MANUAL ON FLOOD FORECASTING

CENTRAL WATER COMMISSION
RIVER MANAGEMENT WING
R.K. PURAM, NEW DELHI
MARCH 1989

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PREFACE

The Central Water Commission has gained three decades of solid experience in the field of flood forecasting. This has been a period of constant endeavours on extending and improving the system. By now the CWC forecasting services have been extended to most of the interstate rivers of India. The forecasting system has also improved to a great extent. Hydrological and hydrometeorological observation network, communication facilities, formulation of forecasts as well as dissemination of forecasts to various State Govt. agencies have all expanded. With the procurement of more VHF/HF wireless sets and experience of operation, the data communication system has improved a good deal. Over 400 wireless stations are operating at present. The various State Governments and Flood Control Organisations concerned in the country are also bestowing greater attention on dissemination of forecasts through their "control rooms", which receive the CWC forecasts and issue flood warnings to flood affected areas. The methods and procedures of flood forecast formulation have also undergone refinements. Very close monitoring of the forecasts particularly during the last three years have further improved the flood forecast performance. This figure had improved to about 94 to 95% accuracy during 1987 & 1988.

Eventhough higher costs are involved, the Central Water Commission have not lagged behind in adoption of modern technological developments wherever possible subject to availability of funds. A pilot scheme for improvement of river flood forecasting system in India was commenced in 1980. Phase-I of this scheme is over. Phase-II has also been completed and is presently under testing for commissioning. Several Engineers and Hydrometeorologists have been imparted training in various flood forecasting models. They have received training in mathematical models and have used it in development of models for use in our operations.

In the light of the complexities of the issues involved and the need for training and retraining the CWC personnel in the matter, a "Manual on Flood Forecasting" was brought out in 1980. During the last 8 years modern technology has been increasingly adopted and advanced technology utilised for flood forecasting on the Yamuna at Delhi and inflow forecasting for the reservoirs in the DVC system. Automatic communication systems have been installed for acquisition of hydrometeorological data. Use of Satellite for data acquisition has recently become operational. The need had arisen, therefore, to revise and update the "Manual on Flood Forecasting".

To achieve this objective a Committee was constituted in December, 1986 under the chairmanship of Shri R. Rangachari, the then Member (River Management), CWC with the following composition:-

* 1. Shri R. Rangachari	Member (River Management)	Chairman
* 2. Shri G.S. Singh	Chief Engineer (Eastern Region)	Member
* 3. Shri T. Kumara Das	Chief Engineer(Southern Region)	Member
* 4. Shri L.S. Saini	Chief Engineer(Northern Region)	Member
5. Shri R.S. Prasad	Director (Upper Yamuna Circle)	Member
6. Shri M.E. Haque	Dy. Director (Middle Ganga Division)	Member
7. Shri N.Ram Gopal	Dy. Director (Krishna Division)	Member
8. Dr. P.B. Kukreja	Dy. Director (Hydromet)	Member-Secretary

* Shri R. Rangachari, Member (River Management), CWC has since retired. Shri G.S. Singh, Chief Engineer, went back to his parent Department, namely, Irrigation Department, Govt. of Bihar, and recently has come again on deputation to the Govt. of India as the Chairman, Ganga Flood Control Commission, Patna. Sarvashri T. Kumar Das and L.S. Saini, Chief Engineers, have also retired.

Various other Officers were coopted from time to time as considered necessary and different chapters were drafted by small groups. The material was then reviewed by Shri Rangachari as Chairman of the Group.

A deliberate attempt has been made to include illustrative examples to make the suggested techniques and procedures clear and useful by flood forecasters working at far flung locations in the field. These examples are only illustrative and need not be taken to be wholly comprehensive. The computer programme developed by the C.W.C. has also been incorporated. The Manual has been subsequently updated taking into consideration the information on the appraisal of the performance of the flood forecasting and warning network during the year 1988.

It is gratefully acknowledged that Shri R. Rangachari provided the inspiration and encouragement for preparation of this Manual.

Also, many other individual officers and staff of CWC (River Management Wing) helped in giving the present shape to this Manual. I am sincerely grateful to them for their devotion and assistance. Particular mention may, however, be made of the contribution by S/Shri G.S. Singh and T. Kumara Das, Chief Engineers; S.P. Rao and R.S. Prasad, Superintending Engineers and

T.K. Mukhopadhyay, Jagendra Singh, M.E. Haque, N. Ram Gopal, H.C. Upadhyay and A.K. Mohinta, Dy. Directors. Shri P.B. Kukreja as the Member-Secretary made significant contributions.

Thanks are also due to S/Shri P.K. Uppal, R.S. Rawat and N.D. Sharma, Mrs. Krishna Sharma and Mrs. Alamelu for typing the manuscript.

(M.S. RAO)
CHAIRMAN
Central Water Commission

New Delhi
March 21, 1989

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CHAPTER-I

INTRODUCTION

1.1. General

Since time immemorial, floods have been responsible for untold misery in major portions of the world and India is no exception. Valuable property, loss of crops, human lives and livestock continue to get washed away during flood times.

Due to ever increasing pressure of population and due to economic considerations, encroachments on flood plains have gone almost unabated. This is causing progressive increase in flood damages. During the monsoon season, when about 80% of the total annual run-off occurs in the Indian rivers, floods of varying intensities are experienced in one or other part of the country. The Rashtriya Barh Ayog had assessed that an area of around 40 million ha was prone to floods in the country. However, due to topographical and economic factors protection against floods can be provided only to around 32 million ha. An area of about 13 million ha has been protected from floods till 1985 through flood protection works, mainly embankments. In the context that flood protection still remains to be given to vast areas as the non-structural measure of Flood Forecasting and Warning is of immense help. Even where structural measures have been undertaken non-structural measures will enhance their usefulness in flood loss mitigation. That is why the Central Water Commission have undertaken the work of flood forecasting and alerting the State Govt. authorities concerned with a view to enable prevention of loss of life and property.

1.1.1. Causes of Floods :

Floods are caused by excessive rainfall in the catchment area while the magnitude and severity thereof depends on the nature and extent of rainfall and the characteristics of the specific watersheds. River floods are caused when heavy and wide spread rain persists in and around the river basins and water flows above the banks. Flash floods in smaller rivers occur due to concentrated spells of heavy rain in short periods when the volume of water that flows is beyond the defined drainage limit of their channels. Floods are also locally caused when drainage facilities are inefficient. Many factors influencing incidence of floods are :-

(i) Intensity and duration of rainfall

(ii) Wetness of the underlying surface at the time of rainfall

(iii) Course of monsoon system causing rain and the direction of river flow.

1.1.2. Floods due to Meteorological Situation

Meteorological situations that are likely to cause heavy rainfall over an area and potentially important for causing floods are as follows:

(i) Tropical storms and depressions

(ii) Active monsoon situation

(iii) Break monsoon situation.

Persistent thunder showers in pre-monsoon season in North Eastern parts of India and in parts of Kerala cause floods during May. during South-West monsoon season when monsoon is active and depressions form in Head-Bay and move Westwards in rapid succession, floods are caused in coastal rivers of Orissa and Central part of the country. During break monsoon situation floods are caused in the rivers emanating from the Himalayas. Tropical storms and depressions cause heavy rainfall during the course of their movement. These moving systems cause heavy rainfall and consequently floods when they are stationary systems or when their speed is reduced at the time of recurvature.

1.1.3. Floods due to breaches

Besides natural causes, some areas get flooded when failure of structural measures occur such as of breaches in the embankments of the rivers. Floods may also get aggravated due to release of water from reservoirs when the rivers are already in floods. Catastrophies could also occur due to failure of reservoirs or sudden release from detention storages or due to blockages of river water on account of land slides.

1.1.4. Flood Management Techniques

"Flood Management" is a broad-based term which includes planned engineering measures aimed at not only controlling the flood but also providing optimum utilisation of stored surplus water during lean season. Wise application of engineering science has afforded ways of mitigating the ravages due to floods; and providing reasonable measure of protection of life and property. Structural engineering measures include the following:-

(i) Multipurpose reservoirs and retarding structures for storage of flood waters.

(ii) Channel improvements to increase flood carrying capacity of the river.

(iii) Embankments and levees for keeping the water away from flood prone areas.

(iv) Detention basins for retarding and absorbing flood waters.

(v) Flood ways for diverting flood flows from one channel to another.

(iv) Improvements in the drainage system.

These structural measures have the effect of restricting the movement of flood water into lood plains.

It has been recognised that loss of human life and property can be reduced to a considerable extent by giving reliable advance warning about the incoming floods. The people could be moved to safer places in an organised manner as soon as the flood warnings are received. Valuable moveable property and cattle could be saved by shifting them to places of safety.

The effectiveness of non-structural measure like flood forecasting and warning, in reducing flood damage would depend upon how accurately the estimation of future stage or flow of incoming flood and its time sequence at selected points along the river, could be predicted.

In passing it may be also mentioned that other non-structural measures like soil conservation, flood proofing, flood plain management, flood plain zoning, flood insurance etc. also play significant role in reducing flood damages but are beyond the scope of this manual.

For optimum operational efficiency, of reservoirs, the accurate hydrological forecast is of vital importance. Thus if a reliable estimate of river flow is available, it is possible to schedule reservoir releases during lean season optimally satisfying power and irrigation needs. Similarly, with a reliable flood forecast, it may be possible to draw down a combined power and flood control reservoir without seriously reducing power generation.

In general, the hydrological forecasts (short range and seasonal forecasts) could contribute towards considerable flood damage reduction as well as efficient operation of reservoirs. With increased emphasis on multipurpose projects and regional development of water resources, the need of hydrological forecasts is more and more felt and reliable forecast service becomes a necessity both for a means of flood damage reduction and reservoir regulation.

Central Water Commission is playing a key role in planning, design and implementation of such flood control projects in the country. Alongwith the structural measures, the Govt. of India lays parallel emphasis on non-structural measures for flood damage reduction and one such measure is flood forecasting including inflow forecasting/level forecasting.

1.1.5. National Water Policy

The National Water Policy of India, adopted in 1987 by the National Water Resources Council has asked that "an extensive network of flood forecasting should be established for timely warning to the settlements in the flood plains, alongwith the regulation of settlements and economic activity in the flood plain zones, to minimise the loss of life and property on account of floods." It has further urged that "the emphasis should be on non-structural measures for their minimisation of losses — such as flood forecasting and warning and flood plain zoning, so as to reduce the recurring expenditure on flood relief."

1.1.6. Flood Forecasting for Flood Mitigation Planning

Emergency measures require prediction of incoming floods, rapid warning to flood plain inhabitants and advance preparation involving provision of flood fighting materials and a plan of action for intense activity immediately prior to, during and immediately after flooding takes place. Once the flood forecasts are received by the Civil and Engineering authorities, the chain of activities start. Warnings are issued, people and moveable property are evacuated immediately prior to floods, patrolling of the affected area is organised, protection works including sealing off inlet points are effected and rescue operations are done during the flood. After the flood recedes, rapid drainage, clean up and rehabilitation follow.

In the long-range plan of flood protection and flood damage, the flood forecasting services assume considerable significance not only in unprotected flood prone areas for organising rescue and relief operations but also in the area already protected by giving advance warning to the engineering authorities, for keeping desired vigil and safeguarding various structures.

1.2.1 Definition of Flood Forecast

Flood forecasting may be defined as "the process of estimating the future stages or flows and its time sequence at selected points along the river during floods". Flood forecasts refer to prediction of "the crest and its time of occurrence" and logical extension to the stages of river above a specified water level called the "Warning Level". This warning level is generally 1 metre below the Danger Level fixed in consultation with the beneficiary, i.e., the concerned State authorities.

1.2.2 Utility of Forecast

Utility of a forecast is dependent on both accuracy and time of prediction. At the time of issue of the forecast, the value and utility of the forecast can be considered to be zero. The entire operation of the "Flood Forecasting Service" has to be planned around a time-factor keeping in view the following factors:-

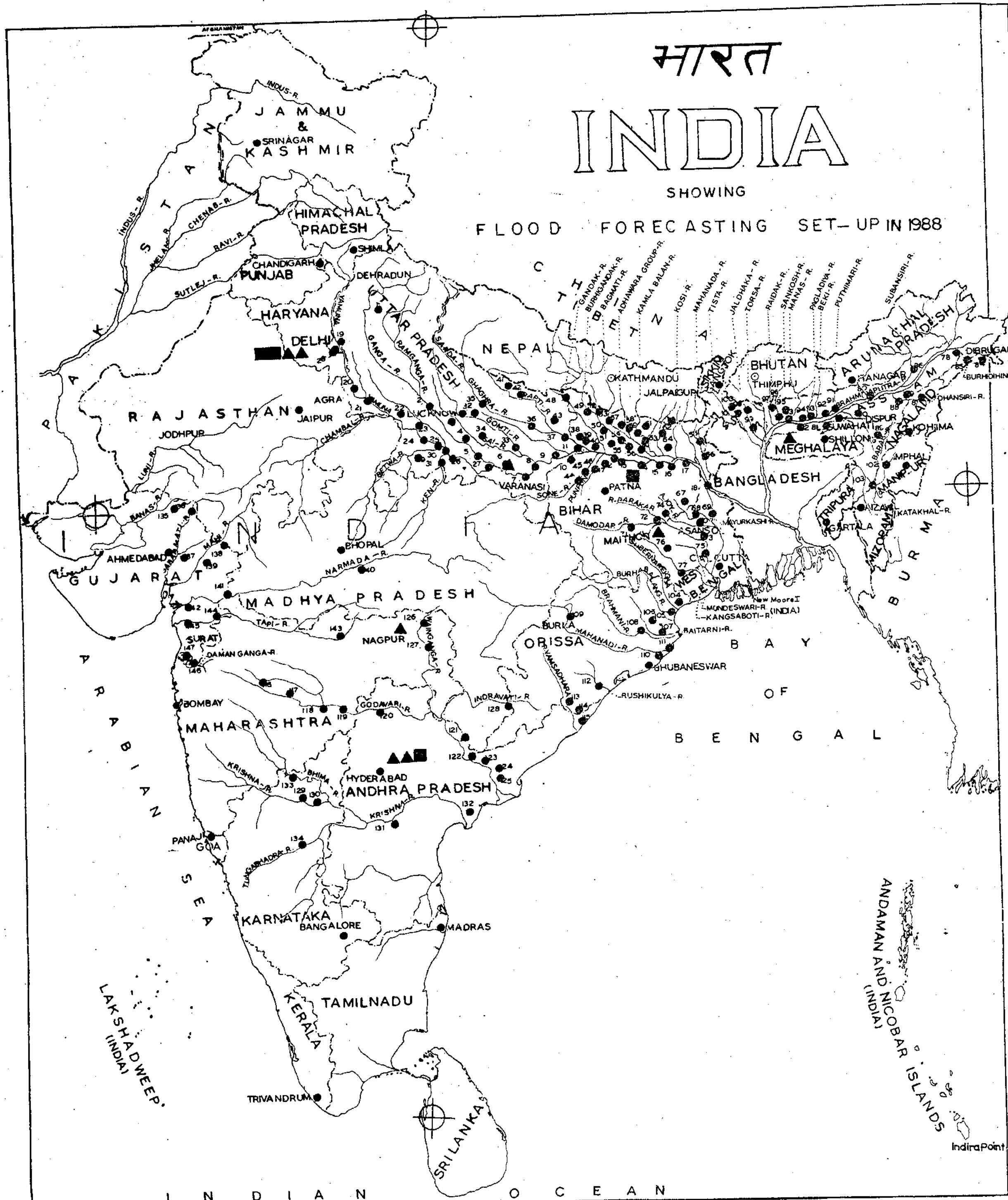
(i) Availability of Operational Data

The operational data being generally poor and incomplete, a compromise between a theoretically desirable and operationally available is required.

(ii) Adoption of appropriate technique

A call for use of appropriate techniques or methods to meet the requirements of accuracy

Fig. 1.1.



S NO.	FORECASTING SITES	RIVER
1.	HARIDWAR	GANGA
2.	ANKINGHAT	,,
3.	KANPUR	,,
4	DALMAU	,,
5.	PHAPHAMAU	,,
6.	CHHATNAG (ALLAHABAD)	,,
7.	MIRZAPUR	,,
8	VARANASI	,,
9.	GHAZIPUR	,,
10.	BUXAR	,,
11.	BALLIA	,,
12.	DIGHAGHAT (PATNA)	,,
13.	GANDHIGHAT (PATNA)	,,
14.	HATHIDAH	,,
15.	MUNGER	,,
16.	BHAGALPUR	,,
17.	COLGONG	,,
18.	FARAKKA	,,
19.	DELHI RAILWAY BRIDGE	YAMUNA
20.	MATHURA	,,
21.	AGRA	,,
22.	ETAWAH	,,
23.	AURAIYA	,,
24	KALPI	,,
25.	HAMIRPUR	,,
26.	CHILLAGHAT	,,
27.	NAINI (ALLAHABAD)	,,
28.	DHANSA REGULATER	SAHIBI
29.	MOHANA	BETWA
30.	SHAHJINA	,,
31.	BANDA	KEN
32.	HANUMAN SETU (LUCKNOW)	GOMTI
33	JAUNPUR	,,
34.	RAE BARELLY	SAI
35	ELGIN BRIDGE	GHAGHRA
36.	AYODHYA	,,
37.	TURTIPAR	,,
38.	DARAULI	,,
39.	GANGPUR SISWAN	,,
40.	CHHAPRA	,,
41.	BALRAMPUR	RAPTI
42.	BANSI	,,
43.	BIRDGHAT (GORAKHPUR)	,,
44.	INDERPURI	SONE
45.	KOELWAR	,,
46	MANER	,,
47	SRIPALPUR	PUNPUN
48.	KHADDA	GANDAK
49.	CHATIA	,,
50.	REWAGHAT	,,
51.	HAZIPUR	,,
52.	LALBEGHIAGHAT	BURHI GANDAK
53	SIKANDARPUR (MUZAFFARPUR)	,,
54.	SAMASTIPUR	,,
55.	ROSERA	,,
56.	KHAGARIA	,,
57.	BENIBAD	BAGMATI
58.	HAYAGHAT	,,
59.	KAMTAUL	ADHWARA GROUP
60.	EKMIGHAT	,,
61.	JHANJHARPUR	KAMLA BALAN
62.	BASUA	KOSI
63.	BALTARA	,,
64	KURSELA	,,
65.	DHENGRAGHAT	MAHANANDA
66.	JHAWA	,,
67.*	MASSANJORE DAM	MAYURAKSHI
68.*	TILPARA BARRAGE	,,
69.	NARAYANPUR	,,
70.	GHEROPARA	AJOY
71.*	TENUGHAT DAM	DAMODAR
72.*	PANCHET DAM	,,
73.*	DURGAPUR BARRAGE	,,
74.*	MAITHON DAM	BARAKAR

S NO.	FORECASTING SITES	RIVER
75.	HORINKHOLA	MUNDESWARI
76.*	KANGSABATI DAM	KANGSABATI
77.	MOHANPUR	,,
78.	DIBRUGARH	BRAHMAPUTRA
79.	NEAMATIGHAT	,,
80.	TEZPUR	,,
81.	GUWAHATI	,,
82.	GOALPARA	,,
83.	DHUBRI	,,
84.	NAHARKATIA	BURHI DIHING
85	KHOWANG	,,
86.	BADATIGHAT	SUBANSIRI
87.	GOLAGHAT	DHANSIRI
88.	NUMALIGARH	,,
89.	KAMPUR	KOPILI
90.	DHARAMTUL	,,
91.	N.H. ROAD X-ING	PUTHIMARI
92.	ROAD BRIDGE	BEKI
93.	N.T. ROAD X-ING	PAGLADIYA
94.	N.H. CROSSING	MANAS
95.	GOLAKGANJ	SANKOSH
96	TUFANGANJ	RAIDAK
97.	GHUGHUMARI	TORSA
98.	N.H. 31 ROAD BRIDGE	JALDHAKA
99.	MATHABANGA	,,
100.	DOMOHANI ROAD BRIDGE	TISTA
101.	MEKHLIGANJ	,,
102.	ANNAPURNAGHAT	BARAK
103.	MATIZURI	KATAKHAL
104.	RAJGHAT	SUBERNAREKHA
105.	N.H.-5 BRIDGE	BURHABALANG
106.	ANANDPUR	BAITARNI
107.	AKHUAPADA	,,
108.	JENAPUR EXP. WAY	BRAHMANI
109.*	HIRAKUD DAM	MAHANADI
110.	ALIPINGAL DEVI	,,
111.	NIMAPARA	,,
112.	PURUSHOTTAMPUR	RUSHIKULYA
113.	GUNUPUR	VAMSADHARA
114.	KASHINAGAR	,,
115.*	GOTTA BARRAGE	,,
116.	KOPERGAON	GODAVARI
117.*	JAIKWADI DAM (PAITHAN RESERVOIR)	,,
118.	GANGAKHED	,,
119.	NANDED	,,
120.*	POCHAMPAD (SRIRAM SAGAR)	,,
121.	DUMMAGUDAM	,,
122.	BHADRACHALAM	,,
123.	KUNAVARAM	,,
124.	RAJAMUNDRY	,,
125.	DOWALAISWRAM	,,
126.	BHANDARA	WAINGANGA
127.	PAUNI	,,
128.	JAGDALPUR	INDRAVATI
129.*	ALMATTI DAM	KRISHNA
130.*	NARAYANPUR DAM	,,
131.*	SRISAILAM DAM	,,
132.*	VIJAYWADA	,,
133.	DEVANGAON BRIDGE	BHIMA
134.*	TUNGABHADRA DAM	TUNGABHADRA
135.*	DANTIWADA DAM	BANAS
136.*	DHAROI DAM	SABARMATI
137.	SUBHASH BRIDGE (AHMEDABAD)	,,
138.*	KADANA DAM	MAHI
139.	WANAKBORI	,,
140.	HOSHANGABAD	NARMADA
141.	GARUDESHWAR	,,
142.	BHARUCH	,,
143.*	HATNUR DAM	TAPI
144.*	UKAI DAM	,,
145.	SURAT	,,
146.*	MADHUBAN DAM	DAMAN GANGA
147	DAMAN	,,

* INFLOW FORECASTING SITES.

REFERENCE

1. INTERNATIONAL BOUNDARY
2. STATE BOUNDARY
3. STATE CAPITAL
4. RIVER
5. FLOOD FORECASTING STATION
6. DIVISION OFFICE
7. CIRCLE OFFICE
8. CHIEF ENGINEER'S OFFICE
9. MEMBER (RIVER MANAGEMENT)

50 0 50 100 150 200 250 300 Km.

GOVERNMENT OF INDIA
CENTRAL WATER COMMISSION
NEW DELHI

and period of warning to different locations.

(iii) Dissemination of Forecast

Dissemination of Forecast is equally important as accuracy and timeliness of forecast. There should be an efficient arrangement for dissemination of the forecast/information. Attempts should be to convey the forecast/warning to the authorities concerned so that they get adequate time to organise necessary measures, if required, in the light of the forecasts issued.

1.3.1. Historical Background

There has been significant progress in the field of flood forecasting in India during the last three decades. An Index Map showing the location of Flood Forecasting Sites in different basins and location of Divisions, Circles, Chief Engineer's Offices are given in Fig. I.1.A schematic diagram showing the system in operation is given in Fig. I.2. The present setup since 1987 of the field formations under Member (River Management) is given in Fig. I.3.

The Central Water Commission now operate 147 flood forecasting sites and has planned the addition of some sites every year depending upon the requirement from the States and availability of funds. These are backed by about 400 hydrological observation sites and rainfall data of over 500 meteorological stations is collected, processed and utilised for forecast formulation. The operational data are mostly transmitted over radio-telephones/wireless sets. Its wireless network consists of over 400 stations. The sites are mostly having 15 watt H.F. sets single side band, land line communication. The control rooms, Divisional, Circle and Chief Engineer's Offices are connected with 100 watt/500 watt wireless sets.

1.3.2 Present Status

The river wise list of the flood forecasting sites on rivers and their tributaries is given in Fig. -I.1

The following is the statewise distribution for 147 flood and inflow forecasting sites as in October, 1987:

1.	Andhra Pradesh	9	7.	Karnataka	4
2.	Assam	20	8.	Madhya Pradesh	2
3.	Bihar	36	9.	Maharashtra	7
4.	Dadara & Nagar Haveli	2	10.	Orissa	11
5.	Delhi	2	11.	Uttar Pradesh	31
6.	Gujarat	9	12.	West Bengal	14

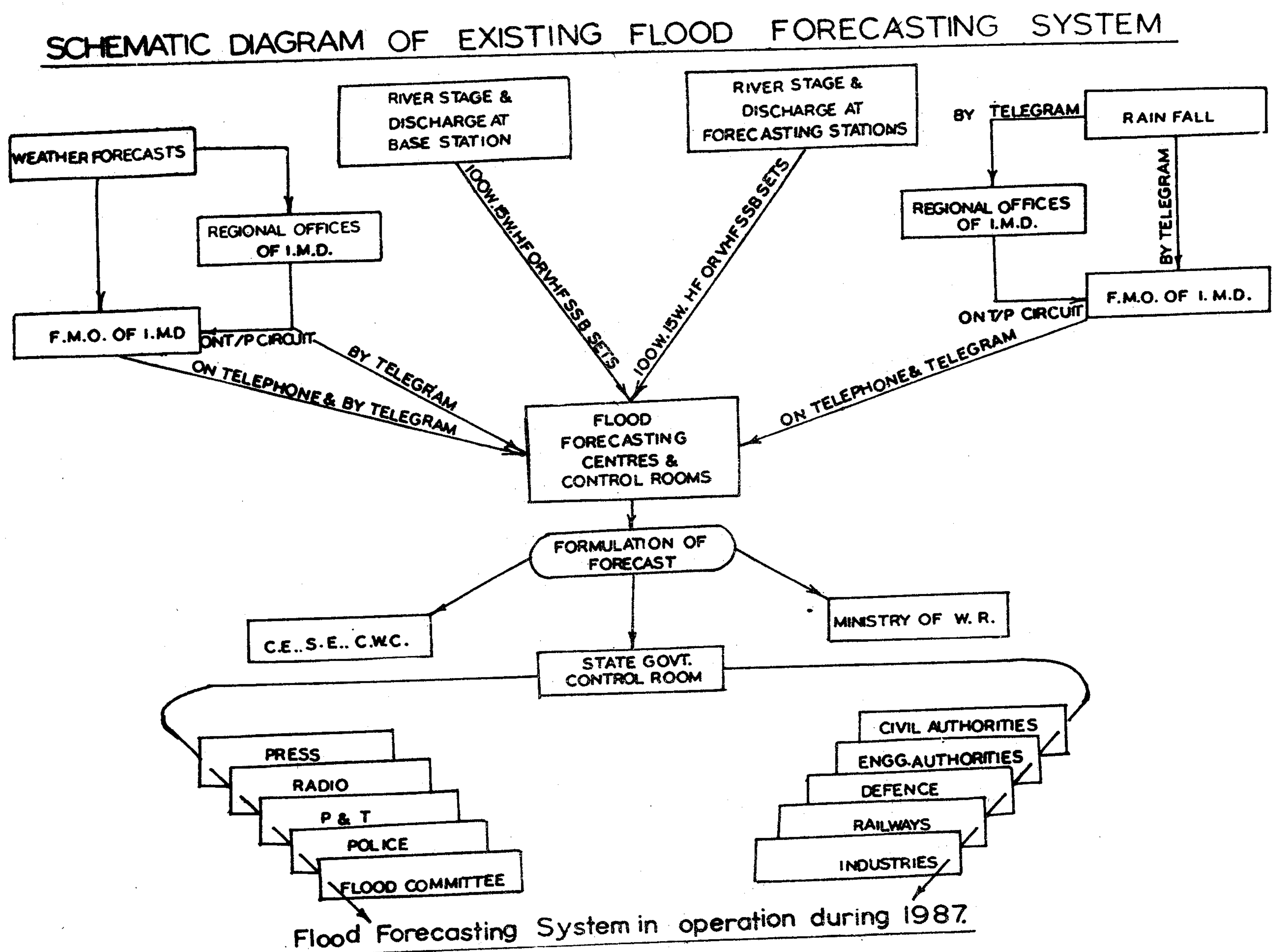
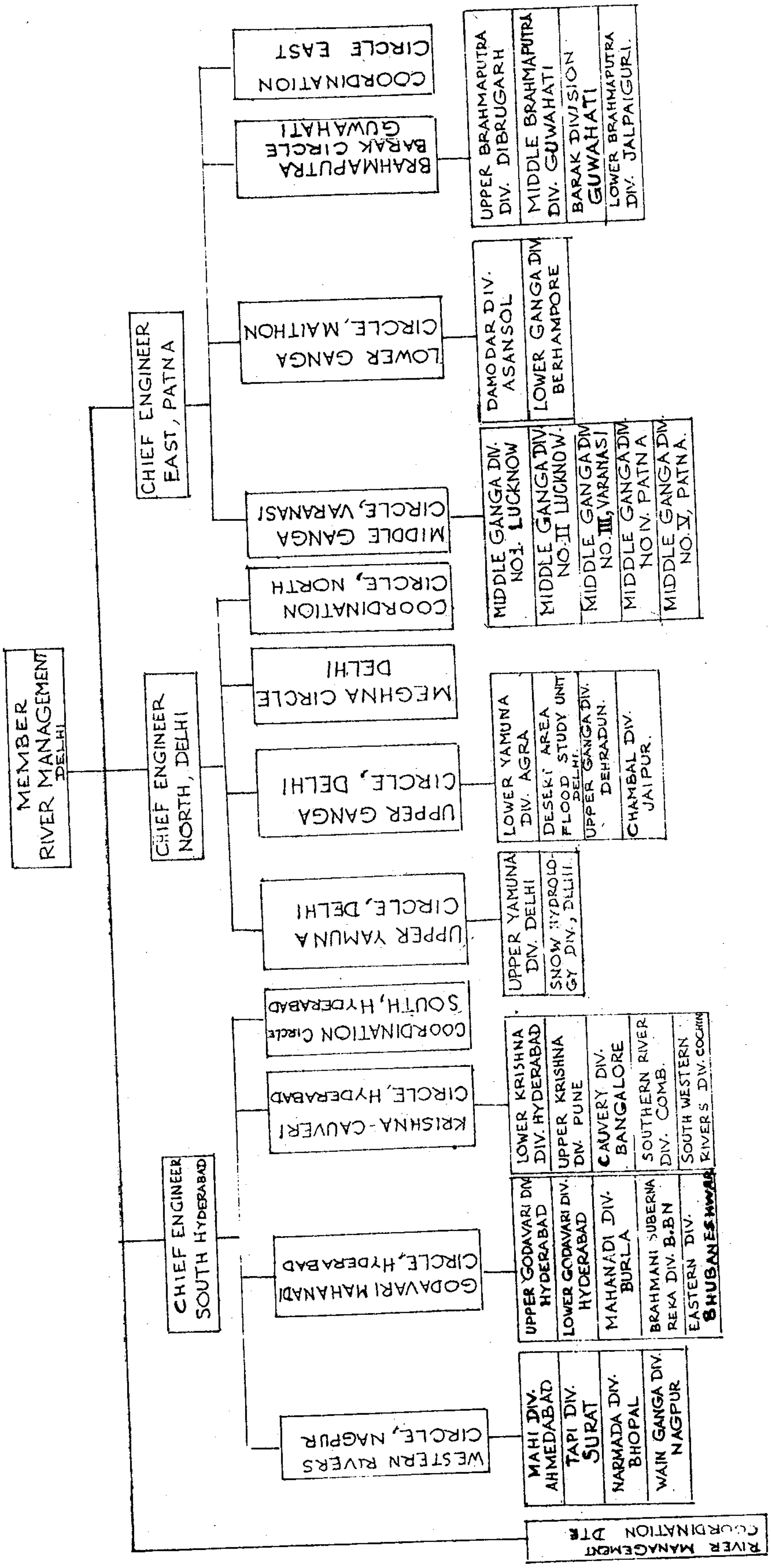

SCHEMATIC DIAGRAM OF EXISTING FLOOD FORECASTING SYSTEM
WEATHER FORECASTS
REGIONAL OFFICES OF I.M.D.
F.M.O. OF I.M.D
ON T/P CIRCUIT
BY TELEGRAM
ON TELEPHONE & BY TELEGRAM
RIVER STAGE & DISCHARGE AT BASE STATION
100W. 15W. HF OR VHF SSB SETS
RIVER STAGE & DISCHARGE AT FORECASTING STATIONS
100W. 15W. HF OR VHF SSB SETS
RAIN FALL
BY TELEGRAM
BY TELEGRAM
REGIONAL OFFICES OF I.M.D.
F.M.O. OF I. M.D.
ON T/P CIRCUIT
ON TELEPHONE & TELEGRAM
FLOOD FORECASTING CENTRES & CONTROL ROOMS
FORMULATION OF FORECAST
C.E.. S.E.. C.W.C.
STATE GOVT. CONTROL ROOM
MINISTRY OF W. R.
PRESS
RADIO
P & T
POLICE
FLOOD COMMITTEE
CIVIL AUTHORITIES
ENGG. AUTHORITIES
DEFENCE
RAILWAYS
INDUSTRIES
Flood Forecasting System in operation during 1987

Fig. 1.2



FLOOD FORECASTING SET-UP, C.W.C.

Fig. 1.3.

It would be noted that there are still no CWC forecast stations in many States of India.

1.3.3. Techniques Employed for issue of Forecasts

1.3.3.1. Various Steps

The various steps involved in the operation before issue of forecasts and warning are as follows:-

(i) Observation and collection of hydrological and meteorological data.

(ii) Transmission/communication of data to the forecasting centres.

(iii) Analysis of data and formulation of forecasts.

(iv) Dissemination of forecasts and warning to the Administrative and Engineering authorities of the States.

1.3.3.2. Data Collection

Observation and collection of hydrological data are done by field formations functioning under Member (RM), Central Water Commission. Flood Meteorological Offices (FMO) of India Meteorological Department collect and transmit the meteorological data. The former is also responsible for planning of river gauge/discharge network, collection of gauge and discharge data and communication of the data to its Flood Forecasting Centres, while the latter is responsible for planning of rain gauge network in consultation with CWC and for collection and transmission of rainfall data to the Flood Forecasting Centres. The F.M.O. provides information regarding general meteorological situation, rainfall amounts of last 24 hours and heavy rainfall warning for the next 24 hours for different regions and quantitative precipitation forecast for various river basins to the concerned flood forecasting centres of Central Water Commission.

The hydrological data are collected everyday and utilised by flood forecasting offices for formulation of forecasts during monsoon period. Similarly, the meteorological data including warning & general synoptic situation and weather forecast are generally being supplied by FMO to the Divisions and Sub-Divisions of Central Water Commission concerned daily by telephone, failing which the information are being collected by the special messengers of CWC field unit from the FMO Office.

1.3.3.3. Data Transmission

Transmission of data on realtime basis from the hydrological and hydro-meteorological sites to the Flood Forecasting Sub-division/Division is a very vital factor in flood forecasting. Time is of essence in the whole exercise. Transmission of data should be, therefore, as fast as possible for enabling organisation of relief measures and protective steps. Transmission of the observed data on realtime basis is, in fact, most crucial for efficient flood forecasting system.

The land-line communication i.e. by telephone/telegram was the earliest and very commonly used mode for data transmission in Flood Forecasting Services till 1970. This system was having the following drawbacks:

(i) The telegraph offices were not always located very close to the data observation sites and consequently a lot of time was wasted in performing the journey between the site and the telegraph office.

(ii) During heavy rainfall period, when timely requirement of the data becomes extremely essential, the telegraph/telephone system became frequently out of order.

The communication system was further improved by installing VHF/HF wireless sets at the data collection sites. The wireless stations are generally operated by the Wireless Operators for transmission of data to the Sub-division. Provision of wireless mechanics has also been made for repairs of the sets and their maintenance. Modern uptodate telecommunication techniques have been introduced in the Upper Yamuna upto Delhi. Planning, operations, maintenance and improvement of the communication network is looked after by CWC officers and supporting staff. Further improvements in the CWC telecommunication system are under execution.

1.3.3.4. Data Analysis and Forecast Formulation

After receipt of the hydrological and hydrometeorological data at the sub-divisions, the data are compiled, scrutinised and analysed by Engineers/Hydrometeorologists engaged in this work. The system of data processing before use in forecast formulation has been introduced to prevent chances of errors. Many forecasting centres have been provided with micro-computer facilities for data processing.

The next important step is the formulation of forecast. In fact, the analysis of data and formulation of forecast is a very important stage in the process of forecasting system.

The various flood forecasting centres are using different forecasting models, based on availability of hydrological and hydrometeorological data, the basin characteristics, computational facilities available at forecasting centres, warning time required and purpose of forecast. However, some of the common methods being used by various centres are given below:

(i) Simple correlation-based on stage-discharge data.

(ii) Co-axial correlation-based on stage, discharge and rainfall data etc.

(iii) Routing by Muskingum method.

(iv) Successive routing through sub-reaches.

(v) Hydrologic models (at selected places).

The forecasts obtained from the correlation diagrams or mathematical models etc. are modified as necessary to arrive at a final forecast based on the prevailing conditions in the river. This requires intimate knowledge of the river by the forecaster. Forecast once issued is further modified and revised forecasts issued, if necessary, on the basis of additional information received after the initial forecast was made.

1.3.3.5. Dissemination

The final forecasts are being communicated to the administrative and engineering authorities concerned of the state and other agencies connected with the flood protection and management work on telephone or by special messenger/telegram/wireless depending upon local factors like vulnerability of the area and availability of communication facilities etc.

On receipt of flood forecasts, the above agencies disseminate flood warnings to the officers concerned and people likely to be affected and take necessary measures like strengthening of the flood protection and mitigation works and evacuation of the people to safer places etc. before they are engulfed by floods. Generally, the State Governments set up control rooms at States and District Headquarters which receive forecasts and then further disseminate the flood warning to the affected areas and organise relief as well as rescue operation. Flood forecasts are also passed on to the All India Radio, Doordarshan and the local newspapers. It must be noted that the ultimate efficiency of the whole flood forecasting and warning system is only as good as the weakest link in the chain between the forecast agency (CWC in this case) and the public who need to respond. This has often been a somewhat neglected part of the total flood warning process. The many potential benefits in improving flood forecasting may not really materialise if the dissemination link is unnecessarily inefficient. Under the existing setup, however, this most important link is in the hands of the State Government agencies to implement even though C.W.C. is vitally interested in this step.

1.4.1. Modernisation Schemes

Two modernisation scheme as per details below are in various stages of completion. These are discussed below:-

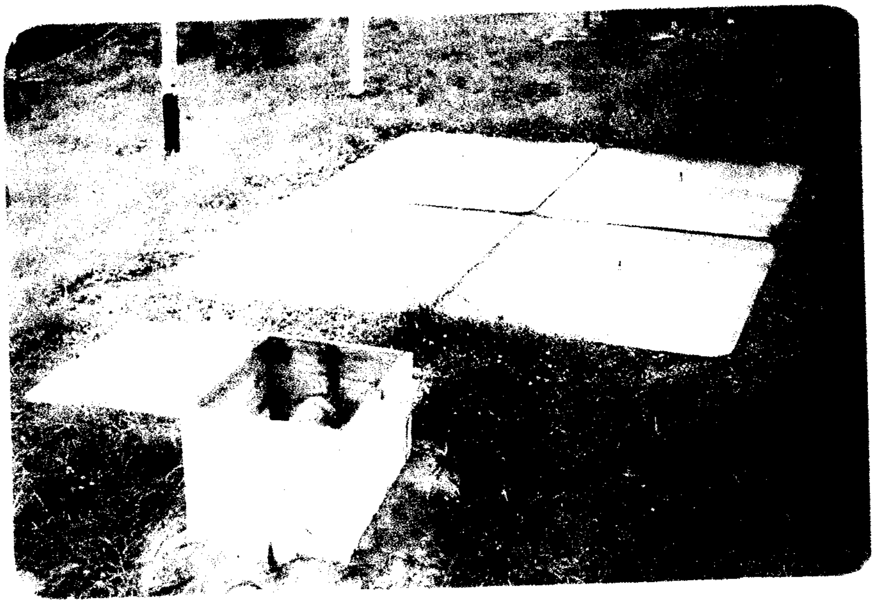
1.4.1.1. Improvement of River & Flood Forecasting System-Pilot Project

A pilot project for the "Improvement of River and Flood Forecasting System in India" was taken up in 1980 as UNDP aided project on the Yamuna river upto Delhi. The executing agency to the UNDP assisted scheme was the W.M.O. The Phase-I of the scheme was completed in 1985. Fourteen data collection-cum-auto-reporting stations which are provided with sensors for precipitation, temperature, water level and micro processors and a VHF telecommunication system besides 7 repeater stations have been installed.

A mini computer, Hewlett Packard 1000 F Series system having 1536 K byte memory, 132 M byte winchester disc storage, thermal printer, graphic plotter, digitizer, display terminal,

TELEMETRY STATION AT TUINI.

COMPUTER CENTRE UPPER YAMUNA CIRCLE, NEW DELHI



SNOW PILLOW AT JHUBBAL, (SHIMLA), HP.

V—NOTCH AT JUBBAL, (SHIMLA), (HP).

teleprinter and matrix printer etc. with a master teleprocessor has also been commissioned at the Central Station located in Sewa Bhawan, R.K. Puram, New Delhi.

The master teleprocessor has been programmed to coordinate the activities of remote stations by directing them to transmit data in a specified sequence and time intervals to store the data on floppy, print the data in a specified format and display the data on CRT, whenever required. The data stored on the floppy is transmitted to the mini-computer through RS-232 interface.

Three computerised hydrological models VIZ; SSARR (Streamflow Synthesis and Reservoir Regulation-Model), NAM-System 11 FF (Flood Forecasting) Model and NLC (Non Linear Cascade-Model) which are continuous models and the HEC-1F (Flood Hydrograph Package Model of Hydrologic Engineering Centre) which is an event type model, have been transferred to the computer. All these models have up-date capabilities and also include rainfall runoff and flood routing routines. These models which are calibrated and are in running mode, were put under test in 1985, 1986 and 1987 monsoon seasons.

As a part of UNDP Pilot Project, an experimental watershed, the Sundli Nala, was selected in the Upper Yamuna basin in Himachal Pradesh for Snow Hydrology studies as detailed earlier.

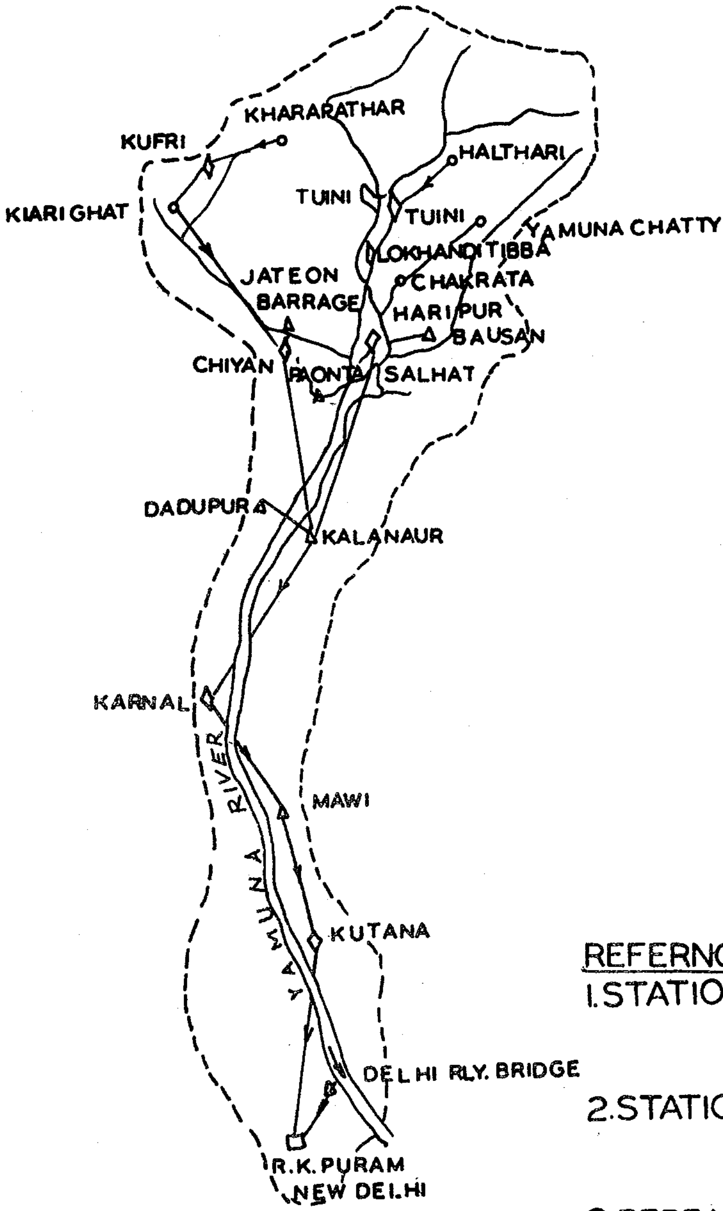
A 'Review Mission' constituted by WMO/UNDP in October, 1984 recommended that Phase-II of the UNDP Project may be taken up to improve the telemetry system by installing 'Data Collection Platform' (DCP) at the data collection sites for communication of data through satellite 'INSAT-1B'. Under Phase-II of this Project which includes the small Snow Research Basin in upper reaches of the catchment, it has been proposed to instal 14 DCSTS (Data Collection, Storage and Transmission Sub-system) with suitable interface to the existing sensors and to communicate the data by using the INSAT- 1B facility to the earth station at Secundrabad and on to the IMD Headquarters at Lodi Road, New Delhi through a micro-wave link and from where the data would be passed on to CWC Control Room at Delhi through VHF wireless link. The implementation of the Phase-II of the project has reached advanced stage of execution and will be completed by December, 1988.

Map showing the Yamuna basin and telemetry system are shown at Fig. I.4 and Fig. I.5. respectively.

1.4.1.2 Snow Hydrology

Snow Hydrology Division for the development of Snow Hydrology sanctioned by the Government of India in May, 1984 is functioning under Upper Yamuna Circle, New Delhi as part of the UNDP/WMO pilot Project for 'Improvement of River and Flood Forecasting System in India". This Division initially started functioning with effect from 21st June, 1984 with its Headquarters at New Delhi. The headquarters were shifted to Shimla w.e.f. 1.10.1987.

A small sub-basin called the Sundli Nala in the Upper Yamuna catchment has been selected as the test region for the development of Snow Hydrology.



REFERNCE :-

1. STATIONS WITH W.T.P. SENSORS.....△
2. STATIONS WITH T. P SENORS...... o
3. REPEATER STATIONS....◇
4. CENTRAL STATION......□

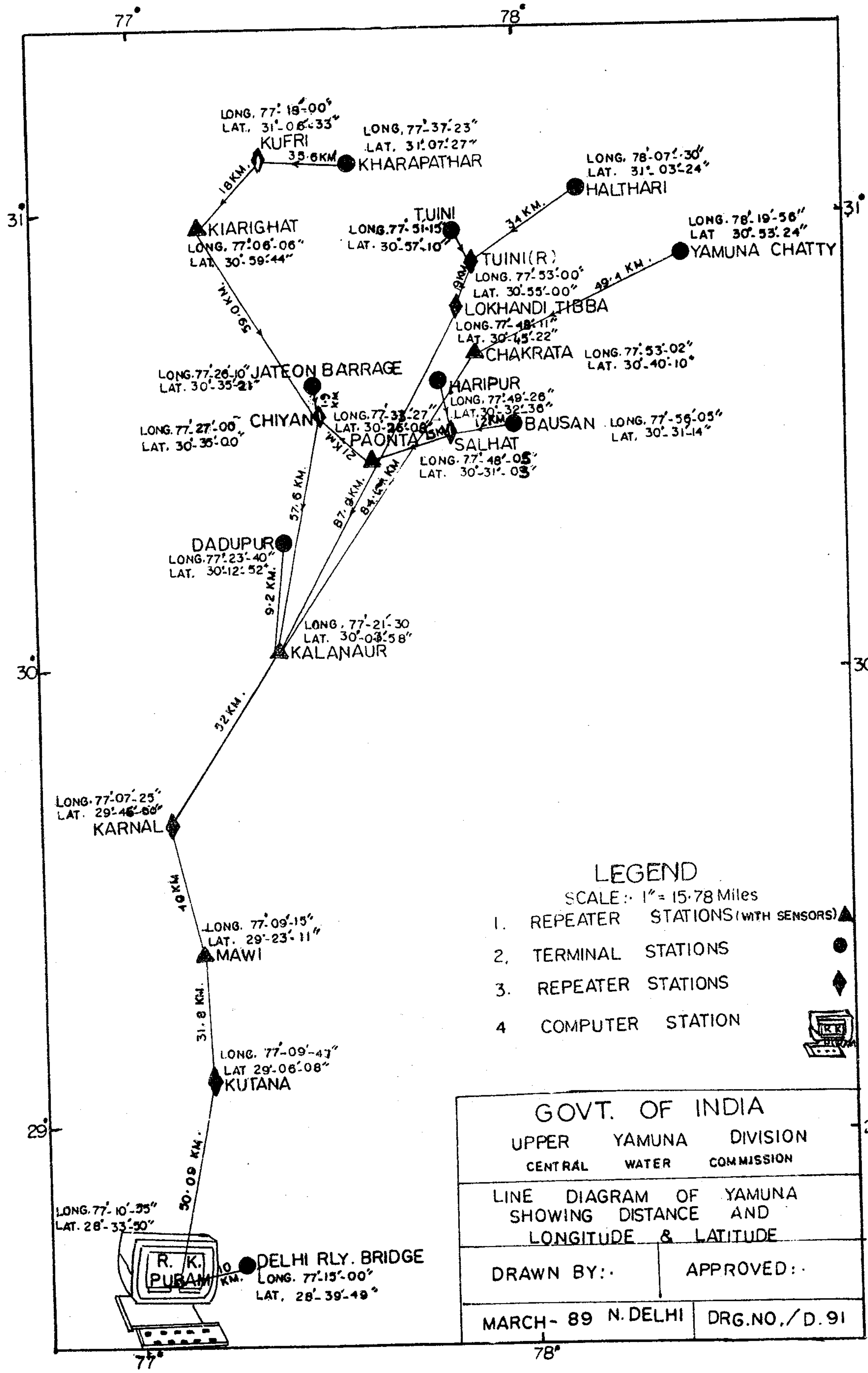
W: Water level sensor,

T: Temperatures sensor.

P: Precipitation sensor.

Yamuna Basin

Fig. 1.4.



TELEMETRY SYSTEM

In all 16 snow courses were selected in the watershed keeping in view the elevation aspect etc. for snow surveys. Snow pillows and other meteorological remote sensors such as Net Radiometer, Precipitation Recorder, Hygrothermograph, dual channel wind recorder, Pan Evaporation Recorder etc. supplied by UNDP were installed at the site. A winter shelter has been constructed at this site which will provide shelter to the snow survey team during winter season. Snow surveys were started during the 1984-85 winter season. For the measurement of snowmelt runoff, a V-notch weir has also been constructed at the exit of the watershed to measure the snowmelt runoff. A self recording river gauge has been installed at the weir for automatic recording of river stages. Suitable snowmelt runoff models for the short range and long range seasonal snowmelt forecasts are being developed.

1.4.2. DHI-CWC Collaboration Project

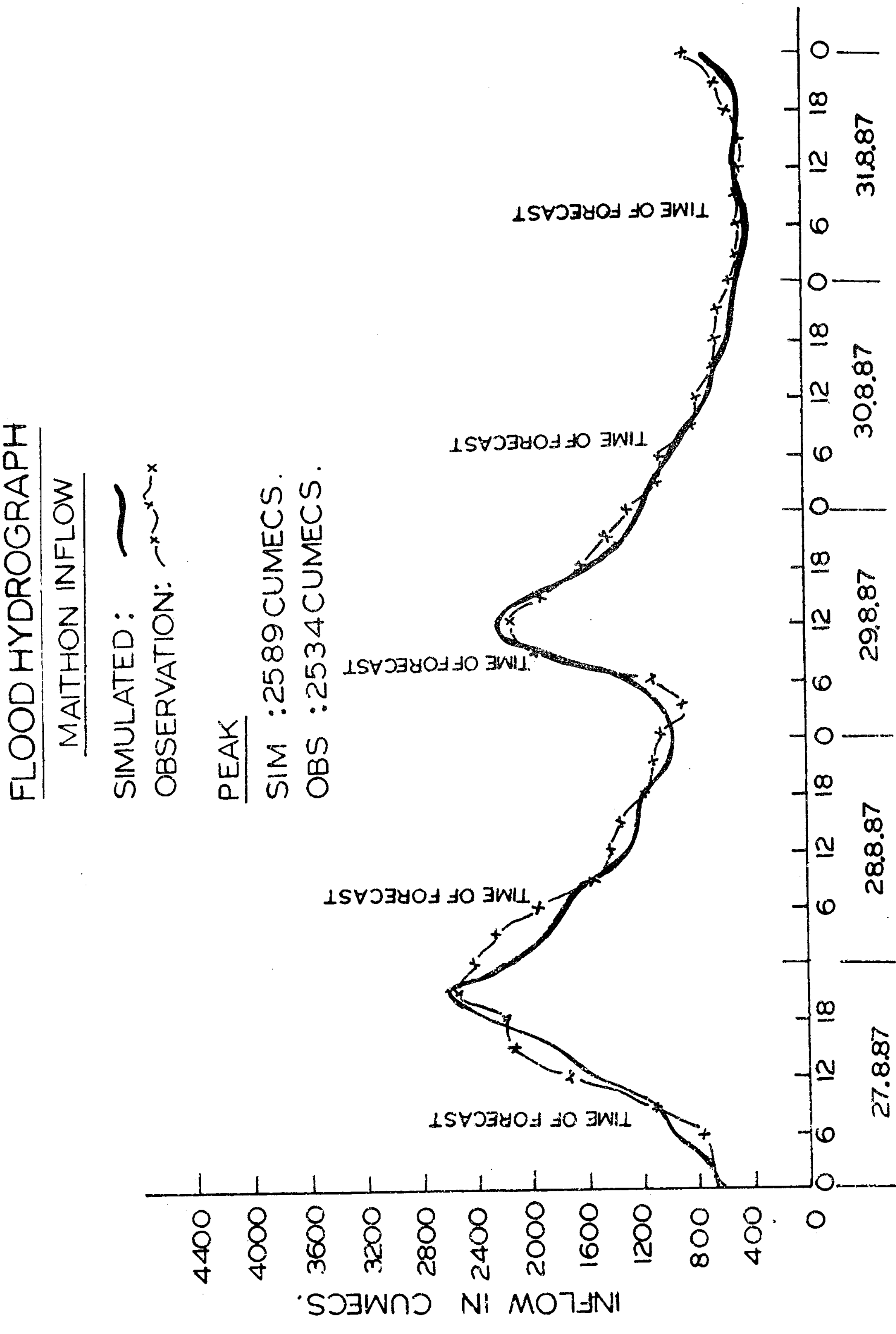
Under the Danish Hydraulic Institute/Central Water Commission Collaboration Project, with Damodar river basins as focus project, computerised mathematical models developed in the DHI were transferred and adapted after modification for inflow forecasting and formulation of flood forecasts using indigenous computer. Phase-II of this scheme was successfully completed in 1986 and models have become operational at Maithon and Delhi. One such result is given in Fig. I.6 . The Phase-III of the Project has been taken up for refinement, upgrading of the existing model, introduction of sediment, transport module and Dambreak model and application of the existing model to another river basin.

1.5.1. Forecast and the Society

The present day society is conscious of the utility of the flood forecasts. Radio, Television, News Papers help in dissemination of the forecasts to a large extent. People are well aware of the utility of these forecasts. Disasters do not just happen because society is well aware of careful planning, adequate training and efficient implementation of well conceived measures at the time of disasterous situations. The choice of measures or combination of measures should always be considered with care and implemented with sincerity. Without this there could be no solution to Disaster Mitigation.

People who have experienced floods are usually ready to pay particular attention to any warnings that are issued and to follow the advice that is given including instruction for evacuation to safer area. It is necessary to ensure that all people, not only those with actual experience, have an awareness of the danger posed by floods. 'Human memory being proverially short', and memories are apt to fade, the awareness must be kept alive and upto date.

Public must be educated and kept fully informed. Educational programmes should be designed for general public as well as school children. It should form part of syllabus. In Japan and Philippines course have been designed for "Typhones" – people are educated to look for tracks. Similar programmes and courses can be designed for floods also. The objective of education should be to create confidence that warnings and instructions receive proper response.


FLOOD HYDROGRAPH
MAITHON INFLOW
SIMULATED :
OBSERVATION:
PEAK
SIM :2589 CUMECS.
OBS :2534 CUMECS.
TIME OF FORECAST
TIME OF FORECAST
TIME OF FORECAST
TIME OF FORECAST
TIME OF FORECAST
INFLOW IN CUMECS.
0
400
800
1200
1600
2000
2400
2800
3200
3600
4000
4400
0 6 12 18
6 12 18 0
6 12 18 0
6 12 18 0
6 12 18 0
27.8.87
28.8.87
29.8.87
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31.8.87

Fig. 1.6

1.5.2. Flood Forecasting- A discussion on Socio-Economic aspects

The value of a forecast lies in its ability to reach the public at the earliest. Enough warning time should be available so that people can move to safer places. A late warning or a warning with less lead time is of no use to the society. Thus warning with sufficient lag time has a utility and men and material can be saved.

On the other hand at many places people have learnt to live with floods. At those places also if real danger of a high flood is communicated well in time people can take appropriate evacuation measures.

At the other extreme is a warning which is not a 'warning in the real sense'. Such warnings should be avoided as it keeps the people "tense" unnecessarily.

1.5.3. Flood Forecasting - A discussion on some practical aspects

When a forecast is issued, public in general are not aware of the effect of particular level in the concerned area. The level information should be translated into its effect on the surroundings. It should be described with reference to the previous historical flood marks and flood events in the area. Prominent marks at various places should be indicated so that they gain confidence in the forecast. These guide posts should be brought to the notice of general public. In the event of real danger due to high floods proper arrangement must be made to keep the public well informed and effort must be made to evacuate them to safer places which are identified before hand.

To have a perfect forecast all the five basic functional components of flood forecasting mentioned below are equally important:-

(i) Reporting network

(ii) Data collection and Processing Facility.

(iii) Forecast preparation Centre.

(iv) Forecast Dissemination Facility.

(v) Forecast procedure development team.

Failure of any one of them will badly dislocate the system. The success of the system will really depend on how smoothly the various functional units run to achieve the following three objectives:-

(a) The loss of time in transmission of data, its processing and analysis, formulation of forecast and dissemination of the same is minimised.

(b) The quality of data is ensured and necessary checks applied at as many points as possible to avoid the errors to the maximum extent.

(c) The forecast is formulated with the help of the most appropriate model using the correctly defined boundary conditions for the model under the current situation.

The best possible forecast can be formulated only when the above objectives are achieved by the forecasting system. However, considerable practical difficulties are encountered in the operation of the system. These problems can be clearly identified from the three objectives described above, and the same are discussed briefly in the following sections.

1.5.4. Delay in Forecast

Some of the factors which are responsible for the loss of time and hence the delay in issue of forecast are:-

(a) Failure of communication system,

(b) processing of huge basic data,

(c) time consuming analytical techniques, and

(d) complicated models for formulation of forecasts.

1.5.5. Failure of Communication System

It is one of the major factors which delays the transmission of data from the observation point to the control rooms where these data are to be processed. The disruption of road/railroad traffic, particularly during heavy flood creates problems where data are sent through messengers. The disruption of telephone lines etc. as a result of storm etc. renders such communication system useless at the time of need. Power breakdown, sudden trouble with the wireless sets and, damages to masts also create serious problems in timely transmission of data. These problems are very frequent in nature and, therefore, it is necessary that the forecasting organisations equip themselves to meet such eventualities by having suitable arrangements to serve as alternative communication system. In India, the flood forecasting units generally have understanding with other agencies (such as police, customs departments etc., who have their own communication network) for their use as regular mode or as an alternative in such situations.

1.5.6. Processing and Analysis of Data and Forecast Formulation

The processing of huge data, their analysis, and formulation of forecast consumes definite amount of time which depends upon:

(a) the number of variables to be used in operational forecasts and their frequencies.

(b) the technique used for analysis of these data (such as the method for assessment of areal distribution from the point rainfalls, the model for time distribution of the amount of direct runoff etc.).

(c) the structure of the models used in the forecast formulation, and

(d) the computing facilities available at the forecasting centre.

The time required in processing, analysis and formulation of forecasts can be considerably reduced by using a reasonably fast computing facility such as a small size computer/micro-processor. In the absence of such facilities, it would be necessary to adopt the techniques/models which are:

(a) as simple as possible,

(b) capable of achieving an accuracy necessary for the purpose, and

(c) expressed in form of functions easy to calculate with the help of available computing facilities.

1.5.7. Sources of Errors in Forecast

The forecast issued in time is not enough. It should also be as accurate as possible. The three main sources of error in forecast are:-

(a) error at source i.e error in observed data,

(b) error during transmission , and

(c) the computational error.

1.5.7.1. Errors at Source

The errors at source may be either the instrumental error or the observational errors or the copying error. The observational and the copying errors can be avoided to a great extent by proper supervision and checks at different levels at frequent intervals. The instrumental errors may be classified in two groups, viz; (i) errors of sudden or emergent nature, and (ii) errors which creep slowly over long time. The errors of the first type are because of sudden problems with the equipments e.g., washing away of gauge post and error during fixation of new gauge posts by staff members not fully trained for the job. A careful processing of data might reveal such errors. Besides, arrangements are generally made for such situations. For example, gauge marks are painted on nearby permanent structures (such as bridge pier, steps of the ghat etc.) near the proper gauge sites. Similarly, the nearby gauge posts/gauge marks of other agencies (state Governments, Railways, etc.) are correlated with the proper gauge sites.

The error of second type are rather difficult to be noticed during the routine data processing. Such errors in the gauge data, for example, may be because of slow settlement of gauge posts in sandy beds or in discharge data, due to deterioration in rating of the current meter, These errors can be detected by frequent checking.

The other hydro-meteorological and hydrological equipments (raingauges, etc.) are also liable to similar errors.

Yet another source of error is due to communication gap. For example, when the gauge post at the proper site is washed away, the observer moves to alternative site for the observation and transmits the data of the new location without mentioning the fact (i.e the change in location of the observation) to the control room. As a result, the variation in the water level at the two location goes unnoticed and hence the error. Such errors are mainly because of lapse of sufficient training to field staff, and they can be avoided and detected by proper training to the concerned officials.

1.5.7.2. Errors during Transmission

The errors during the transmission (which are mainly because of slip of tongue, slip of pen, and absent-mindedness etc.) can be minimised by adhering to the procedure laid down for transmission of data. Further, any error noticed during the processing and the analysis should be immediately checked and rectified. Proper training of the personnels engaged in transmission of data is a must.

1.5.7.3. Computational Errors

The computational errors are of two types: the errors associated with the computational instruments such as calculators etc., and the human error. Such errors are quite possible and to avoid them, it is necessary to check the formulated forecasts at two or more levels preferably by using different approaches. For example a forecast formulated with the help of a co-axial diagram can be rechecked with the help of a mathematical equation representing the co-axial diagram. Further it will be desirable to have another check by some other technique. A water profile diagram — a very basic and rather crude tool for forecasting, may also be used before the issue of the formulated forecast. Such checks will considerably reduce the possibility of computational errors.

1.5.7.4. Unexpected Situations

A very difficult situation arises when the official responsible for formulation of forecast is handicapped because of:

(a) Non-availability of all the desired data/information, and

(b) Deviation from the defined boundary conditions.

Non-availability of All Required Data in time :

The non-availability of the desired information at the time of forecast formulation is very common problem. This generally results in either delay in formulation of forecast (when the receipt of additional information is awaited) or in formulation of forecast with computed data. Obviously, the delay in formulation of forecast will cause loss of precious time which is not desirable. On the other hand, the forecast formulated with computed data may affect the accuracy of forecast.

Therefore, it is necessary that the alternative methods/ techniques are available so that the forecast could be formulated using the available limited data with known degree of accuracy.

Deviation from Defined Boundary Conditions:

A not so common but very important phenomenon is the situation when there is deviation from the defined boundary conditions of the model. Some of the examples of such situations are:

(a) Breach in the flood embankments of river.
Such a situation occured during 1986 in the river Gandak when there was a breach near Pipra Piprasi and river Burhi Gandak when the embankments breached near Samastipur. Under such situations, the commonly used model for forecast do not work any more and the necessary informations about the condition of the breaches etc. are to be collected round-the clock and duly considered while formulating the forecast.

(b) Rain of very high intensity at locations in between the base and the forecasting stations.

This becomes very important when the intermediate catchment is considerable and the same is not incorporated with due weightage in the model.

(c) Unexpected regulation of the control structures.

Such a situation is very often encountered in forecast of sites located just upstream of the control structure or when the base station is located downstream of the control structure. A sudden closure/opening of gates without advance information to the forecasting centre adversely affects the forecast performance.

The conditions arising out of the such situation can be included in the forecasting model in a very limited way. Therefore, they have to be carefully examined and their impacts judiciously incorporated in the model as the situation warrants. Alternatively some other methods are to be identified and adopted depending upon the nature of the problem. Hence, it is necessary that the personnels at the forecasting centre are duly qualified, trained and experienced, and are capable of taking immediate decisions and act accordingly.

One more important aspect which needs to be looked into is the definition of the responsibilities of personnels involved in forecasting process and the organisational set up.

1.5.8. Organisational Set Up

The discussions on the various problems enumerated in the earlier sections repeatedly refer to well defined functional units at the forecasting centre. A forecasting centre should consist of following five well defined functional units.

a. Reporting network headquarters.

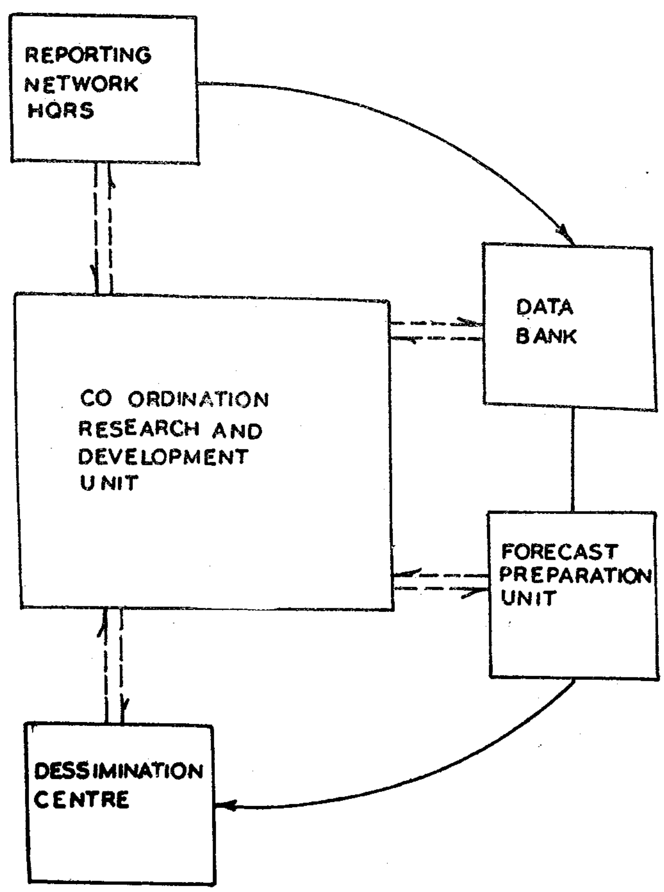
b. Data bank

c. Forecast preparation unit.

d. Forecast dissemination centre, and

e. Co-ordination, research and development unit.

Fig. I.7. illustrates the functional relationship between all the five units.



FUNCTIONAL UNITS OF A FORECASTING CENTRE

Fig. 1.7.

CHAPTER-II

DATA OBSERVATION AND COLLECTION

2.1 Introduction

Data observation and collection constitutes the first phase of the flood forecasting system. A large extent of hydrological, hydrometeorological and other related data are required for development of a hydrological forecasting model for a river system and for operational forecast formulation. For development of the forecasting model, it is desirable to analyse and make use of considerably larger data base for better estimation of the various parameters of the model representing the system behaviour. However, for operational forecasts, the use of all the data may not be feasible because of many constraints such as real time transmission, considerable analytical work due to larger data base and economy of the system. Therefore, the data from very limited number of hydrological and hydrometeorological stations are used for the operational forecast formulations. Hence, the term "Data Observation and Collection" in a broader sense means:

(a) Collection of all the observed historical data from various sources and observation of necessary additional data needed for development of the hydrological forecasting model; and

(b) Identification of representative or key data network from where real time data transmission can be ensured within the constraints of available resources.

Correct observation, systematic collection and proper processing of these data are very essential so that these are available to users without any delay and difficulty.

2.1.1. Data Requirement

There are a number of hydrological and hydrometeorological data which are required to be observed to meet the various objectives. Some of these data which are commonly observed as part of the hydrological observation network in the country include:

(A) Hydrological observations

(i) River water level;

(ii) Discharge;

(iii) Ground water level;

(iv) Water quality;

(v) Sediment load;

(B) Hydrometeorological observations

(i) Rainfall;

(ii) Other forms of precipitation such as snow, Hail etc;

(iii) Evaporation;

(iv) Temperature;

(v) Humidity; etc.

For hydrological forecasting, some or all of the above data may be needed either for model development or for operational use. The data requirements for hydrological forecasting depends on many factors, such as:

(a) Purpose and type of forecast;

(b) Model for forecasting;

(c) Desired degree of accuracy of forecast;

(d) Basin characteristics; and

(e) Economic constraints of the forecasting system.

(i) Purpose and type of forecast

The data requirement will considerably vary with the purpose of the forecast. In the case of a reservoir inflow forecasts for the purpose of its operation, relating to short intervals of time and the volume of water likely to enter the reservoir flow as a result of particular storm flood will be needed. In case of a large river, the water level forecast can be estimated easily with the help of the water level of upstream stations. Therefore, the data input in such cases will be only water level at two or more stations on the main river or its tributaries. On the other hand for a small flashy river, apart from the observations of water level and discharge at relatively small interval, the use of rainfall data almost becomes unavoidable. As a matter of fact, the forecast is to be formulated with the help of quantitative precipitation forecast and other hydrometeorological data in case of very flashy rivers as concentration time is very small.

(ii) Model for forecasting

The hydrological forecasting models are of different types. A specific model is to be selected keeping in view the objectives of the forecast, the basin features and hydrological characteristics, the computing facilities etc. The input data requirement both for calibration as well as for operational forecasts vary considerably from model to model. For example, in case of a simple gauge to gauge co-relation model which may be suitable for large rivers the data requirement may be only water level at two or more stations. On the other hand, the use of a suitable comprehensive catchment model will need a number of data. The data requirement for some of the commonly used hydrological forecasting models are summarised in Table II. 1.1.

TABLE: II. 1.1 : Data Requirement for some of the commonly used Forecasting Models

Data		*Simple Gauge to Gauge Relation*	*SSARR Model*	*Stanford Model*	*Sacromento Model*
1.	Rainfall	No	Yes	Yes	Yes
2.	Gauge/ Discharge	Yes	Yes	Yes	Yes
3.	Evaporation/ Evapotrans-piration	No	Yes	Yes	Yes
4.	Temperature	No	Yes	Yes	No
5.	Basin Area	No	-	-	Yes
6.	Other basin Characteris-tics	No	Yes	Yes	Indirectly

Data	Simple Gauge to Gauge Relation	SSARR Model	Stanford Model	Sacromentc Model
7. Soil water/ Infiltration data	No	Indirectly	Yes	No
8. Vegetation Cover	No	Indirectly	Yes	No
9. Impervious Area	No	Yes	Yes	No
10. Percent Slope	No	Yes	Yes	No

(iii) Desired degree of accuracy of forecast

Although accuracy is of primary concern, the constraints in respect of economy, the relatively lesser importance of the forecasting location etc. may permit for even lesser degree of accuracy and in such a situation a model may be selected where data requirement may be less rigorous. On the other hand, for a location which is densly populated and is highly sensitive to flood damage, it becomes necessary to go for greater accuracy. The use of more and more data may become necessary for improving the model efficiency. However, this is bound to increase the expenditure towards flood forecasting services.

(iv) Basin Characteristics

The basin characteristics both physiographic and hydrologic are one of the major factor in the choice of the forecasting model. Further the basin characteristics sometimes play a vital role in selection of observation of desired data. Therefore, the data availability are greatly restricted on this aspect.

(v) Economic contraints of the forecasting system

This is a major factor which governs the choice for observation, collection and analysis of data to be used for development of suitable model and for operational flood forecasting. More data entails more expenditure and more time in collection and analysis, man power etc. Cost effectiveness of the model vis-a-vis relative accuracy and consequences resulting therefrom are to be duly considered while deciding the data requirements.

Apart from the type of data to be used for the forecasting purposes, the information in respect of frequency of data, the length of record of data and the quality of data are equally important and they are to be duly accounted for in any flood forecasting system planning.

2.1.1.1 Frequency of Data Observation

The hydrological and hydro-meteorological data are generally of a continuous nature. However, these are discretised to facilitate the analysis. Apart from the reduction of analytical work, the discretisation of data also becomes necessary to reduce the cost of observation. No doubt the modern equipments which have been introduced facilitate a continuous record of data, but still all the data can not be and need not be observed on continuous basis. However, the frequency of observation is to be judiciously decided so that;

(i) the desired characteristics of the hydrological and hydrometeorological data frequencies are retained; and

(ii) the necessary results could be obtained from the available frequencies.

Some of the factors which are to be considered in deciding the frequency of observations are as follows:

(i) Economic constraints;

(ii) Availability of automatic equipments and the operation and maintenance facilities for them, and

(iii) the relative importance of the respective data to meet the desired objectives.

These aspects vary from place to place and also from time to time as will be explained from the following discussions.

In large basins where precipitation is not directly used in forecast formulation daily report of rainfall may be adequate in normal circumstances but hourly observations are necessary with the help of Self Recording Raingauges (SRRG) in order that the observed data are used for model development and also for issue of advisory forecasts in heavy and prolonged storms. But forecasts for small basins with rapid concentration time may require reports at interval of one hour or even at lesser interval during heavy storms. For example for slow rising rivers like Brahmaputra in its lower reaches where the maximum rise in water level is of the order of about 30 cm in 24 hours, rainfall data of 12 hours or 24 hours duration for upper catchment may be adequate. On the other hand in flashy rivers like Teesta where the concentration time is small and variation in rainfall intensity is quite considerable, it will be necessary to use 3 hourly or even hourly rainfall data from adequate number of SRRGs. The use of daily rainfall collected through ordinary rain gauge (ORG) is limited to initial water balance studies in process of model design, but for all practical purposes in flood forecasting only the rainfall data collected through SRRGs are used. In respect of river gauges, hourly observation at all sites is desirable. Also, there should be fool-proof arrangement for recording the peak of the floods. Frequency of transmission of river-stage will depend upon the exigencies of the situation. Normally the hourly observed data are transmitted to control room/flood forecasting centres twice or thrice a day, but when the rivers are above Danger Level, the frequency of transmission needs to be increased to meet the requirements.

2.1.1.2 Length of Data Record

The question of length of data record does not arise for the operational flood forecast. For the formulation of the forecast, the required data are needed on continuous basis. However, for development of forecasting model, the past record of data sets (both input and output) are necessary for estimation of the model parameters.

In general, the length of data record should be such that the flow characteristics as well as other features affecting the output are duly represented in the length of the record which is used for model calibration. As such, it may be desirable to have the total length such that at least one or two cycles of low flow and high flow sequences are represented. For the purpose of flood forecasting model, the most important aspect is to estimate and forecast the peak water level or peak discharge and hence every effort should be made to include the highest recorded peak in the recent past.

In general it has been recommended by many researchers that the minimum length of record should be about 10 to 15 years so that some portion may be used for model calibration and the remaining for proving it before the model is put to operation.

2.1.1.3 Quality of Data

The theory of "Garbage in - Garbage out" is applicable to all the models howsoever sophisticated they are. Therefore, the quality of data is of vital importance. As a matter of fact the sources of possible errors are many; particularly in case of manual observation of hydrological data and hence it is extreemly necessary to identify all such sources and take steps to minimise them. Some of the major sources of errors are as follows:

1. Instrumental error,
2. Observational error,
3. Processing error, and
4. Accidental error

The errors may be compensating or cumulative.

These errors can be minimised and the quality of data ensured if some of the following precautions are taken.

(1) Proper Selection of Site

This plays a very important role in ascertaining the desired quality in data observation. The details about the selection of suitable site for hydrological and hydrometeorological stations are discussed elsewhere and its importance need not be overemphasised. Some of the most important features of the site should be stability, easy accessibility and permanency.

(2) Choice of suitable instrument

The instrument should be of desired accuracy. It should be capable of recording the extreme events and the instrument should be such that its maintenance is easily possible under the local conditions. Yet another desirable quality of the instrument should be upgradability.

(3) Well trained and capable Staff

Professionals engaged in the work of data observation and in the maintenance of the site including the equipments etc. should be well trained for their job. There should be frequent checks and there should be arrangement for frequent training through refresher courses etc. to keep the professionals abreast of latest development in their field of activity.

(4) Data processing, checking and storage

The data as observed and collected from the various field stations should be processed as early as possible and appropriate quality control procedure should be applied before the data is finally recommended for use. There should be arrangement for thorough checking of the data at various stages.

From the above discussions, it is amply demonstrated that the type of data requirement, its frequency and duration etc. considerably vary depending on the purposes. The requirement is different for development of the model and its operational use. Further the requirement may vary even for the same purpose i.e. say operational use from site to site. Therefore, it becomes necessary to identify the actual requirement and then to plan the network of hydrological observations.

It is well known that the hydrological observation stations require considerable initial investment as well as substantial running and maintenance cost and therefore it is essential to have a very critical evaluation of the requirement and plan the network of data observation stations in such a way that all the essential informations are available at the minimum cost. It is important to note that increasing the network beyond a certain point with consequent increased costs, will not be commensurate with additional accuracy, if any, obtained thereby.

2.1.2 Objective of Data Network Design

As discussed earlier, the network of hydrological and hydrometeorological observation stations are to be planned for two distinct purposes as follows:

1. To collect sufficiently long record of necessary data for development of a suitable flood forecasting model.

2. To collect necessary data of desired frequency from selected stations on real time basis for the operational flood forecast.

The data requirement for the development of the model may be much larger than the one required for the operational purposes. As a matter of fact, the process of development of flood forecasting model can be initiated with the help of data which are collected under the hydrological observation programme of the region. The primary objective of the hydrological observation system is to have an overall picture of the water availability in the region and hence the data collected for this purpose may not be sufficient for the purpose of flood forecasting where detailed information particularly in respect of time distribution of the river flow are required. The specific requirements are to be evaluated on individual basis and it is not possible to suggest a general guideline for design of data network for flood forecasting. The various factors which essentially govern the data requirement for flood forecasting have been briefly discussed in Section. 2. 1.1 and the detailed requirement for specific model, purpose of forecast, and the degree of accuracy etc. will be discussed subsequently in the respective section.

2.1.2.1 Design of Data Collection Network

Design of data collection network for operational flood forecasting is to evolve the total number of key stations for precipitation (and temperature for snowmelt regions), river stage and discharge measurements, their efficient location based on certain scientific criteria for selection of each station; the time span and frequency of observation; and determination of priority of network establishment.

Network Requirements

For the purpose of operational flood forecasting, the following basic requirement of the data collection network are to be kept in view:

(a) Correct prediction of flood hydrograph resulting from a precipitation event in the catchment.

(b) Study of longitudinal propagation of flood flows upto the forecasting point so that the time of warning may be kept reasonably long.

(c) If the distance between base station and forecasting station is very long, the available warning time may be more but the accuracy in prediction of flood flows will be less.

(d) Availability of data on real time basis. Since the data are to be collected in the least possible time, a compromise may be made between quick areal estimate of rainfall and accuracy attainable in the areal estimate of rainfall.

(e) For a given cost and duration of collection of data, the accuracy obtainable in the areal estimate of rainfall should be optimised by a suitable design of network.

(f) The magnitude of flood is important only so far that the channel capacity should not have hazardous spill over but flood elevations are all important.

(g) Where gauge to gauge correlation of river flow is good, the rain gauge network need not be so dense.

(h) Smaller catchment with flashy river system will require a dense network of self recording raingauges.

(i) The network planning should also take account of snowmelt and reservoir regulation.

(j) Though the basic purpose of this network is for flood forecasting but the network design should make a compromise for the integrated approach towards hydrological network design for other purposes by co-ordinating precipitation, gauge, discharge, sediment flow, water quality, ground water, evapotranspiration, and temperature networks.

2.1.2.2 Number of Key Stations

One objective of the network might be to collect data which maximise the expected net benefits to be derived from the development. A second objective is to collect data from minimum possible number of stations without any significant loss of information. The flood hazard data can be obtained either from crest gauges or fully equipped stream gauging stations.

The second step towards optimum design is to define the conditions which will limit the available resources for data collection. There are three types of constraints; time, money and personnel.

The third step in optimum design is to enumerate the various feasible alternatives i.e., the networks which are within the limits of the constraints and will contribute towards the stated goal. The specifications for each consist of:

(a) The number of sites at which data are to be collected;

(b) their locations;

(c) the type of data which are to be collected at each site;

(d) the frequency of observation and length of record of different type of data at various stations;

(e) the equipment and techniques which will be used to collect the data

(A) For Precipitation network

For the purpose of operational flood forecasting, it is required to estimate the areal mean of individual rainfall event over an area with the help of the following formula:-

$$\overline{P} = \frac{1}{A} \Sigma_{A=0}^{A} P(X_i) \, dA$$

where P (X_i) may be defined as total precipitation recorded by the 'i' th station during the entire duration of event. Obviously, it will be uneconomical and unrealistic to collect rainfall data from all stations on a real time basis in very short time available for flood forecasting. Therefore, one has to select key stations and arrange for real time reception of data only from these stations. Thus, in case of 'n' key station in the catchment being used for estimation of , we may write the $\overline{p}$ estimate $\overline{p}$ as

$$\overline{P} = \frac{1}{n} \Sigma_{i=1}^{n} P(X_i)$$

The variance of the estimate $\overline{p}$ may be expressed as $V(\overline{P}) = \left(\overline{P} - \frac{A}{P}\right)^2$

Rodriguez-Iturbe and Mejia have shown that the variance V ($\overline{p}$) may be given by

$$V(\overline{P}) = \delta p^2 F_2(n)$$

Where F2 (n) is variance reduction factor associated within sampling in space and is given by

$$F_2(n) = \frac{1 - \bar{r}A}{n}$$

where $\bar{r}A$ denotes the average value of correlation coefficient between precipitation at pairs of points situated in the catchment area A; δp is standard deviation computed from precipitation data of all stations for the storm event.

Thus the variance is given by

$$V(\overline{P}) = \delta p^2 \frac{(1 - \bar{r}A)}{A}$$

For known values of V ($\overline{P}$) i.e. the level at which accuracy is desired, p, and $\bar{r}$A, the total number of key station 'n' could be found out.

W.M.O. publication No. 324 (1972) gives the following statement in tabular form (Table II.1.2) as well as in graph shown in Figure.II.1.3 to arrive at optimum number of reporting stations based on standard relative error criteria 'E' of average storm rainfall area catchment relationship to basin area 'A', thunderstorm frequency and mean annual runoff in the basin

$$E = \frac{7.7\,(A)^{0.2}}{(N)^{0.48}}$$

Table. II: 1.2: Estimation of Optimum Number of Precipitation Net work.

Curve No.		*Thunderstorm days per year*		
		< 30	*30-45*	*> 45*
Runoff cm.	>15	1	2	3
per year	≤15	2	3	4

(B) For stream-gauging network

The network design of stream gauging stations for forecasting purpose is relatively simple for two reasons:

(a) The real time reporting, operational stream gauges have to be selected from existing stations, should a forecast procedure based only on correlation of upstream to downstream stage/discharges be selected, or a rainfall-runoff model be calibrated by past observed data.

(b) New Stations are selected predominantly at points for which forecast are needed (the forecast points), so as to permit easy updating and evaluation of the forecast.

2.1.2.3. Cost-Benefit considerations

The economic worth of data collected on real time basis for operational flood forecasting may be determined by corelating the worth of additional data to the benefits foregone as a result of not having that additional data. The benefits foregone in turn are related to the uncertainties in the estimates of value of forecast. W.M.O. publication No. 341 describes benefit and cost analysis of hydrological forecast. Harold J. Day and Walter T. Sittner of U.S.A. have described the methodology on the use of hydrological forecast benefit procedures in lesser developed nations based on flood damage-frequency relationship developed after rigorous efforts to collect data required for the analysis.

2.1.2.4 Criteria for location of key stations

The location of key gauging stations are determined on consideration of the features of the hydrologic region (i.e. by considering the homogeneity of the precipitation, temperature, physiography, and land use), river systems, bifurcations and confluences, international and inter state boundaries, towns, major industries, flood plains etc. Each station has to offer some relevant and useful information and/or jointly with the other stations.

(A) For gauge observation only

(i) At the head of plains on main river and major tributaries below which water spills over natural banks when discharge attains maximum value. For mountainous rivers it is the point where the river leaves the mountain area.

(ii) At places of significant changes in discharge i.e. where excessive gains and losses occur in the river flow.

(iii) To afford more accurate forecast though with reduced time of warning.

(iv) Near places where branches join or separate.

(v) At intermediate points in between two existing stations where distance between the two stations is long.

(vi) At each important tributary near the foothills, in the middle reach and near the confluence with main river but before the point upto which back water effect of main river is felt.

(vii) On less important tributaries, a single gauge in the lower reach.

(viii) The number of self recording gauges should be increased rapidly particularly where needed and where areas are inaccessible.

(ix) Near important cities, towns, industries and very fertile land subjected to frequent heavy flood damages.

(x) Where forecast is being issued on the basis of gauge to gauge correlation the base station and forecasting station must be equipped with gauge.

(xi) In case more than one tributary join the main stream and the forecast is based on multiple coaxial diagram, there should be atleast one gauge on all the tributaries.

(xii) The location of stations on the tributaries should be such that the time of travel from base station to forecasting station in respect of tributaries and main stream is generally same.

(xiii) Where the flood routing model is the basis of formulation of forecast the reaches should be smaller in length and homogeneous in nature such that the assumptions made in the concept of flood routing theory are not violated.

(B) For gauge and discharge observations

(i) At the outlet of main river, major tributaries and important small tributaries.

(ii) At the outlet of each hydrologic region.

(iii) Just below the snowline to assess the snowmelt.

(iv) Near international and inter-state boundaries.

(v) At upstream and downstream of an effluent or influent seepage reach incurring heavy losses or gains on main river and major tributaries.

(vi) On main river and major tributaries upstream or downstream of points of significant permanent withdrawals.

(vii) To reduce ungauged very long reaches on main river and major tributaries.

(viii) Just downstream of confluences of main stream with major tributaries.

2.1.2.5. Priority in network establishment

The priority to establish one particular station over other stations should be decided according to the usefulness of the information supplied by that station. The gradual process of selection of key stations for precipitation measurement may be adopted as below:

At first correlation coefficients between the average of the storm rainfall and individual station rainfall are found. The stations are then arranged in order of their decreasing correlation coefficient and the stations exhibiting higher correlation coefficient are given priority in the network establishment.

Similarly, for river gauging stations, the priority may be given where a good relationship exists for gauge to gauge correlation.

2.1.2.6. Precision and frequency of data measurement

W.M.O. have recommended guide lines for the accuracy and frequency of data measurement which specifies suggested goals for data measurement for the purpose of flood forecasting. These guidelines are presented in Table. II. 1.3

Table. II. 1.3: Desirable precision of observation and frequency of data measurement for Hydrological Forecasting

Element	Precision	Reporting interval	Measure by automatic land station
Precipitation - Total	± 2 mm below 40 mm	6 hours[3]	Yes
amount and form 2	± 5% above 40 mm		
River stage	± 0.01 mm	6 hours5	Yes
Lake level	± 0.01 m	Daily	Yes
Soil moisture	± 10% field capacity	Weekly	Yes
Frost depth	± 2 cm below 10 cm ± 20% above 10 cm	Daily	Yes
Water equivalent of Snow on ground	± 2 mm below 20 mm ± 10% above 20 mm	Daily Daily	Yes
Depth of snow cover	± 2 cm below 20 cm ± 10% above 20 cm	Daily	Yes
Density of snow cover	± 10%	Daily	—
Water temperature[6]	± 0.1°C in 0-4°C — range	Daily	Yes
(rivers and lakes)	otherwise ± 1°C		
Surface temperature snow	± 1°C	Daily	Yes
Temperature profiles (Snow and lakes)	± 1°C	Daily	Yes
River and lake ice	± 0.02 m below 0.2 m ± 10% above 0.2 m	Daily	—
Water level (in wells)	± 0.02 m	Weekly	Yes

Element	*Precision*	*Reporting interval*	*Measure by automatic land station*
Net radiation	± 0.4MJm^{-2}/day below 8 MJm^{-2}/day ± 5% above 3MJm^{-2}/day	Daily	Yes
Air temperature	± 0.1°C	6 Hours	Yes
Wet bulb temperature	± 0.1°C	6 Hours	Yes
Wind-movement	± 10%	6 Hours	Yes
Pan evaporation	± 0.5 mm	Daily	Yes

(1) With respect to actual observations the WMO Technical Regulations use the term "precision of observation or of reading" which is defined as the smallest unit of division on a scale of measurement from which a reading, either directly or by estimation, is possible.

(2) It may be necessary to distinguish solid and liquid forms of precipitation.

(3) Varies from one hour to one day, depending on river response. Event reporting for example, after 2 mm of rain required for flash flood forecasts.

(4) Depends on sensitivity of stage discharge relationship to stage change and can be ± 1 mm accuracy. If possible an accuracy characterised by a relative standard deviation of ± 5 per cent should be arrived at.

(5) See note 3. Event reporting may be appropriate for flash flood forecasts.

(6) Hourly reporting with ± 0.3°C for ice forecasting.

However many years of experience dealing with data collection has shown that quality control of observation is one of the most difficult things to achieve because of practical constraints as mentioned below:—

(1) Hydrological data are to be collected from remote regions, especially in the head water reaches.

(2) Poor initial training, lack of observer motivation, in-efficiently maintained instru-

ments, no follow up procedures to correct observer errors and deficiencies, and lack of quality control when data is processed.

(3) Stream channels erode their banks having loose sand during high floods, thereby making it difficult to instal permanent river gauges. Temporary gauges are frequently washed away.

(4) Gauge observers are very poorly paid for the work. They do not understand the importance of the data they are collecting and some of them may lack interest and motivation for correct observation of data and honesty in observation of data at every hour throughout 24 hours of the day.

(5) Generally the gauge observation sites are established after clearing the jungles and bushes on both banks of the river. But after few years the people of rural mass make a practice to burn dead bodies and perform funerals near such sites because the location of the site is neat and clean. At such places the gauge observers are afraid of even going near the gauge in the night hours because of psychological effects. If the data is not checked during processing in the control room, the flood forecast based on such data will be like "Garbage in-garbage out".

(6) There are also many instances where observers take the observations at a time other than that authorised. This invalidates the synoptic aspects of data when using it to draw isolines, for example.

The above mentioned problems are accentuated when the data must be collected on real time basis such as for operational flood forecasting.

2.1.2.7 Remote Sensing

The need for a detailed and rapid spatial coverage in real time data collection gives particular importance to remote sensing of data. According to W.M.O., remote sensing from ground based radar, satellite, and aeroplanes, offers many distinct advantages for certain classes of forecast, primarily because of possibilities of directly observing areally extensive variables which are otherwise only amenable to point sampling, and providing observations over inaccessible terrain. It can be used to provide direct input to forecasting procedures in the following areas:—

(a) Areal rainfall, both qualitative and quantitative;

(b) Areal extent of flood plain inundation;

(c) Tropical cyclone or hurricane movement; and

(d) Area of snow cover.

Telemetry system of data collection has been found to be very useful in real time data collection. However, the major limitation in installing these systems are their cost. However the state of art of telemetry system has progressed rapidly and has enabled widespread adoption of such techniques round the world.

2.1.2.8 Man-Machine mixed approach

The elimination of man as an observer and transmitter in real time data collection sub-system has some inherent disadvantages. Man has a great mental ability to integrate information and supplements the purely factual and numerical report provided by the sensor. In addition it may cause increase in unemployment to rural masses who await flood season to get job of observers. Under Indian conditions, man-machine mix approach of real time data collection would be most appropriate. Hydrological Stations which are easily accessible throughout 24 hours without any hindrance (mental as well physical) to the observers specially in cities and towns could be manually operated. However most of the sites in villages which are far away from flood forecasting control rooms should be semi-automated so as to ensure reliable data especially during night times and take care of conditions when staff gauges are washed away and observer has to refix the gauges without knowledge of levelling with respect to G.T.S. bench mark. It happens sometime that when the observer reaches in the morning to read the stages he does not find a gauge and fixes another staff gauge arbitrarily. In such cases, errors in stages as high as 1 m have been observed in the field.

In very remote areas which are inaccessible, fully automated self reporting data collection system may be installed.

2.2 Observation network design

As discussed earlier, the data requirement depends upon the specific purpose. There is considerable variation in hydrological and hydrometeorological variables in time as well as in space. While the variations in time are duly accounted by selecting a suitable time interval, the spatial variation is accounted for by establishing the observation stations at sufficient number of locations so that the desired characteristics are duly represented by the observed data.

The choice of suitable stations for various variables is made through proper observation network design.

2.2.1 Raingauge network

Rainfall is one of the basic data required for flood forecasting purposes. Even in case of a long river where the forecasting methods are essentially based on gauge and discharge data, the rainfall data from various stations in the intermediate catchment/catchments are very essentially used as a parameter resulting in marked improvement. So far forecasting in small and flashy rivers is concerned, the techniques of forecasting are primarily based on rainfall runoff method which requires rainfall data from sufficient number of stations in the catchment. The number of raingauge stations in the basin should be such that:

(a) the areal rainfall in the catchment can be estimated with desired accuracy; and

(b) the variation in the areal distribution as well as time distribution can be identified.

Estimation of the number and locations of the raingauge stations which will provide sufficient information regarding rainfall falling over the catchment is referred as Optimal Network Design. For development of a flood forecasting technique, it is desirable that data from all these stations are collected. However for operational purpose the real time collection of rainfall data from all these stations may not be possible because of many factors such as transmission factors, lack of time etc. It is therefore desirable that the number of raingauge station be reduced to the minimum without sacrificing much accuracy in computation of areal average rainfall. These raingauge stations constitute Key or Representative Raingauge Stations.

The detailed procedure for design of Optimal Network as well as Key Network are discussed below:—

2.2.1.1 Design of optimal raingauge network

The design of Optimal raingauge network is intended:

(i) to find out the number of raingauge stations required for the purpose: and

(ii) to suggest suitable locations for these stations in the concerned river basins.

For the area where some raingauges already exist, the optimal raingauge network can be designed by using the coefficient of spatial variation of the rainfall. The optimum number 'N' of raingauge is obtained by using the following relation.

$$N = (C_v/P)^2$$

Where C_v = Coefficient of variation of rainfall based on existing raingauge stations; and

P = percentage error in the estimate of basic rainfall

Alternatively, the existing coefficient of variation of rainfall and the required coefficient of variation can also be utilised for determining the optimal network. If C_{ve} and C_{vr} are the existing coefficient and the required coefficient of variation and 'n' and 'N' are the existing and required number of raingauges, then

$$N = n\frac{V_{ve}}{C_{vr}}$$

Example.II. 2.1

In a particular catchment, there are six raingauge stations as shown in the fig. II. 2.1.

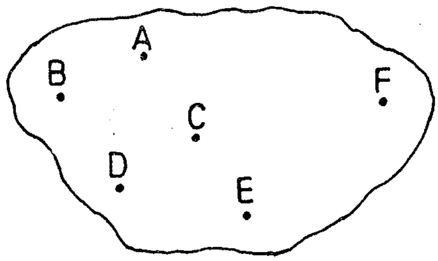


Fig. No.II.2.1

The average annual precipitation at the various stations are given below:—

Station	*Rainfall*
A	700 mm
B	830 mm
C	650 mm
D	710 mm
E	600 mm
F	400 mm

Estimate the Optimal number of raingauge station in the basin. The allowable error may be taken as 5%.

Solution :

$$\bar{P} = \text{Mean of the rainfall} = \frac{\Sigma P}{n}$$

$$= \frac{700+830+650+710+600+400}{6}$$

$$= \frac{3890}{6} = 648\,\text{mm}$$

σ = standard deviation

$$\sigma = \frac{\sqrt{\Sigma(P-\bar{P})^2}}{n-1} = \frac{\sqrt{103484}}{6-1} = 143.86$$

C_v = Coefficient of variation

$$= \frac{\sigma}{\bar{P}} = \frac{143.86}{648} = 0.222$$

Therefore $N = \frac{(0.222)^2}{0.05} = (4.44)^2 = 19.7$ Say 20.

And hence 20 No. raingauge stations should be provided in the catchment if an accuracy upto 5% is desired.

Location of raingauge stations:- In order to locate the station in the catchment the following procedure is adopted:-

(i) Draw isohyets as shown in the figure. II.2.2. It may be seen that various isohyets divided the whole basin area into ten zones.

(ii) The location of raingauges will be such that equal number of raingauge stations are located in each zone i.e 20/10 =2 Raingauge stations in each zone e.g. in zone (I) , one raingauge station already exists and hence only one more is to be established in this zone.

In case of zone (II), 2 raingauge stations are to be established. Similarly the requirement of raingauges in various zones will be determined.

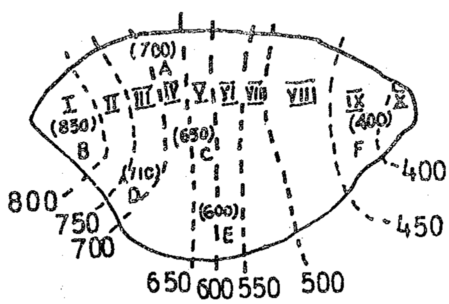


Fig No. II.2.2

(iii) The exact location should be decided keeping in view the following points.

(a) The raingauge station should be located near a village or town.

(b) The site should be accessible throughout the year.

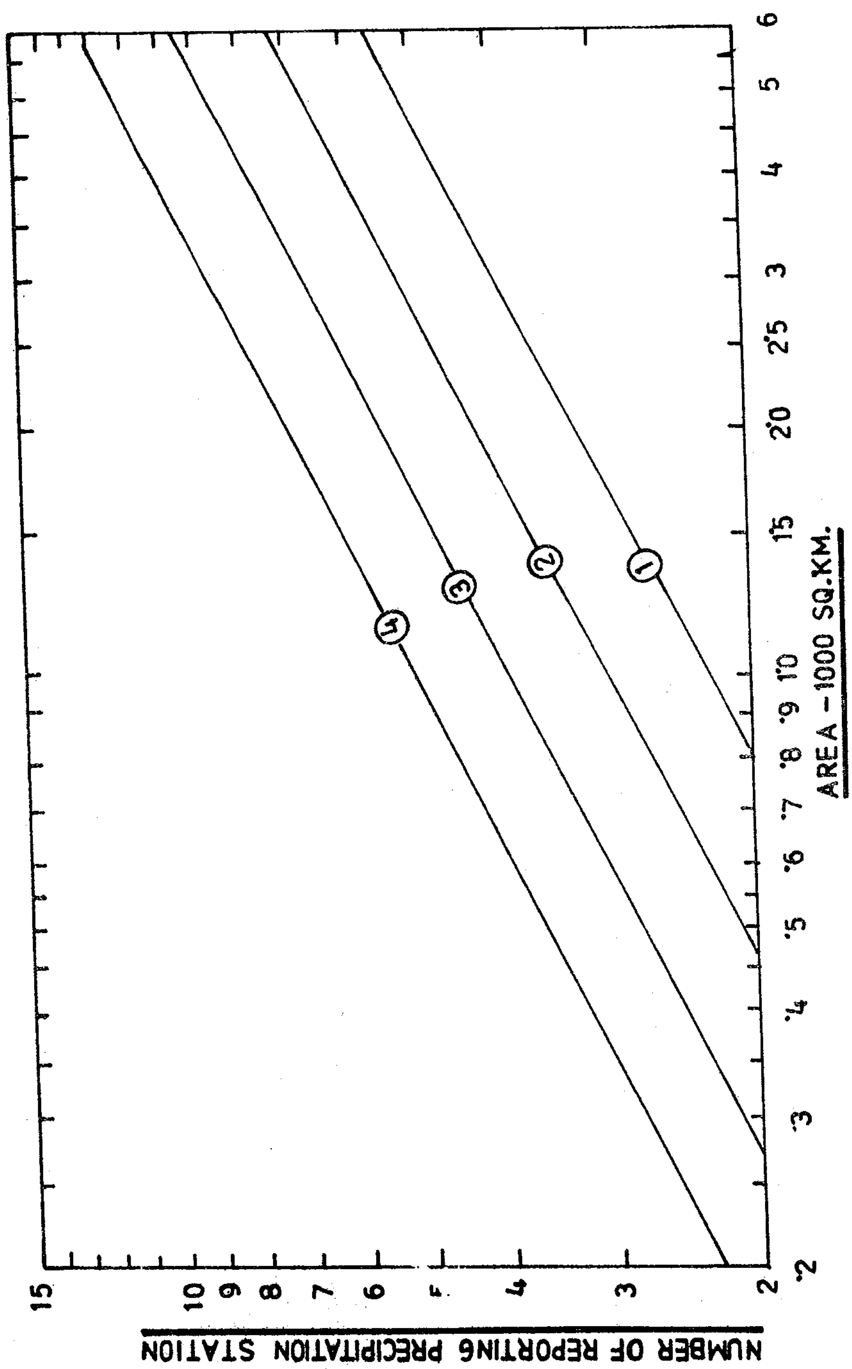


Fig.No.II.2.3 DERIVATION OF OPTIMUM PRECIPITATION NET WORK AS PER W.M.O.

(c) The distribution as a whole should be uniform over the catchment area.

(d) As far as possible each of the sub-catchment should be proportionate to the number of raingauge stations.

2.2.1.2. Determination of Key Station Network

One of the most rational method for determination of the Key Station network is as suggested by Hall (1972). The procedure used for the determination of Key Station network is discussed here.

If P_a be the rainfall required to be estimated from the observed record at selected station $X_1, X_2, X_3, \ldots\ldots\ldots\ldots\ldots X_n$ then P_a can be determined by

$$P_a = C + A_1X_1 + A_2X_2 + \ldots\ldots\ldots\ldots\ldots\ldots + A_nX_n$$

Where, $A_1\ A_2 \ldots\ldots A_n$ are called the regression coefficients and C is a constant, known as intercept.

The method involves a gradual process of elimination of stations. The steps are as follows:

At first correlation coefficients between the average of the storm rainfall and the individual station rainfall are found. The stations are then arranged in order of their decreasing correlation coefficient and the stations exhibiting higher correlation coefficient are considered for further analysis. The next step is to determine the independent order of correlation. For this the station showing the highest correlation coefficient among the stations considered is called the first key station and its data is removed for the determination of second key station. The procedure is repeated by considering the average rainfall of the remaining stations. The station showing the highest correlation coefficient after removing the data of first station is called the second key station. Similarly, the third and further key stations are selected after removing the data of already selected key stations.

As each station gets added to the key station network the total amount of variance which is accounted for by the network at that stage is determined. This would provide a basis for determining the number of stations required for achieving an acceptable degree of error in the areal estimate.

The multiple correlation coefficient increases with the increase of the number of station in the combination and the sum of the squares of the deviations of the estimated values of average rainfall from actual as well as the minimum deviation decreases. A stage will then be reached when improvement in either the multiple correlation coefficient or the sum of the square of deviations will be little. The corresponding number of raingauges at that stage will be taken as the representative network for the purposes of determining areal estimate of rainfall. The next step is to test several combinations of the number of key stations which gives a satisfactory estimate of the areal rainfall. Thus a number of alternative key stations network are determined which may be employed to give estimates of areal rainfall taking into consideration, the possibility of any of the stations in the best combination not reporting the rainfall data.

Alternatively, the key raingauge network can be established by the simple correlation analysis where the rainfall data from the various stations in and around the catchment are correlated with the total runoff, if the correlation coefficient 'r' is less than 0.60, it is best to reject the station as it will reduce the reliability relationship. In case all of the stations exhibit a low correlation coefficient it means that none of the methods for determining the areal estimate of rainfall will work very well. In such a situation it will be desirable to relocate the raingauges.

Various steps, involved in the analysis, are illustrated in Examples II. 2.2. and II. 2.3.

Example II.2.2.

For the catchment of river Brahmani upto Talcher the various storms were analysed and the rainfall observed during these storms at various stations is given in Table II.2..1 Out of the 13 raingauges stations whose data are given above, select Key raingauge stations by using the method suggested by Hall.

Solution:

The key raingauge stations will be determined by using the technique as discussed above. The various steps are explained below:

Step. No. 1

Calculate the average rainfall for each storm i.e. for each storm add the rainfall data of all the 13 stations and divide by 13, the values are given in the last row of Table. II.2.1.

Step. No. 2

Compute the coefficient of correlation between the rainfall data of Panposh (Row. No.1) and the average rainfall (last row). The coefficient of correlation works out to be 0.3296. Similarly compute the correlation coefficient between rainfall data of other stations and the average rainfall.

Table II. 2.2. gives the coefficient of correlation between the data for various stations and the average rainfall.

TABLE. II. 2.1

TABLE SHOWING THE TOTAL R.F. IN DIFFERENT STORMS: ALL R.F. IN mm.

Sl. No.	Name of R.F. Stations	Storm No.												
		1	2	3	4	5	6	7	8	9	10	11	12	13
1.	Panposh	141.5	136.6	55.6	160.0	49.0	123.6	100.7	42.2	120.8	20.8	26.6	12.4	44.2
2.	Bonaigarh	199.0	180.0	103.5	109.0	122.0	52.5	73.0	92.5	10.0	56.0	71.5	10.5	42.0
3.	Chhendipada	87.7	150.0	56.0	51.0	0.0	15.0	46.0	84.0	25.0	84.0	15.5	178.0	60.5
4.	Pallahara	80.0	176.3	80.3	288.0	57.0	46.0	82.8	227.5	63.0	106.7	89.0	71.0	136.0
5.	Kamakhya Nagar	24.4	158.0	22.0	99.0	4.1	50.0	33.4	51.0	19.8	80.7	37.4	122.4	69.0
6.	Talcher	23.4	241.8	60.3	80.7	10.1	36.10	32.7	119.3	30.4	161.4	12.1	210.9	79.7
7.	Dhenkanal	83.2	170.2	74.7	91.9	5.6	50.5	21.5	49.1	53.3	126.3	107.9	163.8	38.1
8.	Sukinda	55.3	77.0	27.8	98.8	4.8	49.5	35.5	95.3	48.7	28.9	19.0	137.5	50.0
9.	Deogarh	216.0	121.4	61.5	83.4	26.2	26.2	25.8	93.1	33.2	28.8	90.6	19.0	201.6
10.	Raemal	59.5	138.2	102.2	78.0	2.5	16.5	59.0	194.0	69.0	36.0	61.0	110.6	77.0
11.	Lohardaga	27.4	11.4	182.4	84.6	13.0	171.8	61.6	26.2	87.8	77.0	136.2	26.0	235.0
12.	Manoharpur	139.0	100.0	16.6	186.6	48.0	139.0	322.0	26.5	62.0	55.0	43.8	25.5	54.0
13.	Angul	19.8	138.2	73.0	32.2	4.6	27.0	46.0	137.1	44.9	70.6	8.4	172.2	28.2
Total R.F. (in m.m.s)		1156.2	1799.1	915.9	1443.2	346.9	803.7	940.0	1237.8	676.4	932.2	719.0	1259.6	1145.3
Average R.F.		88.938	138.392	70.438	111.0	26.68	61.82	72.30	95.21	51.41	71.70	55.30	96.89	88.1

Table : Coefficient of Correlation between the Data for Various Stations and Average Rainfall

Table II.2.2.

Sl.No.	*Raingauge Station*	*Coefficient of Correlation*
1.	2	3
1.	Panposh	0.3296
2.	Bonaigarh	0.3589
3.	Chendipada	0.7647
4.	Pallahara	0.6747
5.	Talcher	0.7313
6.	Kamakhya Nagar	0.8304
7.	Dhenkanal	0.6224
8.	Sukinda	0.7131
9.	Deogarh	0.4305
10.	Reamal	0.6711
11.	Lohardaga	0.2095
12.	Manoharpur	0.1426
13.	Angul	0.6608

From above it may be seen that the best correlation is that of Kamakhya Nagar (0.8304), therefore this station is the first key station of the network.

Step No. 3

Delete the data of this station from Table II.2.1. and calculate the average rainfall on the basis of remaining 12 stations and the earlier values of the mean rainfall as given in the last row of Table II.2.1. is replaced by respective average values.

Step.No.4

Again calculate the coefficient of correlation between rainfall of each station and the average rainfall as in step No.2

The values of respective correlation coefficients are given below:

1.	Panposh	0.3544
2.	Bonaigarh	0.3905
3.	Chhendipada	0.7454
4.	Pallahara	0.6831
5.	Talcher	0.6902
6.	--	--
7.	Dhenkanal	0.5838
8.	Sukinda	0.6999
9.	Deogarh	0.4668
10.	Reamal	0.6863
11.	Lohardaga	(-) 0.2038
12.	Manoharpur	0.1604
13.	Angul	0.6083

Now it may be seen that Chhendipada has the highest correlation coefficient (0.7454) with the average rainfall. This is then the second key raingauge station.

Similarly the 3rd, 4th and 5fth & so N key stations are found out.

Table II.2.3 Coefficient of the Regression Equation

Various Combi-nations	*Kamakhya Nagar*	*Chhendi-pada*	*Pallahara*	*Sukinda*	*Talcher*	*Deogarh*	*Panposh*	*Intercept*	*MCC*
1	2	3	4	5	6	7	8	9	10
1	0.5277	–	–	–	–	–	–	47.78	0.81
2	0.3787	0.1653	–	–	–	–	–	45.94	0.82
3	0.1594	0.2571	0.1756	–	–	–	–	32.45	0.91
4	0.1593	0.2788	0.1837	(–)0.0462	–	–	–	32.68	0.91
5	0.4211	0.5297	0.1985	(–)0.1596	(–)0.2847	–	–	29.49	0.93
6	0.3714	0.3936	0.1645	(–)0.0772	(–)0.1891	0.5628	–	28.10	0.93
7	0.2152	0.3361	0.1436	(–)0.0678	(–)0.0541	0.0614	0.1246	21.26	0.95

Table II.2.4 Regression Coefficients

Various Combi-nations	Kamakhya Nagar	Chhendi-pada	Pallahara	Deogarh	Panposh	Reamal	Manohar-pur	Intercept	MCC
1	2	3	4	5	6	7	8	9	10
8	0.2293	0.1001	0.1366	0.1031	–	–	–	28.41	0.93
9	0.1763	0.2585	0.1255	0.0786	0.1283	–	–	20.73	0.97
10	0.2486	0.1633	0.0729	0.3254	0.1395	0.1078	–	19.21	0.97
11	0.2748	0.1367	0.5601	0.0954	0.0905	0.1413	0.0489	17.11	0.98

Step. No. 5.

Now arrange the stations determined in step 3 and 4 serially in the order as they are found as given below:

Sl.No.	Key Station	Sl.No.	Key Station
1.	Kamakhaya Nagar	8.	Reamal
2.	Chhendipada	9.	Manoharpur
3.	Pallahara	10.	Dhenkanal
4.	Sukinda	11.	Lohardaga
5.	Talcher	12.	Bonaigarh
6.	Deogarh	13.	Angul
7.	Panposh		

Step No. 6

Now the various combinations of stations i.e. 1st + 2nd, 1st + 2nd + 3rd, 1st + 2nd + 3rd + 4th, etc. are considered to constitute the key network and Multiple Correlation Coefficient (MCC) are found out in each case. Table II.2.3 gives the MCC for different combinations of stations alongwith the coefficient of the respective regression equations and their intercept.

It may be seen from Table II.2.3. that the Multiple Correlation Coefficient (MCC) more or less increases with addition of R.G. Station to the Key network. It is seen that the coefficient in respect of stations Sukinda and Talcher are always negative which is physically not justified and hence if these two stations are omitted, then the MCC improves considerably as is indicated in Table. II.2.4.

From Table II.2.3 & II.2.4 it may be seen that MCC in case of combination 9 is 0.97 and there is not much improvement in the MCC by further increasing the number of stations.

Hence the raingauge stations in combination 9 i.e. Kamakhaya Nagar, Chhendipada, Pallahara, Deogarh and Panposh (5 Nos.) may be considered to form key raingauge network.

Example II. 2.3.

The simple correlation between the total rainfall and run off has been used to select representative key raingauge stations for Baitarani basin. Rainfall from twelve different raingauge stations in and around the catchment have been used for the analysis. They are Anandpur, Swampatna, Champua, Ghatgaon, Karanjia, Keonjhar, Thakurmunda, Chandbali, Udala,Kaptipada, Bariapada and Pallahara (Fig. II.2.4). The total rainfall (in mm) recorded at each of the 12 stations during the monsoon period for the years 1973 to 1978 have been computed and are tabulated in Table II.2.5.

The total runoff, (in mm) at Anandpur during monsoon period has been computed for all the years with the help of discharge data. The correlation between the runoff and the rainfall has been worked out for each of the rainfall stations and listed in Table. II .2.5.

Based on the correlation coefficients thus obtained, the following rainfall stations may be included in the forecast development programme.

	Raingauge Stations		*Correlation Coefficient*
1.	Anandpur	..	(0.931)
2.	Udala	..	(0.922)
3.	Swampatna	..	(0.895)
4.	Thakurmunda	..	(0.833)
5.	Keonjhar	..	(0.827)
6.	Champua	..	(0.822)
7.	Karanjia	..	(0.874)
8.	Baripada	..	(0.866)

It is observed from the Table. II.2.5 that Ghatgaon (r=0.35) is so poorly correlated that the station may be abandoned, as serving no useful purpose.

2.2.2. Design of Hydrological Observation Network

The river flow depends upon a number of factors including the basin characteristics, geological features of the catchment, topography, meteorological conditions, storage characteristic etc. These factors vary considerably from place to place. As a result the runoff characteristics of different locations exhibit marked variations. The hydrological observation network should be designed in such a way that these variations are duly represented in the collected data. Because of considerable variations, it is not possible to suggest a standard norm for establishing

Table II. 2.5 Total rainfall from 1973 to 1978 and Correlation Coefficient

Sl. No.	Name of Rain fall Station	Rainfall during Monsoon period in mm						Correction Coefficient (No. Years Data)		
		1973	1974	1975	1976	1977	1978	6 yrs.	5 yrs.	4 yrs.
1	2	3	4	5	6	7	8	9	10	11
1.	Swampatna	1893	1275	1233	702	855	1175	0.895	-	-
2.	Anandpur	2102	1068	1433	973	1354	1228	0.931	-	-
3.	Champua	-	910	1200	915	1128	1112	-	0.822	-
4.	Ghatgaon	1082	1112	1418	927	968	1195	0.354	-	-
5.	Karanjia	1743	1183	-	819	948	1002	-	0.859	-
6.	Keonjhar	1515	816	606	671	508	920	0.827	-	-
7.	Thankurmunda	1859	1191	1809	983	1252	1228	0.835	-	-
8.	Chandbali	1622	1071	1550	1102	1036	1015	0.725	-	-
9.	Udala	1906	1148	1394	1140	1446	1699	0.922	-	-
10.	Kaptipada	-	1121	1580	1105	1485	-	-	-	0.876
11.	Baripada	2269	1223	1724	1149	1903	1774	0.866	-	-
12.	Pallahara	2209	1178	700	856	1563	-	-	0.700	-
	Runoff at Anandpur	919	433	615	352	473	637			

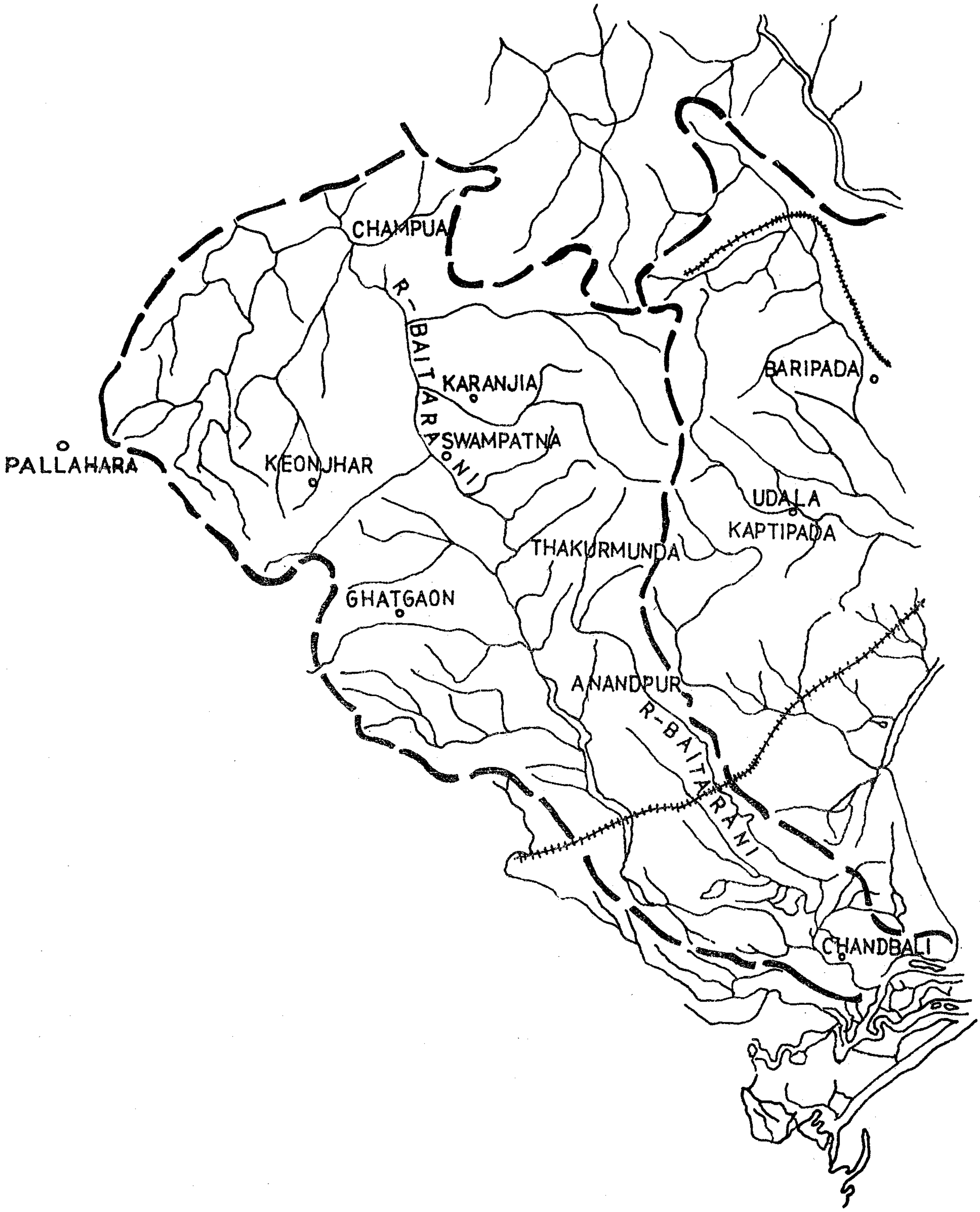


Fig. No.II. 2.4 CATCHMENT MAP OF BAITARANI RIVER BASIN the hydrological observation network. However, for the purpose of overall assessment of the water resources of the region the W.M.O. have suggested a very broad guideline which is reproduced in Table II. 2.6.

Table II. 2.6. Minimum Density of Hydrometric Networks

	Type of region	*Range of norms for minimum net work (Area (km^2) per station.)*	*Range of provisional norms tolerated in difficult conditions. (Area (km^2) per Station)*
I.	Flat regions	1,000-2,500	3,000 -10,000
II.	Mountainous regions	300 -1,000	1,000 - 5,000(4)
	Small mountainous islands with very irregular precipitations, very dense stream network.	140 - 300	
III.	Arid and polar Zones (2)	5,000 - 20,000 (3)	

1. Last figure of the range should be tolerated only for exceptionally difficult conditions.
2. Great deserts are not included.
3. Depending on feasibility
4. Under very difficult conditions this may be extended to 10,000 km^2

A sufficiently detailed network is necessary to achieve the objective of identifying the various parameters representing the basin and other characteristics to be used in the rainfall-runoff modelling. Since the basin characteristics go on changing with time, it becomes necessary to maintain a certain number of hydrological observation stations on long term basis so that the pattern of changes in the characteristics could be established. It is in this background that three types of hydrological observation stations viz. base stations, secondary stations, and temporary stations are considered in any network design of hydrological observations. The base stations are of permanent nature and continue on long term basis. The secondary stations are established with the sole objective of observing sufficient data necessary for identification of parameters of a model which can be used to assess the river flow from other variables such as rainfall, water level etc. The temporary stations are established to observe data necessary for achieving some specific objectives such as planning of a water resources development project, at international and inter-state boundaries etc.

However, the above guidelines are mainly for the purpose of assessment of riverflow and they can not be adopted as such for the design of network for the purpose of flood forecasting. In case of flood forecasting the choice of sites are to be made on entirely different considerations.

2.2.2.1 Location of Hydrological Observation Stations for Flood Forecasting

Some of the factors which are to be given due consideration in the selection of Key hydrological observation stations to be used for operational flood forecasting are as follows:

1. Flood prone towns /villages.
2. Major industrial complex in flood plains.
3. River system.
4. International and inter-state boundaries.
5. Hydraulic regions.
6. Major hydraulic structures.

For the purpose of flood forecasting, the hydrological observation stations (gauge site and gauge and discharge site) are to be selected at following locations:

Gauges only.

(1) At the head of plains on main river and major tributaries below which water spills over natural banks when discharge attains maximum value. For mountainous rivers, it is the point where the river leaves the mountain area.

(2) At places of significant changes in discharge i.e. where excessive gains and losses occur in the river flow.

(3) Near places where branches join or separate.

(4) At intermediate points in between two existing stations where distance between the two stations is long.

(5) At each important tributary near the foothills, in the middle reach and near the confluence with the main river but before the point upto which backwater effect of the main river is felt.

(6) On less important tributaries, a single gauge in the lower reach.

(7) Near important cities, towns, industries and very fertile land subjected to frequent heavy flood damage.

2.2.2.2 For Gauge & Discharge Observation

(1) At the outlet of main river, major tributaries and some small tributaries.

(2) At the outlet of each hydrologic region.

(3) Just below the snowline to assess snowmelt.

(4) Near international and inter state boundaries.

(5) Upstream and downstream of an affluent or influent seepage reach.

(6) Upstream and downstream of significant permanent withdrawals.

(7) On main river downstream of the confluence of tributaries.

2.2.3. Network of other Hydrometeorological Data

The rainfall and runoff constitute the primary input for any flood forecasting model. But at the same time, information in respect of other meteorological and climatological data is also needed. These data are needed both for estimation of the various parameters of the rainfall - runoff model as well as for operational use.

Some of the most commonly used variables are:

(a) Temperature;

(b) Humidity;

(c) Evaporation;

(d) Evapotranspiration;

(e) Sunshine hour; and

(f) Wind direction and velocity.

As a matter of fact, these variables are mostly required for the purpose of estimation of the parameters of flood forecasting model for which the available record of historical data are needed. Further, some of the variables such as temperature and wind direction etc. are generally available from a number of stations. Apart from this, the variation in these data is not very large and therefore it is not necessary to have the data from a very dense network of the stations. Unless these data are exclusively used in operational forecasts, the minor variations in the observations are not of much significance.

In this country, India Meteorological Department has established a network of different class of observatory from where these data can be collected. A brief introduction about the utility of these data and their specific requirement is given below.

A. Temperature

In case of flood forecasting models, the temperature constitutes a very important variable where snowmelt is an important component of the model. Here the variation in temperature plays a very important role. However, for most of the models the daily maximum and minimum temperature is more than enough. As a matter of fact this variable is not directly used in any of the forecasting models currently under use in this country. However, the existing network of stations having record of temperature data are quite sufficient for the purpose of flood forecasting.

B. Humidity

The humidity is also generally used for the purpose of estimation of parameters of the models and is not used as an input for operational system. For a majority of the models, the average monthly values of the relative humidity are used for the purpose. The day to day values of the relative humidity are rarely used in any model.

C. Model Evaporation and Evapotranspiration

Evaporation data are needed to assess rates of losses of water from reservoirs and in studies of the water budget of catchments. The need for evaporation data increases with the degree of aridity. In arid regions, one evaporation station in each 30,000km^2 is recommended. In humid temperate regions, one station in 50,000 km^2 is sufficient. In cold regions one station in 1,00,000 km^2 is recommended for the minimum network.

An evaporation station consists of a pan of standard design where observations of rates of daily evaporation are made, together with daily observations of precipitations, and minimum water and atmospheric temperature, wind movement and relative humidity or dew point temperature.

2.2.4. Data network in India - A Review

The observation, collection, compilation and analysis of hydrological and hydrometeorological data for formulation of flood forecast for almost all the major river systems of India is being carried out by the Central Water Commission. The Central Water Commission started the flood forecasting services in the country with establishment of first scientific flood forecasting unit at Delhi in November 1958 which issued the first forecast for the flood in Yamuna at Delhi Bridge on 25th July 1959. Subsequently the flood forecasting services were introduced to various Inter-State rivers in the year 1969. Initially, the observation of hydrological data used to be the responsibility of other departments or other units of Central Water Commission and the Flood Forecasting Organisations was basically responsible for formulation of forecast with the help of the data observed by other agencies and those observed by Flood Forecasting Units exclusively for the purpose of forecast. In 1978, the work of hydrological observation and flood forecasting in the country was reviewed and the various organisations responsible for the work were re-organised However, the State Government continue to have their own observation network for the specific purposes like preparation of project report for specific water resources development schemes etc.

At present, the Central Water Commission under the Ministry of Water Resources is issuing forecasts for 147 sites along the various rivers of the country. There are about 702 Gauge and Discharge sites for collection of hydrological data.

The salient features of the hydrological observation and flood forecasting activities of the Central Water Commission are given in Appendix II (1)

The names of various forecasting sites are listed in Appendix II (2)

2.3. Observation Techniques

The quality of data is very important and to maintain the quality, it is necessary to follow the recommended observation techniques for the various variables. In this chapter, the observation techniques in respect of commonly used data i.e. water level, discharge, rainfall, snowfall, temperature, humidity, wind velocity and direction etc. are briefly discussed.

2.3.1. Water Level Observation

In flood forecasting operations, the need for correct observation of the river stages can hardly be over-emphasised as at most of the forecasting stations, the forecasts are formulated on the basis of correlations developed between the gauges at the upstream and downstream stations with or without some additional parameters. When combined with discharge measurements, it helps in developing stage-discharge relationship for computing the discharges at different river stages without resorting to costly and cumbersome process of manual measurements.

There are various types of gauges in use and the most commonly used are:

1. Manually Operated Gauges:

(i) Staff Gauge;

(ii) Sloping Gauge or Stepped Gauge;

(iii) Cantilever Gauge; and

(iv) Gauge wells provided in major hydraulic structures.

2. Continuous River Stage Recorders:

(i) Float Type; and

(ii) Pneumatic Type.

2.3.1.1. Staff Gauge

Non-recording graduated staff gauges are generally used for the measurement of river stage. These gauges are commonly made of well seasoned wooden posts of size of 0.150 m x 0.100 m x 3.00 m and founded vertically in concrete blocks (1:2:4) of size 0.600 m x 0.600 m x 0.750m, leaving about 2.15 m length of the gauge above the block. They are installed in steps with an overlapping of about 0.150m and in sufficient numbers to enable the measurement of the entire range of flow from the minimum water level to the maximum water level expected at the site (Fig. II.3.1). To avoid error in measurement due to the afflux the upstream face is streamlined in a cutwater shape. The graduations of the gauge are generally engraved in 0.010 m alternately, the decimetre and full meter markings covering a greater width than the rest. Using synthetic enamel waterproof pint, the gauge post is first painted in white and the engraved graduations are then repainted in black, except that the whole metre marks may be painted in red. The number to indicate the decimetre marks are painted in black and those to read the metre marks in red.

Different type of marking on staff gauges are shown in Fig. II.3.2.

The gauge is read upto the third place ot decimal. The gauge observation is done thrice daily at 08.00,13.00, and 18.00 hrs in lean season and hourly in flood season.

The detailed specifications for the vertical staff gauges are given in IS: 4080 - 1967.

2.3.1.2 Sloping Gauge or Stepped Gauge

Sloping or stepped gauges are provided where the banks are quite stable. This is generally made of concrete blocks fixed along the slope of the banks and provided with steps for reading the gauges, painting them, etc. The marking of the slope gauge is to be made with precise levelling. These gauges are to be checked every year before the floods for possible disturbances. The water levels can be read with greater degree of accuracy because of enlarged scale.

IS 4080-1967

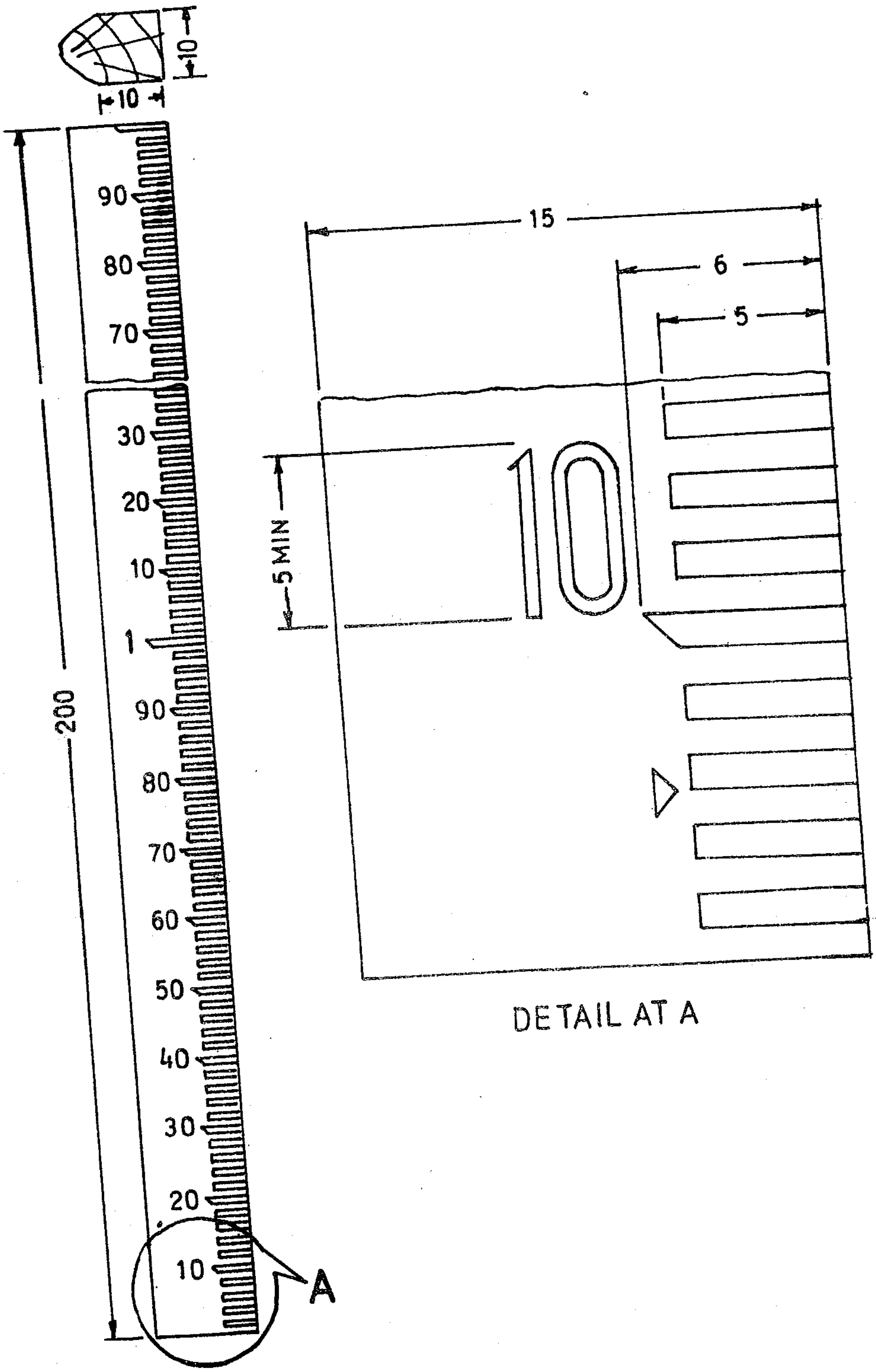
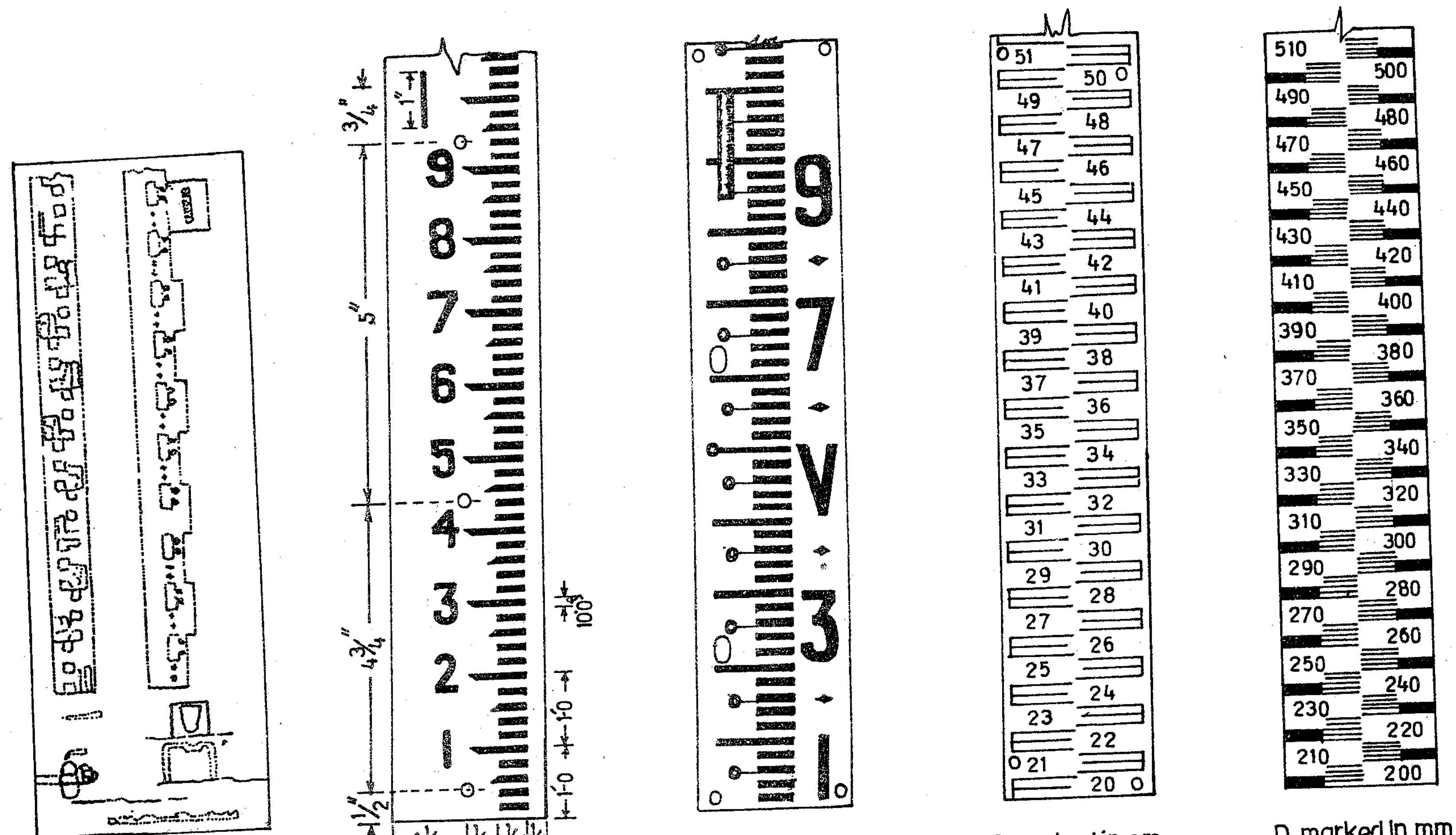


Fig.No.II.3.1 (All Figers in centimeter)



OTT STAFF GAUGE (marked in cm)

A. marked in ft

B. marked in ft

C. marked in cm

D. marked in mm

Note : Gauge plate should be of steel, enamelled on face, numerals and graduations to be sharp in black, rest in white, holes for fixing the gauge plate as shown.

Fig.II .3.2: DIFFERENT TYPE OF MARKINGS ON STAFF GAUGES

2.3.1.3 Cantilever Gauge

Cantilever gauges are generally provided at hill streams where it is not possible to approach the river bed on account of high and steep banks, and it is difficult to install a vertical or inclined staff gauge. In the case of Cantilever gauge, the scale is attached to a beam which cantilevers out over the stream from inshore. A weight is lowered to the water surface with a chain which runs over the pulley at the outer end of the cantilever. The position of a marker on the chain is read on the scale.

2.3.1.4. Gauge Wells

Gauge wells are generally provided in hydraulic structures for recording upstream and downstream water levels. The gauging wells are for the purpose of dampening the disturbances of the water caused in hydraulic structures. Gauges are generally painted on the inside of the well.

Some precautions to be taken in observation of such gauges are briefly indicated below:

(1) The whole number of the gauge may be read wrongly. To avoid this, the markings and numbers to correspond to whole metre graduations are painted in bright colour to distinguish them from the rest.

(2) The water level may be computed erroneously from the gauge reading and zero R.L. of the gauge. This personal error can be avoided by the observer with care and alertness.

(3) The gauge may have been disturbed resulting in the change of the zero R.L. In the case of a temporary gauge fixed with a strut driven into the river bed, it is necessary to check the gauge with reference to a bench mark nearly daily before commencement of the observation. If the gauge is permanently founded in concrete blocks, it may be checked at least once a week or whenever there is suspicion about its disturbance.

(4) There may be abrupt shifting of the measuring site. When it is changed for any reasons, the new gauge should be correlated to the old gauge by simultaneous observations for a suitable period. Such abrupt shifts are easily located by a careful inspection of the relative record or hydrograph.

(5) Errors may occur in computation of water levels when the reference bench marks are submerged in the floods and get disturbed. After the floods recede, it is necessary to check the values of the site bench marks with reference to the G.T.S. bench mark nearby and also to verify the zero levels of the various gauges at the site.

(6) The afflux of the flowing water may cause error in gauge reading. To obviate this, the upstream side of the gauge is streamlined in a cutwater shape and a dampening device like a perforated stilling box (tin canister) is used while observing the gauge.

7) Gauges can be checked by correlating the upstream and downstream level of the same river.

2.3.1.5. Float-Type Gauges

Float-operated gauge requires a stilling facility, i.e. a Well or a G.I. Pipe of suitable diameter, in which the float operates. It senses the fluctuation in the water level and conveys to the recording mechanism through pulley and counter-weight. The strip chart in the recorder is moved by a clock at a speed required for the proper definition of the graphic record. The well is connected to the water outside in the river through an intake pipe. It is preferable to fix two or more intakes at different levels. The installation of a float-type gauge requires a stilling well which is expensive, particularly where the depth is large (Fig. II.3.3.).The intake pipes need to be flushed after every flood, especially where the river carries sufficiently large silt charge. If a hydraulic structure like a bridge or barrage is in the vicinity, a suitable G.I. pipe can be anchored to its pier or abutment for installation of a float-operated gauge . The cost of installation and maintenance in this case is relatively less. (Fig.II.3.4).

The stilling well for the accommodation of the float and recorders should meet the following requirements:

(1) It should be vertical and have sufficient height and depth to allow the float to rise and fall over the full range of water levels.

(2) In stream with fluctuating silt contents inlet pipes should be provided at various stages.

(3) Joints with any inlet pipes should be water-tight.

(4) The dimension of the inlet pipes or of the channel should be large enough for the water level in the well to follow the rise and fall of the stage without delay and also to prevent clogging due to sediment.

(5) If the stage cannot be read on the chart with sufficient accuracy because of short period wave effect, a constriction should be fitted in the inlet pipe to damp out oscillation.

2.3.1.6. Pneumatic Gauge

Pneumatic gauge recorder does not need a stilling well or pipe. Shifting of the equipment of the pneumatic gauge installation from station to station is easier and cheaper. In this type of gauge, the gauge height element is actuated by the static pressure of the head of water above the orifice. It usually has the following three components:

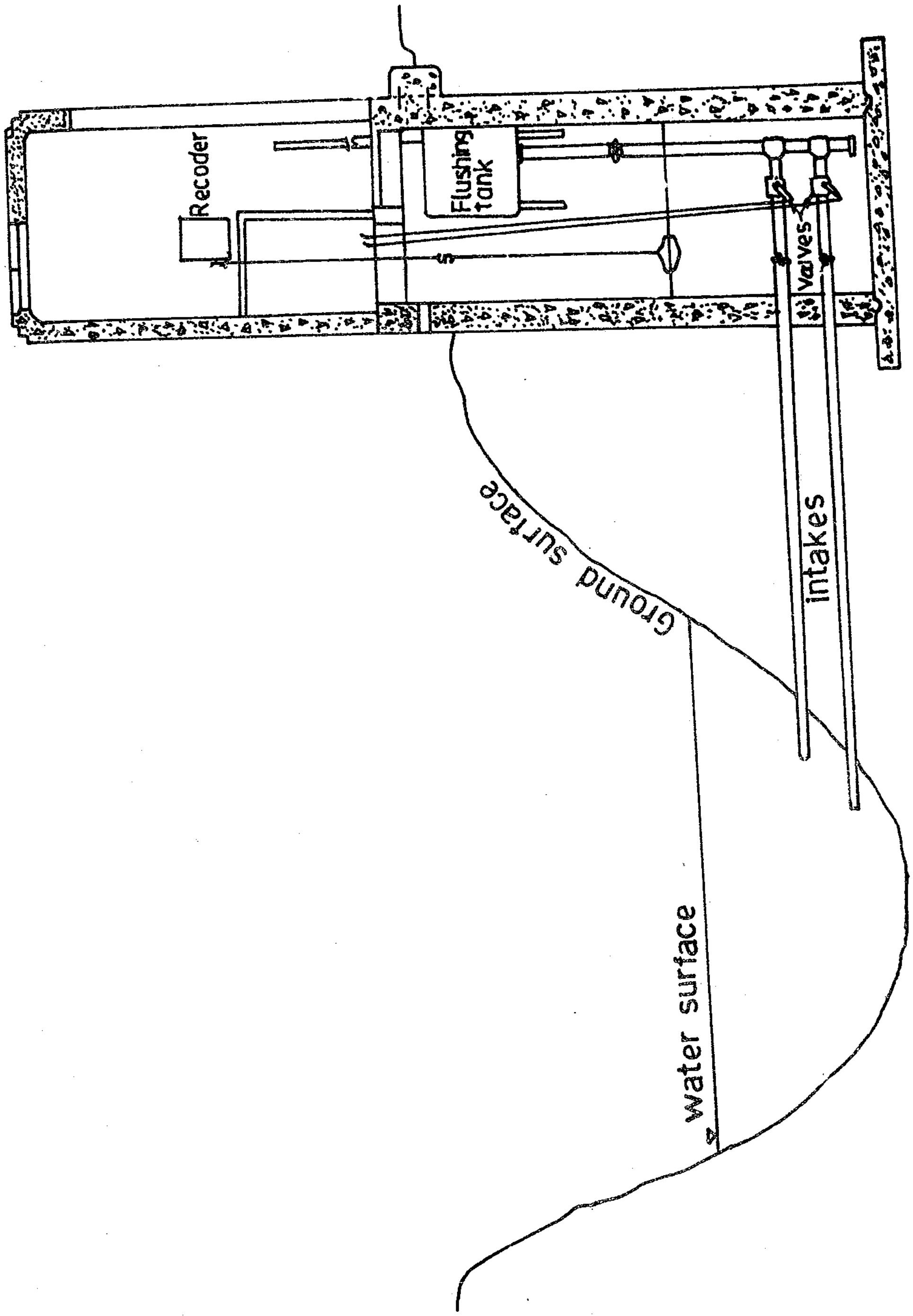


Fig. II.3.3 STILLING WELL

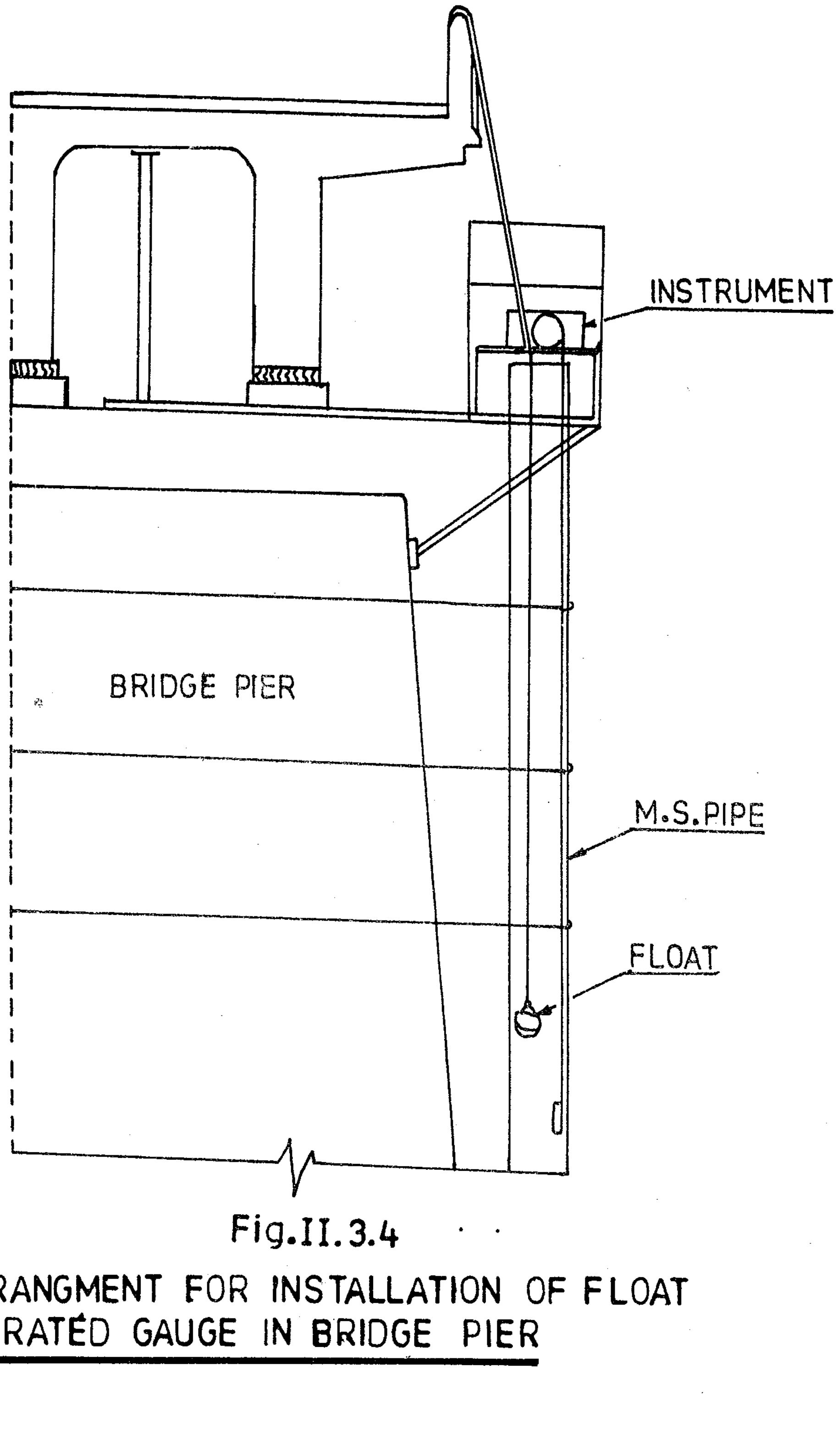


Fig.II.3.4

ARRANGMENT FOR INSTALLATION OF FLOAT OPERATED GAUGE IN BRIDGE PIER

1. Gas purge system;

2. Manometer assembly; and

3. Servo control and amplifier units.

A bubble tube connects the instrument installed inside a shelter on the high bank of river to the orifice tube in the water. When the nitrogen gas bubbles freely into the water through the orifice, a back pressure is created inside the tube which is proportional to the water level and the corresponding difference in hydrostatic pressure is sensed by a mercury manometer or a piston manometer.

When water depth changes, the resulting pressure variation causes the mercury to move from one cup to another unbalancing the system in the case of mercury manometer. This action is detected by a sensitive float switch which closes a set of contacts. The serve control unit amplifies the position change signal and supplies to the serve drive motor. The motor is geared to a movable carriage upon which the pressure reservoir is mounted. When the carriage reaches a point where the mercury column in the two cups is brought into a balance, the float switch opens and stops the motor. The servo motor is also geared to a digital counter and the stylus of the recorder. Errors due to variation in the density of water with temperature and with chemical and salt content can be compensated by adjustment of the inclination of the manometer provided they are directly proportional to the pressure.

In the case of a piston manometer, the movement of the piston on account of difference of pressure disbalances the balance beam. This causes electrical contacts and the servo motor is actuated through the servo amplifier and the balance weight shifted till the beam is balanced when the motor stops.

The measuring tube of the bubble gauge gets silted up which requires occassional purging.

The following requirements should be met in the case of pneumatic recorder:

(1) This recorder measures the stage by means of the pressure exerted at a nozzle which is immersed and solidly fixed at a known elevation inside the stream.

(2) The recorder should have source of compressed air or nitrogen and a device for adjustment of the same.

(3) There should be a device for purging the measuring tube directly from the compressed gas.

(4) The device for measuring the pressure difference should be sufficiently sensitive and accurate.

Besides the above type of gauges, there are also various types of gauges used in stream gauging operation, namely.

(i) Chain gauge;

(ii) Wire Weight Gauge;

(iii) Anchor tape gauge;

(iv) Electric tape gauge;

(v) Hook and painter gauge; and

(vi) Electrical long distance gauge.

The merits and demerits of various types of gauges are detailed as under:-

1. Staff Gauges

(a) The gauge post interferes with the flow so that in the fast flowing stream the reading tends to differ from undisturbed water level. This can be overcome by observing the gauges continuously for a minimum period of two minutes of the period of complete oscillation whichever is longer by using a stilling box. Maximum and minimum readings are averaged.

(b) The floating debris can damage the installation.

(c) A number of gauge posts are required to record the low water level and the high water level.

(d) Due to surface tension effects readings are not likely to be more accurate than ± 3 mm

(e) These are very simple, cheap and easy to maintain.

2. Float-type Gauge

(a) Float-type gauges are indigenously manufactured and the cost of the equipment is much less than that of the imported ones, but the cost of construction of stilling well and intake pipes is very high in case the depth of lowest water level from bank is high and rock foundation is not available for the wells.

(b) Float, counter-weights and tape are the main units of function. These, being mechanical parts, can be maintained with ease.

(c) Recording unit of float-type gauge has to be installed on the stilling well and its installation, therefore, becomes difficult when high banks are not available at the measuring section.

(d) To obviate errors generally found in such gauges, larger diameter of float, reduction of instrumental friction, use of light float line and counter-weight and avoidance of slips for the float line are needed.

3. Pneumatic water level recorder

(a) Though initial cost of the instrument is high, and involves foreign exchange for importing, the use of this equipment does not require costly stilling well. There is no problem of silting of the intake channels etc.

(b) The instrument is very sensitive and, therefore, requires experienced personnel for its maintenance which at times becomes very costly.

(c) Recording unit can be installed at a safer place far away from the bank.

(d) For the operation of this Pneumatic-type of water level Recorder, inert gas nitrogen is required which at remote places is difficult to procure.

2.3.2. Measurement of River Discharge

The methods for measurement of river discharge are broadly classified as follows:

(A) Direct Measurement of River Discharges

(i) *Velocity Area Method*

(a) By Wading

(b) By boat or craft

(c) From a bridge

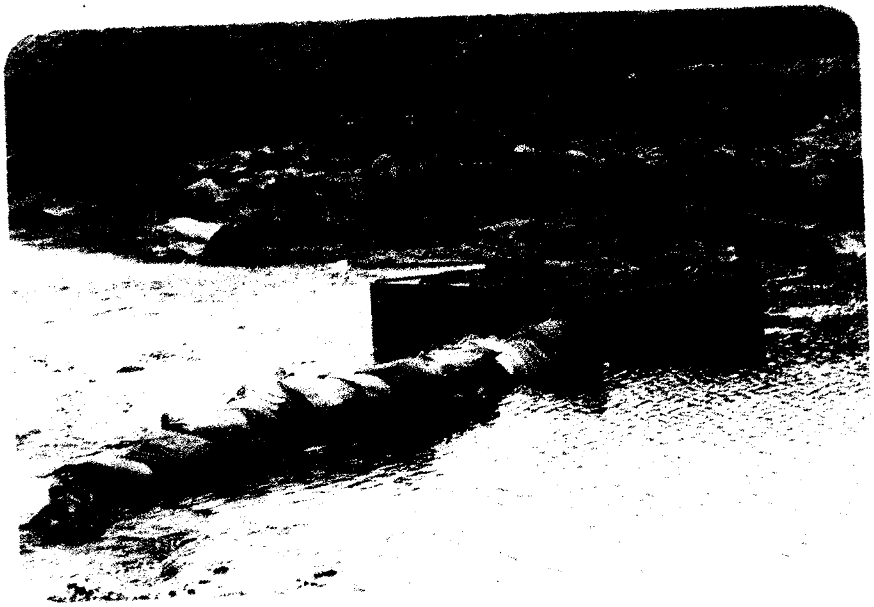
(d) From a cableway

(e) By moving boat method

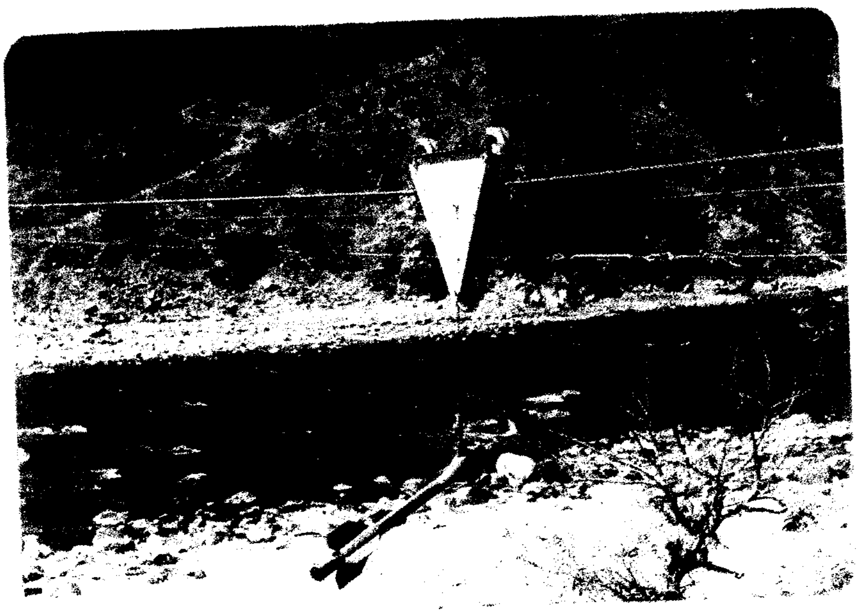
(ii) *Chemical Method*

(a) Salt Dilution method

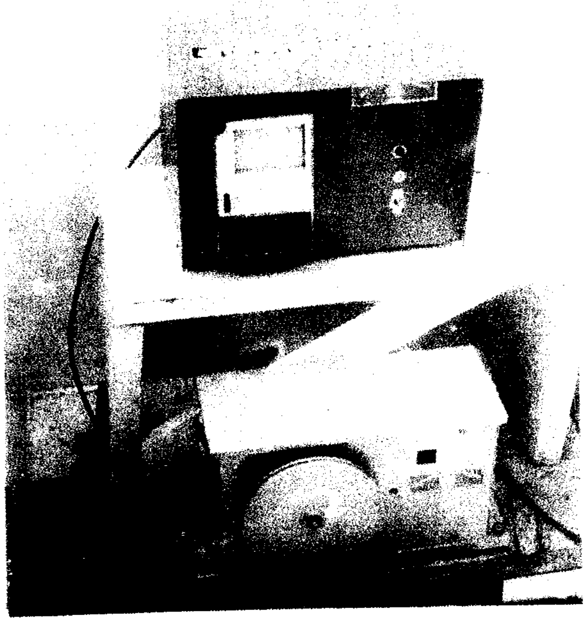
(b) Salt Velocity method



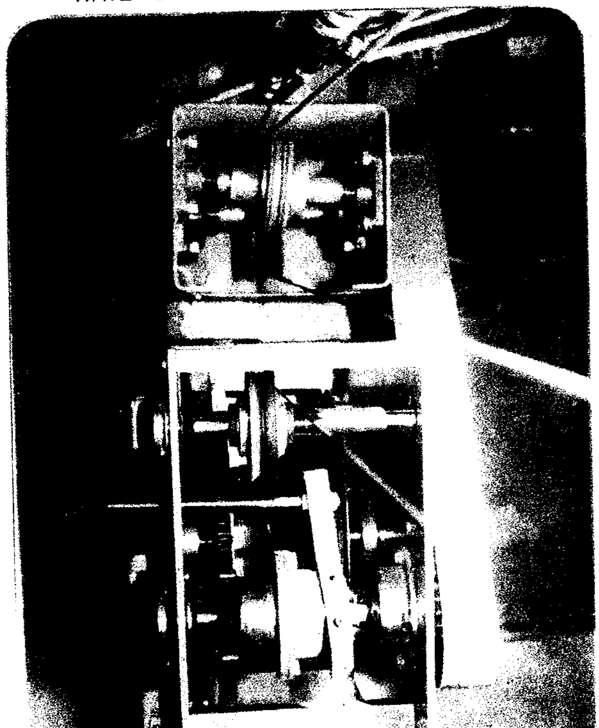
LEAN SEASON DISCHARGE MEASUREMENT BY FLUME AT HRIPUR.



CABLEWAY AT BAUSAN.



WATER LEVEL SENSOR WITH RECORDER.



WINCH MACHINE FOR AUTOMATIC DISCHARGE OBERVATION AT BAUSAN

(iii) *Other Methods* such as accoustic method, ultrasonic methods etc.

(B) Indirect Measurement of River Discharges

(i) Slope Area method

(ii) Stage Discharge Curve

(iii) Assessment of flow passing through Hydraulic structures.

Some of the important equipment required for stream gauging operation are:-

1. Theodolite
2. Levelling instrument
3. Water current meter with accessories
4. Stop Watch
5. Sounding rod
6. Echo Sounder
7. Rigid rod
8. Protactor
9. Measuring tape
10. Binoculars
11. Sextant
12. Fish Weight
13. Anchor with Rope
14. A-Crane
15. Gauging reel
16. Staff gauges

17. Floats

18. Plane table with accessories

19. Prismatic compass

20. Engineer's chain

21. Sounding weight

2.3.2.1. Velocity Area Method

The detailed procedure for measurement of flow of water in open channels are described in IS: 1192 - 1959. However a brief introduction of the method is given below.

In practice the discharge is approximated by the summation.

$$Q_m = \sum_{i=1}^{m} b_i d_i v_i$$

Where Qm= Calculated discharge;

b_i = Width of the i th segment;

d_i = Depth of i th segment;

v_i = Mean velocity in the i th segment; and

m = Number of verticals in a cross section.

The discharge of the stream is determined by measuring the cross sectional area and the velocity by a water current meter. In practice the river section is divided into a number of compartments depending upon the degree of accuracy required.

ISI recommendation in this case is as under:

Description of Channel	*Number of Observation Verticals*	*Maximum width of segment (m)*
Less than 15 m	15	1.5
15 m to 90 m	15	3 to 6
90 m to 180 m	15	6 to 15
Greater than 180 m	25	---

The depth of the stream is measured by sounding rod, log line or echo-sounder. The location of the vertical is usually determined by geometrical loyout on the bank usually called pivot point method. In the case of moving boat equipment, the distance is measured with the help of current meter and the electronic pulse counter.

The velocity is measured by means of a cup type or propeller type current meter by lowering at 0.6 depth where the mean velocity occurs. An average of two readings i.e. at 0.2 and 0.8 D. are also adopted with good result. However, it is better to find out the exact point of mean velocity by actual experiment.

The segmentation is also made in such a way that 10% of the total discharge does not pass through any segment.

The following table shows the mean velocity in a vertical

Type	*Observation point*	*Mean velocity*
Single point	.6 D from Surface	$\bar{v} = v_{.6}$
Two point	.2 and .8 D	$\bar{v} = \frac{1}{2}(v_{.2} + v_{.8})$
Three point	.2D .6D and .8D	$\bar{v} = 1/4\ (v_{.2} + 2v_{.6} + v_{.8})$
Five point	S, .2, .6, .8 and B	$v = 1/10\ (v_s + 3v_{.2} + 2v_{.6} + 3v_{.8} + v_B)$

The discharge of various segments is computed either by mid-section method or by mean section method.

By Mid-Section method $q = \bar{v}.d\left(\frac{b_{left}\ right}{2}\right)$

By Mean Section method $q = \frac{(\bar{v}_1 + \bar{v}_2)}{2} . \frac{(d_1 + d_2)}{2} .b$

2.3.2.2. Discharge Measurement by Moving Boat Method

By this method stream discharge is measured by velocity area method. The data such as depth, velocity and distance of segment is collected while the boat crosses the river. The equipment required are:

(1) Vane and angle indicator

(2) Water current meter

(3) Rate meter

(4) Echo sounder

Procedure

First, a path for the boat to travel is selected which is as nearly perpendicular to the flow direction as is possible. Then, two clearly visible range markers are placed on each bank in line with this path. Anchored floats are placed in the stream 40-50 ft from either bank from the selected path. Finally, the width of the stream is measured by triangulation or other methods and the exact locations of the floats are determined.

During a moving boat discharge measurement the current meter is set at a pre-determined, fixed depth of 3 to 4 ft below the surface; thus, this technique uses the sub-surface method of measuring the velocity. Observed velocities should ideally be multiplied by a co-efficient to adjust it to the mean velocity in its vertical. However, it is assumed that in larger streams where the moving boat technique would be applicable, these co-efficients would be fairly uniform across a section, thus permitting the use of an average cross-section coefficient to be applied to the total discharge. Information obtained from several vertical velocity curves, well distributed across the measuring site, would be needed to determine a representative coefficient for the total cross section.

The depth sounder records a continuous graph of the stream bed during the traverse. At 30 to 40 points along the cross-section a vertical mark is made on depth sounder chart to serve as observation points. It is recommended that at least six such individual runs should be made and the results averaged to obtain the discharge in uniform flow.

Conditions:

This is practicable because of the ease and speed with which the extra runs can be made. For unsteady flow conditions, on tidal streams, it will usually be desirable not to average the results so as to better define the discharge cycle.

2.3.2.3. Discharge Observation by Cableway

In the case of measurement of discharge for small water shed, the cableway system is the best method for adoption. The cableway method of stream flow measurement has got certain distinctive advantages viz, measuring personnel are not exposed to high water and other hazards. Measurement can be done by comparatively less persons and measuring operation can be performed at any time as desired.

In the cableway system the stream is gauged by velocity area method.

There are two main types of cableway systems of measurement of discharge:

(i) Those with a trolley in which the observer goes inside the river and moves in different verticals; and

(ii) Those with an instrument carrier in which the current meter and the sounding weight is moved at different verticals and controlled from the bank.

The first type of cableway is generally used when the span is excessive and on account of sag, it is not possible to move the measuring instrument with the instrument carrier from the bank by winch power alone.

The second type of cableway enables the measuring instrument to be placed at any desired verticals in a stream by means of a combined horizontal and vertical movements for the measurement of velocity sounding etc. from the bank. It also enables to work out the bed profile of the stream by measuring the depth at a number of desired verticals for working out the cross-sectional area.

The equipments used are:

(i) Pier post;

(ii) Track or main cable;

(iii) Tow cable;

(iv) Instrument Carrier;

(v) Cradle; and

(vi) Double Drum winch or single winch

2.3.2.4. Discharge Observation by Dilution Methods/Tracer Technique

Tracer gauging may be divided into two categories:—

(1) Tracer velocity method; and

(2) Tracer dilution method.

Tracer method of discharge measurement as stated earlier is generally most suitable for hilly streams which are highly turbulent. The discharge is determined over a length of the stream rather than at a single point.

2.3.2.5. Discharge Observation by Slope Area Method:

This method employs the principle of energy loss to estimate the velocity. These require measurement of the water surface slope along the stream from which an estimate can be made of the energy gradient. The energy gradient can then be used in the Manning Equation:–

$$\bar{v}=\frac{1.5}{n}r^{2/3}s^{1/2}$$

The cross sectional area is determined by surveying tape and level usually after the passage of the flood. Many times the flood discharge flows over a highway embankment or through a culvert or a bridge opening. These provide opportunities of estimating the velocity using the energy principle. The roughness coefficient value to be used is related to bed material size and condition of the channel. These recommendations are given in Indian Standards Institution IS:2912-1964.

2.3.2.6. Discharge Observation by Stage-Discharge Curve

The purpose in making an area velocity measurement is to obtain a concurrent observation of both stage and discharge. Similar measurements made over a large range of discharge are assembled on a graph called a Rating Curve. If the site for the stream gauging station was well chosen with a stable control, a curved line can be fitted through the data points. The line through the data points is the State-Discharge Relationship. If computer based techniques are used to process the runoff records, it is desirable to define the stage discharge relationship as an equation..

$$Q = K\,(\text{Stage}-a)^{b}$$

Where Q is the discharge in cubic meters per second;

Stage is the water surface elevation in meters,

K, a and b are constants to be determined.

The stage discharge relationship can also be used to obtain an estimate of the discharge of a large flood where only high water marks or other information is available regarding the stage. The stage-discharge relationship may be extended to the stage observed to give an estimate of the discharge. If this involves a large extrapolation, simple graphical extension may not be adequate. Some of the methods of extending the rating curve are:–

(1) Graphical Extension

(2) Logarithmic extension

(3) Stevens $A\sqrt{D}$ Extension etc.

Graphical extension is adequate if the extension is small and if the rating curve is well established at its upper limit. The Logarithmic extension can easily yield an equation for the rating curve.

The detailed procedure for establishing the stage-discharge curve are given in IS: 2914-1964.

2.3.2.7 Errors in Stream Flow Measurements

The difference between the apparent discharge measured and the actual discharge is a measure of accuracy.

In the measurement of stream flow by velocity area method, the accuracy primarily depends on the performance of the water current meter, number of verticals taken for depth and velocity observation, duration of current meter observation, correct measurement of depth and width.

There are two types of errors namely random and systematic. Random errors are caused purely on chance fluctuation and are errors involved in the measurement of depth and width. The systematic errors are associated with a particular instrument or on the methodology of stream flow measurement.

It is necessary to have an idea of the errors involved in streamflow measurements by Velocity-Area method so as to guard against them and for taking corrective steps in the observation. On the basis of analysis of observed data, the range of systematic and random errors has been computed and is given below which may prove useful guide in determining the number of verticals for segmentation and permit velocity measurements etc.

X_b i.e. Errors in Measuring Width:

This type of error is usually not much and a value of ± 0.5% may be taken.

X_m i.e. Errors due to the Choice of Number of Verticals:

m (Number of Verticals)	X_m
8	± 5 percent
15	± 3 percent
25	± 2 percent
50	± 1 percent

X_d i.e. Errors in measuring depth:

This type of error can be reduced by repeated measurement. However, a value of ± 2.5% may be taken for this type of errors.

X_f i.e. Errors on account of duration of exposure of Current Meter:

For an exposure time of 40 secs. The recommended value is ± 6 percent.

X_o i.e. Errors due to the choice of number of points in a vertical.

Method	X_o
Velocity distribution	± 0.5 percent
2 point	± 3.0 percent
1 point (.6d)	± 3.5 percent

X_q i.e. the overall random error

$$X_q = \pm X_m^2 + \frac{1}{m}\left(X_b^2 + X_d^2 + X_v^2\right)$$

$$\text{Where } X_v = \pm\left(\frac{X_f}{P} + X_o^2\right)$$

& P = number of points in the vertical where velocities have been observed.

2.3.3 Rainfall Observation

The following instruments are generally used for rainfall measurement.

(i) Ordinary Raingauge (ORG)

(ii) Self Recording Raingauge (SRRG)

(iii) Automatic Transmission Rain Gauges (ATRG)

A brief description of the different types of rain gauges alongwith the limitations, relative advantage and disadvantages etc. is given below:

2.3.3.1 Ordinary type Raingauge

The most commonly used ordinary type rain gauge is the "Symon's Raingauge", which has been recommended for use by India Meteorological Department also.

Description

A self explanatory diagram is shown in Fig. II.3.5

Rainfall measurement

A measuring cylinder made of clear glass with a low coefficient of expansion and clearly marked with the size of gauge with which it is to be used. Its diameter should not be more than about one-third of that of the rim of the gauge, and can be made less than this.

The graduations should be finely engraved; in general markings should be at 0.2 mm intervals with whole mm lines clearly figured. It is also desirable that the line corresponding to 0.1 mm should be marked. Where it is not necessary to measure rainfall to this degree of accuracy, every 0.2 mm up to at least 1.0 mm and every mm above that should be marked, with every 10 mm line clearly figured. For accurate measures the maximum error of the graduations should not exceed ± 0.05 mm at or above the 2 mm graduation mark and ± 0.02 mm below this mark.

To achieve this accuracy with small amounts of rainfall the inside of the measuring cylinder should be tapered off at the base. In all measurements, the bottom of the water meniscus should be taken as the defining line, and it is important to keep the measure vertical and to avoid parallax errors. It is helpful in this respect if the main graduation lines are repeated on the back of the measure.

Errors and Accuracy of Readings

The errors involved in measuring the catch collected in a gauge are small compared with the uncertainty due to the effect of the exposure of the instrument, provided reasonable care is taken in the readings. Daily gauges should be read to the nearest 0.2 mm and preferably to the nearest 0.1 mm, weekly or monthly gauges should be read to the nearest 1 mm. The main sources of error likely to arise are the use of inaccurate measures spilling of some water when transfering it to the measure, and inability to transfer all the water from the receiver to the measure.

In addition to these errors, losses by evaporation can occur. These are likely to be serious only in hot dry climates and with gauges visited only at infrequent intervals. Evaporation losses can be reduced by placing oil in the receiver (this forms a film over the surface of the water) or by designing the gauge so that (a) only a small water surface is exposed, (b) Ventilation is poor, and (c) the internal temperature of the gauge is not allowed to become excessive. Also, the receiving surface of the gauge is not allowed to become excessive. Also, the receiving surface of the gauge must be smooth, so that the raindrops do not adhere to it. It should never be painted.

In winter where rains are often followed immediately by freezing weather, damage to the

PATTERN OF RAINGAUGE

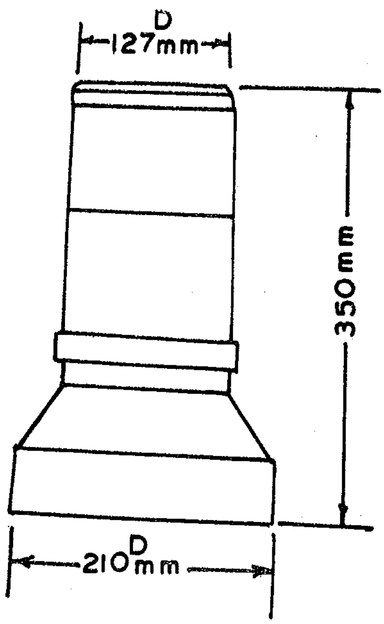


Fig. II 3.5 (a)

Standard Raingaugs (Diameter 127mm: Capacity 175 mm)

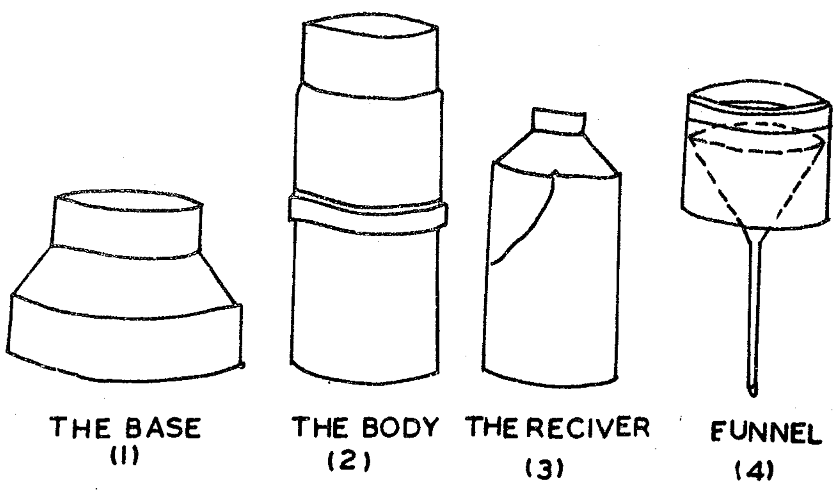


Fig. II 3.5 (b)

receiver, and consequent loss by leakage, can be prevented by the addition of an antifreeze solution. This again mainly applies to gauges visited infrequently. Allowance for the solution added must of course be made when measuring the gauge catch. All gauges should be tested regularly for possible leaks.

2.3.3.2 Self Recording Raingauges

Three most commonly used self recording rain gauges are:

1. Weighing bucket type,
2. Tipping bucket type, and
3. Syphon type.

However the syphon type SRRG is most commonly used in India. A brief description of different types of SRRG's is given below

Weighing bucket type Raingauge

The weighing type gauge weighs the rain which falls into a bucket set on a platform of a spring or level balance, as shown in fig. II. 3.7.

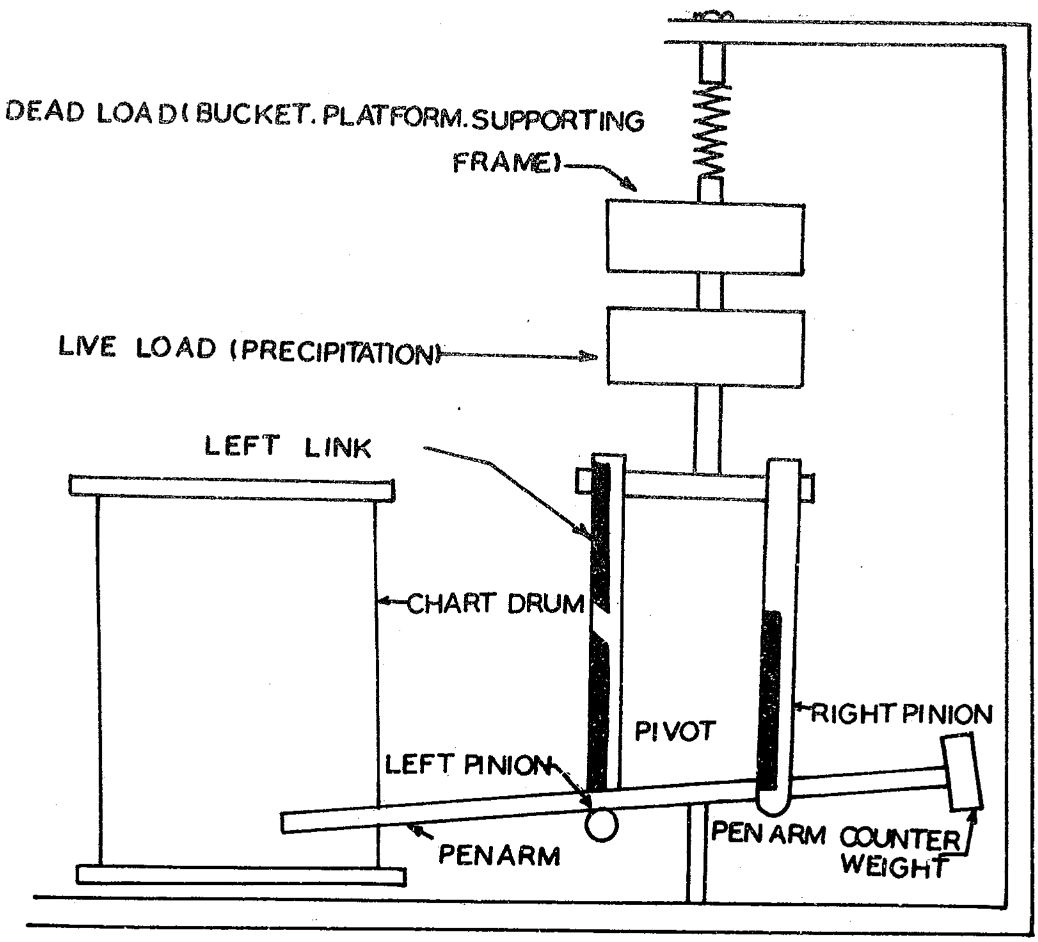


Fig.II.3.7. AUTOMATIC WEIGHING TYPE RAIN GAUGE

The increasing weight of the bucket and its contents are recorded on the chart held by a clock-driven drum. The record thus shows the accumulation of precipitation with time in the shape of a mass curve of precipitation. The gauge must be surviced about once a week when the clock is rewound and the chart is replaced. Since the recorder chart has a limited width, in case of heavy rainfall exceeding a depth represented by the width of the chart within the normal period of renewal of the chart, the recording mechanism reverses the direction of record immediately on reaching the upper edge of the recording chart.

Tipping Bucket Type Raingauge

The tipping bucket raingauge consists of a 30 cm diameter sharp receiver. At the end of the receiver a funnel is provided. A pair of buckets are pivoted under the funnel in such a way that when one bucket receives 0.25 mm of precipitation it tips, discharging its contents into a tank bringing the other bucket under the funnel. Tipping of the bucket completes an electric circuit camming the movement of a pen to mark on a clock driven revolving drum which cammes a record sheet. The same is shown in Fig. II 3.8.

Syphon Type Automatic Rainfall Recorder

The necessary details in respect of installation and use of syphon type SRRG are as follows:

Principle

Rain water from the receiver is led into the float chamber through an entrance tube and as float rises, a recording pen which is mounted to the float rod, records on a chart on a rotating clock drum driven by clock mechanism. The clock drum revolves once in 24 hours. So that a continuous record of the movement of pen is made on chart, syphoning occurs automatically when pen reaches on the top of chart and as the rain fall continues, the pen again rises, from zero line, and if there is no rainfall then recording pen will be given a horizontal line on the chart. A sample record is shown in figure. II 3.9.

Description

A self explanatory diagram of the raingauge recommended by the ISI is given in Fig. II.3.6. Further details are given in IS: 5235 - 1969, and IS: 8389 - 1983.

Installation

The raingauge is installed on a masonry plat form. The 15 cm foundation bolt is fixed in the centre of the platform. The gauge is fixed on the bolt with galvanised iron base. A spirit level is also placed across the rim in three different directions to ensure that the gauge is horizontal. The float chamber can be levelled with the help of three nuts provided.

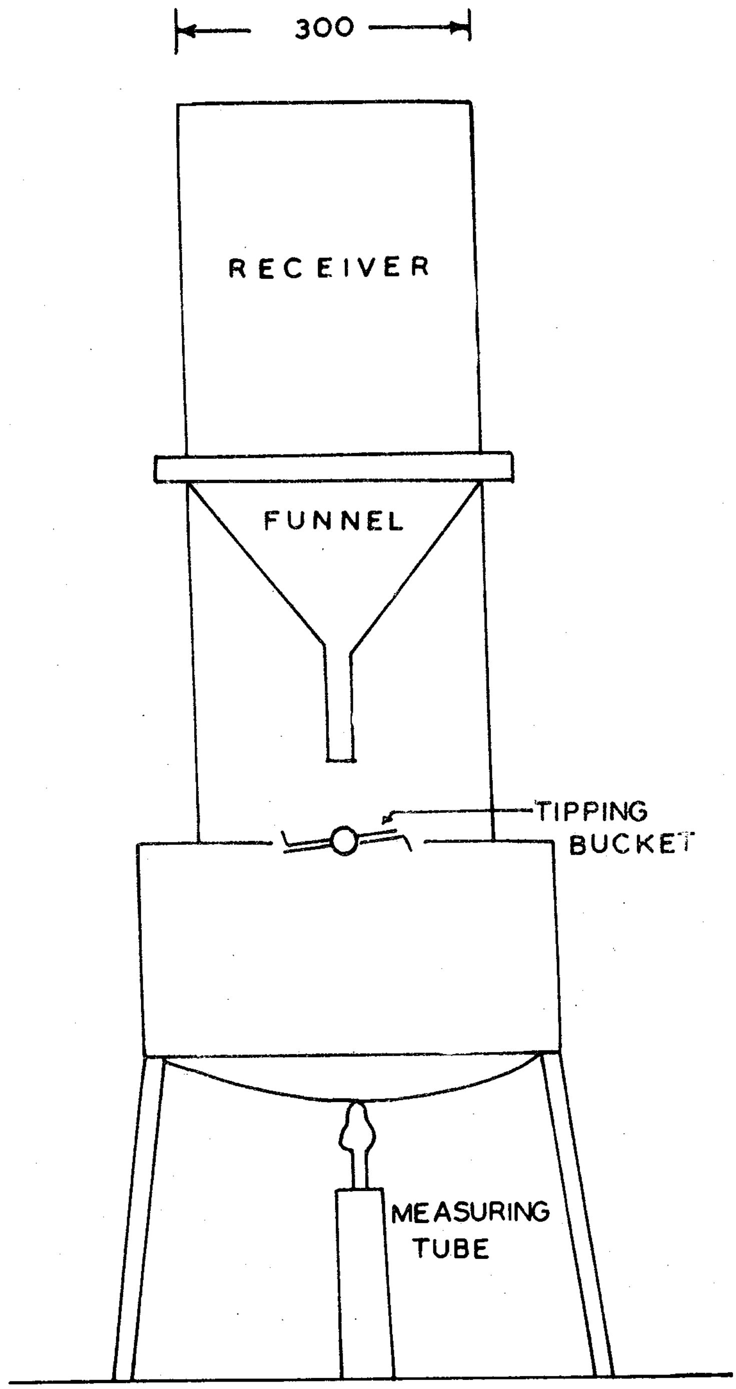


Fig. 3.8 TIPPING BUCKET TYPE RAINGAUGE

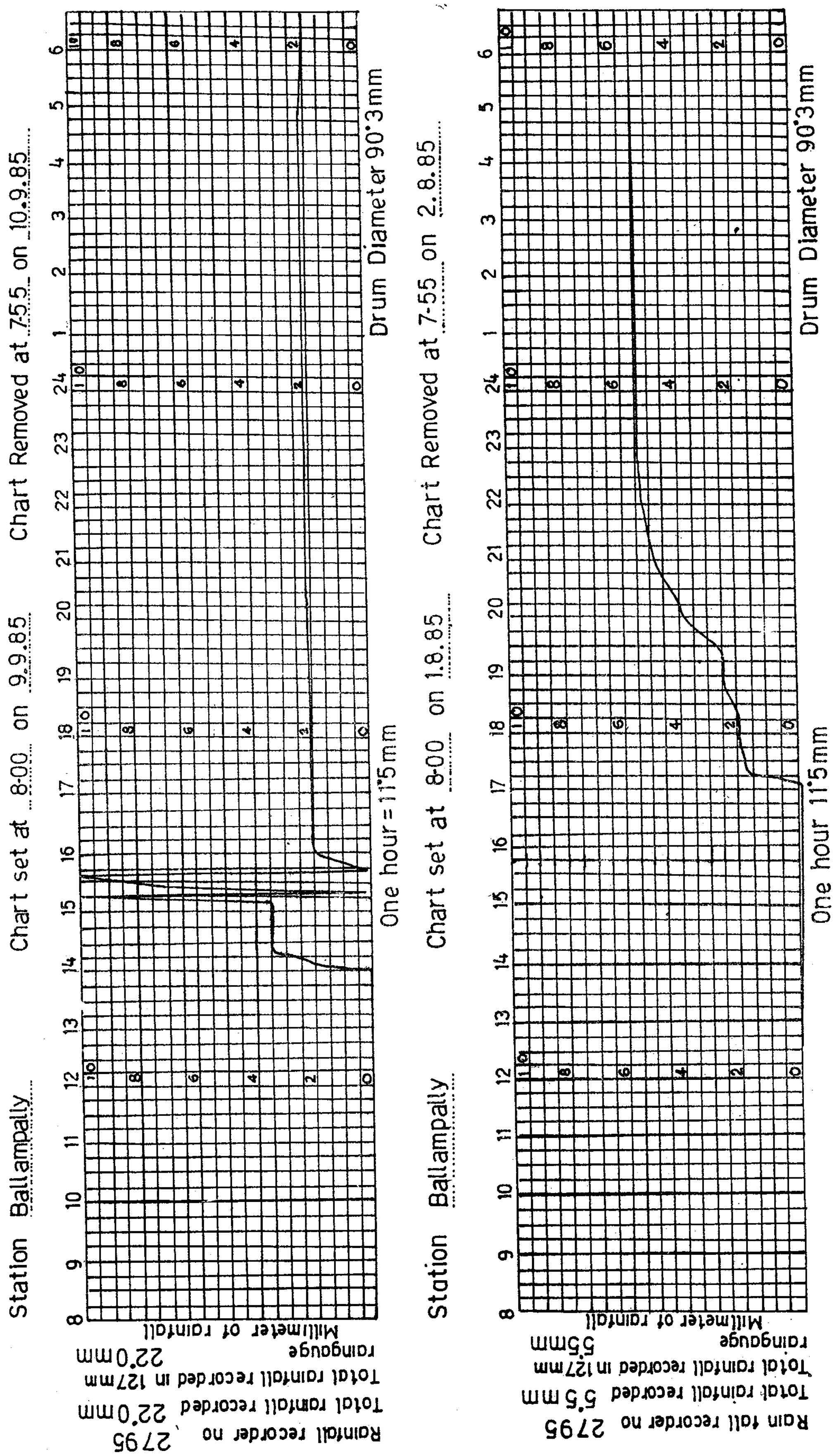


Fig.II.3.9 A SAMPLE RECORD OF S.R.R.G. CHART

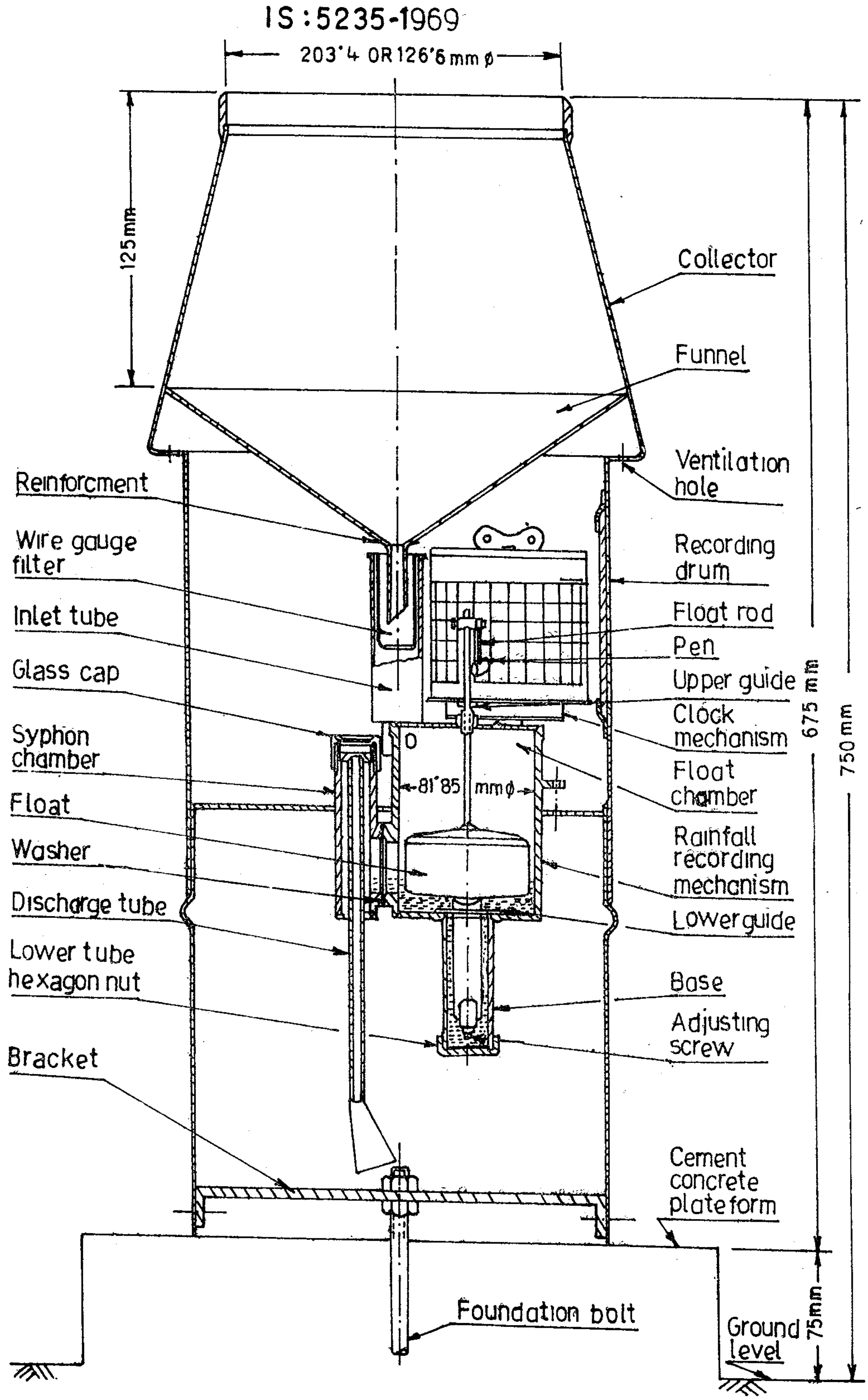


Fig. II.3.6 (Dimensions in mm)

Operation

For daily observations, the following steps are followed:

(a) The old chart is removed from the Clock drum and the new chart is wrapped over it. Care is taken to ensure that the horizontal line on the overlapping portion coincide and the bottom of the chart is as near the flange as possible.

(b) The Clock is wound as per requirement.

(c) The pen is set at zero line as follows:

Sufficient water is poured into the receiver slowly till the pen reaches at top and the syphoning of water starts. After the water is drained out, the pen should be at zero line.

(d) The ink in the pen is checked. There should be sufficient ink.

The chart is required to be changed exactly at the same time every day, usually between 0830 and 0900 Hrs. of IST. The chart should be set in such a way that the pen is set to the correct time.

Hints for Use of the Instrument

1. The instrument should be tested and checked at regular intervals specially when there is no rainfall for some time. This should be done by pouring measured quantities of water from either a 203 or 127 mm. rain gauge measuring jar.

2. If the instrument is dismantled, the receiver, float and the funnel should be properly fitted.

3. The time of syphoning should be checked occasionally to see whether outlet tube is chocked. The time taken should be 15 to 20 seconds.

4. Instrument should always be kept clean and there should be no dust and leaves etc. in the funnel.

5. The tip of the recording pen can be cleaned by passing a piece of thin strong paper between the point. Thick paper must not be used, because of the danger that the point of the pen may be opened.

6. The inside of glass cap should be clean, if not, it should be cleaned with a dry cloth, without any oil or grease on it. The glass cap should be tight.

7. The instrument should be levelled and in vertical position.

8. Special recording ink is used for this purpose. Because normal ink evaporates rapidly.'

2.3.3.3. Radar Measurement of Rainfall

The radar is based on the principle of echo-sounding. High frequency electromagnetic waves are sent out. An extremely small portion of this energy is reflected by objects like rain drops in the sky and is detected by the radar. Two type of radars, namely the X band radar and the S band radar are used. The X band radar transmits on 3 cm wave length (range 50km) while the S band radar uses a wave length of 1 cm. (range 180 Km). The commonest wave lengths are in the 5 cm and 10 cm bands because these wave lengths are best suited to the measurements of precipitation intensity because of the echo power and attenuation characteristics of these wave lengths. By calibration of the echo-intensity with rainfall, it is possible to measure the intensity of rainfall.

The best all purpose weather search radar has the following properties.

i. Wave length is such that the effect of precipitation attennuation is minimised.

ii. Power and pulse length to suit lowest significant precipitation.

iii. Beam width as narrow as possible.

iv. Antenna large enough to receive weakest possible reflected energy.

U.S. weather Brureau WSR-57, is one such type which meets these requirements. Photographic procedures for integrating radar echoes and correlating them with precipitation consist of exposures at 5 min. intervals over a period of 1 to 3 hours. The resulting integrated echo intensities are calibrated with reported precipitation. Digital Video Integrator and Processor (DVIP) has also been utilised at some places for intensity measurement and quantitative estimation of rainfall for flood forecasting purposes.

2.3.3.4. Site Selection

Generally the following difficulties are encountered in accurate measurement of rain-gauges.

(a) The gauges height from ground creates eddy currents which affect the amount of the catch.

(b) The wind affects the amount of the catch.

Since the measurement of rainfall is never subject to a check by repetition and the sample constituting the measurement is almost unbelievably small compared to the actual population, it is always desirable to take every care for correct measurement. The above difficulties can be overcome to a great extent by proper selection of site of raingauges.

The following aspects should be considered in the selection of a site for establishing a raingauge station.

1. The site should be an open place.

2. The distance between the raingauge and the nearest object should be atleast twice the height of the object. In no case should it be nearer to the obstruction than 30 meters.

3. If a suitable site on a level ground can be found, then the gauge should never be situated on the side or top of hill.

4. In the hills where it is difficult to find level space, the site for the gauge should be chosen where it is best shielded from high winds and where the wind does not cause eddies.

5. A fence, if erected to protect the gauge from cattle etc. should be so located that distance of the fence is not less than twice its height.

2.3.4. Observation of other Data

2.3.4.1. Snow Fall

Snowfall is the amount of fresh snow deposited over a limited period (Generally 24 Hours). Measurements are made of depth and water equivalent.

Snow Formation

In many parts of the world, streamflow consists mainly of water released by the melting of snow. This is particularly true for the big rivers of this country Viz. Ganga, Yamuna, Brahmaputra and all the rivers of Punjab. Snow as an appreciable proportion of the total precipitation is limited to the Himalayan region in India.

Measurement of Snow

Snow measurement can be divided into:

(i) Measurement of snowfall which denotes the measurement of each individual storm, and

(ii) Snow survey which determines the accumulated winter snow average and its water equivalent for the purpose of obtaining data for estimating run off.

Thus there are two mainways of measuring the depth of snowfall:

(a) by direct observation of the snow depth on platforms or on the ground with a graduated ruler or scale; and

(b) by measuring the depth of snow caught in a gauge after it has been levelled without compressing.

The first method is more suitable in the absence of strong winds. A mean of several vertical measurements should be made. If the depth of snow which has fallen over a certain period (say in the last 24 hours) is required, special precautions should be taken so as not to measure any old snow. This can be done by sweeping a suitable patch clear before hand or laying covers of suitable material on top of the snow surface and measuring the depth down to these. On the sloping surface, measurements should be made with measuring rod set vertically.

In areas of strong winds. a large number of measurements should be made to obtain a representative depth, but under such conditions the snow depth may be more accurately obtained by measuring the depth of accumulation in a shielded snow gauge.

Details of one form of Modified Nipher Shield

Snow is sometimes measured in an ordinary rain gauge on the weighing type of recording gauge properly equipped with a shield to reduce the effect of wind.

The USBR has developed a precipitation gauge with a heated intake which prevents capping over of the gauge intake.

Snow Run-off, Snow Melt

1. *Objective of Computing Run-off from Snowmelt*

a). Seasonal Water yield forecasting.

b) Run-off forecasting for river regulation

c) Design floods due to snow.

2. *Factors affecting Run-off from Snowmelt*

a) Sources of energy for snowmelt

b) Snow pack characteristics

c) Site conditions

d) Antecedent conditions, and

e) Rainfall.

Water Equivalent of Snowfall

The water equivalent of a fresh snowfall is the amount of liquid precipitation represented by that snowfall. It is determined by the following methods. The relation between depth of fresh snow and its water equivalent should be determined for each climatic region and season. It is important to take several representative samples.

(a) Weighing or melting

Cylindrical samples of fresh snow are taken with a suitable snow sampler and either weighed or melted. It is important to take several representative samples.

(b) Using Raingauges

Snow collected in a non-recording raingauge should be melted immediately and measured by means of an ordinary measuring cylinder graduated for rainfall. The only available and relatively satisfactory recording gauge is one of the weighing type. Float type gauges are unsatisfactory because the heat required to melt the snow as it falls sets up vertical currents above the gauge opening and causes excessive evaporation losses. During the snowfall periods, the funnels of the gauges should be removed so that any precipitation can fall directly into the receiver.

(c) Calculating from depth measurements

The depth of fresh snow is converted using an appropriate relationship. Although the relationship 1 cm of fresh snow - 1mm of water equivalent may be used, it is only valid for a long term average. It is not accurate for a single measurement where the specific gravity of snow may vary from 0.03 to 0.25

Snow Pillows

Snow pillows of various dimensions and materials are used to measure the weight of snow that accumulates on the pillows. The most common pillows are flat circular containers (dia 3.7 m) of rubberized material filled with a non-freezing liquid. The pillow is installed on the surface of the ground, flush with the ground or burned under a thin layer of solid or sand.

Hydrostatic pressure inside the pillow is a measure of the weight of the snow on the pillow. Measurement of the hydrostatic pressure by means of a float operated water level recorder or a pressure transducer provides a means for continuous measurement of the water equivalent of the snow cover. Variations in accuracy of measurements may be induced by temperature changes. Temperature effects may be reduced by installing the access tube to the measurement section in a temperature controlled shelter or by having the access tube and measurement unit in the ground with standard snow tubes, especially during the snowmelt period. They are most reliable when the snow cover does not contain the layers which can cause "bridging" above the pillows. Details of snow measurements are given in Snow Hydrology Project printed by C.W. C in 1986-1987.

2.3.4.2. Wind Measurement

Wind has both speed and direction. The wind direction is the direction from which it is blowing. Direction is usually expressed in terms of 16 Compass points (N, NNE, NE, ENE, etc.) for surface winds and for winds aloft in degrees from North, measured clockwise. Wind speed is usually given in kmph, m/sec or knots. Wind speed is measured by instruments called "anemometers" of which there are several types. Cup type anemometers are commonly utilised.

2.3.4.3. Temperature

Factors influencing Temperature

The temperature of a place is dependant upon some or all of these factors:

1. Latitude, 2. Altitude, 3. Ocean currents, 4. Winds, 5. Aspect 6. Cloud Cover.

Temperature Data Terminology

1. *Average or Mean Temperature* : Arithmetic mean of temperature data.

2 *Normal Temperature:* The arithmetic mean of most recent 10 years Temperature data used as a Standard for Comparison.

3. *Daily Range* : Difference between the Daily Maximum and Minimum temperatures.

4. *Mean Daily Temperature* : Average of the daily maximum and minimum temperatures.

5. *Mean Monthly Temperature* : Average of the mean monthly maximum and minimum temperature.

6. *Mean Annual Temperature* : Average of the monthly means for the year.

7. *Degree Day*: Departure of one degree for one day in the mean daily temperature from a specified base temperature.

Measurement of Temperature

In order to obtain the temperature of the free air, it is very important that the temperature measuring instruments be exposed properly. They must be placed in an open space where the circulation of air is quite unobstructed, but they cannot be exposed freely so to say to the direct rays of the sun. This is usually accomplished by mounting the instruments in an instruments shelter. In order that the observed temperature may be representative of conditions in the free air, it is important that the location of the shelter be typical of the nearby area. The shelters viz. Stevenson screen is so installed that the thermometers are about 1.5 metres. above the ground. Most thermometers for measuring free-air temperature near the surface are of the liquid in glass type. The maximum thermometer automatically registers the highest temperature occurring since its last setting.

The "Thermograph" is an instrument designed to make an autographic record of temperature. The thermometer element is commonly either a bimetallic strip or a metal tube filled with liquid. In the first case, the bimetallic element has the form of a helical coil, one end being rigidly fastened to the instrument and the other linked to the pen. In the second case, the metal tube is made with a tendency to curl. The deformation of the tube, which is generally filled with alochol, actuates a pen. The pen traces the record of temperature, on a ruled sheet which is wrapped around a cylinder revolved by clock work.

Electrical Resistance Thermometers

Electrical Resistance Thermometers are convenient for indicating or recording temperatures at some distance from the point of measurement. They are essentially resistance bridges, with the thermic element forming one arm. The resistance of the thermic element varies with temperature and is indicated on a scale graduated in degrees of temperature. Thermocouples, gas bulb thermometers and other special types of temperature-measuring equipment are frequently used in meeting special problems.

2.3.4.4. Atmospheric Humidity

Water vapour is one of the regular constituents of the atmosphere although at times only very small amounts may be present. The atmospheric moisture has considerable influence on temperature.

Measurement of Humidity

Although numerous instruments have been devised for measuring the amount of water vapor in the atmosphere, the measurement of humidity is one of the least accurate instrumental procedures in meteorolgy.

Instruments

The instrument generally used for making official measurements of humidity in the surface layers of the atmosphere is the psychrometer. It consists of two thermometers, One of which has its bulb covered with a closely fitting jacket of clean muslin saturated with water. The thermometers are ventilated by whirling or by use of a fan or bellows. Because of evaporation, the reading of the moistened (wet-bulb) thermometer is lower than that of the dry bulb, and this difference on degree is called the wet-bulb depression. By reference to appropriate tables, the dewpoint, relative humidity, and vapor pressure may be obtained.

The hair hygrometer consists of a frame in which a strand of hair is kept at approximately constant tension. Changes in length of the hair corresponding to changes in relative humidity are transmitted to a pointer. This instrument is seldom used for meteorological purposes, but it is an inexpensive humidity indicator. The hair hydrograph is essentially a hair hygrometer, but the hair activates a pen, which records on a rotating drum. The hygro thermograph combines the registration of both relative humidity and temperature, on one record sheet.

The Dewpoint hygrometer is a device used to determine the dew point directly. It usually consists of a highly polished metal vessel containing a suitable liquid, which is cooled by any of several methods. The temperature of the liquid at the moment condensation begins to occur on the metal surface is the dew-point. The instrument can be made self recording by observing the metal surface photoelectrically.

The spectroscopic hygrometer measures the selective absorption of light by water vapor in certain bands of the spectrum. This instrument can be used to determine the average moisture content between it and a light source.

Errors in Measurement

All humidity measuring instruments are subject to errors from improper observational technique, but the ordinary psychrometer invites more errors of this type than any other device. First, two thermometers must be read. This doubles the chance of misreading. Second, sling and whirling psychrometers must be stopped to permit reading. Third, the latent heat of fusion causes the mercury in the wet-bulb thermometer to log at the freezing point, which is sometimes mistaken wet-bulb temperature. In addition to these errors are those resulting from the conduction of heat down thé thermometer stem, insufficient ventilation, or thick muslin and inpure water.

2.3.4.5. Evaporation and Evapotranspiration:

General

Estimates of evaporation may be critical in determining the feasibility of a proposed reservoir site and are useful in determining the current operating procedures for a reservoir system. Evaporation and Evapotranspiration are also important elements of any water budget study.

Direct measurement of evaporation and evapotranspiration from large water or land surfaces is not possible at present. However, several indirect methods have been developed which give acceptable results. Evaporation pans and lysi-meters are used in networks. Weighable Lysimeter is shown in fig. 3.10

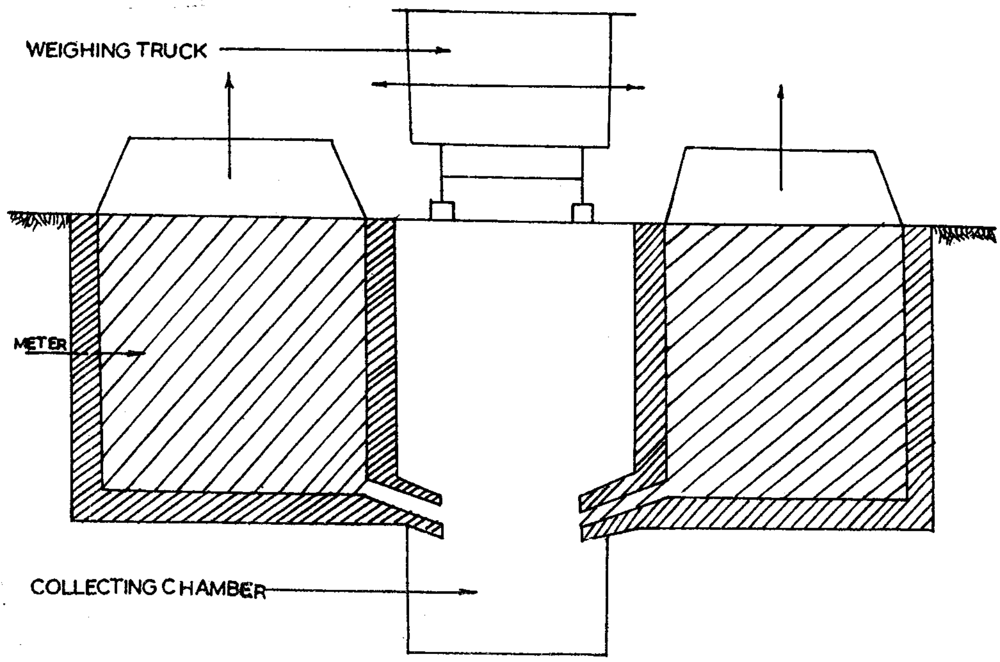


Fig II.3.10 WEIGHABLE LYSIMETER

Pan Evaporation

Evaporation records for pans are frequently used to estimate evaporation from lakes and reservoirs. Many different types of evaporation pans are in use. Among the various types of pans in use there are a few which deserve special attention. There are the U.S. Class- A pan and the USSR CCI-3000 Pan and 20-m^2 tank.

Layout of Evaporation Stations

The site should be fairly level and free from obstructions. At sites where normal climate and soil do not permit the maintenance of a soil cover, the ground cover should be maintained as near as possible to the natural cover common to the area. Obstructions such as trees, buildings, nearby shrubs, or instrument shelters should not be closer than four times the height of the object above the pan. Under no conditions should the pan or instrument shelter be placed on a concrete slab or pedestal, or over asphalt, or layer of crushed rock.

The instruments should be so located on the evaporation station plot as to prevent their casting shadows over the pan. The minimum size of the plot should be 15 by 20 meters. The plot should be fenced to protect the instruments and to prevent animals from drinking the water. The fence should be so constructed that it does not affect the wind structure over the pan.

At unoccupied sites, particularly in arid and tropical regions, it is often necessary to protect the pans from birds and small animals. This may be achieved by the use of :

(a) Chemical repellants. In all cases where such protection is used, care must be taken not to pollute the water in the pan in any way.

(b) A wire-mesh screen of standard design supported above the pan.

In order to get an estimate of the error introduced by the effect of the wiremesh screen on the wind field and the thermal characteristics of the pan, steps should be taken to compare, at the nearest comparable occupied site, readings from the protected pan with those of a standard pan.

Observational Procedure

The water level in the pan must be measured accurately before and after water is added. This may be done in two ways:

(a) The water level may be determined by means of a hook-gauge consisting of a movable scale and vernier fitted with a hook. An alternative arrangement is the use of float. It is essential for the measuring device to be enclosed in a stillwater chamber in the pan; this chamber should be about 10 cm across the top and about 30 cm deep with a fixed point about 6 or 7 cm below the rim of the pan. A calibrated container is used to add or remove water at each observation returning the water level to the fixed point.

(b) The water level may also be determined by the following procedure: (1) a vessel of small diameter fitted with a valve is placed on top of a bench-mark below the water surface in the pan; (2) the valve is opened and the water level in the vessel is allowed to equalize with the water level in the pan: (3) the valve is closed and the volume of water in the vessel is accurately determined in a measuring tube; (4) the height of the water level above the bench-mark is determined from the volume of water in the vessel and the dimensions of the vessel.

Daily evaporation is computed as the difference in water level in the pan on successive days, corrected for the precipitation during the period, if any. When the water level variation in the pan is measured by means of adding water to or removing water from the pan up to the constant water mark in its wall, the evaporation value between the two observations is determined by the formula:

$$E = P \pm d$$

Where P is the depth of precipitation and d is the depth of water added (+) to or removed (-) from the pan.

Several types of automatic evaporation pans are in use. The water level in the pan is automatically kept constant by releasing water into the pan from a storage tank or by removing water from the pain in the case of precipitation. The amount of water added to or removed from the pan is recorded.

2.4. Data Processing and Storage

This constitutes a very important component of the flood forecasting system. Even for the hydrological observation system, the data processing and its proper storage is a very important component. The data observed at the field station is "raw data" and is subject to certain errors - instrumental, observational etc. These raw data are to be processed with a view to:

(a) examine the accuracy of data through various consistency checks etc.; and

(b) remove the probable errors to make the data acceptable for various purposes.

The processed data, which is supposed to be free from all major errors and considered to be acceptable, is then stored in recommended format so that the data is readily available to users as and when required. Further data should be stored with a view to keep them safe for record and future use.

2.4.1. Data Processing

As discussed earlier the different types of errors that are likely to creep in are:

(a) Instrumental error

(b) Observational error

(c) Copying error.

Instrumental Error

The instrumental error may be classified in two groups namely; (i) errors of sudden or emergent nature, and (ii) errors which creep slowly over a long time. Some example of errors of the first type are as follows:

(a) Washing away of gauge post and error during fixation of the new gauge posts by staff members not fully trained for the job.

(b) Slippage of the pen holder on float rod of the self recording raingauge.

A careful examination of the data may reveal such errors. For example an unexpected rise in the water level or sudden drop and again a rising trend in the water level may be because of sudden errors. These can be checked and correction applied after necessary verification. In case of the chart obtained from a raingauge, a sudden drop will be indicative of such errors.

The errors which creep slowly over the long time are rather difficult to be noticed during the routine data processing. Such errors in the water level data, for example, may be due to slow settlement of gauge posts in sandy beds. These errors can be detected by frequent checking of the gauge posts and by comparison of the record with the similar record from the nearby gauge posts/gauge marks of other agencies (Stage Government etc.). Such errors in the rainfall data may creep in as a result of growth of trees around the raingauge stations which may restrict the catch. a consistency check may help in detection of such errors.

Observational Errors

These are the errors made by the observer due to his ignorance or carelessness. Most of the time such errors can be noticed in gauge observations record by a proper check. Such errors can be easily detected and minimised considerably by proper training to the observer, regular supervision and routine checks. It is necessary to ensure that the procedure for observations are strictly adhered to.

Copying Error

Many a time it happens that a person takes a correct reading but records wrongly; for example, a reading of 3.64 may be recorded as 3.46. Sometimes the observations are made and reported to another person who may make wrong entries. For example, during discharge observation, measurements of depth, velocity etc. are made by one person and recorded by another person.

Such errors can again be eliminated when the procedure of repetition is strictly followed.

Yet another source of error is due to communication gap. For example, when the gauge post at the proper site is washed away, the observer moves to alternative site for the observation and transmits the data of the new location without mentioning the fact (i.e. the change in the location of the observation) to the control room. As a result, the variation in the water level at the two locations goes unnoticed and hence the error. Such errors are mainly because of lack of sufficient training to field staff and these can be avoided and detected by proper training to the concerned officials.

To error is human and therefore the errors of one or the other type are bound to creep in despite the best efforts to avoid them. Hence, the processing of the data becomes necessary before these are used for operational purpose. Some of the commonly used steps in processing of hydrological and hydrometeorological data are:

1. Preliminary checks by the field officers;

2. Thorough checkings at control room including
(a) consistency checks and (b) graphical presentations;

3. Rectification of identified "doubtful data" through accepted techniques as well possible field checks ; and

4. Presentation of the observed data in standard format.

2.4.1.1. Preliminary Checking of Data

The term preliminary checking will be used to cover the procedure which are required to be performed by field offices before sending the data to control room/Head quarters. This includes:-

(a) logging of data in receipt form;

(b) ensuring completeness and correctness of indicative informations i.e. dates and stations names including station numbers etc. ;

(c) ensuring completeness of data for station including totals, means, extreme values (they are necessary and record of necessary remarks etc.); and

(d) Checking by local supervisor.

2.4.1.2. Checking at Control Rooms/Head Quarters

The data received from field are to be thoroughly checked at the control room and this checking should include:

(a) Completeness of data - It is to be ensured that all the necessary field observations have been made and duly recorded at appropriate places;

(b) Computational checking - The computations are to be thoroughly checked;

(c) Cross checks - At this stage the various types of data are to be compared with the other possible estimates. For example the observed discharge data can be compared with the estimated value of discharge corresponding to observed stage as obtained from an established rating curve. Similarly, the rainfall data of a self recording rain gauge can be compared from the report of the ordinary raingauge and vice -versa.

(d) Consistency check - The standard methods of consistency check for different types of data should be applied.

(e) Graphical representation - The collected data should be graphically represented (e.g. continuous gauge or discharge hydrograph, bar graph for the daily rainfall data etc.).

2.4.1.3. Rectification of Doubtful Data

Once the data is found to be erroneous it has to be corrected by adopting standard techniques of rectification. However, these corrections are to be made very carefully after making all sorts of checks and rechecks preferably with possible field verifications.

2.4.1.4. Data Recording

The processed data should be recorded in the prescribed form as early as possible. The records in the register should be thorough and complete and the entered data should be checked at least by two persons including certain percent of random check by the Incharge. The format should have complete record in respect of the station, data and time of observation and there should be specific mention of the fact if the recorded data has undergone some modification at processing stage. In case the data has not been observed or observed data has been discharged and estimated value has been recorded in the register then such facts should be clearly mentioned.

2.4.2. Data Storage

The most commonly used means for storage are;

(a) Data register/published data books;

(b) Computer tapes/discs; and

(c) Micro films.

It is always desirable to have more than one copy of the record preferably stored at different locations.

Storage conditions for any of the media should be such as to minimize destruction of stored records by excessive heat, rapid temperature fluctuations, high humidity, dust, insects or pests and by fibro magnetic tapes should be safeguarded from electromagnetic influences. Non-flammable film should be used in microfilming. Basic climatological and hydrological data are irreplaceable and invaluable. Every precaution should be taken to protect them from destruction. Where possible, duplicate sets of records should be kept. The arrangement of data for cataloguing or transferring data to microfilm is of particular concern in hydrology. For example, for flood peak discharge frequency at a stream gauging station, the whole period of record for the individual station should be readily available. Like wise for storm rainfall analysis, all the data for a region for a particular period be filed together. In many cases it is desirable to subject basic observational data to some analysis before or immediately after publication and to put the data in a form most useful to the users.

CHAPTER-III

DATA TRANSMISSION

3.0 Introduction

3.1 General

Any attempt to monitor a randomly varying situation cannot be effective unless it is backed by a very reliable data communication system. Developments in the field of communication technology have played a vital role in better management of such monitoring and decision taking mechanisms, such as those involved in flood forecasting networks. The merit of a communication system is governed by the following features:

1. Reliability of the system
2. Speed of communication
3. Accuracy of communication
4. Redundancy
5. Scope for expansion
6. Economy in installation and maintenance

As we are in the computer age, computer compatibility of the system has also become another important criteria to adjudge the merit of a communication system. Sometimes the ability to communicate coded and classified messages would be also relevant.

3.1.1 Need of Real Time Data Transmission

Since formulation of flood forecast is based on hydrological and hydrometeorological conditions in the river as prevailing at up-stream locations (called base stations) as reflected by measurements of water level, river discharge, rainfall, run off etc, it is imperative that such real time data relating to base stations be made available to the control rooms through the quickest possible channel of communication. Any avoidable loss of time and accuracy in the transmission of data from the base stations would reduce the warning time and adversely affect consequential relief and rehabilitative measures. Thus communication system would constitute the backbone of an effective operational flood forecasting network.

3.2 A Review of Various Systems of Data Transmission

3.2.1 Land Line

In the early years the use of land line communication such as telephone/telegraph channels of communication through P&T Department were widely deployed. The Posts and Telegraph Department authorise different high priority transmission categories depending on the nature of the message. The telegraphic mode of communication cannot however always provide a dependable, speedy communication system due to its inherent limitations. The non-availability of telegraph office nearby all the base stations, which are generally situated in remote areas near the rivers was primary hinderance. Secondly, the speed and fidelity of communication in this mode of communication were not always what was needed for flood forecast systems. Frequent failures of the communication due to faults in cable system, introduction of noise in loop, relaying of message in morse code introducing possible distortion in messages at different stages posed difficulties in flood forecasting. For local communication of flood messages i.e. say from barrage/Dam to local wireless control room and for quick dissemination of forecast or for sending local messages to different civil and engineering authorities, the telephonic system was easily used. The introduction of subscriber Trunk Dialling facility and its expansion linking important cities/towns in the country enhanced its usefulness. Meteorological informations from Flood Meteorological Offices are generally received through telegraphic mode of communication. In recent years, the teleprinter system is also used if F.M.O. and the Divisional Office of flood forecasting are situated at the same place or alternatively 'Telex' system was used.

3.2.2 Wireless

Since the sixties, there was a steep rise in the number of forecasting stations and simultaneously in the number of base stations. It soon became imperative to opt for wireless communication in view of the limitations of land line communication. To begin with, the help of wireless communication system of Police Department was taken. Thereafter, the Central Water Commission established its own dedicated network of wireless stations to facilitate flood forecasting activities. Notwithstanding this wherever necessary the help of Police Wireless network was also taken for the communication of flood data from a number of temporary sites usually established during monsoon only and raingauge stations under some of our field Divisions.

Wireless communication may consist of both H.F. (High Frequency) and V.H.F. (Very high frequency) network. The V.H.F. network is generally used for the communication in short aerial distance range i.e. 30 to 40 km whereas the H.F. network provides communication in long aerial distance range i.e. 250 to 400 km and even more depending upon the radio frequency, ver of the wireless equipments owing to their propagation characteristics.

The easy installation, operation and maintenance property of the transreceiver set as well as its ability to meet the basic requirements of a good communication system have made it the most viable mode of communication. The long distance communication property of the high frequency wireless set with the great degree of reliability and fidelity has made it most popular and widely used mode of communication in flood forecasting in view of the elimination of relay requirements between the number of stations spread over in vast areas.

3.2.3 Telemetry

In addition, there are certain remote locations in the various river basins where personnel cannot be positioned but data from which stations are of vital importance for flood forecasts. Such locations could best be linked up through a telemetry system. In the telemetry system, different sensors read the data such as water level, rainfall, temperature etc. on receiving a command from the master station through teleprocessors and the data are transmitted on realtime basis through VHF set in coded form in a programmed sequence which get decoded by the combination of modem and Master teleprocessor. With the successful Indian entry into the field of space technology, a very bright scope for development of the radio communication has come up at least in those fields which need real time communication of events.

3.3 Data Communication Network in Central Water Commission

There are two types of network structures forming the data communication system for flood forecasting.

1. Internal network of Divisions

2. Interlinking network of Divisions and nodal offices.

3.3.1 Internal Network of Divisions

The internal network of each Division is the independent interference free radio communication network of the Division consisting of wireless stations of base stations, sub-divisional control rooms and Divisional Control room. The real time data are transmitted by base stations to sub-divisional control as well as to Divisional control during the wireless schedules as fixed by Divisional Incharge. The Divisional and sub-divisional controls also have the facility to communicate by telephone during the nonschedule hours. The structure is shown in fig. III. 3.1.

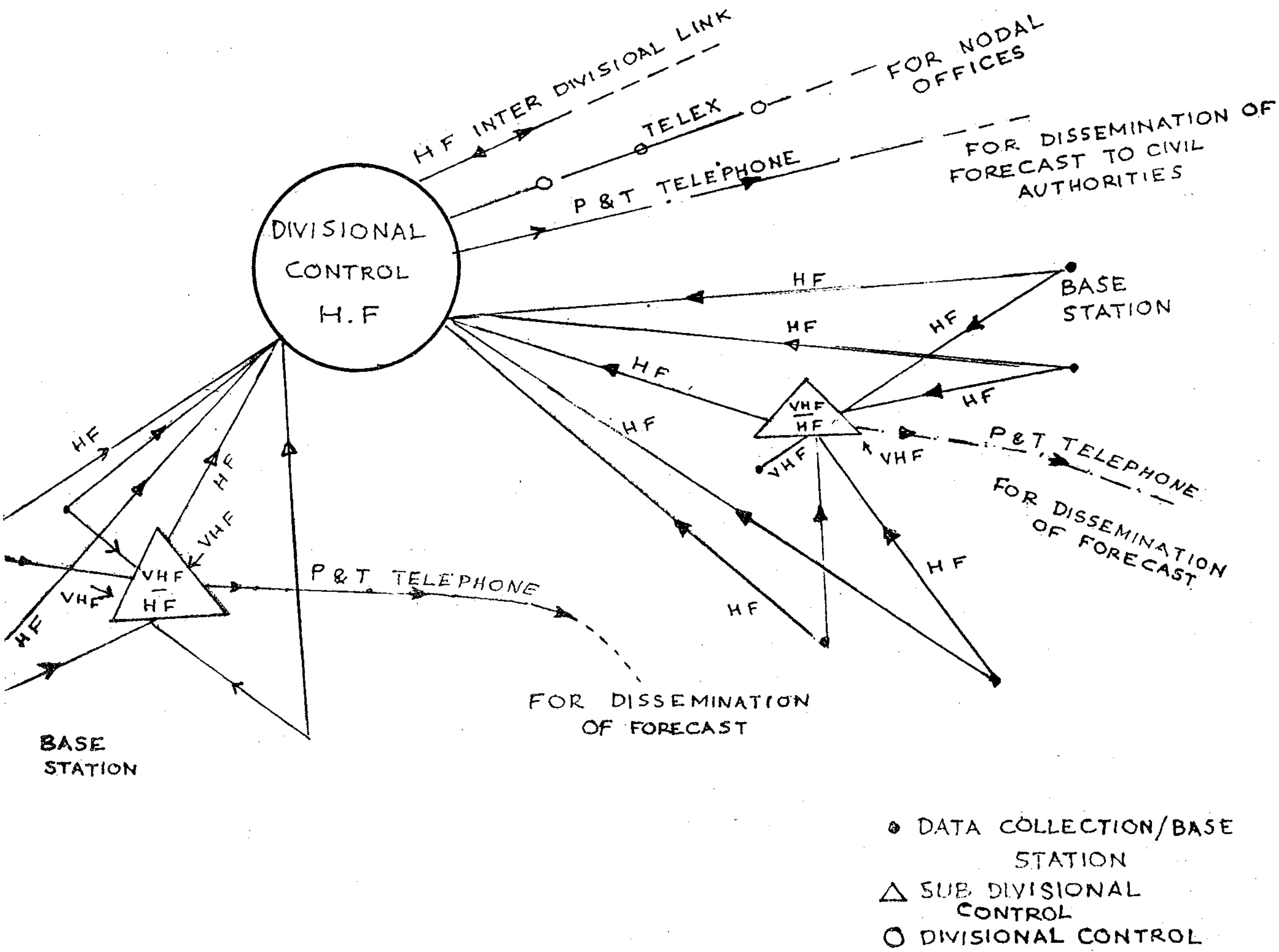


Fig. III 3.1 Internal Network Of Divisions.

3.3.2 Interlinking of Divisions with Nodal Offices

All divisional headquarters are linked with their respective controlling offices for passage of flood messages for their supervision and review of the functioning of flood forecasting system. All nodal offices of CWC i.e. Circles, offices of Chief Engineers and the Commission Headquarters except the Western River Circle, Nagpur are connected on high frequency radio links as shown in fig. III. 3.2. These offices are also provided with telex/STD telephone facilities for land line communication. The network is shown in fig. III. 3.2.

3.3.3 Wireless Communication

The widely adopted method for communicating the flood data is through wireless network in view of its prompt and reliable service.

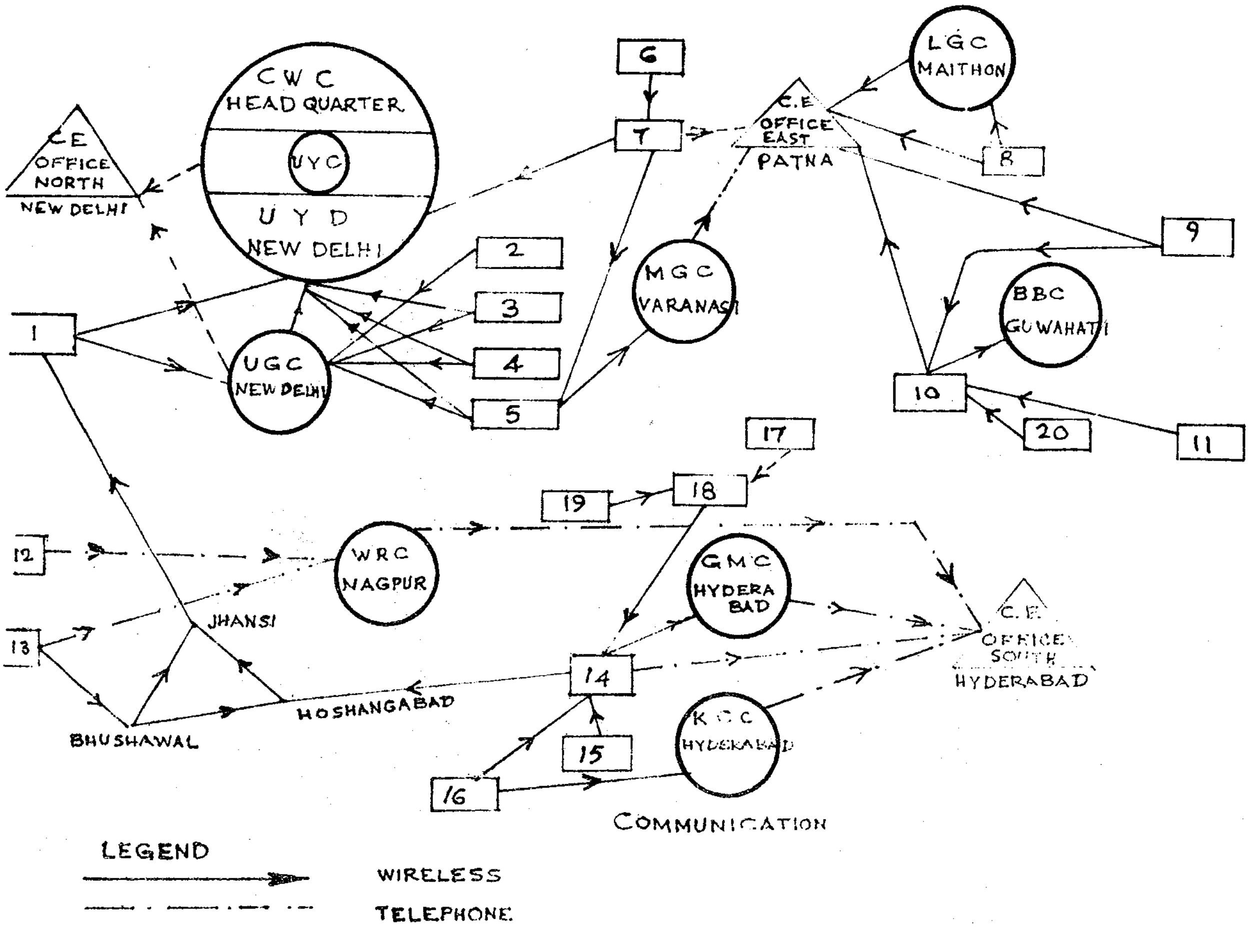
In this system, the communication of speech or telegraphic signal is accomplished by utilising the propagation technique of radio waves which are transmitted and detected by suitable electronic equipments. The essential properties of a radiowave are the frequency, intensity, direction of travel and plane of polarization. Radio waves are classified according to frequency in cycle/sec. or Hertz and wavelength in meters. The distance covered by one complete cycle of such alternating wave is termed as wave length which has the relationship with the frequency in following manner.

$$\frac{\text{Velocity of light}}{\text{Frequency in C/s}} = \text{Wave length}$$

$$\text{i.e.} = \frac{3\text{x}10^8 \text{ meter/s}}{\text{F in C/s.}}$$

Thus a low frequency wave has a long wavelength whereas high frequency corresponds to a short wave length. The role of the wavelength parameter is of very important consideration in ascertaining the size of the radiating element. The intensity of a radio wave is the voltage stress produced in space by the electric field of the wave and is expressed in microvolt per meter. This is the factor which decides the optimum distance of the radio communication coverage. The radio wave travels in the direction at right angle to the wave front which is a plane parallel to the mutually perpendicular lines of electric and magnetic field depending upon the relative direction of these fields. The direction of the electrical field is called the direction of the polarization of wave.

The range of frequencies in use in radio communication extends from 10 KHz to 30,000 MHz; the radio waves are classified in terms of frequency as given in table that follows :



1. LYD, Agra
2. HGD, Dehradun
3. MGDI, Lucknow
4. MGD, II, Lucknow
5. MGD, III, Varanasi
6. MGD, IV, Patna
7. MGD, V, Patna
8. Damodar Divn, Asansol
9. LBD, Jalpaiguri
10. MBD, Guwahati
11. UBD, Dibrugarh
12. Mahi Divn., Ahmedabad
13. Tapti Divn., Surat
14. LG Divn., Hyderabad
15. LKD, Hyderabad
16. UGD, Hyderabad
17. ERD, Bhubaneshwar
18. BSD, Bhubaneshwar
19. MD, Burla
20. BD, Guwahati

Fig. III.3.2 Interlinking of Divisions

S. No.	Designation	Abbre- viation	Frequ- ency Range	Wave Length	Propagation Characteristics	Typical use
1.	Very Low frequency	VLF	10-30KC	30,000-10,000m	Low attenuation.	Long distance point to point communica- tion.
2.	Low Frequency	LF	30-300KC	10,000-1,000m	Less reliable than VLF-day-time absorption greater than VLF but similar in night.	Long distance point to point communication service, naviga- tional aids, marine.
3.	Medium Frequency	MF	300-3000 KC	1000-100m	Attenuation low at night and high in day time, ground wave propagation.	Broadcasting, marine communica- tion navigation, harbour telephone etc.
4.	High Frequency	HF	3-30MC	100-10m	The propagation is through ionosphere so it varies greatly with time of day, season and frequency.	Moderate and long distance communicat- ion of all types.
5.	Very high frequency	VHF	30-300MC	10-1m	Substantially straight line propagation, unaffected by ionosphere	Short distance communicat- ion, television, airplane navigation, radar telemetry.

S. No.	*Designation*	*Abbre-viation*	*Frequ-ency Range*	*Wave Length*	*Propagation Characteristics*	*Typical use*
6.	Ultra high frequency	UHF	300-3000MC	100-10cm	-same-	Short distance communi-cation, Radar, Relay system, television, satellite
7.	Super high frequency	SHF	3000-30,000MC	10-1cm	-same-	Radar, radiorelay, navigation, satellite.

The frequencies higher than 2000 M/s are commonly referred to as microwave frequencies.

Any varying current carrying conductor radiates a certain amount of electrical energy in the form of electro-magnetic waves which depend upon the size and dimension of the conductor compared to wavelength of the current. If the dimensions of the current carrying conductor approach the order of the magnitude of wave length, the radiation will be significant. Thus, high frequency waves can be radiated by a small radiator while low frequency waves require large radiating system for effective radiation. The radiator is often termed as 'Antenna'.

Since the concept of isotropic antenna is hypothetical, all practical antennas have some directional characteristics and maximum energy of radiation would concentrate in a particular direction.

If a radio wave is to convey a message, some feature of the wave must be varied in accordance to the signal representing the information and that is called modulation. The signal whose features are varied is called 'carrier' and the signal responsible for the variation is called 'modulating signal' whereas the resulting signal after such variation is called 'Modulated signal'. The variation of any of the three features, i.e. amplitude, frequency and phase while keeping others constant, in accordance to the instantaneous magnitude of the modulating signal, produces corresponding modulated wave e.g. amplitude modulated, frequency modulated wave etc. In radio telegraphy, this involves turning the radio transmitter on and off in accordance to dots and dashes of the telegraph code as shown in fig. III.3.3. In radio telephone transmission by using amplitude modulation, the amplitude of radio frequency wave is varied in accordance to the pressure of sound wave being transmitted as shown in fig. III.3.4. The amplitude modulation can be used in any range of frequencies whereas the frequency and phase modulation are widely used in very high frequency of communication.

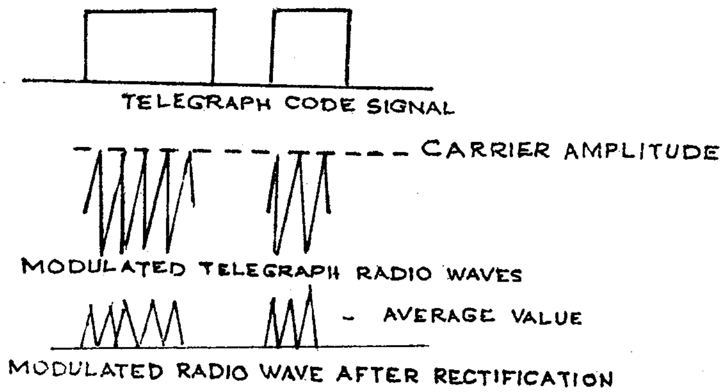


Fig III 3.3 Radio Telegraph and rectification.

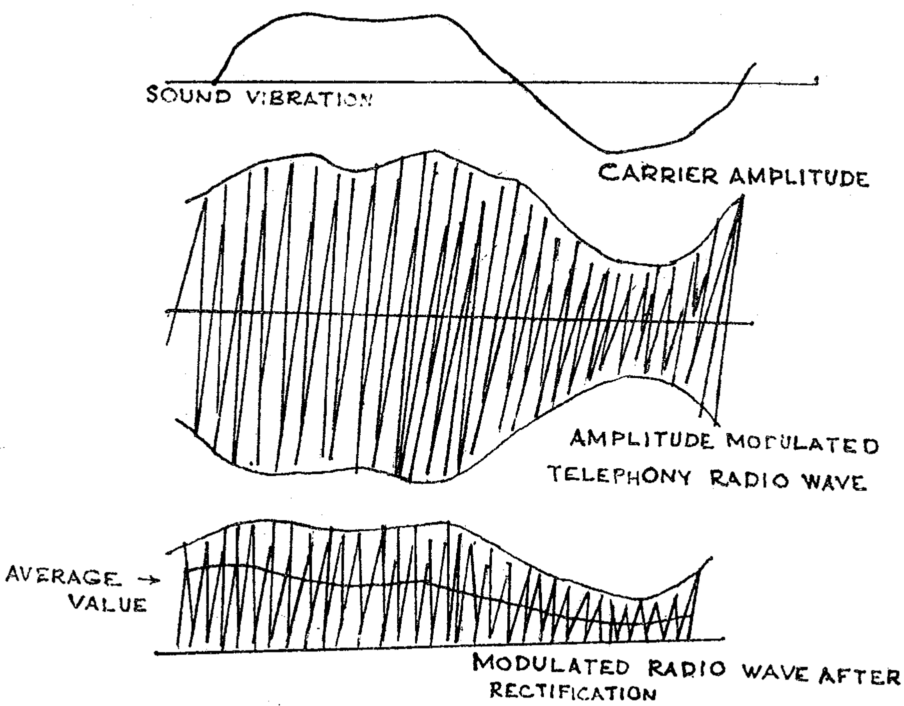


Fig. III 3.4 Amplitude modification and rectification.

The amplitude modulated wave consists of three components. The component whose amplitude is independent of the presence or absence of modulation and is equal to the average amplitude of radio wave is called carrier. The two other components are alike in magnitude but differ in frequency i.e. one of them is less than that of the carrier frequency by an amount equal to the modulation frequency while other is more than that of the carrier by the same amount. These two components are called "side band frequencies" and each of them carry the intelligence that is being transmitted by the modulated wave. Since each side band carries the intelligence, it is possible to convey all the information represented by a modulating signal by transmitting only a single side band by suppressing the carrier and other side band which not only saves over two third of the power but also narrows the requirement of frequency bandwidth to half of the width as required for carrier and two side bands. For these reasons, 'single side band' transmission finds extensive use in carrier line communication and radio telephone communication.

The reception of radio signal is to abstract energy from the radiowave passing the receiving point. Any antenna capable of radiating electrical energy is also able to absorb energy from a passing radio wave. In fact, a time varying voltage is induced by the electromagnetic lines of forces as present in the radio wave when it cuts the antenna conductor and constitutes a current. The time variation of induced current is exactly identical to that of current flowing in antenna radiating the wave. This induced voltage, in association with the current that it produces, represents energy that is absorbed by the passing wave. Since every wave passing across the receiving antenna induces its own voltages in the antenna conductor, it is imperative that the receiving equipment be capable of identifying the desired signal from unwanted signals. The process of adjusting circuits to resonance with the frequency of desired signal is called 'Tuning'.

3.3.3.1 Wireless Set

The transreceiver, popularly known as wireless set, is a device which is capable of both producing radio-frequency energy that is controlled by the intelligence to be transmitted as well as reproducing the intelligence after deducting it from radio frequency energy to be received. It can be used either as radio telephone or a radio telegraph and can use either amplitude modulation or frequency/phase modulation.

A wireless set consists of mainly three units:

1. Power Supply

2. Transmitter

3. Receiver

In a low power wireless set these three units are housed together in one unit as a compact design but this is not true for medium and high power wireless sets in which transmitter and receiver are kept as independent units. In some types of medium and high power sets the power supply unit is housed in transmitter unit otherwise it is also kept as an independent unit.

3.3.3.2 Power Supply

A power supply is a circuit of device which changes the primary electric power into the kind and amount of a.c. and d.c., as needed by the different types of electronic circuits of the transmitter and receiver. This unit derives power from either a.c. mains or battery. In the complete solid state wireless set, only d.c. of different voltage levels are needed whereas in hybrid types of set both a.c. and d.c. are required.

A typical power supply consists of a transformer, a rectifier and filter. The transformer transfers a.c. power from a primary to a secondary circuit by inductive coupling and enables upward or downward stepping of voltage level. The rectifier converts a.c. voltage into pulsating d.c. voltage of nearly same level. The filter facilitates the smoothing of the pulsating D.C. output of rectifier to provide steady D.C. voltage.

Where electric power is derived from battery, the different levels of D.C. voltage are got by DC to DC convertor and potential divider.

3.3.3.3 H.F. Transmitter

The block diagram of typical SSB HF transmitter is shown in fig. III. 3.5. Here the single side band is produced by modulating a low frequency, e.g. 455 KHz. carrier using a balanced modulator and then separating one of the side bands by means of crystal filter or say mechanical filter. The resulting single side band is transformed first to an intermediate frequency say 1145 KHz. by hytrodyne action and then to still higher frequency i.e. channel frequency by a second hytrodyne action. This is all done at low power level, after which the power is increased to the desired value by a chain of linear amplifiers. The frequencies of first and second crystal oscillators are fixed for a particular model of transmitter by its manufacturer whereas the frequency of channel oscillator is fixed by the user as per the authorisation of frequency to him.

The work of various components are described below:

Microphone

It converts sound waves into corresponding electrical waves popularly known as audio signal.

Audio Compressor

It prevents the amplitude of audio signal going beyond some specific level.

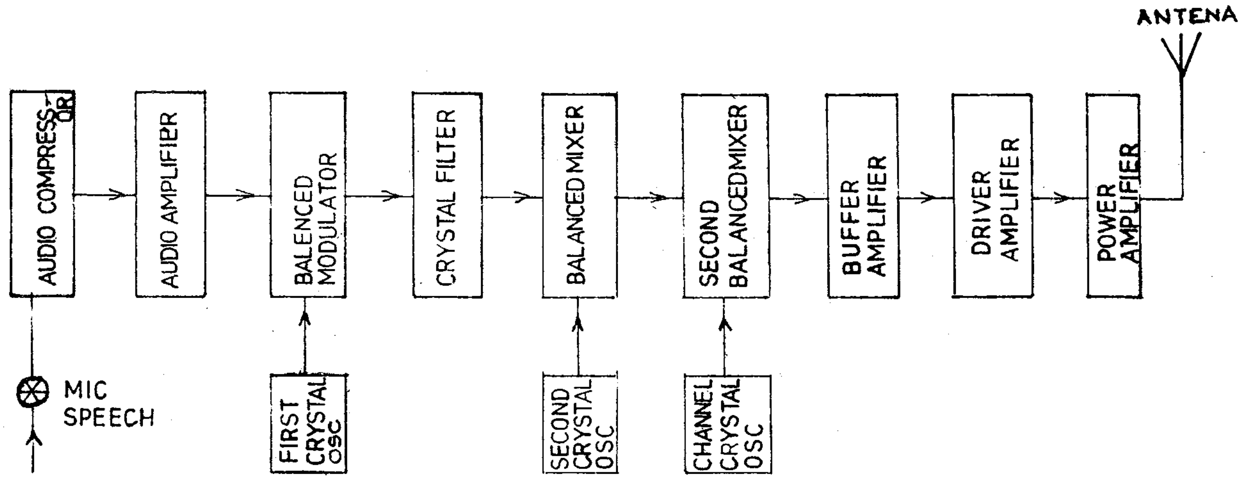


Fig. III.3.5 Block diagram of a typical SSB trasmitter.

Audio Amplifier

It amplifies the audio signal to meet the voltage level requirements of the next stages without adding distortion in its amplitude and frequency.

Crystal Oscillator

It generates signal of a very stable frequency as per the frequency specification of crystal.

Balanced Modulator

In this, the audio signal modulates the signal of first oscillator. The carrier component of the modulated signal is suppressed and only two side bands appear at its output.

Crystal Filter

It sharply filters out the undesired side band enabling only one side band to appear at its output.

Balance Mixer

In this, signals of two frequencies are interacted to produce two signals of frequencies of the value as determined by the summation and difference of interacting frequencies. It means, if frequencies f_1 and f_2 are interacted, the result would be the signals of the frequency $f_1 + f_2$ and $f_1 - f_2$. However, one set of frequency is bypassed depending upon the selection of side band leaving other to appear at its output. The interaction of frequencies for its translation to another frequency is called heterodyne action.

I.F. Amplifier

It amplifies the signal so obtained from the mixer.

Buffer Stage

The stage is used to isolate the driver stage from mixer so that the former may not load the latter.

Driver Stage

It is also called amplifier stages in which the signal from buffer stage is amplified in more than one stage to attain the voltage level to meet the requirements of power amplifier.

Power Amplifier

It is the final stage in which incoming signal from the previous stages gains sufficient power so that it can be transmitted through an antenna system.

3.3.3.4 H.F. Receiver

It is a device which intercepts and demodulates the signal of desired frequency to reproduce the intelligence. Thus, its working is the inverse of the transmitter. The block diagram of a typical receiver is shown in fig. III.3.6

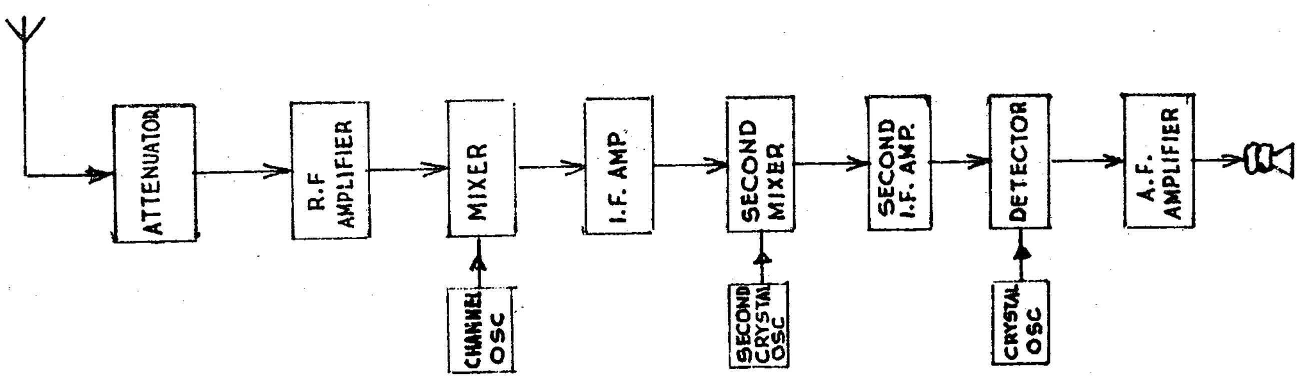


Fig III 3.6 Block diagram of a typical SSB reciver.

The works of these circuit blocks not used in transmitter are explained below:-

Attenuator

It is the unit which helps the signal voltage not to grow beyond some specific value to drive the R.F. stage as well as isolate the R.F. stage from the loading of antenna stage.

R.F. Amplifier

It is tunable radio frequency amplifier which amplifies the intercepted signal of desired frequency rejecting the other sharply.

Detector:

In this stage, the signal is demodulated by rectifying the wave, and thereby obtaining a pulsating direct current varying in amplitude in accordance with the modulation envelope or say intelligence of the wave.

Loud Speaker

It is a transducer which converts electrical audio signal into sound signal.

The reception of the radio telegraph signal is accomplished by injecting an oscillation provided by a fixed oscillator into the intermediate frequency system that differ from the centre of intermediate frequency band by 1000 to 2000 cycles. Hetrodyne action between the incoming signal and this beating oscillator causes the detector output of receiver to be in the form of audio tone. In this case of on-off Keying, this tone is of constant pitch reproducing dots and dashes.

3.3.3.5 V.H.F. Transmitter

The VHF transmitter as used in radiotelephony, works on the phase modulation technique because its primary objective is to transmit intelligible communication which can be accomplished by even low frequency deviation what phase modulation technique provides, as quality of faithful reproduction of the original sound at receiver end is not a major criteria. Further, it has the advantage that the carrier frequency is derived directly from a crystal oscillator and so is inherently very much stable in frequency.

The modulated wave is generated at a low power level, and a chain of amplifiers and harmonic generators are used to develop the required transmitter power and frequency. In the pre-emphasis circuit, the amplitude of the higher frequencies components of the modulating signal are proportionally increased to facilitate the noise suppression at receiver. The block diagram is shown in figure III.3.7

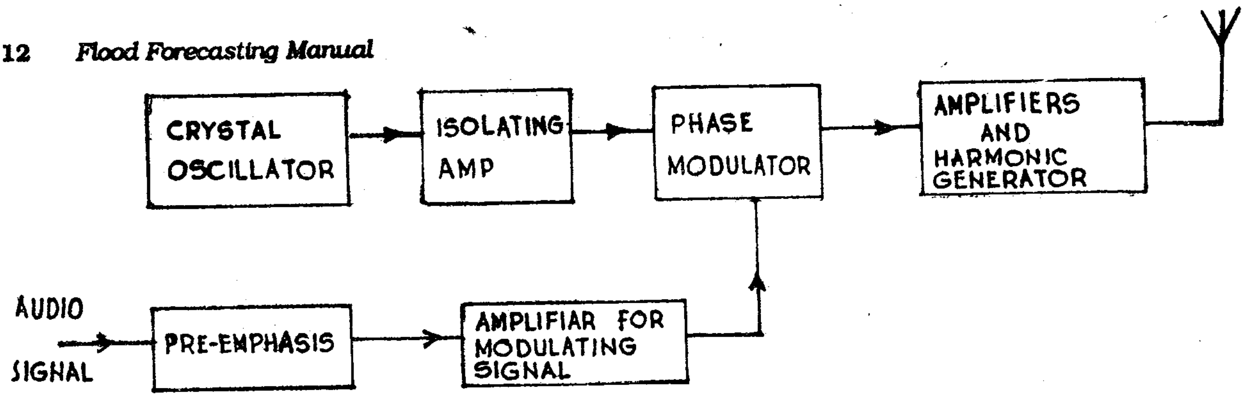


Fig III 3.7 Block diagram of V.H.F. Transmitter.

3.3.3.6 VHF Receiver

The block diagram is shown in fig. III.3.8 which is self explanatory.

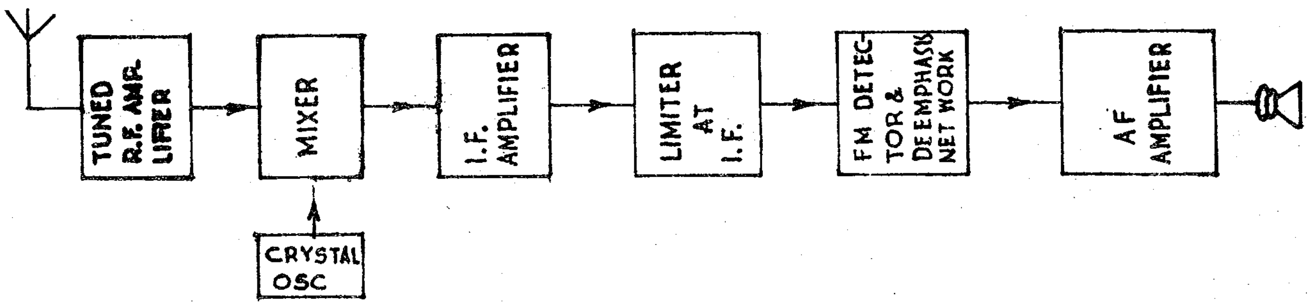


Fig. III 3.8 Block diagram of V.H.F. Receiver.

Limiter is used to reduce the amplitude of signal in order to facilitate the minimization of noise at receiver output.

The deemphasis circuit facilitates the restoration of the amplitude of the higher frequencies components of the intelligence to their proper relative amplitude by proportionately reducing the amount of preemphasis employed at transmitter.

3.4 Wireless Sets in C.W.C.

Wireless Sets being used in Central Water Commission can be classified as under:-

1. H.F. Transreceiver sets
2. V.H.F. Transreceiver sets.

In Central Water Commission, most of the wireless sets are of HF type. Its success lies in the mode of propagation, which facilitates long distance radio communication with low power sets. For the operation of H.F. network, different independent frequencies in the band of 3 to 10 MHz have been allocated by Ministry of Communication to different field Divisions of flood forecasting organisation. However, Ministry of Communication has authorised the CWC to operate on 82.9 MHz frequency throughout the country for VHF network in view of short distance communication.

The usefulness of wireless communication in the flood forecasting can be understood by the number of wireless stations being operated in the organisation which is more than four hundred. At present there are 584 Nos. of H.F. transreceiver sets and 72 Nos. of VHF sets available with the organisation. In addition, to ensure the direct communication by wireless between circles, Chief Engineers' office and Commission's Headquarters, the installation of high power HF sets is under process.

3.4.1 H.F. Transreceiver

100 Watt SSB Set

The 100 W SSB wireless set is a single side band, amplitude modulated, operated from A.C. mains, single phase 50 c/s- 230 volts. The frequency range lies in between 2 to 16 MHz. The set is designed for short to medium range communication on radio telephony, carrier wave (CW) telegraphy. In general the communication range lies in between 400 to 500 km, but under favourable conditions it can work upto 600 km. The maximum power radiated by this set is greater than 80 watt peak envelope power (P.E.P.). It is suitable for stationary as well as for mobile use but in CWC, it is being used as a fixed station. M/s B.E.L., Bangalore have now stopped its production and introduced a new model, Fixed channel transmitter type MHS 117 in conjunction with Fixed channel HF receiver mode HS 419.

General Characteristics

1. Communication range	Skywave propagation, Dipole Antenna-about 500 km.
2. Mode of Operation	A3J (RT) SSB, suppressed carrier (selectable side Band).
3. Power Consumption	245 VA in standby 495 VA in full power.
4. Operating temperature range	0°C to 55°C
5. Audio output	Greater than 400 milliwatt
6. Power output	80 watt (P.E.P.)

15 Watt GE 524 Transreceiver

The 15 watt GE 524 transreceiver is a solid state (fully transistorised), compact, portable and 12 volt D.C. operated set. It can work on fixed channel in frequency band of 2 to 30 MHz and can be used either as a DSB (Double Side Band) or SSB (Single Side Band). It can be used as a fixed/Base station and Mobile station also. When it is used as fixed base station, dipole antenna is used by CWC. When it is used as a mobile station, the communication is done by using whip antenna alongwith loading coil and antenna matching unit; which is used for connecting and matching the impedance of the set with the impedance of the antenna. The mobile set is mostly used for monitoring purpose. This antenna system is workable for short distance coverage.

General Characteristics

(1) Communication Range

(a) Ground wave propagation	Whip aerial-25 kms.
(b) Skywave propagation	suitable aerial system (Dipole antenna used in CWC)-250 kms.

(2) Mode of Operation	SSB, A3J (Radio Telephony in CWC) USB/LSB.
(3) Power consumption	About 30 watt (in Tx condition) about 1.2 watt (in Rx condition)
(4) Audio Output	2.5 milliwatt
(5) Power Output	8 to 15 watt PEP/CW

However, M/s. BEL. Bangalore, manufacturer, have now discontinued its production and introduced a new model named LHP 228 which possess almost the same features except that only USB mode of operation is available in new model and it operates with 24 volt. D.C.

ECIL C-5210

It is a fully solidstate, HF SSB transreceiver, versatile in use operated with 12 volt D.C.

Communication range distance in optimum condition	Sky wave, covering 250 km of crow
Power consumption	Tx 2.5 amp on SSB Rx 125 MA at 11.5 Volt
Frequency range	2 to 18 MHz

Audio output	1 MW into 300 ohm, 200 MW into 20 ohm at 1 KHz
Power Output	20 Watt PEP ± 1 dB into 50 ohms.
Temperature range	0° C to + 55° C

100 Watt HF Fixed Channel Transmitter Type MHS 117

It is compact, medium power solid state general purpose equipment intended to provide radio links for medium ranges i.e. upto 500 km in fixed and mobile role.

Power requirement	230 Volt AC 50 Hz. It can also be operated with 24 Volt DC with proportionate reduction in power output.
Mode of Operation	SSB (USB)/Telegraphy/Teletype.
Frequency range	2 to 21 MHz
Power Output	100 W ± 1 dB (PEP/RMS) for Al, A3J (USB), A2J (USB)
Power consumption	≤600 VA
Temperature range	0° C to 55° C.

Suitable for use with fixed channel HF receiver model HS 419 BEL make.

3.4.2 V.H.F. Transreceiver

MF-753A

The MF-753A is a 50 watt, phase modulated VHF transreceiver set, operated from A.C. mains, single phase, 230 volt supply in the frequency band of 70 to 88 MHz. The transmitter and receiver units of the set are independently crystal controlled and can be operated either on simplex or Duplex basis.

General Characteristics

1. Communication Range	Upto 50-70 kms.
2. Mode of Operation	Simplex, Radio, telephony

3. Power Output	45-80 watt
4. Audio Output	1 watt
5. Operating temperature range	0°C to + 55°C.

GH-650

The GH-650 is a 15 watt, portable, phase modulated VHF transreceiver, operated by 12 volt D.C. source. The range of communication of this set is about 15-20 kms but under favourable conditions, it can be upto 30 kms.

General Characteristic

1. Mode of operation	Simplex, telephony
2. Power consumption	84 watt in Tx condition 7.2 watt in Rx condition
3. Audio output	about 1.0 watt
4. Power Output	about 15 watt
5. Operating temperature range	-10°C to 55°C
6. Frequency Band	68 to 88MHz.

GH-301

The GH 301 is a 15 watt solid state (fully transistorised) portable, phase modulated VHF transreceiver suitable for fixed station as well as mobile role. The set is operated by 12 volt D.C. source (Battery) in the band of 68 to 88 MHz. The whip antenna is used in mobile role whereas flag pole antenna is used in fixed station.

General Characteristics

1. Mode of Operation	Simplex, telephony
2. Power consumption	48 watts in Tx condition
3. Audio Output	About 0.5 watt
4. Power Output	About 15 watt
5. Operating temperature range	–10°C to + 55°C.

LVP-313

The LVP-313 is a 2 watt, fully solid state, light and compact phase modulated man pack VHP transreceiver set. It is operated from self contained 12 volt battery in the frequency range of 68 to 88 MHz. It is used for short distance upto 5-10 kms only.

General Characteristics

1.	Mode of operation	Simplex/Press to talk
2.	Power consumption	9 watt in Tx condition 2.4 watt in Rx condition
3.	Audio Output	500 Milliwatt.
4.	Power Output	2 watt
5.	Operating temperature range	−10°C to +55°C

3.4.3 Accessories

Antenna for Wireless Sets

Antenna plays an important role in wireless communication system. With the selection of a suitable antenna, the distance of communication can be increased. The function of the antenna is to transfer R.F. Energy into space when transmitting and to absorb the same when receiving it.

The different types of antennas used with the above mentioned wireless sets are given below:

1. Dipole Antenna
2. Flag Pole Antenna
3. Whip Antenna
4. Ground plane Antenna
5. Yagi Antenna.

Dipole Antenna

'Horizontal half wave length Dipole antenna, a resonant radiating element, is commonly known as Dipole antenna which can radiate and receive radio signals equally well in two directions. This is also called as Bi-directional antenna and extensively used for H.F. sets for long distance coverage using sky wave propagation. It consists of two single wires or rods or tubes of good conducting material each equal to one quarter of wave length as shown in figure III.4.1. But to get the optimum performance, the total antenna length should be slightly less than half wave length ($\lambda/2$) and the height of antenna above ground should be between 0.15 wave length to 0.3 wave length of radio waves.

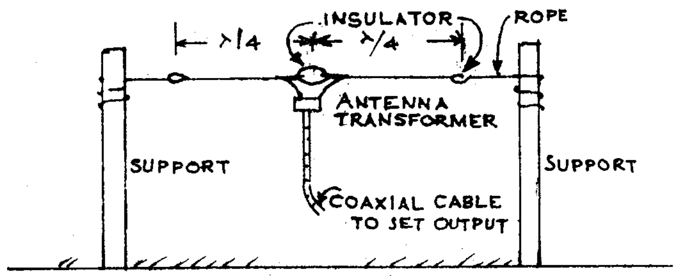


Fig. III. 4.1 Dipole antenna

For practical purpose, the length of antenna is taken 95% of half wave length due to end effects arising out of the slower speed of radio waves in the conductor compared to free space. The characteristic impedance of the dipole is around 73 ohms but the feeder, normally a coaxial cable, as used has the characteristic impedance of 50 ohms. So care must be taken for impedance matching by using aerial transformer in order to minimise the reflection of wave from Load. In some of cases, a single broad band dipole antenna is used for the transmission of different frequencies differing slightly from each other within its band.

The length of antenna can be calculated by using the formula:

Length of antenna in meters = 95% of 1/2 (c/f)

where 'c' is velocity of light in meter/sec.

which is=3×10^8 metre/sec. and 'f' is frequency in cycles/sec.

or alternatively length of antenna in ft. $= \dfrac{468}{\text{frequency in MHz.}}$

Example :

If a wireless set has to work on 5.0 MHz frequency, the total length (L) of Dipole aerial can be calculated as below:

$$L = 95\% \text{ of } 1/2\ (c/f)$$

$$= \frac{95\% \text{ of } 1/2\,(3\times 10^{8})}{5\times 10^{6}}$$

95% of 30 = 28.5 metre

Alternatively $$L = \frac{468}{5} = 93.6 \text{ feet} = 28.5 \text{ meter}$$

Flag Pole Antenna

This is an omni-directional antenna and consists of a straight vertical rod. It responds equally well in all directions. As this type of antenna is always mounted on the top of the pole (must) just like a flag, therefore, it is called Flag Pole antenna. This type of antenna is commonly used with VHF sets.

The length of the flag (antenna element) is always one fourth of wave length of the operating frequency. The VHF frequency allotted to Central Flood Forecasting Organisation is 82.9 MHz. The length of antenna element for this frequency can be worked out as below:

Length of Flag $$= \frac{1/4\,(3\times 10^{8}\,\text{metre/sec})}{\text{frequency in C/s.}}$$

$$= \frac{1/4\,(3\times 10^{8})}{82.9\times 10^{6}} = 0.90 \text{ metre} = 90 \text{ cm.}$$

WHIP Antenna

This is also an omni-directional vertical type antenna and consists of either metal strips or rod. It is always used with antenna matching unit and loading coil to match the impedance of antenna with the wireless set. This antenna is meant for short distance coverage where ease of mobility is required.

With GE-524 sets, 1.25 metre & 3.1 metre long whip antennas are used. 1.25 metre whip antenna is used when maximum mobility is required and range of communication is not the criteria, while 3.1 metre long whip antenna is used when maximum range is required.

Ground Plane Antenna

In some places, ground plane antennas are also used for VHF Communication. This type of antenna is also an omni-directional vertical type antenna which consists of four ground radials besides a whip of 1/4 length as shown in fig. III.4.2. The ground radials facilitate the reinforcement of the field strength of the main lobe by the ground reflection without adding any distortion in the directional pattern. This type of antenna may be used for the frequency range of 30 to 85 MHz.

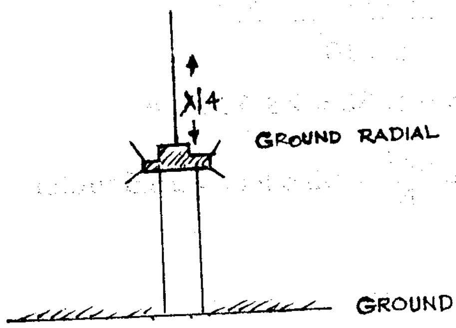


Fig. III. 4.2 Ground plane antenna

Yagi Antenna

It is a compact directional antenna system which provides moderate directive gains i.e. of the order of 6 in a very small space when adjusted for maximum radiation in the indicated direction. It consists of a radiating element, generally a folded dipole, and one or more directors and one reflector arranged along a line. The length of the elements and spacing between them are of vital importance in achieving the directional pattern. The length of director is kept smaller than the radiating element i.e. resonant at higher frequency whereas reflector's length is kept more i.e. resonant at lower frequency in order to concentrate the radiated field in its direction. This is used in VHF communication such as telemetry, satellite communication etc. where a fair amount of directivity is needed. The arrangement is shown in fig. III.4.3.

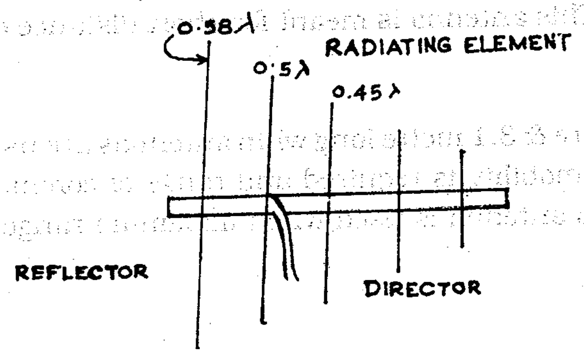


Fig. III. 4.3 Yagi Antenna

The spacing between director and radiating element should be of the order of 0.1 to 0.15 λ whereas the spacing between reflector and radiating element may vary between 0.15 to 0.2 λ.

Power Source to Wireless Sets

The source of power for wireless sets in electrical energy which may be either A.C. supply (single phase, 230 V, 50 MZ) or D.C. supply (24 volt/12 volt), depending upon the design of the set.

A.C. Supply

A.C. supply can be obtained either from mains supply or generators. The mains operated wireless sets are designed to work on a constant 230 volts and fluctuations in the voltage of mains supply adversely affect the working and performance of the set. These voltage fluctuations can be overcome by inserting a 'voltage stabilizer' in between the set and supply socket. At the remote places where the mains power is not available, the use of generator becomes essential. Generator can also be used as standby source of AC supply in case of failure of electricity.

Voltage Stabilizer

It is an automatic electrical device which gives output of the desired constant A.C. voltage despite the varying input AC voltages. Thus, it is a voltage correcting device and consists of a transformer and the error detecting and correcting mechanism besides a front panel voltmeter to monitor the voltages of both ends. However, in some types of stabilizers, provision of manual operation also exists. The error detecting and correcting mechanism may be of either electromagnetic relay switching type or a servomotor type. It detects the drift of input voltage with reference to the set voltage and bring the appropriate tapping of transformer's secondary side in connection to provide the set voltage in steps. A normal stabilizer facilitates the correction of input variation between 180 to 250 volt to 230 volt within 5% of error.

Generator

An electrical generator consists of a prime-mover, alternator and control box shown in fig. III.4.4

Prime Mover can be a petrol or Diesel Engine which produces mechanical energy to drive the alternator. The running cost of petrol engine is high in comparison to Diesel Engine. Therefore, Diesel engines are extensively used as Prime Movers for generator sets. The rotating shaft of Prime Mover (Engine) is mechanically coupled with rotor of the alternator.

Alternator is the main unit of generating set which actually converts mechanical energy into electrical energy and operates on fundamental principles of 'Electromagnetic Induction' i.e. when a conductor moves in a magnetic field cutting the line of force, an e.m.f. is induced, causing flow of current in the conductor. The electrical energy is taken from the alternator to control Box.

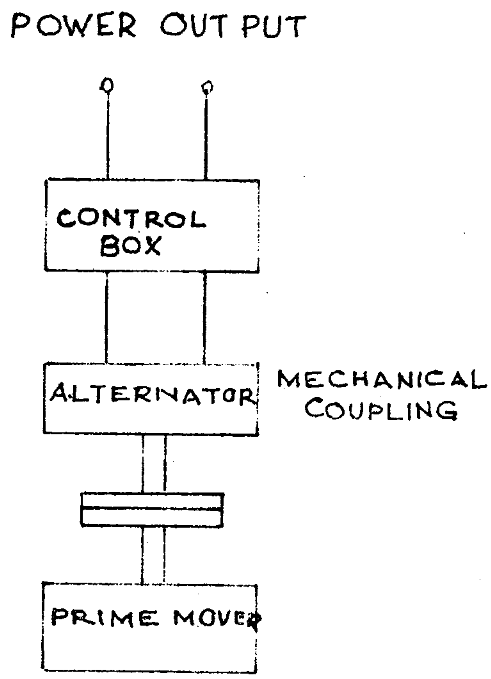


Fig. III. 4.4 Block diagram of an engine driven Alternator

The function of control box is to supply smooth and regulated A.C. voltage through the output terminals and to feed back the D.C. supply to the field of alternator. In Central Flood Forecasting Organisation 2 kw portable, air cooled Diesel Generating sets are used for mains operated wireless sets.

D.C. Supply

D.C. supply can be obtained directly from Battery or through eliminator from A.C. mains.

Storage Battery

It is a rechargeable device which provides D.C. Power by converting the chemical energy into electrical energy and this phenomenon is generally referred as 'discharging' of battery whereas conversion of electrical energy into chemical energy is called 'charging' of battery. For a fixed mobile wireless station, we generally use the lead acid type of storage battery, generally manufactured for an output voltage of 6 or 12 volts as used in automobiles, emergency lights etc. In the manpack role of wireless set, the nickel cadmium type of rechargeable battery finds use. The lead acid type of storage battery provides the advantage of very high capacity to meet the requirements of even a heavy duty use. The capacity of the battery is a very significant parameter of interest as it gives an account of the capability of battery to provide the amount of current at the specified rate of discharge before its voltage falls and is expressed in Ampere-hour. Normally, the capacity of battery depends upon the following factors:

1. Size and Number of plates

2. Quantity of acid in battery.

Thus, a battery of 85 AH capacity is capable of providing one ampere current for 85 hours. Conversely, though the drawal of 85 ampere of current for an hour be correct with mathematical angle but in practice it is not so as it would lead to the complete spoiling of battery plates. As such, the drawal of current should not exceed the value corresponding to some specific hours rate of discharge for which the battery is designed. Generally, 20 hours rate of discharge is specified for battery and so the selection of battery be made considering the average value of current requirement within the permissible value of current of the battery.

Battery Charger

For charging purpose, a storage battery needs a D.C. Supply and the equipment used for this purpose is known as 'Battery charger'. The places where mains is available ordinary types (mains operated) battery chargers can be used while at the remote places where mains is not available, portable engine driven battery charging sets (300/350 watt, 12-18 volts) are used. The charging current should be kept in between 1 to 3 amp. with the help of current selector to accomplish stable charging.

Eliminator

An Eliminator is an electronic device to get the steady DC voltage from AC mains. It consists of mainly three parts:

(i) Step down transformer: It steps down the 230V of mains to the required voltage (12 volt or 24 volt).

(ii) Rectifier circuit: In this unit, the output voltage of transformer is rectified into pulsating DC voltage.

(iii) Filter circuit: Filter circuit converts pulsating DC voltages into steady D.C. voltage.

3.5 Practical Aspects of Wireless Transmission Network

3.5.1 Planning of Wireless Network

The planning of wireless network is done in following three steps before arriving at a conclusion

1. Selection of radio path routes

2. Field survey

3. Radio survey

Though, the HF communication does not necessarily require the selection of radio path routes, the exercise done in this regard may enable the confirmation of its adoption. The following procedures are to be followed for the selection of radio path routes.

(a) Take a map of suitable scale showing all roads, tracks, height contours, villages and towns.

(b) Mark the location and inter-distances of existing and proposed base stations, control rooms and other stations which are to be incorporated into the system.

(c) Locate all hill peaks around each station which generally obstruct the direct VHF communication.

(d) Connect hop between the base station and control station cutting across various height contours. Measure distance from one end for each elevation for plotting line of sight path.

(e) The above line of sight path should be plotted in K=4/3 earth **profile curve sheet** in which the plot covers a small length on the horizontal axis. **Such a short distance** compared to elevation difference between the two points may be **assumed to have** insignificant earth curvature.

If the clear line of sight exists between the stations, the substitution of VHF network may be thought of. Thus, the above procedure provides the idea of network and mast height.

Field Study

The following observations should be made during field survey.

(i) A detailed description of each station in respect of geo-coordinates, M.S.L., height, distance from the data observation points, access roads and location recommended for mast erection.

(ii) Any unusual weather condition expected in the area.

(iii) Terrain condition of adjoining area.

(iv) Nearest location of available electricity supply.

(v) Orientation of high tension electric line, power driven systems, engines in the vicinity.

(vi) The orientation of runway, in the case of nearby airport.

(vii) Details of other communication stations/systems operating on the same band of frequencies.

(viii) Future possibility of construction of structure/buildings along the line of sight path in case of VHF system.

(ix) Availability of land of adequate size for the construction of station's building and masts for suitable accommodation.

Keeping in view the above points, the location may be shifted suitably provided it does not move far away from the data observation points.

Radio survey

The transmitter power is ascertained as per the overall requirements of communication system in respect of maximum hop length path loss and the threshold level of receiver. The gain of antenna depends upon the type of antenna to be used. The path loss is the loss of signal strength in the medium of propagation between the antennae of transmitter and receiver. For space wave propogation the path loss may be computed by following formula. Pathloss in dB = 32.44 + 20 log D + 20 log F where D is path length in km. and F is the frequency of operation in MHz.

The loss attributable to cable and connector losses depending upon its type and length as connected between the equipments and antenna should also be taken into account for computing the received power.

RECEIVED POWER = Transmitted power + gain of transmitting antenna + gain of receiving antenna - (cable loss + connector loss + path loss)

The threshold level of the receiver is the carrier level below which the signal to noise ratio of the receiver will go below 20 dB and this is specified for a particular type of set. The fading margin is the difference between the received power and threshold level. The fading margin of atleast 30dB will ensure uninterrupted communication, even if the signals get attenuated a little more due to unusual disturbances.

In addition to above calculation to arrive on some conclusion, the radio survey should be conducted with the wireless equipments of desired type and output power to check the physical signal strength which can be measured by using field strength meter. The actual communication survey is needed to assess the level of present static and man made noise at the station and the loss of radio signal strength in the terrain around that for which the mathematical calculation does not account for. The actual communication survey can be conducted by deputing two teams with the equipments in the field who would do it by transmitting the test signals only.

After arriving at a conclusion about the type of network and power radiation requirements, steps should be taken to obtain the frequency allocation and licence from the Ministry of Communication, Govt. of India. The procurement of wireless equipments may be initiated immediately after getting the frequencies allocation.

3.5.2 Licencding Procedure

The establishment and operation of a wireless system, require the prior permission of the wireless Advisor to the Govt. of India, Ministry of Communication Operating a wireless set without a valid licence is an offence punishable under the wireless regulations. It is, therefore, necessary that the user should be fully acquainted with the procedures to obtain the licence. The following is a brief summary of the same.

STEP-1

The wireless Advisor of Govt. of India, Wireless Planning and Coordination Wing, Deptt. of Communication, Parliament Street, New Delhi may be requested to send the prescribed application forms to apply for purchase, establishments and operation of wireless sets.

STEP-2

On receipt of application forms the same may be filled up and sent to wireless Advisor, giving details of equipment to be purchased, sources of supply, and the proposed location.

Some of the suppliers are:-

(i) M/s. Bharat Electronics Ltd.
Jalahali,
Bangalore-560003.

(ii) M/s. Punjab Wireless Systems Ltd.
SCO, 54-55-56, Sector 17-A,
Chandigarh-160017.

(iii) M/s. Electronics Corporation of India Ltd.
Industrial Development Area,
Charlapalli,
Hyderabad-500762.

(iv) M/s. West Bengal Electronics Industry
Development Corporation Ltd.,
225-E, Acharya Jagdish Chandra Bose Road,
Calcutta-700020.

Many other firms have come into the field in recent years. The various firms should be contacted even at the stage of preparation of the scheme itself so that while applying to the wireless advisor, the details of the equipment can be given.

STEP-3

On due consideration of the application, the wireless Advisor, will issue a letter of intent to grant licence to establish, maintain and work the wireless stations, allotting a specific frequency. The actual operating licence will be issued by him only after the conditions mentioned in the letter of intent like payment of licence fees, and royalities, site clearance by the Regional Advisory Committee etc. are fulfilled.

STEP-4

After getting the frequency allotment from the wireless Advisor, the equipment manufacturer, should be intimated, so that the manufacturer, can start manufacturing or initiate procurement action for the channel crystals. While some of the firms like M/s. BEL, Bangalore manufacture their own crystals, others have to procure them from others such as M/s. BEL, M/s. KELTRON Crystal Ltd. etc.

STEP-5

If a licence is obtained by the time the sets are ready for despatch by the manufacturers, they may be intimated about the particulars of licence issued. Otherwise, the post master of area may be approached to issue a non dealer's possession licence. With this licence the set can be possessed but can not be operated.

STEP-6

The action for getting a licence should be pursued vigourously and the licence obtained within one year of the issue letter of intent as otherwise the frequency allocation will lapse.

After getting the licence, the sets can be operated as per regulations prescribed by the wireless Advisor.

3.5.3 Installation of Transmission system

For the installation of wireless set, care should be taken in the procurement of crystals to be used in the wireless set to generate channel frequency as per the frequency allocation issued by the Ministry of Communication. The Ministry of Communication generally allocates two parameters of frequency i.e. carrier frequency and assigned frequency, differing from carrier frequency by 1.5 KHz. The choice of crystal frequency is to be made on the basis of carrier frequency, taking into account final intermediate frequency of the particular type of wireless set. For example, 'BEL' make GE 524 type HF transreceiver generates 1.6 MHz as final intermediate frequency. As such, the crystal of frequency increased by 1.6 MHz over allotted carrier frequency be procured for GE 524-type set. In some of other types of wireless sets the final intermediate frequency of 1.75 MHz is used whereas some types of wireless sets use frequency multiplier or divider circuits in conjunction with the channel oscillator. The crystal frequency calculations of a few types of wireless sets under use in Central Water Commission are described below:

1. 100 watt SSB transreceiver/ GE 524 HF set BEL make

   $F \times t = FQ + 1.6$ MHz, $FQ < 12$ MHz

   $F \times t = \dfrac{FQ - 1.6\,\text{MHz}}{2}$, for $FQ \geq 12$ MHz

2. 100 Watt fixed channel W/L MHS 117 and receiver HS 419 BEL make

   $F \times t = FQ + 1.75$ MHz for $FQ < 13$ MHz

   $F \times t = \dfrac{FQ - 1.75\,\text{MHz}}{2}$

   For $FQ \geq 13$ MHz

3. LHP 228 Type BEL make/ECIL make C5210 Type

   $F \times t = FQ + 1.75$ MHz for $FQ < 18$ MHz

   $F \times t = \dfrac{FQ - 1.75\,\text{MHz}}{2}$ for $FQ \geq 18$ MHz

4. GH 650 BEL Make

   Transmitter $F \times t = \dfrac{FQ}{12}$

   Receiver $F \times t = \dfrac{FQ - 11.5\,\text{MHz}}{3}$

5. MPN 8 Watt VHF set

   Transmitter $F \times t = \dfrac{FQ}{18}$

   Receiver $F \times t = \dfrac{FQ - 10.7\,\text{MHz}}{3}$

$F \times t$ – crystal frequency, FQ – Carrier frequency.

The length of dipole antenna should be fixed as per the length derived from the carrier frequency parameter of HF. However, the mode of operation should be understood by the assigned frequency parameter. If the allocation of assigned frequency is greater than carrier frequency, the network is to be operated on USB mode. In the actual installation of wireless set the following points must be kept in view.

1. Orientation and height of antenna.

2. Proper Power supply to set.

3. Impedance of feeder in match with the output impedance of set.

4. Minimization of feeder length without tightening.

5. Proper earthing of the W/L set.

6. Providing of essential accessories for wireless set.

7. Proper accommodation.

The orientation of antenna should be fixed in a compromised direction by successive trials with other stations of the network to ensure the intelligible communication before its actual elevation, to a height of not less than 20 feet for temporary installation. Since antenna requires to be hanged at greater height with firm, supports, tubular type of GI masts are recommended to be erected with minimum spacing between them as 20 feet in excess of maximum antenna length. The mast height may be of the order of 15 to 20 meter. However, for the installation of VHF set only one mast is required.

If the wireless set operates only with mains power supply, the diesel generator set of appropriate capacity be installed either as main source of power or standby source. The inverter of appropriate capacity can also be used as an alternative to generator set. However, for battery operated set, a fully charged storage battery of high capacity be used.

The choice of feeder be made in view of the output impedance of the wireless set. Since most of the wireless sets have 50 ohm output impedance we use 50 ohm coaxial cable as feeder. For matching the feeder impedance with aerial impedance, antenna transformer be used. In order to reduce ohmic loss the length of coaxial cable should be kept minimum as far as practicable but it must not be under tension.

In order to prevent the equipment and the operator from the surges hazard of aerial electrostatic induction, appropriate earthing must be provided to wireless set. This can be accomplished by embedding the wire counter poise in conjunction with one Sq. feet M.S. plate underground three to six feet, amidst the mixture of common salt and charcoal.

The free end of counterpoise may be extended by connecting it with 16 SWG GI wire through hollow pipe for its connection with the ground terminal of wireless set as shown in fig III. 5.1

The essential accessories of wireless set include hand set assembly, head set assembly, telegraphic key, battery/power connector apart from the power source, coaxial cable with BNC connectors and antenna wire.

The wireless set must be housed in safe accommodation free from dirt, dampness and away from any mill or installation using engine, motor, welding equipments, power transformer etc. It should be able to provide natural air cooling to equipments.

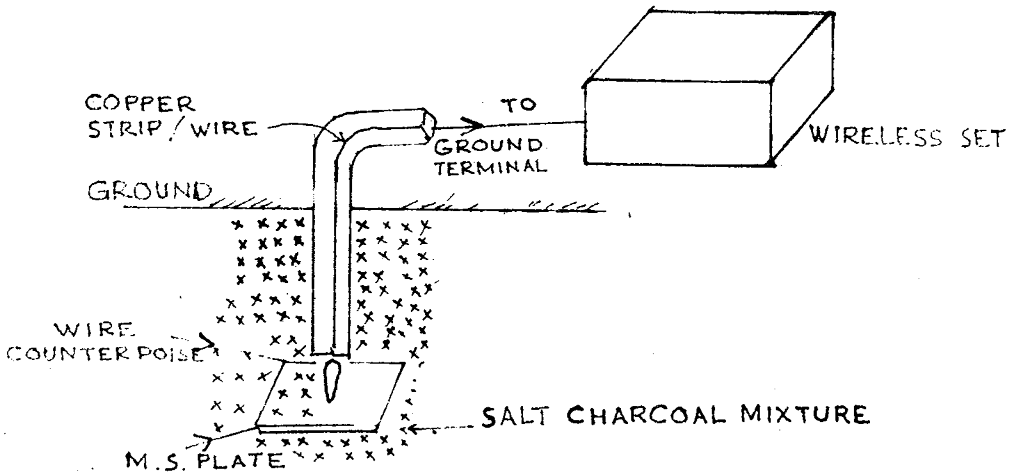


Fig. III 5.1. Earthing mechanism.

3.5.4 Operation of transmission system

In Central Water Commission, the wireless set is generally operated on R/T, A 3J mode. However, in adverse climatic condition telegraphic mode of operation may be used. For the telegraphic mode of operation the headset and telegraphic key are to be used on keeping the function selector switch at 'CW' and for the language of communication 'Morse Code' be used. For voice communication, only handset is used for the receipt and transmission with function selector and mode selector kept at SSB and either of side band selection respectively. The equipment becomes ready for transmission as soon as PTT switch of handset is pressed and its release transforms the equipment to function as receiver. The following alphabet phraselogy be used to spell any word.

A	ALPHA	N	NOVEMBER	1.	UNA
B	BRAVO	0	OSCAR	2.	BISSO
C	CHARLIE	P	PAPPA	3.	TERRE
D	DELTA	Q	QUIBICK	4.	KARTE
E	ECHO	R	ROMEO	5.	PANTA
F	FOXTROT	S	SIARRA	6.	SOXI
G	GOLF	T	TANGO	7.	SETTE
H	HOTEL	U	UMBRELLA	8.	OKTA
I	INDIA	V	VICTOR	9.	NOVE
J	JULLIET	W	WHISKY	10.	NADA
K	KILO	X	X-MAS		
L	LIMA	Y	YANKY		
M	MIKE	Z	ZULLU		

Before the operation of wireless set, the set should be tuned to the carrier frequency in order to bring it on the network. The ways of tuning differs for different types of wireless sets. So technical manual of the respective sets be referred.

When asked about the intellingibility quality of its signal by a station, it should be responded in the manner like 3-4 or 5-5 etc. wherein the first term and second term would represent the degree of the signal strength and clarity respectively. The numbers in expression represent category as noted below:

1. Bad
2. Poor
3. Fair
4. Good
5. Excellent

A complete log of communication at the station should be maintained in log book.

3.5.5 Maintenance of System

As far as practicable, the preventive maintenance should be done at the site of installation which can be accomplished by routine checkup of main equipment as well as allied equipments. The sophisticated electronic equipments are very much susceptible to defect in the presence of dirt and moisture. As such general cleanliness of the exteriors of equipments' body against dirt and moisture must be obeserved. The connectors of the equipments must be tightly fastened with its cable and free from corrosion and rust at both ends. The sophisticated equipments do not need the exertion of pressure for the connection of terminal and switch operation. If it requires so, the user's way of doing connection and connectors' geometry are required to be checked up. The RF output of the set and the battery condition should be watched on the meter of wireless equipment. No heater be used close to the wireless equipment and it should be placed on wooden table to rest on its seat firmly. The antenna wire must be cleaned before and after monsoon. The breakdown maintenance of the equipments must be done only by mechanic or authorities competent to do so. The equipments should be transported by safe means of conveyance after doing proper packing to avoid jerking hazard to equipments. For the maintenance of battery, care should be taken to keep it upright on an insulated block e.g. wooden block. The electrolyte level in the battery should be above the plates in each cell. In case of level running down, only distilled water be added to replenish and a little bench charging be given. The exterior of the battery and terminals must be kept clean and dry. The smearing of terminals with petroleum jelly is recommended. The specific gravity of the electrolyte must be checked by 'Hydrometer' fortnightly. The specific gravity of electrotytex of the fully charged conventional storage battery is $1.210 \pm 5\%$.

The wireless stations are also required to be protected against mishandling, theft and fire. Thus, as far as practicable the wireless set must be operated by only authorised skilled personnel i.e. wireless operator or any personnel authorised by the competent authority to operate. Fire fighting equipments should be provided to each station and the personnel at site must be trained for its operation. Each wireless set must have its own history sheet in which the records of its repair and maintenance be kept.

The equipments to be provided to each wireless stations are listed below:

1. Hydrometer Cell tester
2. Battery charger
3. Mains Power supply unit
4. Fire extinguisher and buckets
5. Blower
6. Voltage stabiliser
7. Petty tools e.g. screw driver, Pliers, line tester etc.
8. Soldering Iron
9. Time piece

The repairing centre/wireless workshop should have following testing and measuring equipments besides the tools kit of mechanic, blower, bench grinder, drill machine etc.

1. AVO meter/Digital voltmeter
2. A.C. Milli Voltmeter
3. Frequency counter
4. Oscilloscope
5. Regulator DC Power supply, 0-30 volt, 0-10A
6. Transistor tester
7. AF signal generator

8. RF signal generator

9. RF output power meter

10. V.T.V.M.

3.6. Guidelines for wireless Operator and other officials engaged in Data Transmission

The wireless operator must ensure himself about the following before establishing the communication:

1. Proper connection of wireless set to battery/eliminator in respect of polarity.

2. Connection of Coaxial cable at both ends i.e. equipments and antenna, and earthing.

3. Proper connection of handset with the audio socket of wireless set.

4. Position of band selector, channel selector, mode selector and tunning.

5. Battery/power supply condition.

6. General correctness of data and message.

The wireless operator is solely responsible for the accuracy of communication. So he must be very much alert and repetitive unless correctness of communication is ensured. He must adhere to the timing of schedules strictly and obey the commands of control wireless station. The operator must neither pass any unauthorised nor edit/distort the message. As far as practicable he should be brief on wireless. He must be aware of the procedures of routine preventive maintenance check up of equipments. Any breakdown of equipments must be promptly reported by him to concerned authorities.

The officials engaged with the supervision and maintenance of data communication e.g. A.E. (W), J.E. (W) etc. must have the clear conception of the network and procedures. He should always keep the vigil over the general communication on the network in respect of signal strength and clarity quality, nature of communication etc. He should visit every station atleast once in two months to ensure the general correctness of equipments and connections for effective maintenance, both preventive and breakdown. The arrangement of standby wireless sets at the norms of 25% of the stations be made for meeting the urgent replacement requirements. As such, its positioning/distribution should be worked out carefully considering the local conditions. The officer should also arrange for the preventive repairs/maintenance of each wireless equipments before monsoon and also arrange for the emergency repairs of wireless sets during monsoon. As far as practicable, he should arrange to provide the genuine original spares and not the equivalent for the use in repair/replacement. The equipments should continue to be repaired and maintained till either its repair become beyond economical repairs or it loses its useful life. Though, the life of equipments vary with the local conditions and operation/maintenance, the following norms may serve as broad guidelines on equipment's useful life time:

Name of Equipment	*Life Expectancy*
1. Wireless set	15 Years
2. Testing & Measuring instrument	
(a) High grade frequency counter, oscilloscope etc.	20 Years
(b) Low grade i.e. multimeter etc.	15 Years
3. Petrol/Diesel engines/generator/Charger	15 Years
4. Secondary battery Lead acid type	8 Years
5. Aerial Mast (GI Tubular or Lattice type)	40 Years

3.7 Telemetry

'Telemetry' is the measurement of any physical quantity from a distant place by transmitting commands and collecting the measured values. Under Central Water Commission Flood Forecast network, a telemetry system has been installed in the Upper reaches of Yamuna basin down to Delhi, with the following primary objectives:

(i) To ensure the performance and accuracy of equipments for automatic measurement of the hydrological and hydrometeorological quantities and its transmission to serve as input in the flood forecast for Delhi.

(ii) To provide computer compatibility to the data communication system enabling the direct utilisation of data input received for the computer based mathematical model programmes for the flood forecasts.

The data collection network under this project comprises a master station and twenty one remote stations, out of which seven stations are mere repeater stations as shown in fig. III.7.1. Hydrological and Hydrometeorological data comprising of water level, aerial precipitation (rain/snow) and ambient temperature are collected from fourteen sensor stations and transmitted to the Master station at CWC Headquarters, Sewa Bhavan, R.K. Puram, New Delhi through a VHF network in real time. In addition to this the battery voltage of all stations are transmitted to monitor the battery condition. The complete unit is a software based system, in which the Master station is programmed to interrogate all stations in time mode of fixed interval. The data reaching Master Station are displayed on the CRT and printed out on a Matrix printer. The data

TELEMETRY NETWORK ON RIVER YAMUNA

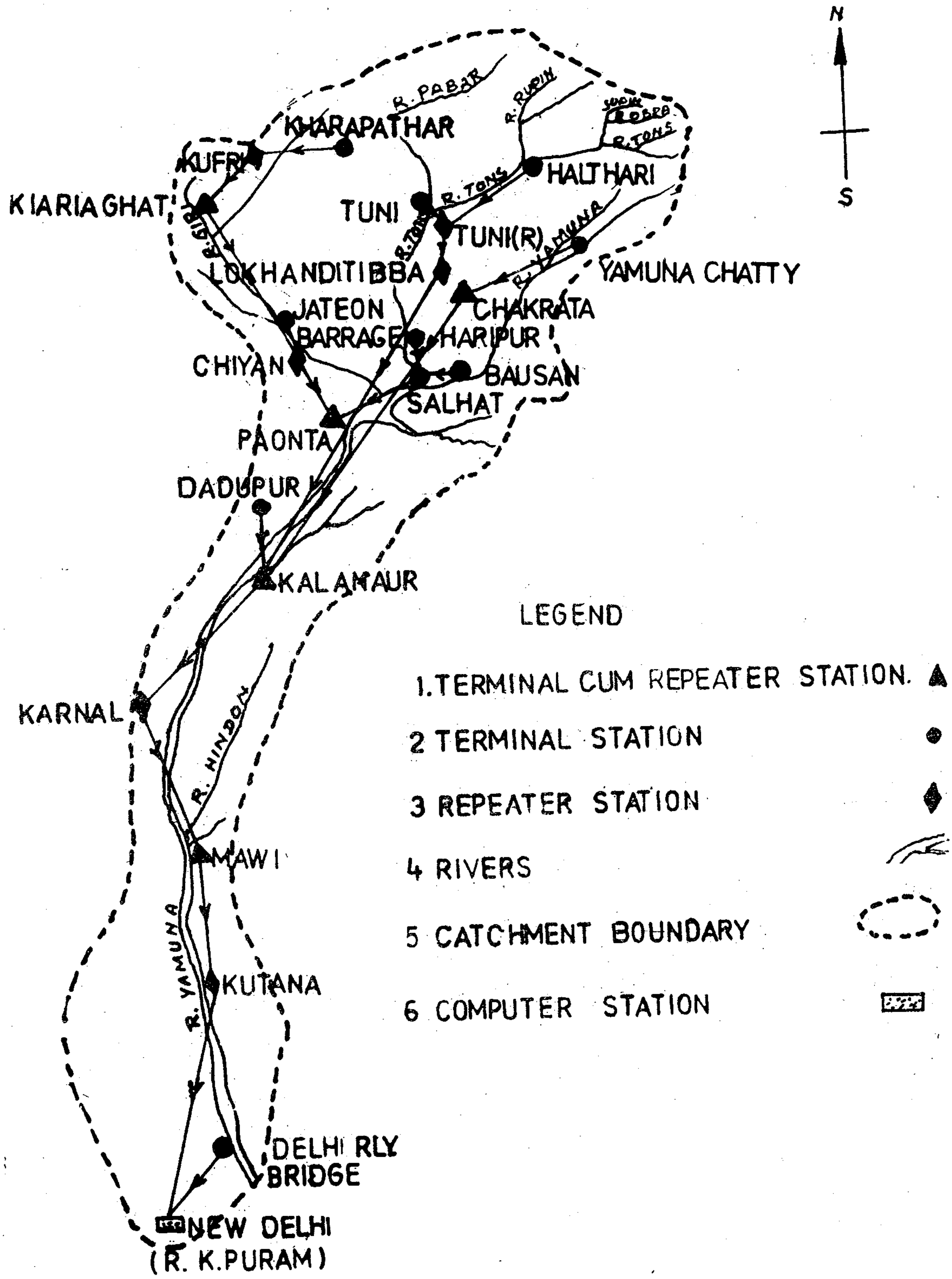


Fig. III.7.1

simultaneously get stored in a floppy and are also 'on line transmitted to (HP-1000 series) mini computer, for utilisation during execution of mathematical model programmes of flood forecasting.

Every station has its own identification number code and radio path code. Thus, any interrogation originating from the Master station for any particular station is received by the immediate station, processed and if it is not meant for that station, it is further transmitted to the next station of the path on the basis of radio path code and station code. The relay process repeats until the interrogations reach the desired destination. The transmitted reply of station on interrogation also reach the master station with the same process. Thus, it is possible to interrogate selectively a particular station or all stations by giving suitable command at Master.

The following equipments are used in Master station to enable it to interrogate the slave stations.

1. Master Teleprocessor

2. Modem

3. VHF transreceiver

4. Peripheral equipments e.g. CRT terminals with key board, printer and floppy disc drive

The Master station is also provided with an uninterrupted power supply system.

The slave teleprocessor and VHF transreceiver set besides the sensors and recorders constitute the main equipments of a slave sensor station. The repeater station possesses only slave teleprocessor and VHF transreceivers. There are two 8 watt, VHF transreceivers at each station except the terminal station which requires only one, to facilitate the transmission of signal in upside and down side directions, at two different frequencies e.g. 159.7 MHz and 161.7 MHz through independent seven elements Yagi antenna systems. The antenna facing towards Master is called upside. The VHF transreceiver sets are connected to slave teleprocessor through 5 pin audio cable. The output of sensors are also connected to slave teleprocessor through a multipin connector. All equipments at slave station derive power from 12 volt battery, rechargeable either by mains operated battery charger or by solar powered chargers.

Master Teleprocessor

It is the brain of the system. The master teleprocessor, 'GCEL' make, has four PCB namely (a) single Board computer (b) RAM, memory board (c) Four channel expansion board (d) Floppy disc interface board.

The single board computer has 4 K Erasable Programmable Read Only Memory (EPROM) wherein the system loader programme resides. It processes the digital form of data to check validity before its storage in RAM memory. It also monitors the switching of VHF Sets. The four channel communication expansion board provides interfaces to CRT, enables to operate floppy drive. The RAM memory PCB has hardware to store 232K bytes of information/data.

Modem

The function of the modem is to convert the digital signal into FSK tone for transmission as well as to convert the received audio tones into the digital signal.

Slave Teleprocessor

It is the heart of the system which collects the data, facilitates the transmission of the same through VHF transreceiver, on request from the Master station. The software for the communication, data collection and local test programme under EPROM facilitates the operation of processor. The signalling conditioner part in the equipment convert the digital outputs of connected sensors into a 12 bit digital signal. The multiplexer unit multiplex the digital data channels, each of 12 bits on to an eight bit data bus connected to processor. It also provides the control outputs. Modem unit is also provided in the teleprocessor to monitor the functioning of communication system.

The above system of telemetry works on relay transmission in step and so failure of any intermediate station causes the link failure of all stations located on the down side of that station on the radio path, thus, posing a big question mark on the reliability of the system. The crux of problem lies in the communication system which uses the troposheric (Spacewave) propagation necessitating the repeaters. The problem can be overcome by the use of statelite communication system to a very large extent.

The Central Water Commission has taken further step to explore the field operation suitability and the degree of reliability and accuracy of the data communication system for telemetry through satellite communication for the improvement of flood forecasting technique by launching a pilot project for the establishment of same in parallel to existing telemetry system in Yamuna basin upto Delhi.

The existing sensors of the telemetry system at fourteen sites will be hooked up by end of 1988 with the proposed satellite communication system by the equipment named Data Collection, Storage and Transmission Sub-system (DCSTS), as shown in Fig. III.7.2. The sensors of Snow Hydrology Receiveer Station, Jubbal would also be hooked up to the network as fifteenth station. The data collected on hourly real time basis and stored in DCSTS will be transmitted with station identification code on hourly mode at predetermined intervals to the geosynchronous satellite (INSAT I B) at 402.75 MHz randomly in burst mode. On the receipt of the singal by the UHF annular slot array antenna of satellite, it would be fed to the DCP transponder where it would be amplified, filtered and upconverted to 4039.1 MHz for its transmission back to earth station, Secundarabad (UP), situated about 60 km. away from New Delhi. At the earth station,

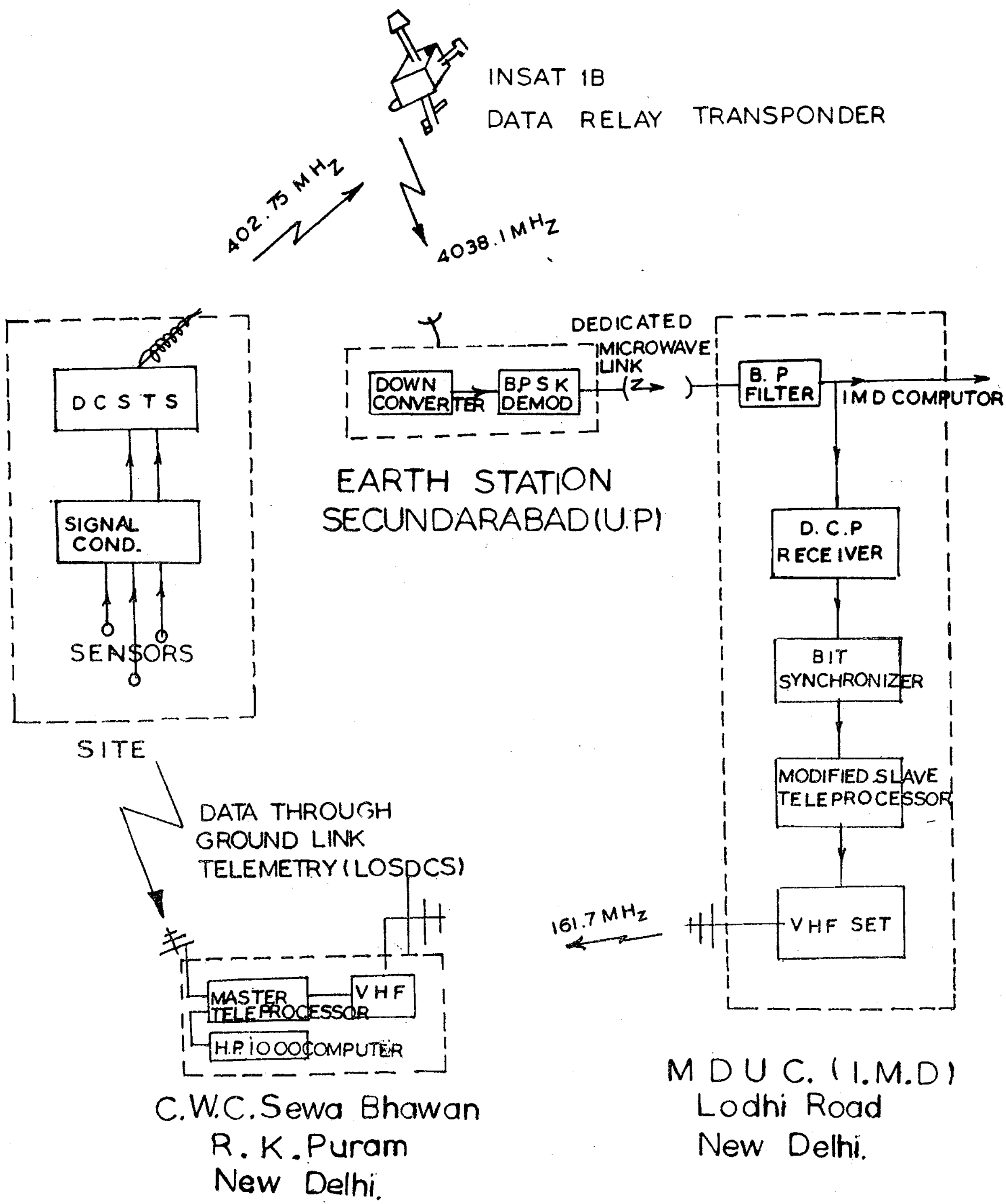


Fig. III.7.2. Configuration of data Communication Through Satellite-INSAT-1B

the signal would be converted to 70 MHz by Down converter and would be fed to Time Division Multiple Access Demodulator i.e. DCP Receiver, BPSK Demodulator. The decoded data would be transmitted from earth station to Meteorological Data Utilisation Centre (MUDC) at Lodhi Road, New Delhi over the same dedicated microwave link as is being utilised by IMD for the collection of similar meteorological data. The demodulated output from the microwave link will directly be taken through clock recovery/data generator unit to a modified slave teleprocessor for storage and subsequent identification of CWC's data. This unit will also bring down the baud rate from 4800 to 300/1200 baud rate for its transmission to Master station at CWC Headquarters, Sewa Bhavan, R.K. Puram, New Delhi, over a VHF link operating on 161.7 MHz without interfering with IMD's computer. This modified slave teleprocessor would have its own identification code to become a part of the existing VHF network of telemetry. The Master teleprocessor would also interrogate it in time mode after interrogating the existing fourteen stations, to collect the data received through satellite. The data so received would be transferred to computer.

CHAPTER-IV

FLOOD FORECASTING BY CONVENTIONAL METHODS

4.1 Introduction

The hydrometeorological, hydrological and other related data which are observed and transmitted to the forecasting centres, constitute the basic input. On the other hand, a forecasting agency is required to estimate and disseminate the variables such as gauges, discharges and/or volume of the inflows etc. at specified locations with sufficient warning time. These desired outputs can be estimated with the help of the available inputs (inputs available at the time of formulation of forecasts) by using suitable computing models, commonly known as forecasting models.

There are a number of forecasting models where the computation technique, the data input requirements and the output vary considerably. The choice of the forecasting model greatly affects the warning time and the accuracy of the forecast. As a matter of fact the available inputs and the desired outputs play a vital role in the choice of the most suitable model for forecasting. Some other factors which also influence the choice of the model for a forecasting site include the degree of accuracy needed; the computational facilities; and the availability of trained personnels to man the forecasting centres. Yet another important factor which needs to be given due consideration in the choice of the model is the river flow and catchment characteristics.

Be that as it may, the forecaster's knowledge and personal comprehension of the behavior of the river, enabling his own judgement, cannot be under-emphasised.

4.1.1 Requirements of a Flood Forecasting Model

As already discussed earlier, the flood forecasting operations are centred around the time factor and the degree of accuracy of the forecast. As a matter of fact, a professional assigned with the responsibility of formulation of forecast has to race against time. Obviously the models which are to be used by forecasting organisations have to be reliable, simple and capable of providing sufficient warning time with, however, the desired degree of accuracy. There are a number of comprehensive models involving very detailed functions which may provide an increased warning time and where the degree of accuracy may be only slightly more, but such models generally have very elaborate requirements for the input data. All the desired input data for a specific model are, in many cases, not available on real time basis and therefore the likely constraints in respect of the availability of data on real time basis are to be given due consideration in the choice of the model.

From the practical point of view, a model for flood forecasting should meet the following requirements.

(i) The model should be capable of providing a reliable forecast with sufficient warning time for organising the flood fighting operations;

(ii) The forecasts formulated by the model must have reasonable degree of accuracy;

(iii) The input data requirement for the model (both for calibration and for operational use) should match the data availability. From practical consideration, it is desirable that the model has the minimum data requirements;

(iv) The model should have functions which are easy to understand; and

(v) The computational procedures involved in the model are simple enough to be operated by the field staff.

As a matter of fact, the choice should never be restricted to one specific model. It is always desirable to select and calibrate as many models as possible with a detailed note on suitability or otherwise of each of the models under different conditions. These models should be applied depending upon the conditions under which they are to be operated.

The comprehensive models which are rather complicated, generally need computational facilities such as computers of suitable size. But at many places, such facilities are not available. In some cases suitably trained staff are not available and many a times these machines can not be operated because of recurring problems such as electricity failures etc. Therefore, it becomes necessary to develop a very practical model which is capable of giving results with desired accuracy and which can be used by simple and readily available computation facilities. Alternatively, both the computer based comprehensive as well as the simple type of models can be developed and a computer based technique can be used in general and in case of emergency the conventional technique which are generally of simple type, may be adopted. Apart from the selection of different models, it is desirable to have a calibration of the models under different conditions. For example a model may be calibrated with suitably large data network but at the same time the model must be calibrated for a smaller network as well giving due consideration to the possible failures in observation and real time transmission of some of the data. This will be helpful in utilising the model even in emergent situations when the data are not available from all the stations. This will require different sets of parameters which are to be adopted under different conditions.

Some of the factors which are required to be considered in selecting a suitable model for a forecasting site are summerised below.

(i) Objective of the forecast/requirements of the forecast.

(ii) The degree of accuracy needed.

(iii) Data availability.

(iv) Availability of the operational facilities.

(v) Availability of trained personnels for development of model and its operational use.

(vi) Upgradability of the model.

4.1.2 Methods of Flood Forecasting

On the basis of the analytical approach for development of the forecasting model, the methods of flood forecasting can be classified as:

1. Methods based on statistical approach, and
2. Methods based on mechanism of formation and propagation of flood.

In the first category, the forecasting methods which are generally graphical or in the form of mathematical relationship, are developed with the help of historical data, using the statistical analysis. These include simple gauge to gauge relationships, gauge to gauge relationships with some additional parameters and rainfall-peak stage relationship. These relationships can be easily developed and are most commonly used in India as well as other countries of the world.

But the most systematic approach for development of forecasting model will be one which is based on the concept regarding the mechanism of formulation and propagation of flood i.e. transformation of rainfall falling over the basin into runoff, its time distribution and further propagation down below.

Based on the data used for formulation of forecast, the various methods of flood forecasting can be classified in three major groups:

1. Forecast on the basis of stage-discharge data;
2. Rainfall-runoff methods; and
3. Meteorological methods.

The detailed classification is illustrated in fig. IV.1.1 and Fig. IV.1.2.

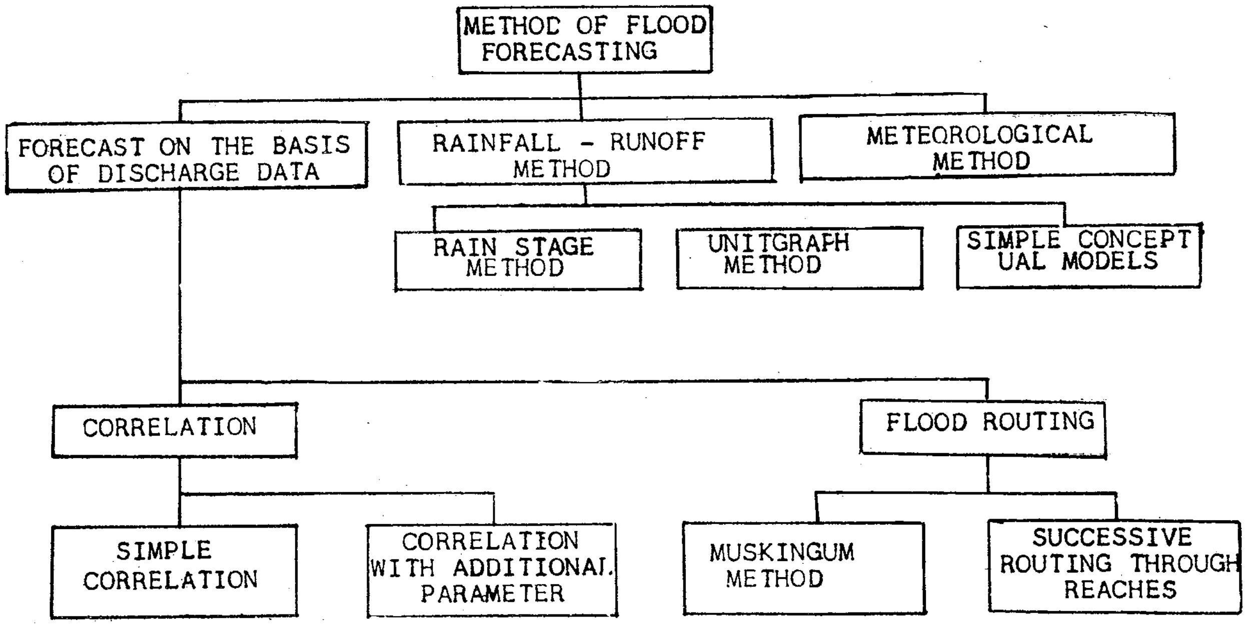


FIG.IV.1.1. METHOD OF FLOOD FORECASTING

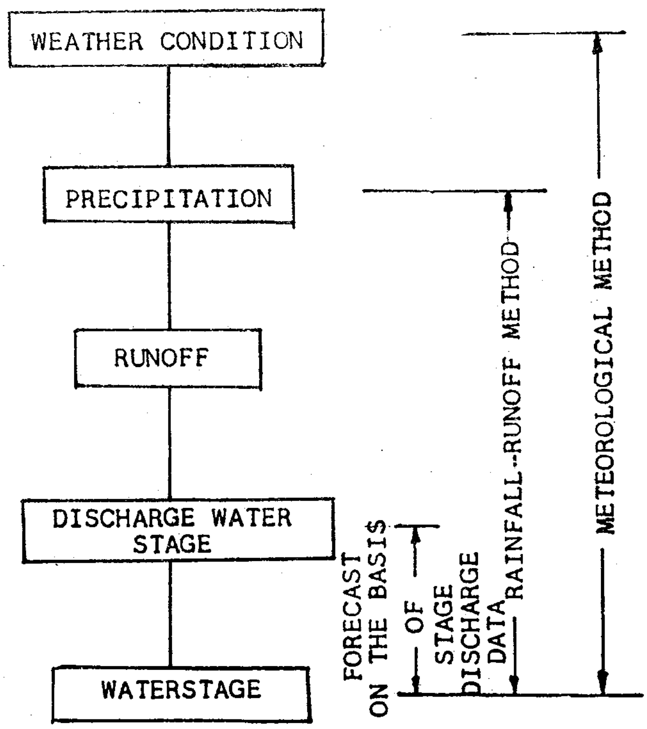


FIG. IV.1.2. METEOROLOGICAL METHOD

For formulation of forecast on the basis of stage-discharge data, the basic data used are gauge and discharge data at various points along the river course. This method can be further classified as:

1. Simple correlation;

2. Correlation with additional parameters;

3. Channel routing by standard methods (e.g. Muskingum);

4. Successive routing through sub-reaches;

These have been discussed in detail in the subsequent sections.

In the sub-basin affected by flash floods, the only effective method of flood forecast will be rainfall-runoff method for which the basic data required is precipitation. This may also be helpful in increasing the warning time of forecasting for the lower reaches of the river as the forecast values of river stage in the upstream, could be used for forecasting down stream stages. Then, it is ideally suited for inflow forecasting into reservoirs and lakes.

For a small catchment where the concentration time is very less, even the use of rainfall data in forecast formulation will not help in getting sufficient warning time. In such cases, hydrometeorological methods are used. When the location, speed and pressure etc. of the tropical depression etc. are informed, the future rainfall can be very roughly estimated with the help of some empirical relationship. This will help in formulating a very rough flood forecast and an advanced warning can be issued. This method is generally applied for flash flood guidance and is needed for a smaller river basin having catchment area of less than 1200 Km^2 and concentration time of 6 hours or less. This type of warning, even with a lack of accuracy, is very important for densely populated or industrialised area lying in smaller basin with rapid concentration time.

Yet another basis for classification may be the requirement of the forecast. Accordingly, the models used for forecast formulation can be basically classified as:

1. Event models, and

2. Continuous models.

If the model is capable of estimating only a particular event, for example the peak flood resulting from a storm, then such models are termed as Event models. A continuous model is capable of predicting the flood hydrograph at specified time interval.

4.1.2.1 A Discussion on Application on Various Methods of Forecasting

The various factors which govern the adoption of a particular method of forecasting are as follows:

1. Physiographic factors;
2. Data availability;
3. Warning time required;
4. Computational facilities at forecasting centre;
5. Purpose of the forecast.

Physiographic Factors

By physiographic factors, we mean basin and channel characteristics of a catchment. These characteristics will help in identifying the method which will be most suitable for the particular point. For example consider the map showing Subernarekha catchment (Fig. IV.1.3). If it is desired to develop a suitable model for forecasting at Chandil dam site (Point P 1), then one has to adopt rainfall-runoff relation only, because the contribution of almost all the tributaries above that point is considerable and being in hilly region with high slopes, the travel time will be comparatively less. On the other hand if the forecast is desired at Rajghat down below (Point P 6), the gauge to gauge relations between point P5 and point P6 is supposed to give a fairly good result. However the use of gauge to gauge relation will allow very less (about 6 hours) warning time. Consider the case of river Burhabalang. This is a very small river. At present forecast is issued for NH5 site with the help of gauge data at Baripada. Warning time available is hardly 8 to 9 hours. It may be observed from the catchment map (Fig. IV.1.4) that two more tributaries join the river just above the forecasting site NH5 and the catchment area of these two tributaries is about 50% of the total catchment.

The catchment above Baripada, the base station, is only about 1/3rd of the total catchment. It may be very easily concluded that the adoption of gauge to gauge relation for such basin is not justified. The only suitable method will be rainfall-runoff method.

On the other hand for lower reaches of large river like Ganga and Brahmaputra, a gauge to gauge correlation, with or without few additional parameters is supposed to give very good results and sufficient warning time.

In general it may be concluded that gauge to gauge relations including simple correlations, multiple correlation and coaxial diagrams etc. are useful in long, slow-flowing rivers. Rainfall-runoff model is very useful for flood forecasting in head water reaches where use of

CATCHMENT OF
SUBARNAREKHA RIVER
SCALE-1: 10,00,000

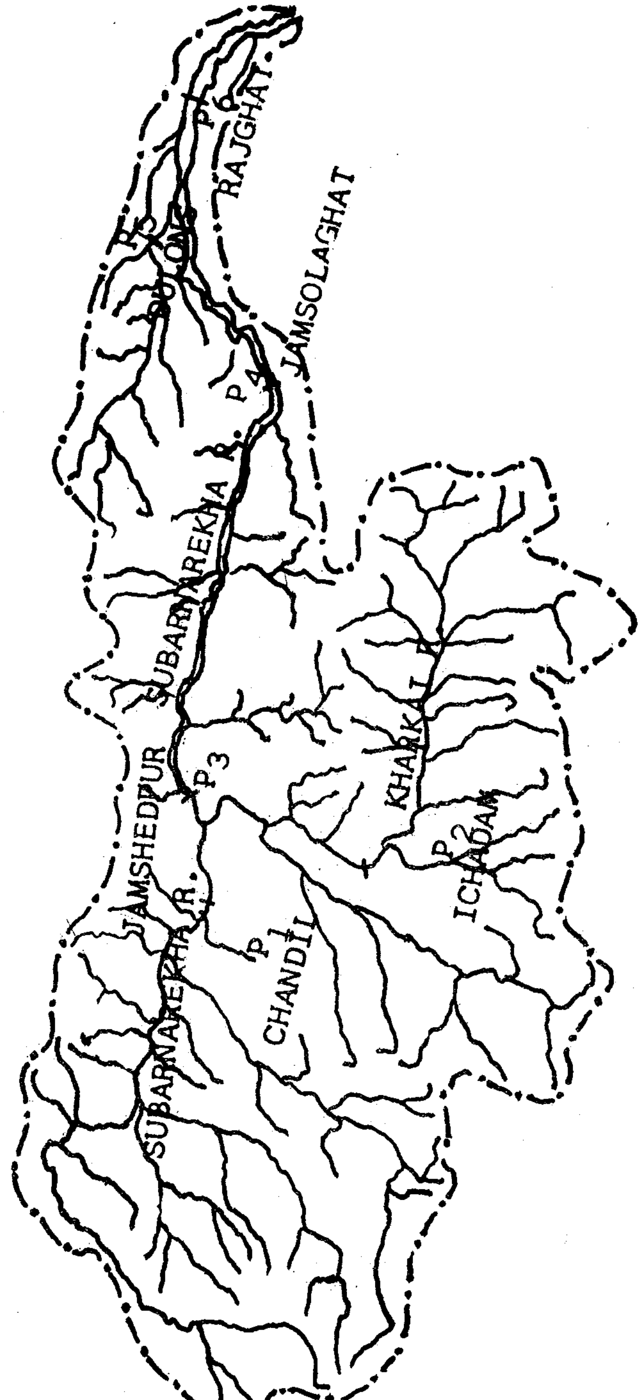


FIG. IV.1.3

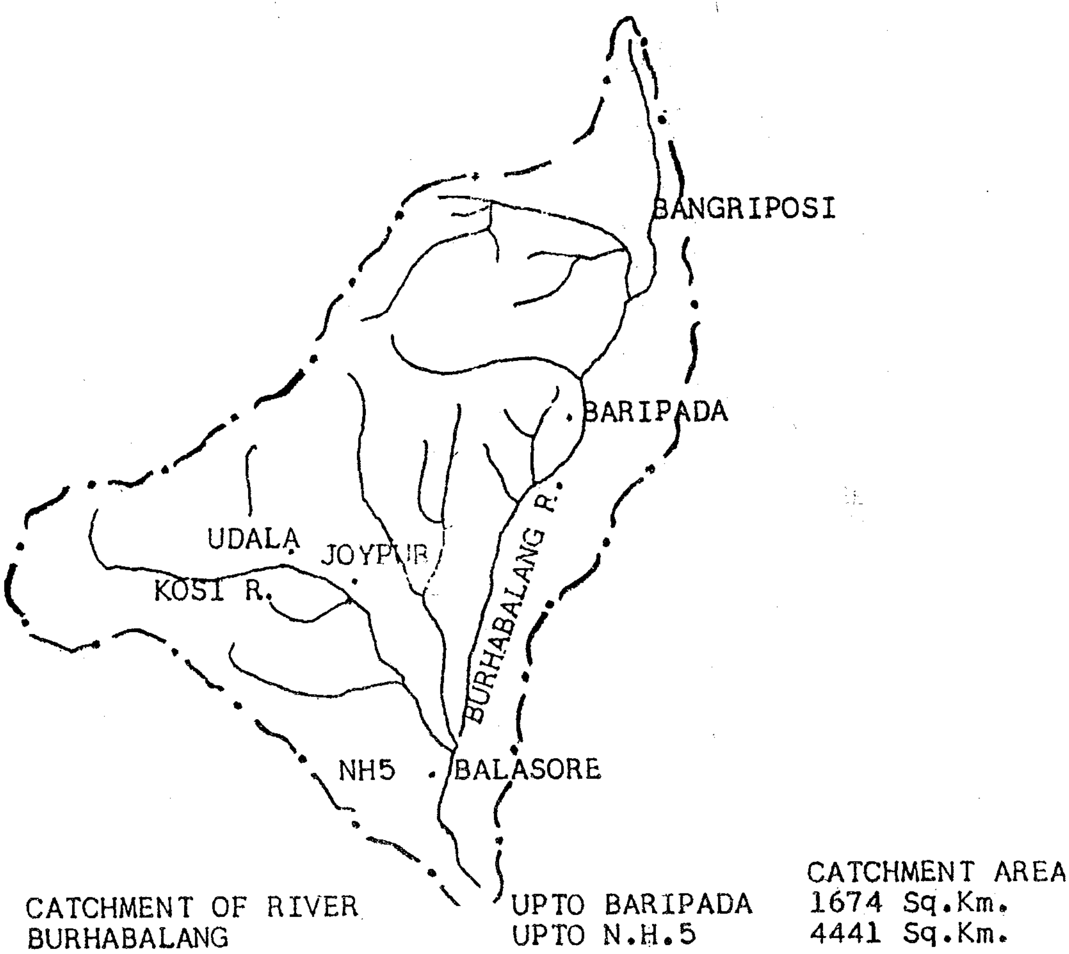


FIG. IV.1.4.

gauge to gauge relation is very difficult, if not impracticable. This is also very effective tool in formulations of Flash Flood Forecasts for the flood prone tributaries as also for increasing the warning time in medium length rivers if appropriately used in conjunction with stream flow routing or gauge to gauge relations for lower reaches.

Data Availability

For development of a flood forecasting model as well as for its operational use, certain specific hydrological and hydrometeorological data are required. For example, for the establishment of gauge to gauge relationship between two stations, only gauge data at different time for the two stations are required. For development of a unit hydrograph at a point the data required will consist of:

(i) Gauge data at specified duration.

(ii) Sufficient number of discharge observations for development of stage discharge curve.

(iii) Rainfall data from sufficiently good number of raingauge stations at specified duration.

On the other hand when a catchment model is to be developed, a large number of hydrological and hydrometeorological parameters are to be defined which need many data. For example the data required for development of SSARR model are:

(a) Several years of precipitation, temperature and discharge data.

(b) Basin area, elevation, location and distribution of hydromet eorological stations.

(c) Information regarding soil water infiltration curve, impervious areas, percent slopes, type and extent of vegetation cover etc.

For the operational use of SSARR model the following data are required:

(a) Precipitation from several stations at given time interval.

(b) Snow melt : either a temperature index or from energy budget approach.

(c) Stream flow input and output from one or more areas.

(d) Daily pan evaporation data.

(e) Rating table for lakes and reservoirs.

The above discussion indicates how the availability of data affects the adoption of a particular method of flood forecasting.

Warning Time Required

As has already been discussed earlier, the gauge to gauge relationship is a good method for forecasting flood at Rajghat site on river Subernarekha (Fig. IV.1.3.) but the warning time available is only about 6 hours which is not at all sufficient for taking precautionary measures. Under such circumstances it is always desirable that some other technique like routing/rainfall-runoff model for upper catchments should be adopted to increase the warning time. For a very small and flashy river, even the rainfall-runoff method does not provide sufficient warning time. In such cases meteorological methods can be used for flash flood guidance even with a lack of accuracy.

Computational Facilities

As far as gauge to gauge relation is concerned no major calculation is involved and forecast can be formulated manually without any loss of time. In case of stream flow routing through sub-reaches a good deal of computation is involved and a micro-computer with smaller capacity will be required for attaining a quick and correct result. Similarly the use of unit hydrograph will require a programmable micro-computer. In case a catchment model is developed, a computer of bigger capacity will be necessary for its operational use. For example, the Sacramento model will need a computer with atleast 16 K core capacity for operational use.

Purpose of the Forecast

For flood purposes, the main requirement is water stage. But for reservoir regulation purposes, the total volume of the incoming flood as well as its time distribution is required and hence for inflow forecasting we have to adopt such a method which will produce the above two informations.

4.2 Methods Based on Statistical Approach

Method based on statistical approach makes use of the statistical techniques to analyse the historical data with an objective to develop methods for the formulation of flood forecasts. The methods thus developed can be presented either in the form of graphical relations or mathematical equations. A large number of data, covering a wide range of conditions, are analysed to derive the relationships which inter-alia include gauge to gauge relationship with or without additional parameter and rainfall-peak stage relationship.

These methods are most commonly used in India as well as other countries of the world. These are discussed in detail in the subsequent sections.

In this method, the variables which affect the stage or the discharge at the forecasting site are identified and the graphical or the mathematical relations are established between the stage or discharge at the forecasting site and the identified variable (s). Some of the variables which affect the stage and/or discharge at the forecasting point are:

(a) Stage and discharge of the base station;

(b) Stage and discharge of previous periods of the forecasting station;

(c) Change in stage and discharge of the base station;

(d) Travel time at various stages;

(e) The rainfall (amount, intensity and duration) in the intercepting catchment;

(f) Topography, nature of vegetation, type of soil, land use, density of population, depth of GW Table, soil moisture efficiency etc. of the intercepted catchment;

(g) The atmospheric and climatic conditions; and

(h) Stage and discharge of any important tributary joining the main stream between the base station and the forecasting station.

Factor (a) to (d) are basic parameters used in developing the correlation curves. Factor (e) and (f) are taken into account by introducing the rainfall and API. Factor (g) is a minor one and can be considered by introducing an additional parameter as week number of year. However, it is not very important for Indian rivers as most of the floods occur during monsoon period only.

The factor (h) is very important and can be neglected only if the contribution of the tributary is very small.

On of the most simple and very useful graphical relation is the "FLOOD PROFILE NOMOGRAM". This diagram indicates the peak stage at each station along the river for a storm. A number of such lines are drawn for various conditions of storms. The various lines should be drawn in different inks and the specific meteorological condition such as heavy concentrated rainfall or other conditions such as breach of embankment etc. should be mentioned. One such Nomogram for river Yamuna is shown in Fig. IV.2.1.Although this diagram does not help in accurate forecast formulation, it serves as a very good guide in checking the formulated forecast, particularly in case of peak level.

The various type of graphs which are used in forecast formulation can be classified as:

(i) Direct correlation between gauges or discharges of U/S and D/S stations.

(ii) Correlation between gauges or discharges at U/S and D/S stations with additional parameters.

Some of the correlation diagrams which are commonly in use are discussed hereunder.

4.2.1 Direct Correlation between Gauge and Discharges at U/S and D/S:

In such a graph basically, only gauge and discharge data of forecasting stations and the base stations are utilised in different forms. The following type of correlations are generally used:

(A) The simplest of all is the correlation between the N^{th} hour stage of base station and $(N+T)^{th}$ hour stage of forecasting stations; where T is the travel time of flood wave between the base station and the forecasting station. Fig. IV.2.2. shows one such graph which is used for forecasting the river stage in river Brahmini in Orissa.

This type of graph can be developed and used for a reach of the river where there is no major tributary with considerable discharge, catchment between the two station is small so that the effect of rain is negligible and the travel time from base station to the forecasting stations is fairly constant for various stages.

However, in most of the cases the travel time is not constant and varies with water level. Apart from this, such relations give considerable errors under different conditions. These relations can be considerably improved if the following aspects are taken into account:

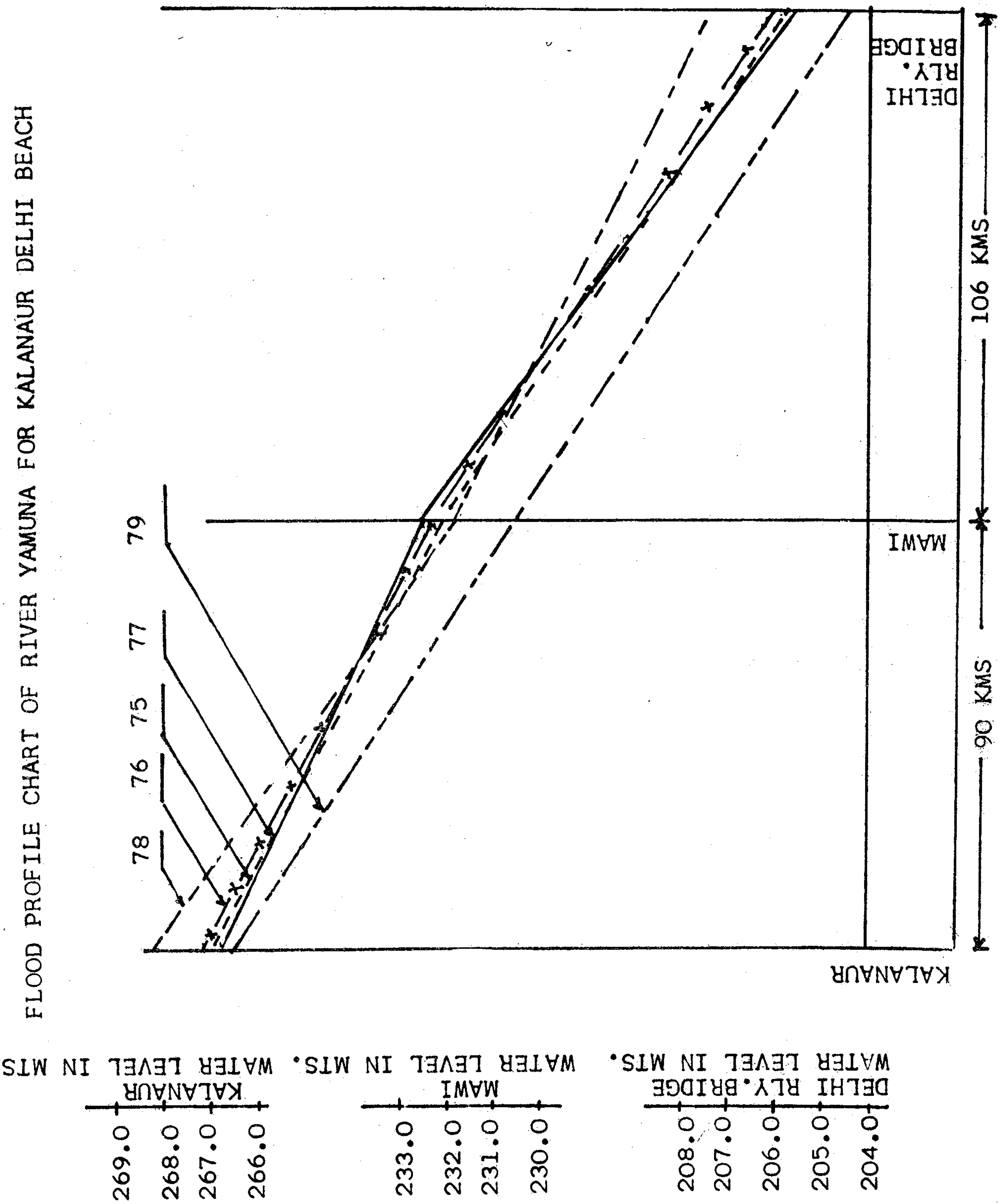
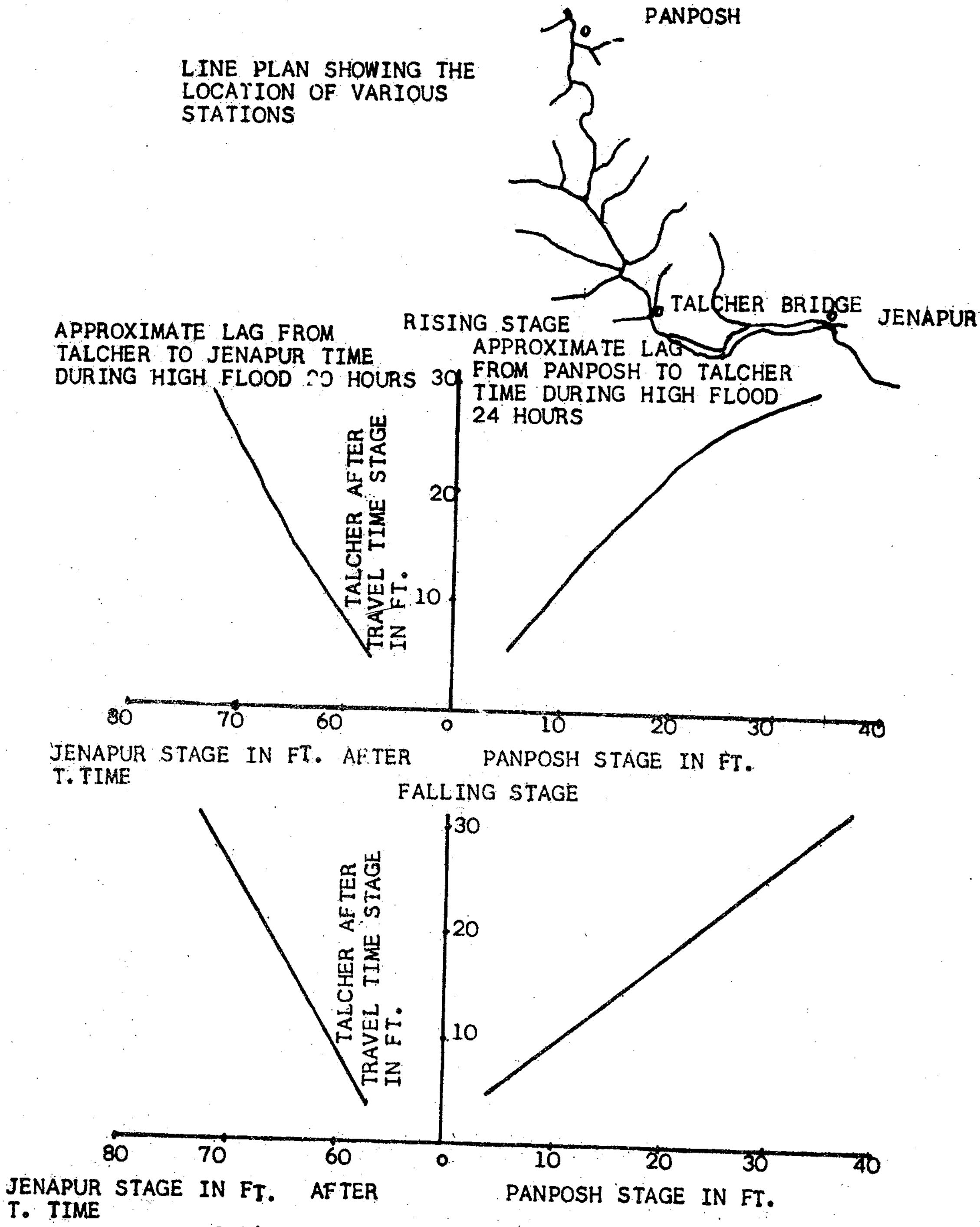

FLOOD PROFILE CHART OF RIVER YAMUNA FOR KALANAUR DELHI BEACH
79
77
75
76
78
DELHI RLY. BRIDGE
MAWI
KALANAUR
106 KMS
90 KMS
KALANAUR WATER LEVEL IN MTS.
269.0
268.0
267.0
266.0
MAWI WATER LEVEL IN MTS.
233.0
232.0
231.0
230.0
DELHI RLY.BRIDGE WATER LEVEL IN MTS.
208.0
207.0
206.0
205.0
204.0

Fig. IV.2.1



CORRELATION DIAGRAM OF RIVER BRAHMANI BETWEEN GAUGE AT PANPOSH, TALCHER AND JENAPUR.

FIG. IV.2.2

(i) The variation in travel time—This can be taken into account by appropriately drawing a travel time—curve (U/S stage Vs travel time).

(ii) Varying conditions during rising and falling stages of the flood—It is always desirable to draw separate curves for rising and falling conditions as already shown in Fig. IV.2.2.

(iii) Antecedent moisture conditions of the stream—It is observed that during first few storms the actual observed water level is generally lower than the forecast level. This is presumably due to the fact that during dry condition of soil, there is more infiltration and hence lesser runoff in the stream. This can be roughly taken into account by drawing two different sets of curve, one for the few initial flood waves and the other for remaining flood waves. Such an attempt has been made for river Sone and a marked improvement has been observed.

(iv) Downstream boundary condition—This is also a very important factor, especially for the forecasting in the lower reaches of the river which falls in the sea or a larger river. When the outfall channel is in high stage or there is high tide in the sea, it will definitely have back water effect and the water level in the falling stream will be different than that in normal conditions. Hence it is always desirable to take into account the tidal effect or the water level of outfall channel. The data from the tidal gauge conjunction with the annual tidal table (published by the Survey of India) will be very useful in determining the tidal influence and back water effect on the lower reaches of the river.

(v) Characteristic of flood wave-- Generally the forecast at D/S stations are quite reliable when the storm results in formation of single peak. But when one flood wave is immediately followed by another, there is considerable effect in the water level at downstream station in different conditions.

For example when a smaller flood wave is followed by a comparatively larger flood wave with high peak, the two flood waves may overlap resulting in slight increase in the level at D/S station than that in normal case. On the contrary, if a larger flood wave is followed by a smaller flood wave, the smaller flood wave may not have any effect by the time it reaches the D/S station. This is very important aspect and can be taken care of, to some extent, by using the modified routing equations.

Various other correlation diagrams which take into account the above discussed and some other factors are described in brief in the subsequent paragraphs.

(B) Direct correlation between the peaks, at forecasting station and base station.

The gauge (peak) at the base station and the gauge (peak) at the forecasting station for the various intensities of flood are plotted. The travel time at various intensities of flood is also plotted corresponding to peak. Such graphs have been successfully used for river Subernarekha in Orissa. The graph is shown in Fig. IV.2.3. Warning time available is about 24-40 hours.

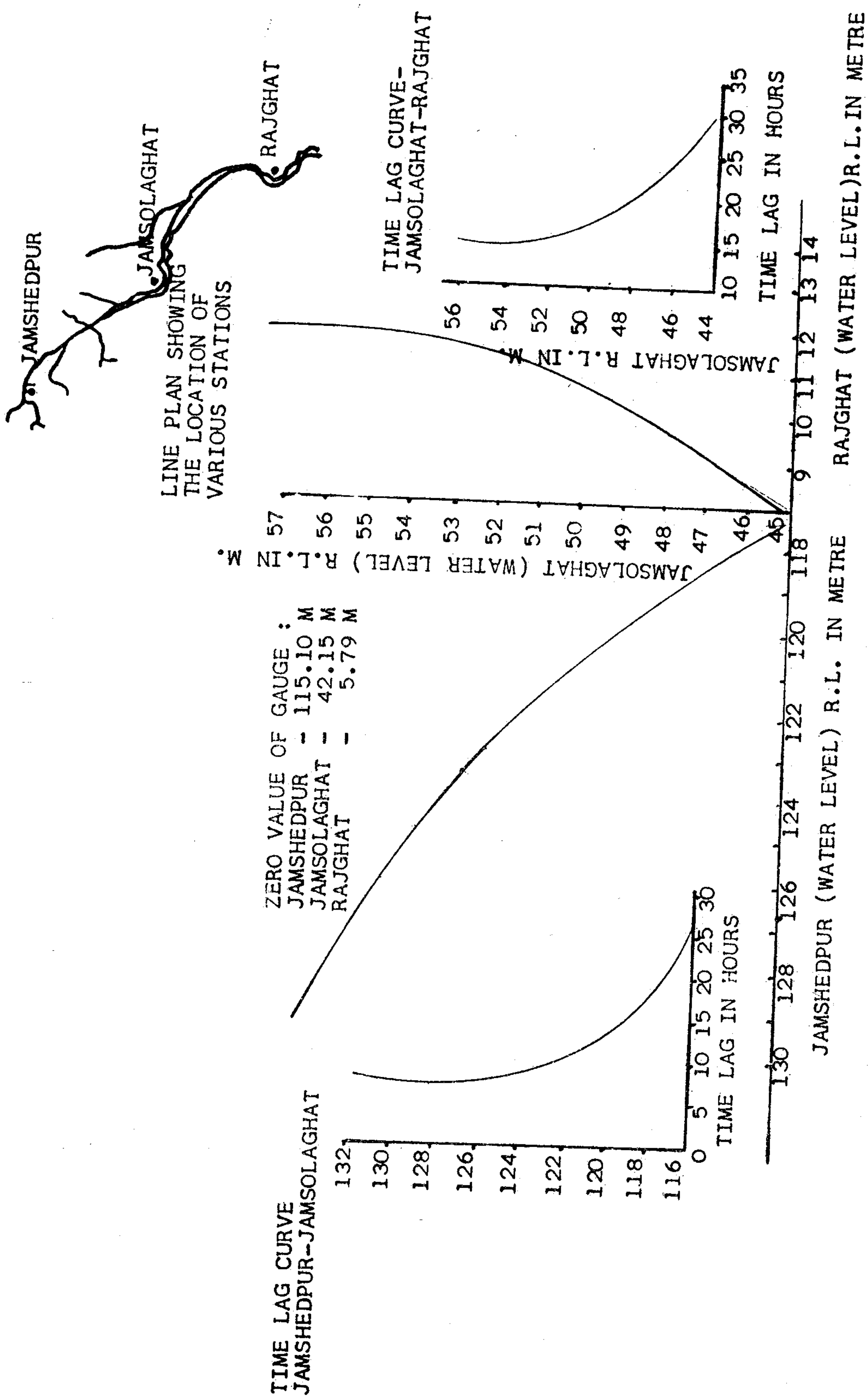

JAMSHEDPUR
JAMSOLAGHAT
RAJGHAT
LINE PLAN SHOWING THE LOCATION OF VARIOUS STATIONS
TIME LAG CURVE- JAMSOLAGHAT-RAJGHAT
56 54 52 50 48 46 44
JAMSOLAGHAT R.L. IN M.
10 15 20 25 30 35
TIME LAG IN HOURS
57 56 55 54 53 52 51 50 49 48 47 46 45
JAMSOLAGHAT (WATER LEVEL) R.L. IN M.
9 10 11 12 13 14
RAJGHAT (WATER LEVEL)R.L.IN METRE
ZERO VALUE OF GAUGE :
JAMSHEDPUR - 115.10 M
JAMSOLAGHAT - 42.15 M
RAJGHAT - 5.79 M
118 120 122 124 126 128 130
JAMSHEDPUR (WATER LEVEL) R.L. IN METRE
TIME LAG CURVE JAMSHEDPUR-JAMSOLAGHAT
132 130 128 126 124 122 120 118 116
0 5 10 15 20 25 30
TIME LAG IN HOURS

FIG. IV.2.3.

(C) Correlation between the change in stage of the base station and change in the stage of forecasting stations during T hours (T = Time of travel of flood wave between the base station and forecasting site).

Such a method obviates errors, to some extent, due to aggradation or degradation in the river section, depending upon flows. This correlation has been found more suited to large rivers with more uniform change in levels and discharges between the base stations and the forecasting stations. Such graphs developed for river Ganga between Dighaghat and Gandhighat is shown in Fig. IV.2.4. Separate graphs have been developed for Rising stage and Falling stages. A multi-tributary model for river Brahmaputra has been developed using the change in stages at the forecasting station and three different base stations on various tributaries which has been discussed subsequently.

(D) Correlation between the N^{th} hour and $(N+T)^{th}$ hour stages of the forecasting station with change in stages at the base station during past 'T' hours as variable. Different sets of graphs are drawn for rising and falling conditions of the river.

Such graphs are used for forecasting river stages at a number of sites. One such correlation used for forecasting Dalmau stage on river Ganga is shown in Fig. IV.2.5.

(E) If there is large fluctuation in the stage of the base stations, the parameter of average gauge within past 'T' hours at the base station is introduced in the Ist quadrant instead of change in stage of the base station. In the 2nd quadrant N^{th} hour stage of the base station is also labelled to account for the intensity of flood. This has been found suitable when the base station is located in the downstream of a control structure on the river through which the flows are released with wide fluctuations. One such correlation is illustrated in Fig. IV.2.6.

(F) In rivers having wide fluctuations in the stages of the upstream stations and relatively much less reduced fluctuations in lower reaches due to large scale inundation/valley storage in between the two points, tendency effect is considered. This is done by correlating N^{th} and $(N+T)^{th}$ hour stage of the forecasting site with the change in the stage of the forecasting site in Past 'T' hours as variable in the Ist quadrant. Then in the 2nd quadrant, the average gauge of the base station is considered as a variable. This type of graph has proved quite useful in Bagmati and Adhwara group of rivers of Bihar in Ganga Basin.

One such graph developed for Kamtaul site of river Adhwara is shown in Fig. IV.2.7.

(G) Gauge to Gauge correlation in coastal rivers—

The coastal rivers pose special problems with regard to formulation of forecast because of the tidal effect. The simple gauge to gauge relation will not yield satisfactory result. Before

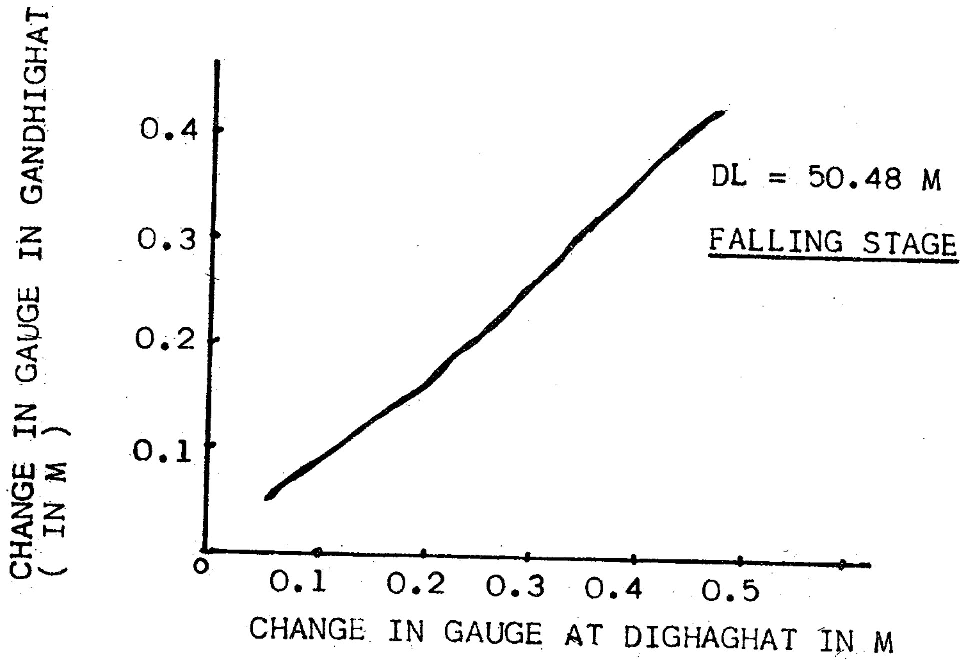
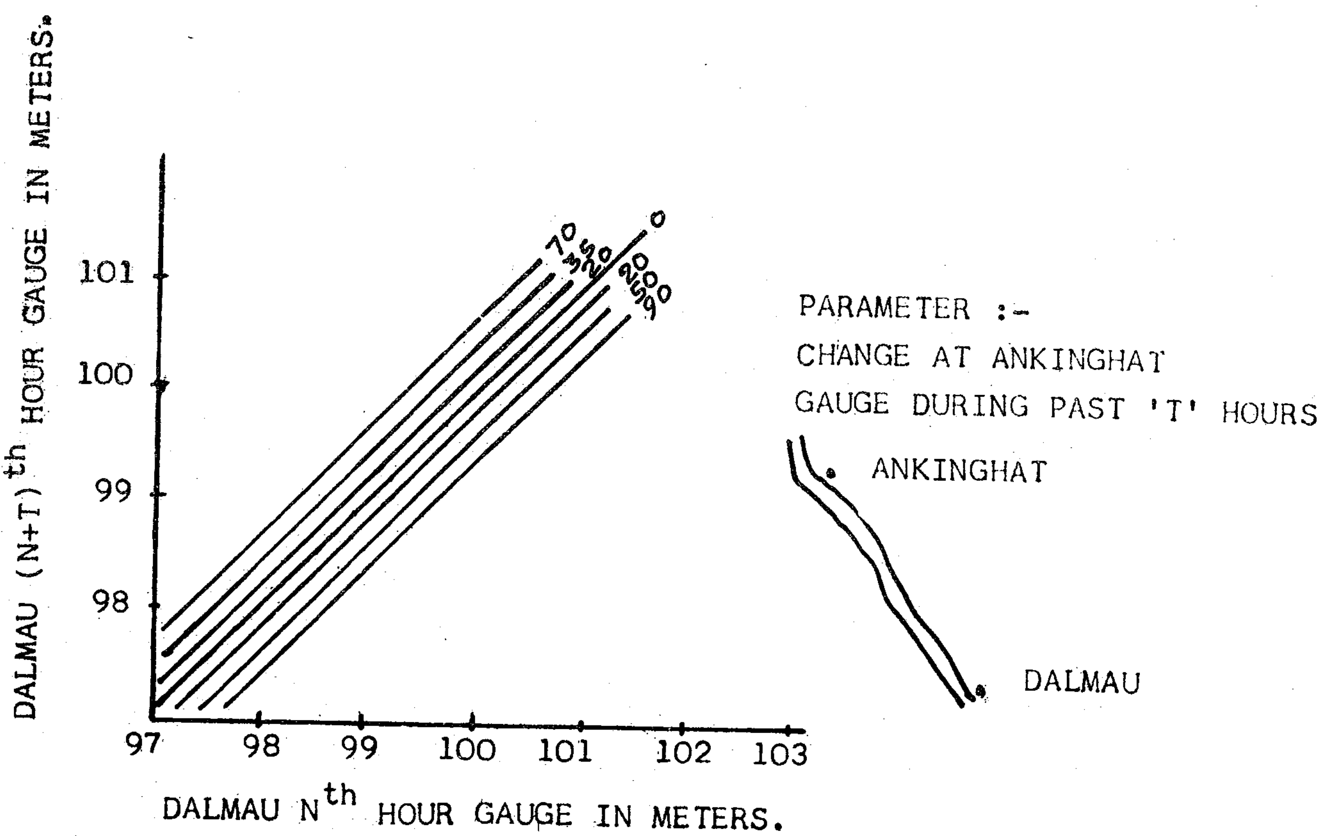


FIG. IV.2.4



CORRELATION GRAPH FOR DALMAU SITE ON RIVER GANGA.

FIG. IV.2.5

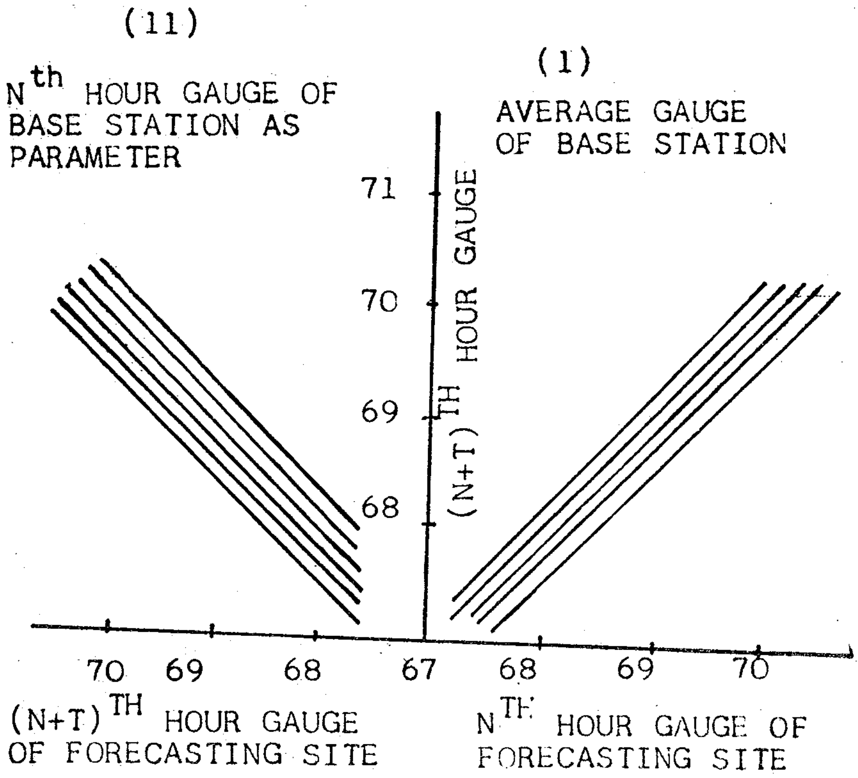


Fig. IV.2.6

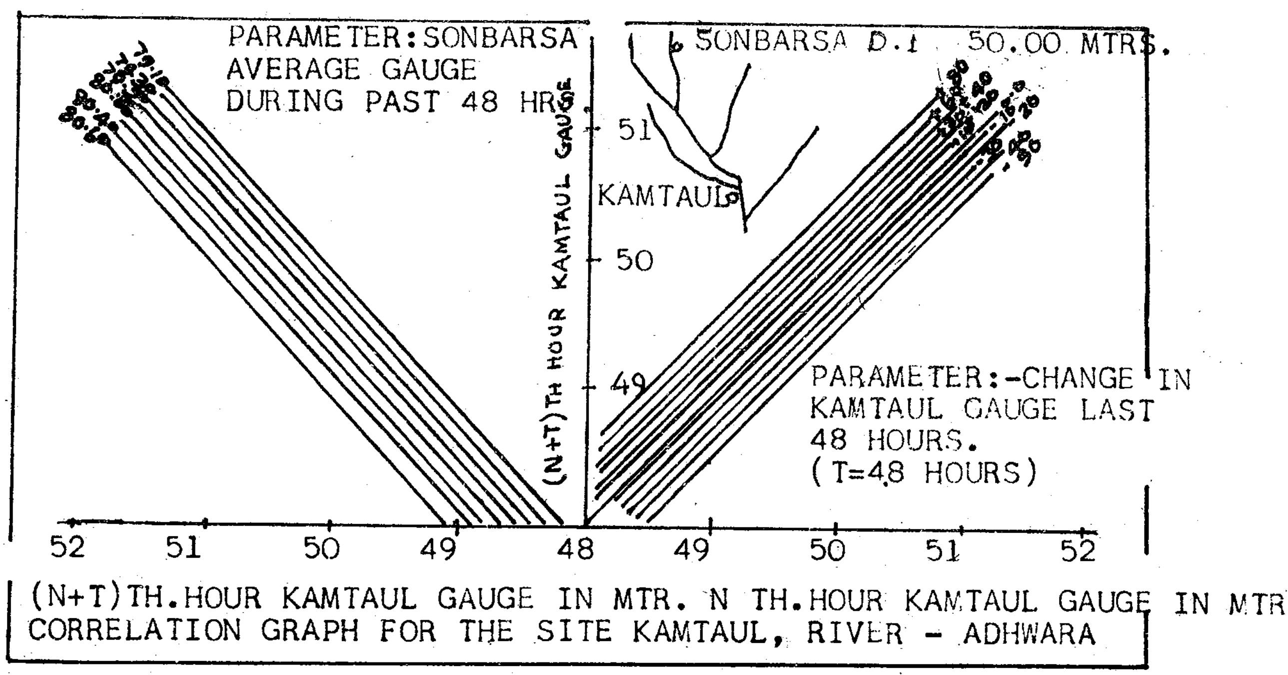


Fig. IV.2.7

developing a gauge to gauge correlation chart, it is considered imperative to analyse and identify the various conditions arising in the coastal rivers, and develop different set of curves for the formulation of forecasts, to suit the various conditions. The various cases, encountered in the coastal rivers are discussed below in brief.

(i) The river is in high stage and there is no tidal effect.

(ii) The river is in low stage and there is a tidal effect. As a result of which there will be backwater effect in the lower reaches of the river. The backwater effect has to be incorporated in the formulation of forecast.

(iii) The river is in high stage and there is a tidal effect. The river will not be able to drain freely and there will be locking effect which will affect the forecast significantly. This aspect has to be considered, while developing the charts etc. for the formulation of forecast.

Thus, it is seen that the three cases mentioned in the preceding sections, call for development of different sets of charts to be utilised for the formulation of forecast.

Realising the necessity of considering the tidal effect in the formulation of forecast for the area below Akhuapada in Baitarni river basin, a tidal gauge has been installed at Chandbali. The data from the tidal gauge in conjunction with the annual tidal table (published every year by the Survey of India) proves to be quite useful in determining the tidal influence and backwater effects on the lower reaches of the river. This will be of particular importance when tides are enhanced by storm surges accompanying the movement of tropical cyclone on shore. The data, obtained from the tidal gauges serves very useful purpose in development of charts for forecast formulation, considering the tidal effect.

Some of the correlation diagrams which have been developed using the upstream and downstream gauges have been discussed above. Besides these the discharges at upstream and downstream stations or the gauges of the upstream station and the discharges of the downstream station or vice-versa are also used for development of forecasting model and some mathematical equations have been developed and are in use. Some of them are discussed below:

(H) Paonta - Tajewala Model (for River Yamuna)

The travel time from Paonta to Tajewala being 2 hours, Paonta gauge in ft. at t hours, Gp (t) is correlated to Tajewala discharge in cusecs, Q_T (t+2).

$$Q_T(t+2) = 7.4045 \times 10^{-5} [Gp(t)]^{4.333}$$

This is for the Rising limb.

Another relation has been developed for falling limb when the travel time of 3 hours is found to be more appropriate and the relationship is:

$$Q_T\ (t+3) = 1.819 \times 10^{-6} \times [Gp\ (t)]^{5.555}$$

The graphical representation is shown in Fig. IV.2.8.

(I) Gauge—Rise models for various reaches of Yamuna.

The height of the flood wave at downstream section is related to its height at the upstream section:

$$(G_{DP} - G_{DO}) = a\ (G_{UP} - G_{UO}) + b$$

where G_{DP} and G_{UP} are the peak gauges at the downstream and upstream sections, respectively. G_{DO} and G_{UO} are the estimated gauges at the time of recorded peak, had the recession prior to the start of the flood wave continued and a and b are constants to be evaluated on the basis of past flood data. One such equation developed for Kalanaur and Delhi reach of the Yamuna is shown in the Fig. IV.2.9.

(J) Discharge—Rise Models.

The discharge rise due to a flood wave at a downstream section is related to that at an upstream section, if the effect of the intermediate catchment contribution is not significant.

$$(Q_{DP} - Q_{DO}) = m(Q_{UP} - Q_{UO}) + n$$

where Q_{DP} and Q_{UP} are the discharges at the downstream and upstream sections, Q_{DO} and Q_{UO} are the estimated discharges at the time of recorded peak, had the recession prior to the start of flood wave continued and m and n are constants to be evaluated on the basis of past flood dat.a One such relation developed for Kalanaur—Mawi reach of Yamuna is shown in Fig. IV.2.10.

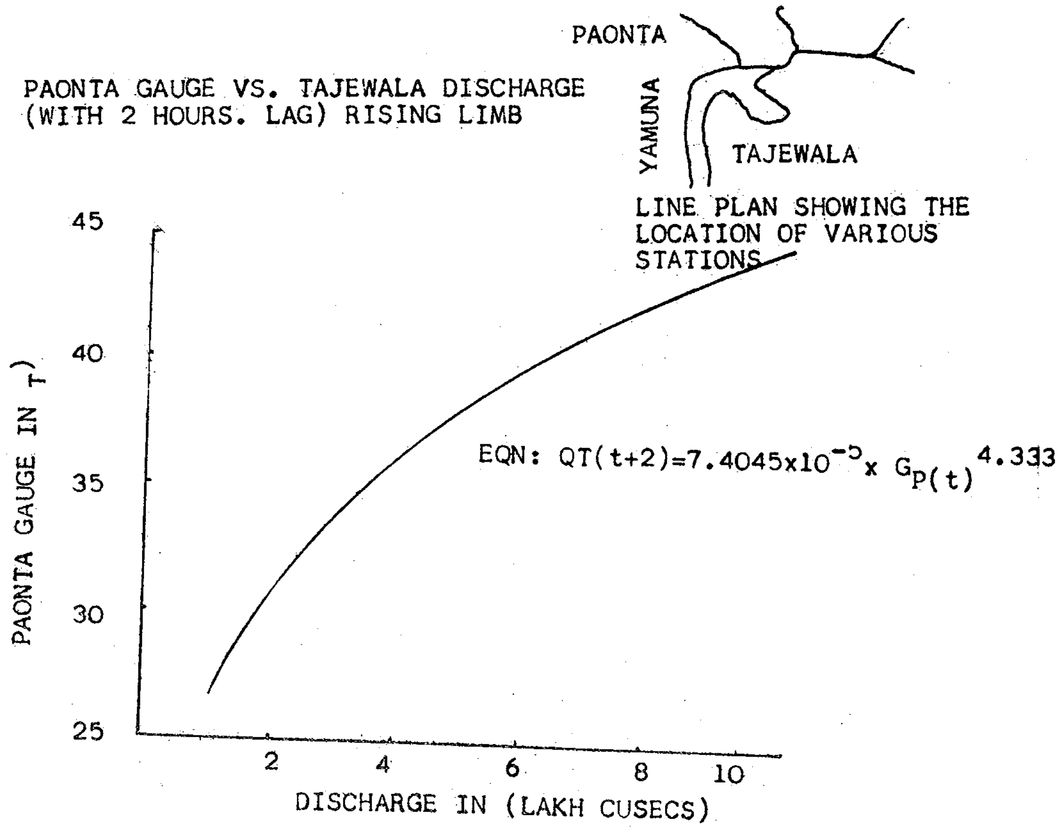
4.2.1.1 Mathematical Equations

The various graphical relations described above, can be suitably represented by a mathematical equation or a set of mathematical equations. Depending upon the plot of data, a linear or non-linear equation can be suitably selected for representation. Some of the commonly used equations are of the form:

(a) $Y = a + b\,X$

(b) $Y = a\,X^b$

(c) $Y = a\,e^{bX}$


PAONTA
PAONTA GAUGE VS. TAJEWALA DISCHARGE
(WITH 2 HOURS. LAG) RISING LIMB
YAMUNA
TAJEWALA
LINE PLAN SHOWING THE LOCATION OF VARIOUS STATIONS
45
40
35
30
25
PAONTA GAUGE IN T)
EQN: QT(t+2)=7.4045x10^-5 x G_P(t)^4.333
2
4
6
8
10
DISCHARGE IN (LAKH CUSECS)

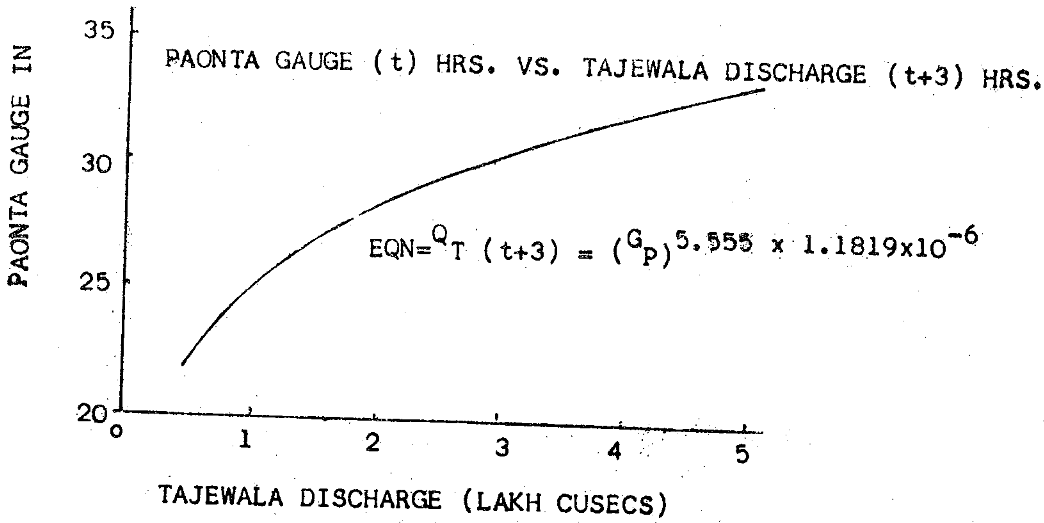
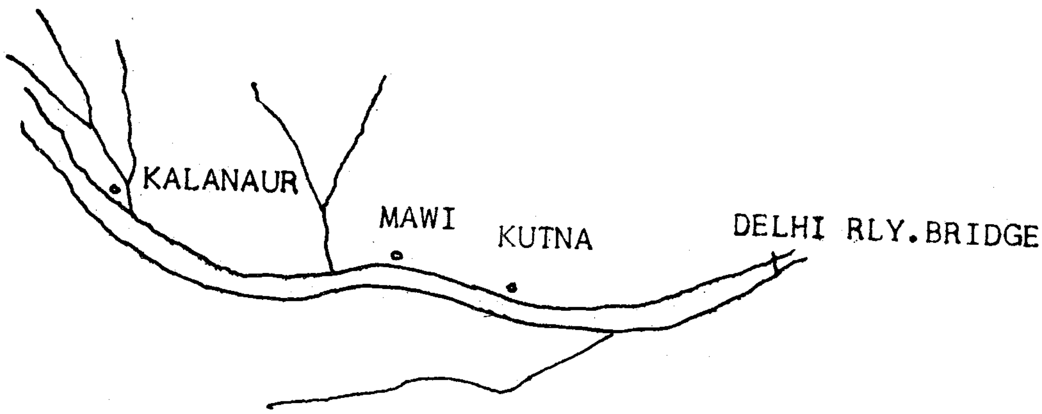

35
30
25
20
PAONTA GAUGE IN
PAONTA GAUGE (t) HRS. VS. TAJEWALA DISCHARGE (t+3) HRS.
EQN= Q_T (t+3) = (G_P)^5.555 x 1.1819x10^-6
0
1
2
3
4
5
TAJEWALA DISCHARGE (LAKH CUSECS)

FIG - IV.2.8



LINE PLAN SHOWING THE LOCATION
OF VARIOUS STATIONS

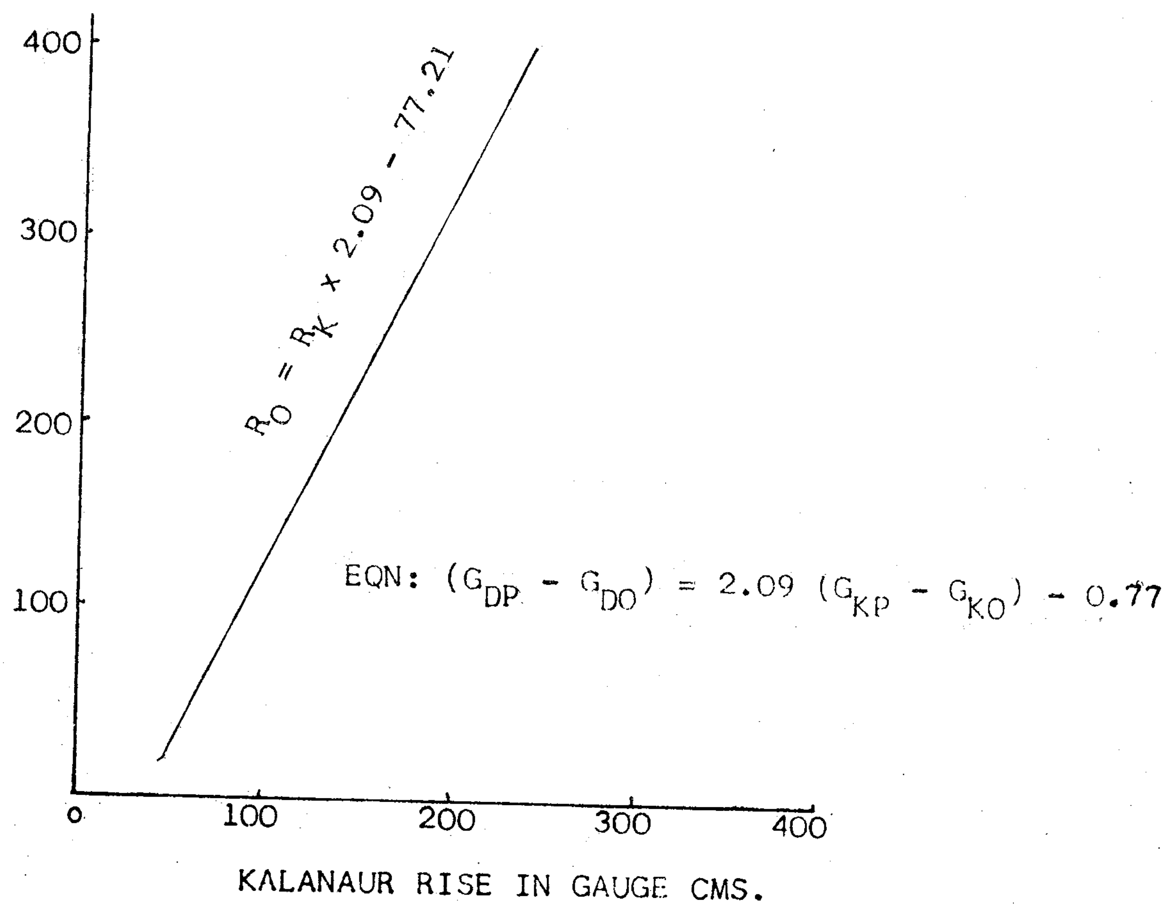
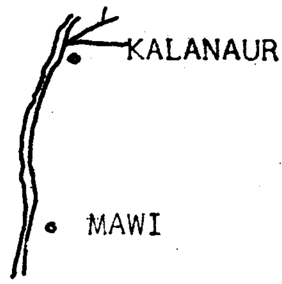
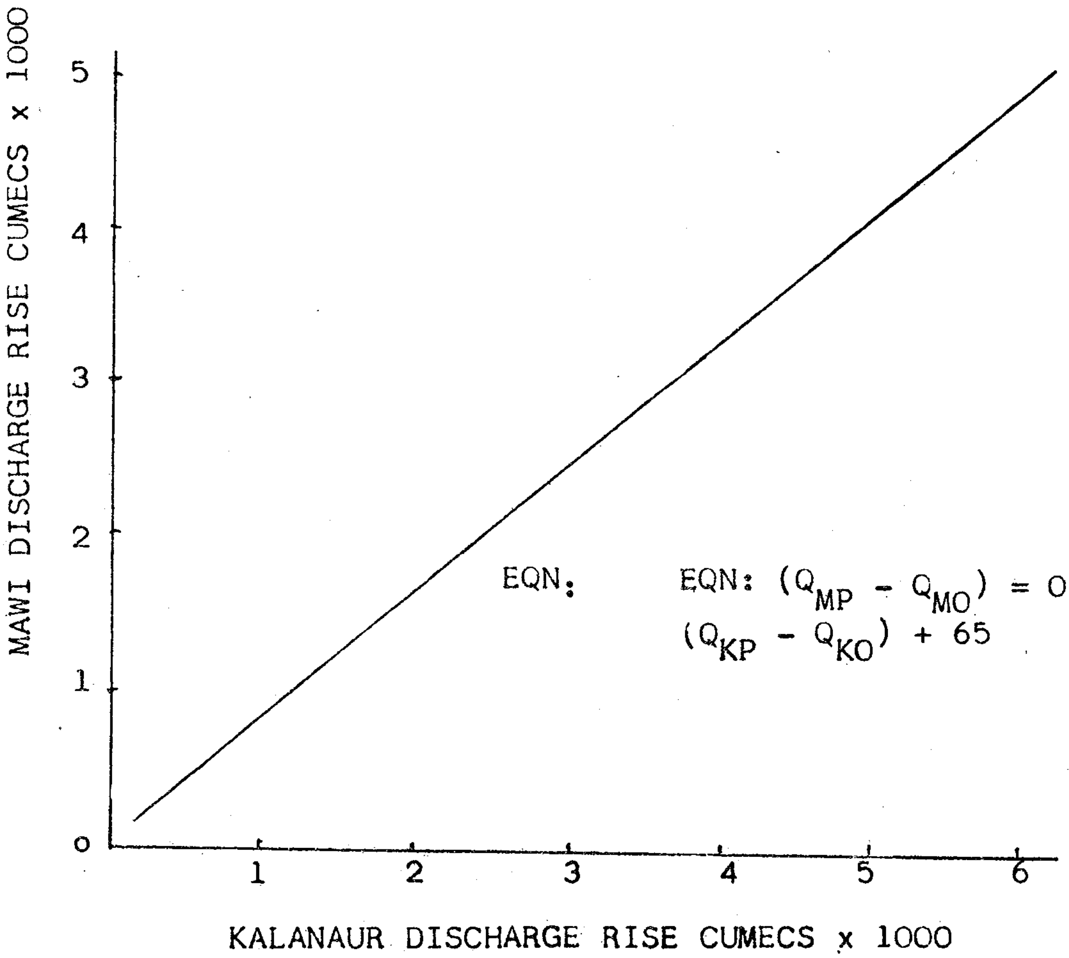


FIG.IV.2.9

LINE PLAN SHOWING THE
LOCATION OF VARIOUS
STATIONS.





CORRELATION BETWEEN RISE IN DISCHARGE AT KALANAUR AND MAWI

FIG. IV.2.10

In some specific cases, the plot can be represented by more than one linear or non-linear equations. The non-linear equations mentioned above can be suitably transformed into linear equations for the purpose of estimation of the constants a and b. For example the equation $Y = a. X^b$ can be transformed to a linear form $\log Y = \log a + b. \log X$. The values of the constants log a and b of the linearly transformed form of the equation can be easily estimated.

The most commonly used method for estimating the values of the constants of a linear equation $Y = a X + b$ is the 'least square method'. If the number of data of the two variables X and Y is n, then the constants a and b can be estimated very easily from the normal equations.

$$\sum Y = a \sum X + b.\, n$$
$$\sum XY = a \sum X^2 + b \sum X$$

The method of development of a mathematical equation correlating the independent and the dependent variables is illustrated in the example IV.2.1 below.

Example IV.2.1

The flood data from 1971 to 1979 for river Sone at Japla and Koelwar site were scrutinised and the peak flood at Japla, the corresponding peak at Koelwar and the travel time as observed are given in table IV.2.1. Establish suitable correlation between the peaks. Also establish relationship to estimate the time of occurrence of the peaks.

Solution

Since only the gauge data for upstream and downstream stations are available, a simple gauge to gauge relation is to be developed. The following steps are to be followed for the development of such relation.

Step No. 1

Plot a graph as shown in Fig. IV.2.11. with gauge at Japla on Y-axis and gauge at Koelwar on X-axis. A look at the graph indicates that a straight line can be conveniently drawn.

Step No. 2.

Instead of drawing a straight line by personal judgement, equation of the straight line $Y = aX + b$ can be developed by using least square technique.

where Y = gauge at Koelwar
X = gauge at Japla.

and a and b are constants to be estimated with the help of observed data.

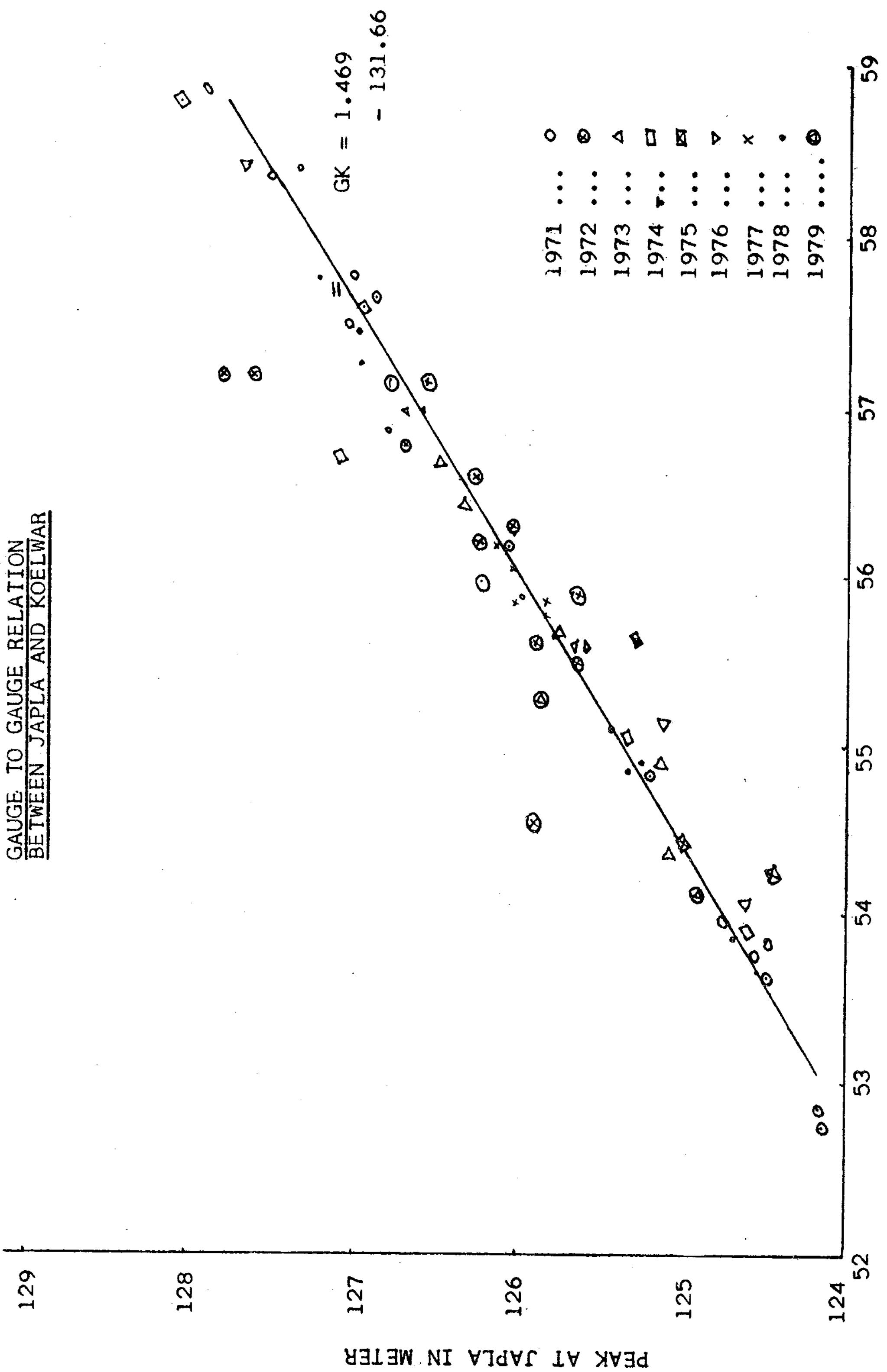


FIG.IV.2.11

GAUGE TO GAUGE RELATION BETWEEN JAPLA AND KOELWAR

Step No. 3.

The gauge at Japla (G_j) and the gauge at Koelwar (G_k) are noted down under columns 2 and 3 respectively of table IV 2.2. The lowest values of the observed peaks at Japla and Koelwar are 124.140 and 52.754 respectively and hence in order to facilitate the computation the reduced variates i.e. X and Y are obtained by deducting 124.00 and 52.00 from peaks at Japla and Koelwar respectively and are noted down under column 4 and 5 of the table IV.2.2.

Step No.4.

Compute the value of X^2, Y^2 and XY and write under column 6,7 and 8 respectively. Compute the sum of the values under column 4 to 8 to get ΣX, ΣY, ΣX^2, ΣY^2 and ΣXY.

Step No. 5.

For the straight line equation Y = aX + b, the normal equations for least square error will be

$$\Sigma Y = a\,\Sigma X + b.\,n$$
$$\Sigma XY = a\,\Sigma X^2 + b\Sigma X$$

Here from Table No. IV.2.2 we get

$$\Sigma X = 108.63$$
$$\Sigma Y = 216.577$$
$$\Sigma X^2 = 263.995$$
$$\Sigma Y^2 = 955.652$$
$$XY = 497.573$$

TABLE — IV.2.1

Date	*Peak at Japlan in metre*	*Peak at Koelwar in metre*	*Travel time (in Hour)*
	<u>Year 1971</u>		
28.6.71	126.060	56.167	30
20.7.71	127.880	58.884	13
29.7.71	127.500	58.369	18
4.8.71	126.990	57.794	18

Date	*Peak at Japlan in metre*	*Peak at Koelwar in metre*	*Travel time (in Hour)*
10.8.71	127.040	57.484	21
1.9.71	126.890	57.644	20
	Year 1972		
6.7.72	124.755	53.959	38
14.7.72	125.900	54.539	33
17.7.72	125.890	55.599	33
8.8.72	127.790	57.174	24
16.8.72	126.050	56.269	26
31.8.72	126.250	56.529	26
15.9.72	125.770	55.884	30
	Year 1973		
14.7.73	124.160	52.834	37
20.7.73	125.090	54.354	33
24.7.73	125.140	54.689	36
20.8.73	126.340	56.404	26
30.8.73	126.490	56.654	26
6.9.73	125.770	55.664	31
26.9.73	125.610	55.594	31
	Year 1974		
20.6.74	124.140	52.754	34
20.7.74	124.615	53.894	35

Date	*Peak at Japlan in metre*	*Peak at Koelwar in metre*	*Travel time (in Hour)*
	<u>Year 1974</u>		
19.8.74	127.080	57.689	14
27.8.74	125.340	55.064	33
	<u>Year 1975</u>		
5.7.75	124.500	53.634	43
20.7.75	126.940	57.594	20
11.8.75	125.020	54.424	34
22.8.75	128.040	58.814	13
8.9.75	125.310	55.634	32
6.10.75	124.450	54.249	46
	<u>Year 1976</u>		
11.7.76	124.480	53.824	43
23.7.76	124.620	54.079	44
29.7.76	125.660	55.514	31
16.8.76	126.710	56.954	21
22.8.76	125.140	55.144	29
17.9.76	127.640	58.434	10
	<u>Year 1977</u>		
27.6.77	125.210	54.820	38
29.6.77	125.840	55.834	29
8.7.77	125.840	55.744	33

Date	Peak at Japlan in metre	Peak at Koelwar in metre	Travel time (in Hour)
	Year 1977		
17.7.77	126.140	56.184	30
30.7.77	126.960	57.464	14
8.8.77	126.580	56.974	22
15.8.77	126.040	56.044	27
10.9.77	126.020	55.814	28
	Year 1978		
26.6.78	125.190	55.144	38
18.7.78	124.700	53.864	42
29.7.78	125.340	54.854	37
5.8.78	126.790	56.854	20
12.8.78	125.990	56.044	30
27.8.78	125.270	54.894	36
4.9.78	127.220	57.774	14
24.9.78	127.340	58.414	23
5.10.78	125.440	55.114	33
	Year 1979		
2.7.79	124.850	53.994	40
19.7.79	124.900	54.154	41
10.8.79	126.060	56.154	27
18.8.79	125.860	55.274	34

TABLE — IV.2.2.

Sl. No.	Gauge at Japla Gj (m)	Gauge at Koelwar G_k (m)	Gj-124 X	G_k-52 Y	X^2	Y^2	XY
1	2	3	4	5	6	7	8
1.	126.060	56.167	2.060	4.167	4.244	17.364	8.584
2.	127.880	58.884	3.880	6.884	15.054	47.389	26.710
3.	127.500	58.369	3.500	6.369	12.250	40.564	22.292
4.	126.990	57.794	2.990	5.794	8.946	33.570	17.324
5.	127.040	57.484	3.040	5.484	9.242	30.074	16.671
6.	126.890	57.644	2.890	5.644	8.352	31.855	16.311
7.	124.755	53.959	0.755	1.959	0.570	3.838	1.479
8.	125.900	54.539	1.900	2.539	3.610	6.446	4.824
9.	125.890	55.599	1.890	3.599	3.572	12.953	6.802
10.	127.790	57.174	3.790	5.174	14.364	26.770	19.609
11.	126.050	56.269	2.050	4.269	4.203	18.224	8.751
12.	126.250	56.529	2.250	4.529	5.063	20.512	10.190
13.	125.770	55.884	1.770	3.884	3.133	15.085	6.875
14.	124.160	52.834	0.160	0.834	0.026	0.696	0.133
15.	125.090	54.354	1.090	2.354	1.188	5.541	2.566
16.	125.140	54.689	1.140	2.689	1.300	7.231	3.065
17.	126.340	56.404	2.340	4.404	5.476	19.395	10.305
18.	126.490	56.654	2.490	4.654	6.200	21.660	11.588

Sl. No.	Gauge at Japla Gj (m)	Gauge at Koelwar G_k (m)	Gj-124 X	G_k-52 Y	X^2	Y^2	XY
1	2	3	4	5	6	7	8
19.	125.770	55.664	1.770	3.664	3.133	13.425	6.485
20.	125.610	55.594	1.610	3.594	2.592	12.917	5.786
21.	124.140	52.754	0.140	0.754	0.020	0.569	0.106
22.	124.615	53.894	0.615	1.894	0.378	3.587	1.220
23.	127.080	57.689	3.080	5.689	9.486	32.365	17.522
24.	125.340	55.064	1.340	3.064	1.796	9.388	4.106
25.	124.500	53.634	0.500	1.634	0.250	2.670	0.817
26.	126.940	57.594	2.940	5.594	8.644	31.293	16.446
27.	125.020	54.424	1.020	2.424	1.040	5.876	2.472
28.	128.040	58.814	4.040	6.814	16.322	46.431	27.529
29.	125.310	54.634	1.310	2.634	1.716	13.206	3.451
30.	124.450	54.249	0.450	2.249	0.203	5.058	1.012
31.	124.480	53.824	0.480	1.824	0.230	3.327	0.876
32.	124.620	54.079	0.620	2.079	0.384	4.322	1.289
33.	125.660	55.514	1.660	3.514	2.756	12.384	5.833
34.	126.710	56.954	2.710	4.954	7.344	24.542	13.425
35.	125.140	55.144	1.140	3.144	1.300	9.885	3.584
36.	127.640	58.434	3.640	6.434	13.250	41.448	23.420
37.	125.210	54.820	1.210	2.820	1.464	7.952	3.412

Sl. No.	Gauge at Japla Gj (m)	Gauge at Koelwar G_k (m)	Gj-124 X	G_k-52 Y	X^2	Y^2	XY
1	2	3	4	5	6	7	8
38.	125.840	55.834	1.840	3.834	3.386	14.700	7.055
39.	125.840	55.744	1.840	3.744	3.386	14.018	6.889
40.	126.140	56.184	2.140	4.184	4.580	17.056	8.954
41.	126.960	57.464	2.960	5.464	8.762	29.855	16.173
42.	126.580	56.974	2.580	4.974	6.656	24.741	12.833
43.	126.040	55.044	2.040	4.044	4.162	16.354	8.250
44.	126.020	55.814	2.020	3.814	4.080	14.547	7.704
45.	125.190	55.144	1.190	3.144	1.416	9.885	3.741
46.	124.700	53.864	0.700	1.864	0.490	3.474	1.305
47.	125.340	54.854	1.340	2.854	1.800	8.145	3.824
48.	126.790	56.854	2.790	4.854	7.784	23.561	13.543
49.	125.990	56.044	1.990	4.044	3.960	16.354	8.048
50.	125.270	54.894	1.270	2.894	1.613	8.375	3.675
51.	127.220	57.774	3.220	5.774	10.368	33.339	18.592
52.	127.340	58.414	3.340	6.414	11.156	41.139	21.423
53.	125.440	55.114	1.440	3.114	2.074	9.697	4.484
54.	124.850	53.994	0.850	1.994	0.723	3.976	1.695

Sl. No.	Gauge at Japla Gj (m)	Gauge at Koelwar G_k (m)	Gj-124 X	G_k-52 Y	X^2	Y^2	XY
1	2	3	4	5	6	7	8
55.	124.900	54.134	0.900	2.134	0.810	4.554	1.921
56.	126.060	56.154	2.060	4.154	4.244	17.256	8.557
57.	125.860	55.274	1.860	3.274	3.460	10.719	6.090
		Total:	108.630	216.577	263.995	955.652	497.573

Substituting the values, we get the equation

$$216.577 = 108.63a + 57b \qquad \text{(i)}$$

$$497.573 = 263.955a + 108.63b \qquad \text{(ii)}$$

or

$$3.7996 = 1.9058a + b \qquad \text{(iii)}$$

$$4.5804 = 2.4302a + b \qquad \text{(iv)}$$

From equation (iii) and (iv)

$$a = \frac{0.7808}{0.5244} = 1.4889$$

and $b = 0.9620$

the equation is

$$Y = 1.4889X + 0.9620$$

substituting $Y = G_k - 52$

and $X = G_j - 124$

$$(G_k - 52) = (G_j - 124) \times 1.4889 + 1.9620$$

or $$G_k = 1.4889\ G_j - 131.6616 \qquad \text{(v)}$$

Equation (v) gives the relation between the peak at Japla and Koelwar.

The co-efficient of correlation (r) is calculated as follows:

$$r = \frac{\Sigma XY - \frac{\Sigma X . \Sigma Y}{n}}{\sqrt{\left[\Sigma X^2 - \frac{(\Sigma X)^2}{n}\right]\left[\Sigma Y^2 - \frac{(\Sigma Y)^2}{n}\right]}}$$

Substituting the values,

$$r = \frac{497.573 - \frac{108.63 \times 216.577}{57}}{\sqrt{\left[263.955 - \frac{(108.63)^2}{57}\right]\left[955.652 - \frac{(216.577)^2}{57}\right]}}$$

$$= \frac{84.8228}{\sqrt{56.9691 \times 132.7468}}$$

$$= \frac{84.8228}{86.9624} = 0.9754$$

The co-efficient of correlation (r) = 0.9754

This indicates a very good correlation and the equation can be used for operational flood forecast. However, from the graph it may be seen that a few points are very much out. A verification of these data indicated that some of these values refer to night observations and hence their accuracy is doubtful. Such data were deleted and other equation was developed in which case the values of co-efficient of correlation works-out to be 0.993 and the equation is

$$G_k = 1.574 \times G_j - 142.362 \qquad \text{(vi)}$$

In order to establish a relation for estimating the time of occurrence of the predicated peak, a similar procedure is adopted as in case of peak to peak relation as already developed above.

A relation has been developed by using least square technique and adopting the similar procedure as already discussed above. The relation works out to be

$$T = 1085.93 - 8.3906 \times Gj$$

The correlation co-efficient works out to be 0.93. Alternatively a suitable graphical relation can also be developed for operational use.

It was observed in the past that while forecasting on the basis of equation (vi), there was substantial errors in the prediction of forecasts for first few unprecedented peaks of the season. Keeping this in view, the first few unprecedented floods of the season were segregated from the rest and attempts were made to fit two separate straight lines which gave the equations as below.

For the unprecedented flood peaks of the season, the equation developed was

$$G_k = 1.587 \times G_j - 143.256 \qquad \text{(vii)}$$

with the correlation coefficient of 0.993 and for the rest of the flood peaks

$$G_k = 10556 \times G_j - 142.102 \qquad \text{(viii)}$$

with the coefficient of correlation as 0.991.

The various steps involved in the development of a simple relationship between the peaks at upstream and downstream stations have been discussed in detail. However, it is not always necessary that this particular method will give the best result and hence it is desirable to attempt various other alternatives as well. The different alternatives which have been tried to improve the forecast performance for river Sone are discussed below in brief.

(A) Correlations between the peak discharges at Japla and Koelwar have also been derived. For the purpose, the peak discharges have been computed from the gauge-discharge curves for Koelwar and Japla. Peak discharges at Koelwar have been plotted on Y-axis and peak discharges at Japla on the X-axis and a straight line equation was developed by using least square techniques. The resultant equation works out to

$$Q_k = 1.524 \times Q_j - 1677$$

where Q_k is the peak discharge at Koelwar in cumecs; and Q_j is the corresponding peak discharge at Japla in cumecs.

The correlation coefficient in this case works out to 0.989 which is quite satisfactory and can be confidently used for operational flood forecasting as well.

(B) A similar correlation was attempted for peak discharges at Koelwar and Chopan, a station further upstream of Japla on river Sone, which is used for giving advisory forecasts for Koelwar, and corresponding peak discharges at Koelwar. An additional feature of this reach is that, North Koel, a tributory of river Sone, joins it, a few kilometers upstream of Japla. Therefore, instead of directly correlating the Koelwar peak discharge with the Chopan peak discharge, it was correlated to peak discharge at Chopan plus contribution of North Koel as obtained from gauge-discharge curve of Daltonganj site on the river, with proper lag time. Peak discharges at Koelwar were plotted on the Y-axis and the peak discharge of Chopan plus corresponding contribution of North Koel at Daltonganj on the X-axis, and a straight line was attempted which gave a coefficient of correlation as 0.92 and the equation as follows:

$$Q_{k\,(N+T)} = (Q_{c\,(N)} + Q_{d(N+T-T')}) \times 0.998 + 400$$

where

$Q_{k(N+T)}$ is the peak discharge of Koelwar at $(N+T)^{th}$ hours in cumecs.

$Q_{c(N)}$ is the corresponding peak discharge of Chopan in Cumecs.

$Q_{d(N+T-T')}$ is the discharge at Daltonganj at $(N+T-T')^{th}$ hours.

where

T is the travel time between Chopan and Koelwar
T is the travel time between Daltonganj and Koelwar.

(C) One of the major difficulties faced in forecasting for Koelwar on river Sone is the forecast for intermediate stage, i.e. stages before the peak is attained. For the purpose, attempts were made to predict the concentration segment of the hydrograph for Koelwar based on the concentration segment of the hydrograph for Japla. In order to predict the concentration segment of Koelwar hydrograph, two parameters need to evaluated. First is the peak which is likely to be attained and second, the time in which this peak will be attained. First parameter can be determined as discussed in the preceding paragraphs. For determination of second parameter i.e. the time of rise at Koelwar, attempts were made to correlate, the time of rise to peak at Japla vis-a-vis time to rise at Koelwar. The following equation with coefficient as 0.85 was developed.

$$T_{RK} = 0.657\, T_{RJ} + 8.28$$

where

T_{RK} is the time of rise to peak at Koelwar in hours.

T_{RJ} is the time of rise to peak at Japla in hours.

The correlation coefficient is not satisfactory for operational use. Attempts to improve the correlation coefficient by taking the average gauge of Japla, peak of Japla and initial gauge of Japla at the time of beginning as parameters were also not successful.

The mathematical equations have been developed for forecasting at a number of sites and some of them are given below.

(i) Mathematical Equations for Chatia on River Gandak.

(A) **For Rising stages.**

$$(CH_{N+28} - CH_N) = 1/3\ (TR_N - TR_{N-28})$$

(B) **For Falling stages.**

$$(CH_N - CH_{N+28}) = 0.5\ (TR_{N-28} - TR_N)$$

where

CH_{N+28} = $(N+28)^{th}$ hour gauge of Chatia in m;

CH_N = N^{th} hour gauge of Chatia in m;

TR_N = N^{th} hour gauge of Triveni in m; and

TR_{N-28} = $(N-28)^{th}$ hour gauge of Triveni in m.

The travel time has been taken as 28 hours.

(ii) Mathematical Equation for Sikanderpur on River Burhi Gandak

(A) **For Rising Stages.**

$$SIK_{N+t} = SIK_N + 0.61055\ (AHR_N - AHR_{N-t})^{1.167048}$$

(B) **For Falling stages.**

$$SIK_{N+t} = SIK_N - 1.1015\ (AHR_{N-t} - AHR_N)^{1.46115}$$

where

SIK_{N+t} = Water level of Sikanderpur at $(N+t)^{th}$ hour in m;

SIK_N = Water level of Sikanderpur at N^{th} hour in m;

AHR_N = Water level of Ahirwalia at N^{th} hour in m;

AHR_{N-t} = Water level of Ahirwalia at $(N-t)^{th}$ hour in m; and

t = Travel time (= 22 hours)

(iii) Mathematical Equation for Samastipur on River Burhi Gandak.

(A) **For Rising Stages.**

$$SMS_{N+t} = SMS_N + (SIK_N - SIK_{N-t}) + 0.05$$

(B) **For Falling Stages.**

$$SMS_{N+t} = SMS_{N+t} - 1.2(SIK_{N-t} - SIK_N)$$

where

SMS_{N+t} = Water level of Samastipur at $(N+t)^{th}$ hour in m;

SMS_N = Water level of Samastipur at N^{th} hour in m;

SIK_N = Water level of Sikanderpur at N^{th} hour in m;l

SIK_{N-t} = Water level of Sikanderpur t $(N-t)^{th}$ hour in m; and

t = Travel time (= 22 hours)

(iv) Mathematical Equation for Basua on River Kosi.

(A) **For Rising Stages.**

$$BAS_{N+t} = BAS_N + 0.64607(BIR_N - BIR_{N-t})^{0.86851}$$

(B) **For Falling Stages.**

$$BAS_{N+t} = BAS_N + 0.64607(BIR_{N-t} - BIR_N)^{0.86851}$$

where

BAS_{N+t} = $(N+t)^{th}$ hour gauge of Basua in m;

BAS_N = N^{th} hour gauge of Basua in m;

BIR_N = N^{th} hour gauge of Birpur in m; and

BIR_{N-t} = $(N-t)^{th}$ hour gauge of Birpur in m; and

t = Travel time (= 16 hours)

4.2.2 Multivariate Correlation

When the direct gauge to gauge correlation are not successful because of appreciable contribution due to rainfall in the inreach catchment, intermediate tributaries or the varying soil moisture condition etc; then the introduction of additional parameters of discharge of the tributary, average rainfall over the intercepting catchment, API etc. become necessary and it gives better results.

With the availability of more and more data and introduction of better data transmission facilities, the correlation diagrams are being developed with more and more additional parameters. The various parameters are introduced in different quadrants.

Some of such diagrams which are at present under use are discussed below in brief

(A) Correlation between Nth hour upstream gauge and $(N+T)^{th}$ hour downstream gauge with N^{th} hour gauge of tributary as a parameter. Such diagram has been developed for forecasting Akhuapada gauge using the Anandpur gauge and Kusai gauge as a parameter for river Baitarani in Orissa. Fig. IV.2.12 shows the correlation diagram.

(B) Correlation between the N^{th} hour, and $(N+T)^{th}$ hour gauge of forecasting station with change in the level of a tributary during past T_1 hours and change in level of the base station during past T hour.

Such correlation diagram has been developed for forecasting Chatnag Gauge and is shown in Fig. IV.2.13 Correlation Diagram has been developed for rising as well as falling stages.

(C) Correlation between Nth hour and $(N+T)^{th}$ hour gauge of forecasting station with following parameters;

(i) Rise/Fall at upstream base station;

(ii) Rainfall observed at the upstream base station.

Example is the diagram for formulating the forecasts at Khowang on river Brahmaputra under Dibrugarh Division which is shown in Fig. IV.2.14. Only rainfall data of upstream station has been used as there is no other SRRG with the transmission facilities in the intervening catchment.

(D) Correlation between Nth Day and (N+1)th Day gauge of Neamtighat with following parameters;

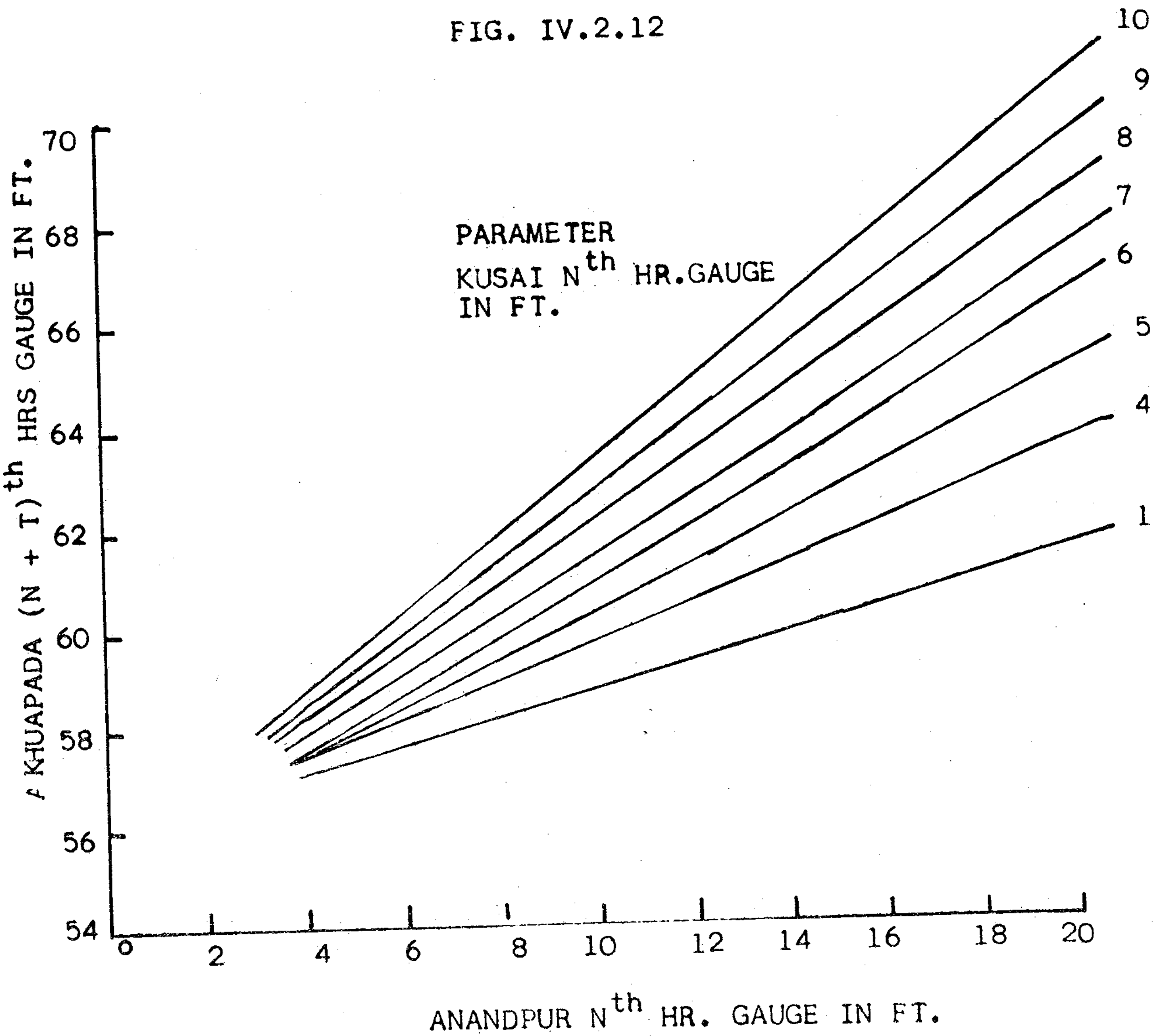
(i) Rise/Fall in Dibrugarh Gauge;

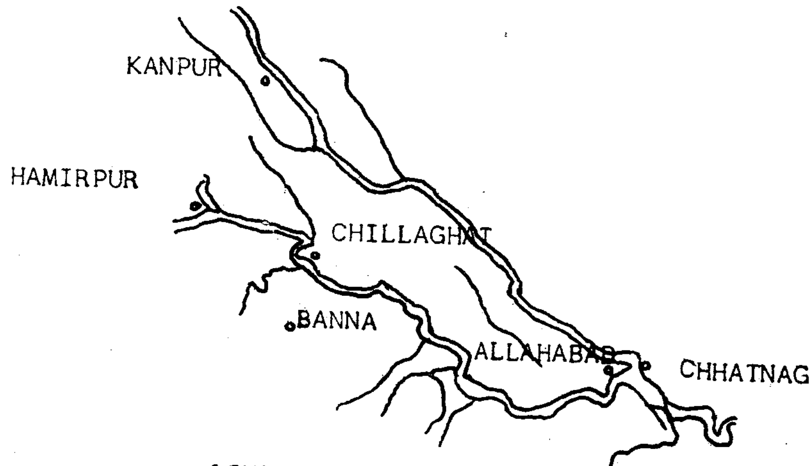
(ii) Rise/Fall in Khowang Gauge;

(iii) Rainfall.

Fig IV.2.15 shows the correlation diagram.

FIG. IV.2.12


PARAMETER
KUSAI Nth HR.GAUGE
IN FT.
KHUAPADA (N + T)th HRS GAUGE IN FT.
ANANDPUR Nth HR. GAUGE IN FT.
10
9
8
7
6
5
4
1
70
68
66
64
62
60
58
56
54
0
2
4
6
8
10
12
14
16
18
20



LINE PLAN SHOWING THE
LOCATION OF VARIOUS
STATIONS

CORRELATION GRAPH FOR FLOOD FORECASTING SITE
CHHATNAG ON RIVER GANGA.
FLOOD SEASON 1978
(YAMUNA RISING STAGE)

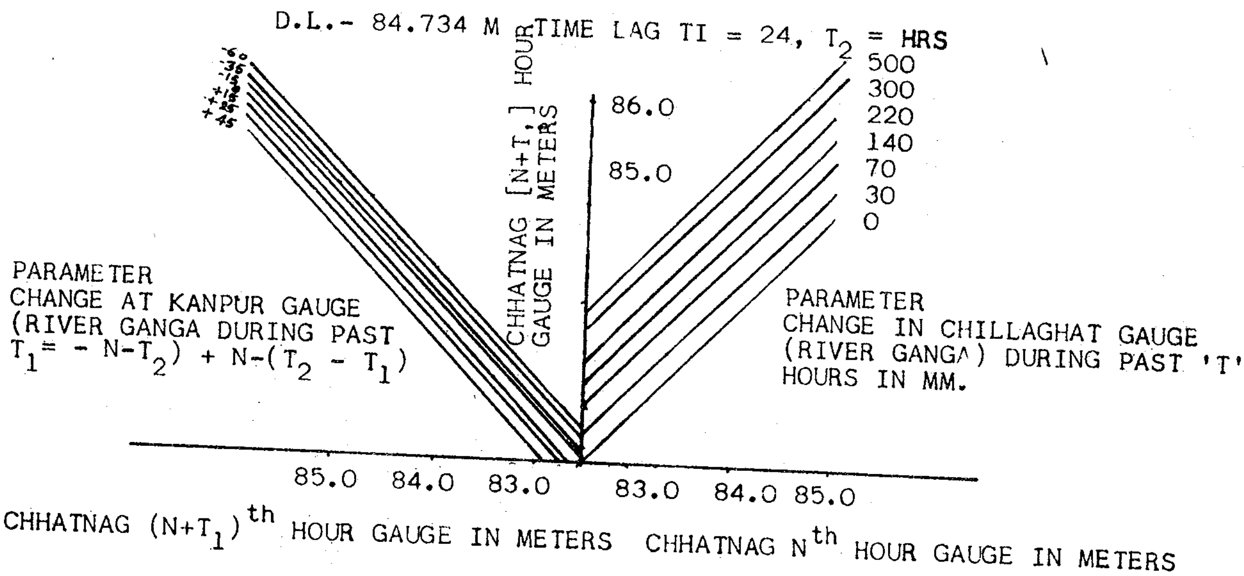


FIG.- IV.2.13

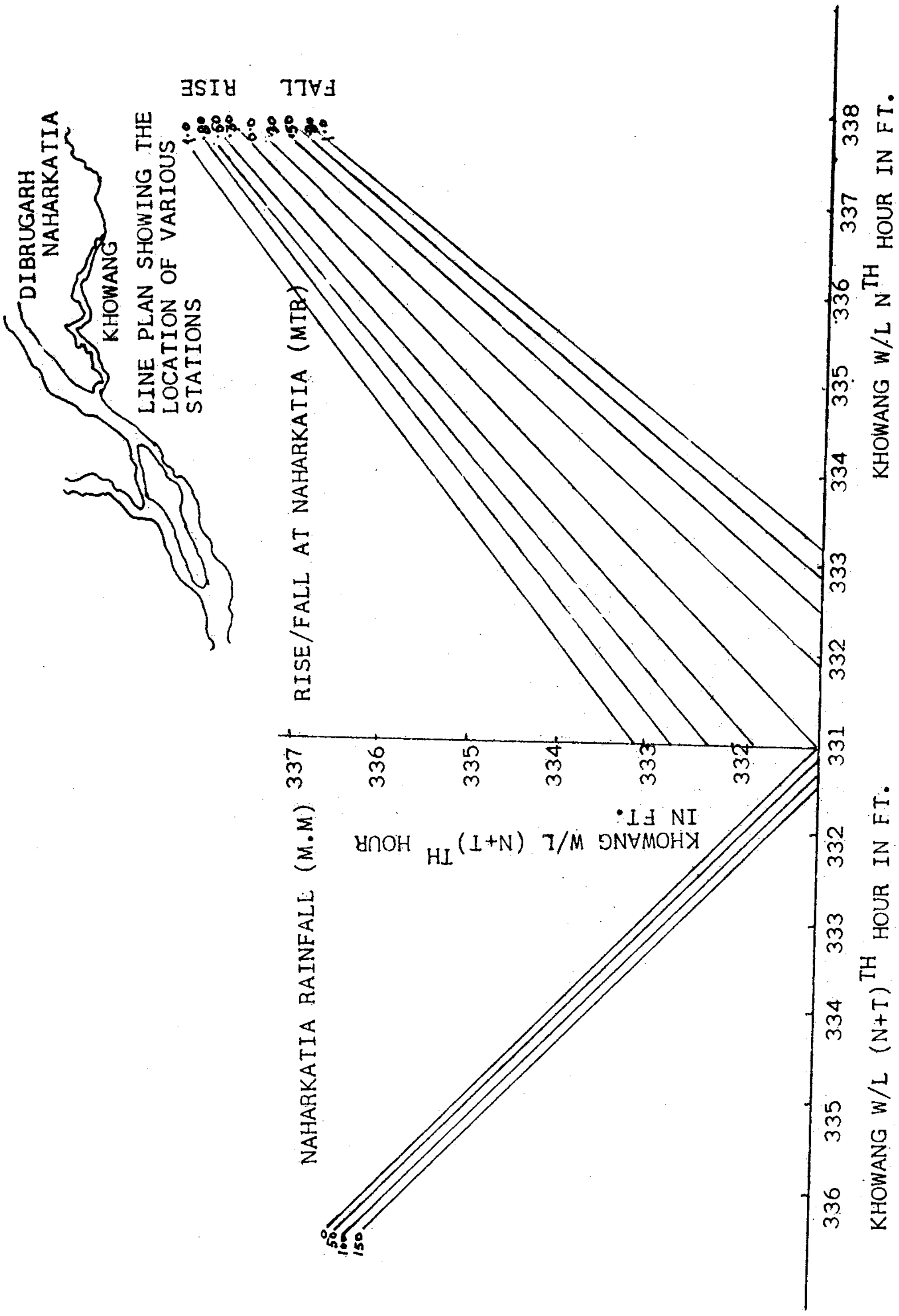
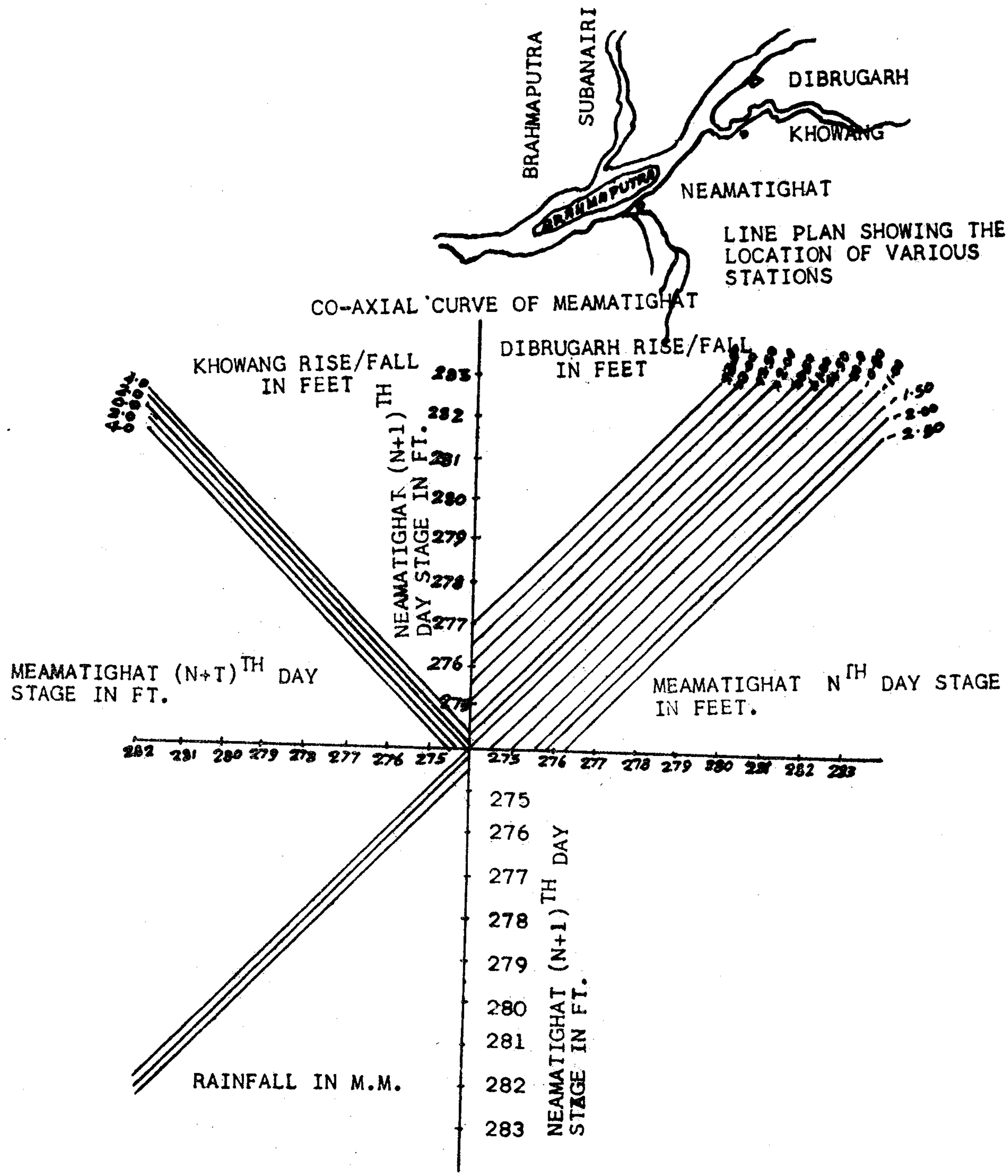

DIBRUGARH
NAHARKATIA
KHOWANG
LINE PLAN SHOWING THE LOCATION OF VARIOUS STATIONS
RISE
FALL
RISE/FALL AT NAHARKATIA (MTR)
NAHARKATIA RAINFALL (M.M)
KHOWANG W/L (N+T)TH HOUR IN FT.
KHOWANG W/L NTH HOUR IN FT.
KHOWANG W/L (N+T)TH HOUR IN FT.
337
336
335
334
333
332
331
338

FIG. - IV.2.14



BRAHMAPUTRA
SUBANAIRI
DIBRUGARH
KHOWANG
NEAMATIGHAT
LINE PLAN SHOWING THE LOCATION OF VARIOUS STATIONS
CO-AXIAL CURVE OF MEAMATIGHAT
KHOWANG RISE/FALL IN FEET
DIBRUGARH RISE/FALL IN FEET
NEAMATIGHAT (N+1)TH DAY STAGE IN FT.
MEAMATIGHAT (N+T)TH DAY STAGE IN FT.
MEAMATIGHAT NTH DAY STAGE IN FEET.
RAINFALL IN M.M.
NEAMATIGHAT (N+1)TH DAY STAGE IN FT.
275
276
277
278
279
280
281
282
283

FIG. IV.2.15

(E) Correlation between N^{th} day Guwahati gauge and $(N+1)^{th}$ day Guwahati gauge with following parameters:

(i) Variation in the Tezpur gauge;

(ii) Average rainfall of (a) Tezpur (b) Guwahati (c) Tangla (d) Mazbat and (e) Dharmatul;

(iii) API.

The Fig. IV.2.16 shows the correlation.

(F) Correlation Diagram for Dowlaiswaram site on river Godavari is based on the correlation between Nth hour Dowlaiswaram Gauge and (N+T)th hour Dowlaiswaram Gauge with following additional parameters.

(i) Variation of gauge during travel time at upstream station Bhadrachalam.

(ii) Variation of gauge during travel time at Konta on River Sabari, a tributary.

(iii) API of the Day for intermediate catchment.

(iv) Average daily rainfall of the day in the intermediate catchment.

Fig. IV. 2.17 shows the correlation diagram under use for forecasting Dowlaiswaram gauge.

When a number of tributaries affect the water level at the forecasting station, then the change in the base station on the main river as well as base stations on the tributary can be considered as additional parameters.

One such diagram (Fig. IV.2.18) has been developed for formulation of forecast at Patna (Gandhighat) on Ganga which takes into account the variation in the level at Buxar on Ganga, Darauli (on Ghaghra), Chopan (on Sone) and Rewaghat (on Gandak). Besides these, some other diagrams have also been developed for other sites. Attempts have also been made to develop mathematical models using regression equation. Some of the mathematical models are described below.

(G) **Multi-Tributary model for Dibrugarh on River Brahmaputra.**

A discrete, linear, time-invariant model has been developed for operational flood forecast of river Brahmaputra at Dibrugarh. This model is based on the difference of the gauge reading at the forecasting station and the upstream base station in the tributary. The use of differences of gauge readings as input in the model takes care of

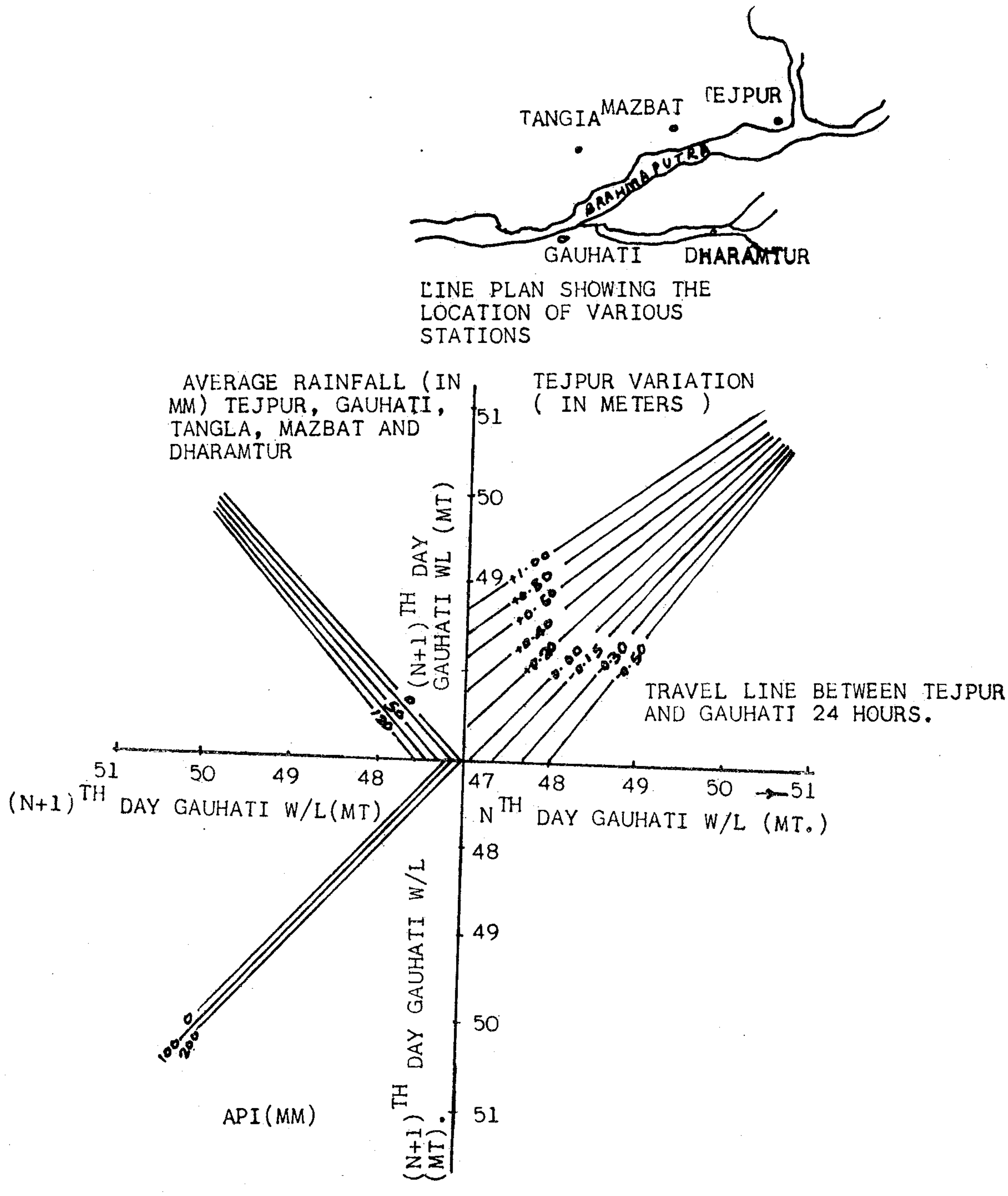

TANGIA
MAZBAT
TEJPUR
GAUHATI
DHARAMTUR
LINE PLAN SHOWING THE LOCATION OF VARIOUS STATIONS
AVERAGE RAINFALL (IN MM) TEJPUR, GAUHATI, TANGLA, MAZBAT AND DHARAMTUR
TEJPUR VARIATION (IN METERS)
(N+1)TH DAY GAUHATI WL (MT)
TRAVEL LINE BETWEEN TEJPUR AND GAUHATI 24 HOURS.
(N+1)TH DAY GAUHATI W/L(MT)
NTH DAY GAUHATI W/L (MT.)
(N+1)TH DAY GAUHATI W/L (MT).
API(MM)

FIG. IV.2.16

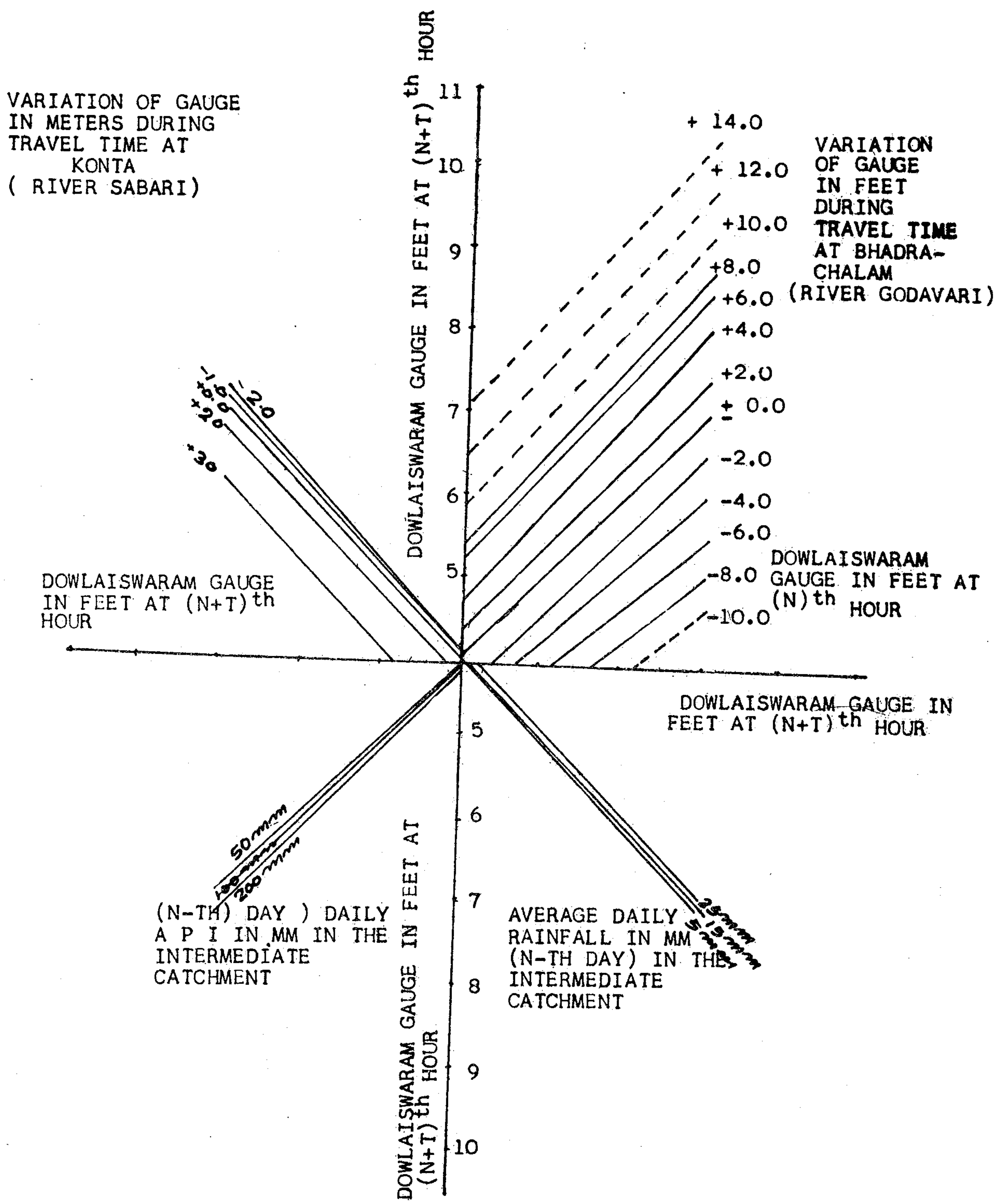
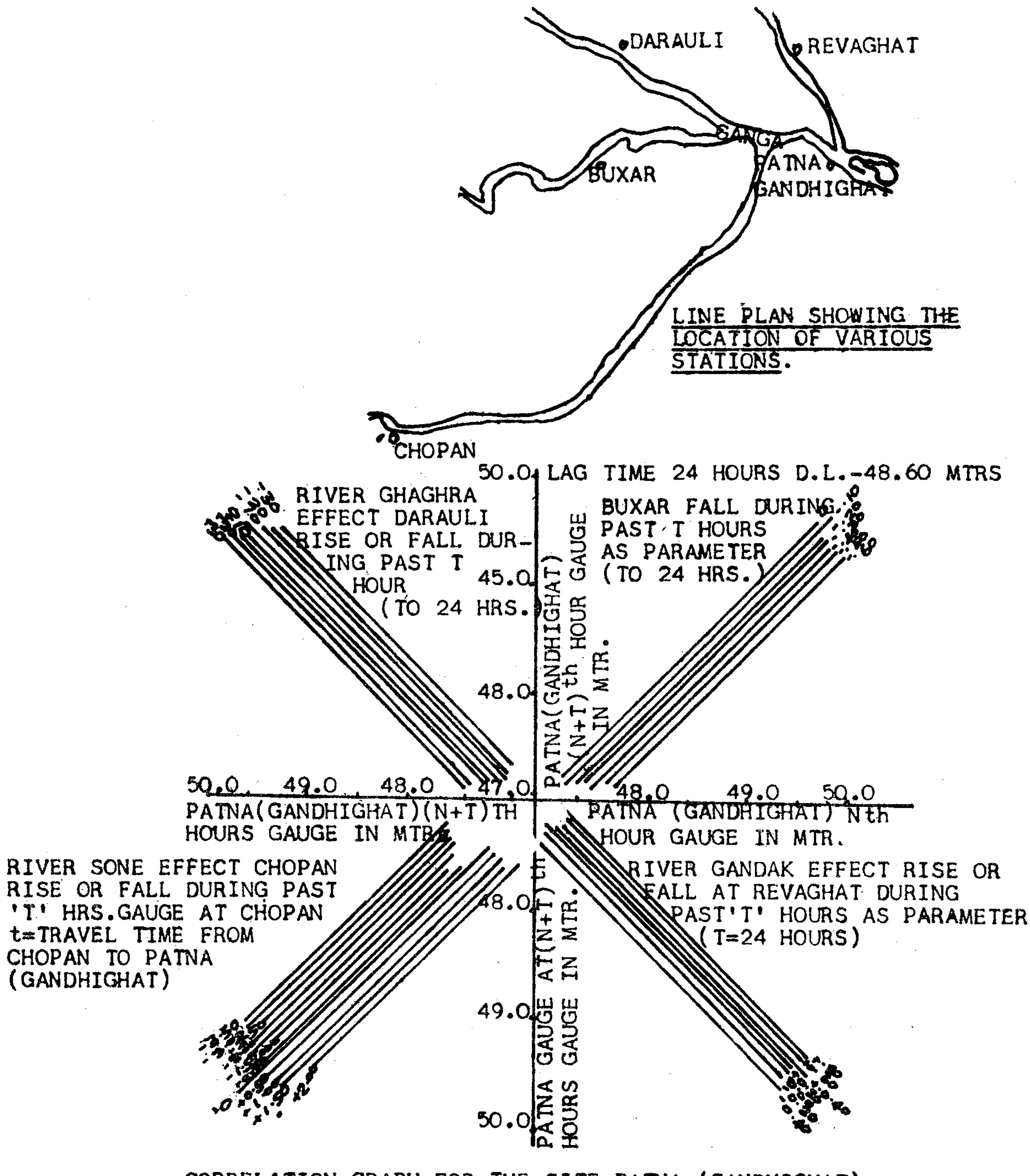

VARIATION OF GAUGE IN METERS DURING TRAVEL TIME AT KONTA (RIVER SABARI)
DOWLAISWARAM GAUGE IN FEET AT (N+T)th HOUR
11
10
9
8
7
6
5
+ 14.0
+ 12.0
+10.0
+8.0
+6.0
+4.0
+2.0
± 0.0
-2.0
-4.0
-6.0
-8.0
-10.0
VARIATION OF GAUGE IN FEET DURING TRAVEL TIME AT BHADRA-CHALAM (RIVER GODAVARI)
DOWLAISWARAM GAUGE IN FEET AT (N)th HOUR
DOWLAISWARAM GAUGE IN FEET AT (N+T)th HOUR
DOWLAISWARAM GAUGE IN FEET AT (N+T)th HOUR
2.0
3.0
50mm
100mm
200mm
(N-TH) DAY) DAILY A P I IN MM IN THE INTERMEDIATE CATCHMENT
DOWLAISWARAM GAUGE IN FEET AT (N+T)th HOUR
5
6
7
8
9
10
AVERAGE DAILY RAINFALL IN MM (N-TH DAY) IN THE INTERMEDIATE CATCHMENT
25mm
15mm
5mm

FIG. IV.2.17



CORRELATION GRAPH FOR THE SITE PATNA (GANDHIGHAT)
RIVER GANGA FALLING STAGE YEAR 1978.

FIG.IV.2.18.

the aggradation or degradation of the river bed of the tributary and the main river. The model in general is expressed as

$$g_{(i+T),\, i} = A_1\, g_{i,\,(i-T)} + \sum_{j=1}^{m} A_{2,j}\, h_{(i-T_j+T),\,(i+T_j)}$$

$$+ \sum_{j=1}^{m} A_{3,j}\, h_{(i-T_j),\,(i-T_j-T)}$$

Where

m	=	Number of tributary (=3 in this case)
T	=	Forecasting time.
T_j	=	Lag time between the forecasting stations ($T < T_j$)
$h_{(i-T_j+T),\ (i-T_j)}$	=	difference in gauges at the upstream station on the tributary between ($i-T_j+T$) and $(i-T_j)^{th}$ instant.
$h_{(i-T_j),\ (i-T_j-T)}$	=	difference in gauges at the upstream station on the tributary between ($i-T_j$) and $(i-T_j-T)^{th}$ instant.
$g_{i},\ (i-T)$	=	difference in gauge at i^{th} and $(i-T)^{th}$ instant of time at the forecasting station.
$g_{(i-T)},\ i$	=	difference in gauge at $(i-T)^{th}$ and i^{th} instant of time at the forecasting station i.e. the forecast value.

A_1,j; A_2,j; A_3,j are the parameters which are to be found out.

The parameters A_1,j, A_2,j and A_3,j can be estimated by the method of least square technique. In this case the forecast of Dibrugarh is formulated with the help of observed gauge data on three major upstream tributaries namely Dihang, Debang and Lohit.

Computer Programmes are readily available for estimating the parameters using method of least square, and subsequently upgrading the same by using method of recursive least square.

(H) Mathematical Equation for Khadda on River Gandak

The equation developed for forecasting at Khadda site on river Gandak is as follows.

$$KHD_{N+t} = 0.6533\ KHD_N + 0.13\ TRV_N + 19.2867$$

Where

KHD_{N+t} = $(N+t)^{th}$ hour gauge of Khadda in m;

KHD_N = N^{th} hour gauge of Khadda in m;

TRV_N = N^{th} hour gauge of Triveni in m; and

t = Travel time (= 7 hours)

The equation is used both for rising as well as falling stages.

(I) **Mathematical Equation for Lalbegiaghat on River Burhi Gandak.**

For Rising stages.

$$LBG_{N+t} = LBG_N + 0.796844 \quad (CHP_N - CHP_{N-t})^{0.63226}$$

$$+ 0.42856 \left(\frac{CHP_N + CHP_{N-t}}{2} - 70\right)^{0.79622} - 0.90$$

For Falling stages.

$$LBG_{N+t} = LBG_N - 0.6 \quad (CHP_{N-t} - CHP_N)$$

$$+ 0.42856 \left(\frac{CHP_N + CHP_{N-t}}{2} - 70\right)^{0.79622} - 0.90$$

Where

LBG_{N+t} = Water level at Lalbegiaghat at $(N+t)^{th}$ hour in m;

LBG_N = Water level at Lalbegiaghat at N^{th} hour in m;

CHP_N = Water level Chanpatia at N^{th} hour in m;

CHP_{N-t} = Water level at Chanpatia at $(N-t)^{th}$ hour in m; and

t = Travel time (approx. equal to 25 hours).

(J) **Forecasting Equation for Baltara on River Kosi.**

For the formulation of forecast of the site at Baltara, the change in the $(N+T)^{th}$ hour gauge and N^{th} hour gauge is correlated with the change in stage at Basua in the past

T hours and the change in water level during $(N-10)^{th}$ hour and $(N-28)^{th}$ hour at Hayaghat, a site on its tributary. The equation is as follows:

Case I : Basua and Hayaghat both Rising.

$$BAL_{N+T} = BAL_N + 0.875876 \ (\Delta BAS)^{1.123508} + 0.93761 \ (\Delta HAY)^{1.302}$$

Case II : Basua Rising and Hayaghat Falling

$$Bal_{N+T} = Bal_N + 0.875876 \ (\Delta BAS)^{1.123508} - 0.5 \ (\Delta HAY)$$

Case III : Basua Falling and Hayaghat Rising

$$BAL_{N+T} = BAL_N - 0.7 \ (\Delta BAS) + 0.93761 \ (\Delta HAY)^{1.302}$$

Case IV : Basua and Hayaghat both Falling

$$BAL_{N+T} = BAL_N - 0.7 \ (\Delta BAS) - 0.5 \ (\Delta HAY)$$

Where

BAL_{N+t} = $(N+T)^{th}$ hour gauge of Baltara in m;

BAL_N = N^{th} hour gauge of Baltara in m;

ΔBAS = Change in gauge of Basua in past T hours in m;

ΔHAY = Change in gauge of Hayaghat during $(N-10)^{th}$ hour and $(N-28)^{th}$ hour in m; and

T = Travel time from Basua to Baltara in hrs.

4.2.3 Rainfall - Stage Method

The relationship for estimating the peak discharge or the peak stage with the help of rainfall data is of great operational significance in the sense that it enables one to find the expected peak discharge or stage which is one of the important requirements in flood warning. In its simplest form, it is the relation between the average rainfall over the catchment and the peak stage. This relation may be either a graphical or mathematical and can be very easily established by using the statistical technique. The result can be further improved by incorporating other parameters such as API etc. These relations are used in many places with quite good result but the deficiency in this method is that the time of occurrence of the peak or the full shape of hydrograph cannot be forecast.

One such relation has been developed for Anandpur site on river Baitarani where the peak discharge is estimated by using the relation.

$$Q_{max} = 1.451 - 0.1698\,X + 0.0129X^2$$

where Q_{max} = peak discharge at Anandpur in lakhs of cusec.

$X = X_1 + X_2$

X_1 = Weighted storm rainfall over the catchment in cms.

X_2 = Effective Antecedent rainfall in cms.

The weighted storm rainfall over the catchment is estimated by assigning certain weights to the various stations, depending upon the area and geographical condition.

$$X = 0.2A + 0.7B + C + 0.6D$$

Where A, B, C, and D represent the rainfall at various stations in cms.

The effective antecedent rainfall is taken as certain percentage of the antecedent rains.

The following table has been assumed and used in all calculations.

Weighted antecedent rainfall in mm	*Percentage of antecedent rainfall to be taken as effective*
0 – 15	NIL
15 – 20	20%
20 – 40	25%
40 – 60	30%
60 – 80	35%
80 – 100	40%

Weighted antecedent rainfall in mm	*Percentage of antecedent rainfall to be taken as effective*
100 — 120	45%
120 — 140	50%
140 — 160	60%
160 — 180	70%
180 — 200	80%
More than 200	90%

The relation has been developed by analysing data of previous 23 storms and the results are quite satisfactory.

4.3 Commonly used Forecasting method with Rainfall as Major Input

In the methods discussed in the preceding section ie. 'Method based on statistical approach', either graphical or mathematical relationships are developed with the help of the historical data. The process of formation of floods or its propagation through the channel is not at all considered. As a matter of fact the most systematic approach for formulation of a flood forecast should be based on the application of concepts regarding machanism of formation and propagation of floods. In Indian rivers the floods are mainly caused due to heavy rainfall during monsoon period and the processes involved in formation and propagation of floods are as below:

(a) Production function (Produces run off)

(b) Distribution function (Produces hydrograph)

(c) Routing function (Routes the hydrograph)

A hydrometeorological model includes all the three functions; whereas the hydrometric model can have (b) and (c) or (b) or (c) only.

It must however, be noted that changes in the catchment characteristics due, for instance, to man's activities change these functions, and consequently their validity.

The production function helps in estimating the amount of run off likely to appear in the stream at the outlet point of the catchment from the observed point rainfalls over the catchment. For practical purpose it involves two steps :

(i) Estimation of average rainfall falling over the catchment; and

(ii) Estimation of the amount of water expected to appear in the stream at the outlet point of the catchment which may be done by establishing rainfall-runoff relations between the average rainfall over the catchment and the runoff or the co-relations considering one or several additional parameters as well.

The most commonly used technique for time distribution of the runoff is the use of unit hydrograph. The standard methods of stream flow routing e.g., Muskingum method with or without suitable modifications and Muskingum-Cunge method are generally used for routing the flood from an upstream point to further down below.

The models, where rainfall is used as imput and production and distribution functions are used to get the flood hydrograph as output, are generally known as rainfall-runoff methods. Such models are based on black box approach in which the process through which rainfall is converted into streamflow, that is, the through put is not analysed. In this case, the complete hydrograph for a particular storm condition is generated. Essentially it is an event model.

The rainfall-runoff model is very useful for flood forecasting in headwater reaches where gauge to gauge method is very difficult if not impracticable.

In a sub-basin affected by flash flood, the only effective method of flood forecasting would be through direct use of precipitations in forecasting flood stages and peaks. This may also be helpful in increasing the warning time of forecast for lower reaches of the river as the forecast values of river stage on the upstream could be used for forecasting down stream stages. Then it is ideally suited for inflow forecasting into reservoirs and lakes.

The various steps involved in this method are discussed hereunder:

4.3.1. Estimation of Mean Rainfall Over the Basin

The rainfall observation at various stations in and around the basin provide"Point Rainfall" at the respective points. But for practical purposes an average rainfall falling over the basin is required. The most commonly used method to estimate the mean or average rainfall over the basin with the help of point rainfall are:

1. Arithmetic Mean Method
2. Thiessen's Polygon Method
3. Isohyetal Method.

The three methods are discussed below in brief :

(i) Arithmetic Mean Method

It is the most simple method in which the mean of the rainfall data from various stations in the basin is taken as the average rainfall of the basin. This method gives a fairly good result when there are considerable number of raingauge stations in the basin which are located at more or less equal distance from each other.

(2) Thiessen's Polygon Method

The wights to different stations are assigned on the basis of the area of the polygon around each station which is constructed as follows:

(i) Mark the location of the several raingauge stations in the basin and a few which are outside but very near the boundary of the basin.

(ii) Join the various stations by straight line. Thus a network of triangles will be formed. Care should be taken that as far possible, each triangle should have acute angles only.

(iii) Draw perpendicular bi-sector of each line and polygons will be formed around stations. The polygons are called the Thiessen's Polygon.

The Fig. IV.3.1. shows the location of the several stations in a catchment and the Thiessen's Polygon around it. If the total catchment area is A and the areas of the polygons around station a, b, c, d, e and f are Aa, Ab, Ac, Ad, Ae and Af respectively, then the weights for respective raingauge stations will be

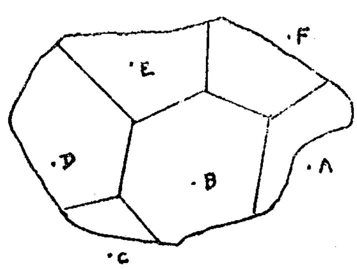


FIG.- IV.3.1.

$$\frac{Aa}{A}, \frac{Ab}{A}, \frac{Ac}{A}, \frac{Ad}{A}, \frac{Ae}{A} \text{ and } \frac{Af}{A}$$

If the point rainfall at stations a, b, c, d, e and f are Ra, Rb, Rc, Rd, Re and Rf **respectively** then the average rainfall over the basin R_{ave} will be.

$$R_{ave} = Ra\frac{Aa}{A} + Rb\frac{Ab}{A} + Rc\frac{Ac}{A} + Rd\frac{Ad}{A} + Re\frac{Ae}{A} + Rf\frac{Af}{A}$$

(3) Isohyetal Method

Isohyets are the lines of equal precipitations . The locations of the various raingauge stations are marked on the basin map and the values of the rainfall at each of the stations is labelled at respective points. Lines of equal rainfall are drawn. The topographic and oreographic condition should be kept in mind while drawing the isohyets.

When the isohyets are drawn, there are two methods of calculating the average rainfall. If the values of isohyets are denoted by P_0, P_1, P_2..... and areas between the two isohyets as A_1, A_2, A_3 ....as shown in the Fig. IV.3.2 the average rainfall can be calculated by any of the following two methods:

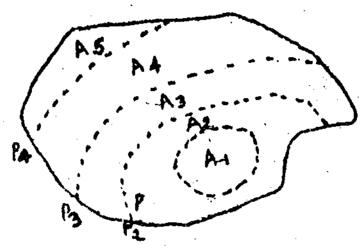


FIG - IV.3.2.

(a) $$R_{ave} = \left[\frac{P_0 + P_1}{2} A_1 + \frac{P_1 + P_2}{2} A_2 + \ldots\ldots\ldots \right] X \frac{1}{A}$$

(b) $$R_{ave} = \left[\frac{A_1}{2} P_0 + \frac{A_1 + A_2}{2} P_1 + \ldots\ldots\ldots \right] X \frac{1}{A}$$

Where A = Total area of the catchment.

An example illustrating the above three methods is given in Appendix IV (I).

Out of these three methods, Isohyetal method though supposed to be the most ideal, is not practical for operational flood forecasting purposes as it is subjective and time consuming. The Arithmetic Mean method is fast and simple but it assumes that each station is representative of surrounding portion of the catchment. The Thiessen's Polygon method takes into account the gauges distribution and is most commonly used method.

4.3.2. Estimation of the Amount of Runoff Expected to appear at the Stream

4.3.2.1. Estimation of the Amount of Runoff Using A.P.I./Base Flow/Soil Moisture as Parameter

Once the average amount of rainfall over the catchment area is known, the next step will be to estimate the rainfall excess i.e the effective rainfall which will actually contribute to Direct

Runoff at the outlet of the basin. The relationship which can be used for estimating the effective rainfall from average rainfall over the catchment is generally known as "Rainfall-runoff relationship." In its simplest form, it is a graphical relation between the observed average rainfall over the catchment (on X-axis) and the effective rainfall (on Y-axis) as shown in Fig. IV 3.3. Such relationship can be developed with the help of the past flood records. It has been found in practice that a simple relation as mentioned above never gives a good result because of the fact that antecedent basin conditions considerably affect the amount of effective rainfall for a particular value of observed average rain over the basin. Under such circumstances, it is desirable to consider one or several additional parameters representing the antecedent basin conditions and other hydrometeorological conditions. Some such cases are represented in Fig. IV.3.4., IV. 3.5. and IV 3.6.

Fig. IV 3.4. represents a case in which Antecedent precipitation Index (API) has been taken as an additional parameter. In the case of Fig. IV.3.5, the additional parameter which has been considered is the base-flow at the beginning of the storm. Such a relationship is very useful for operational purposes and it has been found that it gives good result.

The illustration of Fig. IV. 3.6. takes into account the moisture storage as additional parameter and it has been found that this gives very good results for humid regions.

The basic steps in the development of such relationships are as follows:-

1. Identification of the various floods events and collection of all relevant data such as rainfall from various stations, stage and hence the discharge at specified duration, and other basin parameters etc. corresponding to each flood event.

2. Estimation of average rainfall over the basin during this flood period with the help of available rainfall record from several stations in the catchment by using a suitable technique as already discussed earlier.

3. Plotting of the discharge data to get a flood hydrograph, judicious separation of the base flow component for each of the hydrograph and computation of total direct runoff (effective rainfall) in mms.

4. Estimation of various required parameters such as API, moisture storage etc. by using the normal techniques.

5. Plotting of the total average rainfall and effective rainfall with or without additional parameter to get a relationship. The relations can also be developed by using statistical analysis. (same as in case of development of Gauge to Gauge relationship with or without additional parameter already discussed in Section 4.2). The procedure for development of a few such relationships is explained with the help of the following examples:

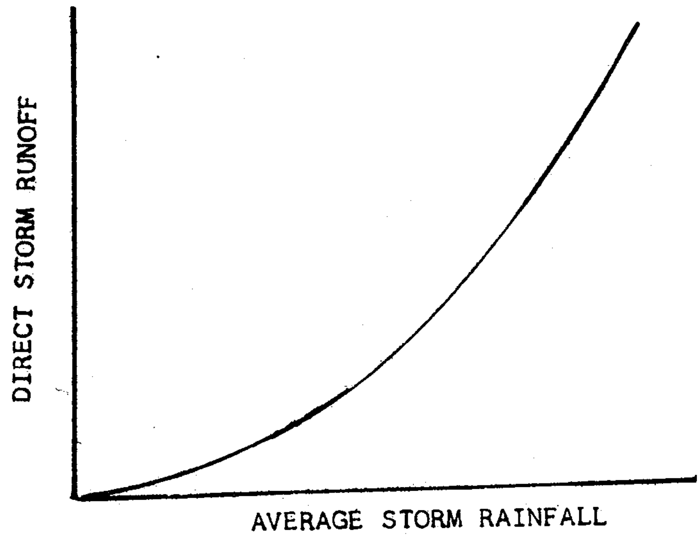


FIG. - IV.3.3

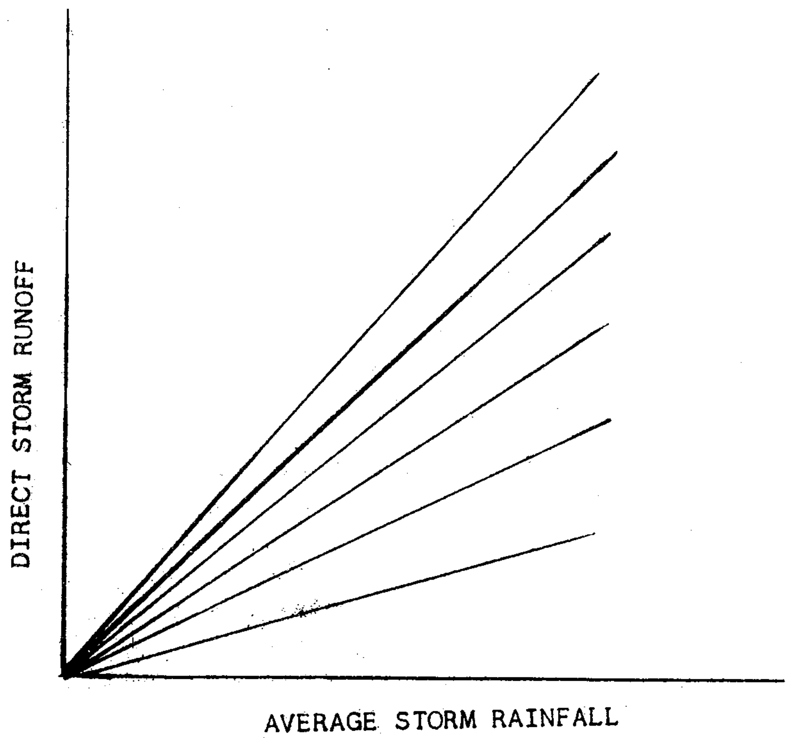
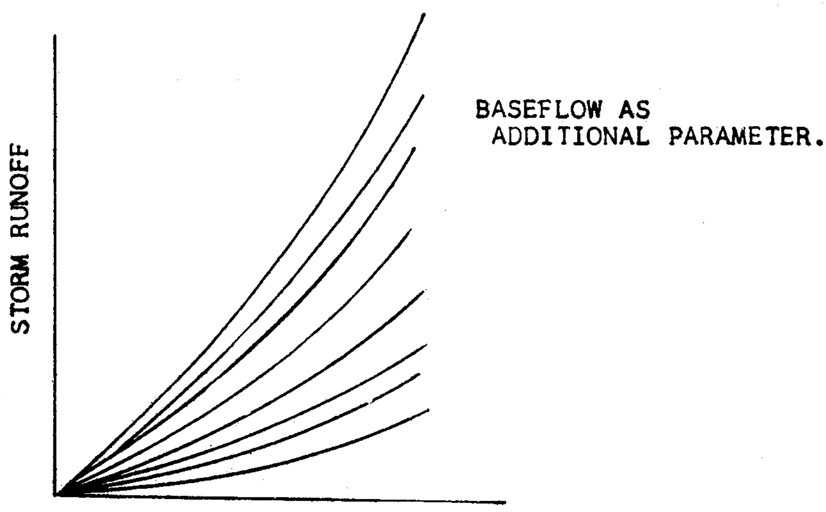
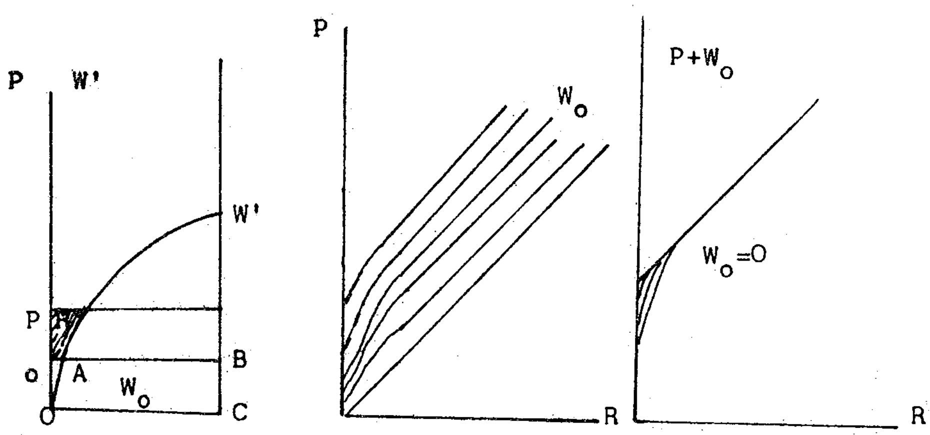


FIG. - IV.3.4



RAINFALL RUNOFF RELATION WITH BASE FLOW ADDITIONAL PARAMETER.

FIG. - IV.3.5



(A) WATERSHED STORAGE CAPACITY CURVE

(B) RAINFALL RUNOFF RELATION

(C) RAINFALL RUNOFF RELATION IN THE FORM OF P+WO+Wo-R

FIG.-IV.3.6

RAINFALL RUNOFF RELATIONS UNDER ORDINARY CONDITIONS.

WHERE P = PRECIPITATION. R = RUNOFF.

W_o = MOISTURE STORAGE OF AERATION ZONE AT BEGINING OF RAINFALL.

W_m = MOISTURE STORAGE OF AERATION ZONE AT TERMINATION OF RAINFALL.

Example IV.3.1.

The record of the discharge data at Anandpur on the river Baitarani was examined and the various flood events were identified. The corresponding discharge hydrograph was plotted, and the average rainfall falling over the basin was estimated for each event. A sample plotting of discharge hydrograph is shown in Fig. IV.3.7. The base flow was judiciously separated from each flood hydrograph and the total direct runoff in mm. was computed. (The method of separation of base flow has been explained in details in the Section on Unit hydrograph).

The total average storm rainfall over the catchment, the corresponding direct runoff and the base flow at the beginning of storm are given in Table IV 3.1. below.

Table -IV.3.1.

Storm No.	*Rainfall (mm)*	*Runoff (mm)*	*Base flow (cumecs)*
1.	133	78	560
2.	8	3	336
3.	233	89	252
4.	55	11	448
5.	14	4	560
6.	84	54	560
7.	18	5	420
8.	64	27	336
9.	28	8	280
10.	21	4	224
11.	28	10	504
12.	164	59	210
13.	88	11	70
14.	59	10	224

Storm No.	Rainfall (mm)	Runoff (mm)	Base flow (cumecs)
15.	24	3	504
16.	36	21	448
17.	88	14	392
18.	247	173	532
19.	48	6	280
20.	19	11	336
21.	21	3	280
22.	79	18	140
23.	33	7	448
24.	76	11	220
25.	67	12	336
26.	89	53	336
27.	62	28	224
28.	106	53	448

With help of the given data, develop a suitable rainfall-runoff relationship with, as well as without baseflow as a parameter.

Solution

(A) Simple Rainfall-Runoff Relation

The rainfall and the runoff data given in the problem can be used to develop the following types of rainfall runoff relations:

(i) A linear relationship by graphical approach.

(ii) A linear relationship by using statistical approach, i.e. regression analysis.

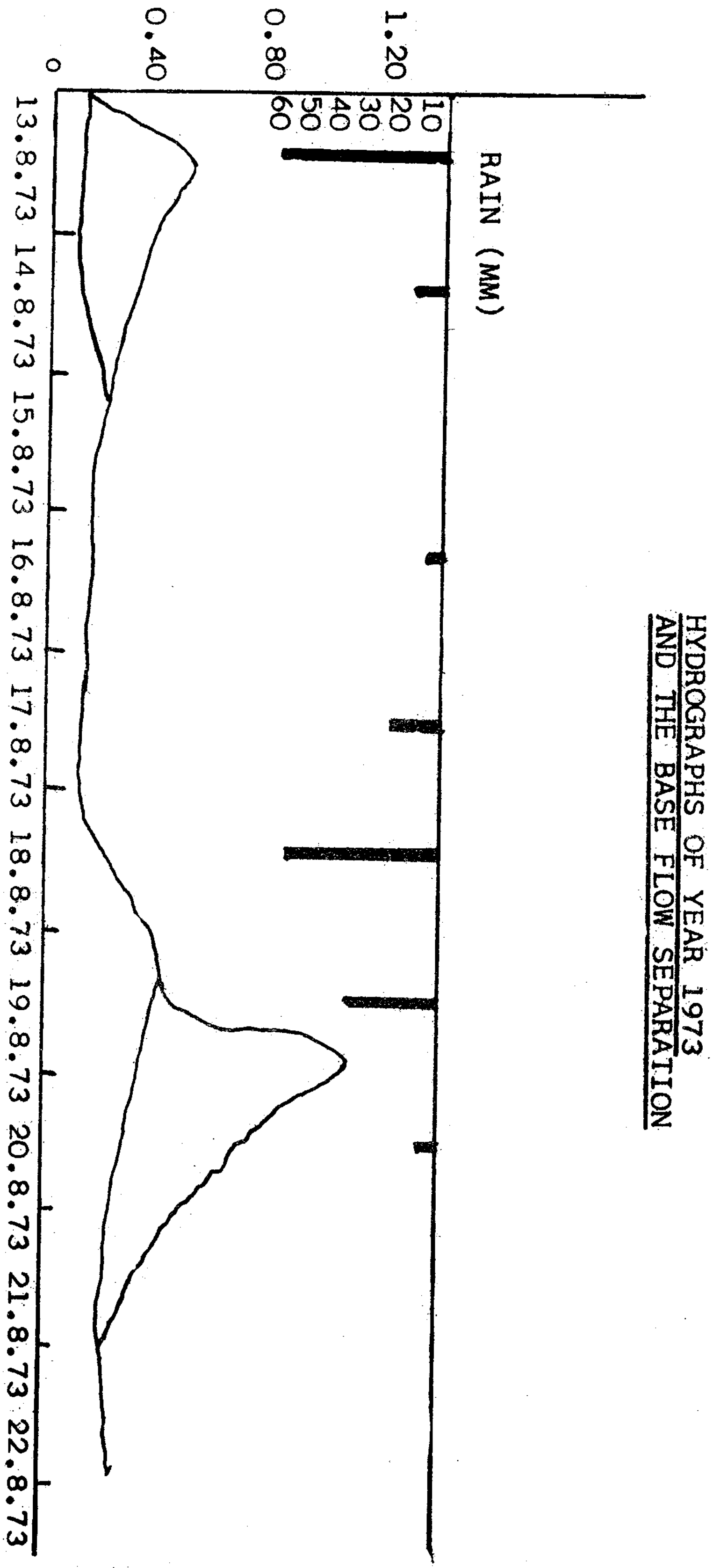

HYDROGRAPHS OF YEAR 1973
AND THE BASE FLOW SEPARATION
RAIN (MM)
10
20
30
40
50
60
1.20
0.80
0.40
0
13.8.73
14.8.73
15.8.73
16.8.73
17.8.73
18.8.73
19.8.73
20.8.73
21.8.73
22.8.73

FIG.IV.3.7

(iii) A non-linear relationship by graphical approach.

(iv) A non-linear relationship by regression analysis.

(i) A linear relationship by graphical approach

A graph is plotted with total rainfall on X-axis and the total runoff on Y-axis as shown in Fig. IV.3.8. A straight line is drawn which is supposed to pass through or very near to most of the points. This line represents the rainfall-runoff relationship.

However, it involves a personal judgement and there are chances of errors. This can be obviated by using regression analysis where the equation for a straight line is developed by the method of least squares which gives minimum value of the square of the deviation. This method is explained below:

Table IV.3.2.

Sl.No.	Rainfall in mm(X)	Runoff in mm (Y)	X^2	Y^2	XY
1.	133	78	17689	6084	10374
2.	8	3	64	9	24
3.	233	89	54289	7921	20737
4.	55	11	3025	121	605
5.	14	4	196	16	56
6.	84	54	7056	2916	4536
7.	18	5	324	25	90
8.	64	27	4096	729	1728
9.	28	8	784	64	224
10.	21	4	441	16	84
11.	28	10	784	100	280
12.	164	59	26896	3481	9676

Sl.No.	Rainfall in mm(X)	Runoff in mm (Y)	X^2	Y^2	XY
13.	88	11	7744	121	968
14.	59	10	3481	100	590
15.	24	3	576	9	72
16.	36	21	1296	441	756
17.	88	14	7744	196	1232
18.	247	174	61009	29929	42731
19.	48	6	2304	36	288
20.	19	11	361	121	209
21.	21	3	441	9	63
22.	79	18	6241	324	1422
23.	33	7	1089	494	231
24.	76	11	5776	121	836
25.	67	12	4489	144	804
26.	89	53	7921	2809	4717
27.	62	28	3844	784	1736
28.	106	53	11236	2809	5618
Total	$\Sigma X=1992$	$\Sigma Y=786$	$\Sigma X^2=241196$	$\Sigma Y^2 = 59484$	$\Sigma XY=110687$

(ii) A Linear relation by using statistical approach i.e. regression analysis.

The details of the computation are given in table IV.3.2. The step by step method of computation follows:

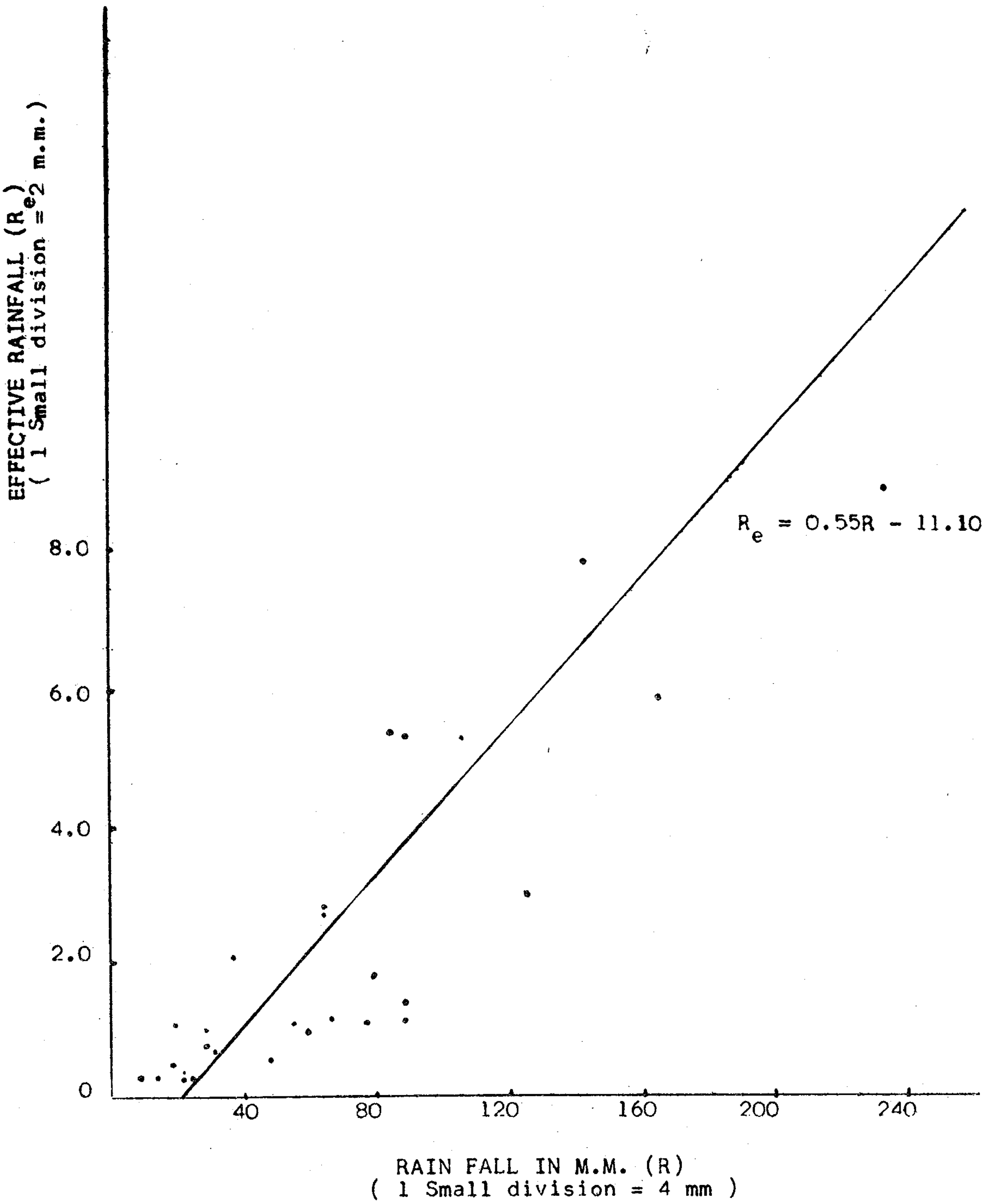

EFFECTIVE RAINFALL (R_e)
(1 Small division = 2 m.m.)
R_e = 0.55R - 11.10
8.0
6.0
4.0
2.0
0
40
80
120
160
200
240
RAIN FALL IN M.M. (R)
(1 Small division = 4 mm)

FIG. IV.3.8

Step No.1.

Note down the value of rainfall (X) under column 2 and the runoff (Y) under column 3 of the table IV.3.2.

Step No. 2.

Compute the values of X^2,Y^2 and XY for each case and write under column 4,5 and 6 respectively.

Step No. 3.

Calculate ΣX, ΣY, ΣX^2, ΣY^2 and ΣXY, which work out to be

$$X = 1992$$

$$Y = 786$$

$$X^2 = 241196$$

$$\Sigma Y^2 = 59484$$

$$\Sigma XY = 110687$$

$$n = 28$$

Step No.4.

Assuming the straight line equation to be

$$Y = aX + b$$

the normal equations are :

$$\Sigma Y = a\Sigma X + bn \quad \text{(i)}$$

$$XY = a X^2 + b\Sigma X \quad \text{(ii)}$$

Substituting the values of X, Y, XY and X^2 the normal equations will be

$$786 = 1992\,a + 28\,b \quad \text{(iii)}$$

$$110687 = 241196\,a + 1992\,b \quad \text{(iv)}$$

After solving the equation (iii) and (iv) we get the values of a and b as follows:

$$a = 0.55$$

$$b = -11.10$$

And hence the regression equation will be $Y=0.55\,X-11.10$ where Y = Runoff in mm. x = Rainfall in mm.

The corresponding line is shown in Fig. IV.3.8.

When different type of equations i.e. linear equation like $Y = aX + b$ or non-linear equation like $Y = aX^b$ are fitted, the coefficient of correlation 'r' is used as a measure of goodness of fit. The value of 'r' in this case is computed as follows:

$$r = \frac{\left[\Sigma XY - \frac{\Sigma X \Sigma Y}{n}\right]}{\sqrt{\left[\Sigma X^2 - \frac{(\Sigma X)^2}{n}\right] \times \left[\Sigma Y^2 - \frac{(\Sigma Y)^2}{n}\right]}}$$

putting the value of $\Sigma X, \Sigma Y, \Sigma X^2, \Sigma Y^2$, ΣXY and n in the above expression, we get the value of r= 0.898.

(iii) Non linear relation by graphical approach:

In this case a suitable curve is judiciously drawn instead of straight line as discussed above. In case of non-linear relationship by graphical approach one such curve which has been drawn is shown in Fig. IV.3.9.

(iv) Non-linear relation by regression analysis:

Suppose we have to fit a non-linear curve expressed by the equation.

$$Y = aX^b$$

where Y = runoff in mm

X = rainfall in mm

a and b are constants.

In order to derive such equations, take Log on both sides.

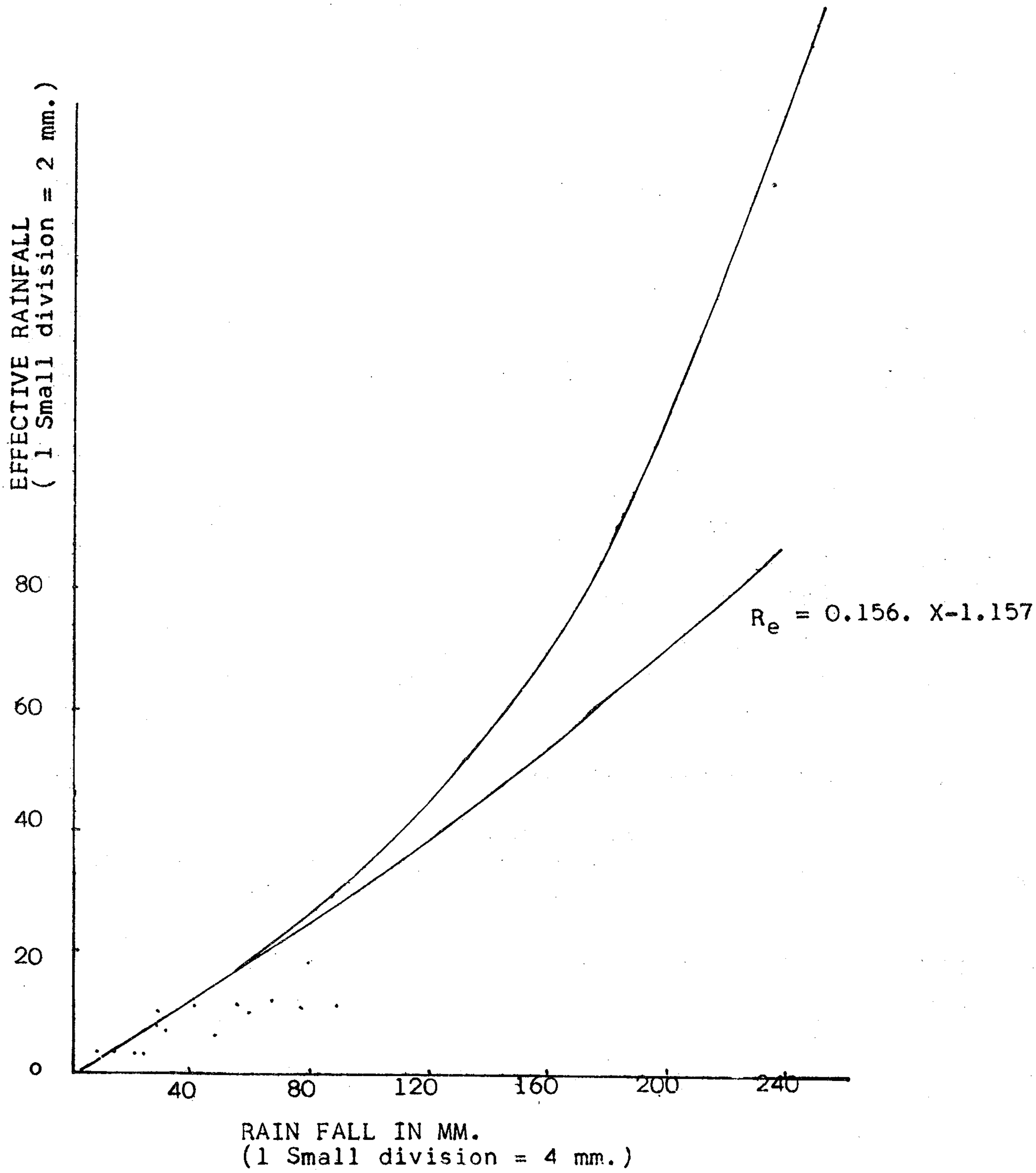

EFFECTIVE RAINFALL
(1 Small division = 2 mm.)
80
60
40
20
0
40
80
120
160
200
240
R$_e$ = 0.156. X-1.157
RAIN FALL IN MM.
(1 Small division = 4 mm.)

FIG. IV.3.9

$$\text{Log } Y = \text{Log } a + b \text{ Log } X$$
$$\text{Log } X = M$$
$$\text{Log } a = A$$
$$b = B$$
$$\text{Log } Y = N$$
$$\text{Then } N = A + B.M$$

This is again an equation of straight line which can be solved for finding A and B very easily as earlier. Knowing A and B, the values of a and b can be estimated and hence the equation $Y = a X^b$ can be developed.

Table IV.3.3

Sl. No.	*Rainfall*	*Runoff*	*Log X*	*Log Y*	*(Log X)^2*	*(Log Y)^2*	*(Log X . Log Y)*
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*
1.	133	78	2.123	1.892	4.511	3.580	4.019
2.	8	3	0.903	0.477	0.816	0.228	0.431
3.	233	89	2.367	1.949	5.604	3.800	4.615
4.	55	11	1.740	1.041	3.029	1.084	1.812
5.	14	4	1.146	0.602	1.314	0.562	0.690
6.	84	54	1.924	1.732	3.703	3.001	3.334
7.	18	5	1.255	0.699	1.576	0.489	0.877
8.	64	27	1.806	1.431	3.262	2.049	2.585
9.	28	8	1.447	0.903	2.094	0.816	1.307
10.	21	4	1.322	0.602	1.748	0.362	0.796
11.	28	10	1.447	1.000	2.094	1.000	1.447

Sl. No.	Rainfall	Runoff	Log X	Log Y	$(Log X)^2$	$(Log Y)^2$	(Log X . Log Y)
1	2	3	4	5	6	7	8
12.	164	59	2.215	1.771	4.906	3.136	3.922
13.	88	11	1.944	1.041	3.787	1.084	2.025
14.	59	10	1.771	1.000	3.136	1.000	1.771
15.	24	3	1.380	0.477	1.908	0.228	0.659
16.	36	21	1.556	1.322	2.422	1.748	2.058
17.	88	14	1.944	1.146	3.781	1.314	2.229
18.	247	174	2.393	2.241	5.725	5.020	5.361
19.	48	6	1.681	0.778	2.827	0.606	1.308
20.	19	11	1.279	1.041	1.635	1.084	1.332
21.	21	3	1.322	0.477	1.748	0.228	0.631
22.	79	18	1.898	1.255	3.601	1.576	2.382
23.	33	7	1.519	0.845	2.306	0.714	1.285
24	76	11	1.887	1.041	3.537	1.084	1.959
25.	67	12	1.826	1.079	3.335	1.165	1.971
26.	89	53	1.949	1.724	3.890	2.973	3.361
27.	62	28	1.792	1.447	3.213	2.094	2.594
28.	106	53	2.025	1.724	4.102	2.973	3.492
Total			47.859	32.742	85.510	44.799	60.250

The details of the computations are given in Table IV.3.3. The step by step method of computation is discussed below:

Step No.1.

Compute the value of Log X and Log Y for each case and write under column 4 and 5 of the Table IV.3.3.

Step No.2.

Compute the values of $(\text{Log } X)^2$, $(\text{Log } Y)^2$ and Log X. Log Y for each case and write under column 6,7 and 8 respectively.

Step No.3.

Calculate $\Sigma \text{Log } X$, $\Sigma \text{Log } Y$, $\Sigma(\text{Log } X)^2$, $\Sigma(\text{Log } Y)^2$ and $\Sigma(\text{Log } X.\ \text{Log } Y)$ which works out to be

$\Sigma \text{Log } X$	=	47.859	=	ΣM
$\Sigma \text{Log } Y$	=	32.742	=	ΣN
$\Sigma(\text{Log } X)^2$	=	85.510	=	ΣM^2
$\Sigma(\text{Log } Y)^2$	=	44.799	=	ΣN^2
$\Sigma(\text{Log } X.\ \text{Log } Y)$	=	60.250	=	$\Sigma(M.N.)$

Step No.4.

The straight line equation is

$$N = A + BM$$

The normal equations will be

$$\Sigma N = B.\Sigma M + A.n \quad \text{(i)}$$
$$\Sigma N.M = B.\Sigma M^2 + A.\Sigma M \quad \text{(ii)}$$

Substituting the values of ΣN, ΣM, ΣM^2 and $\Sigma M.N$, the normal equations will be

$$\mathbf{32.742} = 47.859\ B + 28\ A \quad \text{(iii)}$$
$$\mathbf{60.250} = 85.510\ B + 47.859\ A \quad \text{(iv)}$$

or $\quad -0.0895 = -0.0774\,B$

or $\quad B = \dfrac{0.0895}{0.0774}$

$\quad = 1.156$

Putting the value of B in the equation,

$$\Sigma N = B.\,\Sigma M + A.n$$

we get the value of A as

A = − 0.8066

or a = 0.156

Thus B = b = 1.156

Hence the equation is $Y = 0.156\,X^{1.156}$

Where X = Rainfall in mm.

Y = Runoff in mm.

The corresponding curve is shown in Fig.IV.3.9. The value of coefficient of correlation (r) in this case is computed as 0.872.

It may be seen that the co-efficient of correlation is less in case of a non-linear relationship, the value being 0.872 against that of 0.898 in case of linear equation and hence it may be concluded that in this particular case a linear equation i.e. $Y = 0.55\,X - 11.10$ is better than a non-linear equation of the form $Y = aX^b$

where Y = Runoff in mm.

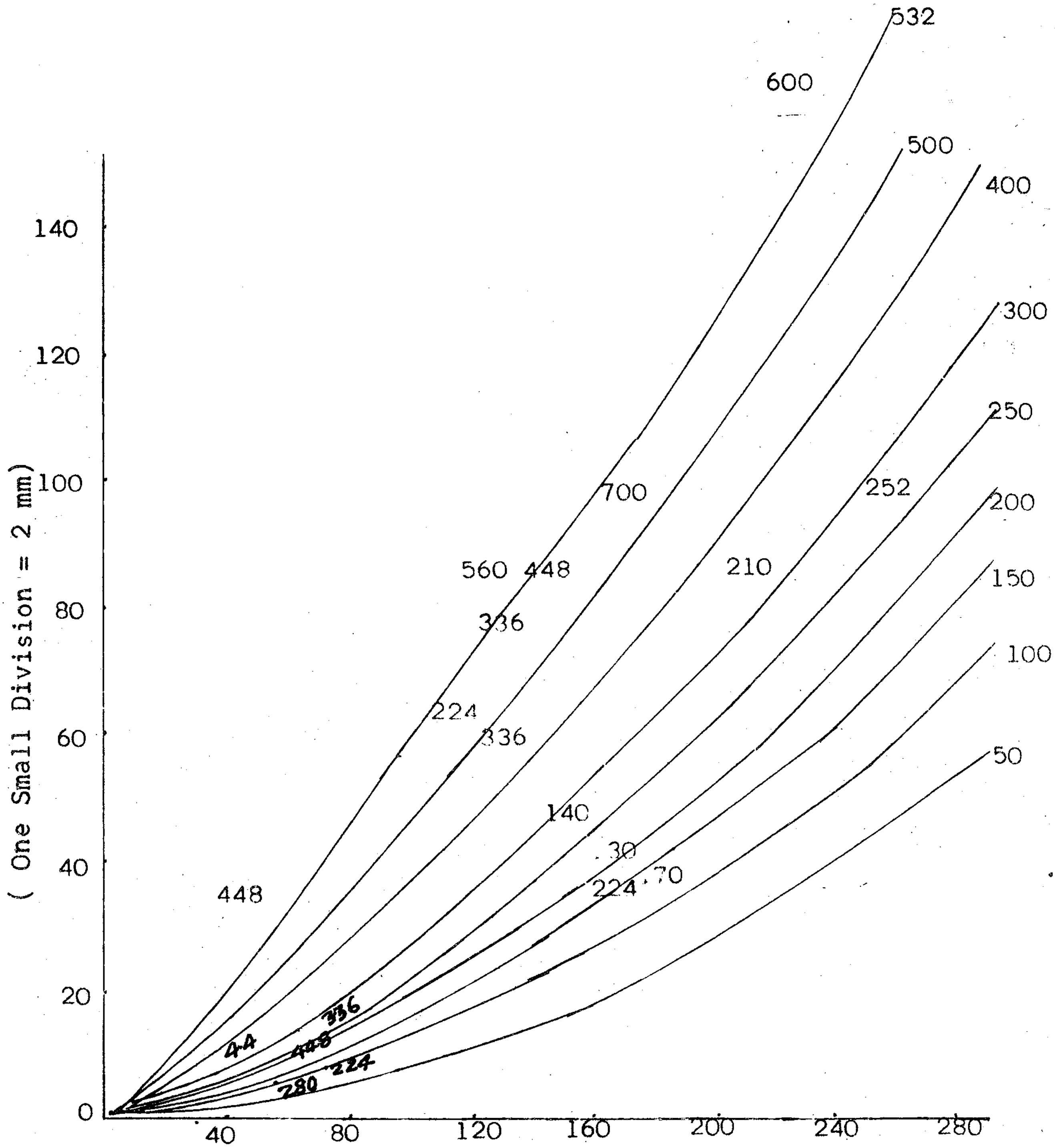
X = Average rainfall over the catchment in mm.

Rainfall - Runoff Relation with Additional Parameter

Apart from rainfall and runoff data, the values of baseflow at the beginning of the storm are also given and hence the base flow can be used as an additional parameter. The step by step procedure for the development of such relationship by graphical method is discussed below.

Step No.1.

Plot the various points corresponding to the given values of the rainfall and runoff values on a graph paper, and note down the values of corresponding baseflow near each point as shown in Fig.IV.3.10. Thus we will get a set of points on the graph paper having different values labelled against each point.



RAINFALL IN MM.
(One Small Division = 4 mm)

Fig - IV.3.10

Step No.2.

Draw a set of curves such that each curve corresponds to one particular value of baseflow. For example all the points labelled by values of 500 or near 500 are identified and a suitable curve is drawn judiciously such that this curve passes through most of the identified points. Similarly curves corresponding to base-flows of 600, 400, 300, 200, 150, 100 and 50 are drawn.

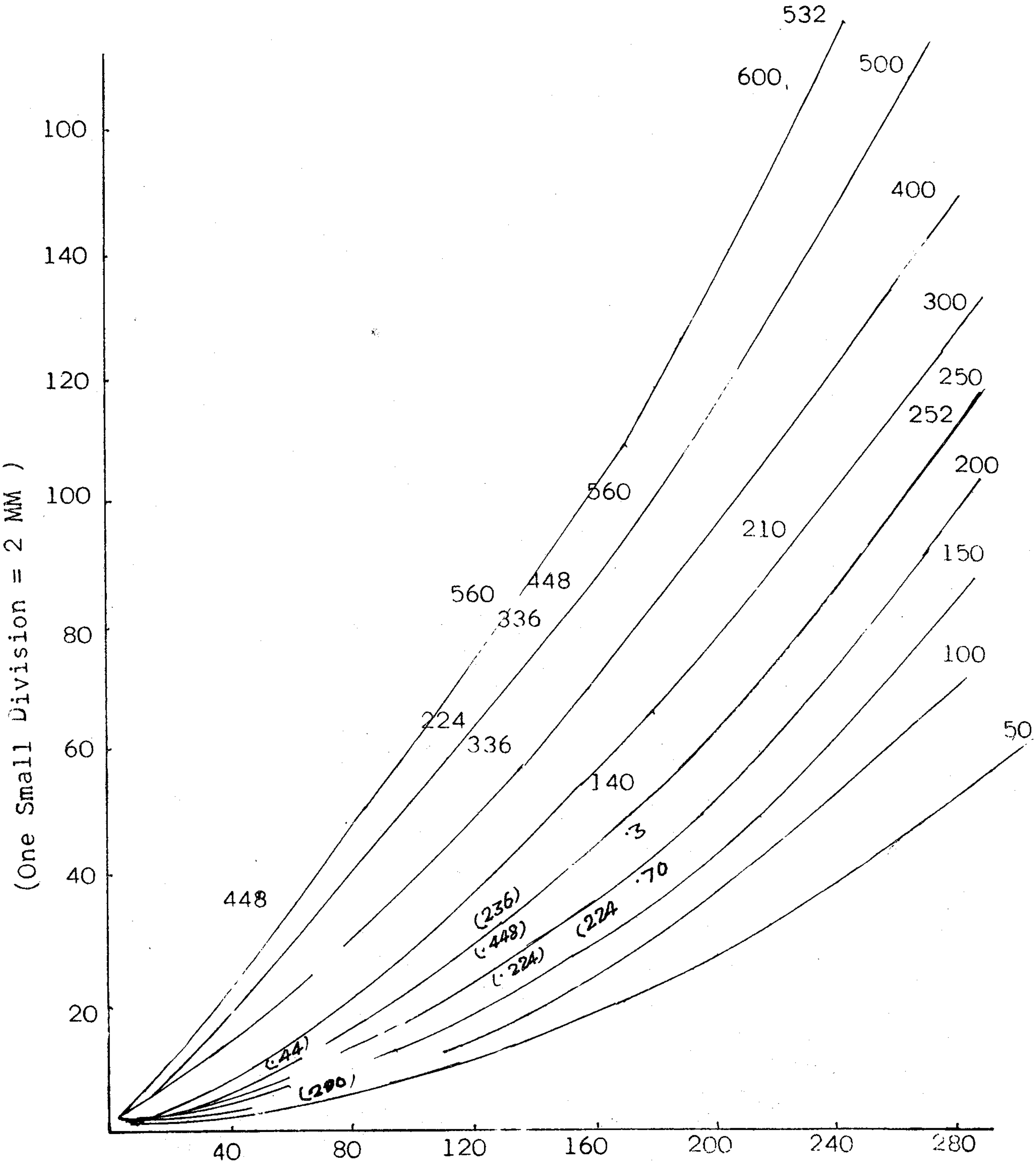
Step No.3.

Since in drawing such curves, personal judgement is involved to a greater extent, it is better to attempt other alternative as well, and to select the most suitable sets of curves. The attempts for drawing other sets of curves is not done arbitrarily. First of all we have to find magnitude and trend of errors in the already drawn curve. For this Table no.IV.3.4 is prepared. Column No.2,3 and 4 give the rainfall, runoff and base flow respectively. With the help of given value of rainfall and baseflow, the runoff is computed from the developed graph and is written under column 5. The difference between the computed runoff (column 5) and the observed runoff (column 3) is obtained which gives the error and is written under column 6 of Table IV. 3.4. The total error (considering the positive and negative signs) as well as the total absolute error are computed. In the first trial it is observed that the error works out to be -40 and the absolute error is 212. The error of -40 indicates a lower computed value. This aspect should be kept in mind while drawing another set of curves i.e. the curves should be slightly upwards so that they give higher computed values.

The set of curves drawn after another attempt is shown in Fig.IV 3.11. This gives an error of -25 and absolute error of 207.

Step No.4.

The procedure in step No.3 is attempted again and again till a set of curves is obtained which gives minimum errors. Finally accepted relation is shown in Fig.IV.3.12 which gives an error of +5.9 and absolute error of 165.



RAINFALL IN MM.
(One Small Division = 4 mm)

FIG. IV.3.11

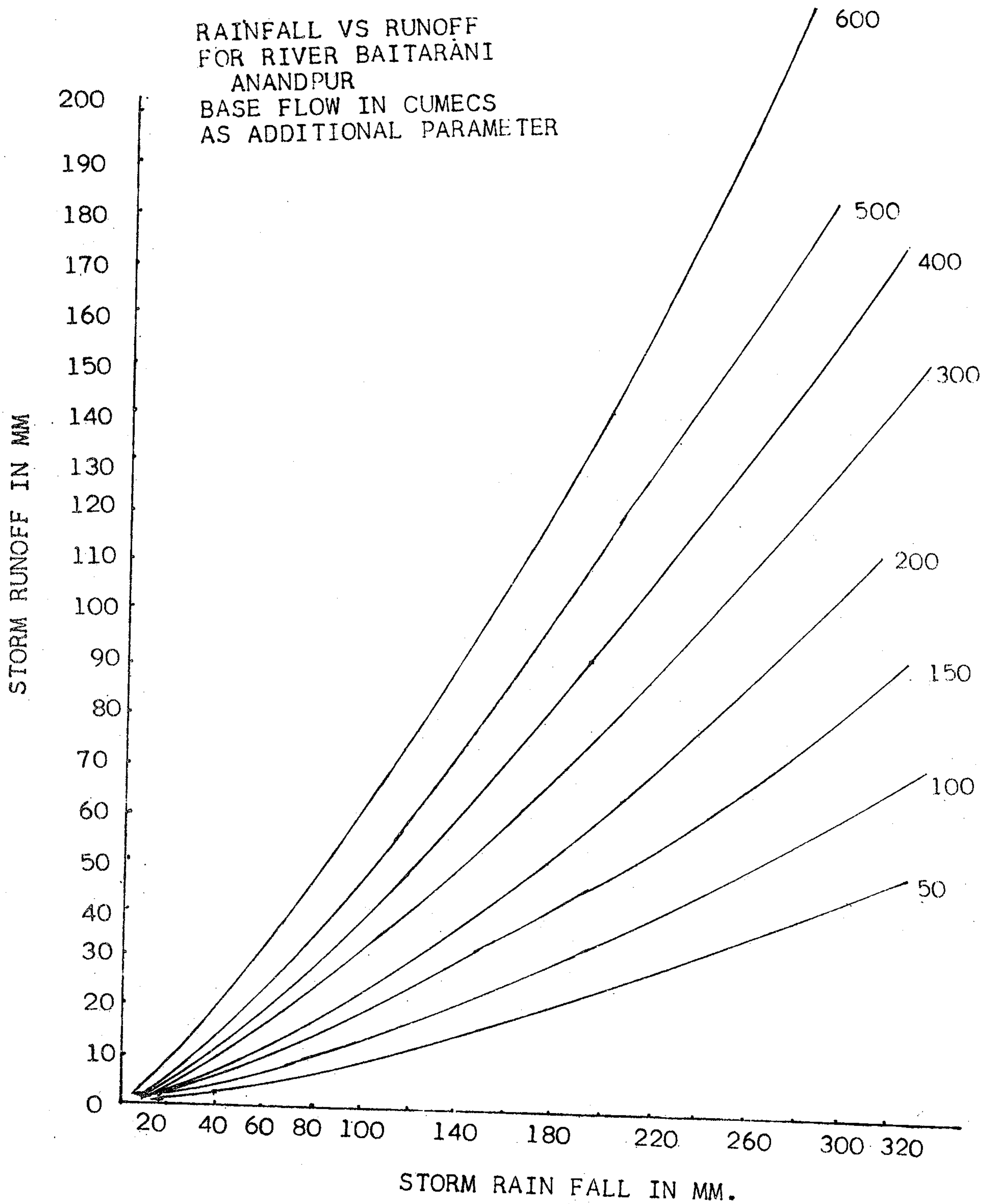

RAINFALL VS RUNOFF
FOR RIVER BAITARANI
ANANDPUR
BASE FLOW IN CUMECS
AS ADDITIONAL PARAMETER
600
500
400
300
200
150
100
50
200
190
180
170
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
STORM RUNOFF IN MM
20
40
60
80
100
140
180
220
260
300
320
STORM RAIN FALL IN MM.

Fig.IV.3.12

Table IV 3.4.

Sl. No.	Rain in mm	Observe-ed run-off(mm)	Base flow (Cum-ecs)	1st Trial Comput-ed run-off(mm)	1st Trial Error [Col] - [Col] (5) (6)	2nd Trial Comput-ed run-off(mm)	2nd Trial Error [Col] - [Col] (7) (8)
1	2	3	4	5	6	7	8
1.	133	78	560	77	– 1	78	0
2.	8	3	336	2	– 1	3	0
3.	233	89	252	82	– 7	82	– 7
4.	55	11	448	28	+17	28	+17
5.	14	4	560	10	+6	10	+6
6.	8	54	560	48	– 6	48	– 6
7.	18	5	420	7	+2	6	+ 1
8.	64	27	336	18	–9	19	– 8
9.	28	8	280	6	– 2	6	– 2
10.	21	4	224	4	0	4	0
11.	28	10	504	14	+ 4	14	+ 4
12.	164	59	210	40	– 19	10	– 19
13	88	11	70	8	– 3	9	– 2
14.	59	10	224	10	0	11	+ 1
15.	24	3	504	12	+ 9	12	+ 9
16.	36	21	448	18	– 3	18	– 3
17.	88	14	392	33	+ 9	31	+17
18.	247	173	532	160	– 13	160	– 13
19.	48	6	280	11	+5	12	+ 6
20.	19	11	336	6	– 6	6	– 5

Sl. No.	Rain in mm	Observed run-off(mm)	Base flow (Cumecs)	1st Trial Computed run-off(mm)	1st Trial Error [Col] - [Col] (5) (6)	2nd Trial Computed run-off(mm)	2nd Trial Error [Col] - [Col] (7) (8)
1	2	3	4	5	6	7	8
21.	21	3	280	5	+ 2	5	+ 2
22.	79	18	140	10	− 8	10	− 8
23.	33	7	448	17	+ 10	17	+ 10
24.	76	11	224	15	+ 4	14	+ 3
25.	67	12	336	20	+10	20	+ 8
26.	89	53	336	26	− 27	27	− 26
27.	62	28	224	12	− 16	11	− 17
28.	106	53	448	48	− 5	46	− 7
				Total error	− 40		− 25
				Absolute total error	212		207

4.3.2.2 **Estimation of the Amount of Runoff Using Antecedent Precipitation Index, Week Number and Storm Duration as Parameter:**

The amount of runoff resulting from a given rainfall in a given basin depends upon many factors such as vegetal cover, soil characteristics, initial moisture deficiencies, and storm characteristics such as areal distribution and intensity of the storm. In order to incorporate these factors five variables, i.e. storm runoff, antecedent precipitation index, season or week of the year, storm duration and storm rainfall are selected. The relationship, thus incorporating the effect of these variables can be represented either by a coaxial diagram or by mathematical equations. Determination of the various parameters, are explained below:

The antecedent precipitation index is expressed by the equation:

$$I = b_1P_1 + b_2P_2 + b_3P_3 + \dots\dots\dots\dots + b_ip_i + \dots\dots\dots\dots$$

where b_i is a constant and p_i is the basin precipitation which occurred i days before the storm under consideration. Such an equation is inconvenient for day-to-day use in flood forecasting. A more stable form of this equation results if it is assumed that 'b' decreases with time before the storm being considered according to the logarithmic recession. During time of no precipitation.

$$I_t = I_o K_t$$

where I_o is the initial value of the Antecedent Precipitation Index (API). I_t is the reduced value t days later, and K is a recession factor. Letting 't' equal unity.

$$I_1 = KI_o$$

The API for any day is equal to that of the day before multiplied by a factor K. When rain occurs on any day, the amount is added to the index and API is given by the equation,

$$I_1 = KI_o + P$$

where I_1 is today's index, I_0 is yesterday's index and P is rain occurring during the intervening 24 hours period.

The value of 'K' should be function of physiographic, climatic and vegetative characteristics of the basin and the actual evapotranspiration but normally it is assumed to be constant, somewhere between 0.85 and 0.95, with 0.90 as the most commonly used value. To evaluate the API, for any storm the computations are to begin from a day usually 25-30 days prior to the occurrence of the storm.

The details of API computation is illustrated in the following example.

Example IV.3.2

The following table shows the precipitation over a catchment (Fig.IV.3.13) on 14 consecutive days. Using a starting value of 14.2 mm for the API on day 1, API is to be computed on days 2 to 14.

Figure IV.3.13

Date	*24 hours precipitation (mm)*	*Date*	*24 hours precipitation (mm)*
1.	0.0	8.	0.0
2.	0.0	9.	2.5

Date	*24 hours precipitation (mm)*	*Date*	*24 hours precipitation (mm)*
3.	34.5	10.	0.0
4.	63.0	11.	2.5
5.	6.4	12.	16.0
6.	0.0	13.	39.6
7.	0.0	14.	2.5

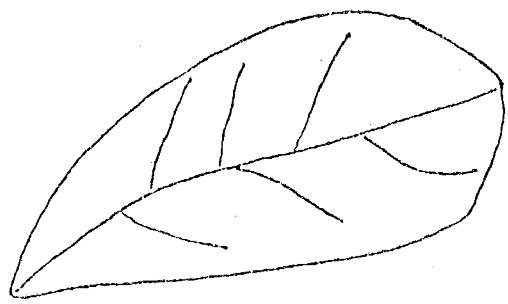


FIG. IV.3.13

Solution:

Antecedent Precipitation Index computation is carried out in standard Tabular Format as illustrated in Table IV.3.5.

In the above computation, each value of API was rounded to the nearest 0.1 mm before computing the next day API.

Storm duration can be defined for short, uniform storm, but becomes quite complicated for long, draw-out storms. One approach, when six hourly precipitation amounts are available currently is to take the sum of all six-hourly periods with 0.20" or more precipitation plus half the sum of intervening period with less than 0.20". This approach assumes that when rainfall of 0.20" or more occurs in six hours, the effective duration is six hours; when less than 0.20" occurs, the effective duration is 3 hours.

Table IV.3.5

Computation of Antecedent Precipitation Index

Month -------- Year --------	1	2	3	4	5	6	7	8	9	10	11	12	13	14
1. Previous day's APIXO. 9.	14.2	12.8	11.5	41.4	94.0	90.4	81.4	73.3	66.0	61.6	55.4	52.1	61.3	90.8
2. Average basin Precipitation	0	0	34.5	63.0	6.4	0	0	0	2.5	0	2.5	16.0	39.6	2.5
3. API = (1) + (2)	14.2	12.8	46.0	104 4	100.4	90.4	81.4	73.3	68.5	61.6	57.9	68.1	100.9	93.3

A convenient means of defining season is the use of the week of the year in which storm begins.

The week number varies from 1 (Jan. 1-7) to 52 (Dec.24-31).

Co-Axial Representation

Fig.IV.3.14. shows the graphical correlation with storm runoff with week number as intermediate variable. In chart B, computed value of storm runoff from chart A is correlated with storm duration to obtain a new computed value of storm runoff. This new value is further modified. In chart C, by correlation with storm precipitation the best computation of storm runoff can be forecast with the five variables. Finally in chart D, the computed values versus the observed ones are plotted to show the reliability of the procedure.

If use of the five veriables could make perfect forecast, all storms would plot on the 45° line in Chart 'D'.

Derivation of Initial Curves

It is helpful to tabulate data of all the storms before starting the analysis. The computed API for the day prior to each storm is used. Week number and duration values are found in the following manner:

Graphical correlation can be started as soon as the tabulation is completed. Scales, such as used in Fig.IV.3.14. should be entered on a blank sheet of graph paper for each of the four quadrants A,B,C and D. As a first approximation, chart B can be drawn with parallel 45° lines spaced about 0.25 inches per 24 hours of duration. After the duration quadrant is complete the development of the rainfall-runoff relation is carried out in following manner:

(i) For each storm, the point in chart 'C', defined by the storms rainfall and runoff is plotted;

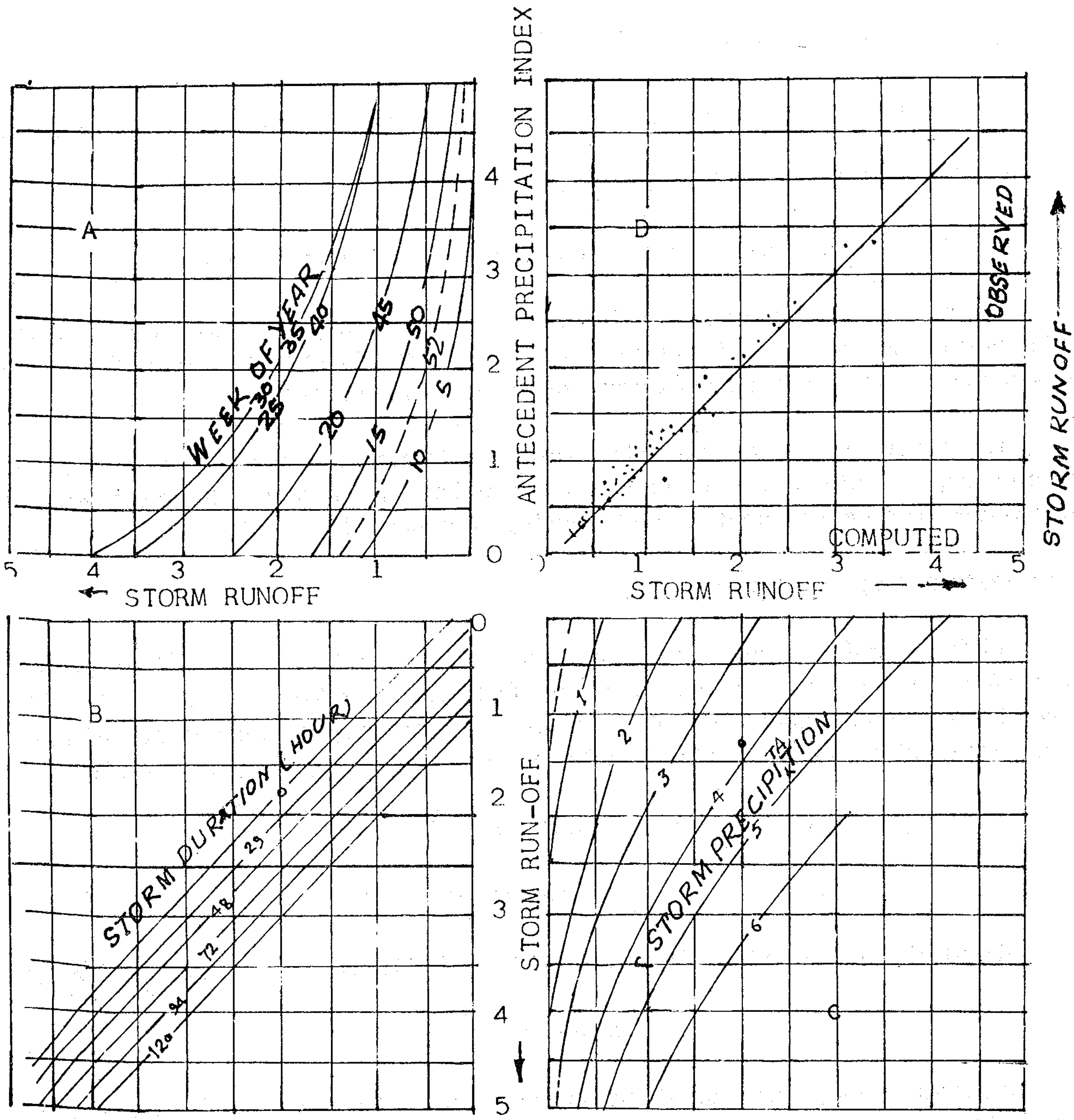
(ii) This runoff value is traced back through chart B to chart A. A point on chart A corresponding to that runoff value and API value for pertinent storm is plotted . The point is labelled with week number.

(iii) Steps (i) and (ii) are repeated for each storm.

(iv) Lines that best fit the plotted data in Chart A are drawn.

Refinement by Successive Approximation

Chart 'C' can now be refined by using the newly drawn lines on chart 'A' while performing the following steps:



DEVELOPMENT OF RUNOFF RELATION

FIG- IV.3.14

(i) Enter 'A' with API and week number.

(ii) Proceed through 'B' with duration.

(iii) The runoff is computed and a new point in chart 'C' with rainfall label is plotted.

(iv) Steps (i) through (iii) are repeated for all storms.

(v) The lines in chart 'C' are adjusted to fit the newly plotted data.

Chart 'B' spacing can be refined at any time by entering the chart sequence from both ends, plotting new points and drawing new lines. Chart 'A' can now be refined by using the revised chart 'C' to repeat the steps first used in preparing chart 'A'.

This whole cut-and-dry procedure can be reiterated as often as is required to define accurate relationship. Chart 'D' should be completed after each refinement so that the improvement can be evaluated. The relationship is presented in Fig.IV.3.15.

Mathematical Representation

Surface runoff can be expressed as function of precipitation, Antecedent Precipitation Index, Week No. and Duration i.e. Surface runoff = f (P, API, WK No., DUR). API and WK. No. are known before the commencement of the storm and represent the antecedent condition. The antecedent condition is denoted by AI and is expressed as a function of API and WK.No.

$$AI = f\ (API, WK.No.)$$

$$SRO = f\ (P, AI, DUR)$$

AI and DUR is grouped together and Final Index (FI) is expressed in terms of AI and DUR.;

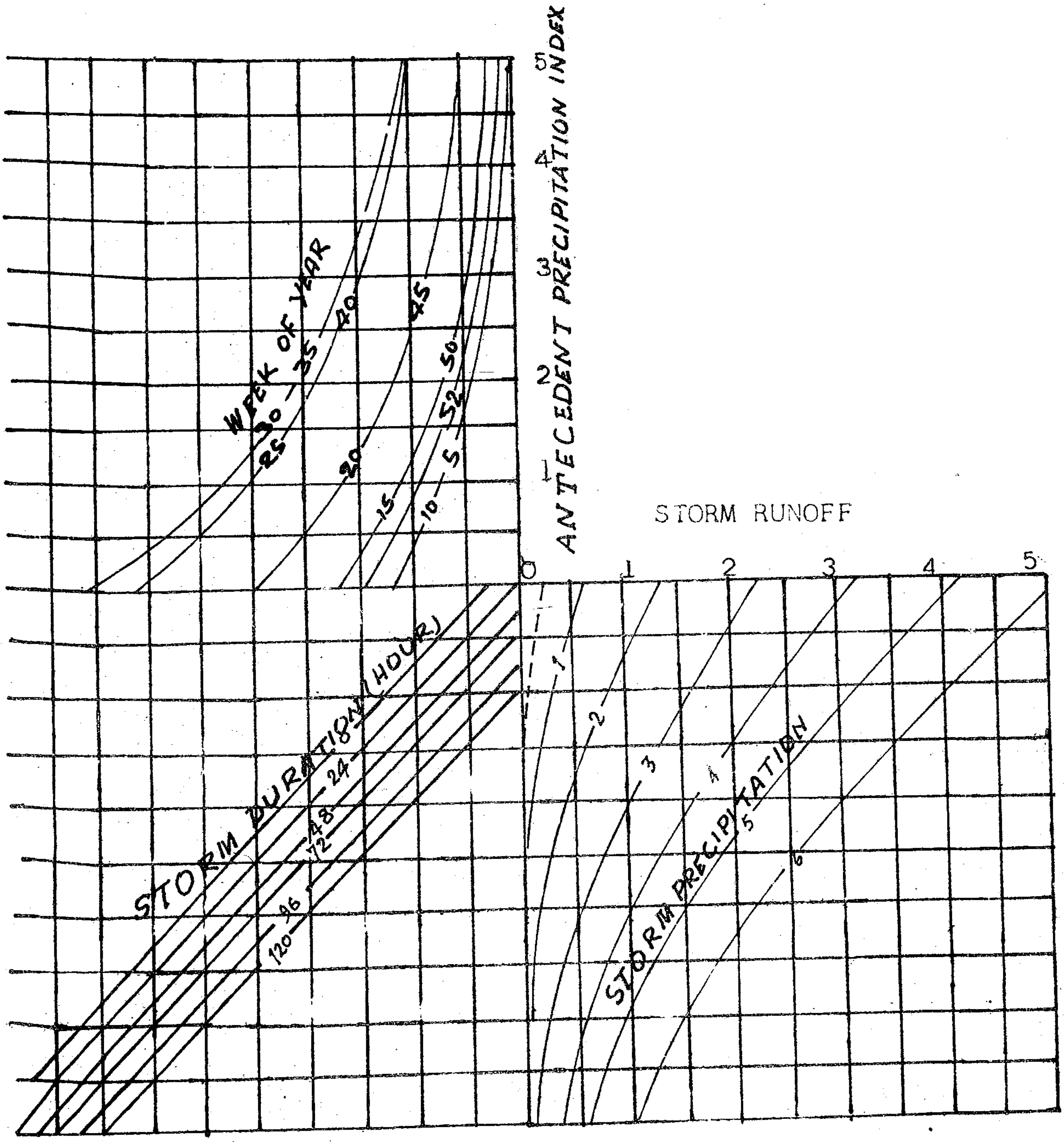
$$\text{i.e. } SRO = f\ (P, FI)$$

In the succeeding sections, attempts will be made to represent the functions mentioned above in the form of mathematical equations.

Season Quadrant:

Season Quadrant is defined by seven parameters. AWK,BWK,ASES,BSES,AX,BX,EX.

Where, AWK = Week represented by wet curve.
BWK = week represented by dry curve



TYPICAL OPERATION RUNOFF RELATION

FIG. IV.3.15

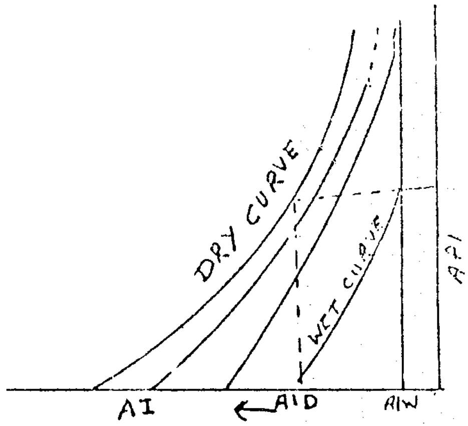


FIG. IV.3.16

ASES = the intercept on the 'AI' axis of the wet curve

BSES = intercept on the AI axis of the dry curve

NWKS = No.Of WKS between the storm and AWK

NWKT = No.Of WKS between AWK and BWK

FRAC = (NWKS/NWKT)

$$FR = \left[\frac{1 + \sin\pi(1.5 - FRAC)}{2}\right]^{EX}$$

EX = Exponent close to 1.0

AI = AIW + FR (AID - AIW)

if AIW > AID then AI = AIW

Duration Quadrant : (Total Storm Relationship)

FI = AI + 0.01 DUR (English Unit)

FI = AI + 0.254 DUR (Metric Unit)

Where DUR is storm duration in hours.

Precipitation Quadrant:

This quadrant is defined by five parameters PA, PB, PC, PD, PE.

$$PF = PA + PB\ (FI)$$

$$PG = PC + PD\ (FI)^{PE}$$

$$RO = [P^{PF} + PG^{PF}]^{1/PF} - PG.$$

Values of these parameters are obtained from the historical record.

The above relations hold good for English Units. The following conversions are required in case metric units are used.

For English Units the input and output variables, API, RI, P and RO are expressed in inches. For metric units, these variables are used in mm.

Of the 13 parameters which define the relationship, 5 have the same numerical values for use with either English or metric units. These are:

AWK, BWK, EX, PA and PE

The remaining 8 may be converted from English to the metric equivalent by use of the following equations:

$$ASES(M) = ASES\ (25.4)$$

$$BSES(M) = BSES\ (25.4)$$

$$AX(M) = AX^{0.03937}$$

$$BX(M) = BX^{0.03937}$$

$$RA(M) = RA^{0.03937}$$

$$PB(M) = PB^{0.03937}$$

$$PC(M)' = PC\ (25.4)$$

$$PD(M) = PD\ [25.4^{(1-PE)}]$$

Example IV.3.3

An API type rainfall-runoff relationship has been developed for a catchment. The parameters for the season quadrant are:

AWK	=	27,	ASES	=	6.30,	AX	=	0.84,	EX	=	1.03
BWK	=	45,	BSES	=	14.15,	BX	=	0.71			

A storm occurs on August 20 (Week No.34). The API just before the storm is 0.92. Compute the Antecedent Index (AI).

The storm lasts for six hours. What is the Final Index (FI).

The parameters for the precipitation quadrant are:

PA = 1.19	PC = 0.10	PE = 1.12
PB = 0.04	PD = 5.90	

If the precipitation value is 2.63", what volume of surface runoff will occur?

Solution:

AWK = 27, BWK = 45

Storm occurs on August 20 i.e. WK No. = 34

NWKS = (34-27) = 7

NWKT = (45-27) = 18

$$\text{FRAC} = \frac{\text{NWKS}}{\text{NWKT}} = 7/18 = 0.38889$$

$$\text{AIW} = (\text{ASES})\,.\,(\text{AX})^{\text{API}} = 5.3663$$

$$\text{AID} = (\text{BSES})\,.\,(\text{BX})^{\text{API}} = 10.3256$$

$$\text{FR} = \left[\frac{1+\text{Sin}\{\pi(1.5-\text{FRAC})\}}{2}\right]^{\text{EX}}$$

$$= \left[\frac{1+\text{Sin}\{\pi(1.5-0.38889)\}}{2}\right]^{1.03}$$

$= 0.3182$

$$\begin{aligned}
AI &= AIW + FR\,(AID - AIW) \\
&= 5.3663 + 0.3182\,(10.3256 - 5.3663) \\
&= 6.9443
\end{aligned}$$

$$\begin{aligned}
FI &= AI + 0.01\,DUR \\
&= 6.9443 + (0.01)\,.\,(6) \\
&= 7.0043
\end{aligned}$$

$$\begin{aligned}
PF &= PA + PB\,(FI) = 1.19 + (0.04)\,(7.0043) \\
&= 1.470
\end{aligned}$$

$$\begin{aligned}
PG &= PC + PD\,(FI)^{PE} \\
&= 0.10 + (5.90)\,.\,(7.0043)^{1.12} \\
&= 52.30
\end{aligned}$$

$$\begin{aligned}
RO &= [P^{PF} + PG^{PF}]^{1/PF} - PG \\
&= [2.63^{1.470} + 52.30^{1.47}]^{1/1.470} - 52.30 \\
&= 0.44'' = 11.18\ \text{mm} \\
&= 11.20\ \text{mm}
\end{aligned}$$

4.3.3 Unit Hydrograph

The Unit Hydrograph, also commonly known as Unit graph is the most simple but at the same time a very powerful tool for hydrological analysis in general and flood forecasting in particular.

The unit hydrograph may be defined as the direct runoff (outflow) hydrograph resulting from one unit of effective rainfall which is uniformly distributed over the basin at a uniform rate during a specified period of time known as unit time or unit duration.

The unit quantity of effective rainfall is taken as 1 mm and the outflow hydrograph is expressed by discharge in cumecs. The unit duration may be 1 hour, 2 hours, 3 hours, 6 hours, 12 hours or so, depending upon the size of the catchment, storm characteristics and operational facilities. However, the unit duration cannot be more than the time of concentration or basin lag or period of rise.

From the above definition of unit hydrograph it is evident that the two basic conditions are:

1. Effective rainfall should be uniformly distributed over the basin, i.e. if there are five raingauges in the basin which represent the areal distribution of rainfall over the basin, then all the five raingauges should record almost same amount of rainfall during specified time.

2. Effective rainfall should be at a uniform rate during the unit duration. If the average rainfall over a particular basin during 6 hours is 126 mm, then a unit hydrograph of 6 hours duration can be derived only if the intensity of rainfall is 21 mm/hour throughout as shown in Fig IV.3.17.

If the same amount of rainfall is distributed as in Fig. IV 3.18 the unit hydrograph cannot be precisely estimated by simple method.

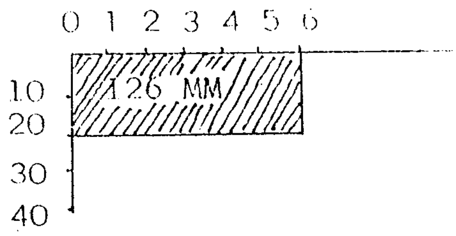


Fig. IV.3.17

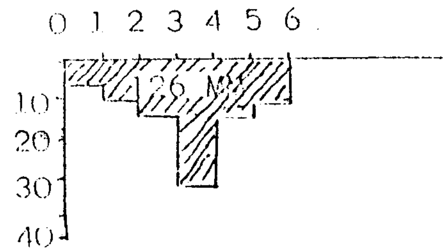


Fig. IV.3.18

4.3.3.1 Assumption in Unit Hydrograph Theory

The following are the basic assumptions in the unit hydrograph theory:

1. The unit hydrograph theory assumes the principle of time invariances. This assumption means that the direct runoff hydrograph from a given drainage basin due to a given pattern of effective rainfall will be always same irrespective of the time, i.e. although the basin characteristics change with season etc., the unit hydrograph remains same.

2. Unit hydrograph theory assumes the principle of linearity, superimposition or proportionality. It means that:

(i) If the ordinates of a unit hydrograph of say 1 hour duration are 0,1,6,4,3,2,1,0 units respectively then the effective rainfall of 2 units falling in 1 hour will produce a direct runoff hydrographs having ordinates of 0, 2, 12, 8, 6, 4 , 2, 0, units as shown in Fig.IV.3.19.

(ii) If the effective rainfall of 2 units occurs in 2 hours i.e. 1 unit per hour then the direct runoff hydrograph ordinates will be obtained by summing up the corresponding ordinates of the two unit hydrograph A and B as shown in the Fig. IV.3.20.

4.3.3.2 Derivation of Unit Hydrograph

The unit hydrograph is best derived from the observed hydrograph resulting from a storm which fulfils the two basic conditions, i.e. the rainfall is more or less uniformly distributed over

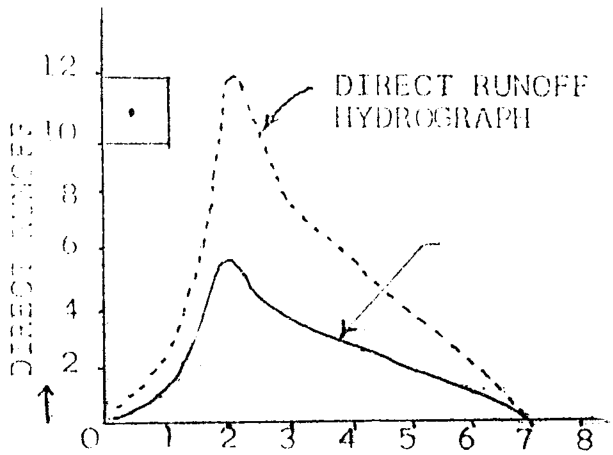


Fig. IV.3.19

the basin and has a reasonably uniform intensity. Such a hydrograph will generally form a single and sharp peak.

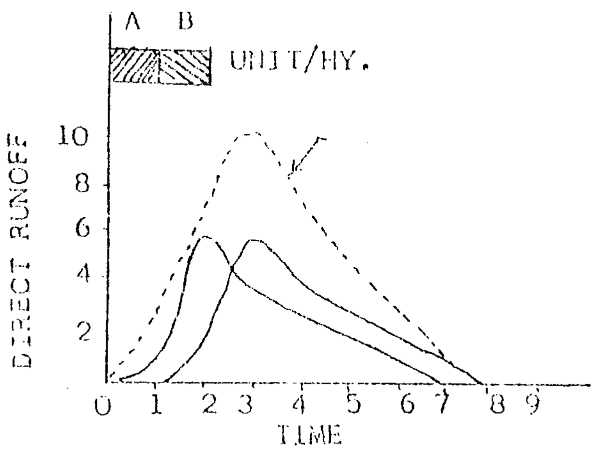


Fig. IV.3.20

The various steps involved in derivation of unit hydrograph are:

1. Separation of base flow and calculation of direct runoff hydrograph ordinates by deducting baseflow ordinates from the corresponding observed flood hydrograph ordinates.

2. Determination of the volume of direct runoff.

Now, the ordinates of the direct runoff hydrograph are divided by observed runoff depth. The resulting ordinates form a unit hydrograph.

It is desirable that the amount of direct runoff of the observed hydrograph is near or greater than 10 mm.

The unit duration of the unit hydrograph will be the effective rainfall duration.

An example illustrating the method of derivation of unit hydrograph from an isolated flood hydrograph is given in Appendix IV (2).

4.3.3.3 Unit Hydrograph from Complex Events

The above mentioned method of derivation of unit hydrograph is valid only when the two basic conditions for unit hydrograph are satisfied. But in actual practice these conditions are rarely satisfied. In most of the cases the rainfall varies with time resulting in a complex event as illustrated in Fig. IV.3.21.

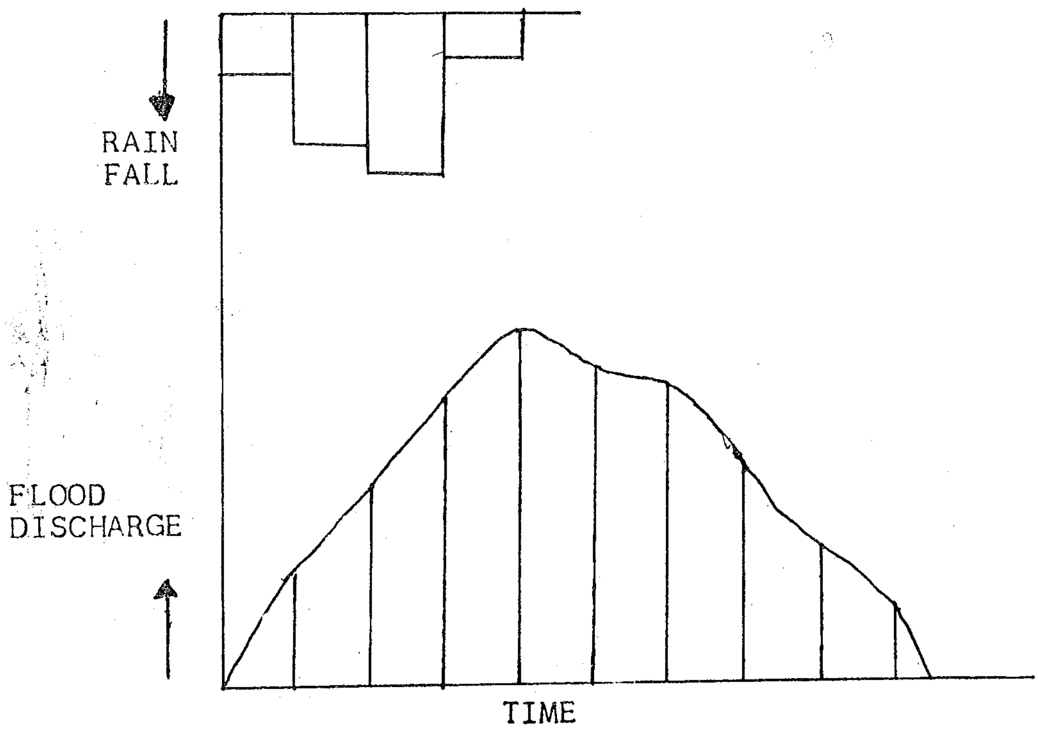


FIG-IV.3.21

There are several methods to derive unit hydrograph in such cases but the two most commonly used methods for analysis of such events are:

1. Instantaneous unit hydrograph.

2. Collin's method for determination of unit hydrograph.

These are discussed in detail in subsequent sections.

4.3.3.4 Instantaneous Unit Hydrograph (IUH)

The Instantaneous Unit Hydrograph is defined as a unit hydrograph of infinitesimally small duration. In other words, IUH is the direct runoff hydrograph at the outlet of the catchment resulting from 1 unit (1 mm) of rainfall falling over the catchment in zero time. Of course, this is only a fictitious situation and a concept to be used in hydrograph analysis.

Derivation of IUH

There are various methods for the determination of an IUH from the given effective rainfall hyetograph and direct runoff hydrograph. But the most common is the model suggested by Nash in 1957. Nash proposed a conceptual model by considering a drainage basin as 'n' identical linear reservoirs in series. By routing a unit inflow through the reservoirs a mathematical equation for IUH can be derived.

The ordinate of the IUH at time t is given by,

$$U(t) = \frac{1}{K(n-1)!}\left(\frac{t}{K}\right)^{n-1} . e^{-t/k}$$

Where n = no. of the reservoir; and

K = a reservoir constant, called storage coefficient.

The values of K and n in Nash model can be evaluated by the method of moments by using the following relations:

$$M_{DRH1} - M_{ERH1} = nK$$

$$M_{DRH2} - M_{ERH2} = n(n+1)\,K^2 + 2\,nk\,M_{ERH1}$$

Where,

M_{DRH1} = first moment arm of DRH

M_{ERH1} = first moment arm of ERH

M_{DRH2} = second moment arm of DRH

M_{ERH2} = second moment arm of ERH.

The unit of the ordinates of IUH is per sec. (Sec^{-1}). When the ordinates are multiplied by the total volume of runoff (in cubic metres) resulting from 1 mm of rainfall over the catchment area, the unit will be cumecs.

Derivation of Unit Hydrograph from IUH

For finding the unit hydrograph from IUH, the area under IUH is plotted with respect to time at the point. Thus line XY in Fig. IV.3.23 represents the area ABB'C in Fig. IV.3.22. This will give a S-Curve.

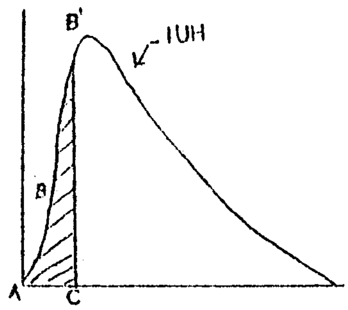


FIG. IV.3.22

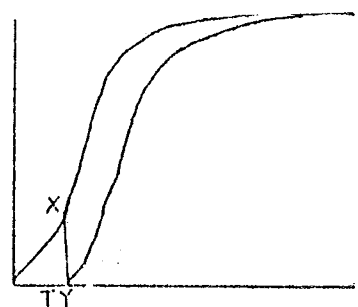
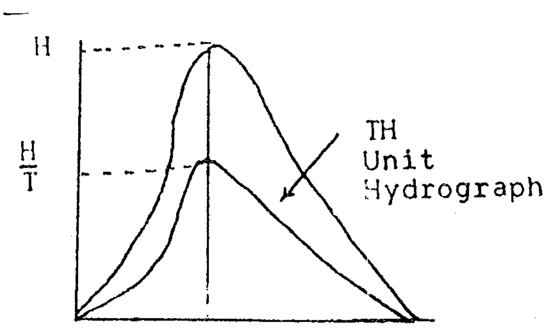


FIG. IV.3.23

If a unit hydrograph of 'T' hour duration is required, the S-Curve is shifted by T-hour and the difference in the ordinates of the two S-Curves is found and divided by T. The resulting curve forms the unit hydrograph of T-hour duration.

The entire procedure of deriving an IUH from known ERH (Effective Rainfall Hyetograph) and DRH (Direct Runoff Hydrograph) and hence the derivation of unit hydrograph is explained below with the help of an example.



IV.3.24

An example illustrating the method of derivation of IUH and that of unit hydrograph of specific duration from the IUH is given in Appendix IV (3).

4.3.3.5 Collin's Method of Unit Hydrograph Determination

The basic steps of this method are:

(i) Assume a unit hydrograph and apply it to all the effective rainfall blocks of the ERH excepting the largest block.

(ii) Find out the resulting hydrograph and substract ordinates from the corresponding ordinates of the actual direct runoff hydrograph.

(iii) Divide the ordinates of the residual hydrograph by the largest block of effective rainfall to get the unit hydrograph.

(iv) Compute the weighted average of the assumed unit hydrograph and the residual unit hydrograph and use it as the revised approximation for the next trial.

(v) Repeat all the previous steps until the residual unit graph does not differ by more than a permissible amount from assumed unit hydrograph.

An example illustrating the various steps involved in derivation of unit hydrograph by Collin's method is presented in Appendix IV (4).

4.3.3.6 Unit Hydrograph of Various Durations

The unit duration of the unit hydrograph derived from the various data sets may not be the same. for example, the data used in examples presented in Appendix IV (2) to IV (4) are for the same site of river Baitarani. It will be observed that the three examples are for unit hydrograph of six hours, one hour and three hours unit duration respectively. In order to compare, the three derived unit hydrographs and also to estimate average of the three, it will be necessary to convert all the three unit hydrograph to the same unit duration. Apart from that, for practical purposes, it is necessary to have unit hydrograph of various durations.

There may be two type of cases.

(i) When unit hydrograph of shorter unit duration 't' is known and a unit hydrograph of longer duration T is to be derived where T is a multiple of t i.e., $T = n.t$ such that n = 1,2, 3...... This can be achieved simply by the principle of super-imposition.

(ii) Otherwise, the unit hydrograph of another duration can be derived by S-curve.

An example presented in Appendix IV (5) illustrates the method for converting a unit hydrograph of t hours unit duration to a unit hydrograph of n.t hours unit duration (n being an integer).

4.3.3.7 S-Curve or S-Hydrograph

The S-hydrograph is a hydrograph produced by a continuous effective rainfall at a constant rate for an indefinite period.

The S-hydrograph can be constructed by summing up a series of identical unit hydrographs spaced at intervals equal to the unit duration of the unit hydrograph. After the S-hydrograph is constructed, the unit hydrograph of a given duration can be derived as follows:

Assume that the S-hydrograph is derived which is due to effective rainfall of 1mm/hour (Fig. IV.3.25).

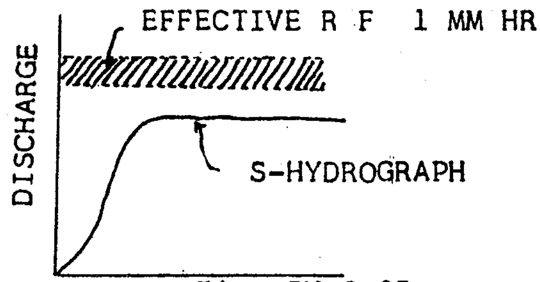


Fig. IV.3.25

Then advance or offset the position of S-hydrographs for a period equal to the desired duration of t hours (Fig. IV.3.26) and find out the difference between the ordinates of the original

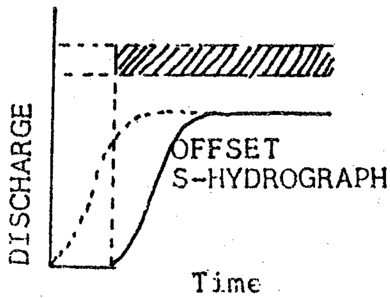


Fig. IV.3.26

S-hydrograph and the offset S-hydrograph (Fig. IV.3.27). This will be the hydrograph due to I. t_o mm of rainfall occurring in t_o hours.

Divide the ordinates of the hydrograph thus obtained (Fig. IV.3.27) by I. to The resulting hydrograph will be the unit hydrograph for t_o hour unit duration as shown in Fig. IV.3.28.

The example presented in Appendix IV (6) illustrates the method of derivation of unit hydrograph of different unit duration by using S-curve.

4.3.3.8 Averaging the Unit Graph

Three different unit hydrographs for Anandpur site on river Baitarani have been developed in the examples presented in Appendix IV. It may be seen that the peak of the unit hydrograph as well as the total base length varies. As a matter of fact, if various storms are considered for development of unitgraph for the same catchment, a marked variation will be

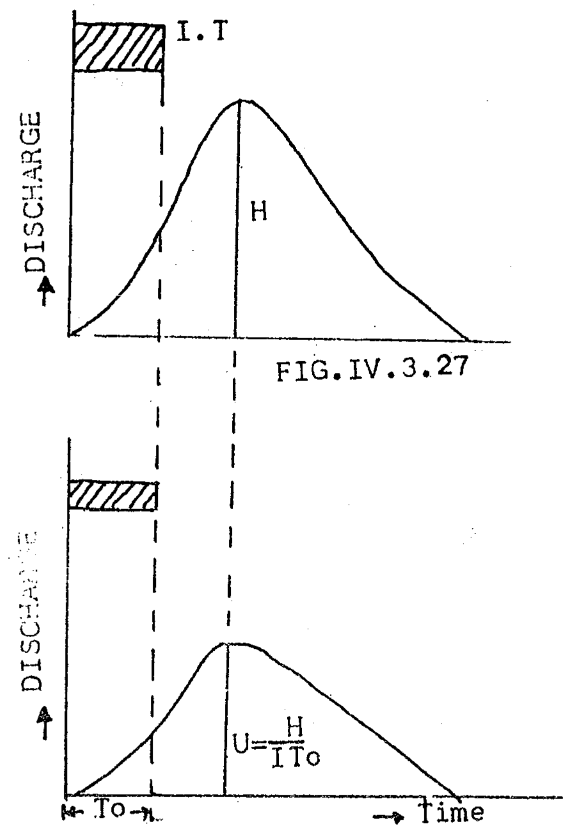


FIG. IV.3.27

FIG. IV.3.28

observed especially in the peak as well as the time of occurrence of the peak. Therefore, it is necessary to derive an average unit hydrograph for practical use.

If several unit hydrographs are averaged by averaging concurrent ordinates, the resulting average unitgraph has a broader, and quite possibly a lower peak than any of the individual graphs.

The correct average unit hydrograph should be obtained by locating the average peak and the average time of occurrence of the peak and sketching a mean unit hydrograph having an area equal to 1 mm of runoff and resembling the individual graph as much as possible.

Fig. IV.3.29. shows the several unit hydrographs of 3 hours unit duration derived for river Baitarani at Anandpur site. To find the average unit hydrograph, the average peak was found to be 141 cumecs and average time of occurrence of peak was estimated to be 15 hours. Similarly the average base length is estimated to be 48 hours. The points A and B are marked as shown in Fig. IV.3.29. Now a suitable unit hydrograph is drawn such that:

AVERAGE PEAK = (156 + 132 + 134)/3
= 141 CUMECS

AVERAGE TIME OF PEAK = (15 + 15 + 12)/3
= 14 HRS SAY 15 HRS

ORDINATES OF AVERAGE U.G.

TIME (HRS)	DISCHARGE (IN CUMECS)
0	0
3	8
6	34
9	103
12	130
15	141
18	119
21	83
24	59
27	40
30	28
33	20
36	14
39	8
42	4
45	2
48	0

$\Sigma U = 793$
$= 1$ MM.

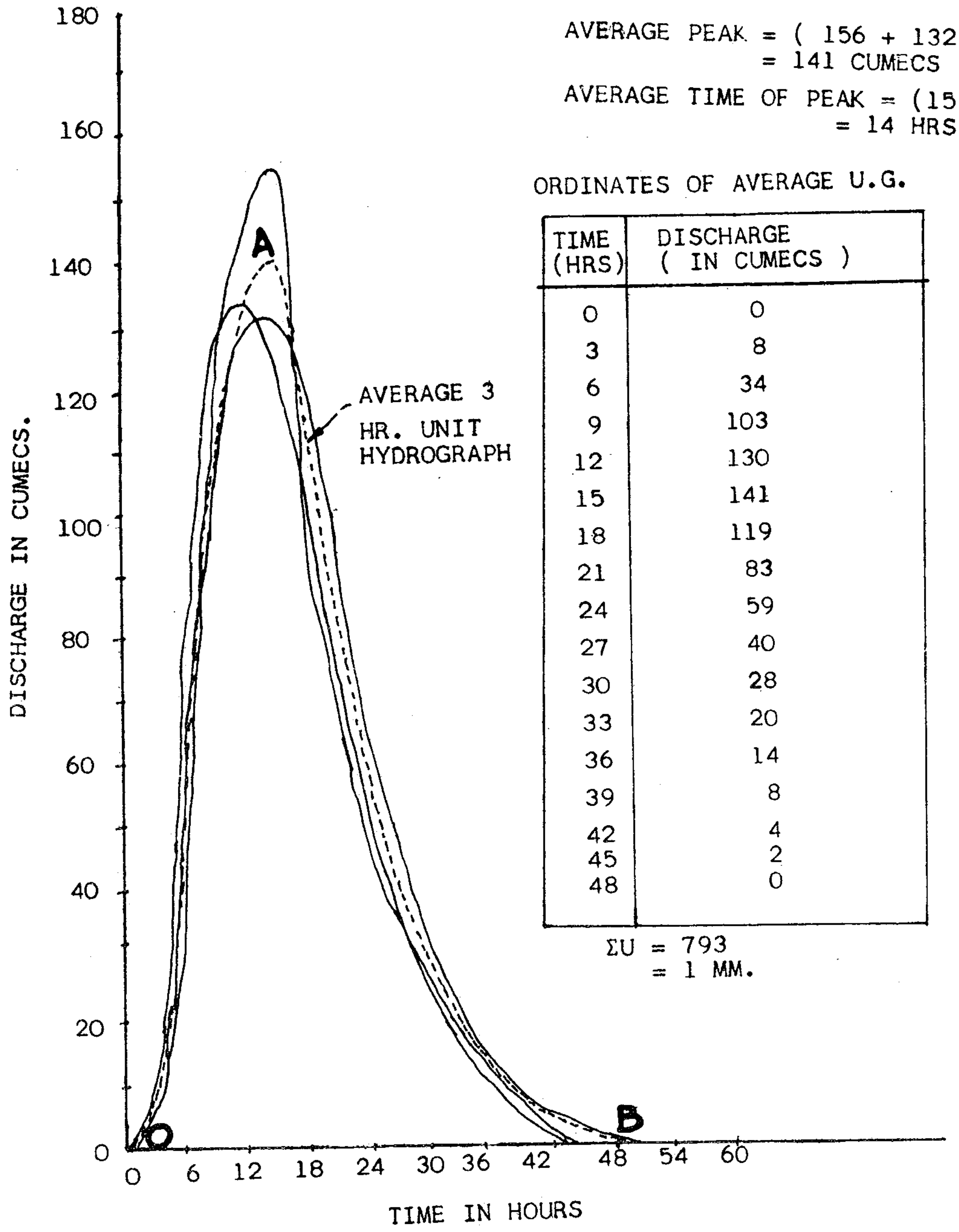


FIG. IV.3.29

1. It passes through the points O, A and B;

2. The area of the unit hydrograph is equal to 1 mm; and

3. The shape resembles the shapes of the three individual unit hydrographs.

The average unit hydrograph thus obtained is shown by dotted line. However, the averaging of the unit hydrograph can not be done in all the cases. It has been observed that for practical purposes the shape of the unit hydrograph is greatly influenced by factors such as amount of effective rainfall, rainfall distribution pattern and the storm movement etc.

Fig. IV 3.30 illustrates, how tne amount of concentrated and heavy effective rainfall affects the shape of the unit hydrographs. It is not necessary that similar features will be reflected in all the storms. As a matter of fact, the formation and distribution of the runoff is quite a complex process in which large number of factors are involved.

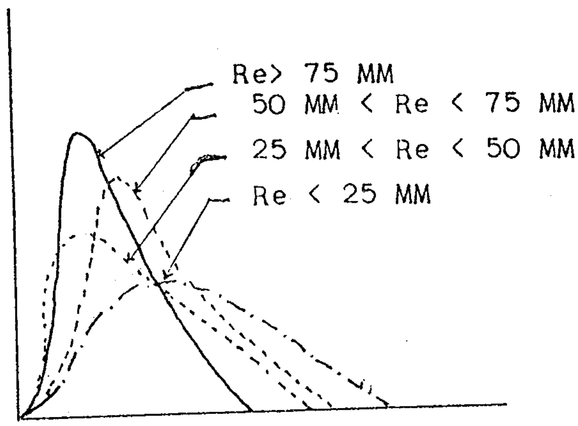
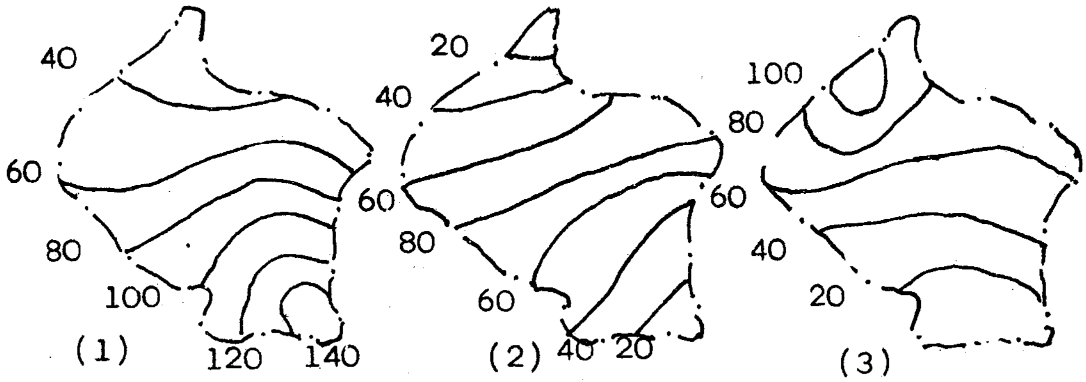


FIG. IV.3.30

Fig. IV.3.31(A) and IV.3.31 (B) illustrate the influence of the pattern of rainfall distribution on the unit hydrographs. Hence for operational use the scheme of the unit hydrograph is to be laid down after taking into account the main influencing factors. It is not sufficient to use one unchangeable unit hydrograph for formulation of flood forecast. Different unit hydrographs should be identified for the various conditions which have major influence on formation and time distribution of the runoff. These unit hydrographs may then be judiciously applied under different conditions.

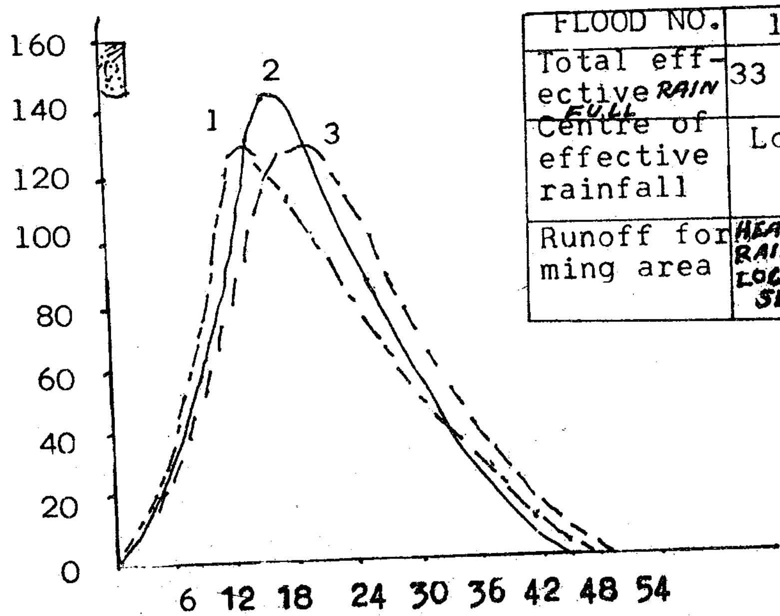
4.3.3.9 Application of Unit Hydrograph in Forecast Formulation

Once a unit hydrograph has been developed for a particular site of a river, it can be very conveniently used for forecast formulation when the actual rainfall data or a dependable QPF is available. This is illustrated with the help of the following example:



(a) ISOHYETALS

FIG. IV.3.31 A



FLOOD NO.	1	2	3
Total effective RAIN FULL	33 mm	21mm	23mm
Centre of effective rainfall	Lower	Midd	Upper
Runoff forming area	HEAVY RAIN LOCALISED	ENTIRE WATER SHED	ENTIRE WATER SHED

(b) UNIT HYDROGRAPH
INFLUENCE OF PATTERN OF RAINFALL DISTRIBUTION

FIG - IV.3.31 B

Example

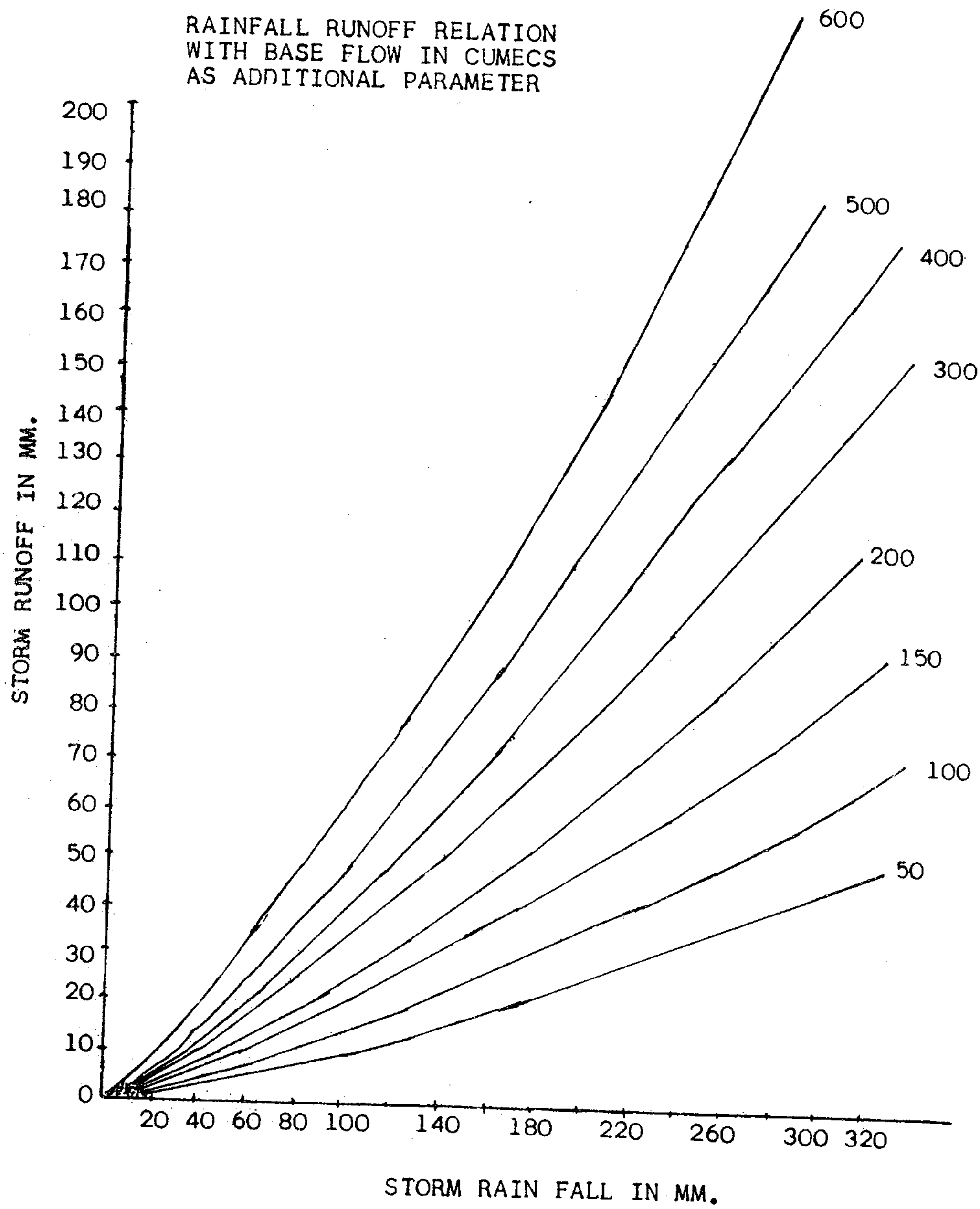
The unit hydrograph for site 'X' on a river has been developed. The ordinates of the three hour unit hydrograph are given below:

Time (Hrs.)	Discharge (Cumecs)
0	0
3	8
6	34
9	103
12	130
15	141
18	119
21	83
24	59
27	40
30	28
33	20
36	14
39	8
42	4
45	2
48	0

The rainfall-runoff relationship which has been developed is given in Fig. IV.3.32

As per the reports received at 1900 hrs. on 13th September, the average rainfall observed at different hours was as follows:

FIG. IV.3.32


RAINFALL RUNOFF RELATION
WITH BASE FLOW IN CUMECS
AS ADDITIONAL PARAMETER
600
500
400
300
200
150
100
50
STORM RUNOFF IN MM.
200
190
180
170
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
20 40 60 80 100 140 180 220 260 300 320
STORM RAIN FALL IN MM.

Duration (Hrs.)	Rainfall Amount
09-12	19.4 mm
12- 15	20.0 mm
15- 18	14.9 mm

The discharge in the river at 09 hrs. was 300 cumecs. Using the above data, compute the ordinates of the expected flood hydrograph.

What will be the peak and its time of occurrence ?

Solution

(i) Total amount of rainfall from 09 to 18 hrs. is 54.3. From the rainfall-runoff relationship given in Fig. IV. 3.32, the amount of runoff corresponding to 54.3 mm of rainfall and 300 cumecs as base flow works out to be 14.5 mm i.e. the loss is of the order of 39.8 mm. Assuming the loss to be at uniform rate, the effective rainfall at various durations will be as follows:

Duration (Hrs.)	*Total rainfall (mm)*	*Loss (mm)*	*Effective rainfall (mm)*
09-12	19.4	13.3	6.1
12-15	20.0	13.3	6.7
15-18	14.9	13.3	1.7

(ii) The ordinates of the unit hydrograph are noted down under col. 3 of the table IV 3.6.

(iii) The values given on the top of columns 4,5 and 6 of the table IV 3.6 are the effective rainfalls in order of time.

(iv) The ordinates of the unit hydrograph are multiplied by 6.1 mm. i.e. the effective rainfall and the results are written under column 4.

(v) The ordinates of the unit hydrograph are multipllied by the next block of effective rainfall i.e. 6.7 mm (as written over column 5) and the results written under col.5 after shifting by 3 hours.

Table IV.3.6

Computation of Flood Hydrograph from Unit Hydrograph and Effective Rainfall Hyetograph

Date	Time	Ordinates of unit hydrograph (in cumecs)	DRH due to rainfall of amount (mm)				base flow	Flood hydro graph	Remarks
			-6.1	6.7	1.7	Total			
1	2	3	4	5	6	7	8	9	10
13.9	09	0	0.00	—	—	0.00	300	300.00	1) Peak of 2206.20
	12	8	48.80	0.00	—	48.80	300	348.80	cumecs will occur
	15	34	207.40	53.60	0.00	261.00	300	561.00	at 00 hrs on 14.9
	18	103	628.30	227.80	13.60	869.70	300	1169.70	
	21	130	793.00	690.10	57.80	1540.90	300	1840.90	2) Time of formulation
14.9	00	141	860.10	871.00	175.10	1906.20	300	2206.20	of forecast about
	03	119	725.90	944.70	221.00	1891.60	300	2191.60	19-20 hrs. when
	06	83	506.30	797.30	239.70	1543.30	300	1843.30	actual rainfall data
	09	59	359.90	556.10	202.30	1118.30	300	1418.30	is available.
	12	40	244.00	395.30	141.10	780.40	300	1080.40	
	15	28	170.80	268.00	100.30	539.10	300	839.10	
	18	20	122.00	187.60	68.00	377.60	300	677.60	

Date	Time	Ordinates of unit hydrograph (in cumecs)	DRH due to rainfall of amount (mm)				base flow	Flood hydro graph	Remarks
			6.1	6.7	1.7	Total			
1	2	3	4	5	6	7	8	9	10
	21	14	85.40	134.00	47.60	267.00	300	567.00	
15.9	00	8	48.80	93.80	34.00	176.60	300	476.60	
	03	4	24.40	53.60	23.80	101.80	300	401.80	
	06	2	12.20	26.80	13.60	52.60	300	352.60	
	09	0	0.00	13.40	6.80	20.20	300	320.20	
	12			0.00	3.40	3.40	300	303.40	
	15				0.00	0.00	300	300.00	

(v) Similarly, the ordinates of unit hydrograph are multiplied by 1.7 mm and the results written under column 6 after shifting the ordinates by 6 hours.

(vii) The corresponding ordinates under column 4, 5 and 6 are added together and written under column 7. This gives the ordinates of the direct runoff hydrograph resulting from given ERH.

(viii) The base flow is assumed to be uniform throughout i.e. 300 cumecs.

(ix) The ordinates of col. 7 and col. 8 are added and written under col. 9. This gives the ordinates of the flood hydrograph.

The value of the peak flood works out to be 2206 cumecs and it will occur at 0000 hrs. of 14th September.

4.3.3.10. Application of Unit Hydrograph when the Rainfall is not uniform over the Catchment

In the above mentioned example on application of unit hydrograph for forecast formulation, the values of average rainfall over the catchment are given. But in real practice the values of the areal rainfall, are to be estimated from the point rainfall. When the spatial variation is considerable, it is not proper to take the average rainfall for estimation of flood hydrograph.

Consider a catchment as shown in the Fig. IV. 3.33. where the rainfall stations are located at A, B and C. The three hourly rainfall data as observed at the three stations is given below in Table IV.3.7.

Fig. IV.3.33

Table IV.3.7

Data	*Time*	*Rainfall at stations*		
		A	*B*	*C*
	09-12	41.0	10.1	7.2
Sept. 13	12-15	14.0	28.8	17.1
	15-18	3.4	7.6	33.8

If the average rainfall is calculated by using the arithmetic mean method the average rainfall at different durations works out to be as follows:

09-12 hrs	—	19.4 mm
12-15 hrs	—	20.0 mm
15 - 18 hrs	—	14.9 mm

which is same as the rainfall values given in the above example. But it may be observed that the spatial variation of the rainfall is too large and simple averaging is not justified. Under such circumstances it becomes necessary to adopt certain techniques such that the spatial variation is taken care of.

One simple approach to tackle such a problem may be by dividing the water-shed into sub-areas. As the main purpose of dividing the water-shed into sub-areas is to deal with the non-uniformity in rainfall distribution, the size of the area resulting from the division should be appropriate so that rainfall distribution over each sub-area is rather uniform and each sub-area has a certain raingauge station. Moreover, the boundary of the sub-area should coincide, as far as possible, with that of the natural basins to facilitate the analysis and treatment of problem as well as the use of available hydrological data of small water-shed.

For this purpose the concept of the dimensionless unit hydrograph is used and the same unit hydrograph is applied to all the sub-areas. The basic steps in estimating the flood hydrograph by this method are illustrated with the help of the following example:

Example

In the above example, substitute the rainfall data as follows:-

Table IV. 3.8

Date	*Time*	*Rainfall at station*		
		A	*B*	*C*
	09-12	41.0	10.1	7.2
	12-15	14.0	28.8	17.1
	15-18	3.4	7.6	33.8

The location of the stations A, B and C are shown in Fig. IV.3.33. The total catchment area is 8570 sq. km.

Steps

Step No. 1. Division of Watershed into Sub-Areas

The watershed is divided into three sub-areas as shown in Fig. IV. 3.34. such that each raingauge station A, B and C represents the rainfall in each of the three sub-areas.

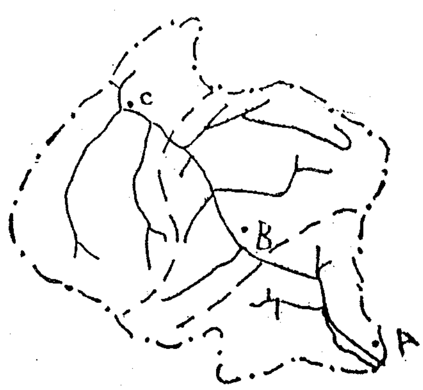


FIG. IV.3.34

Attention has been paid that the division is in accordance with the natural drainage areas. It has been assumed that with respect to the physiographic condition, the three areas belong to the same region and hence the same unit-hydrograph applied to all. The weights to each sub-basin and the number of ~~the~~ sub-reaches to be considered for each sub-basin is given in table IV. 3.9.

Table IV. 3.9

Sub-basin	*A*	*B*	*C*
Area km^2	2040	3470	3060
Weights	0.238	0.405	0.357
No. of sub-reaches in flow concentration of the river network	0	1	2

The weight has been calculated by simply dividing the area of the sub-basin by the total area of the basin, for example:

Area of the sub-basin A = 2040 sq. km.

Total area of the basin = 8570 sq. km.

$$\text{Weight for area A} = \frac{2040}{8570} = 0.238$$

Step No. 2. Computation of Effective Rainfall

Total amount of rainfall from 09 to 18 hours at various stations is as follows:

Table IV. 3.10

Station	*Total runoff mm*	*Effective rainfall mm*	*Loss mm*
Station A	58.4	15	43.4
Station B	46.5	14	32.5
Station C	58.1	15	43.1

From the rainfall-runoff relation in Fig. IV.3.32., the amount of runoff corresponding to the base flow of 300 cumecs for the various values of the total rainfall is computed and is given in column 3 of table IV. 3.10 and hence the losses can be computed and are given under column 4 of the table IV. 3.10. The distribution of the effective rainfall at various durations at different stations is given in the Table no. IV. 3.11.

Table IV. 3.11

Date	Time	A	B	C
	0 12	15	0	0
Sept. 13	12 - 15	0	14	2.7
	15 - 18	0	0	12.3

Step No. 3 Computation of Basin Flow Concentration

The unit hydrograph method is used for the computation of surface runoff hydrograph of sub-area. As already discussed above it can be considered that the dimensionless unit hydrograph of the three sub-areas are identical. In order to simplify the computational procedure the unit hydrograph is not applied to all the sub-areas individually but once to the three sub-areas altogether i.e. to say after multiplying effective rainfall of each sub-area by the corresponding areal weight the computation of the river flow concentration is performed and the results are added together and then the unit hydrograph is applied to get the out flow. The unit hydrograph applied is the same as that for the whole watershed.

To compute the co-efficient of river flow concentration, the Muskingum method is adopted to perform the successive routing through the sub-reaches. For this the value of K and x are calculated for each sub-reach by using the normal technique. In this case, the value of K and x for the two sub-reaches is taken as 3 hrs. and 0.45 respectively. The procedure is same as explained subsequently in case of routing through sub-reaches. Table IV.3.12. gives the computational details.

Step No. 4: Computation of Equivalent Effective Rainfall

The effective rainfall as computed in Table IV. 3.11 above is for sub-basins. First of all the effective rainfall thus calculated are multiplied by respective sub-basin weights and are routed throught sub-reaches to take care of the travel time.

The calculation details are given in Table IV. 3.13. Step by step procedure is explained below:

(I) Write down the effective rainfall as in Table IV. 3.11 under column 2, column 4 and column 12 for station A, B and C respectively.

(II) Multiply the effective rainfall under column 2 by the sub-basin weight i.e. 0.238 and note down the result under column 3. Similarly multiply the rainfall noted under column 4 and column 12 by the respective sub-basin weights i.e. 0.405 and 0.357 and note down the value under column 5 and column 13 respectively.

(III) The value of weighted effective rainfall under column 5 is routed through one sub-reach and the routed value is noted under column 11. Similarly the weighted effective rainfall under column 13 is routed through two sub-reaches and the routed value is noted under column 21.

(IV) Column 22 gives the total of column 5, column 11 and column 21 which is the equivalent effective rainfall and is to be applied to the unit hydrograph of the total catchment to get the Direct Runoff Hydrograph.

Step No. 5: Computation of Flood Hydrograph

With the help of unit hydrograph and the equivalent effective rainfall the ordinates of flood hydrograph can be very easily computed in the same fashion as illustrated in case of earlier example. Table IV. 3.14 gives the details of the calculation.

$$C_o = \frac{0.5\Delta t - Kx}{K - Kx + 0.5\Delta t}, \quad C_1 = \frac{0.5\Delta t + Kx}{K - Kx + 0.5\Delta t}$$

$$C_2 = \frac{K - Kx - 0.5\Delta t}{K - Kx + 0.5\Delta t}$$

or $K = 3, x = 0.45$ and $\Delta t = 3$

$C_0 = 0.048$
$C_1 = 0.904$
$C_2 = 0.048$

$$O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1$$

Results and Discussion

It may be seen that the value of peak flood works out to be 1968 cumecs and its time of occurence will be at 0600 hrs. on 14th of September. When compared with the results obtained in earlier example, it is seen that the peak has reduced by about 238 cumecs and the time of occurrence has advanced by about 6 hours. i.e. there is considerable effect on forecast if the spatial variation in rainfall is considered. The reduction in peak as well as increase in concentration time is quite logical as may be seen from the rainfall pattern. In the second case the rainfall first occurs in the lower basin and the storm advances towards the upper regions. By the time the runoff from upper region reaches the out-let point, the runoff resulting from lower basin must have passed, resulting in reduction in peak.

4.3.3.11 Synthetic Unit Hydrographs

The unit hydrograph for a catchment can be derieved using the above discussed techniques only when sufficient rainfall and runoff data are available but for most of the catchments, very little or no data are available and therefore, it is desirable to develop a procedure for the derivation of synthetic unit hydrograph and hence prediction of storm runoff for ungauged catchments. This can be attempted if the rainfall and runoff data are available for similar catchment in the same geographical region.

Table - IV. 3.12

t	*I*	*Ist Sub-Reach*				*2nd Sub-Reach*			
		$C_o\ I_1$	$C_1\ I_1$	$C_2\ O_1$	O_2	$C_0\ I_2$	$C_1\ I_1$	$C_2\ O_1$	O_2
1.	I	0.048	-	-	0.048	0.0023	-	-	0.0023
2.			0.904	0.0023	0.9063	0.0435	0.0434	0.00011	0.0870
3.				0.0435	0.435	0.0021	0.8193	0.0042	0.8256
				0.0021	0.0021	0.0001	0.0393	0.0396	0.0790
				0.0001	0.0001	-	0.0019	0.0038	0.0057
				0	0	-	0.0001	0.0003	0.0004
						-	0	0	0

Table - IV. 3.13

Ist reach										
Time	*Effective rainfall*	*Weighted effective rainfall*	*Effective rainfall*	*Weighted effective rainfall*	*0.048*	*0.9063*	*0.0435*	*0.0021*	*0.001*	*Total*
1	2	3	4	5	6	7	8	9	10	11
09-12	15	3.57	0	0	0	-				0
12-15	0	-	14	5.67	0.272	0				0.272
15-18		-	0	0	0	5.139	0			5.139
18-21						0	0.247	0		2.247
21-24							0	0.012	0.00	0.012
00-03								0	0.001	0.001
03-06									0.00	0.00
06-09										
09-12										

Table IV. 3.13 (contd.)

		2nd reach								*Total*
Effective rainfall	*Weighted rain fall*	*0.0023*	*0.0870*	*0.8256*	*0.0790*	*0.0057*	*0.0004*	*0*	*Total*	
12	13	14	15	16	17	18	19	20	21	22
0	0	0							0	3.57
2.7	0.964	0.002	0						0.002	0.274
2.3	4.39	0.010	0.235	0					0.245	5.384
			0.382	0.796	0				1.178	1.425
				3.624	0.076	0			3.700	3.712
					0.347	0.005	0		0.352	0.353
						0.025	0.004	0	0.0254	0.0254
							0.002	0	0.002	0.002
								0	0	0

Table IV.3.14

Computation of Flood Hydrograph from Unit Hydrograph and Effective Rainfall Hyetograph

Date	*Time*	*Ordinates of unit hydrograph (Cumecs)*	*D.R.H. due to rainfall of amount (mm)*					
			3.57	*0.274*	*5.384*	*1.425*	*3.712*	*0.353*
1	2	3	4	5	6	7	8	9
13th Sept.	09-12	0	0.00	-				
	12-15	8	28.56	0.00	-			
	15-18	34	121.38	2.192	0.00	-		
	18-21	103	367.71	9.320	43.07	0.00	-	
	21-24	130	464.10	28.22	183.06	11.40	0.00	-
14th Sept.	00-03	141	503.37	35.62	554.55	48.45	29.70	0.00
	03-06	119	424.83	38.63	699.92	146.78	126.21	2.82
	06-09	83	296.31	32.61	759.14	185.25	382.34	12.00
	09-12	59	210.63	22.74	640.70	200.93	482.56	36.36

Table IV. 3.14 (contd.)

Date	*Time*	*Ordinates of unit hydrograph (Cumecs)*	*D.R.H. due to rainfall of amount (mm)* 3.57	0.274	5.384	1.425	3.712	0.353
1	2	3	4	5	6	7	8	9
	12-15	40	142.80	16.17	446.87	169.58	523.39	45.89
	15-18	28	99.96	10.96	317.66	118.28	441.73	49.77
	18-21	20	71.40	7.67	215.36	84.08	308.10	42.01
	21-24	14	49.98	5.48	150.75	57.00	219.01	29.30
15th Sept.	00.03	8	28.56	3.84	107.68	39.90	148.48	20.83
	03-06	4	14.28	2.19	75.38	28.50	103.94	14.12
	06-09	2	7.14	1.10	43.07	19.95	74.24	9.88
	09-12	0	0.00	0.55	21.54	11.40	51.97	7.06
	12-15			0.00	10.77	5.70	29.70	4.94
	15-18				0.00	2.85	14.85	2.82
	18-21					0.00	7.42	1.41
	21-24						0.00	0.71
16th Sept.	00-03							0.00
	03-06							
	06-09							
	09-12							
	12-15							

Table IV. 3.14 (contd.)

0.0254	*0.002*	*0.000*	*Total*	*Base flow*	*Flood hydrograph*	*Remarks*
10	11	12	13	14	15	16
			0.00	300	300.00	1) Peak of 1967.85 cumecs will occur at 0600 hrs. on 14 Sept.
			28.56	300	328.56	
			123.57	300	423.57	
			420.10	300	720.10	2) Time of formulation of forecast about 19-20 hrs. when actual rain fall is available.
			686.78	300	986.78	
-			1171.69	300	1471.69	
0.00	-		1439.19	300	1739.19	
0.20	0.00	-	1667.85	300	1967.35	

0.0254	0.002	0.000	Total	Base flow	Flood hydrograph	Remarks
10	11	12	13	14	15	16
0.86	0.02	0.00	1594.79	300	1894.79	
2.62	0.07	0.00	1347.38	300	1647.38	
3.30	0.21	0.00	1041.86	300	1341.85	
3.58	0.25	0.00	732.45	300	1032.45	
3.02	0.28	0.00	514.82	300	814.82	
2.11	0.24	0.00	351.63	300	651.63	
1.50	0.17	0.00	240.07	300	540.07	
1.00	0.12	0.00	156.52	300	456.52	
0.71	0.08	0.00	93.30	300	393.30	
0.51	0.06	0.00	51.67	300	351.67	
0.36	0.04	0.00	20.92	300	320.92	
0.20	0.03	0.00	9.07	300	309.07	
0.10	0.02	0.00	0.82	300	300.82	
0.05	0.01	0.00	0.06	300	300.06	
0.00	0.00	0.00	0.00	300	300.00	
	0.00	0.00	0.00	300	300.00	
		0.00	0.00	300	300.00	
			0.00			

The basic approach used is as follows:

(i) to derive the unit hydrographs for the catchment in the region for whose records are available.

(ii) to find a correlation between some defined parameters of these unit hydrographs and the catchment characteristics, and

(iii) to use this correlation to predict the parameters of the unit hydrograph for catchments which have no records of stream flow but for which the catchment characteristics can be derived from topographical maps. Some of the methods suggested by various authors for development of synthetic unit hydrograph include:

(1) Use of time area concentration curve

(2) Snyder's method

(3) Transposition of unit hydrograph.

Time Area Concentration Curves

The time area concentration curves are estimated on the basis of the time of travel from various parts of the catchment to the outlet as computed by hydraulic equations for steady flow. However, for practical purposes the curve may be developed merely on the basis of the estimate of the time of translation over the ground and channels.

Consider a catchment as shown in Fig. IV. 3.35 The dotted lines indicate the isochrones. The number marked near each isochrone indicates the time taken to drain out the water from all the points on the line to outlet point O.

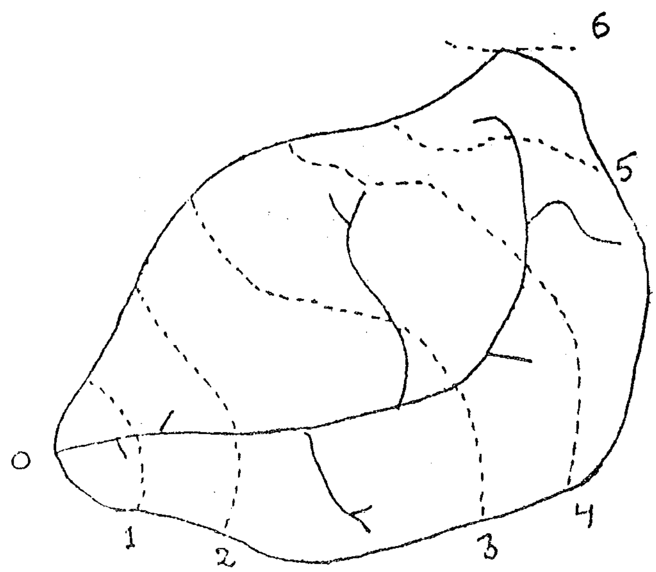


FIG - IV. 3. 35

If a rainfall of 1 mm falls over the entire basin at a time then, during Ist hour the runoff from the area upto isochrone 1 will reach the outlet. During 2nd hour the runoff from the area between the isochrone 1 and isochrone 2 will reach the outlet point O and so on. And hence the runoff at different hours can be easily computed which will be proporational to the respective areas between two isochrones. The plot of hourly runoff thus computed is the same as the instantaneous unit hydrograph in the linear system approach. Thus, the time area concentration curve, whose shape is computed on the basis of catchment characteristic is in fact a synthetic unit hydrograph.

However, in this case simply the translation effect is considered and hence the results obtained using this method generally give higher peak rate of discharge from the catchment due to neglecting the effect on the runoff process of attenuation due to surface storage, soil storage and channel storage. In order to take into account the storage effect, Clark (1945) suggested that the IUH could be derived by routing the time area concentration curve through a single element of linear storage. Subsequently several other models suggesting modifications in the above method have been proposed which are discussed in detail in chapter V while introducing the conceptual models for flood forecasting.

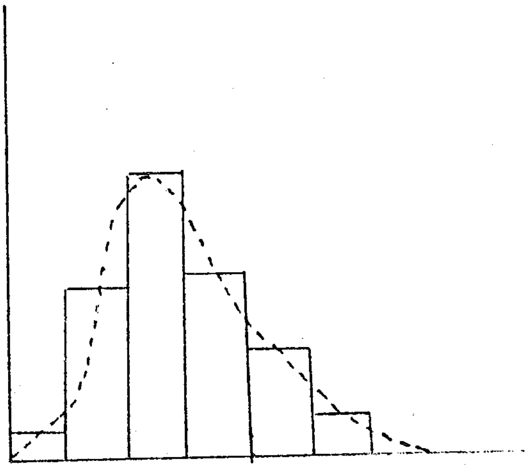


FIG. IV. 3. 36

Snyder's Method

Among several known methods for development of synthetic unit hydrograph the one suggested by F.F. Snyder (1938) is most commonly used. Snyder analysed a large number of hydrographs from drainage basins in the Applichion Mountain region in the United States, ranging in area from 25 sq. km. to 25,000 sq. km.

To sketch a unit hydrograph, it is necessary to know the time of the peak, the peak flow and the time base. The elements must be determined for every particular or regional location of the drainage basin. Snyder proposed the following empirical formula for the lag time (Hr.) from mid point of effective-rainfall duration t_r to peak of a unit hydrograph:

$$t_p = C_t \, (L. \, Lc)^{0.3}$$

In which t_p = the basin lag in hours, from mid point of effective rainfall duration t_r to peak of a unit graph:

L = the length of the main stream from the outlet to the divide in kms;

L_c = the distance from the outlet to a point on the stream nearest to the centroid of the basin; and

C_t = a co-effecient.

The location of the centre of-area may be determined by cutting the basin outline from card board and marking the point of intersection of plumb lines drawn with the map suspended from different corners. The co-efficient C_t varies from 1.0 to 2.2 with lower values associated with basins of steeper slopes.

For the standard duration of effective rainfall t_r, Snyder proposed:

$$t_r = \frac{t_p}{5.5}$$

For the rains of this duration, he found that synthetic unit hydrograph peak Q_p in cumecs may be obtained from the equation:

$$Q_p = \frac{C_p . A}{t_p}$$

And in cumecs/sq km by the relation

$$Q_p = \frac{C_p}{t_p}$$

Where A = the drainage area in square kms.

C_p = co-effecient ranging from 4.0 to 5.0

Q_p = Peak flood in cumec.

For the time base T (in days) of the synthetic unit hydrograph U.S. Army Crops of Engineers adopted the following expression

$$T = 3 + 3.\frac{t_p}{24}$$

These equations are sufficient to construct a synthetic unit hydrogrph for a storm of duration t_r.

The value of Snyder's coefficients C_t and C_p are found to vary considerably depending upon the topography, geology and climate. Snyder indicated that the coefficient C_t is affected by basin slopes S. Linseley, Kohler and Paulhus have suggested an expression for t_p in which the basin slope S has been considered.

$$t_p = C_t \left[\frac{L.Lc}{\sqrt{S}} \right]^n$$

Where n = 0.38 and C_t = 1.2 for mountaineous drainage areas; 0.72 for foothills; and 0.35 for valley areas.

Transposition of Unit Hydrograph

If unit hydrographs are available for several areas adjacent to a basin for which a unit hydrograph is required but for which necessary data are lacking, then transposition of available unit hydrograph will ordinarily give better results than resorting to a wholly synthetic procedure. Sherman originally proposed that the ordinates and abscissas of unit hydrograph for similar basins might be assumed to be proportional to the square roots of the respective drainage areas. Further details are available in any text book on Applied Hydrology.

4.3.4 Stream Flow Routing

As already discussed above, the Muskingum method of stream flow routing is most commonly used. The method has been improved subsequently and a very effective method is the successive routing through sub-reaches.

These two methods are discussed hereunder:

4.3.4.1 Muskingum Method of Stream Flow Routing

The two basic equations for stream flow routing by Muskingum method are:

(a) Storage equation

$$S = K\,[\,x I + (1 - x)\, O\,]$$

Where

S = Storage.

K = A constant expresses the ratio between storage and discharge. It is, in fact, a measure of the lag or time of travel through the reach and the slope of the storage — discharge curve.

x = A dimension-less factor which defines the relative weights given to inflow and outflow in determining stroage.

I = Inflow rate.

O = Outflow rate.

(b) Water Balance Equation:

$$\frac{I_t + I_{t+1}}{2}\Delta t - \frac{O_t + O_{t+1}}{2}\Delta t = S_{t+1} - S_t$$

Where I_t = Inflow rate at time t

I_{t+1} = Inflow rate at time t + Δ t

O_t = Outflow rate at time t

O_{t+1} = Outflow rate at time t + Δt

S_t = Storage at time t

S_{t+1} = Storage at time t + Δt

combining the two equations, we get

$$O_{t+1} = C_0 I_{t+1} + C_1 I_t + C_2 O_t$$

The co-efficients C_0, C_1, and C_2 are given respectively by

$$C_0 = \frac{0.5\Delta t - Kx}{K - Kx + 0.5\Delta t}$$

$$C_1 = \frac{0.5\Delta t - Kx}{K - Kx + 0.5\Delta t}$$

$$C_2 = \frac{K - Kx - 0.5\Delta t}{K - Kx + 0.5\Delta t}$$

The co-efficients are connected by the relation :

$$C_0 + C_1 + C_2 = 1$$

This is the routing formula of the MUSKINGUM method.

It may be seen that only the parameters K and x are unknown. These parameters can be derived by two ways namely;

(i) Empirical method

(ii) Analytical method

Empirical method to obtain K and x

The figure IV.3.37 shows the inflow and the outflow hydrograph at upstream and downstream respectively of a channel reach.

First of all, a certain value of **x** is chosen and the values of

$[xI_t + (1 - x) O_t]$ and $S = \int_o^t (I - O)\, dt$.

are calculated and these are plotted as shown in Fig. IV.3.38. If the value of x is chosen correctly, the plot in Fig. IV.3.38 will appear to be a straight line. Hence, a number of values of x are tried and finally the value of x which results in a loop very near to a straight line is chosen. The slope of the line provides the value of K.

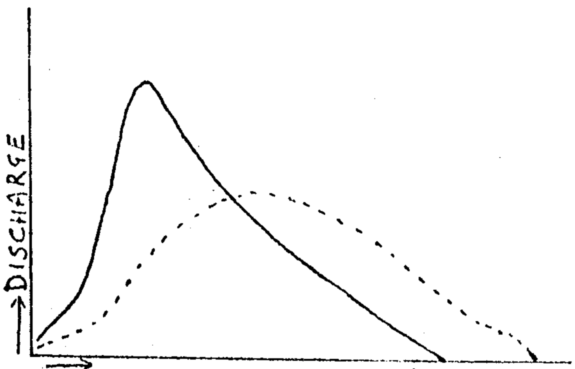


FIG IV.3.37

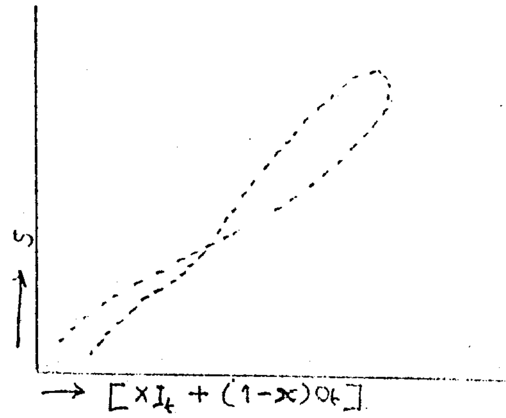


FIG. IV.3.38

Once the values of K and x are estimated, the co-efficient C_o, C_1, C_2 of the routing equation

$$O_{t+1} = C_0 I_{t+1} + C_1 I_t + C_2 O_t$$

can be easily calculated.

Example

Fig. IV.3.39 shows the observed flood hydrograph of Japla and Koelwar respectively on the lower reach of river Sone. Using these data the analysis is carried out in subsequent sections:

1. to estimate K and X for the reach; and

2. to develop routing equation.

Estimation of K and X

The various steps involved in the process of estimation of K and X are explained below:

1. The observed discharge at 2 hours interval at Japla (the upstream station) which is considered as inflow (I) in subsequent discussions is noted down in col. 3 of Table IV 3.15.

2. The observed discharge at 2 hour interval at Koelwar (the downstream station) which is considered as outflow (O) in subsequent discussion is noted down in col. 4 of Table IV. 3.15.

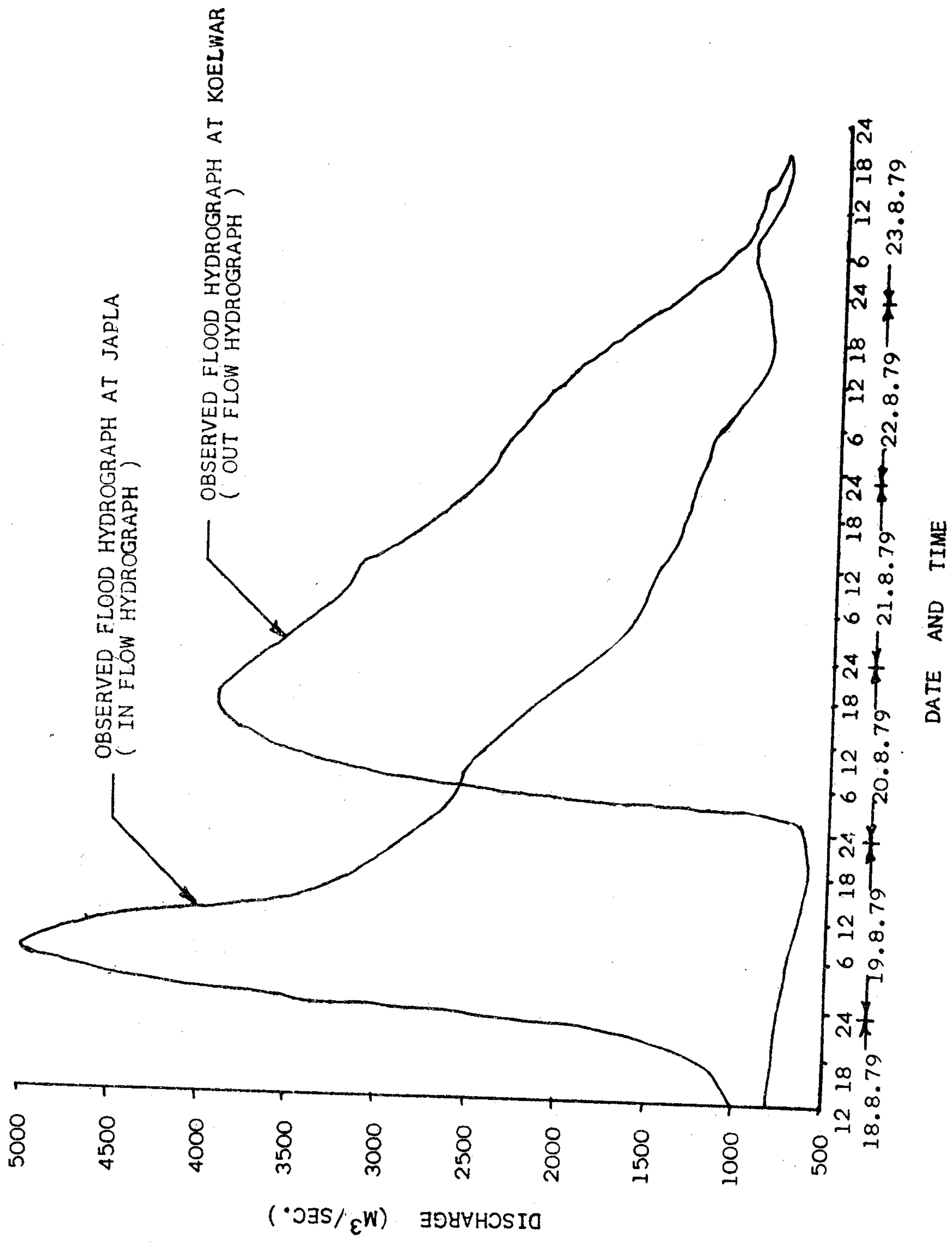

OBSERVED FLOOD HYDROGRAPH AT JAPLA
(IN FLOW HYDROGRAPH)
OBSERVED FLOOD HYDROGRAPH AT KOELWAR
(OUT FLOW HYDROGRAPH)
DISCHARGE (M3/SEC.)
5000
4500
4000
3500
3000
2500
2000
1500
1000
500
12 18 24 6 12 18 24 6 12 18 24 6 12 18 24 6 12 18 24 6 12 18 24
18.8.79
19.8.79
20.8.79
21.8.79
22.8.79
23.8.79
DATE AND TIME

FIG. IV.3.39

3. The value of S at various time is calculated by using the relation.

Explanation:

S at 2nd hour $= S_2 = S_1 + \frac{(I_2 - O_2) + (I_1 - O_1)}{2} \Delta t$

$= 0 + \frac{(1075 - 800) + (1025 - 800)}{2} \times 2$

$= 0 + \frac{275 + 225}{2} \times 2 = 500$

S at 4th hour $= S_3 = S_2 + \frac{(I_3 - O_3) + (I_2 - O_2)}{2} . \Delta t$

$= 500 + \frac{(1120 - 800) + (1075 - 800)}{2} \times 2$

$= 500 + \frac{320 + 275}{2} \times 2$

$= 500 + 595$

$= 1095$

S at 16th hour $= S_9 = S_8 + \frac{(I_9 - O_9) + (I_8 - O_8)}{2} \Delta t$

$= 15540 + \frac{(4380 - 730) + (3985 - 750)}{2} \times 2$

$= 15540 + \frac{3650 + 3235}{2} \times 2$

$= 15540 + 6885$

$= 22425$

The values of S thus computed are noted down in column 5 of Table IV.3.15

4. Next step is to calculate the value of $[XI_t + (I\text{-}X) O_t]$ for various assumed values of X. The value of $[XI_t + (I\text{-}X) O_t]$ is calculated at different time for X = 0.3, X = 0.4, X= 0.45 and X = 0.48 and noted down in column 6,7,8 and 9 respectively.

Explanation:

At eighteenth hour of 19.8.1979.

$I = 3110\ m^3/sec.$

$0 = 610\ m^3/sec.$

For X = 0.3, $XI + (1 - X)\,O$ $= 0.3 \times 3110 + (1 - 0.3) \times 610$

$= 933 + 0.7 \times 610$

$= 933 + 427$

= 1360 which is indicated in col. 6.

For X = 0.4, $XI + (1 - X)\,O$ $= 0.4 \times 3110 + (1 - 0.4) \times 610$

$= 1244 + 0.6 \times 610$

= 1610 which is indicated in col . 7.

For X = 0.45, $XI + (1 - X)\,O$ $= 0.45 \times 3110 + (1 - 0.45) \times 610$

$= 1399.5 + 335.5$

= 1735 which is indicated in col .8.

Table - IV. 3.15

Date	*Time*	*I*	*O*	*S*	*XI + (1 - X) O*			
					X=0.3	*X=0.4*	*X=0.45*	*X=0.48*
1	2	3	4	5	6	7	8	9
18.8.79	1200	1025	800	0	867.5	890	901.25	908
	1400	1075	800	500	882.5	910	923.75	932
	1600	1120	800	1095	896	923	944	953.6
	1800	1225	790	1850	920.5	964	985.75	998.8
	2000	1520	780	3025	1002	1076	1113	1135.2
	2200	2455	760	5460	1268.5	1438	1522.75	1575.6
19.8.79	0000	3425	750	9730	1522.5	1780	1908.75	1986
	0200	3985	750	15540	1720.5	2044	2205.75	2302.8

Date	Time	I	O	S	XI + (1 − X) O			
					X=0.3	X=0.4	X=0.45	X=0.48
1	2	3	4	5	6	7	8	9
	0400	4380	730	22425	1825	2190	2372.5	2482
	0600	5035	710	30400	2007.5	2440	2656.25	2786
	0800	4880	790	38915	1947	2366	2575.5	2701
	1000	4625	660	47070	1849.5	2246	2444.25	2563.2
	1200	4180	640	54575	1702	2056	2233	2339.2
	1400	3615	630	61100	1525.5	1824	1973.25	2062.8
	1600	3270	620	66735	1415	1680	1812.5	1892
	1800	3110	610	71885	1360	1610	1735	1810
	2000	3005	610	76780	1328.5	1568	1687.75	1759.6
	2200	2905	620	81460	1305.5	1534	1648.25	1716
20.8.79	0000	2805	630	85920	1282.5	1500	1608.75	1674
	0200	2710	650	90153	1268	1474	1577	1638.8
	0400	2615	1450	93380	1799.5	1916	1974.25	2009.2
	0600	2515	2400	94690	2443.5	2458	2465.25	2469.6
	0800	2500	3100	94235	2920	2860	2830	2812
	1000	2455	3400	92690	3116.5	3022	2974.75	2946.4
	1200	2410	3650	90505	3278	3154	3092	3054.8
	1400	2305	3850	87720	3386.5	3232	3154	3108.4
	1600	2240	3930	84485	3423	3254	3169.5	3118.8

Date	Time	I	O	S	XI + (1 − X) O			
					X=0.3	X=0.4	X=0.45	X=0.48
1	2	3	4	5	6	7	8	9
	1800	2140	3920	81015	3386	3208	3119	3065
	2000	2020	3870	77385	3315	3130	3037.5	2982
	2200	1910	3790	73655	3226	3038	2944	2887.6
21.8.79	0000	1800	3650	69925	3095	2910	2817.5	2762
	0200	1715	3530	66260	2985.5	2804	2713.5	2658.8
	0400	1630	3400	62675	2869	2692	2603.5	2550.4
	0800	1540	3320	55725	2716	2548	2464	2413.6
	1000	1505	3188	52378	2677.5	2510	2426.25	2376.6
	1200	1475	3150	49020	2647.5	2480	2396.25	2346
	1400	1415	2950	45810	2489.5	2336	2259.25	2213.2
	1600	1360	2850	42785	2403	2254	2179.5	2134.8
	1800	1330	2700	39925	2289	2152	2083.5	2042.4
	2000	1305	2600	37260	2211.5	2082	2017.5	1978.4
	2200	1275	2500	34740	2132.5	2010	1948.75	1912
22.8.79	0000	1250	2440	32325	2083	1964	1904.5	1868.8
	0200	1235	2380	29990	2036.5	1922	1864.75	1830.4
	0400	1225	2320	27750	1991.5	1882	1827.25	1794.4
	0600	1225	2280	25600	1963.5	1858	1885.25	1773.6
	0800	1085	2220	23410	1879.5	1766	1709.25	1675.2

Date	Time	I	O	S	XI + (1 − X) O			
					X=0.3	X=0.4	X=0.45	X=0.48
1	2	3	4	5	6	7	8	9
	1000	995	2150	21120	1803.5	1688	1630.25	1595.6
	1200	950	2050	18865	1720	1610	1555	1522
	1400	915	1960	16720	1646.5	1542	1489.75	1458.4
	1600	885	1850	14710	1560.5	1464	1415.75	1386.8
	1800	865	1750	12860	1484.5	1396	1351.75	1325.2
	2000	875	1630	11220	1403.5	1328	1290.25	1267.6
	2200	895	1500	9860	1318.5	1258	1227.75	1209.6
23.8.79	0000	915	1380	8790	1240.5	1194	1170.75	1166.8
	0200	950	1280	7995	1181	1148.5	1131.5	1121.6
	0400	980	1200	7445	1134	1112	1101	1094.4
	0600	1025	1100	7150	1077.5	1070	1066.25	1064
	0800	995	1040	7030	1026.5	1022	1019.75	1018.4
	1000	950	1000	6935	985	980	977.5	976
	1200	865	970	6780	938.5	928	922.75	919.6
	1400	845	930	6590	994.5	896	891.75	889.2
	1600	825	880	6450	863.5	858	855.25	853.6
	1800	805	850	6350	836.5	832	829.75	828.4
	2000	825	820	6310	821.5	822	822.25	822.4

For X = 0.48, $XI + (1 - X)\,O = 0.48 \times 3110 + (1 - 0.48) \times 610$

$= 1492.8 + 317.2$

$= 1810$ which is indicated in col. 9.

5. The graph with XI + (1- X) O on the X -axis and S on the Y-axis is plotted for different values of X i.e. for X = 0.3, 0.4, 0.45 and 0.48. The values are shown in Fig. IV. 3.40.

6. Although none of the plottings shown in Fig. IV.3.40 form a straight line, the plotting which resembles maximum to a straight line is selected. For this purpose, the lines of best fit by using the least square technique is developed and the calculations given the highest correlation coefficient in case of X = 0.48 with slope as 32.

7. Accordingly, for developing the routing equation, the values of K and X are taken as

$$K = 32 \text{ and } X = 0.48$$

Routing Equation

As aready mentioned above, the routing equation is:

$$O_{t+1} = C_0 I_{t+1} + C_1 I_t + C_2 O_t$$

where, $C_0 = \dfrac{0.5\,\Delta t - KX}{K - KX + 0.5\,\Delta t}$

for Δt = 2 hours ;

K = 32 hours; and

Y = 0.48

$$C_0 = \frac{0.5 \times 2 - 32 \times 0.48}{32 - 32 \times 0.48 + 0.5 \times 2}$$

$$= \frac{1 - 15.36}{32 - 15.36 + 1} = \frac{-14.36}{17.64}$$

$$= -0.81405$$

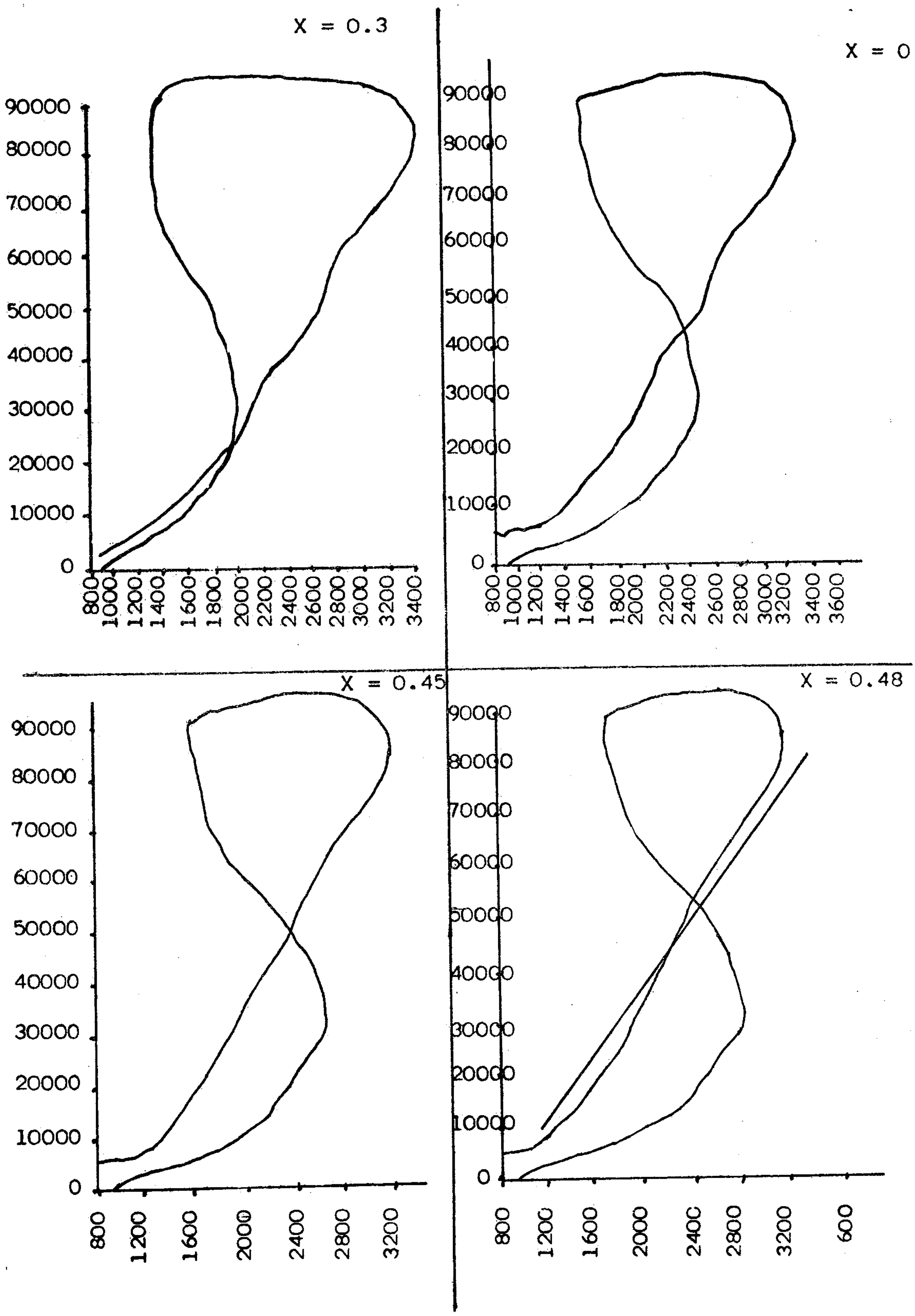

X = 0.3
90000
80000
70000
60000
50000
40000
30000
20000
10000
0
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
2800
3000
3200
3400
X = 0.4
90000
80000
70000
60000
50000
40000
30000
20000
10000
0
800
1000
1200
1400
1600
1800
2000
2200
2400
2600
2800
3000
3200
3400
3600
X = 0.45
90000
80000
70000
60000
50000
40000
30000
20000
10000
0
800
1200
1600
2000
2400
2800
3200
X = 0.48
90000
80000
70000
60000
50000
40000
30000
20000
10000
0
800
1200
1600
2000
2400
2800
3200
600

FIG. IV.3.40

$$C_1 = \frac{0.5\,t + KX}{K - KX + 0.5\,\Delta t}$$

$$= \frac{0.5 \times 2 + 32 \times 0.48}{32 - 32 \times 0.48 + 0.5 \times 2}$$

$$= \frac{16.36}{17.64}$$

$$= 0.92743$$

$$C_2 = \frac{K - KX - 0.5\Delta t}{K - KX + 0.5t}$$

$$= \frac{32 - 32 \times 0.48 - 0.5 \times 2}{32 - 32 \times 0.48 + 0.5 \times 2}$$

$$= \frac{15.64}{17.64}$$

$$= 0.88662$$

Therefore,

$$O_{t+1} = C_0 I_{t+1} + C_1 I_t + C_2 O_t$$

or $$O_{t+1} = -0.81405\, I_{t+1} + 0.92743\, I_t + 0.88662\, O_t$$

is the routing equation.

Computation of Discharge at Koelwar

By using the routing equation as derived above, the observed hydrograph at Japla has been routed to compute the discharge at Koelwar. The results are given in Col. 3. of the Table IV. 3.20 and shown in Fig. IV.3.41. However, the detailed calculations for routing are given below with a few data for illustration.

Table IV.3.16

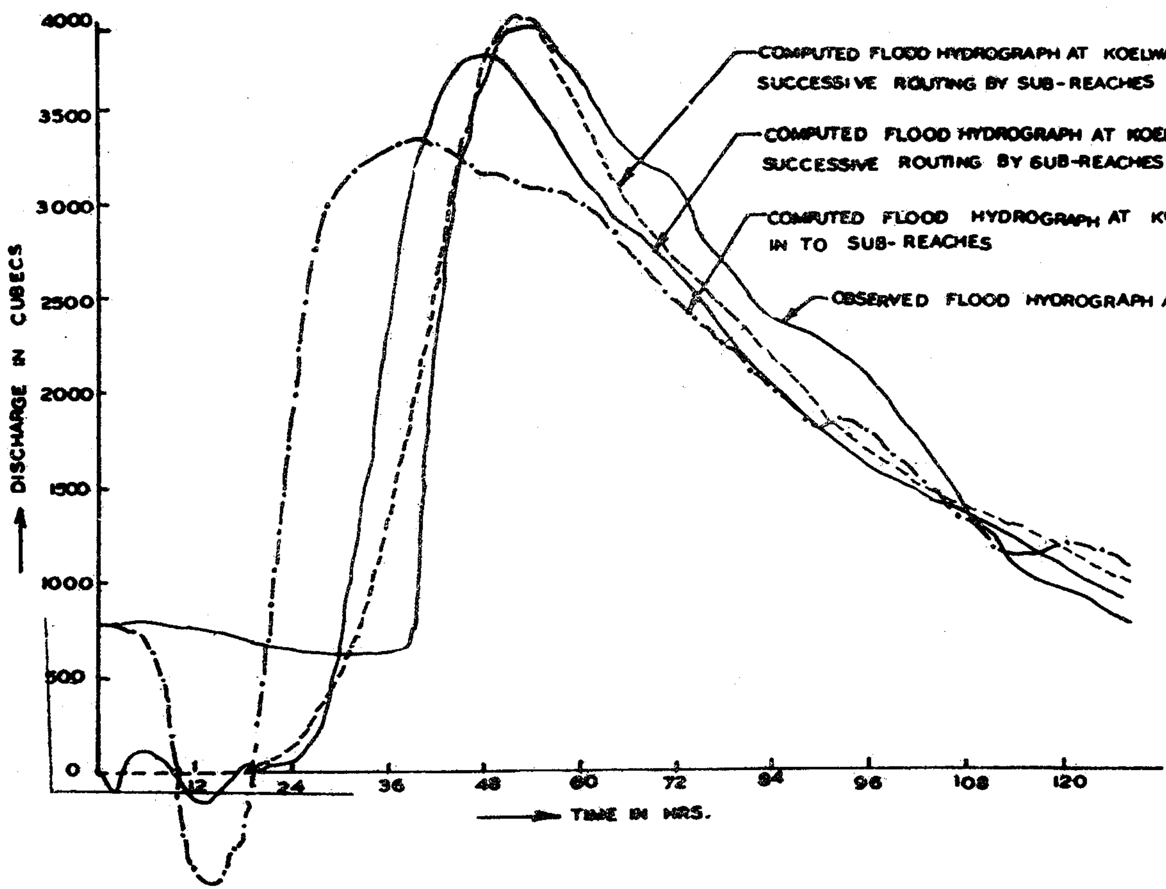
$C_o = -0.81405$, $C_1 = 0.92743$, $C_2 = 0.88662$

Date/Hour	*I*	$C_0 I_{t+1}$	$C_1 I_t$	$C_2 O_t$	O_{t+1}	*Observed O*
18.8.79						
1200	1825	-834	–	–	800	800
1400	1075	-875	951	709	785	800

Date/Hour	*I*	$C_0 I_{t+1}$	$C_1 I_t$	$C_2 O_t$	O_{t+1}	*Observed O*
1600	1120	-912	997	696	781	800
1800	1225	-997	1039	692	734	790
2000	1520	-1237	1136	651	550	780
2200	2455	1998	1410	487	-101	760
19.8.79						
0000	3325	-2707	2277	-89	-520	750
0200	3985	-3244	3084	-421	-621	750
0400	4380	-3566	3696	-551	-421	730
0600	5035	-4099	4062	-373	-410	710
0800	4880	-3973	4670	-363	334	690
1000	4625	-3765	4526	296	1057	600
1200	4180	-3403	4290	937	1824	640
1400	3615	-2943	3877	1617	2551	630
1600	3270	-2662	3353	2961	2952	620
1800	3110	-2532	3033	2610	3119	610
2000	3005	-2446	2884	2765	3203	610
2200	2905	-2365	2787	2840	3262	620

The above discussion explains the method of estimating the parameter K and X and to compute the discharge at a down stream station of a particular reach when discharge hydrograph at the upstream station and the parameters K and X of the reach are known.

It may be seen from Fig. IV. 3.41 that in the computed outflow hydrograph using Muskingum method, the flow decreases markedly during the initial period and then increases



	PEAK (M^3/SEC)	TIME OF PEAK (HRS)
1. OBSERVED HYDROGRAPH	3930	54
2. ROUTED HYDROGRAPH WITHOUT SUBREACHES	3332	42
3. ROUTED WITH 4 SUBREACHES	3763	48
4. ROUTED WITH 8 SUBREACHES	3939	54
5. ROUTED WITH 12 SUBREACHES	38 84	54
6. ROUTED WITH 16 SUBREACHES	3992	54

Fig-IV.3.41

rapidly afterwards. This is because of the fact that the value of K (=32 hours) is much larger than the routing period Δt which is only 2 hours for the reach under consideration.

To avoid such situations, the method of successive routing by sub-reaches is adopted. The method of successive routing through sub-reaches is discussed below.

4.3.4.2. Successive Routing Through Subreaches

For the purpose, the reach between the upstream station and the downstream station is divided into several reaches and the successive routing is carried out through each sub-reach.

The number of the sub-reaches should be so as to make the value of K close to that of the routing period Δt for each sub-reach.

The values of K and X for each sub-reach are evaluated in the following way:

Let the total reach of the river be divided into N sub-reaches and let the two parameters for the total reach be N and X.

Then the two parameters K_e and X_e for each of the sub-reaches will be

$$K_e = \frac{K}{N}$$

$$K_e = \frac{1}{2} - \frac{N(1-2X)}{2}$$

Once, K_e and X_e are estimated, a unit inflow is applied at zero time and the outflow hydrograph is evaluated by successively routing through all the N sub-reaches. The ordinates of this outflow hydrograph at interval of Δt give the flow-concentration coefficients 'P'. Knowing the flow concentration co-efficient 'P', flood hydrograph can be easily computed.

The procedure is explained with the help of the same problem which has been discussed earlier.

Example

Carry out the successive routing through sub-reaches for the data given in the previous example

$K = 32$ Hrs.

$X = 0.48$

$\Delta t = 2$ Hrs.

In order that K_e and Δt are almost equal, the total reach is divided into 16 sub-reaches i.e. N = 16.

$$K_e = \frac{K}{N} = \frac{32}{16} = 2$$

$$X_e = \frac{1}{2} - \frac{N(1-2X)}{2}$$

$$X_e = \frac{1}{2} - \frac{16(1-2\times 0.48)}{2}$$

$$= 0.5 - 8 \times 0.04$$

$$= 0.5 - 0.32$$

$$= 0.18$$

Calculation of Co-efficient C_o, C_1 and C_2 for each sub reach.

$$C_e = \frac{0.5\times 2-2\times 0.18}{2-2\times 0.18+0.5\times 2}$$

$$= \frac{1-0.36}{2-0.36+1} = \frac{0.64}{2.64} = 0.24242$$

$$C_1 = \frac{0.5\times 2+2\times 0.18}{2-2\times 0.18+0.5\times 2}$$

$$= \frac{1+0.36}{2-0.36+1} = \frac{1.36}{2.64} = 0.51515$$

$$C_2 = \frac{2-2\times 0.18-0.5\times 2}{2-2\times 0.18+0.5\times 2}$$

$$= \frac{0.64}{2.64} = 0.24243$$

Computation of flow-concentration co-efficients:

The Table IV.3.17 is self-explanatory and it explains the procedure for computation of flow concentration coefficients. Detailed calculations are given for sub-reach 1 and 2 and only the final results i.e. the outflow after routing through various reaches are tabulated.

Computation of outflow hydrograph at Koelwar :

The details of the computations of outflow hydrograph at Koelwar with the help of the flood hydrograph at Japla and the flow concentration coefficients are given in table IV.3.18 and the computed hydrograph for Koelwar by using this method is shown in Fig. IV.3.41.

TABLE 6·28

$C_0 = 0.24242$ $C_1 = 0.51515$, $C_2 = 0.24243$

t	I	1st Sub reach				2nd Sub reach				O_2 after routing through 3rd sub-reach	O_2 after routing through 4th sub-reach	O_2 after routing through 5th sub-reach	O_2 after routing through 6th sud-reach	O_2 after routing through 7th sub-reach	O_2 after routing through 8th sub-reach	O_2 after routing through 9th sub-reach	O_2 after routing through 10ih sub-roach	O_2 after routing through 11th sub-reach	O_2 after routing through 12th sub-reach	O_2 after routing through 13th sub-reach	O_2 after routing through 14th sub- each	O_2 after routigh through 15th sub-reach	O_2 after routing through 16th sub-reach
		C_0I_2	C_1I_1	C_2O_1	O_2	C_0I_2	C_1I_1	C_2O_1	O_2														
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
0	1	0.24242			0.24242	0.058767			0.058767	0.014247	0.003454	0.000836	0.000204	0.000048	0.000012	0.000002	0	0	0	0	0	0	0
1			0.51515	0.058769	0.57392	0.139120	0.124880	0.014246	0.278256	0.101185	0.032709	0.009910	0.002884	0.000813	0.000225	0.000061	0.000015	0.000002	0	0	0	0	0
2				0.139135	0.139135	0.033729	0.295650	0.067457	0.396836	0.264078	0.124076	0.049332	0.017763	0.005986	0.001924	0.000599	0.000180	0.000049	0.000012	0.000002	0	0	0
3				0.033731	0.033731	0.008177	0.071675	0.096205	0.176050	0.311133	0.241545	0.134434	0.062307	0.025705	0.009785	0.003505	0.001204	0.000396	0.000126	0.000037	0.000010	0	0
4				0.008177	0.008177	0.001982	0.017376	0.042681	0.062039	0.181165	0.262756	0.220718	0.137864	0.071751	0.033010	0.013890	0.005467	0.002040	0.000731	0.000253	0.000084	0.000019	0.000005
5				0.001982	0.001982	0.000481	0.004212	0.015040	0.019733	0.080663	0.176583	0.231676	0.203290	0.137695	0.078344	0.039364	0.018022	0.007681	0.003090	0.001186	0.000438	0.000153	0.000048
6				0.000481	0.000481	0.000117	0.001021	0.004784	0.005922	0.031154	0.091912	0.169411	0.209702	0.188942	0.135727	0.082803	0.044718	0.021986	0.010036	0.004313	0.001764	0.000690	0.000256
7				0.000117	0.000117	0.000028	0.000248	0.001436	0.001712	0.011017	0.041001	0.098357	0.161954	0.193092	0.177046	0.132916	0.085718	0.049148	0.025676	0.012443	0.005664	0.002446	0.001011
8							0.000060	0.000415	0.000475	0.003668	0.016506	0.048968	0.101804	0.154918	0.179947	0.167054	0.129751	0.087528	0.052763	0.029036	0.014821	0.007101	0.003229
9									0.000115	0.001175	0.006178	0.021875	0.055209	0.103383	0.148493	0.169197	0.158529	0.126491	0.088546	0.055685	0.032051	0.017126	0.008593
10									0.000028	0.000375	0.002196	0.009020	0.026837	0.060011	0.103803	0.142676	0.160184	0.151163	0.123272	0.088999	0.058029	0.034730	0.019327
11									0.000007	0.000106	0.000749	0.003502	0.012001	0.031284	0.063662	0.103494	0.137423	0.152476	0.144719	0.120162	0.089046	0.059901	0.037098
12										0.000026	0.000246	0.001296	0.005028	0.014985	0.035181	0.066413	0.102727	0.132661	0.145794	0.139026	0.117191	0.088805	0.061380
13										0.000006	0.000077	0.000462	0.002000	0.006709	0.017878	0.038558	0.068468	0.101681	0.128334	0.139917	0.133951	0.114372	0.088356
14											0.000024	0.000158	0.000761	0.002841	0.008479	0.020614	0.041469	0.069976	0.100457	0.124383	0.134706	0.129388	0.111705
15											0.000006	0.000051	0.000279	0.001148	0.003797	0.010285	0.012084	0.043942	0.071056	0.099132	0.120762	0.130038	0.125260
16												0.000018	0.000098	0.000447	0.001622	0.004838	0.005949	0.025512	0.046046	0.071803	0.097749	0.117431	0.125824
17												0.000004	0.000034	0.000167	0.000666	0.002170	0.002786	0.013850	0.027662	0.047835	0.072282	0.096347	0.114355
18													0.000008	0.000058	0.000264	0.000934	0.001247	0.007099	0.015562	0.029618	0.049349	0.072556	0.094948
19													0.000002	0.000019	0.000100	0.000385	0.000534	0.003461	0.008269	0.017201	0.031392	0.050622	0.072669
20														0.000005	0.000034	0.000152	0.000219	0.001612	0.004177	0.009444	0.018759	0.032991	0.051691
21														0.000001	0.000008	0.000055	0.000084	0.000717	0.002017	0.004932	0.010606	0.020234	0.034431
22															0.000002	0.000018	0.000031	0.000307	0.000935	0.002462	0.005707	0.011755	0.021619
23																0.000004	0.000007	0.000123	0.000415	0.001178	0.002937	0.006504	0.012875
24																		0.000048	0.000175	0.000543	0.001451	0.003443	0.007304
25																		0.000017	0.000073	0.000236	0.000688	0.001749	0.003967
26																		0.000004	0.000028	0.000101	0.000315	0.000855	0.002069
27																			0.000007	0.000042	0.000138	0.000401	0.001037
28																			0.000002	0.000015	0.000059	0.000184	0.000501
29																				0.000002	0.000025	0.000080	0.000234
30																					0.000006	0.000032	0.000105
31																					0.000001	0.000013	0.000043
32																						0.000003	0.000016
33																							0.000004

TABLE 6.29

Date	Time (Hrs)	I	P—	0	0	0	0	0.000005	0.000048	0.000256	0.001011	0.003229	0.008593	0.019327	0.037098	0.061380	0.088356	0.111705	0.125260	0.125828	0.114355	0.094948	0.072669	0.051691	0.034431	0.021619	0.012875	0.007304	0.003967	0.002069	0.001037	0.000501	0.000234	0.000105	0.000043	0.000016	0.000004	TOTAL computed flood hydrograph	Observed flood hydrograph
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
18-8-79	1200	1025	0	0	–	–	–	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	0	800
	1400	1075	0	0	0	–	–	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	0	800
	1600	1120	0	0	0	0	–	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	0	800
	1800	1225	0	0	0	0	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	0	790
	2000	1520	0.000005	0	0	0	0	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	0	780
	2200	2455	0.000048	0	0	0	0	0	0.05	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	0.05	760
19-8-79	0000	3325	0.000256	0	0	0	0	0	0.05	0.26	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	0.32	750
	0200	3985	0.001011	0	0	0	0	0	0.05	0.27	1.04	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	1.37	750
	0400	4380	0.003229	0	0	0	0	0	0.06	0.29	1.09	3.31	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	4.75	730
	0600	5035	0.008593	0	0	0	0	0	0.07	0.31	1.13	3.47	8.81	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	13.81	710
	0800	4880	0.019327	0	0	0	0	0.01	0.12	0.39	1.24	3.62	9.24	19.81	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	34.43	690
	1000	4625	0.037098	0	0	0	0	0.01	0.16	0.63	1.54	3.96	9.62	20.78	38.03	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	74.73	660
	1200	4180	0.061380	0	0	0	0	0.02	0.19	0.85	2.48	4.91	10.53	21.65	39.88	62.91	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	143.42	640
	1400	3615	0.088356	0	0	0	0	0.02	0.21	1.02	3.36	7.93	13.06	23.68	41.55	65.98	90.56	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	247.98	630
	1600	3270	0.111705	0	0	0	0	0.03	0.24	1.12	4.03	10.74	21.10	29.38	45.45	68.74	94.98	114.50	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	390.30	620
	1800	3110	0.125260	0	0	0	0	0.02	0.23	1.29	4.43	12.87	28.57	47.45	56.39	75.19	98.96	120.08	128.39	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	573.87	610
	2000	3005	0.125824	0	0	0	0	0.02	0.22	1.25	5.09	14.14	34.24	64.26	91.08	93.30	108.24	125.11	134.65	128.97	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	800.57	610
	2200	2905	0.114355	0	0	0	0	0.02	0.20	1.18	4.93	16.26	37.64	77.02	123.36	150.69	134.30	136.84	140.29	135.26	117.21	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	1075.19	620
20-8-79	0000	2805	0.094948	0	0	0	0	0.02	0.17	1.07	4.68	15.76	43.27	84.65	147.84	204.09	216.91	169.79	153.44	140.92	122.93	97.32	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	1402.86	630
	0200	2710	0.072669	0	0	0	0	0.02	0.16	0.93	4.23	14.93	41.49	97.31	162.49	244.60	293.78	274.24	190.39	154.13	128.08	102.07	74.49	—	—	—	—	—	—	—	—	—	—	—	—	—	—	1783.77	650
	0400	2615	0.051691	0	0	0	0	0.01	0.15	0.84	3.65	13.50	39.74	94.32	186.79	268.84	352.10	371.42	307.51	191.25	140.08	106.34	78.12	52.98	—	—	—	—	—	—	—	—	—	—	—	—	—	2207.66	1450
	0600	2515	0.034431	0	0	0	0	0.01	0.14	0.80	3.31	11.67	35.92	89.39	181.04	309.05	387.00	445.14	416.49	308.90	173.82	116.31	81.39	55.57	35.29	—	—	—	—	—	—	—	—	—	—	—	—	2651.24	2400
	0800	2500	0.021619	0	0	0	0	0.01	0.14	0.77	3.14	10.56	31.06	80.79	171.58	299.53	444.87	489.27	499.16	418.36	280.74	144.32	89.02	57.89	37.01	22.16	—	—	—	—	—	—	—	—	—	—	—	3080.40	3100
	1000	2455	0.012875	0	0	0	0	0.01	0.13	0.74	3.04	10.64	28.10	69.87	155.07	283.88	431.18	562.43	548.64	501.41	380.23	233.10	110.46	63.32	38.56	23.24	13.20	—	—	—	—	—	—	—	—	—	—	3456.66	3400
	1200	2410	0.007304	0	0	0	0	0.01	0.13	0.72	2.94	9.70	26.72	63.20	134.11	256.57	408.65	545.12	630.68	551.11	455.70	315.70	178.40	78.57	42.18	24.21	13.84	7.49	—	—	—	—	—	—	—	—	—	3745.76	3650
	1400	2305	0.003967	0	0	0	0	0.01	0.13	0.69	2.84	9.38	25.82	60.11	121.31	221.89	369.33	516.64	611.27	633.52	500.87	378.37	241.62	126.90	52.34	26.48	14.42	7.85	4.07	—	—	—	—	—	—	—	—	3925.86	3850
	1600	2240	0.002069	0	0	0	0	0.01	0.12	0.67	2.74	9.06	24.96	58.08	155.37	200.71	319.41	466.93	579.33	614.02	575.78	415.87	289.59	171.87	84.53	32.86	15.77	8.18	4.26	2.12	—	—	—	—	—	—	—	3992.24	3930
	1800	2140	0.001037	0	0	0	0	0.01	0.12	0.64	2.64	8.75	24.10	56.14	111.48	190.89	288.92	403.81	523.59	581.94	558.05	478.06	318.29	205.99	114.48	53.07	19.57	8.95	4.44	2.22	1.06	—	—	—	—	—	—	3957.25	3920
	2000	2020	0.000501	0	0	0	0	0.01	0.12	0.64	2.54	8.44	23.29	54.11	107.77	184.45	274.79	365.28	452.81	525.94	528.89	463.35	365.89	226.41	137.21	71.88	31.61	11.10	4.86	2.32	1.11	0.51	—	—	—	—	—	3845.43	3870
	2200	1910	0.000234	0	0	0	0	0.01	0.12	7.63	2.53	8.12	22.47	52.38	104.06	178.31	265.51	347.40	409.60	454.85	478.00	439.13	354.62	260.26	150.81	86.15	42.81	17.93	6.03	2.53	1.16	0.54	0.24	—	—	—	—	3686.22	3790
21-8-79	0000	1800	0.000105	0	0	0	0	0.01	0.11	0.62	2.48	8.07	21.61	50.54	100.54	172.17	256.67	335.67	389.56	411.44	413.39	396.88	336.09	252.25	173.36	94.69	51.31	24.29	9.74	3.14	1.27	0.56	0.25	0.11	—	—	—	3506.84	3650
	0200	1715	0.000043	0	0	0	0	0.01	0.11	0.59	2.44	7.93	21.48	48.61	97.01	166.34	247.84	324.50	376.41	391.31	373.94	343.24	303.76	239.07	168.02	108.85	64.83	29.11	13.19	5.08	1.58	0.61	0.26	0.11	0.04	—	—	3327.83	3530
	0400	1630	0.000016	0	0	0	0	0.01	0.10	0.57	2.33	7.78	21.10	48.32	93.30	160.51	239.44	313.33	363.88	318.10	355.64	310.48	262.70	216.07	159.24	105.50	62.83	31.99	15.81	6.88	2.55	0.76	0.29	0.12	0.05	0.02	—	3161.69	3400
	0600	1570	0.000004	0	0	0	0	0.01	0.10	0.55	2.26	7.44	20.71	47.45	92.75	154.37	231.05	302.72	351.35	365.52	343.64	295.29	237.63	186.86	143.92	99.99	59.55	36.78	17.38	8.24	3.45	1.23	0.36	0.13	0.05	0.02	0	3014.06	3320
	0800	1540	0	0	0	0	0	0.01	0.09	0.52	2.16	7.23	19.81	46.58	91.08	153.45	222.22	292.11	339.45	352.94	332.20	285.32	226.00	169.03	124.47	90.37	53.82	35.64	19.97	9.06	4.13	1.67	0.57	0.16	0.05	0.02	0	2885.86	3220
	1000	1505	0	0	0	0	0	0.01	0.09	0.49	2.04	6.91	19.25	44.55	89.41	150.69	220.89	280.94	327.55	340.98	320.77	275.82	218.37	160.76	112.59	78.15	46.54	33.78	19.36	10.42	4.54	2.00	0.79	0.26	0.07	0.02	0	2775.29	3180
	1200	1475	0	0	0	0	0	0.01	0.08	0.46	1.93	6.52	18.39	43.29	85.51	147.93	216.91	279.26	315.03	329.03	309.90	266.33	211.10	155.33	107.08	70.69	42.10	30.53	18.35	10.10	5.22	2.19	0.93	0.35	0.11	0.02	0	2679.15	3150
	1400	1415	0	0	0	0	0	0.01	0.08	0.44	1.82	6.17	17.36	41.36	83.10	141.48	212.94	274.24	313.15	316.45	299.04	257.31	203.84	150.16	103.47	67.24	40.04	26.40	16.58	9.57	5.06	2.52	1.02	0.42	0.14	0.04	0.01	2593.50	2950
	1600	1360	0	0	0	0	0	0.01	0.07	0.42	1.73	5.81	16.41	39.04	79.39	137.49	203.66	269.21	307.51	314.56	287.60	248.29	196.93	144.99	100.02	64.97	38.69	23.88	14.34	8.65	4.80	2.44	1.18	0.46	0.17	0.05	0.01	2514.16	2850
	1800	1330	0	0	0	0	0	0.01	0.07	0.40	1.65	5.54	15.47	36.91	74.94	131.35	197.92	257.48	301.88	308.90	285.89	238.79	190.03	140.08	96.58	62.80	37.40	22.72	12.97	7.48	4.33	2.32	1.14	0.53	0.19	0.06	0.01	2437.13	2700
	2000	1305	0	0	0	0	0	0.01	0.07	0.39	1.59	5.26	14.74	34.79	70.86	123.59	189.68	250.22	288.72	303.24	280.74	237.37	182.76	135.17	93.31	60.64	36.11	21.95	12.34	6.77	3.75	2.09	1.08	0.51	0.22	0.07	0.02	2359.14	2600
	2200	1275	0	0	0	0	0	0.01	0.07	0.39	1.56	5.07	14.01	33.15	66.78	117.24	178.48	239.05	280.58	290.02	275.60	233.10	181.67	130.00	90.04	58.59	34.89	21.22	11.92	6.43	3.39	1.81	0.98	0.49	0.21	0.08	0.02	2278.04	2500
22-8-79	0000	1250	0	0	0	0	0	0.01	0.07	0.38	1.52	4.97	13.49	31.50	63.62	110.48	168.76	225.64	248.06	281.85	263.59	128.82	178.40	129.23	86.59	56.53	33.67	20.49	11.52	6.22	3.23	1.64	0.85	0.44	0.20	0.08	0.02	2193.09	2400
	0200	1235	0	0	0	0	0	0	0.06	0.36	1.49	4.86	13.23	30.34	60.47	105.27	159.04	213.36	253.03	269.26	256.16	218.86	175.13	126.90	86.08	54.37	32.38	19.79	11.13	6.01	3.12	1.56	0.76	0.38	0.18	0.07	0.02	2104.97	2380
	0400	1225	0	0	0	0	0	0	0.06	0.35	1.43	4.76	12.93	29.76	58.24	100.05	151.53	201.07	239.25	254.16	244.72	212.68	167.50	124.58	84.53	54.05	32.19	19.10	10.75	5.80	3.01	1.51	0.73	0.34	0.16	0.07	0.02	2015.53	2320
	0600	1225	0	0	0	0	0	0	0.06	0.34	1.37	4.57	12.67	29.09	57.13	96.37	144.02	191.57	225.47	240.32	231.00	203.19	162.78	119.15	82.98	53.07	31.61	18.37	13.37	5.61	2.91	1.46	0.70	0.33	0.14	0.06	0.02	1927.31	2280
	0800	1085	0	0	0	0	0	0	0.06	0.33	1.34	4.39	12.16	28.51	55.83	94.53	138.72	182.08	214.82	226.48	218.42	191.79	155.51	115.79	79.36	52.10	31.03	18.26	9.98	5.41	2.81	1.41	0.68	0.32	0.13	0.05	0.01	1842.91	2220
	1000	995	0	0	0	0	0	0	0.06	0.33	1.32	4.29	11.69	27.35	54.72	92.38	136.07	175.38	204.17	215.79	205.84	181.35	146.79	110.62	77.13	49.83	29.68	17.93	9.92	5.20	2.71	1.36	0.66	0.31	0.13	0.05	0.01	1764.41	2150
	1200	950	0	0	0	0	0	0	0.06	0.32	1.29	4.21	11.43	26.28	52.49	90.54	132.58	172.03	196.66	205.09	196.12	170.91	138.80	104.42	73.68	48.43	28.84	17.60	9.94	5.17	2.61	1.31	0.63	0.29	0.12	0.05	0.01	1692.95	2250
	1400	315	0	0	0	0	0	0	0.06	0.32	1.26	4.12	11.21	25.70	50.45	86.85	130.33	168.12	192.90	197.54	186.40	162.84	130.80	98.73	69.55	46.26	27.55	16.84	9.56	5.08	2.59	1.26	0.61	0.28	0.12	0.05	0.01	1628.70	1960
	1600	885	0	0	0	0	0	0	0.06	0.31	1.25	4.04	10.96	25.22	49.34	83.48	125.02	164.76	188.52	193.77	179.54	154.77	124.63	93.04	65.76	43.67	26.01	16.36	9.14	4.99	2.55	1.25	0.59	0.27	0.12	0.04	0.01	1571.02	1850
	1800	865	0	0	0	0	0	0	0.05	0.31	1.24	3.99	10.74	24.64	48.41	81.64	120.16	158.06	184.76	189.37	176.11	149.07	118.45	88.65	61.68	41.29	24.59	15.63	8.89	4.77	2.50	1.23	0.59	0.26	0.11	0.04	0.01	1518.96	1750
	2000	875	0	0	0	0	0	0	0.05	0.28	1.24	3.96	10.61	24.16	47.30	80.10	117.51	151.92	177.24	185.59	172.10	146.22	114.09	84.26	59.05	38.91	23.18	14.75	8.49	4.63	2.39	1.21	0.57	0.26	0.11	0.04	0.01	1471.66	1630
	2200	895	0	0	0	0	0	0	0.05	0.25	1.10	3.96	10.53	23.87	46.37	78.26	115.30	148.57	170.35	178.04	168.67	142.90	111.91	81.15	56.12	37.08	22.08	13.95	8.01	4.43	2.32	1.15	0.56	0.25	0.11	0.04	0.01	1428.51	1500
23-8-79	0000	915	0	0	0	0	0	0	0.04	0.24	1.01	3.50	10.53	23.68	45.82	76.72	112.65	145.78	166.60	171.12	161.81	140.05	109.37	79.60	54.06	35.24	20.99	13.15	7.58	4.18	2.22	1.12	0.54	0.24	0.11	0.04	0.01	1389.08	1380
	0200	950	0	0	0	0	0	0	0.04	0.23	0.96	3.21	8.55	23.68	45.45	75.80	110.45	142.42	163.46	167.35	155.52	134.35	107.19	77.79	53.02	33.94	20.21	12.53	7.14	3.95	2.09	1.07	0.52	0.24	0.10	0.04	0.01	1352.89	2280
	0400	980	0	0	0	0	0	0	0.04	0.23	0.93	3.07	8.16	20.97	45.45	75.19	109.12	139.63	159.71	164.20	152.09	129.13	102.83	76.24	51.82	33.29	19.83	11.91	6.80	3.72	1.98	1.01	0.50	0.22	0.10	0.04	0.01	1319.01	1200
	0600	1025	0	0	0	0	0	0	0.04	0.22	0.89	2.95	7.86	19.23	40.25	75.19	108.24	137.96	156.58	160.42	149.23	126.28	98.83	73.14	50.79	32.54	19.38	11.47	6.47	3.55	1.87	0.96	0.47	0.21	0.10	0.04	0.01	1285.93	1100
	0800	995	0	0	0	0	0	0	0.04	0.22	0.87	2.86	7.60	18.36	36.91	66.60	108.24	136.84	154.70	157.28	145.80	123.91	96.65	70.30	48.72	31.89	18.99	11.25	6.23	3.37	1.78	0.90	0.45	0.20	0.09	0.04	0.01	1251.76	1000
	1000	950	0	0	0	0	0	0	0.04	0.23	0.88	2.79	7.43	17.68	35.24	61.07	95.87	136.84	153.44	155.39	142.94	121.06	94.83	68.75	46.83	30.59	18.22	10.99	6.11	3.25	1.69	0.86	0.42	0.19	0.09	0.03	0.01	1214.73	1040
	1200	865	0	0	0	0	0	0	0.04	0.23	0.90	2.83	7.52	17.10	33.94	58.31	87.91	121.20	153.44	154.13	141.38	118.69	92.65	67.46	45.79	29.40	17.51	10.77	5.97	3.19	1.63	0.82	0.40	0.18	0.08	0.03	0.01	1174.02	970
	1400	845	0	0	0	0	0	0	0.04	0.24	0.93	2.89	7.69	16.72	32.83	56.16	83.94	111.15	135.91	154.13	140.08	117.26	90.84	65.91	44.93	28.75	17.12	10.34	5.85	3.11	1.60	0.79	0.38	0.17	0.08	0.03	0.01	1130.11	930
	1600	825	0	0	0	0	0	0	0.04	0.25	0.96	2.95	7.86	16.91	32.09	54.32	80.85	106.12	124.65	136.52	140.08	116.31	89.75	63.84	43.90	28.21	16.80	9.93	5.61	3.05	1.56	0.77	0.37	0.16	0.07	0.03	0.01	1084.92	880
	1800	805	0	0	0	0	0	0	0.04	0.26	0.99	3.07	8.16	17.30	32.46	53.09	78.20	102.21	119.00	125.19	124.08	116.31	89.02	63.32	43.04	27.56	16.42	9.71	5.40	2.93	1.53	0.75	0.36	0.16	0.07	0.03	0.01	1041.28	830

$C_o = -0.09890$, $C_1 = 0.64835$, C_2 - 0.45054

TABLE 6.30

t	I	1st Sub reach				2nd Sub reach				O_2 after routing through 3rd sub-reach	O_2 after routing through 4th sub-reach	O_2 after routing through 5th sub-reach	O_2 after routing through 6th sud-reach	O_2 after routing through 7th sub-reach	O_2 after routing through 8th sub-reach
		C_oI_2	C_1I_1	C_2O_1	O_2	C_oI_2	C_1I_1	C_2O_1	O_2						
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
0	1	−0.098900			−0.098900	0.009781			0.009781	−0.000967	0.000096	−0.000009	0.0000009	0	0
1			0.648350	−0.044558	0.603792	−0.059715	−0.064122	0.004407	−0.119430	0.017717	−0.002336	0.000289	−0.000034	0.000004	0
2				0.272032	0.272032	−0.026904	0.391469	−0.053808	0.310756	−0.100184	0.020343	−0.003397	0.000508	−0.000071	0.000009
3				0.122561	0.122561	−0.012121	0.176372	0.140008	0.304259	0.126251	−0.068275	0.018411	−0.003794	0.000673	−0.000108
4				0.055219	0.055219	−0.005460	0.079462	0.137081	0.211082	0.233271	0.028023	−0.038743	0.014059	−0.003547	0.000738
5				0.024878	0.024878	−0.002460	0.035801	0.095101	0.128442	0.229250	0.141194	−0.013250	−0.017474	0.409245	−0.002882
6				0.011209	0.011209	−0.001109	0.016130	0.057868	0.072889	0.179353	0.194510	0.066336	−0.023024	−0.004887	0.005179
7				0.005050	0.005050	−0.000499	0.004712	0.032839	0.039607	0.124146	0.191640	0.137045	0.019082	−0.019017	0.001046
8				0.002275	0.002275	−0.000225	0.003274	0.017845	0.020894	0.079546	0.158965	0.170272	0.080610	−0.004168	−0.011446
9				0.001025	0.001025	−0.000101	0.001475	0.009414	0.010787	0.048318	0.118415	0.168068	0.130092	0.037520	−0.011570
10				0.000462	0.000462	−0.000046	0.000665	0.004860	0.005479	0.028221	0.081887	0.144397	0.153298	0.086088	0.010599
11				0.000208	0.000208	−0.000021	0.000299	0.002469	0.002747	0.015995	0.053609	0.112846	0.151526	0.123191	0.048407
12				0.000094	0.000094	−0.000009	0.000135	0.001238	0.001363	0.008853	0.033648	0.082271	0.133296	0.140561	0.087779
13				0.000042	0.000042	−0.000004	0.000061	0.000614	0.000671	0.004806	0.020424	0.056862	0.107772	0.139092	0.116925
14				0.000019	0.000019	−0.000002	0.000027	0.000302	0.000328	0.002568	0.012064	0.037668	0.081697	0.124461	0.130550
15				0.000009	0.000009	−0.000000	0.000012	0.000148	0.000159	0.001354	0.006966	0.024104	0.058846	0.103223	0.129303
16							0.000006	0.000072	0.000077	0.000705	0.003947	0.014986	0.040658	0.080637	0.117206
17								0.000035	0.000037	0.000364	0.002199	0.009093	0.027135	0.060007	0.099152
18								0.000017	0.000018	0.000186	0.001208	0.005403	0.017586	0.042889	0.079336
19								0.000008	0.000008	0.000095	0.000656	0.003153	0.011115	0.029626	0.060621
20								0.000004	0.000004	0.000048	0.000352	0.001811	0.006873	0.019874	0.044555
21								0.000002	0.000002	0.000024	0.000187	0.001026	0.004169	0.012998	0.031674
22										0.000012	0.000099	0.000574	0.002487	0.008313	0.021875
23										0.000006	0.000052	0.000317	0.001461	0.005213	0.014730
24										0.000003	0.000027	0.000174	0.000847	0.003212	0.009699
25											0.000014	0.000094	0.000485	0.001948	0.006260
26											0.000007	0.000051	0.000275	0.001165	0.003968
27											0.000004	0.000027	0.000154	0.000688	0.002475
28											0.000002	0.000014	0.000085	0.000401	0.001521
29												0.000008	0.000047	0.000232	0.000923
30												0.000004	0.000026	0.000132	0.000553
31												0.000002	0.000014	0.000075	0.000327
32													0.000008	0.000042	0.000192
33													0.000004	0.000023	0.000111
34													0.000002	0.000013	0.000064
35														0.000007	0.000037
36														0.000004	0.000021
37														0.000002	0.000012
38															0.000006
39															0.000004

It may be seen that a lot of computation is involved in estimating the flow concentration coefficient as well as in computing the flood hydrograph when the number of sub-reaches is 16. Hence an attempt has been made to see the variation in the accuracy of the result with lesser number of sub-reaches. Accordingly, the whole computation has been repeated with various number of sub-reaches i.e. N as 12, 8 and 4 also.

Table IV.3.19 gives the details of computations of flow concentration coefficients with N = 8, it indicates that for practical purposes about 50% of the computational work has been reduced.

The finally computed outflows in different cases are tablulated in Table IV.3.20.

TABLE IV. 3.20

Comparison of the Observed Discharges and Computed Discharges by Various Methods

Sl. No.	*Observed discharge at Koelwar (m^3/sec)*	*Routed discharge at Koelwar using Muskingum method without sub-reaches*	*Routed discharge at Koelwar using successive routing through Four sub-reaches*	*Routed discharge using successive routing through Eight Sub-reaches*	*Routed discharge at Koelwar using successive routing through Twelve sub-reaches*	*Routed discharge at Koelwar using successive routing through Sixteen sub-reaches*
1	2	3	4	5	6	7
1	800	800	31	0	0	0
2	800	785	-118	0	0	0
3	800	781	51	0	0	0
4	790	734	126	0	0	0
5	780	550	73	0	0	0
6	760	-101	-32	-2	0	0
7	750	-520	-188	3	0	0
8	750	-521	-175	4	0	1

Sl. No.	Observed discharge at Koelwar (m^3/sec)	Routed discharge at Koelwar using Muskingum method without sub-reaches	Routed discharge at Koelwar using successive routing through Four sub-reaches	Routed discharge using successive routing through Eight Sub-reaches	Routed discharge at Koelwar using successive routing through Twelve sub-reaches	Routed discharge at Koelwar using successive routing through Sixteen sub-reaches
1	2	3	4	5	6	7
9	730	–421	–66	–7	2	5
10	710	–410	41	–19	9	14
11	690	334	–21	–10	26	34
12	660	1057	36	41	65	75
13	640	1824	17	133	136	143
14	630	2551	113	250	244	247
15	620	2952	308	382	389	390
16	610	3119	665	546	572	574
17	610	3203	1188	760	795	801
18	620	3202	1795	1040	1068	1075
19	630	3303	2385	1398	1399	1403
20	650	3304	2887	1829	1791	1784
21	1450	3332	3271	2309	2228	2208
22	2400	3332	3587	2796	2684	2651
23	3100	3251	3681	3240	3120	3080
24	3400	3203	3755	3592	3494	3457

Sl. No.	*Observed discharge at Koelwar (m^3/sec)*	*Routed discharge at Koelwar using Muskingum method without sub-reaches*	*Routed discharge at Koelwar using successive routing through Four sub-reaches*	*Routed discharge using successive routing through Eight Sub-reaches*	*Routed discharge at Koelwar using successive routing through Twelve sub-reaches*	*Routed discharge at Koelwar using successive routing through Sixteen sub-reaches*
1	2	3	4	5	6	7
25	3650	3155	3763	3824	3773	3746
26	3850	3156	3726	3935	3936	3926
27	3930	3112	3634	3940	3984	3992
28	3920	3094	3526	3886	3935	3957
29	3870	3084	3408	3743	3817	3845
30	3700	3053	3286	3594	3658	3685
31	3650	3013	3170	3436	3484	3507
32	3530	2945	3062	3280	3313	3228
33	3400	2874	2970	3134	3154	3162
34	3320	2782	2884	3001	3011	3014
35	3220	2669	2801	2884	2886	2886
36	3180	2570	2725	2782	2778	2775
37	3150	2473	2644	2691	2683	2679
38	2950	2409	2580	2608	2598	2594
39	2850	2341	2463	2529	2518	2514
40	2700	2254	2556	2450	2441	2437

Sl. No.	Observed discharge at Koelwar (m^3/sec)	Routed discharge at Koelwar using Muskingum method without sub-reaches	Routed discharge at Koelwar using successive routing through Four sub-reaches	Routed discharge using successive routing through Eight Sub-reaches	Routed discharge at Koelwar using successive routing through Twelve sub-reaches	Routed discharge at Koelwar using successive routing through Sixteen sub-reaches
1	2	3	4	5	6	7
41	2600	2170	2263	2367	2362	2359
42	2500	2096	2157	2281	2279	2278
43	2400	2025	2067	2190	2193	2193
44	2380	1948	1980	2097	2103	2105
45	2320	1875	1899	2005	2013	2015
46	2280	1802	1819	1916	1925	1927
47	2220	1850	1767	1835	1841	1843
48	2150	1837	1690	1760	1763	1764
49	2050	1778	1609	1692	1693	1693
50	1960	1712	1549	1630	1629	1629
51	1850	1646	1509	1573	1572	1571
52	1750	1576	1480	1521	1520	1519
53	1630	1488	1450	1472	1472	1472
54	1500	1402	1421	1428	1429	1429
55	1380	1328	1390	1390	1390	1353
56	1280	1253	1248	1357	1354	1353

Sl. No.	*Observed discharge at Koelwar (m^3/sec)*	*Routed discharge at Koelwar using Muskingum method without sub-reaches*	*Routed discharge at Koelwar using successive routing through Four sub-reaches*	*Routed discharge using successive routing through Eight Sub-reaches*	*Routed discharge at Koelwar using successive routing through Twelve sub-reaches*	*Routed discharge at Koelwar using successive routing through Sixteen sub-reaches*
1	2	3	4	5	6	7
57	1200	1194	1302	1327	1320	1319
58	1100	1133	1245	1295	1288	1286
59	1040	1145	1197	1258	1254	1252
60	1000	1165	1139	1217	1216	1215
61	970	1210		1170	1174	1174
62	930	1187		1122	1128	1130
63	880	1164	982	1074	1082	1085
64	850	1142	943	1030	1038	1041
65	820	1088	928	992	999	1002

Comparison of Observed Discharges and Computed Discharges for Various Cases:

The observed discharge and the computed discharge for various cases at different time are tabulated in table IV.3.20 and are plotted in Fig. IV.3.41 It may be seen that the results obtained by using the method of successive routing through sub-reaches are definitely better than that in case of routing by Muskingum method.

4.3.4.3 Muskingum—Cunge Method of Flood Routing

The most commonly used Muskingum method of streamflow routing is primarily a hydrologic flow routing method where the parameters are estimated by using the past observed data. In case the flood magnitude is higher than the observed records, the estimated parameters may not give the correct results. Cunge suggested an improvement in the Muskingum method of flood routing by relating the parameters K and X of the method with the flow and channel characteristics. He could arrive at this relationship by arguing that the one to one relationship between the stage and the discharge does not lead to attenuation and the attenuation obtained in the conventional Muskingum method is due to numerical diffusion. Cunge related the numerical diffusion of the Muskingum scheme with the physical diffusion of the linear convective diffusion scheme. This enabled the relationship of K and X with the channel and flood characteristics. With introduction of these features, the forecasting capability of the Muskingum-Cunge method is improved.

4.3.5 Inflow Forecast

Inflow forecasts are very important for flood control operations of a reservoir system. A long range inflow forecast for a reservoir will also help in better utilization of the available water resources. In this case, the total amount of the runoff as well as its time distribution is to be forecast and for operational purposes the expected rise/fall in the reservoir level due to incoming flood and/or releases is forecast.

Basic principles for inflow forecasts are almost same as that for forecasting methods based on formation and propagation of floods.

A beginning has been made in India with introduction of inflow forecasting for a few dams in Damodar river basin and also for a few storage reservoirs in Godavari and Krishna basins. The procedure followed for inflow forecasts at these places is as follows:

(a) Estimation of total depth of runoff with the help of the observed rainfall data from several stations in the basin. Rainfall-Runoff correlation diagrams have been developed for the purpose.

(b) Time distribution of the runoff by unit hydrograph method.

The procedure followed for inflow forecast for a particular reservoir in China is briefly described below which illustrates the basic principles of inflow forecast.

(i) The total catchment area is divided into a reasonable number of sub-areas, each measuring approximately 1000 sq.kms. The reservoir area is treated separetely. Thiessen's polygon method is used for computation of the daily average rainfall.

(ii) Calculation of Runoff Volume R for each single flood. The formula used are:

$$\frac{f}{F} = 1 - \left(1 - \frac{W'}{W'_m}\right)$$

when $$P + a < W'_m ; R = P - W_m \left[\left(1 - \frac{a}{W'_m}\right)^{1+b} - \left(1 - \frac{P+a}{W'_m}\right) \right] \quad \text{(ii)}$$

when $$P + a \geq W'_m ; R = P - (W_m - W_o) \quad \text{(iii)}$$

$$a = W'_m \left[1 - \left(1 - \frac{W_o}{W_m}\right)^{\frac{1}{1+b}} \right] \quad \text{(iv)}$$

$$E_m = V \left(0.0286 + 0.1625 \frac{S_\omega}{S_o} + 0.0013\,T + 0.00975 \frac{S_\omega}{S_o} T \right) \quad \text{(v)}$$

$$E = \frac{W_o}{W_m} \cdot E_m \quad \text{(vi)}$$

Wherein E = Evaporation; V = External radiation in mm on top surface

S_o = max. probable sunshine duration

S_ω = Observed sunshine duration (actual).

T = Daily mean temperature (observed)

E_m = Potential evaporation.

Based on the observed data, b = 0.3 and W_m = 80 mm.

(a) S_o, V, T and Sw are observed values.

(b) "E_m" is calculated by formula (v)

(c) "E" is calculated by formula (vi)

(d) "a" is calculated by formula (iv)

(e) W_1 = Moisture storage on the Ist day and W_2 (Initial Moisture storage for next day) is calculated by the formula $W_2 = W_1 + P - E - R$;

(f) Then, total runoff 'R' is separated into R_s (Surface runoff) and R_g (Ground water runoff)

(g) "f_c" Constant infiltration rate for the whole area is calculated on the basis of observed data, f_c = 0.8 mm per hour.

(iii) Computation of Outflow Hydrograph of the sub-areas.

Generally Unit Hydrograph method is used and the unit hydrograph of a representative watershed is transposed to the sub area. The representative watershed chosen is such that it is one with observed hydrograph data, its physiographic and geologic conditions and size of the area are similar to that of the sub-area and is situated in the watershed in question or in an adjacent watershed. In that case, it can be assumed that the dimensionless unit hydrograph of surface runoff over the individual sub-areas, i.e. the flow concentration co-efficients are all the same, when the ordinates of the empirical unit hydrograph, expressed in m^3/sec, are direct proportional to the drainage area.

(iv) Flow Concentration Computation for the whole area.

The flow concentration in the river network ranges from the outlet of a given sub-area to that of the watershed concerned, and as computations are to be performed accordingly. It is appropriate to use the successive routing through sub-reaches for such kind of computations, and the method of flow concentration co-efficients can be used. The parameters K and x in the Muskingum method used for the computation may be obtained using the analytical method.

(v) Calcluation of Reservoir Inflow.

The reservior inflow hydrograph, Q_s obtained by the flow concentration computation discussed above is only due to runoff from the total area. In order to compute the total inflow the following are also considered:

1. Ground water runoff of the sub-areas, Q_g,

2. Inflow of the antecedent recession, Q_a, and

3. The rainfall amount falling over the reservior surface.

For the purpose of calculating the ground water runoff the routing is not carried out for each such area, but for the entire watershed as à whole. Thus, the relation $W_g = KQ_g$ of the entire watershed is applicable. In this case also, the Muskingum method is adopted to perform successive routing through the sub-reaches.

The inflow to the antecedent recession Q_a is taken as $Q_a = f\,(T)$.

There is hardly any loss for the rainfall falling on the reservoir surfaces, and the travel time of this part of rainfall can be approximately regarded as zero. Therfore, the rainfall R_s, falling on the reservoir surfaces may be directly taken as the increment to the reservoir inflow.

The inflow to Reservoir = $Q = Q_s + Q_g + Q_a + R_s$

CHAPTER-V

CATCHMENT MODELS AND CHOICE OF MODELS

5. Introduction

5.1 General

The distribution of rainfall both in time and space is subject to a great degree of variation. The analysis leading to the forecast of floods or determination of yield from a catchment as such requires large amount of relevant data for performing such an analysis. The advent of digital computer providing the facilities of carrying out vast numbers of iterative calculations involving large volume of data and the ability to answer yes or no to specifically designed interrogations has simulated the building of mathematical programs describing the land phase of hydrological cycle in space and time.

5.1.1 Principle of a model

In principle, mathematical model is a simplified and logical representation of the complex natural system of hydrological cycle. The model is framed as a set of mathematical expressions and logical statements combined together to represent the behaviour of the catchment and river channel system. Development and application of such models have increased tremendously during the last two decades, so that engineering and operational hydrology today most often involves consideration of some kind of mathematical model or other. With the current rapid development in computer technology and sophisticated techniques of automatic data collection and transmission system, the application of computer based mathematical models are increasing day by day. In order to be able to categorise the numerous mathematical models existing today, some definitions are given in the following paras:

5.1.2 Model

A model can be described as an artificial representation of a natural or a man-made system, to simulate the system behaviours and to predict the likely future behaviour of the system, when the input is known. A model can be of the following types:

1. Physical Model
2. Analogue Model
3. Mathematical Model

5.1.2.1 Physical Model

It is a true representation of the prototype in a reduced scale. These models are very much useful when true representation of any system is very much complicated by other means. However, such modelling is difficult and cumbersome and therefore they are not used often.

5.1.2.2 Analogue Model

Analogue model is a representation of the system by analogous means say through a net work of resistors and capacitors. In a water resources problem current flow in model represents flow of water in prototype.

5.1.2.3 Mathematical model

Mathematical Model is a set of mathematical expressions and logical statements combined together to simulate the behaviour of a given system. Mathematical models can be categorised in various ways.

1)	a)	Hydraulic	b)	Hydrologic		
2)	a)	Conceptual	b)	Statistical	c)	Black box
3)	a)	Deterministic	b)	Stochastic		
4)	a)	Time variant	b)	Time invariant		
5)	a)	Lumped	b)	Distributed		

Although all the models cannot be put into one definite class, the classification is general with respect to fundamental principles. The model may pertain to combination of two or more types.

5.1.2.4 Hydrologic Model and Hydraulic Model

Hydrological Model

Hydrologic model is a simplified description of the Hydrological cycle representing a qualitative description of the various forms of water flow in nature. The processes involved in the transformation of rainfall into Evaporation, River Discharge and sub-surface runoff are many and the search for the quantitative connection of these processes under normal as well as extreme condition, is one of the main tasks within the modelling activities in Hydrology. The term is often used as the computer based mathematical model for simulation of runoff from rainfall.

Hydraulic Model

Hydraulic model is the simplified description of river mechanics which simulates flow condition in the channel based on the formulation and solution of mathematical relationship/ expression by known hydraulic principles. The technique is based on the conservation of mass and momentum of unsteady flow equations of Saint Venant.

5.1.2.5 Conceptual, Statistical, Black box Model

A conceptual model follows the actual behaviour of the physical processes involved in the hydrologic cycle. The interaction between the different physical processes are explained through experimentally derived equations, empirically derived equations or mathematical representation of the actual process. In a conceptual model first of all the various physical processes involved are identified e.g. interception, evaporation, evapotranspiration, percolation, surface flow, inter flow, ground water flow etc. The efficiency of the model depends upon the proper identification of the physical processes in a particular system and their interactions.

In a statistical model the relation between the input and output or any other intermediate process is determined with the help of statistical correlations. For this purpose correlation coefficient between various inputs and outputs of the physical processes are found out and the model is framed on the basis of correlation coefficient.

Black box models are empirical, involving some mathematical equations which are assessed not from the physical processes in the catchment system, but from analysis of concurrent input and output time series. The black box models have been developed and extensively applied before the computer technology made it possible to use more physically conceptualised models. Today, black box principles are often used as some component of a complete river-system model.

5.1.2.6 Deterministic model

A deterministic model seeks to simulate the physical process in the catchment, involved in the transformation of rainfall to streamflow, or routing of flow through hydrological time series of the several measured variables such as rainfall, evaporation, and streamflow, involving statistical distribution in probability.

Deterministic models can be classified according to whether the model gives a lumped or a distributed description of the system, and whether the description of the hydrological process is an empirical, conceptual, or fully physical based. Deterministic models can be classified in the following categories.

a) Black box models

b) Lumped conceptual models

c) Fully distributed physically based models.

While most of the deterministic models are based on a physical approach, the statistical methods are often based on more direct approach, developing functional relationships between various types of measured data. The statistical methods in hydrology have been developed extensively with support from basic statistical theory developed and applied in other fields. The following sub-division can be made in the field of modelling.

(a) Regression and correlation models.

(b) Probabilistic models.

(c) Stochastic models.

5.1.2.7 Time Variant and Time Invariant Models

All the models, whether it is conceptual model, Black box model, physically based model or statistical model; are associated with one or more parameters which are unique feature of the system. In most of the models the parameters are stable and do not change with change of time. They are called time invariant models. In some models the parameters describing the system do not remain constant, but go on changing continuously with time to reflect any change in the system characteristics. These models are called time variant models.

5.1.2.8 Lumped and Distributed Models

A lumped model is a model where the system is regarded as one unit. The variables and parameters are thus representing average value for the entire catchment. A distributed model includes spatial variations in all variables and parameters.

Contrary to the lumped conceptual models, a distributed physical based model does not consider the water flows in a catchment to take place through some storage reservoirs. Instead, the flows of water and energy are directly calculated from the governing differential equations, for instance the Saint Venant equations for overland flow, Richard's equation for unsaturated zone flow and Bussinesq's equation for ground water flow.

5.1.3 Choice of Appropirate Model

In recent years, the application of mathematical models for simulation of hydrological and hydrometeorological process has become an important part of engineering services in water resources studies and real time flood forecasting, being an integral part of this technique, further development in the design and application of data base system has proceeded. Since a large number of hydrological models exist and many of them are functionally of the same type, the question comes how to select a particular type of model for use for a specific purpose. For instance, atleast 20 different rainfall runoff models of lumped, conceptual type exist. Although these models at a first superficial glance may look very different, they are basically functioning along the same principle. Thus, the difference in performance between the lumped conceptual rainfall runoff models are believed to be mostly dependent on the experience and efficiency of the Hydrologist who has carried out the calibration and operation, when the models themselves are basically of the same quality.

So the question raised earlier about the choice of appropriate model cannot be answered simply by giving the name of one model. Instead a general write up is being given below which will give an account of different types of models in use and are most appropriate for different kinds of hydrological problems. For some hydrological problems, selection of model type more or less is obvious, e.g. probabilistic model for frequency analysis or stochastic time series model for generation of long synthetic stream flow data. Therefore, only the fields of applicability of the different deterministic simulation models are being discussed here.

The Empirical Black Box models are mainly of interest as single event model or as sub components of more complicated models.

The Lumped conceptual models are especially well suited for simulation of the rainfall runoff process when hydrological time series sufficiently long for a model calibration exists. Thus typical fields of application are:

1. Extension of short inflow records based on long rainfall records.

2. Real Time rainfall runoff simulation for flood forecasting.

3. Other fields of possible application where the lumped conceptual models are not specially well suited, but can be used if no better model is available.

4. Prediction of runoff from ungauged catchments i.e. in the catchment where calibration is not possible. In such cases the model parameters would typically be estimated from the calibration on hydrologically similar and neighbouring catchment.

5. General water balance studies, availability of ground resources, irrigation needs, analysis of variation in water availability due to climatic variability.

The distributed physically based models can, in principle, be applied to almost any kind of hydrological problem, obviously there are many problems for which necessary solution can be obtained by using the cheaper lumped, conceptual model. However, for more complicated problems there may be no alternative but to use a distributed physically based model. Some examples of typical fields of application are:-

1. Predication of the effects of the catchment changes due to human interference in the hydrological cycles such as, changes in land use, urbanisation, ground water development and irrigation. The parameters of the model are direct physical parameters. Therefore the changes in physical value corresponding to the catchment changes can be obtained directly.

2. Prediction of runoff from ungauged catchments and from catchments with very short record (up to 2 years) available unlike the lumped conceptual model which requires long historical time series of rainfall, runoff and evaporation data for the parameter assessment. The parameters of the distributed, physically based model can be assessed directly from intensive, short term field investigations.

3. Water quality and soil erosion modelling for which detailed and physically correct simulation of water flow is essential. The main problem usually mentioned in connection with the application of the distributed, physically based models are:

   (a) Large requirement of data, especially of meteorology, geology, soils and vegetation.

   (b) In-sufficient knowledge of physics of the hydrological processes which are modelled in great details.

   (c) Large computer requirements.

The distributed, physically based models are directly able to utilise the distributed information usually available in topographical, geological and soil map and other information on vegetation and land use distribution. In addition, these models are able to make use use of satellite imageries and other remotely sensed information, including continuous updating of catchment conditions. Thus the data requirements for these models are huge but this merely reflects that the model can utilise all these data if they are made available. On the other hand, model can be operated with less data, but the prediction will be more uncertain.

Similarly, with respect to argument that there still is an insufficient knowledge of hydrological process, it can be stated that all the existing knowledge can be built into the distributed physically based models but not into the lumped conceptual models.

With the rapid developments in the field of computer science, requirements are no longer prohibitory for practical application of large complex models like distributed, physically based model. It is envisaged that these models can easily be operated even on micro or personal computer.

5.2 Description of Models in use in CWC

5.2.1 NAM S11F Model

5.2.1.1 Model components

This modelling system consists of 4 components of mathematical elements:

1. Hydrological model (NAM)

It is a conceptual model developed by the Hydrological Section of the institute of Hydrodynamics and Hydraulic-Engineering at the Technical University of Denmark in the early seventies. NAM is an abbreviation of the Danish "Nedbor-Afstromnings- Model" meaning precipitation-runoff-Model.

2. Hydro-dynamic modelling system (System 11)

System 11 (11 for one dimensional, one layer) is a mathematical system developed by Danish Hydraulic Institute (DHI) for simulation of flows and water quality in estuaries and rivers and similar water bodies.

3. Linear Model for updating

A traditional linear noise model is being used for updating discharges before making the forecast.

4. Data management system

A package of programmes for storage, processing and retrieval of data has been added for preparation of input to the model.

5.2.1.2 Capability of the Model

1. Hydrological model (NAM)

It is for accounting the runoff from rainfall. It simulates runoff from rainfall by operating continuously accounting for the moisture content in different sub-catchments in four different and mutually interrelated storages representing physical elements in the sub-catchment.

2. Hydrodynamic modelling system (System 11)

The system simulates unsteady one dimensional flows and transports in one layer (vertically homogenous) fluids. System 11 can accommodate any system of channel, taking into account topographical effect, lateral discharges, resistance, effects of structures in the rivers, flooding etc. Here the runoff from respective sub-catchments predicted by the NAM model will be routed to the point of forecast by System 11.

3. Linear model for updating

In order to get optimal benefit for real time measurements of discharges in the forecast, some updating of the hydrological and hydrodynamic models is required. A traditional linear noise model is being used for updating discharge before making the forecasts. The philosophy is to lump all the uncertainties in a black box model of the deviations between the simulated (non-updated) and observed discharges. The catchment's response to a rainfall event is approximating the deviation by the action of linear reservoir.

When deviations between discharge (simulated) and discharge (observed) is experienced, a noise term is added to the simulated discharge.

The steps involved in it are as follows:

(a) To calculate, deviation = Σ =Q (observed) − Q (simulated)

(b) To multiply deviation by an updating factor α ($o \le \alpha \le 1$)

(c) To distribute the discharge $\alpha\Sigma$ to grid points located between the discharge stations used for updating

(d) The updating discharge(= noise term) $\alpha\Sigma$ will then decrease exponentially with time until it is changed by the next updating.

4. Data Management

Data management package of programmes was developed for data processing and transformation of hydrometeorological data into formats which are readable as model input.

5.2.1.3. Formulation

1. NAM

The model structure has been shown in Fig V.2.2. Accounting of moisture is being done in two storages namely surface storage (U) and lower Zone (L). Between these storages, the main hydrological and meteorological processes take place. Routing of water as accounted for to the river, is done through a set of parallel linear reservoirs. These four interrelated reservoirs are called overland flow, inter flow, the upper base flow and the lower base flow. If there is a high storage or pondage along a river, a lake reservoir is incorporated at the end of the system. Each of these routing reservoirs is described by only one parameter which is called reservoir time constant.

2. System 11 model

The system 11 is based upon the equations of conservation of mass and momentum (the Saint Venant equations):

$$\frac{dQ}{dx}+\frac{dA}{dt}=0$$

$$\underbrace{\frac{d}{dx}\left(\frac{\alpha Q^2}{A}\right)+\frac{dQ}{dt}+\underbrace{gA.\frac{dE}{dx}+\underbrace{\frac{gQ|Q|}{M^2AR^{4/3}}=gAI_o}_{\text{Kinematic wave}}}_{\text{Diffusive Wave}}}_{\text{Dynamic wave}}$$

Where

Q - Discharge (m^3/Sec)

A - flow area (m^2)

g - acceleration of gravity ($m.s^{-2}$)

M - Manning's roughness coefficient ($m^{1/3}\ .s^{-1}$)

E - Water depth (m)

Q - Discharge (m^3s-^{1})

R -resistance radius (m)

α - momentum distribution coefficient

I_o- slope of river bed

X -Horizontal coordinate (m)

The resistance radius is derived from Engelund's formulation (Of Lundgren and Jonsen, 1964). $R = 1/4 \int_0^b y^{3/2}$ in which y is the local water depth and b is the channel width. This formulation ensures a description where the resistance coefficient is independent of the variation in depth. For the bed friction Manning formula may be used instead of the default chezy formulation ($C = M \times R^{1/6}$). The equation shown above applies to a single branch but in nodal point (junction of three different branches), a continuity equation is used with an energy compatibility condition relating the energy levels of connecting branches.

5.2.1.4. Solution Technique

The method of solution is a fully time centered, implicit finite difference method, based on 6 points Abbott schème. In order to account for changes in both speed and time of preparation the space increments may change automatically in lengths as flood plains etc. The unknown variables are h (stage) and Q (discharge) in grid points, which can be set whenever required with odd numbers for h points and even numbers for Q points. The method of solution is the double sweep method based on Gauss' elimination. The basic equations were converted into quasilinear form and solved by "EF" sweep and backward sweep.

5.2.1.5. Linear Model updating

In the flood forecasting model, in order to have optimal benefit for real time measurement of discharges in the forecast of best accuracy, updating of the hydrological and hydraulic models is done by a linear noise model, by updating the discharges before making the forecasts. This is done by a black box model by lumping all uncertainties of the deviations between the simulated (updated) and observed discharges through a sub-routine to System 11.

For each time, where real time measurements are obtained, a correction (lasting until the next expected time measurements) is derived and equally distributed on the upstream grid points upto next measuring station.

The basic philosophy in the updating routine is deviations between observed Q and simulated Q. Deviations are due to inaccuracies in rainfall input data either by measurement or averaging or different pattern of concentration of precipitation. The deviation between observed and simulated discharge is described by the action of a linear reservoir, i.e. as an exponentially decaying function viz response function for a linear reservoir with time constant K.

5.2.1.6. Data requirement for the model

1. NAM Model

The input requirements are time series of precipitation and potential evapotranspiration plus temperature data if snow routing is included. Further for the calibration period recorded discharges are required.

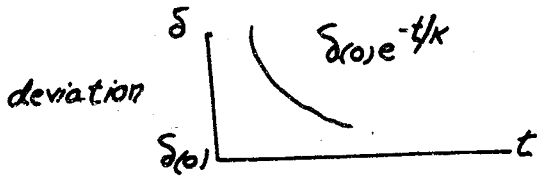


Fig.V.2.1.

2. System 11- input parameters

Impute parameter	The parameter can be specified as a function of		
	space	Time	Level
1. Initial levels	X		
2. Initial discharges	X		
3. Boundary levels	X	X	
4. Boundary discharge	X	X	
5. Level discharge	X	X	
6. Boundary relation	X		
7. Cross sectional areas	X		X
8. Hydraulic radius	X		X
9. Velocity distribution coeff.	X		X
10. Chezy resistance Coeff.	X		X
11. Flooded areas	X		X
12. Space step length	X		X
13. Lateral inflow discharge	X	X	X

Item No. 9 and 10 are calculated by "PARM" programme. The only input will be the cross sections at different gauge stations. Item No. 11 is calculated by "FLOOD" programme giving item No. 7, 8, 9 and 12 as input data.

5.2.1.7. Computer requirement for running the model on real time basis

For running this model on real time basis, computer of the following configuration will be suitable. A computer system of similar configuration was installed at Maithon for running the model on real time basis on River Damodar.

I. *Hardware*

1.	Central processing unit (CPU) with 512 KB Random Access Memory (RAM) supported with 16 bit intel 8088 and 8087 coprocessor	1 no.
2.	Online interactive terminals (video)	2 nos.
(i)	Console (ii) remote	2 nos.
(iii)	Winchester disk drive with 32 MB storage	2 nos.
(iv)	Tape drive, 9 tracks, 16 bpi 2400 ft. (streamer)	1 no
(v)	Dot matrix printer with 132 CPL and 180 CPS	1 no.

II. *Software*

1.	Multiuser, Multi tasking operating system allowing one programme to utilise the entire memory.	1 no.
2.	Fortran 77 version	1 no.
3.	77 Compiler with CP/M-86 OS from DIGITAL RESEARCH	1 no.

The computer was installed in an airconditioned room with vinyl flooring and false ceiling with flushed high fittings suitable for avoiding dust accumulation. Approximately 6 nos. of 1.5 ton standard capacity package type airconditioners are to be installed to provide temperature 22°C ± 3°C and 50% to 60% humidity. For regular supply of 230 ± 5% volts power supply, a servo controlled voltage stabilizer of 10 KVA was installed with range of 180-270 volt.

PC version of the model is being arranged from DHI when the following configuration of the computer system will be sufficient to run the model.

1. 640 KB CPU with 8087 math co-processor
2. 40 MB Disc drive
3. One $5\frac{1}{4}$" floppy drive
4. Monochrome terminal with graphics facility
5. 150 CPS Dot Matrix printer
6. Operating-system MS DOS and C.C. P.M. or UNIX

5.2.1.8. Model calibration

1. Introduction

Model calibration, in principle, involves manipulation of a specific model to reproduce the response of the catchment under study within some range of accuracy. In a calibration procedure an estimation is made for the parameters, which cannot be assessed directly from field data. All empirical (Black box) models and all lumped conceptual models contain parameters whose values have to be estimated through calibration. The fully distributed physically based models only contain parameters which can be assessed through field data, so that in theory, a calibration should not be necessary if sufficient data is available. However, for all practical purposes also the distributed physically based models require some kind of calibration, although the allowed parameter variations are restricted to relatively narrow intervals compared to the empirical parameters in the empirical or lumped, conceptual models.

5.2.1.9. Sources of uncertainties in deterministic simulation

The concept of deterministic simulation can be illustrated in Fig. V.2.2. The model is a simplified representation of all physical systems.

The time and space variability of the input and output is quantified by measurement. However, this measurement cannot provide absolute knowledge of the mass and energy exchange between the catchment and its environment. This is primarily due to the inability to completely measure the area variations in magnitude and timing of the mass and energy exchange, and secondly to the measurement errors . As a result the model will use data which contains disturbances. On the other hand, the physical system experiences the real data and responds to the real input. This response is then measured with some uncertainty.

When the model is used to simulate the behaviour of the physical system, it produces simulated output containing the effects of the input disturbances. This simulated output is then compared with recorded output disturbances. In order to test and verify the accuracy of the model. To achieve the required model accuracy the parameter values are adjusted until the agreement between simulated and recorded output is satisfactory. This parameter adjustment process is the calibration.

Basically four sources of uncertainties in deterministic simulation exist to which the disagreements between recorded and simulated output can be classed with:

1. Random or systematic errors in the input data e.g. precipitation, temperature, or evapotranspiration used to represent the input conditions in time and space for the catchment.

2. Random or systematic errors in the recorded output data e.g. water level or discharge data used for comparison with the simulated output.

3. Errors due to non-optional parameter values.

4. Errors due to incomplete or biased model structure.

Thus during the calibration process only error source 3 is minimised. The disagreement between simulated and recorded output is due to all four error sources. The measurement errors serve as a "background noise" and give a minimum level of disagreement below which further parameter or model adjustments will not improve the results. The objective of a calibration process is then to reduce the error source 3 until it is insignificant as compared to the error source 1 and 2.

During calibration process it is of utmost importance to ensure that a clear distinction be made between the different error sources, so that it is not attempted to compensate e.g. a data error by parameter adjustments. Otherwise the calibration will degenerate to curve fitting which may result in a reasonable fit within the calibration period, but inevitably will give poor simulation results for other periods. In the following five examples it would be physically incorrect and fatal for future predictions to try to compensate the following discrepancies between recorded and simulated flows by parameter adjustments.

- A flood peak during a snow melt season is simulated too low and with too less volume due to an under estimation of the snow precipitation. Error source 1.

- Discrepancies between simulated and recorded flow in a period where the recorded flow is known to be very uncertain due to problems with rating curve. Error source 2.

- A flood peak is simulated too low due too embankment breaches. The model has been developed assuming non-breaching embankments. Error source 4.

- The travel time of high flows are smaller than the travel time for low flows while the routing model is linear with the travel time independent of flow regime. Error source 4.

- O The base flow in the low flow periods is decreasing during the calibration period due to groundwater abstraction and lowering of the ground water tables. Ground water abstraction cannot directly be accounted for in the applied model. Error source 4.

5.2.1.10 Criteria for calibration of Model

During the calibration procedure accuracy criteria are often used when simulated and recorded output is compared. The purpose of these measures is to make a goodness of fit associated with each set of parameters and the most optimal parameter values can be found. To draw a fixed criteria for accuracy is very difficult because of different sources of errors associated with deterministic simulations. Further more most models produce output representing several variables such as water levels, discharge etc. Hence a single criterion on accuracy cannot be derived to represent all variables.

The most widely used criterion is the sum of the squares of the deviations between recorded and simulated values of a variable.

$$F^2 = \sum_{i=1}^{n} \left(QOBS_i - QSIM_i\right)^2$$

where

F^2 = index of disagreement, or objective function

$QOBS_i$ = Observed value at time step i.

$QSIM_i$ = Simulated value at time step i.

n = Number of values (time steps) within the considered time period.

A calibration based on blind optimisation of a single numerical criterion will often result in physically unrealistic parameter values to another time period and will give poor simulation result. Therefore, it is recommended to use numerical criteria as guidance only and a combination of the following four criteria may be used during a calibration process:

1. A good agreement for average simulated and recorded flows.
2. A good agreement for peak flows both with respect to volume, rate and timing.
3. A good agreement for low peaks.
4. A good overall hydrograph agreement with emphasis on a physically correct model simulation.

LINE DIAGRAM OF NAM MODEL STRUCTURE

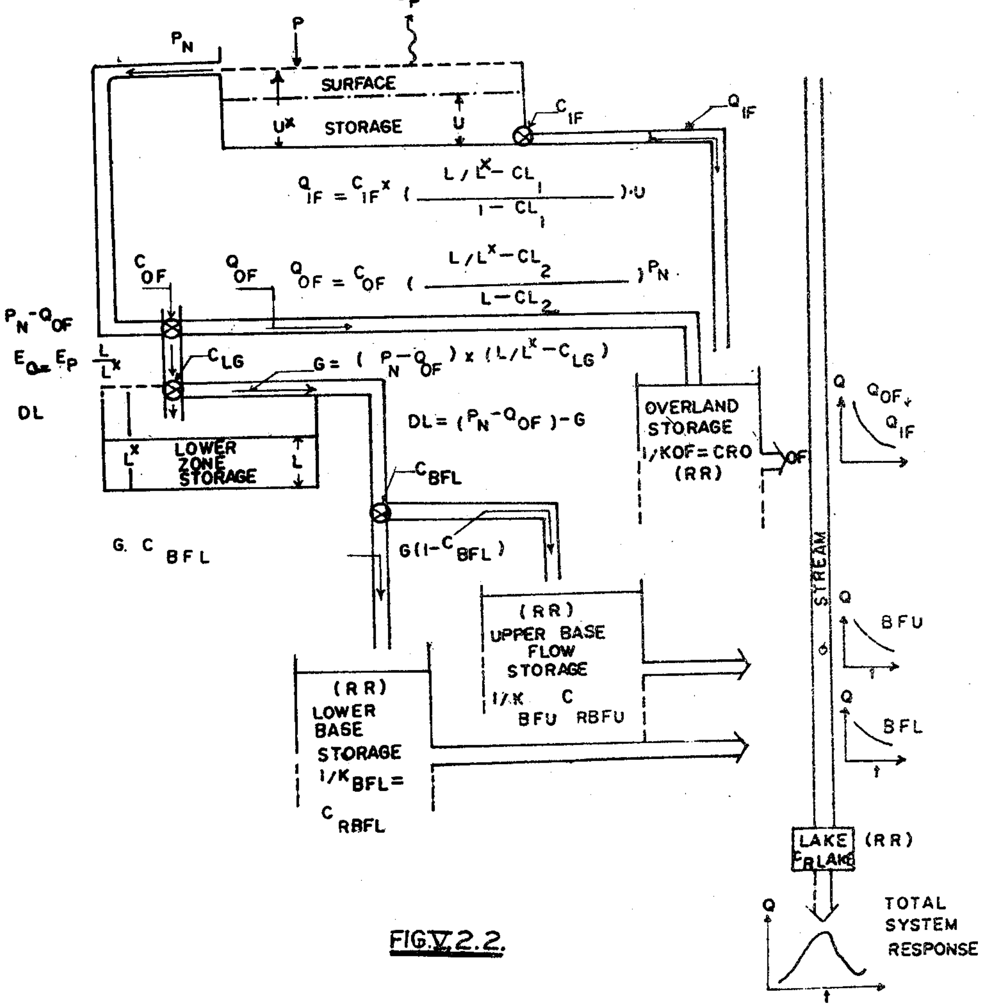


FIG. V.2.2.

P	PRECIPITATION
E_P	POTENTIAL EVAPOTRANSPIRATION
U^x	MAX^M SURFACE STORAGE OPEN TO E_P
U	STATE OF SURFACE STORAGE $0 \leq U \leq U^x$
L^x	ROOT ZONE STORAGE
$\frac{L}{L^x}$	RELATIVE MOISTURE CONTENT IN ROOT ZONE
L	STATE OF ROOT ZONE STORAGE $0 \leq L \leq L^x$
Q_{IF}	IT IS THE INTERFLOW DEPENDS ON U AND GOVERNED AT THRESHOLD VALUE C_{LI}
P_N	NET PRECIPITATION AFTER U IS SATISFIED WITH MOISTURE AND IS DIVIDED INTO:—

I. Q_{OF} — OVER LAND FLOW

II. G. — GROUND WATERFLOW INTO TWO RESERVOIRS.

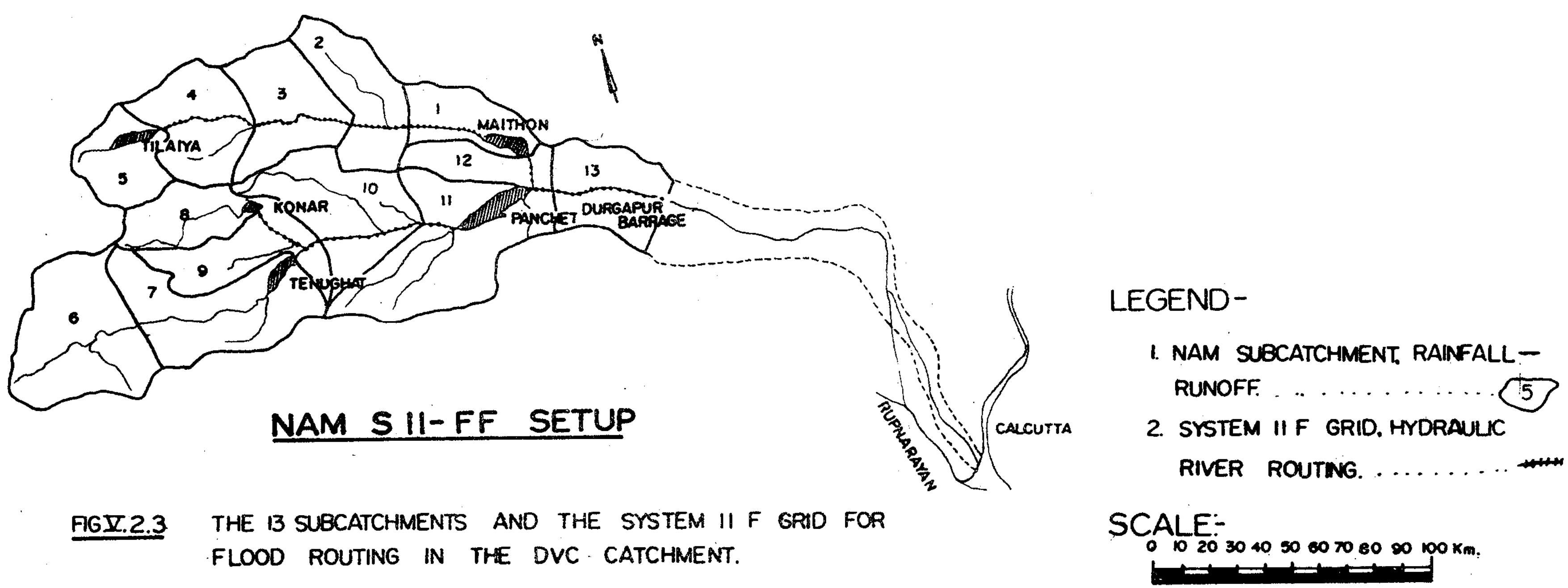
III. DL — INTO ROOT ZONE STORAGE

ACTUAL EVAPOTRANSPIRATION $= E_P \cdot \frac{L}{L^x}$

NOTE : T

(RR) ROUTING LINEAR RESERVOIRS

DAMODAR CATCHMENT



NAM S 11-FF SETUP

FIG V.2.3 THE 13 SUBCATCHMENTS AND THE SYSTEM 11 F GRID FOR FLOOD ROUTING IN THE DVC CATCHMENT.

5.2.1.11 Application of the modelling system

The model can handle the following activities:

1. Simulations of guage and discharge at required stations.
2. Simulation of run-off from rainfall and routing downstream.
3. Updating of the forecast with the incoming information of the latest gauge and discharge.
4. Utilisation of qualitative precipitation forecast for increasing the warning time.
5. It is a valuable tool for reservoir regulation. As the model is easy and flexible to use, various alternatives for solving a critical situation can be investigated and judgement can be taken on the consequence of the decision.

The modelling system has been applied to the Damodar River System and Yamuna River Basin and models have been set up on the two basins and are being used on real time basis since 1985 monsoon season.

The following data has to be collected to set up the model in a particular river basin.

1. Rainfall data, preferably hourly, of all the stations representing the basin for ten years. In case of new basin, atleast 4 years continuous data should be compiled; 2 years for calibration and 2 years for testing.
2. Evaporation data for the period for which rainfall data could be compiled.
3. Time series of guage and discharge data atleast for 5 years for all the observation stations from origin to forecast points.
4. Rating curve at all observation sites.
5. Cross-section of the river with spacing of 5 kilometer and at all locations where there is observation station, sudden change in slope and meandering of river occurs.
6. Basin map of scale 1 cm = 2.5 km., detailing all engineering structures and flood plains with its contours.

5.3.1 HEC-1F Model

5.3.1.1 Introduction

The HEC-1F model has been developed by the Hydrologic Engineering Centre (HEC), Davis, California which is a branch of the United States Army Corps of Engineers. The HEC Water

Resources Computer programd comprise a number of models within hydrology, river and reservoir hydraulics, reservoir operation stochastic hydrology, water quality, sedimentation etc. The HEC-1F model, being HEC's flood hydrograph package, is a comprehensive package of computer programs comprising the following elements;

- Catchment rainfall and snow melt calculation from gauge data.
- Hydrological routing.
- Calculation of rainfall runoff processes.
- Automatic estimation of unit hydrograph, interception/infiltration and streamflow routing parameters.
- Computation of modified frequency curves and expected annual drainage for any location in the stream system and automatically for several flood control plains throughout the watershed.
- Simulation of flood through a reservoir and spillway for dam safety analysis.
- Simulation of dambreak hydrographs.
- Optimization of flood control system components.

HEC-1F Program forecasts runoff from unregulated flows, but can also be used in conjuctions with HEC-5 for real time simulation of reservoir system operation. It is intended for short term forecasts of flood runoff confined to a single flood event and is not suitable for continuous long term forecasting.

5.3.1.2 Model Structure

A typical catchment with subcatchments and routing reach components is shown in Fig. V.2.3. The model simulates the precipitation, precipitation loss and runoff processes in each sub-basin, routes stream flow downstream and combines tributary inflows.

A river basin is sub-divided into an inter-connected system of stream network components using topographic maps and other geographic information. Each sub-basin is intended to represent an area which, on an average, has the same hydraulic/hydrologic properties. The following factors are taken into consideration while dividing the basin into sub-basins:

1. Size of the sub-basin.
2. Topography.

3. Stage data (historical)
4. Stage data (real time)
5. Precipitation data (historical).
6. Precipitation data (real time).
7. Forecast points
8. Rating curves (location)
9. Man-made devices (location and operation) e.g. reservoir.
10. Land use.
11. Meteorological conditions.

5.3.1.3 Description of Model Processes

1. Evapotranspiration

The evapotranspiration process is not calculated and is only accounted for indirectly in terms of initial conditions for the interception/infiltration calculations.

2. Interception/infiltration

This model component due to interception and infiltration is crucial for a correct simulation of flood events. The interception and infiltration losses can be calculated by four different methods shown below:-

Method	*Parameter*	*Description*
Initial and constant	Initial volume loss and a constant infiltration rate.	Initial loss volume is satisfied, then constant loss rate begins.
HEC exponential	Infiltration rate, antecedent moisture condition, rate of change of infiltration with wetness.	Initial infiltration rate adjusted for antecedent condition and continuous function of soil wetness.

Method	*Parameter*	*Description*
SCS Curve number	Curve number	Initial interception loss satisfied before computing cumulative runoff as a function of cumulative rainfall.
Holtan	Infiltration rate capacity, available soil moisture storage.	Infiltration rate computed as exponential function of available soil moisture storage and is limited by ultimate infiltration rate for saturated soil.

3. Land Surface Runoff Component

Precipitation excess over sub-basin is computed by subtracting infiltration and detention losses based on soil water infiltration function. The resulting rainfall excesses are then routed by the unit hydrograph to the outlet of the sub-basin. A unit hydrograph can be computed from users supplied Snyder's parameters. Baseflow is computed relying on an empirical method and is combined with the surface runoff hydrograph to obtain flow at the sub-basin outlet. The Program combines or links together outflows for different sub-basins.

4. Routing Component

A river routing component is used to represent flood wave movement in a river channel. The input to the component is an upstream hydrograph resulting from individual or combined contributions of sub-basin runoff and river routing. The Hydrograph is routed to downstream point based on the characteristics of the channel, using Muskingum, modified Puls or Kinematic wave method.

5.3.1.4 Data Requirements

The data requirements are point rainfall values and discharge data for base flow estimation. There is a high flexibility in respect of form and time resolution of input data.

5.3.1.5 Calibration

5-10 individual storms are required for calibration. The parameter calibration option has the capability to automatically determine a set of unit hydrographs, loss rate and base flow parameters that best reconstitute an observed runoff hydrograph for a sub-basin. Initial estimate of parameters to be determined can be input by the users or chosen by the program optimisation procedure. The best reconstitution is considered to be that which minimises the objective function described in the subsequent sections. This is found by systematically changing the values of the parameters until the objective function is minimum. To help the assessment of the optimisation, the program provides graphical and statistical comparison of the observed/simulated hydrographs. The six optimised parameters are (1) base flow or initial flow (2) Recession constant equal to the ratio of recession flow at T hour and recession flow at (T + 1) hour (3) Initial loss (4) Constant loss per hour (5) Snyder's lag (6) Peaking co-efficient.

The above parameters are calibrated for gauged sub-basins. The calibration of the model for the entire basin is done by trial and error adjustment of runoff and routing parameters.

5.3.1.6 Capability of the Model

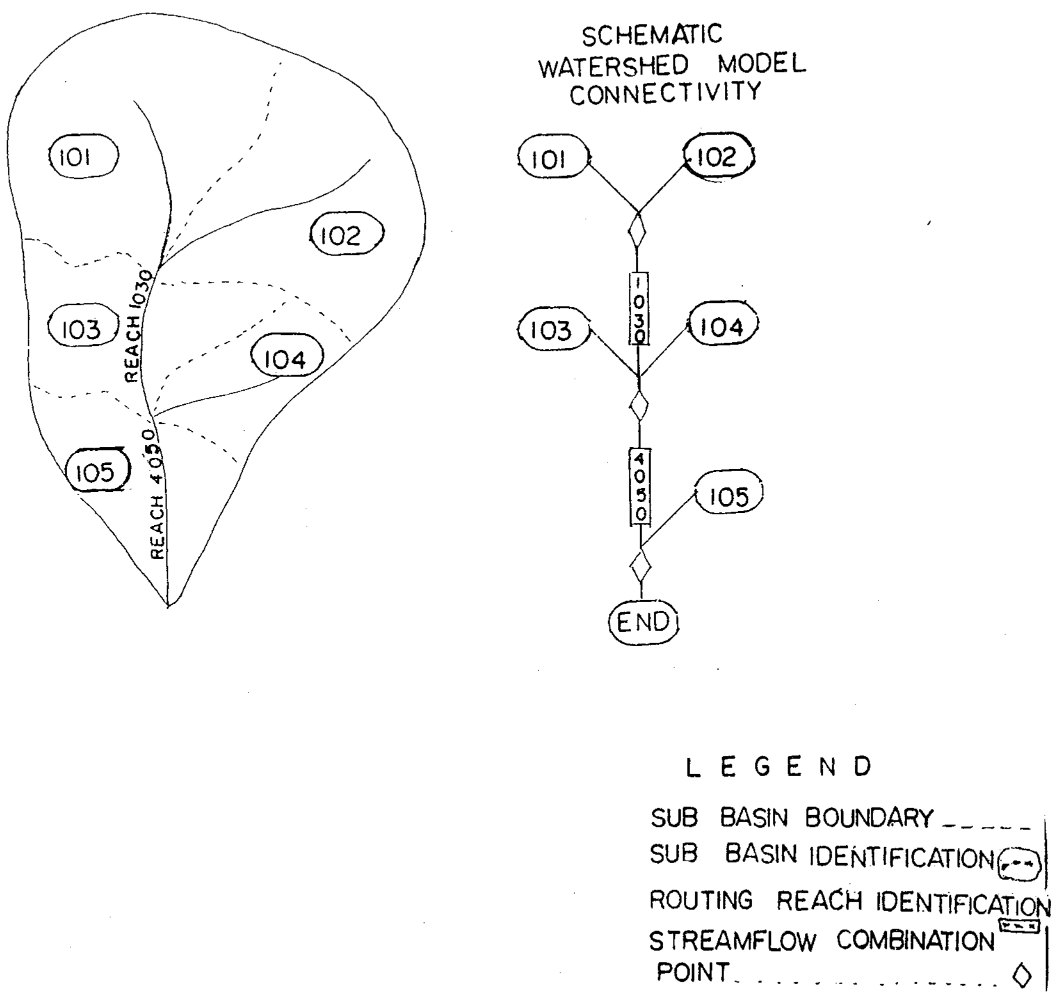
The advantages and disadvantages of the model are detailed below:-

1. It is a simple model with a smaller number of parameters.
2. Due to its simplicity, it is faster than other models to set up and calibrate.
3. The infiltration losses cannot be directly included in the runoff hydrograph. The infiltrated water is not used in any further computation.
4. There is no provision for drying out of the soils between storms.
5. The channel routing is not so general and cannot be accounted for so many hydraulic situations such as flooding of flood plains etc.
6. There is no procedure for updating the simulated flows during a flood event.

5.3.1.7 Application of the Model on Real Time Situation

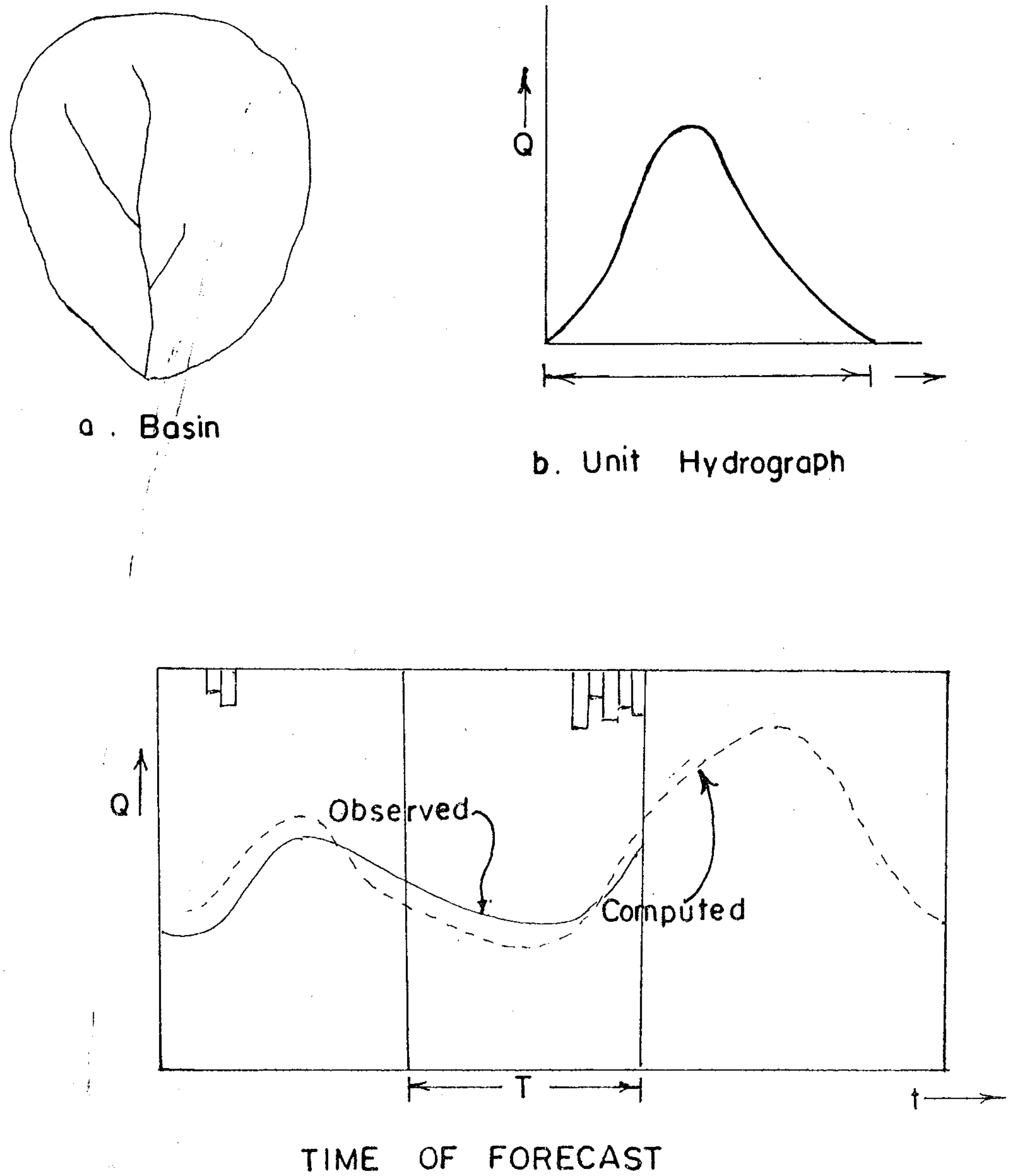
In the Yamuna catchment, data (rainfall/gauge) are being recorded by sensors and automatically transmitted to the Master Teleprocessor with a HP-1000 computer system by the telemetery system. The telemetery system supplies cumulative rainfall values for individual raingauge stations, whereas HEC-1F accepts incremental rainfall values for each raingauge or for the sub-basin. There is provision for another Program "PRECIP" which accepts cumulative rainfall values for each raingauge, converts them into incremental values and computes the sub-basin average incremental values.

After collecting the cumulative rainfall values from the computer, input files are being updated mannually at every forecast operation time. In this respect it is mentioned that Program has been developed for entry and retrieval of the data and preparation of input data base for different models.



TYPICAL HEC-I CATCHMENT MODEL COMPONENTS
(FIGURE FROM FELDMAN (1981))

FIG.V.3.1



C. FRAMEWORK

FRAMEWORK FOR PARAMETER ESTIMATION

FIG.V.3.2.

The execution of the PRECIP program gives the sub-basin weighted incremental rainfall values which forms the basis of updating file "input" for HEC-1F program. This file must also be updated with respect to observed water levels which are being converted into discharges for every gauge station in the catchment. Two input decks viz. E-model and F-Model are generally prepared for running the model on real time situation. E-Model optimises the parameters and F-Model prepares the output file of the forecasted file.

5.4.1 NLC-NLM Model

5.4.1.1 Introduction

Like other conceptual mathematical models NLC-NLM (Non Linear Cascade-Non Linear Model) has also been recently developed under Indian condition and applied to the Yamuna catchment area. The computer program has been tailored to suit the HP-1000 computer environment and used for issue of experimental flood warning since 1985 flood season for Delhi Rly. bridge. The model has been partially utilised in the recent years, and more refinements have been done recently, testing of which will be carried out in the coming monsoons.

5.4.1.2 Model Structure

NLC model stands for Non-Linear Cascade model. This model has two components: Rainfall-runoff watershed model (NLC) and river routing model developed by Dr. A. Svoboda, Institute of Hydrology and Hydraulics, Czechoslovakia and Mr. P. Gabris of same Institute.

1. Rainfall runoff model NLC

It is a simple, two component moisture accounting conceptual model. It can simulate both ground water and direct runoff. The input to the model is the lumped total rainfall over the catchment. The model is structurally represented by the following equation.

$$F^1 = [\{\exp(E) - \exp(-E)\} / \{\exp(E) + \exp(-E)\}] \quad ....(1)$$

$$E = (6.EF/EN) - 3 \quad ......(2)$$

where

PA = Precipitation (input)

PE = Effective precipitation

QS = Direct runoff

GI = Ground water

QG = Ground runoff

QS = Total runoff

EN = Maximum water content

EF = Actual water content > 0 < EN

F^1 = Runoff factor > 0 < 1

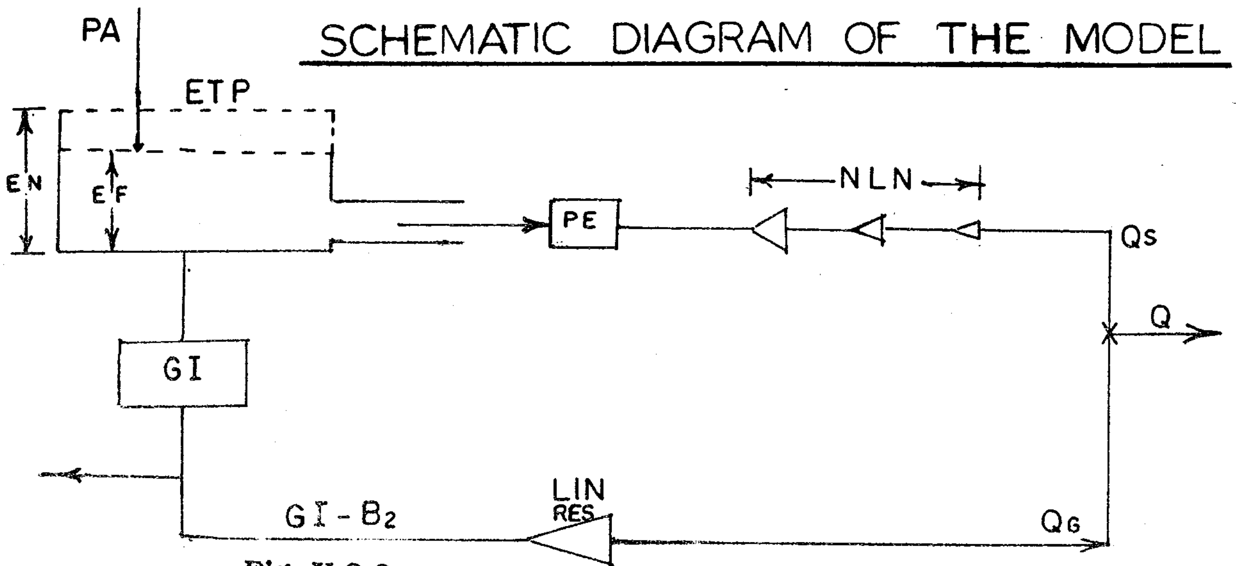


Fig. V.3.3.

The effective precipitation is calculated from the actual precipitation, by the following lation:

$$PE = PA. F^1.$$

The inflow into the ground water storage GI is equal to EM. EF/EN were EM is a parameter pressed in mm/hr. To allow for water loss through deep seepage a parameter B2 is introduced m/hr.) which is substracted from ground water storage input. Evapotranspiration can be termined by the relation ETP=B.EF/EN,B being actual evaporation (mm/hr). Ground water orage is represented by a single linear reservoir, defined by its time constant (BZ). BZ is termined from the recession of the hydrograph in periods with no direct runoff contribution cording to : BZ= (DTx. Kg)/(1-Kg), Kg being the recession coefficient according to

$$Kg = \frac{QG,t}{QG,t-1}$$

ith ordinates QG,t and QG,t-1 located by DT apart on the hydrograph recession. Parameters the direct runoff model are determined by NONLIN part of the model, described in para-2. This done best by using isolated rainfall-runoff events with high direct runoff component by trial d error method. Similarly, the parameters EM, EN, B0, B2 are determined by trial and error so as to simulate the observed runoff volume correctly and proportion of direct and ground water runoff. Another parameter, initial soil water content (in mm), B1 is to be defined by the modeller.

For the snowmelt (or snowmelt and rainfall) modelling, the output from snowmelt routing represents the input into the model.

Because of its simplicity and the required input (mean areal rainfall), model is suitable for real time forecasting. It is updated automatically according to the observed discharge at the catchment outlet.

For more detailed studies, the block determining effective rainfall can be replaced by a more sophisticated subroutine for description of water balance in the unsaturated zone which would produce the rainfall excess of ground water inflow directly, but would require more data which may not be available in real time.

2. Routing Model (Nonlin)

It is based on the concept of a cascade of N nos. equal nonliner reservoirs. The input to the model is the inflow to the first reservoir in series, the output of the reservoir is the input to the second one and so on until the output from the Nth or the last reservoir is the output from the model.

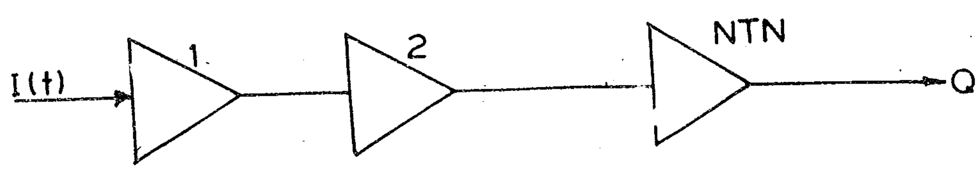


Fig V.3.4 Fictitious Reservoirs

The model preserves the mass continuity and does not take into consideration the lateral inflow. The lateral inflow, if , there be any, is to be added to or super-imposed upon the input part in case the lateral inflow occurs in the lower part of the section. Based on reservoir continuity equation the solution of output Q in the discrete form is /I-Q/dt=S ..... (a)/ where I is the input for each of N fictitious reservoirs in series and S is the change in storage during the respective time interval. The input and output are the same values during the time interval.

The basic reservoir routing equation is:

$Q = K.S^{EX}$ ....(b)

where K is a time constant (parameter) depending upon time step dt (Hrs.) and EX, nonlinearity parameter. Equation (a) and (b) constitute a base for calculation of Q in each of the time interval.

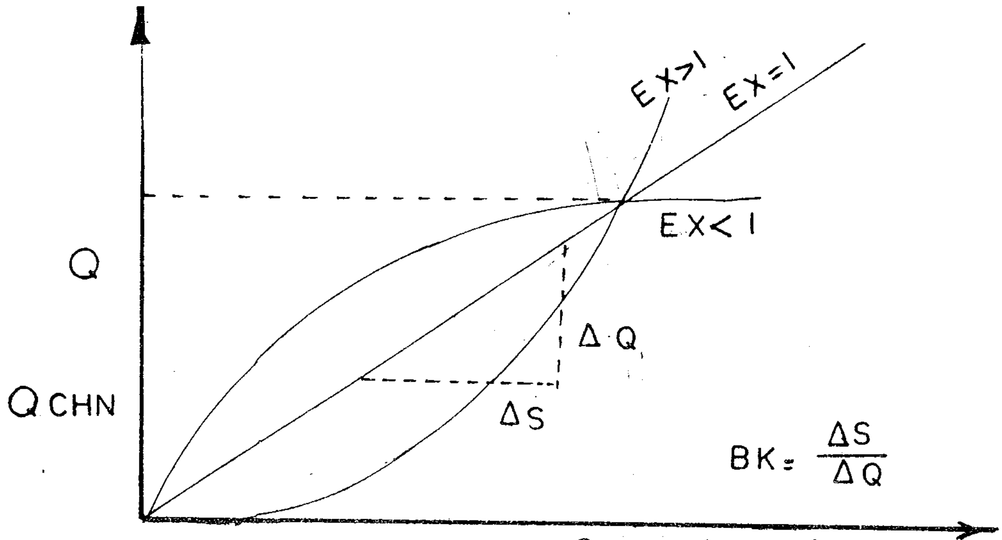


Fig V.3.5

BK (hrs.) is a parameter, which determines the slope of the line passing through the origin of the ordinates of equation (b) and through the point where all curves of equation (b) for different values of EX pass. The output QCHN, which corresponds to the point of intersection of all curves of equation (b), determines the shape of the function (b).

QCHN is also an another parameter.

Using equations (a) and (b) and using a unit time interval we get the following expression:

$I_n - Q_n = (1/K)^{1/EK} \; (Qn^{1/EK} - Q_{n-1}{}^{1/EK})$.......(c)

Where Q_n = output Q at nth time step

$Qn-_1$ = Output Q at previous (n – 1)th time step

I_n = Inflow I at current (nth time step)

Graphical solution of this equation is represented in the following figures.

Ex being a real number, it is not possible to solve the equation (c) for Qn explicitly and there is a fast iteration procedure incorporated in the model computer programme solving for Qn under condition that both sides of equation differ by less than ± 0.001.

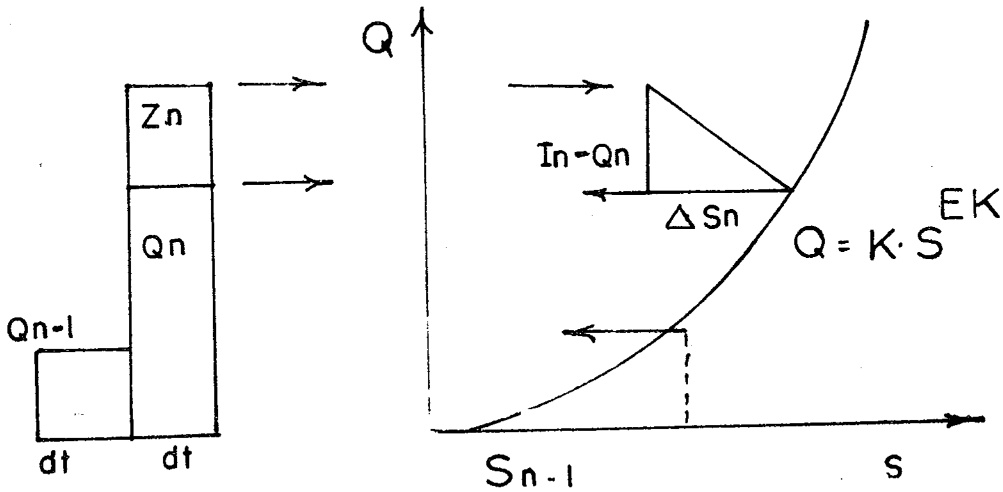


FIG.V.3.6.

To preserve the same accuracy for events which may differ numerically by orders, inflows and outflows in (c) are expressed as dimensionless quantities i.e. In=In/QCHN, so the computer programme becomes general and equally accurate for a wide range of inflow-outflow events and for the use of various units m^3/Sec. and mm/h etc. By the above procedure inflow is transformed through all N reservoirs in series, and the outflow from the last (nth) one represents the modelled outflow from the river section. Besides the time step dt/DT, model is defined by the parameters BK, QCHN, EK and N, which determine the complexity of the model, each elements of the routing equation (b) and number of reservoirs in series. An additional parameter ITR (integer) is used to shift the routed outflow hydrograph, linearly by ITR x DT time steps along the time axis forward. Use of ITR is necessary for a river reach showing a low transformation and high translation effect. Since, the equation (b) is nonlinear, it is possible to better describe the routing effect of river reaches than the similar conceptual models. If the parameter EX is put equal to 1, the model becomes a linear cascade model (Nash model).

5.4.1.3 Computer Programme and Operation

The computer programmes are written in standard FORTRAN-77 language software system consisting of 5 basic operational programmes, 5 basic data base programmes, 9 simulation and system maintenance programmes, 4 command files and 39 sub-programmes. Operational hydrological forecasting system software for real time consists of three parts:-

1. Operational system to maintain real time data, produce and disseminate forecasts.

2. Archive for storage and retrieval of historical data and

3. Simulation system for calibration, tuning and testing of forecasting models.

5.4.1.4 Operational System of Forecasting

1. Operational Part of Data Base

This part of data base is used for operational forecasting. It comprises data file for each observed (discharge and rainfall) and forecasted elements, each file having the length upto ten days. These files are augmented by the flow of incoming data with respect to time, the old data being sent to archive data base. The values stored in the files are supplemented by indicators separating measured, computed, estimated or missing data. By means of indicators, both the measured and forecasted data are kept in the same file and with passage of time the estimated or forecasted data are replaced by measured values. This data base also includes files with information on topography of forecasted river system, on the model execution control and files containing digitized rating curves for storage-discharge conversion. Real time data are measured and collected by an automated system controlled by a dedicated processor called" "Master". Hourly data are printed and recorded into a diskette and sent to HP-1000 computer, installed at Upper Yamuna Circle, Central Water Commission, New Delhi.

2. The Regular System Performance

The principal task of the forecasting software system is to produce operational real time forecasting. It utilizes mathematical models and data from operational data bank. The forecast is displayed on the screen dynamically during computation and dissemination of forecast is done by a standardised Forecast Table which is printed and can also be seen on the screen.

3. Forecast Procedure

The telemetry system sends hourly measured data to the computer which merges the values into one month all station records file called YMNDT. At the end of the month, this file is renamed as YMNDT1 and a new data file is created. Since, this procedure accepts only data of last hour, delay in starting the receiving programme will result in gaps in the data records and filling them up needs manual activity. Data file YMNDT1 is not suitable for direct use by forecasting system because of its size; and length of station records is highly variable during the month and at the end of month, it consists of two files. The missing values cannot be replaced in these files. Hence, data are transferred from YMNDT and YMNDT1 files to operational databases by DSR programme which also converts rainfall data from cumulative to hourly precipitation and water stages to discharges. This also helps in checking the data and filling the missing values. The forecasting procedure is invoked by the command: FOREC the forecast procedure starts its execution by display of the forecasting table which shows the information as are known before the computation.

Below the table is given information about the model which is working and immediately after its execution the appropriate places of screen are rewritten. The final table is printed on printer and hydrographs of observed and forecasted levels and discharges are plotted on the screen or plotter as per forecaster's option in the execution control programme.

Before making the model operational for real time forecasting, the model was set up on Yamuna river basin from the reach Kalanaur to Delhi Rly. Bridge and calibrated with historical data of flood seasons of 1976-84.

5.4.1.5 Calibration of Model

For calibration of the model, historical data are processed and different input files consisting of hourly rainfall, hourly gauge and discharge and rating curves are made for events of high peaks. The files start in its first column with one character to label type of recorded data as Q for discharge, H for level and T for precipitation etc.

The second value is the number of values stored in the file and rest of first line used to label the event. The next lines of the file contain data in free format. Another file contains the approximate parameters of river reach. The model will simultaneously display the table of input parts and simulated parts and its hydrographs.

The calibration of the model is required for finding the best suited value of the different parameters of the model so that it can be utilised for the purpose of forecasting discharge at site from the precipitation and inflows. The model is said to perform well if the calibration gives good response to:-

(1) Water balance difference between observed and simulated hydrographs.

(2) Similarity of peaks of two hydrographs

(3) Overall hydrograph behaviour of observed and simulated discharges.

(4) Nash co-efficient of agreement.

$$R_{NASH} = 1 - \left[\sum_n (QS_n - Qm_n)^2 \Big/ \sum_n (QS_n - Q_n)^2 \right]$$

Where Q_s = Simulated discharge ordinates of hydrographs.

Q_m = Observed discharge ordinates of hydrographs.

Q_n = Average observed hydrograph ordinates.

n = time step

The calibration had to show reasonable results by following the above four criteria. It is difficult to meet all the four criteria in all calibrations, but one had to change the parameters so that many of these conditions are met or at least model could simulate a hydrograph similar to the shape of observed one.

5.4.1.6 Maintenance of the Operational System

The operational system data base includes information about the hydrologic system structure and model parameters.

The hydrological system is divided into components, with its own name with 3 characters abbreviation of the name of the station. The naming was done as follows:-

NAMQCH	discharge file
NAMHTH	water level file
NAMR1H	precipitation file
TPNAM*	topography file which contains information of interface and values of model parameters.
IONAM*	Initial condition file

Operational files are sequential binary files with
368 records, 120 bytes each.

Before the regular forecasting system operation, all files of operational data base be created filling with appropriate data. The process is called initialization of the operational system.

(1) Creation of operational data base files by CCOF programme.

(2) Insertion of current values by DSR programme

(3) Invoke shift programme to create IC file.

5.4.1.7 SETTING UP OF OPERATIONAL SYSTEM FOR YAMUNA BASIN

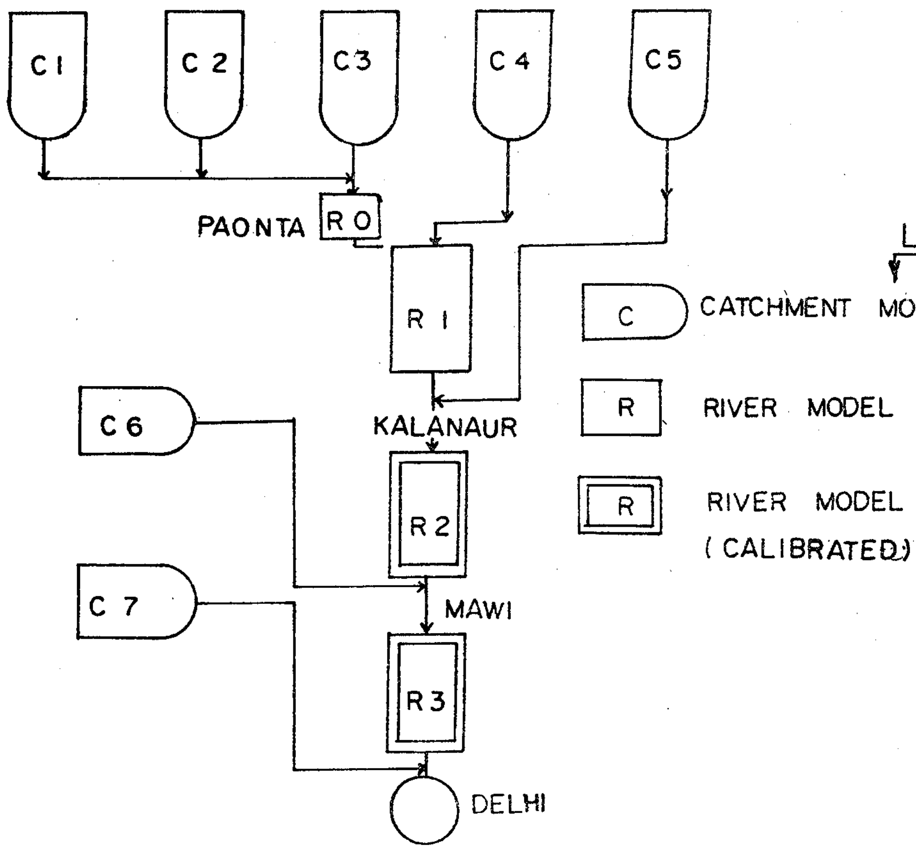
The operational system for flood forecasting in Yamuna catchment area upto Delhi Rly. Bridge from the origin was set up by constructing the time invariant operational files. The structure of the hydrological system for Yamuna is shown in Fig. V.4.1 schematically giving the programme components only.

5.5.1 SSARR Model

5.5.1.1 Introduction

The Streamflow Synthesis and Reservoir Regulation Model (SSARR) has been in the process of development and application since 1956. It was developed initially to meet the needs of the North Pacific Division of the U.S. Corps of Engineers to provide mathematical hydrological simulations for system analysis as required for the planning, design, and operation of water

YAMUNA FORECASTING MODEL SCHEME



LEGEND

C1. GIRI — JATEON

C2. TONS — KOTI

C3. UPPER YAMUNA — BAUSAN

C4. SUBCATCHMENT FROM JATEON-KOTI-BAUSAN TO PAONTA

C5. SUBCATCHMENT PAONTA—KALANAUR

C6. SUBCATCHMENT KALANAUR — MAWI

C7. SUBCATCHMENT MAWI — DELHI

R1. RIVER SECTION PAONTA — KALANAUR

R2. RIVER SECTION KALANAUR — MAWI

R3. RIVER SECTION MAWI — DELHI

RO. FICTITIOUS RIVER SECTION JATEON + KOTI + BAUSAN TO PAONTA

FIG.V.4.1

control works. The SSARR model has been further developed for operational river forecasting and river management activities. In recent years, numerous river systems in the United States and abroad have been modelled with SSARR by various agencies, organisations, and universities. In India, the SSARR Model has been adopted for real time flood forecasting at Delhi for Upper Yamuna catchment.

5.5.1.2 Computer requirement

The original program has been segmented and made to run in a HP-1000 F series computer with RTE6 VM operating system. This work is accomplished by the Indian Engineers in Upper Yamuna Circle, Central Water Commission. In current version of SSARR, called SSARR-8 (as developed by U.S Army Corps of Engineers), has come up recently. Major enhancements include additions designed for interactive operation of the program to assist the hydrologist in forecast operations; long term data storage and retrieval suitable for holding 50 years or more of data and the "Integrated snowband" watershed option, a model formulated to provide the capability to simulate on a continuous basis rainfall-runoff, snow accumulation and ablation, and snowmelt runoff in mountainous areas. The program source language is IBM-VS FORTRAN-77 versions and the programs are also available for the VAX-11/780 computer and IBM-PC-Compatible micro computers.

5.5.1.3 Over view of the Model

The SSARR is a numeric model of the hydrology of a river basin system. Streamflow at headwater points in the system can be synthesized evaluating rainfall, snow accummulation, and snowmelt. Stream flows throughout the basin can be synthesized by simulating the effects of channel routing, diversions and reservoir regulation and storage. The "Model" is comprised of a generalized watershed model and a streamflow and reservoir regulation model.

1. The water model

Simulated rainfall-runoff, snow accumulation, and snowmelt-runoff algorithms are included for modelling of snow peak cold content, liquid water content, and seasonal conditioning for melt, interception, evapotranspiration, soil moisture, baseflow infiltration and routing of runoff into the stream system are accounted for.

2. The river system and Reservoir regulation

Model routes streamflows from upstream to downstream points through channel and lake storage, and reservoirs under free flow or controlled-flow mode of operation. Flows may be routed as a function of multivariable relationships involving backwater effects from tides or reservoirs. Diversions and overbank flows may be simulated.

3. Compute-period length

The simulation proceeds through time by computing the model stage at successive intervals. The time increment may be as short as 0.1 hour or as long as 24 hours, depending on the goals and objectives of the modeller and model-dependent factors such as drainage area, hydrological response time, and the availability of hydro-meteorological data with which to operate and calibrate the model.

4. Applicability

As a general purpose mathematical model of a water system, the SSARR model is a useful tool for streamflow and run-off forecastings as well as for long term studies of the hydrology of a river system. Some examples of applications for which this program is applicable are:

— The analysis of the effects of multiple-purpose reservoir operations.

— Simulation of design storms.

— The investigation of streamflow diversion schemes as needed for irrigation management.

— Daily streamflow forecasting at many points throughout a river system.

— Seasonal streamflow forecasting.

The model has been developed around the requirements of relatively large drainage basins with a sparse network of observed data. However, it has been applied to a wide variety of large and small catchments, for both snowmelt and rain-only simulations in many different regions of the world.

It is a deterministic conceptual model rather than a hydraulic model and as such, may not be suitable for small drainages such as those encountered in studies of urban hydrology. The minimum computation step is one tenth of an hour, which may be too long for accurate simulation of runoff peaks from very small areas.

5. Data requirements

Input data needed for operation of the model include:

Non-variable characteristics data

Which describe physical features such as drainage area, reservoir storage capacity and watershed characteristics that affect runoff.

Initial condition data

Initial condition data for specifying current conditions of all watershed-runoff indices, flow in each increment of each channel reach, and initial reservoir or lake elevations and outflows.

Time variable data

Which includes physical data expressed as time series; for example, precipitation, air temperature, streamflow, reservoir-regulation data and other hydrometeorological elements.

Miscellaneous JOB control and time control data

Which specify such items as total computation period, routing intervals, and special computer instructions to control print-outs and other input-output alternatives.

5.5.1.4 Model Formulation

5.5.1.4.1 BASIC Routing Method

The basic routing method used in the watershed and river models is a "cascade of reservoirs" technique, wherein the lag and attenuation of the flood wave are simulated through successive increments of lake-type storage. A watershed or channel can be visualised as a series of small "lakes" which represent the natural delay of runoff from upstream to downstream points. The user specifies the routing characteristic of the prototype "lake" as well as the number of "lake" increments.

Routing through watershed, river system, and reservoir component of the model relies on the law of continuity in the storage equation:

$$\left[\frac{(I_1+I_2)}{2}\right]t-\left[\frac{(O_1+O_2)}{2}\right]t=S_2-S_1$$

where

I_1 = Inflow at the beginning of the compute period.

I_2 = Inflow at the end of the period.

O_1 = Outflow at the beginning of the period.

O_2 = Outflow at the end of the period.

S_1 = Storage at the beginning of the period.

S_2 = Storage at the end of the period.

t = Time duration, compute period length.

In differential form, the inflow (I_t) is expressed by

$$I_t = O_t + ds/dt$$

In natural lakes where storage is a function of outflow at any given elevation, T_s represents the proportionality factor between storage and outflow:

$$S = T_s O$$

By substitution, the following form of the equation may be derived. This is the form used to compute outflow from prototype "lake" increment:

$$O_2 = O_1 + t\,[(I_m - O_1)/(T_s + t/2)] \;.......... (1)$$

t = time duration of computational period.

T_s = Time of storage.

5.5.1.4.2 Precipitation Input for Watershed Model

1. Station weighting

Precipitation on a watershed may be computed from period precipitation amounts at one or more stations as follows:

$$WP_n = (P_1 * W_1 + P_2 * W_2 + ...P_n * W_n)/n$$

where

$P_1, P_2.....P_n$ = Period precipitation amount at stations 1, 2......n respectively in inches/mm.

n = Total number of stations; a maximum of 30 stations may be used for any one watershed.

WPn = Weighted period net watershed precipitation in inches/mm.

Stations weights are usually calculated on the basis of either percent of area indexed by the station using Thiessen Polygon's, or by a ratio of normal annual precipitation of that portion of the basin to normal annual precipitation of the index station.

Daily precipitation can be distributed over eight three hour periods as specified by the user. The net weighted precipitation distributed for a period is determined as follows:

$$WP_d = WP_n * DIST$$

Where

WPd = Weighted net precipitation distributed for a given period.

DIST = The percentage of daily precipitation for that period, in percent.

2. Soil Moisture — Runoff Relationships

Rain falling on a watershed must either (a) runoff, (b) be retained in the soil system, (c) be intercepted and evaporated from trees and other plants, (d) evaporated from ponds, lake and stream surface, (e) evaporate from the soil, (f) return to the atmosphere by transpiration from trees and other plants, or (g) percolate into the ground water system and thereby loss to the surface water system. If item (g) is considered negligible, the rainfall may be divided into:

(1) Runoff

(2) Soil moisture increase

(3) Evapotranspiration losses.

Runoff

The percent of total rainfall input available for runoff is found from empirically derived relationships of soil moisture index (SMI) versus runoff percent (ROP). If desired, rainfall intensity may be included as third variable in SMI-ROP relationship. The total generated runoff for a period (RGP) is computed as follows:

$$RGP = ROP * (WPn \text{ or } WP_d)$$

where

RGP = Generated runoff for the period in inches/mm

ROP = Runoff percent (from SMI + table).

WP_n = Weighted net precipitation or distributed.

WPd = net precipitation for the period.

The generated runoff (RGP) is then partitioned into three components; surface, sub-surface and base flow.

3. Soil Moisture Changes

The soil moisture index (SMI) is indicator of the relative soil wetness and is used to determine runoff. The SMI is depleted only by the evapotranspiration index (ETI). The ETI can be specified either in table form as month versus daily potential evapotranspiration (inches/mm per **day**), or as weighted daily pan evaporation data (inches/mm per day) at one or more stations. **SMI is** calculated at the end of each period as follows:

$$SMI_2 = SMI_1 + (WP-RGP)\left[\frac{PH}{24} * KE * ETI\right]$$

where

SMI_1 = Soil moisture index at beginning of the period

SMI_2 = Soil moisture index at end of period.

PH = Period length in hours.

ETI = Evapotranspiration index.

KE = A factor for deducing ETI on rainy days, specified to the computer in a table of KE versus rate of precipitation in inches/mm per day.

When accurate accounting of soil moisture changes is required, daily ETI values are used instead of monthly average value. The weighted ETI (ETI_d) can be distributed over eight periods per day as previously described for precipitation.

The effects of rainfall on ETI are inherent to the daily pan evaporation data. Using ETI as daily input, SMI is calculated as follows:

$$SMI_2 = SMI_1 + (W_p - RGP) - (DKE * ETI)$$

where

DKE = A factor for reducing the daily ETI when soil moisture is depleted.

4. Base Flow Separation

The portion of runoff that contributes to base flow is a function of base flow infiltration index (BII). A relationship is provided to the computer as a continuous function in the form of a table. The base flow component is computed as the product of base flow percent (BFP) and runoff rate (RG). The balance is input to surface and subsurface runoff (RGS) is computed as follows:

$$RGS = RG\ (1.0 - BFP)$$

The BII versus base flow percent (BFP) table may have a third element, a base flow input limit, which is specified in inches/mm per hour. The base flow infiltration index goes on changing continuously and at the end of each computed period is computed as follows:-

$$BII_2 = BII_1 + (24^* RG - BII_1)\frac{PH}{TSBII + PH/2}$$

where

BII_1 = Base flow infiltration index (in inches/mm per 24 hours) at the beginning of the period.

BII_2 = Base flow infiltration index at end of the period.

RG = RGP/PH runoff rate in inches/mm per hour.

TSBII = Time delay or time of storage for calculation of change in BII. Separate values may be designed for rising and falling flows: Typical values range from 30 to 60 hours.

5. Surface Sub Surface Flow Separation

A table of surface runoff input (RS) versus total input to surface and subsurface runoff (RGS) is specified to the computer for a particular basin. The relationship commonly used is based on the following assumptions:

1. The minimum surface component is 10% of the total generated runoff (RGS).

2. The subsurface flow component reaches a maximum (KSS) and remains constant for input rates (RGS) above 200% of KSS where

   KSS = Maximum sub surface input rate (in inches/mm per hour)

6. Routing of surface subsurface, and base flow

Each component of runoff excess to surface, sub-surface and base flow is computed as input rate, expressed in inches/mm per hour. Each period value is converted to the equivalent inflow rate in cubic *feet/meter* per second, based on the drainage area and the length of the period, in hours. Each component of inflow is routed through a specified number of increments of storage. These increments can be considered as a series of small lakes which delay runoff.

Channel routing

The time rate of change of streamflow in a river reach is evaluated by first dividing the reach into a series of increments or a "chain of lakes". The routing method for each of the "lakes" is described by equation. Outflow from each increment is used as inflow to the next downstream increment and the step-by-step procedure completed for each increment for each time period.

Time of storage for channel routing increments may be specified in a table of time of storage versus discharges, it may vary inversely or directly as a power function of discharge. Normally when flows are confined to the channel, the time of storage decreases as discharge increases; and it is convenient to express the relation from the equation;

$$TS = \frac{KTS}{QN}$$

where

TS = The time of storage per increment in hours.

Q = Discharge in cubic feet/meter per second.

n = a coefficient usually between -1 and 1

KTS = A constant determined by trial and error or estimated from physical measurement or flow and corresponding routing times.

Natural lake routing, reservoir routing and regulation, and routing streamflow in backwater mode also can be accomplished conveniently.

5.6 CWCFF1 Model

Based on the experience gained in real time application and the short comings of the previously mentioned four mathematical models e.g. SSARR, HEC 1F, NLC and NAMS11F, on Yamuna, a new conceptual model CWCFF1 (Central Water Commission Flood Forecasting 1) has been developed in the Upper Yamuna Circle, Central Water Commission. The Model has been developed by Shri T.K. Mukhopadhyay, Deputy Director and Shri Inderjit Rishiraj, Sr. Prof. Asstt., under the guidance of Shri R.S. Prasad, Director and Shri R. Rangachari, Member (RM). A paper "New Conceptual Flood Forecasting Model "CWCFF1" by S/Shri R. Rangachari, R.S. Prasad, T.K. Mukhopadhyay and I. Rishiraj, has been submitted to Technical Conference on the Hydrology of Disasters, Geneva, 2-3 Nov. 1988 for publication.

Concept of the CWCFF1 Model is based on the fact that water tends to maintain uniform level under free condition. The entire river reach is divided into small slices. Balancing of water is dependent on the average volume of water in two adjoining slices and a factor connecting the slope of the river beds and its characteristics. Losses of water into the dry river beds and the effect of the spreading/spilling of the flood water is also taken into consideration. This model has been able to simulate continuous floods consisting of both low flows and high flows in the Yamuna upto Delhi. It was also able to accurately simulate the high peak after a prolonged low flow, particularly the effect of the varying travel time, which the other models are lacking.

The model is proposed to be tested in river Yamuna during 1988 monsoon and thereafter in other Indian rivers.

The details of the model are given in Appendix V

5.7 Data Base Management System

Data base management system is an organised collection of related information of data that makes it possible and convenient to perform one or more tasks like processing, verification and retrieval of data for use for a particular purpose.

Data base management system can be maintained manually or through digital computers. For a manual system, it involves lots of registers, files, ledgers which cause very much inconvenience to the user. Due to its volume, the process is very slow and may often lead to errors beyond detection. But in the case of a digital computer, the process is very fast and lots of error checking procedures can be suitably applied for verification and processing of data. The retrievel process is also very fast.

In Water Resources/River Management, the problems of data can be of different natures.

(i) Time series data like precipitation, temperature, water level, river discharge etc.

(ii) Physical data like catchment areas, topography, river cross-section and drainage path etc.

(iii) In River Management system, the problems encountered, may be of the following types:

1. Real time use in operational Hydrology for flood forecasting and reservoir regulations.

2. Studies for planning purposes, maximum probable storms analysis, calibration of model parameters and other related matters in connection with design of flood control structures and river valley projects.

For operational hydrology, the data should be available with the computer on real time basis. The processing, verification and retrieval of data should be very fast and does not involve wide range of data. Whereas for the other purposes, the data requirement is quite large and of the order of several years. Keeping in view the above facts, a real time data management system has been developed to facilitate storage of data on real time basis and its use for flood forecasting purposes. This system supports retrieval of historical data for wide range of time period.

The family of the programmes "DMMR" (DMMR is Data Management system by Mukhopadhyay and Rishiraj) is a generalised data management system for collection, storage, processing and retrieval of hydrometeorological data on real time basis for application in operational hydrology. These programmes have been developed in FORTRAN-77 language on HP-1000 F-series computer to support real time hydrometeorological data collection from remote stations through telemetry system and processing of the same for formulation of flood forecast by using various mathematical/ simultation models, particularly in Yamuna catchment above Delhi. The programmes accomplish the following functions:

(i) *Initialisation of System*

Data base can be created for any river basin system to support collection/ manipulation of different types of data.

(ii) *Transfer of data from telemetry system*

If there is a provision for telemetered data collection link, the programme transfers data from telemetry system to operational files.

(iii) *Transfer of data from other sources*

This is done manually in a alternative way to enter data as soon as it is received by the operator.

(iv) *Verification of data*

Correctness and continuity of data is verified and operator is given an option for supply of current observed/simulated/estimated data. These types of data are stored in the data file with an appropriate source code from which source the data is received.

(v) *Retrieval of data*

Data can be copied on screen, output device or any computer file.

(vi) *Preparation of input Data*

Given the current time, the required data is retrieved and input data deck prepared for real time run of various flood forecasting models as opted by operator. The models that can be run are SSARR, NLC, HEC-1F and NAM-S11F.

(vii) Ar*chiving*

Data from operational file are stored in archival file for future record.

(viii) *Operation of flood forecast*

The flood forecast issued by different mathematical models are stored in a file for comparative studies. The performance of flood forecasting models can be plotted in the same paper.

(ix) *Graphics library*

The system supports a group of sub-programmes for plotting of the real time flood forecast as well as plotting of the simulated hydrographs which are needed for calibration of the models.

5.7.1 Data Management File Structure

Operational data files are created during the installation of "DMMR System". Data files are created for each individual station for a particular type of data e.g. hourly water level observed through telemetry for a particular station, incremental rainfall data through telemetry or manual observations, daily rainfall data through telemetry or manual observations, and daily rainfall data, for a particular station etc.

The existing system can create approximately 300-Files. Each data file is Direct Access File to store data of one type for one station. The name of the file is automatically created by the programme from the information supplied by the users during initialisation. Each file contains 368 records of 120 characters long. Twenty four data values and 24 source codes for each day's data are stored in free FORMAT in one record for real time operation. To keep the storage and retrieval operation fast, the operational data files are created as Random Access Files but these files cannot be stored directly on tape for retrieval. For this purpose, file archiving programme converts the random access files to sequential files for easy storage and future retrieval.

5.8 Dambreak model

5.8.1 Introduction: The modelling for Dambreak studies involve the simulation of flood wave propagation following a dam failure. The following fields are required to be modelled for detailed study of the system;

1. Near field – area immediately up and down stream of the dam wall.

2. Far field – the area, a few kilometer down stream of the dam to the river including the flood plain.

The routing model (system 11) already set up for flood forecasting and flood control models is capable of taking care of far field situation as it involves one direction and one layer system.

The modelling of near field's areas involve two horizontal dimensions, hence system 11 is not capable to describe the situation. System 21 (two dimensions and one layer) model is used for simulation of hydrodynamic flow in near field. The details of system 11 have been explained in section 5.2.1.1. Before explaining the system 21, it would be nice to describe different aspects of dambreak simulation.

The estimation of flows through dambreak is very sensitive to variations in the flood input. These inputs depend on the delineation of the mode, rate and extent of the failure. An insight can be gained by listing a number of failure modes:-

1. Instantaneous total failure — a possible failure mode for concrete area dam and a very conservative assumption for any gravity dam.

2. Leakage around abutments or through foundation of concrete gravity dams. In an extreme case a large gap may be eroded around abutment, but the failure will be far from instantaneous or total.

3. Piping failures of earth and rockfill dams.

4. Landslides into dams. This will cause a large wave to spillover the dam wall with subsequent flooding.

5. Earth Quake — This is not implict in the above cases from 1 to 4. It may cause all or any of the causes.

5.8.2 Description of System 21

The hydrodynamic stage of system 21 involves the equation of conservation of volume and momentum in two horizontal dimensions. The following effects are in the equations:-

1. Conservative and cross momentum

2. Wind shear stress at the surface

3. Barometric pressure gradients

4. Corblis forces

5. Momentum dispersion

6. Evaporation

7. Break water porosity

8. Radiation boundary conditions

9. Boussinesq terms for short wave simulations.

10. Sources and sinks (both mass and impulse).

The equations are solved by implicit finite difference technique with variables defined on a space staggered rectangular grid.

A 'fractioned step' technique combined with an Alternating Direction Implicit (ADI) algorithm is used to avoid the necessity for iteration. Third order accuracy is ensured through the centering in time and space of all deviations and co-efficients. ADI algorithms mean that at each time step a solution is first made in the X-direction using the continuity and X-momentum equation followed by a similar solution in the Y-direction.

Input data requirement for these studies are as under:-

1. Dam failure mode
2. Maximum flooded areas
3. Rate of fall of reservoir's water level after start of the breach.
4. Time series of water levels and discharges.
5. Topographical data.

The model has three calibration factors namely bed resistance, wind friction factor and momentum dispersion coefficient (normally called the eddy co-efficient). The eddy coefficient which is a function of flow field can be calculated by various methods. Using these factors alone, calibration of a model is normally quite easy, in practice, the calibration of the model depends far more on the accuracy of data. (e.g. bathymetry, boundary data and wind speeds) than on an exact determination of the coefficients.

A number of mathematical models for dambreak analysis are available. The model developed at Danish hydraulic Institute at Denmark is one of them. One important reason for proposed DHI model is that CWC is already using very similar models through cooperation between CWC and DHI. This model will be set up and knowhow will be transferred to CWC during phase-III of the Project.

The other models like the DAMBRK model from US National Weather Service is probably just as adequate and easier to use.

This modelling system will help for assessment of the following points judiciously:

1. Review dam safety measures for old and new dam
2. Forecasting the effect immediately downstream of the dam as well as its effect in the far off areas in flood plain.

3. Demacration of flood plain in the valley with reference to warning time available.

5.9 Sediment Transport Model

5.9.1 Introduction:

As most of the Indian rivers are transporting sediment all along its course, the beds of the rivers get silted thereby changing the topographical condition of the river system. That is why sediment transport model is a prerequisite for long term predictions of consequences of alternative flood control schemes in alluvial rivers. Furthermore, a sediment transport model may improve the real-time forecasting of water levels at sites with movable river beds and thus changing rating curves. It has been visualized that the computerized or the conventional tool for flood forecasting cannot produce results of high accuracy, if rating curves are not stable. The modelling system has been taken up for application on Lower Damodar in the phase-III of the ongoing CWC-DHI collaboration Project as an extension of the system 11 hydrodynamic model which was set up in the Damodar and Yamuna for improvement of techniques for flood forecasting and flood control.

5.9.2 Theoretical background:

The present version of sediment transport modules is based on equations for noncohesive sediment transport in alluvial rivers, but due to the presence of the full transport dispersion description, it is easy to extend the formulations to account for cohesive sediments also. The model can simulate either the total sediment transport or the suspended and the bed load separately. A sediment budget module makes it possible to assess the sediment/erosion rates. The module is formulated according to the present state of the art of theoretical sediment transport research. At present, not all aspects of the complicated physics for sediment transport are well understood and existing formulations are all founded upon simplified hypothesis.

As a consequence thereof, the module is not limited to a single sediment transport formulation, but can apply alternative descriptions. In this way, it is not designed as fixed tool but rather as a flexible tool, which can support a sediment transport expert in his analysis.

This module may run in parallel with hydrodynamic module so as to allow for a feedback from redistribution of materials to hydrodynamic phenomena. The variation in bed form can cause significant change in bed resistance, which can be calculated from dune dimensions, water depth and the specific discharge.

5.9.3 Sediment Transport Equations:

The theoretical background for the sediment transport description in module is the comprehensive experimental and theoretical research which has been carried out at Technical University of Denmark by Engelund and Hansen Fredsoe. Two different sediment transport equations are applied in this model namely (i) The Engelund-Hansen equation and (ii) Engelund-Fredsoe theory. The first provides only the total sediment transport, while the second devides the sediment transport into suspended and bed load transport.

As per the Engelund-Hansen transport equation

$ø$ = dimensionless sediment transport

$$= 0.08\,\theta\,5/2/f = qt\sqrt{(S-1)gd^3}$$

where f is the friction factor: $f = 2\,U^2 f/v^2$

$$\text{and } \theta = \frac{\tau/p}{(S-1)gd} = \text{dimensionless total bed shear stress.}$$

As per the Engelund-Fredsoe theory for bed load transport

$$ø = 5\left[1 + \left(\frac{\pi/6\beta}{\theta' - \theta_c}\right)^4\right]^{-\frac{1}{4}}\left(\sqrt{\theta'} - 0.7\sqrt{\theta_c}\right)$$

Where $\theta' = \tau/\rho\ (S-1)\ gd$

$\theta_c = \tau_c/\rho\ (S-1)\ gd$ = critical shields parameter

and for the suspended sediment transport

$$q_s = \int_a^D C.U.dy$$

$$\text{where } U = 2.S.U'f\,\text{In}\left(\frac{30Y}{K}\right) = \text{current velocity profile}$$

$$C = C_b\left(\frac{D-\gamma}{\gamma}\cdot\frac{a}{D-a}\right)^z = \text{concentration profile}$$

The near bed concentration, C is found on the basis of the dimensionless bed shear stress τ and is again used to calculate the associated hydraulic resistance. The equilibrium dune height and dune length are formulated on the basis of the suspended and bed load transport. If the dune dimensions are known, the bed shear stress can be calculated from the water depth and specific discharge. Total shear stress τ can be divided into a part resulting from skin friction on the gently curved upstream side of the dune and a portion which is caused by the form drag of the dune.

The equations are solved as fully time centred implicit finite difference equations by using the double sweep method. For non-cohesive sediment transport where upstream and down-stream influence on the local transport capacity is negligible, the solution degenerated to a simple explicit calculation.

The equations for flow resistance are solved by iteration on each time step. Convergence is obtained within a few iterations.

5.9.4 Input Data Requirements:

Input data to the system consists of the following:-

(1) Sediment inflow at upstream boundaries (may be omitted if not readily available).

(2) Geometric standard deviation of the sediment grain size.

(3) Fall diameter.

(4) Initial dune height and dune length (only important during the initial part of the simulation).

(5) Viscosity of water.

(6) Calibration parameters for suspended bed deposition, sediment distribution at model points and bottom resistance.

If the Engelund-Hansen theory is applied, only the sediment diameter has to be specified.

5.9.5 Organisation of the System:

Both hydrodynamic and sediment transport module run in parallel and subsequent steps. Different time steps can be applied in the two modules. Normally larger time steps are used in the sediment transport module. For each time-step, a sediment budget calculation is performed, calculating the net deposition or erosion from the local transport capacity and transport received from neighbour elements. Where several branches meet at nodal points, the total inflowing sediment transport can be divided amongst the outflowing branches either in proporation to the outflowing discharges or on basis of any user specified distribution pattern.

5.9.6 Output from the Model:

Output from the system consists of time series for sediment concentration, net deposition and erosion, dune pattern and inflow resistance at each grid points. From the net deposition and erosion rates, the river cross-sections can be updated according to a chosen distribution pattern, or preferably to a distribution pattern based on actual observations from the particular location.

With the help of these outputs, the impact of the changing beds on cross-section of rivers can easily be found out for modification of the rating curves at the time of formulation of forecasts or examining the flood control schemes.

CHAPTER-VI

FORECAST EVALUATION

6. Forecast Evaluation

6.1. Purpose of Evaluation

In the ideal situation the actual flood level and its time of occurance should be same as had been predicted. This may not happen every time and some of the forecast levels may either vary from the predicted levels or from the predicted time. This variation within some limits might even be advantageous under certain circumstances like the actual level attained being less than the predicted level or high peak was recorded later than the predicted time but if it is other way this could be dangerous. Evaluation of the forecasts is an important step to study whether the forecasts had served the purpose or not.

6.2 Verification of forecast

Verification of the forecasts is possible either graphically or by numerical means. Subjectivity in graphical methods can be dispensed with by adopting numerical evaluation procedure. The main purposes of forecast verification are :-

1. To find the degree of accuracy and inaccuracy of the forecast.

2. To find out possible reasons for error and to suggest improvements in the existing system.

3. To select the most appropriate forecasting methods: and

4. To determine the most appropriate model parameter(s).

A forecast is considered to be accurate if the difference between the forecast level and the corresponding observed level is within a permissible extent of deviation. But the permissible range of deviation need not be the same in every case.

6.3 Present criteria

A simple and common criteria is being adopted presently in CWC to evaluate the forecast performance. In the case of river stage, forecast of ± 15 cm. variation between forecast level and actual level is allowed and similarly 20% of inflow is allowed in case of inflow forecasts. In real life, pattern of peak flood could differ from river to river. In rives like Ken at Banda, Vamsadhara at Gunupur etc. which get flash floods, the travel time, is generally very short viz., a few hours only. In some of the forecast sites like Banda on Ken, the actual rate of rise in flood level has been found to be very rapid (3.5 m. in 24 hours). Hence is such cases the margin of 15 cm may be too low and not be justified. In some other sites like the Ganga main stem downstream of Allahabad the rate of rise is 5-10 cm. in 24 hours and as such 15 cm. may be a liberal and high figure to be allowed for variation. Similar consideration regarding volume of inflow can lead to different yard sticks for different inflow forecasting sites. However, the general practice is to follow the 15 cm. (or variation of 20% inflow) and make further studies as well depending on the circumstances.

6.4. Evaluation of forecasts by simple methods

6.4.1. Criteria for verification

The commonly used criteria for forecast verification are as follows:-

i) Relative Error

$R = (Y_o - Y_f)$

ii) Absolute Error

$A = I\ (Y_o - Y_f)\ I$

iii) $E = 1 - \dfrac{(Y_0 - Y_f)^2}{(Y_0 - Y_n)^2}$

Where Y_o = Actual level at (N +T)th time

Y_f = Forecast level at (N+T)th hour

Y_n = Nth hour level on the basis of which the forecast has been formulated

E = A measure of efficiency.

6.4.2. Examples

Some of the typical forecasts were used as examples to explain the evaluation criteria and interpreting its results.

Example-I

i)	Water level at 'N'th hour at forecasting station	44.38 m.
ii)	Predicted water level for (N +T)th hour (T = Travel time in hours)	45.50 m.
iii)	Water level actually observed	44.78 m.
iv)	Difference between actual and forecast level (iii–ii)	(–) 0.72 m.

As per present practice the evaluation is considered with respect to the level, which is minus 72 cm in the present case. The difference is beyond the limit of ±15 cm. The deviation is, therefore, unacceptable.

Example- II

i)	Water level at 'N'th hour at forecasting station	46.78 m
ii)	Predicted water level for (N+T)th hour	46.76 m.
iii)	Water level actually observed	46.77 m.
iv)	Difference between actual & Forecast level (iii–ii)	(+) 0.01 m.

The deviation of 1 cm is well within the permissible limit of ± 15 cm.

Example-III

i)	Level/capacity at Nth hour at forecasting site	$\frac{99.35\,\text{m}}{5360.70\,\text{MCM}}$
ii)	Predicted inflow upto (N +T)th hour at forecasting site	200MCM
iii)	Level/capacity at (N+T)th hour at forecasting site	$\frac{99.93\,\text{m}}{5592.88\,\text{MCM}}$
iv)	Actual storage received in the Reservoir during 'T' hours.	232.18 MCM
v)	Actual outflow from the Reservoir during 'T' hours.	4.16 MCM.
vi)	Net inflow received in the Reservoir during 'T' hours	236.34 MCM
vii)	Difference between actual inflow & Forecasted inflow (vi–ii)	(+) 36.34 MCM
viii)	Error in inflow forecast $\frac{36.34}{236.34} \times 100 = 15.38\%$	

(Actual inflow is more than forecast inflow by 15.38% but is within ± 20% limit of accuracy).

Example IV

During the year 1978, a total of 16 forecasts were issued at Coronation Road Bridge site on Tista river. The forecast and the corresponding observed values of the water stages are given in Table 6.1 below:

Table VI.4.1

S.No.	*Forecast level in m(Y_f)*	*Corresponding observed level in m (Y_o)*	*Relative Error in m (Y_o - Y_f)*
1.	149.50	149.40	-0.10
2.	149.95	149.75	-0.20
3.	149.50	149.40	-0.10
4.	149.60	149.00	-0.60
5.	149.90	149.80	-0.10
6.	149.90	149.95	+0.05
7.	149.90	149.20	-0.70
8.	149.75	149.30	-0.45
9.	150.10	149.50	-0.60
10.	149.85	149.70	-0.15
11.	150.50	150.00	-0.50
12.	149.50	149.60	+0.10
13.	149.45	150.40	+0.95
14.	150.60	150.70	+0.10
15.	149.65	149.10	-0.55
16.	149.55	149.50	-0.05

The summary of the forecast performance is given below in Table VI.4.2.

Table VI.4.2

Range for error	*No. of forecast*	*%age Accuracy for given range*	*Cumulative percentage of accuracy*	*Remarks*
0 to ± 5 cm	2	13	13	Very accurate
> ± 5 cm to ± 10 cm	5	31	44	Fairly accurate
> ± 10 cm to ± 15 cm	1	6	50	Sufficiently accurate
Beyond ± 15 cm	8	50	100	Incorrect

From the data, it is observed that for the same forecast error (Y_o–Y_f) a fast rising river will have a higher 'E' value. Closer the value 'E' to 1, higher is the effectiveness of the forecast issued.

6.4.3. Warning Time & Forecast

The availability of warning time is very closely related to the accuracy of forecast. For example, consider the case of river Ganga at Farakka. For this site a warning time of about 40 hours is available. Therefore, one can very easily wait for a few hours to collect necessary data and the forecast can be subsequently modified with availability of more and more data, increasing the accuracy of forecast. But in case of Teesta the warning time available is only about 6 hours, the forecast has to be formulated with the help of whatever meagre data are available which may lead to the forecast not being so accurate. Again for very small and flashy rivers, the accuracy of the advisory forecasts may not be measured with the same yardstick.

And hence, the measure for forecast accuracy should be such that:-

(i) The varying nature of flood characteristics are duly taken care of

(ii) Some weightage is given to the available warning time; and

(iii) Various types of forecasts viz. advisory forecasts, revised forecasts etc. are not treated with the same yardstick of accuracy while judging the overall performance of a forecasting centre.

6.4.4 Trend of Errors

Using the various criteria for forecast verification, it is possible to find out the trend of error and possible reasons for the same and suggest possible improvement. As an example the forecasts at Coronation Road Bridge are considered.

It is observed from Table VI.4.1 that most of the errors are negative i.e. the forecast values are more than the observed values. The sum of relative errors R i.e. (Y_o-Y_f) works out to be (-) 2.90 m. In other words, on an average, relative error for forecasts are 18 cm. (=290/16) higher. It may be seen that in one case (Sl.No. 13) the error in forecast is about 95 cm. It is considered as an abnormal case for some specific reason, and if this is deleted then the average relative error works out to be (-) 25 cm. which is also much beyond the permissible range.

In order to identify the possible reasons for error, the forecasting technique was examined. For issuing the forecast on River Teesta at Coronation Road Bridge, the discharge at Khanitar at Nth hour, discharge at Singla Bazar at Nth hour and the discharge at Rangpo at Nth hour are individually calculated through corresponding G & D curves. These discharges are added up and assumed to reach the Coronation Bridge site after four hours in normal conditions. In this case simple translation is considered and no attenuation effect is taken into account. Theoritically, there is bound to be some attenuation. This may be one of the reasons for errors.

So attempts may have to be made to account for reduction in the discharge due to attenuation effect of the channel by developing routing equation at some of the important forecast stations.

6.5. Comparison of various methods.

The forecasts for Dibrugarh site on river Brahmaputra are at present issued with the help of a coaxial relation. A mathematical model (described elsewhere) has also been developed. During the year 1979, both the methods where attempted. Table VI.5.1 gives the forecast level by the two methods and the corresponding observed level.

Table VI. 5.1

Sl.No.	*Date*	*Observed water level in m.*	*Forecast level as per graphical relation in m*	*Forecast level as per mathematical model in m.*	$(Y_o-\bar{Y}_o)$	(Y_o-Y_{fg})	(Y_o-Y_{fm})
1	2	3	4	5	6	7	8
		Y_o	Y_{fg}	Y_{fm}			
1.	17.5.79	104.24	103.94	103.72	-0.31	+0.30	+0.52
2.	11.6.79	103.78	103.78	104.27	-0.77	0.00	-0.49

Sl.No.	Date	Observed water level in m.	Forecast level as per graphical relation in m	Forecast level as per mathematical model in m.	$(Y_o-\bar{Y}_o)$	(Y_o-Y_{fg})	(Y_o-Y_{fm})
1	2	3	4	5	6	7	8
3.	30.6.79	104.73	104.67	104.60	+0.18	+0.06	+0.13
4.	3.7.79	105.19	105.16	105.06	+0.64	+0.03	+0.13
5.	21.7.79	104.36	104.55	104.49	–0.19	+0.19	–0.13
6.	4.9.79	104.82	104.79	104.81	+0.27	+0.03	+0.01
7.	7.9.79	104.73	104.76	104.79	+0.18	–0.03	–0.06
	$\Sigma(\ )^2$				1.2724	0.1324	0.5649

$\bar{Y}_o = 104.55$ m

6.5.1. Evaluation of Forecasts by other methods

Verification Criteria

When two or more methods of forecasting are to be compared or the parameters of a particular method are to be optimized, it can be very easily done by evaluating "measures of efficiency" R^2.

$$R^2 = 1 - \frac{F^2}{F_d^2}$$

Where $F^2 = (Y_o - Y_f)^2$

$F_d^2 = (Y_o - \bar{Y}_o)^2$

Where $\bar{Y}_o$ = mean of the observed values

The use of the above parameters is illustrated with the help of following example:

6.5.2. The methodology to compare the efficiency of the two forecasting methods is illustrated below for Dibrugarh site.

The procedure to evaluate R^2, the measure of efficiency is also described below:-

a) For Graphical Method

$\bar{Y}_o = 104.55$

$\Sigma(Y_o - \bar{Y}_o)^2 = F^2_{dg} = 1.2724$

F_{dg} refers to deviation in graphical method

$\Sigma(Y_o - Y_{fg})^2 = F_g^2 = 0.1324$

F_g refers to graphical method

Therefore, efficiency R_g^2 for graphical method

$$R_g^2 = 1 - \frac{F_g^2}{F_{dg}^2}$$

$$= 1 - \frac{0.1324}{1.2724}$$

$$= 0.89$$

(b) For mathematical Model

$\bar{Y}_o = 104.55$

$\Sigma(Y_o - \bar{Y}_o)^2 = F_{dm}^2 = 1.2724$

$\Sigma(Y_o - Y_{fm})^2 = F_m^2 = 0.5649$

F_{dm} refers to deviation in values of F by mathematical model
F_m is value of F by mathematical model

$$R_m^2 = 1 - \frac{F_m^2}{F_{dg}^2}$$

$$= 1 - \frac{0.5649}{1.2724}$$

$$= 1 - 0.44 = 0.56$$

Hence, out of two methods applied to forecast for Dibrugarh site, graphical method appears to be more reliable.

CHAPTER-VII

FORECAST DISSEMINATION

7.1 Introduction

Dissemination of flood forecasting and warning system is very important as the users are apprised of the notice of the incoming floods well in time for making arrangement for safeguarding the life and the properties from the destructive activities of flood and arrangement for flood fighting etc. The forecasting operation must be thought as a "System" embodying all the things which are necessary to reduce flood damages, not only hydrological analysis, computer operations, or data collection etc. This system has been linked to a chain of many links.

Links in the Chain

The important links can be indicated as:

1. Reliable field data must be collected well in time.
2. Field data must be transmitted to the forecast office quickly.
3. The forecaster must make a *Forecast.*
4. The forecast must be *Accurate.*
5. The forecast must be transmitted to the *Ultimate User.*
6. The forecast must be transmitted *Quickly.*
7. The forecast must be transmitted *Completely and Accurately.*
8. The user must be able to *Understand the Forecast* without warbling.
9. The user must *Believe* the forecast.

10. The user must *Know* what to do further.

11. The user must *Do it.*

Just as any chain becomes in-effective even if one link is missing, the forecast operation accomplishes nothing in a particular flood situation if any of these links of forecast is missing. In order to reduce flood damage all the processes must happen. Forecast dissemination includes links 5,6,7,8 & 9.

Link-5 The flood forecast organisation of CWC should not only disseminate the formulated forecast to the user through a series of people/organisations but the forecasting unit should also maintain a continuing and close relationship with all of them so that it can keep itself be aware of the importance of forecast properly, completely and accurately.

Link-6. During floods, dissemination should be made frequently as often as the forecasts are made, updated or revised. If the revision is done frequently, the effect will confuse people rather than inform them. So the forecaster should be cautious while issuing a forecast, as forecast once issued cannot be taken back. The forecaster should be aware of the schedules and the deadlines of dissemination systems as delay of even a few minutes in issuing the forecast may delay its receipt by the public for hours.

Link-7. The utility of the flood forecast is dependent on both accuracy and timeliness. Underprediction may lead to a dangerous situation resulting in damage of life and properties in the areas, victim of mistaken and misplaced confidence. On the other hand, overprediction may result in avoidable and unnecessary basic evacuation, flood fighting measures including costly engineering measures and distress among the people. Timeliness is also equally important. It should be borne in mind that the value of flood forecasting is zero at the time of actual occurence of the event. Those who handle the flood forecast statement on its way from the forecaster to the user often have rigid space and time constraints. It is important to take note of likely constraints. If the intermediary agencies modify and bridge and reword the statement, the meaning of the forecast may perhaps change. Therefore, it is some times necessary to prepare different forecast statements for different modes of transmission.

Link-8. The forecast statement should be kept as simple as possible. The following type of statements may be considered:

"The stage of the river Yamuna at Delhi Rly. Bridge is 206.0 metre at 0800 hours of 16 July. It is presently rising at a rate of 10 cm. per hour and will reach a peak of approximately 207.10 m at 0700 hours on 17th July morning. This is 1.17 m above danger level and 0.39 metre lower than the level reached during flood of 6th Sept. 1978". This contains a great deal of useful information. The comparison of a forecast peak with recent floods is a useful technique to help people who do not know how their own location relates to the river gauge. It is a fact that an average person receiving information by spoken words over radio for instance, cannot mentally

ASSAM FLOODS DURING AUGUST, 1988

A scene from the flood waters which entered in middle Brahmaputra Division, Central Water Commission, Guwahati, submerging the complete wireless section.

ASSAM FLOODS DURING AUGUST 1988.

People seen taking refuge on the main Guwahati Tezpur road for fear of flood waters at Kolangpara near Sonapur in the District of Kampur, (Assam).

absorb and retain more than a few numbers while the above statement contains eight numbers. So the message for transmission by radio, television or microphone could be better worded somewhat like this.

"The river Yamuna at Delhi Railway Bridge is rising at a rate of 10 cm per hour and is expected to rise about 110 cm today and be near 39 cm below the highest flood level of Sept. 1978 by tomorrow morning."

Link-9. It is also the duty of the forecaster to issue forecast in such a way that the user may have reliance on them. A forecast has also to take into account psychology as much as hydrology. If the user does not believe the forecast he will not act upon it and no damage reduction will be accomplished.

7.1.1 Flood Fighting

The organisation responsible for flood warning and flood fighting should be informed about the incoming flood as early as possible so that the required action is planned and activities set into motion with least possible delay. They should also be kept informed of the propagation of flood wave and any change in the present as well anticipated flood situation with respect to time. This information which is supplied by the flood forecasting unit of CWC in the form of *Flood Forecast Bulletin* must be very clear and include necessary details so that a very realistic picture of the incoming danger is depicted. The forecast should be properly checked before issue in order to avoid dissemination of wrong forecast including even inadvertent mistakes because hardly any time is left for review between receipt of forecasts and start of chain of activities which follow:

The measures adopted for flood forecasting and disaster prevention are related to:

(i) Education and Warnings.

(ii) Emergency actions.

(iii) Road traffic disruption

(iv) Public utility disruption

(v) Raising level of Houses

(vi) Land use regulation and

(viii) Flood warnings.

(i) Education, publicity and warning in flood prone AREAS

Preflood awareness by education and publicity is very important which extends to link the warning system concerning dissemination or communication of warnings. The effectiveness and the awareness of flooding and appropriate action are among two main groups. These are firstly, the officials in the dissemination process who receive and then relay messages and, secondly the recipient public members, who are in danger.

Although the awareness of both the groups i.e. the officials and the public, cannot stop the flood it will reduce substantially the flood damages and will save human lives and cattle.

The first group of officials can further be divided in two subgroups.

(i) Officials within the flood forecasting agencies issuing the flood forecast.

(ii) Officials in different agencies who receive and are responsible for relaying messages onwards and members of the general public who are in imminent danger.

Each of these groups of the "Warning response system" must be aware of the likely areas of flood damage and appropriate action. Pre-flood education primarily comprises, firstly, receiving awareness of the possibility of a flood event and, secondly receiving awareness of what actions should be taken prior to and during a flood. The latter involves people's knowing their personal roles in the event of flood.

(ii) Pre-Flood Education Techniques for Increasing Public Awareness of Flood Risks and Appropriate Responses

Local newspaper articles,

Circulation of flood information brochures and leaflets,

Placing information and telephone numbers in the "*Yellow Pages*",

Providing public information via water bills,

Dissemination of flood hazard maps,

Placing posters in public places,

Mobile exhibitions,

Radio broadcasts (including operative sessions with "Experts"),

Telephone answering service which is published (Pre-recorded or live responses),

Films/Video tape given to local groups/School children Television programmes,

Television advertisements,

Public lectures,

Society, Club, School visits to forecasting centres,

Public seminars,

Rehearsals and tests,

Providing flood hazard materials in towns or beside roads,

Painting lines on buildings to designate elevation of the greatest historical flood,

Requiring estate agents to disclose flood prone properties,

Print notices of flood vulnerability on maps for sale.

7.1.2 Type of Forecast

The dissemination system should be able to reflect the forecast capability of the flood forecasting organisation. Thus for example the speed with which a forecast can be prepared, influences the medium chosen for the dissemination of the warning. It is implicit that dissemination of the message and the institutional arrangement and anticipation of the public response must be tailored to forecast capability and vice-versa.

The dissemination to warn is undoubtedly the most tiring process. The credibility is extremely important for the forecasting unit. The extent to which a flood warning will be taken seriously is strongly influenced by the credibility of the officials or organisation from whom the warning originates. In case flooding does not take place as per the warning the concerned officials/organisation will face criticism and this may cause the officer to hesitate in issue of further warning in future. So standardisation of the procedure to be adopted is most important.

7.1.3 Dissemination Medium

While selecting a dissemination medium for communication between organisations along the dissemination chain, prime consideration should be given to reliability, and accuracy. The medium of warning dissemination used by flood forecasting units are the telephone, telex, messenger and own (captive) wireless network. The police also use their own information transmission media to disseminate warning from the central headquarter to local police stations and thence to the local policemen on the beat. The local policemen could warn residents by loud-speakers, knocking at the doors and by siren etc. The local radio and television network is also used to that extent within dissemination process. At some places signalling by different colours are also used to alert the people. Newspapers are also used for circulation of flood warning.

7.1.4 Flood Message and Forecast

Generally, the flood message is formulated in the morning and gets disseminated to the concerned authorities. For repeating the message to the users to maintain alertness and ensure response from those who might have missed the first message, it is also necessary to update the message, as the flood or floods progress. Messages should also incorporate in their format the time element by mentioning specific time of likely or actual occurence of flood. In this respect final message should clearly indicate as "*Danger Over*" or some such Phrase. The warning message should contain reference to local feature, which will make it easy for the general public to understand. So it should be one of the components in the message format. The colour coded maps of the areas liable to risk from events of different severity of flood should be made well known.

7.1.5 Emergency Action for Flood Fighting

The different steps for flood fighting are tabulated below:

Phase	*Action by district authority*	*Action by police*	*Action by Engg. authority*	*Action by Military*
1	*2*	*3*	*4*	*5*
Phase I				
Prep-aratory.	1. Establishment of emergency flood control room.	Prepare police emergency plan/annual emergency exercise	Arrangement for strengthening of the flood protection works.	Prepare emer-gency plan for evacuation etc. & keep close liaison with civil authority.
	2. Arrangement for pumps, boats etc.		Arrangements of sand bags.	Arrangements of conducting annual exercise.
	3. Purchase of stores, supply like poly-thene cloth, blankets etc. storage of food, medicines.		Workout diver-sion roads, liaison with district administration. Annual exercise.	
Alert advance	Convenient emer-gency control	Pass warning to concerned	Make up filled sand bags stock,	Making arran-gement for

ASSAM FLOODS DURING AUGUST, 1988

Flood water of Brahmaputra seen entered in one of the streets of Machhkhowa bus stand, Guwahati, (Assam)

BRAHMAPÚTRA FLOODS DURING AUGUST, 1988

A major portion of main road connecting D.C. Court & Bhorolimukh, Guwahati under flood waters.

Phase	*Action by district authority*	*Action by police*	*Action by Engg. authority*	*Action by Military*
1	*2*	*3*	*4*	*5*
warning that condition has arisen which may result in a flood.	room, check communication & contact members. Liaison with electricity Board and different engineering & volunteering services.	authorities, deploy extra man power in flood prone area.	patrolling of embankment & structure, monitoring rivers data.	boats & emergency plan for evacuation.
Phase II				
Situation worsening, high probability of severe flooding in specific locations.	Round the clock monitoring of flood situation, keep relief personnel in readiness, make contact with the supplier of food stuff and medicine.	Deploy extraman power in the area, traffic routing plan put in action & pass on warning to public & utility.	Patrol threatened areas, emergency sand bagging, call out contractor to standby.	Enclosed liaison with civil admn. & keep boats & other accessories in readiness.
Phase III				
Flooding occurs, seepage & localised ponding.	Relief aid to the marooned people to reach higher places. Arrangement for food, drinking water & medical facilities & medicines arrangement of shelter in school, colleges & mobilisation of transport for quick supply.	Patrolling the engineering & other establishment, helping the civil Admn in relief and rescue of high risk residences	Filling of seepage areas by Sand bags, patrolling of flood protection works, pumping of local ponding & inspection of engineers for strengthening of structures, shut down electricity in high risk areas.	Emergency activities started, and keep engineers wait for call from civil administration.

Phase	*Action by district authority*	*Action by police*	*Action by Engg. authority*	*Action by Military*
1	*2*	*3*	*4*	*5*
Phase IV				
Flooding, over topping and breaches.	Running round the clock control room & information centre, supply of relief materials military aid invoked, order additional disinfectant, arrangement of vehicle and shelter.	Set up traffic diversion and restrict movement of people. Rescue trapped residents. Identifying casualties, organise volunteer rescues, call out small boats, organise evacuation of residences& who wished to leave.	Repairing of breaches, pumping out water trapped areas, extinguish electrical fire. Arrangement for emergency road & culverts, call contractors to take up emergency works.	Rescue trapped residents & help the civil administration in relief & flood fighting operation.

7.2 Flood Forecasting & Warning System

The forecast formulation is done in the respective river basins at sub-divisional levels and at some places the same is directly done by the division office. Immediately after the formulation of the forecast, these are disseminated to the offices concerned viz. District Administration (District Relief Officer) or Sub-Divisional offices, offices of Irrigation, Flood control, waterways and other Engineering Departments and the police Department of the State Govts. etc. for taking necessary action for flood fighting as detailed in the previous Section. The State authorities, on the basis of incoming flood information, take various steps for alerting the habitants in flood plain or places where flood is anticipated. The State Govts. generally operate round the clock flood control cells in Irrigation and revenue Departments for receiving the forecast messages issued by the Flood Forecasting Organisation of Central Water Commission. Various State govts. generally designate officers of the rank of Superintending Engineers, in Irrigation Deptt. as focal officers for various river basins during monsoon period. They are responsible for keeping constant watch on the operation and effects on different engineering structures.

In addition to giving messages to the concerned Deptt. of the State Govts. messages are also disseminated to the P&T Deptt. Railways Deptt. Military Authority and important industrial establishments situated along the river bank. These messages are also communicated through All India Radio and Television centre in that zone for broadcasting in the local languages. A summarised report is also broadcast in the evening by the All India Radio centre at Head Quarters in English, Hindi and regional languages. Flood bulletins are also communicated to the news agencies and local and all India news papers for publication in their latest publications. After every week the forecast summary and the performance are communicated to the Head Quarters of Central Water Commission. In addition to these a Central Control room is operational at New Delhi for collection of flood data and forecast received from all field formations. This control room prepares daily bulletins depicting the flood levels of each site and forecast with special mention of forecasting stations which are above danger level. This daily bulletin along with the brief status report of flood situation and weather situation are sent to Member (RM), Chairman, CWC, Minister of Water Resources and the officials connected with the management of floods in the Ministry of Water Resources.

At the end of the monsoon season, a report is prepared analysing the performance of the all forecasting stations in India.

Flood Forecast Bulletin

At present generally forecast giving an advance warning of few hours to about two days is issued by the CWC flood forecasting centres. Almost all the flood forecasting centres of the flood forecast organisation in India issue daily bulletins in the morning hours indicating the present position of river at various stations and the forecasts with respect to water level and expected trend of occurrence for the forecasting stations. This is also accompanied by the descriptive bulletins indicating the anticipated flood situation in the river basin. The bulletins also include the weather condition and qualitative weather forecast in the river catchment.

Presently different formats are being used by different flood forecasting centres. It is felt necessary to standardise the proforma for use in different conditions uniformly by all flood forecasting centres. The following proformae for different situations are suggested in Annex.

Annex. VII.1. Daily Water Level and Forecast Bulletin (English)

Annex. VII.2. दैनिक जल स्तर एवं पूर्वानुमान बुलेटिन (हिन्दी)

Annex. VII.3. Daily Inflow Forecast Bulletin.

Annex. VII.4. Daily Water Level and Forecast Bulletin (Yellow Bulletin)

Annex. VII.5. Daily Water Level and Forecast Bulletin (Red Bulletin)

Annex. VII.6 Special Flood Message (Sample for rising stage)

Annex. VII.7 Special Flood Message (Sample for receding stage)

7.3 Flood Warnings

Flood warning to Public is primarily the responsibility of Civil authcrities. They have to adopt suitable media to warn the people and then to take up measures for evacuation and subsequent flood fighting operations. The flood forecasting centres are directly not issuing warning to public except those covered under the 'Flood News' broadcast of All India Radio and throse Doordarshan and News Papers. However, the various methods of communicating warning to people are discussed below in brief:

Communication of Flood Warning to the people

This is the responsibility of the State Govt. Administration. There are several methods of communicating the warning to the people such as radio, visual signals, sirens, local emergency communication etc. However, the suitability of a particular method or methods to be used for warning the people of area will depend on the following factors:-

(i) Vulnerability of the area;

(ii) Available communication facilities, and

(iii) Social awareness.

The method which have been used successfully in various countries are:-

(a) Radio

During the flood season, radio stations should issue flood bulletins in addition to their news bulletins with the frequency of these bulletins increasing as the possibility of floods increase. When an actual flood situation develops, flood warnings can be broadcast. These can indicate the areas likely to be flooded, the approximate depth of flooding and the approximate time of flooding. Instructions can be issued regarding evacuation and flood fighting operations. This is an excellent method for giving flood warning, and is inexpensive, easy and reliable.

(b) Visual signals

When electricity is available, yellow, red and green light signals may be displayed from towers erected in high spots where they can be seen from all points in the locality. If there is no electric power, the lights can be replaced by flags and lanterns. This should be raised on the top of high poles. The yellow signal would indicate likelyhood of arrival of a flood so that the people can take all necessary precautionary measures for evacuation and flood fighting and be alert for the signals. The red signal would tell people to evacuate to higher ground with their livestock and movable personal belongings, in accordance with the previously arranged plan. The green signal would indicate that flood danger is over. It is most important that the people be educated to understand and interpret the meanings of the three signals.

Such an arrangement exists for North Bengal rivers system. This is a very good arrangement for a flashy river where warning time is very less.

(c) Sirens

In addition to the visual signals, sirens can be used effectively, where electric power is available electric sirens can be used, otherwise hand operated sirens can be used. The sounding of sirens in different manners can indicate flood danger and all clear situations as is being done in many places for civil defence.

(d) Local emergency communication facilities

In densely populated area, it may be advisable to furnish emergency communication facilities such as telephone or wireless sets for dissemination of information.

7.4 Warning System—A Review

At present the forecasts issued by flood forecast organisation of CWC are being interpreted by the State authorities on the basis of the past experience of the flooding, to the extent, the flood plain will be affected by the incoming flood. The forecasts issued by different centres indicate the present and forecast level accompanied by a flood bulletin. As there is no flood risk map available with the warning authority, it creates confusion on several occasions. The area likely to be inundated and affected corresponding to various stages of flood needs to be delineated so that the warning can be made more specific. Regular testings of the flood warning system should also be exercised but in practice same are not done. However the present warning system is functioning effectively and to a great extent, it has helped in reducing flood damages in the last 3 decades.

7.5 Socio-Economic Aspects of Flood Warning and Priorities

Due to ever increasing pressure of population and due to economic considerations, encroachment of flood plain in India, the poorer people have started habitation and cultivation and these people are not so much educated to interpret the correct meaning of flood warning. So education of the people is essential in the process of warning system. To minimise the flood damage, flood plain inhabitation must be restricted and legislation be made by the respective state Govt. to this effect.

7.6 Some Suggestions

The inhabitants in the flood prone areas are to be made conscious about the destructive nature of the flood fury. For this purpose, the following steps should be taken to reduce the flood damage in the flood plain.

(i) Education of the people

This education can be accomplished by organising meetings, classes, distribution of pamphlets and placing of large size posters in prominent places. This education should be centered around the steps to be taken to safeguard own properties by moving moveable properties and cattle to safer places and to save the immovable properties. This also includes to inform people of the officials to be contacted for evacuation and relief, the places which, people should evacuate and where they should take shelter. The village Panchayat and the municipal councils can also play important role for increasing public awareness in this regard. Govt. agencies and the voluntary organisations will generally extend possible help for relief and evacuation but people should know how to live in case of disaster and to use water for drinking and keep themselves away from epidemic diseases.

(ii) Flood Plain Zoning

Flood plain zoning is one of the non-structural methods which can be used to reduce the expected average annual damage. Zoning is an action which is frequently recommended by the planner for reducing flood damages by flood insurance, emergency relief etc.

The implementation of the zoning lies with the policy makers. The reluctance of the policy makers to implement the zoning is based on their judgement and will, that the occupants of flood plain are not sufficiently convinced that the restrictions associated with zoning are worth the cost. Zoning is an alternative method strongly dependent upon the willingness of the the flood plain occupants to change their way of life. The components of flood zoning are:-

(i) Building codes

(ii) Wet Land protection

(iii) Subdivision regulation

(iv) Permanent evacuation

(v) Flood grouping

(vi) Control of utility location

(vii) Flood plain encroachment lines.

To start with, steps may be made for preparation of flood risk maps and restriction of activities in the flood risk areas.

(iii) Co-ordination of different Agencies

The warning system existing in India is not foolproof. Sometimes the flood fighting arrangement lags not for resources and man power but for proper coordination among different agencies. Hence a joint team of all the agencies connected with the flood fighting and warning system should be constituted to have the best result out of the forecasts issued by the flood forecasting organisation of CWC.

Annex. VII.1

Daily Water Level and Forecast Bulletin (English)

Flood Immediate
बाढ़ तत्काल

Gram: JALSANSADHAN भारत सरकार Phone: 27331, 26578

GOVERNMENT OF INDIA
केन्द्रीय जल आयोग
CENTRAL WATER COMMISSION

के० ज० आ० – बाढ़ पूर्वानुमान
CWC - FLOOD FORECAST

नदी का नाम Name of River	गंगा GANGA	जारी करने की तिथि Date of issue......................................
स्थान Site	हरिद्वार Haridwar	जारी करने का समय Time of issue......................................
खतरे का चिन्ह Danger Level	294.00 (M),	चेतावनी का चिन्ह Warning Level 293.00 अधिकतम बाढ़ चिन्ह Highest Flood Level 296.23 (1978)

As per Central Water Commission, the Water Level of River Ganga at Haridwar site at.........hrs. on...................was.....................metres. As per present indication/data available with CWC, it is expected that the water level will rise/fall/remain stationary and be near about............metres in the morning/forenoon/afternoon/evening/night of .... There-after, the level is likely to rise/fall/remain stationary. Further forecast will be issued on receipt of fresh data.

Executive Engineer
Himalayan Ganga Division
7-B, Sewak Ashram Road,
Dehradun - 248001

Conformation Copy by post:
No. HGD/HMT-15/

Dated......................................

To,

The Chief Engineer, CWC, Northern Region, 177-B, Srikrishna Puri, Patna.

The S.E., Upper Ganga Circle, CWC, New Delhi-66

The Director, Flood Control Co-ordination Dte., CWC, Sewa Bhawan, R.K. Puram, New Delhi.

The Executive Engineer, Middle Ganga Div.–I, CWC, Lucknow.

The Executive Engineer, Middle Ganga Div.-II, CWC, Lucknow.

The Asstt. Director, Upper Ganga Canal, Mayapur, Haridwar.

The Distt. Magistrate, Pauri/Bijnore.

The Resident Magistrate, Rishikesh/Haridwar.

Annex. VII.2

दैनिक जल स्तर एवं पूर्वानुमान बुलेटिन (हिन्दी)

बाढ़ तत्काल

भारत सरकार
केन्द्रीय जल आयोग
बाढ़ पूर्वानुमान

नदी का नाम	:	यमुना
स्थल	:	मथुरा
चेतावनी का चिन्ह	:	164.20 मीटर
खतरे का चिन्ह	:	165.20 मीटर
अधिकतम जल चिन्ह	:	169.73 (1978)

केन्द्रीय जल आयोग के आंकड़ों के अनुसार यमुना नदी में मथुरा स्थल पर आज दिनांक 3.8.86 को 0800 बजे नदी का जल स्तर 164.80 मीटर था। अब तक की प्राप्त सूचनाओं के अनुसार नदी का जल स्तर (बढ़कर)/घटकर/स्थिर रहकर दिनांक.................... के पूर्वान्ह/अपरान्ह/(सांयकाल)/रात्रि में 164.90 मीटर होने की संभावना है।

अधिशासी अभियन्ता
निचली यमुना मण्डल, आगरा

जल मौसम/आगरा

दिनांक 4/8/86

प्रतिलिपि :

Annex. VII.5

Daily Inflow Forecast Bulletin

FLOOD IMMEDIATE

Phone 228 (Office)
229 (Residence)

Gram: FORECAST, BURLA

भारत सरकार
GOVERNMENT OF INDIA

केन्द्रीय जल आयोग
CENTRAL WATER COMMISSION

INFLOW FORECAST

नदी का नाम Name of River.	महानदी Mahanadi.	जारी करने की तिथि Date of issue.
स्थान Site	हीराकुड बांध Hirakud Dam.	जारी करने का समय Time of issue.

(Full Reservoir level)

F.R.L. 192.00 m.

अ० जल तल
(Maximum water level)
M.W.L. 192.21 (1961)

As per Central Water Commission, the Reservoir level of Hirakud Dam on River Mahanadi at 0800 hrs. on 26.8.86 (date) was 186.87 metres.

As per present indications/data available with CWC ...... Cubic metres of water is likely to flow into the reservoir between 0800 hrs. of 26.8.86 to 0800 hrs. of 27.8.86.

Inflow is likely to increase/decrease/remain steady thereafter.

Executive Engineer
Mahanadi Division.
SD-7/1 Burla (Orissa)

Date: 26.8.86.

Annex. VII.4

Daily Water Level and Forecast Bulletin (Yellow Bulletin)

Flood Immediate
बाढ़ तत्काल
Phone: 27331, 26578

Gram: JALSANSADHAN

भारत सरकार
Government of India
केन्द्रीय जल आयोग
Central Water Commission

के० ज० आ० – बाढ़ पूर्वानुमान
CWC - FLOOD FORECAST

नदी का नाम Name of River	गंगा GANGA	जारी करने की तिथि Date of issue....................................
स्थान Site	हरिद्वार Haridwar	जारी करने का समय Time of issue....................................
खतरे का चिह्न Danger Level	294.00 (M),	चेतावनी का चिह्न Warning Level 293.00 अधिकतम बाढ़ चिह्न Highest Flood Level 296.33 (1978)

As per Central Water Commission, the water level of River Ganga at Haridwar site at...............hrs. on.......................was.................... metres and is within 0.5 metre of highest flood level. As per present indications/data available with CWC, it is expected that water level will rise/fall/remain stationary and be near about...................... metres in the morning/ forenoon/afternoon/evening/night of................

Thereafter, the level is likely to rise/fall/remain stationary

Executive Engineer
Himalayan Ganga Division
7-B, Sewak Ashram Road,
Dehradun-248001.

Confirmation copy by post:

No. HGD/HMT-15/

Dated. ....

To

The Chief Engineer, CWC, Northern Region, 177-B, Srikrishna Puri, Patna.

The Supdtg. Engineer, Upper Ganga Circle, CWC, New Delhi-66.

The Director, Flood Control Coordination Dte, CWC, Sewa Bhawan, R.K. Puram, New Delhi.

The Executive Engineer, Middle Ganga Divn. I, CWC, Lucknow.

The Executive Engineer, Middle Ganga Division II, CWC, Lucknow.

The Chief Engineer, Ganga Valley Hydro Electric Projects, Yamuna colony, Dehradun.

The Executive Engineer, Madhya Ganga Canal, construction Divn. 5, Bijnore.

The Asstt. Director, Upper Ganga Canal, Mayapur, Haridwar.

Annex. VII.6

Daily Water Level and Forecast Bulletin (Red Bulletin)

Executive Engineer
Himalayan Ganga Division
7-B, Sewak Ashram Road,
Dehradun-248001.

Flood Immediate
बाढ़ तत्काल
Phone: 27331, 26578

Gram: JALSANSADHAN

भारत सरकार
Government of India
केन्द्रीय जल आयोग
Central Water Commission

के0 ज0 आ0 – बाढ़ पूर्वानुमान
CWC - FLOOD FORECAST

नदी का नाम Name of River	गंगा GANGA	जारी करने की तिथि Date of issue....................................
स्थान Site	हरिद्वार Haridwar	जारी करने का समयं Time of issue....................................
खतरे का चिन्ह Danger Level	294.00 (M)	चेतावनी का चिन्ह Warning Level 293.00 अधिकतम बाढ़ चिन्ह Highest Flood level 296.33 (1978)

As per Central Water Commission, the water level of River Ganga at Haridwar site at...............hrs. on.......................was.................... metres and has crossed the previous highest flood level. As per present indications/data available with CWC, it is expected that water level will rise/fall/remain stationary and be near about....................... metres in the morning/forenoon/afternoon/evening/night of................

Thereafter, the level is likely to rise/fall/remain stationary

Executive Engineer
Himalayan Ganga Division
7-B, Sewak Ashram Road,
Dehradun-248001.

Confirmation copy by post:

No. HGD/HMT-15/ Dated,

To

The Chief Engineer, CWC, Northern Region. 177-B, Srikrishna Puri, Patna.

The Supdtg. Engineer, Upper Ganga Circle, CWC, New Delhi-66.

The Director, Flood Control Coordination Dte, CWC, Sewa Bhawan, R.K. Puram, New Delhi.

The Executive Engineer, Middle Ganga Divn. I, CWC, Lucknow.

The Executive Engineer, Middle Ganga Division II, CWC, Lucknow.

The Chief Engineer, Ganga Valley Hydro Electric Projects, Yamuna colony, Dehradun.

The Executive Engineer, Madhya Ganga Canal, construction Divn. 5, Bijnore.

The Asstt. Director, Upper Ganga Canal, Mayapur, Haridwar.

Executive Engineer
Himalayan Ganga Division
7-B, Sewak Ashram Road,
Dehradun-248001.

Annex. VII.6

Special Flood Message
(Sample for rising stage)

15th August, 1987.

SPECIAL FLOOD MESSAGE NO. 3

AT A GLANCE

Several rivers of North Bihar continue to reel under high floods. As had been correctly forecast earlier by Central Water Commission Control rooms in Bihar, the water level of Burhi Gandak, Bagmati, Adhwara group, Kamla Balan and Kosi at several sites after having crossed their previously recorded Highest Flood levels continue to be in high floods. River Brahmaputra which had crossed the previous HFL at Neamatighat has started receding, though the river continues to be under floods.

Burhi Gandak As per the forecast made by the control room of Central Water Commission at Patna, the water level of this river at Sikandarpur (Muzafarpur) had crossd the previously recorded HFL (53.93m in 1975) and has reached 54.29 m this morning. It is expected to fall and reach around 54.20 m by the morning of 16.8.87.

At Samastipur, the water level this morning was 49.32 m after crossing the previous HFL and it is expected to rise further and reach around 49.40 m by the early hours of 16.8.87.

At Rosera, the water level reached 46.23 m after crossing HFL and is expected to rise further and reach 46.35 m by the morning on 16.8.87 which will be about 35 cm above the past HFL.

Bagmati As was forecast by CWC control room, the water level of river Bagmati at Benibad crossed the previous HFL (49.56 m in 1979) and was at 49.65 m this morning. The water level has started receding and is expected to fall by 25 cm by 16.8.87 morning.

At Hayaghat also the previously recorded HFL (48.06 m 1975) was exceeded and water level stood at 48.82 m this morning. The water level is expected to fall considerably by 40 cm further till 16.8.87.

Adhwara Group The water level of Adhwara at Ekmighat (Darbhanga) also exceeded the previously recorded HFL (48.57 m during 1975) by 69 cm and was 49.26 m this morning. The CWC base station for the forecast which is at Kamtaul had become in-operational due to flooding and continues to be in-operational.

Kamla Balan The water level of this river at Jhanjharpur which crossed the previously recorded HFL on 11.8.87 was at 52.09 m this morning as was forecast by CWC. The river is expected to recede and fall below previous HFL by 16.8.87.

Kosi The water level of river Kosi at Baltara has risen as per the forecast of CWC and this morning it was at 36.37 m which is 1 meter above the previously recorded HFL of 35.37 m in 1974. The water level at this site is expected to fall by tomorrow morning. A breach in the right embankment in the Badlaghat-Sagarpara reach as reported earlier is likely to affect rail and road communication.

Brahmaputra As was forecast by CWC, the water level of river Brahmaputra at Neamatighat which was above the previously recorded HFL on 13.8.87 has started falling. The water level at this site is much below the 1977 HFL. The water level at this site is expected to fall further by 16.8.87 morning.

A number of wireless stations of flood forecasting offices of CWC in North Bihar are submerged in the floods. However, they are managing to function and continue to send base data to their respective control rooms.

The CWC offices are continuously giving forecasts to the local authorities to enable them to take appropriate measures.

Annex. VII.7

Special Flood Message
(Sample for receding Stage)

Special Flood Message no. 21 Dt. 23.9.1987

At a Glance

River Ganga has receded all the way from Ballia down stream. Only at Ballia, Colgong and Farakka it is above danger level but receding. Similarly water levels of all tributaries of Ganga are receding.

Some Major Details

Ganga at Farakka has receded to a level of 24.02 mts. today morning which is 50 cm below the previous HFL of 24.52 mts. (1948). It will recede by about 30 cm by tomorrow night. At Colgong, the river which was flowing 28 cm above the danger level of 31.09 mts., will recede about 22 cm by tomorrow night. All other sites on Ganga are generally below danger levels and are falling. Only Sripalpur on river Punpun, Jhanjharpur on river Kamla Balan and Baltara on river Kosi are expected to rise by tomorrow morning.

River Brahmaputra and its tributaries are flowing below danger levels except for Numaligarh on river Dhansiri.

CHAPTER-VIII

CONCLUSIONS

8. Conclusions:

Recognising the great importance of non-structural measures like flood forecasting and warning in mitigating the loss of lives and property due to floods, Central Water Commission have set up a network of flood forecasting and warning stations in different interstate rivers of India. This set up functions under the overall direction of Member (River Management). By 1987 the CWC flood forecasting organisation had a network of 147 flood forecasting stations spread over the country (Plate-I). The operation and maintenance of existing flood forecasting network is done according to the Budget allotment done each year under Non-Plan head and is thus subject to restrictions and cuts applied to items under "Non-Plan". Expansion of the network with a view to cover additional flood prone areas and strengthening/improvement of the existing system come under 'Plan' head. Work on such plan schemes are subject to Plan allocation for such schemes and approval of specific schemes by the Govt. as also budget allocation of funds. Even though we receive a large number of requests from the State Govt., we have the limit of availability of funds. In terms of relief assistance the budget provision is only 2.83%. Our performance has also improved to a large extent as far as accuracy is concerned.

The demand for river forecasting viz; forecasts throughout the year has increased with a view to evolve measures for optimal utilisation of Water Resources.

Conventional Methods based on correlation diagrams with or without parameter have been described in detail. Selection of a particular method depends upon the data availability and peculiarity of site. Several mathematical models have been applied for Delhi Railway Bridge forecast but each one of them has its own peculiarity. It cannot be said which model is the best. In fact a new model called CWCFF1 has been developed which might be the best between the reach Kalanaur to Delhi. All the models are awaiting testing in a real flood situation both low flows and high flows.

The Data transmission, back bone of flood forecastings activities, has been dealt in third chapter. Central Water Commission has always tried to adopt the modern technology available in the field of communication engineering for the improvement of fidelity and speed of communication, a primary requirement of flood forecast. Apart from the conventional mode of communication, i.e. teleprinter/telegraph/telex etc. being used, CWC is having its own independent wireless network spread almost throughout the country. An attempt has been made to make the theoritical and practical aspects of such wireless system comprehensive. The need for computer compatibility and the optimum speed of data transmission with accuracy from the remote base stations for the improvement of flood warning time has paved the way for induction of telemetry through surface and satellite communication. Under Phase-I/Phase-II of a pilot project the installation of such telemetry system has been taken up in the Yamuna catchment for the flood forecast at Delhi Railway Bridge. The configuration of such telemetry has also been dealt to provide the basic idea of the system as its success would definitely open a new horizon in the field of flood forecast.

The present evaluation technique needs modification depending upon the nature of river and peculiarity of site. An error of about ± 50 cm. might be adopted for a river site which is flashy in nature while it be reduced to ± 10 cm. for a river like Ganga in plain area. Similar modification is needed for inflow forecasting site.

After the forecast is disseminated by the Central Water Commission office for a particular site, the responsibility of its utilisation rests with the State Engineering authorities and other administration machinery. Central Water Commission has the responsibility to convey at a time sufficient in advance so that full advance warning time is available with the State Authorities.

APPENDIX II (1)

Salient Features of Hydrological Observation and Flood Forecasting Activities in Central Water Commission

1.	Establishment of 'First Scientific Flood Forecasting Unit' (F.F.U. Delhi)	November 1958
2.	Date of issue of 'first flood forecast'	25th July, 1959
3.	Name of first forecasting site/river	Delhi Railway Bridge/Yamuna
4.	Year of commencement of flood forecasting system on the Inter-state rivers.	1969
5.	No. of Chief Engineers' offices.	3
6.	No. of Superintending Engineer's Offices including one River Management Co-ordination Directorate	11
7.	No. of present Flood Forecasting Divisions, excluding Snow Hydrology Division and also other divisions which are engaged in Hydrological observations only.	22
8.	No. of Control Rooms/Sub-Divisions engaged in flood forecasting work under above divisions.	64
9.	No. of Inter-State rivers (Main/Tributaries) covered by flood forecasting programme.	59
10.	No. of States including Union Territories covered under F.F. Programme.	12

11.	No. of forecasting Sites.	147
12.	No. of gauge/gauge and discharge sites	700 (Approximately)
13.	No. of rain gauge stations (ordinary/Self recording)	500
14.	No. of wireless stations.	402
15.	Maximum No. of forecasts issued in any one year.	**7385 (in 1978)**
16.	Seventh Five Year Plan outlay.	Rs 1000 lakhs

APPENDIX II (2)

Flood Forecasting Sites Under Central Water Commission

Sl. No.	*Name of river and forecasting sites*	*Warning level (Mts.)*	*Danger level (Mts.)*	*Highest Flood Level recorded so far during previous years upto 1988.*	
				(Mts.)	*Year.*
1	*2*	*3*	*4*	*5*	*6*
I.	*Ganga*				
1.	Haridwar	293.00	294.00	296.23	1978
2.	Ankinghat	123.00	124.00	124.31	1978
3.	Kanpur	113.00	114.00	113.47	1967
4.	Dalmau	98.36	99.36	99.84	1973
5.	Phaphamau	83.73	84.73	87.98	1978
6.	Chhatnag (Allahabad)	83.73	84.73	88.39	1948
7.	Mirzapur	76.72	77.72	80.34	1978
8.	Varanasi	70.26	71.26	73.90	1978
9.	Ghazipur	62.11	63.11	65.22	1978
10.	Buxer	59.32	60.32	62.09	1948
11.	Ballia	56.62	57.62	59.92	1982

Sl. No.	*Name of river and forecasting sites*	*Warning level (Mts.)*	*Danger level (Mts.)*	*Highest Flood Level recorded so far during previous years upto 1988.*	
				(Mts.)	*Year.*
1	*2*	*3*	*4*	*5*	*6*
12.	Dighaghat (Patna)	49.45	50.45	52.52	1975
13.	Gandhighat (Patna)	47.60	48.60	50.12	1987
14.	Hathidah	40.76	41.76	43.15	1971
15.	Munger	38.33	39.33	40.99	1976
16.	Bhagalpur	32.68	33.68	34.18	1978
17.	Colgong	30.09	31.09	32.50	1987
18.	Farakka	21.25	22.25	24.85	1987
II.	*Yamuna*				
19.	Delhi Rly. Bridge	204.00	204.83	207.49	1978
20.	Mathura	164.20	165.20	169.73	1978
21.	Agra	151.40	152.40	154.76	1978
22.	Etawah	120.92	121.92	126.13	1978
23.	Auraiya	112.00	113.00	116.86	1982
24.	Kalpi	107.00	108.00	112.49	1982
25.	Hamirpur	102.63	103.63	108.59	1983
26.	Chillaghat	99.00	100.00	105.16	1978
27.	Naini	83.73	84.73	87.99	1978

Sl. No.	Name of river and forecast-ing sites	Warning level (Mts.)	Danger level (Mts.)	Highest Flood Level recorded so far during previous years upto 1988. (Mts.)	Year.
1	2	3	4	5	6
III.	*Sahibi*				
28.	Dhansa	211.44	212.45	213.58	1977
IV.	*Betwa*				
29.	Mohana	121.66	122.66	133.35	1983
30.	Shahjina	103.55	104.55	108.67	1983
V.	*Ken*				
31.	Banda	103.00	104.00	110.85	1978
VI.	*Gomti*				
32.	(Hanuman Setu) Lucknow	108.50	109.50	110.85	1971
33.	Jaunpur	73.07	74.07	77.74	1971
VII.	*Sai*				
34.	Rae Bareilly	100.00	101.00	104.81	1982
VIII.	*Ghaghra*				
35.	Elgin Bridge	105.07	106.07	107.18	1983
36.	Ayodhya	91.73	92.73	93.64	1988
37.	Turtipar	63.01	64.01	65.09	1983
38.	Darauli	59.82	60.82	61.66	1948
39.	Gangapur-Siswan	56.04	57.04	58.01	1983

Sl. No.	Name of river and forecasting sites	Warning level (Mts.)	Danger level (Mts.)	Highest Flood Level recorded so far during previous years upto 1988. (Mts.)	Year.
1	2	3	4	5	6
40.	**Chhapra**	52.68	53.68	54.59	1982
IX.	***Rapti***				
41.	**Balrampur**	103.62	104.62	105.15	1981
42.	**Bansi**	83.90	84.90	85.57	1981
43.	**Birdghat (Gorakhpur)**	73.98	74.98	76.84	1974
X.	*Sone*				
44.	Inderpuri	107.20	108.20	108.85	1975
45.	Koelwar	54.52	55.52	58.88	1971
46.	Maner	51.00	52.00	53.79	1976
XI.	*Punpun*				
47.	Sripalpur	49.60	50.60	53.91	1976
XII.	*Gandak*				
48.	Khadda	95.00	96.00	96.85	1978
49.	Chatia	68.15	69.15	69.75	1954
50.	Rewaghat	53.41	54.41	55.41	1986
51.	Hazipur	49.32	50.32	50.93	1948
XIII.	*Burhi Gandak*				
52.	Lalbeghiaghat	62.20	63.20	67.09	1975

Sl. No.	*Name of river and forecast-ing sites*	*Warning level (Mts.)*	*Danger level (Mts.)*	*Highest Flood Level recorded so far during previous years upto 1988.* (Mts.)	Year.
1	2	3	4	5	6
53.	Sikandarpur (Muzaffarpur)	51.53	52.53	54.29	1987
54.	Samastipur	45.02	46.02	49.38	1987
55.	Rosera	41.63	42.63	46.35	1987
56.	Khagaria	35.58	36.58	39.22	1976
XIV.	*Bagmati*				
57.	Benibad	47.68	48.68	49.66	1987
58.	Hayaghat	44.72	45.72	48.96	1987
XV.	*Adhwara Group*				
59.	Kamtaul	49.00	50.00	52.99	1987
60.	Ekmighat	45.94	46.94	49.27	1987
XVI.	*Kamla Balan*				
61.	Jhanjharpur	49.00	50.00	52.73	1987
XVII.	*Kosi*				
62.	Basua	46.75	47.75	48.65	1987
63.	Baltara	32.85	33.85	36.40	1987
64.	Kursela	29.00	30.00	31.85	1978
XVIII.	*Mahananda*				
65.	Dhengraghat	34.65	35.65	38.09	1968

Sl. No.	*Name of river and forecasting sites*	*Warning level (Mts.)*	*Danger level (Mts.)*	*Highest Flood Level recorded so far during previous years upto 1988.*	
				(Mts.)	*Year.*
1	*2*	*3*	*4*	*5*	*6*
66.	Jhawa	30.40	31.40	33.51	1987
XIX.	*Mayurakshi*				
67.*	Messanjore Dam	–	121.31	121.60	1959
68.*	Tilpara Barrage(P.L.)	–	62.79	67.05	1978
69.	Narayanpur	26.99	27.99	29.16	1978
XX.	*Ajoy*				
70.	Gheropara	38.42	39.42	43.94	1978
XXI.	*Damodar*				
71.*	Tenughat Dam	–	268.83	265.56	1985
72.*	Panchet Dam	–	132.58	132.89	1959
73.*	Durgapur Barrage(P.L.)	–	64.47	64.47	Several times
XXII.	*Barakar*				
74.*	Maithon Dam	–	150.88	151.79	1959
XXIII	*Mundeshwari*				
75.	Harinkhola	11.80	12.80	14.58	1978
XXIV.	*Kangsabati*				
76.	Kangsabati Dam	–	134.11	134.71	1978
77.	Mohanpur	24.73	25.73	29.87	1978

Sl. No.	Name of river and forecasting sites	Warning level (Mts.)	Danger level (Mts.)	Highest Flood Level recorded so far during previous years upto 1988. (Mts.)	Year.
1	2	3	4	5	6
XXV.	*Brahmaputra*				
78.	Diburugarh	103.24	104.24	106.32	1988
79.	Neamatighat	84.04	85.04	87.28	1988
80.	Tezpur	64.23	65.23	66.59	1988
81.	Guwahati	48.68	49.68	51.37	1988
82.	Goalpara	35.27	36.27	37.29	1988
83.	Dhubri	27.50	28.50	30.36	1988
XXVI.	*Burhi Dihing*				
84.	Naharkatia	119.40	120.40	122.68	1988
85.	Khowang	101.11	102.11	103.92	1988
XXVII.	*Subansiri*				
86.	Badatighat	81.53	82.53	86.84	1972
XXVIII.	*Dhansiri*				
87.	Golaghat	88.50	89.50	91.30	1986
88.	Numaligarh	76.42	77.42	79.87	1985
XXIX.	*Kopili*				
89.	Kampur	59.50	60.50	61.86	1973
90.	Dharamtul	55.00	56.00	57.68	1988

Sl. No.	Name of river and forecasting sites	Warning level (Mts.)	Danger level (Mts.)	Highest Flood Level recorded so far during previous years upto 1988. (Mts.)	Year.
1	2	3	4	5	6
XXX.	*Puthimari*				
91.	N.H.Crossing	50.81	51.81	54.81	1988
XXXI.	*Beki*				
92.	Road Bridge	43.38	44.38	46.08	1988
XXXII.	*Pagladiya*				
93.	N.T. Rd. Crossing	51.75	52.75	55.38	1984
XXXIII.	*Manas*				
94.	N.H. Crossing	46.95	47.56	50.08	1984
XXXIV	*Sankosh*				
95.	Golakganj	28.51	29.51	32.00	1968
XXXV.	*Raidak*				
96.	Tufanganj	34.22	35.30	36.06	1987
XXXVI.	*Torsa*				
97.	Ghughumari	39.80	40.41	41.40	1984
XXXVII.	*Jaldhaka*				
98.	N.H. 31 Rd. Bridge	80.00	80.90	81.33	1972
99.	Mathabhanga	48.20	48.70	49.60	1972

Sl. No.	Name of river and forecast-ing sites	Warning level (Mts.)	Danger level (Mts.)	Highest Flood Level recorded so far during previous years upto 1988. (Mts.)	Year.
1	2	3	4	5	6
XXXVIII.	*Tista*				
100.	Domohini Rd. Bridge Road Bridge	85.30	85.80	89.30	1968
101.	Mekhliganj	65.45	65.95	65.62	1986
VXLIV.	*Barak*				
102.	Annapurnaghat (Silchar)	18.83	19.83	21.77	1986
XL.	*Katakhal*				
103.	Matizuri	19.27	20.27	22.32	1988
XLI.	*Subernarekha*				
104.	Rajghat	9.45	10.36	12.13	1978
XLII.	*Burhabalang*				
105.	N.H.5 Bridge.	7.21	8.13	9.50	1973
XLIII.	Baitarni				
106.	Anandpur	37.44	38.36	41.20	1975
107.	Akhuapada	18.29	19.20	21.95	1960
XLIV.	*Brahmani*				
108.	Jenapur Exp. Way	22.00	23.00	24.78	1975
XLV.	*Mahanadi*				
109.*	Hirakud Dam	–	192.02	192.17	1965

Sl. No.	Name of river and forecasting sites	Warning level (Mts.)	Danger level (Mts.)	Highest Flood Level recorded so far during previous years upto 1988. (Mts.)	Year.
1	2	3	4	5	6
110.	Alipingal Devi	10.85	11.76	12.86	1982
111.	Nimapara	9.85	10.76	11.60	1982
XLVII.	*Rushikulya*				
112.	Purushottampur	15.15	16.15	16.78	1988
XLVII.	*Vamsadhara*				
113.	Gunupur	81.90	82.90	87.67	1980
114.	Kashinagar	53.60	54.60	57.93	1980
115.*	Gotta Barrage (P.L.)	–	34.84	39.87	1980
XLVIII.	*Godavari*				
116.	Kopergaon	482.22	485.00	490.49	1969
117*.	Jaikwadi Dam	–	463.91	463.60	1983
118.	Gangakhed	374.00	375.00	377.57	1947
119.	Nanded	353.00	354.00	355.65	1983
120*.	Pochampad (Sriram Sagar)	–	332.54	332.63	1988
121.	Dummagudam(P.S)	53.00	55.00	60.25	1986
122.	Bhadrachalam	45.72	48.77	55.66	1986
123.	Kunavaram	39.11	40.32	51.30	1986
124.	Rajamundry Rly. Bridge	17.68	19.51	20.48	1986

Sl. No.	Name of river and forecasting sites	Warning level (Mts.)	Danger level (Mts.)	Highest Flood Level recorded so far during previous years upto 1988. (Mts.)	Year.
1	2	3	4	5	6
125.	Dowlaiswaram	14.25	16.08	18.36	1986
XLIX.	*Wainganga*				
126.	Bhandara	244.00	244.50	246.82	1986
127.	Pauni	N.F.	232.50	235.80	1979
L.	*Indravati*				
128.	Jagdalpur	539.50	540.80	544.68	1973
LI.	*Krishna*				
129.*	Alamati Dam		500.00	494.69	1979
130.*	Narayanpur Dam		492.25	490.50	1987
131.*	Srisailam Dam	–	269.75	269.87	1984
132.*	Vijayawada(P.L) (Prakasam Barrage)		18.30	21.30	1903
LII.	*Bhima*				
133.	Deongaon	404.46	407.00	405.26	1988
LIII.	*Tungabhadra*				
134*.	T.B.Dam	–	497.74	497.74	1984
LIV.	*Banas*				
135*.	Dantiwada Dam	–	184.10	186.04	1973
LV.	*Sabarmati*				
136.*	Dharoi Dam	–	189.58	189.40	1984

Sl. No.	Name of river and forecasting sites	Warning level (Mts.)	Danger level (Mts.)	Highest Flood Level recorded so far during previous years upto 1988. (Mts.)	Year.
1	2	3	4	5	6
137.	Subhas Bridge (Ahmedabad)	43.51	45.34	44.85	1988
LVI.	*Mahi*				
138*.	Kadana Dam	–	127.71	127.43	1983
139.	Wanakbori	69.49	71.93	73.00	1981
LVII.	*Narmada*				
140.	Hoshangabad	292.83	293.83	300.90	1973
141.	Garudeshwar	30.48	31.10	41.65	1970
142.	Bharuch	6.71	7.31	12.65	1970
LVIII.	*Tapi*				
143*.	Hatnur Dam	–	214.00	213.65	1987
144*.	Ukai Dam	–	105.16	105.23	1981
145.	Surat	29.18	30.18	31.01	1968
LIX.	*Damanganga*				
146.*	Madhuban Dam	–	79.86	71.10	1983
147.	Daman	2.60	3.40	2.41	1981

1. *Inflow forecasting site. Full Reservoir level is listed in column 4 (four)
2. P.L. : Pond Level
3. N.F. : Not fixed

APPENDIX IV (1)

EXAMPLE ON ESTIMATION OF AREAL RAINFALL FROM POINT RAINFALL

Example

The catchment of River Baitarani at Anandpur is given in Fig. IV(1). 1. The figure also shows the location of the several raingauge stations in and around the basin. The rainfall at the various stations on 20.7.73 was recorded and is given below:

Station	*Rainfall in mm.*
Anandpur	210.0
Ghatgaon	98.2
Thakurmunda	114.3
Swampatna	92.0
Baripada	68.4
Champua	32.0
Keonjhar	77.0

Obtain the average rainfall over the basin by means of the following methods:

(1) Arithmetic Mean Method.

(2) Thiessen Polygon Method.

(3) Isohyetal Method.

Solution

(1) Arithmetic Mean Method

Out of the seven rainfall stations listed above, two are outside the basin. Therefore compute the sum of the rainfall observed at the five stations in the catchment i.e. 210.0 + 98.2 + 92.0 + 32.0 + 77.0 = 509.2 mm.

Therefore, the average rainfall over the catchment $= \frac{509}{5} = 101.8$ mm

(2) Thiessen Polygon Method

1. The various points indicating the raingauge stations were joined with the help of dotted lines as shown in Fig. IV(1).2.
2. The perpendicular bisector of each line is drawn and thus Thiessen Polygon is formed.
3. The area of each polygon is computed and the Thiessen weights are calculated as shown in the following table.

Totoal area of the basin A = 8570 sq. km.

Rainfall Station	*Area in Km² of Thiessen Polygon around the rainfall station.* *(Ax)*	*Thiessen weights* $W_x = \frac{A_x}{A}$	*Rainfall x Thiessen weight.*
Anandpur	270	0.031	6.51
Ghatgaon	910	0.106	10.41
Thakurmunda	710	0.083	9.49
Swampatna	1950	0.228	20.98
Baripada	510	0.059	4.02
Champua	2090	0.244	7.81
Keonjhar	2130	0.249	19.17
Total		1.000	78.39
		Say	78.4 mm.

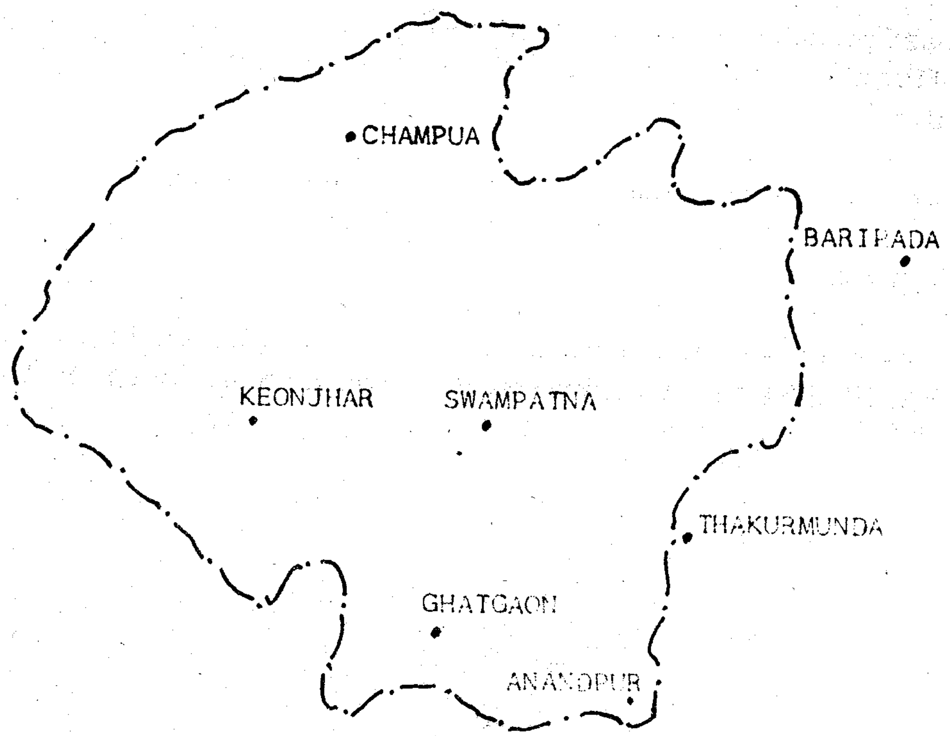


FIG.IV (1).1

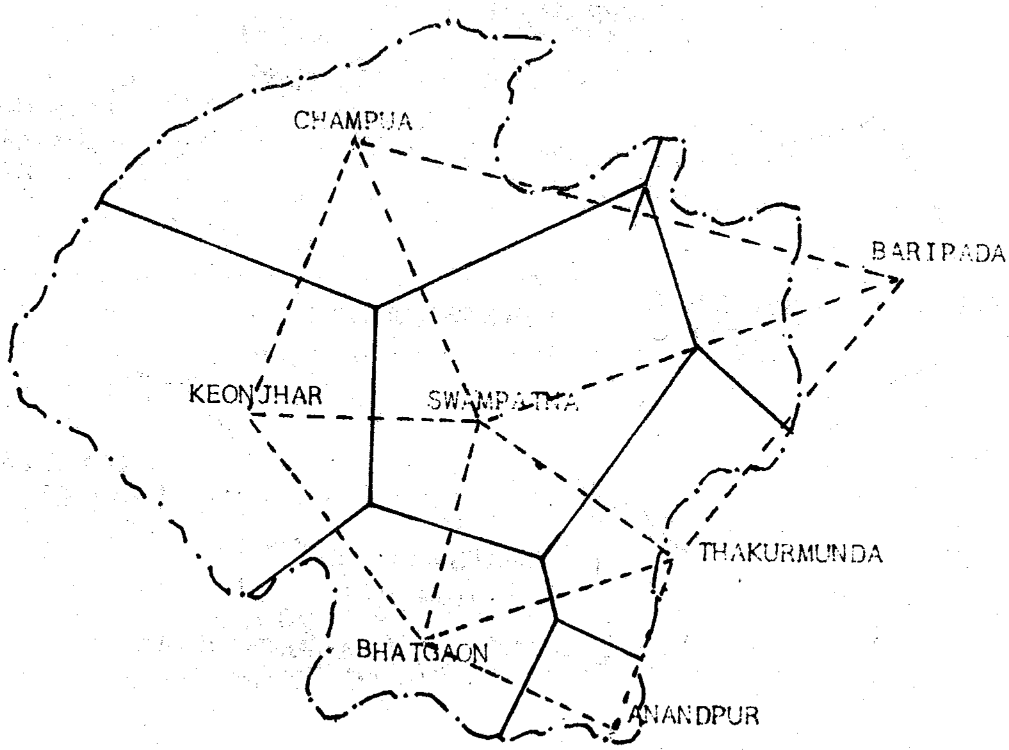


FIG.IV (1).2

The rainfall at each station is multiplied by the Thiessen weight of the station and written under Col. 4 of the above table. The sum of all the weighted rainfall will give the average rainfall over the catchment.

Therefore, the average rainfall over the catchment = 78.4 mm.

3) Isohyetal Method

The values of the point rainfalls are plotted at the respective points on the basin map as shown in the Fig. IV(1).3 and the isohyets are drawn for rainfall of 200 mm, 150 mm, 100 mm and 50 mm.

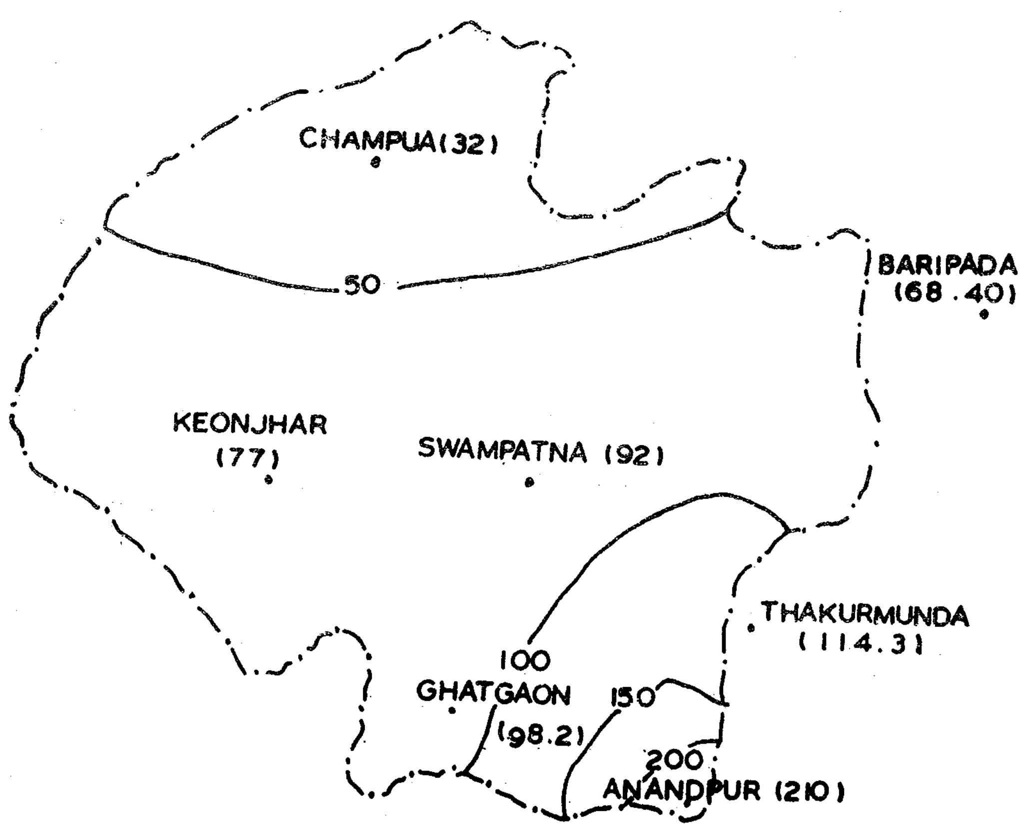


FIG.IV.(1).3

The area between the two isohyets is measured. The average rainfall is estimated as follows:

Isohyets limits.	*Area enclosed (Km²)*	$\frac{A_x}{A}$	*Average precipi-tation*	*Weighted precipi-tation (Col. 3 × Col. 4)*
1	*2*	*3*	*4*	*5*
>200 mm	25	0.003	205	0.62
150-200	240	0.028	175	4.90
100-150	780	0.091	125	11.38
50-100	5545	0.647	75	48.52
>50 mm	1980	0.231	35	8.89
Total	8570	1.000		73.51

Therefore, the average rainfall over the basin as per isohyetal method = 73.5 mm.

The average rainfall obtained by different methods are tabulated below:

Methods	*Average rainfall*
Arithmetic mean method	101.8 mm
Thiessen Polygon method	78.4 mm
Isohyetal method	73.5 mm

APPENDIX IV (2)

EXAMPLE ON DERIVATION OF UNIT HYDROGRAPH FROM AN ISOLATED FLOOD HYDROGRAPH

Example

In order to develop the unit hydrograph for river Baitarani at Anandpur site, the several plotted flood hydrographs were examined. An isolated flood hydrograph was found during the period 13.8.76 to 15.8.76. The flood hydrograph is shown in Fig. IV (2).1. A study of the rainfalls recorded at the various stations in the basin indicated that the rainfalls were almost same at the various stations. The rainfall records also indicated that the effective rainfall duration may be taken as 6 hrs. As such, this particular flood hydrograph was selected for developing a 6 hrs. duration unit hydrograph.

The various steps involved in the development of the unit hydrograph are discussed below:

1. The observed gauge data for the storm period (including few data before the rise of gauge and a few data after the end of recession limb) were converted to discharge data with the help of gauge discharge relations.

2. The discharge hydrograph was plotted as shown in Fig. IV(2).1.

3. The base flow separation was done in the following manner:

   (i) The anticipated recession curve of the smaller peak just before this flood was continued till the time of the peak i.e. upto point 'B'.

   (ii) A suitable point 'C' was chosen on the falling limb at a distance of about 2 t to 2.5 t where t is the time from rise of flood hydrograph to the peak.

   (iii) The points B and C were joined by a straight line. The curve ABC separated the base flow from the total flood.

4. The base flow thus separated was deducted from the corresponding ordinates of the flood hydrograph to get a direct runoff hydrograph as illustrated in the following table:

5. The various ordinates of the DRH (Direct Runoff Hydrograph) were added together and Σ O found to be 14510 cumecs.

6. The total direct runoff in mm was calculated by using the relation:

$$R = \frac{\Sigma O \times t \times 3.6}{A}$$

where R = Direct runoff in mm

ΣO = Summation of the ordinates of DRH.

A = Area of the catchment in sq. km.

t = time interval of the ordinates.

In this particular case, ordinates were taken at three hourly interval.

ΣO = 14510 cumecs

A = Catchment area of Baitarani upto Anandpur

= 8570 sq. km.

t = time interval = 3 hrs.

$$R = \frac{14510 \times 3 \times 3.6}{8570}$$

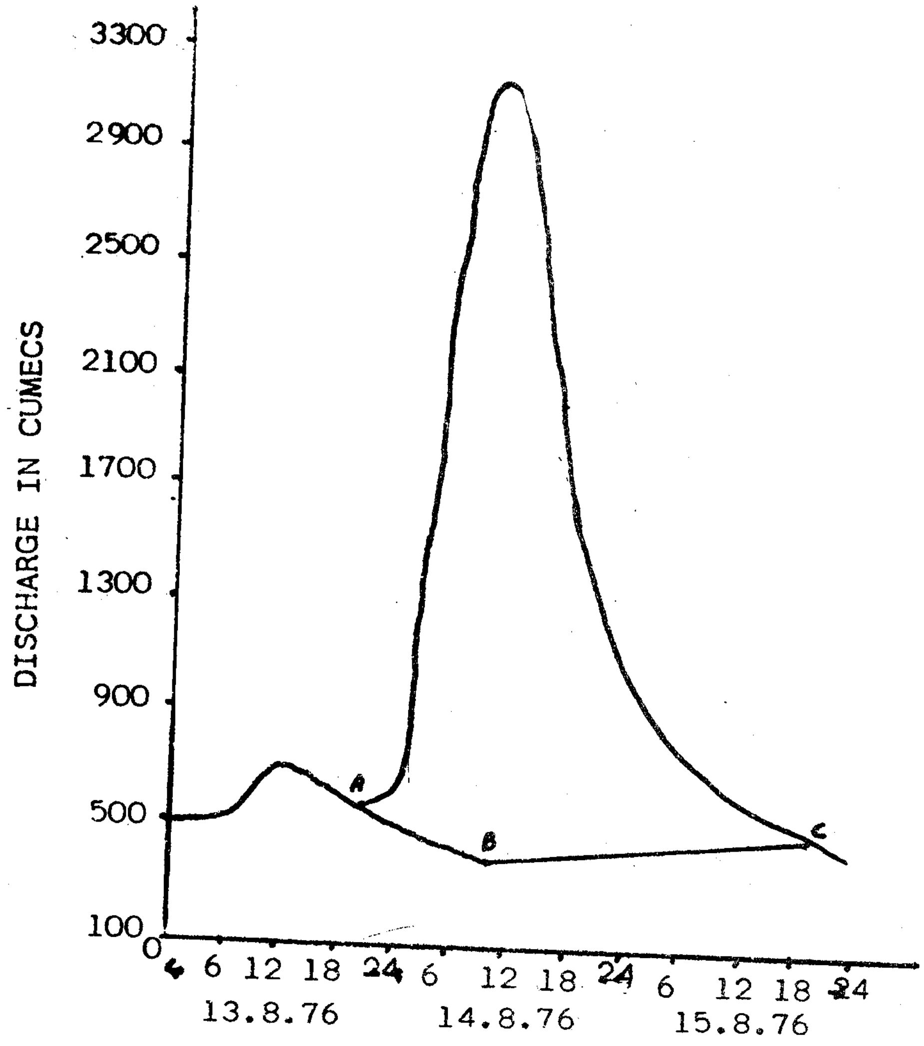
= 18.29 mm.

7. All the ordinates were divided by the total direct runoff i.e. 18.29 mm. The values are given in Col. 7 of the following table.

The column 7 of the table gives the ordinates of the six hour duration unit hydrograph at Anandpur site for river Baitarani. The unit hydrograph is shown in the Fig. IV(2).2.

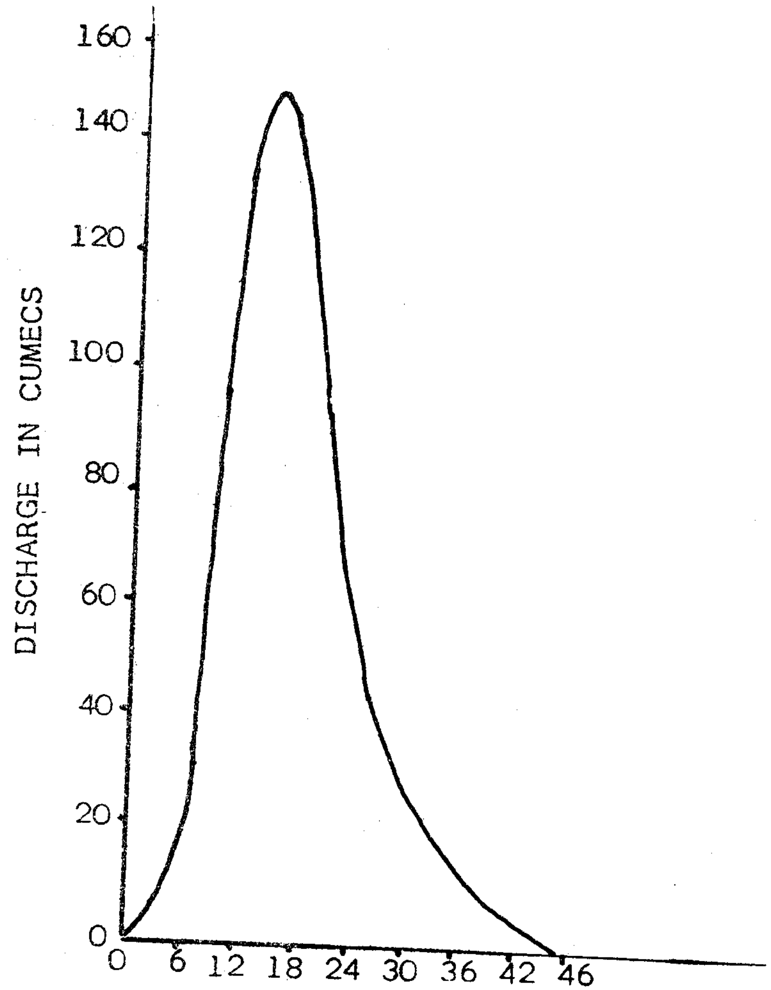
Date	*Time*	*Reduced Time (Hrs.)*	*Ordinates of Flood Hydrograph (Cumecs)*	*Baseflow (Cumecs)*	*Ordinates of Direct Runoff Hydrograph (Cumecs)*	*Ordinates of the 6 Hrs. Duration Unit Hydrograph (Cumecs)*
1	*2*	*3*	*4*	*5*	*6*	*7*
13.8.76	1900	0	610	610	0	0
	2200	3	620	545	75	4

Date	Time	Reduced Time (Hrs.)	Ordinates of Flood Hydrograph (Cumecs)	Baseflow (Cumecs)	Ordinates of Direct Runoff Hydrograph (Cumecs)	Ordinates of the 6 Hrs. Duration Unit Hydrograph (Cumecs)
1	2	3	4	5	6	7
14.8.76	0100	6	880	505	375	20
	0400	9	1875	475	1400	77
	0700	12	2875	450	2425	133
	1000	15	3175	430	2745	150
	1300	18	2850	420	2430	133
	1600	21	2080	430	1650	90
	1900	24	1560	440	1120	61
	2200	27	1220	450	770	42
15.8.76	0100	30	1015	460	550	30
	0400	33	840	470	370	21
	0700	36	755	480	275	15
	1000	39	670	490	180	10
	1300	42	600	500	100	5
	1600	45	545	505	40	2
	1900	48	510	510	0	0
					$\Sigma O = 14510$	



OBSERVED FLOOD HYDROGRAPH
ANANDPUR ON R.BAITARANI

FIG.IV(2).1



6 HOUR UNIT HYDROGRAPH AT ANANDPUR

FIG.IV(2).2

APPENDIX IV (3)

EXAMPLE ON DERIVATION OF INSTANTANEOUS UNIT HYDROGRAPH

Example

A storm of mild intensity was experienced in the catchment of river Baitarani during the period from 26.9.75 to 28.9.75. The rainfall was rather non-uniform. The average hourly rainfall over the catchment and the resulting observed discharge at Anandpur site are plotted in Fig. IV (3).1.

Use data pertaining to this storm and derive IUH and then the unit hydrograph of one hour unit duration.

Solution

The various steps involved in the procedure are as follows:

1. Separation of baseflow to find DRH

   (i) The anticipated recession curve of the smaller peak just before this flood was continued till the time of the peak i.e. upto point B.

   (ii) A suitable point C was chosen on the falling limb at a distance of about 2t to 2.5t where 't' is the time from the rise of flood hydrograph to the peak.

   (iii) The points B and C were joined by a straight line. The curve ABC separated the baseflow from total flood.

   (iv) The baseflow thus separated was deducted from the corresponding ordinates of the flood hydrograph to get the direct runoff hydrograph (DRH). DRH is shown in the Fig. IV (3).2.

DATE	TIME	ORDINATE OF FLOOD HYDROGAPH	BASEFOW	ORDINATES OF DRH
26 9/75	1200	475	475	0
	1500	479	450	29
	1800	485	432	54
	2100	854	422	432
27 9/75	0000	950	410	540
	0300	992	400	592
	0600	934	405	529
	0900	899	410	489
	1200	854	413	432
	1500	793	416	377
	1800	726	418	308
	2100	654	421	235
28 8/75	0000	572	424	154
	0300	523	426	97
	0600	482	429	53
	0800	455	432	23
	1200	435	435	0

$\Sigma O = 4344$ CUMES
$= 5.47$ CM.

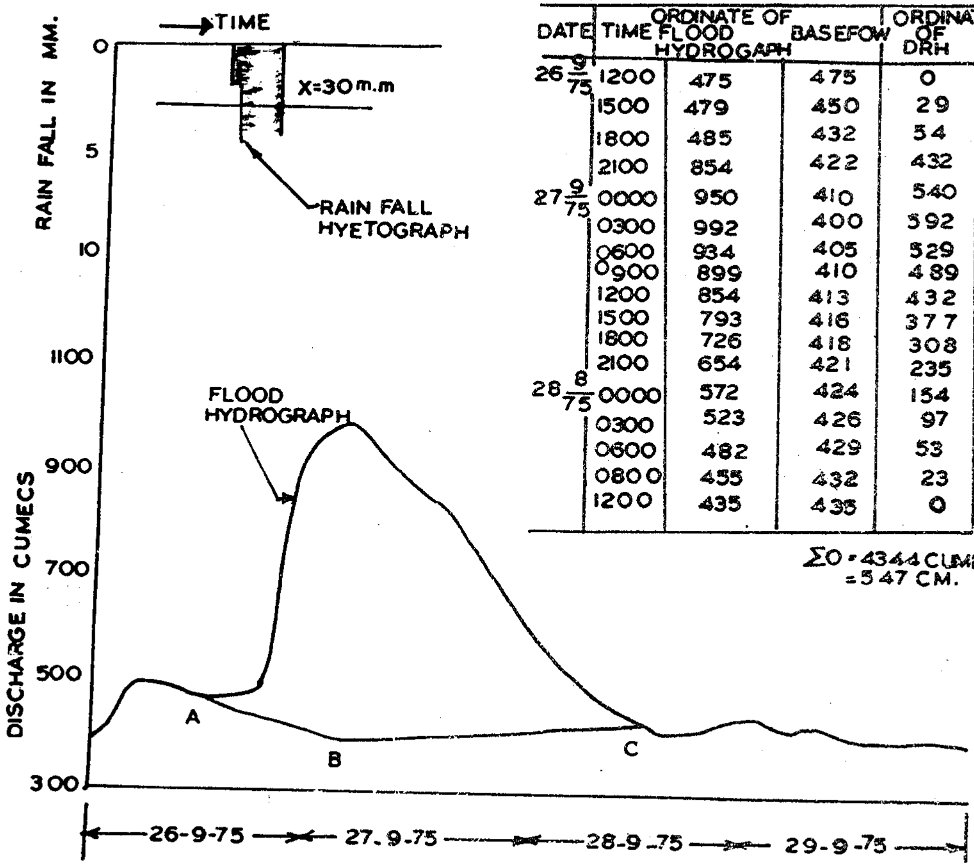


FIG. IV(3).1.

TIME	ORDINATES OF ERN IN MM/SEC.
1	1.30
2	0.525

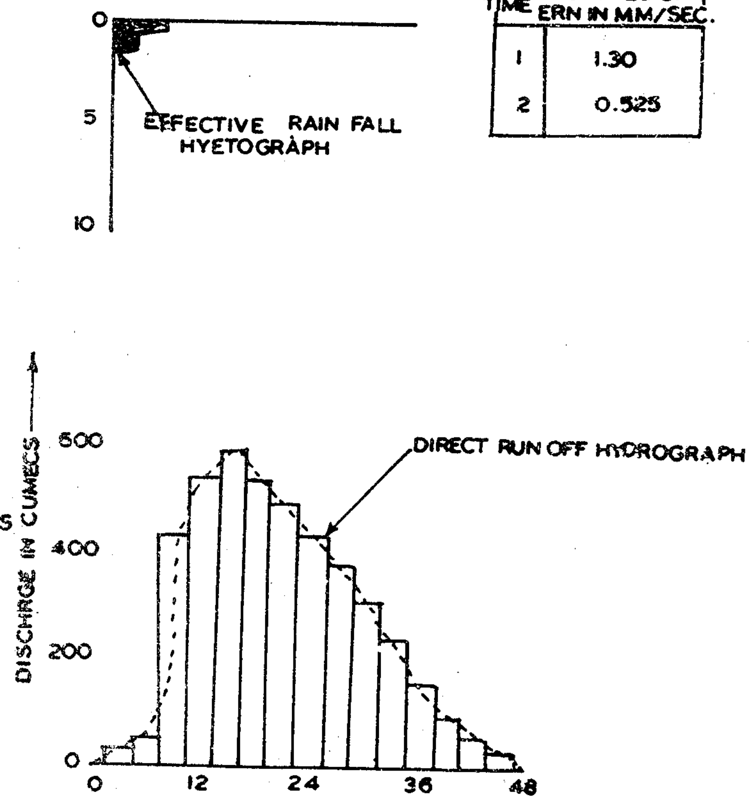


FIG. IV(3).2

2. Separation of Rainfall Excess from Total Rainfall:

The total hourly rainfall (average over the basin) is shown in the Fig. IV (3).1. Since the storm under consideration is in the month of September and there was a heavy storm in August and yet another storm of smaller intensity in early September, the loss may be considered to take place at a uniform rate.

Let the loss be @ X mm/hour.

Now, the total of the hourly ordinates of DRH is found out. It works out to be 13033 cumecs. Hence, the total direct runoff $\frac{13033 \times 3.6}{8570} = 5.475$ mm.

where 8570 is the area of catchment of River Baitarani up to Anandpur in Km^2.

Now let the loss rate, X be more than 2 mm/hour.

Time	*Total Rainfall*	*Loss*	*Effefective Rainfall*
1.	2	2	0
2.	4.65	X	4.65 − X = 1.605
3.	5.34	X	5.34 − X = 2.295
4.	0.1	0.1	0
5.	0.1	0.1	0
6.	4.62	X	4.62 − X = 1.575
		Total :	14.61 − 3 X

Equating 14.61 − 3 X = 5.475

We get, X = 3.045 mm

Substituting the value of X, the effective rainfall at different time is obtained.

For the purpose of analysis, the hourly ordinate will involve a lot of computational work, and therefore, only the three hourly rainfall ordinates have been considered. The hourly rainfalls are assumed to be uniform during three hours and accordingly, the DRH and ERH is drawn which is shown in Fig.IV.(3).2

3. Calculation of n and K

$$M_{ERH1} = \text{first moment arm of ERH}$$

$$= \frac{\text{first moment arm of ERH about 0}}{\text{Area of ERH}}$$

$$= \frac{1.3\times3\times\left(\frac{3}{2}\right)+0.525\times3\times\left(3+\frac{3}{2}\right)}{1.3\times3+0.525\times3}$$

$$= \frac{5.85+7.088}{3.9+1.575} = \frac{12.938}{5.475} = 2.363$$

$$M_{ERH2} = \text{Second moment arm of ERH}$$

$$= \frac{\text{Second moment of ERH about 0}}{\text{Area of ERH}}$$

$$= \frac{1.3\times3\times\left(\frac{3}{2}\right)^2+0.525\times3\times\left(3+\frac{3}{2}\right)^2}{5.475}$$

$$= \frac{8.775+31.894}{5.475} = 7.428$$

$$M_{DRH1} = \text{First moment arm of DRH}$$

$$= \frac{\text{First moment of DRH about 0}}{\text{Area of DRH}}$$

$$= \frac{\begin{array}{c}(29\times3\times3)+(54\times3\times6)+(432\times3\times9)+(540\times3\times12)+\\(592\times3\times15)+(529\times3\times18)+(489\times3\times21)+\\(432\times3\times24)+(377\times3\times27)+(308\times3\times30)+\\(235\times3\times33)+(154\times3\times36)+(97\times3\times39)+\\(53\times3\times42)+(23\times3\times45)\end{array}}{\begin{array}{c}(29\times3)+(54\times3)+(432\times3)+(540\times3)+\\(592\times3)+(529\times3)+(489\times3)+(432\times3)+\\(377\times3)+(308\times3)+(235\times3)+(154\times3)+\\(97\times3)+(53\times3)+(23\times3)\end{array}}$$

$$= \frac{268740}{13032} = 20.622$$

M_{DRH2} = 2nd moment arm of DRH

$$= \frac{\text{2nd moment of DRH about 0}}{\text{Area of DRH}} = 20.622$$

$$= \frac{\begin{gathered}(29\times3\times3^2) + (54\times3\times6^2) + (432\times3\times9^2) + (540\times3\times12^2) + \\ (592\times3\times15^2) + (529\times3\times18^2) + (489\times3\times21^2) + \\ (432\times3\times24^2) + (377\times3\times27^2) + (308\times3\times30^2) + \\ (235\times3\times33^2) + (154\times3\times36^2) + \\ (97\times3\times39^2) + (53\times3\times42^2) + (23\times3\times45^2)\end{gathered}}{13032}$$

$$= \frac{6537510}{13032} = 501.651$$

now, $nK = M_{DRH1} - M_{ERH1}$

$= 20.622 - 2.363 = 18.259$

and $n(n+1)K^2 = M_{DRH2} - M_{ERH2} - 2\ nK\ M_{ERH1}$

$= 501.651 - 7.428 - 2\times18.259 \times 2.363 = 407.931$

From the above two equations, $K = 4.083$
$n = 4.471$

Hence, for analysis the values of K and n may be taken as 4 and 4 respectively (In case of very sandy soil characteristics of the catchment 'n' may be taken as 5 if its value works out to be 4.47, whereas for hilly or semi-hilly regions of catchment, n should be 4).

4. To estimate the ordinates of IUH:

Once n and K are found out, the ordinates of the IUH can be found very easily by using the relation

$$U(t) = \frac{1}{K(n-1)!} \cdot \left(\frac{t}{K}\right)^{n-1} . e^{-t/K}$$

This will give ordinates in units of Sec^{-1}

To find the ordinates in cumecs, it is multiplied

by $\frac{1}{1000}$ m $\times$ (8570 $\times$ 1000 $\times$ 1000) $\times \frac{1}{60} \times \frac{1}{60}$ m^3

i.e. by 2380

$$\therefore U(t) = \frac{2380}{4 \times 31}\left(\frac{t}{4}\right)^{4-1} . e^{\frac{-t}{4}}$$

$$= \frac{2380}{124 \times 64} . t^3 . e^{\frac{-t}{4}}$$

where U(t) = ordinate of IUH at t^{th} hour.

By substituting the t = 1, 2, 3, 4 .... in the above equation, the hourly ordinate of the IUH can be obtained. The ordinates of IUH are given below:

HOURLY ORDINATES OF IUH (IN CUMECS)

1.21	133.34	60.01
7.52	132.03	53.10
19.77	128.42	46.74
36.49	123.01	40.95
55.50	116.27	35.71
74.70	108.61	31.02
92.38	100.41	26.84
107.39	91.97	23.14
119.08	83.54	19.88
127.22	75.31	17.03
131.37	67.44	14.55

12.39	2.64	0.49
10.52	2.20	0.40
8.92	1.83	0.33
7.54	1.52	0.27
6.35	1.26	0.22
5.35	1.05	0.18
4.50	0.87	0.15
3.77	0.72	0.12
3.16	0.59	0.10
		0.08

It will be seen that the last ordinate is never zero. This is because of the fact that the recession of IUH is generally defined by an exponential function which has zero value only at infinity. Hence, the recession is terminated at a suitable point.

5. Derivation of 1 Hr. duration Unit Hydrograph from IUH:

The procedure to derive 'T' hour duration unit hydrograph from the IUH has already been discussed above.

However, the t^{th} hour ordinates of the 1 hour duration unit hydrograph can be very easily found by simply taking the average of t^{th} hour and $(t-1)^{th}$ hour ordinates of the IUH.

Column 2 of the following table gives the ordinates of the IUH (rounded off figures) and the column 3 gives the ordinates of the 1 hour duration unit hydrograph.

Figure IV(3).3 shows the IUH and the 1 hour duration unit hydrograph for river Baitarani at Anandpur site.

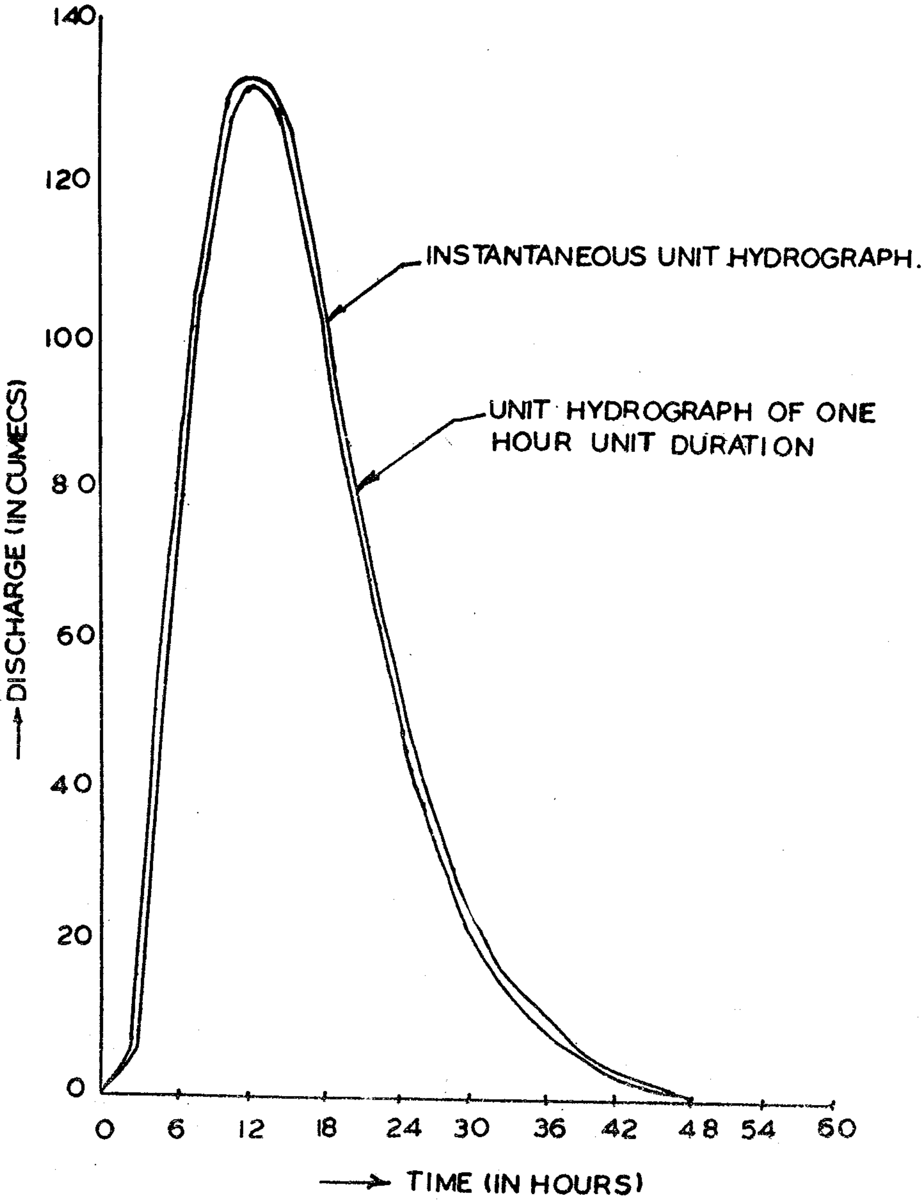

140
120
100
80
60
40
20
0
DISCHARGE (IN CUMECS)
INSTANTANEOUS UNIT HYDROGRAPH.
UNIT HYDROGRAPH OF ONE
HOUR UNIT DURATION
0 6 12 18 24 30 36 42 48 54 60
TIME (IN HOURS)

FIG. IV(3).3.

Time (Hours)	Ordinates of IUH (in cumecs)	Ordinates of unit hydrograph of 1 hr. duration (in cumecs)
1.	2.	3.
0	0	0
1	1	1
2	8	4
3	20	14
4	36	28
5	56	46
6	75	65
7	92	84
8	107	100
9	119	113
10	127	123
11	132	130
12	133	132
13	132	133
14	128	130
15	123	126
16	116	120
17	109	112
18	100	105

Time (Hours)	*Ordinates of IUH (in cumecs)*	*Ordinates of unit hydrograph of 1 hr. duration (in cumecs)*
1.	*2.*	*3.*
19	92	96
20	84	88
21	75	79
22	67	71
23	60	64
24	53	57
25	47	50
26	41	44
27	36	38
28	31	33
29	27	29
30	23	25
31	20	22
32	17	19
33	14	16
34	12	13
35	10	11
36	9	9
37	8	8

Time (Hours)	*Ordinates of IUH (in cumecs)*	*Ordinates of unit hydrograph of 1 hr. duration (in cumecs)*
1.	*2.*	*3.*
38	6	7
39	5	6
40	4	5
41	4	4
42	3	3
43	3	3
44	2	2
45	2	2
46	2	2
47	2	2
48	1	1
49	1	1
50	0	0

APPENDIX IV(4)

EXAMPLE ON DERIVATION OF UNIT HYDROGRAPH BY COLLIN'S METHOD

Example

The direct runoff hydrograph at Anandpur and the effective rainfall hydrograph due to a particular storm over the catchment of river Baitarani are shown in Fig. IV(4).1.

Use the given DRH and ERH to derive unit hydrograph for river Baitarani at Anandpur by using Collin's method.

Solution

The effective rainfall hyetograph blocks are for 3 hour intervals. Therefore, the unit duration of the unit hydrograph thus derived will be of three hour unit duration.

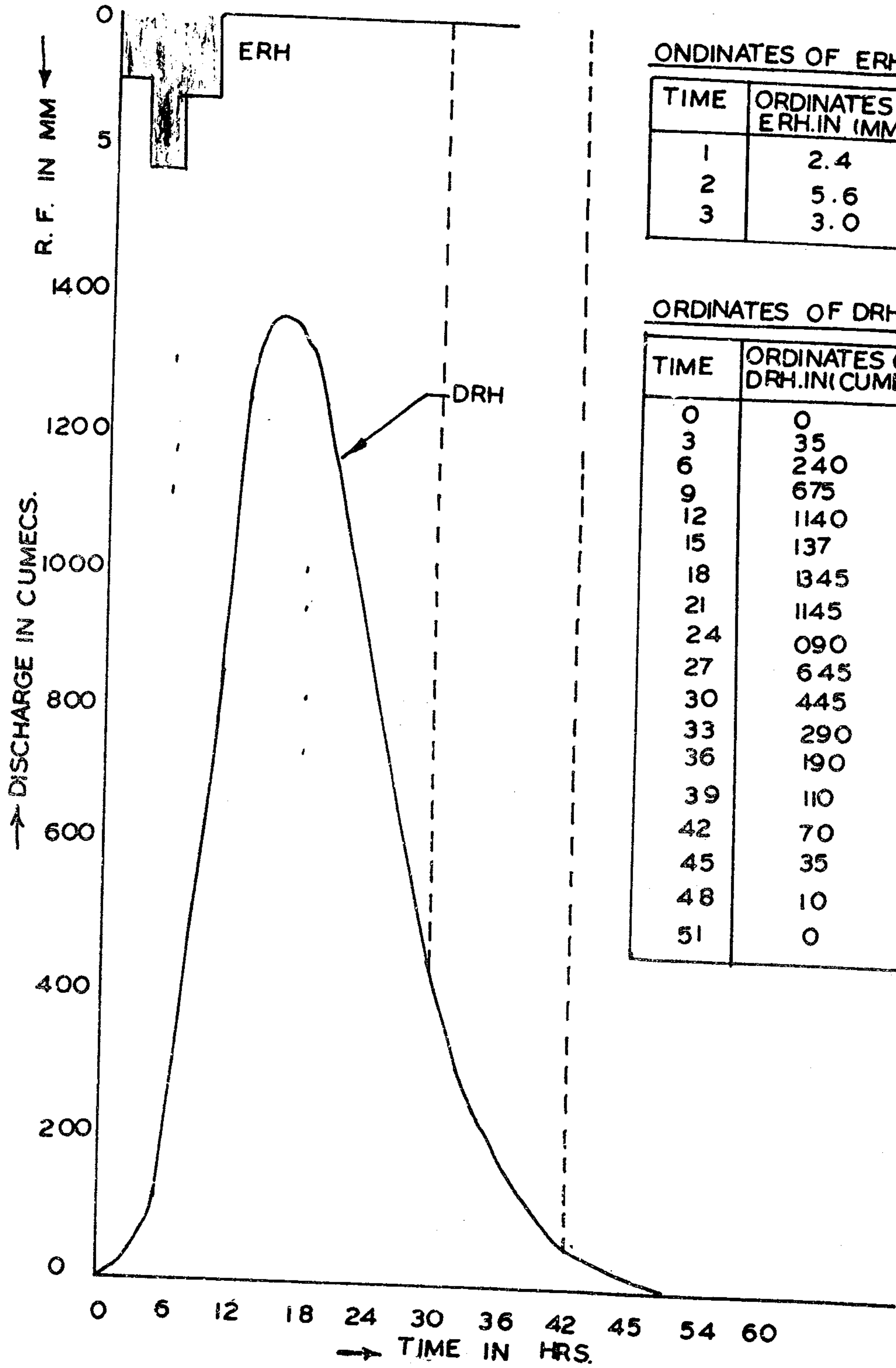
The various steps involved in the procedure are explained below:

(i) From the observed flood hydrograph and observed rainfall hyetograph DRH and ERH were separated as already explained in Appendix IV (3). The DRH and the ERH are shown in Fig. IV(4).1.

(ii) The ordinates of the DRH at different time were written under Column 2 of Table IV (4).1.

(iii) The ordinates of the unit hydrograph were assumed and were written under column 3 of the Table IV (4).1.

(iv) The ordinates of the assumed unit hydrograph were summed up and found to be 752 cumecs. But the sum of ordinates of the unit hydrograph at 3 hour interval, ΣU for a catchment area of 8570 km^2 should be:



ONDINATES OF ERH.

TIME	ORDINATES OF ERH.IN (MM)
1	2.4
2	5.6
3	3.0

ORDINATES OF DRH.

TIME	ORDINATES OF DRH.IN(CUMES)
0	0
3	35
6	240
9	675
12	1140
15	137
18	1345
21	1145
24	090
27	645
30	445
33	290
36	190
39	110
42	70
45	35
48	10
51	0

FIG. IV(4).1.

Unit Hydrograph by Collin's Method

1st Iteration

Time	Direct Runoff Hydro-graph	Assumed unit Hydro-graph	Adjusted value of assumed unit Hy-drograph	Hydrograph due to effective Rainfalls excepting the largest one				Difference between 2 & 8	Unit hydro-graph	Adjusted value of calcula-ted U.G.	Weighted Average of ass-med & calcula-ted U.G.	Re-marks
				2.4 mm	5.6 mm	3.0 mm	Total					
1	2	3	4	5	6	7	8	9	10	11	12	13
0	0	0	0	0	-	-	0	0	0	0	0	
3	35	20	21.1	50.6	0	-	50.6	-15.6	6.7	6.8	13.8	
6	240	80	84.4	202.6	0	0	202.6	37.4	50.5	51.3	67.5	
9	675	130	137.1	329.0	0	63.3	392.3	282.7	90.6	92.1	114.2	
12	1140	150	158.2	370.7	0	253.2	632.9	507.1	122.4	124.4	140.9	
15	1375	110	116.0	278.4	0	411.3	689.7	685.3	119.3	121.2	118.7	
18	1345	80	84.3	202.3	0	747.6	676.9	668.1	115.2	117.0	101.0	
21	1145	60	63.3	151.9	0	348.0	499.9	645.1	95.7	97.2	80.6	
24	890	40	42.2	101.3	0	252.9	354.2	535.8	67.7	68.8	55.7	
27	645	30	31.6	75.8	0	189.9	265.7	379.3	47.8	48.6	40.3	
30	445	20	21.1	50.6	0	126.6	177.2	267.8	28.1	28.6	24.9	
33	290	15	15.8	37.9	0	94.8	132.7	157.3	18.1	18.4	17.1	
36	190	10	10.5	25.2	0	63.3	88.5	101.5	8.9	9.0	9.8	
39	110	5	5.3	12.7	0	47.4	60.1	49.9	6.0	6.1	5.7	

Table - IV(4).1 Contd.

Time	Direct Runoff Hydrograph	Assumed unit Hydrograph	Adjusted value of assumed unit Hydrograph	Hydrograph due to effective Rainfalls excepting the largest one				Difference between 2 & 8	Unit hydrograph	Adjusted value of calculated U.G.	Weighted Average of assumed & calculated U.G.	Remarks
				2.4 mm	5.6 mm	3.0 mm	Total					
1	2	3	4	5	6	7	8	9	10	11	12	13
42	70	2	2.1	5.0	0	31.5	36.5	33.5	3.4	3.5	2.8	
45	33	0	0	0	0	15.9	15.9	19.1	0	0	0	
48	10	-	-	-	0	6.3	6.3	3.7	-	-	-	
51	0	-	-	-	-	0	0	0	-	-	-	
		752	793						780.4	793	793	

When the corresponding ordinates of unit Hydrograph under column (4) and column (12) are compared, it is found that the difference in the ordinates are quite significant and hence the second iteration is to be carried out.

$$\Sigma U = \frac{A}{3.6 \times t} \quad \text{where A = area of catchment in sq. km.}$$

(= 8570 km^2)

$$= \frac{8570}{3.6 \times 3} \quad t = \text{time interval in hour}$$

(= 3 hours)

$$= 793$$

(v) All the assumed ordinates were multiplied by $\frac{793}{752}$ and the adjusted value written in col. 4 of the table IV(4).1.

(vi) The figures on the top of column 5, 6 and 7 of the Table IV(4).1 are the effective rainfalls in order to time.

The ordinates on adjusted unitgraph (col.4) were multiplied by 2.4 and written under col. 5. Since the effective rainfall of 5.6 mm is the highest, the ordinates of adjusted unitgraph (col. 4) were multiplied by zero and written in col. 6 after shifting by 3 hours. Further the ordinates of adjusted unit hydrograph (col. 4) were multiplied by 3.0 mm and written in col. 7 after shifting by 6 hours.

(vii) The ordinates in col. 5, 6 and 7 were added together and written under column 8. This gives the flood hydrograph resulting from effective rainfalls excepting the largest one i.e. 5.6 mm.

(viii) The flood hydrograph ordinates obtained in col. 8 were deducted from the ordinates of DRH in col. 2 and this was noted down in col. 9. This was supposed to give hydrograh resulting from 5.6 mm of rainfall.

(ix) The values in col. 9 were divided by 5.6 mm to give the unit hydrograph and were noted in col. 10. The sum of the ordinates of calculated U.G. as written in col. 10 were found to be 780.4. These ordinates were multiplied by $\frac{793}{780.4}$ to give the adjusted values of the calculated unit hydrograph which are written in col. 11.

(x) The weighted average of the two unit hydrographs (the assumed one as in column 4 and calculated one as in col. 11) were found in the following manner:

$$\begin{bmatrix}\text{Weighted}\\ \text{average of}\\ \text{the ordinates}\\ \text{of the unit}\\ \text{hydrograph}\end{bmatrix} = \frac{\left[\begin{Bmatrix}\text{Ordinates}\\ \text{adjusted}\\ \text{assumed}\\ \text{unitgraph}\end{Bmatrix} \times \begin{Bmatrix}\text{Sum of effec-}\\ \text{tive r.f. exce-}\\ \text{pting lar-}\\ \text{gest one}\end{Bmatrix} + \begin{Bmatrix}\text{Ordinates}\\ \text{of calculat-}\\ \text{ed unit hyd-}\\ \text{rograph}\end{Bmatrix} \times \begin{Bmatrix}\text{Largest}\\ \text{effective}\\ \text{rainfall}\end{Bmatrix}\right]}{\text{Total effective rainfall}}$$

for example, first ordinate will be,

$$\frac{0 \times 5.4 + 0 \times 5.6}{(5.4 + 5.6)} = 0$$

the 2nd ordinate will be,

$$\frac{21.1 \times 5.4 + 6.8 \times 5.6}{(5.4 + 5.6)} = 13.8$$

The weighted average ordinates are written under column 12 of the Table IV(4).1.

xi) The ordinates of the unit hydrograph in column 12 and column 4 were compared; the differences were found to be quite considerable, hence the 2nd iteration was carried out by using the calculated value of unit hydrograph (column 12) as the assumed unit hydrograph (column 4) in the 2nd iteration.

The calculations for 2nd iteration are given in Table IV(4).2. Standard computer programmes are available for deriving unit hydrograph by using the Collin's Method.

Table - IV(4).2

Unit Hydrograph by Collin's Method

2nd Iteration

Time in hrs.	Direct Runoff Hydro-graph	Assumed unit Hydro-graph	Adjusted value of assumed unit Hy-drograph	Hydrograph due to effective Rainfalls excepting the largest one				Difference between 2 & 8	Unit hydro-graph	Adjusted value of calcula-ted U.G.	Weighted Average of ass-med & calcula-ted U.G.	Re-marks
				2.4 mm	5.6 mm	3.0 mm	Total					
1	2	3	4	5	6	7	8	9	10	11	12	13
0	0	0	0	0	-	-	0	0	0	0	0	
3	35	13.8	13.8	33.1	0	-	33.1	1.9	13.9	14.2	14.0	
6	240	67.5	67.5	162.0	0	0	162.0	78.0	64.2	65.8	66.6	
9	675	114.2	114.2	274.1	0	41.4	315.5	359.5	107.0	109.6	111.9	
12	1140	140.9	140.9	358.1	0	202.5	540.6	599.4	133.5	136.8	138.8	
15	1375	118.7	118.7	284.9	0	342.6	627.5	747.5	121.4	124.3	121.6	
18	1345	101.0	101.0	242.4	0	422.7	665.1	679.9	102.8	105.3	103.2	
21	1145	80.6	80.6	193.4	0	376.1	569.5	575.5	81.0	83.0	81.8	
24	890	55.7	55.7	133.6	0	303.0	436.6	453.4	54.7	56.0	55.9	
27	645	40.3	40.3	96.7	0	241.8	338.5	306.5	39.0	40.0	40.1	
30	445	24.9	24.9	59.7	0	167.1	226.8	218.2	22.9	23.5	24.2	
33	290	17.1	17.1	41.0	0	120.9	161.9	128.1	16.4	16.8	16.9	
36	190	9.8	9.8	23.5	0	74.7	98.2	91.8	8.0	8.2	9.0	
39	110	5.7	5.7	13.7	0	51.3	65.0	45.0	6.1	6.2	6.0	

Table - IV(4).2 Contd.

Time in hrs.	Direct Runoff Hydro-graph	Assumed unit Hydro-graph	Adjusted value of assumed unit Hy-drograph	Hydrograph due to effective Rainfalls excepting the largest one				Difference between 2 & 8	Unit hydro-graph	Adjusted value of calcula-ted U.G.	Weighted Average of ass-med & calcula-ted U.G.	Re-marks
				2.4 mm	5.6 mm	3.0 mm	Total					
1	2	3	4	5	6	7	8	9	10	11	12	13
42	70	2.8	2.8	6.7	0	29.4	36.1	33.9	3.2	3.3	3.0	
45	35	0	0	0	0	17.1	17.1	17.9	0	0	0	
48	10	-	-	-	0	8.4	8.4	1.6	0	0	-	
51	0	-	-	-	-	0	0	0	-	-	-	
		793	793						774.1	793		

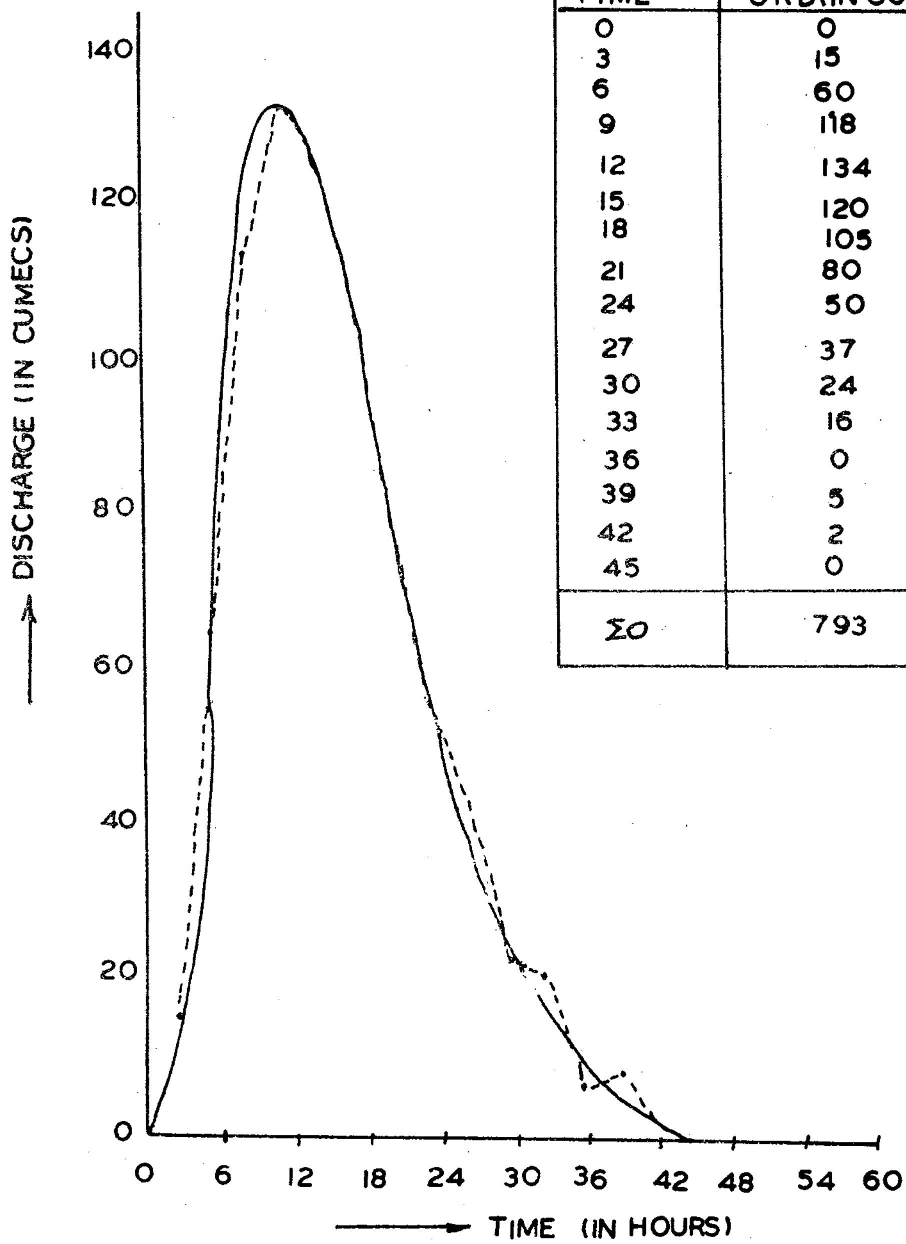
A comparison of the column (4) and (12) indicates that the differences are still significant and hence another iteration is to be carried out.

The above mentioned problem was solved on the computer and the differences in the assumed and calculated unit hydrographs were found to be insignificant after 17th iteration. The value of unit hydrograph ordinates after 17th iteration were found to be as follows:

Time (Hrs.	*Unit hydrograph ordinates (Cumecs)*
0	0
3	15
6	65
9	113
12	134
15	126
18	105
21	80
24	56
27	41
30	22
33	20
36	5
39	9
42	2
45	0

The ordinates of the derived unit hydrograph are plotted and shown as dotted line in Fig. V(4).2. The lower part of the unit hydrograph was not uniform and therefore a smooth curve was edrawn smoothly with least variations and such that the sum of the ordinates was 793. This as been considered as the unit hydrograph of 3 hour unit duration for Anandpur site for river Baitarani.

ORDINATES OF UNIT GRAPH	
TIME	ORD(IN CUMECS)
0	0
3	15
6	60
9	118
12	134
15	120
18	105
21	80
24	50
27	37
30	24
33	16
36	0
39	5
42	2
45	0
ΣO	793



3-HOUR UNIT HYDROGRAPH AT ANANDPUR

FIG- IV(4).2

APPENDIX IV (5)

EXAMPLE ON CONVERSION OF UNIT HYDROGRAPH OF 't' H R. DURATION TO THAT OF 'nt' HOUR DURATION (n BEING AN INTEGER)

Example

A unit hydrograph of one hour unit duration has been derived in the example presented in Appendix IV (3). Convert this unit hydrograph to a unit hydrograph of 3-hour unit duration.

Solution

The various steps involved in the method are discussed below:

(i) The hourly ordinates of the one hour unit duration unit hydrograph are written in col. 2 of the table IV (5).1.

(ii) The ordinates are re-written under col. 3 after shifting the ordinates by one hour.

(iii) The ordinates are re-written under col. 4 after shifting the ordinates by two hours.

(iv) The sum of the ordinates in columns 2, 3 and 4 are written under col. 5. This will give the hydrograph resulting from the effective rainfall of 3 mm in 3 hours.

(v) When the ordinates given under col. 5 are divided by 3, it will give the ordinates of unit hydrograph of 3 hour unit duration. This is given in col. 6 of the table IV (5).1.

TABLE IV (5).1

Time in Hrs.	Ordinates of 1 Hr. duration unit graph.	Ordinates of 1 Hr. duration U.G. shifted by 1 Hr.	Ordinates of 1 Hr. duration U.G. shifted by 2 Hrs.	Sum of Col. (2) + Col. (3) + Col. (4). The flood hydrograph due to 3 mm of effective rain in 3 Hour.	Ordinates of 3 Hr. U.G. = ordinate $\frac{\text{in Col. 5}}{3}$
1.	2.	3.	4.	5.	6.
0	0	-	-	0	0
1	1	0	-	1	0.3
2	4	1	0	5	1.7
3	14	4	1	19	6.3
4	28	14	4	46	15.3
5	46	28	14	88	29.3
6	65	46	28	139	46.3
7	84	65	46	195	65
8	100	84	65	249	83
9	113	100	84	297	99
10	123	113	100	336	112
11	130	123	113	366	122
12	132	130	123	385	128.3
13	133	132	130	395	131.7
14	130	133	132	395	131.7
15	126	130	133	389	129.7
16	120	126	130	376	125.3

Time in Hrs.	*Ordinates of 1 Hr. duration unit graph.*	*Ordinates of 1 Hr. duration U.G. shifted by 1 Hr.*	*Ordinates of 1 Hr. duration U.G. shifted by 2 Hrs.*	*Sum of Col. (2) + Col. (3) + Col. (4). The flood hydrograph due to 3 mm of effective rain in 3 Hour.*	*Ordinates of 3 Hr. U.G. = ordinate $\frac{\text{in Col. 5}}{3}$*
1.	*2.*	*3.*	*4.*	*5.*	*6.*
17	112	120	126	358	119.3
18	105	112	120	337	112.3
19	96	105	112	313	104.3
20	88	96	105	289	96.3
21	79	88	96	263	87.7
22	71	79	88	238	79.3
23	64	71	79	214	71.3
24	57	64	71	192	64
25	50	57	64	171	57
26	44	50	57	151	50.3
27	38	44	50	132	44
28	33	38	44	115	38.3
29	29	33	38	100	33.3
30	25	29	33	87	29
31	22	25	29	76	25.3
32	19	22	25	66	22
33	16	19	22	57	19
34	13	16	19	48	16

Time in Hrs.	*Ordinates of 1 Hr. duration unit graph.*	*Ordinates of 1 Hr. duration U.G. shifted by 1 Hr.*	*Ordinates of 1 Hr. duration U.G. shifted by 2 Hrs.*	*Sum of Col. (2) + Col. (3) + Col. (4). The flood hydrograph due to 3 mm of effective rain in 3 Hour.*	*Ordinates of 3 Hr. U.G. = ordinate in Col. 5 / 3*
1.	2.	3.	4.	5.	6.
35	11	13	16	40	13.3
36	9	11	13	33	11
37	8	9	11	28	9.3
38	7	8	9	24	8
39	6	7	8	21	7
40	5	6	7	18	6
41	4	5	6	15	5
42	3	4	5	12	4
43	3	3	4	10	3.3
44	2	3	3	8	2.7
45	2	2	3	7	2.3
46	2	2	2	6	2
47	2	2	2	6	2
48	1	2	2	5	1.7
49	1	1	2	4	1.3
50	0	1	1	2	0.7
51	-	0	1	1	0.3
52	-	-	0	0	0

APPENDIX IV (6)

EXAMPLE ON S-HYDROGRAPH

Example

A unit hydrograph of six hour unit duration has been derived for river Baitarani at Anandpur in the example presented in Appendix IV (2). Derive the S-hydrograph and then the unit hydrograph of three hour unit duration.

Solution

The various steps are discussed below :

(i) The 6-hour duration unit hydrograph is Fig. IV (6).1. The ordinates are given in col. 2 of Table IV (6).1.

(ii) Since the unit duration is 6-hours, the unit hydrograph is shifted by 6 hours and drawn as shown in the Fig. IV (6).2. The col. 3, 4, 5... contain the ordinates of 6 hour duration unit hydrograph each shifted by 6, 12, 18... hours respectively.

(iii) The sum of the ordinates of the several unit hydrographs of 6 hour duration shifted by 6 hour are given in col. 11 of Table IV (6).1. and the points are plotted on the graph.

(iv) A smooth curve is drawn and its values noted down in col. 12 of Table IV(6).1. This gives the S-hydrograph due of effective rainfall intensity of 1/6 mm per hour for infinite duration.

(v) In order to derive the unit hydrograph of 3 hour duration, another S-hydrograph (offset S-hydrograph) is drawn after shifting it by 3 hours. Its ordinates are given in col. 13.

(vi) The difference of col. 12 and col. 13 gives the hydrograph due to $(1/6 \times 3=)$ 1/2 mm of effective rainfall in 3 hours.

(vii) The ordinates in col. 14 are divided by 1/2 to get the ordinates of unit hydrograph of 3 hour duration.

The unit hydrograph of 3 hour unit duration thus obtained for Anandpur site on river Baitarani is shown in Fig. IV (6).2.

Table IV (6).1

Derivation of Unit Hydrograph of 3 Hours Duration from 6 Hours Duration Unit Hydrograph

Time in hrs.	Ordinates of 6 hrs. unit Hydrograph	6 hrs. duration unit Hydrograph shifted by 6 hrs.	6 hrs. duration unit Hydrograph shifted by 12 hrs.	6 hrs. duration unit Hydrograph shifted by 18 hrs.	6 hrs. duration unit Hydrograph shifted by 24 hrs.	6 hrs. duration unit Hydrograph shifted by 30 hrs.	6 hrs. duration unit Hydrograph shifted by 36 hrs.	6 hrs. duration unit Hydrograph shifted by 42 hrs.	6 hrs. duration unit Hydrograph shifted by 48 hrs.	S-curve due to R.F. @ 1/6 mm per hour	Adjusted values of S-curve after smoothening	S-curve shifted by 3 hrs.	Hydrograph due to 1/2mm of R.F. in 3 hrs.	Unit Hydrograph of 3 hrs. duration
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
0	0	-	-	-	-	-	-	-	-	0	0	-	0	0
3	4	-	-	-	-	-	-	-	-	4	4	0	4	8
6	20	0	-	-	-	-	-	-	-	20	20	4	16	32
9	77	4	-	-	-	-	-	-	-	81	81	20	61	122
12	133	20	0	-	-	-	-	-	-	153	153	81	72	144
15	150	77	4	-	-	-	-	-	-	231	231	153	78	156
18	133	133	20	0	-	-	-	-	-	286	286	231	55	110
21	90	150	77	4	-	-	-	-	-	321	321	286	35	70
24	61	133	133	20	0	-	-	-	-	347	347	321	26	52
27	42	90	150	77	4	-	-	-	-	363	364	347	17	34
30	30	61	133	133	20	0	-	-	-	377	377	364	13	26
33	21	42	90	150	77	4	-	-	-	384	386	377	9	18
36	15	30	61	133	133	20	0	-	-	392	392	386	6	12
39	10	21	42	90	150	77	4	-	-	394	395	392	3	6
42	5	15	30	61	133	133	20	0	-	397	396.5	395	1.5	3

Table - IV(6).1 Contd.

Time in hrs.	Ordinates of 6 hrs. unit Hydrograph	6 hrs. duration unit Hydrograph shifted by 6 hrs.	6 hrs. duration unit Hydrograph shifted by 12 hrs.	6 hrs. duration unit Hydrograph shifted by 18 hrs.	6 hrs. duration unit Hydrograph shifted by 24 hrs.	6 hrs. duration unit Hydrograph shifted by 30 hrs.	6 hrs. duration unit Hydrograph shifted by 36 hrs.	6 hrs. duration unit Hydrograph shifted by 42 hrs.	6 hrs. duration unit Hydrograph shifted by 48 hrs.	S-curve due to R.F. @ 1/6 mm per hour	Adjusted values of S-curve after smoothening	S-curve shifted by 3 hrs.	Hydrograph due to 1/2mm of R.F. in 3 hrs.	Unit Hydrograph of 3 hrs. duration
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
45	2	10	21	42	90	150	77	4	-	396	396.5	396.5	0	0
48	0	5	15	30	61	133	133	20	0	397	396.5	396.5	0	-
		2	10	21	42	90	150	77	4	396	396.5	396.5	0	-
		0	5	15	30	61	133	133	20	397	396.5	396.5	0	-
			2	10	21	42	90	15	77	396	396.5	396.5	0	-
			0	5	15	30	61	133	133	397	396.5	396.5	0	-
				2	10	21	42	90	150	396	396.5	396.5	0	-

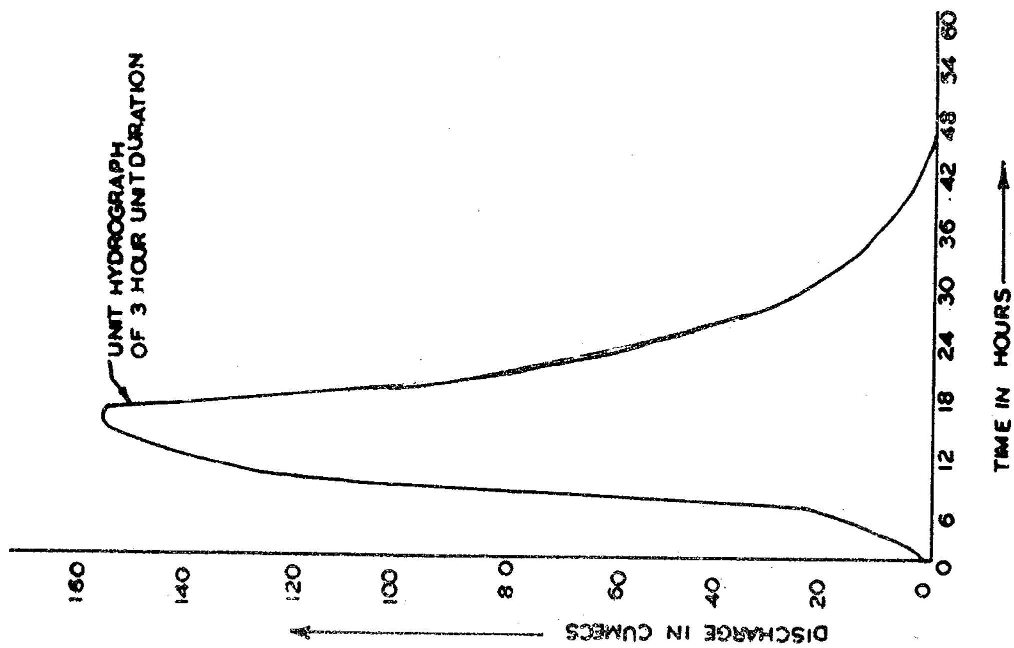


FIG. IV(6).2

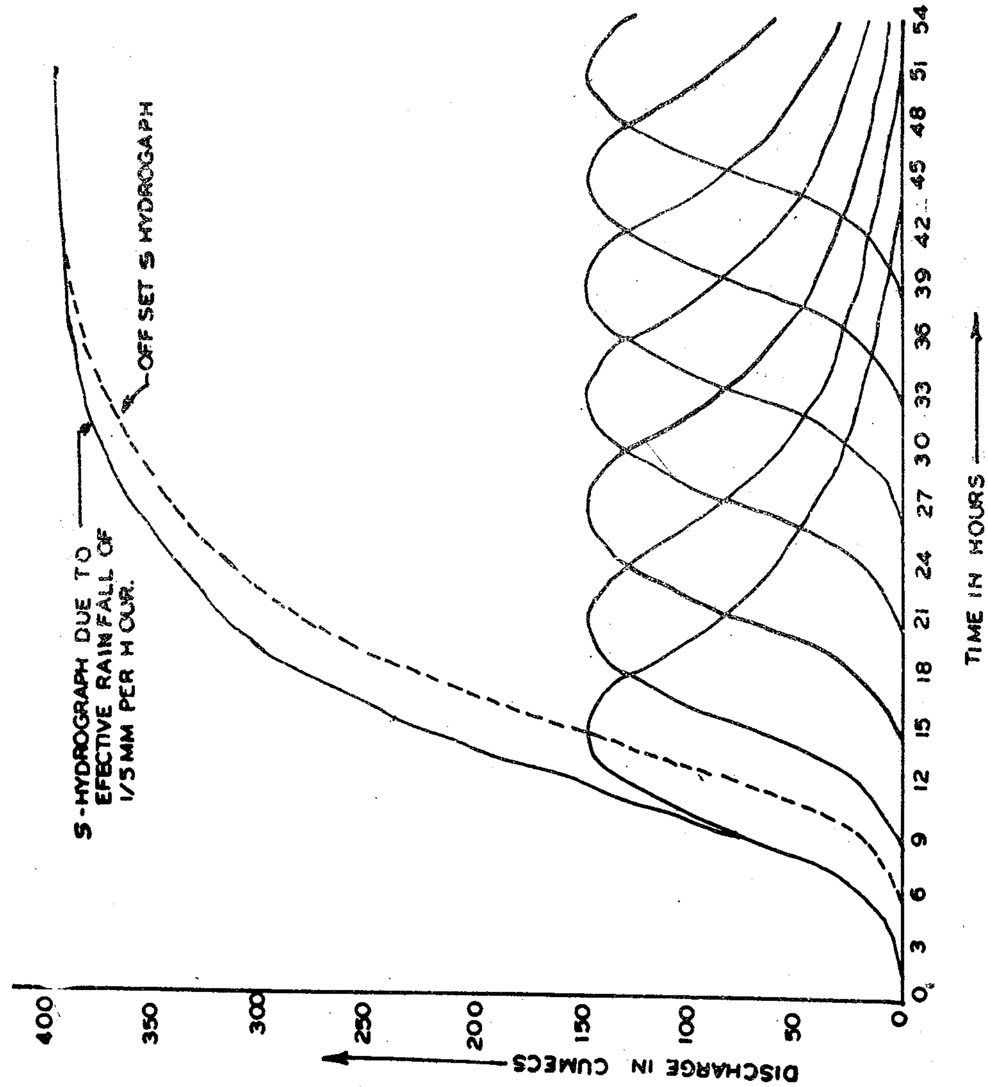


FIG. IV(6).1.

APPENDIX IV (7)

EXAMPLE ON FORMULATION OF STAGE FORECAST

Example

FORECAST : No. 3

Specimen Example of Calculations for Flood-Stage Forecast to DEONGAON Bridge

DEONGAON Date	Time	T.T.	TAKLI (SITE-23) Date	Time	G.R.	Q^1 Discharge in Cumecs. (M^3/sec)	T.T.	WADAKBAL (SITE-26) Date	Time
1	2	3	4	5	6	7	8	9	10
12.8.86	1500	15	11.8.86	24	8.040	2304	27	11.8.86	12
	1800	15	12.8.86	03	8.110	2347	27	11.8.86	15
	2100	15	12.8.86	06	8.145	2371	27	11.8.86	18
	2400	15	12.8.86	09	8.165	2385	27	11.8.86	21

G.R.	Q^2 Discharge in Cumecs (M^3/sec)	Total Discharge in Q^1+Q^2	Local flow.	Total.	F.C. Level Deongaon.	Actual M.L. at Deongaon Bridge.	Remarks.
11	12	13	14	15	16	17	18
2.600	16	2320	50	2370	400.450	400.500	
2.500	13	2360	50	2410	400.500	400.500	

G.R.	Q^2 Discharge in Cumecs (M^3/sec)	Total Discharge in Q^1+Q^2	Local flow.	Total.	F.C. Level Deongaon.	Actual M.L. at Deongaon Bridge.	Remarks.
11	12	13	14	15	16	17	18
2.405	10	2381	50	2431	400.550	400.550	
2.340	8	2393	50	2443	400.550	400.550	

Final Forecast **Issue Time : 1000 Hours**

12.8.1986 2400 400.550 Mtrs.

Future trend : -Future trend likely to rise slightly.

Notes:- 1. Column 3 and column 8 are from Table-IV(7).1.

2. Column 7 from Table-IV(7).2.

3. Column 12 from Table-IV(7).3.

4. Column 14 depending on local conditions for rain fall in the intermittant catchment.

5. Column 16 is from Table-IV(7).4.

6. The actual levels in column 17 are filled later from actual observations.

Table-IV(7).1

Travel Time From Site.23 Takli to Devangaon			**Travel Time From Site. 26 Wadakbal to Devangaon**		
T.T. in hours	**Gauges in Metres**		**T.T. in hours**	**Gauges in Metres**	
	From	**To**		**From**	**To**
27 hours	2.85	metres and below	27 hours	3.15	and below
24 hours	2.85	3.60	24 hours	3.16	4.20
21 hours	3.61	4.50	21 Hours	4.21	5.60
18 hours	4.51	6.00	18 hours	5.61	10.0
15 hours	6.01	8.40	15 hours	Above 10 metre	
12 hours	Above 8.40 Metres		–	-	–

TABLE-IV (7).2

G and D Ready Reckoner for the Site-23 Takli

(Discharge in Cumecs)

Gauge readings in Meters	*0*	*1*	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*
3.0	180	200	220	240	260	280	304	330	358	386
4.0	416	446	478	510	545	580	615	650	686	723
5.0	760	796	832	872	912	954	998	1044	1090	1136
6.0	1182	1228	1274	1320	1370	1420	1470	1520	1575	1630
7.0	1685	1740	1800	1860	1920	1980	2040	2100	2160	2220
8.0	2280	2340	2410	2480	2550	2620	2700	2780	2860	2940
9.0	3020	3100	3180	3260	3340	3420	3500	3590	3680	3770
10.0	3860	3950	4040	4130	4220	4320	4430	4530	4630	4730
11.0	4830	4930	5040	5170	5300	5450	5600	5760	5910	6070
12.0	6240	6410	6580	6750	6920	6990	7175	7375	7600	7810
13.0	8040	8290	8610	9000	9460	10000	10600	11200	11650	12100

Table-1V (7).3

G and D Ready Reckoner for the Site-26 Wadakbal

(Discharge in Cumecs)

Gauge readings in Meters	*0*	*1*	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*
2.0	0	2.5	5.0	7.5	10	13	16	19	22	26
3.0	30	35	41	48	55	62	69	77	85	95

Table IV (7).3 Contd.

Gauge readings in Meters	*0*	*1*	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*
4.0	105	115	125	135	145	158	171	184	197	210
5.0	225	240	256	272	288	304	322	340	358	376
6.0	394	412	430	448	466	485	505	525	545	565
7.0	585	605	625	645	668	691	714	738	762	786
8.0	812	839	866	893	920	948	978	1010	1045	1080
9.0	1105	1140	1175	1210	1255	1305	1355	1405	1460	1530
10.0	1600	1675	1750	1830	1910	2000	2100	2225	2400	2575

Table No. IV (7).4

Correlation Table Between Combined Discharge of Site 23 Takli and Site 26 Wadakbal with Water Level at Deongaon

Zero of Gauge : 394.000 M

Combined Discharge Site 26 + 23	*Water Level at Deongaon Bridge (in Metres)* 0	100	200	300	400	500	600	700	800	900
0				396.00	.40	.30	397.15	.40	.70	.90
1000	398.10	.35	.55	.70	.90	399.05	.25	.40	.60	.70
2000	399.90	400.10	.20	.35	.50	.65	.80	.95	401.10	.20
3000	401.35	.50	.60	.70	.80	.90	402.00	.15	.25	.35
4000	402.45	.55	.65	.75	.85	.95	403.05	.15	.25	.35
5000	403.45	.55	.65	.70	.80	.90	404.00	.05	.10	.20
6000	404.30	.40	.45	.50	.60	.70	.75	.80	.85	.90
7000	404.95	405.00	.10	.15	.25	.30	.35	.40	.45	.50
8000	405.58	405.60	.65	.70	.75	.80	.85	.90	406.00	406.05
9000	406.10	.15	.20	.25	.30	.35	.40	.43	.45	.50
10000	406.15	.60	.65	.67	.70	.75	.78	.80	.85	.90
11000	406.95	.98	407.00	.05	.10	.13	.15	.20	.23	.25
12000	407.30									

3 Hourly Gauge Data

Date	*Name of station/site*	*3 Hourly Gauge Data*							
		03.00	*06.00*	*09.00*	*12.00*	*15.00*	*18.00*	*21.00*	*24.00*
11.8.86	Takli.23.	7.780	7.800	7.800	7.800	7.840	7.900	7.975	8.040
	Wadakbal.26	2.190	2.230	2.580	2.600	2.500	2.405	2.340	2.310
12.8.86	Takli. 23	8.110	8.145	8.165	8.155	8.100	8.050	7.910	7.760
	Wadakbal. 26	2.270	2.250	2.430	2.505	2.480	2.440	2.425	2.410

EXAMPLE ON FORMULATION OF INFLOW FORECAST

Specimen *Calculation of Flood Stage Forecast at Sirsailam Dam 1986*

Forecast No. 19
Dated : 16.08.1986 Time : 09.30

Srisailam Date	Srisailam Time	T.T from DSR to SSM(Hrs)	Deosugur - Krishna Date	Deosugur - Krishna Time	G.R.	Discharge in 1000 cusecs.	Corrected discharge in 1000 cusecs
1	2	3	4	5	6	7	8
16.8.86	1500	15	15.08.86	24	7.400	267.8	– –
	1800		16.08.86	03	7.370	265.3	– –
	2100		16.08.86	06	7.320	261.2	– –
	2400		16.08.86	09	7.300	259.6	– –

T.T. from MTM to SSM(hrs)	Mantralayam – Tungabhadra Date	Time	G.R.	Discharge in 1000 cusecs Q^2	$Q^1 + Q^2$	Local in flow from Hundri	Total inflow col. 14+15
9	10	11	12	13	14	15	16
18	15.08.86	21	4.700	93.8	361.6	–	361.6
18.	15.08.86	24	4.700	93.8	359.1	–	359.1
18.	16.08.86	03	4.660	92.0	353.2	–	353.2
18.	16.08.86	06	4.595	89.3	348.9	–	348.9

Final Forecast Issue Time : 1000 Hours

16.08.1986 – 24.00 hrs. 3,45,000 cusecs.

Future Trend : The inflow is likely to decrease further.

Note: 1. Column 3 and Col. 9. are from Table No. IV (8) .1.

2. Column 7 from Table No. IV (8).2.

3. Column 13 from Table No. IV (8).3.

4. Column 15 depends on local condition such as rainfall and discharge in Hundri.

Table IV (8). 1

Travel Time Curves for 1984

Deosugur			*Mantralayam*		
T.T.in Hours	*G.R. in Meters* *from*	*To*	*T.T. in Hours.*	*G.R.in Meters* *From :*	*To:*
30	Below	3.25	30	Below	2.40
27	3.26	3.80	27	2.41	3.10
24	3.81	4.60	24	3.11	3.80
21	4.61	5.50	21	3.81	4.60
18	5.51	6.75	18	4.61	5.80
15	6.76	8.00	15	5.81	7.20
12	Above	8.0	12	Above	7.20

Table No.IV (8) 2

G and D Ready Reckoner for the Site No.6 Deosugur for 1983
Discharge in 1000 cusecs

Gauge reading in Meters.	*0.0*	*0.1*	*0.2*	*0.3*	*0.4*	*0.5*	*0.6*	*0.7*	*0.8*	*0.9*
1.0	3.5	4.2	5.0	6.0	8.0	10.0	12.2	14.4	16.6	18.8
2.0	21.0	23.4	25.8	28.2	30.6	33.0	35.6	38.2	40.8	43.4
3.0	46.0	49.0	52.0	55.0	58.0	61.0	64.8	68.6	72.4	76.2
4.0	80.0	84.0	88.0	92.0	96.0	100.0	104.2	108.4	112.6	116.8
5.0	121.0	125.8	130.6	135.4	140.2	145.0	150.6	156.2	161.8	167.4
6.0	173.0	178.6	184.2	189.8	195.4	201.0	207.8	214.6	221.4	228.2
7.0	235.0	243.2	251.4	259.6	267.8	276.0	284.8	293.6	302.4	311.2
8.0	320.0	330.4	340.8	351.2	361.6	372.0	382.6	393.2	403.8	414.4
9.0	425.0	435.0	445.0	455.0	465.0	475.0	485.0	495.0	505.0	515.0
10.0	525.0	535.0	545.0	555.0	565.0	575.0	586.0	597.0	608.0	619.0
11.0	630.0									

Table No.IV. (8).3

G and D Ready Reckoner - Mantralayam For - 1983

Zero of Gauge - 306. 000 Meters

Discharges in 1000 cusecs.

Gauge in Meters.	*0.0*	*0.1*	*0.2*	*0.3*	*0.4*	*0.5*	*0.6*	*0.7*	*0.8*	*0.9*
1.0	0.9	1.7	2.4	3.2	4.0	4.8	6.2	7.7	9.1	10.6
2.0	12.0	14.0	16.0	17.9	19.9	21.9	24.5	27.1	29.7	32.3
3.0	34.9	37.7	40.5	43.2	46.0	48.7	52.3	55.9	59.5	63.2
4.0	66.7	70.4	74.1	77.7	81.4	85.1	89.5	93.8	89.2	102.6
5.0	107.0	111.5	116.1	120.7	125.3	129.9	134.4	138.8	143.2	147.7
6.0	152.1	157.7	163.3	168.9	174.5	180.0				

3 Hourly Gauge Data

Data	Name of Site/ Station.	3 Hourly Gauge Data							
		03.00	06.00	09.00	12.00	15.00	18.00	21.00	24.00
15.8.86	Mantralayam	2.900	3.510	4.170	4.480	4.650	4.700	4.700	4.700
	Deosugur.	7.620	7.600	7.580	7.540	7.500	7.460	7.430	7.400
16.8.86	Mantralayam.	4,660	4.595	4.430	4.300	4.190	4.140	4.080	4.040
	Deosugur.	7.370	7.320	7.300	7.270	7.260	7.240	7.230	7.240

APPENDIX V

CENTRAL WATER COMMISSION FLOOD FORECASTING MODEL 1 "CWCFF1"

Model Formulation

The basic concept of the CWCFF1 (Central Water Commission Flood Forecasting 1) Model is based on the fact that fluid tends to maintain uniform level under free condition. When fluids of two different chambers having different levels are allowed to mix freely, their new levels, after a certain time interval, will depend on their levels at the beginning of the time interval and the time lapse.

In this model a river channel is visualised to be comprising of a number of parallel slices. During flow of water through these slices various processes take place simulatenously between two adjoining slices, and major processes are identified as follows:

1. Water flows from higher elevation to lower elevation.

2. Some of the water is absorbed and retained in the dry river banks.

3. Some of the water gets evaporated.

4. Some of the water spreads away from the river bank and returns to the main stream when the river stage falls.

5. Some of the spilled water crossed the bank and is held up in pits and low lying areas and never returns to the main stream.

6. At some places water takes separate course and joins the river in the down stream.

The above processes are dependent on the following physical properties:

(a) Process 1 depends on the difference of levels of water in the two adjoining slices and the slope of the river bed.

(b) Process 2 depends on the dryness of the river banks, and the dryness of river banks depends on the previous maximum water level and the lapse of time since its occurance.

(c) Process 3 depends on the dryness of the air and the air temperature.

(d) Processes 4,5 and 6 depend on the slope of the river, bank conditions and its characteristics.

In this model processes 1,4,5 and 6 are combined together as characteristics of a river channel at a particular location remain almost unchanged.

Slicing of the river channel is shown in Fig. V.1 e.g. slice A A_1 B_1 B, slice B B_1 C_1 C, slice C C_1 D_1 D etc. Two consecutive slices A A_1 B_1 B and B B_1 C_1 C are taken out and their end elevation is shown in Fig V.2 with river bed horizontal. The vertical planes, A A1, B-B_1 and C-C_1 are assumed to be sealed at the beginning of the time period. Now the vertical plane B-B_1 is removed and the two reservoirs are allowed to balance during period 't' (Fig. V.2) . If the initial volume of waters in the two reservoirs at the beginning of the period are Q1 and Q_2 respectively, after belancing, volume of water in each reservoir will be equal to $(Q_1 + Q_2)/2$. Now the system of the two reservoirs is given a tilt equal to slope of the river bed. This operation is done along with the removal of the vertical and continued for time period 't'. A volume of water equal to $\frac{(Q_1 + Q_2)}{2} \times R_n$ will flow from slice-1 to slice-2 (here R_n is a slope factor). At the end of time period 't' the vertical partition B-B_1 is put back. This balancing process continues in series starting from beginning of the river channel to the end of the river channel.

In the procedure explained above, for a fixed time period 't' the new volumes of water in each chamber can be computed and can be continued for the entire period of computation 'T' . The volume of water passing from one slice to the other is a function of the average of the initial volumes in the two slices and a slope factor R_n, which represents the river characteristics.

For process no. 2 record of previous maximum volume for each slice is stored in record and whenever the new volume exceeds the previous volume, the loss of water due to absorption in dry banks is obtained as under:

$$V_f = V_i - (V_i - B) \times Z_n$$

where V_f is the volume after absorption.
V_i is the initial volume

B is previous maximum recorded volume and Z_n is the factor for water absorption. This will be executed during rising trend only.

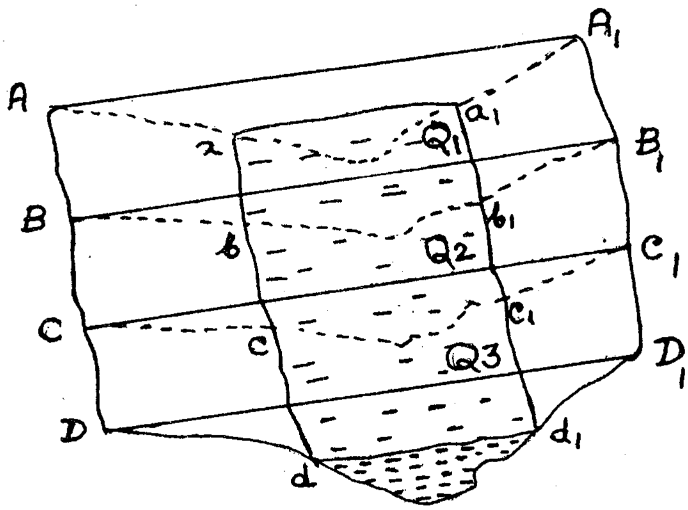


FIG. V. 1

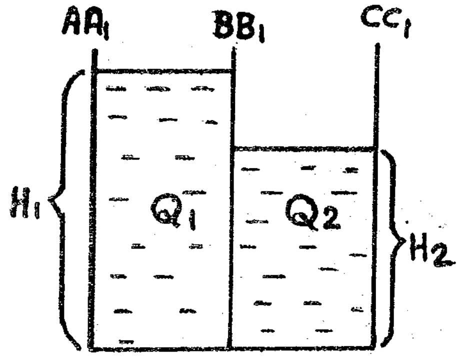


FIG. V. 2

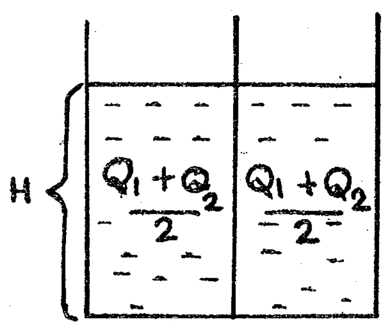


FIG. V. 3

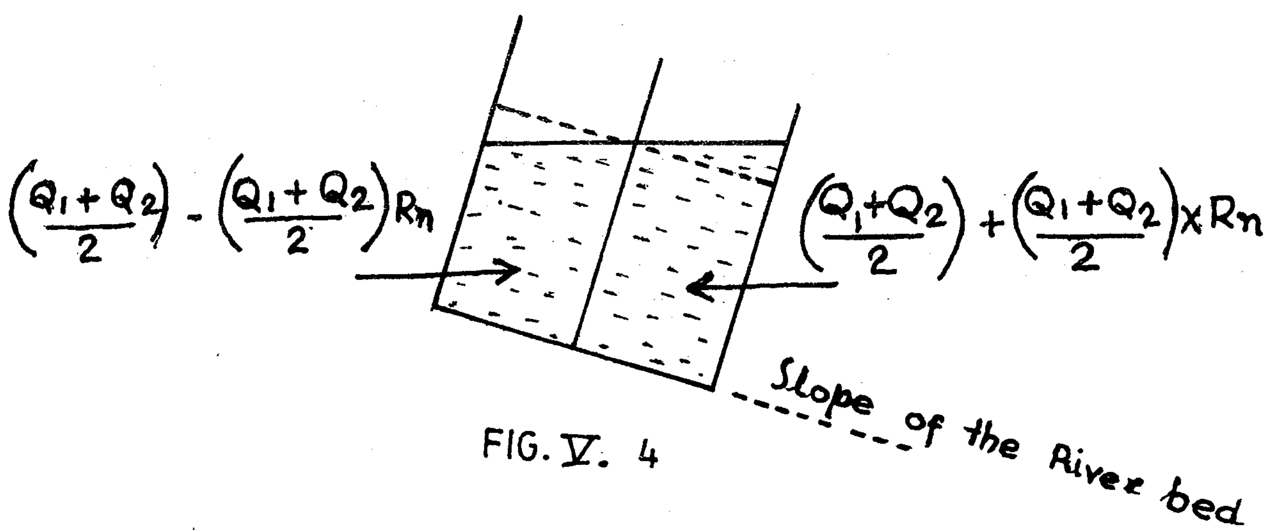


FIG. V. 4

When V_i does not exceed B the volume of B is depleted by an amount equal to $B \times E_f$ where E_f is an evaporation factor.

Parameters Involved

Parameters involved in CWCFF1 Model are following:

R_n; A Routing co-efficient, which indicates what fraction of the average volumes of the two adjacent slices will be transferred from one slice to the other.

Z_n; Is the factor for absorption in the dry banks. It indicates what fraction of the total volume of water, during rising of water level, will be absorbed in the dry banks of the river.

E_f; It is a co-efficient for reduction of the water in the banks of the river during recession. This much fraction of the total previous volume is decreased in every computation period.

NSTP; It indicates how many computations to be done in one hour i.e. computation period for one operation is (60/NSTP) minutes.

NC; Number of slices along the river channel.

Model Calibration

In the Yamuna catchment above Delhi, the CWCFF1 Model is being tried for river reach routing in the reach between Kalanaur to Delhi. The length of the reach is about 196 km. with a catchment area of 6630 sq. km. along the river. The model parameters are calibrated by trial and error method. To start with, the width of each slice was taken up as 1 km. The final parameters found out are stated below:

Total No. of slices = 200

NSTP, no of computation steps in one hr. = 12

$E_f = 0.00002$

Values of R_n and Z_n are found to vary with average water content of the two compartments.

Average water content	R_n *for first 100 slices*	R_n *for next 100 slices*	Z_n
00000.0	0.0001	0.0001	0.001
00025.0	0.0150	0.0150	0.001

Average water content	R_n *for first 100 slices*	R_n *for next 100 slices*	Z_n
00050.0	0.0900	0.0700	0.001
00100.0	0.1100	0.0900	0.001
00200.0	0.1200	0.1000	0.001
00400.0	0.1300	0.1100	0.002
00500.0	0.1560	0.1360	0.050
01000.0	0.1820	0.1565	0.080
01500.0	0.1852	0.1570	0.090
02000.0	0.1875	0.15550	0.095
02500.0	0.1865	0.1600	0.100
03000.0	0.1720	0.1620	0.120
03500.0	0.1640	0.1620	0.125
04000.0	0.1450	0.1420	0.130
05000.0	0.1360	0.1360	0.130
08000.0	0.1300	0.1300	0.130
11000.0	0.1300	0.1300	0.130
20000.0	0.1300	0.1300	0.130

As a thin layer of water will adhere to the river channel, there will be no transfer of water from one slice to the other if the discharge limit is not exceeded for the number of steps (NSTP) as indicated below:

No. of steps per hr. (NSTP)	*Discharge limits*
12	1200
11	1000
10	950
9	850
8	600
7	200
6	0

Effect of Parameters

While calibrating the model the following effects of different parameters are being observed:

1. When number of slices is reduced i.e. the width of the slice is increased, the simulated peak is formed earlier and the rising limb becomes steeper. When number of slices is increased, the peak forms later with slightly reduced slope (Ref. Fig. V.9. and Fig. V.10)

2. When NSTP is reduced the peak is delayed with lower value and when NSTP is increased the peak is formed earlier with higher magnitude.

   (Ref. Fig. V.7 and Fig. V.8)

3. When Z_n value is reduced, excess volume of water is available in simulated peak. (Fig.V.6)

4. When R_n value is increased, the peak reaches faster with reduced value.

   (Ref. Fig.V.5)

Future Developments

CWCFF1 is not only a river channel routing model but the methodolgy can be extended to work as a complete water-shed model, and is described below:

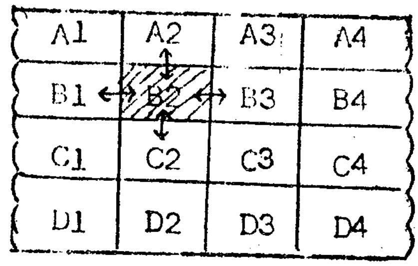


FIG.A

A river catchment is to be divided into a grid of equal areas. Starting from one corner of top of the catchment the water contents of each square compartment will have to be balanced considering adjacent four compartments as shown in Fig.-A. This procedure will have to be continued upto the end of the river channel for each computation period. While applying this model to a catchment, to use it as a complete catchment model, the following procedure is to be followed:

1. Elevation of centre of each square grid is to be found out from topographic map.

2. Boundary conditions for each square are to be identified.

3. Proper separation of surface runoff and base flow it to be done, and both types of flows are to be routed separately upto river channel.

4. For square grids containing river channel there will be no base flows and the base flows from the adjoining square grids will end up as surface flow in the river channel.

5. Rain fall over each grid is to be computed in a simplified way by clustering a group of square grids together, based on the topography of the catchment.

Summary and Conclusions

Most of the models work satisfactorily, for simulation and forecasting of flood, when there is a continuous high flow. But all the four models (as tried in Yamuna catchment) fail to simulate the river flows in the beginning of the flood season or when a high flood comes after a long recession. CWCFF1 Model is developed with a capability to adjust for the absorption losses in the river bed. A comparative hydrograph has been shown in Figure V.12 for simulation of two and half months flows. It is observed that the Model works satisfactorily during both low flows and high flows.

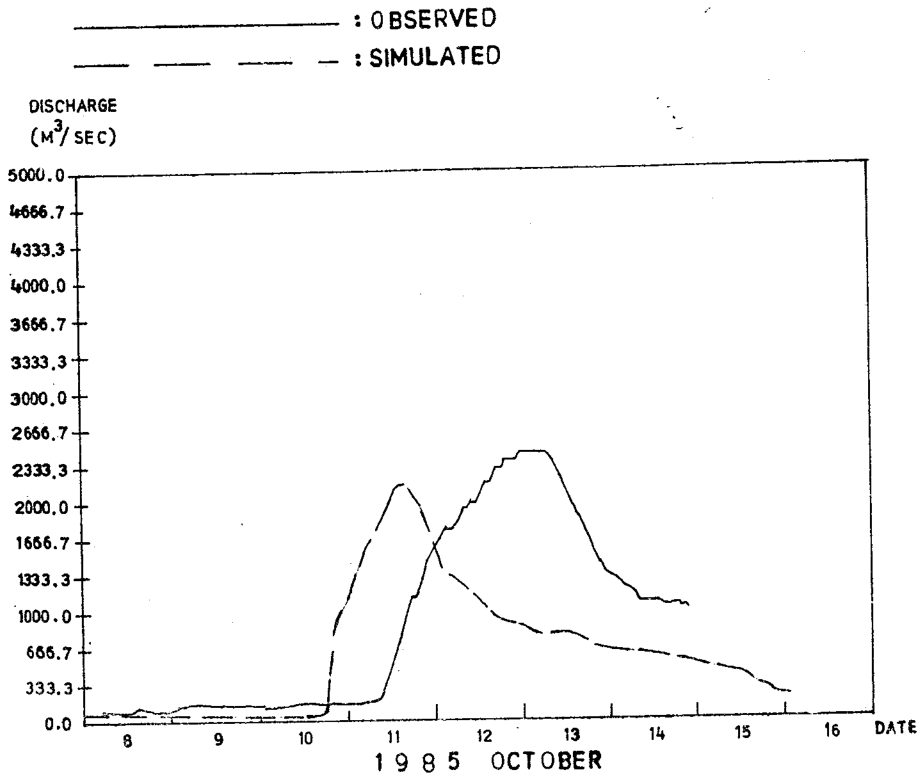


FIG. V 5 - R_n INCREASED BY 0·3

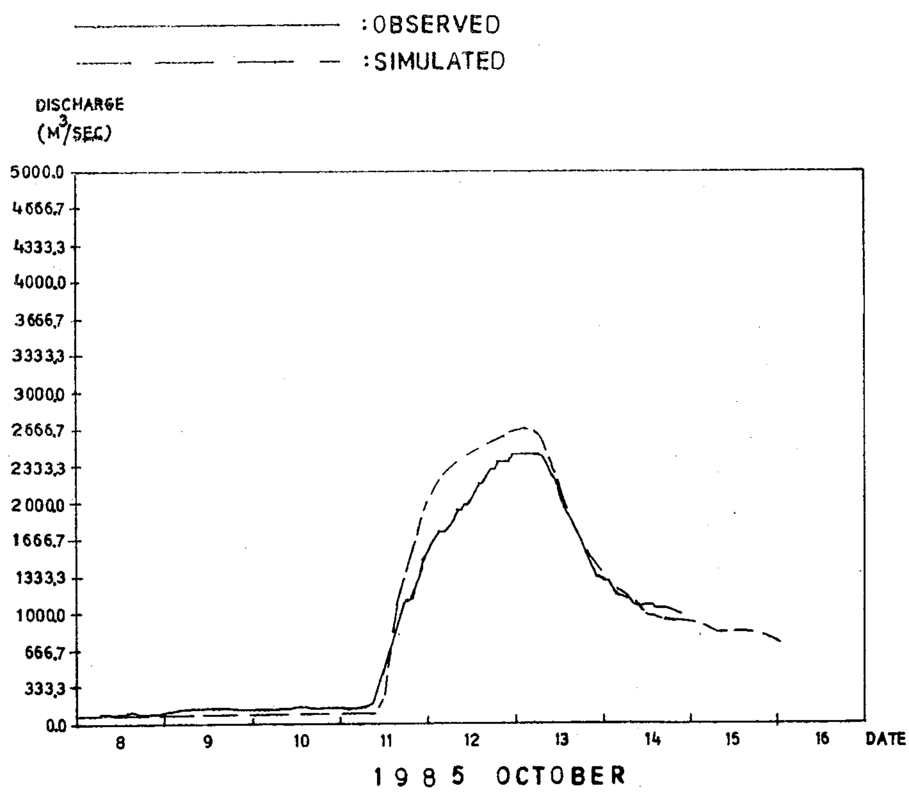


FIG. V. 6 - Z_n REDUCED TO 0·001

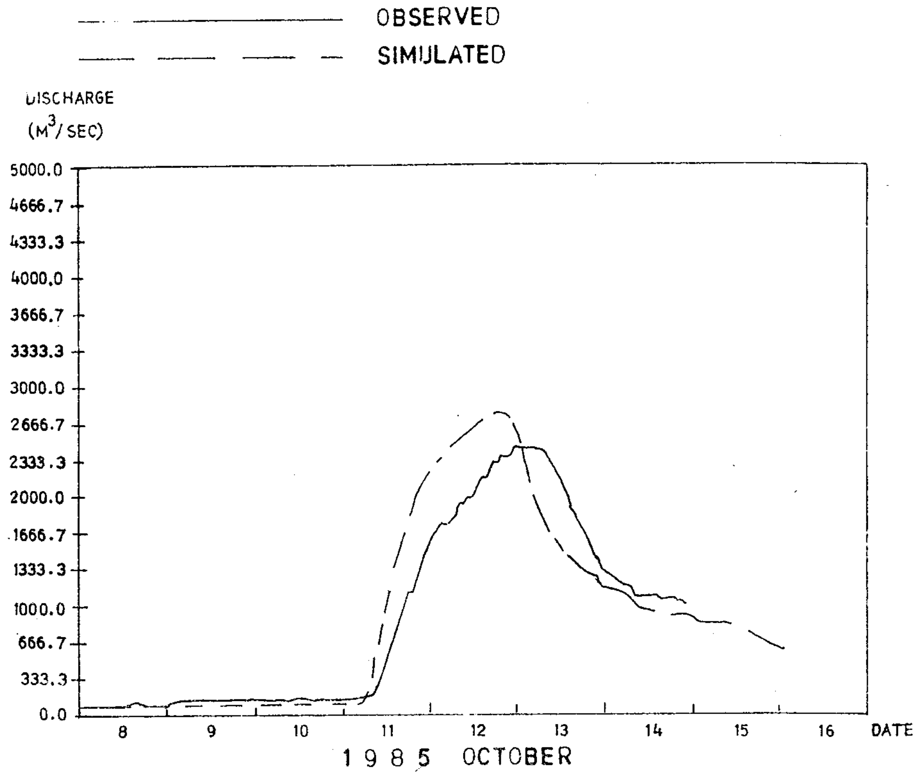


FIG. V. 7-NSTP 16

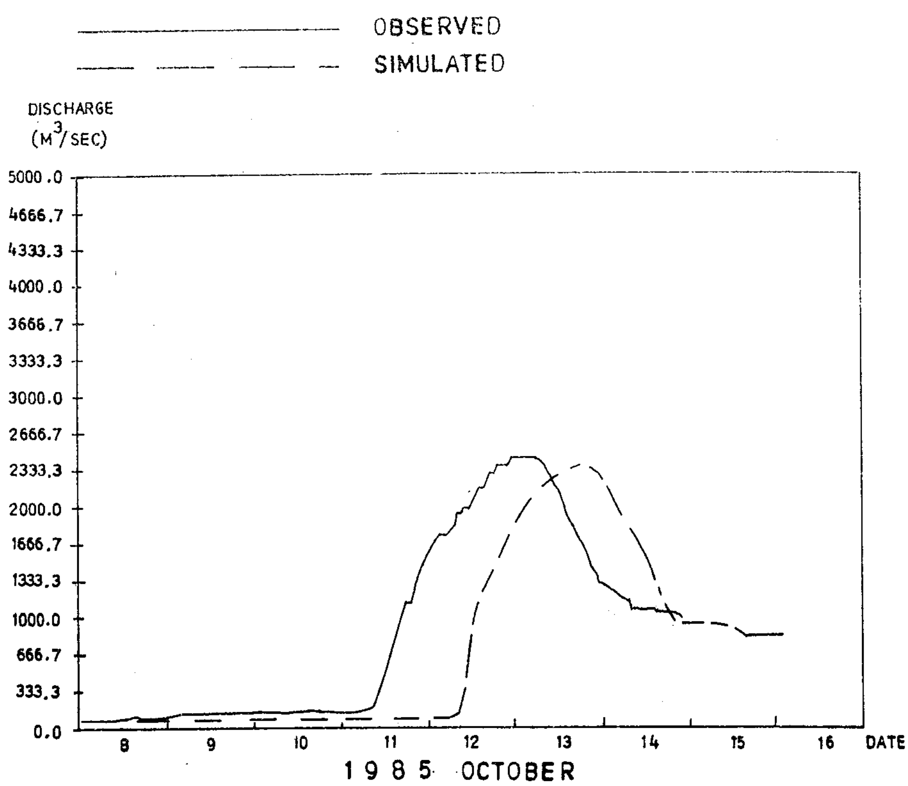


FIG. V. 8-NSTP 8

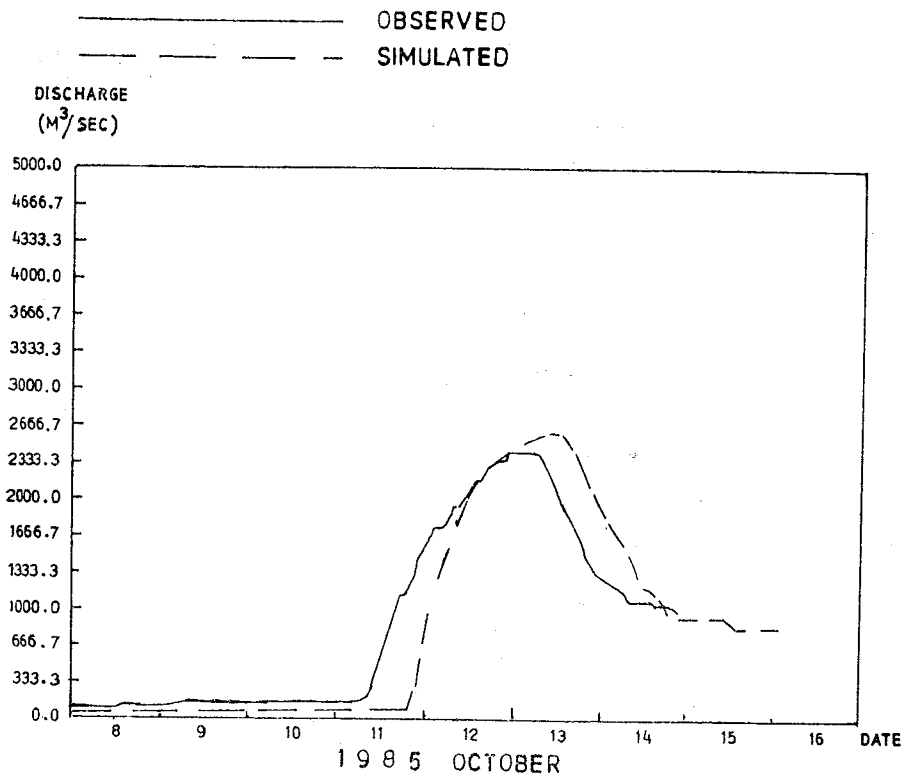


FIG. V. 9-NS 250

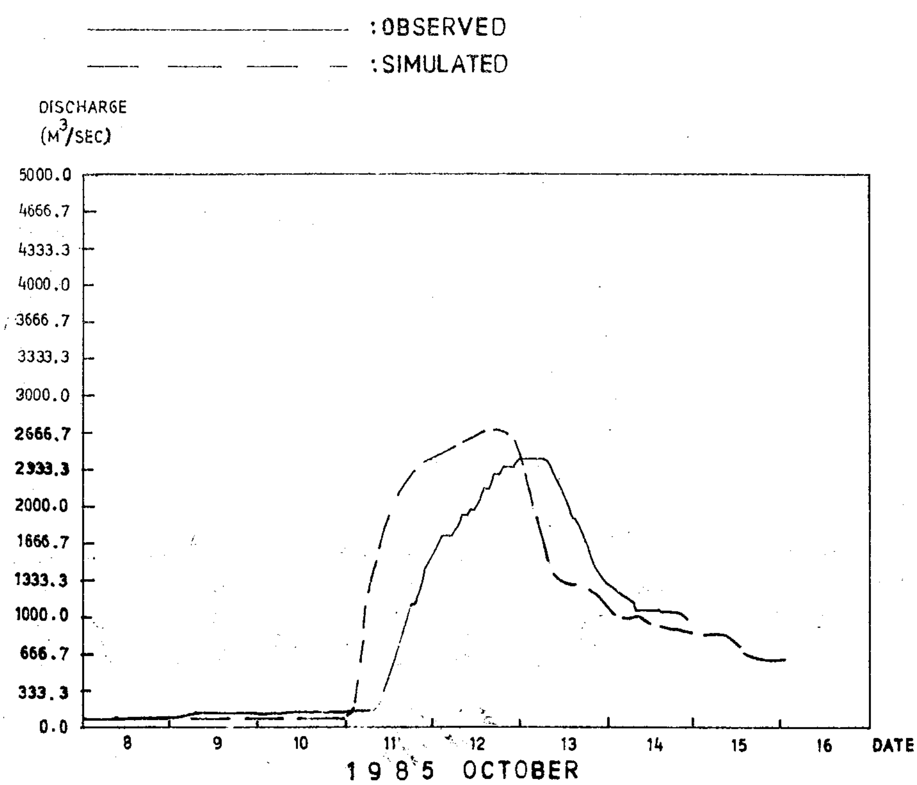


FIG. V. 10-NS 150

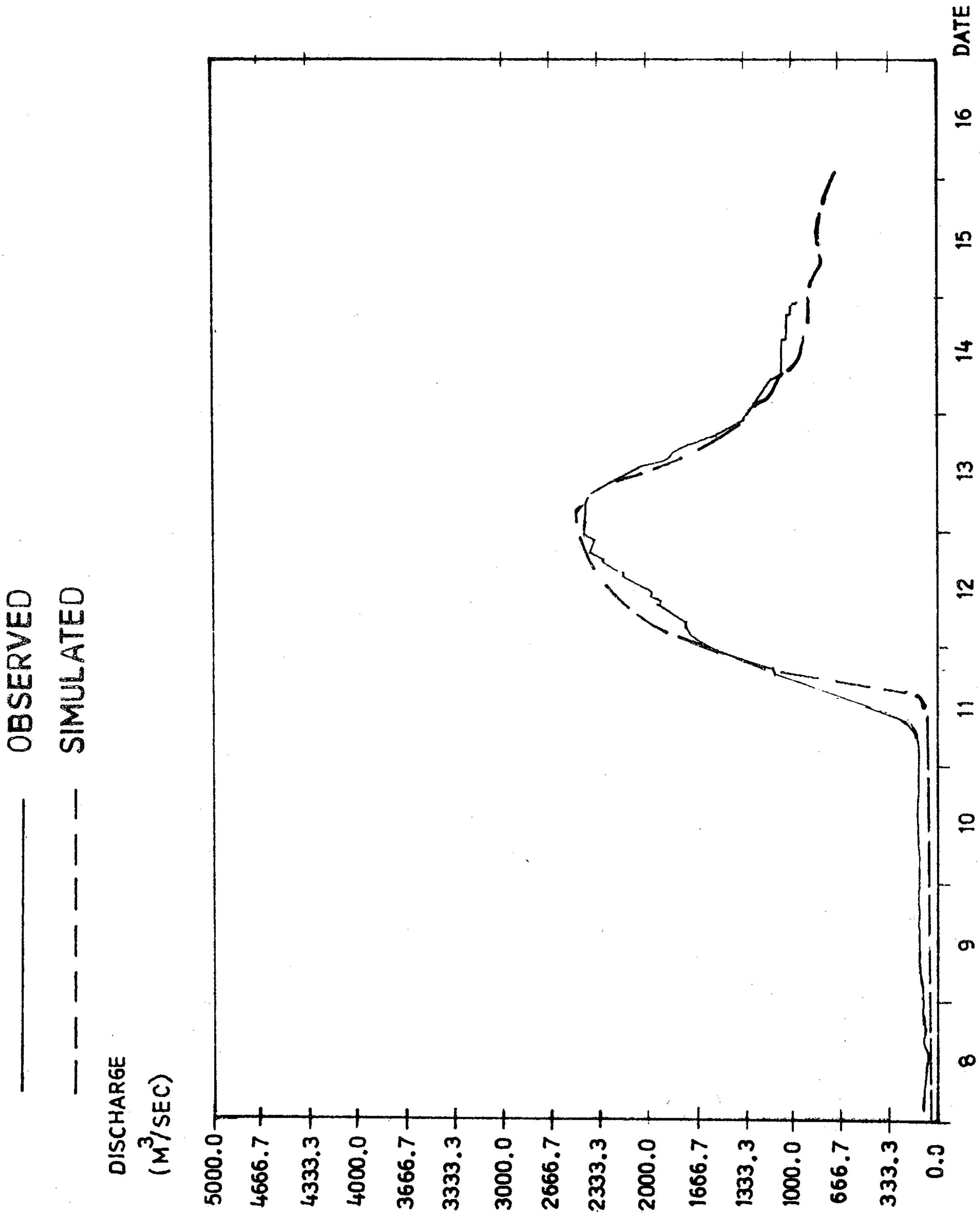


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FIG. V.11-NSTP 12

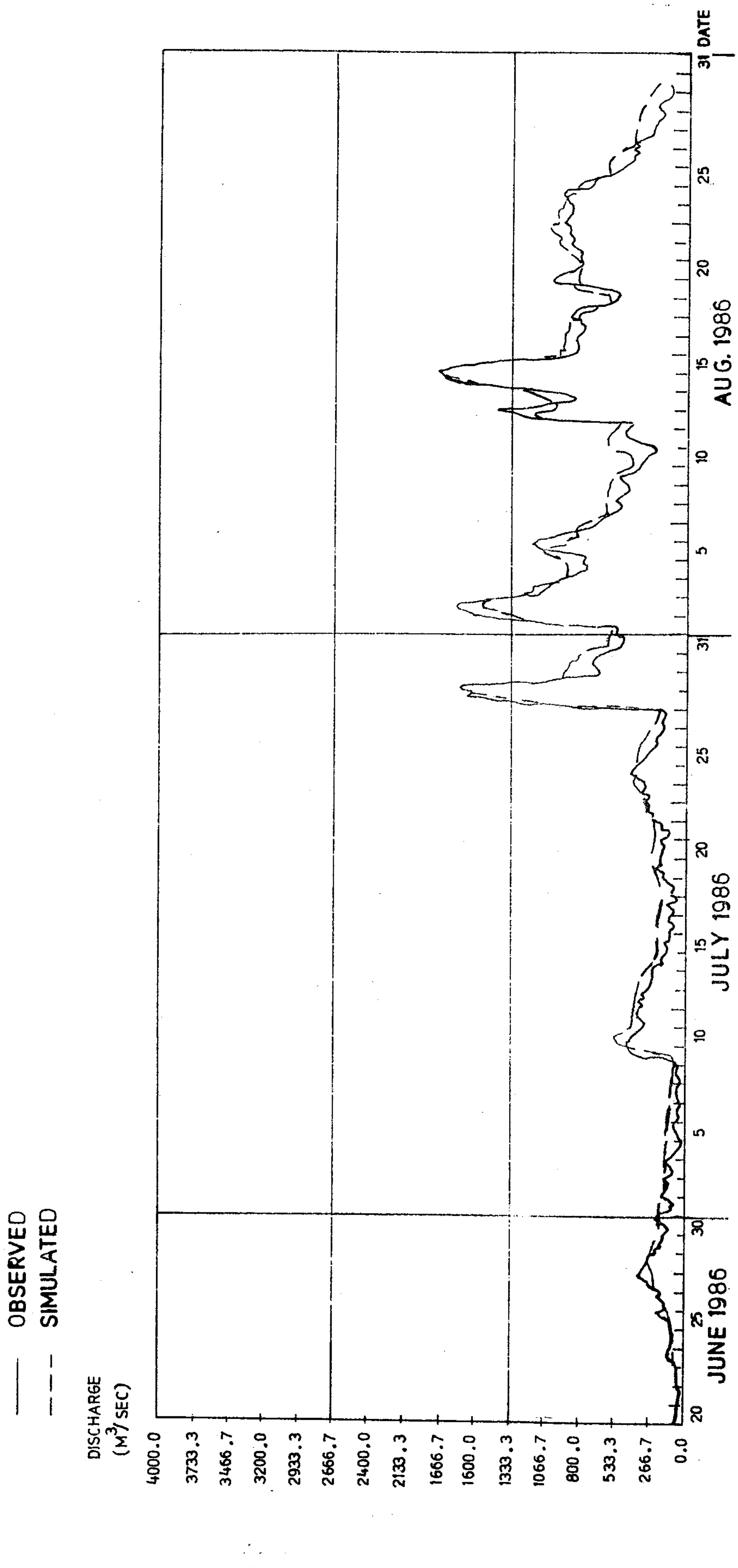

FIG. V. 12 - FINAL PARAMETERS

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