Research report no. 92

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Rainfall-runoff event model using curve numbers (KINFIL)

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July 1989

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RAINFALL-RUNOFF EVENT MODEL
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ABSTRACT

The task of estimating direct runoff for a small ungauged watershed is very often employed by using the Curve Number method which has been developed by the U.S. Soil Conservation Service many years ago. The evident shortcoming of this method is its weak physical background. The presented model KINFIL uses the advantages of this method in a form of the CN-values in which the soil-cover complex, land use and antecedent moisture conditions are coded. Then the Morel-Seytoux equations based on the Green and Ampt theory have been adapted for description of an infiltration process. Runoff formation and routing can be computed in the KINFIL model either as the lumped option using the unit hydrograph model or as the distributed sub-model based on a kinematic wave theory and its numerical implementation. The kinematic wave routing has been briefly reported here.

1 INTRODUCTION

One of the most popular methods for a design flood estimation on small ungauged catchments is the Curve Number method (CN method) developed by the U.S. Soil Conservation Service. The CN-method based on the soil types, vegetation cover, land use and antecedent moisture conditions is widely used due to its easy application.

The simplicity of the CN-method brings on the other hand some limits in its practical use. One of them is the disregard of both intensity and duration of rainfall which causes flood runoff. The second possible source of errors might be the stereotyped estimation of initial water abstraction before runoff starts. These imperfections can be removed through the substitution of the conventional CN-method by the physically based infiltration approach. For the solution of infiltration process, the Morel-Seytoux theory [10], [11] based on the Green and Ampt infiltration equation introducing a ponding time has been used. The correspondence between CN-values and the soil parameters implemented in the infiltration equations has been found using many soil and rainfall data. It has been shown [6] that the correspondence between CN and hydraulic conductivity (K_S) and sorptivity (SP) being originally developed by Morel-Seytoux [10] is in a good agreement with our results. The model has been further improved by introducing continuous changes of CN-values depending upon antecedent moisture conditions. Computation of antecedent moisture conditions is simple and it needs only daily rainfall data and those needed for the calculation of average daily depletion.

There are two possible options for a runoff transformation and routing: either to use the unit hydrograph method and to convolute it with the effective rainfall or to use a kinematic wave routing procedure for geometrically simplified catchment topography. The second option was prefered here.

The previous version of the model (INFIL) using the unit hydrograph method has been implemented in several small catchments in Czechoslovakia with satisfactory results [6], [7]. A new version KINFIL is being currently tested with encouraging results.

2 THE CURVE NUMBER METHOD

A direct storm runoff estimation using the Curve Number method is an international technique. The basic equations are:

$$Pe = \frac{(P - I_a)^2}{P - I_a + S} = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad (1)$$

where P....rainfall depth (mm)

Pe...effective rainfall depth (mm)

S....catchment storage (mm)

I_a...initial abstraction (mm)

Eq. (1) holds for all $P \geq 0.2S$. Parameter representing the storage of an active zone is then transformed by the relationship:

$$CN = \frac{25400}{S + 254} \qquad (2)$$

in which CN <0,100> is called the "curve number". Any catchment condition can be defined by a value of S that can then be described by the CN-value varying between 0 (for $S \to \infty$) and 100 (for $S = 0$).
The CN-value is determined from a knowledge of the soil types and land use. The first and the most important step in the determination of CN for a catchment is the identification of the hydrologic soil groups occurring therein. There are four basic soil groups A, B, C, and D from high to low infiltration rates. The description of soils including maps contains the basic source of the U.S. Soil Conservation Service [12]. Similarly, it contents the necessary tables for evaluation specific hydrologic soil-cover complexes and various kinds of land use. Hydrologic conditions refer to the relative quality of a land cover.
If more than one soil-cover complex is present in a catchment, a composite curve number may be determined by weighting each CN by its respective fraction of the total catchment area.
Another table of the source literature [11] defines separately for both growing and dormant seasons three discrete levels of antecedent moisture condition according to the sum of rainfall for five days previous to the date of interest.

2.1 Advantages in application

In general, the CN-method is very easy to apply. The single parameter CN is all that is needed to compute excess rainfall for an event of given depth and duration. The undoubt advantage of the method is that no runoff data are required for its use, only readily identifiable physical characteristics of the catchment. In turn, for watershed where some recorded runoff is available, the method can serve for curve number determination and it brings a feedback between those and specific soil types and land covers.
A procedure of estimation of CN-values for small catchments has been also developed by using Landsat imagery with the combination of the conventional CN model [14], [19].

2.2 Shortcomings of the method

The first weak points of the CN-method is the estimation of initial abstraction using eq. (1): $I_a = 0.2S$. It is obvious that the value of initial abstraction I_a which should comprise interception and depression storage (both micro and macro) will vary not only with a catchment storage S, but also with a catchment topography and with antecedent conditions.
The next naturally occurring problem in applying this method is the effect of catchment wetness on CN. Values of CN are expected to vary with soil and site moisture. This is handled [12] by introducing three soil moisture classes I, II and III which modify CN suddenly according to the depth of antecedent 5-day rainfall. Changes in CN-values should be continuous as the changes in soil moisture are continuous too.
Another problem with the CN-method arises when the rainfall rate is variable in time. Considering cumulative infiltration W:

$$W = P - P_e + W_e - I_a \qquad (3)$$

where P_e....effective rainfall
W_e....cumulative infiltration at time t_e (i.e. end of initial abstraction)

Eq. (3) applies for any time after the initial abstraction has been satisfied. Combining eq. (1) and (3) one gets:

$$W = P - \frac{(P - I_a)^2}{P - I_a + S} + W_e - I_a, \text{ after re-arrangement:}$$

$$W = \frac{S(P - I_a)}{P - I_a + S} + W_e \qquad (4)$$

Derivating eq. (4) with respect to time, for times greater than t_e:

$$I = \frac{dW}{dt} = \frac{d}{dt}\left[\frac{S\,(P - I_a)}{P - I_a + S}\right] = \frac{S^2 \cdot r}{(P - I_a + S)^2} \qquad (5)$$

where r....rainfall intensity.

Equation (5) holds only after reaching the initial abstraction I_a [2]. The evident shortcoming of the CN-method is that when the rainfall rate r increases, the infiltration rate I increases too and vice versa. Linear relationship between r and I does not reflect the physical principle of phenomenon.

Nevertheless, in spite of shortcomings of the CN-method from oversimplification of the rainfall-runoff description, hydrological practice has shown [4], [12], [14] its satisfactory applicability. The objective of this report is to show how to use hydrological information coded in CN-values for simulation rainfall-runoff process based on physical parameters. One reasonable way is the infiltration approach [10] using correspondence between CN and infiltration parameters, as hydraulic conductivity and sorptivity [11], [7].

3 INFILTRATION APPROACH

Computation of a direct runoff using the infiltration approach is based on the theory of Mein and Larson [9] who extended the Green and Ampt approach to compute the ponding time. Then the theory of Morel-Setoux [10] of iniltration process under condition of variable rainfall has been applied.

The Green and Ampt equation is usually written as:

$$I = K_s \left(1 + \frac{H_f(\theta_s - \theta_o)}{W}\right) \qquad (6)$$

where H_f....effective capillary drive (wetting front suction)

θ_s, θ_o....saturated and intial moisture content

$S_f = H_f(\theta_s-\theta_o)$....storage suction factor (capillary potential)

W.....cumulative infiltration

K_s....hydraulic conductivity

Mein and Larson [9] have introduced a ponding time t_p when:

$$t = t_p,\ \theta_o \rightarrow \theta_s \text{ and } I = r$$

Then cumulative infiltration at ponding time:

$$W_p = I \cdot t_p$$

Substituting these conditions to eq. (6):

$$I = K_s \left(1 + \frac{(\theta_s-\theta_o)H_f}{I \cdot t_p}\right)$$

t_p can be expressed:

$$t_p = \frac{H_f\ (\theta_s-\theta_o)}{r\left(\frac{r}{K_s} - 1\right)} = \frac{S_f}{r(r_*-1)} \qquad (7)$$

where $r_*=r/K_s$ (the normalized rainfall rate with respect to the hydraulic conductivity).

For post-ponding infiltration for the case of constant rainfall, Morel-Seytoux has derived a general equation:

$$W = W_p + S(\theta_o)AR\ [\sqrt{t-tp + \frac{tp}{2}\ (AR)^3} - \sqrt{\frac{tp}{2}\ (AR)^3}] + K_s(t-tp) \qquad (8)$$

where t is any time after ponding to the end of rainfall, W_p is a pre-ponding infiltration and it is equal to $r \cdot tp$. The variable $S(\theta_o)$ is equal to $\sqrt{2K_s \cdot S_f}$ which is the Green and Ampt sorptivity and W is the cumulative depth of infiltration at time t. Variable $AR = r_*/(r_*-1)$.

Derivation of eq. (8) is based on the fact that for short times infiltration capacity varies inversely with the square root of time, and also on the requirement that at ponding time the rainfall rate and the infiltration rate are the same. As the detailed derivation of eq. (6) has not been published

yet, it is given in the Annex, herein.
Similarly, for the case of variable rainfall:

$$W = W_p + S(W_p,\theta_o)\ [\sqrt{t-t_p + BR} - \sqrt{BR}] + K_s(t-t_p) \qquad (9)$$

where rainfall sorptivity

$$S(W_p,\ \theta_o) = \sqrt{\frac{2K_s(S_f+W_p)}{S_f}}$$

$$BR = \frac{1}{2} \frac{(S_f + W_p)^2}{K_s\ S_f \left(\frac{r_p}{K_s} - 1\right)^2}$$

where r_p....rainfall intensity which produced ponding. Parameter BR results from the requirement that at ponding time, the rainfall rate r_p is equal to the infiltration rate. For the ponding time:

$$t_p = t_{j-1} + \frac{1}{r_j}\ [\frac{S_f}{r*_{j-1}} - \sum_{k=1}^{j-1} r_k\ (t_k - t_{k-1})] \qquad (10)$$

where j....subscript of time step of consideration
k....subscript over which all rainfall occurring previous to t_j is summed

Eq. (10) must be applied iteratively. The instantaneous infiltration rate for time $t>t_p$ can be expressed by derivation of eq. (9):

$$I = \frac{1}{2}\ S(W_p,\ \theta_o) \cdot \frac{1}{\sqrt{t-t_p + BR}} + K_s \qquad (11)$$

From this analysis is evident that the infiltration approach being compared with the CN method better respects the physical principles of rainfall-runoff process. On the other hand, for the implementation of the infiltration approach one has to know soil parameters: θ_s, θ_o, K_s, H_f for an experimental spot or as representative parameters for a catchment.

3.1 Combination of infiltration approach with the CN-method

The merit of the infiltration approach consists in its more realistic view on a runoff event particularly when rainfall intensity is not uniform. Since a time dependent infiltration capacity is established, this defines the maximum rate at which water can enter the soil. If the rainfall rate falls to a value below this capacity curve, no runoff occurs. On the contrast to it, it can be easily shown that the CN-model-dynamics will predict excess rainfall as long as rainfall lasts. When the eq. (1) is derivated with respect to time one can get the evident proof of it:

$$\frac{dPe}{dt} = r_e = \frac{(P-Ia)\ (P + 2S - I_a) \cdot r}{(P - I_a + S)^2} \tag{12}$$

Similar problem connected with the initial abstraction which is always fixed as 20% or the total catchment storage is only revised due to antecedent moisture conditions (AMC). There are only three distinct possible values of I_a, although in real conditions the value of I_a could appear between those for AMC I and AMC III.

For historical rainfall-runoff events to be reconstructed the areal variability of a catchment storage S [4] as well as its seasonal variability var (S) can be considered using a water balance analysis [6] and real lysimeter data [7]. The seasonal storage S_s can be computed:

$$S_s = 1.3\ \left[\frac{(\sin(JD-5+180^0) + 1}{2}\right] + 0.2\ S \tag{13}$$

where JD is the Julian date:

$$JD = 30\ (MONTH - 1) + DAY \tag{14}$$

Then the seasonal curve number CN_s is calculated:

$$CN_s = \frac{25400}{S_s + 254} \tag{15}$$

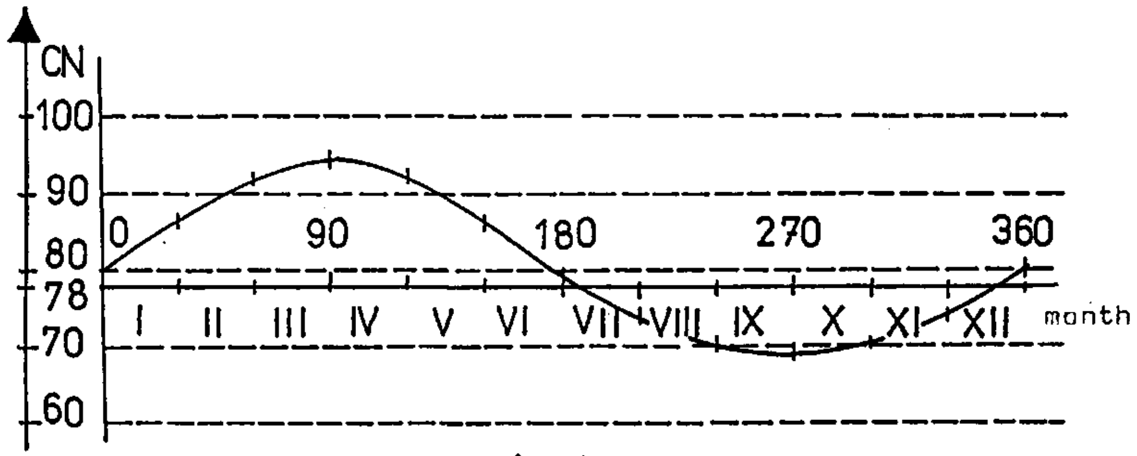

Fig. 1

Figure 1 illustrates the course of the variations of CN_s for arbitrarilly chosen value of (Ex.: CN=78).

Then a simple balance model for the calculation of the actual value of curve number CN_A has been compiled [6], [7] combining the Hawkins method [4] and the Williams and La Seur method [18] in our modification.

The structure of this simple balance submodel comes from the following consideration. Substituting S_s for S and performing the division in eq. (1) one can get:

$$P_e = P - S_s \left(1.2 - \frac{S_s}{P + 0.8\ S_s}\right) \qquad (16)$$

It follows from eq. (16) that the highest possible difference between the rainfall depth P and the effective rainfall depth P_e is not a seasonal storage S_s but 1.2 S_s (if $P \rightarrow \infty$), so RMAX = 1.2 S_s. These relationships are given in Fig. 2.

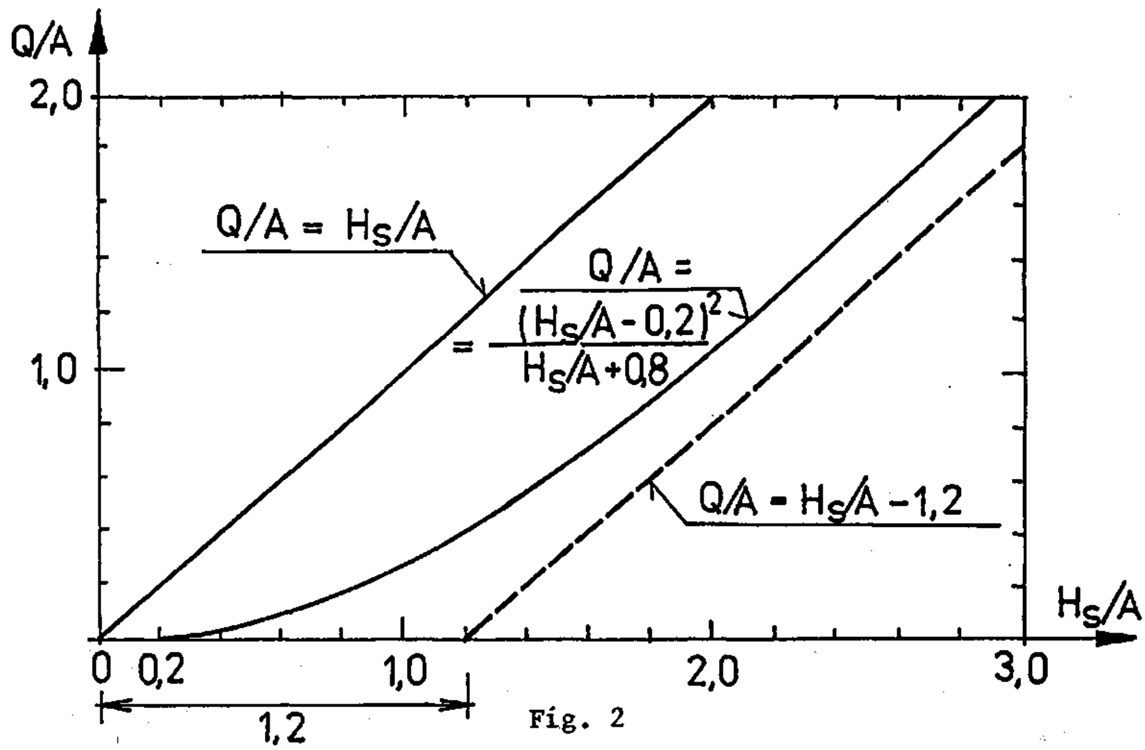

Fig. 2

The value of $RMAX_1$ at the beginning of the 5 day-period t_1 is:

$$RMAX_1 = 1.2\ S_{s1} = 1.2\left(\frac{25400}{CN_{s1}} - 254\right) \qquad (17)$$

where $CN_{s1} = \frac{25400}{S_{s1} + 254}$ which is the seasonal curve number at time t_1. During 5 day-period before storm starts the changes of $RMAX_1$ will be caused by daily precipitation, evapotranspiration infiltration and surface runoff, the last only if $P > 0.2\ S_s$. This hydrological balance at the end of 5 day-period t_2 can be evaluated in accordance with Fig. 3:

$$RMAX_2 = RMAX_1 + 5DEP - (AP15 - Q_T) = 1.2\ S_{s2} \qquad (18)$$

where API is the 5 day antecedent precipitation index:

$$API_5 = \sum_{t=1}^{5} P_t \cdot C^{t-1}$$

C....evapotranspiration constant $(C<0.87,0.96>)$. Infiltration losses are difficult to balance but for the rough estimate of them, daily depletion values DEP usually would do. These can be balanced using yearly average of precipitation AVP and those of runoff AVQ. Then:

$$Q_T = \sum_{t=1}^{5} Q_t, \quad Q_t = \frac{(P_t - 0.2\ S_{st})^2}{P_t + 0.8\ S_{st}} \text{ if } P_t > 0.2\ S_{st}$$

but $Q_T = 0$ otherwise

The actual storage at the end of the balanced period one can get by substituting of eq. (17) to eq. (18):

$$RMAX_2 = 1.2\left(\frac{25400}{CN_{s1}} - 254\right) + 5DEP - (API_5 - Q_T) = 1.2\ S_{s2} \qquad (19)$$

Introducing the actual value of curve number CN_A:

$CN_A = CN_{s2} = \frac{25400}{S_{s2}+254}$ to eq. (1a) we get:

$$CN_A = \frac{30480}{\frac{30480}{CN_{s1}} + 5DEP - (API_5 - Q_T)} \qquad (20)$$

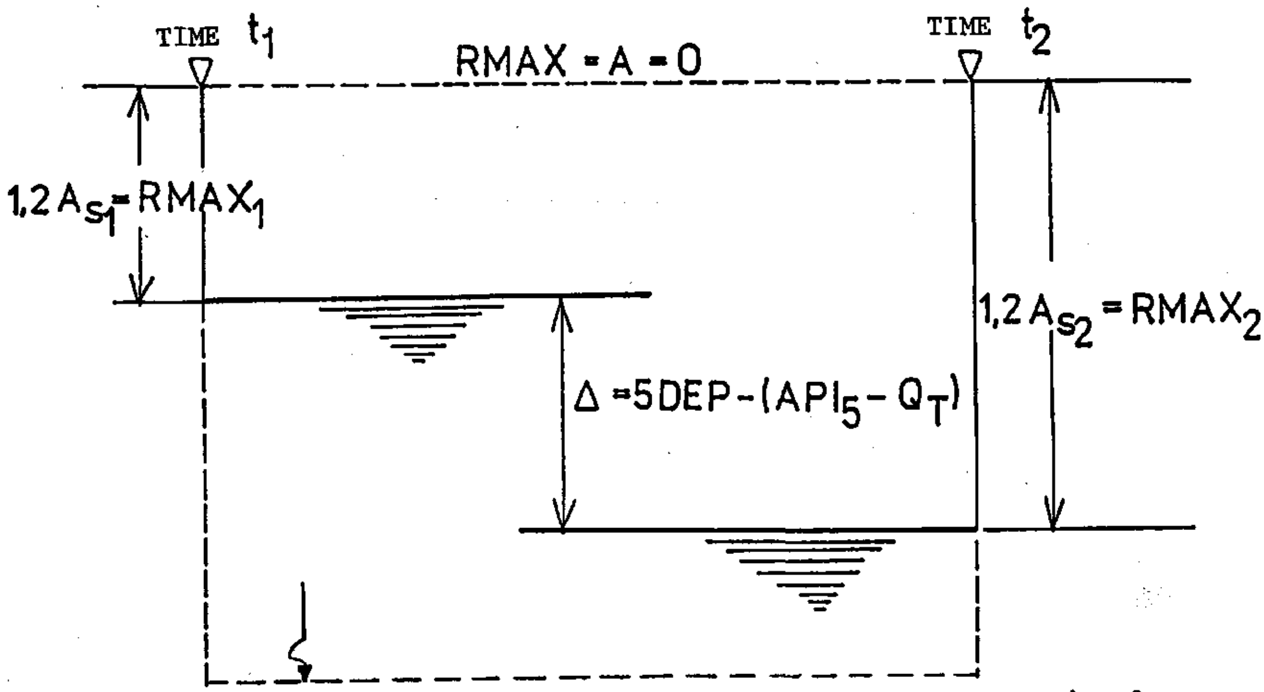

Fig. 3

Eq. (20) is a simple hydrological balance model for better estimate of the curve number value in a concrete historical situation. The Figure 3 visualizes the concept of eq. (20). For the evaluation of CN_A values only the most commonly available data have been used. Verifying the KINFIL model on historical data the CN_A value should be put instead of CN value for any reconstructed event. The practical experience has tested this procedure [7].

3.2 Parameter equivalence

There is a need to define an equivalence between a given (K_s, S_f)-pair and such a curve number CN which should correspond to the pair. The criterion of the same quantity of water being abstracted from a storm if a computation is performed using the convential CN-method as well as the infiltration approach [11], [7] was accepted. This can be done with fair results only for the rainfalls of constant intensity which have been used as the basis of a CN-(K_s, S_f) equivalence.

3.2.1 Equivalence for a single event

It follows from the previous text that for a single event the equivalence equation, making use of eq. (4), can be written:

$$R + W = I_a + \frac{S(P-I_a)}{P-I_a+S} \tag{21}$$

where W is a cumulative infiltration which can be expressed by eq. (8) and R is the surface retention of a catchment comprising both interception and depletion storages. The second term on the right side of eq. (21) is $P-P_e-I_a$, so that fully corresponds to eq. (4) except that retention R has been added to both sides of eq. (21).

Solution of eq. (21) requires assumption for S to be at the field capacity (FC) which represents "average" conditions (i.e. central value between θ(WP and θ_s). Assuming the specific rain (P,t_d) of a constant intensity: $r=P/t_d$, the only unknowns in eq. (21) are I_a and S. For times between 0 and t_e, which is time to satisfy initial abstraction holds:

$t<0,t_e>$: $t_e=I_a/r$, then one may define the function F:

$$F = R + WP + S(\theta_0)\cdot AR \left[\sqrt{\frac{I_a}{r} - t_p + \frac{t_p}{2}(AR)^3} - \sqrt{\frac{t_p}{2}(AR^3)}\right]$$

$$+ K_s \left(\frac{I_a}{r} - t_p\right) - I_a \tag{22}$$

Eq. (22) is a non-linear one with only unknown I_a. The proper value of I_a is that makes F=0, which requires an iterative solution via ΔI_a to reach F=min (using Newton iteration):

$$F = 0 = (F)_o + \left(\frac{dF}{d\ I_a}\right)_o \cdot \Delta I_a \tag{23}$$

After substituting I_a to eq. (21) one can compute the value of S.

3.2.2 Equivalence for the group of events

The first step which should be proceeded is to calculate I_a separately for each event using procedure described above (eq. (21), (22), (23)). Then one must find the best value of S over a group representative storm values. Using values of I_a it is possible to write an equation for each event with a single unknown S. Then the sum of residuals (ρ) of S can be minimized through function G:

$$G = \frac{d}{dS}\left(\sum_{i=1}^{N}\rho_i^2\right) = 0 \tag{24}$$

using Newton iteration:

$$G = 0 = (G)_o + \left(\frac{dG}{dS}\right)_o \cdot \Delta S \tag{25}$$

The quantity ΔS in eq. (25) is solved for and added to the previous estimate of S. This is done until the magnitude of the correction ΔS is insignificant.

For this analysis the design rainfalls of the return period of 1,2,5,10,20,50 and 100 years have been used with ten basic soil texture groups varying from clay to sand. For this correlation the rainfall data of the Czech Hydrometeorological Institute have been applied with soil data

taken from the literature source [20]. Each soil texture being characterized by
hydraulic conductivity K_s and sorptivity $S(\theta_{FC})$ at field capacity has provided the best fitted storage, S value. Then, using a linear regression method and applying eq. (2) the following relationships have been derived [7]:

For hydraulic conductivity K_s (mm/hr):

$$\left.\begin{array}{ll} K_s = \dfrac{100\text{-}CN}{12.4} & \text{if } CN \geq 75 \\ K_s = 31.4 - 0.4\ CN & \text{if } 36 < CN < 75 \\ K_s = 47.1 - 0.8\ CN & \text{if } CN \leq 36 \end{array}\right\} \quad (26)$$

For sorptivity $S(\theta_{FC})$ (mm/hr$^{1/2}$):

$$\left.\begin{array}{ll} S(\theta_{FC}) = \dfrac{100\text{-}CN}{1.66} & \text{if } CN \geq 65 \\ S(\theta_{FC}) = 30.25 - 0.15\ CN & \text{if } CN \leq 65 \end{array}\right\} \quad (27)$$

The storage suction factor S_f is then expressed using definiton:

$$S_f = (S(\theta_{FC}))^2/2K_s \quad (28)$$

Differencies of our resulting relationships and the results of Morel-Seytoux are not significant [11] and confirm his original method.

4 SURFACE RUNOFF ROUTING

The second basic component in deterministic stormwater modelling is overland and open channel flow which can be represented by surface runoff submodel. It can be handled either as a lumped model which permits the direct calculation of hydrographs for a catchment from rainfall excess or as a distributed model solving simultaneously to equation of motion and continuity in a geometrically distributed catchment network.

4.1 Use of lumped models

Many lumped models are based on the unit hydrograph theory. The US Soil Conservation method [12] proposes the dimensionless S-curve for ungauged catchments. The lag time t_l can be expressed:

$$t_l = \frac{2.59\ L^{0.8} \cdot (0.04S + 1)^{0.7}}{1900 \cdot I^{0.5}} \tag{29}$$

where t_l...lag time [hr]

L....hydraulic length of catchment [m]
I....average catchment slope [%]
S....catchment storage (=25400/CN-254) [mm]

Time to peak T_p is:

$$T_p = \frac{\Delta t}{2} + t_l \tag{30}$$

where Δt is the time step length.

The Q-ordinates of the standard dimensionless S-curve are contained in the original literature [12] and they are proportional:

$$\frac{Q}{Q_p} = \alpha \frac{t}{T_p} \tag{31}$$

where α is the coefficient of scaling proportionality. Then unit hydrograph ordinates for the chosen Δt-duration of effective rainfall can be derived. Hydrograph at the outlet of catchment can be computed using the convolution procedure. This routine was used in the former version of the model, INFIL. Another possibility is to derive the geomorphological instantaneous unit hydrograph, GIUH from the physical catchment characteristics [21], [22], Nevertheless, the preference is given to the kinematic wave approximation herein.

4.2 Use of distributed models

The kinematic flow approximation belongs to the group of distributed models and they are experienced to be a very efficient tool in storm water modelling [5], [13], [15]. The chief advantage of this hydraulic approach is its good physical background and therefore a proper approximation to the natural phenomena of runoff formation and routing.

The governing equations of motion for spatially varied unsteady flow over a plane surface are the equation of continuity and momentum which were derived by applying principles of mass conservation and momentum. The one-dimensional continuity equation with lateral inflow (i.e. effective rainfall) is written as:

$$\frac{\partial h}{\partial t} + \frac{\partial q}{\partial x} = q = re(t) \qquad (32)$$

where h....depth of flow at time t and position x[m]

q....lateral inflow per unit width $[m^2/s]$

re(t)....effective rainfall rate as a function of time only [m/s]

Accepting that the bed slope S_o equals to the friction slope S_f, so that:

$$S_o = S_f \qquad (33)$$

This simplification is known as the kinematic wave approximation to the momentum equation. Eq. (33) can be used to write a parametric equation for discharge at any point and time as a function of the water depth only:

$$q = \alpha \cdot h^m \qquad (34)$$

where m....parameter for the type of flow, approximately 5/3 for turbulent flow and 3 for laminar flow

α....hydraulic coefficient, $\alpha = \sqrt{S_o/n}$

S_o...slope of the plane

n....Manning's roughness coefficient

Eq. (34) can be substituted into eq. (32) and yields:

$$\frac{\partial h}{\partial t} + \alpha m h^{m-1} \cdot \frac{\partial h}{\partial x} = re(t) \qquad (35)$$

The kinematic approximation assumes that kinematic wave is propagated downstream only and the term $\alpha m \cdot h^{m-1}$ in eq. (35) is its forward characteristic. Kinematic flow does not exist where there is a backwater effect, so that the rating curve (Q-h relationship) is considered as a non-looped one.
The solution of eq. (35) assumes that the boundary and initial conditions on the plane lenght L are:

$$h = 0 \begin{cases} 0 \leq x < L & t = 0 \\ x = 0 & t > 0 \end{cases} \tag{36}$$

and re(t) is spatially constant but could vary with time.

4.2.1 Cascade of planes

Equation (35) which describes surface flow over one plane has been solved numerically using an explicit finite difference scheme known as the single-step Lax-Wendroff method. The resulting explicit expression for the depth h is [15], [8]:

$$\begin{aligned} h_j^{t+1} = {} & h_j - \frac{\Delta t}{2\Delta x} [(\alpha h^m)_{j+1} - (\alpha h^m)_{j-1} - 2\Delta x \cdot re_j] + \\ & + \frac{\Delta t^2}{4\Delta x^2} [(\alpha h^{m-1})_{j+1} + (\alpha m h^{m-1})_j] [(\alpha h^m)_{j+1} - (\alpha h^m)_j - \Delta x \cdot re_j] - \\ & - \frac{\Delta t^2}{4x^2} [(\alpha h^{m-1})_j + (\alpha m h^{m-1})_{j-1}] [(\alpha h^m)_j - (\alpha h^m)_{j-1} - \Delta x \cdot re_j] + \\ & + (re_j^{t+1} - re_j)/\Delta t \end{aligned} \tag{37}$$

where all non-superscripted variables are evaluated at the time t, and j denotes a space level in the x direction.

The initial conditions are specified as: h(x,0)=0 for all x. The upstream boundary depth is determined by the position of the plane in a cascade.

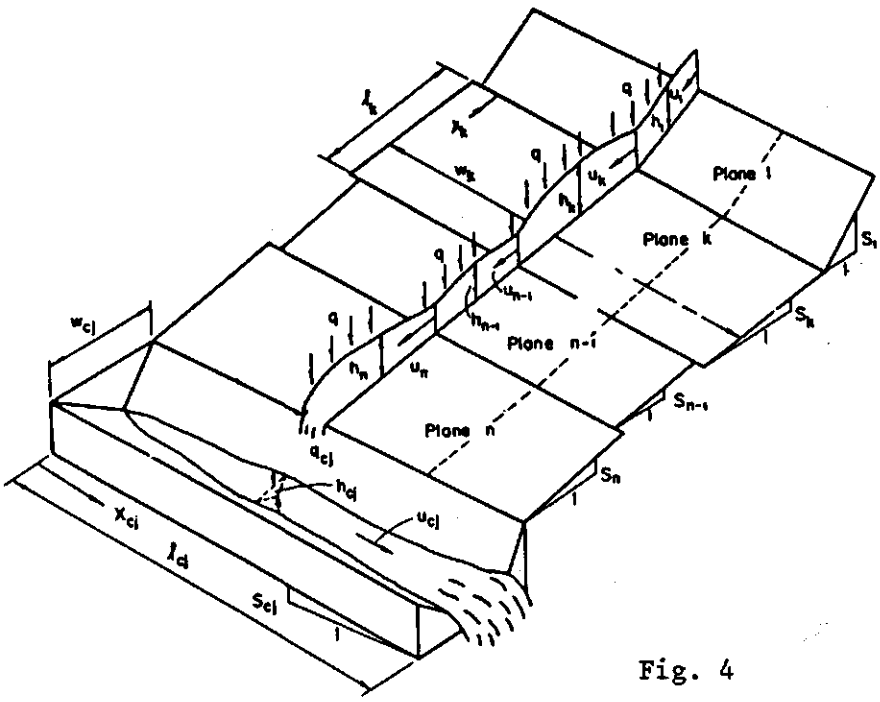

Fig. 4

Considering a cascade of n planes (see Fig. 4) where k is the order of the plane in the cascade, l the lenght of a plane each of the constant width w, then:

$$h(0,t)_k \begin{cases} = 0 \text{ if } k=1 \\ = f[h(1,t)_{k-1}] \text{ if } k>1 \end{cases} \tag{38}$$

The upstream boundary depth for the k-th plane which receives inflow from the (k-1)-th plane is found:

$$h(0,t)_k = \left[Q(1,t)_{k-1} \cdot \frac{1}{\alpha_k}\right]^{1/n_k} \tag{39}$$

Use of the Lax-Wendroff scheme assumes to maintain the following numerical stability criterion:

$$\frac{\Delta t}{\Delta x} \leq \frac{1}{\alpha \cdot mh^{m-1}} \tag{40}$$

4.2.2 Converging surface

Overland flow on a converging surface that has uniform properties is described using the equation [16]:

$$\frac{\partial h}{\partial t} + \alpha m \cdot h^{m-1} \frac{\partial h}{\partial x} = re(t) + \frac{\alpha h^m}{L-x} \tag{41}$$

where L....radius of flow region (see Fig. 5a) [m]

x....space coordinate [m]

Product c·L [m] in Fig. 5 is the outlet radius of the bottom segment and c is the ratio of both radii (i.e. convergence degree)

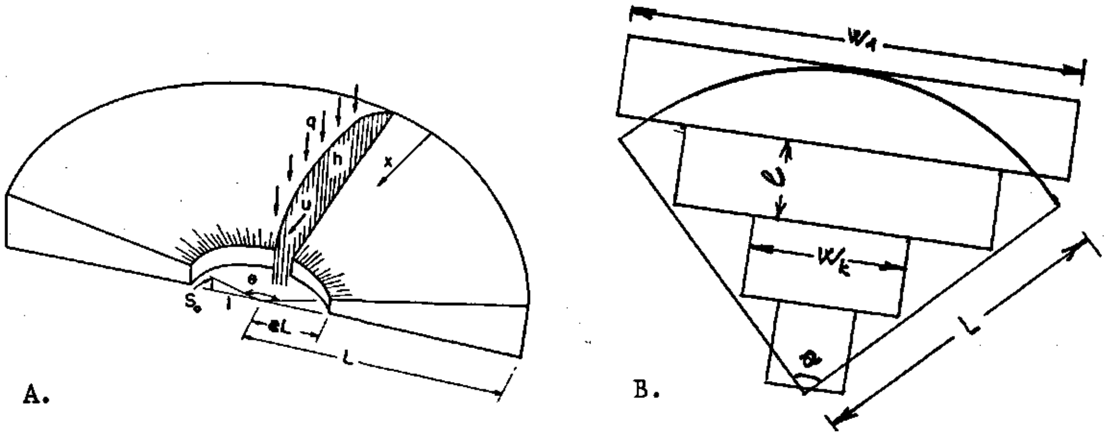

Fig. 5

For the solution of eq. (41) an explicit backwards finite difference scheme has been used [8]:

$$h_j^{t+1} = h_j - \frac{\Delta t}{\Delta x} \alpha m h_j^{m-1} (h_j - h_{j-1}) + \Delta t \left[re_j + \frac{\alpha h_j^m}{(L-(j-1)\Delta x)} \right] \tag{42}$$

The boundary and initial conditons are:

$$h(0,t) = 0 \text{ and } h(x,0) = 0 \tag{43}$$

The stability criterion obeys the eq. (40).

As the numerical scheme (42) is not very accurate and stable the Lax-Wendroff numerical scheme with the plane cascade approximation of converging section has been preferred later. The width of planes changes gradually plane by plane (see Fig. 5b) to simulate properly the convergence. In that case the boundary conditions slightly differ from that given by eq. (38) and (39) by introducing the change of width:

$$h(0,t)_k \begin{cases} = 0 \text{ if } k=1 \\ = f[h(1,t)_{k-1},\ w_{k-1},\ w_k)] \text{ if } k>1 \end{cases} \tag{44}$$

The upstream boundary depth for the k-th plane is:

$$h(0,t)_k = \left\{ \left[Q(1,t)_{k-1} \cdot \frac{w_k}{w_{k-1}} \right] \cdot \frac{1}{\alpha_k} \right\}^{1/n_k} \tag{45}$$

The approximation of converging segment by the variable-width-cascade of planes has been found in accordance with [5] as fair enough.

4.2.3 Main stream routing

Free surface flow in streams can be computed using the kinematic approximation to the equations of unsteady, gradually varied flow. The Muskingum-Cunge method has been accepted here [1]. The basic continuity equation for each stream reach is:

$$\frac{dV}{dt} = I - Q \tag{46}$$

where V....storage of flow in a reach $[m^3]$

I,Q....inflow, outflow rates $[m^3/s]$

The storage V is expressed as

$$V = K \cdot Q + K \cdot X(I-Q) = K\,[X \cdot I + (1-X)Q] \tag{47}$$

in which K is a storage constant with time dimension [s] and X is a weighting coefficient $0 \leq X \leq 1$.

Eq. (46) is the special form of a general continuity equation (32) assuming the depth-discharge relationship as a single-valued (no backwater effect), and not very mild sloping waterways.

The solution of ordinary differential eq. (46) can be written as:

$$Q_{j+1}^{m+1} = C_1 Q_j^n + C_2 Q_j^{n+1} + C_3 Q_{j+1}^n + C_4 \quad (48)$$

where $C_0 = K - KX + \Delta t/2$ (49)

$C_1 = -(KX - \Delta t/2)/C_o$ (50)

$C_2 = (KX + \Delta t/2)/C_o$ (51)

$C_3 = (K - KX - \Delta t/2)/C_o$ (52)

$C_4 = 0.5\ (q_1 + q_2)\Delta x \cdot \Delta t/C_o$ (53)

q is the lateral inflow

For the parameters K,X Cunge [1] has derived:

$$K = \Delta x/cw \quad (54)$$

$$X = 0.5\ [1-Q_p'/(cw\ S_o \cdot \Delta x)] \quad (55)$$

in which cw is the kinematic wave speed corresponding to a unit width upstream peak discharge Q_p', Δx is the reach length and S_o is the stream bottom slope. Eq. (54) may be expressed in an alternative form, i.e.:

$$cw = 1.27\beta\ S_o^{0.3}/(Q_p'^{0.4} \cdot n^{0.6}) \quad (56)$$

$$\beta = 5/3 - 2/3\ \frac{A_o}{B_o^2}\ dB/dy \quad (57)$$

in which A_o is the cross-sectional area corresponding to the total peak discharge Q_p, B_o is the channel width corresponding to Q_p. Depending on the cross-section shape, β may have values in the range $1<\beta\leq5/3$. The selection of Δx affects the accuracy of the solution. It is related to Δt and is limited by the following inequality [1]:

$$\Delta x \leq 0.5\ [cw\Delta t+Q_p'/(cwS_o)] \quad (58)$$

5. STRUCTURE OF THE KINFIL MODEL

Model KINFIL consists of two basic parts:

I. INFIL

II. CPLANE, CONVER, CSTREAM

The first part INFIL is a lumped model (time variant only) computing infiltration process based on the Morel-Seytoux and Mein and Larson equations using the parameter equivalence CN - f(Ks, Sf) as it is described above. The subroutines of the INFIL are required as follows:

PONTI : Ponding time calculation
CONST : Constant intensity rainfall infiltration
PPINF : Variable intensity rainfall infiltration
TABLE : Parameter assignment

The second part of the KINFIL computes the surface runoff formation and routing. The subprogram CPLANE is supposed to be used for the simulation of catchment topography by a cascade of planes being solved by the numerical scheme of Lax-Wendroff. Similarly CONVER represents flow over a converging surface using an explicit backward numerical scheme. CSTREAM subprogram is based on the Muskingum-Cunge method of flood routing. All three subprograms are mutually compatible and they represent the distributed surface runoff model. Associated subroutines and functions are:

WRTR : for writing the interim results of the CPLANE
WRMC : for writing the interim results of the CSTREAM
HKIN : for computing the Lax-Wendroff scheme

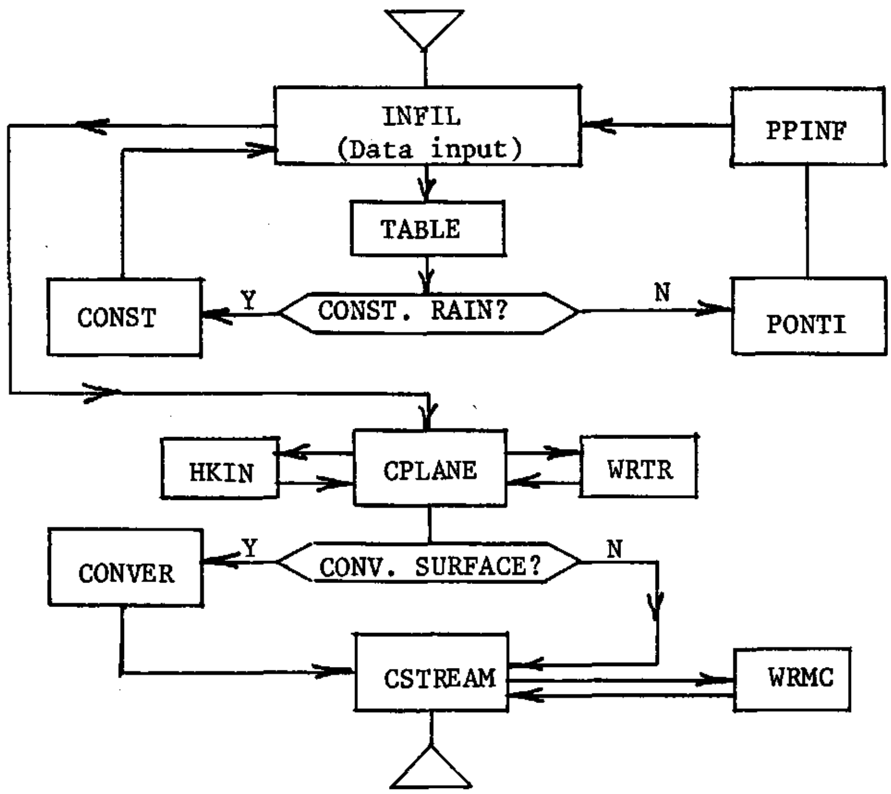

Fig. 6

The KINFIL is written in FORTRAN 77. Data input provides the INFIL subprogram; each subroutine gives the appropriate output data. Both input and output data sets go through the mass balance check controlling whether a mass conservation obeys. The listing of KINFIL is available on the Department of Catchment Hydrology, Soil Physics and Hydraulics, AU Wageningen. The basic imagination of the model structure is given on Fig.6.

6. PRACTICAL APPLICATION AND CONCLUSSIONS

The data of two catchments for debugging and testing the KINFIL model have been implemented: PLANE and HUPSELSE BEEK.

6.1 Hypothetical catchment PLANE

The PLANE is a hypothetical catchment of the symmetrical V-shaped form with the stream of rectangular cross-section at the bottom and with the converging segment in the upstream part of it. Each side of catchment consists of a cascade of three planes with variable length and constant width (see Fig.7).

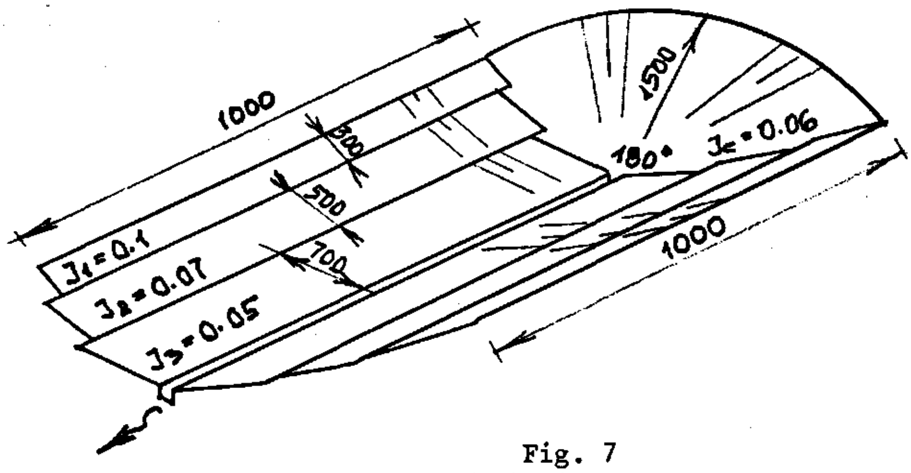

Fig. 7

The main reason why the hypothetical catchment has been used was mainly that all three basic routing elements could be tested: plane, segment, open channel. Two data sets have been implemented here:

INFIL:

PLANE 1: The rainfall of constant intensity r= 15.0 mm/hr, depth 90.0 mm, duration 6 hrs, CN= 85.0 (arbitrarily chosen).

PLANE 2: The rainfall of variable intensity with r max= 28.0 mm/hr, depth 38.3 mm, duration 3.5 hrs, CN= 90.0.

The main parameters were:

CPLANE: 3 PLANES (J=3) (from each side)
slope: SO(J)= 0.20, 0.10, 0.01
length: DLN(J)= 700.0, 500.0, 300.0 m
width: WID(J)= 1000.0 m
Manning's n: FRNM(J)= 0.035, 0.030, 0.025
type of flow m: AM(J)= 1.67 (allways)

time step: DT= 900.0 s for input/output data
DTC= 30.0 s for computation
space step: DX(J)= DLN(J)/10.0 m

CONVER: CONVERGING SEGMENT
slope: SLC=0.06, length: DLNC= 1500.0 m,
angle: TH= 180 deg., inner radius: DC= 3.0 m,
Manning's n: FRN= 0.030, flow type m: AMC= 1.67,
time step: DT= 900.0 s for input/output data
DTC= 30.0 s for computation
space step: DX= 60.0 m

CSTREAM : 5 REACHES (J=5) of the rectangular cross-section
slope: SOS(J)= 0.02 (allways)
length: DLNS(J)= 150.0, 150.0, 200.0, 250.0, 250.0 m
width: WIDS(J)= 3.0 m (allways)
Manning's n: FRS(J)= 0.025 (allways)
area of planes: AREAA= 3.00 km2
area of segment: AREAC= 3.53 km2
time step: DT= 900.0 s for input/output data
DTC= 30.0 s for computation
space step: DX(J)= DLNS(J)/10.0 m

The MUSKINGUM-CUNGE parameters have been computed by the CSTREAM respecting the attenuation parameter, wave speed and curvature of upstream hydrographs. The computatin of reference peak discharge was made in the same way as it is in the Flood Studies, 1975 (Institute of Hydrology, Wallingford, UK). The M-C parameters for the individual reaches were:

PAK= 75.0, 75.0, 100.0, 125.0, 125.0 s
PAX= 0.186, 0.186, 0.265, 0.312, 0.312

The PLANE synthetic data were used first for debugging the KINFIL and for the numerical stability tests. The scheme of Lax-Wendroff was found as stable as the Courant number DTC/DX<1.0, while the simple backwards scheme used for the CONVER was very sensitive getting reliable results only when DTC/DX<=0.5. There were no stability problems with the Muskingum-Cunge procedure. The results are given on the Tab.1, Tab.2, and on the Fig.8 and Fig.9, resp.

6.2 Application to the Hupselse data

The Hupselse Beek catchment covers about 6.5 km2. The main stream runs from east to west through a slightly undulating rural landscape in the eastern part of the Netherlands. The altitude changes from 33 m a.s.l. to 24 m a.s.l. (the outlet). The average slope of catchment is 0.8%. The main stream is 4.2 km long with the average slope 0.1%. There are 7 small tributaries varying in length from 300 m to 1500 m. Concerning the land use 70% is grassland, 21% arable land, 6% woodland and 3% without vegetation (buildings, roads, etc.). The map below (Fig.14) gives the survey of various land use on the catchment.

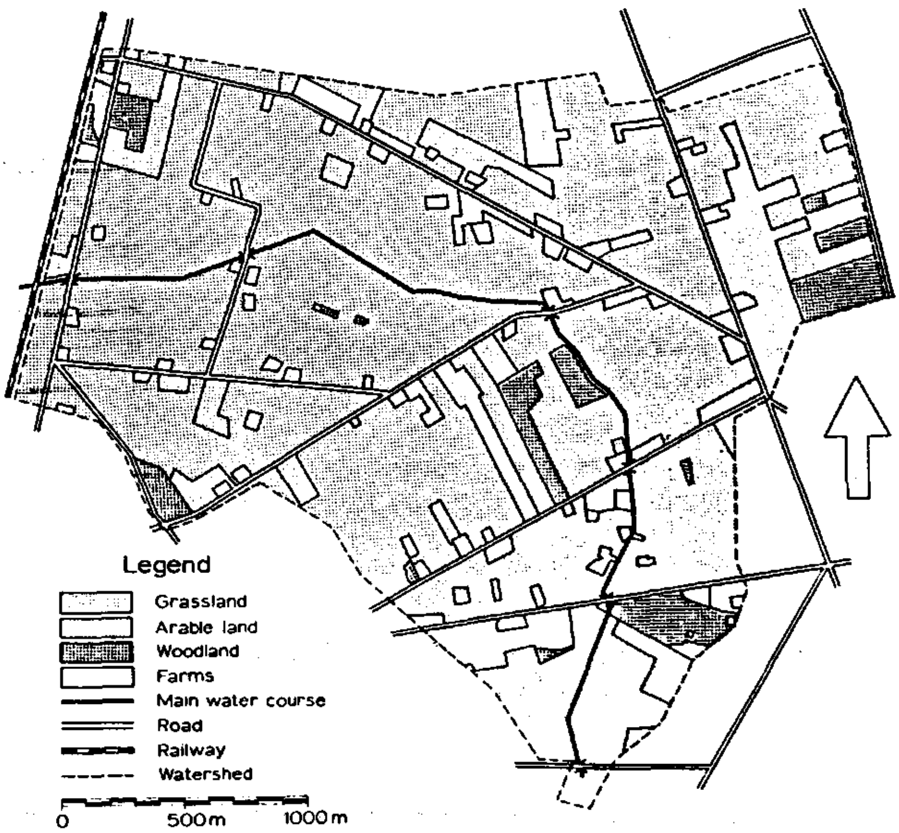

Fig. 14 (⇒ map)

The Hupselse area consists mainly of slightly loamy fine sand and partly of poor loam. In general, the permeability of soil varies from middle to high. There are only two levels with low permeability occurrence. Miocene clay is found at or near to surface in the eastern part of the area. In a western direction it is covered by younger deposits, in which remnants of glacial till are found at about 1 meter deep. Much detailed information on soil science, geology and hydrology is published in the periodical booklet IHP [24].

The selection of rainfall-runoff data for the KINFIL model has been made with a particular reference to high intensity rainfalls. The time step was used no longer than 15 min to be able to describe an infiltration process. First, the CN value has been evaluated using the convential way.

The procedure for estimating the CN value [12] for Hupselse catchment was as follows:

The hydrologic soil group was found as B. The estimation was made that pastures belong in 80% to fair hydrologic conditions and 20% to the poor ones, all other land use belongs to the B group, too. The composite CN can be calculated:

Pastures (80% fair hydrologic conditions)	... 0.56 x 69=	38.64
Pastures (20% poor hydrologic conditions)	... 0.14 x 79=	11.06
Arable land (good hydrologic conditions)	... 0.21 x 78=	16.98
Woods (poor hydrologic conditions)	... 0.06 x 66=	3.96
Roads (hard surface)	... 0.03 x 98=	2.94
	Composite CN =	73.56
	Rounded to CN =	74.00

It is well known that any CN- based model does not give a very good match with reconstructed events unless they are caused by very intensive rainfall. This is obvious as the CN- based models have been developed for design purposes with low probability of occurence (and consequently with high return period). From the 15 year rainfall-runoff records on the Hupselse Beek, eight significant events with the highest intensity rainafall have been selected for reconstruction. The chief characteristics of these rainfall events with the highest 15 mm intensities max. r(mm/hr), total depths P(mm) and five day antecedent precipitation sum AP(mm) were measured as (all records from Assink st.):

		max.r(mm/hr)	P(mm)	AP(mm)
EVENT 1	23/5/1972	27.00	30.12	9.6
EVENT 2	27/5/1972	13.68	44.00	35.5
EVENT 3	28/7/1972	10.80	38.36	17.4
EVENT 4	10/7/1980	11.16	30.60	21.9
EVENT 5	29/9/1984	16.80	24.80	14.7
EVENT 6	14/8/1985	23.52	29.64	19.1
EVENT 7	6/5/1986	10.92	18.48	3.6
EVENT 8	1/3/1987	9.01	75.53	19.7

For all 8 events the antecedent moisture conditions were assigned as the AMC II. To compare the measured rainfall intensities with the design ones, the Table 5 can give the outlook on the parameters of design rainfalls with the return period Te= 2, 10, and 100 years (for De Bilt station). For applying the KINFIL the curve number CN= 74.00 has been considered for all events. This CN value gives the corresponding values of the hydraulic conductivity Ks= 0.041 mm/min and the sorptivity So= 2.018 mm/min**0.5 (the storage suction factor Sf= 49.68 mm). Table 3 gives the comparison of results obtained from the INFIL part of the model with observed depths of runoff. Tab.3 also shows to what extent is sensitive the selection of time step. It gives the results for DT= 15 min and DT= 30 min both of that have been used. Because of the nature of infiltration process, the shortest DT the more reliable results. Anyhow, the aim of this work was not to simulate a rainfall-runoff process in details but to prove the applicability of the KINFIL for design purposes. Then four events No. 1, 2, 5, and 6 have been selected for reconstructing the runoff process with the satisfactory match of the both computed and measured effective rainfall depths. For this purpose the Hupselse catchment has been geometrically approximated by the V- shaped planes with the main stream symmetrically situated between. As there is very small slope of the catchment and a simple topography of it, neither cascade nor converging surface were considered. The main parameters of the model are:

```
CPLANE: 1 PLANE (J=1) (from each side)
        slope: SO(J)= 0.008
        length: DLNJ(J)= 810.0 m
        width: WID(J)= 4000.0 m
        Manning's n: FRNM(J)= 0.15
        type of flow m: AM(J)= 1.67
        time step: DT= 900 s for input/output data
                   DTC= 30 s for computation
        space step: DX(J)= 81.0 m
```

The value of Manning's n has been assigned in accordance to the field experiments [3] for a grassland catchment. The type of flow was considered turbulent as it has been shown [25] that even low flows when they are treated as turbulent would not induce a serious loss of accuracy. Considering all flows turbulent would also negate the need for a computational system to locate the laminar-turbulent transition [25].

For the channel routing process the main stream has been divided into 7 reaches respecting its longitudinal profile. For the sake of simplicity the rectangular cross-section has represented the natural trapezoidal one.

```
CSTREAM: 7 REACHES (J=7) (from upstream to downstream)
         slopes: SOS(J)= 0.0025, 0.0015, 0.0015, 0.0015,
                         0.0014, 0.0010, 0.0006
         lengths: DLNS(J)= 400, 350, 500, 350, 1000, 1100, 600 m
         widths: WIDS(J)= 1.5, 2.0, 2.0, 2.5, 2.5, 3.0, 3.5 m
         Manning's n: FRS(J)= 0.025 (for all reaches)
         time step: DT= 1 hour for input/output data
                    DTC= 60 s for computation
         space step: DX(J)= DLNS(J)/10.0
```

The MUSKINGUM-CUNGE parameters have been computed using the following values:

- Attenuation parameter αp= 0.84 .10**6 (-)
- Wave speed cel= 0.3 m/s (using old records)
- Curvature of upstream hydrograph: was neglected
- Reference peak discharges varied from 0.6 m3/s to 0.8 m3/s for individual events.

The values of the M-C parameters were:

PAK= 22.2, 19.4, 27.7, 19.4, 55.5, 61.1, 33.3 min

PAX= 0.219, 0.178, 0.275, 0.178, 0.387, 0.398, 0.312

All results from events No. 1,2,5,and 6 are tabled on the Tab.4/1, 4/2, 4/3 and 4/4. The hydrographs are plotted on the Fig. 10 A, B, C and D. No evaluation of fitting was computed but the applicability of the model is evident.

The next step in testing KINFIL was to implement the data from infiltrometer which have been recently collected and handled in large extent on the experimental spot in the Hupselse catchment. During last years many field measurements of soil hydraulic properties across the 6.5 km2 catchment area were taken. There are reported [23] three different sampling schemes, the first involves 7 measurement sites over the catchment, the second scheme consists of the 0.5 ha spot and the last one covers the rectangular of 1 ha with the network density of 20 x 20 m in the central part of the catchment. In each of these 36 sites (1 ha) the data on hydraulic conductivity, sorptivity and in some samples also on diffusivity have been obtained by different methods [23]. The scaling theory has been applied to evaluate the spatially scattered values of saturated hydraulic conductivity, Ks and sorptivity So. The scaled infiltration curve and also the scaled cumulative infiltration curve are plotted on the Fig.12. The Ks and So values from 1 ha experimental spot are given in the Table 6 and they have been used for testing conditions for a surface runoff formation caused by design rainfalls of the return period Te= 2, 10 and 100 years. There were more than 1,200 synthetic events being tested whether an overland flow is produced. The results are listed in Table 6 and on Fig.11. From the 36 soil samples there were 19 samples indicating no surface runoff in the case of 2 year rainfall, 6 samples for 10 year rainfall and 3 samples for 100 year rainfall all for various rainfall duration (see Tab.6). From the other side, when subtracting the 5 min design rainfalls the number of those which have produced some surface runoff decreases drastically: 6 events for Te= 2 y, 7 for Te= 10 y and 12 for Te= 100 year rainfall.

Only the soil sample A/1 has shown the different properties (see Tab.6). Fig.11 gives the examples of the "worst" event (A/1) as well as the "average" one. The mean Ks value was 0.795 mm/min which should mean that CN< 1.0 (CN= 1 for Ks= 0.771 mm/min). This can be hardly true for a whole catchment. Considering the scaled average value of hydraulic conductivity from all three groups of experimental spots Ks= 0.234 mm/min for the horizon A, it can be shown that the corresponding CN= 44 which seems to be low anyway. The Ks= 0.234 mm/min has been determined by scaling technique as one of the van Genuchten parameter [23].

To formulate the final conclussions, one can say that due to the strong space variability of infiltration capacities across 6.5 km2 catchment, there are still required some more measurements taking into account also so called "source areas" where a surface runoff is predominantly produced. Nevertheless, the KINFIL as a design flood model does not deal with an interflow component which is undoubtedly a significant form of flow in the Hupselse area.

The last step in the KINFIL application was to implement the design rainfalls for the catchment under the averige -design conditions. There were found most likely as:

CN= 74.00, so that
Ks= 0.041 mm/min
So= 2.018 mm/min**0.5
Sf= 49.68 mm

and the antecedent moisture conditions as AMC II at field capacity.

The computation was made first by the INFIL only for the Te= 2, 10, 100 year for all 11 selected rainfall durations from 5 min to 24 hours with the corresponding intensities (Tab.7).

For the return period Te= 2 years the design flood hydrographs have been computed by KINFIL model. Table 8 gives the hydrographs ordinates for the two years design floods caused by the design rainfall of 30, 60, 120, 180 and 240 min. These durations have been chosen because of the high depth of effective rainfall and also due to fact that the time of concentration of the catchment lies within those range. The hydrographs have been plotted on the Fig.13. The peak flow Qmax is about 1.82 m3/s for the 120 min rainfall. The design flood hydrographs for Te= 10 and 100 year can be determined in the same way.

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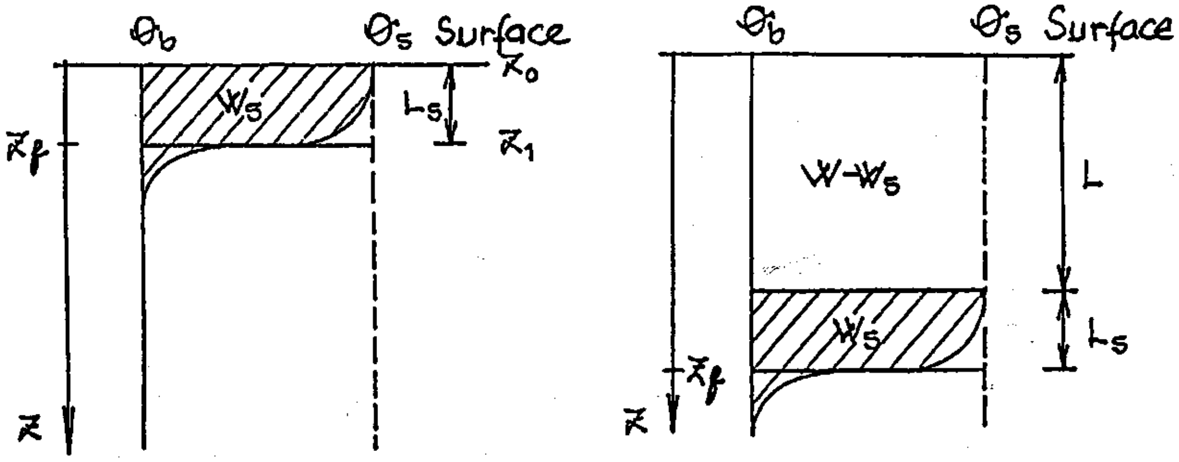
A N N E X : Derivation of the Morel Seytoux equations for infiltration

The Green and Ampt equation (6) considerring the sketch below can be also written in the form:

$$I = Ks \frac{zf + H + Hf}{zf} \tag{1A}$$

where zf ... depth of infiltration front
Hf ... effective capillary drive
H ... depth of ponded water



Initial state:
Surface saturation

Later state:
Propagation of the wetting front

Fig. 1A

Eq.(1A) can be also written as the time propagation of the wetting front:

$$I = (\Theta s - \Theta o) \frac{dzf}{dt} \tag{2A}$$

Eq.(1A) and (2A) putting together give:

$$\frac{zf}{zf + H + Hf} dzf = \frac{Ks}{(\Theta s - \Theta o)} dt \tag{3A}$$

L.h.s. of eq.(3A) can be arranged as

$$\frac{zf}{zf + H + Hf} dzf = (1 - \frac{H + Hf}{zf + H + Hf}) dzf \tag{4A}$$

If eq.(3A) with the arrangement of (4A) is integrated:

$$\int_0^{zf} \left(1 - \frac{H + Hf}{z + H + Hf}\right) dz = \int_0^{t} \frac{Ks}{(\Theta s - \Theta o)} d\tau$$

we get after integration and after some algebraic arrangement:

$$(\Theta s - \Theta o)zf - (H + Hf)(\Theta s - \Theta o).\ln\left(1 + \frac{zf}{H + Hf}\right) = Ks.t \qquad (5A)$$

or applying the expression for cumulative infiltration
$W = (\Theta s - \Theta o)zf$, then:

$$W - (\Theta s - \Theta o)(H + Hf).\ln\left[1 + \frac{W}{(\Theta s - \Theta o)(H + Hf)}\right] = Ks.t \qquad (6A)$$

Introducing time to ponding tp, substituting the storage suction coefficient $Sf=(\Theta s - \Theta o).Hf$, and neglecting the variable H (for its small value comparing with Hf) one can get:

$$Ks(t - tp) = W - Wp - Sf.\ln\left(1 + \frac{W - Wp}{Sf + Wp}\right) \qquad (7A)$$

For the development of $\ln(1 + x)$ for x variables the Taylor series $x - \frac{x2}{2} + \ldots$ has been used.

Eq.(7A) then gets the approximative form after some truncation:

$$Ks(t - tp) = W - Wp - Sf\left[\frac{W - Wp}{Sf + Wp} - \frac{1}{2}\left(\frac{W - Wp}{Sf + Wp}\right)^2\right] \qquad (8A)$$

Then:

$$Ks(t - tp) = \frac{Wp}{Sf + Wp}(W - Wp) + \frac{Sf}{2}\left(\frac{W - Wp}{Sf + Wp}\right)^2$$

and

$$Sf(W - Wp)^2 + 2Wp(Sf + Wp)(W - Wp) - 2(Sf - Wp)^2 Ks(t - tp) = 0 \qquad (9A)$$

For middle values of W - Wp it is possible to neglect the central term in eq.(9A), then:

$$W - Wp = \sqrt{\frac{2(Sf + Wp)^2 . Ks}{Sf}} . \sqrt{t - tp} \qquad (10A)$$

Derivating in time the infiltration rate can be expressed:

$$i = \frac{1}{2}\sqrt{\frac{2(Sf + Wp)^2.Ks}{Sf}} . \frac{1}{\sqrt{t - tp}} \qquad (11A)$$

Introducing sorptivity $S(\Theta o) = \sqrt{2Ks.Sf}$ to eq.(11A) one can write:

$$\sqrt{\frac{2(Sf + Wp).Ks}{Sf}} = S(r*,\Theta o)$$

or:

$$S(r*,\Theta o) = \sqrt{2Sf\left(1 + \frac{Wp}{Sf}\right) Ks} = S(\Theta o).\left(1 + \frac{Wp}{Sf}\right) \qquad (12A)$$

The term $(1 + \frac{Wp}{Sf})$ can be expressed as:

$$1 + \frac{Wp}{Sf} = 1 + \frac{r.tp}{Sf} = 1 + \frac{1}{r^* - 1} = \frac{r^*}{r^* - 1}$$

where r ... rainfall intensity
r*... = r/Ks (normalized rainfall intensity)
Then the infiltration rate is:

$$i = \frac{1}{2} S(\Theta o) \frac{r^*}{r^* - 1} \frac{1}{\sqrt{t - tp}} + Ks \qquad (13A)$$

Instantneous ponding sorptivity has to be corrected for the cases when ponding is not instantneous. Eq.(13A) does not hold for small values (t-tp) as $i \rightarrow \infty$ when t=tp. It can be corrected introducing the correction factor a:

$$i = \frac{1}{2} S(\Theta o) \frac{r^*}{r^* - 1} \frac{1}{\sqrt{t - tp + a.tp}} + Ks \qquad (14A)$$

The correction factor a should guarrantee that i=r at time t=tp, so that:

$$r = \frac{1}{2} S(\Theta o) \frac{r^*}{r^* - 1} \frac{1}{\sqrt{a.tp}} + Ks \qquad (15A)$$

The correction factor a can be expressed after some algebraic re-arrangement of the (15A) as:

$$a = \frac{1}{2} \left(\frac{r^*}{r^* - 1}\right)^3 \qquad (16A)$$

After substitution of eq.(16A) back to eq.(14A) the infiltration rate is:

$$i = \frac{1}{2} S(\Theta o) \frac{r^*}{r^* - 1} \frac{1}{\sqrt{t - tp + \frac{1}{2}\left(\frac{r^*}{r^* - 1}\right)^3 .tp}} + Ks \qquad (17A)$$

and then the cumulative infiltration at time between tp and t is:

$$W - Wp = Ks(t - tp) + S(\Theta o) \frac{r^*}{r^* - 1}\left(\sqrt{t - tp + \frac{1}{2}\left(\frac{r^*}{r^* - 1}\right)^3 tp} - \sqrt{\frac{1}{2}\left(\frac{r^*}{r^* - 1}\right)^3 tp}\right) \qquad (18A)$$

which is the eq.(8) in the Report. The eq.(9) for a variable rainfall can be derived in a similar but more complicated way.

LIST OF TABLES:

LIST OF FIGURES:

Tab. 1

RAINFALL - RUNOFF EVENT "PLANE 1"

(constant intensity rainfall)

No.	Time hr	Rain mm/hr	Infil mm/hr	Ef.rain mm/hr	Q-plane m3/s	Q-conv m3/s	Q-stream m3/s
1	0.25	15.00	13.79	0.00	0.00	0.00	0.00
2	0.50	15.00	10.29	0.00	0.00	0.00	0.00
3	0.75	15.00	7.84	6.13	0.34	0.00	0.03
4	1.00	15.00	6.69	8.31	1.46	0.01	0.51
5	1.25	15.00	5.99	9.01	3.32	0.04	1.40
6	1.50	15.00	5.50	9.49	5.92	0.13	2.72
7	1.75	15.00	5.14	9.86	7.74	0.32	4.01
8	2.00	15.00	4.86	10.14	8.14	0.65	4.68
9	2.25	15.00	4.63	10.37	8.42	1.16	5.01
10	2.50	15.00	4.44	10.56	8.64	1.91	5.43
11	2.75	15.00	4.28	10.72	8.78	3.46	6.02
12	3.00	15.00	4.14	10.86	8.94	6.75	7.11
13	3.25	15.00	4.02	10.98	9.04	8.52	9.46
14	3.50	15.00	3.91	11.09	9.16	9.88	11.91
15	3.75	15.00	3.81	11.19	9.24	11.09	13.39
16	4.00	15.00	3.72	11.28	9.34	11.35	14.60
17	4.25	15.00	3.64	11.36	9.42	11.30	15.23
18	4.50	15.00	3.57	11.43	9.46	11.21	15.26
19	4.75	15.00	3.51	11.49	9.52	11.15	15.18
20	5.00	15.00	3.45	11.55	9.60	11.17	15.11
21	5.25	15.00	3.39	11.61	9.62	11.27	15.04
22	5.50	15.00	3.34	11.66	9.70	11.38	14.97
23	5.75	15.00	3.28	11.72	9.72	11.39	14.90
24	6.00	15.00	3.24	11.76	9.78	11.22	14.84
25	6.25	0.0	0.0	0.0	6.26	10.39	14.14
26	6.50	0.0	0.0	0.0	3.86	9.31	12.60
27	6.75	0.0	0.0	0.0	2.42	8.20	10.90
28	7.00	0.0	0.0	0.0	1.56	7.20	9.41
29	7.25	0.0	0.0	0.0	1.04	6.30	8.12
30	7.50	0.0	0.0	0.0	0.72	5.50	7.03
31	7.75	0.0	0.0	0.0	0.50	4.81	6.11
32	8.00	0.0	0.0	0.0	0.32	4.20	5.30
33	8.25	0.0	0.0	0.0	0.20	3.69	4.61
34	8.50	0.0	0.0	0.0	0.10	3.26	3.98
35	8.75	0.0	0.0	0.0	0.04	2.91	3.44
36	9.00	0.0	0.0	0.0	0.0	2.64	3.07
37	9.25	0.0	0.0	0.0	0.0	2.45	2.75
38	9.50	0.0	0.0	0.0	0.0	2.33	2.55
39	9.75	0.0	0.0	0.0	0.0	2.25	2.45
40	10.00	0.0	0.0	0.0	0.0	2.15	2.30

Symmetrical V-shaped cascade of three planes and one converging surface segment of 180 deg, catchment area is 6.53 km sq.
Storm depth: 90.00 mm
Storm duration: 6.00 hr
Hydraulic conductivity of soil: 1.21 mm/hr
Sorptivity of soil: 9.02 mm/hr**0.5

Tab.2

RAINFALL - RUNOFF EVENT "PLANE 2"

(variable intensity rainfall)

No.	Time hr	Rain mm/hr	Infil mm/hr	Ef.rain mm/hr	Q-plane m3/s	Q-conv m3/s	Q-stream m3/s
1	0.25	15.00	10.05	0.00	0.00	0.00	0.00
2	0.50	16.00	6.35	6.92	0.42	0.00	0.10
3	0.75	18.00	4.96	13.04	2.50	0.01	0.79
4	1.00	14.00	4.28	9.72	4.97	0.03	2.17
5	1.25	28.00	3.85	24.15	12.89	0.12	5.29
6	1.50	20.00	3.55	16.45	14.70	0.37	8.20
7	1.75	12.20	3.33	8.87	11.87	0.77	7.94
8	2.00	10.00	3.15	6.85	8.69	1.32	6.38
9	2.25	2.00	2.00	0.0	5.18	2.81	4.82
10	2.50	2.00	2.00	0.0	3.10	5.53	4.24
11	2.75	0.00	0.00	0.0	1.99	7.07	5.47
12	3.00	1.00	1.00	0.0	1.33	8.08	7.12
13	3.25	5.00	5.00	0.0	0.92	7.91	8.13
14	3.50	3.00	3.00	0.0	0.66	7.48	8.37
15	3.75	0.0	0.0	0.0	0.48	7.10	7.91
16	4.00	0.0	0.0	0.0	0.36	6.81	7.44
17	4.25	0.0	0.0	0.0	0.28	6.42	7.08
18	4.50	0.0	0.0	0.0	0.22	5.72	6.70
19	4.75	0.0	0.0	0.0	0.18	4.67	6.13
20	5.00	0.0	0.0	0.0	0.15	3.49	5.21
21	5.25	0.0	0.0	0.0	0.12	2.40	4.04
22	5.50	0.0	0.0	0.0	0.10	1.52	2.93
23	5.75	0.0	0.0	0.0	0.09	0.89	1.98
24	6.00	0.0	0.0	0.0	0.08	0.48	1.22
25	6.25	0.0	0.0	0.0	0.06	0.24	0.70
26	6.50	0.0	0.0	0.0	0.05	0.12	0.38
27	6.75	0.0	0.0	0.0	0.05	0.10	0.20
28	7.00	0.0	0.0	0.0	0.04	0.08	0.13
29	7.25	0.0	0.0	0.0	0.04	0.07	0.10
30	7.50	0.0	0.0	0.0	0.03	0.06	0.09

Symmetrical V-shaped cascade of three planes, one converging surface segment of 180 deg, catchment area is 6.53 km sq.
Storm depth: 90.00 mm
Storm duration: 6.00 hr
Hydraulic conductivity: 1.21 mm/hr
Sorptivity: 9.02 mm/hr**0.5

Tab.3

STORM RAINFALL-RUNOFF DATA ON THE HUPSELSE CATCHMENT

(DT= 15 min, 30 min)

EVENT NO.	DATE	RAIN DEPTH MM	CUMUL.INF. DEPTH MM 15'	CUMUL.INF. DEPTH MM 30'	EFFECT.RAIN DEPTH MM 15'	EFFECT.RAIN DEPTH MM 30'	OBSERVED RUNOFF DEPTH MM
1	23/5/1972	30.12	25.46	20.94	4.66	9.18	2.50
2	27/5/1972	44.00	38.34	42.16	5.66	4.91	4.50
3	27/7/1972	38.36	34.51	36.52	3.85	1.84	8.40
4	10/7/1980	30.60	30.06	30.57	0.44	0.04	7.25
5	29/9/1984	24.80	20.66	24.20	4.14	0.60	3.34
6	14/8/1985	29.64	15.68	24.15	13.45	5.49	4.10
7	6/5/1986	18.48	18.48	18.18	0.00	0.30	5.40
8	1/3/1987	73.53	65.66	68.44	7.87	7.09	15.75

Tab.5

THE DESIGN RAINFALLS (De Bilt)

(For the reccurence period Te= 2, 10, 100 years)

No.	TIME min	Reccurence period 2 Y INTEN. mm/min	2 Y DEPTH mm	10 Y INTEN. mm/min	10 Y DEPTH mm	100 Y INTEN. mm/min	100 Y DEPTH mm
1	5	1.35	6.75	2.50	12.50	3.40	17.00
2	10	0.90	9.00	1.60	16.00	2.20	22.00
3	15	0.75	11.25	1.20	18.00	1.80	27.00
4	30	0.45	13.50	0.80	24.00	1.10	33.00
5	60	0.25	15.00	0.42	25.20	0.65	39.00
6	120	0.18	21.60	0.30	36.00	0.40	48.00
7	180	0.14	25.20	0.20	36.00	0.30	54.00
8	240	0.10	24.00	0.16	38.40	0.22	52.80
9	360	0.075	27.00	0.11	39.60	0.15	54.00
10	720	0.04	28.80	0.065	46.80	0.085	61.20
11	1440	0.03	43.20	0.04	57.60	0.05	72.00

H U P S E L S E B E E K (MODEL KINFIL) Tab 4/1

RECONSTRUCTION OF THE RAINFALL-RUNOFF EVENT No.1
23/05/1972

No.	TIME HR	RAIN MM/HR	EF.RAIN MM/HR	Q-PLANE M3/S comp.	Q-STREAM M3/S comp.	Q-H.B. M3/S meas.
1	1.0	27.00	4.66	0.312	0.109	0.034
2	2.0	3.12	0.00	0.632	0.108	0.054
3	3.0	0.0	0.0	0.632	0.326	0.189
4	4.0	0.0	0.0	0.630	0.449	0.359
5	5.0	0.0	0.0	0.630	0.458	0.439
6	6.0	0.0	0.0	0.626	0.460	0.400
7	7.0	0.0	0.0	0.610	0.454	0.341
8	8.0	0.0	0.0	0.576	0.439	0.293
9	9.0	0.0	0.0	0.496	0.411	0.253
10	10.0	0.0	0.0	0.456	0.363	0.224
11	11.0	0.0	0.0	0.388	0.315	0.202
12	12.0	0.0	0.0	0.324	0.274	0.186
13	13.0	0.0	0.0	0.268	0.228	0.170
14	14.0	0.0	0.0	0.224	0.189	0.159
15	15.0	0.0	0.0	0.188	0.157	0.148
16	16.0	0.0	0.0	0.162	0.132	0.141
17	17.0	0.0	0.0	0.144	0.113	0.132
18	18.0	0.0	0.0	0.126	0.100	0.126
19	19.0	0.0	0.0	0.108	0.089	0.121
20	20.0	0.0	0.0	0.098	0.078	0.116

H U P S E L S E B E E K (MODEL KINFIL) Tab. 4/2

RECONSTRUCTION OF THE RAINFALL-RUNOFF EVENT No. 2
27/05/1972

No.	TIME HR	RAIN MM/HR	EF.RAIN MM/HR	Q-PLANE M3/S comp.	Q-STREAM M3/S comp.	Q-H.B. M3/S observed
1	1.0	1.56	0.0	0.000	0.000	0.036
2	2.0	12.40	2.06	0.466	0.163	0.036
3	3.0	13.68	3.60	0.844	0.162	0.040
4	4.0	1.56	0.0	0.844	0.455	0.051
5	5.0	0.0	0.0	0.844	0.598	0.146
6	6.0	0.0	0.0	0.842	0.607	0.432
7	7.0	2.52	0.0	0.826	0.597	0.706
8	8.0	0.24	0.0	0.784	0.568	0.746
9	9.0	0.0	0.0	0.708	0.517	0.690
10	10.0	0.72	0.0	0.610	0.448	0.632
11	11.0	0.36	0.0	0.506	0.372	0.560
12	12.0	0.12	0.0	0.412	0.302	0.514
13	13.0	0.0	0.0	0.336	0.243	0.469
14	14.0	0.0	0.0	0.274	0.197	0.432
15	15.0	0.0	0.0	0.226	0.161	0.403
16	16.0	0.12	0.0	0.188	0.134	0.374
17	17.0	0.0	0.0	0.158	0.113	0.347
18	18.0	1.08	0.0	0.136	0.097	0.323
19	19.0	0.48	0.0	0.116	0.084	0.302
20	20.0	2.04	0.0	0.100	0.073	0.285

H U P S E L S E   B E E K   (MODEL KINFIL)   Tab. 4/3

RECONSTRUCTION OF THE RAINFALL-RUNOFF EVENT No. 5
29/09/1984

No.	TIME HR	RAIN MM/HR	EF.RAIN MM/HR	Q-PLANE M3/S comp.	Q-STREAM M3/S comp.	Q-H.B. M3/S observed
1	1.0	0.12	0.0	0.000	0.000	0.036
2	2.0	16.80	4.14	0.500	0.176	0.036
3	3.0	5.88	0.0	0.500	0.173	0.042
4	4.0	1.44	0.0	0.500	0.344	0.134
5	5.0	0.24	0.0	0.500	0.356	0.379
6	6.0	0.0	0.0	0.498	0.366	0.525
7	7.0	0.0	0.0	0.499	0.368	0.529
8	8.0	0.0	0.0	0.484	0.363	0.477
9	9.0	0.0	0.0	0.462	0.350	0.426
10	10.0	0.0	0.0	0.428	0.328	0.379
11	11.0	0.0	0.0	0.382	0.300	0.343
12	12.0	0.0	0.0	0.332	0.266	0.314
13	13.0	0.0	0.0	0.282	0.230	0.287
14	14.0	0.0	0.0	0.240	0.195	0.267
15	15.0	0.0	0.0	0.202	0.164	0.253
16	16.0	0.0	0.0	0.170	0.139	0.240
17	17.0	0.0	0.0	0.146	0.118	0.228
18	18.0	0.0	0.0	0.124	0.101	0.218
19	19.0	0.0	0.0	0.108	0.087	0.208
20	20.0	0.0	0.0	0.094	0.076	0.200

H U P S E L S E   B E E K   (MODEL KINFIL)   Tab. 4/4

RECONSTRUCTION OF THE RAINFALL-RUNOFF EVENT No. 6
14/08/1985

No.	TIME HR	RAIN MM/HR	EF.RAIN MM/HR	Q-PLANE M3/S comp.	Q-STREAM M3/S comp.	Q-H.B. M3/S observed
1	1.0	23.52	5.49	0.000	0.000	0.014
2	2.0	5.88	0.0	0.268	0.094	0.015
3	3.0	0.12	0.0	0.800	0.093	0.033
4	4.0	0.0	0.0	0.800	0.371	0.244
5	5.0	0.12	0.0	0.800	0.567	0.648
6	6.0	0.0	0.0	0.798	0.576	0.852
7	7.0	0.0	0.0	0.786	0.578	0.740
8	8.0	0.0	0.0	0.748	0.568	0.596
9	9.0	0.0	0.0	0.680	0.542	0.484
10	10.0	0.0	0.0	0.590	0.495	0.404
11	11.0	0.0	0.0	0.494	0.431	0.343
12	12.0	0.0	0.0	0.404	0.361	0.303
13	13.0	0.0	0.0	0.330	0.295	0.271
14	14.0	0.0	0.0	0.270	0.238	0.242
15	15.0	0.0	0.0	0.222	0.193	0.220
16	16.0	0.0	0.0	0.186	0.158	0.202
17	17.0	0.0	0.0	0.156	0.132	0.188
18	18.0	0.0	0.0	0.134	0.111	0.175
19	19.0	0.0	0.0	0.116	0.095	0.164
20	20.0	0.0	0.0	0.100	0.083	0.155

INFILTRATION ON THE HUPSELSE CATCHMENT   Tab. 6

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36 INFILTROMETERS RUN ON 1 HA SPOT AT 20x20 m NETWORK

SOIL GROUP		SOIL PARAMETERS		CRITICAL RUNOFF DEPTH IN Te = 2,10,100 years					
				Te= 2 years		Te= 10 years		T= 100 years	
No.	Symb.	Ks	So	Time	Excess	Time	Excess	Time	Excess
		mm/min	mm/min**1/2	min	mm	min	mm	min	mm
1	A/1	0.06148	0.018	120	13.46	120	28.00	180	42.10
2	A/2	0.92620	5.384	-	-	5	1.75	5	2.93
3	A/3	0.55379	2.489	15	1.00	5	5.91	15	11.17
4	A/4	2.34717	0.719	-	-	-	-	5	0.59
5	A/5	1.91081	4.608	-	-	-	-	5	0.51
6	A/6	0.70116	2.173	-	-	5	4.86	5	9.54
7	B/1	0.26909	1.803	15	6.01	30	14.32	30	23.48
8	B/2	0.32852	0.078	15	4.94	30	12.19	30	21.44
9	B/3	0.63408	3.689	5	0.25	5	4.56	5	8.30
10	B/4	0.24077	-0.497	-	-	-	-	-	-
11	B/5	0.36983	-0.188	-	-	-	-	-	-
12	B/6	1.37206	2.910	-	-	5	0.54	5	3.51
13	C/1	0.61007	0.395	-	-	-	-	5	0.59
14	C/2	0.76008	3.232	5	0.07	5	3.74	5	8.01
15	C/3	0.25687	0.804	15	4.36	30	11.80	30	21.07
16	C/4	0.88191	3.592	-	-	5	2.66	5	6.79
17	C/5	0.96684	-0.502	-	-	-	-	-	-
18	C/6	1.24816	5.018	-	-	5	0.50	5	3.23
19	D/1	0.41467	0.865	5	2.58	10	8.70	15	16.89
20	D/2	0.61036	3.403	10	0.44	5	4.90	5	9.31
21	D/3	1.17833	5.868	-	-	5	0.57	5	3.36
22	D/4	1.49280	7.079	-	-	5	0.03	5	1.30
23	D/5	0.37080	2.316	5	2.08	10	7.82	15	15.80
24	D/6	0.34685	7.210	5	1.08	5	6.15	15	12.89
25	E/1	1.14032	0.366	5	0.01	5	3.91	5	8.60
26	E/2	0.89494	5.997	-	-	5	1.77	5	5.49
27	E/3	0.69151	1.327	5	0.52	5	5.66	5	9.90
28	E/4	0.59246	1.774	5	0.80	5	6.07	15	11.26
29	E/5	0.38835	5.436	5	1.08	5	6.21	15	12.76
30	E/6	1.10277	9.262	-	-	5	0.39	5	2.84
31	F/1	0.50350	0.696	5	2.08	5	7.88	15	15.40
32	F/2	1.05522	7.577	-	-	5	0.71	5	3.64
33	F/3	0.73138	0.279	5	1.15	5	7.03	15	12.34
34	F/4	0.95938	6.347	-	-	10	1.67	5	4.81
35	F/5	0.94687	2.713	-	-	5	2.65	5	6.85
36	F/6	0.76948	5.726	-	-	5	2.68	5	6.70
AR. MEAN		0.79525	3.055	5	0.05	5	3.60	5	7.90

HUPSELSE BEEK - D E S I G N   R U N O F F S   Tab. 7

(Design rainfalls, station De Bilt)
MODEL INFHUP

No.	RAINFALL DURATION min	RETURN PERIOD T= 2 YEAR RAIN mm	T= 2 YEAR EF.RAIN mm	T= 10 YEAR RAIN mm	T= 10 YEAR EF.RAIN mm	T= 100 YEAR RAIN mm	T= 100 YEAR EF.RAIN mm
1	5.0	6.75	5.67	12.50	11.40	17.00	15.90
2	10.0	9.00	7.33	16.00	14.32	22.00	20.32
3	15.0	11.25	9.08	18.00	15.82	27.00	24.82
4	30.0	13.50	10.02	24.00	20.53	33.00	29.53
5	60.0	15.00	9.17	25.20	19.46	39.00	33.31
6	120.0	21.60	11.63	36.00	26.26	48.00	38.35
7	180.0	25.20	11.27	36.00	22.36	54.00	40.64
8	240.0	24.00	6.28	38.40	20.98	52.80	35.74
9	360.0	27.00	2.92	39.60	14.86	54.00	29.76
10	720.0	28.80	0.00	46.80	3.23	61.20	15.91
11	1440.0	43.20	0.00	57.60	0.00	72.00	0.22

THE DESIGN CONDITIONS:

CURVE NUMBER CN= 74.00
HYDRAULIC CONDUCTIVITY KT= 0.041 MM/MIN
SORPTIVITY SO= 2.018 MM/MIN**1/2
STORAGE SUCTION FACTOR SF= 49.68 MM
ANTECEDENT MOISTURE CONDITIONS: THE FIELD CAPACITY

M o d e l   KINFIL

Tab. 8

H U P S E L S E   B E E K - D E S I G N   F L O O D S

(Design rainfalls - station De Bilt)
RETURN PERIOD Te= 2 YEARS

ORD. No.	TIME hours	Q-30MIN m3/s	Q-60MIN m3/s	Q-120MIN m3/s	Q-180MIN m3/s	Q-240MIN m3/s
1	1.00	0.813	0.690	0.236	0.080	0.007
2	2.00	0.843	0.722	0.243	0.083	0.007
3	3.00	1.578	1.350	1.180	0.459	0.062
4	4.00	1.494	1.284	1.823	1.223	0.204
5	5.00	1.461	1.277	1.823	1.726	0.466
6	6.00	1.387	1.272	1.808	1.774	0.663
7	7.00	1.209	1.188	1.663	1.751	0.712
8	8.00	0.971	1.016	1.375	1.574	0.754
9	9.00	0.750	0.813	1.057	1.282	0.772
10	10.00	0.577	0.633	0.796	0.986	0.752
11	11.00	0.449	0.493	0.603	0.747	0.687
12	12.00	0.356	0.388	0.466	0.572	0.595
13	13.00	0.286	0.311	0.368	0.446	0.495
14	14.00	0.233	0.253	0.294	0.354	0.403
15	15.00	0.192	0.207	0.238	0.284	0.327
16	16.00	0.159	0.172	0.196	0.231	0.267
17	17.00	0.134	0.144	0.163	0.190	0.220
18	18.00	0.113	0.121	0.136	0.158	0.183
19	19.00	0.097	0.103	0.115	0.132	0.153
20	20.00	0.083	0.089	0.098	0.112	0.129
21	21.00	0.072	0.077	0.085	0.096	0.110
22	22.00	0.064	0.067	0.074	0.082	0.094
23	23.00	0.059	0.061	0.064	0.071	0.081
24	24.00	0.054	0.056	0.055	0.062	0.071
25	25.00	0.050	0.052	0.049	0.055	0.061
26	26.00	0.046	0.048	0.044	0.049	0.053
27	27.00	0.042	0.044	0.039	0.043	0.047
28	28.00	0.039	0.040	0.035	0.039	0.042
29	29.00	0.035	0.037	0.032	0.035	0.038
30	30.00	0.032	0.033	0.029	0.032	0.035

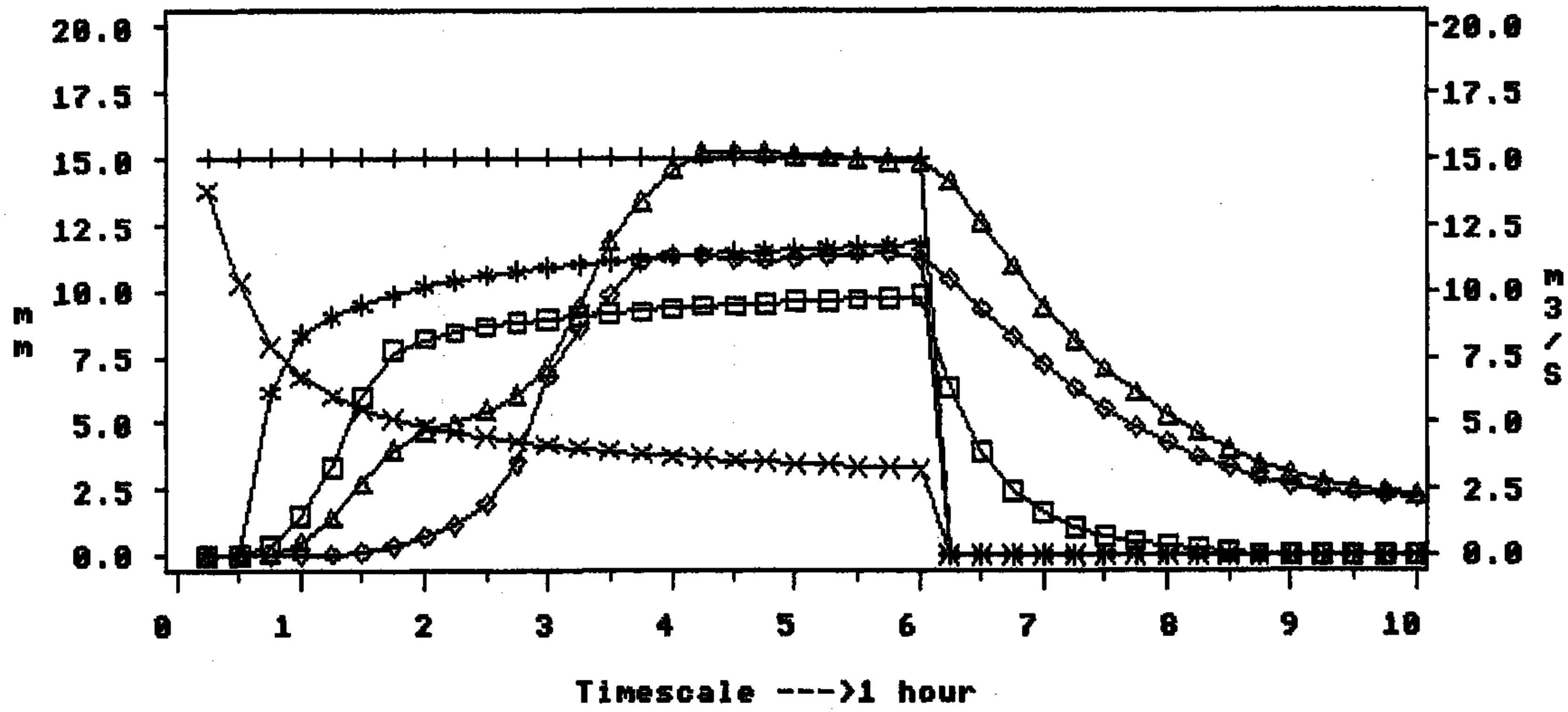
DESIGN CONDITIONS:

CURVE NUMBER CN= 74
HYDRAULIC CONDUCTIVITY KT= 0.041 MM/MIN
SORPTIVITY SO= 2.018 MM/MIN**1/2
STORAGE SUCTION FACTOR SF= 49.68 MM
ANTECEDENT MOISTURE CONDITIONS: THE FIELD CAPACITY

Fig. 8

RAINFALL–RUNOFF EVENT "PLANE 1"

CONSTANT–INTENSITY RAINFALL



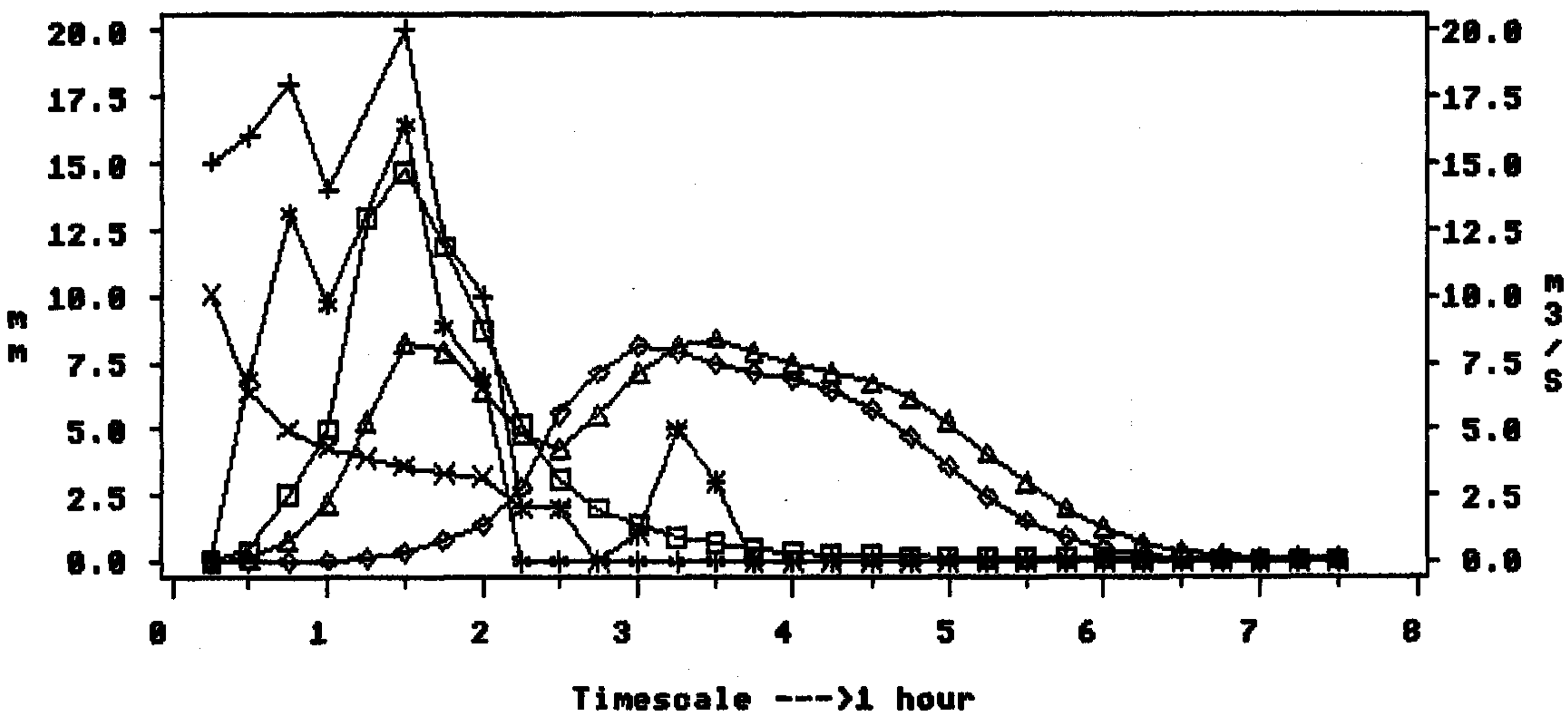
LEGEND:
- + Rainfall hyetograph (mm/hr)
- x Infiltration curve (mm/hr)
- * Effective rainfall (mm/hr)
- □ Plane - hydrograph (m3/s)
- ◇ Conver - hydrograph (m3/s)
- △ Stream - result hydrograph (m3/s)

SOURCE: KINFIL MODEL

RAINFALL–RUNOFF EVENT "PLANE 2"

Fig. 9

VARIABLE–INTENSITY RAINFALL



LEGEND:

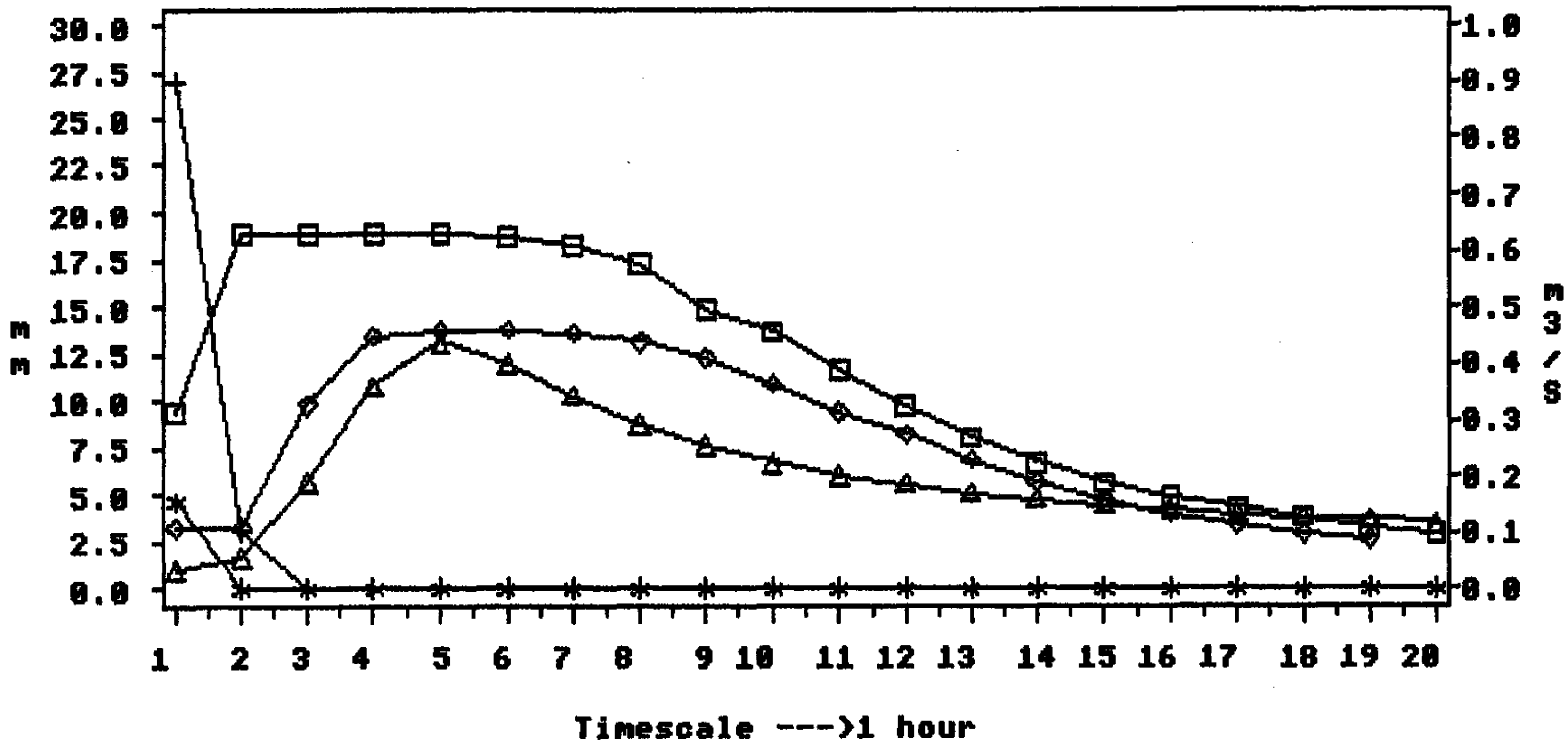
- \+ Rainfall hyetograph (mm/hr)
- x Infiltration curve (mm/hr)
- * Effective rainfall (mm/hr)
- □ Plane - hydrograph (m3/s)
- ◇ Conver - hydrograph (m3/s)
- △ Stream - result hydrograph (m3/s)

SOURCE: KINFIL MODEL

HUPSELSE BEEK

Fig. 10 A

EVENT 1: 23 / 5 / 1972



LEGEND:

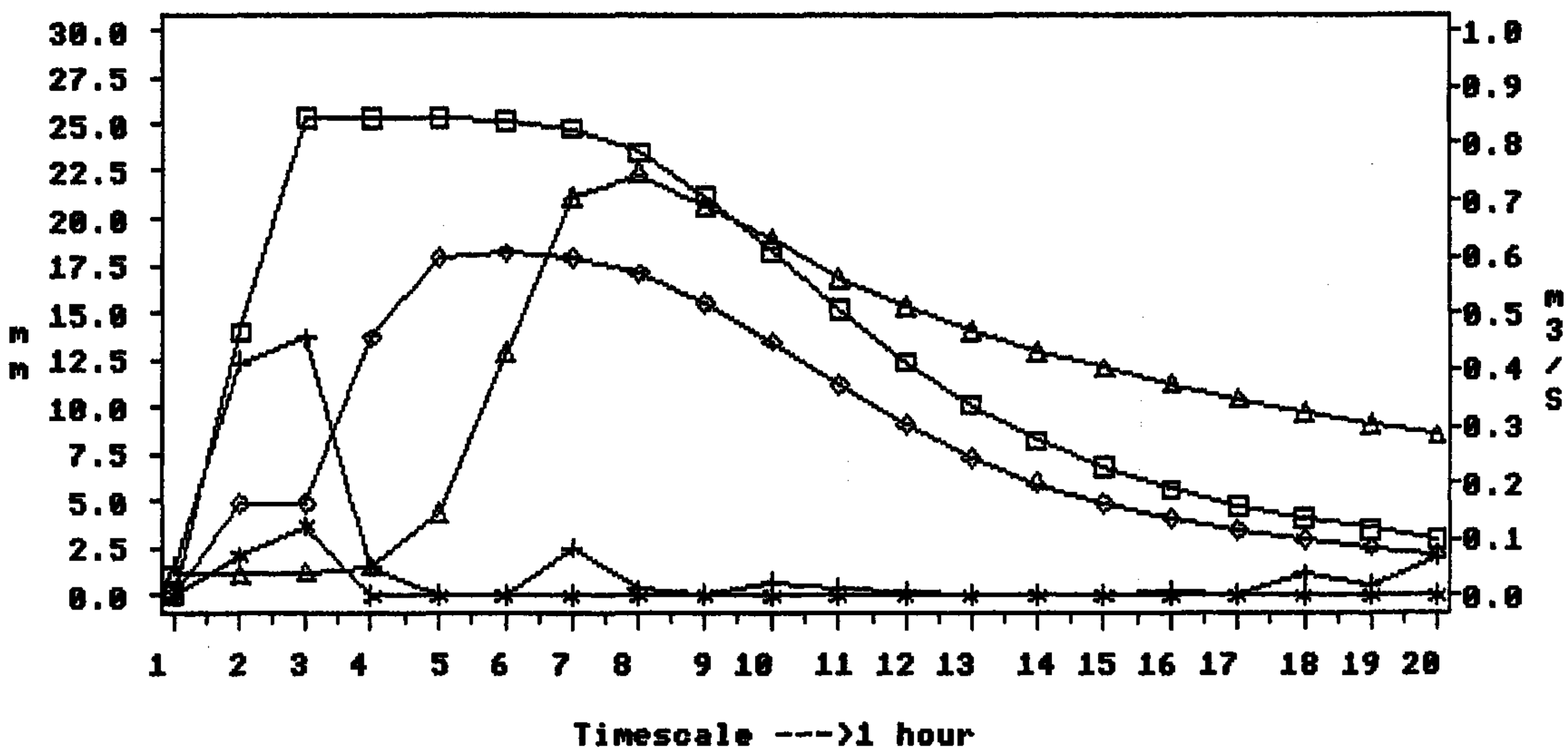
- \+ Rainfall hyetograph (mm/hr)
- * Effective rainfall (mm/hr)
- □ Q - Plane hydrograph (m3/s)
- ◇ Q - Stream hydrograph (m3/s)
- △ Q - Observed hydrograph (m3/s)

SOURCE: KINFIL MODEL

HUPSELSE BEEK

Fig. 10 B

EVENT 2: 27 / 5 / 1972



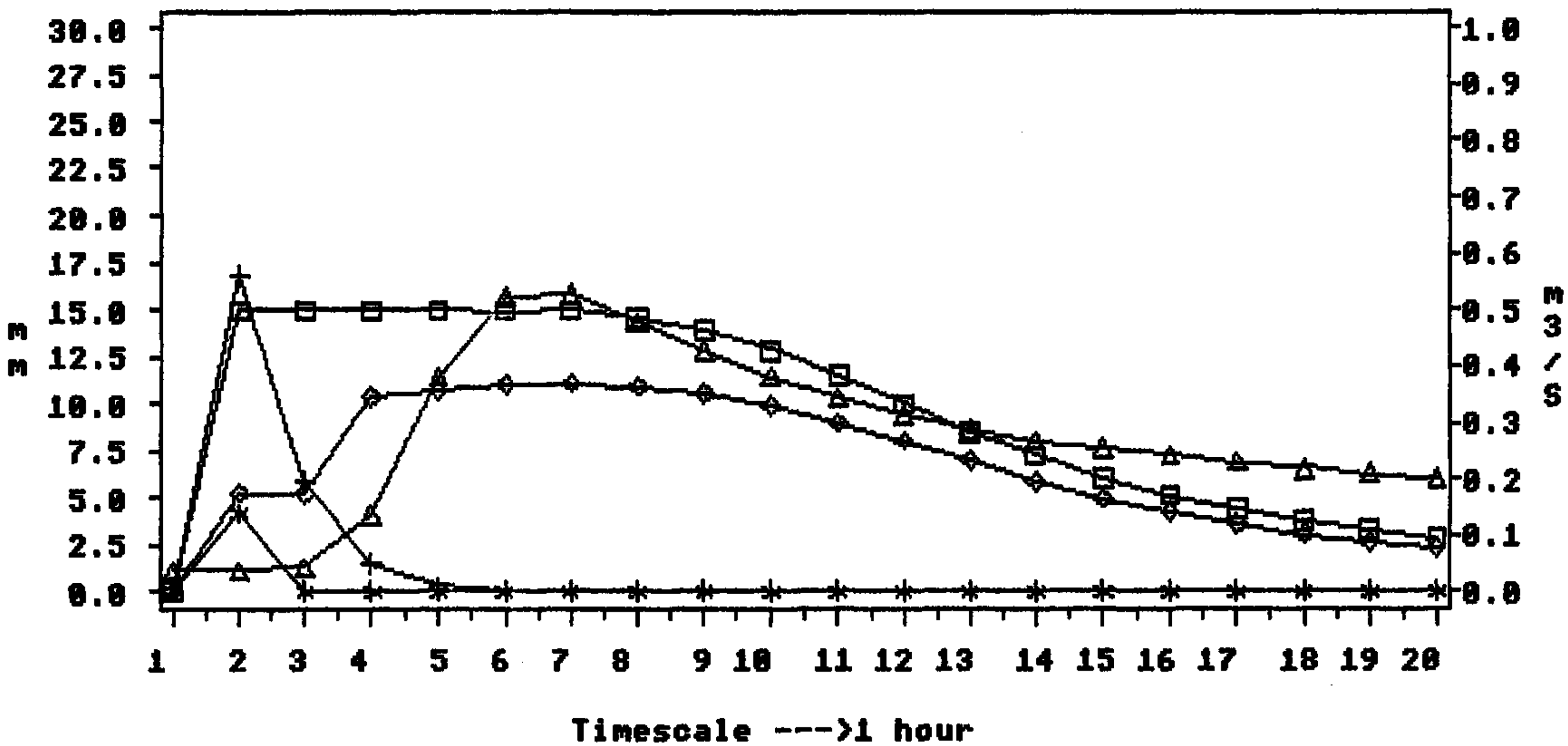
LEGEND:
- \+ Rainfall hyetograph (mm/hr)
- * Effective rainfall (mm/hr)
- □ Q - Plane hydrograph (m3/s)
- ◇ Q - Stream hydrograph (m3/s)
- △ Q - Observed hydrograph (m3/s)

SOURCE: KINFIL MODEL

HUPSELSE BEEK

Fig. 10 C

EVENT 5: 29 / 9 / 1984



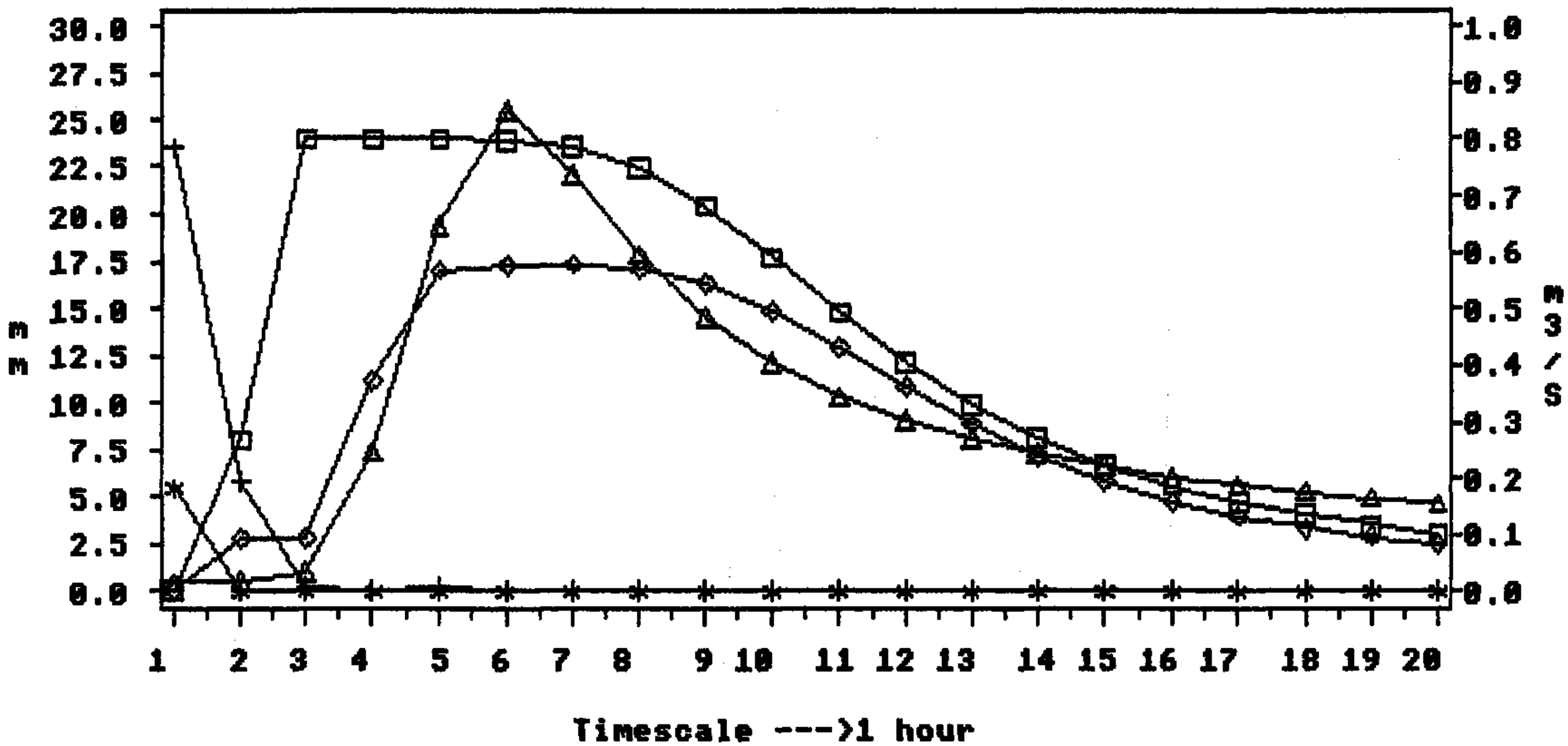
LEGEND:
- + Rainfall hyetograph (mm/hr)
- * Effective rainfall (mm/hr)
- □ Q - Plane hydrograph (m3/s)
- ◇ Q - Stream hydrograph (m3/s)
- △ Q - Observed hydrograph (m3/s)

SOURCE: KINFIL MODEL

HUPSELSE BEEK

EVENT 6: 1 / 3 / 1987

Fig. 10 D

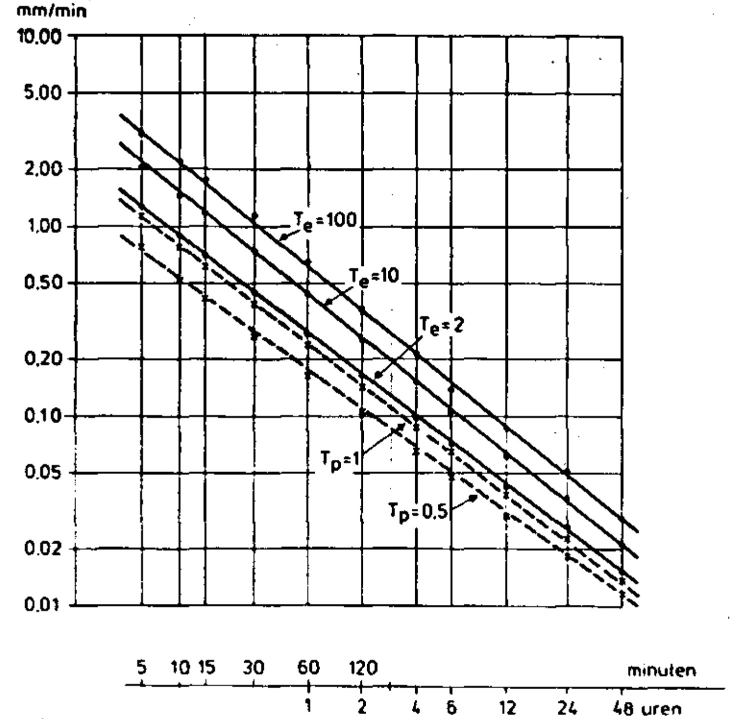


LEGEND:

- \+ Rainfall hyetograph (mm/hr)
- * Effective rainfall (mm/hr)
- □ Q - Plane hydrograph (m3/s)
- ◇ Q - Stream hydrograph (m3/s)
- △ Q - Observed hydrograph (m3/s)

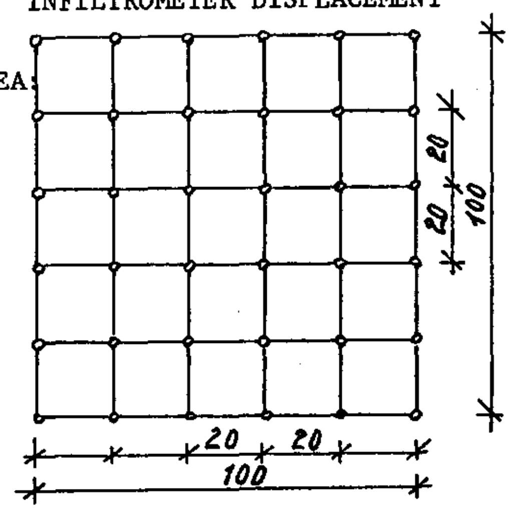
SOURCE: KINFIL MODEL

THE DESIGN RAINFALL
INTENSITY DURATION CURVES



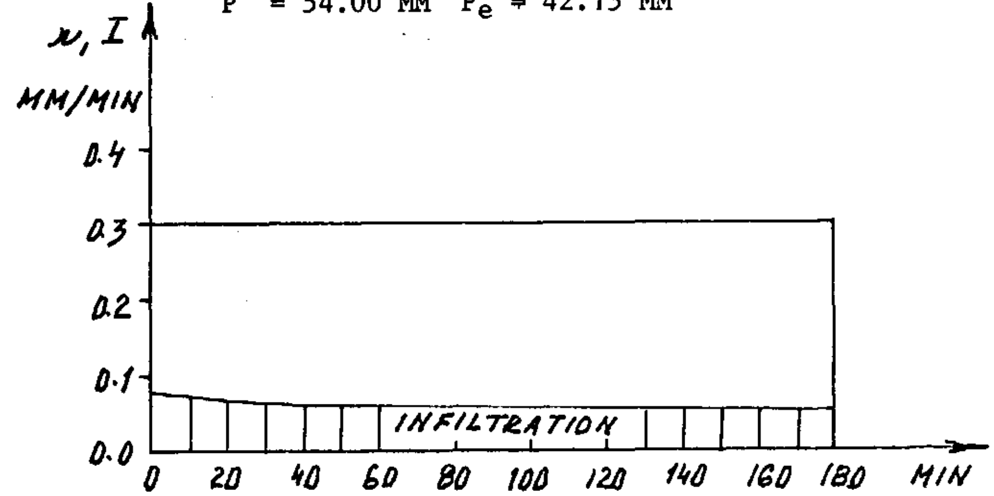
INFILTROMETER DISPLACEMENT

TOTAL AREA:
1 HA

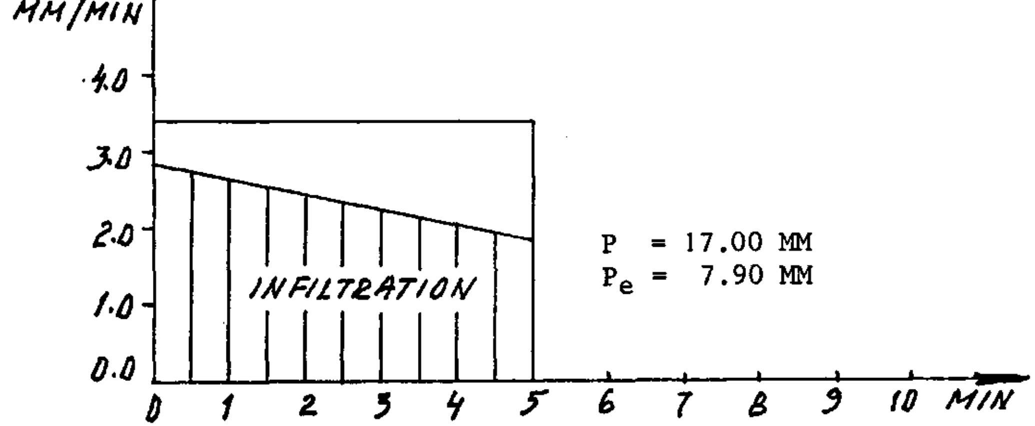


INFILTRATION STUDY ON THE HUPSELSE BEEK

THE "WORST" EVENT: A/1
K_s = 0.06148 MM/MIN
S_o = 0.018 MM/MIN1/2
T_e = 1004, t_d = 180.0 MIN
P = 54.00 MM P_e ≑ 42.15 MM

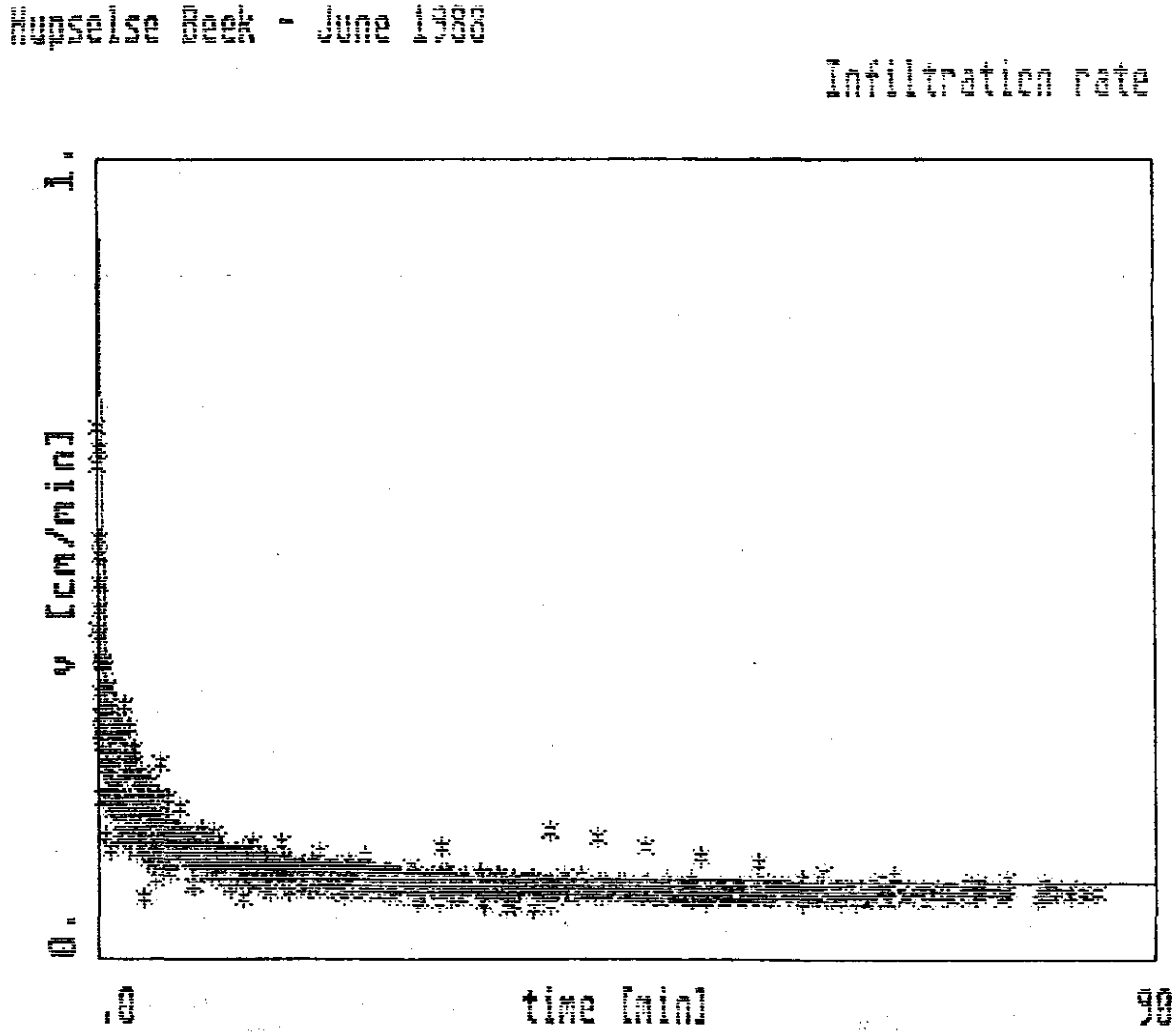


THE "AVERAGE" EVENT: MEAN
K_s = 0.79525 MM/MIN
S_o = 3.055 MM/MIN1/2
T_e = 1004, t_d = 5.0 MIN



P = 17.00 MM
P_e = 7.90 MM

Fig. 11


Hupselse Beek - June 1988
Infiltration rate
1.
v [cm/min]
0.
.0
time [min]
90

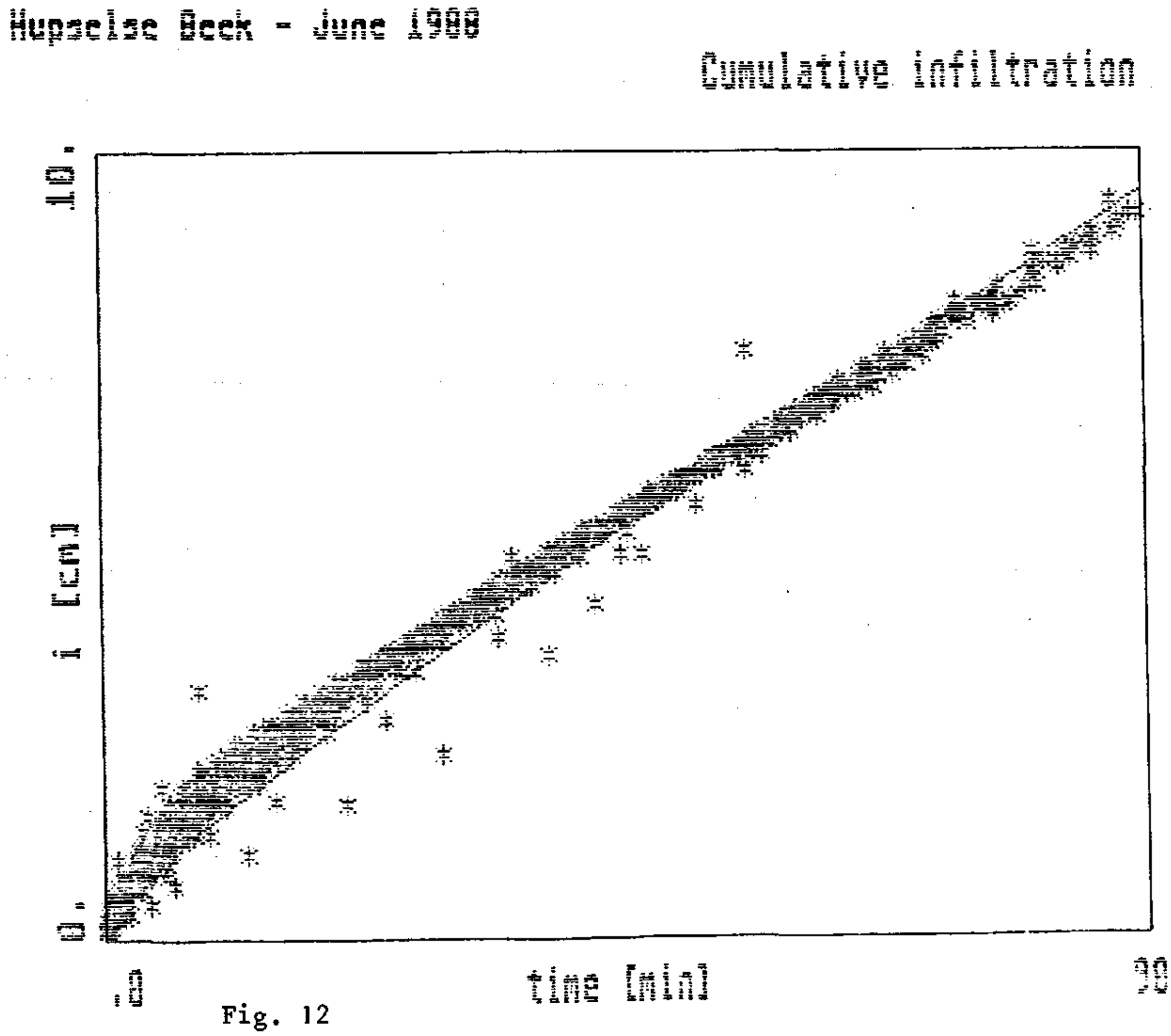
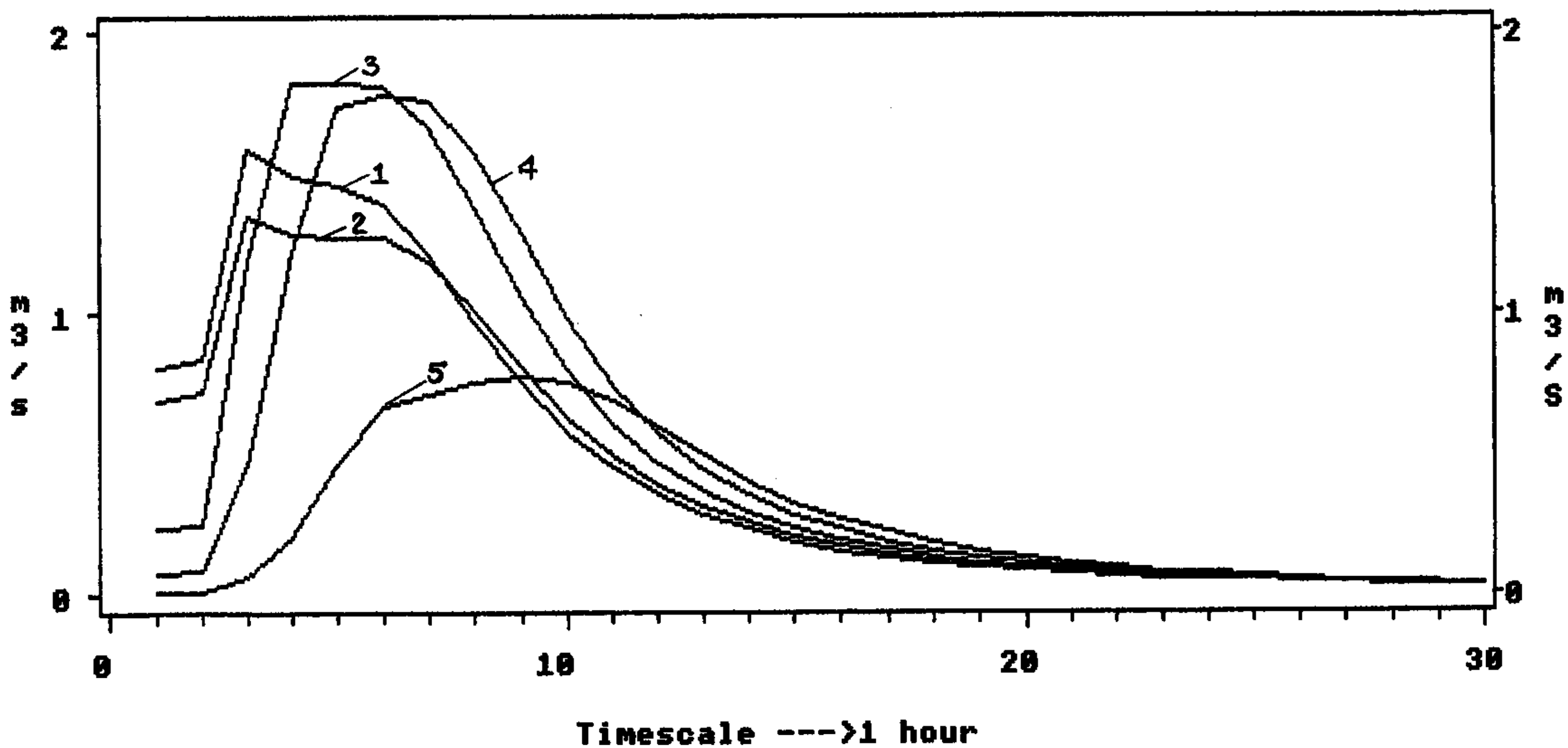

Hupselse Beek - June 1988
Cumulative infiltration
10.
i [cm]
0.
.0
time [min]
90

Fig. 12

HUPSELSE BEEK – DESIGN FLOODS

Fig. 13

RETURN PERIOD TE = 2 YEARS



LEGEND:

1 Hydrograph due to rain td = 30 min
2 Hydrograph due to rain td = 60 min
3 Hydrograph due to rain td = 120 min
4 Hydrograph due to rain td = 180 min
5 Hydrograph due to rain td = 240 min

SOURCE: KINFIL MODEL