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STORM DRAINAGE ESTIMATION IN URBAN AREAS

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ABSTRACT

In the last few decades, the world has witnessed rapid urbanisation. One of the many complex problems which have come up with increased urbanisation is that of quick drainage of storm water from the inhabited areas. For the design of an efficient and economic urban drainage system, it is important to estimate the design runoff with a good degree of accuracy.

In India, the present practice of urban runoff estimation is based on empirical formulae and not adequate for the analysis and design of complex drainage systems. A good amount of research work has been done in economically developed countries in the field of urban drainage and a number of models have been developed, tested and varified, conceiving the urban watersheds as systems.

In this technical note, an attempt has been made to review the various urban drainage models developed in different countries with their merits and demerits. The present practices of urban runoff estimation in India have been examined and the necessity of recording & maintenance of hydrological data for urban areas has been emphasised.

1.0 INTRODUCTION

1.1 Preliminary Remarks

Water may threaten the urban settlements in different ways. In a wider sense, the urban areas are under the possible impact of following:

i) Floods originating from rural and suburban areas and flowing through the urban zones.

ii) Penetration of surface water from rivers and other streams to the city either through streets and open areas or under ground sewerage system.

iii) Flooding caused by tides & surges.

(iv) Flooding caused by the the rise of ground water level.

(v) Flooding caused by the rainfall over the area considered.

Although all the five cases mentioned above are important to be studied but except the last case all others are specific in nature and seriously affect the urban inhabitants in case of long duration heavy rainfall over a large area. Flooding of urban area due to local intense storm over the area considered is a common problem and the solution is a properly designed storm water drainage system.

1.2 Need for Estimation of Urban Runoff

The hydraulic design of an urban storm water drainage system comprises of two parts.

(i) The Selection a suitable rainfall input.

(ii) Calculation of the design flows of water in various parts of the system for the selected rainfall input, which lead to the determination of the appropriate conduit sizes.

To be in error in these aspects means that either the drainage system is undersized or oversized. While the former will result in the flooding of the urban areas and hence inconvenience to the local people, the latter will give an uneconomical design which is equally undesirable. Thus, it is important to estimate the design runoff with a good degree of accuracy.

1.3 Growth of Urbanization

During the last few decades the world has witnessed rapid urbanization. It is basically because of immigration of ruralites to urban areas in search of employment and to enjoy the facilities of city life. The data of UNESCO are shown in Fig. 1. Figure 2 shows the different stages of urbanization of Belgrade from 1912 to 1985 (11).

In India the percentage share of urban population in the total population was only in the order of 10.84 in 1901 but it has increased to 23.71 percent in 1981. According to 1981 census about 60% of the total urban population is living in classe I cities. Table 1 gives some characteristics of the urban population in India (16). To house the increased population and to provide other civil facilities, more structures are built which considerably changes the land use pattern and the soil cover of the area. These changes affect the runoff from these area considerably which has been discussed in the subsequent paragraphs. Figure 3 shows the interaction between urbanisation impact and data and models.

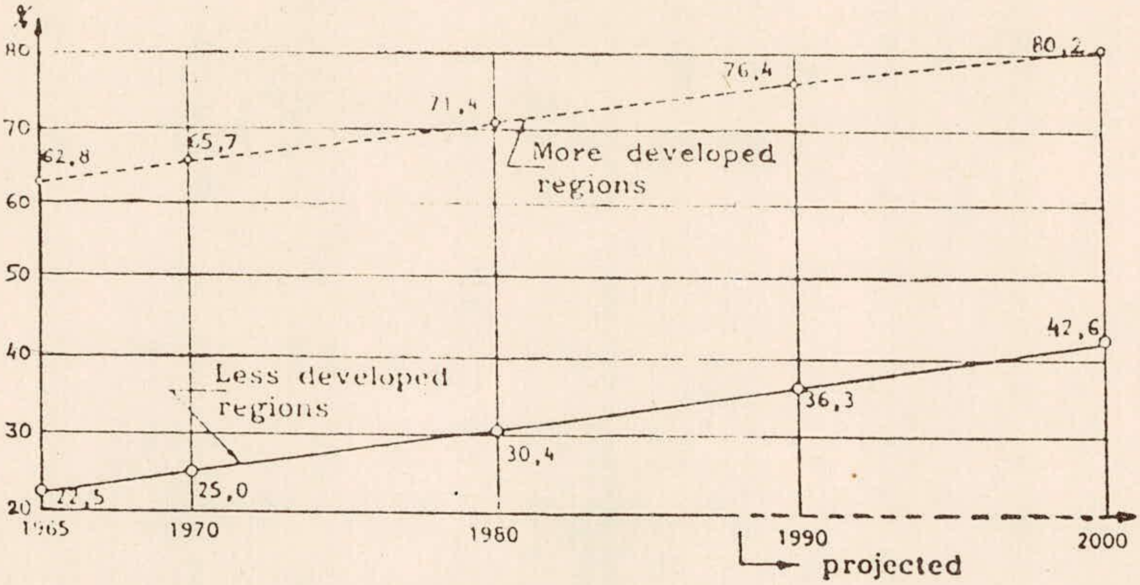


Fig. 1. Percentage of population in urban areas

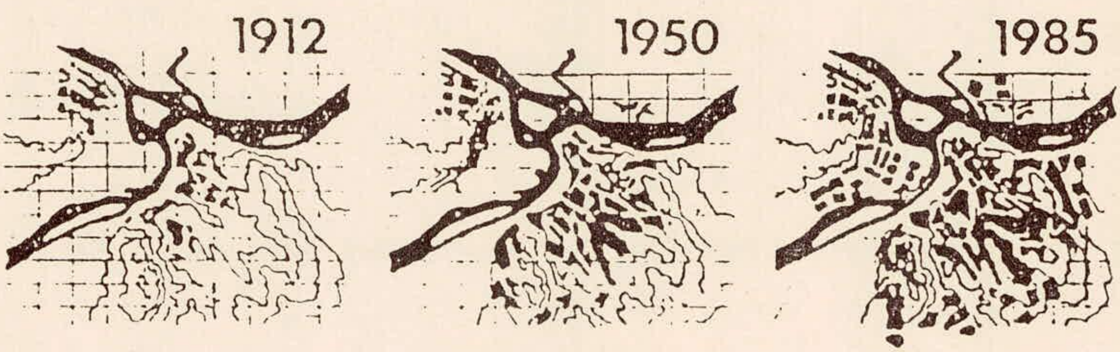


Fig.2 The Spatial Increase of Belgrade from 1912 to 1985

Increased
Urbanization

Need for
Complex
Drainage
Systems

Need for better
Methods of
Assesment and
Design

Need for more

Reliable data

Need for
Training the
Engineers

Fig. 3

Interaction between urbanization impact and data and models

TABLE 1

SOME CHARACTERISTICS OF THE URBAN POPULATION OF INDIA

POPULATION RANGE	NUMBER OF CITIES IN RANGE			1971 POPULATION IN RANGE, MILLIONS OF PERSONS
	1971	1961	1951	
1,000,00 or more	142	113	81	57.0
50,000 to 1,000,00	198	138	102	13.2
20,000 to 50,000	617	484	353	18.9
10,000 to 20,000	931	748	630	13.1
5,000 to 10,000	756	760	1158	5.7
Sub-total	2644	2243	2324	107.9
Less than 5,000	277	218	599	0.9
Total	2921	2461	2923	108.8

1.4 Effect of Urbanization on Runoff

Modifications of the land surface during urbanisation produce changes in the magnitude of runoff process. The major factor which affect the runoff processes is that the part of the area of the catchment is covered with impervious roofs, side walk, roadways and parking lots. The proportion of the catchment that is rendered impervious, increases with the population density. The infiltration capacity of these areas is lowered to almost zero and many areas that remain soil covered are trampled to an almost impervious state so that volume and rate of overland flow is increased.

Another factor is that the natural channels which were existing before urbanization are often straightened, deepened and lined to make them hydraulically smoother. Gutters, drains and storm sewers are laid in the urbanized area to convey runoff rapidly to stream channels. The combined effect of all these changes is to reduce the lag time of a runoff hydrograph i.e. the peak discharge is obtained more quickly.

Urbanisation also affects considerably the climate of the area. It has been found that precipitation, evaporation and local temperature all increase due to urbanization.

In short, urbanization causes the following changes in the runoff:

(i) Increase of runoff peak and volume

(ii) Decrease of time to peak

(iii) Decrease of infiltration

(iv) Reduction of base flow

References (13) and (14) give some case studies of hydrological effects of urbanisation in FRG, Netherlands, Swedan, USA, USSR and Japan.

1.5 Modelling Approaches

A model consists of a mathematical representation of a process to transform an input to produce an output. There are two obvoiously different philosophies in formulating the transformation to represent the process. One is the physical process based modelling which follows as closely as possible the spatial and temporal sequences of the process of the physical system. The other philosophy is to hypothetically consider the transformation, conceptually analogous to something else, not the true physical process but adequately simulating the transformation to produce satisfactory outputs. This approach is called conceptual modelling.

The choice of the type of modelling largely depends upon availability of data. For physically based modelling data must be available at the site of application and should be available in sufficient while for conceptual modelling, the model can be fitted to data at locations where it is available and the optimum model parameters are estimated. Then the model can be used for the desired locations with some modifications.

2.0 LITERATURE REVIEW

2.1 Preliminary Remarks

The central problem in surface water hydrology is the determination of time distribution of runoff caused by a storm event. Transformation of rainfall into runoff is a complex phenomenon as it is affected by the interaction of several processes such as interception, evaporation, surface detention, and infiltration which are listed by Chow (4). Because of lack of understanding of many of these processes and the interaction among them, pioneering hydrologic investigations were limited to the development of methods to determine only the magnitude of peak runoff. Consequently in the course of time, several emperical formulas to predict the magnitudes of peak runoff have resulted One of the major drawbacks of empirical formulas is the subjective selection of coefficients and parameters which are to be used with them.

Very often the design of urban drainage systems involves consideration of flood storage, permanent storage, off channel storage, inter drainage diversions, pumping installations and silting of drains. This requires a knowledge of flood hydrographs rather than only flood peak. Although rainfall runoff process is complicated, the effective rainfall-direct runoff process has been traditionally thought to be simpler. Consequently a good deal of attention has been concentrated on simulating the effective rainfall-direct runoff process as a system.

2.2 System Classification

A system is defined as a dynamic system if the input and the output are functions of time, in contrast of the static system in which the input and the output are independent of time. In a distributed system the input and/or the output are functions of both time and space. If the spatial distribution of input and output are either unimportant or are ignored to simplify the analysis, such systems can be modelled as "Lumped Systems" in which the input and the output are functions of time only. Use of lumped system models is closer to reality for urban basins than for rural basins because the former are often smaller in area and are more uniform in their characteristics. A system is said to be a linear system if the principles of superposition and proportionality can be applied to it.

Systems can also be treated as deterministic or stochastic systems. In the former method, attempts are made to develop relationships among the model parameters, the rainfall characteristics and physiographic characteristics of the watershed. This analysis is conducted using observed data. These relationships are then used to predict future runoff. On the otherhand in the stochastic systems, statistical measures of hydrologic variables are used to generate future events to which probability levels are attached. Long term records, which in many instances are not available are needed to estimate the parameters of stochastic models, in order to obtain a proper representation of their stochastic nature.

2.3 Modelling of Water Shed System

In the analysis and synthesis of rainfall-runoff process, watersheds are conceived as systems with rainfall as input to the system and runoff as output from the system. As both rainfall and runoff are functions of time and space, watershed systems are by definition distributed dynamic systems. Consequently, mathematical models for the rainfall-runoff process should be distributed dynamic models. Dawdy, D.R., et al. have developed and documented a computer program of such a watershed model for routing urban flood discharges through a branched system of pipes of natural channels using rainfall as input (5).

2.3.1 Physically based models

Physically based models mathematically simulate the physical processes occuring in the catchment during the transformation of rainfall into runoff, i.e. there is some degree of reality. This approach requires the consideration of the processes in the elements or components and their relative distribution within the catchment. Therefore, it is a distributed system approach.

In formulating a physically based model, the following process phases are considered after the mode of rainfall input has been determined.

(a) Decomposition of the catchment into subcatchment and components.

(b) Selection of methods to calculate the losses due to interception, depression storage and infiltration.

(c) Selection of methods of transforming rainfall excess water to runoff and routing of runoff on the surfaces and subcatchments to the sewer inlet.

(d) Routing of the flow in sewer systems.

2.3.2 Conceptual models

In conceptual models the catchment runoff is represented in terms of hypothetical parameters instead of real physical parameters. The transformation of excess rainfall to surface runoff is considered as a combination of translation and storage effects. Using the principle of linearity, these separate elements are combined to formulate a conceptual model of the excess rainfall-direct runoff systems. The linear hydrologic components used in the formation of these models are (i) linear reservoirs and (ii) linear channel.

A linear reservoir is a fictitious reservoir in which the storage S is directly proportional to the outflow Q i.e.,

$$S = K Q \qquad \text{...(i)}$$

Where K is a constant called storage coefficient. This equation when solved with hydrologic continuity equation

$$I - Q = ds/dt \qquad \text{...(ii)}$$

Where I is the inflow rate to the reservoir, gives the Instanteneous unit hydrograph of a linear reservoir as

$$h(t) = (1/K)\, e^{(-t/K)} \qquad \text{...(iii)}$$

Where h(t) is the ordinate of Instanteneous unit hydrograph at time, t.

In a similar conceptual way, a linear channel is simply a hypothetical stretch that delays the input by a constant length of time without changing its magnitude. Nonlinear reservoir model, Non linear reservoir model with time lag, Nash multiple linear reservoir model are other examples of conceptual models. Figure 4 illustrates fitting of a conceptual model.

2.3.3 Continuous simulation models

Most storm water models simulate a single storm event but a continuous simulation model works on a long term rainfall record-months or years instead of a single event. The hydrologic input to continuous simulation is the measured rainfall records. The antecedent conditions are handled automatically by the program in continuous simulation.

It is advantageous to use continuous simulation in large planning studies. It enables the user to examine the relative usefulness of control measures on a broad scale and eliminates the need to select a design storm before the design conditions have been determined. STORM and HSPF are the best known contiuous simulation models (24).

2.4 Critical Review of some Specific Models

The list of models and versions available to the potential user seems endless. There is however, a handful of programs that are fairly well documented and available to the public. Table 2 and Table 3 give a summary of selected physically based and conceptual urban surface runoff models respectively. Table 4 gives a comparitive study of major model characteristics. These models have been tested by many users and proved quite satisfactory. Some of thes models with their merits and demerits are described below.

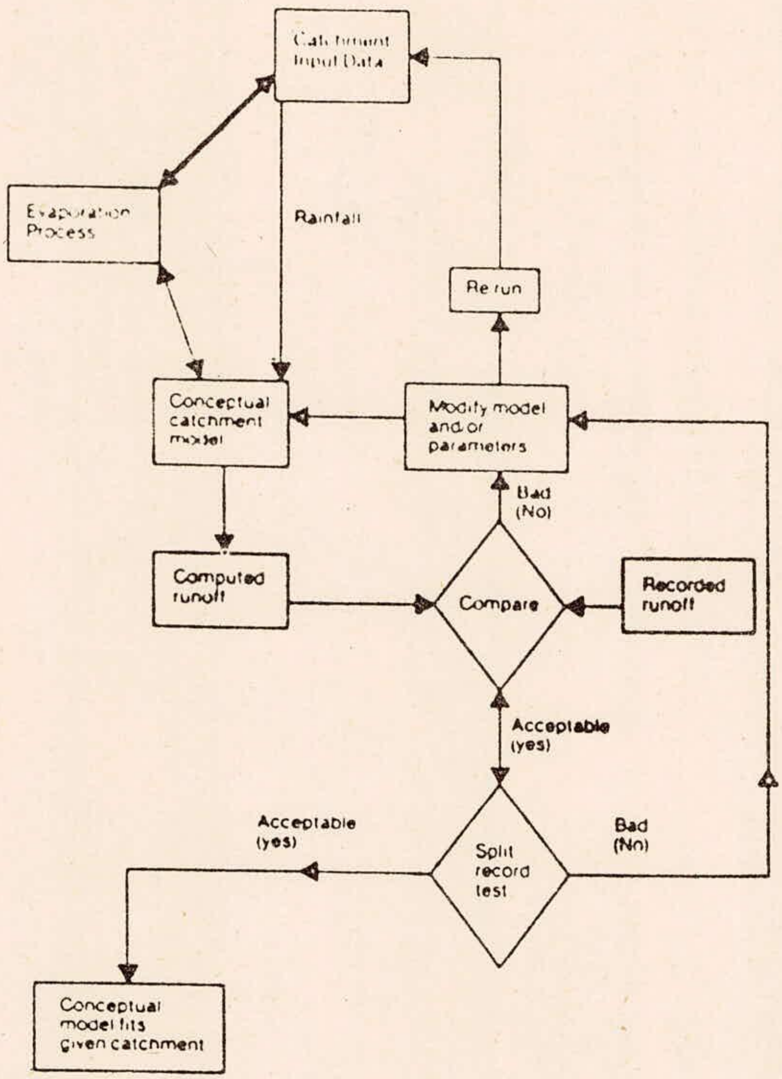


Fig.4 Fitting a conceptual catchment model

TABLE 2 Hydrological Properties of Overland Flow Simulation of Selected Physically Based Urban Runoff Models

S.No.	Model	Rainfall		Abstractions		Surface Runoff				Computer Program Manual Available	Selected References
		Input	Allow areal distribution	Initial losses	Pervious area Infiltration	Impervious area Contribution	Pervious area Contribution	Routing method	Street gutter		
1.	Los angeles hydrograph	Single hyetograph	No	Local empirical graph of losses considering evap., depression storage and infiltration		Yes	Yes	Storage routing with modified Manning's eqn. proposed by Horner & Jeans (1942)	Yes, similar to overland routing	No	Hicks (1944)
2.	Chicago Hydrograph	- do -	No	Depression storage by exp.function	Horton's	Yes	Yes	Modified Issard's	Linear kinematic wave storage	Yes	Tholin & Keifer (1960) Keifer et.al.(1978)
3.	HSP	Hyetographs	Yes	Interception & depression storage	Coupled with subsurface flow	Yes	Yes	Storage routing with constant depth similar to Izzard's, based on Manning's eqn.	No	Yes	Crawford & Linsley (1966); Hydrocomp (1976)
4.	Cincinnati	Single Hyetograph	No	Depression storage by exp.function	Horton's only from rain	Strips	Strips	Storage routing with constant depth detention storage function & Manning's eqn.	Continuity eqn. of steady spatially varied flow	Yes	Papadakis & Preul (1972); Univ. of Cincinnati (1970)
5.	TRRL	single Hyetograph	No		100%	Direct contributing only	No	Time-area	No	Yes	Watkins (1962); TRRL (1976)

........contd.

.......contd.(Table 2)

S.No.	Model	Rainfall		Abstractions		Surface Runoff				Computer Program Manual Available	Selected References
		Input	Allow areal distribution	Initial losses	Pervious area Infiltration	Impervious area Contribution	Pervious area Contribution	Routing method	Street gutter		
6.	ILLUDAS	single Hyetograph	No	Different constants for impervious & pervious surfaces	Horton's only from rain	Area & entry time of direct contributing surface, area of supplemental surface	Area & entry time of direct contributing surfaces	Time-area with Izzard's time formula	No	Yes	Terstreip & Stall (1974)
7.	ILSD	single Hyetograph	Yes, by multiplier & delay time formula	Different values for impervious & pervious surfaces & for different subcatchments	Horton's from rain & Depression storage	Area & entry time of direct surfaces & area for supplemental surfaces	Area & entry time of direct & supplemental surfaces	Time area with Yen-Chow's time formula	No	Yes	Yen et.al. (1984)
8.	IUSR	Hyetographs	Yes	Depression storage	Horton's from rain only	Divided in to strips with input length, width, slope & roughness		Nonlinear kinematic wave routing	Nonlinear kinematic wave routing with Manning's formula	Yes	Chow & Yen (1976)
9.	SWMM	Hyetographs	Yes	- do -	Horton's or Green & Ampt	- do -		Linear kinematic wave, storage routing with uniform depth continuity eqn.& Manning's formula	linear kinematic wave, storage eqn. with Manning's formula & continuity eqns.	Yes	Metcalf & Eddy, Inc. (1971); Huber & Heaney (1982); Huber et.al.(1984)

.......contd.(Table 2)

S.No.	Model	Rainfall		Abstractions		Surface Runoff				Computer Program Manual Available	Selected References
		Input	Allow areal distribution	Initial losses	Pervious area Infiltration	Impervious area Contribution	Pervious area Contribution	Routing method	Street gutter		
10.	USGS	single Hyetograph	No	- do -	Phillip's from rain only	Direct surfaces represented as strips supplemental surfaces add to direct perv. surfaces	strips	Nonlinear kinematic wave	No	Yes	Dawdy et.al.(1972, 1978?);Leclerc & Schaake (1973)
11.	Echime Stormwater Runoff	single Hyetograph	No	Depression storage by graph	By graph based on Horton's	Divided into strips		Kinematic wave	No	No	Toyukuni & Watanabe (1984)
12.	CTH	single Hyetograph	No	Depression storage for impervious surface by exp.function	Horton's	Divided into unit width strips with input length, slope & roughness		Kinematic wave	continuity	No	Arnell (1980)
13.	Belgrade	single Hyetograph	No		Horton's or coupled with subsurface flow	Strips	Strips	Nonlinear kinematic wave	Nonlinear kinematic wave	No	Radojkovic & Maksimovic (1984)

TABLE 3 : Summary of Selected Conceptual or Unit Hydrograph Based Urban Surface Runoff Models

S. No.	Model	Rain Input: Allow temporal variation	Rain Input: Allow areal variation	Abstructions: Initial losses	Abstructions: Infiltration	Surface Runoff: Impervious area contribution	Surface Runoff: Pervious area contribution	Surface Runoff: Simulation Scheme	Computer Program User's Manual Available	Selected References
1.	WASSP HYD & SIM	Yes	No	Depression storage by exp.function	percentage of rain by soil index equation	Yes	Yes	Nonlinear reservoir routing	Yes	Working party on the Hydraulic Design of strom Sewers (1981); Price (1982)
2.	(Desbordes)	Yes	No		Horton's	Lumped	Lumped	Single linear reservoir	No	Desbordes (1978)
3.	RORB	Yes	Yes	Yes	Philip's	Yes	Yes	Nonlinear reservoir	Yes	Laurenson and Mein (1983)
4.	SCS-TR55		No	Yes	SCS curve No.Method	Yes	Yes	Based on SCS non-dimensional Unit Hydrograph	No	SCS (1975)
5.	HEC-1	Yes	Yes	Yes	4 methods: constant, Horton's, SCS-CN, or Holtan's	Yes	Yes	Unit Hydrograph (kinematic wave routing option available)	Yes	HEC (1981a,b)

Note:- Street gutters are not considered seperately in all the above models.

TABLE 4: COMPARISON OF SOME MAJOR MODEL CHARACTERISTICS

	CATCHMENT HYDROLOGY							SEWER HYDROLOGY								WATER QUALITY									MISCELLANEOUS				
	MULTIPLE CATCHMENT INFLOWS	DRY WEATHER FLOW	INPUT OF SEVERAL HYETOGRAPHS	SNOWMELT	RUNOFF FROM IMPERVIOUS AREAS	RUNOFF FROM PERVIOUS AREAS	WATER BALANCE BETWEEN STORMS	FLOW ROUTING IN SEWERS	UPSTREAM AND DOWNSTREAM FLOW CONTROL	SURCHARING AND PRESSURE FLOW	DIVERSIONS	PUMPING STATIONS	STORAGE	PRINTS STAGE	PRINTS VELOCITIES	DRY WEATHER QUALITY	STORM WATER QUALITY	QUALITY ROUTING	SEDIMENTATION AND SCOUR	QUALITY REACTIONS	TREATMENT	QUALITY BALANCE BETWEEN STORMS	RECEIVING WATER FLOW SIMULATION	RECEIVING WATER QUALITY SIMULATION	CONTINUOUS SIMULATION	CAN CHOOSE TIME INTERVAL	DESIGN COMPUTATION	REAL TIME CONTROL	COMPUTER PROGRAM AVAILABLE
BRITISH ROAD RESEARCH LABORATORY	•	•	•		•			•															•			•			•
ENVIRONMENTAL PROTECTION AGENCY & VARIANTS	•	•	•		•	•		•	•	•	•	•	•	•	•	•	•	•	•	•	•		•	•		•	•		•
UNIVERSITY OF CINCINNATI CINCINNATI	•				•	•		•															•			•			•
HYDROCOMP	•	•	•	•	•	•	•	•	•		•		•	•	•	•	•	•		•		•	•	•	•	•		•	
MASSACHUSETTS INSTITUTE OF TECHNOLOGY	•	•	•	•	•	•		•		•	•		•	•	•								•			•			
UNIVERSITY OF ILLINOIS	•	•		•	•	•		•	•		•	•	•	•	•											•	•		•
BATTELLE NORTHWEST	•	•	•		•	•		•		•	•		•	•	•	•	•	•			•					•	•	•	•
CHICAGO FLOW SIMULATION	•	•	•	•	•	•	•	•					•	•									•		•	•			•
CHICAGO HYDROGRAPH METHOD	•	•	•		•	•	•	•															•			•	•		•
CORPS OF ENGINEERS (STORM)				•	•	•	•				•		•				•					•			•				•

2.4.1 Soil Conservation Services (TR-55) Procedure

Soil Conservation Services (SCS) has given the following relation between the accumulated volumes of storm rainfall runoff and catchment retention

$$Q = (P - 0.2S)^2 / (P + 0.8S) \quad \text{...(iv)}$$

where, Q = Actual direct runoff (inches)

P = Total storm rainfall (inches)

S = Potential catchment retention (inches)

Potential catchment retention (S) is related to the soil and cover condition of a watershed. These watershed characteristics are taken into consideration by an index called curve Number which is related to potential catchment retention as follows:-

$$CN = 1000 / (S + 10) \quad \text{...(v)}$$

or

$$S = 1000/CN - 10 \quad \text{...(vi)}$$

SCS developed a soil classification system that consists of four groups. which are identified by the letters A, B, C, and D. Soil characteristics that are associated with each group are as follows:

Group A : deep sand, deep loess, aggregated silts

Group B : shallow loess, sandy loam

Group C : clay loam, shallow sandy loam, soils low in organic content and soilsusually high in clay.

Group D : Soils that swell significantly when wet, heavy plastic clays and certain saline soils

The soil group can also be identified by using following minimum infiltration rate values.

Group	Minimum Infiltration Rate (in/hr.)
A	0.30 - 0.45
B	0.15 - 0.30
C	0.05 - 0.15
D	0 - 0.05

The effect of antecedent moisture condition has been taken into consideration by developing three antecedent moisture conditions, labelled as I, II and III. The following table gives seasonal rainfall limits for the three antecedent soil moisture condition.

Table 5 : Seasonal Rainfall Limits for AMC Conditions

AMC	Total 5 days Antecedent Rainfall (inches)	
	Dormant Season	Growing Seasons
I	less than 0.5	Less than 1.4
II	0.5 to 1.1	1.4 to 2.1
III	Over 1.1	Over 2.1

For a known soil group and land use pattern the curve number can be determined from Table 6 for AMC II. This curve number is modified for other antecedent moisture conditions as per the table 7.

For more complex areas a composite value of CN can be computed by knowing the percent areas of different types of land use and their corresponding curve numbers.

SCS has given charts for estimating peak rates of runoff from small watersheds of areas 1 to 2000 acres. These charts are prepared for the regions of united states having a particular type of rainfall distribution.

Table 6 - Runoff curve numbers for selected agricultura, suburban, and urban land use. (Antecedent moisture condition II, and I_a (0.2S)

LAND USE DESCRIPTION		HYDROLOGIC SOIL GROUP			
		A	B	C	D
Cultivated land : without conservation treatment		72	81	88	91
: with conservation treatment		62	71	78	81
Pasture or range land: poor condition		68	79	86	89
good condition		39	61	74	80
Meadow: good condition		30	58	71	78
Wood or Forest land: thin stand, poor cover no mulch		45	66	77	83
good cover		25	55	70	77
Open Spaces, lawns, parks, golf courses, cemeteries, etc.					
good condition: grass cover on 75% or more area		39	61	74	80
fair condition: grass cover on 50%-75% of area		49	69	79	84
Commercial and business areas (85% impervious)		89	92	94	95
Industrial districts (72% impervious)		81	88	91	93
Residential:					
Average lot size	Average % Impervious				
1/8 acre or less	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
Paved parking lots, roofs, driveways, etc.		98	98	98	98
Streets and roads:					
paved with curbs and storm sewers		98	98	98	98
gravel		76	85	89	91
dirt		72	82	87	89

Different charts are given for flat, moderate and steep catchment slope. Fig. 5 shows such a chart to estimate peak discharge for a small water-shed (area 1 to 2000 acres) having moderate slope. To adjust peak rates of runoff for ranges of flat , moderate and steep slopes, for conditions where swamps or ponding areas exist and for taking into account the variation of water-shed shape factor (ℓ/w) different adjustment factors to peak discharge are determined and applied.

Table 7 : Modified Curve Numbers for AMC I & AMC III

CN for Conditions II	Corresponding CN for condition I	III
100	100	100
95	87	99
90	78	98
85	70	97
80	63	94
75	57	91
70	51	87
65	45	83
60	40	79
55	35	75
50	31	70
45	27	65
40	23	60
35	19	55
30	15	50
25	12	45
20	9	39
15	7	33
10	4	26
5	2	17
0	0	0

The adjusted peak discharge determined by using the above procedure is modified to include the effect of urbanization. The modification factors are applied to the peaks using future condition runoff curve numbers as follows:-

$$Q_{MOD} = Q\,[\,Factor_{IMP}\,]\,[\,Factor_{HLM}\,] \qquad ..(vii)$$

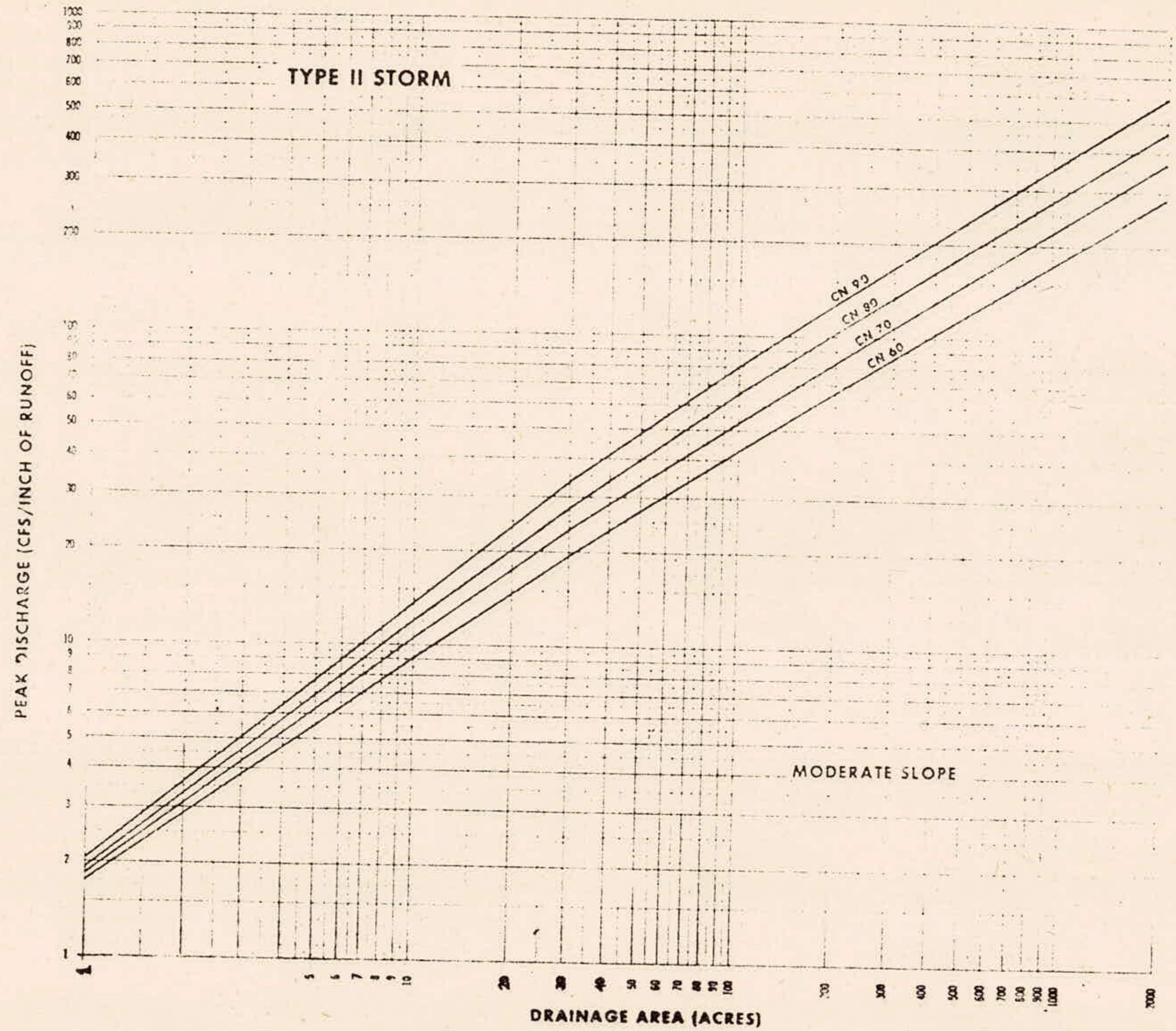


Figure 5 - Peak rates of discharge for small watersheds (24-hour, type-II storm distribution).

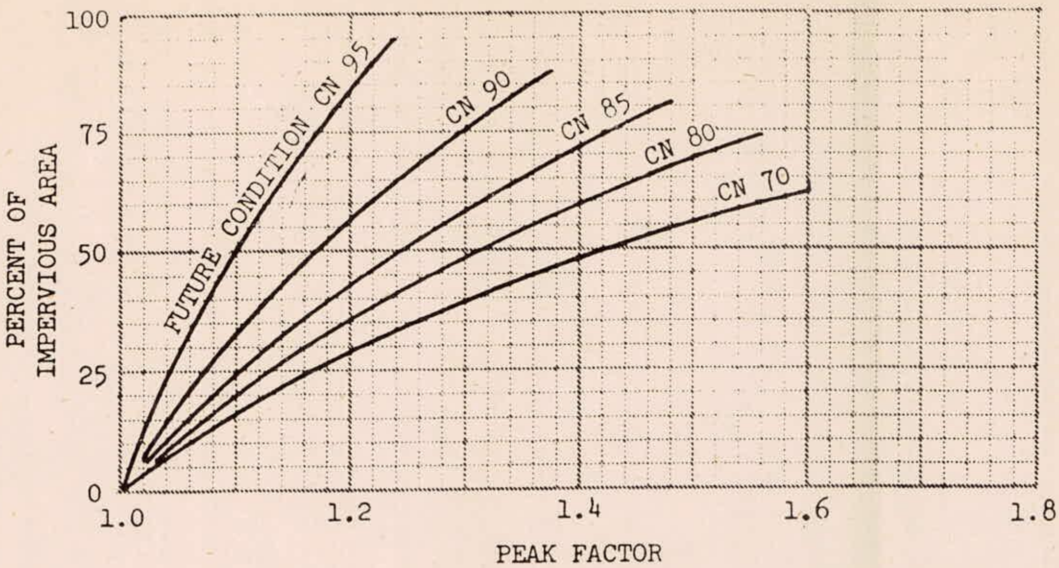


Figure 6 - Factors for adjusting peak discharges for a given future-condition runoff curve number based on the percentage of impervious area in the watershed.

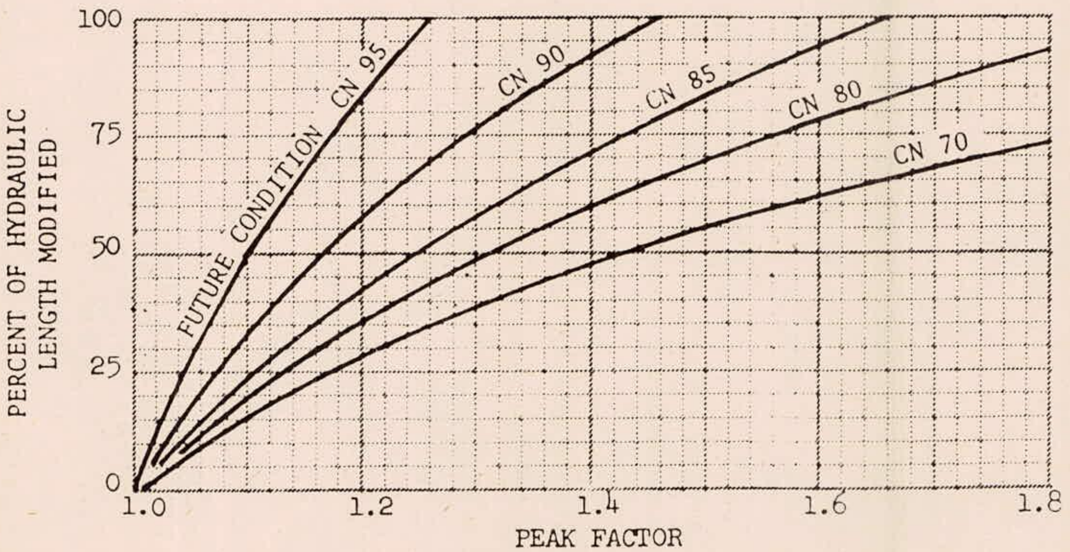


Figure 7 - Factors for adjusting peak discharges for a given future-condition runoff curve number based on the percentage of hydraulic length modified.

where

Q_{MOD} = modified discharge due to urbanization

Q = discharge for future CN adjusted for various factors

$Factor_{IMP}$ = adjustment factor for percent impervious areas

$Factor_{HLM}$ = adjustment factor for percent of hydraulic length modified.

The charts for determining these adjustment factors are shown in figures 6 and 7.

The SCS TR-55 procedure is very much simplified as it involves reading various values from charts and tables and simple calculations, but a careful understanding of charts is required. The major limitation of the method is that it can not be applied for the regions and for the conditions for which charts are not developed. The other limitation of the method is that it can be used only for small watersheds of area less than 2000 acres (20).

2.4.2 TRRL hydrograph model

This model computes the flood hydrograph considering only runoff from paved areas directly connected to the sewer system. This model has been widely used in United Kingdom where urban flooding principally results in summer from short duration, high intensity thunder storms. Due to high soil moisture deficit in summers the runoff volumes from unpaved areas are very small.

The rainfall input to the program can either be a recorded storm or a theoritical rainfall profile. The surface hydrograph is calculated using the area time diagram for the area. The surface effects are allowed for by the time of entry in addition to the time of flow. The hydrograph thus obtained is routed for the storage of water in the drainage system. The routed hydrograph represent the desired hydrograph at the outlet.

The calculations involving the rainfall profile and the time/area diagram are illustrated in Fig. 8. Diagram (1) in figure 8 shows the area divided into sub areas a,b,c, & d, and diagram (2) shows the area-time diagram which relates the area contributing to the rate of flow with the time after the start of the rainfall. This diagram is built up by the linear addition of the time area diagrams representing the sub areas (the lines labelled 4,3,2 and 1 in the diagram). The intercepts between these lines on the time axis correspond to the time of flow along the trunk sewers in sub area a,b, and c. Diagram (3) shows a rainfall profile and both this and the area-time diagram are divided into unit times normally one minute for the very large areas when longer unit times may be used. Diagram (4) shows the unrouted hydrograph obtained through the calculations as shown in the figure.

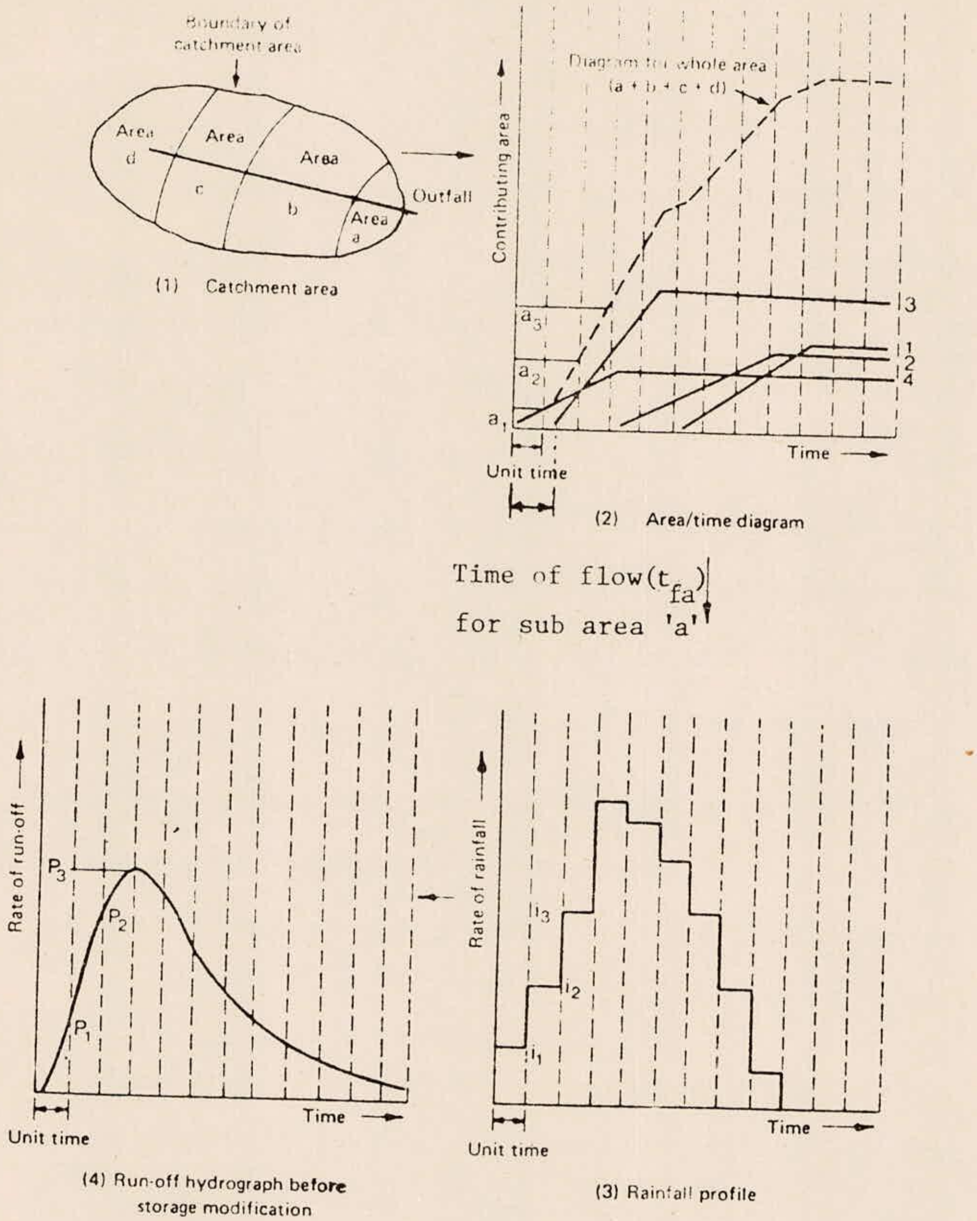
The next step is to route the hydrograph thus calculated through storage. The maximum storage being the volume in the sewers occupied by water at the peak rate of runoff.

The reservoir equation for routing the hydrograph is

$$P - Q = dR/dt \qquad \text{...(viii)}$$

Where P & Q are the rates of runoff at any time t for the unrouted and routed hydrographs respectively, and R represents the storage in the system at that time.

If the increment of time is taken to be equal to unit time 't' then, in general, the above equation can be written as



$P1 = i_1 \, a_1$

$P2 = i_2 \, a_1 + i_1 \, a_2$

$P3 = i_3 \, a_1 + i_2 \, a_2 + i_1 \, a_1$

etc.

or $\begin{bmatrix} P1 \\ P2 \\ P3 \end{bmatrix} = \begin{bmatrix} i_1 & & \\ i_2 & i_1 & \\ i_3 & i_2 & i_1 \end{bmatrix} \begin{bmatrix} a_1 \\ a_2 \\ a_3 \end{bmatrix}$

etc.

Figure 8 - Method of cauculation of runoff hydrograph unmodified for storage.

$$1/2\,(P_{n-1} + P_n)t - 1/2(Q_{n-1} + Q_n)t = R_n - R_{n-1}$$

or $$1/2\,t(P_{n-1} + P_n - Q_{n-1}) + R_{n-1} = 1/2\,t\,Q_n + R_n \qquad \ldots(ix)$$

This equation can be solved if the relation between Q and R is known for the system. Such a relation for the entire system that is yet to be designed, is not easily available. It is, therefore,assumed that at any instant the ratio of the depth of water to the maximum possible depth is the same for all the pipes in the system, i.e. the proportional depth of water in the whole system is constant at any instant. This assumption helps to find a satisfactory solution to the above equation (22). Alternatively a more general solution is possible if the assumption of constant proportional depth is maintained over only a single pipe and the hydrograph is calculated as follows -

(i) The area/time diagram for the uppermost pipe is calculated in the normal manner. The unmodified or surface hydrograph is computed and routed through the pipe storage giving an outflow hydrograph which is stored in the computer.

(ii) For the second pipe a surface hydrograph is computed as if were the first pipe on a branch. The inflow from the upper pipe is then recalled and added to this hydrograph which is then routed through the storage of the second pipe, and the resulting outlfow once again stored.

(iii) The procedure is repeated for all the pipes in the system. The Fig. 9 shows the method of routing the runoff hydrograph and fig. 10 shows the effect of storage routing on the hydrograph.

As reported by Hall, M.J. (8), the TRRL method has not been with-

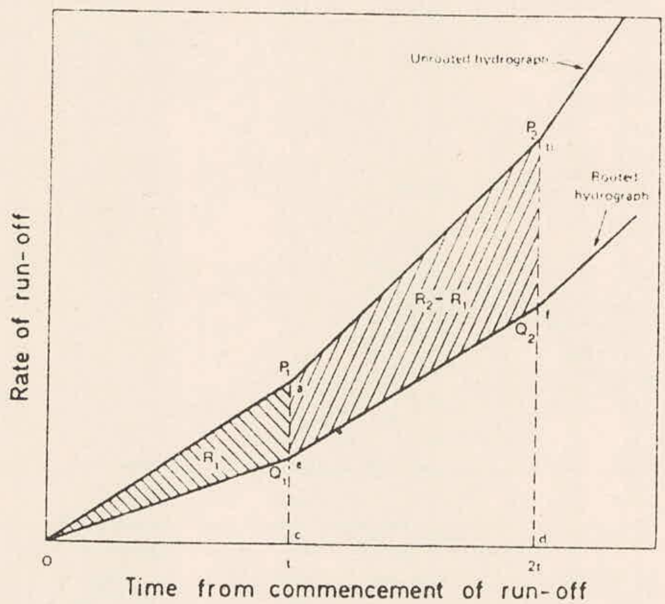
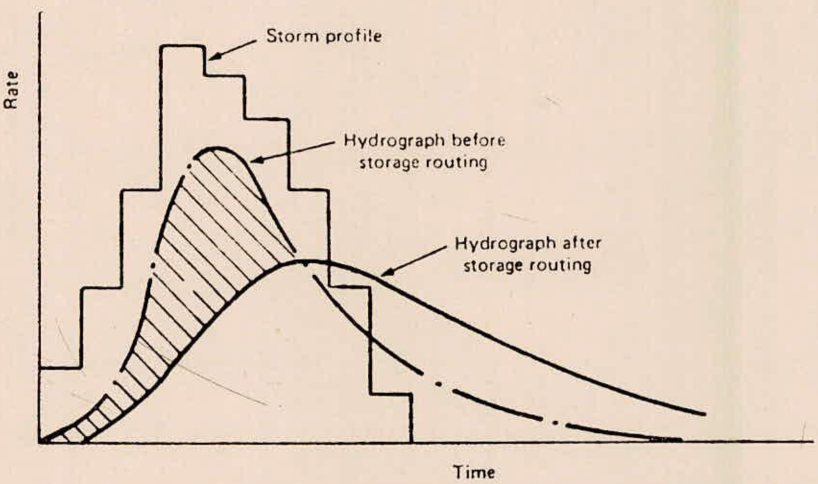


Fig. 9 Routing the runoff hydrograph



The shaded area represents the maximum retention in the sewer system ie the volume of water in the sewers at time of maximum rate of run-off

Fig. 10 The effect on the hydrograph of the storage routing procedure out its critics. Escritt and Young (1963) contended that the time-area diagram implicitly allowed for storage in the sewerage system and to route the unmodified hydrograph was to allow for the same volume of storage twice. According to these authors the attenuation observed in recorded storm hydrographs was attributable to surface storage. However the form of the conceptual model underlying the TRRL Hydrograph method has been amply justified by the comparison between the observed and reconstituted hydrograph carried out during the original study (Watkins, 1962, 1963), and by the encouraging results of independent trials of the method in the United States (Terstriep and Stall, 1969), Australia (Aitken, 1973, Heeps and Mein, 1973, 1974) and Canada (Marsalek et.al., 1975). In addition, an appraisal by Colyer (1977) of the published information has shown that the TRRL Hydrograph method appeared to be more reliable in simulating recorded flood hydrographs than several other computer based procedures developed subsequently in other countries. However, these extensive trials of the method have shown that, other than in temperate climates, neglecting the runoff from the pervious areas within the catchment can lead to an underestimation of peak discharges. Watkin (1976) has since suggested an additional modification to the TRRL Hydrograph Method for tropical climates in which the runoff from the pervious portion of the catchment is modelled by means of a linear reservoir (Watkins & Fiddes, 1978).

The major points of criticism of TRRL Hydrograph Method are

(i) The representation of the above ground phase of runoff by a time of entry.

(ii) The assumption of 100% runoff from the paved and no runoff from the pervious areas of a catchment.

(iii) Storage allowances based solely on the pipe system with no attenuation attributed to the above ground storages.

(iv) The assumption that the storm profile of a selected return period produces a peak discharge of the same return period.

Other limitations of TRRL method are that it does not have a satisfactory facility for computing enhanced flow due to surcharge. Also the TRRL method was developed initially as a design method rather than a simulation method and it follows that although the assumption of 100% runoff from paved areas may be valid for some specific regions like U.K. but not valid for simulation & a procedure for calculating percentage runoff is required.

2.4.3 Distributed routing rainfall-runoff model (U S G S model)

This watershed model routes the urban flood discharges through a branched system of pipes or natural channels using rainfall as input. The model developed and documented by Dawdy et. al. (5) combines soil moisture accounting and rainfall-excess components with the kinematic-wave routing method presented by Leclere & Schaake.

This model can be divided into four major components -

(a) a soil moisture - accounting component

b) a rainfall-excess component

c) a routing component

d) an optimization component.

The antecedent moisture condition is an important parameter which has been taken into account in this model. The figure 11 shows how significantly the runoff is affected by the antecedent moisture condition. Impervious surfaces as well as pervious surfaces both have been taken into account

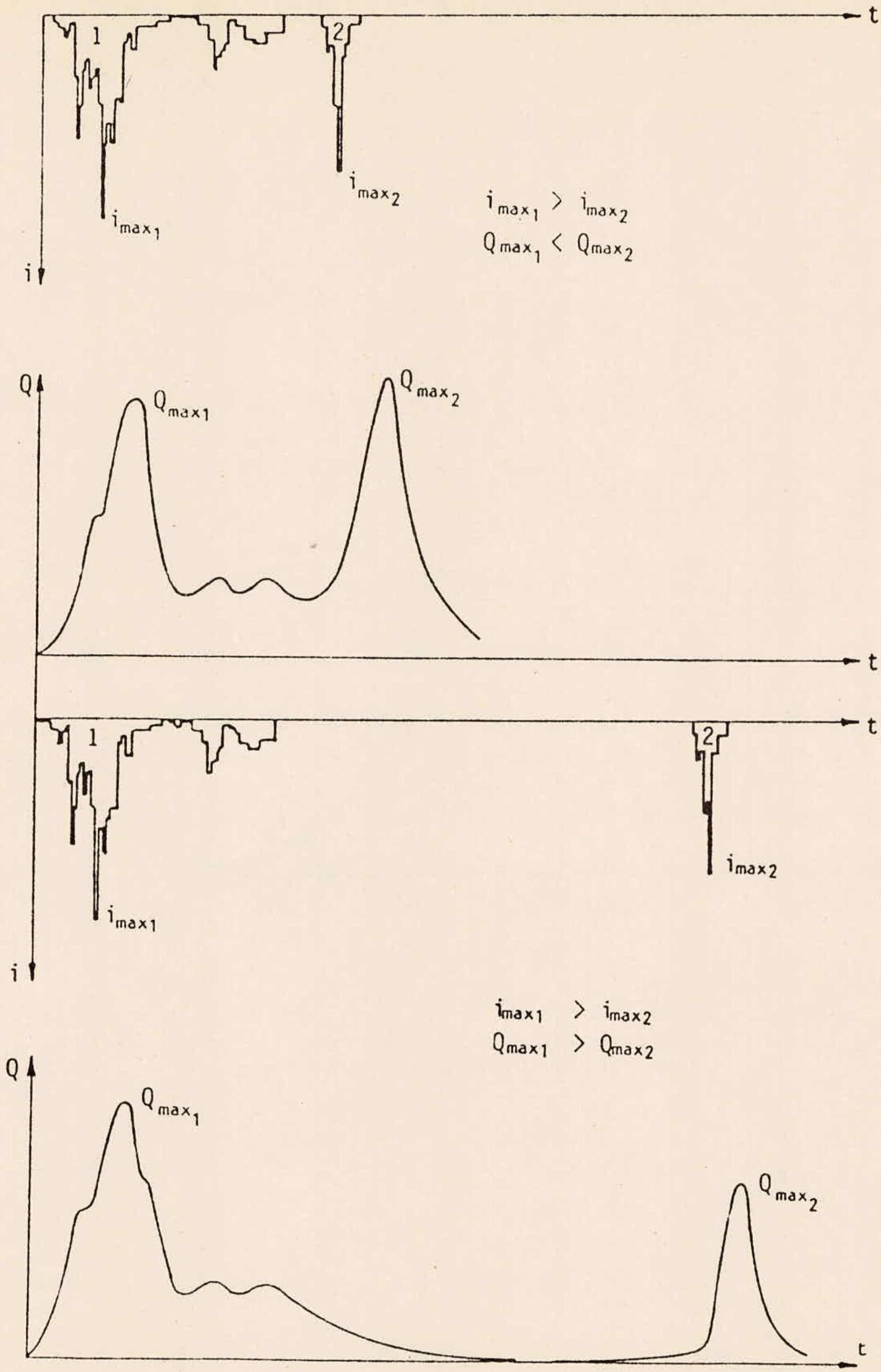


Figure 11 - Effect of antecedent moisture on runoff

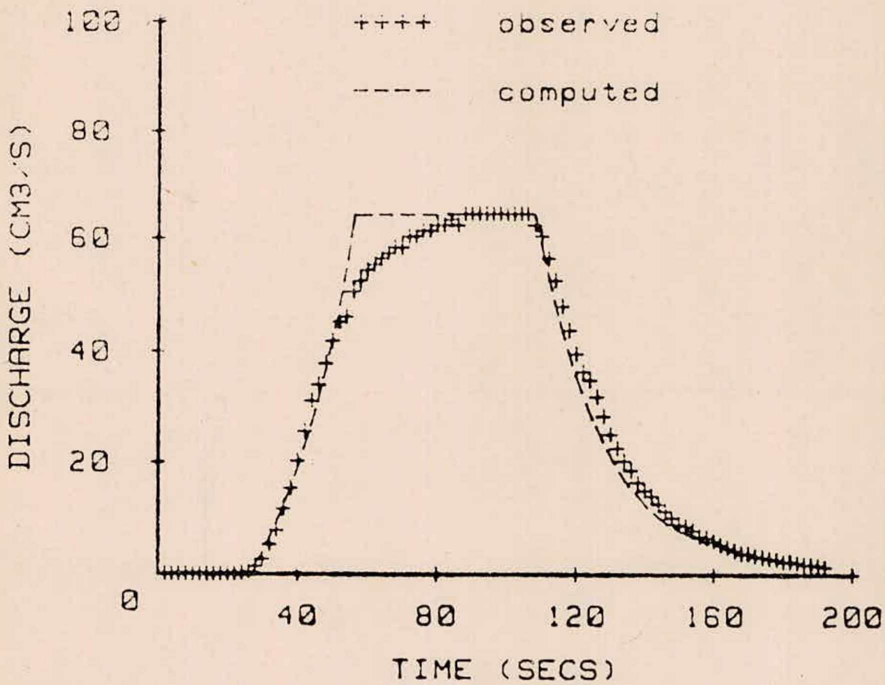


Figure 12 - Comparision of observed and computed prototype hydrographs using kinematic wave equation

while determining the rainfall excess. Two types of impervious surfaces have been considered by the model, first type, effective impervious surfaces are those impervious areas which are directly connected to the channel drainage system and second type, non effective impervious surfaces are those areas which drain to pervious areas.

Kinematic wave theory is applied for both overland-flow and channel routing. Muzik conducted some experiments on laboratory catchment and found that the kinematic wave equations represent individual hydrographs reasonably well. Figure 12 compares the observed and the computed hydrographs for a laboratory catchment. The USGS model employs four point finite difference scheme to solve the kinematic wave equations.

The model includes an option to calibrate the soil moisture and infiltration parameters for drainage basins having observed rainfall-runoff data. Rosenbrock's optimization technique is used to determine the optimum parameter values. Further details about the model are given in Reference(5) along with the listing of the program.

2.4.4. Hydrocomp simulation program - fortron (HSPF)

This model is a version of the Stanford Watershed Model and includes a complete water balance within the study area. It accounts for both surface water and ground water and for exchanges and interactions between them. The kinematic wave method is used for all surface water routing. The water quality aspects are taken into account in this model. Special emphasis has been given to the nutrient cycle and the lower forms of plant and animal life. This model is expensive to operate because it required an extensive data base for proper calibration.

2.4.5 Storm water management model (SWMM)

This model is not a single model but a package of models linked together by an executive program. It was originally developed under sponsorship of the U.S. EPA and now maintained by university of Florida for updating, documentation and user assistance. It is divided into a number of blocks, some of which may be run on their own or in series with others. These blocks are briefly described below -

(i) Executive Block, which controls the running and links other blocks.

(ii) Runoff Block, which models flood flows off pervious or impervious ground, in gutters, drains and channels. It is based on a numerical solution of the kinematic equations. The quantity and quality may be simulated and hydrograph at any point in the system may be displayed. The structure of the model is illustrated in Fig. 13.

(iii) Transport Block - This is a more refined routing subroutine and allows for overflowing manholes, backwatering and flow in non uniform channels and rivers.

(iv) Storage/Treatment Block - The waters may be stored to alleviate floods, and treated to reduce pollutants. This model simulates the effects of such storages and treatments.

(v) Receiving water block - The circulation in lakes may be studied considering hydraulic gradients, wind effects, overflows and numerous sources of inflow.

This model translate rainfall hyetographs into complete hydrographs but the model requirements are difficult to understand. The cost of model operation is also high.

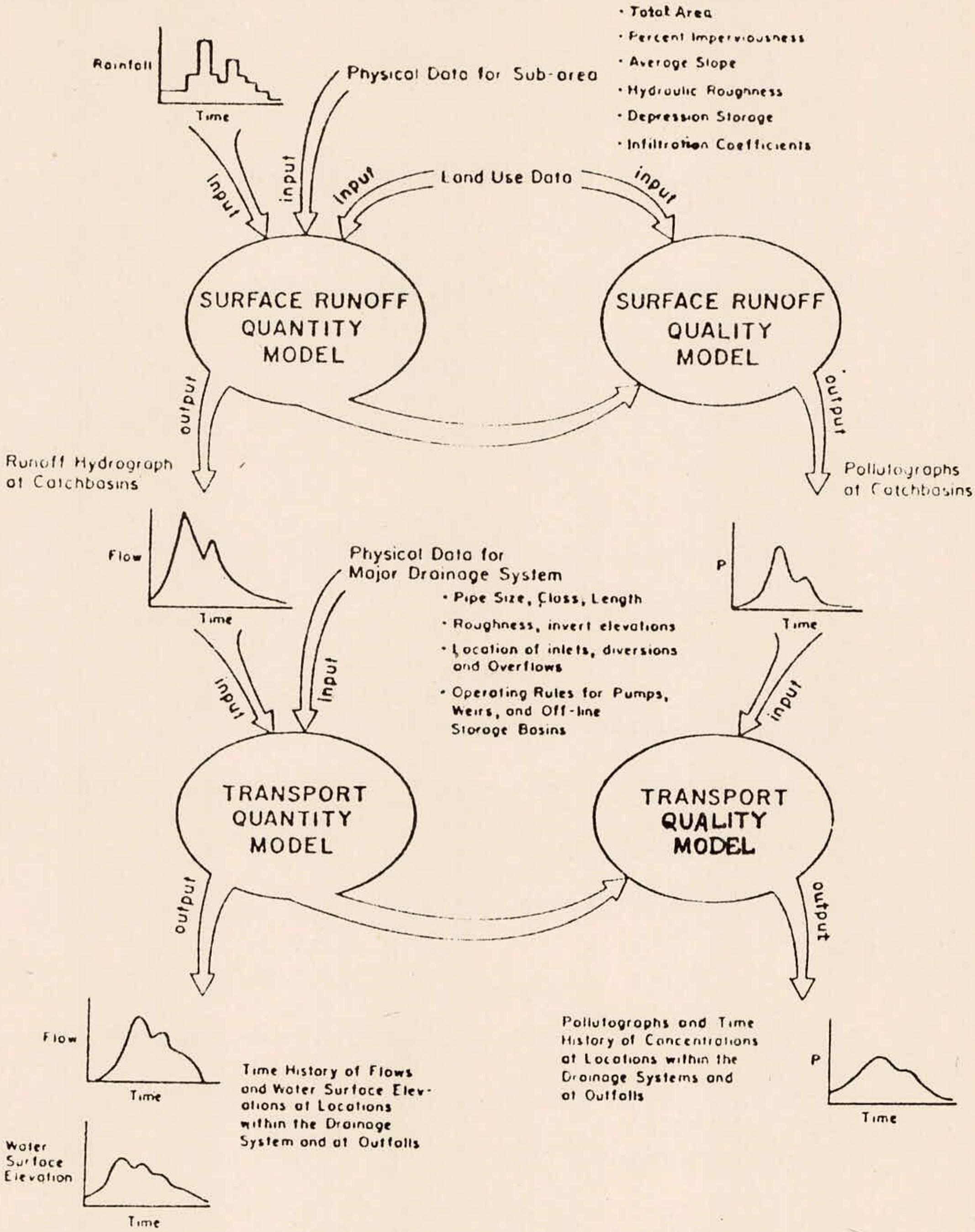


Figure 13 - Sturcture of SWMM surface runoff and transport models

2.4.6 Illinois urban drainage area simulator (ILLUDAS)

This model is a derivative of TRRL hydrograph model and among the more widely used storm sewer design methods in the USA. This model has a facility for computing runoff from unpaved areas also. The procedure of this facility is first to calculate soil infiltration and storage losses and then to develop an area/time relationship for the unpaved areas in a form in which it can be fed into the TRRL procedure.

ILLUDAS assumes that depression storage is filled before any infiltration takes place. Standard infiltration curves based on Horton equation

$$f = fc + (fo - fc)\ e^{-kt} \qquad \ldots(x)$$

where,

fo = initial infiltration rate

fc = final steady state infiltration rate

f = infiltration rate at any time t from the beginning of rainfall

k = shape factor

t = time from start of rainfall

are used.

ILLUDAS then proceeds to compute a hydrograph for the unpaved area and to combine it with the paved area hydrograph and then routes the contribution through storage in the manner of the TRRL method.

The advantages of this model are that both the paved and unpaved areas are considered, data input is straight forward and storage effects are simulated and running costs are low.

2.4.7 Wallingford procedure

It is a complete package containing four methods:

(i) Wallingford rational method:

A modified version of the Rational Method which is intended for use on homogeneous areas of upto 150 hactares:

In this method the peak discharge is determined as

$$Q_p = 2.78\ C_v\ C_R\ i\ A \qquad \text{...(xi)}$$

where C_v is the volumetric runoff coefficient

C_R is a routing coefficient which allows for non linearity in the shape of the time area diagram and variations in the rainfall intensity within the time of concentration

Q_p is peak rate of flow (m^3/sec)

A is catchment area (Km^2)

i is rate of rainfall (cm/hr)

The value of C_v is computed from

$$C_v = PR/100 \qquad \text{...(xii)}$$

where PR, the percentage runoff, is given by

$$PR = 0.829\ IMP + 25.0\ SOIL + 0.078\ UCWI - 20.7 \qquad \text{...(xiii)}$$

Here IMP is the percentage Impervious area of catchment draining to the sewer.

SOIL is a soil Indix taken from a map published by the Institute of Hydrology for U.K.

UCWI is an antecedent wetness Index which for design purposes is obtained from a relationship with the annual average rainfall.

If only impervious area is considered

$$C_v = PR/IMP \qquad \text{...(xiv)}$$

For C_R a value of 1.3 has been recommended for design purposes.

(ii) Wallingford hydrograph method - It is a computer based approach which models the above ground and below ground runoff separately. This method may be employed for both design and simulation and allowances may also be made for the action of storm water overflows, on line and off line detention tanks and pumping stations. This method considers the criticisms made towards TRRL method. The relationship between the return period of the causative design storm is maintained by the use of a stable set of design inputs. The latter have been chosen by applying a technique described by Packman and Kidd involving the comparison of observed and computed probability distributions of peak flow rates. Criticisms regarding the ground phase of runoff, assumption of 100% runoff from paved area and storage allowances, all are countered by the separate modelling of the above ground and below ground phases of runoff. Figure (14) illustrates the modelling of the above ground phase of runoff in Wallingford Hydrograph Method. Hall, M.J. (8) gives further details about the wallingford Hydrograph Method.

(iii) Wallingford optimising method - It is a computer based technique for obtaining the pipe diameter, depth and gradient associated with the minimum construction cost using the discrete differential dynamic programming technique.

(iv) The wallingford simulation program - It is also a computer based method with which the performance of both an existing system and a proposed design may be examined under surcharged conditions. The storm water overflows, on-line and off-line detention tanks and pumping stations may also be taken into account.

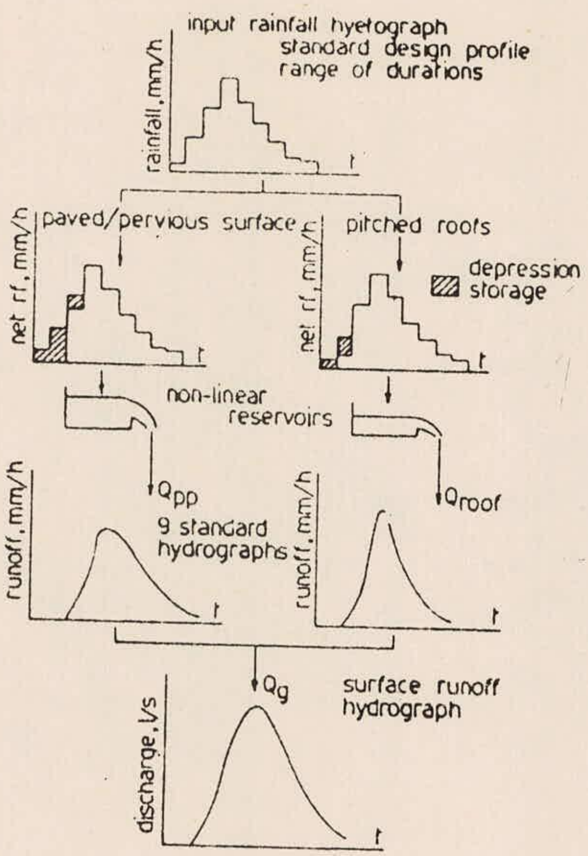


Figure 14 - The Wallingford Hydrograph Method: the above-ground phase of runoff.

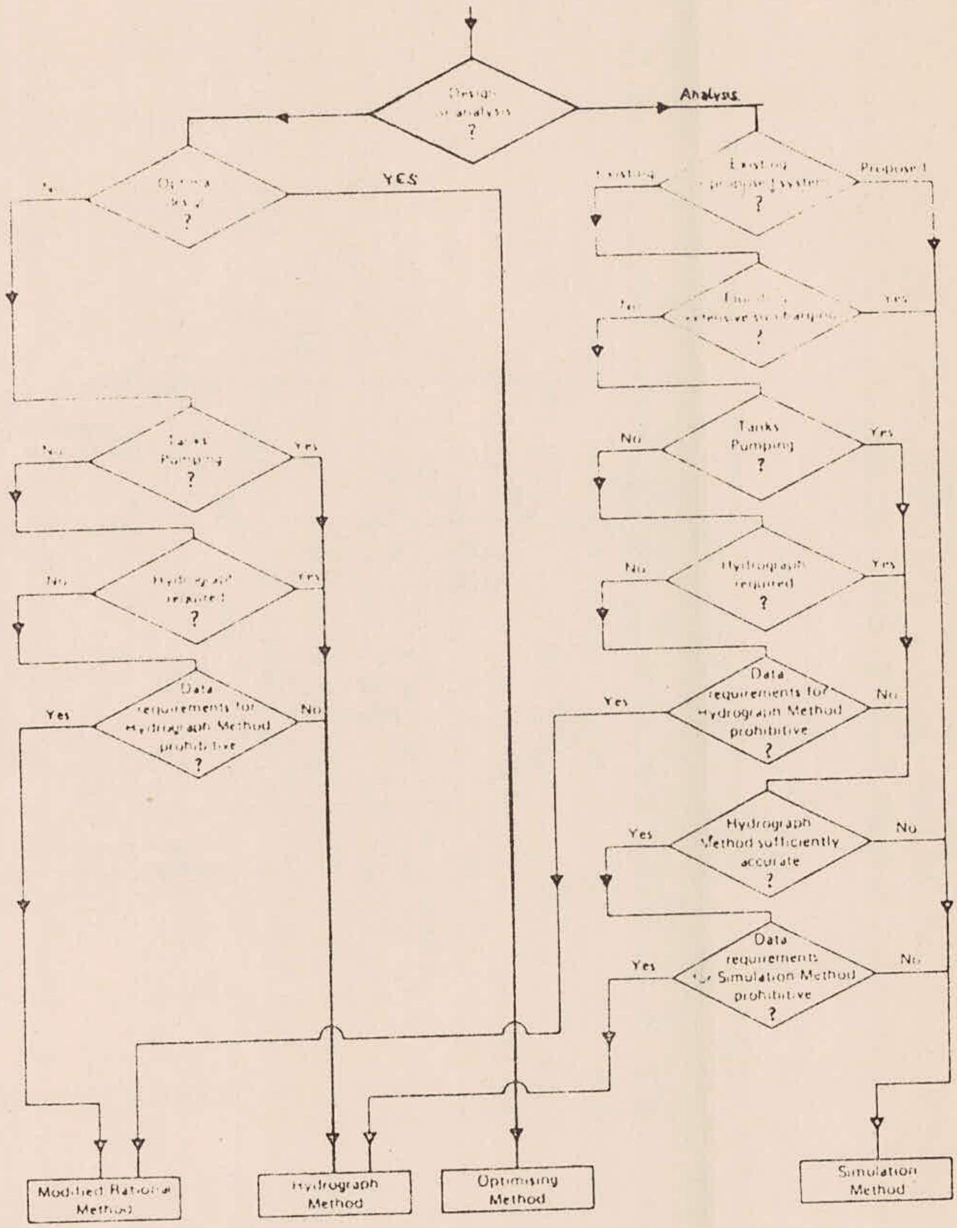


Figure 15 - Selection of a method in Wallingford procedure

These methods may be applied to both separate and combined sewerage systems. The flow chart illustrating the selection of the Wallingford method is shown in Fig. 15.

The procedure provides a very pr cise and cómplete package which is responsible for increased complexity and cost. Where insufficient data are available to permit the modelling of above ground and below ground phase of runoff of every subcatchment and pipe length, a simplified sub-area model is available. Using this sewered sub-area model substantial savings on input data are possible (8).

2.4.8 TVA continuous daily stream flow model:

TVA daily streamflow model is basically a simple water budget model for estimation of storm water runoff. Daily rainfall is budgeted among a series of conventional cascading compartments or reservoirs. It differs from some flow models in that interflow is not included and there is only a single soil moisture reservoir. Input consists of daily rainfall and streamflow and monthly evapotranspiration for analysis runs. Outputs from the system consists of daily, monthly and annual Stream Flows. The model parameters and constants are listed in Table 8. A schematic diagram for continuous daily stream flow model is shown in Figure 16.

i) Interception Storage: It has a deterministic variation in the model. All incoming moisture enters interception storage until a preassigned volume is filled. Values from 0.13 to 0.64 cm have been found to be reasonable for forested watersheds.

ii) Storm Runoff Volumes (Pervious Area)

The following relationship has been used for predicting storm runoff from urban areas based upon the portion of watershed that is impervious:

TABLE-8

CONTINUOUS DAILY STREAMFLOW MODEL

PARAMETERS AND CONSTANTS

Primary Model Parameters

1. B = a volumetric parameter used to preserve mass balance,
2. AW = a winter storm runoff volume parameter
3. DS = a summer storm runoff volume parameter
4. GWK = a groundwater volume parameter
5. TDSRO = a storm runoff routing parameter

Model Constants

1. SROK = storm runoff recession constant
2. GROKW = winter ground-water recession constant
3. GROKS = summer ground-water recession constant
4. GWDOR = dormant season ground-water reservoir allocation constant
5. AHORD = soil A horizon moisture storage capacity
6. BHORP = soil B horizon daily permeability
7. DLF = bypass loss constant
8. TLP = transmission loss parameter
9. PKARST = pervious-area runoff loss parameter

Model Descriptors

1. ACREIN = drainage area in square miles
2. WCEPT = winter interception capacity
3. SCEPT = summer interception capacity
4. PIMP = fraction of watershed impervious
5. FALL, WINTER, SUMMER, SPRING = day of year for beginning of respective season (beginning October 1)

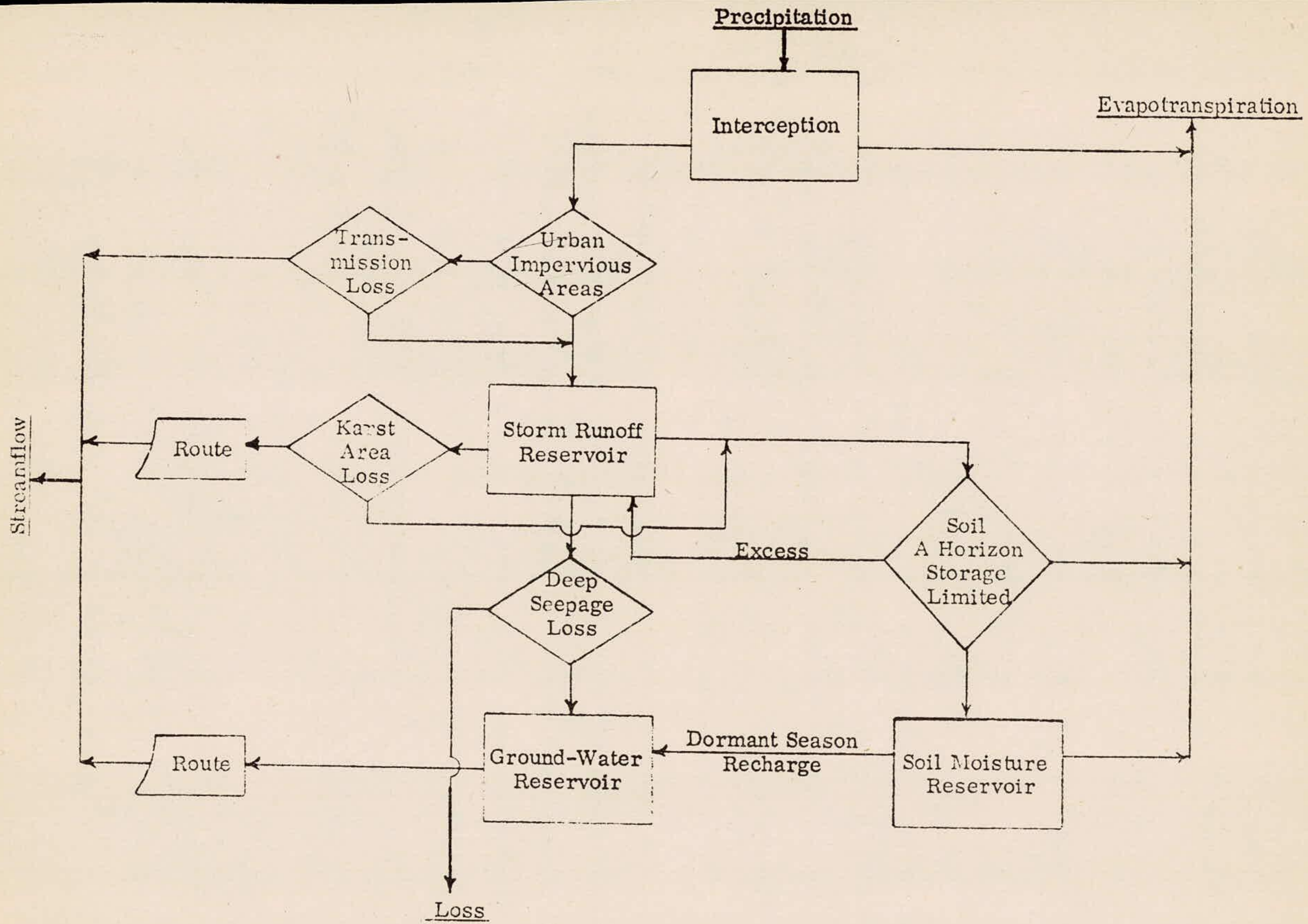


Figure 16 - Continuous Daily Streamflow Model Schematic

$$PSRO = RFr \times 1.165 \times PIMP \qquad \ldots(xv)$$

$$PIMP = (IMP-0.17);\ PIMP > 0 \qquad \ldots(xvi)$$

where

PSRO = Storm runoff from impervious area, cm

RFr = residual rainfall, cm

PIMP = impervious fraction of watershed >0.17

IMP = total impervious fraction of watershed.

Impervious area runoff is assumed to become streamflow on the day of the rain. It is not delayed through routing because at small watersheds where urbanization can be an important factor it runs off rapidly and at large watersheds the impervious area is usually only a small fraction of watershed.

iii) Storm runoff volume (Pervious Areas)

The algorithm used in this part, allocates storm runoff from pervious areas in proportion to the amount of moisture stored in the soil moisture and ground water reservoirs of the model. The algorithm is an adoptation of a rational storm runoff model presented by Betsen et al (1969):

$$RI = (AW + (DS-AW)^{*} SI)e^{-B(SMI + GWR)} \qquad \ldots(xvii)$$

$$SURVOL = (RF_r^2 + RI^2)^{0.5} - RI \qquad \ldots(xviii)$$

where,

RI = retention index, cm

AW = a parameter associated with winter storms, cm

Ds = a parameter associated with summer storms, cm

B = a parameter used to force continuity, cm^{-1}

SI = a seasonal phenolgic index that equals one in summer and zero in winter

SMI = the moisture stored in the soil moisture compartment.

GWR = the volume of water stored in the ground water reservoir, cm

SURVOL = daily storm runoff to be routed, cm

RF_r = residual rainfall, cm

The retention index, RI, is related to physical watershed characteristics and to antecedent conditions. The two coefficients AW and DS are parametric seasonal indices of the moisture storage capabilities of the soil. The parameter B is determined in the model to conserve mass balance between the predicted and the observed total runoff volumes when the model is used analytically. The seasonal variable SI is associated with crop conditions and is used to differentiate between winter and summer. Interpolations between zero (winter) and one (summer) are made for different seasons.

(iv) Groundwater Runoff Volumes

After interception storage and storm runoff volume have been estimated, the remaining precipitation then becomes a potential for groundwater runoff. This portion is assumed to be proportional to the yield of storm runoff :

$$GWV = (SURVOL * GWK/RF) * RF^r \text{ and } GWV < RF^r \qquad \ldots(xix)$$

where

GWV = a volume to be added to the groundwater reservoir, cm

GWK	=	a parameter which relates the yield of groundwater runoff to the yield of pervious area storm runoff.
RF	=	Rainfall-interception, cm
RF^r	=	the available moisture after interception and storm runoff have been removed from precipitation, cm

(v) Dormant season recharge:

For watershed with a high soil water holding capacity (clay & loam soils) a recharge of the ground water can occur as vegetation becomes doemant. During the period moisture held in the soil under tension by the vegetation is released and becomes groundwater runoff. In the model these accretions are taken from the soil moisture reservoir at a daily rate, GWDOR and added to the ground water reservoir.

(vi) Potential runoff volume losses:

Losses of potential runoff volumes can occur for a variety of reasons. Deep losses are those that bypass the streamgauge and thus are lost from the system.

$$GWL = GWV \times DLF \qquad \ldots (xx)$$

where

GWL	=	by pass losses
DLF	=	a parameter equal to zero where no losses occur and equal to one where no groundwater runoff occurs.,

Transmission losses occur when potential storm runoff originating from impervious areas does not reach the streamgage. This effect is most pronounced when runoff volumes originating from roof, roads, etc. infiltrate into lawns or other pervious surfaces or into dry stream channel. The equation for correcting runoff volume is:

$$PSRO = (PRSO/TLP)* PSRO = PSRO^2/TLP \qquad \ldots(xxi)$$

$$PSRO/TLP < 1.0$$

where

TLP = a transmission loss parameter

(vii) Evapotranspiration

Monthly evapotranspiration values are used as input to the model.

$$RF - RO = ET = K \quad (EP_c * GI_i) \qquad \ldots(xxii)$$

where,

RF = average annual rainfall

RO = average annual streamflow

ET = annual evapotranspiration

K = factor, preserves mass balance of evapotranspiration according to long term records

EP_i = average monthly pan evaporation

GI_i = growth index of crop

(vii) Runoff routing:

The daily storm runoff and groundwater runoff volumes are determined using conventional exponential routing coefficients. Storm water runoff volumes originating from impervious areas become streamflow on the day of the rain. Runoff volume originating from pervious areas are estimated as follows:

$$SRO \quad TDSRO * SURVOL_i + SURES_i * (1-SROK) \qquad \ldots(xxiii)$$

where,

SRO = routed storm runoff, cm

TDSRO = a model parameter

SURES = storm runoff reservoir, cm

SPOK = a storm runoff recession parameter

Groundwater is routed daily from the groundwater reservoir by using a recession constant

$$GRO_i = GWR_i * (1\text{-}GROK) \quad \ldots(xxiv)$$

where

GWR = groundwater reservoir

GROK = groundwater recession constant

(ix) Optimization

A modified version of the pattern search technique is used to determine a optimal set of parameters during analytic runs with the model. The objective function used in the model is a minimization of the sums of squares of the errors between predicted and observed daily streamflow values.

3.0 RUNOFF ESTIMATION PRACTICES IN INDIA

3.1 Preliminary Remarks

In a developing country like India, the priorities for economic development and investment are for food, shelter, clothing, health and education hence urban drainage is generally not given serious attention except when it affects significantly any of the above factors. However, recently, there is an increased awareness in India in the problems of urbanization particularly with reference to the impact of urbanization on ecology and environment. This has given rise to greater interest in urban drainage and related aspects.

3.2 General Design Practice

The Rational Formula which was given by Kuichling in 1989 is generally used in India to estimate the design peak flow. The three factors affecting the design flow in the use of the rational formula are the coefficient of runoff, the rainfall duration or the time of concentration, and the frequency of the design rainfall. There is no uniformity in the estimation of these parameters in India (16). The practice of estimation of these parameters is as follows:

(i) Coefficient of runoff:

The coefficient of runoff defines the ratio of runoff to rainfall and usually estimated on an empirical basis. The values used vary from around 15% for predominantly agricultural area to 60% to 70% in the case of densely paved and hilly areas. The recommendations differ widely, and they are much larger than values deduced from observations (16). Table 9 shows the range of values in use.

TABLE 9 : RUNOFF FACTORS FOR URBAN AREAS IN INDIA

	Example	Runoff Factors, percent		
		Population Density, Persons/hectare		
		< 370	370-618	>618
i)	Delhi Master Plan	35 %	45%	60%
ii)	Najafgarh Drain	45%	60%	60%
iii)	Shahdara Drain	60% to 40% depending upon soil type		
iv)	Uttar Pradesh	50%	50%	50%
v)	Bihar	40% to 60% depending upon built-up area		
vi)	Patna	50% to 70% depending upon built-up area.		
vii)	Calcutta	Composite values as function of duration of storm and percentage of impervious area.		

(ii) Rainfall duration:

Because rainfall intensity decreases with increasing duration, the duration of a design rainfall becomes an important factor. When surface drainages are used, the rainfall duration is assumed to be equal to the time of concentration for the basin, which is generally estimated by empirical equations. Generally the response time of sewered urban catchments is small and in such cases the small error in the estimation of time of concentration affects the peak runoff discharge largely. As an alternative to the use of empirical equations, an inlet time of 15 to 30 minutes and a travel velocity of 0.6 to 0.9 meters per second are recommended (16). Sometimes the duration is specified arbitrarily, for example 6 hours in Uttar Pradesh and 1 hour for Delhi.

(iii) Frequency of rainfall - The frequency of rainfall for rural and urban areas is generally adopted as five years and two years respectively. Table 10 shows the range of values in use. However a recurrence interval of two months has been adopted in the case of Calcutta because an improvement to a three month frequency capacity would have been 70% more costly

Table 10 - Frequency of Design Rainfall

Example	Rainfall frequency for Urband Drainage
1. Najafgarh & Shahdara Drains	5 Yrs
2. Patna	5 Yrs
3. Delhi-larger drains (area,20 ha)	5 Yrs
-Small drains	2 Yrs
4. Uttar Pradesh	2 Yrs
5. Gauhati	6 months
6. Calcutta	2 months

than for two months frequency capacity. Since flooding occurs only in three monsoon months, therefore actual return period for the design flood is only a fortnight. Similarly, in Delhi many closed drains had been designed for a two years frequency because of the reason of high cost. These experiences indicate that a combined system in India is very costly. In view of this, design standards for Delhi and Bihar recommended only open drains, lined if necessary, in urban drainage systems.

3.2.1 Limitations of the rational method

The developments in the field of hydrology since the introduction of Rational Formula have revealed that the Rational Method is adequate for approximating the peak rate of runoff from a rain storm in a given basin with in certain limits. The greatest drawback of the Rational Method is that it normally provides only one point on the runoff hydrograph. Viz the peak runoff. In complex storm water drainage systems which involve diversions from one catchment to another, flood storage, off-channel storage, permanent storage, pumping installations, land use planning etc., the knowledge of the entire flood hydrograph rather than peak flow alone is required. More over the assumptions of taking averaged uniform intensity as the design rainfall intensity is justified only for small basins. Normally the application of Rational Formula is justified for the basins of area 40 hectares or less.

3.3. Mathematical Modelling

A non linear hydrologic model [eqn. (xxv)] has been developed

$$S = K_1 q^N + K_2 \, dq/dt \qquad .(xxv)$$

for the storage in the combined sewer and drainage system of Calcutta town, by relating the effective rainfall during a storm to the record of pumpage from the storm water pumps. The parameter N was nearly constant but the parameter K_1 & K_2 were not constant and therefore were correlated by regression analysis in terms of storm characteristics such as total rainfall excess, the duration of rainfall excess, and the time distribution of rainfall excess in terms of the time to centroid of the rainfall excess hyetograph and a shape factor. The results indicated that the hydraulic capacity of the system is very inadequate, leading to frequent flooding of streets. This agrees with the fact that the design capacity provided corresponds to a two months recurrence interval.

3.3.1 Physical Simulation:

The data requirement for mathematical simulation of an urban drainage system is very exhaustive. Since sufficient hydrological data are not available for urban areas in India, therefore, no significant progress in this direction has been made so far.

Though the hydrologic model ILLUDAS has been attempted for analysis and design of some urban drainage systems in the country but in general, the academic and research institutions have yet to evince commensurate interest in this area.

4.0 CONCLUSION

In this Technical Note, the problem of urban drainage in its complexity has been identified. The various approaches to the problem, evolved in different countries and as reported in English literature have been reviewed. It follows from the study that the urban watersheds are generally considered as systems with rainfall as input and runoff as output. The available models for the estimation of urban runoff have been described with particular emphasis on some of the important models like TRRL Hydrograph Model, USGS Distributed Routing Rainfall Runoff Model, Storm Water Management Model etc.

The present practices of urban runoff estimation in India have been examined. It has been indicated that the existing practice of urban runoff estimation is empirical in nature and not reliable for the analysis and design of complex drainage systems. Since a number of models for the estimation of urban runoff have already been developed, tested and verified in economically developed countries, therefore, it is not necessary to develop new methodology in India, however there is a necessity to apply these models in Indian conditions with some modifications if required. It has also been pointed out that recording and maintanance of hydrological data for urban areas need special consideration.

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