TIME-AREA METHOD OF FLOOD ESTIMATION

FOR SMALL CATCHMENTS

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PREFACE

The purpose of the research reported here was to establish whether the simple time-area routing procedure is adequate for small catchment flood estimation. The results were affirmative and emphasized that improved means of estimating catchment parameters should be sought before any more complex routing procedure ought to be attempted.

The method is conceptually simple and promises to become a valuable design tool. It supplements the work reported in HRU Report 1/72 by providing a means of estimating flood hydrographs for catchments smaller than 15 km^2. Complex catchments can also readily be analysed and the estimation of rainfall losses is enhanced by using a deterministic approach which can be readily calibrated against short term rainfall/ runoff records.

D. C. Midgley 1 October 1981.

Director:
HYDROLOGICAL RESEARCH UNIT

ABSTRACT

The Time-Area Method of small catchment flood estimation is adapted for use on programmable calculators. Detailed algorithms are presented as well as programs for the Hewlett Packard HP-97 and HP-41C(V) calculators.

The technique is verified against 60 observed runoff events on 14 small catchments (8 urban and 6 rural). The maximum catchment size is 140 ha. Results are pleasing and warrant adoption of the method as a design tool.

Tentative recommendations are made for the estimation of design parameters.

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CHAPTER 1 INTRODUCTION

Flood estimation is a vital early step in the design of a wide range of civil engineering works. Techniques in common use, however, do not provide the user with a sound understanding of the rainfall/runoff process on which to base his design decisions. Most techniques are of the handbook type and do little to instill appreciation of the underlying principles and philosophies. This report aims to make good this deficiency.

In South Africa at present the Rational Method and Unit Hydrograph techniques are the most commonly used for estimating design floods - the Rational Method for peak discharge and the Unit Hydrograph Method for establishing the temporal distribution of runoff. The former has the advantage of ease of application and is therefore a valuable design tool. Unfortunately it has many inadequacies, the most important being the poor manner in which it accounts for rainfall losses.

Estimation of the runoff coefficient, C, is highly subjective and cannot readily be improved by analysing available rainfall/runoff data. The Unit Hydrograph Method though theoretically sounder is more cumbersome to apply and is limited by availability of the data needed for establishing unitgraphs. Application is therefore restricted to fairly large rural catchments for which regional unitgraphs may be available.

Computer modelling techniques have also recently been applied in South Africa and these go a long way towards facilitating appreciation of the runoff process. The U.S. Soil Conservation Service (SCS) technique has been used in a research project in Natal (Cousens and Burney, 1977) and has been strongly advocated by Schulze and Arnold (1979) for design application. Application of the Illinois Urban Drainage Area Simulator (ILLUDAS) to local urban catchments has been investigated by Watson (1981a) with promising results. The U.S. Environmental Protection Agency Stormwater Management Model (SWMM) has also been applied locally.

No reliable desktop technique is available, however, for estimating small catchment flood hydrographs. The present study expands on a technique that had largely fallen into disuse and demonstrates how it can be successfully applied to flood estimation, namely the time-area method. Several variations of the technique were in use in Britain during the inter-war period (Colyer and Pethick, 1976) but the method was discredited by Escritt (1977) on the grounds that it provided minimal improvement in the estimation of peak discharge and moreover involved excessive hand calculation. His criticism was valid in that use was then still made of the runoff coefficient concept for determining losses. The proposed method, however, considers losses as an abstraction from rainfall and embodies a loss rate that decays with time.

The main application of the time-area method at present is an overland flow sub-routine in digital runoff models, e.g. the Transport and Road Research Laboratory, TRLL, model (Watkins, 1962) and the Illinois Urban Drainage Area Simulator, ILLUDAS, (Terstriep and Stall, 1974). As a desktop technique its use seems to have declined.

With the widespread use of programmable calculators, however, the technique takes on a new light. It is of moderate complexity and easily adapted for use on programmable calculators. This report demonstrates the adaptation of the method to Hewlett Packard HP-97 and HP-41C programmable calculators and shows how it can be a convenient and reliable design tool. The method may easily be programmed on other calculators, even some of those with relatively small capacity. Detailed algorithms are presented to assist the user in adapting the technique to suit his own needs.

CHAPTER 2 DESCRIPTION OF THE METHOD

2.1 Overview

Overland flow is assumed to be the sole source of storm runoff. Surface losses are subtracted from rainfall to determine excess rain which is routed over the catchment without further loss. Heterogeneous catchment conditions are accounted for by dividing the catchment into homogeneous zones. Runoff from each zone is determined separately and the results combined at the outfall. Routing assumes flow velocities to be constant with time.

The steps in computing the hydrograph resulting from a given storm on a particular catchment follow:

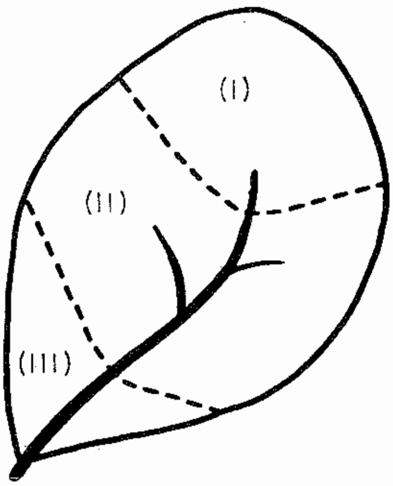
(i) Divide the catchment into zones considered to be subject to the same temporal distribution of excess rain

(ii) For each zone:
   (a) compute the temporal distribution of excess rain
   (b) determine the time-area diagram
   (c) route the excess rain through the time-area diagram to obtain the contributing hydrograph for the zone

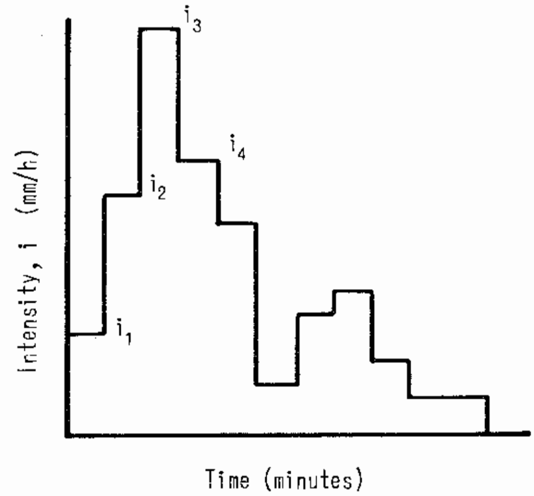
(iii) Add these hydrographs to obtain the outfall hydrograph for the total catchment.

The time-area diagram referred to is a convenient device for flow routing. It is a curve that represents the cumulative catchment area contributing flow to the outfall as a function of time.

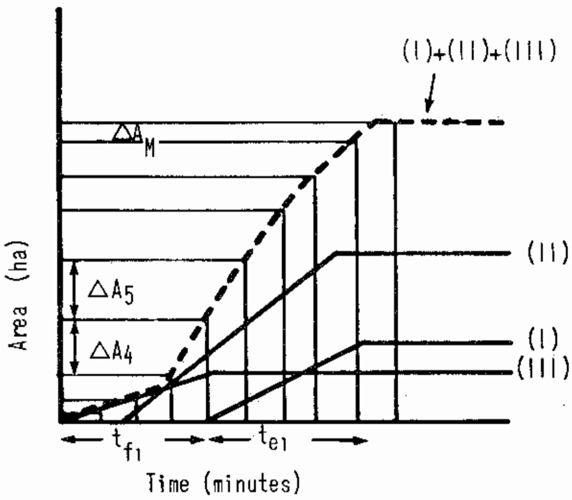
The basic steps in developing a hydrograph for a homogeneous catchment (or zone) are described in Fig. 2.1. The catchment shown in Fig. 2.1(a) is divided into subcatchments, each of which is assumed to have a linear increase with time of contributing area. The time taken for the total subcatchment area to contribute runoff to the adjacent reach is termed the entry-time, t_e. The subsequent travel time in the reach to the outfall is termed the flow-time, t_f. Each subcatchment time-area diagram



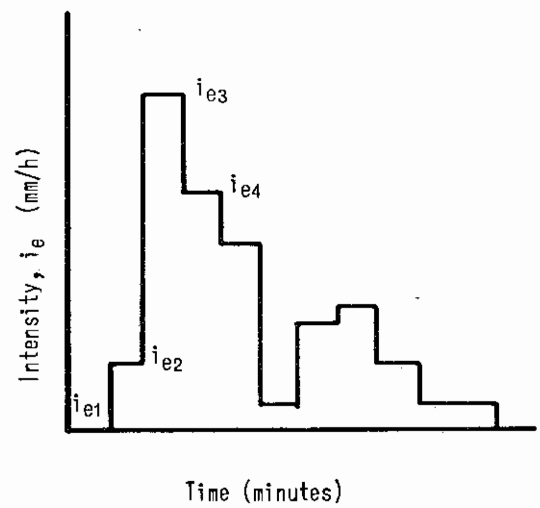
(a) Catchment



(b) Rainfall



(c) Time-area diagram



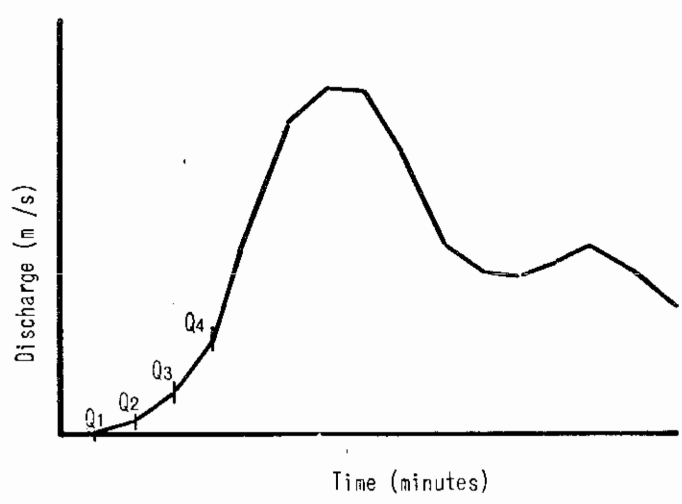
(d) Excess rainfall

$$Q_1 = \frac{1}{360}(i_{e1} \cdot \Delta A_1)$$

$$Q_2 = \frac{1}{360}(i_{e2} \cdot \Delta A_1 + i_{e1} \cdot \Delta A_2)$$

$$Q_3 = \frac{1}{360}(i_{e3} \cdot \Delta A_1 + i_{e2} \cdot \Delta A_2 + i_{e1} \cdot \Delta A_3)$$

$$Q_n = \frac{1}{360}(i_{en} \cdot \Delta A_1 + i_{en-1} \cdot \Delta A_2 + \cdots\cdots + i_{en-a} \cdot \Delta A_M)$$



(e) Computation of hydrograph

Fig. 2.1 Description of the method

is defined in terms of its area, entry time and flow time. The time-area diagram for the whole catchment is obtained by summating the subcatchment diagrams as illustrated in Fig. 2.1(c).

Excess rain (Fig. 2.1(d)) is obtained by subtracting losses from the hyetograph (Fig. 2.1(b)). Isochronal areas (ΔA_1, ΔA_2, ...) are determined from the time-area diagram and used to route the excess rain to the outfall of the catchment as described in Fig. 2.1(e).

The homogeneous zones within a catchment need not be geographically distinct, but can be very much intermingled. A typical example of this is in an urban catchment where paved and unpaved areas would be selected as distinct zones. In large catchments where consideration of spatial non-uniformity of rainfall becomes important, zones can be subdivided to create sub-areas with an average rainfall input. Considerations of accuracy, available data and computational effort will determine the degree of subdivision.

The following sections describe the various elements of the method in detail. Algorithms used in the calculator programs are also presented.

2.2 Infiltration

Infiltration is the loss to runoff through absorption of water by the soil. The rate of loss is governed by the availability of surface water and the capacity of the soil to absorb this water (i.e. its infiltration capacity). This is usually relatively large at the onset of rainfall and decreases to a nearly constant value as the ground becomes saturated.

Horton (1939) proposed an equation to describe the variation in infiltration capacity with time, viz:

$$f_{cap} = f_\infty + (f_o - f_\infty)\, e^{-kt} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.1)$$

where
f_{cap} = infiltration capacity (mm/h)
f_o = infiltration capacity at time t=0 (mm/h)
f_∞ = infiltration capacity at time $t = \infty$ (mm/h)
k = recession constant (h^{-1})
t = time (h)

The equation is based, however, on the limiting assumption that the available water is always equal to or greater than the infiltration capacity. If water is supplied at a lower rate than infiltration capacity eq. 2.1 will imply that infiltration capacity decreases too rapidly. This is illustrated in Fig. 2.2 in which infiltration capacity according to eq. 2.1 is shown as a solid line. As illustrated here, infiltration capacity decreases even when no water is absorbed by the soil, i.e. during periods of no rainfall. This is illogical as one should expect the infiltration capacity to decrease only with increasing wetness of the soil. A more reasonable distribution is shown by the dashed line, the shaded area below which represents the total infiltration. The periods of zero or low-intensity rainfall are assumed to be sufficiently short to render insignificant any recovery infiltration capacity.

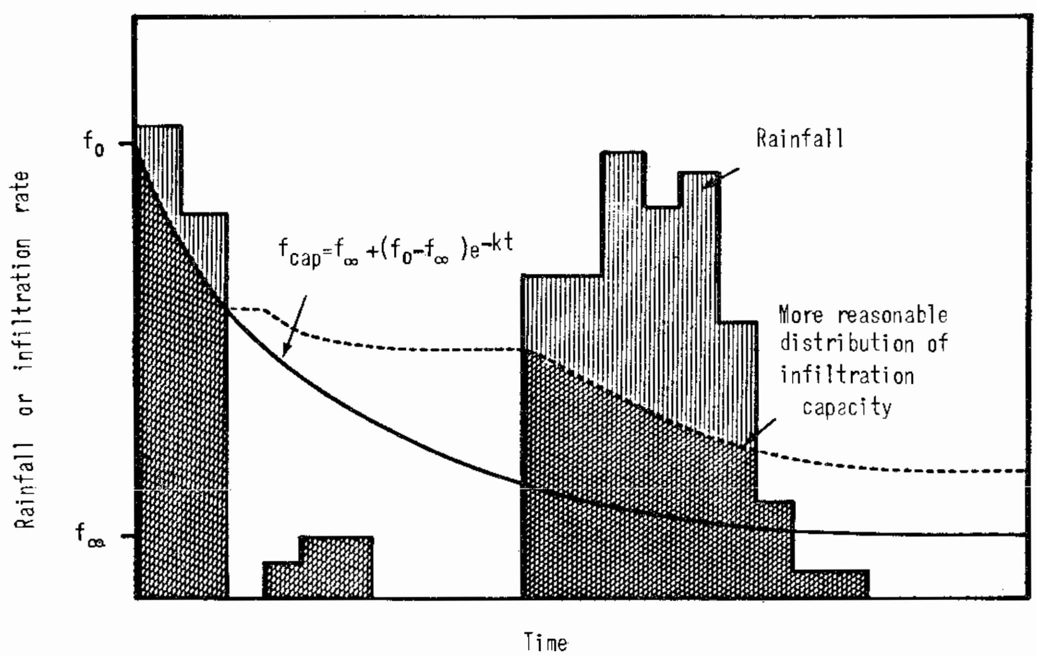


Fig. 2.2 Distribution of infiltration capacity with time

Horton's equation can be corrected to take account of this defect. By letting the accumulated depth of infiltration equal the integral of the infiltration capacity with respect to time, the effective time along the capacity curve can then be determined. A numerical solution is described by Huber et al. (1977) but is practicable only by digital computer since the solution for time is implicit. A similar method is described by Watson (1981a). Both techniques, however, are too time-consuming for efficient handling by a programmable calculator.

A simple explicit solution can be obtained as follows:

(i) split eq. 2.1 into two components, viz. a diminishing component and a constant component
(ii) assume infiltration rate to be constant over each computational time interval.

This assumption is not unreasonable as it is the same as that for the discretization of rainfall.

The two components are:

$$f_{dcap} = (f_o - f_\infty)e^{-kt} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.2)$$

and

$$f_{ccap} = f_\infty \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.3)$$

where the subscripts d and c represent the diminishing and the constant component respectively. Integrating eq. (2.2) to obtain the diminishing infiltration capacity in terms of incremental depth, ΔF_{dcap}, over the time interval, Δt, gives

$$\Delta F_{dcap} = \int_t^{t+\Delta t} (f_o - f_\infty)e^{-kt}$$

$$= \frac{1}{k}(f_o - f_\infty)(1 - e^{-k\Delta t})e^{-kt} \quad \ldots\ldots\ldots (2.4)$$

The accumulated diminishing infiltration capacity with respect to time is :

$$F_{dcap} = \int_o^t (f_o - f_\infty)\ e^{-kt}\ dt$$

$$= \frac{1}{k}(f_o - f_\infty)(1 - e^{-kt}) \quad \ldots\ldots\ldots\ldots (2.5)$$

from which

$$e^{-kt} = 1 - k\ F_{dcap}/(f_o - f_\infty) \quad \ldots\ldots\ldots\ldots\ldots (2.6)$$

Letting time t be adjusted such that the actual accumulated diminishing infiltration, F_d, is equal to the accumulated diminishing infiltration capacity, F_{dcap}, then substituting for e^{-kt} in eq. (2.4) gives

$$\Delta F_{dcap} = \frac{1}{k}(f_o - f_\infty)(1 - e^{-k\Delta t})(1 - \frac{kF_d}{f_o - f_\infty}) \quad \ldots\ldots\ldots\ldots (2.7)$$

This equation gives the infiltration capacity of the diminishing component for the next time increment. In order to obtain the increment of total infiltration capacity, ΔF_{cap}, we must add the constant component. Thus

$$\Delta F_{cap} = \frac{1}{k}(f_o - f_\infty)(1 - e^{-k\Delta t})(1 - \frac{kF_d}{f_o - f_\infty}) + f_\infty \Delta t$$

$$= (1 - e^{-kdt})(f_o - f_\infty)/k - F_d + f_\infty \Delta t \quad ....(2.8)$$

The actual depth of infiltration during the time interval, Δt, is either the available depth of rainfall or the infiltration capacity, ΔF_{cap}, whichever is the lesser, i.e.

$$\Delta F = \text{lesser of} \begin{cases} i.\Delta t \\ \Delta F_{cap} \end{cases} \quad ....(2.9)$$

In order to determine the accumulated diminishing infiltration, F_d, we must apportion the actual infiltration depth, ΔF, between the diminishing and the constant component. Letting ΔF_d be the increase in F_d and referring to Fig. 2.3 we have

$$\Delta F = \alpha.\Delta F_{cap}$$

$$= \frac{\Delta F_d . \Delta F_{cap}}{\Delta F_{cap} - f_c . \Delta t}$$

$$\Delta F_d = \frac{\Delta F}{\Delta F_{cap}}(\Delta F_{cap} - f_\infty . \Delta t) \quad ....(2.10)$$

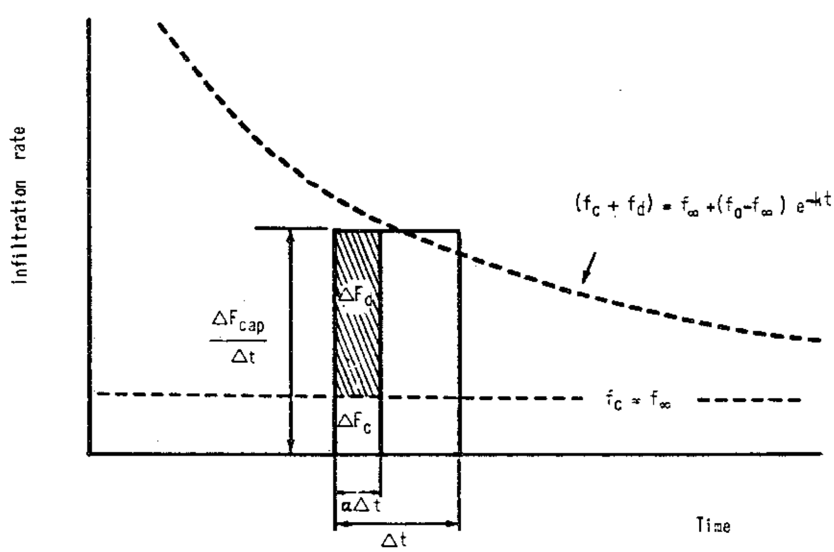


Fig. 2.3 Incrementing accumulated diminishing infiltration

Eqs. 2.8, 2.9 and 2.10 permit depth of infiltration for any time interval, Δt, to be determined explicitly. The only restriction on the use of these equations is that Δt must not be chosen so large as to render unreasonable the assumption of a constant infiltration rate over the interval.

Portions of catchments which are impervious but which drain onto pervious areas can be accounted for by proportionately increasing the rainfall on the pervious areas, i.e.

$$i_p = (1 + \%A_s/100)\ i \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.11)$$

where i_p = the effective rainfall intensity on the pervious area

$\%A_s$ = the supplementary impervious area as a percentage of the pervious area

i = the rainfall intensity

This approximation is adequate when the impervious areas have relatively small response times. Examples of such areas are rocky outcrops in rural catchments and houses with roof drains discharging onto gardens in urban areas.

A flow chart for the computation of excess rain is presented in Fig. 2.4. At first glance the method appears complex but computationally it is highly efficient since iteration is completely eliminated.

When simulating runoff from observed storms there is sometimes a fair amount of rain falling at the beginning of the storm at an intensity which is obviously lower than infiltration capacity. In these cases it is often easier to sum the low intensity rainfall and use this to determine the amount of accumulated diminishing infiltration directly. This is done by determining the time position on the infiltration curve for cumulative depth of infiltration, F_o, equal to the total depth of low intensity rainfall, P_o, as adjusted to account for supplementary impervious-area runoff, i.e.

$$F_o = (1 + \%A_s/100)\ P_o \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.12)$$

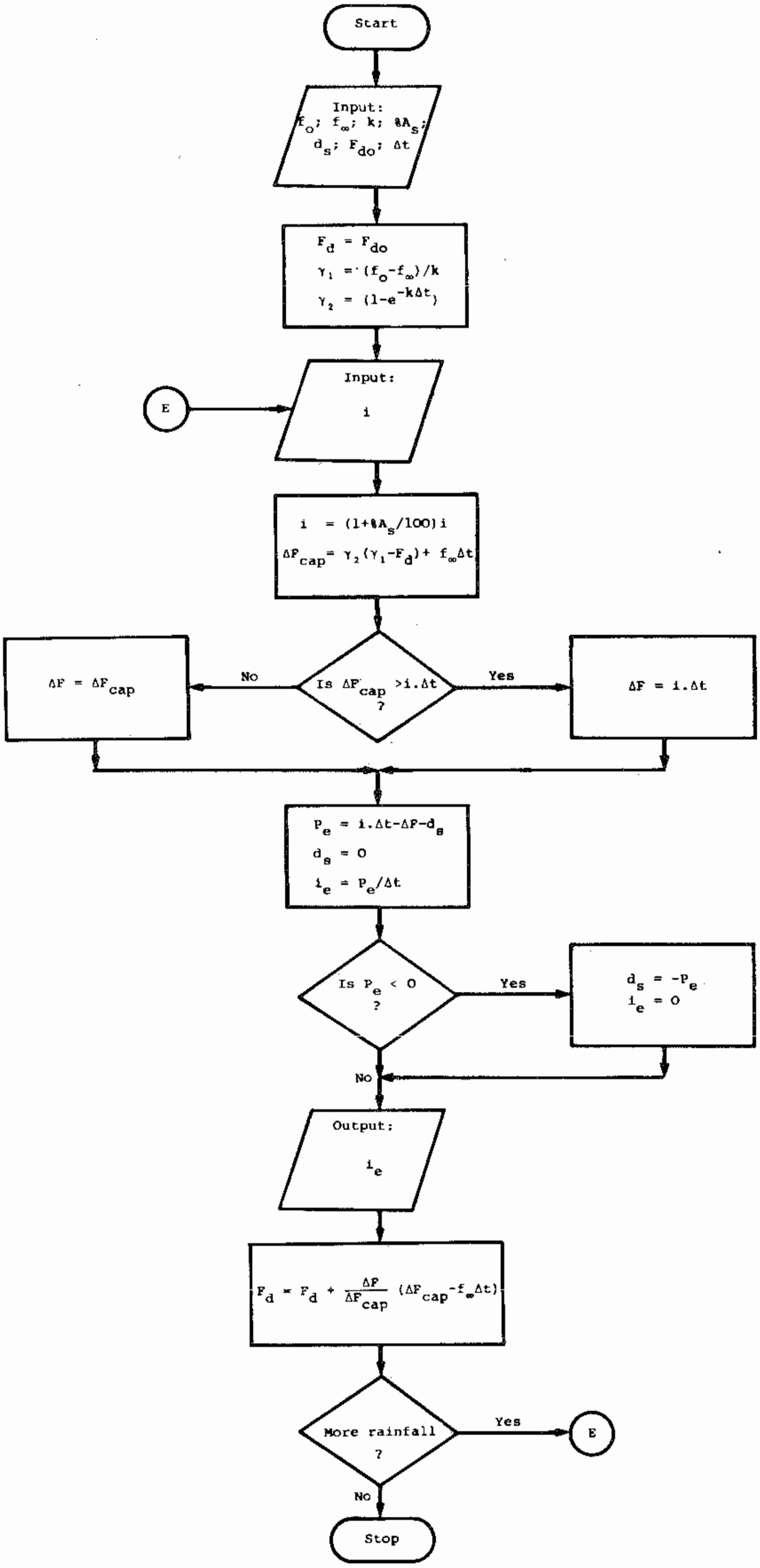


Fig. 2.4 Flow chart for computation of excess rainfall

The solution for time is by necessity implicit and can be conveniently obtained using the Newton-Raphson iterative technique, i.e.

$$t = t - \frac{g(t)}{g'(t)} \quad \text{.............................(2.13)}$$

where in this case g(t) is the zeroed integral of eq. 2.1.
The solution for t is

$$t = t - \frac{f_\infty t + (f_o - f_\infty)\ (1-e^{-kt})/k - F_o}{f_\infty + (f_o - f_\infty)\ e^{-kt}} \quad \text{......(2.14)}$$

The cumulated diminishing infiltration, F_d, is then determined as

$$F_d = F_o - f_c . t \quad \text{.............................(2.15)}$$

2.3 Depression storage

Depression storage is the loss to runoff caused by the ponding of water in shallow surface depressions. In the calculator programs this is considered as an initial loss to be subtracted from rainfall in excess of infiltration (as shown in Fig. 2.4). No regeneration of this loss is accounted for in periods where rainfall is less than infiltration capacity. This is only occasionally significant in single-event simulation and of no consequence when using a typical design storm.

2.4 Time-area diagram

The catchment (or zone) time-area diagram represents the accumulated contributing area with time and is determined by summating the linear subcatchment curves as illustrated in Fig. 2.1. To facilitate program computations the abscissae are rendered dimensionless by dividing through by the computational time step, Δt. The linear subcatchment curves are then characterized by a dimensionless flow-time, $\tau_f = t_f/\Delta t$, a dimensionless entry-time, $\tau_e = t_e/\Delta t$, and subcatchment area. This is illustrated in Fig. 2.5.

The isochronal areas, ΔA_τ, for each subcatchment as determined from the geometry of Fig. 2.5 can be computed as follows:

$$\Delta A_\tau = A_\tau - A_{\tau-1}$$

$$= A_\tau - (A/\tau_e)\,[\tau-1 - \tau_f] \quad \text{......................}(2.16)$$

where $[\tau-1-\tau_f] = 0$ when $\tau-1-\tau_f < 0$

These areas are determined for each subcatchment and summated at each dimensionless time step to obtain the total catchment time-area diagram.

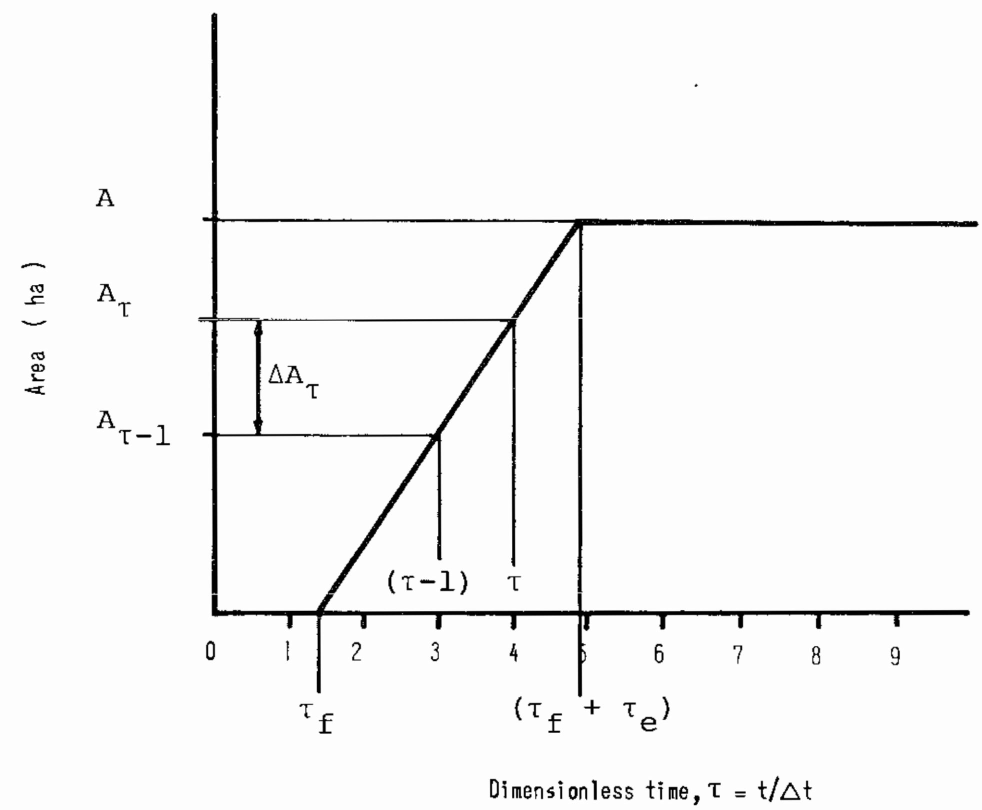


Fig. 2.5 Dimensionless subcatchment time-area curve

2.5 Time-area routing

The principles of the routing technique are shown in Fig. 2.1(e). For programming purposes it is convenient to consider two arrays of size equal to the maximum dimensionless time value, M, of the catchment time-area diagram, one array containing the isochronal areas, ΔA_{τ}, and the other the runoff, $R_{\tau,t}$, on each area at time t. The runoff to each isochronal area from its upstream neighbour is computed by mass balance for each time increment, i.e.

$$R_{\tau,t} = R_{\tau+1,t-\Delta t} + i_{et}.\Delta A_{\tau+1} \qquad \text{..............(2.17)}$$

and for the area furthest from the outfall

$$R_{M,t} = 0 \qquad \text{.......................................(2.18)}$$

For excess rain intensity, i_{et}, in mm/h and area, $\Delta A_{\tau+1}$, in ha, the outfall discharge at time t is

$$Q_t = (R_{1,t-\Delta t} + i_{et}.\Delta A_1)/360 \qquad \text{..............(2.19)}$$

2.6 Design storm

Design storms are synthetic temporal distributions of rainfall used by the engineer to facilitate the sizing of structures, and are based on representative properties of real storms. For flood peak prediction the three most important properties to consider are the total volume of rainfall, the maximum average intensity for the critical catchment response time, and the depth of rainfall antecedent to the peak intensity.

These properties are taken into consideration in the convenient Chicago design storm (Keifer and Chu, 1957) which is based on intensity-duration-frequency (IDF) curves; the distribution is such that for any time interval the maximum average intensity is equal to that from the IDF curves. This means that when one applies the storm to a catchment the critical intensity for all possible sub-areas is used and the necessity of determining the critical storm duration for the catchment is eliminated. The position of the peak intensity within the storm is based on local storm characteristics.

Using an IDF equation of the form:

$$I = \frac{a}{(b+t)^c} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.20)$$

where I is the average rainfall intensity for duration, t, and a, b and c are parameters dependent upon the locality and design frequency, the equation for the Chicago design storm can be derived as follows:

$$i = 60 \frac{dP}{dt} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.21)$$

where i = rainfall intensity (mm/h) at time t (minutes)

P = depth of rainfall (mm)

$= I.\ t/60$

$= \left(\frac{a}{(t+b)^c}\right) t/60$

Thus,

$$i = \frac{a((1-c)t+b)}{(t+b)^{c+1}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.22)$$

This is the equation for an advanced storm pattern, i.e. the peak occurs at the beginning of the storm. If the peak occurs at some later time, then the storm can be described by considering the duration, t, as being composed of a time t_b before, and a time t_a after, the peak, i.e.

$$t = t_b + t_a$$

Now if r is the ratio of the time-to-peak, t_p, to the total duration of the storm, t_d, then

$$r = \frac{t_p}{t_d}$$

$$= \frac{t_b}{t} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.23)$$

$$= \frac{t-t_a}{t} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.24)$$

Substituting for t from eqs. 2.23 and 2.24 in eq. 2.22 gives the following relationships for intensities before and after the peak:

$$i_b = \frac{a\left((1-c)\frac{t_b}{r} + b\right)}{(t_b/r + b)^{c+1}} \quad \text{.............................(2.25)}$$

$$i_a = \frac{a\left((1-c)\frac{t_a}{1-r} + b\right)}{\left(\frac{t_a}{1-r} + b\right)^{c+1}} \quad \text{.........................(2.26)}$$

To use the Chicago storm it is necessary to reduce the storm-hyetograph to a set of discrete values. Use of eqs. 2.25 and 2.26 is inconvenient since average intensities over each interval are required. A simple method is as follows:

(i) Select the time step Δt.

(ii) Compute the discrete point representing the peak rainfall from the equation:

$$i = \frac{a}{(\Delta t + b)^c} \quad \text{...........................(2.27)}$$

(iii) Distribute the time interval selected (Δt) around the peak as $r\Delta t$ before the peak and $(1-r)\ \Delta t$ after the peak.

(iv) Compute the points before and after the peak by integrating the design curve and calculating the discrete intensity ordinates from the volumes for each increment of t.

The general integral form of the hyetograph before the peak is given by:

$$\int_{t_{b1}}^{t_{b2}} i_b dt_b = \left[\frac{at_b/60}{\left(\frac{t_b}{r}+b\right)^c}\right]_{t_{b1}}^{t_{b2}} \quad \text{.........................(2.28)}$$

and after the peak by:

$$\int_{t_{a1}}^{t_{a2}} i_a dt_a = \left[\frac{at_a/60}{\left(\frac{t_a}{1-r}+b\right)^c}\right]_{t_{a1}}^{t_{a2}} \quad \text{.........................(2.29)}$$

An algorithm based on this technique is presented in Fig. 2.7. The variables used are illustrated in Fig. 2.6 and described in Appendix C. So that intensities can be computed in order of occurrence, the starting time, t_o, is first determined. Eq. 2.28 is then used to compute intensities up to the peak. The peak intensity is computed using eq. 2.27 and intensities after the peak using eq. 2.29. Calculations stop when $t_a = t_o + (1-r)t_d$.

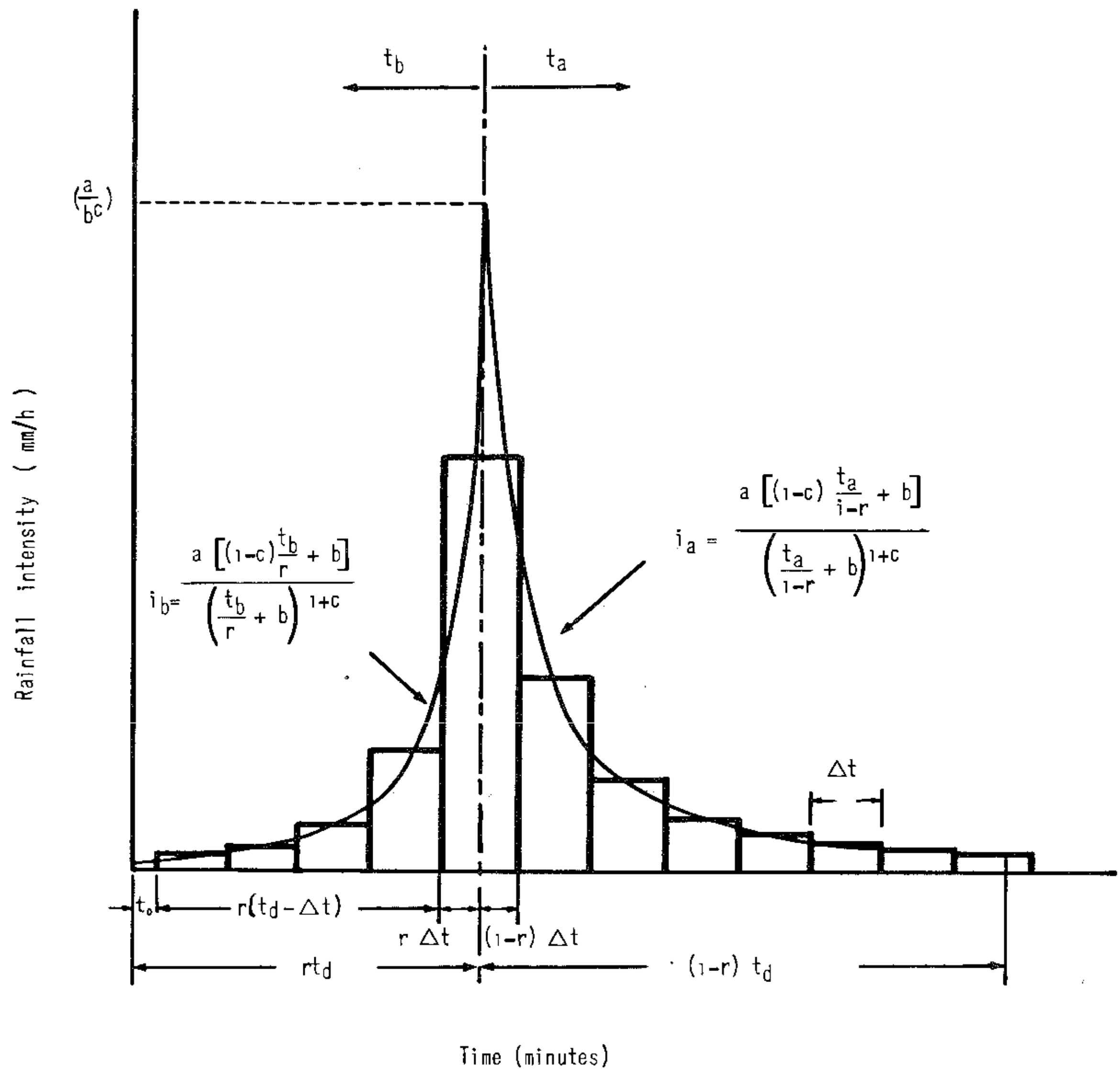


Fig. 2.6 Discretization of the Chicago storm

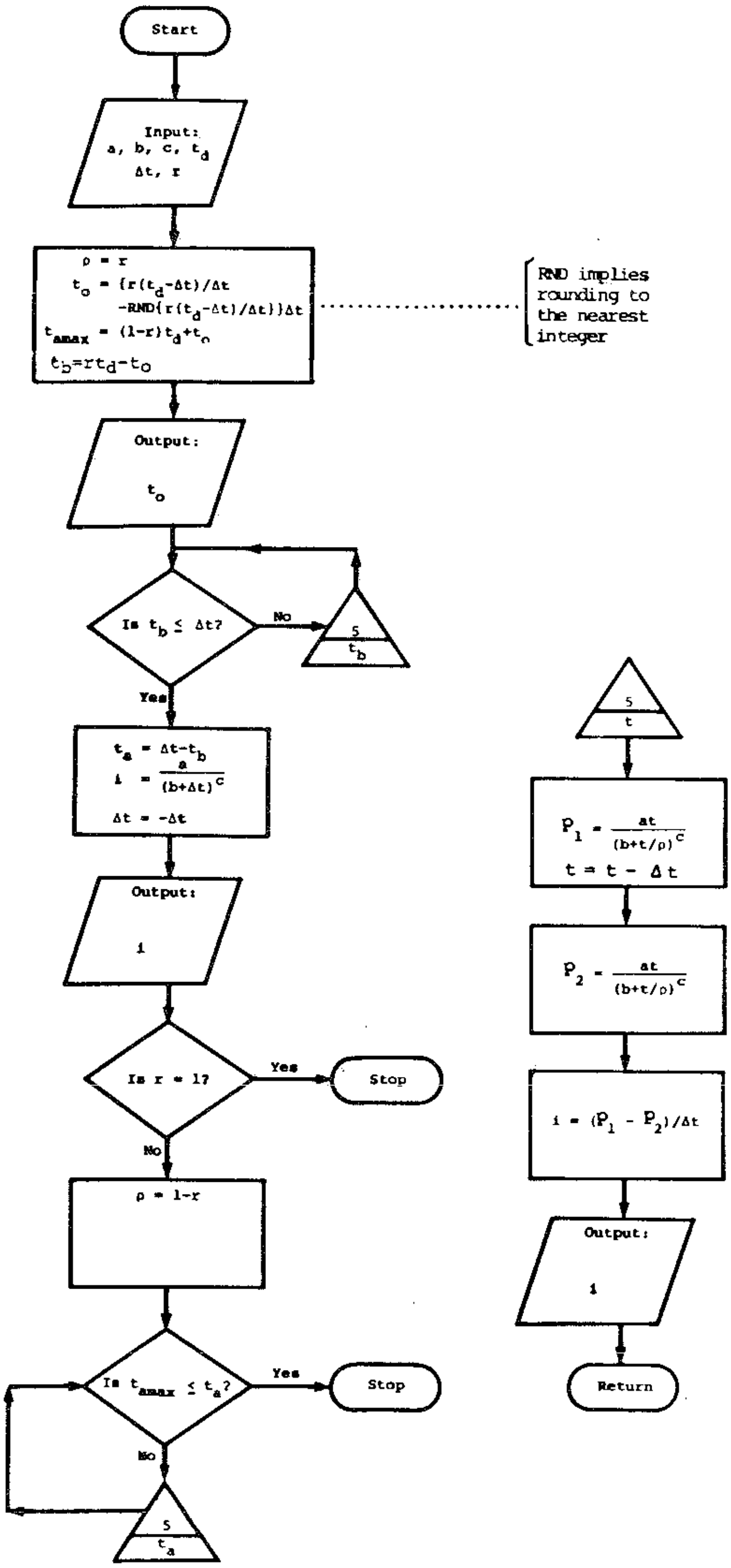


Fig. 2.7 Algorithm for discretizing Chicago storm

2.7 Theoretical limitations

The method described in this chapter does not take account of certain phenomena which may in some circumstances be important. Only those factors felt to be most significant to small catchment flood estimation have been considered. Factors relevant to the determination of low flows (viz. subsurface-flow, evapotranspiration, interception and partial area contributions) are largely ignored. Losses to runoff are allowed for by decreasing rainfall input whereas it would be more nearly correct to subtract losses from surface flow depths. The regeneration of depression storage on pervious areas during low rainfall intensities is not accounted for directly. Only discrete events can be considered since recovery of infiltration capacity between events is not taken into account.

Routing is rather simplistic since account is not taken of changes in velocity with flow depth. A constant velocity representative of the significant portion of the flow is assumed. The types of resulting error that can be expected are illustrated in Fig. 2.8 for overland flow and Fig. 2.9 for pipe (or channel) flow. The solid line in Fig. 2.8 is the observed runoff hydrograph obtained in a laboratory study by Izzard (1946) while the shaded area represents the simulated rainfall input and the dashed line the computed hydrograph using the time-area method. As can be seen the overall shape of the hydrograph is reproduced fairly well, but the shapes of the rising and recession limbs are not well mimicked. The computed hydrograph initially underestimates surface detention on both limbs. This is due to the constant velocity assumption. The sharp peak on the observed hydrograph following termination of rainfall input is due to decreased flow resistance upon cessation of rain and is probably significant only in the laboratory.

The effect of the constant flow assumption on channel flow can be seen in Fig. 2.9 which illustrates attenuation of flow in a circular pipe. The triangular hydrograph represents the inflow and the other two hydrographs represent outflows for different reach lengths. These were computed by MacLaren Ltd. (1976)

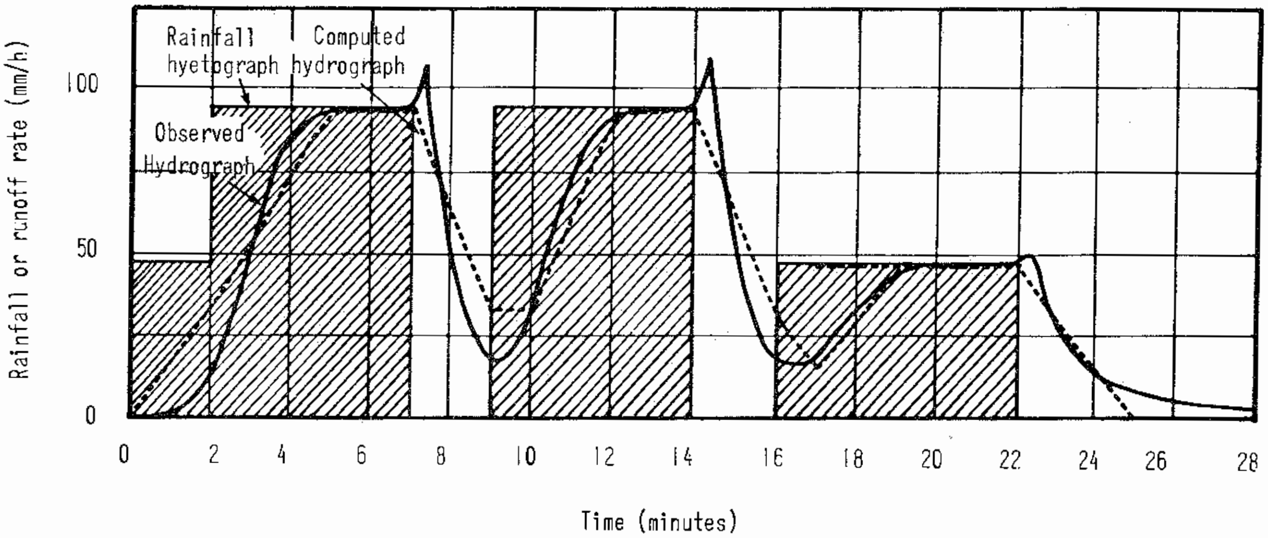


Fig. 2.8 Simulation of Izzard's overland flow hydrograph

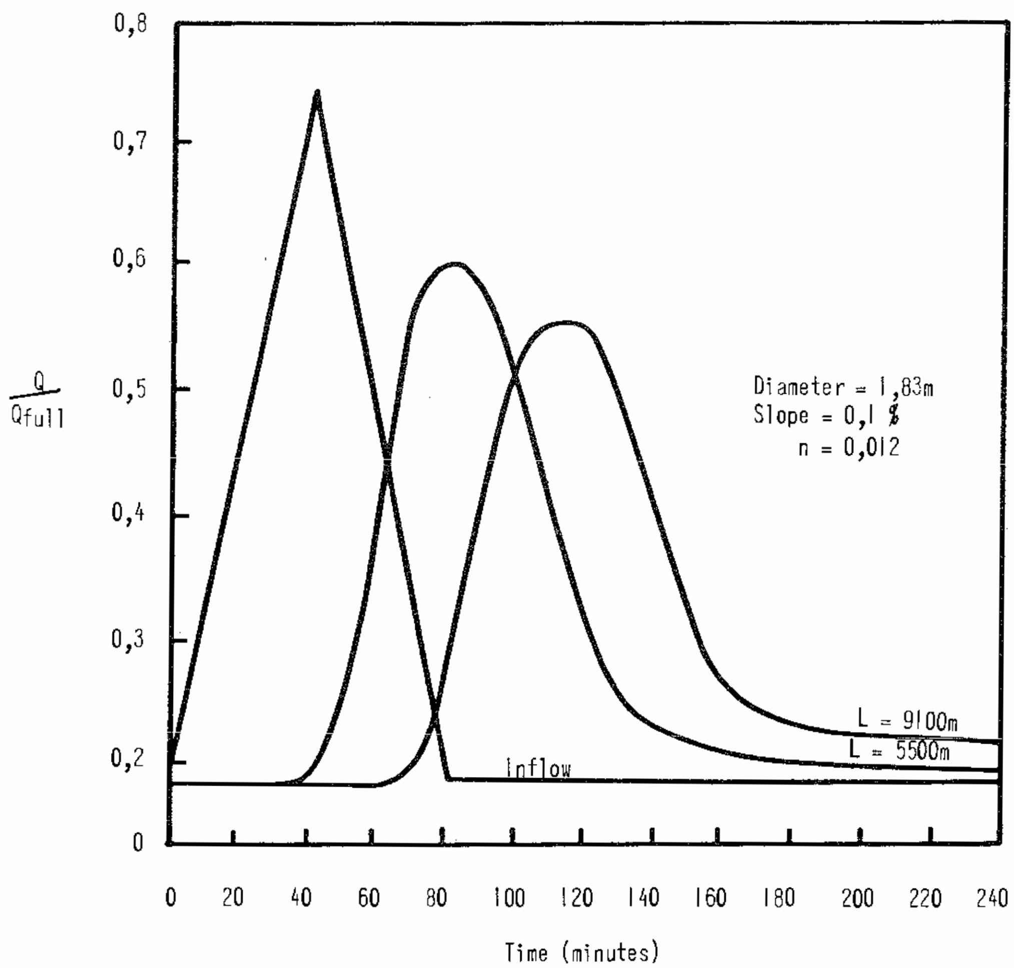


Fig. 2.9 Theoretical pipe flow routing (MacLaren Ltd., 1975)

using the method of characteristics. The constant velocity assumption used in the time-area method would generate outflow hydrographs of the same shape as those of the inflow, but displaced along the time axis. For the example illustrated, peak discharge would have been overestimated by 23% for a reach length of 5500 m and by 32% for a reach length of 9100 m.

CHAPTER 3 ESTIMATION OF PARAMETERS

3.1 Introduction

The time-area method presented here is very similar in principle to ILLUDAS and parameter estimation is in many cases the same. Tentative guides for the estimation of parameters for ILLUDAS have been presented in HRU 1/81 (Watson, 1981a). Much of the material presented there is repeated here for convenience. The form has, however, often been changed to accommodate dissimilar program input requirements.

Recommended parameter values have largely been selected from available literature. Further rainfall/runoff monitoring and analysis will no doubt result in improved values.

3.2 Infiltration

The absorption of water by the soil is termed infiltration. Water enters the soil through cracks, pores or orifices in the surface. Through the larger openings it may flow freely in appreciable quantities under the influence of gravity. Through fine pores movement is much slower and is governed principally by capillary forces. Infiltration rate is usually high at the onset of a storm and decreases to a nearly constant value with lapse of time. The rate of decrease is a function of the volume of water absorbed, the compaction of the surface due to the impact of raindrops, and soil swelling in the case of clays. The final constant infiltration rate is generally controlled by the rate at which water can percolate through the soil profile.

Soil type is the most important factor determining infiltration capacity. Soils with a large percentage of well-graded fines will have low infiltration capacities. In contrast, poorly graded sandy soils will generally have high infiltration capacities.

Soil cover also plays an important role in determining infiltration capacity. Vegetation tends to loosen the surface soil and at the same time protects it from rainfall compaction. Decaying roots create capillary channels which facilitate the flow of water through the soil. In general, the denser the vegetation cover the greater the infiltration capacity. Compaction of the soil surface, e.g. in some urban areas, also reduces infiltration capacity.

The wetter the soil profile at the onset of rainfall the lower will be the initial infiltration rate. Rainfall on days prior to the storm under consideration determines the antecedent moisture condition (AMC) of the soil. It has been shown by Hope (1980) for small catchments that rainfall occurring even 20 days prior to a storm event influences the amount of surface runoff.

Other factors influencing infiltration include: surface slope, depth and uniformity of the soil profile and, in the case of clays, presence of surface cracks.

Horton's equation as modified in Section 2.2 allows for the decrease in infiltration capacity with volume of water absorbed by the soil. Three parameters have to be estimated, viz:

initial infiltration capacity, f_o (mm/h)
final infiltration capacity, f_∞ (mm/h)
recession constant, k (h^{-1})

All three parameters can vary from catchment to catchment, while f_o can also vary considerably for different storms on the same catchment, depending on the AMC. Values of f_o for different soils and AMCs can range from virtually zero to about 500mm/h. Typical values of f_∞ fall between zero and 50 mm/h while the range of k is typically 1 h^{-1} to 8 h^{-1}. The effect on infiltration capacity of variations in the value of k is illustrated in Fig. 3.1

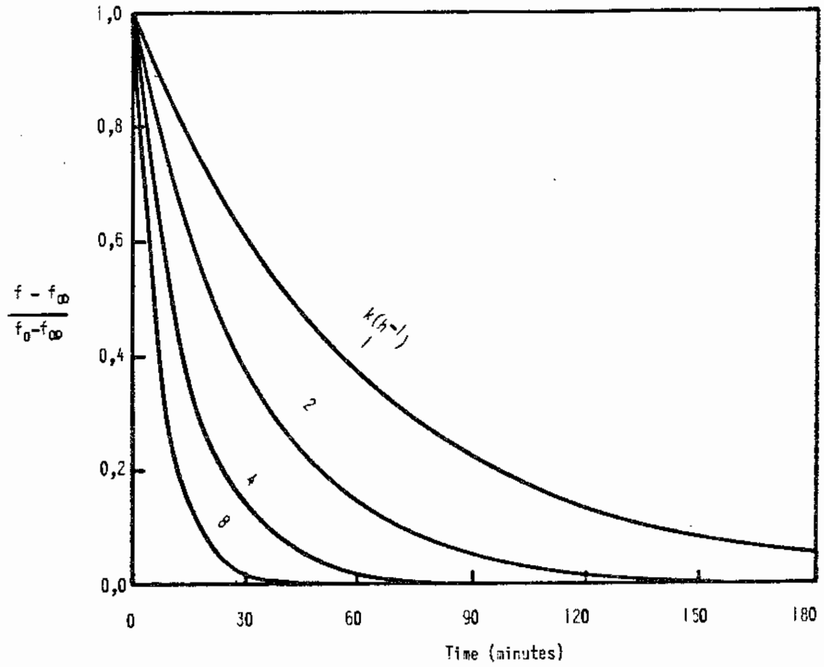


Fig. 3.1 The influence of the parameter k on infiltration capacity

At the current state of knowledge the parameters recommended by the Illinois State Water Survey for use with their urban runoff model ILLUDAS (Terstriep and Stall, 1974) are perhaps the most reasonable. These are described in Table 3.2 as functions of soil type and AMC and are applicable to soils with lawn cover. The AMC values adopted by Terstriep and Stall (1974) are described in Table 3.2 while cover factors for adjusting final infiltration rates are presented in Table 3.3 (ASCE, 1949). The soil types are those defined by the U.S. Soil Conservation Service (1972) and can be briefly described as follows:

- A - High infiltration, typically coarse textured soils (e.g. sands and gravels)
- B - Moderate infiltration rates and moderately well-drained, typically moderately fine to moderately coarse textured soils
- C Slow infiltration rates, typically moderately fine to fine textured soils and soils with layers that impede the downward movement of water
- D Very slow infiltration rates, typically clays or soils with permanent high water tables.

A list of hydrological groupings for South African soil series is presented by Schulze and Arnold (1979).

Table 3.1 Infiltration parameters for use in Horton's equation

Soil type	f_o (mm/h) for AMC:				f_∞ (mm/h)	k (h^{-1})
	1	2	3	4		
A	250	162	84	33	25	2
B	200	130	66	31	13	2
C	125	78	34	7	6	2
D	75	41	7	3	3	2

Table 3.2 Antecedent moisture conditions

AMC number	Description	Total rainfall during 5 days preceding storm (mm)
1	Completely dry	0
2	Rather dry	0 to 12,5
3	Rather wet	12,5 to 25
4	Saturated	over 25

Table 3.3 Infiltration cover factors

Cover		Range in value of cover factor
Type	Condition[1]	
Permanent (forest and grass)	good	1,5 - 3,8
	medium	1,0 - 1,5
	poor	0,6 - 0,9
Close growing crops	good	1,2 - 1,5
	medium	0,8 - 1,1
	poor	0,5 - 0,7
Row crop	good	0,7 - 0,8
	medium	0,6 - 0,7
	poor	0,5 - 0,6

[1] good - high cover density

medium - cover density from 80% to 30% of that for "good" areas

poor - sparse cover, less than 30% of the density on "good" areas

3.3 Depression storage

Rainfall that collects in small surface depressions and does not become runoff is termed depression storage. This is usually described in terms of an average depth over the whole surface. Typical values range between 0,5 mm and 7,5 mm depending on land use and ground slope. In special instances (e.g. contour-tilled land) values as large as 75 mm are possible (Musgrave and Holtan, 1964). In the particular case of contour-tilled land, however, smaller values are more probable because of breakage of contour furrows.

Estimation of this parameter is usually not critical for design since it generally forms a small percentage of the total rainfall. Values of 1 mm and 5 mm are recommended for paved and unpaved areas respectively.

3.4 Entry time and flow time

Entry time is the time taken for runoff from the hydraulically most distant point in the sub-catchment to enter the reach. Flow time is the subsequent travel time in the reach to the catchment outfall assuming flow at a constant velocity. Both parameters are functions of the depth of flow and therefore can vary both within a storm as well as between storms. Assuming these parameters to be constant for a particular storm greatly simplifies the analysis without significantly affecting simulation of storm hydrograph characteristics (see Chapters 4 and 5).

Combination of these two parameters for the hydraulically most distant subcatchment is analogous to application of the time of concentration in the Rational Method. The empirical formulae in common use for estimating time of concentration, however, are mutually inconsistent and of dubious value. Fig. 3.2 compares four commonly-used estimation techniques with the theoretically-based kinematic wave method for overland flow. The figure shows a wide spread of the variation of time of concentration with the ratio of length to square root of slope ($L/\sqrt{s}$). Only the U. S. Soil Conservation Service (SCS) method and the Bransby-Williams method show comparable relationships.

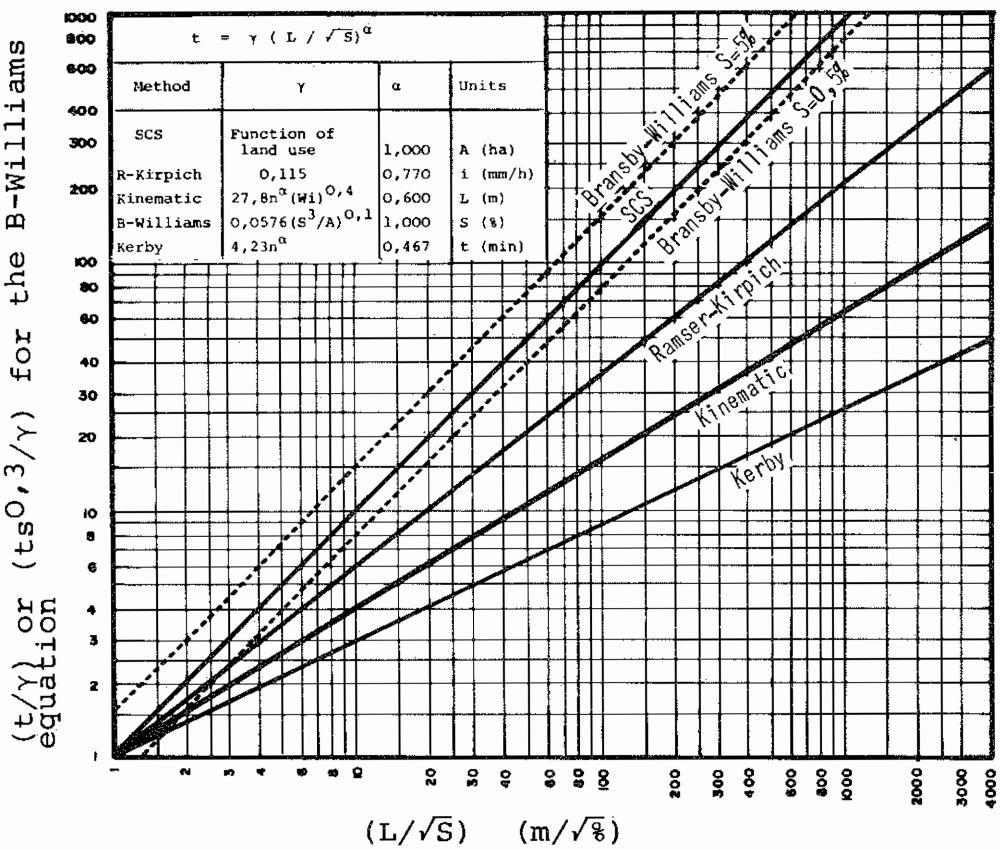


Fig. 3.2 Comparison of time of concentration estimation techniques

The differences are mainly due to the use of different data bases in deriving the formulae. Applicability of each formula, as for any empirical method, is limited to the bounds of the data base.

The Ramser-Kirpich equation (Ramser, 1927, and Kirpich, 1940) is based on average hydrograph rise times for storms on seven agricultural catchments ranging in size from 0,5 to 45 ha with average slopes ranging from 2,7 to 9,8%. The Bransby-Williams formula, on the other hand, was published in a paper on spillway design in India (Williams, 1922). No derivation is given, and it can but be assumed that it was based on river flow measurements. Both these formulae have been shown by French et al.(1974) to be poor predictors of rise time. The SCS method is presented as a plot of flow velocity versus slope for different land uses (SCS, 1972). No empirical or theoretical basis is given for the plot.

Kerby's formula (Kerby, 1959, and Hathaway, 1945) is simply an approximation of the semi-theoretical equation for overland flow by Horton (1938). The kinematic wave equation is a theoretical solution for time to equilibrium for a uniform rainfall intensity on a rectangular plane assuming flow velocity to be a function of depth only. The kinematic equation is to be preferred to the equations of either Horton or Kerby.

Though empirical techniques may be useful as a standard of comparison, entry time and flow time should be determined on the basis of hydraulic principles. For many catchments this is not a simple matter and one is forced to make gross simplifications of the hydraulic response of the catchment. The approach does have the advantage, though, of forcing an awareness of the lack of accuracy of one's estimates.

The velocity of unsteady, non-uniform flow is not the same as that of steady uniform flow. Increments in discharge cause waves to proceed downstream at velocities greater than the mean water velocity. The wave velocity for upstream inflow can be approximated as:

$$V_w = \frac{1}{B} \frac{dQ}{dy} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.1)$$

where V_w = wave velocity (celerity)

B = width of flow at the surface

$\frac{dQ}{dy}$ = differential of discharge with respect to depth

This relationship was derived by Sneddon (1900) and was shown by Pitman and Midgley (1966) to give reasonable estimates of flood travel times in local rivers. The ratio of wave velocity to uniform velocity varies from 1 to 5/3 for various trapezoidal channel cross-sections as shown in Fig. 3.3.

For reaches with only lateral inflow the wave velocity is less than in channels with only upstream inflow. For a wide rectangular channel subject to a uniform lateral inflow the wave velocity is the same as the uniform flow velocity at equilibrium discharge.

For overland flow the situation is the same as for wide rectangular channels with lateral inflow. Travel time is conveniently computed using the kinematic wave equation:

$$t = 27{,}8 \left(\frac{nL}{\sqrt{s}}\right)^{0{,}6} (Wi_e)^{-0{,}4} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.2)$$

$$\frac{V_w}{V} = \frac{1}{3}\left[5 - \frac{4\sqrt{1 + z^2}\,(b/y + z)}{(b/y + 2z)\,(b/y + 2\sqrt{1 + z^2}\,)}\right]$$

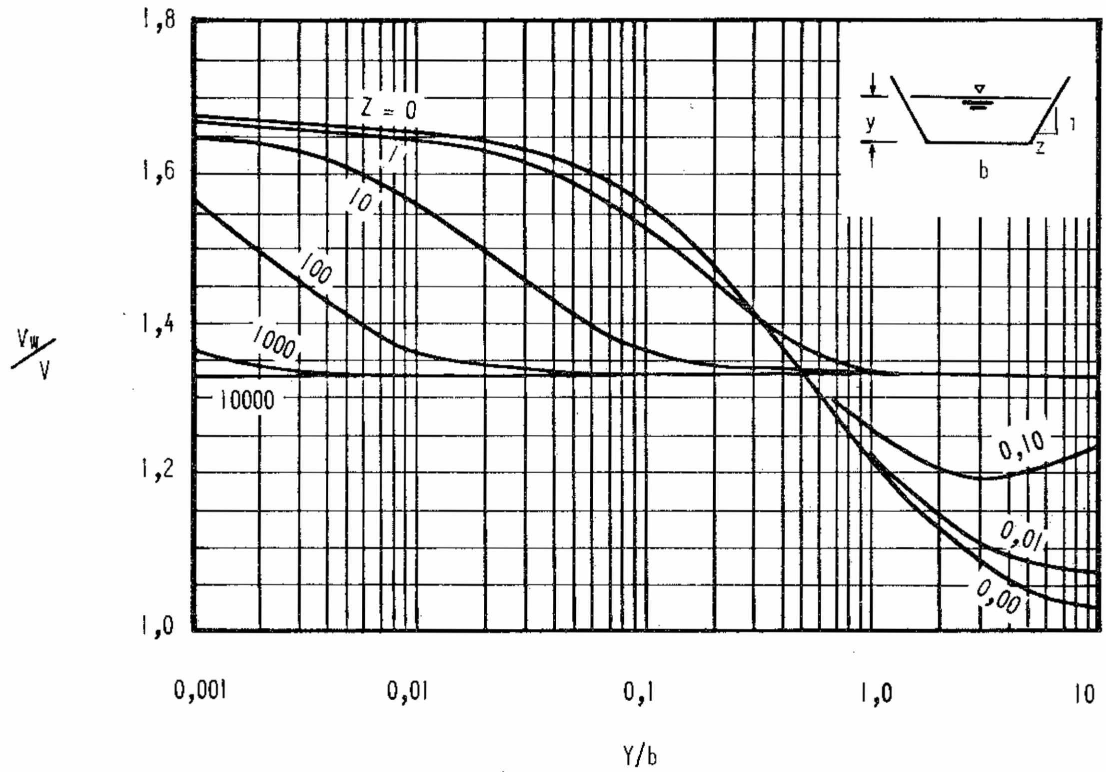


Fig. 3.3 Ratio of wave velocity to uniform flow velocity for flow in a trapezoidal channel

where t = travel time (minutes)
n = Manning's n
L = flow length (m)
s = slope (%)
i_e = excess rain intensity (mm/h)
W = ratio of subcatchment width to flow width

The width ratio, W, is introduced to allow for the concentration of runoff in small gullies or gutters. For channel flow Wi_e would be equal to the lateral inflow rate per unit area of channel. A nomograph for the solution of eq. 3.2 is presented by Watson (1981a). Values of Manning's n for overland flow are given in Table 3.4.

Eq. 3.2 requires an estimate of excess rain intensity representative for the whole storm. The average intensity for a duration approximately equal to the catchment time of concentration would be adequate. Fig. 3.4 is provided to assist in assessing the effect on travel time of variations in rainfall intensity.

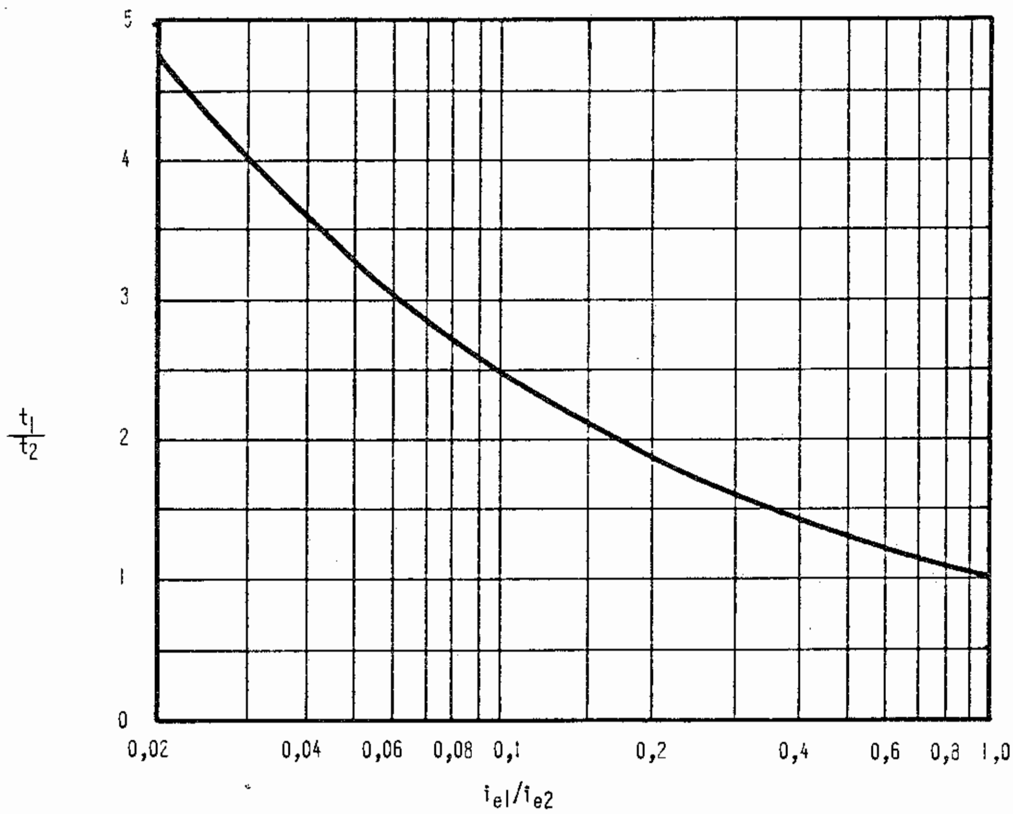


Fig. 3.4 Variation in entry time with variation in excess rain

Table 3.4 Manning's retardance coefficient, n, for overland flow (adapted from Woolhiser, 1975)

Surface	Range in n
Concrete or asphalt	0,010 - 0,013
Bare sand	0,010 - 0,016
Gravelled surface	0,012 - 0,030
Bare clay-loam soil (eroded)	0,012 - 0,033
Sparse vegetation	0,053 - 0,130
Veld	0,100 - 0,200
Lawns (and forest litter)	0,170 - 0,480

3.5 Chicago design storm

The intensity-duration-frequency (IDF) coefficients in eq. 2.20 can readily be evaluated for local conditions by regression analysis of available IDF curves. Simple techniques are described by Watson (1981a and 1981b). In the absence of local IDF relationships, the coefficients given by Midgley and Pitman (1978) can be used. These coefficients, in units compatible with those used in this work, are given in Table 3.5 and Fig. 3.5. The parameters b and c vary only with region while the parameter a also varies with return period. An equation for a in terms of the average 60-minute intensity for a 10-year return period, $I_{10,60}$, is

$$a = \gamma_I \; I_{10,60} \; T^{0,3} \qquad \text{.............................(3.3)}$$

where γ_I = a regional constant given in Table 3.5

and T = the return period (years)

Alternatively, a can be expressed in terms of mean annual precipitation (MAP) as follows:

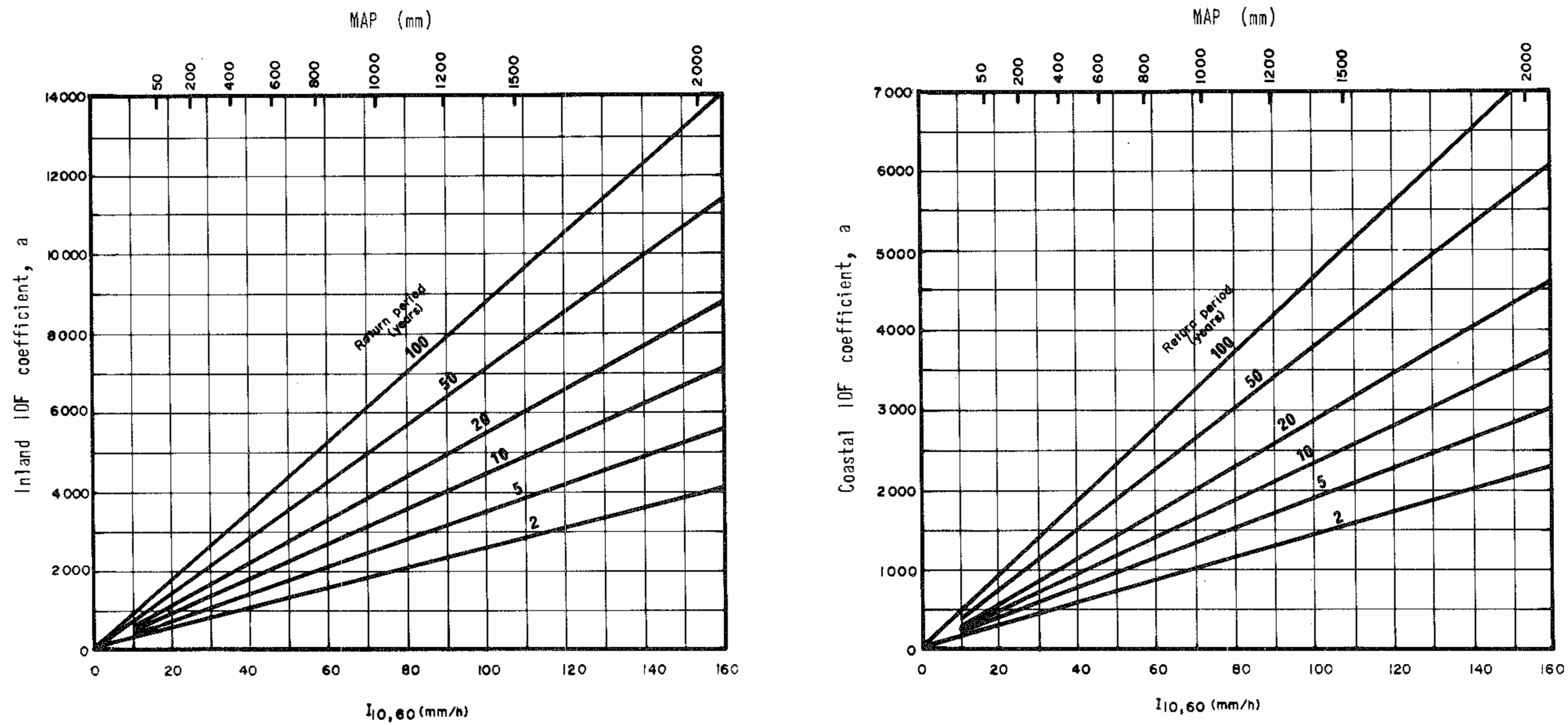
$$a = \gamma_R \; \exp(0{,}06\,\sqrt{MAP}) \; T^{0,3} \qquad \text{.......................(3.4)}$$

where γ_R is a different regional constant with values also given in Table 3.5.

Table 3.5 Regional parameters for Chicago storm

Region	b	c	r	γ_I	γ_R
Inland	14,4	0,883	0,40	22,5	241
Coastal	12,6	0,737	0,40	11,8	84

Eqs. 3.3 and 3.4 are both limited by the data base described in HRU Report 2/78 (Midgley and Pitman, 1978), viz. 50mm<MAP<1050 mm. For MAP greater than 1050 mm HRU 2/78 uses a linear extrapolation which can reflect values that differ by up to 20% from those given by eq. 3.4 for MAP less than 2000 mm. Fig. 3.5 is based on this linear extrapolation.



(a) Inland regions

(b) Coastal regions

Fig. 3.5 Intensity-duration-frequency coefficient, a

The time-to-peak ratio, r, determines the depletion of rainfall losses prior to the peak intensity. Thus the greater the value of r, the larger the volume of runoff. This ratio can be determined from an analysis of local storm hyetographs as described by Keifer and Chu (1957) or Watson (1981a). The ratio varies with storm duration, decreasing with increasing duration. In the absence of local data a value of r equal to 0,40 should be reasonable for storm durations of 2 to 3 hours.

Storm durations should not be varied for every catchment. As long as the duration is substantially longer than the catchment time of concentration it will be adequate. A duration of 2 hours is suggested for catchments with concentration times shorter than 1,5 hours.

CHAPTER 4 VERIFICATION ON URBAN CATCHMENTS

4.1 Introduction

The time-area technique makes certain gross simplifications of the rainfall/runoff process. Surface and channel flow velocities are assumed constant with time, subsurface storm-flow is ignored, losses are subtracted from rainfall instead of from flow depth and are averaged over substantial areas. To establish how reasonable these assumptions are, estimated and observed runoff hydrographs must be compared.

For this purpose data for 36 storms on 8 urban catchments have been assembled. The catchments range in size from 0,2 ha to 143 ha. Only two catchments are local, the remaining six are in the U S A and Canada. Computed hydrographs are compared with observed and in some cases with simulations from other studies which make use of more complex techniques.

Catchments were generally divided into two zones - a directly-connected paved zone and a grassed (unpaved) zone. Paved areas not directly connected to the drainage system (e.g. houses that drain roof water on to gardens) were considered to supplement the rainfall on grassed areas. In all cases parameters were either estimated or taken from published data. Where data were insufficient or processes too complex to analyse, typical parameter values were assumed. For example, in the absence of data to the contrary, depression storage was assumed equal to 1 mm for paved areas and 5 mm for grassed areas. Entry times were estimated using eq. 3.2 for the two small catchments but for the larger ones an entry time of 5 minutes for the paved area and 10 minutes for the grassed area was generally assumed.

Results are presented in the following sections in order of catchment size. Rainfall and AMC data are given in Appendix A.

4.2 South Parking Lot[1]

JohnsHopkins University South Parking Lot catchment no. 1 is shown in Figs. 4.1 and 4.2. It has an area of 0,160 ha and a mean ground slope of 1,8%. It is surfaced with asphalt and bounded by an asphalt curb. Runoff was measured by means of a stage recorder in a calibrated weir-box located in the storm water inlet at the catchment outfall. Rainfall records were obtained from a tipping bucket gauge recording every 0,25 mm (0,01 inch) increment. This was located adjacent to the catchment as shown in Fig. 4.2.

For purposes of simulation the catchment was assumed to have an average depression storage capacity of 1 mm and to be completely impervious. The catchment was discretized into six subcatchments as shown in Fig. 4.3. Entry and flow times were computed using eq. 3.2 with a Manning n of 0,02 and a width ratio of 1,0 for overland flow and 10 for swale flow. An average rainfall intensity of 50 mm/h was assumed for computing entry and flow times for the simulated events. Subcatchment data are given in Table 4.1 and the computed time-area diagram is shown in Fig. 4.4. A one-minute time increment was used for routing.

Fig. 4.1 General view of South Parking Lot no. 1 (Terstriep and Stall, 1974)

Rainfall and runoff data were available for six events. Computed and observed hydrographs are compared in Figs. 4.5 to 4.10.

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[1] Sources of data: Grace and Eagleson, 1966
Harley, Perkins and Eagleson, 1970

Observed runoff is unaccountably less than observed rainfall. Harley et al. (1970) consider this to be due to data errors caused by faulty setting of recording equipment as well as gauge malfunctions. Runoff volumes and peaks are generally overestimated but computed and observed hydrographs are similar in shape. The average ratio of computed to observed peak discharge is 1,06 with a standard deviation of 0,14.

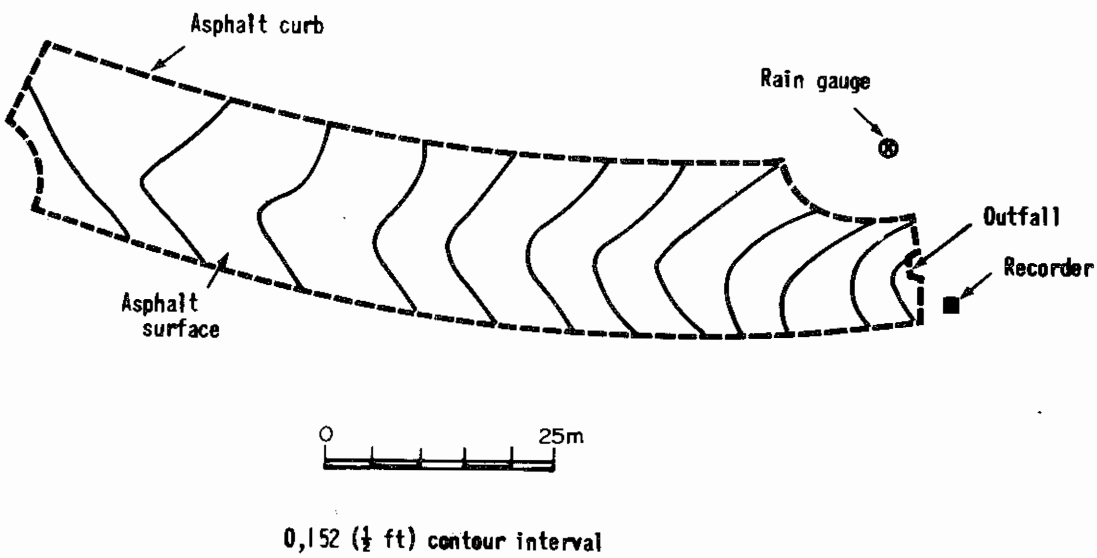

Fig. 4.2 Johns Hopkins University South Parking Lot no. 1

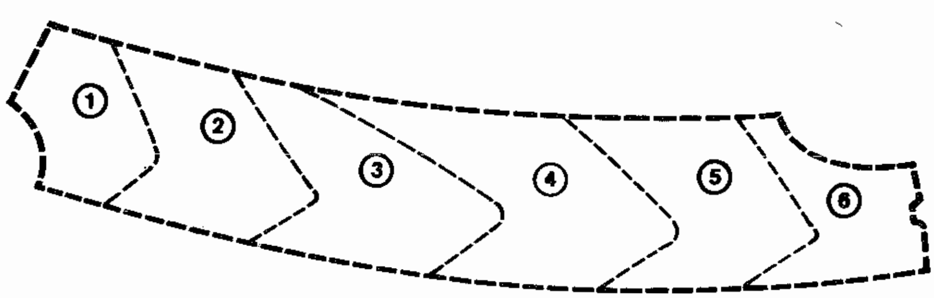

Fig. 4.3 Discretization of South Parking Lot

Table 4.1 South Parking Lot subcatchment data

Sub-catchment	Area (ha)	Entry time (minutes)	Flow time (minutes)
1	0,017	3,8	2,8
2	0,027	3,0	2,2
3	0,035	3,5	1,6
4	0,034	3,0	1,0
5	0,028	2,4	0,5
6	0,019	2,2	0,2
	0,160		

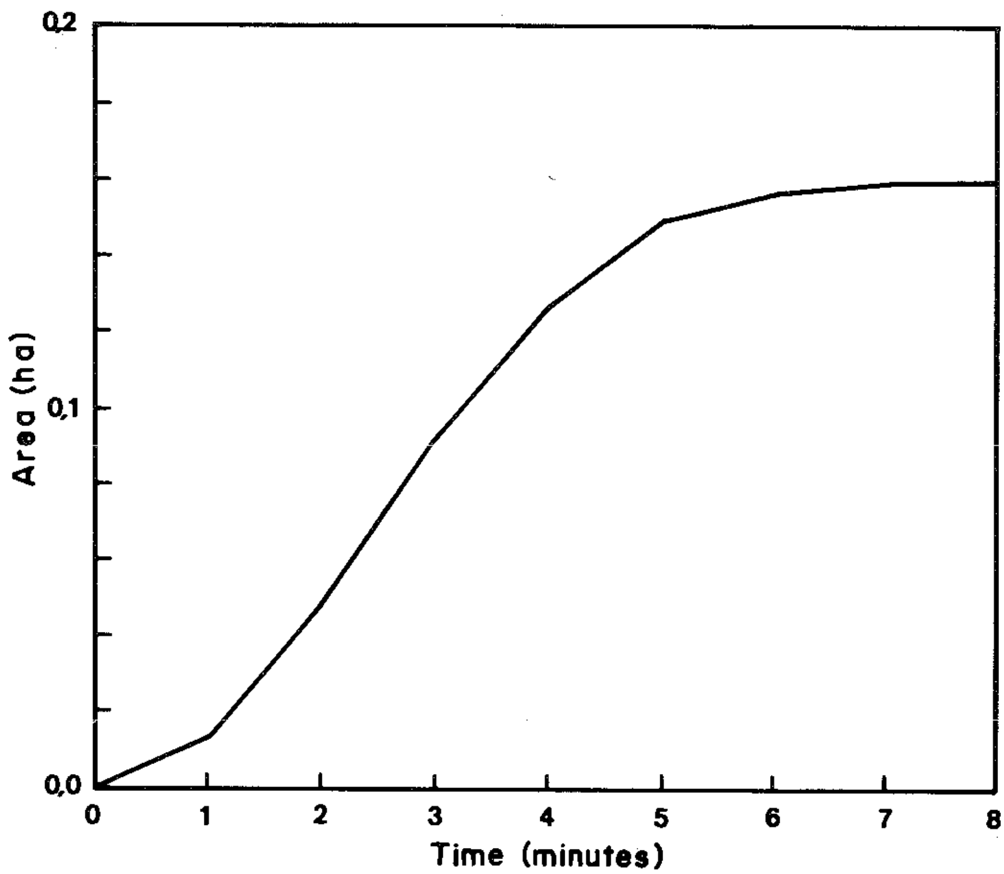

Fig. 4.4 South Parking Lot time-area diagram

Comparisons with hydrographs computed using the more complex kinematic wave routing (Figs. 4.6 and 4.7) are extemely favourable and, generally speaking, the time-area method can be considered to perform adequately on this catchment.

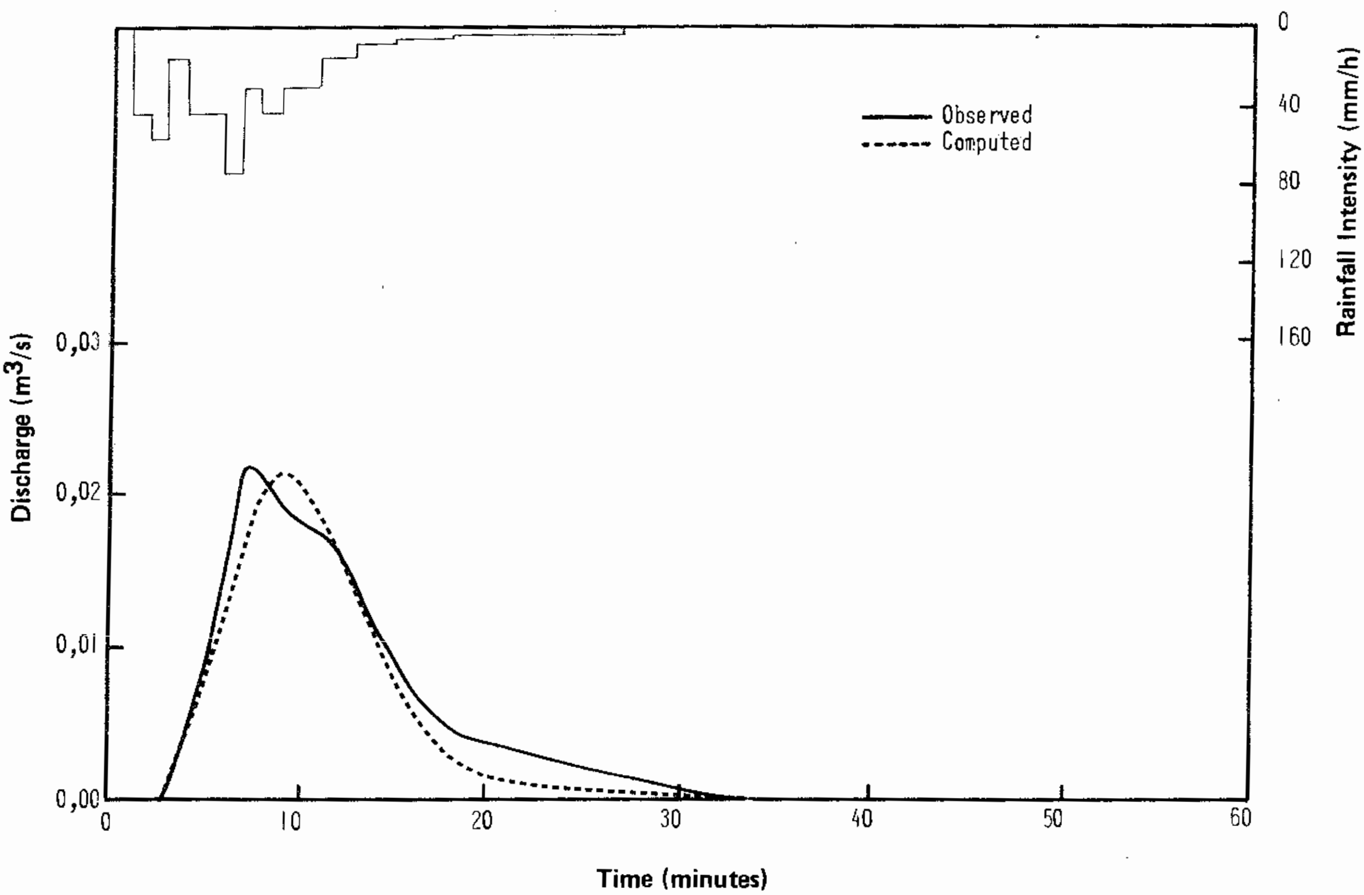

Fig. 4.5 Comparison of computed with observed hydrograph for the storm No. 7 on the South Parking Lot catchment

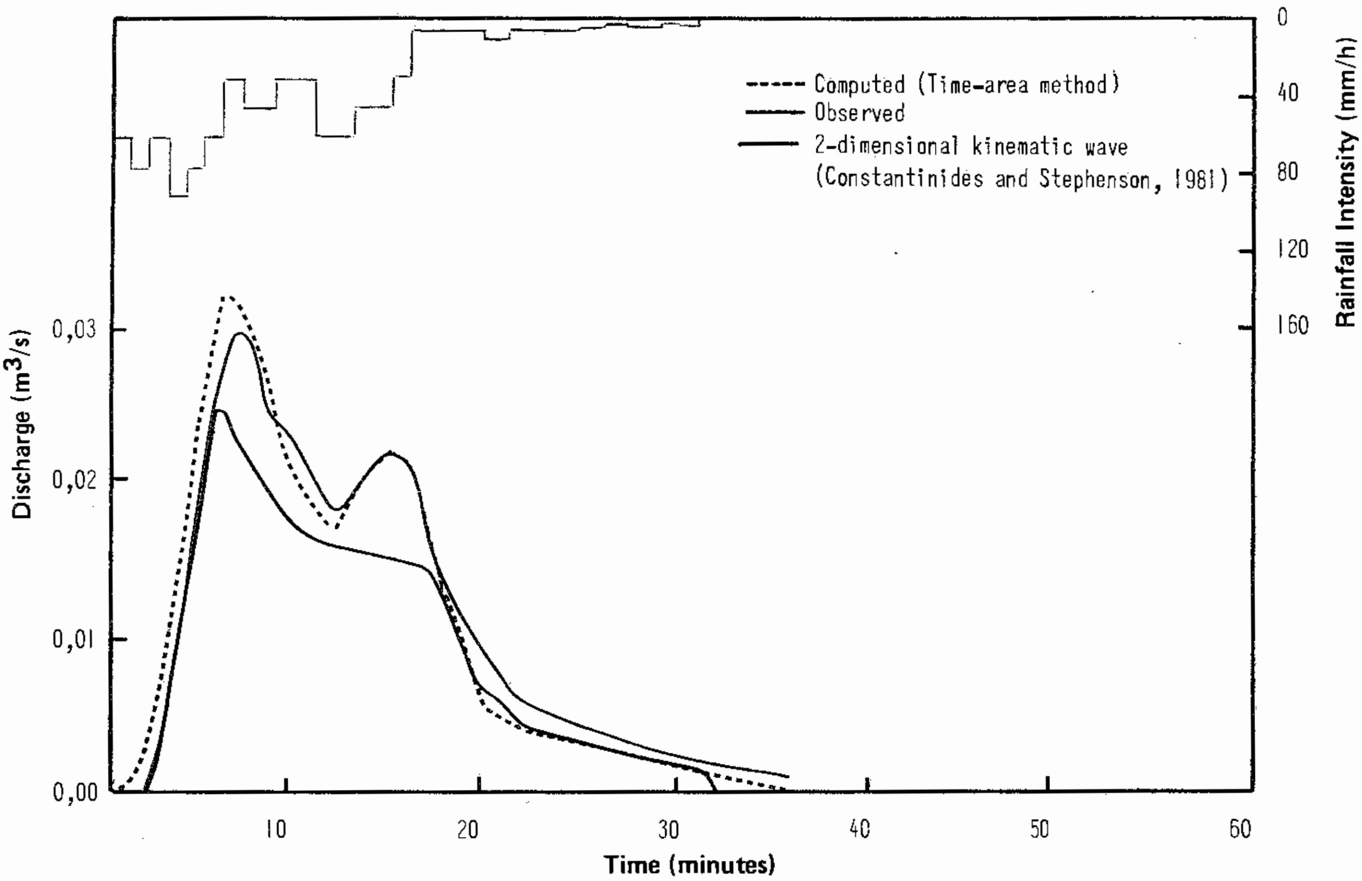

Fig. 4.6 Comparison of computed with observed and kinematic-wave simulated hydrograph for the storm of 9/9/60 on the South Parking Lot catchment

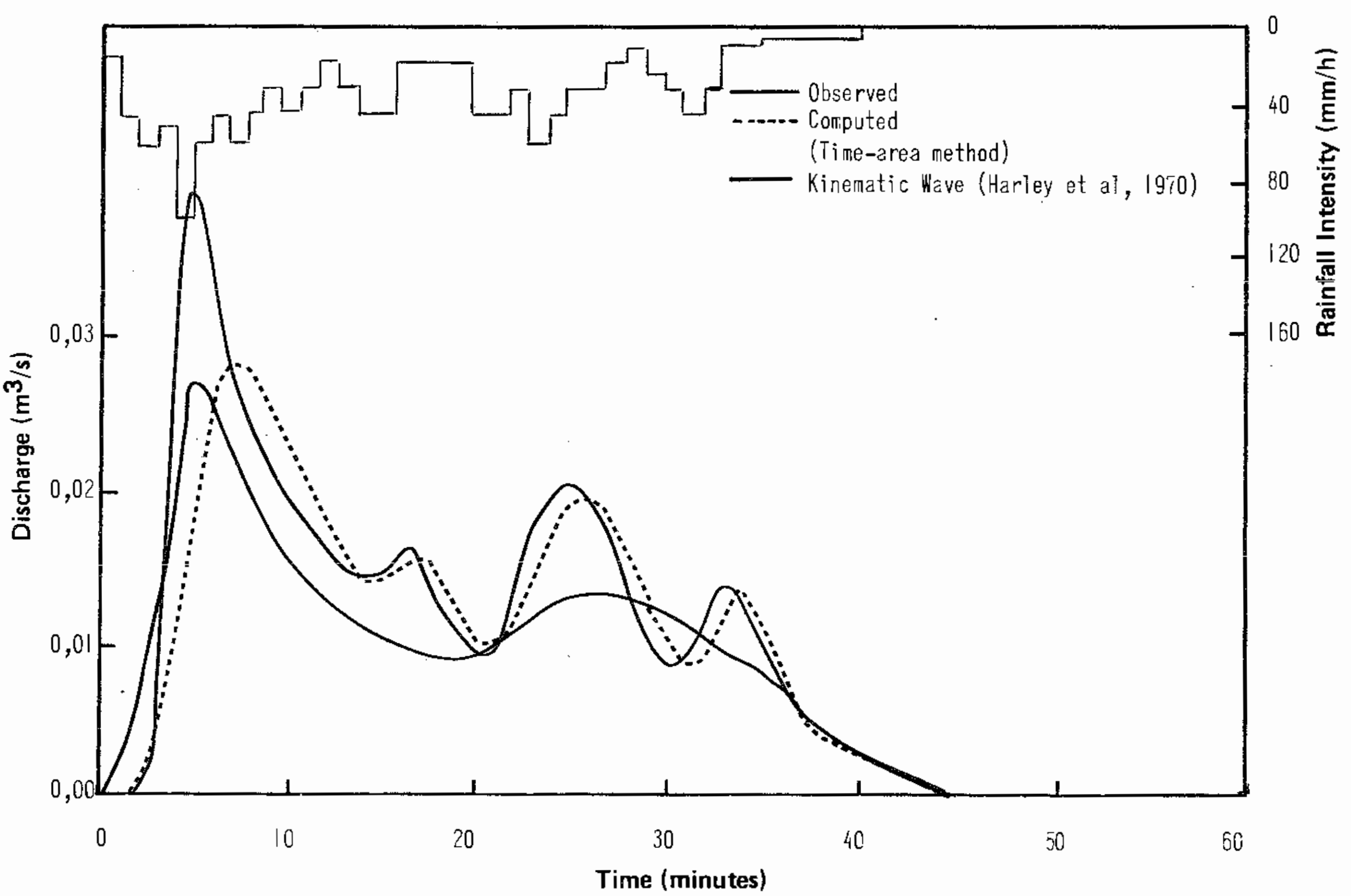

Fig. 4.7 Comparison of computed with observed and kinematic-wave simulated hydrograph for Storm No. 6 on the South Parking Lot catchment

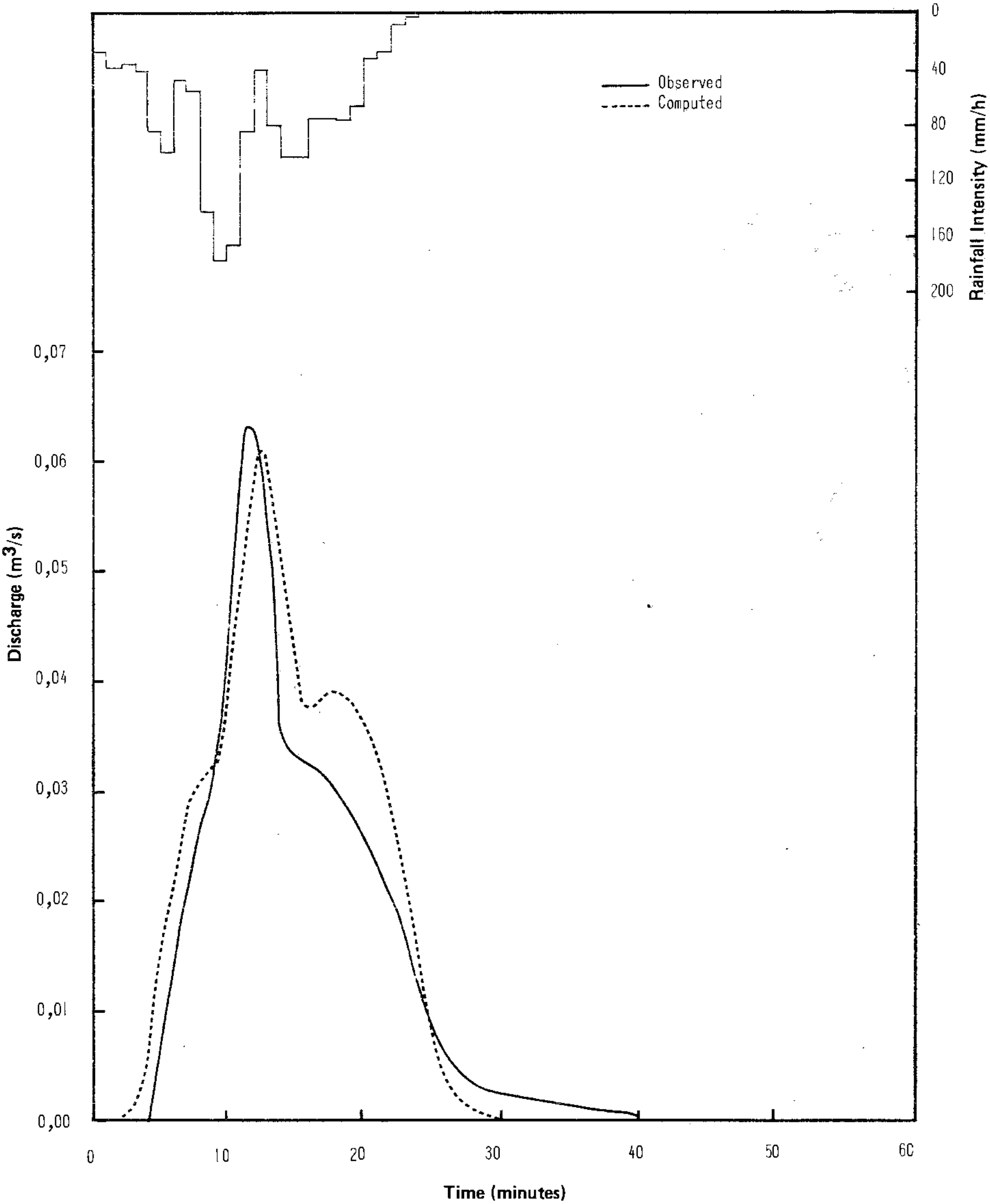

Fig. 4.8 Comparison of computed with observed hydrograph for the storm of 6/8/61 on the South Parking Lot catchment

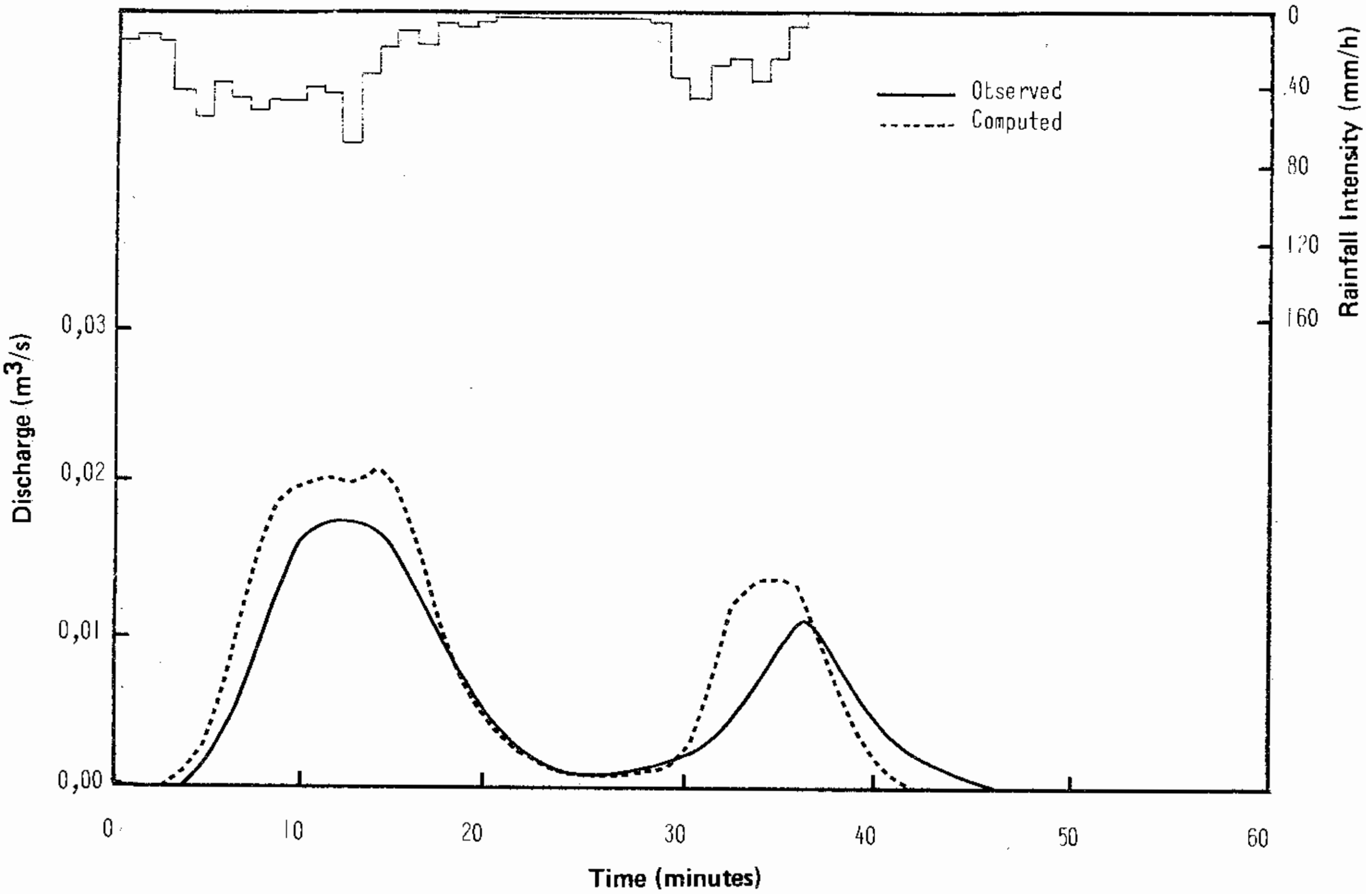

Fig. 4.9 Comparison of computed with observed hydrograph for the storm of 10/8/61 on the South Parking Lot catchment

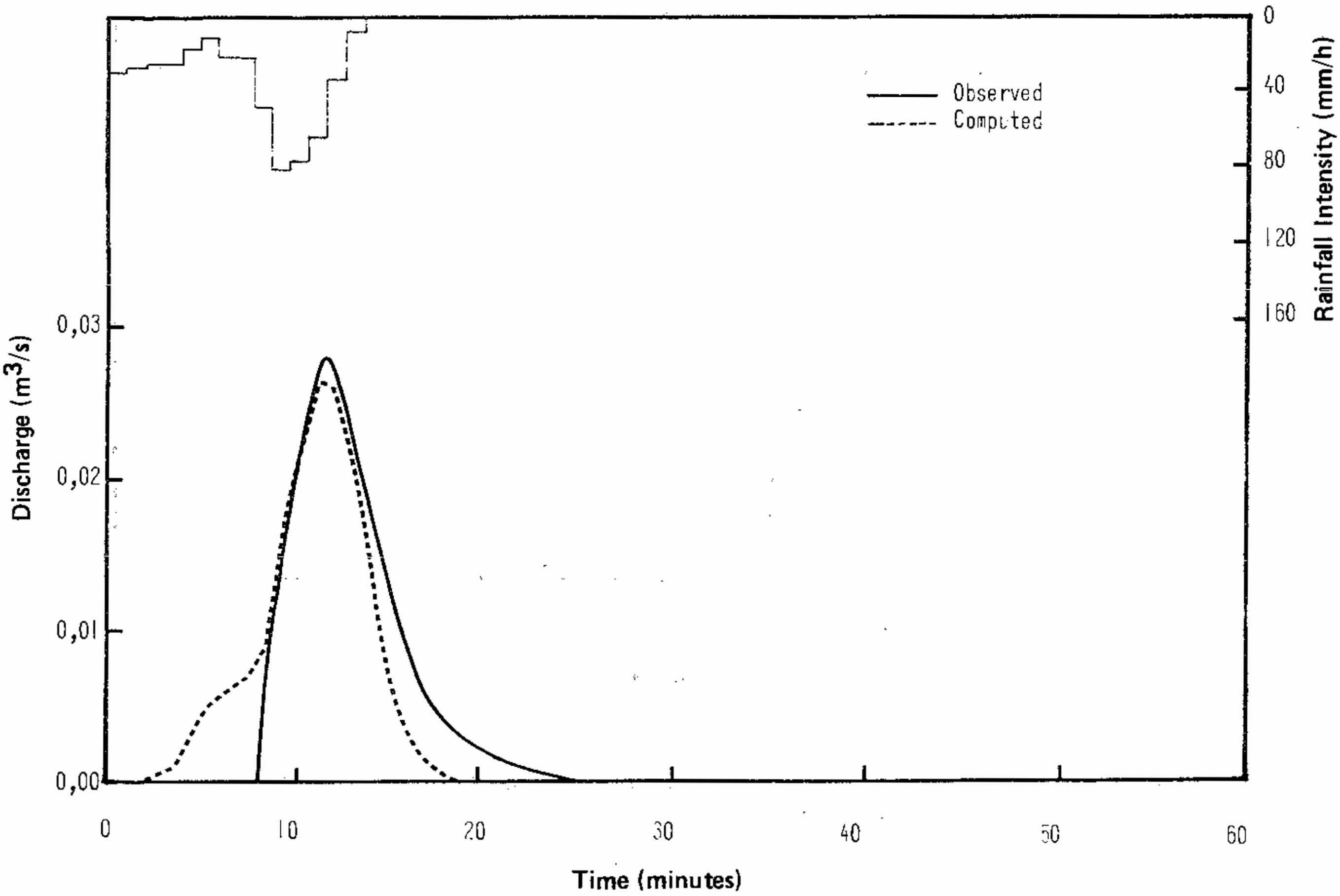

Fig. 4.10 Comparison of computed with observed hydrograph for the storm No. 18 on the South Parking Lot catchment

4.3 Newark Street[1]

A plan and profile of the Newark Street section No. 9 are shown in Fig. 4.11. Like the South Parking Lot catchment this area was gauged as a part of the Storm Drainage Research Project at the Johns Hopkins University. The area of catchment is 0,257 ha, all of which is considered to be impervious. Runoff was estimated from stage measurements in a 230 mm Parshall flume, while rainfall records were obtained from a tipping-bucket gauge, located immediately adjacent to the area, registering every 0,25 mm (0,01") rainfall increment.

For simulation the area was divided into four subcatchments separated from each other by the berm at the change in road slope and the centre-line of the road (Fig. 4.11). An average depression storage of 1 mm was assumed for the whole area. Entry times were computed using eq. 3.2 with a Manning n of 0,02 and an average rainfall intensity of 75 mm/h. Flow width ratios of 1 and 10 were assumed for overland and swale flow respectively. Flow times were estimated assuming full pipe flow velocities and a Manning n of 0,013. Subcatchment data are summarised in Table 4.2 and the computed time-area diagram is shown in Fig. 4.12.

Two rainfall-runoff events by Harley et al (1970) are presented. Computed runoff hydrographs for these events are compared with observed in Figs. 4.13 and 4.14. Hydrographs compare favourably, the average ratio of computed to observed peak discharges being 0,97 and standard deviation 0,02.

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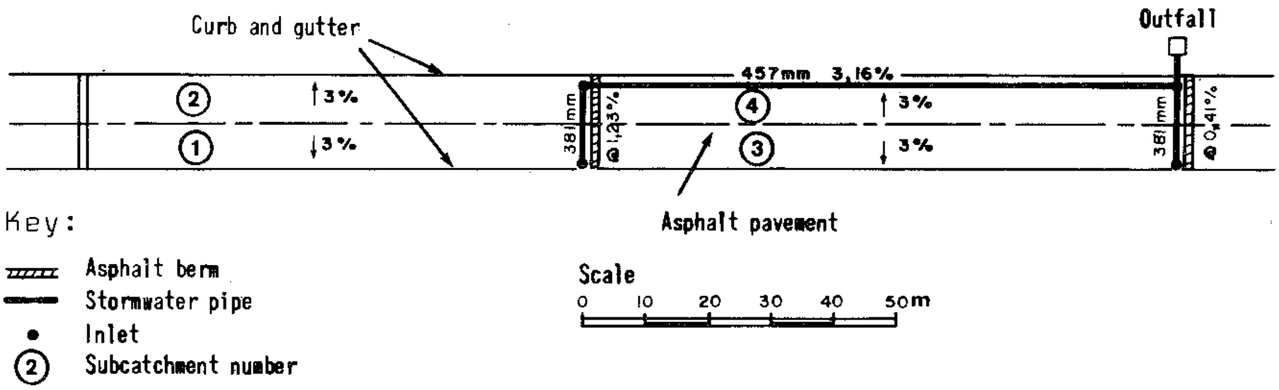
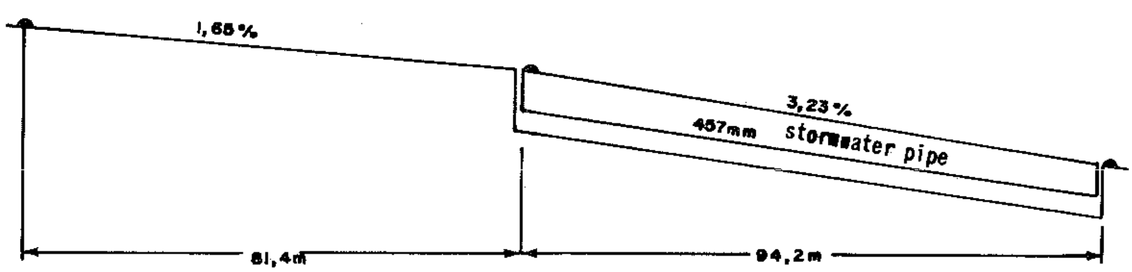
[1] Source of data : Harley, Perkins and Eagleson, 1970



(a) Plan



(b) Profile

Fig. 4.11 Newark Street section No. 9

Table 4.2 Newark Street subcatchment data

Sub-catchment	Area (ha)	Entry time (minutes)	Flow time (minutes)
1	0,0597	3,7	0,7
2	0,0597	3,7	0,5
3	0,0690	4,6	0,2
4	0,0690	4,6	0,0
	0,2574		

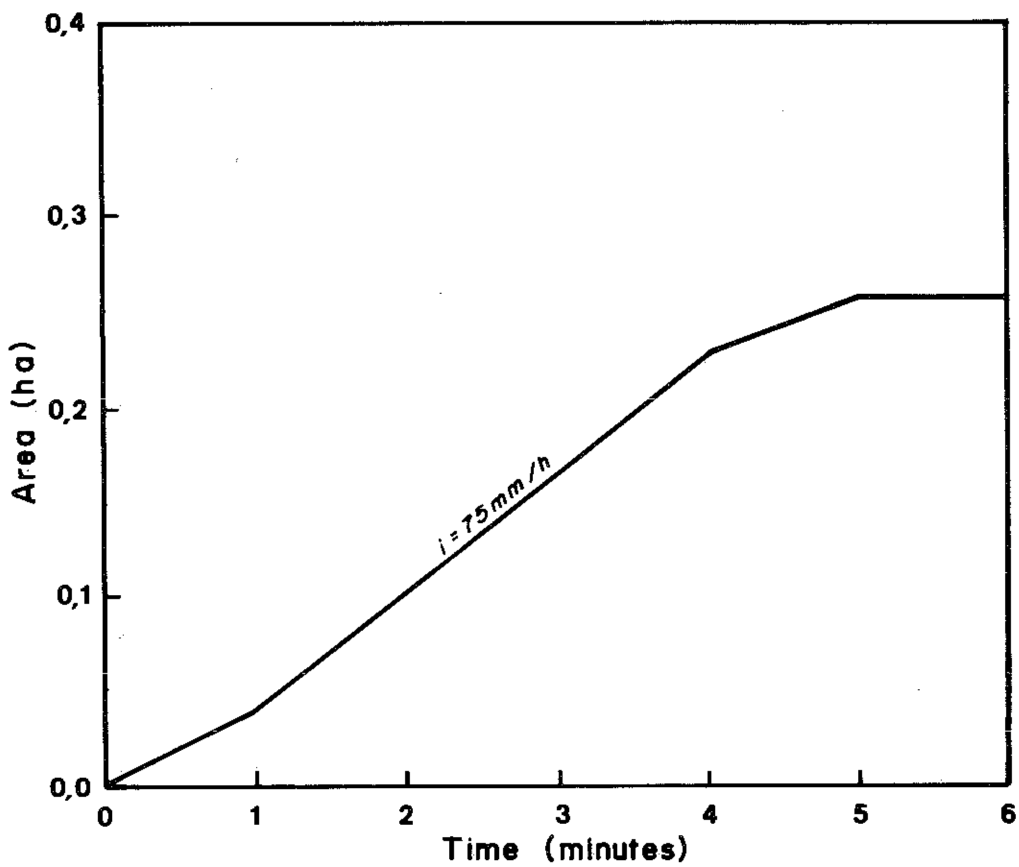


Fig. 4.12 Newark Street time-area diagram

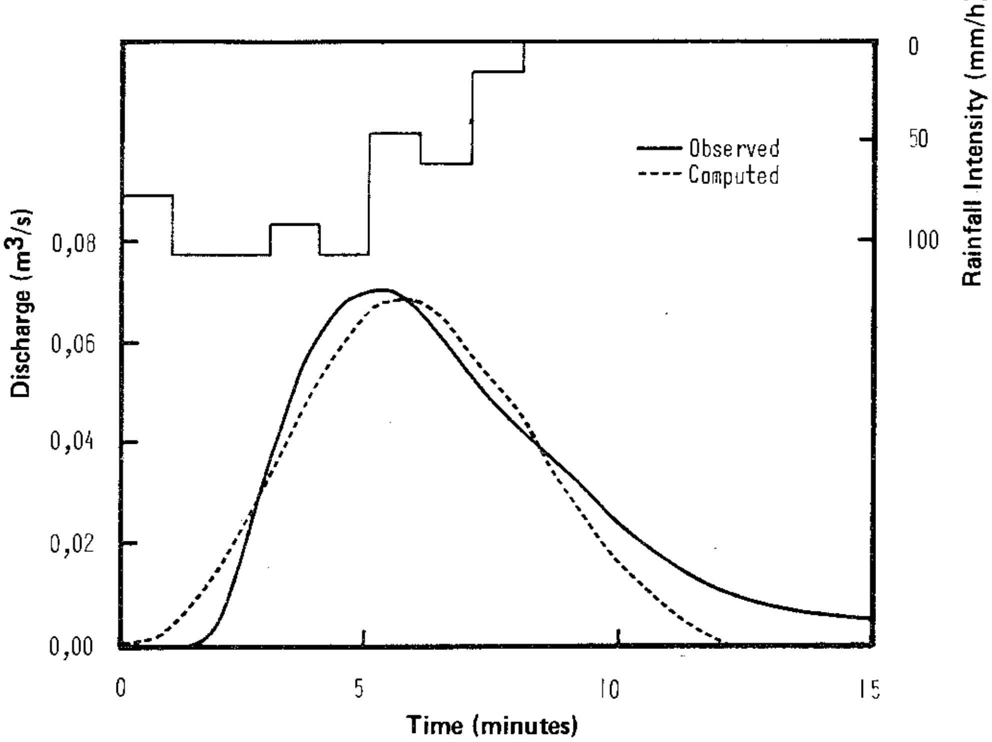


Fig. 4.13 Comparison of computed with observed hydrograph for storm No. 15 on the Newark Street catchment

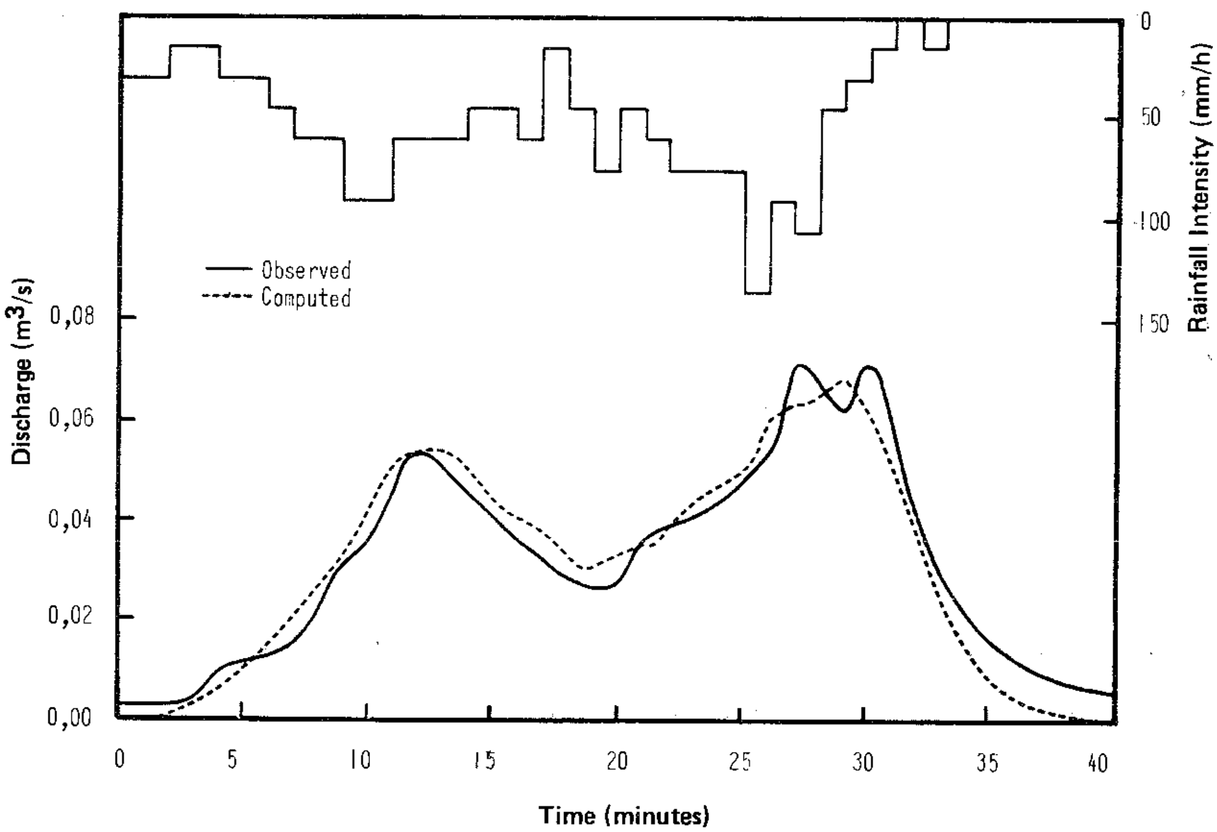


Fig. 4.14 Comparison of computed with observed hydrograph for storm No. 23 on the Newark Street catchment

4.4 Oakdale Avenue[1]

The Oakdale Avenue catchment is located in a residential area of Chicago, USA, and consists entirely of residential lots and adjoining street. The catchment area is 5,22 ha, 39,8% of which is paved and directly connected to the drainage system. A further 5,6% supplements the runoff from unpaved areas. Ground slopes range from 0,4 to 0,9%. Fig. 4.15 is a plan of the catchment showing land use and sewer layout.

Runoff measurements were conducted using a 760 mm parabolic flume located in a vault at the outfall. Rain was measured by means of a tipping-bucket raingauge located on a school roof about one block north of the catchment. Both flow transducer and raingauge were connected to remote recorders over leased telephone lines. Instrumentation operated only during periods of rainfall.

The catchment discretization used by Brandstetter (1976) for verification of SWMM is shown in Fig. 4.16 and for convenience the same discretization has been used here. Subcatchment characteristics are summarized in Table 4.3. Entry times were assumed constant and equal to five and ten minutes for sub-catchment paved and grassed areas respectively. Directly-connected paved area was assumed to be 86% of the whole paved area for each subcatchment. Time-area diagrams for the paved and grassed zones are shown in Fig. 4.17. Loss parameters used by Brandstetter (1976) were adopted, viz. d_{sp} = 2 mm, d_{sg} = 5 mm, f_o = 63,5 mm/h, f_∞ = 11,4 mm/h and k = 4,14 h^{-1}.

The three more intense storms presented by Brandstetter (1976) plus one presented by MacLaren Ltd., 1975 (i.e. 29/4/63) were selected for analysis. One storm comprised of two events separated by 54 minutes and has been considered here as two individual storms. The observed hydrograph for the storm of 2/7/60 is incomplete due to submergence of the measuring flume. The

[1] Source of data: Brandstetter (1976); MacLaren Ltd. (1975).

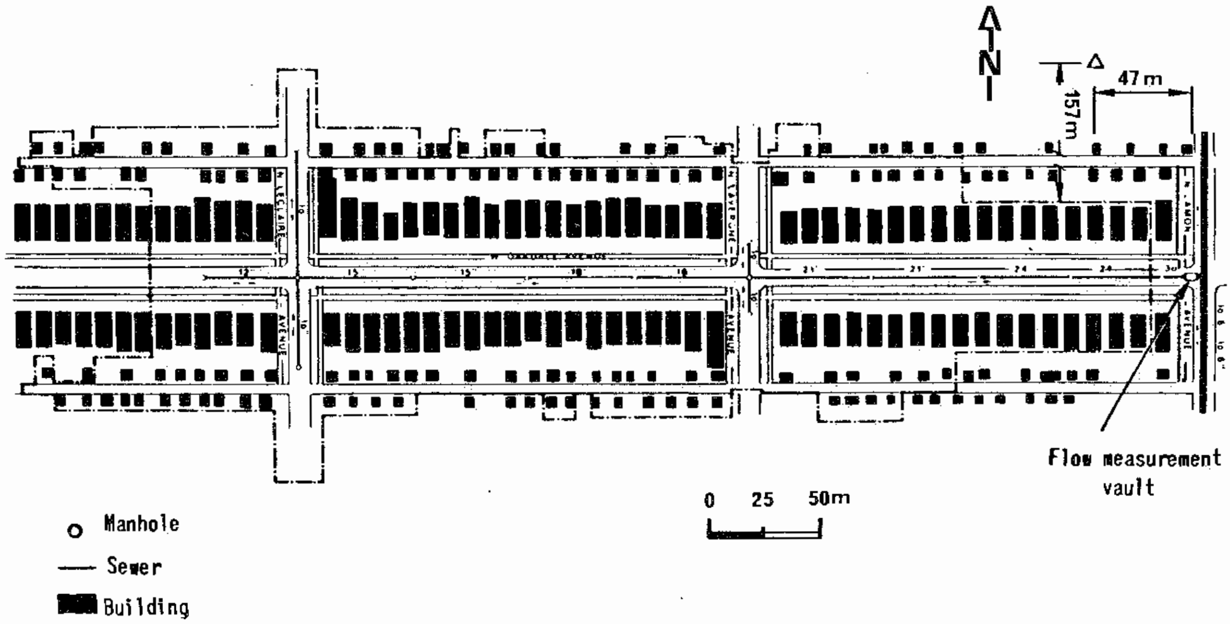


Fig. 4.15 Oakdale Avenue catchment showing land use and sewer layout

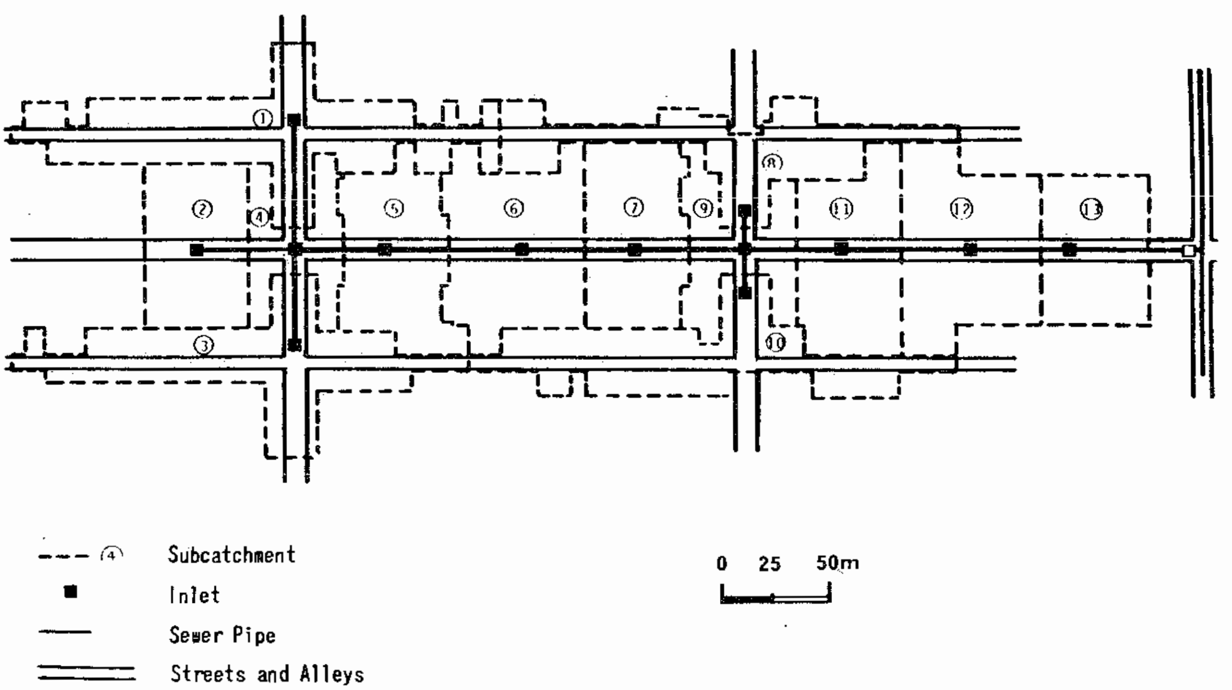


Fig. 4.16 Discretization of the Oakdale Avenue catchment

SWMM simulated hydrograph by Brandstetter (1976) for this event is presented as a basis for comparison (Fig. 4.18). Computed and observed hydrographs for the remaining four events are compared in Figs. 4.19 to 4.22.

The computed hydrograph for the larger runoff event, i.e. that on 2/7/60, compares favourably with both observed and SWMM-simulated hydrographs. Computed hydrographs for the remaining events are reasonable but it seems that surface detention is underestimated and longer travel times would be appropriate. The average ratio of computed to observed peak discharge is 1,11 with a standard deviation of 0,15.

Table 4.3 Oakdale Avenue subcatchment data

Sub-catchment	Total paved area (ha)	Grassed area (ha)	Flow time (minutes)
1	0,285	0,363	6,1
2	0,150	0,190	5,8
3	0,268	0,250	5,7
4	0,112	0,090	5,3
5	0,152	0,233	4,6
6	0,199	0,278	3,8
7	0,149	0,219	3,2
8	0,194	0,166	2,6
9	0,135	0,104	2,5
10	0,226	0,251	2,6
11	0,156	0,228	1,8
12	0,199	0,291	1,1
13	0,143	0,189	0,6
	2,368	2,852	

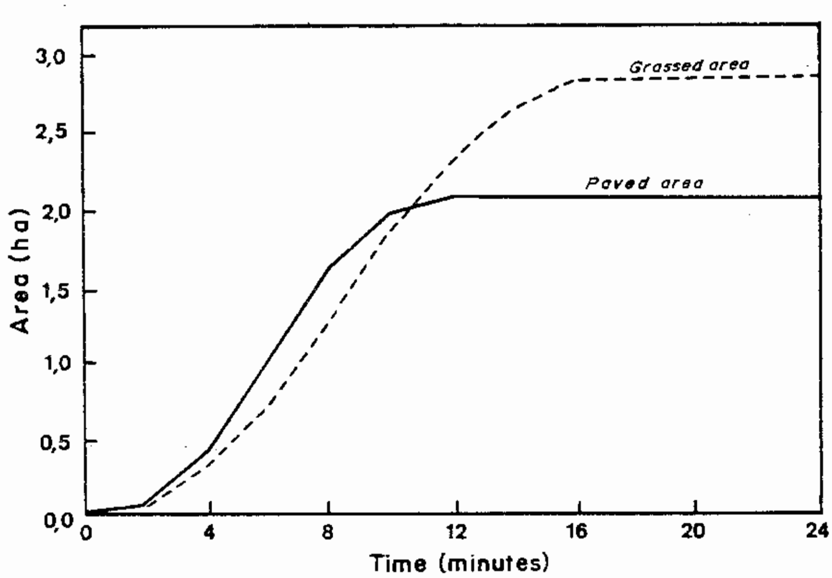


Fig. 4.17 Oakdale Avenue time-area diagram

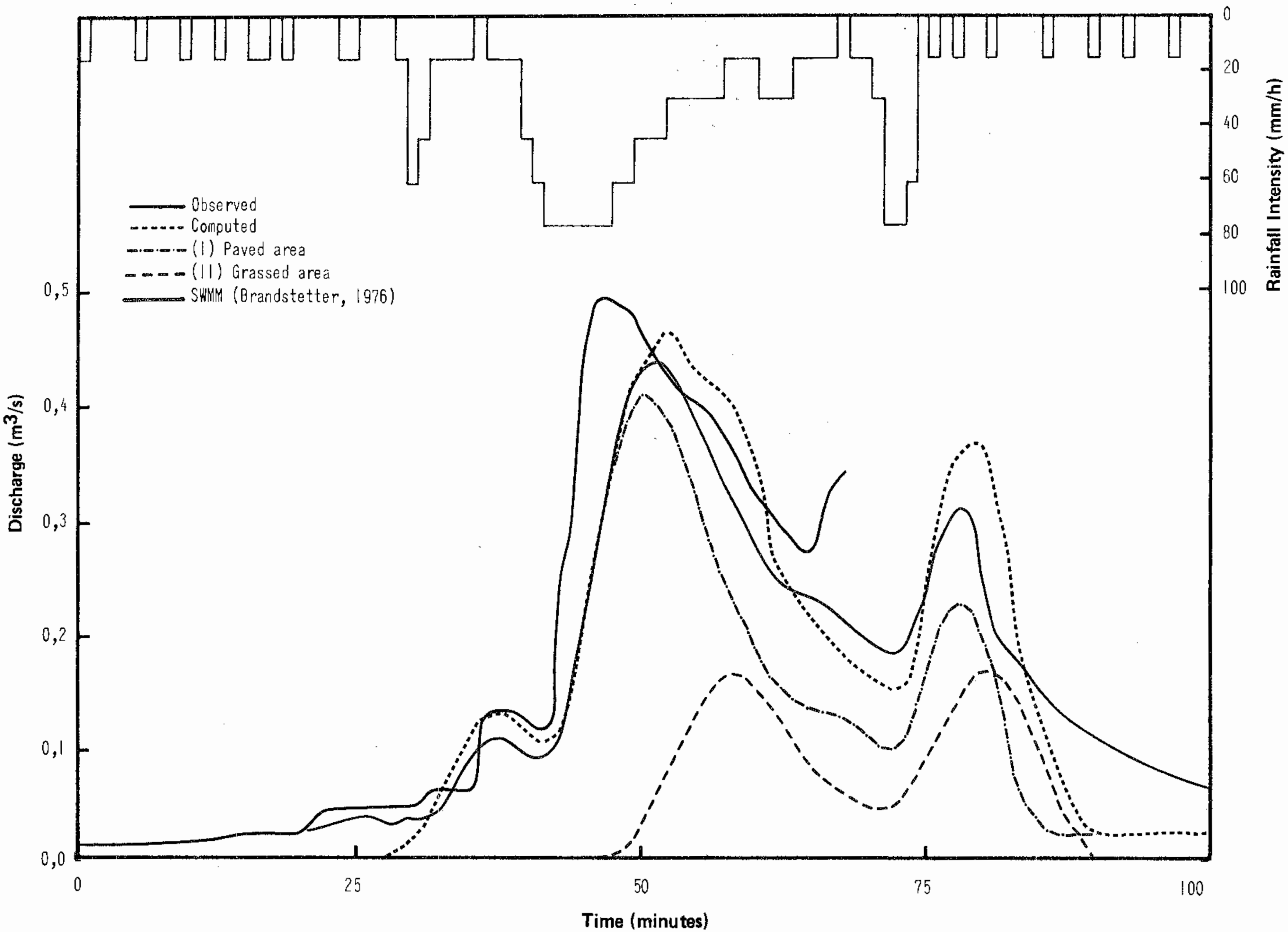


Fig. 4.18 Comparison of computed with observed and with SWMM-simulated hydrograph for the storm of 2/7/60 on the Oakdale Avenue catchment

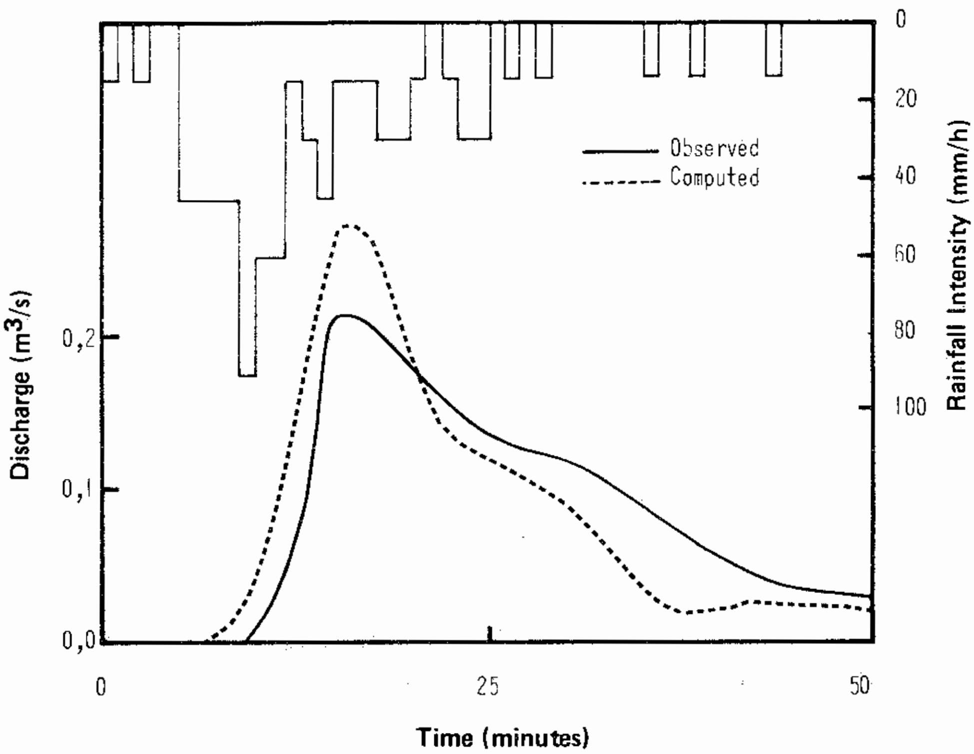


Fig. 4.19 Comparison of computed with observed hydrograph for the storm of 19/5/59 on the Oakdale Avenue catchment

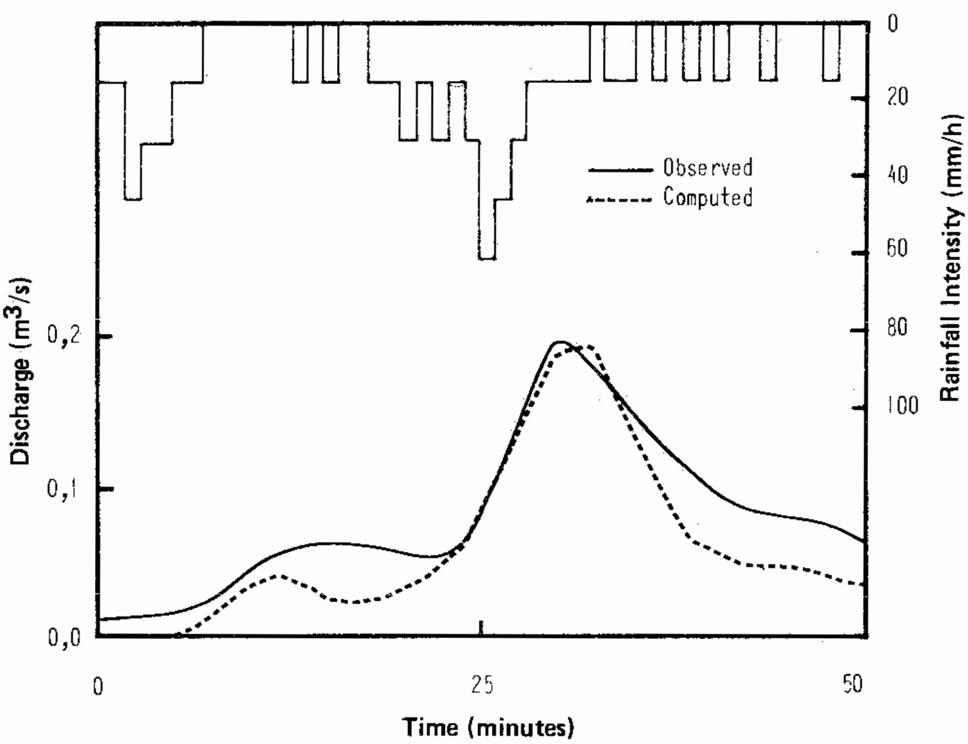


Fig. 4.20 Comparison of computed with observed hydrograph for the storm of 29/4/63 on the Oakdale Avenue catchment

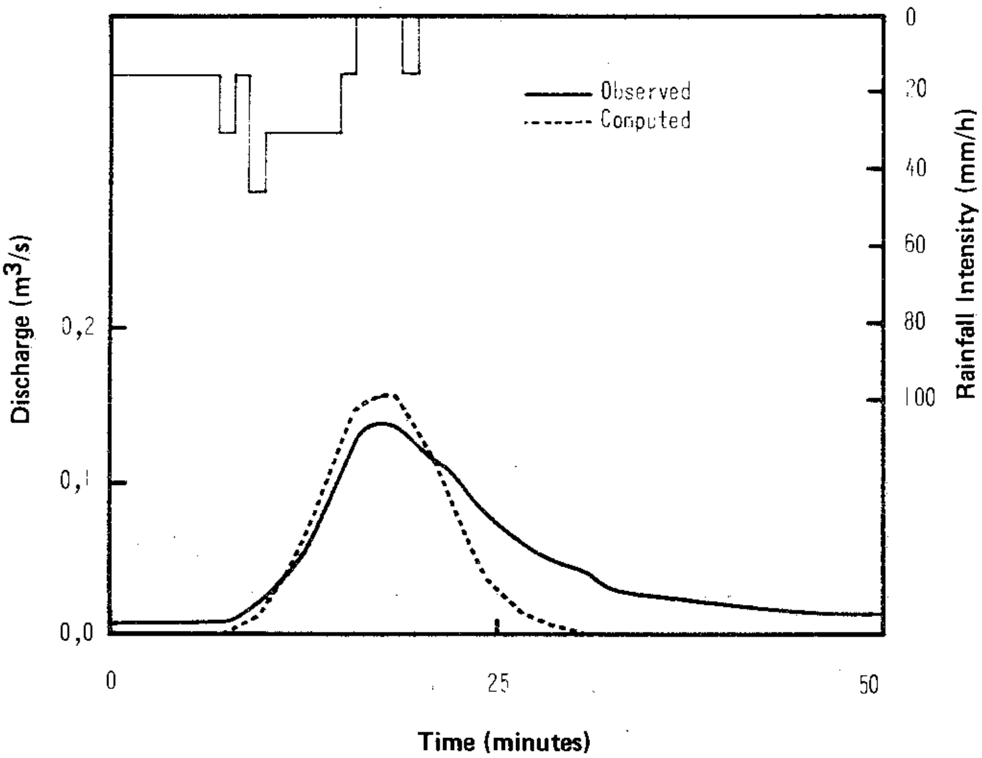


Fig. 4.21 Comparison of computed with observed hydrograph for the storm of 2/8/63 (1) on the Oakdale Avenue catchment

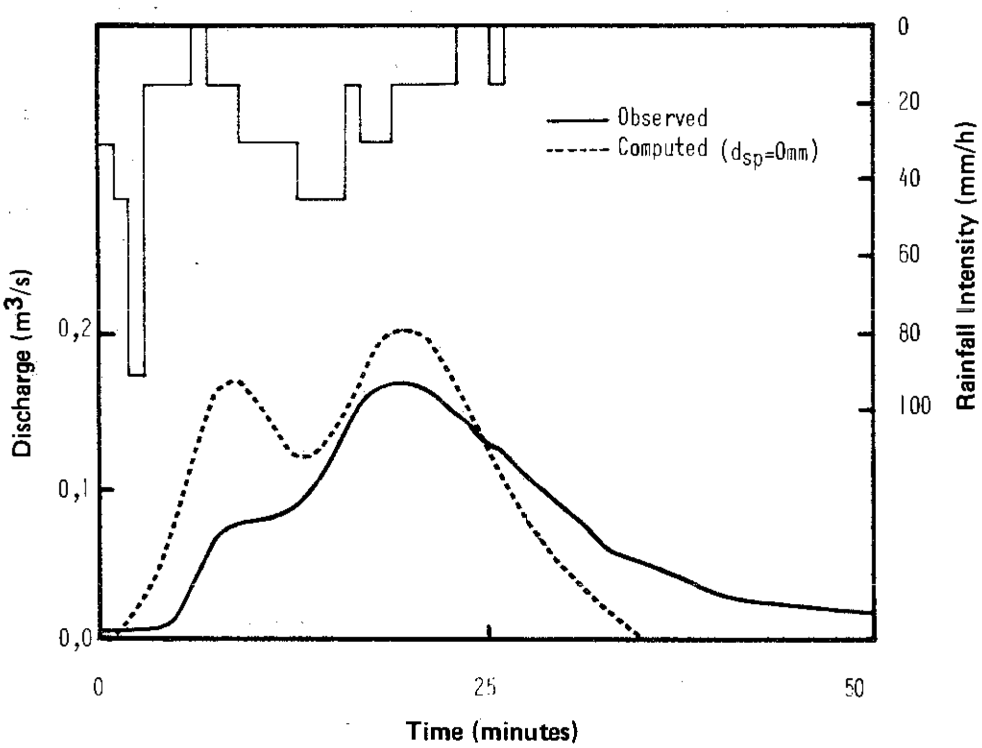


Fig. 4.22 Comparison of computed with observed hydrograph for the storm of 2/8/63 (2) on the Oakdale Avenue catchment

4.5 Gray Haven[1]

The Gray Haven catchment is a homogeneous residential area of 9,43 ha in Baltimore, U S A. The total area of paved surface is 4,90 ha (52%) of which 4,17 ha (44%) is directly connected to the drainage system. Ground slopes are gentle, averaging about 0,5%. The soils are generally of the U S Sassafras series and are classified as hydrological soil type B. Fig. 4.23 is a plan of the catchment with a schematic diagram of the drainage system.

Stage measurements at a Parshall flume at the outfall were recorded synchronously with rainfall measurements from a nearby tipping-bucket gauge. Data for three events were available from the quoted sources.

The distribution of paved area within the catchment was not described in the quoted sources. Linear time-area diagrams were therefore assumed for both the paved and the grassed zones. The time bases of these diagrams were computed by assuming entry times of 5 and 10 minutes for the paved and grassed areas respectively. Pipe flow velocities were assumed equal to 2 m/s and a flow time of 3 minutes was obtained for flow from the top of the catchment. Runoff from the grassed area was assumed to flow on to the paved area before entering the drainage system. The time bases computed in this simple fashion were 8 minutes for the paved area and 18 minutes for the grassed area.

Depression storage was assumed equal to 1 mm for the paved area and 5 mm for the grassed area. The infiltration parameters given in Table 3.1 for soil type B were used. AMC values were available for two of the three storms, viz. AMC = 3 for the storm of 1/8/63 and AMC = 2 for the storm of 14/8/63. For the remaining storm (14/6/63) an AMC of 2 was assumed. A computational time increment of 1 minute was used for calculation of runoff from paved areas and 2 minutes for that from grassed areas.

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[1] Sources of data: MacLaren Ltd., 1975;
Patry _et al._, 1979;
Terstriep and Stall, 1974.

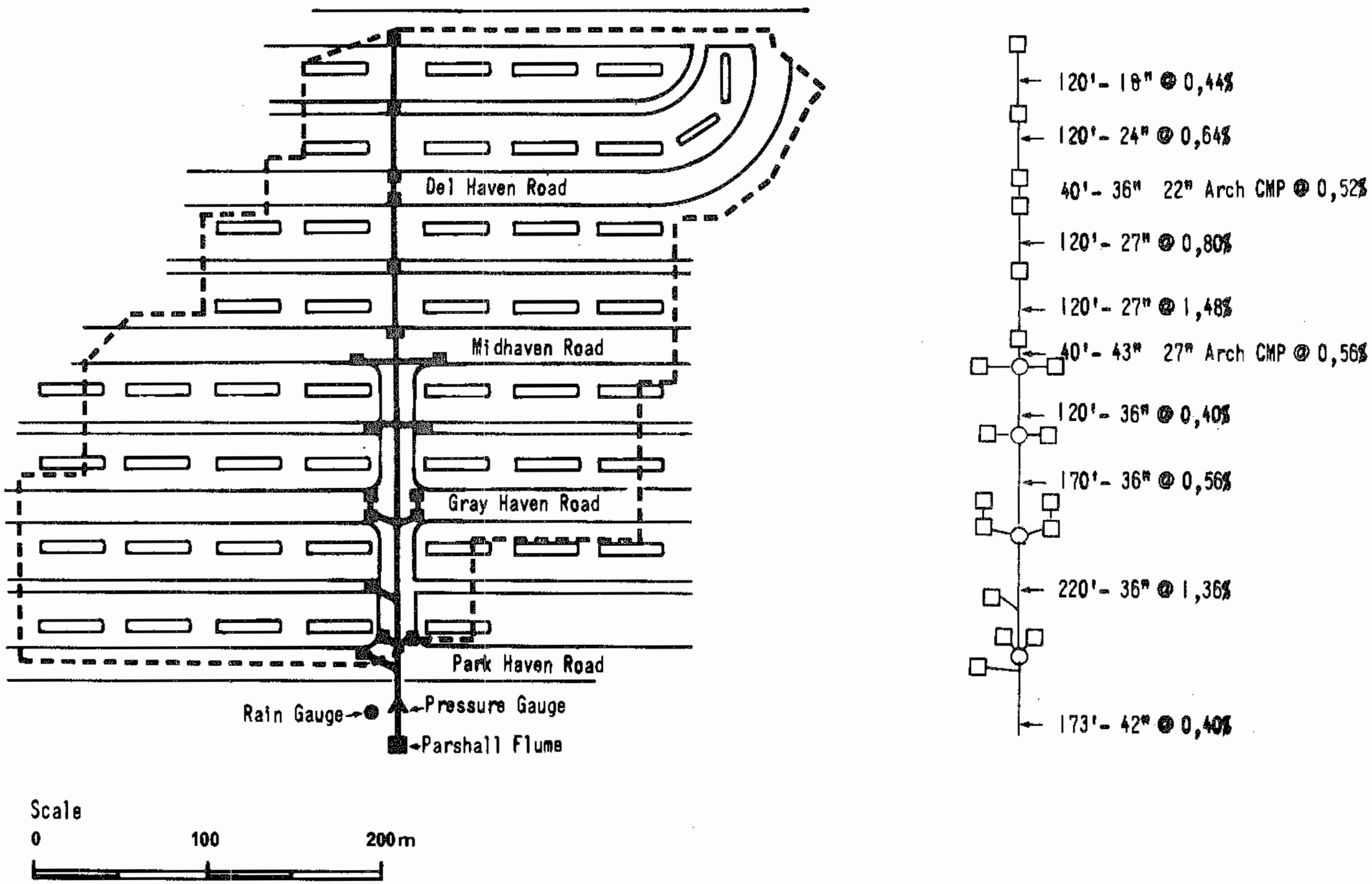

Fig. 4.23 Gray Haven catchment, Baltimore

Computed and observed hydrographs for the three events are compared in Figs. 4.24 to 4.26. The results are fair and could no doubt be improved if more data were available for constructing the time area diagrams. The average ratio of computed to observed peak is 0,91 with a standard deviation of 0,11.

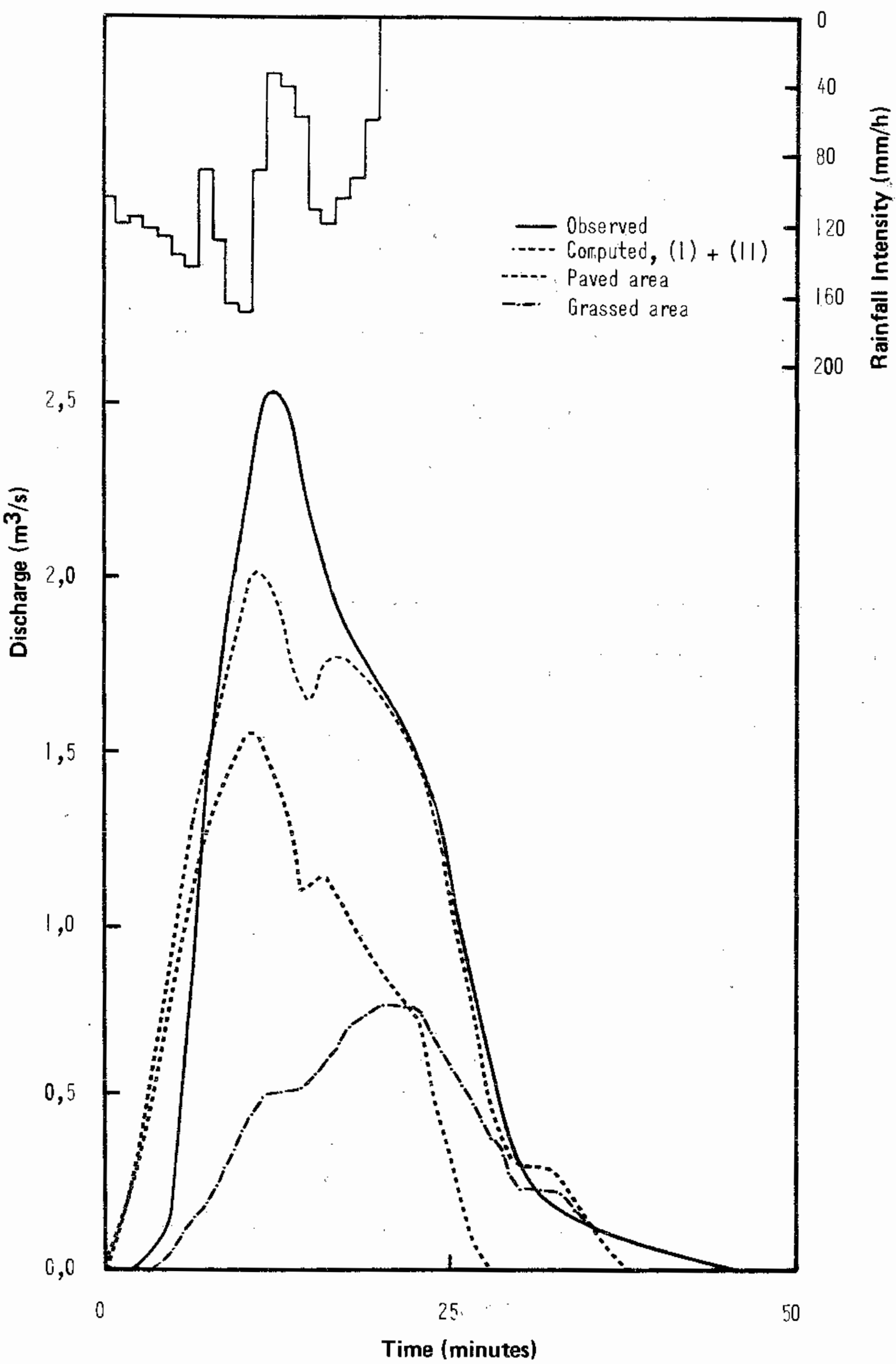

Fig. 4.24 Comparison of computed with observed hydrograph for the storm of 1/8/63 on the Gray Haven catchment

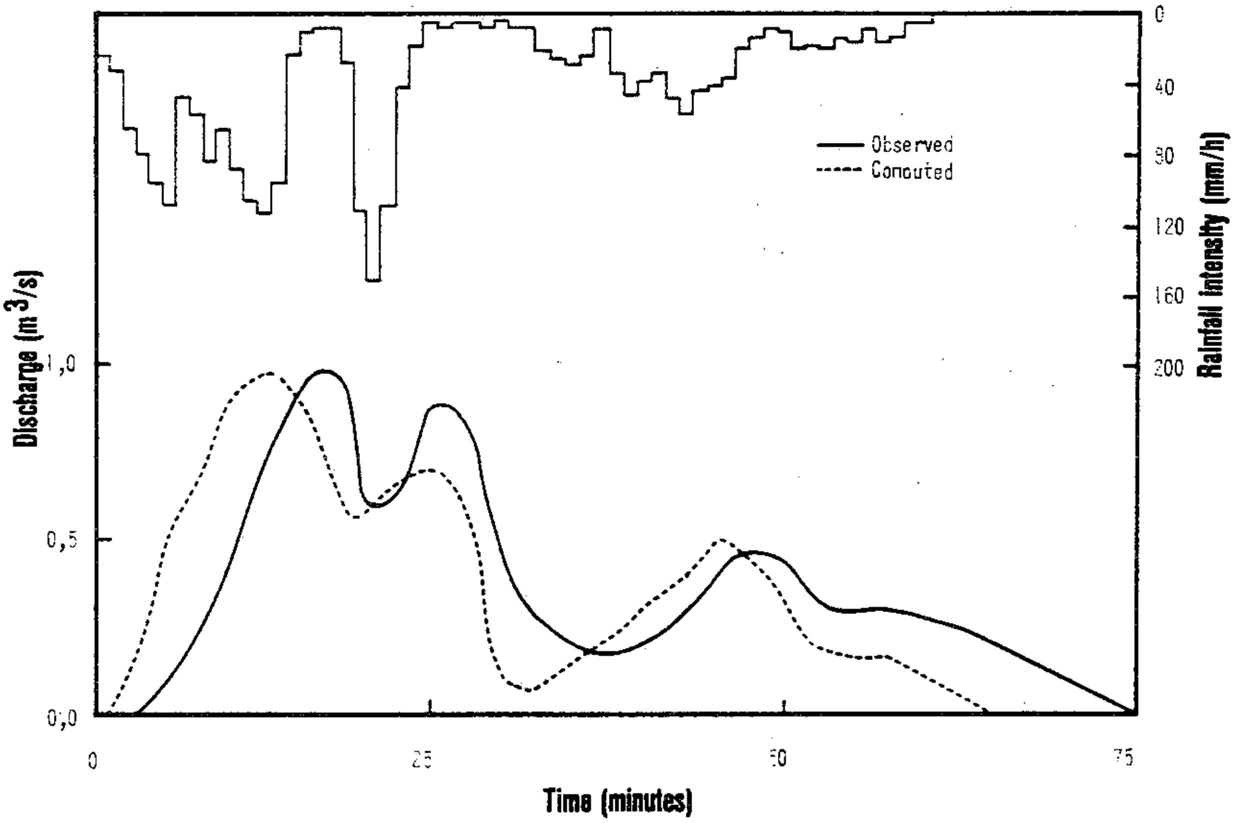

Fig. 4.25 Comparison of computed with observed hydrograph for the storm of 14/6/63 on the Gray Haven catchment

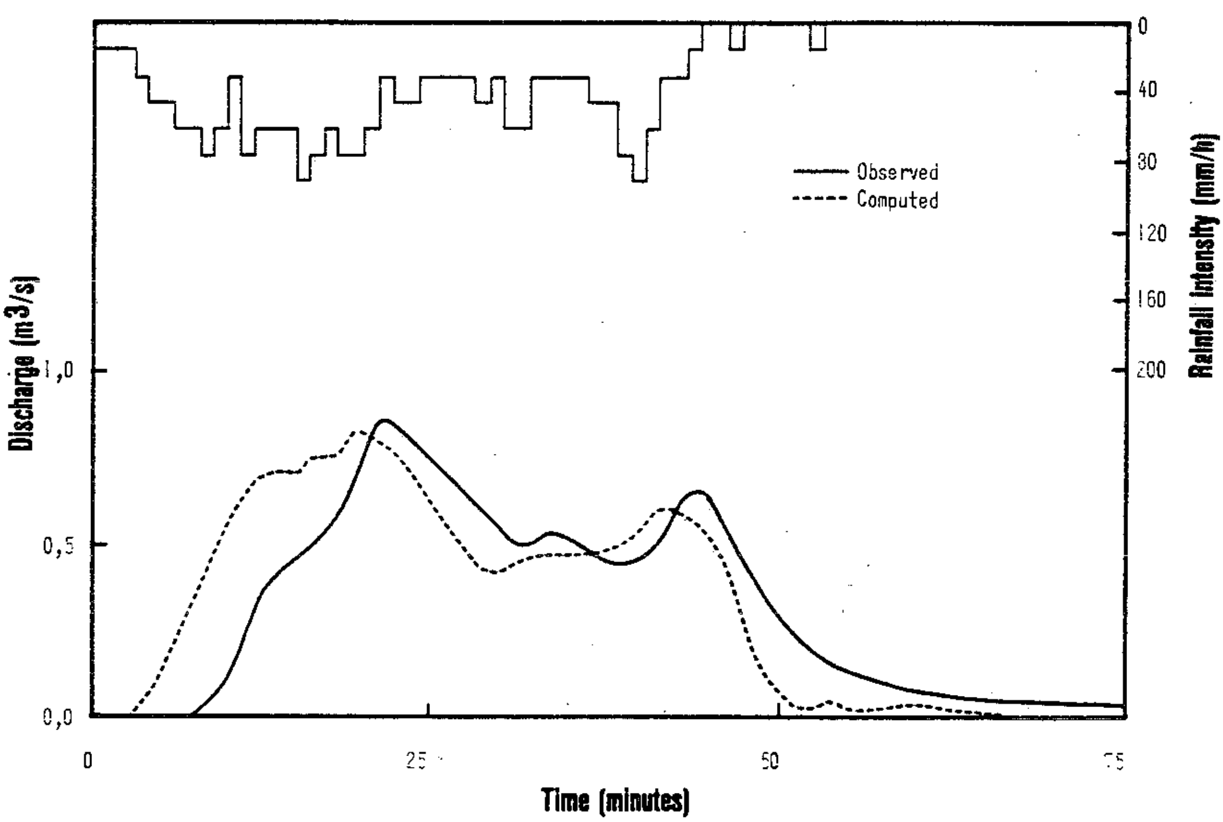

Fig. 4.26 Comparison of computed with observed hydrograph for the storm of 14/8/63 on the Gray Haven catchment

4.6 Pinetown[1]

The catchment is situated in the shopping centre of Pinetown, approximately 20 km inland from Durban, and is monitored by the National Institute for Water Research (NIWR) Durban, South Africa. Fig. 4.27 is a typical view of the catchment while Fig. 4.28 is a plan of the area showing the boundaries and the stormwater drainage system. The total area is 11,9 ha of which 9,0 ha (75%) is directly-connected impervious surface, comprising roads, sidewalks, car parks, office blocks and shopping complexes. The remaining area comprises lawns, unpaved parking areas and small buildings that discharge on to pervious areas. The ground slopes are moderately steep (up to 5%); approximate ground level contours are shown in Fig. 4.28. The soils are sandy.

Fig. 4.27 A typical view of the Pinetown catchment (looking up Crompton Street from raingauge no. 2)

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[1] Sources of data: Simpson et al., 1980;
Simpson, 1981;
Watson, 1981a.

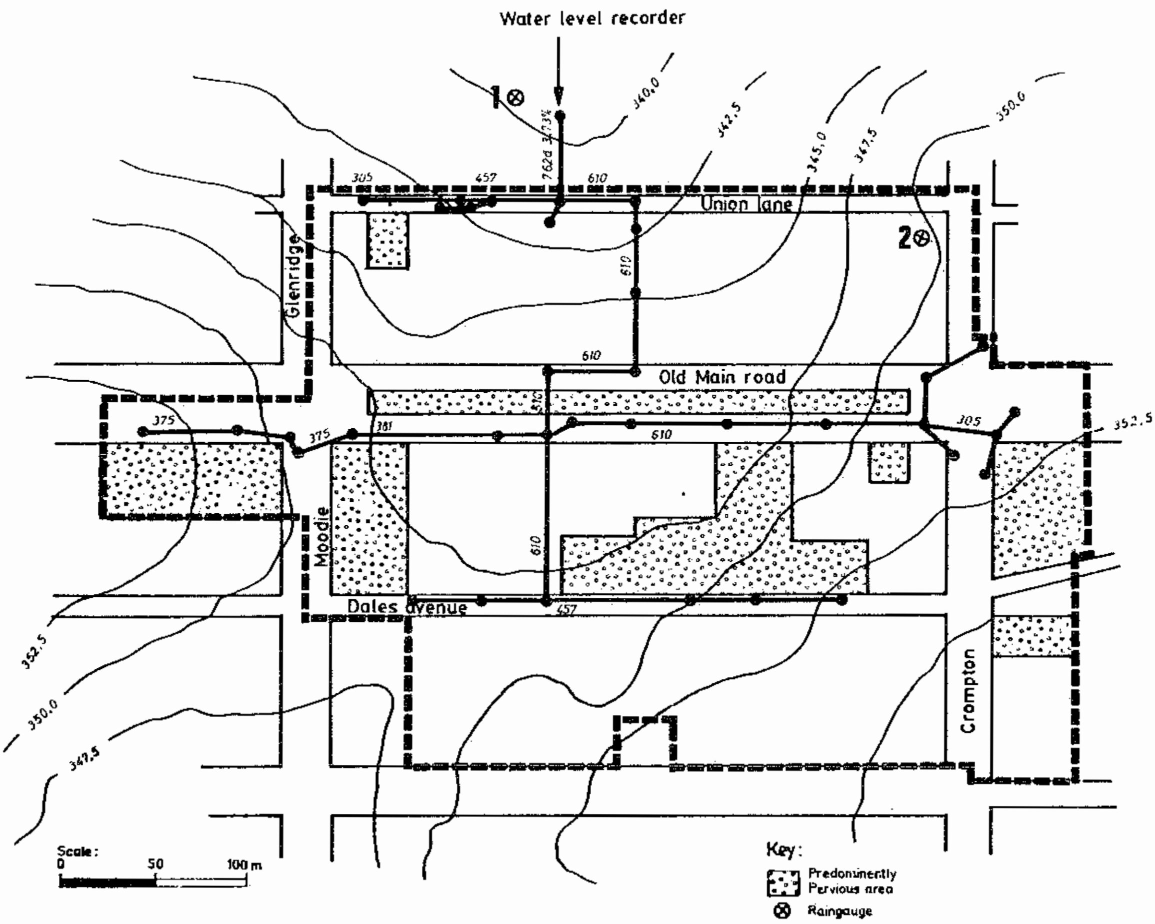

Fig. 4.28 Plan of the Pinetown catchment

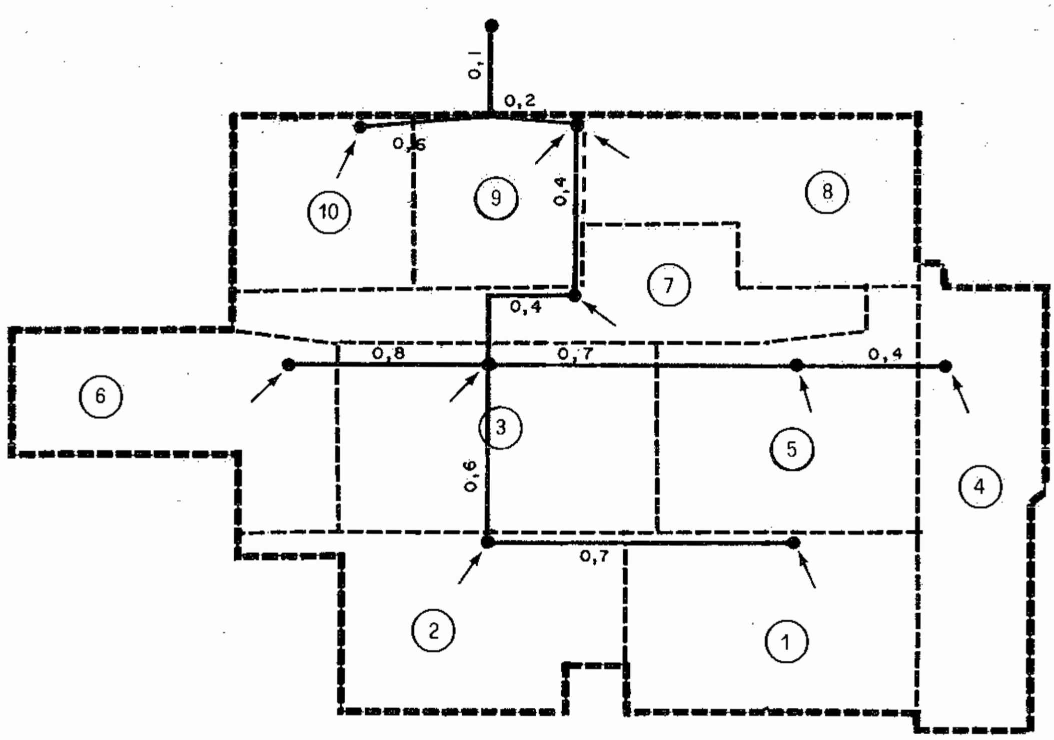

Fig. 4.29 Discretization of the Pinetown catchment showing reach travel times (minutes)

Rainfall was measured by two Casella siphon recorders, one located within the catchment and the other immediately beyond the boundary near the outfall (Fig. 4.28). Water level was measured in the outfall pipe by a Wesmar ultrasonic level detector and rated by salt dilution gauging. The rainfall and runoff data at the outfall were recorded on a punched tape. The raingauge within the catchment recorded rainfall depth on a weekly drum chart and was used to correct rainfall recorded at the outfall. The average total depth was accepted.

The paved and grassed areas were assumed to have average depression storages of 1 mm and 5 mm respectively. Soils were classed as type B and assigned the relevant infiltration parameters from Table 3.1. The supplementary paved area is not significant and was considered as part of the grassed area. For the events considered no grassed-area runoff was computed.

The catchment was discretized into ten subcatchments as shown in Fig. 4.29 and described in Table 4.4. Paved-area entry time was assumed to be 5 minutes for all subcatchments. The time-area diagram for the paved area is shown in Fig. 4.30. A computational time interval of 2 minutes was used for all events except the storm of 4/11/79 for which a 1 minute interval was used.

The three storms used in HRU Report 1/81 (Watson, 1981a), plus another two for which data were made available by the NIWR during 1981, were selected for analysis. These storms represent the more severe of the recorded storms on this catchment during the study period.

Comparisons of computed with observed hydrographs are shown in Figs. 4.31 to 4.35. The results are good; computed hydrographs follow the shapes of the observed hydrographs well and the average ratio of computed to observed peak discharge is 1,12, with a standard deviation of 0,15.

For the two events on the 22/5/79 depression storage was considered to have been completely filled by prior rainfall. If partial depletion of storage space had been assumed the

Table 4.4 Pinetown subcatchment data

Sub-catchment	Paved area (ha)	Grassed area (ha)	Flow time (minutes)
1	1,37	nil	2,4
2	1,26	nil	1,7
3	1,15	0,43	1,1
4	0,99	0,43	2,2
5	0,86	0,50	1,8
6	0,30	0,89	1,9
7	0,60	0,42	0,7
8	1,14	0,08	0,3
9	0,73	nil	0,3
10	0,60	0,19	0,7
	9,00	2,94	

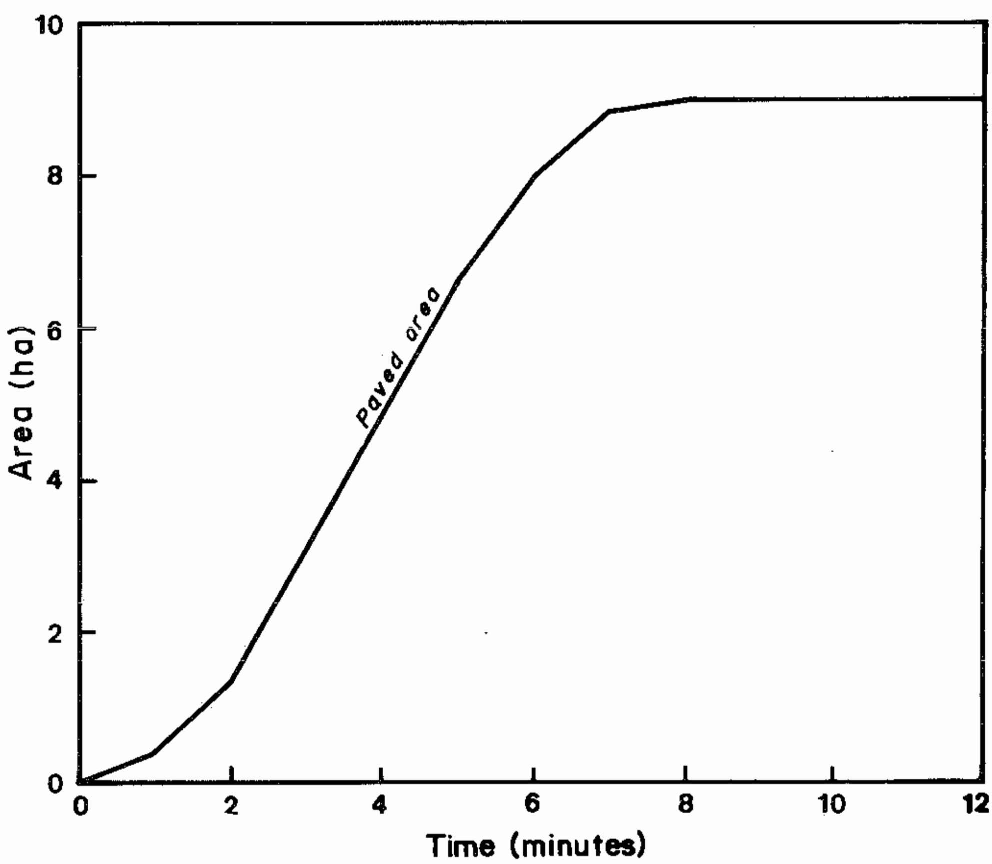

Fig. 4.30 Pinetown time-area diagram

results would have been noticeably improved. This suggests that depression storage is regenerated through slow outflows from surface ponding. For the first event, antecedent rainfall was only slightly larger than the average depression storage and could therefore not completely fill the depression storage where this was larger than the average.

For the second event on the 22/5/79 the low magnitude peaks during the earlier part of the storm are overestimated. This is due to an underestimation of surface detention and can be corrected by increasing flow travel times. The blue line shown in Fig. 4.33 was computed after doubling of the travel times. This corresponds to a rainfall intensity ratio of 0,2 in Fig. 3.3.

The discrepancy for the event of 29/9/79 cannot be explained in this fashion. The volume is overestimated and this could be due to rainfall sampling errors.

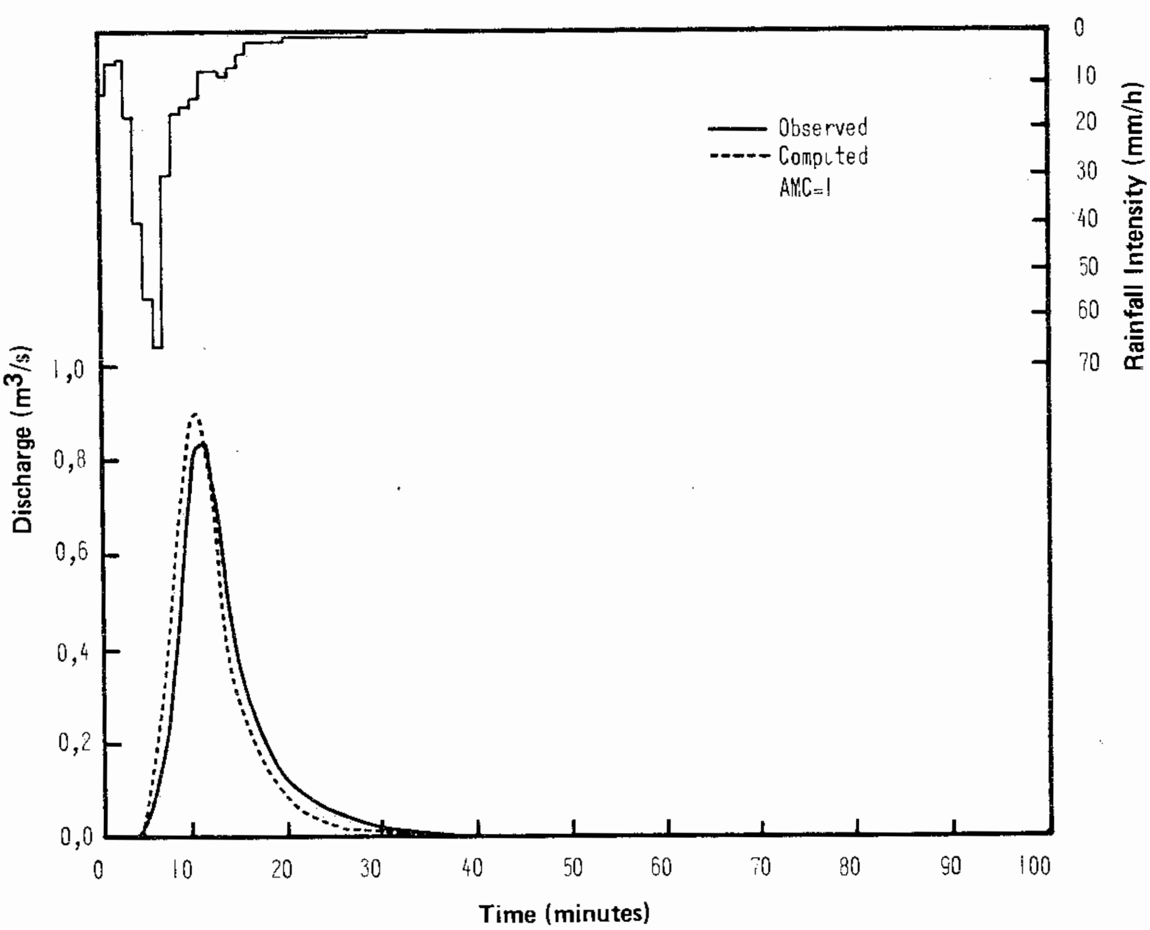

Fig. 4.31 Comparison of computed with observed hydrograph for the storm of 4/11/79 on the Pinetown catchment

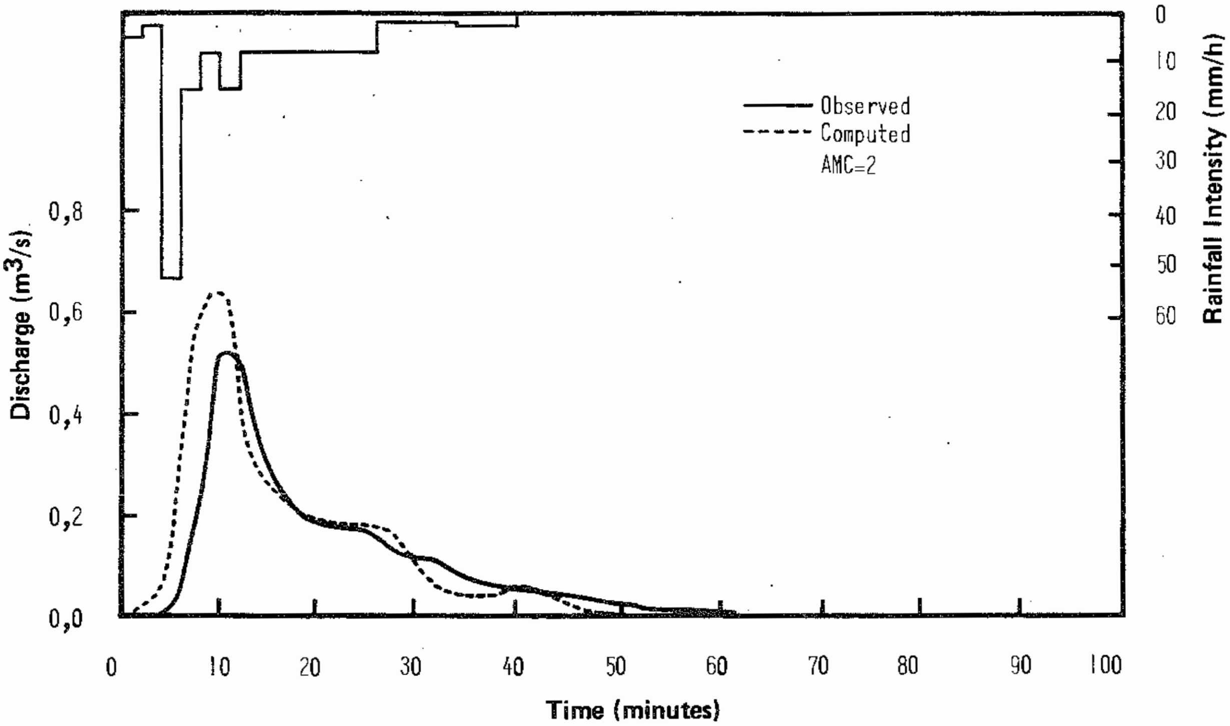

Fig. 4.32 Comparison of computed with observed hydrograph for the storm of 22/5/79 (1) on the Pinetown catchment

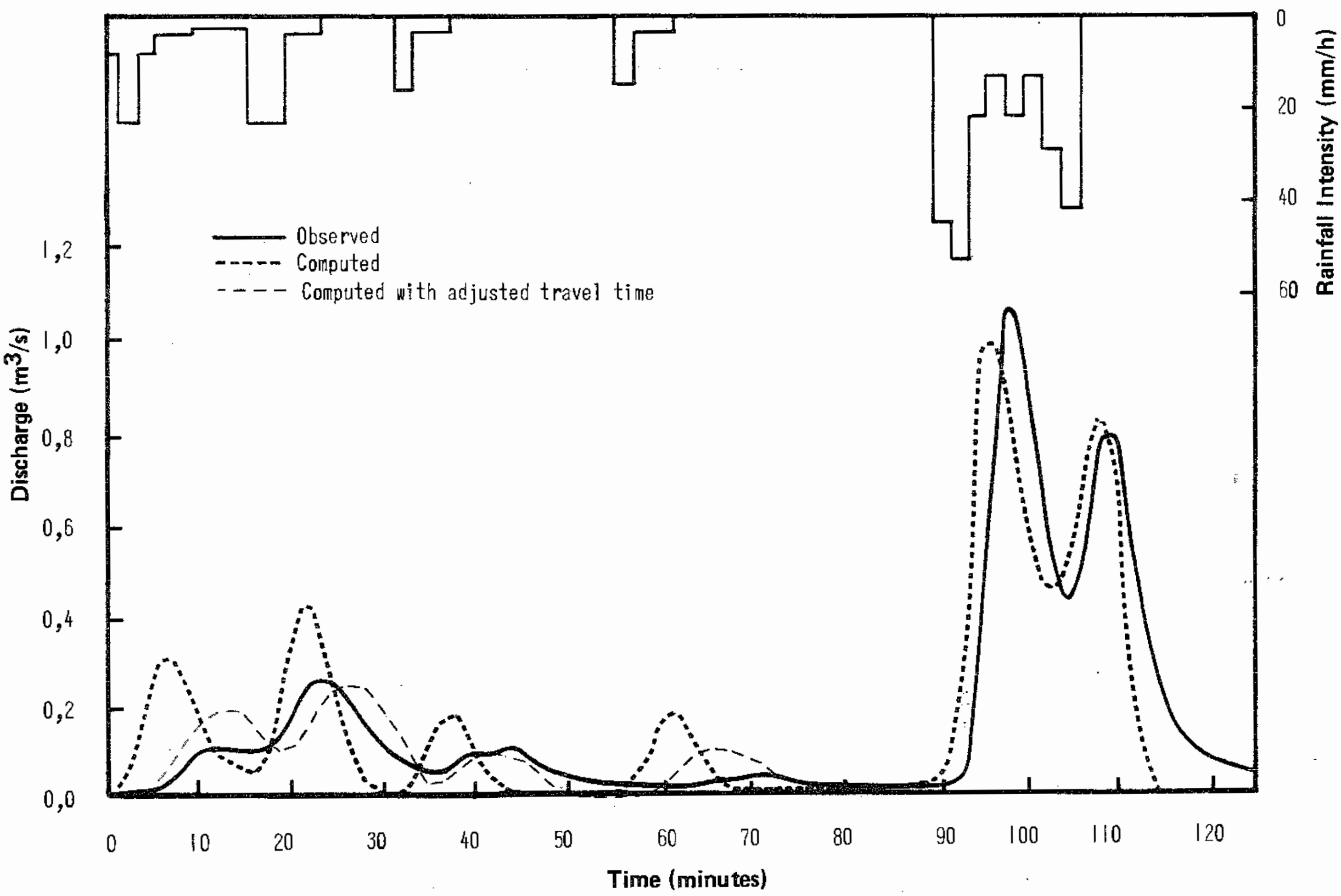

Fig. 4.33 Comparison of computed with observed hydrograph for the storm of 22/5/79 (2) on the Pinetown catchment

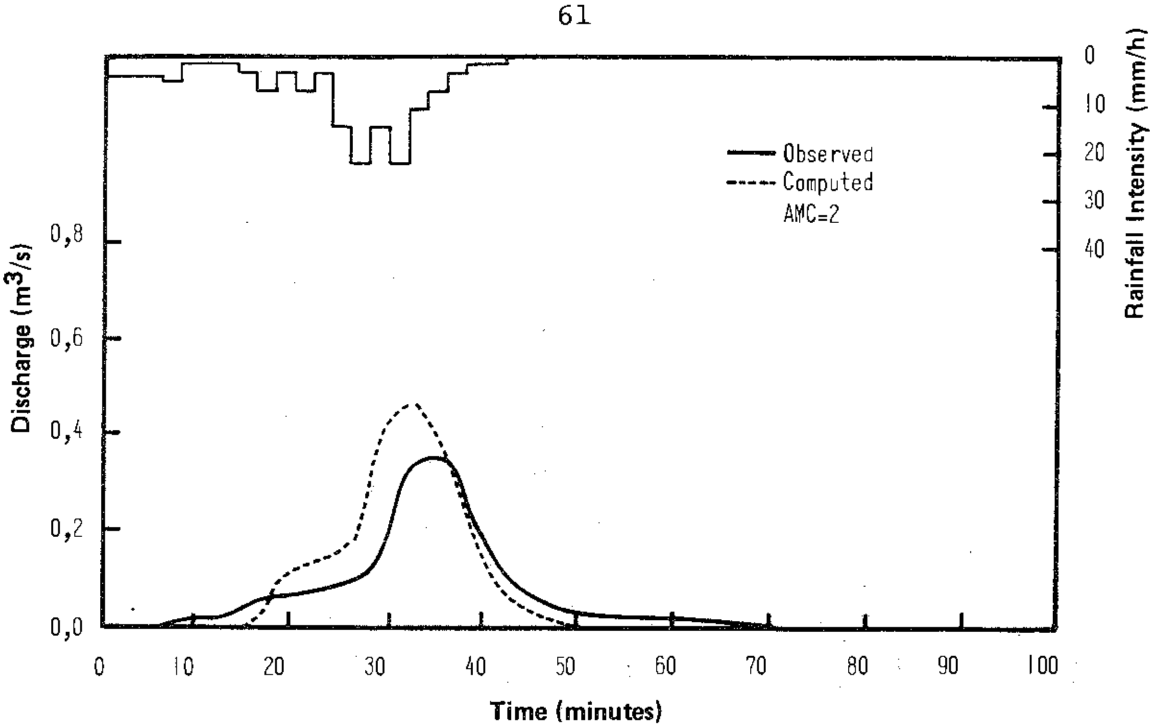

Fig. 4.34 Comparison of computed with observed hydrograph for the storm of 29/9/79 on the Pinetown catchment

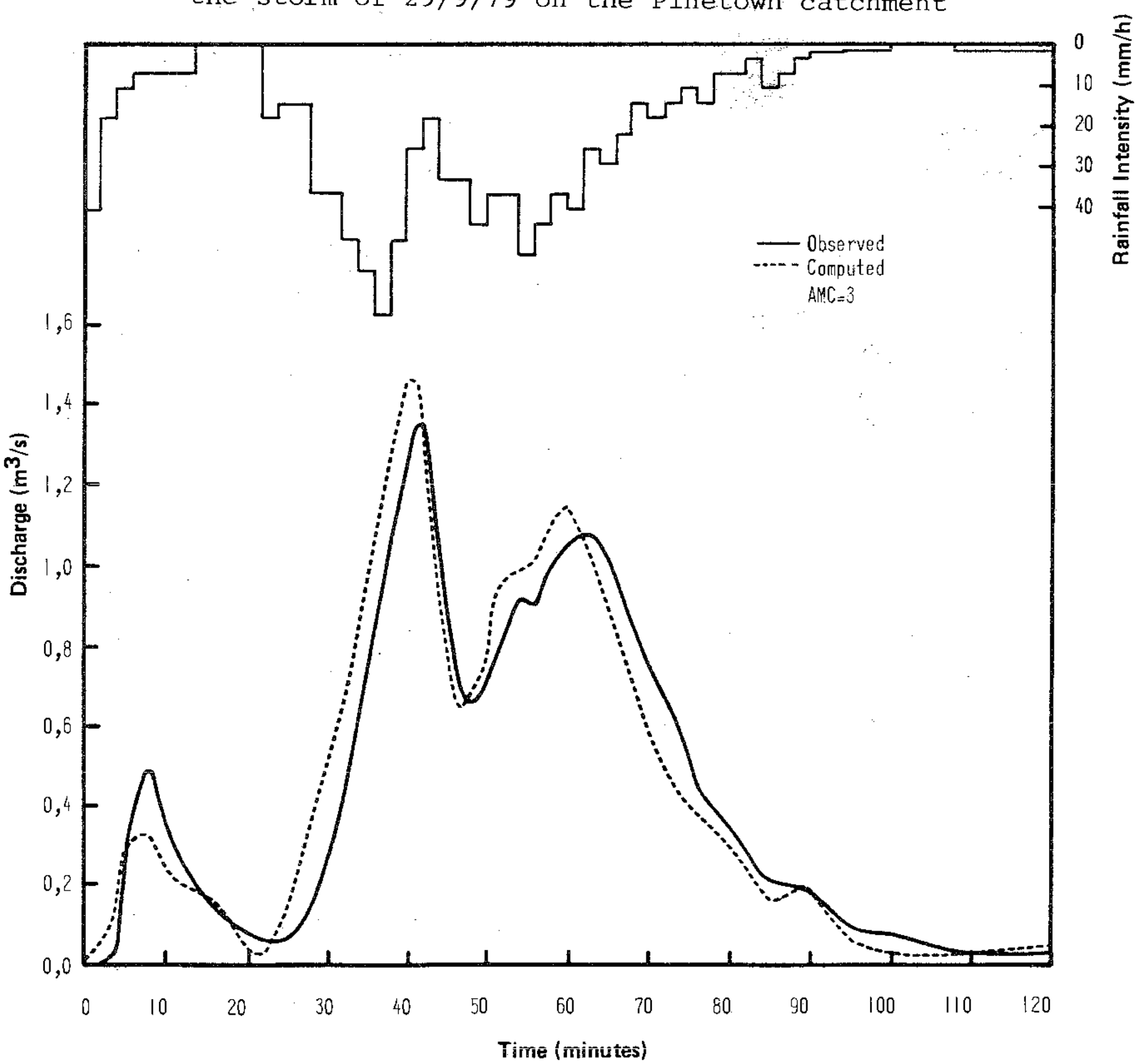

Fig. 4.35 Comparison of computed with observed hydrograph for the storm of 18/2/80 on the Pinetown catchment

4.7 Brucewood[1]

The Brucewood catchment is a 19,5 ha residential subdivision in Toronto, Canada. The area is fully developed and has 169 single-family and 43 detached residences. An aerial view of the catchment is shown in Fig. 4.36. Roof drains from all buildings are connected directly to the storm sewer system. Surface slopes are moderate, in the order of 3%. Fig. 4.37 is a topographic map showing the sewer system.

Rainfall quantity and quality were monitored for about two years by J.F. MacLaren Ltd. (1980) as part of a computer modelling feasibility study for Environment Canada. Rainfall was measured by a tipping-bucket gauge located on the roof of a school approximately 0,4 km from the centre of the catchment. The gauge registered every 0,25 mm (0,01 inch) increment of rainfall. The first bucket tip initiated the operation of the recorder. This had a chart speed of 152 mm/h (6 inch/h) giving a chart resolution of one minute.

Discharge was determined from stage measurements at a sharp-crested weir, rated in a laboratory. Depth was recorded on a chart operating at the same speed as that for the rainfall measurements - this facilitated synchronization of rainfall/runoff data. The flow recorder was set in operation by the first bucket tip of the raingauge and ran for two hours after the last bucket tip.

The paved area was assumed to have an average depression storage of 1 mm. Soil data were not available for estimating infiltration loss parameters. However, for the events considered, computed paved area runoff was approximately equal in volume to observed runoff. It was therefore reasonable to ignore any contribution from the grassed areas.

The catchment was divided into 17 subcatchments as shown in Fig. 4.38. The subcatchments were chosen to coincide approximately with those used in the SWMM study by MacLaren Ltd, 1980 (see Fig. 4.37). This proved convenient since subcatchment data were readily available. The subcatchment data used

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[1] Source of data : MacLaren Ltd., 1980.

Fig. 4.36 Aerial view of the Brucewood catchment looking east

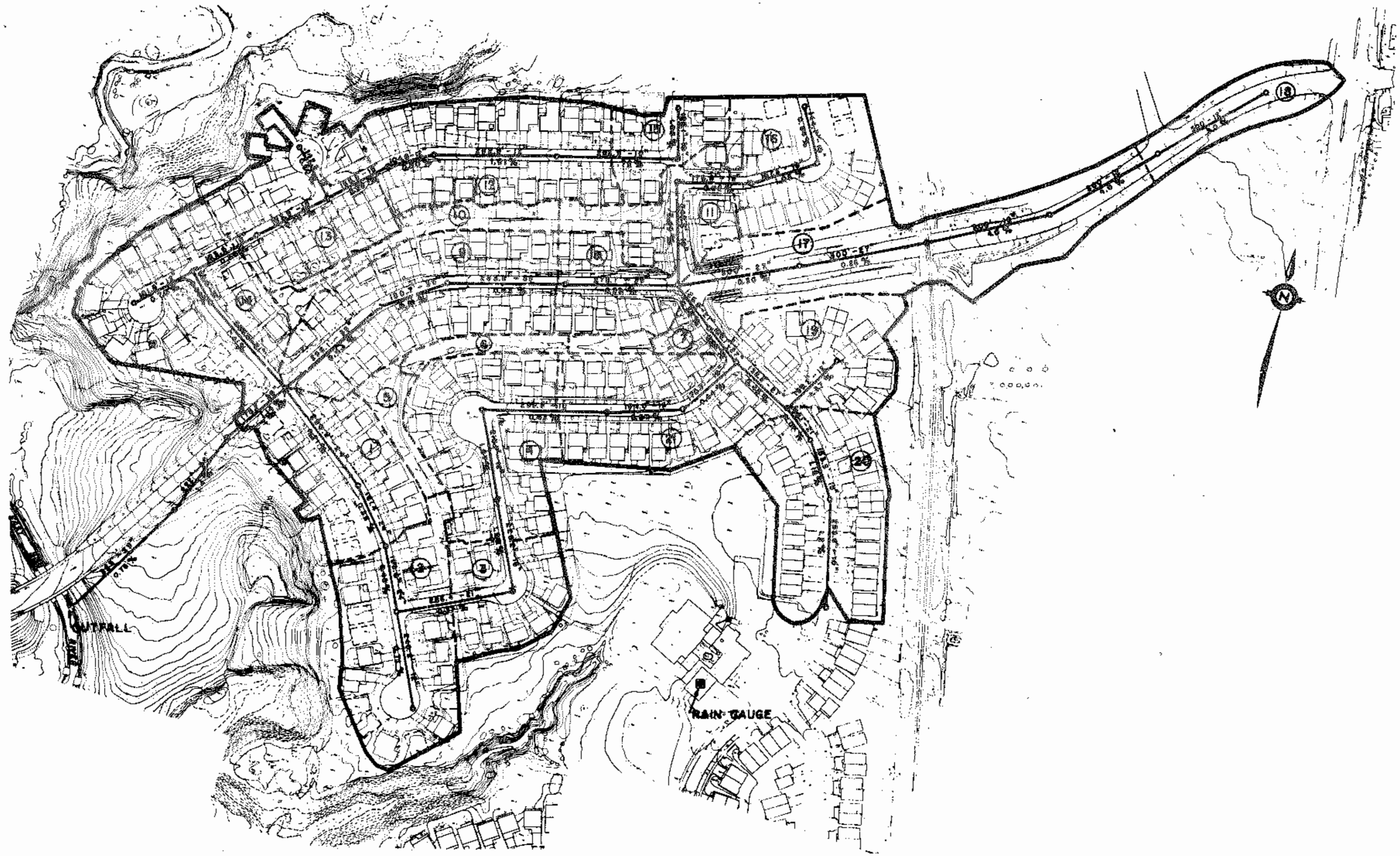

Fig. 4.37 Topographical map of the Brucewood catchment showing the stormwater drainage system and the subcatchments used in the study by J F MacLaren Ltd. (1980)

appear in Table 4.5. Paved-area entry times were assumed to be 5 minutes for all subcatchments and the computed time-area diagram is shown in Fig. 4.39. A computational time increment of 2,5 minutes was used throughout.

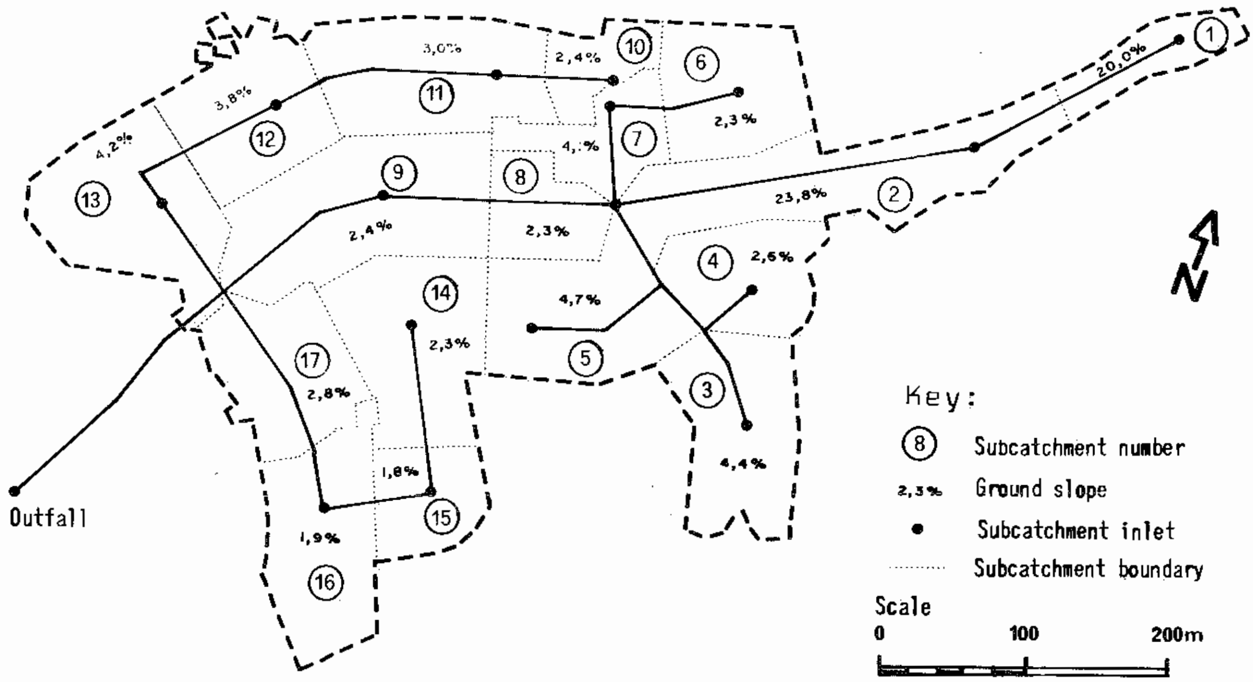

Fig. 4.38 Discretization of the Brucewood catchment

Five rainfall/runoff events are presented by MacLaren Ltd (1980) and the three most severe ones were selected for this study. Comparisons of computed with observed hydrographs and with those simulated by MacLaren Ltd. using SWMM are presented in Figs. 4.40 to 4.42. The results are not as good as for other catchments. In fact, the computed hydrographs differ markedly from the observed. The average ratio of estimated to observed peak discharge is 1,18 with a high standard deviation of 0,28. The SWMM simulations fare no better, with an average ratio of 1,02 and a standard deviation of 0,30. Errors must be largely ascribed to poor rainfall sampling.

The jumpiness in computed hydrographs for low flows is due to underestimation of surface detention. To correct this, longer entry times and flow times would have to be used for the low flows.

Table 4.5 Brucewood subcatchment data

Sub-catchment	Paved area (ha)	Grassed area (ha)	Flow-time (minutes)
1	0,19	0,30	7,9
2	0,48	1,30	6,6
3	0,65	0,60	5,5
4	0,40	1,09	5,4
5	0,58	0,98	5,7
6	0,52	0,52	5,1
7	0,17	0,46	4,0
8	0,33	0,36	3,6
9	0,90	1,00	2,2
10	0,29	0,14	7,5
11	0,84	0,44	6,0
12	0,66	0,46	3,9
13	1,01	0,41	2,3
14	0,76	0,58	5,5
15	0,37	0,27	4,4
16	0,71	0,63	3,2
17	0,57	0,55	2,0
	9,43	10,09	

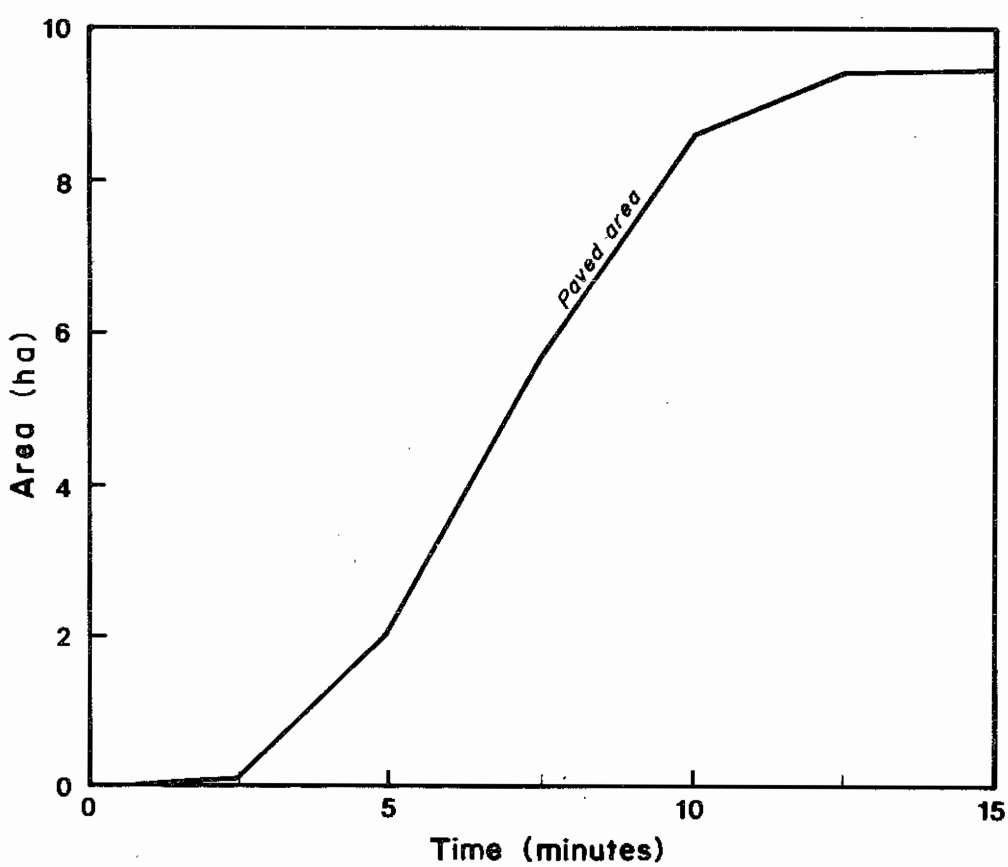

Fig. 4.39 Brucewood time-area diagram

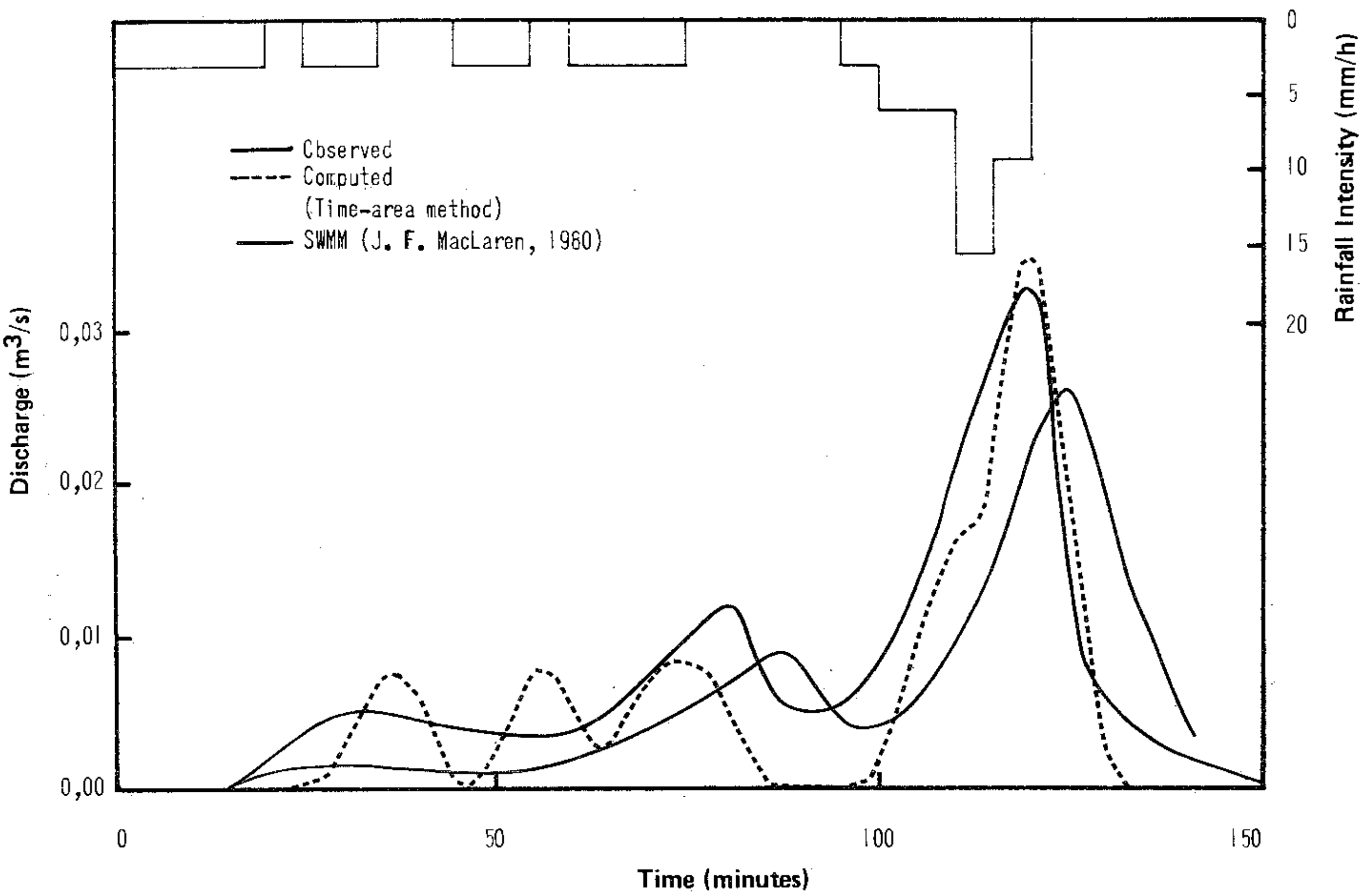

Fig. 4.40 Comparison of computed with observed and SWMM-simulated hydrograph for the storm of 14/5/74 on the Brucewood catchment

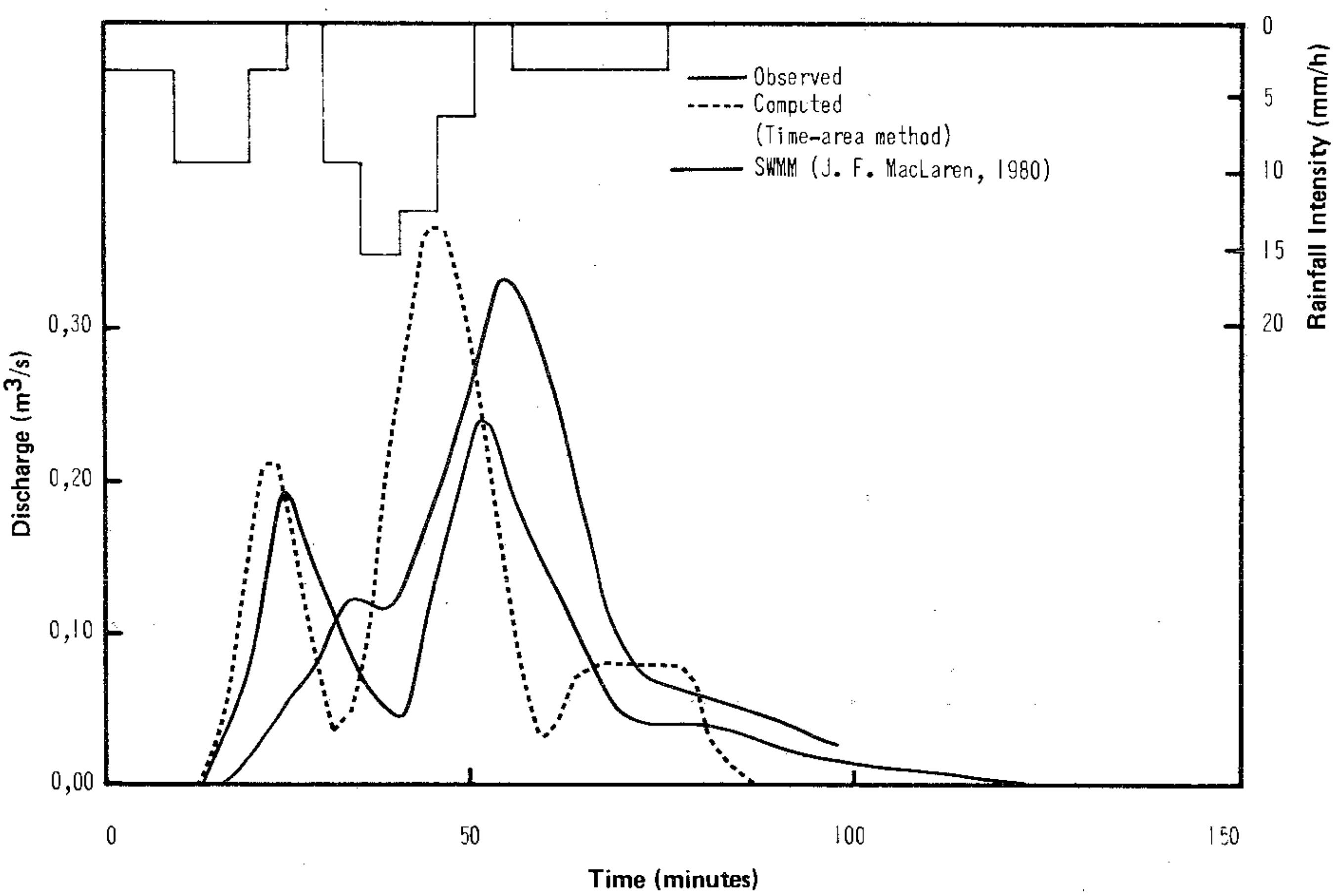

Fig. 4.41 Comparison of computed with observed and SWMM-simulated hydrograpy for the storm of 20/11/74 on the Brucewood catchment

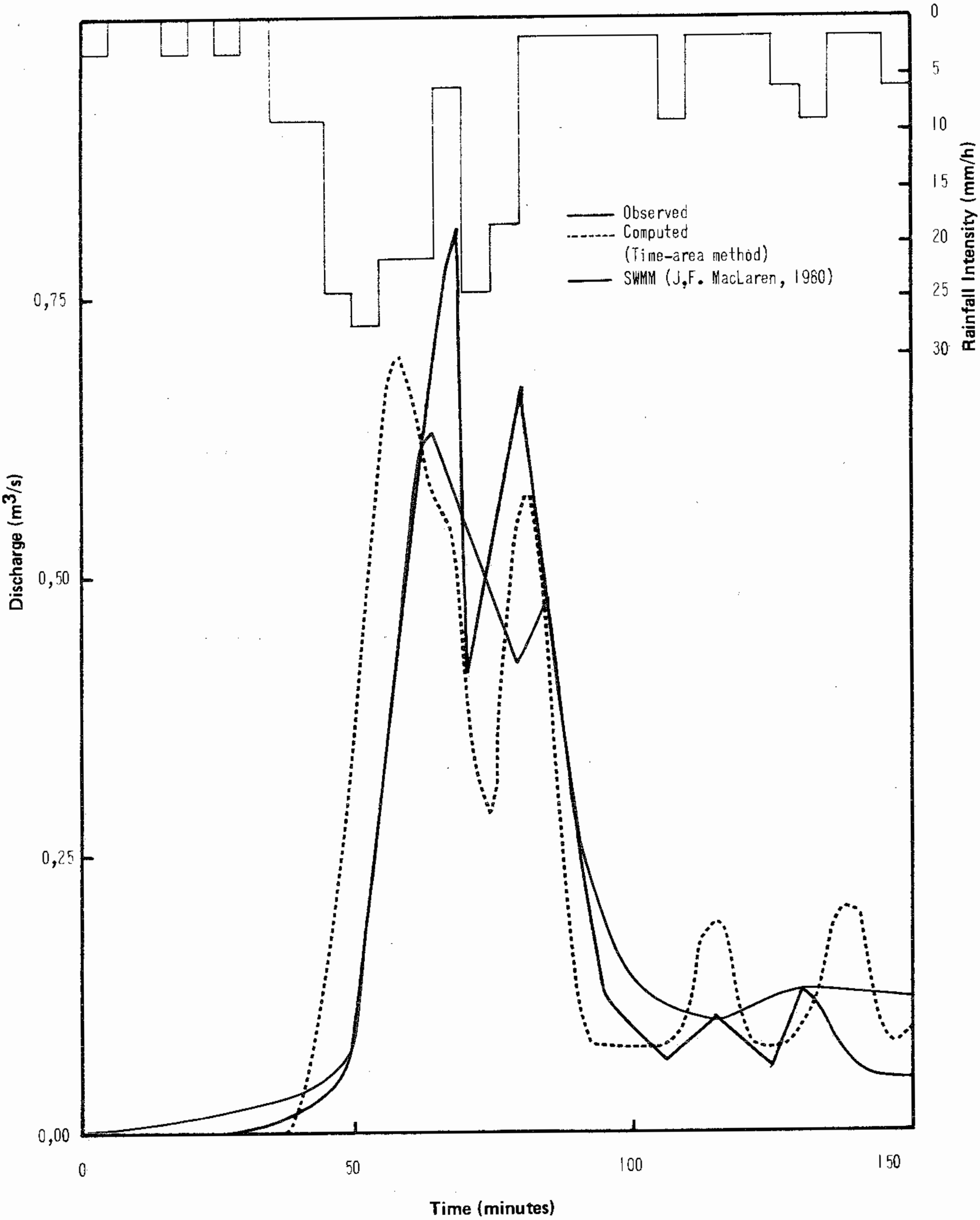

Fig. 4.42 Comparison of computed with observed and SWMM-simulated hydrograph for the storm of 11/9/75 on the Brucewood catchment

4.8 Malvern[1]

The Malvern urban test catchment is located in a residential area of Burlington, Ontario, Canada. The catchment is monitored by the Hydraulics Research Division of the Canadian Centre for Inland Waters in Burlington. Fig. 4.43 is a typical view of the catchment and Fig. 4.44 a plan of the area. The total catchment area is 23,3 ha, of which 31% is paved and directly connected to the sewer system. A further 3% is paved and drains on to pervious areas. The paved area consists of roofs (3,28 ha), roads (2,70 ha), driveways (1,26 ha) and sidewalks (0,66 ha).

The catchment is gently sloping from the north corner towards the drainage outfall located in the southwest corner (Fig. 4.44). The average catchment slope is 1%, but local slopes depend on lot gradings. Typically, front yards slope towards the street, with slopes varying from 2% to 10%. Backyards slope away from the street (2 - 3%) towards drainage swales. Road slopes are on average 1%. Soils are well-drained sandy loams.

The area is served by a tree-type, converging,separate sewer system (Fig. 4.45). All sewers are made of standard concrete pipes which are in good condition. All roof drains are directly connected to the separate sewer system.

Rainfall and runoff were monitored continuously at the outfall of the catchment. Rainfall was measured by a tipping-bucket gauge which tipped at every 0,25 mm (0,01 inch). Runoff was monitored by means of stage measurements at a rectangular weir. The rating curve was obtained by laboratory experiments. Recording chart speeds were such as to allow a one minute discretization of both rainfall and runoff records.

For convenience Marsalek's catchment discretization for simulation with SWMM was used (Fig. 4.46). The 3% supplementary paved area was considered insignificant and accordingly incorporated into the paved area. Subcatchment data are summarised in Table 4.6. Paved entry times were assumed to be

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[1] Sources of data: Marsalek, 1977 and 1979.

5 minutes for all subcatchments. Flow times were computed assuming full pipe flow and a Manning roughness coefficient of 0,013. Individual reach flow times are shown in Fig. 4.4.6 and Fig. 4.47 is the time-area diagram for the paved area.

Soil type B and a 5 mm depression storage was assumed for computation of losses. Since a complete record of antecedent rainfall was not available an AMC of 3 was assumed for all events. On this basis no grassed area runoff was computed. For the paved area an average depression storage of 1 mm was assumed. A time interval of 2 minutes was used for all computations.

Six of the larger rainfall/runoff events presented by Marsalek (1977 and 1979) were chosen for this study. Computed and observed hydrographs are compared in Figs. 4.48 and 4.50. Except for a small time shift, which is ascribed by Marsalek to synchronization errors, the results are good. The average ratio of computed to observed peak discharge is 0,93 with a standard deviation of 0,16. This compares favourably with the SWMM simulations by Marsalek in which he obtained an average ratio of 1,01 with a standard deviation of 0,21 for the same events. The higher average is partly due to rainfall corrections made by Marsalek (1979) to account for unrecorded rainfall during bucket tips. The corrections are generally small and were not made in this study.

Fig. 4.43 Street scene typical of the Malvern catchment (October, 1979)

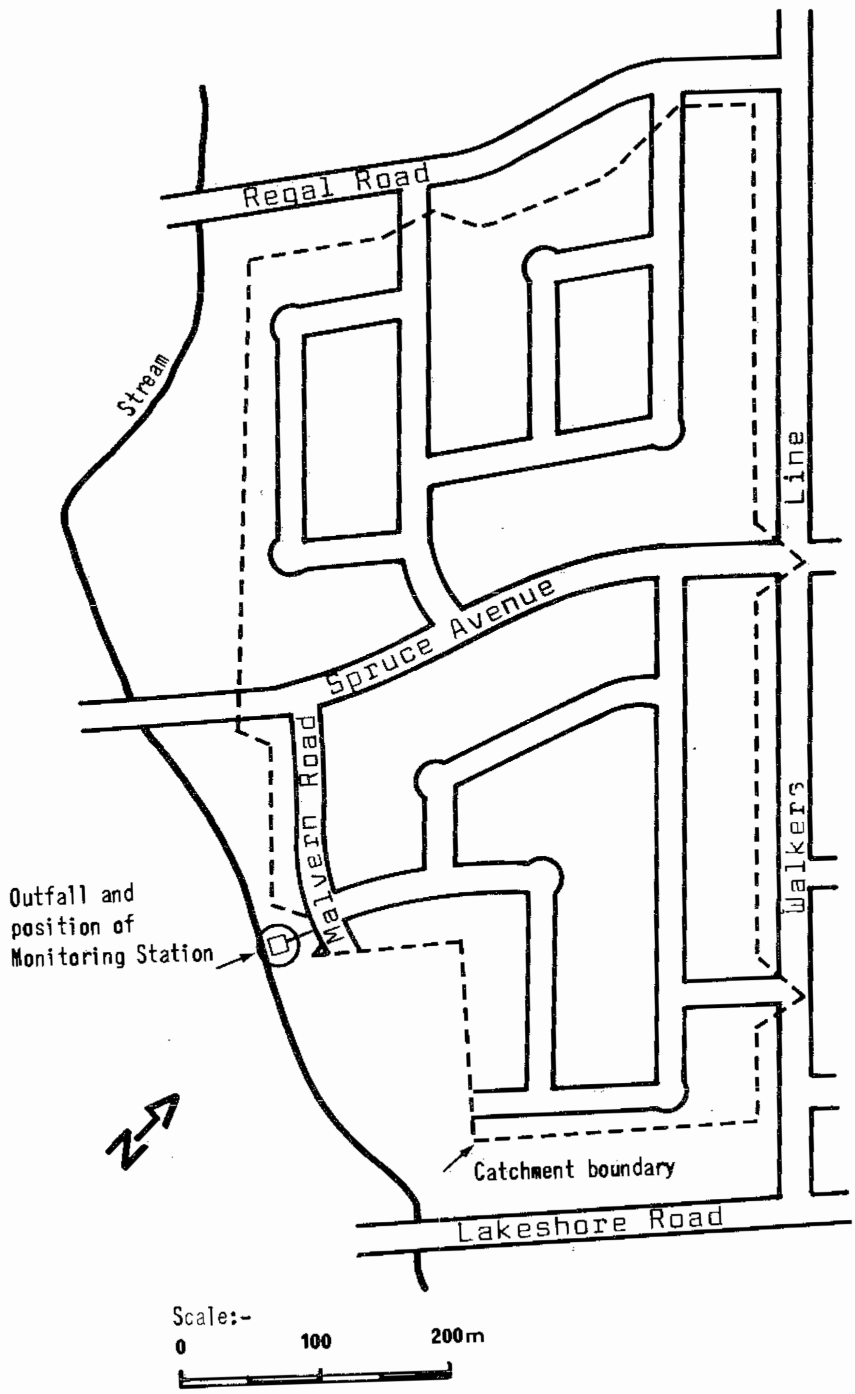


Fig. 4.44 Malvern urban test catchment

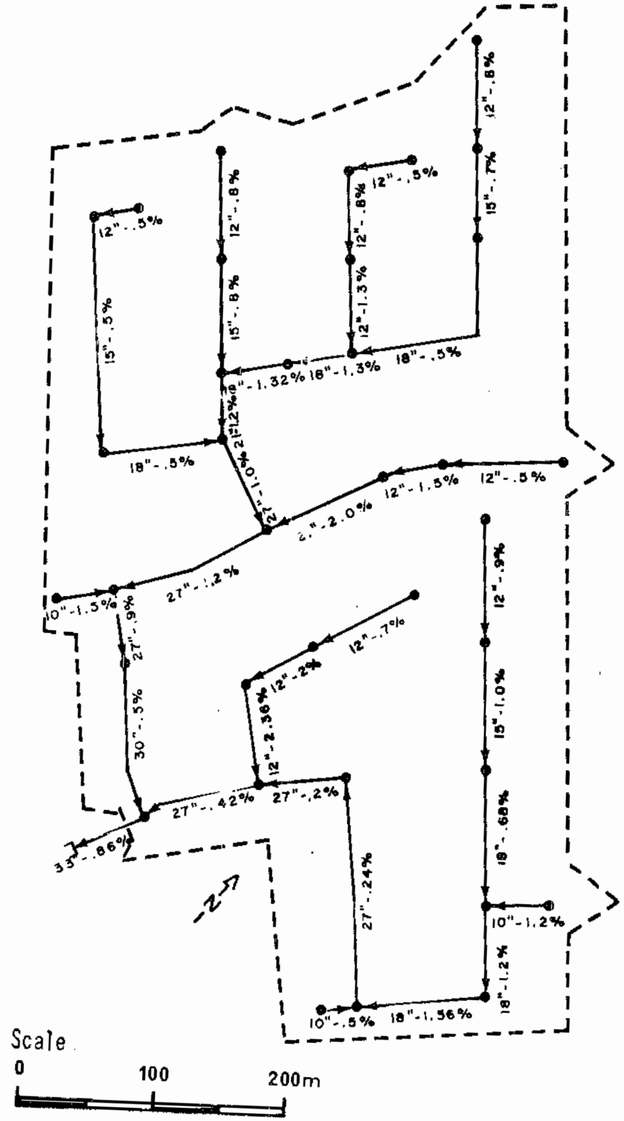


Fig. 4.45 Malvern catchment stormwater network

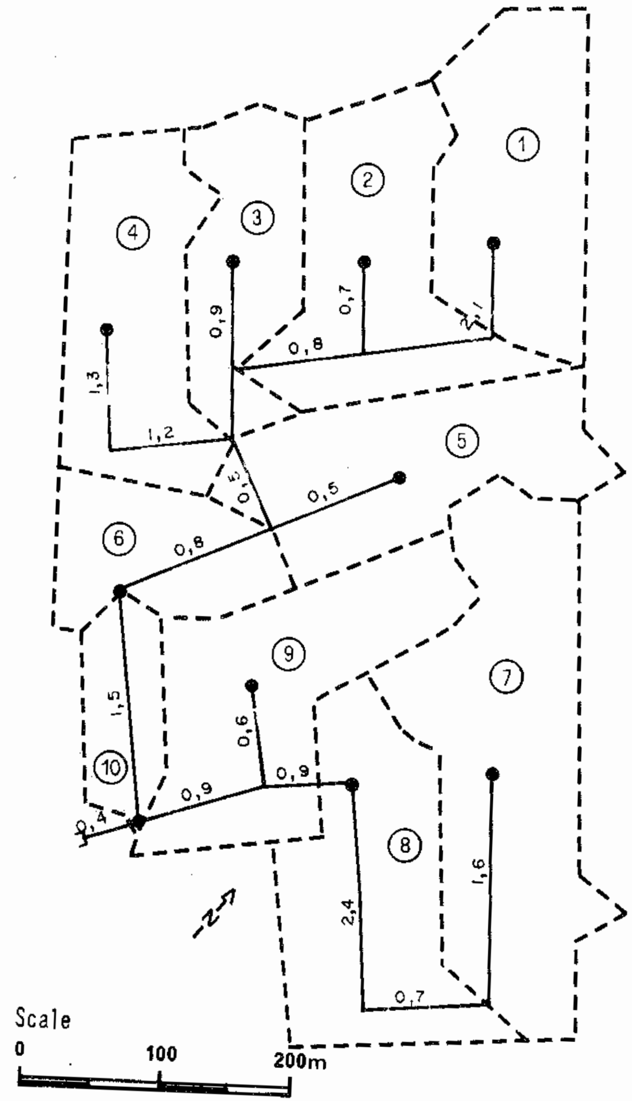


Fig. 4.46 Discretization of the Malvern catchment showing individual reach flow times (minutes)

Table 4.6 Malvern subcatchment data

Sub-catchment number	Paved area (ha)	Pervious area (ha)	Flow time (minutes)
1	0,77	1,52	6,5
2	0,89	1,63	5,1
3	0,67	0,89	4,5
4	1,14	1,29	5,7
5	0,77	1,71	3,3
6	0,49	0,87	1,9
7	1,11	2,72	6,9
8	0,85	1,83	2,2
9	0,76	2,54	1,9
10	0,43	0,43	0,4
	7,88	15,43	

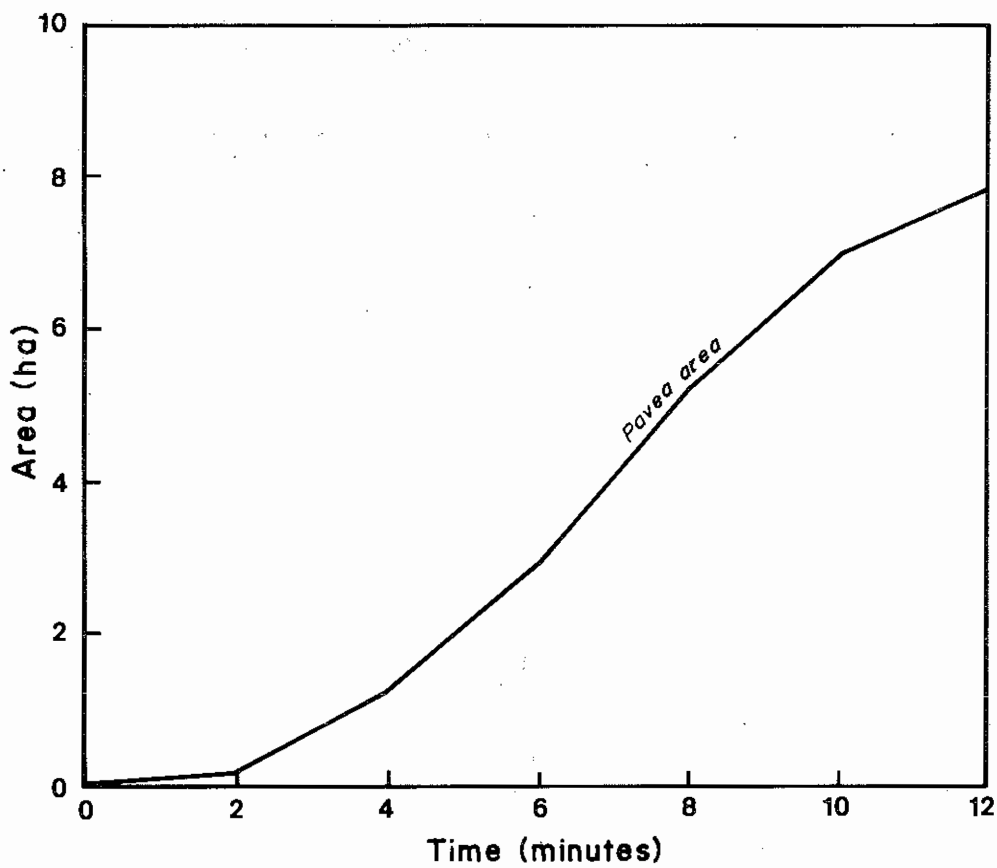


Fig. 4.47 Malvern time-area diagram

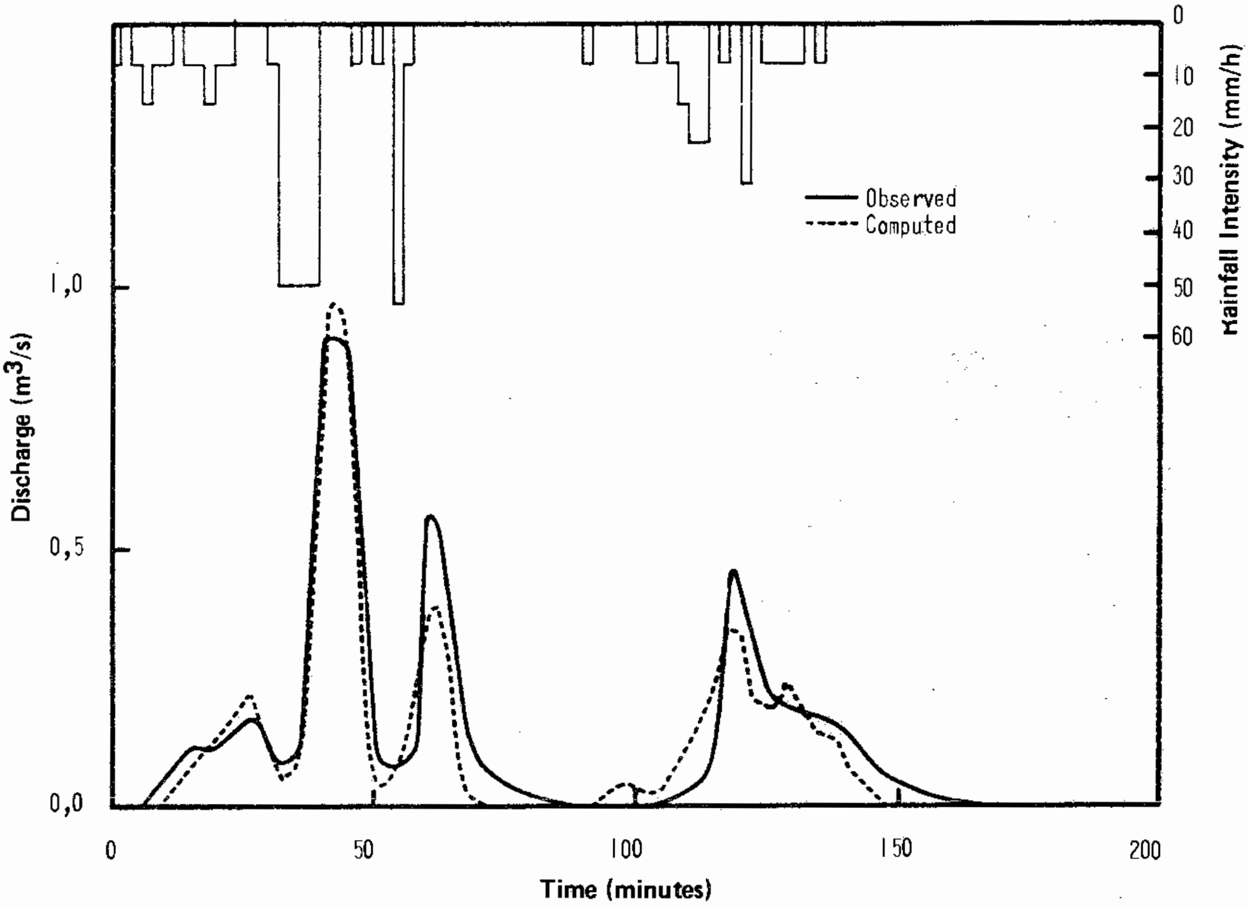


Fig. 4.48 Comparison of computed with observed hydrograph for the storm of 22/9/73 on the Malvern catchment

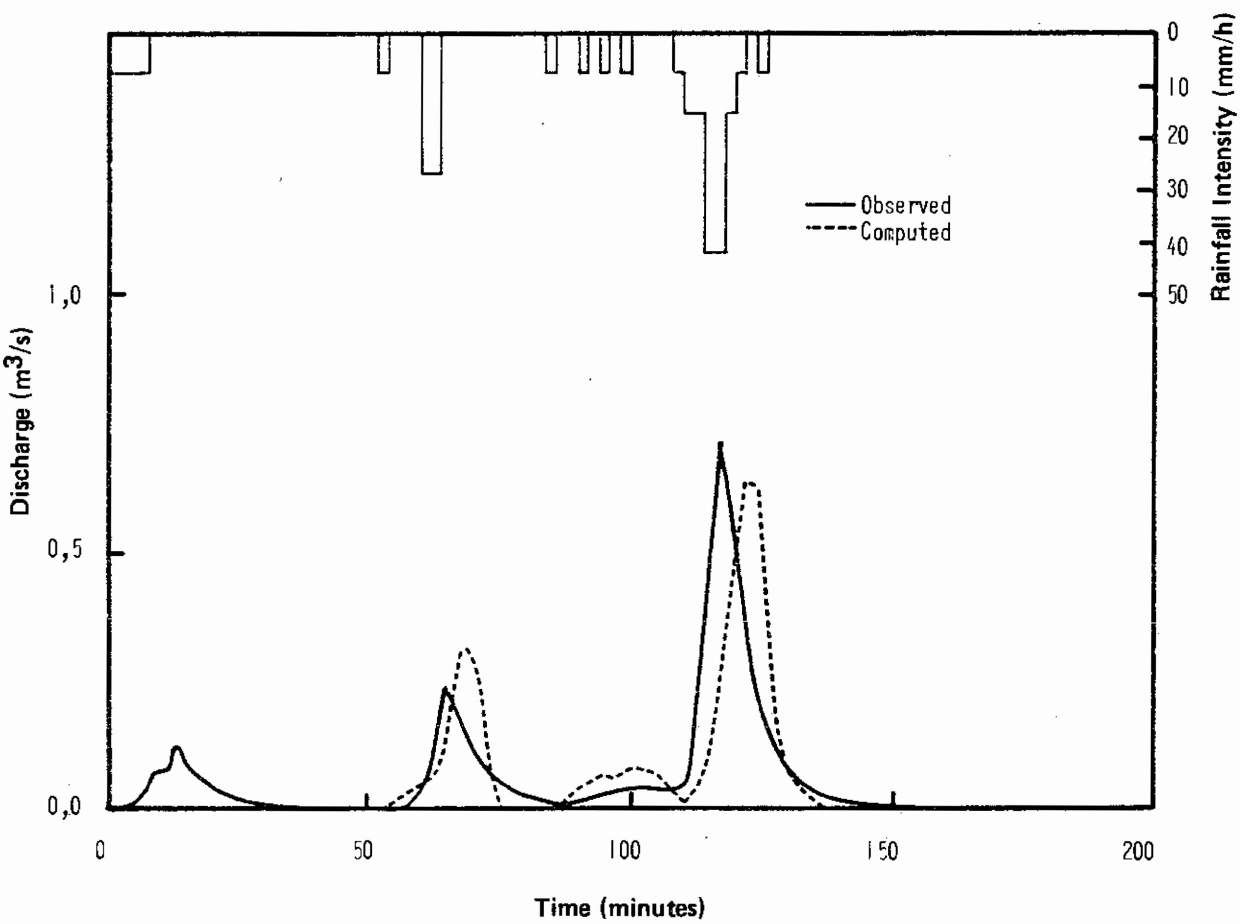
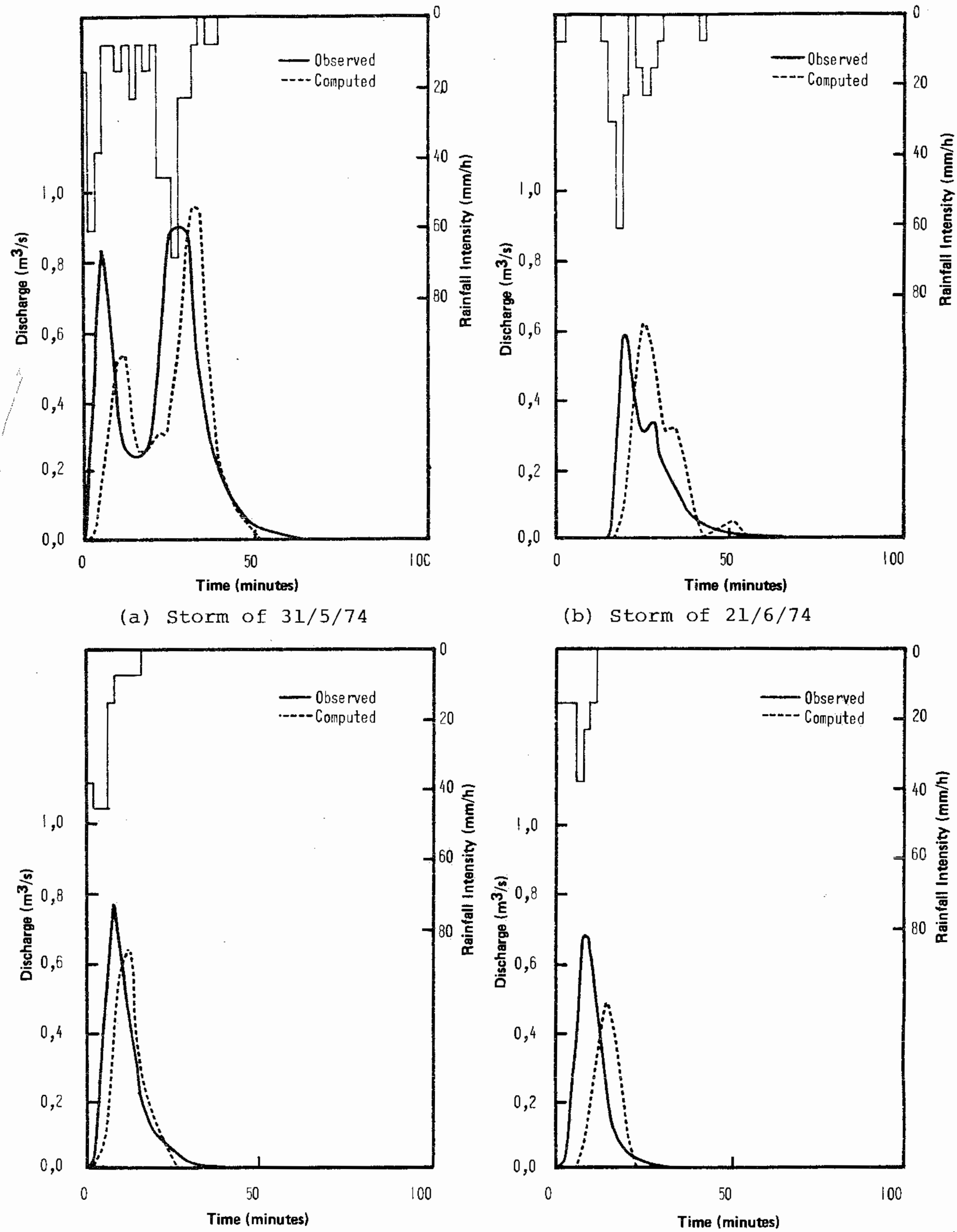


Fig. 4.49 Comparison of computed with observed hydrograph for the storm of 23/9/73 on the Malvern catchment



(a) Storm of 31/5/74

(b) Storm of 21/6/74

(c) Storm of 4/7/74

(d) Storm of 19/7/74

Fig. 4.50 Comparison of computed with observed hydrographs for storms on the Malvern catchment

4.9 Kew

The Kew catchment, situated in the northern suburbs of Johannesburg, has an area of 143 ha. Ground slopes are moderately steep (up to 8%) and soils are residual granodiorite. Although mainly residential, a significant part of the area is occupied by industrial and commercial buildings. The residential sector occupies about 80% of the area, the industrial 10%, the commercial 5% and the remaining 5% open. About 30% of the area is paved though a third of this is estimated to supplement grassed area runoff. The drainage system consists of concrete pipes, concrete channels and a natural stream. A typical residential street scene is shown in Fig. 4.51 while Fig. 4.52 is a topographical map showing the distribution of land use and the storm sewer system.

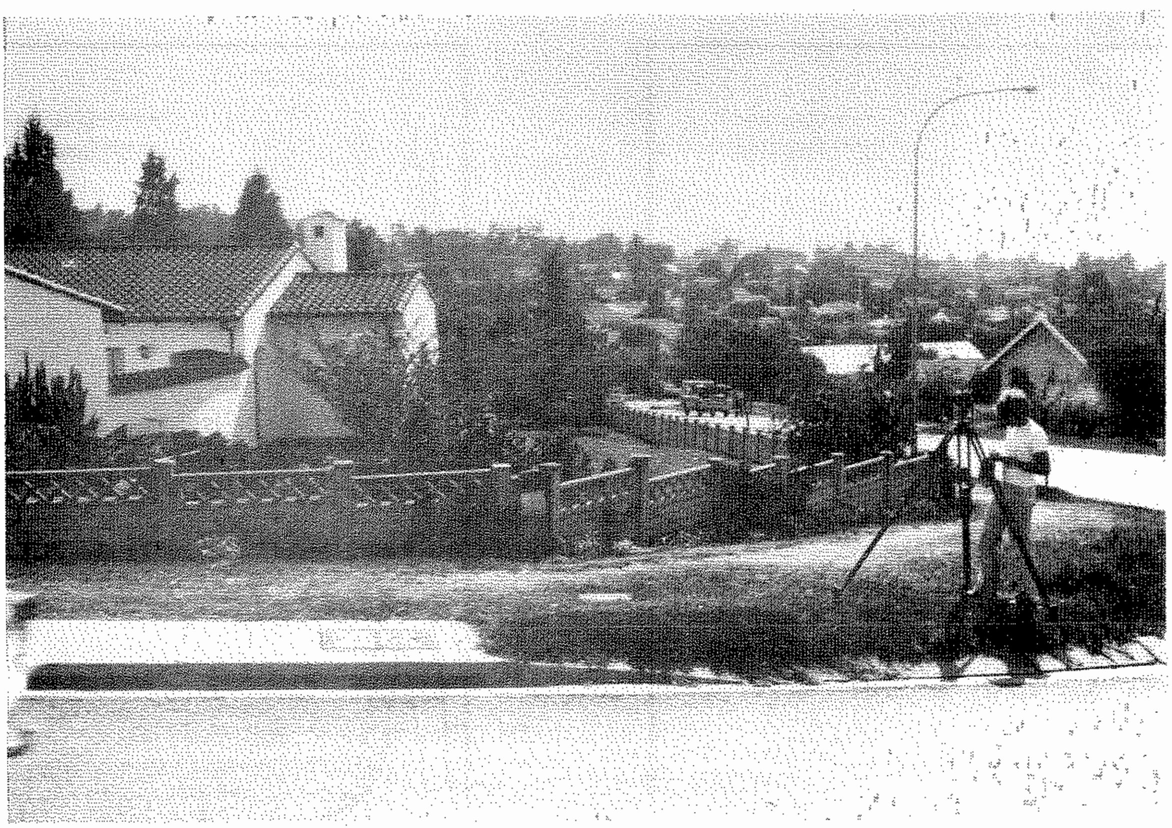


Fig. 4.51 Street scene in the residential portion of the Kew catchment (May 1979)

The catchment was monitored by the author for the 1979/80 rainy season during which rainfall and runoff were measured continuously. The rainfall recorder was a W. Lambrecht type 1509-20 with a 31-day strip chart. This was propelled at 20 mm/h and recorded depth to a scale of 1 : 0,125. The raingauge was located close to the outfall. Discharge was obtained from stage measurements at a V-form Crump weir placed in a culvert. Stage was measured by means of an Ott pneumatic water level transducer and recorded by an Ott R20 strip chart recorder. This had a 32-day chart propelled at 20 mm/h and recorded stage at a scale of 1 : 5. Silting of the weir upset the theoretical rating but corrections were made on the basis of velocity – area measurements.

Rainfall and runoff data are available for seven of the larger recorded storms (Watson, 1981a) but the rainfall data for one of the storms were not representative of the average catchment rainfall. The remaining six events were analysed.

The catchment was discretized into 8 subcatchments as illustrated in Fig. 4.53 and described in Table 4.7. Soils were assumed to be type B and relevant infiltration parameters from Table 3.1 were used. The average depression storage of the paved area was assumed to be 1 mm and of the grassed area 5 mm. Supplementary paved area was found to be fairly uniformly distributed within the grassed area. The catchment was divided into two zones, viz. paved and unpaved. For the events considered paved-area entry times were estimated to be in the order of 10 minutes for most subcatchments. For the sake of simplicity a value of 10 minutes was used throughout. Grassed-area entry times were assigned the value 40 minutes which was the typical value found in a previous study using ILLUDAS (Watson, 1981a). Flow times were estimated assuming uniform flow with Manning n values of 0,012 for pipes, 0,014 for concrete channels and 0,040 for the stream. The resultant time-area diagrams are shown in Fig. 4.54. A computational time increment of 5 minutes was used for simulating all events except that of 18/3/80. For this event a 2-minute time increment was necessary to avoid truncating the peak of the hydrograph.

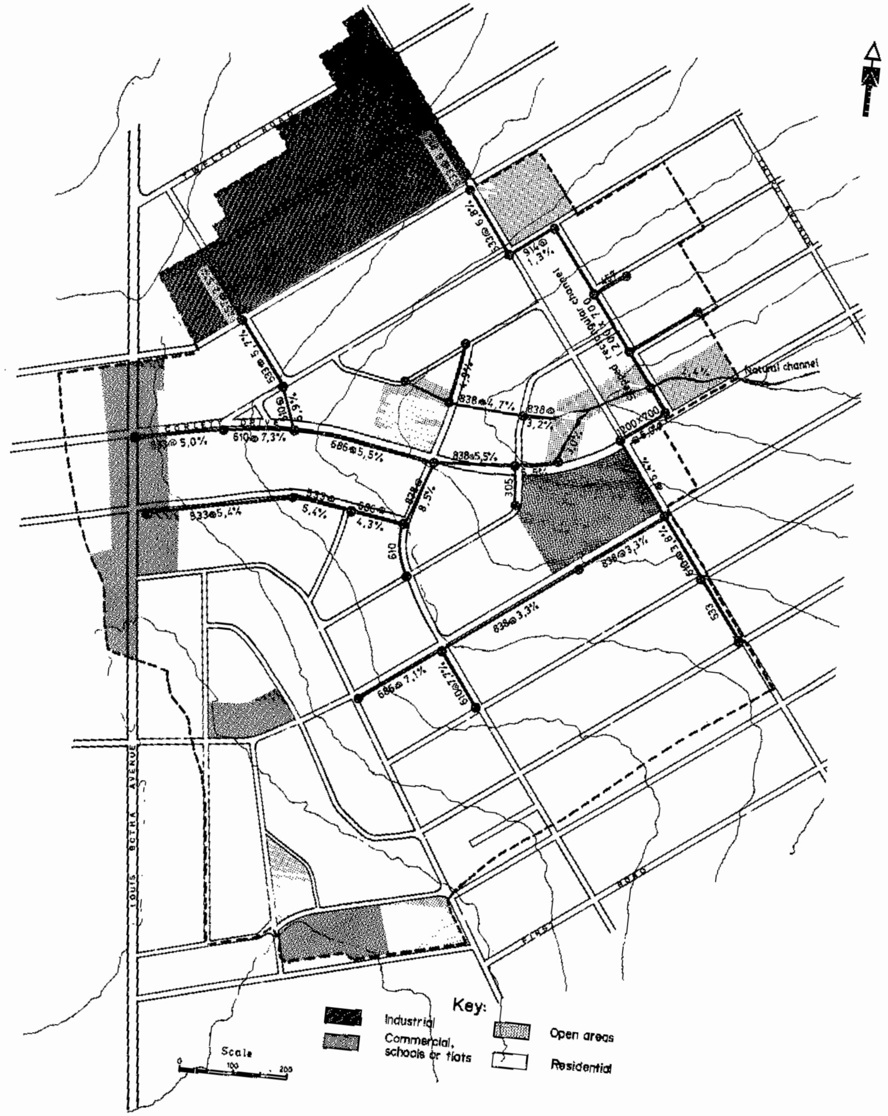


Fig. 4.52 Kew catchment land use and stormwater drainage system

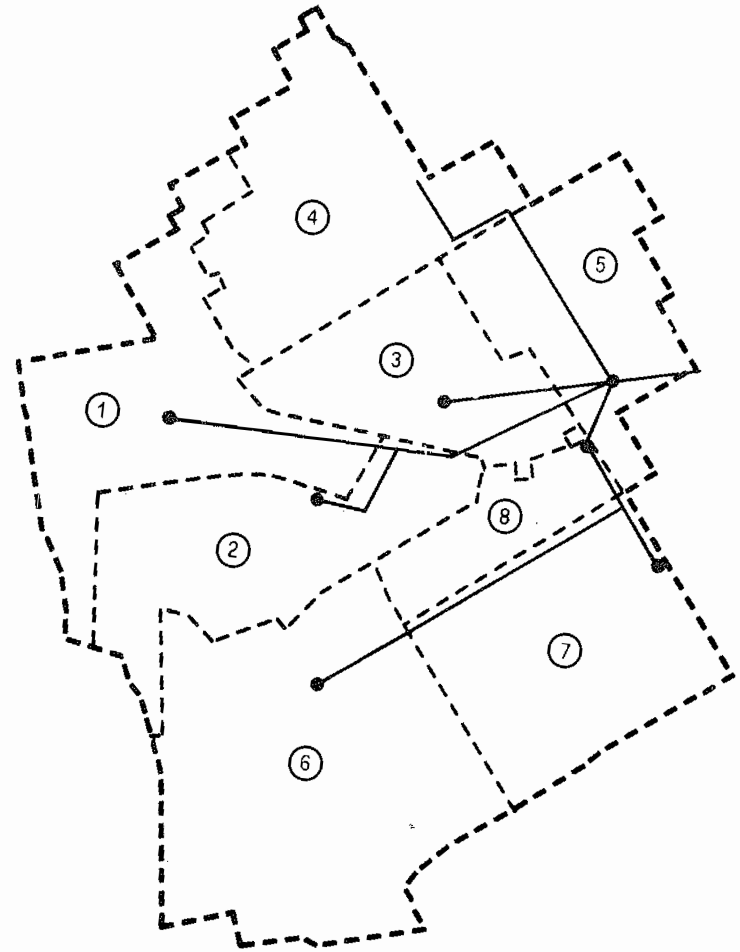


Fig. 4.53 Discretization of the Kew catchment

As shown in Figs. 4.55 to 4.60 simulation of peak discharge is very good; the mean ratio of computed to observed peak is 0,99 with a standard deviation of 0,16. Reproduction of hydrograph shapes is not as good as that of peaks. Discrepancies are due largely to rainfall sampling errors and the simplified manner of accounting for pervious area losses.

Table 4.7 Kew subcatchment data

Sub-catchment	Paved area (ha)	Grassed area (ha)	Supplementary area (ha)	Flow time (minutes)
1	5,3	12,5	2,0	3,0
2	2,5	13,0	2,0	3,3
3	1,5	12,7	1,7	1,8
4	9,2	9,5	1,1	2,5
5	0,6	9,5	1,2	0,5
6	5,1	23,7	3,9	3,3
7	1,4	15,8	2,2	2,0
8	2,5	3,6	0,7	0,9
	28,1	100,3	14,8	

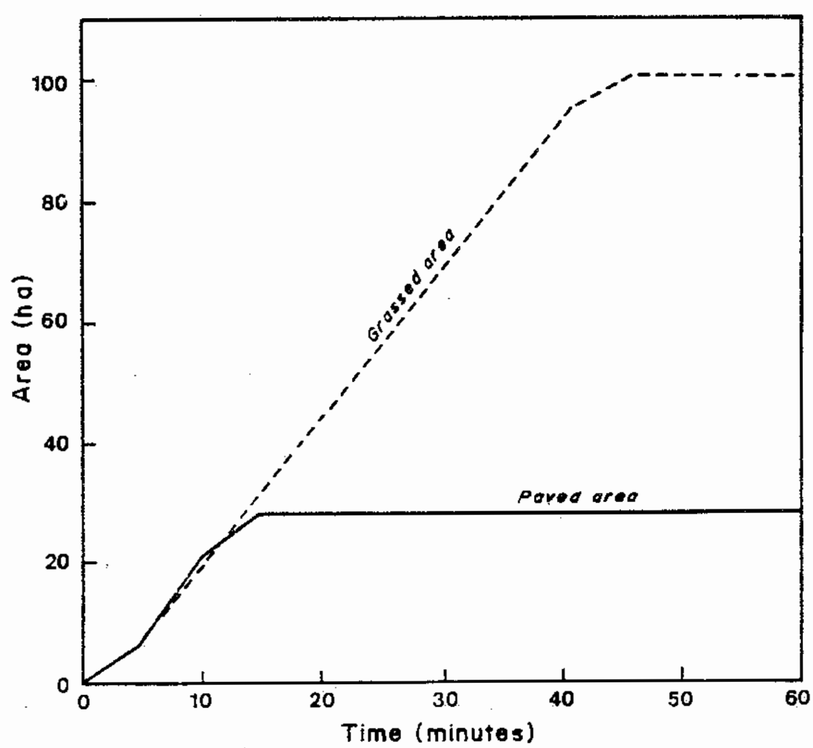


Fig. 4.54 Kew time-area diagrams

The pervious areas were assumed to be uniform with respect to infiltration and depression storage. This is patently untrue, however, as unpaved surfaces in the commercial and industrial areas are often compacted and have reduced infiltration capacity. The same can be said for unpaved driveways in the residential areas. Neglect of this can cause runoff volume to be under-estimated and is a major cause of the too-rapid recession of the computed hydrograph for the storm of 22/3/80 (Fig. 4.57).

The large bulge in the recession of the computed hydrograph for the storm of 18/3/80 (Fig. 4.56) is ascribed to subtraction of losses from rainfall instead of from surface runoff. The correct accounting for losses would result in a much improved runoff distribution, as shown by Watson (1981a) in a verification study of ILLUDAS. This type of discrepancy becomes increasingly important when overland flow occurs over long distances. It loses significance, however, for design events where over-prediction of grassed area runoff affects a small proportion of the total runoff.

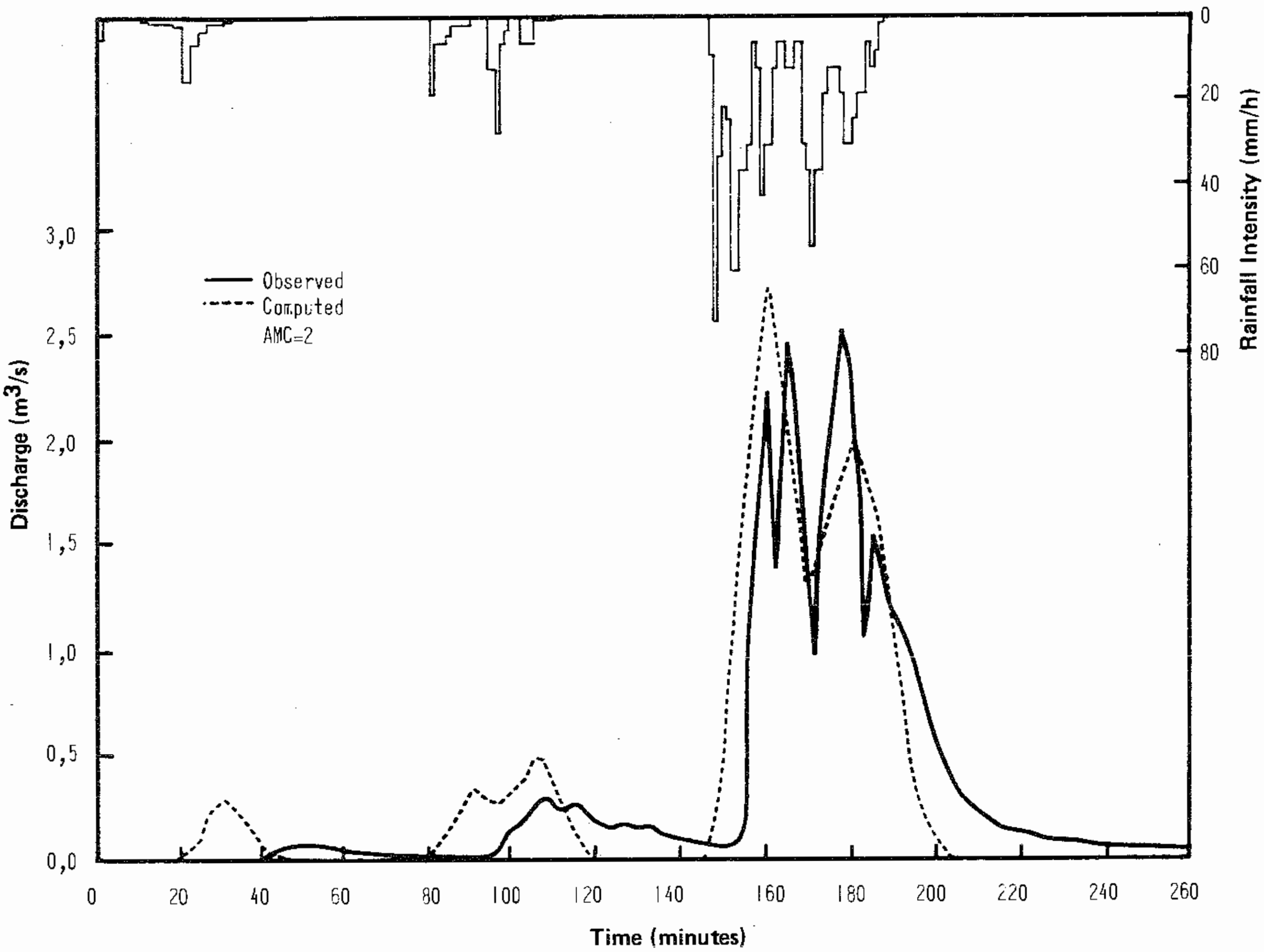


Fig. 4.55 Comparison of computed with observed hydrograph for the storm of 17/3/80 on the Kew catchment

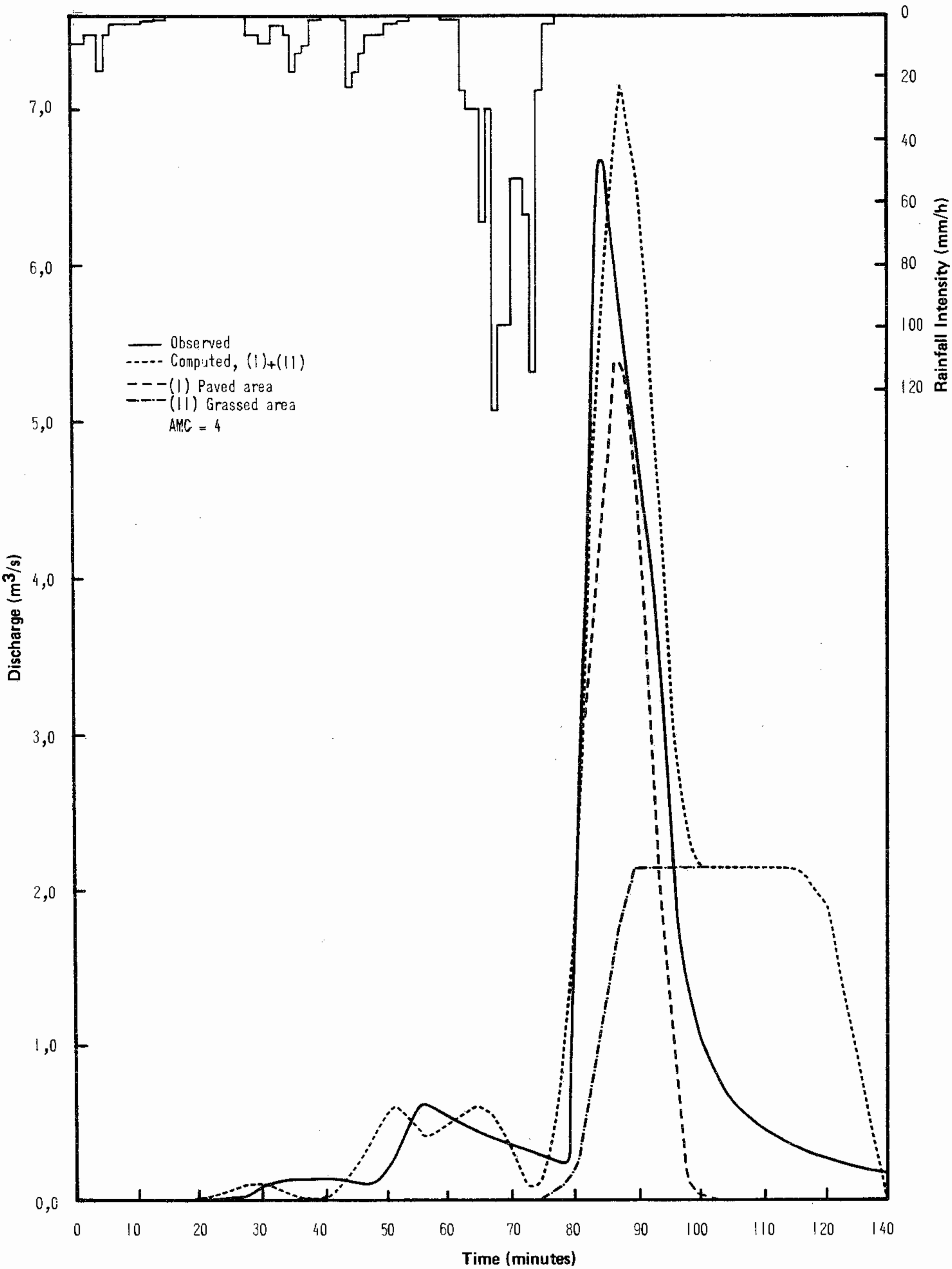


Fig. 4.56 Comparison of computed with observed hydrograph for the storm of 18/3/80 on the Kew catchment

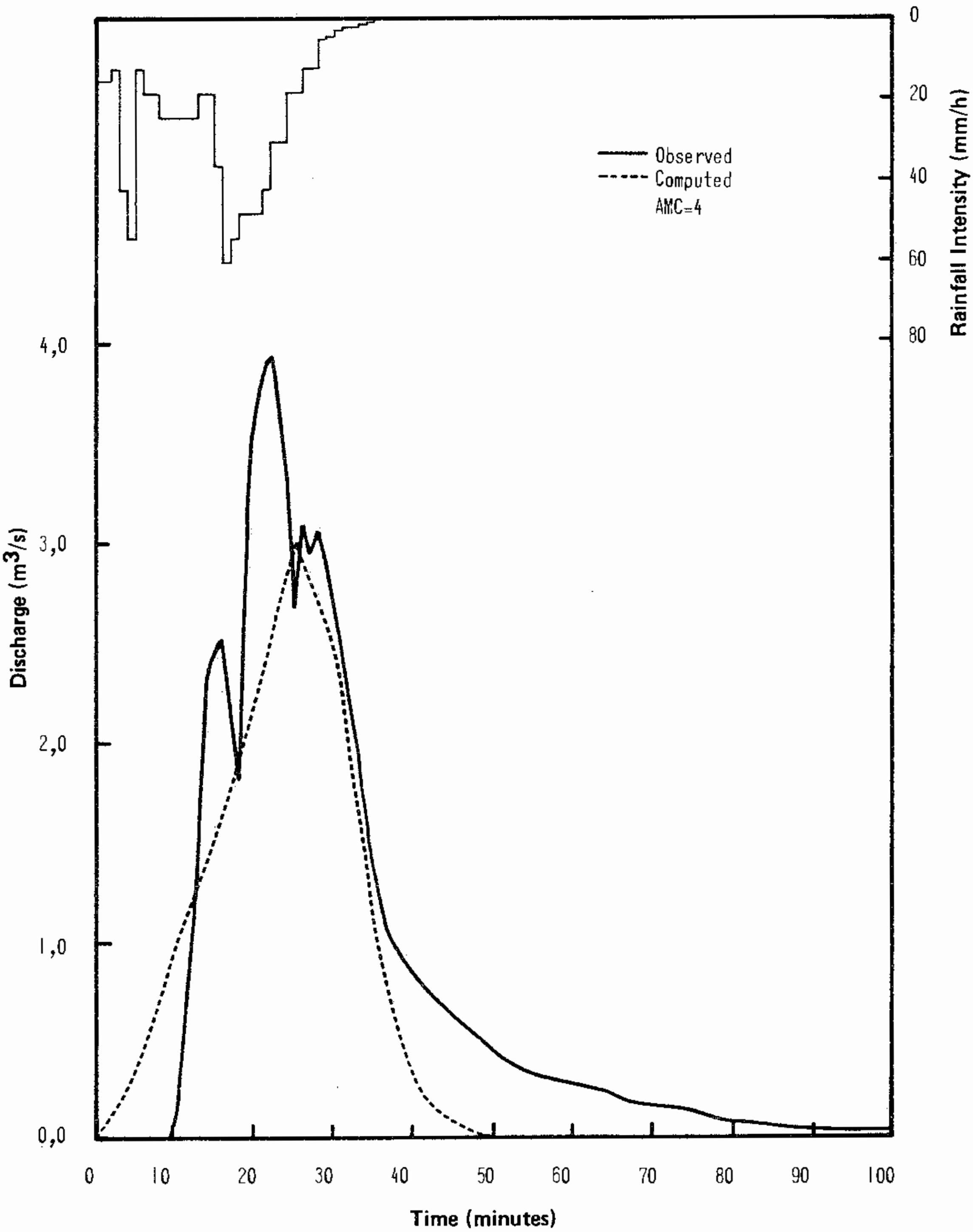


Fig. 4.57 Comparison of computed with observed hydrograph for the storm of 22/3/80 on the Kew catchment

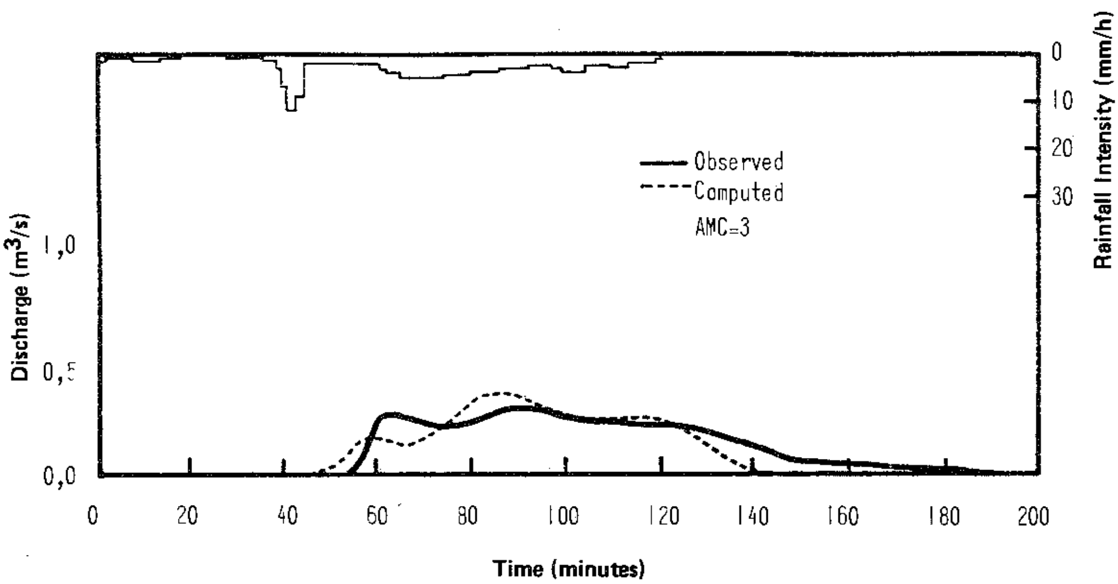


Fig. 4.58 Comparison of computed with observed hydrograph for the storm of 19/2/80 on the Kew catchment

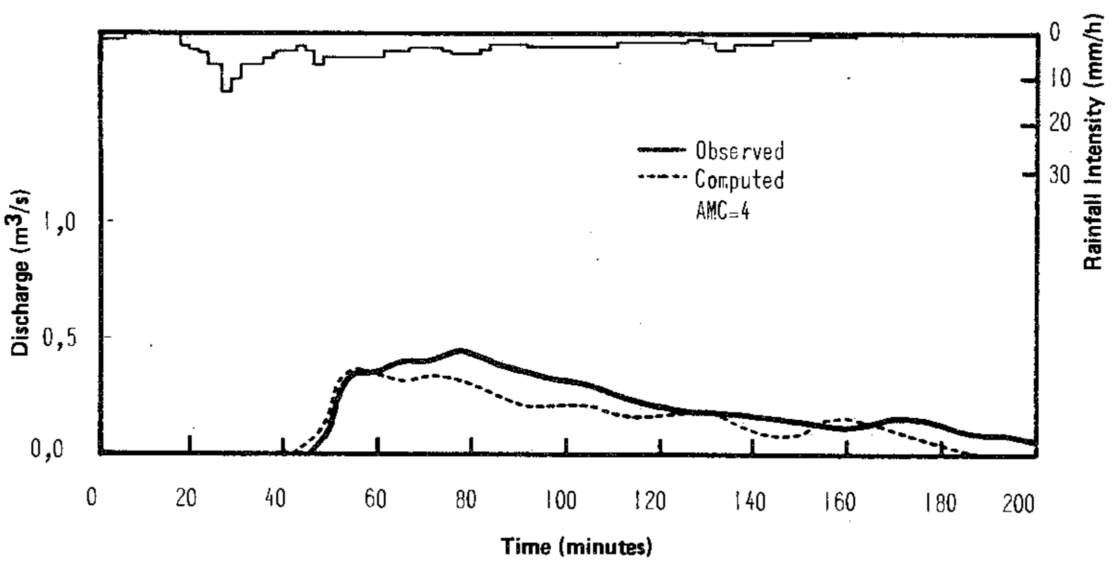


Fig. 4.59 Comparison of computed with observed hydrograph for the storm of 19/3/80 on the Kew catchment

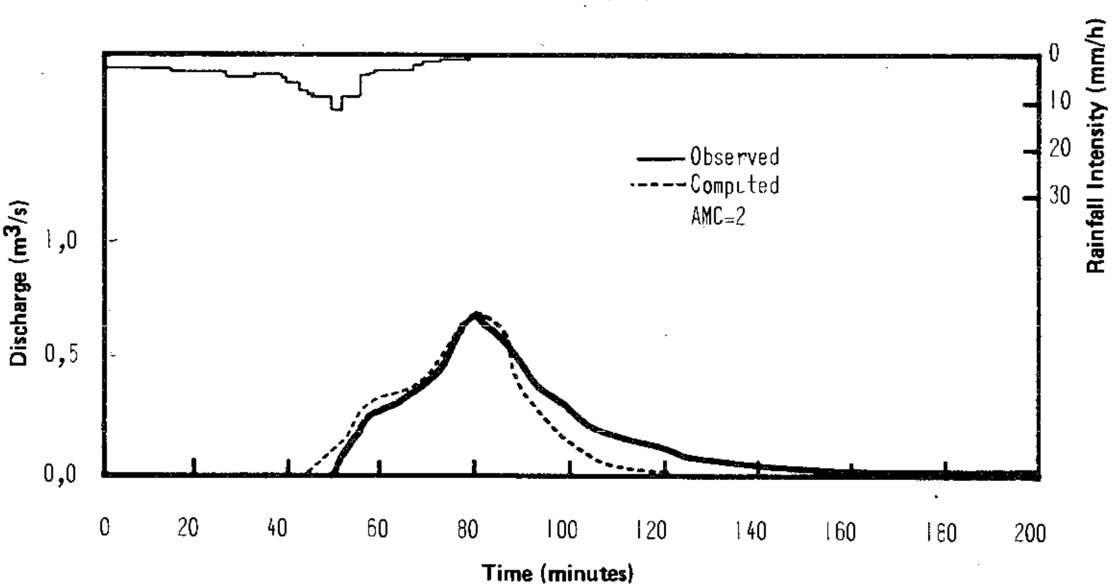


Fig. 4.60 Comparison of computed with observed hydrograph for the storm of 10/4/80 on the Kew catchment

4.10 Discussion of results

The catchments studied range in size from 0,2 ha to 143 ha with percentages of paved area ranging from 20 to 100. Average ground slopes varied from about 0,5% to 5% and computed concentration times ranged from 5 minutes to 45 minutes.

Comparisons of computed with observed hydrographs were in most cases highly satisfactory. Estimations of peak discharge were good, the average ratio of estimated to observed for all 36 events considered being 1,04 with a standard deviation of 0,17. The results are summarized in Table 4.8.

Comparisons were also made with hydrographs derived by SWMM and kinematic wave simulations and in all cases the results were favourable. These models did, however, take better account of the low runoff portions of the hydrographs. The favourable comparisons are very significant since kinematic wave is recognised as being the best computational technique for overland flow. SWMM on the other hand takes detailed account of pipe flow routing.

Table 4.8 Summary of urban catchment verification results

Catchment	Area (ha)	Paved area %	Number of events	λ	s
1. South parking lot	0,2	100	6	1,06	0,14
2. Newart Street	0,3	100	2	0,97	0,02
3. Oakdale Avenue	5,2	45	5	1,11	0,15
4. Gray Haven	9,4	52	3	0,91	0,11
5. Pinetown	12	80	5	1,12	0,15
6. Brucewood	20	48	3	1,18	0,28
7. Malvern	23	34	6	0,93	0,16
8. Kew	143	30	6	0,99	0,16
Overall performance			36	1,04	0,17

λ = mean ratio of computed to observed peak discharge

s = standard deviation of the individual values about λ

Grassed area runoff was computed for only three storms, viz. Oakdale Avenue 2/7/60, Gray Haven 1/8/63 and Kew 18/3/80. The results in all cases were good and served to domonstrate the adequacy of treating paved and grassed areas as two separate zones. The computed hydrograph for Kew (18/3/80) also demonstrated the over-estimation of runoff resulting from subtraction of losses from rainfall instead of from runoff.

CHAPTER 5 VERIFICATION ON RURAL CATCHMENTS

5.1 Introduction

Data have been assembled for 24 storms on 6 rural catchments. The catchments range in sixe from 1,4 ha to 125 ha. Most data came from two publications of the U.S. Department of Agriculture (Hobbs, 1963 and USDA, 1957). Reference was also had to the Ph.D. dissertation of Singh (1974) for soil descriptions and for three storm events. Data for the only local catchment considered (Zululand W1M17) was obtained from the Agricultural Catchments Research Unit of the Universities of Natal and Zululand. Data were selected on the basis of availability of significant runoff events.

Time-area routing parameters were in all cases estimated. Overland flow travel-times were determined from eq. 3.2 with an assumed width ratio , W, of unity. Manning n was assumed to be 0,15 for grasslands and 0,10 for cultivated areas. Channel-flow travel times were computed assuming uniform flow in a triangular channel with side slopes of 30% (i.e. z = 2). A channel roughness coefficient of 0,04 was assumed throughout. In cases where it was found necessary to compute more than one time-area diagram - because of large differences in excess rainfall intensities between storms - only entry times were varied. Flow times were held constant because of the uncertainty involved in estimating and because of their lesser significance for the catchments considered.

The loss parameters could not be accurately estimated, particularly the initial infiltration parameter, f_o, which varies widely with AMC. The parameters f_∞, k and d_s were kept constant for the particular catchment and f_o was allowed to vary between storms. No attempt was made to relate f_o to the depth of antecedent rainfall due to the small sample of events available. Antecedent rainfalls for the selected events are, however, listed in Appendix A.3.

5.2 Hastings 2-H

The USDA experimental catchment 2-H is situated near Hastings, Nebraska (USA). The catchment is 1,38 ha in area and has an average ground slope of 10%. Fig. 5.1 is a contour plan of the area. The topsoil is generally a mixture of silt and clay with silt predominating. Internal drainage is medium, and permeability of the subsoil is moderate. Land use is native grass meadow and surface drainage is good. Rainfall is recorded by a gauge situated about 300 m northwest of the catchment.

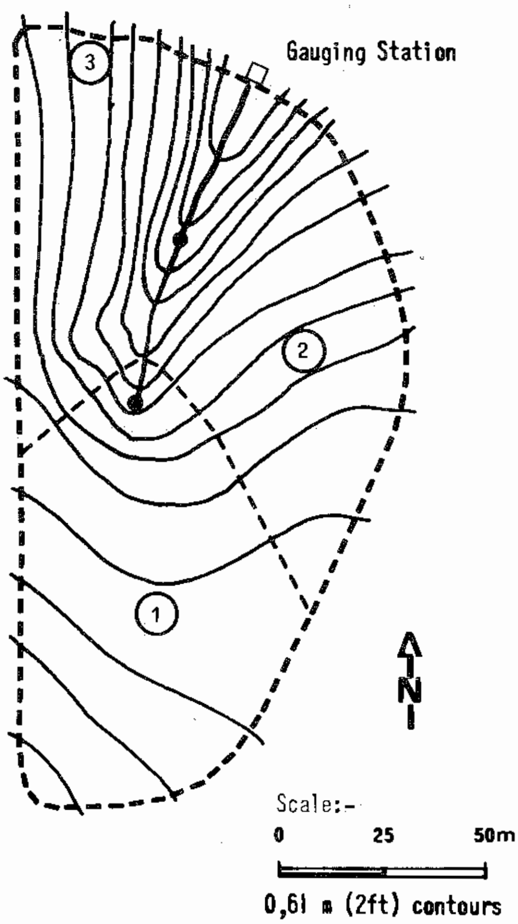


Fig. 5.1 USDA 2-H catchment near Hastings, Nebraska (USA)

Five storm events were selected for simulation. Data for two events (12/6/58 and 3/7/59) were obtained from the USDA publication (Hobbs 1963). Data for the remaining three events were interpolated from figures presented by Singh (1974).

The catchment was divided into three subcatchments as shown in Fig. 5.1. The estimated subcatchment characteristics are presented in Table 5.1 and the computed time-area diagram in Fig. 5.2. Values of the loss parameters f_∞, k and d_s were chosen as 13 mm/h, 6 h^{-1} and 6 mm respectively. Selected values of f_o ranged from 105 mm/h to 190 mm/h for the different storms. The values chosen for each storm are given in Figs. 5.3 to 5.7. A computational time increment of 5 minutes was used throughout.

Table 5.1 Hastings 2-H subcatchment data

Sub-catchment	Area (ha)	Entry time[1] (minutes)	Flow time[2] (minutes)
1	0,54	20	2
2	0,48	15	1
3	0,36	10	1
	1,38		

[1] i = 50 mm/h

[2] Q = 0,1 m^3/s

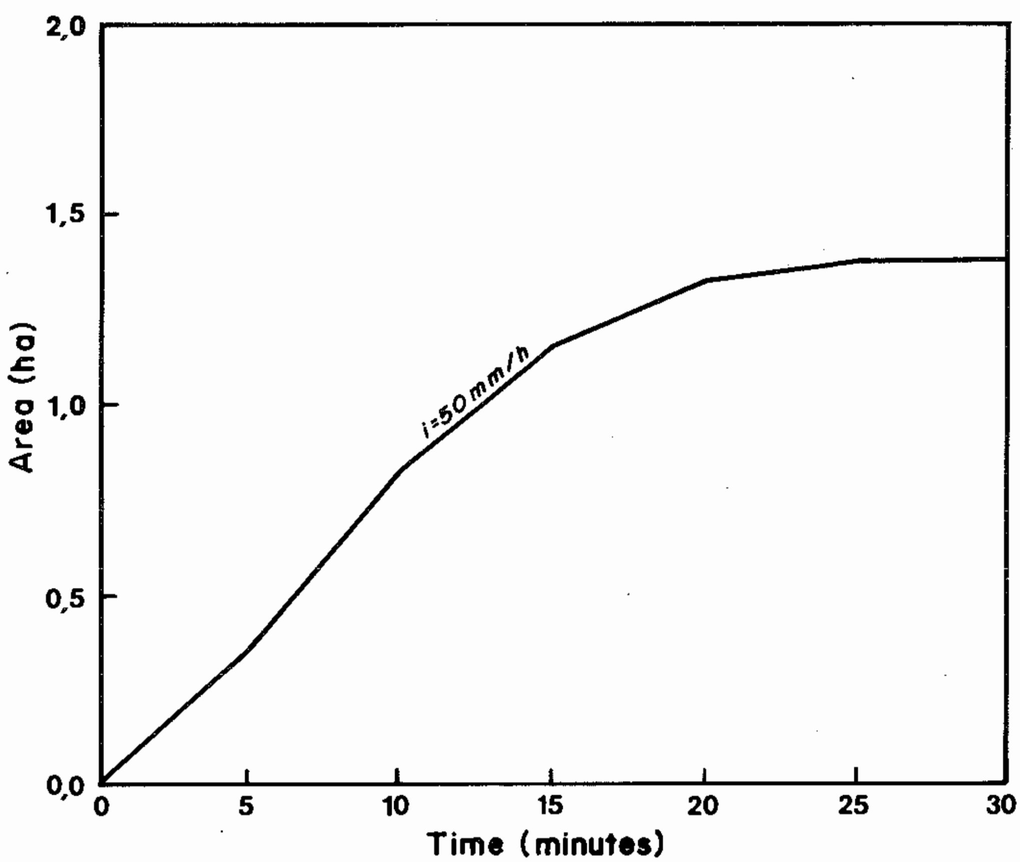


Fig. 5.2 Hastings 2-H time-area diagram

A major portion of the rainfall was absorbed by the soil and as a result of this computed hydrographs were highly sensitive to estimation of the loss parameters. This was particularly so for the multiple-peak events. After calibrating loss parameters, reasonable comparisons of computed with observed hydrographs were obtained (Figs. 5.3 to 5.7). The storm on 26/6/52 was treated as two separate events to allow for regeneration of depression storage. If the recession constant k had been varied between events markedly better results would have been achieved for the storm of 13/7/52 (a k value of about 2 would have been more appropriate for this storm).

To demonstrate the relative significance of loss estimation to routing computations, the computed hydrograph of 15/5/60 is compared with the kinematic wave simulated hydrograph by Singh (1974). The hydrographs are shown in Fig. 5.7. The error in the kinematic wave solution is due to unsatisfactory temporal distribution of losses.

The overall simulation results for this catchment are quite reasonable. The average ratio of estimated to observed peak discharge for the five events is 0,93 with a standard deviation of 0,19.

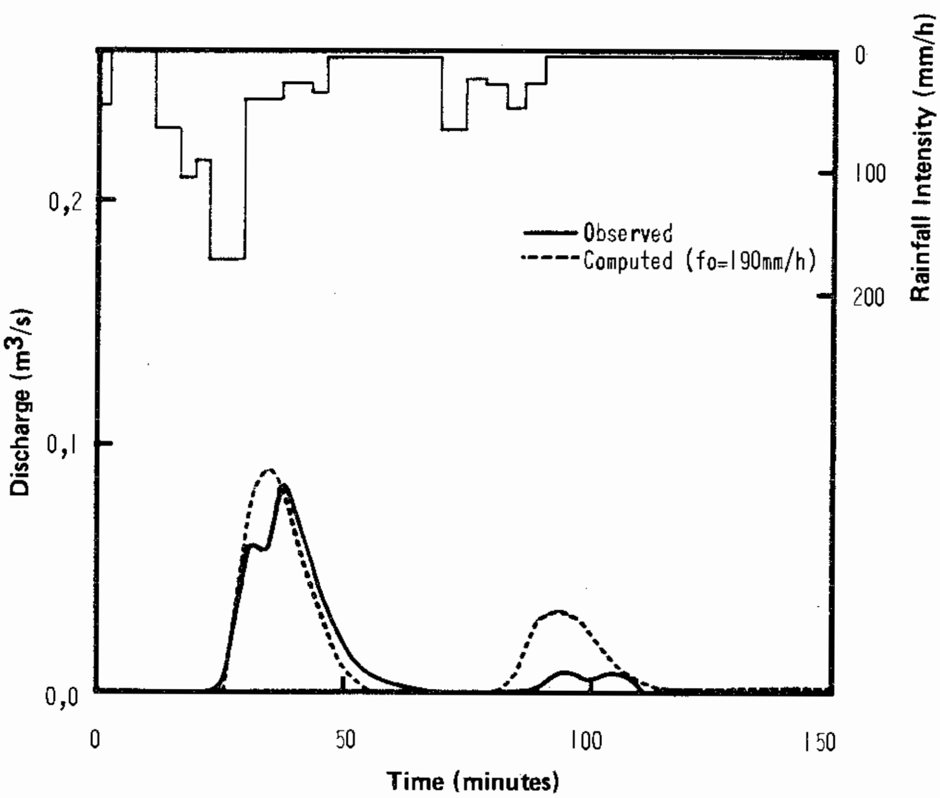


Fig. 5.3 Comparison of computed with observed hydrograph for the storm of 26/6/52 on the Hastings 2-H catchment

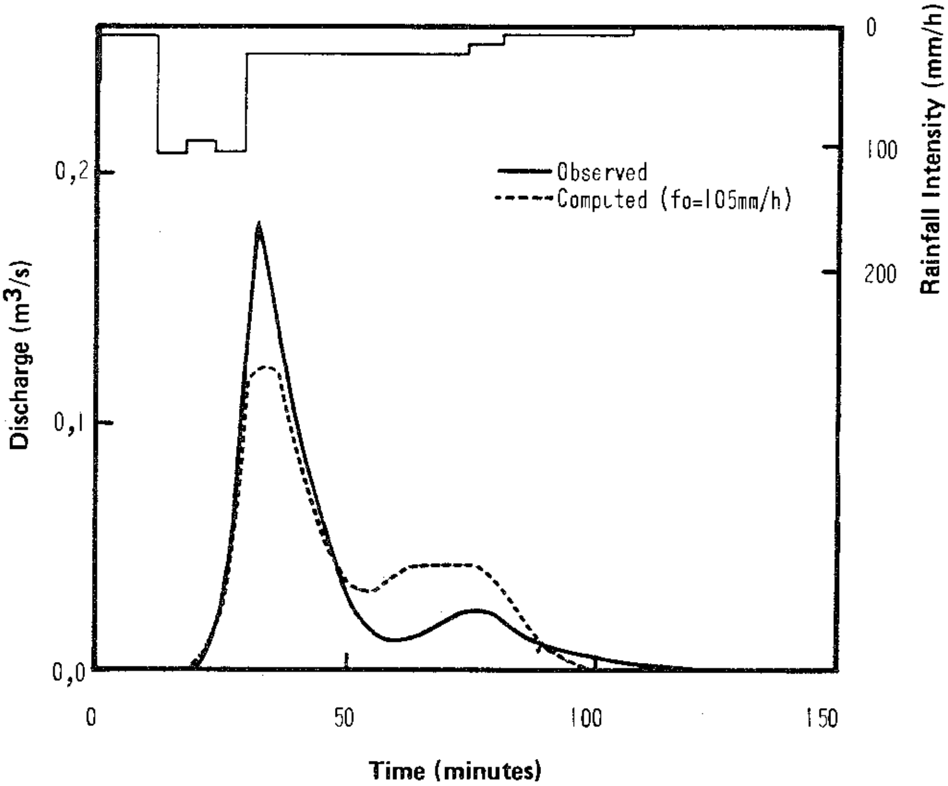


Fig. 5.4 Comparison of computed with observed hydrograph for the storm of 13/7/52 on the Hastings 2-H catchment

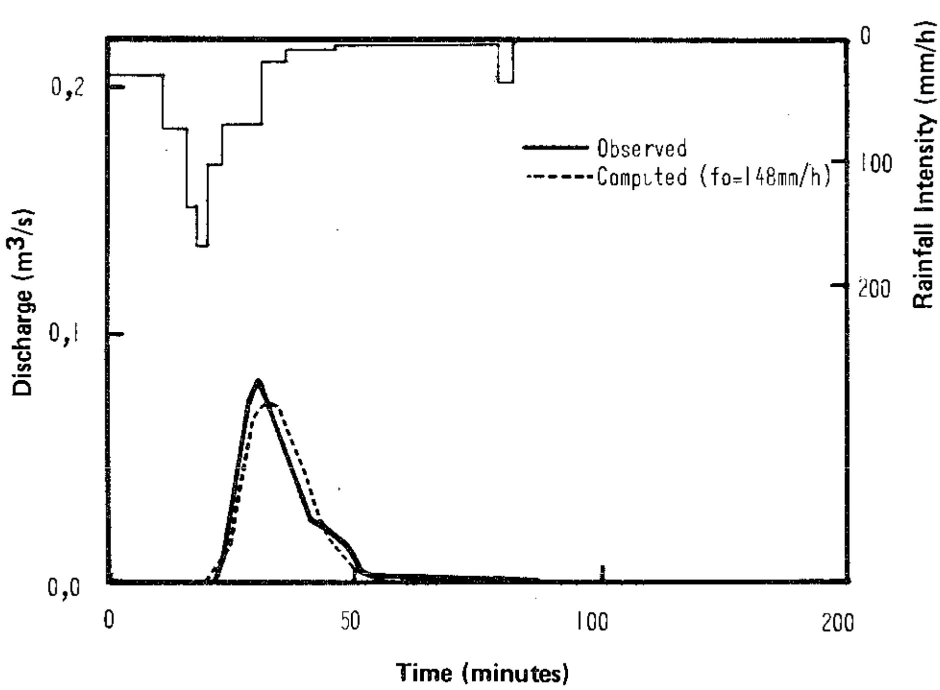


Fig. 5.5 Comparison of computed with observed hydrograph for the storm of 12/6/58 on the Hastings 2-H catchment

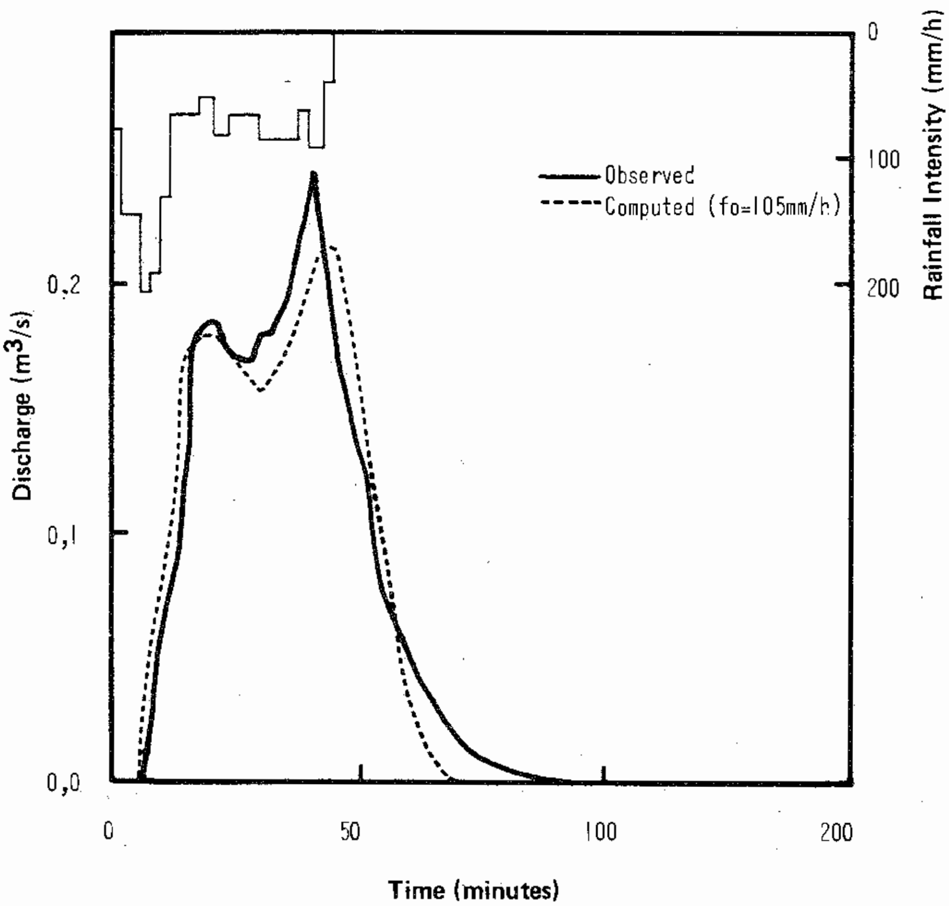


Fig. 5.6 Comparison of computed with observed hydrograph for the storm of 3/7/59 on the Hastings 2-H catchment

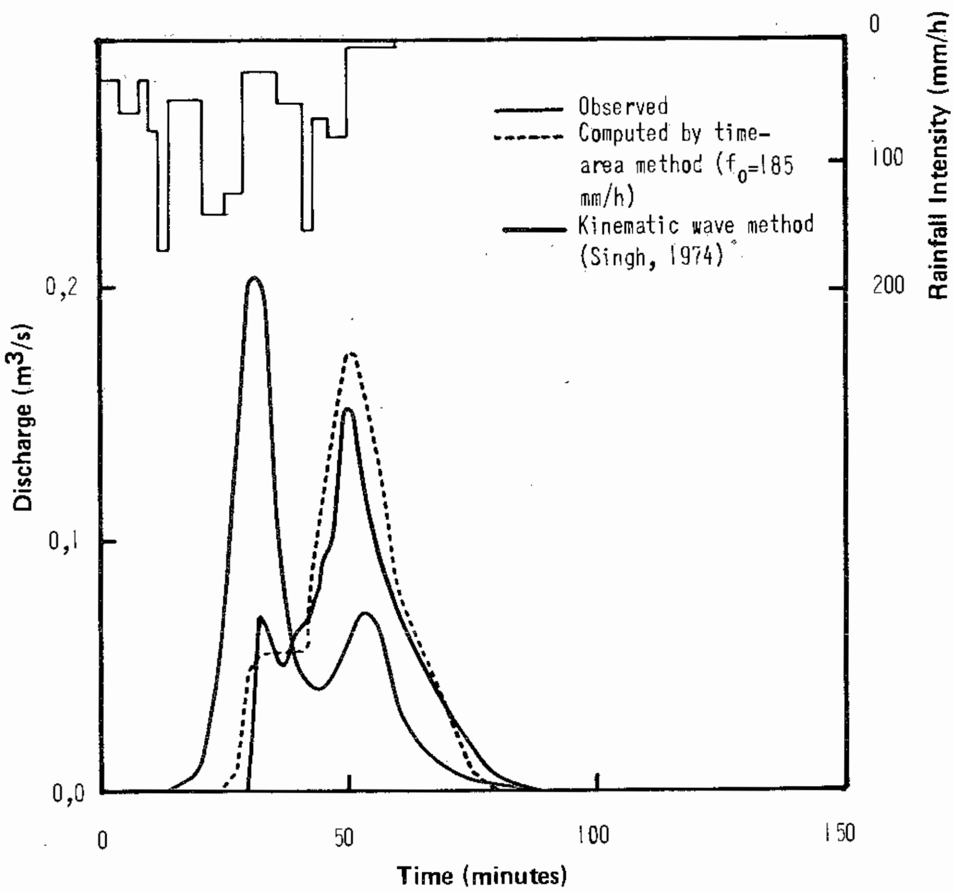


Fig. 5.7 Comparison of computed with observed and kinematic-wave simulated hydrograph for the storm of 15/5/60 on the Hastings 2-H catchment

5.3 Stillwater W-1

The Stillwater W-1 catchment is situated in Oklahoma, USA. It is part of a co-operative research project of the Agricultural Research Service of the USDA and the Oklahoma Agricultural Experiment Station. The catchment is 6,76 ha in area and typical ground slopes are 4%. Fig. 5.8 is a contour map of the catchment. Topsoil is fine-textured with a weak granular structure. The subsoil, which begins at a depth of between 200 and 350 mm, is a silty-clay loam with poor internal drainage and very low permeability.

The catchment was divided into 5 subcatchments as shown in Fig. 5.8. Estimated subcatchment characteristics are shown in Table 5.2. Four storms were available from the USDA publication (Hobbs, 1963). Due to large variations in excess rainfall intensities between storms two time-area diagrams were computed (Fig. 5.9). For the storm of 18/4/57 an average intensity of 100 mm/h was used in eq. 3.2 for determining entry times. For the remaining storms an average intensity of 50 mm/h was used. The loss parameters f_∞, k and d_s were selected as 2 mm/h, $2h^{-1}$ and 5 mm respectively. A computational time increment of 5 minutes was used throughout.

Table 5.2 Stillwater W-1 subcatchment data

Sub-catchment	Area (ha)	Entry time (minutes)		Flow time[1] (minutes)
		50 mm/h	100mm/h	
1	1,35	23	18	2
2	1,08	28	21	2
3	0,56	16	12	2
4	2,62	18	14	1
5	1,15	24	18	1
	6,76			

[1] $Q = 1,0\ m^3/s$

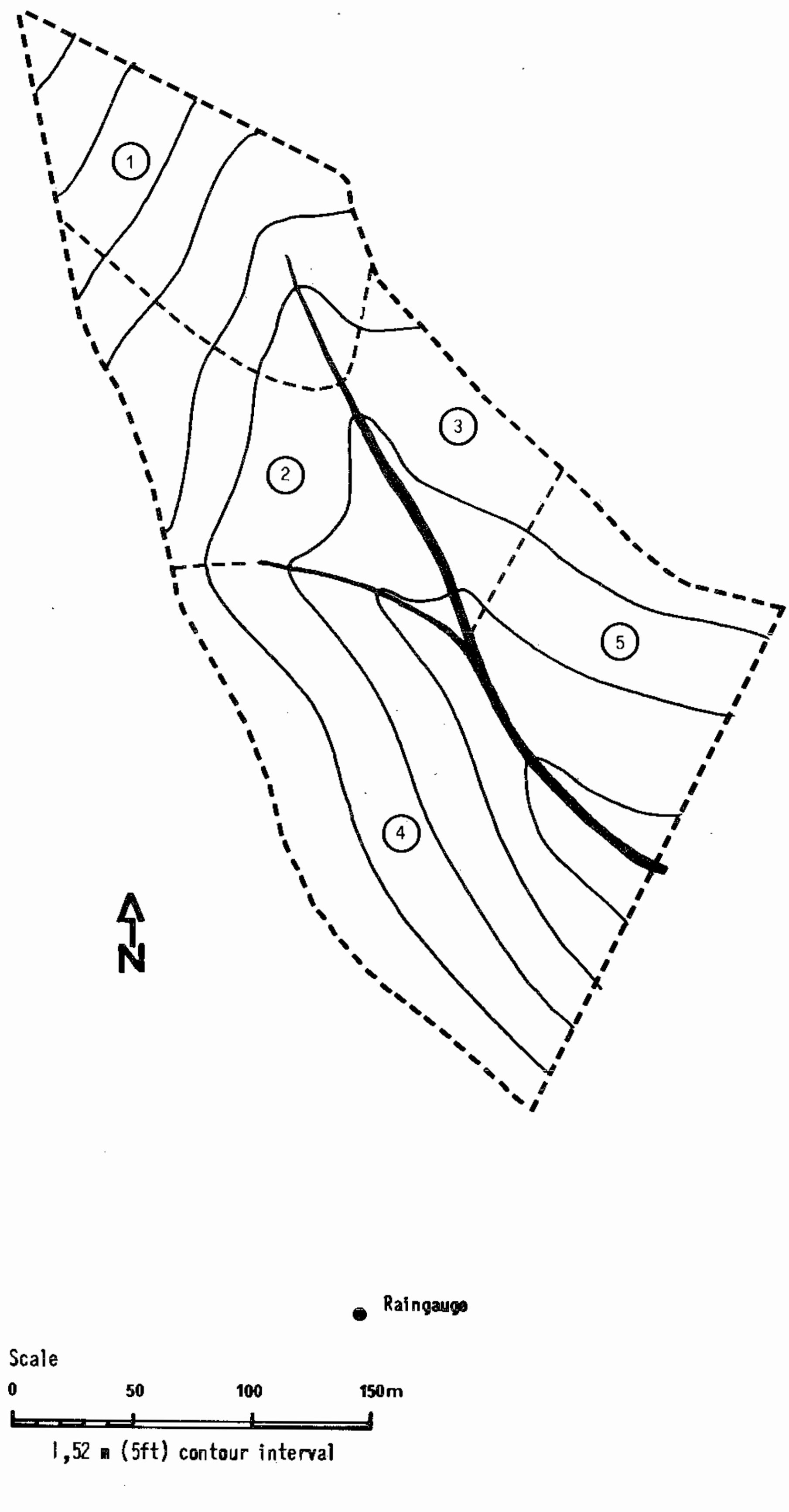


Fig. 5.8 Stillwater catchment W-1, Oklahoma (USA)

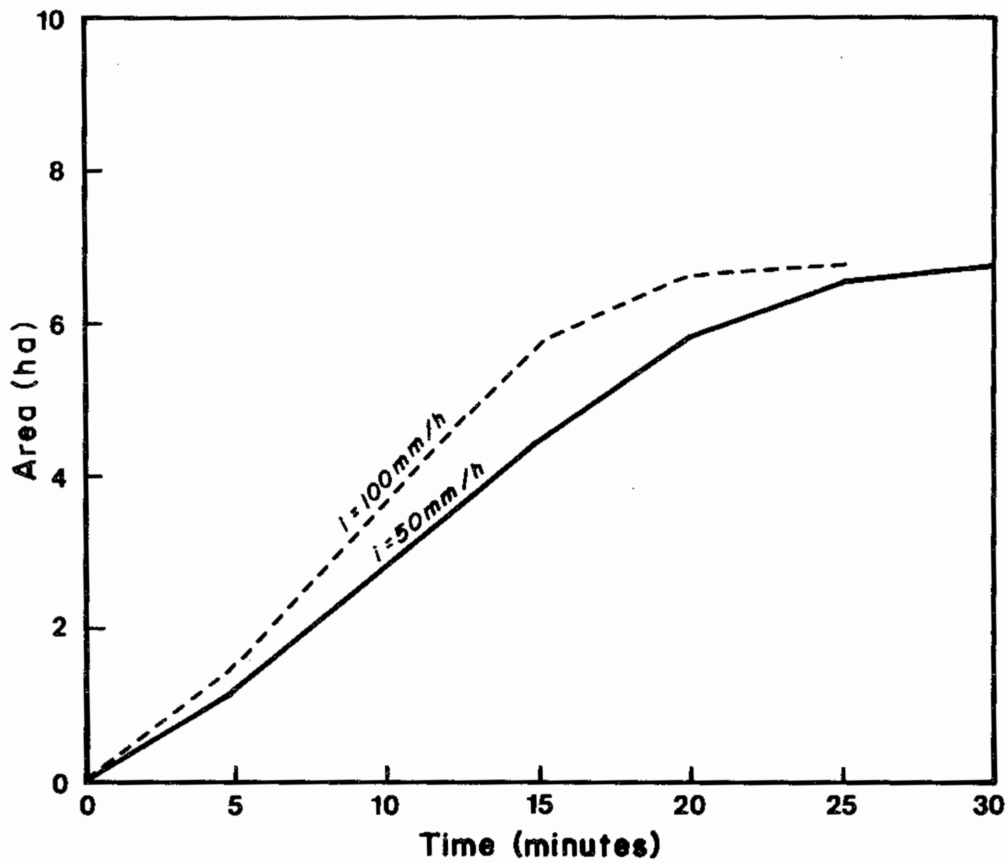


Fig. 5.9 Stillwater W-1 time-area diagrams

Computed and observed hydrographs are compared in Figs. 5.10 to 5.13. Peaks are generally underestimated and the average ratio of computed to observed peak discharge is 0,85 with a standard deviation of 0,17. The high observed peaks could, however, be subject to data errors since the observed peak runoff intensity for the storm of 18/4/57 (Fig. 5.10) was greater than the peak rainfall intensity.

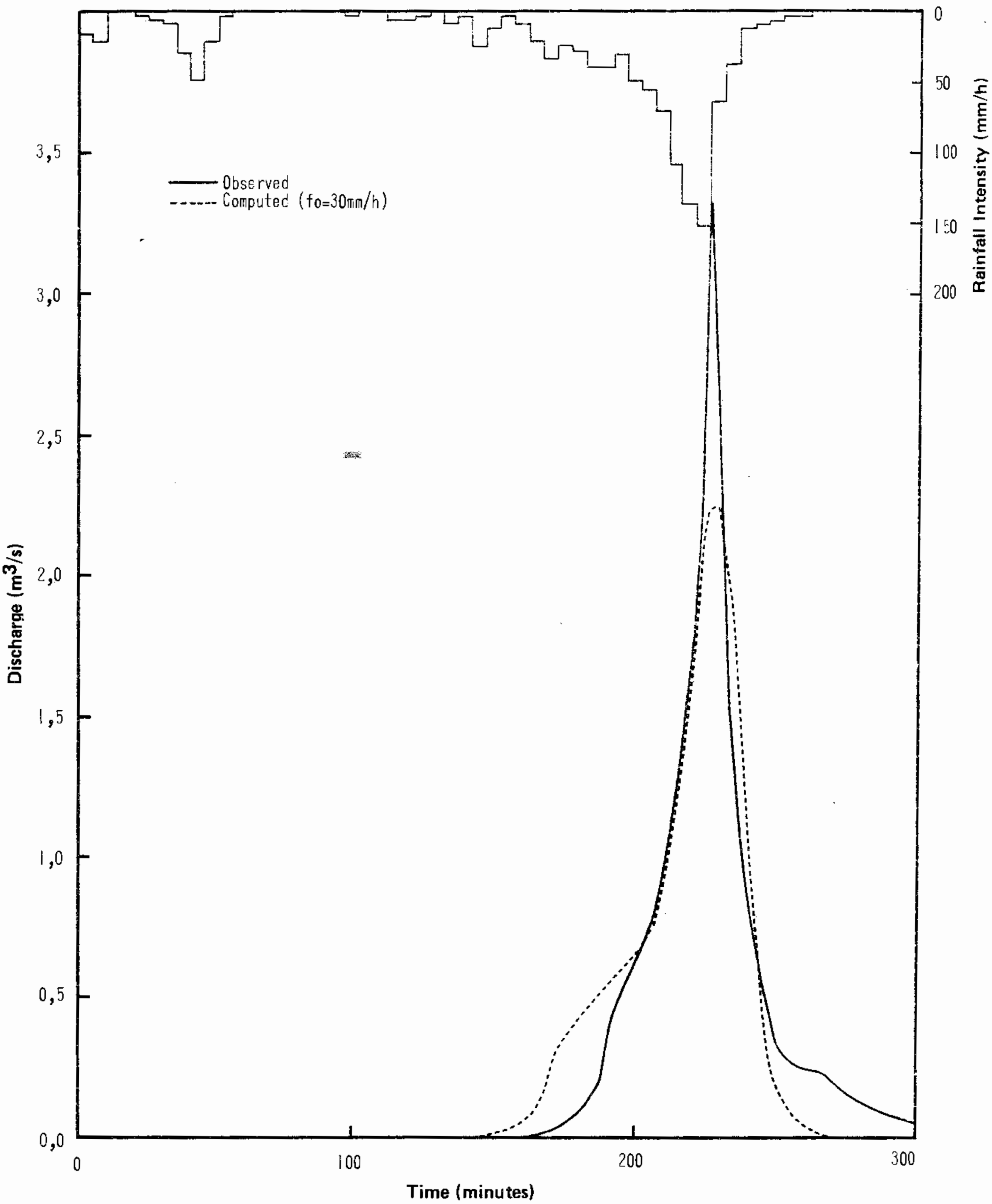


Fig. 5.10 Comparison of computed with observed hydrograph for the storm of 18/4/57 on the Stillwater W-1 catchment

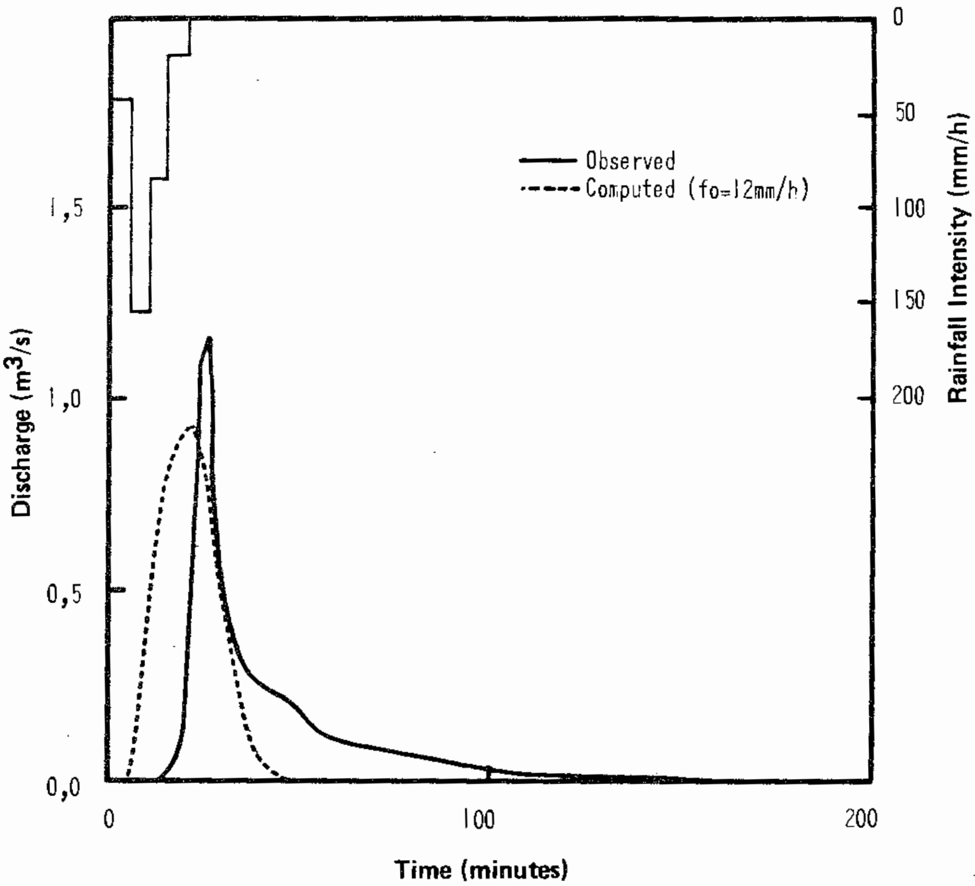


Fig. 5.11 Comparison of computed with observed hydrograph for the storm of 27/6/57 on the Stillwater W-1 catchment

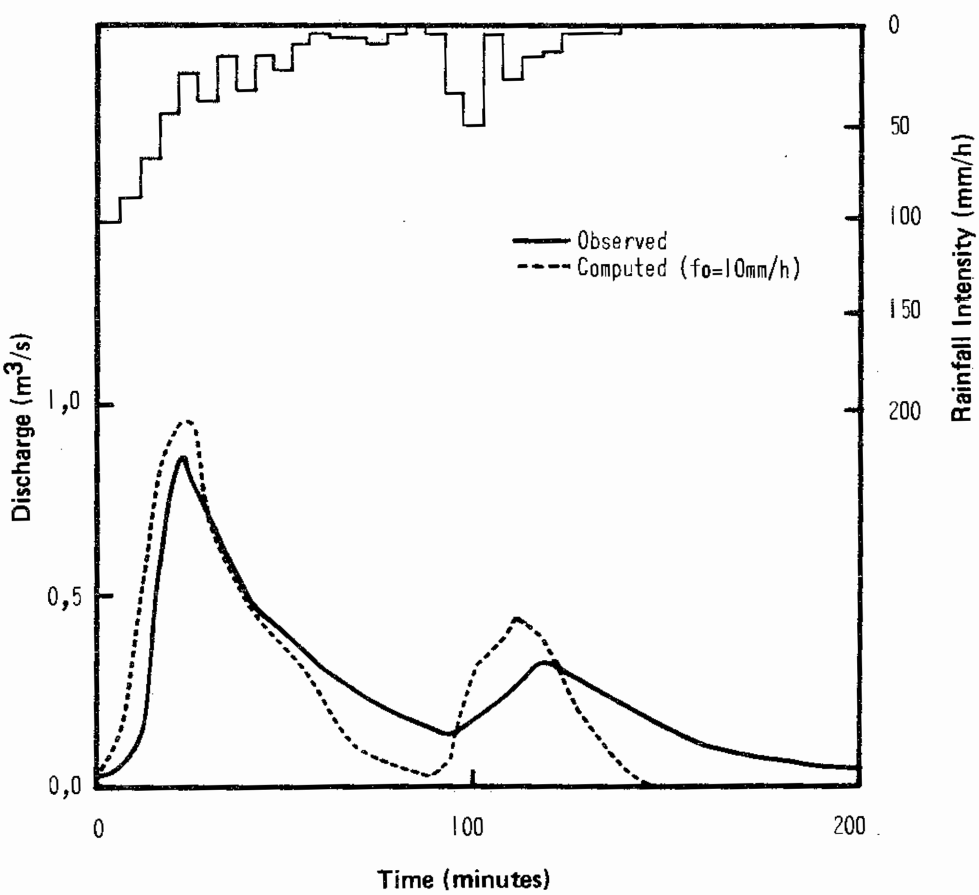


Fig. 5.12 Comparison of computed with observed hydrograph for the storm of 2/10/59(1) on the Stillwater W-1 catchment

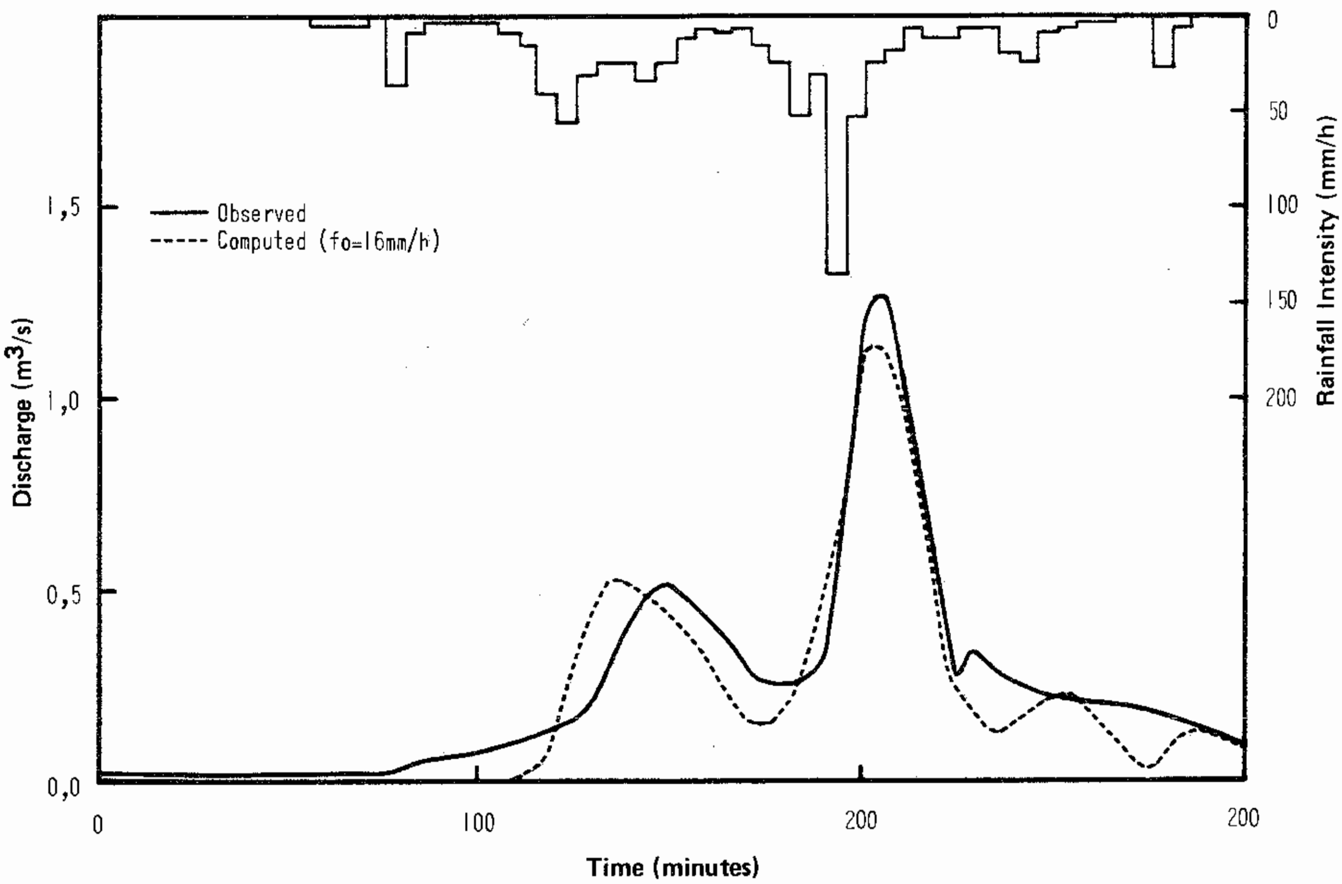


Fig. 5.13 Comparison of computed with observed hydrograph for the storm of 2/10/59(1) on the Stillwater W-1 catchment

5.4 Riesel W-2

The Riesel W-2 catchment is situated in Riesel (Waco), Texas, USA. It is part of a co-operative research project of USDA and Texas Agricultural Experiment Station. The catchment is 52,6 ha in area and has an average ground slope of 2,5%. Fig. 5.14 is a contour map of the area. The soils are deep, fine-textured, granular, of low permeability and alkaline. The internal drainage of the soils is slow. Houston black clay is dominant and the soils are noted for the formation of large extensive cracks upon drying. Approximately 65% of the area is under row crops, 6% native grass pasture, 24% Bermuda grass pasture and 5% gravel roads. The grass pastures are generally located along the waterways.

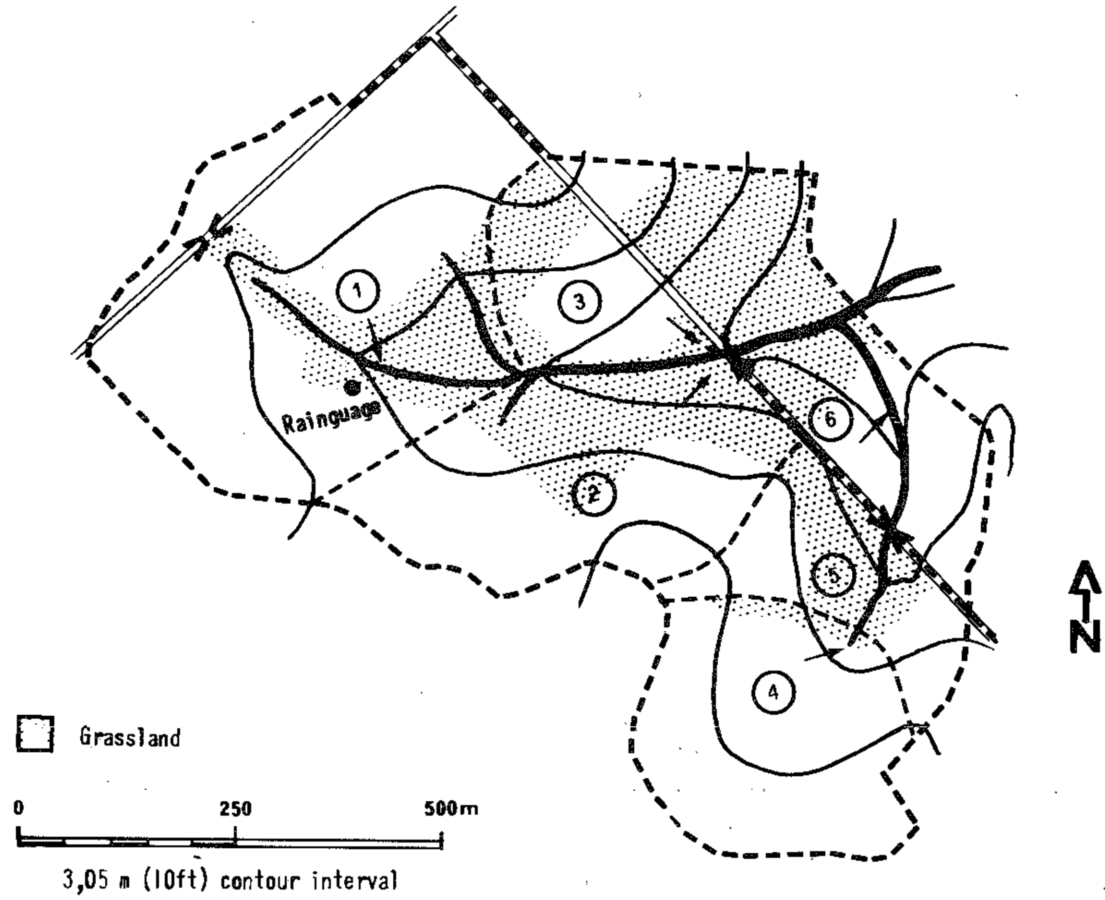


Fig. 5.14 Riesel catchment W-2

The catchment was divided into six subcatchments as shown in Fig. 5.14 and described in Table 5.3. An n value of 0,10 was assumed for overland flow and the computed time-area diagram is presented in Fig. 5.15. The loss parameters f_∞, k and d_s were set equal to 1 mm/h, $2h^{-1}$ and 5 mm respectively. Values for f_o varied between 10 mm/h and 65 mm/h for the different storm events.

Three storms were available from the USDA publication (Hobbs, 1963). Computed and observed hydrographs are compared in Figs. 5.16 to 5.18. There seems to be a synchronization error in the observed data and computed hydrographs had to be shifted about 10 minutes to correspond with observations. A time increment of 5 minutes was used for all computations.

Table 5.3 Riesel W-2 subcatchment data

Sub-catchment	Area (ha)	Entry time[1] (minutes)	Flow time[2] (minutes)
1	17,1	28	7
2	10,3	26	2
3	7,8	27	2
4	8,0	36	4
5	4,5	20	3
6	4,9	18	1
	52,6		

[1] $i = 50$ mm/h

[2] $Q = 2m^3/s$

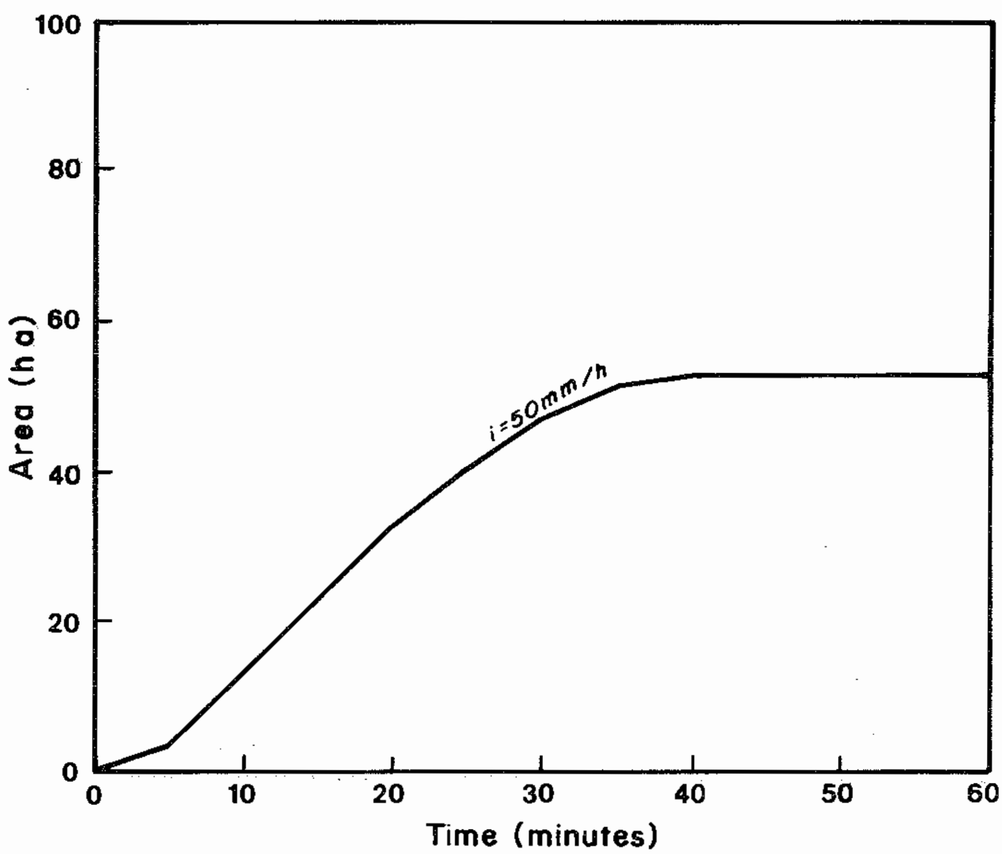


Fig. 5.15 Riesel W-2 time-area diagram

The average ratio of computed to observed peak discharge was 1,17 with a standard deviation of 0,04. The overprediction of peak discharge is due to an underestimation of detention. The Manning n in eq. 3.2 should perhaps have been chosen higher than is typical for row crops, i.e. 0,10, since most of the runoff must pass over the grassed area before reaching the stream. The Manning n for Bermuda grass, which is predominant, is in the range 0,1 to 0,5. Too low a value adopted here could easily account for the increased detention observed.

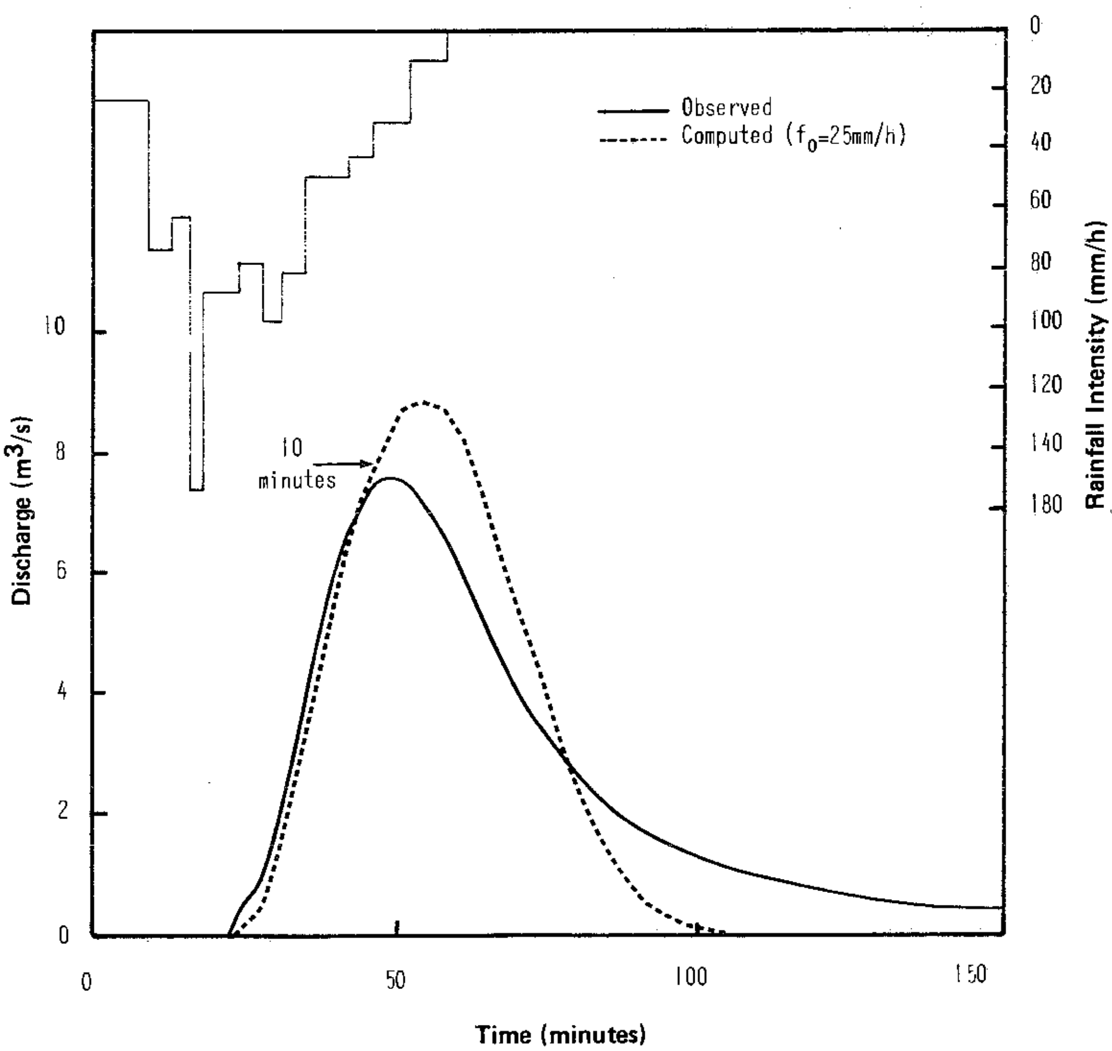


Fig. 5.16 Comparison of computed with observed hydrograph for the storm of 24/4/57 on the Riesel W-2 catchment

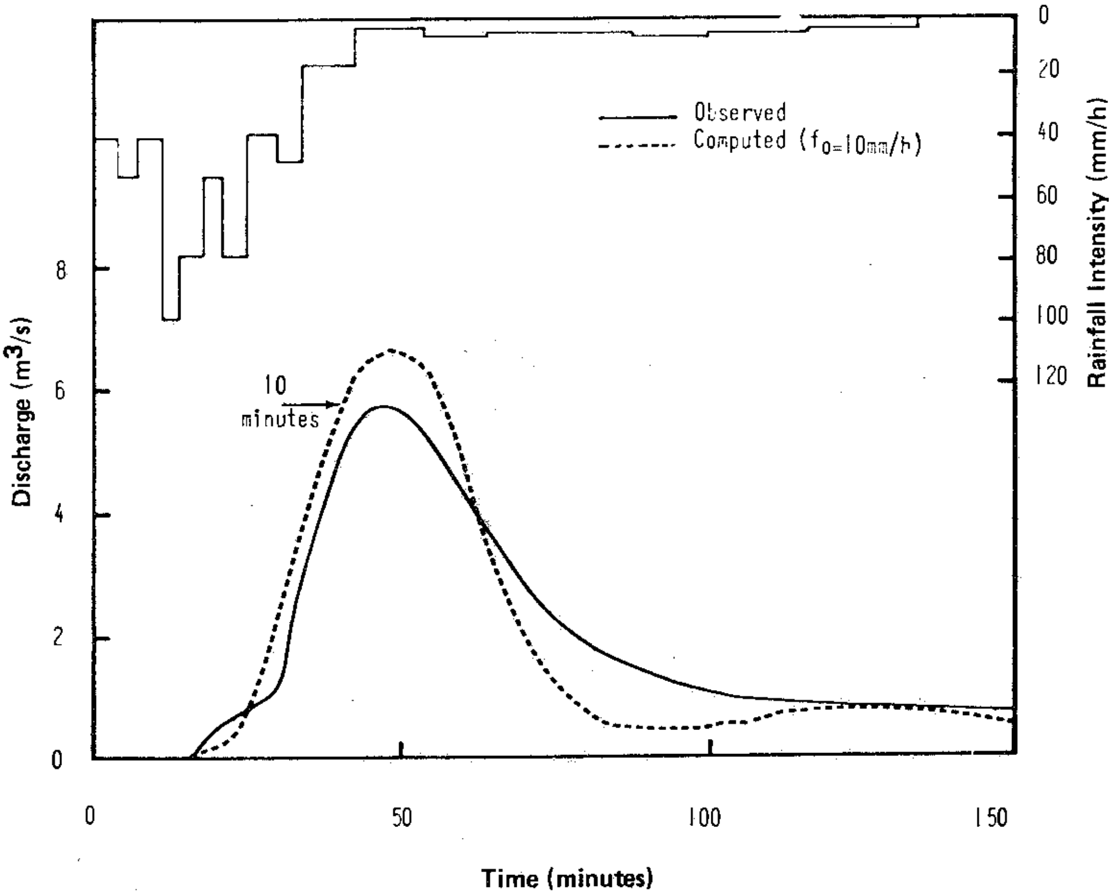


Fig. 5.17 Comparison of computed with observed hydrograph for the storm of 13/5/57 on the Riesel W-2 catchment

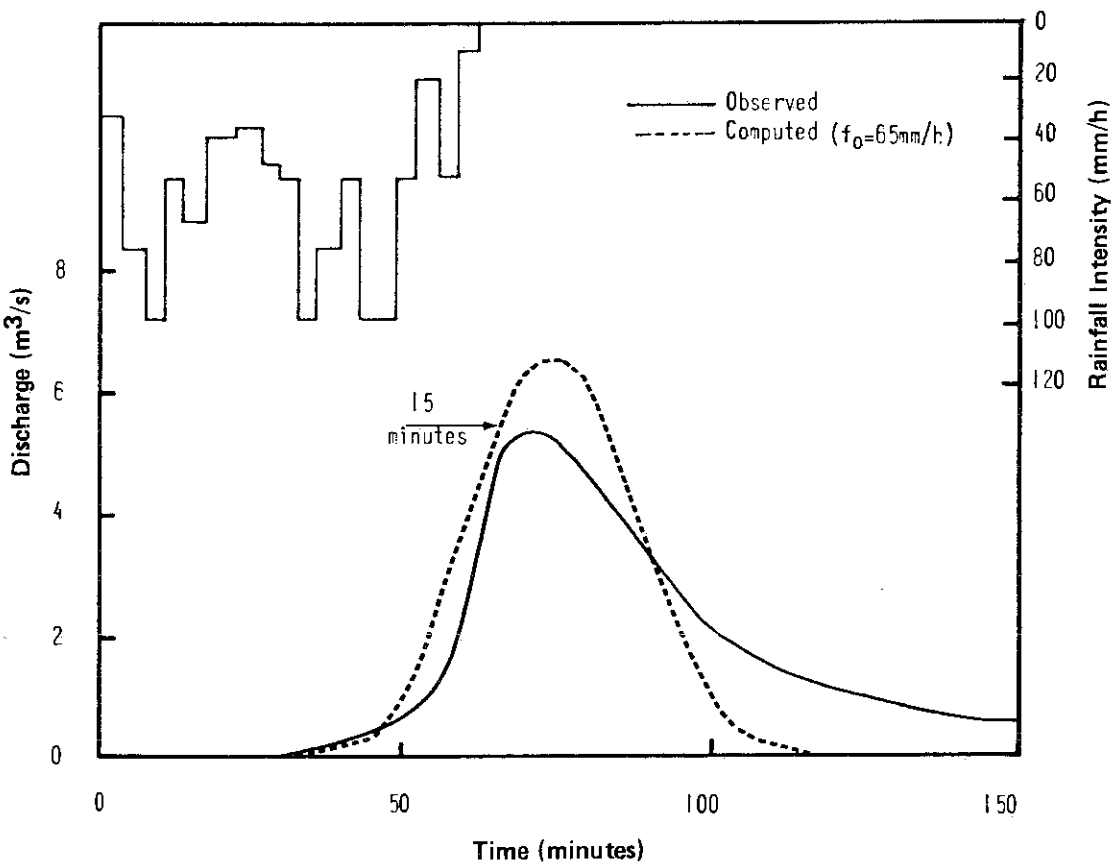


Fig. 5.18 Comparison of computed with observed hydrograph for the storm of 23/6/59 on the Riesel W-2 catchment

5.5 Zululand W1M17

The Zululand W1M17 catchment is one of a number of rural catchments monitored by the University of Zululand over the past few years. The catchments are situated to the northwest of Mtunzini in the Natal coastal belt. Catchment data came from a publication of the University of Zululand (Hope and Mulder, 1979) and rainfall/runoff values were abstracted from the data bank of the Department of Agricultural Engineering of the University of Natal.

The catchment is 66,9 ha in area with typical ground slopes of 12%. Approximately 80% of the surface cover is Ngongoni veld. Most of the remaining area is afforested. The catchment has a rather complex distribution of soil types as illustrated in Fig. 5.20. Rainfall was measured by autographic raingauge located just within the catchment boundary (Fig. 5.19). Runoff was determined from stage measurements at a sharp-crested V-notch.

The complex soil distribution could have been modelled by dividing the catchment into different zones. For simplicity, however, a uniform distribution of losses was assumed throughout the catchment. The loss parameters f_∞, k and d_s were set equal to 3 mm/h, $2h^{-1}$ and 5 mm respectively, while the values of f_o ranged from 3 mm/h to 85 mm/h.

To keep routing assumptions consistent with the simplistic loss assumptions a linear time-area diagram was used. The catchment response time was estimated as 50 minutes.

Five of the larger recorded storms were selected for simulation. Rainfall data were available at 15-minute intervals and for convenience this interval was retained for the computations. Computed and observed hydrographs are compared in Figs. 5.21 to 5.25. Results are pleasing especially considering the gross assumptions made in the analysis. High discharge portions of the hydrographs are well simulated and the average ratio of computed to observed peak discharge is 1,04 with a standard deviation 0,14. The low discharge portions are not particularly well modelled, largely because of neglect of the partial area contribution implicit in

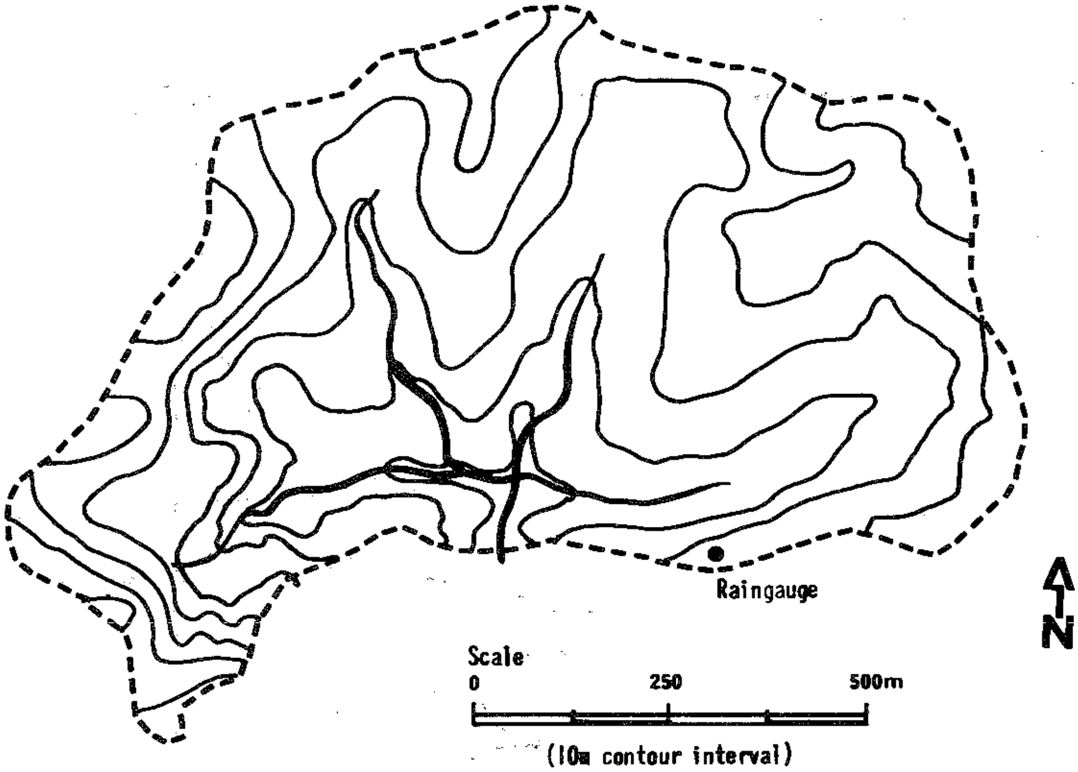


Fig. 5.19 Topography of Zululand catchment W1M17

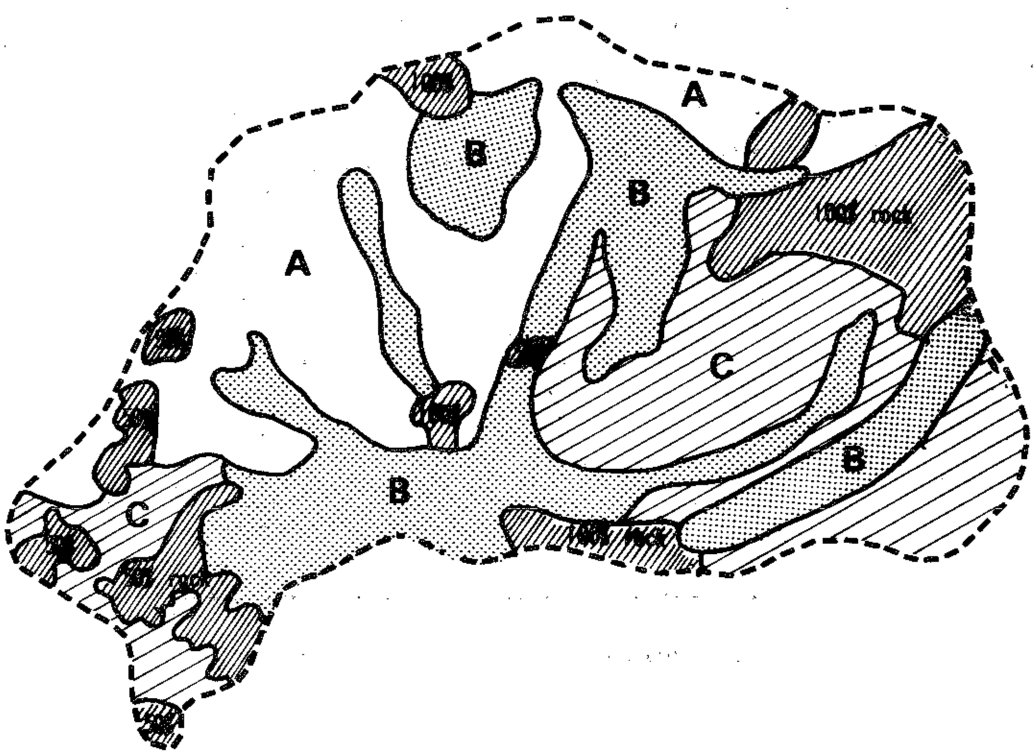


Fig. 5.20 Distribution of soil types for Zululand catchment W1M17

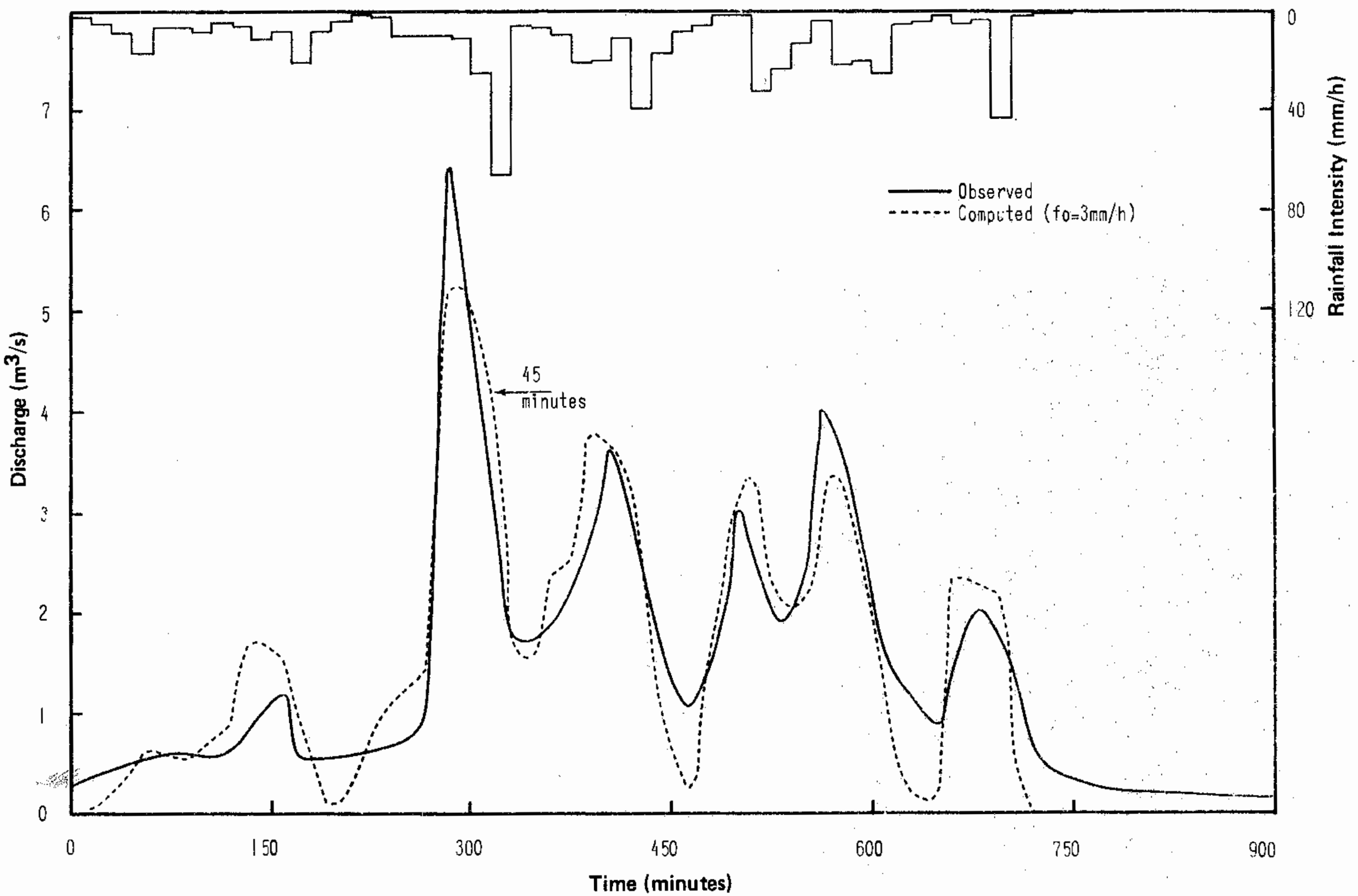


Fig. 5.21 Comparison of computed with observed hydrograph for the storm of 6/2/77 on the Zululand W1M17 catchment

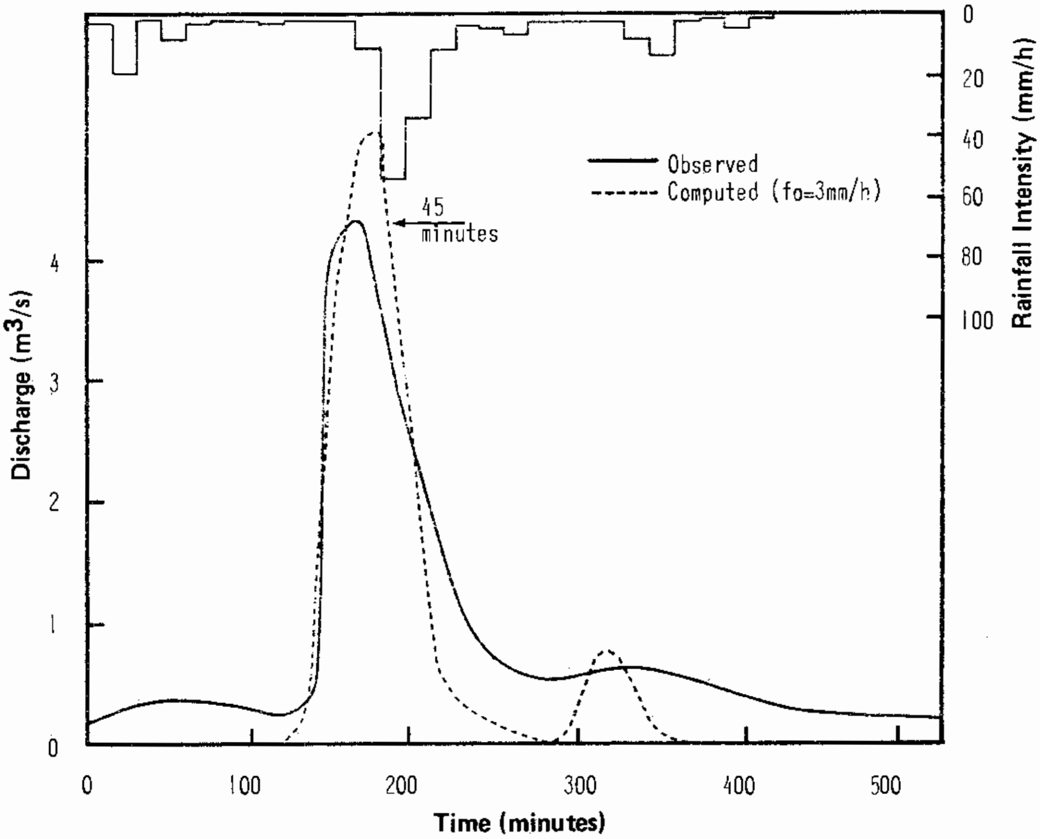


Fig. 5.22 Comparison of computed with observed hydrograph for the storm of 7/2/77 on the Zululand W1M17 catchment

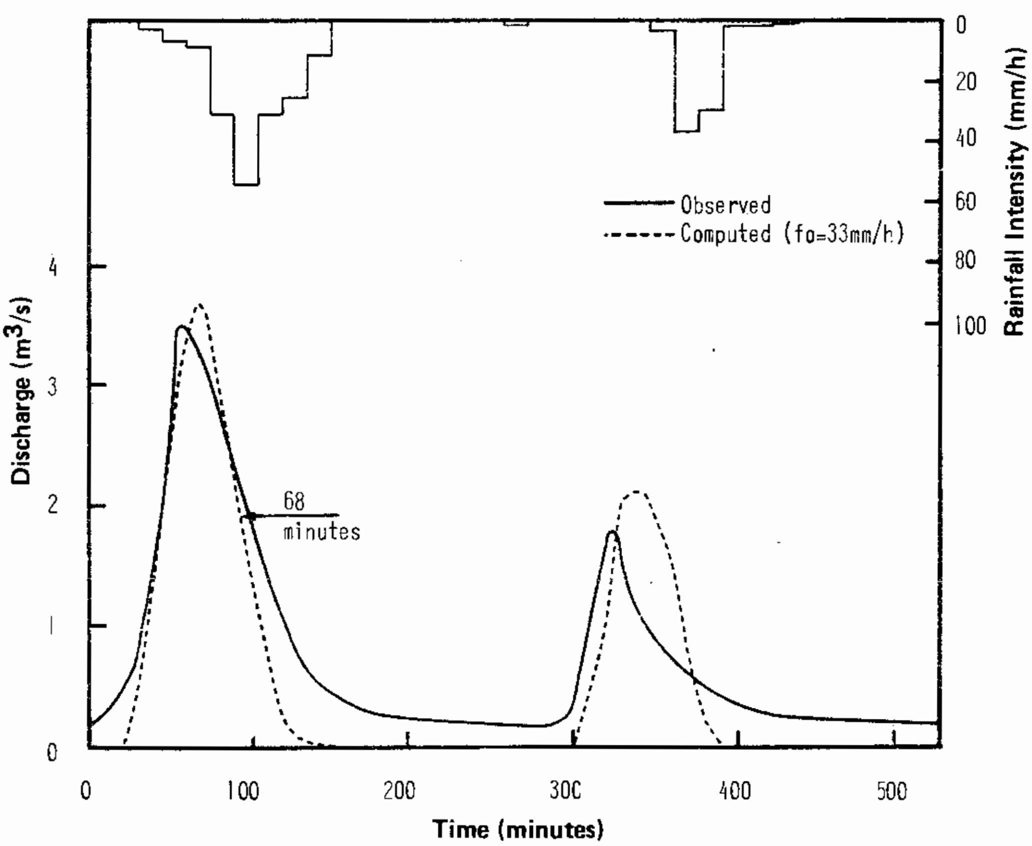


Fig. 5.23 Comparison of computed with observed hydrograph for the storm of 8/2/77 on the Zululand W1M17 catchment

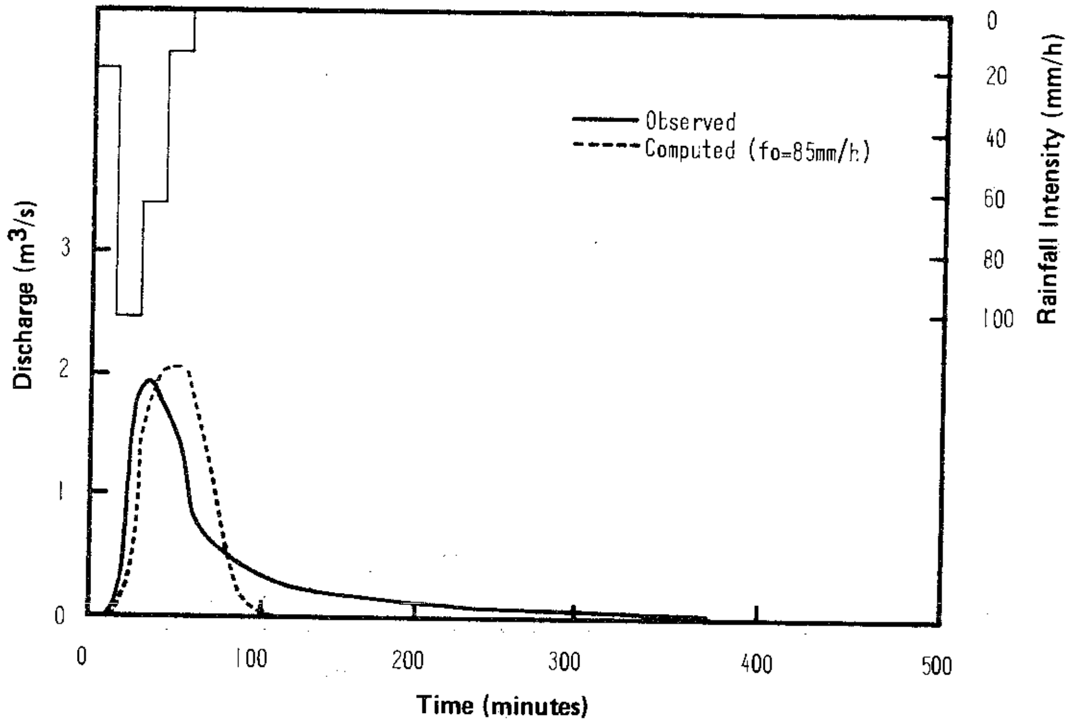


Fig. 5.24 Comparison of computed with observed hydrograph for the storm of 9/11/77 on the Zululand W1M17 catchment

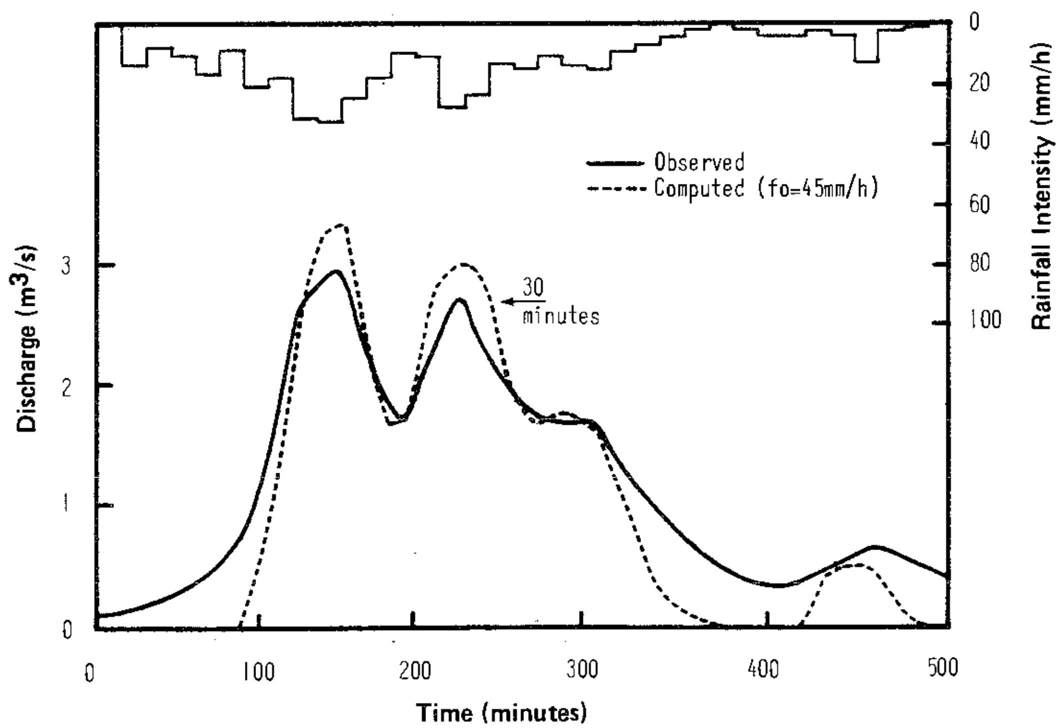


Fig. 5.25 Comparison of computed with observed hydrograph for the storm of 21/1/78 on the Zululand W1M17 catchment

the assumption of uniform loss parameters. The storm of 8/2/77 had to be treated as two separate events to allow for recovery of depression storage.

Synchronization errors are immediately evident in the manner in which rainfall peaks lag behind observed runoff peaks. An average time shift of 40 minutes had to be made to allow for this error.

5.6 Stillwater W-4

The Stillwater W-4 catchment is monitored as part of the same research project as the Stillwater W-1 catchment (section 5.3) and is located in the same vicinity. The area of the catchment is 83,4 ha and typical ground slopes are 5%. Topsoils are fine to medium textured and range from 50 to 300 mm in depth. Subsoils are silty loams and silty clay loams with generally low permeabilities. Surface cover is native grassland which, during the storms analysed, was in poor to fair condition.

The catchment was divided into 10 subcatchments as shown in Fig. 5.26. Estimated subcatchment characteristics are given in Table 5.4 and the computed time-area diagrams are shown in Fig. 5.27. An excess rainfall intensity of 50 mm/h was used in eq. 3.2 for computing entry times for the storms of 18/4/57 and 2/10/59(1). Entry times for the remaining two events were computed assuming an excess rainfall intensity to be 20 mm/h. The loss parameters f_∞, k and d_s were set equal to 3 mm/h, $2h^{-1}$ and 5 mm respectively. Values for the parameter f_o varied between 3 mm/h and 33 mm/h for the various storm events. A time step of 10 minutes was used for all computations.

Computed and observed hydrographs are compared in Figs. 5.28 to 5.31. Once again high discharges are well simulated but not low flows. The average ratio of computed to observed peak discharges is 1,08 with a standard deviation of 0,09.

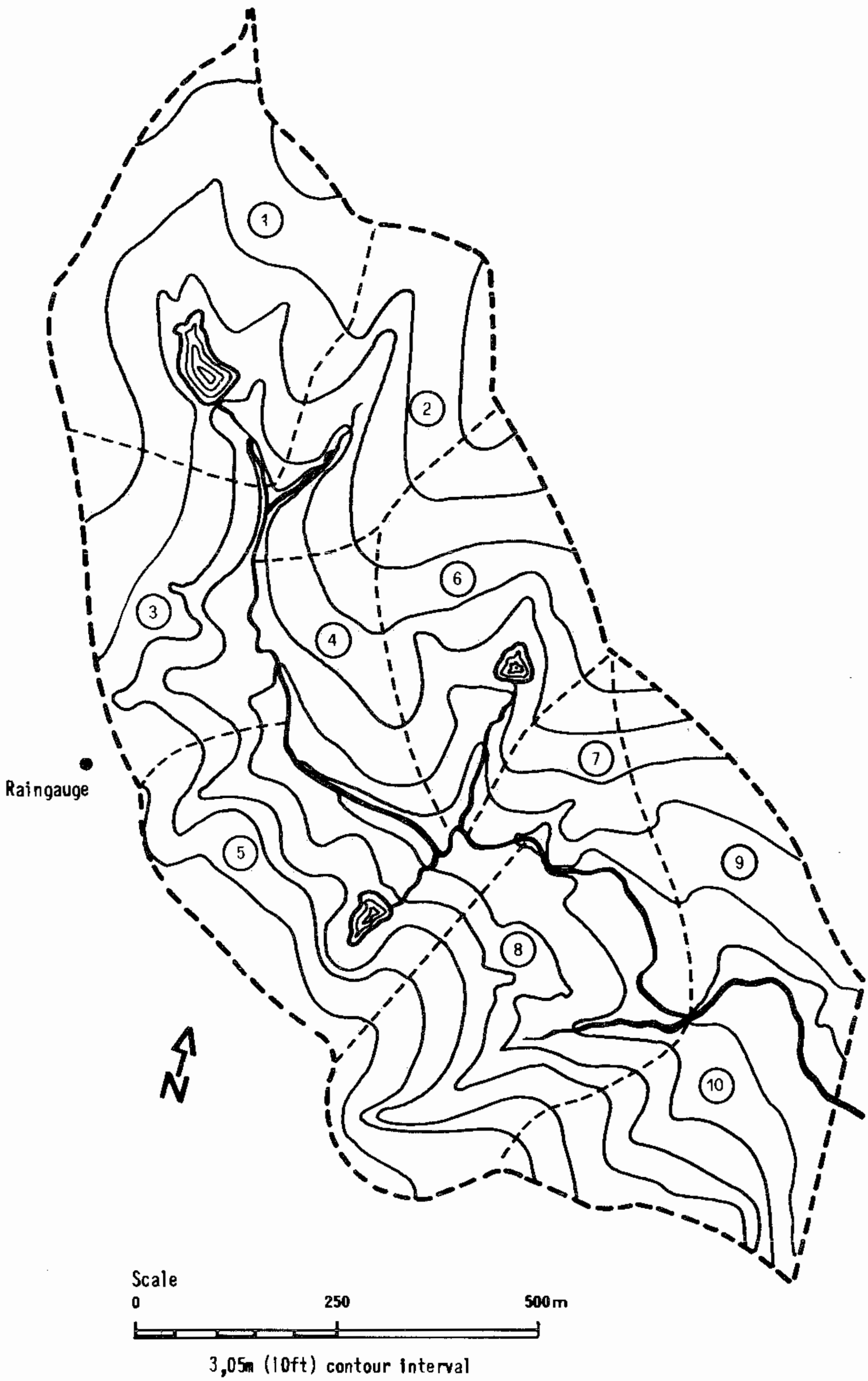


Fig. 5.26 Stillwater catchment W-4, Oklahoma (USA)

Table 5.4 Stillwater W-4 subcatchment data

Sub-catchment	Area (ha)	Entry time (minutes)		Flow time (minutes)
		20 mm/h	50 mm/h	
1	14,2	58	40	12
2	7,3	38	26	12
3	7,5	44	30	10
4	4,5	35	24	8
5	10,2	37	25	8
6	9,6	45	31	8
7	4,6	42	29	5
8	11,4	35	24	5
9	6,3	61	42	2
10	7,8	52	36	2
	83,4			

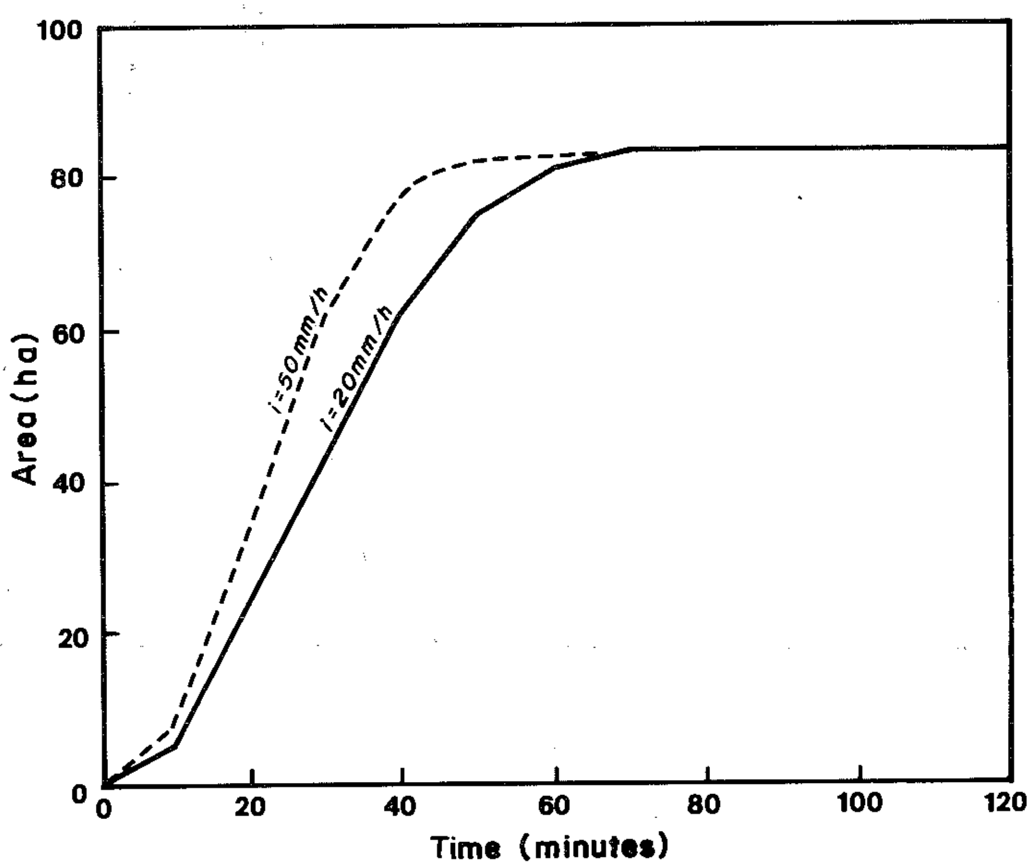


Fig. 5.27 Stillwater W-4 time-area diagram

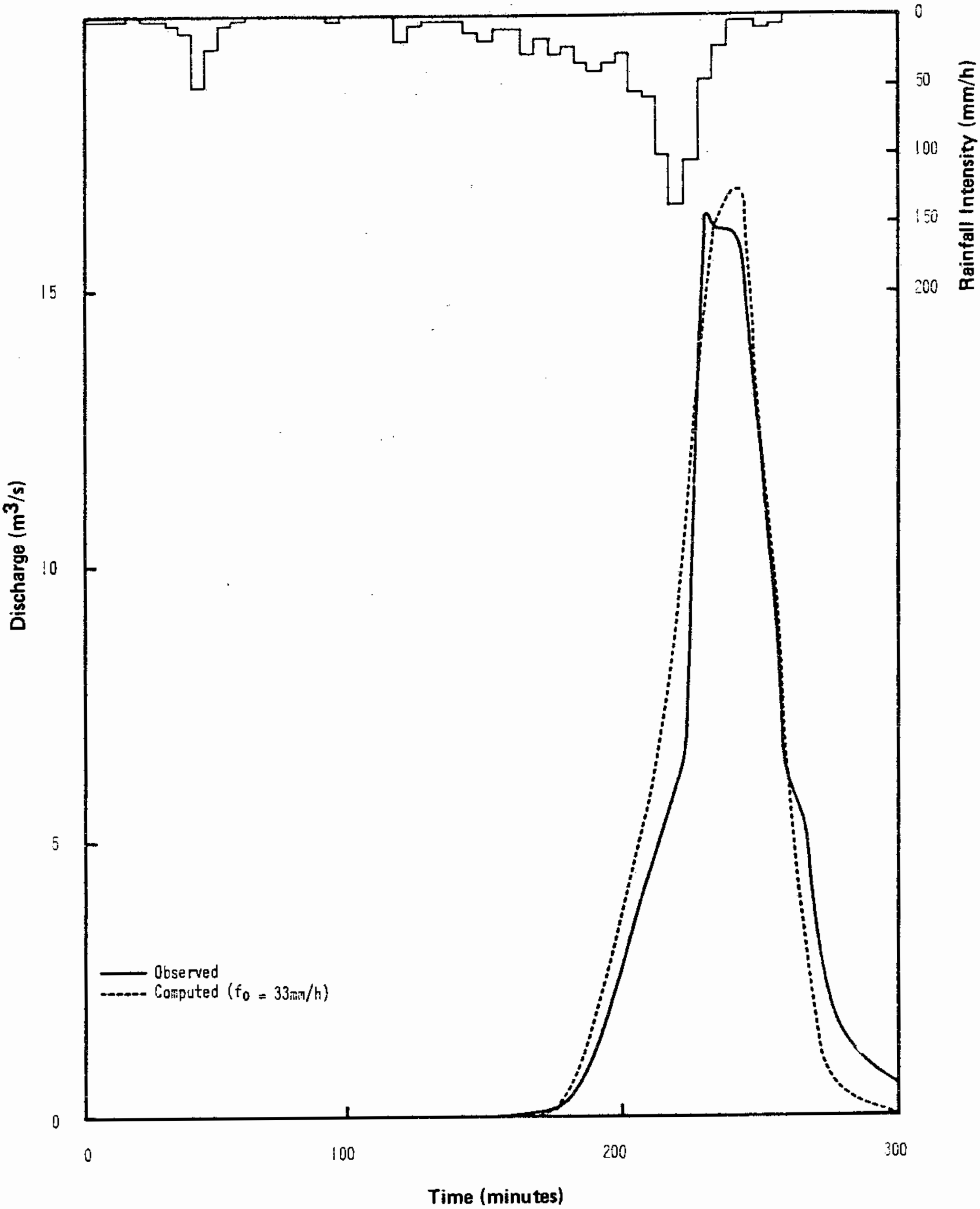


Fig. 5.28 Comparison of computed with observed hydrograph for the storm of 18/4/57 on the Stillwater W-4 catchment

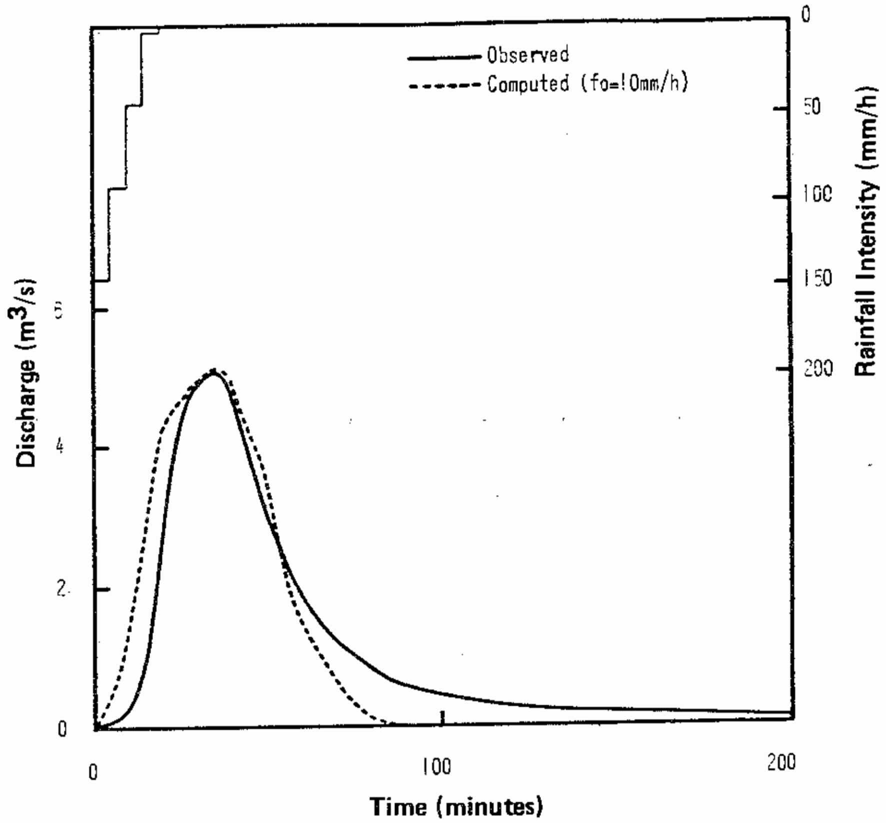


Fig. 5.29 Comparison of computed with observed hydrograph for the storm of 27/6/57 on the Stillwater W-4 catchment

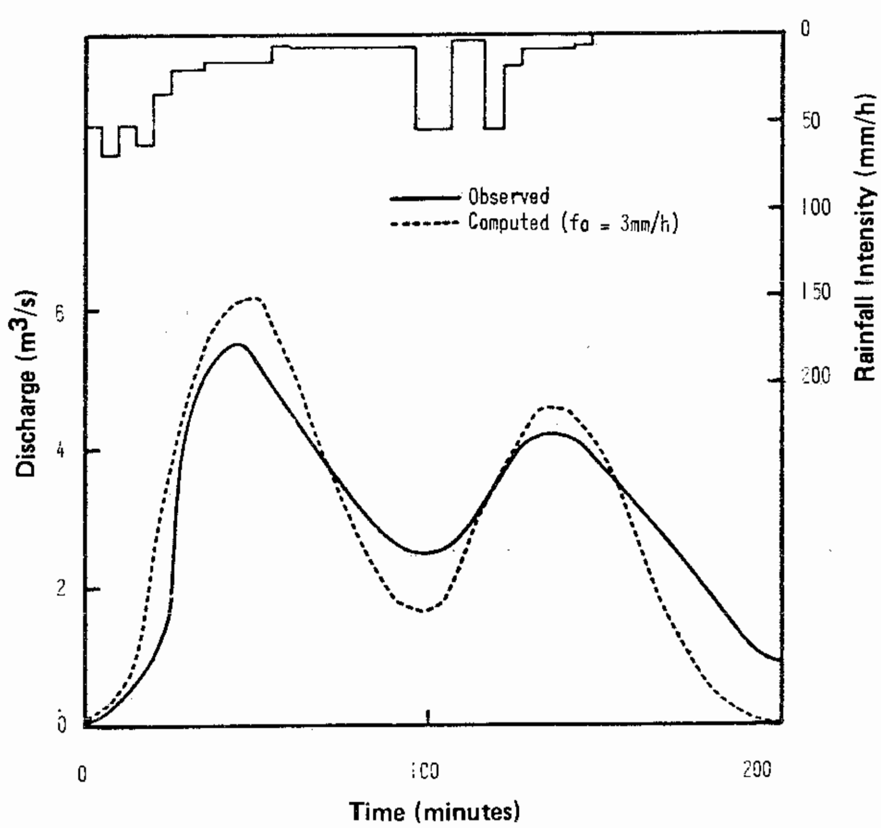


Fig. 5.30 Comparison of computed with observed hydrograph for the storm of 2/10/59(2) on the Stillwater W-4 catchment

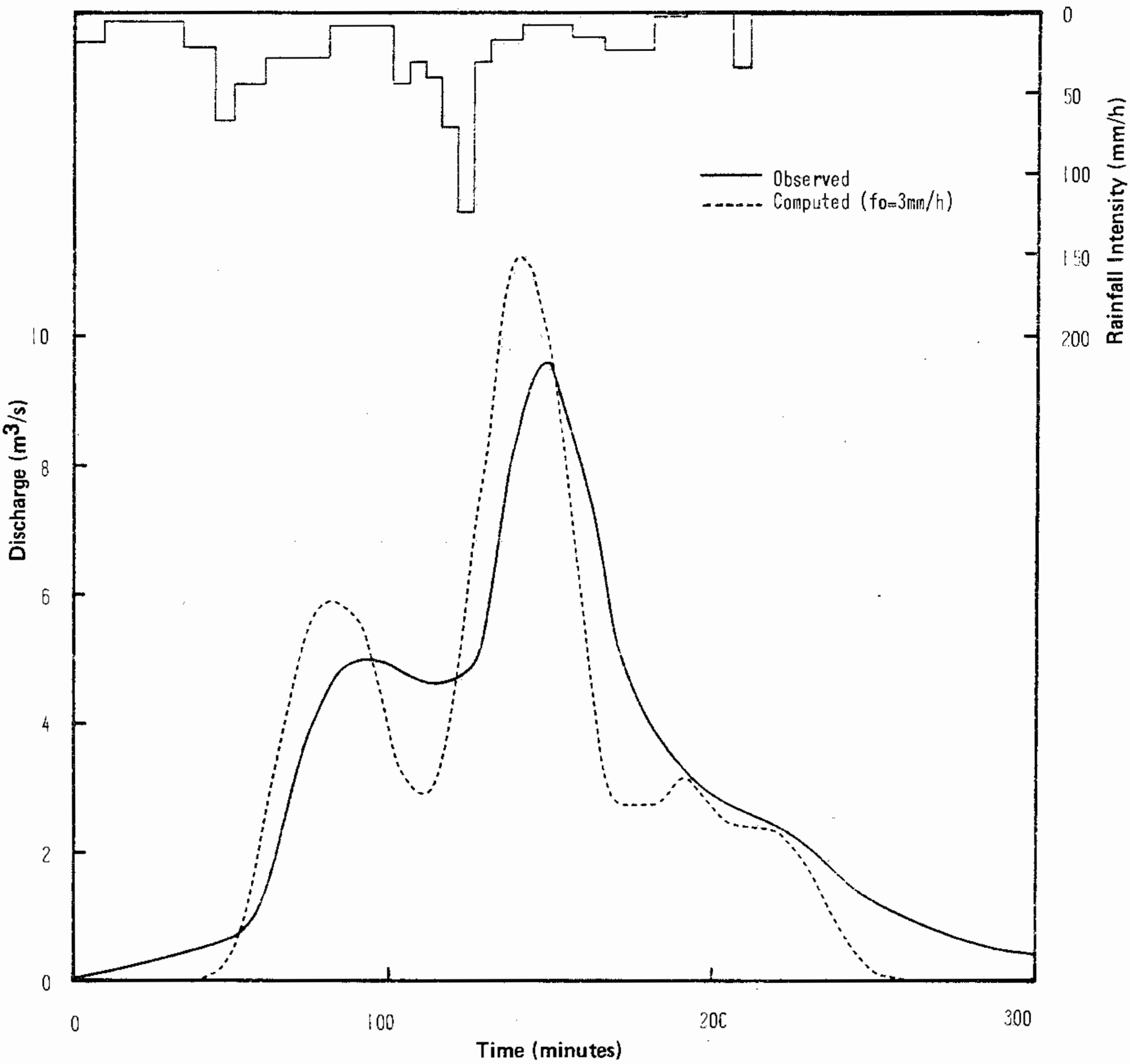


Fig. 5.31 Comparison of computed with observed hydrograph for the storm of 2/10/59(1) on the Stillwater W-4 catchment

5.7 Riesel Y

The Riesel Y catchment is monitored as part of the same research project as the Riesel W-2 catchment and is in the same vicinity. The area of the catchment is 125,1 ha and average ground slopes are 2,4%. Soils are the same as for the W-2 catchment and land use is predominantly agricultural. About 65% of the area is under crops and the remainder given over to Bermuda and native grass pasture. The cultivated land is terraced and contour-tilled. The grasslands are concentrated along the waterways.

The area was divided into ten subcatchments as shown in Fig. 5.32. The estimated subcatchment characteristics are listed in Table 5.5. Manning n was set at 0,10 for overland flow. Because of significant differences in excess rainfall intensities two time-area diagrams were computed. An average intensity of 20 mm/h was used in eq. 3.2 for determining entry times for the event of 23/6/59, while 50 mm/h was assumed for the other events. The loss parameters f_∞, k and d_s were set equal to 1 mm/h, 2 h^{-1} and 5 mm respectively. Values of f_o varied between 10 mm/h and 80 mm/h for the various storm events. A 5-minute time step was adopted for computing the excess rainfall and a 10-minute step for the routing computations. For the storm of 23/6/59 depression storage was assumed to have been filled by antecedent rain.

Three events were selected from the USDA publication (Hobbs, 1963). Computed hydrographs compare reasonably well with observed and are shown in Figs. 5.34 to 5.36. Low flows are underestimated on the recessions of the hydrographs. This could, however, be attributable to data errors. For the storm of 24/4/57 recorded runoff is greater than observed rainfall and it is suspected that the error is in the unnaturally long recession of the hydrograph. Peaks are nevertheless well reproduced - the average ratio of computed to observed peaks is 0,96 with a standard deviation of 0,06.

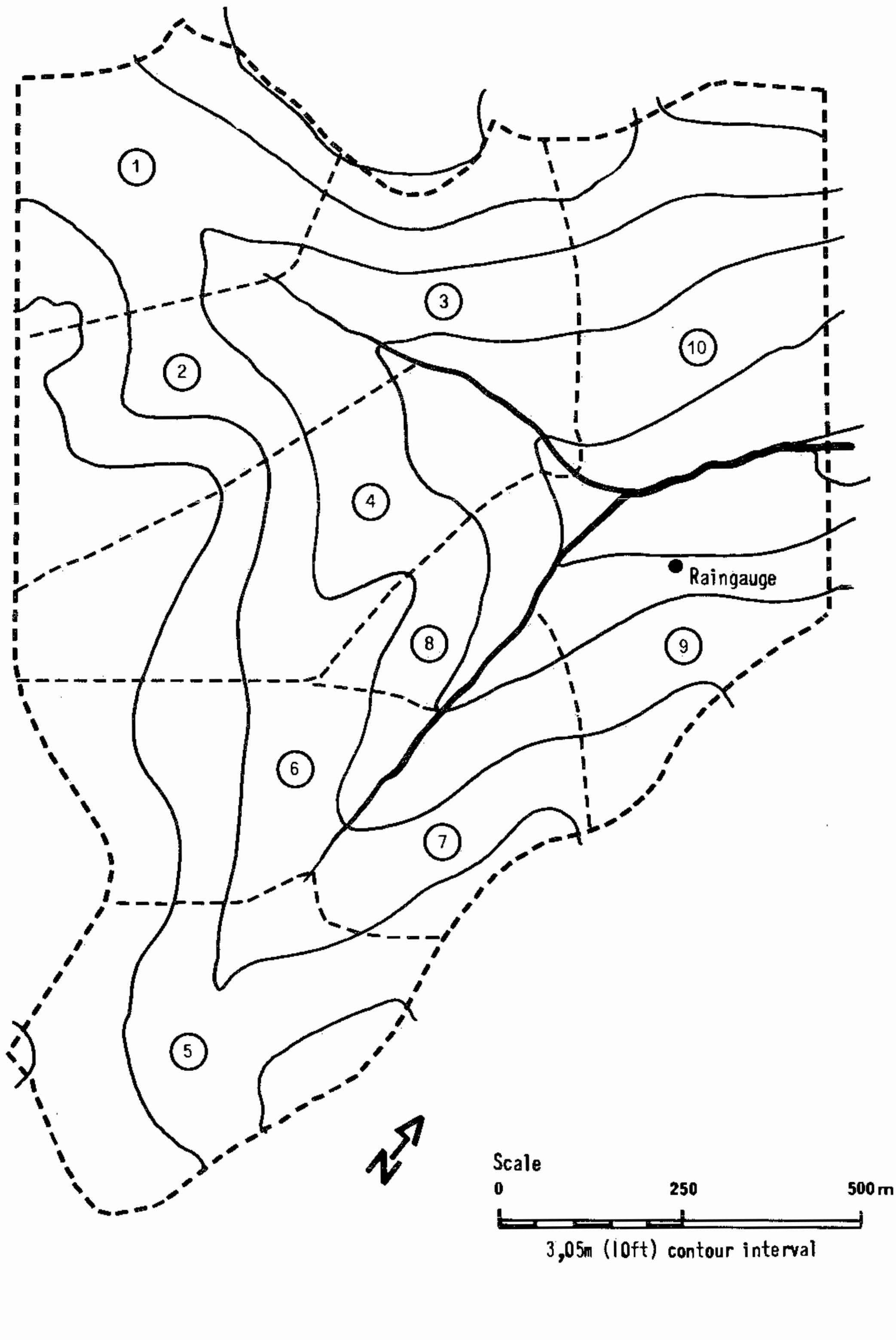


Fig. 5.32 Riesel catchment Y

Table 5.5 Riesel Y subcatchment data

Sub-catchment	Area (ha)	Entry time (minutes)		Flow time (minutes)
		20 mm/h	50 mm/h	
1	13,8	46	32	9
2	12,8	68	47	7
3	10,5	39	27	5
4	17,6	69	48	5
5	16,5	48	33	9
6	10,5	65	45	7
7	8,6	45	31	7
8	6,0	35	24	4
9	11,8	49	34	2
10	17,0	69	48	2
	125,1			

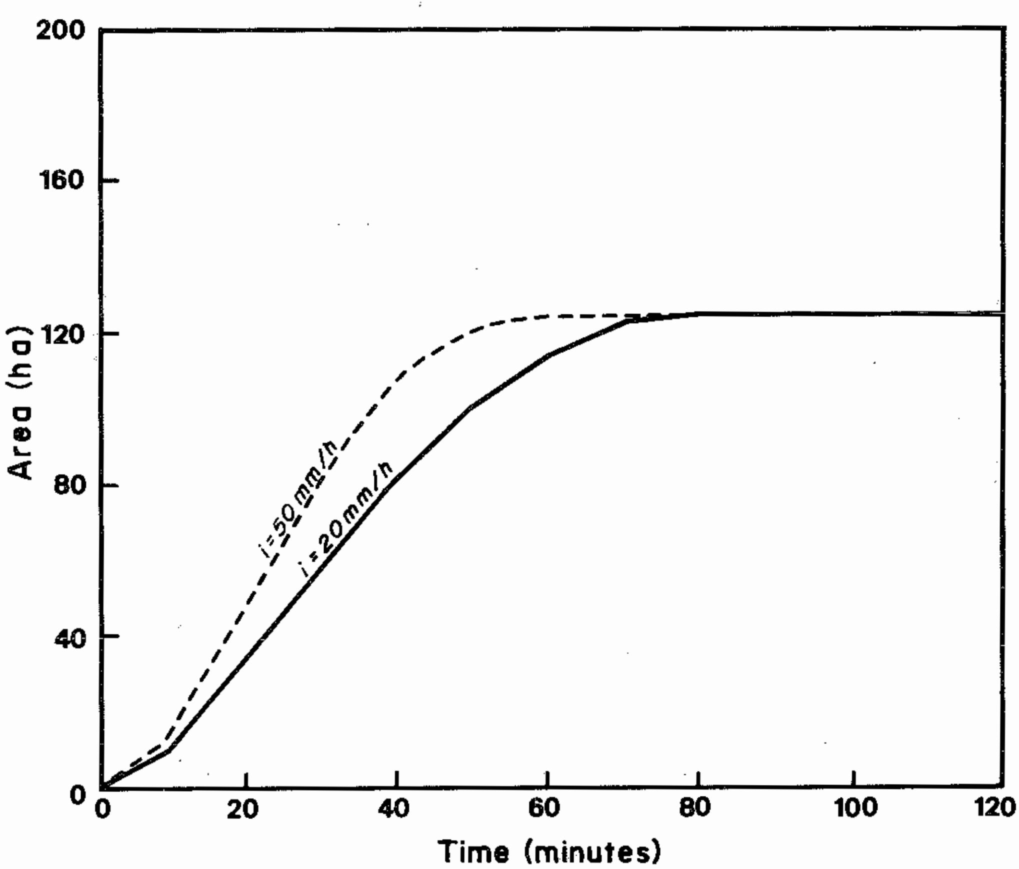


Fig. 5.33 Riesel Y time-area diagrams

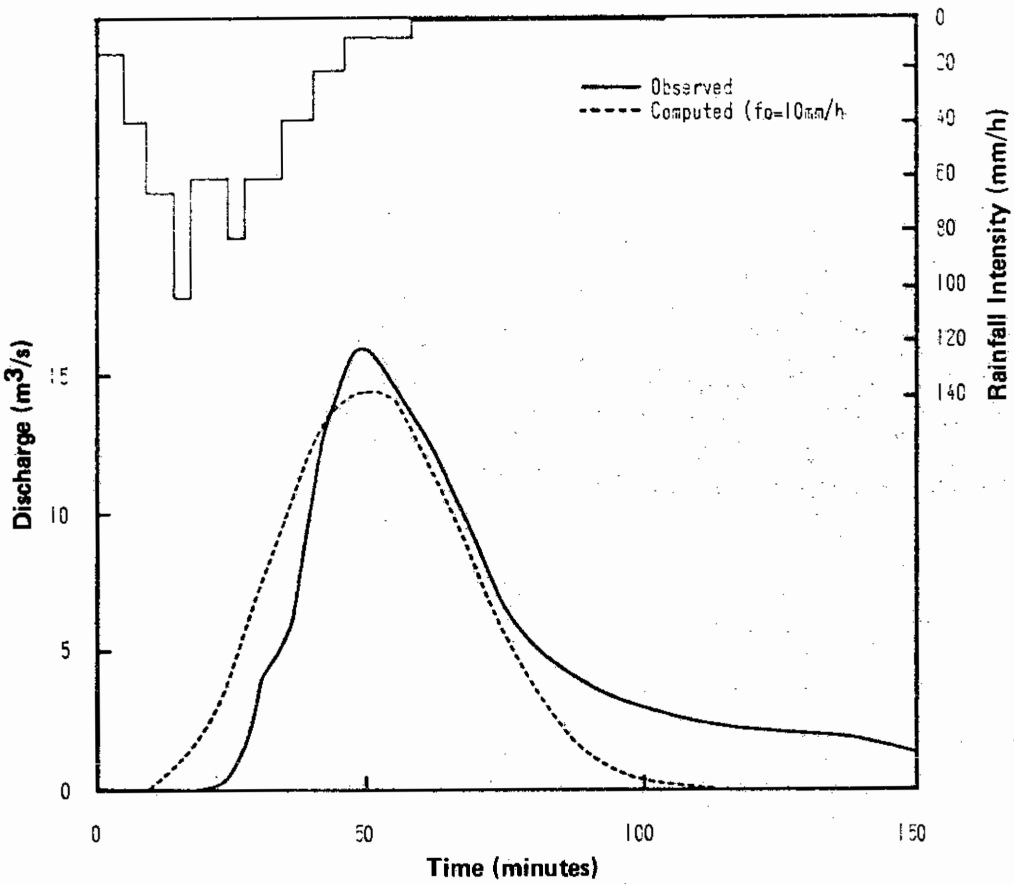


Fig. 5.34 Comparison of computed with observed hydrograph for the storm of 24/4/57 on the Riesel Y catchment

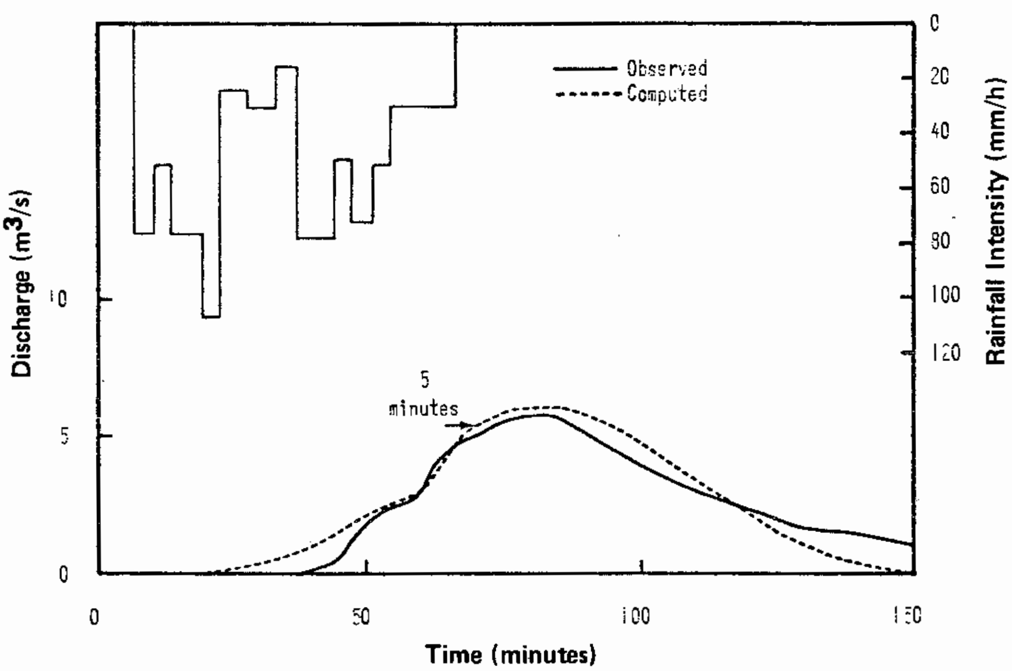


Fig. 5.35 Comparison of computed with observed hydrograph for the storm of 23/6/59 on the Riesel Y catchment

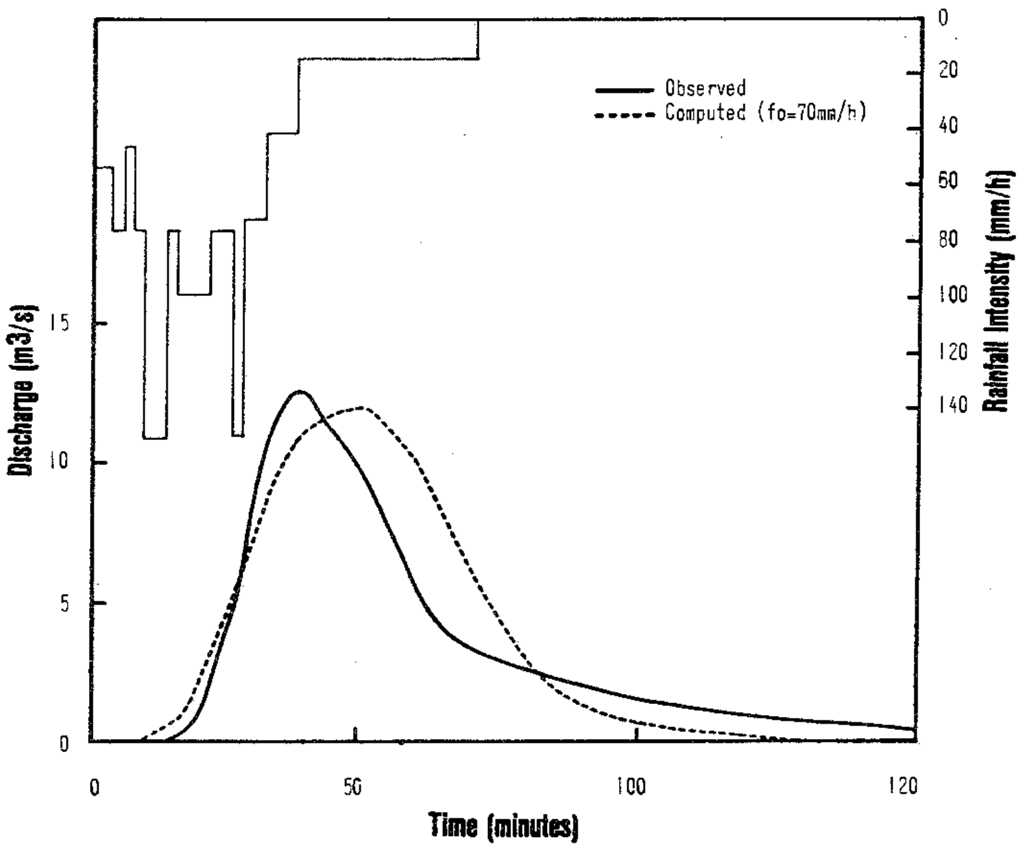


Fig. 5.36 Comparison of computed with observed hydrograph for the storm of 4/6/57 on the Riesel Y catchment

5.8 Discussion of results

The catchments studied range in size from 1,4 ha to 125 ha with average slopes ranging from about 2% to 12%. All four hydrological soil types were present and land use was basically either grassland or crops. Catchment concentration times ranged from 25 minutes to 75 minutes.

The comparisons of computed with observed hydrographs were generally satisfactory. The average ratio of computed to observed peak discharge for the 24 events considered was 1,00 with a standard deviation of 0,16. The results for each catchment are summarized in Table 5.6. The results must, however, be viewed with caution as they are based on calibrations of rainfall loss parameters.

Estimated values of the loss parameters k and d_s of $2h^{-1}$ and 5 mm respectively were found adequate for all catchments except one. The Hastings 2-H catchment exhibited large initial losses and values of d_s and k had to be increased to 6 mm and $6h^{-1}$ respectively. Values of the initial infiltration rate, f_o, had to be calibrated for each storm. The values obtained ranged from 3 mm/h to 190 mm/h.

Table 5.6 Summary of rural catchment verification results

Catchment	Area (ha)	Soil type	Final infiltration rate (mm/h)	Predominant cover	No. of events	λ	s
1. Hastings 2-H	1,4	C	13	Native grass meadow	5	0,93	0,19
2. Stillwater W-1	6,8	D	2	Native grass pasture	4	0,85	0,17
3. Riesel W-2	53	D	1	Row crops	3	1,17	0,04
4. Zululand W1M17	67	A,B, C	3	Ngongoni veld	5	1,04	0,14
5. Stillwater W-4	83	D	3	Native grass pasture	4	1,08	0,09
6. Riesel Y	125	D	1	Row crops	3	0,96	0,06
Overall performance					24	1,00	0,16

Establishment of the final infiltration rate, f_∞, from the latter parts of storms with high AMCs was generally straightforward. Only the Hasting's catchment presented difficulties in the form of interdependence of parameter values. The adopted values of f_∞ are listed in Table 5.6 alongside the catchment soil types and generally do not differ appreciably from the values recommended in section 3.2. The value adopted for Zululand W1M17, however, is unexpectedly low but could be due to parts of the catchment not contributing to runoff.

CHAPTER 6 CONCLUSIONS

The general lack of small catchment runoff data makes the use of process models essential for flood estimation. Models of this type permit land use changes to be analysed and facilitate assessment of errors due to uncertainty in parameter estimation. They also form a sound basis for the transfer of experience from one locality to another.

The time-area method is a simple process model, convenient for desktop application. It has been shown to be capable of reproducing runoff hydrographs for both urban and rural catchments up to 1,5 km^2. Application of the method to larger catchments is mainly limited by the simplifications of its channel routing procedure. As shown in section 2.7 the lag-routing procedure employed over-estimates peak discharge, the error being intensified for wide flood plains and flat channel slopes. In all cases, however, the error will be on the conservative side and in many instances will not be as significant as the uncertainties in other design assumptions. Pitman and Basson (1979), for example, found lag-routing adequate for flood prediction for the 4000 km^2 Hartebeespoort dam catchment.

Though this study has highlighted difficulties in the estimation of loss parameters for pervious areas, this should not deter one from using the technique since this problem is common to all methods. Improvement of parameter estimates is, however, feasible from short term rainfall/runoff measurements.

In summary, the method is applicable under the following conditions:

(1) that the catchment can be divided into a manageable number of zones subject to the same excess rainfall

(2) that partial area contribution to runoff within a zone is negligible

(3) that channel storage can be accounted for by simple lag-routing

(4) that continuous accounting for soil moisture between events is not required

(5) that subsurface storm-flow is an insignificant proportion of total runoff.

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APPENDIX A

RAINFALL DATA

A.1 Urban catchments

SOUTH PARKING-LOT

Rainfall intensity (mm/h) at 1 minute intervals

Storm no. 6

14,3	45,2	62,4	49,7	99,9	59,0	45,2
58,6	44,5	31,1	43,7	30,8	16,5	30,8
44,7	44,7	16,5	16,5	16,5	16,5	45,4
45,4	31,1	59,8	45,4	31,1	45,4	31,1
9,1	9,1	4,8	4,8	4,8	4,8	4,8

Storm no. 7

0	44	57	16	44	44	74
31	43	30	30	14	14	8
8	5	5	5	2	2	2
2	2	2	2	2	2	0
2						

Storm no. 8 (10/8/61)

13,7	10,7	13,7	39,6	53,3	35,1	42,7
50,3	44,2	44,2	36,6	41,1	65,5	30,5
16,8	10,7	16,8	4,6	6,1	4,6	1,5
1,5	1,5	1,5	1,5	1,5	1,5	1,5
4,6	33,5	44,2	27,4	22,9	36,6	24,4
6,1						

Storm no. 9 (9/9/60)

61,0	76,2	61,0	91,4	76,2	61,0	30,5
45,7	45,7	30,5	30,5	61,0	61,0	45,7
45,7	30,5	7,6	7,6	7,6	7,6	15,2
7,6	7,6	7,6	7,6	3,0	1,5	3,0
3,0	1,5	3,0				

Storm no. 13 (6/8/61)

26	38	35	41	85	99	47
55	142	178	166	85	40	81
102	102	76	75	78	67	32
29	9	3				

Storm no. 18

27,4	24,4	22,9	22,9	13,7	7,6	18,3
44,2	77,7	73,2	59,4	30,5	4,6	

NEWARK STREET

Rainfall intensity (mm/h) at 1 minute intervals

Storm no. 15

77	107	107	91	107	46	61
15						

Storm no. 23

31	31	15	15	31	31	46
61	61	91	91	61	61	61
46	46	61	15	46	77	46
61	77	77	77	137	61	91
107	46	31	15	0	15	

OAKDALE AVENUE

Rainfall intensity (mm/h) at 2 minute intervals

19/5/59

7,6	7,6	22,8	45,5	68,4	61,0	22,9
30,5	15,2	30,5	7,6	22,9	15,2	7,6
7,6	0,0	0,0	7,6	7,6	0,0	0,0
7,6						

2/7/60

7,6	0,0	7,6	0,0	7,6	0,0	7,6
7,6	7,6	7,6	0,0	7,6	7,6	0,0
38,1	30,5	15,2	7,6	15,2	30,5	68,6
76,2	76,2	68,6	53,3	45,5	30,5	30,5
22,9	15,2	30,5	22,9	15,2	7,6	15,2
53,3	68,6	7,6	7,6	0,0	7,6	0,0
7,6	0,0	7,6	0,0	7,6	0,0	7,6
0,0						

29/4/63

15,2	38,1	22,9	7,6	0,0	0,0	7,6
7,6	0,0	15,2	22,9	22,9	45,7	38,1
15,2	15,2	7,6	7,6	7,6	7,6	7,6
7,6	0,0	7,6	0,0	7,6	0,0	7,6

2/8/63 (1)

15,2	15,2	15,2	22,8	30,5	30,5	30,5
22,8	0,0	7,6				

2/8/63 (2)

38,0	53,2	15,2	7,6	22,8	30,5	38,0
45,5	22,8	22,8	15,2	7,6	7,6	

GRAY HAVEN

Rainfall intensity (mm/h) at 1 minute intervals

14/6/63

15	15	15	31	46	46	61
61	76	61	31	76	61	61
61	91	76	61	76	76	61
31	46	46	31	31	31	31
46	31	61	61	31	31	31
31	46	46	76	91	61	31
31	15	0	0	15	0	0
0	0	0	15	0	0	0
0	15					

1/8/63

102	117	112	119	125	135	142
86	127	163	168	86	31	38
56	109	117	102	91	58	

14/8/63

27	33	66	81	97	109	48
58	84	66	89	107	114	97
23	10	8	8	28	112	152
109	43	18	5	8	5	5
8	3	8	8	20	25	28
23	8	33	46	38	33	48
56	43	41	36	20	13	8
10	20	18	20	13	15	8
15	13	5	5	...		

PINETOWN

Rainfall intensity (mm/h) at 2 minute intervals

22/5/79 (1)

4,8	2,0	53,0	15,0	7,6	15,0	7,6
7,6	7,4	7,6	7,6	7,6	7,4	1,8
1,8	1,8	1,8	2,6	2,6	2,6	0,4
0,4	0,4	0,4	0,4	0,4	0,4	0,4
0,4	0,4	.....				

22/5/79 (2)

7,4	22,6	7,6	3,8	3,8	2,6	2,6
2,6	22,4	22,6	3,8	3,8	0,0	0,0
0,0	0,0	15,0	3,8	3,8	0,0	0,0
0,0	0,0	0,0	0,0	0,0	0,0	0,0
15,0	3,8	3,8	0,6	0,6	0,6	0,6
0,6	0,6	0,6	0,6	0,6	0,6	0,6
0,6	0,6	0,6	45,0	52,6	22,6	15,0
22,4	15,0	30,0	52,6			

Data for the storms of 29/09/79, 4/11/79 and 18/02/80 are given in Appendix D of HRU Report 1/81 (Watson 1981).

BRUCEWOOD

Rainfall intensity (mm/h) at 2,5 minute intervals

14/5/74

3,05	3,05	3,05	3,05	3,05	3,05	3,05
3,05	0,00	0,00	3,05	3,05	3,05	3,05
0,00	0,00	0,00	0,00	3,05	3,05	3,05
3,05	0,00	0,00	3,05	3,05	3,05	3,05
3,05	3,05	0,00	0,00	0,00	0,00	0,00
0,00	0,00	0,00	3,05	3,05	6,10	6,10
6,10	6,10	15,24	15,24	9,14	9,14	

20/11/74

3,1	3,1	3,1	3,1	9,1	9,1	9,1
9,1	3,1	3,1	0,0	0,0	9,1	9,1
15,2	15,2	12,2	12,2	6,1	6,1	0,0
0,0	3,1	3,1	3,1	3,1	3,1	3,1
3,1	3,1					

11/9/75

3,1	3,1	0,0	0,0	0,0	0,0	3,1
3,1	0,0	0,0	3,1	3,1	0,0	0,0
9,1	9,1	9,1	9,1	24,4	24,4	27,4
27,4	21,3	21,3	21,3	21,3	6,1	6,1
24,4	24,4	18,3	18,3	3,1	3,1	3,1
3,1	3,1	3,1	3,1	3,1	3,1	3,1
9,1	9,1	3,1	3,1	3,1	3,1	3,1
3,1	6,1	6,1	9,1	9,1	3,1	3,1
3,1	3,1	6,1	6,1			

MALVERN

Rainfall intensity (mm/h) at 2 minute intervals

22/9/73

7,6	0,0	7,6	15,2	7,6	7,6	0,0
7,6	7,6	15,2	7,6	7,6	0,0	0,0
0,0	7,6	49,5	49,5	49,5	49,5	0,0
0,0	0,0	7,6	0,0	7,6	0,0	53,3
7,6	0,0	0,0	0,0	0,0	0,0	0,0
0,0	0,0	0,0	0,0	0,0	0,0	0,0
0,0	0,0	0,0	7,6	0,0	0,0	0,0
0,0	7,6	7,6	0,0	7,6	15,2	22,9
22,9	0,0	7,6	0,0	30,5	0,0	7,6
7,6	7,6	7,6	0,0	7,6		

MALVERN - cont

23/9/73

7,6	7,6	7,6	7,6	0,0	0,0	0,0
0,0	0,0	0,0	0,0	0,0	0,0	0,0
0,0	0,0	0,0	0,0	0,0	0,0	0,0
0,0	0,0	0,0	0,0	0,0	7,6	0,0
0,0	0,0	26,7	26,7	0,0	0,0	0,0
0,0	0,0	0,0	0,0	0,0	0,0	0,0
7,6	0,0	0,0	7,6	0,0	7,6	0,0
7,6	0,0	0,0	0,0	0,0	7,6	15,2
15,2	41,9	41,9	15,2	7,6	0,0	7,6

31/5/74

15,2	61,0	38,1	7,6	7,6	15,2	7,6
22,9	7,6	15,2	7,6	45,7	45,7	68,6
22,9	22,9	7,6	0,0	7,6	7,6	

21/6/74

7,6	7,6	0,0	0,0	0,0	0,0	0,0
7,6	30,5	61,0	22,9	0,0	15,2	22,9
15,2	7,6	0,0	0,0	0,0	0,0	0,0
7,6						

4/7/74

38,1	45,7	45,7	15,2	7,6	7,6	7,6
7,6						

19/7/74

15,2	15,2	15,2	38,1	22,9	15,2

KEW

Data for all storms are given in Appendix E of HRU Report 1/81 (Watson, 1981a).

A.2 Rural Catchments

HASTINGS 2-H

Rainfall intensity (mm/h) at 5 minute intervals

26/6/52

17	6	63	88	122	170	40	30
20	15	4	4	4	4	62	28
35	35	8	4	...			

13/7/52

8	8	67	101	96	103	25	25
25	25	25	25	25	25	24	18
12	9	9	9	9	7	4	...

12/6/58

28	28	64	137	89	69	28	11
9	5	5	5	5	5	5	11
14							

3/7/59

117	188	90	59	69	51	84	73
70							

15/5/60

37	49	108	48	120	102	15	45
97	73						

STILLWATER, OKLAHOMA, W-1

Rainfall intensity (mm/h) at 5 minute intervals

18/4/57

15	21	0	0	3	6	9	30
49	21	3	0	1	1	1	1
1	1	0	3	0	0	6	6
3	0	9	3	24	12	3	9
21	34	24	27	40	40	30	49
55	70	107	137	152	64	37	12
6	6	3	3				

27/6/57

43	155	85	21	3

STILLWATER, OKLAHOMA, W-1 - cont.

2/10/59 (1)

2	2	2	2	2	3	3	3
3	3	3	5	5	5	1	37
9	3	3	3	3	9	15	40
55	30	24	24	34	24	12	6
9	12	15	24	52	30	134	52
24	18	6	12	12	6	6	18
24	9	6	3	3	0	0	27
6							

2/10/59 (2)

104	91	70	46	24	40	15	34
15	24	9	3	6	6	9	3
0	3	34	52	3	27	15	12
2	2	3	1	1	1		

RIESEL W-2

Rainfall intensity (mm/h) at 5 minute intervals

24/4/57

24	34	68	107	84	84	80	51
44	33	18	6				

13/5/57

41	43	81	66	67	41	40	15
10	3	4	6	6	5	5	5
5	6	8	8	6	6	6	4
4	4	3					

23/6/59

39	82	63	54	36	41	69	77
69	88	32	37	2			

ZULULAND W1M17

Rainfall intensity (mm/h) at 15 minute intervals

6/2/77

1,8	4,4	8,0	17,0	6,3	6,6	7,6	4,1
6,2	10,9	7,1	20,7	8,1	3,2	0,6	2,0
9,8	9,8	9,8	10,4	24,9	65,7	6,4	6,9
9,2	21,2	20,1	11,0	39,8	16,8	8,0	6,4
1,5	1,5	32,4	23,4	13,2	4,1	21,7	21,0
26,4	6,5	4,0	1,2	4,4	3,6	43,0	1,6
0,4	0,4	0,1					

7/2/77

1,7	18,8	0,3	7,6	1,9	0,7	0,6	2,4
0,3	0,3	0,3	10,5	53,3	32,6	11,4	3,4
4,0	6,2	1,7	2,3	2,9	2,4	7,6	13,0
1,7	0,7	4,0	1,2				

ZULULAND W1M17 - cont.

8/2/77

0,0	0,0	2,7	5,9	8,0	31,3	54,1	30,8
24,5	11,6	0,1	0,1	0,1	0,1	0,1	0,1
0,1	0,8	0,1	0,1	0,1	0,1	0,1	4,0
36,2	29,9	1,1	1,0	0,6	0,1		

9/11/77

18,8	101,9	54,7	14,3	0,4	4,0	0,6	2,8
0,1							

21/1/78

0,8	13,9	6,9	10,7	16,9	8,4	20,4	18,0
31,4	31,9	24,4	17,5	9,4	10,6	27,8	23,8
12,8	14,5	11,1	13,3	15,1	9,4	7,0	4,5
1,7	0,7	1,9	3,6	3,7	2,3	3,8	12,4
2,2	1,2	0,4					

STILLWATER W-4

Rainfall intensity (mm/h) at 10 minute intervals

18/4/57

3	2	3	9	38	4	0	0
0	2	0	9	4	3	15	9
21	24	37	31	56	119	75	12
6	3						

27/6/57

117	23	1

2/10/58 (1)

18	5	5	13	44	44	28	28
9	9	37	55	76	17	7	11
19	23	3	0	17	0	1	0
1	1	0	1	1			

2/10/59 (2)

61	58	27	19	16	11	7	7
7	31	29	29	14	9	3	

RIESEL Y

Rainfall intensity (mm/h) at 5 minute intervals

24/4/57

13	33	64	85	59	72	59	38
24	13	7	1	1	1	1	1
1	1	1	1	1			

4/6/57

58	64	137	94	85	108	53	30
14	14	14	14	14	14		

23/6/59

1	46	61	76	74	26	24	53
73	64	51	29	29	6		

A.3 Antecedent Rainfall

Catchment	Storm date	Antecedent depth of rainfall (mm) in prior			AMC classification number
		5 days	10 days	20 days	
Gray Haven	1/8/63	-	-	-	3
	14/8/63	-	-	-	2
Pinetown	22/5/79	1	1	30	2
	29/9/79	10	-	-	2
	4/11/79	0	0	10	1
	18/2/80	23	-	-	3
Kew	19/2/80	19	67	121	3
	17/3/80	5	5	7	2
	18/3/80	25	25	27	4
	19/3/80	42	42	45	4
	22/3/80	46	50	53	4
	10/4/80	1	7	25	2
Hastings 2-H	12/6/58	0	3	14	1
	3/7/59	∿35	59	-	4
Stillwater W-1	18/4/57	0	4	46	1
	27/6/57	87	168	230	4
	2/10/59 (1)	50	193	195	4
	2/10/59 (2)	124	266	268	4
Riesel W-2	24/4/57	244	245	247	4
	13/5/57	125	138	378	4
	23/6/59	47	47	108	4
Zululand W1M17	6/2/77	74	161	279	4
	7/2/77	202	297	430	4
	8/2/77	245	351	481	4
	9/11/77	9	89	124	2
	21/1/78	92	92	139	4
Stillwater W-4	18/4/57	0	5	44	1
	27/6/57	82	151	209	4
	2/10/59(1)	50	196	197	4
	2/10/59(2)	130	276	277	4
Riesel Y	24/4/57	247	248	250	4
	4/6/57	41	44	50	4
	23/6/59	$\geq$27	$\geq$27	$\geq$85	4

APPENDIX B

HEWLETT-PACKARD HP-97 CALCULATOR PROGRAMS

Three inter-related programs are presented. The first is used to compute excess rainfall - either of a user-provided hyetograph or of a Chicago design storm. The second program determines the isochronal areas from a given set of subcatchment data. The third program then uses the isochronal areas to route the excess rainfall to the catchment outfall.

The variables used are consistent with those described in the text. For convenience of reference a complete list of variables with units is provided in Appendix C.

The programs are described in sections B.1 to B.3 and example applications are presented in section B.4.

B.1 Program I : Excess rainfall

This program computes an excess rainfall hyetograph when provided with loss parameters and either of the following:

(i) average intensities for consecutive intervals on a rainfall hyetograph, or

(ii) parameters for a Chicago design storm.

Instructions for using the program are given in Table B.1. In order to record input data it is initially convenient to have the calculator switched to normal mode. Step 2 describes the basic data input. The Chicago storm parameters in step 2(a) need only be input if either of steps 4, 5 and 6 are to be subsequently used.

Steps 4, 5, 6 and 7 are optional. When performing these steps it is usually convenient to switch the calculator back to manual mode. Step 4 determines the average intensity, I, for a specified duration, t_d. The IDF parameters specified in step 2(a) are used in conjunction with eq. 2.20 to compute this. Step 5 discretizes the Chicago storm using the algorithm described in Fig. 2.7. Step 6 does the same as step 5 but also subtracts losses to determine excess rainfall.

Step 7 computes excess rainfall for a sequence of user-provided average intensities. Step 7(a) initializes parameter values. Step 7(b) provides for the input of discrete rainfall intensities while step 7(c) provides for multiple inputs, i.e. if rainfall is constant over a number of subsequent time steps. Steps 7(b) and (c) should be repeated for all intervals on the rainfall hyetograph.

The program is listed in Table B.2 and the calculator status is described in Table B.3.

Table B.1 User instructions for program I

STEP	INSTRUCTIONS	INPUT	KEYS		OUTPUT DISPLAY	OUTPUT PRINTER
1	Switch to normal mode to print input					
	data					
2	Store data:	Δt	STO	E		
	(a) for Chicago storm	a	STO	A		
		b	STO	B		
		c	STO	C		
		t_d	STO	D		
		r	STO	I		
	(b) for excess rainfall	f_o	STO	0		
		f_∞	STO	1		
		k	STO	2		
		$\%A_s$	STO	3		
3	Perform steps 4,5,6 and 7 as required					
4	Determine average intensity, I, for					
	duration, t	t	B		I	
5	Discretize Chicago storm		C			t_o
						i
						⋮
6	Discretize Chicago storm and determine					
	excess rainfall	P_o	ENT			
		d_s	f	c		t_o
						i
						i_e
						⋮
7	Determine excess rainfall					
	(a) Initialize	P_o	ENT			
		d_s	f	d		
	(b) Intensity for next time					
	increment	i	E			i_e
	OR					
	(c) Constant intensity for next	i_m	ENT			
	m increments	m	f	e		i_{e1}
						⋮
						i_{em}
	Repeat steps (b) or (c) for					
	consecutive intensities, i.					

Table B.2 Listing of program I

LINE	KEY ENTRY	COMMENTS
001	*LBLB	Average intensity
002	RCLB	
003	+	
004	RCLC	
005	CHS	
006	Y^x	
007	RCLA	
008	x	
009	RTN	$I = \frac{a}{(t+b)^c}$
010	*LBLc	Chicago-storm
011	SF2	excess rainfall
012	GSBd	
013	*LBLC	Chigago storm
014	CF0	
015	F2?	
016	SF0	
017	P⇄S	
018	1	
019	STO5	
020	RCLI	
021	STO9	$\rho = r$
022	ST-5	$R_{S5} \rightarrow 1 - r$
023	STO8	$R_{S8} \rightarrow r$
024	RCLD	
025	STx5	$R_{S5} \rightarrow (1-r)t_d$
026	STx8	$R_{S8} \rightarrow rt_d$
027	RCLE	
028	STO6	
029	-	
030	x	
031	RCLE	
032	÷	$r(t_d - \Delta t)/\Delta t$
033	ENT↑	
034	DSP0	
035	RND	
036	-	
037	RCLE	
038	x	t_0
039	ST+5	$R_{S5} \rightarrow (1-r)t_d + t_0$
040	ST-8	$R_{S8} \rightarrow rt_d - t_0$
041	STO7	
042	RND	Print t_0
043	PRTX	
044	SPC	
045	*LBL2	Compute i_b
046	RCLE	
047	RCL8	
048	X≤Y?	
049	GTO3	
050	GSB5	
051	GTO2	
052	*LBL3	Peak intensity
053	-	$t_a = \Delta t - t_b$
054	STO8	
055	RCLE	
056	ST-6	
057	ST-6	$R_6 \rightarrow -\Delta t$
058	GSBB	Peak intensity
059	GSB8	
060	1	
061	RCLI	
062	-	
063	X=0?	
064	GTO6	
065	STO9	$\rho = 1 - r$
066	*LBL4	Compute i_a
067	RCL8	
068	RCL5	
069	X≤Y?	Is $t_{a,max} \leq t_a$?
070	GTO6	
071	GSB5	
072	GTO4	
073	*LBL5	Compute average intensity, i, for next time interval
074	RCL8	
075	RCL9	
076	÷	
077	GSBB	
078	RCL8	
079	x	P_1
080	RCL8	
081	RCL6	
082	-	
083	STO8	$R_{S8} \rightarrow t \pm \Delta t$
084	RCL9	
085	÷	
086	GSBB	
087	RCL8	
088	x	P_2
089	-	
090	RCL6	
091	÷	$i = \frac{P_1 - P_2}{\Delta t}$
092	*LBL8	Print control
093	P⇄S	
094	RND	
095	PRTX	
096	F0?	
097	GSBE	
098	F0?	
099	SPC	
100	*LBL6	
101	P⇄S	
102	0	
103	RTN	
104	*LBLd	Initialize for excess rainfall calculations
105	STO4	
106	R↓	
107	RCL3	
108	%	
109	+	
110	STO5	$F_0 = (1 + \%A_s/100)P_0$
111	RCL0	
112	RCL1	

Table B.2 - cont.

LINE	KEY ENTRY	COMMENTS
113	-	
114	RCL2	
115	÷	
116	STO7	γ_1
117	RCL5	
118	X≠0?	
119	GSB7	Compute F_{do}
120	1	
121	RCLE	
122	6	
123	0	
124	÷	
125	STO9	t (h)
126	RCL2	
127	x	
128	CHS	
129	e^x	
130	-	
131	STO8	γ_2
132	0	
133	RTN	
134	*LBLe	Constant intensity
135	P⇄S	
136	INT	
137	STO0	m
138	X⇄Y	
139	STO1	i_m
140	*LBL0	
141	RCL1	
142	RCL0	
143	1	
144	ST-0	m=m-1
145	R↓	
146	P⇄S	
147	X=0?	If m+1=0, then
148	R/S	stop
149	R↓	
150	GSBE	
151	P⇄S	
152	GTO0	
153	*LBLE	Excess rainfall
154	RCL3	
155	%	
156	+	$i = (1+\%A_s/100)i$
157	RCL9	
158	x	$i \cdot \Delta t$
159	RCL7	
160	RCL5	
161	-	
162	RCL8	
163	x	
164	RCL1	
165	RCL9	
166	x	
167	STO6	$\Delta F_c = \gamma_2(\gamma_1 - F_d) + f_c\Delta t$
168	+	

LINE	KEY ENTRY	COMMENTS
169	ST-6	
170	ST÷6	$R_6 \rightarrow (f_c\Delta t - \Delta F_c)/\Delta F_c$
171	X>Y?	If $\Delta F_c > i \cdot \Delta t$, then
172	R↓	$\Delta F = i . \Delta t$
173	ST×6	$R_6 \rightarrow \Delta F_d$
174	-	
175	RCL4	
176	ST-4	$d_s = 0$
177	-	Pe
178	X<0?	If Pe ∠0, then
179	ST-4	$d_s = -Pe$ and
180	X<0?	$d_e = 0$
181	0	
182	RCL9	
183	÷	
184	RND	
185	PRTX	
186	RCL6	
187	ST-5	
188	0	
189	RTN	
190	*LBL7	Computation of F_{do}
191	1	(Iterative solution
192	RCL2	of eq. 2.13)
193	STO8	
194	RCL9	
195	x	
196	CHS	
197	e^x	
198	ST×8	
199	-	
200	RCL7	
201	ST×8	
202	x	
203	RCL1	
204	ST+8	
205	RCL9	
206	x	
207	+	
208	RCL5	
209	-	
210	RCL8	
211	÷	
212	ST-9	
213	ABS	If the absolute
214	EEX	change in the
215	CHS	estimated value
216	3	of $t \geq 10^{-3}$h,
217	X≤Y?	then improve
218	GTO7	estimate further
219	RCL1	
220	RCL9	
221	x	
222	ST-5	$R_5 \rightarrow F_{do} = F_o - f_\infty t$
223	RTN	

Table B.3 Calculator status for program I

REGISTERS

A	a	0	f_o	(mm/h)	S0	m	
B	b	1	f_∞	(mm/h)	S1	i	
C	c	2	k	(h^{-1})	S2		
D	t_d (minutes)	3	$\%A_s$	(%)	S3		
E	Δt (minutes)	4	d_s	(mm)	S4		
		5	F_d	(mm)	S5	$(1-r)\ t_d + t_o$	
I	r	6	$-\Delta F_c$	(mm)	S6	$\pm\ \Delta t$	(minutes)
		7	γ_1		S7	t_o	(minutes)
		8	γ_2		S8	t	(minutes)
		9	Δt	(h)	S9	ρ	

LABELS

A		0	m = m-1
B	Average intensity, I	1	
C	Chicago storm	2	Intensity before peak
D		3	Peak intensity
E	$i \to i_e$	4	Intensity after peak
a		5	Average intensity for time increment
b		6	$P \gtrless S$
c	Chicago – storm excess-rainfall	7	Initial value of F_d
d	Initialize excess rainfall	8	Print control
e	Constant intensity for m increments	9	

FLAGS

0	Chicago-storm excess-rainfall
1	
2	
3	

SET STATUS

FLAGS			TRIG		DISP	
	ON	OFF				
0	☐	■	DEG	■	FIX	■
1	☐	■	GRAD	☐	SCI	☐
2	☐	■	RAD	☐	Eng	☐
3	☐	■			n	0

B.2 Program II : Isochronal areas

Given the time step and the area, entry time and flow time for each subcatchment this program will compute the isochronal areas. The technique described in section 2.4 is used and up to 9 isochronal areas can be accommodated. If more areas are generated from the data the calculator will display an error message (program line 42). If this occurs then a larger time increment must be selected.

The catchment data are automatically printed out. Should any input errors be detected the program provides for subsequent corrections (steps 5(a) and (b) in Table B.4). Isochronal areas are automatically stored in the registers required for program III. A printout of these areas can be obtained by using step 6.

Should the isochronal areas be predetermined, step 3 describes how they should be input for subsequent use in program III.

The program is listed in Table B.5 and the calculator status is described in Table B.6.

Table B.4 User instructions for program II

STEP	INSTRUCTIONS	INPUT	KEYS		OUTPUT DISPLAY	OUTPUT PRINTER
1	Initialize	Δt	f	a	1,000	Δt
2	If isochronal areas are predetermined					
	proceed to step 3, otherwise proceed					
	to step 4.					
3	Number of isochronal areas	M	STO	B		
	Isochronal areas	ΔA_1	STO	1		
		ΔA_2	STO	2		
		⋮				
		ΔA_M	STO	M		
	(Note: $M \ngtr 9$)					
	Go to step 6					
4	Input subcatchment data	N	ENT			
		A	ENT			
		t_e	ENT			
		t_f	A		N+1	N
						A
						t_e
						t_f
	Repeat step 4 for all subcatchments					
5	Correct input errors:					
	(a) Input incorrect data with a					
	negative subcatchment number	-N	ENT			
		A	ENT			
		t_e	ENT			
		t_f	A		N	-N
						A
						t_e
						t_f
	(b) Input correct data	A	ENT			
		t_e	ENT			
		t_f	A		N+1	N
						A
						t_e
						t_f
6	Print isochronal areas (optional)		f	b	$\Sigma\Delta A$	ΔA_1
						ΔA_2
						⋮
						ΔA_M
						$\Sigma\Delta A$

Table B.8 Listing of program II

KEY ENTRY	COMMENTS
*LBLa	Initialize
CLRG	
PRTX	
STOA	
1	
RTN	
*LBLA	Subcatchment data
R↑	
STO0	
CF0	
X<0?	Data correction ?
SF0	
R↓	
PRST	Print input data
RCLA	
÷	
STOE	τ_f
X⇄Y	
RCLA	
÷	
STOD	τ_e
+	
.	
9	
9	
+	
INT	
STOI	Subcatchment M
X⇄Y	
STOC	A
RCLD	
÷	
STOD	A/τ_e
RCLI	
RCLB	
X≤Y?	
X⇄Y	
STOB	Catchment M
9	
RCLI	
-	
√X	Is M>9 ?
RCLC	
*LBL4	
X=0?	Is $A_\tau = 0$?
GTO5	
RCLD	
RCLI	
1	
-	
RCLE	
-	$(A/\tau_e)(\tau-1-\tau_f)$
×	
X<0?	
CLX	
-	ΔA_τ
F0?	
CHS	
ST+i	
LSTX	A_τ
DSZI	
GTO4	
*LBL5	Increment subcatchment number unless data is being corrected
RCL0	
ABS	
F0?	
RTN	
1	
+	
RTN	
*LBLb	Print isochronal areas
0	
STOI	
*LBL6	
ISZI	
RCLB	
RCLI	
-	
X<0?	
GTO7	
R↓	
RCLi	
PRTX	
+	
GTO6	
*LBL7	Print $\Sigma\Delta A$
SPC	
R↓	
PRTX	
SPC	
RTN	

Table B.6 Calculator status for program II

REGISTERS

A	Δt (minutes)		0	N		S0	
B	M		1	ΔA_1	(ha)	S1	
C	A	(ha)	2	ΔA_2	(ha)	S2	
D	A/τ_e	(ha)	3	ΔA_3	(ha)	S3	
E	τ_f		4	ΔA_4	(ha)	S4	
			5	ΔA_5	(ha)	S5	
I	τ		6	ΔA_6	(ha)	S6	
			7	ΔA_7	(ha)	S7	
			8	ΔA_8	(ha)	S8	
			9	ΔA_9	(ha)	S9	

LABELS

A	Input subcatchment data	0	
B		1	
C		2	
D		3	
E		4	Store isochronal areas
a	Initialize	5	N = N+1 or N = ABS(N)
b	Print isochronal areas	6	Print isochronal areas
c		7	Print $\Sigma\Delta A$
d		8	
e		9	

FLAGS

0	Correct data
1	
2	Set flag zero
3	

SET STATUS

FLAGS ON	FLAGS OFF	TRIG		DISP	
0 ☐	■	DEG	■	FIX	■
1 ☐	■	GRAD	☐	SCI	☐
2 ☐	■	RAD	☐	ENG	☐
3 ☐	■			n	2

B.3 Program III : Time-area routing

Before using this program isochronal areas must be stored in the relevant registers by using program II. Excess rainfall intensities are input for each consecutive time interval and the program routes the flow to the outfall using the algorithm described in section 2.5. Multiple inputs are accommodated where rainfall is constant over a number of consecutive time intervals. After excess rainfall has ceased, zero rainfall should be input until discharge becomes negligible.

User instructions are given in Table B.7 and the program is listed in Table B.8. Table B.9 describes the calculator status for the program. The program is fairly short and is conveniently recorded on the same card as program II.

Table B.7 User instructions for program III

STEP	INSTRUCTIONS	INPUT	KEYS		OUTPUT DISPLAY	OUTPUT PRINTER
1	Initialize	t_{start}	f	d	t_{start}	0
2	Input excess rainfall hyetograph					
	to determine outflow hydrograph:					
	(a) Intensity for next time interval	i_e	E			t
						i_e
						Q
	(b) Constant intensity for m increments	i_{em}	ENT			t
		m	f	e		i_e
						Q
	Repeat (a) or (b) until discharge Q					
	is insignificant					

Table B. 8 Listing of program III

LINE	KEY ENTRY	COMMENTS
092	*LBLd	Initialize
093	RCLA	
094	RCLB	
095	P⇄S	
096	CLRG	
097	P⇄S	
098	STOB	
099	R↓	
100	STOA	
101	R↓	
102	STO0	
103	SPC	
104	0	
105	PRTX	
106	R↓	
107	RTN	
108	*LBLe	Multiple input
109	STOD	m
110	X⇄Y	
111	STOE	i_{em}
112	*LBL3	
113	RCLD	
114	X=0?	
115	R/S	
116	1	
117	-	
118	STOD	
119	RCLE	
120	GSBE	
121	GTO3	
122	*LBLE	Compute discharge
123	STOE	i_e
124	RCLA	
125	ST+0	$t = t + \Delta t$
126	CLX	
127	RCL0	t
128	SPC	
129	PRTX	Print t
130	X⇄Y	
131	PRTX	Print i_e
132	X=0?	
133	GTO2	
134	RCLB	
135	STOI	
136	*LBL1	
137	RCLi	
138	RCLE	
139	x	$i_{et} \cdot \Delta A_{T+1}$
140	P⇄S	
141	ST+i	R_{T-1},t
142	P⇄S	
143	DSZI	$j = j - 1$
144	GTO1	
145	*LBL2	
146	P⇄S	
147	RCL1	

LINE	KEY ENTRY	COMMENTS
148	3	
149	6	
150	0	
151	÷	Q
152	RND	
153	PRTX	Print Q
154	RCL2	
155	STO1	
156	RCL3	
157	STO2	
158	RCL4	
159	STO3	
160	RCL5	
161	STO4	
162	RCL6	
163	STO5	Route flow
164	RCL7	$R_{T-1},t \rightarrow$
165	STO6	R_T,t
166	RCL8	
167	STO7	
168	RCL9	
169	STO8	
170	0	
171	STO9	
172	P⇄S	
173	RTN	

Table B.9 Calculator status for program III

REGISTERS

A	Δt	0	t	(minutes)	S0		
B	M	1	ΔA_1	(ha)	S1	R_1	(ha.mm/h)
C		2	ΔA_2	(ha)	S2	R_2	(ha.mm/h)
D	m	3	ΔA_3	(ha)	S3	R_3	(ha.mm/h)
E	i_{et}	4	ΔA_4	(ha)	S4	R_4	(ha.mm/h)
		5	ΔA_5	(ha)	S5	R_5	(ha.mm/h)
I	τ	6	ΔA_6	(ha)	S6	R_6	(ha.mm/h)
		7	ΔA_7	(ha)	S7	R_7	(ha.mm/h)
		8	ΔA_8	(ha)	S8	R_8	(ha.mm/h)
		9	ΔA_9	(ha)	S9	R_9	(ha.mm/h)

LABELS

A		0	
B		1	$R_{\tau,t}$
C		2	Print Q and route flow
D		3	
E	Compute discharge	4	
a		5	
b		6	
c		7	
d	Initialize	8	
e	Multiple input	9	

FLAGS

0	
1	
2	
3	

SET STATUS

FLAGS			TRIG		DISP	
	ON	OFF				
0	□	■	DEG	■	FIX	■
1	□	■	GRAD	□	SCI	□
2	□	■	RAD	□	ENG	□
3	□	■			n	2

B.4 Example applications

Three examples are provided to assist the user in familiarizing himself with the various aspects of the programs.

B.4.1 Example 1

Fig. B.1 is a typical printout obtained in estimating an observed hydrograph from a recorded storm. This printout was obtained in the rural catchment verification study for the Stillwater W-4 catchment. It is one of the shorter printouts obtained yet illustrates all the pertinent features. The information is printed in distinctive formats which are annotated in the figure to aid identification.

B.4.2 Example 2

Fig. B.2 shows the results of a laboratory runoff plot experiment (Izzard, 1946). The plot was rectangular and of length 7,3m with a 1,0% slope. The surface was crushed slate. Simulated rainfall intensities of 9,3 mm/h and 47 mm/h were applied for 5 and 7 minutes respectively, as shown in the figure.

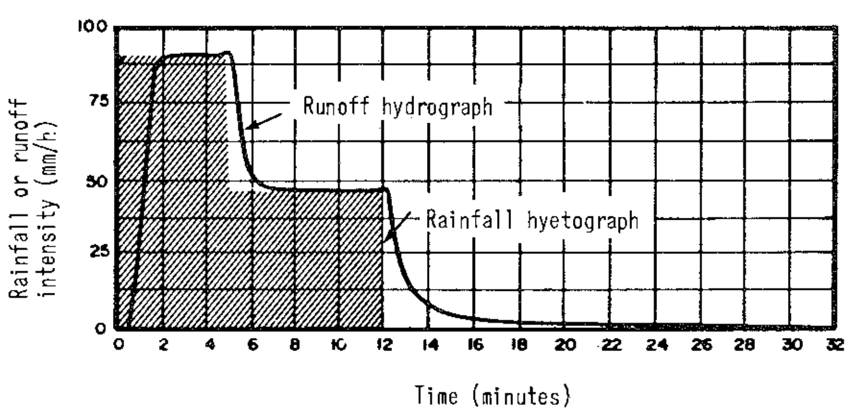

Fig. B.2 Runoff plot experiment (Izzard, 1946)

Assume an entry time of 2 minutes and compute the runoff hydrograph.

Determination of excess rainfall (program I) :-

10.	STOE	Δt
7.	STO0	f_o
6.	STO1	f_∞
2.	STO2	k
0.	STO3	% A_s
0.	ENT↑	P_o
5.	GSBd	d_s
117.	GSBE	i_1
60.	***	i_{e1}
16.	GSBE	i_2
9.	***	i_{e2}
1.	GSBE	i_3
0.	***	i_{e3}

Computation of isochronal areas (program IIa) :-

10.00	***	Δt
1.00	T	Subcatchment No.
14.20	Z	Area
58.00	Y	t_e
12.00	X	t_f
2.00	T	
7.30	Z	
38.00	Y	
12.00	X	
3.00	T	
7.50	Z	
44.00	Y	
10.00	X	
4.00	T	
4.50	Z	
35.00	Y	
8.00	X	
5.00	T	
10.20	Z	
37.00	Y	
8.00	X	
6.00	T	
5.60	Z	
45.00	Y	
6.00	X	
7.00	T	
4.60	Z	
42.00	Y	
5.00	X	
8.00	T	
11.40	Z	
35.00	Y	
5.00	X	
9.00	T	
6.30	Z	
61.00	Y	
2.00	X	
10.00	T	
7.60	Z	
52.00	Y	
2.00	X	

Isochronal areas:

5.44	***	ΔA_1
18.26	***	ΔA_2
15.13	***	ΔA_3
13.13	***	ΔA_4
13.27	***	ΔA_5
5.40	***	ΔA_6
2.76	***	ΔA_7
83.40	***	$\Sigma\Delta A$

Routing excess rainfall to determine hydrograph (program IIb) :-

0.00		⇒ Program initialized
10.00	***	t
80.00	***	i_e
1.21	***	Q
20.00	***	
16.00	***	
4.30	***	
30.00	***	
0.00	***	
5.06	***	
40.00	***	
0.00	***	
5.10	***	
50.00	***	
0.00	***	
3.60	***	
60.00	***	
0.00	***	
1.75	***	
70.00	***	
0.00	***	
0.85	***	
80.00	***	
0.00	***	
0.12	***	
90.00	***	
0.00	***	
0.00	***	

Fig. B.1 Annotated printout for the storm of 27/6/57 on the Stillwater W-4 catchment

Solution: This example requires the use of programs II and III only. The output from these programs with the calculator in normal print mode is shown in Fig. B.3 while observed and computed hydrographs are compared in Fig. B.4.

The catchment area was assumed to be 360 ha so that discharge in m^3/s would be the same as discharge per unit width in mm/h.

```
  1.00 GSBa
  1.00  ***
360.00 ENT↑
  2.00 ENT↑
  0.00 GSBA

  1.00   T
360.00   Z
  2.00   Y
  0.00   X

       GSBb
180.00  ***
180.00  ***

360.00  ***

  0.00 GSBd

  0.00
       DSP0
   93. ENT↑
    5. GSBe

    1.  ***
   93.  ***
   47.  ***

    2.  ***
   93.  ***
   93.  ***

    3.  ***
   93.  ***
   93.  ***

    4.  ***
   93.  ***
   93.  ***

    5.  ***
   93.  ***
   93.  ***
   47. ENT↑
    7. GSBe

    6.  ***
   47.  ***
   70.  ***

    7.  ***
   47.  ***
   47.  ***

    8.  ***
   47.  ***
   47.  ***

    9.  ***
   47.  ***
   47.  ***

   10.  ***
   47.  ***
   47.  ***

   11.  ***
   47.  ***
   47.  ***

   12.  ***
   47.  ***
   47.  ***
    2. GSBe

   13.  ***
    0.  ***
   24.  ***

   14.  ***
    0.  ***
    0.  ***
```

Fig. B.3 Program output for example 2

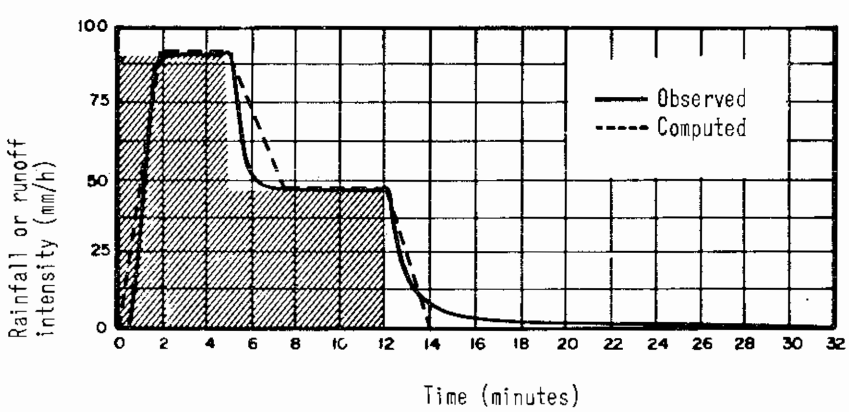

Fig. B.4 Comparison of computed with observed hydrograph from runoff plot experiment

B.4.3 Example 3

Compute a 20-year return period flood hydrograph for the Kew catchment. Use a Chicago design storm of 90-minute duration with a time step of 5 minutes. Assume the mean annual rainfall to be 720 mm and make use of the regional Chicago storm parameters given in Table 3.5 and Fig. 3.5.

Ignore surcharging of pipes and assume surplus runoff to travel overland to the outfall at a velocity comparable with that in the pipes. Use the following 5-minute isochronal areas which were obtained in the verification study (Chapter 4) :

(i) Paved zones: 6,85; 14,05; 7,20 (ha)

(ii) Grassed zone: 6,42; 12,54: 12,54; 12,54; 12,54; 12,54; 12,54; 12,54; 6,12 (ha)

Solution

MAP = 720 mm

From Fig. 3.5 and from Table 3.5

$$a = 3000$$
$$b = 14{,}4$$
$$c = 0{,}883$$
$$r = 0{,}40$$

The rainfall loss parameters assuming AMC = 3 are:

$$\%A_s = 15\%$$
$$f_o = 66 \text{ mm/h}$$
$$f_c = 13 \text{ mm/h}$$
$$k = 2 \text{ h}^{-1}$$
$$d_{sg} = 5 \text{ mm}$$
$$d_{sp} = 1 \text{ mm}$$

The discretization of the Chicago storm at 5 minute intervals and the determination of grassed area excess rainfall is illustrated in Fig. B.5. Steps 1,2 and 6 of program I are used and the calculator has been left in normal mode to illustrate the user's interaction with the program.

```
   5. STOE
3000. STOA
 14.4 STOB
 .883 STOC
  90. STOD
   .4 STOI
  66. STO0
  13. STO1
   2. STO2
  15. STO3
   0. ENT↑
   5. GSBc
  -1.  ***
```

```
 12.  ***
  0.  ***

 15.  ***
  0.  ***

 13.  ***
  0.  ***

 25.  ***
  0.  ***

 35.  ***
  0.  ***

 55.  ***
  0.  ***
```

```
110.  ***
 40.  ***

219.  ***
214.  ***

124.  ***
109.  ***

 74.  ***
 54.  ***

 50.  ***
 30.  ***

 37.  ***
 17.  ***
```

```
 29.  ***
 10.  ***

 23.  ***
  4.  ***

 20.  ***
  2.  ***

 17.  ***
  0.  ***

 15.  ***
  0.  ***

 13.  ***
  0.  ***
```

Fig. B.5 Chicago storm discretization and determination of grassed-area excess-rainfall for example 3

Two figures are printed out for each 5-minute time step. The first represents the Chicago storm rainfall intensity and the second the grassed-area excess-rainfall intensity. The first figures can be used to obtain the excess-rainfall input to the paved zone. All that needs to be done is to subtract the 1 mm (i.e. 12 mm/h for the 5-minute time step) depression storage.

Programs II and III are now used to determine the paved zone hydrograph. The isochronal areas are stored in the relevant registers as described in the user instructions for program II and illustrated in Fig. B.6(a). Program III is then used to route the excess rainfall to the catchment outfall. The steps are illustrated in Fig. B.6(b) with the calculator in normal mode.

```
 5.00 GSBa
 5.00  ***
 3.00 ST06
 6.85 ST01
14.65 ST02
 7.20 ST03
```

(a) Storing isochronal areas in registers 1, 2 and 3

```
 5.00 GSBd

 0.00
15.00 GSBE

10.00  ***
15.00  ***
 0.29  ***
19.00 GSBE

15.00  ***
19.00  ***
 0.95  ***
25.00 GSBE

20.00  ***
25.00  ***
 1.52  ***
35.00 GSBE

25.00  ***
35.00  ***
 2.02  ***
55.00 GSBE

30.00  ***
55.00  ***
 2.91  ***
```

```
110.00 GSBE

 35.00  ***
110.00  ***
  4.94  ***
219.00 GSBE

 40.00  ***
219.00  ***
  9.56  ***
124.00 GSBE

 45.00  ***
124.00  ***
 13.11  ***
 74.00 GSBE

 50.00  ***
 74.00  ***
 10.63  ***
 50.00 GSBE

 55.00  ***
 50.00  ***
  6.32  ***
```

```
37.00 GSBE

60.00  ***
37.00  ***
 4.14  ***
29.00 GSBE

65.00  ***
29.00  ***
 3.00  ***
23.00 GSBE

70.00  ***
23.00  ***
 2.31  ***
20.00 GSBE

75.00  ***
20.00  ***
 1.66  ***
17.00 GSBE

80.00  ***
17.00  ***
 1.56  ***
```

```
 15.00 GSBE

 85.00  ***
 15.00  ***
  1.35  ***
 13.00 GSBE

 90.00  ***
 13.00  ***
  1.17  ***
  3.00 GSBe

 95.00  ***
  0.00  ***
  0.81  ***

100.00  ***
  0.00  ***
  0.26  ***

105.00  ***
  0.00  ***
  0.00  ***
```

(b) Isochronal routing

Fig. B.6 Paved area routing computations for example 3

The next step is to compute the hydrograph for the grassed zone. This is done in the same manner as for the paved zone and the computations are illustrated in Fig. B.7. This time the calculator has been used in manual print mode, since this is the most convenient for general operation. As the isochronal areas are no longer "echoed" on input, step 6 of program II has been used to obtain a printout for checking.

```
  5.00 ***      35.00 ***      65.00 ***      95.00 ***
  6.42 ***      40.00 ***      10.00 ***       0.00 ***
 12.54 ***       6.71 ***      16.34 ***       1.66 ***
 12.54 ***
 12.54 ***      40.00 ***      70.00 ***     100.00 ***
 12.54 ***     214.00 ***       4.00 ***       0.00 ***
 12.54 ***       5.21 ***      16.58 ***       0.85 ***
 12.54 ***
 12.54 ***      45.00 ***      75.00 ***     105.00 ***
  6.12 ***     109.00 ***       2.00 ***       0.00 ***
                16.79 ***      15.97 ***       0.38 ***
100.32 ***
                50.00 ***      80.00 ***     110.00 ***
                54.00 ***       0.00 ***       0.00 ***
  0.00          13.61 ***      11.51 ***       0.14 ***

                55.00 ***      85.00 ***     115.00 ***
                30.00 ***       0.00 ***       0.00 ***
                15.06 ***       5.93 ***       0.03 ***

                60.00 ***      90.00 ***     120.00 ***
                17.00 ***       0.00 ***       0.00 ***
                15.67 ***       3.11 ***       0.00 ***
```

Fig. B.7 Grassed area routing computations for example 3

The outfall hydrograph is now determined by combining the paved and grassed area components. This is done graphically in Fig. B.8.

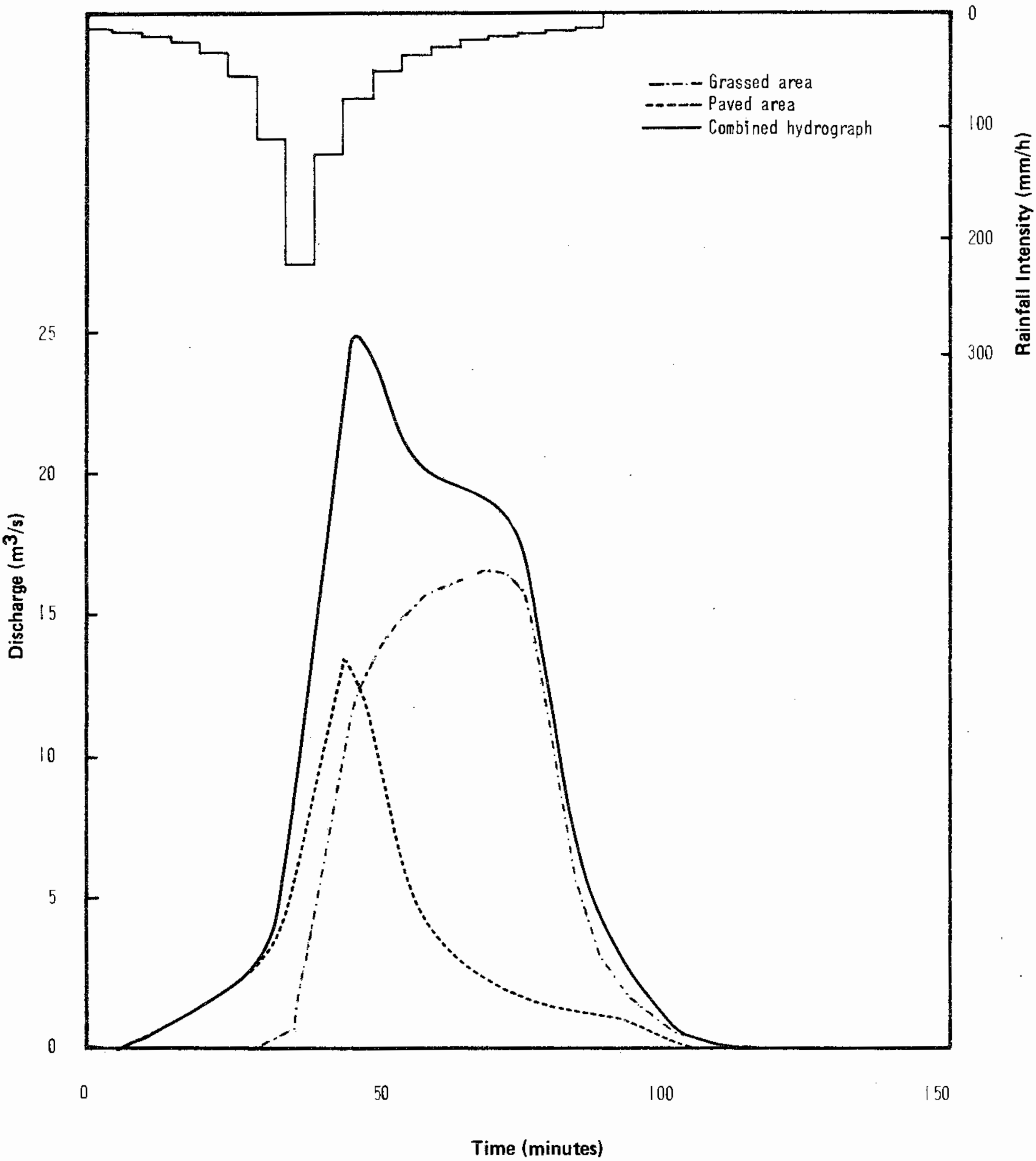

Fig. B.8 Computed hydrograph for example 3

APPENDIX C

LIST OF VARIABLES

Variable name	Description	Units
A	Subcatchment area	ha
A_s	Supplementary impervious area	ha
A_τ	Area contributing flow to outfall within τ time increments	ha
a	IDF coefficient in eq. 2.19	-
B	Width of flow at surface	-
b	Bottom width of trapezoidal channel	-
b	IDF coefficient in eq. 2.19	-
c	IDF coefficient in eq. 2.19	-
d_s	Average depth of depression storage	mm
d_{sp}	Paved area d_s	mm
d_{sg}	Grassed area d_s	mm
F	Cumulated depth of infiltration, $f.\Delta t$	mm
F_o	Initial value of F	mm
F_c	Cumulated $f_o.\Delta t$	mm
F_{cap}	Integral of eq. 2.1	mm
F_d	Cumulated $f_d.\Delta t$	mm
F_{do}	Initial value of F_d	mm
F_{dcap}	Integral of eq. 2.2	mm
f	Infiltration rate	mm/h
f_o	Infiltration capacity at $t = 0$	mm/h
f_∞	Infiltration capacity at $t = \infty$	mm/h
f_{ccap}	Constant infiltration capacity ($=f_\infty$)	mm/h
f_{cap}	Infiltration capacity	mm/h
f_{dcap}	Diminishing infiltration capacity	mm/h
f_d	Diminishing infiltration rate	mm/h
I	Average rainfall intensity	mm/h
$I_{T,t}$	I for return period, T, and duration, t	mm/h
i	Rainfall intensity	mm/h
i_a	Rainfall intensity after peak	mm/h
i_b	Rainfall intensity before peak	mm/h
i_e	Excess rainfall intensity	mm/h
i_{em}	i_e for m time intervals	mm/h
i_m	i for m time intervals	mm/h
i_p	Effective rainfall intensity on pervious areas	mm/h
k	Recession constant in Horton's equation (eq. 2.1)	h^{-1}

LIST OF VARIABLES - cont.

Variable name	Description	Units
L	Flow path length	m
M	Number of isochronal areas	-
m	Number of time intervals of intensity i_m or i_{em}	-
N	Subcatchment number	-
n	Manning roughness coefficient	-
P	Depth of rainfall (precipitation)	mm
P_e	Depth of excess rainfall	mm
P_o	Depth of rainfall prior to major portion of storm	mm
P_1, P_2	Average depths of rainfall for successive durations	mm
Q	Discharge	m^3/s
$R_{\tau,t}$	Runoff onto isochronal area ΔA_τ at time t	ha.mm/h
$R_{M,t}$	Runoff onto the furthermost isochronal area at time t	ha.mm/h
r	Ratio of time-to-peak to duration	-
s	Slope	%
T	Return period	years
t	Time	minutes
t_a	Time after peak for Chicago storm	minutes
$t_{a,max}$	Maximum time after peak for Chicago storm	minutes
t_b	Time before peak for Chicago storm	minutes
t_d	Duration of rainfall	minutes
t_e	Entry time	minutes
t_f	Flow time	minutes
t_p	Time to peak rainfall intensity	minutes
t_{start}	Starting time for routing computations	minutes
t_o	Starting time for discretized Chicago storm	minutes
V	Uniform flow velocity	m/s
V_w	Wave velocity	m/s
W	Ratio of catchment width to flow width	-
y	Flow depth	m
z	Side slopes (horizontal to vertical)	-

LIST OF VARIABLES - cont.

Variable name	Description	Units
α	$\Delta F/\Delta F_{cap}$	-
γ_1	$(f_o - f_\infty)/k$	mm
γ_2	$1 - e^{-kt}$	-
γ_I	Regional rainfall intensity coefficients	-
γ_R	given in Table 3.5	-
ΔA_τ	Isochronal area contributing flow to the outfall in τ time intervals	ha
ΔF	Increment in F	mm
ΔF_c	Increment in F_c	mm
ΔF_{cap}	Increment in F_{cap}	mm
ΔF_d	Increment in F_d	
Δt	Computational time interval	minutes
ρ	r or (1-r)	-
$\Sigma\Delta A$	Sum of isochronal areas	ha
τ	Dimensionless time $(t/\Delta t)$	-
τ_e	Dimensionless entry time $(t_e/\Delta t)$	-
τ_f	Dimensionless flow time $(t_f/\Delta t)$	-
$\%A_s$	Supplementary impervious area, A_s, expressed as a percentage of the pervious area	%

APPENDIX D

HEWLETT PACKARD HP-41C(V) CALCULATOR PROGRAM

by T. op ten Noort

D.1 Program description

D.1.1 General

The three HP-97 programs have been rewritten for the HP-41CV (or the HP-41C with a quad memory-module). The programs have been combined to form a suite of data manipulation subroutines and a "go step" which performs the excess rainfall and routing computations. The enhanced capacity of the HP-41C is effectively used to reduce data input and to render computations more flexible. The available subroutines and their inter-relationship is illustrated in Fig. D.1.

User instructions are presented in Table D.1 and elaborated upon in sections D.1.2 to D.1.8. The program is listed in Table D.2 and the calculator status is given in Table D.3 Variables and units are consistent with those described in Appendix C. Use of both a card reader and a printer is recommended.

D.1.2 Initialization and time step

Subroutine a zeroes the catchment hydrograph and sets the default discharge printing control to show no figures after the decimal point. The user is prompted to input the computational time step.

Subroutine A allows the user to change the computational time step without zeroing the catchment hydrograph or modifying the default print control. When a Chicago design storm has been specified a flag is set to rediscretize the hyetograph at the new time step.

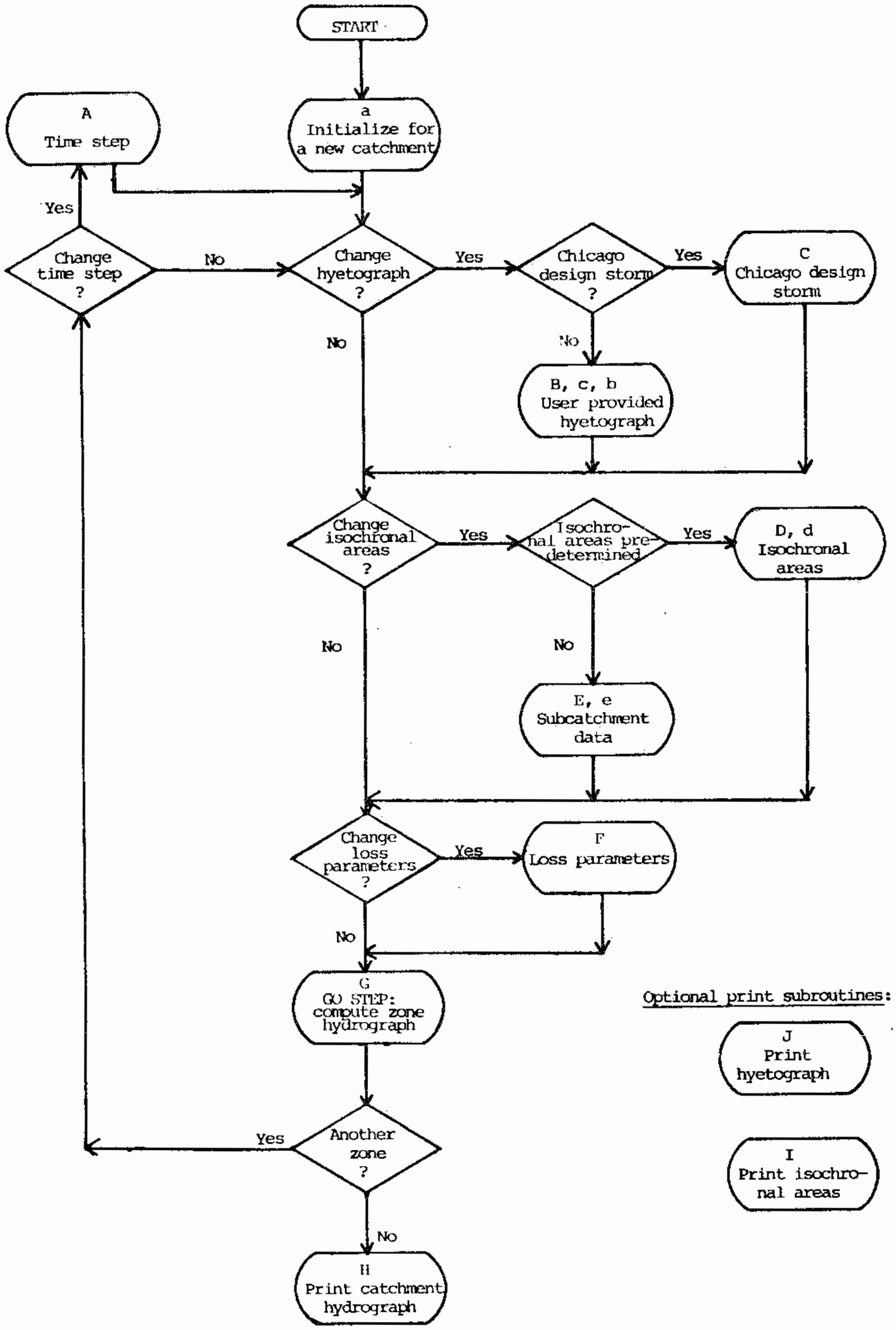

Fig. D.1 Inter-relationship of HP-41C program subroutines

D.1.3 Rainfall

Subroutines B,b and C allow the user to define a rainfall hyetograph while subroutine C allows the user to specify a Chicago design storm. Subroutines B and c allow for input of rainfall intensities at the computational time step; B for discrete values and c for multiple values, i.e. when rainfall intensity is constant over a number of time steps. Subroutine b allows for incorrect values to be overwritten. Up to ninety consecutive three digit integer rainfall intensities can be provided. These can be recorded on a magnetic card for future use. Subroutine J provides for the printing of data read from a magnetic card.

Subroutine C prompts the user for new values of the Chicago-storm parameters. If no value is provided for a particular parameter then the previously defined value is used. If storm duration divided by the computational time step is greater than 89 the program will display "DATA ERROR". If this occurs one must provide a larger time step or a shorter duration.

D.1.4 Isochronal areas

Subroutine E computes isochronal areas given area, entry time and flow time for each subcatchment. The technique used is described in section 2.4 and up to 15 isochronal areas can be accommodated. If more areas are generated from the data the calculator will display "DATA ERROR". If this occurs then a larger time step must be selected.

The catchment data are automatically printed. Should any input errors be detected subroutine e can be used to make corrections. Isochronal areas are printed when data input is complete or when requested using subroutine I.

Should isochronal areas be predetermined then subroutine D is used for input and subroutine d for correction of input errors.

D.1.5 Loss parameters

Subroutine F provides for the input of rainfall loss parameters. The user is prompted for new values of each of the parameters d_s, f_o, f_c, k and $\%A_s$. If no value is provided by the user for any particular parameter then the previously defined value is used.

D.1.6 "Go step"

On entering this subroutine the user is prompted to reset the display control for printing discharge. Merely pressing R/S will keep the control at its present setting. The number of digits to be displayed after the decimal point should be keyed in to reset the control.

If a Chicago design storm has been specified (and has not been discretized in a previous run) it is discretized and stored for subsequent use.

For each time increment the program recalls the rainfall intensity and computes the corresponding excess rainfall. After the first non-zero value is obtained excess rainfall is routed over the catchment and the outfall discharge at the end of the time increment is determined. This discharge is printed and added to the catchment hydrograph. Computation terminates when the first zero discharge is obtained after the termination of rainfall.

D.1.7 Catchment hydrograph

The summated catchment hydrograph is stored with <u>two digits accuracy</u> (in logarithmic form) and can be printed out using subroutine H.

D.1.8 Program interruption

Should the program be interrupted it is essential that the user ensures flags 1, 2 and 4 are cleared before transfering control to another subroutine. Flag 3 should also be checked when it is not used to indicate Chicago-storm discretization in the "go step".

Table D.1 HP-41C program user instructions

			SIZE: 123		OUTPUT	
STEP	INSTRUCTIONS	INPUT DATA/UNITS	KEYS		DISPLAY	PRINTER
1	INITIALIZE (Enter the program, check status,					
	and set USER mode)					
1.1	(a) New catchment		a		DT?	TIME-AREA
						HYDROGRAPH
	or					
	(b) New time step only		A		DT?	
1.2	Computational time step	t	R/S			DT?=(Δt)
2.	RAINFALL					
	For a Chicago design storm go to step 2.2					
	otherwise use step 2.1 to input hyetograph					
2.1	User provided hyetograph:		B		RDTA?	
	(a) Data stored on a card	1	R/S		CARD	
	(Feed in data card)					
	or					
	(b) Data to be keyed in	0	R/S		I1?	
	(i) Single input	i	R/S		I2?	I1?=(i_1)
	and/or				etc	
	(ii) Multiple input		c		M?	
		m	R/S		IM?	M?=(m)
		i_m	R/S		I_j?	IM?=(i_m)
	(c) Data correction:		b		N?	
	Subscript	N	R/S		IN?	
	Intensity	i_N	R/S		IN+1?	IN?=(i_N)
	(d) Terminate input	-1	R/S		WDTA?	
	(i) Record data on a card	1	R/S			
	(Feed in card)					
	or					
	(ii) Data not to be recorded	0	R/S			
2.2	Chicago design storm		C		Td?	
		t_d	R/S		a?	Td? =(t_d)
		a	R/S		b?	a? =(a)
		b	R/S		c?	b? =(b)
		c	R/S		R?	c? =(c)
		r	R/S			R? =(r)
	For parameters that have been previously					
	defined merely press R/S					
3.	ISOCHRONAL AREAS					
	If isochronal areas are pre-determined use					
	step 3.1 otherwise go to step 3.2 to input					
	sub-catchment data					
3.1	Isochronal areas (maximum of 15)		D		DA1?	
	(a) Input isochronal areas	A1	R/S		DA2?	DA1?=(DA_1)
					etc	
	(b) Correct input errors:		d		N?	
	Subscript	N	R/S		DAN?	
	Corrected area	A_N	R/S			
			R/S			
	(c) Terminate input	-1				ΣDA=(ΣΔA)

Table D.1 - cont.

STEP	INSTRUCTIONS	INPUT DATA/UNITS	KEYS (SIZE: 123)		OUTPUT: DISPLAY	OUTPUT: PRINTER
3.2	Subcatchment data		E		A1?	
	(a) Area	$A1$	R/S		Te1?	A1?=(A_1)
	Entry time	t_e	R/S		Tf1?	Te1?=(t_{e1})
	Flow time	t_f	R/S		A2?	TF1?=(t_{f1})
					etc	
	(b) Correct input errors		e		N?	
	Catchment number	-N	R/S		A-N?	
	Key in incorrect data	A	R/S		Te-N?	A-N?=(A_N)
		t_e	R/S		TF-N?	Te-N?=(t_e)
		t_f	R/S		AN?	TF-N?=(t_f)
	Key in correct data as in step 3.2(a)	A	R/S			
		t_e	R/S			
		t_f	R/S			
	(c) Change subcatchment number		e		N?	
		N_{new}	R/S		N_{new}	
	(d) Terminate input	-1	R/S			DA1 =(ΔA_1)
						DA2 =(ΔA_2)
						etc
						ΣDA =($\Sigma\Delta A$)
4.	LOSS PARAMETERS		F		DS?	
		d_s	R/S		FØ?	DS? =(d_s)
		f_o	R/S		Fc?	FØ? =(f_o)
		f_c	R/S		K?	Fc? =(f_c)
		k	R/S		%AS?	K? =(k)
		$\%A_s$	R/S			%AS?=($\%A_s$)
	For parameters that have been previously defined merely press R/S					
5.	HYDROGRAPH COMPUTATIONS					
	Set display for printing discharge		G		DISP?	
	Output: discretized rainfalls for Chicago					TØ =(t_o)
	design storm					I1 =(i_1)
						I2 =(i_2)
						etc
	Output: time					T =(t)
	excess rainfall					Ie =(i_e)
	flow					Q1 =(Q_1)
						etc
6.	PRINTING OPTIONS					
	(a) Catchment hydrograph		H			Q1 =(Q_1)
						Q2 =(Q_2)
						etc
	(b) Isochronal areas		I			DA1 =(ΔA_1)
						DA2 =(ΔA_2)
						etc
						ΣDA =($\Sigma\Delta A$)
	(c) Hyetograph		J			Dt =(Δt)
						I1 =(i_1)
						I2 =(i_2)
						etc

Table D.2 Listing of HP-41C program

LINE	KEY ENTRY	COMMENTS
01	◆LBL "TA"	Program name
02	◆LBL a	Initialise for a new catchment
03	SF 12	
04	"TIME--AREA"	
05	ADV	
06	AVIEW	
07	"HYDROGRAPH"	
08	AVIEW	
09	CF 12	
10	CLD	
11	87,122	Q registers
12	,	
13	STO 01	
14	◆LBL 28	Clear Q registers
15	STO IND Y	
16	ISG Y	
17	GTO 28	
18	◆LBL A	Time step
19	24	
20	STO 00	Δt adress
21	ADV	
22	"dT"	Input Δt
23	XEQ 04	
24	FS? 05	Chicago storm?
25	SF 03	
26	FS? 05	
27	GTO 02	Test $t_d/\Delta t$
28	GTO 03	Terminate input
29	◆LBL 04	Data prompting
30	CF 22	
31	ISG 00	
32	,	
33	"⊢?"	
34	PROMPT	
35	FC?C 22	If no input, then stop, otherwise store data
36	RTN	
37	STO IND 00	
38	◆LBL 05	Print data
39	"⊢= "	
40	ARCL X	
41	AVIEW	
42	CLD	
43	RTN	
44	◆LBL c	Constant intensity
45	12	
46	STO 00	
47	" M"	
48	XEQ 04	
49	"IM"	
50	XEQ 04	
51	80	
52	STO 00	
53	◆LBL 06	Overwrite previous input
54	XEQ 09	
55	CHS	
56	XEQ 08	
57	RCL 14	i_m
58	XEQ 08	Store i_m
59	ISG 12	j=j+1
60	GTO 00	
61	GTO 16	j=90
62	◆LBL 00	
63	DSE 13	m=m-1
64	GTO 06	
65	GTO 15	
66	◆LBL B	Hyetograph input
67	CF 03	
68	CF 05	
69	"RDTA?"	Data card?
70	PROMPT	
71	X=0?	
72	GTO 07	
73	25,056	i registers
74	RDTAX	
75	GTO J	Print Δt and i
76	◆LBL b	Correct hyetograph
77	"N?"	
78	PROMPT	
79	STO Y	
80	90	
81	X<>Y	If N>90, then display : DATA ERROR
82	-	
83	SQRT	
84	RDN	
85	,09	
86	+	
87	RCL 12	Number for store
88	STO 26	routine
89	RDN	
90	STO 12	
91	GTO 15	
92	◆LBL 07	Initialise for hyetograph input
93	,	
94	STO 26	
95	80	
96	STO 00	
97	1,09	
98	STO 12	
99	ADV	
100	FS? 03	
101	RTN	
102	◆LBL 15	Hyetograph input/output
103	"I"	
104	FIX 0	
105	CF 29	
106	ARCL 12	
107	FC? 03	
108	"⊢?"	
109	FC? 03	
110	PROMPT	
111	X<0?	- 1?
112	GTO 16	Finalise input
113	STO 09	
114	XEQ 05	
115	FS? 02	
116	RTN	
117	XEQ 09	Overwrite previous input
118	CHS	
119	XEQ 08	
120	RCL 09	Store hyetograph
121	XEQ 08	
122	FS? 03	
123	RTN	
124	ISG 12	
125	GTO 15	
126	◆LBL 16	Finalise hyetograph input
127	RCL 12	
128	1	
129	-	
130	RCL 26	
131	X<Y?	
132	X<>Y	
133	STO 26	Store final number
134	FS? 03	Chicago storm?
135	GTO 20	
136	"WDTA?"	Write data on to a card?
137	PROMPT	
138	X=0?	No
139	GTO 03	Terminate input
140	RCL 02	Data register number
141	SF 14	
142	WDTAX	Write data
143	GTO 03	Terminate input
144	◆LBL 08	Store routine
145	FIX 0	
146	RND	
147	RCL 12	Counter
148	INT	
149	RCL 00	Starting number
150	+	
151	3	
152	/	
153	FRC	
154	X=0?	
155	GTO 01	
156	,4	
157	X>Y?	
158	GTO 00	
159	X<> Z	
160	1 E7	
161	ST/ Y	
162	GTO 01	
163	◆LBL 00	
164	X<> Z	
165	1 E3	
166	ST/ Y	
167	◆LBL 01	
168	RDN	
169	ST+ IND L	Store
170	RTN	
171	◆LBL 09	Recall routine
172	RCL 12	Counter
173	INT	
174	RCL 00	Starting number
175	+	
176	3	
177	/	
178	FRC	
179	RCL IND	Recall

Table D.2 - cont.

LINE	KEY ENTRY	COMMENTS
	L	
180	X<>Y	
181	X=0?	
182	GTO 01	
183	,4	
184	X>Y?	
185	GTO 00	
186	RDN	
187	1 E4	
188	ST* Z	
189	◆LBL 00	
190	X<> Z	
191	FRC	
192	1 E3	
193	ST* Y	
194	◆LBL 01	
195	RDN	
196	INT	
197	RTN	
198	◆LBL C	Chicago storm
199	SF 03	
200	SF 05	
201	14	
202	STO 00	
203	"Td"	
204	ADV	
205	XEQ 04	
206	"a"	
207	XEQ 04	
208	"b"	
209	XEQ 04	
210	"c"	
211	XEQ 04	
212	"R"	
213	XEQ 04	
214	◆LBL 02	
215	RCL 15	
216	RCL 25	
217	/	
218	89	
219	X<>Y	If $t_d/\Delta t>89$, then display:
220	-	DATA ERROR
221	SQRT	Terminate input
222	GTO 03	
223	◆LBL d	Correct isochronal

LINE	KEY ENTRY	COMMENTS
224	"N?"	areas
225	PROMPT	
226	SF 00	
227	15	
228	X<>Y	
229	X<=Y?	If N>15, then display:
230	GTO 00	
231	,	
232	/	DATA ERROR
233	◆LBL D	Isochronal area input
234	,	
235	STO 07	$\tau_{max}=0$
236	ADV	
237	GTO 00	
238	◆LBL I	Print isochronal areas
239	CF 02	
240	SF 01	
241	ADV	
242	◆LBL 00	Initialise for isochronal area routines
243	FS? 01	
244	RCL 07	
245	FC? 01	
246	15	
247	1 E3	
248	/	
249	STO 00	
250	RDN	
251	FC?C 00	
252	1	
253	ST+ 00	
254	◆LBL 10	Isochronal areas
255	"DA"	
256	◆LBL 11	Input/output routine for isochronal areas and subcatchment data
257	FIX 0	
258	CF 29	
259	FS? 04	
260	"⊢-"	
261	RCL 00	
262	ARCL X	
263	FC? 01	
264	"⊢?"	
265	71	
266	+	Set indirect store/recall counter
267	FIX 3	
268	SF 29	

LINE	KEY ENTRY	COMMENTS
269	FC? 01	
270	PROMPT	
271	FS? 02	
272	RTN	
273	X<0?	
274	GTO 00	Finalise input
275	FS? 01	
276	RCL IND X	ΔA
277	XEQ 05	
278	FC? 01	
279	STO IND Y	ΔA
280	ISG 00	
281	GTO 10	
282	◆LBL 00	Finalise τ_{max}
283	RCL 00	
284	INT	
285	1	
286	-	
287	RCL 07	
288	X<=Y?	
289	X<>Y	
290	STO 00	
291	STO 07	τ_{max}
292	CLX	
293	CF 01	
294	◆LBL 12	Sum isochronal areas
295	RCL 00	
296	71	
297	+	
298	RDN	
299	RCL IND T	
300	+	
301	DSE 00	
302	GTO 12	
303	"ΣDA"	
304	XEQ 05	
305	GTO 03	Terminate input
306	◆LBL e	Correct subcatchment data
307	"N?"	
308	PROMPT	
309	X<0?	
310	SF 04	
311	ABS	
312	STO 00	

LINE	KEY ENTRY	COMMENTS
313	GTO 17	Data input
314	◆LBL E	Input Subcatchment data
315	72,086	
316	,	
317	STO 07	$\tau_{max}=0$
318	◆LBL 13	Clear isochronal area registers
319	STO IND Y	
320	ISG Y	
321	GTO 13	
322	1	
323	STO 00	
324	◆LBL 17	Compute isochronal areas from subcatchment data
325	SF 02	
326	"A"	
327	ADV	
328	XEQ 11	
329	X<0?	
330	GTO I	
331	"⊢ "	
332	XEQ 05	
333	STO 08	A
334	STO 09	
335	"Te"	
336	XEQ 11	
337	XEQ 05	
338	RCL 25	
339	/	
340	ST/ 09	A/τ_e
341	"TF"	
342	XEQ 11	
343	XEQ 05	
344	RCL 25	
345	/	
346	STO 10	τ_f
347	RCL Z	
348	+	
349	,99	
350	+	
351	INT	
352	STO 11	Subcatchment τ_{max}
353	RCL 07	
354	X<=Y?	
355	X<>Y	
356	STO 07	Catchment τ_{max}
357	15	
358	RCL 11	If τ_{max} >15, then

Table D.2 - cont.

LINE	KEY ENTRY	COMMENTS
359	-	display
360	SQRT	DATA ERROR
361	RCL 08	
362	◆LBL 14	
363	X<0?	
364	GTO 00	
365	RCL 09	
366	RCL 11	
367	1	
368	-	
369	RCL 10	
370	-	
371	*	$(A/\tau_e)(\tau-1-\tau_f)$
372	X<0?	
373	CLX	
374	STO Z	
375	-	ΔA_τ
376	FS? 04	
377	CHS	
378	RCL 11	
379	71	
380	+	
381	X<>Y	
382	ST+ IND Y	
383	RCL Z	A_τ
384	DSE 11	
385	GTO 14	
386	◆LBL 00	Increment subcatchment number unless data is being corrected
387	FC?C 04	
388	ISG 00	
389	,	
390	GTO 17	
391	◆LBL F	Loss parameters
392	19	
393	STO 00	
394	"dS"	
395	ADV	
396	XEQ 04	
397	"F0"	
398	XEQ 04	
399	"Fc"	
400	XEQ 04	
401	"K"	
402	XEQ 04	
403	"%AS"	
404	XEQ 04	

LINE	KEY ENTRY	COMMENTS
405	GTO 03	Terminate input
406	◆LBL G	Go step
407	RCL 01	
408	"DISP?"	
409	PROMPT	Set display
410	STO 01	
411	FC? 03	Hyetograph given?
412	GTO 20	Skip discretising
413	XEQ 07	Initialise discretising
414	1	
415	STO 02	
416	RCL 19	
417	STO 06	
418	ST- 02	1-r
419	STO 05	r
420	RCL 15	
421	ST* 02	$(1-r)\ t_d$
422	ST* 05	rt_d
423	RCL 25	
424	STO 03	
425	-	
426	*	
427	RCL 25	
428	/	$r(t_d-\Delta t)/\Delta t$
429	ENTER↑	
430	FIX 0	
431	RND	
432	-	
433	RCL 25	
434	*	t_o
435	ST+ 02	$(1-r)t_d+t_o$
436	ST- 05	rt_d-t_o
437	STO 04	
438	RND	
439	CF 29	
440	"T0"	
441	XEQ 05	Print t_o
442	◆LBL 18	Compute i_b
443	RCL 25	
444	RCL 05	
445	X<=Y?	
446	GTO 00	
447	XEQ 01	
448	GTO 18	
449	◆LBL 00	Peak intensity
450	-	
451	STO 05	$t_3=\Delta t-t_b$

LINE	KEY ENTRY	COMMENTS
452	RCL 25	
453	ST- 03	$-\Delta t$
454	ST- 03	
455	XEQ 00	
456	XEQ 02	
457	1	
458	RCL 19	
459	-	
460	X=0?	
461	GTO 16	
462	STO 06	$\rho=1-r$
463	◆LBL 19	Compute i_a
464	RCL 05	
465	RCL 02	
466	X<=Y?	$t_{a\ max} \leq t_a$?
467	GTO 16	
468	XEQ 01	
469	GTO 19	
470	◆LBL 01	Compute average intensity, i, for next time increment
471	RCL 05	
472	RCL 06	
473	/	
474	XEQ 00	
475	RCL 05	
476	*	P_1
477	LASTX	
478	RCL 03	
479	-	
480	STO 05	$t\pm\Delta t$
481	RCL 06	
482	/	
483	XEQ 00	
484	RCL 05	
485	*	P_2
486	-	
487	RCL 03	
488	/	$i=(P_1-P_2)/\Delta t$
489	◆LBL 02	Print hyetograph
490	STO 09	i
491	XEQ 15	
492	ISG 12	
493	RTN	
494	◆LBL 00	Average intensity
495	RCL 17	
496	+	
497	RCL 18	

LINE	KEY ENTRY	COMMENTS
498	CHS	
499	Y↑X	
500	RCL 16	
501	*	$I=a/(t+b)^c$
502	RTN	
503	◆LBL 20	Initialise for excess rainfall
504	SF 03	
505	SF 04	
506	57,071	
507	,	
508	STO 06	t=o
509	STO 13	F_d=o
510	◆LBL 21	Clear run-off registers
511	STO IND Y	
512	ISG Y	
513	GTO 21	
514	SF 01	
515	RCL 26	Number of intensities
516	INT	
517	1 E3	
518	/	
519	1	
520	+	Counter for Recall routine
521	STO 08	
522	,	
523	STO 11	
524	RCL 20	
525	STO 04	d_s
526	RCL 21	
527	RCL 22	
528	-	
529	RCL 23	
530	X≠0?	
531	/	
532	STO 02	Y_1
533	1	
534	LASTX	
535	RCL 25	
536	*	
537	60	
538	/	
539	CHS	
540	E↑X	
541	-	
542	STO 03	Y_2
543	◆LBL 22	Excess rainfall

Table D.2 - cont.

LINE	KEY ENTRY	COMMENTS
544	80	Starting number for Recall routine
545	STO 00	
546	RCL 08	
547	STO 12	
548	XEQ 09	Recall i
549	RCL 24	
550	%	
551	+	$i=(1+\%A_s/100)i$
552	RCL 25	
553	*	
554	60	
555	/	$i\,\Delta t$
556	RCL 02	
557	RCL 13	
558	-	
559	RCL 03	
560	*	
561	RCL 22	
562	RCL 25	
563	*	
564	60	
565	/	
566	STO 14	$\Delta F_c=Y_2(Y_1-F_d)+f_c\Delta t$
567	+	
568	ST- 14	
569	X≠0?	
570	ST/ 14	$(f_c\Delta t-\Delta F_c)/\Delta F_c$
571	X>Y?	$F_c>i\Delta t$?
572	RDN	$\Delta F=i\Delta t$
573	ST* 14	F_d
574	-	
575	RCL 04	
576	ST- 04	$d_s=0$
577	-	P_e
578	X<0?	$P_e<0$
579	ST- 04	$ds=-P_e$ and $d_e=0$
580	X<0?	
581	,	
582	RCL 25	
583	/	
584	60	
585	*	i_e
586	FS? 01	
587	X=0?	Start computing runoff only after first non zero i_e
588	FC? 01	
589	CF 04	
590	XEQ 23	
591	RCL 14	
592	ST- 13	
593	ISG 08	

LINE	KEY ENTRY	COMMENTS
594	GTO 22	
595	CF 03	
596	♦LBL 23	Compute discharge
597	FC? 03	
598	,	
599	STO 05	i_e
600	RCL 25	
601	ST+ 06	$t=t+\Delta t$
602	ISG 11	
603	,	
604	FS? 04	
605	RTN	
606	CF 01	
607	RCL 07	
608	71	
609	+	
610	1 E3	
611	/	
612	71	
613	+	
614	STO 10	Counter for runoff
615	CF 29	
616	FIX 0	
617	RCL 06	t
618	"T "	
619	ADV	
620	XEQ 05	Print t
621	RCL 05	
622	"Ie"	
623	XEQ 05	Print i_e
624	X=0?	Skip next routine if $i_e=0$
625	GTO 00	
626	♦LBL 24	
627	RCL 10	
628	15	
629	-	
630	RCL IND 10	
631	RCL 05	$i_e t\Delta A_{\tau+1}$
632	*	
633	ST+ IND Y	$R_{\tau-1,t}$
634	ISG 10	
635	GTO 24	
636	♦LBL 00	Compute and Store Q
637	RCL 57	
638	360	

LINE	KEY ENTRY	COMMENTS
639	/	
640	"Q"	
641	ARCL 11	
642	SF 29	
643	STO 09	Q_j
644	FIX IND 01	
645	RND	
646	XEQ 05	Print Q_j
647	263	Starting number for Recall/Store routine
648	STO 00	
649	RCL 11	
650	STO 12	
651	XEQ 09	
652	STO 05	Q_{j-1}
653	CHS	
654	XEQ 08	
655	RCL 05	Add Q_j to Q_{j-1} and store in condensed format
656	1 E2	
657	/	
658	10↑X	
659	1	
660	-	
661	1 E3	
662	/	
663	RCL 09	
664	+	
665	1 E3	
666	*	
667	1	
668	+	
669	LOG	
670	1 E2	
671	*	
672	XEQ 08	
673	,	
674	STO 71	
675	58,07	
676	♦LBL 25	Route runoff
677	RCL IND X	
678	DSE Y	
679	,	
680	STO IND Y	
681	X<>Y	
682	ISG X	
683	ISG X	
684	GTO 25	

LINE	KEY ENTRY	COMMENTS
685	FS? 03	
686	RTN	
687	RCL 09	
688	FIX IND 01	
689	RND	
690	X≠0?	
691	GTO 23	
692	RCL 11	
693	RCL 87	Update total number of Q's
694	X<Y?	
695	X<>Y	
696	STO 87	
697	GTO 03	Terminate output
698	♦LBL H	Print catchment hydrograph
699	263	
700	STO 00	
701	RCL 87	
702	1 E3	
703	/	
704	1	
705	+	
706	STO 12	
707	ADV	
708	♦LBL 26	
709	CF 29	
710	FIX 0	
711	"Q"	
712	ARCL 12	
713	XEQ 09	
714	1 E2	
715	/	
716	10↑X	
717	1	
718	-	
719	1 E3	
720	/	
721	SCI 1	Round Q off to two digits
722	RND	
723	SF 29	
724	FIX IND 01	
725	RND	
726	XEQ 05	
727	ISG 12	
728	GTO 26	
729	GTO 03	Terminate output

Table D.2 - cont.

LINE	KEY ENTRY	COMMENTS	LINE	KEY ENTRY	COMMENTS
730	•LBL J	Print hyetograph			
731	00				
732	STO 00	Initialise			
733	RCL 26				
734	INT				
735	1 E3				
736	/				
737	1				
738	+				
739	STO 12				
740	RCL 25				
741	"dT"				
742	ADV				
743	XEQ 05	Print Δt			
744	SF 02				
745	SF 03				
746	•LBL 27				
747	XEQ 09	Recall i_j			
748	XEQ 15	Print i_j			
749	ISG 12	$j=j+1$			
750	GTO 27				
751	CF 02				
752	CF 03				
753	•LBL 03	Terminate in/			
754	FIX 3	output			
755	SF 29				
756	TONE 9				
757	END				

Table D.3 HP-41C Calculator status

REGISTERS

00	Counter; starting numbers for store/recall	16	a
01	Indirect display	17	b
02	i-registers; $(1-r)t_d+t_o$; γ_1	18	c
03	$\pm\Delta t$; γ_2	19	r
04	t_o; d_s	20	d_s
05	t_a; i_e; Q_{j-1}	21	F_o
06	ρ ; t	22	F_c
07	τ_{max}	23	k
08	A	24	$\%A_s$
09	A/τ_e; i_j; Q_j	25	Δt
10	τ_f ; counter	26	Number of intensities
11	τ ; counter	27–56	i_j , i_{j+1} 0 i_{j+2}
12	Counter for store/recall	57–71	R_τ
13	m; F_c	72–86	ΔA_τ
14	i_m; ΔF_c	87	Number of discharges
15	t_d	88–122	100LOG(1000Q) , 100LOG(1000Q_{j+1}+1) 0 100LOG(1000Q_{j+2}+1)

FLAGS

No	Initial Status	SET INDICATES	CLEAR INDICATES
00	C	Correct isochronal areas	
01	C	Print isochronal areas	
	C	Excess rainfall calcs.	first non-zero i_e encountered
02	C	Compute isochronal areas	print isochronal areas
	C	Print hyetograph	
03	C	Chicago storm	hyetograph
	C	$i_e \neq 0$	$i_e = 0$
	C	Print hyetograph	
04	C	Correct subcatchment data	
	C	Excess rainfall calcs.	runoff calcs
05	C	Chicago storm	no Chicago storm
12	C	Print double width	print normal width
14	C	Overwrite protected data card	
22	C	Numeric data input	no data input
29	C	Digits grouped	digits not grouped

SET STATUS

SIZE	123	TOT. REG	314	USER MODE	___				
ENG	___	FIX	XX	SCI	___	ON	XX	OFF	___
DEG	XX	RAD	___	GRAD	___				

D.2 Example Applications

Printouts for runs on the three examples described in Appendix B (for the HP-97 programs) are presented in Figs. D.2 to D.4. Extensive use of the alpha-numeric capability of the HP-41C renders the printouts easy to interpret. Reference should be made to Appendix B for further explanations.

```
TIME--AREA
HYDROGRAPH

dT?= 10,000

I1?= 117
I2?= 23
I3?= 1

A1? = 14,200
Te1?= 58,000
TF1?= 12,000

A2? = 7,300
Te2?= 38,000
TF2?= 12,000

A3? = 7,500
Te3?= 44,000
TF3?= 10,000

A4? = 4,500
Te4?= 35,000
TF4?= 8,000

A5? = 10,200
Te5?= 37,000
TF5?= 8,000

A6? = 9,600
Te6?= 45,000
TF6?= 8,000

A7? = 4,600
Te7?= 42,000
TF7?= 5,000

A8? = 6,300
Te8?= 61,000
TF8?= 2,000

A9? = 11,400
Te9?= 35,000
TF9?= 5,000

A10? = 7,800
Te10?= 52,000
TF10?= 2,000

DA1= 5,438
DA2= 18,261
DA3= 19,135
DA4= 19,135
DA5= 13,271
DA6= 5,403
DA7= 2,758
ΣDA= 83,400

dS?= 5,000
F0?= 7,000
Fc?= 6,000
K?= 2,000
%AS?= 0,000

T = 10
Ie= 80
Q1= 1,21

T = 20
Ie= 16
Q2= 4,31

T = 30
Ie= 6
Q3= 5,09

T = 40
Ie= 0
Q4= 5,13

T = 50
Ie= 0
Q5= 3,83

T = 60
Ie= 0
Q6= 1,81

T = 70
Ie= 0
Q7= 0,86

T = 80
Ie= 0
Q8= 0,13

T = 90
Ie= 0
Q9= 0,00

Q1= 1,20
Q2= 4,30
Q3= 5,10
Q4= 5,10
Q5= 3,80
Q6= 1,80
Q7= 0,87
Q8= 0,12
Q9= 0,00
```

Fig. D.2 HP–41C printout for example 1

```
TIME--AREA
HYDROGRAPH

dT?= 1,000

M?= 5
IM?= 93
M?= 7
IM?= 47

DA1?= 180,000
DA2?= 180,000
ΣDA= 360,000

dS?= 0,000
F0?= 0,000
Fc?= 0,000
K?= 0,000
%AS?= 0,000

T = 1
Ie= 93
Q1= 47,

T = 2
Ie= 93
Q2= 93,

T = 3
Ie= 93
Q3= 93,

T = 4
Ie= 93
Q4= 93,

T = 5
Ie= 93
Q5= 93,

T = 6
Ie= 47
Q6= 70,

T = 7
Ie= 47
Q7= 47,

T = 8
Ie= 47
Q8= 47,

T = 9
Ie= 47
Q9= 47,

T = 10
Ie= 47
Q10= 47,

T = 11
Ie= 47
Q11= 47,

T = 12
Ie= 47
Q12= 47,

T = 13
Ie= 0
Q13= 24,

T = 14
Ie= 0
Q14= 0,

Q1= 47,
Q2= 93,
Q3= 93,
Q4= 93,
Q5= 93,
Q6= 71,
Q7= 47,
Q8= 47,
Q9= 47,
Q10= 47,
Q11= 47,
Q12= 47,
Q13= 23,
Q14= 0,
```

Fig. D.3 HP–41C printout for example 2

TIME--AREA
HYDROGRAPH

dT?= 5,000

Td?= 90,000
a?= 3000,000
b?= 14,400
c?= 0,883
R?= 0,400

DA1?= 6,850
DA2?= 14,050
DA3?= 7,200
ΣDA= 28,100

dS?= 1,000
F0?= 0,000
Fc?= 0,000
K?= 0.000
%AS?= 0,000

T0= -1
I1= 12
I2= 15
I3= 19
I4= 25
I5= 35
I6= 55
I7= 110
I8= 219
I9= 124
I10= 74
I11= 50
I12= 37
I13= 29
I14= 23
I15= 20
I16= 17
I17= 15
I18= 13

T = 10
Ie= 15
Q2= 0,29

T = 15
Ie= 19
Q3= 0,95

T = 20
Ie= 25
Q4= 1,52

T = 25
Ie= 35
Q5= 2,02

T = 30
Ie= 55
Q6= 2,91

T = 35
Ie= 110
Q7= 4,94

T = 40
Ie= 219
Q8= 9,56

T = 45
Ie= 124
Q9= 13,11

T = 50
Ie= 74
Q10= 10,63

T = 55
Ie= 50
Q11= 6,32

T = 60
Ie= 37
Q12= 4,14

T = 65
Ie= 29
Q13= 3,00

T = 70
Ie= 23
Q14= 2,31

T = 75
Ie= 20
Q15= 1,86

T = 80
Ie= 17
Q16= 1,56

T = 85
Ie= 15
Q17= 1,35

T = 90
Ie= 13
Q18= 1,17

T = 95
Ie= 0
Q19= 0,81

T = 100
Ie= 0
Q20= 0,26

T = 105
Ie= 0
Q21= 0,00

DA1?= 6,420
DA2?= 12,540
DA3?= 12,540
DA4?= 12,540
DA5?= 12,540
DA6?= 12,540
DA7?= 12,540
DA8?= 12,540
DA9?= 6,120
ΣDA= 100,320

dS?= 5,000
F0?= 66,000
Fc?= 13,000
K?= 2,000
%AS?= 15,000

T = 35
Ie= 40
Q7= 0,72

T = 40
Ie= 214
Q8= 5,22

T = 45
Ie= 109
Q9= 10,80

T = 50
Ie= 54
Q10= 13,62

T = 55
Ie= 30
Q11= 15,07

T = 60
Ie= 17
Q12= 15,87

T = 65
Ie= 10
Q13= 16,33

T = 70
Ie= 4
Q14= 16,57

T = 75
Ie= 2
Q15= 15,97

T = 80
Ie= 4,E-2
Q16= 11,51

T = 85
Ie= 0
Q17= 5,93

T = 90
Ie= 0
Q18= 3,11

T = 95
Ie= 0
Q19= 1,66

T = 100
Ie= 0
Q20= 0,86

T = 105
Ie= 0
Q21= 0,40

T = 110
Ie= 0
Q22= 0,16

T = 115
Ie= 0
Q23= 0,04

T = 120
Ie= 0
Q24= 0,00

Q1= 0,00
Q2= 0,29
Q3= 0,95
Q4= 1,50
Q5= 2,00
Q6= 2,90
Q7= 5,60
Q8= 15,00
Q9= 24,00
Q10= 25,00
Q11= 21,00
Q12= 20,00
Q13= 19,00
Q14= 19,00
Q15= 18,00
Q16= 13,00
Q17= 7,20
Q18= 4,30
Q19= 2,50
Q20= 1,10
Q21= 0,40
Q22= 0,16
Q23= 0,04
Q24= 0,00

Fig. D.4 HP-41C printout for example 3