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INTERCOMPARISON OF URBAN WATERSHED MODELS

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1992-93

PREFACE

India has witnessed rapid urbanization since independence. The percentage share of population in Urban areas is of the order of 23.71 percent as per 1981 census. With the increasing urbanization and rapid development of cities, the problem of urban drainage has become more complex. Due to inadequate drainage facilities in urban areas, the rain water gets accumulated in low lying areas causing problems in transportation as well as to inhabitants. Most of the big cities are situated near the river banks. During the high flood period, the river may not be capable of accommodating the drain water. Under this circumstances even if drainage system is adequate, it may not be able to drain the flood water to the river due to back water effect of river. Most of the model available in literature does not provide the solution of back water effect.

The present practices of urban storm runoff estimation in India are empirical in nature. Recently some attention has been made to use already developed mathematical model for the estimation of urban storm runoff. The primary component in designing urban drainage system is the design storm i.e. rainfall value of specified duration and return period. Extreme value of rainfall of various short durations (1 hr to 24 hrs) are required for design of urban drainage system. Calculation of the design flow of water in various parts of the system for selected rainfall input, which lead to the determination of the appropriate conduct sizes is another important component for design of urban drainage system. In this report some of the common urban drainage models like SWMM; Illinois urban drainage area simulator model, SCS Tr-55 procedure, USGS model, Wallingford model, Road Research Lab method and TVA model have been discussed and a comparison has been made regarding the suitability of the model.

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ABSTRACT

The term drainage applies to the process of removing excess water to prevent public inconvenience and to provide protection against loss to property and life. In an urbanised area runoff is contributed by (i) excess surface water after a rainfall from roofs, streets etc. and (ii) wastewater from household, commercial establishments and industries. Past practice was to convey the entire runoff through a single system known as combined sewer system. The present practice is to construct a system to discharge rainfall excess only and a seperate system to transport wastewater. In this report various urban drainage models currently used in various parts have been discussed and a comparison has been made regarding the suitability of each model. Rational method, Illinois Urban Drainage Area Simulator Model, TVA continuous daily stream flow model, Soil Conservation Services (TR-55) model, United State Geological Survey Model, the Wallingford Model consisting of Wallingford Rational Method, Wallingford Hydrograph Method, Wallingford Optimising Method, Road Research Laboratory Method and Storm Water management Model have been described in detail. The choice of which method/model is the most appropriate among the several models available are hardly straight forward. The second factor which may prevent a user from making a clear choice between different method is a lack of information on their relative performance. The results obtained by various users showed that several of the methods were capable of simulating observed event to an accuracy approaching that of the recorded data. It was found that Storm Water Management Model gives best performance but at the expense of large computer storage and time requirements.

1.0 INTRODUCTION

The term drainage applies to the process of removing excess water to prevent public inconvenience and to provide protection against loss to property and life. When a catchment area is urbanised and the amount of impervious cover in the form of roofs, roads and pavement increases, the need arises for the natural drainage network to be supplemented or even replaced completely by man made systems of pipe and paved gutters. In urban area, the runoff is mainly contributed by (i) excess surface water after a rainfall from roofs, roads, paved parking etc. (ii) wastewater from households, commercial establishments and industries. Past practice was to convey the entire runoff through a single system known as combined drainage system. The present practice is to provide system to discharge rainfall excess only and a seperate system to transport waste water. The former is known as the stormwater drainage system and the latter as sanitary drainage system.

The flood estimation methods that have applied to the design of stormwater drainage systems may be considered to fall into two broad categories (i) methods which produce only an estimate of the peak flow rate and (ii) comprehensive approaches that provide the shape of the runoff hydrograph.

1.1 Maximum Discharge Methods

Urban drainage systems in the U.K. were designed on the basis of an average rainfall intensity which was assessed to be independent of duration. However with the publication by the British Rainfall Organisation of statistical summaries of heavy rainfalls in short period from 1988 onwards the inverse relationship between the average rate of rainfall and duration became well established by observation. During the same period the first steps were taken to place urban drainage on a more scientific footing with measurements of rainfall and discharge from several catchment area being undertaken by Kuichling (1889) in the U.S. and Lloyd-Davies (1906) in England. These two studies were similar in form with flow rates being computed from records of depth at the outfall sewer and velocities estimated using a uniform flow formula.

The most common empirical formula used for estimation of peak discharge is Rational formula. The rational method presumes the existence of a time of concentration for every drainage area which is defined as the time taken for flow from the most remote point in the catchment to reach its outfall the peak discharge, Qp, is then assumed to occur when whole of the drainage area contributes to the flow i.e. after an interval from the beginning of rainfall equal to the time of concentration. The magnitude of the peak flow rate is taken to be proportional to the effective rainfall or rainfall excess i.e. the total rainfall minus the losses during the time of concentration.

$$Q_p = 2.78\ CiA$$

where

A	=	total catchment area in ha
i	=	average rainfall rate mm/hr
C	=	runoff coefficient having a value less than unity, and
Q_p	=	peak flow rate lit/sec.

When the Rational method is applied to the design of urban drainage system, the time of concentration is normally estimated from the sum of the time of flow in the sewer and a time of entry.

The Rational method is also known to yield erroneous results under certain design condition in particular, for drainage systems in which the contributing area does not increase uniformly with time, the highest peak runoff rate may be produced by a design storm whose duration is less than the time of concentration. The Rational method does not take into account variations in time of

i) rainfall intensity
ii) flow velocity
iii) temporary storage in the sewer system
iv) the rate of increase in the contributing area.

Watkins (1962) concluded that the rational method is only suitable for design purposes when the drainage areas are sufficiently small for pipe diameters not to exceed 61 cms. Although to refuse the flood estimate provided by the rational method have concentrated largely on the rate of increase in the contributing area, with the use of a plot showing the variation with time from the beginning of the storm of the area of the catchment contributing to the flow at the outfall termed as time area diagram. The time area diagram has formed the basin for two distinct types of methods, the first of which may be referred to as the Tangent Methods. The second group of methods which employs the time area diagram, known as the typical storm methods differs from the Tangent methods in producing a runoff hydrograph and not just an estimate of the peak flow rate.

1.2 Design Hydrograph Method

The development of flood hydrograph estimation methods for urban drainage system may be considered to consist of two seperate phases, namely typical Storm Methods. The latter differe primarily in the distribution of rainfall which is assumed for a specified return period. The method involves the drawing of isochrones i.e. line of equal travel time on a map of drainage area using a time increment. The areas between adjacent isochrones are then measured. Assuming that the storm profile consists of a series of average rainfall intensities, l_1, l_2, l_3 .... with successive time increments of t, the ordinates of the discharge hydrograph may be written as

$$Q_1 = C i_1 A_1$$
$$Q_2 = C i A_2 + C i_2 A_1$$
$$Q_3 = C i A_3 + C i_2 A_2 + C i_3 A_1$$

where C is the runoff coefficient of the drainage area.

1.3 Design by Urban Hydrological Modelling

The simulation of urban runoff is characterized by an attempt to quantify all pertinent physical phenomena from the input (rainfall) to the output runoff. The usually consist of the following steps :

i) determine a design storm
ii) estimation of excess rainfall rate
iii) flow to the gutter by overland flow equations
iv) route gutter flow
v) route the pipe flow, and
vi) determine the outflow hydrograph.

The most widely known of the computer based urban rainfall runoff models is the Storm Water Management Model. The application of SWMM involves the division of the drainage area into a network of idealised elements, each of which consists of a rectangular plans with uniform land use, slope and surface characteristics.

Most urban runoff models deals with individual storm event. With the advent of modern computer, the trend has been more toward the continuous time simulation of many storm and dry periods using the hydrologic process. In this report, the illinois urban drainage models, TVA continuous daily stream flow model, SCS model, USGS model, the wallingford model, the Road Research Lab model and SWMM model have been discussed in detail and comparison regarding the suitabidity of the model has been outlined.

2.0 URBAN DRAINAGE MODEL

Although the general principles underlying the rainfall-runoff process are the same for rural or non-urban watersheds and urban watersheds, urban watersheds usually have different characteristics in comparison with rural watersheds. The urban watershed areas are usually smaller, and also, the stream channels in urban watersheds are more uniform. Furthermore, the storm sewers induce swift conveyance in urban watersheds. Consequently the urban watershed response will usually be much faster in comparison with the rural watershed response. In view of these and other differences urban hydrologic analysis is usually somewhat different from the hydrologic analysis of nonurbanized watersheds. The literature of urban hydrologic analyses pertaining to urban storm modelling and their inter comparisons are discussed briefly in this report.

2.1 Rational Formula

In the hydrologic design of drainage works in urban areas, the most popular empirical formula which is used to compute the peak discharge due to a storm is the Rational Formula, which is given by

$$Q_p = C\ I\ A$$

where Q_p is peak discharge in cfs,

C is a runoff coefficient which depends upon the characteristics of the drainage basin,

I is the intensity of uniform rainfall,

and A is the area of the drainage basin in acres.

There have been several attempts to improve the Rational Formula ever since its introduction in 1887. Metcalf and Eddy used a method called the "Zone Principle" in which the drainage area is divided into zones by isochrones or contours of equal travel time. Each zone is assigned an "appropriate" value of runoff coefficient, the magnitude of which depends upon the imperviousness of the zone, and the distance of the zone from the outlet. An average runoff coefficient applicable for the entire watershed is then estimated and used in the Rational Formula.

Mehn developed a method to evaluate the "composite runoff coefficient" which is similar to the runoff coefficient 'C' of the Rational Formula with the exception that the composite runoff coefficient is applicable only to urban watersheds. In order to compute the composite runoff coefficient, the watershed was divided into subareas and the individual subareas were assigned different values of runoff coefficients. The magnitudes of runoff coefficients depend upon the physiographic characteristics of the subareas and were obtained from the ASCE manual of Engineering Practice. These runoff coefficients were weighted and the weighted average value was adopted as the composite runoff coefficient instead of the coefficient C in the Rational Formula to compute the peak discharge from the entire area.

The frequency of peak runoff obtained by using the Rational Formula is assumed to be the same as the frequency of the rainfall intensity which is selected to compute the peak runoff. An investigation to check this assumption was undertaken by Schaake, et.al. From the analysis of data obtained from six small urban watersheds (each of area less than 150 acres) located in Baltimore, Maryland area, frequencies of both the rainfall intensity and the peak runoff were found to be log-normally distributed. Empirical equations for computing the values of C and the "rainfall intensity averaging time", were also derived in terms of the physiographic characteristics of the watershed.

The accuracy of prediction of peak discharges by using the Rational Formula or its variations depends on the appropriate estimation of the values of the coefficient C, which in turn depends on the judgment of the designer. Thus the results obtained from the Rational Formula have considerable variation. However, the Rational Formula still remains popular in the hydrologic design of urban drainage facilities.

2.2 Hydrograph Synthesis by Routing

Empirical formulas such as the Rational Formula yield only peak discharge estimates which are not too reliable. This drawback, and also the necessity of knowing the time distribution of runoff gave rise to methods of hydrograph analysis in urban watersheds. Horner and Flynt were among the first to use hydrograph methods in the design of storm sewers. They measured the temporal variations in rainfall and runoff on three small (less than 5 acres), heavily urbanized areas in St.Louis, Missouri. Assuming that the abstractions from the rainfall are zero, the "100 Percent Runoff" hydrograph was computed for each storm on a drainage basin by using the unit hydrograph method.

Horner and Jens attempted to synthesize the hydrograph by first computing the excess rainfall distribution for each subarea of a watershed. The infiltration rates were estimated by using Horton's equation,

$$f_i(t) = f_c + (f_o - f_c)e^{-b_3 t}$$

where

$f_i(t)$ is the rate of infiltration at time t,

f_c is the final constant rate of infiltration,

b_3 is a constant dependent on the soil type and vegetation,

and f_o is initial rate of infiltration.

The direct runoff hydrograph for each subarea was then computed by using Horton's equation of overland flow,

$$q = I \tanh^2 [0.922t(I/n_r L_o)^{0.5} S_o^{0.25}]$$

where q is the overland flow at any time t in inches per hour,

n_r is the retardation coefficient representing the surface roughness,

S_o is the average overland flow slope expressed in percentage,

and L_o is the effective length of overland flow in feet.

These direct runoff hydrographs resulting from various subareas were suitably lagged and superposed to obtain the hydrographs of direct runoff at the outlet of the watershed.

Hicks suggested a graphical method called as the "Peak Rate Method" of synthesizing direct runoff hydrographs. By analyzing the data of effective rainfall of 10 year frequency and different intensities and times of concentrations, charts were developed to compute direct runoff hydrographs. These direct runoff hydrographs were supposed to yield the runoff from a completely impervious area and were called "Basic runoff hydrographs". Then, by using a trial and error procedure in which the conduit storage was accounted for, the peak discharges of basic runoff hydrographs were computed. A table of peak discharges of basic runoff hydrographs for different times of concentration was prepared along with charts for "runoff factors". Runoff factors, defined as the ratio of volume of runoff to volume of rainfall, were computed by analyzing data from experimental watersheds which had different land-use classifications and soil types. The peak runoff rate from a given effective rainfall for any drainage area is computed by multiplying the basic peak rate with the appropriate runoff factor taken from the charts. Although the runoff hydrographs can also be computed by this method, the main emphasis is on computation of peak discharges.

For larger times of concentration, a method of "Summing Hydrographs" which is an extension of the "peak rate method" was suggested by Hicks. In the "Summing Hydrographs" method, the watershed was divided into subareas for each of which the direct runoff hydrographs were first obtained by the peak rate method. Then, the hydrographs from all the subareas which drain to a junction point were combined. The resulting combined hydrograph was routed to the next downstream junction point in the basin. The other combined hydrographs from other subareas in the basin which drain to the same downstream junction point were similarly routed. The routing process was continued to obtain the direct runoff hydrograph at the outlet of the drainage basin. The "Summing Hydrographs" method was found to be more useful for large drainage areas with extensive sewer development.

The Storm Drainage Research project was initiated at the Johns Hopkins University in 1949. The objectives of the project were to develop methods of accurate measurement of rainfall and runoff especially in small urban watersheds and to develop methods of predicting runoff hydrographs from urban watersheds by using the given rainfall information and the data of physiographic characteristics of the urban watersheds. Initially, four completely paved watersheds, all of area less than an acre, and which had longitudinal slopes of one to three percent, were instrumented. This program of data collection from urban watersheds was later extended so that currently data are being collected from 29 urban watersheds of areas ranging from 0.1 acre to 153 acres. The percentage of built-up or impervious area in these watersheds varies from 9 to 100 percent, and these watersheds are all located in Baltimore, Maryland.

2.3 Unit Hydrograph Methods

Eagleson applied the unit hydrograph methods to study rainfall-runoff relationships in urban watersheds. The "Volumetric runoff coefficient" which was defined as the ratio of total volume of runoff to the total volume of rainfall", was found to be a constant for the data used in the analysis. By using the volumetric runoff coefficient, Eagleson computed the rainfall excess and thereby derived the 10 minute unit hydrographs. The unit hydrograph characteristics were then related to the physiographic characteristics of the watershed. The relationship between the unit hydrograph peak discharge per square mile of the watershed, q_{um}, and the mean basin slope S, was found to be

$$q_{um} = (2.13 \times 10^{5})S$$

Eagleson observed that for watersheds with appreciable channel storage, q_{um} was a decreasing function of excess rainfall. The unit hydrograph base width, the widths at 50% and 75% of q_{um} were plotted against the maximum unit hydrograph discharge, q_{um}.

Viessman also used the unit hydrograph method to analyze rainfall-runoff process in urban watersheds. The excess rainfall was obtained by using a combination of an initial abstraction deduction and the -index method. For all the storms on a watershed, one minute unit hydrographs were derived, so that the outflow Q at any time t was given by

$$Q = 1(1 - e^{-1/k})\, e^{\frac{(1-t)}{k}}$$

The optimum value of the storage constant k was computed for each storm by minimizing the sum of the squares of the difference between the observed and computed discharges, and also by equating the times to peak of the observed and computed peak discharges.

Viessman concluded that the optimum values of the storage constant varied considerably. However, the hydrographs regenerated by using the average value of the optimum storage coefficients, K, agreed very well with the observed direct runoff hydrographs. Also, the optimum storage coefficients were not found to be significantly correlated with the rainfall characteristics.

2.4 Instantaneous Unit Hydrograph Methods

The possibility of modelling rainfall-runoff process on very small impervious areas (less than 1 acre) by means of conceptual models was investigated by Eagleson and March. For purposes of comparison the instantaneous unit hydrographs were derived by the "Direct Method" and also by using conceptual models proposed by Zoch, Nash and Singh. It was observed that actual direct runoff hydrographs were satisfactorily reproduced by using the Instantaneous Unit Hydrographs derived by the direct method, although there was considerable variation in the shape of the

IUH. Hence it was concluded that no single IUH can be used to obtain the runoff from a watershed for all storms. Another conclusion of this study was that among the three conceptual linear models considered, the Zoch model provided better regeneration of runoff than either the Nash model or the Singh model.

Delleur and Vician have used two conceptual models in their analysis of data from urban watersheds in West Lafayette, Indiana. The storage coefficient K, of the single linear reservoir model, which was the first conceptual model used in the analysis, was determined by a trial procedure. From the data analyzed, it was reported that a value of K which is equal to 0.8 times the observed time lag, gave better regeneration of the runoff hydrograph than the cases in which K was assumed to be equal to the observed time lag or its average value. The second conceptual model which was a series combination of a linear channel and a linear reservoir, was used in an attempt to represent both lag and storage effects in the watershed. The travel time required for obtaining the time-area-concentration curve was estimated by calculating the actual velocities of flow in the storm sewers. The linear-reservoir-channel model consistently predicted lesser peak discharges than the single linear reservoir model for the data analyzed.

2.5 Illinois Urban Drainage Area Simulator Model (ILLUDAS)

This model is used for the hydrologic design of storm drainage system in urban area and is based on a digital model to be known as the Illinois Urban Drainage Area Simulator. This model uses an observed or specific temporal rainfall pattern uniformly distributed over the basin as the primary input. The basin is divided into subbasins. Paved area and grassed area hydrographs are produced from each sub basin by applying the rainfall pattern to the appropriate contributing areas. These hydrographs are combined and routed downstream from one design point to the next until the outlet is reached.

The principal element in the computation of runoff from directly connected paved area are as follows. Equal time intervals of rainfall are applied to the directly connected paved area in small sub basin of the total urban basin. A computation is made of the travel time required for each increament of runoff to reach the inlets at the downstream end of the subbasin and surface hydrograph is provided for each sub basin. ILLUDAS is applied by first dividing the basin into sub basins. A subbasin is normally a homogeneous portion of the basin tributary to a single inlet or set of inlets that constitute a design point in the drainage network. Two physical factors must be evaluated for each subbasin. First the paved area directly connected to the storm drainage system must be determined and secondly the travel time from the farthest point on the paved area to the design point must be calculated. After the directly connected paved area has been determined, the time of travel for the runoff from various parts of the paved area to the inlets at the downstream end of the subbasin are estimated. In this model, travel time on the paved area are computed in two steps. In first step, flow of 0.5 to 1.0 cfs per acre of contributing paved area is assumed. In second step manning equation has been used to compute the velocity of flow. With these velocities, travel times are computed at various points on the paved area in each sub basin.

These travel times are plotted on the paved area and by connecting points of equal travel time a series of isochrones are drawn. The time area curve shows the amount of paved area within the subbasin that is contributing water within the subbasin that is contributing water at the storm drain inlet at any time after the beginning of runoff. The losses considered are initial wetting and depression storage. These losses are computed and treated as an initial loss to be subtracted from the beginning of the rainfall pattern. After subtracting these losses, the remainder of the rainfall will appear as runoff from the paved area.

Computation of grassed area hydrograph for each subbasin closely parallels that of paved area hydrograph. Travel times on the grass strip are equivalent to the time of equilibrium in the equation proposed by Izzard.

$$q_e = .0000231 \; I.L.$$

where

q_e = discharge of overland flow
I = supply rate at inches/hour assumed to be 1
L = length of overland flow in feet

and time of equilibrium is

$$t_c = .003 \; KL \; q_e^{-0.67}$$

where

t_e = time of equilibrium in minutes
K = $(.0007 \; I + C) \; S^{-.33}$
S = surface slope
c = coefficient having a value of 0.046 for bluegrass

After the travel times at various points on the contributing grassed area have been computed, the one minute isochrones are drawn. This time area curve shows the amount of grassed area within the subbasin that is contributing water at the storm drain inlet at any time after the beginning of the runoff. Rainfalling on the supplemental paved area is assumed to runoff onto the surrounding grassed area. The model assumes that this occurs instantly and that the volume of runoff is uniformly distributed over the contributing grassed area.

Infiltration

In an urban basin, the area that is not paved is most often covered with bluegrass turf. When rain falls on this turf, there are two principal losses, the first being depression storage and the second being infiltration into the soil. In ILLUDAS provision is made for depression storage to be filled and satisfied any infiltration takes place. Depression storage is normally taken to be 0.20 inches, but provision is made in ILLUDAS for this to be varied.

Computed Infiltration

The Hortan equation has been used for computing infiltration rate at any given time (t).

$$f = a(S - F)^n + f_c$$

where

- f = infiltration rate at time t, in inches per hour
- a = a vegetative basal factor reflecting the efficiency a crop root system makes of soil porosity for storing water; $a = 1.0$ for bluegrass turf
- n = a constant = 1.4
- S = storage available in the soil mantle in inches (storage at the total soil porosity minus storage at the wilting point)
- F = water already stored in the soil at time t, in excess of the wilting point, in inches (amount accumulated from infiltration prior to time t)
- $(S-F)$ = storage space remaining in the soil mantle at the time t, in inches
- f_c = final constant infiltration rate, in inches per hour (generally equivalent to the saturated conductivity, in inches per hour, of the tightest horizon present in the soil profile)

With the help of above equation it is possible to compute an infiltration curve based on the physical properties of the soil.

The U.S. Soil Conservation Service describes the four hydrologic soil groups as follows:

A - Low runoff potential, high infiltration rates (consist of sand and gravel)

B - Moderate infiltration rates and moderately well drained

C - Slow infiltration rates (may have a layer that impedes downward movement of water)

D - High runoff potential, very slow infiltration rates, (consist of clays with a permanent high water table and a high swelling potential)

Standard infiltration curves have been devised for use in ILLUDAS for soils of hydrologic groups A, B, C and D. These curves were calculated from the Horton equation as given by Chow (1964) as

$$f = f_e + (f_O - f_e)e^{-kt}$$

where

f_O = initial infiltration rate, inches per hour
e = base of natural logs
k = a shape factor selected as $k = 2$
t = time from start of rainfall

This equation is solved in ILLUDAS by the Newton Raphson technique.

Routing Procedure:

ILLUDAS assists the user in the design of detention basins in several ways. First, if an existing system is being analyzed, ILLUDAS accumulates flows greater than the capacity of the existing pipe for each reach in the basin. The maximum volume of flow thus accumulated is equivalent to the detention storage required to keep the system operating at capacity during passage of the design storm. These accumulated flows are reported on the output and serve to pinpoint the location and severity of flooding in the basin.

If a new drainage system is being designed, the user may specify the volume of detention storage allowable at any point in the basin. ILLUDAS will then incorporate that volume of storage into the design by allowing incoming flows to fill the allowable storage. The outlet capacity needed to make effective use of this storage will also be provided by ILLUDAS.

As an additional option the user may limit flow through a given reach by specifying a small outlet pipe size or a maximum discharge through the reach, and ILLUDAS will report the volume of detention storage accumulated during passage of the design storm.

The advantage of this model is that both the paved and unpaved areas are considered, data input is simple and storage effects are simulated.

2.6 TVA continuous daily stream flow model:

TVA daily streamflow model is basically a simple water budget model for estimation of storm water runoff. Daily runoff is budgeted among a series of conventional cascading compartments or reservoirs. The time unit of a day was selected for this model because of the ready availability of daily rainfall and stream flow data. 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 1. A schematic diagram for continuous daily stream flow model is shown in Figure 1.

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 (Impervious Area):

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

$$PSRO = RFr \times 1.165 \times PIMP$$
$$PIMP = (IMP-0.17);\ PIMP>0$$

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 residual precipitation becomes potential storm runoff from pervious areas. One significant departure of the TVA model from other continuous flow models such as Stamford model and the USDA model is that the process of infiltration is not included in this model.

TABLE 1 : 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 groudnwater 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 eyar beginning of respective season (beginning October 1)

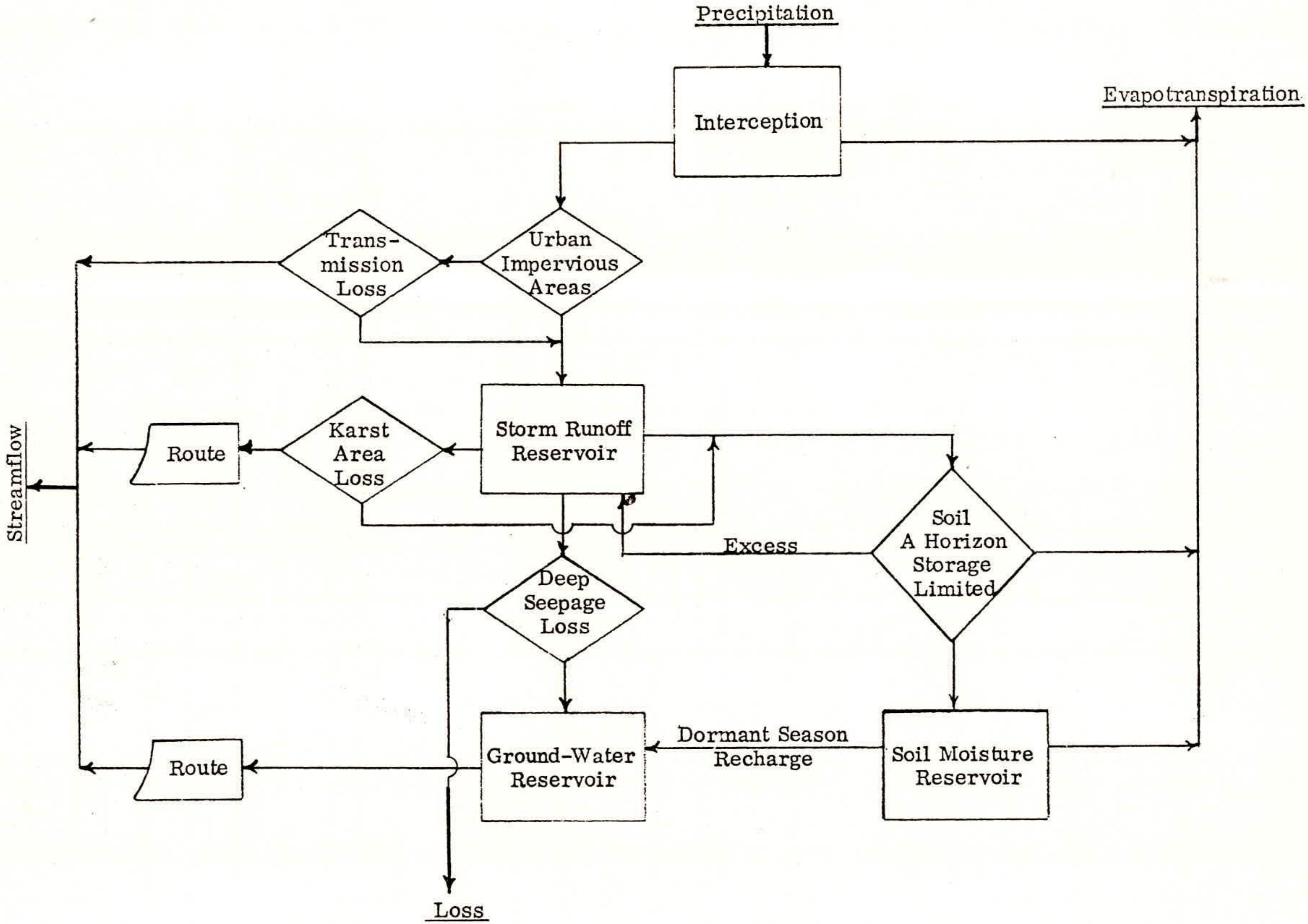

Figure 1 - Continuous Daily Streamflow Model Schematic

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 Betson et al (1969):

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

$$SURVOL = (RF_r^2 + RI^2)^{0.5} - RI$$

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 ground water runoff. This portion is assumed to be proportional to the yield of storm runoff:

$$GWV = (SURVOL * GWR/RF)*RF^{r} \quad \text{and } GWR < RF^{r}$$

where

GWV	=	a volume to be added to the groundwater reservoir cm
GWR	=	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 dormant. During this 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 stream guage and thus are lost from the system.

$$GWL = GWV \times DLF$$

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:

$$PSRO = (PSRO/TLP)*PSRO = PSRO^2/TLP$$

$$PSRO/TLP<1.0$$

where

TLP = a transmission loss parameter

vii) Evapotranspiration

Monthly evapotranspiration values are used as input to the model.

$$RF - RO = ET = K (EP_C * GL_1)$$

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_c	=	average monthly pan evaporation
GL_i	=	growth index of crop

viii) 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:

$$SRO = TDSRO * SURVOL_i + SURES_i * (1-SRKOK)$$

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

$$GRO_i = GWR_i * (1-GROK)$$

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.

x) Regionalization of Model Parameters

One of the end goals in the development of a hydrologic model is to use the model to simulate data at locations where observations do not exist. Conceptually, if a model is complete and correctly formulated, it should be possible to measure all the necessary site characteristics to define, model boundary values and coefficients with the simulation. However the heterogeneities that exist in nature along with the complexities and interactions involved necessitate some idealization of complex natural systems to keep the model tractable and the amount of data required manageable result in losses in generality and in the direct relationshhip between model coefficients and site characteristics. As a consequence, it becomes necessary to correlate the model parameters with site characteristics. This process is termed as regionalization. Using the model parameters obtained from calibration, optimization runs of the model, graphical relationships were developed between, the various model parameters and important climatic/site variables. Equations were derived based on best fit lines. Statistical derivations of these lines could not be justified because some parameters for certain watersheds were known to be off some what.

2.7 Soil Conservation Services (TR-55) Procedure

The soil Conservation Service (SCS) procedure which came into common use in the year 1954 is the product of more than 20 years of studies of rainfall-runoff relationship for small rural watershed areas. The procedure which is basically empirical was developed to provide a rational basis for estimating the effects of land treatment and land use changes upon runoff resulting from storm rainfall.

The SCS has given the following relation between the accumulated volumes of storm rainfall runoff and catchment retention

$$Q = (P - 0.2S)^2 / (P + 0.8S)$$

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:-

$$CN = 1000 / (S + 10)$$

or $$S = 1000/CN - 10$$

SCS developed a soil classification system that consits 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, aggregated silts
Group B : shallow loess, sandy loam
Group C : clay loam, shallow sandy loam, soils low in organic content and soils usually 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 2 : 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 for AMC II. This curve number is modified for other antecedent moisture conditions as per the Table 3.

For more complex areas a composite value of CN can be

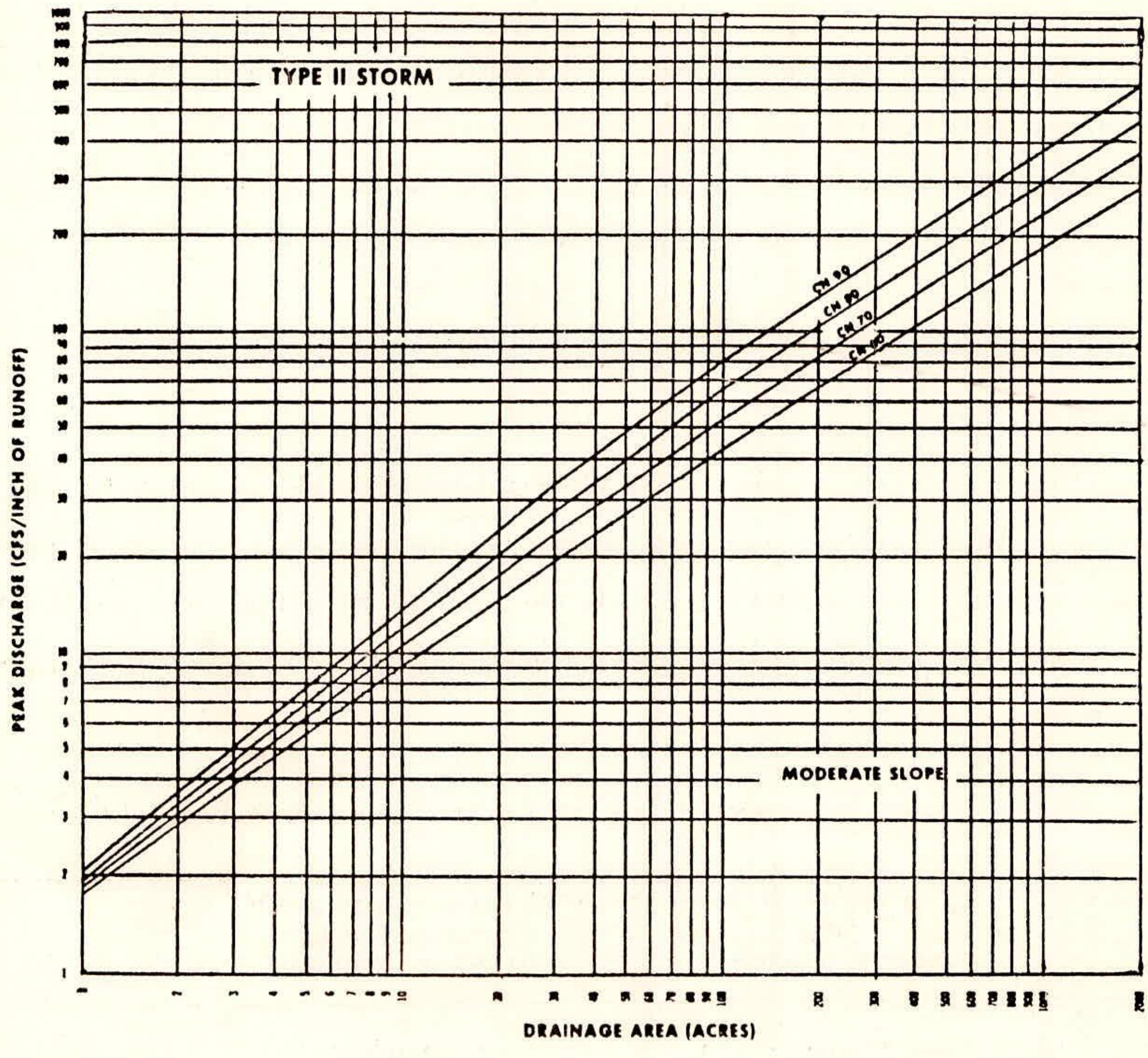

Figure 2: Peak rates of discharge for small watersheds (24 hour, type-II storm distribution).

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.

Different charts are given for flat, moderate and steep catchment slope. Fig. shows such a chart to estimate peak discharge for a small watershed (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 watershed shape factor (l/w) different adjustment factors to peak discharge are determined and applied.

Table 3 : Modified Curve Numbers for AMC 1 & 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:-

$$Q_{MOD} = Q \, [\, Factor_{IMP} \,] \, [\, Factor_{HLM} \,]$$

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 3 and 4 .

The SCS TR-55 procedure is very much simplified as it involves reading various values from charts and tables and simple calculation, 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.

2.8 USGS Model

USGS is a special purpose model used to predict peak flows. The various processes represented in the USGS model are shown in Figure 5 . The USGS model determines rainfall excess over short time interval and routes the rainfall exess to the basin outlet. Rainfall excess is determined by substracting infiltration losses from rainfall occuring during the short unit time intervals. The rate of infiltration is highly dependent upon the soil moisture

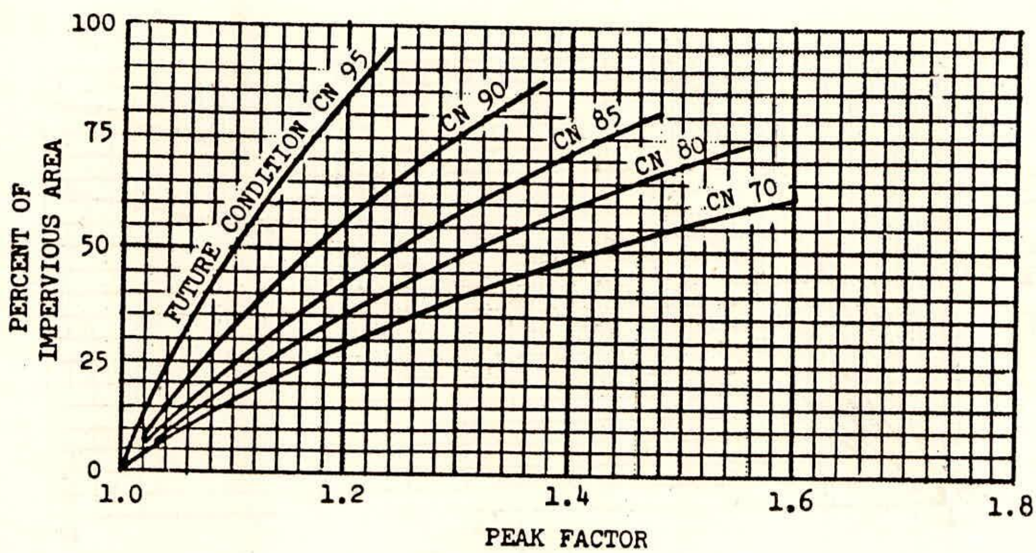

Figure 3: Factors for adjusting peak discharges for a given future-condition runoff curve number based on the percentage of impervious area in the watershed.

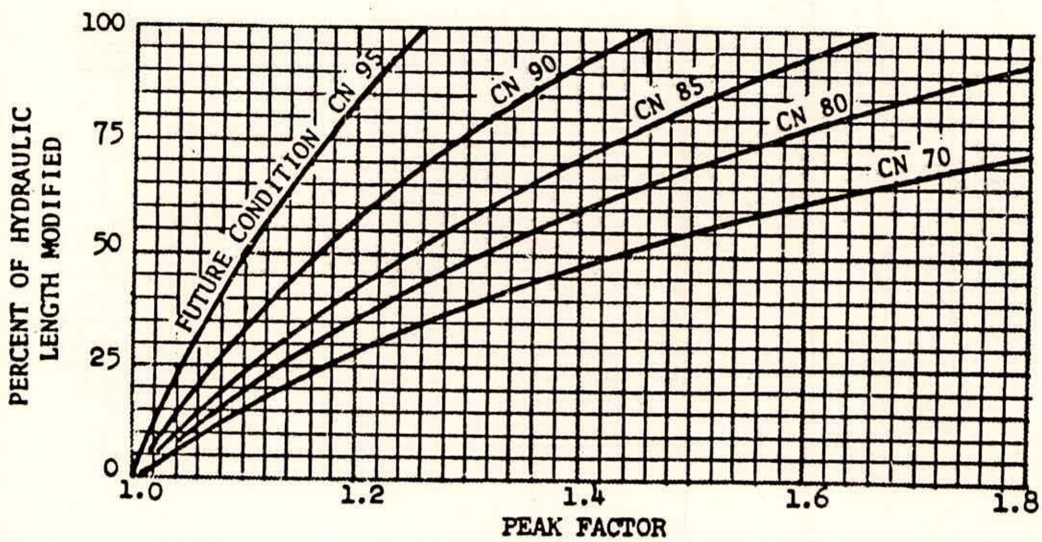

Figure 4: Factors for adjusting peak discharges for a given future-condition runoff curve number based on the percentage of hydraulic length modified.

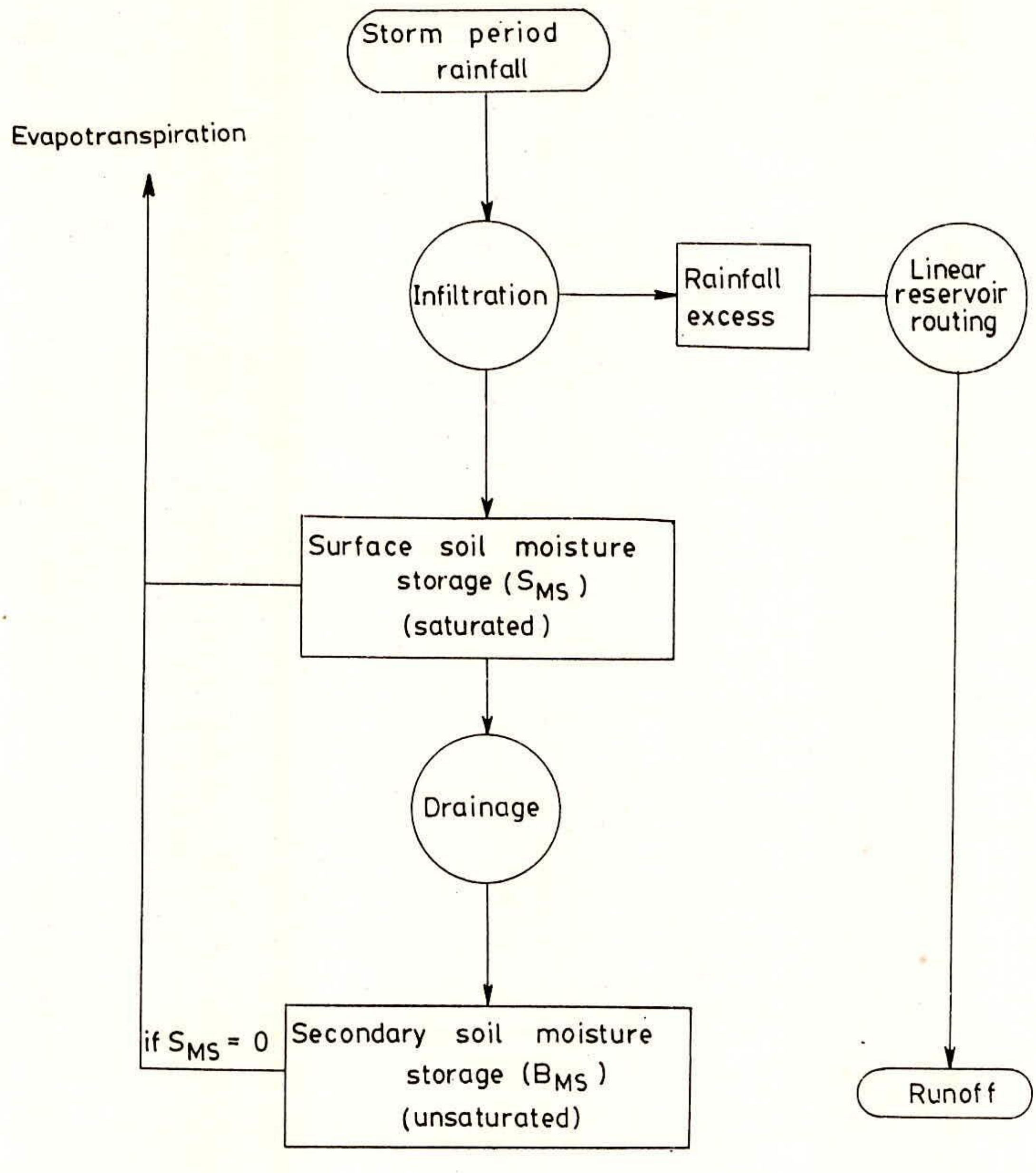

Figure 5: Flow Chart for USGS Model.

condition particularly at the beginning of the storm. Therefore, continuous estimation of soil moisture is very important. Soil moisture accounting is done using the concept of a two layer model. The top layer is called a saturated zone which directly receives all the infiltrated water and its thickness changes depending on rate of infiltration, and vertical drainage which takes place at a constant rate to the bottom zone called unsaturated zone. The unsaturated zone can hold a maximum soil moisture equal to the field capacity of the soil, BMSM, which is a model parameter and anything in excess goes down as gravity drainage which is called percolation. As the processes are simulated in different ways during flood-days and non-flood days, they are briefly described below:

Non-flood days

Processes considered are only infiltration and evapotranspiration and simulation is done with a time increment of one day. Daily infiltration is computed as a constant proportion of the daily rainfall. This constant is designated as R R, which is a model parameter. The daily evapotranspiration is a constant proportion of the daily pan evaporation. This constant, EVC, is also a model parameter. The infiltrated water is added to the saturated zone. The evapotranspiration is met from the saturated zone and if, moisture is not available in the saturated zone, it is met from the unsaturated zone.

Flood days

When there is no rainfall, simulation is done with hourly increments of time. The hourly evapotranspiration is equal to 1/24 of the daily evapotranspiration which is computed as EVC times pan evaporation. The rate of vertical drainage per hour is also 1/24 of DRN which is the daily rate of vertical drainage. All the processes are simulated in the same way as during non-flood days but with hourly increment of time. During the hour when there is rainfall, the simulation is done with 5 minute increment of time. The evapotranspiration during this time interval is ignored.

Computation of rainfall excess

Infiltration loss component is a critical part of USGS model and these losses are computed using a modified form of Philip's equation.

$$\frac{di}{dt} = k \left\{ 1 + \frac{P(\bar{m} - m_o)}{i} \right\}$$

where,

i = accumulated infiltration volume in wetted soil column since the start of infiltration,

k = capillary conductivity of soil,

P = capillary pressure at wetting front in soil column,

m_o = initial moisture content of soil column when infiltration started, and

m = moisture content uniformly distributed through wetted column at the time at which infiltration is computed.

The term $P(m - m_o)$ is assumed to linearly decrease from a maximum, $r.P_s$ at the wilting point of the soil ($M_o = 0$) to a minimum P_s, at the field capacity of the soil ($m_o = m_c$). Thus,

$$P(\bar{m} - m_o) = r.P_s - P_s(r - 1)\frac{m_o}{m_c}$$

Assuming,

$$P(\bar{m} - m_o) = P_m$$

then,

$$\frac{di}{dt} = k \left\{1 + \frac{P}{i^{m}}\right\}$$

The infiltration model has four parameters namely, k, r, P_s and m_c. The above equation describes the infiltration at a point. In order to account for the basin wide variability of soil characteristics and moisture conditions, infiltration capacity is assumed to vary linearly over the basin area from zero to di/dt.

The infiltration rate, FR, is computed at the beginning of every five minutes interval. By knowing the amount of rainfall in five minutes, SR, and using the assumed linear variability of infiltration capacity, the rainfall excess can be computed as,

$$R_e \quad \frac{R^2}{2FR}, \; SR<FR$$

$$R_e = SR - FR/2, \; SR>FR$$

During a period of uninterrupted rainfall, the antecedent moisture content m_o at the start of rainfall is assumed to remain constant as the wetting front advances. During periods of no rainfall, the accumulated infiltration i will diminish due to evapotranspiration and vertical drainage.

Runoff routing

The translation hydrograph has been used in flow routing in USGS model. This model assumes a triangular translation hydrograph of unit area for simplification and generation of the procedure. The triangular translation hydrograph of unit area is defined by the following two model parameters:

i) The first parameter is the time of concentration which is equal to the base of the triangular translation hydrograph. It is designated by X(9).

ii) The second parameter is T_p, the ratio of time to peak of the translation hydrograph to the time of concentration. It is designated by X(10). The translated hydrograph I(t) is obtained by the convolution of the translation hydrograph and the rainfall excess, viz.

$$I(t) = \sum_{j=1}^{n} T_j \, R_e(t-j+1)$$

where,

T_j = Magnitude of the j^{th} ordinate in the translation hydrograph, and

$R_e(t-j+1)$ = Rainfall excess during time t-j+1.

This when converted into discharge units becomes the input hydrograph to a conceptual linear reservoir. The storage coefficient K_s of the linear reservoir is a model parameter and is designated by X (8).

The input for every hour of flood days is computed and routed through the linear reservoir.

The outflow Q(t) from a linear reservoir is a linear function of storage only and is given by

$$S = K_s Q$$

Differentiating above equation, and substituting in the continuity equation

$$\frac{dS}{dt} = I - Q$$

The solution for outflow, for a constant inflow I occuring during an interval t, is found to be

$$Q(t + t) = I - (\{I - Q(t)\} e^{-t/K_s}$$

in which,

$Q(t)$ = outflow at time t = 0

At the cessation of inflow, I = 0 the outflow from the reservoir is

$$Q(t + t) = Q(t)\, e^{-t/K_s}$$

Thus the outflow hydrograph from linear reservoir is the simulated flood hydrograph. The routing procedure is based essentially on unit hydrograph theory. Hourly input of rainfall excess are considered as inputs. The programme stores all the hourly ordinates of the simulated hydrograph. It also finds the maximum value of these simulated hydrograph ordinates which is the simulated peak for the flood event.

Input data

The input data required for peak flow simulation are daily rainfall, daily evaporation, daily runoff and hourly rainfall during flood days. The average value of soil parameters like field capacity of the soil, saturated hydraulic conductivity, infiltration rate and percentage of pervious and impervious area must be known for estimation of peak floods.

Calibration

There are ten model parameters as shown in Table . The programme uses the following three different objective functions for finding the model parameters:

1) $$U_1 = \sum_{i=1}^{n} (\log_e V_{oi} - \log_e V_{si})^2$$

2) $$U_2 = \sum_{i=1}^{n} (\log_e P_{oi} - \log_e P_{si})^2$$

3) $U_3 = \frac{1}{2} U_1 + U_2$, and

4) $U_4 = \sum^{n} \{\log_e (P_{si} \frac{V_{oi}}{V_{si}} - \log_e P_{oi}\}^2$

where,

n = no. of peaks selected for use in calibration,

V_{oi} = observed surface runoff volume during event i,

V_{si} = simulated surface runoff volume for event i,

P_{oi} = observed flood peak i, and

P_{si} = simulated flood peak i.

Initial parameter values are to be given based on average soil characteristics, an estimate of ratio of potential evapotranspiration to pan evaporation and the recession and timing characteristics of observed flood hydrographs. The optimization is done in rounds. In the first round, volume objective function is minimized by adjusting the first seven parameters. In the second round, routing objective function 'U_4' is minimized by adjusting the last three parameters. In the last round, all the ten parameters are adjusted to minimise peak objective function 'U_2' or peak and volume objective function 'U_3'.

The model uses Rosenbrock's rotating coordinate method for parameter identification. Since the problem is highly nonlinear one, it is necessary to specify the range of parameters in order that reasonable solutions are obtained. The initial values and range of all the ten parameters are presented in Table 4.

Table 4 - Details of Parameters in USGS Model

Description	Mnemonic identifier	X-array identifier	Initial value	Lower limit	Upper limit	Unit
Minimum effective magnitude of PS	PSP	X(1)	5.00	1.00	15.00	inches
Hyd. cond. of saturated soil	KSAT	X(2)	0.10	0.01	1.00	inches/hour
Volume of water drained from saturated zone	DRN	X(3)	0.50	0.10	1.00	
Ratio of max. PS to min. PS	RGF	X(4)	10.00	5.00	20.00	
Field capacity of soil	BMSM	X(5)	3.00	1.00	10.00	inches
Parameter to adjust daily pan evaporation to potential ET	EVC	X(6)	0.70	0.50	1.00	
Ratio of daily infiltration to daily rainfall	RR	X(7)	0.85	0.65	1.00	
Time constant for linear reservoir routing	KSW	X(8)	67.00	10.00	150.00	hour
Time of concentration	TC	X(9)	4020.00	600.00	9000.00	minutes
Ratio of time of peak to time base of translation hydrograph	TP/TC	X(10)	50.00	0.10	1.00	

2.9 The Wallingford Model

The Wallingford Procedure for the design and analysis of urban storm drainage networks was based upon the results of a collaborative research programme carried out in the United Kingdom between 1974 and 1981 by the Hydraulics Research Station, the Institute of Hydrology and the Meteorological Office, and coordinated by the National Water Council/Department of the

Environment Working Party on the Hydraulic Design of Storm Sewers. The Procedure consists of four methods:

(i) The Wallingford Rational Method: a modified version of the Rational Method intended for use on outline designs or on homogeneous areas of up to 150 ha.

ii) The Wallingford Hydrograph Method: a computer-based approach which models the above-ground and below-ground phases or runoff separately; this method may be employed for both design and simulation and allowances may also be made for the action of stormwater overflows, on-line and off-line detention tanks and pumping stations.

iii) The Wallingford Optimising Method: a computer-based method with which the performance of both an existing system and a proposed design may be examined under surcharged conditions stormwater overflows, on-line and off-line detention tanks and pumping stations may also be taken into account.

These methods may be applied to both separate and combined sewerage systems, although the calculation of foul sewage flows is not included. No allowances are made for the calculation of runoff from any rural areas that may contribute to an urban drainage network and no water quality modelling is attempted.

The selection of the method most appropriate for a particular design requirement is assited by following the flowchart presented in Fig.6. For the design of new systems, the Modified Rational, Hydrograph or Optimising Methods can be used. The flow calculations for the Optimising Method are carried

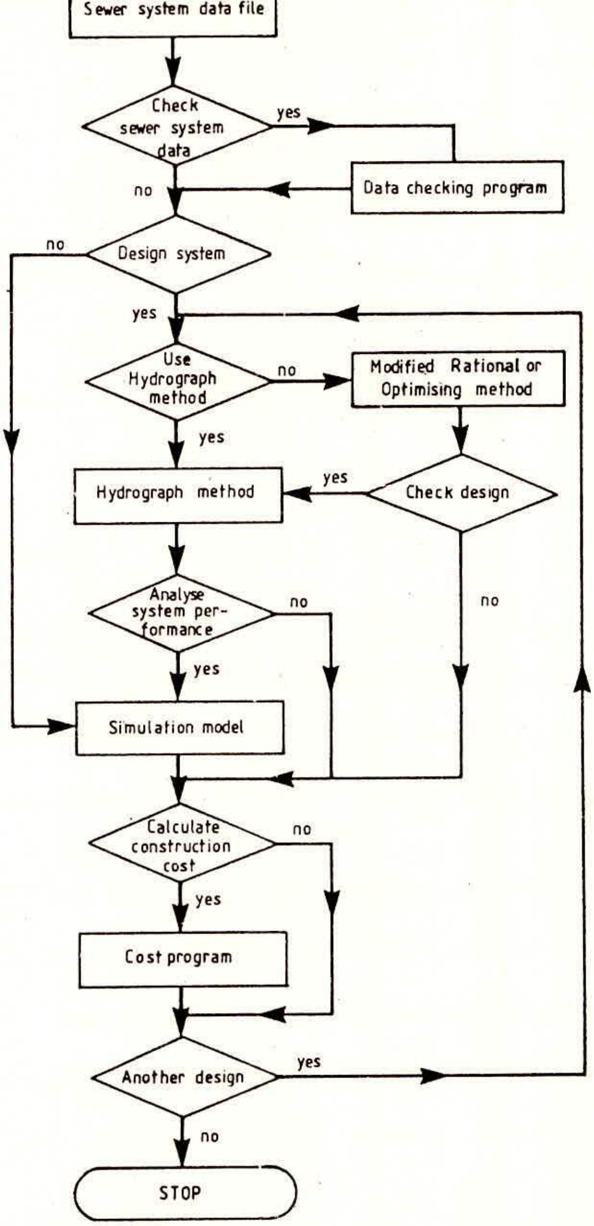

Figure 6: Flowchart for the Wallingford Procedure.

out using the Modified Rational Method, and so the discharge estimates obtained from these approaches are similar, unless the gradient optimisation substantially alters times of concentration. The Optimising Method may not be appropriate if the longitudinal profile of the sewer is constrained by the presence of other underground services.

If optimisation of pipe diameter, depth and gradient is not required, then the designer may use either the Modified Rational or the Hydrograph Methods. The former provides an estimate of peak discharge only, whereas the latter also produces the flood hydrograph. Both methods make allowances for the presence of certain types of ancillary structures.

For the analysis of an existing system, the Modified Rational or the Hydrograph Methods or the Simulation Program may be used. The Modified Rational Method is limited to the estimation of peak flow rates. The Simulation Program incorporates the same algorithm for simulating the above-ground phase of runoff as until surcharging begins, and so both of these methods should yield similar results in non-surcharged pipe systems.

For both the design of new systems and the simulation of existing sewer networks, different methods may be more appropriate at different stages of an investigation. The Modified Rational Method may be applied for both design and analysis in order to provide an initial appreciation of catchment response. For a new sewarage system, the Optimisation Method might then be employed to determine pipe sizes, depths and gradients, which subsequently can be checked using the Hydrograph Method. The latter approach can also be applied to check an existing system for surcharging. Finally, the Simulation program both allows the performance of a proposed sewer network to be evaluated when subjected to rarer events than the selected design storm, and permits a more detailed examination of zones of surcharging in an existing pipe system.

Further details of both the Modified Rational and the Hydrograph Methods are summarised in the following sections.

2.9.1 The Wallingford Rational Method

Modified version of Rational Method is given by

$$Q_p = 2.78\ C_v C_R\ iA$$

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 rainfall intensity within the time of concentration, and Q_p, i and A are peak flow, intensity of rainfall and Area respectively. If the total catchment area is being considered, the value of C_v is computed from

$$C_v = PR/100$$

where PR, the percentage runoff, is given by

$$PR = 0.829IMP + 25.0SOIL + 0.078UCWI-20.7$$

In above equation , IMP is the percentage impervious area of the catchment draining to the sewer, SOIL is a soil index and UCWI is an antecedent wetness index which, for design purposes, is obtained from a relationship with the average annual rainfall. If impervious area alone is being considered,

$$C_v = PR/IMP$$

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

The time of concentration is considered to consist of the sum of the time of flow (based upon full-bore pipe velocities) and a time of entry. The latter varies with both the design return period and the slope and size of the catchment area, ranging from 3-6 min for a 5-year event to 4-8 min for a one-year storm, the smaller values being applicable to the smaller, steeper catchments.

2.9.2 The Wallingford Hydrograph Method

This method was developed partly in response to criticism of the simplifications inherent in the TRRL Method with regard to:

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 above-ground storages; and

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

In the Wallingford Hydrograph Method the relationship between the return period of the peak discharge and 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 (1980) involving the comparison of observed and computed probability distributions of peak flow rates.

The ground model in the Hydrograph Method consists of several components as shown in Fig. 7. For design purposes, the

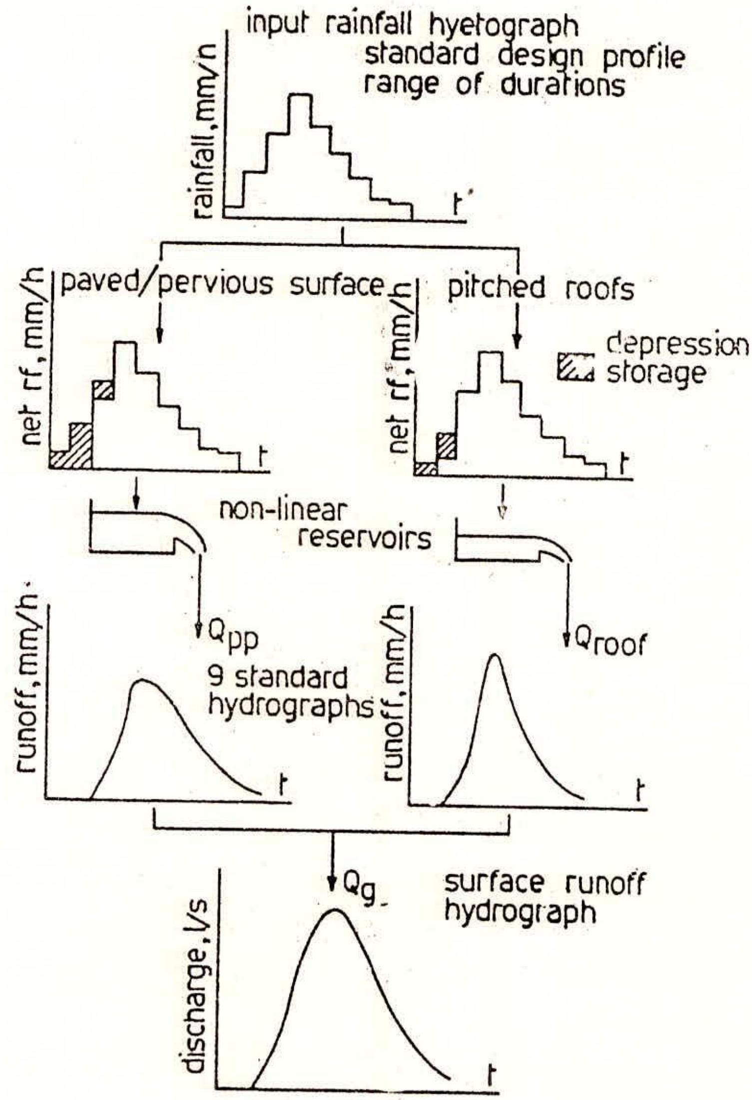

Figure 7: The Wallingford Hydrograph Method: the above-ground phase of runoff.

input to the model consists of a standard summer storm profile whose peakedness is exceeded by 50% of all such events. A duration of 15 min is assumed initially, and the computations are subsequently repeated for further values of 30, 60 and 120 min. In all cases, the total rainfall depth, P (mm), is that corresponding to each duration and the return period of the peak discharge which is to be estimated.

Estimation of the losses on the subcatchments draining to each pipe length begins with the prediction of the percentage runoff, PR, from the whole catchment, which is then distributed between the paved surfaces, pitched roofs and pervious areas within each subcatchment. By observation, the percentage runoff from the impervious surfaces of small drainage areas was found to average 70%. If therefore the value of PR predicted is less than 70% of the proportion of impervious surfaces within a subcatchment, the pervious areas are assumed not to contribute to storm runoff, so that

$$PR_{perv} = 0; \; PR_{pav} = PR_{roof} = 100PR/IMP$$

where the subscripts pav, roof and perv refer to the paved, roof and pervious areas respectively. However, if PR exceeds 70% of the proportion of impervious area, the excess is distributed equally to all surfaces, giving

$$PR_{perv} = PR-0.7IMP$$
$$PR_{pav} = PR_{roof} = 70+PR_{perv}$$

Once the appropriate percentage runoffs for each subcatchment, has been obtained the distribution of effective rainfall is obtained from the storm profile by allowing for both an initial loss to depression storage, DS (mm), and a continuing loss by infiltration. For the paved and pervious areas,

$$DS_{perv} = DS_{pav} = 0.71 SLOPE^{-0.48}$$

where SLOPE is the average overland slope (%) of the subcatchment. In practice, the necessity to take detailed measurements of slopes is avoided by allocating each subcatchment to one of three broad categories, as follows:

Description	Range in slope	Assumed value
Mild	<2%	1.25%
Medium	2-3.5%	2.75%
Steep	>3.5%	4.0%

For pitched roofs, a value of 0.4 mm is recommended for DS_{roof}. Once depression storage has been subtracted from the beginning of the storm, the remaining loss is distributed uniformly throughout the rest of the duration by means of a reduced contributing area. Denoting the actual paved area within a subcatchment as $AREA_{pav}$, the contributing area, AR_{pav}, is given by:

$$AR_{pav} = AREA_{pav} [(PR_{pav}/100)\{P/P-DS_{pav}\}]$$

Similar relationships are applicable for AR_{perv} and AR_{roof}.

The attenuation caused by surface storage is simulated by means of a non-linear reservoir, for which the storage volume, S, is related to the outflow discharge, Q, by the equation

$$S = KQ^{2/3}$$

where K is the storage constant. Using data from a selection of catchments having both paved and pervious surfaces, the following prediction equation was obtained for K:

$$K = 0.051 SLOPE^{-0.23} PAPG^{0.23}$$

where PAPG is the average paved area per gulley. If the number of gulleys in each subcatchment is specified, PAPG may be computed directly. Otherwise, a characteristics value may be obtained by allocating each subcatchment to one of three broad categories, as follows:

Description	Range in area	Assumed value
Small	$<200\ m^2$	$125\ m^2$
Medium	$200-400\ m^2$	$300\ m^2$
Large	$>400\ m^2$	$600\ m^2$

The value of K computed is applied to the effective contributing area, AR_{pav}, of the paved and pervious surfaces together. For pitched roofs, a value of K of 0.04 is recommended.

Even with such a simple model for attenuation, the amount of computation can become excessive with even a modest number of subcatchments. The calculations are therefore simplified by the use of nine 'standard' subcatchments, defined by three values each of SLOPE and PAPG as shown in the above tables. The runoff hydrographs (mm/h) from each of the nine standard subcatchments are computed initially, and every actual subcatchment is then represented by one of the nine. A roof hydrograph may also be synthesised, if required. The gulley hydrograph is obtained by adding the roof hydrograph with its ordinates multiplied by AR_{roof} to the appropriate standard hydrograph with its ordinates multiplied by $AR_{pav} + AR_{perv}$:

$$Q_g = [\, Q_{roof} AR_{roof} + Q_{pp} (AR_{pav} + AR_{perv}) \,]/3600$$

where Q_{roof} and Q_{pp} are the ordinates of the roof and standard hydrographs (mm/h) respectively, and Q_g are the ordinates of the gulley hydrograph (litres/s). The hydrographs obtained for each subcatchment are then routed through the sewer network, pipe by pipe, using the Muskingum-Cunge Method. The calculations are carried out for all four standard durations of storm profile, and the largest computed discharges are taken as the design flows for each pipe length.

Where insufficient data are available to permit the modelling of both the above-ground and the below-ground phases of runoff for every subcatchment and pipe length, or where the costs of data collection for a large drainage area would be prohibitive, a simplified sub-area model is available. In this model, the method of computing the gulley hydrographs is applied to sub-areas of up to 60 ha instead of each pipe length. As shown schematically in Fig.8, the computer sub-area hydrograph is then divided into N equal parts and distributed equally to the N segments of an 'equivalent pipe'. The latter consists of a tapered system of pipes in series, each of which has the same length and slope. The number of segments, N, depends upon the time of flow within the equivalent pipe. The model requires as input data the total length of the major pipe run in the sub-area, the average pipe slope, and the diameter and slope of the outfall pipe. Where no details of the outflow pipe are available, as in a design application, its dimensions must be estimated using the Modified Rational Method. Using this Sewered Sub-area Model, substantial savings on input data are possible, with networks of the order of 100 pipes being reduced to only

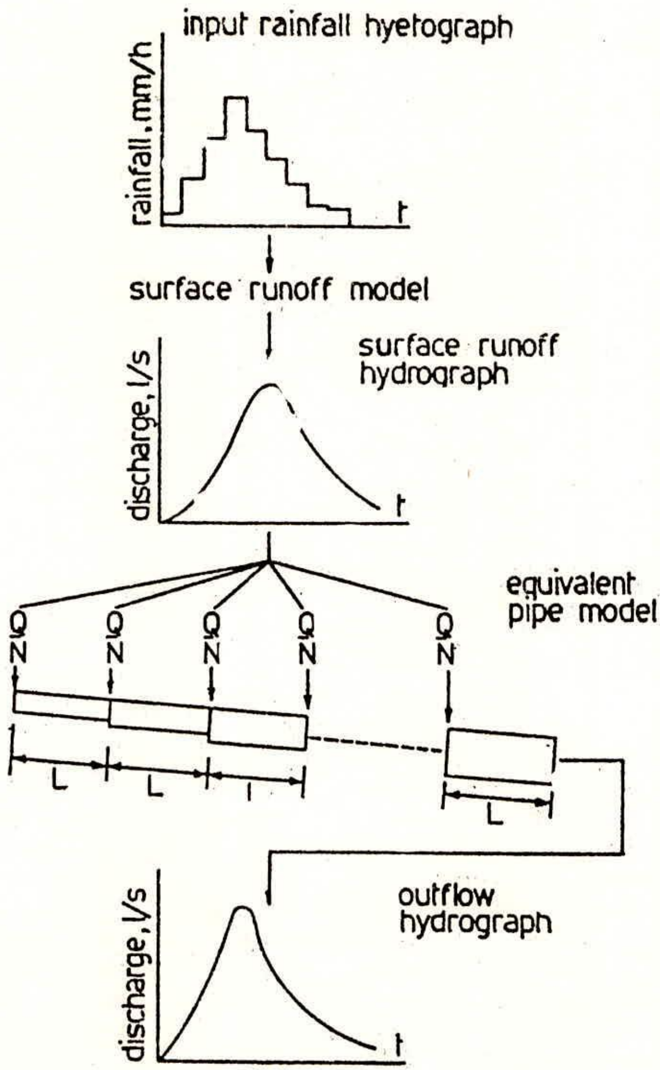

Figure 8: The Wallingford Hydrograph Method: the Sewered Sub-area Model.

four equivalent pipes. Routing of flows through the equivalent pipes is carried out using the Muskingum-Cunge Method.

2.10 Road Research Laboratory Method (RRL)

An urban runoff model (RRL) that utilizes the time-area runoff routing method was developed in England. The technique was developed specifically for the analysis of urban runoff and ignores completely all pervious areas and all impervious areas that are not directly connected to the storm drain system.

The RRL Model could be used for continuous streamflow simulation but tends to be used as an event simulation model. It has been extensively applied in Great Britain, and moderate success has been reported. The Illinois Urban Drainage Area Simulator (ILLUDAS) is an improved version of RRL that has a wider range of capabilities. It incorporates the impervious areas neglected by RRL and is a demonstrated improvement over RRL.

The flow diagram of the processes simulated by RRL is shown in Fig. 9. The major functions of the program involve five principal steps in the development of runoff hydrographs as illustrated in Fig.10. As a first step, the total basin is divided into subbasins similar to the one as shown in Fig.11, and impervious areas that are directly connected to the storm drain system are identified . The remainder of the basin including surfaces such as lawns, floodways, parks, roofs that are not connected to the storm drainage systems, and impervious areas that drain into pervious areas are all ignored by the RRL model.

After hydraulic characteristics such as lengths, slopes, and roughness coefficients are estimated, the second step is the

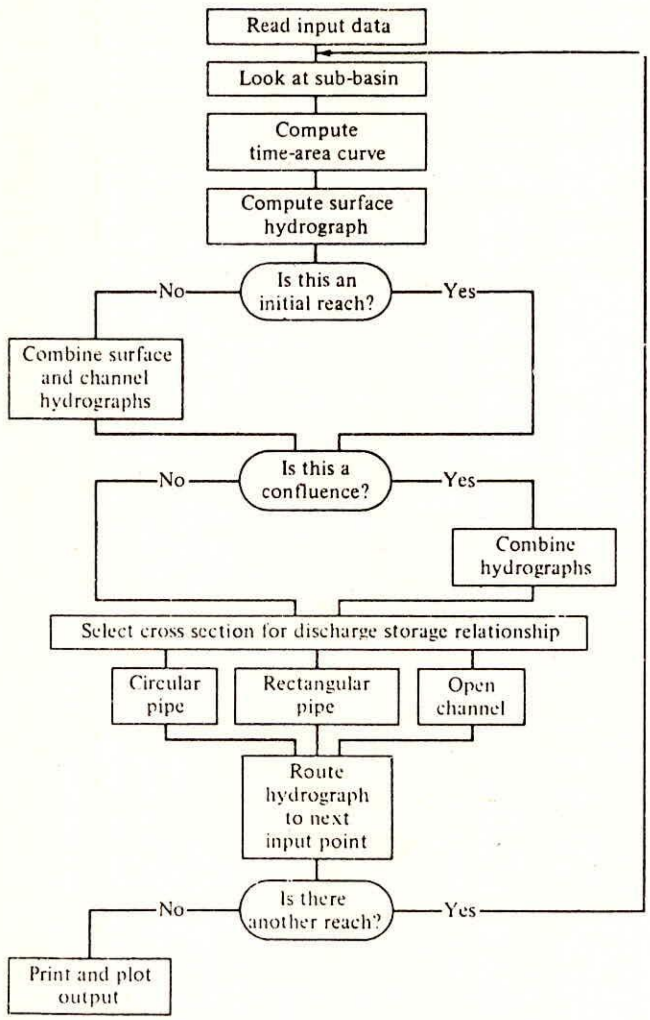

Figure 9: Flow diagram for the computer program of the RRL method.

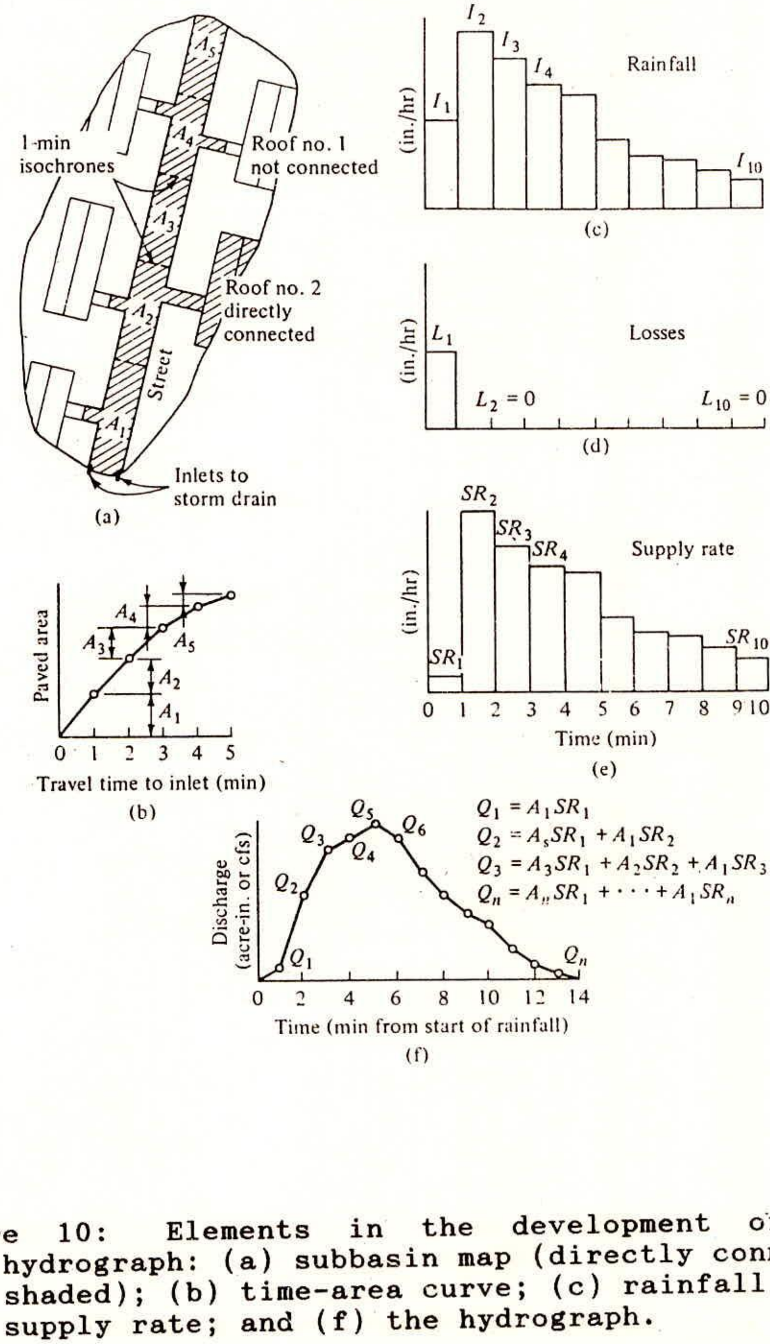

Figure 10: Elements in the development of the time-area hydrograph: (a) subbasin map (directly connected paved area shaded); (b) time-area curve; (c) rainfall; (d) losses; (e) supply rate; and (f) the hydrograph.

calculation of flow velocities for all segments. These velocities are then used to construct lines of equal travel time to the outlet of the basin, called isochrones, on the basin map. The areas between isochrones are then determined and plotted against travel time as shown in Fig. 11.

The third step is to apply the specified rainfall pattern to the directly connected impervious area, and then determine the translated hydrograph at the sub-basin outlet. Excess rainfall hyetograph ordinates are obtained by subtracting the losses from rainfall to give the net supply rate as shown in Fig. 11

Because the routed time-area hydrograph represent translation effects only, the hydrograph must now be routed through reservoir-type storage to account for the effects of storage within the basin. This is accomplished by routing the hydrograph Fig. 12 through a reservoir using the storage-indication method described.

The fifth and final step in the RRL Method is the routing of the subbasin outflow hydrograph to the next confluence or the next input point by a simple storage routing technique. The final result is a total basin runoff hydrograph that would result from the specified storm rainfall.

The merits of the RRL method has been evaluated by applying it to 10 urban watersheds located largely in the east, south, west regions of the United States. The criteria followed in selecting basins for the evaluation where i) basins less than 5 m^2, in size (ii) basins that were intensely urbanized (iii) basins that had extensive storm drainage systems, (4) basins with a high amount of paved area, (v) long records of rainfall and

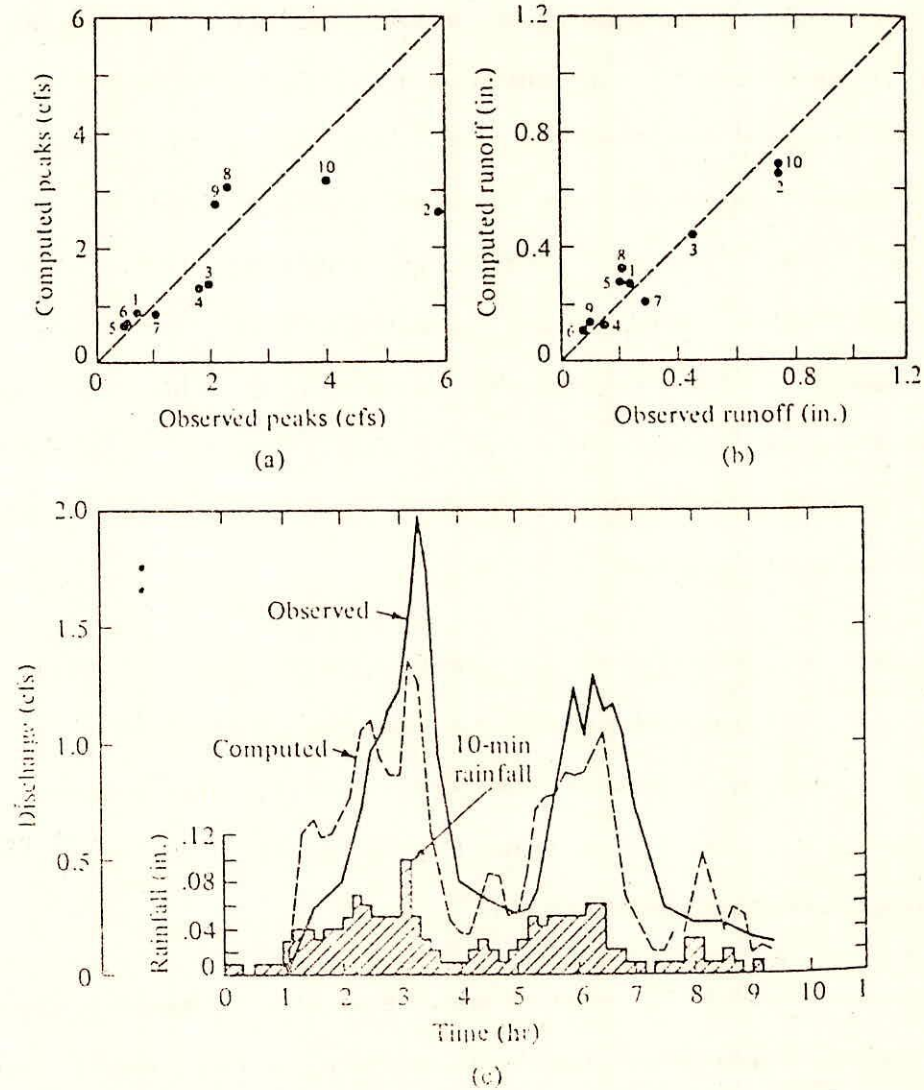

Figure 11: RRL results for Woodoak Drive basin, Long Island, New York, storm of Oct. 19, 1966; (a) peaks; (b) volumes; and (c) the hydrographs.

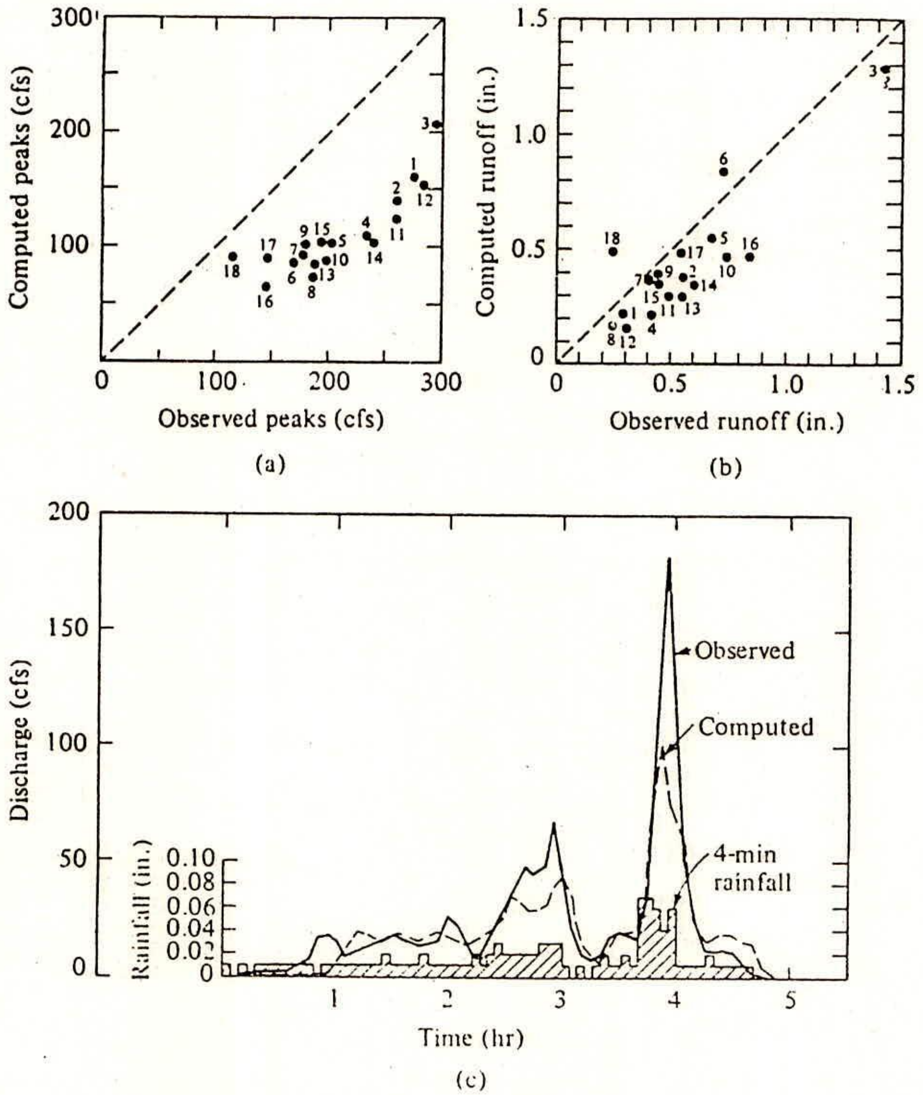

Figure 12: RRL results for Echo Park Avenue basin, Los Angeles, California, storm of April 18, 1965: (a) peaks; (b) volumes; (c) the hydrographs.

runoff, (vi) the degree of quality of the data on storm rainfall and runoff, (vii) the degree of information available on the storm drainage system, and (viii) data that had not already been published in one form or another.

Stall and Terstriep arrived at the following conclusions based on their evaluation of the RRL Method:

1. The RRL method provides an accurate means of computing runoff from the paved area portion of an urban basin.
2. The RRL method adequately represents the runoff from actual urban basins under the following conditions:
   a. The basin area is less than 5 mi^2.
   b. The directly connected paved area is equal to at least 15% of the basin area.
   c. The frequency of the storm event being considered is not greater than 20 yr.
3. The RRL method cannot be used for all urban basins in the United States; the method breaks down when significant grassed area runoff occurs, which happens if one or more of the following conditions exist:
   a. The directly connected paved area is less than 15% of the basin area.
   b. The frequency of the event being considered is greater than 20 yr.
   c. The grassed area of the basin has steep slopes and tight soils regardless of the antecedent moisture conditions.
   d. The grassed area of the basin has steep slopes, moderately tight soils, and an antecedent moisture

condition of 3 or 4.

e. The grassed area of the basin has moderate slopes, moderately tight soils, and an antecedent moisture condition of 4.

4. The principal strength of the RRL method is that, by confining runoff calculations to the paved area of a basin, it utilizes hydraulic functions that are largely determinate such as gravity flow from plain sloping concrete surfaces, gutters, pipes, and open channels. Physical understanding of these flow phenomena is much greater than the present understanding of the many complex phenomena governing runoff from rural areas such as antecedent moisture conditions, infiltration, soil moisture movement, transpiration, evaporation, and so forth.

5. A modification of the RRL method that would provide a function for grassed area contributions to runoff could be developed into a valuable design tool for urban drainage. This is believed to be possible in spite of the many complexities involved. Further flexibility could be offered by the additional provision for routing surface runoff through surface storage.

6. The input data requirements for use of the RRL method on an urban basin are reasonable for the engineering evaluation of a basin for storm drainage design. The necessary data are no more complex or elaborate than the data usually compiled for a traditional storm drainage design.

7. The RRL method is successful and widely used in Great Britain and yet suffers the above-described breakdowns for some of the basins studied in the United States.

8. Better urban rainfall and runoff data are required for the proper testing of all mathematical models. Research basins that do not have hydraulic problems, such as undersized drains or inadequate inlets, should be selected and instrumented.

2.11 Storm Water Management Model (SWMM)

A very widely accepted and applied storm runoff simulation model was jointly prepared by Metcalf and Eddy, Inc.; the University of Florida; and Water Resources Engineers for use by the U.S. Environmental Protection Agency (EPA). This model is designed to simulate the runoff of a drainage basin for any predescribed rainfall pattern. The total watershed is broken into a finite number of smaller units or subcatchments that can be readily described by their hydraulic or geometric properties. A flow chart for the process is shown in Fig.13.

The SWMM model has the capability of determining, for short-duration storms of given intensity, the locations and magnitudes of local floods as well as the quantity and quality of storm water runoff at several locations both in the system and in the receiving waters. The SWMM is an event simulation model and does not keep track of long-term water budgets.

The fine detail in the design of the model allows the simulation of both water quantity and quality aspects associated with urban runoff and combined sewer systems. Information obtained from SWMM would be used to design storm sewer systems for storm water runoff control. Use of the model is limited to relatively small urban watersheds in regions where seasonal differences in the quality aspects of water are adequately documented.

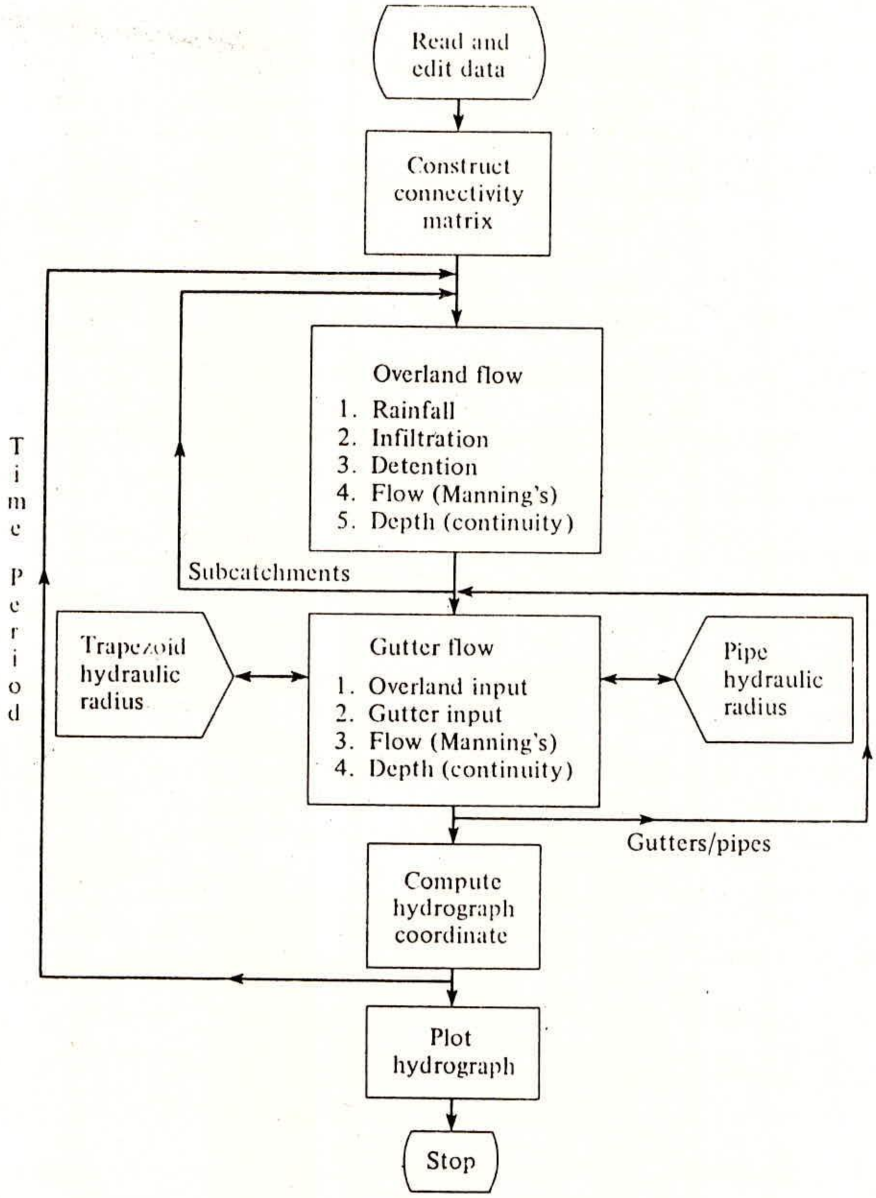

Figure 13: Flow chart for hydrographic computation. "Storm Water Management Model".

The simulation is facilitated by five main subroutine blocks. Each block has a specific function, and the results of each block are entered on working storage devices to be used as part of the input to other blocks.

The main calling program of the model is called the Executive Block. This block is the first and last to be used, and performs all the necessary interfacing between the other blocks.

The Runoff Block uses Manning's equation to route the uniform rainfall intensity over the overland flow surfaces, through the small gutters and pipes of the sewer system into the main sewer pipes, and out of the sewer pipes into the receiving streams. This block also provides time-dependent pollutional graphs (pollutographs).

A third package of subroutines, the transport block, determine the quality and quantity of dry weather flow calculate the system infiltration; calculates the water quality of the flows in the system; and will also calculate the land, capital, and operation and maintenance costs of two optional internal storage tanks through which the combined dry weather and infiltrated flows are routed.

A useful package of subroutines for water quality determination is contained in the Storage Block. The Storage Block allows the user to specify or have the model select sizes of several treatment processes on an optional wastewater treatment facility that receives a user-selected percentage of the peak flow. If used, this block simulates the changes in the hydrographs and pollutographs of the sewage as the sewage passes through the selected sequence of unit processes.

The hydraulic and water quality effects of the effluent from the modeled sewer system on the receiving water body are modeled in the Receiving Water Block. This fifth block of subroutines models the receiving body of water as a network of nodes connected by channel segments. The hydraulics (which determine the resulting water quality) of the flow network are simulated by the Receiving Water Block.

Subcatchment areas, slopes, widths, and linkages must be specified by the user. Manning's roughness coefficients can be supplied for pervious and impervious parts of each subcatchment, or respective default values of 0.250 and 0.013 are assigned by the model.

SWMM is the only event simulation model listed that utilizes Horton's equation for calculating watershed infiltration losses. Infiltration amounts thus determined for each time step are compared with instantaneous amounts of water existing on the subcatchment surface plus any rainfall that occurred during the time step, and if the infiltration loss is larger, it is set equal to the amount available. Input for Horton's equation consists of the maximal and minimal infiltration rates and the recession constant k. Respective default values in SWMM are 3.00 in./hr, 0.52 in./hr, and 0.00115 in./sec.

Urban storm drainage components are modeled using Manning's equation and the continuity equation. The hydraulic radius of the trapezoidal gutters and circular pipes is calculated from component dimensions and flow depths. A pipe surcharges if it is full, provided that the inflow is greater than the outflow capacity. In this case, the surcharged amount will be computed and stored at the head end of the pipe. The pipe will remain full until the sotred water is completely drained.

Necessary inputs in the model are the surface area, width of subcatchment, ground slope, Manning's roughness coefficient, infiltration rate, and detention depth. Gutter descriptions are the length, Manning's roughness coefficient, invert slope, diameter for pipes, and cross-sectional dimensions of the gutter. General data requirements are summarized in Table 5. A step-by-step process accounts for all inflow, infiltration losses, and flow from upstream subcatchment areas, providing a calculated discharge hydrograph at the drainage basin outlet. The following description of the simulation process will aid in understanding the logic of the model.

Table 5: General Data Requirements Storm Water Management Model (SWMM)

Item 1. Define the Study Area.

Land use, topography, population distribution, census tract data, aerial photos, and area boundaries.

Item 2 Define the System

Plans of the collection system to define branching, sizes, and slopes. Types and general locations of inlet structures.

Item 3 Define the System Specialities.

Flow diversions, regulators, and storage basins.

Item 4 Define the System Maintenance

Street sweeping (description and frequency), catchbasin cleaning. Trouble spots (flooding).

Item 5 Define the Receiving Waters.

General description (estuary, river, or lake), measured data (flow, tides, topography, and water quality).

Item 6 Define the Base Flow (DWF)

Measured directly or through sewerage facility operating data. Hourly variation and weekday versus weekend. The DWF characteristics (composited BOD and SS results). Industrial flows (locations, average quantities, and quality).

Item 7 Define the Storm Flow

Daily rainfall totals over an extended period (6 months or longer) encompassing the study events. Continuous rainfall hyetographs, continuous runoff hydrographs, and combined flow quality measurements (BOD and SS) for the study events. Discrete or composited samples as available (describe fully when and how taken.)

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1. Rainfall is added to the subcatchment according to the specified hyetograph:

$$D_1 = D_t + R_t \quad t$$

where

D_1 = the water depth after rainfall

D_t = the water depth of the subcatchment at time t

R_t = the intensity of rainfall in time interval t

2. Infiltration I_t is computed by Horton's exponential function, $I_t = f_c + (f_o - f_c)e^{-kt}$, and subtracted from the water depth existing on the subcatchment

$$D_2 = D_1 - I_t \quad t$$

where

f_c, f_o, and k = coefficients in Horton's equations

D_2 = the intermediate water depth after accounting for infiltration

3. If the resulting water depth of subcatchment D_2 is larger than the specified detention depth D_d, an outflow rate is computed using Manning's equation.

$$V = \frac{1.49}{\quad} (D_2 - D_d)^{2/3} \; S^{1/2}$$

and

$$Q_u = VW(D_2 - D_d)$$

where

V	=	the velocity
n	=	Manning's coefficient
S	=	the ground slope
W	=	the width
Q_u	=	the outflow rate

4. The continuity equation is solved to determine water depth of the subcatchments resulting from rainfall, infiltration, and outflow.

Thus,

$$D_{t+t} = D_2 - \frac{Q_w}{A} t$$

where A is the surface area of the subcatchment.

5. Steps 1 to 4 are repeated until computations for all subcatchments are completed.

6. Inflow (Qin) to a gutter is computed as a summation of outflow from tributary subcatchments ($Q_{w,i}$) and flow rate of immediate upstream gutters ($Q_{g,i}$)

$$Q_{in} = Q_{w,i} + Q_{g,i}$$

7. The inflow is added to raise the existing water depth of the gutter according to its geometry. Thus

$$Y_1 = Y_t + \frac{Q_{in}}{A_s} t$$

where

Y_1 and Y_t = the water depth of the gutter

A_s = the mean water surface area between Y_1 and Y_t

8. The outflow is calculated for the gutter using Manning s

equation:

$$V = \frac{1.49}{n} R^{2/3} S_i^{1/2}$$

and

$$Q_g = VA_c$$

where

R = the hydraulic radius
S_i = the invert slope
A_c = the cross sectional area at Y_1

9. The continuity equation is solved to determine the water depth of the gutter resulting from the inflow and outflow. Thus

$$Y_{t+ t} = Y_1 + (Q_{in} - Q_g) \frac{t}{A_s}$$

10. Steps 6 to 9 are repeated until all the gutters are finished.
11. The flows reaching the point concerned are added to produce a hydrograph coordiante along the time axis.
12. The processes from 1 to 11 are repeated in succeeding time periods until the complete hydrograph is computed.

Three general types of output are provided by SWMM. If waste treatment processes are simulated or proposed, the capital, land, and operation and maintenance costs are printed. Plots of water quality constituents versus time form the second type of output. These pollutographs are produced for several locations in the system and in the receiving waters. Quality constituents handled by SWMM include suspended solids, settleable solids, BOD, nitrogen, phosphorus, and grease. The third type of output is hydrologic related time periods.

3.0 REMARKS

Ever since the digital computers became available to hydrologist, mathematical programming of hydrologic processes has become increasingly popular. The first known hydrologic model of appreciable scale and complexity was the stanford watershed model followed soon by others such as the British Road Research Method, HEC-1 model, Storm Water Management Model, the U.S. Soil Conservation Service TR-20 and many others. Most of the models were written for main frame computers. Urban drainage in india is characterized by its extreme variability of hydrological parameters. When sewerage is designed urban drainage is also provided as a part of combined system. Otherwise separate sewer system are designed and provided with little provision for drainage for example the Calcutta Metropolitan system. The design of drains in rural parts of urban areas, are often approached empirically. The rational formula is commonly used in India to estimate the design peak flow in an urban watershed. Also the hydrologic model, ILLUDAS has been implemented and used for the analysis and design of urban drainage system in India.

The most widely known of the computer based urban rainfall runoff model is the Storm Water management Model (SWMM). In this model, the runoff block is concerned with the derivation of runoff hydrographs and their associated pollutant loadings. The transport block routes both the hydrographs and the time variations of individual pollutants through the sewerage system. The storage and receive blocks simulate the action of a sewage treatment plant and the impact of discharges on the water course receiving the effluent respectively. The Application of SWMM involves the division of the drainage area into a network of idealised elements each of which consists of a uniform land use, slope and surface characteristics. These grid need not to be equal in size but irregular slopes of sub area must be approximated by rectangles of equivalent mean width. The overland flow hydrograph from each plane is derived from water balance computations at each step in which allowance are made for both infiltration and depression storage. These overland flow hydrograph are then routed through gutter storage. The subsequent routing of flows through the lateral may be carried out either in the Runoff block using a simplified approach or where backwater effects are likely to be significant in the transport block using a more sophisticated technique.

Illinois urban drainage model is the testing record for a U.S. adoption of the British Road Research Lab method. The model is based on digital model and used for hydrologic design of storm drainage system. The output from this model are of two types. One for a new design of urban drainage system and other for an evaluation of an existing drainage system. The basic parameters information needed to run the model are; basin parameters like basin area, paved area and grassed area abstraction, information on predominant soil group, manning 'n' value for concrete pipe or clay pipe, Rainfall parameters like duration, return period, time increment, no. of rainfall increments, total rainfall, antecedent moisture condeition, Reach data like Branch number like main branch would be number 1, Reach length, slope, diameter, height, width, lateral, slope, allowable discharge, rainfall ratio and available storage.

The soil conservation service model has the capability of solving many hydrologic problems comprising the formulation of runoff hydrograph, routing hydrograph, combining or seperating of hydrographs at confluence & determine peak discharges and their time of occurrence for individual storm events. These are two approached in the SCS/TR-55 method known as graphical method & tabular method. The tabular method is suitable when a complete hydrograph is desired instead of peak flow or when subdivision of watershed into subareas is involved. For each subarea the following information is required i) weighted curve number, (ii) runoff (iii) time of concentration (iv) travel time downstream of the subarea to the outlet. For selected rainfall distribution type and known Ia/P, the TR-55 provides the hydrograph ordinates for the subarea that correspond to the time of concentration tc and routed to the outlet for the travel time Tt.

Urban hydrologic problems in India differ from those of developed nations in several important respect. They include:

i) limited amount of paved area

ii) preference for open drain over closed one

iii) limited availability of continuous records of precipitation and stream flow

iv) limited number of sewer connection

v) high cost for construction and modification of combined sewer.

The choice of which method is the most appropriate among the several models available are hardly straight forward. A clear distinction must be drawn between design methods and simulation methods. The former are able to calculate the pipe sizes required for a new sewerage system, given the design storm, the layout of the network and other design storm, the layout of the network and other descriptors of the catchment, whereas the latter analyse the performance of an existing system. Using these criteria, the design method are limited to the Rational, TRRL and ILLUDAS. The remainder like SWMM, USGS, TVA are usually simulation methods which are useful in applications such as the renovation and renewal of existing drainage system.

The second factor which may prevent a user from making a clear choice between different design method is a lack of information on their relative performance. The results obtained showed that several of the methods were capable of simualting observed event to an accuracy approaching that of the recorded data.

Several good comparison of RRL, SWMM have been reported in the literature. One of the first was an application of then two models to two urban catchments in Australia for a total of 20 storm events. The following conclusion were drawn:

i) The SWMM was the model with the best overall performance but at the expense of large computer storage and time requirements.

ii) the degree of subdivision of the catchment has a significant influence on the peak discharge.

iii) The Road Research Lab model predicted poorly for storms in which pervious runoff was significant but performed reasonably well for many other type of storms.

iv) A major problem with using noncontinuous models is the prediction of antecedent conditions.

v) Out of the various models, the SWMM simulations were marginally better than those by RRL and both these models were more accurate than UCURM with all models applied in an uncalibrated version.

In summary, an urban stormwater drainage design procedure must consist of several methods, each of which is appropriate to a particular range of drainage area. The choice of model largely depend on the type of problem and input data available. The more complex the design problems, the more sophisticated technique required to obtain the solution. Further subdivision of the procedure is possible according to the need for design or simulation. The hierarchical approach to the design of surface water drainage system is readily shown in the Wallingford procedure.

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