FLOOD PROTECTION METHODS THEIR APPLICATION AND LIMITATIONS

A DISSERTATION
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of the requirements for the award of the Degree
of
MASTER OF ENGINEERING
in
WATER RESOURCES DEVELOPMENT

By

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CERTIFICATE

Certified that the dissertation entitled " Flood Protection Methods Their Application and Limitations " which is being submitted by Er. R.S. AJWANI, in partial fulfilment of the requirements for the award of the Degree of Master of Engineering in Water Resources Development of the University of Roorkee is a record of the candidate's own work carried out by him under my supervision and guidance. The matter embodied in this dissertation has not been submitted for the award of any other degree or diploma.

This is further to certify that Shri R.S. Ajwani has worked for a period of over nine months in the preparation of this dissertation.

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(Dr.P.P.SAHGAL)

ROORKEE
DATED:

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A C K N O W L E D G E M E N T S

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R.S. AJWANI

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C O N T E N T S

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S Y N O P S I S

Floods are natural phenomena which are caused by the concentration of rain-fall in the catchment of the river, and occur, when the resulting run-off in the form of river flow, exceeds that the river channel can carry within its banks. The safeguards against the floods are flood protection methods. The various methods generally employed are (i) works to store or detain the excess water such as storage reservoirs or detention basins (ii) works to confine the flood flow by embankment (iii) works to increase the capacity of the existing channels such as by river training and cut-offs (v) soil conservation.

The above flood protection methods are normally adopted. The limitations of each of the methods which have to be taken into account while planning flood protection in specified areas are discussed in this dissertation.

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Chapter - 1

INTRODUCTION

Flowing waters of the streams and rivers are the source of wealth to the nation if they are utilised by proper control and training. India has immense water resources which are contributed by network of rivers traversing the sub-continent. The total annual water resources have been estimated as about 1.67 billion cu.m. (1356 million acre ft) (7). The rivers carry most of their discharge during south-west monsoon when heavy and wide-spread rains occurs. It is mainly during this period floods of varying intensity are experienced in one or the other part of the country bringing in their wake considerable loss of life, property and disruption of communications.

Excessive precipitation is beyond human control. Man can, therefore, only attempt to reduce the ill effects of excessive precipitation after it has occurred.

Floods in India affect, on an average, 150 lakh acres annually. The area affected is mostly in the Ganga and Brahmaputra vallies and the deltaic areas in Andhra Pradesh and Orissa, which are rich agricultural tracts in the country. The average annual loss of food production is estimated roughly as about 1 million tons. In addition the flood cause loss of human life and cattle and disruption of normal activities. The average annual flood damage is of the order of Rs.100 crores.

Protection of the cultivated areas from floods will provide a sense of security to the farmer, there by giving him incentive for putting more efforts for increased production. Due to the development of speedy communications, and increased pace of daily life, interruptions to communications by floods cause considerable inconvenience. Measures for keeping the communications safe and uninterrupted have, therefore, become an urgent necessity.

A river in floods spills over its banks due to inadequacy of its normal section to carry the high floods discharges. Damage occurs in the areas where such spilling takes place. The magnitude of damage depends upon the depth and duration of the spill and nature of development of the area. Flood protection measures aim at reducing such damage. The degree of protection that can be given to an area depends upon its importance, the extent of damage that is caused annually and the cost of protection measures. It will, therefore, be obvious that " flood control ", as the term generally employed, does not guarantee immunity from flood at all places and at all times, but only implies protection against floods of specified magnitudes or mitigation of the flood losses.

1.1. Flood Control prior to Planned Development :

Even though the importance of flood protection in general had been realised long ago, control measures had been restricted to the construction of embankments to give protection to the commanded areas under the canal systems in Northern India and the deltiac tracts of east flowing rivers in the State of Orissa, Andhra Pradesh and Tamil Nadu.

The total length of embankments thus constructed was about 4,800 km. (3000 miles). Flood damage, therefore, continued to occur in other areas, leading to an average annual loss of about Rs.100 crores.

Measures for stabilising the river channel for prevention of bank erosion had also been taken. These include construction of spurs and groynes, pitching and revetment. Examples of such works are the bank protection works at Banaras constructed hundreds of years ago, known as Banaras Ghats, and protection works undertaken in the first half of this century on both banks of the Krishna river downstream of Vijayawada Anicut (now Barrage) for the prevention of damage to flood embankments.

1.2. Importance of Flood Control :

The continuous short fall in food production to meet the increasing demands has necessitated all measures which are required for reducing the damage caused to crops by recurring floods. The disastrous floods that occurred in the country in 1954 focussed the need for taking effective measures for mitigating the damage due to floods. On this account, flood control and training have received considerable attention during the successive five years plans of the country. The objective of flood control in India, as in other places, is for mitigating the loss of life and damage to property by floods.

Some of the Indian rivers have special problems. Rivers in North and part of Central India and the tail reaches

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Some of the Indian rivers have special problems. Rivers in North and part of Central India and the tail reaches of some of the large Sohth Indian rivers are mostly alluvial in nature and therefore unstable. The position is aggravated in some rivers like the Brahmputra which are in a seismic zone and bring down considerable amount of sediment. The rocks of the hills where this river originate, are friable and susceptible to erosion and thereby cause exceptionally high silt charges in the river. The continuous and nonpredictable shifting of the river courses causes considerable amount of damage and distress. Huge quantities of sand are some times deposited on the fertile lands during floods. Erosion of river margin leads to the loss of land and threat to town and villages. In order to make the river take the desired course and prevent such losses, river training measures are necessary.

Chapter 2

FLOOD STUDY

Selection of a design flood is an important aspect as this affects both the safety and the cost of any structure, not only those primarily for flood control. Too small a design flood for a major works involves a high risk, not only of total failure of structure and the services rendered by it, but also to the safety of persons and properties located downstream. An excessive design flood, on the other hand, will make the structure costly and may affect its economic feasibility.

Hence, probable maximum flood is taken as design flood when virtually no risk can be accepted. Design flood of lesser magnitude is adopted when the release of water due to structural failure or over-topping will not endanger life or cause disastrous damage downstream and to take probable maximum flood for these works would be uneconomical.

There are two aspects in which flood flow can be studied, viz. its volume and its intensity. Both these are included in the flood hydrograph. In the case of structures like bridges, weirs and barrages, where no absorptive capacity is allowed for, the flood peak has to be passed down without any moderation, and therefore peak value is determined in these cases. In the flood control and storage projects where full flood absorptive capacity is provided the aspect of volume of flood is of greater importance. In projects where floods are not fully absorbed but are only moderated, both the volume and the intensity of the flood flow have to be worked out, that is design hydrograph.

The way in which the flood study be made, depends upon the a able data and the stage at which the project planning stands. Ac rdingly, it may be tentative, preliminary or final.

Tentative study is made before the field investigation of the project commences and will be only approximate. It can be based on the formulae or ideas got from design flood figures of other dams in the region. For preparing of preliminary project report, study will be relatively more detailed. This can be carried out by flood formula, enveloping curves and frequency analysis. Detail ed flood studies will be carried out with all the information avail able for the basin, the design hydrograph so obtained is made use of for reservoir flood routing studies, final spillway capacity &c.

Design Criteria to be adopted in the Design of Dams and Embankments for Flood Control

In the design of dams for flood control it is usually not feasible from the economic point of view to provide complete protection against servere floods occurrence. The normal practice is to provide detention storage sufficient to absorb the standard project flood. This can be defined as the flood resulting from a most severe storm or meteorological conditions considered reasonably characteristic of the region excluding extremely rate combinations. All the flood-producing storms in the region are tabulated and about 10 percent of the highest storms of record are omitted. The next lower one is taken as a storm for the standard Project Flood.

For the safety of the dams with the storage of over 61.7 million cum. (50,000 acre-ft) the spillway capacity provided should be sufficient to pass the maximum probable flood which is the maxi-

-mum flood for which there is reasonable chance of occurring at site. This is to be determined by the unit hydrograph method for the maximum storm. The latter is obtained from storm studies of all the storms that occurred in the region, maximised for all conditions including moisture content.

In case of planning, there may be some projects where there is is hardly any discharge data available. In such circumstances, for preliminary studies the peak flood may be estimated by empirical formulae. The empirical formula commonly used in central and northern India is the Dickens formula; in south India, Ryves formula; and in Maharashtra, Inglis formula.

For permanent barrages, and minor dams with less than 50,000 ac-ft storage, the standard project flood or a 100-year flood, s whichever is higher, is to be adopted.

Flood embankments may be designed to pass a flood with a return period of 100 years depending on their importance.

Methods generally applied for Flood Estimation

The determination of the spillway design discharge is of importance because it involves the structual safety of the dam itself. A design flood is the flood adoptedfor the design of a structure after considerations of economic and hydrologic factors. In general, there are three basic approaches to solve the problem of estimation of design flood:

(i) The empirical approach

(ii) The Hydrometeorological approach, utilising the theory of Unit Hydrograph; and

(iii) The statistical approach involving the frequency analysis of observed flood.

(i) Empirical Approach: Based on experience and observations on some catchments, engineers have attempted a correlation between the peak flood flow and the drainage area. Many formulae are available but the most commonly employed are those derived by Dickens, Ryves and Ingli. These formulae help one to get an approximate value of the flood peak and not the complete design flood hydrograph, and their use is restricted to preliminary designs only.

i. Dickens formula

$Q = CM^{3/4}$ $\qquad$ Q = Peak flood in cumecs

C = Coefficient

M = Catchment area in sq. km.

Value of coefficient C varies from $1\frac{2}{3}$ to $10\frac{1}{2}$ on large Indian rivers - actual observed values are upto 22. This formula is applicable to northern and central India.

ii. Ryves formula

$Q = CM^{2/3}$ $\qquad$ (Symbols same as above)

Value of coefficient C according to Ryves 6.8 within 80 km of coast; 8.3 for areas between 80 and 2400 km from coast; 10.0 for limited areas near the hills. Actual observed values upto 37. This formula is applicable to southern India.

iii. Inglis formula

$Q = \frac{125\ M}{\sqrt{M+4}}$ $\qquad$ (Symbols same as above)

Applicable to Bombay region (for fan shaped catchments)

(ii) Unit Hydrograph method: This method is based on the rational analysis of rainfall and runoff data, it gives most reliable estimates of flood for design purposes. Also, this method provides the complete hydrograph and not merely the peak discharge as obtained by empirical and statistical methods and hence for design of storage reservoirs, this has advantage over other methods.

Following steps are involved in application of the unit hydrograph method for calculation of design flood:

i) Analysis of observed flood hydrographs and their subdivision into base flow and direct runoff hydrographs.

ii) Analysis of rainfall data related to observed flood hydrographs.

iii) Derivation of storm rainfall-runoff relationships from observed rainfall-runoff records.

iv) Derivation of unit hydrograph and selection of unit hydrograph for design flood computations.

v) Computation of design flood hydrograph based on data derived from above studies and the design storm.

This method is discussed in detail with illustrative example. ~~in~~

(iii) Frequency analysis method: This statistical method is adopted in cases where sufficiently long observed peak discharge data, say 25 years, are available. These methods are essentially based on the study of past flood data. Making use of behaviour pattern of past events and transposing what has already been experienced, an attempt is made to predict the future pattern. The data are assumed to be a set of independent values, homogeneous and free from any long-term cyclical and trend features.

The method of analysis consists of working out a suitable law of probability or frequencies in regard to the occurrences of different flood intensities and extrapolating it to remote frequencies.

Assumptions made by different authorities are not similar and the law suggested by Gumbel from extreme value theory has been found to be most successful. The expression is written as -

$$X_T = A + B \log_{10} \log_{10} \frac{T}{T - 1}$$

where,

X_T is the peak flood with return period T

A and B are constants to be determined from the data.

Flood Estimation by Unit Hydrograph Method

The probable maximum flood is derived from the probable maximum storm by application of the theory of Unit Hydrograph. The probable maximum storm is an estimate o. the physical upper limit to the storm rainfall over a basin. It is generally determined by storm transposition and maximisation techniques.

Depth-duration curves of the past severe storms which have actually occurred over the catchment area prepared and maximised for maximum moisture charge. The envelopment of the maximised depth.duration curves is taken as depth-duration curve for the probable maximum storm. Maximum possible rainfall depths over the given drainage area can also be found by Isohytal method. By this method, isohytes for the maximum storm that has occurred over the basin area are drawn. Then by placing these isohytes of largest storm over the catchment area of the stream under consideration, the maximum rainfall is calculated over the given area.

Minimum infiltration loss rate applicable to the basin is computed from the rainfall and runoff records of the past. This is applied to the design storm to get the rainfall excess quantities.

Rainfall excess quantities (obtained after allowing infiltration losses) are then arranged in such a way as would be necessary to produce the worst effect. Then design flood is calculated by applying this sequence of excess rainfall depths to the Unit Hydrograph.

An Illustrative Example of Derivation of Maximum Probable Flood by Unit Hydrograph Method

Data: Catchment area of one stream of the whole river basin = 268 sq. miles (695 sq. km)

Two storms have been selected, viz. September 1966 dates 6 and 7, and September 1969 dates 9 to 11. These hydrographs are shown in Fig.2.1. A base flow from the observation of discharge is assumed as 500 cusecs.

In hydrograph of September 1966 dates 6, 7, a recession separation curve is drawn based on discharge observation and after knowing the general recession trend of the river basin.

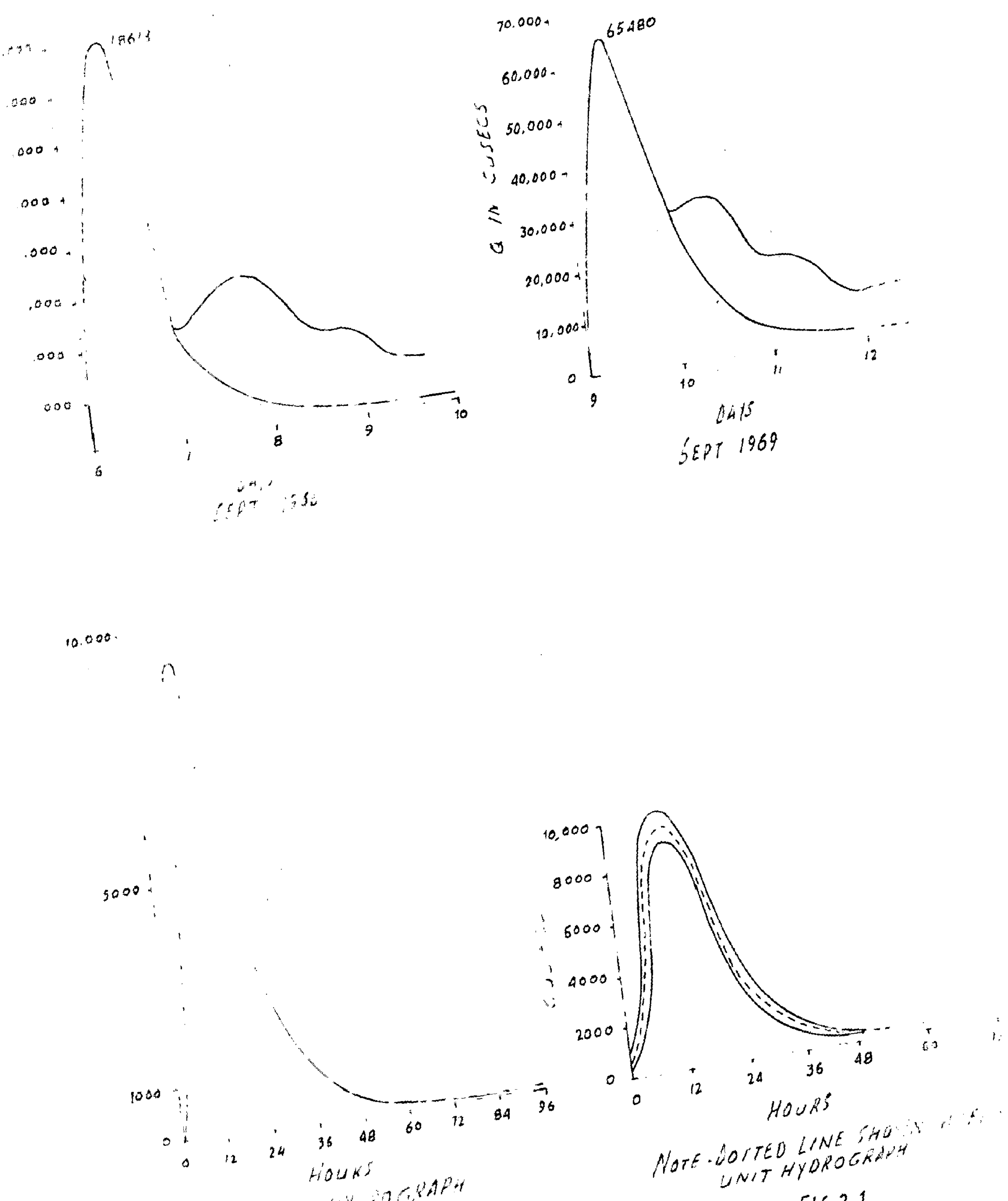
A runoff of 7.73 in. is calculated.

This runoff has been obtained against an average rainfall of 263 mm = 10.35 in. fallen on 7th September.

The percentage of discharge works out to 74.5%.

The Unit Hydrograph ordinates have been worked out by dividing ordinates of the observed hydrograph less base flow to the runoff by 7.73 in.

Similar Unit Hydrograph ordinates are worked out for the other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HOURS
UNIT HYDROGRAPH
10,000
8000
6000
4000
2000
HOURS
NOTE - DOTTED LINE SHOWS
UNIT HYDROGRAPH

FIG 2 1

storm dated 9th to 11th September 1969 as shown in tables 1 and 2 and average unit hydrograph is drawn for both unit hydrographs as shown in Fig. 2.1 ~~p. 13~~.

Table 2.1 - Area of the Catchment = 268 sq. miles.
Hydrograph of September 6 to 10, 1966

Date	Hours	Runoff in cumecs	Base Flow cusecs	Surface flow cusecs	Av. surface flow cusecs	Period between two flows	Discharge in cusecs hours
6	8	2160	2160				
	20	79613	1400	78213	39106	12	469272
7	8	21176	1150	20026	49120	12	589440
	20	8500	750	7750	13888	12	166656
8	8	3500	500	3000	5375	12	64500
	20	1900	500	1400	2200	12	26400
9	8	1100	500	600	1000	12	1200
~~20~~	20	750	250	250	425	12	5100
10	8	500	500	0	125	12	1500
						Total	1334868

$$\therefore \quad \text{Runoff in inches} = \frac{1334868 \times 3600 \times 12}{268 \times 5280 \times 5280} = 7.73 \text{ inches}$$

$\therefore$ Unit Hydrograph ordinates:

Hours	Ordinates in cusecs
0	0
12	10090
24	2585
36	1000
48	388
60	181
72	72
84	32
96	0

TABLE - 2.2

Catchment Area = 268 sq.miles

Hydrograph for 9 to 11 Sept. 1969

Date	Hour	Runoff	Base Flow	Surface runoff	Av.surface run off	Period between two flows hours	Discharge in cusec Hours
9	8 AM	11700	11700	0			
	20	65480	3880	61600	30800	12	3,69,600
10	8 AM	30400	2260	28140	44870	12	5,38,440
	20	9700	1425	8275	18207	12	2,18,484
4	8 AM	3527	954	2573	5424	12	65,088
	20	1920	853	1067	1820	12	21,840
12	8 AM	1026	853	173	620	12	7,440
	20	853	853	0	86	12	1,032
							12,21,924

$$\therefore \quad \text{Runoff} = \frac{12,21,924 \times 3600 \times 12}{268 \times 5280 \times 5280} = 7.06''$$

Unit Hydrograph Ordinates

Hours	Ordinate in Cusecs
0	0
12	8800
24	4020
36	1185
48	368
60	152
72	25
84	0

Maximum storm observed so far pertains to August 6, 7, 8, 1968, not on this stream catchment but some distance away in the basin area. Isohytal maps for 1 day, 2 days and 3 days maximum rainfall are prepared of this area by making use of available rainfall stations in the basin.

By transposition of this storm (isohytal pattern) to a critical position over agiven drainage area without major changes in pattern of chronology of rainfall increments, we get the maximum rainfall by isohytal method for 1 day, 2 days and 3 days as shown below.

Fig. 2.2 shows the transposition of severe storm of 1 day, 2 days and 3 days maximum rainfall isohytals on the given drainage area.

Storm distribution at every 6 hours unit period.

Period	1 day inches	2 days inches	3 days inches
Av. rainfall calculated by isohytal method	17.25	30	32.2
Add 20% moisture maximisation	3.45	6	6.44
	20.70	36	38.64

Storm distribution as per nearby observatory

(i) Max. 3-day rainfall depth = 38.64 in.

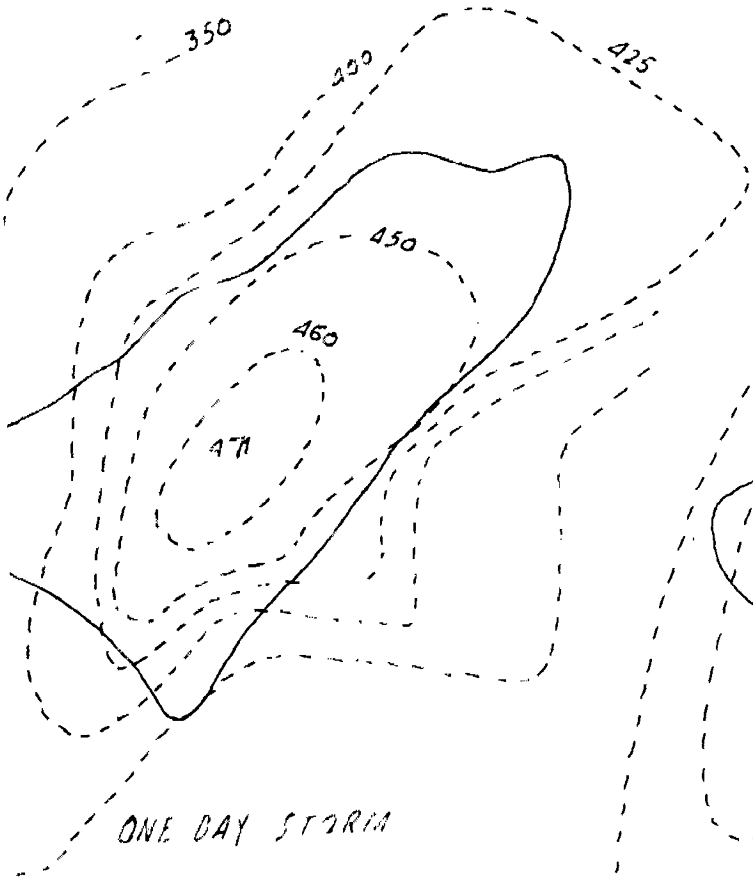
(ii) Max. 2-day rainfall depth = 36.00 in.

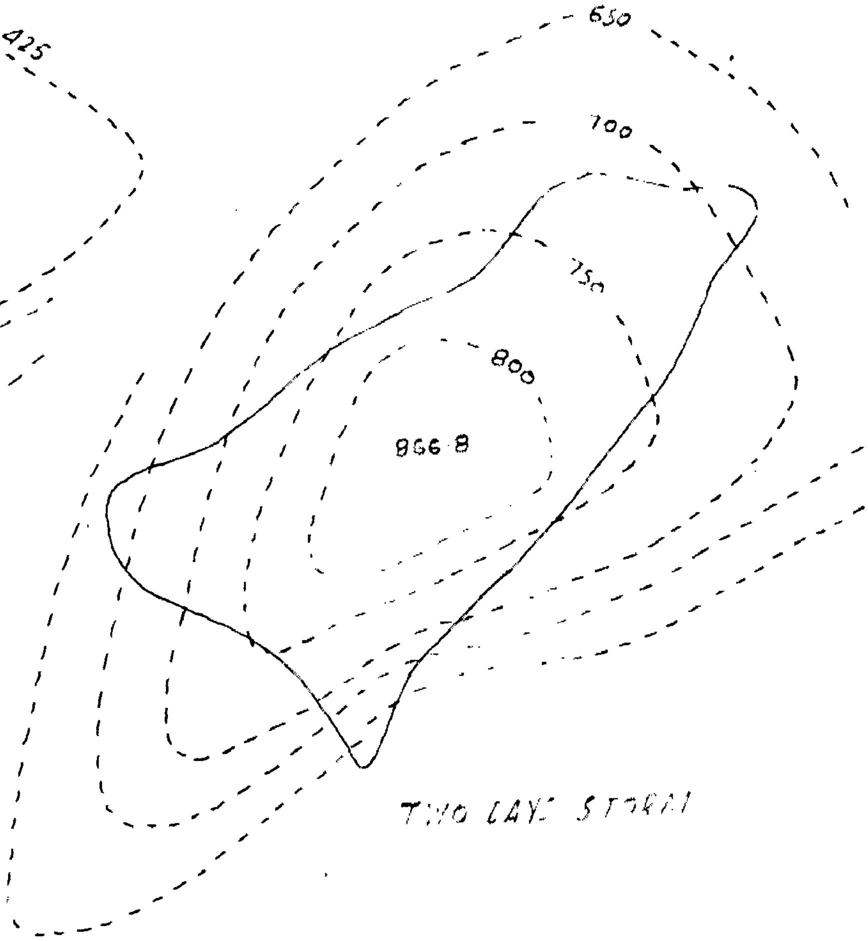
(iii) Max. 1-day rainfall depth = 36 x 0.83 = 29.9 in.

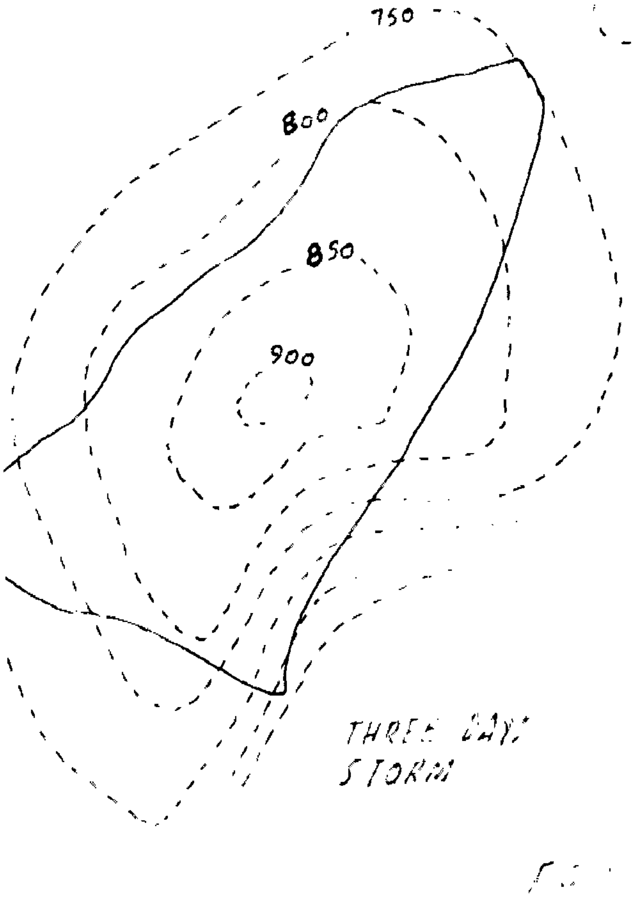
(iv) Max. 12-hour rainfall depth = 36x0.604 = 21.7 in.

(v) Max. 6-hour rainfall depth = 36x0.39 = 14.0 in.

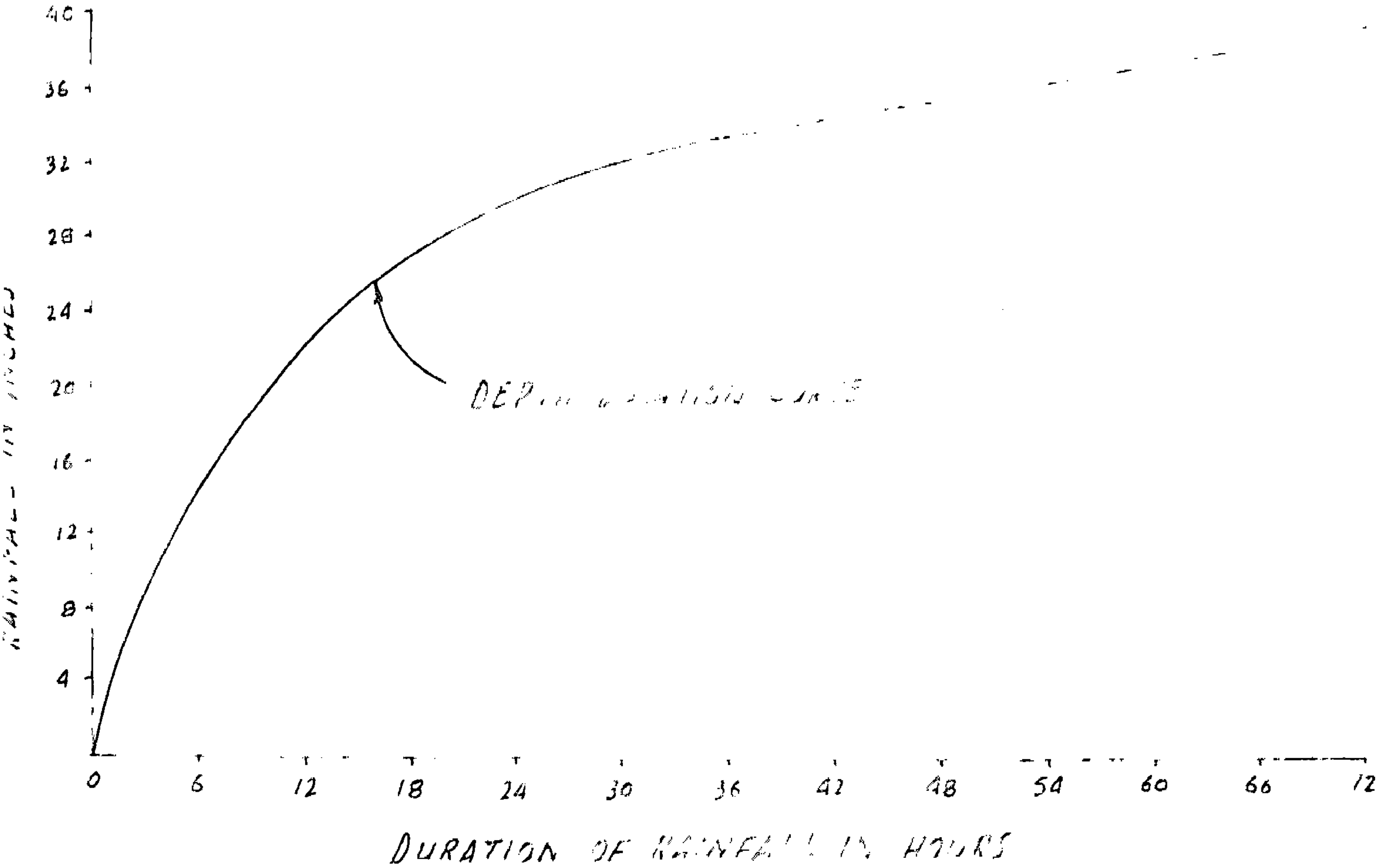
From the graph plotted for above depth versus duration FIG. 2.3 (depth duration curve), rainfall at every 6 hours are worked out.







NOTE - THICK LINE INDICATES BOUNDARY
LINE OF CATCHMENT AREA OF STREAM
UNDER COSIDERATION
CATCHMENT AREA = 268 SQ. MILES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60
66
72
DURATION OF

Table 2.3 - Rainfall Excess

(For this study runoff coefficient is taken as 0.8)

Hours	Rainfall (ins.)	Rainfall excess at 80%	Incremental increase
6	14.00	11.20	11.20
12	21.70	17.35	6.15
18	26.50	21.10	3.75
24	29.90	23.90	2.80
30	32.20	25.70	1.80
36	34.10	27.20	1.50
42	35.20	28.10	0.90
48	36.00	28.70	0.60
54	36.80	29.40	0.70
60	37.50	29.90	0.50
66	38.00	30.30	0.40
72	38.64	30.90	0.60

Derivation of maximum peak

The critical order of rainfall incremental excesses have been fixed as below in order to obtain maximum peak:

Table 2.4

Hours	Incremental rainfall excess inches	Hours	Incremental rainfall excess inches
0	0.0	42	1.50
6	0.4	48	1.80
12	0.5	54	2.80
18	0.6	60	6.15
24	0.6	66	11.20
30	0.7	72	3.75
36	0.9		

Max. flood hydrograph is obtained by applying incremental excess to the unit hydrograph ordinates as shown in table 2.5

Max. peak flood works out to be = 1,91,157 cusecs

Hrs.	Rainfall — Unit Hydrograph Ordinates	11.20	3.75	Total	Base Flow	Maximum probable flood in cusecs
0	0			0	500	500
6	4750			1900	500	2400
12	9500			6175	500	6675
18	8400			10900	500	11400
24	3100			13990	500	14490
30	1800			16335	500	16835
36	1100			19165	500	19665
42	750			25345	500	25845
48	400			34805	500	35305
54	250			48460	500	48960
60	150	0		78847	500	79347
66	120	[illegible]3200	0	145452	500	145952
72	80	[illegible]6300	17800	190657	500	191157
78	40	[illegible]2568	35600	156131	500	156631
84	0	[illegible]4720	31500	82790	500	83290
		20160	11625	42046	500	42546
		[illegible]2320	6750	25668	500	26168
		8400	4125	16235	500	16735
		4480	2813	9623	500	10123
		2800	1500	5762	500	6262
		1680	938	3652	500	4152
		1344	563	2511	500	3011
		896	450	1592	500	2092
		448	300	748	500	1248
		0	150	150	500	650
			0	0	500	500

TABLE -

MAXIMUM FLOOD

Hrs.	Rainfall excesses in inches in the critical designed sequence						
	Unit Hydrograph Ordinates	0.4	0.5	0.6	0.6	0.7	0.9
0	0	0					
6	4750	1900	0				
12	9500	3800	2375	0			
18	8400	3360	4750	2850	0		
24	3100	1240	4200	5700	2850	0	
30	1800	720	1550	5040	5700	3325	0
36	1100	440	900	1860	5040	6650	4275
42	750	300	550	1080	1860	5880	5550
48	400	160	375	660	1080	2170	7560
54	250	100	200	450	660	1260	2790
60	150	60	125	240	450	720	1620
66	120	45	75	150	240	525	990
72	80	32	60	90	150	250	625
78	40	16	40	72	90	125	360
84	0	0	20	48	72	105	225
			0	24	48	84	135
				0	24	56	108
					0	28	72
						0	36
							0

Chapter - 3

FLOOD PROTECTION METHODS

A river in floods spills over its banks due to inadequacy of its normal section to carry the high flood discharges. Damage occurs in the areas where such spilling takes place. The magnitude of the damage depends upon the depth and the duration of the spill and the nature of the development of the area. Flood protection measures aim at reducing such damage. The degree of protection that can be given to an area depends upon its importance, the extent of damage that is caused annually and the cost of the protection measures. It will, therefore, be obvious that " flood control ", as the term generally employed, does not guarantee immunity from flood at all places and at all times, but only implies protection against floods of specified magnitudes or mitigation of the flood losses.

The flood protection methods can be classified under two categories, viz., administrative and engineering.

i) Administrative Methods :

Flood warning system and Flood plane zoning. These aim at reducing the flood damage by the timely evacuation of the population and movable property liable to damage and restricting the use of land and building activity in the areas subject to floods.

Under the flood warning system, the likely damage centres are alerted well in advance of the actual arrival of floods, to enable the people to move to safer places or to raised platforms specifically constructed for the purpose. Flood forecasting is a useful tool in the working of an efficient flood warning system.

The aim of flood plain zoning is to demarcate the areas that are liable to be affected by floods of different magnitudes and frequencies. This facilitates the organisation of developmental activities in the different zones in such a manner that the inconvenience and damage due to floods are minimised.

ii) Engineering methods :

These can be divided into two categories. The first category of works aim at reducing the magnitude of the flood flows at the damaged centre, thereby reducing the spill and consequent damage. Reservoirs, detention basins, diversions and soil conservation measures, come under this type of works. Reservoirs aim at reducing the peak discharges of floods up to a specified magnitude in such a manner that the discharges from the storage reservoir, together with the intermediate contributions, do not exceed the bankfull capacity at the places where the protection is desired. Detention basins moderate the floods downstream by storing part of the flood waters during high peaks. Diversions and flood ways take away the part of the flood flows to another basin or to a depression where it could be stored for use .

Chapter 4

ADMINISTRATIVE METHODS

4.1. Flood Forecasting :

Flood forecasting is used for alerting the likely damage centre in advance of actual arrival of floods to enable the people to move to safer places, for taking advance activities in the petrolling of flood control works and in the operation of flood control reservoirs for optimum benefits.

4.2. Necessity of Flood Forecasting and Flood Warning:

There is always possibility of a flood higher than the design flood coming down in the future.Therefore, while flood protection measures like embankments, reservoir etc., have their own place, it is necessary to take steps to reduce damages to life and movable property from any unexpected floods to the minimum, by being prepared for it. This can be achieved by system of flood forecasting and flood warning, which provides advance notice of an incoming flood, enabling thereby a certain amount of time for living beings and movable property to be taken out to safer places. With this system the petrolling of flood protection works can be intensified during high stages of rivers thereby saving these from breaches, failure etc. With reliable forecasting it is possible to safeguard the dam against an unexpected flood of higher return period than for which the spillway might have been designed. Some times reservoirs can be held at higher level for power or other purposes, pending a higher flood and lowered in time to receive the incoming floods. Similarly, encroachment into the power storage can be made to increase the flood control capacity when the town areas have been flooded due to heavy local rains. Flood forecasting thus increase the overall potentialities of a multi-purpose project. The basis of forecasting is the hydrological and hydrometeorological data in and around the catchment covering rainfall its distribution and river stages from network of reporting stations.

4.3. Illustrative Example :

4.3.1. Flood forecasting on the river Yamuna for Delhi :

On the right bank of river Yamuna is situated an important town Delhi having population of 35 lakhs. Another big town named Shahadara with a population of 5 lakhs is on its left bank. These are protected from the floods of the Yamuna by embankments and in some portions by the National Highway acting as flood bank. Also 45 villages situated along the banks of river which are threatened by floods every year. Whenever the level of the Yemuna at Delhi Railway Bridge exceeds RL 672.00 ft. above sea level there is spilling in the unembanked reaches upstream of the bridge. The villages situated on the river margin get marooned, necessitating measures for evacuating the affected people to safer places. Also, during floods,

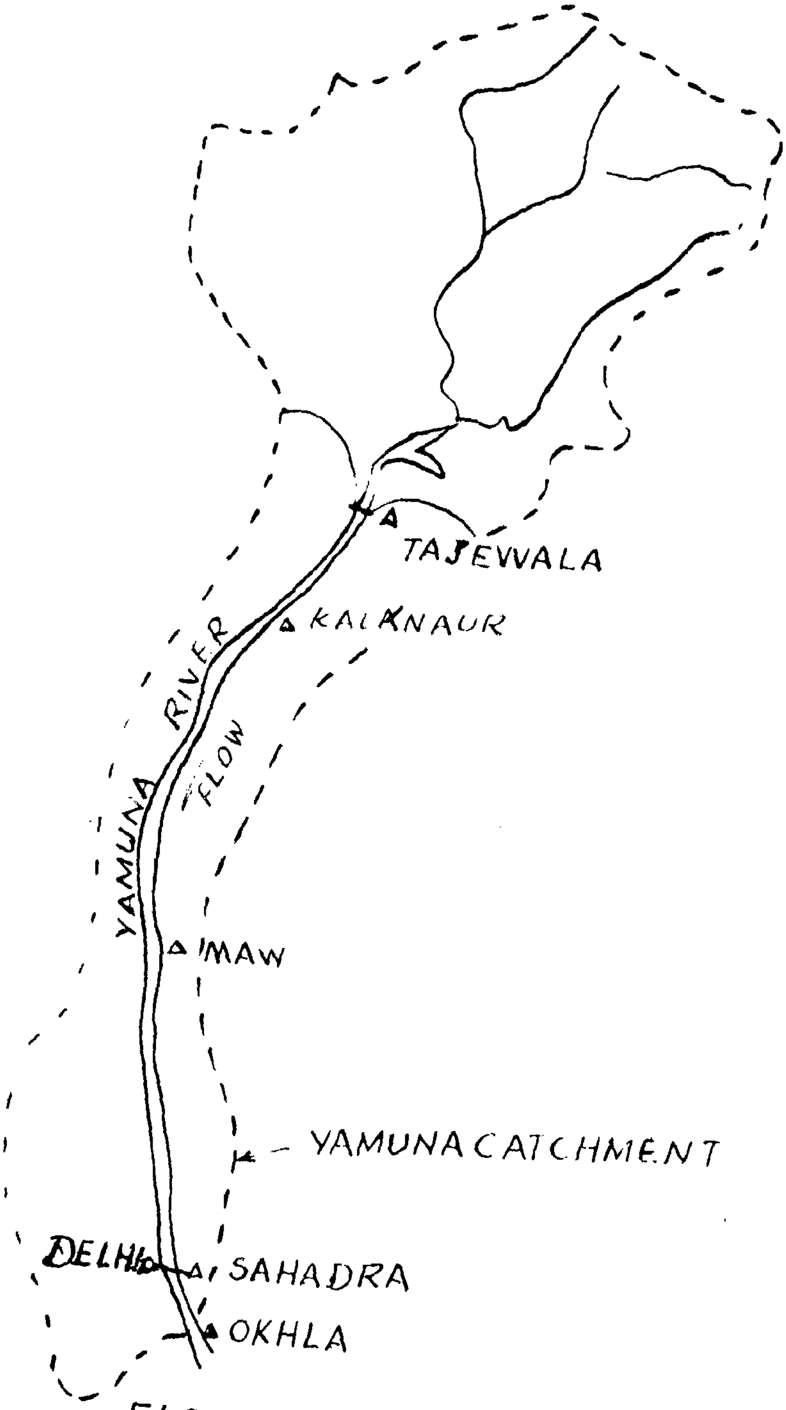

TAJEWALA
KALKNAUR
YAMUNA RIVER
FLOW
MAW
YAMUNA CATCHMENT
DELHI
SAHADRA
OKHLA

FIG 4.3

proper regulation of the Wazirabad and the Yamuna Barrages is required in the interest of their safety and of the embankments. The sluices on the drains out falling into the Yamuna have to be properly operated to prevent back-flow from the Yamuna into the city areas. The patrolling and maintenance operations along the embankments have to be intensified with a view to ensure their maximum safety. These have all pointed to the necessity of having a well-organized system of flood forecasting.

4.3.2. The Yamuna River :

The Yamuna rises from the Jamonotri springs and after flowing in a south-westerly direction for about 160 km (100 miles), it is joined by the Tons and then by Giri. The river debouches into the plains near Tajewala, where a weir has been in existence for the past 100 years. Downstream of Tajewala, no major stream joins the Yamuna and the river slowly winds its way for 240 km (150 miles) in alluvial plains before flowing through Delhi. The catchment area of the river above Tajewala covering about 11060 sq.kms. (4320 sq. miles) is fan-shaped and is hilly. Below Tajewala and upto Delhi, there is a ctachment of 8,244 sq.kms. (3220 sq. miles) which is ribbon shaped and consists of sandy plains. The river experiences floods of high intensity and of short duration at Tajewala. Upto Kalnaur 39 kms. (24 miles) downstream, there is no appreciable change in flood intensities, but lower down, river spread out considerably, causing large reduction in flood peaks. It has been observed that a peak discharge of about 12750 m^3/sec. (4.5 lakh cusecs) at Tajewala gets moderated to about 71,00 m^3/sec. (2.5 lakh cusecs) at Delhi after a travel of about 240 km. (150 miles).

Flood forecasting unit was set up in the Central Water and Power Commission in 1958. This Unit made studies of all the available data of gauges and discharges at Tajewala and Okhla at Delhi. Since 1890, and the gauges at Kolanaur Railway Bridge since 1947. The coaxial correlation between Kalanaur about 192 km. (120 miles) upstream of Delhi have been developed using the daily gauges and daily rainfall data.

With these diagrams, forecasts were issued about 36 hours in advance. The gauge and rainfall data subsequently collected have been used continuously for developing improved correlation. Similar correlation diagrams have also been developed for Delhi and Mawi - 106 kms. (66 miles) upstream of Delhi and between Delhi and Kutana about 64 kms. (40 miles) upstream of Delhi. (Fig. 43).

The hydrological and metrological data received in control room are processed and analysed. The first forecast is made from the correlation diagrams for Kalanaur and Delhi, keeping the following in view :-

1. The current Delhi gauge which may influence the gauges two days later.
2. The rise and fall of Kalanaur gauge in the previous 48 hours.
3. The antecedent precipitation index in the catchment between Kalanaur and Mawi.
4. The rainfall in the catchment between Kalanaur and Mawi.

The forecast on this basis is generally made at about 3.00 p.m. which will be valid approximately 36 to 48 hours later. Correction, if any, in the first forecast is made about 12 hours later based on the gauge data received by wireless from Mawi. The final correction is made after another 12 hours when the flood wave has travel upto Kutna.

4.3.3. Development of Correlation Charts :

Two stations 'A' and 'B' the distance between them along the river as 125 miles are taken into consideration. Time lag has been observed as 48 hours. 'A' being in the upstream side of river. The hydrographs for these two station are plotted from the previous available data. The ordinate at N^{th} hour of flood hydrograph at 'A' is taken corresponding to the ordinate at (N plus 48th) hour in the flood hydrograph at station 'B' and the series of corresponding points are prepared. The points selected are only from the rising limb since the recession curve is not of any importance.

In preparing the chart for the reach 'A' to 'B' the current discharge at 'B' i.e. discharge at 'B' at any Nth hour is plotted on x- axis and discharge at 'B' at (N plus 48th) hour is plotted on Y-axis. Every plotted point is given a value of the corresponding Nth hour discharge at 'A'. After plotting all such points, lines of equal values of 'A' discharge are drawn as contourlines (Fig.4.1).

This graph can be used to obtain the flood discharge at 48 hour later at 'B' from given discharge at 'A'.

In the above chart contribution from the intervening catchment is not taken into account. This can be taken into account by taking two more parameters (i) Rainfall over the intervening catchment and (2) the moisture content of the catchment (Anticedent precipitation Index). Co-axial correlations between Kalanaur and Delhi gauges are shown in Fig.4.2

4.3.4. One Example of forecast issued for the flood is given below :

Kalanaur gauge 120 miles upstream of Delhi is reported to be high, which Delhi gauge is low, with wide spread moderate to heavy rainfall in the catchment. Forecast has been issued on 7th August 1966, for the following day, 8th August on the basis of data available on 7th August.

1.	Delhi gauge on N^{th} day	= 670.1 ft.
2.	Kalanaur gauge on N^{th} day	= 876.0 ft.
3.	°Aug. Kalanaur gauge on N^{th} day	= 873.5 ft.

° Aug. Kalanaur gauge is the average gauge at Kalanaur for the last 48 hours.

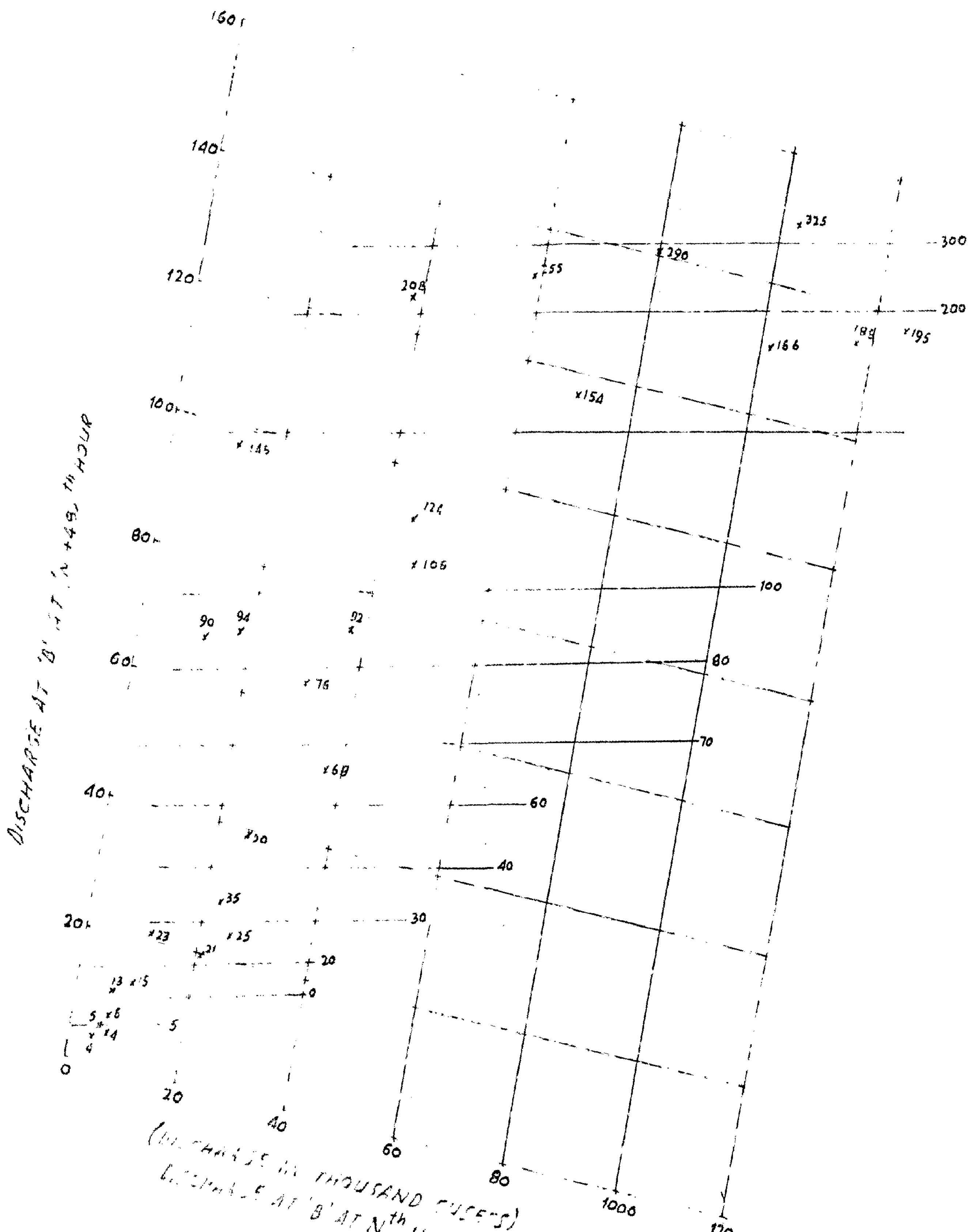

DISCHARGE AT 'B' AT 'N +4' th HOUR
(DISCHARGE IN THOUSAND CUSECS)
DISCHARGE AT 'B' AT N th HOURS

FIG 4.1

4.	Antecedent precipitation index on Nth day (Kalanaur to Mawi)	= 5.08 in.
5.	Rainfall on Nth day (Kalanaur to Mawi)	= 2.15 in.
6.	Forecace from the diagram (Fig.4.2)	= 673.1 ft.
7.	Actual gauge abserved on 8.8.66	= 673.25ft.
8.	Difference between the forecast and actual gauge	= 0.15 ft.

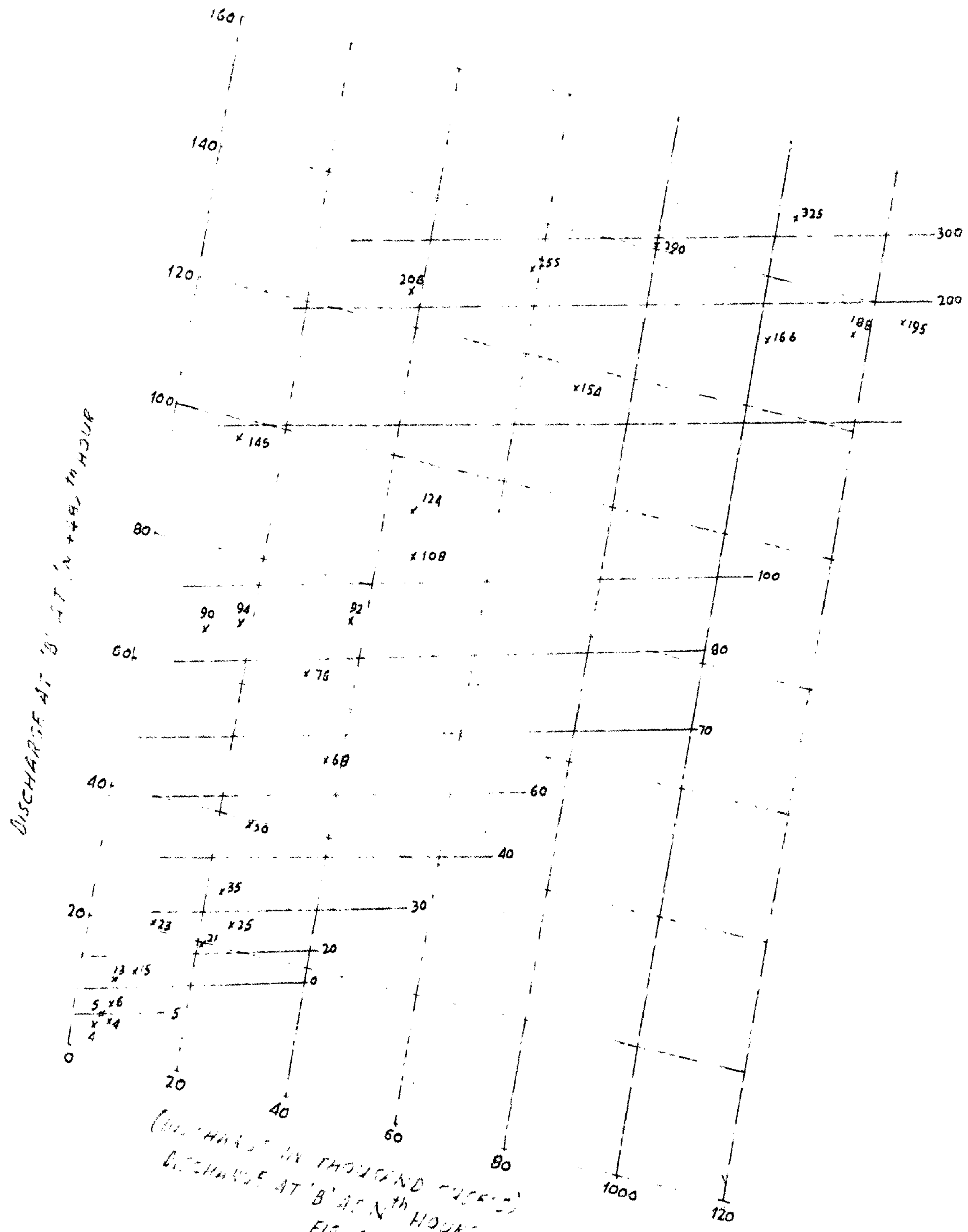
Forecasts formulated and issued about 48 hours in advance on the basis of Kalanaur gauges are subjected to modifications on account of changes in weather conditions in the lower catchment. The revised forecasts are then issued on the basis of conditions at Mawi and Kutana (Fig.4.3) about 34 hours and 12 hours respectively in advance.

4.4. Limitations :

Flood forecasting plays an important role in giving timely warning for evacuation of population and movable property to safer place before the arrival of the floods but it will require efficient telecommunication and wireless systems. Its effectiveness depends upon the time that will be available after the receipt of the warning for evacuation measures in the damaged centre. Adequate data are, however, required for developing a reliable method of flood fore-casting to make it successful.

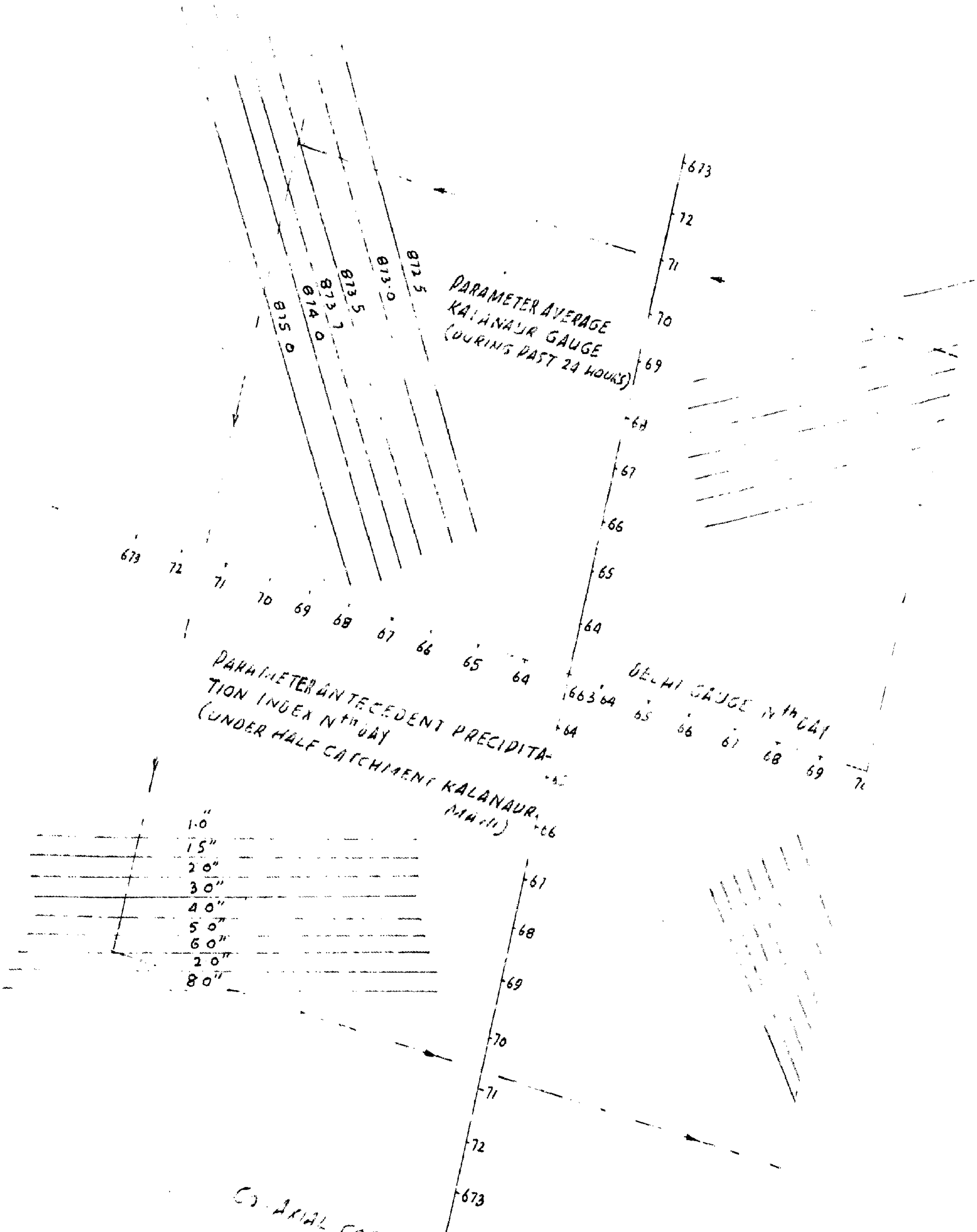
4.5. Flood Plain Zoning :

A flood plain is a portion of river valley which gets covered with water when the river over flows its bunds



DISCHARGE AT 'B' AT (N+4) th HOUR
(DISCHARGE IN THOUSAND CUSECS)
DISCHARGE AT 'B' AT Nth HOURS

FIG. 11



CO-AXIAL CORRELATION BETWEEN KALANAUR AN

FIG 4.

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4.5. Flood Plain Zoning :

A flood plain is a portion of river valley which gets covered with water when the river over flows its bunds during floods. The flood plain is formed by the river during the process of lateral swinging and building up of its valley by silt deposition over a period of time during floods. It is only for a short period during monsoon that the river over flows its banks and occupies the flood plain.

From the agricultural point of view, the flood plain, on account of silt deposition, is the most fertile part of the river valley. The proximity to the navigational facilities is another important factor in case of many big rivers for the popularity in the use of flood plains. Also, rivers are a valuable source of industrial and demestic waters. Thus, for various reasons, the flood plains are generally the most highly developed areas in river basins both from the agricultural and industrial view points.

Flood plain zoning means sensible adjustment of land use that is our efforts to adjust land use to the river instead of following the very costly process of adjusting the river to our use.

4.6. Demarcation of aflood plain :

Zoning of flood plain will have to be decided on the basis of a detailed study of the engineering information mentioned below :

(a) A topographical map of the flood plain, indicating thereon the innundation limits of floods of record.

(b) A map of flood plain, delineating the extent of innundation corresponding to low, medium and high floods respectively. Although there can be any set rules for determining these catagories of flood for all places, depending upon past records of damage by floods, these may be divided as low, medium and high flood categories at any particular place.

(c) A map of flood plain, indicating the magnitude of floods of various frequencies like 5- year, 10-year and 15-year.

(d) A drawing of flood frequency curve, indicating the frequency of past floods of different magnitudes and the probable frequency of future floods.

(e) A hydrograph of some major floods in the past, from which can be obtained such engineering information, as the rate of rise, the period of flood crest, and the duration of flood flow.

(f) Data on monetary extent and type of flood damage previously experienced, and if possible, a graphical correlation of the flood damage on the one hand and the river gauge or discharge on the other.

4.7. Restrictions in Flood Zones :

(i) Flood detention reservoirs stand empty for a considerable period, and it is often desirable to utilise the lands, within the reservoir areas. Such areas can be utilised for the purpose of cultivation, but human habitation should not be permitted on sites located lower than 5 ft. (say), below the spillway crest.

(ii) In unprotected areas or semi-protected areas, public structures, institutions, Government structures, or permanent residential habitations should not be permitted.

(iii) Conservation of areas subject to floods to uses for which the potential loss would be substantially lower in value. This would involve acquisition of lands in question with a view to convert them into parks, seasonal agricultural and grazing plots etc.

The owners of lands vulnerable to floods can not be said to have a right to put their lands to dangerous uses, and then expect the works of rehabilitation and relief of the persons affected by floods to be carried out at the expense of nation at large.

4.8. Conclusion :

Thus, flood plain zoning is very desirable and should in fact, form a part of any integrated flood programme. Action for flood plain zoning should be taken in hand simultaneously with measures for flood protection.

4.9. Limitations :

Occupancy of the flood plain for agricultural, industrial and urban development, has always been attractive due to various advantages such as fertile nature of the soil, water supply and river transport facilities. Consequently, inspite of the threat of occasional flooding, considerable encroachment has taken place in the flood plains in several areas. While it is possible to demarcate the flood zones in such areas and to legally restrict the development, it may not be easy to enforce it in the already developed areas, both on account of the likely public agitation and the magnitude of the rehabilitation programme that it will invalue. Flood zoning can, therefore, be adopted for restricting the future development areas, either suitable engineering measures have to be adopted or the damage reduced by taking timely action for evacuation of people and movable property before the arrival of floods.

Chapter - 5

RESERVOIRS, DETENTION BASINS AND DIVERSIONS

5.01 Concept :

The function of flood control reservoir is to store a portion of flood so that the peak rate of discharge is reduced towards down stream by absorbing a part of the flood volume when the flood is rising and releasing the same volume gradually when the flood is receding. The provision of artificial storage on a river by construction of a reservoir on it is thus the direct method of reducing flood stages and flood damage downstream. With reference to Fig.(5.1) the firm line represents the natural hydrograph of a flood at the dam site and dotted line the hydrograph as moderated by the reservoir. It follows that the reservoir should be able to hold in storage the volume of water represented by the hatched area between the two curves which represents the excess of inflow over outflow during the rising stage, which would then be gradually released during the falling stage.

5.02 Design Criteria :

In the design of dams for flood control it is usually not feasible from the economic point of view to provide complete protection against severe floods occurrances. The normal practice is to provide detention storage sufficient to absorb the standard project flood. This can be defined as the flood resulting from a most severe storm or metrological conditions considered reasonably characteristic of the region.

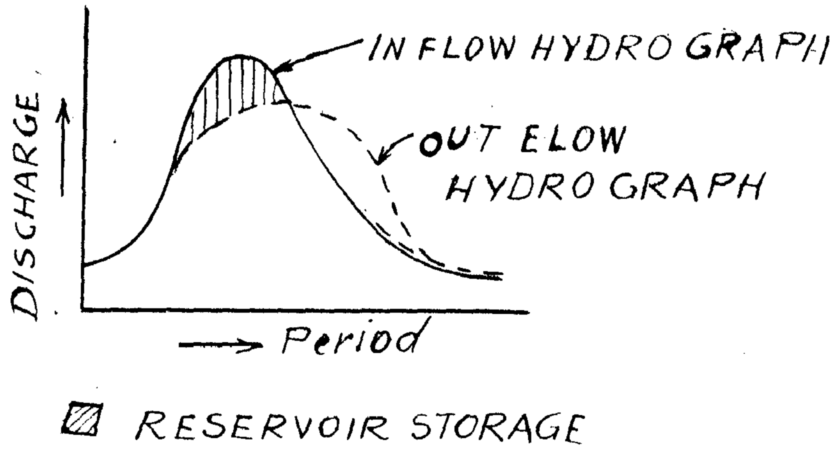

IN FLOW HYDRO GRAPH
OUT FLOW
HYDRO GRAPH
DISCHARGE
Period
RESERVOIR STORAGE

FIG. 5.1

All the flood producing storms are tabulated and about 10 percent of the highest storms of record are omitted. The next lower one is taken as the storm for the standard project flood. Methods of calculation for flood peak and volume have been discussed in the previous chapters.

5.03 Location of reservoirs :

The most effective flood control is obtained from an adequate reservoir located immediately upstream of the area to be protected. However this will in many cases mean that the reservoir be located in a broad plain, where the length of the dam required is excessive. It also entails submergence of vast areas of valuable land. A location farther upstream in a hilly region will generally reduce the both the length of dam as well as the value of land to be submerged. However such location reduces the effectiveness of the reservoir in reducing the flood peaks, because of influence of channel storage and the lack of control over the local inflow between the reservoir and the protected area. If the local area is sufficiently large, it may produce a flood of its own, on which the reservoir has little control. Economic factors generally favour the upstream locations despite its lesser effectiveness.

The effectiveness of the reservoir in reducing peak flow, increases, as its storage capacity increases. The maximum capacity required is the difference in volume between the safe release from reservoir and the maximum inflow. However the economic factors control the size of the reservoir. Thus the size of the reservoir has to be determined by weighing the cost involved in reducing the peak flows on one hand and the benefits estimated on the other hand.

5.04 Estimation of reservoir storage capacity :

For given a run-off data of a river (not less than 25 years) it is possible to determine the storage requirement for a given demand rate for the whole year. The best method is with the help of mass curves that is ploting of accumulated adjusted inflows.

Adjusted inflows are worked by adding rain fall in the reservoir area to the given inflows at u/s of reservoir site with given run-off co-efficient and deducting evaporation losses in the reservoir area.

Figure 5.2 shows the mass curve of calculated adjusted flows. Now to the same scale, accumulated given demand figures for the year are plotted to get the demand line for the annual yield. To find the reservoir capacity for this annual yield, a line parallel to the demand line is drawn tangential to the bulges (crests) on the mass curve. The largest ordinate between the demand line and the mass curve (in flow line) gives the volume which must be held in storage to tide over the dry period.

5.041 Illustrative example for the same is given below :

Given below are calculated the adjusted inflows for the river at reservoir site. What storage capacity would be required to get the constant annual yield of 300 hectare m

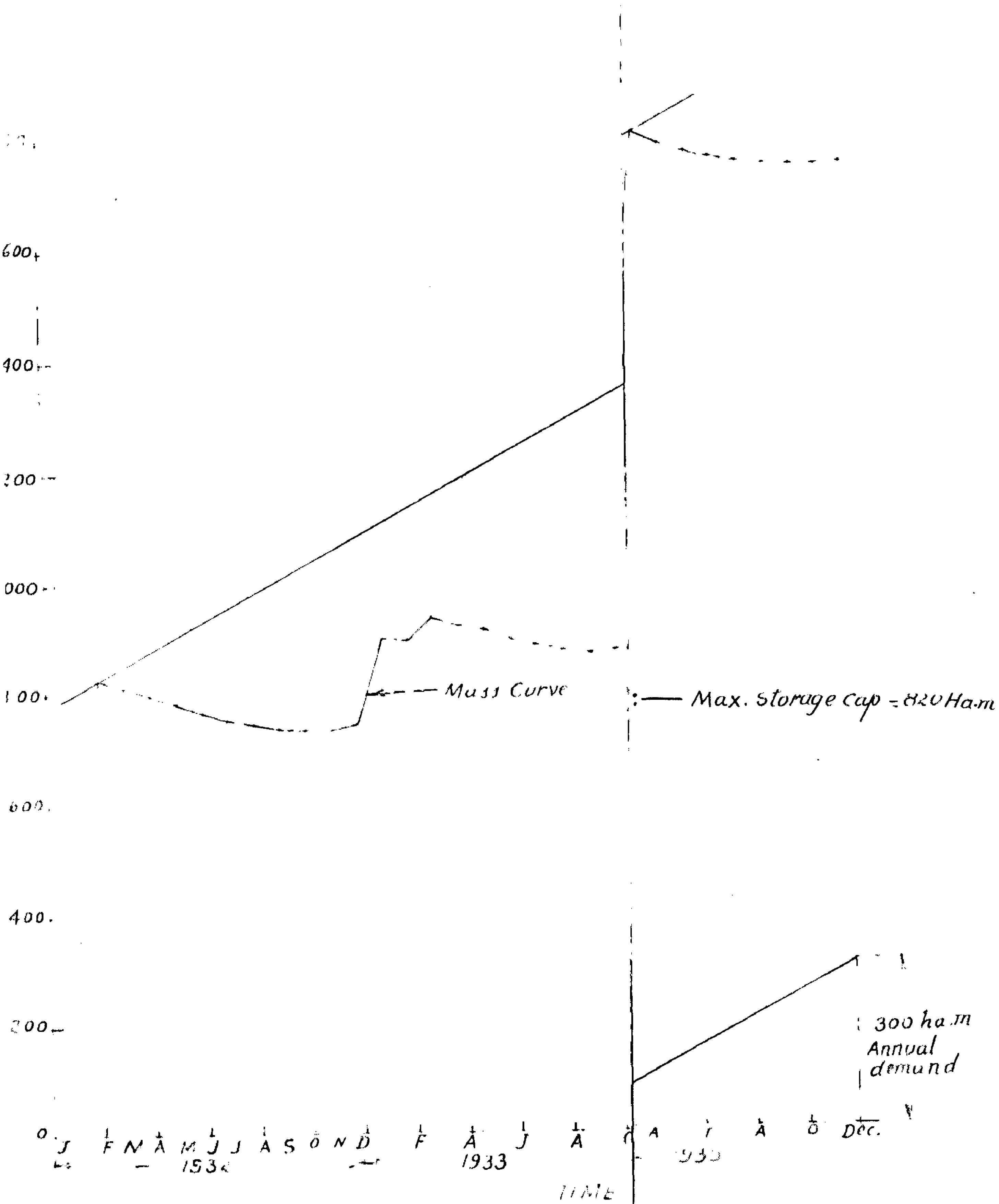

600
400
200
000
100
600
400
200
Mass Curve
Max. Storage cap = 820 Ha.m
300 ha.m
Annual
demand
J F M A M J J A S O N D
F A J A O A J A O Dec.
1933
TIME

FIG. 5

water surface area of reservoir to be 160 ha.m. run of co-efficient to be 0.25 and Pan co-efficient of 0.7.

Month	J	F	M	A	M	J	J	A	S	O	N	D
Normal pan evaporation in cm.	6.0	8.7	12.6	14.4	15.3	13.5	11.4	7.5	4.2	2.1	2.4	3.3

Refer Table 5.1.

Mass curve is drawn with cumulative inflows (Col.5) versus time figure 2. A demand line parallel to the annual demand of 300 ha.m. is drawn tangential to the crests of mass curve. Largest ordinate between these two lines works out to be 820 ha.m.

Result : Max. capacity of storage = 820 hectare meter

5.05 <u>Reservoir Sedimentation</u> :

For the design of the reservoirs, provision is to be made for dead storage and live storage. Dead storage is considered to provide space for the deposition of sediment for a number of years to come.

There are two methods for determining the rate of silting (space to be allotted for sediment in a reservoir).

i) By an assessment of the suspended and bed loads of the stream on which the reservoir is to be constructed.

ii) By analogy to the results of a capacity survey of existing reservoirs.

First method requires sediment load determinations for sufficient long period to ensure adoption of a rational load figure. The second method is based on the assumptions

Table. 5.1

Rainfall in hectare meter = $\frac{\text{rainfall in cm.}}{100}$ x reservoir area x co-eff.

(Col.3) 13.44 = $\frac{11.2}{100}$ x $\underline{160}$ x 0.75

Evaporation losses in ha.m. = Normal pan evap.losses x area x 0.7 = 6/100x160x0.7
= 6.72 ha.m.

(Col.4) Adjusted inflow = Cob.(2) + Col.(3) - Evap. losses
260.72 = 254 + 13.44 - 6.72

Table 5.1.

ha.m

Month	Inflow	Rainfall in ha.m.	Adjusted in flow	Cumulative
1	2	3	4	5
1932				
Jan.	254	13.44	260.72	260.72
Feb.	557	14.16	561.42	822.14
March	0	1.56	-12.55	809.59
April	0	2.16	-13.97	795.62
May	0	0.60	-16.54	779.08
June	0	0	-15.12	763.96
July	0	0	-12.77	751.19
Aug.	0	0	-8.40	742.79
Sept.	0	0	-4.70	738.09
Oct.	0	1.2	-1.15	737.54
Nov.	0	2.4	-0.29	737.[illegible]
Dec.	1	13.8	+11.10	748.35
1933				
Jan.	131	37.36	151.64	899.99
Feb.	3	3.6	-3.14	896.85
March	43	10.2	39.39	936.24
April	1	0.6	-14.53	921.71
May	1	5.76	-10.38	911.33
June	0	0	-15.12	896.21
July	0	0	-12.77	883.44
August	0	0	-8.40	875.04
Sept.	0	0	-4.70	870.34
Oct.	0	5.76	+3.41	873.75
Nov.	0	0	-2.69	871.06
Dec.	0	20.64	+16.94	888.00

Contd.

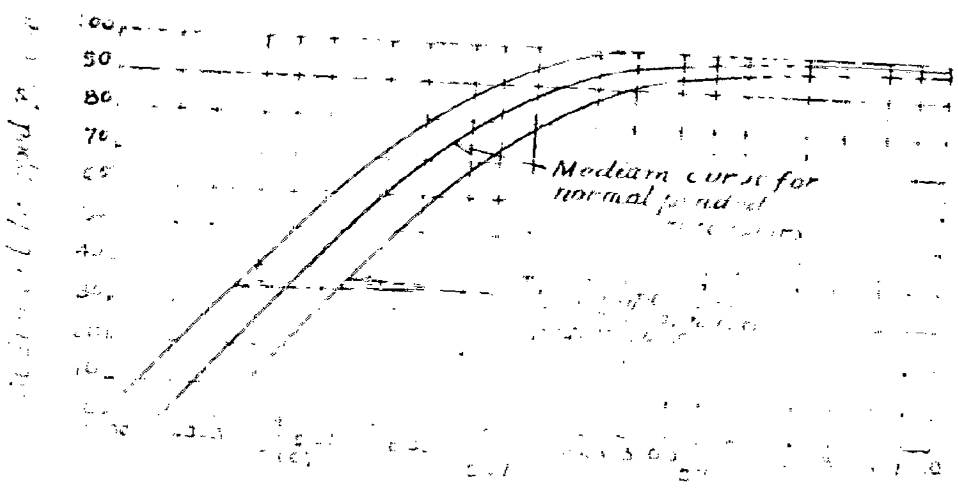
Table 5.1. Contd.

1	2	3	4	5
1934				
Jan.	8	3.96	5.24	893.24
Feb.	38	16.04	44.30	937.54
March	2	0	-12.11	925.43
April	1	2.04	-13.09	912.34
May	1	1.8	-14.34	898.00
June	0	1.8	-13.32	884.68
July	0	0	-12.77	871.91
Aug.	0	0	-8.40	863.51
Sept.	0	3.0	-1.70	861.81
Oct.	0	3.0	+ 0.65	862.46
Nov.	0	15.36	+12.67	875.13
Dec.	0	11.16	+ 7.46	882.59
1935				
Jan.	228	27.36	+248.64	1131.23
Feb.	2	3.96	-3.78	11,27.45
March	204	17.64	+207.53	1334.98
April	460	17.4	+461.27	1796.25
May	3	0	-14.14	1782.11
June	1	0	-14.12	1767.99
July	0	0	-12.77	1755.22
August	0	0	-8.40	1746.82
Sept.	0	0.6	-4.10	1742.72
Oct.	0	2.4	+0.05	1742.77
Nov.	0	1.56	-1.13	17,41.64
Dec.	1	10.04	+ 6.34	1747.98

that the two streams have similar sediment characteristics with regard to the production, transportation and ultimate settlement of sediment.

The surveys of the existing reservoirs indicate the specific weight of the settled sediments and the percentage of entering sediment which is deposited in the reservoir. Specific weights (dry) of sediments samples from reservoirs range from about 40 to 110 lbs/cuft. with an average of about 60 lbs/cuft. for fresh sediments and 80 lb/cuft. for old sediments. Trap efficiency of the reservoir is defined as the ratio of sediment retained in the reservoir to the sediment brought by the stream. This depends on the ratio of reservoir capacity to annual inflow. Curves correlating trap efficiency and capacity inflow ratio are shown in figure 5.3. drawn on the basis of data from surveys of existing reservoirs. It is seen from the figure that the small reservoirs on big rivers passes most of its inflow so quickly that the finer sediments do not settle but are discharged down stream and large reservoirs permit almost complete removal of sediments. Useful life of the reservoir is terminated when the capacity occupied by the sediment is sufficient to prevent the reservoir from serving its intended purpose.

Figure 5.3 may be used for estimating the quantity of sediment being traped in new reservoir.[6] If the average annual sediment load of the stream is known, the volume occupied by this sediment can be calculated by using value of specific weight for the deposited sediment. Usefull life may be calculated by finding the total time required to fill the critical storage volume.


100
50
80
70
Medeam curve for
normal

5.06 Flood routing :

The computation of reservoir storage volumes and outflow rates corresponding to a particular hydrograph of inflow is known as flood routing. Over any internal of time, the volume of inflow must equal the volume of outflow plus the change in the storage during the period. For practical solution step by step method is used for flood routing.

Step by Step method :

For any given flood, the data required for flood routing may be stated as below :

1) The inflow hydrograph
2) The storage elevation curve for reservoir,
3) The outflow elevation curve,

Required :- To construct the out flow hydrograph and to obtain maximum water to be obtained in the reservoir.

The first operation is to divide the inflow hydrograph into steps, so that steps may be taken as straight lines. The time interval should be so choosen as not to miss the peaks. The various steps involved in the process are given below :-

i) Fix the storage level in the reservoir above which the empty flood reserve is provided; this level is called pre-flood pool level.

ii) Work out spillway and outlets discharge rating curves.

iii) Obtain from the inflow hydrograph, the volume of water (to be expressed in cu.m.) entering the reservoir in a short time interval (say 6 to 12 hours) this volume will be

$\frac{i_1 + i_2}{2} \times t$ where i_1, represents the initial inflow and i_2 after lapse of 't' hours.

iv) From the outflow curves, work out outflow $\frac{O_1 + O_2}{2} \times t$ where O_1 is the initial outflow and O_2 after the lapse of time 't' hours. For obtaining O_2 assume a small increase in reservoir level.

v) Deduct the volume of outflow from the volume of inflow to get increment in storage capacity of reservoir.

vi) Add this increment in storage, to the previous pre-flood storage.

vii) Find out storage level corresponding to storage as in step (vi).

viii) Plot outflow hydrograph from the values of O_2 obtained in (iv).

ix) Re-work out steps (iv) to (vii), till the reservoir level assumed in (iv) corresponds nearly to the reservoir level obtained in (vii).

x) Continue step by step process till the outflow curve crosses the inflow curve; this point gives the peak outflow rate. From this time onwards the rate of outflow begins to fall due to decrease in inflow rate.

xi) The various steps should be continued from (iii) to (viii) untill the reservoir level returns to the pre-flow pool level.

5.07 Reservoirs for flood prevention and other uses :

The main obstacle to built a reservoir for flood prevention alone is cost. Therefore to lower the cost of flood prevention, it has always been suggested to use reservoirs for other purposes as well as for flood control.

The demands of power and flood control are more or less conflicting, but by mutual adjustment, a combined use of the same storage is sometimes feasible. This is not in the case of reservoirs having a combined function of flood control and irrigation, since, for the latter, water supply must be dependable and, for the former, there must be a dependable reservoir capacity. Reservation of seperate storage spaces for flood and irrigation within the same reservoir offers greater facilities as a certain over lapping of each space is practicable.

Economics in river valley developments are frequently possible by formulation of a balanced and comprehensive plan involving a combination of purposes. Therefore multipurpose projects permit more complete use of the physical potentialities of individual reservoir sites and fuller utilisation of available facilities than single purpose project. The basic factor in multipurpose operation is compromise. A working table must be devised which permits reasonable efficient operation for each purpose although maximum efficiency is not necessarily attained for any pn single purpose. The success which can be achieved in a joint use of storage space in a multipurpose project depends upon the extent to which the various purposes are compatible. It is helpful, therefore, to review the requirements of the various uses and to consider the ways in which these uses may be coordinated.

5.071 Example :

Fixation of storage capacity of the Hirakud Reservoir :

The Hirakud reservoir on the river Mahanadi has for its principle objectives flood control, irrigation and power generation.

Catchment area upto diameter	= 32,200 sq.miles
Annual silt deposite	= 12,000 ac.ft. per year
∴ Dead storage capacity (for 100 year life)	= 1.20 m. ac.ft.
Net withdrawal from storage for irrigation	= 1.00 m. ac.ft.
Flood storage required (worst flood)	= 3.51 m.ac.ft.
Economic live storage for power	= 3.62 m.ac.ft.
Reservoir losses	= 0.36 m.ac.ft.

Note : Water drawn from storage for the purposes of irrigation will not on the whole be available for power generation as bulk of it is drawn direct from the reservoir.

Capacity of the reservoir would have to be as follows :

	Flood control and power	Flood control and irrigation	Irrigation and power
Flood reserve	3.51	3.51	[illegible]
Power	3.62	-	3.62 m.acft.
Irrigation	-	1.00	1.00 m.acft.
Reservoir losses	0.36	0.36	0.36 m.acft.
Silt reserve	1.20	1.20	1.20 m.acft.
Total	8.69	6.07	6.18 m.acft.

Fortunately however, after the floods are over, there is enough water to fill the reservoir. From records available for the period 1872 to 1946, it was apparent that the river gauge at Haraj (where flood protection are given) never exceeded 89.0 after 24th September. Therefore the entire capacity reserved in the reservoir for flood absorption could be filled up after the 24th September. The river hydrographs further indicated that there would be sufficient water in the river in late September and October to fill the reservoir even in the worst year. Thus the live storage capacity of the reservoir including that reserve for flood control absorption would be available for use in irrigation and power generation. Thus reservoir capacity required for the three purposes of flood control, irrigation and power taken together would be 6.18 m.ac.ft. (with compromise of using flood space after 24th September).

5.08 Reservoir Operation Studies :

There are two major considerations relative to the planning and design stage. One is the selection of storage reservation and the other is the determination of the best plan for using the storage. These considerations are inter-related in as much as the plan of regulation is partly dependent upon the amount of storage and the storage is selected to realise certain broad objectives by following a plan of regulation.

Reservoir operation studies frequently become complicated when several reservoirs are operated in combination for multipurposes. A typical operation study of the Damodar

Valley system of Reservoirs for multipurposes is discussed in the following paragraphs.

5.081 General :

Damodar river rises in the plateau hills of Chota Nagpur at an elevation of about 2000 ft. above M.S.L. It flows in a generally south-easternly direction entering the detaic plains below Raniganj. Near Burdwan the river abrutply changes its course to a southerly direction and joins the Hooghly about 30 miles below Calcutta. The river is fed by six streams of which the principal tributary Barakar joins it where it emerges from the hills.

The area drained by the river at its mouth is about 8500 sq.miles of which 6960 sq. miles is the catchment of upper Damodar just below its confluence with Barakar.

The river used to flood the adjoining areas in the lower valley located in the state of West Bengal.Due to disastrous floods of 1943, the measures for the control of floods in the valley received top priority and an expert Mr. W.L. Voorduin of TVA was invited to prepare a project,primarily for the control of floods, and secondarily for development of water resources for various other uses, e.g. power, irrigation, navigation and water supply for industrial and municipal uses.

Voorduin's plan of development of the water resources of the valley was toconstruct seven storage dams, across Damodar and its tributaries and one diversion dam at Bermo, and a barrage at Durgapur with accompanying net-work of canals.

This development was to be carried out in two stages, the first stage to cover the construction of four dams at Tilaiya, Konar, Maithon and Panchet and a barrage and irrigation system at Durgapur, Fig. 5.4 and the second stage to complete the remaining four dams. The first stage was completed in 1958 and the second stage was not taken up as the Government of West Bengal felt that these dams were not necessary.

Rain fall :

The mean annual rainfall is 50 inches out of this 82% occurs during four monsoon months June to September.

Stream flow :

Maximum annual flow recorded is of the order of 12.m. acft. at Rhondia and average annual flow about 6.1 m.ac.ft.

1966-67 as low as 1.6 m.ac.ft.

Max. flood 1913 - 6,50,000 cusecs - volume 3.238 m.acft.

Max. flood 1959 - 8,10,000 cusecs - volume 1.15 m.acfft.

Bank full capacity at Durgapur - 2,50,000 cusecs

Total storage capacity for four dams is 2.9 m.ac.ft.

Flood control was maintained as primary objective of the scheme, providing a total flood control capacity of 1.51 m.acft. of the 2.48 m.acft. of live storage in these dams. With this flood reserve it is expected that all known floods with peak flow of 6,50,000 cusecs could be moderated to 2,50,000 cusecs, provided conditions in the lower valley were ignored. However till now, the land in the fore shore of the Maithon and Panchet reservoirs has not been acquired up to the designed levels with the result that against the designed flood absorption capacity

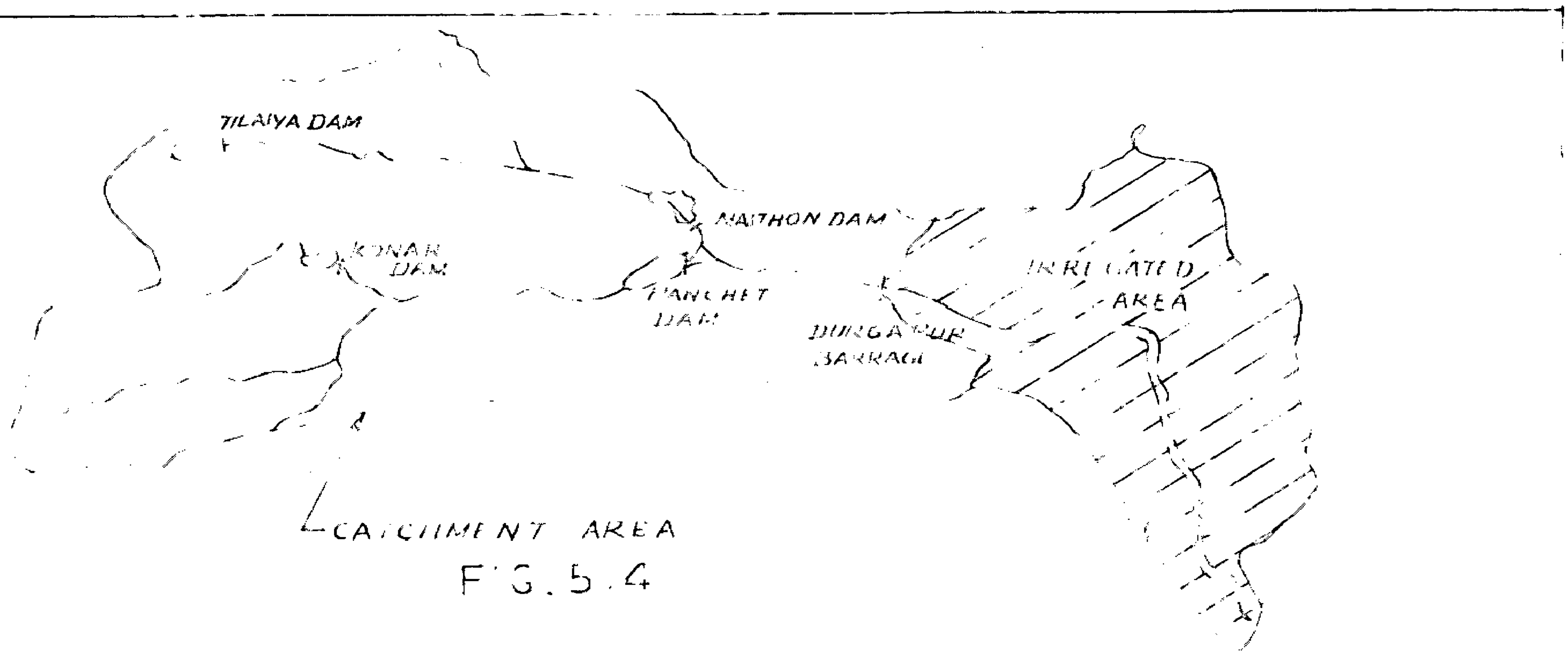

TILAIYA DAM
KONAR DAM
MAITHON DAM
PANCHET DAM
DURGAPUR BARRAGE
IRRIGATED AREA
CATCHMENT AREA

FIG. 5.4

of 1.51 m.acft. only 1.047 m.acft. is available. In view of this moderation of 1 million cusecs flood off the known flood with the peak flow of 6,50,000 cusecs to the bankfull capacity of 2,50,000 cusecs is not possible at present.

However the moderation of flood flows at Durgapur to 1,00,000 cusecs if not less is expected with only 4 dams and with the limited flood reserve available without complete land acquisition up to the design levels.

The general method of operating each reservoir for normal conditions is described below :

	Tilaiya	Konar	Maithon	Panchet
1.Dead storage level	EL 1192	EL 1347	EL 435	EL 392
2.Crest level	EL.1212	EL 1372.5	EL460	EL 405
3.Top of the gate	EL 1222	EL 1404	EL 500	EL 445

5.082. Tilaiya : reservoir is operated primarily for power and irrigation andonly incidently for flood control. Hence during the greater part of the monsoon, one power unit is runn at base load and the other to meet peak loads, thereby allowing the reservoir to fill by the end of the monsoon. Calculations show that by this operation there is seldom any spillage, since the capacity of the reservoir is large enough to accommodate all the inflow in a normal year less the water required to run one unit by 2,000 kw about 4000 cusecs. Thereafter, during the dry period (Oct. to June), the storage barely suffices to run one unit at the base load or both units for peaking purposes for part of each day. The water thus released is re-regulated at Maithon dam, firstly for power and then for irrigation.

5.083. Konar dam is at present operated primarly to meet the cooling water requirements of the steam plant at Bakaro, not exceeding 4002 cusecs, including flow from the uncontrolled Bokaro river. Almost all the water is returned to the river downstream and is available for repetative use by various municipalities, industries and Panchet power plant and thereafter diverted into the irrigation canals, off the barrage at Durgapur.

5.084. Maithon and Panchet hill reservoirs are to be operated for irrigation and power, with the important limitation that prescribed storage levels (EL 480 and EL 410 resp.) are not to exceed, except for the purpose of temporarily absorbing flood flows in excess of downstream channel capacity.

General instructions for the operator for the operation of Maithon and Panchet hill reservoirs are contained in Appendices I and II. P.48,50.

5.085 Outline of the proposed flow regulation for reservoirs- Tilaiya, Konar, Maithon and Panchet

Procedure :

1) Allocation of storage capacities is as given below :

	Tilaiya	Konar (in 1000 acre-ft.)	Maithon	Panchet
Dead storage	61	49	168	148
Capacity for monsoon storage	259	224	496	185
Reserve for flood control	-	-	440	881
Total to top of gates	320	273	1,104	1,214

APPENDIX-I

MAITHON RESERVOIR

Rules of operation for flood control :

With the arrival of a flood inflow in the reservoir, the operator is required first to allow the reservoir level to rise to EL 480, without opening the under sluices or spillway gates. (Two or three turbines are permitted to be operated).

If the reservoir level continues to rise rapidly and approaches EL 495 (land acquisition level) the spillway as well as the under sluices should be operated and water released to ensure the reservoir level not to rise above EL 500 (top of gates)

When the water level falls well below EL 495, the spillway gate should be gradually closed but under sluices should be kept fully open, till reservoir level returns to EL 480, the pre-flood pool level, when the sluices should also be closed.

Notes: The operations at Maithon and Panchet hill are to be co-ordinated so as to keep the combined release within the channel capacity of 250,000 cusecs in the lower reaches of the river.

Rules of operation for irrigation and Power :

Storage levels : EL 435 to EL 480

Period	Reservoir elevation	Irrigation and Power Requirements
July	EL 438 to EL 456	Total irrigation requirement 7,891 cusecs (Maithon & Panchet Hill) Power two or three units as per W.A.*
Aug.	El 456 to EL 480	Total irrigation requirement 6,838 cusecs Power 2 to 3 units as per W.A.
Sept.	El 480	Total irrigation requirement 7,891 cusecs Power 2 & 3 units as per W.A.
Oct. (first half)	El.480	Total irrigation requirement 8,206 cusecs Power 1 or 2 units as per W.A.
Oct. (second half)	El.480 to EL 475	Total irrigation requirement 8,206 cusecs Power 1 or 2 units as per W.A.
Nov. to March	EL 475 to EL 453	Total irrigation requirement 1,500 cusecs Power 1 unit
April to Mid June	El 453 to El 435	Total Irrigation - nil Other uses - 500 cusecs Power 1 unit
June (second half)	El 435 to EL 438	Total irrigation requirement 5,129 cusecs Power 1 or 2 units as per W.A.

Note : 1. The release from Maithon reservoir is to be co-ordinated with that from Panchet Hill to meet full irrigation requirements.

2. Unit capacity 20,000 kw each.

* W.A. stands for water availability

APPENDIX - II

PANCHET HILL RESERVOIR

Rules for operation of flood control :

With the arrival of a flood inflow in the reservoir, the operator is required first to allow the reservoir level to rise to EL 410, without opening the under sluices or spillway gates. (one turbine may be operated at this time).

When the reservoir level begins to rise above EL 410, the operator is required to hold down the water level to EL 410 by opening all the under sluices and spillway gates, limiting the release to safe river capacity (when water level exceeds EL 410, both turbines are permitted to be operated).

If the reservoir level continues to rise rapidly and approaches EL 425 (land acquisition level) the spillway as well as under sluices should be operated and just enough water released to ensure the reservoir level not to rise above EL 445 (top of gates).

When the water level begins to fall below EL 425, the spillway gates should be gradually closed but under sluices should be kept open till reservoir level returns to EL 410, the pre-flood, pool level, when the sluices also should be closed.

Note : The operations at Panchet Hill and Maithon are to be co-ordinated so as to keep the combined release within the channel capacity of 2,50,000 cusecs in the lower reaches of the river.

Rules of operation for Irrigation and Power :

(Storage levels EL 392 to EL 410)

Period	Reservoir elevation	Irrigation requirements and power production
July	El.397 to El.408	Total irrigation requirement - 7,891 cusecs (Panchet Hill & Maithon) Power 1 unit
Aug.	El.408 to El.410	Total irrigation requirement - 6838 cusecs Power 1 or 2 units as per W.A.*
Sept.	EL. 410	Total irrigation requirement - 7,891 cusecs Power 1 or 2 units as per W.A.
October first half	El.410	Total irrigation requirement - 8206 cusecs Power 1 or 2 units as per W.A.
October 2nd half	EL 410 to EL 407	Total irrigation requirement - 8,206 cusecs Power - 1 unit
Nov. to March	EL 407 to EL 401	Total irrigation requirement - 1500 cusecs other uses - 500 cusecs Power - 1 unit
April to Mid June	El 401 to El 392	Total irrigation requirement - Nil Other uses 500 cusecs Power - 1 unit
June (2nd half)	EL.392 to El.397	Total irrigation requirement - 5,129 cusecs Power - 1 unit

Note :- 1. The release from Panchet Hill reservoir is to be co-ordinated with that from Maithon to meet full irrigation requirements.

2. Unit capacity 40,000 KW each

* W.A. stands for water availability

Meeting of the above requirement, the guide for storage and the guide for drawdown are fixed by a few trials to ensure optimum usage of water (Figs.5.5 and 5.6).

ii) For the purpose of estimating the contribution of inflow into Maithon due to the release from Tilaiya and for inflow into Panchet due to release from Konar, allowance has been made for enroute losses as shown below :

@ 10% of release from the middle of June to the middle of December.

@ 20% from the middle of December to end of Feb.

@ 25% for March and April

@ 30% from the beginning of May to the middle of June.

iii) Withdrawal direct from the Tilaiya reservoir for the High Level Irrigation Scheme to irrigate 10,000 acres during the Kharif season and 5,500 acres during the Rabi season is allowed as follows :

June 21-30	- 100 cusecs	equivalent to	2,000 ac.ft.
July	- 100	-do-	6,000 -do-
August	- 50	-do-	3,000 -do-
Sept. 1-20	- 50	-do-	2,000 -do-
21-30	- 100	-do-	2,000 -do-
October	- 100	-do-	6,000 -do-
November	- 33	-do-	2,000 -do-
December	- 33	-do-	2,000 -do-
January	- 33	-do-	2,000 -do-
February	- 33	-do-	2,000 -do-
March	- 33	-do-	2,000 -do-
		Total	31,000 Ac.ft.

iv) Allowance has been made for withdrawal of water for industrial and domestic uses at the rate of 20 cusecs in the reach between Tilaiya and Maithon and at the rate of 450 cusecs in the reach between Konar and Panchet. It will be seen that in some periods the release from Konar and the inflow from intermediate catchment together would not be as high as 450 cusecs, in which case it is assumed that the inflow less than 450 cusecs is all consumed for industrial and domestic water supply in the reach.(8)

v) Evaporation loss from the water surface in the reservoirs is allowed on the basis of average monthly evaporation recorded by Piche evaporimeter installed in the observatories near the dam sites.

vi) Primary supply through the power plant equal to 150 cusecs in Tilaiya, 250 cusecs in Konar and 1,100 cusecs in Maithon is released throughout the year except when such release requires drawing the reservoirs level below the deadstorage when it is assumed that the power house will be shut down completely and the primary supply to the power house cut-off. In case of Maithon in addition to the primary supply, an extra release of 900 cusecs during June 21-30, 2900 cusecs during July 1-10 and 3,400 cusecs during July 11 to Aug.10 is assumed in order to help meeting the Kharif irrigation requirement.

vii) After releasing the primary and assumed supplies as above, if the reservoir level is above the guide curve for storage or the guide curve for draw-down, then secondary supply limited to the capacity of the power plant is permittod. Tho capacity of the power plant is 800 cusecs at Tilaiya, at Konar it is assumed as equal to 500 cumecs and at Maithon it is 1,200 cusecs.

viii) After filling the reservoir upto the permissible limit, the excess water, if any, during the monsoon months will first be released through the power plant and anything in excess of power plant capacity will be released through the sluices or over the spillway as a part of the flood control operation.

ix) The expected indent of the water at canal head works is estimated on the basis of discussings hold with Chief Engineer. According to this, the total requirement for the season out of which the rain fall deficiency plus a certain percent (10% of canal take-off during second half of June, 5% during second half of September and 7% during October) addod to cover the Transit loss would be the indent for supply at the canal headworks. This has been separately worked out and the 10-day summary is taken for the purpose of flow regulation. The extent of this requirement met by releases from Maithon following the normal operation procedure is deducted and the balance is taken as the indent for water to be supplied from Panchet reservoir as shown in table 5.2. The outflow from Panchet will be so regulated as to meet these indents received during each week or shorter period during the Kharif season. As a result of such operation, the dry period release from Panchet is governed by the quantity of water left in the reservoir at the end of the Kharif season.

5.086 Working table :

Following the above procedure, the flow regulation is worked out for the period July 1949 to February 1958. The types of years included one fairly representative . Details of the computations are shown in the working tables 5.3 to 5.6. Reservoir levels are also represented graphically in figures 5.5. and 5.6.

5.087 Summary :

i) Tilaiya reservoir does not fill up to the top of gates in 7 out of 8 years. It fills up to the spillway crest level (EL.1212) in two out of 8 years).

ii) Konar reservoir does not fill up to the top of gates in 6 out of 8 years. It fills up to EL. 1387 in 6 out of 8 years.

iii) Maithon reservoir fills up to EL 480 every year and Panchet fills up to EL 410 every year.

iv) A release of 250 cusecs can be assumed from Konar through out the year even in dry year.

v) A minimum release of 1,100 cusecs can be assured from Maithon through out the year even in dry year.

vi) In case of the Tilaiya the Power House will have to be closed down in some years when the reservoir level goes below the dead storage level and in case of Panchet reservoir. the power house will have to be shut down during the dry periods in 7 out of 8 years because of the extra draw-down below the dead storage in meeting the irrigation requirement mainly during October.

5.9. Limitations :

Effective flood moderation can be achieved only if the storage is adequate to deal with all floods except the abnormal ones. The cost of providing such storage for flood control only is generally very high. Therefore, the flood protection by storage reservoirs can be considered in special cases such as the protection of important centres immediately downstream. For realising the planned benefits from reservoirs, operation has to be skillfully done regulating the releases taking into account the intermediate flows between the dam and the damage centres. Decrease in the normal discharge in the river due to the flood moderation effected by the reservoir, results in deterioration of the channel downstream. The sense of security provided by the reservoir is likely to lead to the development of [illegible] areas affected by floods before the construction of reservoir. Silting will gradually reduce the effective capacity for controlling the floods. All the factors enumerated above may, in due course, lead to a larger loss in the protected area requiring more expensive and elaborate methods to remedy the situation.

Review of the progress made :

Prior to 1951, storage schemes had been planned mainly with the object of providing irrigation water. There was practically no storage dam for flood moderation. Since 1951, a large number of storage dams has been taken up for harnessing the river waters for irrigation and hydroelectric development and also for moderation of floods.

Specific storage for flood moderation has been provided in the Hirakud Dam Project on Mahanadi and dams on the river Damodar. In the case of Hirakud Dam, entire storage of 4.72 million acre feet is utilised for flood moderation during the South West monsoon and later on filled to be utilised for irrigation and power. In the Damodar Valley dams, a storage of 1.3 thousand million cu.m. (1.05 million acre feet) out of a total of 3.3 thousand million cu.m. (2.64 million acre feet) has been reserved for flood moderation. The balance is utilised for irrigation and power during flood season. In the non-flood season, the supplies, if available, are stored in the storage earmarked for flood control. These reservoirs have been in operation for more than a decade now and have been proved very useful in the contributing the overall prosperity of the area. The other large irrigation and multipurpose reservoirs such as Bhakra-Nangal, Rihand, Nagarjunasagar and Tungabhadra, even though they do not have got any specific flood storage, provide incidental flood control benefits in as much as they moderate the flood peaks in most of the years during the filling period.

Even though the flood problem is serious, requiring measures for moderation of peak flood discharges, it has not been possible to construct major storage dams in Brahmaputra basin and on the Northern tributaries of Ganga in the North-central and North-eastern regions. This has been mainly due to the lack of suitable sites within the limits of the country, high seismicity in the region and the high sediment content of the rivers which will reduce the life and the utility of the reservoirs.

5.2. Diversion of excess discharge :

When the flood waters can not be led safely through particular react of river even by embankments or can not be moderated by reservoirs, part of this can be diverted to another channel. Diversions and flood ways take away a part of flood flows to another basins or a depression where it could be stored for use later on, and, consequently reduce the discharge to harmless extent at most of the times.

The excess water of a river, during floods can be diverted in one or more of following ways :-

i) From one river to another river.
ii) Directly into sea
iii) On to arid zone
iv) Into natural lakes or depressions
v) By providing breaching sections.

i) From one river to another river :

When the receiving river has the capacity to take diverted flood discharge without creating any serious problem along its own, diversion of excess water from one river to another river is possible. Examples of this in India are, diversion of flood discharge of Mahanadi into Brahmin, and Budameru river into Krishna river.

ii) Directly into sea :

Flood waters are diverted into sea for reducing flood damage in the lower reaches of the river. It may be mentioned here that spilting up of rivers into number of channels at its delta is a natural process of flood diversion. A number of cut-offs to the rivers near sea are contemplated in Orissa.

iii) On to the arid zone :

The main arid zone in India is in the state of Rajasthan. On this method a scheme for diversion of flood flows of Ghaggar river into the sand dunes near Suratgarh has been completed.

iv) Into natural lakes or depressions :

Damage due to floods can be reduced by diverting the flood flows into natural lakes or depressions, the inundation of which will not result in great loss. This method has been employed on the scheme of supplementry channel taking off from river Jhelum, upstream of Srinagar and outfalling into Wullar lake.

v) By providing breaching sections :

In some cases breaching sections or spillways are being provided in the embankments to allow escape of excess flood water into areas where the damage would be least.

5.21 Diversion for moderating the flood intensities have been adopted to a limited extent so far. One of the notable works which has been executed is the supplementry channel with a capacity of 283 cumecs (10,000 cusecs) taking off from the Jhelum 5 km. (3 miles) upstream of Srinagar and outfalling into Wullar lake. This has been provided to reduce the flood intensities and heights of the Jhelum river at Srinagar which is protected by embankments. Diversion for carrying 210 cumecs (7,500 cusecs) of the Budameru, a small river in Andhra Pradesh, into the Krishna has been done for reducing the discharge of the river, while passing through the Vijayawada town. The diversion takes nearly 25% of the normal peak floods. Another important work which has been completed recently, is the diversion channel from Ghaggar river, which carries away 330 cumecs (12000 cusecs) into the depressions in the sand dunes which have a capacity of about 863 million cu.m. (0.7 million ac.ft.). The accumulated water is lost by percolation into the underground water table and by evaporation. The diversion has been provided to prevent damage to the important towns of Hanumangarh and Suratgarh, and lines of communications which have been aligned along the low area which, prior 1958 hardly used to get any damage causing floods in Ghaggar but, in recent years, with development work in the upper catchment, sustained peaks leading to damage and disruption of communication have become frequent.

5.22. Limitations :

i) Diversion of excess water from one river into another will be feasible if the receiving source has the capacity to take such diverted flood discharge without creating serious problems along its own.

ii) There may be opposition from the land-owners whose land the diversions have to pass.

5.3. Detention Basins :

As distinct from reservoirs created by building dams, detention basins can be formed as the creations of nature, improved and regulated by man to serve the needs of flood moderation. As a rule, rivers flowing through alluvium built up their banks higher than the adjoining lands. Swamps and lakes got created where drainage of adjoining land is obstructed by this process of bank raising by river spills. During high floods, when the river is not embanked, water spills over the river banks and flows into these depressions, later to flow back into the river. This process brings about flood moderation in the river downstream.

The detention basins, to be effective, needs large areas, specially on a large rivers. For example a river of a size of Ganga or the Brahmaputra, would require detention basins of aggregate area of many thousand square miles for bringing about an appreciable flood moderation. Such large swamps do not exist in these river basins. However this method can find application on a number of small rivers in the country. For instance, there are a number of swamps or " beels" along some tributaries of the Brahmaputra, and in the Barak valley in Assam as well as along some of the rivers in North Bihar, which can be put to such use.

The capacity of natural detention basins can be considerably increased by embanking these around. Their utility can be increased by providing regulating devices at the intake and the exit.

Flood moderation by means of detention basins, when feasible, is generally the cheapest form of flood control. The works are relatively inexpensive as the compensation of land is generally small. In India, no sizeable detention basins have been created. Few are contemplated on various rivers in the states of Assam, Bihar and Uttar Pradesh.

Storage tanks in M.P., Rajasthan, U.P. and a good number in South India, built for purposes of irrigation have been contributing towards moderation particularly during early part of rainy season.

5.32. Limitations :

i) Moderation of floods by detention basins wherever feasible, is generally cheap. However to be effective the area of detention basins has to be relatively large, but natural swamps and lakes having a large area are very rare.

ii) The pattern of floods in the river should be such that there is adequate interval between two successive floods to facilitate the clerance of water from the earlier flood before the arrival of the next floods.

THE PLAN OF MEETING KHARIF

Month	Date	CANAL OFF - TAKE						
		1949	1950	1951	1952	1953	1954	1955
July	1 - 10	6.2	7.4	4.1	2.0	1.4	11.5	1.0
	11 - 20	4.7	3.0	6.0	3.4	7.2	5.1	4.7
	21 - 31	0.3	4.6	11.1	4.7	1.3	8.0	0.6
August	1 - 10	2.7	0.0	8.6	9.7	2.8	11.5	5.5
	11 - 20	5.1	1.9	4.1	5.3	3.7	3.2	1.0
	21 - 31	5.2	6.1	3.5	7.6	5.0	11.5	8.3
Sept.	1 - 10	5.2	11.5	0.4	4.0	6.1	2.0	9.8
	11 - 20	1.9	11.9	9.3	6.4	1.4	5.8	7.7
	21 - 30	5.6	9.6	10.5	3.5	0.1	4.3	3.3
Oct.	1 - 10	5.5	5.4	2.0	1.1	5.7	8.4	3.5
	11 - 20	4.4	4.7	3.0	4.9	8.4	8.4	4.7
	21 - 31	3.3	1.9	7.1	8.2	9.1	3.6	6.9
TOTAL		50.1	58.0	69.7	60.8	52.2	83.3	57.0
Delta in Inches		11.5	13.3	16.0	14.0	12.0	19.2	13.1

TABLE - 5.

TILAIYA RESE

WORKING TABLE SHOWING FLO

Month	Date	Guide for Storage	Guide for draw-down	Normal evapo-ration loss	With-drawl direct from reser-voir for Irr-igation	Primary Supply	1949 - 50		
							1	2	3
								65	
July	1-10	80			2	3	12	72	
	11-20	100		2	2	3	7	72	
	21-31	120			2	3	17	84	
Aug.	1-10	130			1	3	38	118	
	11-20	140		2	1	3	30	140	2
	21-31	150			1	3	10	146	
Sept.	1-10	160			1	3	11	153	
	11-20	170		2	1	3	28	170	5
	21-30	180			2	3	11	176	
Oct.	1-10		160	1	2	3	5	170	5
	11-20		155	1	2	3	4	163	5
	21-31		150	2	2	3	2	153	5
Nov.			140	4	2	9	5	140	3
Dec.			130	3	2	9	2	128	2
Jan.			120	3	2	9	2	116	
Feb.			110	4	2	9	1	102	
March			100	6	2	9	2	87	
April			90	8		9	0	70	
May			80	7		9	0	54	
June	1-10		75	1		3	2	55	
	11-20		70	1		3	4	58	
	21-30		65	1	2	3	22	69	5
TOTAL				48	31	108	215		30

1953 - 54				1954 - 55				
1	2	3	4	1	2	3	4	1
	42				133			
55	87	5		3	126	5		9
34	109	5		10	124	5		19
37	136	5		16	130	5		8
21	148	5		9	130	5		14
40	177	5		45	164	5		41
99	267	5		12	167	5		19
16	274	5		19	177	6		7
39	302	5		21	187	5		6
37	320	13	1	4	181	5		3
6	315	5		0	170	5		5
1	305	5		2	161	5		2
14	307	5		1	150	5		2
7	284	15		17	140	12		7
10	265	15		9	130	5		7
10	246	15		9	120	5		10
4	220	15		8	110	3		7
5	192	15		17	100	10		1
9	170	15		11	90	4		0
7	146	15		10	80	4		7
4	141	5		4	75	5		2
5	137	5		3	70	4		9
7	133	5		3	65	2		1
46 7		188	1	233		144		18

NOTES:

Col.(1) Inflow into the reservoir

Col.(2) Reservoir capacity at the end of the period

Col.(3) Secondary Supply

Col.(4) Release through sluices o r over the spillway

TABLE -

KONAR RESE

WORKING TABLE SHOWING F

Month	Date	Guide for Storage	Guide for Draw-down	Normal evaporation loss	Primary supply	1949 - 50 (1)	(2)	(3)	(4)
							55		
July	1-10	80		1	5	4	54		
	11-20	100			5	20	68		
	21-31	120			5	45	108		
Aug.	1-10	140			5	52	150	5	
	11-20	160		1	5	32	171	5	
	21-31	170			5	22	183	5	
Sept.	1-10	180			5	24	197	5	
	11-20	190		1	5	46	232	5	
	21-30	200			5	13	235	5	
Oct.	1-10		160		5	12	237	5	
	11-20		155	1	5	11	237	5	
	21-31		150		5	4	231	5	
Nov.			140	2	15	11	210	15	
Dec.			130	2	15	4	184	15	
Jan.			120	2	15	2	152	15	
Feb.			110	2	15	2	122	15	
March			100	3	15	2	100	6	
April			85	3	15	0	82		
May			70	3	15	4	68		
June	1-10		65		5	1	64		
	11-20		60	1	5	7	60	5	
	21-30		55		5	12	62	5	
	TOTAL			22	180	330		121	

1953 - 54				1954 - 55			
1	2	3	4	1	2	3	4
	48				55		
64	102	5		4	54		
18	109	5		8	56		
31	130	5		20	71		
31	151	5		8	74		
35	175	5		50	118		
90	255	5		9	122		
27	272	5		30	147		
30	273	5	18	8	149		
35	273	5	25	7	151		
9	272	5		5	151		
4	265	5		5	150		
1	256	5		2	147		
4	228	15		4	134		
1	197	15		10	127		
1	166	15		10	120		
2	136	15		8	110	1	
0	103	15		8	100		
0	85			8	85	5	
1	68	[illegible]		43	95	15	
3	65	1		1	86	5	
5	60	4		1	76	5	
3	55	3		2	68	5	
TOTAL							
395		143	43	251		36	

NOTES:

Col.(1) Inflow into the Reservoir
Col.(2) Reservoir capacity at the end of the period
Col.(3) Secondary Supply to the power plant
Col.(4) Release through the sluices or over the spill

TABL

MAITHON

WORKING TABLE SHOWING FLOW

Month	Date	Guide for Storage	Guide for Drawdown	Normal evaporation loss	Primary Supply	Additional assured supply forming part of secondary supply
July	1-10	320		1	22	58
	11-20	370		1	22	68
	21-31	420		1	22	68
Aug.	1-10	470		1	22	68
	11-20	520		1	22	
	21-31	570		1	22	
Sept.	1-10	620		1	22	
	11-20	664		1	22	
	21-30	664		1	22	
Oct.	1-10	664		1	22	
	11-20	664		2	22	
	21-31	664		2	22	
Nov.			620	7	66	
Dec.			570	7	66	
Jan.			500	6	66	
Feb.			430	8	66	
March			360	11	66	
April			280	11	66	
May			200	8	66	
June	1-10		180	3	22	
	11-20	220		1	22	
	21-30	270		1	22	18
	TOTAL			77	792	280

1953 - 54				1954 - 55				1955 -	
1	2	3	4	1	2	3	4	1	2
	196				201				176
287	338	64		47	167			218	313
184	377	54		104	180			170	370
160	420	26		130	219			214	439
173	470	32		47	175			165	470
161	520	88		212	364			353	664
312	664	122	23	86	427			82	601
118	637	122		155	559			73	620
319	664	122	147	116	652			72	664
140	664	117		60	664	25		33	664
51	664	28		25	664	2		46	664
48	658			26	664	2		46	664
12	646			19	659			15	655
62	620	15		32	618	[illegible]		27	609
35	570	12		29	570	4		17	553
48	500	46		38	500	36		16	497
26	430	22		22	430	18		23	430
35	360	28		14	360	7		28	360
22	280	25		12	280	15		15	280
25	200	31		11	200	17		92	200
18	180	13		15	180	10		28	180
53	210			21	178			120	220
32	201			39	176			92	270
2291		967	170	1260		136		1945	

NOTES:

Col.(1) Inflow into the reservoir
Col.(2) Reservoir Capacity at the end of the period
Col.(3) Secondary supply
Col.(4) Release through sluices & over the spillway.

TABLE - 5.6

PANCHET HILL RES...

WORKING TABLE SHOWING FLOW REGUL...

Month	Date	Guide for storage	Guide for draw-down	Normal evaporation Loss	1949 - 50					
					1	2	3	4	5	1
						250				
July	1-10	300		2	170	300	118			237
	11-20	333		1	181	333	134	13		624
	21-31	333		1	357	333	134	222		287
Aug.	1-10	333		1	480	333	134	345		325
	11-20	333		1	574	333	134	439		482
	21-31	333		1	279	333	134	144		162
Sept.	1-10	333		1	256	333	134	121		189
	11-20	333		1	390	333	134	255		209
	21-30	333		1	120	333	119			81
Oct.	1-10	333		1	114	333	113			47
	11-20	333		2	82	333	80			42
	21-31	333		2	33	320	44			23
Nov.			320	6	82	351	45			41
Dec.			300	6	40	340	45			28
Jan.			280	6	22	311	45			23
Feb.			250	7	12	271	45			8
March			220	13	8	221	45			1
April			190	11	0	180	30			0
May			160	10	0	148	22			0
June	1-10	150		2	0	146	-			17
	11-20	200		2	98	227	15			36
	21-30	250		1	248	333	134	7		39
TOTAL				79	3546		1838	1546		2901

1953 - 54					1954 - 55					
1	2	3	4	5	1	2	3	4	5	1
	164					156				
691	333	134	386		28	32	-		150	114
270	333	134	135		82	101	-		12	136
290	333	134	155		130	160	14		56	175
344	333	134	209		59	78	20		120	106
421	333	134	286		266	301	29		13	85
881	333	134	746		116	208	134		74	44
174	333	134	39		494	333	134	234		40
309	333	134	174		352	333	134	217		45
341	333	134	206		133	333	132			23
75	333	74			35	223	134		10	39
15	200	134		12	23	100	82		62	27
10	48	46		114	27	75	-		50	10
49	91	-			21	90				0
20	105				0	84				0
8	107				12	90				0
8	105				0	83				0
0	95				0	70				0
0	84				0	59				0
0	74				0	49				14
17	89				7	54				80
59	146				14	66				55
21	156	10			22	87				41
4003		1470	2336	126	1821		813	451	547	103

NOTES

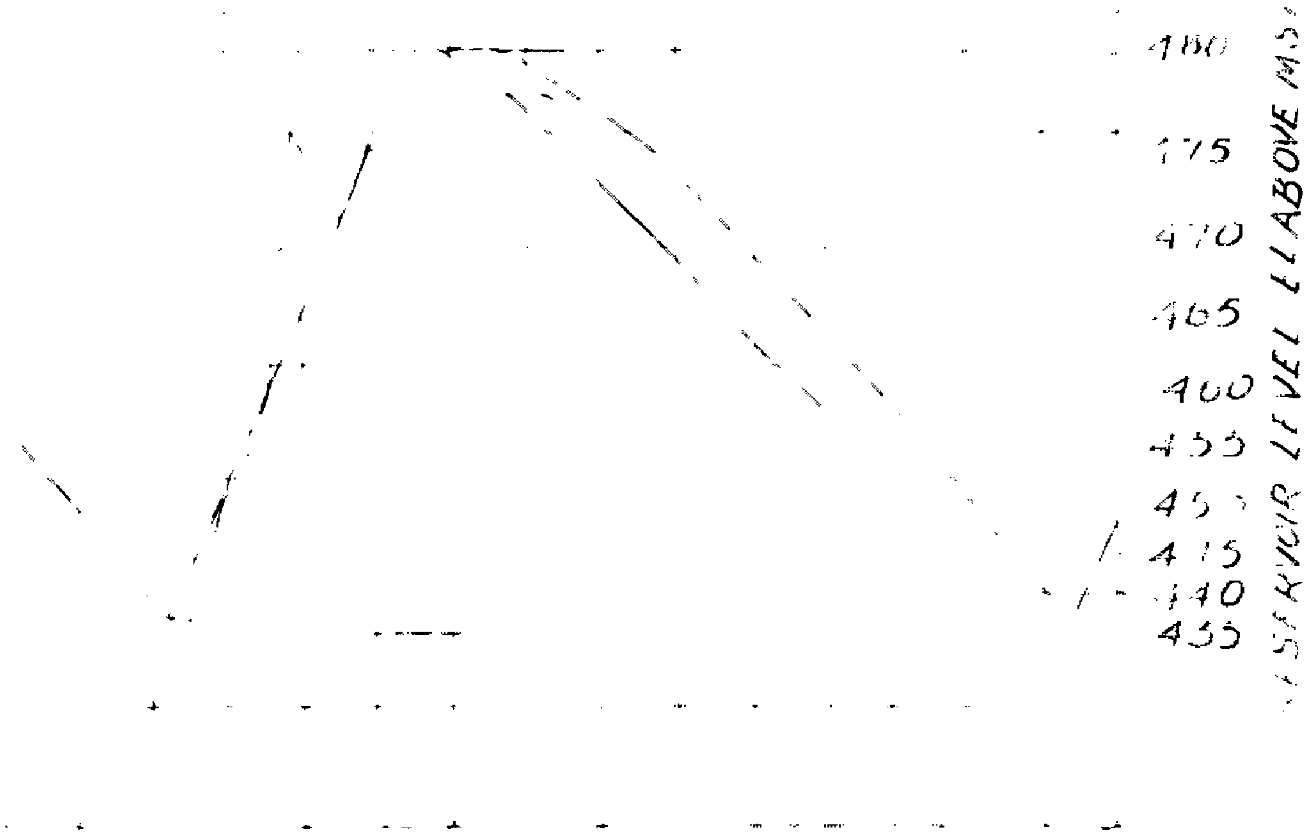
Col.1 Inflow into the reservoir

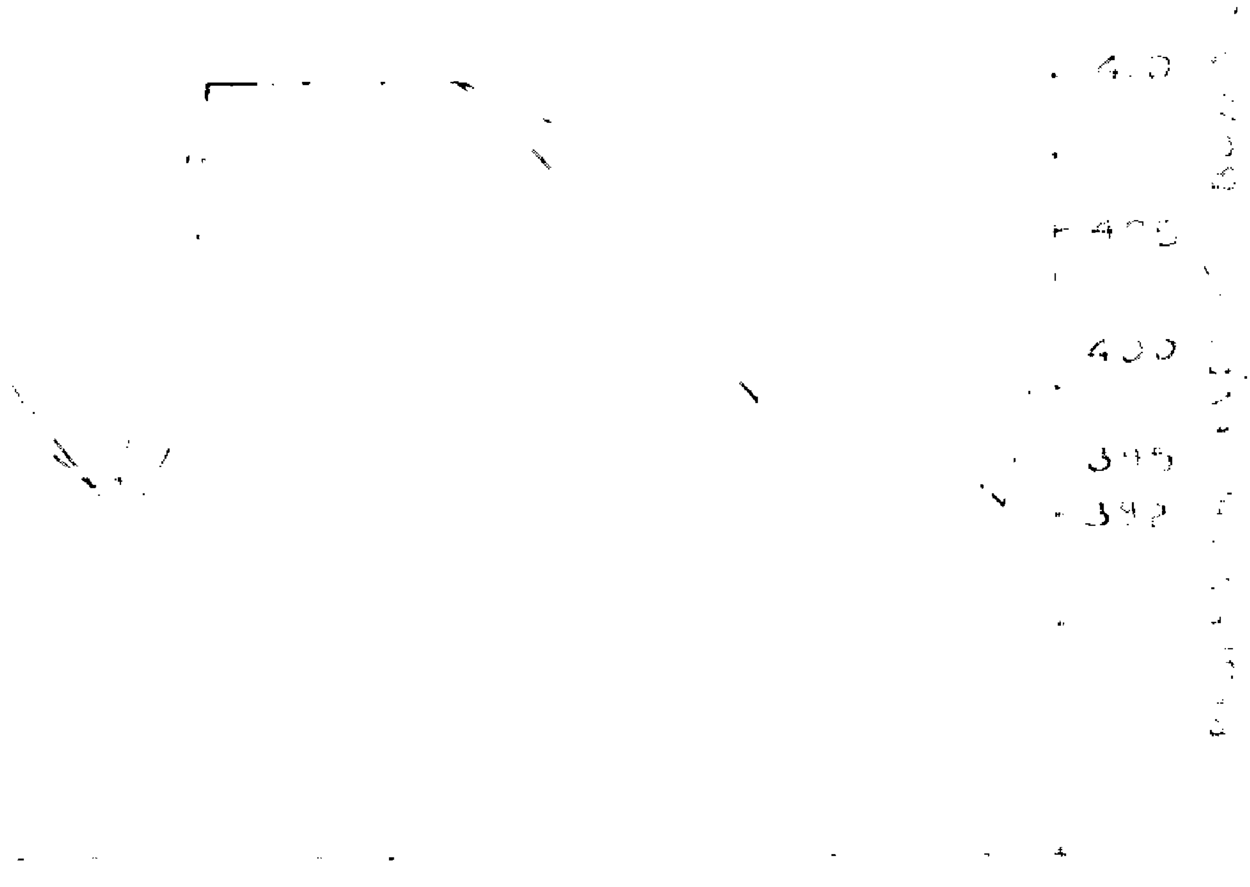
Col.2 Storage capacity at the end of the period

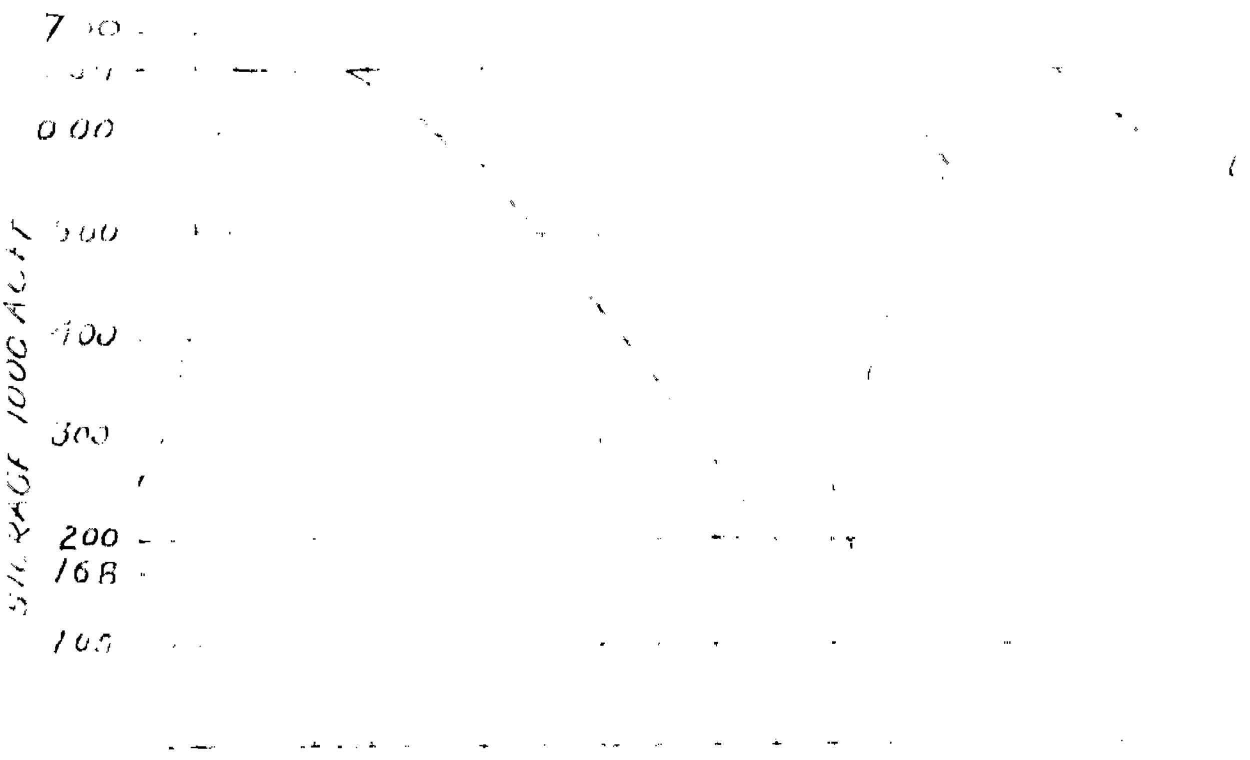
Col.3 Release through power house

Col.4 Release through sluices or over the spillway

Col.5 Part of the i rrigation release which can not be passed through power House.


465
445
440
RESERVOIR LEVEL ABOVE M.S.

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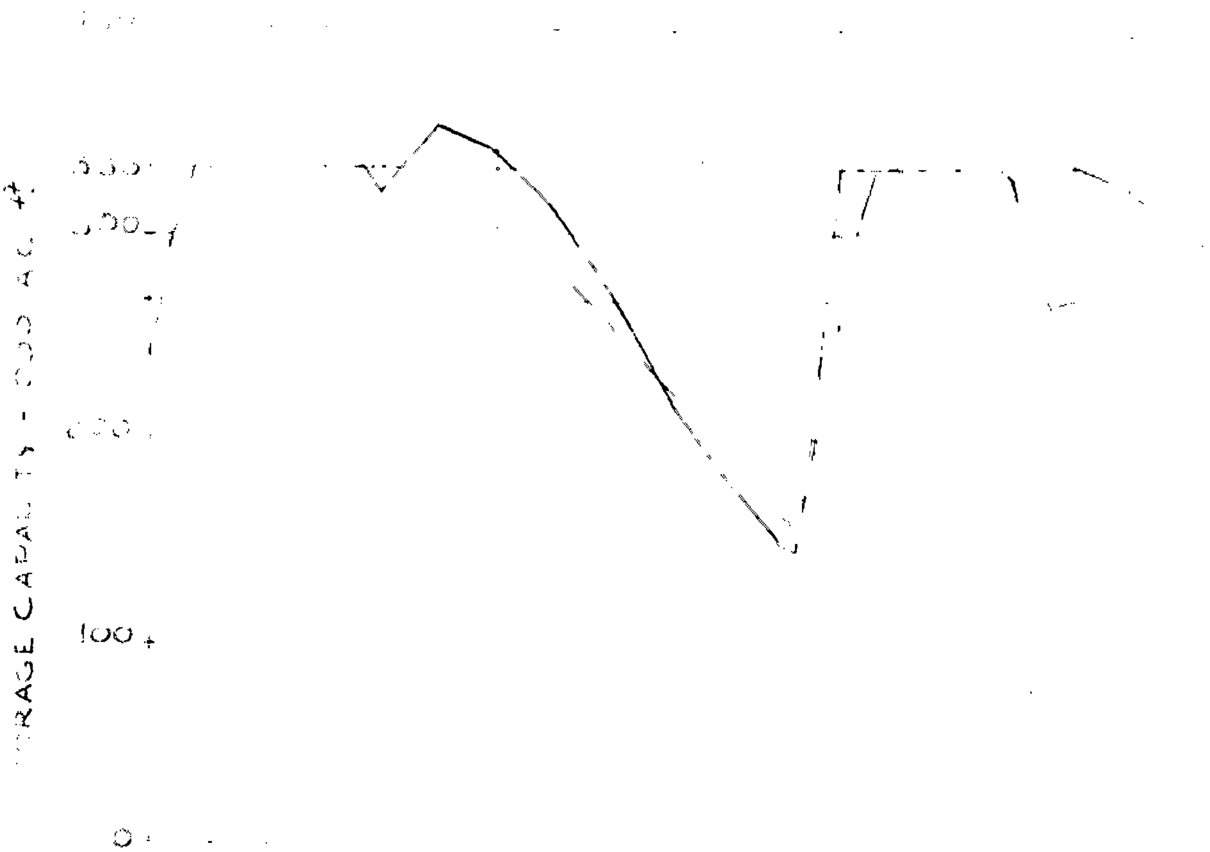
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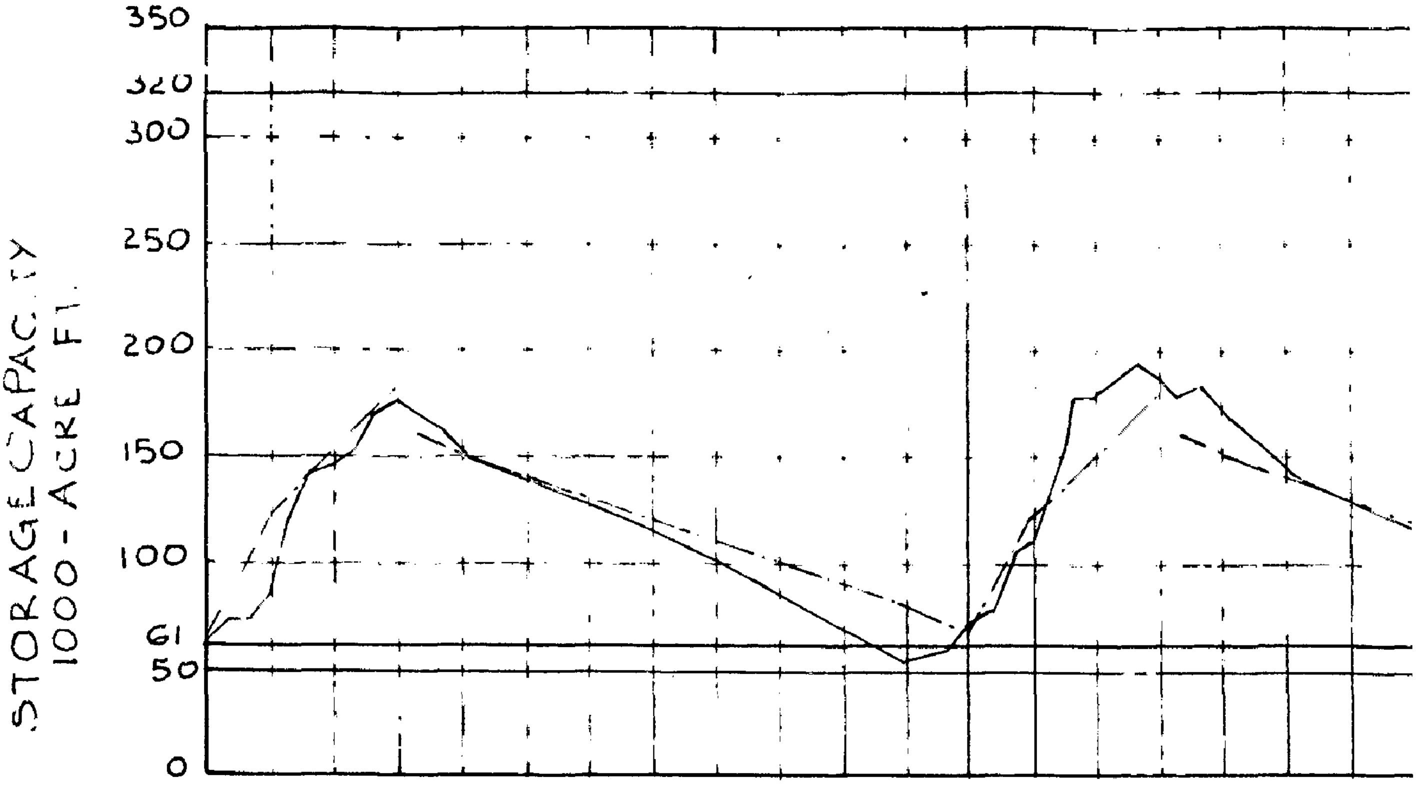
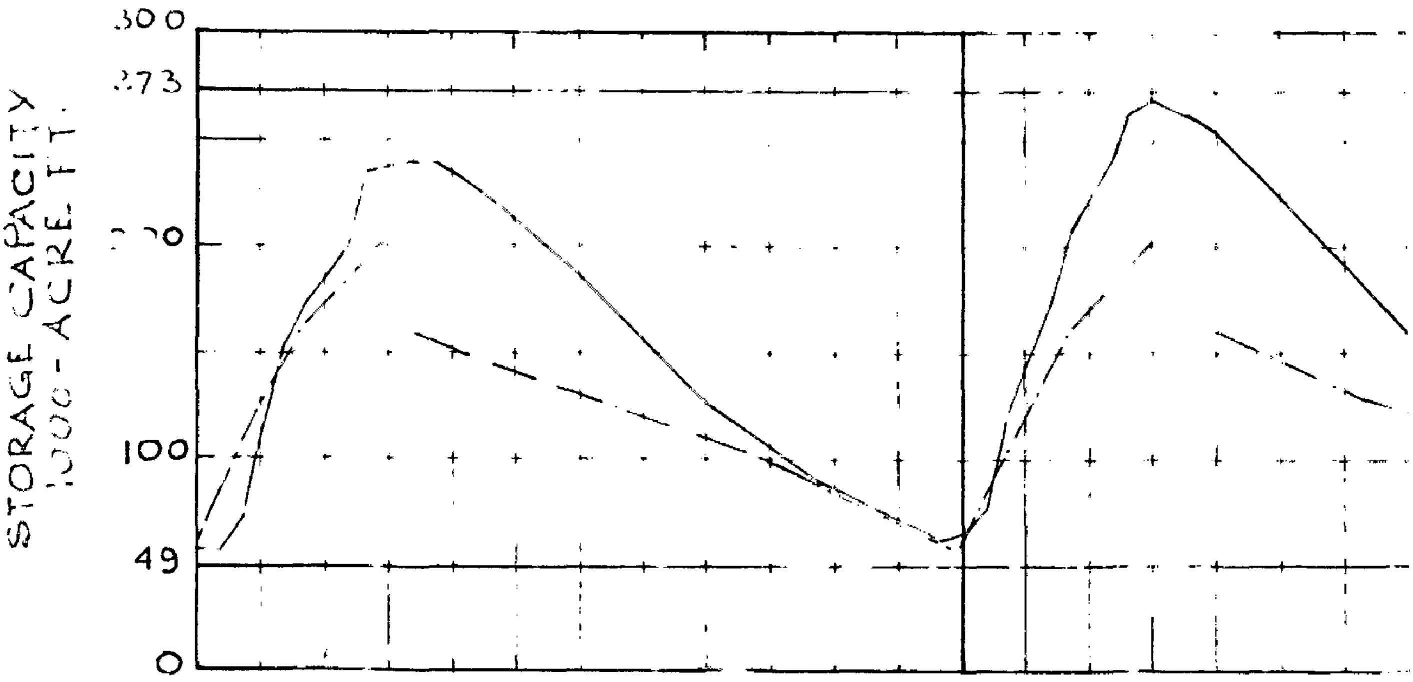

350
320
300
250
200
150
100
61
50
0
STORAGE CAPACITY
1000 - ACRE FT.


300
273
100
49
0
STORAGE CAPACITY
1000 - ACRE FT.

Chapter 6

E M B A N K M E N T S

6.1. L a y o u t

When the low land on both sides of the river is to be protected from flood waters, for justification to the land owners, one-half of the flood-way should be taken from each side. If the river channel is straight and local conditions permit, then only the embankments will be placed approximately equal distance from the channel. But, usually, river channels have lot of curves and it is not desirable that the embankments should follow all the sinuosities of the channel. In this case, it is seen that the alignment of the floodway is straighter than that of channel, which should be followed in deciding the alignment of embankments. Embankments should not be located close to the bend in the channel as in this case undermining will take place by caving of the river bank. Also, its slope will be subjected to the erosive action of high velocities of channel during floods. As far as possible, embankments should be straight and any change in direction should be provided with curves of large radius.

For deciding upon the suitable alignment the most important point is to know the probable limits of erosion and the likely future cut of the river. These can be judged by plotting the river courses and ascertaining the limits of the swings or oscillations the river is likely to take. It also depends, to a great extent, on the safe carrying capacity of the existing river section and the depth of the spill above the bank levels.

The nature of soil for the foundation must be firm enough to support the embankment and possess necessary resistance to harmful percolation and seepage. For an embankment to retain a known depth of water, the material should be suitably compacted to form a dense water-tight stable fill under conditions of saturation and drawdown. The soil should be well-graded and possess high density, low permeability and good cohesion.

Embankments are aligned so that important towns and properties along the river bank are left outside the embankment. Some times ring bund has to be provided to protect a town or urban property, if it has to be within the embankments. In such cases, river side of the ring bund may have to be protected by short permeable or impermeable spurs or stone revetment and pumps to be provided to pump out drainage and rain water into the river when the latter is in spate with high water-level. Such a ring bund has been provided for the town of Nirmali on the Kosi river.

The alignment of embankments has to be determined in such a way that the high velocity flow is sufficiently away from them. For this, hydraulic models are the useful guide.

Even when the alignment of embankments is determined from model tests, it should be emphasized that this will not necessarily be the case which will give satisfactory results for a long time. River conditions change continually, and an embankment which has behaved satisfactorily over a certain period, may be susceptible to attack during subsequent years. The success of an embankment depends upon vigilant and continuous supervision during the floods. When the embankments are likely to be attacked, immediate protection has to be provided. If, however, it is found that providiñn measures such as spurs, revetments, etc., would not give desired security to the embankments, a loop bund behind the existing embankment should be constructed. This will act as a second line of defence should the original embankment fail. Such loop bund has been provided for Chitauni Bund on the river Great Gandak.

6.2. Design of Embankment

For design of embankment, through knowledge of foundation conditions, type of the material to be used, its physical properties and construction methods is needed.

Embankments can be classified into two types: homogeneous type and zoned type.

(i) Homogeneous Embankments

In an embankment where only one type of material is used, it is called homogeneous embankment. This is used for low heights and sometimes for medium heights.

(ii) Zoned Embankments

Where at least two types of materials are used, inner section of impervious outer sections of relatively pervious materials, are called zoned embankments. This is used for high or structure of very great importance.

6.21 Foundation Design

Foundation is not actually designed, but certain provisions for treatment are made in design to ensure that the essential requirements are fulfilled.

Rock or hard shale foundations seldom present any problem of bearing power for embankment but erosive leakage, fault planes, &c, may become unsuitable and fail under continued saturation and hydraulic pressure. Designs for such foundations require particular care to secure tight or bonding with the superimposed embankment and proper control of seepage along the plane of contact.

Gravel, sand, clay and other granular materials are common to earthen embankment construction. Foundations of unconsolidated material very often present many varied and complex problems. Bearing power, local slips, displacements and percolation - are the problems involved.

(i) Control of seepage through foundations

Provisions made for seepage through foundations of unconsolidated and permeable materials are, cutoff trenches, steel sheet piling cutoffs, blankets of impervious material extending upstream from toe of the embankment. Inverted filters of sand and gravel may be employed to cover the area below the downstream toe of the embankment over which percolating water may escape under an appreciable gradient or head. Closing seams and joints and other openings in bed rock is accomplished by injection of grout under pressure usually to form a deep grout curtain or cutoff.

Factors influencing the stability of an embankment are weights of embankment materials, their coefficients of internal friction, stresses resulting from the adopted slopes, settlement and pore pressures which result from consolidation and protection.

Foundations of low bearing power may require extension of an otherwise suitable base width, to bring within allowable limits the bearing pressureof soil.

6.22. Essential requirements for design of Embankments

(1) The crest of the embankment should be above the high flood level and have adequate free-board against wave wash.

(2) For embankments along the banks of the rivers where generally the fetch is not much, a free-board of 4 feet over the designed High Flood level is considered adequate.

(2) The hydraulic gradient should be well within the bank with adequate cover. Hydraulic gradient varies according to the character of the soil and may be assumed as 1:1 in an impermeable clay varying to 1:12 in sandy soil. It should be inside the body of the embankment with a minimum cover of 0.9 m (3 ft).

(3) The structure should be stable under all stages of construction and conditions of saturation and drawdown.

(4) Theupstream slope should be properly protected against wave action, erosion and drawdown, and the downstream slope against action of rain and wind.

6.23. High Flood level

For correctly assessing the highest flood level, the following methods are adopted to suit the particular situation:

(1) Where long-term discharge and gauge data exist.

Where discharge and gauge data is available for about 40-to-50 years or longer period, a relationship between gauge and discharge is first established. The available discharge data is subjected to frequency analysis and a peak discharge of a

100-year flood is determined and adopted for design purposes. The gauge corresponding to this discharge is read off from the gauge discharge curve already established. This is further checked by determining a 100-year peak gauge by subjecting the available gauge data to a frequency analysis.

(2) Where the discharge and gauge data are available for a short period.

A relationship between the available gauge and discharge data is first established, then a relationship between storm rainfall and peak discharge is established for a period for which discharge data is available. Using all available past rainfall data, a suitable value is chosen for design storm rainfall intensity. The design flood peak discharge is then read off from the relation already established, and, finally, a design peak gauge is read from the established relation.

(3) Where no discharge or gauge data is available.

In such cases, all available storm rainfall data is examined and rainfall intensity is worked out. Then the resulting peak discharge is derived on the basis of hydrological characteristics of adjoining river basin for which data might be available.

In all the above three cases, the design flood peak discharge and gauge are further verified by computing the discharge for the corresponding gauge on the basis of available cross-section at the discharge site, observed slopes and velocities under conditions of flood flows. Under conditions of extreme paucity of data, suitable empirical formulae are used.

6.24. Hydraulic Gradient line

6.24. Hydraulic Gradient line

The top flow line or seepage line as per analysis and experiment is very nearly parabolic except for a short distance of reverse curvature at P as shown in Fig. 6.1

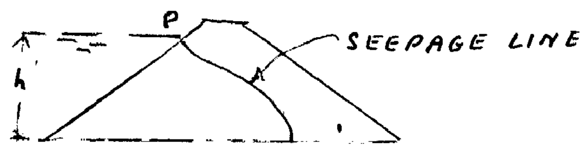


FIG. 6.1

As an approximation, the seepage line may be assumed as a straight line. For impermeable material as clay, it is taken as 1:1 and varying upto 1:12 for sandy soil. For safety of the embankment, water seeping from downstream is avoided and it is recommended that the seepage line should have 3-to-4-ft cover along the downstream slope and about 2-ft cover along the ground.

6.25. Stability analysis

In an earthen embankment, there are forces which cause movement of soil from upper level to lower level. Forces consist of weight of soil particles, frictional resistance between the soil particles and the cohesional resistance between the particles. Swedish engineers have studied that the surface of slippage is close to the circular in most cases. Therefore, the analysis for finding the stability of the slopes is known as slip circle test. The slope is considered safe when the factor of safety between the Resistive forces and the Actuating forces (component of weight of soil particles) is between 1.3-to-1.5 as shown in the following equation:

$$F_S = \frac{(\Sigma N - \Sigma P)\tan\phi + \Sigma CL}{\Sigma T} = 1.3 \text{ to } 1.5$$

where,

- N = normal component of wt. of soil particle
- T = tangential component of wt. of soil particle
- ϕ = angle of internal friction
- C = unit cohesive strength of soil
- L = length of arc of circle
- P = pore pressure

(i) <u>Upstream slope</u>

This lope is tested for reservoir full, sudden drawdown.

In case of reservoir-full condition test, weights of material below seepage line are taken as submerged weight, above seepage line as moist weight.

(a) <u>Sudden drawdown condition</u>

In this case, the intergranular forces that are submerged weight of soil, frictional force and cohesive force are in equilibrium among themselves and also the neutral forces that are weight of water and water pressure on the upstream face.

When the condition of sudden drawdown occurs, that is when the flood subsides quickly, the force due to water pressure on the upstream face of embankment is withdrawn and the equilibrium is thrown out of balance. Withdrawal is assumed instantaneous such thatno drainage occurs from the soil and saturation line is unchanged. In this case, the soil has to put forward additional force, that is unused cohesive force to avoid slippage. Values of C and ϕ in this case are to be taken as undrained shear test values.

For checking the stability of downstream face of embankment, the worst condition is when the reservoir is full. This is done in the same way as reservoir-full condition in upstream slope as described above.

(b) Condition of embankment just after the completion of construction

Due to mechanised construction methods, the height of embankment is raised quickly and the pore pressures in the bottom layer are built up due to superimposed load of the material above it. The above equation may be used for finding the factor of safety. In this case, the top surface of the cohesive soil may develop cracks and no cohesion is possible there. Therefore, reduced cohesion to be effective over the entire length of arch, that is in ratio of uncracked length to total length of arc. This check is performed for both the slopes.

6.26. Freeboard

The vertical distance between the top of the embankment and the maximum water level is called the free board. This extra height is the margin of safety against the floods of greater intensity than that designed for and affords protection against wave action during storms. A freeboard of 4 ft is considered adequate where the fetch is not much.

6.27. Crest

Experience shows that a minimum top width of 15 ft with suitable turning platform is quite desirable for inspection road, transport of material during construction and maintenance. A wide top has the further advantage of providing ample space for taking earth for strengthening any weak site during emergency. In special cases, top width may be reduced to 10-to-12 ft.

6.23. Side slopes

Principal elements influencing the side slopes are material of which it is made, method of construction, embankment height and length of time the embankment is subjected to flood waters. It should be safe against piping action, i.e. width of base must be such that the water percolating through it may not have sufficient force to carry any material with it ultimately resulting in a rush of water and soil from toe of the embankment and work subsiding into the hollow so formed. Also, it must be safe against slipping under worst conditions.

(i) Upstream slope or river side slope

For heights less than 10 feet, steepest slope for good soil which is to be used in most favourable conditions of saturation and drawdown is 2:1 and for heights 10 ft and above, 3:1. Also, it is to be seen that the river side slope is flatter than the under-water angle of repose of the material used for embankment. The slope may be as much as 4:1 in case of sandy material used for embankment, in this case a cover of one foot thick good soil is provided to protect the slope.

(ii) Downstream slope or land side slope

Downstream slope should be provided according to the hydraulic gradient to be determined for the material used in the embankment with the minimum cover of 0.9 m (3 ft). Hydraulic gradient is flatter in the case of more coarse material used. It is 1:12 for sand, and 1:4 for clay. Also, the slope may be made flatter as per the requirement of stability. Slope generally provided is 3:1 for height less than 10 ft and 4:1 for height 10 ft and above. If these slopes are not sufficient to give required cover for hydraulic gradient, berm of required width is provided on downstream slopes.

(iii) Berm

Berm is a horizontal benching provided on the land-side slope to prevent the water from seeping through the base of the embankment and causing the embankment to slough. The height of the berm must be such that its surface will be above the line of saturation (H . G.), and its width such that the lineof saturation will intersect the natural ground within its base width.

(iv) Embankment slope

Slope of the embankment should be the same as the water surface slope at Maximum flood height. Gauge heights which are calculated from the assumed Maximum discharge should be used. On aggrading type of steam, the slope of the water surface is greater than the slope of bed and on degrading type stream it is reverse.

(v) Protection to Earth banks

Upstream slope

The traditional method of protection against wave attack is by use of adequate grass covering. A short well-clipped grass, giving a good turf is also provided as a protective layer against surface erosion.

Sufficient freeboard may be provided for wave action and height of wave, that is from trough to crest of the wave, may be calculated from the Stevenson's formula given below.

$$h_w = 0.032\sqrt{VF} + 0.76 - 0.27F^{1/4}, \text{ for } F < 32 \text{ km.}$$

$$h_w = 0.032\sqrt{VF}, \text{ for } F > 32 \text{ km.}$$

where,

h_w is height of wave from trough to crest in metres

V is max. wind velocity in kms/hr

F is the fetch in km.

If the current is strong, pitching on slopes is provided. It should be taken at least 1 ft above the maximum height of wave length of wave expected. At places where the embankment is under the direct attacks of river, horizontal apron built up of stones in wire-crate or concrete blocks, in addition to slope pitching is provided. Length and the thickness of apron are calculated according to the expected scour which is found by applying the Lacey's scour depth formula -

$$R = 0.472 \left(\frac{Q}{f}\right)^{1/3}$$

where,

R is normal scour depth below H.F.L. in ft.

Q = discharge in cusecs

f = Lacey's silt factor, depending on sides and nature of bed and equals to $1.76\sqrt{dm}$ where dm is the mean size of bed material in inch.

$\therefore$ Depth of scour is taken as D = xR

where,

x = 1 at shank = 1.5 at nose.

Thickness of launching apron = 1.25 times the thickness of horizontal stone pitching (suggested by Spring). Where river is liable to sudden deep scour, it may be increased to 1.5 times.

Slope of launching apron suggested by Spring is 2:1.

$\therefore$ Length of apron when launched will be $\sqrt{5}\,D$

Volume of stone per foot width $= \sqrt{5}\,D\bar{t}$

where $\bar{t}$ is the thickness of apron.

For slope to be safe against sudden-drawdown conditions, stability analysis is to be carried out which is discussed in the ~~following~~ previous paragraphs. For this purpose free draining material or more pervious material is preferred on upstream slope. High embankments are protected with rip-rap and low embankments with willow mattress.

(vi) Rip-rap

Twelve-to-twentyfour inches thick durable stone are hand-packed on the slope of the embankment and joints are filled with spalls or small rock fragments. In case fines may be washed out through the voids in the rip-rap, 6-12 in. thick layer of graded filter is provided below the rip-rap.

(vii) Downstream slope protection

(a) Turfing

The slope to be provided turfing is to be dressed with 6-in. top layer of good soil. Sod consists of set of roots and earth at least two inches thick is laid on the slope in close contact and tamped firmly in place. The slope should be wetted till the plant growth is fully established and should not be allowed to dry out. Grass which grows in bunches on having deep roots should not be used as it will provide easy percolation path for water.

Where growth of grass is not possible such as in arid or semi-arid regions, gravel or rock blanket should be providedfor the protection.

Rain water may cause erosion of the slope and form gullies along the slope, for this purpose longitudinal drains may be provided on berms with reverse slope and cross drains at suitable site for draining out the water safely.

(viii) Toe of the embankment

For preventing any soil particles to be washed away through seepage under the embankment, filter blanket is provided which extends from rock toe upto some distance near heating material. A drain filled with boulders is provided at the end of rock toe for draining out the seepage water.

6.30. Spacing of embankments :

The spacing and the height of embankments are inter-dependent. Either the embankments can be made low and far apart, or they can be made higher and closer together. The correct location of the embankments from an economic standpoint is that which makes the sum of the cost of the embankments and the value of the unreclaimed land in the floodway a minimum. In this connection, the value of the unreclaimed lands should be taken as the difference between the market value of the land, if it were possible to protect it from overflow, and its value for the floodway purposes.

The spacing and height of embankments are determined by a series of trial solutions as shown below.

6,31. Outline of method :

First, a height of water level above the bank of river is assumed and to this a free board of 3 to 4 feet is added to get the height of embankments to carry the estimated discharge. The discharge through and above the channel proper is then computed and this discharge subtracted from the maximum discharge gives the discharge which must pass over the flood ways between the channel proper and the embankments. The latter discharge divided by the computed velocity of flow along the flood-way gives the cross-sectional area to be provided by the flood-way. This area divided by the depth of flow over the flood plaining gives the width of the flood-way, exclusive of the channel width. The cost of the embankments, land acquisition of temporary and permanent lands, value of unpro-tected land in floodway, compensation for the crops and hutments falling under the lands for embankments and borrow pits is then computed. Similar solutions are made for other heights of water level above the river banks; and as previously stated that spacing of embankments which gives a minimum value for the sum of the cost of embankments and the value of unprotected land, is the proper spacing (corresponding the height of embankment) to use.

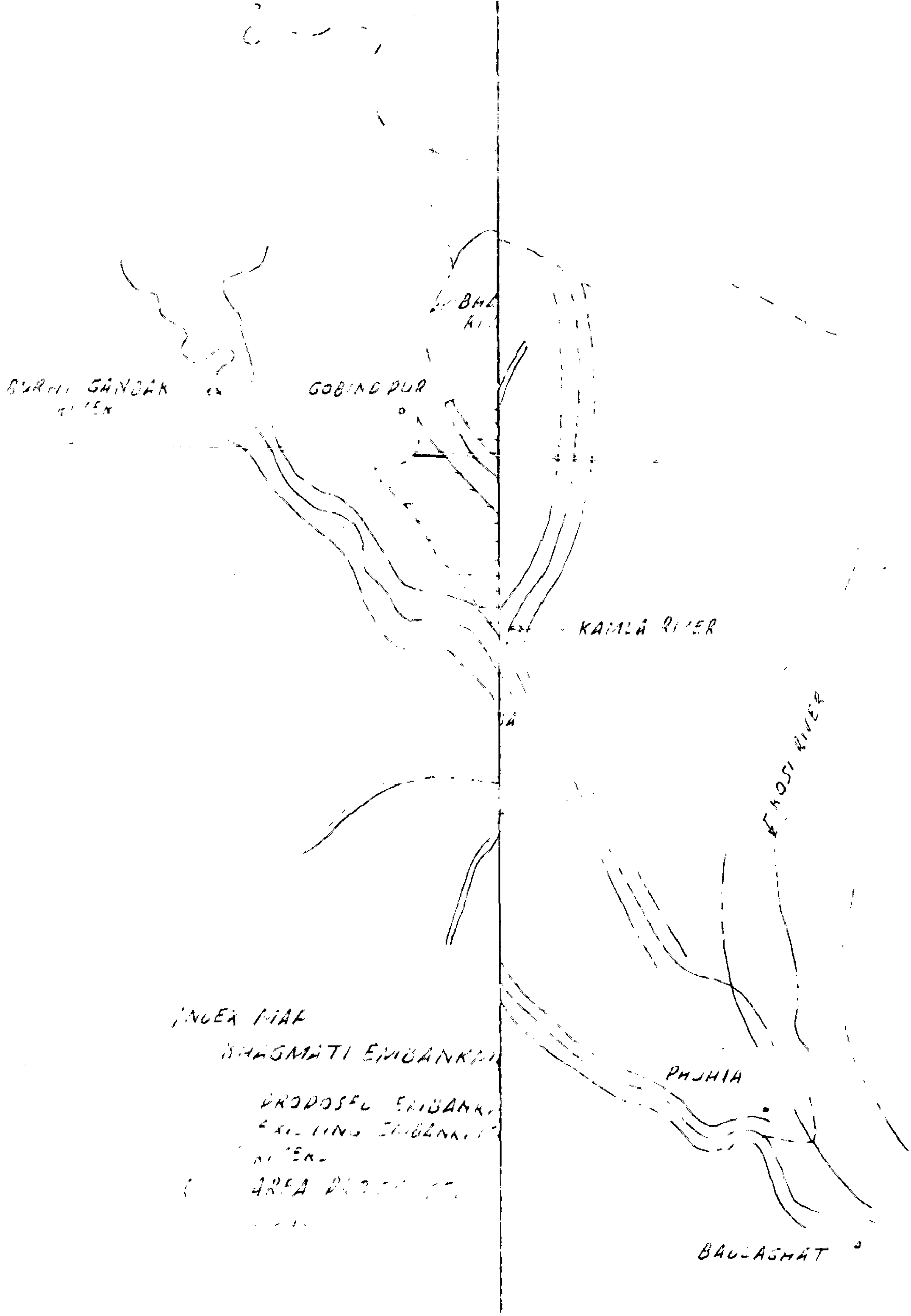
The example of Bhagmati Flood Control Scheme is given below to understand better the several steps involved in the determination of the correct spacing and height of embankments.

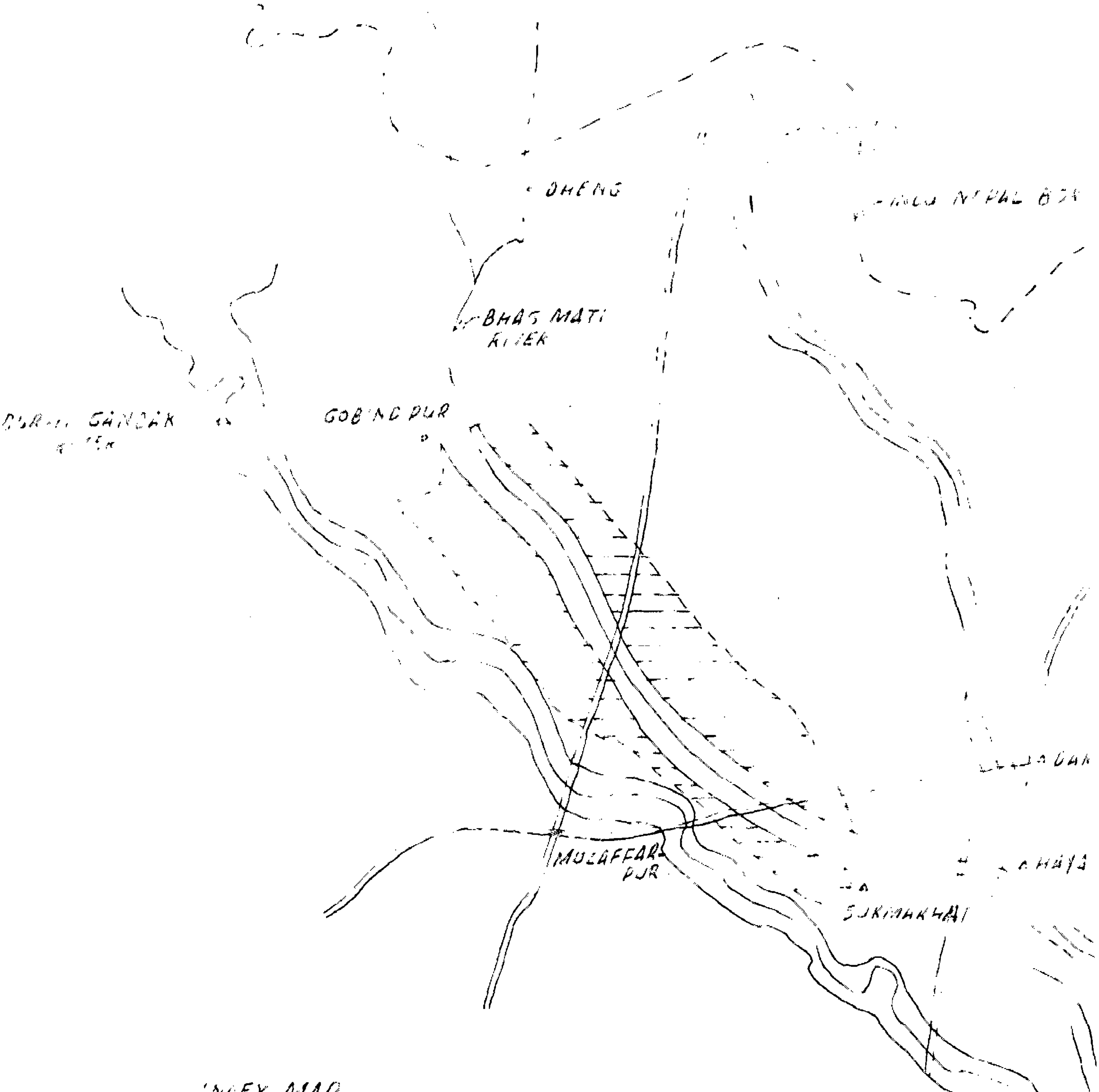
6.32. Bhagmati embankment scheme :

Introduction :

The river Bhagmati, one of the perennial rivers of North Bihar rises in the Sheopuri range of hills in Nepal at an altitude of about 7000 ft. above M.S.L. The total length of the river from its place of origin to the point of outfall is 365 miles, out of which 122 miles fall in Nepal and the rest 243 miles in India.

The catchment is fan shaped at the top. Below Hayaghat it is more or less rectangular in shape. Out of the total catchment of 5,187 sq. miles, 2747 sq. miles lie in Nepal and the balance 2440 sq. miles in Bihar.


GOBIND PUR
KOSI RIVER



INDEX MAP
BHAGMATI EMBANKMENT SCHEME.

PROPOSED EMBANKMENT.
EXISTING EMBANKMENT
RIVER
AREA PROTECTED

FIG 5

The Bhagmati river flows between two big rivers, the Gandak and the Kosi. The flood in Bhagmati is an annual phenomenon and is mainly due to the heavy rains in its hilly catchment, which is often synchronised with heavy rains in plains lower down. Bhagmati has a steep slope of 4.55 ft.per mile between Noon Thore and Dheng Railway bridge. Below Dhung Railway bridge, the steep slope abruptly changes to a flatter slope. The silt carried by the river is therefore deposited in the reach from Dheng Railway bridge to Muzaffarpur- Darbhanga road crossing. This results in raising the bed of the river. It is in this middle reach that the river section is normally wide and shallow and spills over, even in comparatively low floods.

The maximum discharge occurred in 1965 was 90,080 cusecs at Dheng railway bridge.

6.33. Problem :

The river Bhagmati has already been embanked in its lower reaches vis. (a) along its left bank from Hayaghat to Phahia, 46 miles and (b) along its right bank from Surmarhat to Badlaghat, 101 miles.

The main problems of the Bhagmati are that :-

a) it innudates the area of 800 sq. miles in Bihar where the crops are mostly damaged.

b) it has carved out several spill channels on both its bank.

c) it has a tendency to avulge in river Burhi Gandak, through its right spill channels.

d) its section is inadequate and not capable of taking its full discharge.

e) it erodes its banks and fertile lands are engulfed into the river.

f) the command area of the proposed Bhagmati irrigation scheme, below Dewapur, is also to be made flood free.

6.34. Proposals :

For tackling the problem of Bhagmati a preliminary investigation of a 230' high dam at Ramchhu about 6 miles up-stream of Noonthore costing Rs.18 crores was done. This proposed dam, if executed, would have controlled the flood discharge to the extent of one lakh cusecs. This project will no doubt moderate the flood in the Bhagmati basin, but will not make the area flood free completely as the section of the river is not capable of accomodating even a discharge of 25,000 cusecs. Hence in addition to the dam at Noonthore, embankments will have to be provided to prevent spilling.

Embankments in the following reaches of Bhagmati are proposed as Bhagmati flood control scheme.

i. Embankment along the right bank of river Bhagmati from Sheopur Gobindpur road to Surmarhat (Down stream terminal tagged with existing Hayaghat Surmarhat embankment) .

ii. Embankment along left bank of Bhagmati from Sheopur-Gobindpur road to Samastipur Darbhanga road at Jathmalpur.

6.35. <u>Available Data</u> :

Proposed embankment with a 50-year design flood will rise the H.F.L. considerably and consequently may adversely affect the drainage of the country side. Hence the proposed embankment should be designed for 25-year flood 101,000 cusecs and escapes should be provided at approximate places for discharging a portion of the flood discharge above the 25-year flood.

6.35.1. <u>Free Board</u>:

A free board of 4 ft. over the design H.F.L. along the embankment has been provided to arrive at the design crest level of the embankment.

6.35.2. <u>Embankment Section</u> :

It is proposed to keep the top width of the embankment 16ft. with side slopes 2 : 1 on the river side and 3 : 1 on country side. The slope of line of saturation has been taken as 5 : 1.

6.36. <u>Procedure</u> :

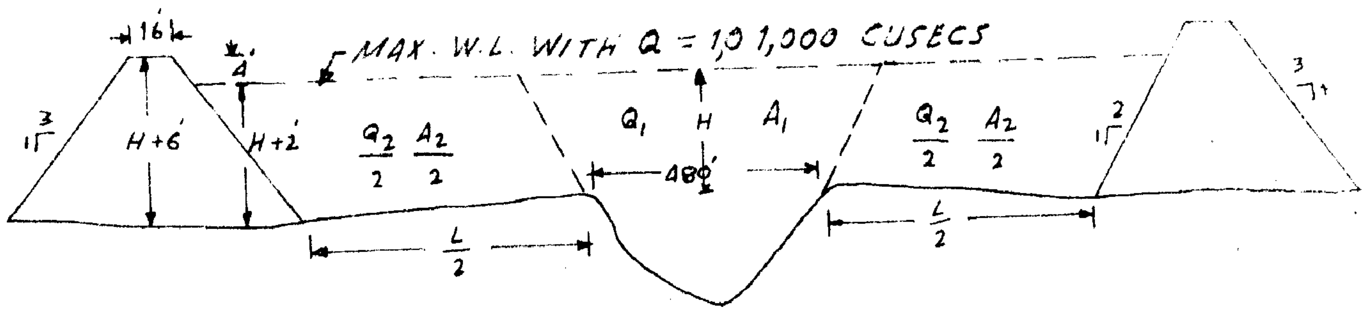


FIG. 6.3

Data given :

Q	=	1,01,000 cusecs, design flood discharge
A	=	channel X-section area (RST)
P_1	=	wetted perimeter (RST)
i	=	slope of channel
n	=	roughness factor for channel, n' = roughness factor for flood-way
A_1	=	X RST Y = effective channel X-sectional area
Q_1	=	discharge through effective channel x-section area.
Q_2	=	discharge through flood-way
L	=	length of flood-way
H	=	depth of water above the banks

Given river channel and flood plain as shown by cross-section PQRSTLM in the above figure. The flood discharge to be carried by the embanked channel cross-section is 1,01,000 cusecs, the area of the channel x-section RST is 4156.8 sq.ft., the wetted perimeter P_1 = RST is 482.69 feet, the slope of the stream is 0.00019; n for the channel is given as 0.025 and for the floodway 0.05. The value of land for permanent acquisition (for seat of the embankments) is Rs.2500/- an acre and for temporary acquisition (for borrow pits) is Rs.300/- an acre. Crop compensation given @ Rs.200 an acre and ~~for~~ compensation for hutments Rs.600 per mile.

Assume a trial height of water 3.5 feet above the bank level and 4 feet as free board. With the water 3.5 ft.

above the bankful level, the area of the effective channel XRSTY, will be increased 1690 sq. ft., making a total cross-sectional area of 5846.8 sq.ft. The wetted perimeter, RST, is 482.69 ft. and hence the hydraulic radius will be 12.1 ft. Substituting in Manning's formula, the mean velocity is found to be 4.3 ft/sec; and with this velocity the discharge Q_1 would be 25,100 cusecs. Since the flood discharge to be carried is 1,01,000 cusecs, the floodway must be designed to carry Q_2 = 75,900 cusecs. The velocity of flow along the floodway for a roughness factor of0.05 average depth of 4.5 ft. and a slope of 0.00019 would be 1.12 ft/sec. To carry a discharge of 75,900 cusecs at a velocity of 1.12 ft/sec., the area of the floodway must be 67,800 sq.ft and since the average depth of flow at floodway is 4.5 ft., the width of the floodway must be 15,050 ft., exclusive of the channel proper.

Cost of the embankments with given top width and side slopes and height 9.5 ft. is worked out for one mile length as Rs.1.39 lakhs. A floodway one mile long and 15,050 ft. wide would contain 1825 acres. Since the potential value of this land for agricultural is only Rs.2500 per acre, while its value for floodway purposes is only Rs.2000 per acre, the loss is Rs.500 per acre or Rs.9.13 lakh per mile of floodway. The cost of permanent land width for one mile long embankments @ Rs.2500 per acre is Rs.0.40 lakh. Cost of temporary land for taking earth for one mile embankments @ Rs.300 per acre is Rs.0.16 lakh. Cost paid for damaging standing crops @ Rs.200 per acre is Rs.0.10 lakhs. Compensation paid for hutments @ Rs.600 per mile is Rs.1200. The total cost for this height is Rs.11.20 lakh.

In like manner the spacing and the total cost for the heights of water depth above bank level for 4.5', 6.5', 8.5', 12.5' and 16.5' are worked out as shown in the following calculations. Values of the total cost have been plotted against the corresponding spacing of embankments. (Fig. 6.4). Hence from the curve the 4,500 ft. spacing is seen to be the most economical. Calculations for the same are given below :

REF. FIG. 6.3 P. 90.

Q = 1,01,000 design flood discharge

A = channel x-sec. area (RST)

P_1 = wetted perimeter (RST)

i = slope of channel

n = roughness factor for channel, n'= roughness factor for floodway.

A_1 = effective channel x-sec. area (XRSTY)

Q_1 = discharge through effective channel x-sec. area (XRSTY)

Q_2 = discharge through floodway.

L = length of floodway

H = depth of water above banks.

6.37. Calculations :

6,37.1. Depth of water level above banks assumed as 3.5 ft.

Area of portion above channel

$= 843.5 \times 3.5 = 1690$ sq.ft.

$Q = 1{,}01{,}000$ cusecs.

$P_1 = 482.69$ ft.

$A_1 = 4156.8 + 1690 = 5846.8$ sq.ft.

$R_1 = \frac{5846.8}{482.69} = 12.1$ ft.

$V_1 = \frac{1.486}{0.025} \times (12.1)^{2/3} \times (0.00019)^{1/2}$

$= 4.3$ ft/sec.

$Q_1 = 5846.8 \times 4.3 = 25{,}100$ cusecs

$Q_2 = 1{,}01{,}000 - 25{,}100 = 75{,}900$ cusecs.

$V_2 = \frac{1.486}{0.005} \times 4.5^{2/3} \times 0.00019^{1/2}$

$= \frac{1.486 \times 2.73 \times 0.01378}{0.05}$

$= 1.12$ ft/sec.

$A_2 = Q_2/V_2 = 75{,}900/1.12 = 67{,}800$ sq. ft.

$L = A_2/4.5 = 67{,}800/4.5 = 15{,}050$ ft.

Spacing $= 15{,}050 + 480 = 15{,}530$

Cost of earthwork :

Cross section area :

$= 16 \times H + 2.5\ H^2$

$= 16 \times 9.5 + 2.5 \times 9.5^2$

$= 152.0 + 2.5 \times 90.25$

$= 152.0 + 225.6$

$= 377.6$ sq.ft.

Quantity of earthwork for both embankments for one mile length.

$= 377.6 \times 5280 \times 2 = 3{,}99{,}00{,}000$ cft.

Cost at Rs.34.9/ %o cft.

$= 39900 \times 34.9 = 139{,}000$ Rs.

B. <u>Cost of land</u> :

Width $= 16 + 5 \times 9.5 = 16 + 47.5 = 63.5$ ft.

area $= \frac{63.5 \times 2 \times 5280}{43560} = 15.35$ acres

Cost at rate of Rs.2500/ ac. $= 2500 \times 15.97$

$=$ Rs.40,000

C. Loss due to value of unprotected land in the floodway.

@ Rs.500/ac.

area $= \frac{15{,}050 \times 5280}{43560} = 1825$ acres.

Cost $= 500 \times 1825 = 9{,}12{,}500$

D. Cost of temporary land for taking earth for embankment

(2 ft. depth of borrow pit).

Area $= \frac{\text{Quantity of earth work/mile}}{2 \times 43560}$ acres

$= \frac{3{,}990{,}000}{2 \times 43560}$ $= 45.8$ ac $\pm$ 24% | 12% for each side for office, barrack, parallel strip etc.

$= 45.8 + 9.16$

$= 54.96$ ac.

Cost @ Rs.300/ac.

= 30 x 54.96 = Rs. 16,488

E. Compensation to be paid for damaging standing crops

(area = 1/2 of permanent land + 3/4 of temporary land)

Area = 1/2 x 15.97 + 3/4 x 54.96

= 7.98 + 41.25 = 49.23

Cost @ Rs.200/ ac.

= 200 x 49.23 = Rs.9,846

F. Compensation to be paid for hutments falling on the alignment of embankment @ Rs.600 per mile for one side.

Cost = 2 x 600 = Rs. 1,200

Total cost = A + B + C + D + E + F

= Rs. 11.20 lakh

6.37.2. <u>Depth of water level above bank assumed as 4.5 ft.</u>

Area of portion above channel

= 484.5 x 4.5 = 2180

Q = 101,000 cusecs

P_1 = 482.69 ft.

A_1 = 4156.8 + 2180 = 6336.8 sq.ft.

R_1 = 6336.8/482.69 = 13.1 ft.

$V_1 = 1.486/0.025 \times 13.1^{2/3} \times 0.00019^{1/2}$

$= 1.486/0.025 \times 5.55 \times 0.01378 = 4.53$ ft/sec.

$Q_1 = 6336.8 \times 4.53 = 28{,}700$ cusecs.

$Q_2 = 101{,}000 - 28{,}700 = 72{,}300$

$V_2 = 1.486/0.05 \times 5.5^{2/3} \times 0.00019^{1/2}$

$= 1.486/0.05 \times 3.12 \times 0.01378 = 1.28$ ft/sec.

$A_2 = 72{,}300/1.28 = 56{,}600$ sq.ft.

$L = 56{,}600/5.5 = 10{,}300$ ft.

$S = 10{,}300 + 480 = 10{,}780$

A. Cost of Earthwork :

Cross-section area $= 16 \times H + 2.5 H^2$

$= 16 \times 10.5 + 2.5 \times 10.5^2$

$= 1680 + 2.5 \times 110.25$

$= 168.0 + 276.0 = 444.0$ sq.ft.

Quantity of E.W. $= 444.0 \times 5280 \times 2 \neq 46{,}80{,}000$

Cost $= 4680 \times 36.2 = 1{,}69{,}500$ Rs.

B. Cost of land : width $= 16 + 5 \times 10.5 = 16 + 52.5 = 68.5$ ft.

Area $= 68.5 \times 2 \times 5280/43560 = 16.6$ acre + 4 % i.e. .66

$= 17.26$

Cost $= 2500 \times 17.26 = 43{,}100$ Rs.

C. Loss due to valve of unprotected land in flood way :

area = 10,300 x 5280/43560 = 1250 acres.

Cost = 500 x 1250 = 6,25,000 Rs.

D. Cost of temporary land

Area = 4680,000/2 x 43,560 = 53.8 + 24 %

= 53.8 + 12.9

= 66.7 ac.

Cost = 300 x 66.7 = Rs.20,010

E. Compensation for crops :

Area = 1/2 x17.26 + 3/4 x 66.7

= 8.63 + 50.0 = 58.63 Ac.

Cost = 200 x 58.63 = Rs.11,726

F. Compensation for hutment = Rs.1200

Total cost = A + B + C + D + E + F = Rs.8.71 lakhs

6.37.3. Depth of water level above banks assumed as 6.5 ft.

Q = 101,000 cusecs.

P_1 = 482.69 ft.

A_1 = 4156.8 + 486.5 x 6.5 = 4156.8 + 3120=7276.8 sq.ft.

R_1 = 7276.8/482.69 = 15.1 ft.

$V_1 = 1.486/0.025 \times 16.1^{2/3} \times 0.00019^{1/2}$

$= 1.486/0.025 \times 6.1 \times 0.01378$

$= 5.0$ ft/sec.

$Q_1 = 7276.8 \times 5.0 = 36,384.0$ cusecs.

$Q_2 = Q - Q_1 = 101,000 - 36,384 = 64,616$ cusecs.

$V_2 = 1.486/0.05 \times 7.5^{2/3} \times 0.00019^{1/2}$

$= 1.486/0.05 \times 3.83 \times 0.01378$

$= 1.57$ ft/sec.

$A_2 = Q_2/V_2 = 64616/1.57 = 41,200$ sq.ft.

$L = 41200/7.5 = 5,500$ ft. $S = 5,500 + 480 = 5980$ ft.

A. Cost of earthwork :

c/s area $= 16 \times H + 2.5\ H^2$

$= 16 \times 12.5 + 2.5 \times 12.5^2$

$= 200 + 390.6 = 590.6$ sq./ft.

Cost @ Rs.37.1/ ‰ cu.ft.

$= \frac{590.6 \times 5280 \times 2}{1000} \times 37.1 = 2,31,500$

B. Width of embankment $= 16 + 5\ H = 78.5$

area of land required $= \frac{78.5 \times 2 \times 5280}{43560}$

$= 19.0$ acres + 4 % i.e. 0.76

$= 19.76$ acres.

Cost @ Rs.2500/ ac. $= 19.76 \times 2500 =$ Rs.49,400

C. Loss due to value of unprotected land in flood ways

$$\text{area} = \frac{5500 \times 5280}{43560} = 666 \text{ acres}$$

Cost = 500 x 666 = 3,33,000 Rs.

D. Cost of tempeorary land

$$\text{Area} = \frac{6,240,000}{2 \times 43560} = 71.2 + 24 \%$$

$$= 71.2 + 17.2$$

$$= 88.4 \text{ ac.}$$

Cost = 300 x 88.4 = Rs.26,520

E. Crop compensation

area = 1/2 x 19.76 + 3/4 x 88.4

= 9.88 + 66. 3

= 76.18 ac.

Cost = 200 x 76.18

= Rs. 15,236

F. Compensation for hutments = Rs. 1200

Total cost = A + B + C + D + E + F = Rs.6.57 lakhs

6.37.4 Depth of water level above banks assumed as 8.5 ft.

Area of portion above channel = 488.5 x 8.5 = 4,150

Q = 101,000 cusecs.

P = 482.69 ft.

A_1 = 4156.8 + 4,150 = 8, 306.8 sq.ft.

R_1 = 8306.8/482.69 = 17.25 ft.

$$V_1 = 1.486/0.025 \times 17.25^{2/3} \times 0.00019^{1/2}$$

$$= 1.486/0.025 \times 6.7 \times 0.01378 = 5.47 \text{ ft/sec.}$$

$$Q_1 = 8306.8 \times 5.47 = 45{,}500 \text{ cusecs.}$$

$$Q_2 = 101{,}000 - 45{,}500 = 55{,}500 \text{ cusecs.}$$

$$V_2 = 1.486/0.05 \times 9.5^{2/3} \times 0.00019^{1/2}$$

$$= 1.486/0.05 \times 4.5 \times 0.01378 = 1.845 \text{ ft/sec.}$$

$$A_2 = 55{,}500/1.845 = 30{,}200$$

$$L = 30{,}200/9.5 = 3{,}180 \text{ ft.}$$

$$S = 3{,}180 + 480 = 3{,}660 \text{ ft.}$$

<u>Cost of earthwork</u> :

A. cross-section area

$$= 16 \times H + 2.5\ H^2$$

$$= 16 \times 14.5 + 2.5 \times 14.5^2$$

$$= 232.0 + 525.6 = 757.6 \text{ sq.ft.}$$

Quantity of E.W.

$$= 757.6 \times 5280 \times 2 = 80{,}00{,}000 \text{ cft.}$$

Cost 8,000 x 43 = Rs.3,44,000

Cost of land.

B. Width $= 16 + 5 \times 14.5 = 16 + 72.5 = 88.5$ ft.

$$\text{Area} = \frac{88.5 \times 2 \times 5280}{43{,}560} = 21.45 \text{ ac.} + 4\%\text{ i.e. } 0.86$$

$$= 22.31 \text{ ac.}$$

Cost = 2500 x 22.31 = Rs.55,700

C. Loss due to value of unprotected land in flood way

Area $= 3180 \times 5280 / 43{,}560 = 386$ ac.

Cost $= 500 \times 386 = 1{,}93{,}000$ Rs.

D. Cost of temporary land

$$\text{Area} = \frac{80{,}00{,}00}{2{,}43560} = 91.8 + 24\ \%$$

$$= 91.8 + 22.0$$

$$= 113.8 \text{ Ac.}$$

Cost $= 300 \times 113.8 =$ Rs.34,140

E. Crop compensation

area $= 1/2 \times 22.31 + 3/4 \times 113.8 = 96.45$ ac.

Cost $= 200 \times 96.45 =$ Rs.19,290

F. compensation for hutments = Rs.1200

Total cost = A + B + C + D + E + F = Rs.6.48 lakhs

6.37.5. Depth of water level above banks assumed as 12.5 ft.

Area of water above channel $= 492.5 \times 12.5 = 6150$ sq.ft.

$Q = 101{,}000$ cusecs.

$P_1 = 482.69'$

$A_1 = 4156.8 + 6150 = 10{,}306.8$ sq.ft.

$R_1 = 10{,}306.8 / 482.69 = 21.3$

$$V_1 = 1.486/0.025 \times 21.3^{2/3} \times 0.00019^{1/2}$$

$$= 1.486/0.025 \times 7.8 \times 0.01378$$

$$= 6.37 \text{ ft/sec.}$$

$Q_1 = 6.37 \times 10{,}306.8 = 65{,}600$ cusecs.

Q_2 = 101,000 - 65,600 = 35,400 cusecs.

$V_2 = \frac{1.485}{0.03} \times 13.5^{2/3} \times 0.00019^{1/2}$

= 2.32 ft/sec.

A_2 = 35,400/ 2.32 = 15, 300

L = 15,300/ 13.5 = 1135 ft.

S = 1135 + 480 = 1615 ft.

Cost of earthwork:

A = Cross section area

$\frac{16 + 103.5}{2} \times 18.6$ = 1154 + 36 = 1190

Quantity of E.W. for one mile length = 2 x 5280 x 1190 = 125,50,000 cuft.

Cost = 125,50,000/1000 x 49 = Rs.615,000

B. Cost of land. Area = $\frac{114.5 \times 2 \times 5280}{43560}$

= 27.7 + 4% i.e. 1.11

= 28.81

Cost = 2500 x 28.81 = Rs.72,000

C. Loss due to valve of unprotected land in flood way.

Area = $\frac{1135 \times 5280}{43560}$

= 138 acres

Cost = 500 x 138 = Rs.69,000

D. Cost of temporary land

area = $\frac{125,50,000}{2 \times 43560}$ = 158.5 + 24%

= 158.5 + 38.0

= 196.5 ac.

Cost = 300 x 196.5 = Rs.58,950

E. crop compensation

area = 1/2 x 28.81 + 3/4 x 196.5

= 14.40 + 147.3 = 161.7 Ac.

Cost = 200 x 161.7 = Rs.32,340

F. Compensation for hutments = Rs.1200

Total cost = A + B + C + D + E + F = Rs.8.48 lakhs

6.37.6. Depth of water level above banks assumed as 16.5 ft.

Area of portion above channel

= 496.5 x 16.5 = 8180 sq.ft.

Q = 1,01,000 cusecs.

P_1 = 482.69 ft.

A_1 = 4156.8 + 8180 = 12, 336.8 sq.ft.

R_1 = 12336.8/482.69 = 25.5 ft.

V_1 = $1.486/0.025 \times 25.5^{2/3} \times 0.00019^{1/2}$

= 1.486/0.025 x 8.7 x 0.01378 = 7.1 ft/sec.

Q_1 = 7.1 x 12, 336.8 = 87,500 cusecs.

Q_2 = 101,000 - 87,500 = 13,500 cusecs

V_2 = $1.486/0.05 \times 17.5^{2/3} \times 0.00019^{1/2}$

= 1.486/0.05 x 6.75 x 0.01378 = 2.76 ft/sec.

A_2 = 13,500/2.76 = 4,900 sq. ft.

L = 4900/17.5 = 280 ft.

S = 480 + 280 = 760 ft.

Cost of earthwork:

A C/s area = 1,732sq. ft. Cost @ Rs.58/ % cuft.

$$= \frac{1,732 \times 5280 \times 2 \times 58}{1000} = Rs. 10,60,000$$

B Width = 128.5 ft. area $\frac{128.5 \times 2 \times 5280}{43560}$ = 31.2 ac. +4%

(i.e. 1.248) = 32.45 ac.

Cost = 2500 x 32.45 = Rs.81,000

C. Loss due to valve of unprotected land in flood way.

$$\text{Cost} = \frac{500 \times 280 \times 5280}{43560} = 17,000$$

D. Cost of temp. land

area = 183,00,000/2x43560 = 210 + 24 %

= 210 + 50.4 = 260.4 Ac.

Cost = 300 x 260.4 = Rs.78,120

E. Crop compensation

area = 1/2 x 32.45 + 3/4 x 260.4

= 16.22 + 195.3 = 211.52 ac.

Cost = 200 x 211.52 = Rs.42,304

F. Compensation for hutments

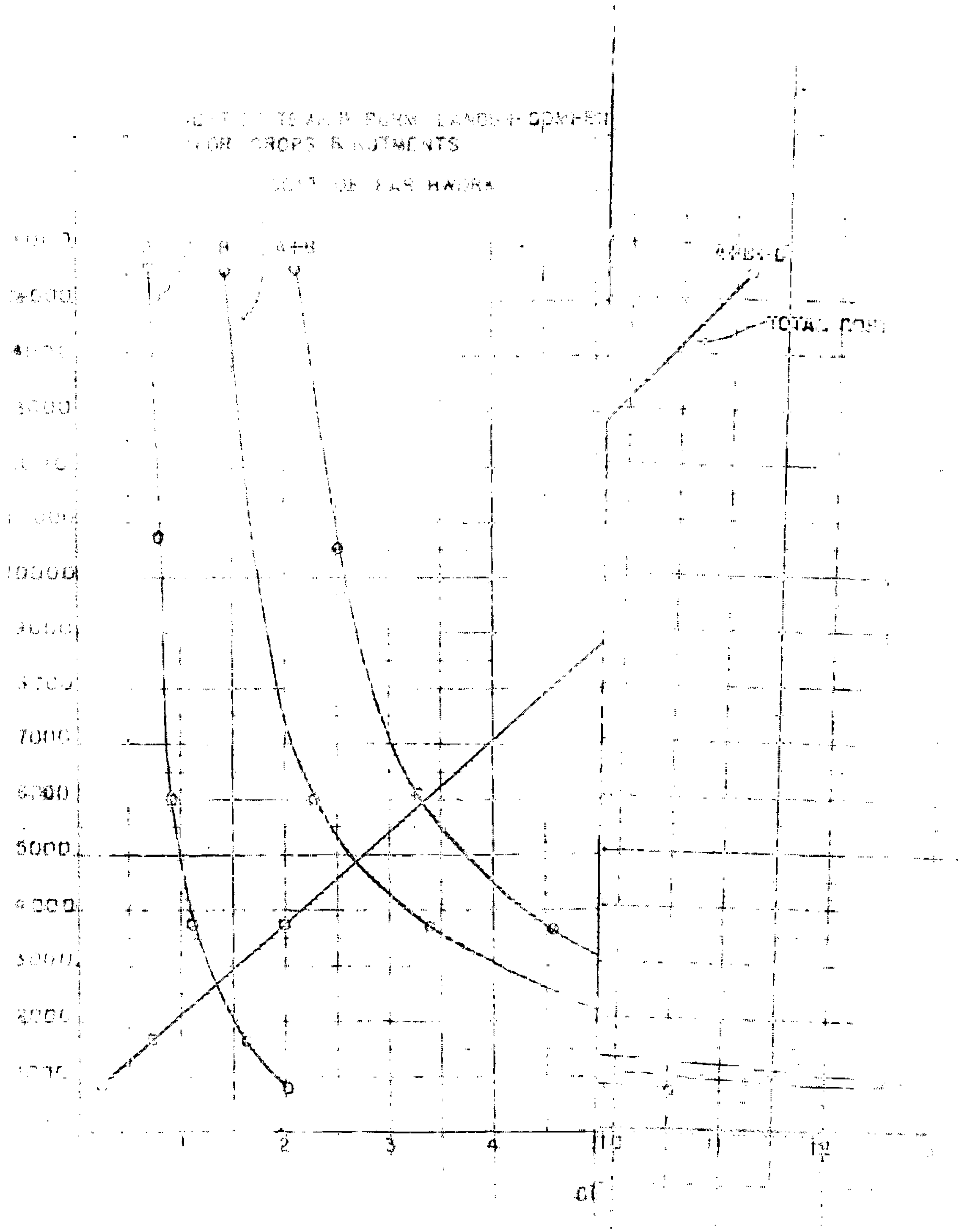
= Rs. 1200

Total cost = A + B + C + D + E + F

= Rs. 12.80 lakhs.

Table 6.1

Spacing of embankments in feet	Cost of earth work in embankments	Cost of land acquisition, compensation for crops and hutments	Loss due to valve of unprotected land for floodway	Cost of emb. + cost of land acquisition, comp. for crop & hutment	Total cost (Col.2+3+4) Rs.lakhs
1	2	3	4	5	6
15,530	1.39	0.68	9.13	2.07	11.20
10,780	1.70	0.76	6.25	2.46	8.71
5,980	2.32	0.92	3.33	3.24	6.57
3,660	3.44	1.11	1.93	4.55	6.48
1,615	6.15	1.64	0.69	7.79	8.48
760	10.60	2.03	0.17	12.63	12.80


A+B
TOTAL COST
10000
7000
6000
5000
4000
1
2
3
4

6.38. Relation between spacing of embankments, hydraulic radius and roughness factor

From the above data of the Bhagmati flood control scheme, taking the channel portion and over banks portion as one composite section, the values of hydraulic radius " r " and roughness factor " n " for the different spacings of the embankments have been worked out as shown in the calculation below. Hydraulic radius is plotted against the spacing of the embankments (Fig. 6.6) and it is seen that it increases with the decrease in the spacings of the embankments. Again hydraulic radius " r " is plotted against roughness factor " n " (Fig. 6.5) here it decreases with the increase in " n ".

Conclusion :

It is seen that the above relations worked out for the composite section, that is hydraulic radius varies inversely with roughness factor and the channel width, are the same as for the normal section of the channel.

Calculations :

A_1 = X-sectional area of channel (RST)

A_2 = X-sectional area above channel portion (XRTY)

A_3 = X- sectional area of floodway (CQRX + YTLD)

A = X-sectional area of composite section (CQRSTLD)

P = wetted perimeter of composite sec. (CQRSTLD)

R = Hydraulic radius for composite section (CQRSTLD)

n = roughness factor for composite section (CQRSTLD)

Q = 101,000 cusecs.

1) <u>Spacing of embankments</u> = 15,530 ft.

Combined cross-section area = $A_1 + A_2 + 2A_3$

A_1 = 4156.8 sq. ft.

A_2 = 480 x 3.5 = 1,680 sq.ft.

A_3 = 15050 x 4.5 = 67,700 sq.ft.

$A = A_1 + A_2 + A_3$ = 73,536.8 sq.ft.

P = 482.69 + (15530 - 480) + 2 x 3.5 x 2.2

= 482.69 + 15050 + 15.4 = 15,548 ft.

$R = A/P$ = 73536.8/15,548 = 4.73 ft.

$V = Q/A$ 1,01,000/73,536.8 = 1.375 ft/sec.

$$V = 1.486/n \times R^{2/3} \times S^{1/2} = 1.375 = 1.486/n \times 4.73^{2/3} \times 0.00019^{1/2}$$

$$= \frac{1.486}{n} \times 2.82 \times 0.01378$$

$$\therefore\ n = \frac{1.486 \times 2.82 \times 0.01378}{1.375} = 0.042$$

(ii) <u>Spacing of embankments = 10,780 ft.</u>

H_1 = 4.5 H_2 = 5.5 Q = 101,000 cusecs.

A_1 = 4156.8 sq.ft. 4156.8

A_2 = 480x4.5 = 2160 sq.ft. 2160.0

A_3 = 10,300 x 5.5 = 56,500 sq.ft. 565000.00

62,816.8

A = 62,816.8

P = 480 + 10,300 + 2 x 6.5 x 2.2 = 10,809 ft.

R = A/P = 62,816.8/10,809 = 5.82 ft.

V = Q/ A = 101,000/62,816.8 = 1.61 ft./sec.

$V = 1.486/n \times 5.82^{2/3} \times 0.00019^{1/2}$

n = 1.486/1.61 x 3.24 x 0.01378 ∴ n = 0.0412

(iii) <u>Spacing of embankments = 5980 ft.</u>

H_1 = 6.5 H_2 = 7.5 Q = 101,000 cusecs.

A_1 = 4156.8 sq.ft. 4156.8

A_2 = 480 x 6.5 = 3120 3120.0

A_3 = 5500 x 7.5 = 41,200 41200.00 sq.ft.

48,476.8

say 48,500

A = 48,500

P = 480 + 5,500 + 2 x 8.5 x 2.2 = 6017 ft.

R = A/P = 48,500/6017 = 8.07

$V = Q/A = \frac{101,000}{48,500}$ = 2.08 ft/sec.

$V = 1.486/n \times 8.07^{2/3} \times 0.00019^{1/2}$

2.08 = 1.486/n x 4.03 x 0.01378

n = 1.486/2.08 x 4.03 x 0.01378 = <u>0.0397</u>

(iv) Spacing of embankments = 3,660 ft.

H_1 = 8.5 H_2 = 9.5 Q = 101,000 cusecs.

A_1	= 4156.8 sq. ft.	4157
A_2	= 480 x 8.5 = 4080	4080
A_3	= 3180 x 9.5 = 30,200	30200
		38437

$A = A_1 + A_2 + A_3 = 38{,}437$ sq.ft.

P = 480 + 3180 + 2 x 10.5 x 2.2 = 3704 ft.

R = A/P = 38.437/3,704 = 10.4 ft.

V = Q/A 101,000/38,437 = 2.63 ft/sec.

$V = 1.486/n \times 10.4^{2/3} \times 0.00019^{1/2}$

2.63 = 1.486/n x 4.77 x 0.01378

n = 1.486/2.63 x 4.77 x 0.01378 = 0.0364

(v) Spacing of embankments = 1.615 ft.

H_1 = 12.5 H_2 = 13.5 Q = 101,000 cusecs.

A_1	= 4156.8 sq.ft.	4157
A_2	= 480 x 12.5 = 6,000	6000
A_3	= 1135 x 13.5 = 15,300	153000
		25,457

$A = A_1 + A_2 + A_3 = 25{,}457$

P = 480 + 1135 + 2 x 14.5 x 2.2 = 1679 ft. say 1679

R = A/P 25.457/1679 = 15.15 ft.

$$V = Q/A = \frac{101,000}{25,457} = 3.97 \text{ ft/sec.}$$

$$V = 1.486/n \times 15.15^{2/3} \times 0.00019^{1/2}$$

$$3.97 = 1.486/n \times 6.1 \times 0.01378$$

$$n = 1.486/3.97 \times 6.1 \times 0.01378 = \underline{0.0314}$$

(vi) <u>Spacing of embankments = 760 ft.</u>

$H_1 = 16.5 \qquad H_2 = 17.5 \qquad Q = 101,000$ cusecs.

A_1	= 4156.8 sq.ft.		4157
A_2	= 480 x 16.5	= 7,920 sq.ft.	7920
A_3	= 280 x 17.5	= 4,800	4900
			16,977

$$A = A_1 + A_2 + A_3 = 16,977 \text{ sq.ft.}$$

$$P = 480 + 280 + 2 \times 18.5 \times 2.2 = 841 \text{ ft.}$$

$$R = A/P = 16977/841 = 20.2 \text{ ft.} \quad V = Q/A = \frac{101,000}{16,977}$$

$$= 5.94 \text{ ft/sec.}$$

$$V = 1.486/n \times 20.2^{2/3} \times 0.00019^{1/2}$$

$$5.94 = 1.486/n \times 7.4 \times 0.01378$$

$$n = 1.486/5.94 \times 7.4 \times 0.01378 = \underline{0.0255}$$

Table 6.2.

Q = 101,000 cusecs (Composite Section)

Spacing of embankments in feet	Hydraulic radius 'r' in feet.	Roughness Co-efficient 'n'
15,530	4.73	0.0420
10,780	5.82	0.0412
5,980	8.07	0.0397
3,660	10.40	0.0364
1,615	15.15	0.0314
760	20.20	0.0255

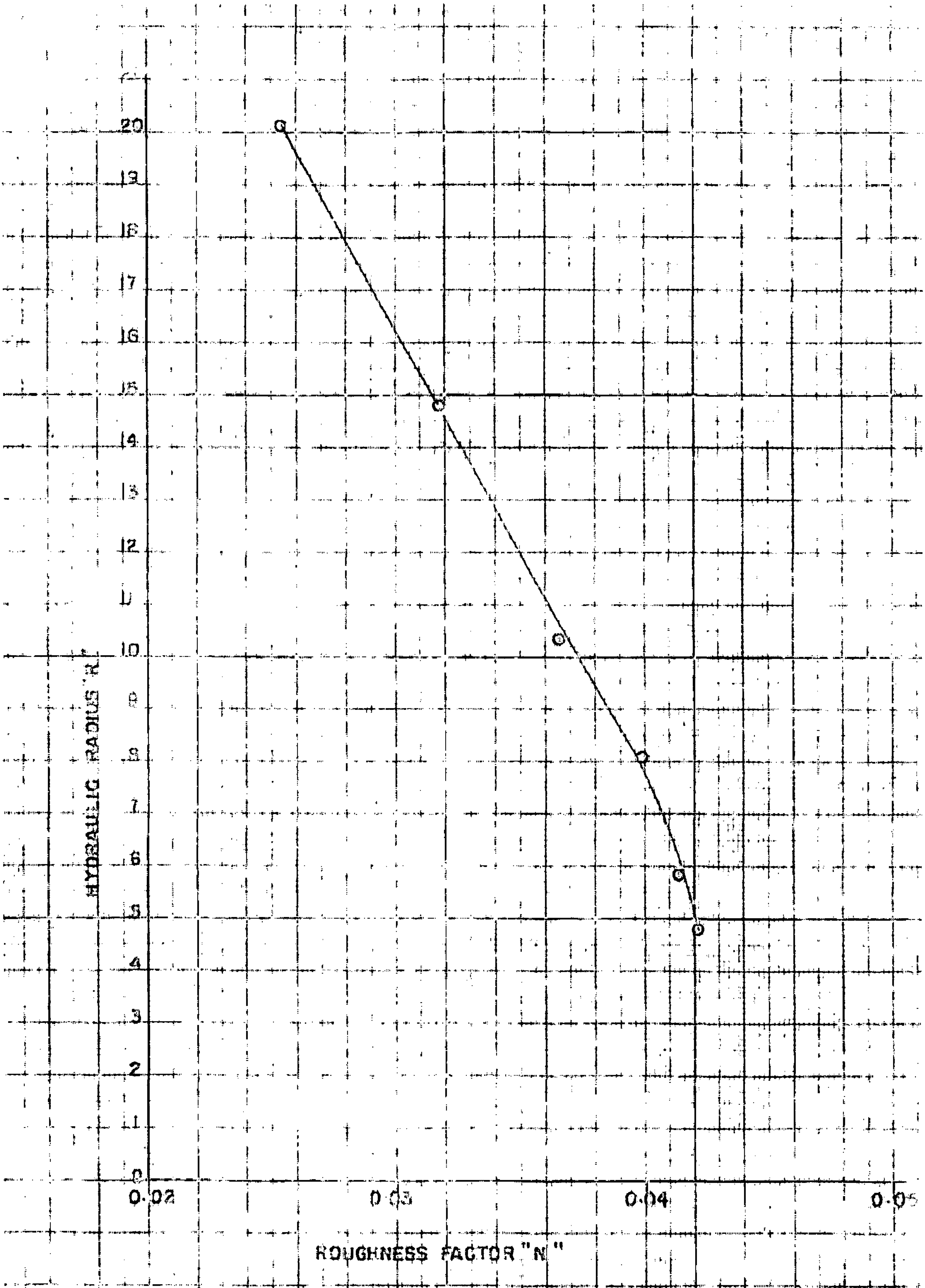

20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
HYDRAULIC RADIUS 'R'
0.02
0.03
0.04
0.05
ROUGHNESS FACTOR "N"

FIG. 6.5

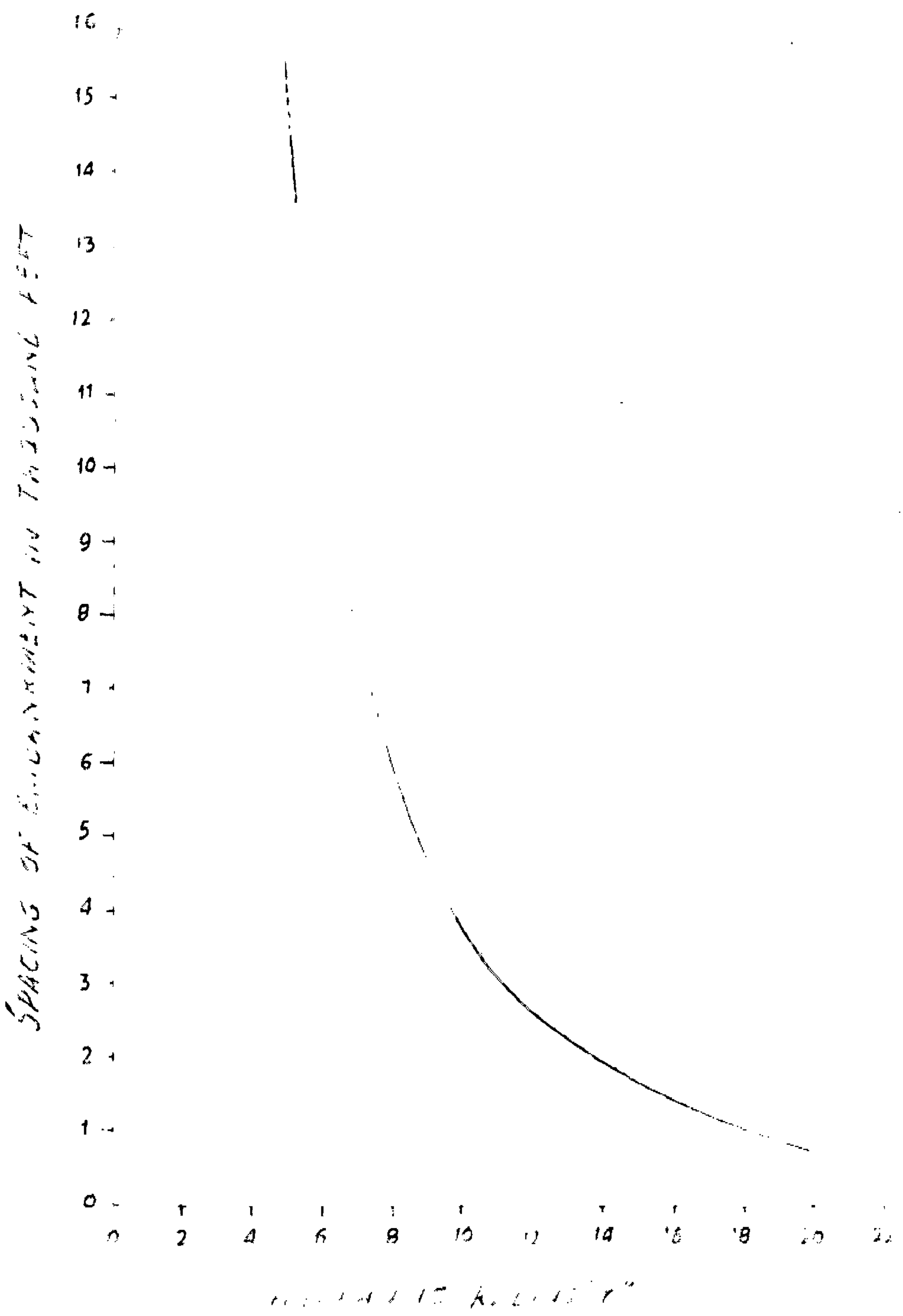


Fig. 6·6

6.4. Silt carrying capacity of river section between embankment :

Sediment deposition is a troublesome process along the rivers and streams, it raises their bed levels, thereby the flood heights are increased and inundation takes place. Also sediment load causes the river to meander and often to leave its original courses and flow along new courses. It silts up irrigation and navigation channels and making them inefficient.

The simplified forms of the following equations give sediment discharge capacity and water discharge capacity of the channel : [7]

1) $Q_s = 0.17/m^{3/4} \quad r^2 \; B.d^2 \; S^2$

2) $Q = 1.49/n \quad B, \; d^{5/3} \; S^{1/2}$

Where

Q_s = rate of sediment transport

m = diameter of sand in mm

r = specific weight

B = width of channel

d = Hydraulic mean depth.

S = slope

n = roughness factor

Q = discharge

We have to establish relation between width of the channel and the sediment discharge capacity from the above two equations.

Keeping the discharge, Q_2 same and taking the change in S negligible, then from equation (2) - $d \propto 1/B^{3/5}$ ....(3)

from equation (1) - $Q_s \propto Bd^2$ ....(4)

Sub value of d from 3 in 4

$= Q_s \propto 1/\; B^{1/5}$

This shows that by decreasing the bed width, the sediment discharge capacity is increased. The above relation is also proved by taking the illustrative example of Bhagmati Flood Control Scheme. The previous paragraphs, spacings of the embankments have been calculated for assuming the different heights of water level over the river bank. Now taking the same data, bed load and total sediment load have been calculated between the different spacings of the embankments calculated above for maximum design flood. Calculation on page 116.

Bed load transport capacity has been calculated by Einstein equation. This equation correlates two dimensionless parameters ϕ and ψ , where

$$\phi = \frac{Gs}{G.r.} \left(1/G-1 \right)^{1/2} \left(1/ gd^3 \right)^{1/2}$$

$$\psi = \frac{(G-1)d}{R'S}$$

Where g_s is the rate of bed load transport in lbs/sec.

ϕ and ψ are dimensionless parameters

G = specific gravity of the grains

r = specific weight (Unit wt. of water)

g = acceleration due to gravity

d = grain diameter

R' = Hydraulic mean radius with reference to grain roughness.

S = slope

Total load has been calculated by Laursen equation

$$\overline{C} = \left(d/D \right)^{7/6} \left(\frac{\tau_o'}{\tau_c} - 1 \right) f \left(u_*/w \right)$$

where $\overline{C}$ is the percentage concentration by weight d = grain diameter

D = depth of water

= Boundary shear

= Critical tractive stress

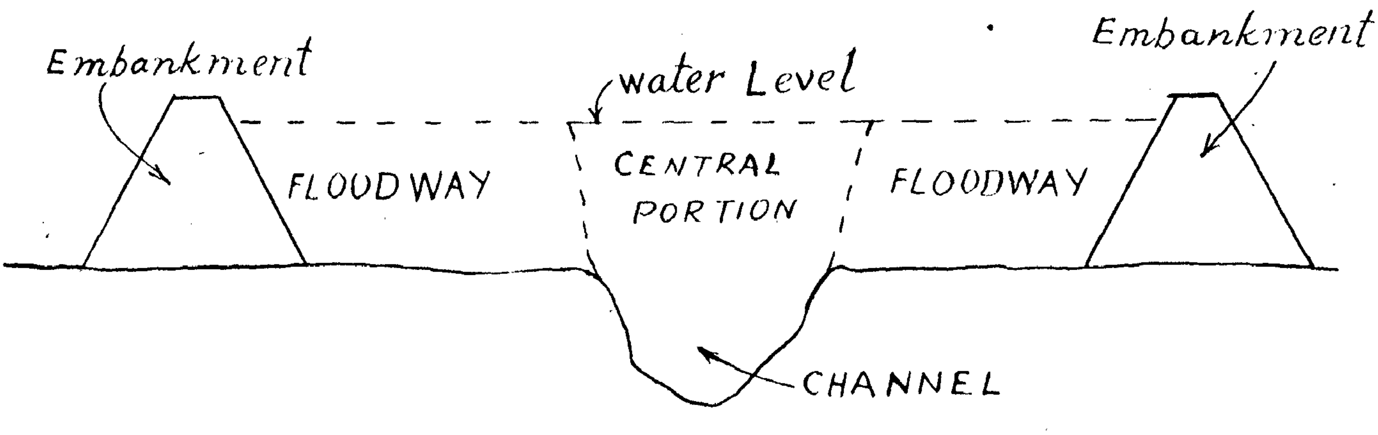
f- stands for function

u_* = shear velocity

w = fall velocity

Total load = $\lesssim \overline{C}$ P found in PPM

Total bed load (i.e. for channel and floodways) ~~and total sediment load~~ have been plotted against the spacings of the embankments (Fig. 6.8). It is seen that the relation comes the same as worked out analytically that is the total load increases with the decrease of the spacings of the embankments. Calculations for bed load and total load are given below.



5.41. Calculations :

6.41. Spacing = 15,530

(i) <u>Central Portion</u> :

$Q = 25,100$ cusecs

$A = 5846.8$ sq.ft.

$R = 12.1$ ft.

$d_{35} = 0.00059$ ft $= 0.18$ mm

$d_{65} = 0.000853$ ft $= 0.26$ mm

mean vel. = 4.3 ft/sec.

Hydraulic mean radius with reference to grain roughness, R' is given by

$$V/V_* = 7.66 \; (R'/K_s)^{1/6}$$

$$V/\sqrt{gR's} = 7.66 \; (R'/d_{65})^{1/6}$$

$$\frac{4.3}{\sqrt{32.2 x R' x 0.00019}} = 7.66 \; (R'/0.000856)^{1/6}$$

$$R'^{2/3} = 4.3 \times 0.513 = 2.21$$

$$R' = (2.21)^{3/2} = 3.28$$

$$\psi = (G-1) \; D_{35}/R'S = \frac{1.65 \times 0.00059}{3.28 \times 0.00019} = 1.56$$

from graph $\phi = 4.4$ for $\psi = 1.56$

$$\phi = gs/G.r \; (1/G-1)^{1/2} \; (1/ g d^3)^{1/2}$$

$$4.4 = \frac{G_s}{2.65 x 62.4} \times \frac{1}{(1.65)^{1/2}} \times \frac{1}{\sqrt{32.2} \quad (0.000853)^{3/2}}$$

$g_s = 4.4 \times 0.0301 = 0.1325$ lbs/ft/sec.

$Gs = 0.1325 \times 480 = 63.50$ lbs/sec.

(ii) Flood-way

Q = 75, 900 cusecs

A = 67,800 sq. ft.

R = 4.5 ft.

d_{65} = 0.000853 ft.

d_{35} = 0.00059 ft.

Mean vel. = 1.12 ft/sec.

Hydraulic mean radius with reference to grain roughness R' is given by

$$\frac{V}{\sqrt{g\ R's}} = 7.66 \ (R'/d_{65})^{1/6}$$

$$\frac{1.12}{\sqrt{32.2 \times R' \times 0.00019}} = 7.66 \ (R'/0.000853)^{1/6}$$

$$R'^{2/3} = 1.12 \times 0.513 = 0.575$$

$$R' = (0.575)^{3/2} = 0.436$$

$$\psi = (G-1)\ D_{35}/R's = \frac{1.65 \times 0.00059}{0.436 \times 0.00019} = 11.75$$

From graph ϕ = 0.050 for ψ = 11.75

$$\phi = gs/G.r.\ (1-G-1)^{1/2}\ (1/gd^3)^{1/2}$$

$$0.05 = gs/2.65 \times 62.4 \times 1/1.2845 \times 1/56745 \times \frac{25}{10^6} \quad \frac{25}{10^6}$$

g_s = 0.05 x 0.0301 = 0.001505 lbs/ft./sec.

G_s = 0.001505 x 15050 = 22.65 lb/sec.

Total G_s = 63.5 + 22.65 = 86.15 lbs/sec.

6.42. Spacings of emb. = 10,780 ft.

(i) <u>Central portion</u>

$Q = 28700$ cusecs.

$A = 6336.8$ sq.ft.

$R = 13.1$ ft.

$d_{35} = 0.00059$

$d_{65} = 0.000853$ ft.

$u_{mean} = 4.53$

$$\frac{V}{\sqrt{g\, R'\, S}} = 7.66\ (R'/d_{65})^{1/6}$$

$$\frac{4.53}{\sqrt{32.2 \times R' \times 0.00019}} = 7.66\ (R'/0.000856)^{1/6}$$

$$R'^{2/3} = 4.53 \times 0.513 = 2.33$$

$$R' = (2.33)^{3/2} = 3.55$$

$$\psi = (G-1)\ D_{50}/R's$$

$$= \frac{1.65 \times 0.00059}{3.55 \times 0.00019} = 1.445$$

from graph $\phi = 3.7$ for $\psi = 1.445$

$$\phi = g_s/ G.r.\ (1/G-1)^{1/2}\ (1/g\,d^3)^{1/2}$$

$$4.8. = g_s/2.65 \times 62.4 \times 1/1.2845 \times 1/5.6745 \times 25/10^6$$

$$g_s = 4.8 \times 0.0301 = 0.1445 \text{ lbs/ft/sec.}$$

$$G_s = 0.1445 \times 480 = 69.3 \text{ lbs/sec.}$$

(ii) Flood ways :

$Q = 72,300$ cusecs

$A = 56,600$ sq.ft.

$R = 5.5$ ft.

$d_{35} = 0.00059$

$d_{65} = 0.000853$

mean $v = 1.28$ ft/sec.

$$\frac{v}{\sqrt{g\, R' s}} = 7.66\ (R'/d_{65})^{1/6}$$

$$\frac{1.28}{\sqrt{32.2 \times R' \times 0.00019}} = 7.66\ (R'/0.000853)^{1/6}$$

$$(R')^{2/3} = 1.28 \times 0.513 = 0.657 \qquad R' = 0.657^{3/2} = 0.533$$

$$\psi = (G-1)\, D_{35}/ R's = \frac{1.65 \times 0.00059}{0.533 \times 0.00019} = 9.6$$

from graph $\phi = 0.1$ for $\psi = 9.6$

$$\phi = g_s/ G.r.\ (1/G-1)^{1/2}\ (1/gd^3)^{1/2}$$

$$0.1 = g_s/2.65 \times 62.4 \times 1/1.2845 \times 1\times10^6/5.6745\times25$$

$$g_s = 0.1 \times 0.0301 = 0.00301 \text{ lbs/ft/sec.}$$

$$G_s = 0.00301 \times 10,780 = 32.4 \text{ lbs/sec.}$$

Total $G_s = 69.3 + 32.4 = 101.7$ "

6.43 Spacings of emb. = 5980 ft.

(i) Central portion :

Q = 36,384 cusecs

P = 482.69

A = 5846.8

V = 5 ft/sec.

R = 15.1

n = 0.025

K_s = 0.000853 ft

d_{35} = 0.00059

d_{65} = 0.000853 ft.

v = 5 ft/sec.

$$\frac{V}{\sqrt{g R' s}} = 7.66 \left(R'/d_{65} \right)^{1/6}$$

$$\frac{5}{\sqrt{32.2 \times R' \times 0.00019}} = 7.66 \left(R'/0.000856 \right)^{1/6}$$

$$R'^{2/3} = 5 \times 0.513 = 2.565 \text{ ft.}$$

$$R' = (2.565)^{3/2} = 4.1$$

$$\psi = (G-1) D_{35}/ R' S$$

$$= \frac{1.65 \times 0.00059}{4.1 \times 0.00019} = 1.25$$

from graph ϕ = 5.7 for ψ = 1.25

$$\phi = g_s/G\text{-}r\text{-} \left(1/G-1 \right)^{1/2} \left(1/g\ d^3 \right)^{1/2}$$

$$5.7 = g_s/2.65 \times 62.4 \times 1/(1.65)^{1/2} \times 1/\ (32.2)^{1/2} (0.000853)^{3/2}$$

$$g_s = 5.7 \times 0.0301 = 0.1715 \text{ lbs/ft/sec.}$$

$$G_s = 0.1715 \times 480 = 82.3 \text{ lbs/sec.}$$

(ii) **Flood-ways :**

Q = 64,616 cusecs

A = 41,200 sq.ft.

R = 7.5

d_{35} = 0.00059

d_{65} = 0.000853

Mean v = 1.57 ft/sec.

$$\frac{v}{\sqrt{g\,R'\,s}} = 7.66\ (\,R'/d_{65}\,)^{1/6}$$

$$\frac{1.57}{\sqrt{32.2 \times R' \times 0.00019}} = 7.66\ (\,R\,/0.000853)^{1/6}$$

$$R'^{2/3} = 1.57 \times 0.513 = 0.805 \qquad \doteq R' = (0.805)^{3/2}$$

$$R' = 0.722$$

$$\psi = (\,G-1)\ D_{35}/\ R'\ s = \frac{1.65 \times 0.00059}{0.722 \times 0.00019} = 7.1$$

from graph ϕ = 0.255 for ψ = 7.1

$$\phi = g_s/\ G.r.\ (\ 1/G-1)^{1/2}\ \ (\ 1/g\ d^3)^{1/2}$$

$$0.255 = g_s/2.65 \times 62.4 \times 1/1.2845 \times 1/5.6745 \times (0.000853)^{3/2}$$

$$= g_s = 0.255 \times 0.0301 \text{ lbs/ft/sec.}$$

$$G_s = 0.007675 \times 5500 = 42.2 \text{ lbs/sec.}$$

Total G_s = 82.3 + 42.2 = 124.5 lbs/sec.

6.44 <u>Bed load by Einstein Bed load equation</u>

At spacing 3660 ft.

(i) <u>Central portion</u>

$Q = 45{,}500$ cusecs.

$A = 8306.8$ sq.ft.

$R = 17.25$ ft.

$d_{35} = 0.18$ mm $= 0.00059$ ft.

$d_{65} = 0.26$ mm $= 0.000853$ ft.

Mean vel. = 5.47 ft/sec.

Hydraulic mean radius with reference to grain roughness, R' is given by

$$\frac{V}{\sqrt{g R' s}} = 7.66 \left(R'/d_{65} \right)^{1/6} \quad \ldots . \; V/V_s = 7.66 (R'/K_s)^{1/6}$$

$$\frac{5.47}{\sqrt{32.2 x R' x 0.00019}} = 7.66 \left(R'/0.000856 \right)^{1/6}$$

$$R'^{2/3} = \frac{5.47}{\sqrt{32.2 x 0.00019}} \quad x \left(\frac{0.000856}{7.66} \right)^{1/6}$$

$$R'^{2/3} = \frac{5.47 \; x \; 0.307}{5.67 \; x \; 0.01378} \; x \; \frac{1}{7.66} \quad = 5.47 \; x \; 0.513 = 2.81 \text{ ft.}$$

$$R' = 2.81^{3/2} = 4.7 \text{ ft.}$$

$$\psi = (G-1) D_{35} / R' s$$

$$= \frac{1.65 \; x \; 0.00059}{4.7 \; x \; 0.00019} \quad = 1.09$$

From graph value of $\phi = 6.6$ for $\psi = 1.09$

$$\phi = g_s / G \gamma \; (1/G-1)^{1/2} \; (1/g \; d^3)^{1/2}$$

$$6.6 = g_s / 2.65 x 62.4 \; x \; 1/(1.65)^{1/2} \; x \; 1/(32.2)^{1/2} (.000853)^{3/2}$$

$$= g_s / 165.5 \; x \; 1/1.2845 \; x \; 1/5.6745 \; x \; 25/10^6 = 0.1985$$

$$= g_s \; 6.6 \; x \; 0.0301 = 0.1985 \text{ lbs/ft/sec.}$$

$$G_s = 0.1985 x 480 = 95.2$$

(ii) Flood-way

$Q = 55,500$ cusecs

$A = 30,200$ sq.ft.

$R = 9.5$ ft.

$d_{35} = 0.00059$

$d_{65} = 0.000853$

Mean V = 1.845 ft/sec.

$$\frac{V}{\sqrt{g\, R' s}} = 7.66\ (R'/d_{65})^{1/6}$$

$$\frac{1.845}{\sqrt{32.2 x R' x 0.00019}} = 7.66\ (R'/0.000853)^{1/6}$$

$$R'^{2/3} = 1.845 \times 0.513 = 0.946 \quad = R' = (0.946)^{3/2}$$

$$= R' = 0.92$$

$$\psi = (G-1)\ D_{35}/\ R's = \frac{1.65 x 0.00059}{0.92 x 0.00019} = 5.57$$

from graph $\phi = 0.47$ for $\cancel{\phi}\psi = 5.57$

$$\phi = g_s/\ G.\gamma.\ (1/G-1)^{1/2}\ (1/g\ d^3)^{1/2}$$

$$0.47 = g_s/\ 2.65 \times 62.4 \times 1/1.2,845 \times 1/5.6745 \times (0.000853)^{3/2}$$

$g_s = 0.47 \times 0.0301$ lbs/ft/sec

$G_s = 0.01415 \times 3,180 = 45.0$ lb/sec.

Total $G_s = 95.2 + 45.0 = 140.2$

6.45 Spacing of emb = 1,615 ft.

(i) Central portion

$Q = 65,600$ cusecs

$P = 482.69$

$A = 10,306.8$ sq.ft.

$R = 21.3$

$n = 0.025$

$K_s = 0.000853$ ft

$d_{35} = 0.18$ mm $= 0.00059$

$v = 6.37$ ft/sec.

$d_{50} = 0.0008$

$$\frac{V}{\sqrt{g R' s}} = 7.66 \left(R'/K_s \right)^{1/6}$$

$$\text{or } R'^{2/3} = \frac{V}{\sqrt{g s}} \times \frac{K_s^{1/6}}{7.66}$$

$$= \frac{6.37 \times 0.307}{0.01378 \times 5.67 \times 7.66}$$

$$R'^{2/3} = 3.27$$

$$R' = 5.9$$

$$\psi = (G-1)\, D/R'S = \frac{1.65 \times 0.00059}{5.9 \times 0.00019} = 0.868$$

$\phi = 8.5$ from graph for $\phi\psi = 0.868$

$$\phi = g_s / G\text{-}r\text{-} \left(1/G-1 \right)^{1/2} \left(1/ g\, d^3 \right)^{1/2}$$

$$8.5 = g_s/2.65 \times 62.4 \times 1/1.2845 \times 1/5.6745 \times (0.000853)^{3/2}$$

$$g_s = \frac{8.5 \times 2.65 \times 62.4 \times 1.2845 \times 5.6745 \times 25.0}{10^6} =$$

$$g_s = 8.5 \times 0.0301 = \quad 0.256 \text{ lb/ft/sec.}$$

$$G_s = 0.256 \times 480 = 123 \text{ lbs/sec.}$$

(ii) **Flood-way** :

$Q = 35{,}400$ cusecs

$A = 15{,}300$ sq.ft.

$R = 13.5$ ft.

$d_{35} = 0.00059$

$d_{65} = 0.000853$

Mean $V = 2.32$ ft/sec.

$$\frac{V}{\sqrt{g\,R'\,S}} = 7.66\ (R'/d_{65})^{1/6}$$

$$\frac{2.32}{\sqrt{32.2 \times R \times 0.00019}} = 7.66\ (R'/0.000853)^{1/6}$$

$$= R'^{2/3} = 2.32 \times 0.5{,}13 = 1.19 = R' = (1.19)^{3/2} = 1.298$$

$$\psi = (G-1)\, D_{35}/\, R'\, S = \frac{1.65 \times 0.00059}{1.298 \times 0.00019} = 3.95$$

from graph $\phi = 1.12$ for $\psi = 3.95$

$$\phi = g_s/\, G.r.\ (1/G-1)^{1/2}\ (1/\, g\, d^3)^{1/2}$$

$$1.12 = g_s/\ 2.65 \times 62.4 \times 1/1.2845 \times 1/5.6745 \times (0.000853)^{3/2}$$

$$g_s = 1.12 \times 0.0301 = 0.0337 \text{ lbs/ft/sec.}$$

$$G_s = 0.0337 \times 1135 = 38.2 \text{ lb/sec.}$$

Total $G_s = 123.0 + 38.2 = 161.2$ lbs/sec.

6.46 At spacing 760 ft.

(i) Central portion

$$Q = 87{,}500 \text{ cusecs}$$

$$A = 12336.8 \text{ sq.ft.}$$

$$R = 25.5$$

Mean vel. = 7.1 ft/sec.

$$d_{35} = 0.00059 \text{ ft.}$$

$$d_{65} = 0.000853 \text{ ft.}$$

$$\frac{V}{\sqrt{g\ R'S}} = 7.66\ (R'/d_{65})^{1/6}$$

$$\frac{7.1}{\sqrt{32.2 \times R' \times 0.00019}} = 7.66\ (R'/0.000856)^{1/6}$$

$$(R')^{2/3} = 7.1 \times 0.513 = 3.64$$

$$R' = (3.64)^{3/2} = 6.95 \text{ ft.}$$

$$\psi = (G-1)\ D_{50}/\ R'\ S$$

$$= \frac{1.65 \times 0.00059}{6.95 \times 0.00019} = 0.738$$

from graph value of ϕ = 10.3 for $\phi\psi$ = 0.738

$$\phi = g_s/\ G.\gamma.\ (1/\ G-1)^{1/2}\ (1/\ gd^3)^{1/2}$$

$$10.3. = g_s/2.65 \times 62.4 \times 1/(1.65)^{1/2} \times 1/(32.2)^{1/2}(0.000853)^{3/2}$$

$$g_s = 10.3 \times 0.0301 = 0.31 \quad \text{lbs/ft/sec.}$$

$$G_s = 0.31 \times 480 = 149.0 \text{ lbs/sec.}$$

(ii) **Flood-ways:**

$Q = 13{,}500$ cusecs

$A = 4{,}900$ sq.ft.

$R = 17.5$ ft.

$d_{50} = 0.0008$ ft.

$d_{65} = 0.00853$ ft.

$d_{35} = 0.00059$

V mean $= 2.76$ ft/sec.

$$\frac{V}{\sqrt{g\,R's}} = 7.66\ (R'/d_{65})^{1/6}$$

$$\frac{2.76}{\sqrt{32.2 \times R' \times 0.00019}} = 7.66\ (R/0.000853)^{1/6}$$

$$R'^{2/3} = 2.76 \times 0.513 = 1.415$$

$$R' = (1.415)^{3/2} = 1.683$$

$$\psi = (G-1)\ D_{35}/R's$$

$$= \frac{1.65 \times 0.00059}{1.683 \times 0.00019} = 3.04$$

from graph $\phi = 1.75$ for $\psi = 3.04$

$$\phi = g_s/G.\gamma.\ (1/G-1)^{1/2}\ (1/g\,d^3)^{1/2}$$

$$1.75 = g_s/2.65 \times 62.4 \times 1/1.2845 \times 1/5.6745 \times (0.000853)^{3/2}$$

$$g_s = 1.75 \times 0.0301 = 0.0527 \text{ lbs/ft/sec.}$$

$$G_s = 0.0527 \times 280 = 14.75 \text{ lbs/sec.}$$

Total $G_s = 149 + 14.75 = 163.75$ lbs/sec.

Table 6.3.

Spacing of emb. in ft.	RIVER PORTION: H.M.R. with ref. to grain roughness R'	RIVER PORTION: Mean vel. v ft/sec.	RIVER PORTION: Discharge of bed sediment Gs-lbs/sec.	FLOOD WAY: H.M.R. R'-in ft.	FLOOD WAY: Mean Vel. v ft/sec.	FLOOD WAY: Discharge of bed sediment Gs-lb/sec.	Total bed sediment discharge Gs lbs/sec.
15,530	3.28	4.30	63.50	0.436	1.12	22.65	86.15
10,780	3.55	4.53	69.30	0.533	1.28	32.40	101.70
5,980	4.10	5.00	82.30	0.722	1.57	42.20	124.50
3,660	4.70	5.47	95.20	0.92	1.845	45.00	140.20
1,615	5.90	6.37	123.00	1.298	2.32	38.20	161.20
760	6.95	7.10	149.00	1.683	2.76	14.75	163.75

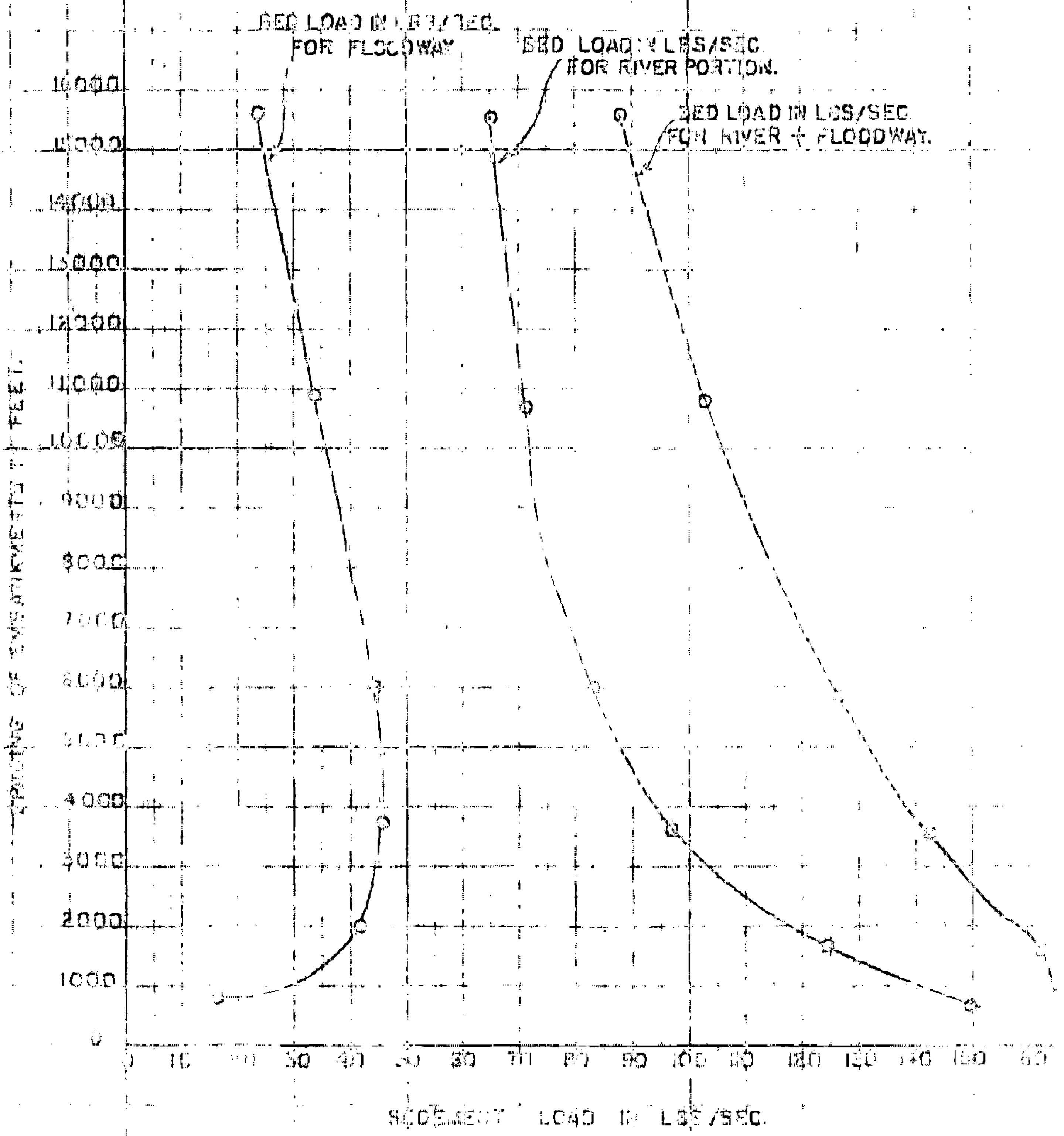


FIG. SHOWING BED LOAD IN LBS/SEC. VS. SPACING OF EMBANKMENTS IN FT.

6.5. Total load computation by Laursen Method :

$$C = (d/D)^{7/6} \left(\frac{\tau_o'}{\tau_c} - 1 \right) f (U_* /W)$$

Discharge in the river = 101,000 cusecs.

i) River portion :

Proportion and mean size :

Size	% fine	% of size	dm (mm)	% to which dm corresponds	Range
0.055 mm -0.1 mm	8	8	0.08	4	< 0.1
0.2mm	43	35	0.155	22	0.1 t 0.2
0.4 mm	88	45	0.25	63	0.2 t 0.4
upto 2.5 mm	100	12	0.52	94	> 0.4

ii) Settling (fall) velocity - W

dm (mm)	W in cm/sec	W in ft/sec.
0.08 mm	0.52	1.70×10^{-2}
0.155 mm	1.50	4.90×10^{-2}
0.25 mm	3.00	9.85×10^{-2}
0.52 mm	8.00	26.2×10^{-2}
0.244 mm	3.0	9.85×10^{-2}

iii) Shear velocity $U_* = \sqrt{g R S_f}$

River Portion

Spacing of embankments	R in ft.	$\sqrt{S_f}$	U_*	U_*/W ($U_* = \sqrt{gRS_f}$) $d_m = 0.08$	U_*/W 0.155	U_*/W 0.25	U_*/W 0.52
15,530	12.1	0.01371	0.94	55.3	19.5	9.54	3.59
10,780	13.1	0.01371	1.02	60.0	20.8	10.35	3.89
5,980	15.1	0.01371	1.175	69.2	24.0	11.95	4.48
3,660	17.25	0.01371	1.34	78.8	27.35	13.60	5.12
1,615	21.3	0.01371	1.66	97.6	33.9	16.85	6.34
760	25.5	0.01371	1.985	117.0	40.5	20.15	7.57

(iv) Computation for dm/D (in 10^{-6})

Spacings of embankments	D (in ft)	dm 0.08	0.155	0.25	0.52
15530	13.89	18.60	36.60	59.00	123.0
10780	14.89	17.65	34.10	55.00	114.5
5980	16.89	15.55	30.01	48.50	101.0
3660	18.89	13.90	26.90	43.40	90.3
1615	22.89	11.50	22.20	35.80	74.5
760	26.89	9.77	18.90	30.50	63.5

(v) Laursen function $f(U_*/W) = C / (d/D)^{7/6} \left(\frac{\tau_o'}{\tau_c} - 1 \right)$

Spacing of embankments	dm 0.08 U^*/W	0.08 $f(U^*/W)$	0.155 U_*/W	0.155 $f(U_*/w)$	0.25 U_*/W	0.25 $f(U_*/w)$	0.52 U_*/w	0.52 $f(U_*/w)$
15530	55.3	18500	19.5	4020	9.54	900	3.59	85
10,780	60.0	20,000	20.8	4670	10.35	1100	3.89	110
5980	69.2	22,000	24.0	6150	11.95	1400	4.48	160
3660	78.8	25,000	27.35	7700	13.60	1720	5.12	210
1615	97.6	29,000	33.9	12,000	16.85	2870	6.34	350
760	117.0	32,000	40.5	15000	20.15	4375	7.57	550

(vi) Critical tractive stress ($\tau_c = 4.5\ dm/304.8$)

dm	0.08	0.155	0.25	0.52
τ_c	0.118×10^{-2}	0.228×10^{-2}	0.369×10^{-2}	0.767×10^{-2}

(vii) Boundary shear $\tau_o' = \bar{V}^2/30 \; (d_m/D)^{1/3}$ (for entire silt sample $d_m = 0.224$

Spacing of embankments	$\bar{V}$ (ft/sec)	(d_m/D)	τ_o'
15,530	4.30	52.8×10^{-6}	2.315×10^{-2}
10,780	4.53	49.3×10^{-6}	2.505×10^{-2}
5,980	5.00	43.5×10^{-6}	2.93×10^{-2}
3,660	5.47	38.9×10^{-6}	3.38×10^{-2}
1,650	6.37	32.1×10^{-6}	4.29×10^{-2}
760	7.10	27.3×10^{-6}	5.34×10^{-2}

(viii) Ratio of Boundary shear to critical tractive stress $\dfrac{\tau_o'}{\tau_c}$

Spacing of embankments	d_m			
	0.08	0.155	0.25	0.52
15530	19.6	10.5	6.27	3.02
10,780	21.2	11.0	6.80	3.27
5,980	24.8	12.85	7.95	3.82
3,660	28.4	14.80	9.17	4.42
1,650	36.3	18.85	11.65	5.60
760	45.3	23.40	14.50	6.97

(ix) Mean concentration $\bar{C} = (d/D)^{7/6} \left(\frac{\gamma_s'}{\gamma_c} - 1 \right) f \; (U_*/W)$

Spacing of embankment	dm			
	0.08	0.155	0.25	0.52
15,530	1.005	0.255	0.0549	0.00472
10,780	1.15	0.285	0.06825	0.00628
5,980	1.275	0.386	0.08940	0.00993
3,660	1.47	0.498	0.11380	0.01420
1,650	1.775	0.792	0.19700	0.02450
760	2.01	1.040	0.31900	0.04170

(x) Fraction multiplied with concentration $\bar{C}$ P

Spacings of embankments	$\bar{C}$P					
	dm=0.08 P=0.08	dm=0.155 P=0.35	dm=0.25 P=0.45	dm=0.52 P=0.12	$\Sigma \bar{C}$ P	PPM
15,530	0.08050	0.08920	0.02475	0.00057	0.19502	1950
10,780	0.09200	0.09950	0.03070	0.00075	0.22295	2230
5,980	0.10200	0.13500	0.04020	0.00119	0.27839	2784
3,660	0.11750	0.17450	0.05120	0.00171	0.34491	3450
1,615	0.14200	0.27700	0.08850	0.00294	0.51044	5104
760	0.16100	0.36400	0.14350	0.00500	0.67350	6735

II. Flood way :

Proportion and mean size -

Size	% fine	% of size	dm(mm)	% to which dm corresponds	Range
0.055 mm - 0.1 mm	8	8	0.08	4	< 0.1
0.2 mm	43	35	0.155	22	0.1 to 0.2
0.4	88	45	0.25	63	0.2 to 0.4
upto 2.5	100	12	0.52	94	> 0.4

i) Settling (fall) velocity = w

dm (mm)	w in cm/sec.	w in ft/sec.
0.08	0.52	1.70×10^{-2}
0.155	1.50	4.90×10^{-2}
0.25	3.00	9.85×10^{-2}
0.52	8.00	26.2×10^{-2}
0.244	3.00	9.85×10^{-2}

ii) Shear velocity $U_* = \sqrt{g\ R\ sf}$

Spacing of embankment	'R' in ft.	$\sqrt{Sf}$	$U_* = \sqrt{gRsf}$	dm in (mm) 0.08 U_*/w	0.155 U_*/w	0.25 U_*/w	0.52 U_*/w
15,530	4.5	0.01371	0.165	9.72	3.37	1.675	0.63
10,980	5.5	0.01371	0.1825	10.75	3.72	1.855	0.697
5,980	7.5	0.01371	0.213	12.55	4.35	2.165	0.813
3,660	9.5	0.01371	0.240	14.10	4.90	2.44	0.917
1,615	13.5	0.01371	0.285	16.80	5.82	2.99	1.09
760	17.5	0.01371	0.326	19.20	6.65	3.31	1.245

Computation for dm/D (in 10^{-6})

Spacing of embankment	D in(ft)	dm 0.08	0.155	0.25	0.52
15,530	4.5	58.3	113.0	182.5	379.0
10,780	5.5	47.7	92.5	149.0	310.0
5,980	7.5	35.0	67.8	109.5	227.5
3,660	9.5	27.6	53.5	86.3	179.5
1,615	13.5	19.4	37.7	60.75	126.5
760	17.5	15.0	29.1	46.8	97.5

Laursen function f (U_o/w) = $\bar{C} / (d/D)^{7/6} \left(\frac{\tau_o'}{\tau_c} - 1 \right)$

Spacing of embankment	dm 0.08 U_o/w	0.08 $f(U_o/w)$	0.155 U_o/w	0.155 $f(U_o/w)$	0.25 U_o/w	0.25 $f(U_o/w)$	0.52 U_o/w	0.52 $f(U_o/w)$
15,530	9.72	950	3.37	80	1.675	25	0.63	12
10,780	10.75	1150	3.72	100	1.855	30	0.697	13
5,980	12.55	1500	4.35	150	2.165	35	0.813	14
3,660	14.10	1800	4.90	180	2.44	40	0.917	15
1,615	16.80	2800	5.82	280	2.99	60	1.09	16
760	19.20	4000	6.65	380	3.31	75	1.245	18

Critical tractive stress (τ_c = $\frac{4.5 \text{ dm (mm)}}{304.8}$

dm	0.08	0.155	0.25	0.52
τ_c	0.118×10^{-2}	0.228×10^{-2}	0.369×10^{-2}	0.767×10^{-2}

Boundary shear $\tau_o' = \bar{V}^2/30 \; (d_m/D)^{1/3}$ (for entire sampbe dm = 0.224)

Spacing of embankment	$\bar{V}$ (ft/sec)	(dm/D)	o
15,530	1.12	163.5×10^{-6}	0.002285
10,780	1.28	133.5×10^{-6}	0.00279
5,980	1.57	98.0×10^{-6}	0.00378
3,660	1.845	77.4×10^{-6}	0.004825
1,650	2.32	54.5×10^{-6}	0.0068
760	2.67	42.0×10^{-6}	0.00827

Ratio of Boundary shear to critical tractive stress $\dfrac{\tau_o'}{\tau_c}$

Spacing of embankmento	dm			
	0.08	0.155	0.25	0.52
15,530	1.935	1.0	0.618	0.298
10,780	2.37	1.225	0.757	0.364
5,980	3.2	1.66	1.025	0.493
3,660	4.08	2.115	1.305	0.63
1,650	5.76	2.98	1.845	0.888
760	7.02	3.63	2.24	1.08

Mean concentration $\bar{C} = (dm/D)^{7/6} \left(\frac{\gamma_o'}{\gamma_c} - 1 \right) f\ (V_*/w)$

Spacing of embankments	dm 0.08	0.155	0.25	0.52
15,530	0.0102	-	-	-
10,780	0.0142	0.000438	-	-
5,980	0.0208	0.001355	0.000021	-
3,660	0.0266	0.00225	0.00022	-
1,650	0.042	0.00382	0.000608	-
760	0.0564	0.0051	0.000818	0.000309

Fraction multiplied with concentration $\bar{C}$ P

Spacing of embankments	$\bar{C}$ P: dm=0.08 P=0.08	dm=0.155 P=0.35	dm=0.25 P=0.45	dm=0.52 P=0.12	$\Sigma \bar{C}$ P	PPM
15,530	0.000815	-	-	-	0.000815	8.15
10,780	0.001135	0.0001545	-	-	0.001289	12.89
5,980	0.001665	0.000475	0.000009	-	0.002149	21.49
3,660	0.002095	0.000787	0.000010	-	0.002892	28.92
1,65	0.003360	0.001340	0.000274	-	0.004973	49.73
760	0.004520	0.001785	0.000368	0.000004	0.006677	66.77

Spacing of embankment	PPM for river portion	PPM for flood way	PPM for whole section
15,530	1950	8.15	1958.15
10,780	2230	12.89	2242.89
5,980	2784	21.49	2805.49
3,660	3450	28.92	3478.92
1,615	5104	49.74	5153.74
760	6735	66.77	6801.77

6.6. Effects of embankments on various types of rivers :

Rivers can be classified as aggrading, degrading and poised rivers.

6.61. Aggrading Rivers :

Usually aggrading rivers ta-ke braded shape, braiding being the result of river's incapacity to transport heavy sediment which it brings from above. At the time of high floods, the river spills over the banks and the silt is deposited from silt laden waters and the level of the banks is raised. Such raising more near the river and less away from it. Due to construction of embankments on aggrading river the silt laden waters are restricted up to the line of embankments. Therefore, unless the silt gets pushed down into the rivers, it has to deposit in the area within the embankments. This may lead to emphasize on aggradation. As such the construction of embankments on a braided river the process of the formation of a more defined channel, takes a fairly long time.

6.62. Degrading rivers :

Degrading rivers seldom require the construction of embankments for preventing of flooding of areas, as they flow in deep channels and do not spill normally. Along the degrading rivers, some of the reaches, having low ground levels do get flooded in a very high floods. If embankments are constructed for such reaches, they will not have the adverse effect on the river channel, which will continue to show the degrading tendency. In course of time as degradation continues, lower flood heights will obtain and increased free board would be available at the embankments. Where long reaches of a degrading river are embanked, increased flood levels resulting from confining flood waters within the embankments must obtain, although the river bed itself would not rise.

6.63. Poised rivers :

One poised reaches of rivers, which are able to transport, by and large, whatever sediment comes in from above, construction of embankments does not have any adverse effect on river bed in such reaches. Within the embankments the confined flood flow tends to deepen and widen the river, to enable it to carry what previously flowed over bigger reach. When embankments are constructed over long reaches on these rivers, initially the flood levels would rise, but in a course of time would tend to fall to pre-embanked flood levels.

6.64. The effect of confining the flood water of the river between embankments is: [1]

1) to increase the rate at which the flood wave travels down the stream.

2) to increase the water surface elevation of the river at flood.

3) to increase the maximum discharge at all the points downstream.

4) to increase the velocity and the scouring action through the embanked sections, and

5) to reduce the surface slope of the stream above the embanked portions.

When flood rise occur in the river, water level rises to the high flood level. Obviously, the water is now occupying a much larger area of the river cross-section. To fill up the valley cross-section from low water level to the high flood level a certain volume of water is required and this volume is held back in the valley from the flood waters coming from above, the balance only passing downstream. This volume of water which is held up by the valley is called ' valley storage '. It follows that as a flood progresses downstream during the rising stage it has to leave apart of its water behind, in order to augment the river cross-section. Correspondingly the peak rate of discharge as the flood progresses downstream is reduced.

The first three points are now self evident. As the marginal embankments are constructed, they restrict the valley storage to a greater or smaller extent in accordance with their spacing from the main river. The reduction in valley storage would lead to quicker passage of the flood crest, higher peak discharge rates and higher flood stages.

At the low velocities prevailing, the bed material of the river itself remain inert and takes no part in the flow. As the surface run off of a flood rise starts coming in, it brings with it a load of sediment resulting from the erosion of the drainage basin. Upto a certain stage of the flood, the river is unable to transport all the sediment coming in and there is deposition of silt in the bed. As the discharge and velocities keep rising there will be a momentary stage of equilibrium above which the scour of the cross-section start as the tractive force exceeds that needed to carry forward the sediment load. As the flood recedes the tractive force will fall below the value to convey the silt cargo after which again there will be deposition of silt in the bed. Thus the total effect of the passage of the flood on the river cross-section will be the net result of silting in the rising and falling stages when the carrying capacity is below the load and scouring at and near the high stage when it is more than the load. In case of low floods, in which lower stages predominate, the silting effect may be more important whereas the higher floods may have the opposite tendency. The over all tendency of any reach of the river in its natural conditions will, therefore, be governed by the intensity, duration and frequency of the floods besides the grade and concentration of sediment charge brought in and the other relevant factors.

Spacing of the marginal embankments is also important. If the embankments are placed far apart, the restriction of the valley storage and the rise in the high flood levels is small, the increase in the relative velocities is also relatively small but on the other hand a part of the flood plain is still available for deposition of silt. The reverse is the case for closely spaced embankments.

It is thus seen that no general statement applicable to the all cases is possible with regard to the effect of marginal embankments on bed levels. It will depend on the factors affecting the natural tendencies of the river mentioned above plus the spacing of the embankments relative to the main channel and apart from each other.

The fifth point also follows because a rise of flood levels within the embanked reach would lead to formation of back water and flattering of slope behind the reach.

From the above paragraph " The effects of confining the flood waters of a river between embankment ", it is seen that the influence of longitudinal embankments is, thus, set out to be one of intensification of the tendency to scour; but the net effect on the bed here again has to be evaluated on the basis how far this tendency is offset by the opposite tendency of silting during the periods of velocities lower than the critical.

The overall effect of embankments on a river bed can, in general, be said to be to minimise the tendency of silt, which, in its turn, is dependent upon a number of factors such as the nature and quantity of bed silt, the velocities attained etc. In case the natural tendency to silt is rather strong, the effect of embankments may not be able to stop altogether the rising of bed that is going on.

Thus, in a scientific assessment of the effect of the embankments on the river bed, factors such as distance between embankments on either side of the natural banks, the critical velocity, the nature of the bed silt, the intensity, the duration of floods, all become important factors which have to be taken into consideration. Without considering all these factors, it is not advisable to attempt to draw a conclusion, in general, either that the embankments tend to scouring of the bed or that they lead to a rising of a bed or even that they have simply no influence. It is, therefore, highly desirable to institute detailed studies on rivers of different characteristics with a view to collect the above information to serve as a basis for a scientific investigation of the problem.

The following is a brief account of the observations made on some of the rivers in the past by engineers in connection with the same problem. [14]

i) River Mississipi in the United States :

Cross-sections of the river in certain reaches taken by the Mississipi Commission, in 1881, in 1894 and again in later years. On the comparison of these cross-sections, Ockerson in 1914 found that there was a marked tendency towards a greater cross-section involving deepening of the bed.

ii) The Yellow River in China :

According to study of S.C. Chang, the bed of this river has been estimated to be rising at the rate of 1 to 1.3 metres a century as compared with the surrounding country.

iii) Indus river (Pakistan)

The Indus in Pakistan has been completely embanked from northern border of Sind to the sea, except where the ground is high. The construction of embankments on the Indus has accelerated the aggrading tendency of the river although it must be recognised that some aggradation would have resulted even without embankments.

iv) Mahanadi :

Professor Mahalanobis from a study of the variations of levels from 1868 to 1928 at Naraj, concluded that the average variation in level is 0.304 mm (0.001 ft) a year.

6.55. Conclusion :

From the above examples it can be seen that in some cases there was scouring of the bed while in some there was a rise in bed level as a result of embankments.

All this goes only to give support to that no single generalization on this question could be universally true.

6.7. Limitations:

Embankments are justified and should be used in any valley where the interest on their first cost and the cost of annual maintenance is less than the net annual increase on the returns from the land which is protected by them.

Embankments are generally cheapest, the quickest and the most immediately effective method of flood control, there are certain inherent difficulties that follow their construction. Inspite of the greatest care during execution, earthen embankments are fragile, liable to be breached by over topping, river erosion, in filtration, leaks and cracks due to shrinkage of soil. These, therefore, need constant and some times expensive maintenance arrangements during the flood season. The breaches tend to become the subject of painfull enquiries, the outcome of which, in most cases, is no discovery, but a reiteration of one or more of the causes listed above.

Protection from embankments is rigidly limited in the sense that in case a particular flood exceeds the designed flood, a dangerous situation can result. This limitation places a heavy burden on the engineer to explain clearly the degree of protection afforded by the embankments. In agricultural tracts, embankments are likely to cut-off fertilising silt and inspite of sluices that may be provided, silt does not travel far enough to the fields. There is likely to be an increase in flood heights upstream, greater peak flows downstream and drainage congestion on the country side of embankments.

Although these disadvantages some times create difference in the public mind, these are inherent to embankments to which occasionally there is no alternative, like in the case of Assam, North Bihar, North Bengal and deltaic area of Central and Deccan rivers.

6.8. Example :

During the last 18 years more than 7,883 km (4,972 miles) of embankments have been constructed to benefit an area of 60 lakh ha (148 lakh acres). The experience of functioning of these embankments, which had been constructed, based on the meagre data, were found to be inadequate to withstand, the higher floods which occurred subsequently. They are being raised and strengthened taking into account the further data. By and large the embankments have given considerable benefit. This can be illustrated from the Kosi embankment scheme. The Kosi, which originates in the Himalayas, was called the " River of Sorrow " on account of the flood havoc it used to cause frequently. In addition, it had a tendency to shift its course. During the period of 130 years, the river had shifted west-ward by about 112 km (70 miles) rendering a vast tract of fertile lands unfit for cultivation by the deposition of course silt. The Kosi embankments, which have been constructed to confine the river, after their completion in 1957, have protected 2 lakh ha (5 lakh acres) from the annual innundation and have also arrested the tendency of the river to more west-ward. The analysis of the data, after the construction of embankments has not shown any appreciable rise in the bed level. The embankments are, of course, frequently subject to attack of the main current at different points but these have been effectively checked by taking up river training works as necessary.

Chapter - 7

CHANNEL IMPROVEMENTS AND RIVER TRAINING WORKS

7,1, River training for preventing bank erosion :

Erosion or scour of the banks occurs due to the instability of the river or disturbances in its regime conditions. These are attributed to the variations in discharge, amount of sediment, slope of the river and composition of bed and bank material. Conditions in a river are constantly changing, and therefore, equilibrium conditions are seldom attained. Depending upon the river characteristics, bank failure takes place, to a lesser or a greater degree, on almost all rivers at one place or the other. But the situation attracts attention only when considerable erosion or loss of land occurs, especially when this endangers important areas, lines of communications and affects large population.

7.2. Causes of bank erosion :

River bank erosion is caused either on account of the bank material not being sufficiently strong or when the river currents are too fast. Strength of some type of soils is very weak and with saturation they are known to cause sloughing of river bank. Bank erosion on the Barak and some other rivers in Assam is of this type. Banks of some of the rivers are composed of relatively stiffer material when dry. The strata, if it is porous, permits easy percolation of the drainage water from the adjoining lands. On the contrary, if the material is somewhat impremeable, the drainage water, could make the river banks composed of such soils, unstable on account of back pressure. A typical example of which is the Pohru, a tributary of the Jhelum in Kashmir.

The river bank erosion also depends on the severity of the river attack. At the foot hills in sub-mountain regions, rivers have braided pattern, i.e. having several inter-twining channels. Bank erosion in such channels is caused on account of oblique attack during floods. Bank erosion on the Kosi river in North Bihar and in Brahmaputra river in Assam are of this type. In the plains, the rivers normally forms a meandering course. These meanders are generally not stationary but have a tendency to move progressively in the downward direction. In such meandering rivers, erosion occurs on the outside of the beds on account of secondary helicoidal flow which removes the material from the eroding bank on the shoal on the opposite side. The bank erosion on Ganga river in Mansi bend is an example of this type of erosion.

Bank erosion could also be caused on account of natural and artificial changes taking place in the river regime when structural like dams, barrages etc. are constructed which arrest sediment movement on the river bed and consequently the river reaches downstream get affected.

7.3. <u>Indirect protection work :</u>

<u>Function:</u>

Indirect protection works are constructed in front of the bank with the object of reducing the erosive force of the current either by deflecting it away from the bank or by inducing silt deposition in front of it. These works include spurs at adequate spacing.

7.31. Types of Spurs :

The type of the work can be further divided according to the durability of the works.

i) Temporary and semi-permanent measures :

Temporary works check erosion at the time of emergency while semi-permanent works will last and will be effective for comparatively short time. These include the construction of bamboo spurs and tree spurs.

ii) Permanent works :

These include construction of permeable spurs made with wooden piles, semi-permeable boulder spurs which are essentially permeable spurs, but strengthened by dumping boulders on both upstream and downstream and impermeable spurs which consist of rockfill or earth core armoured with resistant material like stones, fascine mattress or sausages filled with stones.

Factors which influence the choice and design of spurs are :

i) Fall and velocity of river

ii) Character of bed material carried, such as shingle boulders, sound or silt.

iii) Width of water way at high water, mean water and low water,

iv) Depth of water-way, height of flood rise and nature of flood hydrograph; and

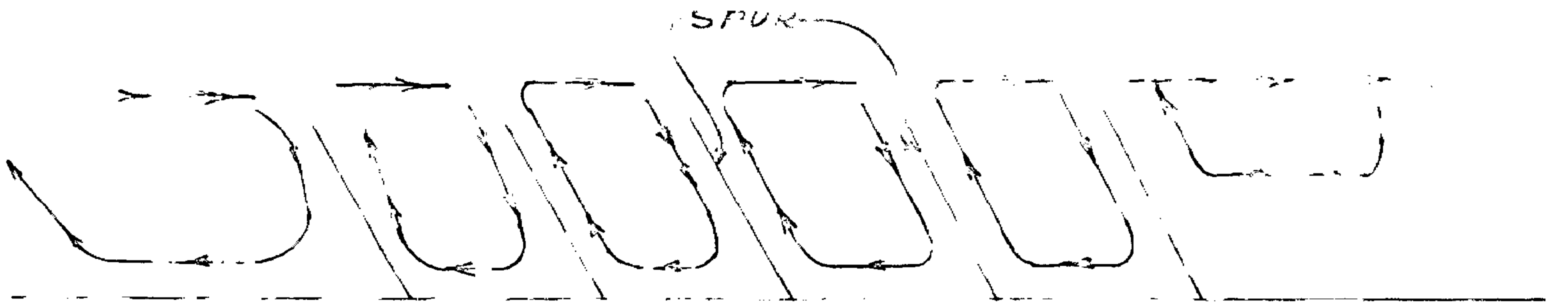
v) Available materials and funds

Spurs for confining a river to a defined channel, especially during high water and low water are usually made impermeable. Permeable spurs are suitable for silt-laden rivers.

Depending upon the purpose spur can be used singly or in series.

Spurs may be aligned either perpendicular to the bank line or at an angle pointing upstream or downstream. A spur pointing upstream has the property of repelling the river flow away from it and hence it is termed as repelling spur (Fig.7.1). A spur pointing downstream attracts the river flow towards it and is called attracting spur. When a spur aligned perpendicular to flow, changes only the direction of flow without repelling it, it is known as a deflecting spur.

7.311. Repelling spurs are usually successful in achieving desired results if they are properly located with due regard to their position in relation to the meander length. It is desirable to test them in hydraulic models before adopting them in practice. The angle of deflection upstream varies from 60^{o} to 80^{o} with the bank. Generally, the head of repelling spur causes disturbance in the flow at its nose and heavy scour accurs downstream due to eddy formation. These spurs should, therefore, have a strong head to resist the direct



SHOWING FLOW LINES BETWEEN SPURS IN SERIES WITH UNIFORM SPACING

FIG. 7.2

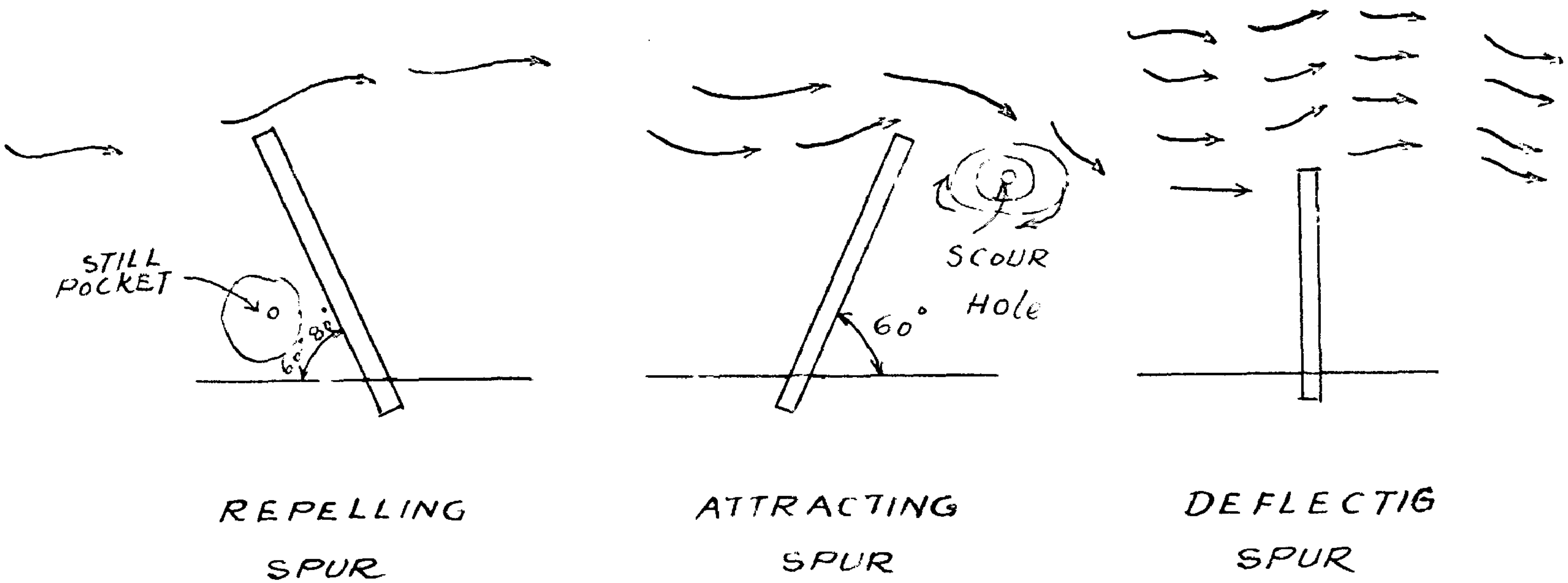


FIG. 7.1

attack of a swirling current. The scour diminishes from the head towards the bank and protection of the slope and apron can be reduced accordingly.

In the case of a repelling spur, a still water pocket is formed upstream of it, and suspended load brought down by the river gets deposited in this pocket. The action of a series of repelling spurs is shown in figure 7.2.

Flow spreads to the side below the hose of the spur, and after flowing through the pocket formed by two successive spurs, it impinges on the lower spur and is turned towards the shore. This back roller action is responsible for sedimentation in the pocket.

7.312. Attracting spur, in this case the angle is usually 60°, within a range of 30° to 60°. An attracting spur bears the full fury of the frontal attack of the river on its upstream face and has therefore, to be adequately armoured; and equally, heavy protection is not necessary on the downstream slope.

7.313. Spacing

Regarding the spacing between adjacent spurs, the general practice is to make it in a certain proportion of their lengths, varying it with the width of the river. Spurs are usually spaced further apart (with respect to their lengths) in a wide river than in a narrow one if the discharges of the two, are more or less same. Larger spacing can be planned for convex banks, and smaller for concave banks, with spacing at crossings in between the two. A spacing of 2 to 2.5 times the length of spurs is the general practice.

7.314. Length :

The length of the spurs depends upon the distance between the original river bank line and the designed normal line of the trained channel. Two long a spur is not desirable on rivers with easily erodible material (as compared with the strength of the flood current). In this case the best way is to start with a shorter length and to extend the spurs after space between them has been silted up. Shorter and temporary spurs constructed in between the long ones may be necessary to induce silt deposition.

7.315. Section :

With their lengths and spacing fixed, other dimensions of spurs will depend on the type of construction, material and the extent of protection required. Any material that would resist both the water and the current might be, and has been, used for construction of spurs. There are two main types of spurs, the solid type and the permeable type, and these are two heights of protection, the high water and low water. Dimensions of spurs should be designed according to (a) the material used; for instance, an earth spur can not be over topped and must be safely above the highest possible water level; (b) the side slopes depend on the angle of repose that the material may assume when dumped or placed in the river; for instance, rock could have steeper side slopes than earth. Also where the attacking force is greatest, the section should be strongest, for instance the head of spur.

7.32. Apron :

The face of the impermeable spur is protected at the river bed level by stone pitching, so that during floods, the slope is not damaged. Scour, however, would occur at the toe with consequent undermining and collapse of the stone pitching. To obviate such damage to the slopes, a stone cover, known as apron, is laid beyond the toe on the horizontal river bed, so that the scour undermines the apron first, starting at its farthest end and works backward towards the slope. The apron then launches to cover the face of the scour with stone, forming a continuous carpet below the permanent slopes of the spurs. Adequate quantity of stone for the apron has to be provided to ensure complete protection of the whole of the scoured face. This quantity will obviously depend on the apron thickness, depth of the scour and the slope of the launched apron.

7.321. Design :

Generally the method adopted for finding out the length of the apron is by determining the scour depth. The formula accepted and in use so far is that of Lacey's scour formula, which is

$$R = 0.472\ (Q/f)^{1/3}$$

Where

R is normal scour depth below HFL in ft.

Q = discharge in cusecs.

Depth of scour is taken as

$$D = x\,R$$

Where x = 1 at the shank

and = 1.5 at nose

f = Laccy's silt factor depends on sides and nature of bed and equal to $1.76 \sqrt{dm}$ where dm is the mean size of bed material in mm.

7.322. Section :

Thickness of launching apron as suggested by Spring is 1.25 times the thickness of stone pitching, and for rivers liable to sudden deep scour, it may be increased to 1.5 times say t'. Slope of the launching apron suggested by Spring and Gales is 2 : 1.

Length of the apron when launched will be = $\sqrt{5}$ D volume of stone per foot width = $\sqrt{5}$ x D x t where t is the thickness of apron after launching.

$$\therefore \text{Length of apron} = \frac{\sqrt{5} \times D \times t}{t'}$$

7.323. Illustrative example :

Flood protection works for village Khajnawar on Chhacha Rao, hilly torrent, district Saharanpur, U.P. Village is situated on the left bank of stream Chhacha Rao, which is on the upstream side of the newly constructed bridge on Kalsai-Fatehpur road. There is a very high land on the upstream side of the Rao on which the village is situated. Stream Rao is eroding its left bank and causing danger to the safety of the village. There is an urgent demand from the villagers to provide some protective measures for their village.

Data :

$Q = 15{,}000$ cuseco

Depth of river during high floods = 7 '

Laccy's silt factor $F = 1$ (for course sandy soil)

$$R = 0.472\ (Q/f)^{1/3}$$
$$= 0.472\ (15000/1)^{1/3}$$
$$= 0.472 \times 24.9 = 11.8 \text{ ft.}$$

Taking deepest scour depth at nose = 1.5 R and at u/s shank = 1 R

R at nose = 1.5 x 11.8 = 17.7 ft. Say 18' below HFL

R at u/s shank = 1 x 11.8 = 11.8 ft. say 12' "

Depth of scour at nose D = 18-7 = 11 ft. below bed

Depth of scour at u/s shank D = 12-7 = 5 ft. below bed

i) Thickness of the Launching Apron :

Taking the thickness of the launching apron as 2.5 ft. at 2 : 1 slope.

Quantity of apron material required per ft.

Length at nose $= \sqrt{5} \times D \times T$
$= 2.23 \times 11 \times 2.5$
$= 61.5$ cuft.

Quantity of apron material required per ft.

length at upstream shank $= \sqrt{5} \times D \times T$
$= 2.23 \times 5 \times 2.5$
$= 27.88$, Say 28 cuft.

ii) Width and thickness of apron :

Taking thickness of apron 3' at nose

width of apron at nose = 61.5/3 = 20.5 ft.

Adopted 25 ft.

Width of apron at u/s shank = 28/3

= 9.3 ft. say 10 ft.

Kept the same.

7.4. Direct bank protection works :

Bank protection is provided for any of the following purposes :

1. Flood embankments are given bank protection due to erosion otherwise huge losses will take place in the protected areas on accidental breaching.

2. Upstream and downstream banks of weir, barrage and bridge are alsoprotected from erosion due to heavy currents.

3. In navigable rivers and canals, due to the movement of vessels, waves are formed which cause bank caving and protection is provided for the same.

4. Bank protection is given to save the agricultural lands and valuable properties in the thickly populated areas from river erosion.

5. Due to bank erosion, the condition of the river gets deterioated and hence bank protection forms the part of river training works.

7.41. Causes of erosion :

1. On the side slope of stream the resistance to motion of the soil particles is further reduced by the sliding force of the particles due to their weight and hence on slope they get washed away easily by strong current.

2. At the time of floods, banks get partially or fully saturated. Shearing resistance for silt or silty sand may be as low as 50 percent of its original value before saturation. If the angle of sloping surface is steeper than the reduced angle of shearing resistance, sloughing may result.

7.42. Types of Works :

Direct bank protection works include vegetal cover, pavement, revetment, grading of slope etc.

If the current is not strong, banks can be well protected by a vegetal cover, either by turfing or by sodding or by other natural growth. Low growth of shurbs and willows is by for the most effective cover but they require a long period to grow. In such cases, a temporary cover with mattresses of woven willow brushes is often necessary for the initial period of one or two years. After the natural growth has taken place, it just needs cutting every year to keep the brush-wood from growing into tall trees. Temporary covering by brush-wood or lumber mattress, weighted down by stones, may be used for emergency purposes.

When the current is very strong, protection has to be provided by stone revetments or various types of mattresses such as willow, lumber, asphalt or articulated concrete.

7.43. Revetment :

Banks are first cleared from trees, brush wood, grass etc. Cleared banks are then properly graded such that the slope is flatter or at least equal to the angle of repose of the soil under water. Drainage behind the pavement may be necessary to prevent the saturation of the banks. Graded stones, i.e. filter should be provided on the back side of the revetment to prevent soil particles being sucked out due to high velocity flow.

7.44. Thickness of rip-rap :

Thickness of rip-rap is governed by the velocity of the current near the bank. Table given below is recommended by Spring[7] for determining the thickness of stone pitching for guide banks, the same can also be used for finding the thickness for revetment. Where stones are dumped under water the thickness shouldbe increased by 50 per cent.

River bed material as classified by Spring	Thickness in inches for river slopes in inches per mile				
	3	9	12	18	24
1	2	3	4	5	6
Very course	16	19	22	25	28
Course	22	25	28	31	34
Medium	28	31	34	37	40
Fine	34	37	40	43	46
very fine	40	43	46	49	52

Pitching with boulders or concrete blocks gives the bank a certain rigidity which induces scour at its toe. Therefore, the pitched bank must be protected at its toe. For toe protection different method are used in practice. The first is a flexible stone apron, designed after taking into account the expected scour at the toe of revetment. The apron launches as the scour developes. Generally the maximum anticipated depth of scour is assumed as 1.5 to 2 D. Method of designing apron is discussed in previous paragraphs under indirect protection works.

7.5. Channel improvement :

Flood protection by way of channel improvement can be done by increasing the x-sectional area of channel or velocity so that more discharge will pass in the first case and in second case it will pass quickly. Hence both will serve in reducing flood height and its duration.

7.51. Increasing the channel x-section area :

If a stream is narrow and shallow, its catchment small, the depth of flooding small, and the period of overflow short, it may be practicable to deepen and widen the channel to a limited extent and to use the excavated material to raise the height of the banks. But for the streams of large catchment area of several thousand square miles, it is not economical to solve the flood problem in this way, since the volume of flood water is so large that it is not feasible to pass the flood water within its banks.

7.52 Increase in velocity :

Velocity of flow depends on three factors (i) the roughness of channel, (ii) the hydraulic radius (iii) the slope of channel.

7.53. Roughness of channel :

This include the obstacles which retard the flow. Any obstackle such as floating trees and bushes backs up the water and reduces the slope and velocity. This happens in the case of small streams but for large streams the gain is negligible. Lining the channel thereby improving roughness co-efficient and thus enabling river to pass greater discharge for the same water level. This is very costly and has not been employed for flood control.

7.54. Hydraulic radius :

This can be increased by increasing the bed width of depth of the channel. It is seen that for same enlargement of x-section area, deepening of the channel has more effect on velocity than widening. Since the outlet of a stream can not be lowered in most instances, the deepening of the channel in upstream portion would decrease the slope of the stream, as such there will be more loss in velocity due to flatter slope than gain in velocity due to increase in hydraulic radius. If however the streams can be shortended by construction of cut-offs, the channel can be deepened as well as slope is increased/at the same time.

Hydraulic radius can also be increased by raising the banks i.e. by construction of embankments instead of lowering the bed by dredging. Cheapest method of construction of embankments is by dredging the material from the bed and in this both the methods of increasing hydraulic radius are considered as one method.

7.55. Slope of Channel :

Velocity of watōr varies as square root of slope of channel. Ifwe have to double the velocity in that case we have to increase the slope by four times. The total fall between two pointsof the stream is constant and the only way to increase the slope or fall per unit length of stream is by reducing its length. The shortening is accomplished by the construction of cut-offs across the narrow necks of land made by sharp loops in the channel.

7.6. Cut-off :

When the meander of the river reaches extreme condition and develops into narrow necks, it is likely that the river may cut across the neck at high flood and create a straight channel for itself.

7.61. Natural cut-offs :

A river flowing along the curved paths, has a shallow side channels beside, which are caused by the floods or they are the remnants of an old course. During high stages these side channels get deepened and growth of bars develops at the curve portions which stress the tendency of water to take to side channels. With the result that main channel continues to silt because of extended bars and cut-off is completely formed.

7.62. Artificial cut-offs :

The cut-offs may be utilised to divert the river from a curved flow which may be endangering valuable land or property or for any other river training purpose. The artificial cut-off is likely to develop when the ratio of the length of the bend to that of the chord is between 1.7 to 3.0. For the design of an artificial cut-off, the alignment should be such that at both ends, the cut is tangential to the main direction of flow. Cut should be preferably aligned on a slight curve to promote speedy development. Upstream end of the cut-off should be made bell-mouthed. According to the require formula of Laccy RS^2 depends wholly upon grade of transported material. (R is hydraulic radius and S is slope). Assuming that the existing channel is reasonably stable at a dominant discharge and the cut will flow through the same grade of material, the cut should be excavated for a section for which RS^2 will be atleast as high as of river section. Since the slope is inverely proportional tothe length of channel as said in previous paragraphs, a workable relation is obtained for the pilot cut to be self-scouring, provided that the expression R/L^2 is greater for the pilot cut than for the river section. (L being the length of a reach). This, however, ignores losses by shocks at curves, the reduction of the discharge down the river course by the amount of flow in the cut, and the fact that material in the cut is usually more firmly compacted than in the river.

In practice, therefore, for R/L^2 to be greater for cut than the main course, pilot cuts are designed to a rather deeper section.

7.7. River Training for Navigation :

River training for maintaining a safe and good navigable channel known as " low water training " or " training for depth " is being done by concentrating the flow in the desired channel and closing other channels. ' Bandelling (has been used for this purpose extensively on the Ganga and the Brahmaputra. Bandals are designed to concentrate the low water flow in a single channel for maintaining required navigation depths. These works are taken up after the flood season as soon as the water levels begin to lower. They check the flow and cause sand to be deposited parallel to and behind the bandals., thus the channel confined between the bandals is formed with sand banks on either side, and the whole discharge of river is directed through this channel. Deeping of the channel is achieved in two or three weeks after the construction of the bandals. The depth thus achieved subsists until the next flood period.

Bottom panelling which has been developed in Chatean laboratories in France has been usedon an experimental basis for low water training in the Brahmputra. This consists of panels of corrugated sheets or any other material resting on river bed and held in position by vertical piles driven 3 to 4.6 m (10 to 15 ft.) below bed level at 1 m (3 ft.) intervals. This has the advantage over the bandalling in that it can stand higher floods and also last longer. Further, the building up of sand banks can be continued by adding more pannels. This has been tried at Hathimura in 1964 and Neamati in 1965.

7.8. Limitations :

A river can be made to carry its discharge at lower levels or within its high banks by improving its hydraulic conditions, but such improvement are generally very costly and can be feasible only in short reaches in the vicinity of towns where other methods of protection can not be resorted to. Such a local improvement may not always be effective, may require expensive maintenance. Although channel improvements can help in reducing flood stage at the point of improvement, but they increase flood heights at down stream points and are likely to increase the damage there.

7.9. Examples :

Examples of the important river training works are the Dibrugarh protection works in Assam where a combination of stone spurs, timber spurs, permeable spurs and revetment has been adopted. These works were undertaken after the floods of 1954 when the Dibrugarh town has almost under a threat of extinction by severe erosion of the river Brahmaputra. The works have withstood the subsequent onslaughts of the river and have provided very good protection to the town. River training by revetment and spurs has also been successfully adopted for the protection of Gauhati and Goalpara towns in Assam. Another notable work which has been carried out recently is the construction of stone spurs on the right bank of the Great Gandak river in Uttar Pradesh. This has not helped in checking the west-ward movoment the river but also diverted the main stream away from the bank thereby removing the threat to the safety of the Western Gandak Canal. The experience of the river training works undertak so far indicates that the temporary measures can only give some immediate protection from the active erosion and can not held on for long. Stone spurs and revetment, which are measures of permar nature, have been generally found satisfactory and have given good results.

Measures such as concrete mattress and timber mattress extensively used on Mississippi in the U.S.A. have not been tried in India so far due to their high cost as well as the heavy investment on plant and machinery that is required for laying them.

Bottom pavelling has also been tried as anti-erosion measure on Brahmaptura, when the bank is subjected to severe erosion but the efficacy of this method is yet to be established. However, it is considered that if the work is planned properly, after study of the changes in the river course for a number of years and if done in addition to other training measures, it can be effective.

Dredging has not been tried so far as an anti-erosion measure. However, it has been considered that it may be advantage ly done to provide a suitable leading channel to divert the main stream away from the vulnerable bank. Therefore, on an experiment basis, two dredgers are being procured to work at selected places along the Brahmaputra where erosion is a serious problem.

Chapter - 8

SOIL CONSERVATION

8.1. Introduction :

It has been observed that the vegetal cover has effect on floods. Vegetal cover removes moisture from the soil by transpiration and that it also promotes loose organic soil, which is favourable for the infiltration of rainfall. A heavy vegetal cover means a high interception loss during storms. Therefore less flood run-off is expected from well-vegetated area than from an area of bare vegetation. Vegetal cover has a decided influence on the reduction of flood of small magnitude; but the effect is much less pronounced for large floods.

Flood problem arises not only due to excessive water in the rivers but also due to excessive sediment loads in them. Some reduction in sediment inflow is possible by use of soil-conservation methods within the drainage area.

Soil conservation is essentially a land development measure, as it increases the agriculture yields and productive potential of forests and grass lands. Its effect on reducing the sediment load in rivers and diminution of flood discharge are additional to the improved capability of the soil.

8.2. The important factors which effect the soil erosion are :

i) The intensity and amount of rain fall;

ii) The topography of catchment

iii) The nature of the soil; and

iv) The vegetative cover.

8.3. Effects of Soil Erosion :

When the rain falls with the intensities greater than the rate at which it can enter the soil, or infiltrate, the water flows down hill towards the streams as surface run-off along with sediment removed from the top layer of the soil. In addition to loss of valuable soil from land the eroded sediment carried into the river streams causes the following damages:

i) Section of the river gets reduced by deposition of sediments and thereby increases the flood heights.

ii) It upsets the regime of the river, causing it to change its course, erode its banks and silt its bed.

iii) Reservoirs get silted up and thereby their life is reduced.

iv) The height of the embankments in case of aggrading rivers is raised and consequently they become more vulnerable to breaches.

v) Navigability of the rivers is reduced on account of excessive sediments deposit.

vi) The course sediment gets deposited on the flood plains, ruins the standing crops and even renders unfit fertile lands for cultivation.

8.4. Soil conservation measures comprises :

8.4.1. (a) Maintaining a mantle of vegetation over the land during periods of heavy precipitation to reduce soil erosion. It has been seen that the soil losses from exposed ground are

50 to 150 times the losses from well-protected areas. Where the land has been unjudiciously made bare, the same should be covered by growing some vegetation on it. The reckless deforestation should be checked by controlling authority like Forest Department. The rights of adjoining villagers regarding timber, fuel, cultivation, grass-grazing in the forests should be regulated by this authority.

8.42.(b) Additional measures to reduce the erosive forces, specially the eroding action of water such as terracing, contour cultivation, Check dams.

i) Cultivation on very steep hilly slopes should be prohibited and a permanent cover of vegetation (i.e. forest or grass) should be maintained on it. That is the land which would inevitably get eroded, if laid bare, should never be uncovered by destroying the vegetation on it.

ii) Terracing : Terraces are made on steep slopes, to breach the velocity of water and hence its erosive force. The peak rate of run-off is 50 to 60 per cent less on terraced area than on unterraced land. The terraces are important in reducing the amount of soil reaching the stream channels from the cultivated land.

iii) Contour cultivation : This kind of cultivation reduces the total run-off from 15 to 47 per cent and soil losses from 30 to 50 percent from the land so cultivated.

iv) Chek -dams : These are constructed across streams to control its gradient and high velocities responsible for bed and bank erosion. These are constructed with brush wood and loose stone.

8.5. Effect of vegetal cover on run-off and sediment flow :

To have some idea about the effect of soil conservation on reduction of run-off and soil erosion, some research was carried out in Bombay State on specially constructed plots during 1934-35 to 1942-43. The results of experiments are as follows :

Sl.No.	Different types of covers	% of runoff to total rain	Soil loss (100 lb/acre)
1.	Plot with natural vegetaion	4.77	11.7
2.	Vegetation removed	19.75	396.3
3.	Shallow cultivation	22.50	556.0

The results of these experiments show a decided increase of rate of run-off and soil erosion on bare lands as compared with those under complete vegetation. It is further increased if the land is cultivated and left fallow.

8.6. Limitations :

Soil conservation measures are intended to reterd the speed of run-off and to minimise soil losses and consequently the sediment laad in the river. While this may moderate medium and low floods, their effect on the high floods is not significant. But soil conservation measures play an important role of preventing excessive loss of soil even during high floods. However, these have to be carried out over large areas, and require considerable time to be effective. These measures, though useful, are slow and costly to implement.

Upto-date Progress of Flood Protection Works :

The physical progress regarding flood protection works achieved upto March 1975 includes construction of 7883 km. of flood embankments, 13395 km. of drainage channels, 4686 villages have been raised above high flood level and 221 town protection schemes have been executed. These works have provided protection to nearly 80.622 lakh ha.

CONCLUSION :

Every method of flood control has its advantages as well as its limitations. In the adoption of flood control methods, emphasis has too often laid on a comparison of the engineering effectiveness of different measures. The general practice adopted is to select, out of the technically feasible methods, one that is the most economical or yields the highest benefits cost ratio. In doing so it is generally found that a combination of different methods rather than a single method will be most economical.

It is beyond doubt that any improvement in the methods of flood control is of major significance to the improvement of living standard in that area. Flood problems affect millions and they can only be solved by the co-operative effort of millions.

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REPORT OF THE EXAMINERS FOR AWARD OF M.E.DEGREE

1. Name of the Candidate SHRI R.C.AJWANI

2. Department W.R.D.T.C.

3. Title of dissertation 'FLOOD PROTECTION METHODS, THEIR APPLICATION & LIMITATIONS.

4. The Viva-Voce Examination was held on November 19,1975 at Roorkee.

EXAMINER'S REPORT

The candidate has brought ~~imagine~~ out the importance of flood Protection measures as a part and parcel of other development activities in flood ~~fo~~prone areas lying in river basis. The usual methods described for flood protection include construction of storage reservoirs (including multipurpose reservoirs), detention basis, construction of embandments, channel imperovement, construction of river training works and cut offs, Soil Conservation etc. which can be provided to sint various sites depending upon the extent of dammage and relative impoptance of the areas to be protected. While complete immunity from flood damages may not be possible to be achived but the losses to life and property (Crops and urban property) could be minimised by dealing the problem in a planned manner.

The dissertation is a v.satisfactory effort on the part of the candidate.

The dissertation is approved.

Sd/-J.P.Gupta,
Superintending Engineer,
Tehri Dam Design Circle, Roorkee.

Dated 26.11.1975

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NO.EX/ /E-139 Dated February ,1976.

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