HYDROLOGY PROJECT
Technical Assistance

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Training module # SWDP - 38

How to do hydrological data validation using hydrological models

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DHV Consultants BV & DELFT HYDRAULICS

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1. Module context

While designing a training course, the relationship between this module and the others, would be maintained by keeping them close together in the syllabus and place them in a logical sequence. The actual selection of the topics and the depth of training would, of course, depend on the training needs of the participants, i.e. their knowledge level and skills performance upon the start of the course.

2. Module profile

Title	:	How to do hydrological data validation using hydrological models
Target group	:	Hydrologists, Data Processing Centre Managers
Duration	:	Two sessions of 90 min each.
Objectives	:	After training, the participants will be able to 1. carry out hydrological data validation using hydrological models 2. filling in missing data using hydrological models
Key concepts	:	• conceptual approach in rainfall-runoff modelling • schematisation of basins for modelling • calibration of hydrological model Sacramento • use of model for data validation • use of model for filling in missing data
Training methods	:	Lecture, exercises
Training tools required	:	Board, OHP, Computer
Handouts	:	As provided in this module
Further reading and references	:	

3. Session plan

No	Activities	Time	Tools
1	***Introduction*** • What is a hydrological model • Storages and hydrological processes • Operation of the model • Optimisation of the model • Model calibration and uncertainty • Calibration and verification • Use of rainfall-runoff models	10 min	 OHS 1 OHS 2 OHS 3 OHS 4 OHS 5 OHS 6 OHS 7
2	***Conceptual modelling using Sacramento model*** • Model characteristics • Segments and channel elements • Transformation of rainfall into runoff • Sacramento model concept overall • Scheme of Sacramento concept for segments • Sacramento segment module • Upper zone storage in scheme • Upper zone storages • Lower zone storage in scheme • Lower zone storages • Percolation (1) • Percolation (2) • Actual percolation demand • Percolation (4) • Baseflow • Drainage parameters • Evaporation • Evapotranspiration, potential and actual • Routing of direct & surface runoff and interflow • Clark method schematically • Clark method parameters • Channel module • Two-layer Muskingum approach	45 min	 OHS 8 OHS 9 OHS 10 OHS 11 OHS 12 OHS 13 OHS 14 OHS 15 OHS 16 OHS 17 OHS 18 OHS 19 OHS 20 OHS 21 OHS 22 OHS 23 OHS 24 OHS 25 OHS 26 OHS 27 OHS 28 OHS 29 OHS 30 OHS 31
3	***Application of Sacramento model*** • Estimation of parameters • Data requirement • Model result example • Application to Jhelum catchment • Schematisation Jhelum • Schematisation and calibration • Clark model parameter estimation (Tc) • Clark model parameter estimation (K) • Result of simulation • Example of application to Bilodra catchment • Schematisation • Model calibration • Use of model for validation • Use of model to fill-in missing data	35 min	 OHS 32 OHS 33 OHS 34 OHS 35 OHS 36 OHS 37 OHS 38 OHS 39 OHS 40 OHS 41 OHS 42 OHS 43 OHS 44 OHS 45

4	***Exercise*** • Create daily rainfall, potential evapotranspiration and runoff series for Bilodra • Catchment schematisation • Model calibration • Apply model for validation • Fill-in mising runoff data	 20 min 20 min 20 min 20 min 10 min	

4. Overhead/flipchart master

5. Handout

Add copy of the main text in chapter 7, for all participants

6. Additional handout

These handouts are distributed during delivery and contain test questions, answers to questions, special worksheets, optional information, and other matters you would not like to be seen in the regular handouts.

It is a good practice to pre-punch these additional handouts, so the participants can easily insert them in the main handout folder.

7. Main text

Contents

How to do hydrological data validation using hydrological models

1. General

1.1 What is a hydrological model

A physical or mathematical model is a simplified version of reality that is amenable to testing.

A hydrological rainfall runoff model is a means of representing the transformation of an input of rainfall over a catchment area to runoff at a specified outflow point. To simplify the complex processes operating over the catchment and beneath its surface, the hydrology of the catchment is conceived as a series of interlinked processes and storages.

Storages are considered as reservoirs for which water budgets are kept and the processes which control the transfer of water from one storage to the next are described mathematically by logical rules and equations to define, initiation, rate and cessation. Storages are allocated a total capacity and an actual content at any particular moment in time.

Complex catchment processes can be simplified and represented in a wide variety of ways and a large number of models have been developed. The selection of a model type depends on the uses to which it will be put and the availability of measured information on inputs, outflows and storages. **The data processing software HYMOS has selected and extended/adapted the Sacramento Model which has had previous wide use and testing. It is physically realistic, it can operate with the amount of information typically available and requires limited computer power.** Many more sophisticated models exist but all are limited by availability and quality of data and for most applications there is little to be gained by the use of more sophisticated models.

1.2 Optimisation, calibration, verification and application

For a particular catchment the operation of the model depends on the selection of the value of storage capacities and the parameters of the linking equations. **This may be done by estimation based on the physical properties of the catchment, e.g. soil type and impermeable area, or they may be computed by the process of optimisation.**

Optimisation is the means by which, using a measured input of rainfall (and evapotranspiration) and successive computer runs, **the parameters of the model are progressively adjusted to improve the correspondence between the gauged outflow (*Q_{gaug}*) and the outflow simulated by the computer run (*Q_{sim}*). Optimisation may be done by manual adjustment of parameters or by automatic optimisation. Optimisation makes use of quantitative measures of goodness of fit (the objective function) such as:**

$$F=\sum_{i=1}^{n}(Q_{t,gaug}-Q_{t,sim})^2$$

for the n values of the time series being optimised. In automatic optimisation the objective function (*F*) is minimised by a search through the parameter space in a defined and efficient way. The model is run with a given set of parameters, the objective function is calculated, the parameters are adjusted and the process repeated until the value of *F* shows no further improvement.

The entire process of parameter estimation and optimisation using measured time series of input rainfall and outflow is referred to as calibration. Calibration is subject to uncertainty in simulation and results in disagreement between recorded and simulated output. Following may be the sources of uncertainties:

(a) random or systematic errors in the input data, e.g. precipitation or evapotranspiration used to represent the input conditions in time and space for the catchment
(b) Random or systematic errors in recorded output data, i.e. measured discharges for comparison with simulated discharges
(c) Errors due to non-optimal parameter values
(d) Errors due to incomplete or biased model structure.

During calibration only error source (c) is minimised, whereas the disagreement between simulated and recorded output is due to all four error sources. Measurement errors (a) and (b) serve as "background noise" and give a minimum level of disagreement below which further parameter or model adjustments will not improve the results. The objective of calibration is therefore to reduce error source (c) until it insignificant compared with the error sources (a) and (b).

It is usual to withhold a part of the measured data from calibration. This can then be used to verify the performance of the model by using the calibrated parameters with the new data (without optimisation) to determine the objective function and goodness of fit. Verification is a means of ensuring that the optimised parameters are a true representation of the physical behaviour of the catchment and not simply a consequence of the model structure.

The calibrated and verified model is then ready for application where the rainfall input is known but the outflow is unknown.

1.3 Uses of hydrological rainfall runoff models

Rainfall runoff models have a wide variety of uses which include:

- filling in and extension of discharge series
- validation of runoff series
- generation of discharges from synthetic rainfall
- real time forecasting of flood waves
- determination of the influence of changing landuse on the catchment (urbanisation, afforestation) or the influence of water use (abstractions, dam construction, etc.)

The use of the model for the HIS is normally limited to the filling in of missing values in discharge series and the correction of suspect values. It is not usually applied to short sequences of missing data but to gaps of several months in length. The time and effort involved in the calibration of the model does not normally justify application to short gaps; though the model may be thus used if it has previously been calibrated for the same catchment.

2. The Sacramento Model

2.1 Outline of model components

The application of the Sacramento model as integrated in HYMOS is based on a semi-distributed approach. It implies that a catchment is divided into a number of segments, which are interconnected by channel reaches as shown in Figure 1.

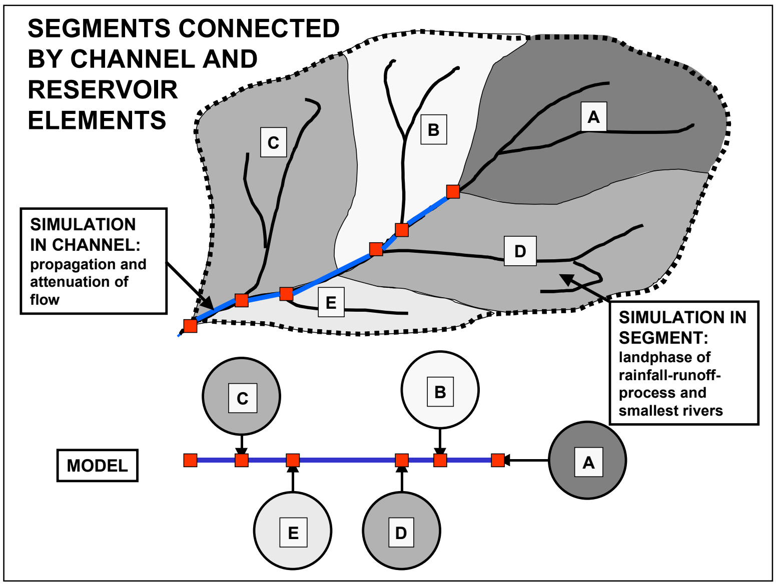

Figure 1: Semi-distributed approach towards rainfall runoff simulation

In a segment rainfall is transformed into runoff to the main river system. An explicit moisture accounting lumped parameter model is used to carry out the transformation. Important elements in the segment phase is the computation of the rainfall abstractions and the response time of the catchment to rainfall input, for which the time of concentration is an indicator, see Figure 2. Within a segment areal homogeneity of rainfall input and basin characteristics is assumed. The contributions of the segments to the main river are routed through the river network where the main features are travel time and flood wave damping. Generally a Muskingum layer approach or unit hydrograph technique is used for the routing.

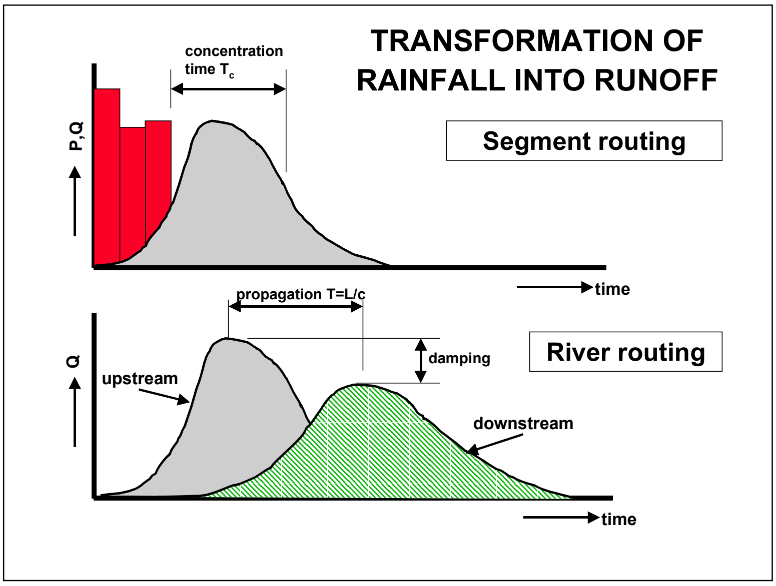

Figure 2: Features of segment and river routing

Basic data input requirements are time series of rainfall, evapo-transpiration and the observed discharge as well as the catchment or segment area. The data time interval depends on the objective of the simulation and is generally taken as 1 hour or 1 day. The model simulates the rainfall runoff process with a time step, which is less than the data time interval.

All parameters and storage capacities have also to be initially estimated on the basis of physical properties of the segment and the river system. Some then remain fixed whilst other are recommended for optimisation.

2.2 The Segment Module

The segment module simulates the rainfall-runoff process in part of the catchment, where the attention is on the land-phase of the rainfall-runoff process. It is assumed that the open water system in the segments contributes little to the shaping of the hydrograph. The conceptualisation of the processes as described in the segment module is presented in Figure 3.

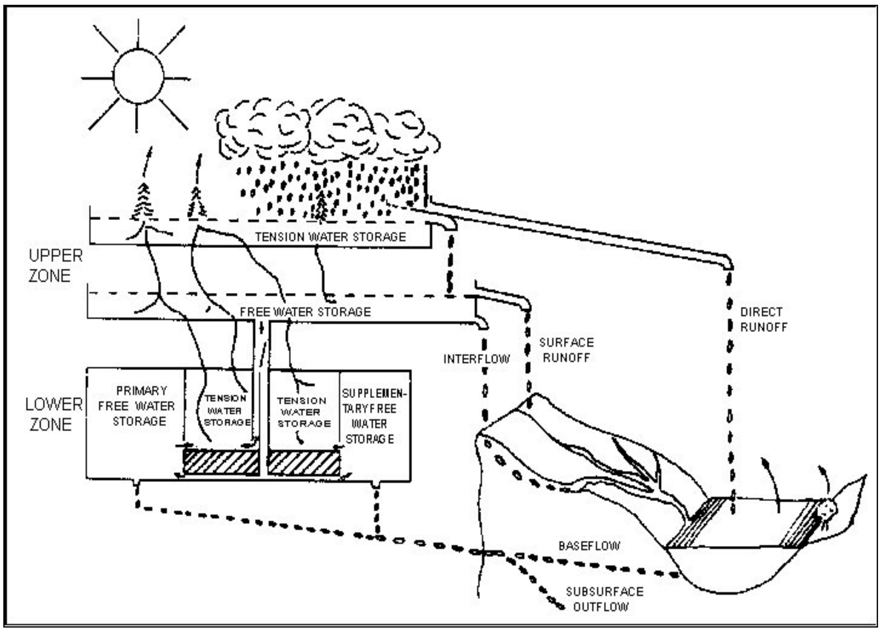

Figure 3: Conceptualisation of the rainfall runoff process in a segment

The segment module is divided into the following components, **(**see also Figure 4):

Impervious area	with transfer to	direct runoff
Previous area		
Upper zone		
Tension storage	with transfer to	evaporation, free water storage
Free water storage	with transfer to	evaporation, percolation, surface runoff and interflow
Lower zone		
Tension storage	with transfer to	evaporation, free water storage
Free water storage	with transfer to	base flow

From the *impervious* areas, precipitation immediately discharges to the channel. However, impervious areas, which drain to a pervious part before reaching the channel, are not considered impervious. Both zones have a tension and a free water storage element. Tension water is considered as the water closely bound to soil particles. Generally first the tension water requirements are fulfilled before water enters the free water storage.

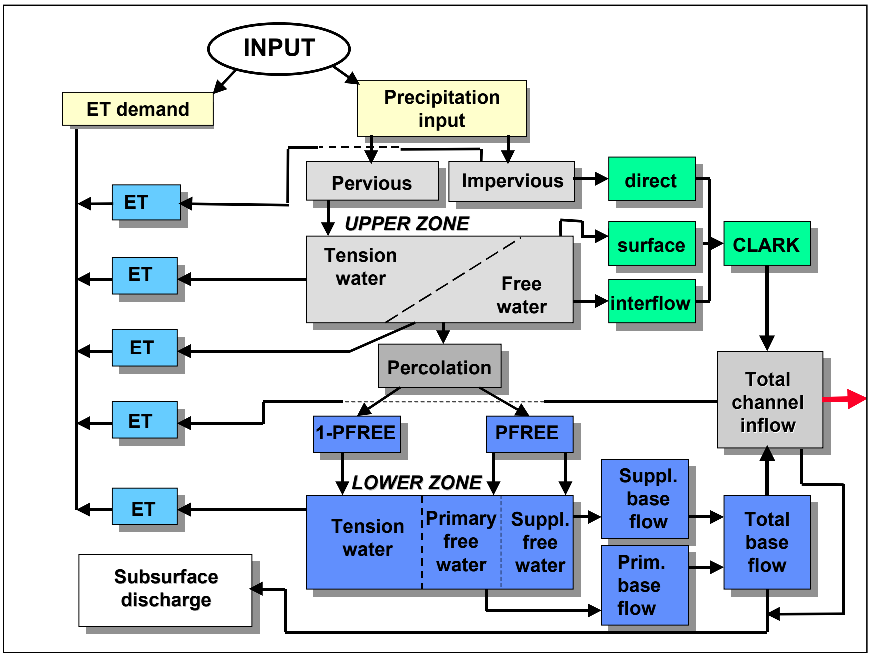

Figure 4: Schematisation of rainfall runoff process in a segment

In the following sub-sections the various components will be described in detail

2.2.1 Upper zone storage

The upper zone tension storage represents that precipitation volume required under dry conditions:

- to meet all interception requirements, and
- to provide sufficient moisture to the upper soil so that percolation can begin.

If the maximum storage capacity of the upper-zone tension storage is exceeded, water becomes available for the upper zone free water storage, a temporary storage from which water percolates to the lower zone system and from which water discharges to the channel via the interflow component. The preferred flow direction from the upper zone is the vertical direction, i.e. percolation to the lower zone system.

Interflow occurs only when the precipitation rate exceeds the percolation rate. The upper zone is treated as a linear storage element which is emptied exponentially: discharge = storage * storage depletion coefficient. The upper zone free water storage depletion coefficient is denoted by *UZK* and the upper zone free water content by *UZFWC* then the interflow takes place at a rate:

$$Q_{interflow} = UZFWC * UZK \qquad (1)$$

When the precipitation intensity exceeds the percolation intensity and the maximum interflow drainage capacity, then the upper zone free water capacity (*UZFWM*) is completely filled and the excess precipitation causes surface runoff.

2.2.2 Lower zone storage

The lower zone consists of the tension water storage, i.e. the depth of water held by the lower zone soil after wetting and drainage (storage up to field capacity) and two free water storages: the primary and supplemental storage elements representing the storages leading to a slow and a fast groundwater flow component, respectively. The introduction of two free lower zone storages is made for greater flexibility in reproducing observed recession curves caused by groundwater flow.

2.2.3 Percolation from upper to lower zones

The percolation rate from the upper zone to the lower zone depends on the one hand on the lower zone demand, i.e. requirements determined by the lower zone water content relative to its capacity and on the other hand on the upper zone free water content relative to its capacity.

The **lower zone percolation demand** is denoted by $PERC_{act.dem}$. The upper zone free water content relative to its capacity is UZFWC/UZFWM. Hence, **the actual percolation intensity** then reads:

$$PERC = PERC_{act.dem} * UZFWC/UZFWM \qquad (2)$$

The lower zone percolation demand has a lower and an upper limit:

- the minimum lower zone percolation demand, and
- the maximum lower zone percolation demand.

The minimum lower zone percolation demand occurs when all three lower zone storages are completely filled. Then by continuity the percolation rate equals the groundwater flow rate from full primary and supplemental reservoirs. Denoting the minimum demand by *PBASE* then it follows:

$$PERC_{min.dem} = PBASE = LZFPM * LZPK + LZFSM * LZSK \qquad (3)$$

where:

LZFPM	= lower zone primary free water storage capacity
LZFSM	= lower zone supplemental free water storage capacity
LZPK	= drainage factor of primary storage
LZSK	= drainage factor of supplemental storage

The maximum lower zone percolation demand takes place if the lower zone is completely dried out i.e. if its content = 0. Then the maximum percolation rate is expressed as a function of *PBASE*:

$$PERC_{max.dem} = PBASE\ (1 + ZPERC) \qquad (4)$$

with: $ZPERC >> 1$ usually.

The actual lower zone percolation demand depends on the lower zone content relative to its capacity. Computationally it means that *ZPERC* has to be multiplied by a function *G* of the relative lower zone water content such that this function:

- equals 1 in case of a completely dry lower zone
- equals 0 in case of a completely saturated lower zone

- represents an approximate exponential decay of the percolation rate in case of a continuous recharge.

In the Sacramento model this function has the following form:

$$G = \left(\frac{\sum (\text{lower zone capacities} - \text{lower zone content})}{\sum (\text{lower zone capacities})} \right)^{REXP} \qquad (5)$$

and the *actual percolation demand* is given by (see Figure 5):

$$PERC_{act.dem} = PBASE\ (1 + ZPERC * G) \qquad (6)$$

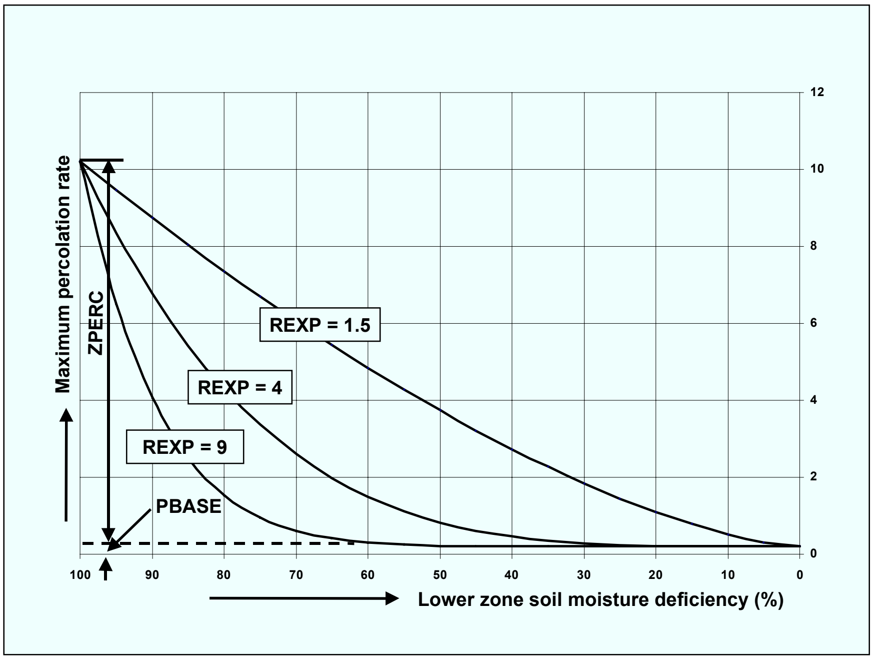

Figure 5: Actual percolation demand representation

2.2.4 Distribution of percolated water from upper zone

The percolated water drains to three reservoirs, one tension and two free water reservoirs. Based on the preceding comments one would expect that the lower zone tension storage is filled first before percolation to the lower zone free water storages takes place. However, variations in soil conditions and in precipitation amounts over the catchment cause deviations from the average conditions. This implies that percolation to the free water reservoirs and hence groundwater flow takes place before the tension water reservoir is completely filled. The model allows for this to let a fraction of the infiltrated water percolate to the two free water storages. **When the tension water reservoir is full, all percolated water drains to the primary and supplemental free water storage in a ratio corresponding to their relative deficiencies.**

2.2.5 Groundwater flow

Baseflow to the river from groundwater depends on the contents of the two lower zone free water storages and two drainage constants expressed in fractions of the content per day. If the actual contents of the primary and supplemental free water zones are denoted by LZFPC and LZFSC respectively then the total base flow QBASE becomes, in accordance with the linear reservoir theory:

QBASE = LZFPC * LZPK + LZFSC * LZSK (7)

The drainage factors LZPK and LZSK can be determined from the recession part of the hydrograph by plotting that part of the hydrograph on semi-logarithmic paper (Fig. 6). In the lowest part of the recession curve only the slow base flow component is acting while in the higher stages both base flow components contribute.

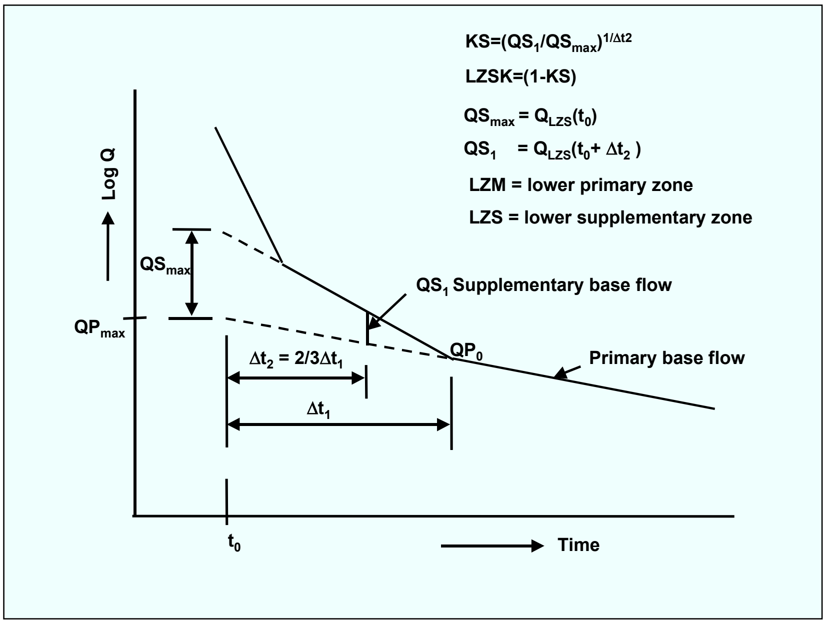

Figure 6: Principle of computation of lower zone recession coefficient

The drainage factor LZPK follows from:

$$K = (QP_{t0+\Delta t} / QP_{t0})^{1/\Delta t} \quad (8)$$

and

$$LZPK = 1 - K \quad (9)$$

where:

- K = recession coefficient of primary base flow for the time unit used
- Δt = number of time units, generally days
- $QP_{t0+\Delta t}$ = a discharge when recession is occurring at the primary base flow rate
- QP_{t0} = the discharge t time units later

If QP_{max} represents the maximum value of the primary base flow, then the maximum water content of the lower zone becomes:

$$LZFPM = QP_{max} / LZPK \quad (10)$$

and similarly the supplemental lower zone free water capacity is determined; at least this procedure provides first estimates of the lower zone free water capacities (Fig. 6).

The total base flow contributes completely or in part to the channel flow. A complete contribution occurs if subsurface discharge (i.e. discharge from the segment, which is not measured at the outlet) is absent. Otherwise a fraction of the total base flow represents the subsurface flow.

2.2.6 Actual evapotranspiration

Evaporation at a potential rate occurs from that fraction of the basin covered by streams, lakes and riparian vegetation. Evapotranspiration from the remaining part of the catchment is determined by the relative water contents of the tension water zones. **If *ED* is the potential evapotranspiration, then the actual evapotranspiration from the upper zone reads:**

$$E_1 = ED * (UZTWC / UZTWM) \quad (11)$$

i.e. the actual rate is a linear function of the relative upper zone water content. Where $E_1 < ED$ water is subtracted from the lower zone as a function of the lower zone tension water content relative to the tension water capacity:

$$E_2 = (ED - E_1) * LZTWC / (UZTWM + LZTWM) \quad (12)$$

If the evapotranspiration should occur at such a rate that the ratio of content to capacity of the free water reservoirs exceeds the relative tension reservoir content then water is transferred from free water to tension water such that the relative loadings balance. This correction is made for the upper and lower zone separately. However, a fraction *RSERV* of the lower zone free water storage is unavailable for transpiration purposes.

2.2.7 Impervious and temporary impervious areas

Besides runoff from the pervious area, the channel may be filled by rainwater from the impervious area. With respect to the size of the impervious area it is noted that in the Sacramento model a distinction is made between **permanent and temporary impervious areas** where temporary impervious areas are created when all tension water requirements are met, i.e. an increasing fraction of the catchment assumes impervious characteristics.

2.2.8 Routing of surface runoff

Before the runoff from the impervious areas, the overland- and interflow reach the channel, they may be transformed according to a unit hydrograph leading to an adapted time distribution of these flow rates.

Use can be made here of the Clark method, which is a combined time-area and storage routing method. The model requires the construction of a time-area diagram. For this isochrones are constructed representing points of equal travel time to the segment outlet, see Figure 7. The areas between successive isochrones is determined and subsequently properly scaled by the time of concentration Tc. The latter is defined as the time required to have the effect of rainfall fallen in the most remote part felt at the segment outlet. The time-area diagram can be thought of as the outflow from the segment if only translation and no deformation takes place of an instantaneous unit supply of rain over the entire segment. Subsequently, the time area diagram flow is routed through a linear reservoir, which characterises the effect of storage in the open drainage system of the segment. This reservoir is represented by the second parameter: the recession coefficient k. It is noted that the output from the reservoir represents the instantaneous unit hydrograph (IUH). This has to be transformed into say a 1-hour unit hydrograph, dependent on the chosen routing interval.

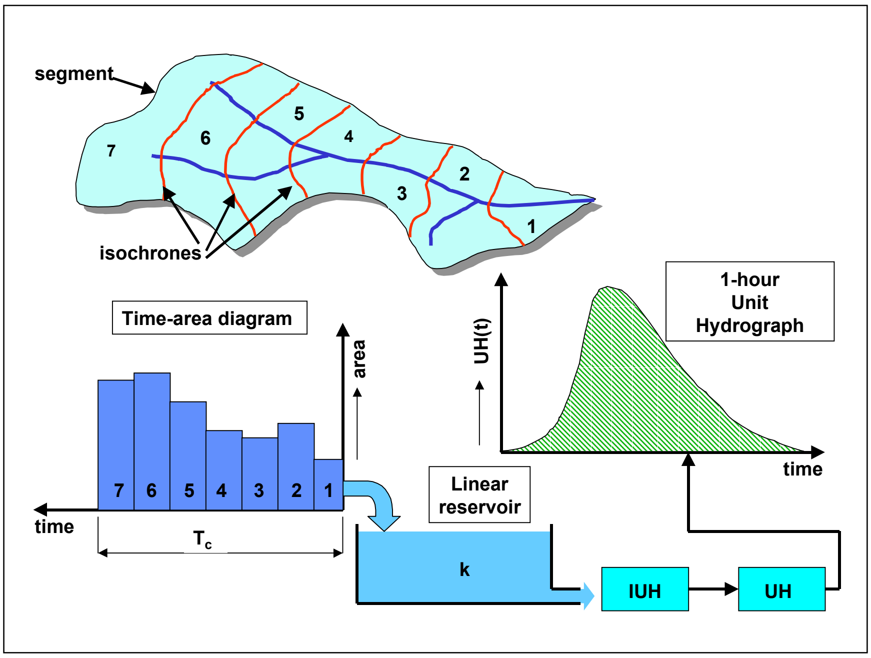

Figure 7: Principles of the Clark method for simulating surface runoff and interflow

The two parameters T_c and k can be obtained from observed rainfall and discharge hydrographs. The time of concentration is equal to the time interval between cessation of rainfall and the time the hydrograph has receded to its inflection point (see Figure 2). Alternatively it is determined from physical features of the segment as length and slope. A large number of empirical formulas are available which relate the time of concentration to topographical features of the basin. It is noted, though, that these formulas have generally only local validity. The best is to estimate the celerity from the flow velocities in the drainage system taking account of the following characteristics of celerity:

- If the rivulet remains inbank the celerity is about 1.5 to 1.7 times the cross-sectional flow velocity
- If the flow becomes overbank the above celerity has to be multiplied with the ratio of the drain width and the total width of the flow at the water surface (i.e. inclusive of the floodplain)

To the time required to travel through the drainage system one has to add the overland flow time.

The recession coefficient k is determined from the slope of recession part of the surface runoff hydrograph, similar to the procedure for groundwater.

2.3 The channel module

Contributions to the channel flow component are made by:

- runoff from impervious areas,
- overland flow from the pervious areas,
- interflow, and
- base flow (completely or in part).

The propagation and attenuation of the segment outflows in channel branches can be described by:

- Unit hydrograph technique
- Muskingum routing
- Structure and reservoir routing

Unit hydrograph technique

To propagate and attenuate riverflow through a channel reach for each channel branch a unit hydrograph can be defined. It describes how the inflow to the branch will be redistributed in time while travelling through the branch. Let the inflow to the branch be denoted by I_i and let the ordinates of the unit hydrograph be U_1, U_2, etc., with $\square U_i = 1$, then the outflow from the branch O_i becomes:

$$O_i = I_i \times U_1 \qquad (13)$$
$$O_{i+1} = I_i \times U_2 + I_{i+1} \times U_1$$
$$O_{i+2} = I_i \times U_3 + I_{i+1} \times U_2 + I_{i+2} \times U_1, \text{ etc.}$$

If e.g. the travel time through the reach is exactly 1 time interval and there is no attenuation then:

$U_1 = 0$, $U_2 = 1$.

This option provides a simple means to combine segment outflows entering the river at different locations, when the computational interval is too large for proper channel routing using the Muskingum approach. E.g. the travel time through a branch is 15 hours, but the computational interval is 1 day as rainfall data were only available as daily totals. Then within that day (24-15)/24x100% arrives and the rest the next day, so $U_1 = 0.375$, $U_2 = 0.625$. Routing with daily intervals is very acceptable when one is interested in 10-daily or monthly flow data and not in the finest details of the hydrograph.

Muskingum routing

The Muskingum procedure is based on the following routing equation:

$$O(t+\Delta t) = c_1 I(t) + c_2 I(t+\Delta t) + c_3 O(t) \qquad (14)$$

$$c_1 = \frac{\Delta t + 2Kx}{2K(1-x)+\Delta t} \qquad c_2 = \frac{\Delta t - 2Kx}{2K(1-x)+\Delta t} \qquad c_3 = \frac{2K(1-x)-\Delta t}{2K(1-x)+\Delta t}$$

where: I = Inflow to channel reach
O = Outflow from a channel reach

Since equation (14) is derived from S = K(xI+(1-x)O), where S = storage in the reach, it is observed that for x = 0 a simple linear reservoir concept follows: S = KO. With x = 0.5 there is no attenuation and the inflow is passed on through the end of the channel reach without any attenuation in time K.

The routing interval should be less than or equal to K as otherwise peaks will be missed at the downstream end of the reach. Often a value of Δt = ½ to ¼ of K is advised. However, taking Δt too small then c_2 becomes negative, which will lead to negative outflows when the inflow hydrograph suddenly rises. To avoid negative outflows the routing interval is conditioned by: $2Kx \le \Delta t \le K$.

Unfortunately, this leaves little freedom in the selection of Δt when x is close to 0.5. Note that when x=0.5 and Δt = K, it follows from (14) that c_1=1, c_2=0 and c_3=0; hence: $O(t+\Delta t) = O(t+K) = c_1 I(t)$, i.e the inflow is passed on to the outlet time K later, unaltered.

Flood wave celerity and attenuation changes drastically when the river reaches the flood plain. To cope with these changes a layered Muskingum approach can be used. The principle of the layered Muskingum procedure is displayed in Figure 8, in which the meaning of the various parameters is explained. By applying different sets of parameters for the inbank flow and overbank flow the reduction of the flood wave celerity in case of wide flood plains can be taken into account.

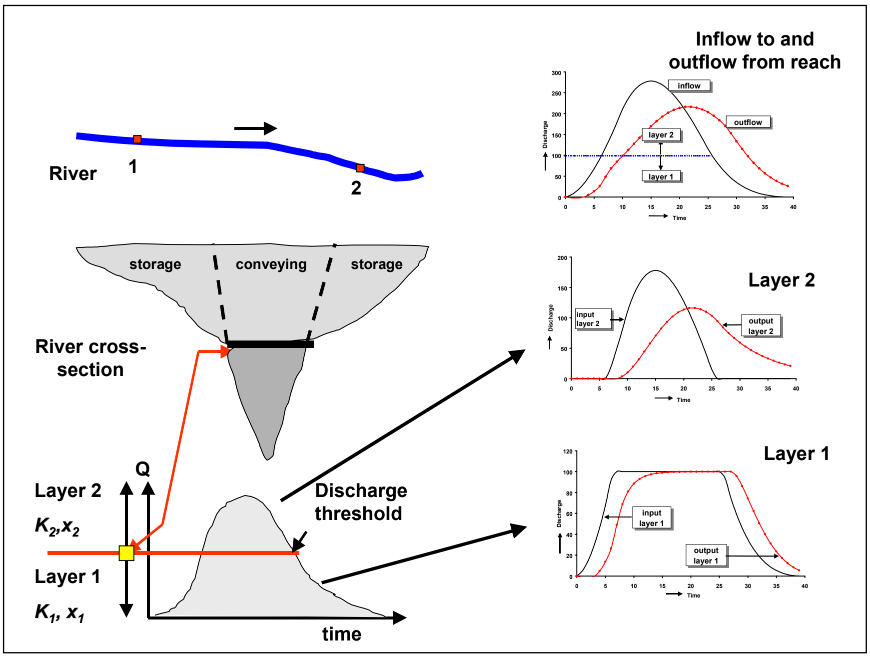

Figure 8: Principle of layered Muskingum approach

Structures and reservoirs

Some features may be present in the river which affect the shape of the hydrograph, like culverts and reservoirs:

- culvert
  A culvert limits the capacity of the river. Basically, it chops the peak of the hydrograph beyond the capacity of the culvert. In the model the shape of the hydrograph is altered such that upon passage of the floodwave the maximum downstream hydrograph value is kept at the culvert capacity until the entire volume in the upstream hydrograph above the capacity of the culvert has passed. It is noted that some old bridges may act also more or less like a culvert.
- reservoir
  The model includes a number of reservoir routing options where the flow is controlled by overflow (ogee and glory type) structures and underflow structures. For routing a third order Runge-Kutta scheme is used.

2.4 Estimation of segment parameters

2.4.1 Overview of parameters

The following groups of parameters can be distinguished for a particular segment:

Segment:
Segment area (km^2)

Direct runoff

PCTIM	Permanently impervious fraction of segment contiguous with stream channels
ADIMP	Additional impervious fraction when all tension water requirements are met
SARVA	Fraction of segment covered by streams, lakes and riparian vegetation

Upper soil moisture zone

UZTWM	Capacity of upper tension water zone (mm)
UZFWM	Capacity of upper free water zone (mm)
UZK	Upper zone lateral drainage rate (fraction of contents per day)

Percolation

ZPERC	Proportional increase in percolation from saturated to dry conditions in lower zone
REXP	Exponent in percolation equation, determining the rate at which percolation demand changes from dry to wet conditions

Lower zone

LZTWM	Capacity of lower zone tension water storage (mm)
LZFPM	Capacity of lower zone primary free water storage (mm)
LZFSM	Capacity of lower zone supplemental free water storage (mm)
LZPK	Drainage rate of lower zone primary free water storage (fraction of contents per day)
LZSK	Drainage rate of lower zone supplemental free water storage (fraction of contents per day)
PFREE	Fraction of percolated water, which drains directly to lower zone free water storages
RSERV	Fraction of lower zone free water storages which is unavailable for transpiration purposes
SIDE	Ratio of unobserved to observed baseflow
SSOUT	Fixed rate of discharge lost from the total channel flow (mm/□t)

Surface runoff

Unit hydrograph ordinates

Internal routing interval

PM	Time interval increment parameter
PT1	Lower rainfall threshold

Basically two procedures are available to get first estimates for the majority of the segment parameters:

- from observed rainfall and runoff records: this method is usually applied and works well provided that the model concepts are applicable and that reliable records are available for some time covering the majority of the range of flows

- from soil characteristics: this method is particularly suitable if no runoff records are available, i.e. for ungauged catchments.

With respect to gauged catchments the following grouping of parameters according to the method of estimation can be made:

1. Parameters computed and estimated from basin map solely:
   segment area and SARVA
2. Parameters estimated from observed rainfall and runoff records:
   readily: LZFPM, LZPK, LZFSM, LZSK, PCTIM
3. approximately: UZTWM, UZFWM, UZK, LZTWM, SSOUT and PFREE
   Parameters estimated from topographic maps and rainfall and runoff records:
   unit hydrograph ordinates obtained from Clark method
   Parameters to be obtained through trial runs:
4. ZPERC, REXP, SIDE, ADIMP, RSERV
5. Internal routing parameters, as per requirement:
   PM, PT1, PT2

In the next sub-sections guidelines are given for the determination and estimation of the segment parameters for gauged catchments.

2.4.2 Segment parameter estimation for gauged catchments.

The estimation of the segment parameters is presented according to their order of appearance in the previous sub-section. The sequence in which the estimation is done in practice is different from this order, for which reference is made to the end of the sub-section.

Segment:

Segment area

To allow a good comparison between the observed and simulated runoff from the basin, the segment area (km^2) should refer to the total segment area draining upstream of the gauging station,. Any difference between total segment area up to the main stream and the area upstream of the gauging station can be accommodated for in the channel routing part.

Direct runoff:

PCTIM

Permanently impervious fraction of the basin contiguous with stream channels. It can be determined from small storms after a significant period of dry weather. Then the volume of direct runoff (=observed runoff - baseflow) divided by the volume of rain gives the percentage impervious fraction of the basin. *PCTIM* should not be close to 1!

An example is given below.

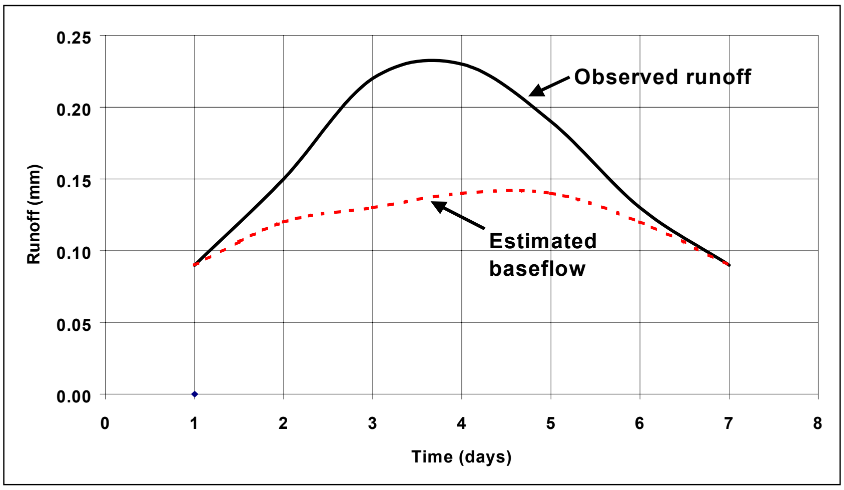

Figure 9: Calculation of PCTIM

ADIMP

Fraction of the basin, which becomes impervious as all tension water requirements are met. It can be estimated from small storms after a very wet period. As before, the volume of direct runoff divided by the volume of rain gives the total percentage of impervious area. The estimate for ADIMP follows from:

ADIMP = Total Percentage Impervious – PCTIM (15)

SARVA

Fraction of the basin covered by streams, lakes, and riparian vegetation, under normal circumstances. The SARVA area is considered to be the same as or less than PCTIM (see below). Detailed maps may be referred to in order to estimate the extent of paved areas, which drain directly to the streams so that differences between PCTIM and SARVA can be approximated. Generally, SARVA appears to range between 40% and 100% of the PCTIM value.

Upper soil moisture zone:

UZTWM - the upper tension storage capacity

The depth of water, which must be filled over non-impervious areas before any water becomes available for free water storage. Since upper zone tension water must be filled before any streamflow in excess of the impervious response can occur, its capacity can be approximated from hydrograph analysis. Following a dry period when evapotranspiration has depleted the upper soil moisture, the capacity of upper zone tension water can be estimated. That volume of rainfall, which is retained before runoff from the pervious fraction is visible, is identified as *UZTWM.* To that rainfall volume the losses to evaporation during the considered period should be added. All periods of rain following a dry period should be checked for estimation of this parameter. Generally the capacity of the upper zone tension will vary between 25 and 175 mm, depending on the soil type.

Following the logic of the Curve Number method, where the initial abstraction before rainfall becomes effective is estimated as 20% of the potential maximum retention, the *UZTWM* becomes:

UZTWM = 50.8(100/CN - 1) (mm) (15a)
CN-values range from 30 to about 90 for rural areas and are a function of:

soil type (soil texture and infiltration rate); hydrological soil groups A-D are distinguished
land use, type of land cover, treatment and hydrologic or drainage condition

It is also a function of antecedent moisture condition, for which the condition “dry” should be taken in view of the meaning of *UZTWM*. Based on this assumption *UZTWM* would vary between 120 and 6 mm, values which are in the range of those given above, particularly if one realises that the 20% of the potential maximum as initial abstraction is an average value. Reference is made to standard textbooks on hydrology for CN-values

UZFWM - the upper free water storage capacity

Upper zone free water represents that depth of water, which must be filled over the non-impervious portion of the basin in excess of *UZTWM* in order to maintain a wetting front at maximum potential. This volume provides the head function in the percolation equation and also establishes that volume of water, which is subject to interflow drainage. Generally its magnitude ranges from 10-100 mm. It is not generally feasible to derive the magnitude of the upper zone free water from direct observations, and successive computer runs are required in order to establish a valid depth.

However, if a rough estimate of UZK is available (see below), then a rough value of UZFWM can be obtained from the hydrograph at the time of the highest interflow, by reducing the flow value with primary and supplemental baseflow.

UZK - the upper zone lateral drainage rate

The upper zone lateral drainage rate is expressed as the ratio of the daily withdrawal to the available contents. Its range is roughly 0.18 to 1.0, with 0.40 generally serving as an effective initial estimate. Though basically, this factor is not capable of direct observation and must be determined by successive computer runs, Peck (1976) suggests the following approximate procedure. *UZK* is roughly related to the amount of time that interflow occurs following a period with major direct and surface runoff. A long period of interflow results in a small value for *UZK*. Assuming that interflow is observed during N consecutive days and that interflow becomes insignificant when it is reduced to less than 10% of its maximum value it follows:

$$(1 - UZK)^N = 0.10 \quad \text{or} \quad UZK = 1 - 0.1^{1/N} \qquad (16)$$

Values for UZK as a function of N can be read from Figure 10.

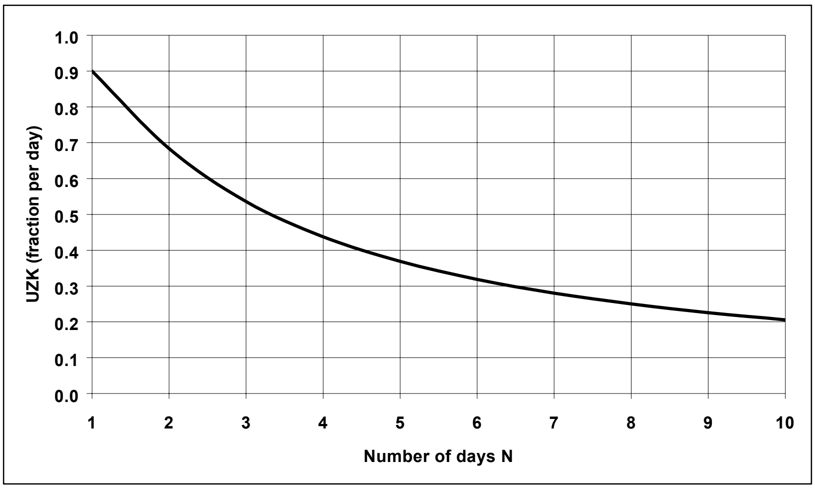

Figure 10:
UZK as function of number of days with significant interflow

Percolation

ZPERC

The proportional increase in percolation from saturated to dry condition is expressed by the term *ZPERC*. The value of *ZPERC* is best determined through computer trials. The initial estimate can be derived by sequentially running one or two months containing significant hydrograph response following a dry period. The value of *ZPERC* should be initially established so that a reasonable determination of the initial run-off conditions is possible.

Amstrong (1978) provides a procedure to derive ZPERC from the lower zone tension and free water reservoir capacities and drainage rates, using equations (3) and (4). The maximum percolation takes place when the upper zones are full and the lower zones are empty. Assuming that the maximum daily percolation will be the maximum contents of the lower zones, from equation (4) it follows for ZPERC:

$$\mathrm{ZPERC} = \frac{\mathrm{LZTWM} + \mathrm{LZFPM} + \mathrm{LZFSM} - \mathrm{PBASE}}{\mathrm{PBASE}} \qquad (17)$$

If data would be available on maximum percolation rates ZPERC can be estimated using equation (4). Values for ZPERC ranging from 5 to 80 have been used.

REXP

The exponent in the percolation equation which determines the rate at which percolation demand changes from the dry condition, *(ZPERC + 1)*PBASE*, to the wet condition, *PBASE*. Fig. 5 illustrates how different values of the exponent affect the infiltration rate. It is recommended that an initial estimate of this exponent is made from the same record which is used in determining an initial estimate of *ZPERC*. The interaction between *PBASE, ZPERC* and *REXP* may require a shift of all three terms whenever it becomes clear that a single term should be changed. Visualising the percolation curve generated by these three terms helps to ascertain the necessary changes. The observed range of *REXP* is usually between 1.0 and 3.0. Generally a value of about 1.8 is an effective starting condition. Values for *REXP* for different soils are given by Amstrong (1978) and are presented in Table 1.

Soil classification	REXP
Sand	1.0
Sandy loam	1.5
Loam	2.0
Silty loam	3.0
Clay, silt	4.0

Table 1: Perlocation exponent REXP for different soil types

Lower zone:

LZTWM - lower zone tension water capacity (mm)

This volume is one of the most difficult values to determine effectively. Inasmuch as carryover moisture in this storage may exist for a period of many years, its total capacity may not be readily discernible from available records. If a drought condition during the period of record in the basin or in the area being studied has been sufficient to seriously affect the transpiration process of deep rooted plants, then the period of record is usually sufficient to determine the maximum storage value of lower zone tension water. Often, however, field data is not adequate for this purpose. As a result, unless great care is taken, the depth of lower zone tension water storage may inadvertently be set near the maximum deficit experienced during the period of record rather than the actual capacity of the zone. It has been noted that the plant growth of an area is a relatively effective indicator of the capacity of the lower zone tension water zone. In heavily forested regions of deep-rooted conifers, this zone may be approximately 600 mm in magnitude. In areas of deep-rooted perennial grasses this depth is more likely to be close to 150 mm. Where vegetation is composed primarily of relatively shallow-rooted trees and grasses, this depth may be as little as 75 mm. It should be realised that this zone represents that volume of water, which will be tapped by existing plants during dry periods.

An approximate procedure to estimate LZTWM from a water balance analysis is presented by Peck (1976). For this a period is selected with direct and/or surface runoff following an extended dry spell. The selected period is bounded by the times t_1 and t_2. At both times t_1 and t_2 only baseflow occurs. The start t1 is selected immediately prior to the occurrence of direct/surface runoff and t_2 immediately following a period of interflow. The times t_1 and t_2 can best be selected from a semi-log plot of the runoff, see Figure 11.

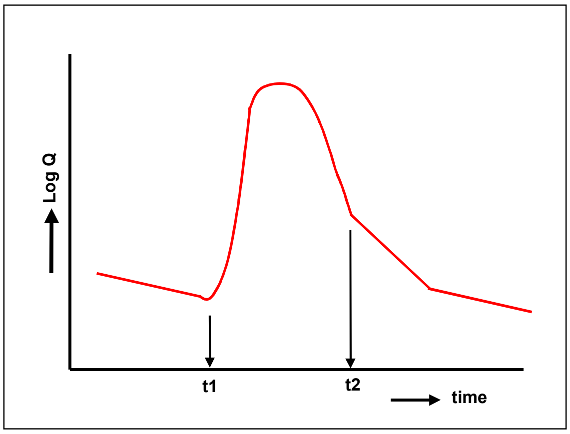

Figure 11:
Selection of period for LZTWM estimation

Assuming that UZTW is full and UZFWC is empty at times t_1 and t_2 the water balance for the period t1- t_2 then reads:

$$P - R - E - \Delta LZFPC - \Delta LZFSC = \Delta LZTWC \qquad (18)$$

Where: P = precipitation from t_1 to t_2 (mm)
R = total runoff from t_1 to t_2 (mm)
E = segment evaporation (mm); this amount would small for most wet period and may be neglected
ΔLZFPC = change in storage in LZ primary free water reservoir from t1 to t2 (mm)
ΔLZFSC = change in storage in LZ supplemental free water reservoir from t1 to t2 (mm)
ΔLZTWC = change in the lower zone tension water (mm)

ΔLZTWC is a lower limit of LZTWM since:

- The lower zone tension water reservoir may not have been fully empty at t_1
- The lower zone tension water reservoir may not have been completely filled at t_2

Hence some 10 to 20% (or more) may be added to the value obtained through (18). If such ideal cases as assumed above cannot be found, water balances for periods of 3 to 4 months may be considered.

In equation (18) ΔLZFPC and ΔLZFSC are computed as follows:

$$\Delta LZFPC = LZFPC(t_2) - LZFPC(t_1), \text{ where } LZFPC(t) = QP(t)/LZPK \quad (19)$$

$$\Delta LZFSC = LZFSC(t_2) - LZFSC(t_1), \text{ where } LZFSC(t) = QS(t)/LZSK \quad (20)$$

The primary baseflows QP at times t_1 and t_2 are estimated by extrapolation from other periods. Let the discharges at t_1 and t_2 be denoted by Q1 and Q2, then the supplemental baseflows follow from:

$$QS(t_1) = Q1 - QP(t_1) \quad \text{and} \quad QS(t_2) = Q2 - QP(t_2) \quad (21)$$

LZFPM - lower zone primary free water storage

The maximum capacity of the primary lower zone free water, which is subject to drainage at the rate expressed by *LZPK*. The value of the lower zone primary free water maximum can be approximated from hydrograph analysis. For this the primary base flow, obtained from a semi-log plot of the lower end of the recession curve, is extended backward to the occurrence of a peak flow. Assuming that the primary free water reservoir is completely filled then, so that it outflow is at maximum (*QPmax*), its value is determined from equation (10). The effectiveness of this computation in determining the maximum capacity is dependent upon the degree to which the observed hydrograph provides a representation of the maximum primary baseflow. If only a portion of the groundwater discharge is observable in the stream channel, the estimated capacity based upon surface flows must be increased to include the non-channel components by applying the term *SIDE* (See below).

LZFSM - lower zone supplemental free water storage

The maximum capacity of the lower zone supplemental free water reservoir, which is subject to drainage at the rate expressed by *LZSK*. A lower limit of the lower zone free water supplemental maximum can be approximated from hydrograph analysis. Fig. 6 illustrates the computation of the lower zone free water supplemental maximum. Note that first the primary base flow has to be identified and corrected for, see also equation (21). The effectiveness of this computation in determining the maximum capacity is dependent upon the degree to which the observed hydrograph provides a representation of the maximum baseflow capability of the basin. If only a portion of the groundwater discharge is observable in the stream channel, the estimated capacity based upon surface flows must be increased to include the non-channel components by applying the term *SIDE* (See below).

LZPK - lateral drainage of the lower zone primary free water reservoir.

Lateral drainage rate of the lower zone primary free water reservoir expressed as a fraction of the contents per day. The coefficient is determined from the primary base flow recession curve. Selecting flow values from this curve at some time interval Δt apart provides with the help of equations (8) and (9) the required estimate, see also Figure 6.

LZSK - lateral drainage of the lower zone supplemental free water reservoir

Lateral drainage rate of the lower zone supplemental free water reservoir, expressed as a fraction of the contents per day. Its computation is outlined in Figure 6. The procedure is similar to that of LZPK, with the exception that the flow values have to be corrected for the primary base flow.

PFREE

The fraction of the percolated water, which is transmitted directly to the lower zone free water aquifers. Its magnitude cannot generally be determined from hydrograph analysis. An initial value of 0.20 is suggested. Values will range between 0 and 0.50. The analysis of early season baseflow allows an effective determination of *PFREE*. The relative importance of PFREE can be determined from storms following long dry spells that produce runoff (UZTW completely filled). If the hydrograph returns to approximately the same base flow as before then little filling of the lower zone free water reservoirs did take place and hence the PFREE-value can be rated small, 0 to 0.2. If, on the contrary, the base flow has increased significantly a PFREE-value as high as 0.5 may be applicable.

RSERV

Fraction of the lower zone free water, which is unavailable for transpiration purposes. Generally this value is between zero and 0.40 with 0.30 being the most common value. This factor has very low sensitivity.

SIDE

Represents that portion of base flow, which is not observed in the stream channel. When the soil is saturated, if percolation takes place at a rate, which is greater than the observable baseflow, the need for additional soil moisture drainage becomes manifest. *SIDE* is the ratio of the unobserved to the observed portion of base flow. When the saturated soils do not drain to the surface channel, *SIDE* allows the correct definition of *PBASE*, in order that the saturated percolation rate may be achieved. In an area where all drainage from baseflow aquifers reaches surface channels, *SIDE* will be zero. Zero or near zero values occur in a large proportion of basins. However, in areas subject to extreme subsurface drainage losses, *SIDE* may be as high as 5.0. It is conceivable that in some areas the value of *SIDE* may be even higher.

SSOUT

The sub-surface outflow along the stream channel, which must be provided by the stream before water is available for surface discharge. This volume expressed in mm/time interval is generally near zero. It is recommended that the value of zero be utilised, and *SSOUT* is applied only if the log *Q* vs. time plot requires a constant addition in order to achieve a valid recession characteristic. If constant volumes of flow are added to observed stream flow, the slope of the discharge plot will be altered. That value, which is required to linearize the primary recession, is the appropriate value of *SSOUT*. It should be realised that where *SSOUT* is required, an effective determination of lower zone free water storages and discharge rates will require inclusion of the *SSOUT* value (mm/Δt)

Surface runoff

Unit hydrograph ordinates for the routing of flow from the impervious and pervious surfaces as well as interflow towards the segment outlet can be obtained through standard unit hydrograph procedures. It requires the selection of rainfall events (corrected for losses) with their resulting flood hydrographs (corrected for base flow). Note that for each event the net rainfall amount should match with the surface runoff and interflow amount. Various procedures are available to arrive at a unit hydrograph. If the rainfall intensity during the storm varies, multiple linear regression and discrete convolution techniques may be applied. The regression technique is readily available in spreadsheet software. The resulting unit hydrographs generally will show some irregularities and hence requires some smoothing afterwards. Unit hydrographs from various storms may appropriately be averaged to arrive at a representative unit hydrograph for the segment.

Another option is to use the Clark method. The principle of the Clark method was dealt with in Sub-section 2.2.8. First requirement is the derivation of a time-area diagram. If a Digital Elevation Model (DEM) is available from a catchment with appropriate software automatic calculation of the time-area diagram is possible. In the absence from a DEM the time-area diagram is derived from a basin map. By estimating travel times to the basin outlet (from river and terrain slopes, assumed roughness and flow depth) isochrones can be determined. The areas between successive isochrones is determined leading to a first estimate of the time-area diagram. The total time base of the time-area diagram should be the concentration time T_c, but due to inaccurate assessment of celerities in the basin it may differ from that. Therefore, the time base of the time-area diagram is scaled by a more appropriate estimate of T_c. An estimate for T_c may be obtained as the time lapse between the cessation of rainfall and the occurrence of recession on the falling limb of the hydrograph of surface runoff. The time base of the time-area diagram is scaled by this time lapse. Alternatively, the concentration time is estimated from an empirical formula applicable to the region. E.g. for a number of small catchments in the Indus basin the following equation applies:

$$T_c = \frac{1}{119}\frac{L}{\sqrt{S}} \tag{22}$$

where: T_c = concentration time (hrs)
L = length of river (km)
S = slope of main river

The units of the time-area diagram (km^2) are converted into m^3/s by multiplication with 0.278/Δt, with Δt in hours. Subsequently, the time-area diagram is routed through a linear reservoir, with reservoir coefficient k, estimated from the slope of the recession curve of the surface water hydrograph. The routing is carried out by the following equation:

$$\begin{aligned} &O_{i+1} = c_1 I_{av} + c_2 O_i \\ &\text{with}: I_{av} = \frac{1}{2}(I_i + I_{i+1});\ c_1 = \frac{\Delta t}{k + \Delta t/2};\ c_2 = \frac{k - \Delta t/2}{k + \Delta t/2};\ c_1 + c_2 = 1 \end{aligned} \tag{23}$$

where: I_{av} = average inflow during Δt (input is in form of histogram)
O = outflow from the linear reservoir

The outflow from the reservoir is the Instantaneous Unit Hydrograph (IUH) for the basin, which has to be transformed by averaging or S-curve technique into the Unit Hydrograph resulting from a rainfall of duration equal to the routing interval.

Internal routing interval

PM Time interval increment parameter
PT_1 Lower rainfall threshold
PT_2 Upper rainfall threshold

In case the time step used in the model is larger than 1 hour, the model simulates the redistribution of water between the various reservoirs with a time step, which is smaller than the time interval of the basic data. Particularly for the infiltration process this effect could be important. Also the rainfall will be lumped to that smaller interval. The number of increments in the time interval is derived from:

$$N_{\Delta t} = 1 + PM * (UZFWC * F + P_{eff}) \quad (24)$$

where:

$$F = 1 \qquad \text{for } P_{eff} < PT_1 \quad (25)$$

$$F = 1/2\ P_{eff}\ /\ PT_2 \qquad \text{for } PT_1 \leq P_{eff} \leq PT_2 \quad (26)$$

$$F = 1 - 1/2PT_2\ /\ P_{eff} \qquad \text{for } P_{eff} > PT_2 \quad (27)$$

The most important parameter is seen to be PM. Taking a very small value for PM (say *PM = 0.01*), then $N_{\Delta t}$ remains approximately1. If e.g. *PM = 0.1* then $N_{\Delta t}$ becomes substantially larger than 1. To limit the increase of $N_{\Delta t}$ a low value for PT_1 is to be chosen in combination with a large value of PT_2, which will reduce the value of *F*.

2.4.3 Sequence of parameter estimation

From the presentation above it will be clear that certain parameters should be estimated before other can be assessed. The following sequence is recommended of which the first three steps are mandatory:

1. Segment area
2. Lower zone primary free water parameters LZPK and LZFPM
3. Lower zone supplemental free water parameters LZSK and LZFSM
4. Impervious fraction PCTIM
5. Upper zone parameters UZTWM, UZK and UZFWM
6. Lower zone tension capacity LZTWM
7. Percolation parameters ZPERC and REXP
8. Remaining parameters

2.4.4 Linear reservoirs

An essential feature of the Sacramento model is that the free water reservoirs are considered as linear reservoirs, i.e. there is a linear relation between the reservoir storage S and the outflow Q:

$$S = kQ \quad (28)$$

If the recharge is indicated by *I*, the continuity equation for the linear reservoir reads:

$$dS/dt = I - Q \quad (29)$$

Eliminating S from above equations results in a linear first order differential equation in *Q*:

$$\frac{dQ}{dt}+\frac{1}{k}Q-\frac{1}{k}I=0 \tag{30}$$

With I constant and at t = t0 Qt = Qt0 the solution to (30) reads:

$$Q_t = I\left(1-\exp\left(-\frac{(t-t_0)}{k}\right)\right)+Q_{t_0}\exp\left(-\frac{(t-t_0)}{k}\right) \text{for} t \geq t_0 \tag{31}$$

When there is no recharge to the reservoir (*I = 0*) equation (31) reduces to:

$$Q_t = Q_{t_0}\exp\left(-\frac{(t-t_0)}{k}\right) \tag{32}$$

This equation can be compared with (8), using the same notation:

$$Q_t = Q_{t_0}K^{(t-t_0)} \tag{33}$$

Hence:

$$K = \exp\left(-\frac{1}{k}\right) \text{or} :k = -\frac{1}{\ln K} \tag{34}$$

Expressing time in days, then the amount of water released from the reservoir in 1day amounts according to equation (28):

$$S_0 - S_1 = kQ_0 - kQ_1 = kQ_0\left(1-\exp\left(-\frac{1}{k}\right)\right) = S_0(1-K) \tag{35}$$

This is seen to match with e.g. the equations for the lower zone primary free water reservoir, where:

S_0 = LZFPC   and   1-K = LZPK   (36)

Equation (34) provides a means to express the lower zone free water parameters in terms of dimensions and physical properties of aquifers. Consider the phreatic aquifer shown in Figure 12, which has the following dimensions and properties:

- The width of the aquifer perpendicular to the channel is L
- The water table at the divides is h_0 above the drainage base
- The specific aquifer yield is μ
- The aquifer transmissivity is T.

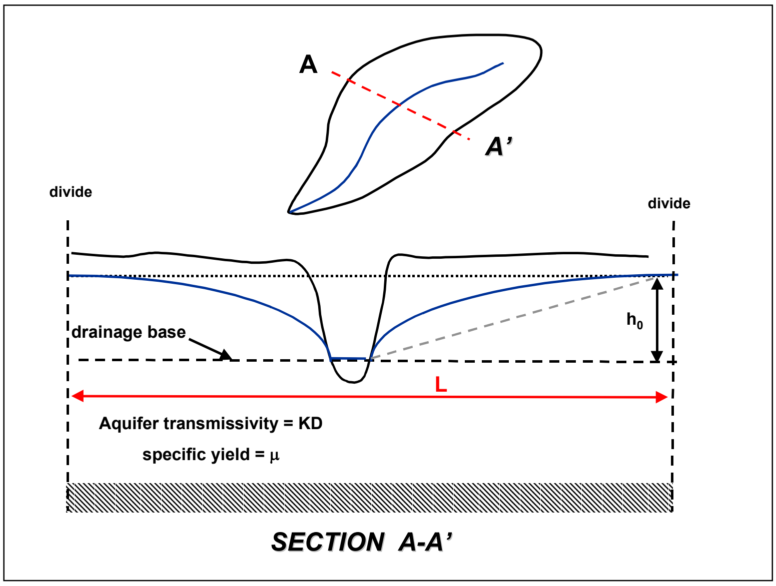

Figure 12: Schematic cross section through basin aquifer

The amount of water stored above the drainage base per unit length of channel available for drainage is:

$S = \mu c_1 L h_0$ with $\frac{1}{2} < c1 < 1$

The discharge to the channel per unit length of channel according to Darcy with the Dupuit assumption

$Q = -2T dh/dx = 2Tc_2h_0/(L/2)$ with $c_2 > 1$

Combining the above two equations by eliminating h_0 and bringing it in the form of the linear storage discharge relation (28):

$$S = \frac{\mu L^2}{4cT} Q \text{ with : } c = \frac{c_2}{c_1} > 1$$

.

Hence for the reservoir coefficient k in (28) it follows:

$$k = \frac{\mu L^2}{4cT} \qquad (37)$$

The reservoir coefficient k is seen to be proportional to the square of the aquifer width and inversely proportional to T, which is logical as k is a measure for the reside-time of the percolated water in the groundwater zone. The value of c varies between 2 and 2.5 dependent on the shape of the water table. For the parameters K and LZPK for the lower zone primary free water storage it then follows:

$$K = \exp(-\frac{4cT}{\mu L^2}) \text{and} : LZPK = 1 - \exp(-\frac{4cT}{\mu L^2}) \qquad (38)$$

A similar story applies for the lower zone free supplemental reservoir, which can be viewed as representing the drainage from the shallower based denser network of the smaller channels, see Figure 13. Since its main difference is with the aquifer width L, which is much smaller than for the deeper based primary channel network, its reservoir coefficient will be smaller than of the primary free water storage and consequently LZSK >> LZPK.

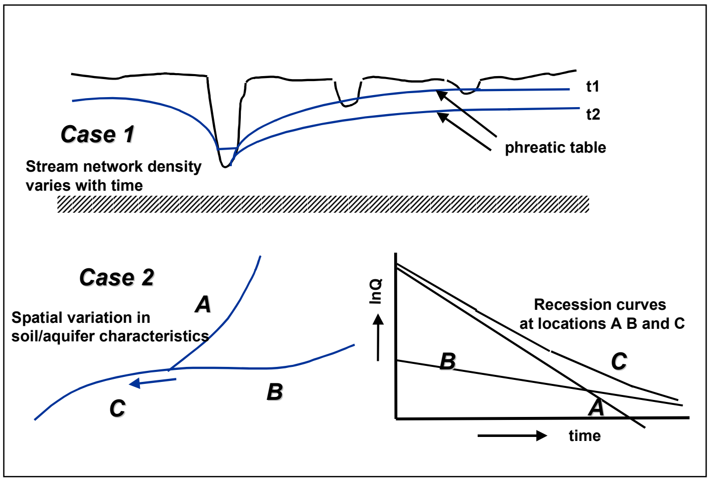

Figure 13:
Cases of multiple exponential decay of recession curve

Note that similar differences in a basin between fast and slow draining aquifers if different soils are present leading to different transmissivities.

Note also that from equation (33) it follows for $t - t_0 = 1$ that $K = Q_1/Q_0$. Hence, by deriving this ratio for the recession part of the hydrograph, the parameter K can be obtained from the lowest part of the recession curve where the ratio becomes constant.

2.5 Required input

The input required to run the model for simulation of rainfall-runoff process in a segment is presented in Figure 14, which shows the HYMOS screen for running the Sacramento model.

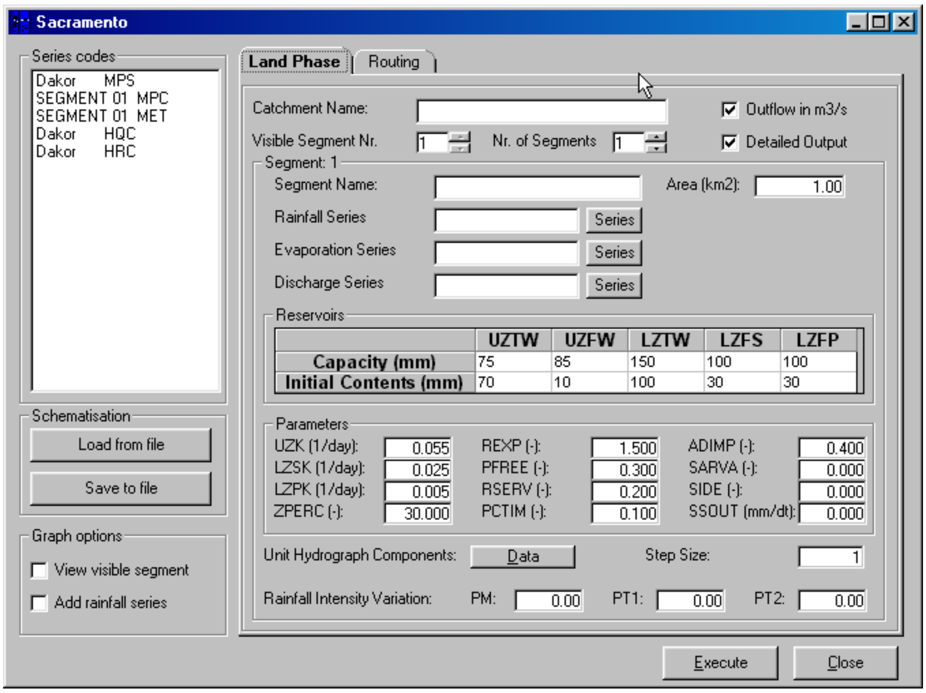

Figure 14: Input screen for running Sacramento rainfall-runoff model in HYMOS

To run the channel routing module the routing parameters as presented in Section 2.3 have to be entered for each distinguished branch. For each branch it has to be specified which hydrograph has to be routed to the next node, which may be:

- Segment outflow from one or more segments, draining at the upstream channel node
- Outflow from one or more upstream channel branches
- Hydrograph presented by the user, e.g. the outflow from a reservoir

3. Case studies

A few case studies will be considered in the course. The data required for cases are comprised in the KHEDA database. The availability of the data in this database is presented in the Annex 1: Data availability in KHEDA catchment. The cases comprise the following.

Case 1
A worked out example has been prepared for a part of the KHEDA catchment. The selected basin is located upstream of the stream gauging station Dakor: ‘Dakor basin’. The case study is described in the Annex 2: ‘Case study 1, Rainfall-runoff simulation for Dakor basin’. The document outlines the procedure how to prepare the basic data required for the simulation and how to estimate the model parameters.

Case 2
The second case study deals with Bilodra catchment of which Dakor segment is a smaller part. The course participants are requested to carry out this case along the lines presented for Dakor basin. The Bilodra catchment is taken as one. Necessary data have been included in the KHEDA database

Case 3
Case study deals with Bilodra catchment similar to Case 2, with segmentation of the basin in two parts: Dakor basin and Bilodra-sub basin (i.e. Bilodra excluding Dakor basin).

Case 4
Study area concerns Watrak river with main emphasis on running the Sacramento model for hourly data and routing the flow through the river.

Finally, reference is made to a few papers presented on the use of the Sacramento model for basins in India, carried out by IMD: S.D.S. Abbi, et. al., which are included in Annex 3. The papers show reliable performance of the model for the selected study basins.

3.1 Case study 1:Rainfall-runoff simulation for Dakor basin

Basin layout and available data

Case study 1 is carried out for Dakor catchment, see Figure 1, located in the south-eastern part of the basin indicated in the database as KHEDA catchment, see Figure 2.

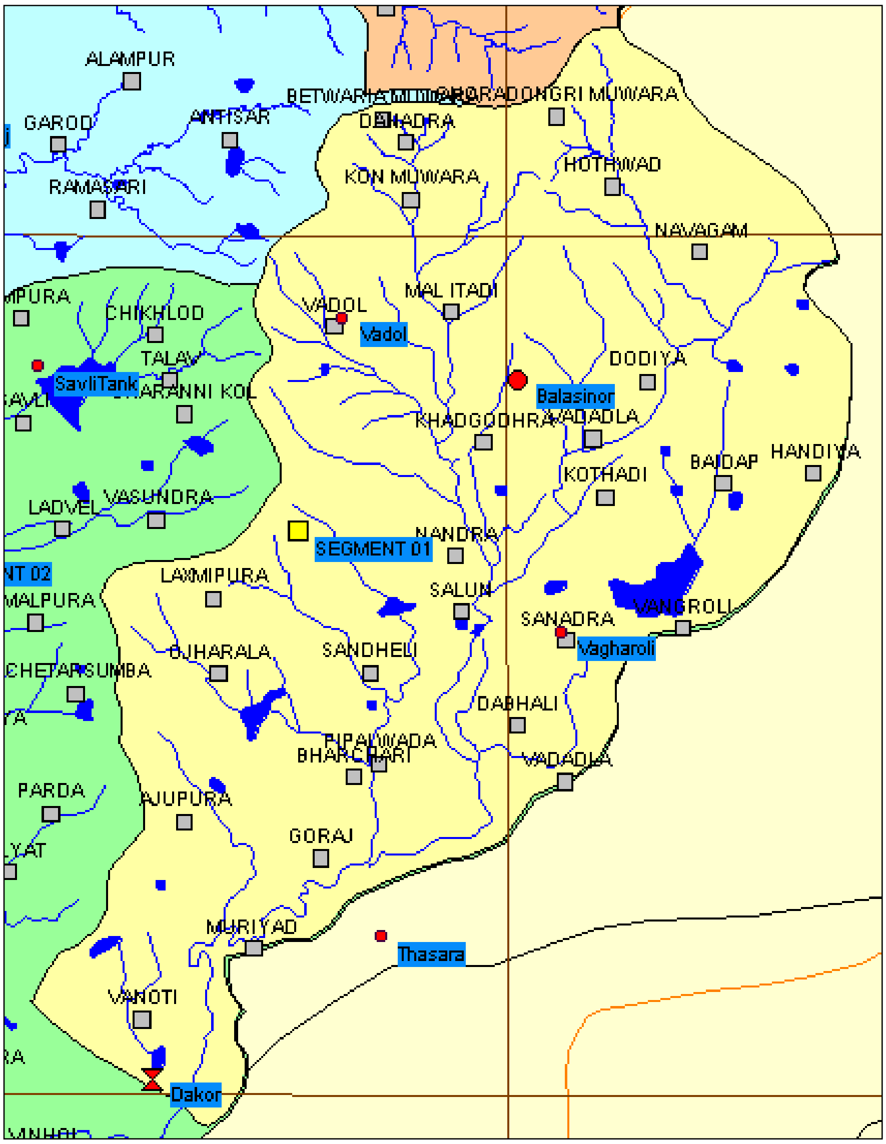

Figure 1: Layout of Dakor catchment

The basin measures 430.59 km^2 upstream of gauging station Dakor. The length of the river is 53 km and the slope is approximately 10^{-3}. At the measuring site at Dakor the river is about 60 m wide. The river bed is at 45 m +MSL. From the basin map it is observed that some storage tanks are present in the area. The area contains sandy soils, which dry out quickly.

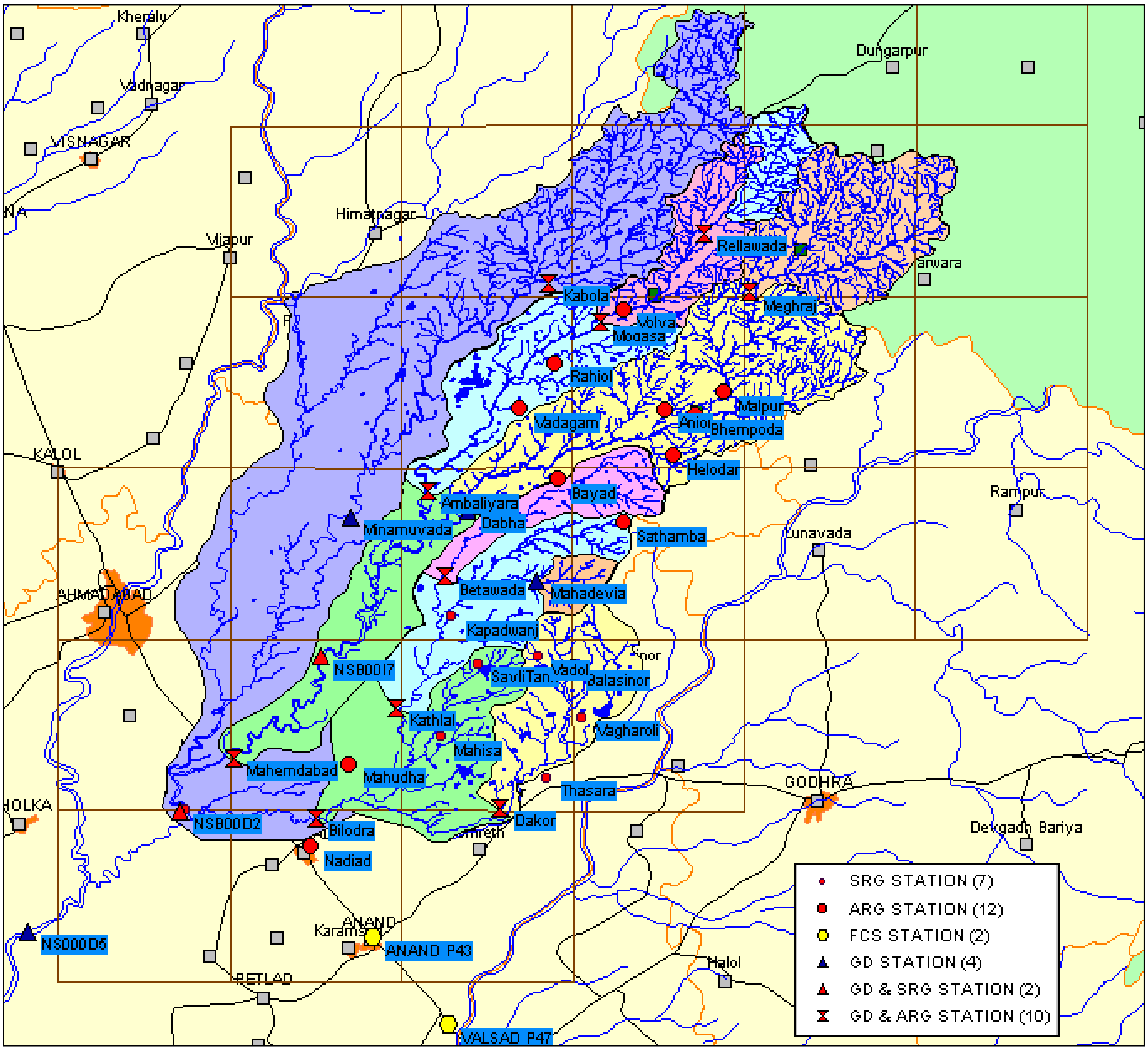

Figure 2: **Map showing the location of the Dakor basin in the KHEDA catchment**

In and around Dakor basin the following stations are of importance:

Rainfall: Vadol
Balasinor
Savli Tank
Mahisa
Vagharoli
Thasara
Dakor

Evaporation: Anand
Valsad

Streamflow: Dakor

Daily rainfall data is available for quite a number of years for the above-mentioned stations, see Annex 1. Hourly rainfall data, however, is lacking. Also a long record of pan evaporation data is available from two stations a little south of the basin, but which are considered representative for the basin. Hourly water level data is available for a large number of monsoon seasons. Prior to and after the monsoon no water level records are available as the river runs generally dry. During the monsoon season four times per day flow measurements are being carried out.

3.2 Objective

The objective of this case study is to demonstrate the development of a basin rainfall-runoff model based on Sacramento model available in HYMOS as a tool for creation of long series of runoff based on climatic data. Emphasis will be on the steps involved in model calibration and verification. Though the final acceptable result may involve a number of trials, this number can be limited if the initial estimates for the parameter values are carefully made. One should also get an indication of the possible range of the parameters for the basin under study.
The model will be developed using daily data on rainfall, evaporation and runoff. In this case we solely concentrate on segment rainfall-runoff simulation. River routing will not be considered as the interval of one day is too large for meaningful routing in such a small basin. For that hourly data should have been present for rainfall. For water resources analysis routing with an interval of one day will be sufficient.

3.3 Basin reconnaissance and input data preparation

After having collected and studied the topographic, geologic, soils and land use maps of the area as well as the characteristics of the hydraulic infrastructure it is imperative that a field visit precedes the model development. Based on differences is drainage characteristics, it may be decided to subdivide the basin in segments. The question on sub-division comes again when dealing with the spatial variability of the rainfall. How far sub-division should take place depends basically on the objective of the study in relation spatial variability. Segment areas in practice vary from a few hundred square kilometers to a few thousand. For water resources assessment studies where an exact reproduction of the shape of the hydrograph is not of importance segments will generally not be small; matching with the locations where flow data are required then also plays a role. Furthermore, practicalities as the availability of a gauging station with calibration data matters. The basin itself acts diffusive and smoothes the differences. Here it assumed that the Dakor catchment is sufficiently homogeneous to be covered by one segment.

Next the input and calibration data are being prepared including catchment rainfall data, potential evapotranspiration and runoff. It is noted here that in view of the objective of the course, being familiarisation with the Sacramento model, **data validation** is not given the attention it deserves **but should be given due attention in actual model development**. Completely erroneous models may result from poorly validated data.

For calibration and verification purposes representative periods have to be selected, which include the full gamma of flows. For the case study the year 1994 will be considered for calibration purposes. Flows have been very large that year and also a distinct recession curve is available for parameter estimation.

Catchment rainfall series 1994

All rainfall stations mentioned in Chapter 1were considered for calculation of the areal rainfall. An example of the variability of the point rainfall data on a daily basis is illustrated in Figure 3. The Figure also shows that occasionally day shifts in the rainfall data seem to be present. Such errors may deteriorate the quality of the catchment rainfall. Though in this area it is hard to say whether such errors are present as the correlation distance of rainfall events here is are rather small, careful analysis of the daily record casts doubt on the time of occurrence of the rainfall events as reported. Another impression of the spatial variability of the data is obtained from the annual totals as listed for the years 1993 and 1994 in Table 1

Station	Annual Rainfall (mm) 1993	Annual Rainfall (mm) 1994	Thiessen weights
Vadol	590	1317	0.17
Balasinor	574	1485	0.33
Savli Tank	837	1193	0.02
Mahisa	700	775	0.02
Vagharoli	924	1577	0.24
Thasara	991	1775	0.14
Dakor	672	1252	0.08

Table 1: Annual rainfall of years 1993 and 1994 and Thiessen weights for areal rainfall computation

From the Table it is observed that the spatial variability in the rainfall amounts even at short distances is rather large. The low annual value for Mahisa in 1994 compared to its neighbours is mainly due to the fact that an extremely large rainfall, which occurred in the region, was not available in the Mahisa record (erroneously or not). From the Table one can also see that rainfall totals from one year to another may vary considerably.

Thiessen method has been applied to compute the daily areal rainfall in the Dakor basin, see also Figure 4 and Table 1, where the station weights are presented. It is observed that the contributions of Savli Tank and Mahisa in the areal total for the Dakor basin are

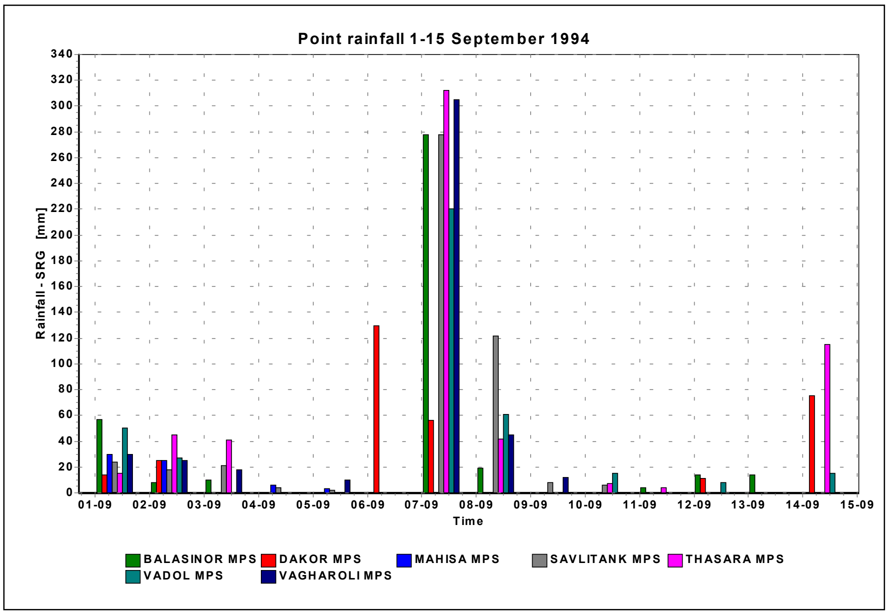

Figure 3: Example of daily point rainfall data at the selected stations

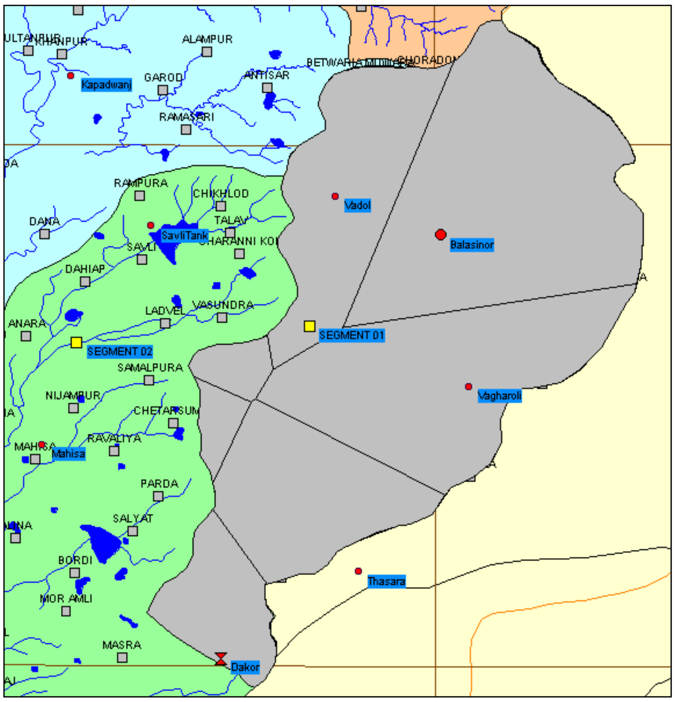

Figure 4:
Thiessen polygon for Dakor basin rainfall

small, hence the doubts on the Mahisa record for 1994 will not greatly affect the computed areal average.

The resulting daily average rainfall for the year 1994 is presented in Figure 5. It may be observed that the rainfall occurs from mid June till mid September only. The annual total amounts 1483 mm.

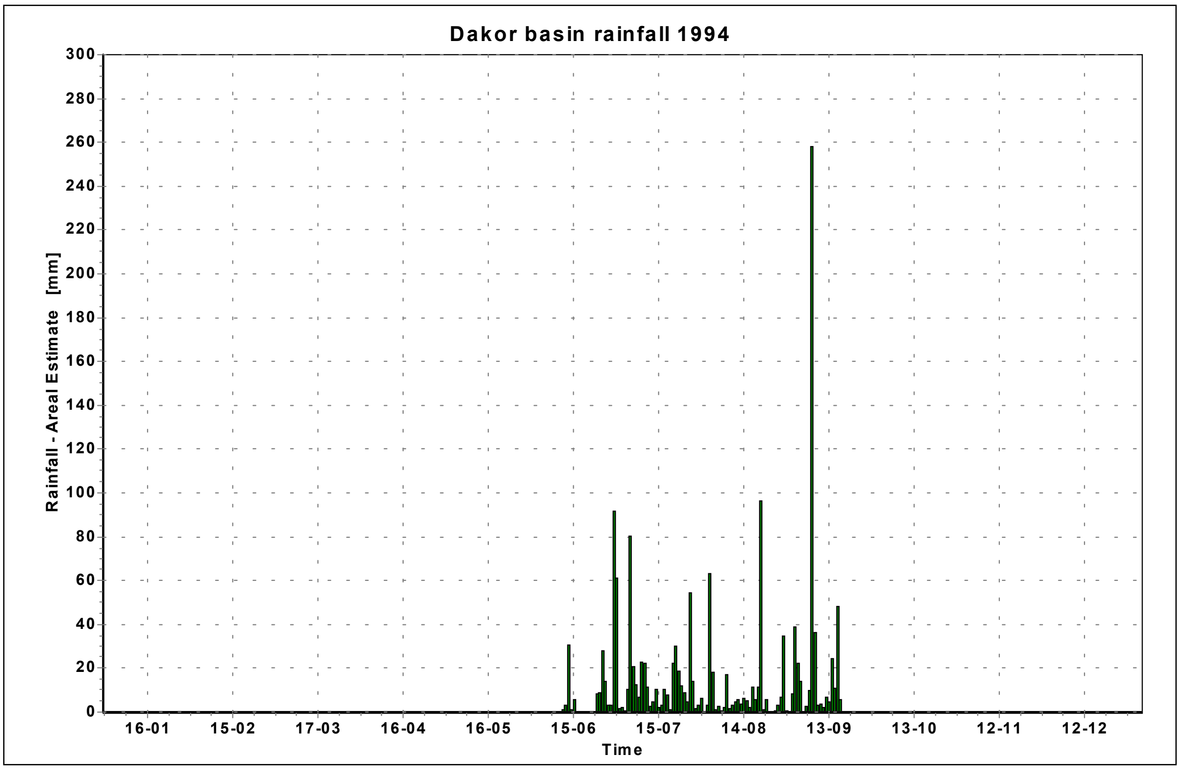

Figure 5: **Daily rainfall in Dakor basin for the year 1994**

Potential evapotranspiration series 1994

The potential evapotranspiration in Dakor basin is derived from the pan-evaporation records available from the stations Anand and Valsad. To transform pan evaporation into potential evapotranspiration generally pan coefficients ranging from 0.6 to 0.8 are being applied. Here, an average value of 0.7 is used. The variation of the potential evapotranspiration through the year is presented in Figure 6. Make sure that the evapotranspiration series does not include missing data. The annual total potential evapotranspiration for 1994 amounts 1483 mm, which is coincidentally exactly equal to the computed basin average rainfall. It is observed that the potential evapotranspiration during the monsoon season drops to about 2 mm/day in July and August, with an average of 2.9 mm/day from June till September.

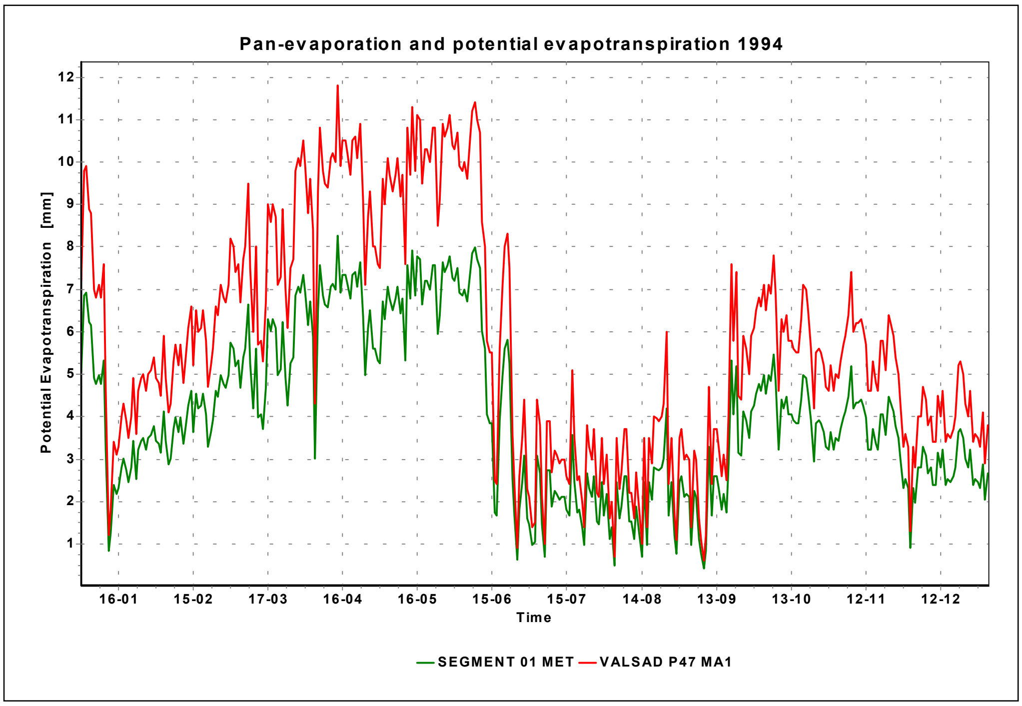

Figure 6: Pan evaporation and potential evapotranspiration near Dakor 1994

Water levels and discharges for 1994

The runoff series for Dakor are derived from the hourly water level record available for that station. The entire water level record for 1994 is presented in Figure 7

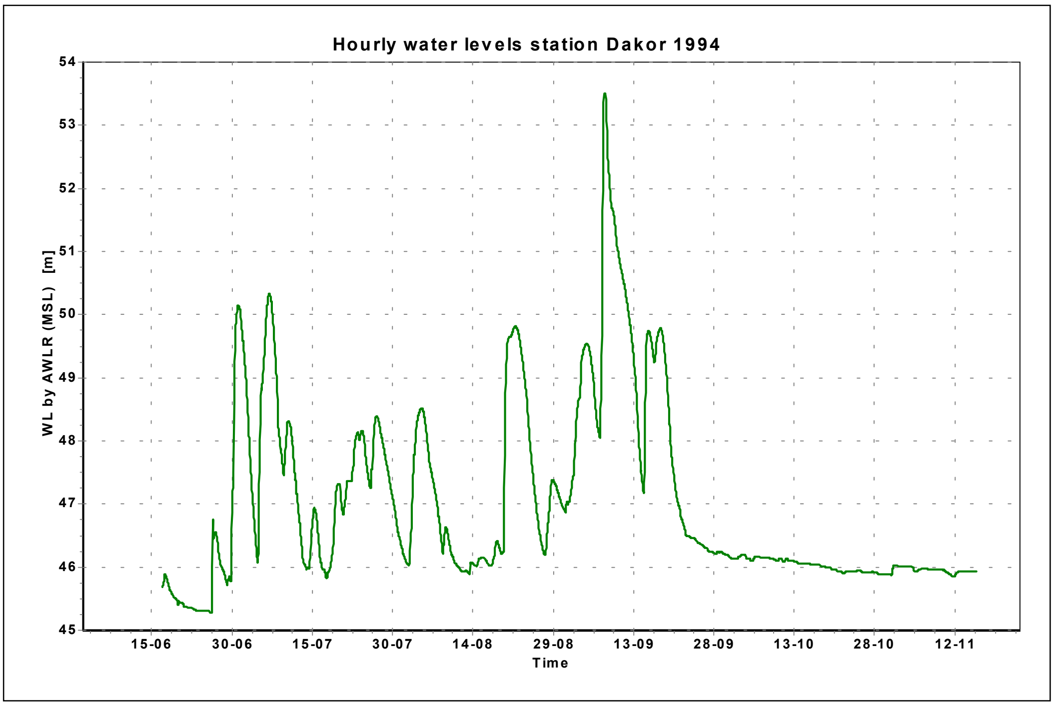

Figure 7: Hourly water levels Dakor 1994

The water levels are seen to vary between 45 to over 53 m+MSL at Dakor, i.e. 8 m difference. One also observes that the peaks are rounded, quite different from the water level record at Bilodra, available in your database.

The water level data are transformed into discharges by means of stage-discharge relations. The relations were fitted to the data presented in Figure 8. Two rating curves were developed, one valid till 14 July and one valid thereafter. About the cause of the change no further investigations were made, but the changes are most likely caused by shifts in the control section due to morphology. Note that the change takes place after the occurrence of the first peaks in 1994. The rating curves fitted to the data from 15 July onward are shown in Figure 9.

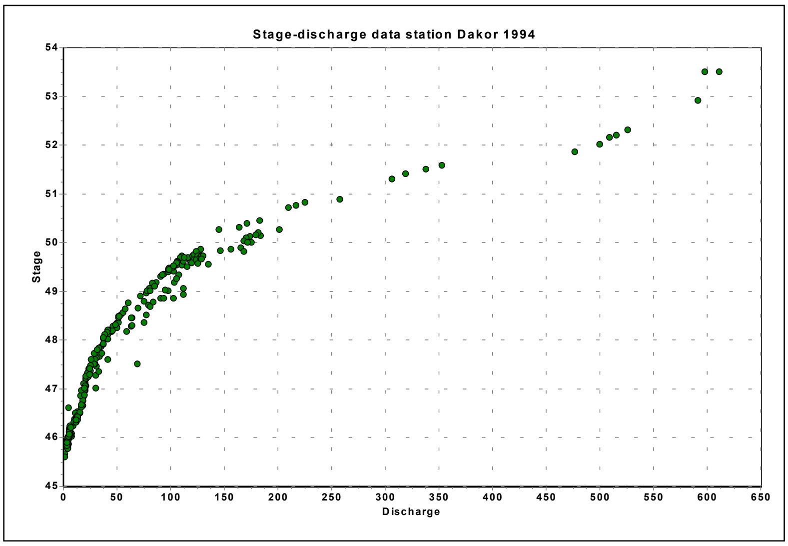

Figure 8: Stage-discharge measurements of 1994 for station Dakor

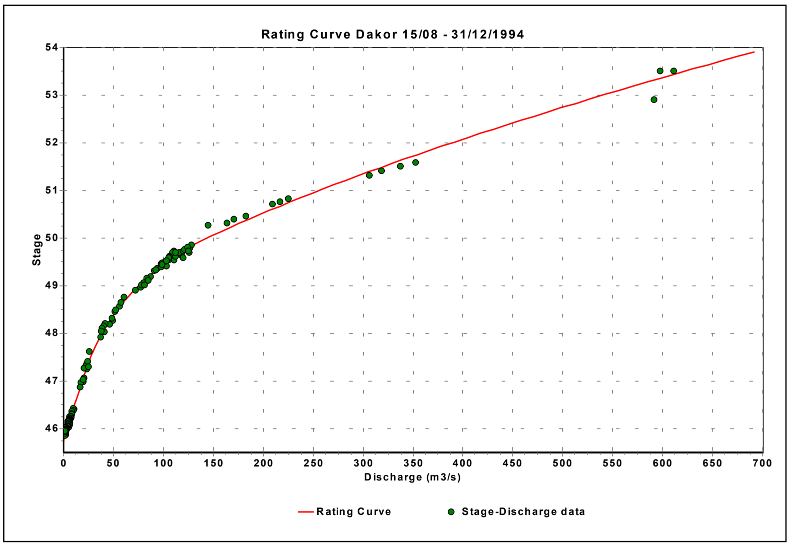

Figure 9:
Stage-discharge relation for station Dakor
(15/8 – 31/12/1994)

Note that a few measurements in the higher flow region have been omitted. These data referred to a falling stages but these data plotted to the right of the curve matching with the highest flows during steady stages. For a stable channel the data should have plotted to the left of the curve and from that point of view were considered inconsistent. One reason for plotting right might have been that the downstream control section has drastically eroded during the passage of the flood wave.

The resulting hourly discharge hydrograph at Dakor for 1994 is shown in Figure 10.

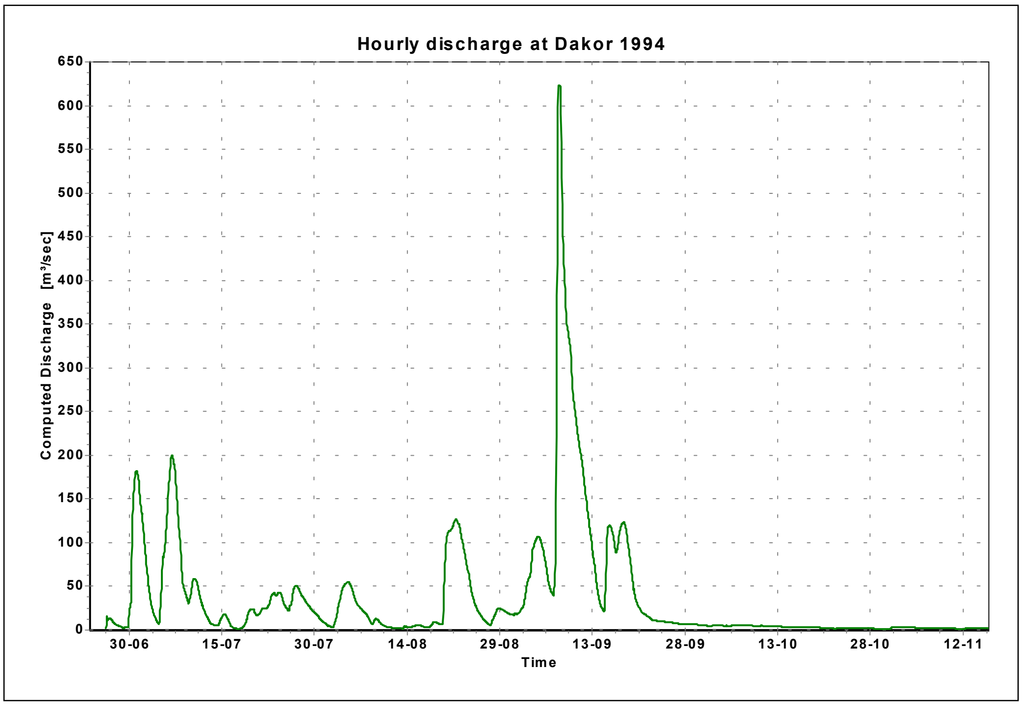

Figure 10: Hourly runoff at Dakor for 1994

The discharges in m^3/s have subsequently been transformed to hourly runoff values in mm/hr by multiplying the discharges with 3600 (s)/area(km^2) x 10^{-3}. **Subsequently, the hourly runoff values have been aggregated to daily values, in a manner equal to the way daily rainfall data are treated, i.e. from 8.00 hrs at day 1 to 8.00 hrs at day 2,**

reported as a daily value for day 2. This requires special attention while executing the aggregation.

The total runoff for the year 1994 amounted 1062 mm, i.e. a runoff coefficient of 72%.

The daily rainfall, potential evapotranspiration and runoff data for 1994 are tabulated in Tables 2, 3 and 4.

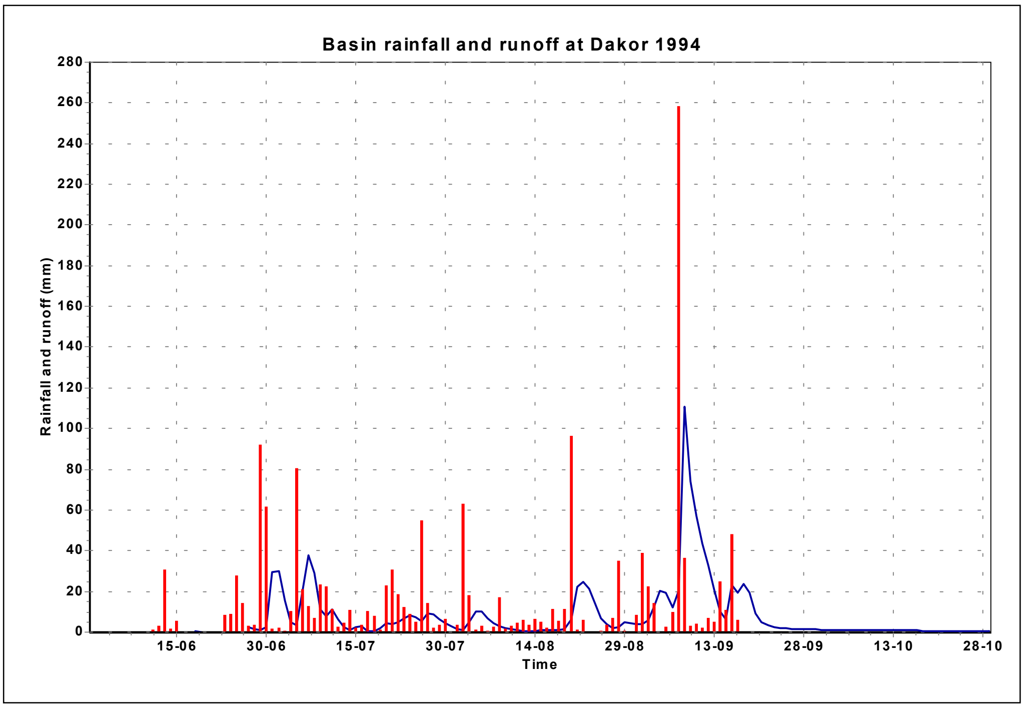

Figure 11: Daily rainfall and runoff for 1994 for Dakor basin

3.4 Estimation of parameters

The model parameter estimation based on the data for the year 1994 is carried along the lines as presented in the text in Chapter 2.4.2

Segment area
Segment area is derived from the basin boundary data in HYMOS:

Segment area = 430.59 km^2.

Lower zone primary free water storage parameters LZPK and LZFPM
Reference is made to the semi-logarithmic plot of the runoff series, shown in Figure 12. Lowest runoff values with an exponential decay showing as a straight line in the plot are present at the end of October and in November, see Figure 13. A straight line is fitted the observations and the drainage factor LZPK is determined from the runoff values at 31/10 and 12/11, which are solely attributed to runoff from the lower zone primary free water storage. It then follows from equation (8):

$$KP = \left(\frac{R_{(12/11)}}{R_{(31/10)}}\right)^{1/12} = \left(\frac{0.23}{0.27}\right)^{1/12} = 0.986$$

Hence the drainage factor becomes with equation (9):

Table 2: Daily rainfall in Dakor segment for 1994

Daily data and statistics of series SEGMENT 01 MPC Year = 1994

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	.00	.00	.00	.00	.00	.00	1.54	3.23	38.90	.00	.00	.00
2	.00	.00	.00	.00	.00	.00	1.88	63.02	22.35	.00	.00	.00
3	.00	.00	.00	.00	.00	.00	.57	18.01	13.99	.00	.00	.00
4	.00	.00	.00	.00	.00	.00	10.19	1.00	.20	.00	.00	.00
5	.00	.00	.00	.00	.00	.00	80.43	2.83	2.52	.00	.00	.00
6	.00	.00	.00	.00	.00	.00	20.56	.41	9.69	.00	.00	.00
7	.00	.00	.00	.00	.00	.00	12.47	2.21	258.01*	.00	.00	.00
8	.00	.00	.00	.00	.00	.00	6.53	17.11	36.08-	.00	.00	.00
9	.00	.00	.00	.00	.00	.00	23.00	1.44	3.08	.00	.00	.00
10	.00	.00	.00	.00	.00	.00	22.04	2.92	3.64	.00	.00	.00
11	.00	.00	.00	.00	.00	1.00	11.28	4.42	1.90	.00	.00	.00
12	.00	.00	.00	.00	.00	3.00	2.38	5.58	6.83	.00	.00	.00
13	.00	.00	.00	.00	.00	30.36	4.53	3.55	4.67	.00	.00	.00
14	.00	.00	.00	.00	.00	1.28	10.52	6.26	24.53	.00	.00	.00
15	.00	.00	.00	.00	.00	5.55	2.08	5.02	10.62	.00	.00	.00
16	.00	.00	.00	.00	.00	.11	3.33	2.10	48.08	.00	.00	.00
17	.00	.00	.00	.00	.00	.00	10.20	11.23	5.60	.00	.00	.00
18	.00	.00	.00	.00	.00	.00	7.94	5.51	.00	.00	.00	.00
19	.00	.00	.00	.00	.00	.00	.84	11.15	.00	.00	.00	.00
20	.00	.00	.00	.00	.00	.00	22.50	96.36	.00	.00	.00	.00
21	.00	.00	.00	.00	.00	.00	30.31	.96	.00	.00	.00	.00
22	.00	.00	.00	.00	.00	.00	18.47	5.93	.00	.00	.00	.00
23	.00	.00	.00	.00	.00	8.25	11.95	.00	.00	.00	.00	.00
24	.00	.00	.00	.00	.00	8.55	8.74	.00	.00	.00	.00	.00
25	.00	.00	.00	.00	.00	27.80	4.92	.70	.00	.00	.00	.00
26	.00	.00	.00	.00	.00	13.94	54.55	3.34	.00	.00	.00	.00
27	.00	.00	.00	.00	.00	3.12	13.91	6.81	.00	.00	.00	.00
28	.00	.00	.00	.00	.00	3.22	1.79	34.75	.00	.00	.00	.00
29	.00	*********	.00	.00	.00	91.97	3.22	.33	.00	.00	.00	.00
30	.00	*********	.00	.00	.00	61.31	6.37	.00	.00	.00	.00	.00
31	.00	*********	.00	*********	.00	*********	.00	8.26	*********	.00	*********	.00
Data	31	28	31	30	31	30	31	31	30	31	30	31
Eff.	31	28	31	30	31	30	31	31	30	31	30	31
Miss	0	0	0	0	0	0	0	0	0	0	0	0
Sum	.00	.00	.00	.00	.00	259.46	409.04	324.44	490.70	.00	.00	.00
Mean	.00	.00	.00	.00	.00	8.65	13.19	10.47	16.36	.00	.00	.00
Min.	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
Max.	.00	.00	.00	.00	.00	91.97	80.43	96.36	258.01	.00	.00	.00

Annual values:

Data	365 *	Sum	1483.63 *	Minimum	.00 *	Too low	0
Effective	365 *	Mean	4.06 *	Maximum	258.01 *	Too high	1
Missing	0						

Table 3: Daily potential evapotranspiration in Dakor segment for 1994

Daily data and statistics of series SEGMENT 01 MET Year = 1994

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	5.04	3.36	4.97	6.93	5.25	7.49	.98	1.12	2.17	4.62	3.43	2.31
2	6.86	3.15	5.74	6.16	6.72	6.93	1.05	1.40	2.10	4.97	3.71	1.96
3	6.93	4.13	5.60	6.72	6.30	6.86	3.08	.49	.98	4.55	3.99	2.80
4	6.23	3.43	5.18	5.88	7.07	7.00	2.66	2.45	2.24	4.97	4.13	2.80
5	6.16	2.87	5.32	3.01	6.79	6.72	1.47	1.61	2.10	4.83	4.48	3.29
6	4.90	3.01	4.69	6.51	6.51	7.42	.70	1.89	1.12	5.46	5.18	3.08
7	4.76	3.71	5.39	7.56	6.79	7.84	2.73	2.59	.77	4.83	4.20	2.66
8	4.97	3.99	5.60	6.86	7.07	7.98	2.73	2.59	.42	3.22	4.34	2.80
9	4.76	3.64	6.65	6.65	6.44	7.70	1.89	1.54	.84	4.41	4.34	2.38
10	5.32	3.99	5.25	6.58	6.79	7.49	2.24	1.54	3.29	4.20	4.41	2.38
11	3.22	3.36	4.20	7.07	5.32	6.02	2.17	1.12	1.68	4.48	4.27	3.15
12	.84	3.71	5.60	7.14	7.56	5.60	2.03	1.89	2.59	4.06	3.99	2.80
13	1.26	4.27	3.99	7.00	6.79	4.06	2.10	1.26	2.59	4.06	3.22	3.22
14	2.38	4.62	4.06	8.26	7.91	3.85	2.10	.70	2.31	3.92	3.22	2.38
15	2.17	3.64	3.71	6.93	6.86	3.85	1.82	2.45	1.82	3.85	3.71	2.52
16	2.31	4.55	4.69	7.35	7.77	1.75	1.68	.98	2.17	3.85	3.36	2.45
17	2.80	4.20	6.30	7.35	7.70	1.68	3.57	2.45	1.75	4.41	3.22	2.59
18	3.01	4.27	6.02	7.14	6.65	3.92	2.45	2.03	2.87	4.97	4.06	2.80
19	2.73	4.55	6.30	6.79	7.21	4.97	1.75	2.80	5.32	4.90	4.06	3.64
20	2.45	4.06	6.09	7.35	7.21	5.60	1.82	2.77	4.06	4.55	3.57	3.71
21	2.80	3.29	4.97	7.42	7.00	5.81	1.40	2.73	5.18	3.92	4.48	3.50
22	3.43	3.64	5.11	7.07	7.56	5.25	.98	2.80	3.15	2.94	4.34	3.01
23	2.52	3.92	6.23	7.63	7.56	2.52	2.66	3.01	3.08	3.85	4.13	2.80
24	3.22	4.62	4.90	6.37	5.95	1.68	2.31	4.20	4.13	3.92	3.78	3.22
25	3.43	4.48	4.27	4.97	6.37	.63	2.10	1.68	3.92	3.85	3.50	2.38
26	3.50	4.97	5.25	6.09	7.63	1.82	2.59	2.45	3.50	3.64	3.01	2.52
27	3.22	4.76	5.39	6.51	7.42	2.45	1.54	1.19	4.13	3.29	2.31	2.45
28	3.50	4.69	6.86	5.60	7.56	3.08	1.47	.77	4.27	3.22	2.52	2.31
29	3.57	*********	7.07	5.60	7.77	1.61	2.45	2.45	4.55	3.64	2.31	2.87
30	3.78	*********	6.93	5.32	7.28	1.47	1.68	2.59	4.76	3.22	.91	2.03
31	3.43	*********	7.35	*********	7.21	*********	2.17	2.10	*********	3.50	*********	2.66
Data	31	28	31	30	31	30	31	31	30	31	30	31
Eff.	31	28	31	30	31	30	31	31	30	31	30	31
Miss	0	0	0	0	0	0	0	0	0	0	0	0
Sum	115.50	110.88	169.68	197.82	216.02	141.05	62.37	61.63	83.86	128.10	110.18	85.47
Mean	3.73	3.96	5.47	6.59	6.97	4.70	2.01	1.99	2.80	4.13	3.67	2.76
Min.	.84	2.87	3.71	3.01	5.25	.63	.70	.49	.42	2.94	.91	1.96
Max.	6.93	4.97	7.35	8.26	7.91	7.98	3.57	4.20	5.32	5.46	5.18	3.71

Annual values:

Data	365	* Sum	1482.56	* Minimum	.42	* Too low	0
Effective	365	* Mean	4.06	* Maximum	8.26	* Too high	0

Daily runoff from Dakor segment in 1994

Daily data and statistics of series Dakor HRC Year = 1994

Day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	29.42	1.48	3.83	1.13	.64	-999.99*
2	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	30.15	.87	5.99	1.01	.64	-999.99*
3	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	15.36	4.82	12.58	1.11	.64	-999.99*
4	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	4.93	10.00	20.27	1.13	.62	-999.99*
5	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	3.55	10.23	19.47	.97	.45	-999.99*
6	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	20.30	6.55	12.29	1.07	.54	-999.99*
7	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	37.48	4.55	20.33	1.07	.54	-999.99*
8	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	29.02	2.68	110.92*	1.06	.51	-999.99*
9	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	11.34	2.14	73.86	1.02	.52	-999.99*
10	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	7.84	1.53	56.92	.95	.47	-999.99*
11	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	11.02	.77	43.44	.94	.33	-999.99*
12	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	6.11	.48	32.67	.96	.23	-999.99*
13	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	2.58	.41	20.85	.90	.41	-999.99*
14	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	1.18	.69	9.99	.81	.42	-999.99*
15	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	2.27	.80	6.51	.77	.42	-999.99*
16	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	2.89	1.05	23.10	.75	-999.99*	-999.99*
17	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	.72	.78	19.58	.73	-999.99*	-999.99*
18	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	.65	.31	1.03	23.90	.69	-999.99*	-999.99*
19	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	.20	1.73	1.65	19.49	.61	-999.99*	-999.99*
20	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	4.56	6.39	9.09	.56	-999.99*	-999.99*
21	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	3.68	22.37	4.67	.49	-999.99*	-999.99*
22	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	4.86	24.74	3.29	.38	-999.99*	-999.99*
23	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	6.62	21.46	2.35	.39	-999.99*	-999.99*
24	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	8.27	14.03	2.07	.41	-999.99*	-999.99*
25	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	7.23	7.00	1.91	.44	-999.99*	-999.99*
26	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	5.18	3.72	1.69	.38	-999.99*	-999.99*
27	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	2.44	9.25	1.82	1.47	.35	-999.99*	-999.99*
28	-999.99*	-999.99*	-999.99*	-999.99*	-999.99*	1.43	8.47	2.18	1.32	.37	-999.99*	-999.99*
29	-999.99*	*********	-999.99*	-999.99*	-999.99*	.73	6.16	4.69	1.32	.30	-999.99*	-999.99*
30	-999.99*	*********	-999.99*	-999.99*	-999.99*	2.23	4.33	4.38	1.28	.28	-999.99*	-999.99*
31	-999.99*	*********	-999.99*	*********	-999.99*	*********	2.79	3.63	*********	.27	*********	-999.99*
Data	31	28	31	30	31	30	31	31	30	31	30	31
Eff.	0	0	0	0	0	6	31	31	30	31	15	0
Miss	31	28	31	30	31	24	0	0	0	0	15	31
Sum	-999.99	-999.99	-999.99	-999.99	-999.99	7.69	289.61	168.95	566.45	22.30	7.37	-999.99
Mean	-999.99	-999.99	-999.99	-999.99	-999.99	1.28	9.34	5.45	18.88	.72	.49	-999.99
Min.	-999.99	-999.99	-999.99	-999.99	-999.99	.20	.31	.41	1.28	.27	.23	-999.99
Max.	-999.99	-999.99	-999.99	-999.99	-999.99	2.44	37.48	24.74	110.92	1.13	.64	-999.99

Annual values:

Data	365 *	Sum	1062.37 *	Minimum	.20 *	Too low	0
Effective	144 *	Mean	7.38 *	Maximum	110.92 *	Too high	1

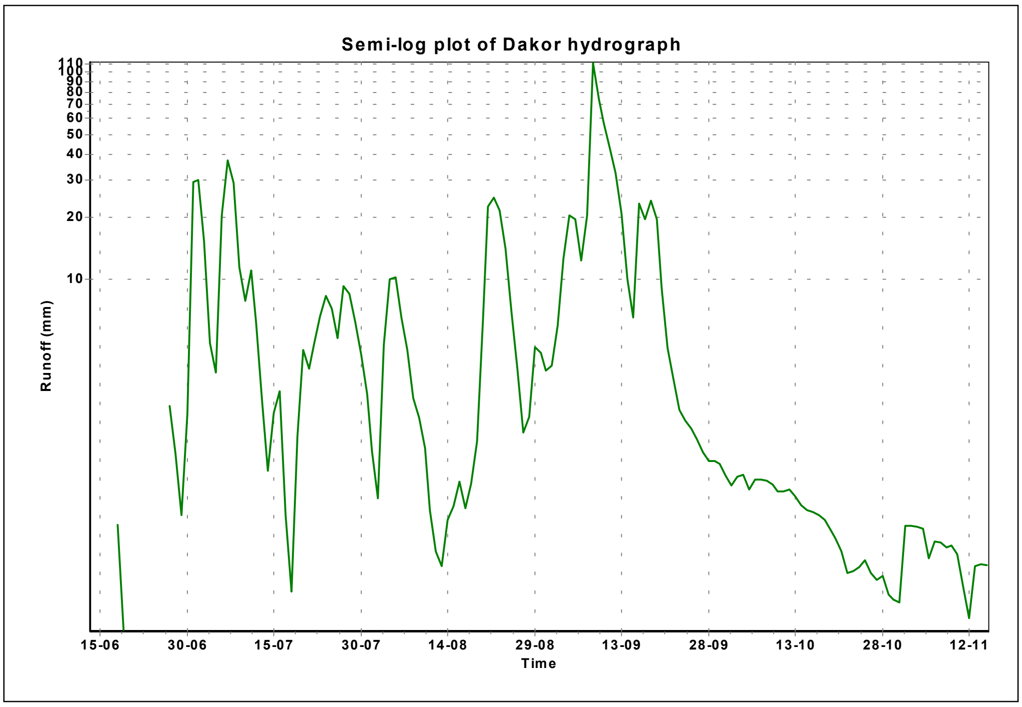

Figure 12: Semi-logarithmic plot of runoff series for Dakor 1994

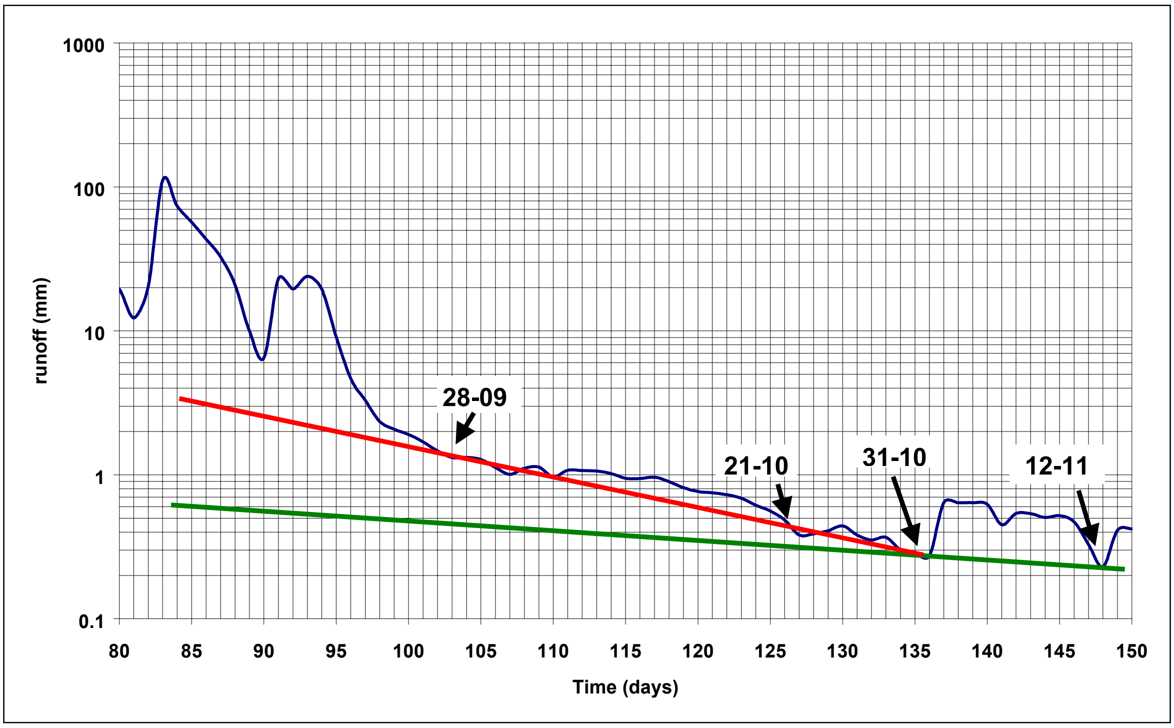

Figure 13: Detail of semi-log plot of Dakor hydrograph for estimation of lower zone free water storage parameters

LZPK = 1 – KP = 1 – 0.986 = 0.014

To arrive at a value for the capacity of the lower zone primary free water storage an estimated maximum runoff value from that reservoir is required RP max. Assuming that after a very wet period this maximum is achieved one can estimate this maximum by extrapolating the primary baseflow recession curve backward in time up to the runoff peak on 8 September. Under the presumption that this storm has completely filled the lower zone primary free water storage a maximum value of approximately 0.6 mm/day can be read from the semi-log plot. (This value can also be computed from the value at 31/11 using KP and the time interval from 8/9 till 31/10: QP(8/9) = QP(31/11)KP^{-53} = 0.57 mm/day).

Hence the LZFPM becomes with equation (10):

$$LZFPM = \frac{QP\max}{LZPK} \approx \frac{0.6}{0.014} \approx 45mm$$

Note that the value is rounded to the nearest 5 mm, in view of the uncertainties involved.

It is noted here that extrapolation of the primary baseflow recession curve backward from 31/10 to 8/9 is not entirely correct as in between some recharge may have occurred by the storm between 16 and 18/9. On the other hand we are not entirely sure that on 8/9 the lower free water zone was completely filled.
Abstractions from river flow may affect the result. Abstractions may be observed from recession on semi-log plot failing to fall to a straight line (it curves downward in the course of time). By adding a constant amount a straight line can often be obtained. The effect is illustrated in Figure 14.

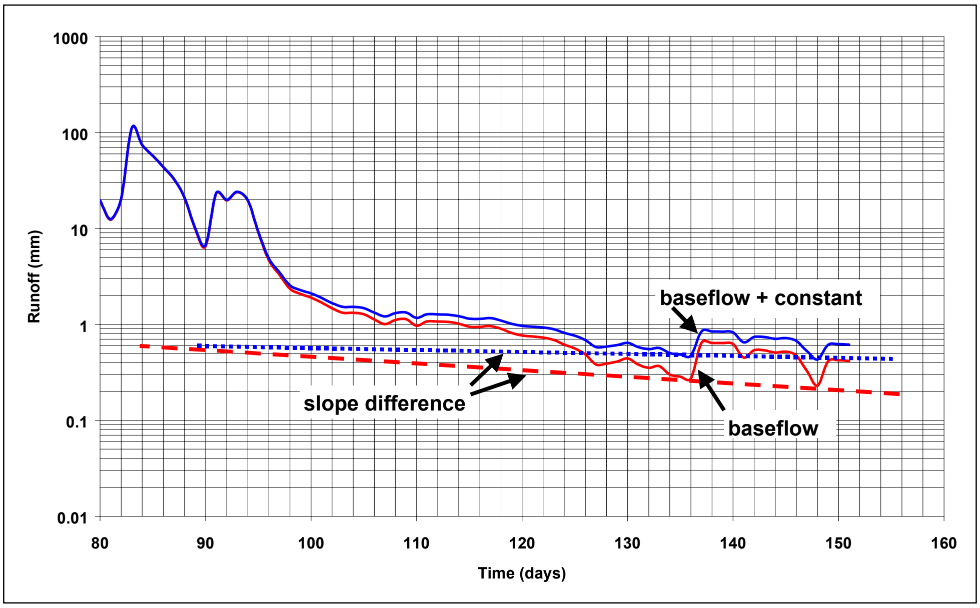

Figure 14: Effect of abstraction of river flow on primary baseflow parameters

It is observed that the result is rather sensitive to abstractions and due care should be given to this phenomenon. If abstractions are present and no corrections are made than the estimated LZPK-value will be too high.

Question: what would be the estimates for LZPK and LZFPM is 0.1 mm/day is added to the recession curve??

Lower zone supplemental free water storage parameters LZSK and LZFSM

The lower zone supplemental free water storage parameters are determine in a fashion similar to the primary base flow parameters. For the supplemental storage the hydrograph between 28/9 and 31/10 is observed. It is assumed that the runoff in this period is entirely due to baseflow: primary and supplemental baseflow

On 28/9 $Q = 1.32$ mm/day
On 21/10 $Q = 0.49$ mm/day
On 31/10 $Q = 0.27$ mm/day, primary base flow

First the primary baseflow component in the flows on 28/9 and 21/10 are estimated by backward extrapolation from 31/10 using equation (8). The time intervals are respectively 33 and 10 days with 31/10. Hence:

28/9 $QS = 1.32 - 0.27 \times 0.986^{-33} = 1.32 - 0.43 = 0.89$
21/10 $QS = 0.49 - 0.27 \times 0.986^{-10} = 0.49 - 0.31 = 0.18$

Hence:

$$KS = \left(\frac{QS_{(21/10)}}{QS_{(28/9)}}\right)^{1/23} = \left(\frac{0.18}{0.89}\right)^{1/23} = 0.933$$

So:

$LZSK = 1 - KS = 1 - 0.933 = 0.067$

The maximum total baseflow is estimated from Figure 13 as about 3.5 mm/day using the same procedure as before: extrapolation backward in time till 8/9 (can also be derived analytically). Since the primary baseflow was estimated here as 0.6 mm/day then QSmax = 3.5 – 0.6 = 2.9 mm/day. Then:

$$LZFSM = \frac{QS\max}{LZSK} = \frac{2.9}{0.067} \approx 45\text{mm}$$

A sensitivity analysis similar to the one carried out for the primary baseflow will indicate that the parameters of the supplemental storage are less sensitive to abstractions.

Upper zone tension storage capacity UZTWM

There has been a long dry spell before 11/6. No response to rainfall is found for the storm between 11/6 and 15/6. Rains thereafter do show increases in flow. Hence this period is suitable for estimation of UZTWM. The rainfall in this period amounted in total 41.2 mm. The potential evapotranspiration is for this period 23.4 mm. The sum of the two would be an upper limit to the storage capacity in the upper zone tension as the actual evapotranspiration has probably been less. Hence a value between 50 and 60 mm is likely. A first estimate of 60 mm will be applied.

Upper zone free water storage UZK and UZFWM

From inspection of falling limbs of the hydrograph above baseflow, it is estimated that the depletion of the interflow takes about 6 days, hence from Figure 10 in the text

$UZK \approx 0.3$

The maximum interflow is difficult to assess. From the hydrograph between 19/9 and 20/9 a value of around 10 mm, hence a first estimate for UZFWM reads

$UZFWM \approx 30$ mm

Lower zone tension water capacity LZTWM

The period between 29/6 and 14/7 which follows a period where the upper zone tension is likely to be about full, but little supply has taken place to the lower zone. The period is enclosed by runoff from the lower zone free water reservoirs only, see Figure 15.

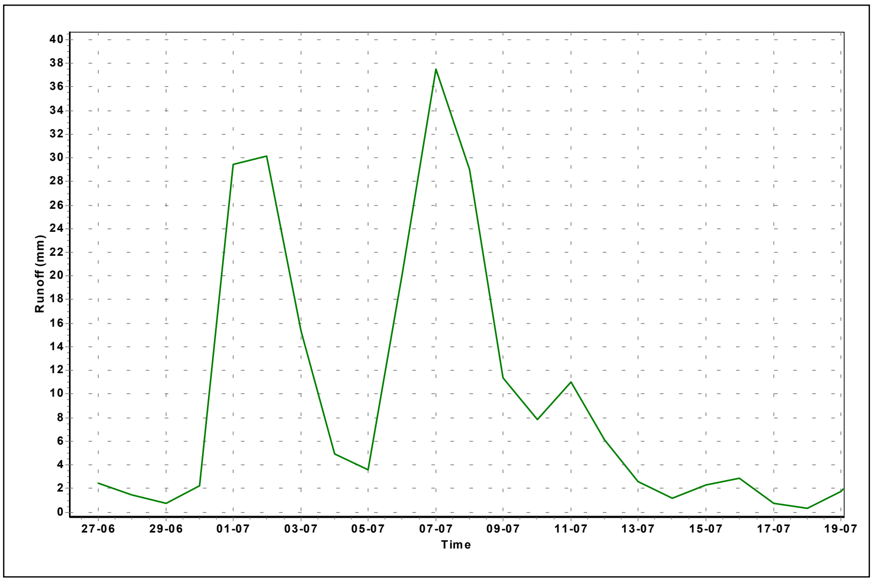

Figure 15: Part of hydrograph used for LZTWM

Assuming that changes in lower zone free water storages as well as the upper zone storages are small, from a simple waterbalance computation for the period 29/6 to 14/7 the following change in the LZTW storage is observed

$\Delta LZTWC = \Sigma P - \Sigma R - \Sigma E = 361 - 217 - 31 = 113$ mm

Note that the evaporation is taken as potential as the UZTW storage was filled.

The value obtained in this manner is certainly a lower limit as by mid July there is still capacity in the lower zone. The first estimate is therefore set as:

LZTWM = 150 mm

Extending the period to 24/9 leads to a value of 1377 – (1016 + 180) = 181 mm, but then corrections for the free lower zone storages have to be taken into account as well. Also the errors may occur, stemming from the fact that the evaporation is not at its potential rate etc.

Percolation parameters ZPERC and REXP

From equation 17 a first estimate for ZPERC can be obtained. It requires the value of PBASE:

PBASE = LZFPM x LZPK + LZFSM x LZSK = 45 x (0.014 + 0.067) = 3.64 mm/day

Hence with equation 17:

$$ZPERC = \frac{LZTWM + LZFSM + LZFPM - PBASE}{PBASE} = \frac{150 + 45 + 45 - 3.64}{3.64} = 57$$

So as a first approximation a value of 60 is assumed.

REXP is estimated at 1.5 as the soils appear to be sandy, see Table 1 in the text.
Unit hydrograph parameters

The concentration time for the Dakor basin can be estimated by assuming a celerity between 2 to 3 m/s or 7 to 10 km/hr during floods. With a total river length of 53 km it implies that the concentration time will be in the order of 5 to 8 hrs, which is much smaller than the time interval to be used in the simulation. The hydrograph though shows that the surface runoff is considerably delayed. An approximation for the unit hydrograph components based on inspection of the runoff compared to the rainfall gives the following hydrograph values:

0.15, 0.40, 0.30, 0.15

The Clark procedure could also be used here. This is discussed below.

From Figure 16 a concentration time is computed from a comparison of the rainfall and runoff record. The time between the cessation of rainfall to the inflection point on the falling limb of the hydrograph is a good indicator for the time of concentration Tc. From Figure 16 a value of 2 days (± 0.5 days) is read.

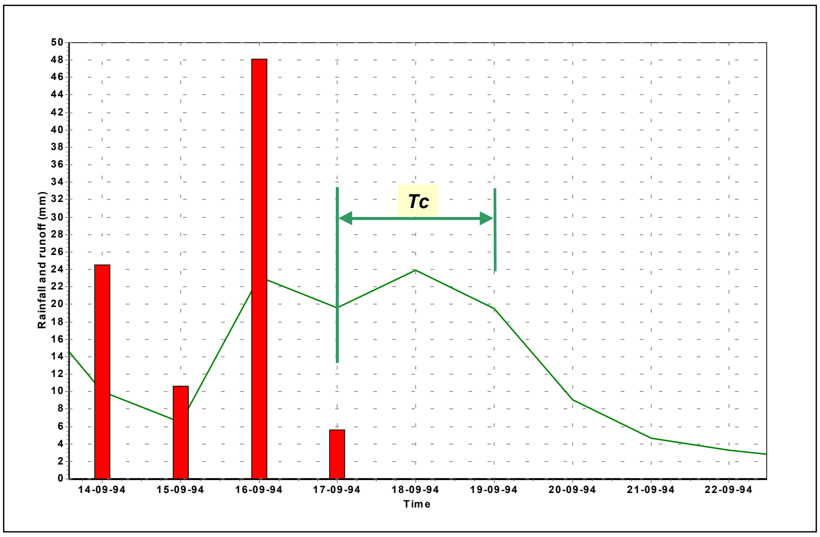

Figure 16: Estimation of the time of concentration from rainfall and runoff record.

In the time area diagram 2 intervals are considered. The time area diagram presents the hydrograph resulting from an instantaneous supply of 1 mm over the catchment. Since we consider two intervals only one isochrone is considered. The isochrone separate the segment into two parts where, given the shape of the segment, the lower part constitutes about 40% of the segment and the upper part 60%. Again great detail is not required as we are dealing with daily data in a small segment, with a Tc value somewhere between 1.5 and 2.5 days. The approximate time area diagram is presented in Figure 18.

The next step is to estimate the reservoir coefficient k. This coefficient is obtained from the slope of the recession of the surface water hydrograph. For this Figure 17 is observed. If the baseflow part is subtracted from the actual flow values then the surface runoff is seen to reduce from 18 mm/day to 2 mm/day in 2 days. Hence, k is obtained from:

$$Q_{t2} = Q_{t1}\exp\left(-\frac{(t2-t1)}{k}\right)$$

$$k = -\frac{(t2-t1)}{\ln(Q_{t2}/Q_{t1})} = -\frac{2}{\ln(2/18)} = \frac{2}{-2.20} = 0.91\text{days}$$

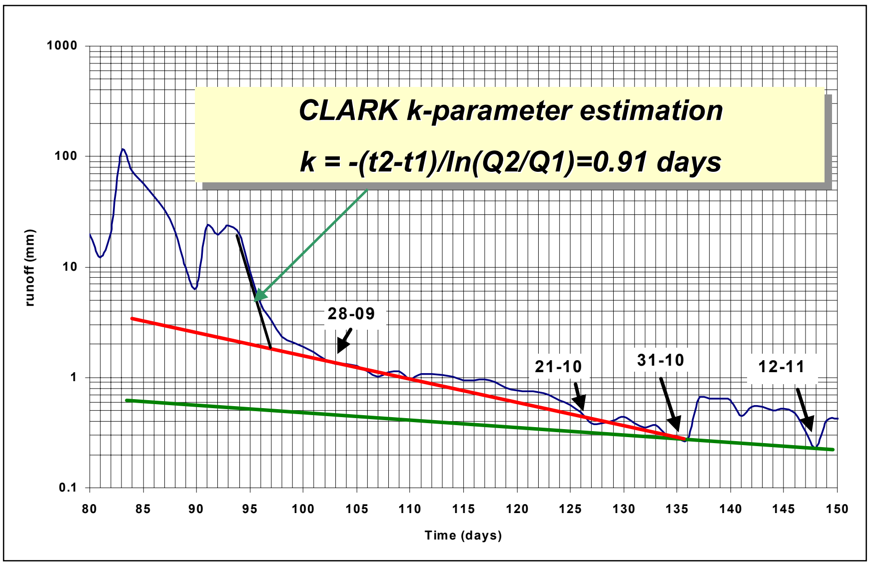

Figure 17: Estimation of reservoir coefficient parameter in Clark method

Note that for the estimation of k one should particularly concentrate on the surface runoff part and not on the interflow part as interflow is already delayed by the UZFW reservoir.

The routing coefficients then become according to equation 23 with Δt = 1 day and k = 0.91 days:

$$c1 = \frac{\Delta t}{k + \Delta t/2} = \frac{1}{0.91 + 1/2} = 0.71;\ c2 = 1 - c1 = 1 - 0.71 = 0.29$$

Hence, the routing equation becomes:

Qi+1 = c1 x Iav + c2 x QI = 0.71 x Iav + 0.29 x QI, etc.

The result is the instantaneous unit hydrograph. The 1-day unit hydrograph is obtained by averaging over successive intervals: Qday, i = ½(Qinst,i + Qinst,i-1).

The routing is carried out in the Table below, and the result is presented in Figure 18.

Time	**Input I_{av}**	**$Q_{out\text{-}inst}$**	**$Q_{out\text{-}day}$**
0	**0**	**0.00**	**0.00**
1	**0.4**	**0.28**	**0.14**
2	**0.6**	**0.51**	**0.40**
3	**0**	**0.15**	**0.33**
4	**0**	**0.04**	**0.10**
5	**0**	**0.01**	**0.03**
6	**0**	**0.00**	**0.01**
7	**0**	**0.00**	**0.00**

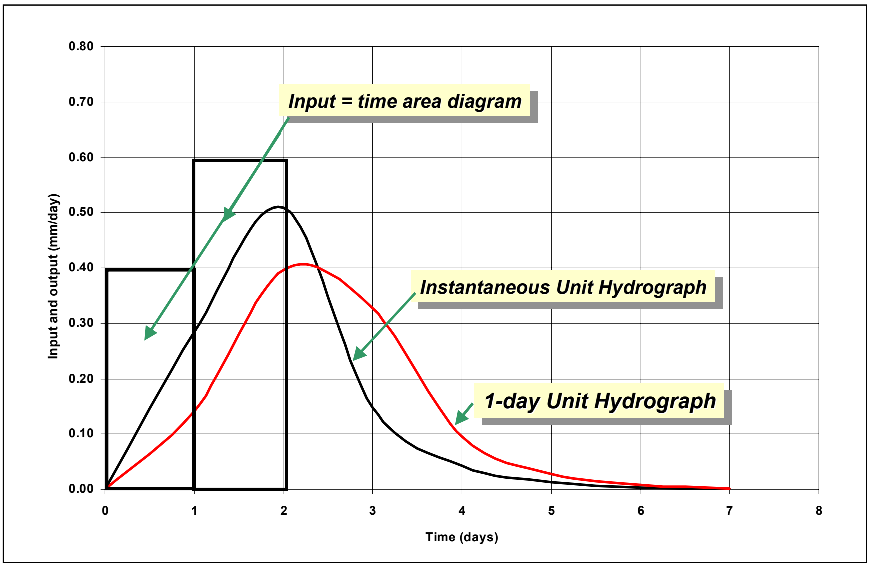

Figure 18: Time-area diagram and instantaneous and 1 day unit hydrograph

The values of the last column in the Table are input to the model as the 1-day unit hydrograph.

Other parameters

The SIDE parameter needs special attention in the Dakor case as the actual groundwater tables are far below the drainage base. It implies that water from the lower zone free water reservoirs will percolate further down to the deep groundwater table. To estimate the value of SIDE the unobserved portion of groundwater should be determined. Say, 100 mm is withdrawn from aquifer by mining and the groundwater tables are declining annually with 2 m, and the specific yield is 0.3. Then 100 mm + 0.3 x 2 m = 160 mm is withdrawn from the aquifer. This value has to be compared with the observed baseflow. Then SIDE follows from:

$$\text{SIDE} = \frac{\text{unobserved base flow}}{\text{observed base flow}} = \frac{160}{\text{observed base flow}}$$

The mined amount of groundwater (100 mm) is to be added to the model as rainfall.

The other parameters are set to their nominal values as the hydrograph do not permit estimation of e.g. PCTIM or ADIMP. For PCTIM the very first period in the monsoon would have been appropriate, but water level observations started too late for that and some days have missing values. The total list of first estimate of the parameters is shown below.

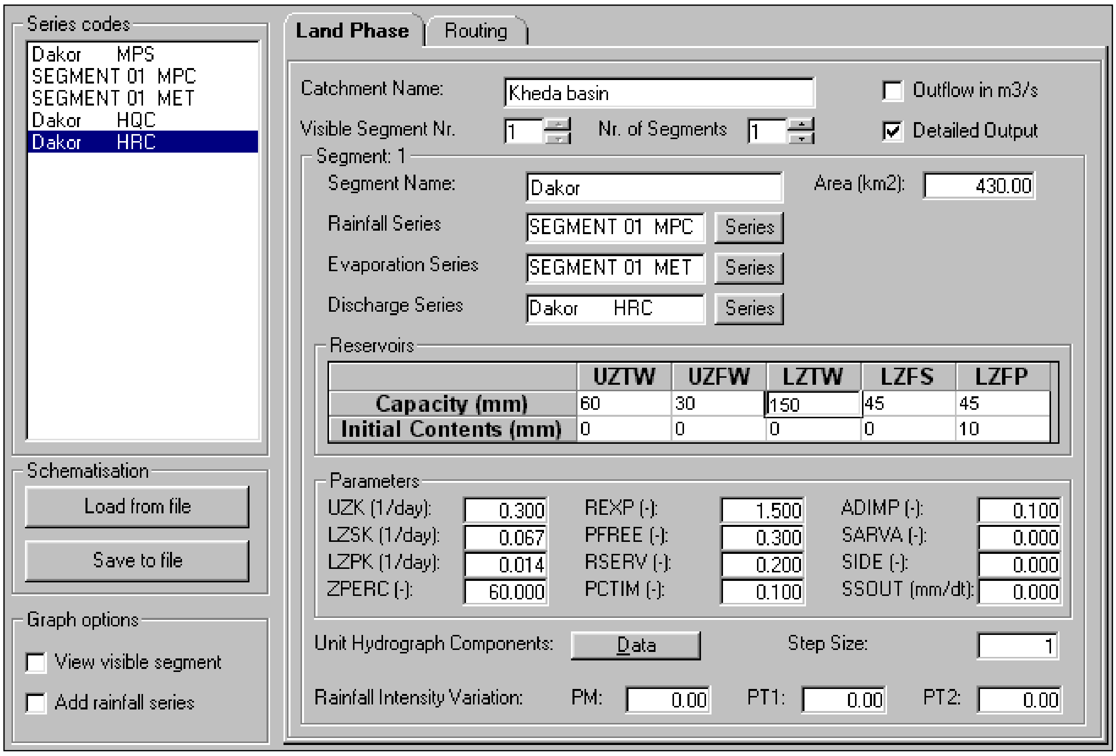

Figure 19: List of input parameters for Sacramento model

The initial contents when starting the run on 1/1/94 by assuming that in September the previous year all free base flow reservoirs were full. The potential evapotranspiration from September to January amounts about 400 mm so the tension water reservoirs are expected to have dried up.

First runs

The results of the first run are shown in Figure 17. The accumulated difference between observed and simulated run off closes at – 41 mm. Matching the water balance is the most important first step while continuing with calibration. By adding 50 mm to the LZTWM the difference is seen to be nearly eliminated, Figure 18. Then the fine-tuning can start. Particular attention is required to parameter SIDE. From the first run an observed base flow of 288 mm. With the assumed removal of 160 mm an estimate for the SIDE parameter would be 160/288 = 0.56.

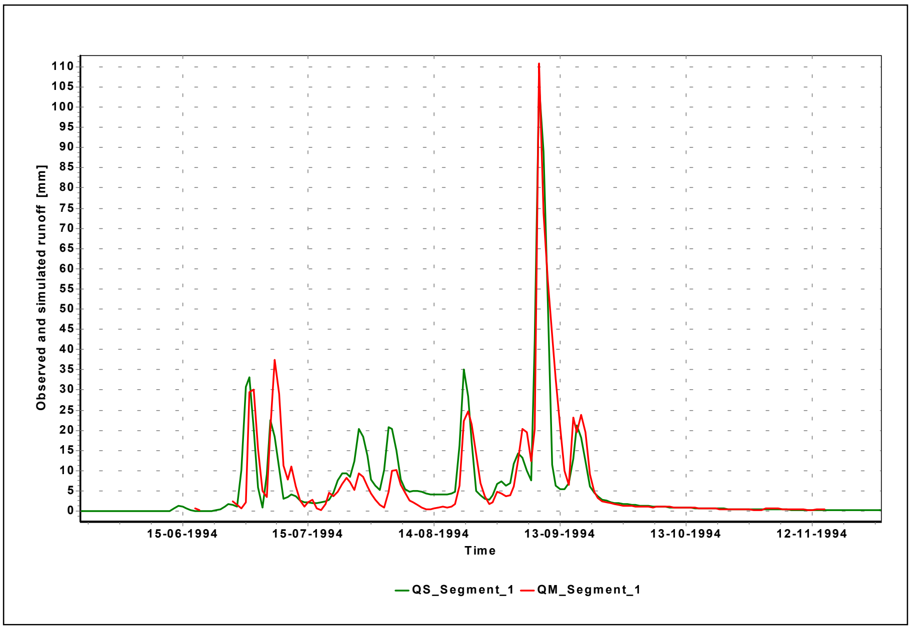

Figure 20: Observed and simulated runoff, first run

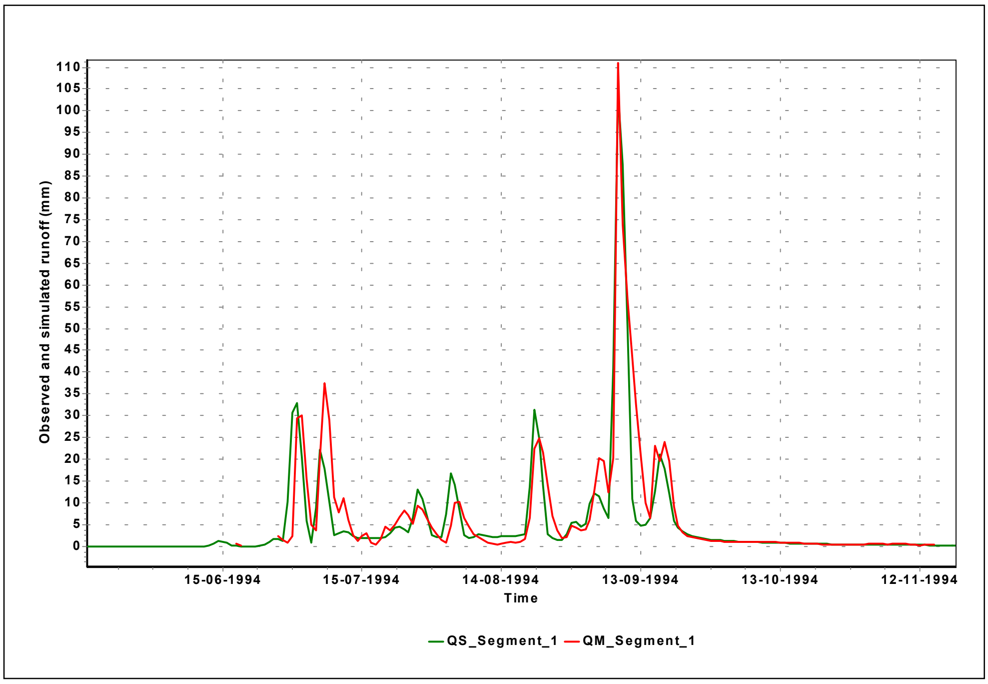

Figure 21: Observed and simulated flow, LZTWM increased to 200 mm

Example output

```
 Simulation of rainfall-runoff process: Land Phase


     Catchment name    = Kheda basin

     Number of segments:  1


 Rainfall-runoff simulation of segment  1

 Catchment: Kheda basin             Segment: Dakor

Rainfall series   : SEGMENT 01  MPC
Evaporation series: SEGMENT 01  MET
Discharge series  : Dakor       HRC

                  UZTW  UZFW  LZTW LZFSW LZFPW
 Capacaty  (mm)    60.0  30.0 200.0  45.0  45.0
 Initial content     .0    .0    .0    .0  10.0

 UZK   =      .3000 (1/DAY)     RSERV =      .2000 (-)
 LZSK  =      .0670 (1/DAY)     PCTIM =      .1000 (-)
 LZPK  =      .0140 (1/DAY)     ADIMP =      .1000 (-)
 ZPERC =    60.0000 (-)         SARVA =      .0000 (-)
 REXP  =     1.5000 (-)         SIDE  =      .0000 (-)
 PFREE =      .3000 (-)         SSOUT =      .0000 (mm/dt)

 Given unit hydrograph components:
     .150    .400    .300    .150

 Applied unit hydrograph components:
     .150    .400    .300    .150    .000

 Given Rainfall Intensity Components (PT1,PT2):
     .000    .000


 Time step results
 Year Mo Da Ho    PRECIP     UZTWC     UZFWC     LZTWC     LZFSC     LZFPC    MDISCH    CDISCH   ACCDIFF

 1994  6 16  0       .11     35.58       .00      1.27       .00       .31   -999.99       .72       .00
 1994  6 17  0       .00     34.59       .00      1.26       .00       .30   -999.99       .19       .00
 1994  6 18  0       .00     32.33       .00      1.26       .00       .30       .65       .09       .56
 1994  6 19  0       .00     29.65       .00      1.24       .00       .29       .20       .00       .76
```

1994	6	20	0	.00	26.88	.00	1.23	.00	.29	-999.99	.00	.76
1994	6	21	0	.00	24.28	.00	1.22	.00	.29	-999.99	.00	.76
1994	6	22	0	.00	22.15	.00	1.20	.00	.28	-999.99	.00	.76
1994	6	23	0	8.25	29.47	.00	1.19	.00	.28	-999.99	.13	.76
1994	6	24	0	8.55	37.20	.00	1.19	.00	.27	-999.99	.46	.76
1994	6	25	0	27.80	60.00	4.60	1.19	.00	.27	-999.99	1.01	.76
1994	6	26	0	13.94	60.00	12.12	4.41	.69	.95	-999.99	1.70	.76
1994	6	27	0	3.12	60.00	.67	12.89	2.47	2.75	2.44	1.61	1.59
1994	6	28	0	3.22	60.00	.14	13.36	2.40	2.81	1.43	1.17	1.85
1994	6	29	0	91.97	60.00	30.00	13.46	2.27	2.80	.73	10.12	-7.55
1994	6	30	0	61.31	60.00	30.00	34.12	6.57	7.15	2.23	30.75	-36.07
1994	7	1	0	1.54	60.00	.89	54.89	10.64	11.45	29.42	33.01	-39.66
1994	7	2	0	1.88	60.00	.83	55.51	10.06	11.42	30.15	20.41	-29.91
1994	7	3	0	.57	57.49	.00	56.09	9.51	11.38	15.36	5.85	-20.40
1994	7	4	0	10.19	60.00	5.13	56.07	8.88	11.22	4.93	.91	-16.38
1994	7	5	0	80.43	60.00	30.00	59.66	9.08	11.80	3.55	8.98	-21.81
1994	7	6	0	20.56	60.00	20.10	80.49	13.14	15.90	20.30	22.39	-23.90
1994	7	7	0	12.47	60.00	9.74	94.56	15.44	18.53	37.48	18.12	-4.53
1994	7	8	0	6.53	60.00	3.80	101.38	15.97	19.63	29.02	10.48	14.01
1994	7	9	0	23.00	60.00	21.11	104.04	15.51	19.88	11.34	2.75	22.60
1994	7	10	0	22.04	60.00	19.80	118.82	17.93	22.48	7.84	3.29	27.15
1994	7	11	0	11.28	60.00	9.11	132.68	20.01	24.82	11.02	3.81	34.36
1994	7	12	0	2.38	60.00	.35	139.06	20.21	25.67	6.11	3.49	36.98
1994	7	13	0	4.53	60.00	2.43	139.30	18.91	25.35	2.58	2.54	37.03
1994	7	14	0	10.52	60.00	8.42	141.00	18.07	25.30	1.18	2.10	36.11
1994	7	15	0	2.08	60.00	.26	146.89	18.33	26.00	2.27	2.13	36.25
1994	7	16	0	3.33	60.00	1.65	147.07	17.15	25.67	2.89	1.99	37.15
1994	7	17	0	10.20	60.00	6.63	148.23	16.29	25.51	.72	1.90	35.98
1994	7	18	0	7.94	60.00	5.49	152.87	16.40	25.95	.31	2.08	34.21
1994	7	19	0	.84	59.09	.00	156.71	16.29	26.24	1.73	2.15	33.80
1994	7	20	0	22.50	60.00	19.80	156.70	15.20	25.87	4.56	2.30	36.05
1994	7	21	0	30.31	60.00	28.91	170.56	17.82	27.81	3.68	3.51	36.22
1994	7	22	0	18.47	60.00	17.49	190.79	21.98	30.74	4.86	4.90	36.18
1994	7	23	0	11.95	60.00	13.96	198.36	22.54	31.52	6.62	5.44	37.36
1994	7	24	0	8.74	60.00	11.43	200.00	24.30	32.98	8.27	5.20	40.43
1994	7	25	0	4.92	60.00	7.42	200.00	25.79	34.26	7.23	4.83	42.82
1994	7	26	0	54.55	60.00	30.00	200.00	25.95	34.79	5.18	8.88	39.12
1994	7	27	0	13.91	60.00	25.64	200.00	31.65	38.14	9.25	16.56	31.82
1994	7	28	0	1.79	60.00	13.19	200.00	34.44	39.95	8.47	15.36	24.92
1994	7	29	0	3.22	60.00	7.80	200.00	34.32	40.35	6.16	11.35	19.73
1994	7	30	0	6.37	60.00	8.86	200.00	33.34	40.31	4.33	6.21	17.84
1994	7	31	0	.00	57.83	4.68	200.00	32.68	40.34	2.79	5.01	15.63
1994	8	1	0	3.23	59.98	2.45	200.00	31.33	40.08	1.48	4.19	12.92
1994	8	2	0	63.02	60.00	30.00	200.00	29.72	39.69	.87	9.29	4.50
1994	8	3	0	18.01	60.00	30.00	200.00	34.09	41.29	4.82	19.79	-10.47
1994	8	4	0	1.00	59.37	15.74	200.00	36.86	42.36	10.00	19.23	-19.69
1994	8	5	0	2.83	60.00	9.45	200.00	36.77	42.49	10.23	14.19	-23.65

1994	8	6	0	.41	58.52	5.31	200.00	35.75	42.31	6.55	7.32	-24.42
1994	8	7	0	2.21	58.20	2.95	200.00	34.17	41.95	4.55	5.15	-25.02
1994	8	8	0	17.11	60.00	14.40	200.00	32.35	41.49	2.68	4.38	-26.72
1994	8	9	0	1.44	59.90	7.63	200.00	32.92	41.67	2.14	4.64	-29.23
1994	8	10	0	2.92	60.00	5.36	200.00	32.13	41.47	1.53	4.83	-32.53
1994	8	11	0	4.42	60.00	6.14	200.00	31.01	41.17	.77	4.52	-36.27
1994	8	12	0	5.58	60.00	6.88	200.00	30.17	40.94	.48	4.07	-39.86
1994	8	13	0	3.55	60.00	5.81	200.00	29.60	40.76	.41	3.93	-43.38
1994	8	14	0	6.26	60.00	8.51	200.00	28.87	40.54	.69	3.94	-46.63
1994	8	15	0	5.02	60.00	6.83	200.00	28.84	40.50	.80	4.03	-49.85
1994	8	16	0	2.10	60.00	4.54	200.00	28.43	40.36	1.05	4.02	-52.83
1994	8	17	0	11.23	60.00	11.03	200.00	27.56	40.09	.78	3.97	-56.01
1994	8	18	0	5.51	60.00	8.86	200.00	28.32	40.27	1.03	4.20	-59.18
1994	8	19	0	11.15	60.00	12.73	200.00	28.45	40.28	1.65	4.61	-62.14
1994	8	20	0	96.36	60.00	30.00	200.00	29.43	40.54	6.39	16.12	-71.87
1994	8	21	0	.96	59.14	14.67	200.00	33.79	41.79	22.37	35.03	-84.52
1994	8	22	0	5.93	60.00	10.24	200.00	34.10	41.93	24.74	28.47	-88.25
1994	8	23	0	.00	56.99	5.56	200.00	33.61	41.84	21.46	17.08	-83.87
1994	8	24	0	.00	53.00	3.00	200.00	32.24	41.50	14.03	5.02	-74.86
1994	8	25	0	.70	52.22	1.58	200.00	30.54	41.04	7.00	3.80	-71.65
1994	8	26	0	3.34	53.43	.81	200.00	28.63	40.50	3.72	3.03	-70.96
1994	8	27	0	6.81	59.18	.40	200.00	26.82	39.97	1.82	2.69	-71.84
1994	8	28	0	34.75	60.00	30.00	200.00	25.11	39.43	2.18	3.90	-73.56
1994	8	29	0	.33	58.70	13.55	200.00	31.08	41.05	4.69	6.59	-75.46
1994	8	30	0	.00	56.17	7.03	200.00	31.70	41.24	4.38	7.31	-78.39
1994	8	31	0	8.26	60.00	6.14	200.00	30.88	41.03	3.63	6.29	-81.04
1994	9	1	0	38.90	60.00	30.00	200.00	30.06	40.81	3.83	6.94	-84.15
1994	9	2	0	22.35	60.00	30.00	200.00	34.24	41.98	5.99	11.83	-90.00
1994	9	3	0	13.99	60.00	29.49	200.00	37.11	42.82	12.58	14.48	-91.90
1994	9	4	0	.20	58.37	16.45	200.00	39.02	43.40	20.27	13.32	-84.95
1994	9	5	0	2.52	58.84	9.51	200.00	38.65	43.37	19.47	9.96	-75.43
1994	9	6	0	9.69	60.00	12.91	200.00	37.39	43.09	12.29	7.43	-70.57
1994	9	7	0	258.01	60.00	30.00	200.00	36.83	42.97	20.33	42.06	-92.30
1994	9	8	0	36.08	60.00	30.00	200.00	38.98	43.51	110.92	105.37	-86.75
1994	9	9	0	3.08	60.00	19.71	200.00	40.51	43.91	73.86	89.00	-101.89
1994	9	10	0	3.64	60.00	11.93	200.00	40.36	43.90	56.92	51.47	-96.43
1994	9	11	0	1.90	60.00	7.23	200.00	39.22	43.65	43.44	11.39	-64.38
1994	9	12	0	6.83	60.00	8.44	200.00	37.59	43.27	32.67	6.29	-38.01
1994	9	13	0	4.67	60.00	6.90	200.00	36.34	42.96	20.85	5.30	-22.46
1994	9	14	0	24.53	60.00	26.10	200.00	35.00	42.62	9.99	5.40	-17.87
1994	9	15	0	10.62	60.00	23.24	200.00	37.07	43.09	6.51	6.84	-18.20
1994	9	16	0	48.08	60.00	30.00	200.00	38.15	43.35	23.10	12.93	-8.03
1994	9	17	0	5.60	60.00	21.31	200.00	39.87	43.77	19.58	21.36	-9.81
1994	9	18	0	.00	57.13	12.45	200.00	40.05	43.83	23.90	18.16	-4.07
1994	9	19	0	.00	52.06	7.27	200.00	38.88	43.57	19.49	12.33	3.09
1994	9	20	0	.00	48.54	4.19	200.00	36.98	43.12	9.09	6.08	6.10
1994	9	21	0	.00	44.35	2.36	199.82	34.70	42.57	4.67	4.55	6.22

1994	9	22	0	.00	42.02	1.29	199.55	32.50	42.00	3.29	3.53	5.98
1994	9	23	0	.00	39.87	.68	199.06	30.40	41.43	2.35	2.88	5.44
1994	9	24	0	.00	37.12	.34	198.14	28.41	40.86	2.07	2.46	5.06
1994	9	25	0	.00	34.70	.16	197.07	26.53	40.30	1.91	2.17	4.80
1994	9	26	0	.00	32.67	.07	195.99	24.77	39.74	1.69	1.97	4.52
1994	9	27	0	.00	30.42	.03	194.60	23.12	39.18	1.47	1.82	4.17
1994	9	28	0	.00	28.26	.01	193.03	21.57	38.64	1.32	1.70	3.79
1994	9	29	0	.00	26.12	.00	191.25	20.13	38.10	1.32	1.60	3.51
1994	9	30	0	.00	24.04	.00	189.27	18.78	37.56	1.28	1.51	3.28
1994	10	1	0	.00	22.19	.00	187.25	17.52	37.04	1.13	1.43	2.98
1994	10	2	0	.00	20.35	.00	185.00	16.35	36.52	1.01	1.35	2.63
1994	10	3	0	.00	18.81	.00	182.86	15.25	36.01	1.11	1.29	2.45
1994	10	4	0	.00	17.25	.00	180.46	14.23	35.50	1.13	1.22	2.36
1994	10	5	0	.00	15.86	.00	178.07	13.28	35.01	.97	1.16	2.17
1994	10	6	0	.00	14.42	.00	175.32	12.39	34.52	1.07	1.10	2.14
1994	10	7	0	.00	13.26	.00	172.85	11.56	34.03	1.07	1.05	2.16
1994	10	8	0	.00	12.55	.00	171.18	10.78	33.56	1.06	1.00	2.22
1994	10	9	0	.00	11.63	.00	168.88	10.06	33.09	1.02	.95	2.29
1994	10	10	0	.00	10.81	.00	166.68	9.39	32.62	.95	.91	2.32
1994	10	11	0	.00	10.00	.00	164.33	8.76	32.17	.94	.87	2.40
1994	10	12	0	.00	9.33	.00	162.19	8.17	31.72	.96	.83	2.53
1994	10	13	0	.00	8.70	.00	160.05	7.62	31.27	.90	.79	2.63
1994	10	14	0	.00	8.13	.00	157.99	7.11	30.83	.81	.76	2.69
1994	10	15	0	.00	7.61	.00	155.97	6.64	30.40	.77	.73	2.73
1994	10	16	0	.00	7.12	.00	153.95	6.19	29.98	.75	.70	2.78
1994	10	17	0	.00	6.60	.00	151.65	5.78	29.56	.73	.67	2.84
1994	10	18	0	.00	6.05	.00	149.07	5.39	29.14	.69	.64	2.89
1994	10	19	0	.00	5.55	.00	146.54	5.03	28.74	.61	.62	2.89
1994	10	20	0	.00	5.13	.00	144.21	4.69	28.33	.56	.59	2.86
1994	10	21	0	.00	4.80	.00	142.23	4.38	27.94	.49	.57	2.77
1994	10	22	0	.00	4.56	.00	140.75	4.08	27.55	.38	.55	2.61
1994	10	23	0	.00	4.27	.00	138.82	3.81	27.16	.39	.53	2.47
1994	10	24	0	.00	3.99	.00	136.88	3.56	26.78	.41	.51	2.37
1994	10	25	0	.00	3.74	.00	134.98	3.32	26.40	.44	.49	2.32
1994	10	26	0	.00	3.51	.00	133.21	3.09	26.03	.38	.47	2.23
1994	10	27	0	.00	3.32	.00	131.63	2.89	25.67	.35	.46	2.12
1994	10	28	0	.00	3.14	.00	130.09	2.69	25.31	.37	.44	2.05
1994	10	29	0	.00	2.95	.00	128.36	2.51	24.96	.30	.43	1.92
1994	10	30	0	.00	2.79	.00	126.85	2.34	24.61	.28	.41	1.79
1994	10	31	0	.00	2.63	.00	125.22	2.19	24.26	.27	.40	1.67
1994	11	1	0	.00	2.48	.00	123.64	2.04	23.92	.64	.39	1.92
1994	11	2	0	.00	2.32	.00	121.95	1.90	23.59	.64	.38	2.18
1994	11	3	0	.00	2.17	.00	120.15	1.78	23.26	.64	.37	2.45
1994	11	4	0	.00	2.02	.00	118.31	1.66	22.93	.62	.36	2.72
1994	11	5	0	.00	1.87	.00	116.34	1.55	22.61	.45	.35	2.83
1994	11	6	0	.00	1.71	.00	114.09	1.44	22.29	.54	.34	3.03
1994	11	7	0	.00	1.59	.00	112.30	1.35	21.98	.54	.33	3.24

1994	11	8	0	.00	1.47	.00	110.48	1.26	21.67	.51	.32	3.43
1994	11	9	0	.00	1.37	.00	108.68	1.17	21.37	.52	.31	3.64
1994	11	10	0	.00	1.27	.00	106.88	1.09	21.07	.47	.30	3.81
1994	11	11	0	.00	1.18	.00	105.16	1.02	20.78	.33	.29	3.84
1994	11	12	0	.00	1.10	.00	103.58	.95	20.49	.23	.29	3.78
1994	11	13	0	.00	1.04	.00	102.32	.89	20.20	.41	.28	3.91
1994	11	14	0	.00	.98	.00	101.07	.83	19.92	.42	.27	4.06
1994	11	15	0	.00	.92	.00	99.66	.77	19.64	.42	.27	4.21
1994	11	16	0	.00	.87	.00	98.39	.72	19.36	-999.99	.26	4.21

Summary of values (in mm)
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YEAR	MO	ND	PRECIP	E-POT	E-ACT	Runoff	Baseflow	Storage	UZTWC	UZFWC	LZTWC	LZFSC	LZFPC	ADIMC
1994														
	1	396	.00	115.50	14.07	.92	.92	26.11	.06	.00	27.14	.00	2.09	26.81
	2	424	.00	110.88	8.56	.54	.54	17.00	.01	.00	17.66	.00	1.41	17.42
	3	455	.00	169.68	7.67	.40	.40	8.93	.00	.00	9.13	.00	.91	9.00
	4	485	.00	197.82	4.41	.25	.25	4.27	.00	.00	4.22	.00	.59	4.16
	5	516	.00	216.02	2.16	.17	.17	1.94	.00	.00	1.82	.00	.38	1.79
	6	546	259.46	141.05	29.13	51.18	.62	181.10	60.00	30.00	34.12	6.57	7.15	136.21
	7	577	409.04	62.37	56.05	237.92	39.78	296.17	57.83	4.68	200.00	32.68	40.34	248.38
	8	608	324.44	61.64	55.20	265.62	66.07	299.79	60.00	6.14	200.00	30.88	41.03	255.94
	9	638	490.70	83.86	71.68	482.12	69.15	236.69	24.04	.00	189.27	18.78	37.56	209.61
	10	669	.00	128.10	76.79	23.91	23.91	135.98	2.63	.00	125.22	2.19	24.26	125.40
	11	699	.00	110.18	40.44	8.23	8.23	87.31	.39	.00	82.42	.27	15.89	81.20
	12	730	.00	85.47	21.04	4.70	4.70	61.57	.09	.00	59.29	.03	10.27	58.22