GOVERNMENT OF INDIA

CENTRAL WATER COMMISSION

GUIDELINES FOR PREPARATION OF DPR FOR FLOOD MANAGEMENT WORKS

FLOOD MANAGEMENT ORGANISATION

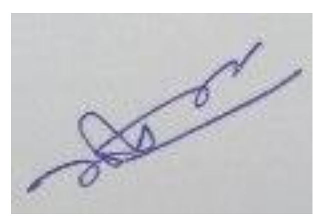
APRIL 2018

Member (River Management) Central Water Commission
&
Ex-Officio Additional Secretary to the Government of India

Foreword

India is one of the most flood-affected countries in the world in terms of affected geographical area. There is not a single year when some or the other parts of the country gets inundated in flood water. How best to cope with floods is an age-old problem. It is a natural disaster. One way is to accept it as inevitable and learn to live with it in the best possible manner. But unlike other natural disasters such as earthquakes it is possible to manage flood to a great extent. As widely known that there are two options for flood management ie. structural & non-structural and modern flood management strategy is a judicious mixture of both of these options.

These guidelines for preparation of Detailed project report for flood management works primarily deals with structural part of flood management. These guidelines deliberated on the outlines of the detailed project report for flood management works for timely appraisal and clearance to provide relief to the flood affected people. I am sure that these guidelines would be of great use for State Engineers at different levels in flood management works.



Chief Engineer(Flood Management)
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Preface

India has peculiar geographical features experiencing flood in some parts and drought in other parts and sometimes they co-exist. India has made huge investment in flood control sector since 1951 in implementing number of flood management schemes which has undoubtedly provided great relief to a large population against floods. The Engineers involved in framing the project report and subsequent implementation for flood protection, anti-erosion and river training works need a handbook for having a comprehensive view of design principles, construction techniques and costing thereof. These guidelines containing various chapters on different aspects of project report would provide a great help for preparation of detailed project report for flood management works.

I place on record the outstanding efforts made by officials of Flood Management Planning Dte especially Sh Piyush Kumar, Director along with other officials in preparation of this handbook. Any suggestion for improvement of the contents will be highly appreciated.

INDEX-OUTLINE OF DPR

Chapter 1: Foreword

1.1 Foreword duly signed by Head of the Department should be attached in the DPR.

1.2 Foreword may contain a brief summary about the master plan of the area, proposed project features along with its scope.

CHAPTER 2: SALIENT FEATURES OF THE PROJECT

SN	ITEMS		Remarks
1	Name of Work	:	
2	Estimated cost & Price level		
3	Reference of State TAC clearance		
4	Master Plan for the basin, fitment of the project and priority		
5	Name of State		
6	Name of District		
7	Name of Basin/sub-basin		
8	Name of rivers/tributaries		
9	Latitude & Longitude of the project		
10	Nearest GD site & its latitude & longitude		
11	Distance along with direction from nearby major district(HQ)/town		
12	B.C. Ratio		
13.	Benefitted population in nos		
14.	Benefitted area in ha		
15	Flood affected area of the State in mha		
16	Protected area of the State; so far in mha		
17.	Details of proposed works along with reach length		
17.1	Embankments		
17.2	Sluices		
17.3	Revetment		
17.4	Retaining wall		
17.5	RCC porcupines		
17.6	Re-sectioning/dredging		
17.7	others		
18	Completion Schedule		
19	Details of already executed works		

CHAPTER 3. Executive Summary of the project

3.1 Introduction

3.2 Location of the project:

3.3 Description of the problem of the area along with the last five years satellite imageries of flood effect

3.4 Details of earlier executed/ongoing works

3.5 Master Plan for the basin, fitment of the project and priority:

3.6 Non- structural measures

3.7 Survey/Investigation conducted in formulation of the scheme:

3.8 Various alternatives mentioning best national and international cost effective practices & present proposal:

3.9 Scope of the project:

3.10 Design featues:

3.11 Basis of rates and cost estimate

3.12 Benefit cost Ratio:

3.13 Constrcution materials and Constrcution program:

3.14 Socio-Economic Aspects:

3.15 Environtal Aspects:

3.16 Recommendations:

3.17 Pre project and post project scenario description with satellite imageries:

CHAPTER 4. Prioritization of schemes

4.1 A description of the project area likes important roads, railways lines , important/vital installations, agriculture, industries, villages/towns, habitation etc may be given in the DPR.

4.2 Long term strategy as well as short term plans in conformity with Master Plan of the basin for providing relief to the flood affected area may be given.

4.3 Socio-economic impact and capacity building of habitation due to implementation of the project may also be discussed.

4.4 Status and brief of earlier executed works viz embankments, raised platforms, drainage development works, anti erosion measures etc along with their cost at current year price level may also be given.

4.5 Annual maintenance cost, actual expenditure along with annual allocation of budget for such works may also be given.

4.5 The scheme should be prioritized on the basis of its features like impact on social life, benefits and protection to the area per unit cost etc.

CHAPTER 5. Hydrology of the project

5.0 GENERAL

The economics and sizing of every structural and non-structural component of the project is dependent on hydrology. Hydrological inputs play a major role in planning, designing, execution and successful operation of a water resources development project. The Hydro-Meteorological data is a basic input to finalize the hydrological design parameters of any hydraulic structure. Estimation of design flood for the design of hydraulic structures is an important component of hydrological studies. The higher value results in increase of the cost of hydraulic structures, an under estimated value likely to place the structure and population involved, at risk. The design flood study is an integral and significant part of safety of any hydraulic structure.

5.1 DESIGN FLOOD SELECTION CRITERIA

As per CWC handbook "Flood Protection, Anti erosion and River Training Works" published in year 2012, design flood selection criteria shall be as follows:

5.1.1 Embankment for predominantly agricultural areas.

The design flood for this type of embankment is kept 25 years for fixation of crest level.

5.1.2 Embankments for township or areas having industrial installations

The design flood for this type of embankment is kept 100 years for fixation of crest level. In the cases where anti erosion measures are proposed along with the embankment then design flood may be kept as 50 years for rural areas and 100 years for urban/industrial areas. In certain special cases, where damage potential justifies, maximum observed flood may also be considered for fixing the crest level.

5.3 BASIC HYDROLOGICAL DATA REQUIREMENT AND COMPILATION

The DPR shall contain the following basic information/data:

(a) Geo-spatial details of various hydrological observations sites maintained by the CWC/States/Other agencies within the drainage area along with their locations.

(b) Details of specific data collected with method of observation along with summary.

(c) The long terms Gauge and Discharge data of nearby sites and hydro-meteorological (rainfall) data within the drainage area.

(d) Annual peak discharges and levels for all G&D sites in and around of project at upstream and downstream shall be furnished for entire period of record. Instantaneous flood peak series may be prepared from hourly water level records available at G&D site for frequency analysis.

(e) Drainage area map showing location detail of all observation sites, proposed location of embankment, location of rain gauge stations, G&D sites and important hydraulic structures, etc.

5.4 DESIGN FLOOD COMPUTATIONS

For selection of design flood, the relevant BIS codes/guidelines for the specific structure shall be referred. For design flood estimation, procedure recommended in CWC's "Manual on Estimation of Design Flood" and other guidelines on the subject shall also be referred. For flood management schemes, design flood of appropriate return period shall be computed using following approaches:

(a) Flood Frequency Analysis
(b) Hydro-meteorological Approach

5.5 FLOOD FREQUENCY ANALYSIS

To estimate the design flood using flood frequency approach, the following procedures shall be adopted:

(a) The flood peak series shall be checked for randomness, homogeneity, trend, jump, outliers etc using appropriate statistical methods.

(b) Flood frequency analysis shall be carried out using time series of instantaneous annual flood peak. Based on the hourly gauge data the observed annual flood peak shall be converted into instantaneous flood peak.

(c) Using the instantaneous annual flood peak time series, the flood frequency analysis shall be carried out using standard frequency distributions such as Gumbel, log Pearson type-III and Log Normal distributions etc. to estimate the desired return period flood.

(d) Goodness of fit test for the frequency distribution shall be carried out using standard statistical tests such as Chi Square, D-Index etc. to assess the appropriate frequency distribution for the data set and decide the appropriate design flood.

5.6 HYDRO-METEROLOGICAL APPROACH

Where the observed flood peak series is not available, desired return period flood shall be estimated by hydro-meteorological approach using the following procedure:

(a) The rainfall of appropriate return period shall be assessed using standard meteorological approaches and tools such as isopluvial maps published by IMD / PMP Atlas of CWC.

(b) The drainage area representative observed concurrent short interval rainfall and runoff data of 4 to 5 flood events shall be collected to develop the catchment response function / unit hydrograph.

(c) Where the observed concurrent short interval rainfall and runoff data is not available, the flood estimation reports for different subzones published by CWC can be used to develop synthetic unit hydrograph.

(d) Infiltration loss rate, base flow and hourly rainfall distribution coefficients of rainfall can be assessed from the relevant reports / flood estimation reports.

(e) Critical sequencing and convolution shall be carried out as per standard procedure. In this regard Manual on Estimation of Design Flood published by Central Water Commission in March, 2001 may be followed.

(f) When the catchment area is very small and unit hydrograph may not be derived, rational formula may be used for design flood estimation.

5.7 WATER SURFACE PROFILE

Water level corresponding to estimated design flood shall be computed using gauge and discharge data of nearest G&D site. If gauge and discharge data is not available, the water surface profile for the estimated design flood shall be computed using hydrodynamic study on appropriate hydrodynamic model such as HEC-RAS, Mike11 etc. For hydrodynamic simulation, sufficient number of surveyed river cross sections shall be used to represent the study river reach.

In case where upstream reservoirs are existing and some flood moderation is possible from these reservoirs, the same should be taken into consideration while computing the water surface profile.

Some illustrations for hydrological analysis are given at annex-5.1

Annex: 5.1

Example-1: Estimation of 100 year return period flood for Rukni river at a proposed water resources project location

1.0 Introduction

A water resources project is proposed on river Rukni, a principal tributary of Barak river at latitude $24^0 29'26''$ N and longitude $92^0 48'15''$ E. The entire catchment of the river is rainfed. The catchment area of the river at proposed project site is 731 sq.km. The 100 year return period flood for the scheme has been estimated using hydro-meteorological approach.

1.1 Physiographic parameters

The physiographic parameters of the river catchment at proposed flood protection site has been estimated by GIS processing of ASTER DEM. The catchment area obtained from the GIS is about 731 sq.km. The catchment area map at the proposed project site comprising of elevation band, drainage/catchment area at diversion site, longest flow path (L), Centroidal longest flow path (Lc) is given at Figure-1.1. The estimated parameters of the river catchment at proposed site are given in Table-1.1. The same have been utilised for working out the unit hydrograph from the relevant Flood Estimation Report (FER) of CWC.

Table-1.1: Sub-basin parameters

Catchment area (km^2)	L (km)	Lc (km)	Equivalent stream slope (m/km)
731	81.55	37	2.1

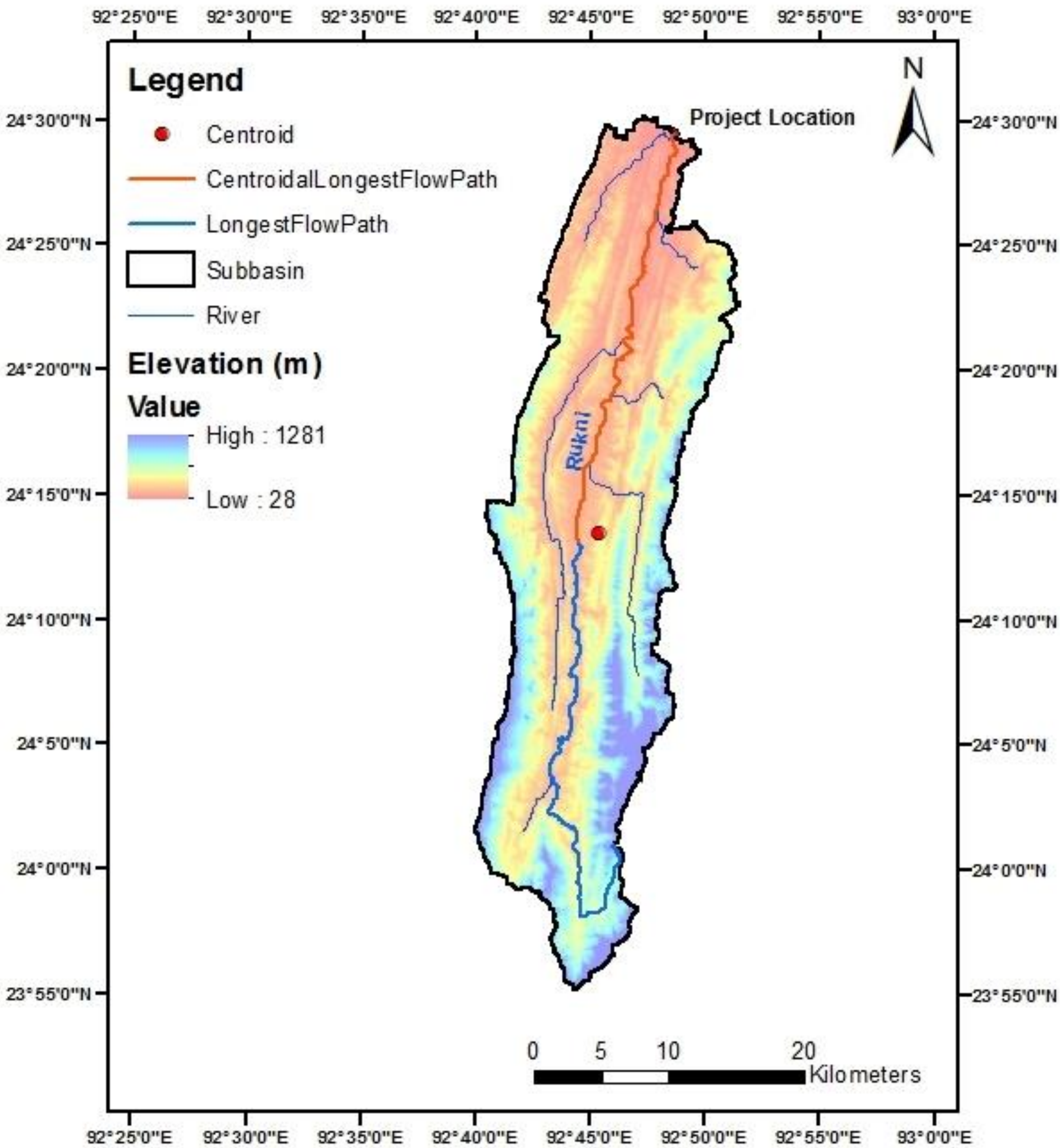

Fig-1.1: Catchment Area map at Project location

1.2 Unit hydrograph

Since, drainage area representative concurrent rainfall and discharge data at short interval is not available, the unit hydrograph (UH) for drainage area has been worked out using Flood Estimation Report FER-2(b) of South Brahmaputra. The estimated UH parameter are given in **Table-1.2**. The unit hydrograph is given in **Table-1.3**.

Table-1.2: Unit hydrograph parameters

q_P (cumec)	0.18
t_P (hr)	11.91
W_{50} (hr)	13.33
W_{75} (hr)	7.03
WR_{50} (hr)	4.57
WR_{75} (hr)	2.54
T_B (hr)	42.98
Q_P (cumec)	134.12
T_m (hr)	10.00

Table-1.3: Unit hydrograph for project catchment

Time (hr)	Discharge (cumec)	Time (hr)	Discharge (cumec)	Time (hr)	Discharge (cumec)
0	0	15	94	30	20
1	4	16	84	31	18
2	10	17	77	32	16
3	18	18	70	33	14
4	32	19	64	34	12
5	52	20	58	35	10
6	72	21	53	36	9
7	92	22	48	37	7
8	115	23	43	38	6
9	129	24	39	39	4
10	134	25	35	40	3
11	132	26	31	41	2

12	128	27	28	42	1
13	116	28	25	43	0
14	104	29	22		

1.3 100 year return period rainfall

From the Isopluvial map of IMD, the 24 hr, 100 year return period point rainfall has been adopted as 28 cm. The same has been converted in areal rainfall by multiplying with appropriate areal reduction factor as per FER-2(b).

For hourly distribution of rainfall normalized distribution coefficient has been worked out for bell of 12 hours using the hourly distribution coefficient. The hourly distribution coefficient of 24 hour rainfall as obtained from IMD and normalized distribution coefficient for 12 hour bell are given in **Table-1.4**.

Table-1.4: Hourly distribution coefficient of 24 hour rainfall for first 12 hour and normalized distribution coefficient 12 hour bell

Time (hr)	Distribution coefficient for 24 hour rainfall (%)	Normalised Distribution coefficient for 12 hour bell (%)
1	19	26
2	29	40
3	36	50
4	41	57
5	45	63
6	49	68
7	53	74
8	57	79
9	61	85
10	65	90
11	69	96
12	72	100
13	76	
14	79	
15	82	
16	84	
17	86	
18	88	
19	90	
20	92	
21	94	
22	96	
23	98	
24	100	

1.4 Loss rate and base flow

A design loss rate of 0.35 cm /hour and design base flow of 0.05 cumec/sq.km as per FER-2(b) has been adopted.

1.5 Critical sequencing of rainfall

Hourly distribution of 100 year rainfall of each bell is given in Table-1.5. Critical sequencing of hourly effective rainfall of each bell is given in Table-1.6. The reverse of critically sequenced effective rainfall has been used for convolution to get 100 year return period.

Table-1.5: Hourly distribution of 100 year rainfall

24 hr point rainfall as per Isopluvial map of IMD	**28 cm**
Areal reduction factor (ARF)	**0.852**
24 hr areal rainfall (ARF x 24 hr point rainfall)	**23.86 cm**
Depth 1st 12 hr bell (0.72 x 23.86)	**17.18 cm**
Depth 2nd 12 hr bell (0.28 x 23.86)	**6.68 cm**

Time (hr)	Dist coeff	Normalised dist coeff	**Cumulative rainfall depth**		**Incremental rainfall depth**		Loss rate 0.35 cm/hr	**Effective rainfall depth**	
			1st 12 hr bell	2nd 12 hr bell	Incremental rainfall 1st bell	Incremental rainfall 2nd bell		Effective rainfall 1st bell	Effective rainfall 2nd bell
	(%)	(%)	(cm)	(cm)	(cm)	(cm)		(cm)	(cm)
1	19	26	4.53	1.76	4.53	1.76	0.35	4.18	1.41
2	29	40	6.92	2.69	2.39	0.93	0.35	2.04	0.58
3	36	50	8.59	3.34	1.67	0.65	0.35	1.32	0.30
4	41	57	9.78	3.80	1.19	0.46	0.35	0.84	0.11
5	45	63	10.74	4.17	0.95	0.37	0.35	0.60	0.02
6	49	68	11.69	4.55	0.95	0.37	0.35	0.60	0.02
7	53	74	12.64	4.92	0.95	0.37	0.35	0.60	0.02
8	57	79	13.60	5.29	0.95	0.37	0.35	0.60	0.02
9	61	85	14.55	5.66	0.95	0.37	0.35	0.60	0.02
10	65	90	15.51	6.03	0.95	0.37	0.35	0.60	0.02
11	69	96	16.46	6.40	0.95	0.37	0.35	0.60	0.02
12	72	100	17.18	6.68	0.72	0.28	0.35	0.37	0.00

Table-1.6: Critical sequencing for effective hourly rainfall

Time	UH ordinate	Critical sequence of hourly effective 100 yr rainfall		Reversed sequence of hourly effective 100 yr rainfall	
		1st 12 hr bell (B1)	2nd 12 hr bell (B2)	1st 12 hr bell (B1)	2nd 12 hr bell (B2)
(hr)	(cumec)	(cm)	(cm)	(cm)	(cm)
3	18				

4	32				
5	52				
6	72	0.37	0.00	0.60	0.02
7	92	0.60	0.02	0.60	0.02
8	115	0.60	0.02	0.60	0.02
9	129	1.32	0.30	0.60	0.02
10	134	4.18	1.41	0.60	0.02
11	132	2.04	0.58	0.84	0.11
12	128	0.84	0.11	2.04	0.58
13	116	0.60	0.02	4.18	1.41
14	104	0.60	0.02	1.32	0.30
15	94	0.60	0.02	0.60	0.02
16	84	0.60	0.02	0.60	0.02
17	77	0.60	0.02	0.37	0.00
18	70				
19	64				
20	58				

Sequence used for convolution B2-B1

1.6 100 year return period flood

The reverse sequence of hourly effective rainfall as given in Table-1.6 has been convoluted with the ordinates of unit hydrograph to get the 100 year direct runoff hydrograph. The base flow contribution has been added to get the 100 year flood hydrograph at proposed diversion site of Rukni Irrigation Project. The estimated 100 year flood is about 1700 cumec. The 100 year flood hydrograph is given in Table-1.7. A plot of the same is given in Figure-1.2.

Table-1.7: 100 year flood hydrograph for Rukni Irrigation Project

Time (hr)	Discharge (cumec)	Time (hr)	Discharge (cumec)	Time (hr)	Discharge (cumec)
0	37	25	1284	50	248
1	37	26	1446	51	222
2	37	27	1592	52	198
3	37	28	1679	53	174
4	38	29	1700	54	153
5	39	30	1671	55	134
6	41	31	1602	56	115
7	46	32	1489	57	99
8	58	33	1364	58	83
9	76	34	1245	59	70
10	101	35	1130	60	60
11	138	36	1028	61	50
12	186	37	937	62	42
13	237	38	854	63	39
14	292	39	777	64	38
15	350	40	707	65	37
16	399	41	641	66	37

17	440	42	580		
18	479	43	526		
19	527	44	474		
20	594	45	426		
21	683	46	383		
22	796	47	343		
23	942	48	307		
24	1115	49	276		

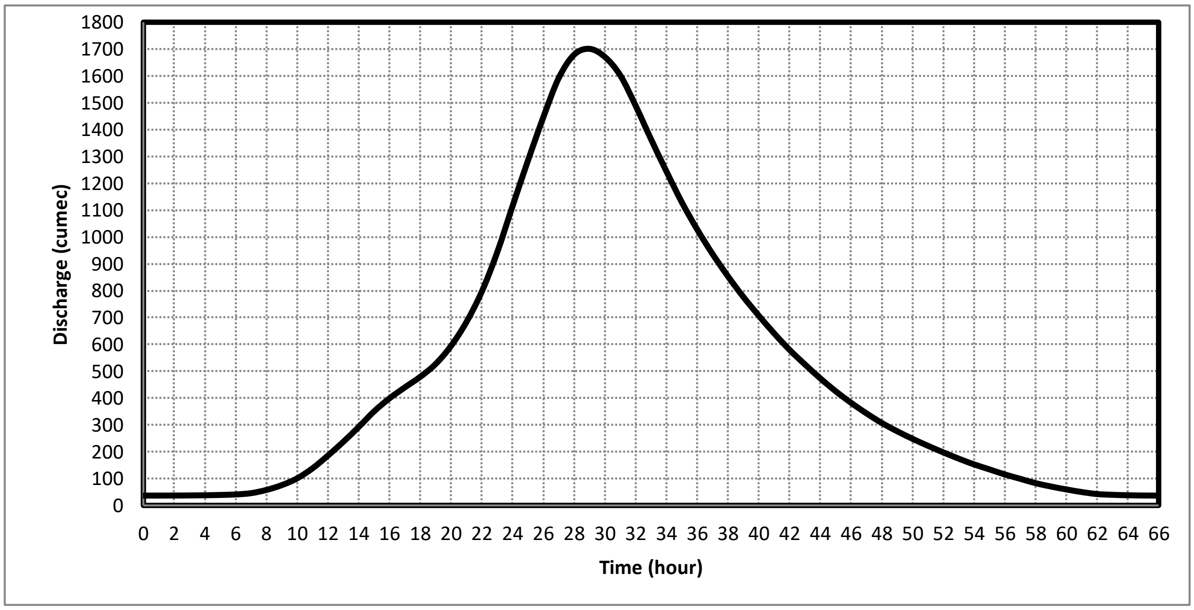

Fig-1.2: 100 yr flood hydrograph for Rukni Irrigation Project

Example-2: Estimation of water surface profile for 100 year return period flood in a river reach of 5 km near proposed project location in Rukni river

2.0 Water surface profile in Rukni river for 100 year return period flood

For estimating the maximum water surface profile corresponding to 100 year flood, the necessary flow simulation has been carried out using one dimensional mathematical model HEC-RAS. The study reach of Rukni river for 3 km upstream and 2 km downstream of proposed project location rivers have been represented in the model by river cross sections taken at an interval of 500 m. Another cross section has also been taken at 2.5 km downstream of proposed project site to apply downstream boundary at that location. The Manning's n for the study reach has been adopted as 0.030.

The upstream boundary of HEC-RAS model set up has been adopted as 1700 cumec constant discharge corresponding to 100 yr return period flood. The downstream boundary has been adopted as normal depth and applied at the downstream most river cross section. The HEC-RAS model set up for one of the river viz Kokila is shown in Fig.2.1.

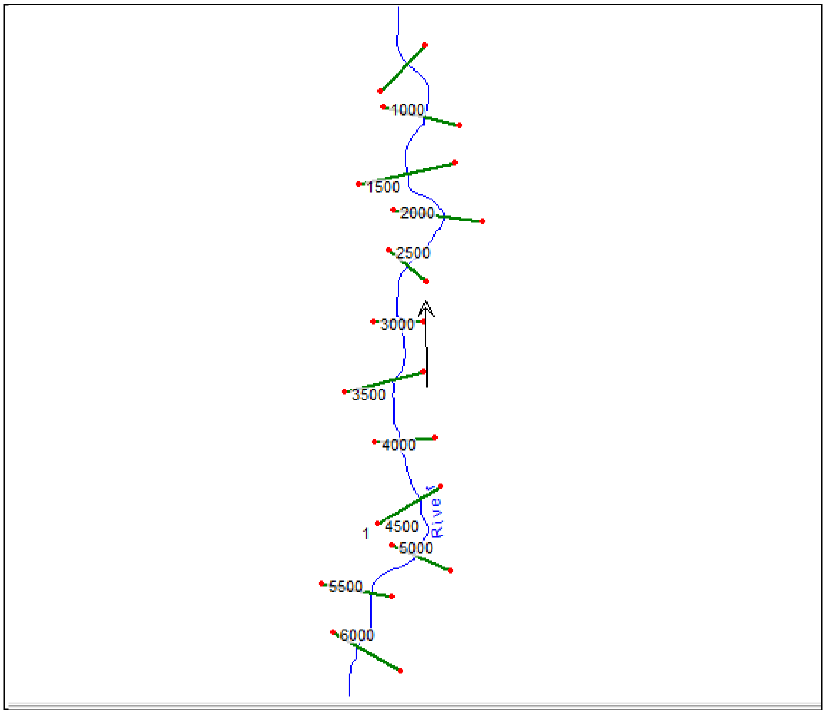

Fig.2.1: HEC-RAS model set up for water surface profile study of Rukni river

The water surface profile of Rukni river for 100 yr flood as obtained from HEC-RAS simulation is presented in Figure-2.2. The maximum water level at different locations of the river reach is given in Table-2.1.

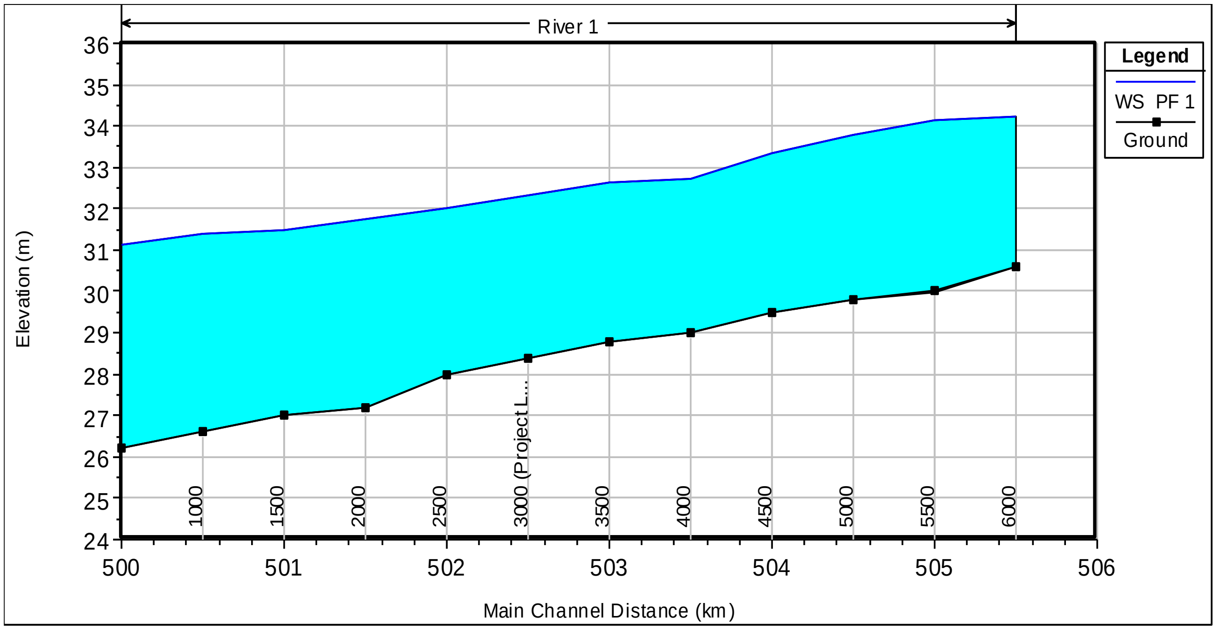

Fig.2.2: Bed and water surface profile of Rukni river for 100 yr flood

Table-2.1: Bed level, maximum water level for 100 yr flood in Rukni river

Reach	River Sta	Profile	Q Total	Min Ch El (Bed level)	W.S. Elev	Vel Chnl	Flow Area	Top Width
			(m3/s)	(m)	(m)	(m/s)	(m2)	(m)
1	6000	PF 1	1700	30.6	34.23	3.12	544.69	229.34
1	5500	PF 1	1700	30	34.13	1.31	1294.32	428.88
1	5000	PF 1	1700	29.81	33.77	1.76	966.09	444.51
1	4500	PF 1	1700	29.5	33.36	1.68	1011.58	390.07
1	4000	PF 1	1700	29	32.72	2.38	715.15	268.6
1	3500	PF 1	1700	28.8	32.64	1.07	1583.61	527.54
1	3000	PF 1	1700	28.4	32.32	1.86	912.3	314.67
1	2500	PF 1	1700	28	32.02	1.75	972.76	276.59
1	2000	PF 1	1700	27.2	31.74	1.77	961.19	303.03
1	1500	PF 1	1700	27	31.49	1.41	1203.59	486.88
1	1000	PF 1	1700	26.6	31.42	0.9	1893.67	543.43
1	500	PF 1	1700	26.21	31.11	1.88	902.3	377.31

Example-3: Estimation of 25 year return period flood using flood frequency analysis

3.0 Introduction

In a river observed flood peak data is available at a nearby G&D site for the period 1990-91 to 2009-10. The catchment area at G&D site is 2350 sq.km. Flood protection is to be provided for some villages. The catchment area of river near proposed flood protection scheme is 1800 sq.km. The flood computations at G&D site has been carried out using probabilistic approach (flood frequency analysis) and the estimated return period flood at G&D site has been transposed at proposed project site using Dicken's formula. The observed annual flood peaks at G&D site, given in Table-3.1 has been subjected to various statistical tests to check its randomness and presence of outliers, homogeneity etc.

Table-3.1: Observed annual flood peak at G&D site (Catchment area 2350 sq.km)

YEAR	Maximum Daily observed Flood Peak (cumec) at Bhalukpong	YEAR	Maximum Daily observed Flood Peak (cumec) at Bhalukpon
1990-91	2190	2000-01	1073
1991-92	2335	2001-02	1430
1992-93	2463	2002-03	1468
1993-94	2391	2003-04	2231
1994-95	1277	2004-05	3679
1995-96	1487	2005-06	2478
1996-97	2016	2006-07	1089
1997-98	1160	2007-08	2020
1998-99	1432	2008-09	2150
1999-00	1781	2009-10	1224

3.1 Statistical characteristics of Observed flood peak series

The various statistical characteristics of the observed flood peaks series are given below:

Mean, Xm =1868.65	SD, Sx = 651.19	Skewness = 0.98	Kurtosis = 1.52

3.2 Test for randomness

To test the randomness of the flood peak series, the observed annual flood peaks of different years has been plotted in Fig-3.1 and total number of troughs and peaks counted. The randomness has been tested by **turning point test** as given below:

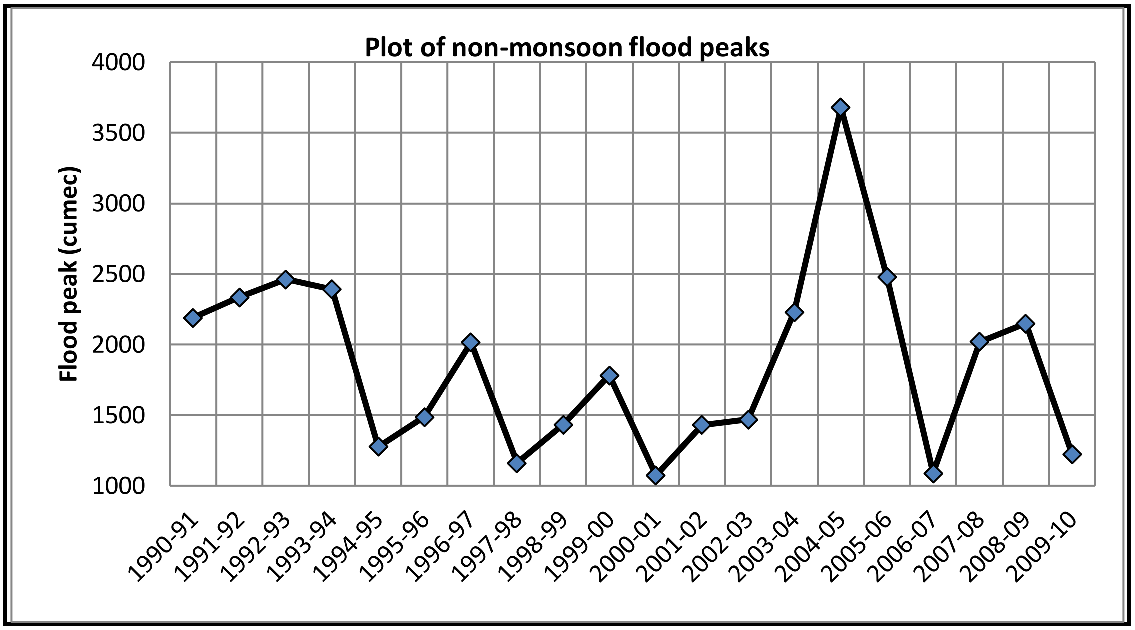

Fig-3.1: Plot of observed non-monsoon flood peak

Test statistics, $z = (p-E(p)) / \sqrt{var(p)}$

For $|z| < 1.96$, the series can be considered random at 5% significance level

p = number of turning points = sum of peaks and troughs of plot of Fig-3.1 = 9

N = Number of data points = 20

$E(p) = 2(N-2)/3 = 2x18/3 = 12$

$Var(p) = (16N-29)/90 = (16x20-29)/90 = 3.23$

$z = (9-12)/\sqrt{3.23} = -1.67$

As $|z|$ < 1.96, Flood peak series may be considered random at 5% significance level.

3.3 Outlier test

Outliers are the points, departing significantly from the trend of the remaining data of the sample. The observed annual flood peak series at G&D site has been tested for high and low outliers as given below:

Mean of log transformed observed flood peaks, x_m = 7.478

Standard deviation of log transformed flood peaks, s= 0.337

10% significance level outlier test, K_N = 2.385 for sample size of 20

Higher outlier, $Q_H = exp\ (x_m + K_N * s) = 3954$, which is greater than the highest observed peak of 3679 cumec. Hence there are no high outliers in the series

Low outlier, $Q_L = exp\ (x_m - K_N * s) = 792$, which is less than lowest observed peak of 1073 cumec. Hence there are no low outliers in the series

3.4 Homogeneity test

Homogeneity of data has been tested by t-test. The t-test (or *student's t-test*) gives an indication of the separateness of two sets of measurements, and is thus used to check whether two sets of measures are essentially different. It is used when there is random assignment and only two sets of measurement to compare. The t-test is a basic test that is limited to two groups. For the present case the two groups, one for the measured non-monsoon peaks from 1990-1991 to 1999-2000 and other for the measured non-monsoon peaks from 2000-2001 to 2009-2010 have been tested to test the null hypothesis that the means of the two groups are equal. The test results are given in Table-3.2. Base on the test results where t Stat < t Critical, it can be said the non-monsoon flood peak series is homogeneous at 5% significance level.

Table-3.2: t-Test: Paired Two Sample for Means

	Variable 1	*Variable 2*
Mean	1853.20	1884.10
Variance	240267.96	654426.16
Observations	10	10
Pearson Correlation	-0.59	
Hypothesized Mean Difference	0.00	
df	9.00	
t Stat	**-0.08**	
P(T<=t) one-tail	0.47	
t Critical one-tail at 5% significance level	**1.83**	
P(T<=t) two-tail	0.94	
t Critical two-tail at 5% significance level	**2.26**	

3.5 Flood Frequency Analysis

The observed annual flood peaks have been made instantaneous on the basis of hourly gauge data. From the observed hourly gauge data it has been found that the observed flood peaks were about 15% less than the maximum discharge. Hence the observed flood peaks have been multiplied with a factor of 1.15 to make them instantaneous. The same are given in Table-3.3.

Table-3.3: Observed and Instantaneous annual flood peaks G&D site

Year	Observed annual flood peak at G&D site (cumec)	Instantaneous flood Peak at G&D site (cumec)
1990-91	2190	2518.50
1991-92	2335	2685.25
1992-93	2463	2832.45
1993-94	2391	2749.65
1994-95	1277	1468.55
1995-96	1487	1710.05
1996-97	2016	2318.40
1997-98	1160	1334.00
1998-99	1432	1646.80
1999-00	1781	2048.15
2000-01	1073	1233.95
2001-02	1430	1643.93
2002-03	1468	1688.20
2003-04	2231	2565.17
2004-05	3679	4230.98

2005-06	2478	2849.35
2006-07	1089	1252.09
2007-08	2020	2323.45
2008-09	2150	2472.50
2009-10	1224	1407.49

The 25 year return period flood has been estimated by fitting Log normal, Log Pearson Type-3 (LPT-3) and Extreme value Type-I i.e. Gumbel distributions on instantaneous flood peak series. The estimated floods are given in Table-3.4.

Table-3.4 : 25 year return period flood at G&D site

Distribution	**25 year non-monsoon flood at Bhalukpong (cumec)**	**Chi Square (X^2) value**	**Chi Square Critical value at 5% significance level**
Log Normal	3672	3.50	5.99
LPT-3	3747	2.50	5.99
Gumbel	3677	3.50	5.99

From the above table it can be seen that all the three estimates are quite close with the maximum value of 3747 cumec corresponding to LPT-3 distribution. Further LPT-3 has been found the best fitting distribution with minimum value of Chi Square. Hence, 25 year return period flood at G&D can be taken as **3747 cumec**. The confidence band for LPT-3 distribution is given in Fig-3.2.

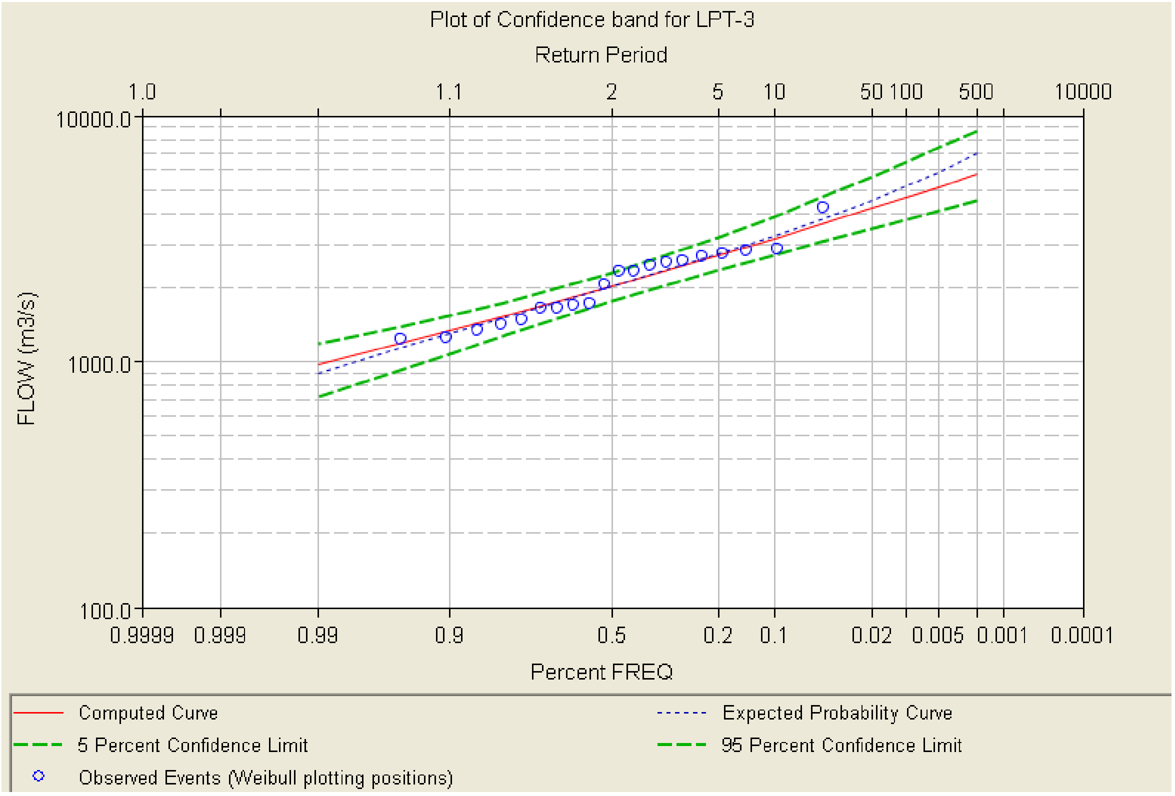

Fig-3.2: Confidence band for LPT-3

3.6 25 year return period flood at proposed project site

To get the 25 year return period flood at proposed project, the estimated flood at G&D site has been transposed Project site using Dicken's formula as given below:

25 year non-monsoon diversion at project site	= 3747x(1800/2350)^0.75 = 3068 say 3100 cumec

Hence, 25 year return period flood at the proposed project site may be considered as 3100 cumec.

CHAPTER 5. Hydrology of the project

5.0 GENERAL

The economics and sizing of every structural and non-structural component of the project is dependent on hydrology. Hydrological inputs play a major role in planning, designing, execution and successful operation of a water resources development project. The Hydro-Meteorological data is a basic input to finalize the hydrological design parameters of any hydraulic structure. Estimation of design flood for the design of hydraulic structures is an important component of hydrological studies. The higher value results in increase of the cost of hydraulic structures, an under estimated value likely to place the structure and population involved, at risk. The design flood study is an integral and significant part of safety of any hydraulic structure.

5.1 DESIGN FLOOD SELECTION CRITERIA

As per CWC handbook “Flood Protection, Anti erosion and River Training Works” published in year 2012, design flood selection criteria shall be as follows:

5.1.1 Embankment for predominantly agricultural areas.

The design flood for this type of embankment is kept 25 years for fixation of crest level.

5.1.2 Embankments for township or areas having industrial installations

The design flood for this type of embankment is kept 100 years for fixation of crest level. In the cases where anti erosion measures are proposed along with the embankment then design flood may be kept as 50 years for rural areas and 100 years for urban/industrial areas. In certain special cases, where damage potential justifies, maximum observed flood may also be considered for fixing the crest level.

5.3 BASIC HYDROLOGICAL DATA REQUIREMENT AND COMPILATION

The DPR shall contain the following basic information/data:

(a) Geo-spatial details of various hydrological observations sites maintained by the CWC/States/Other agencies within the drainage area along with their locations.

(b) Details of specific data collected with method of observation along with summary.

(c) The long terms Gauge and Discharge data of nearby sites and hydro-meteorological (rainfall) data within the drainage area.

(d) Annual peak discharges and levels for all G&D sites in and around of project at upstream and downstream shall be furnished for entire period of record. Instantaneous flood peak series may be prepared from hourly water level records available at G&D site for frequency analysis.

(e) Drainage area map showing location detail of all observation sites, proposed location of embankment, location of rain gauge stations, G&D sites and important hydraulic structures, etc.

5.4 DESIGN FLOOD COMPUTATIONS

For selection of design flood, the relevant BIS codes/guidelines for the specific structure shall be referred. For design flood estimation, procedure recommended in CWC's "Manual on Estimation of Design Flood" and other guidelines on the subject shall also be referred. For flood management schemes, design flood of appropriate return period shall be computed using following approaches:

(c) Flood Frequency Analysis
(d) Hydro-meteorological Approach

5.5 FLOOD FREQUENCY ANALYSIS

To estimate the design flood using flood frequency approach, the following procedures shall be adopted:

(e) The flood peak series shall be checked for randomness, homogeneity, trend, jump, outliers etc using appropriate statistical methods.

(f) Flood frequency analysis shall be carried out using time series of instantaneous annual flood peak. Based on the hourly gauge data the observed annual flood peak shall be converted into instantaneous flood peak.

(g) Using the instantaneous annual flood peak time series, the flood frequency analysis shall be carried out using standard frequency distributions such as Gumbel, log Pearson type-III and Log Normal distributions etc. to estimate the desired return period flood.

(h) Goodness of fit test for the frequency distribution shall be carried out using standard statistical tests such as Chi Square, D-Index etc. to assess the appropriate frequency distribution for the data set and decide the appropriate design flood.

5.6 HYDRO-METEROLOGICAL APPROACH

Where the observed flood peak series is not available, desired return period flood shall be estimated by hydro-meteorological approach using the following procedure:

(g) The rainfall of appropriate return period shall be assessed using standard meteorological approaches and tools such as isopluvial maps published by IMD / PMP Atlas of CWC.

(h) The drainage area representative observed concurrent short interval rainfall and runoff data of 4 to 5 flood events shall be collected to develop the catchment response function / unit hydrograph.

(i) Where the observed concurrent short interval rainfall and runoff data is not available, the flood estimation reports for different subzones published by CWC can be used to develop synthetic unit hydrograph.

(j) Infiltration loss rate, base flow and hourly rainfall distribution coefficients of rainfall can be assessed from the relevant reports / flood estimation reports.

(k) Critical sequencing and convolution shall be carried out as per standard procedure. In this regard Manual on Estimation of Design Flood published by Central Water Commission in March, 2001 may be followed.

(l) When the catchment area is very small and unit hydrograph may not be derived, rational formula may be used for design flood estimation.

5.7 WATER SURFACE PROFILE

Water level corresponding to estimated design flood shall be computed using gauge and discharge data of nearest G&D site. If gauge and discharge data is not available, the water surface profile for the estimated design flood shall be computed using hydrodynamic study on appropriate hydrodynamic model such as HEC-RAS, Mike11 etc. For hydrodynamic simulation, sufficient number of surveyed river cross sections shall be used to represent the study river reach.

In case where upstream reservoirs are existing and some flood moderation is possible from these reservoirs, the same should be taken into consideration while computing the water surface profile.

CHAPTER 6. DESIGN OF WORKS

6.1 Design of embankments:

6.1.1 A levee or dyke may be defined as an earthen embankment extending generally parallel to the river channel and designed to protect the area behind it from overflow of flood waters. Embankments are the oldest known forms of flood protection works and have been used extensively for this purpose. These serve to prevent inundation, when the stream spills over its natural section, and safeguard lands, villages and other properties against damages.

6.1.2 Embankment Manual, CW&PC, 1960 stipulates that an embankment is designated as low, medium or major (according to its height above natural surface level (NSL). The details are as under in **Error! Reference source not found.**.

Table 1-1: Classification of embankment

#	*Classification of embankment*	*Criterion*
1	Low Embankment	Height < 10 ft (3 m)
2	Medium Embankment	10 ft (3 m) <Height> 30 ft (9 m)
3	Major Embankment	Height > 30 ft (9 m)

6.1.3 Design of embankment should be done as per provisions of BIS code 12094: 2000. Some major provisions stipulated in the code are given as under:

6.1.4 The design flood for embankment to protect rural areas & urban areas may be kept as 25 years & 100 years respectively for fixation of crest level. In the cases where anti erosion measures are proposed along with the embankment then design flood may be kept as 50 years for rural areas and 100 years for urban/industrial areas. In cases where gauge & discharge sites are not present, discharge may be worked out using the Empirical formula using the catchment area, extent of rainfall, catchment characteristics etc. Further hydrological data may be adopted using the regional hydrological booklets/manuals, prepared by the Hydrological Studies Organization, CWC.

6.1.5 The embankments should be aligned on the natural bank of the river, where land is high and soil available for the construction of embankments. The alignment should be such that important township, vital installations, properties, cropped area is well protected by the embankment The alignment should be such that high velocity flow is quite distant from the toe of embankment to avoid scouring of the same and if embankments' alignment is near the high velocity flow then slope and toe protection in the form of pitching along with launching apron using the boulders, geo-bags, sand filled geo-mattress may be given. RCC porcupine screens along the toe line may also be used to retard the flow to induce siltation and check scouring of the toe-line. Alignment should also be planned so that land acquisition is feasible and not prolonged.

6.1.6 The spacing of embankments along the jacketed reach of the river should not be less than 3 times Lacey's wetted perimeter for the design flood discharge. The minimum distance of the embankment from the river bank and midstream of the river should be one times Lacey's wetted perimeter and 1.5 times Lacey's wetted perimeter [Lacey's wetted perimeter (P) $=4.75 (Q_{design})^{1/2}$] respectively. In the tidal reach of the river, embankments should be constructed with due regard to their effect on navigation requirements in the channel as embankments in such cases may reduce the tidal influx causing a reduction in available navigation depth.

6.1.7 Free board should be taken as 1.5 m for discharges less than 3000 cumecs and 1.8 m for discharges more than 3000 cumecs.

6.1.8 The top width of the embankment should be sufficiently enough to accommodate the vehicular traffic. The top width of the embankment may be kept as 5.0 m. Turning platform of length 15 m to 30 m and 3 m width at C/S side slope at an interval of 1 km or more may be provided.

6.1.9 It is desirable to know the approximated line of seepage or hydraulic gradient line (HGL). The following guidelines may be used for determining the HGL.

Clayey soil: 4H:1V
Clayey sand: 5H:1V

Sandy soil: 6H:1V

6.1.10 The river side (R/S) slope should be flatter than the under-water angle of repose of the material. Up to an height of 4.5 m, the slope should not be steeper than 2H:1V and in case of high embankments, slope should not be steeper than 3H:1V, when the soil is good and to be used in the most favorable condition of saturation and drawdown.

6.1.11 A minimum cover of 0.6 m over the HGL should be maintained. For embankment up to height of 4.5 m, the country side slope should be 2H:1V from the top up to the point where the cover over HGL is 0.6 m after which a berm of suitable width, with country side slope of 2H:1V from the end of the berm up to the ground level should be provided.

6.1.12 For drainage, longitudinal drains should be provided on the berm and cross drains at suitable places should be provided to drain the water from the longitudinal drains. Toe drain should be provided to prevent sloughing of toe. Perforated pipe embedded in properly designed graded filter with arrangements for disposal of water in the country side should be provided. Use of geo-textile material is also useful for safe drainage.

6.1.13 Sluices with regulating arrangements should be provided for country side drainage. The size of sluice will depend upon the intensity of rainfall and the catchment area to be drained. Sluices may be designed as per provision of BIS code IS 8835:1978.

6.1.14 The criterion for stability analysis for high embankment is based on the stability analysis of embankment dams. Slope stability is generally analyzed by two methods depending upon the profile of failure surface viz. (a) Circular arc method and (b) Sliding Wedge method. In the 'Circular arc' method or 'Swedish Slip Circle' method, the rupture surface is assumed cylindrical or in the cross-section by an arc of a circle. The sliding wedge method assumes that the failure surface is approximated by a series of planes.

6.2 Design of revetment

6.2.1 Protection of banks is a part and parcel of river training works because bank caving is one of the causes of deterioration of river conditions. River passing through populated/agricultural areas necessitates protection of adjacent lands and properties threatened by the erosion.

6.2.2 The design of revetment may be done as per provisions of IS 14262:1995. Main provisions are given as under.

6.2.3 The design flood for pitching/revetment may be calculated for 50 years return period using the flood frequency analysis. The design HFL should be obtained from gauge discharge relationship (G-D curve). In cases where G&D site are not present, discharge may be worked out with the Empirical formula using the catchment area, extent of rainfall, catchment characteristics etc.

6.2.4 **Revetment using boulders:** The weight of stones on slopes (W in kg) may be worked using the formula given below

:W (in kg) = $0.02323 * S_s * V^6 / K * (S_s - 1)^3$ -------------------------------- (1)

Where K (correction factor for slope) =$[1 - \sin^2\theta / \sin^2\Phi]^{1/2}$

S_s=specific gravity of boulders (may be adopted as 2.65)

Φ = Angle of repose of material of protection works (adopted as 30^0 for boulders)

θ= Angle of sloping bank2 (H) :1 (V) (26.56^0)

V= Velocity in m/s

K =$[1 - \sin^2 26.56^0 / \sin^2 30^0]^{1/2}$ = =0.447

Hence weight of stones for 2H:1V slope

W (in kg) = $0.02323*S_s*V^6/0.447*(S_s-1)^3$

For normal bank protection works, a slope of 2H:1V or flatter may be taken.

Size of stone (Ds in m) may be determined from the following relationship.

Ds (in m) = $0.124*(W/S_s)^{1/3}$ -- (2)
Where W= Weight of stone in kg
S_s= Specific gravity of stone (may be adopted as 2.65)
Minimum dimension of stones > Ds

Generally, the size of stone should be such that its length, width and thickness are more or less same ie stones should be more or less cubical. Round stones or very flat stones having small thickness should be avoided.

Minimum thickness of pitching (t) or protection layer is required to withstand the negative head created by the velocity. This may be determined by the following equation.

Minimum thickness of pitching (t in m) = $V^2/2g(S_s-1)$ ------------ (3)
V= Velocity in m/sec
g= Gravitational acceleration in m/sec2
S_s= Specific gravity of stone (may be adopted as 2.65).

Therefore thickness of pitching should be higher than t (as obtained above in equation 3). Two layers of stones of minimum size 't' should be provided, when pitching is being provided with boulders in loose.

At high velocity, required weight of stones (to be found by equation No 1) comes out to be higher, which makes handling and placing of stones a bit difficult. In such cases or in case when requisite sized stones are not available, small size stones filled in GI (Galvanized Iron) wire crates may be used for pitching purpose. In this case single layer of GI wire crates filled with stones having thickness more than 't' may be used as pitching. The specific gravity of the crate is different from the boulders due to presence of voids. Porosity of the crates (e) may be worked out using the following formula.

$E = 0.245 + 0.0864 / (D_{50})^{0.21}$ --- (4)
Where D_{50}= mean diameter of stones used in mm. let us assume D_{50} as 250 mm
$e = 0.245 + 0.0864 / (250)^{0.21}$
$= 0.27$

The opening in the wire net used for crates should not be larger than the smallest size of stone used. The mass specific gravity of protection (S_m) can be worked out using the following relationship.

$S_m = (1-e) * S_s$-- (5)

This mass specific gravity may be used to work out the weight of the crates and this weight should be more than weight of stone required, worked out by the equation No.1.

A graded filter of size 150 mm to 300 mm thickness may be laid beneath the pitching to prevent failure by sucking action by high velocity. Geo-synthetic filter may also be used as that is easy to lay, durable, efficient and quality control is easy. A 150 mm thick sand layer over the geo-synthetic filter may be laid to avoid rupture of fabric by the stones.

Paneling may be provided in the pitching where slope length is more so that slopes may remain more stable. The size of panel may be varied depending upon the length of river reach to be protected and the length of slope length.

In case of revetment on slopes up to NSL, which is below HFL, a top key or capping berm should be provided for allowing flow of water over the top surface of the revetment.

6.2.5 Pitching in mortar.

Stones, bricks or concrete blocks may be used for construction of pitching in mortar. Size of stones/bricks/concrete blocks in this type of pitching is not a critical aspect of design as every individual complement is bounded by mortar. Average size of available stone can be used for this purpose. But thickness of such pitching should be more than 't' (as calculated by the equation No. 3).

Mortar revetment should not be constructed in continuous or monolithic form. To avoid cracks, joints at suitable interval may be provided. Generally revetment may be provided in panels of size 3mx3m or 3mx5m. The size of panel may be varied depending upon the length of river reach to be protected and length of slope. Standard stone filter or geo-synthetic filter may be provided beneath the joints.

Drain holes or weep holes may be provided in each panel for free drainage of pore water from saturated bank soil beneath it. Depending upon the size of panel, one or more weep holes may be provided for a panel. The pipe provided in the drain hole should be up to the natural bank. Stone graded filter or geo-synthetic filter may be provided at the end at the contact of the bank soil.

6.2.6 Revetment using Geo-bags

The pitching may also be provided using sand filled geo-synthetic bags. The size of bags may be 1.1m x 0.7m x 0.15m. The weight of such bags is around 126 kg which is generally safe for the velocity up to 3 m/s. For higher velocities, size of Geo-bag may be higher so that weight of bag is higher than the required weight (worked out by the equation No.1). The geo-synthetic material should be safe against the UV rays and abrasion.

If the pitching is being provided in geo-bags, then generally filter in not provided because material of Geo-bags itself work as filter. But for safety purpose (for taking care of bank soil in joints), a geo-synthetic filter layers beneath the geo-bags may be provided.

6.2.7 Toe protection

To prevent the sliding and failure of the revetment on slope, toe is required to be protected. This may be in the form of simple toe-key, toe wall, sheet pile or launching apron.

Simple key may be provided at the toe (may be called as toe key) when rock or un-erodible strata is available just below the river bed and the overlaying banks are erodible. The key is in the form of stone/bricks or concrete blocks filled in the trench below the hard river bed for depth equal to the thickness of pitching “t” for proper anchorage. Sole purpose of this key is to provide lateral support to the pitching. The key may be of mortar or in geo-bags, if the pitching is provided in mortar or geo-bags.
When hard strata is available below the river bed at a reasonable depth, toe wall is recommended. The thickness of the toe wall depends upon height of wall and height of overlaying pitching. The toe wall may be design as retaining wall and be constructed in masonry along with provisions of weep

holes etc.

When firm strata is not available at reasonable depth below the river bed, toe protection in the form of sheet pile or launching apron may be provided. The sheet pile may be made of RCC, steel or bamboo. The sheet piles may be drilled below the river bed up to maximum scour depth.

Sheet piles are difficult to drive; therefore Launching apron is preferred and provided with revetment. Launching apron should be laid at low water level (LWL). The launching apron may be laid using the stones or geo-bags. The stones/geo-bags in the apron should be designed to launch along the slope of scour and provide a protection layer so that scouring is checked. The size of launching apron should be such that it should form a protection layer up to level of maximum scour depth. Slope of launching apron may be taken as 2H:1V. Filter below the launching apron may also be provided so that river bed material is safe against suction.

6.2.8 Size of Launching apron:

Width of the launching apron depends upon the scour depth below HFL. Depth of scour below HFL (D) may be worked out using the following formula:

$$D = 0.473\ (Q/f)^{1/3} \quad \text{-----} \quad (6.1)$$

and

$$D = 1.33\ (q^2/f)^{1/3} \quad \text{-----} \quad (6.2)$$

Where Q = design discharge in cumecs and q = design discharge per unit width or design discharge intensity in cumecs/m

f is silt factor. Silt factor (f) may be calculated using the following formula

$$f = 1.76\ (d)^{1/2} \quad \text{-----} \quad (7)$$

where d is mean particle diameter of river material in mm

Generally scour depth (D) below HFL should be calculated using the design discharge (equation no. 6.1). In some cases (for braided rivers) scour depth may be calculated using the design discharge intensity (equation no. 6.2).

Maximum scour depth $(D_{max})_{\text{below HFL}} = 1.5 *$ Scour depth $(D_{\text{below HFL}})$.

Maximum Scour depth $(D_{max})_{\text{below LWL}} = (D_{max})_{\text{below HFL}} - (HFL-LWL)$

If the launching apron is being laid at LWL then width of the launching apron should be calculated using the following formula.

Width of launching apron= 1.5 * (D_{max}) below LWL
Thickness of launching apron (T) = 1.5* thickness of pitching (t).
In some cases, thickness of the launching apron is kept different from 'T' due to size of crates etc (if launching apron is being provided in crated stones), then width of the launching apron may be revised keeping the volume of stones/geo-bags same per unit length of the apron.

6.2.9 Anchoring

Proper anchor is required for keeping the revetment in place and serving the desired function. Upstream edge from where the revetmen starts should be secured well to the adjoining bank. Similarly downstream edge where the revetment ends also needs to be secured well to the adjoining bank. Anchorage is also required to be provided on the top of submerged bank. If the top of bank is above HFL, the revetment should be provided above HFL with an adequate free board say 1.0 m. Under such situation, anchorage at top is not required.

6.3 Design of spurs/groynes

6.3.1 Spurs/groynes are structures, constructed transverse to the river flow and extended from the bank into the river. Spurs/groynes, protruding into river come under purview of anti erosion works. These types of works are provided to keep away flow from the erosion prone bank. The spurs are provided along with launching apron to prevent scouring under the water and consequent fall of spurs.

6.3.2 The design of Spurs/groynes may be done as per provisions of IS 8408:1994. Main provisions are given as under.

6.3.3 Spurs may be aligned either normal to flow direction or at angle pointing towards u/s or d/s of the flow. A spur pointing u/s of the flow repels the flow away from the bank and is known as repelling type spur/groyne. When a short length spur changes only direction of flow without repelling, it is known as deflecting spur/groyne. Spur pointing d/s of the flow attracts the flow towards the bank and is known as attracting spur/groyne. Generally repelling type or deflecting spurs are provided for anti erosion measures. Repelling type spurs may be kept at an angle of 5^0 to 10^0 against the direction of flow.

6.3.4 The length of spur should be decided on the basis of availability of land on the bank. Effective length of the spur should be the portion which is likely to face/counter the river flow. Extra length given in the spur only for the purpose of tagging the spur with high ground should not be taken into consideration for adoption as effective length of spur. Length shouldn't be less than that required to keep the scour hole formed at the nose away from the bank. Thus assuming angle of repose of sand to be 2.5H:1V and anticipated maximum scour depth below river bed (d_s), the length should be more than $2.5xd_s$. Short length may lead to bank erosion at u/s and d/s of the groyne due to formation of eddies at nose. On the other hand, too long spur may obstruct the river and may not withstand the attack on account of heavy discharge concentration at the nose.

Normally the effective length of spur shouldn't exceed 1/5th of width of flow in case of single channel. In case of wide, shallow and braided rivers, the protrusion of spur in the deep channel should not exceed 1/5th of the width of channel on which the spur is proposed excluding the length over then bank.The spacing of spurs is normally 2 to 2.5 times its effective length. For site specific cases model studies may be conducted.

6.3.5 The top level of spur will depend on the type namely, submerged, partially submerged or non-submerged and will be best decided by model study. In case of non-submerged spurs, the top level should be above design flood level with adequate free board. Free board may be adopted as 1m/1.5m. In case non-submerged spur is tied with the embankment, then top level of embankment and top level of spur may be kept same with similar free board and design HFL.

The top width of spur should be 3 to 6 m as per requirement. Side slopes of the spur may be kept 2H:1V or 3H:1V depending upon the material being used for construction.

6.3.6 Stones/boulders used in pitching are subjected to hydrodynamic drag and lift forces. These destabilizing forces are expressed in terms of velocity, tractive forces etc. the stabilizing forces acting against these are component of submerged weight of stones and downward component of force caused by contact of the stones.

The weight of stones on slopes (W in kg) may be worked using the formula given below.

W (in kg) = $0.02323*Ss*V^6 / K*(S_s-1)^3$ --------------------------------------- - (1) Where K (correction factor for slope) =$[1-\sin^2\theta/\sin^2\Phi]^{1/2}$ S_s = specific gravity of boulders (may be adopted as 2.65) Φ = Angle of repose of material of protection works (adopted as 30^0 for boulders) Θ = Angle of sloping bank 2 (H) :1 (V) (26.56^0) V = Velocity in m/s K = $[1-\sin^2 26.56^0/ \sin^2 30^0]^{1/2}$ = 0.447 Hence weight of stones for 2H:1V slope W (in kg) = $0.02323*S_s*V^6/0.447*(S_s-1)^3$

6.3.7 Size of stone (Ds in m) may be determined from the following relationship.

Ds (in m) = $0.124*(W/S_s)^{1/3}$ -- - (2) Where: W = Weight of stone in kg S_s = Specific gravity of stone (may be adopted as 2.65) Minimum diminution of stones > Ds

Generally, the size of stone should be such that its length, width and thickness are more or less same ie stones should be more or less cubical. Round stones or very flat stones having small thickness should be avoided.

6.3.8 Minimum thickness of pitching (t) or protection layer is required to withstand the negative head created by the velocity. This may be determined by the following equation.

Minimum thickness of pitching (t in m) = $V^2/2g(S_s-1)$ ------------------ (3) V= Velocity in m/sec g= Gravitational acceleration in m/sec2 S_s= Specific gravity of stone (Generally adopted as 2.65).

Therefore thickness of pitching should be higher than t (as obtained above). Two layers of stones of minimum size 't' should be provided when pitching is being provided with boulders in loose.

6.3.9 At high velocity, required weight of stones (to be found by equation no 1) comes out to be higher, which makes handling and placing of stones a bit difficult. In such cases or in case when requisite sized stones are not available, small size stones filled in GI (Galvanized Iron) wire crates may be used for pitching purpose. In this case single layer of GI wire crates filled with stones having thickness more than ‘t’ may be used as pitching. The specific gravity of the crate is different from the boulders due to presence of voids. Porosity of the crates (e) may be worked out using the following formula.

$e = 0.245 + 0.0864/ (D_{50})^{0.21}$ ---(4)

Where D_{50} = mean diameter of stones used in mm. let us assume D_{50} as 250 mm

$e = 0.245 + 0.0864/ (250)^{0.21}$

$= 0.27$

The opening in the wire net used for crates should not be larger than the smallest size of stone used. The mass specific gravity of protection (S_m) can be worked out using the following relationship.

$S_m = (1-e) * S_s$ -- (5)

This mass specific gravity may be used to work out the weight of the crates and this weight should be more than weight of stone required, worked out by the equation no.1.

Crates should be laid with long dimension along the slope of the bank. Crates must be tied to each other by 5 mm GI wire as additional protection. If crates are being provided in layers then each layers should be tied to each other at suitable interval using the 4 mm GI wire.

6.3.10 A graded filter of size 150 mm to 300 mm thickness may be laid beneath the pitching to prevent failure by sucking action by high velocity. Geo-synthetic filter may also be used as that is easy to lay, durable, efficient and quality control is easy. A 150 mm thick sand layer over the Geo-synthetic filter may be laid to avoid rupture of fabric by the stones.

6.3.11 The pitching may also be provided using sand filled geo-synthetic bags. The size of bags may be 1.1 mx0.7mx0.15 m. The weight of such bags is around 126 kg which is generally safe for the velocity up to 3 m/s. For higher velocities, size of geo-bag may be higher so that weight of bag is higher than the require weight (worked out by the equation no. 1. The Geo-synthetic material should be safe against the UV rays and abrasion.

The thickness of geo-bag pitching may be decided as per procedure given above. To summarize again, thickness of pitching should be more than 't'. Pitching may be provided in double layers of geo-bags (in loose) and in single layer if encased using the nylon/polypropylene ropes.

If the pitching is being provided in geo-bags, then generally filter in not provided because material of geo-bags itself work as filter. But for safety purpose (for taking care of bank soil in joints), a geo-synthetic filter layers beneath the geo-bags may be provided.

6.3.12 IS code 8408:1994 & 14262:1995 mentions following provisions regarding launching apron.

To prevent the sliding and failure of the spur due to scouring action by the river current, provision of launching apron is kept to take care of the scouring at nose and at shank (portion in the river) of the spur.

Launching apron should be laid at low water level (LWL). The launching apron may be laid using the stones or geo-bags. The stones/geo-bags in the apron should be designed to launch along the slope of scour and provide a protection layer so that scouring is checked. The size of launching apron should be such that it should form a protection layer up to level of maximum scour depth. Slope of launching apron may be taken as 2H:1V. Filter below the launching apron may also be provided so that river bed material is safe against suction.

6.3.13 Width of the launching apron depends upon the scour depth below HFL. Depth of scour below HFL (D) may be worked out suing the following formula.

$D = 0.473 (Q/f)^{1/3}$ -- (6.1)

and

$D = 1.33 (q^2/f)^{1/3}$ -- (6.2)

Where Q= design discharge in cumecs and q= design discharge per unit width or design discharge intensity in cumecs/m.

f is silt factor. Silt factor (f) may be calculated using the following formula

f= 1.76 (d) $^{1/2}$--(7) where d is mean particle diameter of river material in mm

Generally scour depth (D) below HFL should be calculated using the design discharge (equation no.6.1). In some cases (for braided rivers) scour depth may be calculated using the design discharge intensity (equation no. 6.2).

Maximum scour depth (D_{max}) below HFL= 1.5* Scour depth ($D_{below\ HFL}$). Maximum Scour depth (D_{max}) below LWL = $(D_{max})_{below\ HFL}$ – (HFL-LWL)

If the launching apron is being laid at LWL then width of the launching apron should be calculated using the following formula at different locations of the groyne.

(i)	Width of launching apron at nose	=	(2-2.5) * (D_{max}) below LWL
(ii)	Width of launching apron at transition from nose to shank and first 30 m to 60 m in u/s	=	1.5 * (D_{max}) below LWL
(iii)	Width of launching apron in shank portion for next 30 m to 60 m	=	1.0 * (D_{max}) below LWL
(iv)	Width of launching apron at transition from nose to shank and first 15 m to 30 m in d/s	=	1.0 * (D_{max}) below LWL

Thickness of launching apron (T) = 1.5* thickness of pitching (t).

In some cases, thickness of the launching apron is kept different from 'T' due to size of crates etc (if launching apron is being provided in crated stones), then width of the launching apron may be revised keeping the volume of stones/geo-bags same per unit length of the apron.

4 Design of RCC porcupines spurs & screens

6.4.1 Protection of banks is a part and parcel of river training works. This protection comes under anti erosion works. Permeable structures envisaging construction of RCC porcupine screens & spurs are a cost effective alternative to the impermeable bank protection works for the rivers carrying considerable amount of silt. RCC porcupine is a prismatic type permeable structure, comprises of six members of made of RCC, which are joined with the help of iron nuts and bolts.

Permeable screens & spurs are the main type of permeable structures in vogue. Prima facie, the purpose, overall behavior and layout of the above mentioned structures can be compared to those of submersible bunds, spurs and revetment respectively. The permeable structures can be used either independently or with a support of other impermeable boulder structures or river training and bank protection measures. Depending upon the purpose, the permeable structures like RCC porcupines may be constructed in transverse or parallel to direction of flow.

6.4.2 Only partial obstruction to the flow of about 15 to 20% only is envisaged in the design. Higher obstruction causes more diversion of flow resulting in undesired scouring around the proposed structures, particularly at the nose portion. Additional protection to the nose and flanks is required to avoid such scour. Therefore, obstruction more than 20% is avoided.

Submergence of RCC porcupine screens & spurs may be kept up to 50% of depth of flow. For example, single layer of RCC porcupines, comprising 3 m long members is sufficient for depth of water till 6 m.

6.4.3 Layout of RCC porcupine spurs

(a) The porcupines (comprising of six members of size 3mx0.1mx0.1m) are laid in a row across the river bank protruding into the river at spacing generally adopted as 3m c/c. If size of member is 2m x 0.1m x 0.1m, then spacing between the porcupines may be kept as 2m c/c.
(b) Each porcupine spur is made up of 3 to 7 rows of porcupines (Higher rows for higher flow). The spacing of rows is kept at same as spacing of each porcupine in each row (3 m or 2 m c/c depending upon size of the member).
(c) If the flow depth is more than 6 m, RCC porcupine spurs may be provided in double vertical layers.
(d) On a straight reach, RCC porcupine spurs are placed at 3 to 5 times the length of spur. On a curved channel, the spacing can be kept as 2 to 4 times the length of spur.
(e) The length of spur into the river shouldn't exceed the 1/5 of the width of the flow. Generally length of spurs may be kept less than 100m to 150 m.
(f) In order to resist the tendency of outflanking, additional porcupines may be provided along the sloping bank at u/s and d/s of the RCC porcupine spurs.
(g) At least three RCC porcupine spurs may be provided for a reach to be protected. A single permeable spur is generally not effective.
(h) At several locations facing severe erosion, where revetment with

apron is not feasible or justified due to space and cost constraint, provisions of RCC porcupine spurs along with porcupine dampeners/screens along the eroded bank may be provided.

The practice of providing one or two additional spurs u/s and d/s of the eroding reach, pointing towards u/s with reference to flow may be followed for the RCC porcupine spurs also.

6.4.4 Layout of RCC porcupine screens

(a) The RCC porcupine screens are used to block the secondary channels.
(b) Each porcupine screen is made up of 5 to 9 rows of porcupines (Higher rows for higher flow). The spacing of rows is kept at same as spacing of each porcupine in each row (3 m or 2 m c/c depending upon size of the member).
(c) At least two screens are provided to block the secondary channel. A single screen is generally not found effective.
(d) One screen is normally provided at the entrance of the secondary channel. The second screen is provided at a distance of 1 to 1.5 times width of the secondary channel.
(e) The screens are constructed covering a part or the whole width of secondary channel. If the screen covers the whole width, the screens are extended on both banks for a length 1/3rd of the channel width.
(f) Depending upon the importance, the possibility of development of bypass channel, a third screen can also be provided further d/s at a suitable location.

If the screens are located near the bank, the extension towards bank should be restricted to the design HFL

5 Design of Drainage Improvement Works.

6.5.1 IS code 8835:1978 stipulates that drains are constructed with the object of relieving excess water from agricultural and other areas and disposing of surplus water which is not required for normal agricultural operations. The proper disposal of surplus rain water is also essential to avoid its percolation down to the water level which may otherwise lead to rise in the water table thereby aggravating or creating the problem of water logging.

6.5.2 IS Code 8835:1978 stipulates that drains may be designed for 3 day rainfall of 5 year return period. However, in specific cases requiring higher degree of protection, return period of 10 or 15 year may also be adopted. Adoption of higher return period rainfall should be justified in term of economics. Cross drainage works should be designed for 3 day rainfall of 50 year return period.

6.5.3 IS code 8535:1978 envisages following guidelines for the alignment of the drainage channel.

The drains should generally follow the drainage line ie. lowest valley line. As far as possible the alignment of the main or outfall drain should be in the centre of the area to be drained. If the alignment crosses any depressions, ponds or marshes, the drain should not pass through these, as apart from the difficulties in excavation, it affects the hydraulic performance of the drain. In such cases, it is preferable to take the drain away from the depression or pond, and suitably connect it to the drain if it is required to drain the pond or depression.

In selecting alignments, care should be taken to see that as far as possible these do not pass through village habitation. In the forced reaches, care should be taken to see that the embankments of the drains are not of an excessive height in order to minimize the danger of flooding in the event of breaches in the embankments.

6.5.4 IS code 8535:1978 envisages following guidelines for capacity/design discharge of the drainage channel.

Normally the drain is provided to accommodate the design discharge where drains follow natural valley lines. In such cases, no embankments should be provided along the drain so as to allow free flow of water from the surroundings areas. Wherever embankments are necessary for accommodating a portion of the design discharge or where disposal of excavated soil will be very costly, large gaps should be provided in the embankments on either side so as to allow unrestricted inflows, and in case of discharges higher than the channel capacity, the water should spill over the area and return to the channel freely when the discharge in it recedes. In the forced or diversion reaches, embankments on both sides are, however, provided as the design discharge cannot be accommodated within the cut section of the drain. However, even in such cases attempts should be made by selecting a proper alignment to keep the height of the embankments to the minimum. In such cases, inlets of adequate size should be provided in the embankments to admit the water from surrounding areas.

6.5.5 Generally the drains should be designed for three day rainfall of 5 year frequency. Studies carried out indicate that 5 year frequency gives optimum benefit cost ratio.

6.5.6 Period of disposal: The period of disposal of the excess rainfall is entirely dependent on the tolerance of individual crops. Crops Like paddy can generally stand submersion for a period of 7 to 10 days without suffering any significant damage. Therefore, in paddy growing areas, the drainage should aim at disposing of the rain water in a period varying from 7 to 10 days. Based on experience the following periods of disposal are recommended.

#	*Crops*	*Period of Disposal*
(i)	Paddy	7 to 10 days
(ii)	Maize, bajra and other similar crops	3 days
(iii)	Sugarcane and bananas	7days
(iv)	Cotton	3 days
(v)	Vegetables	1 day (in case of vegetables, 24 hour rainfall will have to be drained out in 24 hours)

6.5.7 Run-off: Run-off coefficients depends on the type of soil, crops, general topographical conditions like land slopes, etc. In plain areas, the run-off percentage is generally of the order of 15 to 20. In semi-hilly areas the percentage may be higher. Until precise data becomes available, the following run-off coefficients for different soils are recommended for plain areas.

#	*Type of catchment*	*Run-off Coefficient*
(i)	Loam, lightly cultivated or covered	0.40
(ii)	Loam, largely cultivated and suburbs with gardens, lawns, macadamized roads	0.30
(iii)	Sandy soils, light growth	0.20
(iv)	Parks, lawns, meadows, gardens, cultivated area	0.05-0.20
(v)	Plateaus lightly covered	0.70
(vi)	Clayey soils stiff and bare and clayey soils lightly covered	0.55

6.5.8 Design discharge for cross drainage works: IS code 8535:1978 stimulates that cross drainage works are always designed for a higher discharge than the cut sections of the drains. This is mainly on account of the fact that the damage caused to the structures in the event of flows resulting from rainfall higher than the designed rainfall, can be much more than to the drain. Besides, any remodeling of the structures at a later date for higher discharges will not only be costly but time consuming, apart from involving dislocations to facilities like roads, railways, irrigation canals, etc. The drains can, however, be remodeled without much dislocation. All the cross drainage structures should, therefore, be designed for a 3day rainfall of 50 year frequency, time of disposal remaining the same depending on the type of crop. In fixing the waterways, care should be taken to see that afflux is within the permissible limits.

The drain should be designed as per Lacey's regime theory so that no silting/scouring is occurred in the drain section. Design procedure for the drainage channel may be done as per design of irrigation channel by Lacey's theory. The design procedure is as under.

Velocity of the flow (V in m/sec) = $(Qf^2/140)^{1/6}$
Where Q = design discharge in cumecs and, f is the silt factor, which can be worked out using the formula f = 1.76 $(d)^{1/2}$, where d is the average bed material size in mm
Hydraulic mean depth (R in m) = 2.5* (V^2/f)
Area of channel section (A in m2) = Q/V
Wetted perimeter (P in m) = 4.75 $(Q)^{1/2}$
and Bed slope (S) = $(f^{5/3}) / (3340*Q^{1/6})$

6.5.9 The drain section shall be adequate to carry the designed discharge and the velocity shall be non-silting, non-scouring to be determined by Manning's formula. In selecting the side slopes for the drain, it will be necessary to consider the kind of material through which the drain is to be excavated. Generally side slopes of 1.5H : 1V are provided.

6.5.10 Although deeper sections of the drain may be desirable, the width to depth ratio should be so selected that the section is both hydraulically efficient as well as economical in excavation. In the case of drains with embankments, the berm width equal to the depth of the drain, subject to a minimum of 1 m should be provided between the toe of the embankment and the section of the drain. The top of the embankments should be 1 m higher than the design full supply level. Wherever, there is likelihood of backing up effect on account of floods in a river into which the drain outfalls, the top of the embankments should be so designed that the flood levels on account of back water conditions are accommodated within the section over which the minimum freeboard is to be provided.

6.5.11 Whenever the drain is out falling into a river, the FSL should be slightly higher than the dominant flood level. The dominant flood level is the stage of river/outfall which is (a) attained and not exceeded for more than 3 days at a time; and also (b) attained and not exceeded 75% of time over a period of preferably not less than 10 years. In cases where the topography permits, the FSL can be above the highest flood level. However, if such a level results in flatter slopes or in FSL becoming higher than the natural ground level, FSL at outfall should be kept slightly above the dominant flood level. In such cases, there will be backing up in the drain when the river rises above the dominant flood level. Such occurrences being infrequent and of short duration can be tolerated. Care shall, however, be taken in determining the dominant flood discharge and the level.

6.5.12 The FSL of the drain as far as possible should be at or below the ground level. Where it cannot be ensured, the FSL should in no case be more than 0.3 m above the average ground level at the starting point of the drain. The hydraulic should then be determined adopting the stipulation and the criteria laid down for fixation of FSL at outfall.

6.5.13 **Silt Removal & Dredging:** Generally silt removal and dredging should be avoided and if it is necessary then it should be supported by Model Studies.

6 Design of Sluice works

Sluices with regulating arrangements should be provided for country side drainage. The size of sluice will depend upon the intensity of rainfall and the catchment area to be drained. If discharge is high, sluice needs to be replaced with a barrage. For planning and design of a barrage, Central Board of Irrigation and Power (CBIP) "Manual on Barrages and Weirs on Permeable Foundation" and other relevant IS codes may be followed.

A separate volume discussing in details (unless otherwise stated) the following points and additional points if any as relevant to the project shall form an appendix of the project report. It shall include structural 'and hydraulic design calculations for the sluice.

To reduce the bulk of the volume, only essential structural calculations considered absolutely necessary shall be furnished. However, for stability analysis loading diagrams considering various conditions of water level, drainage and other forces considered, shall be included.

Summary of the report appended for the relevant items shall be furnished under this Chapter. Cross reference shall be given to other chapters and appendices wherever necessary.

6.6.1 Structure and layout

6.6.1.1 General— brief

a) Headworks: its site and vicinity--stage of the river (Mountain/sub-mountain/plain with slope of the river in the vicinity of the structure). The sluice should be as near the lowest part of the area to be drained as possible. And should be accessible at the time of need.
b) Reasons for choice of the layout of the project
c) Layout of the sluice/ barrage/approach channel and tail channel in case of spillway is located in other than main river gorge and appurtenant/auxiliary works, reasons for choice of site.

6.6.1.2 Geology, seismicity and foundation—brief

a) Geology of the entire project areas.
b) Geo technical evaluation of foundations, abutments, and other major components.
c) Geological log of bore/drill holes, pits, drifts, geo-physical data etc.
d) Evaluation of foundation and abutments and other major components for treatment(including grouting, drainage etc.)
e) Engineering properties of the foundation materials including results of the in-situ tests like density, permeability, shear, bearing capacity, penetration, modulus of elasticity, bulk modulus, Deformation Modulus etc. and evaluation of design parameters.

6.6.1.3 Alternative studies carried out for selection of sluice/ barrage site

6.6.1.4 Choice of final layout of all major components of the project and reason – details

6.6.2 **River Diversion arrangements**–choice of design flood with Hydrographs.

a) Coffer dames
b) Tunnel(s), Construction Sluices etc.

6.6.3 Construction materials– brief

a) Qualitative and quantitative assessment of availability of construction material. Transport constraints if any.
b) Engineering properties of the materials and evaluation of design parameters(shear/compression/tensile strength, permeability, gradation, density, moisture etc.)

c) Special considerations with regards to the scarce materials, if any,
d) Details of tests undertaken for assessing the suitability of the construction materials

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6.6.4 Details of the model studies, if any, for important structure.

6.6.5 Sluice Design

Waterway of sluice should be sufficient enough to drain out the accumulated water with in evacuation time. Crest level of sluice/ barrage may be kept sufficiently low so that lean period flow also gets drained easily. Exit gradient, Floor thickness should be sufficient enough for both condition of flow i.e., drainage and backwater flow. Hydraulic jump consideration may be checked only for drainage flow condition. Scour depth of d/s cut-off may be decided based on lower of (i) design flood of drain or (ii) design flood of out-fall river. Hydraulic design of sluice regulator may be carried out by referring to "IS 6531: CRITERIA FOR DESIGN OF CANAL HEAD REGULATORS". The design calculations should include the following:

6.6.5.1 Assumed retrogression at maximum and minimum discharges.
6.6.5.2 Looseness factor
6.6.5.3 Scour factor
6.6.5.4 Intensity of discharge under design/super flood condition.
6.6.5.5 Co-efficient of discharge
6.6.5.6 Exit gradient value
6.6.5.7 Stress allowed (Concrete/ Masonry/Steel Foundation etc.
6.6.5.8 Type (concrete/ Masonry) profile cutoffs up-stream and downstream aprons, uplift pressure relief arrangements etc.
6.6.5.9 Various conditions of MWL, TWL, Drainage etc. considered for stability analysis of the different components of Sluice and values of factor of safety.
6.6.5.10 Gates, type, size and hoist arrangement and stop-logs including operating cranes.
6.6.5.11 Detail of spillway bridge Guide and afflux bunds, sheet piles, abutments, divide wall, wings wall, flare out wall, upstream/downstream protection etc.
6.6.5.12 Further, following data shall be appended in the chapter:

- Area to be drained.
- 3 day rainfall of 50 year frequency.
- Dominant flood level from historical hydrograph.

- Capacity – stage curve of area to be protected.
- Evacuation time available based crop data or other relevant data
- Rating curve of drain.
- Flood study Outfall River.
- Details of vital installation and their levels.

Illustrative designs of some works are given at annex 6.1

Annex 6.1

Illustrative Design of various flood management works:

Sample design of works is illustrated as under:

1. Design of embankment

Cross-Section of the embankment may be worked out as under

Design HFL (As per provisions in hydrology chapter given in DPR)= 100 m (adopted for illustration purpose)

Top Level= 100+1.5 (free board)= 101.5 m

(if discharge < 3000 cumec)

Top Level= 100+1.8(free board) = 101.8 m

(if discharge > 3000 cumec)

Top width	=	5.5 m
R/S slope	=	2 :1
C/S slope	=	3 :1
HGL(should be remained within the embankment With minimum cover of 0.5 m)	=	6 :1

2 Design of bank revetment

The design of typical bank revetment has been provided in the following method.

Design Discharge Q (As per provisions in hydrology chapter given in DPR)(adopted 20000 cumec for illustration purpose	=	20000	cumec
Gravitational Acceleration (g)	=	9.81	m/sec^2
Design HFL (for illustration purpose)	=	100.00	m

Observed LWL (for illustration purpose)	=	96.00	m
Stream Velocity V (for illustration purpose)	=	3.00	m/sec
Mean Dia of river bed material d (for illustration purpose)	=	0.30	mm
Silt Factor f= 1.76* (d) $^{1/2}$	=	0.96	
Angle of sloping bank (2H:1V) θ	=	26.56	^{0}
Angle of repose of protection material Φ	=	30	^{0}
Value of K=[1-Sin²θ/ Sin²Φ]$^{1/2}$	=	0.447	
Specific gravity of boulders Ss	=	2.65	
Weight of boulders W=0.02323*Ss*V^6/ (K* (Ss-1) ^{3})	=	22.349	kg
Size of boulder= 0.124 (W/Ss) $^{1/3}$	=	0.25	m
Thickness of pitching (T) for negative head criterion=V2/2g (Ss-1)	=	0.28	m
Thickness of pitching (=2*0.3=0.60m)	=	**0.60**	**m**
Design of Launching Apron (to be laid at LWL)			
Scour Depth below HFL D = 0.473* (Q/f) $^{1/3}$	=	13.015	m
Max. Scour Depth below HFL due to bends etc (D_{max}) = 1.5*D	=	19.523	m
Width of Launching Apron = 1.5*[Dmax- (HFL-LWL)]	=	23.285	m
Adopt 16 crates of size 1.5mx1.5mx0.45 m (total width of launching apron=16*1.5=24)	=	24.000	m
Thickness of Launching Apron (2 layers of crates=2*0.45=0.90)) =1.5* 0.60	=	0.900	m
Size of Launching apron	**=**	**24x0.90**	m

3 Design of groynes

The design of typical boulder spur has been provided in the following method.

Design Discharge Q (As per provisions in hydrology chapter given in DPR)(adopted 20000 cumec for	=	20000	cumecs

illustration purpose)			
Gravitational Acceleration g	=	9.81	m/sec^2
Design HFL (for illustration purpose)	=	100.00	m
Observed LWL (for illustration purpose)	=	96.00	m
Stream Velocity V (for illustration purpose)	=	3.00	m/sec
Mean Dia of river bed material d (for illustration purpose)	=	0.30	mm
Silt Factor f= 1.76* (d) $^{1/2}$	=	0.96	
Angle of sloping bank (2H:1V) θ	=	26.56	^{0}
Angle of repose of protection material Φ	=	30	^{0}
Value of K=[1-$Sin^2θ$/ $Sin^2Φ$]$^{1/2}$	=	0.447	
Specific gravity of boulders Ss	=	2.65	
Weight of boulders W=0.02323*Ss*V^6/ (K* (Ss-1) ^{3})	=	22.349	kg
Size of boulder= 0.124 (W/Ss) $^{1/3}$	=	0.25	m
Thickness of pitching (T) for negative head criterion=V2/2g (Ss-1)	=	0.28	m
Thickness of pitching (=2*0.3=0.60m)	**=**	**0.60**	**m**
Design of Launching Apron (to be laid at LWL)			
Scour Depth below HFL D = 0.473* (Q/f) $^{1/3}$	=	13.015	m
Max. Scour Depth below HFL at Nose ($D_{max)}$ = (2.0-2.5) *D (adopted as 2D)	=	26.030	m
Max. Scour Depth below HFL at transition from nose to shank and 1ST 30 m to 60 m U/S (D'max) = 1.5*D	=	19.523	m
Max. Scour Depth below HFL for next 30 m to 60 m in U/S (D''max) = 1.0*D	=	13.015	m
Max. Scour Depth below HFL for transition from nose to shank and 1ST 15 m to 30 m D/S (D'''max)	=	13.015	m

= 1.0*D			
Width of Launching Apron at nose= 1.5*[Dmax- (HFL-LWL)]	=	33.045	m
Adopt 23 crates of size 1.5mx1.5mx0.45 m (total width of launching apron=23*1.5=34.5)	**=**	**34.50**	**m**
Width of Launching Apron for transition from nose to shank and up to 60-90 m U/S =1.5*[D'max- (HFL-LWL)]	=	23.285	m
Adopt 16 crates of size 1.5mx1.5mx0.45 m (total width of launching apron=16*1.5=24)	**=**	**24.00**	**m**
Width of Launching Apron for next 30 m to 60 m in U/S =1.0*[D''max- (HFL-LWL)]	=	9.015	m
Adopt 6 crates of size 1.5mx1.5mx0.45 m (total width of launching apron=6*1.5=9)	**=**	**9.000**	**m**
Width of Launching Apron for transition from nose to shank and 1st 15 m to 30 m D/S =1.0*[D'''max- (HFL-LWL)]	=	9.015	m
Adopt 6 crates of size 1.5mx1.5mx0.45 m (total width of launching apron=6*1.5=9)	**=**	**9.000**	**m**
Thickness of Launching Apron (loose boulder) =1.5* Thickness of pitching	**=**	**0.900**	**m**

4 Design of regime drainage channel using the Lacey's theory

Design discharge Q= 50 cumecs (to be carried out by the channel)

Silt factor f= 1.1(For illustration purpose)

Velocity $V = (Qf^2/140)^{1/6}$

$= (50*1.1^2/140)^{1/6}$

$V = (50*1.21/140)^{1/6} = 0.8695$ m/sec

Area of cross section (A) = Q/V= 50/0.8695= 57.50 m^2

Hydraulic mean depth (R) = $2.5* (V^2/f)$

$= 2.5^* (0.869^2/1.1)$

$= 1.72$ m

Wetted perimeter (P in m) = $4.75 (Q)^{1/2}$

$= 4.75 (50)^{1/2}$

$= 33.59$ m

For a trapezoidal channel with side slope of 0.5H:1V

$P = b+2^*d (0.5^2+1^2)^{1/2}$

where b = width of channel in m and d is depth of channel in m

$P = b+2.24d$

$33.59 = b+2.24d$ or $b= 33.59-2.24d$

$A = (b+0.5d) ^*d$

$57.50 = bd+0.5d^2$

$(33.59-2.24d) ^*d+0.5d^2 = 57.50$

$33.59d-2.24d^2+0.5d^2 = 57.50$

$1.74d^2-33.59d+57.50 = 0$

$d^2-19.31d+33.05 = 0$

$d = (19.31- (19.31^2-132.2)^{0.5}) /2$ (neglecting + sign for realistic values of d)

$= (19.31-15.51) /2$

$= 1.90$ m

$b = 33.59-2.24^*1.9 = 29.33$ m

channel slope (S) = $(f^{5/3}) / (3340^*Q^{1/6})$

$= (1.1^{5/3}) / (3340^*50^{1/6}$

$= 0.00018$ or 1 in 5459.

Hence design of channel is as under:

Base width (b) = 29.33 m

Depth (d) = 1.90 m

Channel slope (S) = 1 in 5469

Velocity (V) = 0.8695 m /sec

CHAPTER 7: ABSTRACT OF COST

7.1 Preparation of cost estimate:

7.1.1 To work out the total cost estimate of the project in detail, the cost of various sub heads should be in a tabular form according to enclosed **annex-6.1**.

7.1.2 Total cost of the should be mentioned along with the price level year.

7.1.3 Cost estimate should be prepared on the basis of latest Schedule of rates of concerned State/UT.

7.1.4 Analysis of rates of non schedule items should be worked out considering the cost of materials, carriage-handling-storing, labour and share of machines involve in execution of work and overhead charges etc. or providing the genuine quotation of the item.

7.1.5 The wages of workers are periodically revised by the State under the statutory labour law. Daily wage rates, therefore, shall be taken as those prevalent in the State at the time of formulation of the project.

7.1.6 For working out the use rates of machinery, the norms for life, depreciation, repair provision etc. should be adopted as per recommendation by the latest CWC Guide Book on use rate, hire charges and transfer value of equipment and spare parts. Price of various equipment should be taken on the basis of recent quotations/ price list of such equipment.

7.1.7 Provision for contingencies and work charged establishment should be considered up to 3% and 2% respectively of the item rates under the C-Works, K-Building and R-Communication sub heads only. These percentage provisions should not be considered on lump sum items.

7.1.8 Quantity certificate regarding the correctness of quantities as per approved design and drawing should be provided by the project authority from the concerned Chief Engineer or equivalent rank.

7.2 Preparation of detailed estimates of costs-I-Works:_ The various items under this head and detailed sub heads for which estimates should be prepared are indicated below:

7.2.1 A-Preliminary: The provision under this head covers relating to various investigations, surveys, Model tests etc. This shall be based on the actual cost likely to be incurred and shall be limited to 1-2% of the total cost of I-works.

7.2.2 B-Land_: The provisions under this head covers acquisition of land etc. The provision shall be made as per actual requirements.

7.2.3 C-Works_: The provisions under this head covers the civil costs of the all flood protection works, erosion works, embankment works, rising and strengthening of existing works or any similar works etc.

7.2.4 K-Building: The provision under this head covers the construction of temporary buildings for residential/ Non-residential and stores shed etc during the construction period of the project. In addition to the cost of buildings, the provision for land development, fencing /boundary walls, security / observation booths, service connection such as water supply / sanitation drainage and electrification may be made as per norms fixed by the State Government. This should be based on the actual cost likely to be incurred and be limited to 2-3% of the total cost of I-works.

7.2.5 M-Plantation:__ The provisions under this head covers the requirement of plantation to augment the stability of banks as well as catchment area treatment to some extent. The provisions shall be made on lump sum basis keeping in view the experience of other projects.

7.2.6 O-Miscellaneous:_ The following provisions are to be kept in this Sub head:

(a) Capital cost of Electrification :

(i) Water supply, purification and distribution

(ii) Sewage disposal

(iii) Fire fighting equipment

(b) Maintenance and service of:

(i) Electrification

(ii) Water supply, purification and distribution works

(iii) Sewage disposal and storm water drainage works

(iv) Security arrangements

(v) Fire fighting equipment

(C) Other items:

(i) Visits of dignitaries

(ii) Boundary pillars and stones, distance marks and bench marks

(iii) Misc petty items

7.2.7 P-Maintenance : The provisions under this head covers the cost of maintenance of works during construction period of project and should be limited to 0.5 % of the total cost of I-works.

7.2.8 R-Communication: The provisions under this head cover the construction of approach roads and quarry roads etc.

7.3 Sub head II-Establishment Charges: In case works to be execute departmentally, the provision for establishment charges should be kept 8 to 10 percent of I-works excluding B-land. No establishment Charges should be provided if works are to be executed on contract basis. In both case, certificate should be provided by the project authority from the concerned Chief Engineer or equivalent rank.

7.4 Sub head III-T&P: The provision under this sub head covers survey instruments, camp equipments, office equipments etc. A nominal provision of Rs 20-25 lakhs under this sub head may be adequate.

7.5 Sub-head IV-Suspense: Generally no provisions should be made under this sub-head unless necessary and well supported by facts and documentation.

7.6 Sub-head V-Receipt and Recovery: This head is meant to account for estimated recoveries by way of resale or transfer of temporary buildings & vehicles shall be accounted for under this sub head. The recoveries in account of temporary buildings may be taken at 15% of the construction cost of building.

7.7 Audit and Accounts Charges: The provision under this Sub head should be made @ 0.25% of I-Work.

Annexure 7.1

Format for Abstract of Cost Estimate

(Amount in .Rs Cr.)
(Price level. year ....)

A	DIRECT CHARGES		Remarks
I	**I-WORKS**		
1	A-Preliminary		
2	B-Land		
3	C-Works		
4	K-Building		
5	M-Plantation		
6	O-Miscellaneous		
7	P-Maintenance		
8	R-Communication		
9	Y-Losses		
	I-WORKS		
II	Establishment		
III	T&P		
IV	Suspense		
V	Receipts and Recoveries (-)		
	TOTAL DIRECT CHARGES		
B	**INDIRECT CHARGES**		
(i)	Audit & Accounts charges		
	Total Cost including Direct and Indirect Charges (A+B)		

CHAPTER 8: BENEFIT COST RATIO

Benefit Cost Ratio should be calculated by dividing total annual benefits/damages by total annual cost of the schemes as under:

B. C. Ratio = Total annual damages/Total annual Cost

Total annual damages:

Average annual benefits may be comprised of the following.

Average actual annual damages on the basis of past years duly certified damage data.

Average actual annual damage= Past annual damages/nos of years------(a)

Anticipated annual damages in absence of the proposed project considering the economic life of project as 50 years. While calculating the anticipated annual damages, extent of probable damages like cropped area, land use, type of properties etc duly certfied by the Competent authority.

In such cases annual anticipated losses should be worked out by dividing the total value of the all type of properties, which is likely to be damages in absence of the project by economic life of project ie 50 years.

Average anticipated annual damage= value of properties likely to be damaged/50. ------------------------------(b).

Total annual damage= Average actual annual damage (a)+ Average

anticipated annual damage (b)

Total annual cost:

Total annual cost of the scheme should be worked out as under:

Interest charges@10% of cost of the scheme.
Maintenance charges@5% of cost of the scheme.
Depreciation charges@2% of cost of the scheme

Total annual cost of the scheme= sum of the above ie @17% of cost of the scheme.

CHAPTER 9: CONSTRUCTION PLANNING:

9.1 General: The construction planning for works envisaged in any flood management/river training works is a vital component for the timely completion of the works avoiding time and cost overrun. Time is of high essence of flood management works as the same has to be completed in available non-monsoon season. Construction Planning becomes part of the overall activity starting from off-setting of monsoon which include (i) vulnerability/damage assessment; (ii) Type of measures to be taken, (iii) Design of structures, construction planning and preparation of DPR, (iv) Administrative approval of DPR and (v) Implementation of the works while keeping sufficient time for each activity. Time to accommodate unforeseen issues should also be kept in mind.

Implementation of a flood management/river training works include invitation of tenders for various works, site survey like latest river configuration, site clearance etc, mobilization of resources like men, material at the site in pre-organized manner for various works.

9.2 Construction Planning It is understood that construction planning is the key for in-time completion of the flood management and river training works. It is seen from the past experience that most of the projects are delayed in completion due to lack of proper construction planning.

For a proper completion of a project, the storage and installation of new innovative material for construction of embankments, revetment, spurs etc. for project specific problem need to be executed under well trained guidance and accuracy. The planning for the same needs to be done considering all the situations like working season, monsoon season, land acquisition, site survey and clearance, procurement of materials etc. The Implementation of project may involve following steps.

9.2.1 Invitation of Tenders Model tender documents for procurement of materials including geo-textile bags, geo-textile tubes, mattress, wire-mesh for various civil works including earth work, boulder work, launching of RCC porcupines should be prepared and used immediately after administrative approval of the project.

9.2.2 Procurement of construction material Construction materials, required frequently in large quantities including boulders, sand, geo-textile bags, geo-textile tubes, mattress, wire-mesh etc. should be procured well in advance preferably during monsoon season to save time. Any additional quantity as per approved DPR may be procured concurrent to execution of works

9.2.3 Storage of construction material at site: There should be proper space/shed for the storage of construction material. The storage space/shed should be such that, there is no risk of wear-n-tear and theft of the construction material till the works are over

9.2.4 Testing of the material: There should be arrangement of testing of the construction material before the start of the work. Provision for standard testing along with procedure of testing should be made a part of the tender document. All the construction material should posses qualifying standards before construction.

9.3 Bar Chart and phasing of the project:

9.3.1 Bar chart showing activities related with execution of the project should be appended with the DPR.

9.3.2 Phasing of the project comprising of physical progress along with financial progress should also be given in the DPR.

9.3.3 Generally project should be completed within two financial years.

Chapter 10 Index map:

Clear index map consisting of following in Arc-GIS/Autocad format on the basis of latest satellite imagery should be appended in the DPR.

10.1 Annual banklines for past 2-3 years in different colours should be marked in the index map.

10.2 Nearby G&D site along with its distance from the start of the project location may be given in the index map.

10.3 Nearby Executed works (green colour) and proposed works (Red color) along with reach lengths, orientation, layout etc as well as benefitted area in yellow color should be marked on the index map.

Chapter 11: Drawings

Following drawings should be appended in the DPR.

11.1 L-section of the river

11.2 X-section of the river at 250 m c/c

11.3 L-section and X-sections of the proposed works

11.4 Satellite/Google images of the river reach 5 km u/s and d/s of the project site.

Chapter 12: Annexure & Certificates

Following annexure & certificates should be appended in the DPR.

12.1 Minutes of latest State TAC meeting in which the proposal was cleared.
12.2 Clearance/No- objection certificate from the State Forest Department.
12.3 Clearance from State Flood Control Board.
12.4 Certificate regarding past damages from Revenue/Agriculture Department.
12.5 E-flow, longitudinal & latitudinal connectivity certificate.
12.6 Correctness of quantity certificate.
12.7 Certificate that separate schemes are not being proposed/planned on the same reach of the river.
12.8 Certificate of the State Finance concurrence.
12.9 Recent Site visit report by the regional office of CWC.
12.10 Soft copy of the DPR in MS-word and MS-excel format in pen-drive/DVD.
12.11 Power point presentation on the proposal.