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EFFECT OF URBANISATION ON RUNOFF HYDROGRAPH

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NATIONAL INSTITUTE OF HYDROLOGY
JALVIGYAN BHAWAN
ROORKEE - 247 667 (INDIA)

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PREFACE

Man's impact on the hydrological regime is on a whole, nowhere more intense than in urban areas. The effects of urbanization on the human environment transcend by far the considerations of the hydrological cycle. Expected massive increases in urbanization over the next several decades clearly suggest that present problems will probably be alarmingly compounded. Among the hydrological problems associated with urbanization are the continuously increasing demands for various uses, changes in physical environment that alter the natural water balance and the disposal of wastes that may contaminate streams and ground water. This disposal of water from the urban environment will result in an ever increasing runoff through urbanized drainage network.

Flow in urban drainage system is principally by gravity. One of the many complex problems which has come up with increasing urbanization is that the quick disposal of storm water from the inhabited areas. For design of a good drainage system the correct estimation of runoff from urban areas in a prerequisite. In this report Palam urban catchment has been modeled to find out the effect of urbanisation on run off peak and time of concentration. The study has been carried out by Mr. Manoj Kumar Shukla, Scientist 'B', Drainage Division of Institute.

(S. M. Seth)
DIRECTOR

ABSTRACT

Urbanization which may be broadly defined as the process of expanding urban influence, has been taking place for more than 6000 years, its pace has increased markedly since the beginning of the nineteenth century. With this the demand for water also increased. The role of water as a nuisance became more immediate concern to individual, which are, flooding drainage, erosion and sedimentation. This paved the way for study of urban hydrology as a separate consideration within general field of hydrology.

The important highlight of 1981 census in India is the unprecedented rapid growth of urban population. It is estimated that by the end of this century the urban population in the country will be more than 300 million. At this pace, the urban runoff will also increase tremendously. As we know that at present in India the practice of urban storm runoff measurement is with the help of empirical formula which is not adequate for design of a complex drainage system. Instead more emphasis should be given to implement the findings of recent researches and models developed by various countries of the world.

In this report an attempt is made to find out the effect of change in impervious area on the run off peak and time to peak using a deterministic model Kingen for the Palam urban drainage basin a sub basin of Nazafharg drainage basin, N.Delhi.

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1.0 INTRODUCTION

Human life is seldom threatened by the flooding of underground drainage facilities, except as a human hazard. Because the principle local detrimental effects of flooding are damage to the below ground sections of buildings and hindrance of traffic, the consequence of flooding range from clearly assessable property destruction to annoying inconvenience. It follows that provision of complete protection from flooding can only rarely be justified. Instead, facilities are designed which will be overtaxed infrequently. However, because of the marginal level of protection afforded, storm drainage flooding damages are of considerable magnitude. Indirect damages from local drainage flooding are much more extensive than for stream flooding and generally recur more often, and direct damages are usually much more widely disposed throughout a community.

Flood control, damage and the quality of receiving waters are all closely related. Increased volumes of direct runoff from underground drainage conduits clearly can aggravate flooding of urban flood plains. On the other hand, increased receiving-stream stages can cause or induce flooding of underground drainage systems. Considering that urbanization increases the rates and volumes of runoff delivered locally to receiving waters, it is evident that the conveniences of surface cleansing and land drainage are obtained at the expenses of higher stages and greater pollutant burdens in receiving waters. In a study in Sweden, the estimated annual volume of storm water runoff from urban areas is slightly greater than the annual volume of waste water, and according to similar estimates it is about half as large for F.R.G. The impact of such large quantities of water from urban areas entering receiving water over very brief periods and causing high runoff should be thoroughly studied and investigated.

As the land surface is developed for urban use, a region is transformed from the natural state to a totally man made state. New structures add large amounts of impervious areas to watershed, which is general increase slopes and considerably diminishes the water storage capability. As area covered by structures approaches 100%, the amount of vegetation, natural surface and infiltration will be approaching zero.

1.1 Urban Hydrology

The development of rural areas into urban communities has a significant effect on the hydrologic response because the impervious area is drastically changed and conveyance systems for drainage are often installed. Schulz (1971) summarized the salient features of unit hydrographs generally changed by urbanization-increase of peak discharges, reduction of response time, and reduction of hydrograph base length. Runoff volumes also increase. These changes have been observed since the Nineteenth Century but were not quantitatively investigated until the Twentieth Century. In urban hydrology there is still a shortage of accurate rainfall-runoff field data. Considerable emphasis has been placed on this problem in the last 5 years (Shukla and Soni, 1992).

2.0 REVIEW OF LITERATURE

Several authors including Savini and Kammerer (1961), Leopold (1968), Hall (1973), Cordery (1976), have described the changes in flow regime which occur when an initially rural catchment area is subject to urbanization. The particular aspects of urbanization which exert the most obvious influence on hydrological processes are the increase in population density and the increase in building density within urban area.

Johnson (1967) showed that from 1800 to 1950, world urban population grew from less than 40 million people to greater than 700 million; an increase of about 1650%. Hoyt (1963) estimates that in 1800, 1% of the world's population lived in cities of 100,000 or more. In 1960, he estimated that the proportion has increased to 20%. Rugg (1972) demonstrated that 36 of the major cities of the world experienced an average population increase of about 200% from 1800 to 1850, and about 200% again from 1850 to 1890. For example, London and New York, the too largest cities, went from 958,800 and 62,900 respectively, in 1800 to 4.2 million in 1890. Prof. W.F.Geiger of University of Essen (FRG) quoted that in 1800 only one percent of world population lived in cities, in 1970, 37 percent and by 2000 year the urban population is expected to reach 51 percent of the total. Despite the continuing increase in the extent of major towns and cities in many countries the land occupied by the urban population is often less than 5% of the total area (UNESCO, 1979). The concentration of human activities intensities local competition for all types of resources, among the most vital of which is water. As Schneider et al (1973) have noted, water is both an artery and a vein to urban life.

Several Researchers have worked on the effect on runoff due to urbanization.

Horner and Flynt (1936) quantitatively studied the runoff from two different city blocks in St. Louis, Missouri. Izzard (1946) studied flow over paved and turf surfaces and in gutters from which he developed some empirical curves to estimate the maximum rate of runoff. Much of the early work in calculating runoff from urban areas was based upon the well-known rational formula. Introduction of the unit hydrograph permitted its use in urban hydrology. A survey of current practicing engineers indicated they use the rational method for areas less than 5 square miles; while larger areas are calculated by the unit hydrograph method (Committee on Flood Control, 1969).

Several "hydrograph" methods, which permit estimation of runoff, have been developed in particular regions of the country. Caution must be used if these methods are applied to conditions that may be different than the ones under which they were derived. The Los Angeles Hydrograph Method, developed by Hicks (1944) for use in southern California, is based upon a substantial amount of data from that region. The procedure uses two methods of computing discharges-the peak-rate method (which is a rational type method) and summing hydrographs (used when the time of concentration exceeds 60 minutes or a flow retention structure is part of the system) (Chow, 1964). Tholin and Keifer (1960) published one of the classic reports on urban hydrology, the Chicago Hydrograph Method. A step-by-step design procedure based upon a unit size of 10 acres was presented. Abstractions from design rainfalls were calculated. Overland flow was computer by Izzard's procedure. Routing through all sewers was done by a time-offset method because of its simplicity. From the storm sewer hydrographs, it was possible to develop a series of design charts for peak discharge based on percent of directly connected impervious area, type of land use, and travel time. The time-offset method of routing in storm sewers is often used. This conditions, but its limitations have not been fully evaluated. harris (1970) used a progressive average-lag method for routing in storm sewers. He compared this technique with the method of characteristics for the full dynamic equation of motion. He found a satisfactory comparison of the two methods and thus chose the simplified method; however, this method requires observed hydrographs (at least three) to evaluate the routing constants.

Since the mid-1950's, the Johns Hopkins University has conducted extensive research in storm sewer drainage. An inlet hydrograph method was developed, based on a rational type formula for peak flows and an assumed triangular shape. These hydrographs are summed to obtain the total hydrograph, after each inlet hydrograph is reduced by a factor based on the time characteristics of the event (Viessman and Geyer, 1962). Schaake (1970) applied a kinematic wave model by separating the catchment into segments over which the model parameters were assumed uniform. He presented a technique to compute the kinematic parameter, based upon geometrical characteristics and assumed types of flow for a segment. The model was tested on an 0.4 acre experimental catchment in Baltimore, Maryland. The University of Cincinnati developed a runoff model for urban watersheds (1972). Infiltration on pervious segments was computed by Honton's equation with surface retention estimated by an exponential relationship recommended by Linsley, Kohler, and Paulus (1949); average values for impervious and pervious segments given if measured data were not available on the watershed. Overland flow was assumed to be turbulent and computed by a storage routing procedure, while gutter flow was computed strictly by continuity and was assumed to occur over relatively short lengths. Sewer routing was performed by undistorted lagging of the inflow hydrograph. This procedure results in higher peaks at later times than more exact methods. The model was applied to a 13-acre watershed in Chicago with satisfactory agreement between observed and computed hydrographs, except on the recession portion.

In 1969, the Denver Regional Council of Governments published an urban storm drainage criteria manual. This manual outlined design requirements for urban storm drainage projects in the Denver region. Rainfall-frequency maps were prepared up to the 100 year return period. The rational formula was used to compute runoff in areas which did not contain storm sewers and were less than 200 acres. The unit hydrograph method was used for areas larger than 200 acres manual outlined procedures to estimate the rainfall excess and compute runoff by the rational formula with typical coefficients or from the specified unit hydrograph method.

Lighthill and Whitham (1955) considered propagation of flood waves in rivers as mainly kinematic, a balance of bed slope and friction slope; they also investigated kinematic shock waves Wooding (1965) applied kinematic wave theory to a catchment formed by two planes in a V-shape, each discharging into a stream at the center. He concluded that kinematic theory was applicable to gradually varied unsteady flow if the Froude number was less than 2. Woolhiser and Liggett (1967) showed how the use of dimensionless continuity and momentum equations could reduce the number or parameters for overland flow on a plane from 5 to 2. A parameter of the dimensionless momentum equation was used to measure the applicability of kinematic wave theory. The parameter was

$$K = \frac{S\ L}{Ho\ Fo^2} \qquad (2\text{-}1)$$

where S is surface slope; L is length of flow; Ho is normal depth; and Fo is the Froude number for normal flow. Woolhiser and Lggett have shown that for $K > 10$, the kinematic wave solution, labeled $K = \infty$, is a good approximation. The kinematic wave parameter is often several thousand or more for many cases of overland flow.

Foster, et. al. (1968) simulated rainfall on an erodible fallow plot and found that a kinematic wave model satisfactorily predicted overland flow. Observed hydrograph data were analyzed to estimate retention storage and surface roughness. A comparison was made between a constant Darcy-Weisbach f or Manning's n and a variable friction factor of the form

$$f = a R_e^b \tag{2-2}$$

where a and b are constants and Re is the Reynolds number. The constant friction factor gave results as good as a variable factor, which would indicate the flow was turbulent. Henderson (1963) and Eagleson (1970) have utilized the kinematic wave theory, but they limit the kinematic waves to a non-subsiding state. They do not account for the subsidence property of kinematic shock waves that may exist. Kinematic shocks result when waves travel downstream. This phenomenon is represented mathematically by an upstream characteristic intersecting a discontinuity at the intersection (see section on kinematic Equations for a Plane for mathematical definition of a characteristic). Kibler (1968) and Kibler and Woolhiser (1970) developed dimensionless kinematic equations for a cascade of planes and developed a parameter based upon the widths, slope, and roughness of adjoining planes to predict occurrence of kinematic shocks. A method for tracing the shock waves was also presented.

The kinematic wave models that have been formulated have been solved by a variety of finite difference methods, some implicit and some explicit. Brakensiek (1967a) tested three types of finite difference methods on a kinematic model of flood routing and found that a four point implicit scheme, which centered on the two upper points, gave the most satisfactory results. Kibler and Woolhiser (1970) found that an explicit finite difference scheme with second order accuracy was the most satisfactory numerical method for their studies of overland flow.

Kinematic models have usually been used to simulate hydrographs of individual runoff events. Such simulations require that the surface geometry or macro scale features of the watershed, like length, width and slope of overland flow areas and channel lengths, slopes, and cross-sections, be measured from topographical maps and incorporated into the model geometry.

Rao, et.al. in 1972 found that an increase in the area imperviousness from 0 to 40 percent would approximately halve the time to peak discharge and increase its magnitude by 90 percent. Unesco, 1978 also reported that design of urban drainage system in India is based on rational formula because of lack of adequate continuous records of precipitation and stream flow. Yet, there is a vital need of standardization of design procedures based on engineering and economic considerations.

Crippen in 1965 employed unit hydrograph methods on the sharon Creek watershed, a small 245 acre basin and found that peak discharge (Q_p) increased from 180 cubic feet/second to 250 cms/s as urbanization proceeded over the several year study period. Hobert Massing in his study on Hydrological effects of urbanization in Federal Republic of Germany has reported that while the urban area in Maichingen increased from 4 sq.km. in 1972 to 20 sq.km in 1985, the runoff coefficient also increased from 0.15 to 0.6, thereby increasing flood peak runoff of Schwippe river two fold.

Mcpherson in his study on hydrological effects of urbanization in United States of America indicated that in general the volume of flood runoff increases with the increase in impervious areas and by the construction of storm sewers, gutters, catch basins, etc. Runoff peak flows in stream and drains, changes in timing and time distribution of direct runoff from urban areas are striking reflection on influence of urban development. Kuprianov, V.V. in his studies on hydrological effects of urbanization in the former USSR reported that annual runoff from urbanized territory may be greater than 10% or more, than that from rural areas. Maximum discharges of normal rainfall floods may be several times greater than in natural watersheds. As absolute volumes of flood depends on amount and intensity of precipitation as well as the extent of areas of impervious surface.

Furguson and Sucking in 1989 presented preliminary results of annual and base flow studies on Peach tree Creek, comparing rainfall runoff relationships in early and late periods in the watershed's urbanization. This paper extends the authors' previous work by adding peak flow analysis, expanding the base flow analysis, and adding more recently available data to all studies. Matti Melanen and Risto Laukkanen in their study on analysis of rainfall-runoff relationships in Finnish urban test basins have concluded that on an average, the portion of surfaces generating runoff in a urban basin may be estimated as 80-90 percent of the portion of paved (impervious) surfaces with direct assess to the storm drainage system.

Langford and Turner (1973) conducted an experimental test to evaluate the accuracy of kinematic wave theory as applied to overland flow. The close agreement of experimental and calculated recessions showed the accuracy of this theory in predicting the behavior of flows over the rough uneven surface. The hysteresis in the storage - discharge curves was also explained in terms of kinematic wave theory.

Ponce et. al. (1978) assessed the applicability of Kinematic and diffusive wave models and found that larger bed slopes are for those of overland flow and long wave periods are those corresponding to slow rising flood waves.

For the detailed review on the aspect of effect of urbanisation on runoff and on the various urban hydrological models, the reader can refer reports, TN-93 and TR- 144.

2.1 Kinematic Wave Theory

De Saint Venant in 1871 has developed the continuity and momentum equations for gradually varied unsteady flow (Yevjevich, 1960). Many researchers have studied gradually varied unsteady flow and found conditions for which a simplification of the complete momentum equation and the continuity equation are sufficiently accurate.

Kinematic wave models have been used to simulate runoff from agricultural areas and found to give satisfactory results. Woolhiser, Hanson and Kuhlman (1970) modeled runoff as beginning under laminar conditions and changing to turbulent flow at a Reynolds number of 300 for a short-grass, grazed prairie. The average value of the parameter k was about 7,000. Langford and Turner (1973) simulated rainfall on a stabilized fallow surface with a friction relationship in the form of laminar-turbulent Manning's n that varied with rainfall intensity. The surface retention showed a hysteresis effect because of changing hydraulic roughness under conditions of rain and no rain. Brakensiek (1967b) depicted a mixed-cover, agricultural watershed in

Wisconsin as a distributed system by utilizing a hypsometric curve and contour length-elevation curve. He fitted hydrographs by varying Manning's n and obtained satisfactory results with values of 0.08 to 0.10 for n. These values seem low for an agricultural area. Overton and Brakensiek (1970) also applied the kinematic wave model for a V-shape configuration. They derived a lag time based on watershed dimensions, roughness, and rainfall rate. Their relation between lag time and rainfall rate well with observed data for several events on a Hastings, Nebraska, experimental watershed. A sensitivity analysis showed the solution more sensitive to errors in rainfall than to errors in averaging geometry and roughness.

2.2 Infiltration

Any watershed model simulating runoff from a partially or totally pervious surface must have a means of estimating infiltration. The process of infiltration has remained as one of the most complex problems faced by the watershed engineer. Many methods have been developed for estimating infiltration quantitatively-some empirical and some based on theoretical relationships. Horton's (1940) infiltration equation accounts for the time variability of infiltration. The equation is

$$f = f_{\infty} + (f_o - f_{\infty})\, e^{-ct} \tag{2-3}$$

where f is infiltration rate at time t; f_{∞} is the steady-state infiltration rate; and c is a parameter related to the soil cover complex.

Philip (1969) developed a theory of infiltration based upon the governing relationship for movement of a fluid in porous media. An algebraic form of his relationship for infiltration from a ponded surface is where s is the 'sorptivity' of the soil, a measure of the influence of capillarity, and A is an approximate value of the steady state infiltration rate. Several empirical methods are merely indices of infiltration and assume a constant loss rate throughout the entire hydrograph. The Index and W Index are the best known of this type. These indices are best suited for major storms occurring on wet soils or storms when the peak rates and durations occur after infiltration can be approximated as a constant.

$$f = 0.5\ st^{-0.5} + A \tag{2.4}$$

The partial differential equation that governs one-dimensional flow of water in an unsaturated porous medium (ignoring air counter-flow) is often referred to as Richard's equation (Smith and Woolhiser, 1971)

$$\frac{\delta(Sa\ \phi)}{\delta t} = Ks\ \frac{\delta}{\delta z}\left[Kr\ \frac{\delta\varphi}{\delta z} \right] - Ks\ \frac{\delta Kr}{\delta z} \tag{2.5}$$

where Sa is relative saturation, ϕ is porosity, Ks is saturated conductivity, Kr is relative conductivity; φ is soil capillary potential; and z is distance below the surface.

Smith (1972) reported on extensive numerical experiments based on Richard's equation for a range of soils from fine clay with swelling properties to a moderately uniform sand. The infiltration model resulted from analysis of simulation using a uniform rainfall rate for six soils. Initially, the infiltration rate is limited by the rainfall rate, i. Then, soil surface capillary potential goes to zero and surface runoff begins at the time denoted tp. This time marks the beginning of the infiltration decay-type function that has the form:

$$f = f_{\infty} + A\ (t - t_0)^{-\alpha} \tag{2.6}$$

where f is the infiltration rate: f_{∞} is the steady-state infiltration rate; t is time; t_c is the vertical asymptote of infiltration decay function; and A and are parameters unique to a soil, initial moisture, and rainfall rate.

3.0 Mathematical Model

There are mathematical watershed models, such as the Stanford Model, that utilize some aspects of kinematic theory plus other mathematical functions to simulate the hydrologic processes for a continuous period of time, generally, for several months or years. The model that is developed in this study is designed to have the capability of predicting storm runoff from agricultural or urban watersheds for discrete periods of time, generally, for several hours to no more than 1 or 2 days.

The model used herein is classified as non-linear, deterministic, and distributed. Input to the model is: (1) the hyetograph of precipitation as measured on or near the watershed and is assumed constant over the watershed, (2) the geometry and topography as determined from a map of the area, (3) two parameters, which relate to the surface roughness characteristics and the regime of flow (laminar or turbulent) which would be expected to occur, and (4) infiltration characteristics for pervious areas. The watershed is segmented into a series of planes cascading onto other planes or connected with other planes by channels. The planes are either impervious, i.e., streets or parking lots, or are pervious, i.e., rural open areas or lawn areas. The channels are assumed to have either a trapezoidal or circular cross section.

3.1 Surface Water Routing

The governing equations of motion for spatially varied, unsteady flow over a plane surface are derived by applying the principles of conservation of mass and momentum.

The one-dimensional continuity equation with lateral inflow is as below

$$\frac{\delta h}{\delta t} + \frac{\delta (uh)}{\delta x} = q \qquad (3\text{-}1)$$

where h = the depth of flow; u = local average velocity; q = lateral inflow; x = the distance from the upstream end; and t = time.

The momentum equation for one-dimensional gradually-varied, unsteady flow is

$$\frac{1}{g}\left(\frac{\delta u}{\delta t} + u\frac{\delta u}{\delta x}\right) + \frac{\delta h}{\delta x} = S - S_f - \frac{q}{g} * \frac{u}{h} \qquad (3\text{-}2)$$

where g is the acceleration due to gravity; S is the slope of the bed surface; and Sf is the friction slope.

Above equations for gradually varied unsteady flow are based on the following assumptions (Yevjevich and Barnes, 1970):

i) The slope of the bed surface, S, is small and is approximately equal to the sine of the angle of inclination.

ii) The flow is one-dimensional so that the vertical components of velocity and acceleration are negligible.

iii) The pressure in the vertical cross section is hydrostatic.

iv) Boundary friction and turbulence can be accounted for by introduction of a resistance term that is the same as at a corresponding term that is the same as at a corresponding uniform flow depth.

v) The velocity distribution in the vertical cross section is the same as the distribution in steady flow.

Each term in the momentum equation corresponds to a component of the energy gradient as

$\frac{1}{g} \frac{\delta u}{\delta t}$ = Slope due to velocity variation with time

$\frac{1}{g} u \frac{\delta u}{\delta x}$ = Slope due to velocity variation with distance in the direction of flow

$\frac{\delta h}{\delta x}$ = Slope of water surface

$-\frac{q}{g} * \frac{u}{h}$ = Slope due to lateral inflow

and S and S_f are slopes as defined previously.

Lighthill and Whitham (1955), Henderson (1963), and Woolhiser and Liggett (1967)It has reported that the gravity and friction components dominate the other terms of the momentum equation and approximate equilibrium is obtained so that the momentum equation reduces to

$$S = S_f \tag{3-3}$$

This simplification is known as the kinematic wave approximation to the momentum equation.

Kinematic Equations for a Plane

Equation (3-3) can be used to write a parametric equation for the local velocity as

$$u = \alpha h^{N-1} \tag{3-4}$$

where h = local mean depth,

N = parameters related to surface roughness and geometry.

Chezy's turbulent flow formula is

$$u = C (R S_f)^{0.5} \tag{3-5}$$

where R is the hydraulic radius and C is the Chezy friction factor of flow resistance. For planes and wide channels, R = h. This approximation and the substitution of Eq. (3-3) into Eq. (3-5) results in Eq. (3-4) with $\alpha = C* S^{0.5}$, and $N = 3/2$.

For laminar flow, the Darcy-Weisbach friction factor is

$$f = \frac{k}{R_e} \tag{3-6}$$

where k is a dimensionless friction parameter and Re is the Reynolds number. The Darcy-Weisbach formula.

$$S_f = \frac{f}{4R} \frac{u^2}{2g} \tag{3-7}$$

can be rewritten upon substitution of Eq. (3-3) as

$$S = \frac{f}{8g} \frac{u^2}{h} \tag{3-8}$$

The Reynolds number is

$$R_e = \frac{uh}{\nu} \tag{3-9}$$

where ν is the kinematic viscosity. Substituting Eq. (3-9) and (3-6) into Eq. (3-8) yields

$$S = \frac{f \; Re \; \nu \; u}{8g \; h^2} \tag{3-10}$$

or $$u = \frac{8g \; S \; h^2}{k \; \nu} \tag{3-11}$$

Equation (3-11) has the form of Eq. (3-4) with

$$\alpha = \frac{8gs}{k \nu} \quad \text{and} \quad N = 3$$

Equation (3-4) can be substituted into Eq. (3-1) and yields

$$\frac{\delta h}{\delta t} + \frac{\delta(\alpha h^{N-1} h)}{\delta x} = q \tag{3-12}$$

$$\frac{\delta h}{\delta t} + \alpha N\ h^{N-1}\ \frac{\delta (uh)}{\delta x} = q \qquad (3\text{-}13)$$

The total differential of h[x,t] is

$$dh = \frac{\delta h}{\delta t}\ dt + \frac{\delta h}{\delta x}\ dx \qquad (3\text{-}14)$$

Equations (3-13) and (3-14) can be solved simultaneously. The matrix form of the equations is written equating the determinant of the square matrix to zero defines the path of the characteristic in the x-t plane

$$\begin{vmatrix} 1 & \alpha N h^{N-1} \\ dt & dx \end{vmatrix} \begin{vmatrix} \frac{\delta h}{\delta t} \\ \frac{\delta h}{\delta x} \end{vmatrix} = \begin{vmatrix} q \\ dh \end{vmatrix} \qquad (3\text{-}15)$$

$$\frac{dx}{dt} = \alpha\ N\ h^{N-1} \qquad (3\text{-}16)$$

Substituting the column vector of the right hand side of Eq. (3-15) into the second column of the square matrix and equating the determinant to zero defines the rate of change of depth with respect to time along the characteristic

$$\frac{dh}{dt} = q \qquad (3\text{-}17)$$

Equations (3-16) and (3-17) are the characteristic equations. Equation (3-17) can be integrated for constant q, to find the depth along the characteristic as

$$h = h_o + q(t - t_o) \qquad (3\text{-}18)$$

where ho is the initial depth at time to. The uniform flow equation can be written

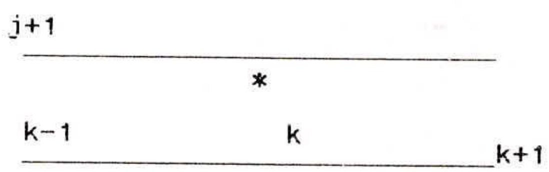
$$Q = \alpha\ h^N \qquad (3\text{-}19)$$

where Q is the discharge rate. Equations (3-16) (3-18), and (3-19) can be used to compute the entire outflow hydrograph for a single plane segment from a constant lateral inflow rate of q.

This development of dimensional kinematic flow equations for a single plane is analogous to the equations for a wide channel and the development for the channels will not be repeated here. Discussion of the equations for a wide channel will follow in a later section.

Finite Difference Method of Solution

Several different methods of numerical solutions including: (1) an upstream differencing scheme, (2) a four-point implicit and (3) Lax-Wendroff scheme. In this model Lax-Wendroff scheme used. This method has second-order accuracy but because it is explicit, it requires a limitation of the time step size to maintain numerical stability. Figure below shows the notation for the Lax-Wendroff scheme.



The strategy of solving the kinematic equation is to find the depth at the advanced time step, $h[x, t+\delta t]$, in terms of known values. Expanding $h[x, t+\delta t]$ in a Taylor's series

$$h[x, t+\delta t] = h[x,t] + (\delta h/\delta t)\delta t + (\delta^2 h/\delta t^2)\delta t^2/2 + 0(\delta t)^3 . \quad (3\text{-}20)$$

where $(\delta t)^3$ is the order of the truncation error. Equation (3-12) can be written as

$$\frac{\delta h}{\delta t} = - \left[\frac{\delta(\alpha h^N)}{\delta x} - q \right] \quad (3\text{-}21)$$

then

$$\frac{\delta^2 h}{\delta t^2} = - \frac{\delta}{\delta t} \left[\frac{\delta(\alpha\ h^N)}{\delta x} - q \right]$$

$$= - \frac{\delta}{\delta x} \left[\frac{\delta(\alpha\ h^N)}{\delta t} \right] + \delta q/\delta t \qquad (3\text{-}22)$$

$$= - \frac{\delta}{\delta x} \left[\alpha\ N\ h^{N-1} \frac{\delta h}{\delta t} \right] + \delta q/\delta t \qquad (3\text{-}23)$$

Now, substitute Eq. (3-21) into Eq. (3-23), then

$$\frac{\delta^2 h}{\delta t^2} = - \frac{\delta}{\delta x} \left[\alpha\ N\ h^{N-1} \frac{\delta(\alpha h)^N}{\delta x} - q \right] + \delta q/\delta t \qquad (3\text{-}24)$$

Substituting equations (3-21) and (3-24) in the equation (3-20)

$$h[x,\ t+\delta t] = h[x,t] - \delta t \left[\frac{\delta(\alpha\ h^N)}{\delta x} - q \right]$$

$$+(\delta^2 t/2)\{ \frac{\delta}{\delta x} \left[\alpha\ N\ h^{N-1} \frac{\delta(\alpha h^N)}{\delta x} - q \right] + \delta q/\delta t \qquad (3\text{-}25)$$

This second order approximation for $h[x,\ t+\delta t]$ provides the basis for the Lax-Wendroff finite difference formulation. This finite differencing formulation permits the evaluation of depths interior of the upstream and downstream boundaries.

The solution for the entire length of flow can be established when the initial and boundary conditions are established. The initial condition must be specified as

$$h[x,\ 0] \left| \begin{array}{l} 0 \\ \text{or, for all } x \\ > 0 \end{array} \right. \qquad (3\text{-}27)$$

The upstream boundary depth is determined by the position of the plane in a cascade. Consider a cascade of planes where i is the order of the plane in the cascade (for the uppermost plane, i - 1), l is the length of a plane, and w is the width. Then,

$$h[0, t]_i = \begin{cases} 0 \text{ if } i=1 \\ f(h[1, t]_{i-1}, W_{i-1}, W_i), \text{ if } i > 1 \end{cases} \quad (3\text{-}28)$$

The discharge from an upper plane is assumed to be modified as the ratio of the upper width to the lower width. The upstream boundary depth for the ith plane which receives inflow from the (i-1)th plane is found by

$$h[0,t]_i = [(Q[1, t]_{i-1} \frac{W_i}{W_{i-1}}) \frac{1}{\alpha_i}]^{1/N_i} \quad (3\text{-}29)$$

Equation (3-29) defines the upstream boundary depth. The downstream boundary depth cannot be obtained from the finite difference scheme because of the nature of the scheme. However, the characteristic equation can be used to obtain the depth at the downstream boundary. Equation (3-18) with $t_0 = 0$, can be substituted into Eq. (3-16) to obtain

$$dx = \alpha\ N\ (h_0 + qt)^{N-1} dt \quad (3\text{-}30)$$

Integration of this equation yields

$$x = (\alpha\ /q)\ (h_0 + qt)^N\ dt \quad (3\text{-}31)$$

where c is a constant of integration. At $x = x_0$ and $t = t_0$, the initial location and time, the constant can be evaluated as

$$c = x_0 - (\alpha\ /q)\ (h_0 + q\ t_0)^N \quad (3\text{-}32)$$

Equation (3-32) can be substituted into Eq. (3-31) with the result

$$x = x_0 + (\alpha\ /q)\ [(h_0 + qt)^N - (h_0 + q\ t_0)^N] \quad (3\text{-}33)$$

but because $h = h_0 + qt$ and if $t_0 = 0$, eq. (3-33) changes to

$$x - x_0 = \frac{\alpha}{q}(h^N - h_0^N) \tag{3-34}$$

If $x - x_0 = \delta x$ rearrangement of equation (3-34) yields

$$h = (h_0^N + \frac{q}{\alpha} \delta x)^{1/N} \tag{3-35}$$

The downstream boundary depth was found by integrating the characteristic equations of the kinematic equation.

Numerical Stability

A disadvantage of the Lax-Wendroff scheme, when compared with the implicit scheme that was analyzed by Kibler and Woolhiser (1970), is that a numerical stability criterion must be maintained, while the implicit scheme is unconditionally stable. A numerically stable finite difference scheme is one that does not allow a small perturbation in the solution to grow without limit until it destroys the calculation. The stability criterion for the Lax-Wendroff finite difference scheme can be derived by an approximate method. A numerically stable scheme is one in which the ratio of successive error terms is less than or equal to unity.

The stability criterion for this finite difference scheme is

$$\alpha N h^{N-1} \frac{\delta t}{\delta x} \leq 1 \tag{3-38}$$

so that for a fixed length increment, Δt, and the largest depth on the surface at time t, h_{max},

$$\Delta t \leq \frac{\Delta x}{\alpha N h_{max}^{N-1}} \tag{3-39}$$

insures that stability exists at all points on the surface. This method of deriving the stability criterion is only approximate since it is based upon a linear analysis. Eq. (3-39) does indicate an appropriate time step for the Lax-Wendroff scheme.

3.2 Channel Routing

Free surface flow in channels can be computed using the kinematic approximation to the equations of unsteady, gradually varied flow. The difference between routing runoff over planes and through channels is that upstream inflow to a plane is given in discharge per foot of width of the plane, while upstream inflow to a channel is the total discharge from the previous segment. For watershed area computation, a channel is assumed to have negligible width. Therefore, rainfall does not fall directly onto the channel. The lateral inflow to a channel is the discharge per foot of width received from an adjacent plane.

The general geometrical shape that is considered in this study is a trapezoidal. The trapezoidal shape can be used to simulate geometry from nearly rectangular to very broad swale-like channels, including triangular, by specifying the proper geometric parameters.

Trapezoidal Open Channels

The continuity equation for a channel with lateral flow is

$$\delta A/\delta t + \delta Q/\delta x = q \qquad (3\text{-}40)$$

where A is the cross-sectional area; Q is the channel discharge; and q is the lateral inflow per foot of length. Assuming that Q can be expressed as a function of A, Eq. (3-40) can be rewritten in the form

$$\delta A/\delta t + \delta Q/\delta A * \delta A/\delta x = q \qquad (3\text{-}41)$$

A Taylor's series expansion of Eq. (3-41) can be performed analogous to that Eq. (3-20) using Lax-Wendroff finite difference scheme for a channel with lateral inflow. The kinematic approximation is entered into the calculation through the discharge relationship. If the Chezy formula is used, then

$$Q = \alpha R^{N-1} A \tag{3-42}$$

where $\alpha = C\ S^{.5}$; R is the hydraulic radius; A is the cross-sectional area; and N is 3/2. Hydraulic radius is A/P where P is the wetted perimeter. Than Q is related to the cross-sectional area by

$$Q = \alpha A^{N} / P^{N-1} \tag{3-43}$$

The downstream boundary solution is found by a first-order finite difference scheme based on Eq. (3-40)

3.3 Computer Program KINGEN

The program, namely KINGEN which has been compiled by Rovey et al has been used in the present study to simulate the overland flow. The model used is a kinematic through a cascade of overland flow planes and channels using kinematic wave theory. It is a nonlinear, distributed deterministic model (Kibler and Woolhiser 1970). KINGEN is a computer program based upon the mathematical model described in section 3.0. Kingen model is capable of computing flow for the following geometrical segments: overland flow over a rectangular impervious surface, overland flow over a rectangular pervious surface with an infiltration component to compute rainfall excess, open channel flow in a trapezoidal-shaped channel, and free surface flow in a circular conduit.

Watershed geometry is represented by combinations of the channel and plane segments. Model parameters are estimated for each watershed under study from available information such as from topographic maps, aerial photographs, soil surveys, property development records, watershed reconnaissance, infiltration tests or any other source that may contain hydrologic information. Input data are utilized by the model to sequentially compute the outflow hydrograph from each segment. The computation begins on the segment at the highest elevation of the watershed and continues down slope to the lowest point on the watershed. The program KINGEN consists of a main program and 19 subroutines.

3.4 Estimating Runoff from Urban Areas

3.4.1. Runoff equation

The amount of runoff from a storm event largely depends on detention, infiltration, evapotranspiration etc. and is related to soil type, type of vegetation, and amount of impervious cover.

The relationship between accumulated storm rainfall P, runoff Q and infiltration plus initial abstraction (F + Ia) and potential abstraction S is given by

$$\frac{F}{S} = \frac{Q}{P_e} \qquad (3.4.1)$$

Assuming Pe = Potential runoff or effective storm r.o. (storm rainfall- initial abstraction)

$$F = P_e - Q$$ eqn (3.4.1) becomes

with

$$Q = \frac{P_e^2}{P_e + S} \qquad (3.4.2)$$

The initial abstraction (Ia) in inches is estimated from an empirical relation based on data of small watershed thus $P_e = P - Ia = P - 0.2S$

where P = Total storm rainfall (inch)

Therefore $$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad (3.4.3)$$

Potential abstraction is related to the soil and cover conditions of a watershed. The runoff curve number, which is also related to soil and cover condition, is related to potential abstraction S by

$$CN = \frac{25400}{S + 254}$$

on $$S = \frac{25400}{CN} - 254 \qquad (2.4.4)$$

In the determination of urban CN's, consideration should be given to the type of land use and the hydrologic soil group of the area. Tables giving the CN values for agriculture, suburban and urban areas can be easily obtained from SCS Hand book no.4. In the present study as given by ministry of Agriculture Ia = 0.3 S is used.

Therefore $$Q = \frac{(P - 0.3S)^2}{P + 0.7S} \qquad (3.4.4)$$

4.0 METHODOLOGY

A watershed is a complex system with stochastic inputs. Once the physical characteristics of the system, its initial state and the inputs are specified the response of the system becomes essentially deterministic. The basic hydraulic element of a watershed comprises on overland flow plane discharging as a lateral inflow to a channel. The flow in this element is unsteady

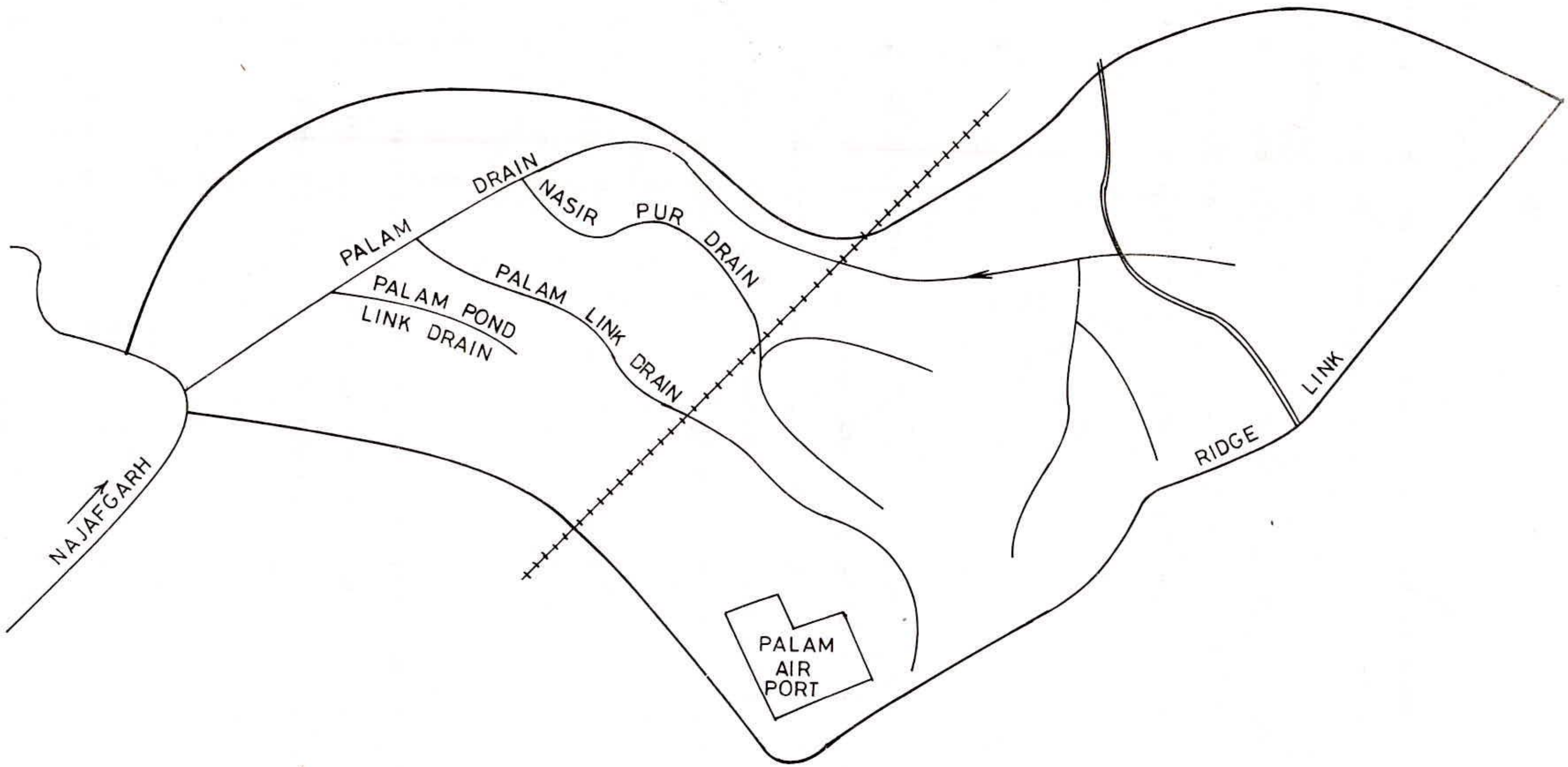


FIG.1 PALAM URBAN CATCHMENT

and gradually varied. The lateral inflow to the plane is rainfall excess (rainfall less infiltration loss). The complex geometry of the watershed can be simplified by a large number of planes and channels. The theory of kinematic model has been reviewed in section 2. This model is best suited for a basin having limited hydrologic data because the input parameter for the model can be estimated from the available topographic maps, areal photographs, soil surveys and from other sources containing hydrologic data.

The aim of the present study is to simulate the surface runoff of the Palam urban catchment of New Delhi (Fig 1) using kinematic cascade model. The study of simulation of runoff from the Palam basin has been carried out for a rainfall events of hourly maximum for two years R.I. The input parameters have been obtained from topographic sheets, irrigation atlas and soil survey map.

4.1 DESCRIPTION OF THE WATERSHED

The Palam drainage is a basin sub basin Nazafgarh drainage basin, New Delhi having a catchment area of about 58.0 Sq.km. The catchment is geographically located between $77^{\circ}0'$ and $77^{\circ}15'$ East longitudes, and $28^{\circ}30'$ North and $28^{\circ}40'$ North Latitudes. The study area lies in the southern part of New Delhi (Fig 1).

4.2 TOPOGRAPHY

The shape of the catchment is fan shaped. The altitudes of the watershed varies from 250 m in eastern part to 210 m to the western part. The Palam drain has three tributaries. The first one called Nasirpur drain flowing from east to west, second one, Palam link drain flowing from east to west of the catchment and third Palam pond link drain also flowing east to west. The total length of the main channel and tributaries of Palam drainage basin is about 35 km. The drainage density of the basin is about 0.6 km/sq.km. The basin has uniform slope towards the channel. The average slope of the basin is about 3 m in 1200 m.

4.3 SOILS

From the soil classification map, the basin soils are identified and classified as Darayapur series, Kakra series and Holambi series, with Holambi series occupying major area of watershed. In general the soil varied from sandy loam to silt loam. Holambi series comprises of well drained, very deep, loam to silt loam soils of yellowish brown to dark yellowish colour. They occur on land surface which are nearly level slope less than 1 %. The soils are silty in texture. The structure is sub granular blocky to angular blocky. The CEC is moderate with good fertility retention capacity. Available moisture holding capacity of soil is 9.0 mm for 60 cm depth and 15.7 cm for 100 cm profile depth from surface. Soils are well drained with moderate permeability. The soil map of Delhi state has been prepared by the Regional Centre of All India land use and soil survey department and is given in Fig 2.

Salient Details of the Palam Urban Watershed

S.No.	PARTICULARS	UNIT
1.	Catchment area	58 Sq.Km.
2.	Shape of the basin	Fan
3.	Name of the drain	Palam
4.	Names of the tributories	1) Nasirpur drain 2) Palam link drain 3) Palam pond drain
5.	Total length of Main and Distributories	35 Km.

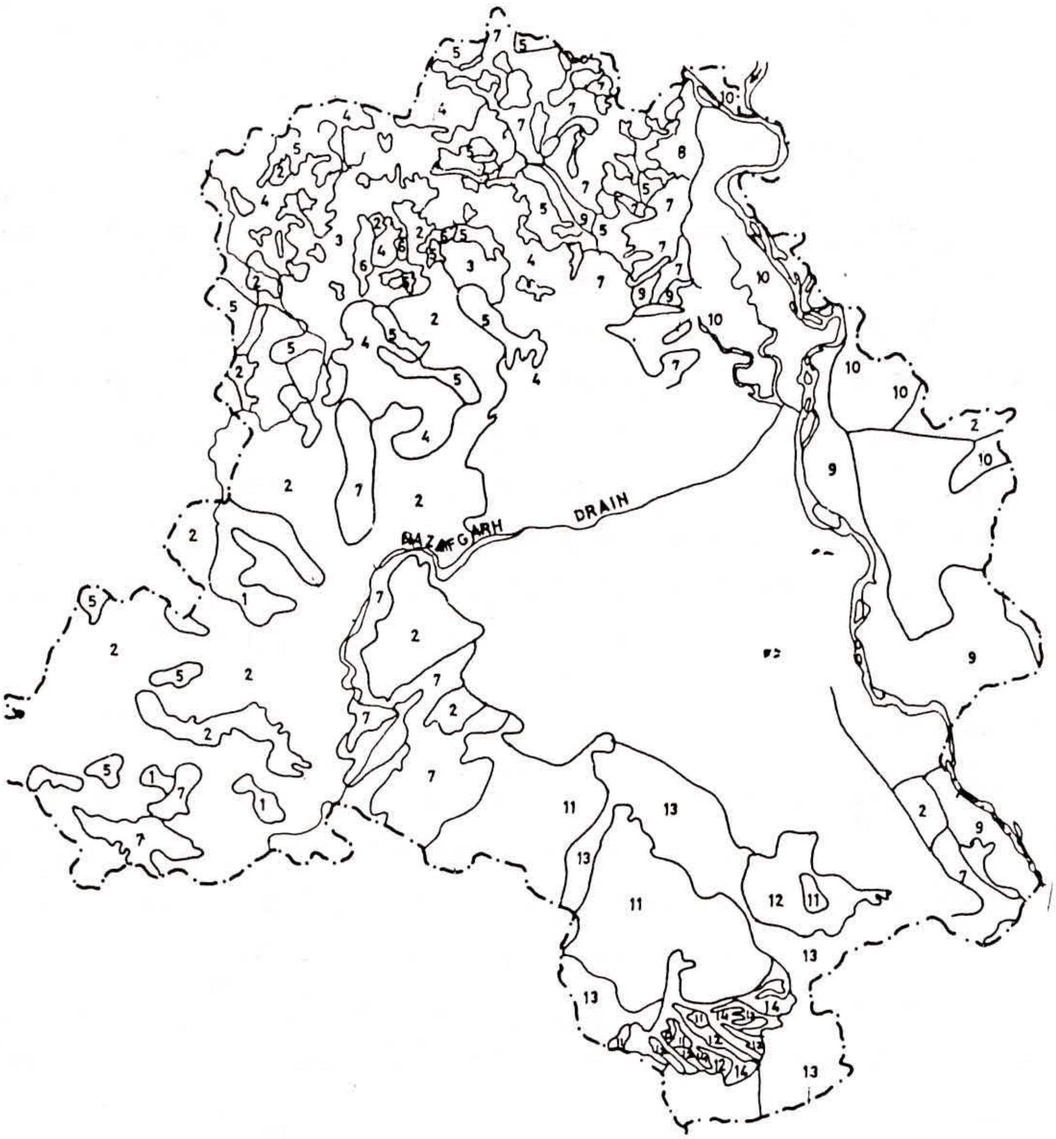


FIG.2-SOIL MAP OF DELHI TERRITORY

REFERENCE
State Boundary
Soil Boundary

6.	Length of Main channel	12 Km.
7.	Drainage density	0.6 Km/Sq.Km.
8.	Land Slope	2.5 m/Km.
9.	Attitude	250 M-210 M
10.	Type of Soil	Silt loam
11.	Land use	Arable
12.	Total Annual Rainfall	714 mm
13.	Climate	Sub humid

4.4 HYDROMETEOROLOGY

The basin belongs to sub humid region. The average annual rainfall is about 1100 mm. The watershed has a continental type of climate, cold in winter and very hot in summer. The basin receives most of the rainfall from south west monsoon. The monsoon rainfall constitutes nearly 80% of the total annual rainfall. The maximum and minimum temperature recorded varies between 45°C in the month of April and 2.5°C in the month of December. The average annual temperature is 26°C. The salient features of the Palam drain are given in Table 3.1.

4.5 DATA USED

The topographic details of Palam urban watershed are collected from the Survey of India's topographic sheet No. 53H/2 surveyed during 1969-1970 and 53H/2/NW, 53H/2/5W 1975-1976 respectively. From the topographic sheets. The data of Palam urban watershed boundaries have been delineated. Stream and drainage channels and the ground level contours at 5 m intervals have been prepared.

The soil classification details of the study area are obtained from the soil map of India published (1972) by All India Soil and Land Use Survey, IARI, New Delhi (Fig 2). The steady state infiltration rate of soil and other infiltration parameters were obtained from the double infiltrometer tests conducted in the study area. Since infiltration is a very important and sensitive parameter it should be determined accurately by infiltration tests. These tests should be performed before and after the first storm at suitable number of places as per the soil series existing in the study area. If observed runoff data is available for the catchment under study, infiltration can be obtained from rainfall runoff relationship. The initial relative moisture content and maximum moisture content under imbibition are also very sensitive parameters. These should be determined accurately either insitu or in the laboratory. The initial volumetric relative moisture content is determined by bringing disturbed and undisturbed soil samples from various locations of catchment after the first rain and determining their moisture content and bulk density. The product of these two gives the volumetric moisture content of soil. The other parameters such as ponding time parameter and infiltration scaling parameters can be obtained from published literature for the types of soil present in the study area. The final values of these parameters can be selected by making a sensitive analysis. If observed rainfall runoff data is available then these parameters should be calibrated. The mannings roughness parameters were obtained from selecting the appropriate values from published literature and field condition of area under study, these values are given in table 5.

4.6 WATERSHED GEOMETRY

In the simplest way a watershed is modeled as a network of channel and inter channel areas of overland flow. Each inter channel (including upland) is modeled as a plane or as a cascade of planes. Each plane is characterised by an area, length, width, slope and roughness coefficient. The length of the plane is decided by considering the uniformity of the parameters such as width, slope and roughness coefficient. A watershed should be discretised to sufficient number of elements to preserve their variability. The geometric shape of the plane element is considered to be rectangular. In the computation of watershed area, a channel is assumed to have negligible width. Channels are assumed to have trapezoidal cross-section. The trapezoidal shape can also be used to simulate flows in channels with rectangular or triangular cross sections. Each channel element is characterised by its length, slope, shape and roughness coefficients (tab 5).

The shape parameters define the cross section of a channel for trapezoidal cross section. The shape parameters are bed width and side slopes.

The study area was divided in 31 cascades of planes and channels. The total area contributing to a drain was obtained from the toposheet. This area was then transformed into a rectangle of same area and according to the land use the area was assigned a weighted infiltration rate and other overland flow parameters. The arrangement of planes and channels for year 1970 and 1975 is given in Fig 3. The model geometry for study area when total area is connected is given in tab 1 & 3 respectively for year 1970 and 1975. The model geometry for same years when only impervious area is connected is given in tab 2 & 4 respectively (Fig 4 & 5).

The validity of the program was tested for the watershed of W-I watershed Colorodo, USA. The simulated runoff was found to be comparable with observed data. After testing the model, the Palam watershed runoff was simulated for a rainfall event of hourly max. for two years RI.

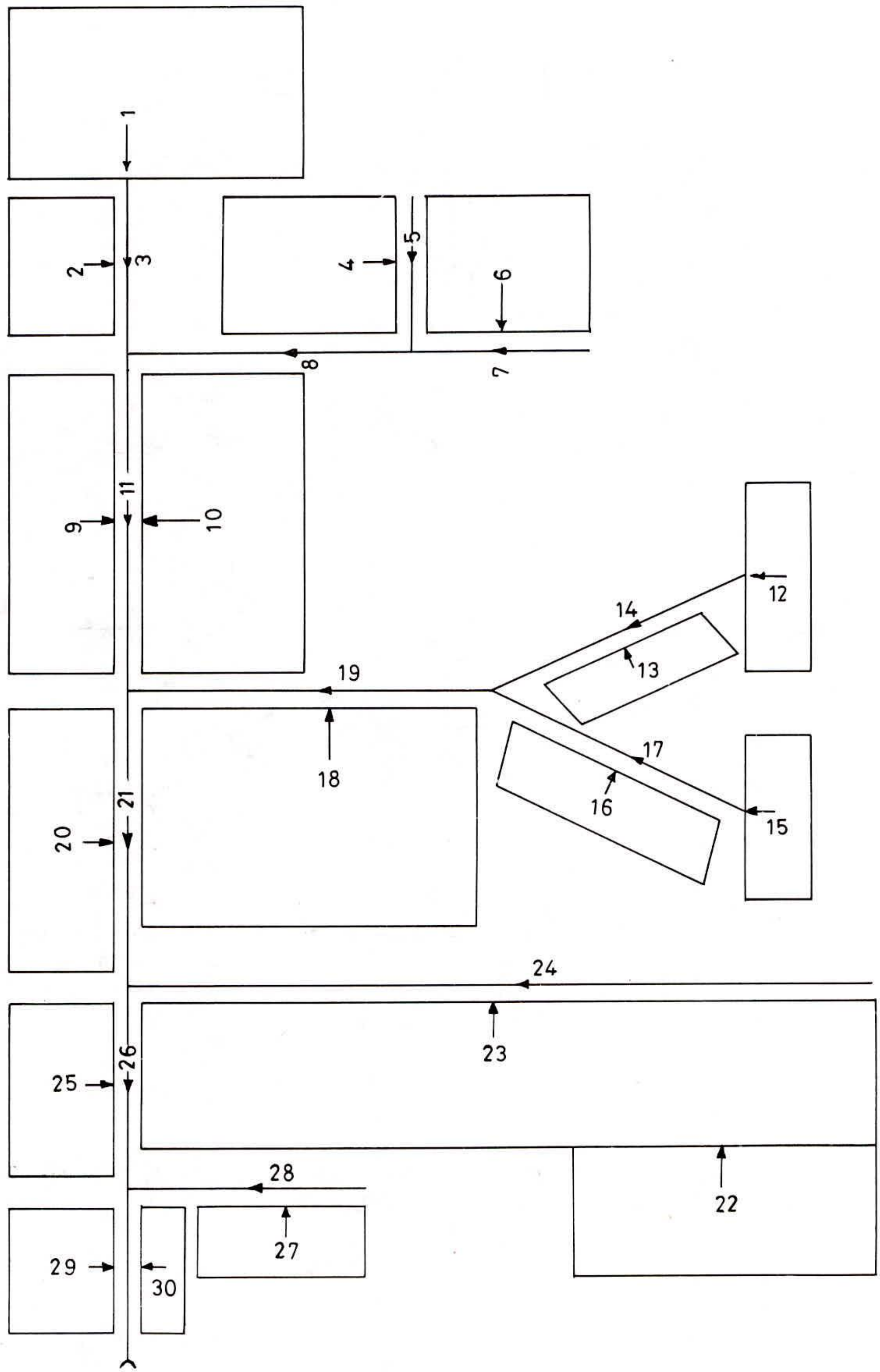


FIG. 3 MODEL GEOMETRY OF PALAM BASIN.

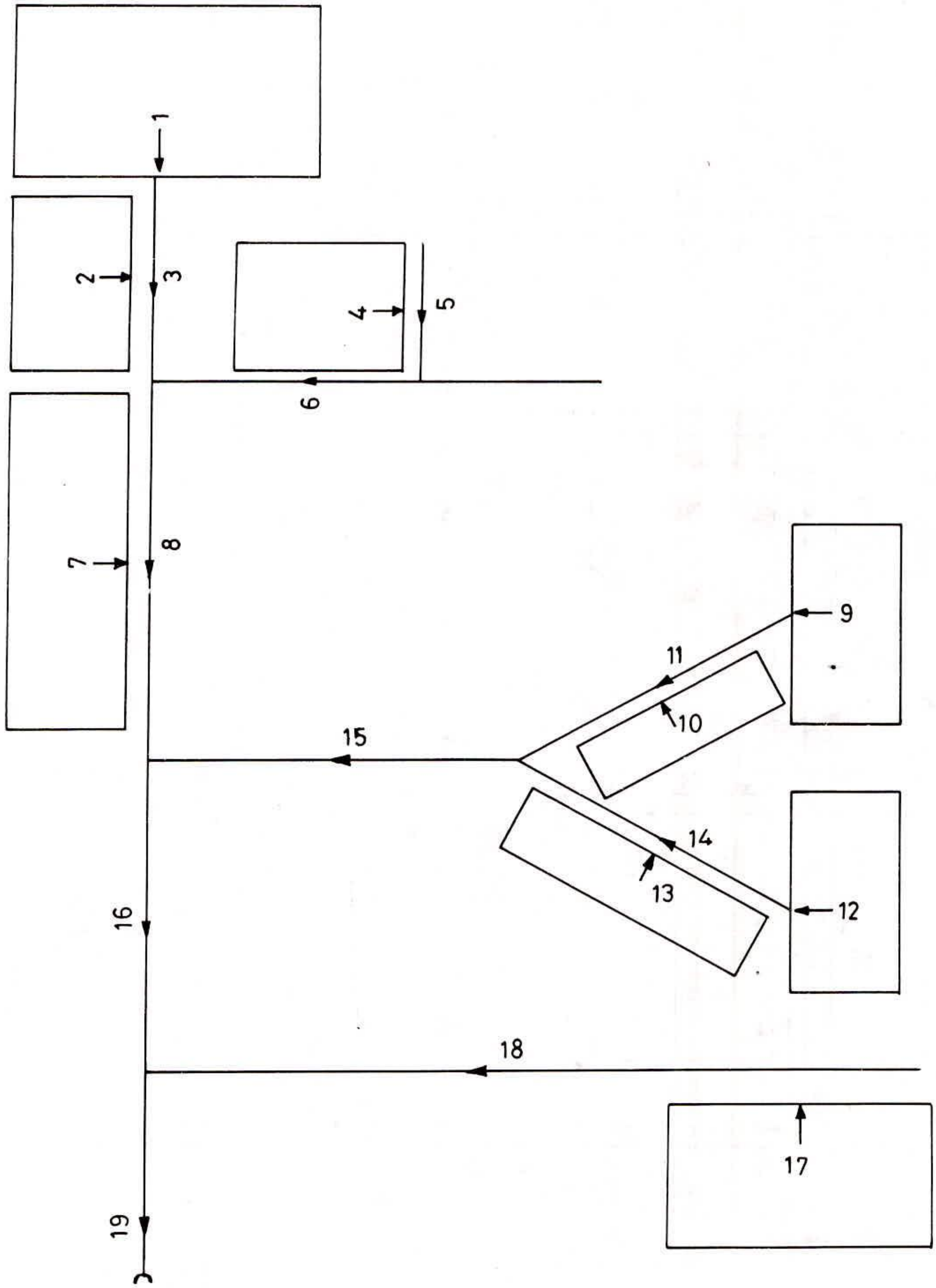


FIG. 4 MODEL GEOMERTRY FOR YEAR 1970 WHEN ONLY IMPERVIOUS AREA CONNECTED

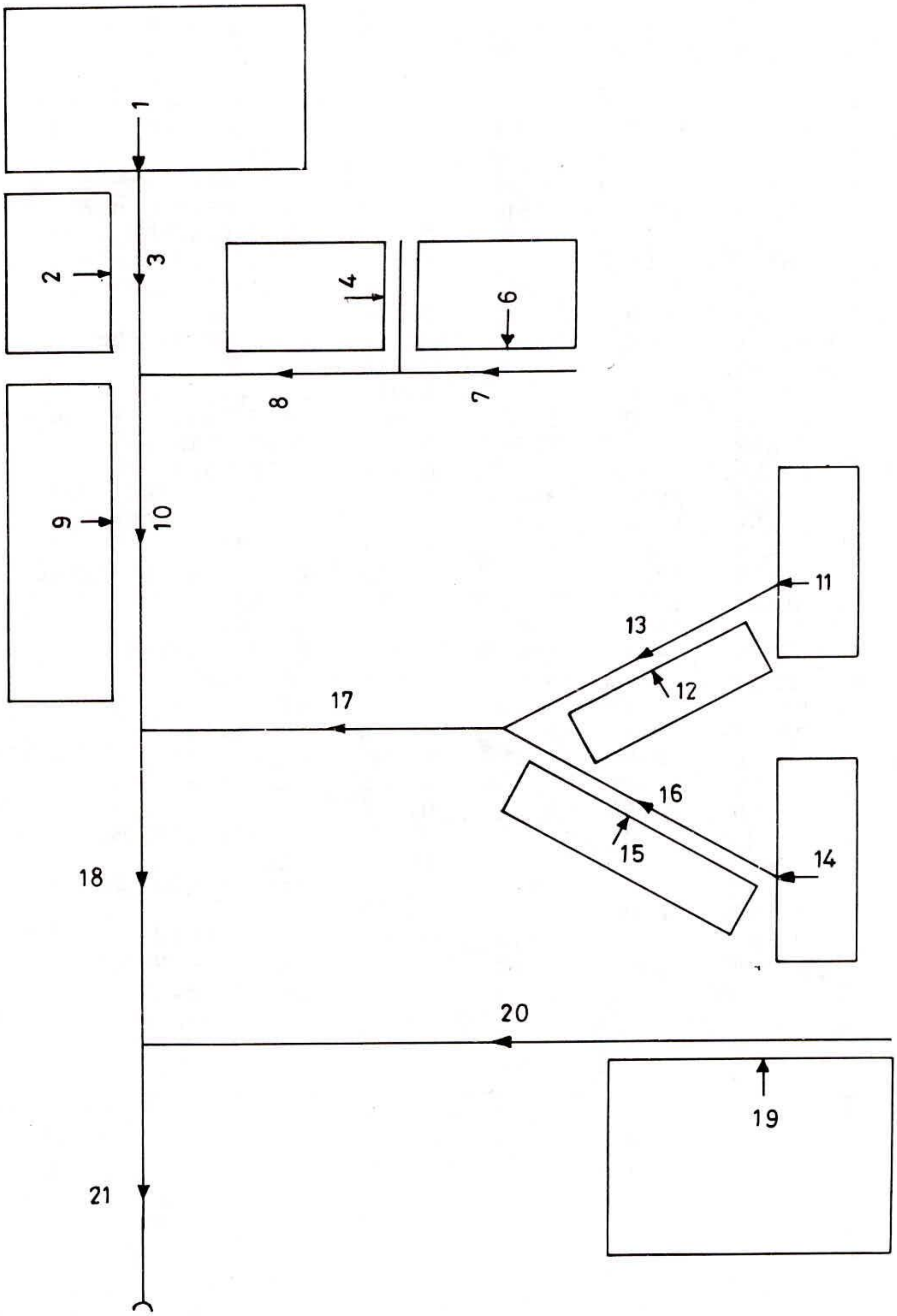


FIG. 5 MODEL GEOMETRY FOR YEAR 1975 (ONLY IMPERVIOUS AREA CONNECTED)

Tab.1 MODEL GEOMETRY OF PALAM URBAN CATCHMENT 1970
TOTAL AREA CONNECTED

SL. NO.	TYPE	LENGTH IN FT	WIDTH IN FT	SLOPE	LATERAL FLOW FROM
1.	PLANE	9590	13860	0.009	RAIN (SEMI IMPERVIOUS 0.2)
2.	PLANE	7260	4620	0.009	RAIN (IMPERVIOUS)
3.	CHANNEL	4620	0	0.018	1,2
4.	PLANE	2725	6600	0.008	RAIN (IMPERVIOUS)
5.	CHANNEL	6600	0	0.018	4
6.	PLANE	2310	7920	0.008	RAIN (PERVIOUS)
7.	CHANNEL	7920	0	0.018	6
8.	CHANNEL	1980	0	0.018	5,7
9.	PLANE	1980	23760	0.004	RAIN (SEMI PERVIOUS 0.2)
10.	PLANE	2980	23760	0.006	RAIN (PERVIOUS)
11.	CHANNEL	23760	0	0.018	9,10, 8,3
12.	PLANE	3960	3960	0.008	RAIN (IMPERVIOUS)
13.	PLANE	1320	5280	0.006	RAIN (IMPERVIOUS)
14.	CHANNEL	5280	0	0.018	12,13
15.	PLANE	3960	1980	0.008	RAIN (IMPERVIOUS)
16.	PLANE	1980	7920	0.006	RAIN (IMPERVIOUS)
17.	CHANNEL	7920	0	0.018	15,16
18.	PLANE	3960	10560	0.006	RAIN (PERVIOUS)
19.	CHANNEL	10560	0	0.018	18,14,17
20.	PLANE	4620	4290	0.004	RAIN (PERVIOUS)
21.	CHANNEL	4290	0	0.018	20,11,19
22.	PLANE	7600	11550	0.006	RAIN (SEMI PERVIOUS 0.1)
23.	PLANE	1980	23760	0.006	22, RAIN (PERVIOUS)
24.	CHANNEL	23760	0	0.018	23
25.	PLANE	5280	3300	0.004	RAIN (PERVIOUS)
26.	CHANNEL	3300	0	0.018	25,21,24
27.	PLANE	3300	5280	0.006	RAIN (PERVIOUS)
28.	CHANNEL	6600	0	0.018	27
29.	PLANE	3640	5940	0.002	RAIN (PERVIOUS)
30.	PLANE	3320	2640	0.002	RAIN (PERVIOUS)
31.	CHANNEL	5940	0	0.018	30,29,26,28

Tab.2 MODEL GEOMETRY OF PALAM URBAN CATCHMENT 1970
ONLY IMPERVIOUS AREA CONNECTED

SL. NO.	TYPE	LENGTH IN FT	WIDTH IN FT	SLOPE	LATERAL FLOW FROM
1.	PLANE	9590	1386	0.009	RAIN (IMPERVIOUS)
2.	PLANE	7260	4620	0.009	RAIN (IMPERVIOUS)
3.	CHANNEL	4620	0	0.018	1,2
4.	PLANE	2725	6600	0.008	RAIN (IMPERVIOUS)
5.	CHANNEL	6600	0	0.018	4
6.	CHANNEL	1980	0	0.018	5
7.	PLANE	1980	5940	0.004	RAIN (IMPERVIOUS)
8.	CHANNEL	23760	0	0.018	7,3,6
9.	PLANE	3960	3960	0.008	RAIN (IMPERVIOUS)
10.	PLANE	1320	5280	0.006	RAIN (IMPERVIOUS)
11.	CHANNEL	5280	0	0.018	9,10
12.	PLANE	3960	1980	0.008	RAIN (IMPERVIOUS)
13.	PLANE	1980	7920	0.006	RAIN (IMPERVIOUS)
14.	CHANNEL	7920	0	0.018	12,13
15.	CHANNEL	10560	0	0.018	11,14
16.	CHANNEL	4290	0	0.018	8,15
17.	PLANE	9580	1155	0.006	RAIN (IMPERVIOUS)
18.	CHANNEL	23760	0	0.018	17
19.	CHANNEL	9240	0	0.018	16,18

Tab.3 MODEL GEOMETRY OF PALAM URBAN CATCHMENT 1975
TOTAL AREA CONNECTED

SL. NO.	TYPE	LENGTH IN FT	WIDTH IN FT	SLOPE	LATERAL FLOW FROM
1.	PLANE	9590	13860	0.009	RAIN (SEMI IMPERVIOUS 0.15)
2.	PLANE	7260	4620	0.009	RAIN (IMPERVIOUS)
3.	CHANNEL	4620	0	0.018	1,2
4.	PLANE	2725	6600	0.008	RAIN (IMPERVIOUS)
5.	CHANNEL	6600	0	0.018	4
6.	PLANE	2310	7920	0.008	RAIN (IMPERVIOUS)
7.	CHANNEL	7920	0	0.018	6
8.	CHANNEL	1980	0	0.018	5,7
9.	PLANE	1980	23760	0.004	RAIN (SEMI PERVIOUS 0.2)
10.	PLANE	2980	23760	0.006	RAIN (PARTLY PERVIOUS 0.2)
11.	CHANNEL	23760	0	0.018	9,10, 8,3
12.	PLANE	3960	3960	0.008	RAIN (IMPERVIOUS)
13.	PLANE	1320	5280	0.006	RAIN (IMPERVIOUS)
14.	CHANNEL	5280	0	0.018	12,13
15.	PLANE	3960	1980	0.008	RAIN (IMPERVIOUS)
16.	PLANE	1980	7920	0.006	RAIN (IMPERVIOUS)
17.	CHANNEL	7920	0	0.018	15,16
18.	PLANE	3960	10560	0.006	RAIN (PERVIOUS)
19.	CHANNEL	10560	0	0.018	18,14,17
20.	PLANE	4620	4290	0.004	RAIN (PERVIOUS)
21.	CHANNEL	4290	0	0.018	20,11,19
22.	PLANE	7600	11550	0.006	RAIN (PARTLY IMPERVIOUS 0.15)
23.	PLANE	1980	23760	0.006	22, RAIN (PERVIOUS)
24.	CHANNEL	23760	0	0.018	23
25.	PLANE	5280	3300	0.004	RAIN (PERVIOUS)
26.	CHANNEL	3300	0	0.018	25,21,24
27.	PLANE	3300	5280	0.006	RAIN (PERVIOUS)
28.	CHANNEL	6600	0	0.018	27
29.	PLANE	3640	5940	0.002	RAIN (PERVIOUS)
30.	PLANE	3320	2640	0.002	RAIN (PERVIOUS)
31.	CHANNEL	5940	0	0.018	30,29,26,28

Tab.4 MODEL GEOMETRY OF PALAM URBAN CATCHMENT 1975
CNLY IMPERVIOUS AREA CONNECTED

SL. NO.	TYPE	LENGTH IN FT	WIDTH IN FT	SLOPE	LATERAL FLOW FROM
1.	PLANE	9590	4158	0.009	RAIN (IMPERVIOUS)
2.	PLANE	7260	4620	0.009	RAIN (IMPERVIOUS)
3.	CHANNEL	4620	0	0.018	1,2
4.	PLANE	2725	6600	0.008	RAIN (IMPERVIOUS)
5.	CHANNEL	6600	0	0.018	4
6.	PLANE	2310	7920	0.008	RAIN (IMPERVIOUS)
7.	CHANNEL	7920	0	0.018	6
8.	CHANNEL	1980	0	0.018	5,7
9.	PLANE	1980	6534	0.004	RAIN (IMPERVIOUS)
10.	CHANNEL	23760	0	0.018	9,8,3
11.	PLANE	3960	3960	0.008	RAIN (IMPERVIOUS)
12.	PLANE	1320	5280	0.006	RAIN (IMPERVIOUS)
13.	CHANNEL	5280	0	0.018	11,12
14.	PLANE	3960	1980	0.008	RAIN (IMPERVIOUS)
15.	PLANE	1980	7920	0.006	RAIN (IMPERVIOUS)
16.	CHANNEL	7920	0	0.018	14,15
17.	CHANNEL	10560	0	0.018	13,16
18.	CHANNEL	4290	0	0.018	10,17
19.	PLANE	9580	1270	0.006	RAIN (IMPERVIOUS)
20.	CHANNEL	23760	0	0.018	19
21.	CHANNEL	9240	0	0.018	19,20

Tab 5 Mannings roughness coefficients

	IMPERVIOUS (ASPHALT)	PERVIOUS (SPARSE VEG.)	CHANNEL
R1	0.013	0.053	0.015 (BRICK LINED)* 0.025 (GRASS WEEDS PRESENT)**
R2	108	2000	-

SOURCE:* OPEN CHANNEL HYDRAULICS BY CHOW
** FLOW IN OPEN CHANNELS SUBRAMANYA

Tab 6 Run off depth using SCS METHOD

Type of land use	1970 Area	CN	1975 Area	CN
1. Urban district (poor cover)	11.27	77	15.6	68
2. Less urban Airport	3.4	49	3.4	49
3. Open area (Grass, cultivated Fields etc)	43.33	39	39	39
Total area	58		58	
Weighted CN		45		48
S [(25400/CN)-254] (cm)		31		27.5
Q $[(P-0.3S)^2/(P+0.7S)]$ (cm)		9.6		11.1
Q (From Model) (cm)		7.07		11.41

4.7 SENSITIVITY ANALYSIS

Sensitivity analysis of the various overland parameters was carried out to find out the behavior of each parameters. The parameters selected for the sensitivity analysis include ponding time parameter (B), infiltration scaling parameter (C), initial and maximum water content (SI & So) and mannings roughness parameters for plane and channel. The analysis was carried out by varying one parameter at a time within it's range of variability while keeping the remaining parameters constant. The response i.e. the affect on discharge for each set of data was obtained using the model. The values of these parameters taken for the base run area 0.58, 0.5, 873, 0.5, 0.85 0.053 and 0.025.

The initial volume of water content was varied from 0.3 to 0.8 (Fig 6) while keeping other parameters same as for base run. It was observed that as water content is increasing the corresponding runoff is also increasing. The maximum water content was varied from 0.6 to 0.95, refer Fig 6, and the corresponding decrease in the runoff is observed. From the figure it is seen that a small change in volume of water content is resulting in large change in the runoff, therefore these parameters are very sensitive and should be determined very accurately for the field condition. This could be done either by insitu measurement at sufficient locations in the catchment or by using the standard values for the type/ types of soil existing in the area.

The parameter C was varied from 100 to 3200. It was observed (Fig 7) that as C is increasing runoff is decreasing and after C is greater than 1100 runoff becomes almost constant. The response of ponding time parameter indicates that increase in parameter B reduces the runoff appreciably but after B crosses a value of 0.8 the runoff remains almost constant (Fig 7).

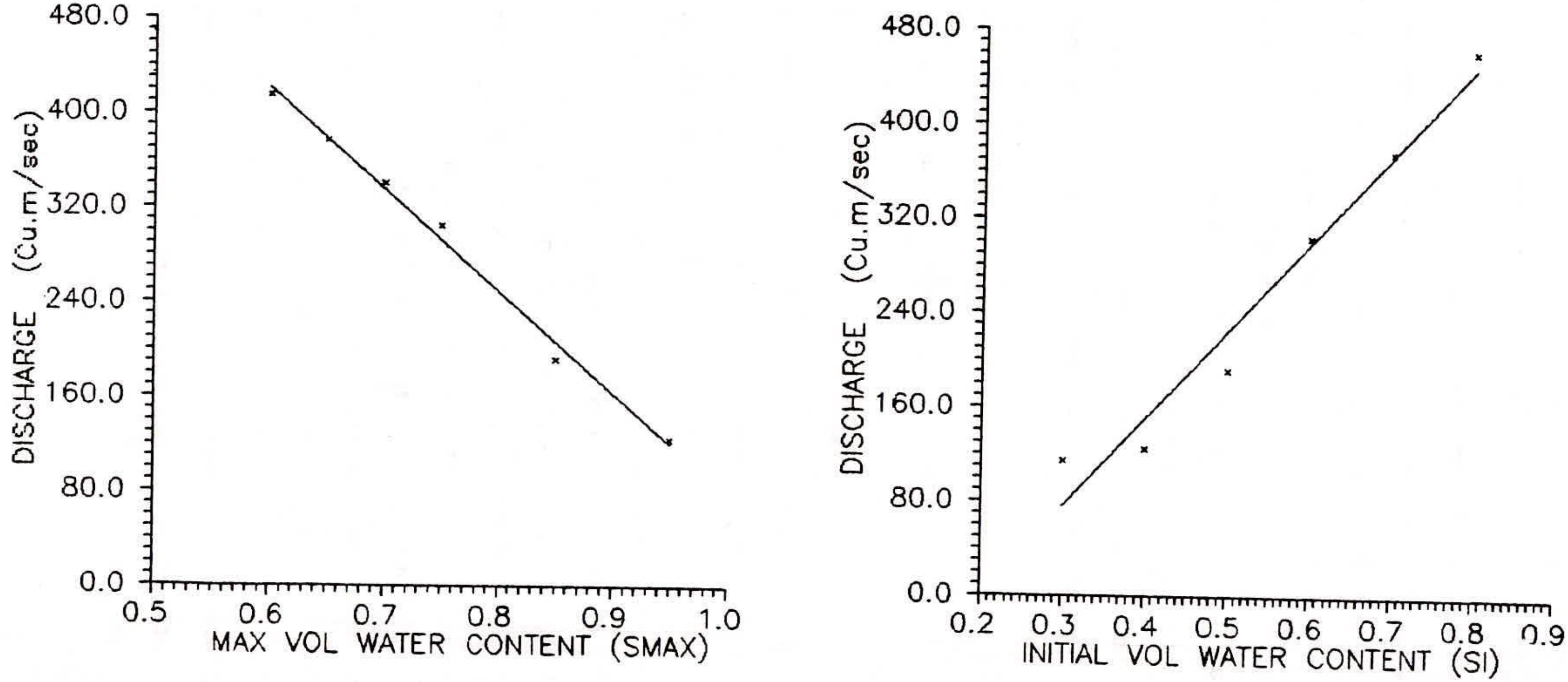


Fig. 6. Maximum and minimum relative water content

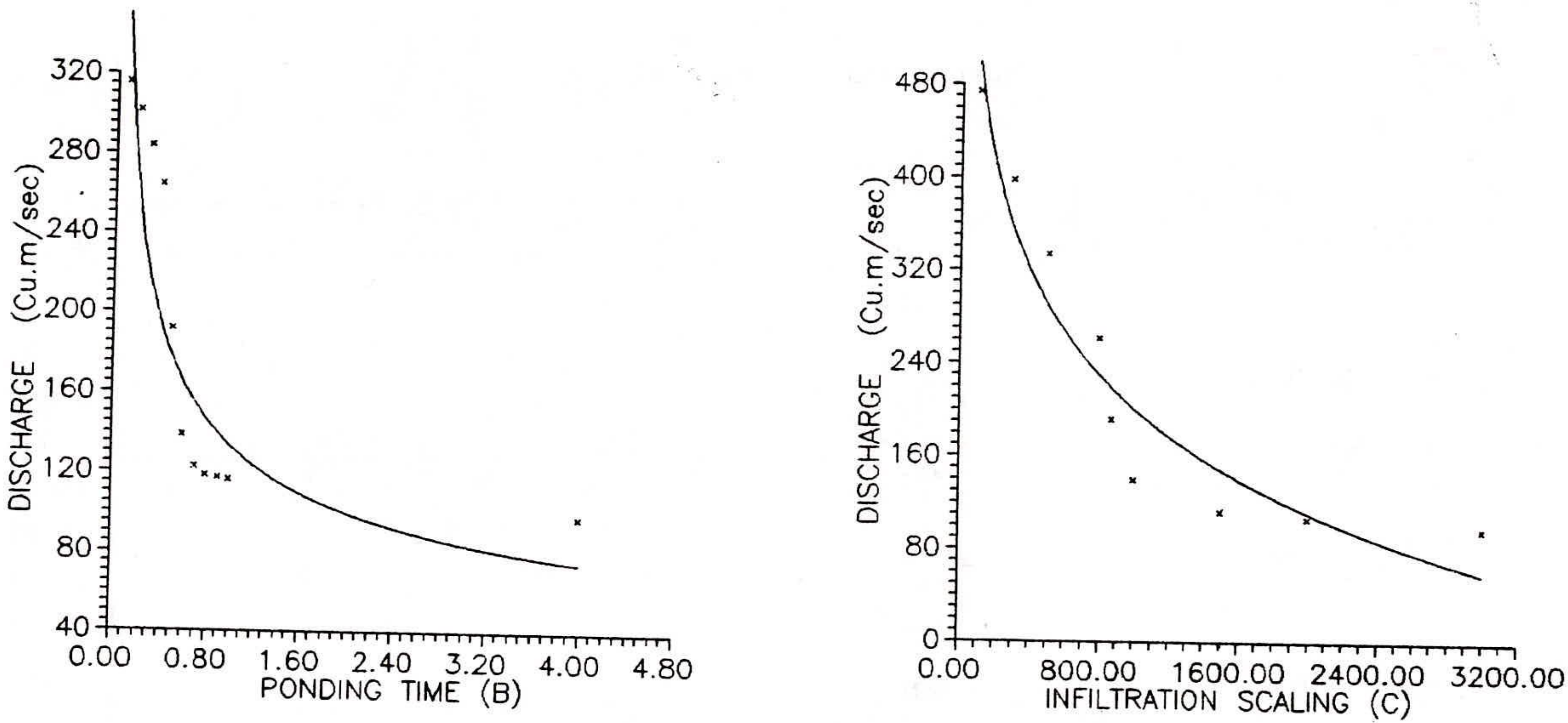


Fig. 7. Ponding time and infiltration scaling parameter

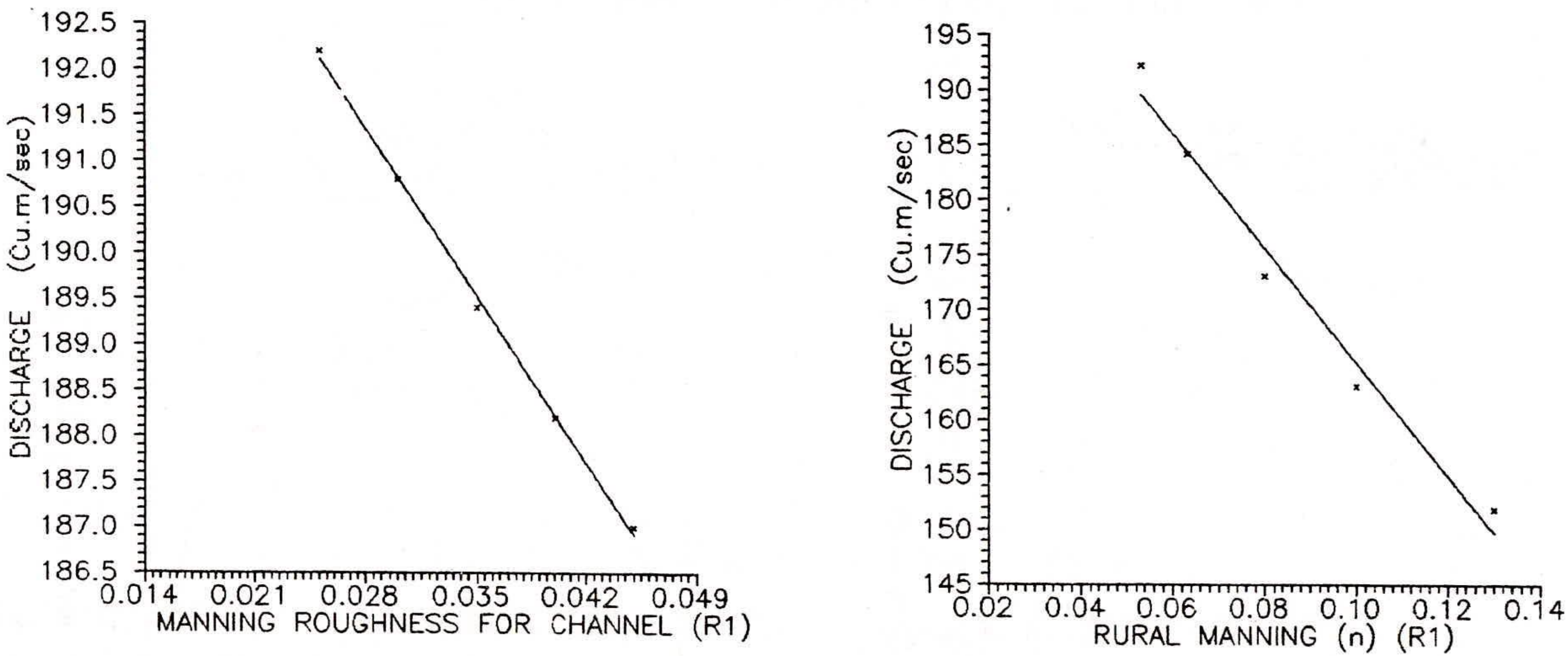


Fig. 8. Mannings roughness coefficient

The analysis of Mannings roughness coefficient for plane and channel (Fig 8) shows that as roughness coefficient is increasing discharge from the catchment is increasing.

5.0 RESULT AND DISCUSSION

Run off hydrographs have been simulated for two rainfall events (hourly maximum for two years and five years RI) (Fig 9). The point rainfall was converted in to areal average using Fig 10. The effect of change in impervious area on the peak of hydrograph was obtained by using toposheets for year 1970 and 1975 and finding out the discharges from the connected (impervious) areas and pervious areas. These hydrographs are shown in Figures from 11 to 14. The model parameters SI, C, and B have been obtained from the published literature for the silt loam soil (CSU paper 93). The sensitivity analysis of these parameters was also carried out and given in Fig 6-8.

The hydrograph given in Fig 11 for year 1970 indicates that three hydrograph peaks are visible. This is mainly due to the quick disposal of run off from impervious areas. These peaks are of 127 cumec, 115 cumec and 190 cumecs occurring at 2 3/4 hrs, 5 3/4 hrs and 10 1/4 hrs respectively from the total area of Palam catchment. For the same rainfall event for year 1970 the three flood peaks of 120 cumec, 123 cumec and 120 cumecs are occurring at 2 3/4 hrs, 5 3/4 hrs and 8 hrs respectively from the directly connected areas (impervious area). Fig 11 indicates that discharge is varying between 123 cumecs and 110 cumecs from 2 1/4 hrs to 10 2/3 hrs i.e. for almost 8 hrs and 25 minutes.

The flood hydrograph for the same rainfall event for year 1975 (Fig 8) indicates that only two distinct peaks are visible. These peaks are of 145 cumec and 243 cumec occurring at 2 3/4 hrs

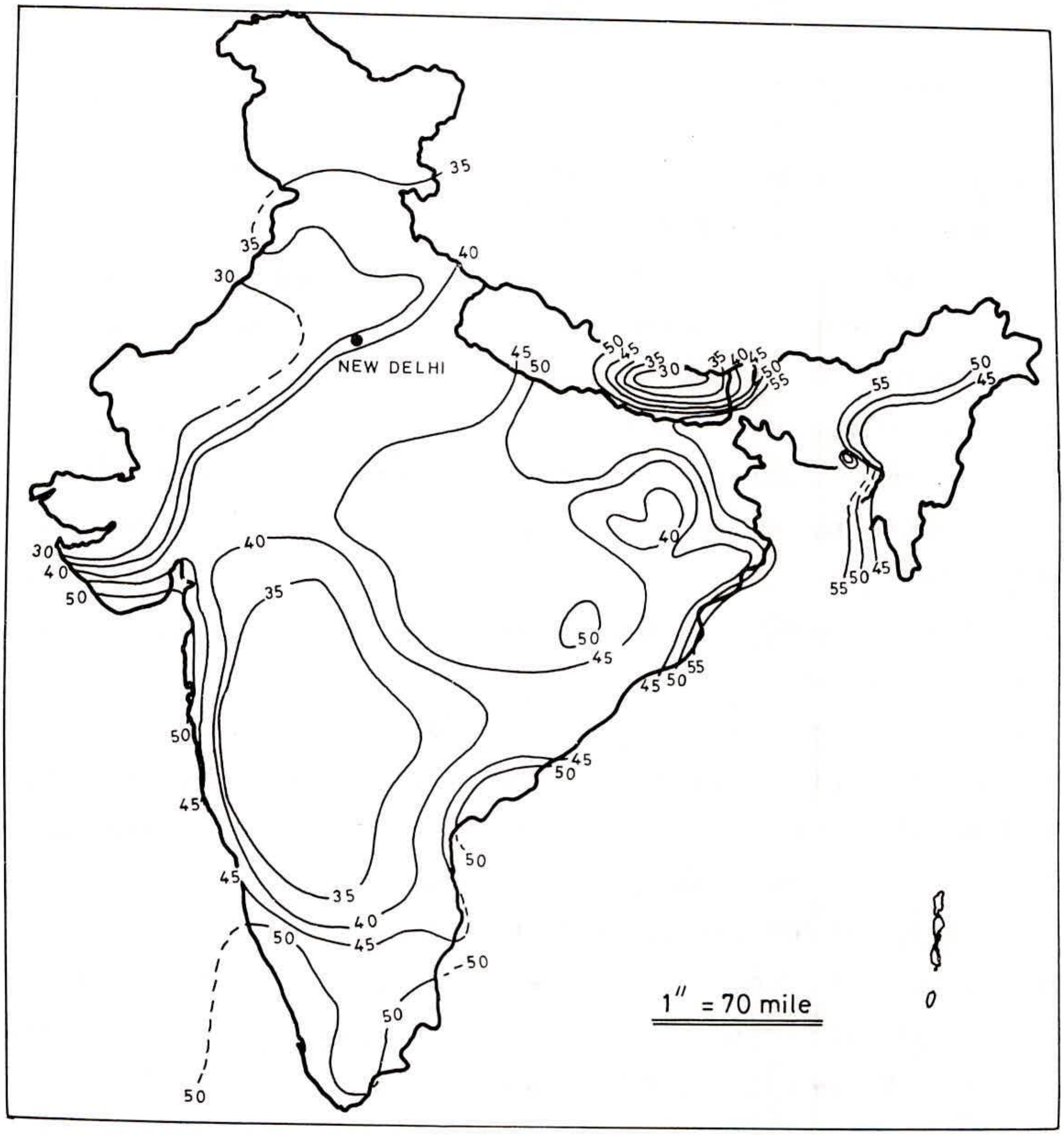


FIG. 9 - 2 YEAR - 1 HOUR MAXIMUM RAINFALL (mm)

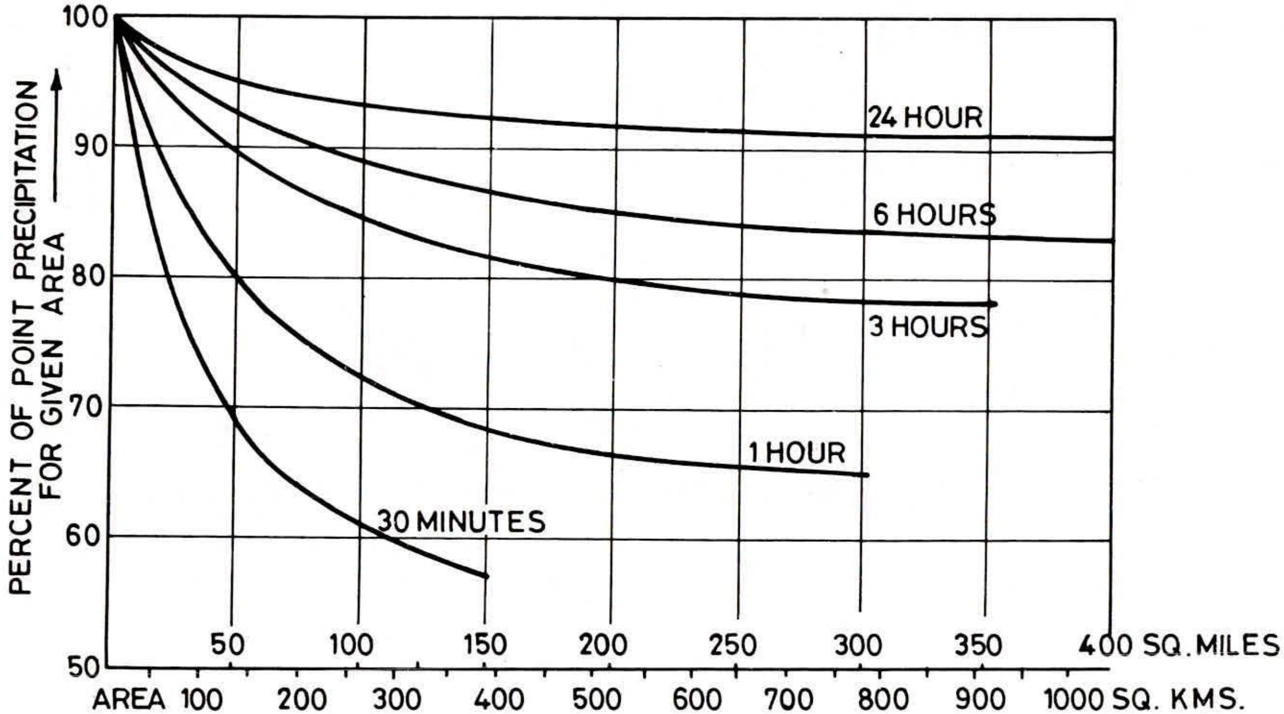


FIG. 10 – AREAL ANALYSIS GRAPH

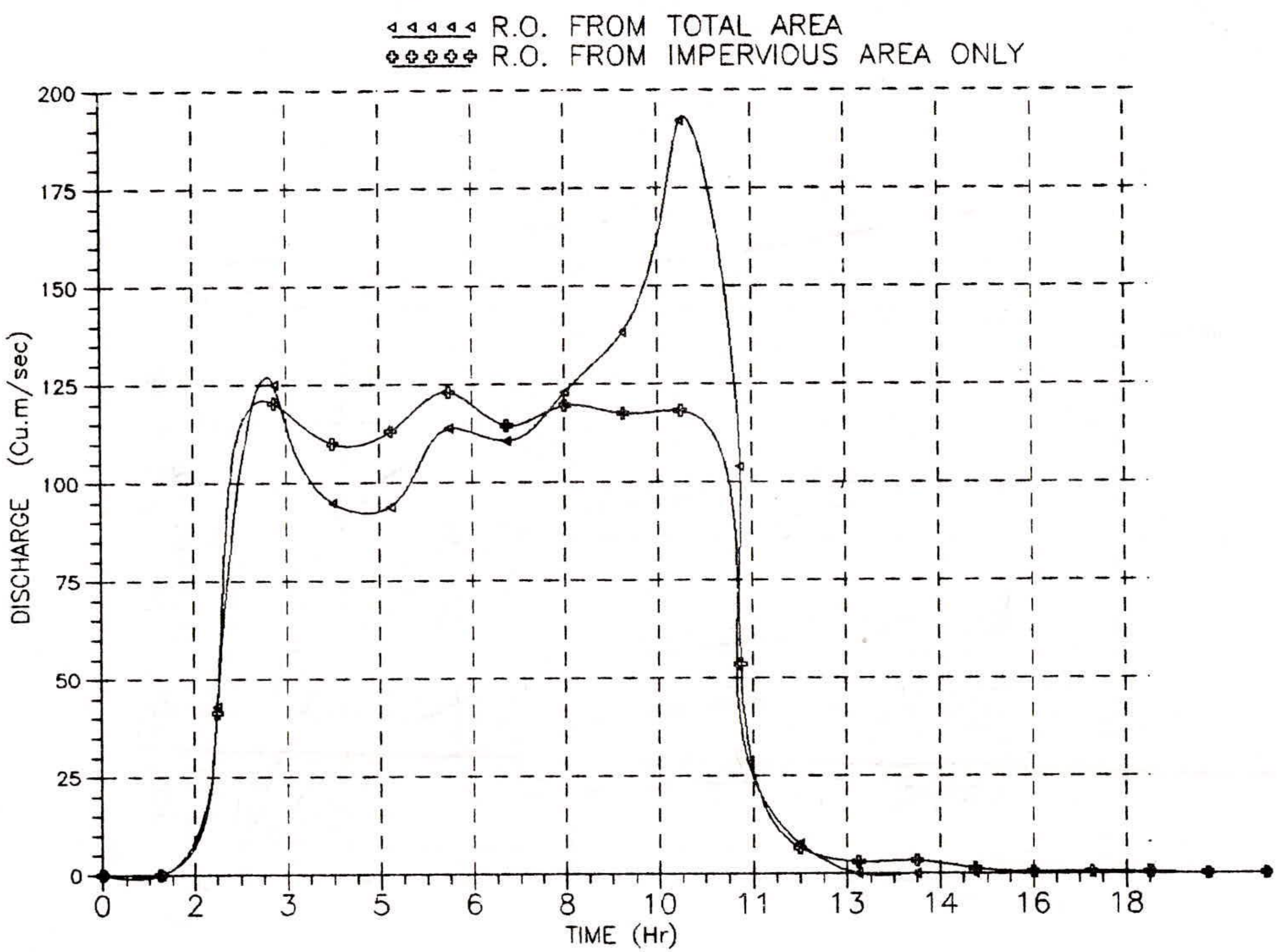


FIG.11-OVERLAND FLOW HYDROGRAPH FOR WATERSHED PALAM FOR YEAR 1970

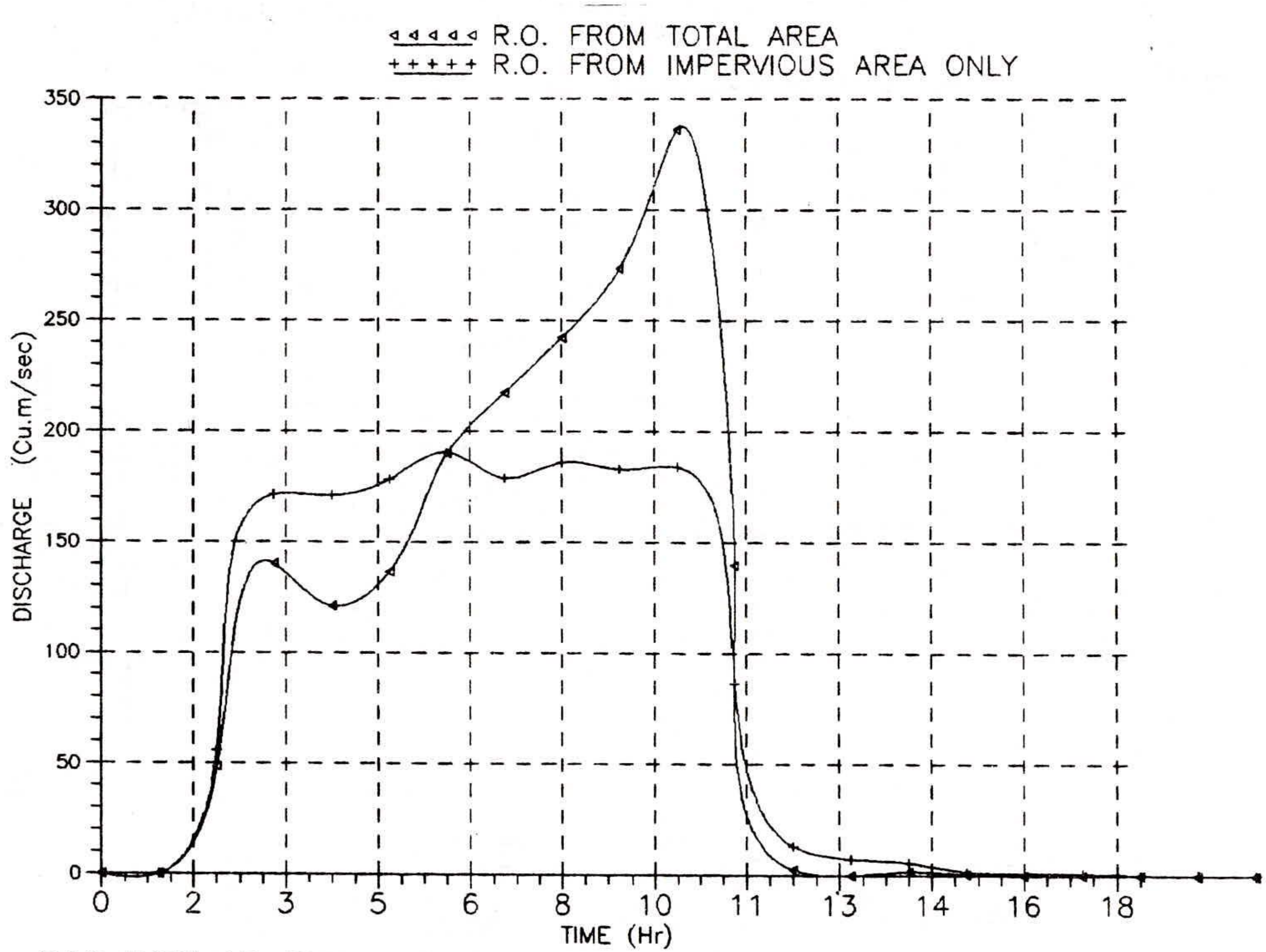


FIG.12-OVERLAND FLOW HYDROGRAPH FOR WATERSHED PALAM FOR YEAR 1975

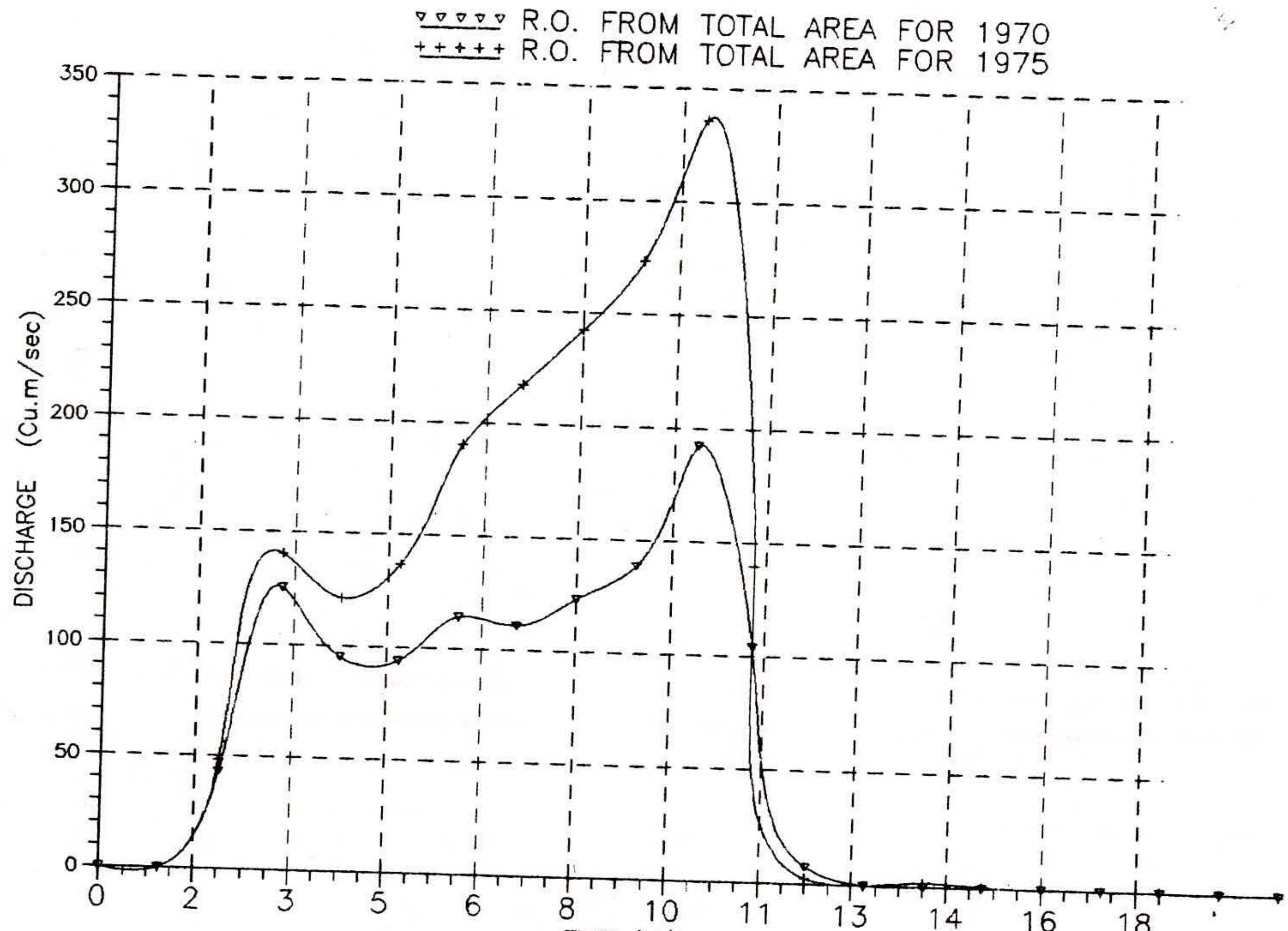


FIG.13-OVERLAND FLOW HYDROGRAPH FOR WATERSHED PALAM (TOTAL AREA CONTRIBUTING)

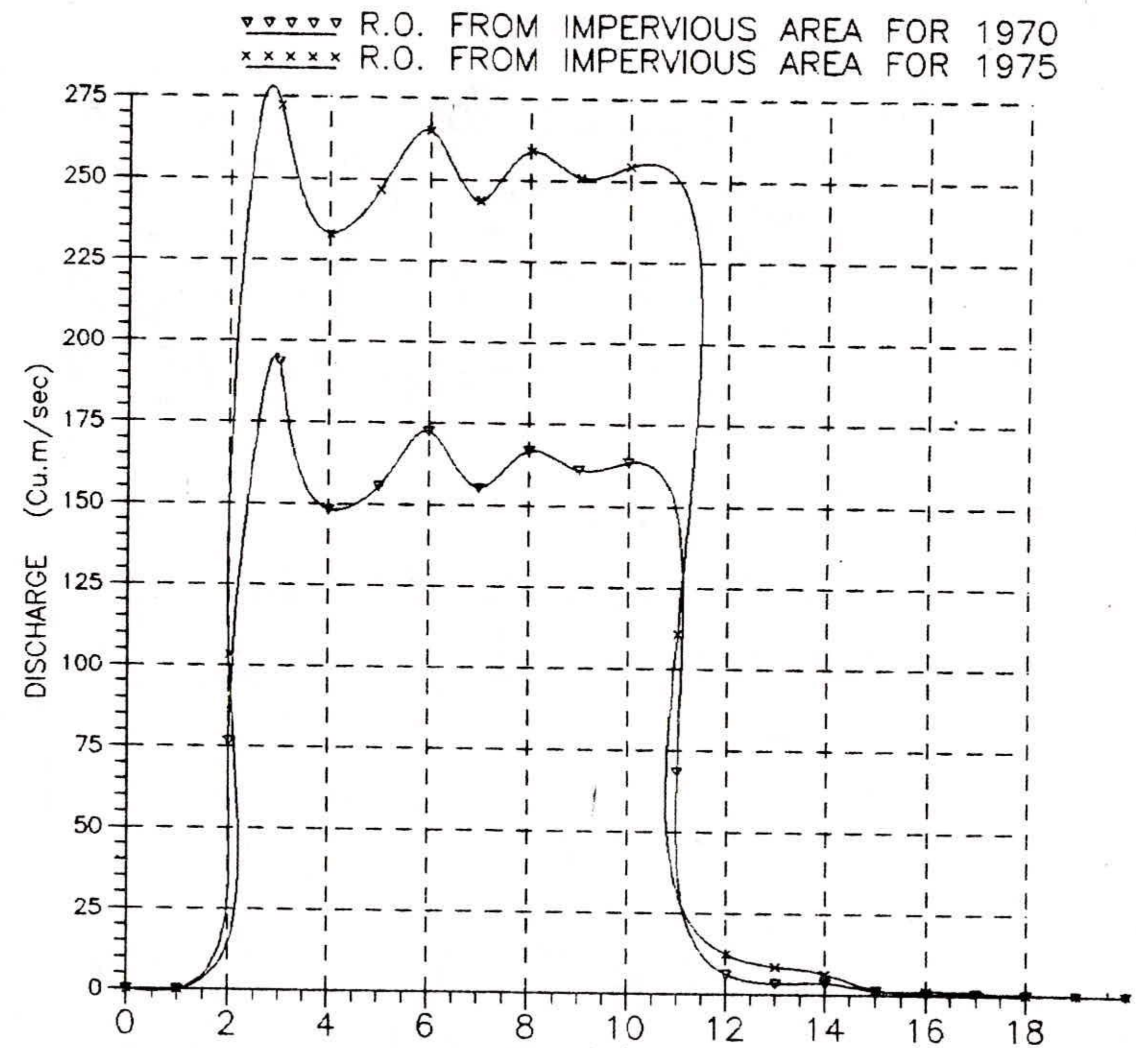


FIG14-OVERLAND FLOW HYDROGRAPH FOR WATERSHED PALAM
(ONLY IMPERVIOUS AREA CONTRIBUTING)

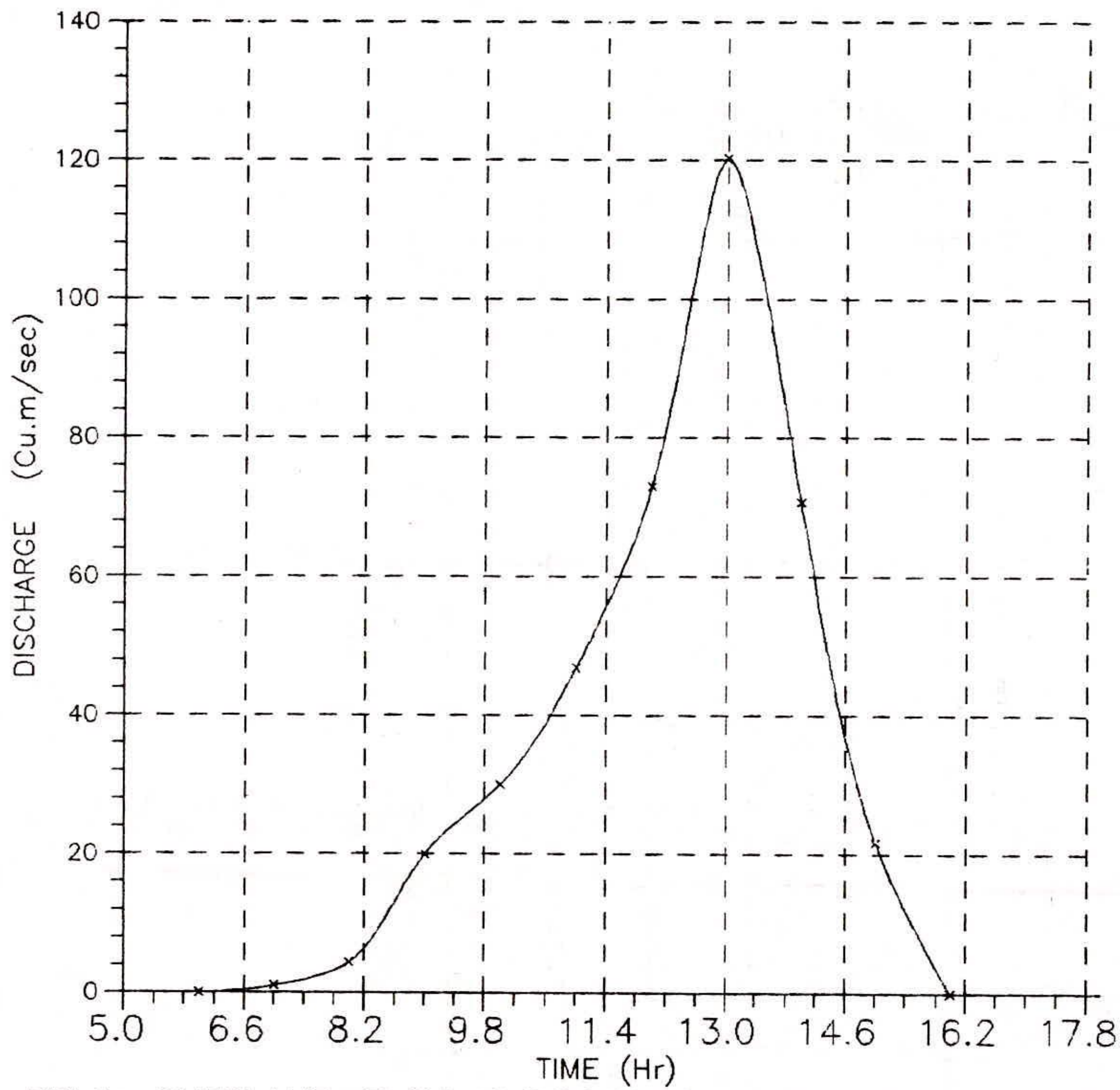


FIG.15-OVERLAND FLOW HYDROGRAPH FOR WATERSHED PALAM
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and 10 1/4 hrs respectively from the total area. However, a clear peak is obtained from directly connected area for same year of magnitude 195 cumecs occurring at 5 1/2 hr (Fig 12).

A comparison of peak discharge and time to peak for year 1970 and 1975 indicates that when entire palam catchment is contributing than the runoff peak has increased from 190 cumecs to 243 cumecs i.e. an increase of about 1.3 times but there is no change in the time to peak for the same period (Fig 13). However for the same rainfall event when only impervious areas are contributing to run off the run off peak has increased from 123 cumecs to 195 cumecs i.e. about 1.6 times and time to peak has decreased from 5 3/4 to 5 1/2 hr i.e. by about 15 minutes (Fig 14). Fig 15 gives the hydrograph from the same area when entire Catchment was assumed to be pervious and contributing to runoff. The runoff peak and time to peak for this situation are found to be 120 cu.m/sec and 13 hrs respectively.

The depth of runoff predicted by model was compared with the SCS method also and it is given in table 6. It is observed that for year 1970 the runoff depth given by both models are different, SCS the runoff depth is obtained as 9.6 cm whereas model is giving 7.07 cm however for year 1975 this difference is very less i.e. 11.1 cm and 11.41 cm. Therefore, the runoff depth predicted by both the models are matching satisfactorily.

6.0 CONCLUSION

From the study it can been inferred that as impervious area is increasing the run off peak is also increasing and at the same time time to peak is decreasing. The runoff is taking place at a faster pace from impervious areas and that is why more than one hydrograph peak are occurring. The total volume of runoff taking place from the urban area is about 80 % of total runoff taking place from the entire area (both pervious and impervious). The results of present study can not be used for design of existing drainage system as it is but it certainly indicates the problems of higher peaks to come in next 10 or 15 years.

7.0 SUGGESTION FOR FUTURE WORK

(1) This study should be taken up on a micro level for the same watershed.

(2) Different types of drains existing in the area should be identified and incorporated in the model.

(3) Using remote sensing imageries, spot data and CCT data the rate of change in the impervious areas should be found out for different years and the run off hydrographs should be simulated for those years.

(4) Results obtained with this model should be verified with actual runoff data for the same rainfall event if available.

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Director	Dr. S. M. Seth
Head	Dr. B. Soni

STUDY GROUP

M.K.Shukla	Scientist B
Documentation staff	Kiran Ahuja