain gauge locations, rainfall and snowfall data and other climatological information such as temperature, wind velocity, evaporation, cloud cover etc. which are available with the India Meteorological Department (IMD) Pune, their regional offices and state revenue department.
(ii) Major rivers and their tributaries are being gauged regularly by the Central Water Commission and State Governments and long term records of water levels, sediment load and daily discharges at gauging sites of rivers / tributaries are available.
(iii) Other sources from where some useful information can be obtained are as below.
  (a) Survey of India
  (b) Irrigation/water resources Department of State Governments
  (c) Agriculture Department of State Governments
  (d) Forest Department of State Governments
  (e) District Revenue Department of State Governments
  (f) Water Year Books of State Governments
  (g) Snow and Avalanche Study Establishment (SASE)

3.2 Collection of Data

The project developer should install gauging station near the intake site and few rain gauges in the catchment. The discharge measurements should be carried out for a minimum of two years covering two lean seasons and one monsoon season on daily basis. Two years discharge data though is short to develop a long term series but it gives an idea about minimum discharge expected to be available and can be used for initiating the planning for development of the site(s). In order to have longer period observed discharges, the gauging site should be established at the earliest and the data till the preparation of detailed project report (DPR) should be used in hydrological studies. For gauging and discharge measurement techniques, guideline for site investigations can be referred. For the SHP projects on existing dams and canal falls, a longer time series 5-15 years normally be used depending on data availability at existing facilities, which normally is available.

3.3 Quality of Data

For assessing the quality of data, details of methods of measurement and observations, the instruments used and the frequency of observations is essential. Adequate length of data is essential for any hydrologic analysis. The longer the length of data more is the confidence on the reliability of the analysis. In view of development on river in the upstream that might have taken place, normalization of discharge data is required. Generally data of 25 to 30 years is considered adequate for any statistical analysis but for small huydropower projects, a 10 years period may be adequate. However, planning may be initiated with minimum of two years daily discharge data.

3.4 Filling-in Missing Data

It is generally observed that rainfall and discharge data in many cases are found missing for some days and even for months. The following techniques are used for filling missing data:

(i) Using values of observed discharges earlier or later of the missing period

(ii) Using the observations of the adjoining stations. Atleast three stations be used. The normal ratio method can be used for estimation of missing rainfall data. In the normal ratio method, the rainfall R_A at station A is estimated as a function of the normal monthly or annual rainfall of the station and those of the neighbouring stations for the period of missing data.

$$R_A = \frac{\sum_{i=1}^{n} \frac{NR_A}{NR_i} \text{ x } R_i}{n}$$

Where,

R_A	is the estimated rainfall at station A
R_i	is the rainfall at surrounding stations
NR_A	is the normal monthly or seasonal rainfall at station A
NR_i	is the normal monthly or seasonal rainfall at station i
n	is the number of surrounding stations.

4.0 FLOW ASSESSMENT FOR AN UNGAUGED CATCHMENT

Many times situation arises when the discharge observations are not available at all for streams and flow assessment has to be made for planning and the preparation of project report of a possible project site. Depending on the availability of data of other sites or basins one of the following methods may be adopted.

4.1 Long Term Data of Some Other Site

When long term flow measurement data of a site on the same stream or adjoining stream are available, they can be transposed to the proposed site in proportion to the catchment areas of the two sites.

$$\frac{Q_1}{Q_2} = \frac{A_1}{A_2} \text{ i.e. } Q_1 = \frac{A_1}{A_2} Q_2$$

1 denotes ungauged site and 2 denotes the site for which flow data is available.

In case the rainfall data are available for both the catchments, the rainfall variability may also be considered.

4.2 Regional Model

A regional model for generating flows of different dependabilities has been developed jointly by AHEC, Department of Earth Sciences of IIT Roorkee and NIH Roorkee under UNDP – GEF and MNRE, Govt. of India, sponsored hilly hydro project which may be useful for generating the flow duration curve of an ungauged small catchment in Himalayas

The model developed for ungauged catchments based on hydro meteorologically similar regional catchments can be used. In this model flow duration curve (FDC) for an ungauged catchment is derived using regionalization procedure. The regions in India are identified and given in Table 1. For each region, based on available data of gauged catchments a mean FDC of that region in terms of Q/Q_{mean} and percentage of time is developed. The regional flow estimated values for $(Q/Q_{mean})_D$ for various dependability levels (D) are given in Table 2. Q_{mean} for each gauged catchment is related with catchment area (A).

$$Q_{mean} = CA^m$$

where C is coefficient and m is exponent. The values of C and m for each region (A to I) are given in Table 1.

Knowing the area of ungauged catchment (A), Q_{mean} can be worked out using the values of C and m of the region. This value of Q_{mean} multiplied by the factor $(Q/Q_{mean})_D$ for that region from Table 2 will give the required dependable flow (Q_D) for that ungauged catchment. After obtaining Q_D for different value of D, the FDC of the ungauged catchment can be plotted for further planning purposes.

Table 1: Values of the parameters of the Regional Models for Mean Flow

S. No.	Region	State covered	Exponent, m	Coefficient, C	Coefficient of correlation (R)
1.	A	Jammu & Kashmir (Except Leh & Kargil)	0.06046	3.8189	0.0808
2.	B	Jammu & Kashmir (Leh & Kargil)	$Q/A = (1/2)(Q/A)_{Leh} + (Q/A)_{Kargil} = 0.05804$		
3.	C	Himachal Pradesh	0.86811	0.1200	0.8759
4.	D	Uttar Pradesh/Uttarakhand	0.89075	0.0463	0.8174
5.	E	Bihar/Jharkhand	0.74795	0.0652	0.7742
6.	F	West Bengal & Sikkim	0.98920	0.0577	0.8467
7.	G	North Assam & Arunachal Pradesh	0.26817	2.2807	0.3706
8.	H	South Assam & Meghalaya	0.48589	1.4136	0.6820
9.	I	Manipur, Nagaland, Mizoram & Tripura	1.22343	0.0151	0.9435

Table 2: Regional Flow Estimates for Various Dependability

Region	Regional Values for $(Q/Q_{mean})_D$ For Various Dependability Levels (D)					
	D = 25%	D = 50%	D = 60%	D = 75%	D = 80%	D = 90%
A	1.1562	0.6584	0.5428	0.4011	0.3577	0.2686
B	1.2240	0.8434	0.7360	0.5888	0.5396	0.4304
C	1.1797	0.6609	0.5399	0.3917	0.3466	0.2544
D	1.2828	0.8364	0.7078	0.5315	0.4729	0.3447
E	0.7374	0.2711	0.1974	0.1226	0.1031	0.0675
F	1.0942	0.5089	0.3896	0.2551	0.2171	0.1444
G	1.3075	0.8500	0.7148	0.5270	0.4640	0.3257
H	1.1436	0.4909	0.3551	0.2053	0.1646	0.0913
I	1.2451	0.5511	0.3957	0.2198	0.1716	0.0856

5.0 WATER AVAILABILITY ASSESSMENT AND FLOW DURATION CURVE

5.1 Water Availability Assessment using Ten Daily Data

The basic time unit used in preparing a flow-duration curve greatly affects its appearance. When mean daily discharges are used, a steep curve is obtaied as the flow duration curve. When the mean flow over a longer period is used (such as ten-daily flows or mean monthly flows), the resulting curve will be flatter due to averaging of short-term peaks with intervening smaller flows during a month. Extreme values are averaged out more and more, as the time period gets larger. The steps to draw flow duration curve from available to-daily average discharge data are given below:

Step 1:

Sort (rank) average daily discharges for period of record from the largest value to the smallest value, involving a total of n values.

Step 2:

Assign each discharge value a rank (M), starting with 1 for the largest ten-daily discharge value.

Step 3:

Calculate exceedence probability (P) as follows:

$$P = \frac{M}{n+1} \times 100$$

where,

P = Probability that a given flow will be equalled or exceeded (% of time)
M = Ranked position on the listing (dimensionless)
n = Number of events for period of record (dimensionless)

The procedure has been explained with the following example. The ten daily average flows of a gauging site are given in table 3:

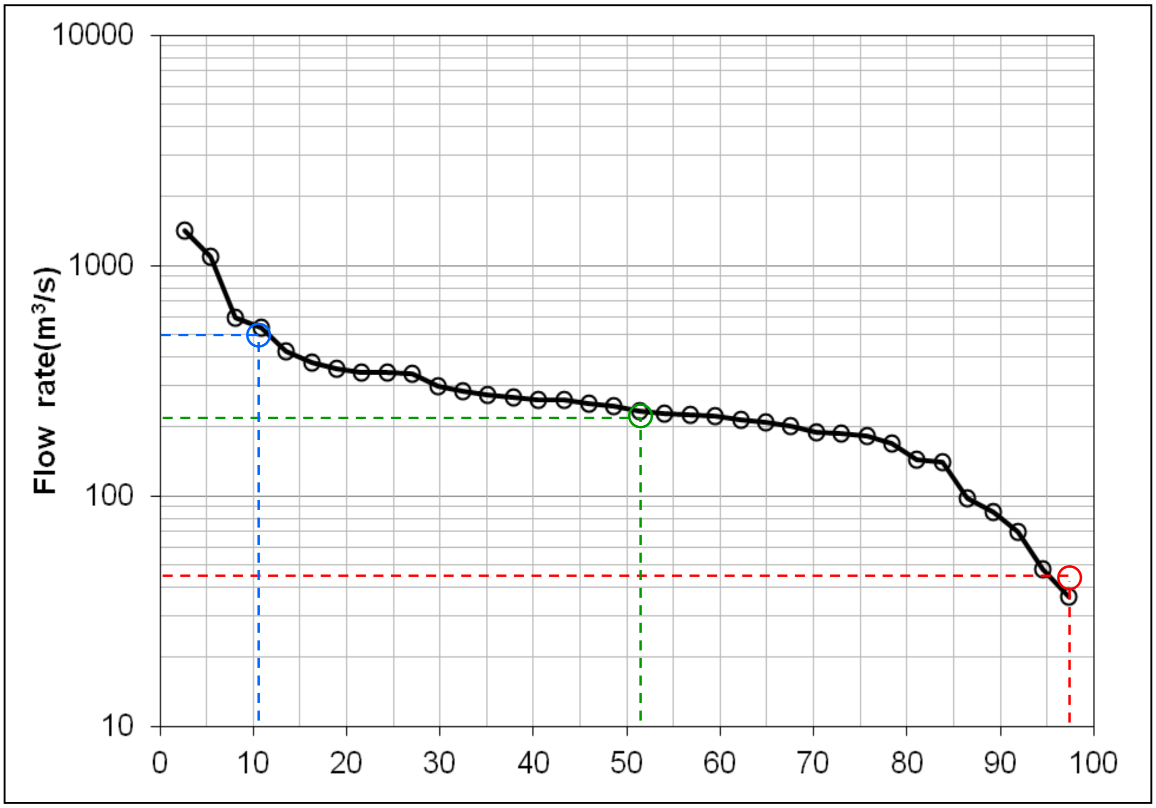
Table 3: Values of ten daily average flows

S. No.	Value	S. No.	Value	S. No.	Value
1	222.7	13	338.3	25	233.5
2	224.5	14	286.8	26	143.7
3	228.9	15	260.7	27	170.1
4	422.3	16	267.4	28	188.4
5	535.2	17	274.9	29	139.6
6	341.8	18	261.2	30	209.3
7	356.7	19	244.7	31	187.9
8	591.7	20	252.7	32	97.5
9	343.1	21	301.2	33	47.9
10	379.4	22	200.1	34	69.7
11	1416.2	23	182.7	35	84.8
12	1097.3	24	213.6	36	36.4

To develop the flow duration curve (FDC) and estimate the flow in river at 10%, 50% and 95% and other percentage of time the discharge values are first arranged in descending order and then the highest value is ranked one. The exceedence probability (P) is calculated using above equation for n=36+1=37. The FDC is plotted as shown in Fig. 1 and its tabular form is given in Table 4.

Table 4: Values, rank and probability of flows

Value	Rank(M)	P (%)
1416.2	1	2.70
1097.3	2	5.41
591.7	3	8.11
535.2	4	10.81
422.3	5	13.51
379.4	6	16.22
356.7	7	18.92
343.1	8	21.62
341.8	9	24.32
338.3	10	27.03
301.2	11	29.73
286.8	12	32.43
274.9	13	35.14
267.4	14	37.84
261.2	15	40.54
260.7	16	43.24
252.7	17	45.95
244.7	18	48.65
233.5	19	51.35
228.9	20	54.05
224.5	21	56.76
222.7	22	59.46
213.6	23	62.16
209.3	24	64.86
200.1	25	67.57
188.4	26	70.27
187.9	27	72.97
182.7	28	75.68
170.1	29	78.38
143.7	30	81.08
139.6	31	83.78
97.5	32	86.49
84.8	33	89.19
69.7	34	91.89
47.9	35	94.59
36.4	36	97.30



Probability of Exceedance (%) or % of time Equalled or Exceeded

Fig. 1: Flow durvation curve

From the Fig. 1, the minimum flow at 10%, 50%, and 95% of time are found to be 560 m^3/s, 230 m^3/s and 47 m^3/s respectively. The minimum flow at 10%, 50%, and 95% or any other dependability may also be obtained/ interpolated from this curve.

5.2 Water Availability Assessment using Annual Data

Temporal water availability at the proposed project site is essential to estimate the power potential and annual energy generation. In run of river small hydro projects the flow duration curve (FDC) is drawn to know the time variability of flow. It shows a discharge which has equalled or exceeded certain percentage of time out of the total time period which is generally taken as one year.

In case discharge data are available for more than 10 years, for water availability studies for a SHP, the FDC is drawn for 90% dependable year. The 90% and 75% dependable year is generally calculated by arranging the annual runoff of all the years in descending order and using Weilbuls' formula:

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

P is dependability percentage, m is the rank of runoff of the desired dependability and N is the number of data. If P is 90%, N = 19, m works out as $\frac{90}{100} \text{ x } (19+1) = 18$. 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-daily discharge series of that year is considered. These 36 discharge values (normally if 10 daily average is used for hydropower project. In case daily discharge data is used these discharge values shall be 365 / 366 and 12 values for monthly discharge data) are arranged in descending order and percentage of time each has exceeded or equaled is worked out using the above Weilbul's formula. Discharge of rank first will be equaled or exceeded by $\frac{1}{36+1} \text{x} \, 100$ i.e. 2.7% of time. Similarly discharge of rank 2 will be equaled or exceeded by 5.4% of time. In this manner percentage of time equaled or exceeded by all the 36 discharges can be worked out and plotted. A typical flow duration curve is shown in Fig. 2. From these curves, discharges of various dependability such as Q_{90}, Q_{75}, Q_{50} etc. may be obtained. In case discharge data is available for less than 10 years, this procedure of developing FDC can be used by averaging discharge data of each year after arranging is descending year. This FDC is plotted between percentage of time and average 10-daily discharge.

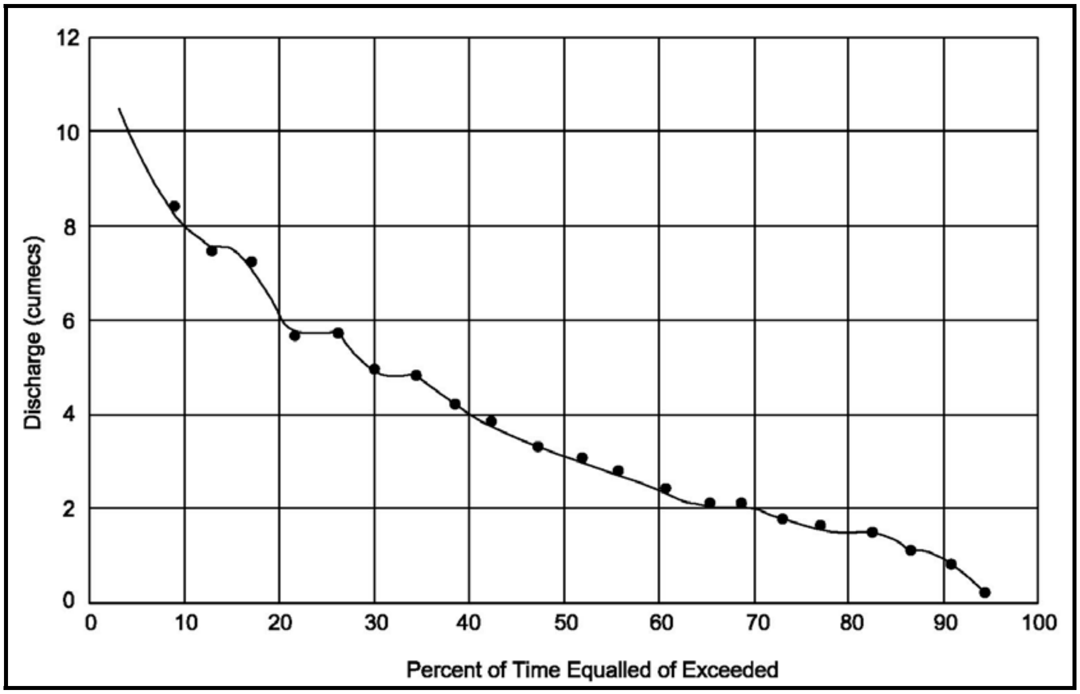


Fig. 2: Typical Flow Durvation Curve

6.0 ESTIMATION OF FLOOD DISCHARGE

Estimation of flood discharge is essential for the safety of the diversion structure of the SHP which is generally a weir or barrage or small dam without large storage capacity. In such a case moderation of flood peak is not possible, thus the capacity of waterway to pass the flood should be adequate. Design flood is fixed after due consideration of economic, hydrologic factors and safety of life and property in the downstream. For barrages / weirs, design flood of 50 to 100 years return period may be adopted. In respect of dams, design flood may be adopted as per IS Code 11223 – 1985. The methods of computation of design flood are given below:

6.1 Flood Frequency Method

For estimating the design flood, one of the standard flood frequency methods may be used. Large number of flood frequency methods are available in the literature. For the use of any method of flood frequency analysis, long term records (about 30 years) of observed flood peak discharges are required. Design flood of desired return period may be worked out using Gumbels, Normal, Log-Normal-2 parameters, Log Normal-3 parameters, Log Pearson Type-III distributions etc. The design flood given by best fit distribution may be adopted after applying the tests for goodness of fit like chi square test etc. Gumbel's method is generally used for small hydro projects.

According to Gumbel's method of moment, the flood frequency equation is given as;

$$X_T = x + s\,(0.78\,Y - 0.45)$$

Where,

X_T is flood peak of return period T.
x is average value of annual flood peaks.
s is standard deviation of flood peak series.
Y is called reduced variate and is a function of T and its values are given in Table 5:

Table 5: Relationship between Return Period (T) and Reduced Variate

Return Period (T) in Years	Reduced Variate (Y)
2	0.37
5	1.5
10	2.25
25	3.2
50	3.9
100	4.6

6.2 Method of least squares can also be used and the flood frequency equation is as given below:

$$X_T = A + B.Y$$

Where,

$$Y = -\log\log\frac{T}{T-1} \text{ and}$$

$$T = \frac{N+1}{m}$$

m is the rank of peak discharge in descending series of flood peaks, N is the number of flood peaks considered, A & B are constants and can be worked out by using method of least square.

After determining A and B, X_T for any value of T can be worked out from the above equation.

6.3 Data Plotting Method

The annual flood peak data are plotted on a semi – log paper. The data events are plotted on the ordinate which has the rectangular scale and the return periods are plotted on the abscissa which has the logarithmic scale. For example, if 20-years flood data is available and arranged in descending order then highest flow is assumed to have a return period of 20 years, the second highest flow a return period of 10-years, the third highest a return period of 6.67 years and so on. The plot is shown in Fig. 3. It is seen that peak flows of watersheds generally produce linear or near linear curves when plotted on semi log paper. The extrapolation of these linear plots can give the peak flood of 100 years return period. This extrapolation can also be done by developing the best fit line using method of least square to determine the constants in the following equation:

$$X_T = A + B \log_{10} T$$

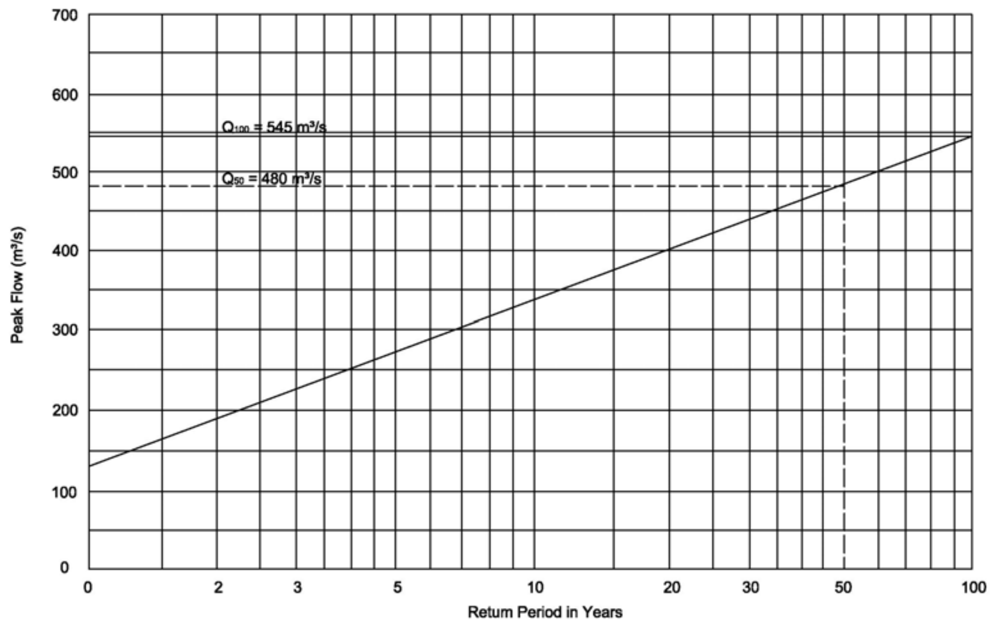


Fig. 3: Flood Frequency Peak Flow Vs. Return Period

6.4 When Peak Flood Data at Site is not Available

Generally, long records of flood peaks at SHP sites are not available. In that case one of the following approaches may be adopted depending on availability of data.

(a) If long term record of flood peaks of some other site on the same stream or a site in adjoining hydro meteorologically similar catchment is available, the flood frequency analysis can be carried out, to determine the peak flood of 100-year return period and the same can be transposed to the ungauged site of the SHP in proportion to area by using following equation:

$$\frac{Q_g}{Q_u} = \left(\frac{A_g}{A_u}\right)^{3/4}$$

Where,

Q_g is flood peak of 100 year return period of site of which record is available.
A_g is catchment area of site of which record is available.
Q_u is flood peak of 100 year return period of ungauged site.
A_u is catchment area of ungauged site.

(b) Using Unit Hydrograph for Flood Estimation

When short interval catchment rainfall and runoff data for flood events at project site are not available, the parameters of unit hydrograph may be evaluated by using CWC sub-zonal reports. For the derivation of regional unit hydrograph and its application to compute design flood hydrograph, reference may be made to relevant CWC study report of the sub zone to which the catchment of proposed SHP belongs. In this approach, concurrent storm rainfall and run-off data of the representative catchments over a period of 5 to 10 years have been used to develop representative unit hydrographs of the catchments located in the region. Using these data, a method based on synthetic unit hydrograph (SUH) principle has been developed for the various subzones of India. For this purpose, the country has been divided into 26 hydro-meteorologically-homogeneous subzones. These reports have been prepared jointly by Central Water Commission (CWC), Bridges and Flood Wing of Research Designs and Standards Organization (RDSO) of Ministry of Railways. Ministry of Transport (MOT) and India Meteorological Department (IMD). The relationship of SUH and catchment characteristics for North Brahmaputra Subzone 2(a), South Bahmaputra Subzone 2(b) and Western Himalayas Zone7 are given in Table 6. The locations of these subzones are shown in Fig. 4.

Table 6: Relationships between SUH parameters and catchment characteristics

SUH Parameters	**Relationships**		
	North Brahmaputra Subzone 2(a)	**South Bahmaputra Subzone 2(b)**	**Western Himalayas Zone7**
Time in hours from the centre of unit rain fall duration to the peak of unit hydrograph.	$t_p = 2.164\left(q_p\right)^{-0.940}$	$t_p = 2.87\left(q_p\right)^{-0.839}$	$t_p = 2.498\left(LL_c/s\right)^{-0.156}$
Peak discharge of unit hydrograph per unit area of catchment ($m^3/s/km^2$)	$q_p = 2.272\left(LL_c/s\right)^{-0.409}$	$q_p = 0.905\left(A\right)^{0.758}$	$q_p = 1.048\left(tp\right)^{-0.178}$
Width of UH in hours at 50 percent of peak discharge	$W_{50} = 2.084\left(q_p\right)^{-1.065}$	$W_{50} = 2.304\left(q_p\right)^{-1.035}$	$W_{50} = 1.954\left(LL_c/s\right)^{0.099}$

SUH Parameters	Relationships		
	North Brahmaputra Subzone 2(a)	**South Bahmaputra Subzone 2(b)**	**Western Himalayas Zone7**
Width of UH in hours at 75 percent of peak discharge	$W_{75} = 1.028 \left(q_p\right)^{-1.071}$	$W_{75} = 1.339 \left(q_p\right)^{-0.978}$	$W_{75} = 0.972 \left(LL_c / s\right)^{0.124}$
Width of the rising limb of UH in hours at 50 percent peak discharge	$WR_{50} = 0.856 \left(q_p\right)^{-0.865}$	$WR_{50} = 0.814 \left(q_p\right)^{-1.018}$	$WR_{50} = 0.189 \left(W_{50}\right)^{1.769}$
Width of the rising limb of UH in hours at 75 percent peak discharge	$WR_{75} = 0.440 \left(q_p\right)^{-0.918}$	$WR_{75} = 0.494 \left(q_p\right)^{-0.996}$	$WR_{75} = 0.419 \left(W_{75}\right)^{1.246}$
Base width of unit hydrograph in hours	$T_B = 5.428 \left(t_p\right)^{0.852}$	$T_B = 2.447 \left(t_p\right)^{1.157}$	$T_B = 7.845 \left(t_p\right)^{0.453}$
Peak discharge of unit hydrograph in m^3/s	$Q_p = q_p \times A$	$Q_p = q_p \times A$	$Q_p = q_p \times A$

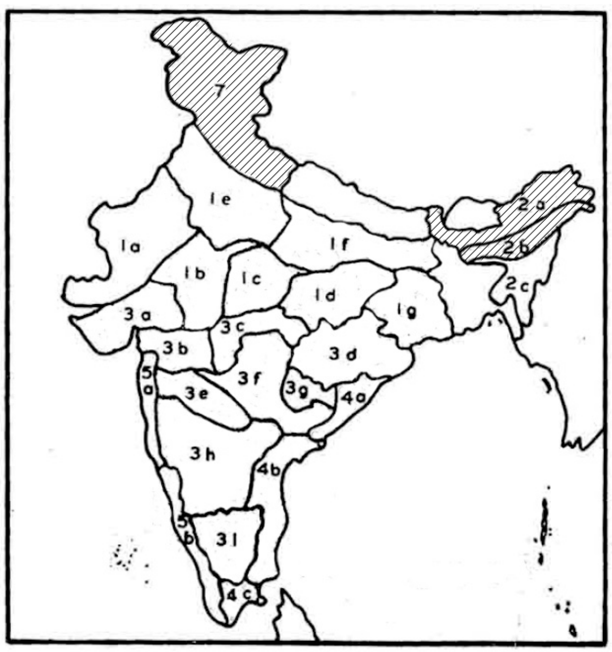


Fig. 4: Locations of North Brahmaputra Subzone 2 (a), South Brahmaputra Subzone 2 (b) and Western Himalayas Zone-7 (Ref. R9)

Example for Design Flood Estimation using SUH is given below:

The 1 day rainfall for the study area lying in Zone-7 for 25, 50 and 100 year return periods obtained from isopluvials supplied by IMD in CWC (1994) manual are 110 mm, 140 mm and 150 mm respectively. The catchment characteristics of a catchment in Western Himalayas region (zone-7) are given in Table 7.

Table 7: Characteristics of a catchment laying in Zone 7

Parameter	Value
A	697.85 km^2
L	44.816 km
L_c	17.131 km
S	3.423 m/km

Where,

A = Catchment area in km^2
L = Length of the longest main stream along the river course in km
L_c = Length of the longest main stream from a point opposite to the centriod of the catchment area to the outlet
S = Equivalent slope in m/km.

Using the given catchment characteristics and the relationships between the SUH parameters and the catchment characteristics for Zone-7, the estimated SUH parameters are given in Table 8.

Table 8: Estimated Synthetic Unit Hydrograph Parameters

S. No.	Parameters & Relationship	Value
1	$t_p = 2.498\left(LL_c/s\right)^{-0.156}$ (h)	5.5
2	$q_p = 1.048\left(tp\right)^{-0.178}$ (m^3/s/km^2)	0.77
3	$W_{50} = 1.954\left(LL_c/s\right)^{0.099}$ (h)	3.34
4	$W_{75} = 0.972\left(LL_c/s\right)^{0.124}$ (h)	1.9
5	$WR_{50} = 0.189\left(W_{50}\right)^{1.769}$ (h)	1.6
6	$WR_{75} = 0.419\left(W_{75}\right)^{1.246}$ (h)	0.93
7	$T_B = 7.845\left(t_p\right)^{0.453}$ (h)	16.98
8	$Q_p = q_p \times A$ (m^3/s)	539.93

Based on the equations given above the unit hydrograph is plotted and the volume of the SUH is computed. In case, the volume of SUH is not preserved as a unit (1 cm) then the SUH is modiofied in the rising and/ or recession limb for preserving its unit volume. The ordinates of derived SUH for the above refered catchment are given in Table 9. It may be noted that the volume (m^3) of derived SUH should be 2.778*A.

Design Storm

The 1 day rainfall for the study area for 25, 50 and 100 year return period obtained from isopluvials supplied by IMD in CWC (1994) manual are 110 mm, 140 mm and 150 mm respectively. These values are increased by 15% to convert them into any 24-hour values (CWC, 2001). In case the 24 hour rainfall values for various return periods are avaialble (as in CWC subzone reports) than increase of 15% is not required.

Table 9: Ordinates of synthetic unit hydrograph

Time (hr)	Discharge (m^3/sec)
0	0.0
1	9.0
2	16.0
3	40.0
4	167.0
5	378.0
6	539.9
7	385.0
8	217.0
9	86.7
10	35.0
11	22.0
12	16.0
13	12.0
14	8.0
15	5.0
16	2.0
17	0.0

Design Storm Duration

The design storm (T_d) is estimated as T_d=1.1 × t_p = 1.1× 5.5 = 6 h (CWC, 1994).

Rainfall Depth Duration Frequency

For 6 h duration the ratio for 24 h point rainfall to short duration rainfall is 0.73. Hence, the given 24 h point rainfall values of various return periods are multiplied by 0.73 (CWC, 1994).

Conversion of Point to Areal Rainfall

The Areal Reduction Factor (ARF) is found to be 0.71 for the study area for 6 h design storm duration. Hence, the 6 h duration rainfall values computed above for various return periods are multiplied by 0.71 (CWC, 1994).

Rainfall distribution

After suitable adjustment of the design storm rainfall of various return periods for short duration of 6 hour and areal extent and coefficients are given in Table 10 (CWC, 1994).

Table 10: Rainfall coefficients

Time (h)	Coefficients
0	0.0000
1	0.5437
2	0.1754
3	0.1228
4	0.0702
5	0.0526
6	0.0353

Critical sequencing

The temporal distribution of the total 6 h rain storm is performed according to the coefficients. The, design storm has been considered in the form of single bell for convolution with the design unit hydrograph as its duration is less than 24 hours. However, for design storm duration of 1-day, 2-days and 3-days or more the procedure given in CWC (2001) for two bells per day may be adopted.

Design Loss Rate

For estimation of design effective rainfall, design loss of 0.5 cm/h has been considered as given in CWC (1994). For computation of design effective rainfall, design loss rate of 0.5 cm/h has been subtracted from the 1-houly distributed rainfall values.

Estimation of Design DSRO

The 25, 50, and 100 year return period rainfall values are distributed into hourly incremental values using the distribution coefficients given in Table 10. The synthetic unit hydrographs and the excess-rainfall hyetographs for 25, 50, and 100 year return period are given below in Table-11. The design unit hydrograph has been convoluted with the effective rainfall for computing design DSRO hydrograph.

Table 11: Ordinates of unit hydrographs and excess rainfall hyetographs

Time (hr)	U.H (m^3/s)	Excess Rainfall (cm)		
		25 year	50 year	100 year
0	0.0	0.00	0.00	0.00
1	9.0	0.00	0.00	0.00
2	16.0	0.00	0.05	0.15
3	40.0	0.45	0.75	0.95
4	167.0	2.55	3.35	3.65
5	378.0	0.15	0.25	0.15
6	539.9	0.00	0.00	0.00
7	385.0			
8	217.0			
9	86.7			
10	35.0			
11	22.0			

Time (hr)	U.H (m^3/s)	Excess Rainfall (cm)		
		25 year	50 year	100 year
12	16.0			
13	12.0			
14	8.0			
15	5.0			
16	2.0			
17	0.0			

Design Base Flow

Base flow is the portion of stream flow that comes from the sum of deep subsurface flow and delayed shallow subsurface flow. CWC (1994) has analyzed total 45 flood events for estimating base flow. The recommended value of base flow 0.05 cumec per sq. km for subzone-7 is used here. On this basis, the base flow works out to be 34.89 m^3/s.

Estimation of Design Flood

For estimation of flood hydrograph, the base flow contribution is added to the design DSRO hydrograph. The ordinates of the flood hydrographs for various return period are given in Table 12 and the same are also shown in Fig. 5.

Table 12: Ordinates of the flood hydrograph

Time (hr)	U.H (m^3/s)	Flood Hydrograph (m^3/s)		
		25 year	50 year	100 year
0	0.0	34.89	34.89	34.89
1	9.0	34.89	34.89	34.89
2	16.0	34.89	34.89	34.89
3	40.0	34.89	35.34	36.24
4	167.0	38.94	42.44	45.84
5	378.0	65.04	79.04	88.94
6	**539.9**	95.04	129.09	157.69
7	385.0	214.44	317.04	398.64
8	217.0	636.84	914.84	1090.53
9	86.7	1266.80	1767.11	2010.30
10	35.0	**1641.58**	**2237.66**	**2460.53**
11	22.0	1195.28	1626.7	1740.28
12	16.0	685.00	924.86	972.31
13	12.0	304.28	406.93	420.44
14	8.0	147.04	191.12	198.95
15	5.0	103.44	129.94	137.44
16	2.0	84.39	103.39	109.19
17	0.0	71.49	85.34	89.44

Time (hr)	U.H (m^3/s)	Flood Hydrograph (m^3/s)		
		25 year	50 year	100 year
18		59.34	68.54	70.94
19		49.74	55.14	56.24
20		40.74	42.84	42.94
21		35.19	35.39	35.19
22		34.89	34.89	34.89
23		34.89	34.89	34.89
24		34.89	34.89	34.89

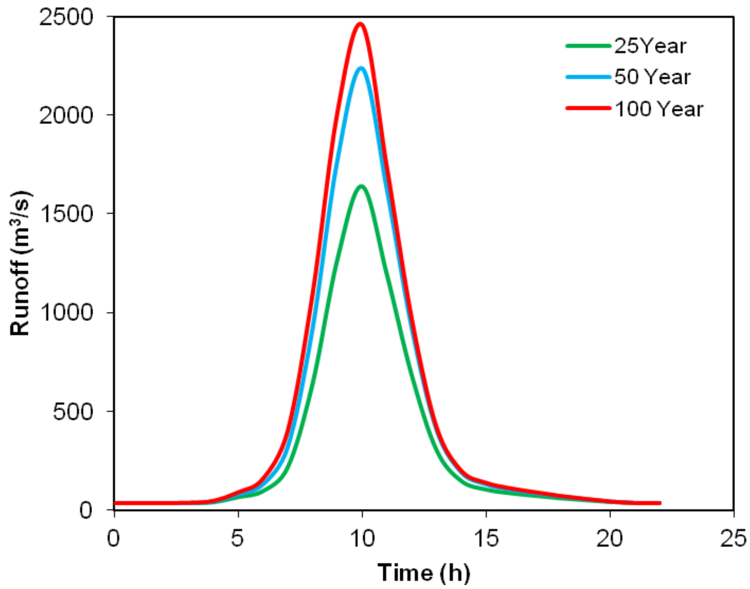


Fig. 5: Design flood hydrographs for 25, 50 and 100 year return periods

(c) Regional Flood Frequency Analysis

For estimation of floods of various return periods regional flood frequency analysis may be carried out using the L-moments approach.

For estimation of floods of various return periods for gauged and ungauged catchments, regional flood frequency relationships/ regional flood formulae have been developed using the L-moments approach for three subzones, out of the 26 hydrometeorological homogeneous sub-zone shown in Fig 4.

For developing the L-moments based regional flood formulae, the annual maximum peak floods data of the stream flow measurement sites of the respective region/ Subzone have been screened using the Discordancy measure (D_i) and homogeneity of the region is tested employing the L-moments based heterogeneity measure (H). Based on the L-moments ratio diagram and $|Z_i^{dist}|$ -statistic criteria, the robust frequency distribution has been identified for each of the Subzones. For the Western Himalayas Zone-7, GLO: for North Brahmaputra Subzone 2 (a), PE3 and for South Brahmaputra Subzone 2 (b) GNO distributions have been found to the robust distributions. Also, for estimation of floods of various return periods for ungauged catchments, the regional flood frequency relationships developed for gauged catchments have been coupled with the regional relationships developed between mean annual maximum peak flood and catchment area of the respective Subzones.

(i) Estimation of floods of various return periods for gauged catchments

For estimation of floods of various return periods for the gauged catchments of North Brahmaputra Subzone 2 (a), South Brahmaputra Subzone 2 (b) and the regional flood frequency relationships/ growth factors ($Q_T/\overline{Q}$) given in Table 13 have been developed.

Table 13: Values of growth factors ($Q_T/\overline{Q}$) for North Brahmaputra Subzone 2(a), South Brahmaputra Subzone 2(b) and Western Himalayas Zone7

Subzone	Return Period (Years)					
	2	10	25	50	100	200
2 (a)	0.873	1.866	2.350	2.699	3.038	3.370
2 (b)	0.830	1.955	2.591	3.091	3.613	4.159
Zone 7	0.911	1.753	2.240	2.646	3.097	3.599

For estimation of flood of desired return period for a gauged catchment, estimate the annual mean maxiumum peak flood of the catchment by taking average value of the available annual maximum peak floods and multiply it with the growth factor of the corresponding return period.

(ii) Estimation of floods of various return periods for ungauged catchments

The regional flood frequency relationships developed based on the L-moments approach as mentioned above have been coupled with the regional relationship between mean annual peak flood and catchment area using the least squares approach. For estimation of floods of various return periods for ungauged catchments of Western Himalayas Zone-7, North Brahmaputra Subzone 2 (a) South Brahmaputra Subzone 2 (b) the following form of regional flood frequency relationships/ flood formula has been developed.

$$Q_T = C_T * A^b$$

Where Q_T is the flood estimate in m^3/s for T year return period, and A is the catchment area of the ungauged catchment in km^2, C_T is regional coefficient for various return periods for different regions and "b"is the regional coefficient for different regionas. Values of C_T and "b" and for some of the commonly used return periods are given in Table 14.

Table 14: Values of regional coefficient C_T and coefficient "b" for North Brahmaputra Subzone 2(a), South Bahmaputra Subzone 2(b) and Western Himalayas Zone-7

Subzone	Coefficient "b"	Return Period (Years)					
		2	10	25	50	100	200
		C_T					
2 (a)	1.046	0.883	1.888	2.378	2.731	3.074	3.410
2 (b)	0.840	1.479	3.484	4.617	5.508	6.438	7.411
Zone-7	0.772	5.064	9.745	12.452	14.709	17.216	20.007

The above regional flood formula with its regional coefficients may be used for estimation of floods of various return periods for the Subzones 2(a), 2(b) abd Zone-7. The solutions of the above mentioned regional flood formulae have been provided in tabular forms in Appendices-1, 3 and 3 for different catchment areas and various return periods. The values of floods of various return periods for ungauged catchments of the given sizes of area may be read from the respective Appendices as a guideline.

(d) When no discharge data and rainfall are available, the following two methods can be used to assess the peak flood at site.

(1) Based on Field Information

A study of physical features near the stream at site be made to find the signs of high flood mark which shall be confirmed from local enquiry from senior persons living near the stream and the records of local revenue officials and department of Bridge and Roads. After ascertaining the high flood level, the flood discharge corresponding to this level can be computed by using Manning's equation. Area of river cross section (A) at site and river slope (S) can be obtained by conducting the surveys. The coefficient of Manning 'n' can be assumed on the basis of physical features of river at site for working out the peak flow from the following equations:

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

Where, $R = \frac{A}{P}$, P is the perimeter below high flood mark.

(2) Use of Empirical Formulae

There are several empirical relations to estimate peak flood flows on the basis of catchment area. In India, the most commonly used is Dicken's formulae and is given below:

$$Q = CA^{3/4}$$

Where, C is the coefficient which varies from region to region. On the basis of discharge observations of long periods at various locations the country is divided into regions and each region is assigned a value of C for the use in Dicken's formulae as given in Fig. 6.

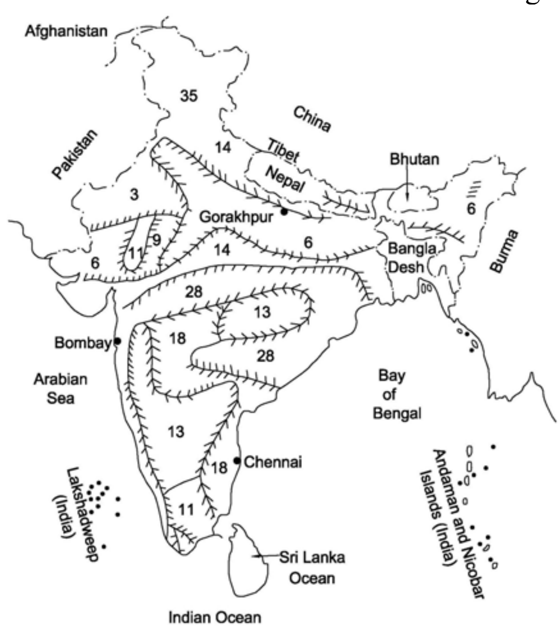


Fig. 6: Dicken's Constant C for Different Parts of India

7.0 SPILLWAY DESIGN FLOOD AND CONSTRUCTION FLOODS

For a run of river project, with no significant storage at river diversion site, peak flood cannot be moderated. Therefore, the water way for the barrage / weir or spill way of low height diversion dams should be provided for the design flood. The design flood is decided on consideration of economic and hydrological factors as well as the safety in the downstream reach. Normally the design flood for barrage / weir is taken as flood of 50 to 100 years return period and standard project flood (SPF) for the spillway of a diversion dam is taken as flood of 100 years.

During construction of a diversion structure the river flow has to be diverted. The diversion arrangement has to be planned and designed for a certain discharge which is always associated with some amount of risk of being exceeded. This again depends on hydrologic, economic factors, the construction sequence and schedule. When the diversion is to be done for non-monsoon flow and monsoon flood can be allowed to pass over in complete barrage / weir, the diversion arrangement can be planned and designed for the maximum non-monsoon flow in the past 10 years. In case the monsoon flood cannot be allowed to pass the incomplete structure the diversion for a SHP project can be planned for a flood a return period of 4 to 5 times the construction period of the project.

8.0 SEDIMENTATION

SHPs generally operate at full installed capacity during monsoon period. During this operation, silt laden water of monsoon flows is diverted into water conductor. It may cause damage to under water components of the generating equipment such as runners, guide vanes etc. resulting in loss of generation and costly repair and maintenance of equipment. The problem is more severe in projects located on Himalayan streams which carry lot of sediment containing quartz during monsoon.

Hence, at the planning stage of a SHP, the characteristics of sediment i.e. size, shape, hardness and concentration, which are site specific, should be assessed with as much accuracy as possible. Sediment sampling at site for concentration, sieve analysis and petrographic analysis (for mineral composition and shape) is essential at the diversion site for their possible removal and sediment characteristics are also important for turbine manufactures.

9.0 WATER QUALITY

Besides the sediment, the chemical analysis of water is important to have details on presence of salts and the nature of water (acidic or alkaline) which will have effect on the metal of gates, penstock and other equipment, and concrete structure. The parameters generally determined in chemical analysis are, Dissolved solids, pH value, Suspended solids, Total hardness, Sulphates, carbonates, bi-carbonates, chlorides, iron, calcium, magnesium and Electrical conductivity.

SECTION -II: INSTALLED CAPACITY FOR SHP

1.0 LOAD SURVEY AND POWER EVACUATION

For isolated/off grid projects, load assessment for power requirement prior to the power potential study is necessary so that installed capacity is sufficient to meet the local demand and growth in power demand for next 5-10 years. The installed capacity should be corresponding to 90% dependable discharge or more, in order to have more reliability in case of isolated schemes. In case, where perennial water is not available, the lower dependability may be chosen but for such plants, a hybrid system in conjunction with biomass, diesel or any other source should be planned. For grid connected schemes, the availability and condition of grid for power evacuation needs to be checked. For an Independent Power Producer (IPP) the plant can be a Captive Plant, a Merchant Plant or as usual to supply the power to a distribution licensee. Hence the load assessment/ grid status has to be ensured accordingly after having studied the power evacuation system plan.

2.0 POWER EQUATION

The equation for Power is as follows

$$P = 9.81\eta QH$$

Where

η = Overall efficiency of turbine, generator and gear box
Q = Discharge in cumec
H = Head available for power generation in m
P = Power in kW

3.0 POWER POTENTIAL STUDIES AND INSTALLED CAPACITY

Prior to the determination of installed capacity, the followings are to be ensured:

i) Applicable efficiencies including part load efficiency of equipment such as turbine, generator and gear box.
ii) Accurate assessment of rated head.
iii) Minimum discharge as environmental flow to be released in the river/stream at diversion structure as per regulations and deducted from the water availability.
iv) Provision has been made for silt flushing discharges and other losses in the water conductor system.
v) Other requirements of water use such as drinking, irrigation, industrial, navigational, recreational, etc., if any are met and deducted from the water availability as applicable.

3.1 Fixation of Installed Capacity

3.1.1 In case discharge data are available for short period say less than 10 years. The steps for fixing installed capacity of a SHP scheme are as follows.

(i) Obtain daily/10-daily average hydrological inflow series (year-wise) in m^3/sec for all hydrological years.
(ii) Deduct environmental flow to have water availability for power generation.
(iii) Arrange discharge data for each year in descending order.
(iv) Calculate average discharge for all the year's data
(v) Calculate % of time corresponding to average discharge values and plot flow duration curve as described in para 5.0 of Section- I.
(vi) Calculate power potential for each discharge value using power equation given in para 2.0.
(vii) Consider number of alternatives of installed capacities in suitable steps say 100, 200, 500, 1000, 1500, 2000, 3000, 4000……… kW .
(viii) Compute incremental energy generation (Δ kWh) for every incremental MW or so (Δ MW) and plot the curve between installed capacity and corresponding annual energy as given in Table 15 and Fig. 7 for a typical project. It is evident from the Fig. 7 that the drop in the curve is sharp at 21 MW; hence the Installed capacity may be around 21 MW. Installed capacity is fixed at a value where the fall in the graph is sharp.
(ix) The alternative for installed capacity, which provides higher energy generation with a acceptable / saleable generation cost as compared to alternate source cost is selected.

For acceptable / saleable generation cost, a cash flow analysis for the life of the plant (may be taken as 35 years) be prepared for different installed capacities of plant. The cash flow series which will give the positive Net Present value (NPV), sufficient Internal Rate of Return (IRR) as per the requirement of developer/financial institutions and permissible generation cost (which will depend on the regulated tariff or the cost of purchase of power for captive use or the cost of sale for Merchant plant etc as the case may be) shall guide the fixation of the Installed capacity.

3.1.2 In case discharge data are available for longer period (say more than 10 years). The steps for fixing installed capacity of a SHP scheme are as follows.

(i) Obtain daily/10-daily average hydrological inflow series (year-wise) in m^3/sec for all hydrological years.
(ii) Compute 90% dependable year. 0.9 $(n+1)^{th}$ year is the 90% dependable year, where n is the number of years for which hydrological inflows data are available.
(iii) Deduct environmental flow from the discharge data of 90% dependable year to have water availability for power generation.
(iv) Arrange discharge data of 90% dependable year in descending order.
(v) Calculate % of time corresponding to average discharge values and plot flow duration curve as described in para 5.0 of Section- I.
(vi) Calculate power potential for each discharge value using power equation given in para 2.0.
(vii) Consider number of alternatives of installed capacities in suitable steps say 100, 200, 500, 1000, 1500, 2000, 3000, 4000……… kW.
(viii) Compute incremental energy generation (Δ kWh) for every incremental MW or so (Δ MW) and plot the curve between installed capacity and corresponding annual energy.
(ix) Installed capacity is fixed at a value where the fall in the graph is sharp.
(x) The alternative for installed capacity, which provides higher energy generation with reasonable generation cost is considered optimum.

Table 15: Installed capacity Vs Incremental Energy

S. No.	Inst Cap.(MW)	Annual Energy (GWh)	Load factor (%)	Incremental Benefit $\Delta MU/\Delta MW$	Potential Exploited (%)
1	10	84.89	96.91		30.12
2	11	91.50	94.95	6.60	32.46
3	12	97.77	93.01	6.27	34.68
4	13	103.83	91.18	6.06	36.83
5	14	109.84	89.57	6.01	38.97
6	15	115.63	88.00	5.78	41.02
7	16	120.84	86.21	5.21	42.87
8	17	125.83	84.50	5.00	44.64
9	18	130.27	82.62	4.44	46.21
10	19	134.60	80.87	4.32	47.75
11	20	138.51	79.06	3.92	49.14
12	21	142.39	77.40	3.88	50.51
13	22	146.01	75.76	3.62	51.80
14	23	149.60	74.25	3.59	53.07
15	24	152.83	72.69	3.23	54.22
16	25	155.94	71.21	3.12	55.32
17	26	159.06	69.84	3.11	56.43

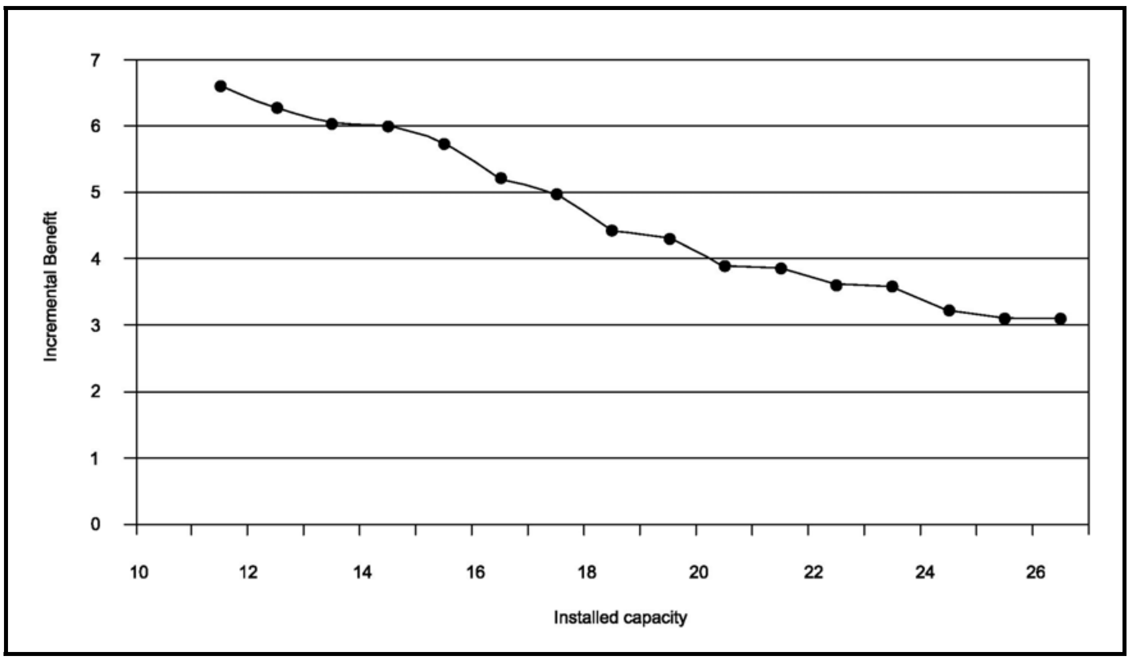


Fig 7: Graphical representation of Installed Capacity Vs Incremental Energy

4.0 SELECTION OF UNIT-SIZE & NUMBER OF UNITS

In general, the steps for selecting size and number of generating units are as under:

(i) Number of generating units should be kept minimum, because the cost of generating units and related equipment and civil works increases with the increase in number of units. Single unit installation has the disadvantage of total loss of power generation in case of unit forced outage, which may not be acceptable in case of isolated / off grid operation units.

(ii) No of units and its size will depend on the discharge requirement of machine, so that it can generate the efficient/ optimum power during lean discharge. In case more than one unit is to be installed in a power house, these should be generally of the same capacity to facilitate inventory management of powerhouse. The superposition of design discharges of an installation with equal unit rating on the flow duration curve of the stream is shown in Fig. 8.

(iii) Unit-size can also be decided based on the transport limitations i.e. maximum size (LxWxH) of package of generating units/transformer which can be transported to the site, if, corresponding access is not available.

(iv) In some cases, there is a tendency to install units of different capacities to suit to the stream flow pattern, but it results in different sizes of units and an increase in inventory. Such a choice is desirable, if there is a substantial accrual of benefit in terms of increased energy generation which can off-set the increased cost of inventory. The superposition of design discharges of an installation with unequal unit ratings on the flow duration curve of the stream is shown in Fig. 9.

(v) In case of run of river schemes without pondage, number of units is decided keeping in view the varying discharge during lean period and turbine operating characteristics.

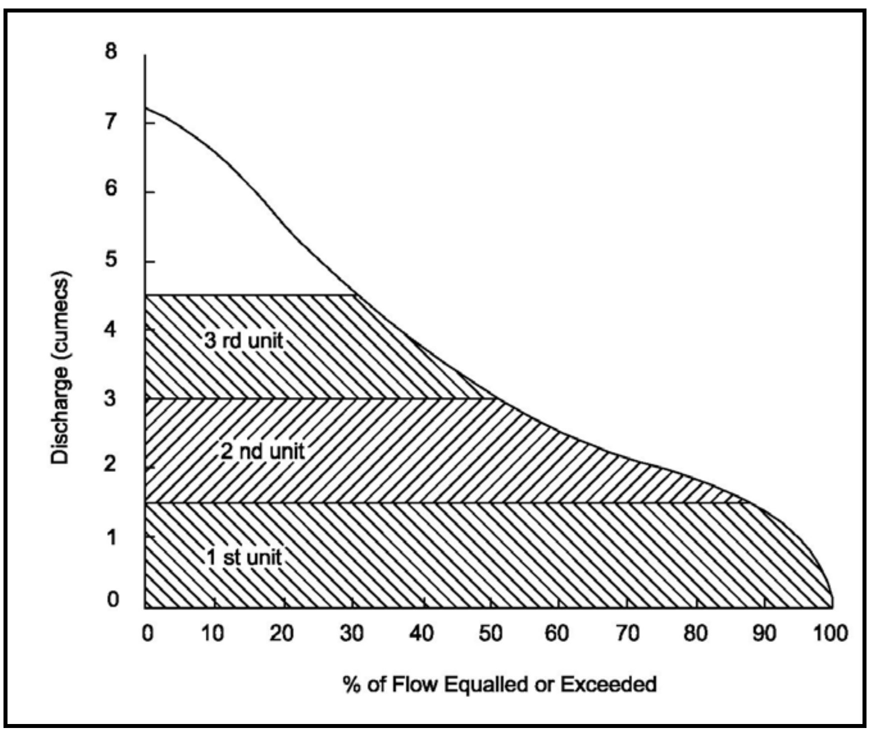


Fig. 8: Equal Sized Turbine Generating Unit

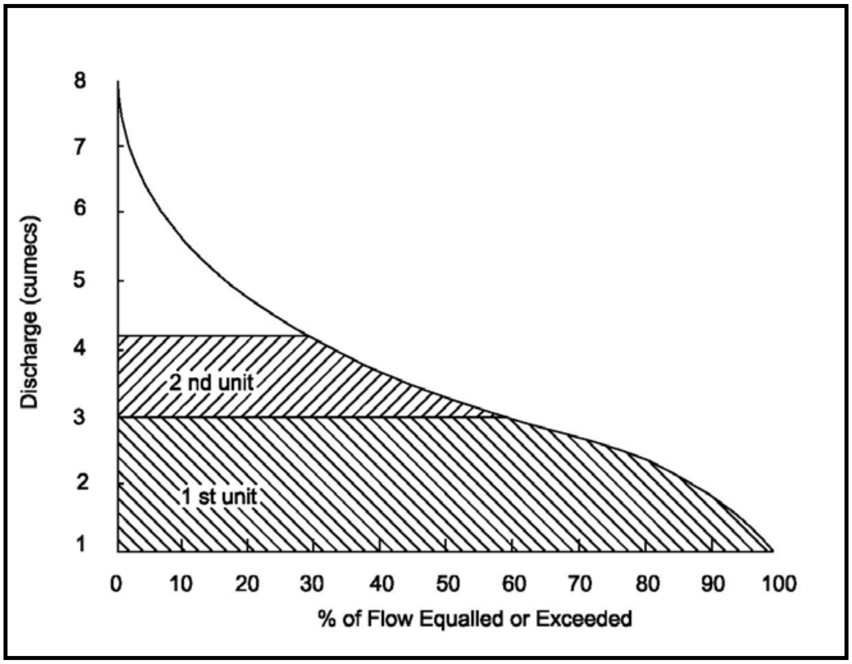


Fig. 9: Unequal Sized Turbine Generating Unit

For specific cases where the installed capacity has been finalized on the basis of monsoon season discharge and there is substantial variation in lean/monsoon discharge, the number of units can be more as compared to the projects where lean season flow is substantial. This shall be governed by the machine requirement of full/partial load in monsoon/lean season. This shall also be governed by the limits of financial analysis as required judiciously by the developer for the sale of power.

APPENDIX-1

Variation of floods of various return periods with catchment area for North Brahmaputra Subzone 2(a)

Catchment Area (km^2)	Return periods (Years)					
	2	10	25	50	100	200
	Floods of various return periods (m^3/s)					
10	9.8	21.0	26.4	30.4	34.2	37.9
20	20.3	43.3	54.6	62.7	70.6	78.3
50	52.9	113.0	142.4	163.5	184.0	204.1
100	109.2	233.4	293.9	337.6	380.0	421.5
200	225.5	481.9	606.9	697.0	784.6	870.3
300	344.6	736.5	927.5	1065.3	1199.0	1330.1
400	465.5	995.1	1253.2	1439.3	1620.0	1797.1
500	587.9	1256.7	1582.6	1817.6	2045.9	2269.5
600	711.4	1520.7	1915.1	2199.5	2475.8	2746.4
700	835.9	1786.8	2250.2	2584.4	2909.0	3226.9
800	961.2	2054.6	2587.5	2971.8	3345.0	3710.6
900	1087.3	2324.0	2926.8	3361.4	3783.6	4197.1
1000	1213.9	2594.7	3267.7	3753.0	4224.4	4686.1
1100	1341.2	2866.7	3610.3	4146.5	4667.3	5177.3
1200	1469.0	3139.9	3954.3	4541.6	5112.0	5670.7
1300	1597.3	3414.1	4299.7	4938.2	5558.4	6165.9
1400	1726.0	3689.3	4646.2	5336.2	6006.5	6662.9
1500	1855.2	3965.4	4993.9	5735.5	6455.9	7161.5
1600	1984.7	4242.3	5342.7	6136.1	6906.8	7661.6
1700	2114.7	4520.0	5692.4	6537.8	7359.0	8163.2
1800	2245.0	4798.5	6043.2	6940.6	7812.4	8666.1
1900	2375.6	5077.7	6394.8	7344.5	8266.9	9170.4
2000	2506.5	5357.6	6747.2	7749.3	8722.6	9675.8
2500	3165.5	6766.1	8521.1	9786.5	11015.7	12219.6
3000	3830.6	8187.7	10311.4	11842.7	13330.2	14787.0
3500	4500.8	9620.3	12115.6	13914.9	15662.6	17374.2
4000	5175.5	11062.3	13931.7	16000.7	18010.4	19978.6
4500	5854.0	12512.7	15758.3	18098.5	20371.8	22598.0
5000	6536.1	13970.6	17594.3	20207.2	22745.3	25230.9

APPENDIX-2

Variation of floods of various return periods with catchment area for South Brahmaputra Subzone 2(b)

Catchment Area (km^2)	Return periods (Years)					
	2	10	25	50	100	200
	Floods of various return periods (m^3/s)					
10	10.2	24.1	32.0	38.1	44.6	51.3
20	18.3	43.2	57.2	68.2	79.8	91.8
50	39.6	93.2	123.5	147.3	172.2	198.2
100	70.8	166.8	221.1	263.8	308.3	354.9
200	126.8	298.6	395.8	472.2	551.9	635.3
300	178.2	419.8	556.4	663.8	775.9	893.2
400	227.0	534.6	708.5	845.3	988.0	1137.3
500	273.8	644.9	854.6	1019.6	1191.7	1371.8
600	319.1	751.6	996.1	1188.3	1389.0	1598.9
700	363.2	855.5	1133.8	1352.6	1581.0	1820.0
800	406.3	957.1	1268.4	1513.2	1768.7	2036.0
900	448.6	1056.6	1400.3	1670.6	1952.7	2247.8
1000	490.1	1154.4	1529.9	1825.2	2133.4	2455.8
1100	531.0	1250.6	1657.5	1977.3	2311.3	2660.5
1200	571.2	1345.5	1783.2	2127.3	2486.5	2862.3
1300	611.0	1439.1	1907.2	2275.3	2659.5	3061.4
1400	650.2	1531.5	2029.7	2421.4	2830.3	3258.1
1500	689.0	1622.9	2150.8	2565.9	2999.2	3452.5
1600	727.4	1713.3	2270.7	2708.9	3166.3	3644.8
1700	765.4	1802.8	2389.3	2850.4	3331.8	3835.3
1800	803.0	1891.5	2506.9	2990.6	3495.7	4023.9
1900	840.4	1979.4	2623.3	3129.6	3658.1	4210.9
2000	877.4	2066.6	2738.9	3267.4	3819.2	4396.3
2500	1058.3	2492.7	3303.6	3941.1	4606.6	5302.8
3000	1233.4	2905.3	3850.4	4593.4	5369.1	6180.5
3500	1404.0	3306.9	4382.7	5228.5	6111.5	7035.1
4000	1570.6	3699.5	4903.0	5849.2	6837.0	7870.2
4500	1734.0	4084.3	5413.0	6457.6	7548.1	8688.8
5000	1894.5	4462.3	5914.0	7055.2	8246.7	9493.0

APPENDIX-3

Variation of floods of various return periods with catchment area for Western Himalayas-Zone 7

Catchment Area (km^2)	Return periods (Years)					
	2	10	25	50	100	200
	Floods of various return periods (m^3/s)					
10	30.0	57.6	73.7	87.0	101.8	118.4
20	51.2	98.4	125.8	148.6	173.9	202.1
50	103.8	199.7	255.2	301.4	352.8	410.0
100	177.2	341.0	435.8	514.7	602.5	700.1
200	302.6	582.3	744.1	879.0	1028.8	1195.6
300	413.9	796.4	1017.6	1202.1	1406.9	1635.0
400	516.8	994.4	1270.7	1501.0	1756.8	2041.6
500	613.9	1181.4	1509.6	1783.2	2087.1	2425.4
600	706.7	1359.9	1737.7	2052.7	2402.5	2792.0
700	796.0	1531.8	1957.3	2312.1	2706.2	3144.8
800	882.5	1698.1	2169.9	2563.1	3000.0	3486.3
900	966.5	1859.7	2376.4	2807.1	3285.6	3818.2
1000	1048.4	2017.3	2577.8	3045.0	3564.0	4141.7
1100	1128.4	2171.4	2774.6	3277.5	3836.1	4457.9
1200	1206.8	2322.2	2967.4	3505.2	4102.7	4767.7
1300	1283.7	2470.3	3156.5	3728.6	4364.2	5071.6
1400	1359.3	2615.7	3342.4	3948.2	4621.1	5370.2
1500	1433.7	2758.8	3525.2	4164.2	4873.9	5664.0
1600	1506.9	2899.7	3705.3	4376.9	5122.9	5953.3
1700	1579.1	3038.7	3882.9	4586.6	5368.4	6238.6
1800	1650.4	3175.8	4058.0	4793.5	5610.6	6520.0
1900	1720.7	3311.1	4231.0	4997.9	5849.7	6797.9
2000	1790.2	3444.9	4401.9	5199.7	6086.0	7072.5
2500	2126.8	4092.5	5229.4	6177.3	7230.2	8402.1
3000	2448.2	4711.0	6019.8	7110.9	8322.9	9672.0
3500	2757.6	5306.4	6780.6	8009.5	9374.7	10894.3
4000	3057.1	5882.6	7516.8	8879.3	10392.7	12077.3
4500	3348.1	6442.6	8232.4	9724.5	11382.0	13226.9
5000	3631.8	6988.5	8930.0	10548.5	12346.5	14347.7