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A Study of the Effect of Urban Development on Streamflow, using Computer Simulation and Statistical Analysis.

by Michael Charles McGovern.

A Thesis Submitted for the Degree of Doctor of Philosophy.

Department of Civil Engineering
The City University,
London.

May 1975.

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LIST OF SYMBOLS

a	parameter in Kostiakov's infiltration equation
A	a parameter
API	Antecedent Precipitation Index
D	depression storage capacity (mm)
e	a constant
f	infiltration capacity rate (mm/hr)
f_c	infiltration capacity when t is large (mm/hr)
F	accumulated soil moisture (mm); an objective function; a statistic
F_{max}	maximum value of the objective function
F_{min}	minimum value of the objective function
F_C, F_E, F_R	value of the objective function at the contracted, expanded, and reflected point on the 'simplex'
k_d	a parameter in the depression storage component
k	a parameter
k_f	infiltration capacity decay constant
k_r	a storage coefficient in the routing component
m	the rank of an annual flood peak
n	number of parameters; number of years
p_1	precipitation rate (mm/hr)
P_1	accumulated precipitation (mm)
p_2	excess from infiltration component (mm/hr)
P_2	accumulated excess from infiltration component (mm)
p_3	excess from depression storage component (mm/hr)

P_3 accumulated excess from the depression storage component (mm)

q discharge

Q flow volume

Q_i observed flow

r rate of inflow to depression storage

R volume of water in depression storage; multiple correlation coefficient

s parameter in Philip's infiltration equation

S storage in the catchment channel system

S_i simulated flow

T_p return period

y a dummy variable used in the derivation of the depression storage component

PREFACE

The research reported in this thesis was carried out to investigate the effects of urban development on streamflow and to summarise previous work on the problem.

The various approaches to the problem are discussed in Chapter 1 where the scope of the research is defined. The hydrological processes which may be affected by urban development are discussed in Chapter 2 and a review is given of previous work in this field.

Two lines of approach were used in the study (1) deterministic simulation of individual storms and (2) multiple regression analysis of mean daily flow and flow peaks. Flood frequency analysis of recorded peaks and peaks predicted by a multiple regression model was also carried out. For the deterministic simulation, a hybrid computer was utilized. The development of the model and the automatic search method used to find the model parameters are discussed in Chapter 3. The model was used to simulate a number of storm events occurring on the catchment of Crawbers Brook, which has undergone a twenty-year period of urbanization. The urban development of the catchment and the collection of the data used in the analysis are described in Chapter 4. This is followed by a discussion of applications of the simulation model in Chapter 5 and a presentation of the results. The multiple regression analysis of mean daily flow and of flow peaks is described in Chapter 6. Chapter 7 summarises the

results of the research.

The work was carried out in the Department of Civil Engineering at the City University, London and was financed by the Natural Environment Research Council.

The writer would like to thank his supervisor, Dr. W.J. Morris, for his help and advice in carrying out the research. Gratitude is also extended to [redacted] for his encouragement; to the computer unit staff at the City University for helping to de-bug the programmes; to the Geography Department at the London School of Economics for the use of their digitizer; to Mr. [redacted] for lending his maps, data and experience; to Mrs.M.Jones for typing this thesis, and to my wife for her help and encouragement.

ABSTRACT

An investigation of the effect of urban development on streamflow was carried out using computer simulation and multiple regression analysis. In addition, a review of past work in this field is presented.

Storms occurring on the Crawters Brook catchment during the twenty-year urban development of Crawley, Sussex were analysed on a hybrid computer. A hybrid computer simulation model was developed to simulate the short-term hydrological processes of the catchment. The four parameters of the model were optimized for each of the 36 storm events using the 'Simplex' direct-search method. The sensitivity of the model to parameter changes was analysed by generating hydrographs for various sets of parameters and by plotting response functions.

Multiple regression analysis was used to link parameter values to physical and meteorological variables. It was found that the percentage of impervious area in the catchment was the most important variable in explaining the variance of three of the model parameters. The regression equations could be used to predict parameter values for hypothetical catchment conditions in order to generate hydrographs from the simulation model.

Daily flow volumes were also analysed using multiple regression and it was found that increases in the impervious area causes significant increases in mean daily flow.

A flood frequency analysis was carried out linking flow peaks to physical and meteorological data using regression analysis. The regression equations were then used to predict annual peak flows for the study period for various levels of urbanization. It was found that changing a rural catchment to an urban catchment with 40 per cent

impervious area would result in the mean annual flood being increased by a factor of 2.64 and the twenty-five year flood by 2.44. The annual number of peaks above a certain threshold was also found to increase.

CHAPTER ONE

PURPOSE AND SCOPE OF THE STUDY

The urbanization of Britain has taken place over hundreds of years, but has tended to be concentrated in the flood-plains. This was a consequence of man's basic needs for water and of his use of valleys as lines of communication. However, although the benefits may be great the cost of settlement in flood-plains is also high. By building homes, industrial works and roads by a river, the risk of damage by flooding is increased and the frequency of floods may also be increased. The primary purpose of the research reported in this thesis was to investigate this problem.

In order to protect cities and towns from flooding, in the most economical way, it is necessary to have an estimate of the frequency of floods of a certain magnitude such that the costs of the flood prevention measures are outweighed by benefits derived from them in the prevention of flood damage.

This is a far from simple task for two reasons : first, even if man had no effect on the complex hydrological system, long term records enabling the calculation of flood frequencies are few and far between. Secondly, by installing flood prevention works such as culverts and embankments, the hydrological system is changed and any long term records prior to urbanization are of little use in calculating the frequency of post-urbanization floods.

A great deal of research effort has been directed toward these problems in the past, and suggested solutions range from

the highly simplified 'Rational Method' of hydrological design, to the simulation of the hydrological processes of the catchment by mathematical modelling (Rantz, 1971). However, the vast majority of these studies have been carried out in the United States (see Chapter 2.VI) and it was therefore decided that a study be made of a catchment in Britain that had undergone urbanization.

The study was carried out with the object of developing a simulation model of the catchment such that the parameters in the model could be physically related to actual changes in the catchment due to urbanization. The model could then be used to synthesize the outflow hydrograph from an urban catchment, given certain meteorological and physical conditions. It was felt that this method would be more useful than a unit hydrograph study because the parameters of the model would be physically meaningful, and the operation of the model would give insight into the operation of the hydrological system. The latter may be achieved by detailed study of the components of model output and of the interaction between model parameters.

In addition to the computer simulation of individual storms, mean daily flow may be studied by regression analysis, linking urbanization and meteorological variables to the flow data. This gives a measure of the importance of urbanization in influencing overall flow volumes. Lastly, flood frequency analysis of recorded peaks may be carried out. This operation is hampered by the fact that, on an urbanized catchment, sets of recorded peaks from a stationary data series before and

after urbanization are difficult to obtain. This may be overcome by removing the non-stationary effect of urbanization using multiple regression analysis, yielding predicted data sets corresponding to conditions before and after urbanization.

Thus, the scope of the study is limited to the effects of urbanization on the quantity of water flowing through the outlet of the catchment. Water quality has been ignored, although it is recognised that a great deal of damage can be caused by water pollution, being more insidious in its effects than flooding damage. Lack of data prohibits an investigation of water quality in the particular catchment chosen for study.

CHAPTER TWO

HYDROLOGICAL PROCESSES

Hydrological processes operate on the water in the hydrological system. They have the effect of transporting it, storing it and changing it from one state to another. Hydrological processes have been recognised as forming parts of a cyclic system and this concept has been useful in understanding the complex nature of the system. It enables the various processes to be viewed as components of the cycle. It is necessary to describe what are understood to be the basic hydrological processes in catchment hydrology because these assumptions determine the final structure of a model.

The major processes are shown in Figure 2.1. Processes acting on water in a catchment can be grouped conveniently into three phases : (1) the atmospheric phase; (2) the land phase; (3) the channel phase.

2.I. THE ATMOSPHERIC PHASE

Processes acting in the atmospheric phase have the effect of causing transport, storage or change of state of the water in the atmosphere. Water vapour in the atmosphere over a catchment is derived from two sources : (1) evaporation from the land and channel phases; and (2) horizontal advection inwards (Sellers, 1968, p.82). Similarly, the two major vapour sinks are : (1) precipitation (including dewfall); and (2)

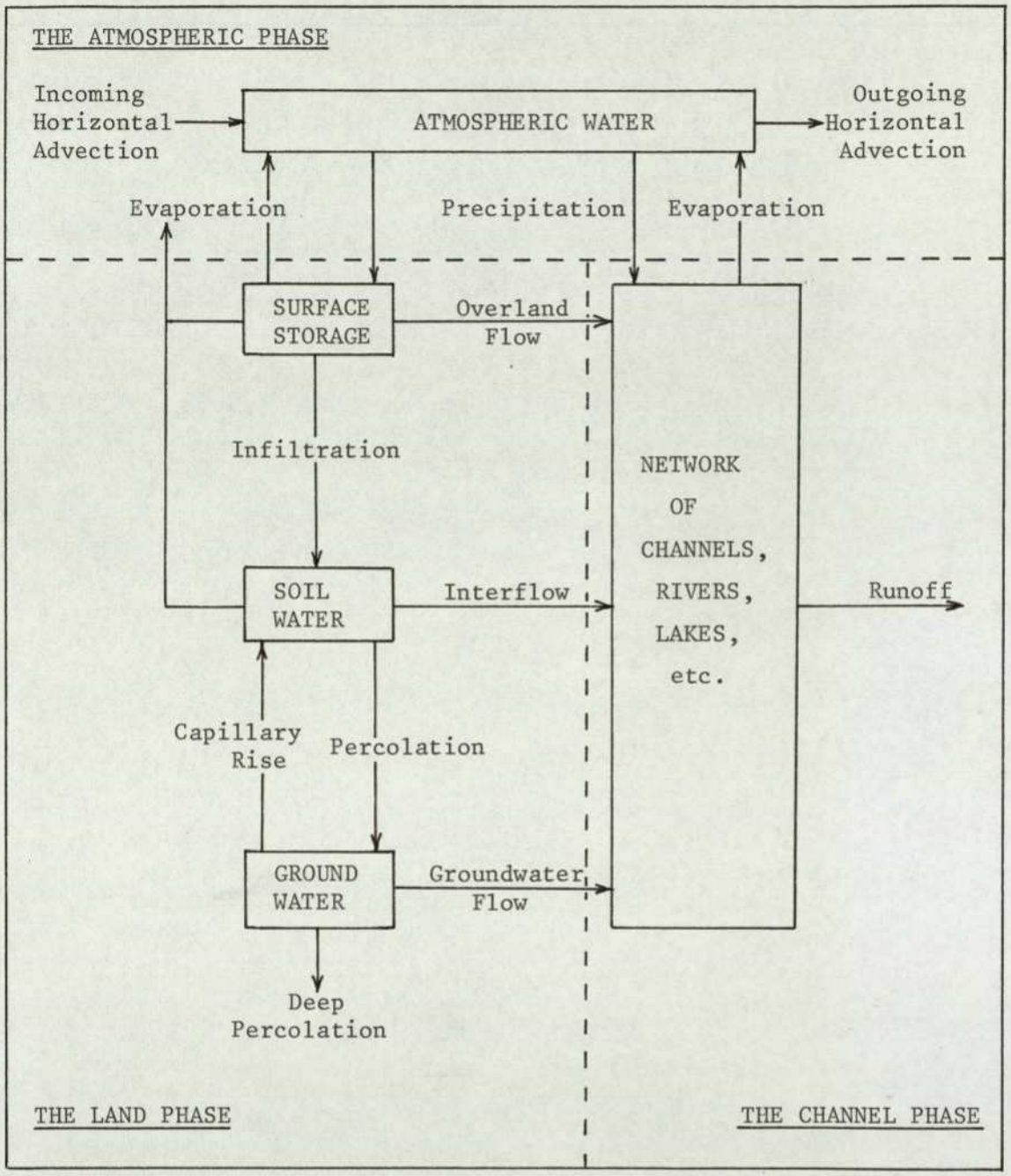


Figure 2.1 Hydrological Processes at the Catchment Scale

horizontal advection outwards. The advection terms shown in Figure 2.1 are important from the point of view of overall moisture balance, but as hydrological studies are mostly concerned with the flow of water in rivers and on the land surface, these terms are generally neglected in catchment hydrology and are studied in the field of meteorology. Clearly, in catchment hydrology, precipitation and evaporation are two important areas of study. The physics of these processes can be fairly complex, but for the purposes of the hydrologist, can be simplified considerably.

2.I.1. Precipitation. Precipitation is the process by which atmospheric water is moved from the atmospheric phase to the land or channel phases of the hydrological system. It can take many different forms - rain, sleet, snow, hail, dew, hoar-frost, fog-drip, rime - but usually only rain and snow make significant contributions to precipitation totals (Barry and Chorley, 1968, p.88).

Various statistics describing the characteristics of precipitation can be derived, such as depth, duration, average intensity and peak intensity. A short period of high intensity rainfall is termed a 'storm' and it is this phenomenon which is of most interest to the hydrologist studying flooding problems. Storms often occur as a result of convectional heat cells in the atmosphere. More prolonged periods of rain of lower intensity can also cause flooding and are often the result of the passage of 'fronts' associated with cyclonic conditions. The calculation of levels of probability associated with the occurrence of

storms of specified sizes is an important part of the study of flooding problems.

2.I.2. Evaporation. Evaporation is the process by which water is moved from the land phase or from the channel phase to the atmospheric phase. It occurs whenever energy is transported to an evaporating surface, provided the vapour pressure in the air is below the saturated value. The saturated vapour pressure is directly proportional to air temperature and thus if an evaporating surface is available, temperature is an important variable affecting evaporation. Air turbulence has the important function of allowing the mixing of saturated air in contact with the water surface with the unsaturated air above it. Wind speed is therefore a second important variable in the study of the evaporation process.

Water loss from vegetation occurs when the vapour pressure in the leaf cells is greater than the atmospheric vapour pressure, and is termed 'transpiration'. In hydrological studies at the catchment scale, it is difficult to separate the two processes of evaporation and transpiration and they are usually lumped under the term 'evapotranspiration'.

In practice, evapotranspiration is not as important as precipitation in the study of flooding problems except in the calculation of conditions antecedent to the storm. The fact that a catchment can respond quite differently to two identical storms is evidence of the influence of evapotranspiration in determining the 'water-balance' of the catchment during relatively dry periods. In studies of water yield and drought, evapotranspiration is often

incorporated in a water-balance model such as :

Initial Storage + Rainfall =

Evaporation + Runoff + Final Storage

This type of model was used by Penman (1950) to estimate the monthly water balance of the Stour catchment.

2.II THE LAND PHASE

The land phase has an important function in the hydrological system. It acts as an intermediary between the atmospheric phase and the channel phase. As shown in Figure 2.1 there are two basic processes engaged in the transfer of water between the atmospheric and land phases, namely precipitation and evaporation. Although these two basic processes also transfer water between the atmospheric phase and the channel phase, the main source of water to the network of channels and rivers is from the land phase.

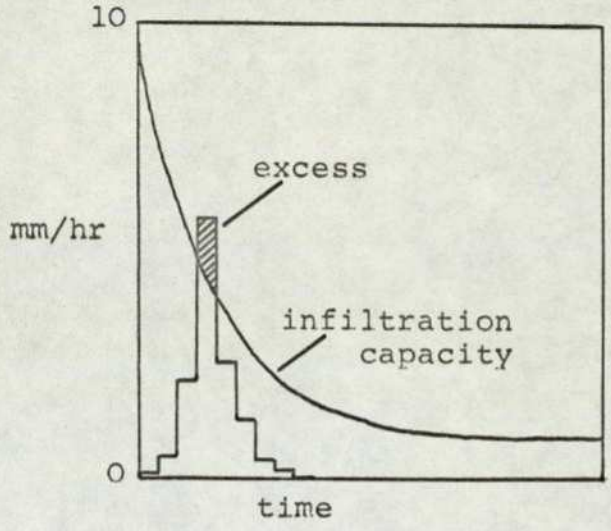
The precipitation received from the atmospheric phase by the land phase first encounters the surface storage component. This comprises interception storage, depression storage (sometimes termed 'retention') and overland flow storage (sometimes termed 'detention'). Water in surface storage may be lost to the atmospheric phase by evaporation, to the channel phase by overland flow, or to another component of the land phase - soil water storage - by infiltration.

When precipitation enters the land phase it can fall on a number of surfaces : (1) vegetation; (2) bare soil; (3) impervious areas such as roads; and (4) water already present on the land surface. Each surface gives a different hydrological response.

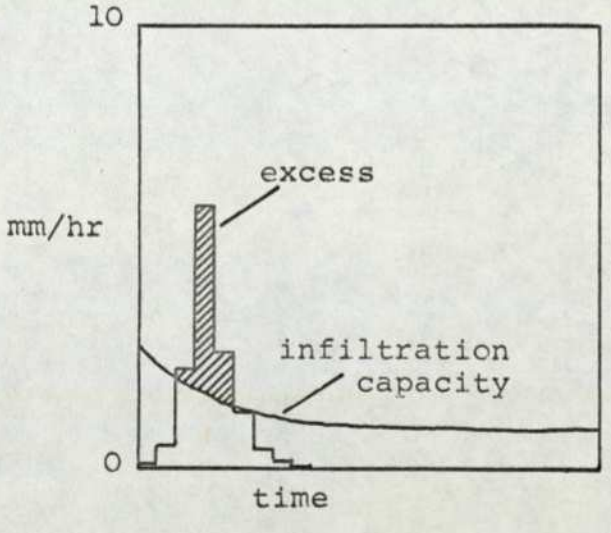
2.II.1. Interception Storage. Interception occurs when precipitation falls on vegetation, where it is temporarily stored. The volume of water in interception storage is then depleted by evaporation to the atmosphere or by through-drip and stem-flow to the soil below. In general, interception is not an important process in large storms (Linsley, Kohler and Paulhus, 1958, p.162) but can have a significant effect on the long-term water balance of a catchment, particularly in forested areas,

2.II.2. Infiltration. Precipitation falling on bare soil, water reaching the soil from interception storage, and water already in depression storage, is subject to the infiltration process. This is the process by which water moves downward from the land surface, through the soil, into the lower soil horizons. Given a constant rate of water supply, the rate at which water infiltrates into the soil has been shown to decrease approximately exponentially with time, asymptotically approaching a constant value (Dunin, 1973). Infiltration governs the short-term response of the catchment to precipitation and is therefore an important process in the study of floods. It is also of importance in water-balance studies because it controls the ratio between water passing to long-term groundwater storage and short term surface runoff and interflow.

The infiltration process is highly variable with respect to the moisture content in the soil, so that a given amount of precipitation may give rise to a large or small short-term response in the channel phase, depending on whether the infiltration rate is low or high respectively. This is shown in Figure 2.2.



(a) High initial infiltration capacity.



(b) Low initial infiltration capacity

Figure 2.2 Infiltration Effects on Streamflow

Because infiltration has such an influence on streamflow, it is incorporated in the simulation model used in this study and is discussed in more detail in Chapter 3.II.2.

2.II.3. Depression Storage. The second storage component to be encountered in the land phase is depression storage. This component represents the numerous small depressions on the land surface which serve to prevent water from flowing directly to the channel system. If the rate of water supply to the land surface exceeds the infiltration rate at a particular instant, the excess water either flows into depression storage or contributes to overland flow. The latter case occurs either when the depression storage component is full and cannot accommodate more water or when there are no depressions in the land surface and depression storage capacity is zero.

At any point in time there will be a wide variety of depression storage capacities throughout the catchment, depending on such factors as antecedent precipitation filling some of the depressions, variability of slope, variability of geological conditions, etc. It is to be expected therefore, that part of the catchment (probably that part adjacent to the river channel) would yield an excess of water almost as soon as the supply of water to depression storage has started. As supply continues, a progressively larger part of the catchment would be expected to contribute water to overland flow and subsequently to the channel system. This complex process is therefore important in its effect on the rate of water supply to overland and channel flow, and is incorporated in the simulation model used in this study. The depression

storage component, and its mathematical representation is further discussed in Chapter 3.II.3.

2.II.4. Overland Flow. The process by which water flows across the surface of the land is termed 'overland flow', and the water entering the channel system via the land surface is termed 'surface runoff'. Overland flow occurs when the rate of precipitation exceeds the rate of inflow to depression storage, and is one of the processes by which water passes from the land phase to the channel phase. The overland flow component usually contributes the most rapid response of a channel system to an input of precipitation. The period of time between the occurrence of precipitation excess and the overland flow contribution to streamflow ranges from a few minutes to a few hours. In storms with relatively low rainfall intensity, however, the infiltration capacity of the soil may never be exceeded by the precipitation supply and overland flow will not occur. It should be noted that overland flow usually starts to contribute to the streamflow hydrograph in the area nearest to the river channel. This is a consequence of the low ground slopes in this zone, inhibiting the flow of water in the soil, and leading to saturation.

Overland flow usually occurs during periods of high intensity rainfall, when the infiltration rate is exceeded and surface depressions are filled. Consequently in temperate maritime climatic zones, within which this study was carried out, overland flow is not a frequent occurrence. The major exception to this observation occurs when impervious areas are present in the

catchment. These may take the form of roads, buildings, airstrips, etc. Any precipitation falling on impervious areas by-passes the infiltration component, reaching the overland flow component more rapidly than precipitation falling on bare soil or vegetation.

2.II.5. Soil Water and Interflow. Water entering the soil by infiltration is stored in the soil and may be lost by evapotranspiration, percolation or interflow (see Figure 2.1). Evapotranspiration losses are dominantly affected by the action of deep-rooted vegetation such as trees and bushes. Percolation is an extension of the infiltration process and occurs in the lower zones of the soil, transferring water vertically by gravity. It is much slower than infiltration because the medium through which the water passes is more densely compacted and often saturated.

Interflow is the process by which water is transferred laterally through the soil towards the channel system. The contribution of interflow to streamflow in the channel is considerably delayed compared with overland flow because the velocity of the water through the soil is relatively slow. The time lag between the supply of water to the interflow component and the contribution to streamflow ranges from a few hours to a few days.

The relationship between interflow and overland flow can be described as follows (Kirkby, 1969). Initially, water entering the soil passes downward into the lower zones, where it moves laterally to the channel system as interflow. As the supply from the surface proceeds and the sub-surface storages begin to fill, the boundary between water travelling downwards and water travelling laterally shifts upward. Eventually the boundary reaches the

surface and provided the local surface depressions are filled, overland flow occurs.

The relationship between interflow and ground-water flow is equally close, but generally the two processes are defined on the basis of the type of flow occurring. Interflow is unsaturated flow whereas groundwater flow occurs under saturated conditions. Clearly the two processes become indistinguishable under certain conditions when the passage of 'fronts' of moisture downward through the soil occurs during multiple storms. However the concept of interflow is useful in simplifying the complex reality of sub-surface hydrological processes.

2.II.6. Groundwater. Water which percolates from the soil into the underlying strata is known as groundwater. The groundwater storage component is usually identified separately from the soil water storage component because the medium through which the water flows is saturated and is beyond the reach of the roots of vegetation. Water is lost from groundwater storage in three ways : (1) by capillary rise to the soil above; (2) by deep percolation to inactive groundwater storage; (3) by horizontal groundwater flow to the channel system. The first two losses are usually very small compared with the third.

Groundwater flow to the channel system is the contribution to streamflow which suffers the longest delay between inflow to groundwater storage from soil water and subsequent inflow to the channel phase. The delay ranges from several days to several months and is therefore of little importance in flooding problems.

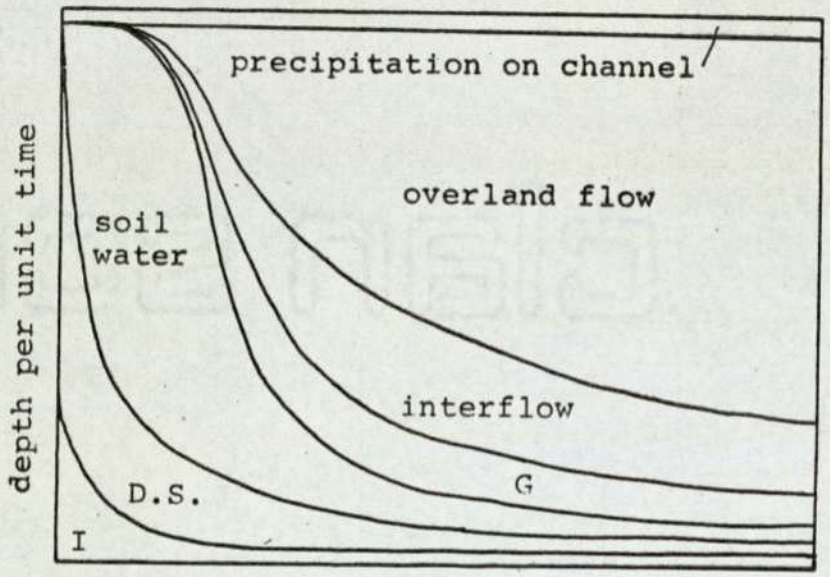
However, it is of major importance in catchments suffering from water pollution, where groundwater flow in the summer months helps to keep the concentration of pollutants low.

The processes acting in the land phase redistribute the incoming precipitation from the atmospheric phase. An example of the way in which storm rainfall can be redistributed by these processes is illustrated in Figure 2.3.

2.III THE CHANNEL PHASE

Water enters the channel phase from both the land and atmospheric phases. It is then transported through the network of river channels and lakes until it reaches the sea or is evaporated. Each of the various links in the channel network has a contributing area which supplies interflow and groundwater flow by lateral seepage through the banks and overland flow from the land surface. The process by which water is transported through the network is termed 'routing'. It has the effect of causing a change in shape of the hydrograph as the flood peak travels downstream due to the amount of storage available both in the channel system and on the flood plain. The inflow hydrograph is also translated in time due to the time taken for the water to travel through the network. The reduction in peak rate of discharge per unit area, and the 'spreading' of the recession curve is known as 'attenuation'. This effect is shown in Figure 2.4.

The mathematical representation of routing in the channel phase is discussed in Chapter 3.II.4, where the derivation of the



Key

I Interception

D.S. Depression Storage

G Groundwater

Figure 2.3 Hypothetical Distribution of Storm Rainfall through Time.

Source: Adapted from Linsley, Kohler and Paulhus, 1949, p.410.

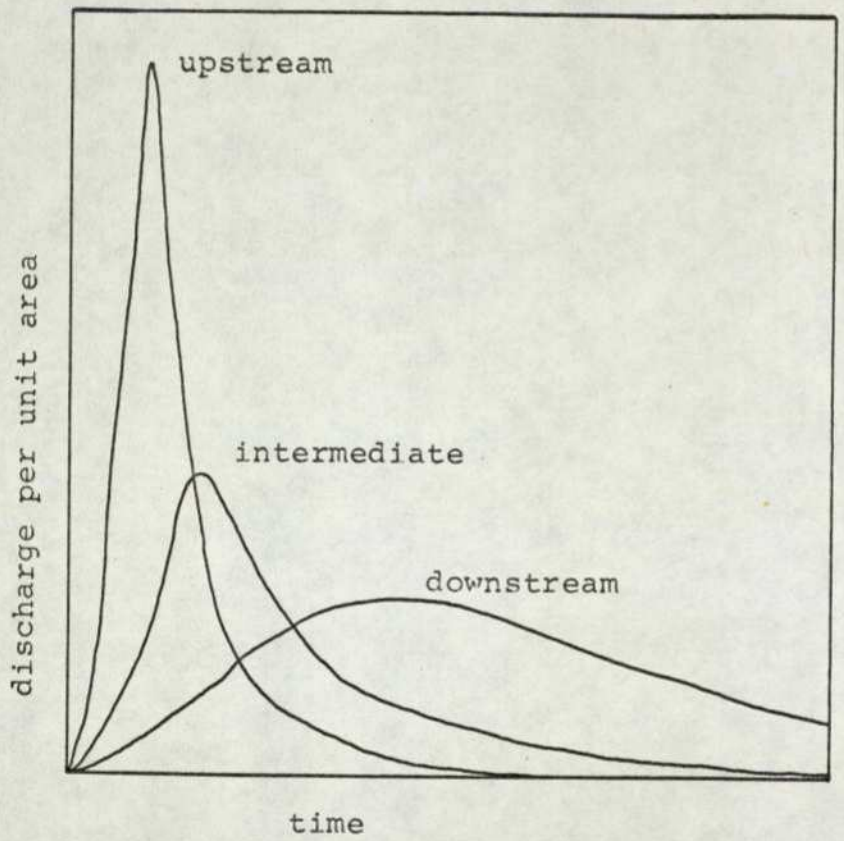


Figure 2.4

Attenuation Fffect of Catchment Storage.

equation used in the simulation model is presented.

2.IV MEASUREMENT OF HYDROLOGICAL PROCESSES

The collection of hydrological data is a prerequisite for the quantitative analysis of hydrological problems. Ibbitt (1971) distinguishes the following information as comprising a representative data set for a catchment :-

(a) precipitation

(b) meteorological conditions

(c) streamflow

(d) groundwater levels

(e) soil moisture contents

(f) surface storage variation

(g) discharge capacity of drainage

(h) quantitative inventory of cultural features

(i) economic data for future use

The first three items in the list are regarded as being essential in the study of flooding problems. Groundwater levels and soil moisture contents are considered to be desirable but not essential. Ibbitt also notes that the time interval used in recording hydrological information must be short when the data is to be used in flood studies.

In general, the size of the catchment to be studied and the purpose of the study dictate the amount and quality of data to be measured and collected. A small homogeneous catchment obviously will require precipitation data from a smaller number of stations than a large heterogeneous one, although the time interval will

probably need to be shorter. Similarly, a study of floods in a particular catchment will require data of a different type and quality than that required for a groundwater study. The testing and evaluation of a hydrological catchment model are therefore influenced by the measurement and collection of the basic hydrological data.

2.IV.1. Precipitation Measurement. There are two basic methods used for the measurement of precipitation. The 'standard' or 'storage' raingauge is the most common although the least useful method because it provides only scant information on the short-term distribution of precipitation, The second type is the 'autographic' or 'recording' raingauge, where the accumulated depth of rainfall is recorded continuously and displayed in graphical form. An alternative type of recording raingauge records the time at which a small receptacle is successively filled during a storm. This digital information can be stored easily on computer-compatible paper-tape or magnetic-tape.

The spatial density of precipitation measuring devices is an important consideration in the design of a data-collecting network. Intense convectional storms may be very small in area, and may go unrecorded in a sparsely covered catchment. This type of storm is also often difficult to record accurately because of insufficient time resolution on the recorder mechanism. It is usually very difficult to extract precipitation data from a recorder chart at less than five to fifteen minute intervals when the time scale on the chart is less than about one centimetre to the hour.

Synchronisation of recorders at different stations is important in flood studies but is difficult to achieve when chart scales are inadequate.

Other problems encountered with autographic equipment are : charts not changed; charts fitted incorrectly; instrument incorrectly calibrated; ink reservoir not refilled; clockwork mechanism not rewound. Rodda (1969) reviews the efficiency of various precipitation gauges. The most common problem in the long term is that of changing conditions surrounding the gauge, such as vegetation growth. This can usually be identified with double mass analysis (see Chapter 4.II.1.).

2.IV.2. Meteorological Conditions. Humidity, temperature, solar radiation and wind all affect evapotranspiration and thus determine the condition of a catchment prior to a storm. A storm occurring after a period with high levels of summer evapotranspiration will cause less runoff than a similar storm occurring in winter.

Evaporation is much less spatially variable than precipitation, and estimates derived from humidity temperature, radiation and wind-speed measurements may be applicable over a fairly wide area.

2.IV.3. Streamflow Measurement. The most useful way to measure streamflow is by automatic recorder. This is usually effected by means of a float connected to a chart pen, giving a continuous record of river level. Alternately, the river level can be sampled at fixed intervals. If a daily interval is used the information may be of little use and the method is less effective than the

similar method of measuring precipitation because there would be little evidence in the readings if the river experienced a rapid rise and fall between sampling times. However, river levels can be recorded automatically at much shorter intervals and may be stored and processed in a similar manner to automatic digital rainfall recorders. Naturally, automatic recorders are subject to similar errors and inadequacies as autographic rainfall recorders.

The readings of river level (stage) can be converted to streamflow per unit time (discharge) by reference to the 'rating curve' or stage-discharge relationship for the station. The establishment of a rating curve entails measuring the discharge in the river, using a current meter and measuring apparatus, for as wide a variety of river levels as possible. Accuracy can usually be claimed for the middle portions of the curve, but measurements of river level and discharge at times of flood may be sparse, leading to an ill-defined curve for high flows. This is complicated by the fact that the cross-section of the river often changes markedly at high levels when it is in flood, so that extrapolation of the rating curve may lead to serious errors.

A major source of error in using a rating curve to obtain discharge from stage is sedimentation or erosion of the stream channel. This can cause a rating curve to become obsolete, and as nearly all channel cross sections exhibit some variation with time, inaccuracies in the rating curve inevitably occur. It is therefore important to maintain the curve by taking frequent readings of stage and corresponding discharge.

Another source of error is caused by channel storage and the variation in surface slope (Wilson, 1969). This causes stage to be lower for a given discharge when the river level is rising and higher when the level is falling. The rating curve is therefore in the form of a loop, which is usually approximated by a straight line.

2.IV.4. Hydrological Data Series. Having discussed briefly the measurement and errors encountered in hydrological data, it is stressed that, for most studies, the period of data collection should be as long as possible. This is necessary for the study of both the deterministic and stochastic aspects of hydrological processes. A long record enables the non-stationary component to be separated from the stationary component of a series.

In a daily or monthly hydrological series, the non-stationary component is usually seasonal, super-imposed on a long-term trend which may be the result of influences such as man's activity. Yearly series, such as annual peak flow magnitude, may exhibit a marked trend, and this can be identified by means of such techniques as regression analysis, moving average analysis and double mass analysis. The subtraction of the long-term trend from the original data series leaves the seasonal or cyclic component, which may be identified by harmonic analysis, and the residual series, which may be represented by a simple stochastic model. A long period of record is therefore necessary to estimate the statistical characteristics of the residual series with a reasonable level of confidence.

2.V. THE INFLUENCE OF MAN'S ACTIVITIES

Man's activities can have a profound influence on the hydrology of the catchment, affecting both the quantity and quality of streamflow (Moore and Morgan, 1969, Preface). These activities are often broadly divided into urban and rural activities (United Nations, 1973), which may have intentional and unintentional effects on catchment hydrology.

2.V.1. Man's Influence in the Atmospheric Phase. The processes of evaporation and precipitation can be affected by man's activities in a number of ways. Man's rural activities, such as forest clearance, crop changes, and vegetation burning, can change evaporation rates and precipitation. Changes in evaporation are due mainly to different transpiration rates for different plants. Changes in precipitation are more hypothetical as little research has been carried out in this field, but it is to be expected that the increase in particulates in the atmosphere (as a result of fires and of wind erosion associated with 'prairie farming') provides extra condensation nuclei around which droplets of rain can form. Also, attempts have been made in rural areas to increase precipitation by seeding prospective rainclouds with silver iodide crystals, but these attempts have very local effects and are very few in number.

The effects of urban development are probably more marked than rural activities because they are highly localised, yet very numerous. Several studies have shown an increase in precipitation associated with urban areas (Chandler, 1965; Parry, 1956; Atkinson, 1968, 1969; Changnon, 1969), although the high spatial variability

of rainfall and the time-lag between the effect of towns and raindrop formation makes such studies difficult.

An urban area can have four main effects on the precipitation processes in the atmospheric phase (Chandler, pers.comm.) :

(1) The aerodynamic effect , due to increased turbulence associated with tall buildings;

(2) The thermal effect, due to heat effluent from factories, homes, etc.;

(3) The combustion effect, which puts moisture into the air;

(4) The addition of nuclei generated by the city, changing the processes of condensation and droplet formation.

Before the advent of the internal combustion engine, particles of carbon from the combustion of coal and wood probably contributed the most important influence on the atmospheric phase, but in the twentieth century the numerous motor vehicles inject hygroscopic nuclei (e.g. lead and iodine vapour) into the atmosphere, increasing the tendency for moisture droplets to form.

Quantitative studies have shown that precipitation is almost always increased around cities. Landsberg (1956) initiated studies in this field, and found that rainfall was increased by 10% in the cities he studied. Changnon (1969) presented results to show that precipitation can increase by up to 63% (see Table 2.1), although this increase for La Porte was thought to be rather anomalous. Similarly Chandler (1965) and Atkinson (1968, 1969) showed increases in precipitation associated with the conurbation of London, particularly in the incidence of summer thunderstorms. It would seem from these studies, that the size of the city and

	Chicago	La Porte	St Louis	Tulsa	Champagna-Urbana
Annual Precipitation	5	31	7	8	5
Summer Precipitation	4	30	-	5	4
Winter Precipitation	6	33	-	11	8
Thunderstorm Days	6	28	11	-	7
Summer Thunderstorm Days	13	63	21	-	17

Table 2.1 Effect of Urbanization on Precipitation (Percentage Increase over surrounding Rural Area).

Source: Changnon, 1969.

the weather type is important. Atkinson (pers.comm.) considered that urban areas must be larger than about 60,000 population to have a significant effect on precipitation totals, although small urban areas may trigger summer thunderstorms in certain circumstances.

2.V.2. Man's Influence in the Land Phase. Changes in land use are very important in affecting processes in the land phase. Forest clearance results in a reduction in the interception storage capacity with a consequent increase in the amount of water reaching the ground. Rainfall that was held in the crowns of trees and later evaporated is able to reach the ground and contribute to streamflow. This can be investigated by the hydrological analysis of an area before and after a major forest fire (Fleming, 1971), after major cutting activities (Hewlett and Hibbert, 1961; Patric and Reinhart, 1971) or after renewed planting (Law, 1957; Swank and Miner, 1968). It was estimated (Fleming,1971) that for a 30% devegetation of a catchment, either by cutting or fire, the amount of water yield increased by about 17.5% due to the decreased evapotranspiration and interception. However, if the vegetation is allowed to recover, interception storage capacity rapidly increases to its previous levels.

Crop changes may have an intentional effect on the hydrology of a catchment in order to manage the water resources, causing variations in water yield and distribution (Harrold, et.al. 1962; Jones, 1966; Hibbert, 1969). Black (1968) showed that abandonment of farms in Eastern New York resulted in an increase in streamflow. Irrigation and drainage of agricultural land and extraction for

water supply, also come under this category of intentional changes to the water regime.

Urban development has a more marked, though local effect on the hydrological processes of the land phase. Da Costa (1970) defines urbanization as the concentration of people in a relatively small area, providing them with houses, gardens, roads and drainage systems. It is clear from this definition that urbanization must have a considerable impact on processes in the land phase. As a result of the creation of areas that are virtually impervious to water, infiltration becomes slight and overland flow is greatly increased. Even pervious areas such as lawns and parks are compacted and experience a reduction in infiltration capacity (Lull and Sopper, 1969). Overland flow velocities increase due to the smoothness of the surfaces.

Many studies have been made of the effects of urban development in the land phase and most find that interception storage and infiltration are reduced and overland flow amounts and velocities are increased (James, 1965; Lull and Sopper, 1969). These studies are discussed in more detail in Chapter 2.V.4, with reference to the cumulative effects on streamflow.

2.V.3. Man's Influence in the Channel Phase. Changes made by man to the channel system include the construction of flood banks (levees) to protect arable or urban land on the floodplain, the clearance of vegetation from banks, the installation of field drains and sewers, the straightening of meanders, the building of dams, and the dredging and aggradation of the channel bed.

The two most important consequences of these activities are : (i) confining the flow, thereby increasing depth and velocity and decreasing flood-plain storage; (ii) decreasing the roughness of the channel, thereby increasing velocity of flow (da Costa, 1970). The installation of sewers and the straightening of channels in urban areas also tends to alter the configuration of the channel system by the addition of extra tributaries and the alteration of catchment boundaries.

2.V.4. Cumulative Effects of Man's Activities - A Summary. The streamflow record can be analysed in order to detect any changes in the characteristics of the record due to man's activities. These characteristics, some of which overlap, include (Bendat and Piersol, 1966):

(a) instantaneous peak flow magnitudes,

(b) mean flow,

(c) flow volume in storms,

(d) flow variance,

(e) flow range,

(f) number of exceedences of a given flow,

(g) autocorrelation properties, and recession characteristics,

(h) spectral properties

(i) cross correlation properties and time lag,

(j) yield of flow from a given amount of precipitation.

All these characteristics may be influenced by man in some way, but obviously some are economically more important to man than others. Therefore the investigation of problems concerning

these characteristics is more urgent. Flooding is probably the most pressing problem because flood damage is the main cost of man's interaction with rivers. Consequently, the analysis of peak flow magnitudes has been at the fore in most studies. Flow volumes and lag times between storm rainfall and storm runoff are also most important in this respect.

The benefits of man's interaction with rivers accrue mainly from the supply of water, transportation and the supply of power (hydro-electric and mechanical). Mean flow and flow variance are probably more important in these cases.

Most investigations into the effects of man's activities on streamflow therefore concentrate on these characteristics of the hydrometeorological records. As the research reported in this thesis was carried out in order to investigate the effects of urban development on streamflow, the following review is limited to work on this aspect of the problem.

2.VI. REVIEW OF LITERATURE ON EFFECTS OF URBAN DEVELOPMENT

Some of the effects of urban development on streamflow have long been appreciated (Task Force, 1969) and investigations into the factors involved were begun by Horner and Flynt (1934) with a study carried out in St.Louis, Missouri. Since then, many studies have contributed to the growing amount of data on and knowledge of the effects of urban development, although a large proportion of the studies are devoted to the U.S.A. It is now generally accepted that there is an increase in volume and peak flow due to urbanization, but estimates of the magnitude of this

effect differ greatly.

There are basically two approaches to the problem. The first approach is to study the hydrometeorological data from catchments with differing degrees of urban development. Differences in the statistical characteristics of the record may then be broken down into those attributable to catchment constants such as area, slope, vegetation and climate, and those attributable to urbanization.

The second approach is to study an urbanizing catchment through time and to detect changes in the record either by statistical analysis or by simulation (see Chapter 3.I) of streamflow. This is achieved by using parameters representing conditions before and after urbanization, and simulating two or more sequences of flow.

2.VI.1. The First Approach - Statistical studies of rural and urban catchments. This type of investigation requires that the conditions at each catchment be fairly constant during the period of study so that representative statistics may be calculated using the stationary data series.

Carter (1961) derived an equation to predict the mean annual flood for the built-up areas that he studied. The mean annual flood ($Q_{2.33}$) was a function ofcatchment area, lag-time and a coefficient of imperviousness, as follows :

$$Q_{2.33} = 230.K.A.^{0.85}.T^{-0.45} \qquad (i)$$

where A is the area in square miles

T is the lag between the centroid of precipitation contributing to the runoff and the centroid of runoff

K is related to percentage impervious area (I) by the formula : $K = \frac{0.3 + 0.0045.I}{0.3}$

The data used in the study was obtained from catchments in the United States.

Wiitala (1961) studied two adjacent catchments in Michigan, a rural one of 22.9 square miles and an urban one of 36.5 square miles. He found that lag times were 12 hours and 3 hours respectively, while the urban flood peaks were about three times larger than the rural peaks. Total flow volumes were about the same. Similarly Waananen (1961) studied streams on the Eastern Seaboard of the United States and in California, where he found that mean annual flows were about three to four times larger in the urban areas than in the rural areas. Total flow volumes were found to be doubled.

Also published in 1961 were the results of a study by the U.S. Geological Survey on Little Sugar Creek, North Carolina. With an impervious area of 15%, the stream was compared with surrounding rural streams and the differences in flood magnitude calculated (see Table 2.2.).

TABLE 2.2. Increases in Peak Flow due to urbanization on Little Sugar Creek, North Carolina

Mean Annual Flood	58%	increase
10 - year Flood	30%	"
20 - year Flood	17%	"

(source : U.S.Geological Survey, 1961)

These figures demonstrate that the relative effect of urban development decreases with increased flow magnitude.

In 1966, Espey, Morgan and Masch presented the results of a comprehensive study involving data from eleven rural catchments and 24 urbanized catchments. Unit hydrographs were derived from each of the 35 catchments and their characteristics were linked to catchment parameters by multiple linear regression analysis. The unit hydrograph characteristics used in the study are shown in Figure 2.5. The time of rise parameter (T_r) was obtained from catchment length, slope and impervious cover. This was then used as an independent variable in the prediction equation for peak flow (Q_{max}), which was of the form :

$$Q_{max} = c.A^n.T_r^m \qquad \text{(ii)}$$

where A is catchment area,

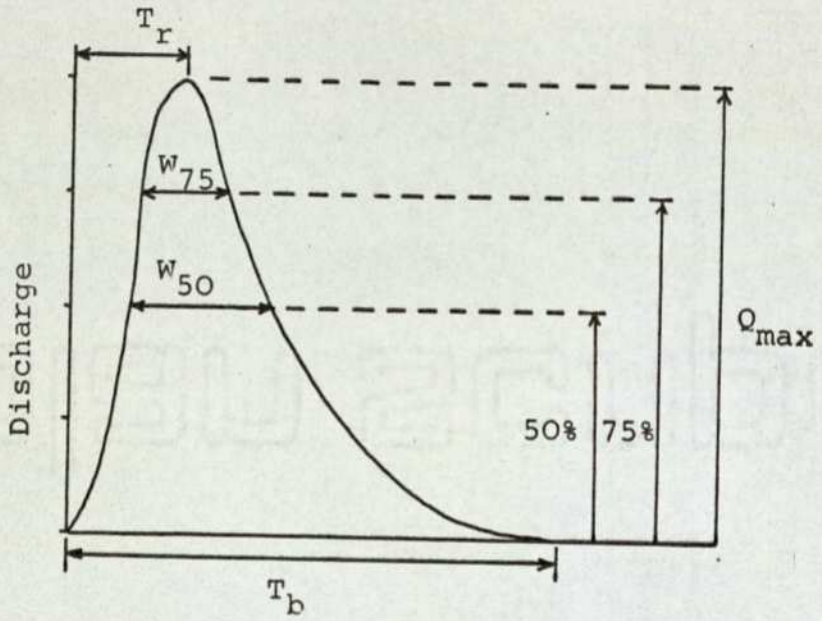
c, n and m are constants. The shape of the unit hydrograph could then be obtained by substituting catchment area and peak flow (obtained from the above equation) into the linkage equations, which were of the form :

$$UHC = c.A^n.Q_{max}^m \qquad \text{(iii)}$$

where UHC is the unit hydrograph characteristic (for example, hydrograph width at 50% peak flow)

c, n and m are constants (different from equation (ii)). The results of this study indicated that urbanization causes higher peak flows, occurring in shorter time intervals, with greater unit yields (flow per unit area).

Wilson (1967) compared three catchments in Mississippi, and



Q_{max} = Maximum Discharge

T_r = Time of Rise

T_b = Time length of Base

W_{50} = Hydrograph width at 50 per cent Peak Discharge

W_{75} = Hydrograph width at 75 per cent Peak Discharge

Figure 2.5 Hydrograph characteristics used by Espey, et al., 1966.

calculated flood frequency curves for each, based on 13 years of data. The mean annual flood on the urban catchment was found to be about 4.5 times that on the rural catchment, while the 50-year flood was about three times larger.

A study of the urbanization of forested catchments in the United States was made by Lull and Sopper (1969). They found that the streamflow was redistributed through the hydrological system as a result, with much more runoff coming off as overland flow (see Table 2.3).

Table 2.3. Flow and Storage (in inches) on three types of catchment

	Forest	Urbanized 50%	Urbanized 100%
Precipitation	40	40	40
Interception	5	3	2
Overland Flow	(0)	(18)	(36)
Soil Moisture	(6)	(3)	(2)
Evapotranspiration	15	12	2
Runoff	20	25	36

(brackets indicate temporary storage)

(source : Lull and Sopper, 1969)

Peak flows in an area with about 12% impervious cover were found to be nearly twice as large as those in rural areas nearby. They also found vast differences in sediment yield and water turbidity from various types of land use (see Table 2.4).

Anderson (1970) studied the effects of urban development on streamflow in Northern Virginia by using Carter's (1961) equations. Dimensionless frequency curves were derived by dividing flow by

Type of Land Use	Sediment Yield (tons/sq ml/yr)	Turbidity (p.p.m.)
Forest	50	45
Urban	50-100	90
Farm Buildings	1000-5000	4500
Building Site	25000-50000	45000

Table 2.4 Effects of Urbanization on Sediment Yield and Turbidity (in Eastern U.S.A.)

Source: Lull and Sopper, 1969.

mean annual flood so that flood frequencies of different catchments could be compared. The ratios found for rural catchments are shown in Table 2.5.

Table 2.5. Ratio of peak flow to mean Annual Flood for various recurrence intervals on rural catchments

	Recurrence Interval				
	2.33	10	25	50	100
Ratio:	1.0	2.2	3.3	4.4	5.5

(source : Anderson, 1970)

Sets of ratios were then derived for catchments with various degrees of urban development. The ratios read from these tables could then be substituted into an equation to predict flood peaks for any degree of urbanization and for any return period using :

$$Q_x = Q_{2.33}R = 230.K.R.A^{0.82}.T^{-0.48}$$

where Q_x is the peak flow for recurrence interval x

$Q_{2.33}$ is the mean annual peak flow (cfs)

R is the flood frequency ratio

K is the coefficient of imperviousness

A is the catchment area (square miles)

T is lag time.

The value of K was calculated from percentage impervious area (I) using :

$$K = 1.0 + 0.015.I$$

and the lag time for the equation was calculated from :

$$T = 4.64\ (L/S^{\frac{1}{2}})^{0.42} \quad \text{(for unsewered basins)}$$

$$\text{or } T = 0.55\ (L/S^{\frac{1}{2}})^{0.52} \quad \text{(for fully developed basins)}$$

where L is channel length and S is slope.

Narayana, et al (1970) derived equations to predict the peak rate and volume of runoff, and evaluated the relative effects of storm and catchment characteristics. Seventy catchments in the United States were included in the analysis, of which fifty were rural. Multiple linear regression was used to relate the dependent variables - peak runoff and total volume - to the independent variables. The latter were divided into catchment characteristics (area, slope, channel length, degree of channelization, impervious cover factor) and storm characteristics (duration, amount, maximum 30-minute intensity). The resulting equations were converted to nomogram form for rapid substitution of independent variables for prediction purposes.

It is noticeable that all the studies in the above category have been carried out in the United States. It should also be noted that the approach has its disadvantages. These are (1) that data from many catchments with differing degrees of urbanization must be obtained if a reliable prediction equation is to be produced; (2) that the record from each catchment must be stationary during the study period; and (3) the variance in streamflow must be separated into that attributable to urbanization and that attributable to other variables.

2.VI.2. The Second Approach - Statistical and Simulation Studies of an Urbanizing Catchment

Sawyer (1961) compared two adjacent streams, one of which had experienced urbanization over a period of ten years. The rural catchment was used as a control catchment to detect any changes in runoff due to factors other than urbanization, e.g.

precipitation changes. It was found that surface runoff increased by 123% in the urban catchment compared with 6-7% in the rural catchment.

Van Sickle (1962) studied a catchment in Texas which experienced steady urbanization between 1939 and 1961. He found that peak flow of unit hydrographs derived from successive periods of record increased by 200% to 500%. The derived unit hydrographs for six period of record are shown in Figure 2.6.

Increases in the volume of storm runoff from a stream in California were detected by Harris and Rantz (1964). The impervious cover had increased from 4% in 1945 to 19% in 1968.

Another unit hydrograph study was carried out by Crippen (1965) on an urbanizing catchment in California. He found that the peak flow from a particular storm type increased from 180 cfs to 250cfs in three years as a result of the urban development.

A simulation study was carried out on a catchment in Ohio (Curtis, et. al. 1964). A simple routing model was verified using existing streamflow data. The input to the model was then changed to represent channel inflow with urbanized conditions with the result that the 10-year flood peak experienced an increase of 50%. It was found that the effect of urbanization decreased with increasing flood magnitude.

A more sophisticated computer simulation model was used by James (1965) to investigate factors of importance in urban floods. The model used was the Stanford Watershed Model (Crawford and Linsley, 1962), which was calibrated using data from Morrison

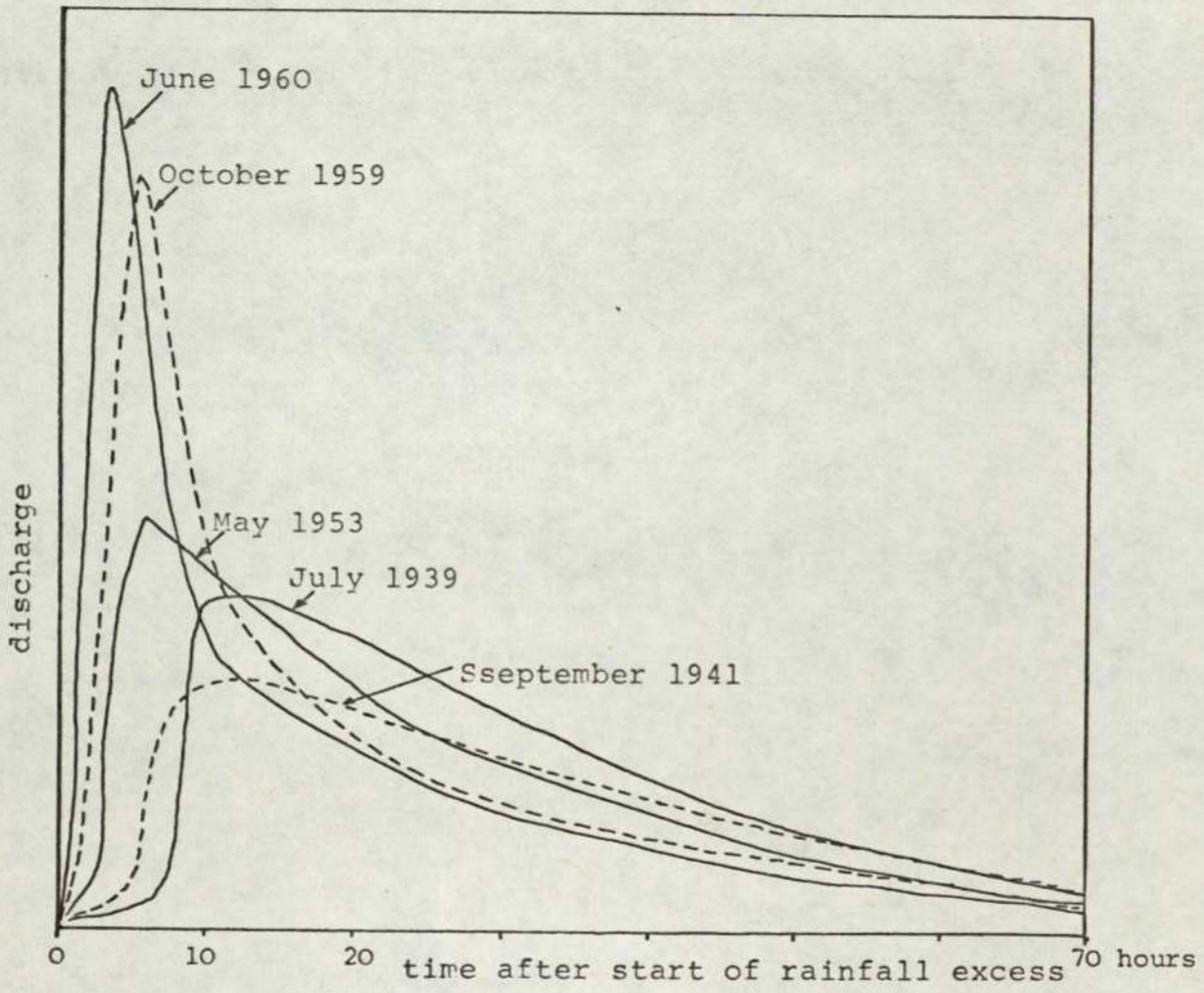


Figure 2.6 Changes in Unit Hydrographs due to Urbanization.

Source: Van Sickle, 1965.

Creek, California. The parameters representing urban development were then changed in increments to generate sets of hydrographs for various urbanized conditions. Graphs derived from these hydrographs were used to estimate peak flows for various areas, frequencies and degrees of urbanization.

Rantz (1971) used the graphs resulting from James's study to develop flood frequency curves for Californian streams for different degrees of urbanization. He also provided tables for use in conjunction with the 'rational formula' and for deriving unit hydrograph charactertistics for catchments with selected degrees of urban development. The effect of the position of an urbanized area of land was noted in this study. An urban area at the head of a catchment would have a greater effect than one at the downstream end becausd the urban and rural peaks would combine in the former case, whereas in the latter the urban peak would have passed before the arrival of the rural peak.

An analogue computer simulation model was used at the Utah Water Research Laboratory (Narayana, et al. 1969; Evelyn, et al. 1970) to synthesize hydrographs for Waller Creek, Texas. The model was calibrated using storms from periods of increasing urban development. The resulting changes in parameter values were then linked to urban development variables by regression analysis.

Seaburn (1969) used unit hydrographs to investigate the effects of urban development on Long Island, New York. He found that peak flows were increased by 50%. A double mass analysis showed that yield also increased.

The opportunity to study the effect of urban development on streamflow in the U.K. was provided by the building of new towns in the 1950's and 60's. Studies by Hollis (1970) of Canons Brook in Harlow, Essex were made using a computer simulation model. Daily flow sequences were derived for the catchment with parameters representing various levels of urban development. Flow duration curves were then obtained from these sequences, showing an increase in the frequency of high daily flows and a decrease in the frequency of low flows. Shaw and Hall (1972) derived unit hydrographs from storms occurring early in the development of Crawley, Sussex and from storms occurring at later stages. Peak flow of the unit hydrographs was increased by about 45% during the period.

A study of peak flows on streams in New Zealand by Howe and Waugh (1972) showed increases due to urbanization over a nine year period. The annual peaks and the number of exceedences of given levels of discharge all exhibited increases.

In addition to studies on water quantity, a few studies have been made on the effect of urbanization on water quality. The results of the study by Lull and Sopper have been mentioned earlier. Wolman and Schick (1967) noted massive increases in the amount of sediment in streams, particularly as a result of building construction. Anderson and McCall (1968) carried out a similar study in New Jersey and came to similar conclusions.

Periodically, separate reviews of work on the effect of urban development on streamflow and of various models used to simulate urban streamflow appear. These publications are useful in collating

the literature so that undue duplication of research does not occur. Reference may be made to the following papers to extend the range of literature on the subject, although it is hoped that no important works have been omitted from the above review :

Savini and Kammerer, 1961; Leopold, 1968; Task Force on Effect of Urban Development on Flood Discharges, 1969; Moore and Morgan, 1969; Rao et al., 1972; Sarma et al., 1973; Papadakis and Preul, 1973; United Nations, Food and Agricultural Organisation, 1973; Espey and Winslow, 1974; McPherson and Schneider, 1974.

Studies involving the simulation of urbanized catchments without reference to differences between urban and rural conditions are discussed in the following chapter.

CHAPTER THREE

DEVELOPMENT OF THE SIMULATION MODEL

The first part of this chapter reviews the types of models that may be used in hydrological investigations. The application of electronic computers is then discussed in relation to deterministic simulation models. Secondly, the structure, components and implementation of the hybrid computer simulation model developed to simulate the response of the catchment under study is presented. Finally, a review of techniques available for the optimization of the model parameters is given and the direct-search 'Simplex' method used in the study is described in detail.

3.I THE USE OF MODELS AND COMPUTERS IN HYDROLOGY

Hydrology involves the creation of models in order to reconstruct past occurrences or to envision future conditions (Dawdy and Thompson, 1966). The definition of the term 'model' in science is fairly loose, and is suitably defined by Skilling (1964) : 'A model can be a theory or a law or an hypothesis or a structured idea. It can be a role, a relation or an equation. It can be a synthesis of data.' In short, a model serves to represent reality so that it is more easily interpreted and understood.

3.I.1 Hydrological Models. The classification of hydrological models has been discussed exhaustively in recent literature, but a basic division is perhaps that between physical and conceptual

models (Chorley and Haggett, 1967, p.25).

Physical or hardware models are often used in the place of the real world because the hydrological phenomena under study can be controlled. Rainfall simulators operating over physical models of a catchment enable the study of individual mechanisms within the hydrological cycle to be carried out in semi-isolation (Thorpe, 1973). A catchment simulator was used by Thorpe to study the effects of the location of impermeable (urban) areas with respect to the catchment as a whole. He found that the peak rate of runoff is amplified when urban areas are located at the upstream end of a catchment.

Another type of physical model is the analogue model, where the properties of the hydrological system (e.g. flow of water) are represented by the properties of another system (e.g.flow of electricity). Electrical analogue models are often used in groundwater studies and have been used in the analysis of flood runoff (Shen, 1965; Harder Mockros and Nishizaki, 1960). Analogue computers are an extension of this concept and are discussed separately in Chapter 3.1.2.

The major drawback with hardware models is their expense. They are both expensive to construct and expensive to alter. The application of a hardware model to the investigation of the effects of changing levels of urbanization on a particular, rather than idealized, catchment was therefore thought to be unfeasible.

Conceptual models form a much larger class of models. They can be broken down into ideographic models and mathematical models. Ideographic models often form the basis upon which mathematical

models are built. Indeed, it could be argued that ideographic models (e.g. the idea of a 'hydrological cycle') always precede mathematical models (e.g. the Standard Watershed Model of the hydrological cycle). Mathematical models are of more practical use in hydrology because they enable reality to be represented quantitatively.

Dawdy (1969) recognises that mathematical models can be classified in many different ways, but that a realistic division in hydrology is between deterministic and stochastic models. This division is by far the most common, although equally valid divisions are 'analytic'/'synthetic' (Dawdy, 1969), 'analytic'/'parametric' (Nash and Sutcliffe, 1970), 'general systems'/'physical hydrology' (Amorocho and Hart, 1964), 'deterministic'/'statistical'/'systems' (Fleming, in press) and 'black box'/'white box' or 'bulk parameter'/'distributed parameter' (Dawdy, 1969).

It is not intended to discuss the significance or validity of these various terms because, in many cases, a model will fit into whichever class the user wishes. Dooge (1972) points to the fact that many apparently diverse models have strong underlying similarities.

The type of problem to be solved usually dictates the class of model to be used (Dooge, 1972). The degree of interest in the status of the hydrological variables in the model, for example, dictates whether a black box (no interest) or a white box model is chosen. Similarly a stochastic model is rarely used to forecast streamflow which may occur a few hours in the future, but

it can give an idea of the error associated with such a forecast by a deterministic model. In short, the model should suit the problem. One of the purposes of the research into the effect of the urban development of Crawley reported in this thesis was to reconstruct past flood events and to ascertain how the catchment changes affected the hydrological system generating these floods. It was therefore decided that a deterministic type of model should be used. Modelling philosophy is further discussed in the Addendum.

Hydrological variables exhibit both deterministic and stochastic components. This is because the laws which govern the complete hydrological system are so numerous and complex that it is impossible to form a set of equations which would exactly predict the form of a hydrograph from precipitation and other measured data (Dawdy and O'Donnell, 1965). Thus when deterministic reconstructions of hydrographs are attempted, stochastic or random components remain. Using a deterministic model, deviations in observed phenomena which cannot be represented by the model equations are deemed to be 'chance' elements and as well as being the result of the omission of certain processes in the model, they may also be due to errors in the data. The magnitude of these deviations is often used as a measure of the accuracy or fidelity of the model. If, however, it is required to reproduce what is as equally likely to occur as the actual record, the probability distribution of the chance elements can be represented by a stochastic model (Bloomer and Sexton, 1972). A stochastic model is therefore useful in generating long sequences of synthetic flows which can, for example, be used in the design

of reservoirs.

Dawdy's (1969) division into analytic and synthetic models is an important one in deterministic modelling, but the complexity of the hydrological system makes analytic models of complete catchment behaviour difficult to use successfully. The unit hydrograph concept is probably the most widely used model in this class because the simplicity of its use makes it popular. However, its derivation is much more difficult, and a large degree of subjectivity is inherent in defining the distribution of rainfall excess (Rantz, 1971, p.55) from which the unit hydrograph is partially derived. Several examples of the use of unit hydrographs were mentioned in the previous chapter, but because the parameters (e.g. the moments of the unit hydrograph (Nash, 1960)) are not physically meaningful it was decided that this type of model was not suited to the aims of the present research.

3.I.2. Simulation. Synthetic or simulation models, which represent each process in the hydrological system by a set of equations, permit more realistic representations of the catchment. However, until computers became widely available in the early 1960's, the tedium and expense of simulation was very great. One of the first overall catchment models took 34,000 man hours to complete, while the same amount of work performed today on a computer would take a few minutes (Dawdy and Thompson, 1966). Catchment simulation models are therefore designed around the capabilities of the computer, and are often only fully defined in terms of a set of statements in a computer programme.

There are two basic ways to approach the formulation of a catchment simulation model. The first is to develop the model by making use of the known physical laws governing the system (Nash and Sutcliffe, 1970) which are usually the laws of continuity and momentum. This type of model is often referred to as a true deterministic simulation model, as distinct from models of the second approach, based on empiricism, and termed parametric models. However, it could be argued that the division is not a meaningful one in that all 'known physical laws', according to the philosopher Karl Popper, are provisional(Magee, 1973, p.26). Most overall catchment models have 'known physical laws' embedded in them, usually in the routing component.

A simulation model is built up from which are thought to be the most relevant components of the hydrological system (O'Donnell, 1966). Whether a component is relevant or not depends on the problem under study. For example, models simulating urban floods often have relatively simple moisture accounting procedures and groundwater components but have complex routing procedures - gutter flow, overland flow, pipe flow and channel flow being given separate consideration (Chen and Shubinski, 1971; Tholin and Kiefer, 1960).

A cautionary note is sounded by Dawdy (1969) in emphasising that although relevant processes should be included, processes which are known intuitively to exist, but for which there are little or no equations and data, should be avoided. However, good results have been achieved by complex models simulating conjectural or little documented processes. It could therefore

be argued that guessing at the relationships involved in a component of the hydrological cycle (e.g. interception) is more likely to give good results than excluding the process from the model on the grounds that the relevant relationships are unknown. Ibbitt (1970, p.203) found that although some components in the Stanford Watershed Model were perhaps redundant, the model reproduced hydrographs remarkably accurately. However, he did point out that this could be due to the extra parameters in the model allowing a better fit than the O'Donnell model with few parameters, and that these extra parameters may have greatly different values if the model were fitted to another set of data.

Advocating the use of computer simulation, Dawdy, Lichty and Bergmann (1972) indicate the practical uses of the technique : rainfall records can be used to create or add to streamflow records; verified parameters may be transferred to ungauged sites; if man-made changes can be related to model parameter changes, 'before' can be compared with 'after'.

Therefore it would seem that computer simulation techniques are certainly the most rational approach to the study of the flood hydrology of a catchment (Rantz, 1971, p.56).

3.I.3. Application of computers to simulation. The simulation of the processes in the hydrological cycle would not be practical without using electronic computers. To make use of a computer, the problem must be presented in a form compatible for computer solution.

The two basic types of computer available are termed digital and analogue computers, after the way in which the variables are

manipulated. In digital machines, the variables are in discrete form and solution times are relatively long, whereas in analogue machines the variables are in continuous form and solution of the system equations is almost instantaneous, regardless of the complexity of the model (Bekey and Karplus, 1968). A digital computer programme consists of a series of statements, usually in a 'high-level' language such as FORTRAN and PL/1, which must be 'compiled' by translating the statements into 'machine language'. Execution of the programme then proceeds in a series of steps and the results are usually printed at the end of execution. The programme statements are stored on punched cards or paper tape.

An analogue computer programme consists of an electrical circuit, with various components performing the functions of integration, addition, multiplication, etc. The components are arranged in modular form behind a 'patch panel' and may be connected with wire loops to form the required circuits. The programme is executed by putting the analogue computer into the 'initial condition' mode and then into the 'operate' mode for the duration of the simulation period. Timing may be controlled by logic elements. Setting up the analogue computer programme for execution is more laborious than setting up a digital programme, which only requires the submission of a batch of cards.

The analogue computer is highly suited to the simulation of systems characterised by differential equations as they can be solved continuously and easily. Processes simulated may be selectively observed on an oscilloscope by connecting leads

from the appropriate analogue components, and parameters may be changed on the basis of these observations. Subjective parameter optimization may therefore be achieved much more easily than on a digital computer, and valuable insight into the interaction of the various processes may be gained. On the other hand, digital computers may be used to implement automatic optimization procedures. Another advantage in using digital computers for simulation is that any desired level of accuracy within the capabilities of the computer may be specified. This can be up to sixteen decimal places, whereas analogue computers are subject to random errors when an accuracy of more than three or four decimal places is required.

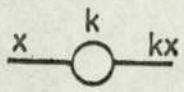
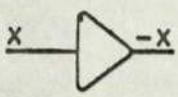
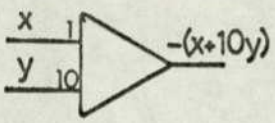
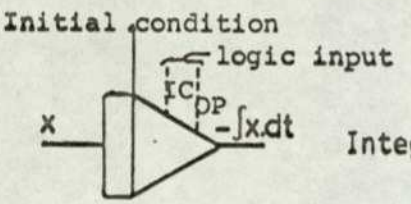
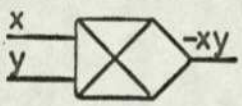
It can be appreciated that each type of computer has its advantages and disadvantages. However, the combination of an analogue computer to perform the simulation and a digital computer to control the simulation and optimize the parameters yields a valuable tool. It was therefore decided to utilize a hybrid computer to simulate the hydrological system of the urbanizing catchment under study. The various components of a hybrid computer are shown in Figure 3.1. The use of these components in the simulation of the hydrological system under study is discussed in the following section.

3.II THE HYBRID COMPUTER SIMULATION MODEL.

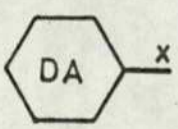
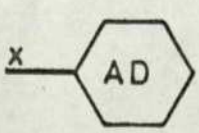
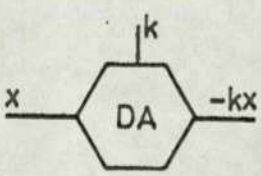
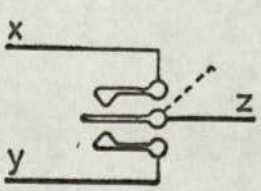
This section describes the structure of the simulation model used in the study, and the development of the equations representing

ANALOGUE COMPONENTS

(Note, Analogue signals shown : ——— , logic signals: ---)

Symbol	Name	Purpose
	Potentiometer	Multiplication by constant k
	Inverter	Inversion of variable x
	Summer	Summation of x and y
	Integrator	Integration of variable x
	Multiplier	Multiplication of x and y

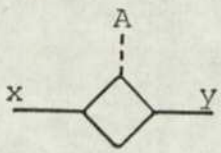
INTERFACE COMPONENTS

	Digital to Analogue Converter
	Analogue to Digital Converter
	Digtal to Analogue Multiplier (variable x multiplied by digital value k)
	Relay (digitally controlled analogue switch)

Contd. overleaf

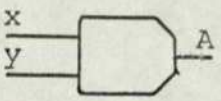
Figure 3.1 Analogue Computer and Interface Components.

INTERFACE COMPONENTS (Contd.)



Electronic Switch

A	0	1
y	0	x

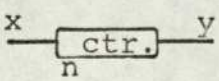
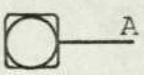
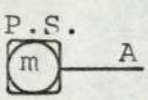
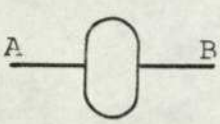
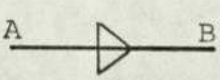
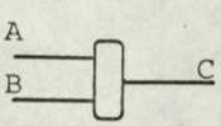
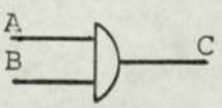


Comparator

x+y	>0	<0
A	1	0

LOGIC COMPONENTS

(Note, logic signals shown as solid lines)

Symbol	Name	Purpose
	BCD Counter	If x has m pulses per sec., y has m/n pulses per sec.
	Terminal on Logic Panel	
	Pulse Source	Emits m pulses per second.
	Flip-flop	When A becomes '1', B changes state.
	Inverter	$B = \overline{A}$
	OR Gate	C = A .OR. B
	AND Gate	C = A .AND. B

(Figure 3.1 continued)

the components. The implementation of the model using a hybrid computer and the operation of the model using hydrological data is also discussed.

3.II.1. The Structure of the Model. The model used in this study was developed so that the internal processes and parameters have physical meaning. As with most models developed to simulate the response of a catchment, it is a partial-systems model in that processes and information are selected from the total available information.

A simulation model is structured in that its component parts are selected and connected in a definite pattern (Skilling, 1964). The hydrological processes to be simulated have been selected in order both to simplify the system and to minimise the number of parameters. These criteria are stressed by Dawdy (1969) as being desirable in the modelling of hydrological systems. At the same time it is hoped that despite simplification, the components are physically meaningful.

In an investigation of urban streamflow the part of the hydrograph representing the short term response to precipitation is most important. This part is generally regarded as being composed of surface runoff and interflow, and the hybrid computer simulation model simulates only this part of the hydrological system. Sub-surface processes have been ignored because they usually only affect the hydrograph long after the main peak has passed. Evaporation during the simulation period is also neglected because it is considered to be very small.

The first component of the model (see Figure 3.2) is infiltration, which has a great impact on surface runoff, especially during the initial part of the storm when a large amount of rainfall may infiltrate the soil and not contribute to surface runoff. Rainfall that does not infiltrate is subject to the second component of the model - depression storage. Excess water from this component, which becomes increasingly available as the surface depressions are filled, is routed through the third component of the model - a linear reservoir.

3.II.2. The Infiltration Component. The general requirements for an infiltration function for use in a simulation model are : (1) it must be dependent on a variable rate of rainfall and not on time alone; (2) the number of parameters must be minimised; and (3) it must be easily solvable.

The first requirement is necessary in the simulation of naturally occurring storms because the rate of rainfall is never uniform and is often highly variable, with perhaps short breaks between bursts of rain. Many infiltration functions have been formulated on the basis of laboratory experiments with a constant rate of rainfall supply, and therefore would not apply to natural conditions. The infiltration function for the simulation model must be a general one because such measures as sorptivity only apply to small plots, and the catchment to be simulated is fairly large. The function must also be capable of handling data at 15-minute intervals.

Experiments have shown that, with a fixed rate of supply,

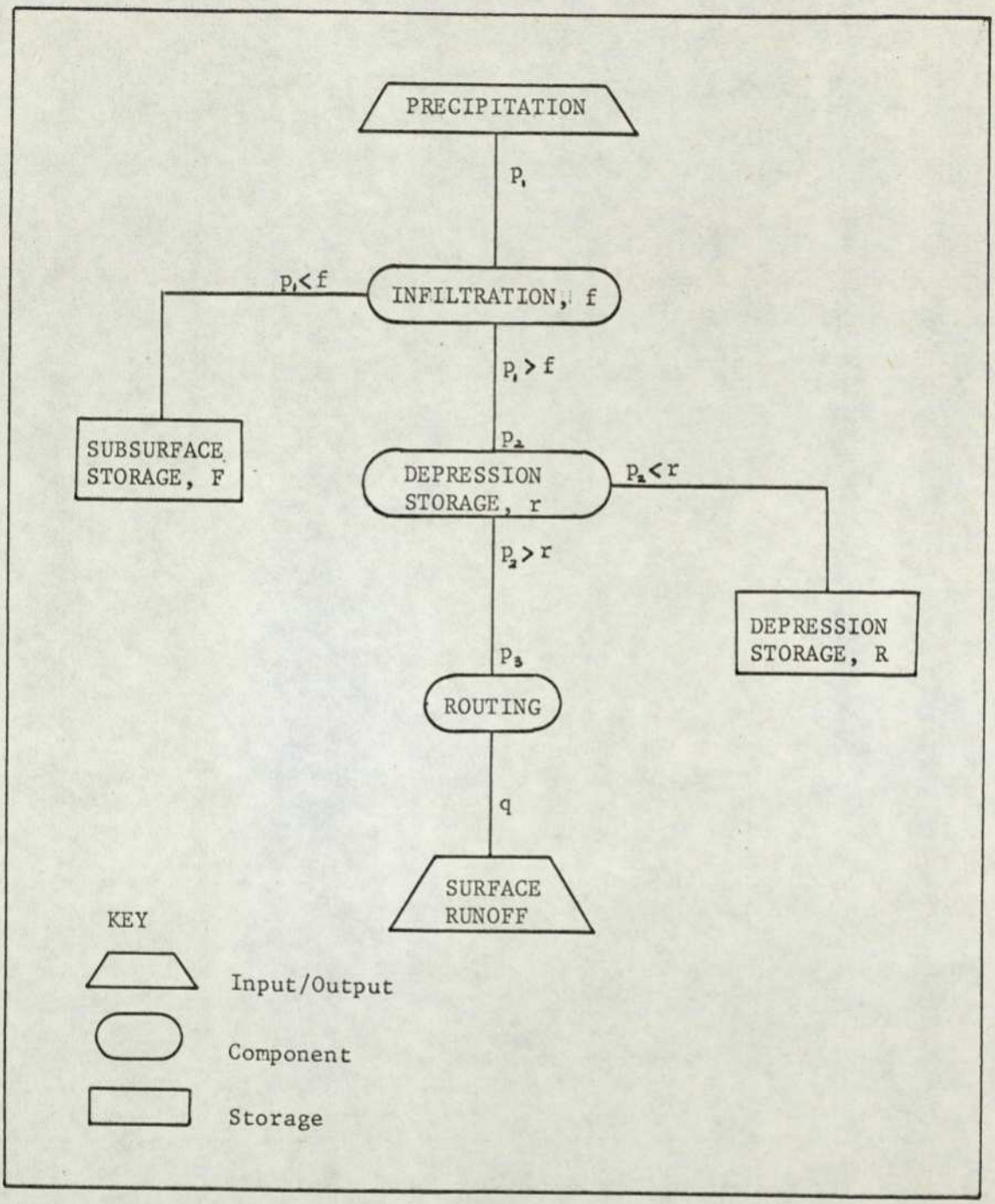


Figure 3.2 Structure of the Simulation Model

infiltration into the soil exhibits an asymptotic approach to a constant value. Rubin (1966) discusses the theoretical aspects of the infiltration process. He recognises three types of infiltration :

1. non-ponding
2. incipient ponding
3. ponded.

The first two types have the infiltration rate controlled by rainfall intensity whereas in the third type, it is controlled by water depth. The equations given by Rubin are all partial differential equations and are, as noted by Dunin (1972), rather tedious to solve. Information is required on such variables as hydraulic conductivity, diffusivity and soil moisture content, and although this may be feasible on a small experimental plot, it is not suitable for the simulation of a catchment with highly variable soil characteristics.

Simpler empirical equations have been developed to represent the infiltration process with respect to only one independent variable. Horton's equation is perhaps the most commonly known. It relates the infiltration capacity rate (f) to time (t) :

$$f = f_c + (f_o - f_c)\, e^{-k_f \cdot t} \qquad (3.i)$$

where f_o = initial infiltration capacity when t = o

f_c = infiltration capacity when t is large,

k_f = an empirical constant

This equation assumes a rate of supply at the surface in excess

of the infiltration rate. Philip (1957) notes that Horton's equation is unrealistic when t is small because f_o is not high enough, but it could be argued that this is only important when the time increment is small, because the rate of infiltration very rapidly decays to a more moderate level. Philip also notes that another equation relating infiltration rate to time, that of Kostiakov, is unsatisfactory because the parameters in the equation vary with time. Kostiakov's equation is of the form :

$$f = a.k.t.^{a-1} \qquad (3.ii)$$

where a and k are parameters.

Philip developed an equation which was claimed to correct the faults of the above equation :

$$f = \frac{1}{2}. st^{-\frac{1}{2}} + f_c \qquad (3.iii)$$

where s = a parameter.

This equation was found to be rather unsuitable for solution on the hybrid computer model because of instability problems at low values of t.

Crawford and Linsley (1966) developed a function to represent mean infiltration capacity in the catchment at any time t by the following expression :

$$f = A \left(\frac{F}{k}\right)^n \qquad (3.iv)$$

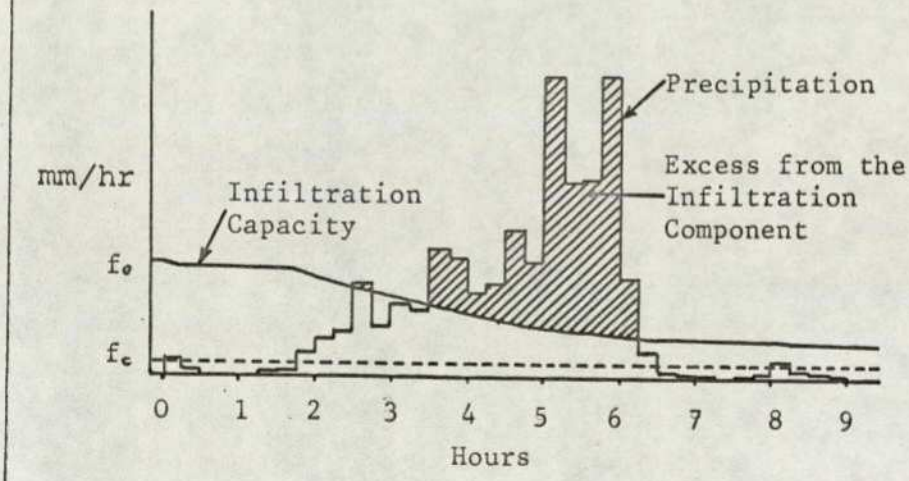
where F = soil moisture content,

A, k and n are parameters (n = 2)

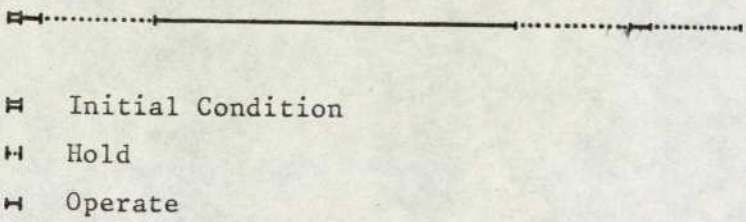
This equation was found by Crawford and Linsley to be very reliable for simulating infiltration throughout the year, but was thought to be too complex for a model simulating single storms.

The equation chosen for the hybrid computer simulation model was a modified form of Horton's equation. This equation is exponential in form and is thus easily solvable on an analogue computer. With a 'lumped parameter' model, an equation requiring measured soil characteristics is unjustified and as such the Horton equation is acceptable, but the main disadvantage of the function is that it is dependent on time and not on soil moisture. This is not a serious problem in the simulation of single storms because soil moisture usually experiences a fairly steady saturation with time and drying by evapotranspiration is not important during the relatively short time span of the simulation. However, it was decided to make the infiltration capacity partially dependent on precipitation rate. This is achieved by delaying the start of the exponential decay until the minimum infiltration capacity (f_c) is exceeded by the precipitation rate. If the precipitation rate later decreases below the minimum infiltration capacity, the infiltration capacity is held constant until precipitation again reaches a sufficiently high rate. This process is shown in Figure 3.3.

The analogue circuit for generating this function is shown in Figure 3.4 together with the logic elements for testing the relative magnitudes of precipitation rate and infiltration capacity. Integrator 95 generates the exponential decay and is put into the 'operate' mode whenever the precipitation rate



(a) Variation in infiltration capacity with time.



(b) Mode of Integrator 95 (representing infiltration capacity) during simulation

Figure 3.3 Function of the Infiltration Component.

exceeds the minimum infiltration rate. This is tested by comparator 64 (see Figure 3.4 for the analogue circuit and Figure 3.12 for the logic circuit) which compares the values of the two variables. The analogue circuit was derived as follows :

$$f = f_c + (f_o - f_c)\, e^{-k_f t} \qquad (3.v)$$

$$\text{let } f = f_c + y \qquad (3.vi)$$

$$\text{differentiating y, } \frac{dy}{dt} = -k_f (f_o - f_c)\, e^{-k_f t} \qquad (3.vii)$$

$$= -k_f . y \qquad (3.viii)$$

Thus, if the output of integrator 95 is multiplied by the constant $-k_f$, it forms the first derivative of y. This can be fed into the integrator to generate the exponential decay part of the function, which is represented by y. It is then necessary to input the initial condition of the function, which is represented by $(f_o - f_c)$ in Equation 3.v, into the integrator using potentiometer 95. The output from the integrator (y) is then added to f_c to form the complete function using summer 66. It should be noted that the use of an integrator or summer always causes a sign inversion of the input.

In order to derive the excess moisture from the infiltration function, the infiltration rate must be subtracted from precipitation. Amplifier 36 is used for this and obviously, if this difference is negative, rainfall excess is zero, i.e.

$$\text{Excess } p_2 = p_1 - f \quad \text{when } p_1 > f \qquad (3.ix)$$

$$\text{Excess } p_2 = o \quad \text{when } p_1 < f \qquad (3.x)$$

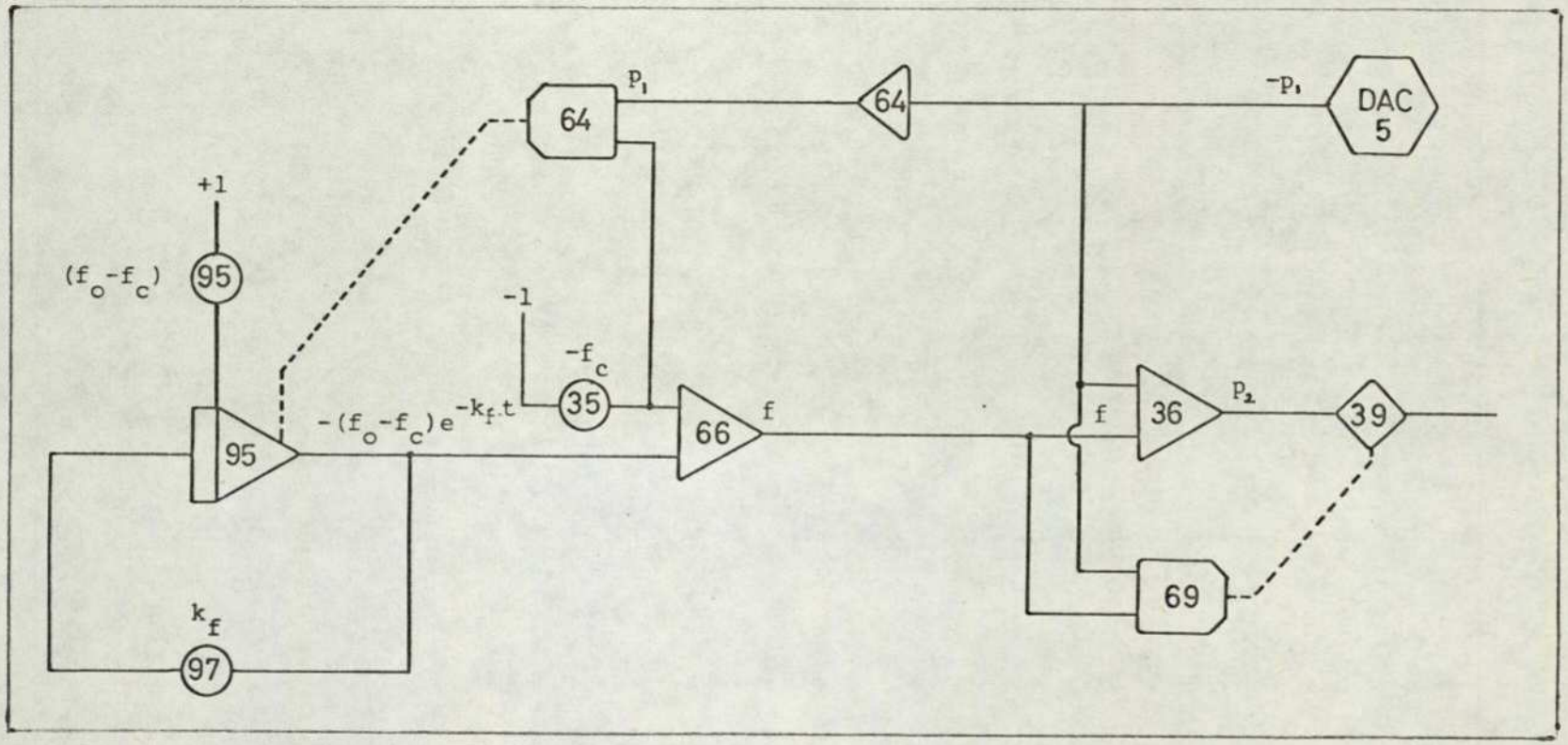


Figure 3.4 Analogue Circuit for the Infiltration Component.

This condition is tested by comparator 69, comparing the values of precipitation and infiltration. If the difference is positive, switch 39 is closed.

The effect of this infiltration function on the rate of excess water from the infiltration component of the model was shown in Figure 2.2.

3.II.3. The Depression Storage Component. This component can be viewed as representing the flow of water into the many small depressions which cover the surface of the catchment. As the surface depressions fill, overland flow commences and continues at an ever increasing rate as long as the supply of excess water is maintained. The rate of inflow into the depressions decreases as the available volume is filled and is therefore a function of available depression storage capacity. The net rate of inflow decreases and, assuming no evaporation, asymptotically approaches zero as the depressions are filled and overflow. In this context, the net rate is theinflow minus infiltration loss. All precipitation not infiltrating the soil therefore becomes surface runoff when depression storage is full. This can be represented conceptually by an exponential function such that the volume of water in depression storage (R) increases as the volume of excess from the infiltration component (P_2) increases (Linsley, Kohler and Paulhus, 1958). This is represented by the following equation :

$$R = D - D.e^{-k_d P_2} \qquad (3.xi)$$

where D = available depression storage on a dry catchment

k_d = a constant related to depression storage capacity.

Given a constant rate of excess from the infiltration component (p_2), the volume of water in depression storage exhibits the change shown in Figure 3.5.

In order to keep the number of parameters as small as possible the decay parameter k_d was eliminated by equating it with the inverse of the depression storage capacity (D). This means that when depression storage capacity is small, k_d is large and the storage fills up quickly. This is shown in Figure 3.6, with the converse case of a large depression storage capacity.

Thus $$R = D - D.e^{-\frac{1}{D}.P_2} \qquad (3.xii)$$

To derive the rate of inflow to depression storage (r) this equation is differentiated with respect to time :

$$r = \frac{dR}{dt} = \frac{dP_2}{dt} \cdot \frac{dR}{dP_2} \qquad (3.xiii)$$

where dP_2/dt = rate of rainfall excess (p_2) from the infiltration component.

Differentiating Equation 3.xii :

$$\frac{dR}{dP_2} = \left(\frac{-1}{D}\right) . \left(-De^{-\frac{1}{D}.P_2}\right) \qquad (3.xiv)$$

$$= e^{-\frac{1}{D}.P_2} \qquad (3.xv)$$

Combining Equations 3.xiii and 3.xv yields :

$$r = p_2 . e^{-\frac{1}{D}.P_2} \qquad (3.xvi)$$

This variable represents inflow to depression storage and is subtracted from the excess water from the infiltration

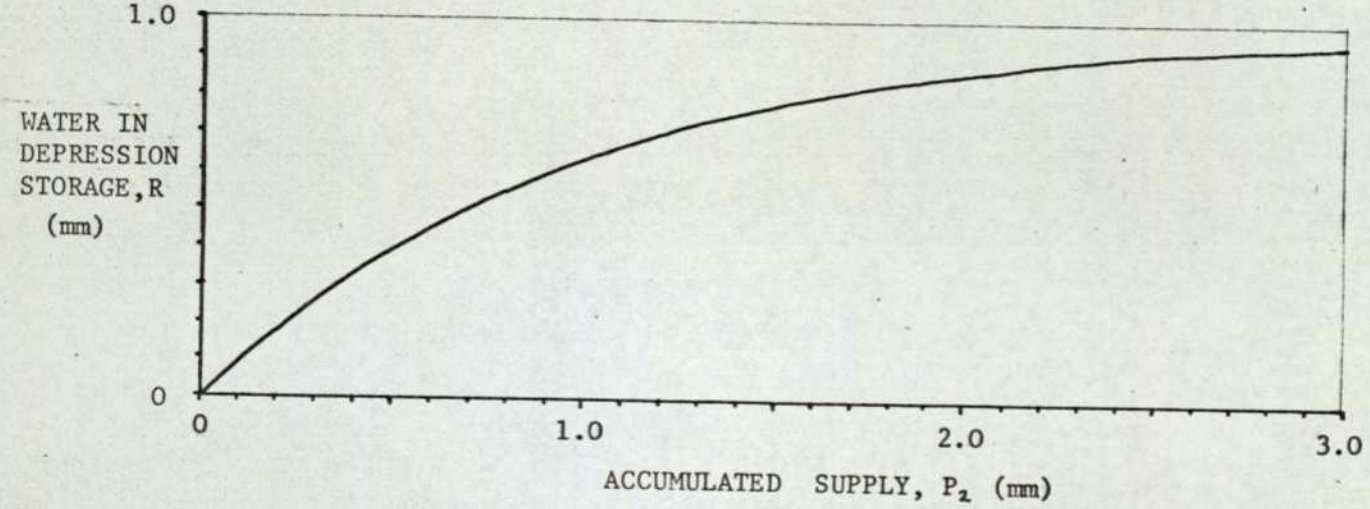


Figure 3.5 Inflow to Depression Storage.

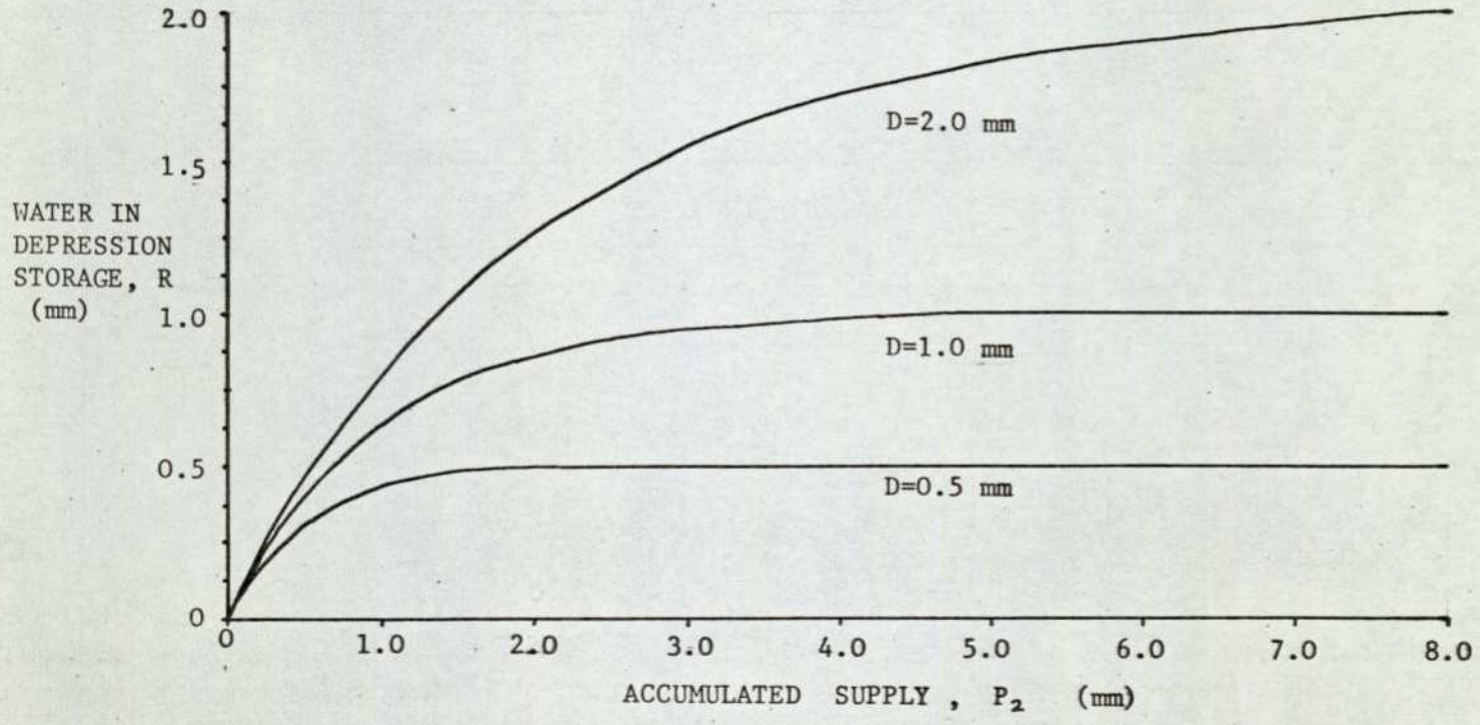


Figure 3.6 Effect of Different Depression Storage Capacities.

component to yield excess from the depression storage component (p_3) :

$$p_3 = p_2 - p_2 e^{-1/D \cdot P_2} \qquad (3.xvii)$$

$$= p_2 (1 - e^{-1/D \cdot P_2}) \qquad (3.xviii)$$

The analogue computer circuit for generating this function is shown in Figure 3.7(b). The function represented by Equation 3.xvi is generated in two stages. First the exponential part of the equation is generated using Integrator 70. This is then multiplied by the excess from the infiltration component (p_2) to yield the inflow to depression storage (r) as follows :

$$\text{from Equation 3.xiv, let } y = e^{-1/D \cdot P_2} \qquad (3.xix)$$

$$\frac{dy}{dt} = \frac{-1}{D} \cdot e^{-1/D \cdot P_2} \cdot \frac{dP_2}{dt} \qquad (3.xxi)$$

$$= \frac{-1}{D} \cdot y \cdot \frac{dP_2}{dt}$$

$$= \frac{-1}{D} \cdot r \qquad (3.xxii)$$

Therefore if the first derivative of y (as given by Equation 3.xxi) is fed into Integrator 70, the output will be y. This is then multiplied by the excess from the infiltration component (p_2) to give inflow to depression storage (r) from Multiplier 38. This can then be subtracted from the excess from the infiltration component using amplifier 3, to yield excess from depression storage. The output (r) from the multiplier must also be multiplied by the parameter 1/D to give input to the integrator as given by

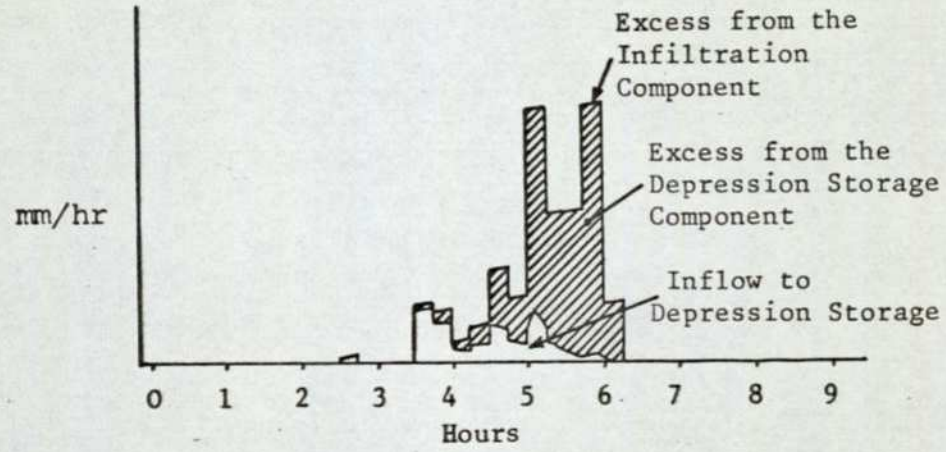


Figure 3.7(a) Function of the Depression Storage Component.

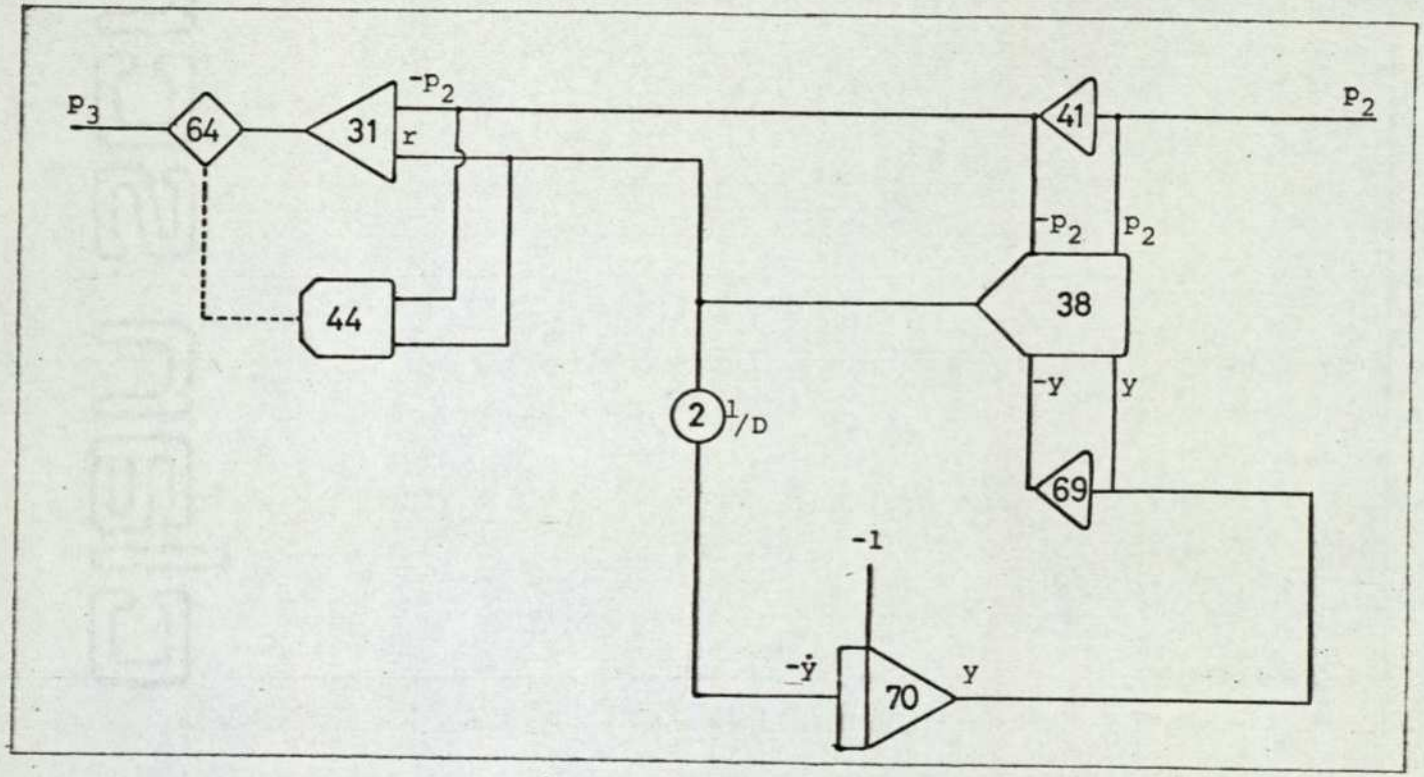


Figure 3.7(b) Analogue Circuit for the Depression Storage Component

Equation 3.xxx. It should be noted that the initial condition for Integrator 70 has a value of unity because at time t = o, accumulated excess from the infiltration component is assumed to be zero and hence the value of y in Equation 3.xix is unity.

The value of parameter 1/D is set on potentiometer 2, and emplifiers 41 and 69 are used only to reverse the sign for the purpose of multiplication.

3.II.4. The Routing Component. In simulating floods rather than daily or monthly flow volumes, routing becomes an important component of a simulation model. The basic problem is one of converting rainfall excess from the various other components into streamflow via catchment storage (Chow, 1964, p.25.35). The routing component must therefore take account of both overland flow and channel flow.

The most detailed and accurate way to represent routing is by the use of the St.Venant equations (Price, 1973), but these equations are complex, requiring the values of several coefficients and parameters to be measured and they must be solved numerically. They are therefore more suitable for use with small homogeneous reaches and flow planes. A much simpler method, but one in which the parameters are 'lumped' is the storage method. Prasad (1967) developed a model to represent the non-linear effects of streamflow routing, which required the calculation of three parameters. He related these to various catchment characteristics. However, on small catchments where storms are arealy uniform, fairly short in duration and of high intensity, a linear storage model would suffice (Nash, 1960).

The linear storage model assumes that the inflow to the system is related to the outflow by a constant linear operator. By assuming firstly that the outflow (q) from the channel system is proportional to the storage (S) within it :

$$q = k_r S \qquad (3.xxiii)$$

where k_r = a storage coefficient,

and secondly that the rate of change in storage is equal to the difference between the inflow (I) and the outflow :

$$\frac{dS}{dt} = I - q \qquad (3.xxiv)$$

a general routing equation may be derived, relating inflow to outflow. The differentiation of Equation 3.xxiii and substitution into Equation 3.xxiv gives :

$$\frac{dq}{dt} = k_r .(I-q) \qquad (3.xxv)$$

This function was used in the simulation model and the analogue circuit for generating this function is shown in Figure 3.8(b). The excess from the previous components (p_3) is added to the outflow using summer 60. This is then multiplied by the parameter k_r to give the first derivative of the outflow (q). Integration of this variable (using integrator 30) gives outflow from the catchment,

3.II.5. Implementation on the Hybrid Computer. All the components described in the preceding sections are connected on the patch panel of the analogue computer (see Figure 3.9). Control of the simulation, input and output of hydrometeorological

for imput to the optimization procedure of the simulation model.

4.III.2. Meteorological Data. Tables of daily rainfall amounts were obtained from the Meteorological Office for the gauges at The Beeches, Broadfield and Tinsley Green. These were used in the analysis of the daily flow on Crawters Brook, with The Beeches being used to represent daily rainfall over the catchment.

Copies of the Dines raingauge charts for Broadfield were obtained for selected storms, including the largest flood in every year. Rainfall amounts at 15-minute intervals were calculated by graphical means rather than by digitization because the amount of data needed was relatively small and did not warrant any extra programming involved. The derived rainfall amounts were punched onto paper tape for input to the hybrid computer simulation model. Variables for the analysis of peak flow magnitudes and for the analysis of model parameter variations were also measured from the charts. These were maximum 15-minute and 30-minute storm rainfall intensities, storm duration and rainfall depth.

Monthly values of temperature, sunshine hours, daylight hours, relative humidity and a daily antecedent precipitation index (API) were punched onto cards for the analysis of daily flow. The API was derived from the rainfall data using the formula :

$$API_t = c.(API_{t-1} + P_t) \qquad (4.i)$$

where API_t = antecedent precipitation index on day t

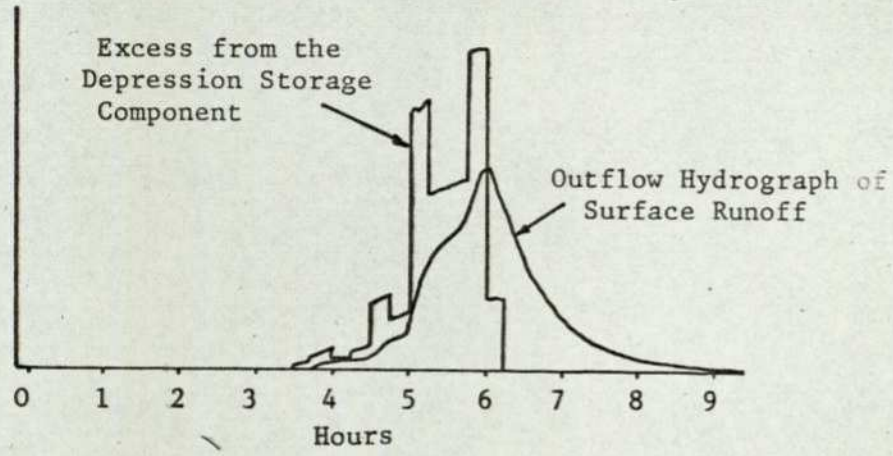


Figure 3.8(a) Function of the Routing Component

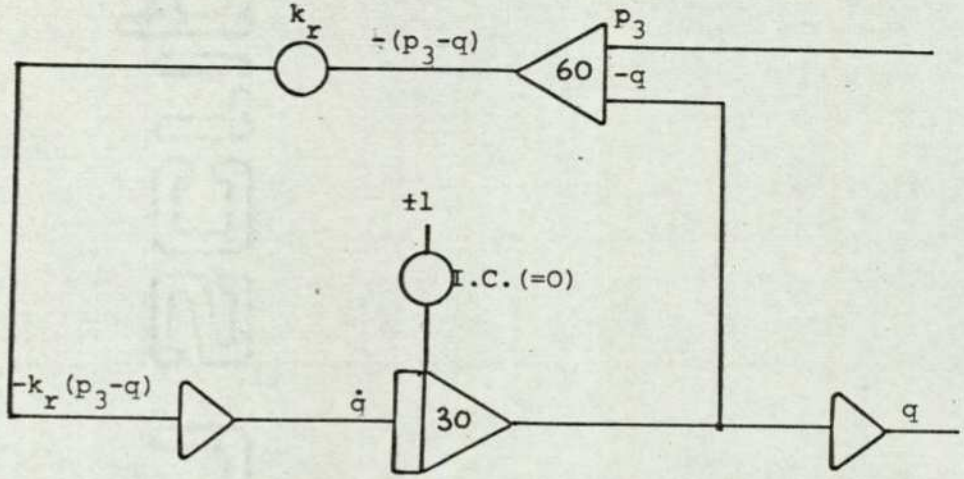


Figure 3.8(b) Analogue Circuit for the Routing Component.

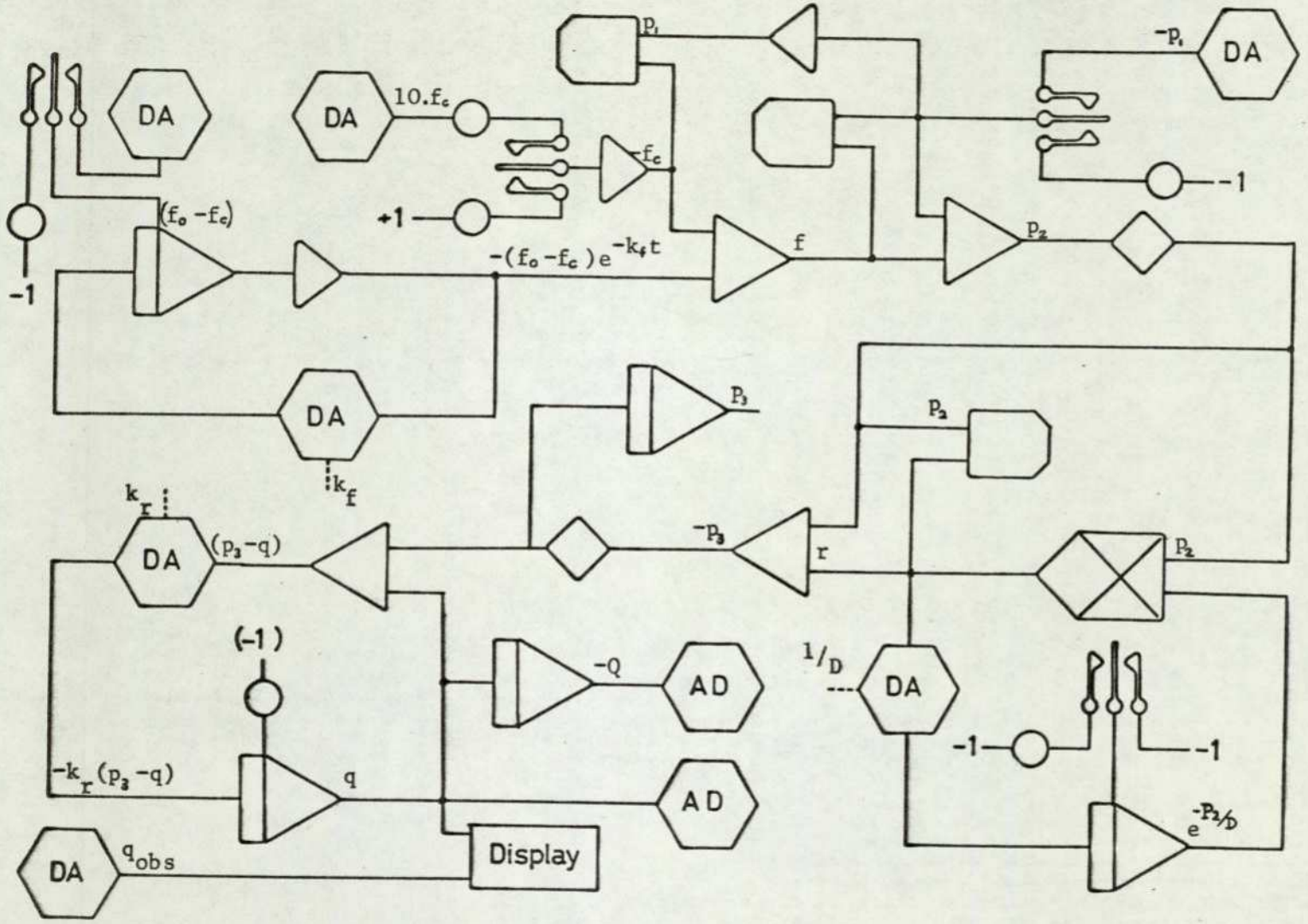


Figure 3.9 Analogue Circuit for the Complete Simulation Model

data, and the optimization of the parameter values, is performed by the digital computer. Figure 3.10 shows the major components of a typical hybrid computer system. The various interface components allow logical data to be transferred via sense lines (analogue to digital) and control lines (digital to analogue), and allow numeric data to be transferred via the converters.

In the final version of the model used for the simulation, potentiometers were replaced by digital to analogue multipliers. This allowed a much more rapid operation of the parameter optimization programme because potentiometers often take a relatively long time to adjust to the correct value. Several additional components were also added to the model, which allowed such variables as cumulative rainfall excess, total runoff volume, etc. to be recorded.

The timing diagram for the simulation is shown in Figure 3.11. The precipitation data was accumulated over 15-minute intervals and after several test runs with a sample storm, it was decided that a scale factor of 15 minutes real time to 0.2 seconds computer time was suitable. In order to synchronize the data transfer a logic pulse was generated using the timing circuit shown in Figure 3.12. (see Figure 3.1 for definition of logic symbols). This was then used to initiate the various digital computer commands and operations.

These are as follows :

(a) set the initial parameter values,

(b) put the analogue computer into the 'Initial Condition' mode (call IC),

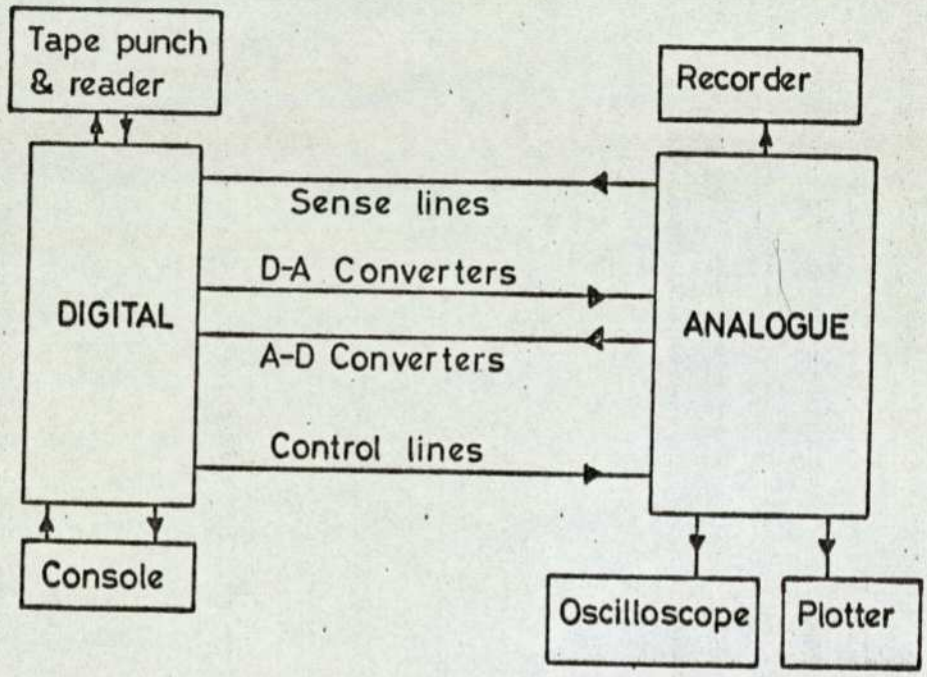


Figure 3.10 Major Components of a Hybrid Computer System

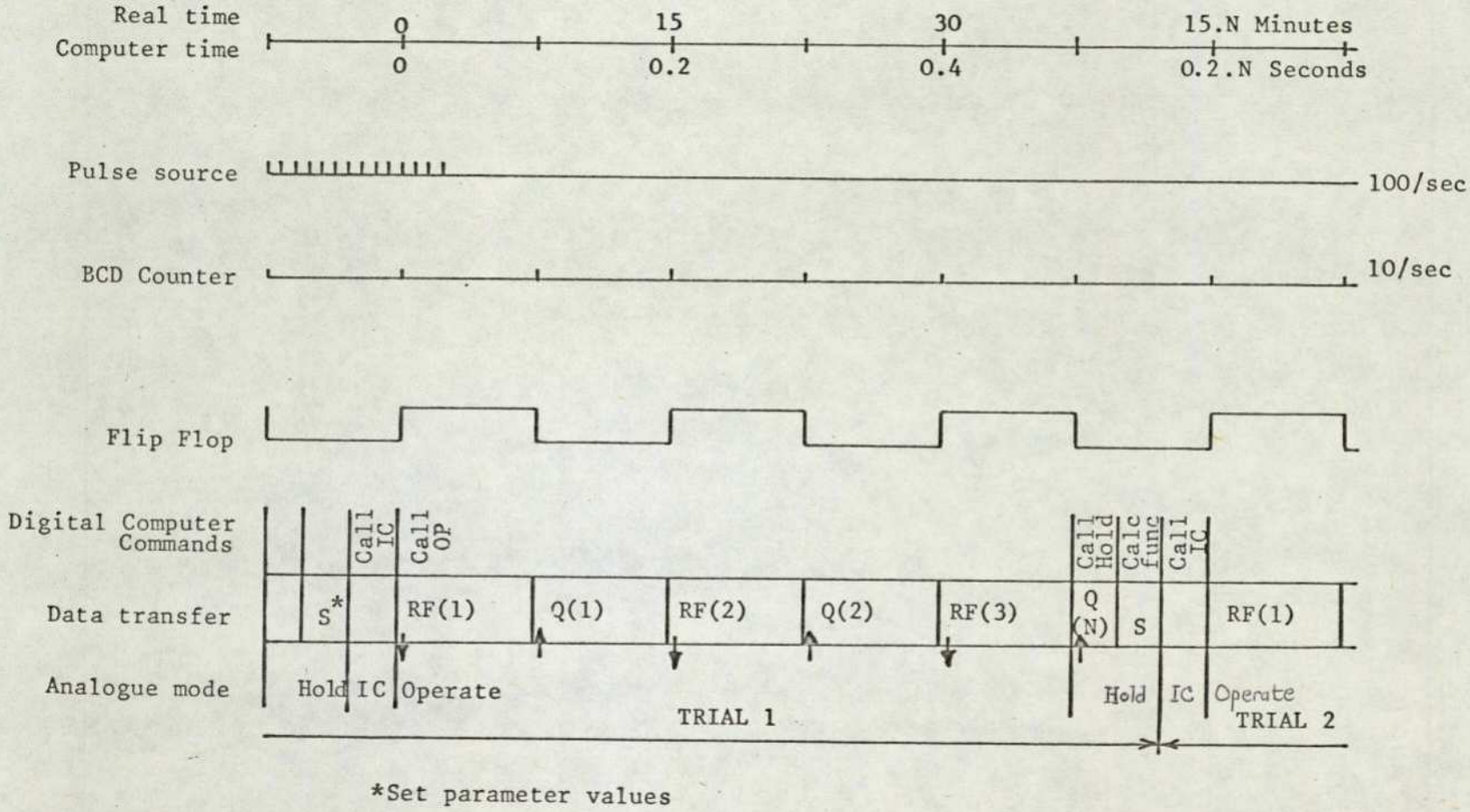


Figure 3.11 Timing Diagram for Simulation

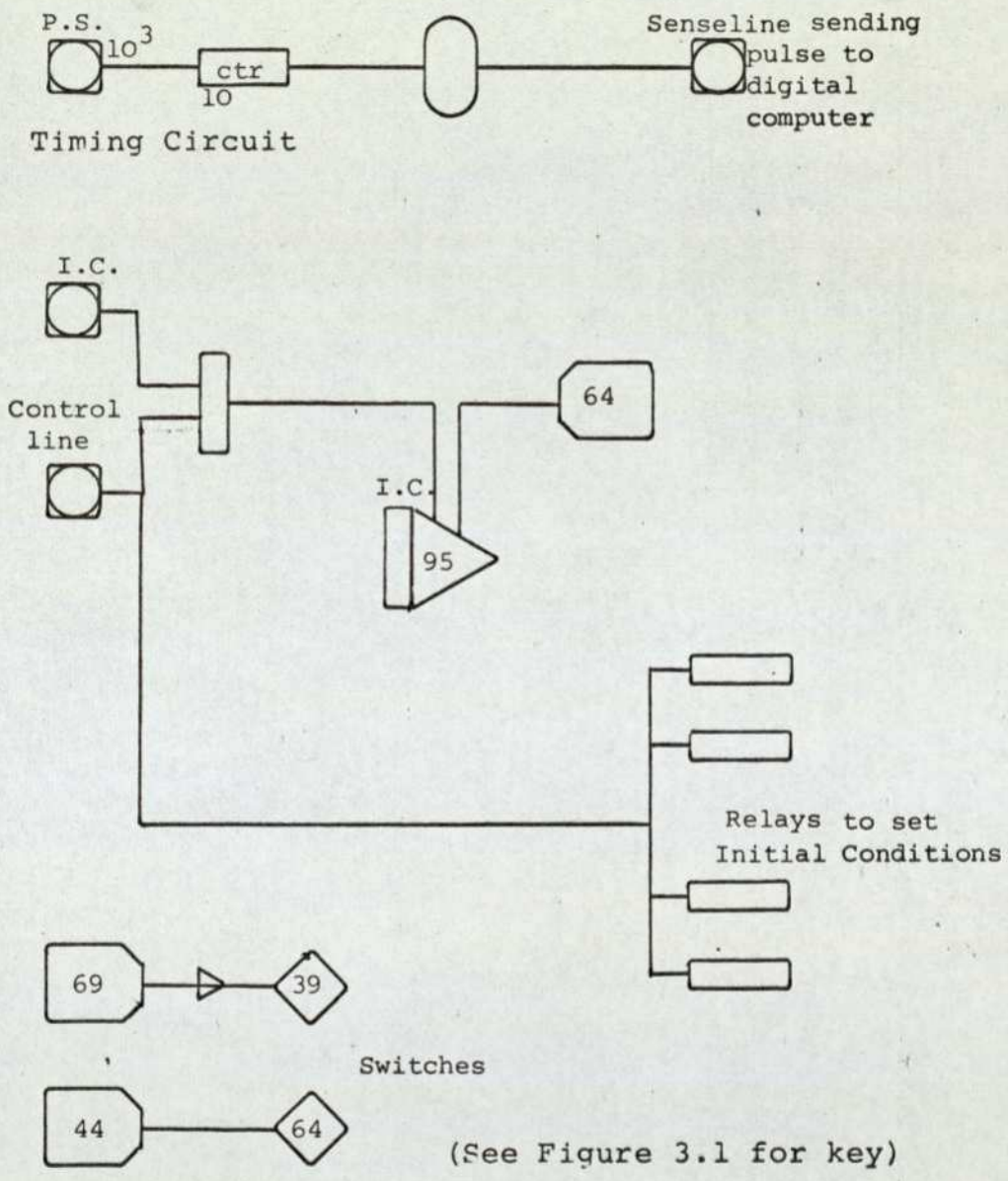


Figure 3.12 Logic circuit to control Simulation.

(c) transfer the first rainfall value and put the analogue computer into the 'operate' mode (call OP),

(d) read the first simulated streamflow value,

(e) repeat the data transfers until the end of the simulation period,

(f) put the analogue computer into the 'Hold' mode (call Hold),

(g) calculate the objective function (see Chapter 3.III) and change the parameters accordingly,

(h) repeat from (b).

The hybrid computer was found to be a useful tool in developing the model due to the high degree of so-called 'man-machine interaction'. Adjustments to parameters and to model structure could be made quite easily and the results displayed almost instantaneously on the oscilloscope or graph plotter.

3.III. PARAMETER OPTIMIZATION

A general mathematical model simulating catchment hydrological processes is made specific when the model parameters are assigned numeric values. The 'correct' parameter values are those which allow the model to reproduce observed hydrological variables as accurately as possible. The values may be measured in the field, laboratory or from maps or they may be derived from the observed hydrometeorological data. The methods which utilize the latter by comparing them with simulated variables are termed 'parameter optimization techniques'. The assignation of parameter values by means of optimization techniques is often termed

model calibration.

3.III.1. The Criterion of Fit. The process of optimizing a simulation model entails comparing the simulated streamflow with observed or recorded streamflow and adjusting the model parameters until a better 'fit' is obtained. The criterion of fit may be a subjective visual assessment of the two hydrographs, or it may be an 'objective function' such as the area between the two hydrographs. An objective function (F) is a function of the n parameters $(x_1, x_2, \ldots, x_n)$ in the model, i.e.

$$F = f(x_1, x_2, \ldots, x_n)$$

A typical objective function for a two-parameter model is shown in Figure 3.13. Such function surfaces are often called 'response surfaces'.

The purpose of parameter optimization techniques is to adjust the parameters in order to minimize or maximize the objective function. Once the optimum set of parameter values has been found, the model can be used for the purposes for which it was designed, e.g. prediction.

The most commonly used objective function is the 'sum of squares' :

$$F = \sum (S_i - Q_i)^2$$

where S_i = simulated flow

Q_i = observed flow

n = number of evenly spaced values in the record.

This criterion has certain advantages over other criteria

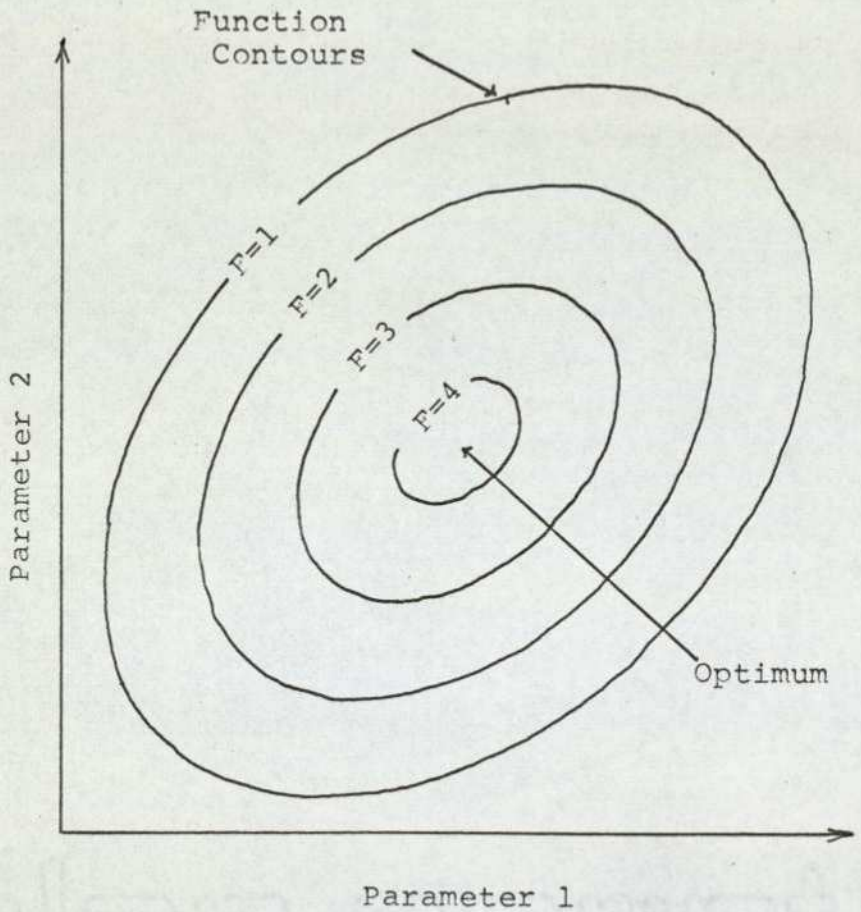


Figure 3.13 Typical Objective Function

(Ibbitt, 1970) because : (1) for linear models the contours of the surface are quadratic and are therefore amenable to certain powerful optimizing techniques; (2) it has statistical significance since $F/(n - 1)$ is the variance of the individual functional values; (3) it is simple to calculate; (4) it gives greater proportional weight to flood peaks. For these reasons, the sum-of-squares criterion was chosen as the objective function for the optimization of the simulation model used in the study.

3.III.2. Automatic Optimization Techniques. Although subjective optimization of model parameters has been predominant in past studies, automatic methods have three distinct advantages : (1) parameter values do not vary if different people carry out the optimization; (2) they are more accurate; (3) they are more rapid.

The most obvious way to find the optimum set of parameters would be to increment each parameter in turn by a small amount and to record the smallest functional value, but this would obviously require much superfluous effort. Therefore more effective methods have been developed. These fall into two classes, deterministic or sequential methods, and random or non-sequential methods. Ibbitt (1970) tested the various methods available and concluded that the random search method, in which successive sets of parameter values are randomly sampled and substituted into the model equations, found reasonable sets of parameter values when the initial parameter values were wildly inaccurate. Deterministic methods were found to be more

efficient when the parameter values were not too far from their optimal values because they were able to follow the function contours until the optimum was found. Thus when the objective function has more than one local optimum but only one 'global' optimum, the random method ensures a more extensive search than deterministic methods.

Deterministic methods can be broken down into gradient search, least-squares search, and direct search methods. Gradient search methods require the calculation of the derivatives of the objective function with respect to each of the parameters and follow the steepest gradient of the response surface until the optimum is found. Least-squares methods find the best possible starting set of parameters by solving the quadratic equation of the objective function in x_i (Ibbitt and O'Donnell, 1971). Direct search methods are diverse but most calculate the objective funtion for a number of sets of points near the initial set and then continue the search on the basis of that part of the response surface so revealed.

Ibbitt found that the most effective deterministic method was that due to Rosenbrock (1960). This is a direct search method based on a 'ridge-following' technique. The initial set of parameter values is chosen (point A on Figure 3.14) and a unidirectional stepwise search made until the maximum is found (point B). This process is repeated in an orthogonal direction until a further maximum is found (point C). The line connecting these two points is then used as a new search direction with further trial directions orthogonal to this new direction.

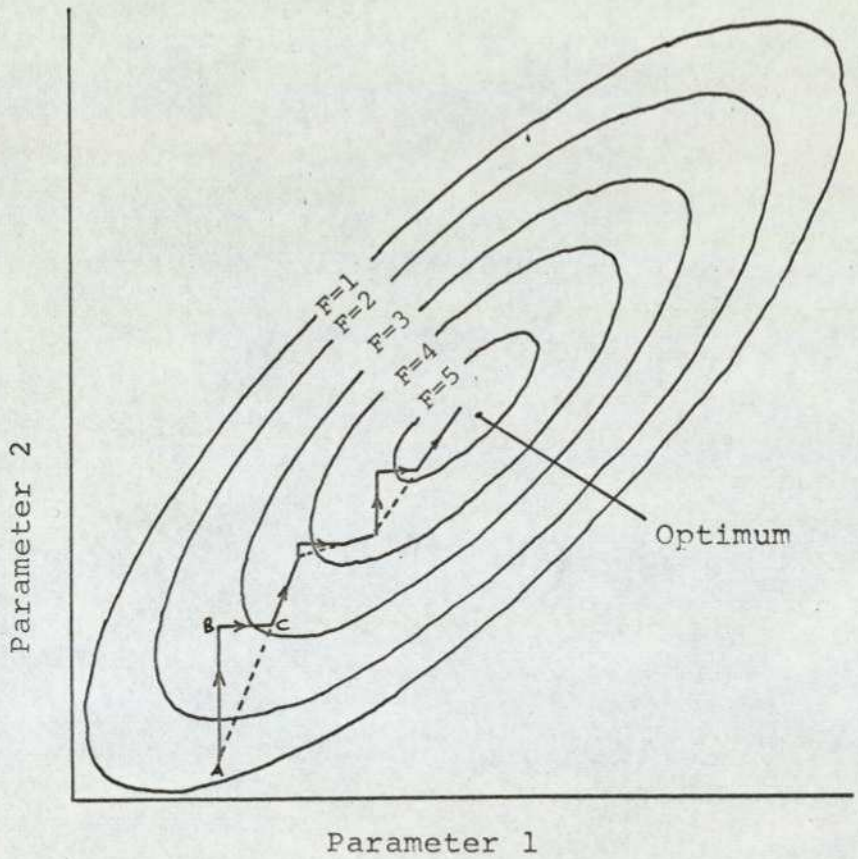


Figure 3.14 Typical Rosenbrock-method (Ridge-Following) Search

Source: Ibbitt, 1970, p.62.

This technique was found to converge rapidly on the optimum. A similar direct search technique was developed by Nelder and Mead (1965) after being suggested by Box (1957). This 'Simplex' method was available for use on the City University EAI 690 Hybrid computer as a library programme and was therefore used to optimize the hybrid computer simulation model.

3.III.3. The Simplex Method. This method is best explained by reference to the flow-chart in Figure 3.15. The sequence of operations is as follows, (assuming minimization is required):

(a) The initial set of n parameter values is arbitrarily chosen by the operator and the other n sets of values are calculated by the computer programme to form the initial 'Simplex' with n + 1 points (in the two-parameter case, the Simplex would therefore be a triangle superimposed on the response surface). The relative size of the initial Simplex is determined by a control parameter 'SCALE'.

(b) The objective function is evaluated for each point on the Simplex by repetitive simulation.

(c) The functional values are sorted and the minimum (F_{min}) and maximum (F_{max}) values found.

(d) the co-ordinates of the 'centroid' of the Simplex opposite the point corresponding to F_{max} is calculated by averaging the co-ordinate values for each parameter in turn, omitting the co-ordinates at F_{max}. (Thus for the two-parameter case, the centroid is midway between

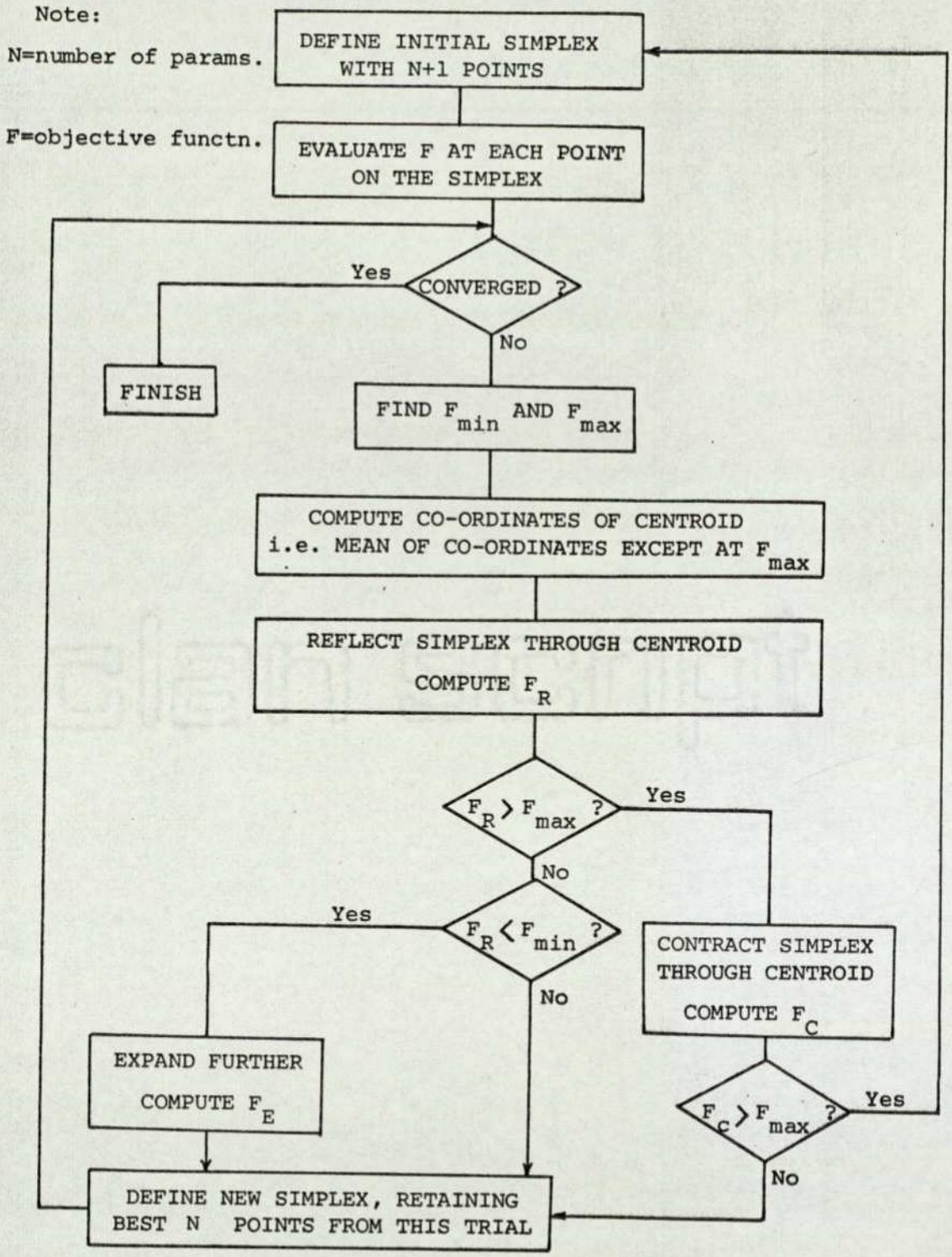


Figure 3.15 Optimization Flow Chart for the 'Simplex' Method.

the two points on the Simplex opposite F_{max}).

(e) The Simplex is reflected through the centroid and the objective function (F_R) at the new reflected point is evaluated.

(f) If $F_R > F_{max}$,

(i) the Simplex is contracted and the objective function (F_c) evaluated at the contraction point,

(ii) then if $F_c < F_{max}$, this point is used in the construction of a new Simplex; otherwise all co-ordinates are changed and the sequence is begun again.

(g) If $F_R < F_{max}$,

(i) but $F_R > F_{min}$, a new Simplex using F_{min} and F_R is defined;

(ii) otherwise, when $F_R < F_{min}$, the Simplex is expanded further and the objective function (F_E) evaluated at the expansion point. The points corresponding to the newly found F_{min} the the previous F_{min} are then used in the definition of a new Simplex.

(h) A new trial is initiated, starting the sequence from (b).

The terminology used in the above description may be clarified by reference to the two-parameter case shown in Figures 3.16 and 3.17. The three basic operations are reflection, contraction and expansion (Figure 3.16). Reflection occurs first and may be followed by expansion or contraction in the same direction, or by reflection in another direction. Three of the four possible results of a trial are shown in Figure 3.17, illustrating the

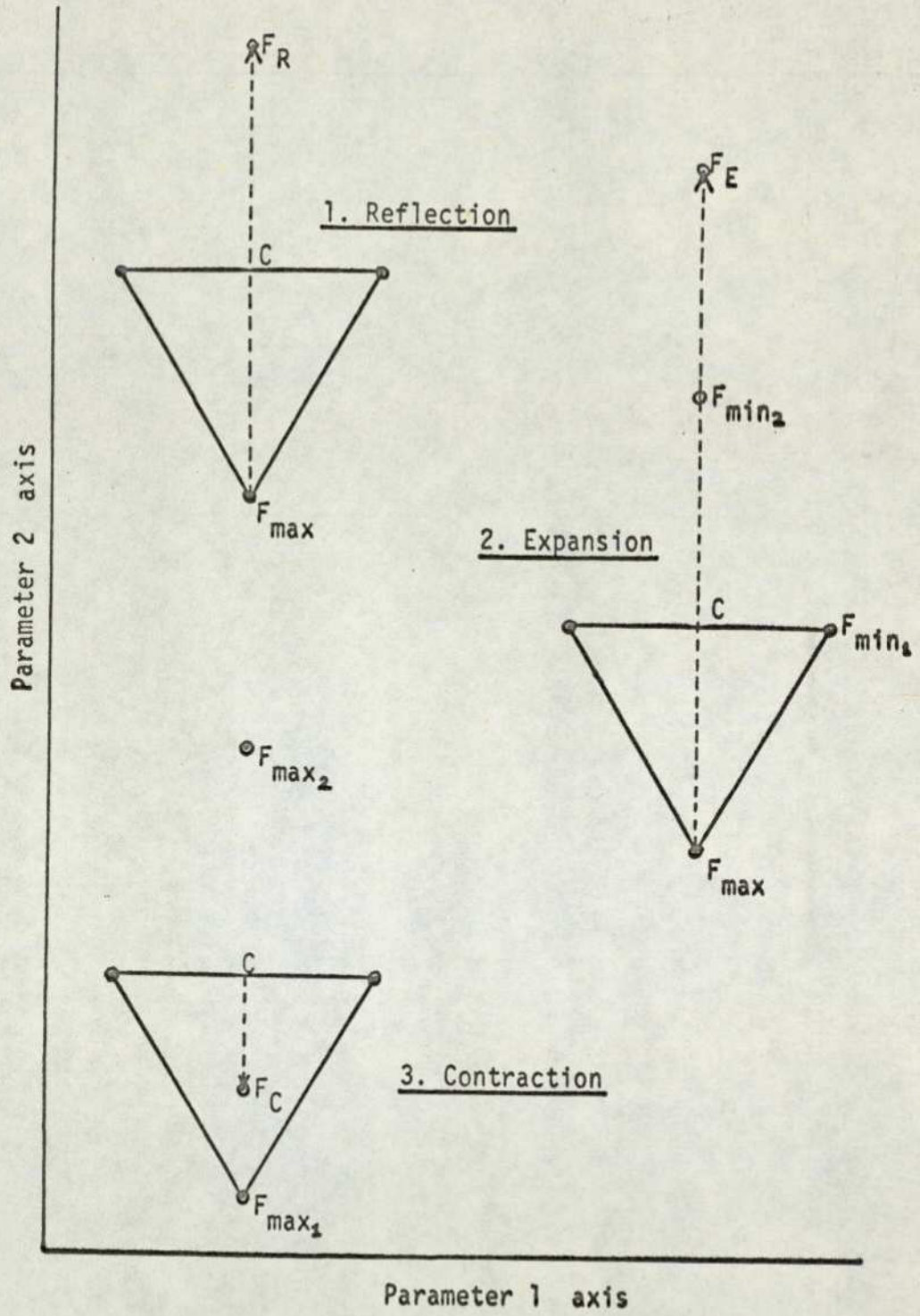


Figure 3.16 Basic Operations in the Simplex Method

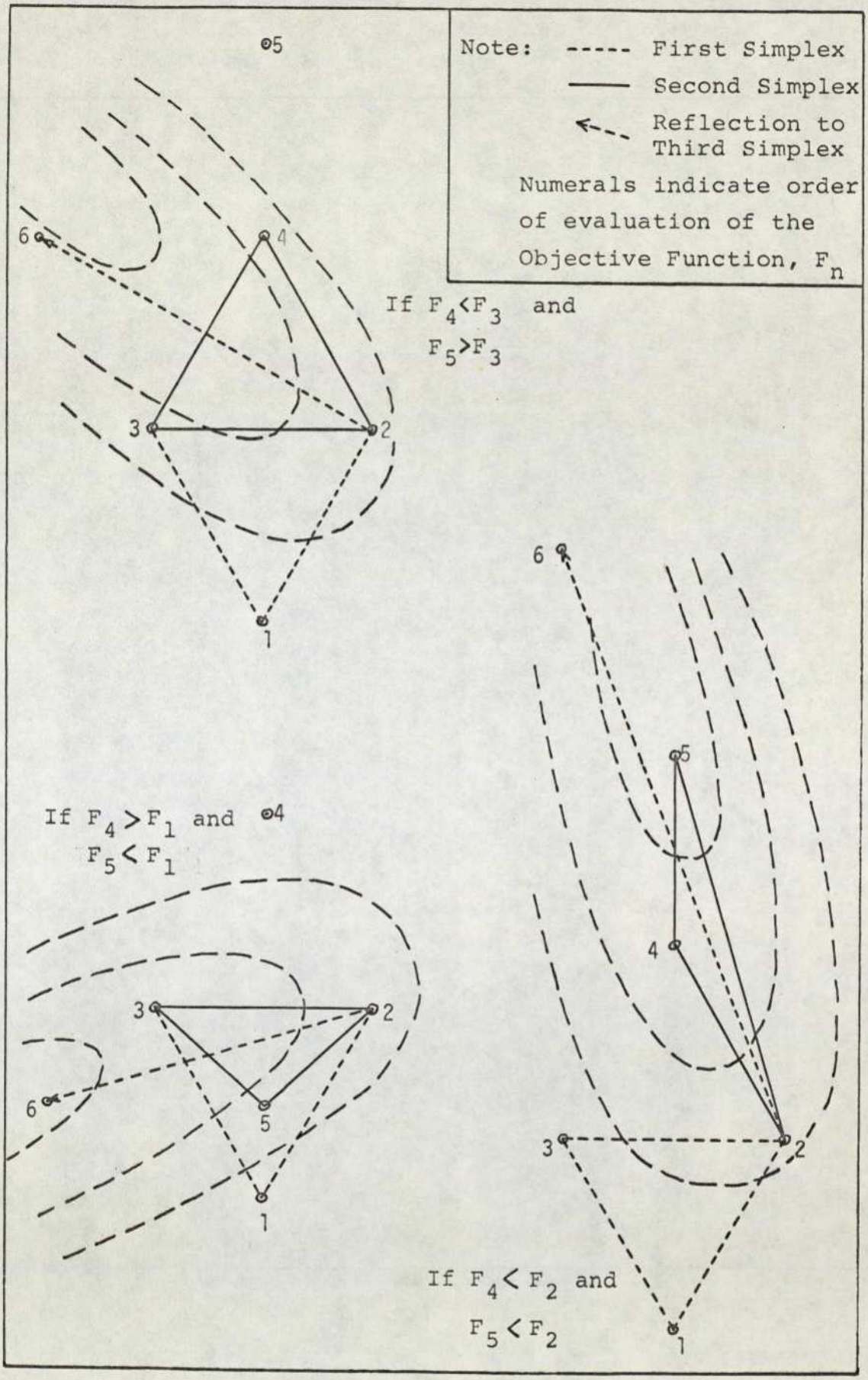


Figure 3.17 Possible Results of a Simplex Trial.

new direction of search for each case. The fourth possibility is when the trial fails to find a value less than F_{max}, in which case the procedure is started with a new initial point. As the optimal functional value is converged upon, the Simplex gets smaller and smaller because contraction occurs more often than expansion. The smallness of the Simplex is therefore an indicator of convergence and is tested against a control parameter 'CONV'. This test may cause the programme to terminate, or the programme may also be terminated by the control parameter 'INUM'. This tests whether the maximum number of allowable trials has been exceeded. The three control parameters SCALE, CONV and INUM therefore allow a great deal of control to be exercised over the optimization procedure.

The procedure was tested on a simple two-parameter model (Morris and McGovern, 1973) in order to study the search paths generated by the programme and to test the effect of varying the control parameters. A typical search path is shown in Figure 3.18 indicating the way in which the 'trough' is followed to the optimum. It was found that appropriate values of the control parameters for a first set of trials were :

SCALE = 0.1

INUM = 50

CONV = 0.1

The optimum set of simulation parameters found were then used as the starting point in a second set of trials with :

SCALE = 0.01

INUM = 100

CONV = 0.001

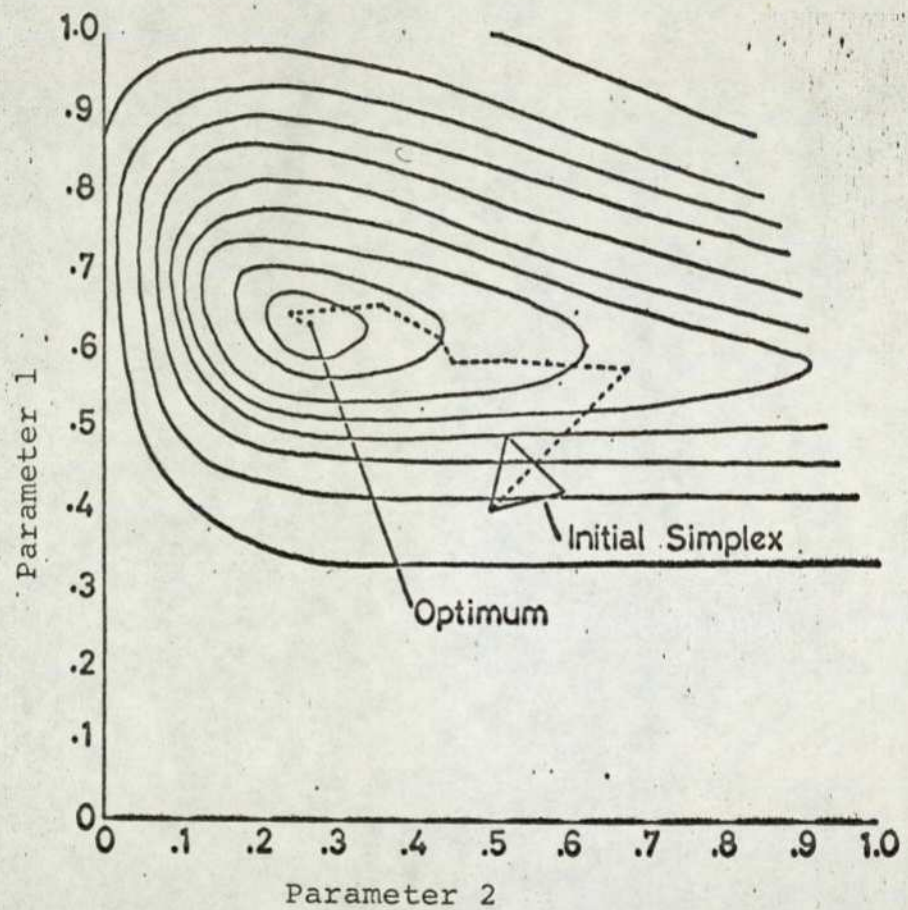


Figure 3.18 Typical Two-parameter Search Path using the Simplex Method.

Source: Morris and McGovern, 1973.

The first set of trials was carried out to find the approximate area of the optimum, while the second set of trials pinpointed the optimum with some accuracy. However, if the approximate value of the simulation parameters was known, only one set of trials was found to be necessary.

Detailed results of the optimization of the hybrid computer simulation model are discussed in Chapter 5.

CHAPTER FOUR

THE STUDY AREA AND DATA SOURCES

Section 4.I of this chapter provides a description of the catchment studied and a background to the urban development of Crawley. The subsequent sections deal with the collection and preparation of data needed for the study.

4.I. CRAWTERS BROOK AND THE URBANIZATION OF CRAWLEY

The two approaches to the study of the effect of urban development on streamflow were discussed in Chapter Two and it was concluded that a study of the variation in streamflow characteristics through time of an urbanizing catchment was to be made. The requirements of a study of this kind are : (1) the existence of an area with recent rapid urban development; (2) appropriate hydrometeorological data in the area; (3) a catchment in the area that had experienced a rapid catchment-wide growth in its proportion of urbanized area.

An enquiry into the possible areas suitable for study revealed that remarkably few area fulfilled these requirements. Most catchments either had insufficient data due to their relatively slow urbanization over several decades, or they were not yet sufficiently urbanized to have much effect on streamflow. It was concluded that the New Town of Crawley in Sussex

offered the best opportunity for study because (1) it had experienced steady urbanization for the past twenty years, (2) it had continuous precipitation and streamflow records for most of the 20-year period of urban development, (3) it was in the headwaters of a catchment and one of the streams, Crawters Brook, became entirely urbanized during the study period, and (4) the Crawters Brook catchment was small enough to be a manageable size for the purposes of data collection and simulation.

4.I.1. Description of the Catchment. The study area lies in the south-east of England approximately 55km south of London (see Figure 4.1). The Crawters Brook catchment is underlain by Lower Cretaceous strata, mainly Weald Clay, and is in an area of undulating topography. The streams in the area drain to the River Mole (Figure 4.2), which in turn is a tributary to the River Thames.

The catchment covers an area of about 2.24km^2 and the catchment boundary (Figure 4.3) was delimited by reference to the maps of the Drainage Department of Crawley Urban District Council. The highest point on the catchment is a little below 100m O.D., while the gauging station at Woolborough Road is at about 64m O.D. The land use in the area prior to the urban development was mainly arable farming, with patches of woodland in places less suitable, such as the southern part of the catchment. The size of the catchment was thought to be small enough to have fairly homogeneous physical characteristics such

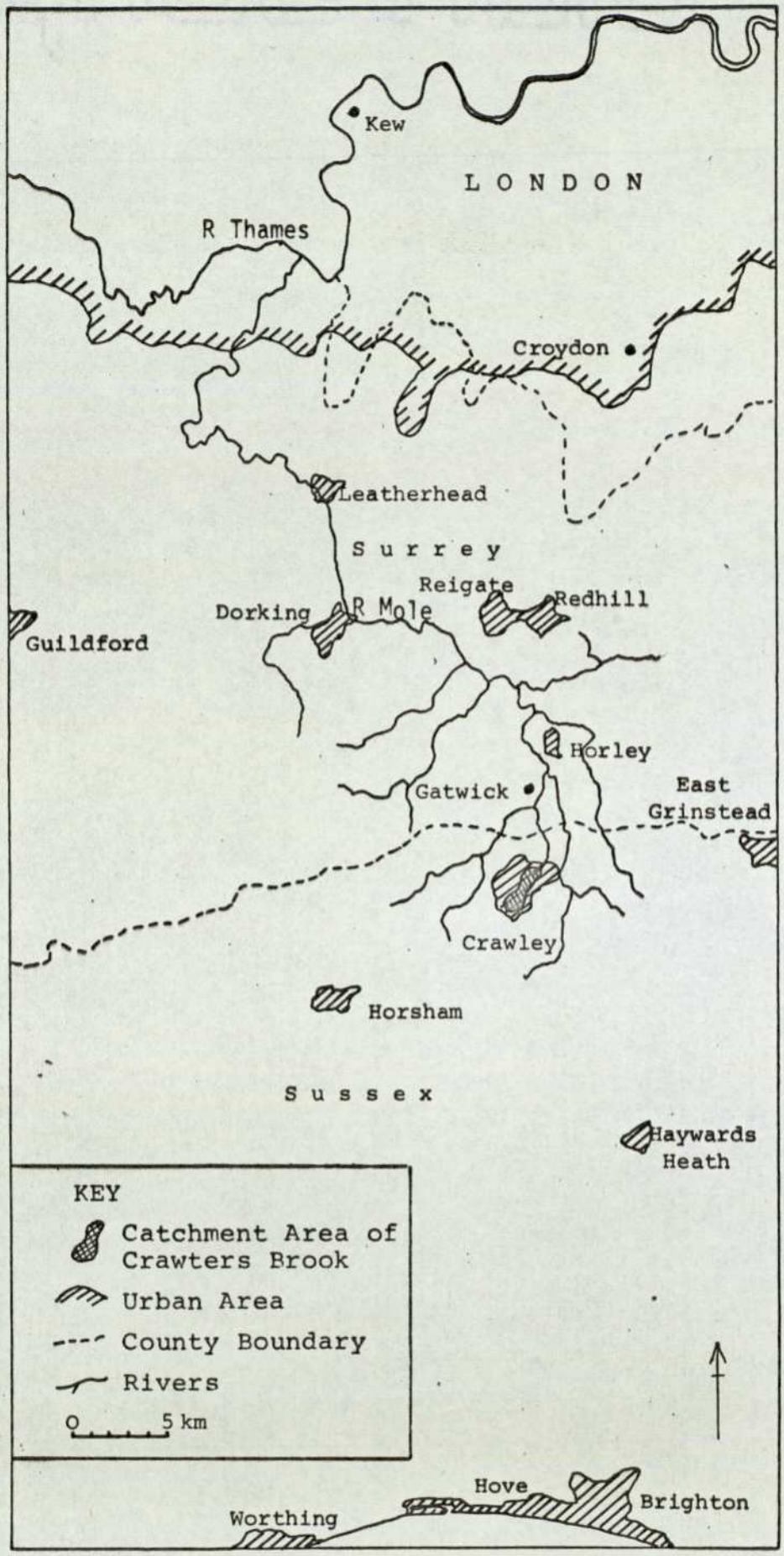


Figure 4.1 Location Map

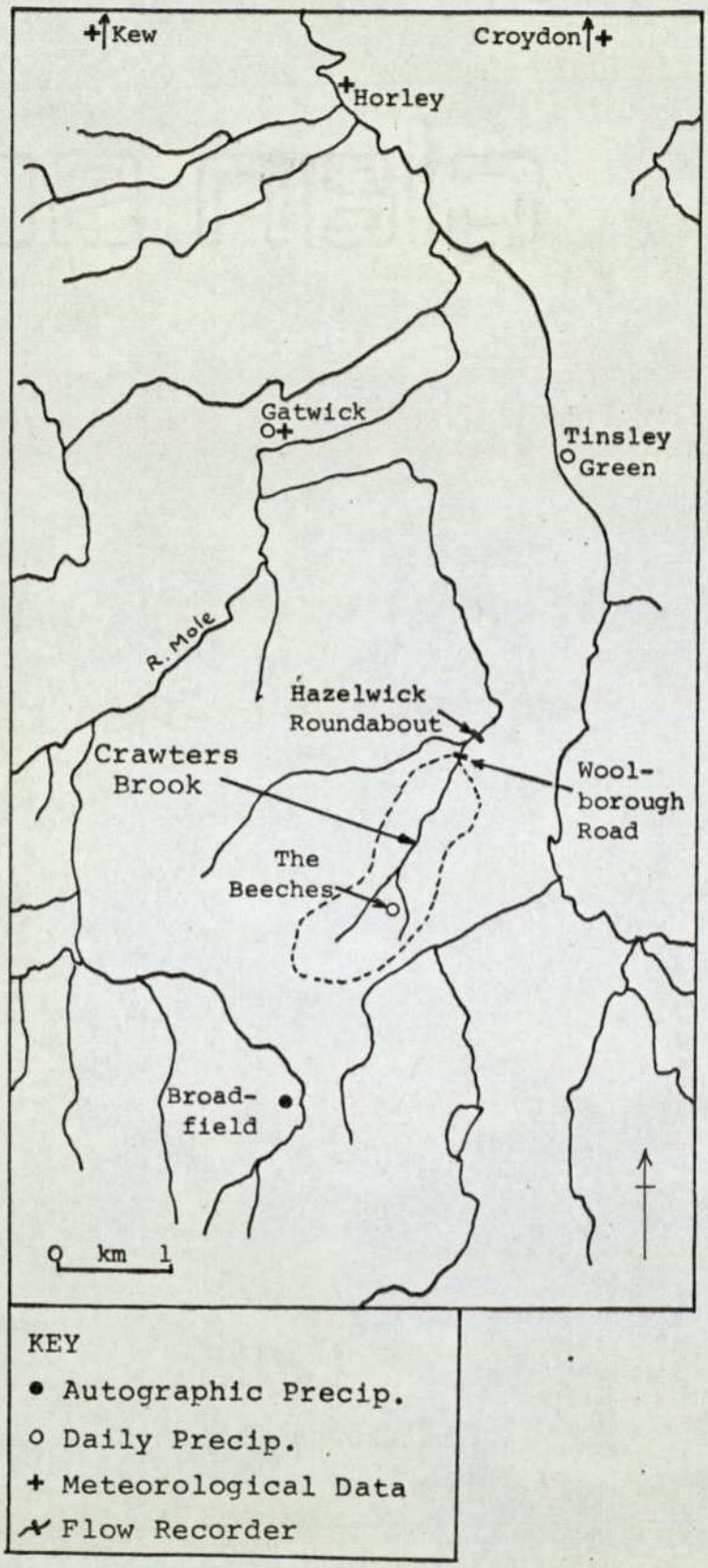


Figure 4.2 Hydrometeorological Stations used in the Study

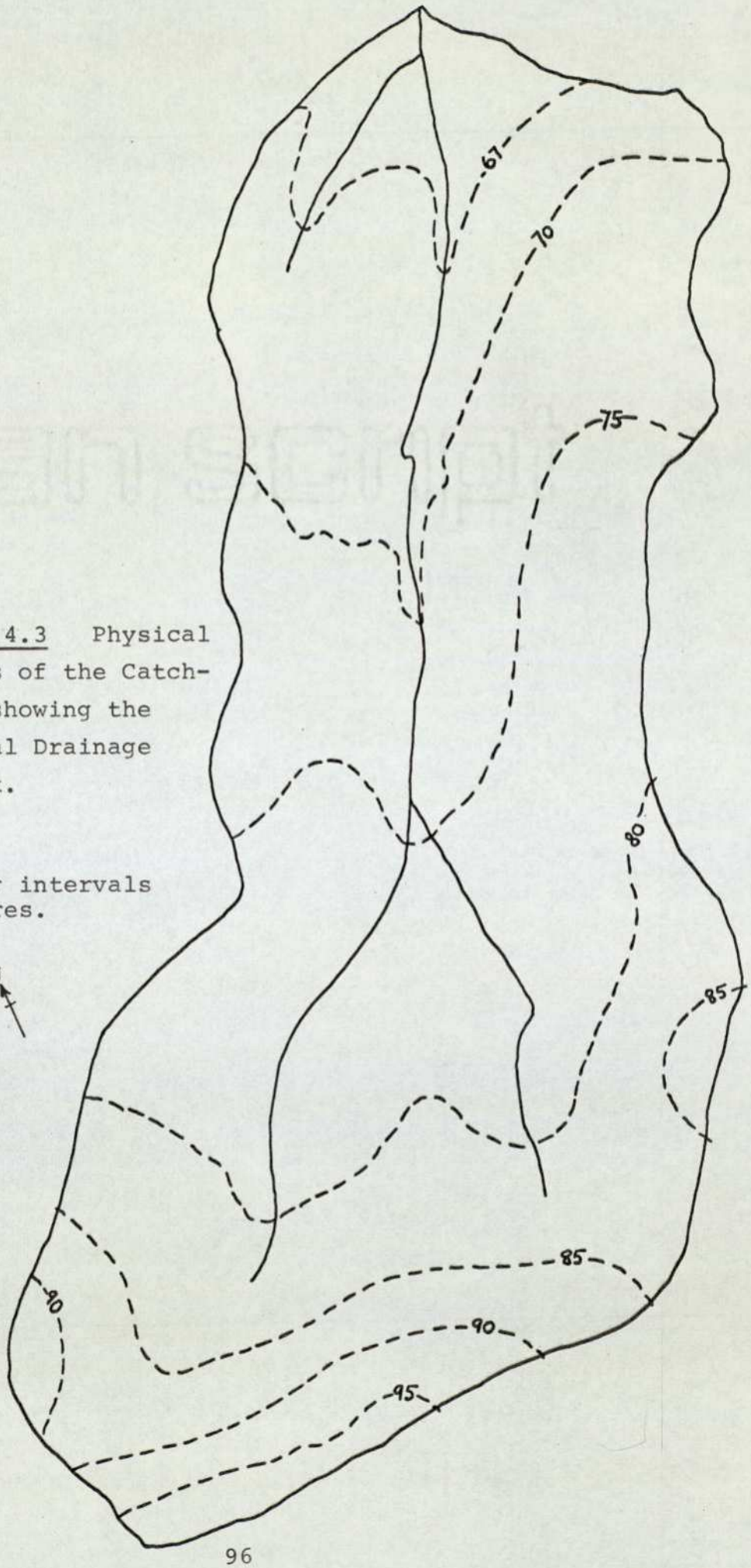


Figure 4.3 Physical Aspects of the Catchment, showing the original Drainage Network.

Contour intervals in metres.

as soil type, overland flow slope and roughness, micro-topography, etc. A longitudinal section of the main channel is shown in Figure 4.4.a.

4.I.2. The Building of the New Town. The New Town of Crawley was designated on 9th January 1947, soon after the passing of the New Towns Act in 1946. The provisional plans for the town of 50,000 people appeared in February 1950, although the size of the town was later to exceed this figure. By the end of 1952 some 1,500 homes had been constructed. Typical development in the early phase of urbanization is depicted in Plate I; the line of trees in the picture marks the stream course.

The new town was built around the old villages of Crawley, Ifield and Three Bridges, which had a combined population of 8,000 in 1948. The nine neighbourhood units were each to have a population of between 5,000 and 8,000, and the four inner units encompassed the catchment of Crawters Brook. Urbanization continued at up to 2,500 homes per year until 1958, when the rate dropped to about 500 per year. A graph of the growth in the number of new houses in Crawley is given in Figure 4.5. The distribution of buildings in the catchment is shown in Figure 4.6. The central area of the New Town was built adjacent to the old town centre of Crawley. This central area, part of which is shown in Plate 2, was built within the Crawters Brook catchment. A transection through the town along the line of the main channel brings out the distribution of the percentage of impervious cover within the catchment towards the end of

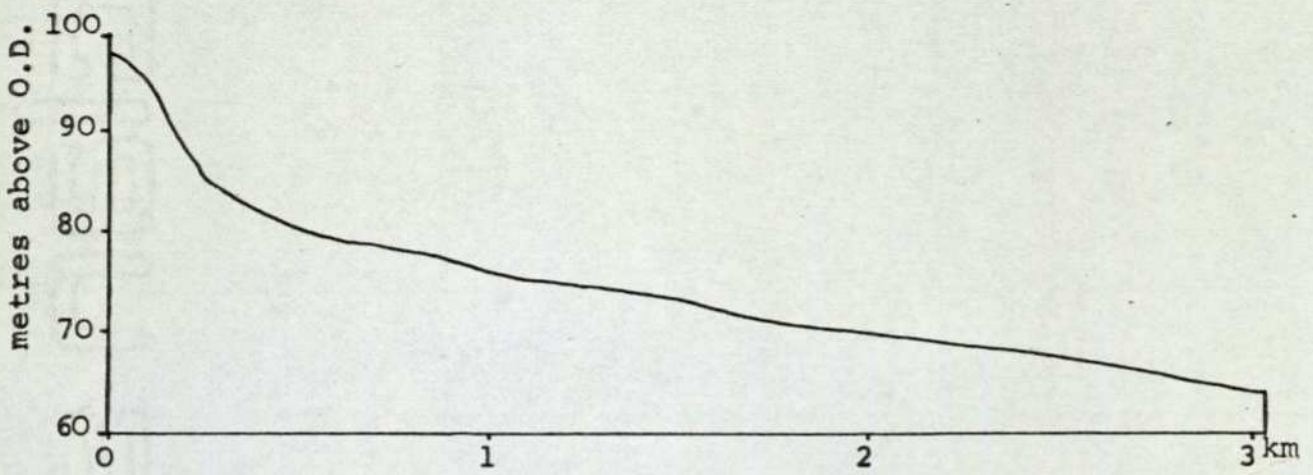


Figure 4.4(a) Long Section of the Main Channel

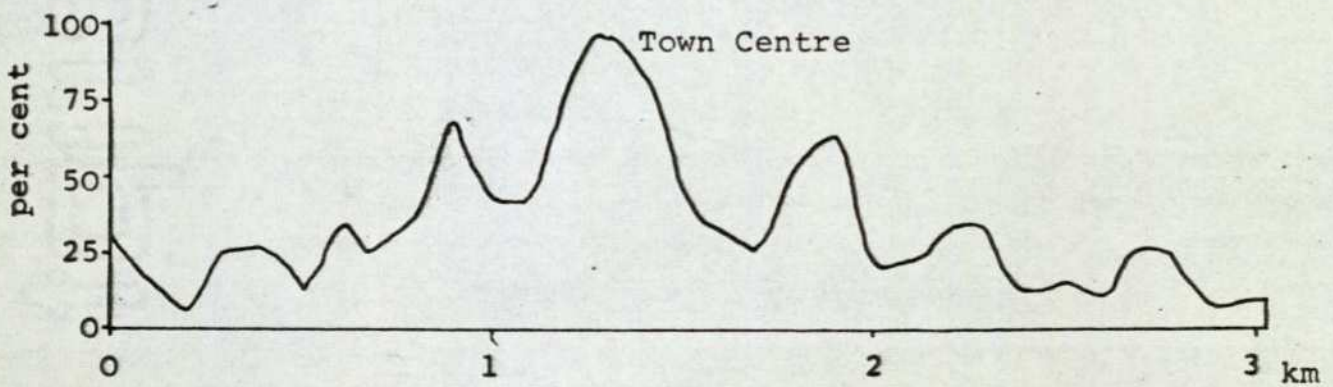


Figure 4.4(b) Long Section Showing Distribution of Impervious Area

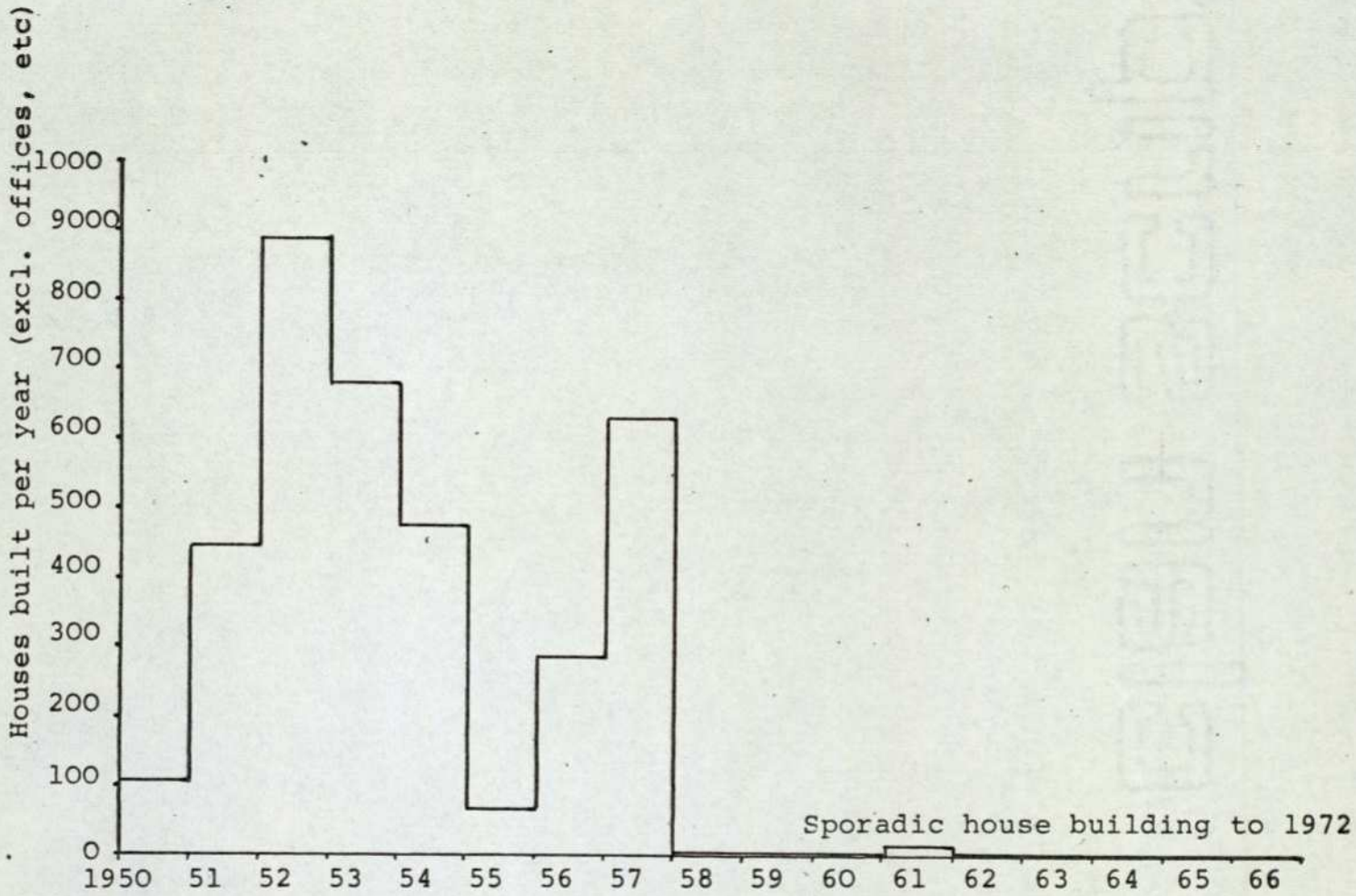


Figure 4.5 Graph of Urban Development of Houses through time.

Source: Adapted from 'Crawley Official guide', 1968.

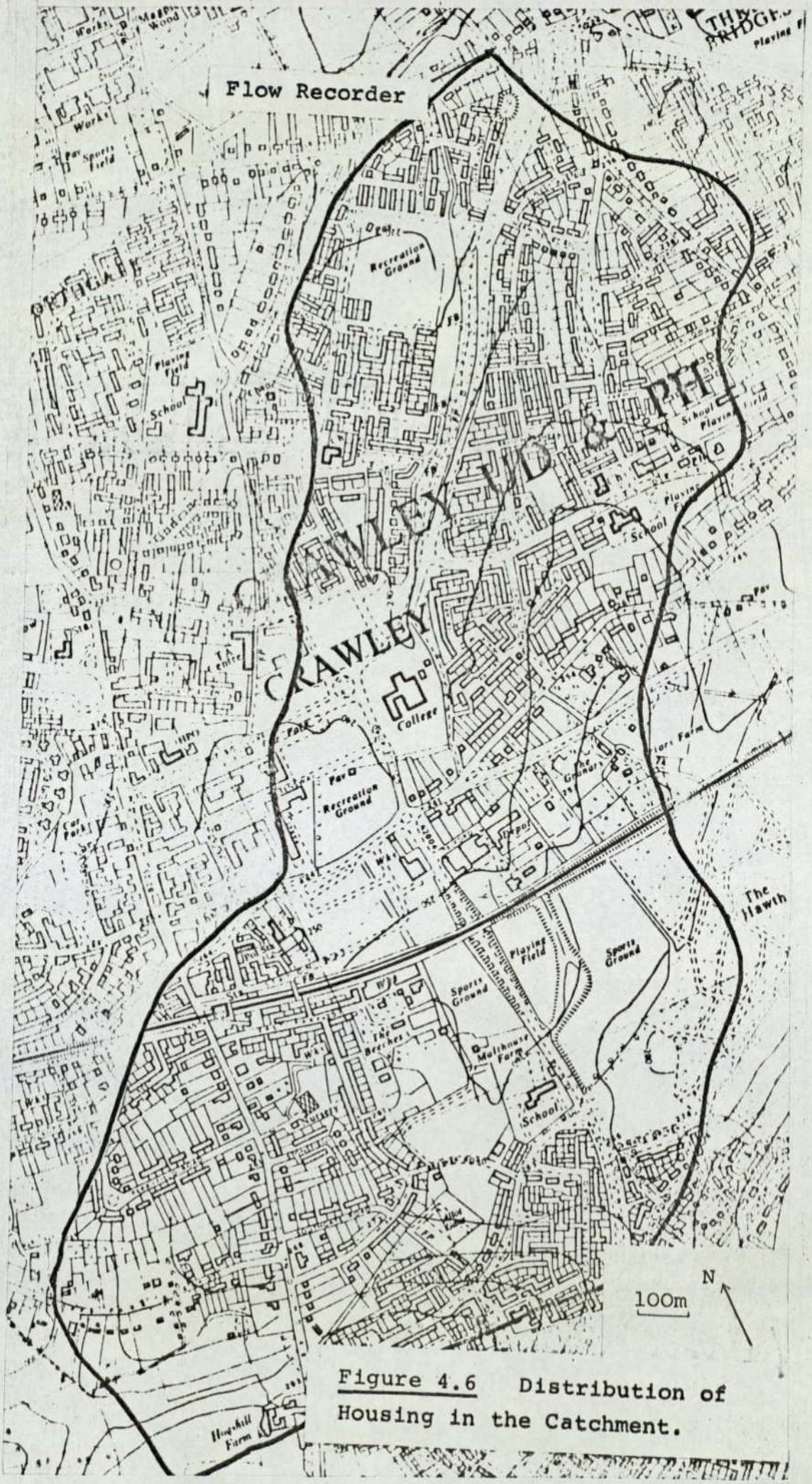


Figure 4.6 Distribution of Housing in the Catchment.



Plate 1. Early developments in the northern part of the catchment (Northgate).



Plate 2. Crawley town centre, near the centre of Crawters Brook catchment, showing a high percentage of impervious cover (80 per cent).



Plate 3. An area with a fairly high percentage of impervious cover (about 45 per cent).

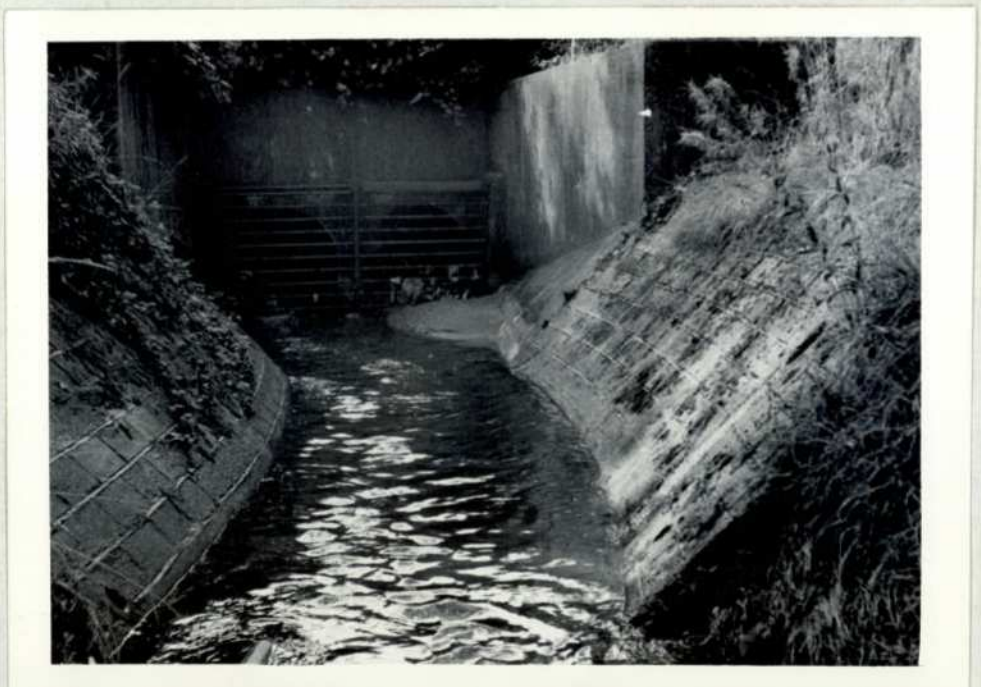


Plate 4. Channel system modifications - the emergence of Crawters Brook from the town centre culvert into a lined open channel.



Plate 5. Channel system modifications, showing a piped tributary to the main channel and channel straightening and lining.



Plate 6. A recent (1973) modification to the hydrology of Crawters Brook - the installation of a water retaining structure in a recreation area.

the study period (Figure 4.4.b). It can be seen that the urbanized area is fairly central within the catchment.

As well as covering much of the catchment with impervious material (as shown on Plates 1, 2 and 3), the urban development of Crawley necessitated the installation of a surface water sewer system and the canalization of the main channel. A map of the channel network after urban development is shown in Figure 4.7; some of these alterations took place before the beginning of the period of study.

The point of emergence of the stream from the culvert below the town centre is shown in Plate 4. Here the stream enters a straightened course which is concrete lined in places (see Plates 5 and 6). The earth embankment across the flood plain shown in Plate 6 was constructed after the study period in an attempt to create a small temporary reservoir to hold back water during times of flood.

4.II. AVAILABILITY OF DATA

The data needed for the investigation can be broken down into hydrometeorological data and physical catchment data.

4.II.1. Hydrometeorological data. The distribution of hydrometeorological stations around the catchment is shown in Figure 4.2. The streamflow data extends from 9th September 1952 until the end of the study period, which was taken as 30th September 1972, giving a 20-year period of record. The critical depth flume is situated in the Northgate area of Crawley at Woolborough

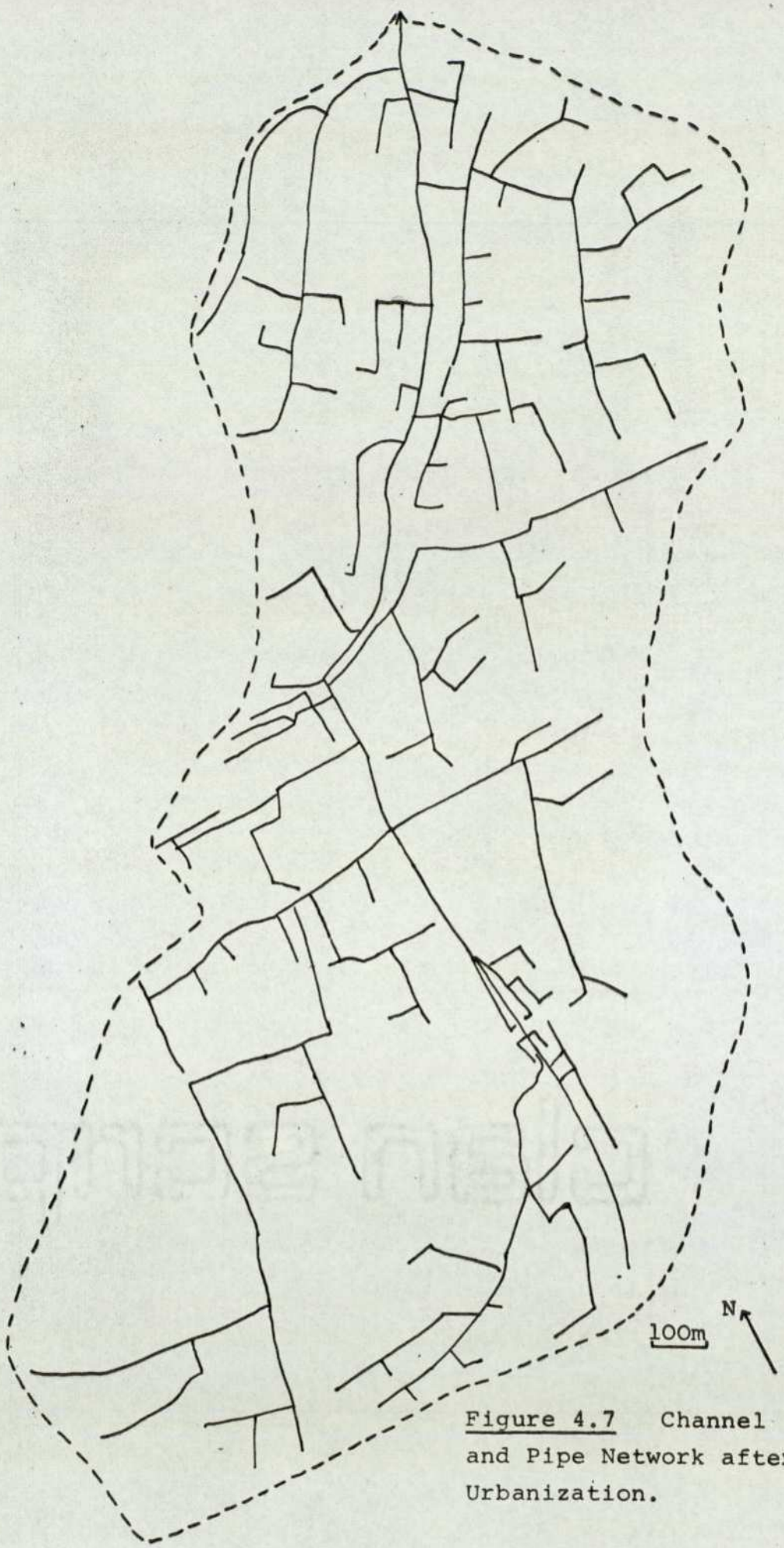


Figure 4.7 Channel and Pipe Network after Urbanization.

Road. It is equipped with a Lea Recorder, giving a continuous record of discharge. The use of a critical depth flume obviates the need for a frequent updating of the stage-discharge relationship. The data consists of weekly charts, but because of the errors discussed in Chapter 2.IV, there are gaps in the record. A chart showing the gaps in the streamflow record is given in Figure 4.8.

Precipitation data for the catchment was required on a continuous basis but unfortunately, there is no autographic raingauge within the catchment. The nearest gauge to the streamflow gauging station is at Tinsley Green, but the gauge at Broadfield is nearer to the centroid of the catchment (see Figure 4.2) and was therefore chosen to represent catchment precipitation. It was also found that when Thiessen polygons (see Wilson, 1969, p.16) were constructed, the catchment was almost entirely within the Broadfield area. Precipitation data used in the simulation of the storms occurring on the catchment was therefore obtained from the Dines Standard Recording Gauge at Broadfield.

The daily precipitation records at Broadfield, Tinsley Green, Gatwick Airport and The Beeches were used to check the consistency of the data. The raingauge at The Beeches is very near to the centroid of the catchment and was used to adjust the data at Broadfield so that the latter was more representative of the catchment as a whole. A double mass plot of accumulated precipitation at The Beeches and at Broadfield (Figure 4.9) confirmed that no changes in the relative amounts of precipitation

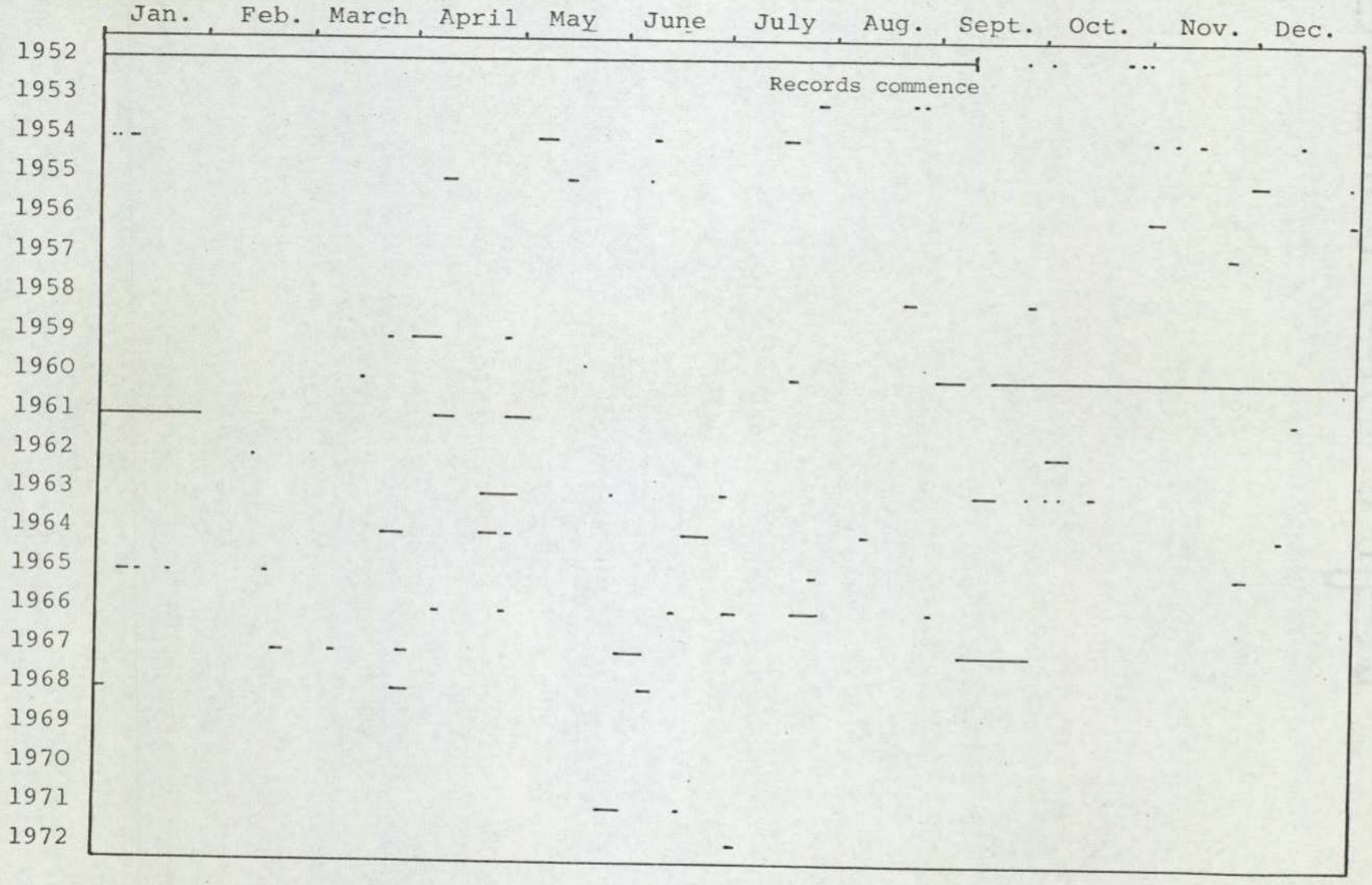


Figure 4.8 Gaps in the Streamflow Record

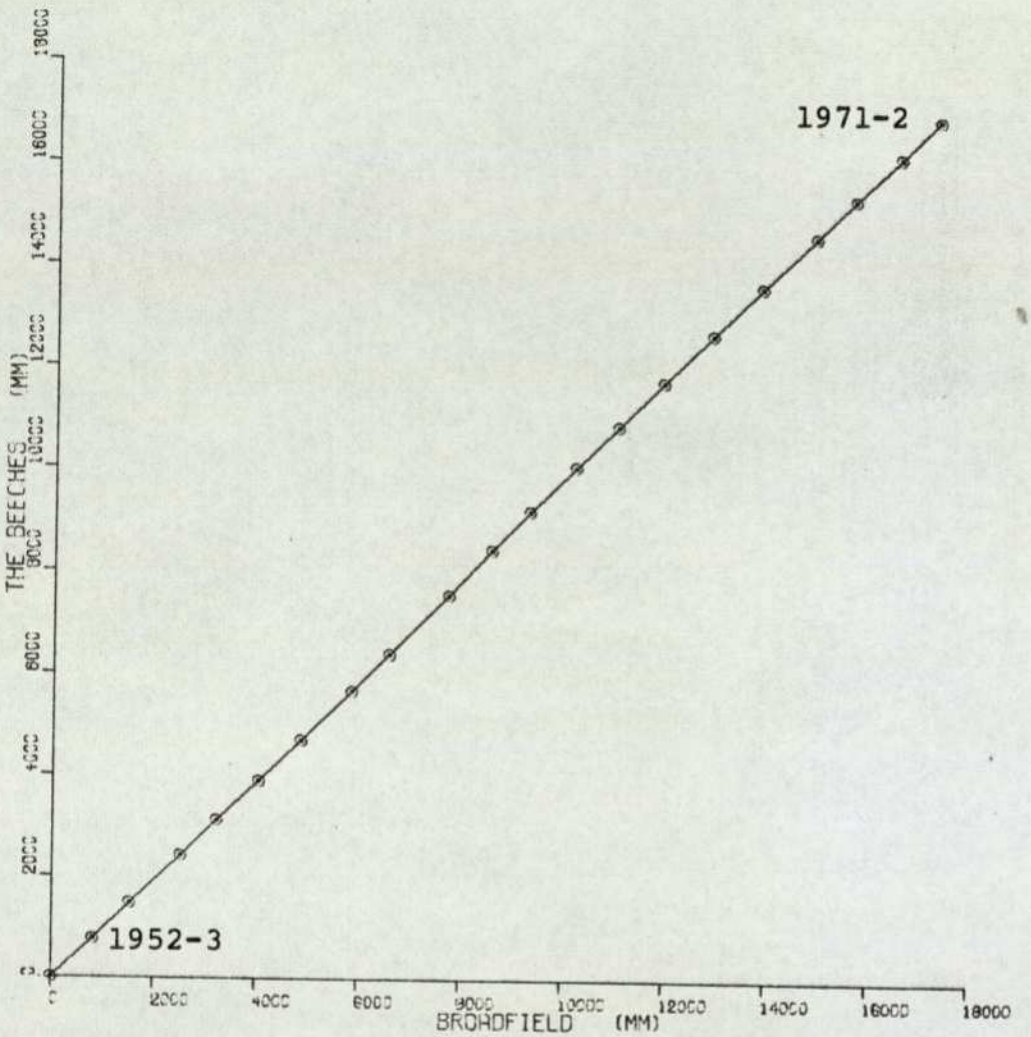


Figure 4.9 Double Mass Plot - Broadfield vs. The Beeches

falling at each gauge had occurred. The daily record at The Beeches was used in the statistical study of the daily flow for the catchment, and a double mass plot against Tinsley Green (Figure 4.10) showed no appreciable change in slope.

Potential Evapotranspiration data was not available for the area, the nearest station being at Kew, about 65km. away. In order to overcome this, data on variables affecting evaporation were collected. These comprised average monthly values of sunshine hours, relative humidity and temperature. Windspeed data was not available. The number of hours of daylight for each day of the year was also calculated. It was not possible to use the records from only one station throughout the study period because the records were discontinuous. The data was therefore collected from several stations and adjustment factors were used to give continuous and consistent records.

4.II.2. Physical Catchment Data. In order to utilize the simulation model to reconstruct past streamflow and to predict future streamflow, values of the model parameters must be found. These comprize infiltration parameters (f_o, f_c and k_f), depression storage (D) and the routing parameter (k_r). To simulate historical storm events, contemporaneous parameter values need to be substituted into the model. As infiltration data was not collected in the past, and depression storage and routing parameters are almost impossible to measure, the parameter values were derived from the historical precipitation and streamflow data using the optimization procedure discussed in Chapter 3.III.3.

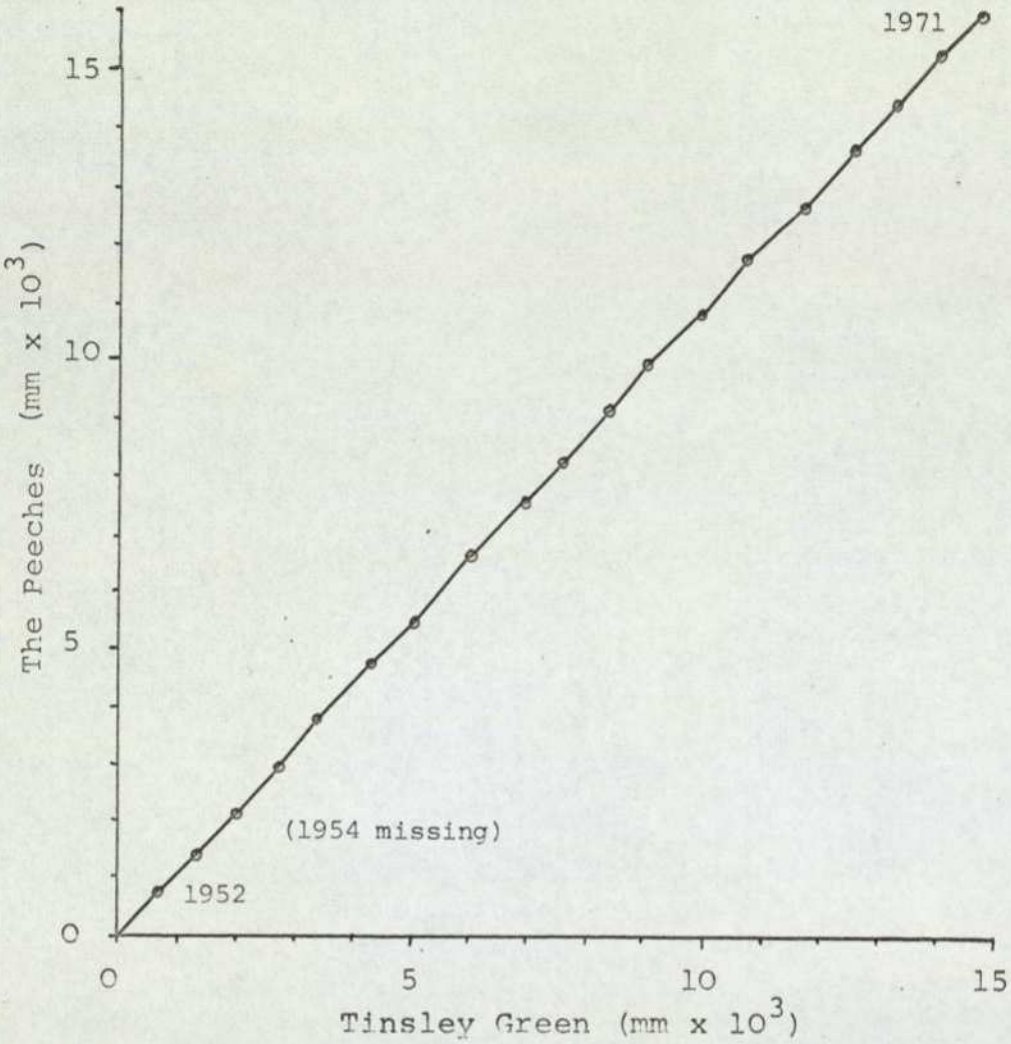


Figure 4.10 Double Mass Plot - The Peeches vs. Tinsley Green.

Correlation of the derived parameter values with contemporaneous hydrometeorological and catchment data in order to use the model as a predictive tool necessitates the collection of data representing physical catchment conditions affecting streamflow. Some of this information, such as catchment area, relative relief, etc. is constant throughout the study period and would not therefore be responsible for variations in parameter values. The most important changes were in the proportion of impervious area and its location within the catchment. This data can be measured in three ways : (1) by reference to aerial photographs of the catchment taken at various times through the study period; (2) by reference to maps drawn at various times through the study period, and (3) by reference to local records of the dates of building construction. The first two methods give impervious area at discrete points in time and interpolation methods must be used. In this study the latter two methods were used as the relevant information was available at Crawley U.D.C. Town Hall. Unfortunately, the spatial distribution of housing was not recorded with sufficient precision to be easily measurable so it was assumed that the distribution at any point in time was uniform throughout the catchment.

A set of 1 : 1250 Ordnance Survey maps covering the area was obtained and the area covered by impervious material was measured using a planimeter. This area consisted of roads, pavements, houses, car-parks, etc., and it was found that the percentage of the catchment covered by impervious material increased from about 5% prior to the building of the new town,

to about 30% in 1972. The Map Room at the British Museum was utilized to assist in tracing the urbanization of the catchment through time. A graph of the percentage of impervious cover in the catchment through time is given in Figure 4.11.

4.III DATA PREPARATION

Before the data can be analysed it must be converted to a form suitable for input to a computer. A flow chart showing the stages of data preparation prior to data analysis is given in Figure 4.12.

4.III.1. Streamflow. The discharge axis on the original weekly charts is divided into three linear stages, recording 0-200, 200-2000, and 2000 to 10,000 cubic feet per minute respectively. Measuring the area under the curve to obtain daily streamflow volumes would therefore be a more time consuming process than it would be if the chart had a single linear axis. Taking 15-minute ordinates of many storms for simulation would also be tedious. It was therefore decided to process the entire 20-year record so that the data would be available for analysis in a computer-compatible form. This was achieved using a digitizer, so that discharge for any selected time interval would be quickly available for use by using a computer programme to interpret the digitized record.

Digitizing the charts entails the punching of discharge-time co-ordinates on computer cards. The frequency of punching with respect to the time axis dictates the amount of detail in the original record retained in the punched cards. If points

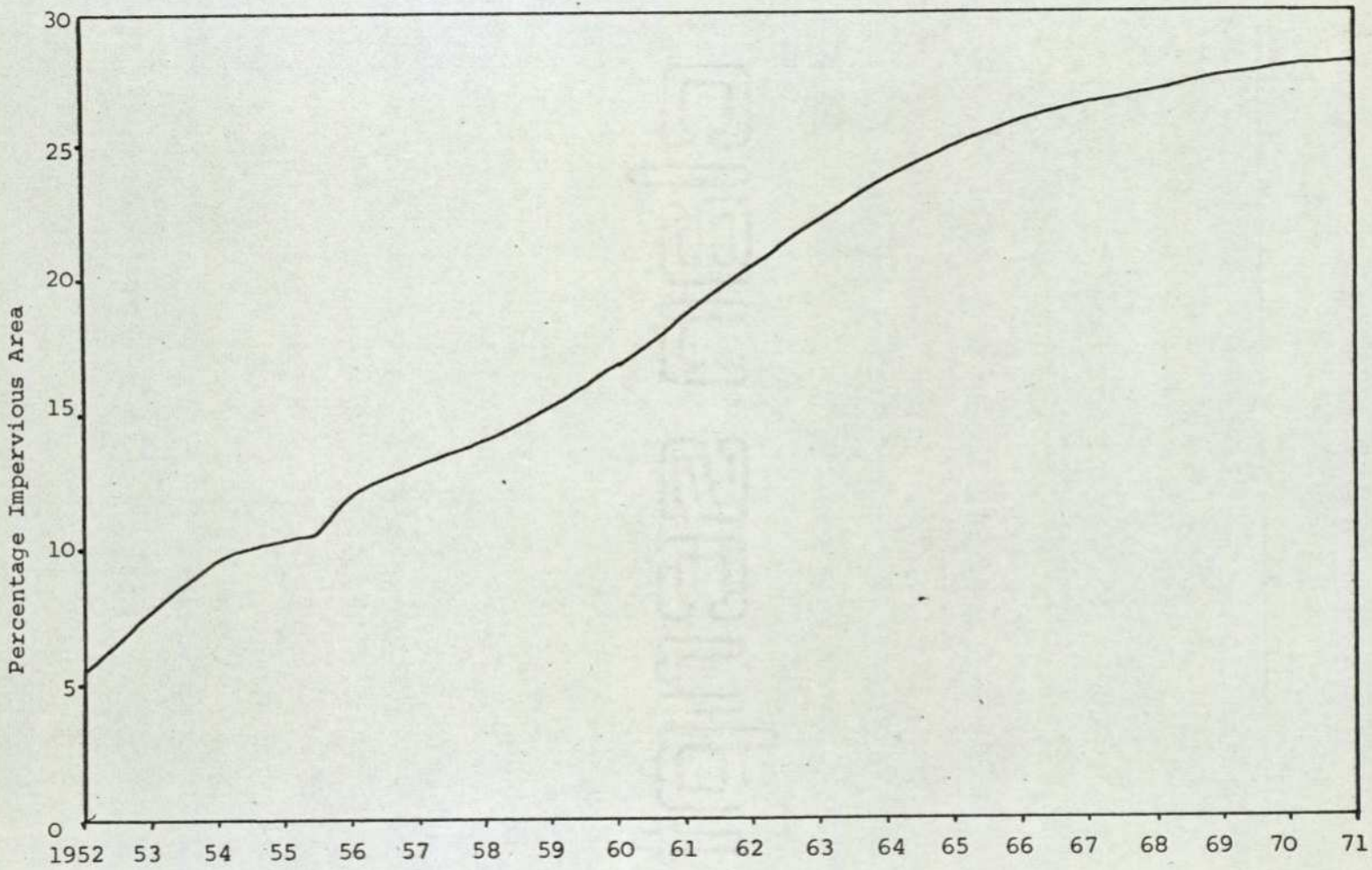


Figure 4.11 Graph of Impervious Area through Time

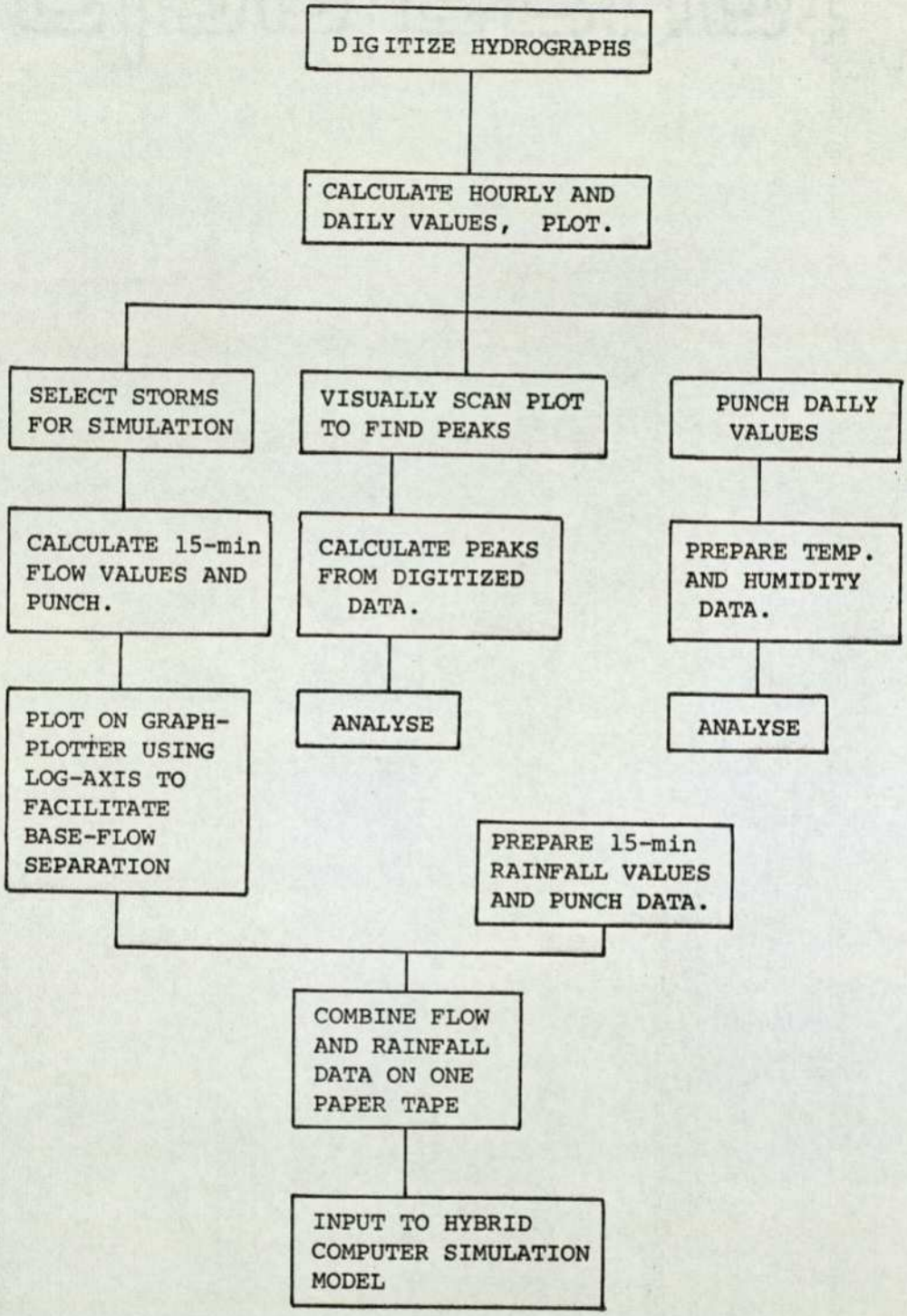
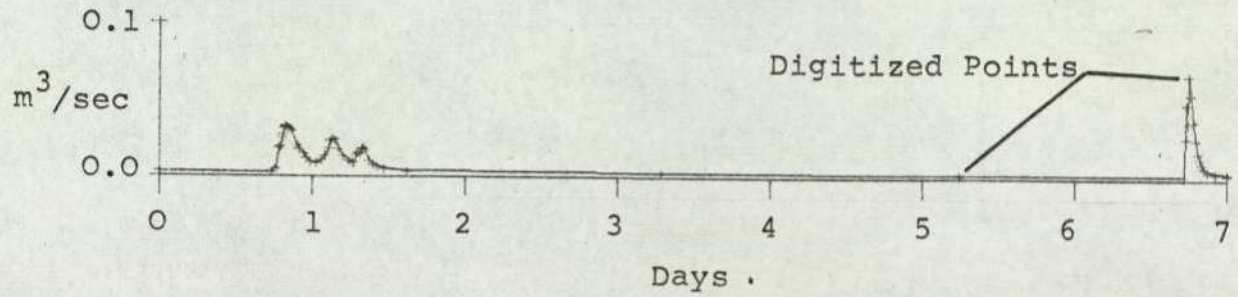


Figure 4.12 Flow Chart of Sequence of Data Preparation and Analysis.

were digitized at regular five-minute intervals, a great deal of detail would be retained at the cost of a vast number of computer cards and time spent digitizing. Conversely, a six-hour interval would miss important parts of many storms and would not give an accurate measure of mean daily flow. Considering these disadvantages of regular sampling it was decided to sample at sufficient points to define rising and falling trends in the hydrograph. A similar procedure was described by Hibbert and Casner (1971), and in their case, all 5-minute data was punched at the gauging station and a computer programme was employed to reduce the redundant data by automatically selecting the best points for retention based on a series of tests.

The type of digitizer used was a D-MAC Pencil Follower Type. It consists of a reading table and an electronic console connected to an IBM card punch. It is a high-speed device capable of converting data from graphical form (the hydrograph) into a form suitable for computer processing (punched cards). The reading table is 100 x 100 cm. and has a servo-positioning mechanism below a plastic-coated surface. This mechanism is accurate to within $\pm$ 0.1 mm. which means that the largest error would be two minutes on the time axis, 0.26 cubic feet per minute on the lower discharge scale, 2.6 c.f.m. on the middle discharge scale, and 26 c.f.m. on the upper discharge scale. A coil in the reading pen radiates a signal which is picked up by coils in the table. The position of the cross-wires on the reading pen is registered in binary coded decimal form which is converted to true decimal form in the electronic console.



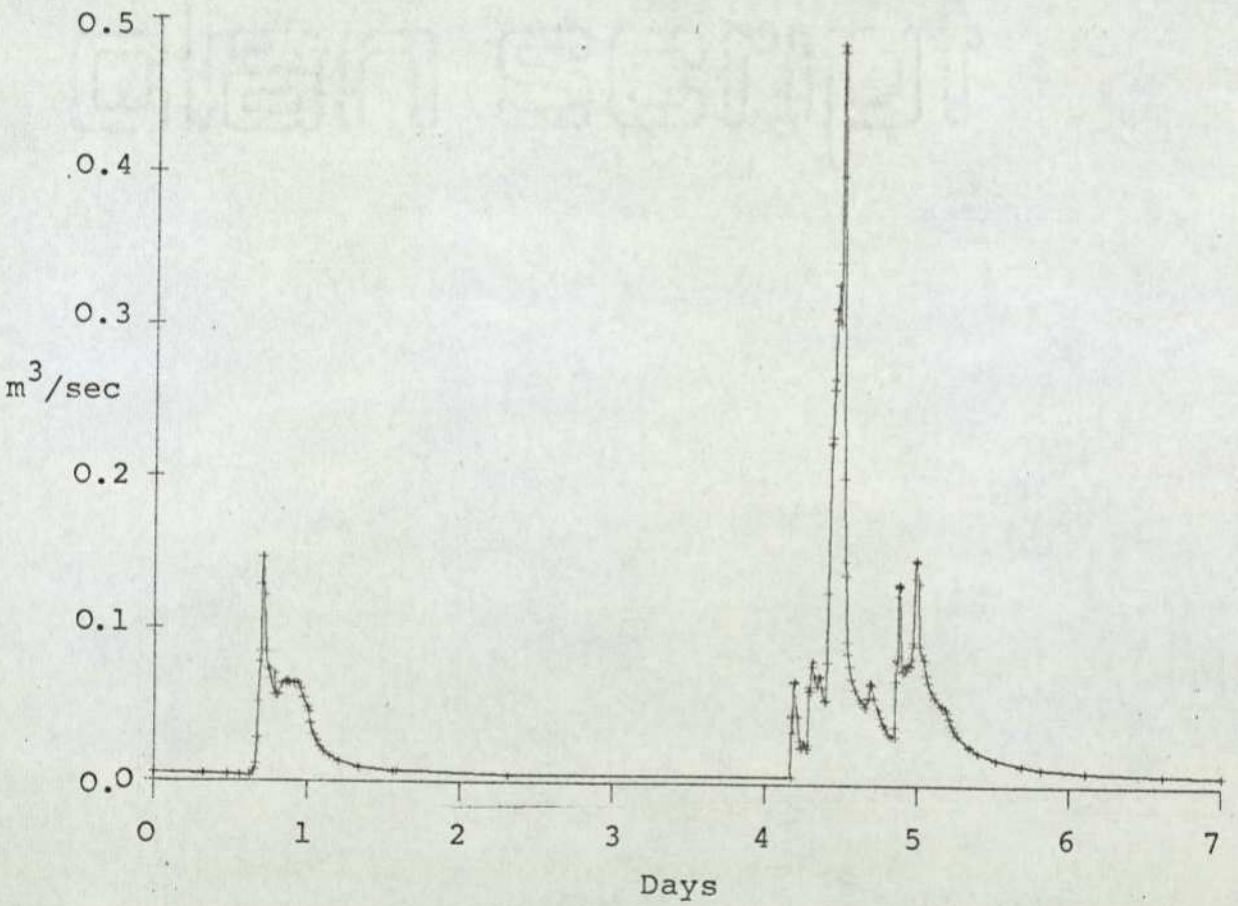


Figure 4.13 Digitizing the Hydrograph Records.

Here it is displayed as two four-digit co-ordinates giving the location of the cross-wires in tenths of a millimetre. The console gives a rapid serial output of the displayed numbers when the readout switch on the side of the reading pen is depressed. These numbers are then automatically punched on computer cards in a pre-determined format.

In practice it was found that about one data point per second could be punched using the digitizer. This compares very favourably with the time taken to read off discharge ordinates by conventional methods. The procedure followed was to tape one or two hydrograph charts to the reading table and to digitize them consecutively. A certain number of control points were then punched, along with a manual punching of the time and date of the chart. This enabled the computer programme to interpret the punched information correctly. Two typical digitized records are shown in Figure 4.13. The following information (see Figure 4.14 for terminology) is coded in a deck of cards for one record chart :

CONTROL CARD

Col.	Description	Name
1 - 3	Chart Number	IPL
5 - 8	Start time	IT
10-15	Date	ID
17-20	Number of Points	NP
22-23	Number of Cards	NC
25-32	Left Corner Co-ordinates	LX, LY

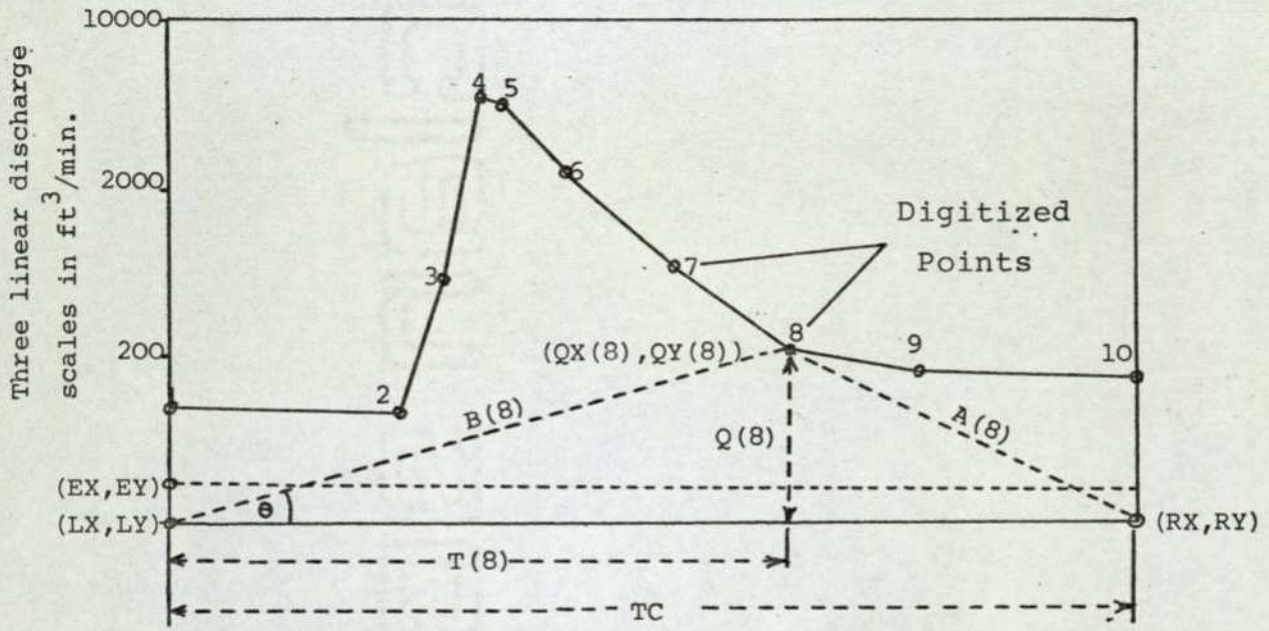


Figure 4.14 Coded Information used to interpret the Digitized Record.

33-40	Right Corner Co-ordinates	RX, RY
41-48	Error Co-ordinates	EX, EY

FIRST DATA CARD (of NP/8 data cards)

1 - 4	First Point, Time Co-ord.	QX(1)
5 - 8	First Point, Discharge Co-ord.	QY(1)
9 -12	Second Point, Time Co-ord.	QX(2)
.		
.		
.		
61-64	Eighth Point, Discharge Co-ord.	QY(8)
65-67	Record Number	IPL
68-69	Card Number	NCP

SECOND DATA CARD

1 - 4	Ninth Point, Time Co-ord.	QX(9)
.		
.		
.		

The basic information needed to reconstruct the original hydrograph is contained on the computer cards. The digitized points corresponding to the original chart are sufficiently close together to define the hydrograph : the more complex the hydrograph, the more points are needed to define it accurately. It was found that a one-week chart required between five and 650 points, with an average of about 100. The EX,EY co-ordinates in the list above correspond to the estimated silting error,

to the error due to incorrect alignment of the chart, or to any other error causing a displacement of the true discharge axis. They were used by the computer programme to adjust the values accordingly. Figure 4.15 gives an example of this, where a storm has caused siltation in the flume at the gauging station. The trace from the time of peak to the clearance of the silt is adjusted by punching the appropriate EX,EY co-ordinates. The computer programme then subtracts the calculated 'error' from the discharge ordinates.

The computer programme (called STREAM) developed to interpret the digitized data is listed in Appendix I. The values of discharge (Q(I)) at each digitized point are calculated and the values at the regular intervals required (hourly or 15-minute) are then interpolated. The calculations proceed as follows :

1. calculate the length of the time axis (TC) :

   $$TC = (RX - LX)^2 + (RY - LY)^2$$

2. calculate the lengths A(I) and B(I) to complete the triangle :

   $$A(I) = (QX(I) - (RX)^2 + (QY(I) - RY)^2$$

   $$B(I) = (QX(I) - LX)^2 + (QY(I) - LY)^2$$

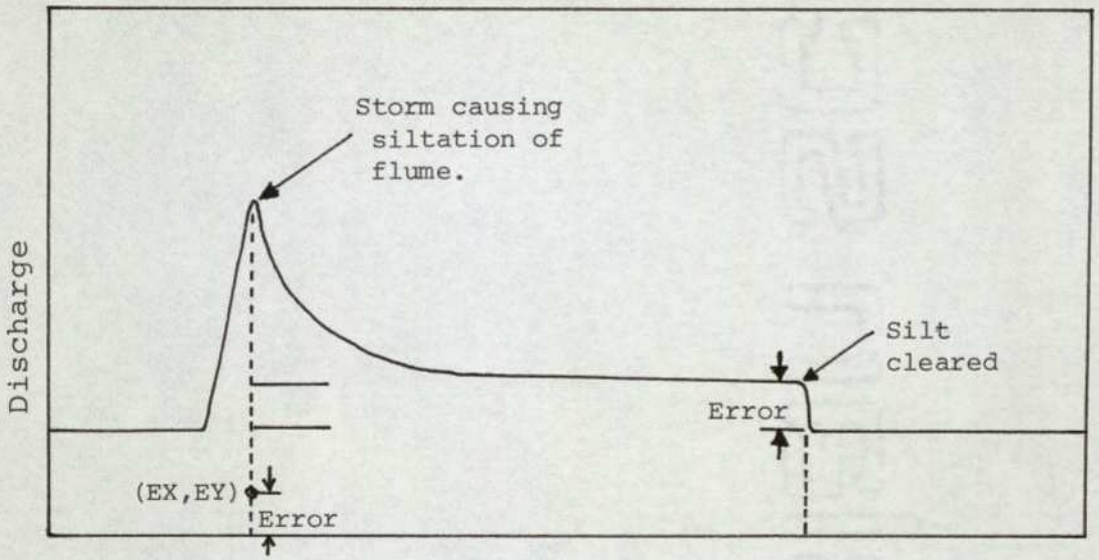


Figure 4.15 Calculation of the Error Co-ordinates.

3. calculate the actual time co-ordinate T(I) :

using the cosine rule,

$$A(I)^2 = TC^2 + B(I)^2 - 2.B(I).TC.\text{Cos}\ \theta$$

As $T(I) = B(I).\text{Cos}\ \theta$

then $$T(I) = B(I).\ \frac{B(I)^2 + TC^2 - A(I)^2}{2.B(I).TC}$$

or $$T(I) = \frac{B(I)^2 + TC^2 - A(I)^2}{2.TC}$$

4. calculate the actual discharge (Q(I)) at time T(I) :

$$Q(I) = B(I)^2 - T(I)^2$$

The three discharge scales in Figure 4.14 were measured from a number of charts and the mean value found. These were then incorporated in the programme with a conversion factor to convert the co-ordinates to units of m^3/sec. The programme prints values of discharge in graphical form as a series of asterisks at intervals of one hour using a logarithmic axis. Mean daily flow is also calculated and printed. An example of programme output is shown in Figure 4.16.

Several other programmes were used to process the streamflow data. CHECK was written to check the digitized data for error, to print the time, date and number of points on each chart and to print the duration of any gaps in the data. An example of printed output is given in Figure 4.17. Program DT15 was adapted from STREAM to punch and print streamflow at 15-minute intervals for selected storms and to plot the data accurately using a Calcomp plotter. This was to prepare data

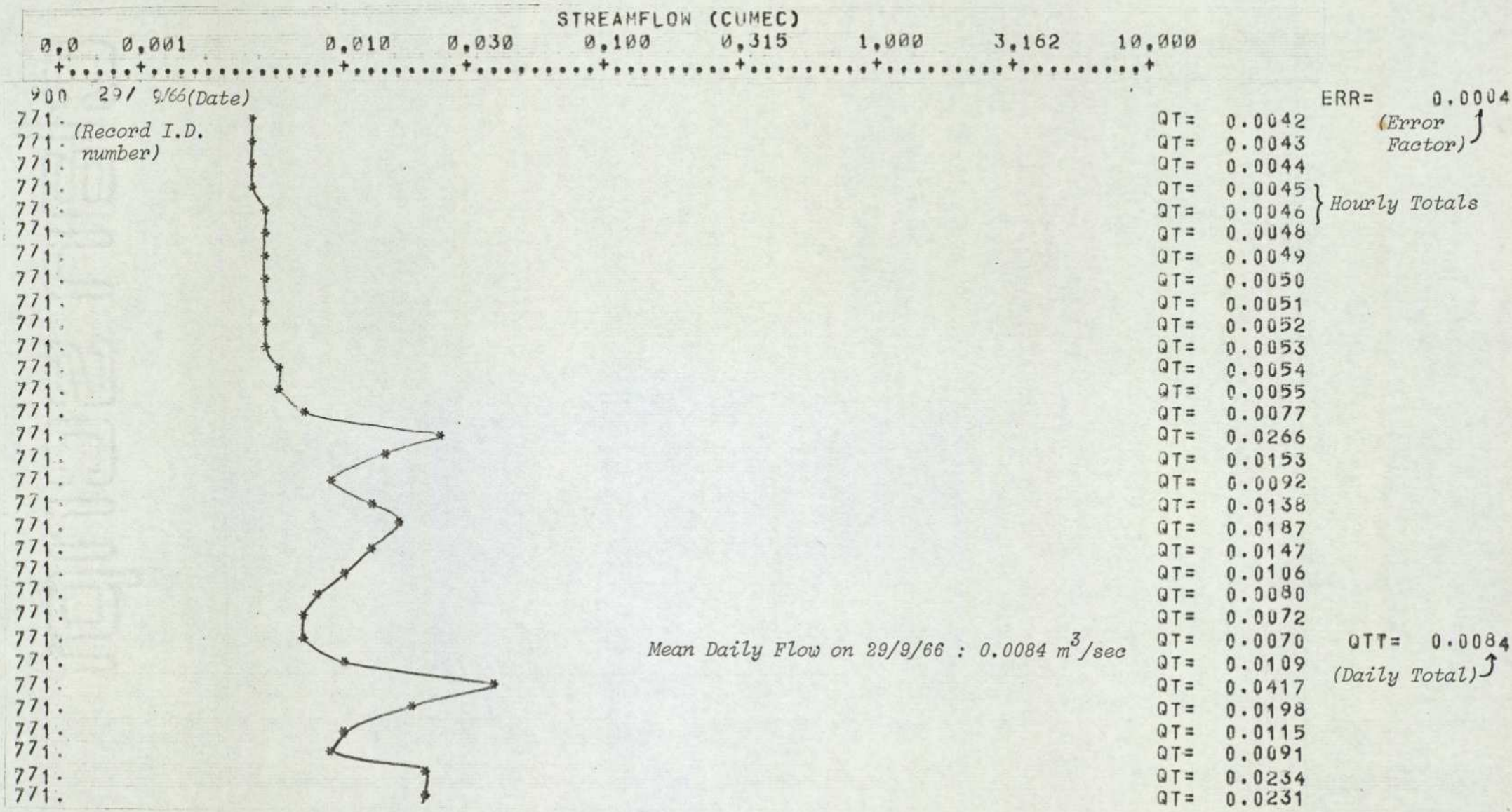


Figure 4.16 Example of Computer Output from Programme to Interpret Digitized Record

```
DOCUMENT CHEK*151

ERR 2     R=169     C= 6        Indicates error in Record 169
ERR 2     R=175     C=20        Card 6

PERIOD TIME    DATE NO.PTS.
  168   0900   040455     25   (number of digitized points)
.....GAP OF   4 DAYS 15.00 HRS
                FROM 1500  070455 TILL 0600 120455
  169   0600   120455     62
  170   0900   180455     30
  171   0900   250455     80
  172   0900   020555    182
  173   0900   095555     49

  etc.
```

Figure 4.17 Example of Computer Output from the Programme used to check the Digitized Data.

for imput to the optimization procedure of the simulation model.

4.III.2. Meteorological Data. Tables of daily rainfall amounts were obtained from the Meteorological Office for the gauges at The Beeches, Broadfield and Tinsley Green. These were used in the analysis of the daily flow on Crawters Brook, with The Beeches being used to represent daily rainfall over the catchment.

Copies of the Dines raingauge charts for Broadfield were obtained for selected storms, including the largest flood in every year. Rainfall amounts at 15-minute intervals were calculated by graphical means rather than by digitization because the amount of data needed was relatively small and did not warrant any extra programming involved. The derived rainfall amounts were punched onto paper tape for input to the hybrid computer simulation model. Variables for the analysis of peak flow magnitudes and for the analysis of model parameter variations were also measured from the charts. These were maximum 15-minute and 30-minute storm rainfall intensities, storm duration and rainfall depth.

Monthly values of temperature, sunshine hours, daylight hours, relative humidity and a daily antecedent precipitation index (API) were punched onto cards for the analysis of daily flow. The API was derived from the rainfall data using the formula :

$$API_t = c.(API_{t-1} + P_t) \qquad (4.i)$$

where API_t = antecedent precipitation index on day t

P_t = precipitation on day t

c = a recession constant between 0 and 1

Thus if no rainfall occurs during several days, the API decreases by amount c each day. This allows for the depletion of moisture in the catchment during dry periods (see Linsley, Kohler and Paulhus, 1958, p.171-172 for a more detailed discussion).

For the analysis of peak flow magnitudes, the API for the previous day of the storm was used and any rain falling on the day of the storm was added to the API if it occurred before the storm to be analysed. This was done to maintain a degree of independence between the API variable and the storm rainfall variables.

The data collected from the study area and prepared on computer cards and tape comprised daily streamflow and precipitation; monthly temperature, sunshine hours and relative humidity interpolated to daily intervals; streamflow and precipitation at 15-minute intervals for selected storms and various statistics describing storm rainfall; the proportion of impervious cover in the catchment interpolated to a daily interval. This data was used (1) to analyse daily flow variations; (2) to analyse the variation in model parameters throughout the study period; and (3) to analyse the factors affecting the number and magnitude of peak flood events.

CHAPTER FIVE

APPLICATION OF THE SIMULATION MODEL

The application and results of the simulation study are presented in this chapter. The practical aspects of the optimization of the model parameters are discussed in the first section. This is followed by an analysis of the response of the model to variations in the model parameters and in the input data. The final section is concerned with the explanation of parameter variation and the use of statistical techniques to enable the simulation model to be used as a predictive tool.

5.I. OPTIMIZATION OF THE MODEL PARAMETERS

The Simplex Direct Search Optimization procedure was used to find sets of model parameter values for a number of storms during the study period.

5.I.1. Selection of storms for simulation. About one hundred storms were arbitrarily selected for simulation, including the storm responsible for the highest peak flow in each year of the study period. However, many of these storms were unsuitable due to errors in the streamflow data (such as siltation) or in the precipitation data (such as a frozen gauge). The final number of storms simulated was thirty-six, representing a variety of hydrological conditions and being spread over the study period.

It was decided to simulate more storms at the beginning of the study period than at the end as this was the period when the percentage of impervious cover on the catchment was changing most rapidly.

As the simulation model was designed to simulate surface runoff only, this had to be separated from baseflow. It is a well-known fact in hydrology that baseflow separation is a highly subjective and artificial practice, but it enables the model to be reasonably simple. If baseflow separation is not carried out, all the processes contributing to the streamflow hydrograph must be simulated. Many methods have been used to separate surface runoff from baseflow; perhaps the most rigorous method was devised by Singh and Stall (1971). This method consists of calculating the recession characteristics of the hydrograph by trial and error, constructing a baseflow hydrograph and subtracting this from the total hydrograph. It was decided that this method would be too time-consuming to apply and that errors in baseflow separation would not give large proportional errors in the derived surface runoff hydrographs for the major storms used in the study. Such a complex method would therefore not be justified.

The method used by Nash (1960) was simple to apply and was used in this study. It consists of plotting the streamflow hydrograph on a logarithmic scale and marking the point where the recession curve becomes a straight line (see Figure 5.1). This is assumed to represent the exponential depletion of subsurface storages, i.e. baseflow. The line joining this

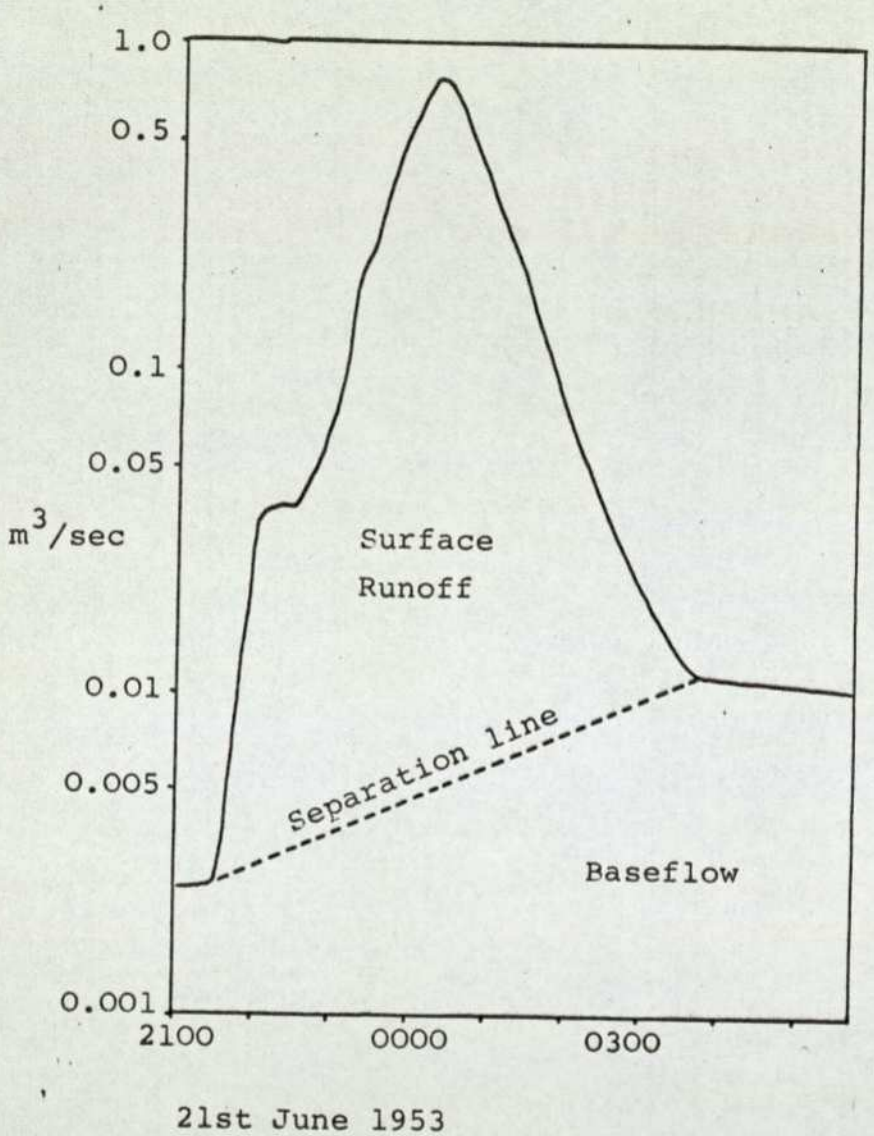


Figure 5.1 Baseflow Separation.

point with the point of rise of the storm hydrograph is taken to be the separation line. A short computer programme (TRAN) was written to perform the calculations and to punch the surface runoff ordinates on paper tape. The corresponding storm rainfall was also punched on the tape, preceded by the date of the storm and the number of data points in the series, for input to the hybrid computer.

5.I.2. Application of the Optimization Procedure. First the hybrid computer was initialized and the output from the components given a 'static check' using the programme 'STAT'. This programme automatically selected the correct time scale on the analogue computer and set the scaling coefficients on the potentiometers. It also printed reminders to check that certain manual operations were performed (e.g. setting the pulse source, closing switches, etc.) A static check was then carried out by automatically feeding known values into the analogue circuit and checking that the printed output values from the various components were correct. This detected any loose wires or erroneous connections in the circuit.

The next step after the static check was to load the optimization programme (MAIN) and its associated subprogrammes into the memory of the computer. The MAIN programme (see Figure 5.2) initializes the hybrid computer system and reads in the first storm to be simulated. Next the scaling factor is typed on the computer keyboard and the MAIN programme scales the data accordingly. This is followed by the input

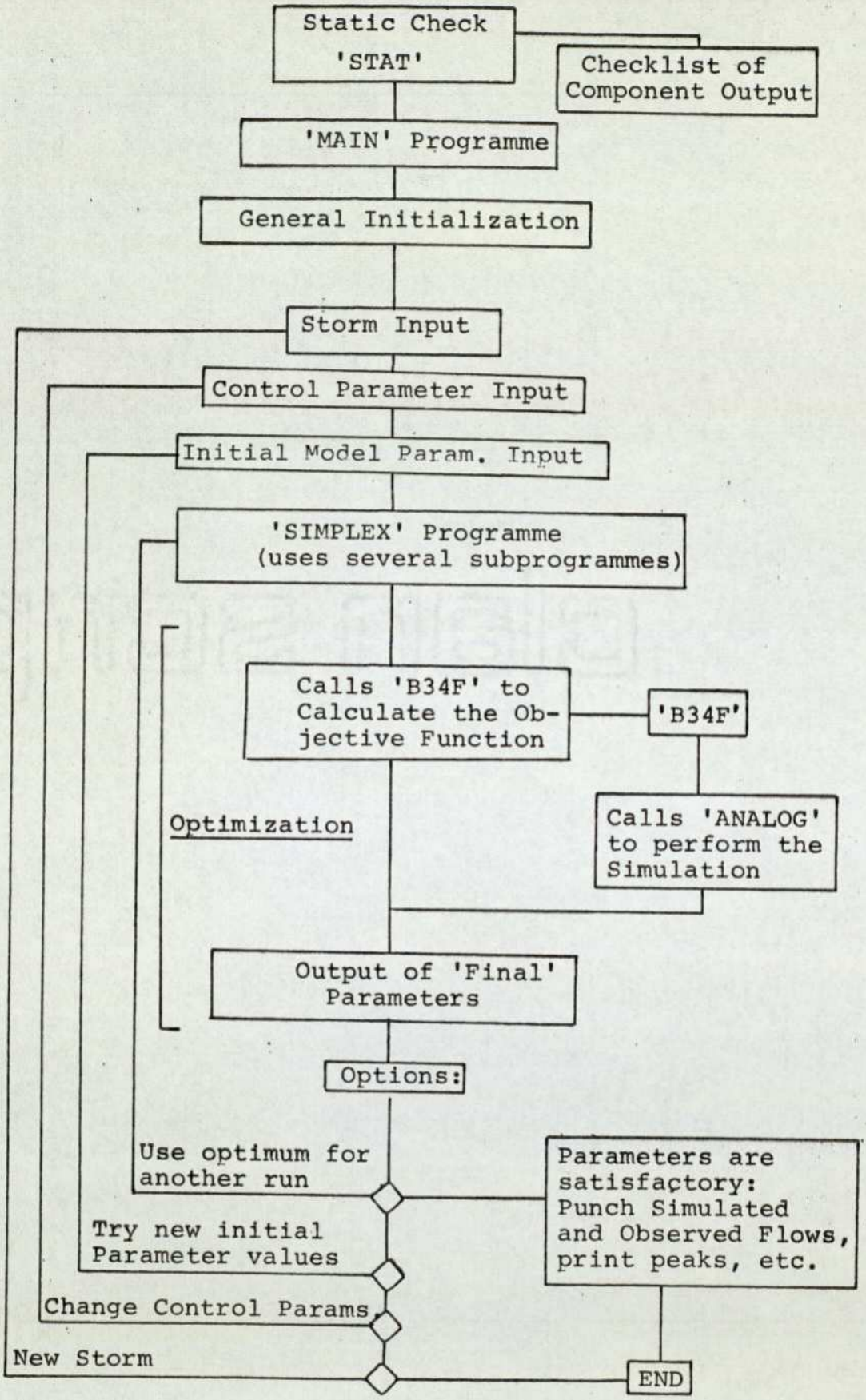


Figure 5.2 Interaction of Computer Programmes during Optimization.

of the control parameters required for the SIMPLEX subprogramme and by the initial parameter estimates for the model. Control then passes to the SIMPLEX subprogramme which performs the optimization of the model parameters.

The objective function (called FUNC in the programme) for each point on the Simplex is calculated by the B34F subprogramme. This is carried out by simulating the storm for each set of parameters using the ANALOG subprogramme and summing the squares of the differences between observed and simulated hydrographs. When the SIMPLEX subprogramme has finally calculated the optimum parameter values, control passes back to 'MAIN' and a number of options can be chosen by the operator by typing the appropriate code number. The first option is to use the optimum parameter values in a new run; the second option allows a new set of initial parameter values to be chosen; the third option allows different control parameters to be selected, the final option is to output the simulated and observed hydrographs, the peak flows and volumes and to start again with a new storm.

By using the hybrid computer, a great deal of flexibility of operation was achieved. The digital programme could be stopped at any point and re-started at the same or another point. The output from any of the analogue computer components could be viewed on the visual display unit and up to four hydrological processes could be viewed on the screen simultaneously. This enabled the operator to see how the optimization was progressing and how the different parameter values affected the hydrological processes represented by the model.

The number of trials required to find the optimum set of parameters varied considerably from storm to storm. This depended mainly on the initial parameter values chosen for the optimization procedure and on the nature of the response surface of the model. This is discussed in the following section. A plot of the objective function value against the number of optimization trials performed is given in Figure 5.3.

5.II. ANALYSIS OF THE MODEL RESPONSE

The response of the model to hydrological data and to various parameter values can be examined in a number of ways : individual parameter sensitivity can be studied by plotting parameter values against the objective function, peak flow or flow volume, and by generating hydrographs for various sets of parameters; interdependence between parameters can be studied by plotting response surfaces.

5.II.1. Parameter Sensitivity. The measure of variation in model output resulting from a change in a parameter of the system is termed parameter sensitivity. It can be used to determine the relative importance of particular types and forms of data and individual processes within the hydrological system (McCuen, 1973). Sensitivity will depend not only on the structure of the model but on the particular period simulated. All the components of the model for which sensitivity is to be studied should come into play. In the following analysis, storms were chosen so that all processes were operative. Plots of parameter values against objective function values for a sample storm

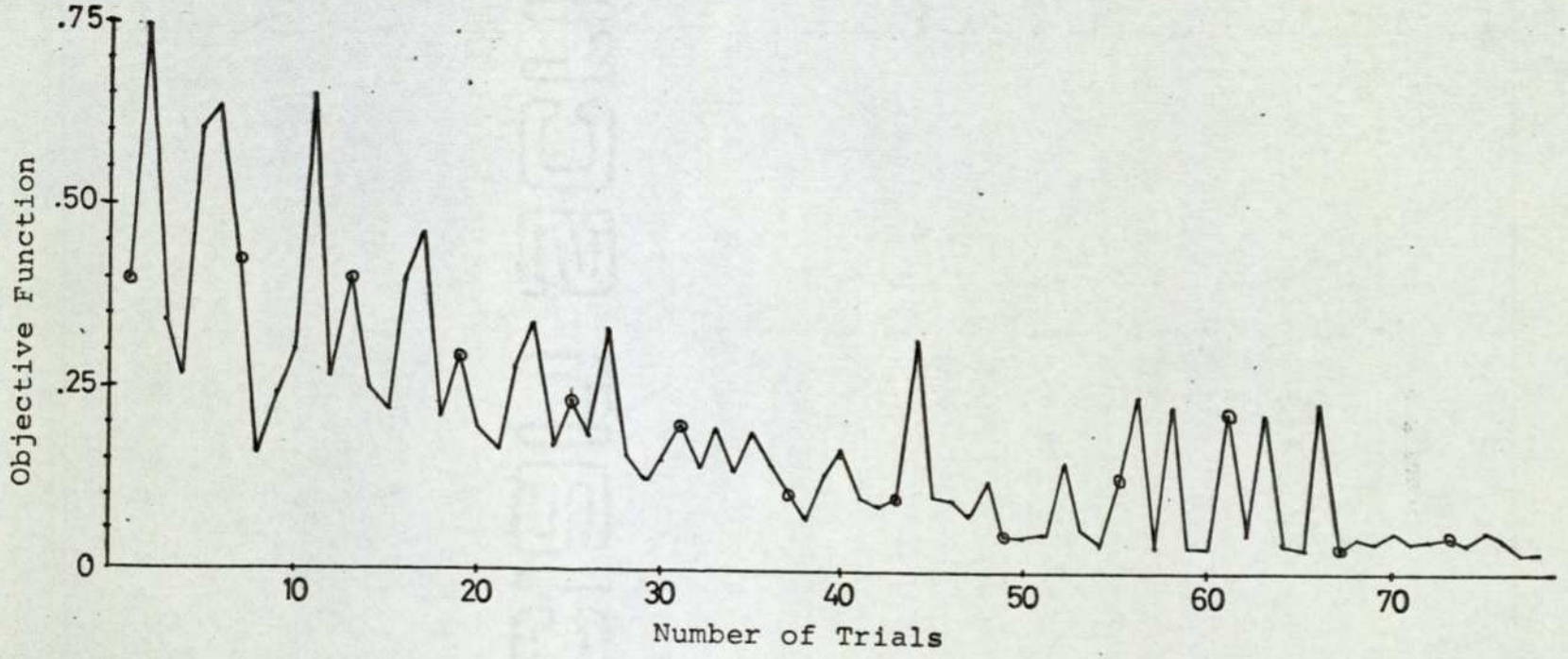


Figure 5.3 Plot of Objective Function for each new Trial vs. Number of Trials.

(21st June 1953) are given in Figure 5.4 to 5.8. The response functions shown were calculated by incrementing a single parameter while the others were kept constant at their optimum values. The system was simulated and the objective function calculated for each increment. The resulting functions were plotted on a Calcomp graph plotter.

In interpreting the response functions, certain considerations must be borne in mind. Firstly, they depend to a large extent on the storm chosen for the simulation and the catchment conditions prevailing at the time. For example, if the storm occurred when the catchment was very wet, the initial infiltration capacity (f_c) would be low and hence different values of the infiltration decay constant (k_f) would make very little difference to the objective function. The parameter k_f would therefore be insensitive in this particular case.

A second consideration is that of scale. The ranges of parameters vary greatly and consequently the parameters must be scaled to be within their realistic range. For example, f_o could be allowed to range up to several thousand millimetres per hour but in practical terms, any value above say 100 mm would result in zero rainfall excess and the parameter would be completely insensitive above this threshold. Limiting the values of certain parameters would therefore allow the optimization procedure to be much more efficient. On the other hand, the parameter f_c has only a limited range and the optimization procedure would have difficulty in obtaining an accurate optimum. In practice this was overcome by multiplying the parameter by a

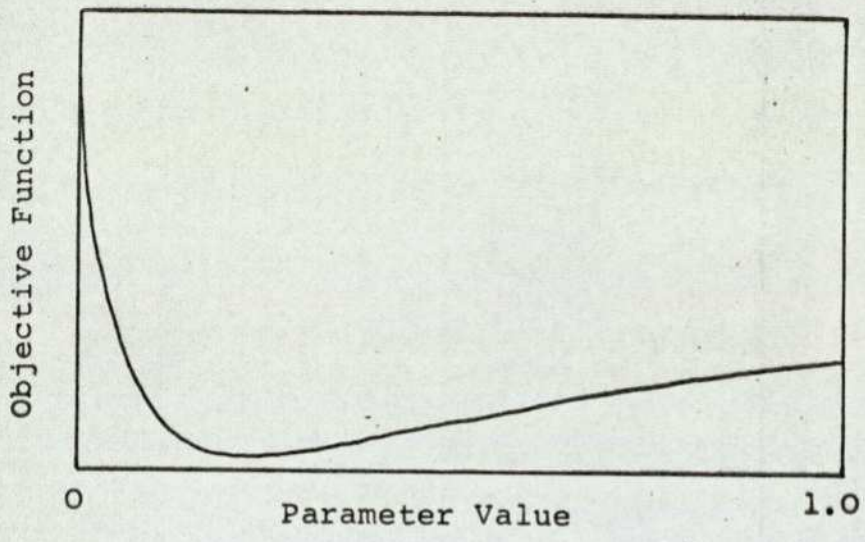


Figure 5.4 Response Function for the Routing Parameter, k_r.

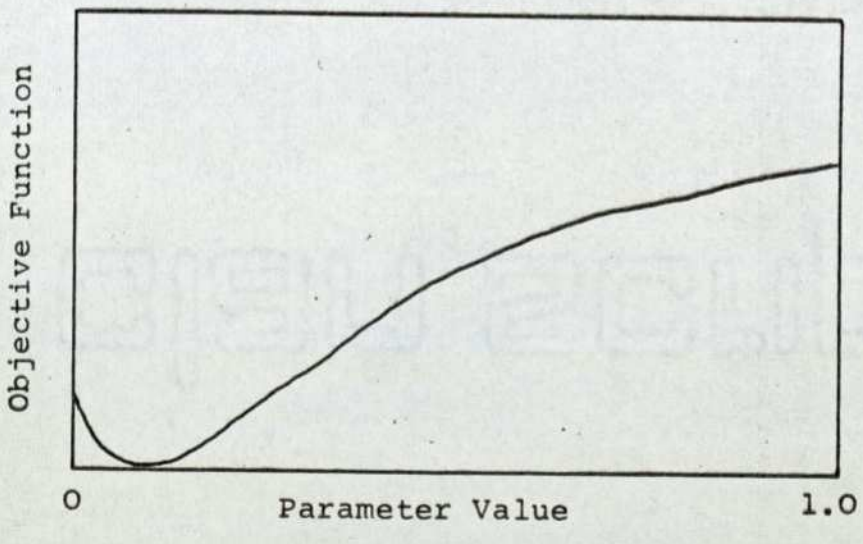


Figure 5.5 Response Function for the Maximum Infiltration Capacity Parameter, f_o.

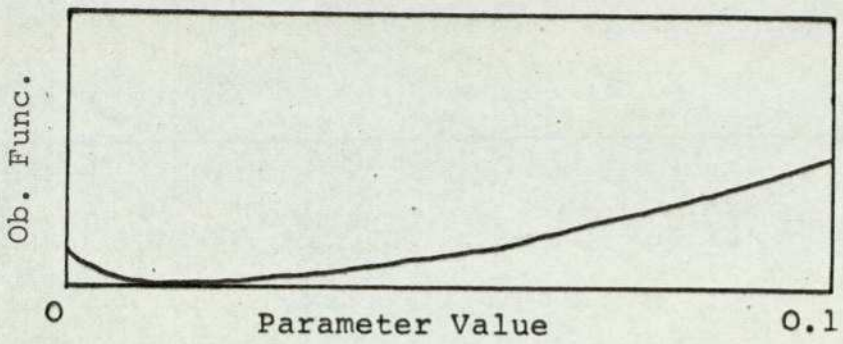


Figure 5.6 Response Function for the Minimum Infiltration Capacity Parameter, f_c.

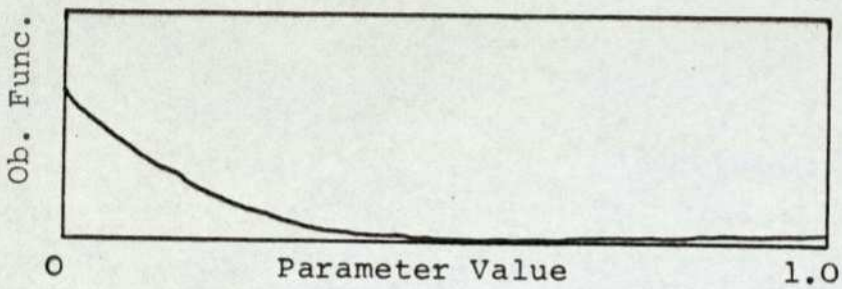


Figure 5.7 Response Function for the Infiltration Capacity Decay Parameter, k_f.

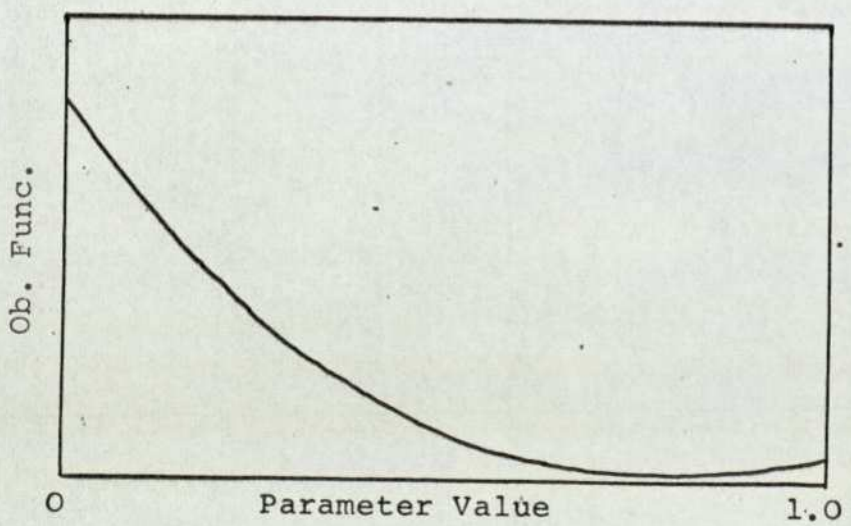


Figure 5.8 Response Function for the Depression Storage Parameter, $1/D$

factor of ten for the purpose of optimization and converting back to the correct units on the analogue circuit.

A third consideration, different from the practical constraints discussed above, is the case of absolute constraints. None of the parameters can have values less than zero. The parameter f_c is also absolutely constrained in the positive direction because it cannot have a value greater than the parameter f_o.

A fourth consideration is the interdependence of parameters. Although the model might appear to be sensitive to variations in a parameter, very similar model output might be obtainable by the variation of another parameter. Single parameter response functions may therefore conceal important correlations between parameters. This is discussed in Section 5.II.2.

These considerations make the direct comparison of the sensitivity of different parameters a difficult task. However, in optimizing the model parameters, knowledge of the typical response functions derived from many storms would be useful. Although sensitivity of model parameters may not be directly comparable, the range over which a parameter has a significant effect on model output and the range over which the parameter is most sensitive would aid both subjective and automatic optimization methods.

The response functions presented give a continuous quantitative measure of the effects of parameter variation on the objective function, but they do not show the effects on hydrograph shape. This can be demonstrated by plotting the simulated

hydrographs generated by successively incrementing individual parameters. Two storms were selected for this analysis, one with multiple peaks (19th September 1953) and one with essentially a single peak (21st June 1953). The simulation parameters were optimized on each storm. The optimum parameter values were then set on the model and each parameter was incremented in turn while the others remained at their optimum values. Figures 5.9 to 5.16 illustrate the results. Parameter k_f was not used in this analysis as it was shown to be fairly insensitive (Figure 5.7) and was assumed to have a constant value throughout the study period.

Figures 5.9 and 5.10 show how the routing parameter (k_r) affects hydrograph shape. Its main effect is to alter the 'Peakedness' of the hydrograph. With a low parameter value, separate peaks on the September storm are not distinguishable. It can be seen that the model is most sensitive to changes in k_r when the parameter values are low. This is confirmed by the response function (Figure 5.4) which shows a steep gradient for low parameter values.

The variations in hydrograph shape attributable to the initial infiltration capacity (f_o) parameter are shown in Figures 5.11 and 5.12. The parameter has a strong influence on flow volumes and on the duration of flow. Comparison with 5.15 and5.16 showing the effects of the depression storage capacity parameter (D) indicates that although both parameters control flow volume, they do so in different ways. Low values of f_o cause flow to occur earlier than low values of D because

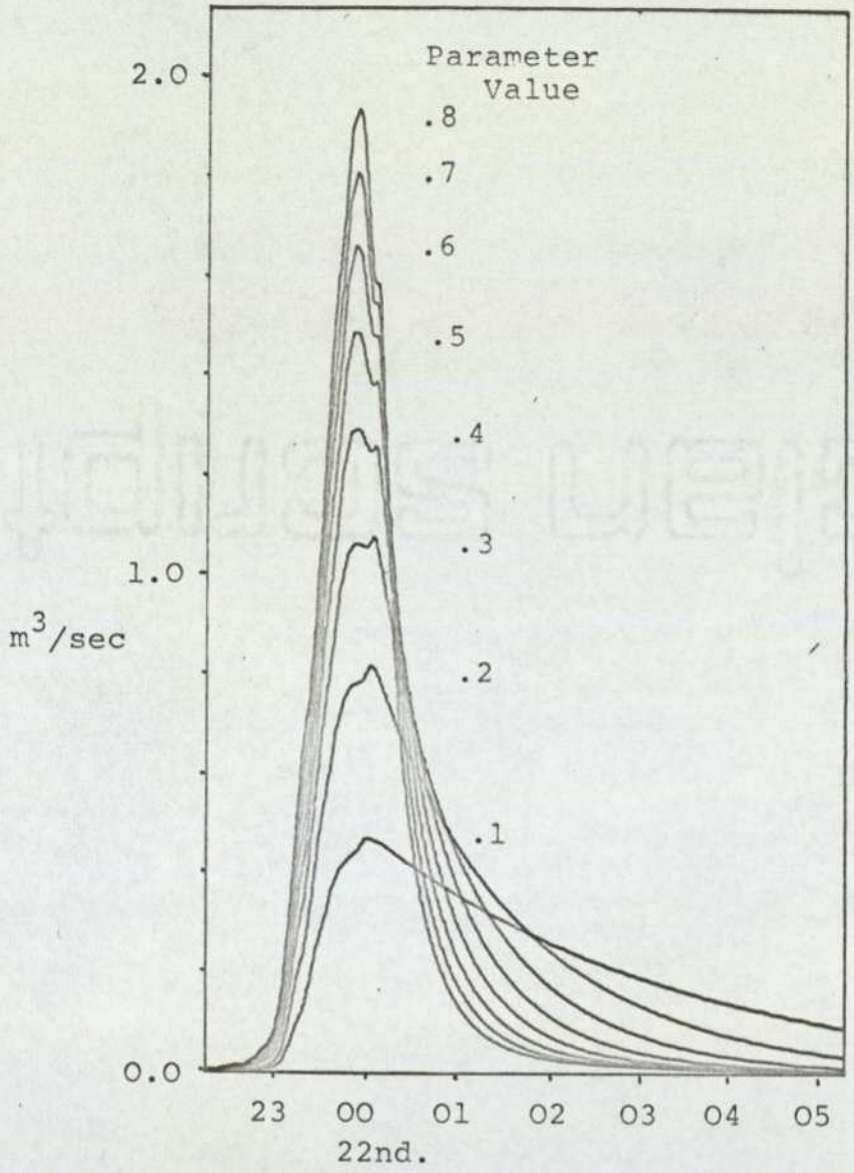


Figure 5.9 Sensitivity of the Routing Parameter k_r using the June 1953 Storm.

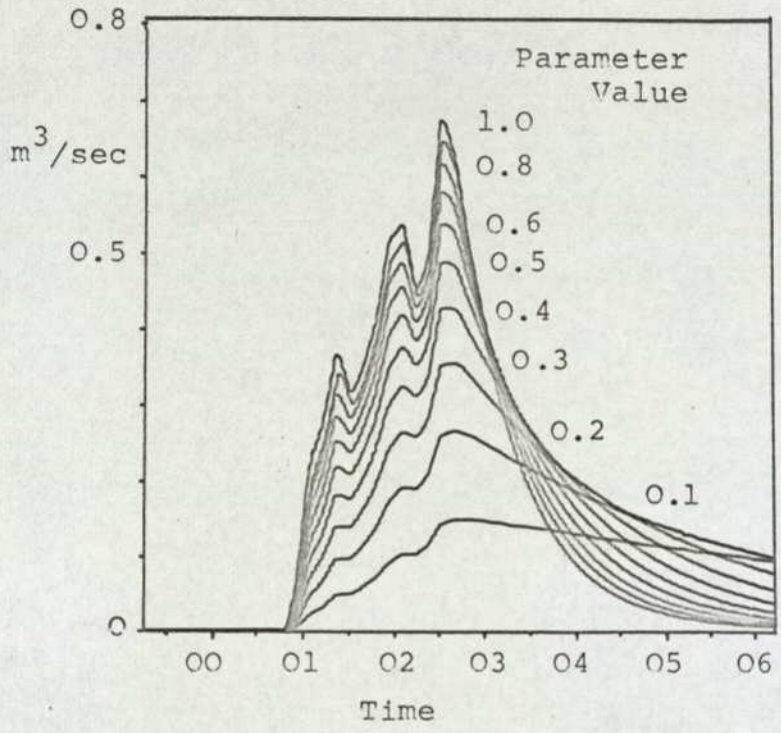


Figure 5.10 Sensitivity of the Routing Parameter, k_r using the September 1953 Storm.

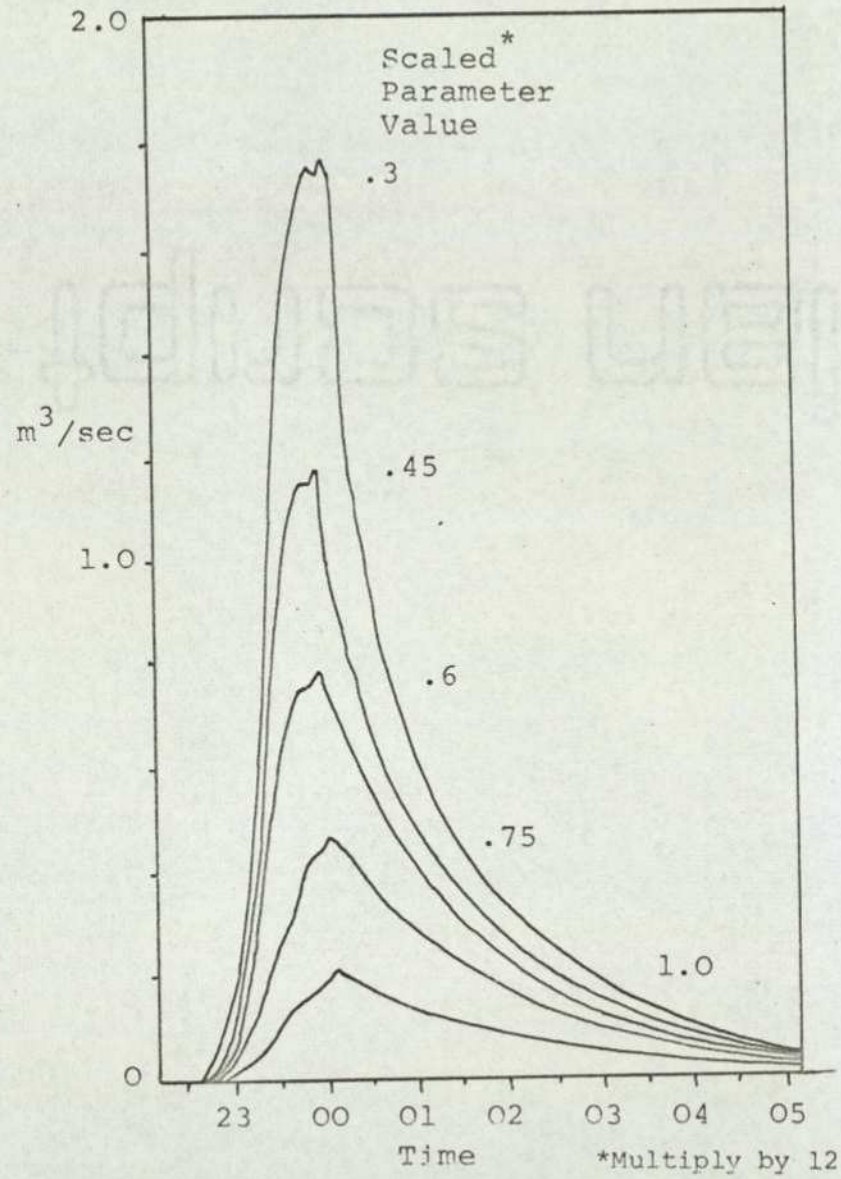


Figure 5.11 Sensitivity of the Maximum Infiltration Capacity Parameter, f_o using the June 1953 Storm.

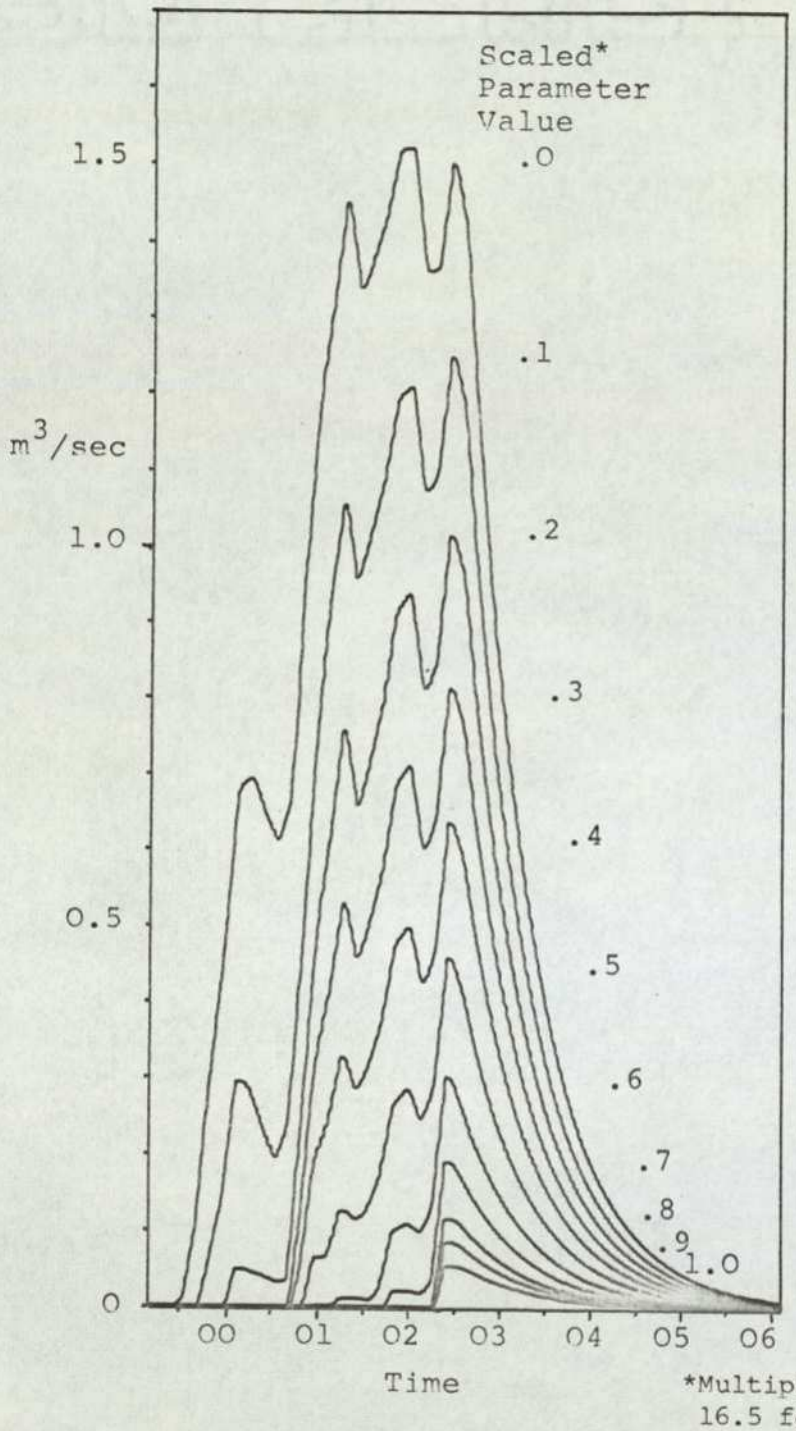


Figure 5.12 Sensitivity of the Maximum Infiltration Capacity Parameter, f_o using the September 1953 Storm.

the infiltration component is encountered before the depression storage component and water is able to move more rapidly to the subsequent components. The sensitivity of the model is greater for low values of f_o than for high values, giving rise to large volumes of runoff, but the response function (Figure 5.5) indicates that values of the objective function are much higher for high f_o values. This is due to the observed flow being very much higher than the simulated flow.

The minimum infiltration capacity (f_c) parameter has a similar effect on the hydrographs but is more limited in its range (Figures 5.13 and 5.14). Unlike the initial infiltration parameter, f_c is unable to effect a large increase in flow volume.

The depression storage capacity (D) parameter has an important influence on flow volumes as shown in Figures 5.15 and 5.16. It should also be recalled that the parameter affects the speed with which depression storage is filled because it is also present in Equation 3.xii as an exponent. A small value of D represents a small depression storage capacity which is filled relatively rapidly. The flow consequently rises quickly. In the September storm (Figure 5.16) it can be seen that the early part becomes increasingly dominant as depression storage capacity decreases. For low levels of depression storage, variations in the parameter do not have much effect on the hydrograph.

The effects of parameter variation on flow peaks and volumes can be quantified in a similar manner to the response functions. These plots would aid in the modification of a simulation model

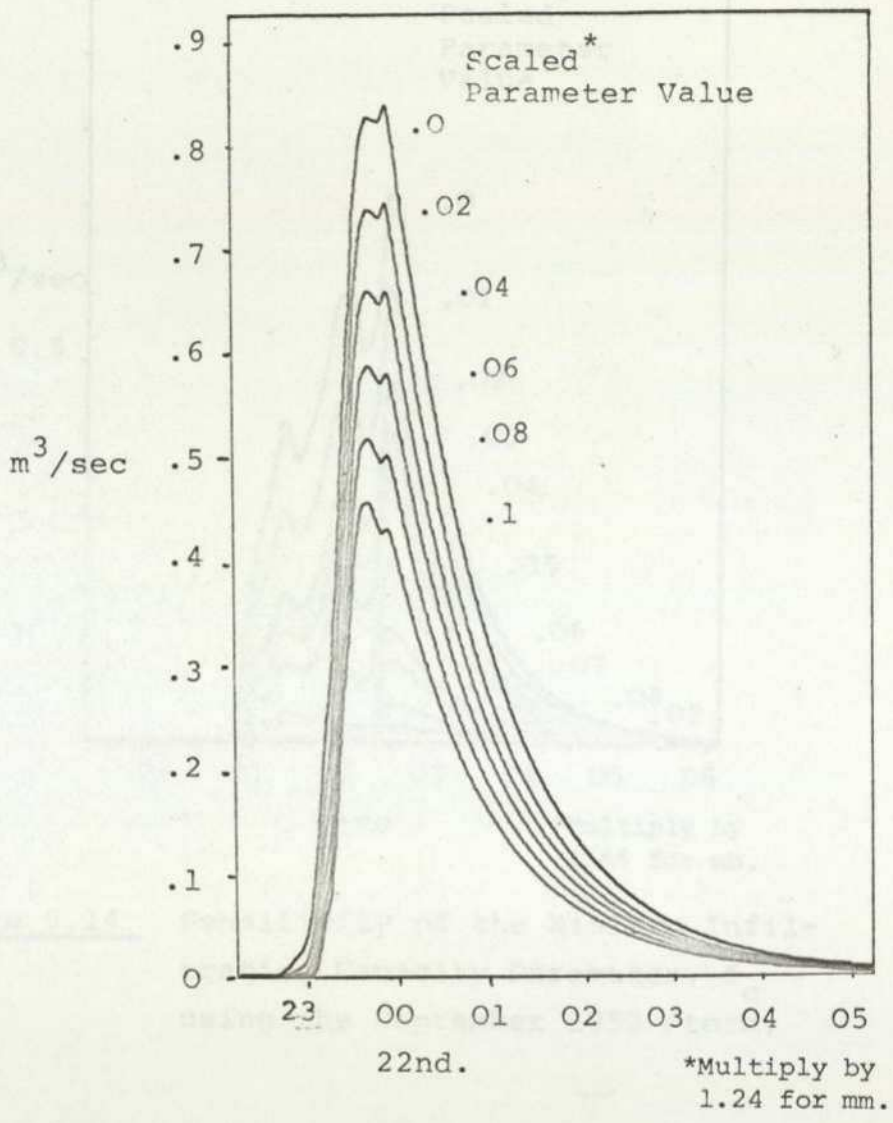


Figure 5.13 Sensitivity of the Minimum Infiltration Capacity Parameter, f_c using the June 1953 Storm.

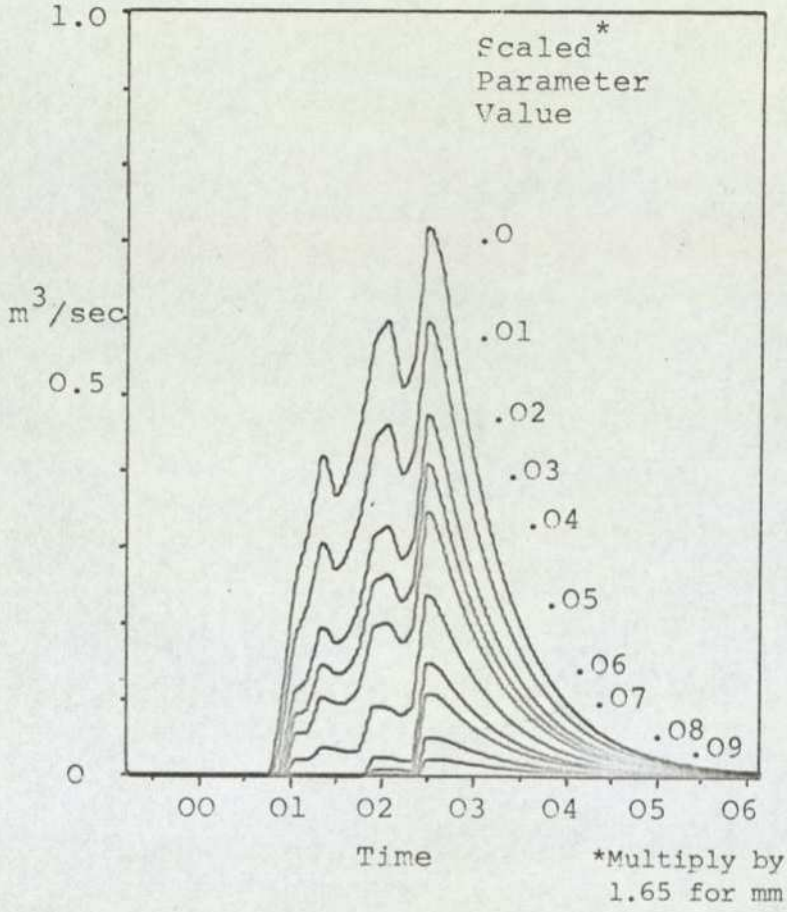


Figure 5.14 Sensitivity of the Minimum Infiltration Capacity Parameter, f_c using the September 1953 Storm.

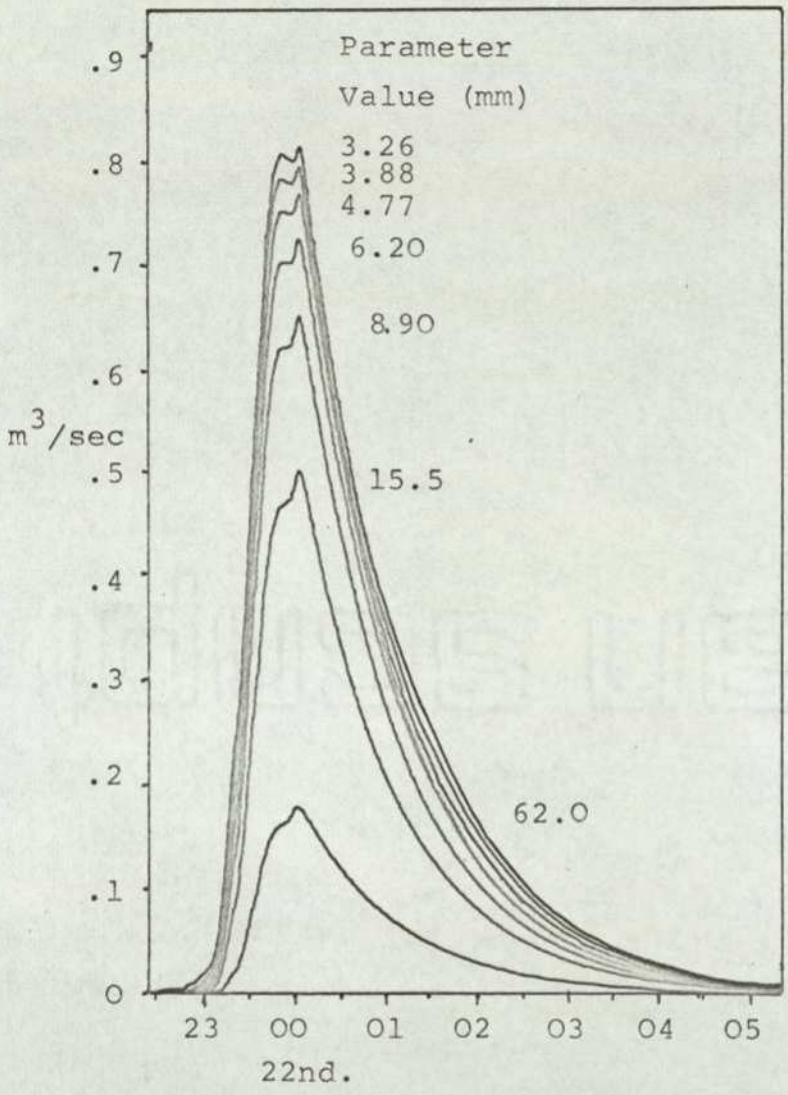


Figure 5.15 Sensitivity of the Depression Storage Capacity Parameter D using the June 1953 Storm.

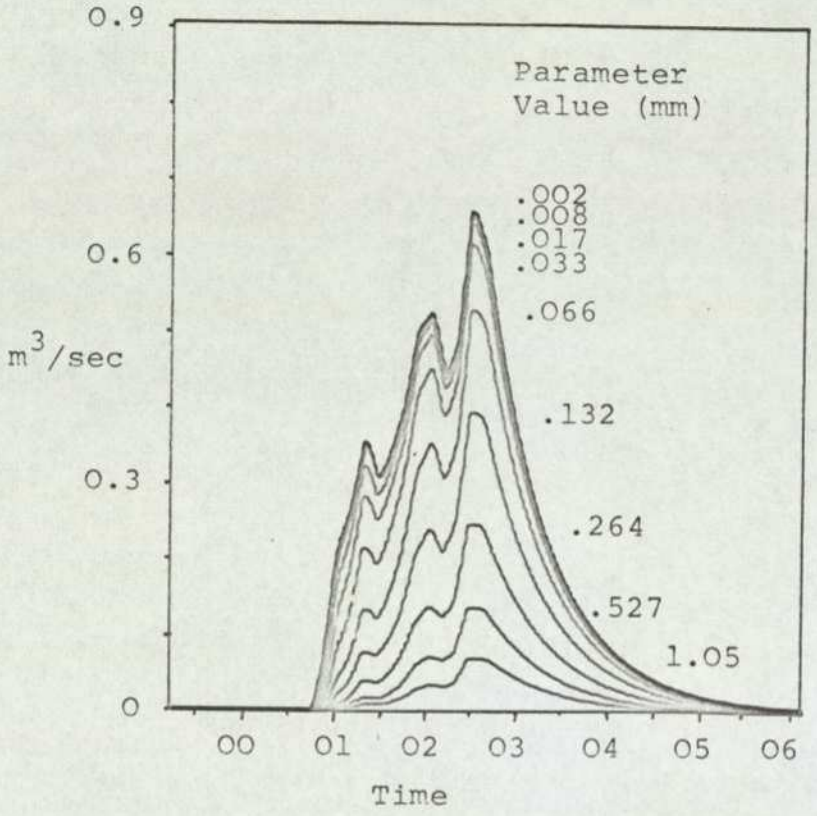


Figure 5.16 Sensitivity of the Depression Storage Capacity Parameter D using the September 1953 Storm.

according to the purpose for which it was built, enabling parameters to be deleted which have insignificant effects on the variable of interest (such as peak flow in flood studies). The plots in Figures 5.17 to 5.20 illustrate the effect of varying the parameter values by plotting the value against magnitude of peak and volume.

It can be concluded that sensitivity analysis is useful in demonstrating the ability of a parameter to change the streamflow hydrograph and to give an indication of the type of change that would occur. This information is useful in determining redundant parameters and processes, that is, those which have little effect on model output and which perform operations having identical effects to other parameters. With reference to the hybrid computer simulation model, it was concluded that the parameters were sufficiently sensitive in their effects to justify inclusion in the model, with the exception of the infiltration recession parameter k_f. As mentioned previously, this parameter was fairly insensitive, and it was therefore assigned a constant value throughout the study. This value (0.36) was derived from the optimization of several storms.

5.II.2. Response surfaces. Two-dimensional representations of the values of the objective function corresponding to variations in two parameters are known as response surfaces. The response functions discussed earlier can be viewed as 'Cross-sections' of the response surface, with the other parameter held constant at its optimized value.

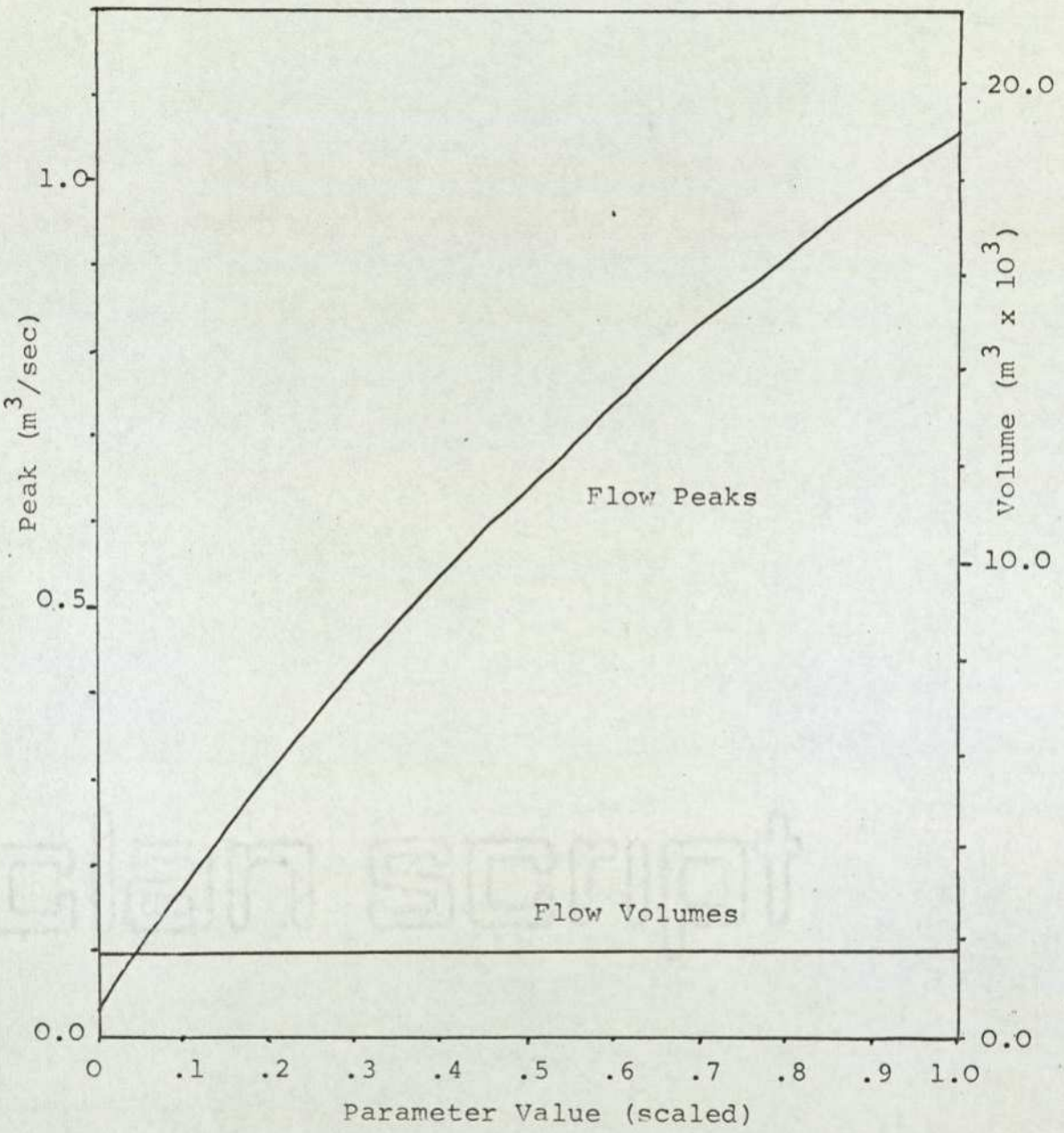


Figure 5.17 Example of the Effect of the Routing Parameter, k_r on Flow Peaks and Volumes.

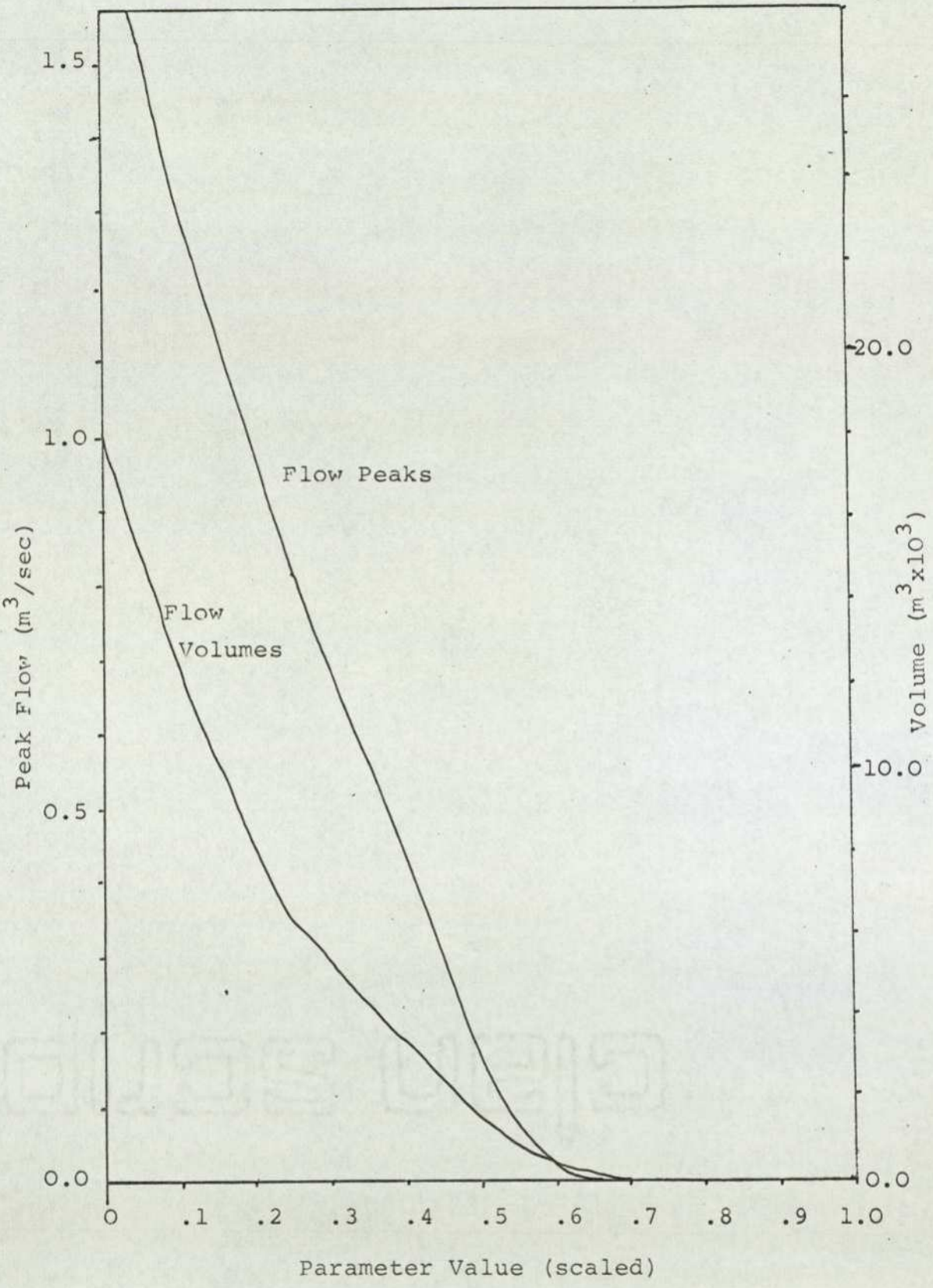


Figure 5.18 Example of the Effect of the Maximum Infiltration Capacity Parameter, f_o on Flow Peaks and Volumes.

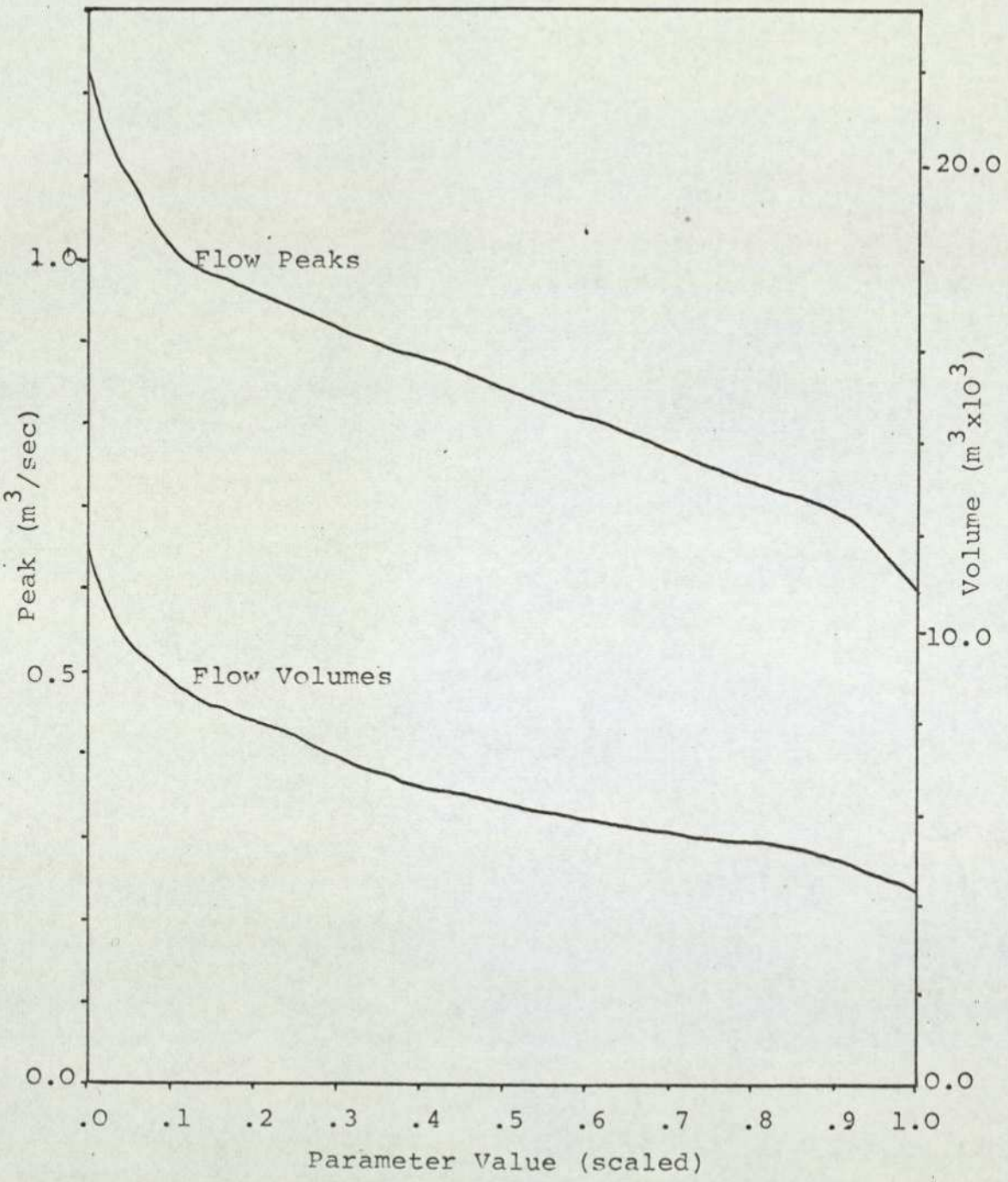


Figure 5.19 Example of the Effect of the Minimum Infiltration Capacity Parameter, f_c on Flow Peaks and Volumes.

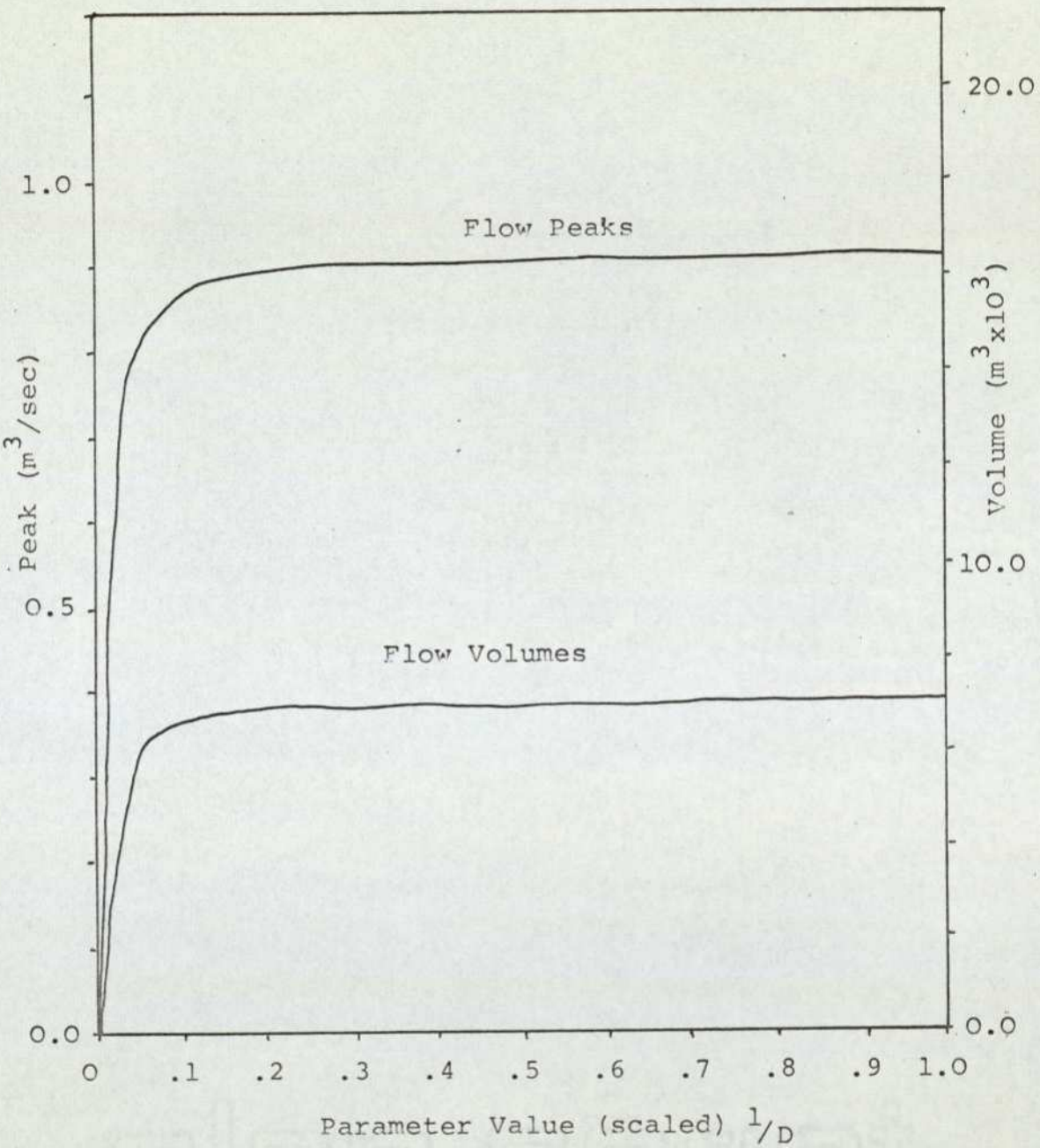


Figure 5.20 Example of the Effect of the Depression Storage Capacity Parameter, D on Flow Peaks and Volumes.

Response surfaces have several important uses in hydrological modelling. They allow investigation into the problems besetting automatic optimization procedures (Ibbitt, 1970, p.41). These include the problems of multiple optima, threshold values, parameter correlation and parameter sensitivity. Figures 5.21 to 5.30 illustrate the response surfaces calculated by the programme RESP. This programme systematically incremented each of the two parameters, calculated the resulting objective functions and punched the matrix of values onto paper tape. This was then used to plot the contours of the response surface on a Calcomp graph plotter. The storm used in the example was the 21st June 1953 storm. Table 5.1 gives a key to the figures for ease of reference and a list of the contour levels plotted.

It can be seen from the plots that multiple optima were not present on the response surfaces for this particular storm. Shaded areas represent 'plateaux', that is, areas where the objective function is constant. These are caused by parameters having threshold values, a fairly common occurrence in hydrological models. Figure 5.26 shows the response surface for variations in the two infiltration capacity parameters. Both these parameters are capable of causing a threshold such that no excess precipitation is generated. This occurs when their values are high. Similarly, when the inverse of the depression storage capacity parameter (1/D) is very small, depression storage capacity is very large and no water reaches the routing component. This is indicated by the shaded areas on Figures 5.22, 5.25, 5.28 and 5.29. Fortunately, these areas can be avoided

Contour Levels Plotted

(values of the Objective Function)
2.00
1.50
1.00
.750
.500
.400
.300
.250
.200
.150
.100
.075
.050
.025 (encircles optimum)

Figure Numbers

	f_o	l/D	f_c	k_f
k_r	5.21	5.22	5.23	5.24
f_o		5.25	5.26	5.27
l/D			5.28	5.29
f_c				5.30

Table 5.1 Key to Figures 5.21 to 5.30 illustrating response surfaces.

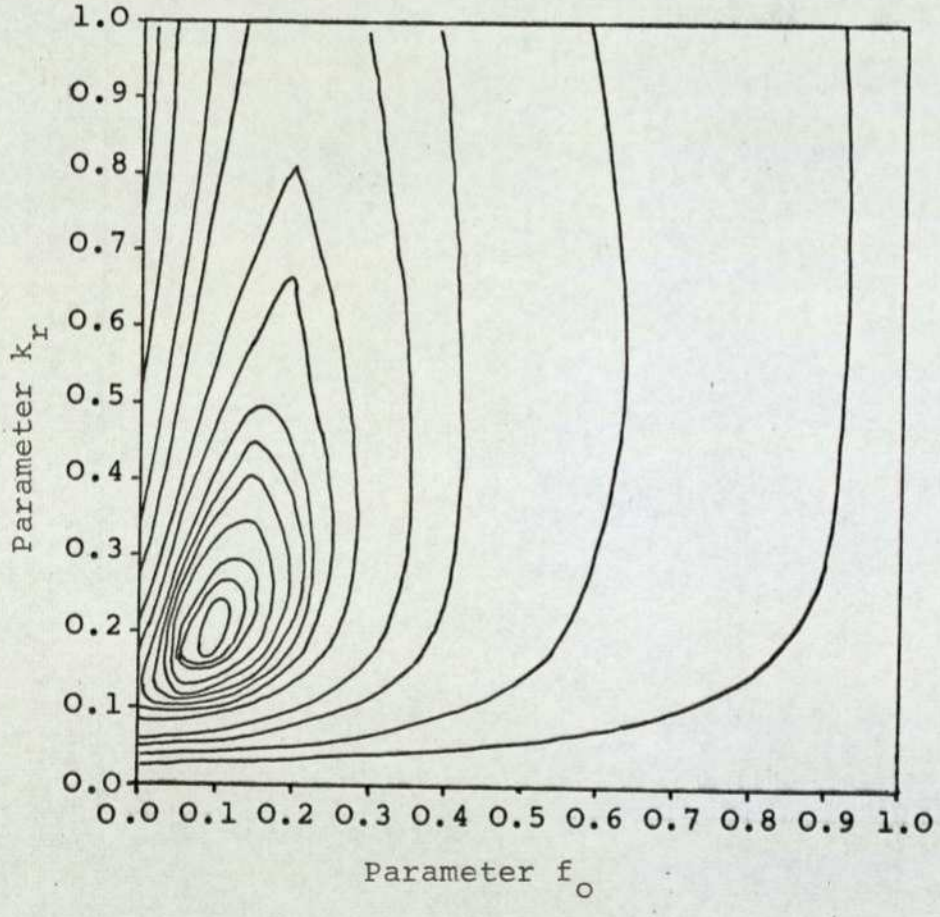


Figure 5.21 Response Surface for the Routing Parameter, k_r and the Maximum Infiltration Capacity Parameter, f_o.

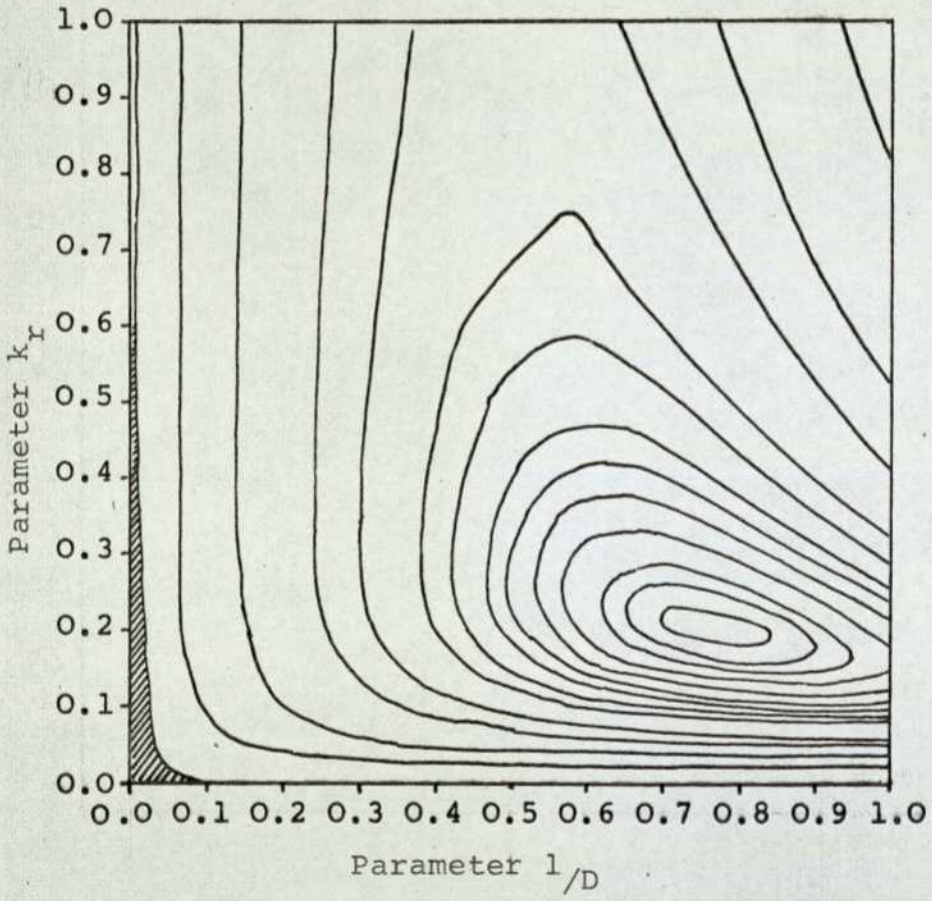


Figure 5.22 Response Surface for the Routing Parameter, k_r and the Depression Storage Capacity Parameter, $1/D$

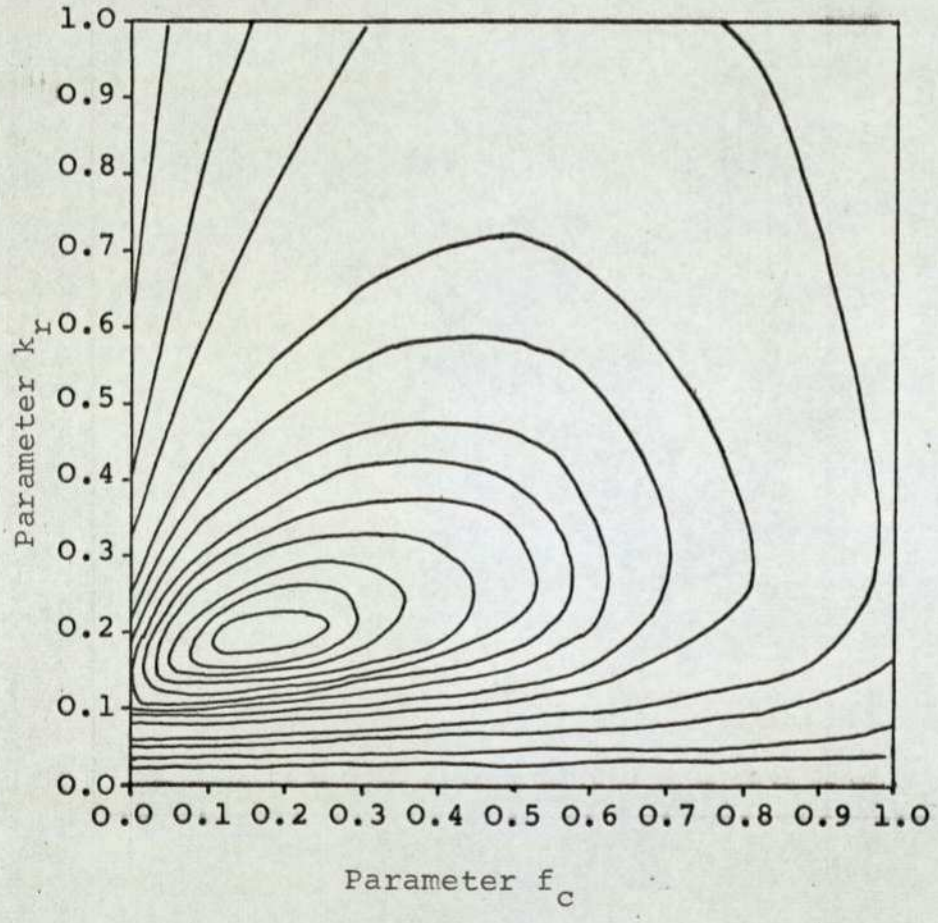


Figure 5.23 Response Surface for the Routing Parameter, k_r and the Minimum Infiltration Capacity Parameter, f_c.

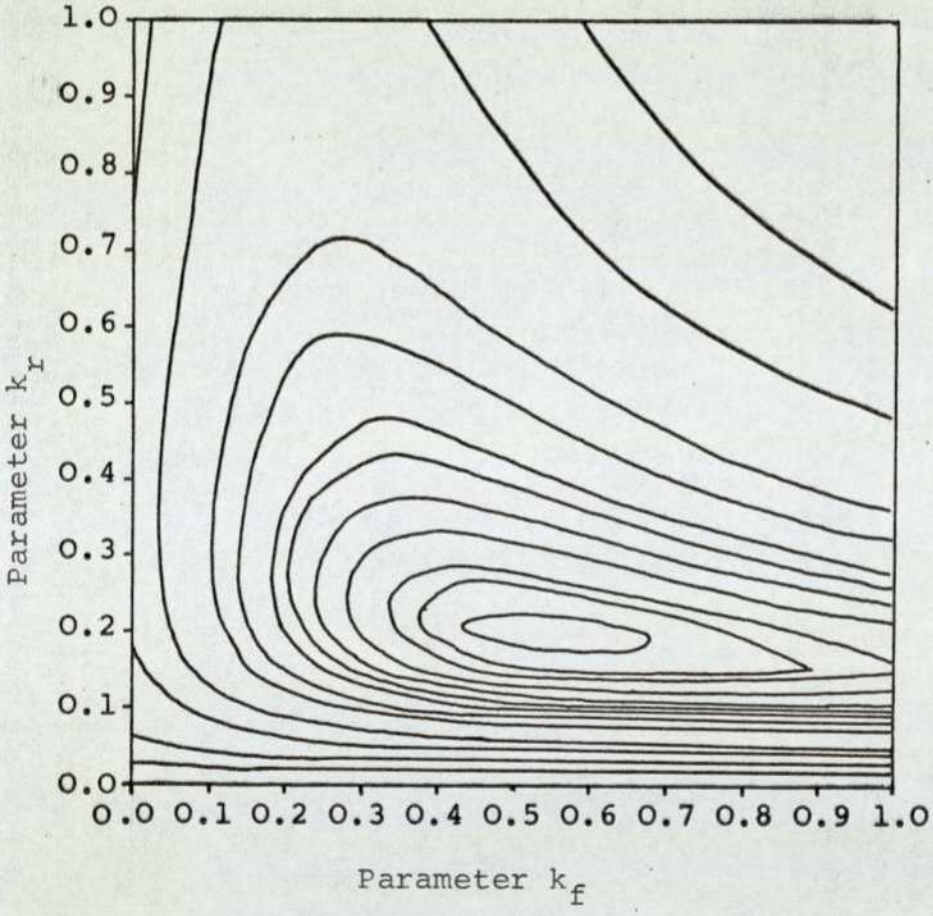


Figure 5.24 Response Surface for the Routing Parameter, k_r and the Infiltration Capacity Decay Parameter, k_f.

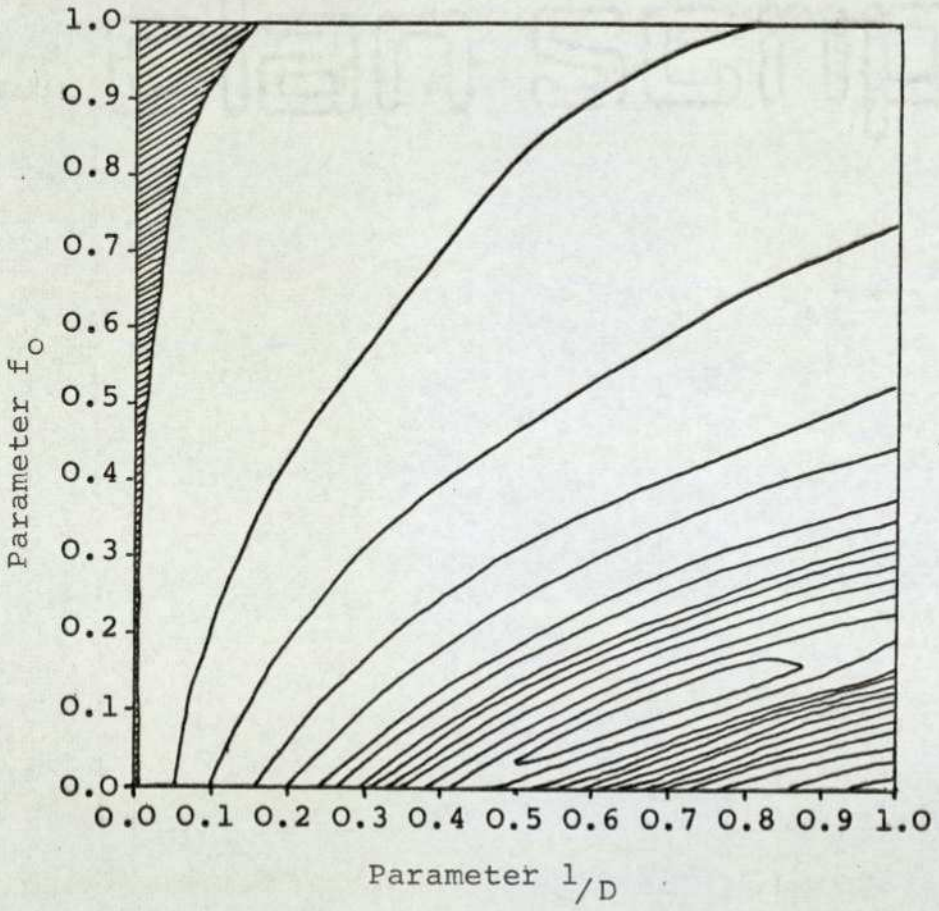


Figure 5.25 Response Surface for the Maximum Infiltration Capacity Parameter, f_o and the Depression Storage Capacity Parameter, $1/D$

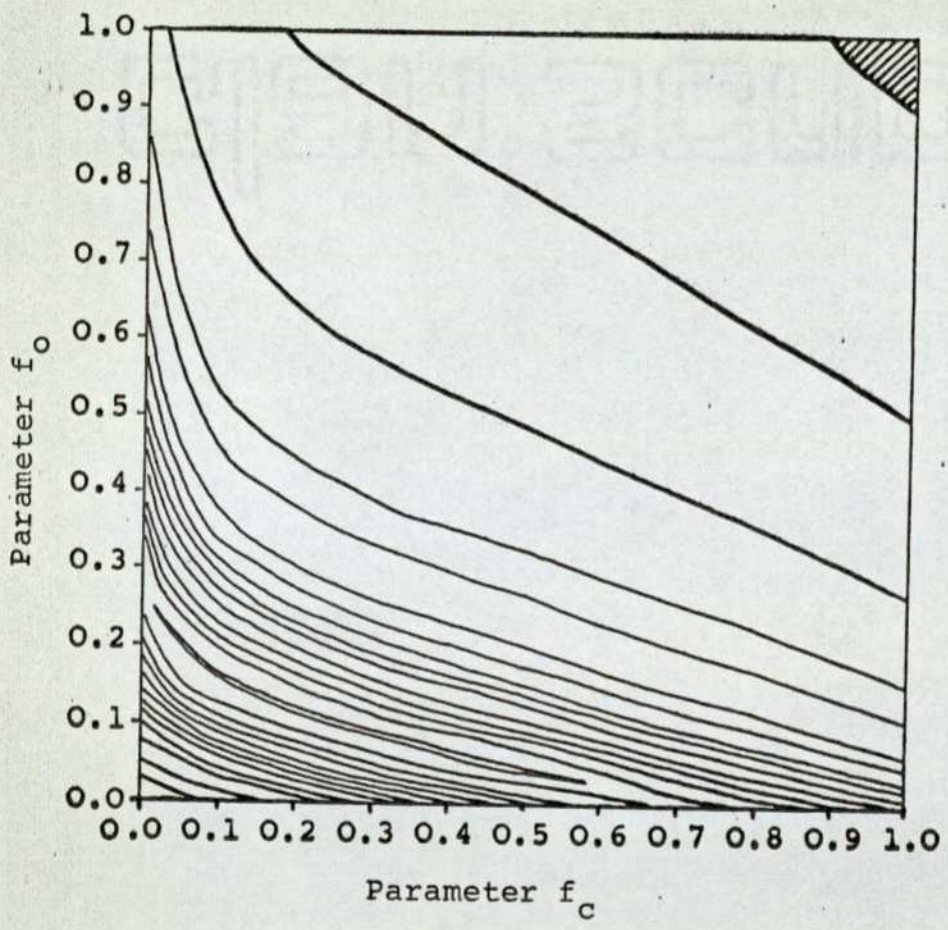


Figure 5.26 Response Surface for the Maximum Infiltration Capacity Parameter, f_o and the Minimum Infiltration Capacity Parameter, f_c.

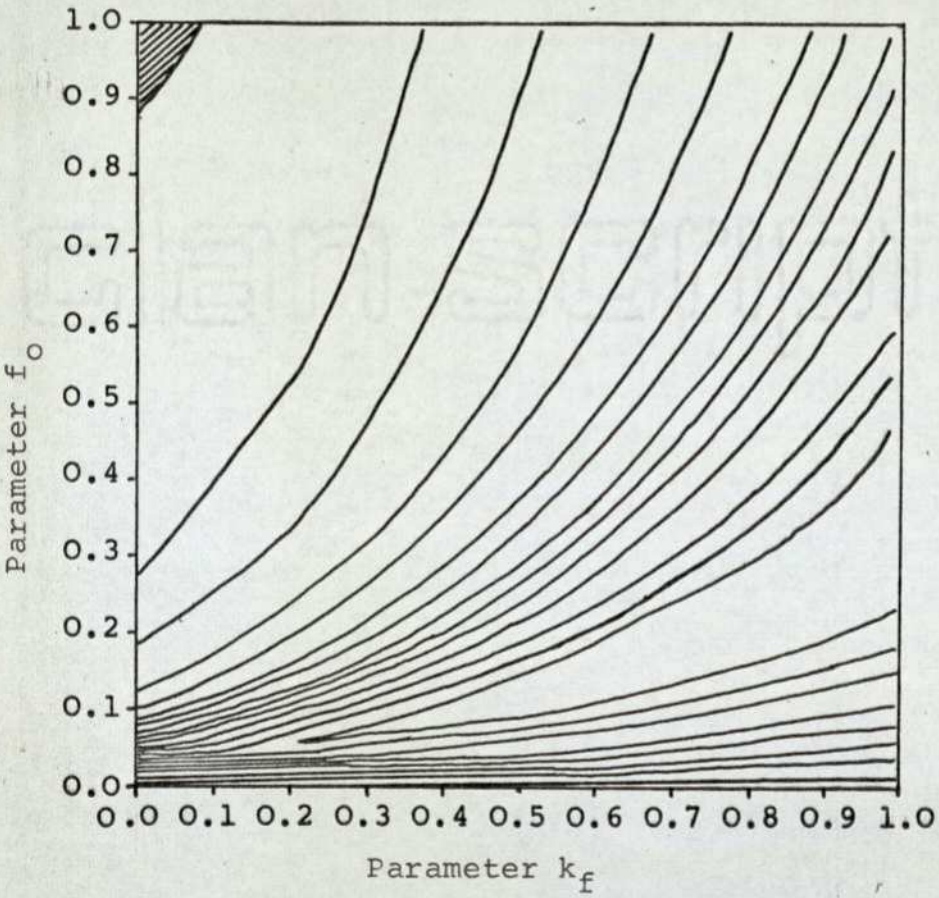


Figure 5.27 Response Surface for the Maximum Infiltration Capacity Parameter, f_o and the Infiltration Decay Parameter, k_f.

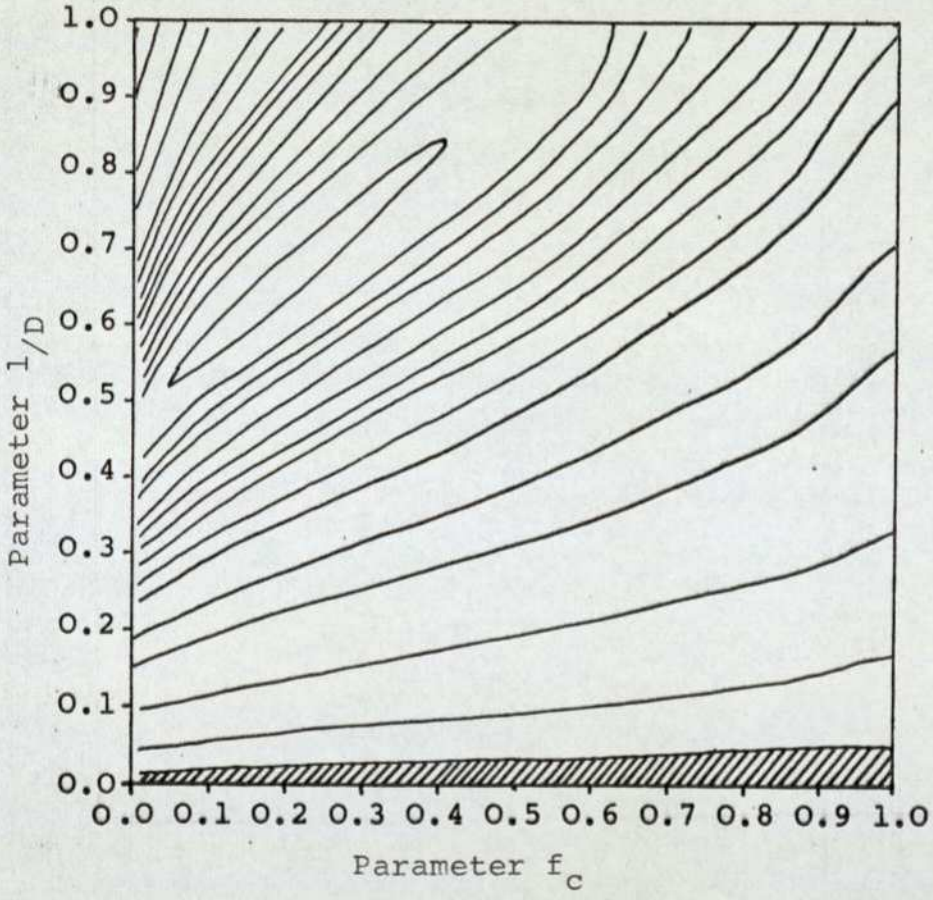


Figure 5.28 Response Surface for the Depression Storage Capacity Parameter, $1/D$ and the Minimum Infiltration Capacity Parameter, f_c.

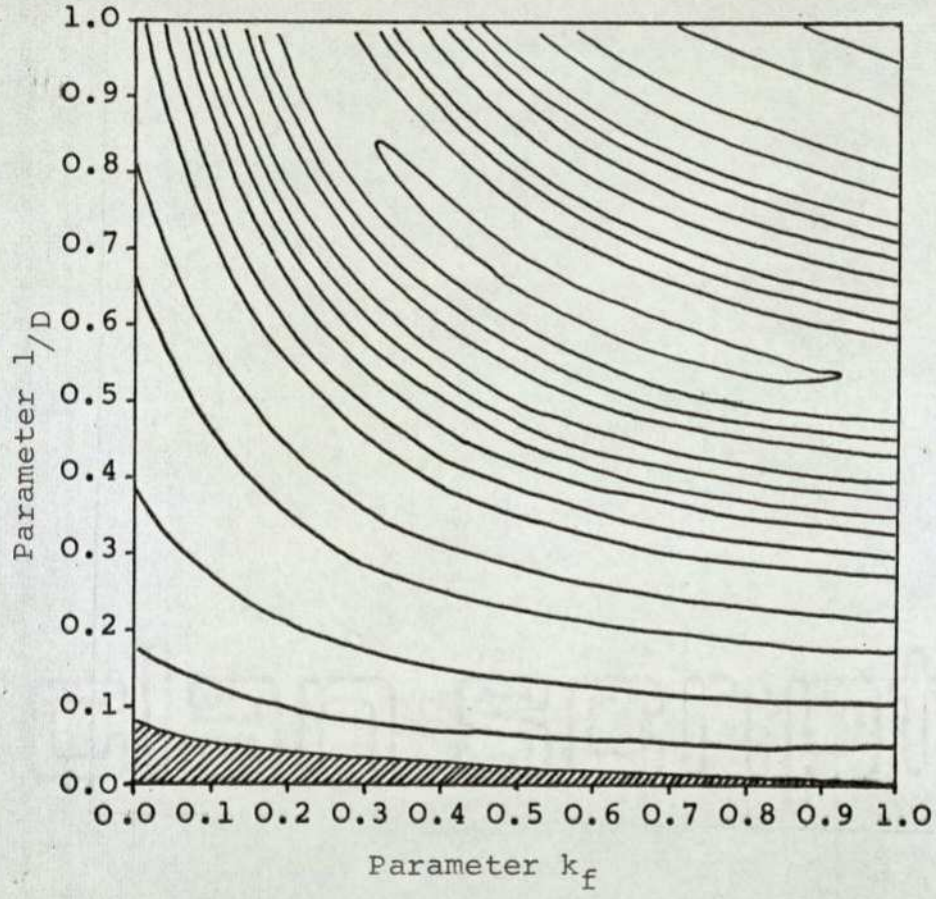


Figure 5.29 Response Surface for the Depression Storage Capacity Parameter, $^1/_D$ and the Infiltration Capacity Decay Parameter, k_f.

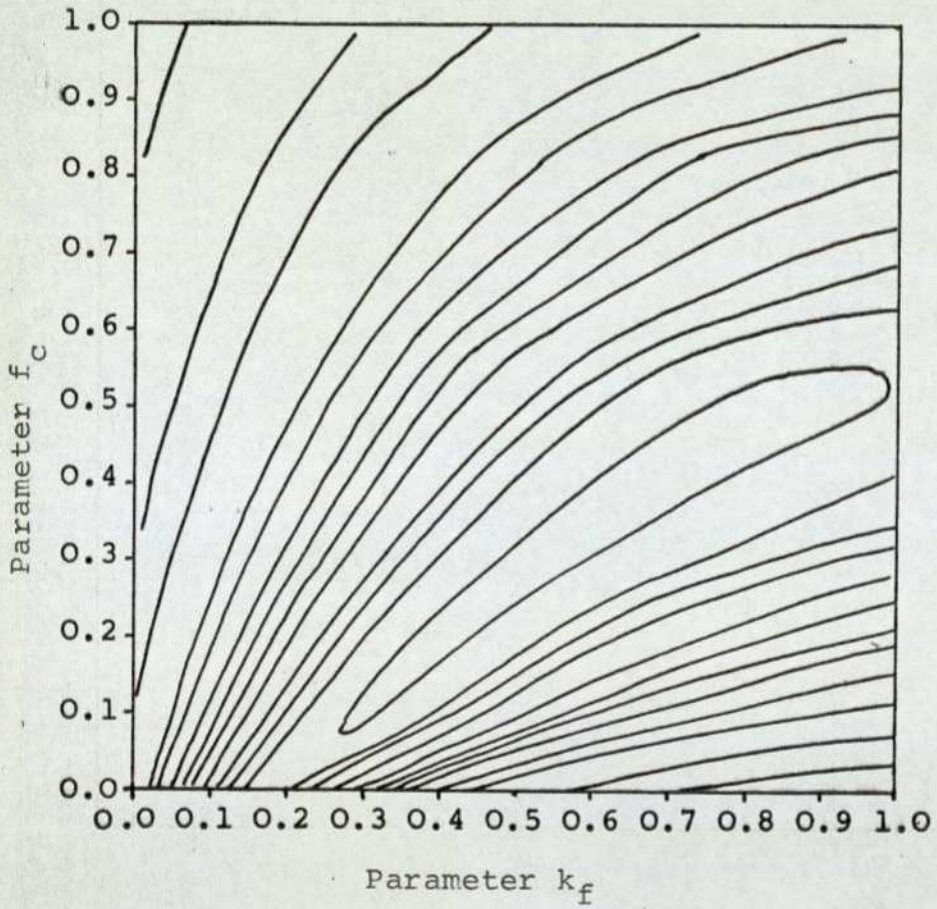


Figure 5.30 Response Surface for the Minimum Infiltration Capacity Parameter, f_c and the Infiltration Capacity Decay Parameter, k_f.

by choosing reasonable values for the parameters.

Parameter correlation is indicated in some of the plots by a 'trough' or valley of contours at an angle to the parameter axes. All the parameters, except perhaps the routing parameter, exhibit some degree of correlation. The exception is due to the fact that the routing parameter (k_r) is the only one which does not control flow volume. Hence the contours on the response surfaces for k_r exhibit a rounded pattern ideal for most optimization techniques. A parameter with function contours parallel to its axis is insensitive. Figures 5.24 and 5.27 indicate that the infiltration recession parameter (k_f) is insensitive in the region of the optimum values of k_r and f_o.

In summary, the response surfaces assist in gaining an understanding of the interaction between processes within the model and of the optimization process. The correlation between pairs of parameters is brought out in the plots and underlines the dangers of subjective optimization. A pair of parameter values at one end of a trough such as that depicted in Figure 5.25 give simulation results which are almost as good as those corresponding to the true parameter values in the centre of the trough. Without quantitative estimates of the objective function within the trough, subjective optimization will nearly always lead to parameter values which are sub-optimal. On the other hand, the response surfaces represent only one characteristic of the simulated and observed hydrographs, namely the sum of the squares of the 'error' terms, and the hydrologist has very little control over the final set of parameter values chosen

by an automatic procedure. It may be that good estimates of peak flows, combined with an overall approximate fit between simulated and observed hydrographs give better results for a particular purpose than the minimization of the objective function.

5.II.3. Model Performance. The adequacy of the model for simulating the hydrological processes of Crawters Brook is best demonstrated by comparing the observed and simulated hydrographs. Six examples are given for storms throughout the study period (Figures 5.31 - 5.36). It can be seen that the model simulates the system reasonably well, assuming that the streamflow and precipitation data are correct. Higher quality data and better baseflow separation techniques may have resulted in better model performance.

The mean of the simulated peaks was 0.762 m^3/sec and the mean observed peak was 0.806 m^3/sec, indicating an under-simulation of the peaks of about 5 per cent. Flow volumes were under-simulated, being 10,000 m^3 on average, compared to 11,200 m^3 for the observed volumes. Graphs comparing the peak flows and volumes are given in Figures 5.37 and 5.38.

5.III. SIMULATION RESULTS

The results of the optimization of the simulation model showed considerable variation in the values of the model parameters from storm to storm. In order to determine the effects of urbanization on the model parameters and to enable the model

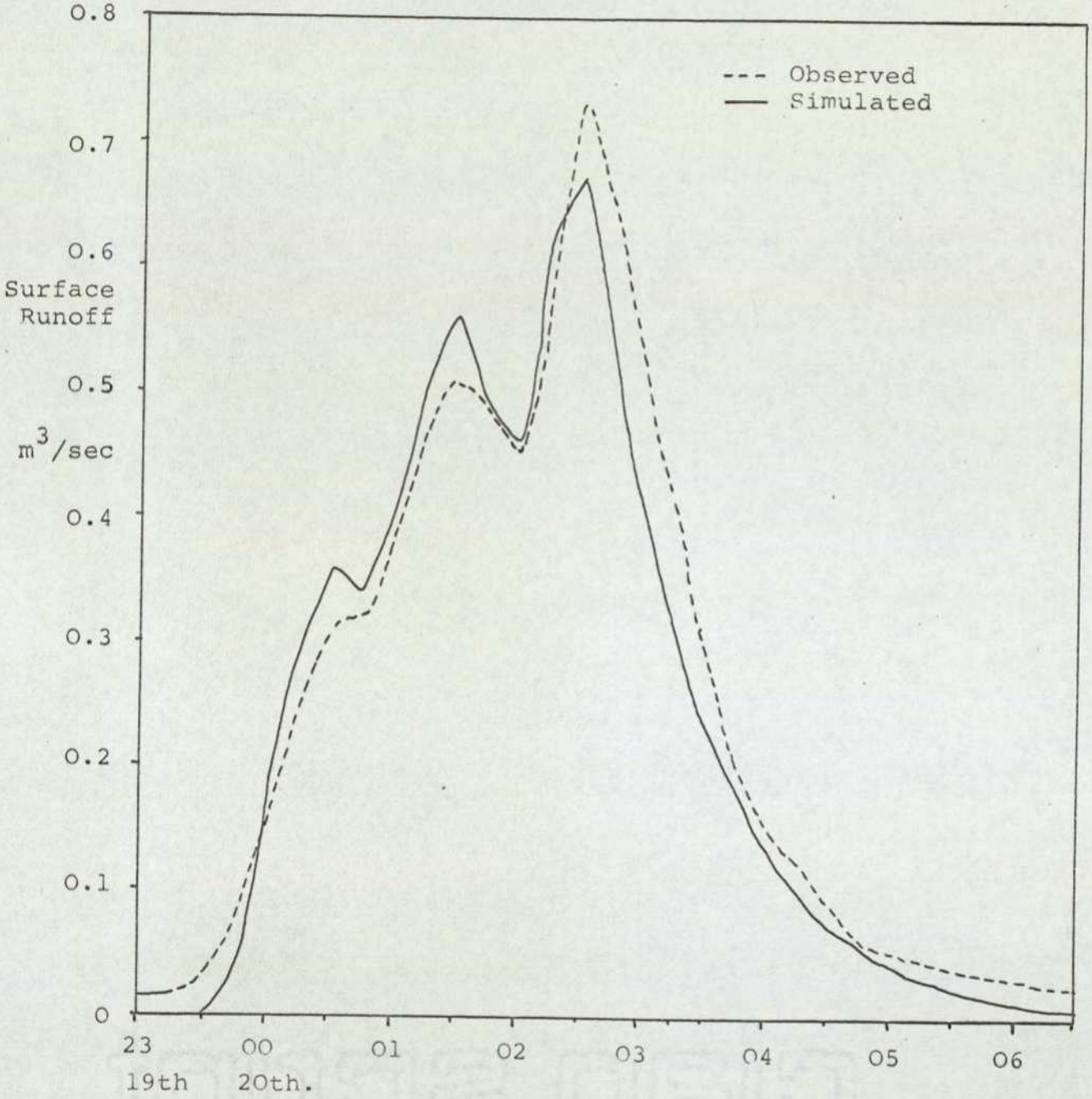


Figure 5.31 Observed and Simulated Hydrographs, 20th September 1953.

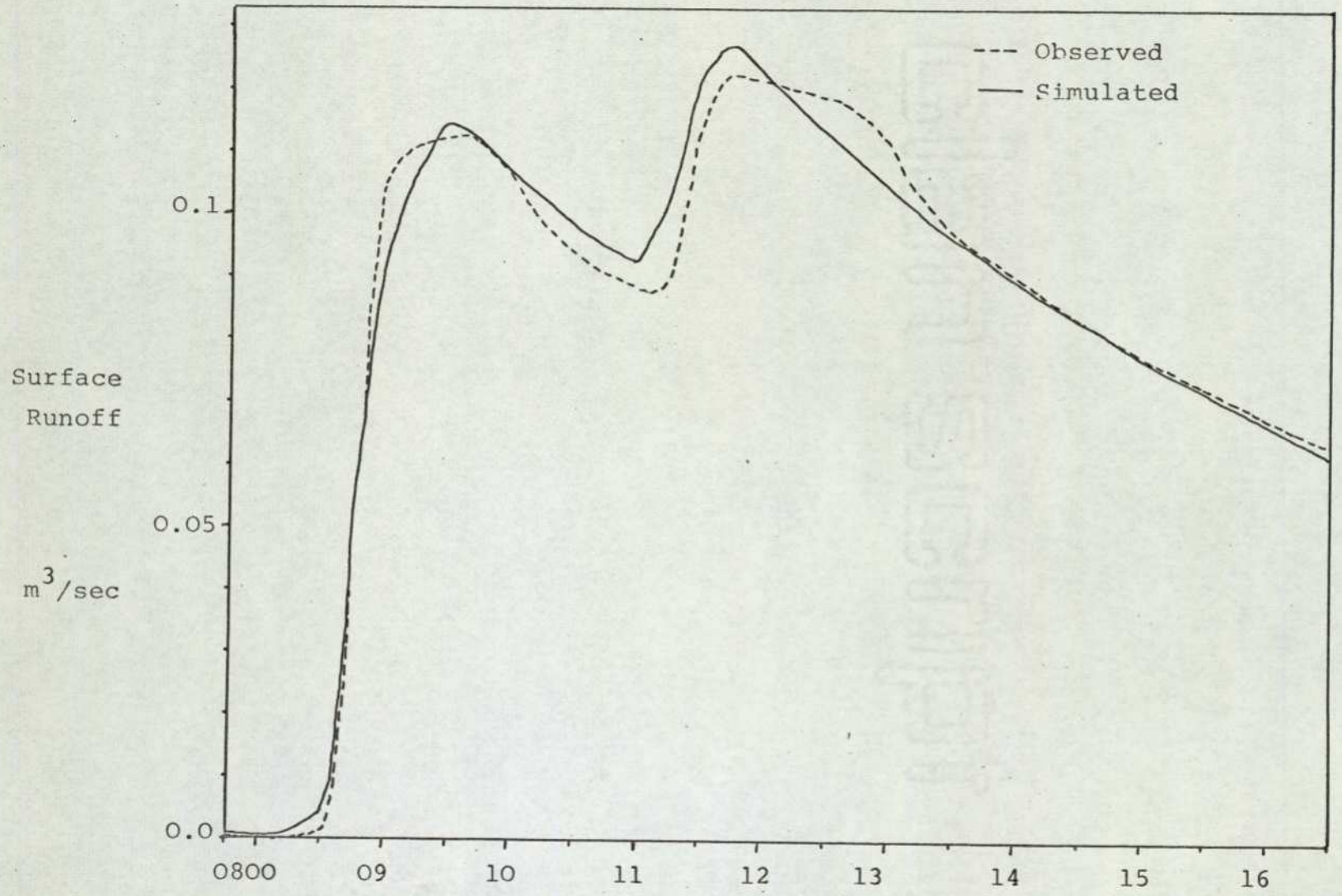


Figure 5.32 Observed and Simulated Hydrographs, 28th October 1953.

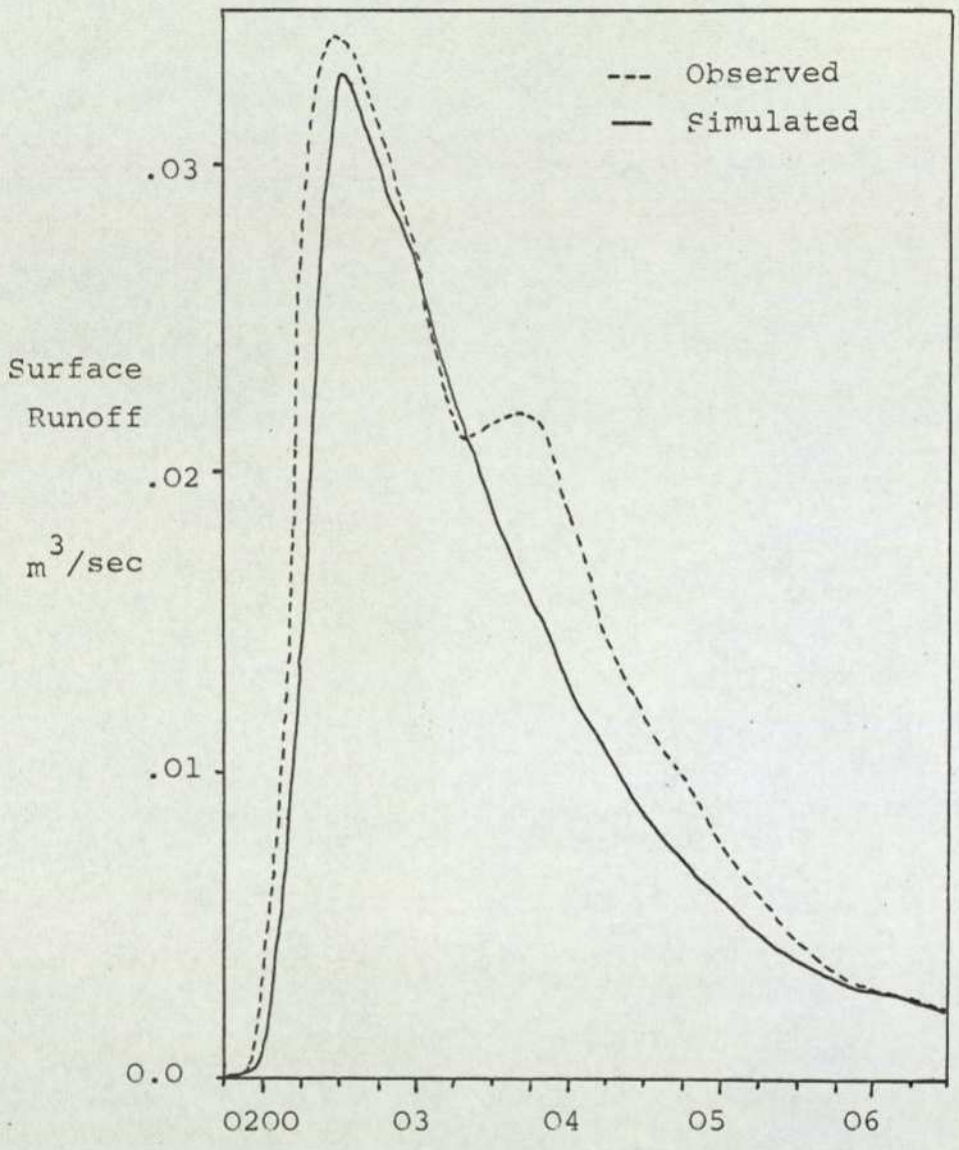


Figure 5.33 Observed and Simulated Hydrographs, 27th July 1954.

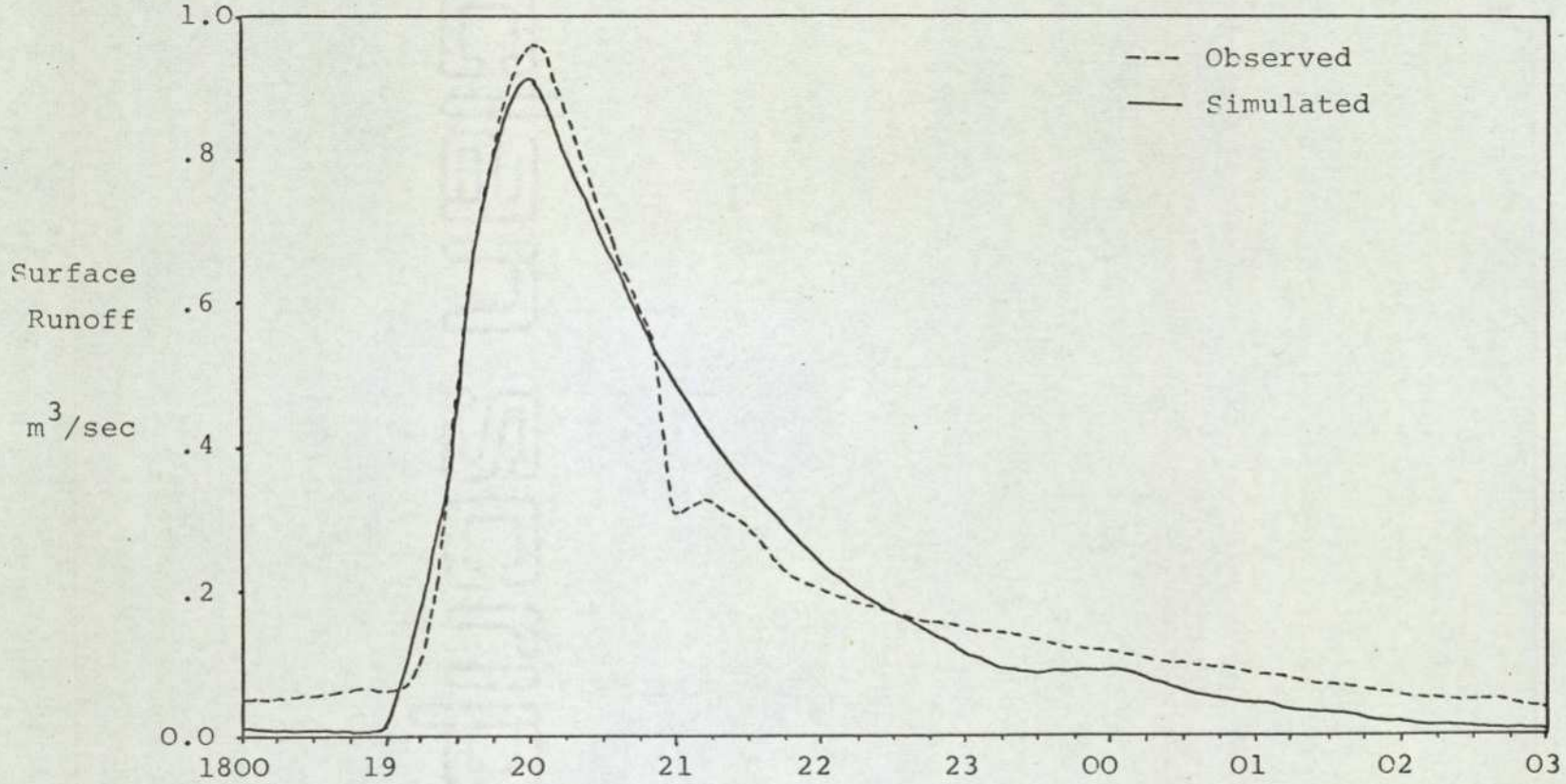


Figure 5.34 Observed and Simulated Hydrographs, 13th December 1958.

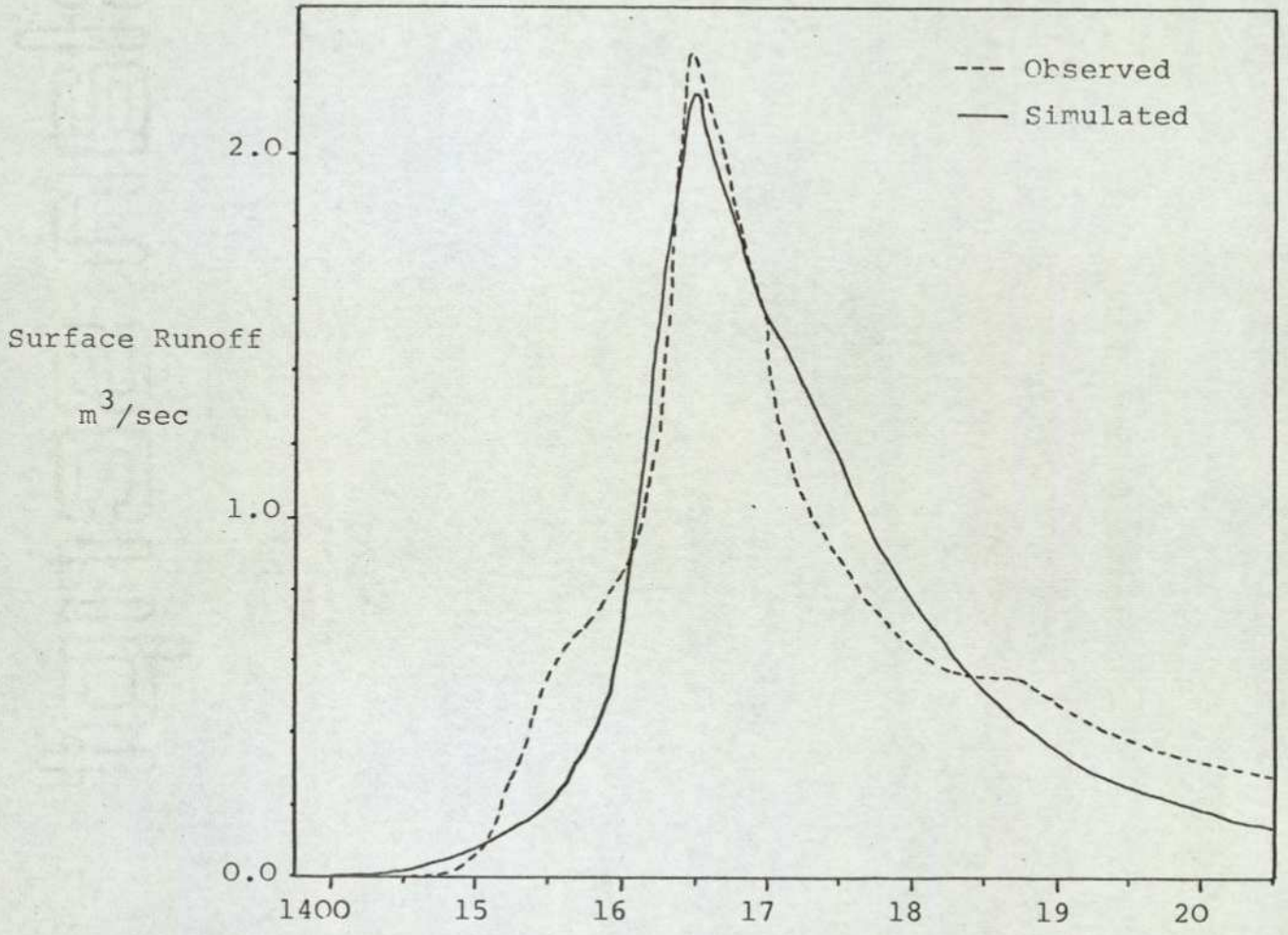


Figure 5.35 Observed and Simulated Hydrographs, 24th September 1963.

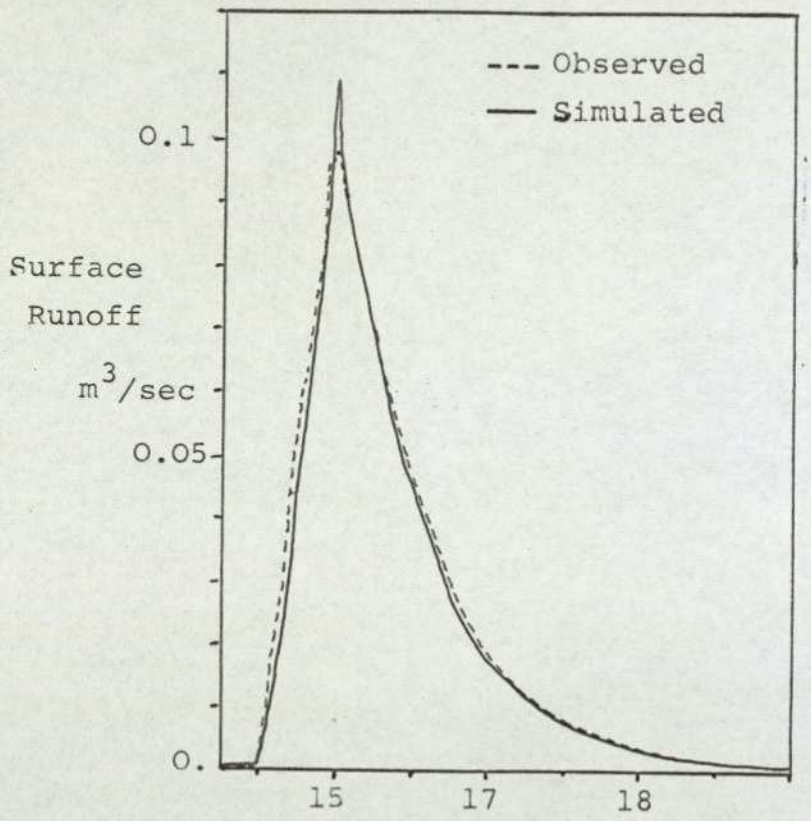


Figure 5.36 Observed and Simulated Hydrographs, 18th September 1969.

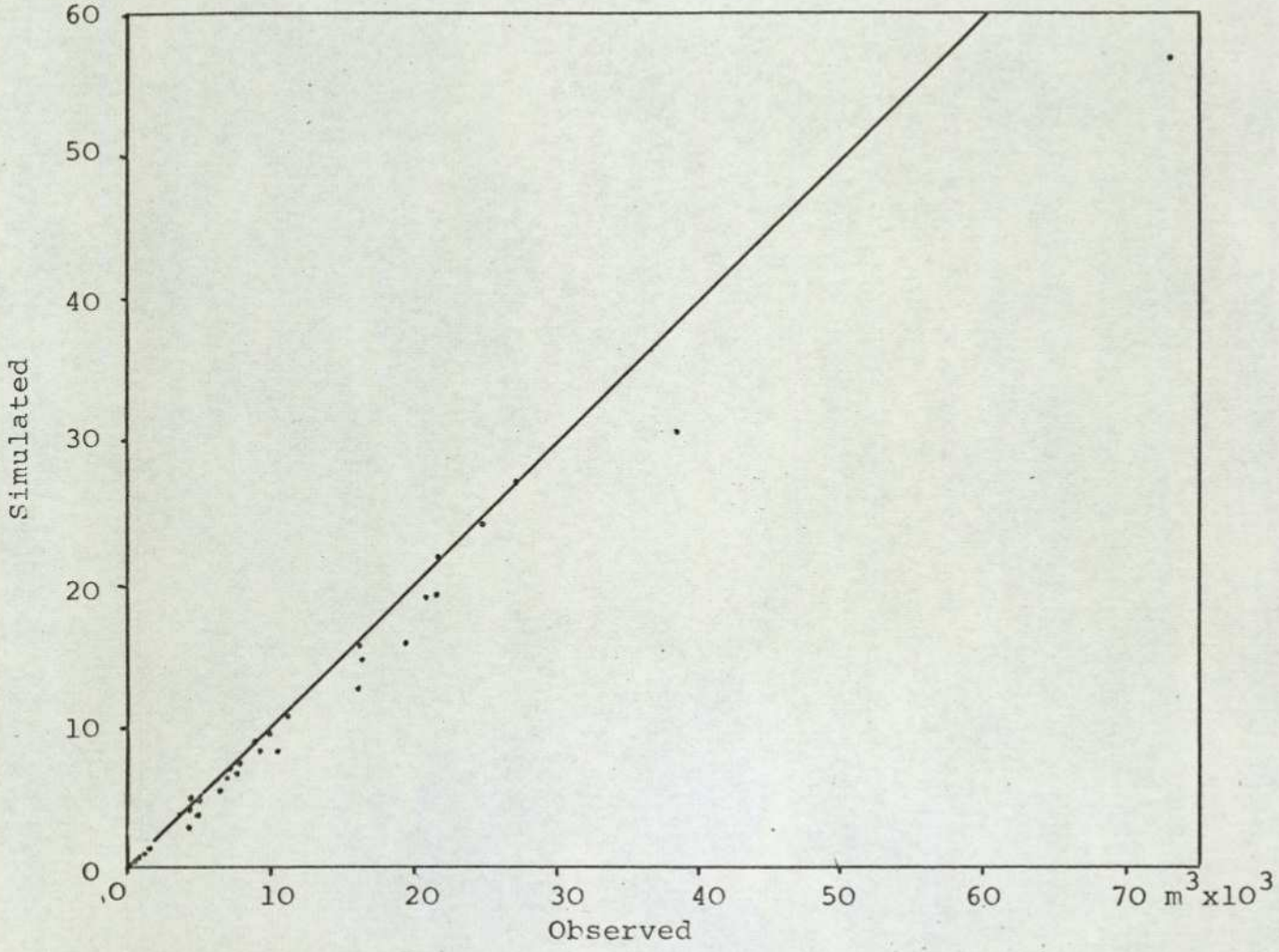


Figure 5.37 Plot of Simulated vs. Observed Flow Volumes.

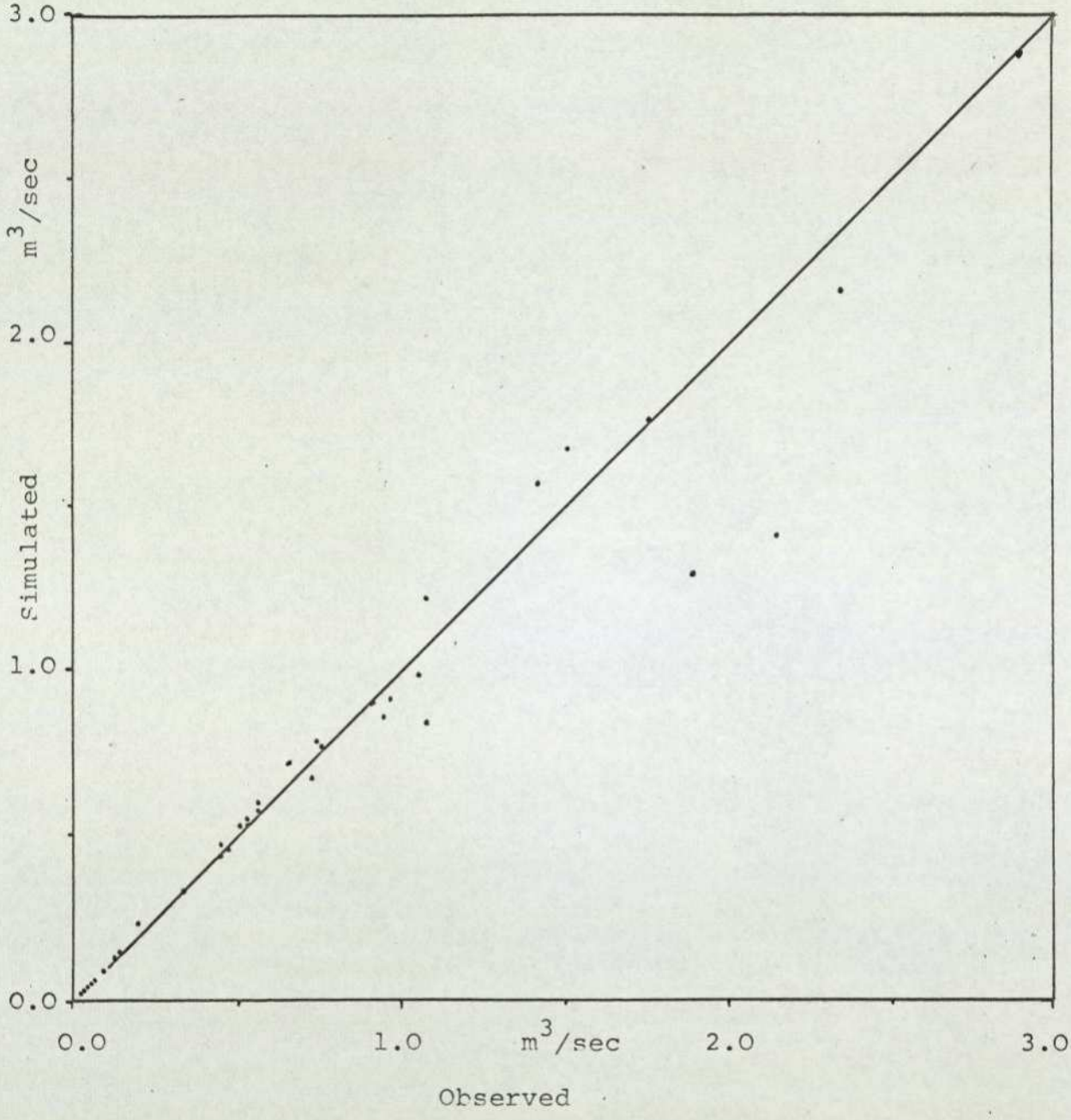


Figure 5.38 Plot of Simulated vs. Observed Flow Peaks

to be used as a predictive tool, it is necessary to break down the variance of each parameter into its constituent parts and assign each part to a measurable variable. This can be achieved using statistical techniques.

5.III.1. Explanation of Parameter Variance. The variance of a parameter value may be due to the urbanization of the catchment, to the hydrological condition of the catchment at the time of the storms and to the intrinsic characteristics of the actual storms. The relative importance of these variables in explaining the variance can be assessed using the statistical techniques of correlation and regression analysis (Dixon and Massey, 1969). These techniques are widely used in the analysis of hydrological data (Stammers, 1966).

To carry out the analysis a set of data for each storm must be assembled, and according to statistical theory the storms should be randomly chosen. As the storms were not in fact randomly chosen but were biased towards large storms and to storms occurring early in the study period, this should be borne in mind when interpreting the results. The data must include variables describing the factors which are thought to affect the variance in parameter values. The variables were broken down into two classes : those which represent catchment conditions and those which represent storm characteristics. The former consisted of catchment impervious cover, an antecedent precipitation index (A.P.I.) and temperature. The A.P.I. gives an indication of the moisture content of the catchment as a result of precipitation occurring in the time

period before the storm. On the other hand, temperature gives an indication of the seasonal influence of evaporation, this being highly correlated with temperature.

The variables chosen to represent storm characteristics were precipitation depth, duration and maximum 15-minute intensity. The mean and standard deviation of each variable in the analysis is listed in Table 5.2.

The techniques of regression and correlation are very similar, the former requiring control of the independent variables while the latter does not. The basic methods of analysis are identical although the assumptions are different. The basic assumption in correlation analysis is that the distribution of the two variables to be correlated form a normal distribution (Dixon and Massey, 1969, p.202). This can be tested by plotting frequency histograms of the variables and visually checking that the shape of the curve approximates the 'Bell-shaped' normal distribution. The graphs (Figure 5.39) indicate that some of the variables are not normally distributed. This can be corrected to a certain extent by using a transformation. However, it was decided not to do this as an appropriate transformation for every variable is difficult to find. It was therefore decided to carry out the analysis with the untransformed variables and interpret the results with an appropriate degree of caution.

The objective of the analysis was to form predictive equations such that the model parameters could be defined in terms of the independent variables. The resulting predictive equation

Variable	Mean	Stnd. Devn.	Units
k_r	1.06	0.77	15-min^{-1}
f_o	3.23	3.98	mm/hr.
f_c Parameters	0.61	0.75	mm/hr.
D	1.39	2.05	mm
Impervious Area	14.9	7.77	%
A.P.I.	15.2	9.38	mm
Temperature	9.96	4.25	C
Precip. depth	14.8	10.6	mm
Max. 15-min Prec.	1.93	1.24	mm/hr.
Duration	7.25	6.40	hrs.

Table 5.2 Means and Standard Deviations of Variables in the Analysis of Parameter Values.

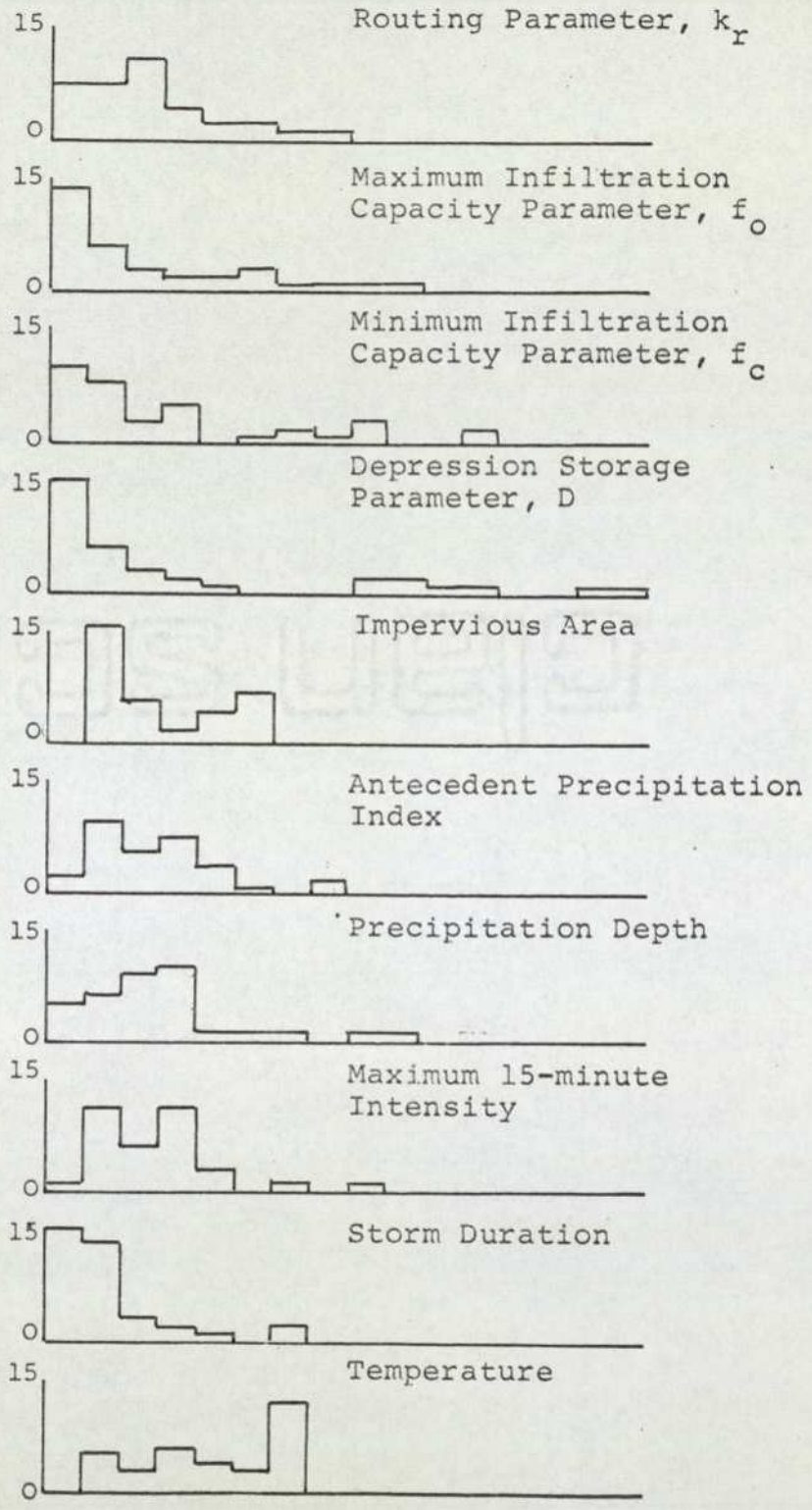


Figure 5.39 Histograms of Variables in the Regression Analysis of Parameter Values.

is of the form :

$$\bar{y} = a_o + a_1 x_1 + a_2 x_2 + \ldots\ldots a_n x_n$$

where $\bar{y}$ = the estimate of the dependent variable,

the a's are regression coefficients,

the x's are n independent variables.

The computer programme used was the Stepwise Multiple Linear Regression programme (BMD02R) of a suite of programmes written by the Biomedical Department of the University of California (Dixon, 1968). The BMD02R programme utilizes the least-squares method for calculating regression coefficients.

The regression calculations proceed by steps. The independent variable which would explain the largest amount of the variance of y is regressed in the first step, forming an equation of the simple linear type :

$$y = a_o + a_1 x_1$$

This variable is the variable in the correlation matrix with the highest correlation coefficient with y. To take the routing parameter as an example, it can be seen in Table 5.3 that impervious cover is the independent variable with which it is most highly correlated (r= 0.547). The second and subsequent steps entail the calculation of F-ratios for each of the remaining independent variables. In each step, the variable with the highest F-ratio is added to the independent variables in the equation. With each additional variable the amount of explained parameter variance increases, but the F-ratio for the

			k_r	f_o	f_c	D
Catchment conditions	Impervious area	(%)	.547	-.301	-.258	.351
	A.P.I.	(mm)	-.373	-.104	-.145	-.209
	Temperature	(°C)	.329	.350	.193	.051
Storm conditions	Duration	(hrs)	-.401	.603	.204	-.081
	Max. intensity	(mm/hr)	.397	.002	.056	.295
	Depth	(mm)	-.273	.601	.286	.050

Table 5.3 Correlation Coefficients between variables in the Regression Analysis of Parameter Values.

equation decreases due to the loss of degrees of freedom (for each addition, the number of degrees of freedom is reduced by one). When the F-ratio becomes very low, additional variables explain very little additional variance.

In the analysis, regression was carried out in three stages. The first stage involved the regression of the impervious area variable as this was the variable of most interest. In the second stage the catchment condition variables were allowed into the equation if their F-ratios merited inclusion. Finally, the storm variables were added. Equations for each stage (Equations 1, 2 and 3 in column 1) are presented in Tables 5.4 to 5.7, together with the corresponding standard errors of the coefficients (Std.err.), coefficients of multiple correlation (R), standard errors of the estimate (S), F-ratios (F) and the significance of the regression coefficients (signif).

The square of the coefficient of multiple correlation represents the amount of variance explained by the variables in the equation. Table 5.8 gives the explained variance (i.e. the coefficient of determination, R^2) of each of the four parameters using the regression equation with all independent variables. Regression results for individual parameters are discussed below.

The routing parameter, k_r. The first equation in Table 5.4 is a simple linear equation quantifying the effect of urbanization on k_r. The coefficient of multiple correlation (R= .547) indicates that about 30% of the variance of k_r is explained

Eqn.	Variable	Reg. coef.	Std. err.	R	$\hat{S}$	F	Signif.
1.	a_o	.2498		.547	.658	14.1	
	a_1 Impervious area	.0545	.0145				99%
2.	a_o	.0902		.657	.611	7.86	
	a_1 Impervious area	.0473	.0141				99%
	a_2 Temperature	.0530	.0248				95%
	a_3 A.P.I.	-.0171	.0117				90%
3.	a_o	.5681		.735	.579	5.49	
	a_1 Impervious area	.0276	.0155				95%
	a_2 Temperature	.0475	.0238				95%
	a_3 A.P.I.	-.0241	.0120				95%
	a_4 Depth	-.0210	.0238				80%
	a_5 Duration	-.0039	.0388				
	a_6 Max. intensity	.1629	.1107				90%

Table 5.4 Regression Equations for the Routing Parameter, k_r.

by the impervious area variable (R^2 = .299).

In order to test the significance of the regression coefficients a one-sided t-test was used using tabulated distributions of the t-statistic. This can be used to test whether a regression coefficient is significantly different from zero, in which case it can be said that the dependent variable is not independent of the other variables in the equation. In the case of Equation 1, the regression coefficient was found to be significantly different from zero with a confidence level of 99 per cent. In interpreting these figures, however, it should be noted that this test relies on the distribution of the values of the dependent variable being a normal distribution for any given value of the independent variable.

The variables in Equation 2 would also seem to have a significant contribution to the variance of k_r. Equation 3 accounts for 54 per cent of the variance of k_r. It is interesting to note that the storm variables (depth, duration and intensity) are almost entirely independent of k_r. It can be concluded that impervious area is the most important variable in explaining the variance of k_r.

The initial infiltration capacity, f_o. The correlation between f_o and the impervious area variable is 0.301 and consequently the explained variance is only about nine per cent. Obviously other variables are responsible for variations in f_o. One would suspect that the antecedent precipitation index would be important because it indicates the relative dryness of the catchment and

Eqn.	Variable	Reg. coef.	Std. err.	R	$\hat{S}$	F	Signif.
1.	a_o	5.5156		.301	3.849	3.28	
	a_1 Impervious area	-.1539	.0850				95%
2.	a_o	3.9353		.502	3.603	3.47	
	a_1 Impervious area	-.1883	.0832				95%
	a_2 Temperature	.3266	.1461				95%
	a_3 A.P.I.	-.0763	.0692				80%
3.	a_o	-2.7423		.725	2.967	6.41	
	a_1 Impervious area	-.0418	.0776				
	a_2 Temperature	.3307	.1213				99%
	a_3 A.P.I.	.0117	.0615				
	a_4 Depth	.0150	.0949				80%
	a_5 Duration	.2158	.1576				90%
	a_6 Max. intensity						

Table 5.5 Regression Equations for the Maximum Infiltration Capacity Parameter, f_o.

hence the relative level of the initial infiltration capacity. However, the data for Equation 2 in Table 5.5 would seem to suggest that temperature is more important than the A.P.I., indicating that the seasonal cycle of catchment moisture conditions is important. Equation 3 includes two of the storm variables (the F-ratio for maximum intensity was too low for inclusion) and these seem to be relatively independent of f_o as was the case for k_r.

The minimum infiltration capacity f_c. The first equation in Table 5.6 explains less than seven per cent of the variance of f_c. Once again, other variables must be responsible for the variation. The regression coefficients for the catchment variables (temperature and A.P.I.) in Equation 2 have rather meagre levels of significance and Equation 3 indicates that the storm variables have very little effect. It must be concluded that the final equation would give a rather poor estimate of f_c.

The depression storage capacity, D. Impervious area is the most important variable in explaining the variance of D. This is indicated by the relatively high coefficient of multiple correlation (R = .351) compared with f_o and f_c. The other variables in the analysis contribute very little although their F-ratios are high enough to warrant inclusion in Equation 3. It is to be concluded from Table 5.7 that D, like f_c, is inadequately explained by the variables in the analysis.

In summary, the regression analysis emphasised the

Eqn.	Variable	Reg. coef.	Std. err.	R	$\hat{S}$	F	Signif
1.	a_o	.9791		.258	.7312	2.36	
	a_1 Impervious area	-.0248	.0161				90%
2.	a_o	1.0337		.393	.7181	1.89	
	a_1 Impervious area	-.0319	.0166				95%
	a_2 Temperature	.0325	.0291				80%
	a_3 A.P.I.	-.0180	.0138				80%
3.	a_o	.7973		.427	.743	1.04	
	a_1 Impervious area	-.0278	.0199				90%
	a_2 Temperature	.0300	.0305				80%
	a_3 A.P.I.	-.0134	.0154				80%
	a_4 Depth	.0245	.0305				
	a_5 Duration	-.0271	.0498				
	a_6 Max. intensity	-.0183	.1421				

Figure 5.6 Regression Equations for the Minimum Infiltration Capacity Parameter, f_c.

Eqn.	Variable	Reg. coef.	Std. err.	R	$\hat{S}$	F	Signif.
1.	a_o	.0119		.351	1.946	4.63	
	a_1 Impervious area	.0924	.0430				95%
2.	a_o	.3883		.369	1.993	1.63	
	a_1 Impervious area	.0833	.0460				95%
	a_2 Temperature	.0135	.0808				
	a_3 A.P.I.	-.0247	.0383				
3.	a_o	-.4801		.429	2.038	1.05	
	a_1 Impervious area	.0841	.0546				90%
	a_2 Temperature	-.0013	.0837				
	a_3 A.P.I.	-.0109	.0423				
	a_4 Depth	.0417	.0838				
	a_5 Duration	-.0376	.1366				
	a_6 Max. intensity	.2321	.3898				

Table 5.7 Regression Equations for the Depression Storage Capacity Parameter, D.

importance of the impervious area in a catchment in changing the catchment parameters. The sign of the regression coefficients for impervious area in the tables is indicative of the direction of the resulting change. Increasing impervious area causes an increase in the routing parameter (k_r) and the depression storage capacity (D) while it causes a decrease in the initial infiltration capacity (f_o) and the minimum infiltration capacity (f_c).

However, it was somewhat disappointing to conclude that the variation in depression storage capacity and in minimum infiltration capacity were inadequately accounted for by the variables in the regression analysis (see Figure 5.8). This can be explained by considering the following factors.

Firstly, the parameters were obtained, not from physical measurements through time, but by extraction from the hydrometeorological data using automatic parameter optimization. Although this method may be better than subjective optimization, both are subject to the problems mentioned earlier in this chapter, namely parameter correlation and suboptional function minima (local optima). The correlation between D and f_c may well have resulted in the finding of erroneous values for these two parameters.

Secondly, the data used in the study was not of the highest quality and the baseflow separation technique was rather simple. This may have resulted in the model being fitted to erroneous data which would lead to erroneous parameter values.

Thirdly, the independent variables chosen to explain the parameter variance may have been too few in number. Some measure

Parameter	R	R^2 x100
k_r	0.735	54 %
f_o	0.725	53 %
f_c	0.43	18 %
D	0.43	18 %

Note: R is the Multiple Correlation Coefficient,

R^2 is the Coefficient of Determination

Table 5.8 Explained Parameter Variance Using the best Regression Equations.

of the spatial distribution of impervious cover may have resulted in an improved equation for the prediction of the routing parameter. More complex indices representing the moisture conditions of the catchment prior to the storms may also have resulted in better equations for the prediction of the other three parameters, which are considerably dependent on catchment moisture content.

5.III.2. Hydrograph Prediction. In order to visualise the effects of variations in parameter values due to urbanization, the equations developed using the multiple linear regression analysis were used to derive model parameter values for a sample storm for various levels of urbanization. All other independent variables were assumed to be fixed at their actual values (occurring on 19th September 1953). The levels of urbanization chosen were expressed as the following percentages of impervious area : 45 per cent, representing maximum urban development, 40 per cent, 30 per cent, 20 per cent, 10 per cent and one per cent. The highest percentage was not exceeded because such figures would be unrealistic over such an area. The lowest percentage represents essentially rural conditions. Table 5.9 gives the parameter values for various levels of urbanization. The simulation model could be used to construct the streamflow hydrographs which would occur under any urbanized conditions within the range of the equations.

Although it was noted in the previous subsection that the regression equations were rather inadequate, the urbanization variable (percentage of impervious area) explained the highest proportion of the variance of each parameter.

Parameter	Percentage Impervious Area					
	1	10	20	30	40	45
k_r (15-min^{-1})	0.856	1.104	1.380	1.656	1.932	2.070
f_o (mm/hr)	4.345	3.969	3.552	3.134	2.717	2.507
f_c (mm/hr)	1.166	0.916	0.638	0.360	0.082	0.000
D (mm)	0.060	0.817	1.658	2.499	3.340	3.761

Table 5.9 Predicted Parameter Values for various percentages of Impervious Area.

CHAPTER SIX

THE EFFECTS OF URBAN DEVELOPMENT ON DAILY AND PEAK FLOWS

The purpose of this part of the study was to analyse the effects of the urban development of the catchment on the recorded mean daily flow and peak flows. Regression and frequency analysis were used to quantify the effects of independent catchment and meteorological variables on streamflow.

6.I. ANALYSIS OF DAILY FLOW

The two main approaches that can be used to study daily flow are simulation and statistical analysis.

The simulation approach entails the calibration of a simulation model for a period of time when the catchment is essentially rural. After the model has been calibrated, synthetic flow data may be generated using the observed meteorological data. This flow data can be compared with the recorded flow in order to quantify the changes resulting from urban development. This approach has two major requirements : (1) a period (usually not less than three years for adequate calibration) during which the catchment was in a rural condition; (2) adequate hydrometeorological data to carry out the simulation, the most important being hourly precipitation. Unfortunately, the catchment under study satisfied neither of these requirements. Firstly, streamflow measurements started at about the same time as the building of the new town of Crawley and so no calibration

period was available. Secondly, the precipitation was in autbgraphic form and no tabulated values of rainfall for intervals less than daily were in existence. The simulation approach was thus clearly impractical and it was decided to use statistical methods.

Multiple Linear Regression has been used by previous workers to relate flow volumes to independent hydrometeorological variables (e.g. Sharp, et al., 1960) and this method would be suitable for the present study as it would enable the impervious area of the catchment to be included as an independent variable. A discussion of the use of multiple linear regression was given in Chapter 5.III.1.

6.I.1. Selection of variables. The daily streamflow data used as the dependent variables in the analysis was that calculated by the computer programme (STREAM) from the digitized data. The daily time period used was 0900 to 0900 as this is the standard observation time for daily hydrometeorological data. The most important variable affecting daily streamflow is probably daily precipitation. It was mentioned in Chapter 4 that the raingauge at The Beeches in Crawley is representative of catchment rainfall as it is located centrally within the catchment, and so this data was used in the regression analysis.

The catchment moisture condition is a second important variable in that it controls the proportion of rainfall flowing to the stream and to sub-surface storage. An antecedent precipitation index was used to represent this variable. The

variation in this index should account for some of the variance in stream baseflow - when the A.P.I. is low after a dry period, baseflow would be low, and vice versa. However, this variable alone cannot represent catchment moisture status because the moisture in the catchment would be depleted more rapidly in summer than in winter due to the seasonal variation in evapotranspiration. To account for this annual cycle, several variables were used, the sources of which were discussed in Chapter 4.II.1. They consisted of monthly values of sunshine hours, relative humidity and temperature, and the number of daylight hours for each day. The monthly data were interpolated to give estimates of daily figures. Because the streamflow on a particular day may be highly dependent on precipitation occurring on the previous day (due to the time lag of the catchment), the latter was used as a separate independent variable. The percentage of impervious area in the catchment was used to represent the degree of urban development.

A matrix giving the correlation coefficients between the variables used in the analysis for the first four years of data is presented in Table 6.1. For the main part of the analysis, 6,774 daily values were available. The remaining days making up the full twenty years were unsuitable due to gaps in the data. Unfortunately a gap of only two or three hours rendered the rest of the day's record unsuitable. Table 6.2 gives the means and standard deviations for the data.

6.I.2. Regression Analysis Results. The first four years of

	Log. Flow	Prec.	Prec. (t-1)	A.P.I.	Temp.	Sun.	Rel H.	Day. hrs.	Imp. area
Log. Flow	1.000	.648	.461	.709	-.264	-.288	.210	-.275	.160
Precipitation		1.000	.248	.623	.017	-.046	.104	-.032	.010
Prec. (t-1)			1.000	.610	.020	-.044	.104	-.028	.007
A.P.I.				1.000	.036	.105	.225	-.070	.000
Temperature					1.000	.759	-.342	.795	.212
Daily sunshine						1.000	.800	.918	.146
Relative humidity							1.000	-.759	-.289
Daylight hours								1.000	.214
Impervious area									1.000

Table 6.1 Correlation Matrix - October 1952 to September 1956 Data (Daily)

Variable	Mean	Stnd. Devn.	Units
Mean Daily Flow	0.030	0.056	m^3/sec *
Daily Precipitation	2.182	4.773	mm
A.P.I.	8.805	9.277	mm
Temperature	9.390	4.629	°C
Daily Sunshine	4.131	1.960	hours
Daylight Hours	12.000	3.304	hours
Relative Humidity	87.867	5.517	%
Impervious Area	19.567	7.349	%

Table 6.2 Means and Standard Deviations of Daily Data - October 1952 to September 1972.

* Note: To avoid very small regression coefficients, units used in the regression equations are $m^3/sec \times 10^{-3}$ or litres/sec.

the study period were used as a trial period to assess the relative importance of the independent variables in the hope that one or two of those representing evapotranspiration could be deleted if they were found to be insignificant.

A random sample of 100 days was used to determine whether a transformation was necessary. A histogram of mean daily flow (Figure 6.1) indicated that the distribtion of the data was skewed. Initially a logarithmic transformation was used to counteract this. An important effect of a transformation to counteract positive skewness of data is that it gives more weight to lower values of daily flow than would otherwise be the case. As the simulation study discussed in the previous chapters analysed the effect of urbanization on the hydrological processes associated with individual storms, and the flood frequency study in Chapter 6.II analyses the effect on peak flows, this part of the study can be viewed as assessing the effects of urbanization on the lower flow regimes (mainly baseflow).

The pertinent results of the regression analysis of the first four years are summarised in Table 6.3. It is interesting to note that the impervious area explains an additional five per cent of the variance in the logarithm of daily flow. The last three variables, representing a measure of evapotranspiration explain very little additional variance and therefore two of these variables could be omitted from the analysis with almost no loss in the amount of explained variance. As the relative humidity was the last variable to be entered into the regression equation and therefore had the smallest contribution to the

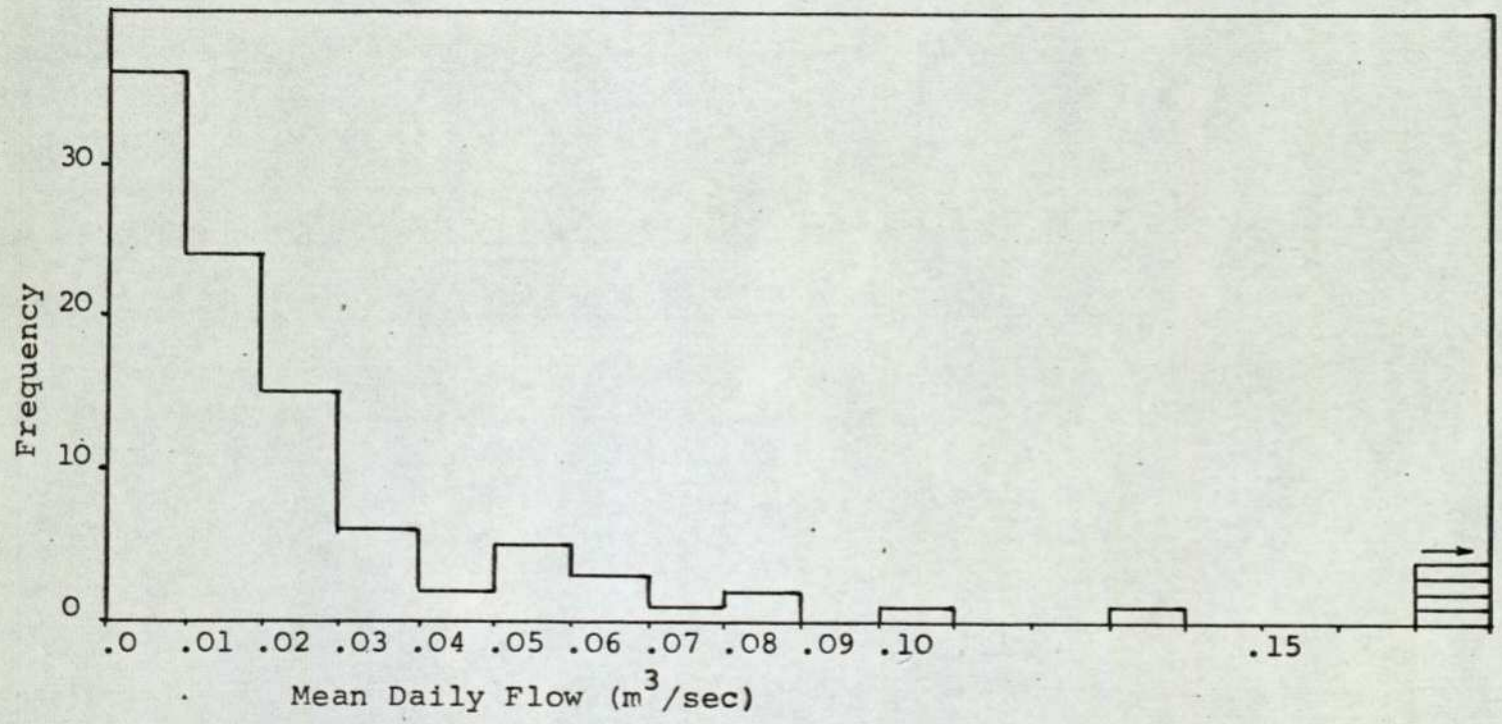


Figure 6.1 Histogram of Mean Daily Flow (Random Sample, N=100).

explained variance of mean daily flow, it was omitted from subsequent analysis. The number of daylight hours was also omitted because firstly, it had a very high correlation with sunshine hours ($r = 0.918$), and secondly it was a seasonal variable which, unlike sunshine hours, would not be related to short-term changes in meteorological conditions. The regression coefficient of sunshine hours also had the highest level of significance of the three (as shown in the last column of Table 6.3).

A second regression analysis was carried out without the two variables and with the full twenty years of observations. The correlation matrix for this analysis is given in Table 6.4. The results are presented in Table 6.5. With the 6,774 days of data the impervious area variable is less important, explaining only an additional 2.4 per cent. This is probably because this variable was changing more rapidly on average during the first four years than over the full twenty-year study period.

In order to try to improve the explained variance, other transformations of the variables were tried. Alterations were also made to the recession constant used to calculate the A.P.I. (in Equation 4.i), which in the first two analyses, was set equal to 0.8. The reason for changing the constant was that it is an estimated value only, and another value may be more representative of the true antecedent conditions. Alternative values tried were 0.75 and 0.70. The mean daily flow was transformed using the square root and cube root instead of the logarithm.

Table 6.6 shows the results of the analysis with a square

Step	Variable Entered	R	R^2.100	Increase in R^2	Signif. of Reg. Coef.
1	A.P.I.	.709	50.2	50.2	99.9%
2	Temperature	.766	58.6	8.4	99.9%
3	Precip.	.809	65.5	6.9	99.9%
4	Imperv. Area	.839	70.4	5.0	99.9%
5	Precip(t-1)	.844	71.3	0.8	99.9%
6	Daylight Hrs	.845	71.3	0.0	-
7	Sun. Hrs.	.845	71.4	0.1	99%
8	Relative H.	.845	71.5	0.1	95%

Note: R is the Multiple Correlation Coefficient
R^2.100 is the Coefficient of Determination

Table 6.3 Steps in the Regression Analysis on the October 1952 to September 1956 Data (N=1389)

	log. Flow	$\sqrt[2]{\text{Flow}}$	$\sqrt[3]{\text{Flow}}$	Prec	Prec (t-1)	API	Flow (t-1)	Temp.	Sun.	Imp. Area
Log. Flow	1.000	.925	.966	.654	.433	.513	.513	-.319	-.344	.153
$\sqrt[2]{\text{Flow}}$		1.000	.991	.769	.460	.487	.531	-.257	-.297	.083
$\sqrt[3]{\text{Flow}}$			1.000	.742	.460	.506	.536	-.283	-.319	.105
Prec.				1.000	.238	.204	.171	-.013	-.078	.006
Prec(t-1)					1.000	.618	.762	-.014	-.079	.007
A.P.I.						1.000	-	-.045	-.186	.010
Flow(t-1)							1.000	-.171	-.212	.039
Temp.								1.000	.790	.033
Sun. hrs.									1.000	.037
Imp. Area										1.000

Table 6.4 Correlation Matrix - October 1952 to September 1972 Daily Data.

Step	Variable Entered	R	R^2.100	Increase in R^2	Signif. of Reg. Coef.
1	Precip.	.654	42.7	42.7	99.9%
2	A.P.I.	.760	57.8	15.0	99.9%
3	Temperature	.815	66.4	8.7	99.9%
4	Imperv. Area	.830	68.9	2.4	99.9%
5	Precip(t-1)	.833	69.4	0.5	99.9%
6	Sun. Hrs.	.833	69.4	0.0	-

Table 6.5 Steps in the Regression Analysis of the October 1952 to September 1972 Data (N=6774)

Step	Variable Entered	R	R^2.100	Increase in R^2	Signif. of Reg. Coef.
1	Precip.	.769	59.1	59.1	99.9%
2	A.P.I.	.840	70.5	11.4	99.9%
3	Temperature	.871	75.9	5.4	99.9%
4	Precip(t-1)	.878	77.1	1.2	99.9%
5	Imperv. Area	.882	77.8	0.7	99.9%
6	Sun Hrs.	.882	77.8	0.0	-

Table 6.6 Steps in the Regression Analysis of the October 1952 to September 1972 Data with Square Root Transform of Flow.

root transform. The explained variance has risen to 78 per cent, while the cube root transform gave an explained variance of 77 per cent (Table 6.7). As the square root transform gives the best level of explained variance, it was used in all subsequent regression equations.

It can be seen from Table 6.8 that variations in the antecedent precipitation index had little effect on the explained variance. In addition, transforming the independent variables, whose distributions were known to be skewed, did not improve the estimate, in fact the standard error of the estimate worsened considerably.

It could be argued that the streamflow on the previous day (t-1) would give a good representation of antecedent moisture conditions on day t. This was therefore used in place of the A.P.I., to improve the explained variance. With this equation, almost 80 per cent of the variance in mean daily flow was explained by the independent variables. However, it would be useless as a predictive equation for two reasons. Firstly, if the equation were used to predict mean daily flow for hypothetical conditions, obviously the flow on day (t-1) for each day to be predicted would not be available. Secondly, if the equation were used sequentially to predict daily flow, which would then be used as an independent variable to predict the next day's flow, the errors in the prediction would be cumulative. The regression coefficients and standard errors for this equation are given in Table 6.9, showing that all independent variables have a highly significant effect on mean

Step	Variable Entered	R	R^2.100	Increase in R^2	Signif of Reg. Coef.
1	Precip.	.742	55.1	55.1	99.9%
2	A.P.I.	.826	68.2	13.1	99.9%
3	Temperature	.865	74.9	6.7	99.9%
4	Imperv. Area	.872	76.0	1.1	99.9%
5	Precip(t-1)	.877	77.0	1.0	99.9%
6	Sun. Hrs.	.877	77.0	0.0	-

Table 6.7 Steps in the Regression Analysis of the October 1952 to September 1972 Data with Cube Root Transform of Flow.

	Stnd. Err. of Est.	R	$R^2 \times 100$
(a) A.P.I. recession constant values:			
c = 0.8	1.453	.882	77.8
c = 0.75	1.456	.881	77.7
c = 0.7	1.461	.881	77.5
(b) using log. transforms	1.750	.822	67.6
(c) using Flow(t-1)	1.390	.891	79.4

Table 6.8 Results of Regression Analyses of Daily Data using (a) various values of the A.P.I. recession constant, c; (b) log. transforms of precipitation and A.P.I.; (c) previous day's flow as an independent variable replacing A.P.I.

Variable	Regression Coefficient	Signif.
(Constant)	3.5437	
Precipitation	0.4494	99.9%
Flow (t-1)	0.0206	99.9%
Temperature	-0.0970	99.9%
Sunshine	-0.0785	99.9%
Imp. Area	0.0298	99.9%

Standard Error of the Estimate = 1.396

Multiple Correlation Coefficient = 0.891

Coefficient of Determination = 79.4 per cent.

Table 6.9 Regression Equation for Predicting Daily Flow, using previous day's Flow as an Independent Variable and Square Root Transform of Flow.

daily flow.

It was concluded from the foregoing analysis, that the equation with a square root transform of mean daily flow, no transforms of the independent variables, and with the antecedent precipitation index recession constant equal to 0.8, gave the best results. The steps in the derivation of this equation were given in Table 6.6, and the regression coefficients and significance levels are given in Table 6.10, without the redundant sunshine hours variable. The regression coefficient for impervious area (and for the other independent variables) is different from zero with a high level of significance, and it can be concluded that an increase in the percentage of impervious area in the catchment has a significant effect on mean daily flow.

6.II. ANALYSIS OF PEAK FLOWS

The object of this part of the study was to estimate the effect of urban development on the magnitude of peak flows of a given frequency. The problem was tackled in three different ways using the data from Crawters Brook. Firstly, the available annual maximum flows for the water-years 1952/53 to 1971/72 were analysed. Secondly, multiple linear regression was used to form predictive equations for the peak flows and to construct flood frequency curves for various percentages of impervious area. Thirdly, the numbers of peaks in each year exceeding a fixed magnitude were analysed.

Variable	Regression Coefficient	Signif
(Constant)	3.3254	99.9%
Precipitation	0.4396	99.9%
Prec. (t-1)	0.0896	99.9%
A.P.I.	0.0738	99.9%
Temperature	-0.1583	99.9%
Imp. Area	0.0348	99.9

Standard Error of the Estimate = 1.453

Multiple Correlation Coefficient = 0.882

Coefficient of Determination = 77.8 per cent.

Table 6.10 Best Regression Equation for Predicting Daily Flow, using A.P.I. recession Constant of 0.8 and Square Root Transform of Flow.

6.II.1. Frequency Analysis of Annual Maxima. The recorded annual maximum flows are given in Table 6.11. In order to compare flood frequency curves for rural and urban conditions it is necessary to have two sets of data, collected during periods when no change occurred in the percentage of impervious area in the catchment. Ideally, these sets of data should be at least twenty years long to give reliable flood frequency curves. This requirement of stationary data is very difficult to meet and the analysis must be carried out using non-stationary sets of data. If the study period is split into two parts, representing rural and urban conditions a dilemma arises. Maximising the difference between the rural and urban sets of data requires taking the observations from each extremity of the study period, yet this small number of observations is insufficient to define a flood frequency curve. Conversely, if more observations are taken at either end of the study period, the effect of urbanization is masked. A compromise was therefore made in taking the last four years of data (from Table 6.11) and the first four years.

In order to derive the flood frequency curves for these two periods, the data must be plotted on a probability paper representing the assumed distribution of the annual maxima. A commonly used distribution in hydrological analysis (Chow, 1964, p.8.35) is the Type I Extremal Distribution. The peak flows were ranked and the return period (or recurrence interval)

Water Year	Flow in m^3/sec
1952	2.330
1953	2.956
1954	1.064
1955	1.316
1956	1.321
1957	1.861
1958	1.312
1959	gap
1960	4.729+
1961	1.687
1962	2.573
1963	4.729+
1964	1.798
1965	gap
1966	gap
1967	4.729+
1968	3.682
1969	1.830
1970	1.689
1971	0.958

Table 6.11 Annual Maximum Flows on Crawter's Brook for Water Years 1952/3 - 1971/2

calculated using the Weibull formula :

$$T_p = \frac{N + 1}{m} \qquad (6.1)$$

where T_p = return period

N = number of years of record

m = rank of flood.

These values represent the mean time periods between flows of given magnitudes. The return periods for the eight peaks in this analysis are shown in Table 6.12, the percentages of impervious area for the rural and urban periods being 8.7 and 28.0 respectively. The plotted points are shown in Figure 6.2 and the flood frequency curves were fitted by eye.

Although the frequency curves suggest a difference between the return periods of flows under rural and urban conditions, the difference is far from convincing. This may be due to the following factors : (1) the small sample of observations; (2) the semi-urbanized condition of the catchment during the 'rural' period; (3) the effect of meteorological conditions, perhaps masking the true effects of urbanization. In order to overcome these problems, multiple linear regression analysis was employed.

6.II.2. Regression Analysis of Peak Flows. By linking the magnitude of peak flow with the causative variables, predictions may be made of the twenty-year annual maximum series resulting from chosen percentages of impervious area.

Rank	Return Period	Recorded Flow Peak (m^3/sec)	
		First 4 years	Last 4 years
1	5 years	2.956	3.682
2	2.5	2.330	1.830
3	1.67	1.316	1.689
4	1.25	1.064	0.958
	Imp. Area:	8.7 %	28.0 %

Table 6.12 Split Sample Flood Frequency Analysis of First four and Last four Years.

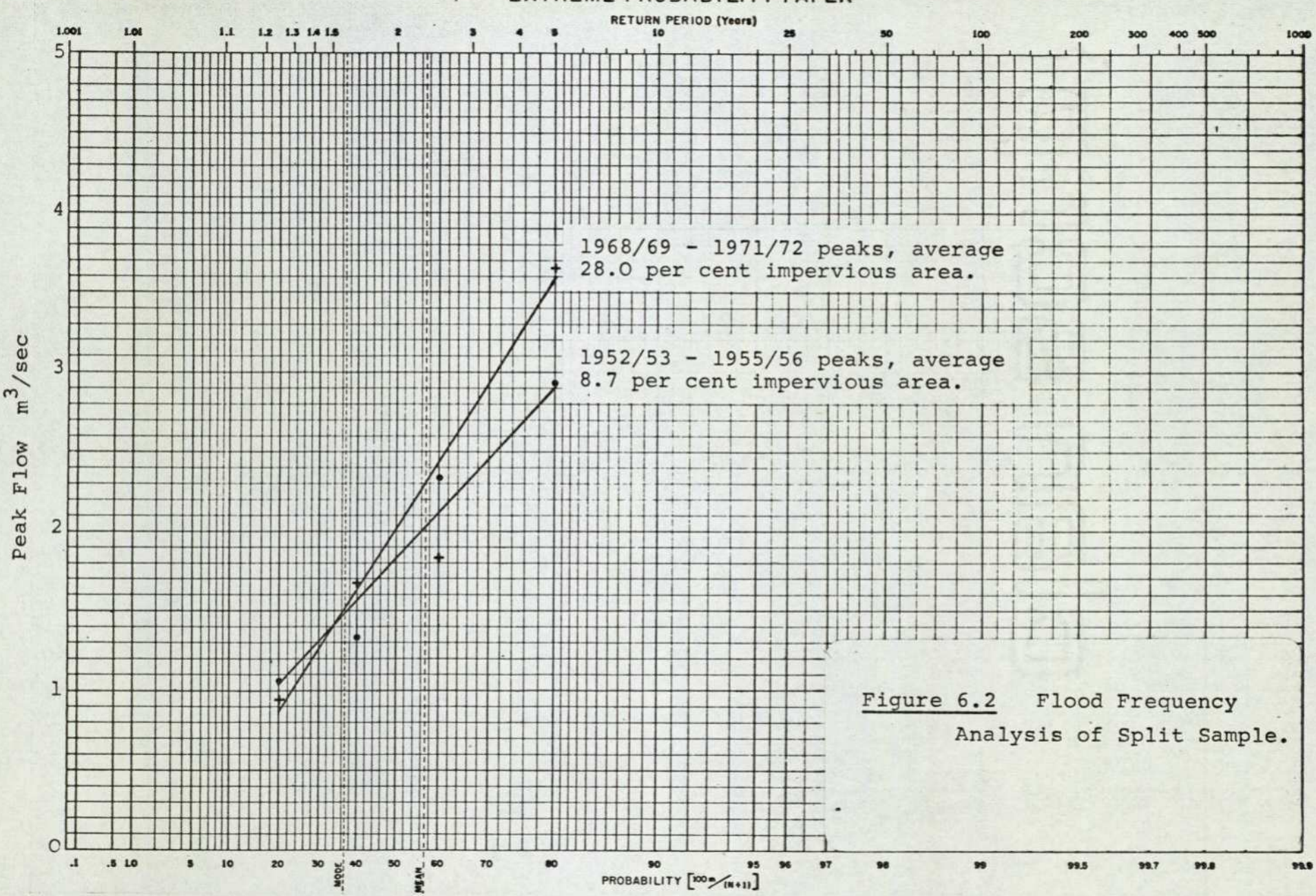


Figure 6.2 Flood Frequency Analysis of Split Sample.

The independent variables chosen for the analysis are listed in Table 6.13, with their means, standard deviations and units of measurement. In order to increase the reliability of the estimates, the data for thirty other storms were added to the fifteen available annual maxima. These storms were selected from those used in the simulation study discussed earlier, and all had peak flows in excess of 0.475 m^3/sec. This was to ensure that there was no undue bias toward smaller peaks as the regression equation was primarily to predict large peaks.

The measurement of most of the independent variables was straightforward, but the storm duration and A.P.I. variables require some explanation. Duration was defined as the length of time during which rain was falling, with no time gaps greater than one hour. The A.P.I. used was that for the day previous to the storm. This was added to rainfall falling on the day of the storm and this latter rainfall was multiplied by a factor inversely proportional to its time of occurrence before the start of the storm (i.e. 0.8 for 24 hours, 0.85 for 12 hours, 0.875 for 6 hours, etc.). Plotting histograms of the data showed (see Figure 6.3) that all distributions except impervious area and temperature were positively skewed. Logarithmic transformations were therefore used to give a more normalized distribution.

The correlation coefficients between the variables in the analysis are presented in Table 6.14. Because the maximum 15-minute and 30-minute precipitation intensities were not available for three of the annual peaks listed in Table 6.11,

Variable	Mean	Stnd. Devn.	Units
Peak Flow	1.047	0.891	m^3/sec
Precip. Depth	15.621	9.998	mm
Max. 30-min Prec.	3.784	2.972	mm
Max. 15-min Prec.	2.494	2.133	mm
Duration	7.422	6.025	hours
A.P.I.	14.202	9.579	mm
Temperature	8.947	4.387	^oC
Imperv. Area	13.673	7.889	%

Table 6.13 Means and Standard Deviations of Variables in the Regression Analysis of Peak Flow.

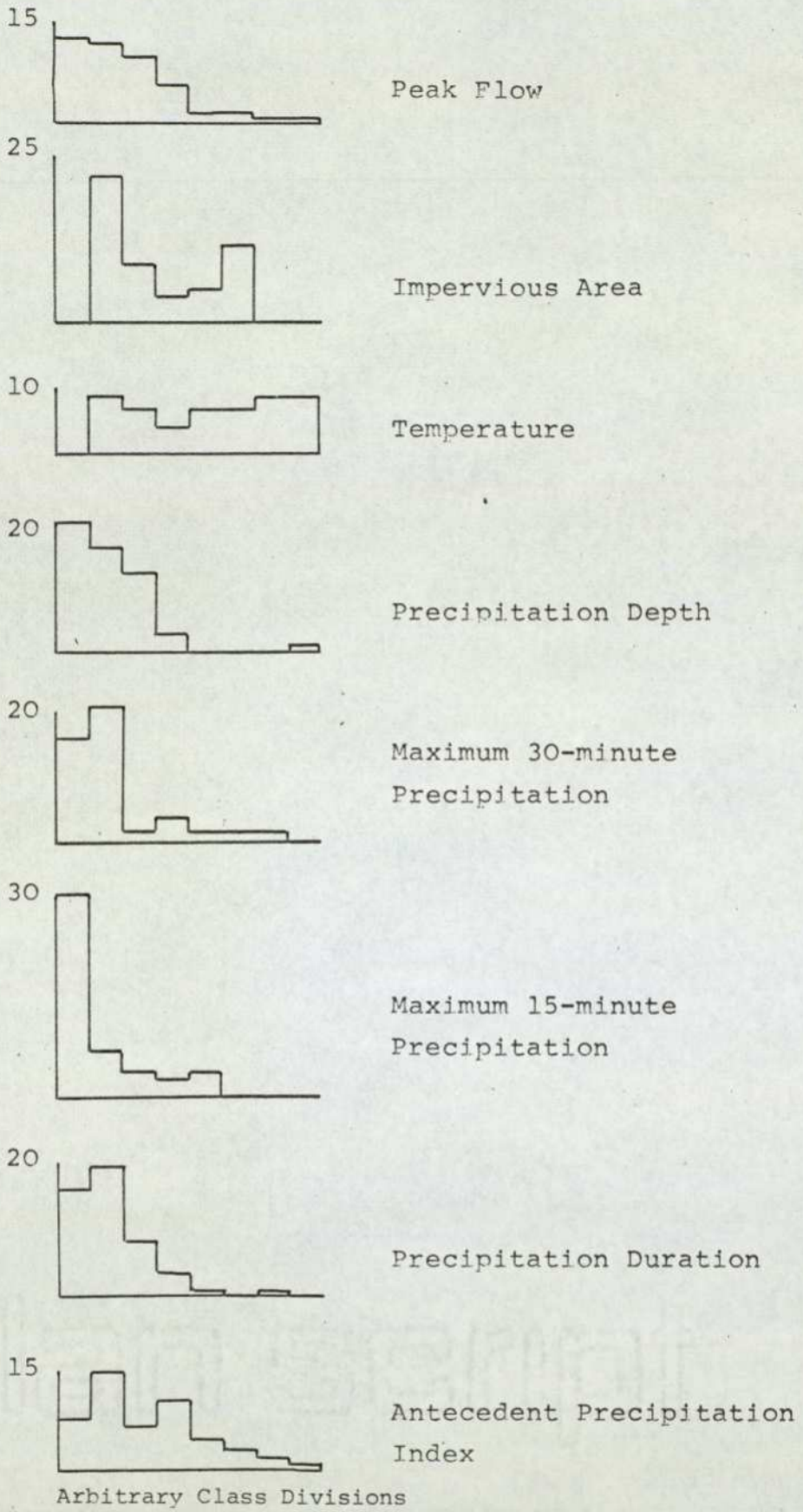


Figure 6.3 Histograms of Variables in the Regression Analysis of Peak Flow.

	Variable number													
	1.	2.	3.	4.	5.	6.	7.	8.	9.	10.	11.	12.	13.	14.
1. Peak Flow	1.000	.548	.579	.599	.133	-.242	.284	-.004	.853	.584	.638	.623	.068	-.154
2. Prec Depth		1.000	.249	.229	.787	-.165	-.024	.247	.574	.935	.304	.262	.684	-.001
3. 30-min Prec			1.000	.974	-.285	-.245	.205	.344	.532	.340	.917	.904	-.368	-.158
4. 15-min Prec				1.000	-.296	-.246	.218	.329	.513	.313	.900	.923	-.383	-.180
5. Duration					1.000	-.065	-.195	.033	.156	.652	-.265	-.289	.899	.040
6. A.P.I.						1.000	-.055	-.060	.009	-.119	-.131	-.131	-.037	.885
7. Imp. Area							1.000	.060	.313	.027	.281	.278	-.154	.002
8. Temp.								1.000	.025	.239	.318	.304	-.025	-.071
9. Log(1)									1.000	.679	.693	.646	.088	.153
10. Log(2)										1.000	.424	.375	.630	.039
11. Log(3)											1.000	.981	-.357	-.037
12. Log(4)												1.000	-.384	-.063
13. Log(5)													1.000	.024
14. Log(6)														1.000

Table 6.14 Correlation Matrix - Variables in Regression Analysis of Peak Flow. (N = 45)

it was necessary to derive two prediction equations, one for the years with all seven independent variables, and one for those with five variables. In deriving the latter equation, the three annual peaks with only five independent variables could be used, making a total of 48 storms.

The sequence of steps in the Stepwise Multiple Linear Regression analysis for the seven-variable case is given in Table 6.15. It is interesting to note that precipitation depth, temperature (representing time of year) and impervious area are most significant. The regression coefficient for the 15-minute precipitation intensity is not significantly different from zero, suggesting that the high correlation coefficient between this variable and 30-minute intensity ($r = 0.981$ in Table 6.13) has made the former somewhat insignificant. However, it did contribute to the explained variance and was therefore retained in the equation for predictive purposes. The table of regression coefficients and standard errors is given in Table 6.16.

The sequence of steps in the analysis for the five-variable case is given in Table 6.17. All variables contribute to the variance in peak flows, explaining 75.7 per cent altogether. This is not much less than the 78.5 per cent explained by the seven-variable equation. The regression coefficients and standard errors for the five-variable equation are given in Table 6.18. All coefficients are significantly different from zero.

The equations developed were used to predict the annual peak flows which would have occurred under various urbanized conditions.

Step	Variable Entered	R	R^2x100	Increase in R^2	Signif.
1.	Log. Max 30-min Prec.	.693	48.0	48.0	90 %
2.	Log. Precip. Depth	.813	66.1	18.1	99.9 %
3.	Temperature	.853	72.8	6.8	99.9 %
4.	Impervious Area	.869	75.5	2.7	95 %
5.	Log. A.P.I.	.879	77.3	1.8	90 %
6.	Log. Duration	.885	78.3	1.0	80 %
7.	Log. Max 15-min Prec.	.886	78.5	0.3	-

Table 6.15 Steps in the Regression Analysis of Peak Flow using Seven Independent Variables.

Variable	Regression Coefficient	Signif
(Constant)	-1.7277	
Impervious Area	0.0115	95.0%
Temperature	-0.0309	99.5%
Log. Precip Depth	1.3046	99.5%
Log. Max 30-min Prec.	1.0899	90.0%
Log. Max 15-min Prec.	-0.4897	-
Log. Duration	-0.4077	80.0%
Log. A.P.I.	0.1360	90.0%

Table 6.16 Best Regression Equation for Predicting Peak Flow (Log) using Seven Independent Variables.

Step	Variable Entered	R	R^2x100	Increase in R^2	Signif.
1.	Log. Precip. Depth	.655	43.0	43.0	99.9 %
2.	Log. Duration	.764	58.4	15.4	99.9 %
3.	Impervious Area	.829	68.8	10.4	95.0 %
4.	Temperature	.864	74.6	5.8	99.9 %
5.	Log. A.P.I.	.870	75.7	1.1	90.0 %

Table 6.17 Steps in the Regression Analysis of Peak Flow using Five Independent Variables.

Variable	Regression Coefficient	Signif.
(Constant)	-1.5796	
Impervious Area	0.0105	95.0 %
Temperature	-0.0280	99.9 %
Log. Precip Depth	1.8570	99.9 %
Log. Duration	-0.9211	99.9 %
Log. A.P.I.	0.1250	90.0 %

Table 6.18 Best Equation For Predicting Peak Flow (Log) using Five Independent Variables.

Table 6.19 lists the ranked flows predicted by the equations, for three percentages of impervious area : 0 per cent, representing rural conditions, 20 per cent, and 40 per cent, representing fully urbanized conditions. Although the 40 per cent limit could be exceeded in the actual catchment, it was felt that the equations would not be reliable in this range as they were based on data with a range of impervious area of 5.8 per cent to 29.2 per cent.

The return periods were calculated using Equation 6.i with N = 20. The plotted points are shown in Figure 6.4 and the frequency curves shown were fitted by eye. After comparison with Figure 6.2 it can be seen that multiple regression offers a better approach to the problem. The three sets of points are noticeably distinct, despite the somewhat wide scattering of points for high flows. The effect of urbanization on the floods of selected recurrence intervals can be extracted from Figure 6.4. These are presented in Table 6.20. As the high flows were fairly widely scattered, extrapolation beyond the 25-year flood was not attempted.

In order to indicate whether the curves are significantly different, confidence limits (or control curves) may be placed around them. These were calculated using the equations and functions given by Chow (1964, p.8.32), and are presented in Figure 6.5. As the biggest storm on record (September 1968) was ignored in the fitting of the frequency curves, it was also ignored in the calculation of the confidence limits, the magnitude of which depends to a large extent on the variance of the

Return Period	Rank	Impervious Area 0%	20%	40%
21.0	1	7.431	12.596	21.352
10.5	2	3.165	5.365	9.095
7.00	3	2.466	4.181	7.087
5.25	4	2.426	4.113	6.971
4.20	5	2.251	3.816	6.469
3.50	6	1.656	2.688	4.444
3.00	7	1.547	2.622	4.404
2.63	8	1.533	2.598	4.364
2.33	9	1.446	2.451	4.155
2.10	10	1.439	2.440	4.136
1.91	11	1.356	2.300	3.897
1.75	12	1.294	2.193	3.717
1.62	13	1.269	2.152	3.647
1.50	14	1.047	1.716	2.909
1.40	15	1.012	1.700	2.837
1.13	16	0.987	1.674	2.760
1.24	17	0.958	1.555	2.531
1.17	18	0.881	1.493	2.524
1.11	19	0.795	1.348	2.284
1.05	20	0.625	1.060	1.767

TABLE 6.19

Annual Flow Peaks Predicted by Multiple Regression Equation for various percentages of impervious area

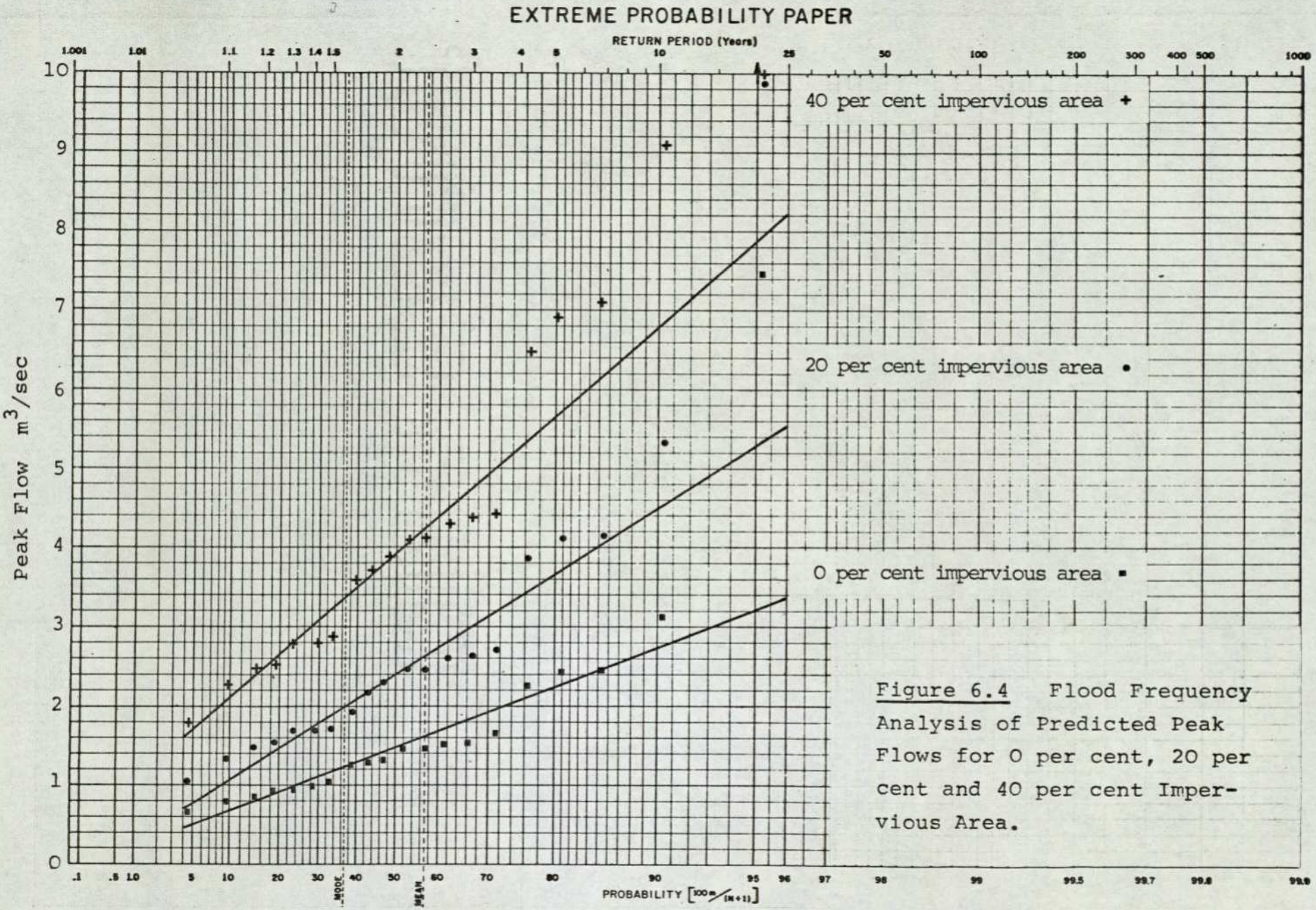


Figure 6.4 Flood Frequency Analysis of Predicted Peak Flows for 0 per cent, 20 per cent and 40 per cent Impervious Area.

Return Period		Impervious area. 0%	20%	40%
$Q_{2.33}$	m^3/s	1.60	2.60	4.23
	ratio	1.00	1.62	2.64
Q_5	m^3/s	2.22	3.63	5.61
	ratio	1.00	1.64	2.53
Q_{10}	m^3/s	2.72	4.48	6.80
	ratio	1.00	1.65	2.50
Q_{25}	m^3/s	3.36	5.52	8.20
	ratio	1.00	1.64	2.44

Table 6.20 Peak flow magnitudes and ratios for three levels of impervious area.

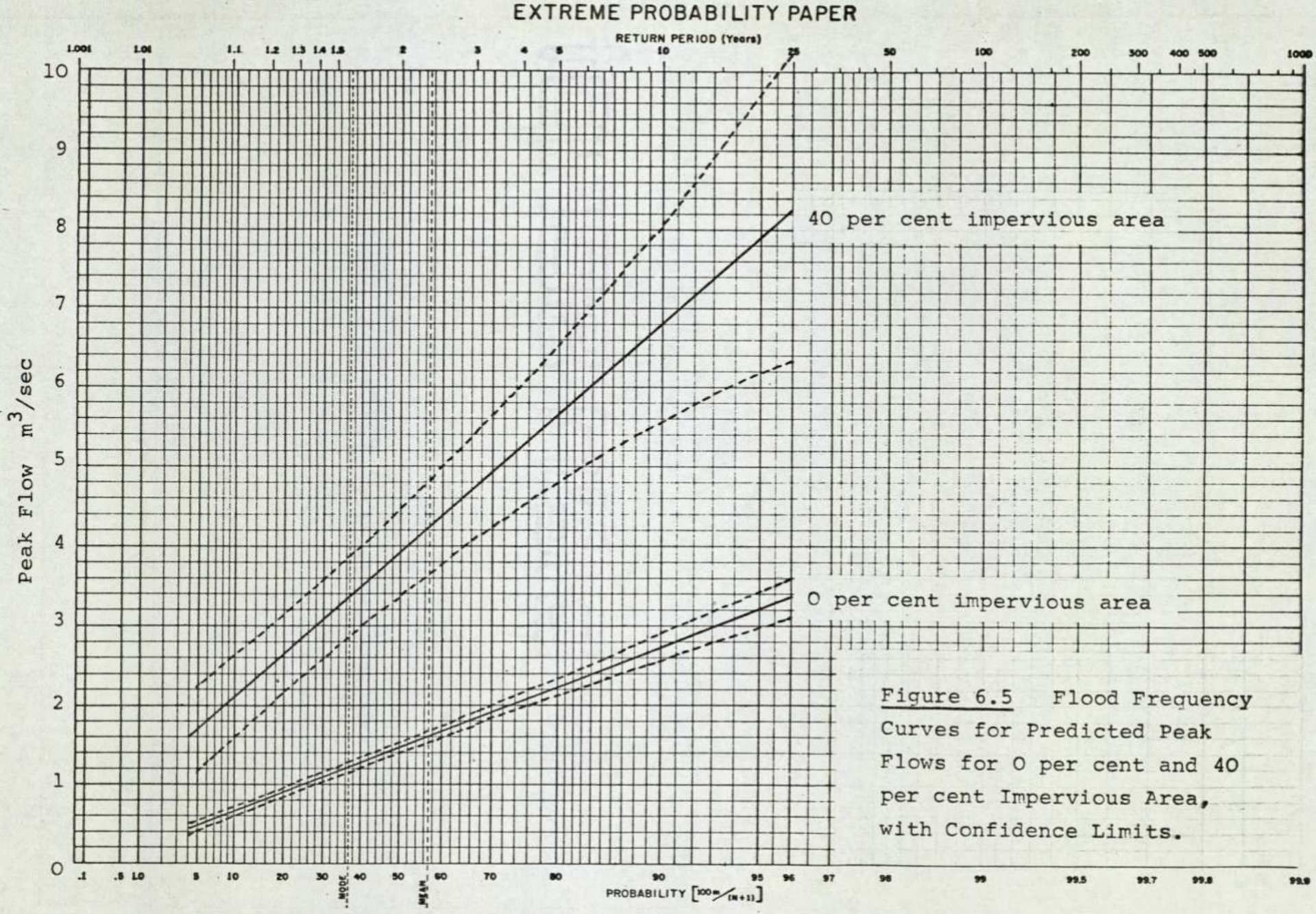


Figure 6.5 Flood Frequency Curves for Predicted Peak Flows for 0 per cent and 40 per cent Impervious Area, with Confidence Limits.

original data.

Table 6.21 gives the standard errors for various points on the two curves. These points were joined by smooth curves to form the control curves, which represent confidence limited corresponding to one standard error, or 68.3 per cent. The limits for the mean annual flood ($Q_{2.33}$) are $\pm$ 0.6 m^3/sec for the urban curve and $\pm$ 0.067 m^3/sec for the rural curve. Assuming that the calculated values of $Q_{2.33}$ for a number of data series are normally distributed, it is possible to state that the estimated mean annual flood for urbanized conditions has a 95 per cent chance of being within 1.2 m^3/sec (two standard errors) of 4.24 m^3/sec, and that the mean annual flood for rural conditions has a 95 per cent chance of being within 0.134 m^3/sec of 1.6 m^3/sec. Clearly, the 95 per cent confidence limits of these two values do not overlap and it is highly probable that the two curves are significantly different. At higher return periods the control curves veer away from the straight line, but even here, the 95 per cent confidence bands do not overlap. To take the twenty-year flood as an example, for the rural curve, the limits are 2.76 to 3.64 m^3/sec, while for the urban curve, they are 5.98 to 13.82 m^3/sec.

It can be concluded that the multiple regression analysis demonstrates that urban development has an important effect on flood frequency for Crawters Brook, causing the mean annual flood to be increased by a factor of 2.64 and the 25-year flood by a factor of 2.44.

Return Period	Confidence limits for stated Impervious Area (One Standard Error)	
	0 %	40 %
20	1.96	0.22
10	1.43	0.16
7.0	1.02	0.11
3.0	0.67	0.076
2.33	0.60	0.067
1.25	0.48	0.054
1.05	0.56	0.063

Table 6.21 Confidence Limits for the Flood Frequency Curves for 0% and 40% Impervious Area.

Source: Derived from equations given in Chow, 1964, pp. 8.32-8.33.

6.II.3. Analysis of Flow Exceedences. In this part of the study, the number of peak flows in each year exceeding a certain value were analysed. A flow of 0.5 m^3/sec was chosen as the threshold level because it gives a good distribution of peaks, with at least seven in each year. The distribution is shown in Figure 6.6, with a graph of the increase in the percentage of impervious area in the catchment. It would appear that the number of peaks increases with urbanization, and this hypothesis was tested by regression analysis. Gaps in the flow data were augmented by using data from the gauge at Hazelwick Roundabout, a few hundred metres away (as shown in Figure 4.2).

A simple linear regression was carried out using only impervious area as the independent variable and the number of peaks in each year as the dependent variable. This equation yielded a correlation coefficient of 0.48, indicating that 23 per cent of the variance was explained by impervious area. As impervious area is obviously not the only variable contributing to the variance, it was decided to introduce the annual rainfall in an attempt to increase the explained variance. This resulted in a multiple correlation coefficient of 0.59 as shown in Table 6.22. Furthermore, as the distribution of the number of exceedences was positively skewed, the data was transformed using a logarithmic transformation. This resulted in a multiple correlation coefficient of 0.65, or an explained variance of 43 per cent, a considerable improvement on the first equation.

The regression coefficients, standard errors and significance levels are presented in Table 6.23. It can be concluded

Independent Variables	Dependent Variable	
	Number of Peaks (N_p)	Log(N_p)
Impervious Area	.477	.552
Impervious Area and Annual Rainfall	.591	.652

Table 6.22 Correlation Coefficients between Variables in the Regression Analysis of Number of Exceedences (N_p).

	Regress. Coeff.	Stnd. Error	Signif
(Constant)	.3376		
Impervious Area	.0151	.00537	99%
Annual Rainfall	.000652	.000345	95%

Table 6.23 Best Regression Equation for Predicting Number of Exceedences (Log.N_p)

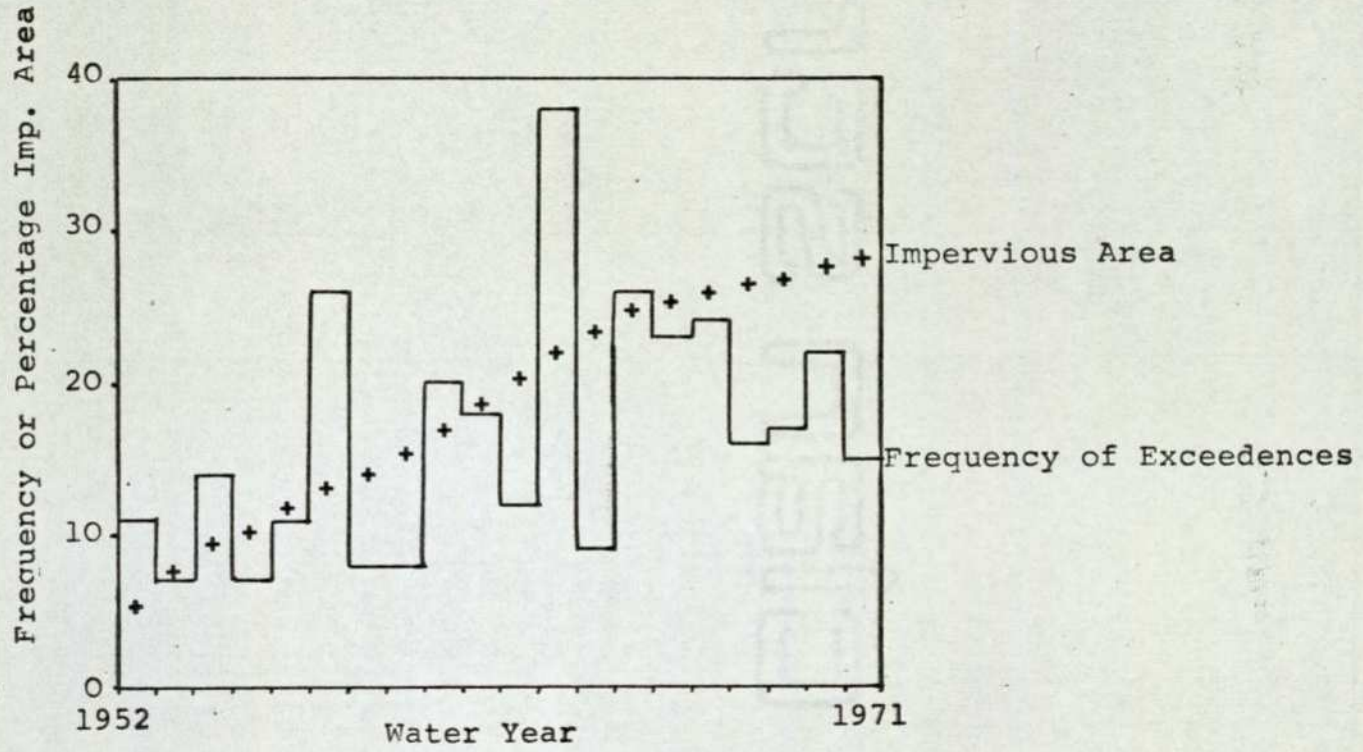


Figure 6.6 Histogram of annual Exceedences of 0.5 m^3/sec and percentage Impervious Area.

that the percentage of impervious area in the Crawters Brook catchment contributes significantly to the annual number of exceedences of the threshold flow rate of 0.5 m^3/sec.

CHAPTER SEVEN

SUMMARY AND CONCLUSIONS

The simulation of storm events on the Crawters Brook catchment and the optimization of the simulation model are discussed in the first section of this chapter. The second section presents the conclusions to be drawn from the multiple regression analysis of mean daily flow and flow peaks. Finally, recommendations are made regarding fruitful areas of research.

7.I. SIMULATION AND OPTIMIZATION The Hybrid computer simulation model and the associated optimization procedure performed well, and on the whole, storms were adequately simulated. The regression analysis of parameter values showed that urbanization had an effect on all four parameters in the simulation model. The regression equations for this part of the study are summarized in Table 7.1.

The prediction equation for the routing parameter had the best multiple correlation coefficient of the four, and the percentage of impervious area in the catchment was the most significant independent variable. The equation for the maximum infiltration capacity also had a relatively high multiple correlation coefficient, but impervious area was less important than the seasonal effects of soil-moisture content represented by temperature. This is perhaps to be expected for this para-

$$k_r = 0.5681 + .0276(I) + .0475(T) - .0241(A) - .0210(R) - .0039(D_n) + .1629(M)$$

$$f_o = -2.742 - .0418(I) + .3307(T) + .0117(A) + .0150(R) + .2158(D_n)$$

$$f_c = 0.7973 - .0278(I) + .0300(T) - .0134(A) + .0245(R) - .0271(D_n) - .0183(M)$$

$$D = -0.4801 + .0841(I) - .0013(T) - .0109(A) + .0417(R) - .0376(D_n) + .2321(M)$$

where I = Impervious Area (%)

T = Temperature

A = Antecedent Precipitation Index

R = Precipitation Depth

D_n = Precipitation Duration

M = Maximum Precipitation Intensity

k_r = Routing Constant

f_o = Max. Infiltration Capacity

f_c = Min. Infiltration Capacity

D = Depression Storage Capacity

Table 7.1 Regression Equations for predicting values of the Model Parameters.

meter. The equations for depression storage capacity and minimum infiltration capacity were respectively less significant than the equations for the other two parameters, but impervious area was again the most significant independent variable.

It can be seen from the signs of the regression coefficients in Table 7.1. that urbanization caused an increase in the value of the routing parameter (increasing 'peakedness') and the depression storage capacity, while the infiltration capacity parameters were decreased. All these effects confirm what would intuitively be expected. The equations given may be used to predict the values of the parameters of the hybrid computer simulation model, given the design storm and the design catchment conditions. With these parameter values substituted into the model, the design storm can be input to the model to yield the predicted surface runoff hydrograph.

In order to extend the model for use on other ungauged catchments of similar character, an adjustment factor could be applied to the computed precipitation excess from the model to take account of differing catchment areas. The routing parameter would also need to be changed to take account of variables such as catchment area and slope. These latter variables would therefore have to be included in the prediction equation for the routing parameter by applying the model to several catchments, optimizing the routing parameter and carrying out a further regression analysis. This would be a useful area of study for further research.

The hybrid computer proved to be a good research tool.

It enabled the hydrological processes represented by the model to be viewed simultaneously on a visual display unit, and this probably gave more insight into the working of the model and the interaction of various parameters and processes than attempting to interpret reams of digital computer printout. Using the computer 'on-line' also meant that the optimization of the model could be halted and restarted if convergence was not taking place. Conversely, if convergence had taken place but the convergence was not sufficient to halt the programme on the basis of the convergence criterion, the procedure could be terminated and the parameter values displayed.

The practical disadvantages of the hybrid computer were found to be lack of accuracy at small magnitudes, leading to difficulties in optimizing very near to the optimum of the objective function. Another disadvantage was the inconvenience of the computer programme on the analogue patch-panel compared with a digital programme consisting of a deck of cards.

The sensitivity analysis was easily performed on the hybrid computer, and combined with the response surfaces, demonstrated the interaction between model processes. The resulting sets of hydrographs and plots could also be used to aid subjective optimization of the model in that they graphically depict the effect of varying individual parameters.

The Simplex optimization procedure performed well (although it was not compared with other methods) and a considerable degree of control could be exercised over the optimization by the use of the control parameters and the capability of the

hybrid computer. The philosophy of deriving model parameters representing actual physical values from only two data series (precipitation and streamflow) could be questioned. If parameters are strongly correlated, or if a parameter has no effect on streamflow for a particular storm, then unrealistic values may be obtained by automatic optimization. These values may give a better 'fit' than those obtained using subjective optimization, but they may be inconsistent from storm to storm. There is really no way to solve this problem, but the adverse effects of depending only on automatic optimization methods and a single criterion of fit may be lessened by applying subjective constraints on certain parameters.

7.II. REGRESSION ANALYSIS OF DAILY AND PEAK FLOW. The analysis of mean daily flow showed urbanization to have a highly significant effect. The equation with a square-root transform of mean daily flow (summarized in Table 7.2) gave the best results, explaining 78 per cent of the variance. It showed that urbanization caused an increase in mean daily flow. The equation could be used to predict long term monthly yields from an urbanizing catchment, but it is felt that its use in predicting daily values would be hampered by the inadequate representation of short term processes affecting flow.

The use of regression analysis to form a predictive equation for peak flow (summarized in Table 7.3) enabled flood frequency curves to be derived for rural and urbanized conditions. The analysis showed that impervious area was a significant

$$Q_d^{\frac{1}{2}} = 3.325 + .4396(R) + .0896(R_r) + .0738(A) - .1583(T) + .0348(I)$$

where Q_d = Daily Flow

R = Daily Precipitation

R_r = Precipitation on Previous Day

A = Antecedent Precipitation Index

T = Temperature

I = Impervious Area (%)

Table 7.2 Regression Equation for Predicting Daily Flow.

$$\text{Log}(Q) = -1.7277 + .0115(I) - .0309(T) + 1.305(\text{Log}(R)) - 1.090(\text{Log}(M_{30})) - .4897(\text{Log}(M_{15})) - .4077(\text{Log}(D_n)) + .1360(\text{Log}(A))$$

where Q = Peak Flow

I = Impervious Area

T = Temperature

R = Precipitation Depth

M_{30} = Maximum 30-minute Precipitation Intensity

M_{15} = Maximum 15-minute Precipitation Intensity

D_n = Precipitation Duration

A = Antecedent Precipitation Index

Table 7.3 Regression Equation for predicting Peak Flow

variable affecting peak flow, although rainfall depth and duration, and the time of year (represented by temperature) were, not surprisingly, more significant. The equation explained about 76 per cent of the variation in peak flow. In addition to the calculation of flood frequency curves the equation may also be used to predict peak flow from design storm characteristics and catchment conditions. The analysis of flow exceedences demonstrated that urbanization had a significant effect on the annual number of exceedences of the threshold flow rate of 0.5 m^3/sec.

7.III. RECOMMENDATIONS. The study has demonstrated the effect of urbanization on streamflow. The potential use of the simulation could be broadened by studying other catchments which have undergone urbanization in order to bring more catchment variables into the prediction equations. These variables would comprise area, slope, shape factors, drainage density and vegetation, for example. Some measure of changes in the spatial distribution of impervious area within the catchment through time may also be useful. Unfortunately, data quality and availability are the main stumbling blocks in broadening the scope of the study. The extensive data collection network and documentation associated with the construction of the new city of Milton Keynes in Central England should provide valuable data in the years to come. This data network should include precipitation stations which are representative of the

subcatchments in the new town, and evaporation stations to enable modelling of a more complete representation of the catchment hydrological processes.

APPENDIX

Principal Computer Programmes		Page
STREAM	to convert digitized records to values of streamflow	245
CHECK	to check digitized records for errors	250
TRAN	to subtract baseflow from the hydrograph	252
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ANALOG	to operate analogue computer	254
B34F	to calculate the objective function	255
SIMPLEX	to perform the optimization	256

```
      PROGRAM STREAM
     A(INPUT, OUTPUT, TAPE 1 = INPUT, TAPE 2 = OUTPUT)
C
C     THIS PROGRAM CONVERTS THE CONTINUOUS DIGITIZED
C     DATA TO HOURLY AND DAILY RUNOFF VALUES AND
C     PLOTS THESE ON THE LINE PRINTER
C
C       VARIABLES:
C       IPL     RECORD I.D. NUMBER
C       IH      TIME OF START OF RECORD
C       ID/IM   DAY AND MONTH OF START
C       NDAY    NUMBER OF DAYS IN MONTH
C       IFEB    EQUALS '1' IF LEAP YEAR
C       NP,IP   NUMBER OF POINTS DIGITIZED IN RECORD
C       NPER    NUMBER OF RECORDS TO BE RUN
C       LXY     LEFT-HAND CORNER CO-ORDINATES
C       RXY     RIGHT-HAND CORNER CO-ORDINATES
C       QXY     HYDROGRAPH TRACE CO-ORDINATES
C       EXY     CO-ORDINATES OF SILTING ERROR
C       TC      LENGTH OF TRACE IN D-MAC UNITS
C       HRS     LENGTH OF TRACE IN HOURS
C       RLT     LENGTH OF TRACE IN MINUTES
C       ERR     SILTING ERROR IN D-MAC UNITS
C       HERR    SILTING ERROR IN CUMEC
C       Q0,Q1   SUCCESSIVE RUNOFF VALUES IN CUMEC
C       T0,T1   TIME OF OCCURENCE OF Q0 AND Q1
C       TOT,TT,MIN1,MIN2,QM,QT ARE VARIABLES USED
C               TO CALCULATE HOURLY AND DAILY RUNOFF
C
      DIMENSION IPL(10),IH(10),ID(10),IM(10),NP(10),
     1LXY(10),RXY(10),EXY(10),QXY(200,10)
      REAL LXY,LX,LY
      DOUBLE PRECISION TOT,TT,T
      COMMON/BLK/HR,TOT,QTT
      DATA S1,S2,S3/752.11,758.53,756.36/
     1,HI,HQT,HF/3H I=,3HQT=,3H F=/
C
C     READ NUMBER OF RECORDS AND IFEB
      READ(1,1)NPER,IFEB
      DO 5 L=1,NPER
C     READ LEAD CARD
      READ(1,2)IPL(L),IH(L),ID(L),IM(L),NP(L)
     1 ,LXY(L),RXY(L),EXY(L)
C     GAP IN RECORD IF IPL = 0
C     IF THERE IS A GAP IN RECORD, CONTINUE
      IF(IPL(L).LE.0)GOTO5
      IP=NP(L)
C     READ DATA CARDS
      READ(1,3)(QXY(I,L),I=1,IP)
5     CONTINUE
C     PRINT AXIS AND LABELS
      WRITE(2,26)
      WRITE(2,28)
      WRITE(2,27)
```

```
C
C       CALCULATE FLOWS AND PLOT USING SUBROUTINE 'FLOW'
C
        NPE=NPER-1
         DO 7 L=1,NPE
        QTT=0.
        QH=0.
        Q1=0.
        T1=0.
        TT=0.
        IF(IPL(L).LE.0)GOTO4
C
C       CALCULATE BASE DIMENSIONS OF EACH HYDROGRAPH
        IF(IPL(L+1).GT.0)GOTO20
        CALL SEP(LXY(L+1),LX,LY)
        CALL SEP(RXY(L+1),RX,RY)
        IH(L+1)=IFIX(LX)
        ID(L+1)=IFIX(LY)
        IM(L+1)=IFIX(RX/100.)
20        CALL SEP(LXY(L),LX,LY)
        CALL SEP(RXY(L),RX,RY)
        CALL SEP(EXY(L),EX,EY)
        TC=SQRT((LX-RX)**2+(LY-RY)**2)
C       CALCULATE SILTING ERROR
        ERR=SQRT((LX-EX)**2+(LY-EY)**2)
        HERR=200.*ERR/S1/2118.8862
C       PRINT RECORD HEADING
        WRITE(2,21)IH(L),ID(L),IM(L),HERR
C
C       CALC. LENGTH OF TIME AXIS IN MINUTES USING TIME
C       OF START OF THIS AND NEXT RECORD
        IH2=IH(L)/100
        IH1=IH(L)-IH2*100
        P=33.
        IF(IH2.LT.9.OR.IH(L).EQ.900)P=9.
        HRS=P-FLOAT(IH2)-FLOAT(IH1)/60.
        HR=-HRS
        MINDIF=60*(IH(L+1)/100-IH2)+IH(L+1)-IH(L+1)/
     1  100*100-IH1
        IF(IM(L).NE.IM(L+1))GOTO30
        NDAY=0
        GOTO32
30      IF(IM(L).EQ.12)GOTO31
        IF(IM(L).EQ.2)GOTO33
31      NDAY=31
        IF(IM(L).EQ.4.OR.IM(L).EQ.6.OR.IM(L).EQ.9.OR.IM(L)
     1  .EQ.11)NDAY=30
        GOTO32
33      NDAY=28
        IF(IFEB.EQ.1)NDAY=29
32      LT=MINDIF+(ID(L+1)+NDAY-ID(L))*1440
        MIN1=IFIX((HRS-FLOAT(IFIX(HRS)))*60.)
        MIN2=LT-MIN1-(LT-MIN1)/60*60
        RLT=FLOAT(LT)
        P=2.*TC
```

```
C
C       BEGIN CALCULATION OF FLOWS
        IP=NP(L)
        DO 6 I=1,IP
C       CALCULATE FLOW RATE FROM D-MAC COORDS.
        CALL SEP(QXY(I,L),QX,QY)
        AS=(QX-RX)**2+(QY-RY)**2
        BS=(QX-LX)**2+(QY-LY)**2
        T0=T1
        Q0=Q1
        T1=(BS+TC**2-AS)/P
        Q1=SQRT(BS-T1**2)
C
C       CONVERT D-MAC UNITS TO CUMEC
8       IF(Q1.GT.S1)GOTO9
C       LOWER SCALE
        Q1=.125499212E-03*(Q1-ERR)
        GOTO10
9       IF(Q1.GT.S1+S2)GOTO11
C       MIDDLE SCALE
        Q1=.09438921+.11199332E-02*(Q1-S1)-HERR
        GOTO10
C       UPPER SCALE
11      Q1=.94389213+.4991761ZE-02*(Q1-S1-S2)-HERR
10      IF(I.EQ.1)GOTO6
C       CALCULATE TIME AXIS INTERVALS
        TOT=(RLT*T1-RLT*T0)/TC
        IF(TOT.LT.0.)TOT=0.0001
16      IF(MIN1.NE.0)GOTO14
        IF(TOT+TT.LT.60.)GOTO13
C       FOR HOURLY FLOW
        QUAN=HQT
        T=60. -TT
17      QM=Q0+T*(Q1-Q0)/TOT
        QT=T*(QM+Q0)/2.+QH
        TOT=TOT-T
        Q0=QM
        T=T+TT
C
C       CALL PLOTTING SUBROUTINE
        CALL FLOW(QT,T,QUAN,IPL(L))
        TT=0.
        QH=0.
        GOTO16
C       ADJUST FOR RECORD OF LESS THAN ONE WEEK
14      IF(TOT+TT.LT.FLOAT(MIN1))GOTO13
C       FOR FLOW UP TO INITIAL WHOLE HOUR
        QUAN=HI
        T=FLOAT(MIN1) -TT
        MIN1=0
        GOTO17
13      QH=TOT*(Q0+Q1)/2.+QH
        TT=TOT +TT
6       CONTINUE
```

```
      IF(TT.LT.1.)GOTO7
C     FOR FLOW AFTER FINAL WHOLE HOUR
      CALL FLOW(QH,TT,HF,IPL(L))
      GOTO7
C     PRINT MESSAGE FOR GAP IN RECORD
4     WRITE(2,29)
7     CONTINUE
C
C     PRINT AXIS AND LABELS
      WRITE(2,27)
      WRITE(2,28)
      STOP
C
1     FORMAT(I3,I2)
2     FORMAT(I3,1X,I4,1X,I2,I2,3X,I4,3X,1X,3F8.0)
3     FORMAT(8F8.0)
21    FORMAT(1X,I4,I4,1H/,I2,85X,4HERR=,F10.4)
26    FORMAT(1H2,40X,18HSTREAMFLOW (CUMEC))
27    FORMAT(4X,38H+.....+.............+.......+.......
     1,43H..+.........+.........+.........+.........+)
28    FORMAT (3X,37H0.0   0.001           0.010    0.030
     15X,44H0.100     0.315     1.000     3.162     10.00
     2,1H0)
29    FORMAT(1H ,10HGAP.......)
      END



      SUBROUTINE SEP(XY,X,Y)
C
C     THIS SUBPROGRAM SEPARATES ONE 8-FIGURE NUMBER
C     INTO TWO 4-FIGURE CO-ORDINATES
C
      X=FLOAT(IFIX(XY/10000.))
      Y=XY-X*10000.
      RETURN
      END
```

```
      SUBROUTINE FLOW(QT,T,QUAN,IPI)
C
C     THIS SUBPROGRAM PRINTS THE GRAPH
C
C       VARIABLES
C       A       ARRAY CONTAINING BLANKS AND ASTERISK
C       BK      BLANK SPACE
C       AST     ASTERISK
C       MAG     POSITION OF ASTERISK IN HYDROGRAPH PLOT
C       QTC     AVERAGE HOURLY RATE OF FLOW
C       QTT     AVERAGE DAILY OF FLOW
C
      DIMENSION A(80)
      DOUBLE PRECISION TOT,TT,T
      COMMON/BLK/HR,TOT,QTT
      DATA AST,BK,A/1H*,1H ,80*1H /,HF/3H F=/
      QTC=QT/T
      IF(QTC.LT.0.)QTC=0.
C
C     CALCULATE POSITION OF ASTERISK IN ARRAY 'A'
      MAG=IFIX(20.*ALOG10(1000.*QTC+1.))
      IF(MAG.LT.1)MAG=1
      IF(MAG.GT.80)MAG=80
      A(MAG)=AST
      HR=HR+T/60.
      IF(T.GT.59.99.OR.QUAN.EQ.HF)GOTO2
C     PLOT AND PRINT HOUR'S FLOW
5     WRITE(2,3)IPI,(A(J),J=1,80),QUAN,QTC
      A(MAG)=BK
      RETURN
2     IF(HR.LT.0.)GOTO5
      QTT=QTT+QTC
      IF(HR.LT.24..AND.QUAN.NE.HF)GOTO5
      QTT=QTT/HR
C     PLOT AND PRINT HOUR'S AND DAY'S FLOW
      WRITE(2,4)IPI,(A(J),J=1,80),QUAN,QTC,QTT
      A(MAG)=BK
      QTT=0.
      HR=0.
      RETURN
3     FORMAT(1H ,I3,1H.,80A1,A3,F8.4)
4     FORMAT(1H ,I3,1H.,80A1,A3,F8.4,/H    QTT=,F8.4)
      END
```

```
      PROGRAM CHECK
     A(INPUT, OUTPUT, TAPE 1 = INPUT, TAPE 2 = OUTPUT)
C
C     THIS PROGRAM CHECKS THE DATA CARDS OUTPUT FROM
C     THE D-MAC DIGITIZER
C
C       VARIABLES:
C         IPL     RECORD I.D. NUMBER ON LEAD CARD
C         T       TIME AXIS CO-ORDINATE
C         NP      NUMBER OF POINTS DIGITIZED IN RECORD
C         NCP     NUMBER OF CARDS IN RECORD
C         T       TIME OF START OF RECORD
C     ID/IM/IY    DATE OF START OF RECORD
C         NC      TIME AXIS CO-ORDINATE OF FIRST POINT
C                 ON EACH CARD
C         NPER    NUMBER OF RECORDS TO BE CHECKED
C         G=H     DURATION OF GAP IN RECORD
C         K       NUMBER OF GAPS IN RECORD
C         IPD     RECORD I.D. NUMBER ON DATA CARDS
C
      DIMENSION IPL(200),T(200),ID(200),IM(200),IY(200),
     1G(6),H(30,6),NC(80),NP(200)
      L=0
      K=0
C
C     READ NUMBER OF RECORDS TO BE CHECKED
      READ(1,2)NPER
      DO 3 I=1,NPER
      IF(L.GT.0)GOTO10
C
C     READ LEAD CARD
      READ(1,1)IPL(I),T(I),ID(I),IM(I),IY(I),NP(I),NCP,
     1(G(J),J=1,6)
10    L=0
C     IF IPL=0 THERE IS A GAP IN THE RECORD
      IF(IPL(I))4,4,5
4     K=K+1
C     STORE DURATION OF GAP IN RECORD
      DO 6 J=1,6
6     H(K,J)=G(J)
      GOTO3
C
C     READ DATA CARDS
5     DO 7 J=1,NCP
      READ(1,8)NC(J),IPD
C
C     CHECK IF I.D. NUMBERS ARE THE SAME
      IF(IPD.NE.IPL(I))GOTO20
      IF(J.LT.3)GOTO7
C
C     CHECK ORDER OF CARDS
      IF(NC(J-1).LT.NC(J-2).OR.NC(J-1).GT.NC(J))GOTO22
      GOTO7
```

```
C       PRINT "ERROR 2" FOR CARDS OUT OF ORDER
22      WRITE(2,23)IPL(I),NC(J-1)
        GOTO26
C       PRINT "ERROR 1" FOR I.D. NUMBERS NOT MATCHING
20      WRITE(2,21)IPL(I),NC(J)
26      L=I+1
C       READ CARDS UNTIL NEXT RECORD, THEN CONTINUE
27      READ(1,1)IPL(L),T(L),ID(L),IM(L),IY(L),NP(L),NCP
       1,(G(J),J=1,6)
        IF(IPL(L).NE.IPL(I)+1.AND.IPL(L).NE.0)GOTO27
        GOTO3
7       CONTINUE
3       CONTINUE
C
C       PRINT RESULT OF CHECK
        K=0
        WRITE(2,11)
        DO 12 I=1,NPER
C       PRINT DETAILS OF RECORD
        IF(IPL(I).EQ.0)GOTO13
        WRITE(2,14)IPL(I),T(I),ID(I),IM(I),IY(I),NP(I)
        GOTO12
C       PRINT DETAILS OF GAPS, IF ANY
13      K=K+1
        HR=(FLOAT(IY(I))*10000.+FLOAT(NP(I)))/10000.
        WRITE(2,15)ID(I),HR,(H(K,J),J=1,6)
12      CONTINUE
C
1       FORMAT(I3,1X,A4,1X,A2,2I2,1X,I4,I3,1X,6A4)
2       FORMAT(I3)
8       FORMAT(I4,60X,I3)
11      FORMAT(1H0,28HPERIOD TIME    DATE  NO.PTS.)
14      FORMAT(1H ,I4,3X,A4,3X,A2,2I2,I6)
15      FORMAT(1H ,35H.....
       1,7HGAP OF ,A2,5H DAYS,F7.2,5H HRS.,7H FROM  ,3A4
       2,8H  TILL  ,3A4)
21      FORMAT(1H ,5HERR 1,5X,2HP=,I3,5X,2HC=,I3)
23      FORMAT(1H ,5HERR 2,5X,2HP=,I3,5X,2HC=,I3)
        END
```

```
      PROGRAM TRAN
        DIMENSIØN IRØ(500)
      ACCEPT 4,NPTØT,NPSRØ,VAL1,VAL2
4     FØRMAT(I3/I3/F4.0/F4.0)
      READ(4,3)(IRØ(I),I=1,NPTØT)
3     FØRMAT(1216)
      BASE=(VAL2-VAL1)/FLØAT(NPSRØ)
      NS=NPTØT-NPSRØ
      DØ 1 I=1,NS
1     IRØ(I)=0
      NS1=NS+1
      DØ 2 I=NS1,NPTØT
2     IRØ(I)=IRØ(I)-IFIX(VAL1+BASE*FLØAT(I-NS1+1)
     1)
      PAUSE 2
      WRITE(53)(IRØ(I),I=1,NPTØT)
      STØP
      END
```

```
      PROGRAM STAT
        DIMENSIØN P(26),A(24),V(26)
       DATA P(1),P(2),P(3),P(4),P(5),P(6)/4HP032,4HP035,4HP036,4HP091
      1,4HP095,4HP075/
      2,A(1),A(2),A(3),A(4),A(5),A(6),A(7),A(8),A(9),A(10),A(11),A(12)
      3,A(13),A(14)/4HA030,4HA031,4HA032,4HA036,4HA038,4HA041,4HA044
      4,4HA060,4HA064,4HA066,4HA069,4HA070,4HA095,4HA096/
      5,V(1),V(2),V(3),V(4),V(5),V(6)/.1,.2,.5,.95,.6,.15/
      6,P(7),P(8),V(7),V(8)/4HP077,4HP090,.1,.1/
       TYPE 102
102   FØRMAT(10H SSW CLØCK/)
      CALL QSHYIN(IERR,680)
      CALL QSC(1,IERR)
      CALL QSRUN(IERR)
      CALL QSSECF(IERR)
      CALL QWCLL(0,.TRUE.,IERR)
      CALL  QWPR(4HP045,0.0004,IERR)
      DØ 10 I=1,8
10    CALL QWPR(P(I),V(I),IERR)
      CALL QWPR(4HP061,0.5,IERR)
      CALL QSST(IERR)
      DØ 20 I=1,14
      CALL QRAR(A(I),U,IERR)
20    TYPE 100,A(I),U
100   FØRMAT(1H ,A4,F8.4)
      CALL QSPC(IERR)
      CALL QWCLL(0,.FALSE.,IERR)
      STØP
      END
```

```
      PROGRAM MAIN
      LOGICAL SENSW
      DIMENSION RF(250),RO(250),
     1X(16,15),S(15),TES(16),FRF(5),FRO(5)
     2,Q(250)
      COMMON/BLK1/SCALE,NTRUE
      COMMON/BLK2/X,S
      COMMON/BLK3/TES
      COMMON/BLK4/I,J,K,L,N,IN,IO
      COMMON/BLK5/N1,IPR,INUMA,CONV1
      COMMON/BLK6/KT,FUNC
      COMMON/BLK7/NN,INUM
      COMMON/BLK8/RF,RO,Q,NP,QMS
      COMMON/BLK9/VAR
      CALL QSHYIN(IERR,680)
      CALL QSC(1,IERR)
      IN=4
      IO=3
      NTRUE=5
      QMO=0.
      PTOT=0.
      QOT=0.
      QST=0.
      VAR=0.
      TYPE 8888
8888  FORMAT(1HS)
      ACCEPT 9998,SC
      READ(IN,2)NP,(FRF(L),L=1,5),(FRO(L),L=1,5)
2     FORMAT(I3/5A4/5A4)
      READ(IN,FRF)(RF(I),I=1,NP)
      READ(IN,FRO)(Q(I),I=1,NP)
      CALL QWPR(4HP097,0.36,IERR)
      V40=2./(FLOAT(NP)*0.2)
      CALL QWPR(4HP040,V40,IERR)
      DO 73 I=1,NP
      Q(I)=Q(I)/10000./SC
      IF(Q(I).GT.QMO)QMO=Q(I)
      RF(I)=RF(I)*12.309905/SC
      QOT=QOT+Q(I)
73    PTOT=PTOT+RF(I)
      XQ=QOT/FLOAT(NP)
      DO 75 I=1,NP
75    VAR=VAR+(XQ-Q(I))**2
      VAR=VAR/FLOAT(NP)
1     TYPE 9995
      ACCEPT 9999,INUMA,IPR,N1
9995  FORMAT(4HINUM/3HIPR/2HN1)
9999  FORMAT(I5)
      TYPE 9993
9993  FORMAT(3HCON/3HSCA)
      ACCEPT 9998,CONV1,SCALE
9998  FORMAT(F9.4)
100   TYPE 9994,NTRUE
9994  FORMAT(I3,7H PARAMS)
      ACCEPT 9998,(S(I),I=1,NTRUE)
      IF(SENSW(5))NTRUE=4
      PAUSE 3
99    CALL SIPLEX
      TYPE 9991
```

```
9991   FORMAT(1HM)
       ACCEPT 9999,M
       NTRUE=5
       IF(SENSW(5))NTRUE=4
       GO TO(100,10,1,10),M
10     DO 20 J=1,NTRUE
       S(J)=X(1,J)
20     CONTINUE
       GO TO (100,99,1,13), M
13     CALL B34FUN
       PAUSE 20
11     WRITE(5,9992)(RO(IR),IR=1,NP),(Q(IR),IR=1,NP)
9992   FORMAT(10F7.4)
       DO 74 I=1,NP
74     QST=QST+RO(1)
       TYPE 9997,QMO,QMS,PTOT,QOT,QST
9997   FORMAT(4HQMO=,F9.4,5H QMS=,F9.4,6H PTOT=,F9.4/
      1F9.4)                          4HQOT=,F9.4,5H QST=,
       QST=0.
       STOP
       END
```

```
        SUBRØUTINE ANALØG
       DIMENSIØN RF(250),RØ(250),Q(250)
       LØGICAL SENSW,ITEST
       CØMMØN/BLK8/RF,RØ,Q,NP,QMS
       CALL QSIC(IERR)
       I2=NP+1
       DØ 5 I=1,I2
       CALL QRSLL(0,ITEST,IERR)
 4     CALL QRSLL(0,ITEST,IERR)
       IF (ITEST) GØ TØ 4
       IF(I.EQ.1)GØTØ7
       CALL QRBADR(RØ(I-1),0,1,IERR)
       CALL QWJDAR(Q(I-1),4,IERR)
       IF(I.EQ.I2)GØTØ5
 7     CALL QRSLL(0,ITEST,IERR)
 6     CALL QRSLL(0,ITEST,IERR)
       IF (.NØT.ITEST) GØ TØ 6
       CALL QWJDAR(RF(I),5,IERR)
       IF(I.EQ.1)CALL QSØP(IERR)
 5     CØNTINUE
        CALL QSH(IERR)
       RETURN
       END
```

```
      SUBROUTINE B34FUN
      DIMENSION RF(250),RO(250),Q(250),X(16,15),S(15),TES(16)
      LOGICAL SENSW,OVL
      COMMON/BLK1/SCALE,NTRUE
      COMMON/BLK2/X,S
      COMMON/BLK4/II,JJ,K,L,N,IN,IO
      COMMON/BLK6/KT,FUNC
      COMMON/BLK8/RF,RO,Q,NP,QMS
      COMMON/BLK9/VAR
      FUNC=0.0
      IF(.NOT.SENSW(4))GOTO12
      WRITE(IO,100)N,(S(J),J=1,N)
12    DO 13 IX=1,4
10    IF(ABS(S(IX)).GE.1..OR.S(IX).LT.0.)GO TO 20
13    CONTINUE
100   FORMAT(1H0,I4,5F8.4)
      DO 78 I=1,4
      ICH=I-1
78    CALL QWJDAR(S(I),ICH,IERR)
      CALL ANALOG
      IV=IFIX(10.*S(5))
      I2=NP
      I1=1
      IF(IV)1,2,3
1     KO=-IV
      KC=0
      I2=NP+IV
      GOTO4
2     KO=0
      KC=0
      GOTO4
3     KO=0
      KC=-IV
      I1=1+IV
4     QMS=0.
      DO 79 I=I1,I2
      IF(RO(I).GT.QMS)QMS=RO(I)
      IA=I+KO
      IB=I+KC
79    FUNC=FUNC+(RO(IA)-Q(IB))**2
      FUNC=FUNC*10./FLOAT(I2-I1+1)/VAR
52    IF(.NOT.SENSW(4))GOTO50
      WRITE(IO,102) FUNC
102   FORMAT(2H F,E12.5)
  50  KT=KT+1
51    CALL QRSLL(3,OVL,IERR)
      CALL QRSLL(3,OVL,IERR)
      IF(OVL)GOTO51
      RETURN
  20  FUNC=10.0**10
      GO TO 50
      END
```

```
C          SIMPLEX HILL CLIMBING PROCEDURE
C      MAX. NUMBER OF PARAMETERS  15
C      MAIN PROGRAMME MUST INITIALISE THE FOLLOWING BEFORE CALLING
C      THE SIMPLEX HILL CLIMBING PROCEDURE.
C      INUMA=MAX NUMBER OF TRIALS PER PASS
C      IPR=CONTROL INTEGER TO CONTROL THE TYPE OUT OF INTERMEDIATE
C          1-TYPE OUT COMPLETE HISTORY                          RESULT
C          2-TYPE OUT INTERMEDIATE VALUES OT TIMES DETERMINED BY N1
C      N1=CONTROLS THE TIMES WHEN INTERMEDIATE RESULTS WILL BE TYPED
C       NTRUE=NUMBER OF DIMENSION    MAX. ALLOWED IS 15
C       CONV1= CONVERGENCE CRITERION
C       SCALE=CONSTANT USED BY SUBROUTINE B34STT TO CONSTRUCT FIRST
C      S(J) = INITIAL STARTING POINT FOR OPTIMISATION ROUTINE
       SUBROUTINE SIPLEX
       DIMENSION X(16,15),S(15),TES(16)
       DIMENSION XC(15)
       COMMON/BLK1/SCALE,NTRUE
       COMMON/BLK2/X,S
       COMMON/BLK3/TES
       COMMON/BLK4/I,J,K,L,N,IN,IO
       COMMON/BLK5/N1,IPR,INUMA,CONV1
       COMMON/BLK6/KT,FUNC
       COMMON/BLK7/NN,INUM
C      INITIALISE
       N=NTRUE
       NN=N+1
C      IDOUB=0 FIRST PASS
       IDOUB=0
       ICU=0
       IDCU=N*(N+1)
C      INUM= COUNT OF NUMBER OF SIMPLEX TRIALS.
       INUM=0
C      INUM1= COUNT FOR FREQUENCY  OF PRINTING
       INUM1=0
       IF(N-15)1,2,2222
2222   WRITE(IO,2983) N
2983   FORMAT(4H  N=,I3)
C      N TOO LARGE, RETURN
       RETURN
C      ZERO X(I,J) ELEMENTS WHICH ARE NOT USED
1      DO 3 J=NN,16
       DO 3 K=1,15
3      X(J,K)=0.0
       KT=0
C       DETERMINE POINTS TO DEFINE THE INITIAL SIMPLEX
2      CALL B34STT
       WRITE(IO,941)
C      TYPE OUT THE CO-ORDINAVES OF THE INITIAL POINTS
       CALL OUTSIM(1)
941    FORMAT(//8H  IN.PTS)
5      DO 402 I=1,NN
       IF(IDOUB)513,401,513
513    IF(I-2)401,402,401
401    DO 323 J=1,N
323    S(J)=X(I,J)
C      EVALUATE FUNCTION AT EACH COORDINATE OF SIMPLEX AND STORE
C      RESULTS IN TES(I)
       CALL B34FUN
```

```
      TES(I)=FUNC
402   CONTINUE
      IDOUB=0
5570  IF(INUM-INUMA)350,350,301
C     HAVE MAX. NUMBER OF TRIALS BEEN EXCEEDED
350   IF(INUM1-N1)81,40,81
81    GO TO (40,41),IPR
40    WRITE(IO,951)
C     OUTPUT COORDINATES OF SIMPLEX AND FUNCTION VALUES
      CALL OUTSIM(2)
951   FORMAT(/8H SIMPLEX)
      INUM1=0
41    IF(INUM-1)54,54,441
C     CHECK FOR CONVERGENCE
441   DO 300 I=1,NN
      DO 300 J=1,N
      CU=ABS((X(I,J)-XC(J))/(X(I,J)+XC(J)))
      IF(CU-CONV1)53,53,300
53    ICU=ICU+1
300   CONTINUE
C     IF CONVERGENCE IS ACHIEVED , RETURN
      IF(ICU-IDCU)54,301,301
C     NO CONVERGENCE, INCRENENT COUNT AND CONTINUE
54    ICU=0
      INUM=INUM+1
      INUM1=INUM1+1
      NNN=NN+1
C     ZERO ELEMENTS TES(N+2) TO TES(16) OF ARRAY TES(I)
      DO 6 I=NNN,16
      TES(I) = -(10.**+38)
6     CONTINUE
C     FIND ELEMENT IN ARRAY TES(I) WITH MAX. VALUE
      TESMA=AMAX1(TES(1),TES(2),TES(3),TES(4),TES(5),TES(6),TES(7),
     1TES(8),TES(9),TES(10),TES(11),TES(12),TES(13),TES(14),TES(15),
     2TES(16))
      DO 7 I=1,NN
      IF(TES(I)-TESMA)7,8,7
8     K=I
      GO TO 9
7     CONTINUE
C     PLACE MAX. POSSIBLE NUMBER IN ELEMENTS TES(N+2) TO TES(16)
C     OF ARRAY TES(I)
9     DO 10 I=NNN,16
      TES(I)=10.**38
10    CONTINUE
C     FIND ELEMENT IN ARRAY TES(I) WITH MIN. VALUE
      TESMI=AMIN1(TES(1),TES(2),TES(3),TES(4),TES(5),TES(6),TES(7),
     1TES(8),TES(9),TES(10),TES(11),TES(12),TES(13),TES(14),TES(15),
     2TES(16))
      DO 11 I=1,NN
      IF(TES(I)-TESMI)11,13,11
13    L=I
      GO TO 14
11    CONTINUE
C     COORDINATES OF MAX. TO X(1,1)
C     MAX. FUNCTION VALUE TO TES(1)
C     COORDINATES OF MIN. TO X(2,1)
C     MIN. FUNCTION VALUE TO TES(2)
14    CALL SORT
C     CALCULATION OF CENTROID
```

```
C       THE COORDINATES OF CENTROID XC(I)ARE THE AVERAGE VALUE OF
C       COORDINATES EXCLUDING COORDINATES DEFINING MAX.
200     XN=FLOAT(N)
907      FORMAT (4E18.8)
43      DO 118 I=1,N
        SUM=0.
        DO 17 J=2,NN
        XC(I)=SUM+X(J,I)
17      SUM=XC(I)
118     XC(I)=XC(I)/XN
        GO TO (403,45),IPR
403     WRITE(IO,405)
C       TYPEOUT COORDINATED OF CENTROID
        WRITE(IO,907) (XC(IWRITE),IWRITE=1,N)
405     FORMAT(/6H  CTRD)
404     CONTINUE
C       REFLECTION
45      DO 117 I=1,N
117     S(I)=2.*XC(I)-X(1,I)
C       COMPUTE FUNCTION VALUE AT REFLECTION POINT
        CALL B34FUN
        TEST 1=FUNC
        GO TO (46,47),IPR
C       TYPEOUT FUNCTION VALUE AND COORDINAYES OF REFLECTED POINT
46      WRITE(IO,911)
        CALL OUTSIM(3)
911     FORMAT(/8H  REF.PT)
C       TEST FOR A NEW MAX. OR NEW MIN.
47      IF(TEST 1-TESMA)19,38,35
19      IF(TESMI-TEST 1)210,210,18
210     DO 211 I=2,NN
C       COMPARE FUNCTION VALUE AGAINST PREVIOUS VALUES EXCLUDING THE
C       PREVIOUSLY OBTAINED MAX. AND MIN. VALUES
C       IF REF. POINT IS  .GT. TES(2) TO TES(N+1), CONTRACT
C       IF REF. POINT IS .LT. OR .EQ. TES(2) TO TES(N+1), RETAIN THIS
                                                                POINT
        IF (TEST 1-TES(I))31,31,211
211     CONTINUE
        GO TO 38
C       NEW MINIMUN
C       IF REFLECTED POINT PRODUCES NEW MIN. TRY EXPANSION IN THE
C       SAME DIRECTION OF REFLECTED POINT
18      DO 23 I=1,N
        X(1,I)=S(I)
23      S(I)=2.*X(1,I)-1.*XC(I)
C       COMPUTE FUNCTION VALUE AT EXPANSION POINT
        CALL B34FUN
        GO TO (48,49),IPR
48      WRITE(IO,912)
C       TYPEOUT FUNCTION VALUE AND COORDINATES OF EXPANSION POINT
        CALL OUTSIM(3)
912     FORMAT(/8H  EXP.PT)
49      IF(TESMI-FUNC)24,24,31
C       IF FUNCTION VALUE AT EXP. POINT IS LESS THAN MIN.RETAIN THIS
C       POINT AND START ANOTHER SIMPLEX TRIAL
C       IF FUNCTION VALUE AT EXP. POINY IS .GT. MIN. RETAIN THE
C       REFLECTED POINT AND START ANOTHER SIMPLEX TRIAL
24      TES(1)=TEST 1
        GO TO 5570
C       REFLECTION HAS PRODUCED A NEW MAX.
C       CONTRACT THE SIMPLEX
```

```
35      DO 37 I=1,N
37      S(I)=X(1,I)
38      DO 28 I=1,N
28      S(I)=0.5*S(I)+0.5*XC(I)
C       COMPUTE THE FUNCYION AT CONTRACTION POINT
        CALL B34FUN
        GO TO (55,56),IPR
55      WRITE(IO,913)
C       TYPEOUT FUNCTION VALUE AND COORDINATES OF CONTRACTION POINT
        CALL OUTSIM(3)
913     FORMAT(/8H  CON.PT)
56      IF(FUNC-TEST 1)30,29,29
30      IF(FUNC-TESMA)31,29,29
C       FUNCTION VALUE AT CONTRACTION POINT .LT.MAX. AND REF. POINT
C       RETAIN THIS POINT AND START ANOTHER TRIAL
31      DO 32 I=1,N
32      X(1,I)=S(I)
        TES(1)=FUNC
        GO TO 5570
C       FUNCTION VALUE AT CONTRACTION POINT IS .GT. MAX. AND REF.
C       CHANGE ALL COORDINATES AND START AGAIN                  POINT
29      DO 33 I=1,NN
        DO 33 J=1,N
33      X(I,J)=(X(I,J)+X(2,J))/2.
        IDOUB=1
        GO TO 5
C       END OF HILL CLIMBING ROUTINE
301     WRITE(IO,902)
902     FORMAT(//11H  FINAL PTS)
C       OUTPUT COORDINATED OF FINAL SIMPLEX, VALUE OF FUNCTION AT EACH
C       COORDINATES, AND ITERATION NUMBER
        CALL OUTSIM(2)
C       OUTPUT TOTAL NUMBER OF FUNCTIONAL EVALUATION
        WRITE(IO,1901) KT
1901    FORMAT(8H  FN EVS,I4)
        RETURN
        END

C       SUBROUTINE TO                    TIONAL POINTS TO DETERMINE
C       INITIAL SIMPLEX
        SUBROUTINE B34STT
        DIMENSION X(16,15),S(15)
        COMMON/BLK1/SCALE,NTRUE
        COMMON/BLK2/X,S
        COMMON/BLK4/I,J,K,L,N,IN,IO
        XN=FLOAT(N)
        P=SCALE*((XN-1.)+SQRT(XN+1.))/(XN*1.4142)
        Q=SCALE*(SQRT(XN+1.)-1.)/(XN*1.4142)
        DO 610 J=1,N
610     X(1,J)=S(J)
        NALPH=1
        NA=N+1
        DO 620 I=2,NA
        DO 630 J=1,N
        IF(J-NALPH) 640,650,640
640     X(I,J)=Q+S(J)
        GO TO 630
650     X(I,J)=P+S(J)
630     CONTINUE
620     NALPH=NALPH+1
        RETURN
        END
```

```
C       THIS SUBROUTINE WILL OUTPUT VALUES FROM THE SIMPLEX OPTIMISATIO
C       ROUTINE . THE QUANTITIES IT OUTPUTS ARE DETERMINED BY IFLAG
C       IFLAG = 1 OUTPUT STARTING COORDINATES
C       IFLAG = 2 OUTPUTCOORDINATES OF SIMPLEX, FUNCTION VALUE AT EACH
C       COORDINATE AND THE ITERATION NUMBER
C       IFLAG = 3 OUTPUT FUNCTION VALUE AND COORDINATES OF A POINT
        SUBROUTINE OUTSIM(IFLAG)
        DIMENSION X(16,15),S(15),TES(16)
        LOGICAL SENSW
        COMMON/BLK2/X,S
        COMMON/BLK3/TES
        COMMON/BLK4/I,J,K,L,N,IN,IO
        COMMON/BLK6/KT,FUNC
        COMMON/BLK7/NN,INUM
        IF(.NOT.SENSW(3).AND.IFLAG.EQ.1)RETURN
        GOTO(2,2,3),IFLAG
3       WRITE(IO,910)
910     FORMAT(6H  FUNC)
        WRITE(IO,907) FUNC
        WRITE(IO,911)
911     FORMAT(7H  CO-OD)
        WRITE(IO,907)(S(I),I=1,N)
907     FORMAT(5F8.5)
1       RETURN
2       WRITE(IO,911)
        WRITE(IO,907) ((X(I,J),J=1,N),I=1,NN)
        GOTO (1,4,1),IFLAG
4       WRITE(IO,910)
        WRITE(IO,907) (TES(I),I=1,NN)
        WRITE(IO,908)INUM
908     FORMAT(6H -INUM,I5)
        RETURN
        END



        SUBROUTINE SORT
        DIMENSION X(16,15),S(15),TES(16),A(15),B(15),C(15),D(15),
       1X1(16),X2(16)
        COMMON/BLK2/X,S
        COMMON/BLK3/TES
        COMMON/BLK4/I,J,K,L,N,IN,IO
        IF(K-1)60,61,60
60      IF(L-1)62,63,62
62      IF(K-2)64,65,64
64      IF(L-2)66,67,66
66      DO 15 I=1,N
15      A(I)=X(1,I)
        DO 101 I=1,N
101     B(I)=X(2,I)
        DO 100 I=1,N
100     C(I)=X(K,I)
        DO 201 I=1,N
201     D(I)=X(L,I)
        DO 203 I=1,N
203     X(1,I)=C(I)
        DO 16 I=1,N
16      X(2,I)=D(I)
        DO 51 I=1,N
51      X(K,I)=A(I)
        DO 52 I=1,N
52      X(L,I)=B(I)
```

```
      AX=TES(1)
      BX=TES(2)
      CX=TES(K)
      DX=TES(L)
      TES(1)=CX
      TES(2)=DX
      TES(K)=AX
      TES(L)=BX
      RETURN
61    DO 68 I=1,N
      X1(I)=X(2,I)
      X(2,I)=X(L,I)
68    X(L,I)=X1(I)
      AX=TES(2)
      TES(2)=TES(L)
      TES(L)=AX
      RETURN
63    DO 69 I=1,N
      X1(I)=X(1,I)
      X2(I)=X(2,I)
      X(1,I)=X(K,I)
      X(K,I)=X2(I)
69    X(2,I)=X1(I)
      AX=TES(1)
      BX=TES(2)
      TES(1)=TES(K)
      TES(K)=BX
      TES(2)=AX
      RETURN
65    DO 70 I=1,N
      X1(I)=X(1,I)
      X2(I)=X(2,I)
```

```
      X(1,I)=X2(I)
      X(2,I)=X(L,I)
70    X(L,I)=X1(I)
      AX=TES(1)
      BX=TES(2)
      TES(1)=BX
      TES(2)=TES(L)
      TES(L)=AX
      RETURN
67    DO 71 I=1,N
      X1(I)=X(1,I)
      X(1,I)=X(K,I)
71    X(K,I)=X1(I)
      AX=TES(1)
      TES(1)=TES(K)
      TES(K)=AX
      RETURN
      END
```

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ADDENDUM

The overall aim of formulating a catchment simulation model is to represent each of the interacting processes in the hydrological system as accurately as possible, so that the causes and effects of any perturbations in the system can be analysed. Ideally, the model should be able to simulate the atmospheric processes over the catchment and the natural and man-induced processes within the catchment. These would include primarily water quantity processes and artificial water transfers, but also water quality processes and processes related to the erosion and deposition of sediment.

Recent developments in the fields of physical hydrology and ecology have made possible the mathematical representation of catchment processes which, in the early days of catchment modelling, were thought to be unworthy of inclusion in a catchment model. In addition, the major processes have been continually re-examined with the aim of providing more accurate representations. Thus, with the computing power available from large electronic computers and with a large amount of systems analysis and programming, it is possible to simulate the interrelationships between many diverse catchment processes.

As hydrological research continues, the potential accuracy of hydrological models increases. For example, processes may be treated as distributed rather than lumped parameter systems, enabling the spatial distribution of variables such as vegetal cover, soil characteristics, impervious area and relief to be represented more accurately. In the field of urban hydrology it becomes feasible to break the system down into small sub-units, comprising individual flow planes, gutters and pipes or culverts. Flow through the soil may be simulated in three dimensions using partial differential equations and the effects of overbank storage and irregular channels may be represented using non-linear functions. These techniques allow the most accurate representations of natural catchments, which are non-linear, distributed parameter, time-variant systems.

As possible processes and solution procedures which should be included in a catchment model are investigated, it soon becomes apparent that there are certain constraints to be borne in mind. Firstly, there is the constraint of knowledge. Can a process be identified and formulated in a mathematical way? A process can often be identified in the field but measuring the process and developing general models of it are much more difficult. It has taken many years to develop the mathematical techniques with which to analyse one of the most easily measurable processes, namely channel flow. The development of realistic equations for sub-surface flow has been slower because it is difficult to measure and, unlike channel flow, is not easily simplified into one- or two-dimensional flow. The constraint of knowledge also applies to problem solution. A process may be identified and represented by a non-linear partial differential equation, but can the equation be solved?

The second constraint is one of accuracy. Is it possible to obtain an accurate solution from the model? The problem of model accuracy is linked to measurement accuracy. The inclusion of an additional process such as interception in a catchment model may lead to more accurate results, but the improvement in accuracy may well be negligible compared with the accuracy with which the model variables were measured. Thus it may be difficult to ascertain whether additional processes improve the model or not. This is complicated by the fact that certain processes come into operation only during infrequent events, which may not be present in the data used to test the model.

The third constraint is one of time and money. Does the model merit the inclusion of every process and every advanced technique or would a satisfactory solution be obtainable from a much simpler model? Clearly this depends to a large extent on the purpose for which the model is required. A particular problem might be amenable for solution using a simple model, but if the model is too simple, then a similar or related problem might require the building of completely new model. In research therefore, the best strategy might be to design a model for a particular requirement, but in the commercial environment, where

time is strictly limited, a more comprehensive model might be more useful. Although parts of the model may be redundant in a particular problem, it could be applied to many diverse water resources problems without repetitive model building. Indeed, if hydrological models are to be used to their full potential in national and local water resources planning, the ideal would be to have a comprehensive general water quantity/quality model in modular form, so that the various processes and functions are compatible and may be selected to suit a particular problem.

In relation to the research reported in this thesis, the formulation of a model to simulate the response of the catchment under study was carried out with the above constraints in mind. The final model was simple, but enabled the hydrographs of surface runoff (insofar as this is a phenonmenon separable from base-flow) to be reproduced. A certain amount of accuracy was sacrificed for the sake of flexibility in using a hybrid computer, although the accuracy in measurement of the variables under study was thought to be such that no great disadvantage was incurred.

A valuable extension to the study might be to build a model to simulate sub-surface processes and to model surface processes (gutter flow, pipe flow, etc.) in more detail. This would entail the introduction into the model of time-variant parameters which, if not directly measurable from maps, drawings and historical documents, would prove to be an interesting problem in parameter optimization. The calibration and verification of a continuous-process model with distributed parameter would be difficult in the catchment studied in Crawley because intermediate points on the catchment were not gauged. In addition, no evaporation data are available in the area. Thus the verification of a water balance would be difficult. More general problems may be encountered in trying to obtain parameters for the model. The most effective way to obtain parameter values is to measure them in the field (if possible). This would be impossible to do if the parameters varied through time and no measurements were made in the past. The other alternative is to obtain parameters from the hydrological data

series by optimization. A more complicated model with more than ten parameters would make optimization difficult due to parameter correlation.

It can be appreciated therefore, that adequate data are a prerequisite for the building, calibration and verification of an advanced simulation model of the kind suggested above. The hydrological data collection network should be of sufficient density to define the distribution of hydrological processes within the catchment. In addition, a record should be kept of all land use changes in order to relate parameter values to land use. Obviously such a data collection scheme would be expensive to maintain, but in the long term the benefits would accrue in terms of more accurate simulation models. These could then be used to aid in the planning, design and operation of hydrological systems.