MODELING AND MANAGEMENT OF FLOOD FLOWS

A DISSERTATION

Submitted in partial fulfilment of the requirements for the award of the degree
of
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in
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By

SALAH EL-DIN MOHAMMED ABUAGLA

DEPARTMENT OF HYDROLOGY
INDIAN INSTITUTE OF TECHNOLOGY ROORKEE
ROORKEE -247 667 (INDIA)
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CANDIDATES DECLARATION

I hereby certify that the work which is being presented in this dissertation entitled **MODELING AND MANAGEMENT OF FLOOD FLOWS** in partial fulfillment of the requirements for the award of the degree of Master of Technology in Hydrology, and submitted in the Department of Hydrology, Indian Institute of Technology Roorkee, Roorkee is an authentic record of my own work carried out during a period from July 2007 to June 2008 under the supervision of Dr. B. S. Mathur, Professor (Retd.) and Emeritus Fellow, Department of Hydrology, Indian Institute of Technology Roorkee, Roorkee.

The matter presented in this thesis has not been submitted by me for the award of any other degree of this or any other Institute.

Date: 27.06.2008

(Salah El-Din Mohammed Abuagla)
Trainee Officer's Signature

This is to certify that the above statement made by the candidate is correct to the best of our knowledge.

(B.S.Mathur)
Professor (Retd.) and Emeritus Fellow
Department of Hydrology
Indian Institute of Technology Roorkee
Roorkee

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SYNOPSIS

In most developing countries of the tropics tremendous growth in population has been recorded during the last four to five decades. This has resulted into increased water demand in the domestic, agricultural, commercial and industrial sectors. With limited water available during the four monsoon months, it has become necessary that each unit of water be either judiciously consumed or else be stored. In majority cases, this water demand is met through the ground water supplies. Consequently, lowering of ground water table has become a common phenomenon which is observed throughout. Suitable plans need be drawn for natural recharge of groundwater table by storing the excess flood waters in the surface depressions which may not be economically viable. Also, there is a need to explore where ever possible, the river bed itself be utilized for providing a safe passage to floods as well as to utilize it for storage of water for recharging the aquifers. With these objectives in view this dissertation has been worked out.

The objectives of the study have been defined along with the introductory remarks (Chapter 1). The literature survey relevant the problem pertaining to the hydrologic design and models to be adopted has been reported in Chapter 2 of the thesis. Since most of the catchments of rivers in developing countries happen to be ungauged, therefore, a hydrologic model utilizing the properties of the Rational Formula and the Manning's equation has been developed. This is resulted into the development of a nomograph involving the runoff coefficient, rainfall excess intensity, discharge per unit catchment area per unit channel width, Manning's roughness, runoff depth in channel, velocity of flow in channel, Froude's number and the time of concentration. The proposed concepts have been discussed in detail in Chapter 3.

A case study has been reported in Chapter 4. A Catchment area of 3200 km^2 has been considered. Flood produced is allowed to pass through a constricted section of the river. The levees on the two banks have been strengthen to form the dykes. The flood in excess of safe discharge has been diverted through the broad crested weir sections to fill the depression on the two sides of the river. The proposed management plan saves the two economic sectors located in the downstream. The diverted water in the depressions will infiltrate, percolate and will ultimately recharge the aquifers.

The proposed concepts have been applied on to the Solani river catchment in the Uttarakhand state of India with its outlet located at the site of existing aqueduct which carry water of Upper Ganga canal. The total catchment area is 547 km^2. The rainfall data of the hydro-meteorological observatory of the department of hydrology, Indian Institute of Technology Roorkee had already been analyzed in previous studies. The 100 years return period rainfall of 2 hours duration has been adopted for design flood computations. In order to compute the design flood, the 2 hour design unit hydrograph (UH) has been worked out. This design unit hydrograph has been obtained by increasing the peak ordinate of the average synthetic unit hydrograph by 50 percent. The average unit hydrograph has been computed by averaging three synthetic unit hydrographs one UH has been computed using the recommended approach of the Central Water Commission of India for this hydro-meteorological subzone. The other two UHs were computed by using the Snyder's method as well as the hydrologic model proposed by Clark. It is proposed to tame the flood waters corresponding to the worked out design flood for the above mentioned design rainfall. The two pairs of central dykes divide the river span into one central section to allow the floods to pass through the Solani river and the two parallel auxiliary channel sections each of 50 m width and 6 km long for storing water. Some treated waste water will also come to the auxiliary channel sections which are provided with gates at the downstream. From the central section of the river, flood waters will be allowed to enter through the gated sections of broad crested weirs. The gates provided in the broad crested weir sections may be opened for a design period of 4 hours for filling the auxiliary recharging storages with a head of 1.0 m of water. This water will be stored from the occasional floods which are received in the river after the flushing the waste waters coming to the sections. It is proposed that 0.6 MCM of water be stored in the two auxiliary sections of the river for recharging the ground water regime on which the entire water supply of Roorkee depends.

The last Chapter deals with the conclusions. Which emphasizes that such schemes need be properly investigated from all environmental considerations to make them eco-friendly.

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LIST OF SYMBOLS

Symbol	Description
i	Rainfall of intensity
t_c	Time of concentration
A, A_d	Watershed area, drainage area
Q	Peak discharge
C	Runoff coefficient
t_0	Inlet time, i.e. time of travel on overland
t_f	Flow time, i.e. time of travel in channel system
L	Length of channel/ditch from headwater to outlet
S	Average watershed slope
H	Elevation difference between divide and outlet
c	Retardance coefficient
n	Manning roughness coefficient
CN	SCS runoff curve number
v	Average velocity in feet per second from Fig. 3-1 of TR 55 for various surfaces
A_c	Channel water area
R_c	Hydraulic radius
P_c	Channel effective perimeter
S_c	Channel slope
V	Velocity of water along the flow direction
b	Bottom width
h	Depth of water
z	Side slope of the channel ($z\mathrm{H}:1\mathrm{V}$)
d_0	Diameter
Q_n	Direct runoff ordinate
P_m	Excess rainfall depth
U_{n-m+1}	Unit hydrograph ordinate
M	Pulses of excess rainfall
N	Pulses of direct runoff
t_p	Lag time
L	Basin length measured along the main channel from the basin divide to the outlet
$\bar{L}$	Distance along the main channel from the gauging station to the point nearest to the centroid of the basin
C_t	Regional constant representing watershed slope and storage
t_r	Duration of effective rainfall (hours)
C_p	Regional constant

t_R	Non-standard rainfall duration (hour)
t_p'	Basin lag in hours for an effective duration of t_R hour
t_B	Time base
W_{50}	Width of unit hydrograph in hour at 50% peak discharge
W_{75}	Width of unit hydrograph in hour at 75% peak discharge
q	Peak discharge per unit catchment area in cumec/km2.
$g(t)$	Step response function
$h(t)$	Unit pulse response function
$A_1, A_2, A_3, \ldots\ldots, A_N$	Inter-isochrone areas
K	Storage time constant
x	Weighting factor
C_0, C_1, C_2	Muskingum coefficients
I	Translation hydrograph ordinates
h'	Maximum allowable water depth
q'	Discharge per unit channel width
q	Discharge passing through outlet per unit width of the channel and per unit drainage area
Fr	Froude's Number
h_{c_new}, h_{c_old}	Water depths used for iteration
$C_{\max}$	Maximum runoff coefficient value in nomograph
$i_{\max}$	Maximum rainfall excess intensity value in nomograph
$n_{\max}$	Maximum roughness value in nomograph
$Q_{\max}$	Maximum peak discharge value in nomograph
$h_{c\max}$	Maximum channel water depth value in nomograph
$V_{\max}$	Maximum average flow velocity value in nomograph
$t_{c\max}$	Maximum time of concentration value in nomograph
$Fr_{\max}$	Maximum Froude's Number value in nomograph
$q_{\max}$	Maximum discharge passing through outlet per unit width of the channel and per unit drainage area used in nomograph
P	Levee height
h_o	Excess river stages more than the levee height
L_w	Levee length
L_b	Levee width
$d_1, d_2, d_3, \ldots, d_m$	Successive contour intervals
Δd	Successive difference in contour intervals
ΔQ	Discharge through the weir per unit length under the head h_0 above the weir
T	Allowable time is to fill the storage S
T_m	Time to peak in SUH

$T_{50}, TT_{50}, T_{75}, TT_{75}$	Time widths used in SUH
d	Runoff depth
h_L	Head loss
y_c	Critical depth
V_c	critical flow velocity
C_d	Coefficient, accounts for the head loss
C_v	Coefficient, accounts for the approach velocity head
k_w	Broad-crested weir discharge coefficient

LIST OF ABBREVIATIONS

Symbol	Description
SWMM	Storm Water Management Model
SCS	Soil Conservation Service
TR 55	Technical Release 55
GIS	Geographical Information System
HEC	Hydrologic Engineering Center
HMS	Hydrologic Modeling System
TR-20	Technical Release 20, computer program for the simulation of runoff
ILLUDAS	Illinois Urban Drainage Area Simulator
STORM	Storage Treatment and Overflow Runoff Model
SSARR	Stream-flow Synthesis And Reservoir Regulation model
RAS	River Analysis System
DRH	Duration of Direct Runoff
UH	Unit Hydrograph
SUH	Synthetic Unit Hydrograph
CWC	Central Water Commission of India
IMD	India Meteorological Department
FORTRAN	**For**mula **Tran**slation System
TUH	Triangular Unit Hydrograph
TH	Translation Hydrograph

CHAPTER 1 INTRODUCTION: OBJECTIVE AND SCOPE

In the hydrologic designs for flood estimation and its control, the efforts are to mitigate the adverse effects of high flows causing floods. In a high flood, river flows tend to overtop either a natural or an artificial embankment along the stream. The magnitudes of floods are described in terms of flood discharges, river stages, and flood volumes. Each of these factors plays an important role in the hydrologic design of different flow control measures.

Hydrologic design for water use is concerned with the development of water resources to meet human needs and with the conservation of the natural life in water environments. As the population and economic activities increase, so increases the demand for use of water for its various uses. But these must be balanced against the limited supply of water provided by the nature and the desire to maintain healthy plant and animal life in rivers, lakes, and estuaries etc. Hydrologic information plays a vital role in managing a balance between supply and demand for water, and also in planning water resource development projects as a whole. In contrast to hydrologic design for water control which is concerned with mitigating the adverse effects of high flows, hydrologic design for other uses water use is directed at utilizing average flows and also in mitigating the effects of low flows.

In this context, it becomes essential to apply appropriate methodologies for the estimation of flows. Particularly in tropical countries the situation is much more diverse in nature. The low flows continue over a pretty long time of nearly 8 to 9 months and rainy season which is the wet period in the hydrologic cycle restricts itself only 3 to 4 month. In many countries of tropics the wet season is concentrated during the monsoon periods. Similar condition prevails in Sudan as in India. During the dry period when lean flows are observed, agricultural activities along with shelter accommodation of farmers do extend into the river bed. This is mainly due to easy availability of water in shallow wells, temporarily constructed in the river bed, which serve the purpose of drinking water as well as for irrigating crops. As the monsoon season approaches around mid June, river flows suddenly increase and the inhabitant and the agriculturists are often caught unawares. Increasing stages of the river tend to cover the entire river bed and as water levels go further up, the stages register high flows and water tends to overtop levees to enter the flood plain. Such conditions may prevail for shorter durations as well as over longer periods covering more and more areas in the flood plain as the monsoon proceeds and intensifies.

Keeping the above in view, the present study has been undertaken. This study concentrates on the river flows leading to higher river stages and ultimately engulfing the flood plains. Suitable methodologies, which need to be simple enough, to be easily understood by field engineers and technical persons, are aimed to be devised. Also, suitable measures of river training works are proposed to this studies which may lead to proper management practices for issuing warnings and making people aware to avoid damages in terms of lives, of livestocks and crops etc. Accordingly, the following objectives have been set for the present study.

1.1 OBJECTIVES

Existing hydrologic computational procedures and models need be studied to have proper evaluation as well as to determine the suitability of their applications under the present situation. In Sudan, in majority cases rainfall data is available in different regions more or less continuously for longer duration. On the other hand, the river flow data at various points of the river system is not available. Therefore, methodology to be recommended must be suitable for applying on to ungauge catchments.

It is proposed to use the rational formula commonly used in hydrology as it involves only the drainage area and the rainfall. This is to be used in combination with Manning's relationships which accounts for the flow parameters of river stages, flow velocities, roughness conditions and slopes.

It is proposed to evolve relationships among these parameters and finally to incorporate them in the form of a nomograph for easy and handy computation of river stages.

Severe rainfall conditions in the form of major severe storms as well as of storm rainfalls of high return periods have been used to have an estimation of river flows which threaten to enter into the flood plains.

Corresponding to different stages, flood control measures are to be described to deal with floods. Further, once the flows overtops the bank and enter the river plain it becomes necessary to map the flood effects.

Keeping the above objective in view the following approach has been undertaken to carryout the study.

1.2 THE APPROACH

In spite of best efforts, the data from upper parts of Sudan could not be procured. Since climatologically the conditions are much similar to the conditions prevailing in the lower parts of the sub-mountainous region of India, the basin of the Solani river has been taken and its data is used to develop the model. It is anticipated that the study done on the Solani river basin may possibly be reemployed in Sudan.

1.3 PLANNING OF THE DISSERTATION REPORT

As discussed above, in Chapter 1, the problem has been introduced and the objectives of this study are set. The Chapter 2 deals with review of literature of methodologies relevant to the objectives set in. Chapter 3 discusses the model development. Mathematical formulations using the rational methods and the Manning's formula are used to study and developed relationships between the flow parameters and also to develop a nomograph for the same. For different magnitudes of floods, a strategic river training work is proposed and under overtopping conditions for the calculated risk the flood mapping has been discussed. Synthetic Unit Hydrograph (SUH) and Instantaneous Unit Hydrograph (IUH) approaches are used to estimate the flood peak and its stage at the outlet. A case study has also been taken up to see the working of the proposed model (Chapter 4). In Chapter 5, the proposed model has been applied onto Solani river catchment with its outlet at Roorkee. Detailed geographic analyses have been carried out using the ArcGIS software in geographical information system environment. The estimated parameters are used to compute the unit hydrograph by following the procedure suggested firstly, by the Central Water Commission (CWC) of India and secondly, by following the Snyder's approaches. Finally a 2 hour unit hydrograph has been recommended for Solani at its outlet. The unit hydrograph thus obtained is crosschecked by Clark's Method and result is found quite acceptable. The rainfall data registered at Roorkee has been analyzed by earlier researches (Gautam, 1997; Shahoei, 1997). A few cases of severe most storms registered in Roorkee and also the rainfall depths of different return periods have been utilized to compute the flood hydrographs. By adopting suitable river management technique for the calculated risk the flood mapping in the vicinity of Roorkee has been carried out.

The last chapter (Chapter 6) deals with the conclusions. Where entire work has been summarized and suitable conclusions have been drawn.

CHAPTER 2 LITERATURE REVIEW

2.1 INTRODUCTION

This dissertation deals with river flows, flooding of the flood plains when river overtops the natural levees under different stages of flooding. Accordingly in this chapter, some of the concepts, terminologies, basic equations of river flow measurement and their theories with their solution techniques have been discussed. Some of currently used watershed models along with their important features and applications have also been discussed along with classic concepts e.g. the Rational formula, Unit hydrograph theory and Manning's concept of the flow of water.

2.2 DESIGN FLOWS

Hydrologic design for water control concentrates on mitigating the adverse effects of high flows or floods. A severe flood is any high river stage flow that overtops either natural or artificial embankments forming the river banks. The magnitudes of floods are described by flood discharge, flood elevation, and flood volume. Each of these factors is important in the hydrologic design of different types of flow control structures. A major portion of this chapter thus, deals with design flows or design flood computations for flow regulation structures (detention basins, flood control reservoirs, etc.) and flow conveyance structures (storm sewers; drainage channels, flood levees, diversion structures, etc.). The purpose of flow regulation structures is to smooth out peak discharges regulating the parameters controlling peak discharges, thereby decreasing downstream flood elevation peaks, and the purpose of flow conveyance structures is to safely convey the flow to downstream points where the adverse effects of flows are controlled or are minimal. This chapter discusses methods and models that are in vogue for hydrologic design of flow control structures.

One view of the typical drainage system can be considered as consisting of two major types of elements: location elements and transfer elements. Location elements are the places where the water stops and undergoes changes as a result of humanly controlled processes, for example, water storage, water treatment, water use, and wastewater treatment. Transfer elements connect the location elements; these elements include channels, pipelines, storm sewers, sanitary sewers, and streets. The system is fed by rainfall, influent water from various sources, and imported water in the pipes or channels.

The receiving water body can be a river, or a large water body.

Models can be used as tools for planning and management. The determination of the runoff volume and peak discharge rate are important issues in urban water management, and methods for calculating these variables range from the well-known rational formula to advanced computer simulation models such as the Storm Water Management Model (SWMM) (Huber, et al., 1975).

2.2.1 Design Philosophy

Drainage design can be divided into two aspects: runoff prediction and system design. In recent years, rainfall-runoff modeling for urban watersheds has been a popular activity and a variety of such rainfall-runoff models are now available, as described by American Society of Civil Engineers (1960), Chow and Yen (1977), Heeps and Mein (1974), Brandstetter (1976), McPherson (1975), Colyer and Pethick (1977), Yen (1978), and Kibler (1982). Computer models are described briefly in Section 2.4. Many models, in one form or the other, use the concept of rational formula which is used next. Bennis, S. and Crobeddu, E. (2007) proposed a runoff simulation model based on the improvement of the rational hydrograph method.

2.2.2 Rational Method

The rational method, which can be traced back to the mid-nineteenth century, is still probably one of the most widely used methods for design of storm water (Pilgrim, 1986; Linsley, 1986; Chow et al., 1988). Although valid criticisms have been raised about the adequacy of this method, it continues to be used for design because of its simplicity. In urban hydrology it is still used extensively particularly where detailed hydrologic data is not available.

The idea behind the rational method is that if a rainfall of intensity i begins instantaneously and continues indefinitely covering the entire area, the rate of runoff will increase until the time of concentration t_c is reached. When the entire watershed starts contributing to flow at the outlet the maximum ordinate is reached. The product of rainfall intensity i and watershed area A is the inflow rate for the system, iA, and the ratio of this rate to the rate of peak discharge Q (which occurs at time t_c) is termed the runoff coefficient C ($0 \le C \le 1$). Thus the rational formula is expressed as follows

$$Q = 0.2778 C i A \tag{2.1}$$

where, the maximum discharge Q is in cubic meter per second for a rainfall intensity i in mm per hour over the watershed area A which is in square km. The duration of rainfall intensity i in Equation (2.1) is equal or more than the time of-concentration of the watershed. If the drainage area consists of sub-areas or sub-catchments due to different surface characteristics or land use and soil covers, a composite analysis is required that must account for the various surface characteristics. If the areas of the sub-catchments are denoted by A_j and the runoff coefficients of each sub-catchment are denoted by C_j. The peak runoff is then computed using the following form of the rational formula:

$$Q = i \sum_{j=1}^{m} C_j A_j \tag{2.2}$$

where m is the number of sub-catchments drained by a drainage system.

The assumptions associated with the rational method are:

(i) The computed peak rate of runoff at the outlet point is a function of the average rainfall rate during the time of concentration, that is, the peak discharge does not result from a more intense storm of shorter duration, during which only a portion of the watershed is contributing to runoff at the outlet.

(ii) The time of concentration employed is the time for the runoff to establish till the flows from the most remote parts of the drainage area reach the outlet.

(iii) Rainfall intensity is constant throughout the storm duration.

2.2.3 Runoff Coefficient

The runoff coefficient C is the least precise variable of the rational method. Its use in the formula implies a fixed ratio of peak runoff rate to rainfall rate for the drainage basin, which in reality may not the case. Proper selection of the runoff coefficient requires judgment and experience on the part of the hydrologist. The proportion of the total rainfall that will reach the drainage channels depends on the percent imperviousness, slope, and ponding character of the surface. Impervious surfaces, such as asphalt pavements and roofs of buildings, will produce nearly 100 percent runoff after the surface has become thoroughly wet, regardless of the slope. Field inspection and aerial photographs are useful in estimating the nature of the surface within the drainage area.

The runoff coefficient is also dependent on the character and condition of the soil. The infiltration rate decreases as rainfall continues, and is also influenced by the antecedent moisture condition of the soil. Other factors influencing the runoff coefficient are rainfall intensity, proximity of the water table, degree of soil compaction, porosity of the subsoil, vegetation, ground slope, and depression storage. A reasonable coefficient must be chosen to represent the integrated effects of all these factors. Suggested coefficients for various surface types as used are given in Table 2.1 (Chow, 1964).

Table 2. 1 Runoff coefficients for use in the rational method

Type of drainage area	Runoff Coefficient, C
Lawns:	
Sandy soil, flat, 2%	0.05-0.10
Sandy soil, average, 2-7%	0.10-0.15
Sandy soil, sleep, 7%	0.15-0.20
Heavy soil, flat, 2%	0.13-0.17
Heavy soil, average, 2-7%	0.18-0.22
Heavy soil, sleep, 7%	0.25-0.35
Business:	
Downtown areas	0.70-0.95
Neighborhood areas	0.50-0.70
Residential:	
Single-family areas	0.30-0.50
Multi unites, detached	0.40-0.60
Multi unites, attached	0.60-0.75
Suburban	0.25-0.40
Apartment dwelling areas	0.50-0.70
Industrial:	
Light areas	0.50-0.80
Heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25
Palygrounds	0.20-0.35
Railroad yard areas	0.20-0.40
Unimproved areas	0.10-0.30
Streets:	
Asphaltic	0.70-0.95
Concrete	0.80-0.95
Brick	0.70-0.85
Drives and Walks	0.75-0.85
Roofs	0.75-0.95

2.2.4 Rainfall Intensity

The rainfall intensity i is the average rainfall rate in mm per hour for a particular drainage basin or sub-basin. The intensity is selected on the basis of the design rainfall duration. The design duration is equal to the time of concentration for the drainage area under consideration. In case of risk analysis, the return period of rainfall is considered as design period. The return period is established by design standards or chosen by the hydrologist as a design parameter.

2.2.5 Time of Concentration

Runoff is assumed to reach a peak at the time of concentration t_c when the entire watershed is contributing to flow at the outlet. The time of concentration is the time for a drop of water to flow from the remotest point in the watershed to the point of interest. A trial and error procedure can be used to determine, the critical time of concentration where there are several possible flow paths to consider. The time of concentration in a storm drainage system is the sum of the inlet time t_0 i.e. time of travel on overland and the flow time t_f i.e. time of travel in channel system; thus the time of concentration, t_c, is given as follows

$$t_c = t_0 + t_f \tag{2.3}$$

The flow time is given as

$$t_f = \sum_{i=1}^{n} \frac{L_i}{v_i} \tag{2.4}$$

Various relationships developed for the computation of time of concentration are given in Table 2.2.

Table 2. 2 Summery of time of concentration formulas

Method and Date	Formula for t_c (min)	Remarks
Kirpich (1940)	$t_c = 0.0078L^{0.77}S^{-0.385}$ $L =$ length of channel/ditch from headwater to outlet, ft $S =$ average watershed slope, ft/ft	Developed from Soil Conservation Service (SCS) data for seven rural basins in Tennessee with well-defined channel and steep slopes (3% to 10%); for overland flow on concrete or asphalt surfaces multiply t_c by 0.4; for concrete channels multiply by 0.2; no adjustments for overland flow on bare soil or flow in roadside ditches.
California Culverts Practice, Rowe, K. R. and Thomas R. L. (1942)	$t_c = 60(11.9L^3 / H)^{0.385}$ $L =$ length of longest watercourse, mi $H =$ elevation difference between divide and outlet, ft	Essentially the Kirpich formula; developed from small mountainous basins in California (U. S. Bureau of Reclamation, 1973, pp. 67-71).

(Continued)

Izzard (1946)	$t_c = \frac{41.025(0.0007i + c)L^{0.33}}{S^{0.333}\, i^{0.667}}$ i = rainfall intensity, in/h c = retardance coefficient L = length of flow path, ft S =slope of flow path, ft/ft	Developed in laboratory experiments by Bureau of Public Roads for overland flow on roadway and turf surfaces; values of the retardance coefficient range from 0.0070 for very smooth pavement to 0.012 for concrete pavement to 0.06 for dense turf; solution requires iteration; product i times L should be $\leq$ 500.
Federal Aviation Administration (1970)	$t_c = 1.8(1.1 - C)L^{0.50} / S^{0.333}$ C = rational method runoff coefficient L = length of overland flow, ft S = surface slope, %	Developed from air field drainage data assembled by the Corps of Engineers; method is intended for use on airfield drainage problems, but has been used frequently for overland flow in urban basins.
Kinematic wave Formulas Morgali and Linsley (1965) Aran and Erborge (1973)	$t_c = \frac{0.94L^{0.6}n^{0.6}}{(i^{0.4}S^{0.3})}$ L = length of overland flow, ft n = Manning roughness coefficient i = rainfall intensity, in/h S = average overland slope , ft/ft	Overland flow equation developed from kinematic wave analysis of surface runoff from developed surfaces; method requires iteration since both i (rainfall intensity) and t_c are unknown; superposition of intensity-duration-frequency curve gives direct graphical solution for t_c
SCS lag equation (1973)	$t_c = \frac{100L^{0.8}[(1000/CN) - 9]^{0.7}}{1900S^{0.5}}$ L = hydraulic length of watershed (longest flow path), ft CN = SCS runoff curve number S = average watershed slope, %	Equation developed by SCS from agricultural watershed data; it has been adapted to small urban basins under 2000 acres; found generally good where area is completely paved; for mixed areas it tends to overestimate; adjustment factors are applied to correct for channel improvement and impervious area the equation assumes that $t_c = 1.67 \times$ basin lag.
SCS average velocity charts (1975, 1986)	$t_c = \frac{1}{60}\sum \frac{L}{v}$ L = length of flow path, ft v = average velocity in feet per second from Fig. 3-1 of TR 55 for various surfaces	Overland flow charts in Fig. 3-1 of TR 55 show average velocity as function of watercourse slope and surface cover.

Source: Kibler. 1982, Copyright by the American Geophysical Union.

2.2.6 Drainage Area

The size and shape of the catchment or subcatchment under consideration must be determined carefully. The area may be determined by planimetering topographic maps. Modern tools like the application of Geographical Information System (GIS) may be handy tools where topographic changes take places or where the mapped contour interval is too great to distinguish the direction of flow. The drainage area contributing to the system to each inlet point must be measured. The outline of the drainage divide must follow the actual watershed boundary, rather than commercial land boundaries.

2.2.7 Assessment of the Rational Method

The rational method is criticized by some hydrologists because of its simplified approach to the calculation of design flow rates. Nevertheless, the rational method is still widely used for the design of storm sewer systems in the United States and other countries because of its simplicity and the fact that the required dimensions of the storm sewers are determined as the computation proceeds. More realistic flow simulation procedures involving the routing of flow hydrographs require the dimensions of the flow conveyance structures to be predetermined. The storm sewer system design produced by the rational method can be considered as a preliminary design whose adequacy can be checked by routing flow hydrographs through the system. Flow velocity computations whether on the overland or in channel, use the Manning's formula which is discussed next.

2.3 MANNING'S EQUATION

The Manning's formula, also known as Stricker's Equation, was first introduced in 1891 by Flamant (Henderson, 1966). The Manning's formula, meant for fully rough turbulent flow, is written as

$$V = \frac{1}{n} R_c^{2/3} S_c^{1/2} \tag{2.5}$$

$$Q = \frac{1}{n} A_c R_c^{2/3} S_c^{1/2} \tag{2.6}$$

where, A_c is the channel water area (m^2), R_c is hydraulic radius (m) which is equal to the term A_c / P_c [channel water area (L^2) / channel effective perimeter (L)] are the geometric elements of the open channel, S_c is channel slope, n is channel roughness and V is the velocity of water along the flow direction.

2.3.1 Geometric Elements of Open Channel

Detailed information of the geometric elements of open channel was presented by Chow (1959), which is briefly stated by Akan (2006) as given in Table 2.3.

Table 2. 3 Geometric elements of different channel section

Section Type	Water Area (A_c)	Wetted Perimeter (P_c)	Top Width (T)	Hydraulic Depth (D)
Rectangular	by where b is bottom width, h is depth of water	$b+2h$	b	h
Trapezoidal	$(b+zh)h$ where z is the side slope of the channel (zH : 1V)	$b+2h\sqrt{1+z^2}$	$b+2zh$	$\frac{(b+zh)h}{b+2zh}$
Triangular	zh^2	$2h\sqrt{1+z^2}$	$2zh$	$\frac{h}{2}$
Circular	$\frac{1}{8}(2\theta-\sin 2\theta)d_0^2$ where d_0 is the diameter and $\theta=\pi-\mathrm{arcCos}\left[(h-\frac{d_0}{2})/(d_0/2)\right]$	θd_0	$(\sin\theta)d_0$ or $2\sqrt{h(d_0-h)}$	$\frac{1}{8}(\frac{2\theta-\sin 2\theta}{\sin\theta})d_0$

2.3.2 Manning's roughness factor (*n*)

In practice, for a given channel, the Manning's roughness factor is assumed not to vary with the flow conditions (Yen, 1992; Hager, 2001; Strum, 2001). The Manning's roughness factor is well documented and published in the literature. Chow (1959) presented an extensive table of minimum, normal and maximum n values for a variety of channel material. Chow's table also reported by French (1985), Chen and Cotton (1988), Strum (2001); US Army Crop Engineers (2002). Manning's roughness factor value for variety of channel material is given in Table 2.4 (Chow, 1964).

Table 2. 4 Mannings Roughness Coefficients for Various Boundaries

Boundary	Manning's Roughness coefficient, n
Very smooth surface such as glass plastic or brass	0.010
Very smooth concrete and planned timber	0.011
Smooth concrete	0.012
Ordinarily concrete lining	0.013
Good wood	0.014
Verified clay	0.015
Shot concrete, untroweled and earth channels in best condition	0.017
Straight unlined earth canals in good condition	0.020
Rivers and earth canals in pore condition - some growth	0.025
Winding natural streams and canals in poor condition - considerable moss growth	0.035
Mountain streams with rockey beds and rivers with variable sections and some vegetation along banks	0.040 - 0.050
Alluvial channels, sand bed, no vegetation	
1.Lower regime	
Ripples	0.017 – 0.028
Dunes	0.018 – 0.035
2. Washed-out dunes or transitions	0.014 – 0.024
3. Upper regime	
Plane bed	0.011 – 0.015
Standing waves	0.012 – 0.016
Antidunes	0.012 – 0.020

2.4 THE UNIT HYDROGRAPH

The unit hydrograph (UH) is the unit pulse response function of a linear hydrologic system. First proposed by Sherman (1932), the unit hydrograph (originally named unit-graph) of a watershed is defined as a direct runoff hydrograph (DRH) resulting from 1 in (usually taken as 1 cm in SI units) of excess rainfall generated uniformly over the drainage area at a constant rate for an effective duration. Sherman originally used the word "unit" to denote a unit of time, but since that time it has often been interpreted as a unit depth of excess rainfall. Sherman classified runoff into surface runoff and groundwater runoff and defined the unit hydrograph for use only with surface runoff.

2.4.1 Unit hydrograph derivation

The unit hydrograph is a simple linear model that can be used to derive the hydrograph resulting from any amount of excess rainfall. The discrete convolution Equation (2.7) allows the computation of direct runoff Q_n when excess rainfall P_m and the unit hydrograph U_{n-m+1} are given as

$$Q_n = \sum_{m=1}^{n\leq M} P_m U_{n-m+1} \tag{2.7}$$

The reverse process, called deconvolution, is needed to derive a unit hydrograph, when rainfall excess values P_m and resulting flood hydrograph ordinate values Q_n are available. Considering M pulses of excess rainfall and N pulses of direct runoff in the storm N equations can be written for Q_n, $n = 1,2,\ldots, N$, in terms of $N-M+1$ unknown values of the unit hydrograph. If Q_n and P_m are given and U_{n-m+1} is required, the set of equations is overdetermined, because there are more equations (N) than unknowns ($N-M+1$).

Unit hydrograph can be calculated using linear regression method, though it may result least square error (Snyder, 1955). Matrix method is usually used to calculate the unit hydrograph ordinates, however, the solution is not easy to determine by this method as many repeated and blank entries in the matix of rainfall data create problem in execution in inverse process involved in it (Bree, 1978). Newton and Vinyard (1967) and Singh (1976) gave alternative methods of obtaining the least square solution, but method do not ensure that all the unit hydrograph ordinates will be non negative. Linear programming is an alternative method for solving for unit hydrograph ordinate which minimizes the absolute error and ensure the non-negativity of unit hydrograph ordinates (Eagleson, Mejia and March, 1966; Deininger, 1969; Singh, 1976; Mays and Coles, 1980; Hiller and Lieberman, 1974; Bradley, Hax, and Magnanti, 1977) . Multistorm analysis may also be carried out using the least squares method (Diskin and Boneh, 1975; Mawdsley and Tagg, 1981). Mays and Taur (1982) used non linear programming to simultaneously determine the loss rate for each storm period and the composite unit hydrograph ordinates for a multistorm event. Unver and Mays (1984) extended this nonlinear programming method to determine the optimal parameters for the loss functions, and the composite unit hydrograph.

In general the unit hydrographs obtained by solutions of the set of equations for different rainfall pulses are not identical. To obtain a unique solution a method of successive approximation (Collins, 1939) can be used, which involves four steps: (i) assume a unit hydrograph, and apply it to all excess-rainfall blocks of the hyetograph except the largest; (ii) subtract the resulting hydrograph from the actual DRH, and reduce the residual to unit hydrograph terms; (iii) compute a weighted average of the assumed unit hydrograph and the residual unit hydrograph, and use it as the revised approximation for the next trial; (iv) repeat the previous three steps until the residual unit hydrograph does not differ by more than a permissible amount from the assumed hydrograph.

The resulting unit hydrograph may show erratic variations and even have negative values. If this occurs, a smooth curve may be fitted to the ordinates to produce an approximation of the unit hydrograph. Erratic variation in the unit hydrograph may be due to nonlinearity in the effective rainfall-direct runoff relationship in the watershed, and even if this relationship is truly linear, the observed data may not adequately reflect this. Also, actual storms are not always uniform in time and space, as required by theory, even when the excess rainfall hyetograph is broken into pulses of short duration.

2.4.2 Assumptions of unit hydrograph

The following basic assumptions are inherent in this model.

(i) The excess rainfall has a constant intensity within the effective duration.

(ii) The excess rainfall is uniformly distributed throughout the whole drainage area.

(iii) The base time of the DRH (the duration of direct runoff) resulting from an excess rainfall of given duration is constant.

(iv) The ordinates of all DRH's of a common base time are directly proportional to the total amount of direct runoff represented by each hydrograph.

(v) For a given watershed, the hydrograph resulting from a given excess rainfall reflects the unchanging characteristics of the watershed.

Under natural conditions, the above assumptions cannot be perfectly satisfied. However, when the hydrologic data to be used are carefully selected so that they come close to meeting the above assumptions, the results obtained by the unit hydrograph model are generally acceptable for practical purposes (Heerdegen, 1974). Although the model was originally devised for large watersheds, it has been found applicable to small watersheds from less than 0.5 hectares to 25 km^2 (about 1 acre to 10 mile^2). Some cases do not support the use of the model because one or more of the assumptions are not well satisfied. For such reasons, the model is considered inapplicable to runoff originating from snow or ice (Chow, 1988).

Concerning assumption (i), the storms selected for analysis should be of short duration, since these will most likely produce an intense and nearly constant excess rainfall rate, yielding a well-defined single-peaked hydrograph of short time base.

Concerning assumption (ii), the unit hydrograph may become inapplicable when the drainage area is too large to be covered by a nearly uniform distribution of rainfall. In such cases, the area has to be divided and each subarea analyzed for storms covering the whole subarea.

Concerning assumption (iii), the base time of the direct runoff hydrograph (DRH) is generally uncertain but depends on the method of baseflow separation. The base time is usually short if the direct runoff is considered to include the surface runoff only; it is long if the direct runoff also includes subsurface runoff.

Concerning assumption (iv), the principles of superposition and proportionality are assumed so that the ordinates Q_n of the DRH may be computed. Actual hydrologic data are not truly linear; when applying Equation (2.7) to them, the resulting hydrograph is only an approximation, which is satisfactory in many practical cases.

Concerning assumption (5), the unit hydrograph is considered unique for a given watershed and invariable with respect to time. This is the principle of time invariance, which, together with the principles of superposition and proportionality, is fundamental to the unit hydrograph model. Unit hydrographs are applicable only when channel conditions remain unchanged and watersheds do not have appreciable storage. This condition is violated when the drainage area contains many reservoirs, or when the flood overflows into the flood plain, thereby producing considerable storage.

2.4.3 Synthetic Unit hydrograph

The catchments, which are at remote locations and rainfall-runoff data are scanty to develop UH conventionally needs alternative way to create UH. In order to construct unit hydrograph characteristics for such areas, empirical equations of regional validity which relate the salient hydrograph characteristics to the basin characteristics should be available. UH derived from such relationships are known as synthetic unit hydrograph (SUH). Synthetic unit hydrograph procedures are used to develop unit hydrographs for other locations on the stream in the same watershed or for nearby watersheds of a similar character. There are three types of synthetic unit hydrographs (i) those relating hydrograph characteristics (peak flow rate, base time, etc.) to watershed characteristics (Snyder, 1938; Gray, 1961), (ii) those based on a dimensionless unit hydrograph (Soil Conservation Service, 1972), and (iii) those based on models of watershed storage (Clark, 1943).

In a study of watersheds located mainly in the Appalachian highlands of the United States, and varying in size from about 10 to 10,000 mile^2 (30 to 30,000 km^2), Snyder (1938) found synthetic relations for some characteristics of a standard unit hydrograph. Additional such relations were found later by U.S. Army Corps of Engineers (1959). In India, Central Water Commission of India (CWC) has divided entire India into some of hydro-meteorological subzone and developed some empirical relations for SUH.

2.4.4 Snyder's Synthetic Unit Hydrograph

The most important characteristic of a basin affecting a hydrograph due to a given storm is basin lag. Basin lag, better known as lag time t_p is the time difference between the centroid of the input i.e. rainfall excess and the output i.e. surface runoff. Physically it represents the mean time of travel of water particles from all parts of the catchment to the outlet during a given storm. Its value is determined essentially on the physical feature of the catchment, such as size (area), length, stream density and vegetation. For its determination, however, only a few important catchment characteristics are considered. Snyder proposed t_p as follows

$$t_p = C_t (L\bar{L})^{0.3} \tag{2.8}$$

where, lag time t_p is in hour, L is the basin length measured along the main channel from the basin divide to the outlet, $\bar{L}$ is the distance along the main channel from the gauging station to the point nearest to the centroid of the basin and C_t is a regional constant representing watershed slope and storage. The value of C_t in Snyder's study ranged from 1.35 to 1.65 (Subramanya, 2003). However, studies by many investigators have shown that C_t depends upon the region under study and wide variations with the value of C_t ranging from 0.3 to 6.0 have been reported (Sokolov et al., 1976). Linsley et al. (1958)

found that the basin lag t_p is better correlated with basin slope S and proposed the following equation

$$t_p = C_{t'}\left(\frac{L\overline{L}}{\sqrt{S}}\right)^{n'} \text{ in hour} \tag{2.9}$$

where $C_{t'}$ and n' are basin constants. For the basins in the USA values of n' was found equal to 0.38 and same of $C_{t'}$ were 1.715 for mountainous drainage areas, 1.03 for foothill drainage areas and 0.50 for valley drainage areas.

Snyder adopted a standard duration t_r hours of effective rainfall given by

$$t_r = \frac{t_p}{5.5} \text{ in hour} \tag{2.10}$$

the peak discharge Q_p (m^3/s) of a unit hydrograph of standard duration t_r hour is given by Snyder as follows

$$Q_p = \frac{2.78\, C_p A}{t_p} \tag{2.11}$$

where, A is the catchment area in km^2, C_p is a regional constant. The equation is based on the assumption that the peak discharge is proportional to the average discharge of (1 cm$\times A$/duration of rainfall excess). The values of the coefficient C_p range from 0.56 to 0.69 for Snyder's study areas and is considered as an indication of the retention and storage capacity of the watershed. Like C_t, the values of C_p also vary quite considerably depending on the characteristics of the region and values of C_p in the range 0.31 to 0.93 have been reported.

If a non-standard rainfall duration t_R hour is adopted, instead of the standard value t_r to derive a unit hydrograph the value of the basin lag is affected. The modified basin lag is given as follows

$$t_p' = t_p + \frac{t_R - t_r}{4} \tag{2.12}$$

where t_p' is the basin lag in hours for an effective duration of t_R hour.

Taylor and Schwartz (1952) proposed a relation of time base t_B which was recommended by Butler (1957) also

$$t_B = 5\left(t_p' + \frac{t_R}{2}\right) \text{ in hours} \tag{2.13}$$

where t_B is taken as the next larger integer value divisible by t_R, i.e. t_B is about five times the time-to-peak.

The time widths of unit hydrographs at 50% and 75% of the peak have been found for US catchments by the US Army Corps of engineers. These widths are correlated to the peak discharge intensity and are given as follows

$$W_{50} = \frac{5.87}{q_p^{1.08}} \tag{2.14}$$

$$W_{75} = \frac{W_{50}}{1.75} \tag{2.15}$$

where, W_{50} is the width of unit hydrograph in hour at 50% peak discharge, W_{75} is the width of unit hydrograph in hour at 75% peak discharge and $q_p (= Q_p / A)$ is the peak discharge per unit catchment area in cumec/km^2.

Since the coefficients C_t and C_p vary from region to region, in practical application it is available that the value of these coefficient are determined from known unit hydrographs of a meteorologically homogeneous catchment and then used in the basin under study.

2.4.5 Unit hydrographs for different rainfall durations

When a unit hydrograph of a given excess-rainfall duration is available, the unit hydrographs of other durations can be derived. If other durations are integral multiples of the given duration, the new unit hydrograph can be easily computed by application of the principles of superposition and proportionality. However, a general method of derivation applicable to unit hydrographs of any required duration may be used on the basis of the principle of superposition. This is the S-hydrograph method.

The theoretical S-hydrograph is that resulting from a continuous excess rainfall at a constant rate of 1 cm/h (or 1 in/h) for an indefinite period. This is the unit step response function of a watershed system. The curve assumes a deformed S shape and its ordinates ultimately approach the rate of excess rainfall at a time of equilibrium. This step response function $g(t)$ can be derived from the unit pulse response function $h(t)$ of the unit hydrograph, as follows. The response at time t to a unit pulse of duration at beginning at time 0 is

$$h(t) = \frac{1}{\Delta t}[g(t) - g(t - \Delta t)] \tag{2.16}$$

Similarly, the response at time t to a unit pulse beginning at time Δt is equal to $h(t - \Delta t)$, that is, $h(t)$ lagged by Δt time units:

$$h(t - \Delta t) = \frac{1}{\Delta t}[g(t - \Delta t) - g(t - 2\Delta t)] \tag{2.17}$$

and the response at time t to a third unit pulse beginning at time $2\Delta t$ is

$$h(t - 2\Delta t) = \frac{1}{\Delta t}[g(t - 2\Delta t) - g(t - 3\Delta t)] \tag{2.18}$$

Continuing this process indefinitely, summing the resulting equations, and rearranging, yields the unit step response function or S-hydrograph as follows

$$g(t) = \Delta t\ [h(t) + h(t - \Delta t) + h(t - 2\ \Delta t) + \ldots] \tag{2.19}$$

where the summation is multiplied by Δt so that $g(t)$ will correspond to an input rate of 1, rather than $1/\Delta t$ as used for each of the unit pulses.

Theoretically, the S-hydrograph so derived should be a smooth curve, because the input excess rainfall is assumed to be at a constant, continuous rate. However, the summation process will result in an undulatory form if there are errors in the rainfall abstractions or baseflow separation, or if the actual duration of excess rainfall is not the derived duration for the unit hydrograph. A duration which produces minimum undulation can be found by trial. Undulation of the curve may be also caused by nonuniform temporal and areal distribution of rainfall; furthermore, when the natural data are not linear, the resulting unstable system oscillations may produce negative ordinates. In such cases, an optimization technique may be used to obtain a smoother unit hydrograph.

After the S-hydrograph is constructed, the unit hydrograph of a given duration can be derived as follows: Advance, or offset, the position of the S-hydrograph by a period equal to the desired duration $\Delta t'$ and call this S-hydrograph an offset S-hydrograph, $g'(t)$ defined by

$$g'(t) = g(t - \Delta t') \tag{2.20}$$

The difference between the ordinates of the original S-hydrograph and the offset S-hydrograph, divided by $\Delta t'$, gives the desired unit hydrograph.

$$h'(t) = \frac{1}{\Delta t'}[g(t) - g(t - \Delta t')] \tag{2.21}$$

2.4.6 Clark's method for deriving instantaneous unit hydrograph

Clark's method, also known as Time-area histogram method aims at developing an instantaneous unit hydrograph due to an instantaneous rainfall excess over a catchment. It is assumed that the rainfall excess first undergoes pure translation and then attenuation. The translation is achieved by a travel time-area histogram and the attenuation by routing the result of the above through a linear reservoir at the catchment outlet.

Time here refers to the time of concentration t_c.The time interval between the end of the rainfall excess and the point of second point of contra flexion of the resulting surface runoff provides a good way of estimating t_c from a known rainfall-runoff data.

Total catchment area drains into outlet in t_c hours. If points on the area having equal time of travel are considered and located on a map of the catchment, a line joining them is called an isochrone (or runoff isochrone). The inter-isochrone areas $A_1, A_2, A_3, \ldots\ldots, A_N$ are used to construct a travel time-area histogram. If a rainfall excess of 1 cm occurs instantaneously and uniformly over the catchment area, this time-area histogram represents the sequence in which the volume of rainfall will be moved out of the catchment and arrive at the outlet. If a sub-area A_r km^2 represent a volume of A_r km^2.cm (= $A_r \times 10^4$ m^3) moving out in time $\Delta t_c = t_c / N$ hours. The hydrograph of outflow obtained by the time-area histogram, while properly accounting for the sequencing of arrival of flows, do not provide for the storage properties of the catchment.

To overcome this deficiency, Clark (1943) assumed a linear reservoir to be hypothetically available at the outlet to provide the requisite attenuation. Linear reservoir at the outlet is assumed to be described as follows

$$S = KQ \tag{2.22}$$

where, K is the storage time constant. The value of K can be estimated by considering the point of inflection of a surface runoff hydrograph. At this point the inflow into the channel has ceased and beyond this point the flow is entirely due to withdrawal from the channel storage. The continuity equation (2.22) can be rewritten as follows

$$I - Q = \frac{ds}{dt} \tag{2.23}$$

$$-Q = \frac{ds}{dt} = K\frac{dQ}{dt} \tag{2.24}$$

Hence, $K = -Q_i / (dQ/dt)_i$ (2.25)

where, i refers to the point of the inflection, and K can be estimated from a known surface runoff hydrograph of the catchment. The constant K can be estimated from the data on the recession limb of a hydrograph. Knowing K of the linear reservoir, the inflows at various times are routed by the Muskingum method (Chow et al., 1988), which is stated as follows

$$Q_n = C_0 I_n + C_1 I_{n-1} + C_2 Q_{n-1} \tag{2.26}$$

$$C_0 = \frac{-Kx + 0.5\Delta t}{K - Kx + 0.5\Delta t} \tag{2.27}$$

$$C_1 = \frac{Kx + 0.5\Delta t}{K - Kx + 0.5\Delta t} \tag{2.28}$$

$$C_2 = \frac{K - Kx - 0.5\Delta t}{K - Kx + 0.5\Delta t} \tag{2.29}$$

where, $C_0 + C_1 + C_2 = 1.0$ and x is a weighting factor having the range $0 \le x \le 0.5$. The value of x depends on the shape of the modeled wedge storage. The value of x ranges from zero for reservoir- type storage to 0.5 for a full wedge. When the value is $x = 0$, there is no wedge hence no backwater ,i.e. level-pool reservoir. In natural stream the values of x varies from 0 to 0.3.

The inflow rate between an inter-isochrones area A_r km^2 with a time interval Δt_c (hr) can be sated as follows

$$I = \frac{A_r \times 10^4}{3600\,\Delta t_c} \quad (\text{m}^3/\text{s}) \tag{2.30}$$

2.5 SIMULATING DESIGN FLOWS

Since the early 1960s, a host of deterministic hydrologic simulation models have been developed. These models include event simulation models for modeling a single rainfall-runoff event and continuous simulation models, which have soil moisture accounting procedures to simulate runoff from rainfall in hourly or daily intervals over long time periods. Examples of event simulation models include: the U. S. Army Corps of Engineers (1981) HEC-1 flood hydrograph model; later it is updated to Hydrologic Modeling System HEC-HMS (2006). The HEC-HMS model is probably the most widely used hydrologic event simulation model. The acronym HEC stands for Hydrologic Engineering Center, the U. S. Army Corps of Engineers research facility in Davis, California, where this model was developed. The Soil Conservation Service (1965) TR-20 computer program for project hydrology; the U. S. Environmental Protection Agency (1977) SWMM storm water management model; and the Illinois State Water Survey ILLUDAS model, by Terstriep and Stall (1974) are the other event simulation models. Examples of continuous simulation models include: the U. S. National Weather Service runoff forecast system (Day, 1985); the U.S. Army Corps of Engineers (1976) STORM model; and the U.S. Army Corps of Engineers (1972) SSARR streamflow synthesis and reservoir regulation model. This is by no means a complete list of available models, but it covers most of the models commonly used in hydrologic practice.

A subarea land surface runoff component is used to represent the movement of water over the land surface and into stream channels. The input to this component is a rainfall hyetograph. Excess rainfall is computed by subtracting infiltration and detention losses, based on an infiltration function that may be chosen from several options, including the Soil Conservation Service (SCS) curve number loss rate. Rainfall and infiltration are assumed to be uniformly distributed over the subbasin. The resulting rainfall excesses are then applied to the unit hydro-graph to derive the subarea outlet runoff hydrograph. Unit hydrograph options include the Snyder's unit hydrograph and the SCS dimensionless unit hydrograph. Alternatively, a kinematic wave model can be used to find subbasin runoff hydrographs.

A stream routing component is used to represent flood wave movement in a channel. The input to this component is an upstream hydrograph resulting from individual or combined contributions of subarea runoff, streamflow routings, or diversions. This hydrograph is routed to a downstream point, using the characteristics of the channel. The techniques available to route the runoff hydrograph include the Muskingum method, level-pool routing, and the kinematic wave method.

2.6 FLOOD PLAIN ANALYSIS

A flood plain is the normally dry land area adjoining rivers, streams, lakes, bays, or an ocean, that is inundated during flood events. The most common causes of flooding are the overflow of streams and rivers and abnormally high tides resulting from severe storms. The flood plain can include the full width of narrow stream valleys, or broad areas along streams in wide, flat valleys. The channel and flood plain are both integral parts of the natural conveyance of a stream. The flood plain carries flow in excess of the channel capacity and the greater the discharge, the further the extent of flow over the flood plain.

The first step in any flood plain analysis is to collect data, including topographic maps, flood flow data if a gauge station is nearby, rainfall data if flood flow data are not available, and surveyed cross sections and channel roughness estimates at a number of points along the stream.

A determination of the flood discharge for the desired return period is required. If gauged flow records are available, a flood flow frequency analysis can be performed. If gauged data are not available, then a rainfall-runoff analysis must be performed to determine the flood discharge. The rainfall hyetograph is determined for the desired return period, a synthetic unit hydrograph is developed for each subarea of the drainage basin, and the direct runoff hydrograph from each subarea is calculated. The subarea direct runoff hydrographs are routed downstream and added to determine the total direct runoff hydrograph at the most downstream part of the drainage basin. The peak discharge of the most downstream hydrograph is used as the design flood discharge.

Once the flood discharge for the desired return period has been determined, the next step is to determine the profile of water surface elevation along the channel. This analysis can be carried out assuming steady, gradually-varied, non-uniform flow using a one-dimensional model such as HEC-2 (U. S. Army Corps of Engineers, 1982), which is updated to HEC-RAS (2005); or a two-dimensional model based upon either finite differences or finite elements (Lee and Bennett, 1981; Lee, et al., 1982; Mays and Taur, 1984). One-dimensional models allow the flow properties to vary along the channel only, while two-dimensional models account for changes across the channel as well. Unsteady flow models are necessary for flood plain delineation in large lakes because the storage in the lake alters the shape and peak discharge of the flood hydrograph as it passes through.

After the water surface elevations have been determined, the area covered by the flood plain is delineated. The lateral extent of the flood plain is determined by finding ground points on both sides of the stream that correspond to the flood profile (water surface) elevations. Ground elevations in the flood plain can be determined from topographic maps, street maps, or stereo aerial photos. Topographic maps are the most convenient, with the elevations given by contour lines. The flood plain boundary is determined by following the contour line that corresponds to the flood profile elevation for a particular area. Of course, the flood plain delineation is only as accurate as the topographic maps used. After flood levels have been, determined for a particular reach of stream the actual location of the flood plain boundaries should be checked by field surveys.

Encroachment on flood plains, such as by artificial fill material, reduces the flood-carrying capacity, increases the flood heights of streams, and increases flood hazards in areas beyond the encroachment. One aspect of flood plain management involves balancing the economic gain from flood plain development against the resulting increase in flood hazard.

2.7 FLOOD CONTROL STRUCTURE DESIGN (LEVEE)

Urbanization increases both the volume and the velocity of runoff, and efforts have been made in urban areas to offset these effects. Levee can provide one means of managing flood water. The main purpose of a levee is to prevent flooding of the adjoining countryside; however, they also confine the flow of the river resulting in higher and faster water flow. Levees are usually built by piling earth on a cleared, level surface. Broad at the base, they taper to a level top, where temporary embankments or sandbags can be placed. Because flood discharge intensity increases in levees on both river banks, and because silt deposits raise the level of riverbeds, planning and auxiliary measures are vital. Sections are often set back from the river to form a wider channel, and flood valley basins are divided by multiple levees to prevent a single breach from flooding a large area (Petroski, 2006). When a river/ channel floods over its banks, the water spreads out, slows down, and deposits its load of sediment. Over time, the river's banks are built up above the level of the rest of the floodplain.

Sometimes levees are said to fail when water overtops the crest of the levee. Levee overtopping can be caused when flood waters simply exceed the lowest crest of the levee system; Levee at this stage is acting as a weir. Overtopping can lead to significant landside erosion of the levee or even be the mechanism for complete breach. Properly built levees are armored or reinforced with rocks or concrete to prevent erosion and failure. Most levees failed due to water overtopping them but some failed when water passed underneath the levee foundations causing the levee wall to shift and resulting in catastrophic sudden breaching.

Following the concepts discussed in the previous sections a model has been conceived which is discussed in the following chapter.

CHAPTER 3 MODEL DEVELOPMENT

3.1 INTRODUCTION

In the previous Chapter, the basic concepts along with the literature review were briefly discussed. In present days, a practicing hydrologist has put resilience on models which are readily available in different software packages as stated in Section 2.5. Though these models handled lumped as well as distributed characteristics of the flow process over a drainage basin, in this study, an attempt has been made to develop a model using proper combination of the hydrology and hydraulics condition to manage flood water over a natural as well as urbanized catchment.

A hydrologic model is considered as a quantitative expression of a hydrologic process or phenomenon which one is observing, analyzing, and wants to use for prediction purpose. Mathematical modeling of hydrologic process provides a tool by means of which one can study and gain an understanding of hydraulic flow phenomenon, select and design engineering structures to manage flood water. An attempt is made to develop a model based on hydraulic equations which represent the flow characteristics. There are many factors which influence the runoff process. These affect the movement of water on the overland plane, in the channels and as well as in flood plain areas. Different hydrologic and hydraulic facets have been investigated in order to get an insight into complexities involve in it.

To achieve this objective, various techniques and available models were studied. It was decided that investigation of the watershed response be carried out in two phases. The first phase may deal with the flood water movement in the upstream of the control structure, which includes water movement over the overland surface, and in the stream channels; the second phase may deal with the flood water movement in the flood plain areas.

Rational Method and Manning's Equation have been employed for determination of different parameters associated with flow characteristics and a general dimensionless multivariable nomograph has been prepared. This nomograph is pretty much helpful in determining different flow situations for different types of parameter combination, and allows user to adjust some of the parameters to control the flood water movement. In the end, using area-volume-elevation analysis and levee (broad crested weir) design provide a flood management system to handle the excess flood water especially for urbanized watershed.

Thus, the present study is aimed at developing a mathematical model utilizing the characteristics of hydrology as well as hydraulics for flood management system of urban watershed.

3.2 ELEMENTS USED IN THE PROPOSED MODEL

To meet the objectives stated in Chapter 1, a suitable model need to be developed. The model should properly take into account the mechanics of water on the plane, in the channel as well as in the flood plain area. Also, intention of the study was multipurpose to maintain a particular depth of water in a channel for environmental purposes. A moderate depth was kept in most of times of the year for irrigation purposes as well. In extreme condition, excess water in the channel can be a cause of flood in flood prone area. In that case a control structure (levee) is provided for a maximum allowable water depth h' as a precaution of flood management. Exceeding that depth h' water may overtop and enter into flood prone areas.

So the entire model was divided into main two elements. (i) Elements in the upstream of control structure (Section 3.2.1), and (ii) Elements in the downstream of control structure, (Section 3.2.2).

3.2.1 Elements in the Upstream of Control Structure

Elements in the upstream of control structure are linked with flow on the overland plane and flow in the channel upto the outlet. For a catchment of drainage area A_d peak discharge rate at the outlet of the catchment Q_p can be calculated at time of concentration t_c using Rational Formula (equation 2.1), where hydrologists or engineers have a clear idea about the land use nature of the catchment (runoff coefficient C, Section 2.2.3, Table 2.1) and about a constant rate of rainfall excess intensity i of a particular duration

$$Q_p = 0.2778\, C\, i\, A_d \tag{3.1}$$

where, Q_p is peak discharge is in m^3/s. A weighted runoff coefficient , C ,can be used in case of complex nature of the catchment land use area, rainfall intensity, i, is in mm/hr and can be obtained for a particular duration and return period from a intensity duration frequency (IDF) curve, and drainage area are in km^2. Weighted runoff coefficient C_w can be calculated as follows

$$C_w = \frac{\sum_{i=1}^{m} C_i A_i}{\sum_{i=1}^{m} A_i} \tag{3.2}$$

where, C_i is the runoff coefficient of drainage area A_i, $\sum_{i=1}^{m} A_i = A_d$ and m is the number of sub- catchment. From the equation 3.1, a relations between runoff coefficients and Q_p for different rainfall excess intensities can be worked out.

The graphical relationship between q (i.e. peak discharge per unit area and per unit width of channel of the control section) and C_j for different rainfall intensity is shown in Figure 3.1. This relationship subsequently forms the first quadrant of the nomograph. This relationship holds good for different shapes of the control section.

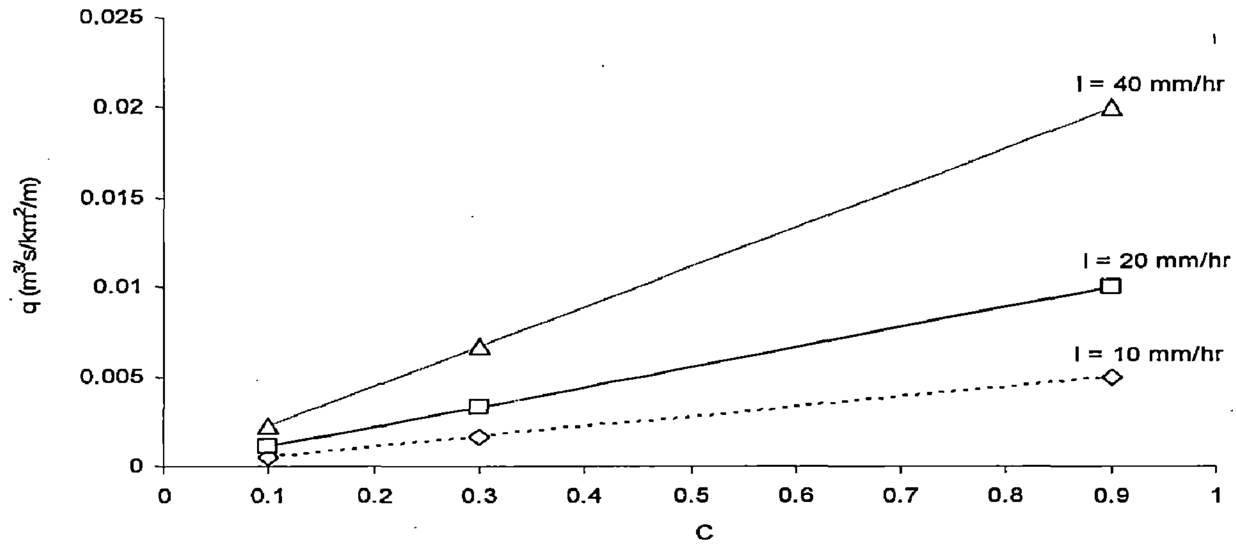

Figure 3. 1 Relation between runoff coefficient and peak discharge per unit catchment area per unit channel width for different rainfall intensity values.

3.2.2. Elements in the Downstream of Control Structure

Once Q_p are known at the outlet of the catchment for a rainfall intensity and a particular value of C, the Manning's relationship is being used for different slopes of channels for the given slope value (S_c) and given roughness (n). Corresponding to it the water depth h in the channel is found out as follows.

Case I - Wide Rectangular Channel

In a wide rectangular channel, corresponding to a particular water depth h_c (m), having a bottom width of channel b (m); the water area of a wide rectangular channel is calculated as bh_c. The wetted perimeter is approximate b and let its hydraulic radius be h_c. Thus, the Manning's relationship (Section 2.3, equation 2.6) may be rewritten as follows

$$Q = \frac{1}{n} b h_c^{5/3} S_c^{1/2} \tag{3.3}$$

From equation (3.3) water depth in the channel was obtained as

$$h_c = \left[\frac{Qn}{b S_c^{1/2}} \right]^{3/5} \tag{3.4}$$

For one percent slope value for different roughness, the graphical relationship is shown in Figure 3.2.

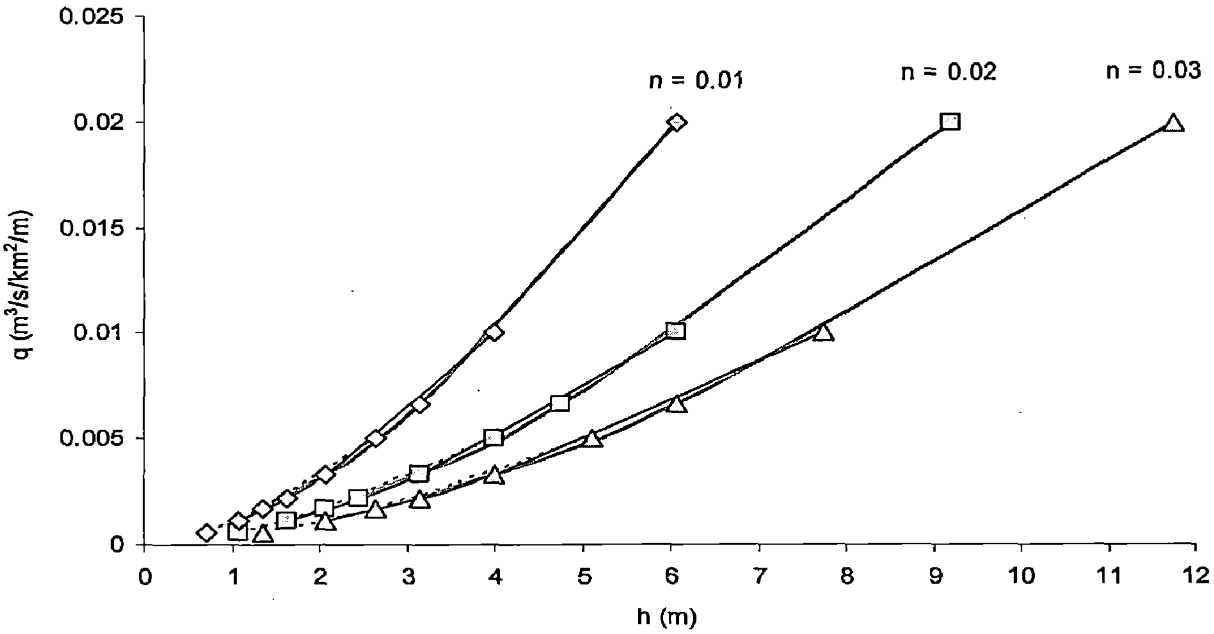

Figure 3. 2 Relation between water depth and peak discharge per unit catchment area per unit channel width for different values of roughness.

In the proposed nomograph this will form the second quadrant. Discharge per unit channel width is calculated as under.

$$q' = Q / b = \frac{1}{n} h_c^{5/3} S_c^{1/2} \quad (3.5)$$

Discharge passing through outlet per unit width of the channel and per unit drainage area is given as under.

$$q = q' / A_d = \frac{1}{A_d} \frac{1}{n} h_c^{5/3} S_c^{1/2} \quad (3.6)$$

From equation (3.4) velocity of the flow in the channel is obtained as follows

$$V = \frac{1}{n} h_c^{2/3} S_c^{1/2} \quad (3.7)$$

The relationship between V (m/s) and h_c (m) is shown for different n values in Figure 3.3.

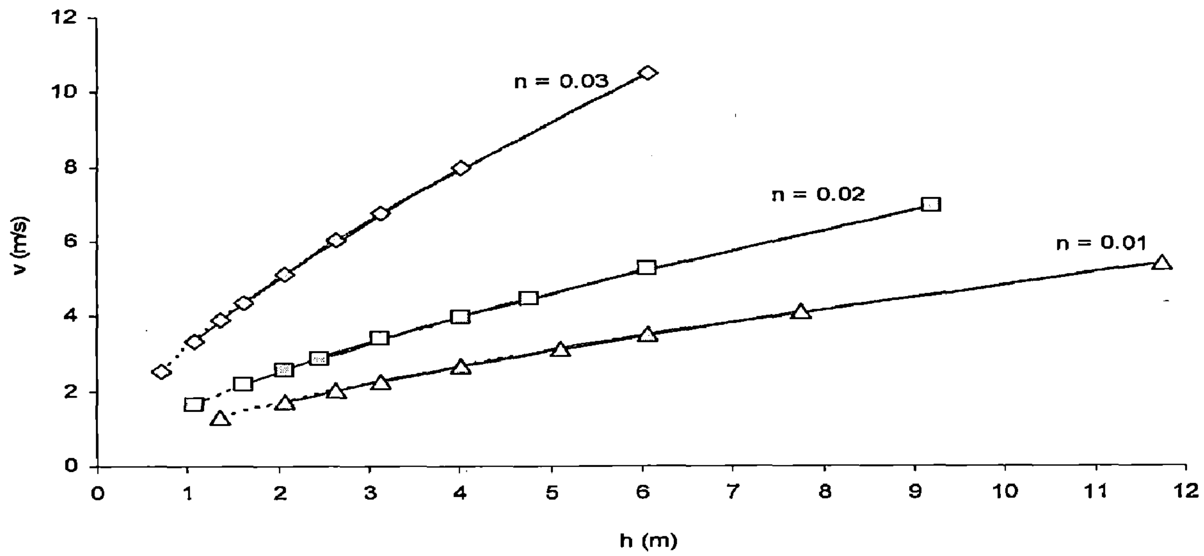

Figure 3. 3 Relation between water depth and Manning's velocity for different values of roughness.

This forms third quadrant of the proposed nomograph. Also, time of concentration for a catchment is worked out as given below

$$t_c = L / V \quad (3.8)$$

where L is the longest Channel length (m). The ratio of inertial to gravitational forces acting on the flow (Froude Number) is represented as

$$Fr = \sqrt{V^2 / g h_c} \quad (3.9)$$

where g is acceleration of gravity, The flow is said to be at critical state when $Fr = 1.0$, the flow is subcritical when $Fr < 1.0$, and it is supercritical when $Fr > 1.0$ (Akan, 2006).

The relationship between V (m/s) with Froude's number and time of concentration t_c is shown in Figure 3.4.

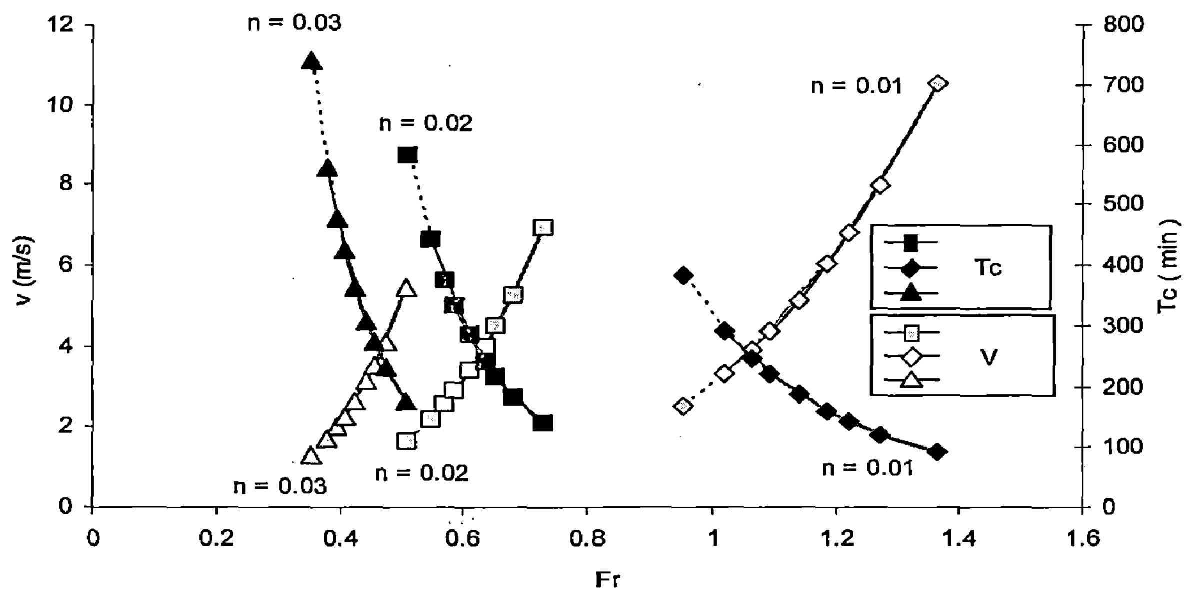

Figure 3. 4 Relation between Froude number with Manning's velocity and time of concentration and for different values of roughness.

This forms the fourth quadrant of the proposed nomograph. Combining the Figures 3.1 through 3.4, the nomograph for computing the hydraulic parameters for wide rectangular channel is shown in Figure 3.5.

Relationships developed and given earlier are meant for wide rectangular channel. For the trapezoidal section the mathematical formulations are described in the following section.

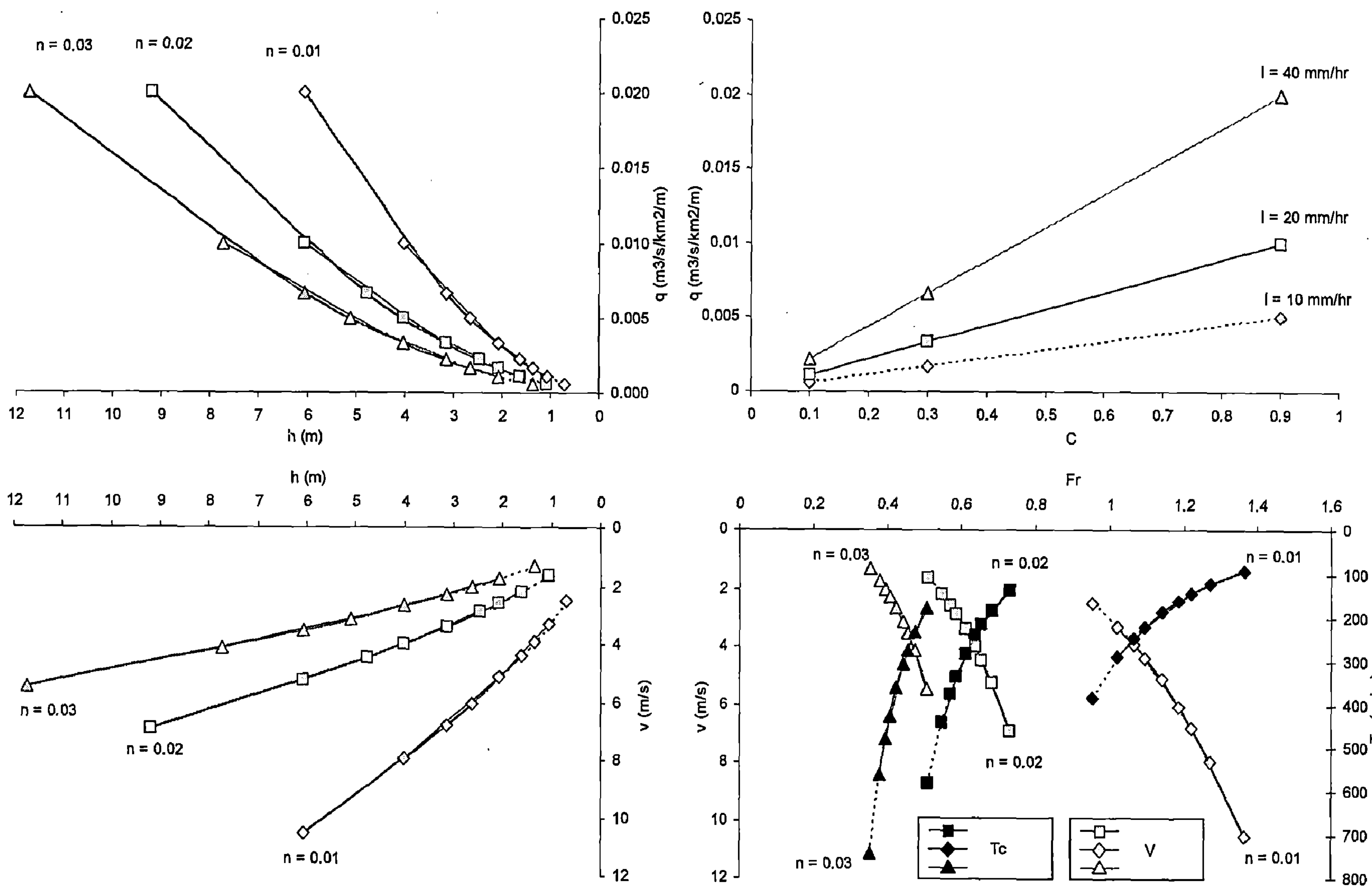

Figure 3. 5 Nomograph of discharge per unit channel bottom width and per unit drainage area and runoff coefficient for different rainfall intensity; and water depth, velocity, Froude number and time of concentration (in secondary axis) for different roughness coefficient for a particular channel slope of 1% for wide rectangular cross-sectional channel.

Case II - Trapezoidal Channel Section

Corresponding to a particular water depth h_c (m), having a bottom width of channel b (m) and side slope of the channel z, the water area, wetted perimeter, hydraulic radius etc. for a trapezoidal channel are calculated as given in Table 2.3 (Section 2.3.1). Manning's formula (Section 2.3, equation 2.6) can be rewritten as follows

$$Q = \frac{1}{n} \frac{[(b+zh_c)h_c]^{5/3}}{\left[b+2h_c\sqrt{1+z^2}\right]^{2/3}} S_c^{\,1/2} \tag{3.10}$$

To compute h_c the above non-linear equation is solved using Newton-Raphson Method. Using equation (3.10) a function f of h_c is considered as follows

$$f(h_c) = Q - \frac{1}{n} \frac{[(b+zh_c)h_c]^{5/3}}{\left[b+2h_c\sqrt{1+z^2}\right]^{2/3}} S_c^{\,1/2} \tag{3.11}$$

Derivative of the function $f(h_c)$ with respect to h_c is as follows

$$f'(h_c) = \left(-\frac{S_c^{\,1/2}}{n}\right) \frac{\left\{ \begin{array}{l} \left[b+2h_c\sqrt{1+z^2}\right]^{2/3} \left(\dfrac{5}{3}\right)[(b+zh_c)h_c]^{2/3}(b+2zh_c) \\ -[(b+zh_c)h_c]^{5/3}\left(\dfrac{2}{3}\right)\left[b+2h_c\sqrt{1+z^2}\right]^{-1/3}(2\sqrt{1+z^2}) \end{array} \right\}}{\left[b+2h_c\sqrt{1+z^2}\right]^{4/3}} \tag{3.12}$$

The optimization algorithm for Newton-Raphson method is expressed as given below

$$h_{c\_new} = h_{c\_old} - \frac{f(h_{c\_old})}{f'(h_{c\_old})} \tag{3.13}$$

The optimization terminated when the following condition is achieved

$$(h_{c\_new} - h_{c\_old}) \le 0.0001 \tag{3.14}$$

Once h_c is obtained, q, V, t_c, Fr are calculated on similar lines as of equation (3.6) to equation (3.9). The nomograph for computing hydraulic parameters of trapezoidal section is developed on similar lines and is given in Figure 3.6.

3.2.2. Dimensionless Analysis

For the various parameters of the nomographs (Figures 3.5 and 3.6), the maximum attainable values are assigned as C_{max}, i_{max}, n_{max}, Q_{max}, h_{cmax}, V_{max}, t_{cmax} and Fr_{max} . The plots of the nomograph using the parameters have been rationalized with respect to these values and the ratios used are C/C_{max}, i/i_{max} , n/n_{max} , Q/Q_{max}, h_c/h_{cmax}, V/V_{max}, t_c/t_{cmax} and Fr/Fr_{max}. The nomographs so arrived at are shown in Figure 3.7 and Figure 3.8.

3.2.3. Regression Relations

The regression relationships for the dimensionless plots of different quadrants were also worked out and are given as follows for the wide rectangular channel. One may use these relationships as an alternative of the nomograph

$$q/q_{max} = i/i_{max} \times C/C_{max} \tag{3.15}$$

$$q/q_{max} = n_{max}/n \times (h_c/h_{cmax})^{1.667} \tag{3.16}$$

$$V/V_{max} = \frac{0.5173}{n/n_{max}} \times (h_c/h_{cmax})^{0.667} \tag{3.17}$$

$$V/V_{max} = (3 \times n/n_{max})^3 \times (Fr/Fr_{max})^4 \tag{3.18}$$

$$t_c/t_{cmax} = \frac{0.1234}{(3 \times n/n_{max})^3} \times (Fr/Fr_{max})^{-4} \tag{3.19}$$

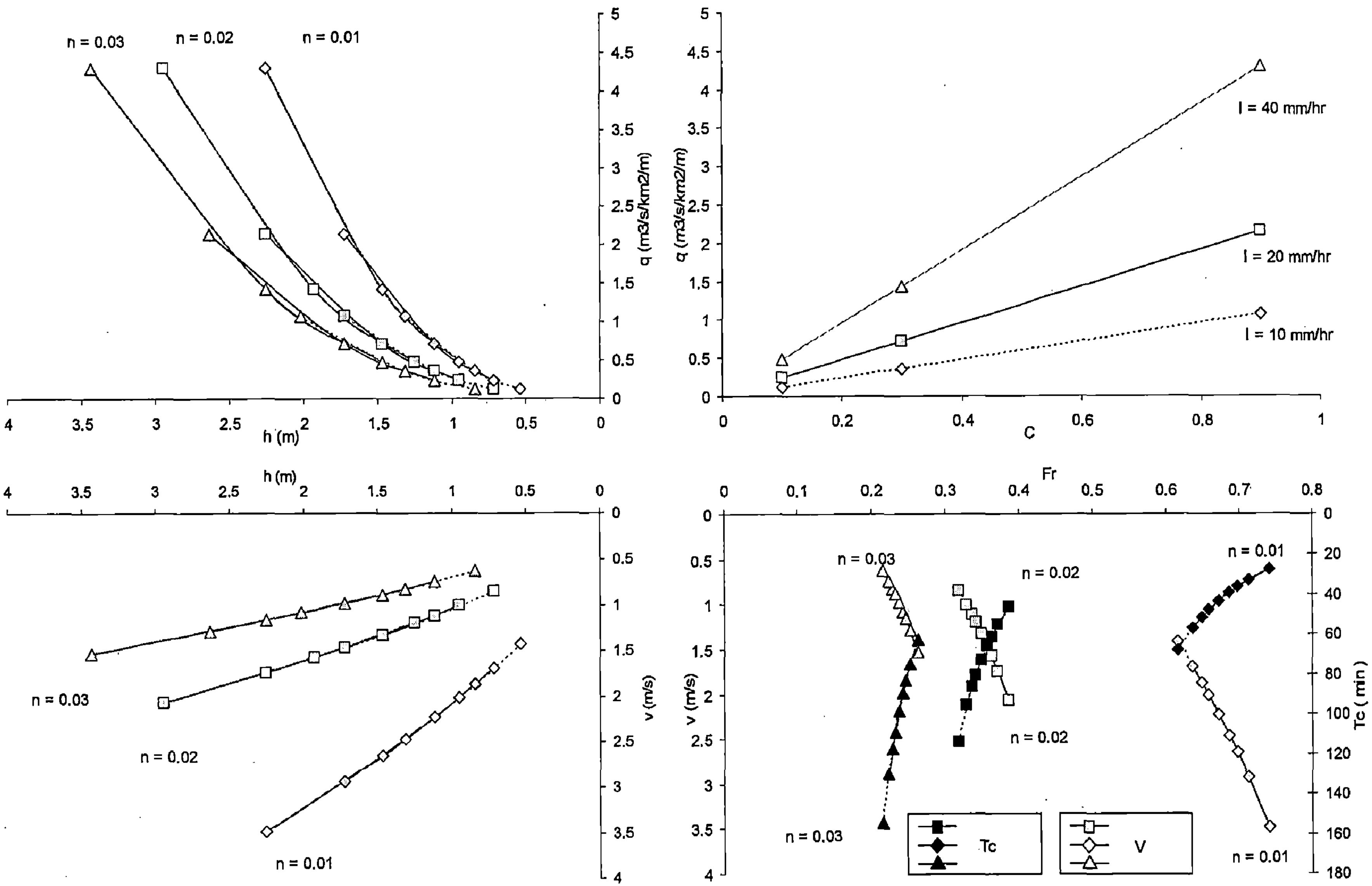

Figure 3. 6 Nomograph of discharge per unit channel bottom width and per unit drainage area and runoff coefficient for different rainfall intensity; and water depth, velocity, Froude number and time of concentration (in secondary axis) for different roughness coefficient for trapezoidal cross-sectional channel.

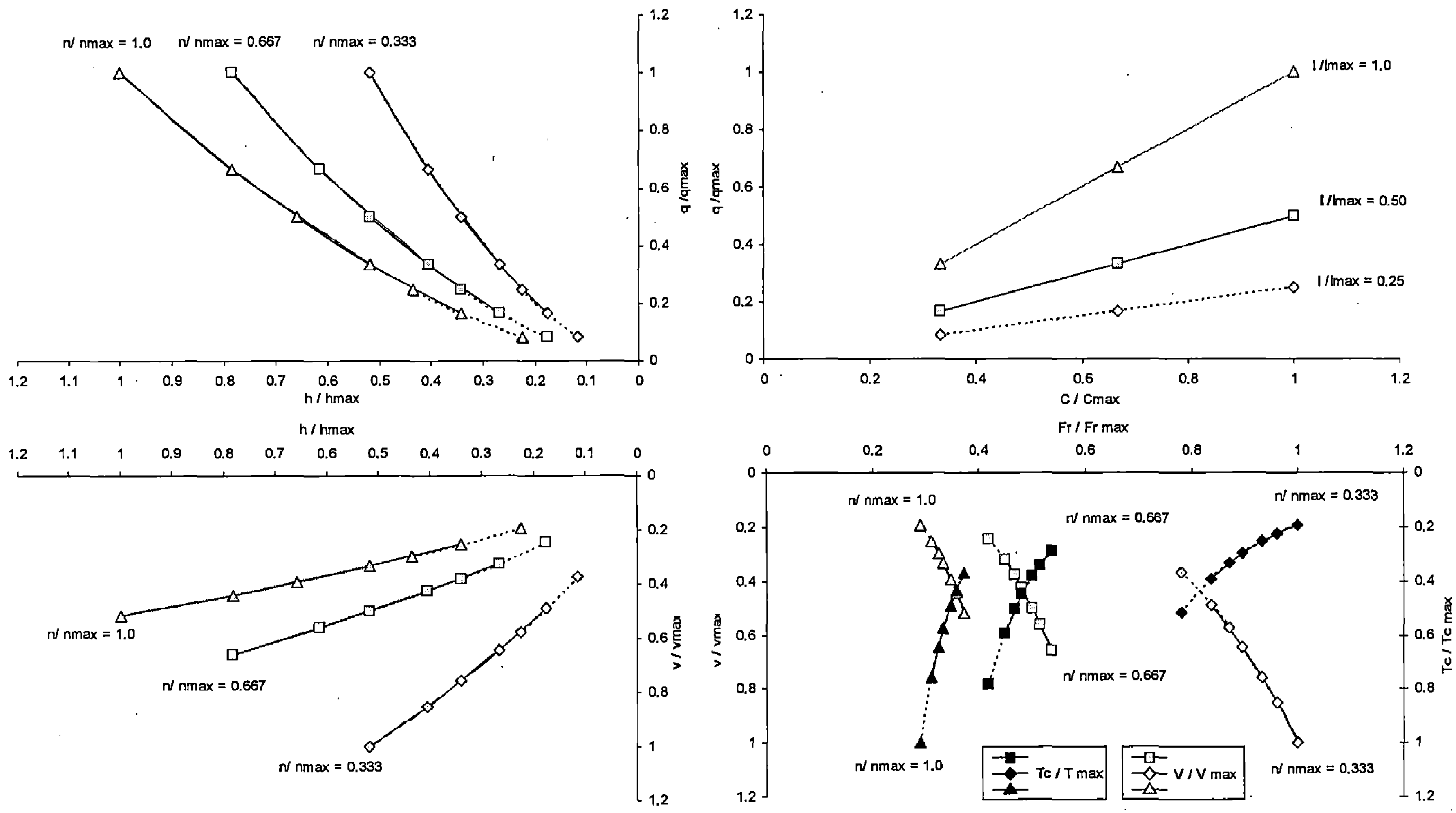

Figure 3. 7 Nomograph of dimensionless discharge per unit channel bottom width and per unit drainage area (q/q_{max}) and runoff coefficient (C/C_{max}) for different rainfall intensity (i/i_{max}); and water depth (h/h_{max}), velocity (v/v_{max}), Froude number (Fr/Fr_{max}) and time of concentration ($T_c/T_{c\ max}$) (in secondary axis) for different roughness coefficient (n/n_{max}) for wide rectangular cross-sectional channel.

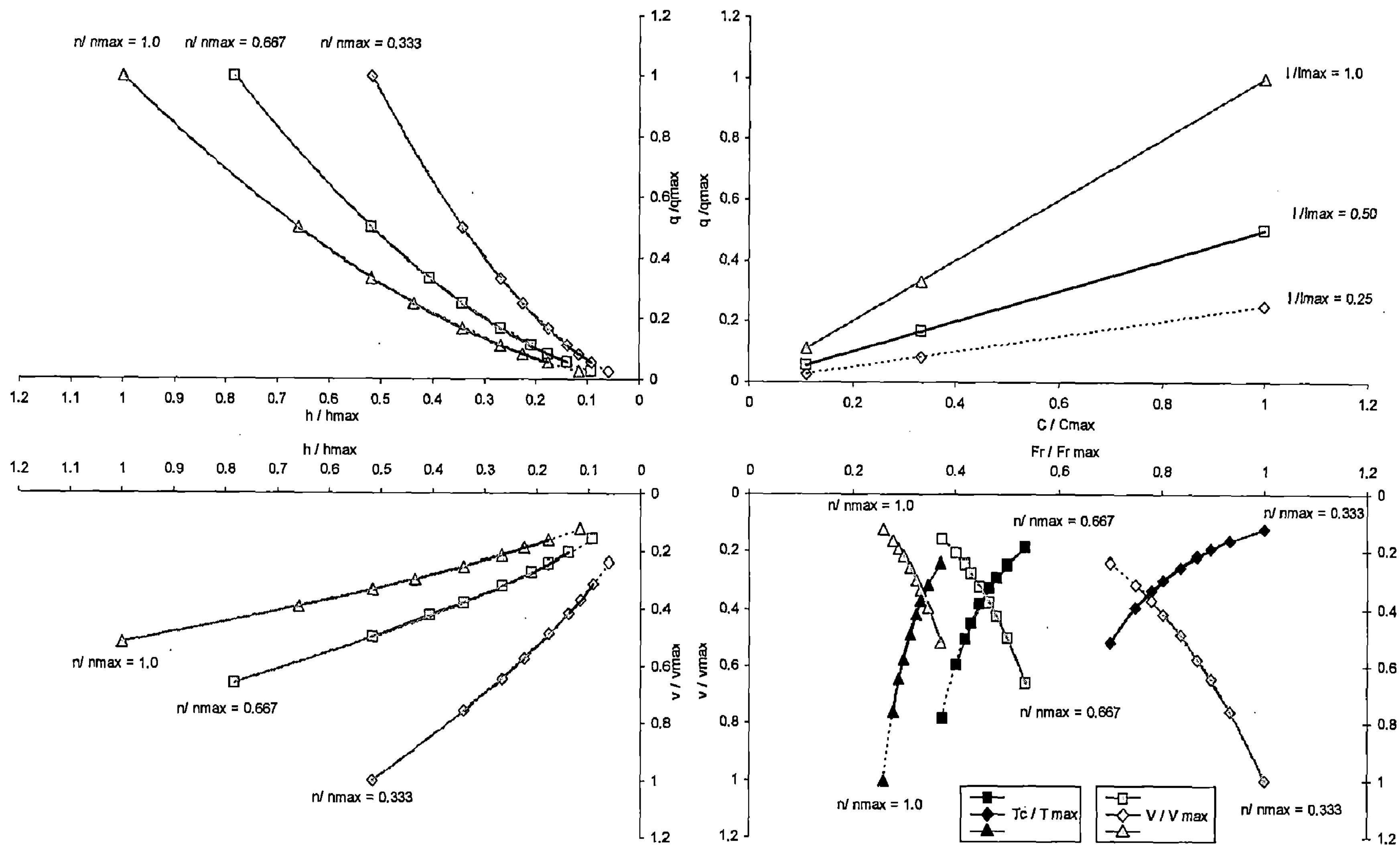

Figure 3. 8 Nomograph of dimensionless discharge per unit channel bottom width and per unit drainage area (q/q_{max}) and runoff coefficient (C/C_{max}) for different rainfall intensity (i/i_{max}); and water depth (h/h_{max}),velocity (v/v_{max}), Froude number (Fr/Fr_{max}) and time of concentration ($T_c/T_{c\ max}$) (in secondary axis) for different roughness coefficient (n/n_{max}) for trapezoidal cross-sectional channel.

For the nomograph given in Figure for the trapezoidal section of the channel, the regression relationships for side slope of the channel z as 1:1 were worked out and are given as follows

$$q/q_{max} = i/i_{max} \times C/C_{max} \tag{3.20}$$

$$q/q_{max} = \left[\frac{1}{(n/n_{max})} - \frac{0.0062}{(n/n_{max})^{2.87}}\right] \times (h_c/h_{cmax})^{\left[1.667+0.05(n/n_{max})^{1.27}\right]} \tag{3.21}$$

$$V/V_{max} = \left[\frac{1}{2.(n/n_{max})} - \frac{0.0373}{(n/n_{max})^{0.47}}\right] \times (h_c/h_{cmax})^{\left[0.5+\frac{0.0339}{(n/n_{max})^{0.667}}\right]} \tag{3.22}$$

$$v/v_{max} = \left[\begin{array}{l} \left[\left\{3.8\left(\frac{n}{n_{max}}\right)-\left(\frac{n}{n_{max}}\right)^3\right\}^{8.327\left(\frac{n}{n_{max}}\right)} - 0.885\right] \times \\ (Fr/Fr_{max})^{\left[\frac{0.51\times n}{n_{max}}+\left\{\frac{3.14\times n}{n_{max}}\right\}^{\frac{n}{0.667\times n_{max}}}+5.93\right]} \end{array}\right] \tag{3.23}$$

$$t_c/t_{cmax} = \left[\begin{array}{l} \left(\frac{1}{6}\right)\left[\left\{3.8\left(\frac{n}{n_{max}}\right)-\left(\frac{n}{n_{max}}\right)^3\right\}^{8.327\left(\frac{n}{n_{max}}\right)} - 0.8\right]^{-1} \times \\ (Fr/Fr_{max})^{-\left[\frac{0.51\times n}{n_{max}}+\left\{\frac{3.14\times n}{n_{max}}\right\}^{\frac{n}{0.667\times n_{max}}}+5.93\right]} \end{array}\right] \tag{3.24}$$

3.3 COMPUTER PROGRAMMES

For the model discussed in Section 3.2, the Rational Method and Manning's Formula computation on the overland as well as in the channel of rectangular and trapezoidal cross sectional, dimensionless nomographs were done in Microsoft Excel and FORTRAN.

3.4 USE OF SYNTHETIC UNIT HYDROGRAPH

In cases, where detailed physiographic analysis is possible instead of using quadrant 1 of the nomograph it would be better to use Synthetic unit hydrograph (SUH) approaches for the computation of peak discharge and subsequently the parameter q ($m^3/s/km^2/m$). The following approaches have been taken up for the development of SUH.

(i) Central water commission of India approach (Appendix-II).

(ii) Snyder's approach (Section 2.4.4).

(iii) Clark's Model (Clark, 1945) (Section 2.4.6).

CHAPTER 4 FLOOD CONTROL AND MANAGEMENT

The following aspects have been considered for flood control and management during the high flows of a particular river.

(i) Computation of flood discharges

(ii) Computation of water stages resulting into overtopping the banks.

(iii) Inundation schedule of the flood plain.

The following procedure has been adopted for the above three aspects.

4.1 COMPUTATION OF FLOOD DISCHARGES

As discussed in the previous section for the given rainfall values over the catchment the peak flood estimates have been arrived by using either quadrant 1 of the nomograph (Figure 3.7 and 3.8) or using Synthetic Unit hydrograph approaches (Section 3.4).

4.2 COMPUTATION OF WATER STAGES RESULTING INTO OVERTOPPING THE BANKS

As the river emerges out from the outlet of the catchment, its section get diversifies as shown in Figure 4.1(a). This causes reduction in velocities resulting into dropping of the sediments leading to the formation of levees. As the river stages overtop the levee water start entering into the flood plain. In order to protect larger areas to be submerged a proper design of the levees is required. This will allow water to enter into the flood plain through the recommended portion of the levee shown as hatched part W-W in Figure 4.1(b). Its design can be done on the lines of a broad crested weir as discussed in the following section.

Using the nomograph for different values of discharges Q (or q), the river stages h are computed for given values of roughness n. It is to be noted that slope of the river may not be a very sensitive parameter over a short stretch of length downstream of the outlet. A minimum stage $h_{\min}$ is proposed to maintain for perennial rivers from environmental consideration. As the river stages go up and cross the levee height (h_o) water is allowed to enter into the flood plain through the broad crested weir section W-W (Figure 4.1(a)).

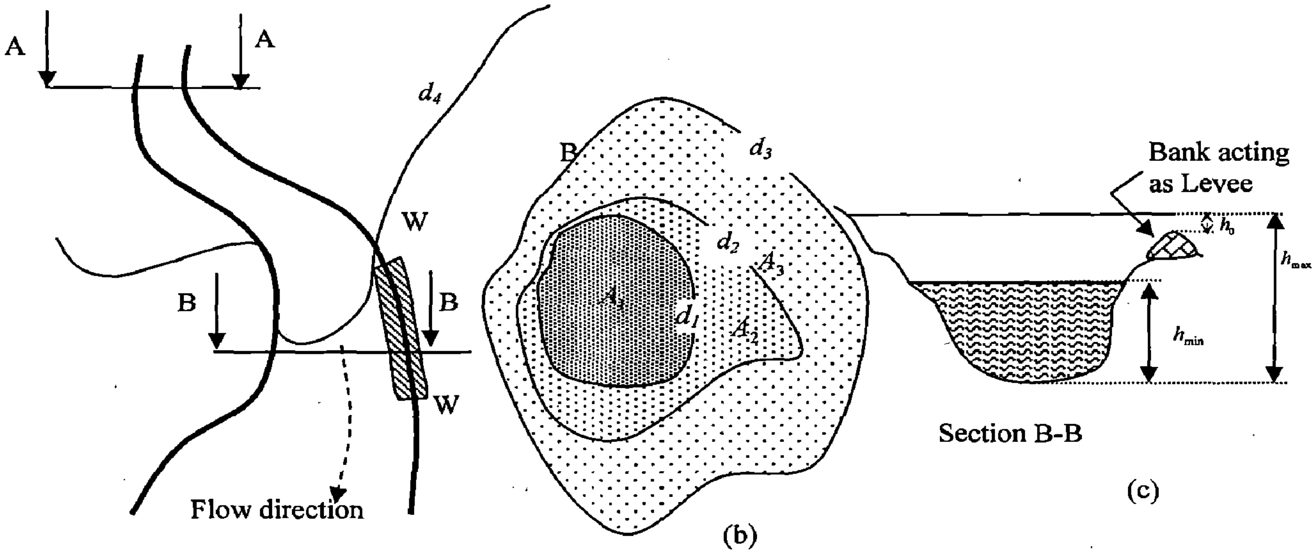

Figure 4. 1 Schematic diagram of channel bank acting as levee and probable flood plain area

4.3 INUNDATION SCHEDULE OF THE FLOOD PLAIN

The detailed description of a broad crested weir is given in Appendix III. As shown in Figure 4.1 (c), under a head h_0 water is allowed to enter through a levee length, L_w (i.e. W-W in Figure 4.1 (a) which is designed as a broad crested weir and having a width L_b which is at a height p above the river bed. In the relationships given in Appendix III, the parameter L_b is adopted in accordance with the field condition i.e. the top width of the levee. The length of the weir is so arrived at that it inundates the flood plain over designed period of time.

The inundation scheduled of the flood plain will be a function of the topography of flood plain beyond the levee (Figure 4.1(b)). The area-elevation storage curve is prepared for incremental heights of 0.25 m or so. If the areas of the depressions are A_1, A_2, A_3, ... , A_m ; the successive contours are d_1, d_2, d_3, ... , d_m and the intervals is Δd, the storage for the area to be inundated is worked out using (i) Prismoidal formula or (ii) Simpson formula or (iii) Trapezoidal formula as given in the Table 4.1 (Patra , 2002).

Table 4. 1 Formulae for calculating storage confined within the contours

Methods	Storage Calculations
Prismoidal formula	$V=\frac{\Delta d}{6}\{A_1+A_2+4A_m\}$ where $A_m=0.5(A_1+A_2)$
Simpson formula	$V=\frac{\Delta d}{3}\{A_1+A_4+...+A_m+2(A_1+A_3+...+A_{m-1})\}$
Trapezoidal formula	for two successive contours, $V=\frac{\Delta d}{3}\{A_1+A_2+\sqrt{(A_1A_2)}\}$ for three successive contours, $V=\frac{\Delta d}{3}\{A_1+4A_2+A_3\}$

For better accuracy, the contour interval should be as small as possible. In most cases among the three equations, the Prismoidal formula gives better results. The above formulas are meant for equal interval of contour. If contours are not at equal interval then following method may be adopted.

Volume of flooded water V' (m^3) is confined under a set of contours at interval d' (m) and covering areas A_1 and A_2, worked out as follows

$$V'=d'\{(A_1+A_2)\times 0.5\}\times 1000^2 \tag{4.1}$$

where, $d'=d_2-d_1$.

4.4 DESIGN LENGTH OF WEIR

If ΔQ is the discharge through the weir per unit length under the head h_0 above the weir and allowable time is T to fill the storage S then the length of the weir is worked out as given under.

$$L=\frac{S}{T\Delta Q} \tag{4.2}$$

The time of filling the storages can be worked out over shorter intervals of periods (ΔT) to cover the entire time span T. In this flood management plan, the downstream areas can be protected to the extent the duration of the flood above the crest level.

The above proposal can be suitably be briefed and its working may be understood as given in Figure 4.2. The two economic sectors namely Economic Sector-I and Economic

Sector-II need be protected from the floods produced by the upstream catchment. In between the catchment and the economic sectors the channel length A-B is constrained and its width is designed. Also, the two banks the levees are reinforced to channelize the flow. In order to allow only the safe flows. The flood plains on the two sides of the channel section A-B are utilized for diverting the excess flows to avoid problems in the economic sectors. For the purpose, in the levee on the two banks broad crested weirs may be provided. The excess flows above the safe river stage will enter into depressions. The water in the depressions may be used for recharging the aquifers down below and will form a part of natural recharging basin.

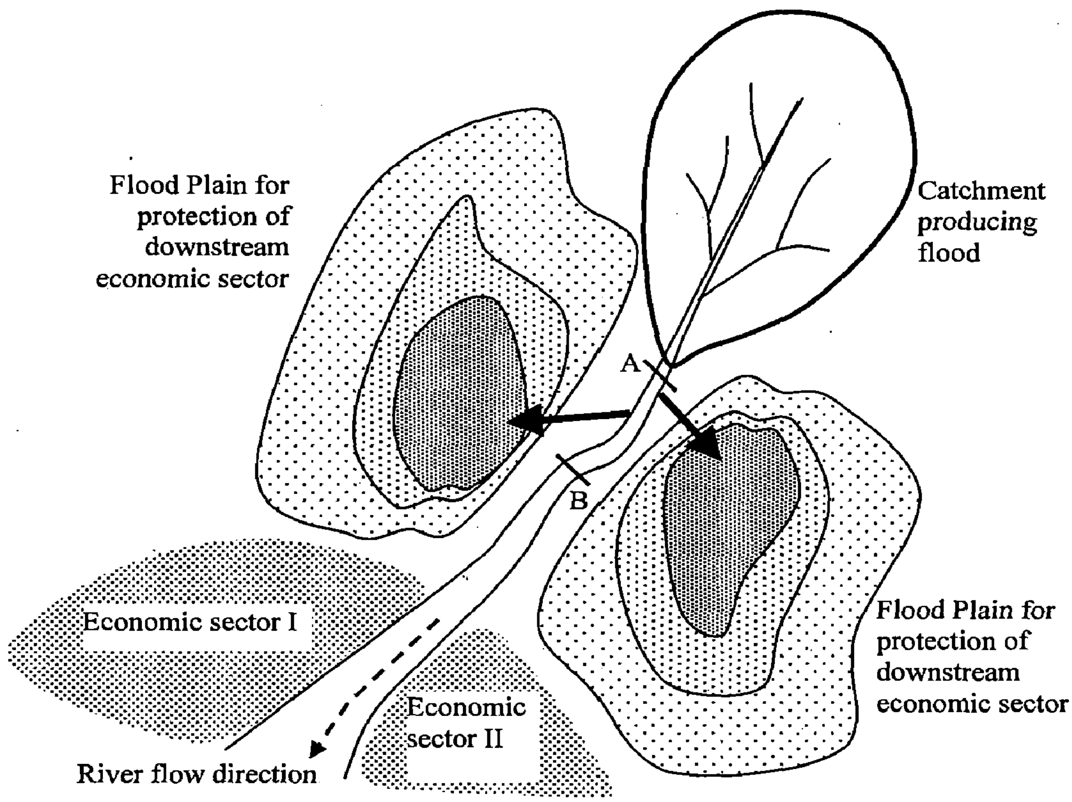

Figure 4. 2 Schematic diagram of flood protection scheme for economical sectors.

The above mentioned principle of flood control and management is illustrated and explained in details in the case study presented in the following section.

4.5 THE CASE STUDY

A case study has been taken up for a catchment which may or may not exist somewhere in India. It is shown in Fig 4.3. The upper most part shown as cross hatching produces flood which may cause damages in the economic sector I and economic sector II located southwards. In between the flood producing area and the economic sectors the river travels over a distance more than 75 km having depressions on the two sides of the channel. After the contributing area, the channel is approximately tamed by strengthen the levees and providing a channel width of 500 m. The levees have a section of 2.5 m in height, 2.0 m in the top width as shown in Figure 4.4(a). Within the levee section at a height from 1.0 m from the ground a broad crested weir having a length L_w equal to the width of the channel (Figure 4.4(b)) i.e. 500 m and a top width (L_b) of 2.0 m is provided on the two banks as the escape routes for the water when the river in high flows. On the two sides of the constructed section of the river A-B, the location of the overtopping weirs at A is shown in its expanded form as X-Y. The two weirs which may be provided with gates are shown at location P-Q and R-S. The position of the depressions for storing water on the two sides of the river along with the dimensions are shown in Figure 4.4(a). The location of broad crested weir in the levees is shown in Figure 4.4(b). The hydraulics of the diversion of the overtopping weirs is shown in Figure 4.4(c).

The physiographic parameters of the uppermost contributing catchment are given in Table 4.2. The hydraulic parameters as obtained from the nomograph (Section 3.2.2) for a peak discharge of 3555 m^3/s are given in Table 4.3. This peak discharge corresponds to an excess rainfall intensity of 20 mm/hr, which continues up to the time of concentration t_c equal to 5.0 hours for the drainage area 3200 km^2. Thus the hydraulic parameters obtained are given in Table 4.3. The design hydrograph so worked out is shown in Figure 4.5. Following the principle of triangular hydrograph, the peak is kept at the time of concentration i.e. 5.0 hours and the total time base has been calculated as t_c equals to 5.0 hours for the rising limb and 1.67 t_c i.e. equal to 8.0 hour for the recession part, i.e. making at total time base of 13.0 hours (approx).

Table 4. 2 Catchment physiographic parameters of the area contributing to runoff

Area	**Channel bottom width**	**Channel length**	**Slope of the channel**	**Runoff coefficient**	**Excess rainfall intensity**	**Roughness coefficient**	**Overall bank height**	**bank height**	**Crest width**
(A_d)	(b)	(L)	(S_c)	(C)	(i)	(n)		(p)	L_b
(km^2)	(m)	(km)	(%)		(mm/hr)		(m)	(m)	(m)
3200	**500**	**60**	**0.1**	**0.2**	**20**	**0.015**	**2.5**	**1.0**	**2.0**

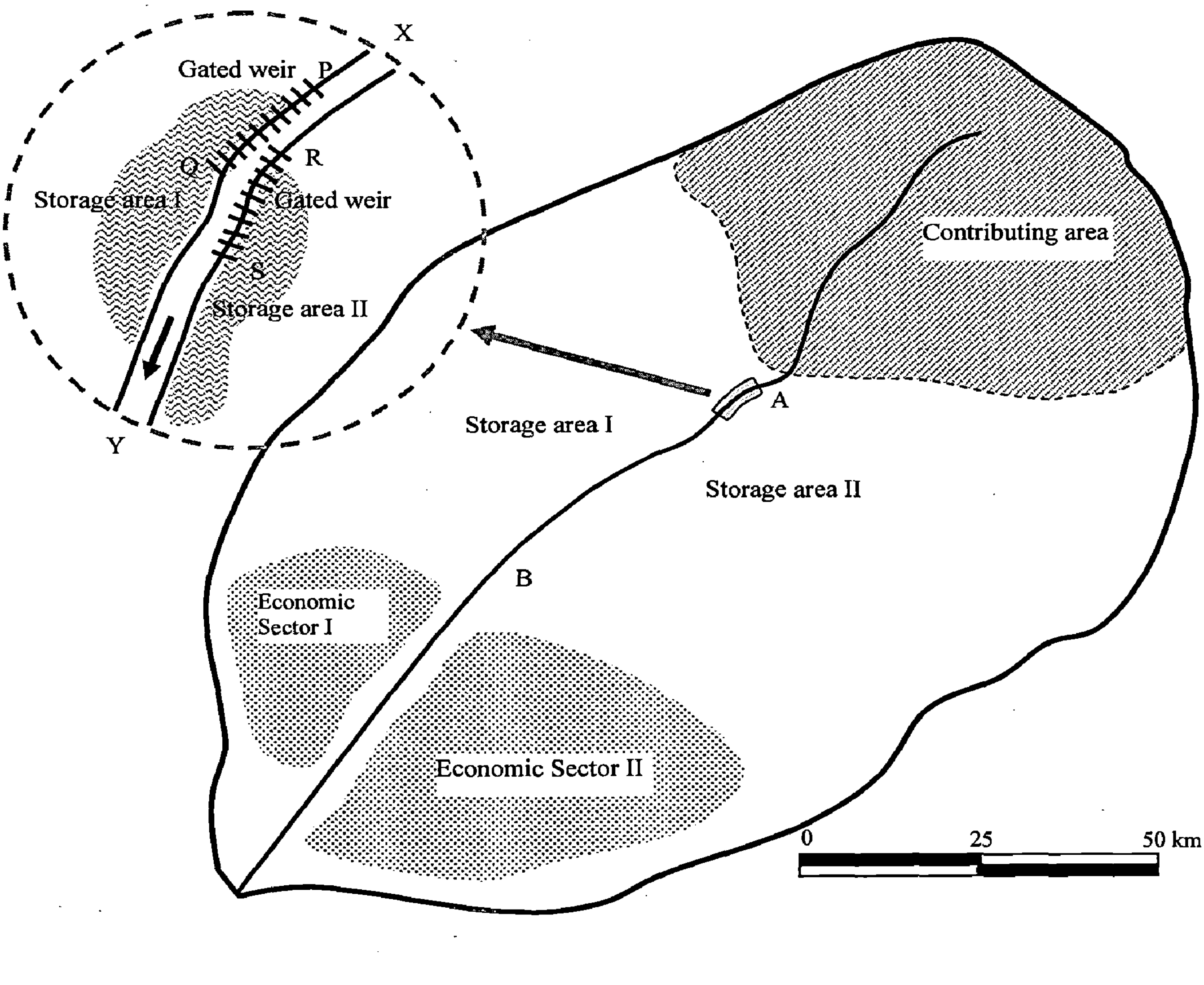

Figure 4. 3 Study area of case study

Table 4. 3 Hydraulic parameters obtained from the nomograph

Peak discharge (Q_p) (m^3/s)	Water depth in the channel (h_c) (m)	Manning's Velocity (V) (m/s)	Froude Number (Fr)	Time of concentration (t_c) (min)
3555	2.1	3.43	0.76	300

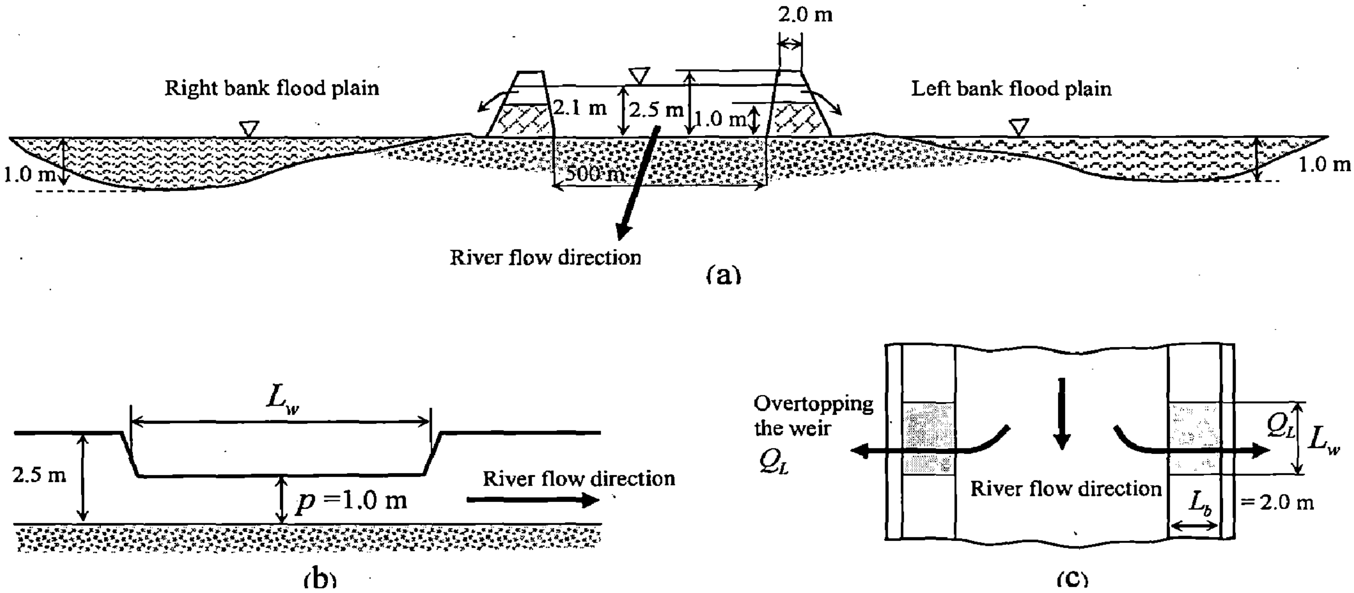

Figure 4. 4 Schematic representation of river flow system with design of weir as levee

Table 4. 4 Weir length calculation

Crest width L_b (m)	Design weir length L_w (m)	bank height p (m)	Water depth in the channel h_c (m)	Excess depth of water above safe level h_0 (m)	Weir discharge at each side of the bank Q_L (m^3/s)
(1)	(2)	(3)	(4)	(5) = (4) - (3)	(6)
2.0	500.0	1.0	2.1	1.1	1037.1

Note: Values of columns (1), (2) and (3) are assumed and adjusted. Column (4) is worked out from the nomograph (Table 4.3 and Equation 3.16). Value in Column (6) is computed using Equation III-3 through Equation III-8 .

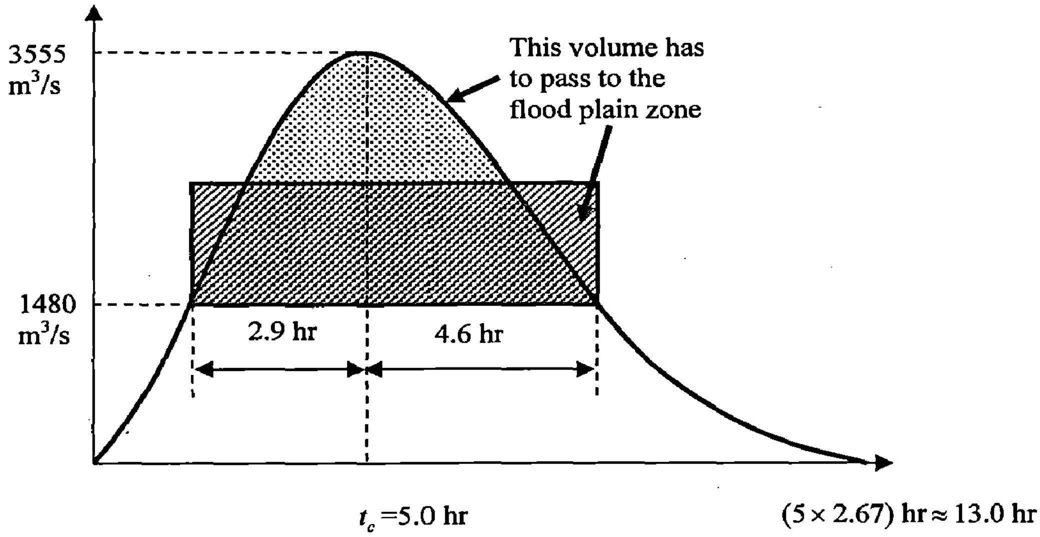

Figure 4. 5 Design hydrograph for case study

Diversion of flows through weir

The stage discharge to pass through the channel was worked out is 1480 m^3/s against the flow peak of 3555 m^3/s. Thus the dotted portion of the hydrograph given in Figure 4.5 need be diverted to the flood plain storage areas I and II to save the economic areas. This volume has been approximated through the rectangle as shown in the Figure 4.5. Its time base is worked out proportionately and is 7.5 hours. This gives a total volume of 28×10^6 m^3. Volume to be diverted towards the two flood plain will be 14×10^6 m^3. Assuming an allowable depth of inundation as 1.0 m on each side, the inundated water will flood 14×10^6 m^2 i.e. 14 km^2.

Concludingly, it may be remarked that in all 28 km^2 area will be required to protect the economic area for the assumed figure of 20 mm/hr rainfall excess intensity which continued 5.0 hours. This area will serve as recharging zone for the ground water regime.

Calculations

Discharge from the contributing catchment = 3555 m^3/s

Total discharge through the weir on the two sides of the banks = 2075 m^3/s (for 1.1 m head above the crest level of the weir, Appendix III, Equation III-3 through Equation III-8).

Thus, the safe discharge in the channel = (3555-2075) m^3/s =1480 m^3/s.

Using Triangular hydrograph concept, base time is obtained as 13 hr.

From 0 to 2075 m^3/s volume will pass to flood plain within $\frac{2075}{3555} \times 13 = 7.5$ hr.

Within 7.5 hr, (2075/2) m^3/s = 1037.5 m^3/s discharge will pas to flood plain zone.

Total volume = $1037.5 \times 7.5 \times 3600\ m^3 = 28 \times 10^6\ m^3$.

Volume for each side of flood plain = $14 \times 10^6\ m^3$.

So, approximately 14 km^2 area is required with inundation depth of 1.0 m in order to protect the economic sector downstream.

4.6 COMPUTER PROGRAMMES

Area-volume-elevation calculation is performed using GIS and a separate Microsoft Excel and FORTRAN programme. Broad Crested weir design has also been calculated in a separate Microsoft Excel and FORTRAN programme (Appendix IV).

CHAPTER 5 APPLICATIONS OF PROPOSED CONCEPTS ON SOLANI RIVER CATCHMENT

During the last fifty years, the small city of Roorkee covering an area of approximately 4 sq. km has expanded in all the directions. Now Roorkee and the adjoining areas cover more than 16 sq. km. Its population has increased from nearly 66,000 to 2,00,000 in the last five decades. During the last ten years and especially after the carving of Uttarakhand state, the adjoining areas are being developed as the industrial belt in which many industries have already come up and some are likely to come up in near future. The Solani river originates from the Lower Himalayas known as Shiwalik. It covers a distance nearly 60 km till it approaches Roorkee as shown in Figure 5.1. At Roorkee two aqueducts exists. The old aqueduct is 150 years old and the new aqueduct has been constructed recently and is in operation during the last 10 years. Due to this vast expansion the following hydrologic problems have surfaced which are likely to become more acute in years to come.

5.1 WATER SUPPLY

The entire city of Roorkee and the adjoining areas are dependent for the domestic, commercial and industrial water supply from the aquifers. A considerable stretch of the Ganga canal has already been lined in order to reduce seepage. It is likely that the entire canal bed during its passage through Roorkee will be lined in future. This will drastically reduce the seepage which recharges the ground water regime resulting into near water mining conditions in and around Roorkee. There is a need for planning of recharge of ground water through the Solani flows.

5.2 PROTECTION FROM FLOODS

Since more and more areas are coming under construction, the percentage of built-up areas in an around Roorkee has increased over 4 to 6 times. This is increasing the runoff from the urban sub-watersheds. Also, due to mismanagement of Shiwalik watershed, the runoff into the Solani river has considerably increased and it is further likely to increase in future.

5.3 THE WATER QUALITY ASPECTS

At present, the domestic waste is not being treated and is being directly released in to the Solani river. Solani being a tributary of Ganga river it ultimately pollutes the Ganga water which is the lifeline of Indian mainland in the north. Also, releases of the chemicals into the Solani river are going to be a cause of concern. It is expected that in due course these will be treated along with the domestic waste water and the safe water will be released into the Solani river.

In order to tackle the above mentioned present and futuristic problems, the following plan has been envisaged.

5.4 SOLANI RIVER DEVELOPMENT PLAN AT ROORKEE

It is proposed that in order to tackle the aforesaid hydrologic problems, as shown in Figure 5.2, the Solani river be tamed over a distance of 6 km around Roorkee. The 4 km stretch (or even larger) may be upstream of the Solani aqueduct. Downstream of the aqueduct the stretch may extend up to 2 km or more. This stretch of river will receive the waste as shown through the routes P-Q-R-S-T-U etc. Under the proposed plan, the span of the Solani river may be constricted to a width of 300 m guided by the two levee which may be 1.5 m above the bed having a top width of 2 m. Beyond this span of levees on the two sides, two auxiliary channels I and II each having a width of 50 m be provided which will be guided by another pair of levees which be above 3.5 m above the ground. The main channel will carry the normal flows of the Solani river. However, during the high flows water may overtop through the broad crested weir to be provided at regular intervals to allow high flows to enter into the auxiliary channels. In the downstream section, the auxiliary channels may be provided gates which are shown as A-B, C-D. The waste water to be received in the auxiliary channels will be flushed out during the first floods to be received in the Solani. Towards the end of the rainy season the auxiliary gates will be closed and water may stand up to a height of 1.5 m in the 6 km long stretch which will ultimately recharge the ground water. During the dry season, the treated waste water coming to the auxiliary channels will recharge the aquifers. The proposed plan needs a careful scrutiny from the considerations of other environmental factors. Its validity may also be checked through hydraulic model study of the Solani river stretch. However, the surface hydrologic regime study will require the following investigations and computations.

Flood computations pertaining to peak discharge, maximum stage, and volume associated with the flood events corresponding to desired return periods of rainfalls, (i.e.

100 years) need be arrived at. Since no measured flow data is available for the Solani flows SUH approach has been adopted for its computation. These aspects have been discussed in the forthcoming sections.

5.5 SUH COMPUTATIONS FOR THE SOLANI RIVER AT ROORKEE

The following three methods have been used for working out design UH of Solani river at Roorkee.

(i) CWC recommended procedure for a 2-hour Synthetic Unit Hydrograph.

(ii) Snyder's approach in combination with Triangular UH (TUH) concept.

(iii) Clark's model to compute Instantaneous Unit Hydrograph (IUH).

All the above mentioned methodologies require a careful study and analysis of the Solani river catchment ($77^{\circ}45'22''E$, $30^{\circ}16'21''N$ to $78^{\circ}00'30''E$, $29^{\circ}53'00''N$). The location of Solani river catchment in India is shown in Figure 5.3.

A shuttle radar topography mission (SRTM) digital elevation model (DEM) data with a resolution of 90 m (available on http://srtm.csi.cgiar.org/) has been used for drawing the catchment plan of river Solani upstream of Roorkee township using ARC/INFO GIS [ESRI Inc., 2005] software. The same is given in Figure 5.1. Using the technology semi-urbanized watershed around the township was explored and the same is given in Figure 5.4. Accordingly Indian meteorological department (IMD)-CWC classification, the Solani river catchment belongs to hydro-meteorological subzone 1(e) (Figure 5.3).

The physiographic parameters i.e. catchment area A, length of the longest stream L, and length of the stream starting from outlet to the nearest point of the center of gravity (C.G.) of the catchment $\bar{L}$ for the Solani river catchment at Roorkee site are given in Table 5.1. The physiographic parameter S i.e. the equivalent slope of Solani stream works out to be 3×10^{-4} m/m or ft/ft (Appendix V).

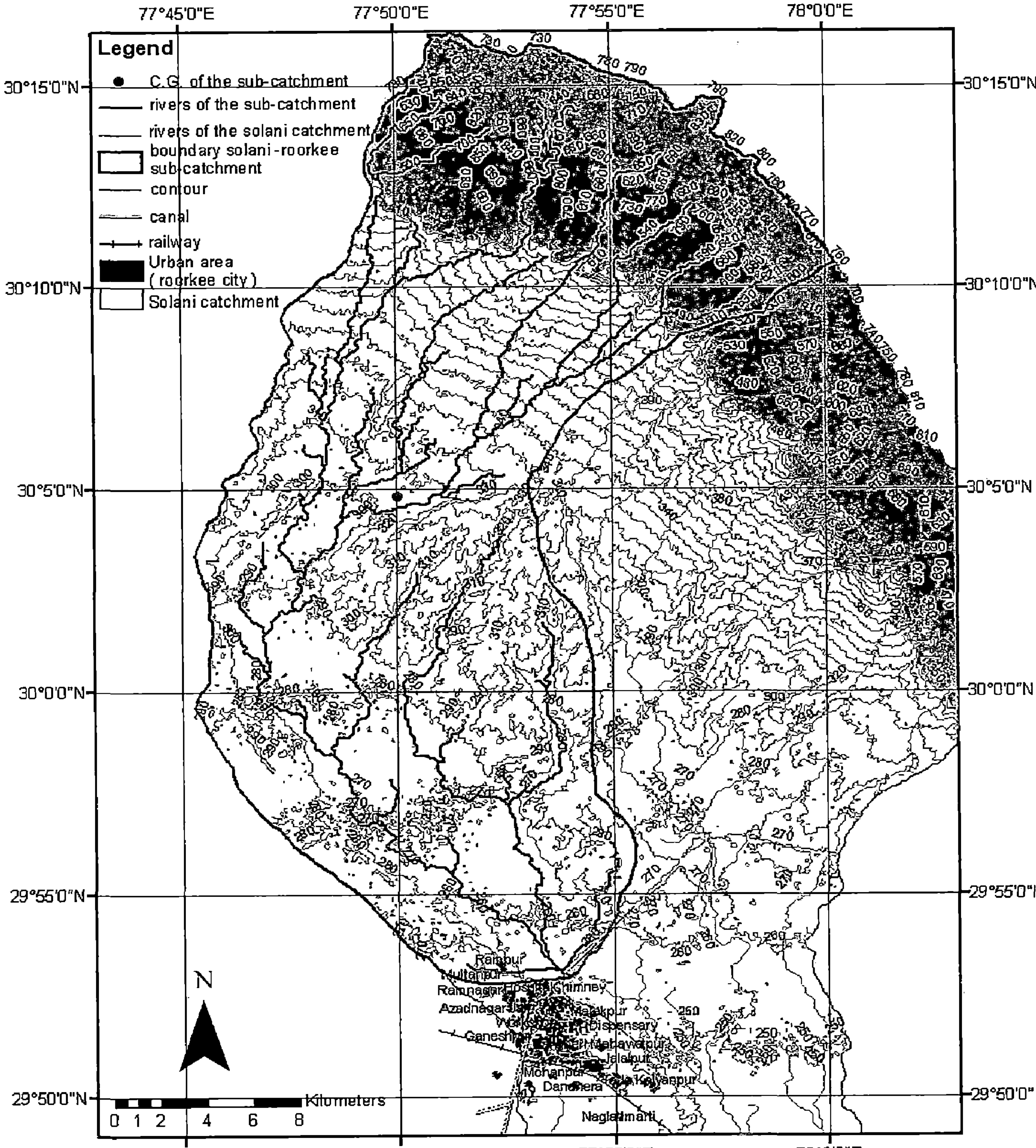

Figure 5. 1 SRTM based catchment plan of river Solani upstream of Roorkee township

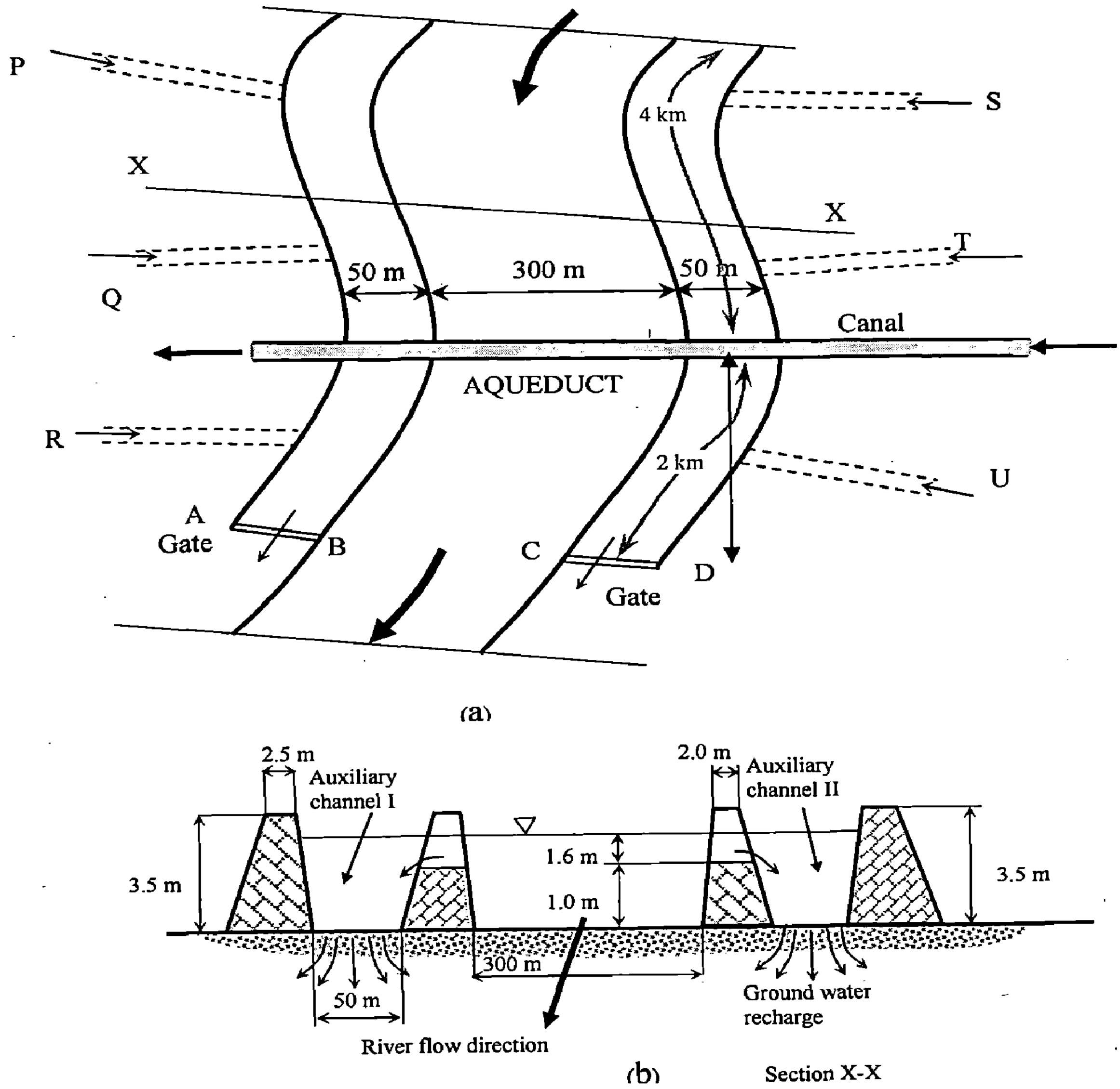

Figure not to Scale

Figure 5. 2 Schematic diagramme of proposal for the development of Solani river system

Table 5. 1 Physiographic parameter values of Solani river catchment

Parameters	S.I Unit	F.P.S. unit
A	546.67 km^2	213.67 $mile^2$
L	59.2 km	37 mile
$\overline{L}$	33 km	20.63 mile

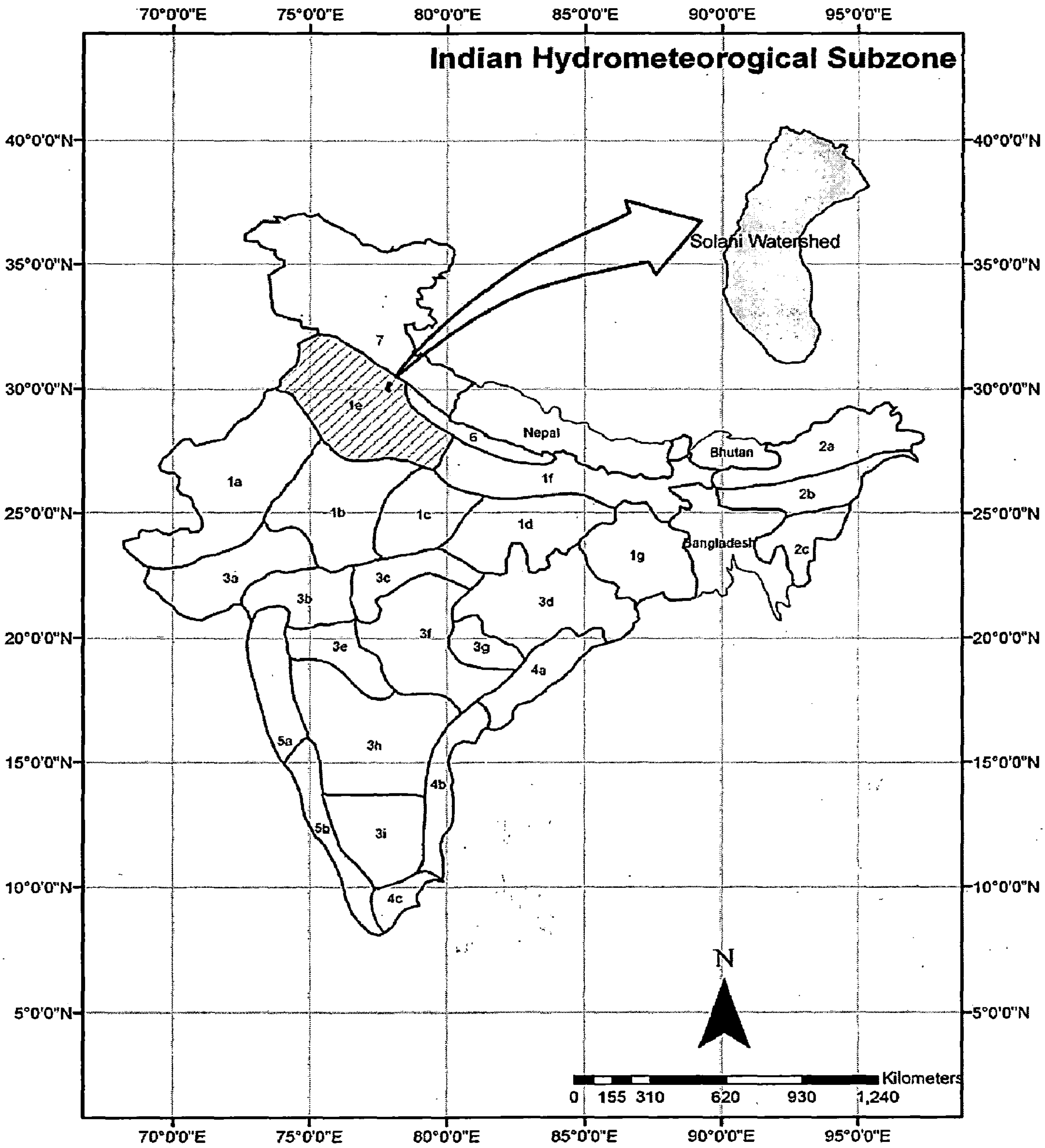

Figure 5. 3 Location of Solani catchment in Indian hydro-meteorological subzone.

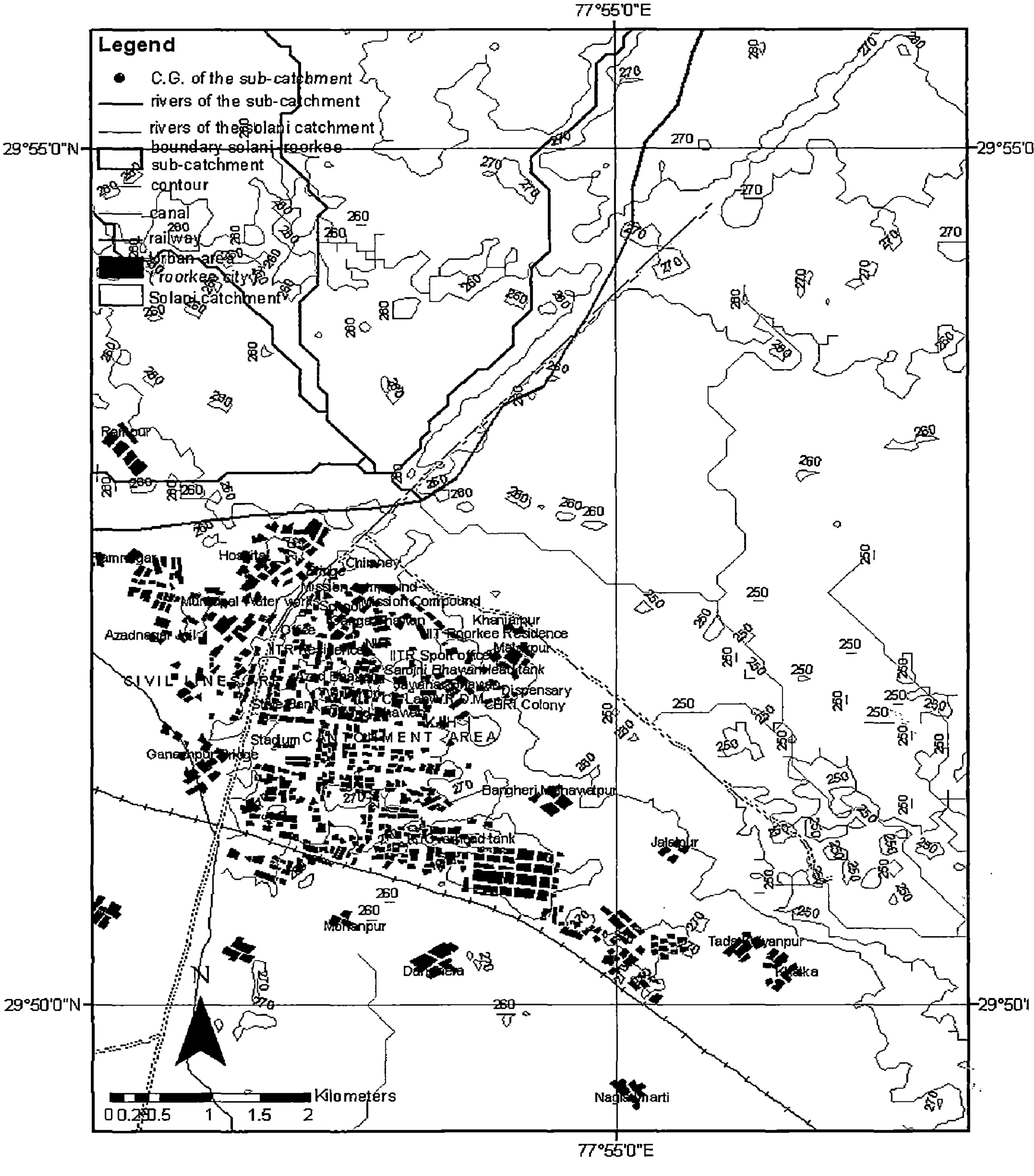

Figure 5. 4 Semi-urbanized watershed of part of Solani catchment in the vicinity of Roorkee township.

5.5.1 CWC recommended 2 hour SUH computation

As mentioned in Appendix II, the 2 hour SUH parameters for Solani river at Roorkee are given in Table 5.2.

Table 5. 2 Computed values of 2 hour SUH parameters for Solani rivers at Roorkee

tr (hr)	q_p (cumec/km^2)	Qp (cumec)	tp (hr)	Tm (hr)	T_B (hr)	W_{50} (hr)	W_{75} (hr)	W_{R50} (hr)	W_{R75} (hr)
2	0.097	52.92	20.97	21.9733	82.89	22.37	11.42	6.75	3.84

Plotting of 2 hour SUH parameters of Solani at Roorkee

As proposed in the CWC method, for Solani river, the ordinates of 2 hour SUH parameters have been calculated. The time widths at 50% and 75% of peak discharge ordinates have been worked out as given in Appendix II, Figure II.1. The computed values of these parameters are given in Table 5.3.

Table 5. 3 Computed time widths of 2 hour SUH parameters of Solani at Roorkee (notations as shown in Appendix II, Figure II.1)

Percentage value of Q_p (%)	Time (T) (hr)	Rising Limb (m^3/s)	Time (TT) (hr)	Recession Limb (m^3/s)
$50\ Q_p$	15.22	26.46	37.59	26.46
$75\ Q_p$	18.13	39.69	29.55	39.69
$100\ Q_p$	21.97	52.92	21.97	52.92

Using the parameters obtained in Table 5.3, the 2 hour SUH is plotted (Figure 5.5) and UH ordinates at 2 hour interval are read. Depth under the SUH is worked out to be 1.1 cm, which is 10 percent more. In order to arrive at the unit volume (= 1 cm $\times$ A), a cross check is done by making use of the properties of 'triangular unit hydrograph' (TUH) as well as by following the Snyder's approach (1938).

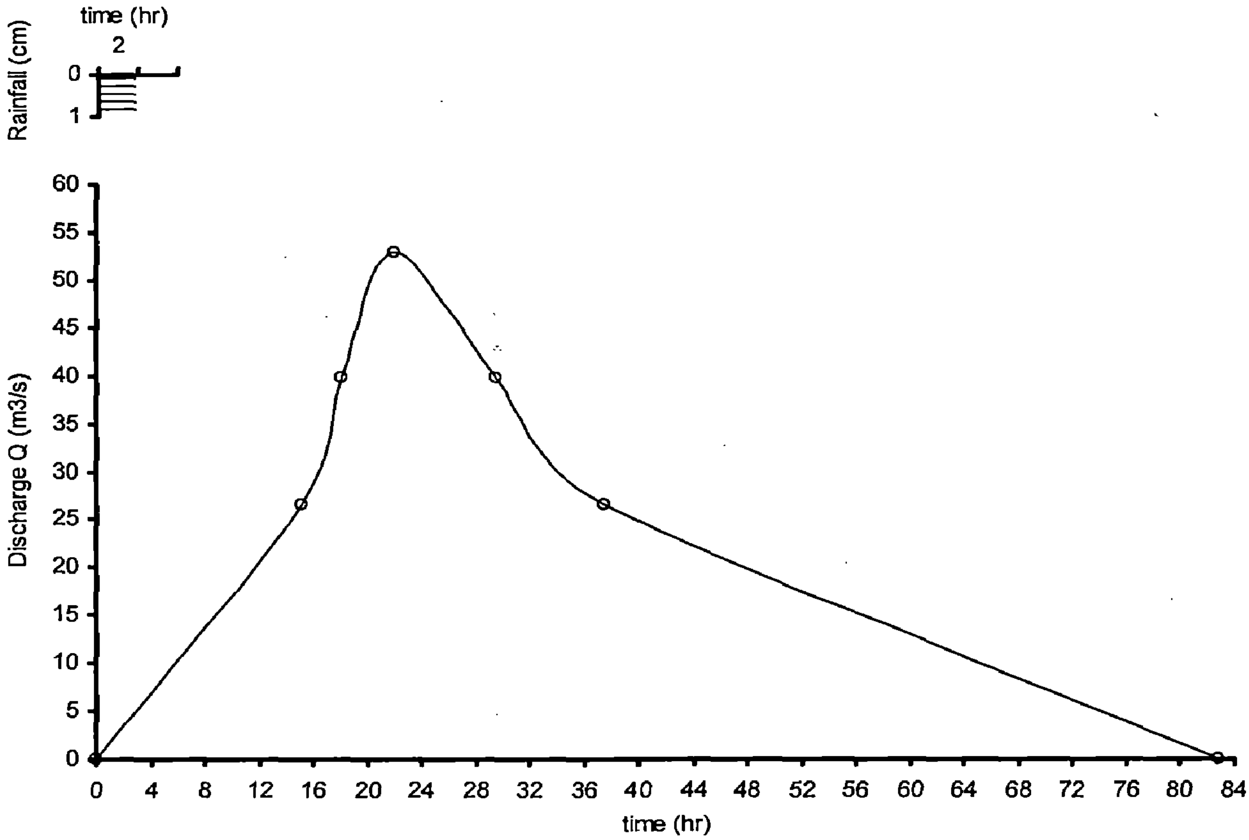

Figure 5. 5 Initially calculated 2 hour UH for Solani catchment (CWC method).

5.5.2 Cross checking by 'triangular unit hydrograph (TUH) method'

Conventionally the time base T_B of a triangular unit hydrograph is adopted as follows.

$$T_B = 2.67 \times T_m \tag{5.1}$$

For the Solani river catchment this worked out to be as under

$T_B = 2.67 \times 21.97 = 58.66$ hr ≈ 60 hr

For the computed peak discharge Q_p (52.9 cumec) the same unit volume has been calculated as follows

$$\text{Unit volume} = \frac{1}{2} \times T_B \times Q_p \tag{5.2}$$

where T_B is in sec and Q_p is in cumec

$$\text{Volume} = \frac{1}{2} \times 60 \times 3600 \times 52.9\,\text{m}^3 = 5.71\ \text{MCM}$$

This approximately matches reasonably well with the unit volume (= 1 cm $\times$ A = 1 cm $\times$ 546.7 km^2 = 5.467 MCM). The SUH for the Solani river at Roorkee after some correction in the recession part is shown in Figure 5. 6 and its ordinates are given in Table 5.4.

Table 5. 4 Corrected 2 hour SUH ordinates for Solani river at Roorkee by CWC method.

Time (hr) (i)	0	2	4	6	8	10	12	14	16	18	20	22	24	26	28	30
Corrected SUH Ordinates (CWC) (ii)	0	4	7.8	11.7	16.2	20.8	26.2	32	39	46.5	52	54	53	49.5	45.2	40.6

(i)	....	32	34	36	38	40	42	44	46	48	50	52	54	56	58	60
(ii)	....	36.2	32.6	29.1	25.9	23	20.5	17.9	15.4	12.8	10.2	7.8	5.5	3.2	1.5	0

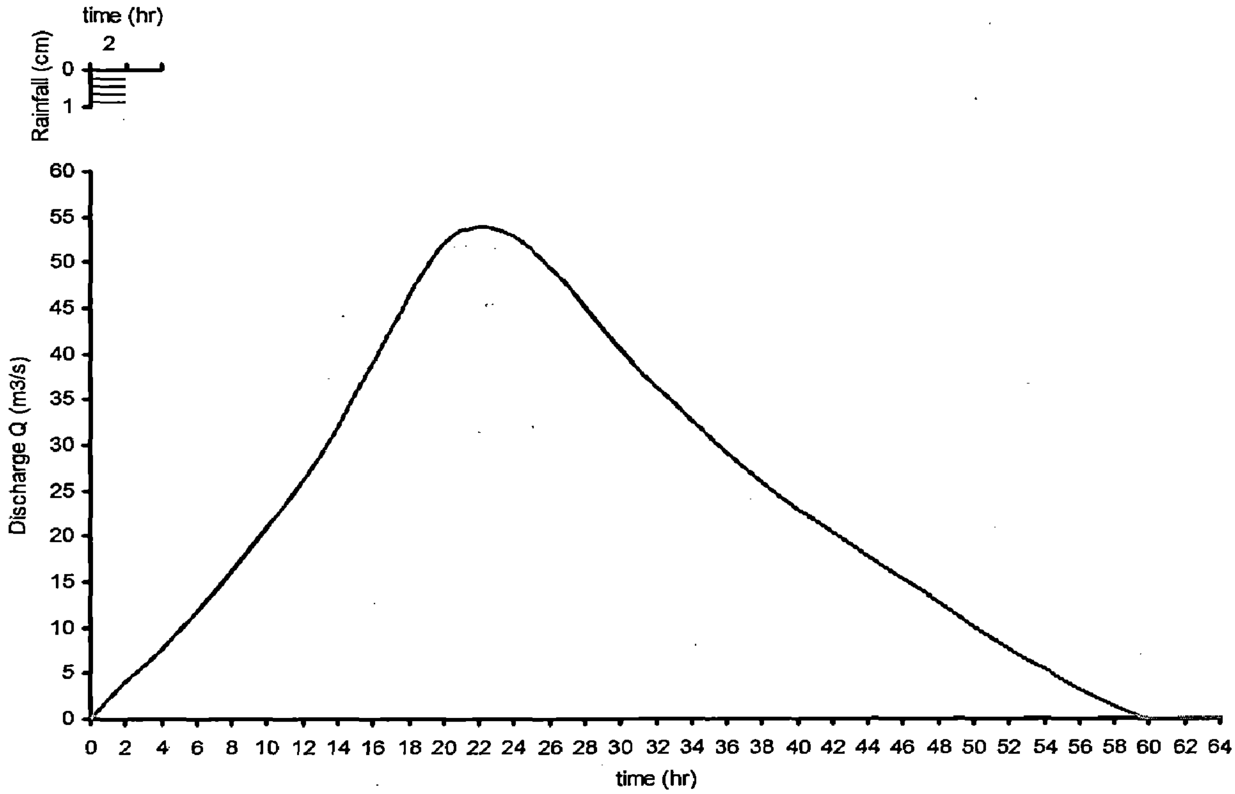

Figure 5. 6 Corrected 2 hour unit hydrograph for Solani river catchment

5.5.3 Synthetic unit hydrograph computations using 'Snyder's method'

The computed physiographic parameters for the Solani river are given in Table 5.1 and Table V.1 (Appendix V). Adopting the coefficients C_t as 0.9 [range 0.7 to 1.0 and 1.8 to 2.2 (Linsley et al., 1949)] and C_p as 0.6 [range 0.35 to 0.5 and 0.56 to 0.69 (Linsley et al., 1949)] the lag time t_p and the peak ordinate Q_p of SUH are worked out as follows.

$$t_p = C_t\left(\frac{L\bar{L}}{\sqrt{S}}\right)^{0.3} \text{ hr} \tag{5.3}$$

where, $L, \bar{L}$ are in mile, and S is in ft/ft. The value of t_p for Solani river catchment at Roorkee is worked out as 22.26 hr.

$$Q_p = \frac{C_p \times 640 \times A}{t_p} \text{ cusec} \tag{5.4}$$

where A is in mile^2 and t_p is in hr. The value of t_p for Solani river catchment at Roorkee is worked out as 2150.13 cusec or 59.73 cumec. Also, the duration of SUH t_r, time to peak T_m and time base T_B are adopted as follows

$$t_r = t_p / 5.5 \tag{5.5}$$

$$T_m = t_r / 2 + t_p \tag{5.6}$$

$$T_B = 2.67\, T_m \tag{5.7}$$

where t_p is 22.26 hr.

The worked out parameters are summarized in Table 5.5.

Table 5. 5 Worked out SUH parameters of Solani river.

Sl. No.	parameters	Symbol	Values	Remarks
1	Peak discharge (m^3/s)	Q_p	59.73	
2	Time lag (hr)	t_p	22.26	
3	Duration of SUH (hr)	t_r	4.05	the value of t_r is adopted as 4 hour
4	Time to peak (hr)	T_m	24.26	
5	time base (hr)	T_B	64.77	

Using the above parameter the 4 hour SUH is plotted in Figure 5.7 and its ordinates are given in Table 5.6.

Table 5. 6 Worked out 4 hour SUH ordinates of Solani river by Snyder's method

Time (hr) (i)	0	2	4	6	8	10	12	14	16	18	20	22	24	26	28	30	32
4 hr SUH ordinates (m^3/s) (Snyder) (ii)	0	5	9.4	14	19.2	25	30.5	36.4	41.4	47.2	52.5	57	59.7	57.5	51.8	44	38
(i)	34	36	38	40	42	44	46	48	50	52	54	56	58	60	62	64	66
(ii)	33	28.5	24	20	15.2	12	9	7	5	3.5	2.8	2	1.8	1.2	1	0.5	0

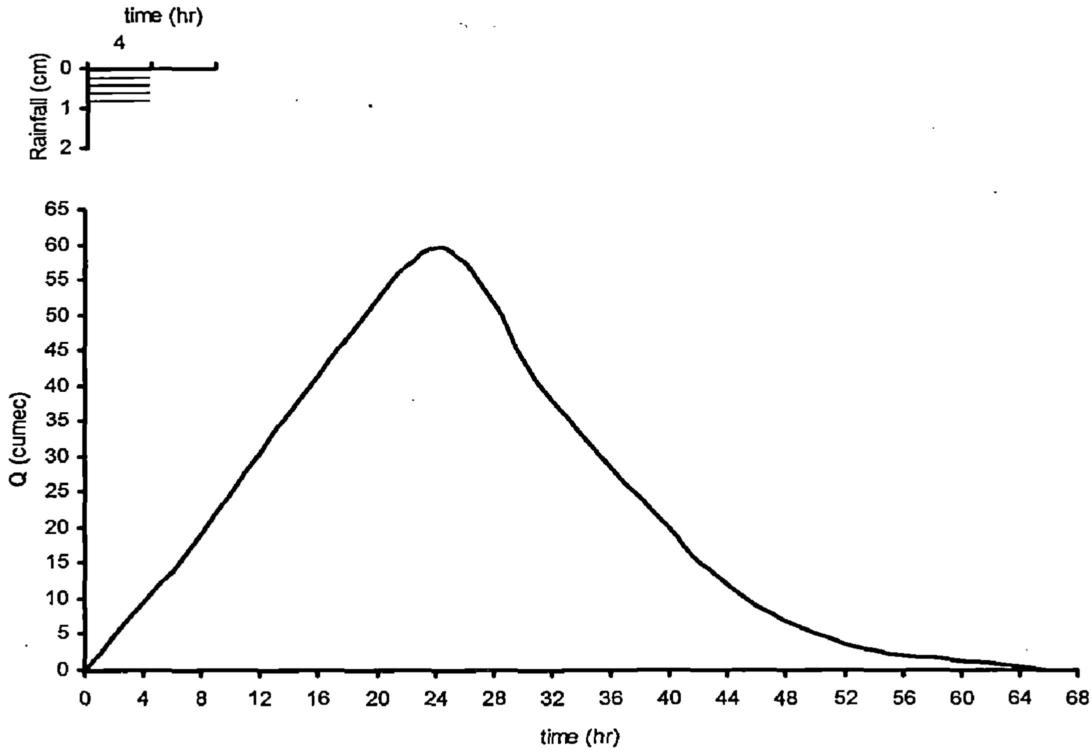

Figure 5. 7 Four hour SUH using the Snyder's SUH method

5.5.4 Adoption of synthetic unit hydrograph

The unit hydrograph arrived at by the CWC approach is having the unit duration of 2 hour whereas the SUH by Snyder's approach shows the unit duration as 4 hour. Using the S-hydrograph approach and the data as Table 5.6, a 2 hour SUH is worked out and its computations are shown in Table 5.7. The SUH so computed is given in Figure 5.8 and its ordinates are mentioned in Table 5.8.

Table 5. 7 Two hour SUH (Snyder's Method) computations using S-hydrograph approach

Time (hr)	Adjusted 4 hr Q (cumec)	S-curve ordinates	S-curved lagged by 2hr	(iii) - (v)	2 hr UH ordinates (=(v)/(2/4))	2 hr UH (correc) ordinates
(i)	(ii)	(iii)	(iv)	(v)	(vi)	(vii)
0	0	0	0	0	0	0
2	5	5	0	5	10	6.2
4	9.4	9.4	5	4.4	8.8	11.5
6	14	19	9.4	9.6	19.2	17
8	19.2	28.6	19	9.6	19.2	23
10	25	44	28.6	15.4	30.8	28
12	30.5	59.1	44	15.1	30.2	34
14	36.4	80.4	59.1	21.3	42.6	39.2
16	41.4	100.5	80.4	20.1	40.2	46
18	47.2	127.6	100.5	27.1	54.2	52

(Continued)

20	52.5	153	127.6	25.4	50.8	58.5
22	57	184.6	153	31.6	63.2	**61.5**
24	59.7	212.7	184.6	28.1	56.2	59
26	57.5	242.1	212.7	29.4	58.8	53
28	51.8	264.5	242.1	22.4	44.8	47
30	44	286.1	264.5	21.6	43.2	41
32	38	302.5	286.1	16.4	32.8	36
34	33	319.1	302.5	16.6	33.2	30
36	28.5	331	319.1	11.9	23.8	25.5
38	24	343.1	331	12.1	24.2	21
40	20	351	343.1	7.9	15.8	17
42	15.2	358.3	351	7.3	14.6	13.2
44	12	363	358.3	4.7	9.4	9.4
46	9	367.3	363	4.3	8.6	7
48	7	370	367.3	2.7	5.4	5
50	5	372.3	370	2.3	4.6	4
52	3.5	373.5	372.3	1.2	2.4	3
54	2.8	375.1	373.5	1.6	3.2	2.5
56	2	375.5	375.1	0.4	0.8	1.8
58	1.8	376.9	375.5	1.4	2.8	1.2
60	1.2	376.7	376.9	-0.2	-0.4	1
62	1	377.9	376.7	1.2	2.4	0.5
64	0.5	377.2	377.9	-0.7	-1.4	0
66	0	377.9	377.2	0.7	1.4	0

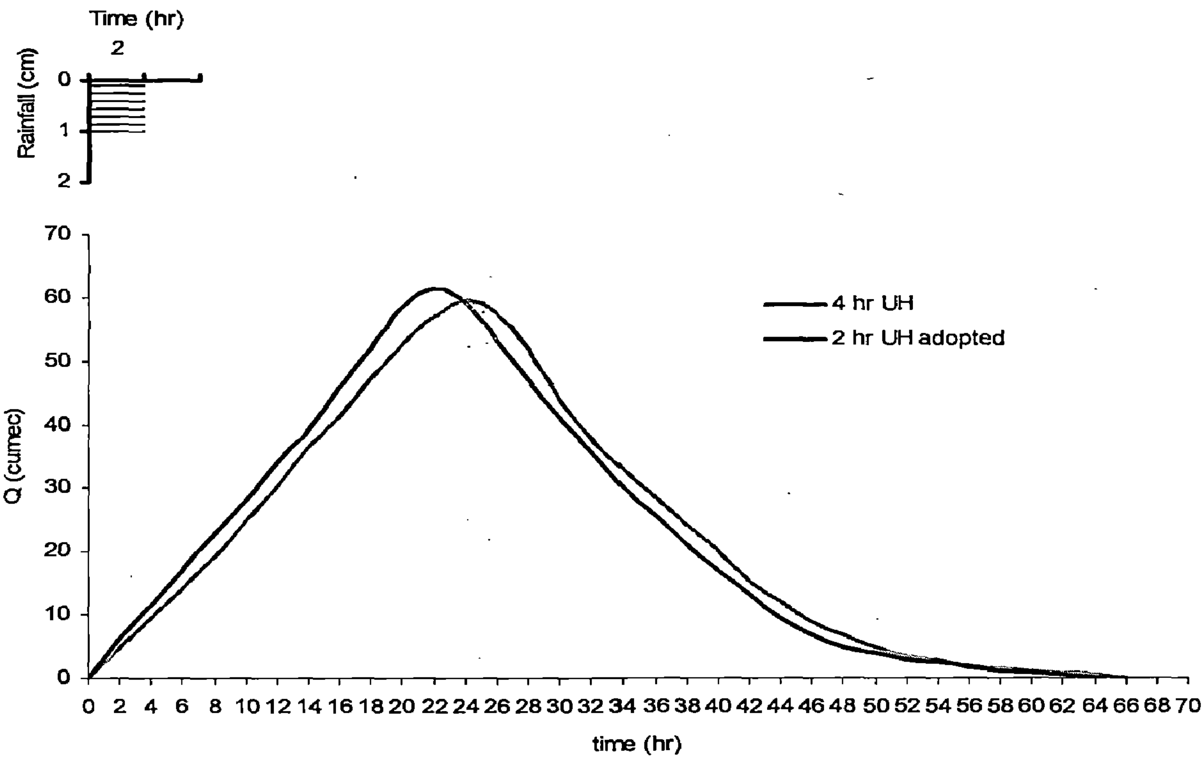

Figure 5. 8 Adopted 2 hour SUH using the Snyder's SUH method

Table 5. 8 Two hour SUH ordinates by using Snyder's method

Time (hr) (i)	0	2	4	6	8	10	12	14	16	18	20	22	24	26	28	30	32
2 hr SUH ordinates (m^3/s) (Snyder) (ii)	0	6.2	11.5	17	23	28	34	39.2	46	52	58.5	61.5	59	53	47	41	36

(i)	.....	34	36	38	40	42	44	46	48	50	52	54	56	58	60	62	64
(ii)	.....	30	25.5	21	17	13.2	9.4	7	5	4	3	2.5	1.8	1.2	1	0.5	0

5.5.5 Two hour UH computations using Clark's method

For the Solani river catchment instantaneous unit hydrograph (IUH) has also been developed using Clark's model. The time of concentration t_c for this method has been obtained from the known hydrograph (i.e. the UH for Solani river at Roorkee developed earlier). The time interval between the end of the rainfall excess and the point of second point of contra flexion of the Design UH is taken as t_c (Figure 5.9).

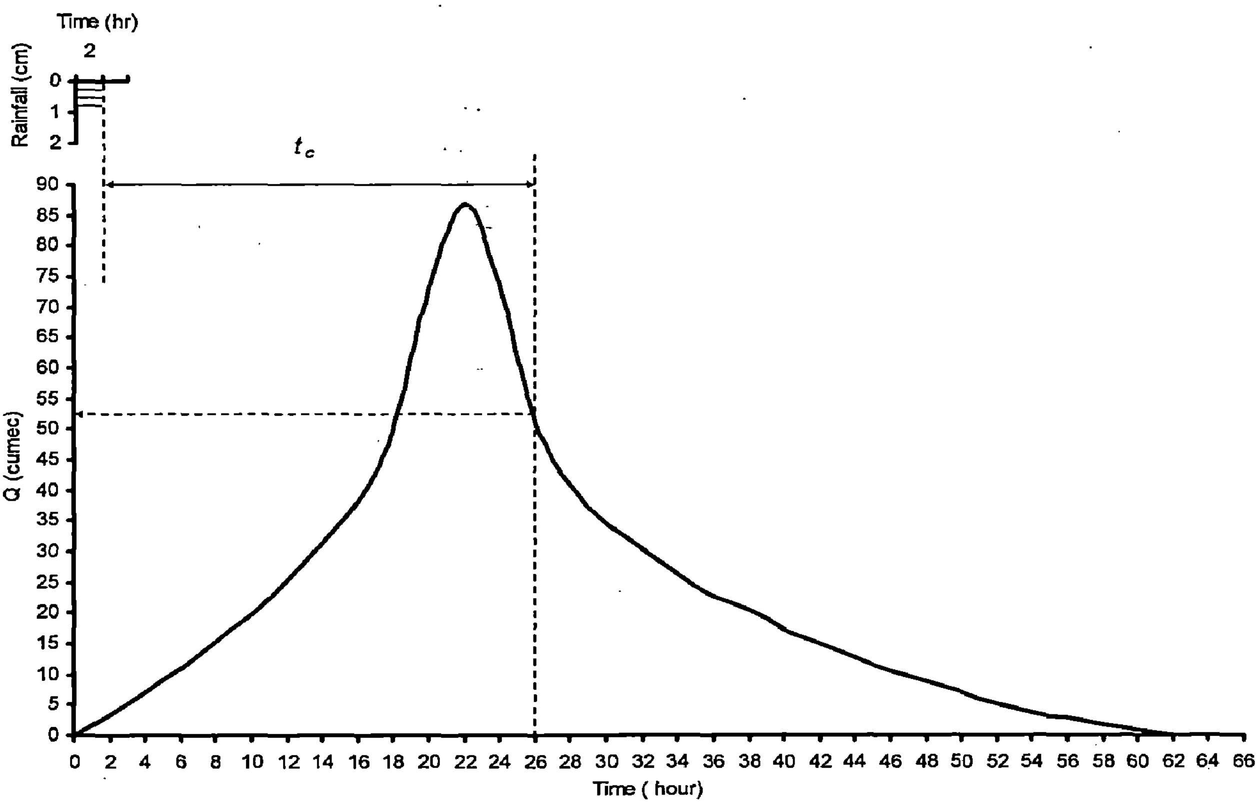

Figure 5. 9 Time of concentration for the Clark's method.

The time of concentration is found to be 24 hour (= 26 – 2 hour), Twelve inter-isochronal areas have been worked out for developing the translation hydrograph (Figure 5. 10). For an impulse input of unit depth the computation for translation hydrograph are given in Table 5.9.

To obtain the IUH for Solani river, the translation hydrograph is to be routed through linear reservoir ($S = KQ$) with storage coefficient as K. The value of K has been worked out to be 13 hours (Appendix VI).

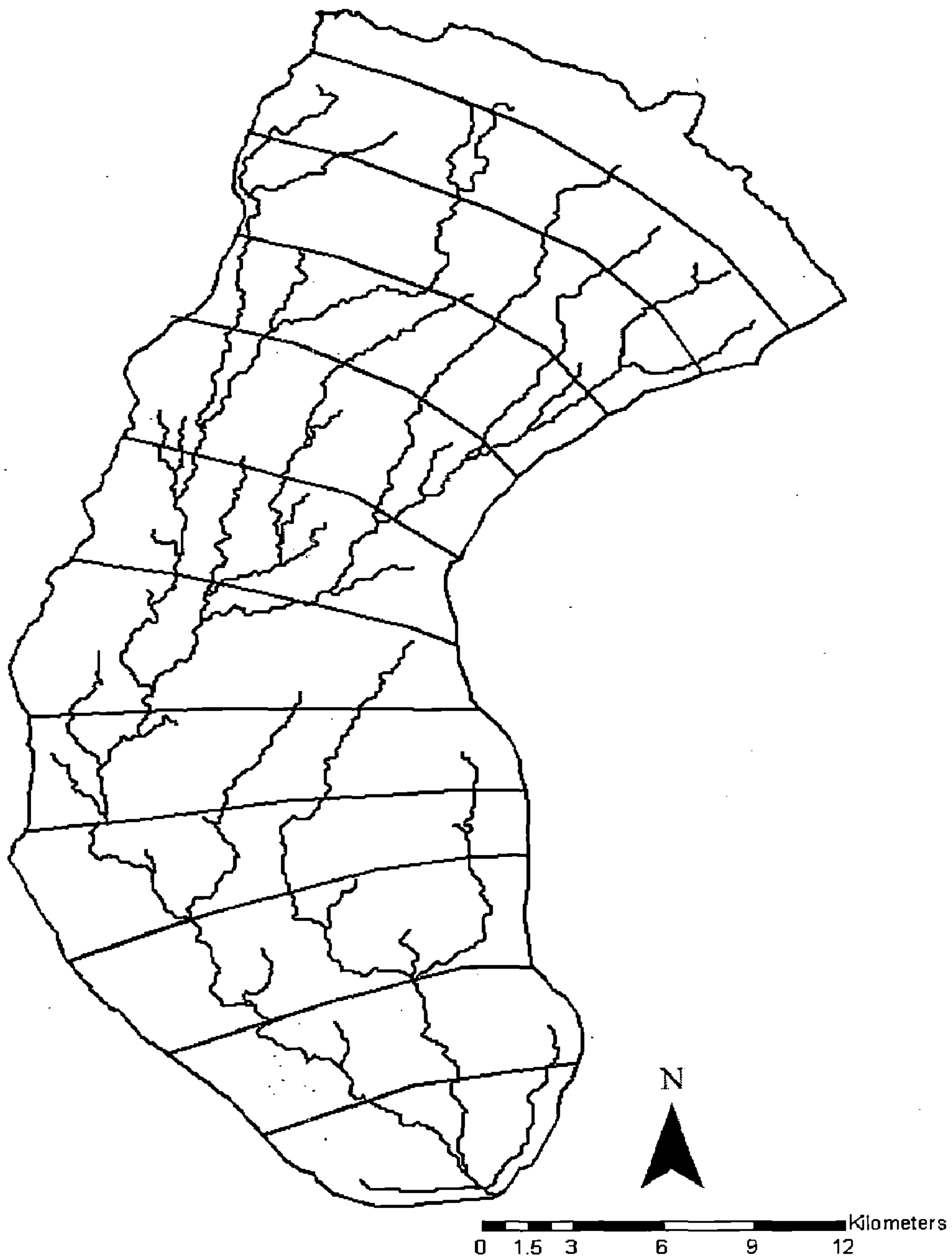

Figure 5. 10 Sub-catchment for Clark's method

5.5.6 Routing through Linear reservoir (K=13 hours)

The sub-catchment areas obtained from Figure 5.10 are plotted in Figure 5.11 as time-area histogram and used in the calculation of ordinates of Translation Hydrograph (TH) (Table 5.9).

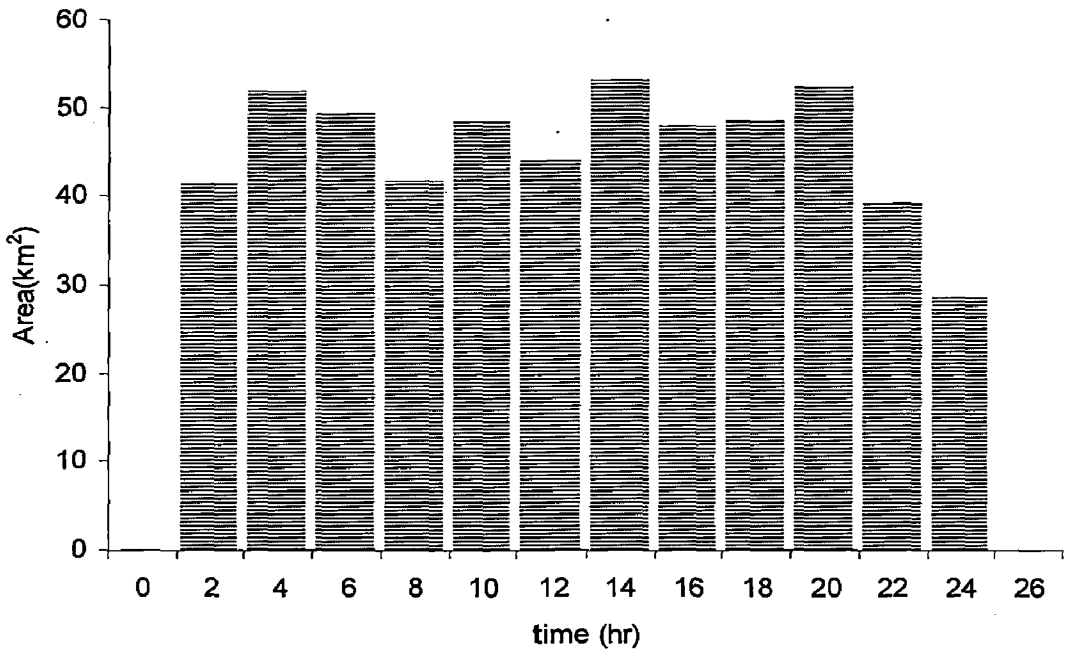

Figure 5. 11 Time-area histogram

Table 5. 9 Translation Hydrograph for Solani river

Time (hr)	Area (A_{nr}) km^2	TH (m^3/s)
0	0	0
2	41.53	57.6806
4	51.71	71.8195
6	49.42	68.63894
8	41.76	58.00005
10	48.28	67.05561
12	44.02	61.13894
14	53.19	73.87506
16	47.84	66.4445
18	48.62	67.52783
20	52.1	72.36117
22	39.48	54.83338
24	28.7	39.86114
26	0	0

The following equations are used for routing the translation hydrograph through the linear reservoir with storage coefficient value as 13 hours to calculate the IUH ordinates.

$$Q_n = C_0 I_n + C_1 I_{n-1} + C_2 Q_{n-1} \tag{5.8}$$

$$C_0 = \frac{0.5\Delta t}{K + 0.5\Delta t} \tag{5.9}$$

$$C_1 = \frac{0.5\Delta t}{K + 0.5\Delta t} \tag{5.10}$$

$$C_2 = \frac{K - 0.5\Delta t}{K + 0.5\Delta t} \tag{5.11}$$

where, $C_0 + C_1 + C_2 = 1.0$. Finally, IUH obtained is plotted as in Figure 5.12 and its ordinates are given in Table 5.10.

Table 5. 10 Ordinates of IUH for Solani river (4 hour interval)

Time (hr) (i)	0	4	8	12	16	20	24	28	32
Ordinates of IUH (m^3/s) (ii)	0	12.76	26.99	36.63	45.19	51.389	52.309	40.89	30.06
(i)	....	36	40	44	48	52	56	60	64
(ii)	....	22.09	16.24	11.94	8.78	6.45	4.74	3.49	2.566

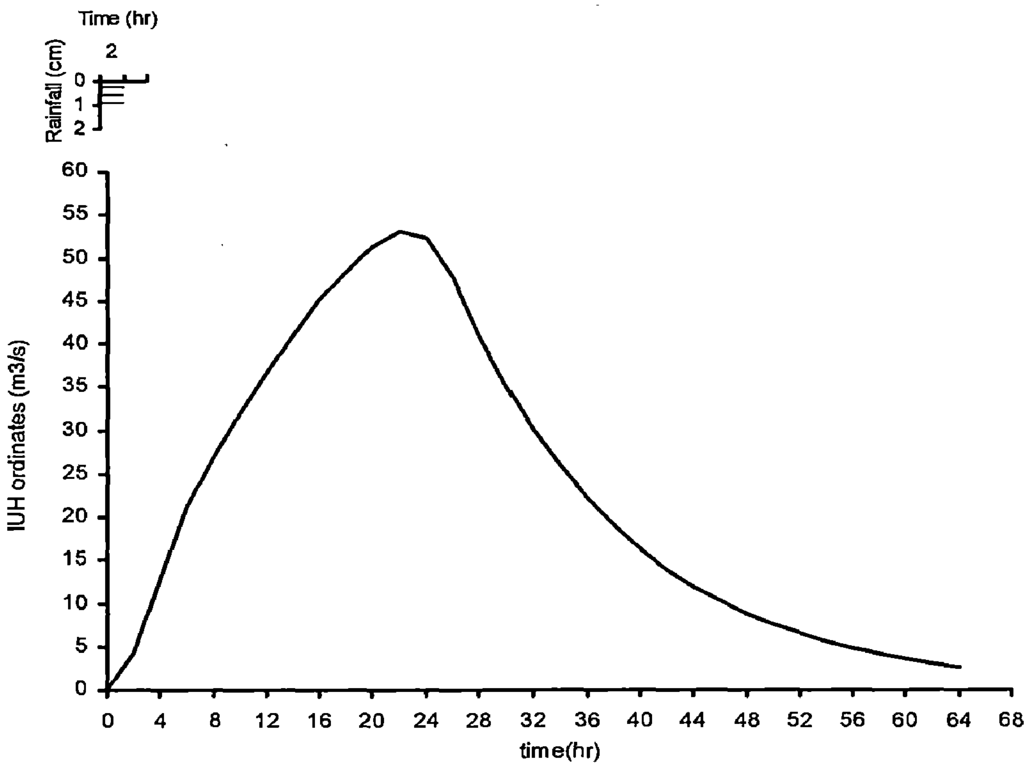

Figure 5. 12 IUH obtained by Clark's Method

The 2 hour UH obtained from the IUH is shown in Figure 5.12 and its ordinates are given in Table 5.11.

Table 5. 11 Ordinates of 2 hour UH obtained from Clark's method.

Time (hr) (i)	0	4	8	12	16	20	24	28	32
Ordinates of UH (m^3/s) (ii)	0	8.44	23.98	34.35	43.11	49.84	52.71	44.29	32.56
(i)	......	36	40	44	48	52	56	60	64
(ii)	......	23.93	17.59	12.93	9.51	6.99	5.14	3.78	2.78

5.5.7 Design of unit hydrograph for Solani river at Roorkee

Three numbers of 2 hour UH have been arrived at in the previous sections (Table 5.4, Table 5.8 and Table 5.11). These are plotted in Figure 5.13.

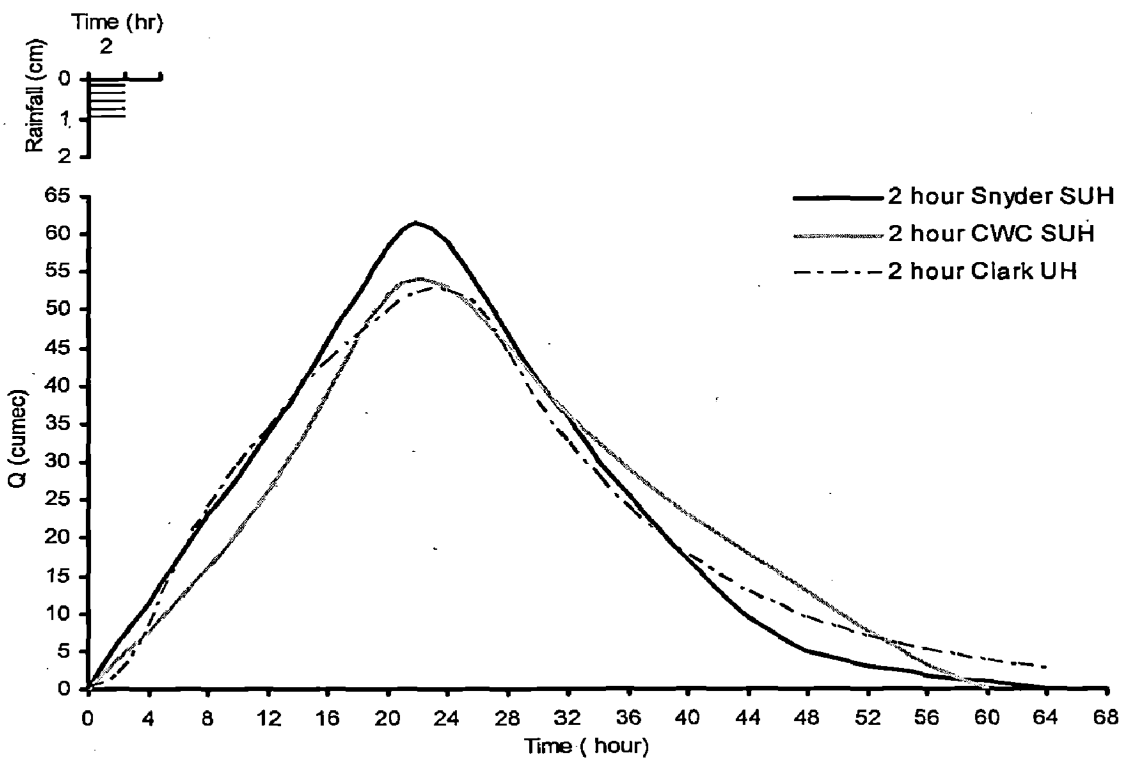

Figure 5. 13 Comparison of three methods to produce 2 hour UH.

Since these match quite closely the average unit hydrograph has been arrived at and it is shown in Figure 5.14.

5.5.8 Design of 2 hour UH

Following the recommended procedure of the CWC the design UH is arrived at by increasing the peak ordinate by 50 percent and adjusting the volume from the two limbs (i.e. beyond the crest segment). The 2 hr design UH so arrived at is shown in Figure 5.14. Two hour design UH ordinates are given in Table 5.12.

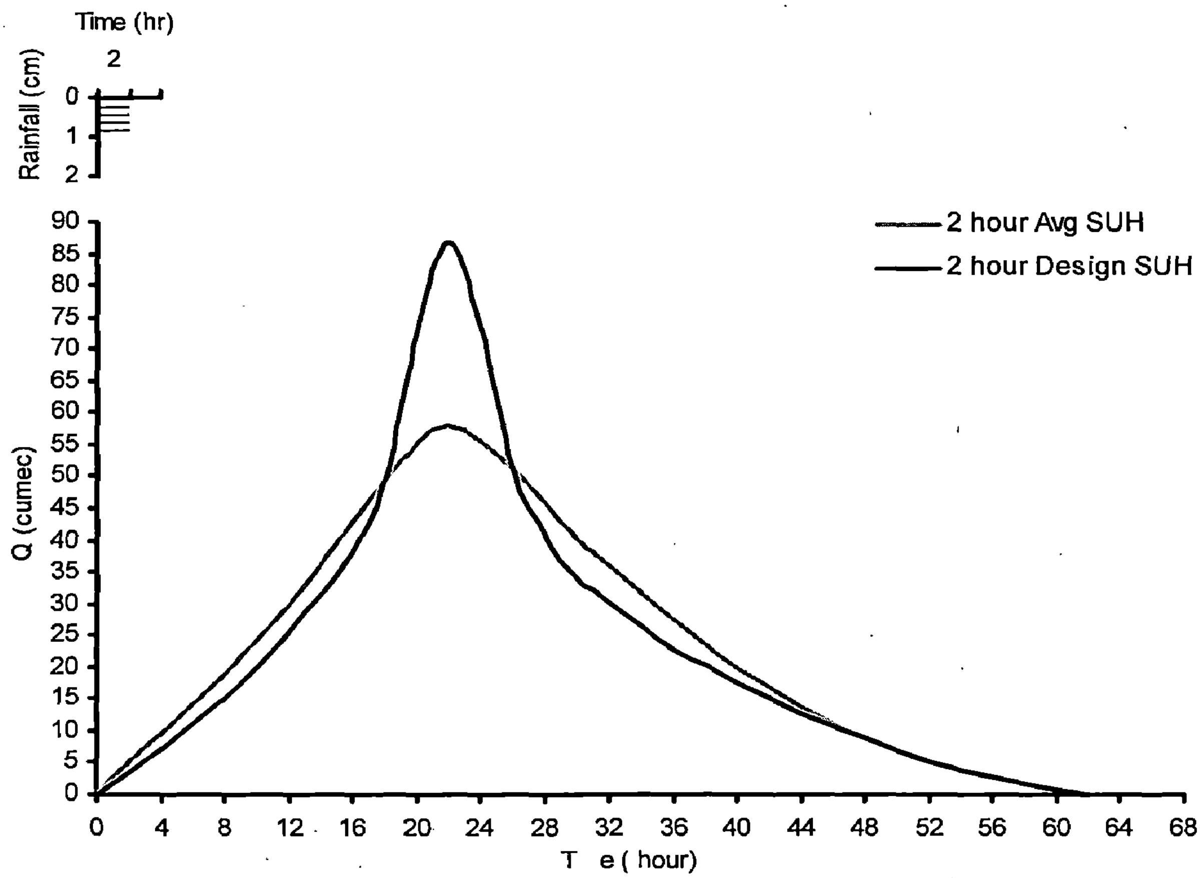

Figure 5. 14 Design 2 hour UH developed from average 2 hour UH.

Table 5. 12 Ordinates of design 2 hour UH developed from average 2 hour UH

Time (hr) (i)		0	2	4	6	8	10	12	14	16	18	20	22	24	26	28	30
2 hr Design UH ordinates (m^3/s) (ii)		0	3.5	7.2	11	15.2	19.8	25.6	31.4	38.3	49.4	73	87	74	51.2	41	34.4
(i)	.....	32	34	36	38	40	42	44	46	48	50	52	54	56	58	60	62
(ii)	.....	30.4	26.4	22.6	20.4	17.4	15	12.6	10.6	8.8	7	5.2	3.8	2.8	1.8	0.8	0

5.5.9 Design flood computation for 100 years return period rainfall

Intensity – Duration- Frequency analysis of rainfall data were carried out for rainfall durations starting from 2.5 minutes to 3.0 hours using the data of the observatory of the Department of Hydrology at Roorkee (Gautam, 1997; Shahoei, 1997). From these studies, a rainfall depth of 2 hour duration for a 100 years return period as worked out to be 163.8 mm has been adopted and the same is used for the computation of design flood. Using the 2 hour design UH (Table 5.12), the flood computations have been carried out following the principles of UH theory. The design flood ordinates are given in Table 5.13 and its plot is shown in Figure 5.15. The peak discharge value as adopted is 1430 m^3/s. It is available at 22nd hour from the start.

Table 5. 13 Design flood ordinates for 100 years return period rainfall for Solani river catchment

Time (hr) (i)	0	4	8	12	16	20	24	28
Flood hydrograph (m^3/s) (ii)	0	118.08	249.28	419.84	628.12	1197.2	1213.6	672.4

(i)	32	36	40	44	48	52	56	60
(ii)	498.56	370.64	285.36	206.64	144.32	85.28	45.92	13.12

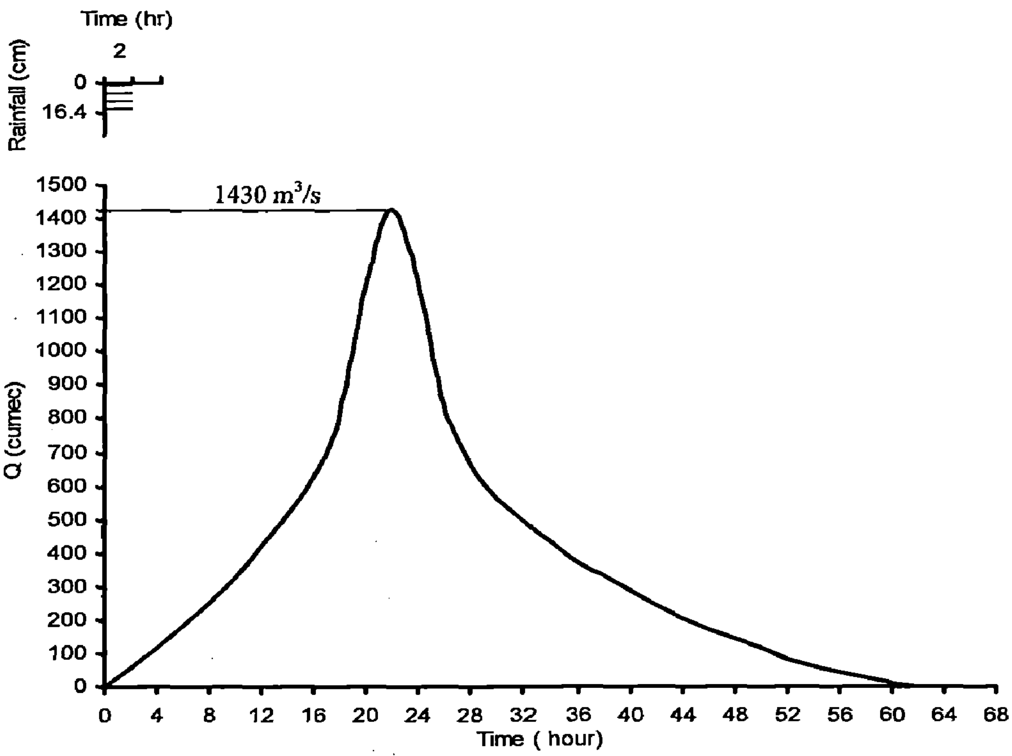

Figure 5. 15 Design hydrograph for 100 years return period for Solani river catchment

5.5.10 Computations for peak stages

For the constricted section of 300 m width (Figure 5.2), the roughness value of 0.018 and the slope 0.0003, the peak river stage is computed using Equation 3.4 which is reproduced as given below.

$$h_c = \left(\frac{Q_p n}{b\sqrt{S}} \right)^{3/5} = 2.61 \text{ m} \tag{5.12}$$

Within the central pair of embankments height of 3.5 m, the design flood having peak stage of 2.61 m will pass smoothly. However, the objective is to store some flood water into the two auxiliary channels having width of 50 m on the two sides of the main channel. This will serve the purpose of flushing off the stored effluents and the ground water recharge.

5.5.11 Flood diversion into auxiliary channels

The central dykes are to be provided with gated broad crested weirs having widths of 2.5 m. The length of weir is worked out for a sill height of 1.0 m above the bed level of the river (Figure 5.2). Manning's relationship is used to compute the discharge which will pass through the 300 m wide main channel of the Solani river having depth of 1.0 m. This works out to be 290 m^3/s . As shown in Figure 5.16, discharges shown by dotted part and having a time base of 35 hours will cause the flow through the broad crested weir sections to the auxiliary channels.

Calculation

Weir discharge is calculated using Equation III-3 through Equation III-8 (Appendix III) and it works out to be 3.92 m^3/s for 1 m levee height and h_0 = 1.61 m . So, length required to pass 1140 m^3/s flood water is 291 m.

In the auxiliary channel, total volume available is 0.6×10^6 m^3. If this space is filled within 4 hours, then gate opening scheduled will be given as Table 5.14 (using Equation III-3 through Equation III-8).

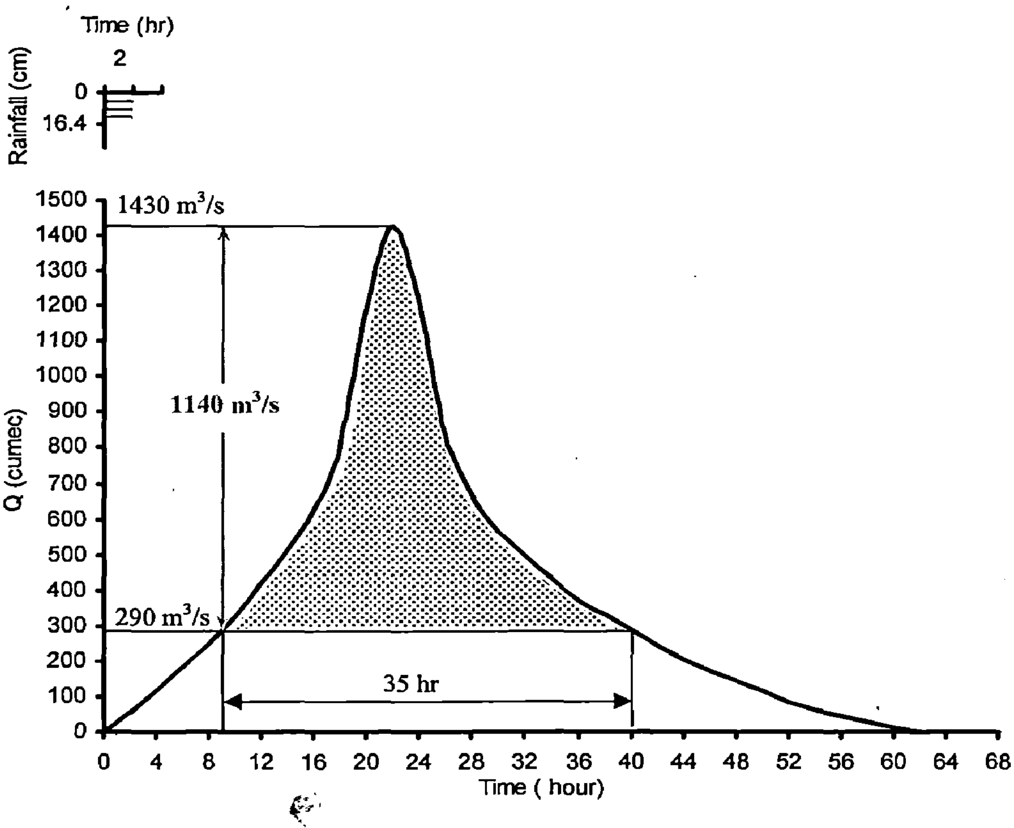

Figure 5. 16 Design flood hydrograph showing flood diversion

Table 5. 14 Gate opening schedule in order to fill up the auxiliary channel with 4 hours time

Sl. No	h_0 (m)	L_w in each side of the main channel (m)
1	0.25	104
2	0.50	36
3	0.75	19
4	1.00	12
5	1.25	8
6	1.50	6
7	1.61	5.5

CHAPTER 6 CONCLUSIOONS

Due to tremendous growth of human population along with increased industrial activities, the demand for water has increased manifolds. In most tropical countries where wet season continues during the four monsoon months and the rest of the year remains almost dry, there is an imperative need to use the water judiciously and store it when it is in excess supply. In Chapter 1, these aspects have been briefly discussed and the objectives of the present study are also stated.

In Chapter 2 of this thesis, a literature survey is reported which briefs of different methodologies for the computation of design flood for ungauged catchments. These methodologies have been used in the forthcoming section for developing a suitable model for computation of floods for ungauged areas.

Since in most cases the catchments of the river remains ungauged, it was considered desirable to synthesize the available techniques for computing the floods. Efforts were concentrated to evolve methodologies which may be convincely used by the field personals. To achieve this objective, the Rational Formula was used to compute the discharge from the catchment for per unit drainage area per unit width of the channel (q) for a common slope which is generally encountered. Its graphical presentation is given in Figure 3.1. Further, the Manning's equation was studied in depth for the computed design discharge as a function of the overall roughness factor (n). The graphical relationship so arrived as in given in Figure 3.2. Further, corresponding to the head of water h_c in the channel, average flow velocity (V) as a function of the roughness was worked. The plot of this is shown in Figure 3.3. In order to determine the nature of flow and time of concentration of the water to reach the outlet, the relationship between velocity (V), the Froude number (Fr) and the time of concentration (t_c) were explored and shown in Figure 3.4. Nomographs were arrived at by combining all these individual plots and the same is shown in Figure 3.5 and Figure 3.6. The Figure 3.5 is drawn for a channel section which can be approximated by a rectangle, whereas, the nomograph given in Figure 3.6 has been arrived at for a trapezoidal section. Attempts were made for a more generalized usage for these nomographs. Consequently, for both types of channel sections dimensionless nomographs were worked out and the same are given in Figures 3.7 and 3.8. The regression analyses were carried out for all the four plots of these nomographs. These regression equations are given in Section 3.2.3 of the thesis (Equation 3.15 through 3.24). These nomographs have satisfactorily been employed as a model to determine the design floods corresponding to different rainfall excess intensities. The details of the approach are given in Chapter 3 of this thesis.

Chapter 4 gives a case study carried out over a large catchment (3200 km^2) having a rainfall excess intensity of 20 mm/hr with runoff coefficient of 0.2. The peak discharge worked out to be 3555 m^3/s. To manage this flood and to protect the economic sectors downstream of the outlet (Fig 4.3) a stretch of 60 km long river has been constricted to a width of 500 m. The floodwaters are allowed to flow through the tamed section of the river. This is achieved by strengthening the levees on the two banks in the form of dykes (Figure 4.4(a)). The dyke sections have been provided with broad crested weir sections (500 m long) having sluice gates to allow the excess flood waters to enter into the flood plain depressions (Section 4.5). The total water stored upstream in depressions to protect the economic sectors worked out to be 28 MCM which will serve the purpose of recharging the aquifers for increased ground water supply. In short, this study emphasizes on a flood management plan for protection of economic sectors against the excessive river flows by diverting the same into upstream depressions. This water also replenishes the ground water table for increased ground water supplies during the dry period.

Further in Chapter 5, another flood management plan has been reported. This plan envisages the usage of the river bed width for passing the safe discharges downstream as well for storing water for replenishment of ground water table. For this purpose, two pairs of the dykes have been proposed over a 6 km long Solani river stretch with its outlet at Roorkee aqueduct. This aqueduct carries water of the Upper Ganga canal. The Solani river catchment in the Uttarakhand state of India ($77^o45'22''E, 30^o16'21''N$ to $78^o00'30''E, 29^o53'00''N$) has a drainage basin of 547 km^2 with a channel length of 60 km for its outlet at Roorkee. The physiographic parameters have been worked out by applying the GIS. Three methodologies have been employed for computing Synthetic unit hydrograph (SUH). These are the Central Water Commission of India (CWC) recommended approach for this hydro-meteorological subzone [i.e. 1(e)], Snyder's approach and the Clark's model. A two hour average unit hydrograph was worked out and its peak was increased by 50 percent as recommended in the for design flood computation by the CWC (CWC, 1982). A design flood was worked out for a 2 hour rainfall with a return period of 100 years. The design flood peak was computed as 1430 cumec (Section 5.5.9). The proposed plan showing structure of the dykes for constricting Solani river bed width to 300 m along with two parallel auxiliary channels each having 50 m bed width is shown in Figure 5.2(a). The treated domestic and industrial waste will be allowed come to the auxiliary channels. This is to be flushed out during the wet period from the out flows of the excessive flood waters. These flows have been planned through the gated section (sluice gates) to be provided in the broad crested weirs in the inner pair of dykes (Figure 5.2(b)). The auxiliary channels in the downstream section are to be provided with gates which will be used for storing water for recharging the aquifer (Section 5.5.11). In short, in this plan proposal, approximately 0.6 MCM of water is proposed to be stored for ground water recharge.

Concludingly, it may be remarked that a careful scrutiny of the flood management plans is necessary keeping all the environment constraints in view. The plans will also require verification through physical model studies. Depending upon the physiography, nature of the river behavior and availability of the storage space, any of the two proposed plans discussed in the thesis (viz. transfer of water to the flood plains or storage of water within the river width itself through the auxiliary channels) may be thought of. Whichever plan is to be adopted it should be environmentally sound and need be eco-friendly.

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APPENDIX I NOMOGRAPH PARAMETER CALCULATIONS

I.1 Nomogragh parameter calculations for wide rectangular channel

```
$DEBUG
   DIMENSION C(50),AI(50),AN(50)
   DIMENSION QP(50,50),QQ(50,50),AHC(50,50)
   OPEN(UNIT=1,FILE='NOMOGRAPH_INPUT.IN',STATUS='OLD')
   OPEN(UNIT=2,FILE='NOMOGRAPH_OUTPUT.OUT',STATUS='UNKNOWN')
C
C  C= RUNOFF COEFFICIENT
C  AI= RAINFALL EXCESS INTENSITY (MM/HR)
C  AN= ROUGHNESS COEFFICIENT
C  AD= DRAINAGE AREA (KM2)
C  SC= CHANNEL SLOPE
C  B= CHANNEL WIDTH (M)
C  AL= CHANNEL LENGTH (KM)
C  QP= PEAK DISCHARGE (M3/S), OBTAINED FROM RATIONAL METHOD
C  HC= CHANNEL WATER DEPTH (M), OBTAINED FROM MANNING'S FORMULA
C  V= MANNING'S VELOCITY IN CHANNEL (M/S)
C  TC= TIME OF CONCENTRATION (MIN)
C  FR= FROUDE NUMBER
C  QQ= DISCHARGE PER UNIT WIDTH PER UNIT DRAINAGE AREA
C
   READ(1,*)
   READ(1,*)NC
   READ(1,*) (C(I),I=1,NC)
   READ(1,*)
   READ(1,*)
   READ(1,*)NAI
   READ(1,*) (AI(I),I=1,NAI)
   READ(1,*)
   READ(1,*)
   READ(1,*)NAN
   READ(1,*) (AN(I),I=1,NAN)
C  --------------------------
   AD=71.53
   SC=0.0009
   B=14.1
   AL=12.85
C  --------------------------
   DO I=1,NC
      DO J=1,NAI
      QP(I,J)=C(I)*AI(J)*AD*0.2778
      END   DO
   END      DO
C  --------------------------
   WRITE(2,100)
100   FORMAT(1X,'RUNOFF C',3X,'I(MM/HR)',3X,'D AREA(KM2)',3X,
   1'PEAK DIS(M3/S)')
   WRITE(2,*)'---------------------------------------------------'
   DO I=1,NC
      DO J=1,NAI
      WRITE(2,101)C(I),AI(I),AD,QP(I,J)
      END DO
   WRITE(2,*)
   END DO
101   FORMAT(2X,F7.3,5X,F7.3,5X,F7.3,5X,F7.3)
```

```
C ----------------------------
   WRITE(2,102)
102    FORMAT(1X,'PEAK DIS(M3/S)',5X,'QQ',5X,'HC(M)',5X,
   1'PEAK DIS(M3/S)')
   WRITE(2,*)'-----------------------------------------------------------'
   DO K=1,NAN
      DO I=1,NC
            DO J=1,NAI
            QQ(I,J)=QP(I,J)/AD/B
           AHC(I,J)=((QP(I,J)*AN(K))/(B*(SC**0.5)))**(0.6)
            WRITE(2,103)QP(I,J),QQ(I,J),AHC(I,J),AN(K)
            END   DO
      WRITE(2,*)
      END   DO
   END DO
103    FORMAT(2X,F7.3,5X,F7.3,5X,F7.3,5X,F7.3)
C ----------------------------

   STOP
   END
```

I.2 Nomogragh parameter calculations for trapezoidal channel

```
$DEBUG
   DIMENSION C(50),AI(50),AN(50)
   DIMENSION QP(50,50),QQ(50,50),AHCT(50,50)
   OPEN(UNIT=1,FILE='NOMOGRAPH_INPUT.IN',STATUS='OLD')
   OPEN(UNIT=3,FILE='NOMOTRAPE_OUTPUT.OUT',STATUS='UNKNOWN')

C
C  C= RUNOFF COEFFICIENT
C  AI= RAINFALL EXCESS INTENSITY (MM/HR)
C  AN= ROUGHNESS COEFFICIENT
C  AD= DRAINAGE AREA (KM2)
C  SC= CHANNEL SLOPE
C  AZ= CHANNEL SIDE SLOPE
C  B= CHANNEL WIDTH (M)
C  AL= CHANNEL LENGTH (KM)
C  QP= PEAK DISCHARGE (M3/S), OBTAINED FROM RATIONAL METHOD
C  HC= CHANNEL WATER DEPTH (M), OBTAINED FROM MANNING'S FORMULA
C  V= MANNING'S VELOCITY IN CHANNEL (M/S)
C  TC= TIME OF CONCENTRATION (MIN)
C  FR= FROUDE NUMBER
C  QQ= DISCHARGE PER UNIT WIDTH PER UNIT DRAINAGE AREA
C
   READ(1,*)
   READ(1,*)NC
   READ(1,*) (C(I),I=1,NC)
   READ(1,*)
   READ(1,*)
   READ(1,*)NAI
   READ(1,*) (AI(I),I=1,NAI)
   READ(1,*)
   READ(1,*)
   READ(1,*)NAN
   READ(1,*) (AN(I),I=1,NAN)
C ----------------------------
   AD=31.94
   SC=0.0009
   AZ=17.0
```

```
      B=2.33
      AL=5.78
C     ---------------------------
      DO I=1,NC
         DO J=1,NAI
         QP(I,J)=C(I)*AI(J)*AD*0.2778
         END   DO
      END      DO
C     ---------------------------
      WRITE(3,100)
100      FORMAT(1X,'RUNOFF C',3X,'I(MM/HR)',3X,'D AREA(KM2)',3X,
      1'PEAK DIS(M3/S)')
      WRITE(3,*)'-----------------------------------------------------'
      DO I=1,NC
         DO J=1,NAI
         WRITE(3,101)C(I),AI(I),AD,QP(I,J)
         END DO
      WRITE(3,*)
      END DO
101      FORMAT(2X,F7.3,5X,F7.3,5X,F7.3,5X,F7.3)

C     ---------------------------
      WRITE(3,*)'TRAPEZOIDAL CROSS SECTION'
      WRITE(3,*)
      WRITE(3,104)
104      FORMAT(1X,'PEAK DIS(M3/S)',5X,'QQ',5X,'HC(M)',5X,
      1'ROUGHNESS')
      WRITE(3,*)'-----------------------------------------------------'
      HCT=1.0
      DO K=1,NAN
      KK=1
         DO I=1,NC
               DO J=1,NAI
               QP(I,J)=C(I)*AI(J)*AD*0.2778
               QQ(I,J)=QP(I,J)/AD/B
               QP1=QP(I,J)

               AN1=AN(K)

505                  HCTNEW=HCT-FN(HCT,QP1,SC,AN1,B,AZ)/DF(HCT,SC,AN1,B,AZ)
               IF(ABS(HCTNEW-HCT).LE.0.0005.OR. KK.GT.100)GOTO 506
               KK=KK+1
               HCT=HCTNEW
               GOTO 505
506                  AHCT(I,J)=HCTNEW
               WRITE(3,510)QP(I,J),QQ(I,J),AHCT(I,J),AN(K)
               END   DO
         WRITE(3,*)
         END   DO
      END DO
510      FORMAT(2X,F7.3,5X,F7.3,5X,F7.3,5X,F7.3)


      STOP
      END

C
C
C     NEWTON RAPHSON METHOD FOR COMPUTING HC
C
C     SUB-PROGRAM TO COMPUTE FN(HC)
C
         FUNCTION FN(HCT,QP1,SC,AN1,B,AZ)
      FN=QP1-((SC**0.5)/AN1)*((((B+AZ*HCT)*HCT)**(5.0/3.0))/
      1((B+2.0*HCT*SQRT(1+AZ**2.0))**(2.0/3.0)))
```

```
      RETURN
      END
C     ------------------------------------
C
C     SUB-PROGRAM TO COMPUTE THE DERIVATIVE FN(HC)
C
      FUNCTION DF(HCT,SC,AN1,B,AZ)

      DF=((-((SC**0.5)/AN1))* ((B+2.0*HCT*SQRT(1+AZ**2.0))**(2.0/3.0))*
     1(5.0/3.0)*(((B+AZ*HCT)*HCT)**(2.0/3.0))*(B+2.0*AZ*HCT)-
     2(((B+AZ*HCT)*HCT)**(5.0/3.0))*(2.0/3.0)*
     3((B+2.0*HCT*SQRT(1+AZ**2.0))**(-1.0/3.0))*(2.0*SQRT(1.0+AZ**2.0)))
     4/(B+2.0*HCT*SQRT(1+AZ**2.0))**(4.0/3.0)

      RETURN
      END
```

APPENDIX II TWO HOUR SYNTHETIC UNIT HYDROGRAPH DERIVATION FOR AN UNGAUGED CATCHMENT [CENTRAL WATER COMMISSION OF INDIA APPROACH FOR SUBZONE 1(E)]

Considering the hydro-meteorological homogeneity of subzone 1(e), the relations established between physiographic and unit hydrograph (UH) parameters developed by CWC (CWC,1984) are applicable for derivation of 2 hour Synthetic Unit Hydrograph (SUH) for an ungauged catchment in the same subzone. The following steps are involved for derivation of 2 hour unit hydrograph. The parameters used for developing UH as proposed by CWC are given in the Figure II.1. Using the suggested procedure a 2 hour SUH has been developed for the Solani river at Roorkee.

Step-1: Analysis of 'catchment physiography'

Physiographic parameters of the ungauged catchment viz. the catchment area A, length of the longest stream L and equivalent stream slope S are determined from the catchment area plan for the estimation of $L/\sqrt{S}$, where the equivalent stream slope S can be calculated as follows.

$$S = \frac{\sum L_i (D_{i-1} - D_i)}{L^2} \qquad \text{(II-1)}$$

where L_i is the length of the ith segment in km, D_{i-1}, D_i are the depths of the river at the point of intersection of $(i-1)$ and ith contours respectively from the base line (datum) drawn at the level of the point of study in meters and L is the length of the longest stream.

Step-2: Determination of 'peak value for 2 hour SUH'

The following equations for the unit hydrograph peak per unit catchment area q_p (cumec/sq.km) have been used to arrived at the different parameters of the 2 hour UH of Solani river catchment. The parameter $L/\sqrt{S}$ has been worked out in the previous steps.

$$q_p = 2.030/(L/\sqrt{S})^{0.649} \qquad \text{(II-2)}$$

The peak discharge Q_p (cumec) for the 2 hour SUH is obtained as follows

$$Q_p = q_p \times A \qquad \text{(II-3)}$$

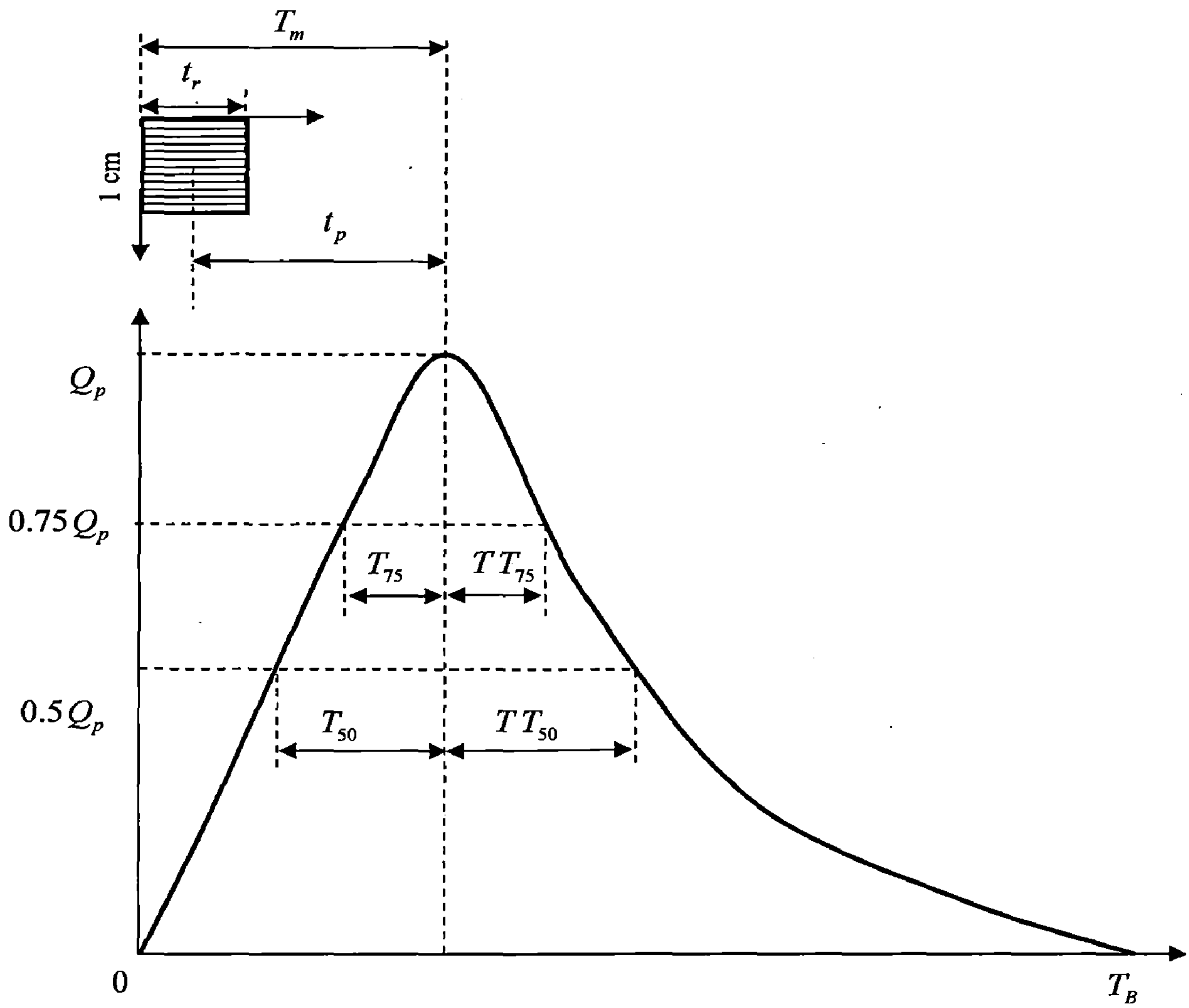

Figure II. 1 The details of the parameters of UH developed by CWC.

Step-3: Estimation of the 'time to peak (T_m) for 2hour SUH'

The worked out value of q_p is substituted in the following relationship for computing the time lag t_p (hr), i.e. the distance from the c.g. of the rainfall excess depth upto the peak discharge .

$$t_p = 1.858/(q_p)^{1.038} \qquad \text{(II-4)}$$

The time to peak T_m of 2 hour UH (Figure II.1) is thus computed as follows.

$$T_m = t_p + t_r/2 \qquad \text{(II-5)}$$

Step-4: Estimation of 'time base for 2 hour SUH'

The t_p value is substituted in the following equation to work out time base T_B in hour.

$$T_B = 7.744\left(t_p\right)^{0.779} \tag{II-6}$$

Step-5: Estimation of 'time widths for different % values of peak discharge of 2 hour SUH'

Substituting q_p value in the following equations, the time width parameters (in hr) W_{50}, W_{75}, W_{R50} and W_{R75} (Figure II.1) have been calculated.

$$W_{50} = T_{50} + TT_{50} = 2.217/(q_p)^{0.990} \tag{II-7}$$

$$W_{75} = T_{75} + TT_{75} = 1.477/(q_p)^{0.876} \tag{II-8}$$

$$W_{R50} = T_{50} = 0.812/(q_p)^{0.907} \tag{II-9}$$

$$W_{R75} = T_{75} = 0.606/(q_p)^{0.791} \tag{II-10}$$

(Note: the values in the parenthesis refer to Figure II.1)

Step-6: Computation of 'unit depth'

The parameter computed in earlier equations viz. Q_p, T_m, t_p, $T_B, T_{50}, TT_{50}, T_{75}, TT_{75}$ are used and a 2 hour SUH is plotted on a graph. The runoff depth (d) under the SUH is computed as follows.

$$d = \frac{0.36 \times Q_i \times t_r}{A} \text{ cm} \tag{II-11}$$

In case, the depth of runoff (d) for the SUH is not equal to 1.0 cm, suitable modifications may be made by smoothening the graph so as to contain a unit volume (i,e, 1 cm × catchment area).

APPENDIX III FLOW OVER A BROAD CRESTED WEIR

Broad Crested Weir has a horizontal crest with a finite length L_w , in the flow direction of water, Its width is L_b as shown in Fig III.1. A weir is classified as broad-crested if 12.5 > L_b / h_o >3.0 (Strum, 2001). Streamline become straight and parallel over a broad-crested weir, with the critical depth occurring at some point over the crest. Various cross-sectional shapes, such as parabolic and triangular, are possible for broad-crested weir. However, we will limit our discussion to rectangular broad-crested weirs.

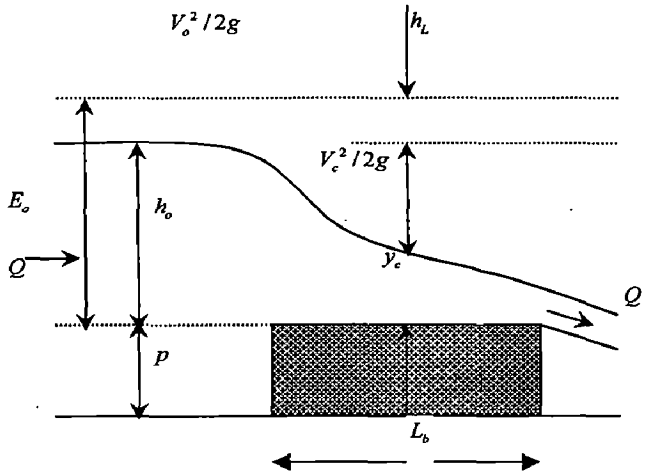

Figure III. 1 Broad-crested weir

The energy equation between approach section and the critical flow section as

$$E_o - h_L = h_o + \frac{V_o^2}{2g} - h_L = y_c + \frac{V_c^2}{2g} \qquad \text{(III-1)}$$

where h_L is the head loss, y_c is the critical depth and V_c is the critical flow velocity. $V_c = q / y_c$ and $y_c = (q^2 / g)^{1/3}$, where q is the discharge per unit width. Substituting these into equation (III-1), noting that $q = Q / L_w$. For this case, and rearranging the equation we obtain

$$Q = \frac{2}{3}\left(\frac{2}{3}g\right)^{1/2} L_w \left(h_o + \frac{V_o^2}{2g} - h_L\right)^{3/2} \qquad \text{(III-2)}$$

In the field installation we measure the flow depth, h_o , and therefore an equation expressing Q in terms of h_o is practical. For this purpose, traditionally, equation (III-2) is simplified as (Bos, 1989)

$$Q = C_d C_v \frac{2}{3}\left(\frac{2}{3}g\right)^{1/2} L_w h_o^{3/2} \qquad \text{(III-3)}$$

The coefficient C_d accounts for the head loss, and is expressed as

$$C_d = 0.93 + 0.10\frac{E_o}{L_b} \qquad \text{(III-4)}$$

The coefficient C_v accounts for the approach velocity head, and is expressed as

$$C_v = \left(\frac{E_o}{h_o}\right)^{3/2} \qquad \text{(III-5)}$$

Here we will further simplify Equation (III-3) to the form of

$$Q = k_w \sqrt{2g} L_w h_o^{3/2} \qquad \text{(III-6)}$$

where k_w is the broad-crested weir discharge coefficient, expressed as

$$k_w = k_d C_v \qquad \text{(III-7)}$$

with $k_d = (2/3)(1/3)^{1/2} C_d$. Substituting Equation (III-4) for C_d ,

$$k_d = 0.358 + 0.038\frac{E_o}{L_b} \qquad \text{(III-8)}$$

Equations (III-5) to equation (III-8) are used to determine the discharge over a broad-crested weir. However, because E_o is not measured, an iterative scheme is needed to solve these equations.

Iteration Scheme

(i) Initially assume $E_o = h_o$, in other words neglect the velocity head of the approach flow.

(ii) With this assumption, C_v =1.0 from equation (III-5), and by using Equation (III-8) find k_d .

(iii) Find k_w using equation (III-7) and substitute the value to find out $Q_{initial}$ in m^3/s.

(iv) Now refine the solution by taking the approach velocity head into account based on calculated discharge. Total depth of the approach section is $p + h_o$. Thus, $V_o = Q_{initial} / [(p + h_o) \times L_w]$, The corresponding velocity head become $V_o^2 / 2g$ thus

$E_o = h_o + V_o^2 / 2g$.

(v) Recalculate C_v , k_d , k_w and then Q_{new}

(vi) Update velocity head again using this discharge Q_{new} and repeat the calculations. If result having insignificant difference has obtained with the previous one then accept the value of Q_{new} .

APPENDIX IV WEIR DISCHARGE CALCULATION

```
$DEBUG
   DIMENSION AHC(500),AQWEIR(500)
   DIMENSION AQWFINAL(500)
   OPEN(UNIT=1,FILE='WEIR_INPUT.IN',STATUS='OLD')
   OPEN(UNIT=3,FILE='WEIR_OUTPUT.OUT',STATUS='UNKNOWN')

C
C  ALB= CREST LENGTH (M)
C  ALW= CREST WIDGTH (M)
C  AP= CREST HEIGTH (M)
C  AHC= HEIGHT OBTAINED FROM NOMOGRAM (M)
C  AHO= HEIGHT ABOVE THE CREST LEVEL = HC-AP
C  EO= ENERGY LEVEL = INITIALLY EQUAL TO HO
C  CV=  coefficient accounts for the approach velocity head
C  CD=  coefficient accounts for the head loss
C  AQWEIR= DISCHARGE (M3/S) OVEROPPING WEIR
C  AVO=approach velocity
C  AVOH=approach velocity head
C
   READ(1,*)
   READ(1,*)NHC
   READ(1,*) (AHC(I),I=1,NHC)

   AP=0.3
   ALW=1000.0
   ALB=1.0
C  --------------------------
      WRITE(3,*)'-------------------------------------------------------
   1------------------------'
      WRITE(3,500)
500   FORMAT(5X,'STEP',7X,'HC(M)',7X,'EO',9X,'CV',8X,'AKD',8X,'AKW',5X,
   1'AQWEIR(M3/S)',7X,'AVO',7X,'AVOH')
      WRITE(3,*)'-------------------------------------------------------
   1------------------------'

C  --------------------------


   DO I=1,NHC
      AHC1=AHC(I)
      AHO=AHC1-AP
      EO=AHO

      K=1

      CV=(EO/AHO)**(3.0/2.0)
      AKD=0.358+0.038*(EO/ALB)
      AKW=AKD*CV
      AQWEIR(1)=AKW*SQRT(2.0*9.81)*ALW*(AHO**(3.0/2.0))
      AVO=AQWEIR(1)/(AHC1*ALW)
      AVOH=(AVO**2.0)/(2.0*9.81)

      WRITE(3,502)K,AHC1,EO,CV,AKD,AKW,AQWEIR(K),AVO,AVOH

      EO=AHO+AVOH

      K=2

300         CV=(EO/AHO)**(3.0/2.0)
      AKD=0.358+0.038*(EO/ALB)
      AKW=AKD*CV
```

```
      AQWEIR(K)=AKW*SQRT(2.0*9.81)*ALW*(AHO**(3.0/2.0))


      IF(ABS(AQWEIR(K)-AQWEIR(K-1)).LE.0.0001.OR. K.GT.100)GOTO 302


      AVO=AQWEIR(K)/(AHC1*ALW)
      AVOH=(AVO**2.0)/(2.0*9.81)

      WRITE(3,502)K,AHC1,EO,CV,AKD,AKW,AQWEIR(K),AVO,AVOH

      K=K+1

      EO=AHO+AVOH

      GOTO 300

302          AQWFINAL(I)=AQWEIR(K)

      WRITE(3,*)
      WRITE(3,*)

   ENDDO


502   FORMAT(2X,I4,5X,F7.3,5X,F7.3,4X,F7.3,4X,F7.3,4X,F7.3,4X,F12.3,
   14X,F7.3,4X,F7.3)


   WRITE(3,*)
   WRITE(3,*)' CHANNEL HC(M) WEIR DISCHARGE(M3/S)'
   WRITE(3,*)'--------------------------------------------'
   DO I=1,NHC
      WRITE(3,*)AHC(I),AQWFINAL(I)
   ENDDO

   STOP
   END
```

APPENDIX V EQUIVALENT SLOPE CALCULATION FOR SOLANI RIVER USING GIS

Table V. 1 Equivalent stream slope computation for Solani river at Roorkee

Reduced Distance starting from gauging site (km)	Reduced levels of river bed (m)	Length of each Segment L_i (km)	Height above datum = difference between the 1st & the ith R.L.(D_i) (m)	$(D_{i-1}+D_i)$ (m)	$L_i\left(D_{i-1}+D_i\right)$ (km.m)
0	260	0	0	0	0
14.5	270	14.5	10	10	145
22.4	280	7.9	10	20	158
28.9	290	6.5	10	20	130
34.2	300	5.3	10	20	106
37.3	310	3.1	10	20	62
39.14	320	1.84	10	20	36.8
40.6	330	1.46	10	20	29.2
42.1	340	1.5	10	20	30
43.4	350	1.3	10	20	26
44.8	360	1.4	10	20	28
45.4	370	0.6	10	20	12
46.3	380	0.9	10	20	18
47	390	0.7	10	20	14
47.9	400	0.9	10	20	18
48.5	410	0.6	10	20	12
49.3	420	0.8	10	20	16
50	430	0.7	10	20	14
50.6	440	0.6	10	20	12
51.3	450	0.7	10	20	14
51.8	460	0.5	10	20	10
52.2	470	0.4	10	20	8
53.8	480	1.6	10	20	32
53.9	490	0.1	10	20	2
54.2	500	0.3	10	20	6
54.6	510	0.4	10	20	8
54.9	520	0.3	10	20	6
55.3	530	0.4	10	20	8
55.6	540	0.3	10	20	6
55.9	550	0.3	10	20	6
56.2	560	0.3	10	20	6
57	570	0.8	10	20	16
57.3	580	0.3	10	20	6
58	590	0.7	10	20	14
58.1	600	0.1	10	20	2
58.7	610	0.6	10	20	12
59.2	620	0.5	10	20	10

APPENDIX VI COMPUTATION OF STORAGE COEFFICIENT FOR CLARK'S MODEL APPLICATION OF SOLANI RIVER

Determination of the value of K has been estimated by considering the point of inflection of a surface runoff hydrograph. At this point the inflow into the channel has ceased and beyond this point the flow is entirely due to withdrawal from the channel storage (Table VI.1). Storage at different time interval is shown in Figure 5.14 and a regression line has also been drawn. Storage at different discharge has been plotted in Figure 5.15 and value of K is found from the regression line drawn indicating the relation between storage and discharge comparing with equation $S = KQ$.

Table VI. 1 Storage calculation using the recession limb of the design UH

Time on Design UH (hr)	Discharge (m^3/s)	Time (hr)																				$\sum$ (iv) to (xxii) (m^3/s)	Storage (m^3)
(i)	(ii)	(iii)	(iv)	(v)	(vi)	(vii)	(viii)	(ix)	(x)	(xi)	(xii)	(xiii)	(xiv)	(xv)	(xvi)	(xvii)	(xviii)	(xix)	(xx)	(xxi)	(xxii)	(xxiii)	(xxiv)
26	51.2	0	51.2	41	34.4	30.4	26.4	22.6	20.4	17.4	15	12.6	10.6	8.8	7	5.2	3.8	2.8	1.8	0.8	0	312.2	2247840
28	41.0	2		41	34.4	30.4	26.4	22.6	20.4	17.4	15	12.6	10.6	8.8	7	5.2	3.8	2.8	1.8	0.8	0	261.0	1879200
30	34.4	4			34.4	30.4	26.4	22.6	20.4	17.4	15	12.6	10.6	8.8	7	5.2	3.8	2.8	1.8	0.8	0	220.0	1584000
32	30.4	6				30.4	26.4	22.6	20.4	17.4	15	12.6	10.6	8.8	7	5.2	3.8	2.8	1.8	0.8	0	185.6	1336320
34	26.4	8					26.4	22.6	20.4	17.4	15	12.6	10.6	8.8	7	5.2	3.8	2.8	1.8	0.8	0	155.2	1117440
36	22.6	10						22.6	20.4	17.4	15	12.6	10.6	8.8	7	5.2	3.8	2.8	1.8	0.8	0	128.8	927360
38	20.4	12							20.4	17.4	15	12.6	10.6	8.8	7	5.2	3.8	2.8	1.8	0.8	0	106.2	764640
40	17.4	14								17.4	15	12.6	10.6	8.8	7	5.2	3.8	2.8	1.8	0.8	0	85.8	617760
42	15.0	16									15	12.6	10.6	8.8	7	5.2	3.8	2.8	1.8	0.8	0	68.4	492480
44	12.6	18										12.6	10.6	8.8	7	5.2	3.8	2.8	1.8	0.8	0	53.4	384480
46	10.6	20											10.6	8.8	7	5.2	3.8	2.8	1.8	0.8	0	40.8	293760
48	8.8	22												8.8	7	5.2	3.8	2.8	1.8	0.8	0	30.2	217440
50	7.0	24													7	5.2	3.8	2.8	1.8	0.8	0	21.4	154080
52	5.2	26														5.2	3.8	2.8	1.8	0.8	0	14.4	103680
54	3.8	28															3.8	2.8	1.8	0.8	0	9.2	66240
56	2.8	30																2.8	1.8	0.8	0	5.4	38880
58	1.8	32																	1.8	0.8	0	2.6	18720
60	0.8	34																		0.8	0	0.8	5760
62	0.0	36																			0	0.0	0

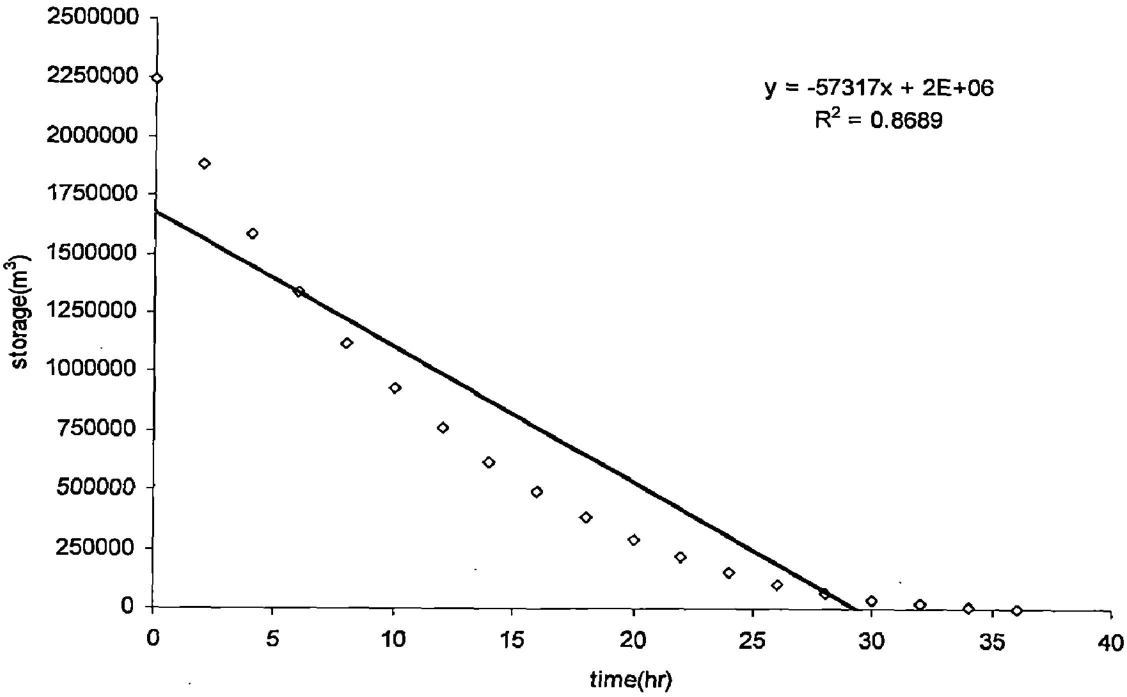

Figure VI. 1 Storage at different time interval

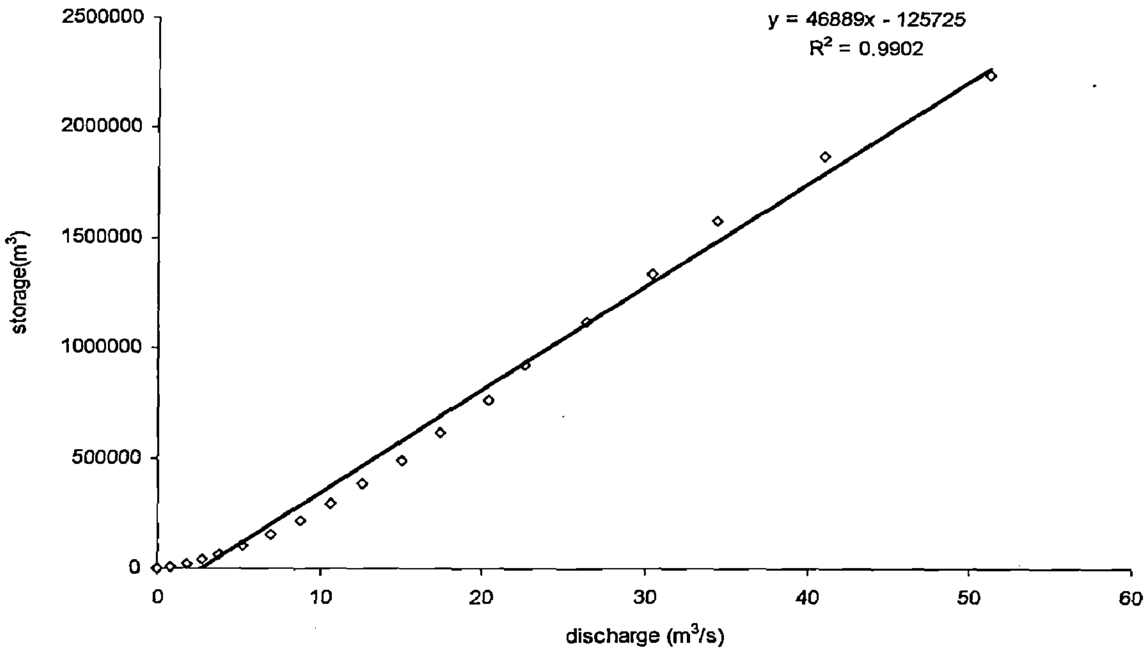

Figure VI. 2 Storage at different discharge

The value of K is found to be 46889 sec (= 13.024 hr).