CONCEPTUAL HYDROLOGIC MODELLING FOR MASTER PLANS IN URBAN DRAINAGE

by

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A thesis
presented to the School of Graduate Studies and Research
of the University of Ottawa
in partial fulfillment of the requirements
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ABSTRACT

In recent years government agencies in Canada have developed policies to implement "stormwater management" techniques in master drainage plans which are designed to regulate watershed development in a manner compatible with watershed needs. In order to do this, the post-development flows need to be assessed long before the detailed design is done. The general objective of this thesis is to develop a planning model for urbanizing watersheds in the framework of master drainage plans. The model should be capable of determining the outflow hydrograph for the post-development watershed using only the limited data available at the planning stage without any calibration.

Lumped models are appropriate where the outflow hydrograph is required at only a few critical points in the watershed. HYMO, a lumped model which is very popular in Canada for planning purposes, due to its facility of application, is found to underestimate the peak flows for urban watersheds.

A literature review indicates that various conceptual models such as the single linear reservoir (SLR) model have been used by several investigators for urban watersheds.

However there has not been a systematic assessment of these lumped models as compared to the detailed simulation using design models under Canadian design conditions. Therefore a methodology is developed using mathematical watersheds to compare the performance of conceptual models under different conditions such as the size and the imperviousness levels of the urban watersheds. Based on the objective function of minimizing the differences in the peak flows (as compared to those obtained by detailed simulation), the 'optimal' parameters of these conceptual models are obtained by means of Rosenbrock's hill-climbing technique. They are then compared with the parameters predicted by various relations. The one parameter of the SLR model which is the storage coefficient K has been related to watershed and storm characteristics by means of multiple regression analysis by some researchers while others have related it to the time of equilibrium.

From the analysis of eight relations it is observed that the single linear reservoir model using a relation derived by Desbordes or one derived by Sarma, Delleur and Rao predicts reasonably accurate peak flows for the larger storms for watersheds up to 100 acres (40 ha) in size. A version of this model has also been programmed on a calculator.

For short duration high intensity storms used in design, there is a significant contribution of rainfall excess from the pervious or undeveloped areas. The approach taken in some models is to simulate separately the runoff hydrographs from the impervious and pervious portions of the watershed. Therefore a model consisting of two linear reservoirs in parallel is proposed here and tested.

It is found that the 2 parallel linear reservoirs (2PLR) model is a better tool than the SLR model. For planning purposes the model can predict acceptable peak flows for the whole range of storms tested for watersheds ranging from 2 to 300 acres without calibration. The storage coefficients of the 2 reservoirs are related with the input rate i, length L, slope S and surface roughness n using relations based on the kinematic wave theory. The model computes the losses in the pervious areas by means of the cumulative form of Horton's infiltration equation and has been programmed as a ready-to-use tool in the HYMO structure. The optimization subroutine with which it has been interfaced can be used for the calibration of model parameters where rainfall-runoff data is available.

Tests with several storms on two residential watersheds, the Gray Haven watershed in Baltimore, Maryland and the Malvern watershed in Burlington, Ontario have validated the performance of the 2PLR model for predicting the outflow hydrographs from urban watersheds.

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Chapter I

INTRODUCTION

1.1 MASTER DRAINAGE PLANS

Traditionally the philosophy in the design of drainage systems is to allow the runoff to drain away as fast as possible. i.e. convenience on-site but little consideration for downstream effects. Increased runoff would be diverted away by means of 'improved' channels. As development proceeded, the runoff from upstream urbanized areas began to cause flooding, erosion and pollution problems downstream, necessitating the building of expensive downstream control and conveyance works. The flooding problems are aggravated by the fact that houses are often built on the floodplains of the rivers.

In view of this situation, "stormwater management" techniques have been developed in Canada, U.S.A. and other countries. In Ontario, the Ministry of Natural Resources (MNR) and the Ontario Ministry of the Environment (MOE) recognised the need for a new and comprehensive approach to urban drainage management and have proposed policies (Urban Drainage Policy Committee,1978) to regulate the use of the floodplain and to mitigate the hydrologic effects of new developments. The first of 5 main policies is that every municipality is required to develop master drainage plans for all watersheds in their boundaries. It states

> "that the purpose of this policy is to foster master drainage planning for rapidly developing municipalities to ensure that stormwater drainage systems are developed in a manner compatible with watershed needs, to identify existing water quality and flooding problems and to avoid future problems."

It is suggested that master drainage plans should be developed for the entire watershed. For new developments, policy 4 states that

> "The draft plan of subdivision for new developments shall indicate the predicted effects of the proposed subdivision on the water resources and the proposed measures to mitigate or prevent the adverse effects ."

The subdivision plan shall

- include the expected change in the runoff hydrograph as compared to predevelopment conditions for both frequent and infrequent storms.
- The effects the altered hydrograph would have on the receiving watercourse, such as channel erosion and flooding.

Legislation is currently being drafted to support the policies. The concept behind these policies is to maintain the natural pre-development hydrologic conditions where possible e.g. by maintaining the water table and allowing for groundwater recharge.

Some of the simple guidelines in these policies such as 'zero runoff increase' which means that post-development flows should not have any increase over pre-development flows are not easy to implement. First of all, the assessment of pre-development flows is hampered by the fact that most of the areas where development is to take place are ungauged. The modelling of post-development flows is usually difficult because urbanization has not yet taken place or is an ongoing process.

<u>In view of the above needs the general objective of this thesis is to develop a planning model for urbanizing watersheds in the framework of master drainage plans. The required model should be able to predict the runoff hydrograph for future urbanized conditions using the limited data available at the planning stage</u>.

The thesis is part of a more comprehensive research program conducted at the University of Ottawa in the following directions:

1. Analysis of the interface between simplified and sophisticated computer models and the development of a new standard method for the preliminary analysis of stormwater management projects (Level I).
2. Development of a conceptual hydrologic model for master drainage plans for use in both small and large areas (Level II).
3. Development of a model for the simultaneous routing of runoff in major and minor systems for the detailed design of drainage systems in new subdivisions (Level III).

4. Development of a modified procedure for the application of the SCS Curve Number method in rural watersheds.

While each of the studies is basically independent of the others, the common goal is to develop a package of models that is applicable to a broad range of drainage studies. An attempt is also made to derive at each level, results which are consistent with the other levels.

1.2 MODELLING REQUIREMENTS FOR MASTER DRAINAGE PLANS

Urban runoff models can be classified as distributed system or lumped system models . A distributed system model can be defined as one in which an attempt is made to model the complex details of the physical flow processes at every part of the watershed. An example of a distributed system model is the EPA-SWMM. Where detailed data is available, distributed models can give good results. However practical applications necessitates the use of a model with a limited number of parameters. Where the outflow hydrograph is only required at a few points in the watershed, then a lumped system approach can be used. For planning purposes this is generally the case. Also, as has been pointed out by McPherson(1979) almost all sewer application model verifications have been at the outlet of the watershed due to the limited data available. Therefore for verification purposes, a distributed system model is thus reduced to a lumped system model. <u>Hence the approach taken in this study is to develop a lumped system model that can give outflow hydrographs with peak flows that are close to those from a more sophisticated distributed system model</u>. The planning model should be able to account for the two major changes on the hydrological regime due to urbanization i.e. increased imperviousness and channelization.

Due to the limited data available at the planning stage, the number of parameters for a planning model should be minimized. Watershed characteristics which are more easily measurable should be related to model parameters as much as possible. Some of these characteristics are discussed below.

1. <u>Area</u>

   This is often determined by just a topographic map and a planimeter. However Alley and Veenhuis (1979) have shown that the ratios of drainage area determined by this method and those determined by field-verified boundaries for 12 basins in Denver ranged from 0.13 to 4.90. This may be due to irrigation ditches overflowing into the watershed or storm sewers which discharge into an adjacent watershed. Therefore field inspection is important. The perimeter of the watershed is another characteristic that can be measured.

2. Future land use

Land use can be broadly classified into categories such as residential, commercial and industrial. Different percentage of imperviousness can be assigned to these categories e.g. industrial areas should have about 90 % imperviousness.

3. Soil type

The Soil Conservation Service have classified soils into four broad categories, A,B,C,D depending on their runoff potential. This soil classification has been used with the SCS Curve Number (CN) procedure. In the ILLUDAS model the soil classification is used for selecting Horton's infiltration parameters (Terstriep and Stall,1974). The CN procedure will be assessed in later sections.

4. Channel characteristics

The type of channel e.g. sewers, man-made open channels, natural channels is usually known at the planning stage. Manning's roughness coefficient n for these channels can be estimated. At the early planning stage only the major system i.e. the roads and the natural channels is known.

1.3 THE PRESENT PRACTICE OF MODELLING FOR MASTER DRAINAGE PLANS

The most frequently used models for urban drainage in Canada are SWMM, ILLUDAS and HYMO (Perks,1978). The only model which has been extensively tested by comparison with measurements in Canada is SWMM (Proctor and Redfern, James F MacLaren, 1976, James F Maclaren, 1975). ILLUDAS has been extensively tested in the U.S. (Terstriep and Stall, 1974, Wenzel and Terstriep, 1976, Burke and Gray, 1979). However both SWMM and ILLUDAS have been used mainly for design purposes. SWMM is often used to check the design of a sewer system calculated by means of the Rational Method or to evaluate an existing system. ILLUDAS has the sizing of pipes as one of its objectives. HYMO which is the most frequently used model for master drainage plans has not been systematically verified in Ontario.

Both SWMM-RUNOFF and ILLUDAS have been used for planning purposes by considering the watershed as a lumped watershed (Proctor and Redfern, James F. MacLaren, 1976, Purenne et al, 1979, Ahmad, 1980). For planning, these models have certain limitations which will be briefly discussed. Some of the detailed steps, such as computing the overland flow hydrograph and then routing along the gutters for each sub-watershed, would no longer be done. For lumped watersheds average values of the paramaters such as width and slope would be used. The parameters would then lose their physical correspondence and become just calibration parameters.

ILLUDAS computes the outflow hydrograph by adding up the hydrographs from the impervious and pervious areas. The hydrograph from the impervious area is obtained by convoluting the paved area rainfall excess supply rate with the time-area curve. The time-area curve is assumed to be a straight line joining the origin with the travel time from the farthest point on the impervious portion of the watershed to the inlet. This assumption of a linear function for the time-area curve ignores the watershed shape and is equivalent to assuming a regular shape like a rectangular watershed. Actually this is also equivalent to using a rectangular unit hydrograph. The travel time is calculated by using Manning's equation for the condition of uniform sheet flow. A similar procedure is followed in determining the hydrograph from the pervious portion of the watershed. The travel time in this case is computed by a modified form of Izzard's equation (1946). ILLUDAS does not consider surface detention and thus may result in higher peak flows than other models. The only parameter for calibration is the inlet time and it has been observed that the peak flow is not very sensitive to it (P'ng, Liang, Ray, Moss,1980).

HYMO (Hydrologic Model) (Williams and Hann,1973) is a very popular model because it is easy to use and requires only limited input data. Basically HYMO is a computer package made up of a series of subroutines each of which relates to a specific hydrologic command. It is intended as a structure to perform hydrologic modelling. The program is written with hydrologists, who are not computer-oriented, in mind as it requires little computer training. It uses the SCS CN procedure to compute the rainfall excess. It has a built-in dimensionless unit hydrograph which is converted to a unit hydrograph once the two parameters K and tp are known (see section 2.2). The major portion of the unit hydrograph from the beginning of rise to the inflection point is calculated by means of the two-parameter gamma equation. The other portions of the unit hydrograph follows an exponential decay curve. HYMO has gained some acceptance as being adequate for rural or pre-development conditions. For ungauged watersheds, the K and tp parameters should however be calibrated instead of being determined by the built-in empirical relationships (Cheung, Sautier, Wisner,1981). However its applicability to small urban watersheds has not been established to date (Perks,1978, Wisner and Cheung,1979).

1.4 APPLICATION OF HYMO FOR URBAN WATERSHEDS

1.4.1 MTRCA Study

James F Maclaren Ltd (1979) have applied HYMO to urban watersheds by deriving a procedure that modifies K and tp to account for different levels of imperviousness for use on larger watersheds in a study, for the Metropolitan Toronto and Region Conservation Authority (MTRCA). The K and tp values are calibrated for highly urbanized and completely rural watersheds in the Toronto region. Using computer simulations for hypothetical watersheds, the parameters are then interpolated for other imperviousness levels. Since the procedure is based on the calibration of K and tp for the rural conditions in Toronto it is not applicable in other places. It is also applied for fairly large areas (about 5 square km or greater) and therefore is not recommended for use with smaller areas. Results from detailed design studies done by various consultants have shown serious discrepancies when compared to those obtained by this procedure, namely lower flows by HYMO (Mather,1980). Comparisons between the flows simulated by HYMO using the parameters given by James F MacLaren (1979) and the SWMM model confirm that for small areas and intense storms these relations are not applicable. These observed differences between an instantaneous unit hydrograph (IUH) model and detailed design models are also found for other planning models (see section 1.5).

1.4.2 <u>Assessment of HYMO for urban runoff control studies</u>

HYMO is assessed for application in urban drainage planning in a research study conducted by Wisner and Cheung (1979). It is attempted to calibrate CN and the unit hydrograph parameters K and tp in order to obtain peak flows closer to those from detailed SWMM simulations. K and tp are initially estimated from watershed data like area and slope using empirical relationships derived from data from rural watersheds in the Southern U.S. The parameters are calibrated by taking different percentages of the initial K and tp estimates and then comparing the resulting peaks with those obtained from SWMM. The conclusions from the study are as follows:

1. HYMO, as presently used, can underestimate the peak flows significantly. Three sets of relationships for K and tp (including the MTRCA study) are used and found to give lower peak flows than SWMM.
2. A 50 % reduction in the K and tp values results in increased peak flows comparable to SWMM for certain storms.

The objective of the study is mainly to point out the limitations of HYMO. The calibration of the parameters is done for very limited conditions. Therefore, as pointed out in the study, the validity of the second conclusion for wide planning purposes is limited. Also other unit hydrograph relationships are not tested.

Another major limitation of HYMO for urban watersheds is the use of the SCS CN procedure. It is found to underestimate the rainfall excess for small rainfalls. Wisner, Gupta and Kassem (1980) have shown that the runoff volumes obtained from measurements on two residential watersheds, Gray Haven and Northwood, are greater than the runoff volumes computed with the SCS curve number method. McCuen (1981) has computed the mean CN for 29 storms on the Gray Haven watershed and found it to be 94 (fig.1.1). The watershed's CN obtained by means of the weighting procedure (National Engineering Handbook,1971) is 86.5. This shows that the runoff volumes obtained from the CN procedure are less than the measured values for small storms. A 1:2 year, 3-hour storm with a rainfall of 1.2 in. would yield only 0.07 in. of runoff volume from a watershed with CN = 75.

It can be concluded that despite its wide use in Canada and its advantages as a computer user oriented language, the present hydrologic procedures in HYMO makes it unsuitable for urban watersheds. Its use in master drainage plans will lead to discrepancies with detailed design models. For urban conditions, other relationships for synthetic unit hydrographs would have to be incorporated. A better procedure for determining the rainfall excess distribution is also needed.

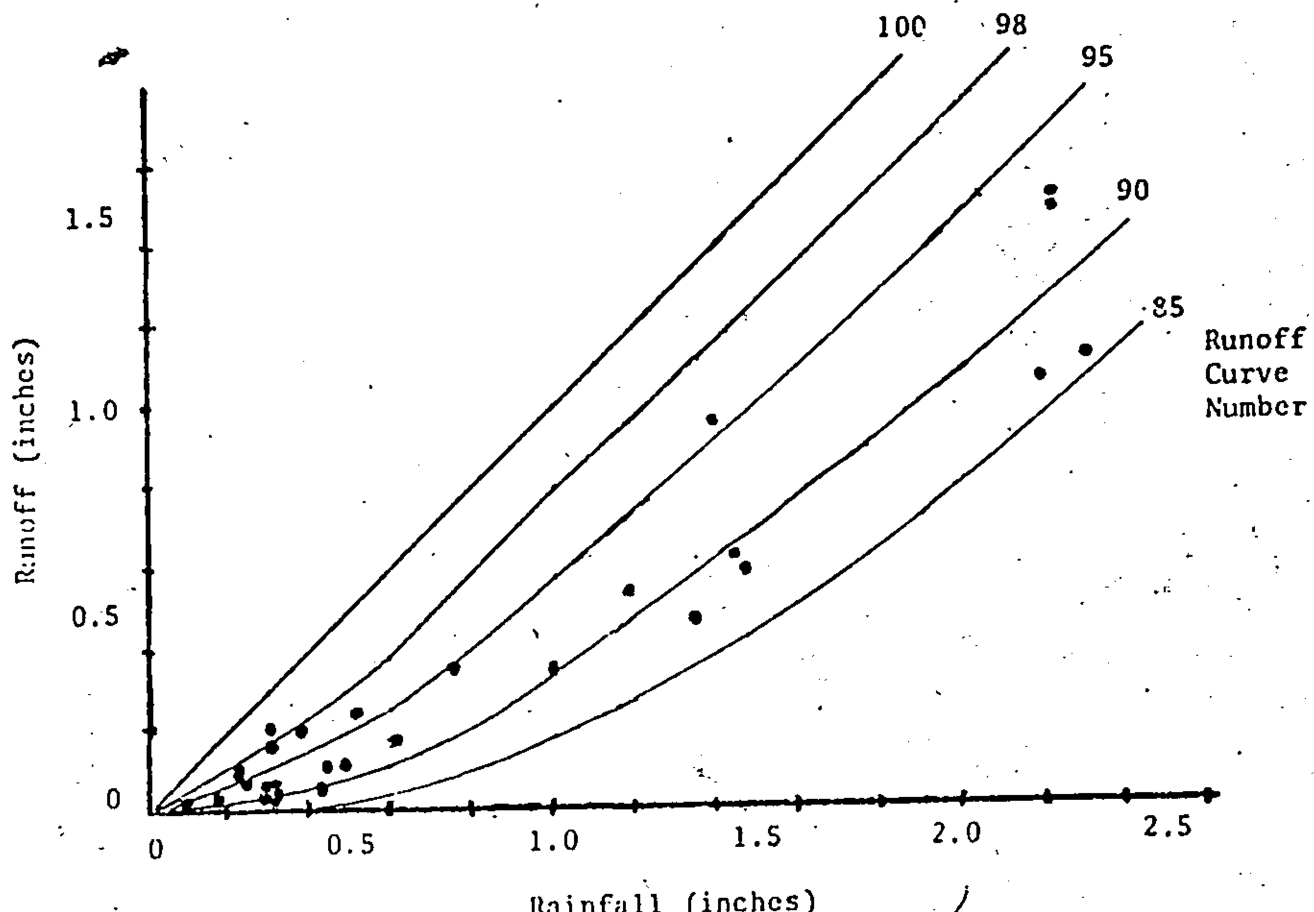


FIGURE 1.1 Relationship between Rainfall and Runoff: TR-55 and Real Measurements on the Gray Haven Watershed

(Ref : McCuen, 1981)

1.5 LIMITATIONS OF OTHER CONCEPTUAL PLANNING MODELS

Differences between planning and design models have also been reported for a study conducted in the U.S.A. Burke and Gray (1979) have compared methods that are used in the design of storm sewer systems such as the Rational Method and ILLUDAS with the instantaneous unit hydrograph (IUH) method of Sarma, Delleur and Rao (1969,1972,1973) (SDR). The SCS Curve Number (CN) procedure (applied in the same manner as in HYMO, section 3.2) is utilized to compute the rainfall excess increments for use in conjunction with the SDR method. Two watersheds, one 13.4 acres in size and the other, 121.4 acres are used for testing. The peak flows estimated by the IUH method of SDR are found to be much lower than those predicted by the design models such as ILLUDAS and the Rational Method.

Discrepancies may be due to the method of rainfall excess determination (which is felt to be the cause in the SDR method with the CN procedure) or the subjective element in the selection of model parameters. This situation happens quite frequently in the 'real' world as can be attested to, by government agencies which have to review hydrologic studies (Mather,1980). In order to assess the performance of an IUH model as compared to a design model it is necessary to use the same effective rainfall.

This is attempted in the preliminary phase of this study with another U.S. planning model, the Santa Barbara Urban Hydrograph (SBUH) method, which is recommended in a widely used urban hydrology textbook (Wanielista,1979).

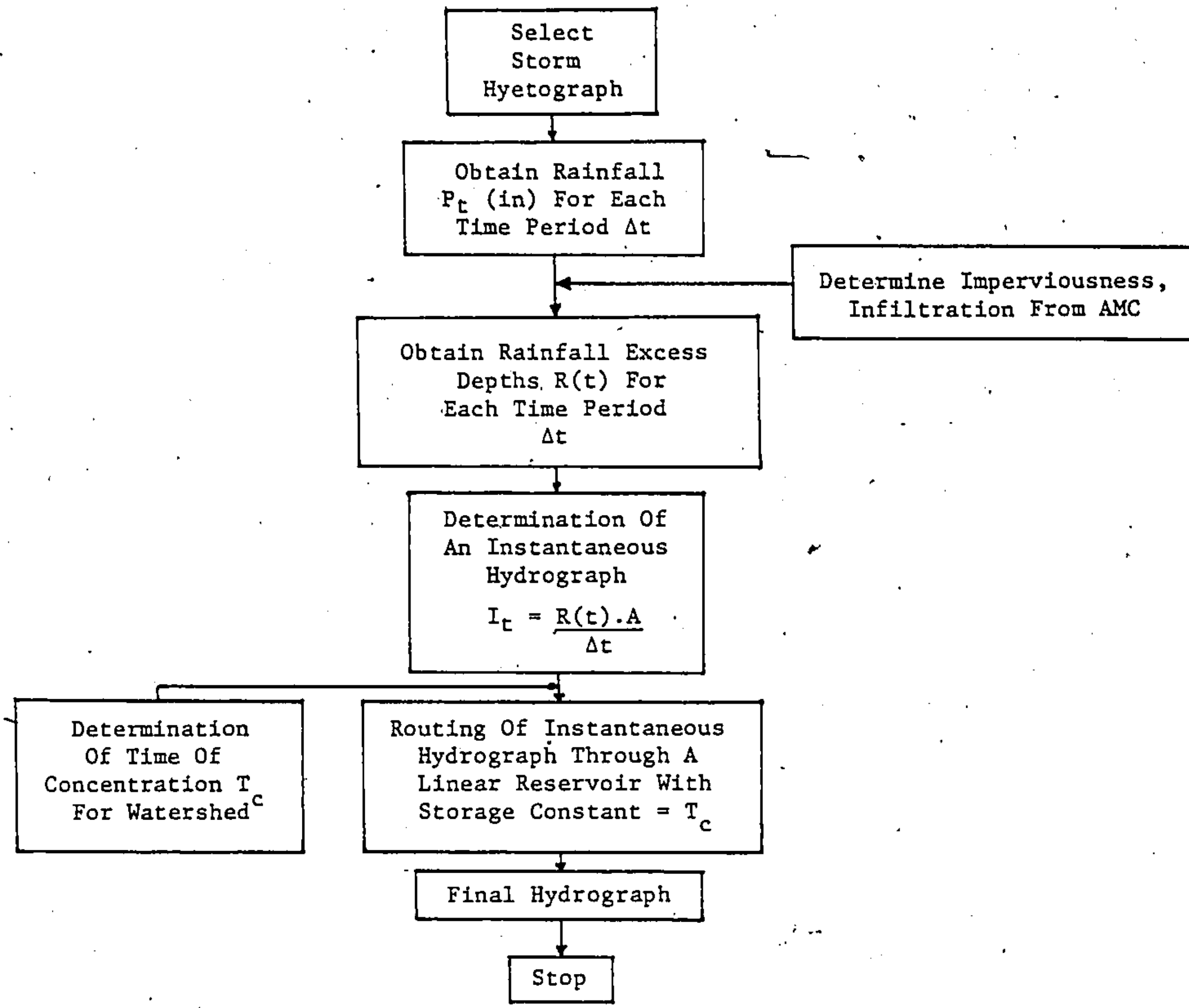
The Santa Barbara Urban Hydrograph Method is proposed by Stubchaer (1975) as a simple means of obtaining the outflow hydrograph from an urban watershed. Basically the method consists of determining the rainfall excess from both the impervious and pervious areas for each time step, adding them together and routing through a single linear reservoir with a storage coefficient K equal to the <u>time of concentration</u>. The flowchart for the model is shown in Fig.1.2. The method has also been adapted by Golding (1980) for use on the HP-67 and HP-97 programmable calculators and is known as the Howard Needles Version of SBUH (HNV-SBUH). The original SBUH method calculates the rainfall excess in the pervious area by a ϕ-index which is related to the antecedent moisture condition of the soil while HNV-SBUH calculates the infiltration by means of Horton's equation. The transformation of rainfall excess to runoff, by routing it through a single linear reservoir, has been performed successfully for small impervious areas by Willeke (1966). For a linear reservoir,

$$S = KQ \tag{1.1}$$

where S = storage, K = storage coefficient and Q = outflow

FIGURE 1.2

FLOW CHART FOR HNV-SBUH METHOD



$$\Delta S = K(Q_2 - Q_1)$$

where the subscripts 1 and 2 refer to the beginning and end of the time increment respectively. From the continuity equation,

$$I - Q = \frac{ds}{dt} \tag{1.2}$$

$$\frac{I_1+I_2}{2} - \frac{Q_1+Q_2}{2} = \frac{\Delta S}{\Delta t} \tag{1.3}$$

Substituting eqn.1.1 in eqn.1.3

$$K(Q_2 - Q_1) = \Delta S = \frac{I_1 + I_2}{2}\Delta t - \frac{Q_1 + Q_2}{2}\Delta t \tag{1.4}$$

$$2K(Q_2 - Q_1) = \Delta t(I_1 + I_2 - Q_1 - Q_2)$$

Add $\Delta t(Q_2 - Q_1)$ to both sides

$$(2K + \Delta t)(Q_2 - Q_1) = \Delta t(I_1 + I_2 - 2Q_1)$$

$$Q_2 - Q_1 = \frac{\Delta t}{2K+\Delta t}(I_1 + I_2 - 2Q_1) \tag{1.5}$$

which is the routing equation used in the SBUH method. S = KQ is actually a special case of the Muskingum routing equation in which x = 0 in the equation

$$S = K[xI + (1-x)Q] \tag{1.6}$$

When combined with the continuity equation, the following routing equation is obtained.

$$Q_2 = C_oI_2 + C_1I_1 + C_2Q_1 \qquad 1.7$$

where

$$C_o = \frac{0.5\Delta t}{K + 0.5\Delta t}$$

$$C_1 = \frac{0.5\Delta t}{K + 0.5\Delta t}$$

$$C_2 = \frac{K - 0.5\Delta t}{K + 0.5\Delta t}$$

Eqn.1.7 can be shown to be equal to eqn.1.5.

In order to test the model, the peak flows calculated by HNV-SBUH and SWMM are compared for various watersheds at 2 levels of imperviousness, 25% and 35%, using typical Southern Ontario design storms at 2 recurrence intervals, 1:2 year and 1:5 year (P'ng,Wisner,Cheung,1980). The results for the two storms are shown in figs.1.3 and 1.4 respectively.

1. For the conditions tested, HNV-SBUH is found to give lower peak flows than SWMM for the same total runoff volume (obtained by adjusting f_o). For the 1:2 year storm, the peak flow ratio(HNV-SBUH/SWMM) varied from 0.58 to 0.73 while for the 1:5 year storm, it varied

from 0.69 to 0.80. The differences in the peak flows are partly due to some differences in calculating the rainfall excess. (Although both models use Horton's equation, SWMM uses the cumulative form of Horton's infiltration procedure while in HNV-SBUH the infiltration is solely a function of time).

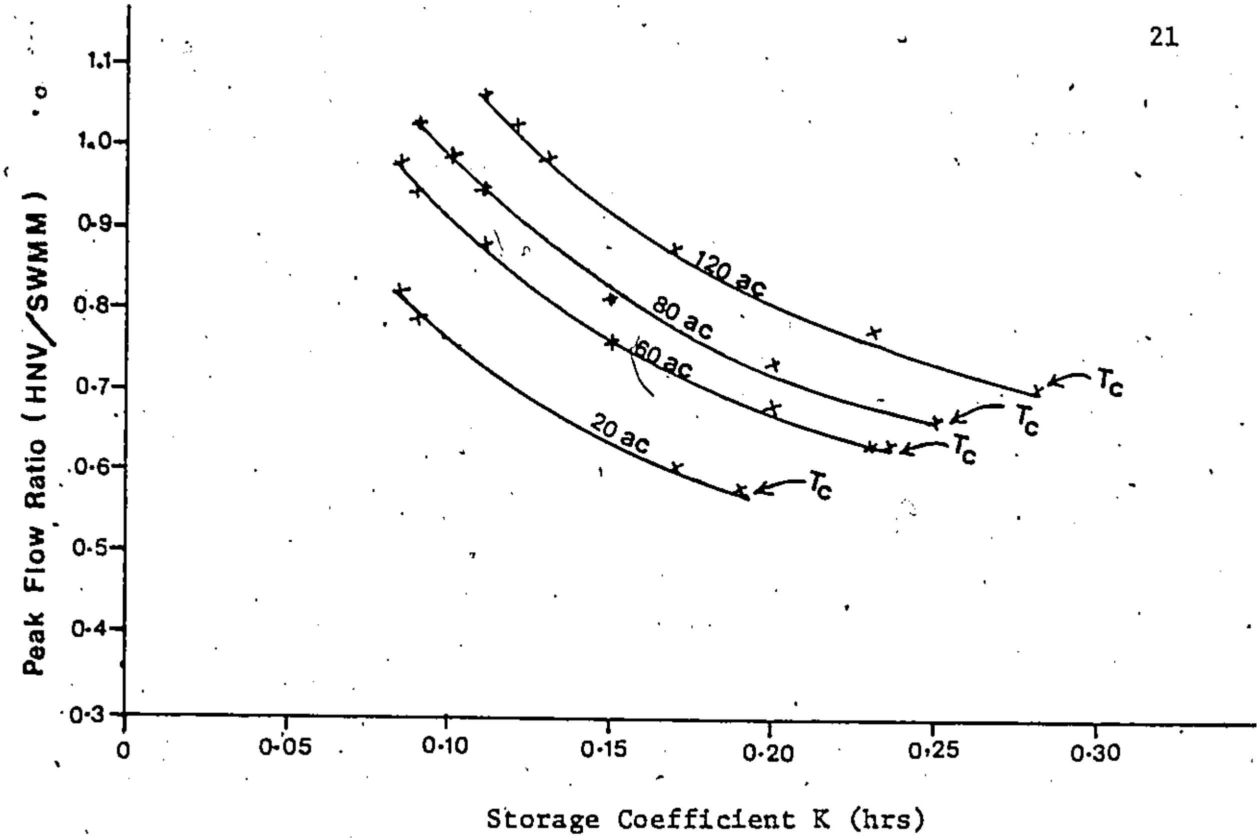
2. The peak flow is found to be very sensitive to the storage coefficient K for the storms tested which are of high intensity and short duration. By calibrating K it is possible to obtain peak flows close to those from SWMM only under certain conditions.
3. In order to be useful for planning purposes, K has to be related to watershed and/or storm characteristics. The use of $K = T_c$ in the SBUH method is not adequate since engineers may use different values of T_c for the same watershed as observed by Rossmiller, 1980. K would also have to be calibrated with real measurements.

This brief review shows that in Canada and the U.S.A., the present practice of using IUH models for large urban watersheds in master plans without systematic comparisons with design models may lead to significant discrepancies between the planning and the design stages of projects. As will be indicated in chapter II, single linear reservoir models have been developed and tested for small inlet areas for design purposes. The hydrographs for small inlet areas are, in this case, routed through the sewer network. A typical example is the QUURM model (Watt and Kidd,1972) developed in Canada. They have also been applied on a lumped basis for larger watersheds (Sarma et al, 1972). These models will be reviewed in the next chapter for suitability of application in master planning studies. Prior to this review, a brief summary of the basic concepts in linear conceptual models will also be given.

FIG. 1.3

The Peak Flow Ratios of HNV-SBUH to SWMM Corresponding to Different Storage Coefficients K

1:2 year storm 25% imperv.



1:2 year storm 35% imperv.

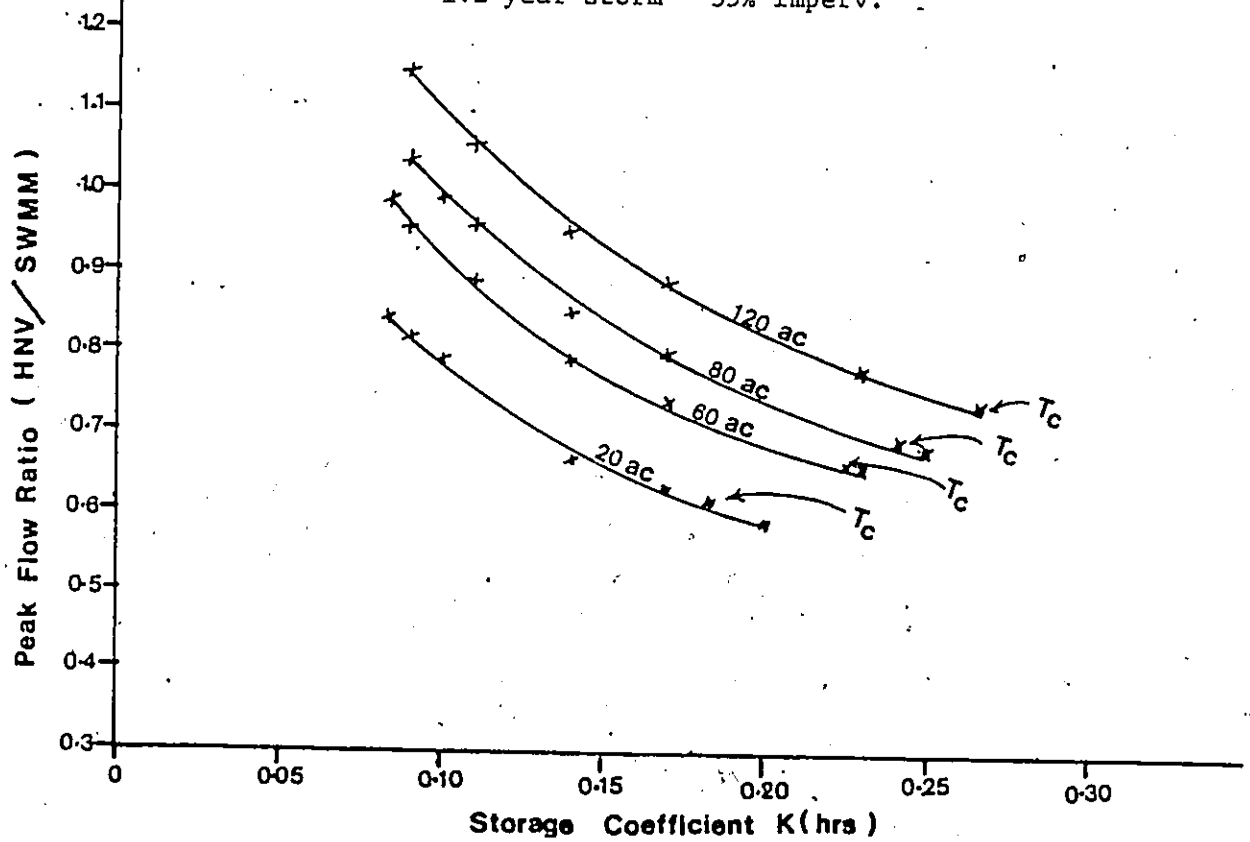
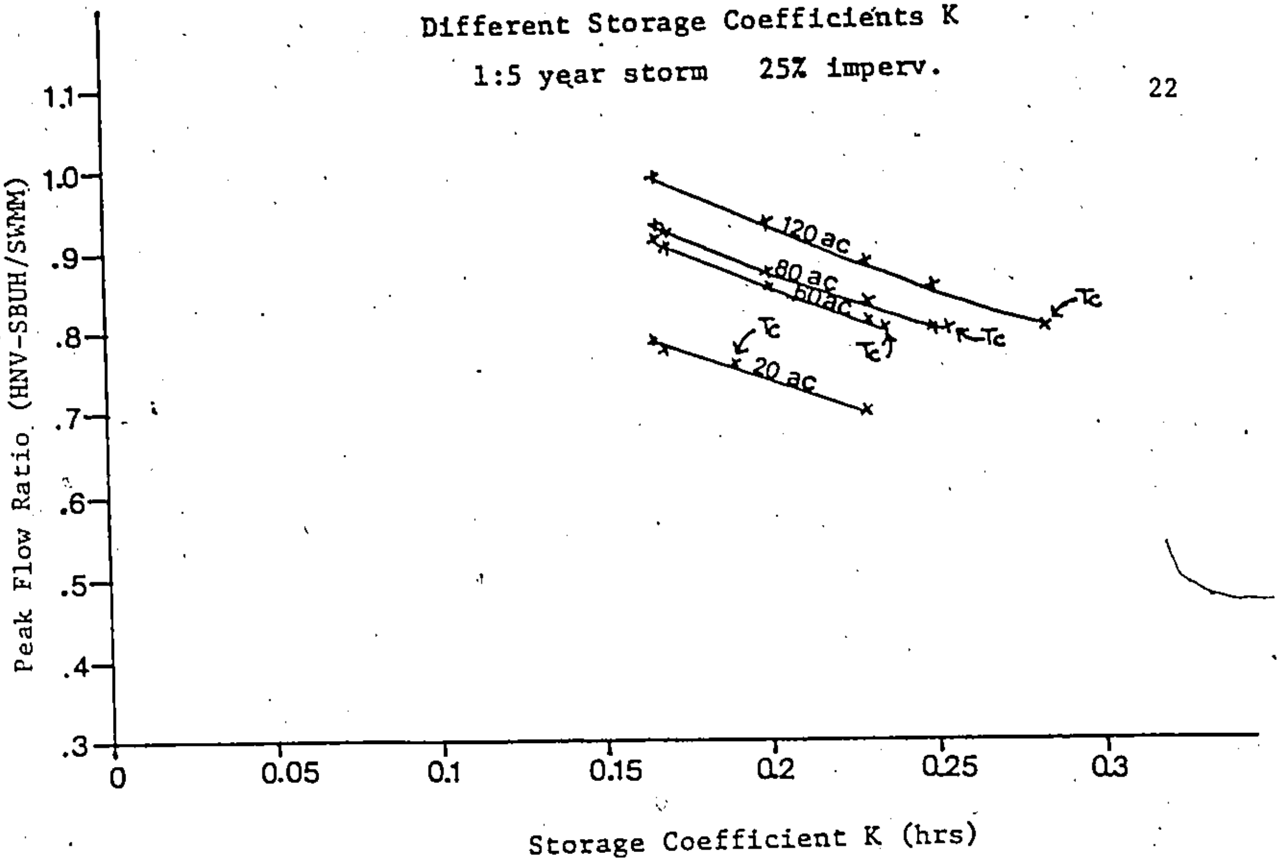


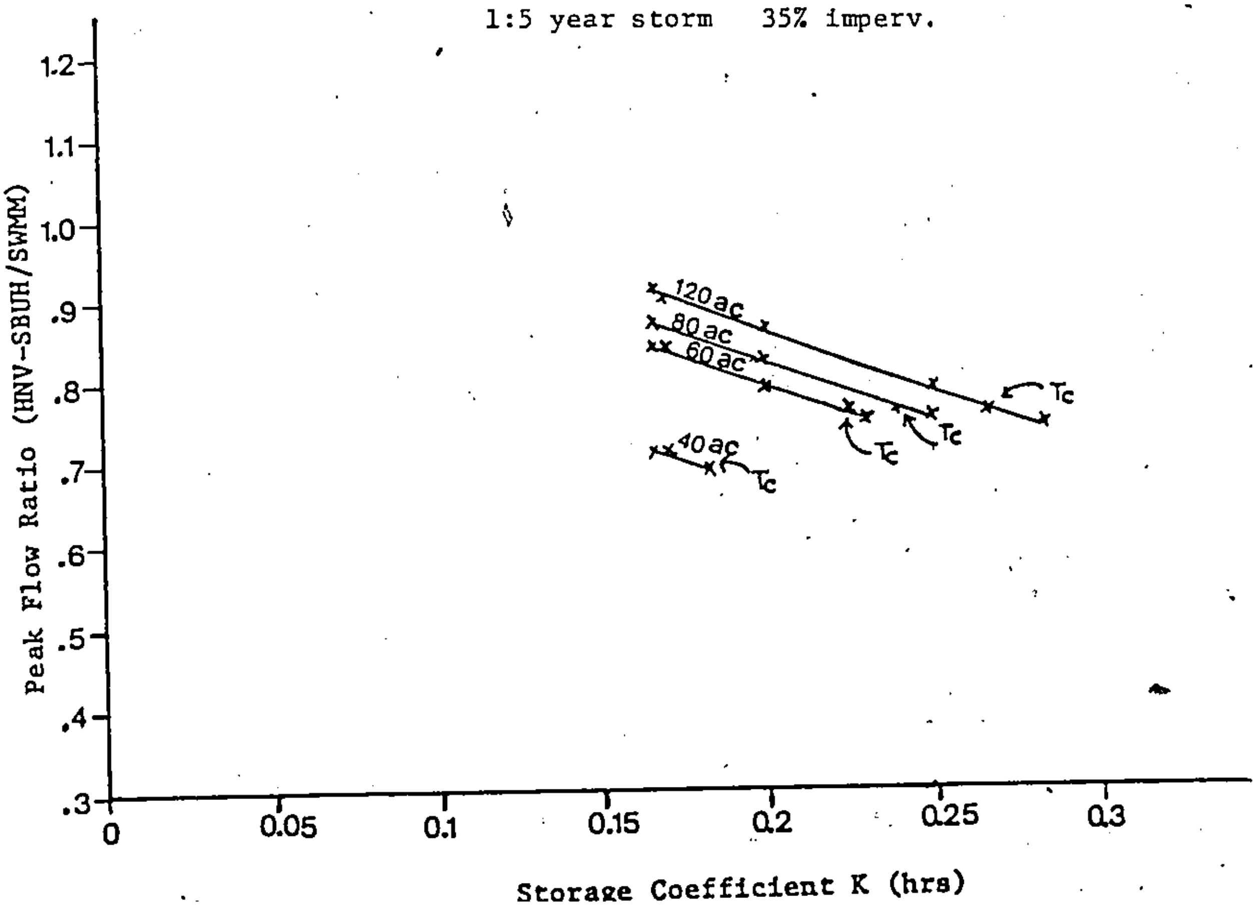
Fig. 1.4

The Peak Flow Ratios of HNV-SBUH to SWMM Corresponding to Different Storage Coefficients K

1:5 year storm   25% imperv.



1:5 year storm   35% imperv.



Chapter II

LITERATURE REVIEW AND RESEARCH NEEDS

2.1 CONCEPTUAL MODELS

2.1.1 System Models

Hydrologic models are models that represent the processes and systems of the hydrologic cycle by using simplified concepts. A system can be defined (Diskin,1981) as an ordered assembly of interconnected elements that transform in a given time reference, certain measurable inputs into outputs. The inputs and outputs are usually functions of time which means that they are 'dynamic' systems as opposed to 'static' systems which are independent of time. If the input and output are functions of both time and space, they are called 'distributed' systems. If the spatial distribution is ignored, then the systems are known as 'lumped' systems.

Systems can be further classified into linear and nonlinear systems. A linear system is one that has the property of superposition. Take a system with an input function that equals the sum of other input functions. If the output function due to this input function is equal to the sum of the individual outputs then the system is said to be linear.

Consider three input functions $X_1(t)$, $X_2(t)$ and $X_3(t)$ and their corresponding outputs $Y_1(t)$, $Y_2(t)$ and $Y_3(t)$ where

$$Y_1(t) = \phi[x_1(t)]$$

$$Y_2(t) = \phi[x_2(t)] \qquad 2.1$$

$$Y_3(t) = \phi[x_3(t)]$$

ϕ is the linear operator.

If

$$X_3(t) = aX_1(t) + bX_2(t) \qquad 2.2$$

where a and b are any two scalar quantities and

$$Y_3(t) = aY_1(t) + bY_2(t) \qquad 2.3$$

then the system is a linear system. A nonlinear system by definition is one that is not linear. One very important advantage of linear systems is that the tools of linear mathematics are very well-developed.

The assumption of time-invariance in a system means that its parameters are constant with respect to time. The output would depend only on the form of the input and not the time of its application. With this assumption it is then possible to predict the output from past records of input and output.

If

$$X(t) \rightarrow Y(t)$$

then for a time-invariant system

$$X(t+\tau) \rightarrow Y(t+\tau)$$

where τ is a time shift (positive or negative).

For rainfall-runoff modelling two systems can be defined. The first one can be called the watershed system where the input is the total rainfall hyetograph and the output is the total output hydrograph. The second system, known as the surface runoff system is a subsystem of the watershed system. Here the input is the rainfall excess hyetograph and the output is the direct runoff hydrograph. Shown in Fig.2.1 is a block diagram (Kulandaiswamy,1964) of the watershed system with the surface runoff system outlined in dotted lines.

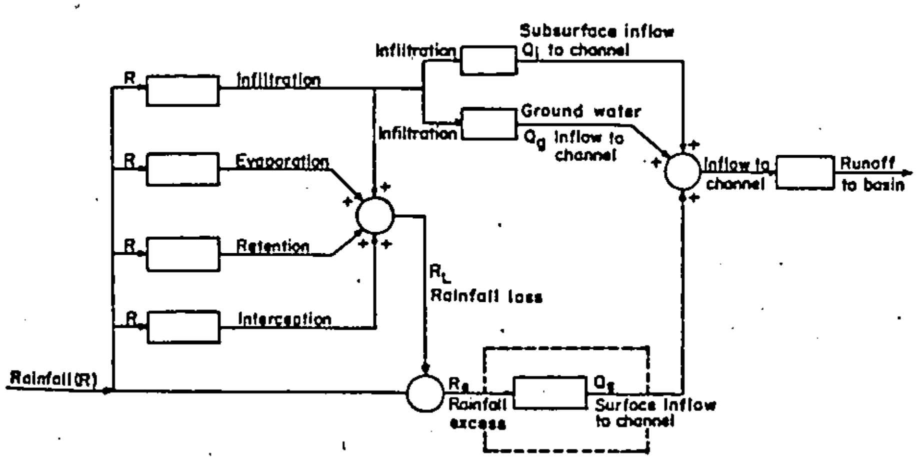


FIGURE 2.1 Kulandaiswamy's block diagram.

(Watershed System)

The watershed system is highly nonlinear while the surface runoff system is only slightly nonlinear. That is why many successful studies have been completed based on the assumption of a linear system for surface runoff. But the determination of the rainfall excess is not an easy task and is one of the weak links in rainfall-runoff modelling. To obtain the direct runoff hydrograph, usually, arbitrary methods of baseflow separation are applied because little is known about the physical processes involved. However it is generally accepted that the shape and volume of the direct runoff hydrograph is relatively insensitive to the method of baseflow separation.

2.1.2 Instantaneous Unit Hydrographs

The basic theory concerning the unit hydrograph, the instantaneous unit hydrograph and its historical development has been well reviewed by Chow (1964) and Dooge (1973). This has been summarized in Appendix A which also includes a review of the basic linear conceptual models. Basically the unit hydrograph theory assumes that the surface runoff system is a lumped, linear system. The input-output relation of a lumped, linear system can be expressed by the convolution or Duhamel integral.

$$Q(t) = \int_{0}^{t'<t_o} U(t-\tau)I(\tau)d\tau \qquad 2.4$$

where $Q(t)$ = output

$I(t)$ = input

$U(t-\tau)$ = impulse response or IUH

t_o = duration of rainfall excess

$t' = t$ when $t \leq t_o$ and $t' = t_o$ when $t > t_o$.

The instantaneous unit hydrograph or IUH is the impulse response function of the surface runoff system. It is defined as the hydrograph resulting from one unit of rainfall excess generated in zero time over the entire watershed. In other words it is the unit hydrograph that results from an infinitesimally small duration of rainfall excess.

2.1.3 Unit Hydrographs by the Analysis Approach

There are two approaches to derive synthetic unit hydrographs. The 'synthesis' approach correlates the parameters of a conceptual model with watershed and storm characteristics while the 'analysis' approach correlates the unit hydrograph characteristics directly with the watershed and storm characteristics. The ordinates of the unit hydrograph or the IUH can be derived from the rainfall excess and direct runoff record. Deriving the unit hydrograph corresponds to solving a set of linear algebraic equations while for the IUH it corresponds to solving an integral equation for the continuous case. Various methods have been used for this problem of determining the system response. They range from trial-and-error methods to transform methods such as the Laplace transform. This has been discussed by Dooge (1973). In practice this is not easy as errors in the data will cause errors in the unit hydrograph resulting in an irregular shape.

An example of the 'analysis' approach is provided by Espey et al (1977) who developed 10-minute unit hydrographs for use with small urban watersheds. Five hydrologic parameters are selected to describe the shape of the 10-minute unit Hydrographs. Two of the hydrologic parameters are the time to rise and the peak discharge. Relations are developed relating the five hydrologic parameters with watershed characteristics such as area, imperviousness, length and slope of main channel and conveyance factor Φ. It is concluded that the effects of urbanization can be accounted for, by the percentage of impervious cover and the conveyance factor. The main disadvantage with this method is that the unit hydrographs have to be sketched by hand after determining the hydrologic parameters. This also means that there is no mathematical equation describing the shape of the unit hydrograph and therefore the unit hydrograph cannot be programmed on a computer. Most of the data is from Texas and therefore the conclusions may not be applicable for local Canadian conditions. Therefore the use of conceptual models to delineate the unit hydrograph has certain advantages.

First of all, conceptual models automatically provide a number of constraints on the form of the unit hydrograph which are as follows

- All of the unit hydrographs are normalized automatically to unit area.
- All of them produce only non-negative ordinates.
- All of them produce unimodal shapes.

These constraints will filter out some of the magnification of data errors inherent in the inversion process of unit hydrograph derivation. The shape of the IUH from conceptual models is mathematically defined and this is an advantage over unit hydrograph methods like Espey et al (1977) where they have to be sketched by hand from the unit hydrograph characteristics.

Another advantage is that the information content of the data can be concentrated in a fixed, small number of parameters which will improve the chances of a satisfactory correlation with watershed characteristics (Dooge,1977).

2.1.4 Linear Conceptual Models

Various conceptual models have been proposed to delineate the IUH Some of the basic linear conceptual models are the (1) linear reservoir and the (2) linear channel or various combinations of these. The linear reservoir and the linear channel account for storage and translation effects respectively.

a. <u>Linear Reservoir</u>

A linear reservoir is one in which the storage S is directly proportional to the outflow Q.

$$S = KQ$$

The IUH of the linear reservoir is

$$U(t) = \frac{1}{K} e^{-t/K} \qquad 2.5$$

which is represented by the hydrograph for the outflow from the first reservoir in Fig.2.2

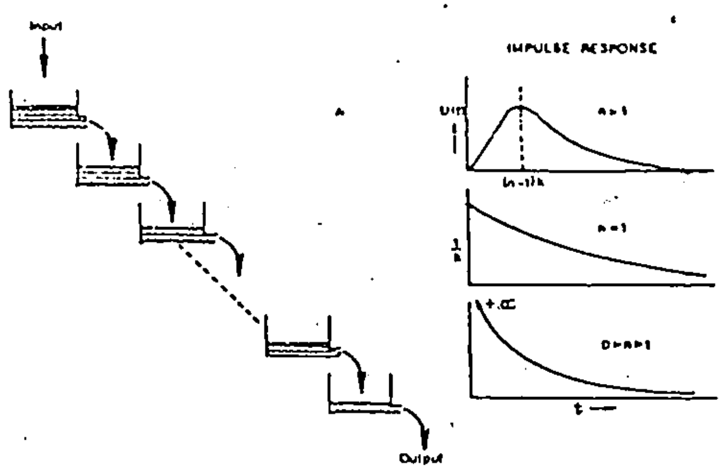


Fig.2.2 Representation of the Nash model
(Ref. Institute of Hydrology, Wallingford, 1978)

b. <u>Linear Channel</u>

The concept of a linear channel represents pure translation in a linear catchment. A linear channel is defined as a fictitious reach in which the time of travel T through the reach for a discharge Q of any magnitude is constant. At any point in the reach,

$$A = CQ \tag{2.6}$$

where A = cross-sectional area

Q = outflow

C = translation coefficient

When an inflow hydrograph is routed through a linear channel, the shape of the inflow hydrograph will remain the same but it will be lagged by the time of translation T.

c. Nash's Model

Nash (1957) has proposed that the IUH can be obtained by routing an instantaneous unit input through a cascade of equal linear reservoirs (fig.2.2). The IUH is given by

$$U(t) = \frac{1}{K\Gamma(n)} \left(\frac{t}{K}\right)^{n-1} e^{-t/K} \tag{2.7}$$

which represents a gamma distribution.

Other researchers who have used the gamma distribution as a representation of the form of the unit hydrograph are Wu (1963), DeCoursey (1966) and Williams (1968). The two-parameter gamma distribution equation can be written as

$$q/q_p = [t/t_p]^{n-1} \, e^{(1-n)(t/t_p-1)} \tag{2.8}$$

which can be derived from eqn.2.8 as shown by Wu (1963). DeCoursey (1966) and Williams (1968) have used eqn.2.8 to describe the shape of the unit hydrograph from the beginning of rise to the inflection point on the falling limb and a recession depletion equation for the remainder of the hydrograph. The recession depletion equation is written as

$$q = q_o e^{\left(\frac{t_o - t}{K}\right)} \qquad 2.9$$

where q_o and t_o are the flow rate and time at the inflection point respectively. The relation used in HYMO which is also described in Williams (1972) is almost the same except for a slight modification. In HYMO the recession depletion equation is separated into two parts. From t_1 ($t_1 = t_o + 2K$) to ∞ the recession depletion equation becomes

$$q = q_1 e^{\left(\frac{t_1 - t}{K}\right)} \qquad 2.10$$

where q_1 = flow rate at t

$K_1 = 3K$

The parameters K and n in eqn.2.7 can be evaluated by the method of moments (Nash,1959).

For a large number of reservoirs, K becomes very small. For n reservoirs, the first moment of the IUH about the origin is

$$M_1 = nK \qquad 2.11$$

The second central moment of the IUH is

$$MU_2 = nK^2 \tag{2.12}$$

All additional moments approach zero. At the limit, the gamma distribution becomes a straight line parallel to the y-axis at the time lag (t = L), which is pure translation (Overton,1970).

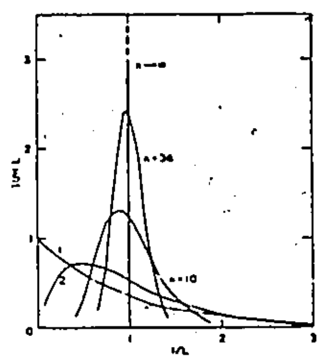


Fig. 2.3 *IUH* as a function of number of equal linear reservoirs.

(Ref: Overton, 1970)

2.2 APPLICATION OF CONCEPTUAL MODELS IN URBAN WATERSHEDS

Different investigators have applied conceptual models successfully to various urbanized watersheds ranging from small inlet areas to larger partially urbanized watersheds. Small inlet areas refer to areas less than 5 acres in size.

2.2.1 Small inlet areas

Willeke (1966) has been among the first to apply conceptual models for synthesizing the runoff hydrographs in urban areas. In that study the rainfall excess is routed through a single linear reservoir with a storage coefficient equal to the time lag t_L. The rainfall excess is obtained by subtracting first an initial loss and then a constant loss rate i.e. ϕ-index technique. The routing of the rainfall excess is accomplished by a special case of the Muskingum routing procedure (eqn.1.7) The data used are for 9 small inlet areas, less than 2 acres in size, with imperviousness ranging from 8.7% to 100%. Only a weak relationship is found to exist between storm intensity and time lag and it is concluded that the time lag is essentially constant for small urban watersheds. Viessman (1966a) in a discussion of the paper has challenged this conclusion due to the small number of storms used and the magnitude of the standard deviations of the time lag. He has also found that the storage coefficient varies according to the rainfall loss equation applied by as much as 20% to 30 %. Another criticism is that K can only be determined if the rainfall and runoff records are available. However the study did demonstrate that the single linear reservoir could be used for simulating the runoff hydrograph from very small inlet areas. Viessman (1966b) has applied the single linear reservoir to some small impervious areas of less than 1 acre in size. From the two equations for outflow from the linear reservoir,

$$Q = I(1-e^{-t/k}) \tag{2.13}$$

$$Q = Q_1 \; e^{-\tau/k} \tag{2.14}$$

where $\tau = t - t_1$,

he has derived the 1-minute unit hydrographs. t in eqn.2.13 is 1-minute and Q is equal to Q_{max}.

$$Q_{max} = I(1-e^{-1/k}) \tag{2.15}$$

$$Q_\tau = Q_{max} \; e^{-\tau/k} \qquad ; \tau = t - 1 \tag{2.16}$$

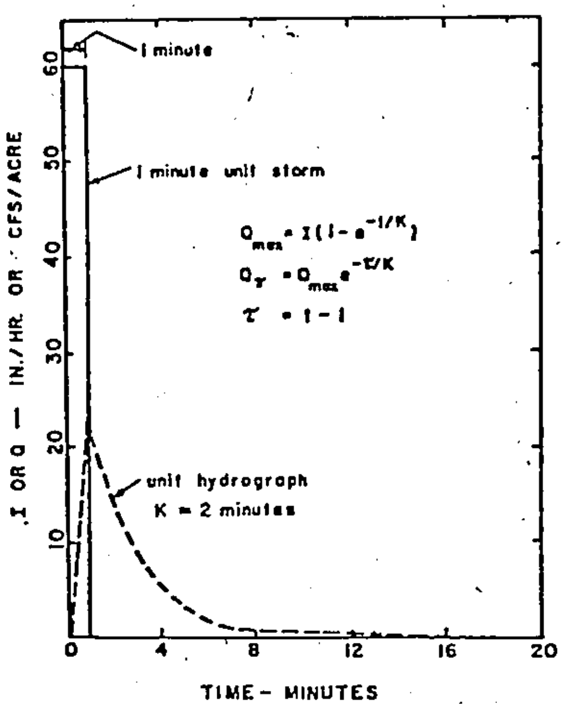


Fig.2.4 Unit hydrograph for $K = 2$ minutes.
(Ref. Viessman, 1966)

Viessman (1966) differs from Willeke (1966) in that he has convoluted the rainfall excess with the 1-minute unit hydrographs instead of routing it through the linear reservoir. However Overton (1970) has shown that routing is mathematically equivalent to convolution. Good simulations of the observed runoff hydrographs are obtained. In a continuation of the previous paper, Viessman (1968) has found that although K varies from storm to storm, the calculated output is not highly sensitive to K for variations of less than 1/2 minute. The U.K. Institute of Hydrology Workshop (1978) also has concluded that variations of 1/2 minute or less are not important. A relationship between K and the watershed characteristics, slope, length and Manning's roughness (eqn.4.1) is then derived by Viessman. The equation is derived from data from 4 impervious areas. It is also applied to a pervious area and the results obtained are good. However this limited testing does not make the relationship generally applicable. No important relationship between the storage coefficient K and the storm characteristics are found. For larger peak flows the K values are found to be more stable. Neumann (1977) has related the storage coefficient K with the watershed characteristics, slope, length, roughness and rainfall excess intensity based on the equation derived by Morgali and Linsley (1965) for time to equilibrium which is derived from kinematic wave theory. The

Institute of Hydrology Workshop (1978) has also found that the hydrographs from small impervious areas can be simulated by means of the single linear reservoir (SLR) model.

The studies reviewed so far have indicated that the single linear reservoir model has successfully been used for very small impervious inlet areas. The next step is to apply them in larger urban watersheds.

2.2.2 Larger Urbanized Watersheds

For urban watersheds where the channel or sewer system is known, the inlet hydrographs can be routed and added together to form the outflow hydrograph. With an adequate routing procedure such a model can then be used for design purposes. Watt and Kidd (1975) has attempted this in the Queens University Urban Runoff Model (QUURM). The inlet hydrographs are routed through the channel system using a time-offset method. Viessman et al (1970) have also applied a linear reservoir-linear channel model to the Gray Haven watershed. The inlet hydrographs from the 17 subwatersheds are computed by means of the single linear reservoir model. Each inlet hydrograph is then routed through another reservoir with a storage coefficient equal to the estimated travel time to the outfall, to account for channel storage. The routed hydrograph is then translated to the outfall by means of a time-offset method. Linear conceptual models have also been applied on a lumped basis to larger urban watersheds.

Delleur and Vician (1966) have found that for watersheds which are only partially urbanized, the storage coefficient of the SLR model is slightly less than the time lag and that it can vary from storm to storm depending on the antecedent moisture conditions. They have applied the single linear reservoir model to a 29-acre watershed of 38 % imperviousness and have found that the peaks will match well if the time lag is reduced by 20 %. But the times to peak of the calculated hydrographs are less than those of the observed. They have then attempted to separate the storage and translation effects by applying a conceptual model consisting of a linear reservoir in series with a linear channel for each subwatershed. For each subwatershed the time-area diagram is approximated by a trapezoid or an isoceles triangle depending on whether the rainfall duration is greater or less than the travel time in the channel. The times to peak of the observed and calculated hydrographs are found to coincide indicating that the translation by the linear channel is valid. However the peak flows are found to be lower. This means that the storage coefficient which is estimated by the travel time in the channel is too large.

Pedersen et al (1980) have applied the single linear reservoir model to urban watersheds and have also related K to the time of equilibrium t_e (eqn.4.4) Sarma, Delleur and Rao (1969,1972,1973) in a study involving 13 watersheds, 8 of which are partially urbanized have tested 4 linear conceptual models

1. Single linear reservoir model
2. Nash model
3. Holtan-Overton model or double reservoir in series
4. linear reservoir-linear channel model

They have found that the IUH obtained from rainfall-runoff data by means of the Fourier transform method is of the exponential decay type which is the same as that of the single linear reservoir. They have concluded that for small urbanized watersheds of less than 5 square miles the single linear reservoir model gives the best performance in reproducing the runoff hydrographs while the Nash model gives better performance for larger watersheds. However the regeneration performance for the Nash model is not considered to be very good. Of the 8 urbanized watersheds, 4 are less than 1 sq. mile and the rest range from 2.31 to 19.31 square miles in area. The highest imperviousness percentage is 38 %. Many of the storms analyzed are of low intensity. The storage coefficient K is correlated with the area, imperviousness and storm characteristics, rainfall excess P_e and the duration of rainfall excess T_r. The mean basin slope and the mean basin length are not found to be significant. The peak flow and the time to peak are also correlated with these 4 parameters. Due to the small range of imperviousness it remains to be seen whether the relations derived can be extended for other levels of imperviousness. The relations also need to be tested for more intense storms. To be used as a planning tool, a submodel for determining the rainfall excess distribution has to be linked with the single linear reservoir model.

The two parameters of the Nash model, n and K are related by

$$nK_n = t_L \qquad 2.17$$

t_L and K_n are correlated with watershed and storm characteristics.

$$t_L = 0.831\ A^{0.458}(1+U)^{-1.662}\ P_e^{-0.267}\ T_r^{\ 0.371} \qquad 2.18$$

$$K_n = 0.575\ A^{0.389}(1+U)^{-0.662}\ P_e^{-0.106}\ T_r^{0.222} \qquad 2.19$$

Desbordes (1978) has also successfully applied the single linear reservoir model to urban watersheds. The storage coefficient K is correlated with both watershed and storm characteristics. Two of the watershed characteristics are the slope and the length which are not found to be significant in the study of Sarma et al (1969). The data base that is used includes the watershed data of Sarma et al (1969) and other watersheds in France. This relation is further discussed in section 4.1.

Since the storage coefficient K is theoretically equal to the time lag t_L, various other relations correlating t_L with the watershed characteristics are also reviewed with a view to applying them in the SLR model. Schaake et al (1967) have used multiple regression analysis to correlate t_L with the slope, length and the imperviousness ratio using data from 29 urban watersheds. Schulz and Lopez (1974) derived unit hydrographs from 9 urban watersheds around Denver. The changes in these unit hydrographs are found to be related to the decrease in watershed response time which can be defined as the significant time required for a watershed to respond to a uniform input of rainfall excess. Out of 8 definitions of response time, they have found that the time lag t_L is the response time that has the highest correlation with the physical watershed and storm characteristics. The relation is given in section 4.1.

2.3 SEPARATE SIMULATION OF RUNOFF FROM PERVIOUS AND IMPERVIOUS AREAS

For urban areas the runoff during the early part of the storm comes from the impervious areas. For large intense storms there will be some significant contribution from the pervious areas. In one series of their simulations for the Gray Haven watershed, Viessman et al (1970) have assumed that the contribution comes only from the impervious areas. For this assumption, the peak flows for 13 of the 15 storms are simulated quite well by the model. For two of the storms however, the peak flows are underpredicted. These two storms are both large and intense with a runoff/rainfall ratio greater than 0.50 which is the imperviousness ratio of the drainage area. This shows that a separate consideration of the pervious and impervious areas may be important. However this can be done in different ways.

a) Some models like the SBUH method calculates the rainfall excess separately from the impervious and pervious areas and adds them together for each time increment before routing them through the single linear reservoir.

b) Other models perform the separate simulation of hydrographs from the pervious and impervious areas and take into account the faster rate of response of the impervious areas. Both SWMM and ILLUDAS follow this approach of simulating separately the hydrographs from the pervious and impervious areas. ILLUDAS has two rainfall excess hyetographs derived from the total rainfall hyetograph and two time-area curves. The runoff hydrograph is obtained by convoluting each rainfall excess hyetograph with the corresponding time-area curve and then adding the results. The time-area curve is the impulse response function and corresponds to the IUH of a linear channel.

The parallel cascades model, first proposed by Diskin (1964) is made up of 2 parallel cascades of equal linear reservoirs. In applying the model to urbanized areas, Wittenberg (1975) has interpreted each cascade of equal linear reservoirs as representing the impervious and pervious portions of the watershed. It is an example of the simulation of separate hydrographs by using the IUH's from the cascades of reservoirs. The IUH's are convoluted with the respective rainfall excess distributions. In deriving the two rainfall excess inputs, Wittenberg has assumed that the impervious area has initial abstractions but no infiltration, while for the pervious area a specified constant percentage of each rainfall increment will become lost to direct runoff. Diskin et al (1978) have also applied the parallel cascades model to an urban watershed of 3.5 square miles and their model differs from that of Wittenberg only in the method of obtaining the rainfall excess. The ϕ-index method is used in the model of Diskin et al (1978) for both pervious and impervious areas. The instantaneous unit hydrograph for the model is

$$U(t) = \frac{\alpha}{K_1\Gamma(n_1)} \left(\frac{t}{K_1}\right)^{n_1-1} e^{-t/K_1} + \frac{\beta}{K_2\Gamma(n_2)} \left(\frac{t}{K_2}\right)^{n_2-1} e^{-t/K_2} \qquad 2.20$$

where $\alpha + \beta = 1.0$ for a unit input.

α and β are scaling factors corresponding to the portions of impervious and pervious areas respectively. The rainfall excess inputs for the two portions are multiplied by the corresponding scaling factors. The input and output for the impervious area is related by the following convolution integral

$$Y_A(t) = \alpha \int_o^t X_A(\tau) h_A(t-\tau) d\tau \qquad 2.21$$

in which $h_A(t)$ is the impulse response function of the first cascade of reservoirs. It should be noted that both portions of the watershed receive the same total rainfall hyetograph but have different rainfall excess distributions. The advantage of the model is that the increase in imperviousness is reflected in α while the improvement of the drainage system is reflected in K_1, the storage coefficient of each of the reservoirs in the first cascade (Delleur, 1981).

2.4 SUMMARY AND RESEARCH NEEDS

None of the models discussed above have been systematically compared with detailed simulation using design models and most have been tested just for limited conditions. The review of planning procedures in Ontario also indicates that most of the research results reviewed above, have not been used by engineers for practical modelling in planning studies.

HYMO which is a form of the Nash model, when applied as a planning model (using its original or modified parameters) for urban watersheds in Ontario, is found to underestimate the flows significantly as compared to detailed SWMM simulations (Wisner and Cheung,1979).

A widely used single reservoir U.S. planning model, the SBUH method, investigated by the author as described in Section 1.5. has not been found to give for all conditions, results comparable with SWMM. Its assumption that the storage coefficient K equals the time of concentration, T_c, may make the method too subjective due to the wide variation possible in estimates of T_c. For a planning model, it is necessary to predict the parameters of the model from the watershed characteristics.

From the preceding literature review, it is found that the advantages of linear, time-invariant models are, however, significant. The mathematical computations are facilitated and system behaviour can be compared directly since it is expressed in the response functions. The influences of different components of rainfall excess can be identified since superposition is assumed. The single linear reservoir model which is simpler than the Nash model has been found to be acceptable for small watersheds. There are, however, many different relations for the storage coefficient K. Empirical relations for the storage coefficient for larger partially urbanized areas of up to a few square miles (Sarma et al, 1969, Desbordes, 1978) have also been developed.

A table listing the watershed and storm characteristics correlated with the storage coefficient K in different studies is shown in Table 2.1. Due to the various relations available and the different parameters involved it is difficult for a practising engineer to select an appropriate relation. Another approach is to use two parallel cascades of linear reservoirs which then increases the number of parameters from one to four. There are also no general relations for the model parameters.

In order to meet the objective of developing a planning model for Canadian urban watersheds, the following questions need to be answered.

(1) Which of the relations for model parameters are compatible with Canadian design conditions?

Some of these relations are developed from local data and therefore may not be transferable to Canadian conditions. This may be more true for larger watersheds with the accompanying larger variation of watershed characteristics. The relations also may not be applicable for design conditions that is, high intensity storms.

(2) Which relation will be the 'optimal' one for each specific application e.g. a certain range of size of watersheds?

There has not been a systematic comparison of these relationships to assess their applicability under different conditions. Therefore a methodology needs to be developed to compare these different IUH relations under design conditions at different imperviousness levels for various watershed sizes. A systematic comparison will enable the limiting conditions of each relation to be more clearly defined and will also determine the 'optimal' relation for each specific condition.

The rainfall excess volume and the distribution of rainfall excess play very important roles in the determination of the outflow hydrographs. In deriving the various correlations for K, different rainfall excess distributions have been utilized. Some have used the ϕ-index while others have used the constant proportional loss model. For comparing the K relations, the rainfall excess submodel has to be the same in each case. Therefore a rainfall excess submodel will be selected and incorporated for testing.

Another important research need is to compare the effects of using the rainfall excess distribution obtained by adding both impervious and pervious rainfall excess contributions versus the separate modelling of the direct runoff hydrographs as done in models such as ILLUDAS. This is especially relevant for intense storms where the contribution from the pervious areas is significant.

Table 2.1 WATERSHED AND STORM CHARACTERISTICS RELATED TO THE STORAGE COEFFICIENT K

		WATERSHED CHARACTERISTICS						STORM CHARACTERISTICS			
Investigations	Type of Relation	Area	Length	Slope	Imperviousness	Surface Roughness	Hydrologic Radius	Rainfall Excess	Duration of Rainfall Excess	Duration of Storm	Rainfall Excess Intensity
VIESSMAN (1968)	Empirical		X	X		X					
NEUMANN (1977)	Kinematic Wave		X	X		X					X
INTERNATIONAL WORKSHOP, INSTITUTE OF HYDROLOGY, WALLINGFORD (1978)	Kinematic Wave & Regression		X	X							X
PEDERSEN, PETERS, HELWEG (1980)	Kinematic Wave		X	X		X					X
SARMA, DELLEUR, RAO (1969,1972,1973)	Regression	X			X			X	X		
DESBORDES (1978)	Regression	X	X	X	X			X	X		
SCHAAKE, GEYER, KNAPP (1967)	Regression		X	X	X						
SCHULZ & LOPEZ (1974)	Regression				X		X	X		X	

2.5 DETAILED OBJECTIVES OF THE STUDY

The general objective of this thesis is to develop a planning model for urbanizing areas in the framework of master drainage plans.

Following the preliminary analysis and the literature review this general objective can be further expounded as more clearly defined, detailed objectives which are listed as follows.

1. To review previous research on conceptual models which have been applied in urban hydrology.
2. To develop a methodology for the comparison of linear conceptual models under different watershed conditions such as different imperviousness levels and different sizes.
3. To propose an IUH model for master drainage planning in both small (25 acres or 10 ha) and larger urban watersheds up to 300 acres (121 ha) in size. The proposed model when applied for lumped areas as in planning studies should give results consistent with a frequently used distributed design model such as SWMM. This is to avoid the presently found discrepancies which hinder the implementation of modelling policies.
4. To interface the model with HYMO such that it can be used as a comprehensive model for drainage studies in small urban watersheds.

HYMO is selected for this purpose for the following reasons

a) easy to use

b) already popular among engineers in Ontario.

c) has reservoir routing subroutine.

d) has channel routing capability (variable travel time (VTT) method).

The model resulting from the study will be called URBHYMO.

Chapter III

STUDY METHODOLOGY

3.1 DEVELOPMENT OF MATHEMATICAL WATERSHEDS FOR THE COMPARISON OF IUH MODELS

3.1.1 Mathematical vs Real Watersheds

The methodology used in the study and the development of the model is described in this chapter. Basically it can be divided into 4 parts:

1. development of the mathematical watersheds for the comparison of IUH models
2. selection of a distributed model for the detailed simulation of the outflow hydrographs from the mathematical watersheds
3. selection of a rainfall excess submodel
4. selection of an optimization scheme.

The outline of the methodology is discussed in the first part. A methodology needs to be developed in order to compare the performance of different relations for the storage coefficient K of the SLR model. This may be done by means of two possible approaches.

The first approach is to use measured rainfall-runoff data from some selected watersheds. There are, however, very few urban watersheds with measured high intensity storms corresponding to design conditions (In fact there are also very few Canadian urban watersheds that have been gauged for a relatively longer period of time). The second approach is to compare the models on mathematical watersheds and simulate runoff data using design storms as input and modelling the inlet hydrographs by means of tested design models. The main advantage of the last approach is that the watershed and channel characteristics can be varied for widely differing conditions. Different storms, both real and hypothetical can be used as input.

Various researchers have used mathematical models to simulate data for their investigations. Machmeier and Larson (1968) have developed a mathematical model of a watershed with the entire channel system to assess whether such a model can be used for experimental purposes. The watershed model is used to represent small watersheds in South-eastern Minnesota. The equations of momentum and continuity are used for routing unsteady flow through the channel system. The model watershed is then used to investigate the effect of supply rate on various hydrologic time parameters such as time to virtual equilibrium, time to peak etc. and hence to assess the validity of the unit hydrograph assumptions. It is concluded that the mathematical watershed model is a useful research tool for testing the principles of runoff on a watershed basis, especially the channel routing phase. Other researchers who have used simulation models to simulate data for experimental purposes instead of using measured data are Ragan et al (1975) and Marsalek (1976).

It is decided to use detailed simulation by a tested design model on mathematical watersheds instead of real watershed data. This decision is based on the objective to propose a lumped model giving results consistent with detailed simulation. Real watershed data will only be used for the validation of the model. The mathematical watersheds will be built in two stages. In the first stage, 2.5-acre (1 ha) inlet areas will be used to build 25-acre (10 ha) mathematical subwatersheds. These 2.5-acre inlet areas will be connected by the lateral pipe system. In the second stage the 25-acre (10 ha) subwatersheds will be joined together by the main trunk system to form larger watersheds of up to 300 acres (121 ha).

The hypothesis here is that if the following two conditions are fulfilled :

a) the 2.5-acre inlet hydrographs are accurately simulated

b) the routing model is adequate

<u>then the outflow hydrograph for the entire watershed can be accurately simulated as well</u>. This outflow hydrograph can thus be considered as equivalent to one obtained by means of a distributed model.

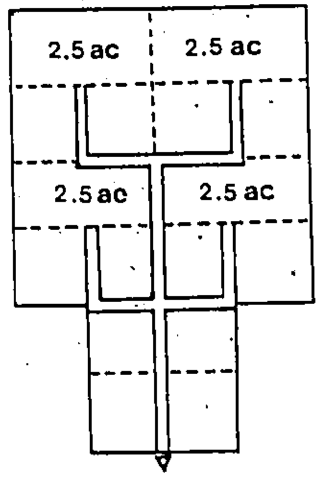


Fig. 3.1 25-acre test watershed

The outflow hydrographs from various mathematical watersheds will be used for assessing the performance of different IUH models. In order to assess the single linear reservoir model and to select the relation for K, the model is applied and the optimized K values corresponding to the outflow hydrographs from the mathematical watersheds are derived. These optimized K values can be based on different objective functions. Various relations predicting K can then be compared with these optimized values (e.g. fig.4.3).

3.1.2 Large Watersheds

For the second stage the outflow hydrographs from the 25 acre subwatersheds are routed and added together to obtain the final outflow hydrograph for the entire watershed.

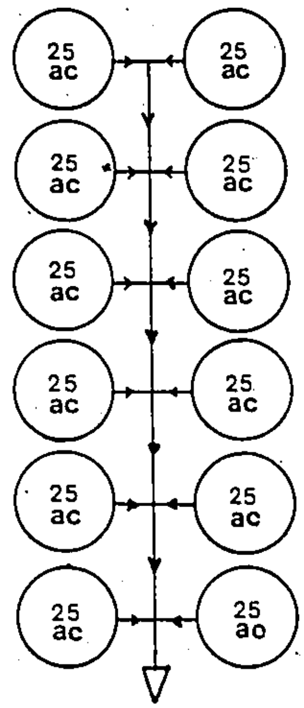


Fig.3.2 Mathematical Watersheds up to 300 acres

The routing of the trunk sewer system will be done by means of the variable travel time hydrologic routing procedure which is available in HYMO (Williams and Hann,1973). This routing procedure has been found by previous comparisons with more sophisticated routing techniques to be acceptable (Wisner and Kassem,1980). HYMO is capable of accepting hydrographs as input at different points in the watershed. The stage discharge curve in HYMO is computed by using Manning's equation. The travel time for various discharges is computed by using the variable travel time. The three channel characteristics that affect the runoff are the length, the slope and the Manning's roughness. The cross-sectional shape of the channel is trapezoidal with a side slope of 2.

3.2 SELECTION OF DISTRIBUTED (BASE) MODEL

To build the watershed, the inlet hydrographs first need to be verified and then an accurate routing procedure has to be selected. After a review of various model comparisons it is decided that the inlet hydrographs of the 2.5-acre inlet areas will be simulated by means of the SWMM-RUNOFF including the gutter routing subroutine for routing through the lateral pipe system. SWMM is selected because it is a widely used model and good results as compared with measurements have been reported by many investigators (Proctor & Redfern, J.F.MacLaren, 1976, Heeps and Mein, 1974). In a comparison of 4 urban runoff models (1) SWMM (2) UCURM (3) RRL (4) QUURM, it is concluded (J.F.MacLaren,1975) that "flows simulated by all urban runoff models agreed fairly well with measured runoff hydrographs for small urban catchments". The study has utilized many storm events from 4 urban watersheds which are less than 100 acres in size. It is also concluded that "SWMM gives the best overall performance of the models considered." An independent evaluation of the RRL, UCUR and SWMM models has been done by Heeps and Mein (1974) for a 172-acre test area in Melbourne. The study indicates that the SWMM model gives the best results with the

UCUR model giving consistently higher flows and the RRL model lower flows. Proctor and Redfern, J.F.Maclaren, 1976 in a comprehensive study testing the SWMM model for Canadian conditions have concluded that "SWMM flow simulation has proven to be accurate in a large number of applications for areas of greatly differing size, land use and sewer system configurations, and over a range of meteorological conditions. Consequently it is considered that the use of SWMM can be recommended for the simulation of flows, where required, in most drainage planning and design studies, although particular advantages offered by other models in certain specific situations should not be overlooked."

The gutter routing in SWMM-RUNOFF is also assessed. Cheung and Kassem (1979) have compared the routing in SWMM-RUNOFF with that of EXTRAN, which is a dynamic routing model, for a 52-acre watershed shown in fig. 3.3. The outflow hydrographs are shown in Fig.3.4. It is concluded that the routing in SWMM-RUNOFF is adequate if there is no surcharge. More recent improvements of channel routing by Kassem (1982) have not been considered for this study.

In order to check the accuracy of SWMM for simulating the inlet hydrographs for high intensity storms, an independent test is conducted. The flows simulated by SWMM-RUNOFF are compared with those simulated by the single linear reservoir model developed at the International Workshop, Institute of

Hydrology, Wallingford (1978) (IWW) for a 100 % impervious, 25-acre test watershed. This 25-acre test watershed is built from 2.5-acre inlet areas. For each storm, two different intensities, the peak rainfall intensity and half the peak rainfall intensity are used to assess the effect of using different intensities. Channel routing is done with the variable travel time method. A 100 % impervious test watershed is used in order to eliminate any differences in the rainfall excess distribution. The IWW model is selected because the relation is derived from data obtained from many small inlet areas. The results are shown in Table 3.1.

It can be seen that the peak flows and the times to peak agree very well for both the 2.5-acre inlet areas as well as the 25-acre subwatersheds. This confirms that SWMM can be used for simulating the inlet hydrographs for 100 % impervious areas. SWMM will also be used for other levels of imperviousness. In urban watersheds, most of the runoff comes from the impervious areas. It is felt that if the runoff from the impervious areas can at least be accurately simulated, this will overshadow the errors in simulating the runoff from the pervious areas.

The input storm used is the "Chicago design storm", which gives a conservative, high peak intensity for the design conditions. The storms are derived from typical IDF curves in Ontario. The use of a design storm for urban drainage

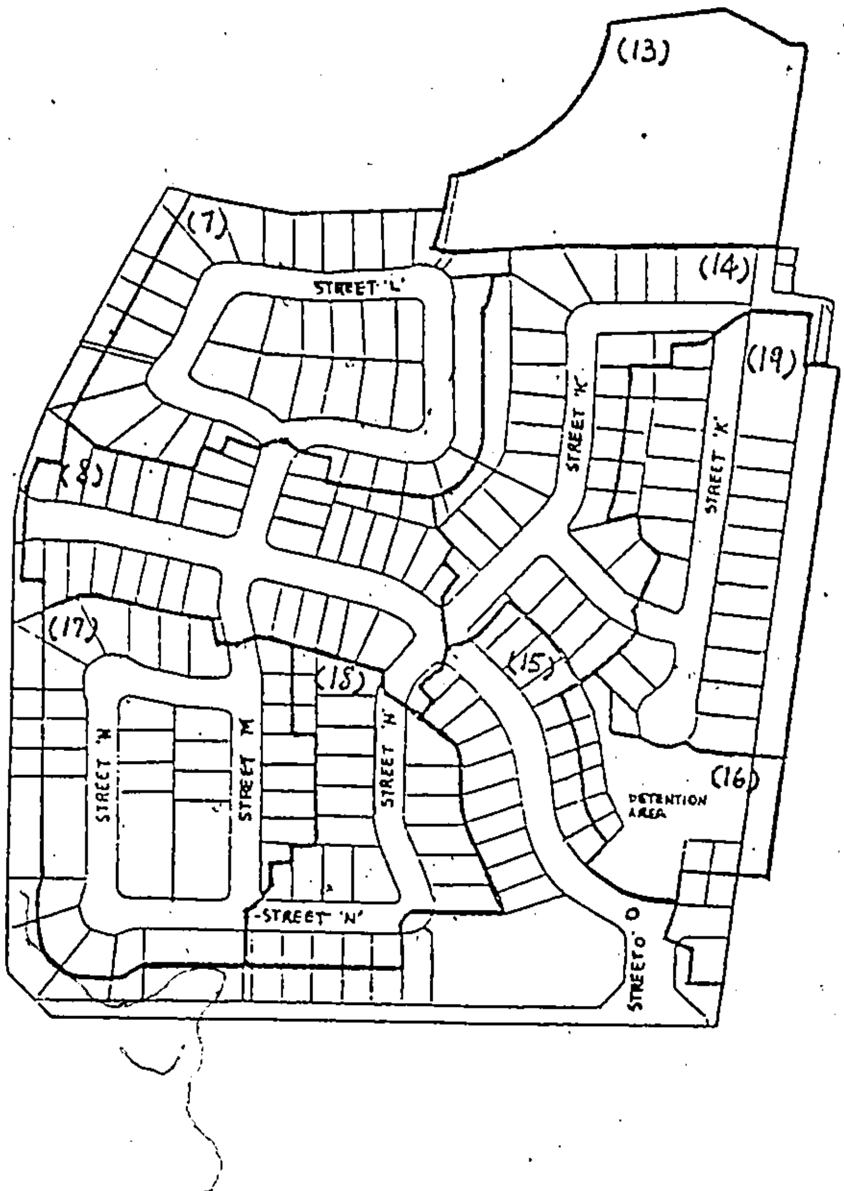


Fig. 3.3 Test Area 'B' 52 Acres

(Ref. Cheung and Kassem, 1979)

design is well accepted in engineering practice in Canada. Although the design storm concept has its shortcomings, it will continue to be popular with engineers because of its facility. Wisner and Gupta (1981) have shown that model simulations using Chicago design storms yield reasonable estimates of the flows when compared to those obtained with a series of real storms.

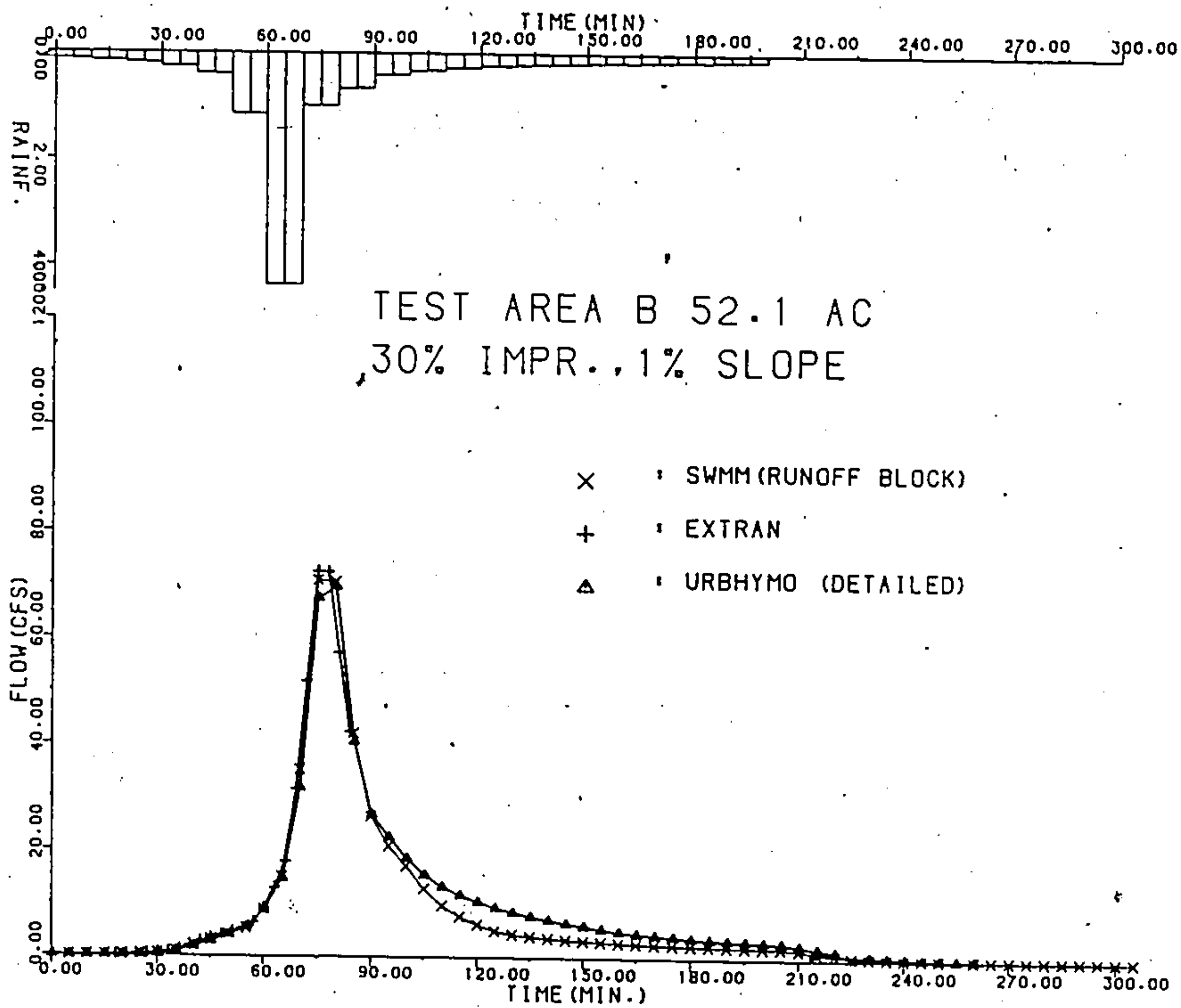


Fig. 3.4 Comparison of hydrographs simulated by SWMM-RUNOFF, EXTRAN and URBHYMO in Test Area B.

Table 3.1

COMPARISON OF PEAK FLOWS SIMULATED BY

SWMM-RUNOFF and the IWW MODEL

25 ac. test watersheds, 2% slope, 100% imperv.

	SWMM		IWW		Differences in
	Peak(cfs)	Time to peak(hr)	Peak(cfs)	Time to peak(hr)	Peak Flows %
1.2 year					
2.5 acres	8.49	0.917	7.78	0.917	8%
25.0 acres	76.94	0.917	76.82	0.917	0
1.2 year (half peak intensity)					
2.5 acres	4.04	0.917	3.87	0.917	4%
25.0 acres	37.66	0.917	38.01	0.917	1
1.5 year					
2.5 acres	12.01	1.75	10.90	1.25	9%
25.0 acres	112.53	1.25	108.18	1.25	4
1.5 year (half peak intensity)					
2.5 acres	5.70	1.25	5.40	1.25	5%
25.0 acres	53.10	1.25	53.37	1.25	1

3.3 SELECTION OF A RAINFALL EXCESS SUBMODEL

Rainfall-runoff modelling can be divided into 2 main parts.

1. determination of rainfall excess distribution.
2. transformation of rainfall excess to runoff

An accurate determination of the rainfall excess is necessary for the successful simulation of runoff. In an urban area, there are 2 general types of surface, impervious areas and pervious areas. The impervious areas can be further subdivided into impervious areas directly connected (hydraulically connected) to the drainage system and those which are non-directly connected. Non-directly connected impervious areas are those that drain onto pervious areas. Some of the runoff from these areas will undergo infiltration and the rate of runoff will be slower than that from the directly connected impervious area. Examples of such areas are rooftops that drain to the backyard and road surfaces that drain to earth ditches. Delleur et al (1975) found that "the proper estimation of the impervious area directly connected to storm sewers is paramount in the estimation of urban runoff." Except for initial losses due to wetting and depression storage, impervious areas are generally considered to have 100 % runoff. Therefore these areas will contribute to runoff even for small storms. The pervious areas will experience a higher depression storage as well as infiltration losses. The rate of infiltration will depend on the soil moisture conditions and the soil type. The soil moisture conditions depend on the antecedent rainfall prior to the beginning of the storm. As the soil gets more and more saturated the infiltration capacity rate decreases. When the rainfall intensity exceeds the infiltration capacity rate, the rainfall excess starts to fill up the depression storage. When the depression storage is filled then runoff occurs.

Previous model comparisons (J.F.MacLaren,1975) did not consider the effect of different rainfall excess in the comparison of peak flows. Due to different infiltration models, the rainfall excess distributions for different models are not the same. Therefore some models may produce higher peak flows because they have a higher rainfall excess volume. In the methodology used here therefore, the same rainfall excess submodel will be used. In some models this rainfall excess distribution is obtained by subtracting a constant rainfall loss (ϕ-index) after accounting for initial abstractions. Or it may be obtained by deducting a constant percentage of the rainfall input. Other models like the SBUH method have calculated the rainfall excess from the impervious and pervious areas separately and then added them together at each time increment. In order to select the rainfall excess submodel three procedures are reviewed.

3.3.1 The Soil Conservation Service Curve Number Procedure

One of the most widely used procedures for estimating the rainfall excess is the Soil Conservation Service Curve Number (CN) procedure. It is used in the SCS models such as TR-55 and TR-20. It is also used in HYMO. It is popular mainly because it requires very little data in order to estimate the CN and it can also reflect land use conditions. Another reason is that it has been recommended in several sewer design manuals (Soil Conservation Service, 1975, American Iron and Steel Institute,1980). In this section the CN procedure is critically assessed as to its suitability for use in urban areas.

CN has no intrinsic meaning but is only a nonlinear transformation of S which is a watershed storage parameter. Although the CN procedure is intended for use in estimating the total direct runoff, it is now used as an infiltration equation with time steps as small as 5 minutes (Aron et al 1979). The CN procedure is also extended to compute the rainfall excess increments for each time step which are obtained as follows. The cumulative direct runoff Q is calculated from the cumulative rainfall P up to the end of the time step. The rainfall excess increment for that time step is obtained by subtracting the cumulative direct runoff up to the end of the preceding time step. The data used for developing the SCS CN procedure are daily precipitation and runoff records from agricultural watersheds and infiltration curves from infiltration studies. Therefore the validity of its use for estimating rainfall excess increments can be questioned (Hawkins,1978). It is found that the CN values for rural watersheds for large storms cannot be applied for small storms (Cheung et al,1981).

In urban areas where there are 2 general types of land use (impervious and pervious portions) a weighted CN procedure has been suggested (Soil Conservation Service, 1975, Miller and Viessman,1972). However it has been found that the SCS CN procedure may severely underestimate the runoff volume particularly for small rainfalls (Wisner, Gupta, Kassem,1981). There may also be significant errors for areas with low CN. Golding (1980) has utilized the SCS CN procedure to simulate runoff from a gauged urban basin in South Florida (58.3 acres, Group A soil, 36 % imperviousness (18 % directly connected)). He has found that the computed initial abstraction Ia amounts to 0.86 in. (CN = 70) which is greater than the total recorded rainfall on the basin which has peak flows of up to 40 cfs in many cases. Reduction of the initial abstraction may give a more realistic runoff volume. Golding has suggested the following values;
(a) $CN < 70$, $I_a = 0.075\ S$; (b) $70 < CN < 80$, $I_a = 0.10\ S$;
(c) $80 < CN < 90$, $I_a = 0.15\ S$; (d) $90 < CN$, $I_a = 0.20\ S$;
Aron et al (1980) recommend a value of 0.10 S or lower for

I_a. Sautier (1979) has determined Ia by measurements and found that it is significantly lower than 0.2 S. Jobin (1981) has obtained good results with measured Ia values.

Burke and Gray (1979) have found that the CN is very sensitive to AMC conditions. For an urban watershed which is 30 % impervious with type B soil, there are 8 to 18 fold variations in runoff volume (from 23000 cu.ft. to 186,000 cu.ft. and 422,000 cu.ft.) for AMC I, II and III.

Based on this review, it is concluded that despite its popularity the CN procedure should not be considered. For short duration high intensity storms which are critical in urban watersheds, the SCS CN procedure underestimates the rainfall excess. Another reason is that the rainfall excess from the pervious and impervious areas are not considered separately.

3.3.2 ϕ-Index Method

One simple method that is used for determining the rainfall excess is the ϕ-index method which is defined as the rate of rainfall above which the rainfall volume equals the runoff volume. For rural watersheds this method generally gives a poor time distribution of the rainfall excess. The error for urban areas may be reduced due to the larger portion of impervious area with zero infiltration. If the rainfall rates are close to the infiltration capacities which vary with time, then the ϕ-index method can give misleading results. Another disadvantage is that rainfall-runoff data is required to determine the ϕ-index. For planning purposes a more physically based approach would be more appropriate. The ϕ-index is used in the Santa Barbara Urban Hydrograph Method where it is estimated as a function of the antecedent precipitation index. The method is also used in the parallel cascades model by Diskin et al (1978).

3.3.3 Horton's Infiltration Procedure

The infiltration rate is the actual rate at which water enters the soil while the infiltration capacity rate is the maximum possible infiltration rate for the prevailing conditions at a certain point in time. If the rainfall rate exceeds the infiltration capacity rate then the infiltration rate will be equal to the infiltration capacity rate. Various infiltration equations have been proposed. One of the most widely used is that of Horton. It is used in models such as SWMM, UCURM, QUURM and ILLUDAS. The Horton equation considers that the infiltration capacity rate is an exponential function of time which decays to a constant rate. It is written as follows

$$f_t = f_c + (f_o - f_c)e^{-\alpha t} \qquad 3.1$$

where

f_t = infiltration capacity rate (in or mm/hr)

f_o = initial infiltration capacity rate (in or mm/hr)

f_c = final infiltration capacity rate (in or mm/hr)

α = decay rate (1/hr)

e = base of natural logarithm

The equation is only satisfactory for the condition that the rainfall intensity is higher than the infiltration capacity rate. To overcome this problem, the cumulative form of the equation can be used. It has the advantage that the infiltration rate becomes a function of the amount of water accumulated into the soil.

$$F = \int_o^t f_t \, dt$$

$$= f_c t + \frac{(f_o - f_c)}{\alpha} (1 - e^{-\alpha t}) \qquad 3.2$$

where F = cumulative infiltration volume at time t

The average infiltration capacity rate during the next time step is

$$\bar{f}_t = \frac{F(t+\Delta t) - F(t)}{\Delta t} \qquad 3.3$$

In order to determine the actual infiltration rate f, the average infiltration capacity rate is then compared with the average rainfall intensity i during the period Δt.

$$f = \begin{cases} \bar{f_t} & \text{if} \quad i \geq \bar{f_t} \\ i & \text{if} \quad i < \bar{f_t} \end{cases} \qquad 3.4$$

If $f = \bar{f_t}$, then the calculation proceeds to the next time step with the cumulative infiltration volume at $F(t+\Delta t)$. If $f = i$, then the actual cumulative infiltration would be

$$F_{actual} = F(t) + i \, t \qquad 3.5$$

where

$$F_{actual} < F(t+\Delta t)$$

The new time t_1 which would correspond to the cumulative infiltration F_{actual} is determined by means of an iterative process. The calculation then continues from this point for the next time step.

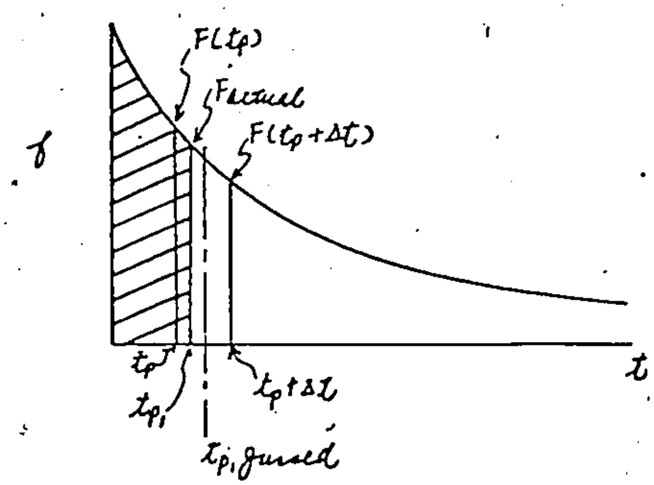


Fig. 3.5 Cumulative form of Horton's Infiltration equation (Ref. Cheung, 1980)

Since the cumulative form of Horton's infiltration equation has a better physical interpretation than the other two rainfall loss models, it will be used for determining the rainfall excess in the planning model. The antecedent moisture condition can be represented by the water, F, accumulated into the soil before the start of the storm. F can be directly specified as input. The other infiltration parameters also need to be specified.

3.3.4 Depression Storage

Depression storage represents that portion of the rainfall that is abstracted by the ground before runoff occurs. Due to its variability it is usually assumed to be a constant over the entire watershed. Some models use different depression storage constants for the impervious and pervious portions of the watershed. Several investigators have related the depression storage of impervious areas to the slope S of the watershed.

$Ds = 4.1 - 1.00 \cdot S$ (Willeke,1966)

$Ds = 3.30 - 0.77 S$ (Viessman,1968)

$Ds = 0.86 - 0.14 S$ (Falk and Niemczynowiecz,1970)

where Ds = depression storage (in.)

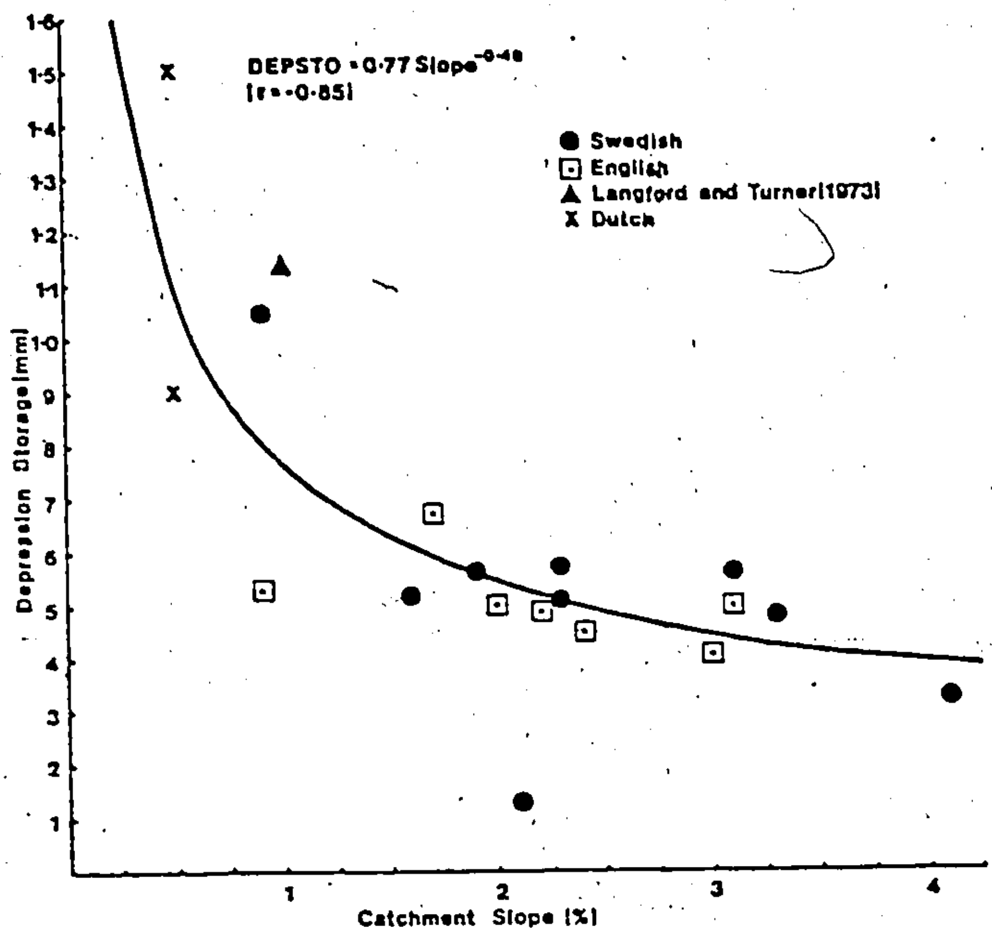


Fig. 3.6 Relationship between depression storage and Catchment Slope (Ref. Institute of Hydrology, Wallingford, 1978)

However, the data base for the above relationships are limited and therefore cannot be recommended for general use. Another study (Institute of Hydrology,1978) involving 22 watersheds has resulted in a relationship for depression storage as a function of slope.

$$d = 0.77 \text{ slope}^{-0.49} \qquad 3.6$$

where d = depression storage (mm), slope = %

This relationship shown in Fig.3.6 can be used to estimate the depression storage for impervious areas since it is derived from the largest data base. Consequently it is incorporated as a default equation in the model. The depression storage for both the impervious and pervious areas can also be directly specified for input.

The rainfall excess submodel programmed in URBHYMO has the capacity of considering 3 types of areas, i.e. the directly and non-directly connected impervious areas and the pervious areas. The directly connected impervious areas undergo only initial losses due to depression storage. The non-directly connected impervious areas are assumed to run onto pervious areas. Therefore it means that the pervious areas receive an extra amount of rainfall that can undergo infiltration. The rainfall on the pervious area is thus increased by a percentage equal to the ratio of the non-directly connected impervious area to the pervious area.

3.4 SELECTION OF AN OPTIMIZATION PROCEDURE

Delleur (1981) in a recent literature review of mathematical models in urban hydrology noted that the current trend is towards more exact models and towards automatic calibration of the models. Optimization can be defined as the process by which the "best" estimates of model parameters are determined according to some specified criterion of how well the calculated output fits the observed data. In this study the "observed" data will be the outflow hydrographs obtained from the mathematical watershed model. An optimization procedure therefore needs to be utilized in order to obtain the "best" estimates of the parameters of the planning model corresponding to the "observed" outflow hydrographs. Once obtained, these parameter estimates can then be compared with the parameters predicted by various relations in order to determine the best relations for the planning model. In optimization there are two aspects to be considered.

1. the selection of the optimization method itself i.e. the systematic variation of the parameters.
2. the selection of the objective function or measure of goodness-of-fit by which the model is judged.

Ibbitt and O'Donnell (1971) have examined nine leading optimization techniques to estimate the parameters of a conceptual model and concluded that Rosenbrock's method (1960) is the most effective parameter estimation method. Sorooshian (1980) also has compared two direct search algorithms used in the calibration of rainfall-runoff models and concluded that for the estimation of sensitive parameters in rainfall-runoff models, Rosenbrock's method is to be preferred. Han and Rao (1980) have utilized Rosenbrock's method to optimize the parameters in the single event version of ILLUDAS. Wenzel and Voorhees (1980) have also utilized Rosenbrock's method for their continuous version of ILLUDAS. Han and Rao (1980) also incorporated the optimization procedure in the SWMM-RUNOFF Block. There is found to be a significant improvement in the fit between the observed and optimized hydrographs as compared to those estimated by following the user's manual. The distributed rainfall-runoff model of the USGS (Dawdy,Schaake,Alley,1978) has a modified Rosenbrock optimization technique for optimizing the soil moisture and infiltration parameters. Therefore the Rosenbrock optimization technique is selected for use with the planning model. The subroutine that is used is a subroutine called CLIMB developed at the University of Waterloo Computing Centre.

Diskin and Simon (1977) have analyzed the procedure to be used for the selection of an objective function and concluded that the selection depends on the problem encountered. For example, for the design of storage facilities the objective function should emphasize the total runoff volume while for pipe sizing the peak flows should be emphasized. In this study the objective function used will be the differences in peak flows of the computed and "observed" hydrographs. Another objective function that may be used is the sum of the squares of the differences between calculated and observed hydrograph ordinates. It should be noted that the optimal parameter estimates are optimal only with respect to the objective function used. An objective function is needed even for trial-and-error methods. An example of the comparison between an optimized hydrograph using the SLR model and the hydrograph obtained from the detailed SWMM simulation is shown in fig.4.17. The runoff volumes need not be considered since they are the same from both the mathematical watersheds and the planning model. For comparison with measured data on real watersheds, however, the runoff volumes would have to be considered.

The optimization procedure has been interfaced with URBHYMO. Actually, for the objective function of minimizing the differences in peak flows and the optimization of one parameter, trial-and-error methods will suffice. However, the use of the optimization procedure will be comparatively more efficient if several parameters have to be optimized. Several objective functions have been programmed in the 'ERROR' subroutine of HYMO. An example of the output using the optimization procedure is shown in Appendix D which shows the comparison between the optimized hydrograph corresponding to a specified objective function and the observed hydrograph for a storm on the Gray Haven watershed.

Chapter IV

SINGLE LINEAR RESERVOIR MODELS

4.1 25-ACRE (10 HA) TEST WATERSHEDS

4.1.1 Introduction

The first series of experiments is to analyze the applicability of the single linear reservoir models under design conditions for watersheds of 25 acres in size. The outflow hydrographs for the test watersheds of 25 acres are obtained by simulating the 2.5-acre inlet hydrographs and then routing them by means of the SWMM-RUNOFF Block. The ground slope for the inlet areas is 2 %. The values for the infiltration parameters are the SWMM default values. The watersheds are at 5 levels of imperviousness, 25 %, 40 %, 60 %, 80 % and 100 %. The single linear reservoir model with the incorporated rainfall excess submodel is then applied in a lumped manner. Using the Rosenbrock optimization subroutine the storage coefficient K is optimized based on the objective function of minimizing the differences between the peak flows. It can be seen that the optimized K values decrease with increasing imperviousness. The K values predicted by various relations are compared with the optimized K values and analyzed below.

The relations used in the analysis are divided into two groups as shown in table 4.1. The K values are given in hours. The first group of relations for K (except K1) are derived from the kinematic wave theory. The storage coefficient is a function of length l, slope S, surface roughness n and input rate i. To use these relations, it is assumed that the watershed is a planar surface. For partially impervious watersheds an average 'n' reflecting the level of imperviousness would be required. It is considered that the average 'n' should be decreasing for increasing imperviousness. The K values are calculated by using different n values for different percentages of imperviousness. The 'n' values used are 0.040, 0.027, 0.020, 0.017 and 0.013 for imperviousness levels of 25 %, 40 %, 60 %, 80 % and 100 % respectively. The 'n' values are for overland flow which may be different from open channel values. The second group of relations are derived from multiple regression analysis. K5, K9 and K10 are relations for obtaining the time lag which is theoretically equal to the storage coefficient of the single linear reservoir. One purpose of the tests is to see whether these relations can be extended to conditions beyond their original application. The rainfall excess distribution used in all the tests are obtained by adding up the contributions from the impervious and pervious areas for each time step.

Table 4.1 RELATIONS FOR STORAGE COEFFICIENT K (hrs.) OF THE SLR MODEL

Author	Relation
1st. Group	
VIESSMAN (1968)	$K1 = 0.004 \dfrac{L^{0.66} n^{0.66}}{S^{0.330}}$
NEUMANN (1977)	$K2 = 0.00719 \dfrac{L^{0.593} n^{0.605}}{i^{0.388} S^{0.38}}$
INTERNATIONAL WORKSHOP, INSTITUTE OF HYDROLOGY, WALLINGFORD (1978)	$K3 = 0.02588 \dfrac{L^{0.22}}{i^{0.40} S^{0.40}}$
PEDERSEN, PETERS, HELWEG (1980)	$K4 = 0.00775 \dfrac{L^{0.60} n^{0.60}}{i^{0.40} S^{0.30}}$
2nd. Group	
SARMA, DELLEUR, RAO (1969,1972,1973)	$K5 = 0.831 \dfrac{A^{0.458} T_R^{0.371}}{(1+U)^{1.662} P_e^{0.267}}$
	$K6 = 0.887 \dfrac{A^{0.490} T_R^{0.294}}{(1+U)^{1.683} P_e^{0.240}}$
	$K7 = 0.788 \dfrac{A^{0.409} T_R^{0.156}}{(1+U)^{2.06} P_E^{0.15}}$
DESBORDES (1978)	$K8 = 0.11386 \dfrac{A^{0.18} L^{0.15} T_D^{0.21}}{(1+U)^{1.90} S^{0.36} H_p^{0.07}}$
SCHAAKE, GEYER, KNAPP (1967)	$K9 = 0.175 \dfrac{L^{0.24}\, Ximp^{0.26}}{S^{0.16}}$
SCHULZ & LOPEZ (1974)	$K10 = 0.0041837 \dfrac{T_s^{0.31} P_e^{0.323} H_r^{0.80}}{U_r^{0.342}}$

The performance of each relation as compared to optimized values will be discussed first and the general conclusions given next. Both the K values and the resulting peak flows are compared.

4.1.1.1 Viessman (1968,1970)

$$K_1 = 0.00025\,[66.6667\ n.L.S^{-0.5}]^{0.66} \qquad 4.1$$

where n = Manning's roughness coefficient

L = max. gutter flow distance (ft)

S = slope (ft/ft)

The relation is originally derived from 4 small impervious areas with slopes ranging from 1 to 2.5 %. The equation is also tested on a pervious watershed and found to give good results (Viessman,1970). K1 is not a function of any storm characteristic or rainfall excess. It is not a function of imperviousness and would result in a constant value for different imperviousness levels. However by changing 'n' values corresponding to different imperviousness levels, this will be reflected in different K values. The computed K1 values are compared with the optimized K values in figs. 4.1, 4.3 and 4.5 for the 1:2 year, 1:5 year and 1:25 year storms respectively, with the corresponding peak flows plotted in figs.4.2,4.4 and 4.6. The computed K values are slightly higher than the optimized K values for the 1:2 year storm. However the resulting peak flows are close to the peak flows from SWMM being within 11 % as shown in table 4.2. For the 1:5 year storm the computed peak flows are overpredicted for the watersheds with lower imperviousness. For a 25 % impervious watershed, the peak flow is overpredicted by 39 % for the 1:5 year storm. A similar trend is also observed for the 1:25 year design storm (fig.4.6).

The surface roughness n may be used as a calibration parameter to improve the results for the more intense storms at lower imperviousness levels. The results indicate that for some conditions, K1 is not considered a suitable relation for estimating the storage coefficient for the SLR model.

4.1.1.2 Neumann (1977)

$$K_2 = \frac{0.00719\ L^{0.593} n^{0.605}}{i^{0.388} S^{0.38}} \qquad 4.2$$

where L = length of flow (ft)

n = Manning's roughness coefficient

S = slope (ft/ft)

i = rainfall excess intensity (in/hr)

Neumann adapted K2 from the equation derived by Morgali and Linsley (1965), relating the time to equilibrium t_e with the same parameters as indicated above. The equation for outflow is

$$Q = I(1 - e^{-t/K})$$

When $t = \infty$, the above equation gives $Q = I$, which means that the outflow approaches an equilibrium condition becoming equal to inflow. Neumann assumed that equilibrium occurs when $Q = 0.90\ I$.

$$\frac{Q}{I} = 1 - e^{-t/k} \qquad ;t = t_e$$

$$e^{-t_e/K} = 1-0.90 = 0.10$$

$$t_e/K = \ln 10$$

$$\frac{K}{t_e} = \frac{1}{2.3025} = 0.4343 \tag{4.3}$$

Therefore K can be obtained from t_e. The model is quasi-linear since it is a function of the rainfall excess intensity. The rainfall excess intensity i is assumed to be the maximum 10-minute rainfall excess intensity. This is also done for K3 and K4. It can be applied to different types of surfaces since it includes the parameter 'n'. The 'n' values that are used for different levels of imperviousness are the same as those used for K1. The computed storage coefficients and the corresponding peak flows are shown in figs.4.1 - 4.6. For the 1:2 year storm, the computed K2 values decrease with increasing imperviousness and are higher than the optimized K values. The resulting peak flows are significantly underpredicted. The peak flow ratios (K2/SWMM) increase from 0.64 for 25 % imperviousness to 0.93 for 100 % imperviousness. The results improve as imperviousness levels increase since the differences in rainfall excess distributions are minimized. For the 1:5 year storm, the peak flow is 22 % overpredicted at 25 % imperviousness but the differences are within 11 % for 40 % imperviousness and above. For the 1:25 year storm the peak flow is also overpredicted at 25 % imperviousness. Of those in the first group, the K2 relation results in the best prediction for peak flows for the 1:5 year storm. But for the 1:2 year storm it does not predict peak flows as accurately as those of Viessman (K1).

4.1.1.3 International Workshop, Institute of Hydrology, Wallingford (1978) (IWW)

$$K_3 = 0.02588\ S^{-0.40}\ L^{0.22}\ i^{-0.40} \qquad 4.4$$

where S = ground slope (%)

L = overland flow length (ft)

i = average intensity of rainfall excess during the most intense 10 minutes (in/hr)

At this International Workshop several research groups from Europe have pooled together data from 20 small urbanized watersheds in the Netherlands, Sweden and the United Kingdom and compared the performance of various models. The data is collected at the inlet or the interface between the above-ground phase of runoff and the below-ground phase which is the sewer-routing component of the rainfall-runoff process. 13 of the watersheds are fully-paved (100 %). However this does not mean that they are fully impervious. The Dutch, for example, pave some portion of the watershed with bricks which may result in infiltration losses of up to 40 %. Even for watersheds paved with asphalt or concrete there is a certain amount of infiltration occurring. Most of the watersheds are small inlet areas of less than 1 acre. The models are compared and it is concluded that the single linear reservoir model performs satisfactorily. It is a quasi-linear model since K is a function of the rainfall excess intensity. Morgali and Linsley (1965) related the time to equilibrium t_e to several watershed characteristics like slope, length, roughness and rainfall excess intensity i. From this it is assumed that the storage coefficient K is of the form $C.i^{-0.40}$ The parameter C is then optimized and regressed on watershed characteristics to obtain

$$C = 0.02588\, S^{-0.40}\, L^{0.22}$$

which results in the relation for K3.

The watershed tested is 25 acres which is larger than the watersheds used in the derivation of the relation. This test is to see whether the relation can be extended for larger watersheds. The length parameter L is assumed to be 850 feet which is the width of a 25-acre rectangular watershed of equivalent area. Another test is done where L is estimated by the length (which is 1.5 times the width) of the watershed. The slope parameter is taken to be the ground slope.

K3 does not appear to be very sensitive to length L. It will be affected however by the method used in the determination of rainfall excess. The rainfall excess submodel used here adds up both the contributions from the pervious and impervious portions of the watershed though in reality there is a lagging effect on the rainfall excess from the pervious areas. Therefore the peak 10-minute rainfall excess intensity, and consequently K3, is a function of not only the rainfall intensity but also the infiltration characteristics and the percentage of impervious area. However the results in figs.4.1,4.3 and 4.5 indicate that K3 does not change significantly for different percentages of imperviousness. The calculated K values are very close to the optimized K values for the 1:2 year storm. This results in peak flows which are within 11 % of those of SWMM. But for both the 1:5 year and the 1:25 year storms, the calculated K values are found to be much lower than the optimized K values. The peak flows are found to be highly overpredicted as shown in figs.4.4 and 4.6.

Among the relations in the first group, K3 gives the best prediction for peak flows for the 1:2 year storm and the worst for the 1:5 year and 1:25 year storms. The overprediction of peak flows for the 1:5 year and 1:25 year storms is due to the high contribution from the pervious areas which is not lagged since K3 is not a function of roughness 'n'. It can be concluded that K3 cannot be used for predicting the peak flows from partially impervious watersheds.

4.1.1.4 Pedersen, Peters and Helweg (1980)

$$K_4 = 0.00775\ L^{0.60}\ n^{0.60}\ i^{-0.40}\ S^{-0.30} \qquad 4.5$$

where L = length of plane (ft)

n = Manning's roughness coefficient

i = constant rainfall excess intensity (in/hr)

S = slope of planar surface (ft/ft)

This relation is based on the following equation for the time to equilibrium t_e derived from kinematic wave theory assuming turbulent flow conditions (Ragan and Duru,1972).

$$t_e = 0.93\ \frac{(L.n)^{0.60}}{i^{0.40}S^{0.30}} \qquad 4.6$$

Overton and Meadows (1976) derived the expression

$$t_e = 1.6\ t_{50} \qquad 4.7$$

where t_{50} is the time difference between 50 % of rainfall excess and 50 % of the direct runoff volume, and t_e is defined by eqn.4.6. Implicit in eqn.4.7 is the relation

$$t_{50} = \frac{Seq}{i} \qquad 4.8$$

where Seq is the storage at equilibrium and i is the constant rainfall excess intensity. This relation is derived in Overton (1970). However it is found that t_e obtained by eqn.4.7 and eqn.4.8 is about 0.8 times t_e obtained from eqn.4.6.

$$\frac{1.6 \frac{Seq}{i}}{t_e} = 0.8 \qquad 4.9$$

$$2 \frac{Seq}{i} = t_e$$

Pedersen et al (1980) set Seq /i equal to time lag t_L to obtain

$$2t_L = t_e$$

This means that $t_L = 0.8\ t_{50}$. Therefore

$$t_L = \frac{t_e}{2} = \frac{0.93(L.n)^{0.60}}{2i^{0.40}S^{0.30}} \qquad 4.10$$

which is equal to K4 after conversion to hours. For a storm input, the rainfall excess intensity is not constant.

FIG.4.1

COMPARISON OF OPTIMIZED AND PREDICTED

STORAGE COEFFICIENTS K FOR SLR MODEL

25 AC. WATERSHED

1:2 YEAR STORM

K1,K2,K3,K4

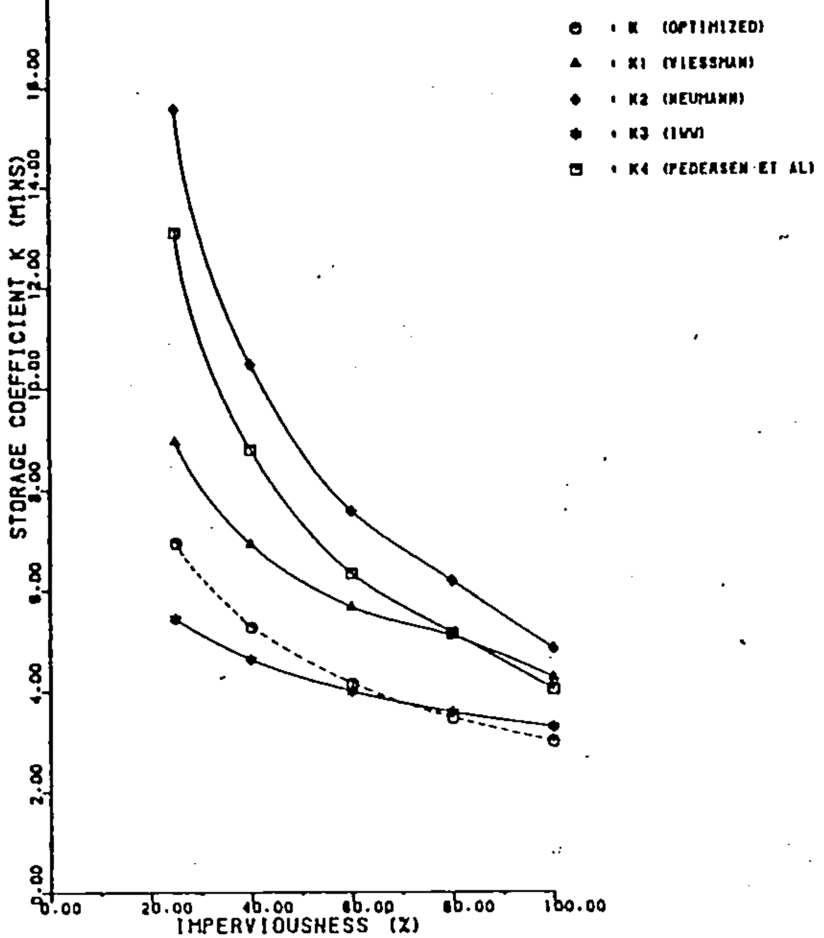


FIG.4.2

COMPARISON OF PEAK FLOWS FOR SLR MODEL

25 AC. WATERSHED, 1:2 YEAR STORM

K1,K2,K3,K4

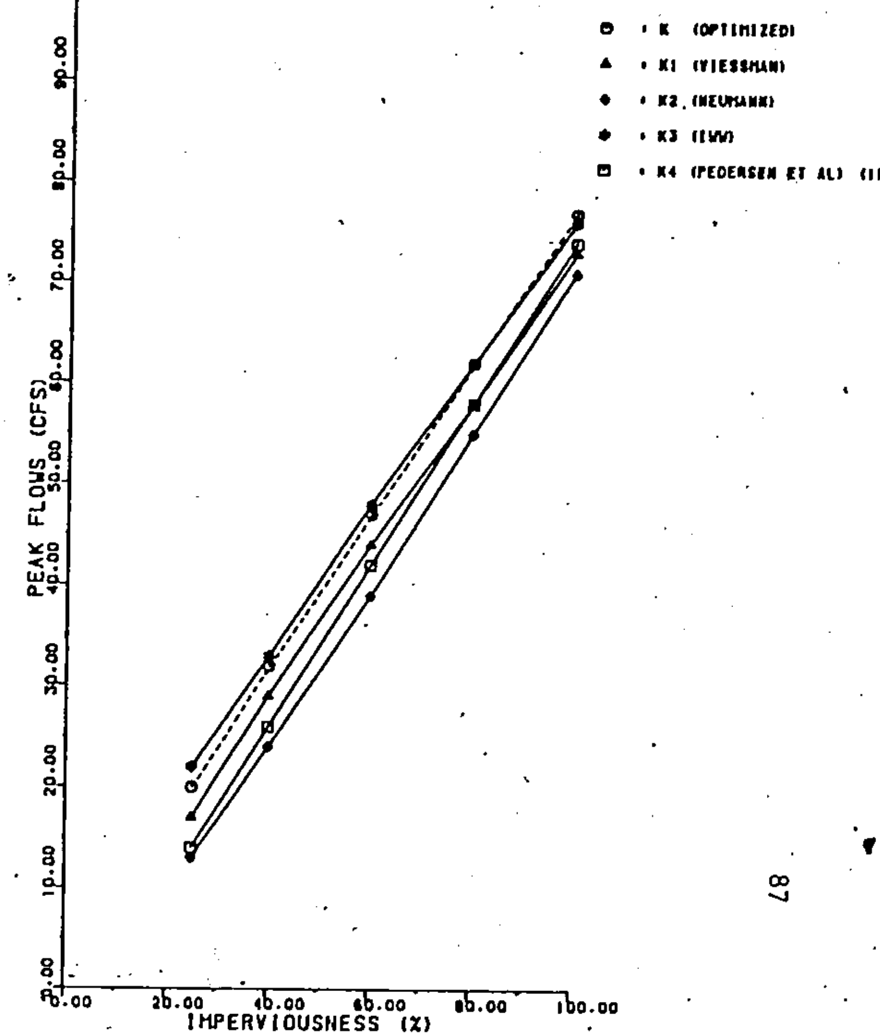


FIG.4.3

COMPARISON OF OPTIMIZED AND PREDICTED

STORAGE COEFFICIENTS K FOR SLR MODEL

25 AC. WATERSHED

1:5 YEAR STORM

K1,K2,K3,K4

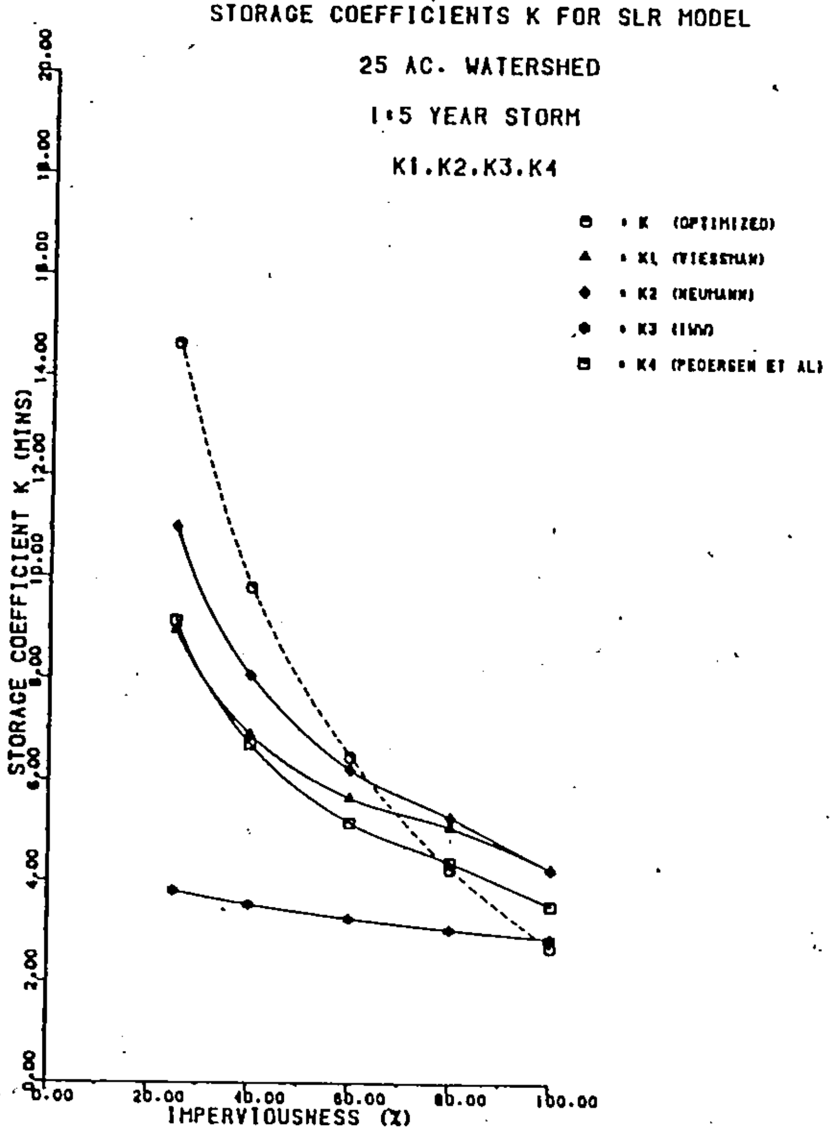


FIG. 4.4

COMPARISON OF PEAK FLOWS FOR SLR MODEL

25 AC. WATERSHED, 1:5 YEAR STORM

K1,K2,K3,K4

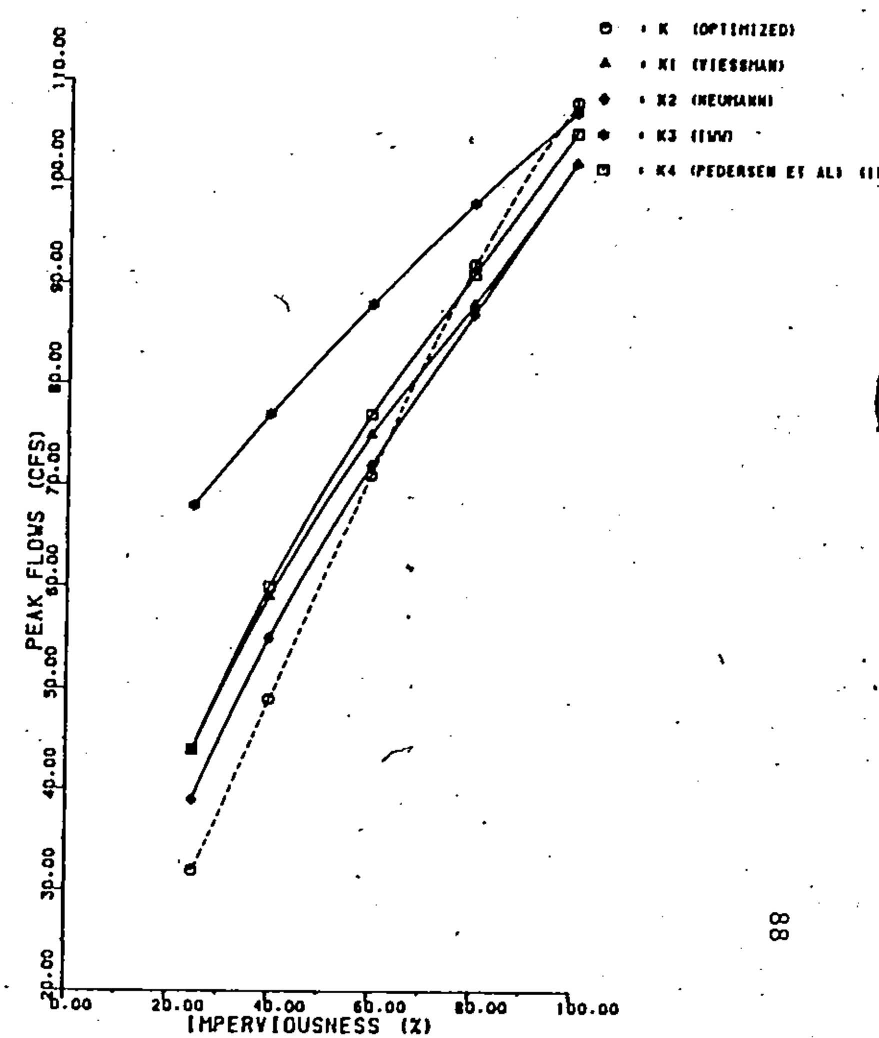


FIG. 4.5

COMPARISON OF OPTIMIZED AND PREDICTED STORAGE COEFFICIENTS K FOR SLR MODEL

25 AC. WATERSHED

1:25 YEAR STORM

K1, K2, K3, K4

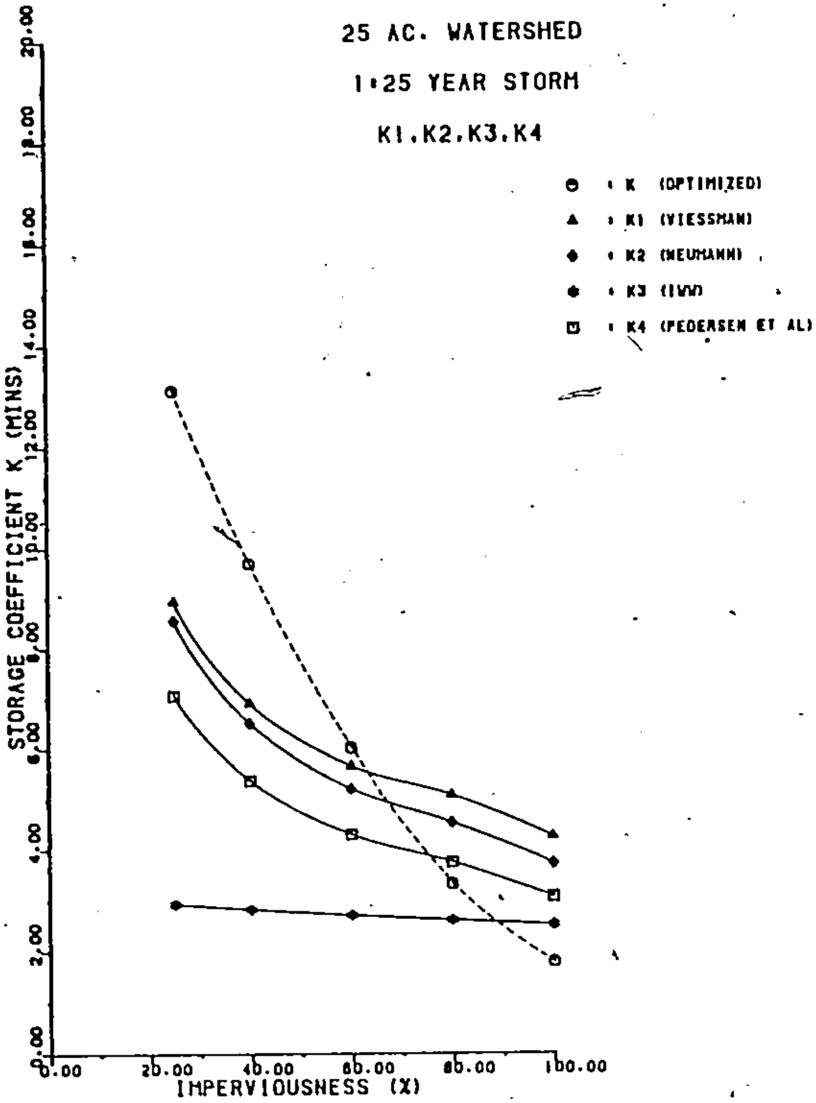
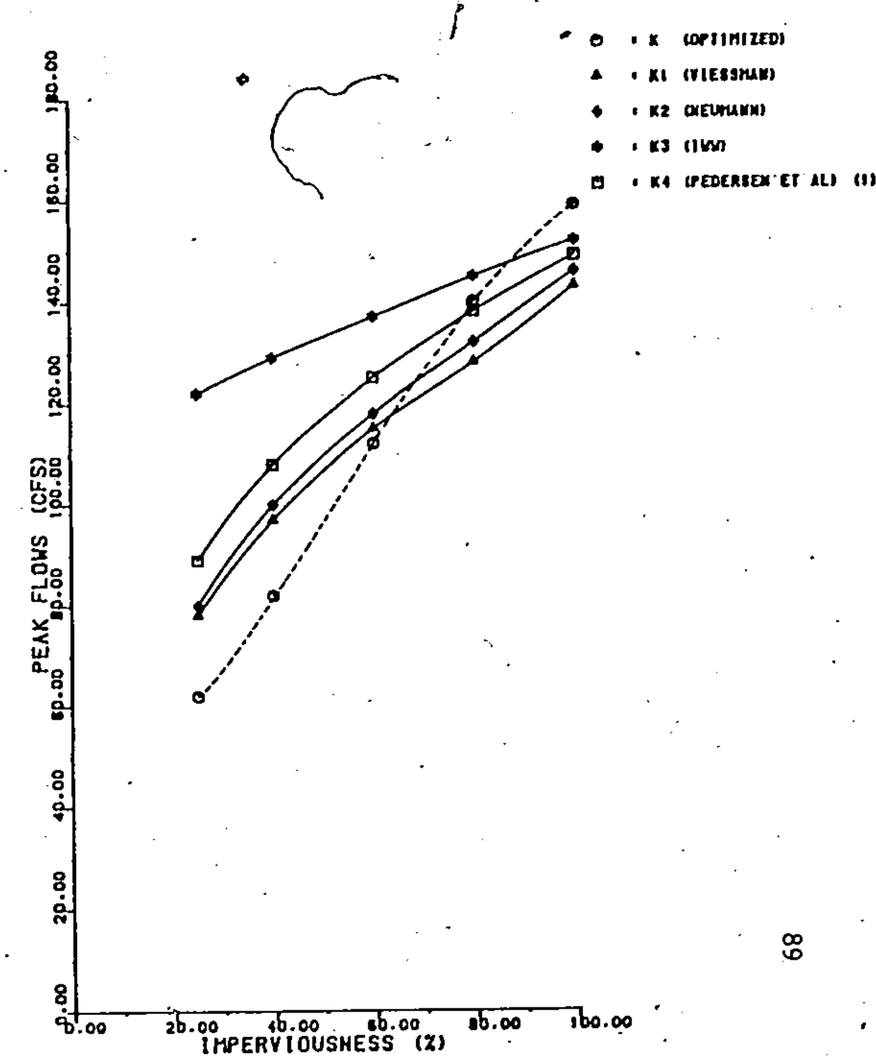


FIG. 4.6

COMPARISON OF PEAK FLOWS FOR SLR MODEL

25 AC. WATERSHED. 1:25 YEAR STORM

K1, K2, K3, K4



Therefore it is assumed to be the maximum 10-minute rainfall excess intensity.

The peak flows are underpredicted for the 1:2 year storm and are overpredicted for both the 1:5 year and 1:25 year storms. The peak flow ratios are tabulated in table 4.2. For a 40 % impervious watershed, the peak flow ratio for the 1:2 year storm is 0.82 while for the 1:5 and 1:25 year storms it is 1.22 and 1.32 respectively. These differences are considered too high and therefore it is concluded that K4 cannot be used for predicting the storage coefficient for the SLR model.

4.1.2 Second Group of Relations

The second group of relations for the storage coefficient is obtained from multiple regression analysis. The relations derived by Sarma, Delleur and Rao (1969) (K5,K6,K7) and Desbordes (1978) (K8) have been applied in the single linear reservoir model. The relations derived by Schaake et al (1967) (K9), Schulz and Lopez (1974) are actually relations for the time lag t_L and have not been applied previously for the SLR model. Therefore this extension of their original use is to determine whether they can be applied.

4.1.2.1 Sarma, Delleur, Rao (1969,1972,1973) (SDR)

According to this study the storage coefficients are

$$K_5 = 0.831\ (A*0.0015625)^{0.458}Uf^{-1.662}P_e^{-0.267}T_r^{0.371} \quad 4.11$$

$$K_6 = 0.887\ (A*0.0015625)^{0.490}Uf^{-1.683}P_e^{-0.240}T_r^{0.294} \quad 4.12$$

$$K_7 = 0.788(A*0.0015625)^{0.409}Uf^{-2.06}P_e^{-0.15}T_r^{0.156} \quad 4.13$$

where A = area (acres) [0.0015625 is a factor for conversion of acres to square miles]

Uf = 1 + timp [timp = ratio of total impervious area to total area].

Pe = rainfall excess (in.)

Tr = duration of rainfall excess (hr.)

Theoretically the storage coefficient K is equal to the time lag which is defined as the time difference between the centroids of the rainfall excess and the direct runoff which is K5 in this case. K6 is obtained from optimization based on minimizing the objective function of the sum of the squares of the differences between observed and calculated hydrographs. K7 is obtained from optimization based on minimizing the objective function of the differences between the peaks of the observed and calculated hydrographs.

The total impervious area or built-up area is defined as "the area in the watershed which does not contribute to direct infiltration. Paved roads, parking lots, building roofs, lawns, and sidewalks have been considered impervious areas."(Sarma et al,1969 ,pg.76) A total of 131 storms are analyzed.

It can be observed from eqns.4.11 - 4.13 that the storage coefficient decreases with increasing urbanization and increasing rainfall excess. It increases with the area and the duration of the rainfall excess. This is consistent with physical processes. Since the storage coefficient is a function of the storm characteristics Pe and Tr this makes it a quasi-linear model i.e. the storage coefficient and consequently the IUH is constant during each storm but varies from storm to storm. Different methods of determining the rainfall excess volume will result in different rainfall excess which will affect the storage coefficient. The time of rise of the direct runoff hydrograph is taken to be the start of rainfall excess. The duration of rainfall excess, Tr is measured from this point to the cessation of rainfall. For urban areas, rainfall excess usually commences when the initial wetting losses on the impervious portion of the watershed is satisfied. Therefore Tr will depend on the value of the depression storage on the impervious area. This shows that the storage coefficient K of the SDR method is affected not only by the watershed and storm characteristics but also by the rainfall excess submodel.

Two other regression equations relating the peak flow and the time to peak to the same parameters used in the determination of the storage coefficient have also been developed.

$$Q_p = 484.1(A*0.0015625)^{0.723} U_f^{1.516} P_e^{1.113} T_r^{-0.403} \quad 4.14$$

$$T_p = 0.775(A*0.0015625)^{0.323} U_f^{-1.285} P_e^{-0.145} T_r^{0.634} \quad 4.15$$

where Qp = peak flow (cfs)

Tp = time to peak (hr)

The K values computed from the SDR formulas are compared with the optimized K values in figs.4.7, 4.9 and 4.11. The corresponding peak flows are plotted in figs.4.8, 4.10 and 4.12. It can be seen that for the 1:2 year storm, the peak flows predicted by K7, out of the three relations, are the most accurate. The differences in the peak flows are within 20 % for the imperviousness levels tested. For the 1:5 year storm, however, K7 overpredicts the peak flows at lower imperviousness levels. For a 25 % impervious watershed the peak flow is overpredicted by 28 %. For this storm, K5 gives the best results at the lower imperviousness levels, the differences in the peak flows being within +10 %. For the 1:25 year storm, the peak flow ratios for K5 range from 0.86 to 1.00 while for K6, they vary from 0.91 to 1.10. For K7, the peak flow ratios vary from 0.96 to 1.20. For highly impervious watersheds, K7 produce the best prediction for all three storms. K6 predicts peak flows which are in between those predicted by K5 and K7. A hydrograph computed by using the SLR model for the K7 relation is shown in fig.4.18. It is observed that the general fit of the hydrographs is very good. Since the peak flows are reasonably well predicted for the 2 larger storms, the model is programmed on a calculator. The flow chart and the listing for the program are shown in Appendix E.

4.1.2.2 Desbordes (1978)

$$K_8 = 0.11386\, A^{0.18} S^{-0.36} U_f^{-1.90} TD^{0.21} L^{0.15} H_p^{-0.07} \qquad 4.16$$

where A = area (acres)

S = mean slope (%)

L = watershed length (ft.)

Uf = 1 + Timp

TD = duration of 'heavy' part of rainfall (hr.)

Hp = rainfall depth during TD (in.)

K8 is derived from multiple regression analysis from rainfall-runoff data of 21 French and American urban watersheds. The American watersheds are those used in the study of Sarma, Delleur and Rao (1969). The range of values for the parameters are

1 acre < A < 12355 acres ; 360.9 ft < l < 58402 ft

0.4 % < slp < 5 % ; 0.16667 hr < TD < 3.0 hr

0.20 in < Hp < 9.45 in

The range of data covered by this regression equation is wider than that of SDR and includes smaller watersheds as well. Tests by Desbordes with the model have shown good results in almost all cases (less than 20 % error in the peaks and 10 % in the times to peak). For partially urbanized watersheds, Timp is replaced by the instantaneous runoff coefficient. This model is also quasi-linear since K is a function of rainfall characteristics TD and Hp. Since TD is not defined exactly, in the tests conducted here, it is assumed that TD equals 10 minutes since that is the duration of the peak rainfall intensity.

The watershed length used is 850 feet. The K8 values are also computed using a length of 1280 feet. The storage coefficients are not found to be sensitive to the watershed length L. The predicted storage coefficients are plotted in figs.4.7, 4.9 and 4.11 and the corresponding peak flows are plotted in figs.4.8, 4.10 and 4.12 for the 1:2 year, 1:5 year and the 1:25 year storms respectively. The computed K8 values are found to be very close to the computed K5 values. For the 1:2 year storm, the peak flows are underpredicted with the peak flow ratios ranging from 0.71 for the 25 % imperviousness level to 0.92 for the completely impervious watershed. For the 1:5 year and 1:25 year storms, the predicted peak flows are found to be close to the optimized peak flows with the differences being within +11 % as shown in table 4.2. The computed K8 values are found to be very close to the computed K5 values.

FIG. 4.7

COMPARISON OF OPTIMIZED AND PREDICTED STORAGE COEFFICIENTS K FOR SLR MODEL

25 AC. WATERSHED

1:2 YEAR STORM

K5,K6,K7,K8

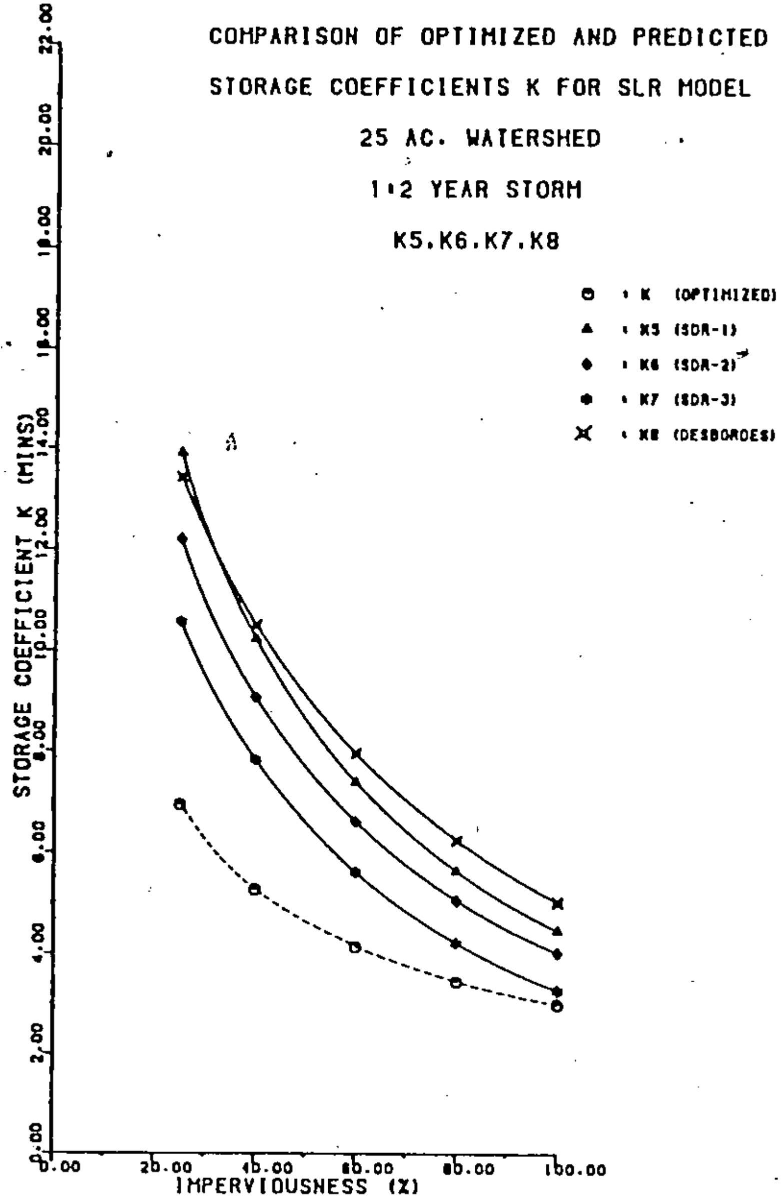


FIG.4.8

COMPARISON OF PEAK FLOWS FOR SLR MODEL

25 AC. WATERSHED, 1:2 YEAR STORM

K5,K6,K7,K8

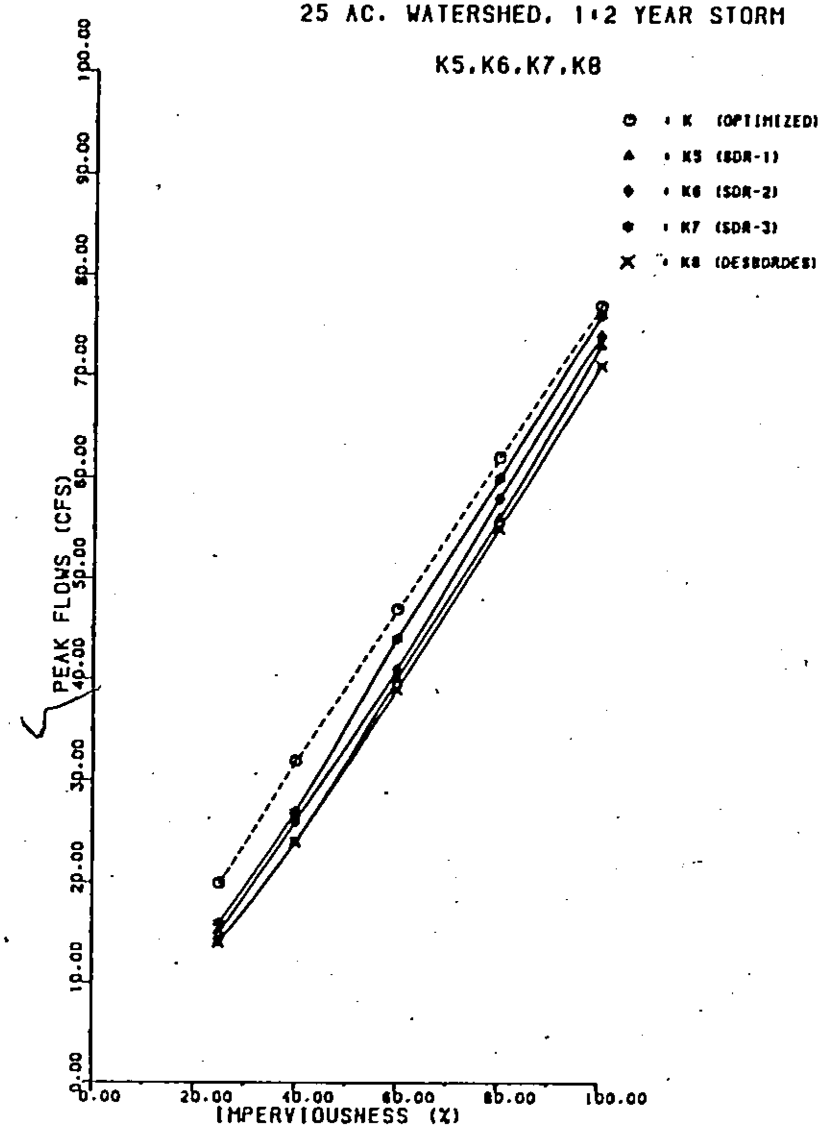


FIG. 4.9

COMPARISON OF OPTIMIZED AND PREDICTED

STORAGE COEFFICIENTS K FOR SLR MODEL

25 AC. WATERSHED

1:5 YEAR STORM

K5.K6.K7.K8

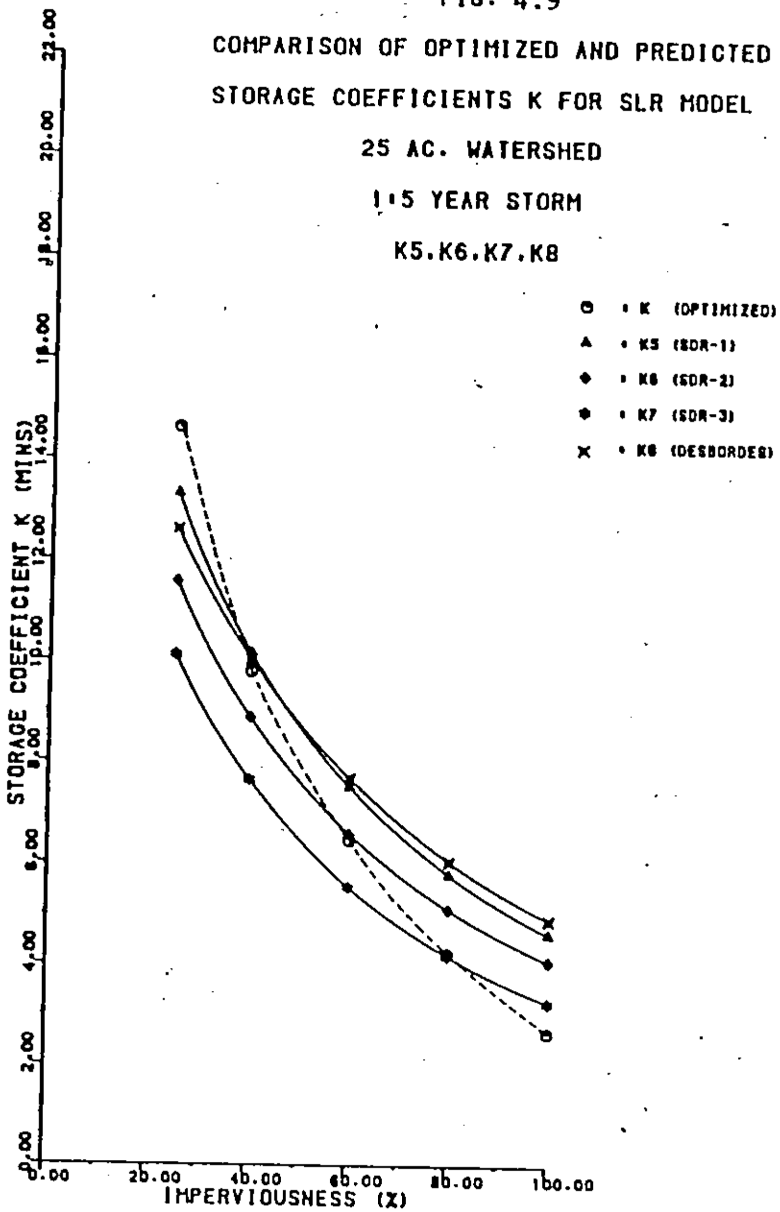
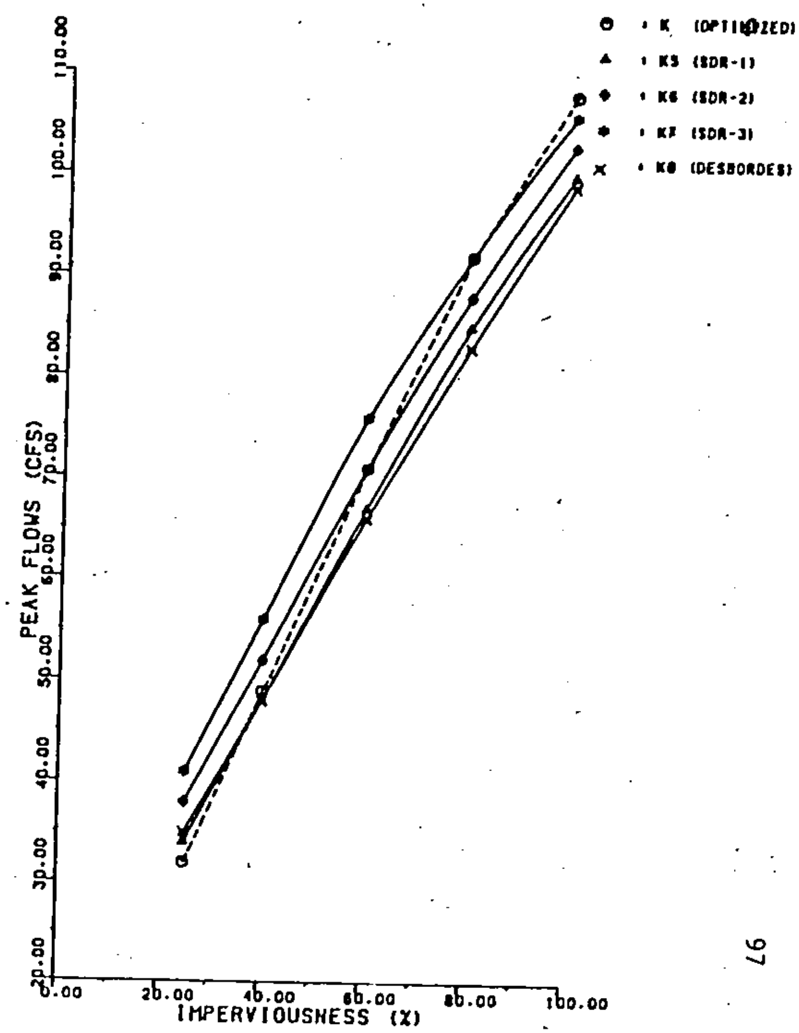


FIG. 4.10

COMPARISON OF PEAK FLOWS FOR SLR MODEL

25 AC. WATERSHED, 1:5 YEAR STORM

K5.K6.K7.K8



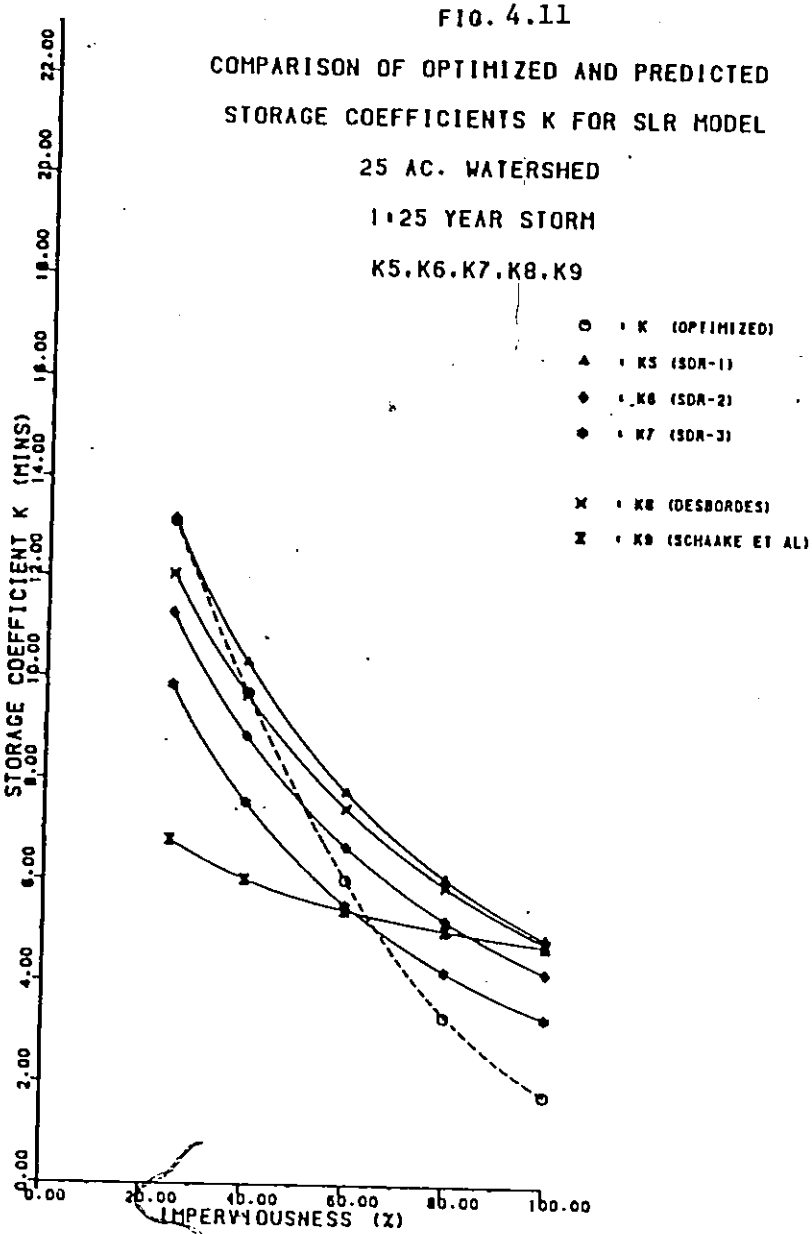


FIG. 4.11

COMPARISON OF OPTIMIZED AND PREDICTED STORAGE COEFFICIENTS K FOR SLR MODEL

25 AC. WATERSHED

1:25 YEAR STORM

K5, K6, K7, K8, K9

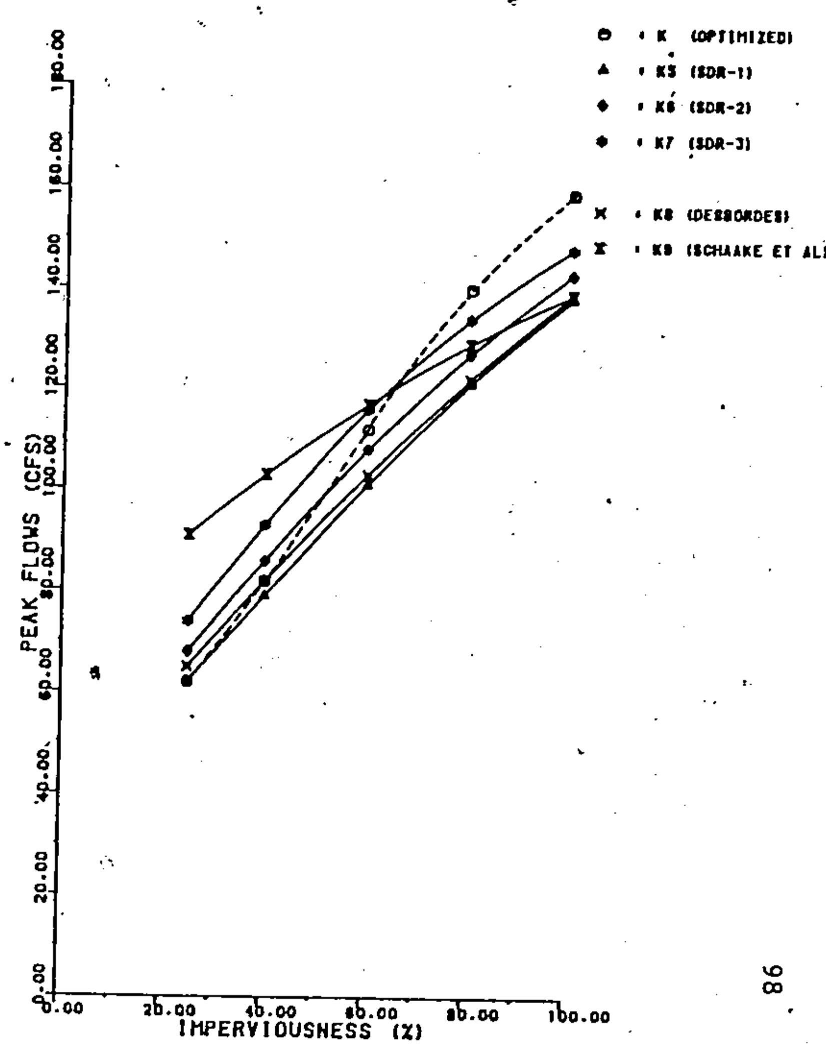


FIG. 4.12

COMPARISON OF PEAK FLOWS FOR SLR MODEL

25 AC. WATERSHED, 1:25 YEAR STORM

K5, K6, K7, K8, K9

In general, the peak flows are reasonably well predicted and further tests are done to test K8 for larger watersheds as described in the next section.

4.1.2.3 Schaake, Geyer and Knapp (1967)

$$K_9 = 0.0175\ L^{0.24} S^{-0.16} X_{imp}^{-0.26} \qquad 4.17$$

where L = length of main drainage channel (ft)

S = average slope of main drainage channel (%)

Ximp = directly connected imperviousness (%)

K9 is an equation for time lag which is derived by multiple regression analysis of the data for 19 gauged urban drainage areas. Schaake et al (1967) have stated that about two-thirds of the estimates of time lag by using K9 is expected to be between -22 % and +13 % of the true value of the time lag. K9 is not correlated with storm characteristics and therefore is the estimate for the average value of the time lag for a number of storms. The estimate may vary for individual storms on a watershed. The range of values for which the correlation is based is as follows:

imperviousness > 8 % ; 0.5 % < S < 6 %

150 ft < L < 6000 ft

The gutters draining to storm water inlets are paved and the house roofs are not directly-connected to the storm drain.

Two different values for slope are used in the computations. For the 1:2 year storm, the K9 values (fig.4.13) due to the two slopes are found to be close to the optimized values. The resulting peak flows are found to be very close to those computed for the detailed watershed by SWMM as seen in fig.4.14. The storage coefficients for the 1:5 year and 1:25 year storms are plotted in figs.4.15 and 4.11 with the corresponding peak flows in figs.4.16 and 4.12. For both the 1:5 year and 1:25 year storms the K9 values are not close to the optimized K values at all, except between 60 % and 70 % imperviousness. The resulting peak flows are significantly different from those computed for the detailed watershed.

K9 is not a function of the rainfall characteristics. Therefore it predicted the same values for all three storms. But the optimized K values change significantly from storm to storm. Therefore K9 cannot be extended to predict the storage coefficient for different storms.

4.1.2.4 Schulz and Lopez (1974)

$$K_{10} = \frac{0.0041837\ Ts^{0.31} Pe^{0.323} Hr^{0.80}}{Ur^{0.342}} \tag{4.18}$$

where Ts = duration of storm rainfall (hr.)

Pe = rainfall excess (in.)

Hr = hydrologic radius (area/perimeter)

Ur = 1 + Ximp (Ximp = ratio of directly-connected impervious area to the total area.)

Schulz and Lopez have derived unit hydrographs from events on 9 urban watersheds in the Denver Metropolitan Region which are from 0.10 to 1.0 sq. mile. The imperviousness range from 25 % to 100 %. The unit hydrograph parameters are then correlated with watershed and storm characteristics. The changes in the unit hydrograph are related to the decrease in watershed response time. They have concluded that the lag time t_L is the best way of defining the response time for the determination of peaks of the unit hydrograph. Since the lag time is theoretically equal to the storage coefficient of the single linear reservoir it is incorporated into the model for testing as K10. It can be seen that K10 is also a function of the watershed shape in terms of a parameter called the hydrologic radius which is defined as the ratio of area to perimeter of the watershed.

From figs.4.13 and 4.15 it can be seen that the computed K values do not follow the same trend as the optimized K values since they increase with increasing imperviousness. This is because K10 increases with increasing rainfall excess Pe, which is in contrast with other relations like K7 or K8 where K is inversely proportional to Pe. The predicted peak flows for the 1:2 year and the 1:5 year storms are shown in figs.4.14 and 4.16 respectively. It can be seen

FIG 4.13

COMPARISON OF OPTIMIZED AND PREDICTED

STORAGE COEFFICIENTS K FOR SLR MODEL

25 AC. WATERSHED

1:2 YEAR STORM

K9, K10

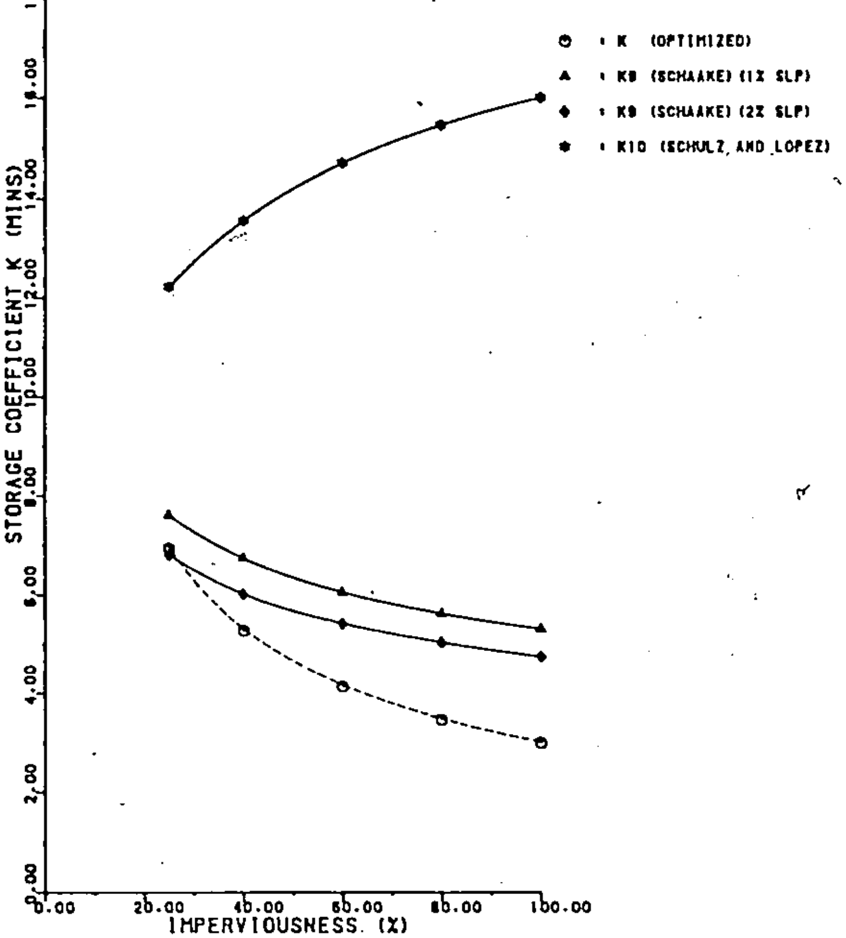


FIG. 4.14

COMPARISON OF PEAK FLOWS FOR SLR MODEL

25 AC. WATERSHED, 1:2 YEAR STORM

K9, K10

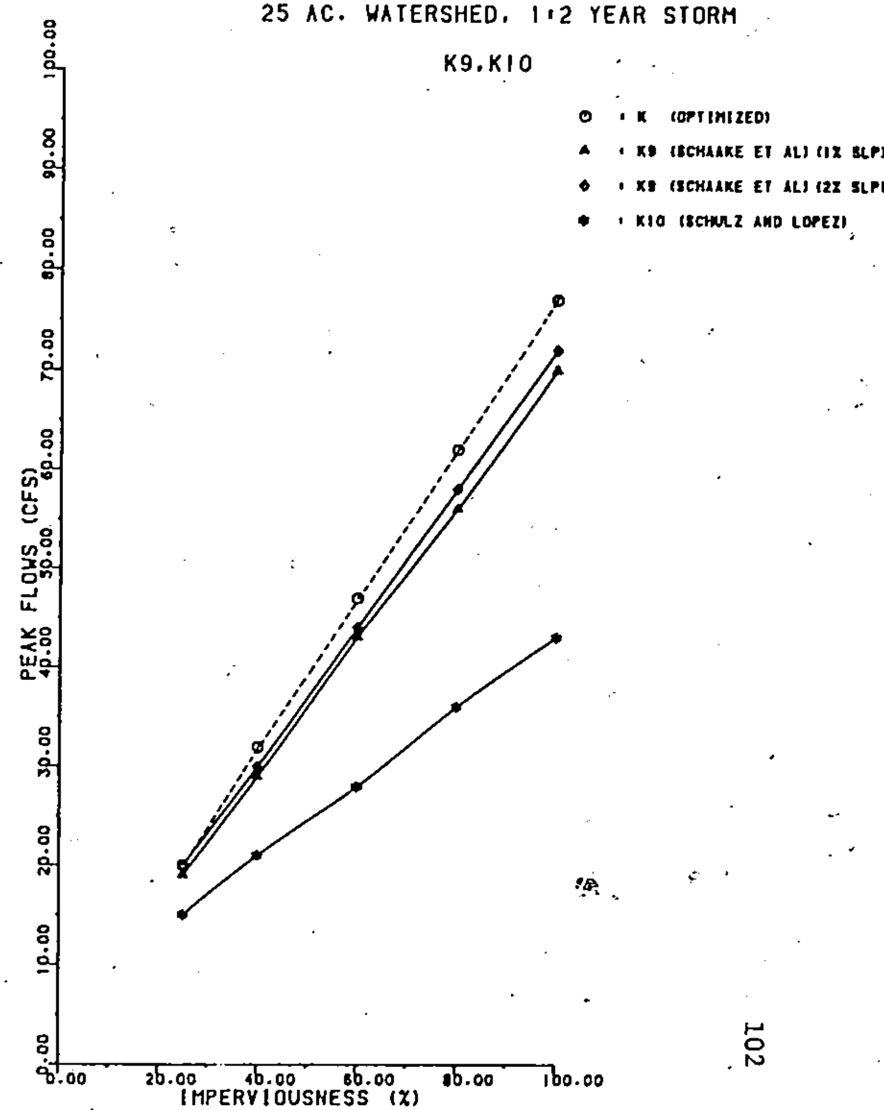


FIG. 4.15

COMPARISON OF OPTIMIZED AND PREDICTED STORAGE COEFFICIENTS K FOR SLR MODEL

25 AC. WATERSHED, 1:5 YEAR STORM

K9, K10

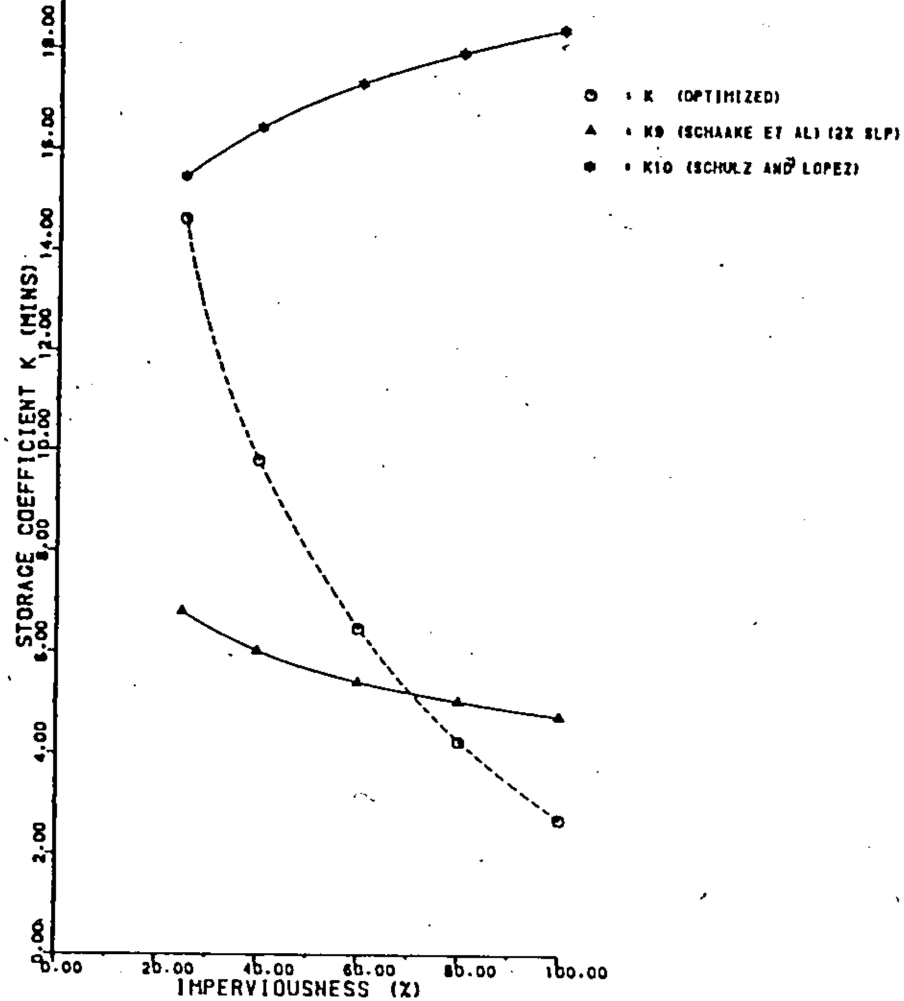
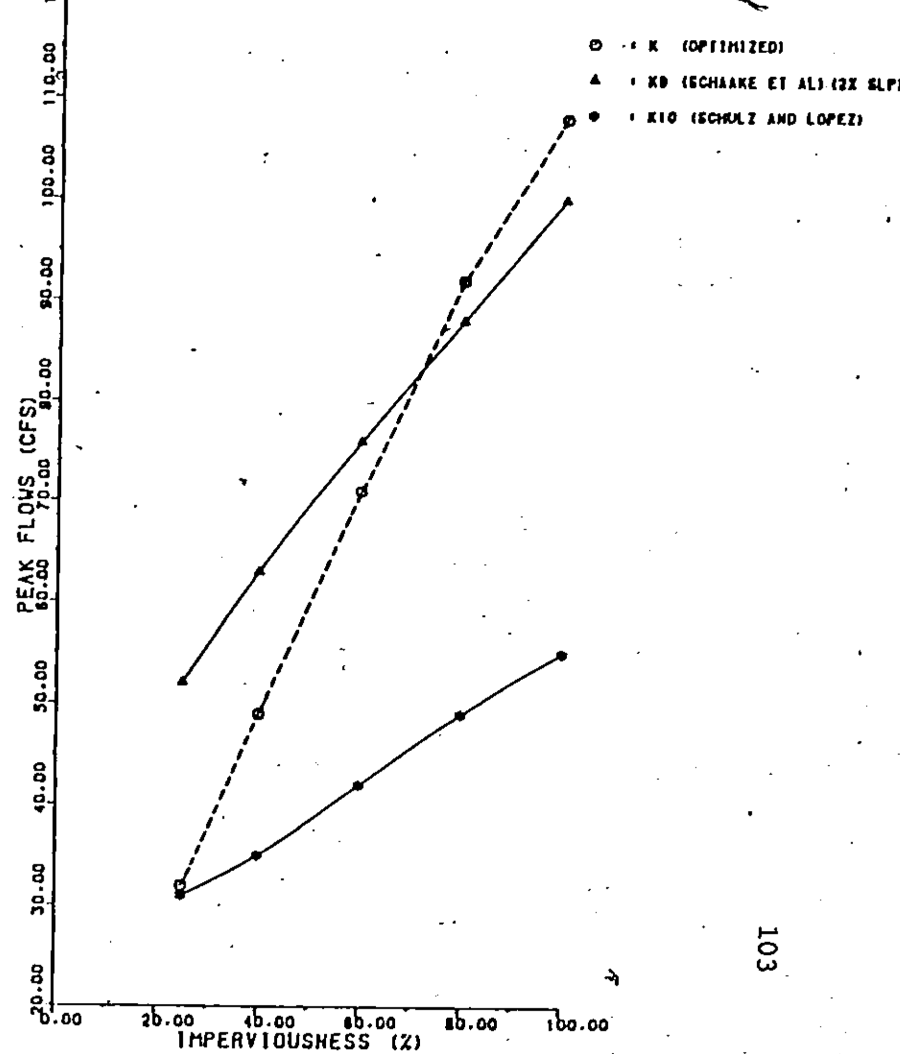


FIG. 4.16

COMPARISON OF PEAK FLOWS FOR SLR MODEL

25 AC. WATERSHED, 1:5 YEAR STORM

K9, K10



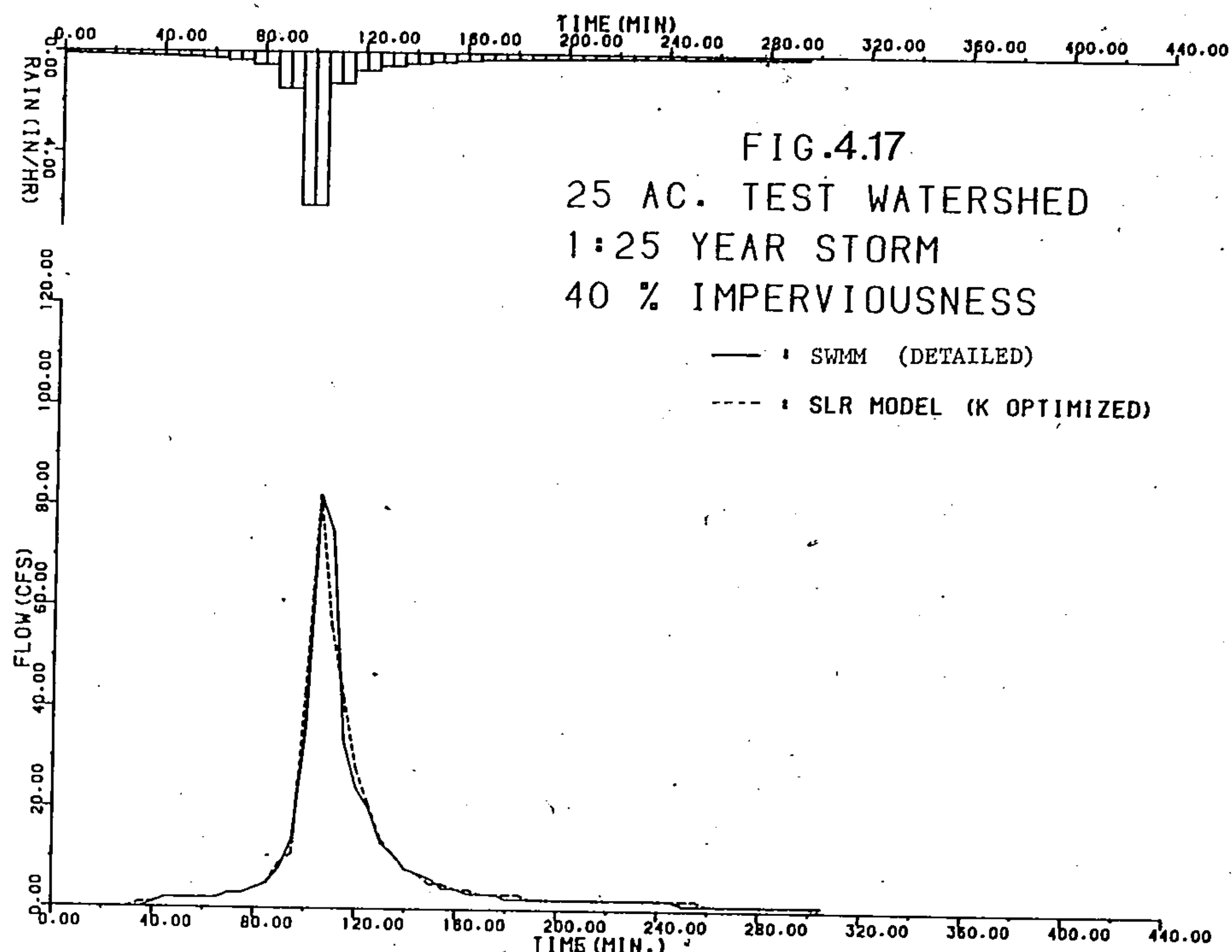


FIG.4.17
25 AC. TEST WATERSHED
1:25 YEAR STORM
40 % IMPERVIOUSNESS

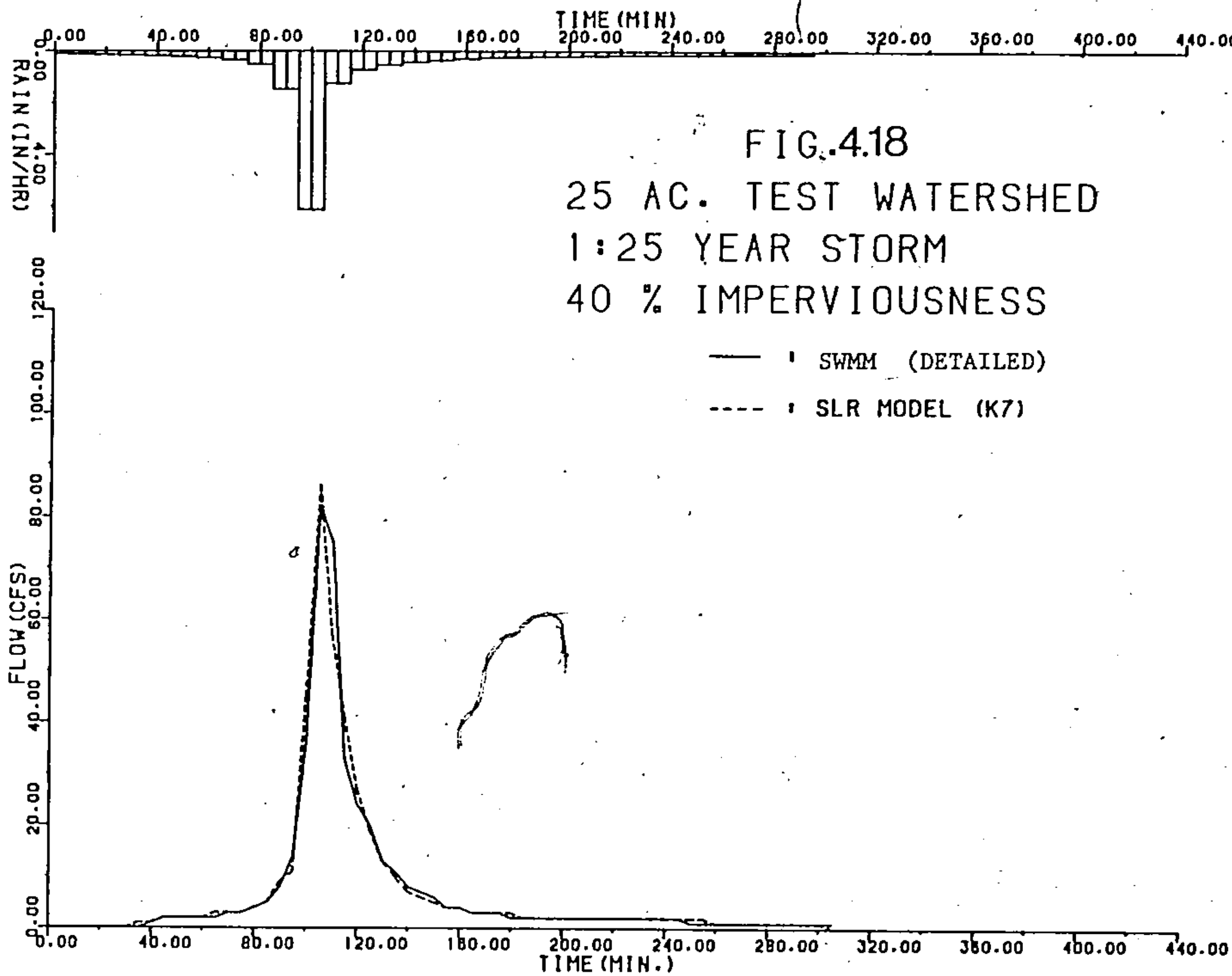


FIG.4.18
25 AC. TEST WATERSHED
1:25 YEAR STORM
40 % IMPERVIOUSNESS

that the predicted peak flows are significantly underpredicted, except at the 25 % imperviousness levels. The peak flow ratio is from 0.56 to 0.75 for the 1:2 year storm and from 0.51 to 0.96 for the 1:5 year storm. It can be concluded that the storage coefficient K of the SLR model cannot be adequately predicted by means of K10 for planning purposes.

4.1.3 Discussion of results for the 25-acre watersheds

In the previous section, various relations for computing the storage coefficient K of the single linear reservoir with storm and watershed characteristics have been analyzed. These various models, when coupled with a rainfall excess submodel are then compared as to their predictive performance for watersheds of 25 acres under planning conditions. Some of the relations for K are however applied for conditions beyond the data base from which they are derived. For example, K2, K3 and K4 are based on the kinematic wave theory in which a homogeneous planar surface is assumed. It is attempted to extend their use to non-homogeneous watersheds by varying the surface roughness parameter 'n' for watersheds at different imperviousness levels.

The optimized K values for the 1:2 year storm are very different from those for the 1:5 year or the 1:25 year storms for the same watersheds. Therefore for the SLR mod-

Table 4.2 THE RATIOS OF THE PEAK FLOWS OF THE SLR MODEL TO DETAILED SWMM SIMULATION FOR DIFFERENT RELATIONS FOR K (25-acre watersheds)

1:2 year storm

Imperviousness %	K1	K2	K3	K4	K5	K6	K7	K8	K9	K10
25	0.89	0.64	1.11	0.72	0.69	0.75	0.81	0.71	1.01	0.75
40	0.91	0.75	1.04	0.82	0.76	0.81	0.86	0.75	0.96	0.65
60	0.92	0.83	1.01	0.89	0.84	0.88	0.92	0.82	0.93	0.60
80	0.93	0.88	1.00	0.92	0.90	0.93	0.97	0.88	0.93	0.58
100	0.95	0.93	0.99	0.96	0.94	0.96	0.99	0.92	0.93	0.56

1:5 year storm

Imperviousness %	K1	K2	K3	K4	K5	K6	K7	K8	K9	K10
25	1.39	1.22	2.12	1.37	1.07	1.18	1.28	1.11	1.62	0.96
40	1.20	1.11	1.56	1.22	0.98	1.06	1.14	0.99	1.28	0.72
60	1.05	1.02	1.23	1.09	0.94	0.99	1.06	0.93	1.07	0.59
80	0.96	0.94	1.06	0.99	0.92	0.95	1.00	0.90	0.96	0.53
100	0.94	0.95	0.99	0.97	0.93	0.95	0.98	0.92	0.92	0.51

1:25 year storm

Imperviousness %	K1	K2	K3	K4	K5	K6	K7	K8	K9	K10
25	1.27	1.30	1.98	1.44	1.00	1.10	1.20	1.06	1.47	0.77
40	1.19	1.22	1.58	1.32	0.97	1.05	1.14	1.00	1.26	0.63
60	1.02	1.05	1.22	1.11	0.90	0.96	1.03	0.92	1.04	0.51
80	0.91	0.94	1.03	0.98	0.86	0.91	0.96	0.87	0.92	0.46
100	0.93	0.95	0.99	0.97	0.90	0.93	0.96	0.91	0.91	0.46

el, relations for the storage coefficient which are functions of just watershed but not storm characteristics, may not be adequate. For 100 % impervious watersheds, all the 4 relations in the first group gave relatively very good results since the differences in the rainfall excess distribution are minimized. The Viessman storage coefficient K1 which is a function of just watershed characteristics, predicts peak flows which are acceptable for the 1:2 year but not the 1:5 and 1:25 year storms. The IWW storage coefficient K3 which does not have surface roughness as a parameter changes with the peak rainfall excess intensity. The peak rainfall excess intensity is a function of the imperviousness level. K3 is found to overpredict the peak flows for the 1:5 year and 1:25 year storms. The differences in the peak flows are found to be about 100 % for the 25 % imperviousness level which is typical for residential developments. Therefore this shows that K3 cannot be extended for use on partially impervious watersheds. The Neumann (K2) and Pedersen et al (K4) relations, which both have rainfall excess intensity and surface roughness as parameters, perform better than the previous ones. However they also do not result in acceptable peak flows at all imperviousness levels for the three storms tested. For conditions corresponding to residential developments, they overpredict the peak flows by 30 - 40 % for the 1:5 year and the 1:25 year storms.

For the second group, the relations that predicted the K values closest to the optimized K values are those of Sarma, Delleur, Rao (SDR). K6 predicts peak flows that are in between those predicted by K5 and K7. For this relation, the range of discrepancies in the peak flows is from -25 % to +18 % for the three storms tested. The relation derived by Desbordes (K8) is also found to predict the peak flows reasonably accurately. The peak flow ratios range from 0.71 to 1.11.

From the results there is not one relation that has differences in peak flows of less than 20 % as compared with the detailed SWMM simulations for all 3 storms at different levels of imperviousness. All the relations in the first group are found to give significant errors. Some of the discrepancies are due to the contribution of rainfall excess from the pervious areas. It is felt that the results can be improved by considering the different response times of the impervious and pervious portions of the watershed.

4.2 LARGER WATERSHEDS

4.2.1 100-acre (40 ha) watersheds

Following the methodology outlined in Chapter III, the mathematical watersheds are built up to 300 acres in size and the outflow hydrographs are then obtained. Since the tests are for urban watersheds, a value of n = 0.013 is assumed for the channel roughness. The channel slope is taken to be 2 %.

Only the relations derived by Desbordes (K8) and Sarma et al (K5,K6,K7) are tested since they are derived from larger watersheds. These regression equations are functions of the area and the conclusions derived from the tests with the 25-acre watersheds are not necessarily applicable for these larger watersheds. The predicted peak flows for a first series of tests with 100-acre watersheds are shown in figs.4.19,4.20 and 4.21 for the 1:2 year, 1:5 year and 1:25 year storms respectively. The peak flow ratios are tabulated in table 4.3.

The best results are given by K7 for all three storms. The peak flows predicted for the 1:5 year and 1:25 year storms are considered acceptable. For the 1:5 year storm, the ratios of the predicted peak flows to those using detailed SWMM simulations from the mathematical watersheds are above 0.60 for all levels of imperviousness tested. For the 1:25 year storm, the peak flow ratios are above 0.80 for all imperviousness levels above 40 %. The results improve with increasing imperviousness because the errors due to the rainfall excess distribution will be reduced. For the 1:2 year storm, it underpredicts the peak flows at the lower imperviousness levels. A plot of the predicted runoff hydrograph from a 40 % impervious, 100-acre watershed is shown in

FIG. 4.19

COMPARISON OF PEAK FLOWS FOR SLR MODEL

100 AC. WATERSHED, 1:2 YEAR STORM

K5,K6,K7,K8

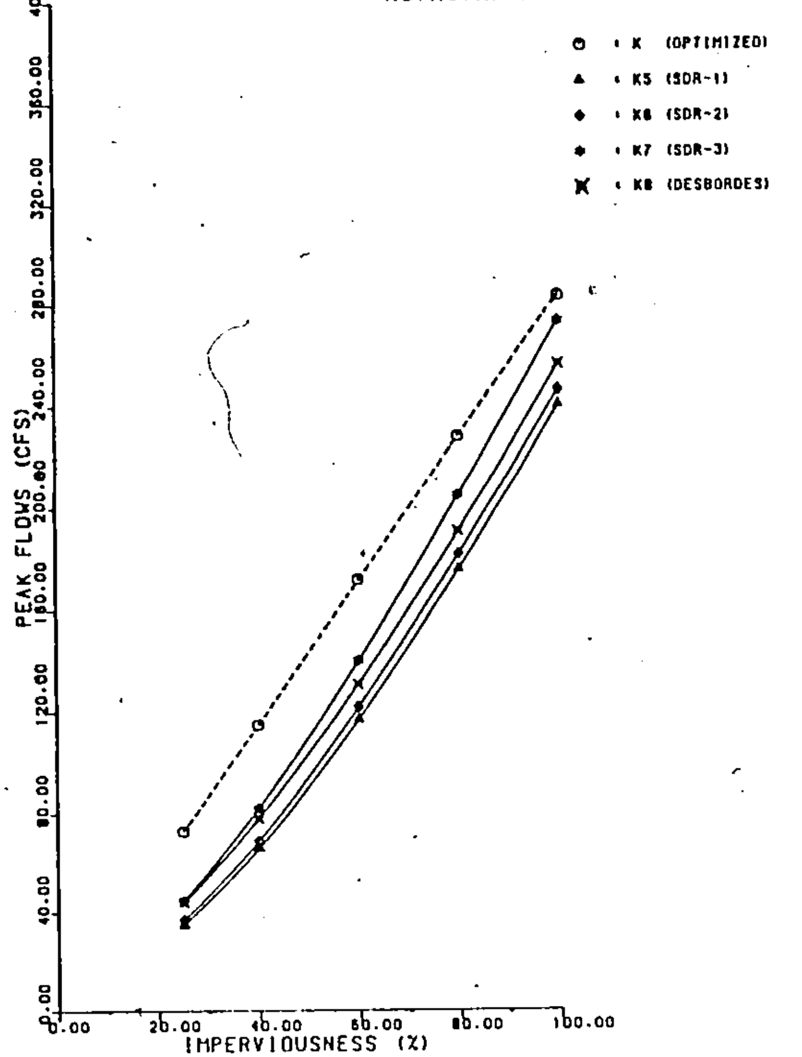


FIG. 4.20

COMPARISON OF PEAK FLOWS FOR SLR MODEL

100 AC. WATERSHED, 1:5 YEAR STORM

K5,K6,K7,K8

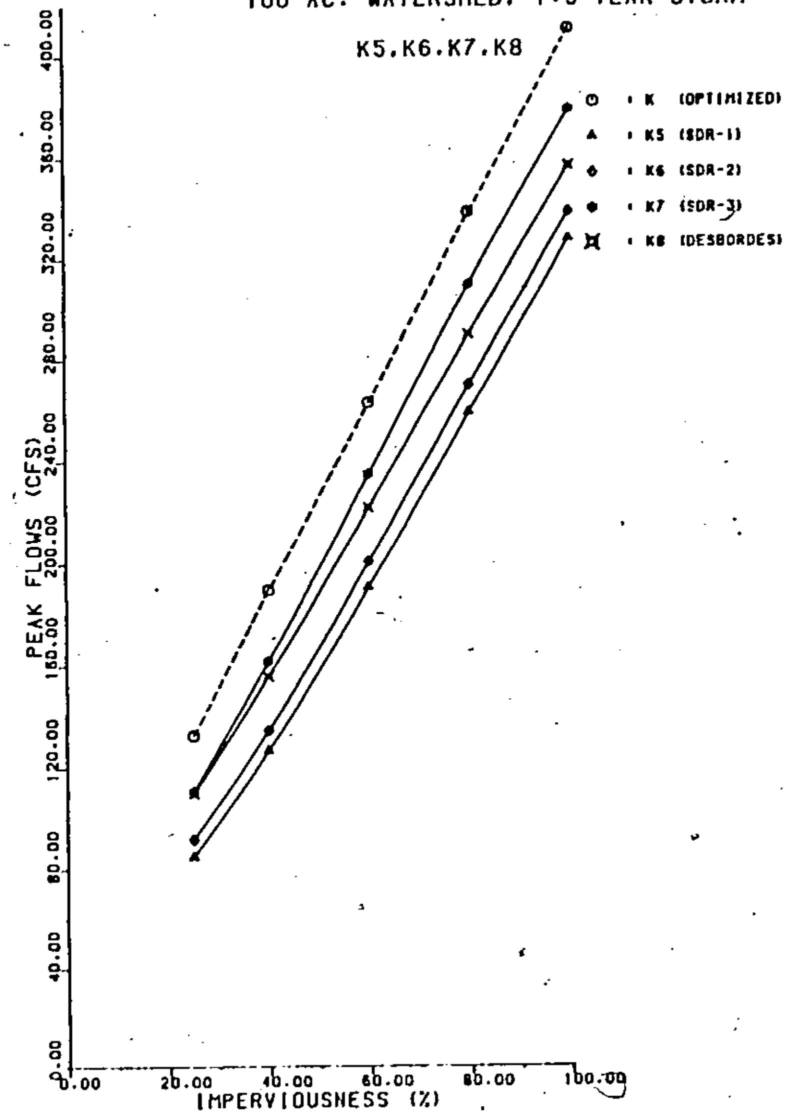


FIG. 4.21

COMPARISON OF PEAK FLOWS FOR SLR MODEL

100 AC. WATERSHED, 1:25 YEAR STORM

K5, K6, K7, K8

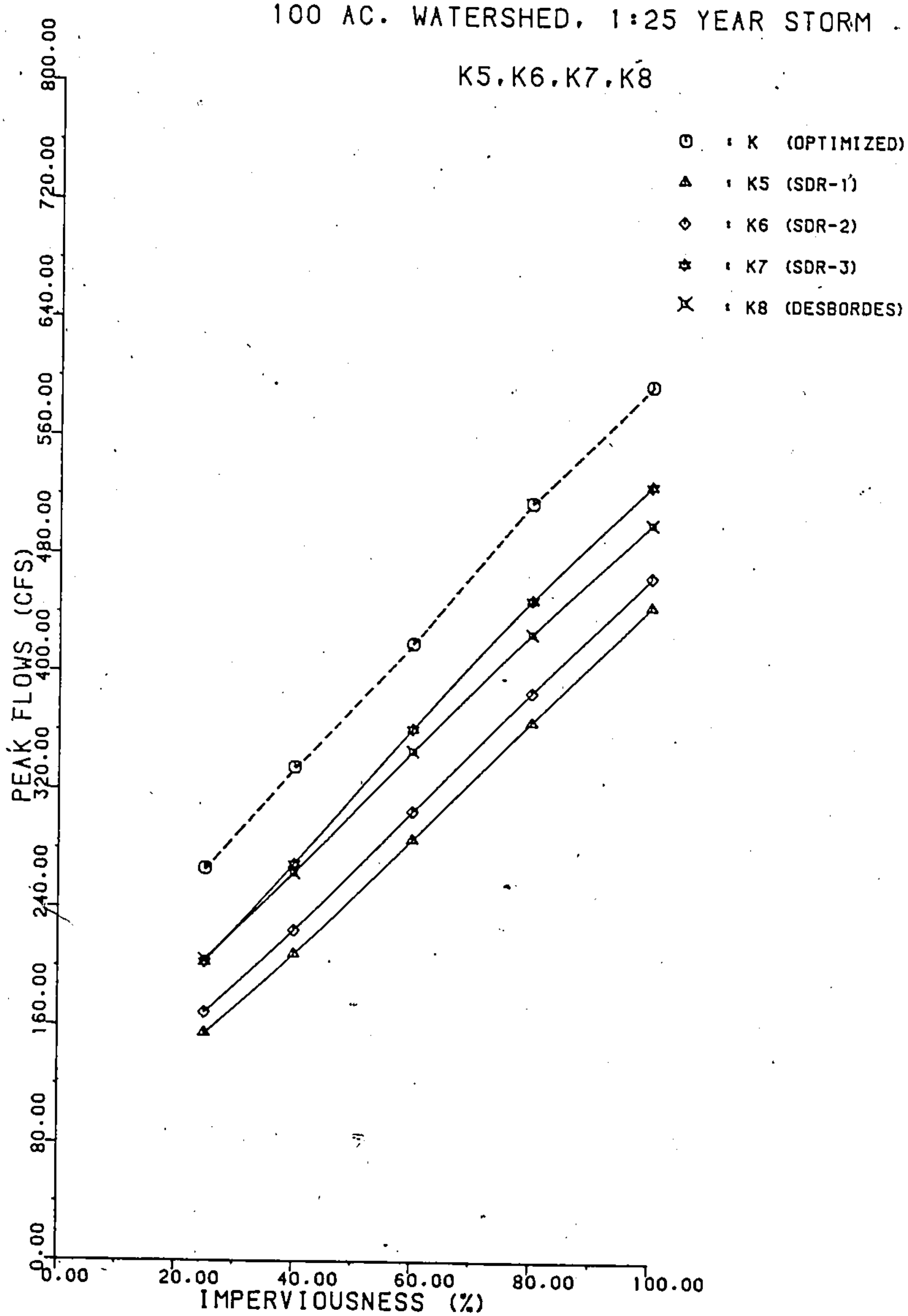


fig.4.22. K5 and K6 predicts lower peak flows than K7 for the three storms tested at all levels of imperviousness and, should not be used in conjunction with the SLR model. The peak flows predicted by using K8 are also considered acceptable. They are very close to those predicted by K7, the biggest discrepancy between them being only 7 %.

4.2.2 300-acre (121 ha) watersheds

Only K7 is used in obtaining the computed outflow hydrographs for 300-acre watersheds since it predicted the best results for the 100-acre watersheds. Table 4.4 shows the peak flow ratios for the three storm inputs. The results show that the peak flows are badly underpredicted for all three storms. The peak flows are plotted in figs.5.17 - 5.19 (in the next chapter). Therefore the SLR model using K7 is not recommended for urban watersheds larger than 100 acres.

4.3 GENERAL CONCLUSIONS

The conclusions from this chapter can be summarized as follows:

1. The second group of relations derived from multiple regression analysis provide better estimates of the storage coefficient for the single linear reservoir model. The first group of relations based on the

kinematic wave theory cannot be directly applied for the SLR model for watersheds with low imperviousness levels.

2. There is not one formula that can give peak flows that have differences of less than 20% as compared to a detailed simulation with SWMM for all three storms at the different levels of imperviousness tested. However the K7 and K8 relations are found to provide reasonable estimates of the peak flows for watersheds up to 100 acres in size for the two larger storms. For these two storms, the peak flows predicted by these two relations are within 25 % less than those obtained by detailed SWMM simulation. For the 1:2 year storm, the peak flows are significantly under-predicted at the 25 - 40 % imperviousness levels, with the differences ranging from 29 % to 39 %. It is observed from tables 4.2 - 4.4 that the peak flow ratios for the SDR relations decrease as the area of the watersheds increase in size.
3. A single linear reservoir model using K7 has been programmed on a calculator for use in runoff control studies for watersheds of less than 100 acres (40 ha) in size.

Table 4.3 THE RATIOS OF THE PEAK FLOWS OF THE SLR MODEL TO DETAILED SWMM SIMULATION FOR DIFFERENT RELATIONS FOR K (100-acre watersheds)

Imperviousness %	K5	K6	K7	K8
		1:2 year storm		
25	0.48	0.51	0.62	0.61
40	0.57	0.60	0.71	0.68
60	0.68	0.71	0.81	0.76
80	0.77	0.80	0.90	0.84
100	0.85	0.87	0.97	0.90
		1:5 year storm		
25	0.64	0.69	0.83	0.83
40	0.67	0.71	0.85	0.82
60	0.73	0.76	0.90	0.84
80	0.76	0.80	0.91	0.86
100	0.80	0.82	0.92	0.87
		1:25 year storm		
25	0.58	0.63	0.76	0.77
40	0.62	0.67	0.80	0.79
60	0.68	0.73	0.86	0.83
80	0.71	0.75	0.87	0.83
100	0.75	0.78	0.89	0.84

Table 4.4 THE RATIOS OF THE PEAK FLOWS OF THE SLR MODEL USING K7 TO DETAILED SWMM SIMULATION (300-acre watersheds)

Imperviousness %	1:2 year storm	1:5 year storm	1:25 year storm
25	0.43	0.60	0.57
40	0.51	0.61	0.58
60	0.61	0.65	0.63
80	0.70	0.71	0.68
100	0.78	0.77	0.76

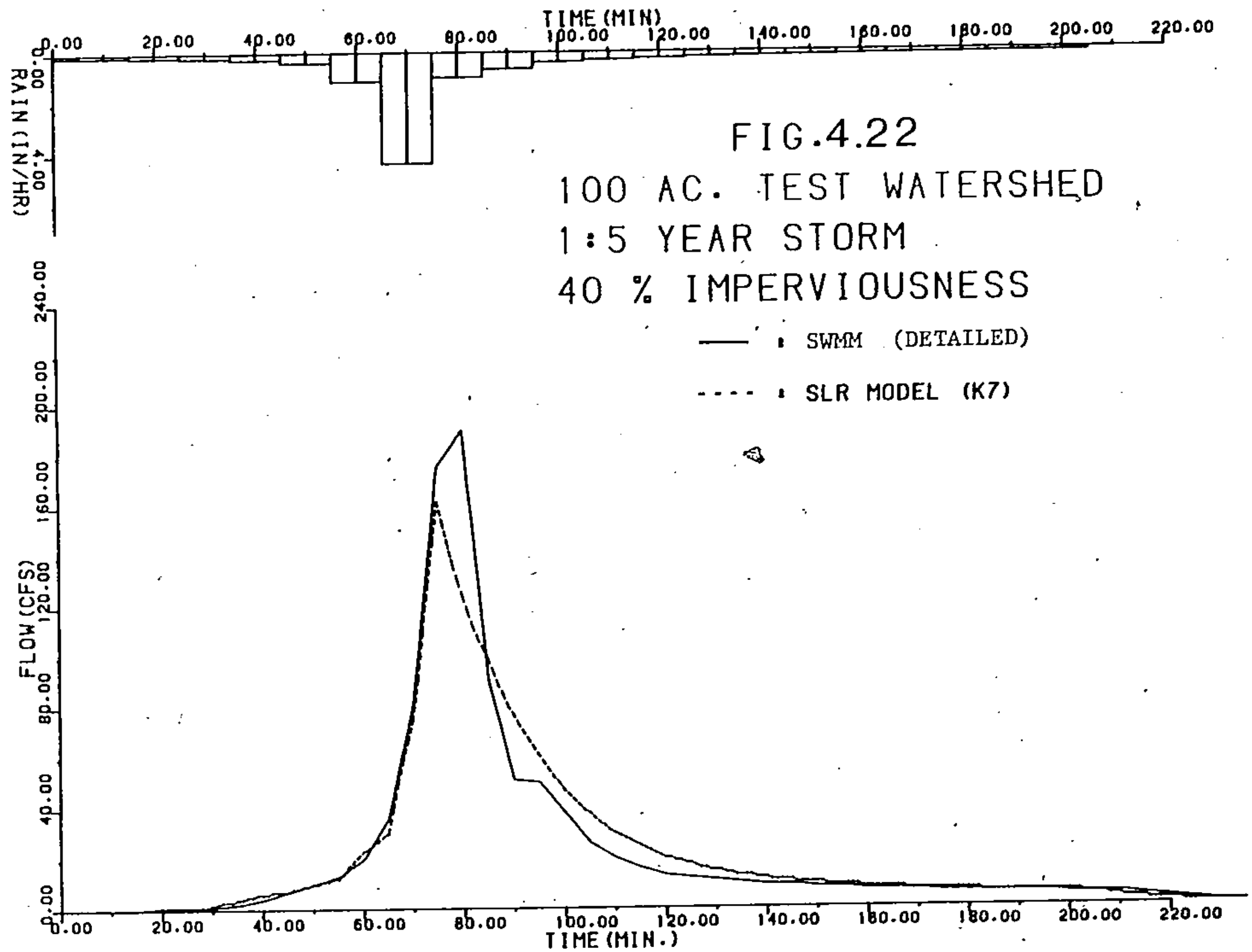


FIG.4.22
100 AC. TEST WATERSHED
1:5 YEAR STORM
40 % IMPERVIOUSNESS

Chapter V

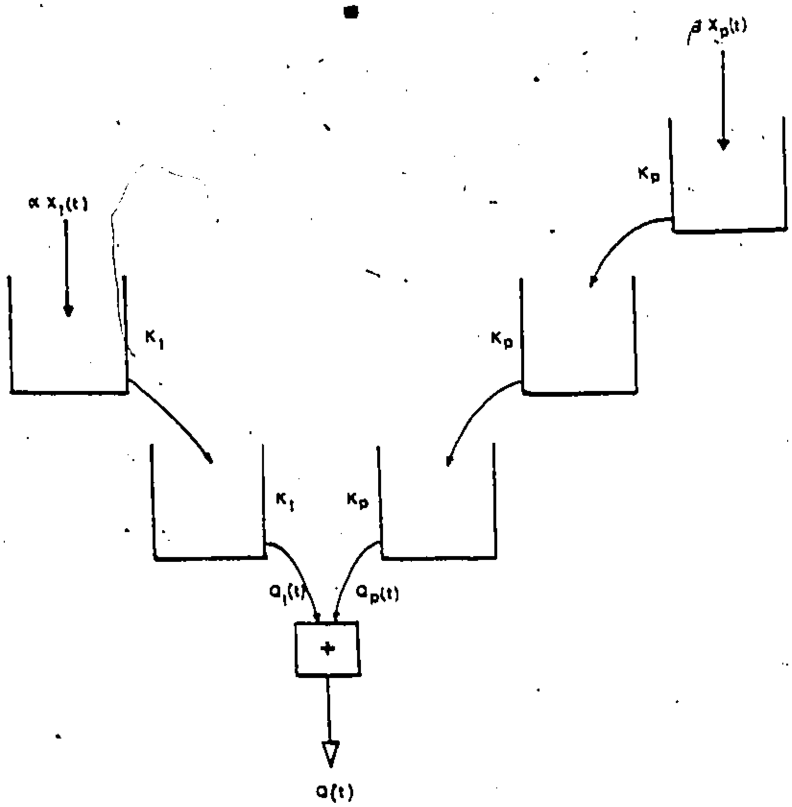
THE 2 PARALLEL LINEAR RESERVOIRS MODEL (2PLR)

5.1 INTRODUCTION

The relations for predicting the storage coefficient of the SLR model are found to be adequate for some storms but not the entire range of small as well as large storms tested. Some relations which can predict acceptable peak flows for the 1:5 year and the 1:25 year storms are found to overpredict for the smaller 1:2 year storm. This may be due to the method used in obtaining the rainfall excess distribution by adding the contributions from the impervious and pervious areas for each time increment. To improve the results, it seems that the separate simulation of the hydrographs from both the impervious and pervious portions of the watershed would be a more logical approach as it can account for the different response times from these two portions of the watershed.

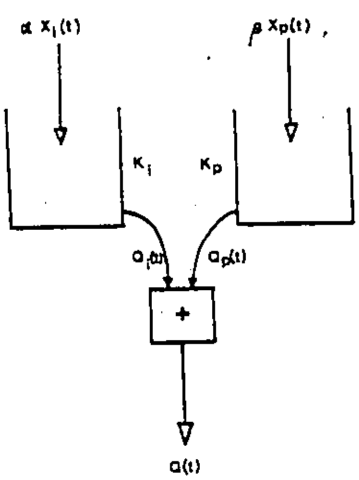
In both the SWMM-RUNOFF and ILLUDAS models the outflow hydrograph is obtained by adding the hydrographs from the impervious and pervious areas. For lumped models, it has been mentioned in the literature review in chapter II that the parallel cascades model (fig.5.1) has been used by some investigators (Wittenberg 1975, Diskin et al 1978). It is proposed here to develop a model that is made up of 2 linear reservoirs in parallel (fig.5.2). The reservoirs would transform the input to the impervious and pervious areas into the corresponding hydrographs from each area. These hydrographs would then be added together to form the outflow hydrograph from the entire watershed.

The 2 parallel linear reservoirs model is a special case of the parallel cascades model in which $N_i = N_p = 1$. The main advantage of the 2 parallel linear reservoirs model is that it eliminates 2 parameters which are the number of reservoirs (N_i, N_p) in each cascade. To use it as a planning model, the storage coefficients would have to be related with watershed characteristics. The first group of relations (K1,K2,K3,K4) are used to estimate the storage coefficients of the 2 linear reservoirs. K1,K2 and K4 are functions of the surface roughness 'n'. Therefore the different response times of the impervious and pervious portions can be reflected in different 'n' values. The storage coefficient of the linear reservoir for the impervious area is represented by Ki while that for the pervious area is represented by Kp.



PARALLEL CASCADES MODEL

FIG.5.1



2 PARALLEL LINEAR RESERVOIRS MODEL

FIG.5.2

5.1.1 25-acre (10 ha) test watersheds

In the first series of experiments, Ki is estimated by K1 (Viessman) while Kp is estimated by each of the four relations mentioned above. In the second series of experiments, the storage coefficient proposed by IWW, K3 is used to estimate the impervious area storage coefficient, Ki. This results in eight possible combinations of estimates for the storage coefficients. The experiments are conducted for 25-acre watersheds with different imperviousness ratios. The Chicago design storms for three return periods, 1:2 year, 1:5 year and 1:25 year are used as input. The overland flow length is taken to be 850 feet. The slope is taken to be the slope of the overland flow plane. The length and slope parameters are assumed to be the same for both impervious and pervious areas. The surface roughness is taken to be n = 0.013 for the impervious area and n = 0.25 for the pervious area. The input rate i for the impervious area is taken to be the same as the rainfall intensity since there are no infiltration losses. For the pervious area the rainfall excess intensity is obtained after subtracting the infiltration losses and averaging over the most intense 10 minutes. The computed storage coefficients are independent of the imperviousness. For the same storm, they would remain constant for different imperviousness ratios.

It can be seen in fig.5.3 that the 2PLR model predicts very good peak flows for the 1:2 year storm with the combinations using Ki = K3 giving slightly better results than those using Ki = K1. Although four different relations are used to estimate Kp, the resulting peak flows are very close together. Therefore not all the results are plotted in the figure. The storage coefficients estimated by these four relations are significantly different, though, from each other and from the optimized Kp values. However the peak flows are not sensitive to the Kp because the contribution of the rainfall excess from the pervious area is small for the 1:2 year storm.

For the 1:5 year storm, the results shown in fig.5.4 again demonstrates that the 2PLR model can predict accurate peak flows. When Kp is estimated by K3 however, the peak flows are overpredicted except for watersheds with imperviousness ratios higher than 0.80. Again the combinations using Ki = K3 estimate slightly better peak flows than those using Ki = K1. The results for the 1:25 year storm are shown in fig.5.5 and support the same conclusions as reached for the 1:5 year storm. Due to the higher contribution of rainfall excess from the pervious area the peak flows are found to be more sensitive to the Kp values.

The overall results indicate that for the 3 storms tested, the best combination of relations out of the 8 combina-

FIG. 5.3

COMPARISON OF PEAK FLOWS FOR 2PLR MODEL

25 AC. WATERSHED, 1:2 YEAR STORM

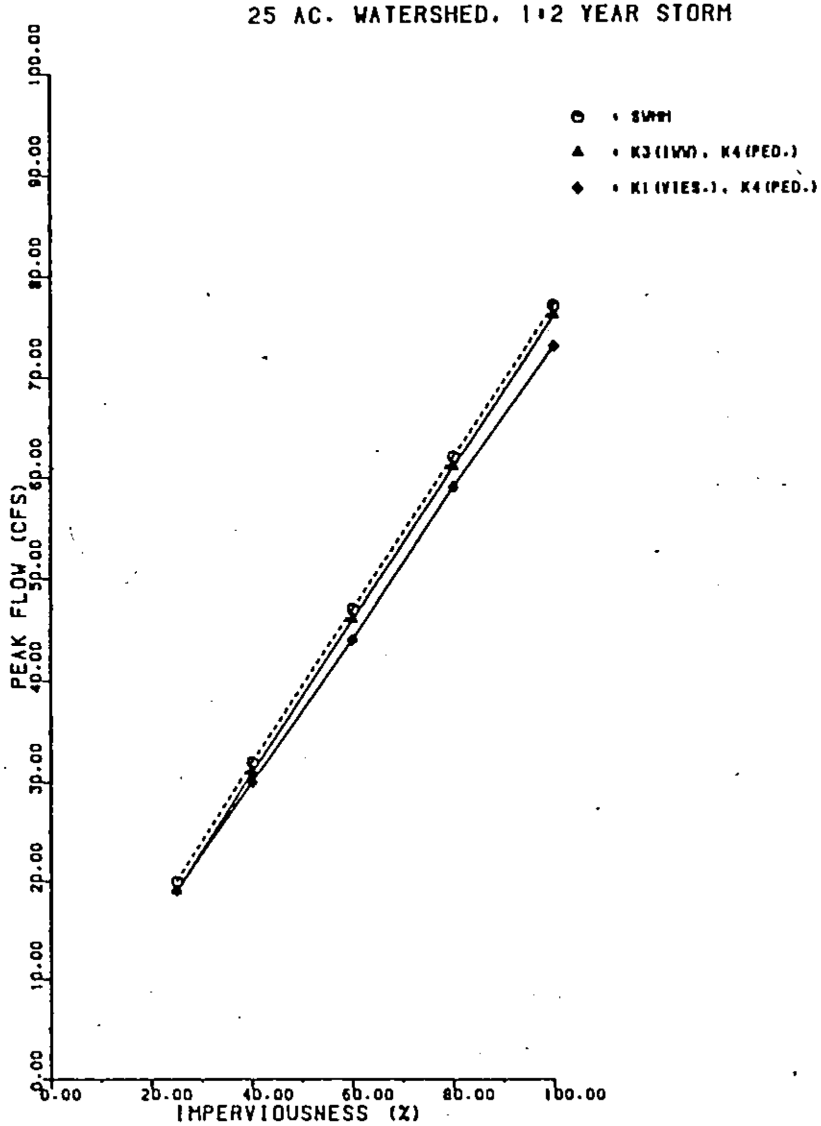


FIG. 5.4

COMPARISON OF PEAK FLOWS FOR 2PLR MODEL

25 AC. WATERSHED, 1:5 YEAR STORM

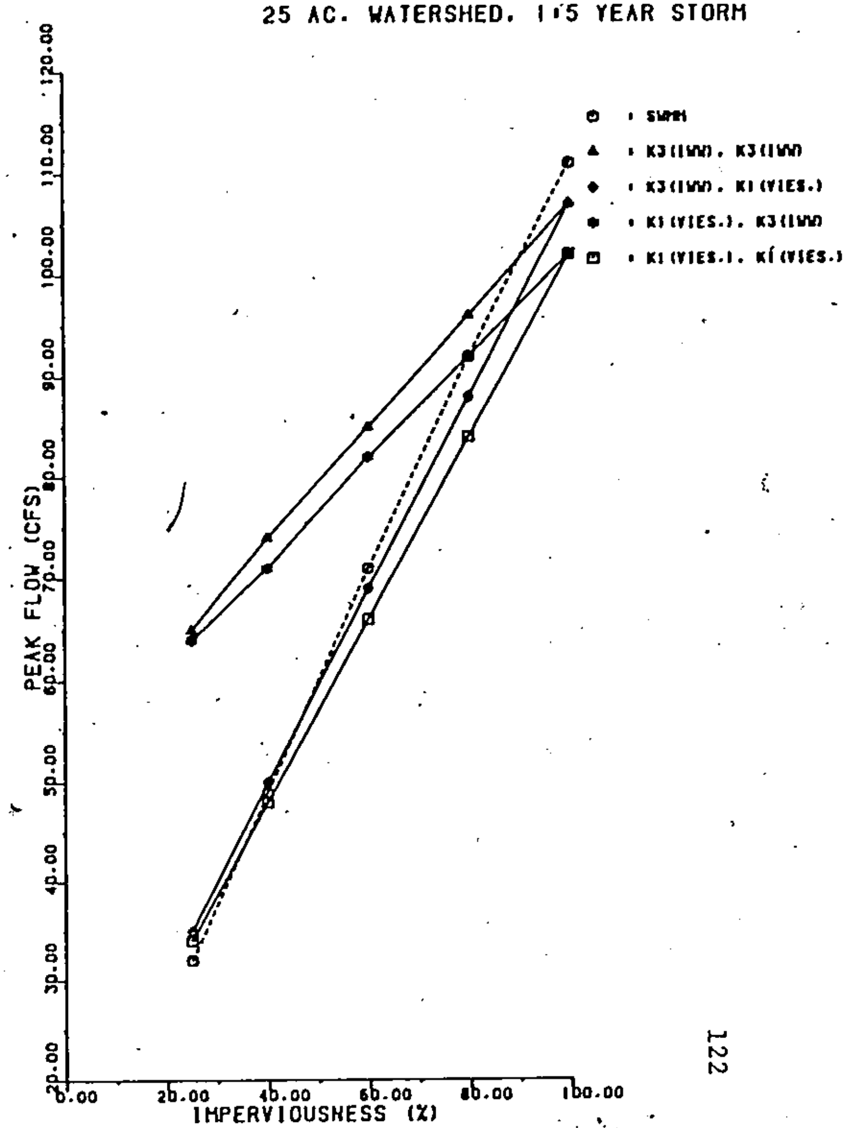
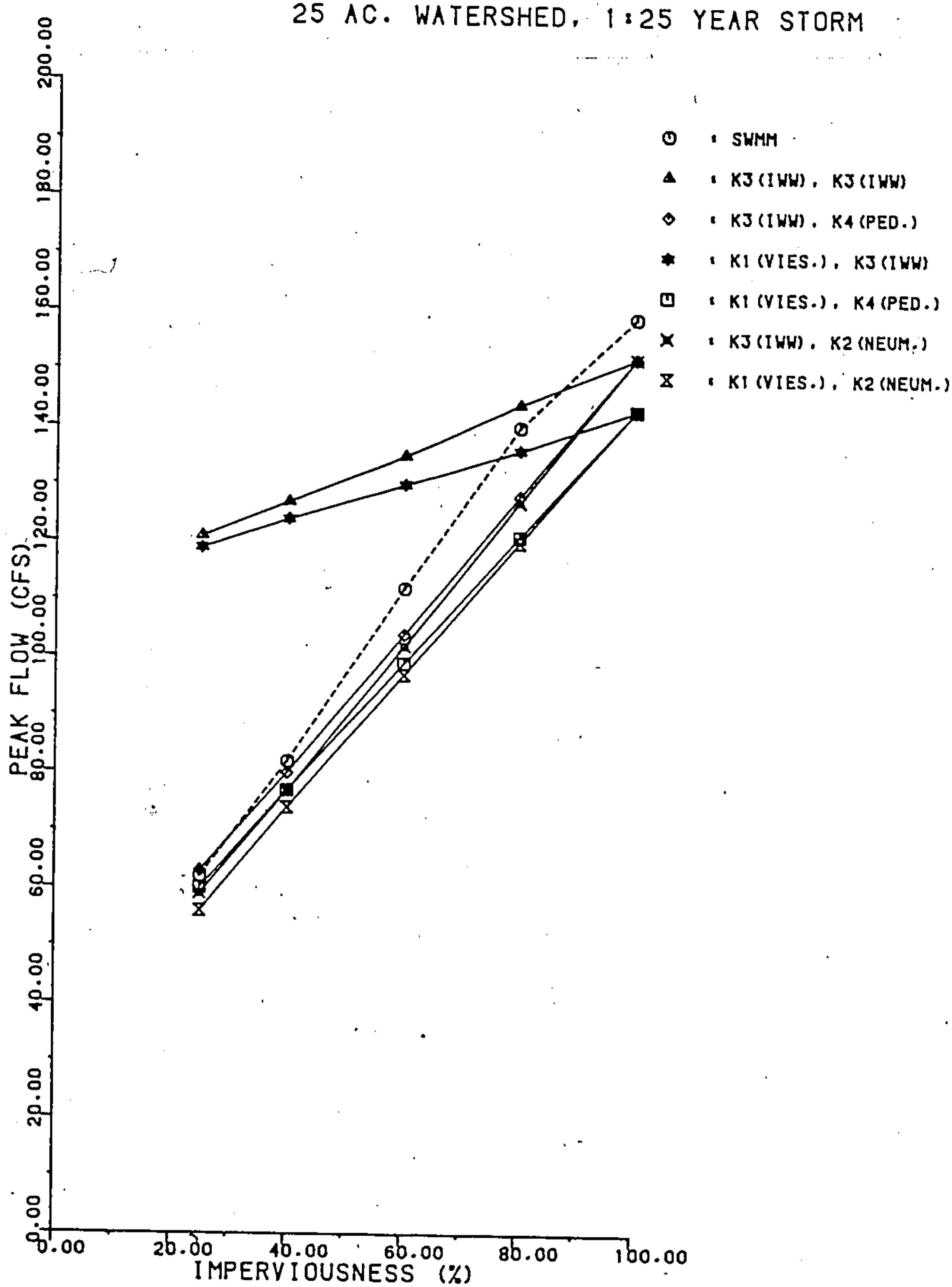


FIG.5.5

COMPARISON OF PEAK FLOWS FOR 2PLR MODEL

25 AC. WATERSHED, 1:25 YEAR STORM



tions tested is K3 for Ki and K4 (Pedersen et al,1980) for Kp. Two of the hydrographs simulated by the 2PLR model are compared with the hydrographs simulated in detail by SWMM-RUNOFF in figs.5.6 and 5.7.

The 2PLR model using the relations (K3,K4) is then tested on another urban watershed, test area B which is a residential development of 52.1 acres (fig. 3.3). Detailed simulation by SWMM is also done on the watershed. The hydrographs simulated by the 2 models for the 1:5 year storm are shown in fig.5.8. The 2PLR model is applied on a lumped basis. The peak flow is very accurately predicted.

5.2 LARGER WATERSHEDS

The 2PLR model is also tested on a lumped basis for the 100-acre (40 ha) and 300-acre (121 ha) test watersheds to see whether it can be applied to larger watersheds. The outflow hydrographs from these test watersheds are simulated in detail by means of SWMM-RUNOFF. The testing procedure is similar to the one applied in testing the model for the 25-acre watersheds. Most of the rainfall excess in urban areas come from the impervious areas. Therefore the peak flows are more sensitive to changes in Ki. This time four different relations are used to estimate Ki.

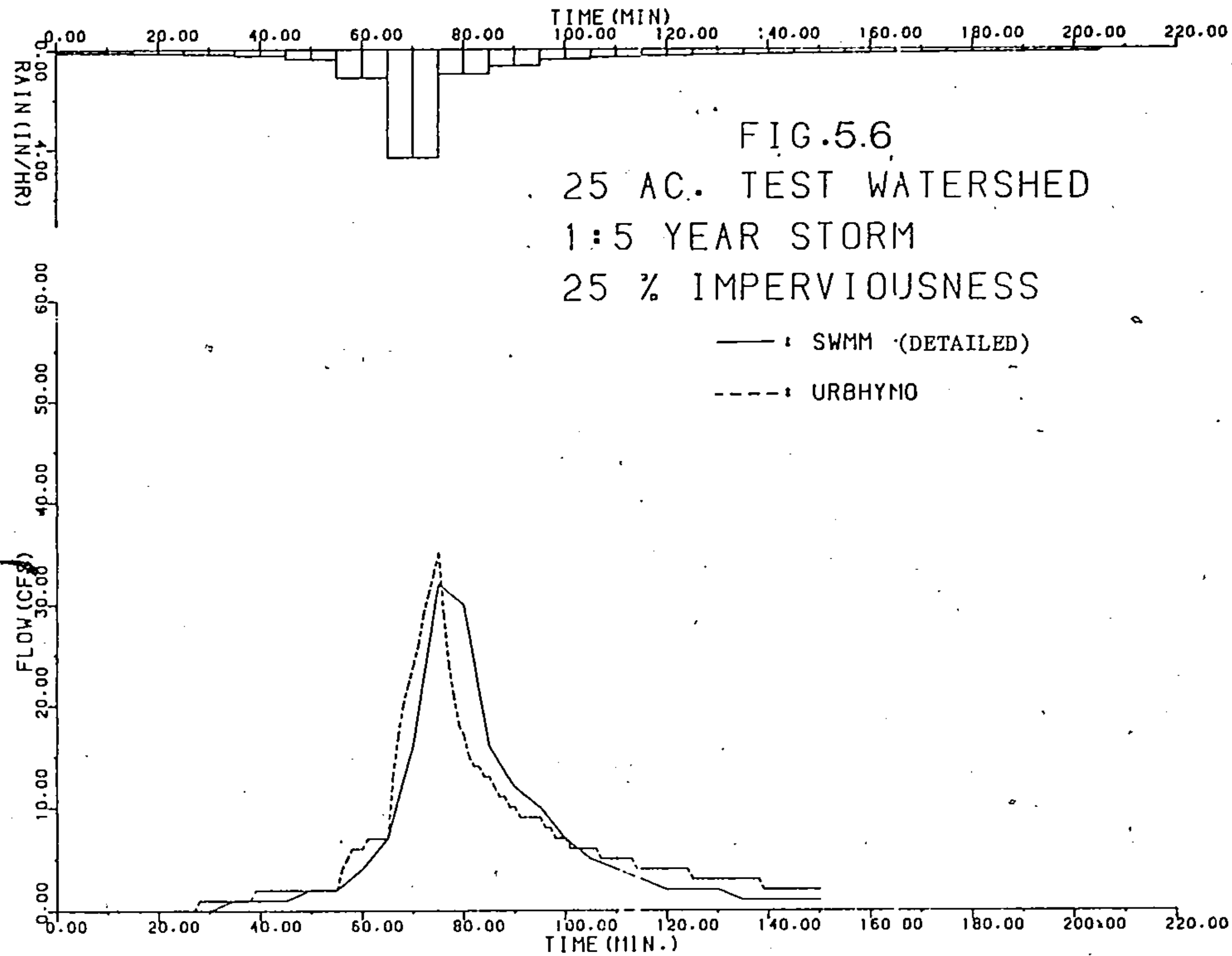


FIG.5.6
25 AC. TEST WATERSHED
1:5 YEAR STORM
25 % IMPERVIOUSNESS

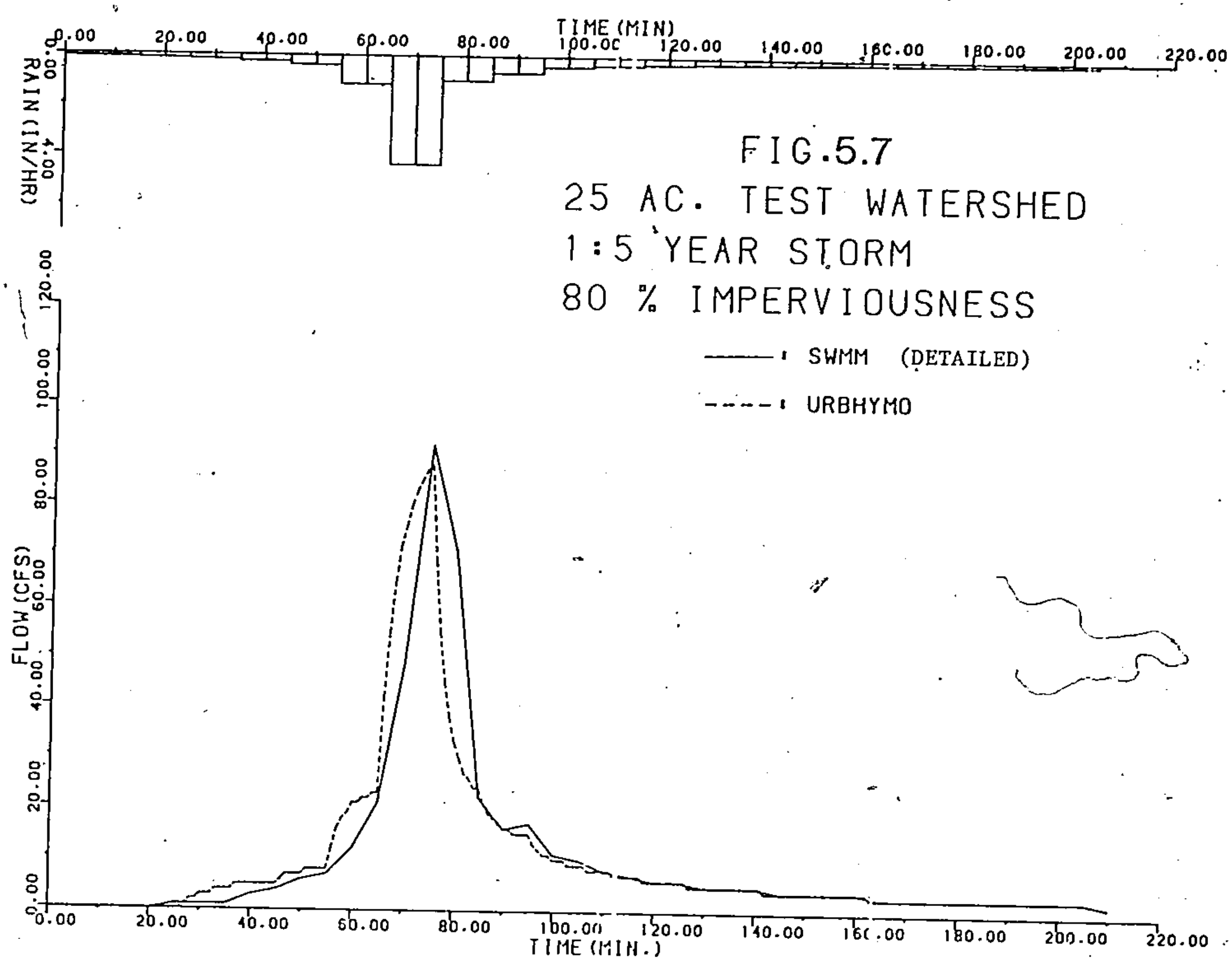


FIG.5.7
25 AC. TEST WATERSHED
1:5 YEAR STORM
80 % IMPERVIOUSNESS

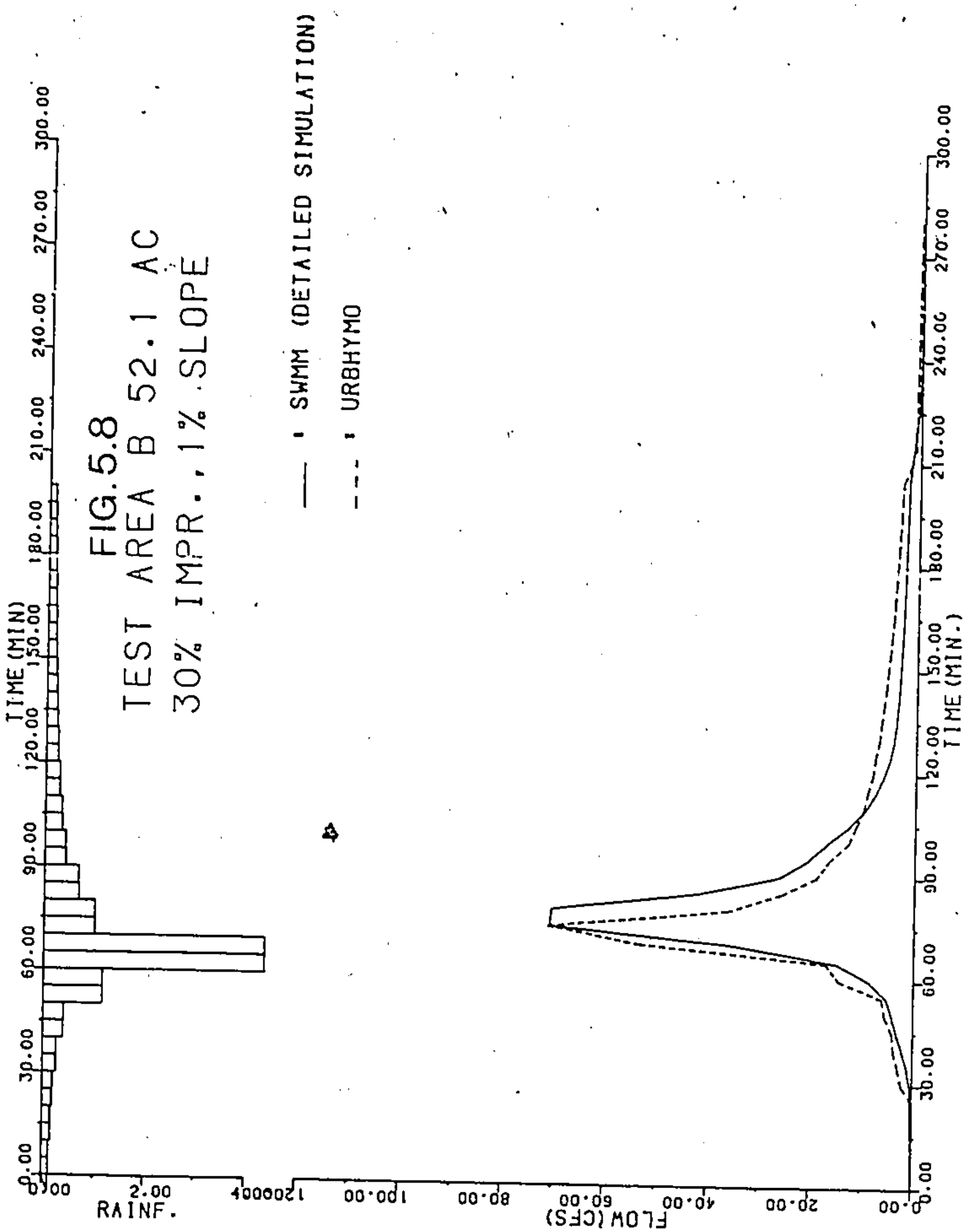


FIG.5.8
TEST AREA B 52.1 AC
30% IMPR., 1% SLOPE

5.2.1 100-acre (40 ha) watersheds

For the 1:2 year storm as shown in fig.5.9, it is observed that all the four combinations produced good peak flows, being within about 10 % of those from the detailed SWMM simulations. The best results are given by the relations of Ki = K3 and K_p = K4.

For the 1:5 year storm, in the first series of tests Ki is estimated by four different relations with Kp being estimated by K4. The results are presented in fig.5.10. It is observed that the best results are obtained when Ki = K3, with the next closest results being produced by Ki = K4. In the second series of tests therefore, Ki is kept constant and estimated by K3. Kp is estimated by K1,K2 and K4. It is found that the computed values of Kp do not vary by much, ranging from 47.3 to 58.7 minutes. The resulting peak flows are very close to each other.

The input rate parameter i used in the computation of the storage coefficients has been taken to be the rainfall excess intensity during the most intense 10 minutes since the rainfall maintains the peak intensity for a duration of 10 minutes. This assumption is based on the premise that the peak discharge in an urban watershed is often caused by a high intensity,short duration,burst of rainfall. This type of storm can also be characterized by the Chicago design storm which is used as the input for these tests. For the impervious area, the rainfall excess intensity is the same as the rainfall intensity while for the pervious portion of the watershed, the rainfall excess intensity is dependent on the infiltration parameters. Therefore i for the pervious area changes for every time step and depends on the duration over which it is averaged. It is attempted to find out the effect of determining i by the maximum Δt-minute rainfall excess intensity in which Δt is the time increment used in the computations. Fig.5.11 shows the results of two different options of determining the intensity i parameter used in the calculations. Option 1 is for the case when i is the peak rainfall excess intensity over 10 minutes. Option 2 is when i is determined by the maximum Δt-minute rainfall excess intensity in which Δt = 5 minutes. The results for the 1:5 year storm does not demonstrate any appreciable difference in the peak flows for the two options.

The results for the 1:25 year storm are shown in figs.5.12 - 5.14. Fig.5.12 show the peak flows when 4 different relations are used to estimate Ki and Kp is estimated by K4. The best results are given by Ki = K3. For a 25 % impervious watershed, the peak flow ratio is 0.83. For Ki = K4, the peak flow ratio is 0.79. Fig.5.13 show the results when 4 different relations are used to estimate Kp and Ki is estimated by K3. It is observed that the use of K3 for estimating Kp will result in overpredicted peak flows. The

FIG. 5.9

COMPARISON OF PEAK FLOWS FOR 2PLR MODEL

100 AC. WATERSHED, 1:2 YEAR STORM

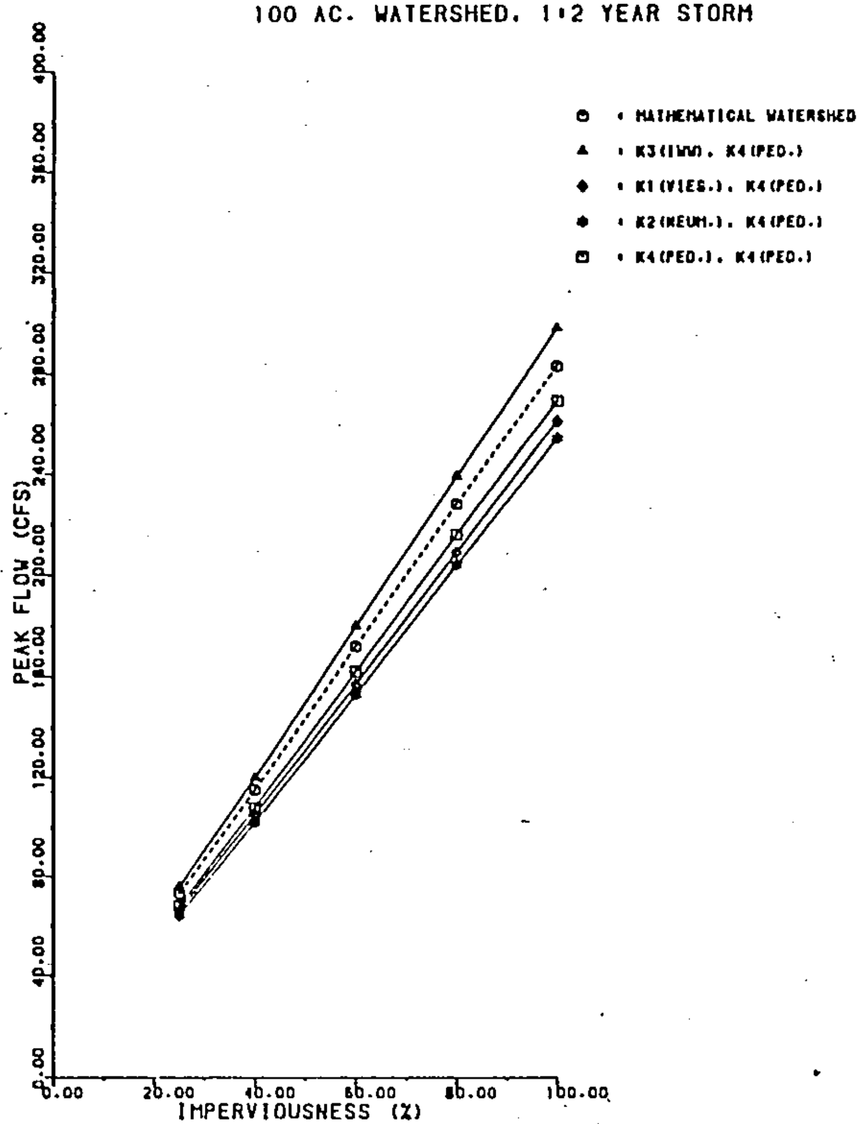


FIG. 5.10

COMPARISON OF PEAK FLOWS FOR 2PLR MODEL

100 AC. WATERSHED, 1:5 YEAR STORM

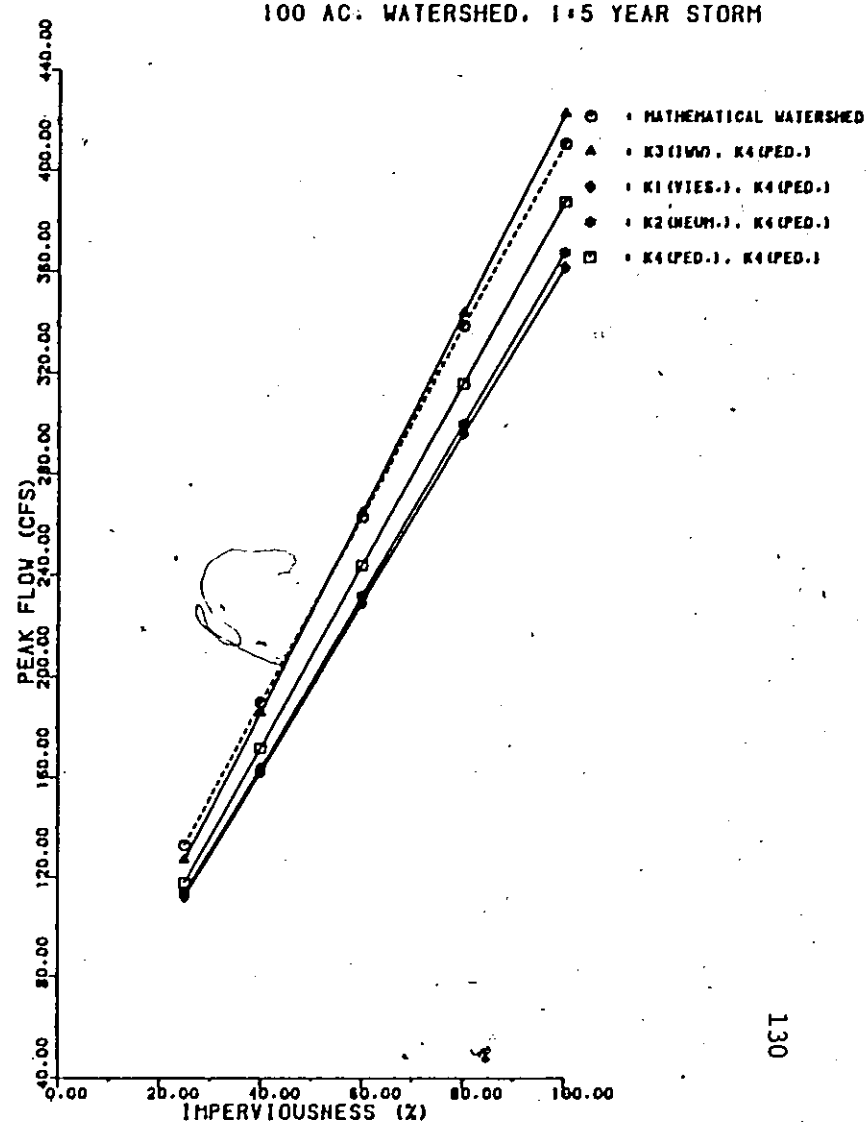


FIG. 5.11

COMPARISON OF PEAK FLOWS FOR 2PLR MODEL

100 AC. WATERSHED, 1:5 YEAR STORM

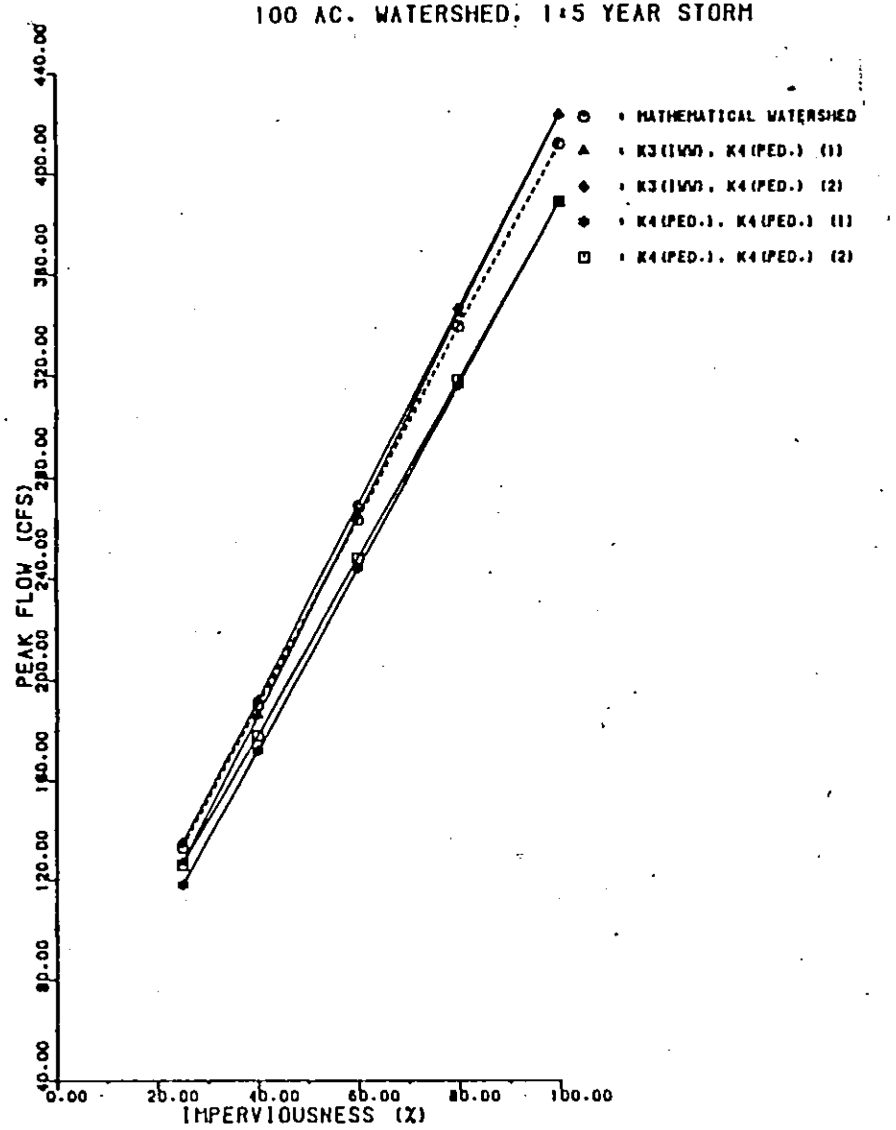


FIG. 5.12

COMPARISON OF PEAK FLOWS FOR 2PLR MODEL

100 AC. WATERSHED, 1:25 YEAR STORM

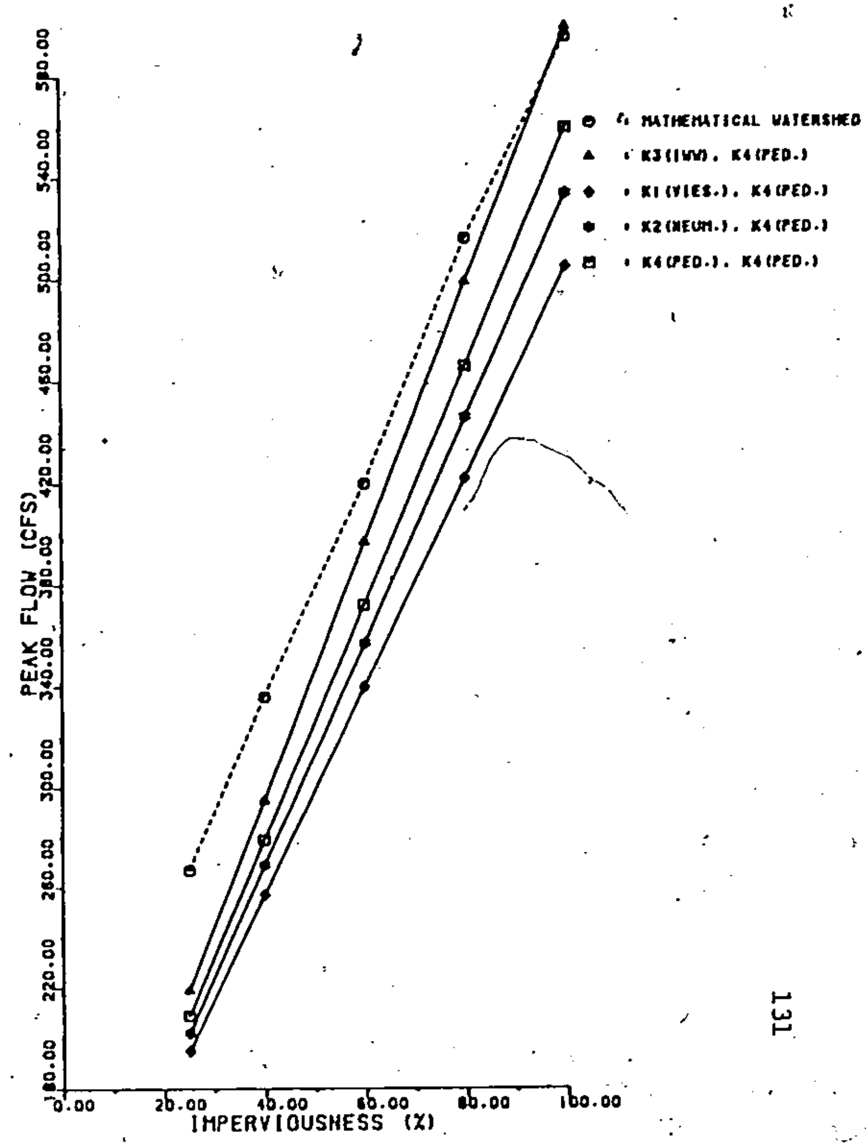


FIG. 5.13

COMPARISON OF PEAK FLOWS FOR 2PLR MODEL

100 AC. WATERSHED, 1:25 YEAR STORM

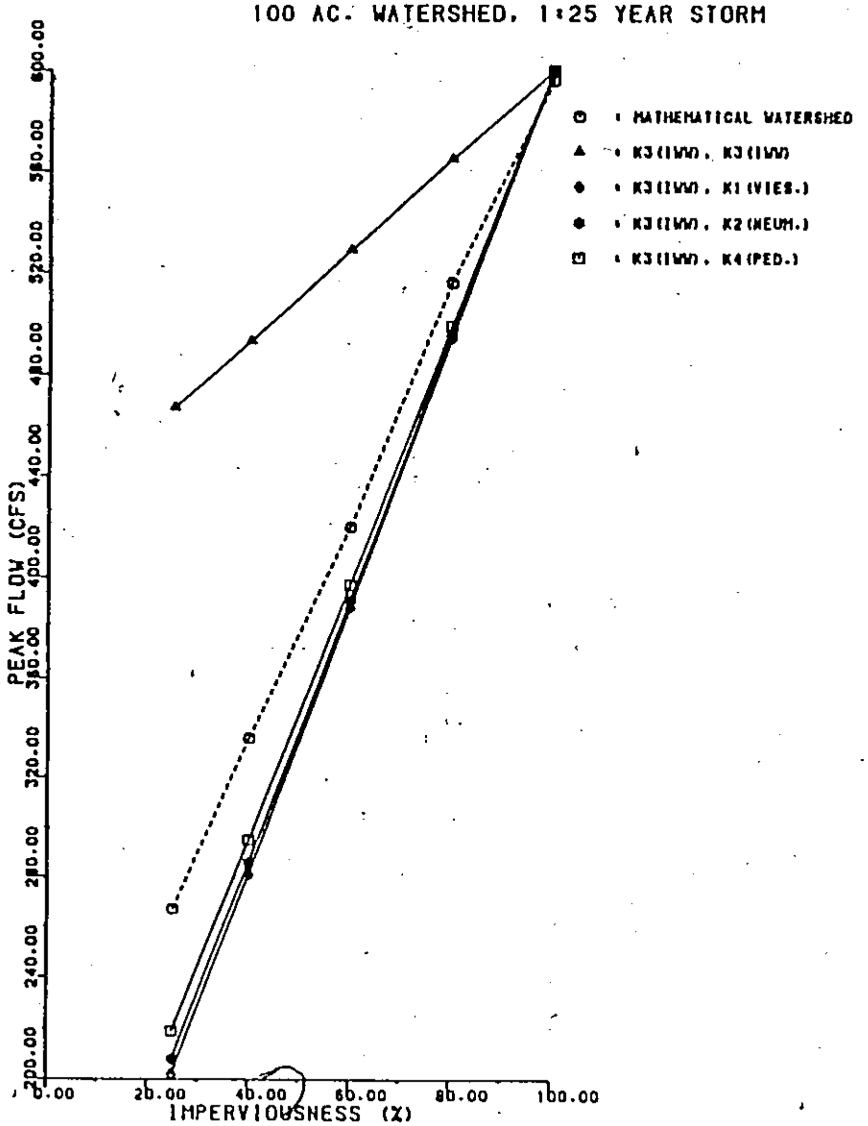
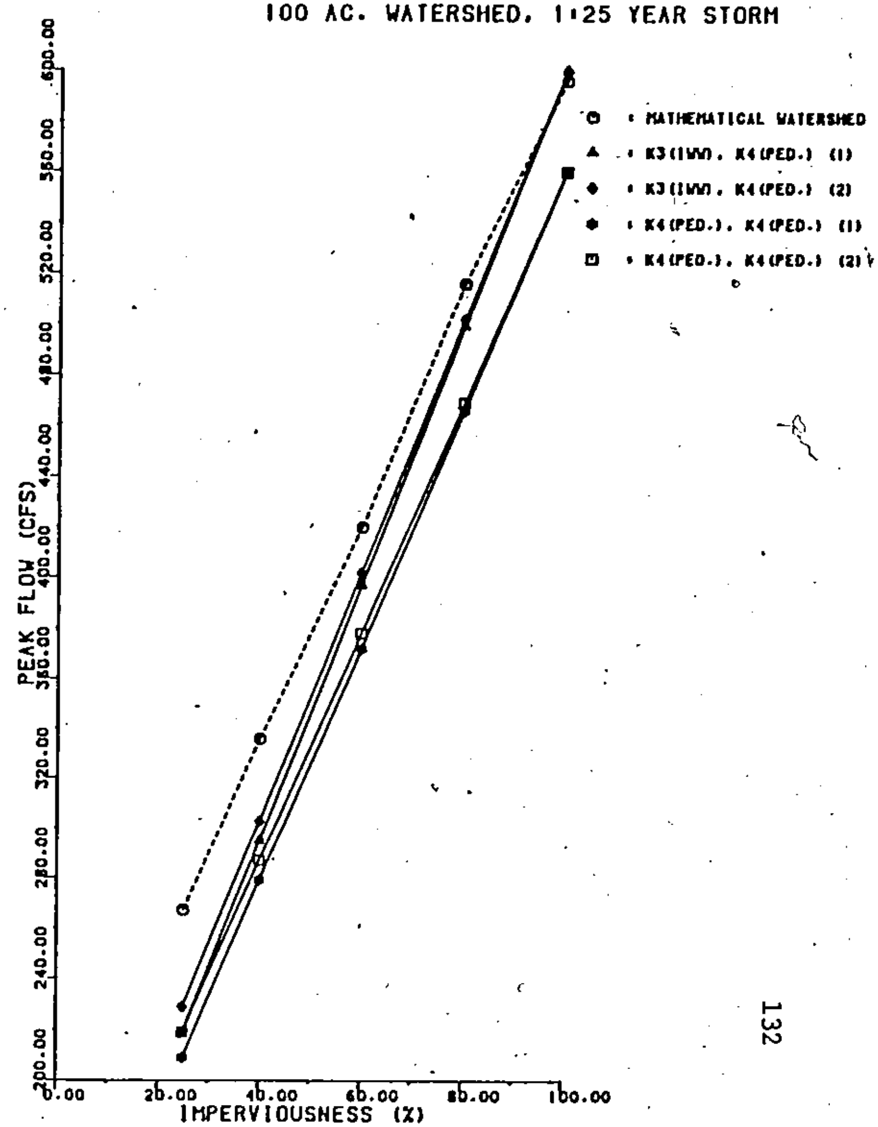


FIG. 5.14

COMPARISON OF PEAK FLOWS FOR 2PLR MODEL

100 AC. WATERSHED, 1:25 YEAR STORM



results given by the other three relations are very close together. The peak flows are all within 20 % of the peak flows from the mathematical watershed model.

Fig.5.14 compares the results when the two options of determining the intensity i is used. The second option when i is determined by the maximum Δt-minute rainfall excess intensity gives slightly better results but which are not significantly different from those of the first option.

The results demonstrate that the 2PLR model can estimate the peak flows for the 100-acre watersheds even for intense storms such as the 1:25 year storm. Two hydrographs simulated by the 2PLR model using K3 to estimate Ki and K4 to estimate Kp are shown in figs.5.15 and 5.16. These hydrographs are compared with the hydrographs obtained from detailed SWMM simulations from the mathematical watershed. The hydrographs generally fit well together. The hydrograph predicted by the 2PLR model is also compared with the hydrograph predicted by the SLR model as shown in fig.5.17. The peak flow simulated by the 2PLR model is much better than that predicted by the SLR model.

5.2.2 300-acre (121 ha) watersheds

The 2PLR model is also applied to 300-acre test watersheds at different levels of imperviousness. The comparison of peak flows are shown in figs.5.18, 5.19 and 5.20 for

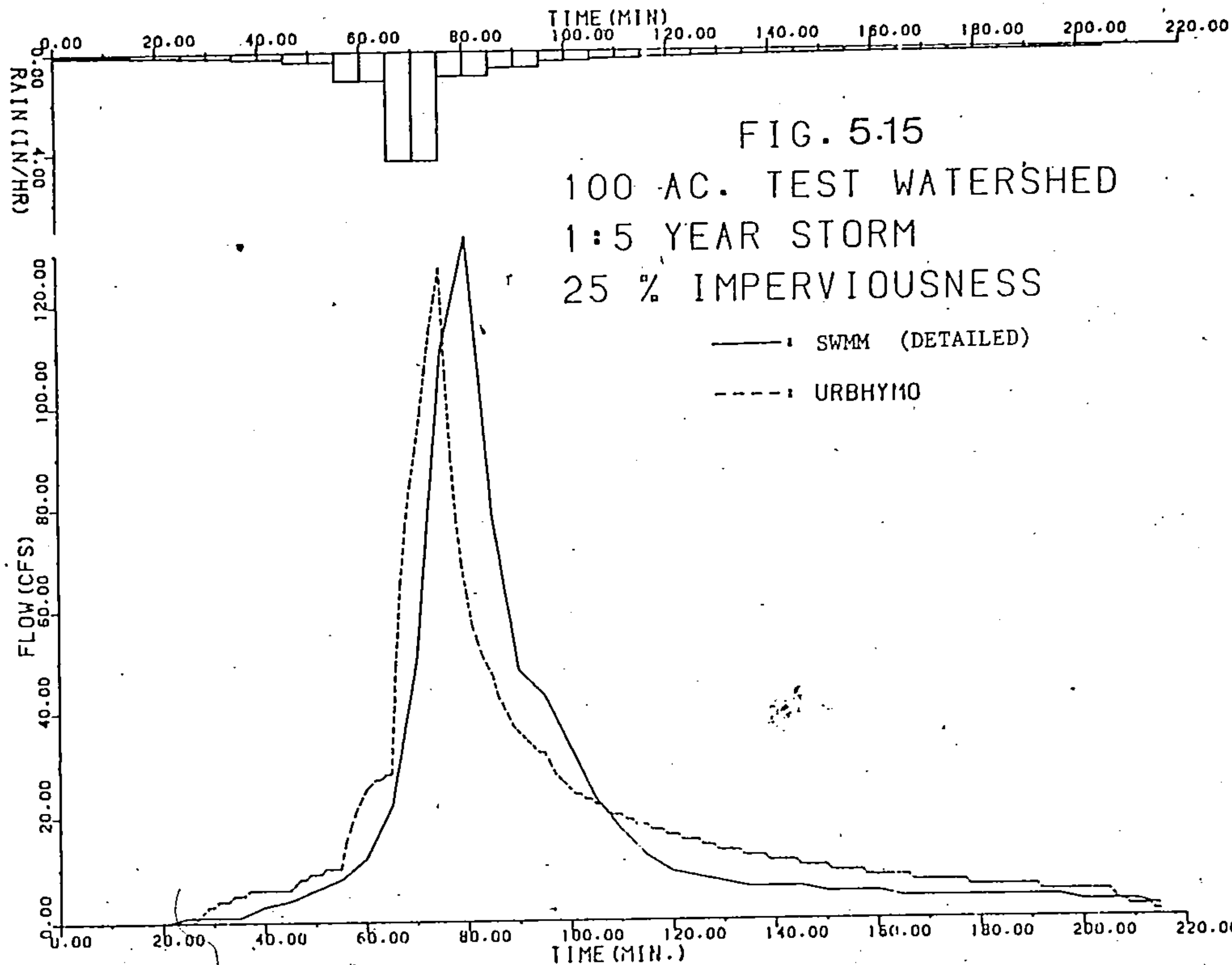
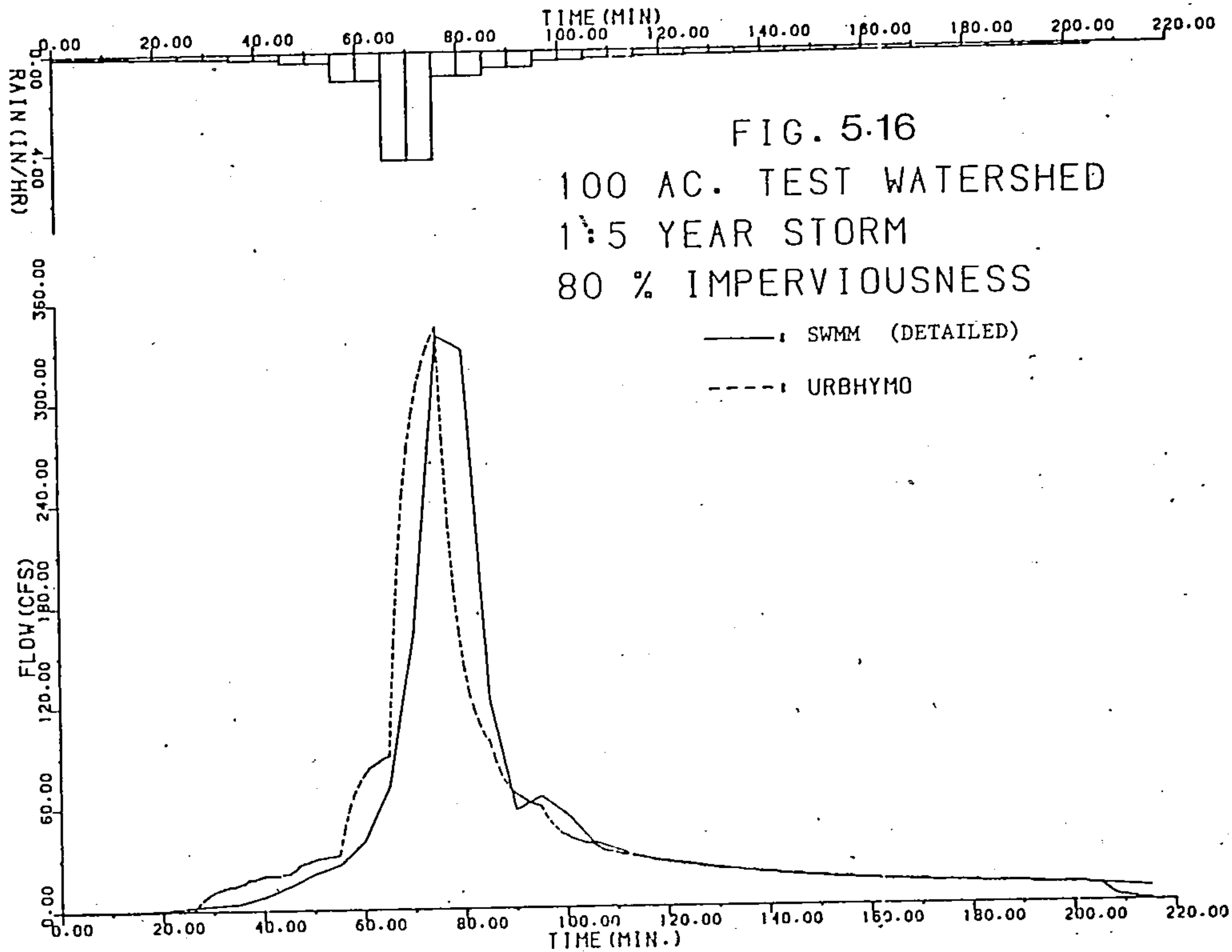
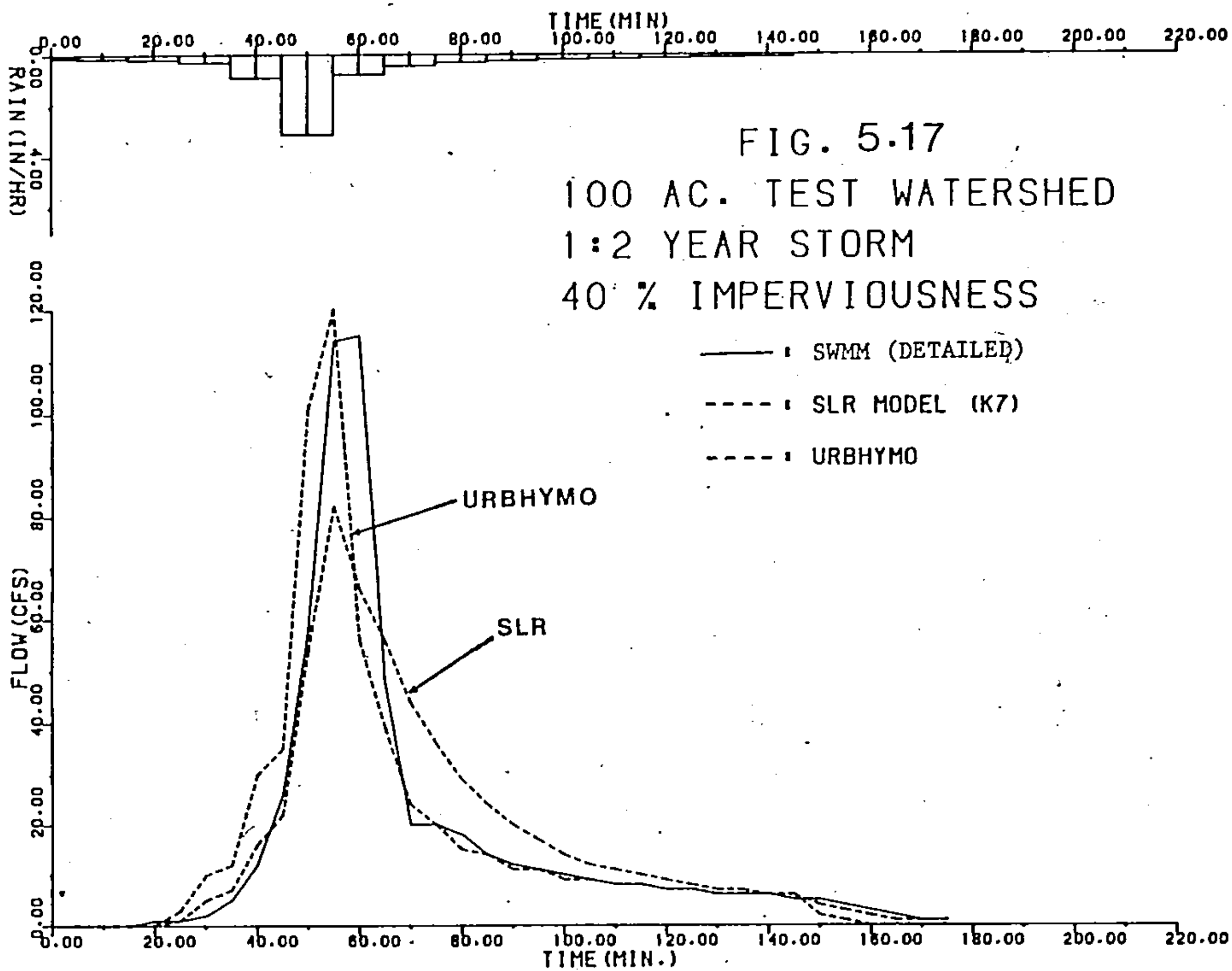


FIG. 5.15
100 AC. TEST WATERSHED
1:5 YEAR STORM
25 % IMPERVIOUSNESS

FIG. 5.16
100 AC. TEST WATERSHED
1:5 YEAR STORM
80 % IMPERVIOUSNESS




TIME (MIN)
RAIN (IN/HR)
FLOW (CFS)
TIME (MIN.)
FIG. 5.17
100 AC. TEST WATERSHED
1:2 YEAR STORM
40 % IMPERVIOUSNESS
SWMM (DETAILED)
SLR MODEL (K7)
URBHYMO
URBHYMO
SLR

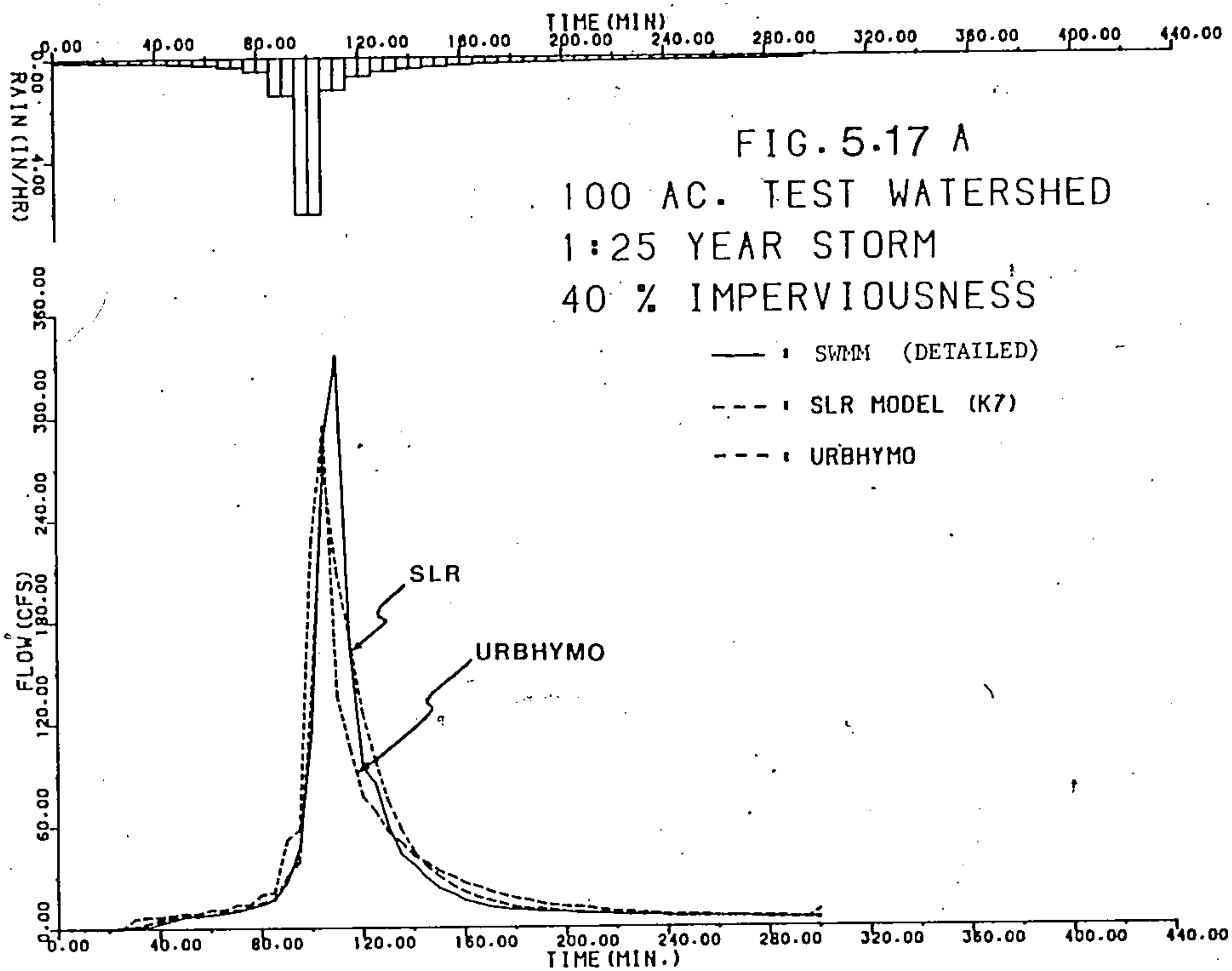


FIG. 5.17 A
100 AC. TEST WATERSHED
1:25 YEAR STORM
40 % IMPERVIOUSNESS

the 1:2 year, 1:5 year and 1:25 year storms respectively. The relations used for predicting the storage coefficient are K3 for the impervious area and K4 for the pervious area. The plotted peak flows show that for all the three storms, the predicted peak flows are very close to the ones from the mathematical watershed. The peak flows obtained by using the SLR model using K7 are also shown in the same figures for comparison. The 2PLR model gives much better results than the SLR model. The times to peak are 5 minutes less than those obtained by the mathematical watershed. The simulated hydrographs are plotted and compared in fig.5.21 - 5.22.

5.3 GENERAL CONCLUSIONS

It is found that the 2PLR model can predict peak flows close to those obtained from detailed SWMM simulation from the mathematical watersheds for areas of 25 acres up to 300 acres. This has been done without calibration of the model parameters. For watersheds larger than 100 acres, the peak flows simulated by this model are more accurate than those simulated by the SLR model. The nonlinearity in the urban watershed system is accounted for, in the model by the producing of the two different rainfall excess hyetographs from one total rainfall hyetograph. The transformation of each rainfall excess hyetograph to a runoff hydrograph by a linear reservoir is quasi-linear since the storage coefficient is a function of the rainfall excess intensity. The 2PLR model will be called URBHYMO.

FIG. 5.18

COMPARISON OF PEAK FLOWS

300 AC. WATERSHED, 1:2 YEAR STORM

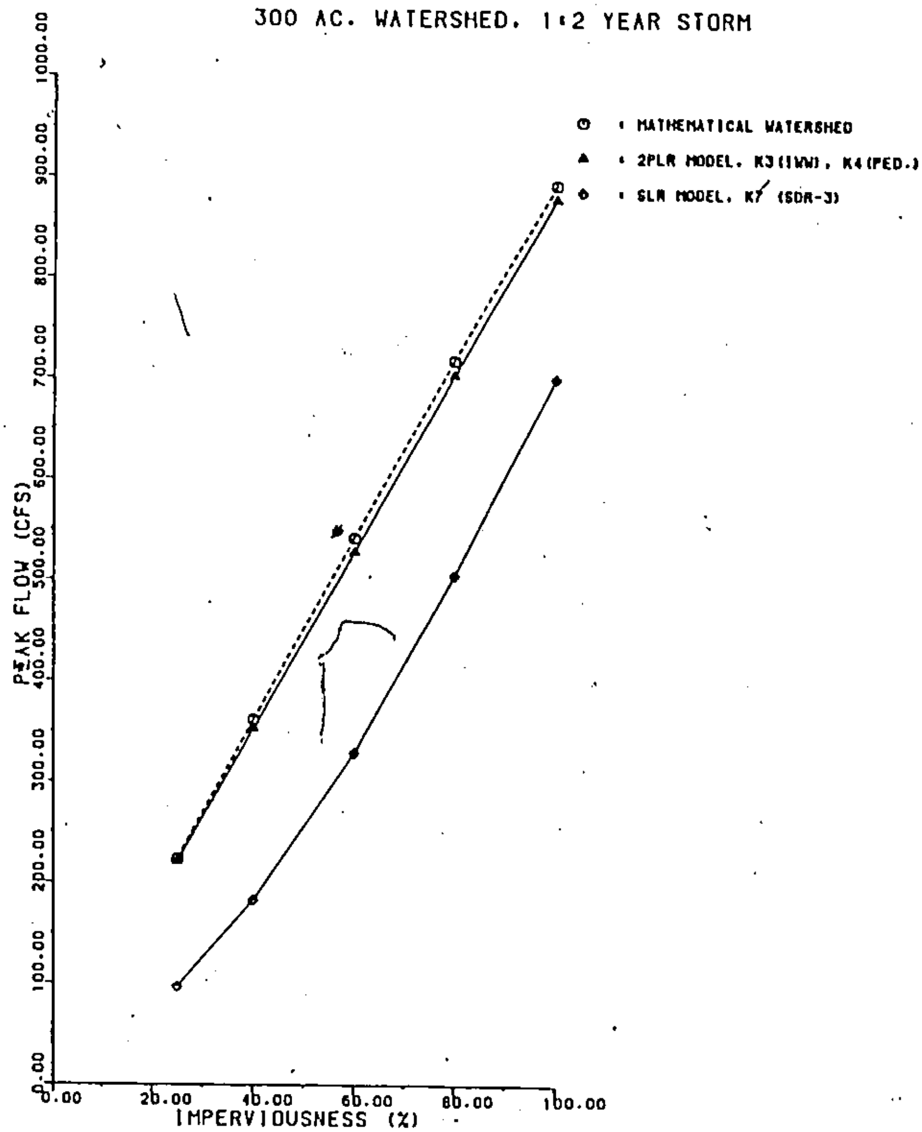


FIG. 5.19

COMPARISON OF PEAK FLOWS

300 AC. WATERSHED, 1:5 YEAR STORM

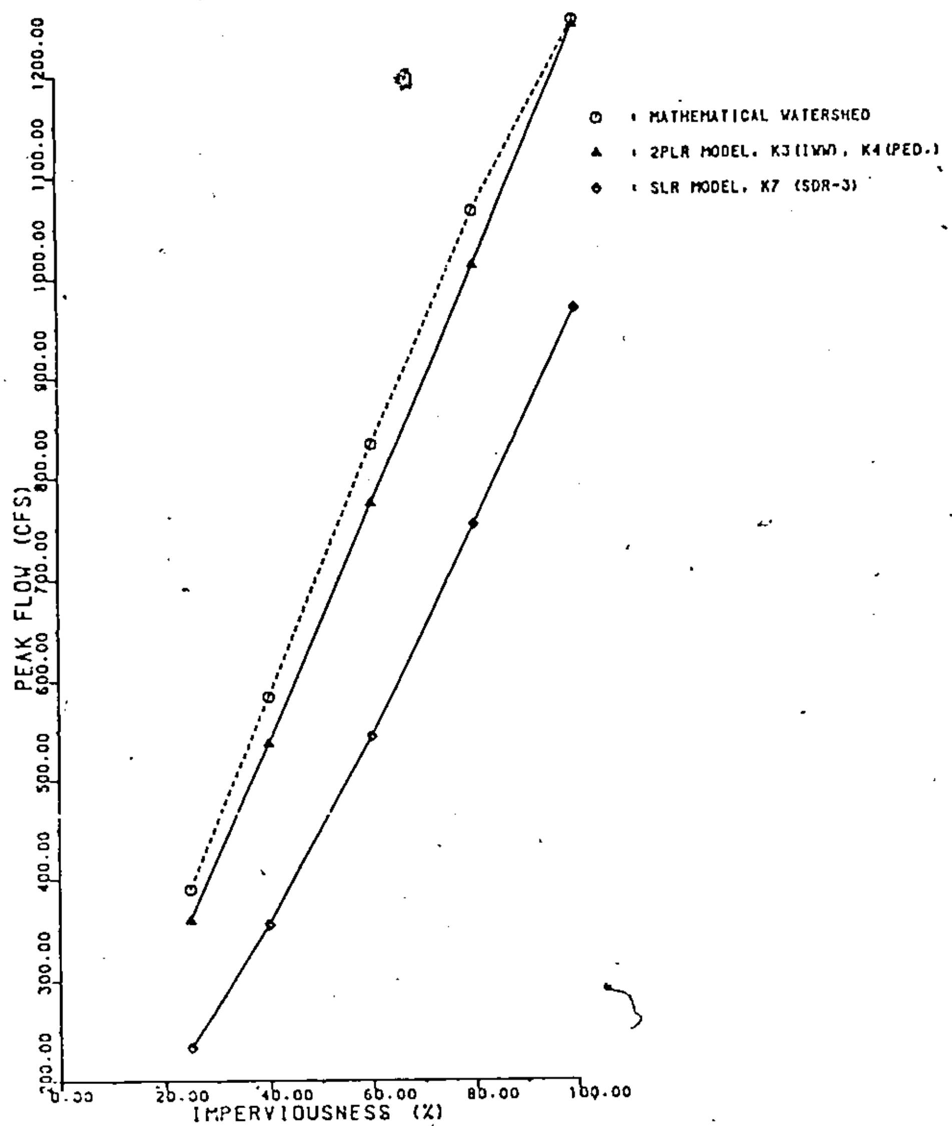
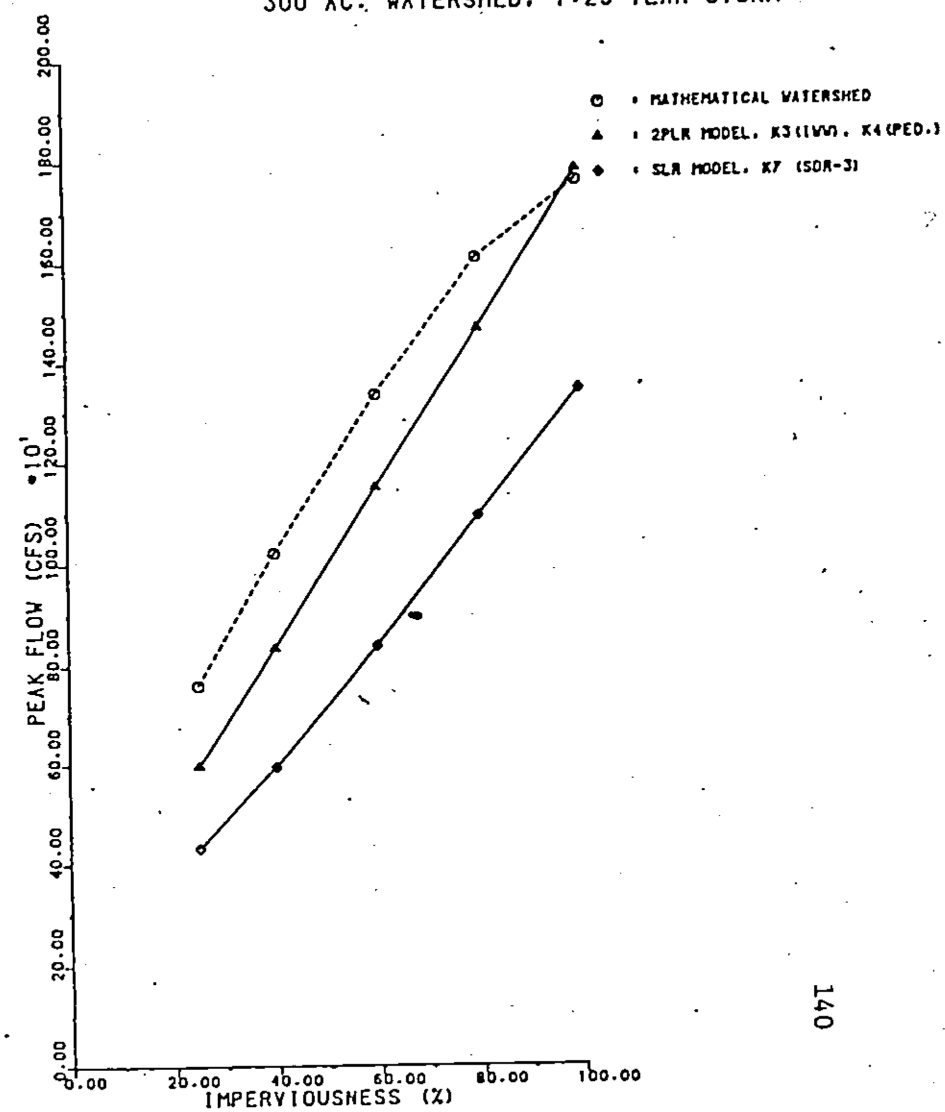


FIG. 5.20

COMPARISON OF PEAK FLOWS

300 AC. WATERSHED, 1:25 YEAR STORM



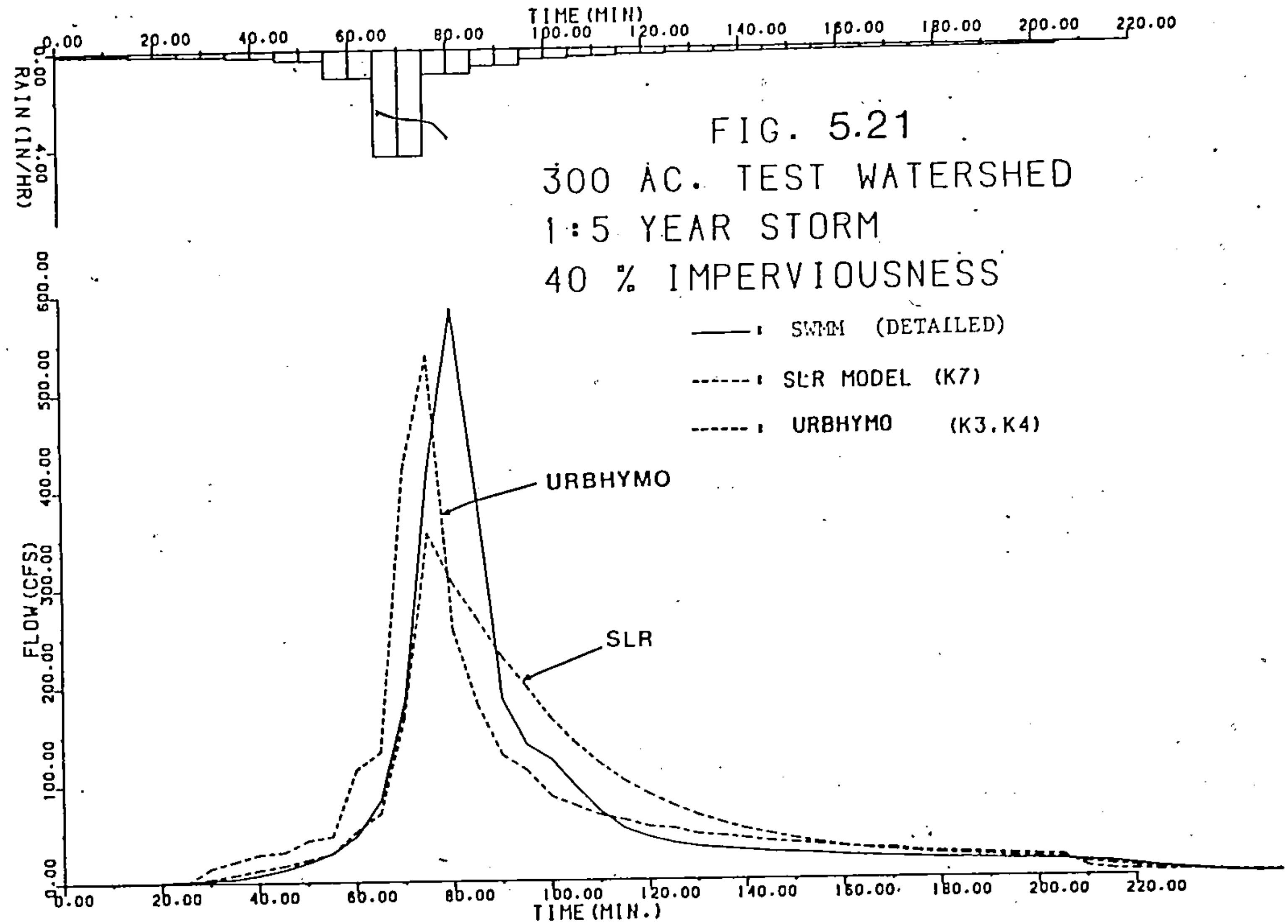
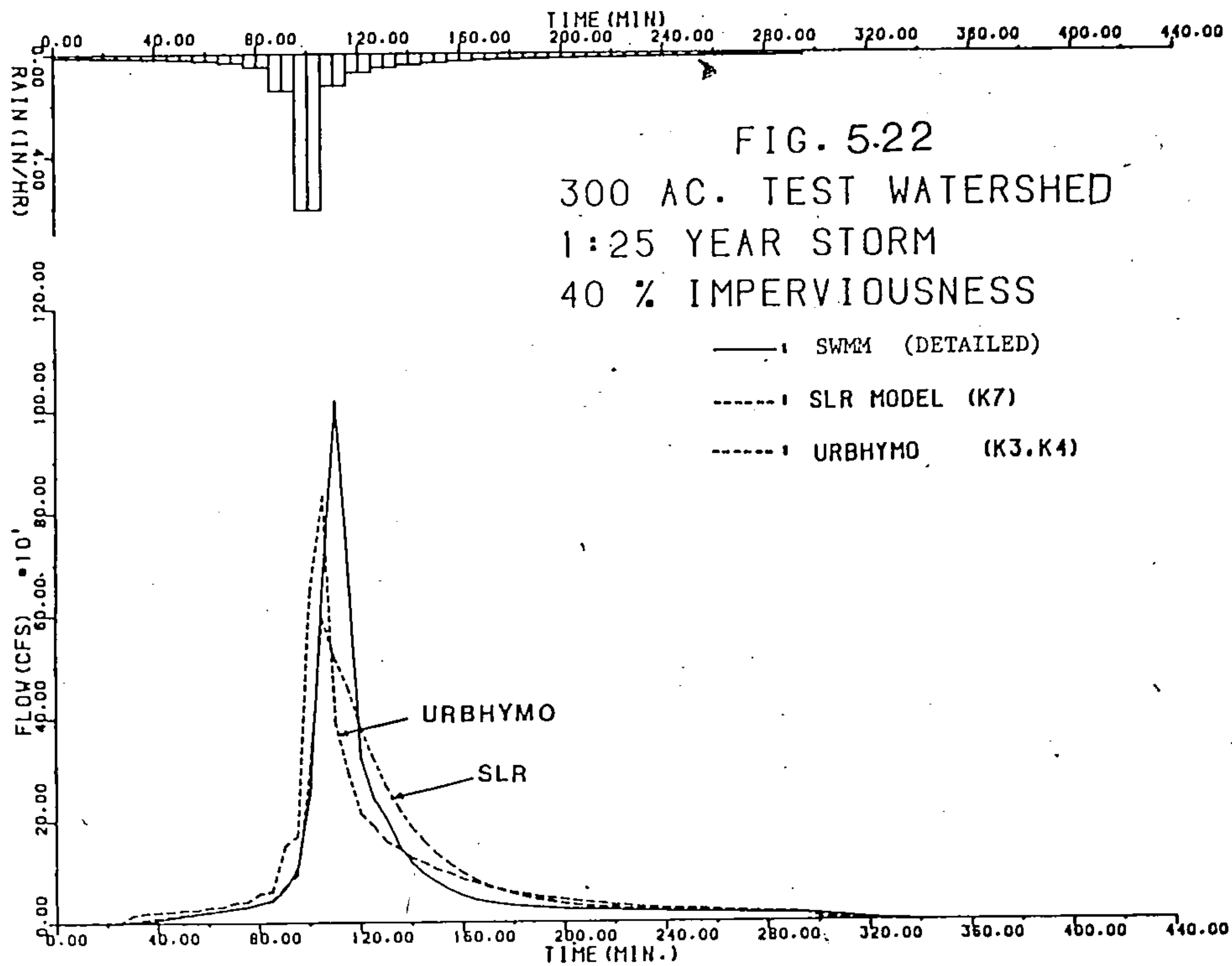


FIG. 5.21
300 AC. TEST WATERSHED
1:5 YEAR STORM
40 % IMPERVIOUSNESS



TIME (MIN)
RAIN (IN/HR)
FIG. 5.22
300 AC. TEST WATERSHED
1:25 YEAR STORM
40 % IMPERVIOUSNESS
SWMM (DETAILED)
SLR MODEL (K7)
URBHYMO (K3,K4)
URBHYMO
SLR
FLOW (CFS) *10¹
TIME (MIN.)

Chapter VI

VALIDATION OF URBHYMO AND FURTHER RESEARCH

6.1 INTRODUCTION

The development of the planning model is done by using hypothetical mathematical watersheds and simulation of the runoff from these watersheds by using SWMM-RUNOFF for the small areas and routing the hydrographs through the channel system. It is attempted to validate URBHYMO by comparing how well the simulated hydrographs can fit the observed hydrographs on two urban watersheds. The two watersheds selected are the Gray Haven watershed and the Malvern watershed. In both cases, only the more intense storms are selected for validation because measurement errors are reduced and these storms are important for design.

6.2 GRAY HAVEN WATERSHED

This watershed has good documented rainfall-runoff data and has been used for the validation of other models such as SWMM (J.F.MacLaren,1975) and ILLUDAS, including the recent continuous version (Wenzel and Voorhees,1980). It is a 23.3-acre residential watershed with group houses and is located in Baltimore, Maryland. 52 % of the area is classified as impervious of which 44 % is directly-connected to the sewer system. It has a separated storm sewer system connected to an outfall. Shown in fig. 6.1 is a detailed plan of the watershed together with a schematic layout of the drainage system. Seven of the most intense storms are selected for simulation.

URBHYMO (2PLR model) is applied in a lumped manner to the watershed. Ki is estimated by K3 and Kp is estimated by K4. The slope and length of the watershed is taken to be the same in both the impervious and pervious portions of the watersheds. The slope is taken as the average slope of the main channel system since it runs right through the watershed. The watershed is roughly rectangular in shape and the "length" in the K relations is calculated by assuming that it is the width of a 1.5x by 1x rectangle with equivalent area. It is found to be 822 feet. This is taken to be the length for both the impervious and pervious portions. The surface roughness n is assumed to be 0.25 for the pervious areas. Since the intensity of the rainfall changed from minute to minute, the intensity i is estimated by the peak rainfall excess intensity. The infiltration parameters used are f = 3.0 in./hr., f = 0.52 in./hr., = 4.14/hr. No calibration is done to the model parameters.

Results comparing observed and simulated hydrographs are tabulated in tables 6.1 which also include the results obtained by other models such as SWMM, RRL and MIT (MacLaren,1974). Observed and simulated hydrographs for several storms are plotted in figs.6.2 - 6.8.

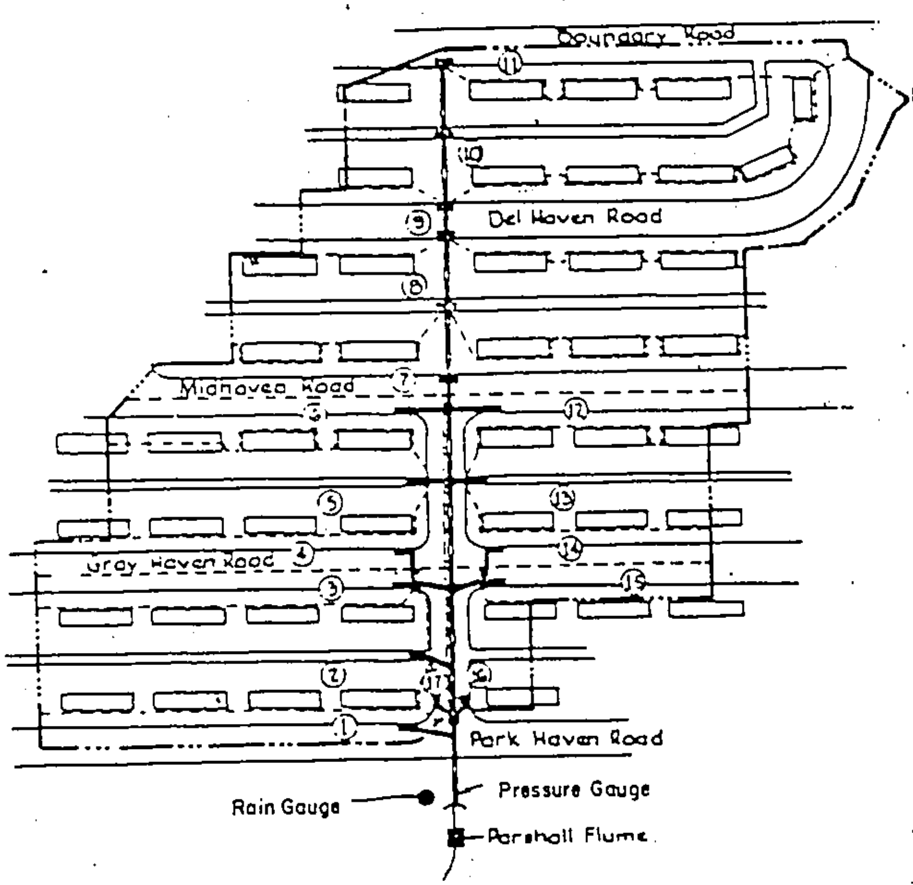


Fig.6.1 Gray Haven Test Area

Table 6.1 COMPARISON OF OBSERVED AND SIMULATED PEAK FLOWS BY VARIOUS MODELS

(Gray Haven Watershed)

STORM DATE	Qp 1 Observed Peak (cfs)	SWMM		RRL		MIT		URBHYMO	
		Qp 2	2/1	Qp 3	3/1	Qp 4	4/1	Qp 5	5/1
1. 5 June 1963	80.7	78.0	0.97	59.4	0.74	–	–	72.8	0.90
2. 10 June 1963	79.0	78.1	0.99	58.9	0.75	82.5	1.04	72.9	0.92
3. 14 June 1963	30.8	31.4	1.02	32.9	1.07	34.5	1.12	30.4	0.97
4. 20 June 1963	29.6	34.1	1.15	35.7	1.21	–	–	32.3	1.09
5. 1 August 1963	88.1	63.3	0.72	59.2	0.67	83.0	0.93	67.8	0.77
6. 14 August 1963	34.7	38.5	1.11	40.5	1.17	–	–	41.7	1.20
7. 2 August 1965	30.3	34.7	1.15	36.6	1.21	–	–	33.7	1.11

COMPARISON OF OBSERVED AND SIMULATED TIMES TO PEAK (min.) BY VARIOUS MODELS

1. 5 June 1963	41	39	.95	34	.83	–	–	37	0.90
2. 10 June 1963	41	39	.95	34	.83	34	.83		
3. 14 June 1963	22	21	.96	22	1.00	23	1.05	20	0.91
4. 20 June 1963	22	21	.96	22	1.00	–	–	20	0.91
5. 1 August 1963	12	13	1.08	13	1.08	14	1.17	11	0.92
6. 14 August 1963	16	15	.94	16	1.00	–	–	14	0.88
7. 2 August 1965	35	34	.97	35	1.00	–	–	32	0.91

Note: The results for the other models besides URBHYMO are obtained from J.F.MacLaren, 1975.

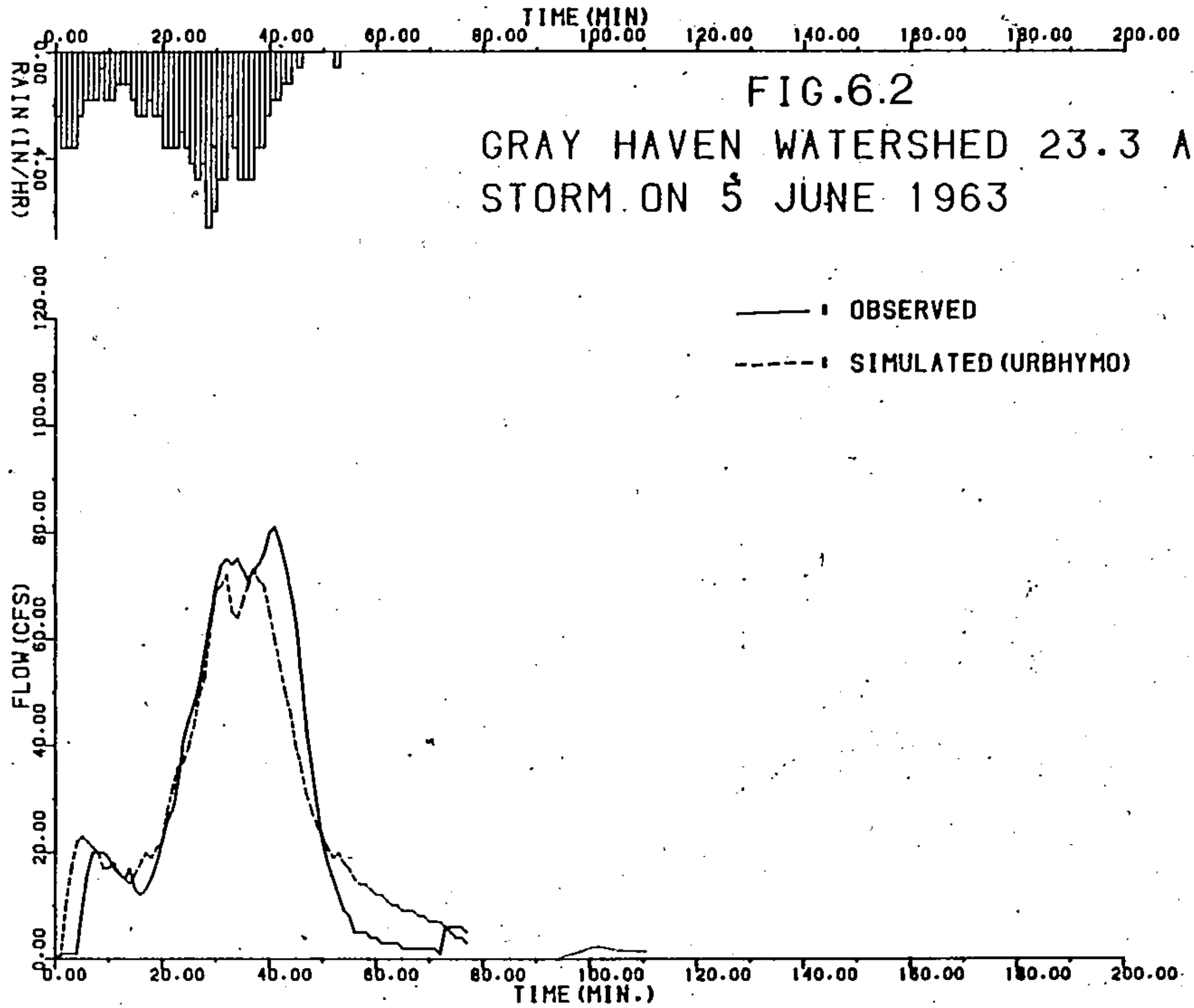


FIG.6.2
GRAY HAVEN WATERSHED 23.3 AC.
STORM ON 5 JUNE 1963

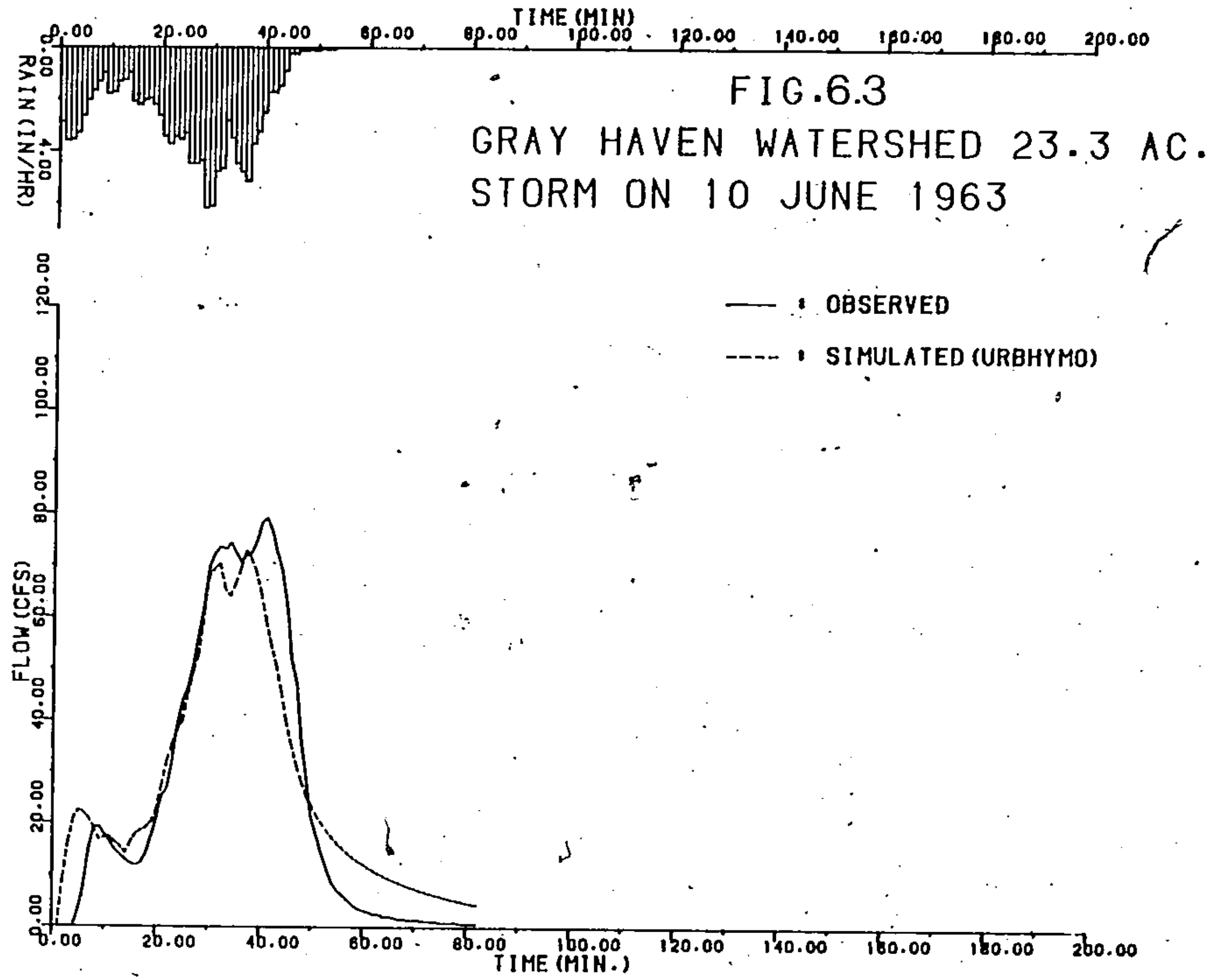


FIG.6.3
GRAY HAVEN WATERSHED 23.3 AC.
STORM ON 10 JUNE 1963

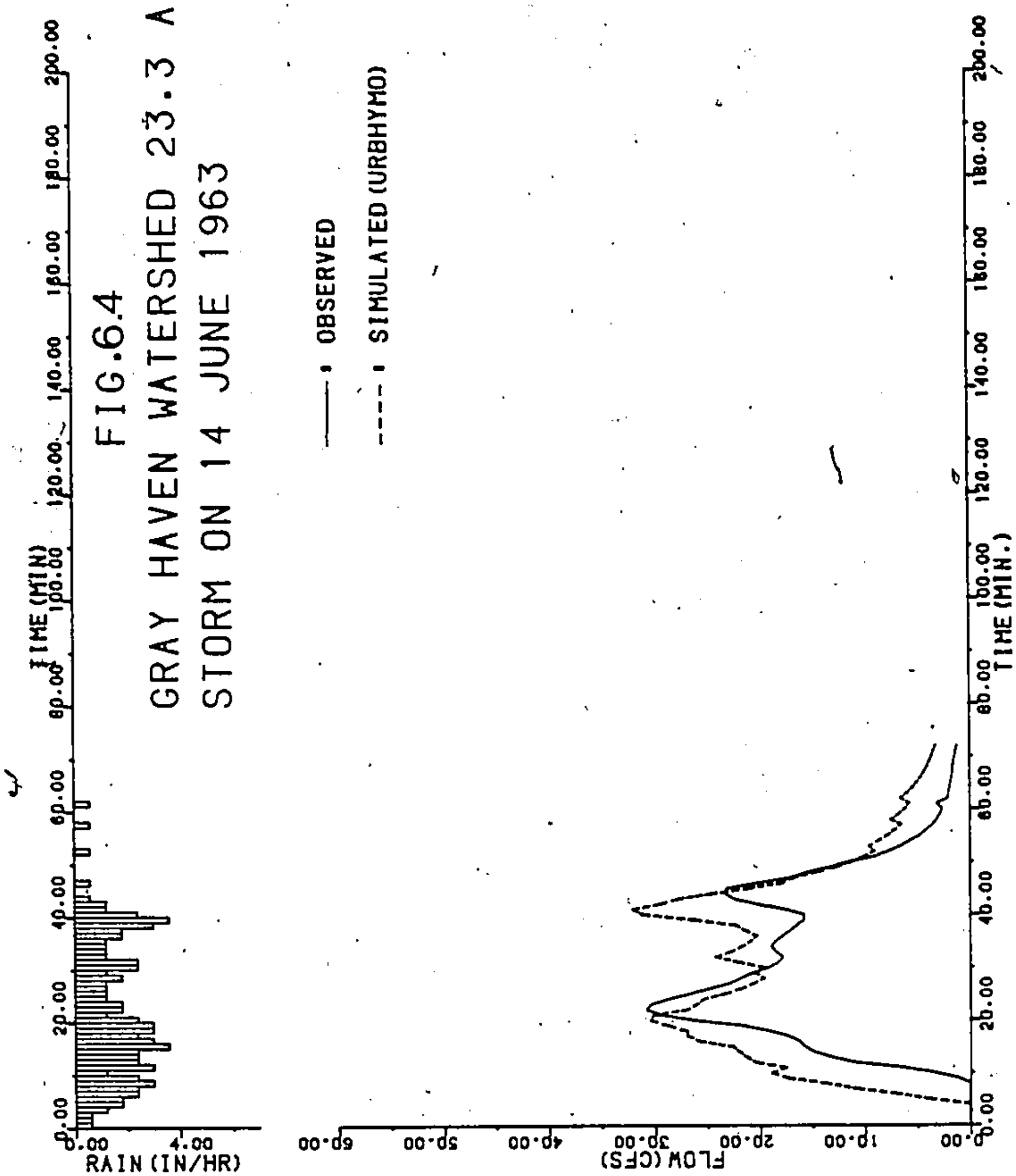


FIG.6.4
GRAY HAVEN WATERSHED 23.3 AC.
STORM ON 14 JUNE 1963

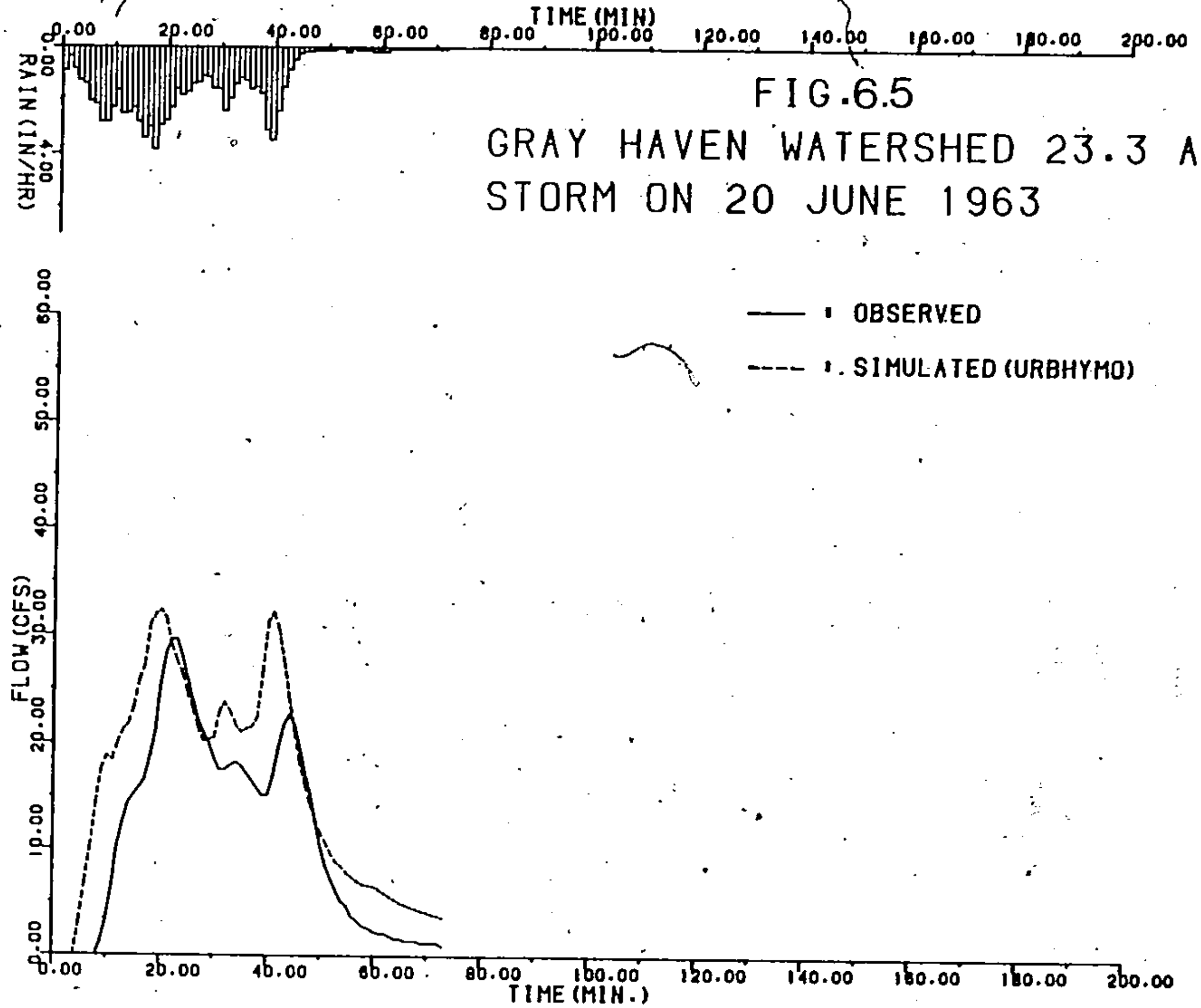


FIG.6.5
GRAY HAVEN WATERSHED 23.3 AC.
STORM ON 20 JUNE 1963

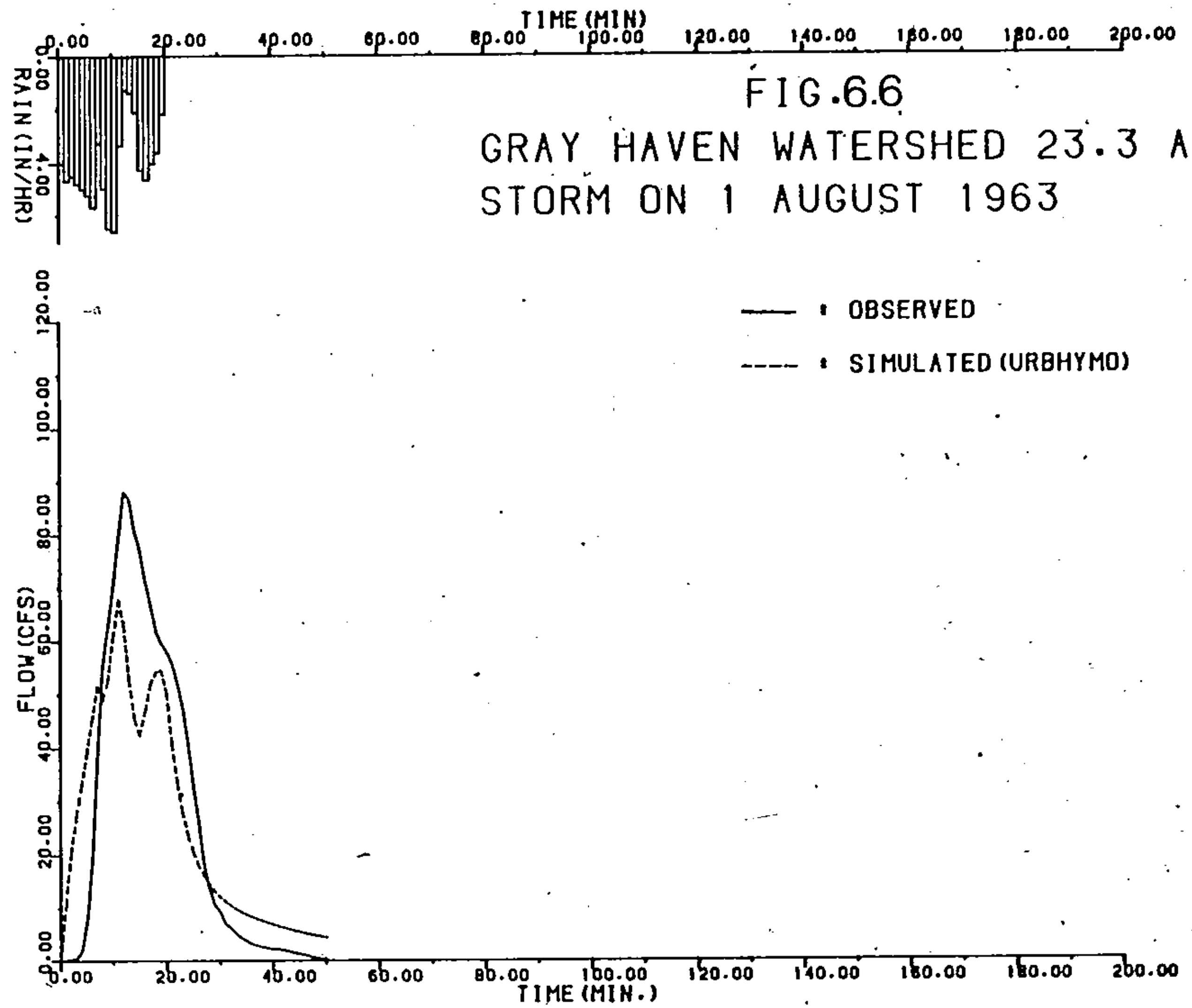


FIG.6.6
GRAY HAVEN WATERSHED 23.3 AC.
STORM ON 1 AUGUST 1963

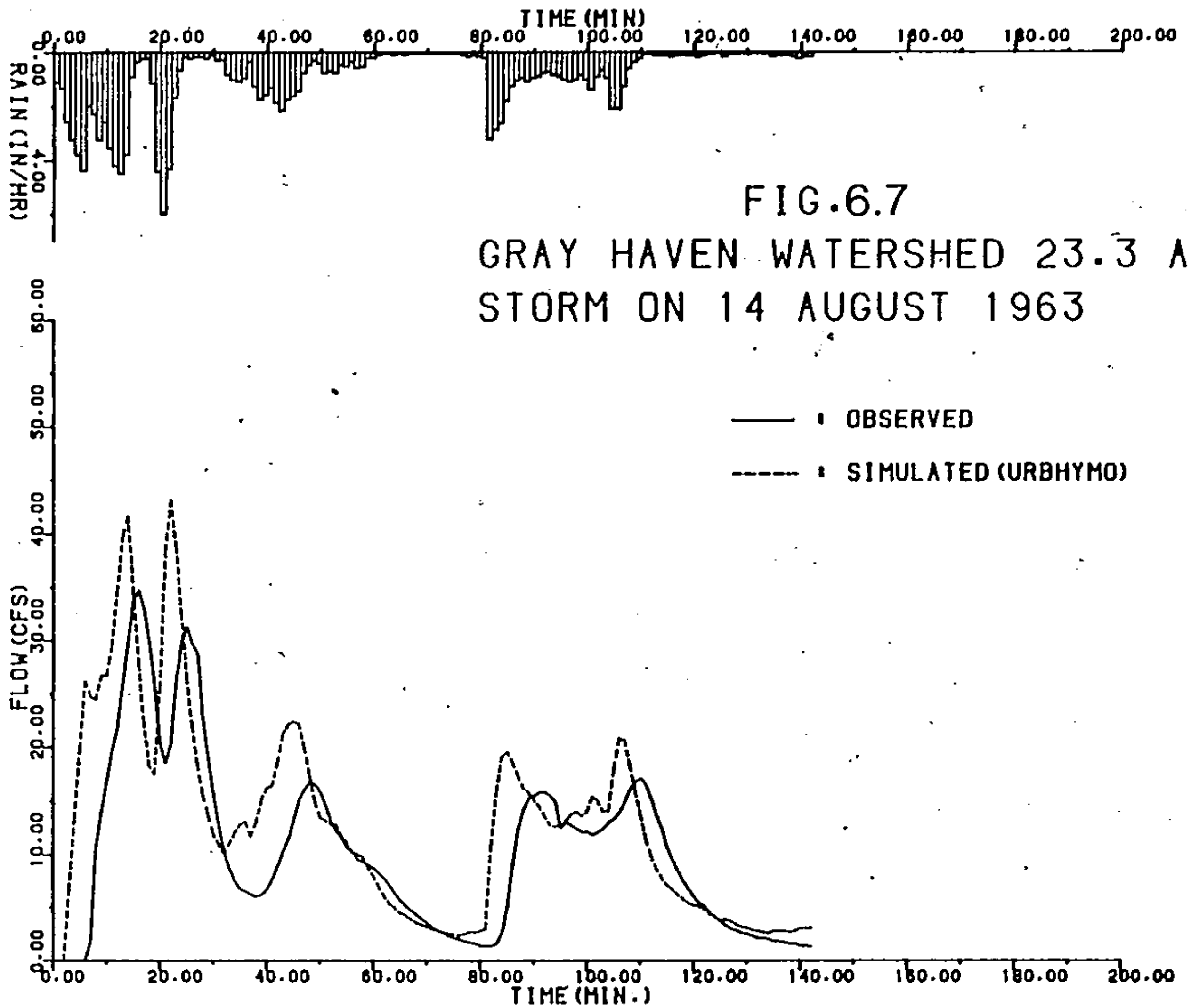


FIG.6.7
GRAY HAVEN WATERSHED 23.3 AC.
STORM ON 14 AUGUST 1963

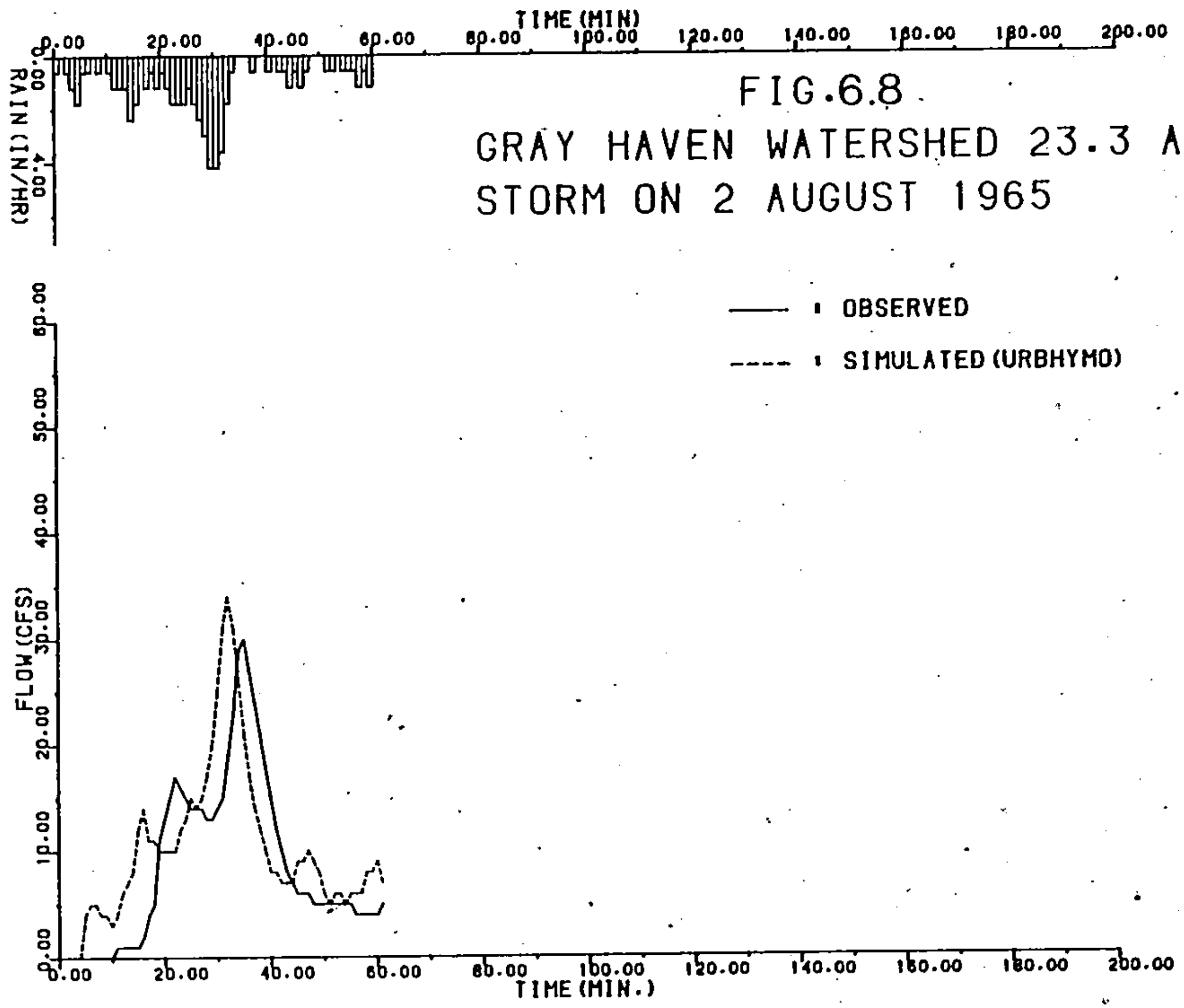


FIG.6.8

GRAY HAVEN WATERSHED 23.3 AC.

STORM ON 2 AUGUST 1965

6.3 MALVERN URBAN TEST CATCHMENT

This watershed is located in Burlington, Ontario. The data has been collected by the Hydraulics Research Division, Canada Centre for Inland Waters (Marsalek,1977,1979). Two reports which tabulate the data collected in 1973 and 1974 have been written by Marsalek. The reports also contain a description of the watershed. Rainfall data is measured by means of a flow-measuring weir at the drainage outfall. Many of the storms collected are small and in many cases, the runoff comes only from the impervious areas.

The contributing area of the watershed is 57.6 acres which is determined from a topographical map as well as field inspection. The watershed is a residential subdivision drained by a separate sewer system. The average slopes of the drainage area have been determined as follows.

northeast - southwest direction 1.0 %

northwest - southeast direction 0.7 %

highest elevation point - drainage outlet 1.0 %

Front yards and back yards have varying slopes depending on local conditions. Road slopes are on the average 1 %. 19.51 acres of the area is impervious which is 34 % of the watershed. The impervious area consists of roofs, roads, driveways and sidewalks and all the roof leaders are connected to the sewer systems. The storm sewer system is shown in fig.6.9. The soil types are well-drained, fox

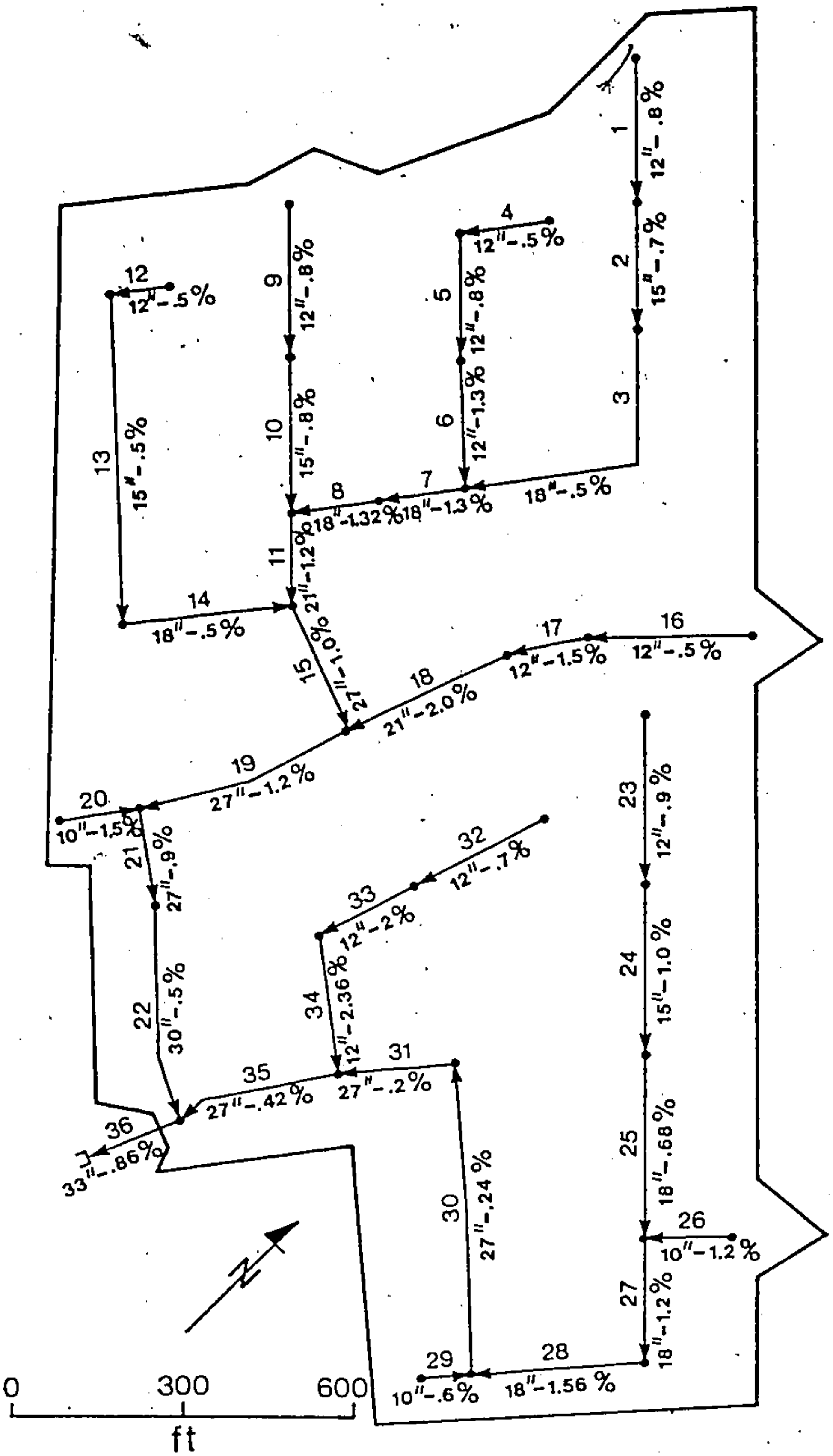


FIG.6.9 MALVERN CATCHMENT-STORM SEWER SYSTEM

(Ref. Marsalek, 1977)

sandy loam. Marsalek has used the SWMM-RUNOFF Block to simulate the runoff hydrographs and the results are presented in table 6.2.

The infiltration default values in SWMM are used in the simulations. The default values for surface roughness are also used. For surface depression storage, the pervious area value of 0.184 in. is used while for the impervious area, it is estimated as 0.02 in. from the runoff simulation instead of the default value of 0.062 in. For URBHYMO (2PLR model) the same infiltration parameters as those used in SWMM are utilized. The slope is taken to be 1 % for both the impervious and pervious areas. The overland flow length is taken to be the width of a 1.5x by 1x rectangular watershed. The width x is found to be 1293 feet. The roughness values are the same as those of SWMM.

6.4 SUMMARY OF RESULTS

The results show that for planning purposes the peak flows are simulated very well by URBHYMO. For five of the seven storms in the Gray Haven watershed, the simulated peak flows are within 11 % of the observed peak flows. The other two storms have simulated peak flows within 25 % of the observed peak flows. The results from URBHYMO are also compared with the detailed simulations of SWMM. The SWMM simulations are done by routing the inlet hydrographs through

Table 6.2 COMPARISON OF OBSERVED AND SIMULATED PEAK FLOWS

(Malvern Watershed)

Storm	Observed Qp1(cfs)	SWMM Qp 2	SWMM 2/1	URBHYMO Qp 3	URBHYMO 3/1
1. 22 Sept. 1973	32.4	34.5	1.06	29.8	.94
2. 23 Sept. 1973	25.2	22.4	.88	19.2	.76
3. 31 May 1974	31.8	39.3	1.23	33.9	1.07
4. 21 June 1974	20.4	25.7	1.27	23.9	1.17
5. 4 July 1974	27.2	23.8	0.88	21.4	.79
6. 19 July 1974	24.1	18.2	0.75	16.3	.68

COMPARISON OF OBSERVED AND SIMULATED TIMES TO PEAK (min.)

1. 22 Sept. 1973	39	37	.95	34	.94
2. 23 Sept. 1973	113	118	1.04	118	1.0
3. 31 May 1974	32	27	0.84	27	.93
4. 21 June 1974	16	13	0.81	20	1.05
5. 4 July 1974	8	5	0.63	6	.75
6. 19 July 1974	7	8	1.0	8	1.14

Note: The results for SWMM are obtained from Marsalek, 1977, 1979.

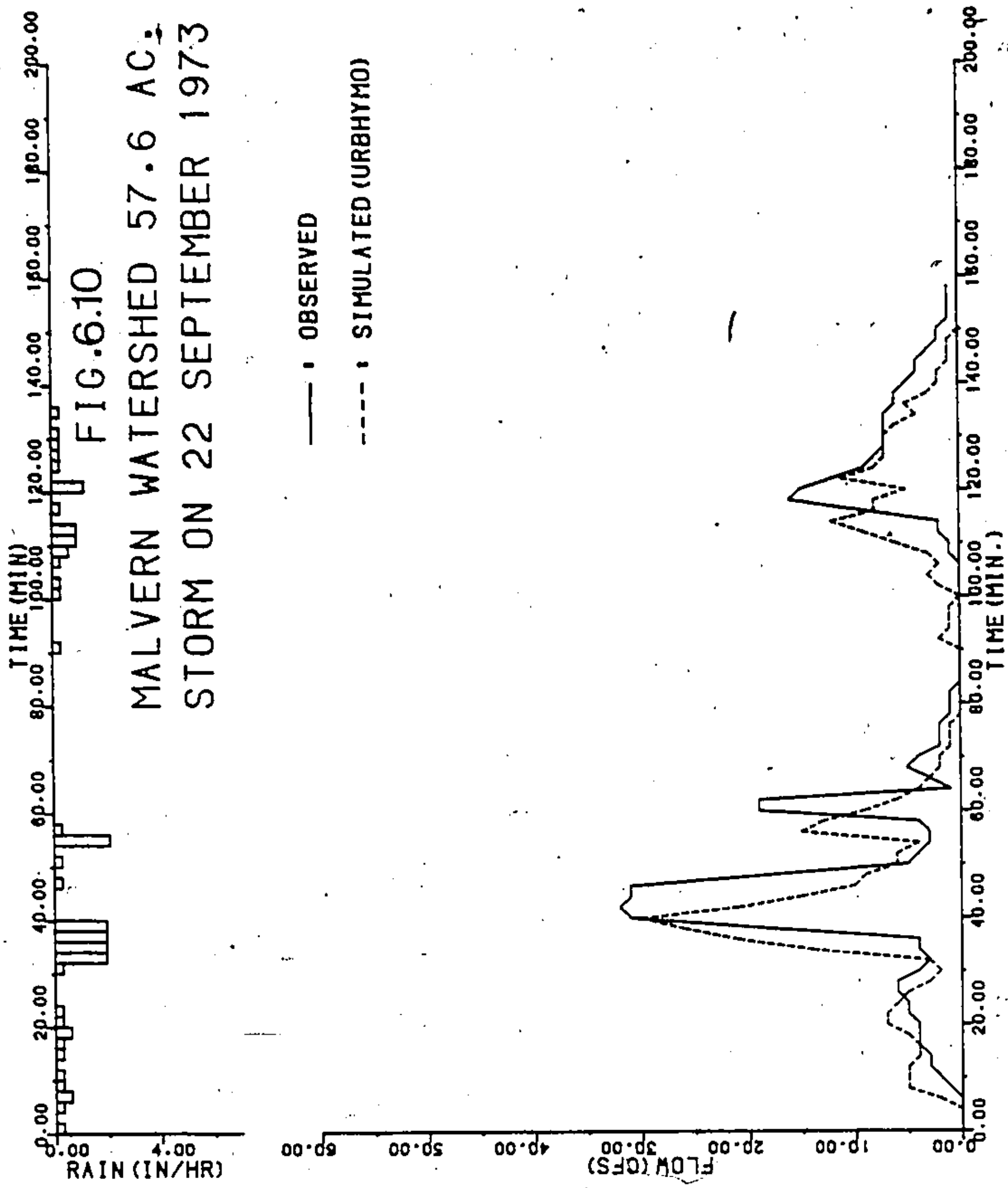


FIG. 6.10
MALVERN WATERSHED 57.6 AC.
STORM ON 22 SEPTEMBER 1973

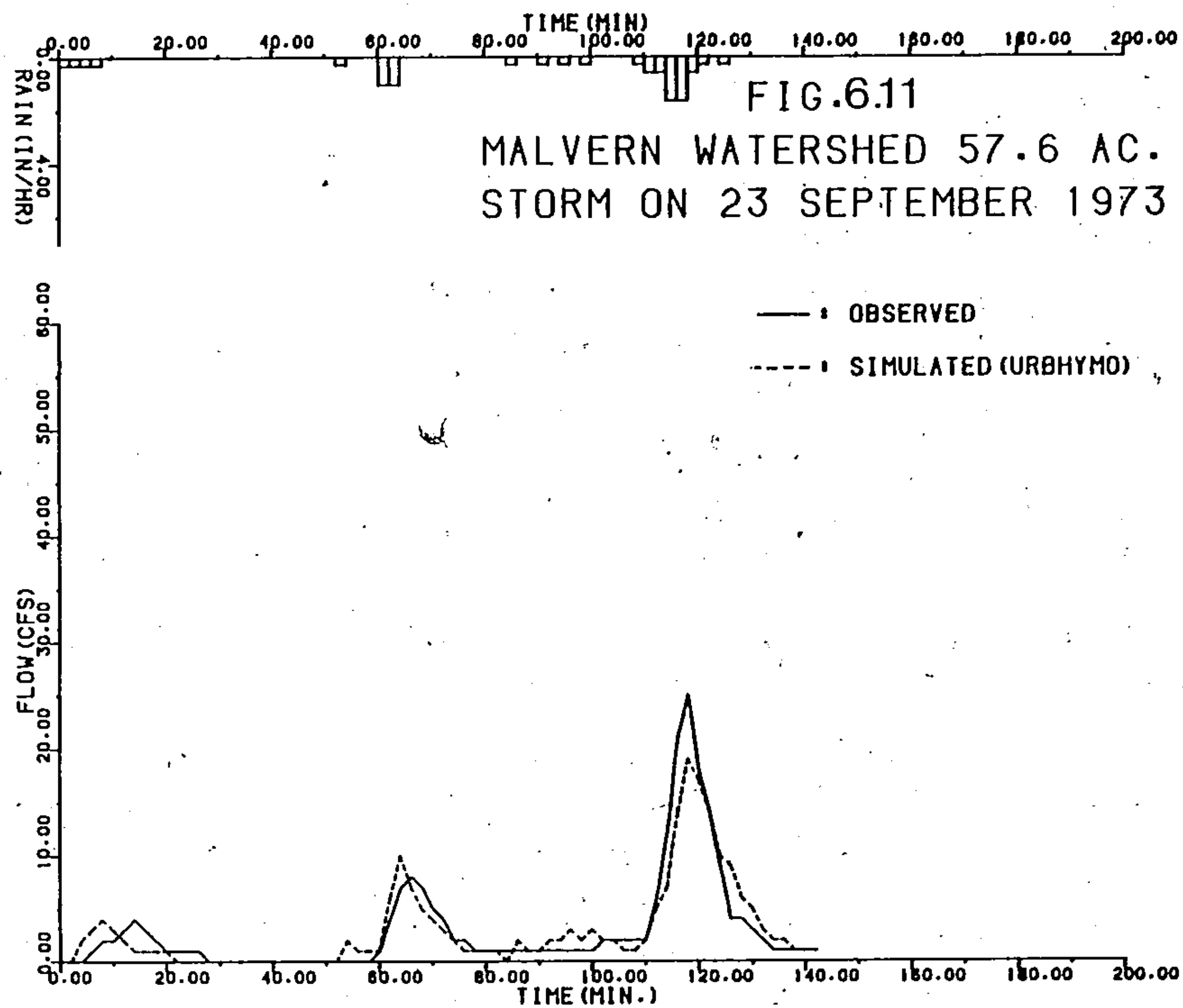


FIG.6.11
MALVERN WATERSHED 57.6 AC.
STORM ON 23 SEPTEMBER 1973

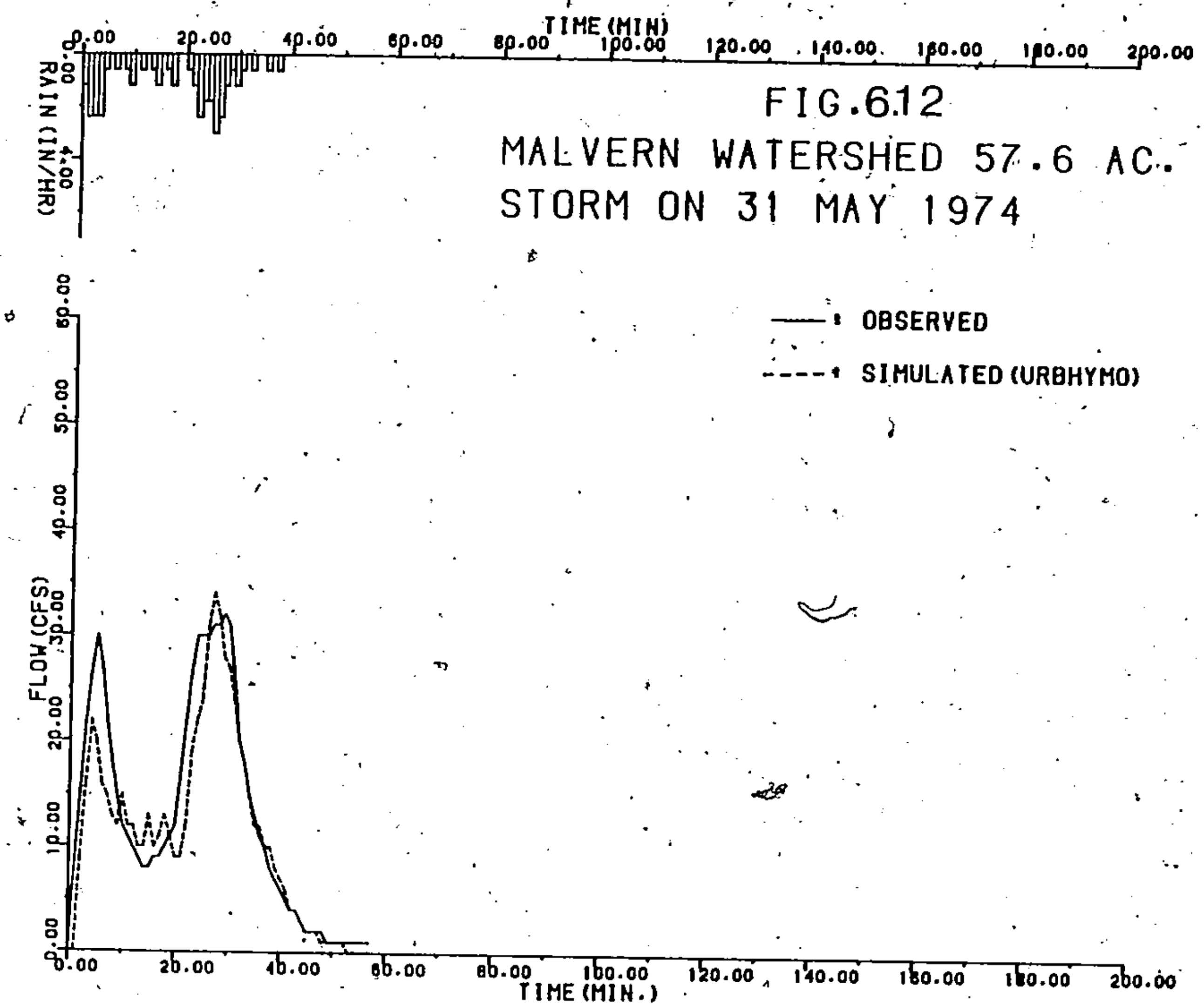


FIG. 6.12
MALVERN WATERSHED 57.6 AC.
STORM ON 31 MAY 1974

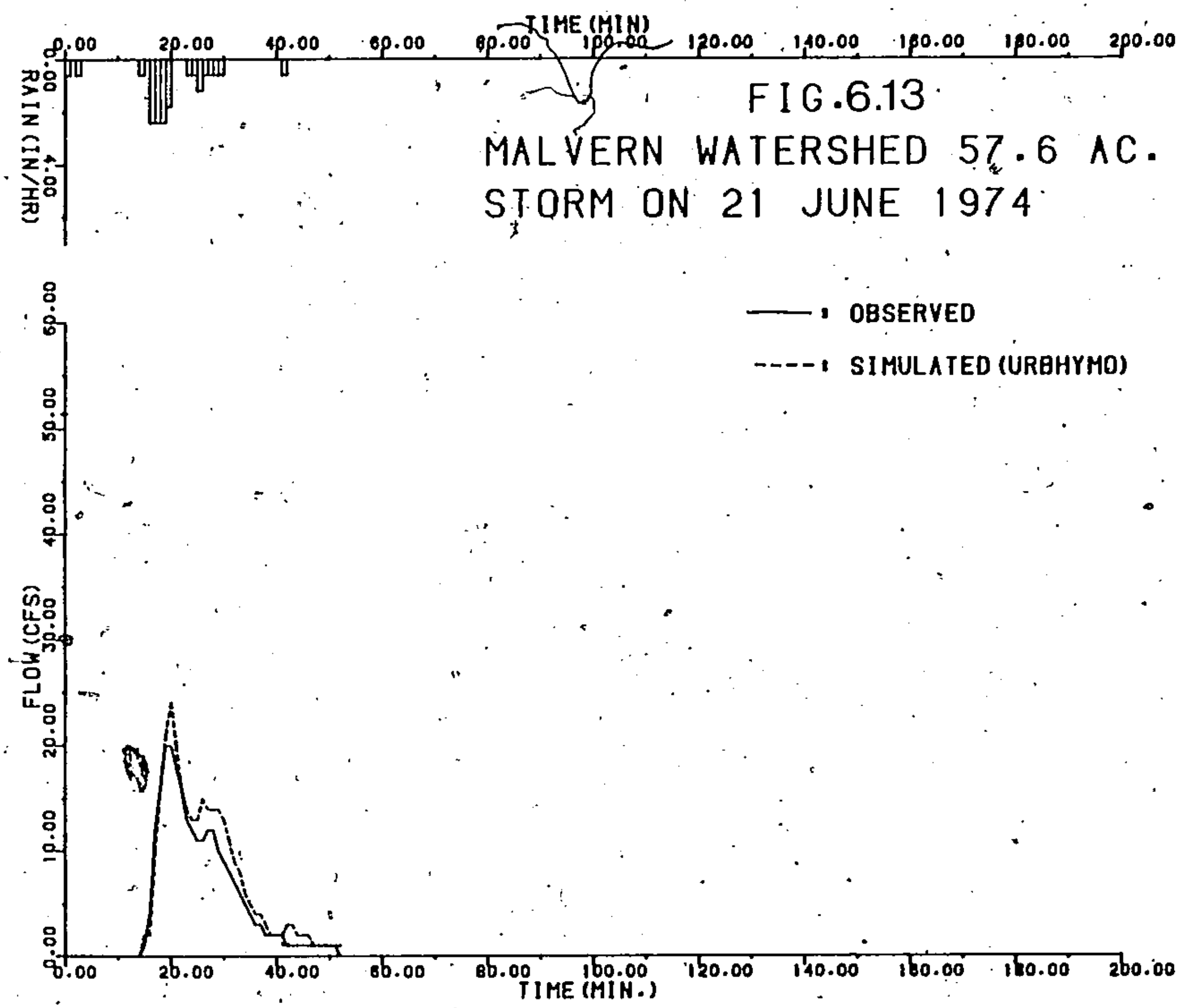


FIG.6.13
MALVERN WATERSHED 57.6 AC.
STORM ON 21 JUNE 1974

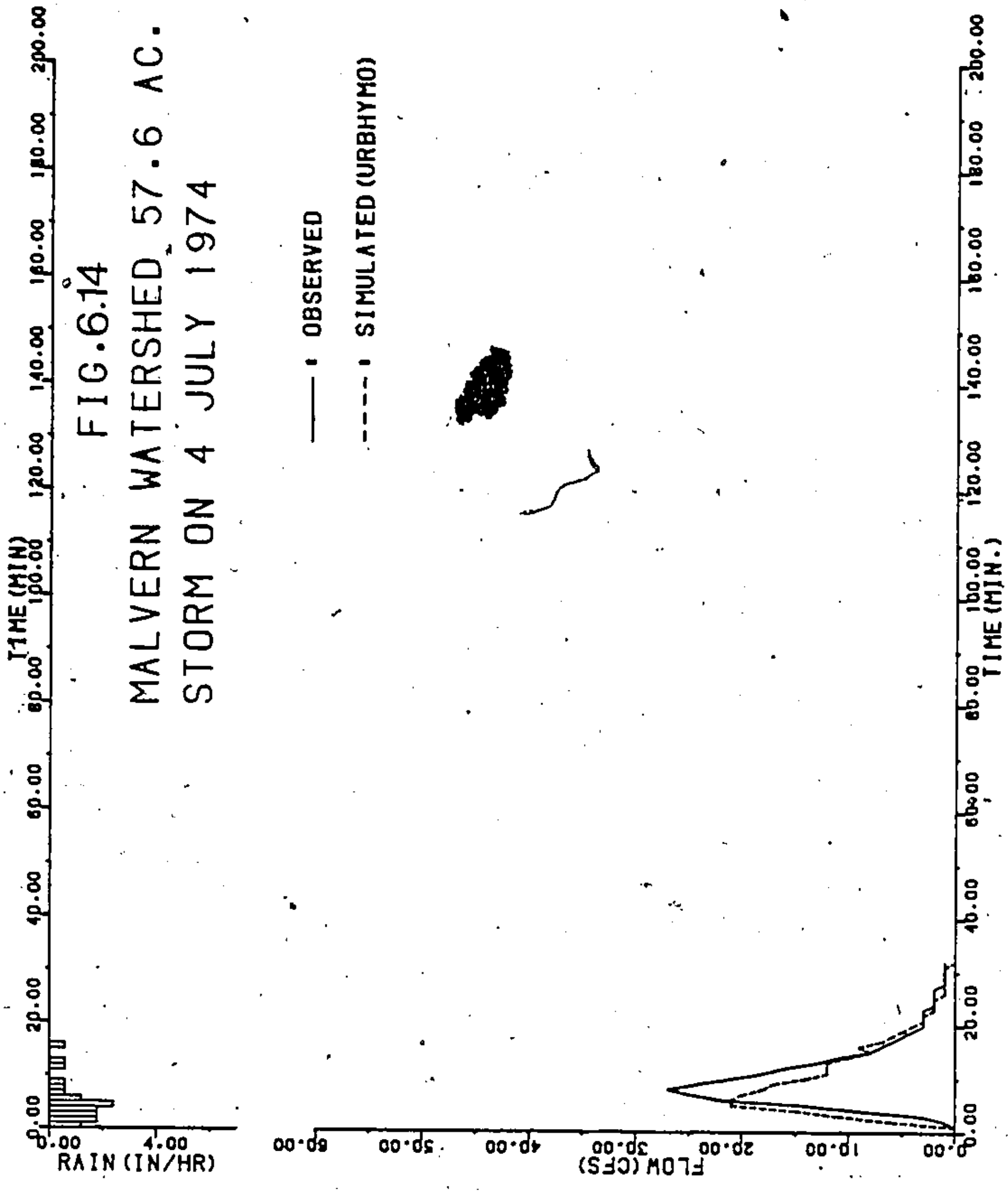
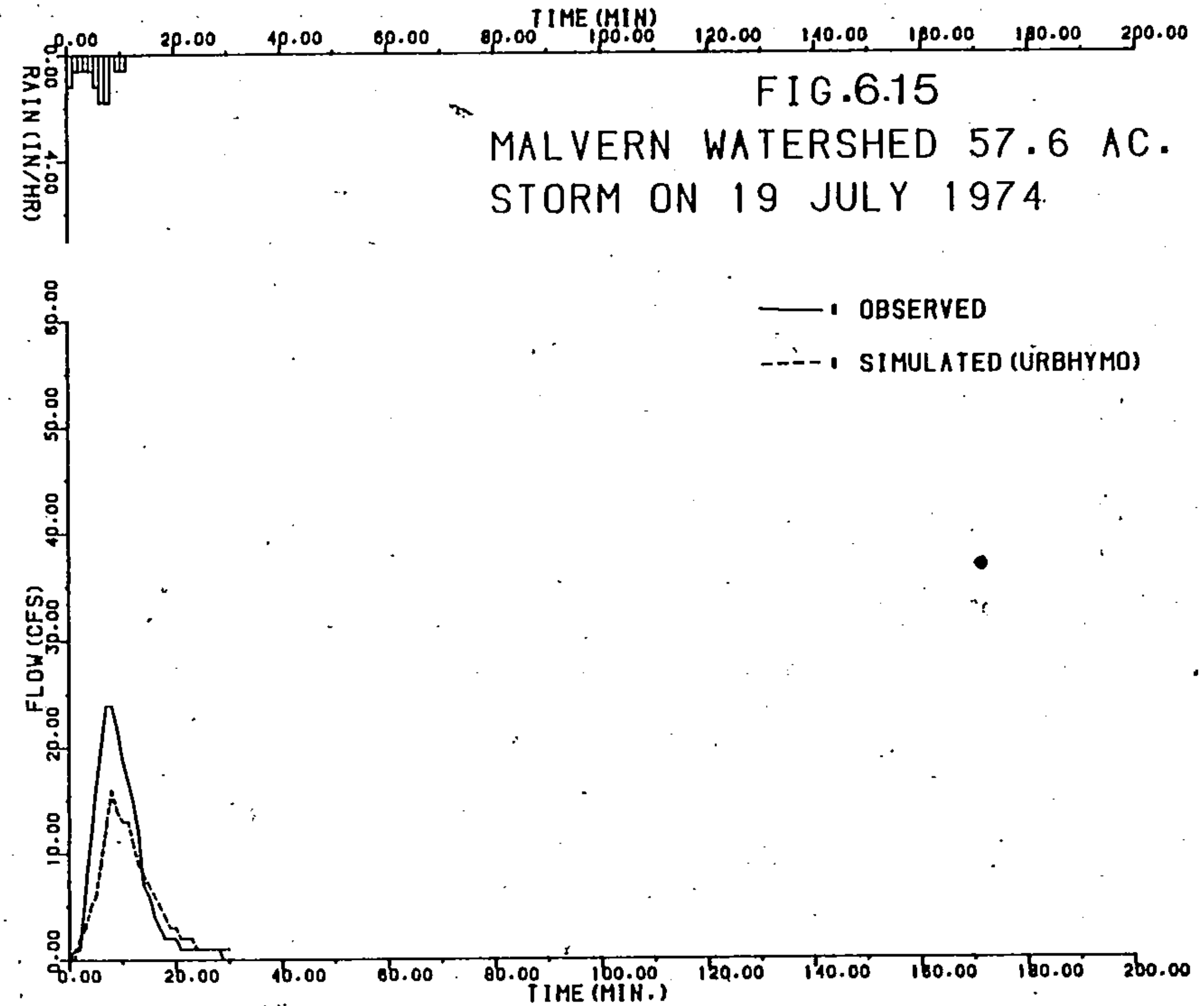


FIG. 6.14
MALVERN WATERSHED 57.6 AC.
STORM ON 4 JULY 1974

FIG.6.15
MALVERN WATERSHED 57.6 AC.
STORM ON 19 JULY 1974



the sewer system while URBHYMO is applied on a lumped basis. The times to peak are also simulated very well, being within 10 % of the observed values for six of the seven storms. Some multi-peaked hydrographs are simulated as well.

For the Malvern watershed, the results show that for five of the six storms, the simulated peak flows are within 25 % of the observed peak flows. Two of these storms are within 10 % of the observed peak flows. The storm of 19 July, 1974 (fig.6.15) is highly underpredicted, with the simulated peak flow being only 68 % of the observed peak flow. The data reveals that for this storm, the simulated runoff volume is much lower than the observed runoff volumes, being only 75 % of it. It is not clear where the extra runoff is coming from. Since the storm is of low intensity (peak intensity is less than 2 in./hr.) there is not expected to be any runoff from the pervious areas. The six storms that are simulated are plotted and compared with the observed hydrographs in figs.6.10 - 6.15. The fit of the hydrographs is generally considered to be good. The times to peak are also simulated well. Four of the six simulated hydrographs have times to peak that are within 10 % of the observed times.

6.5 FURTHER RESEARCH

6.5.1 Test for URBHYMO as a design model

Watt and Kidd (1975) have developed a design-oriented urban runoff model called QUURM which utilized the linear reservoir to simulate the inlet hydrograph and a time-offset method to perform the channel-routing. However no tests are done on how to predict K from watershed and/or storm characteristics and this limited its utility. Burke and Gray (1979) in their assessment of several methods of storm sewer design felt that the IUH method of SDR has good potential for further development as a design model. They have suggested that if there is an improved method for determining the rainfall excess instead of the CN procedure and if the model is coupled with a suitable routing technique the method might be an alternative to ILLUDAS. Since some of the relations for K are derived from small areas, and Horton's infiltration procedure has been incorporated into URBHYMO, it is decided to test URBHYMO as a design model.

A test watershed of 25 acres made up of 2.5 ac. inlet areas is used to test for HYMO's channel routing procedure by comparison with SWMM-RUNOFF's gutter routing procedure. The runoff for a 1:5 year design storm is simulated by using SWMM with detailed routing. The SWMM hydrograph for the 2.5 acre inlets is then routed to the outlet by the variable travel time (VTT) routing subroutine. Open channel gutters are used instead of pipes. The bottom width of the gutter is taken to be the same as that of the diameter of the pipe with a side slope of 2:1 (vertical/horizontal). It is assumed that the gutters have a Manning's n of 0.013. It can be seen in fig.6.16 that the hydrographs from the two different routing procedures are very close. This demonstrates that the VTT channel routing in HYMO is adequate for routing the flows from small urban areas. The runoff for the 2.5 ac. inlets are then simulated by URBHYMO and routed by the VTT channel routing. The resulting outflow hydrograph is also plotted on fig.6.16. It is found to compare very well with the other 2 hydrographs. This demonstrates that URBHYMO can be used for simulating small inlet hydrographs

URBHYMO is applied to simulate the subwatershed hydrographs in Test Area B (fig. 3.3) with detailed routing performed. The results are found to compare very well with the detailed SWMM simulations (fig. 3.4). This demonstrates that URBHYMO may be used as an alternative to SWMM-RUNOFF as a design-oriented model for urban watersheds. However further tests in applying URBHYMO for small watersheds with detailed routing is still required to confirm this.

URBHYMO has the potential to be a design-oriented model if further improvements are made to the channel routing procedure. The specially-weighted finite difference formulation of the kinematic wave model for channel routing (Kassem,1982) is being considered. The method can be used for pipe routing or open channel flow.

6.5.2 Larger Watersheds

The methodology that is developed can be used to extend the watershed size tested up to 5 square km. For larger watersheds, the parallel cascades model may also be tested.

For rural watersheds, where rainfall losses are sensitive, improvements in the CN procedure by considering a variable initial abstraction correlated with the antecedent precipitation index API (Jobin,1981), may be incorporated into HYMO. Other unit hydrograph models such as Nash's model may also be tested.

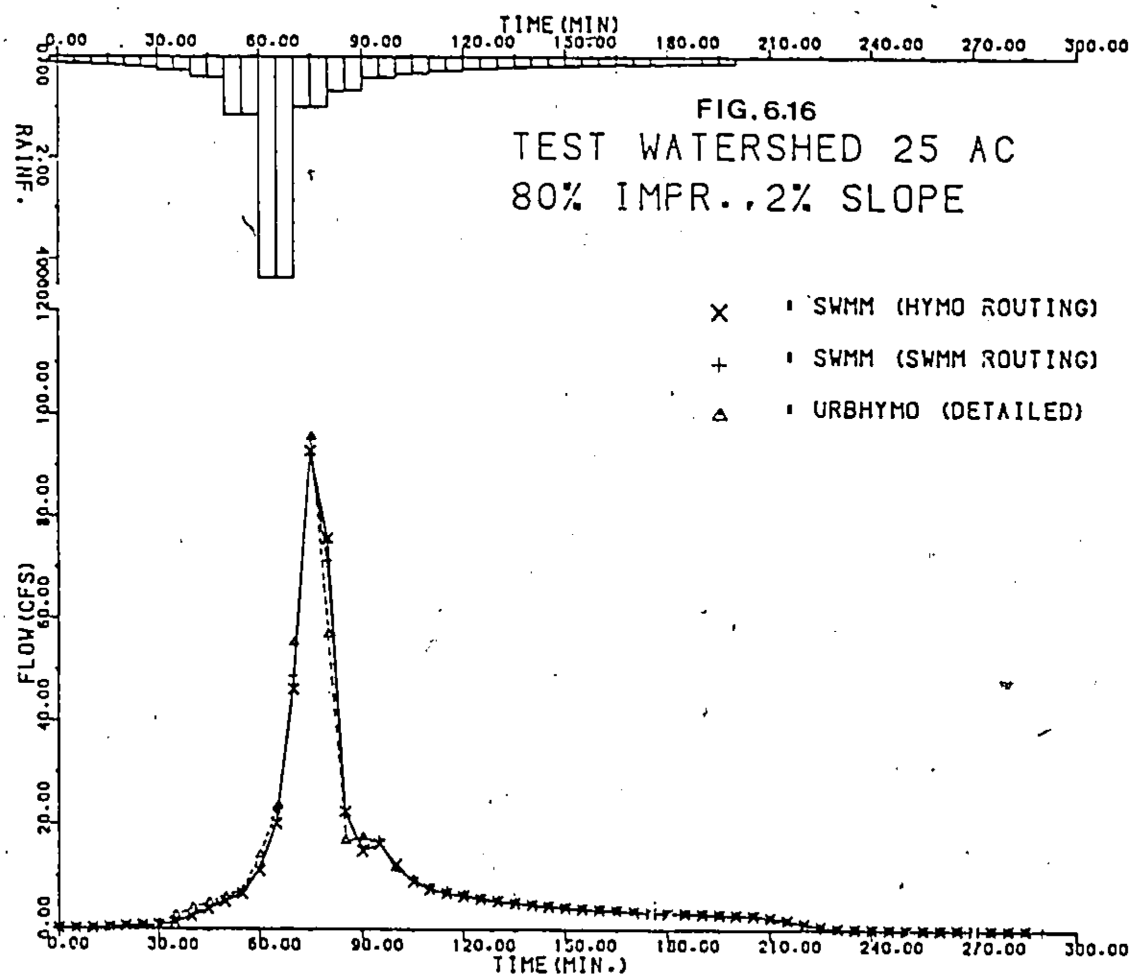


FIG.6.16

Chapter VII
CONCLUSIONS

The conclusions derived from this study can be summarized as follows:

1. A review of hydrologic planning studies has indicated that the present Canadian practice of using HYMO with empirical relations for the K and tp parameters is not adequate for urbanizing watersheds and gives significant discrepancies in flows when compared with detailed design models.
2. A procedure has been developed, by using mathematical watersheds, for the comparison of conceptual models in urban watersheds under different watershed conditions such as imperviousness levels.
3. Eight relations for the storage coefficient of the Single Linear Reservoir model have been tested. It is found that only two empirical relations derived by Desbordes and Sarma, Delleur, Rao (SDR) can predict the peak flows with discrepancies of less than 25 %, under limited conditions such as larger storms and watersheds of less than 100 acres (40 ha). A version of the single linear reservoir model and the relation derived by SDR has been programmed on a calculator.

4. A series of tests with a proposed model which is composed of two linear reservoirs in parallel (2PLR) has shown that it is more appropriate for urban watersheds. It is found that under the testing conditions, the model can simulate accurate peak flows with discrepancies of less than 20 % as compared to the detailed SWMM simulations without calibration.
5. For the 2PLR model, of the relations for the storage coefficients of the linear reservoirs which were tested under the study conditions, those that produce the best results are derived from the time to equilibrium obtained from kinematic wave theory. This model can be used for small inlet areas of 2 acres as well as larger areas of up to 300 acres.
6. The performance of the model for predicting the outflow hydrographs from urban watersheds has been validated using real measurements from two residential watersheds, Gray Haven in Baltimore, Maryland and Malvern in Burlington, Ontario.
7. The 2PLR model has been incorporated into the HYMO structure as a ready-to-use package. It will be known as URBHYMO. Since URBHYMO has been interfaced with the Rosenbrock optimization scheme, this will enable the model parameters to be calibrated automatically where rainfall-runoff data is available.

8. Although the model is chiefly for planning purposes a limited investigation indicates that it can also be used for design purposes. This can be done by simulating the inlet hydrographs and routing with the variable travel time (VTT) routing procedure in HYMO. Further research needs to be done in this direction.

The main advantages of URBHYMO as compared to other models are the following:

- for planning studies the results are found to be acceptable even without calibration;
- the model parameters can be related to watershed and storm characteristics which are easily determined. The watershed characteristics are length L, slope S and surface roughness n. The storm characteristic is the peak rainfall excess intensity.
- it gives better results than the SLR model for a broader range of watershed conditions;
- it requires fewer parameters than the parallel cascades model of Wittenberg and Diskin et al.

The main limitation of the model is that it has only been tested for watersheds of up to 300 acres (121 ha). The introduction of URBHYMO will reduce the problems due to inconsistency between the flows simulated during the planning and design stages of drainage studies.

REFERENCES

1. Ahmad,M., "Methodology for 'lumped' SWMM Modelling," Proceedings SWMM User's Group Meeting, Toronto, Ontario, June 19-20, 1980.

2. Alley,W.M., and Veenhuis,J.E., "Determination of Basin Characteristics for an Urban Distributed Routing Rainfall-Runoff Model," Proceedings, SWMM User's Group Meeting, EPA-600/9-79-026, May 24-25, 1979.

3. American Iron and Steel Institute, "Modern Sewer Design," Canadian Edition, First edition, 1980.

4. Aron,G.,Miller,A.C., and Lakatos,D.F., "Infiltration Formula Based on SCS Curve Number," Journal of the Irrigation and Drainage Division, ASCE, Vol.103, No.IR4, December 1977, pp. 410-426.

5. Burke,C.B., and Gray,D.D., "A Comparative Application of Several Methods for the Design of Storm Sewers," Tech. Report No.118, Water Resources Research Center, Purdue University, West Lafayette, Indiana, August 1979.

6. Cheung,P., "Notes on Update of EPA-SWMM", Technical Memorandum, Dept. of Civil Engineering, University of Ottawa, February 1980.

7. Cheung,P., and Kassem,A.M., "Computer Workshop on SWMM-RUNOFF and WRE-TRANSPORT Models," in short course on "Storm Water Management for Runoff Control in New Developments," Dept. of Civil Engineering, University of Ottawa, October 16-18, 1979.

8. Cheung,P.,Sautier,J.L., and Wisner,P.E., "Rapport Hydrologique sur le Bassin de la Seymaz," (non-publie), Departement de Genie Civil, Universite d'Ottawa, Aout 1981.

9. Chow,V.T., "Runoff," Section 14, Handbook of Applied Hydrology, McGraw Hill, 1964.

10. Clark,C.O., "Storage and the unit hydrograph," Transactions, ASCE, Vol.110, 1945, pp.1419-1446.

11. Dawdy,D.R.,Schaake,J.C.,Jr. and Alley,W.M., "User's Guide for Distributed Routing Rainfall-Runoff Model," Water Resources Investigations, 78-90, U.S. Geological Survey, 1978, 146 pp.

12. DeCoursey,D.G., "A Runoff Hydrograph Equation," U.S. Dept. of Agriculture, Agricultural Research Service, ARS 41-116, 1966, 23 pp.

13. Diskin,M.H., "A Basic Study of the Linearity of the Rainfall-Runoff Process in Watersheds," Ph.D. Thesis, University of Illinois, Urbana, Illinois, 1964.

14. Diskin,M.H., "Nonlinear Hydrologic Models," International Symposium on Rainfall-Runoff Modelling, Mississippi State University, Mississippi State, Mississippi, May 18-21, 1981.

15. Diskin,M.H.,Ince,S., and Oben-Nyarko,K.,"Parallel Cascades Model for Urban Watersheds," Journal of the Hydraulics Division, ASCE, Vol.104, No. HY2, February 1978, pp.261-276.

16. Diskin,M.H., and Simon,E., "A Procedure for the Selection of Objective Functions for Hydrologic Simulation Models," Journal of Hydrology, Vol. 34, 1977, pp. 129-149.

17. Delleur,J.W., "Mathematical Modeling in Urban Hydrology," International Symposium on Rainfall-Runoff Modeling, Mississippi State University, Mississippi State, Mississippi, May 18-21, 1981.

18. Delleur,J.W.,Rao,A.R., and Hossain,A., "On Modeling the Runoff Process in Urban Areas," National Symposium on Urban Hydrology and Sediment Control, University of Kentucky, Lexington, Kentucky, July 28-31, 1975.

19. Delleur,J.W., and Vician,E.B., Discussion of "Time in Urban Hydrology," Journal of the Hydraulics Division, ASCE, Vol. No. HY5, September 1966, pp. 243-251.

20. Desbordes,M., "Urban Runoff and Design Storm Modeling," in "Urban Storm Drainage," Proceedings of the International Conference, University of Southampton, April, 1978, ed. P.R.Helliwell, Pentech Press, London.

21. Dooge,J.C.I., "A General Theory of the Unit Hydrograph," Journal of Geophysical Research., Vol.64, No.1, 1959, pp.241-256.

22. Dooge,J.C.I., "Linear Theory of Hydrologic Systems," Technical Bulletin No. 1468, Agricultural Research Service, U.S. Dept. of Agriculture, October 1973, 327 pp.

23. Dooge, J.C.I., "Problems and Methods of Rainfall-Runoff Modelling," in "Mathematical Modeling in Surface Water Hydrology," edited by Ciriani,T.A.,Maione,U., and Wallis,J.R., John Wiley and Sons, 1977.

24. Espey,W.H.,Jr.,Altman,D.G., and Graves,C.B.,Jr., "Nomographs for Ten-Minute Unit Hydrographs for Small Urban Watersheds," Appendum 3 of the ASCE Program report "Urban Runoff Control Planning," June, 1977.

25. Falk,J., and Niemczynowicz,J., "Characteristics of the Above-Ground Runoff in Sewered Catchments," Proceedings of the International Conference, University of Southampton, April, 1978, Ed. P.R.Helliwell, Pentech Press, London.

26. Golding,B.L., Discussion of "Runoff Curve Numbers with Varying Site Moisture," Journal of the Irrigation and Drainage Division, ASCE, Vol.105, No.IR4, December 1979, pp.441-442.

27. Golding,B.L., "Hydrograph Synthesis by the HNV-SBUH Method utilizing a Programmable Calculator," Proceedings, SWMM User's Group Meeting, Toronto, Ontario, June 19-20, 1980.

28. Han,J., and Rao.A.R., "Optimal Parameter Estimation and Investigation of Objective Functions of Urban Runoff Models," Tech. Report 135, Water Resources Research Center, Purdue University, West Lafayette, Indiana, September, 1980.

29. Hawkins,R.H., Discussion of "Infiltration Formula based on SCS Curve Number," Journal of the Irrigation and Drainage Division, ASCE, Vol.104, No.IR4, December 1978, pp.464-467.

30. Heeps,D.P., and Mein,R.G., "Independent Comparison of Three Urban Runoff Models," Journal of the Hydraulics Division, ASCE, Vol.100, No.HY7, July 1974, pp.995-1009.

31. Ibbitt,R.P., and O'Donnell,T., "Mathematical Models of Catchment Behaviour," Journal of the Hydraulics Division, ASCE, Vol.91, No.HY4, July 1965, pp.123-139.

32. Institute of Hydrology, "Rainfall-Runoff Processes over Urban Surfaces," Proceedings of an International Workshop, Institute of Hydrology, Wallingford, Oxon., U.K., April 1978.

33. Izzard,C.F., "Hydraulics of runoff from developed surfaces," Proceedings, Highway Research Board, Vol.26, 1946, pp.129-150.

34. James F. MacLaren Ltd., "Review of Canadian Design Practice and Comparison of Urban Hydrologic Models," Research Report No.26,Canada-Ontario Agreement on Great Lakes Water Quality, Project No.74-8-31, October 1975.

35. James F. MacLaren Ltd., "Report on Hydrologic Model Study, Humber, Don and Rouge Rivers, Highland, Duffin, Petticoat and Carruther's Creeks for the Metropolitan Toronto and Region Conservation Authority," October 1979.

36. Jobin,D., "Application et Modification de la Methode du Taux D'Ecoulement du S.C.S. pour les Evenements Frequents," Dept. de Genie Civil, University of Ottawa, September 1981.

37. Kassem, A.K., "Development and Application of A Simultaneous Routing Model for Dual Drainage Systems," Ph.D Thesis, Dept. of Civil Engineering, University of Ottawa, January 1982.

38. Kulandaiswamy,V.C., "A Basic Study of the Rainfall Excess-Surface Runoff Relationship in a Basin System," Ph.D. Thesis, University of Illinois, Urbana, Illinois, 1964.

39. Machmeier,R.E., and Larson,C.L., "Runoff Hydrographs for Mathematical Watershed Model," Journal of the Hydraulics Division, ASCE, Vol.94, No.HY6, Proc.Paper 6248, November 1968, pp.1453-1474.

40. MacPherson,M.B., "Overview of Urban Runoff Tools of Analysis," EP 79-R-20, GREMU 79/01, Ecole Polytechnique de Montreal, April 1979.

41. Marsalek,J., Discussion of "Dimensionless Inlet Hydrograph Model," Journal of the Hydraulics Division, ASCE, No.HY12, December 1976, pp. 1773-1776.

42. Marsalek,J., "Malvern Urban Test Catchment, Vol.I," Research Report No.57, Canada-Ontario Agreement on Great Lakes Water Quality; Project No.73-3-12, 1977.

43. Marsalek,J., "Malvern Urban Test Catchment, Vol.II," Research Report No.95, Canada-Ontario Agreement on Great Lakes Water Quality, Project No.73-3-12, 1979.

44. Mather,C., "Personal Communication," 1980.

45. McCuen,R.H., Discussion of "Considerations regarding the Application of SCS TR-55 Procedures for Runoff Computations," Proceedings, SWMM User's Group Meeting, September 1981.

46. Miller,C.R., and Viessman,W.,Jr., "Runoff Volumes from Small Urban Watersheds," Water Resources Research, Vol.8., No.2, 1972.

47. Morgali,J.R., and Linsley,R.K., "Computer Analysis of Overland Flow," Journal of the Hydraulics Division, ASCE, Vol.95, No.HY3, May 1965, pp.81-100.

48. Nash,J,E., "The Form of the Instantaneous Unit Hydrograph," International Association of Scientific Hydrology, Pub.45, 3(1957), pp.114-121.

49. Nash,J.E., "Systematic Determination of Unit Hydrograph Parameters," Journal of Geophysical Research, Vol.64:111-115, 1959.

50. Nash,J.E., "A Unit Hydrograph Study with Particular Reference to British Catchments," Proceedings, Institution of Civil Engineers, Vol.17, November 1960, pp.249-282.

51. National Engineering Handbook, Section 4, Hydrology, U.S. Dept. of Agriculture, Soil Conservation Service, U.S. Government Printing Office, Washington,D.C., 1971.

52. Neumann,W., "Zur Prognose des Niederschlag - Abflussverhalten in urbanen Einzugsgebieten," Wasserwirtschaft, 67(4), 1977.

53. O'Kelly,J.J., "The Employment of Unit-Hydrographs to determine the flows of Irish Arterial Drainage Channels," Inst. Civil Engin. (Ireland) Proc. 4(3): 365-412.

54. Overton,D.E., "Route or Convolute?" Water Resources Research, Vol.6, No.1, February 1970, pp.43-52.

55. Overton,D.E., and Meadows,M.E., "Stormwater Modeling," Academic Press, New York, N.Y., 1976.

56. Perks,A., "The Development of Storm Drainage Modelling in Canada," in "Urban Storm Drainage," Proceedings of the International Conference, University of Southampton, April 1978, ed. P.R.Helliwell, Pentech Press, London.

57. Pedersen,J.T.,Peters,J.C., and Helweg,O.J., "Hydrographs by Single Linear Reservoir Model," Journal of the Hydraulics Division, ASCE, Vol.106, No.H5 , Proc.Paper 15430, May 1980, pp.837-852.

58. P'ng,C.E.,Liang,W.Y.,Ray,S.K., and Moss,D., "The Application of HYMO,SWMM and ILLUDAS for the Study of Urbanization in a Watershed," Report submitted for the course CVG 5172, Dept. of Civil Engineering, University of Ottawa, Ottawa, Ontario, November 1980, 163 pp.

59. P'ng,C.E.,Wisner,P.E., and Cheung,P., "Preliminary Assessment of a Simplified Linear Reservoir Model for Programmable Calculators (The Santa Barbara Urban Hydrograph Method (HNV-SBUH))," IMPSWM Progress Report No.11 (unpublished), Dept. of Civil Engineering, University of Ottawa, May 1980, 30 pp.

60. Proctor and Redfern Ltd., and James F. MacLaren Ltd., "Storm Water Management Model Study Volume 1," Research Report.47, Canada-Ontario Agreement on Great Lakes Water Quality, Project No.73-5-10, September 1976.

61. Purenne,P.,Courchesne,G., and Leclerc,G., "Simplified Segmentation of an Urban Catchment for Urban Runoff Simulation with SWMM," Proceedings, SWMM User's Group Meeting, EPA 600/9-79-026, May 24-25, 1979.

62. Ragan,R.M.,Root,M.J., and Miller,J.F., "Dimensionless Inlet Hydrograph Model," Journal of the Hydraulics Division, ASCE, Vol.101, No.HY9, Proc.Paper 11580, September 1975, pp.1185-1195.

63. Rao.A.R.,Delleur,J.W., and Sarma,P.B.S., "Conceptual Hydrologic Models for Urbanizing Basins," Journal of the Hydraulics Division, ASCE, Vol.92, No.HY7, Proc.Paper 9024, July 1972, pp.1205-1220.

64. Rosenbrock,H.H., "An Automatic Method of finding the greatest or least value of a function," Computer Journal(3), 1960.

65. Rossmiller,R.L., "The Rational Formula Revisited," International Symposium on Urban Storm Runoff, University of Kentucky, Lexington, Kentucky, July 28-31, 1980.

66. Sarma,P.B.S.,Delleur,J.W., and Rao,A.R.,."A Program in Urban Hydrology.Part II.An Evaluation of Rainfall-Runoff Models for Small Urbanized Watersheds and the Effect of Urbanization on Runoff," Tech. Report No.9, Water Resources Research Center, Purdue University, West Lafayette, Indiana, October 1969.

67. Sarma,P.B.S.,Delleur,J.W., and Rao,A.R., "Comparison of Rainfall-Runoff Models for Urban Areas," Journal of Hydrology, Vol.18, 1973, pp.329-347.

68. Schaake,J.C.,Jr.,Geyer,J.C., and Knapp,J.W., "Experimental Examination of the Rational Method," Journal of the Hydraulics Division, ASCE, Vol.93, No.HY6, Proc. Paper 5607, November 1967, pp.353-370.

69. Schulz,E.F., and Lopez,O.G., "Determination of Urban Watershed Response Time," Hydrology Paper No.71, Colorado State University, Fort Collins, Colorado, December 1974.

70. Sherman,L.K., "Stream flow from rainfall by the Unit-graph Method," Engineering News-Record, Vol.108, April 7, 1932, pp.501-505.

71. Soil Conservation Service, "Urban Hydrology for Small Watersheds," Technical Release No.55, U.S. Dept. of Agriculture, January 1975.

72. Soroosian,S., "Comparison of Two Direct Search Algorithms used in Calibration of Rainfall-Runoff Models," Preprints, IFAC Symposium on Water and Related Land Resources Systems, Cleveland, Ohio, May 28-31, 1980, Pergamon Press, pp.441-449.

73. Stubchaer,J.M., "The Santa Barbara Urban Hydrograph Method," National Symposium on Urban Hydrology and Sediment Control, University of Kentucky, Lexington, Kentucky, July 28-31, 1975.

74. Terstriep.M.L., and Stall,J.B., "The Illinois Urban Drainage Area Simulator, ILLUDAS," Illinois State Water Survey Bulletin 58, 1974.

75. Urban Drainage Policy Committee, "Proposed Model Policies for Urban Drainage Management," Report of the Urban Drainage Policy Committee to the Urban Drainage Subcommittee, Canada-Ontario Agreement on Great Lakes Water Quality, December 1978.

76. Viessman,W.,Jr., Discussion of 'Time in Urban Hydrology', Journal of the Hydraulics Division, ASCE, Vol.92, No.HY5, September 1966(a), pp. 242-243.

77. Viessman,W.,Jr., "The Hydrology of Small Impervious Areas," Water Resources Research, Vol.2, No.3, 1966(b) , pp.405-412

78. Viessman,W.,Jr., "Runoff Estimation for Very Small Drainage Areas," Water Resources Research, Vol.4, No.1, February 1968.

79. Viessman,W.,Jr.,Keating,W.R., and Srinivasa,K.N., "Urban Storm Runoff Relations," Water Resources Research, Vol.6, No.1, February 1970, pp.275-279.

80. Wanielista,M.P., "Stormwater Management,Quantity and Quality," Ann Arbor Science Publishers Inc., 1979.

81. Watkins,L.H., "The Design of Urban Sewer Systems," Road Research Technical Paper No.55, Department of Scientific and Industrial Research, London, 1962.

82. Watt,W.E., and Kidd,C.H.R., "QUURM - A Realistic Urban Runoff Model," Journal of Hydrology, Vol.27, 1975, pp.225-235.

83. Wenzel,H.G.,Jr., and Terstriep,M.L., "Sensitivity of Selected ILLUDAS Parameters," Illinois State Water Survey, Urbana, Illinois, Contract Report 178, August 1976.

84. Wenzel,H.G.,Jr., and Voorhees,M.L., "Adaption of ILLUDAS for Continuous Simulation," Journal of the Hydraulics Division, ASCE, Vol.106, No.HY11, Proc.Paper 15840, November 1980, pp.1795-1812.

85. Willeke,G.E., "Time in Urban Hydrology," Journal of the Hydraulics Division, ASCE, Vol.92, No.HY1, Proc.Paper 4615, January 1966, pp.13-29.

86. Williams,J.R., "Runoff Hydrographs from Small Texas Blacklands Watersheds," U.S. Dept. of Agriculture, Agricultural Research Service, ARS 41-143, October 1968, 24 pp.

87. Williams,J.R., "Concept of a Technique for an analysis of Watershed Runoff Events," Proceedings of the Second International Symposium in Hydrology, Colorado State University, Fort Collins, Colorado, September 1972.

88. Williams,J.R., and Hann,R.W., "HYMO, A Problem-Oriented Computer Language for Hydrologic Modeling," User's Manual, ARS-S-9, U.S. Dept. of Agriculture, May 1973.

89. Wisner,P.E., and Cheung,P., "Application of the HYMO Model for Runoff Control Studies, Comparison with SWMM, Limitations and Recommendations," IMPSWM Progress Report No.2 (unpublished), Dept. of Civil Engineering, University of Ottawa, August 1979, 29 pp.

90. Wisner,P.E., and Gupta,S., "Validity of Chicago Design Storms," unpublished report, Dept. of Civil Engineering, University of Ottawa, 1981.

91. Wisner,P.E.,Gupta,S., and Kassem,A.K., "Considerations regarding the Application of SCS TR-55 Procedures for Runoff Computations," Proceedings, SWMM User's Group Meeting, Toronto, Ontario, June 1980.

92. Wisner,P.E., and Kassem,A.K., "Review and Comparison of Routing Methods in Storm Water Modelling," IMPSWM Progress Report No.13 (unpublished), Dept. of Civil Engineering, University of Ottawa, October 1980, 48 pp.

93. Wisner,P.E., "Modelling Procedures for Master Drainage Plans and their Verification by Reviewing Agencies," 5th IMPSWM Meeting, University of Ottawa, Ottawa, Ontario, January 1981.

94. Wittenberg,H., "A Model to Predict the Effects of Urbanization on Watershed Response," National Symposium on Urban Hydrology and Sediment Control, University of Kentucky, Lexington, Kentucky, July 28-31, 1975.

95. Wu,I.P., "Design Hydrographs for Small Watersheds in Indiana," Journal of the Hydraulics Division, ASCE, Vol.89, No.HY6, November 1963, pp.35-66.

Appendix A

UNIT HYDROGRAPH THEORY AND LINEAR CONCEPTUAL MODELS

A.1 UNIT HYDROGRAPH THEORY

The unit hydrograph which was introduced by Sherman in 1932 is regarded as one of the major contributions to hydrology. It is defined as the hydrograph of direct runoff resulting from 1 unit of rainfall excess generated uniformly over the basin area at a uniform rate during a specified duration of time. Basically the unit hydrograph theory assumes that the surface runoff system is a lumped linear system. The other assumptions and limitations of the unit hydrograph have been well-reviewed in Chow (1964) and will not be repeated here. A unit hydrograph of duration Δt can be represented by $U(\Delta t,t)$ where t is any time after the beginning of rainfall excess. The ordinate of the direct runoff hydrograph for a given storm can be expressed as

$$Q(t) = \sum_{i=1}^{n} U[\Delta t,\ t-(i-1)\Delta t\ ]\ I_i \Delta t \qquad 1$$

A.1.1 Instantaneous Unit Hydrograph Theory

The concept of the instantaneous unit hydrograph or IUH, first introduced by Clark (1945), is a further development of the unit hydrograph. The IUH is defined as the hydrograph resulting from one unit of rainfall excess generated in zero time over the entire watershed. In other words it is the unit hydrograph that results from an infinitesimally small duration of rainfall excess. This is just a concept and does not occur in reality. Mathematically it is written as

$$U(0,t) = \lim_{\Delta t \to o} U(\Delta t,t) \qquad 2$$

The ordinates of the instantaneous unit hydrographs have the dimension of discharge rate per unit depth where $U(\Delta t,t)$ is the unit hydrograph of duration Δt. An IUH can be converted to a unit hydrograph of any duration Δt by

$$U(\Delta t,t) = \frac{1}{\Delta t}\int_{t-\Delta t}^{t} U(0,t)\, dt \qquad 3$$

For a linear system with an input of a unit impulse function or a unit pulse function the output is defined as the unit impulse response and unit pulse response respectively. A unit pulse function is defined as

$$\delta(t,b) = \left[\begin{array}{ll} 1/b & 0 \le t \le b \\ 0 & \text{otherwise} \end{array} \right] \qquad 4$$

When b approaches 0, the equation becomes a unit impulse function $\delta(t)$ which is also known as a Dirac-Delta function.

$$\delta(t) = \left[\begin{array}{ll} 0 & : \text{if } t \neq 0 \\ \infty & \text{if } t = 0 \end{array} \right] \quad (5)$$

$$\int_{-\infty}^{\infty} \delta(t)\, dt = 1 \quad (6)$$

The IUH is the impulse response function of the surface runoff system. The input-output relation of a lumped, linear system can be expressed by the convolution or Duhamel integral.

$$Q(t) = \int_{0}^{t' \leq t_o} U(t-\tau) I(\tau) d\tau \quad (7)$$

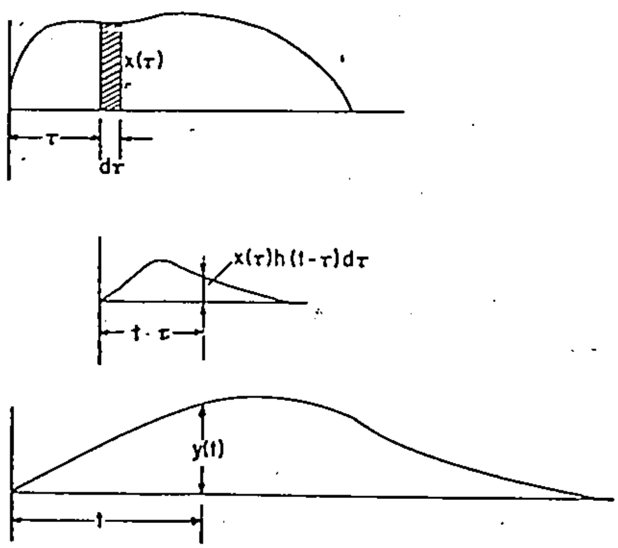
where $Q(t)$ = output

$I(t)$ = input

$U(t-\tau)$ = impulse response or IUH

t_o = duration of rainfall excess

$t' = t$ when $t \leq t_o$ and $t' = t_o$ when $t > t_o$.



Convolution of inflow with IUH.

If the input and output i.e. rainfall excess and direct runoff in the convolution integral are measured in the same units, then the ordinates of the IUH must have a dimension of [$time^{-1}$].

$$\int_0^\infty u(t)\, dt = 1.0 \qquad 8$$

$$\int_0^\infty u(t)t\, dt = t_L \qquad 9$$

where t_L is the time lag of the IUH. The time lag is also equal to the time interval between the centroids of the rainfall excess and the direct runoff.

A.2 LINEAR CONCEPTUAL MODELS

The linear conceptual models are made up of combinations of linear hydrologic components such as (1) linear reservoir and (2) linear channel which account for storage and translation effects respectively. These components are described below.

a. Linear Reservoir

A linear reservoir is one in which the storage S is directly proportional to the outflow Q.

$$S = KQ \qquad 10$$

When combined with the continuity equation (eqn.1.2), this results in the linear differential equation

$$I - Q = K\frac{dQ}{dt} \qquad 11$$

For a linear differential equation

$$a\frac{dx}{dt} + x = F(t) \qquad 12$$

If $F(t) = M$ the general solution is

$$x = M + Ce^{-t/a} \qquad 13$$

Therefore the solution for eqn. 11 is

$$Q = I + Ce^{-t/k} \qquad 14$$

When I is the inflow (effective precipitation) to the reservoir for the boundary condition $t=0$, $Q=0$

$$C = -I$$

$$Q = I - Ie^{-t/k} \qquad 15$$

$$= I(1-e^{-t/k})$$

For $t = \infty$ the above equation gives $Q = I$ which means that the outflow approaches an equilibrium condition becoming equal to inflow.

If $F(t) = 0$ the general equation is

$$x = Ce^{-t/a} \qquad 16$$

If the inflow ceases at some time t after the beginning of runoff i.e. $I = 0$, then the solution for eqn. 11 is

$$Q = Ce^{-t/k} \qquad 17$$

When $t_1 = 0$, $Q_1 = C$

$$Q = Q_1 e^{-\tau/k} \qquad 18$$

where $\tau = t - t_1$, i.e. equal to the time since inflow terminated.

For an instantaneous inflow which fills the reservoir S in $t = 0$, eqn. 10 gives $Q = S/K$

Eqn. 18 gives the outflow as

$$Q = \frac{S}{K} e^{-t/k} \qquad 19$$

For a unit input or S = 1 the IUH of the linear reservoir is therefore

$$U(t) = \frac{1}{K} e^{-t/k} \qquad 20$$

which is represented by the hydrograph for the outflow from the first reservoir in Fig.A.1

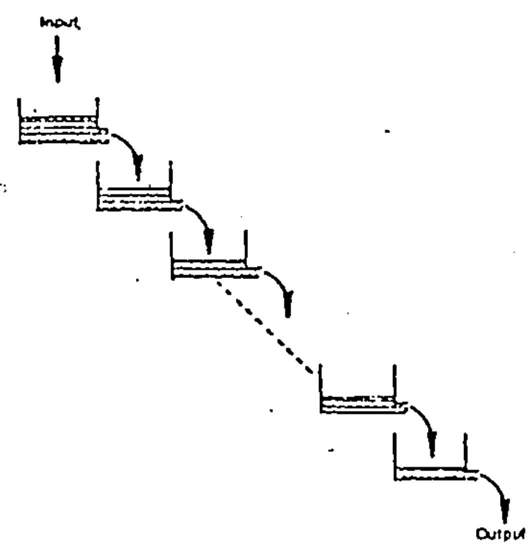


Fig.A.1 Representation of the Nash model

b. Linear Channel

The concept of a linear channel represents pure translation in a linear catchment. A linear channel is defined as a fictitious reach in which the time of travel T through the reach for a discharge Q of any magnitude is constant. At any point in the reach,

$$A = CQ \tag{21}$$

where A = cross-sectional area

Q = outflow

C = translation coefficient

In other words the rating curve is a linear relationship between discharge and area. The velocity is constant for all discharges at any point but may vary from point to point along the reach. When an inflow hydrograph is routed through a linear channel, the shape of the inflow hydrograph will remain the same but it will be lagged by the time of translation T. If the input function is f(t) then the output function is f(t-T). The outflow for an inflow to the linear channel of volume S and duration Δt is given by

$$Q = S\delta\ (t,\Delta t) \tag{22}$$

where $\delta\ (t,\Delta t) = \frac{1}{\Delta t}$

for $0 \leq \tau \leq t$ and $t = \tau + T$. It is zero otherwise. τ is the time measured from the beginning of the segment while T is the translation time. The Dirac-delta function $\delta(t)$ as Δt approaches zero is the IUH for the linear channel.

A watershed can be considered as a linear channel receiving spatially varied flow. The watershed can then be divided into 'n' subwatersheds by isochrones which are contours joining all points in the watershed which are separated from the outlet by the same translation time. Isochrones cannot cross one another, cannot close, and can only originate or terminate on the boundary of the watershed.

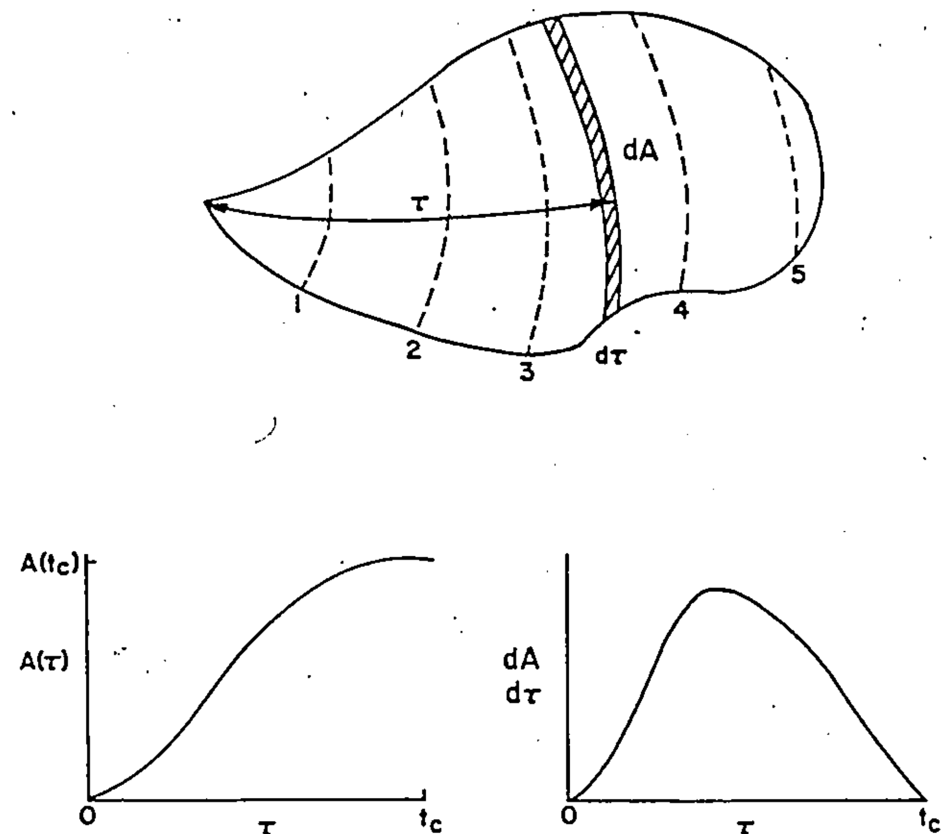


FIGURE A.2.—*Top*: Isochromes of travel time. *Bottom*: *Left*, time-area curve; *right*, time-area-concentration curve.

(Ref: Dooge, 1973)

Assume that the isochrones are separated from each other by equal travel times Δt. When the area enclosed by each succeeding isochrone to the outlet is plotted against the travel time, the time-area curve is obtained. If the derivative of the time-area curve is divided by the watershed area then it is known as the time-area-concentration curve which has a base-length equal to the time of concentration t_c. The time-area-concentration curve has ordinates directly proportional to the shape of the drainage area projected onto the main channel. The time-area-concentration curve $w(t)$ is given by

$$W(t) = \begin{cases} \frac{1}{A}\frac{dA}{dt} & 0 < t < t_c \\ 0 & \text{otherwise} \end{cases} \tag{23}$$

$$\int_0^\infty W(\tau).dt = 1.0 \tag{24}$$

For this case where only translation effects are considered, Dooge (1959) has shown that the IUH has the same shape as the time-area-concentration curve.

Clark (1945) has suggested that the IUH can be derived by routing the time-area-concentration curve through a single storage element such as a linear reservoir. O'Kelly (1955) has found that if the time-area-concentration curve is replaced by an isoceles triangle and then routed through a linear reservoir, the resulting IUH is not significantly different. This is due to the smoothing effect of the linear reservoir such that the variations in watershed characteristics are removed by routing.

c. Nash's Model

Nash (1957) has proposed that the IUH can be obtained by routing an instantaneous unit input through a cascade of equal linear reservoirs (fig.A.1). The IUH is given by

$$U(t) = \frac{1}{K\Gamma(n)} \left(\frac{t}{K}\right)^{n-1} e^{-t/k} \qquad 25$$

which represents a gamma distribution. For integers the gamma function (n) = (n-1)!. If n is not an integer then

$$\Gamma(n) = \int_0^\infty t^{n-1} e^{-t} dt \ , \qquad n > 0 \qquad 26$$

The parameters K and n in eqn. 25 can be evaluated by the method of moments (Nash,1959). Nash (1960) has applied the model to rural watersheds in Britain and correlated the moments with watershed characteristics. The first and second moments of the IUH about the origin t = 0 are respectively

$$M_1 = nK \qquad 27$$

$$M_2 = n(n+1)K^2 \qquad 28$$

The second central moment of the IUH is

$$MU_2 = nK^2 \qquad 29$$

The first moment M_1 represents the time lag of the centroid of the IUH. The time difference between the centroids of the rainfall excess hyetograph and the runoff hydrograph = $t_L = nK$.

The Nash model does not involve the concept of translation of flow. Dooge (1959) has attempted to develop a general conceptual model of the unit hydrograph made up of a series of linear reservoirs and linear channels. The linear channel is used to account for translation. The order of occurrence of the linear components do not matter since it is a linear process.

Appendix B

LISTING OF URBHYMO PROGRAM

```
      SUBROUTINE CMPHY
C     THIS PROGRAM IS KNOWN AS URBHYMO. IT IS A CONCEPTUAL MODEL
C     MADE UP OF TWO LINEAR RESERVOIRS IN PARALLEL FOR COMPUTING
C     RUNOFF HYDROGRAPHS FROM URBANIZED WATERSHEDS.
C     IT ACCEPTS A RAINFALL HYETOGRAPH (IN/HR) AS INPUT AND COMPUTES
C     INFILTRATION LOSSES IN THE PERVIOUS AREA BY MEANS OF HORTON'S
C     INFILTRATION EQUATION.
C     THE ORIGINAL HYMO VERSION HAS NOT BEEN DELETED AND CAN BE
C     USED AS AN OPTION.
C
C
C     HYMO
C     THIS PROGRAM DEVELOPS A UNIT HYDROGRAPH, CONVERTS MASS RAINFALL
C     TO POINT RUNOFF, AND COMPUTES STORM HYDROGRAPHS BY SUMMATION.
C
      REAL CF(200)
      COMMON OCFS(300,6),DATA(310),CFS(200),CTBLE(25,11),RAIN(200),
     1A(20,6),Q(20,6),DEEP(20,6),ITBLE(25,2),DP(20),SCFS(20),C(20),
     2ZALFA(20),IEND(6),DA(6),DIST(6),SEGN(6),DT(6),PEAK(6),ISG(6),
     3NPU,NHD,NER,MAXNO,NCOMM,ICC,NCODE,TIME,ROIN,TPEAK(6)
      INTEGER STA,STP,START,NURI
      DIMENSION RIM(300),RP(300),RAINM(300),RAINEX(2,300)
C
C
      CUMINF(TT)=FC*TT+(FO-FC)/DC*(1.0-EXP(-DC*TT))
C
C
      IOUT = 6
      ID=DATA(1)
      NHD=DATA(2)
      DT(ID)=DATA(3)
      DA(ID)=DATA(4)
C
C     IS URBHYMO TO BE USED
C
      CKI=DATA(5)
      IF (CKI.GT.0.9) GO TO 100
      CN=DATA(6)
C
C     ARE K AND TP FURNISHED OR WILL THEY BE COMPUTED
C
      IF (DATA(7)) 1,2,2
```

```
1       XK=-DATA(7)
        TP=-DATA(8)
        GO TO 3
2       HT=DATA(7)
        XL=DATA(8)
        SLOPE=HT/XL
        XLDW=(XL**2.)/DA(ID)
        XK=27.0*(DA(ID)**.231)*(SLOPE**(-.777))*(XLDW**.124)
        TP=4.63*(DA(ID)**.422)*(SLOPE**(-.46))*(XLDW**.133)
3       PEAK(ID) = 1.
        DO 4 I=1,300
4       OCFS(I,ID)=0.
C       COMPUTE N BY ITERATION.
        XN=5.0
        XKTP=XK/TP
        DO 6 I=1,50
        TINF=1.+SQRT(1./(XN-1.))
        XN1=.05/(XKTP*(ALOG(TINF/(TINF+.05))+.05))+1.
        DIFF=ABS(XN1-XN)
        IF (DIFF-.001) 7,7,5
5       XN=XN1
6       CONTINUE
        WRITE (IOUT,29)
        GO TO 28
C       DETERMINE C1.
7       DELT=TINF/100.
        TC1=0.
        XN1P=XN-1.
        XN1M=1.-XN
        DO 8 I=2,101
        TC1=TC1+DELT
      8 CFS(I)=(TC1**XNIP)*EXP(XNIM*(TC1-1.))
        SUM=CFS(101)/2.
        DO 9 I=2,100
      9 SUM=SUM+CFS(I)
        C1=SUM*DELT
        CFSII=CFS(101)
        TTINF=TINF*TP
        TREC1=TTINF+2.*XK
        EEE=EXP((TTINF-TREC1)/XK)
        XK1=3.*XK
        B=645.333/(C1+CFSII*(XKTP*(1.-EEE)+EEE*(XK1/TP)))
C       COMPUTE B, QP, AND CFSI.
        QP=(B*DA(ID))/TP
        CFSI=QP*CFS(101)
        CFSR1=CFSI*EEE
        WRITE (IOUT,30) XN,QP
C       DETERMINE INCREMENTAL RUNOFF
        R=1000./CN-10.
        B1=.2*R
        J=9
        IF (DATA(J)) 13,10,10
```

```
10     RAIN(1)=DATA(J)
       DO 11 I=2,200
       J=J+1
       RAIN(I)=DATA(J)
       IF (RAIN(I)-RAIN(I-1)) 12,11,11
11     CONTINUE
12     NUMB=I-1
13     DO 15 I=1,NUMB
       IF (RAIN(I)-B1) 33,33,14
33     DATA (I)=0.
       Q1=0.
       GO TO 15
14     Q2=((RAIN(I)-B1)**2.)/(RAIN(I)+.8*R)
       DATA (I)=Q2-Q1
       Q1=Q2
15     CONTINUE
C      COMPUTE UNIT HYDROGRAPH
       UHV=645.333*DA(ID)/DT(ID)
       SUM=0.
       T2=0.
       CFS(1)=0.
       DO 20 I=2,200
       T1=T2
       T2=T2+DT(ID)
       IF (T2-TTINF) 16,16,17
   16 CFS(I)=QP*((T2/TP)**XNIP)*EXP(XNIM*(T2/TP-1.))
       GO TO 198
   17 IF (T2-TREC1) 18,18,19
   18 CFS(I)=CFSI*EXP((TTINF-T2)/XK)
       GO TO 198
   19 CFS(I)=CFSR1*EXP((TREC1-T2)/XK1)
  198 IF (T1.GE.TP.OR.T2.LE.TP) GO TO 202
       IF (CFS(I)-CFS(I-1)) 197,197,199
  197 SUM=SUM-CFS(I-1)+QP
       CFS(I-1)=QP
       GO TO 202
  199 CFS(I)=QP
  202 SUM=SUM+CFS(I)
       IF (CFS(I).LE.0.1.OR.SUM.GE.UHV) GO TO 21
   20 CONTINUE
       I=200
   21 ICND=I
C
C      COMPUTE STORM HYDROGRAPH
C
       DO 24 J=2,NUMB
       N=J+ICND-2
       IF (N-300) 23,23,22
  22   N=300
  23   I=2
       DO 24 K=J,N
       OCFS(K,ID)= OCFS(K,ID)+DATA(J)*CFS(I)
```

```
      I=I+1
 24   CONTINUE
      GO TO 200
C
C     URBHYMO COMPUTATION BEGINS
C
C
C     READ IN DATA
C     NURI=NUMBER OF RAINFALL INCREMENTS
C
  100 CKP=DATA(6)
      STOCOI=DATA(7)
      STOCOP=DATA(8)
      XIMP=DATA(9)
      TIMP=DATA(10)
      NURI=DATA(11)
      FO=DATA(12)
      FC=DATA(13)
      DC=DATA(14)
      F=DATA(15)
      DEPSTI=DATA(16)
      DEPSTP=DATA(17)
      SLOPEI=DATA(18)
      XLENGI=DATA(19)
      XINTEI=DATA(20)
      XMANNI=DATA(21)
      SLOPEP=DATA(22)
      XLENGP=DATA(23)
      XINTEP=DATA(24)
      XMANNP=DATA(25)
      TDUR=DATA(26)
      HP=DATA(27)
      HYDRAD=DATA(28)
      SLPI=SLOPEI/100.0
      SLPP=SLOPEP/100.0
C
C     INITIALIZE TO ZERO
C
      DO 94 I=1,300
      OCFS(I,ID)=0.0
      RAINM(I)=0.0
      RAINEX(1,I)=0.0
      RAINEX(2,I)=0.0
      RIM(I)=0.0
 94   RP(I)=0.0
      STP=0
C
C     DETERMINE RAINFALL EXCESS (URBHYMO)
C     TTRAIN = TOTAL RAIN (IN)
C
      J=29
      IF (DATA (J)) 113,110,110
```

```
 110 TOTRN=0.0
     RAIN(1)=DATA(J)
     TOTRN=TOTRN+RAIN(1)
     DO 121 I=2,NURI
     J=J+1
     RAIN(I)=DATA(J)
     TOTRN=TOTRN+RAIN(I)
 121 CONTINUE
     TTRAIN = TOTRN*DT(ID)
     WRITE (IOUT,253) TTRAIN
 113 IF (DEPSTI.NE.5.0) GO TO 114
     DEPSTI=0.0303*(SLOPEI**(-0.49))
 114 DEPTI=0.0
     DO 111 I=1,NURI
     DEPTI=DEPTI+(RAIN(I)*DT(ID))
     IF (DEPTI.GE.DEPSTI) GO TO 116
     DATA(I)=0.0
 111 CONTINUE
 116 XKEEP=RAIN(I)
     RAIN(I)=(DEPTI-DEPSTI)/DT(ID)
     STA=I
C
C    RUNOFF FROM IMPERVIOUS AREA
C
     DO 130 I=STA,NURI
     RIM(I)=XIMP*RAIN(I)
 130 CONTINUE
     RAIN(STA)=XKEEP
C
C    INFILTRATION COMPUTATION FOR PERVIOUS AREA
C
     T=0.0
     DO 185 I=1,200
     TEMP=CUMINF(T)
     WRITE (IOUT,67) I,TEMP
C
C    COMPARE F WITH TEMP=ACCUM. INFIL.
C
     IF (F.LE.TEMP) GO TO 201
     T=T+0.01
 185 CONTINUE
C
C    ASSUME FLAT INFILTRATION CURVE AFTER TLIM
C
 201 TLIM=960.0/DC
     TP=T
C
C    TP IS TIME ON HORTON CURVE THAT CORRESPONDS TO CUMULATIVE
C    INFILTRATION
C
     DEPTH=0.0
     DO 150 I=1,NURI
```

```
      RAINM(I)=(1.+(TIMP-XIMP)/(1.-TIMP))*RAIN(I)
      DEPONE=DEPTH+(RAINM(I)*DT(ID))
      TP1=TP
      IF (TP.LT.TLIM) GO TO 160
      FIL=FC
      TP=TLIM
      GO TO 155
C
C     TO OBTAIN AVERAGE INFILTRATION RATE
C
  160 FIL=(CUMINF(TP+DT(ID))-CUMINF(TP))/DT(ID)
C     WRITE (IOUT,64) I,FIL
C
C     COMPARE RAINFALL INTENSITY WITH INFILTRATION RATE
C     RAINFALL EXCESS IN IN/HR
C     RIM=RAINFALL EXCESS (IMPERVIOUS AREA)
C     RP=RAINFALL EXCESS (PERVIOUS AREA)
C
  155 IF (DEPONE.LE.(FIL*DT(ID))) GO TO 156
      ACFIL=FIL
      DEPTWO=DEPONE-(FIL*DT(ID))
      IF (DEPTWO.GE.DEPSTP) GO TO 101
      RP(I)=0.0
      DEPTH=DEPTWO
      GO TO 157
 101  IF (STP.EQ.0) STP=I
      RP(I)=(1.-TIMP)*((DEPTWO-DEPSTP)/DT(ID))
      DEPTH=DEPSTP
 157  TP1=TP+DT(ID)
      GO TO 175
C
  156 ACFIL=DEPONE/DT(ID)
      DEPTH=0.0
      RP(I)=0.0
      IF (TP.GE.TLIM) GO TO 180
C
C     FIND ACCUMULATED INFILTRATED VOLUME FROM INTEGRATED HORTON'S
C     EQUATION
C
      CUMI=CUMINF(TP)
      CUMI=CUMI+ACFIL*DT(ID)
      TP1=TP+DT(ID)*0.5
C
      DO 170 J=1,11
      EE=EXP(-TP1*DC)
      FT=FC+(FO-FC)*EE
      FF=FC*TP1+(FO-FC)/DC*(1.0-EE)-CUMI
      DELF=FF/FT
      TP1=TP1-DELF
      IF (ABS(DELF).GT.0.001*DT(ID)) GO TO 170
      IF (TP1.LT.TP+DT(ID).AND.TP1.GT.TP) GO TO 175
  170 CONTINUE
```

```
C
C      NO CONVERGENCE. AS DEFAULT USE TP=TP+0.9*DT
C
       TP=TP+0.9*DT(ID)
       GO TO 180
  175  TP=TP1
  180  IF (TP.GT.TLIM) TP=TLIM
  150  CONTINUE
C      TOTAL RAINFALL EXCESS
       PEI=0.0
       PEP=0.0
       PETO=0.0
C
C      IF CKP IS EQUAL TO 3.0 THEN THE SINGLE
C      LINEAR RESERVOIR MODEL WILL BE USED
C
       IF (CKP.NE.3.0) GO TO 220
       DO 135 I=STA,NURI
       RAINEX(1,I)=RIM(I)*DT(ID)
       RAINEX(2,I)=RP(I)*DT(ID)
       DATA(I)=(RIM(I)+RP(I))*DT(ID)
       PETO=PETO+DATA(I)
 135   CONTINUE
       GO TO 221
 220   DO 221 I=STA,NURI
       RAINEX(1,I)=RAIN(I)*DT(ID)
       RAINEX(2,I)=(RP(I)*DT(ID))/(1.-TIMP)
       DATA(I)=(RIM(I)+RP(I))*DT(ID)
       PEI=PEI+RAINEX(1,I)
       PEP=PEP+RAINEX(2,I)
       PETO=PETO+DATA(I)
 221   CONTINUE
C
       DTEN=1./6.
       NRANIT=((DTEN/DT(ID))+0.5)
       SDUR=NURI*DT(ID)
       TRI=(NURI-STA+1)*DT(ID)
       TRP=(NURI-STP+1)*DT(ID)
       URBF=1.+TIMP
       URDC=1.+XIMP
       NBER=2
       IF (CKP.EQ.3.0) NBER=1
       DO 140 M=1,NBER
       IF (CKP.NE.3.0) GO TO 213
C
C      COMPUTATION OF RAINFALL EXCESS INTENSITY
C
       PKDATA=0.01
       DO 211 I=STA,NURI
       IF (DATA(I)-PKDATA) 211,211,212
 212   PKDATA=DATA(I)
       XTIP=PKDATA/DT(ID)
```

```
      IXTIP=I
211   CONTINUE
      IF (NRANIT.EQ.1) GO TO 261
      IF (NRANIT.EQ.2) GO TO 262
      IF (NRANIT.EQ.10) GO TO 263
261   XINTEN=XTIP
      GO TO 264
262   XTIPA=DATA(IXTIP-1)/DT(ID)
      XTIPB=DATA(IXTIP+1)/DT(ID)
      XINTEN=(XTIPA+XTIP)/2.
      IF ((XTIPA+XTIP).LT.(XTIPB+XTIP)) XINTEN=(XTIPB+XTIP)/2.
      GO TO 264
263   NXTIPA=IXTIP-9
      SMXTIP=0.0
      DO 265 I=NXTIPA,IXTIP
      SMXTIP=SMXTIP+DATA(I)
265   CONTINUE
      XINTEN=SMXTIP*6.
264   WRITE (IOUT,242) XINTEN
      GO TO 133
213   IF (M.EQ.1) GO TO 133
134   IF (XINTEP.GT.0.0) XINTEN=XINTEP
      IF (XINTEP.GT.0.0) GO TO 133
      PKRP=0.0
      DO 138 I=STP,NURI
      IF (RP(I)-PKRP) 138,138,139
139   PKRP=RP(I)
      TIP=PKRP/(1.-TIMP)
      ITIP=I
138   CONTINUE
      IF (NRANIT.EQ.1) GO TO 218
      IF (NRANIT.EQ.2) GO TO 219
      IF (NRANIT.EQ.10) GO TO 223
218   XINTEN=TIP
      IF (XINTEN.EQ.0.0) XINTEN=0.0001
      GO TO 133
219   TIPA=RP(ITIP-1)/(1.-TIMP)
      TIPB=RP(ITIP+1)/(1.-TIMP)
      XINTEN=(TIPA+TIP)/2.
      IF ((TIPA+TIP).LT.(TIPB+TIP)) XINTEN=(TIPB+TIP)/2.
      IF (XINTEN.EQ.0.0) XINTEN=0.0001
      GO TO 133
223   NTIPA=ITIP-9
      SUMTIP=0.0
      DO 222 I=NTIPA,ITIP
      SUMTIP=SUMTIP+RP(I)
222   CONTINUE
      XINTEN=SUMTIP/(10*(1.-TIMP))
      IF (XINTEN.EQ.0.0) XINTEN=0.0001
133   IF (M.EQ.1) GO TO 141
      CK=CKP
      SLOPE=SLOPEP
```

```
      XLENG=XLENGP
      XMANIN=XMANNP
      SLP=SLPP
      RATIO=1.-TIMP
      TR=TRP
      PE=PEP
      GO TO 142
141   CK=CKI
      SLOPE=SLOPEI
      XLENG=XLENGI
      XMANIN=XMANNI
      IF (XINTEI.GT.0.0) XINTEN=XINTEI
      SLP=SLPI
      RATIO=XIMP
      IF (CKP.EQ.3.0) RATIO=1.0
      TR=TRI
      PE=PEI
      IF (CKP.EQ.3.0) PE=PETO
142   IF (CK.GT.1.0) GO TO 74
      IF (M.EQ.1) GO TO 143
      SC=STOCOP
      GO TO 85
143   SC=STOCOI
144   GO TO 85
C
C     COMPUTATION OF STORAGE COEFFICIENT K
C
 74   IF (CK.GT.1.1) GO TO 75
      SC=0.00025*((66.6667*XMANIN*XLENG*(SLP**(-0.5)))**0.66)
      GO TO 85
 75   IF (CK.GT.1.2) GO TO 76
      SC=0.00719*(XLENG**0.593)*(XMANIN**0.605)
     C*(SLP**(-0.38))*(XINTEN**(-0.388))
      GO TO 85
 76   IF (CK.GT.1.3) GO TO 77
      SC=0.02588*(SLOPE**(-0.40))*(XLENG**0.22)*(XINTEN**(-0.40))
      GO TO 85
 77   IF (CK.GT.1.4) GO TO 78
      SC=0.00775*(XLENG**0.6)*(XMANIN**0.6)*
     C(XINTEN**(-0.4))*(SLP**(-0.3))
      GO TO 85
 78   IF (CK.GT.1.5) GO TO 79
      SC=0.831*((DA(ID)*0.0015625*RATIO)**0.458)*(URBF**(-1.662))*
     C(PE**(-0.267))*(TR**0.371)
      GO TO 85
 79   IF (CK.GT.1.6) GO TO 80
      SC=0.887*((DA(ID)*0.0015625*RATIO)**0.490)*(URBF**(-1.683))*
     C(PE**(-0.240))*(TR**0.294)
      GO TO 85
 80   IF (CK.GT.1.7) GO TO 81
      SC=0.788*((DA(ID)*0.0015625*RATIO)**0.409)*(URBF**(-2.06))*
     C(PE**(-0.15))*(TR**0.156)
```

```
      GO TO 85
 81   IF (CK.GT.1.8) GO TO 82
      SC=0.11386*((DA(ID)*RATIO)**0.18)*(SLOPE**(-0.36))*(URBF**(-1.9))
     C*(TDUR**0.21)*(XLENG**0.15)*(HP**(-0.07))
      GO TO 85
 82   IF (CK.GT.1.9) GO TO 83
      SC=0.0175*(XLENG**0.24)*(SLOPE**(-0.16))*(XIMP**(-0.26))
      GO TO 85
 83   SC=0.0041837*(SDUR**0.031)*(PE**0.323)
     C*(HYDRAD**0.80)*(URDC**(-0.342))
 85   SCMIN=SC*60
      QP=(645.333*DA(ID)*0.0015625*RATIO)/SC
      QPP=1.0/DT(ID)
C
C     COMPUTE UNIT HYDROGRAPH.
C
      SUM=0.
      T2=0.
      CFS(1)=0.0
      QMX=QPP*(1-EXP(-DT(ID)/SC))
      DO 70 I=2,200
      T2=T2+DT(ID)
      CFS(I)=QMX*EXP(-(T2-DT(ID))/SC)
      IF (CFS(I).LE.0.001) GO TO 71
70    CONTINUE
      I=200
71    ICND=I
      IMD=ICND-1
C
C
      WRITE (IOUT,251) PETO,TR
      WRITE (IOUT,301) SC,QMX,XINTEN,RATIO
      WRITE (IOUT,241) SCMIN
C     WRITE (IOUT,40) URBF
      SUMCF=0.0
      DO 53 I=1,ICND
      SUMCF=SUMCF+CFS(I)
53    CONTINUE
      WRITE(IOUT,43) SUMCF
      NUMB=NURI+1
C     CONVERT CFS/ACRE TO CFS
      DO 59 I=1,ICND
      CFS(I)=CFS(I)*DA(ID)*RATIO*1.00833
 59   CONTINUE
C
C     COMPUTE STORM HYDROGRAPH.
C
      IF (CKP.EQ.3.0) GO TO 34
      DO 96 J=2,NUMB
      L=J-1
      N=J+ICND-2
      IF (N-300) 98,98,97
```

```
97     N=300
98     I = 2
       DO 96 K= J,N
       OCFS(K,ID)=OCFS(K,ID)+RAINEX(M,L)*CFS(I)
       I=I+1
  96   CONTINUE
 140   CONTINUE
       WRITE (IOUT,44) DEPSTI,DEPSTP
C      WRITE (IOUT,57) STA,STP
       GO TO 200
  34   DO 224 J=2,NUMB
       L=J-1
       N=J+ICND-2
       IF (N-300) 226,226,225
 225   N=300
 226   I=2
       DO 224 K=J,N
       OCFS(K,ID)=OCFS(K,ID)+DATA(L)*CFS(I)
       I=I+1
 224   CONTINUE
C
C      COMPUTE RUNOFF VOLUME
C      DETERMINE PEAK
C
 200   TPEK=TIME
C      WRITE (IOUT,210) TPEK
       PEAK(ID)=0.2
       RO = 0.
       DO 26 I = 2,N
       RO = RO + OCFS(I,ID)
       IF (OCFS(I,ID)-PEAK(ID))26,26,25
25     PEAK(ID)= OCFS(I,ID)
       TEPE=(I-1)*DT(ID)
26     CONTINUE
       TPEK=TPEK+TEPE
C      WRITE (IOUT,250) TEPE,TPEK
       TPEAK(ID)=TPEK
       IF (CKI.LE.0.9) GO TO 205
C      WRITE (IOUT,58) XKEEP
C      WRITE (IOUT,193) RAIN(STA)
C      WRITE (IOUT,192) XMANNI,XLENGI,SLOPEI
C      WRITE (IOUT,196) XMANNP,XLENGP,SLOPEP
C      WRITE (IOUT,69) (I,RIM(I),I,RP(I),I=STA,NURI)
C      WRITE (IOUT,55)
C      WRITE (IOUT,56) (I,DATA(I),I=1,NURI)
       WRITE(IOUT,39) RO
 205   IEND (ID) = N
       ROIN=(RO*DT(ID))/(DA(ID)*0.0015625 * 645.333)
       WRITE (IOUT,63) PEAK(ID),ROIN,TPEAK(ID)
C      PUNCH CODE
       IF (NPU) 28,28,27
   27  WRITE (IOUT,31)ID,NHD,DT(ID),DA(ID),PEAK(ID),  ROIN,IEND(ID)
```

```
      WRITE (IOUT,32) (OCFS(I,ID),I=1,N)
28    RETURN
C
29    FORMAT('N DID NOT CONVERGE AFTER 50 ITERATIONS.')
30    FORMAT(T10,'SHAPE CONSTANT, N = ',F6.3/T10,'UNIT PEAK = ',F10.1,'C
     1FS'/)
31    FORMAT(   'RECALL HYD',T21,'ID=',I1,T29,'HYD NO=',I3,T42,'DT=',F9.
     16,' HRS',T61,'DA=',F8.3,' SQ MI'/T21,'PEAK=',F7.0,'CFS',T40,'RO=',
     2F6.3,' INCHES',T59,'NO PTS=',I3/T21,'FLOW RATES')
32    FORMAT (T21,7F8.0)
301   FORMAT(T10,'STORAGE COEFF. SC = ',F8.4,' HOURS'/T10,
     1'UNIT PEAK(QMX) = ',F10.4,' CFS'/T10,
     2'INTENSITY= ',F10.4,' IN/HR'/T10,
     3'RATIO= ',F10.4,/)
37    FORMAT(T10,'CO-ORDINATES OF THE UNIT HYDROGRAPH ARE')
38    FORMAT(T10,F10.4)
39    FORMAT(T10,'SUM OF RUNOFF CO-ORDINATES = ',F10.3)
40    FORMAT(T10,'URBANIZATION FACTOR URBF=',F8.3)
41    FORMAT(T10,'IMPERVIOUSNESS RATIO IMP=',F8.3)
44    FORMAT(T10,'DEPRESSION STORAGE(IMP)=',F8.3,' (PERV)=',F8.3)
43    FORMAT(T10,'SUM OF UNIT HYDROGRAPH CO-ORDINATES=',F10.3)
50    FORMAT(T10,'CFS(I+1)=',F10.3)
51    FORMAT(T10,'CF(I)=',F10.3)
52    FORMAT(T10,'CF(I+1)=',F10.3)
55    FORMAT(T10,'EXCESS RUNOFF IN INCHES')
56    FORMAT(T10,I3,F10.4)
57    FORMAT(T10,' STA=',I3,2X,' STP=',I3)
58    FORMAT(T10,' XKEEP=',F8.4)
61    FORMAT(T10,'PE-1(',I3,')=',F12.4)
62    FORMAT(T10,'PE-2(',I3,')=',F12.4)
63    FORMAT(T10,'PEAK DISCHARGE=',F10.4,' CFS',2X,'RUNOFF VOLUME=',
     CF8.4,' INCHES',' TIME TO PEAK=',F8.4)
64    FORMAT(T10,'FIL(',I3,')=',F12.4)
65    FORMAT(T10,'ACF-1(',I3,')=',F12.4)
66    FORMAT(T10,'ACF-2(',I3,')=',F12.4)
67    FORMAT(T10,'F (',I3,')=',F12.4)
68    FORMAT(T10,'CUMI-1(',I3,')=',F12.4)
69    FORMAT(T10,'RIM(',I3,')=',F10.4,'   RP(',I3,')=',F10.4)
191   FORMAT(T10,'PEAK DISCHARGE(SDR)=',F12.4,'  CFS'/T10,
     C'TIME TO PEAK (SDR) = ',F12.4,' HOURS')
192   FORMAT(T10,'XMANNI=',F10.4,'  XLENGI=',F10.3,'  SLOPEI=',F10.4)
193   FORMAT(T10,'RAIN(STA)=',F10.4)
194   FORMAT(T10,'OCFS(',I3,')=',F12.4)
196   FORMAT(T10,'XMANNP=',F10.4,'  XLENGP=',F10.3,'  SLOPEP=',F10.4)
195   FORMAT(T10,'CUMI-2(',I3,')=',F12.4)
210   FORMAT(T10,' TPEK(1)=',F12.4)
214   FORMAT(T10,' RAINEX(1)( ',I3,')=',F12.4)
215   FORMAT(T10,' RAINEX(2)( ',I3,')=',F12.4)
216   FORMAT(T10,' CFS(',I3,')=',F10.4)
217   FORMAT(T10,' OCFS(',I3,')=',F10.4)
241   FORMAT(T10,'STORAGE COEFF. K = ',F10.4,' MINS')
242   FORMAT(T10,' XINTEN = ',F10.4)
250   FORMAT(T10,' TEPE= ',F10.4,'  TPEK(2)= ',F10.4)
251   FORMAT(T10,' RAINFALL EXCESS PETO = ',F12.4,' INCHES'/T10,
     C'RAINFALL EXCESS DURATION TR = ',F8.4,' HOURS')
253   FORMAT(T10,' TOTAL RAINFALL = ',F12.4,' INCHES')
      END
```

Appendix C

EXAMPLE OF URBHYMO COMPUTER SIMULATION

```
ZALFA = 1234567890 *./-

      COMMAND TABLE

      START                    1  2
      STORE HYD             2310
      RECALL HYD            3310
      COMPUTE HYD           4310
      PRINT HYD             5  2
      PUNCH HYD             6  1
      PLOT HYD              7  2
      ADD HYD               8  4
      STORE RATING CURVE    9100
      COMPUTE RATING CURVE 10310
      STORE TRAVEL TIME    11100
      COMPUTE TRAVEL TIME  12  5
      ROUTE                13  4
      ROUTE RESERVOIR      14100
      ERROR ANALYSIS       15  2
      SEDIMENT YIELD       16  5
      FINISH               17  0
* REAL STORM TESTING - GRAY HAVEN CATCHMENT
* STORM ON 2 AUGUST 1965
* GH02AU65
* URBHYMO TESTING
* RUN NO.1 - ONE MINUTE DT
START              RAINFALL BEGINS AT 0.0 HOURS
STORE HYD          ID=1 HYD NO=101 DT=0.01666667 HR DA=23.3 ACRES
                    0.0  0.0  0.0  0.0  0.0  0.0  0.0  0.0
                    0.0  0.0  0.3  0.6  1.2  1.1  1.1  1.4
                    2.0  3.5  5.3 10.6 13.0 14.6 16.7 15.5
                   14.6 14.0 13.7 13.5 13.0 13.0 13.9 15.3
                   18.9 23.3 28.8 30.3 27.0 24.3 20.9 17.9
                   14.6 11.8  9.8  8.2  7.2  6.3  5.7  5.5
                    5.0  5.0  4.9  4.9  4.9  4.8  4.7  4.5
                    4.4  4.2  4.1  4.1  4.2  4.5
    TPEAK(   1)=     0.5833
PRINT HYD          ID=1
                              PARTIAL HYDROGRAPH  101

    TIME      FLOW          TIME     FLOW          TIME     FLOW          TIME     FLOW         TIME
     HRS       CPS           HRS      CPS           HRS      CPS           HRS      CPS          HRS
    0.0        0.0          0.217     1.10         0.433    13.70         0.650    17.90        0.867
    0.017      0.0          0.233     1.10         0.450    13.50         0.667    14.60        0.883
    0.033      0.0          0.250     1.40         0.467    13.00         0.683    11.80        0.900
    0.050      0.0          0.267     2.00         0.483    13.00         0.700     9.80        0.917
    0.067      0.0          0.283     3.50         0.500    13.90         0.717     8.20        0.933
    0.083      0.0          0.300     5.30         0.517    15.30         0.733     7.20        0.950
    0.100      0.0          0.317    10.60         0.533    18.90         0.750     6.30        0.967
    0.117      0.0          0.333    13.00         0.550    23.30         0.767     5.70        0.983
    0.133      0.0          0.350    14.60         0.567    28.80         0.783     5.50        1.000
    0.150      0.0          0.367    16.70         0.583    30.30         0.800     5.00        1.017
    0.167      0.30         0.383    15.50         0.600    27.00         0.817     5.00
    0.183      0.60         0.400    14.60         0.617    24.30         0.833     4.90
    0.200      1.20         0.417    14.00         0.633    20.90         0.850     4.90
    RUNOFF VOLUME =        0.371 INCHES
    PEAK DISCHARGE RATE =      30.30 CFS


COMPUTE HYD        ID=2 HYD=102 DT=0.01666667 HR DA=23.3 ACRES
                   CKI=1.3 CKP=1.4 STOCOI=1.000 STOCOP=1.000
                   XIMP=.44 TIMP=.52 NURI=60
                   FO=3.0 INPHR FC=0.52 INHR DECAY=4.14 PHR F=0.0 IN
                   DEPSTI=0.062  DEPSTP=0.184
                   SLOPEI=0.9  LENGI=822 XINTEI=4.20 MANNI=0.013
                   SLOPEP=0.9  LENGP=822 XINTEP=3.18 MANNP=0.250
                   TDUR=1.0 HP=1.0 HYDRAD=1.0
                   RAINFALL HYETOGRAPH INPHR
                   0.60 0.00 0.60 1.20 1.80 0.60 0.60 0.00
                   0.60 0.00 0.60 1.20 1.20 1.20 2.40 1.80
                   0.00 1.20 0.60 1.20 0.60 1.20 1.80 1.80
                   1.80 1.20 1.80 2.40 3.00 4.20 4.20 3.60
                   1.80 0.60 0.00 0.00 0.00 0.60 0.00 0.00
                   0.60 0.00 0.60 0.60 1.20 0.60 1.20 0.60
                   0.00 0.00 0.00 0.60 0.60 0.00 0.60 0.60
                   0.60 1.20 0.60 1.20
    TOTAL RAINFALL =         0.9900 INCHES
    F (   1)=         0.0
    STORAGE COEFF. SC =    0.0666 HOURS
    UNIT PEAK(QMX) =      13.2900 CFS
    INTENSITY=        4.2000 IN/HR
    RATIO=        0.4400

    STORAGE COEFF. K =       3.9936 MINS
    SUM OF UNIT HYDROGRAPH CO-ORDINATES=      59.997
    STORAGE COEFF. SC =    0.4895 HOURS
    UNIT PEAK(QMX) =       2.0086 CFS
    INTENSITY=        3.1800 IN/HR
    RATIO=        0.4800

    STORAGE COEFF. K =     29.3694 MINS
    SUM OF UNIT HYDROGRAPH CO-ORDINATES=      59.931
    DEPRESSION STORAGE(IMP)=    0.062 (PERV)=    0.184
    SUM OF RUNOFF CO-ORDINATES =       680.022
    PEAK DISCHARGE=    33.7169 CFS  RUNOFF VOLUME=  0.4824 INCHES  TIME TO PEAK=  0.5333
```

TIME HRS	FLOW CFS	TIME HRS	FLOW CFS	TIME HRS	FLOW CFS	TIME HRS	FLOW CFS	TIME HRS
0.0	0.0	0.867	5.69	1.733	0.26	2.600	0.04	3.467
0.017	0.0	0.883	6.09	1.750	0.25	2.617	0.04	3.483
0.033	0.0	0.900	5.02	1.767	0.24	2.633	0.04	3.500
0.050	0.0	0.917	5.55	1.783	0.23	2.650	0.04	3.517
0.067	0.0	0.933	5.95	1.800	0.23	2.667	0.04	3.533
0.083	4.12	0.950	6.26	1.817	0.22	2.683	0.04	3.550
0.100	4.58	0.967	7.86	1.833	0.21	2.700	0.04	3.567
0.117	4.94	0.983	7.73	1.850	0.20	2.717	0.03	3.583
0.133	3.85	1.000	8.99	1.867	0.20	2.733	0.03	3.600
0.150	4.37	1.017	7.22	1.883	0.19	2.750	0.03	3.617
0.167	3.40	1.033	5.83	1.900	0.18	2.767	0.03	3.633
0.183	4.02	1.050	4.74	1.917	0.18	2.783	0.03	3.650
0.200	5.89	1.067	3.89	1.933	0.17	2.800	0.03	3.667
0.217	7.33	1.083	3.22	1.950	0.17	2.817	0.03	3.683
0.233	8.45	1.100	2.69	1.967	0.16	2.833	0.03	3.700
0.250	12.07	1.117	2.27	1.983	0.16	2.850	0.03	3.717
0.267	13.52	1.133	1.94	2.000	0.15	2.867	0.03	3.733
0.283	10.53	1.150	1.68	2.017	0.15	2.883	0.02	3.750
0.300	10.94	1.167	1.46	2.033	0.14	2.900	0.02	3.767
0.317	9.89	1.183	1.29	2.050	0.14	2.917	0.02	3.783
0.333	10.45	1.200	1.16	2.067	0.13	2.933	0.02	3.800
0.350	9.51	1.217	1.05	2.083	0.13	2.950	0.02	3.817
0.367	10.15	1.233	0.95	2.100	0.12	2.967	0.02	3.833
0.383	12.02	1.250	0.88	2.117	0.12	2.983	0.02	3.850
0.400	13.48	1.267	0.82	2.133	0.11	3.000	0.02	3.867
0.417	14.62	1.283	0.76	2.150	0.11	3.017	0.02	3.883
0.433	14.13	1.300	0.71	2.167	0.11	3.033	0.02	3.900
0.450	15.12	1.317	0.67	2.183	0.10	3.050	0.02	3.917
0.467	17.27	1.333	0.64	2.200	0.10	3.067	0.02	3.933
0.483	20.31	1.350	0.61	2.217	0.10	3.083	0.02	3.950
0.500	25.43	1.367	0.58	2.233	0.09	3.100	0.02	3.967
0.517	30.83	1.383	0.55	2.250	0.09	3.117	0.02	3.983
0.533	33.72	1.400	0.53	2.267	0.09	3.133	0.01	4.000
0.550	31.28	1.417	0.51	2.283	0.08	3.150	0.01	4.017
0.567	26.27	1.433	0.49	2.300	0.08	3.167	0.01	4.033
0.583	20.98	1.450	0.47	2.317	0.08	3.183	0.01	4.050
0.600	16.84	1.467	0.45	2.333	0.08	3.200	0.01	4.067
0.617	13.61	1.483	0.44	2.350	0.07	3.217	0.01	4.083
0.633	12.44	1.500	0.42	2.367	0.07	3.233	0.01	4.100
0.650	10.15	1.517	0.41	2.383	0.07	3.250	0.01	4.117
0.667	8.35	1.533	0.39	2.400	0.07	3.267	0.01	4.133
0.683	8.30	1.550	0.38	2.417	0.06	3.283	0.01	4.150
0.700	6.88	1.567	0.37	2.433	0.06	3.300	0.01	4.167
0.717	7.13	1.583	0.35	2.450	0.06	3.317	0.01	4.183
0.733	7.32	1.600	0.34	2.467	0.06	3.333	0.01	4.200
0.750	8.82	1.617	0.33	2.483	0.06	3.350	0.01	4.217
0.767	8.60	1.633	0.32	2.500	0.05	3.367	0.01	4.233
0.783	9.80	1.650	0.31	2.517	0.05	3.383	0.01	4.250
0.800	9.34	1.667	0.30	2.533	0.05	3.400	0.01	4.267
0.817	7.60	1.683	0.29	2.550	0.05	3.417	0.01	4.283
0.833	6.23	1.700	0.28	2.567	0.05	3.433	0.01	4.300
0.850	5.16	1.717	0.27	2.583	0.05	3.450	0.01	

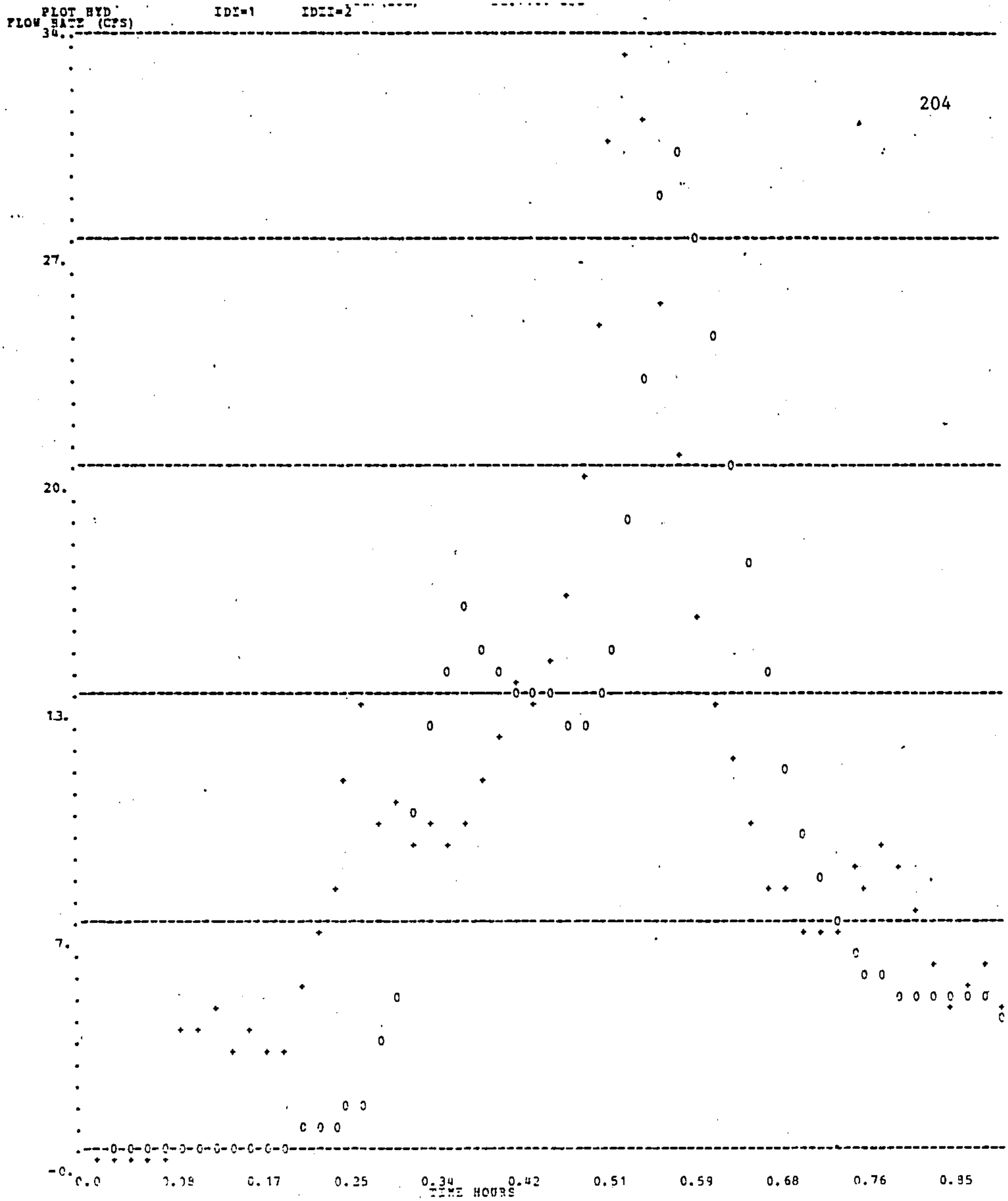
RUNOFF VOLUME = 0.482 INCHES
PEAK DISCHARGE RATE = 33.72 CFS

ERROR ANALYSIS IDI=1 IDII=2

TIME HRS	FLOW 1 CFS	FLOW 2 CFS	ERROR CFS
0.0	0.	0.	0.
0.017	0.	0.	0.
0.033	0.	0.	0.
0.050	0.	0.	0.
0.067	0.	0.	0.
0.083	0.	4.	-4.
0.100	0.	5.	-5.
0.117	0.	5.	-5.
0.133	0.	4.	-4.
0.150	0.	4.	-4.
0.167	0.	3.	-3.
0.183	1.	4.	-3.
0.200	1.	6.	-5.
0.217	1.	7.	-6.
0.233	1.	8.	-7.
0.250	1.	12.	-11.
0.267	2.	14.	-12.
0.283	3.	11.	-7.
0.300	5.	11.	-6.
0.317	11.	10.	1.
0.333	13.	10.	3.
0.350	15.	10.	5.
0.367	17.	10.	7.
0.383	15.	12.	3.
0.400	15.	13.	1.
0.417	14.	15.	-1.
0.433	14.	14.	-0.
0.450	13.	15.	-2.
0.467	13.	17.	-4.
0.483	13.	20.	-7.
0.500	14.	25.	-12.
0.517	15.	31.	-16.
0.533	19.	34.	-15.
0.550	23.	31.	-8.
0.567	29.	26.	3.
0.583	30.	21.	9.
0.600	27.	17.	10.
0.617	24.	14.	11.
0.633	21.	12.	8.
0.650	18.	10.	8.
0.667	15.	8.	6.
0.683	12.	8.	3.
0.700	10.	7.	3.
0.717	8.	7.	1.
0.733	7.	7.	-0.
0.750	6.	9.	-3.
0.767	6.	9.	-3.
0.783	5.	10.	-4.
0.800	5.	9.	-4.
0.817	5.	8.	-3.
0.833	5.	6.	-1.
0.850	5.	5.	-0.
0.867	5.	6.	-1.
0.883	5.	6.	-1.
0.900	5.	5.	-0.
0.917	4.	6.	-1.
0.933	4.	6.	-2.
0.950	4.	6.	-2.
0.967	4.	8.	-4.
0.983	4.	8.	-4.
1.000	4.	9.	-5.
1.017	4.	7.	-3.

SUM OF SQUARES OF ERRORS= 2013.1289
ERROR VARIANCE= 32.4698
ERROR STANDARD DEVIATION = 5.698
PEAK DISCHARGE ERROR = 11.28 PERCENT

PLOT HYD IDI=1 IDII=2



PLOT HYD ID I=1 ID II=2
FINISH

```
ZALFA = 1234567890 *./-

        COMMAND TABLE

     START                     1  2
     STORE HYD              2310
     RECALL HYD             3310
     COMPUTE HYD            4310
     PRINT HYD              5  2
     PUNCH HYD              6  1
     PLOT HYD               7  2
     ADD HYD                8  4
     STORE RATING CURVE     9100
     COMPUTE RATING CURVE  10310
     STORE TRAVEL TIME     11100
     COMPUTE TRAVEL TIME   12  5
     ROUTE                 13  4
     ROUTE RESERVOIR       14100
     ERROR ANALYSIS        15  2
     SEDIMENT YIELD        16  5
     FINISH                17  0
* MALVERN STORM 31 MAY, 1974
* MALVERN3
* URBHYMO MODEL TESTING
* RUN NO.1 - ONE MIN DT
START                  RAINFALL BEGINS AT 19.08333333 HOURS
STORE HYD              ID=1 HYDNO=101 DT=0.016666667 HR DA=57.6 ACRES
                       .20 7.76 15.11 21.60 26.73 29.57 26.73 21.60 18.24
                       14.88 11.52 10.75  9.99  9.22  8.45  8.45  8.81
                        9.18  9.54 10.96 12.37 16.99 21.60 26.73 29.57
                       30.02 30.47 30.92 31.37 31.82 31.06 25.74 20.42
                       17.77 15.11 13.27 11.44  9.60  7.76  6.82  5.87
                        5.16  4.46  3.75  3.10  2.44  2.17  1.90  1.63
                        1.36  1.09  1.01  0.93  0.86  0.78  0.70  0.63
                        0.57  0.50  0.44  0.37  0.35  0.33  0.32  0.30
                        0.28  0.26  0.25  0.23  0.22  0.20
    TPEAK(   1)=         19.5667
PRINT HYD              ID=1
                                        PARTIAL HYDROGRAPH  101 -
```

TIME HRS	FLOW CFS	TIME HRS	FLOW CFS	TIME HRS	FLOW CFS	TIME HRS	FLOW CFS	TIME HRS
19.083	0.20	19.333	8.45	19.583	31.06	19.833	2.44	20.083
19.100	7.76	19.350	8.81	19.600	25.74	19.850	2.17	20.100
19.117	15.11	19.367	9.18	19.617	20.42	19.866	1.90	20.116
19.133	21.60	19.383	9.54	19.633	17.77	19.883	1.63	20.133
19.150	26.73	19.400	10.96	19.650	15.11	19.900	1.36	20.150
19.167	29.57	19.417	12.37	19.667	13.27	19.916	1.09	20.166
19.183	26.73	19.433	16.99	19.683	11.44	19.933	1.01	20.183
19.200	21.60	19.450	21.60	19.700	9.60	19.950	0.93	20.200
19.217	18.24	19.467	26.73	19.717	7.76	19.966	0.86	20.216
19.233	14.88	19.483	29.57	19.733	6.82	19.983	0.79	20.233
19.250	11.52	19.500	30.02	19.750	5.87	20.000	0.70	20.250
19.267	10.75	19.517	30.47	19.766	5.16	20.016	0.63	
19.283	9.99	19.533	30.92	19.783	4.46	20.033	0.57	
19.300	9.22	19.550	31.37	19.800	3.75	20.050	0.50	
19.317	8.45	19.567	31.82	19.816	3.10	20.066	0.44	

```
     RUNOFF VOLUME =        0.213 INCHES
     PEAK DISCHARGE RATE =        31.82 CFS


COMPUTE HYD            ID=2 HYD=102 DT=0.016666667 HR DA=57.6 ACRES
                       CKI=1.3 CKP=1.4 STOCOI=1.000 STOCOP=1.000
                       XIMP=.34 TIMP=.34 NURI=38
                       FO=3.0 INPHR FC=0.52 INHR DECAY=4.14 PHR F=0.0 IN
                       DEPSTI=0.02 DEPSTP=0.18
                       SLOPEI=1.0  LENGI=1293 XINTEI=3.00 MANNI=0.013
                       SLOPEP=1.0  LENGP=1293 XINTEP=0.00 MANNP=0.250
                       TDUR=1.0 HP=1.0 HYDRAD=1.0
                       RAINFALL HYETOGRAPH INCHES PER HR
                         1.20 2.40 2.40 2.40 0.60 0.00 0.60 0.00
                         0.60 1.20 0.00 0.60 0.00 0.60 1.20 0.00
                         0.60 1.20 0.00 0.00 0.60 1.20 2.40 1.80
                         1.80 3.00 2.40 1.20 0.60 1.20 0.60 0.00
                         0.60 0.00 0.00 0.60 0.00 0.60
     TOTAL RAINFALL =          0.5700 INCHES
    F (  1)=         0.0
    STORAGE COEFF. SC =    0.0807 HOURS
    UNIT PEAK(QMX) =      11.2005 CFS
    INTENSITY=       3.0000 IN/HR
    RATIO=       0.3400

    STORAGE COEFF. K =        4.8397 MINS
    SUM OF UNIT HYDROGRAPH CO-ORDINATES=      59.996
    STORAGE COEFF. SC =   39.3571 HOURS
    UNIT PEAK(QMX) =        0.0254 CFS
    INTENSITY=       0.0001 IN/HR
    RATIO=       0.6600

    STORAGE COEFF. K =  2361.4287 MINS
    SUM OF UNIT HYDROGRAPH CO-ORDINATES=      4.849
    DEPRESSION STORAGE(IMP)=    0.020  (PERV)=    0.180
    SUM OF RUNOFF CO-ORDINATES =      651.608
    PEAK DISCHARGE=    33.9100 CPS   RUNOFF VOLUME=   0.1870 INCHES   TIME TO PEAK= 19.5333
```

HYD ID=2

PARTIAL HYDROGRAPH 102

TIME HRS	FLOW CFS	TIME HRS	FLOW CFS	TIME HRS	FLOW CFS	TIME HRS	FLOW CFS	TIME HRS	FLOW CFS
19.083	[illegible]	19.883	1.30	20.683	0.0	21.483	0.0	22.283	0.0
19.100	[illegible]	19.900	1.05	20.700	0.0	21.499	0.0	22.299	0.0
19.117	6.85	19.916	0.86	20.716	0.0	21.516	0.0	22.316	0.0
19.133	16.04	19.933	0.70	20.733	0.0	21.533	0.0	22.333	0.0
19.150	21.89	19.950	0.57	20.750	0.0	21.549	0.0	22.349	0.0
19.167	[illegible]	19.966	0.46	20.766	0.0	21.566	0.0	22.366	0.0
19.183	16.28	19.983	0.37	20.783	0.0	21.583	0.0	22.383	0.0
19.200	15.45	20.000	0.30	20.800	0.0	21.599	0.0	22.399	0.0
19.217	12.57	20.016	0.25	20.816	0.0	21.616	0.0	22.416	0.0
19.233	12.43	20.033	0.20	20.833	0.0	21.633	0.0	22.433	0.0
19.250	14.54	20.050	0.16	20.850	0.0	21.649	0.0	22.449	0.0
19.267	11.83	20.066	0.13	20.866	0.0	21.666	0.0	22.466	0.0
19.283	11.83	20.083	0.11	20.883	0.0	21.683	0.0	22.482	0.0
19.300	9.62	20.100	0.09	20.900	0.0	21.699	0.0	22.499	0.0
19.317	10.04	20.116	0.07	20.916	0.0	21.716	0.0	22.516	0.0
19.333	12.59	20.133	0.06	20.933	0.0	21.733	0.0	22.532	0.0
19.350	10.34	20.150	0.05	20.950	0.0	21.749	0.0	22.549	0.0
19.367	10.34	20.166	0.04	20.966	0.0	21.766	0.0	22.566	0.0
19.383	12.99	20.183	0.03	20.983	0.0	21.783	0.0	22.582	0.0
19.400	10.57	20.200	0.03	21.000	0.0	21.799	0.0	22.599	0.0
19.417	8.60	20.216	0.02	21.016	0.0	21.816	0.0	22.616	0.0
19.433	9.23	20.233	0.02	21.033	0.0	21.833	0.0	22.632	0.0
19.450	11.91	20.250	0.01	21.050	0.0	21.849	0.0	22.649	0.0
19.467	10.53	20.266	0.01	21.066	0.0	21.866	0.0	22.666	0.0
19.483	21.71	20.283	0.01	21.083	0.0	21.883	0.0	22.682	0.0
19.500	24.39	20.300	0.01	21.100	0.0	21.899	0.0	22.699	0.0
19.517	30.82	20.316	0.00	21.116	0.0	21.916	0.0	22.716	0.0
19.533	33.91	20.333	0.00	21.133	0.0	21.933	0.0	22.732	0.0
19.550	32.00	20.350	0.00	21.149	0.0	21.949	0.0	22.749	0.0
19.567	28.24	20.366	0.00	21.166	0.0	21.966	0.0	22.766	0.0
19.583	27.39	20.383	0.00	21.183	0.0	21.983	0.0	22.782	0.0
19.600	24.49	20.400	0.00	21.199	0.0	21.999	0.0	22.799	0.0
19.617	19.92	20.416	0.00	21.216	0.0	22.016	0.0	22.816	0.0
19.633	18.41	20.433	0.00	21.233	0.0	22.033	0.0	22.832	0.0
19.650	14.98	20.450	0.00	21.249	0.0	22.049	0.0	22.849	0.0
19.667	12.18	20.466	0.00	21.266	0.0	22.066	0.0	22.866	0.0
19.683	12.12	20.483	0.00	21.283	0.0	22.083	0.0	22.882	0.0
19.700	9.86	20.500	0.0	21.299	0.0	22.099	0.0	22.899	0.0
19.717	10.23	20.516	0.0	21.316	0.0	22.116	0.0	22.916	0.0
19.733	8.32	20.533	0.0	21.333	0.0	22.133	0.0	22.932	0.0
19.750	6.77	20.550	0.0	21.349	0.0	22.149	0.0	22.949	0.0
19.766	5.50	20.566	0.0	21.366	0.0	22.166	0.0	22.966	0.0
19.783	4.48	20.583	0.0	21.383	0.0	22.183	0.0	22.982	0.0
19.800	3.64	20.600	0.0	21.399	0.0	22.199	0.0	22.999	0.0
19.816	2.96	20.616	0.0	21.416	0.0	22.216	0.0	23.016	0.0
19.833	2.41	20.633	0.0	21.433	0.0	22.233	0.0		
19.850	1.96	20.650	0.0	21.449	0.0	22.249	0.0		
19.866	1.59	20.666	0.0	21.466	0.0	22.266	0.0		

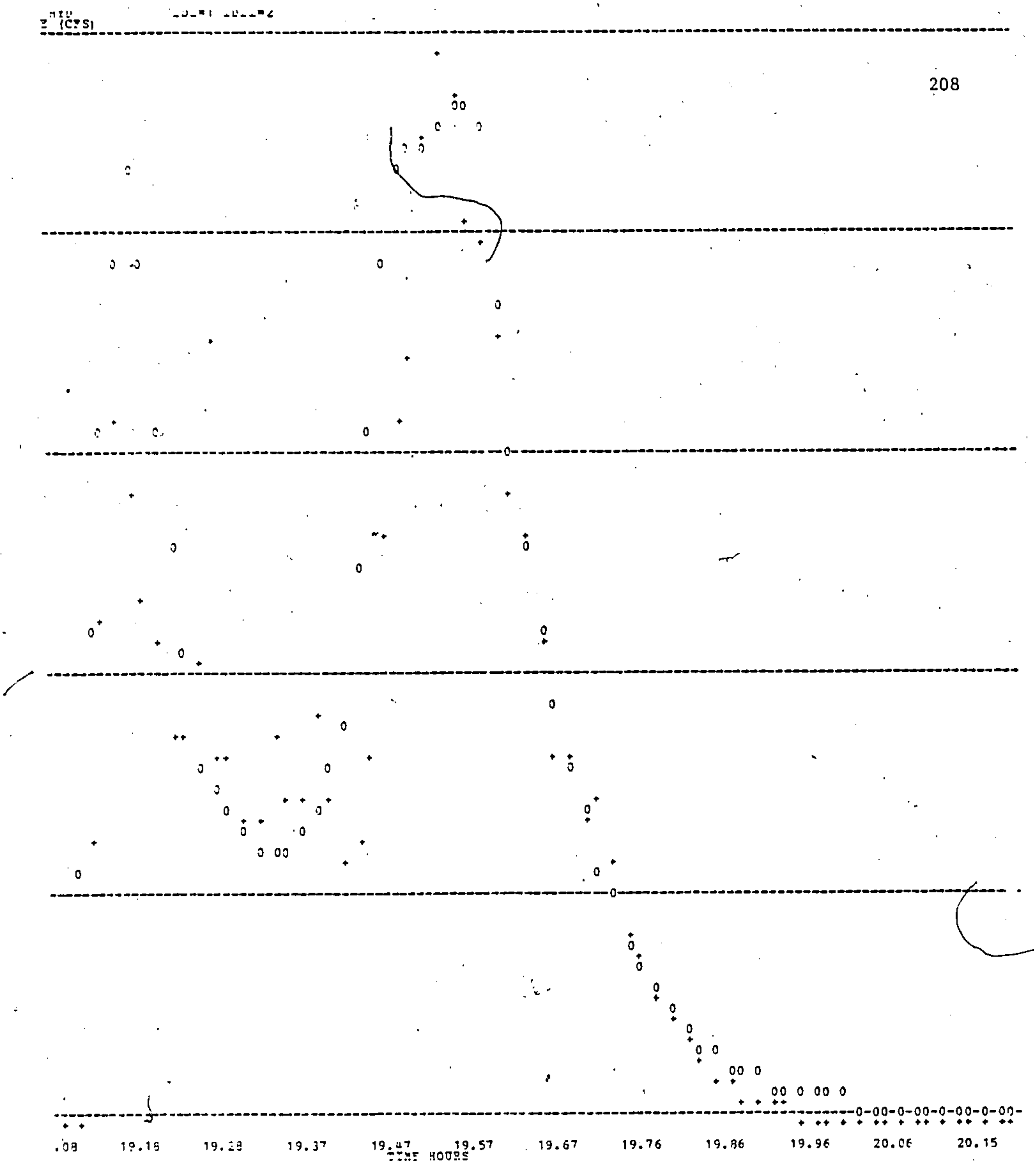
RUNOFF VOLUME = 0.187 INCHES
PEAK DISCHARGE RATE = 33.91 CFS

ERROR ANALYSIS   ID1=1 ID2=2

TIME HRS	FLOW 1 CFS	FLOW 2 CFS	ERROR CFS
19.083	0.	0.	0.
19.100	8.	0.	9.
19.117	15.	9.	5.
19.133	22.	16.	6.
19.150	27.	22.	5.
19.167	30.	20.	10.
19.183	27.	16.	10.
19.200	32.	15.	6.
19.217	18.	13.	6.
19.233	15.	12.	2.
19.250	12.	15.	-3.
19.267	11.	12.	-1.
19.283	10.	12.	-2.
19.300	9.	10.	-0.
19.317	8.	10.	-2.
19.333	8.	13.	-4.
19.350	9.	10.	-1.
19.367	9.	11.	-1.
19.383	10.	13.	-3.
19.400	11.	11.	0.
19.417	12.	9.	4.
19.433	17.	9.	9.
19.450	22.	12.	10.
19.467	27.	19.	8.
19.483	30.	22.	9.
19.500	30.	24.	6.
19.517	30.	31.	-0.
19.533	31.	34.	-3.
19.550	31.	32.	-1.
19.567	32.	28.	4.
19.583	31.	27.	4.
19.600	26.	24.	1.
19.617	20.	20.	1.
19.633	18.	18.	-1.
19.650	15.	15.	0.
19.667	13.	12.	1.
19.683	11.	12.	-1.
19.700	10.	10.	-0.
19.717	8.	10.	-2.
19.733	7.	8.	-1.
19.750	6.	7.	-1.
19.766	5.	6.	-0.
19.783	4.	4.	-0.
19.800	4.	4.	0.
19.816	3.	3.	0.
19.833	2.	2.	0.
19.850	2.	2.	0.
19.866	2.	2.	0.
19.883	2.	1.	0.
19.900	1.	1.	0.
19.916	1.	1.	0.
19.933	1.	1.	0.
19.950	1.	1.	0.
19.966	1.	0.	0.
19.983	1.	0.	0.
20.000	1.	0.	0.
20.016	1.	0.	0.
20.033	1.	0.	0.
20.050	0.	0.	0.
20.066	0.	0.	0.
20.083	0.	0.	0.
20.100	0.	0.	0.
20.116	0.	0.	0.
20.133	0.	0.	0.
20.150	0.	0.	0.
20.166	0.	0.	0.
20.183	0.	0.	0.
20.200	0.	0.	0.
20.216	0.	0.	0.
20.233	0.	0.	0.
20.250	0.	0.	0.

SUM OF SQUARES OF ERROR= 861.2124
ERROR VARIANCE= 12.1298
ERROR STANDARD DEVIATION = 3.483
PEAK DISCHARGE ERROR = 6.57 PERCENT

PEAK DIFFERENCE/PEAK(ID1) = 0.0657
TPEAK DIFFERENCE = 0.0333 HRS
TPEAK DIFFERENCE/TPEAK(ID1) = 0.0017
ERROR IN PEAK AND TP - PEEP= 0.0657
TPEAK1(1)= 19.5667TPEAK2(2)= 19.5333
PEAK(ID1) = 31.8200 CFS PEAK(ID2) = 33.9130 CFS



PLOT HYD   ID I=1   ID II=2

Appendix D

EXAMPLE OF URBHYMO SIMULATION WITH OPTIMIZATION PROCEDURE

MINIMUM OF FUNCTION OF 2 VARIABLES WITH 2 CONSTRAINTS

NO. OF CYCLES	CURRENT TRIAL VALUE OF IND.VARIABLES	LAST SUCCESSFUL FUNCTIONAL VALUE	TOTAL DISTANCE MOVED DURING STAGE	CHANGE IN DIRECTION OF MOTION FROM LAST STAGE
0	0.7999998D-01 0.2000000	2.036455	0.0	0.0
1	0.7999998D-01	2.03646		
2	0.7999998D-01	2.03646		0.2000000
3	0.7999998D-01	2.03646		0.2000000
4	0.7999998D-01	2.03646		0.2000000
5	0.7999998D-01	2.03646		0.2000000
6	0.6749998D-01	1.28834		0.2000000
7	0.6749998D-01	0.176452		0.2000000
				0.1875000
8	0.6749998D-01 0.1875000	0.1764515	0.1767767D-01	0.7071068
9	0.6749998D-01	0.176452		
10	0.6749998D-01	0.176452		0.1875000
11	0.6749998D-01	0.176452		0.1875000
12	0.6749998D-01	0.176452		0.1875000
13	0.6749998D-01	0.176452		0.1875000
14	0.6749998D-01	0.176452		0.1875000
15	0.6915726D-01	0.117551		0.1875000
16	0.6915726D-01	0.117551		0.1891573
17	0.6915726D-01	0.117551		0.1891573
18	0.6998591D-01	0.657974D-01		0.1891573
19	0.6998591D-01	0.657974D-01		0.1883286
				0.1883286
20	0.6998591D-01 0.1883286	0.6579745D-01	0.2620392D-02	0.4472136
21	0.6998591D-01	0.657974D-01		
22	0.6998591D-01	0.657974D-01		0.1883286
23	0.6998591D-01	0.657974D-01		0.1883286
24	0.7026384D-01	0.186420D-01		0.1883286
25	0.7026384D-01	0.186420D-01		0.1874968
26	0.7026384D-01	0.186420D-01		0.1874968
27	0.7047229D-01	0.183522D-02		0.1874968
28	0.7047229D-01	0.183522D-02		0.1875643
				0.1875643
29	0.7047229D-01 0.1875643	0.1835219D-02	0.9059558D-03	0.9701425
30	0.7047229D-01	0.183522D-02		
31	0.7047229D-01	0.183522D-02		0.1875643
32	0.7047229D-01	0.183522D-02		0.1875643
33	0.7047229D-01	0.183522D-02		0.1875643
34	0.7047229D-01	0.183522D-02		0.1875643
35	0.7047229D-01	0.183522D-02		0.1875643
36	0.7049441D-01	0.110113D-02		0.1875643
37	0.7049441D-01	0.110113D-02		0.1875296
38	0.7049441D-01	0.110113D-02		0.1875296
39	0.7049441D-01	0.110113D-02		0.1875296
40	0.7049441D-01	0.110113D-02		0.1875296
41	0.7049441D-01	0.110113D-02		0.1875296
42	0.7049441D-01	0.110113D-02		0.1875296
43	0.7049441D-01	0.110113D-02		0.1875296
44	0.7048612D-01	0.386362D-04		0.1875296
45	0.7048612D-01	0.386362D-04		0.1875426
46	0.7048612D-01	0.386362D-04		0.1875426
47	0.7048612D-01	0.386362D-04		0.1875426
48	0.7048612D-01	0.386362D-04		0.1875426
49	0.7048612D-01	0.386362D-04		0.1875426
50	0.7048612D-01	0.386362D-04		0.1875426
				0.1875426
51	0.7048612D-01 0.1875426	0.3863619D-04	0.9059558D-03	0.9701425

CONVERGENCE TO 5 SIGNIFICANT FIGURES REACHED IN ALL INDEPENDENT VARIABLES.

PARTIAL HYDROGRAPH 102

TIME HRS	FLOW CFS	TIME HRS	FLOW CFS	TIME HRS	FLOW CFS	TIME HRS	FLOW CFS	TIME HRS	
0.0	0.0	0.517	71.43	1.033	5.61	1.550	0.33	2.067	
0.017	0.0	0.533	73.49	1.050	5.08	1.567	0.30	2.083	
0.033	7.84	0.550	69.46	1.067	4.60	1.583	0.42	2.100	
0.050	13.89	0.567	68.69	1.083	4.18	1.600	0.33	2.117	
0.067	18.15	0.583	71.43	1.100	3.79	1.617	0.31	2.133	
0.083	20.08	0.600	74.83	1.117	3.45	1.633	0.21	2.150	
0.100	20.29	0.617	78.99	1.133	3.14	1.650	0.19	2.167	
0.117	19.67	0.633	77.05	1.150	2.86	1.667	0.18	2.183	
0.133	18.40	0.650	75.46	1.167	2.61	1.683	0.16	2.200	
0.150	16.62	0.667	71.21	1.183	2.37	1.700	0.15	2.217	
0.167	17.04	0.683	65.53	1.200	2.17	1.717	0.14	2.233	
0.183	17.28	0.700	60.32	1.217	1.98	1.733	0.12	2.250	
0.200	16.88	0.717	55.50	1.233	1.80	1.750	0.11	2.267	
0.217	15.75	0.733	49.53	1.250	1.65	1.767	0.10	2.283	
0.233	14.53	0.750	42.84	1.267	1.50	1.783	0.09	2.300	
0.250	16.04	0.767	37.28	1.283	1.37	1.800	0.08	2.317	
0.267	17.49	0.783	32.19	1.300	1.26	1.817	0.08	2.333	
0.283	18.25	0.800	28.29	1.317	1.15	1.833	0.07	2.350	
0.300	18.75	0.817	24.42	1.333	1.05	1.850	0.07	2.367	
0.317	19.61	0.833	21.84	1.350	0.97	1.867	0.06	2.383	
0.333	21.23	0.850	19.26	1.367	0.88	1.883	0.06	2.400	
0.350	24.38	0.867	17.04	1.383	0.80	1.900	0.05	2.417	
0.367	27.50	0.883	15.10	1.400	0.73	1.917	0.05	2.433	
0.383	30.62	0.900	13.43	1.417	0.67	1.933	0.04	2.450	
0.400	34.54	0.917	11.92	1.433	0.61	1.950	0.04	2.467	
0.417	38.40	0.933	10.23	1.450	0.56	1.967	0.04	2.483	
0.433	38.77	0.950	8.88	1.467	0.51	1.983	0.03	2.500	
0.450	45.30	0.967	7.51	1.483	0.47	2.000	0.03		
0.467	49.37	0.983	6.64	1.500	0.43	2.017	0.02		
0.483	61.29	1.000	6.58	1.517	0.39	2.033	0.02		
0.500	68.53	1.017	6.21	1.533	0.36	2.050	0.02		

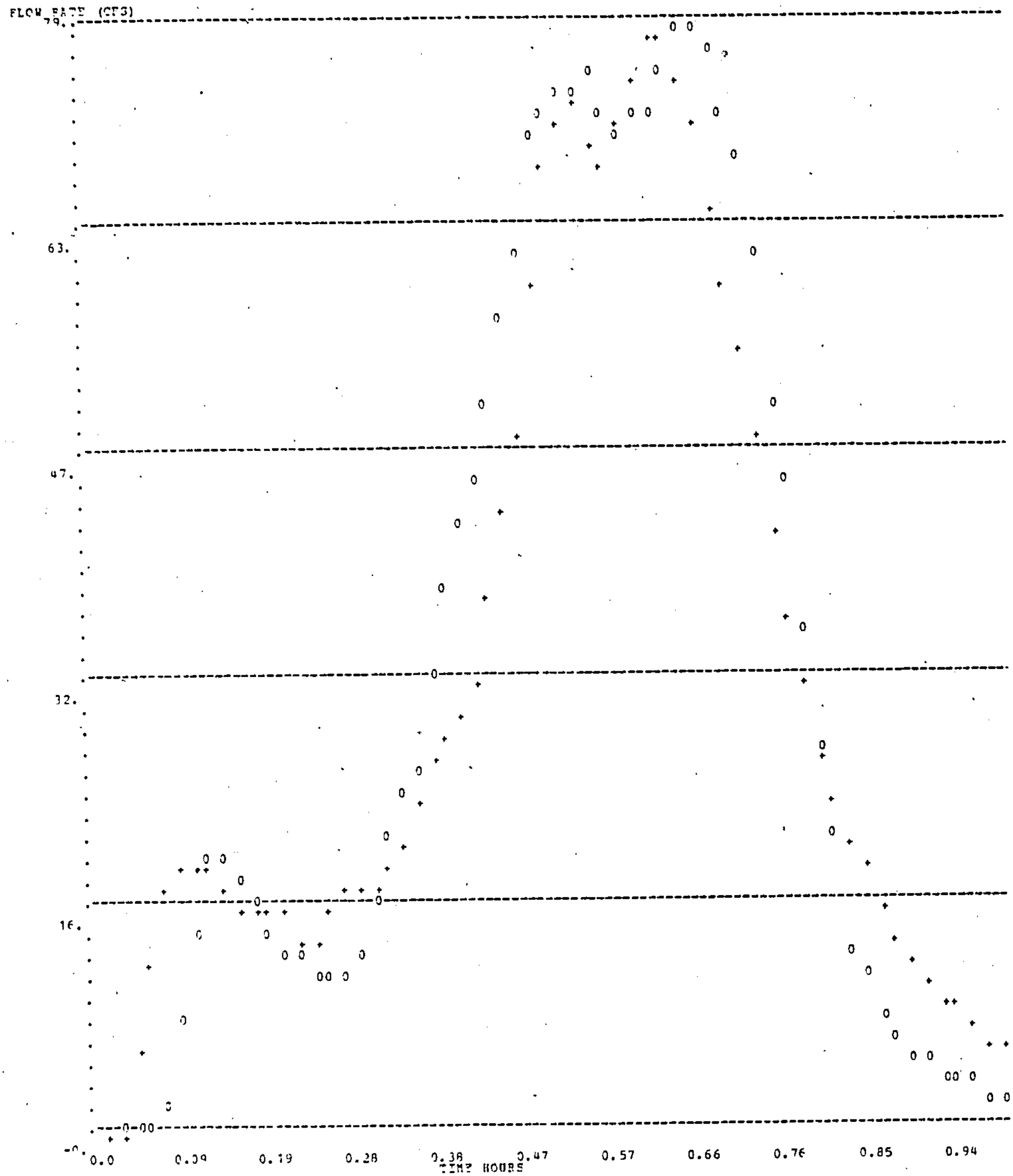
RUNOFF VOLUME = 1.471 INCHES
PEAK DISCHARGE RATE = 78.99 CFS

PLOT HYD   ID I=1 ID II=2

TIME HRS	FLOW 1 CPS	FLOW 2 CPS	ERROR CPS
0.000	0.	0.	0.
0.017	0.	0.	0.
0.033	0.	8.	-8.
0.050	0.	14.	-14.
0.067	2.	18.	-16.
0.083	8.	20.	-12.
0.100	15.	20.	-5.
0.117	19.	20.	-0.
0.133	19.	18.	1.
0.150	18.	17.	1.
0.167	17.	17.	-0.
0.183	15.	17.	-2.
0.200	14.	16.	-3.
0.217	13.	16.	-3.
0.233	12.	15.	-2.
0.250	12.	16.	-4.
0.267	12.	17.	-5.
0.283	14.	18.	-4.
0.300	17.	19.	-1.
0.317	21.	20.	1.
0.333	25.	21.	4.
0.350	27.	24.	3.
0.367	32.	27.	5.
0.383	39.	29.	10.
0.400	44.	31.	13.
0.417	47.	32.	14.
0.433	51.	39.	12.
0.450	57.	45.	12.
0.467	62.	50.	12.
0.483	70.	60.	10.
0.500	72.	69.	4.
0.517	74.	71.	2.
0.533	73.	74.	-1.
0.550	74.	70.	5.
0.567	72.	69.	4.
0.583	71.	71.	-1.
0.600	72.	75.	-3.
0.617	72.	79.	-7.
0.633	75.	78.	-3.
0.650	78.	75.	3.
0.667	79.	71.	8.
0.683	77.	65.	11.
0.700	72.	60.	12.
0.717	69.	55.	14.
0.733	62.	50.	13.
0.750	52.	43.	9.
0.767	47.	37.	10.
0.783	36.	32.	3.
0.800	28.	28.	-0.
0.817	21.	25.	-4.
0.833	14.	22.	-8.
0.850	11.	19.	-8.
0.867	9.	17.	-8.
0.883	7.	15.	-8.
0.900	6.	13.	-7.
0.917	5.	12.	-7.
0.933	4.	11.	-6.
0.950	4.	9.	-6.
0.967	3.	9.	-5.
0.983	3.	8.	-5.
1.000	3.	7.	-4.
1.017	2.	6.	-4.
1.033	2.	6.	-4.
1.050	2.	5.	-3.
1.067	2.	5.	-3.
1.083	2.	4.	-3.
1.100	2.	4.	-2.
1.117	1.	3.	-2.
1.133	1.	3.	-2.

```
SUM OF SQUARES OF ERROR=          3421.48
ERROR VARIANCE=      49.5866
ERROR STANDARD DEVIATION =         7.042
PEKPE=       0.0000
TPKEP=       0.0755ZERTPK=       0.0503
ERROR IN PEAK AND TP - PERS=        0.0755
PEAK(ID1)  =         78.9870 CPS    PEAK(ID2)  =      78.9870 CPS
DIFF. IN P.O.VOL =        0.0061 IN    % DIFF. IN P.O.VOL =       0.4156
PEAK DISCHARGE ERROR =      0.00 PERCENT
```

✓


FLOW RATE (CFS)
79.
63.
47.
32.
16.
-0.
0.0
0.09
0.19
0.28
0.38
0.47
0.57
0.66
0.76
0.85
0.94
TIME HOURS

```
0.01666667
    0.0
    0.0
    0.0
    0.00
    2.26
    7.92
   15.15
   19.34
   19.34
   17.94
   16.54
   14.91
   13.98
   13.05
   12.35
   11.88
   12.35
   13.98
   17.24
   20.74
   25.16
   26.80
   32.15
   39.14
   43.80
   46.83
   50.79
   57.09
   62.44
   70.13
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   73.63
   73.16
   74.33
   72.46
   70.83
   72.00
   72.46
   74.79
   78.30
   78.99
   76.66
   72.46
   69.20
   62.44
   51.73
   47.30
   35.65
   27.96
   20.74
   13.98
   11.18
    8.62
    7.22
    6.29
    5.36
    4.43
    3.73
    3.26
    3.03
    2.56
    2.10
    2.10
    2.10
    1.86
    1.63
    1.63
    1.40
    1.40
    1.40
     78.99    69      1.4650      0.6670
      0.44         0.52
   X(I)          G(I)          H(I)
     0.070         0.030         0.500
     0.188         0.060         1.000
```

Appendix E

A SINGLE LINEAR RESERVOIR MODEL FOR PROGRAMMABLE CALCULATORS

FLOW CHART FOR IMPSWM-SLR MODEL

Enter F (infiltrated vol.) (in.) due to AMC

Calculate infiltration rate
$f_{cap}(t) = f_o - K[F(t) - f_c . t]$
$f(t) = P(t) \quad P(t) \leq f_{cap}(t)$
$f(t) = f_{cap}(t) \quad P(t) > f_{cap}(t)$

Enter Storm Hyetograph

Subtract depression storage

Obtain Rainfall rate $P(t)$ (in/hr.) for each time period Δt

Calculate Rainfall Excess Rate $R(t)$ (in/hr.) for each time period Δt

Directly Connected Impervious Area Runoff
$R(0) = I . P(t)$ in./hr.

Pervious Area Runoff
$R(1) = (1-I)[P(t) - f(t)]$ in./hr.

Total Runoff
$R(t) = R(0) + R(1)$ in./hr.

Determination of an 'instantaneous' hydrograph.
$I(t) = R(t) . A$ cfs.

Calculate storage co-efficient K of single linear reservoir
3 possible options

$K_1 = 0.0431\ A^{.458} (1 + I_{tot.})^{-1.662} P_E^{-.267} T_R^{.371}$

$K_2 = 0.0374\ A^{.49} (1 + I_{tot.})^{-1.683} P_E^{-.24} T_R^{.294}$

$K_3 = 0.0561\ A^{.409} (1 + I_{tot.})^{-2.06} P_E^{-.15} T_R^{.156}$

Routing of 'instantaneous' hydrograph through a single linear reservoir with storage coefficient K.

$$Q(2) = Q(1) + \frac{\Delta t}{2K + \Delta t}[I(1) + I(2) - 2Q(1)]$$

Obtain outflow hydrograph

STOP

User Instructions

SLR MODEL (1)

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STEP	INSTRUCTIONS	INPUT DATA UNITS	KEYS		OUTPUT DATA UNITS
A	Enter rainfall increments (maximum of 27 in				
	registersR11-R19.				
	(2 decimal places; min.=0.01 in/hr.,				
	max.=9.99 in/hr.				
	1. Load side 1 of card no. 1				
	2. Place first rainfall increment in	P(1)	A		
	X-register.	(in/hr.)			
	3. Place second rainfall increment in	P(2)	R/S		
	X-register.	(in/hr.)			
	4. Place third rainfall increment in	P(3)	R/S		
	X-register.	(in/hr.)			
	5. Continue to place successive rainfall	P(n)	R/S		
	increments in X-register.	(in/hr.)			
B	To record rainfall data for subsequent future		f	W/DATA	
	use, press f W/DATA and pass blank magnetic				
	card through calculator (not necessary if data				
	not to be saved).				
C	To compute rainfall excess				
1.	Load sides 1 and 2 of card no. 2				
2.	Input rainfall increments into calculator by	P's			
	loading both sides of rainfall data card — only	(in/hr.)			
	if rainfall increments not entered in steps A				
	and B above.				

User Instructions

SLR MODEL (2)

STEP	INSTRUCTIONS	INPUT DATA UNITS	KEYS		OUTPUT DATA UNITS
3.	Input known values				
			f	P⪌S	
	a) Depression Storage	DEPSTO(in)	STO	0	
			f	P⪌S	
	b) Accumulated Infiltration due to AMC	F (in.)	STO	1	
	c) Time step	Δt (hr.)	STO	3	
	d) Initial infiltration rate	fo(in/hr.)	STO	C	
	e) Final infiltration rate	fc(in/hr.)	STO	D	
	f) Directly connected Impervious Area	I(decimal	STO	A	
4.	To obtain rainfall excess (in 3 dec. places)		E		Δt (hrs.)
					R(1)(in/hr.)
	(several minutes running time before				
	output begins)				2Δt (hrs)
					R(2)(in/hr)
					3Δt (hrs)
					R(3)(in/hr)
					nΔt (hrs)
					R(n)(in/hr)
5.	To print supplementary information		f	c	P_E (in)
	P_E - rainfall excess				TR (hr)
	TR - duration of rainfall excess				F (in)
	F - accumulated infiltration				ΣΔt (hr)
	Δt - total time in which infiltration				
	takes place				
D	For n>27, require 2nd. run. To continue				
	computation of rainfall excess				
	1. Clear registers		f	d	
	2. Load side 1 of card no. 1				
	3. Place rainfall increment in X-register	P (in/hr)	A		

User Instructions

SLR MODEL (3)

STEP	INSTRUCTIONS	INPUT DATA UNITS	KEYS		OUTPUT DATA UNITS
	4. Continue to place successive rainfall	P	R/S		
	increments in X-register	(in/hr.)			
	5. To record rainfall data for subsequent				
	future use, press f W/DATA and pass blank				
	magnetic card through calculator.				
	6. Load sides 1 and 2 of card no. 2				
	7. Input rainfall increments into calculator				
	by loading both sides of rainfall data card				
	(only if not entered in steps 2-4 above)				
	8. Input known values				
	(Depression storage = o and is not required				
	as input since it is already subtracted				
	at the beginning of the storm in the				
	first run).				
	a) Accumulated infiltration	F (in)	STO	1	
	(from supplementary information of 1st. run)				
	b) Time step	Δt (hr)	STO	3	
	c) Total time in which infiltration takes place	$\Sigma\Delta t$ (hr)	STO	8	
	(from supplementary information of 1st.run)				
	d) Last time step in the 1st. run	t (hr)	STO	9	
	e) Initial infiltration rate		STO	C	
	f) Final infiltration rate		STO	D	
	g) Directly connected impervious area	I(decimal)	STO	A	
	7. To obtain rainfall excess ordinates.		f	b	$n\Delta t$ (hrs)
					R_n (in/hr)
	8. To print supplementary information		f	c	P_E (in)
					T_R (hr)
	(Sum of P_E, sum of T_R from both runs are				F (in)
	used as input for card no. 3)				$\Sigma\Delta t$ (hr)

User Instructions

1 SLR MODEL (4) 2

STEP	INSTRUCTIONS	INPUT DATA UNITS	KEYS		OUTPUT DATA UNITS
A	Enter rainfall excess increments (maximum of 26) which are obtained from output of card no.2 (3 decimal places; min. - 0.001 in/hr., max. - 9.999 in/hr)				
	1. Load side 1 of card no. 1A (Card no. 1A is the same as card no. 1 except that it takes input of 3 decimal places).				
	2. Place first rainfall excess increment in X-register	R(1) (in/hr)	A		
	3. Place second rainfall excess increment in X-register.	R(2) (in/hr)	R/S		
	4. Continue to place successive rainfall increments in X-register.	R(n) (in/hr)	R/S		
B	To record rainfall excess data for subsequent future use, press f W/DATA and pass blank magnetic card through calculator (not necessary if data not to be saved).		f	W/DATA	
C	To compute hydrograph ordinates				
1.	Load sides 1 and 2 of card no. 3.				
2.	Input rainfall excess increments into calculator by loading both sides of rainfall excess data card - only if rainfall increments not entered in steps A and B above.	R's (in/hr)			

User Instructions

SLR MODEL (5)

220

STEP	INSTRUCTIONS	INPUT DATA UNITS	KEYS		OUTPUT DATA UNITS
3.	Input known values				
	a) Area	A (acres)	STO	0	
	b) Total impervious Area	I_m (decimal)	STO	1	
	c) Rainfall excess	P_E (in)	STO	2	
	d) Drration of rainfall excess	T_R (hr)	STO	3	
	e) Time step	Δt (hr)	STO	5	
	f) Time before the start of runoff	$\Sigma\Delta t$ (hr)	STO	9	
4.	To obtain hydrograph ordinates		E		Δt (hrs)
					Q(1)(cfs)
					$2\Delta t$ (hrs)
					Q(2) (cfs)
					$n\Delta t$ (hrs)
					Q(n) (cfs)
5.	To compute runoff volume		A		R.O. Vol (in)
6.	To print supplementary information		B		I(t) (hrs)
	I(t) - last inflow into linear reservoir				ΣQ(t) (cfs)
	Q(t) - sum of hydrograph ordinates				K (hrs)
	K Storage Coefficient				
D	If n >26, require 2nd. run				
	Continue computation of hydrograph ordinates				
1.	Clear registers		D		
2.	Load side 1 of card no. 1A -				
3.	Place rainfall excess increment in X-register		A		
4.	Continue to place successive rainfall increments in X-register		R/S		

User Instructions

SLR MODEL (6)

221

STEP	INSTRUCTIONS	INPUT DATA UNITS	KEYS		OUTPUT DATA UNITS
5.	To record rainfall excess data for subsequent				
	future use press f W/DATA and pass blank				
	magnetic card through calculator.				
6.	To compute hydrograph ordinates				
	Input known values				
	a) Area	A (acres)	STO	0	
	b) Total impervious area	I_{tot} (decimal)	STO	1	
	c) Rainfall excess	P_E (in)	STO	2	
	d) Duration of rainfall excess	T_R (hr)	STO	3	
	e) Time step	Δt (hr)	STO	5	
	f) last inflow into linear reservoir	I(t) (cfs)	STO	6	
	(from supplementary information of 1st.run)				
	g) sum of hydrograph ordinates	$\Sigma Q(t)$ (cfs)	STO	8	
	(from supplementary information of 1st.run)				
	h) Last time step printed on 1st.run		STO	9	
	i) Last hydrograph ordinate printed on 1st.run		STO	E	
7.	To obtain hydrograph ordinates		E		$n\Delta t$ (hr)
					Q(n) (cfs)
8.	To compute runoff volume		A		R.O.Vol
					(in)

Listing of HP - 97 programs for the SLR model

CARD NO.1

```
001  *LBLA     21 11
002     1         01
003     1         01
004  STOI      35 46
005    R↓        -31
006  *LBL1     21 01
007     1         01
008     0         00
009     ÷        -24
010  STOI      35 46
011     1         01
012  ST+0   35-55 00
013  RCL0      36 00
014  DSP0     -63 00
015   PSE      16 51
016  RCLI      36 46
017  DSP9     -63 09
018   R/S         51
019  ENT↑        -21
020   EEX        -23
021     4         04
022     ÷        -24
023  ST+I   35-55 46
024     1         01
025  ST+0   35-55 00
026  RCL0      36 00
027  DSP0     -63 00
028   PSE      16 51
029  RCLI      36 46
030  DSP9     -63 09
031   R/S         51
032  ENT↑        -21
033   EEX        -23
034     7         07
035     ÷        -24
036  ST+I   35-55 46
037     1         01
038  ST+0   35-55 00
039  RCL0      36 00
040  DSP0     -63 00
041   PSE      16 51
042  RCLI      36 46
043  DSP9     -63 09
044   R/S         51
045  ISZI   16 26 46
046  GTO1      22 01
047   R/S         51
```

CARD NO.1A

```
001  *LBLA     21 11
002     1         01
003     1         01
004  STOI      35 46
005    R↓        -31
006  *LBL1     21 01
007     1         01
008     0         00
009     ÷        -24
010  STOI      35 46
011     1         01
012  ST+0   35-55 00
013  RCL0      36 00
014  DSP0     -63 00
015   PSE      16 51
016  RCLI      36 46
017  DSP9     -63 09
018   R/S         51
019  ENT↑        -21
020   EEX        -23
021     5         05
022     ÷        -24
023  ST+I   35-55 46
024     1         01
025  ST+0   35-55 00
026  RCL0      36 00
027  DSP0     -63 00
028   PSE      16 51
029  RCLI      36 46
030  DSP9     -63 09
031   R/S         51
032  ISZI   16 26 46
033  GTO1      22 01
034   R/S         51
```

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Card No. 2

001	*LBLE	21 15
002	.	-62
003	3	03
004	1	01
005	GSB3	23 03
006	RCL4	36 14
007	×	-35
008	STO4	35 04
009	.	-62
010	0	00
011	1	01
012	×	-35
013	ST×2	35-35 02
014	RCL3	36 03
015	RCL1	36 01
016	X>Y?	16-34
017	GTOE	22 15
018	2	02
019	STO3	35 03
020	STO4	35 04
021	*LBLb	21 16 12
022	1	01
023	1	01
024	STOI	35 46
025	*LBL1	21 01
026	RCLi	36 45
027	EEX	-23
028	3	03
029	×	-35
030	INT	16 34
031	1	01
032	0	00
033	0	00
034	÷	-24
035	X=0?	16-43
036	GTO2	22 02
037	STO7	35 07
038	GSB4	23 04
039	RCLi	36 45
040	EEX	-23
041	3	03
042	×	-35
043	FRC	16 44
044	EEX	-23
045	3	03
046	×	-35
047	INT	16 34
048	1	01
049	0	00
050	0	00
051	÷	-24
052	X=0?	16-43
053	GTO2	22 02
054	STO7	35 07
055	GSB4	23 04
056	RCLi	36 45
057	EEX	-23
058	3	03
059	×	-35
060	FRC	16 44
061	1	01
062	0	00
063	×	-35
064	X=0?	16-43
065	GTO2	22 02
066	STO7	35 07
067	GSB4	23 04
068	1	01
069	5	05
070	RCLI	36 46
071	X=Y?	16-33
072	R/S	51
073	ISZI	16 26 46
074	GTO1	22 01
075	*LBL2	21 02
076	STO7	35 07
077	*LBL3	21 03
078	GSB4	23 04
079	GTO3	22 03
080	*LBL4	21 04
081	DSP3	-63 03
082	RCL5	36 05
083	P⇄S	16-51
084	RCL0	36 00
085	P⇄S	16-51
086	X>Y?	16-34
087	GTOa	22 11
088	GTOb	22 12
089	*LBLa	21 11
090	RCL7	36 07
091	RCL3	36 03
092	×	-35
093	ST+5	35-55 05
094	P⇄S	16-51
095	RCL0	36 00
096	P⇄S	16-51
097	RCL5	36 05
098	X≤Y?	16-35
099	GTOc	22 13
100	P⇄S	16-51

CARD NO.2

101	RCL0	36 00
102	P⇌S	16-51
103	-	-45
104	STO7	35 07
105	GTOD	22 14
106	*LBLB	21 12
107	RCL7	36 07
108	ST+8	35-55 08
109	*LBLD	21 14
110	RCL0	36 00
111	RCLD	36 14
112	×	-35
113	CHS	-22
114	RCL1	36 01
115	+	-55
116	4	04
117	.	-62
118	1	01
119	4	04
120	×	-35
121	CHS	-22
122	RCLC	36 13
123	+	-55
124	STO0	35 00
125	RCL7	36 07
126	X≤Y?	16-35
127	GTO5	22 05
128	RCL0	36 00
129	STOB	35 12
130	RCL3	36 03
131	×	-35
132	ST+1	35-55 01
133	GTO6	22 06
134	*LBL5	21 05
135	STOB	35 12
136	RCL3	36 03
137	×	-35
138	ST+1	35-55 01
139	*LBL6	21 06
140	RCLB	36 12
141	RCL7	36 07
142	X⇌Y	-41
143	-	-45
144	1	01
145	RCLA	36 11
146	-	-45
147	×	-35
148	[illegible]	16-45
149	GTO7	22 07
150	RCL7	36 07
151	RCLA	36 11
152	×	-35
153	+	-55
154	STO9	35 09
155	GTO9	22 09
156	*LBL7	21 07
157	RCL7	36 07
158	RCLA	36 11
159	×	-35
160	STO9	35 09
161	*LBL9	21 09
162	RCL3	36 03
163	ST+3	35-55 03
164	ST+4	35-55 04
165	RCL9	36 09
166	PRTX	-14
167	RCL6	36 06
168	ST+2	35-55 02
169	PRTX	-14
170	SPC	16-11
171	.	-62
172	0	00
173	0	00
174	1	01
175	X>Y?	16-34
176	GTOc	22 16 13
177	RTN	24
178	*LBLC	21 13
179	RCL3	36 03
180	ST+9	35-55 09
181	RCL9	36 09
182	PRTX	-14
183	2	02
184	PRTX	-14
185	SPC	16-11
186	RTN	24
187	*LBL3	21 03
188	ST+3	35-55 03
189	RCL3	36 03
190	4	04
191	.	-62
192	1	01
193	4	04
194	^	-35
195	CHS	-22
196	e^x	33
197	RCLC	36 13
198	RCLD	36 14
199	-	-45
200	×	-35
201	RTN	24
202	*LBLd	21 16 14
203	CLX	-51
204	CLRG	16-53
205	P⇌S	16-51
206	CLRG	16-53
207	RTN	24
208	*LBLc	21 16 13
209	SPC	16-11
210	RCL2	36 02
211	RCL3	36 03
212	×	-35
213	PRTX	-14
214	RCL4	36 04
215	PRTX	-14
216	RCL1	36 01
217	PRTX	-14
218	RCL8	36 08
219	PRTX	-14
220	R/S	51

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Card No. 3

001	*LBLE	21 15
002	RCL0	36 00
003	.	-62
004	4	04
005	0	00
006	3	03
007	Y^X	31
008	1	01
009	RCL1	36 01
010	+	-55
011	2	02
012	.	-62
013	0	00
014	6	06
015	CHS	-22
016	Y^X	31
017	×	-35
018	RCL2	36 02
019	.	-62
020	1	01
021	5	05
022	CHS	-22
023	Y^X	31
024	RCL3	36 03
025	.	-62
026	1	01
027	5	05
028	6	06
029	Y^X	31
030	×	-35
031	×	-35
032	.	-62
033	0	00
034	5	05
035	6	06
036	1	01
037	×	-35
038	STO4	35 04
039	1	01
040	1	01
041	STOI	35 46
042	*LBL1	21 01
043	RCLI	36 46
044	EEX	-23
045	4	04
046	×	-35
047	INT	16 34
048	1	01
049	0	00
050	0	00
051	0	00
052	÷	-24
053	X=0?	16-43
054	GTO4	22 04
055	STO7	35 07
056	GSB3	23 03
057	RCLI	36 46
058	EEX	-23
059	4	04
060	×	-35
061	FRC	16 44
062	EEX	-23
063	4	04
064	×	-35
065	INT	16 34
066	1	01
067	0	00
068	0	00
069	0	00
070	÷	-24
071	X=0?	16-43
072	GTO4	22 04
073	STO7	35 07
074	GSB3	23 03
075	2	02
076	3	03
077	RCLI	36 46
078	X=Y?	16-33
079	R/S	51
080	ISZI	16 26 46
081	GTO1	22 01
082	*LBL4	21 04
083	STO7	35 07
084	*LBL5	21 05
085	GSB3	23 03
086	GTO5	22 05
087	*LBL3	21 03
088	DSP2	-63 02
089	RCL7	36 07
090	RCL0	36 00
091	×	-35
092	STO7	35 07
093	RCL6	36 06
094	+	-55
095	RCLE	36 15
096	2	02
097	×	-35
098	-	-45
099	RCL4	36 04
100	2	02
101	×	-35
102	RCL5	36 05
103	+	-55
104	RCL5	36 05
105	X⇄Y	-41
106	÷	-24
107	×	-35
108	RCLE	36 15
109	+	-55
110	STOE	35 15
111	ST+8	35-55 08
112	RCL7	36 07
113	STO8	35 08
114	RCL5	36 05
115	ST+9	35-55 09
116	RCL9	36 09
117	PRTX	-14
118	RCLE	36 15
119	PRTX	-14
120	SPC	16-11
121	.	-62
122	0	00
123	1	01
124	X≤Y?	16-35
125	R/S	51
126	RTN	24
127	*LBLD	21 14
128	CLX	-51
129	CLRG	16-53
130	P⇄S	16-51
131	CLRG	16-53
132	RTN	24
133	*LBLA	21 11
134	RCL5	36 05
135	.	-62
136	5	05
137	5	05
138	1	01
139	7	07
140	4	04
141	×	-35
142	RCL8	36 08
143	×	-35
144	RCL9	36 09
145	÷	-24
146	RTN	24
147	*LBLE	21 15
148	RCL8	36 08
149	PRTX	-14
150	RCL9	36 09
151	PRTX	-14
152	RCL4	36 04
153	PRTX	-14
154	R/S	51