SIMULATION OF GROUND WATER RECHARGE AND MOVEMENT IN ALLUVIAL AQUIFERS ON THE BLACK MESA

by

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STATEMENT BY AUTHOR

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ABSTRACT

Alluvial aquifers associated with ephemeral washes on the Black Mesa of Arizona are an undeveloped and somewhat overlooked water resource in an arid region. A model is developed to simulate conditions on such a stream-aquifer system in order to evaluate characteristics in the natural state, as well as evaluate the effects of a strip mining operation on the watershed and through parts of the aquifer.

The model generates precipitation stochastically on an event basis. Storms are considered convective, and duration and areal distributions are calculated. The watershed model computes hydrographs from rainfall on several subwatersheds and routes and adds these outputs in the correct sequence to determine total hydrographs at selected cross-sections. Runoff is determined through use of SCS curve numbers. Recharge is modeled by the Green and Ampt infiltration formula and a one dimensional form of the differential ground water equation. A ground water model then determines head fluctuations between recharge events.

Three runs simulating natural, actual post-mining, and hypothetical post-mining conditions show few adverse effects to the stream-aquifer system if large portions of the aquifer are not overturned. Examination of the natural conditions shows evapotranspiration dominating drainage losses from the alluvial aquifer.

CHAPTER 1

INTRODUCTION

Major drainages on the Black Mesa in northeastern Arizona are associated with alluvial deposits which represent a potential water resource in this arid region. The initiation of extensive coal strip mining on the Black Mesa has threatened to disturb the natural state of these alluvial aquifers as well as the runoff characteristics of the drainage basins. Runoff, in turn, recharges these aquifers through infiltration into the sandy wash bottoms. The purpose of this study is to develop a model that quantifies the rainfall, runoff, recharge, and ground water system in order to determine the effects of mining and characteristics of the natural system as a water resource.

To this point in time, exploitation of these ground water reservoirs has been limited to use by livestock at a very few, hand-dug wells and at points where the water table intersects the channel bottom of the wash. However, the possibility of more extensive use of these aquifers at some future time (possibly as a water source for irrigation of reclaimed mine spoils) calls for an evaluation before disturbances take place.

The watershed chosen as a case study was that drained by Coal Mine Wash and is depicted in Figure 1. It is one of several streams that drain southwestward, away from the Mesa's rim. Although the associated alluvial deposits are not over 400 feet wide at any place, they

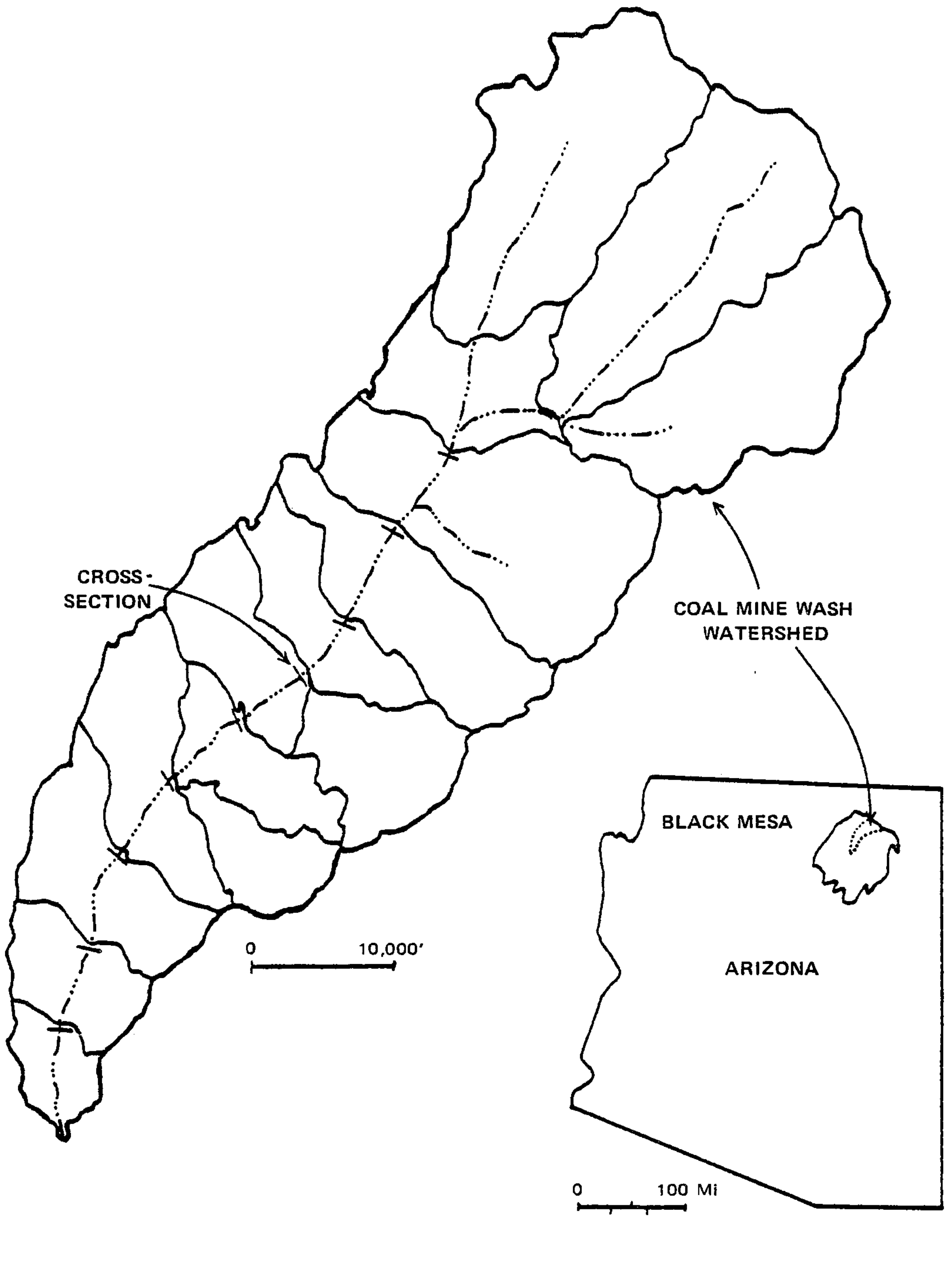

Figure 1. Location and maps of the Black Mesa and Coal Mine Wash Watershed.

are associated with 9 miles of the stream. Several observation wells have shown the aquifer to be between 5 and 25 feet deep. At a specific yield of 0.28, the volume of water held in Coal Mine Wash alone would be approximately 1400 acre feet. Intuitively, in a region such as the Black Mesa, which gets 9-13 inches of rain per year, the value of this amount of water would seem high. The significance of this amount of water would increase if the water now used in the mining operation depletes the supply of deep water or if water becomes needed for uses other than livestock watering and domestic consumption. Such would be the case if irrigation was to be practiced along the washes. The length of the alluvial deposits makes them an accessible water resource to many Navajo Indians living in higher, more remote portions of the Black Mesa. It should also be noted that, besides the deep wells drilled by the mining operation, the water supply along these washes is the only major, dependable source readily available on the Black Mesa. Water quality analysis indicates a soluble salt content around 2000 ppm, which is suitable for livestock and irrigation if the proper leaching methods are used. It seems, therefore, that before permanent changes are made in the hydrologic characteristics of such stream-aquifer systems, their effects should be evaluated to prevent unintentional, permanent destruction of a potential future resource.

The Black Mesa is a prominent physiographic feature covering over 2500 square miles on the Colorado Plateau (see Figure 1). The highest point is over 8100 feet, and the lowest point of the 44.1 square mile Coal Mine Wash Watershed is 6500 feet. Ponderosa Pine is the dominant vegetation type above 7500 feet, and Pinyon-Juniper dominate below. Grazing and strip mining are now the two most important land uses.

These conditions differ from those of other studies examining recharge to alluvial systems through wash bottoms. Several authors have examined ground water recharge, infiltration, transmission losses and the like in the more heavily populated areas of southeastern Arizona where the presence of several gauging stations has provided data for analysis. Burkham (1970) measured the inflow and outflow of several reaches of alluvial channels in the Tucson Basin and developed an empirical equation for infiltration loss as a function of inflow. Anderson (1972), using an electrical analog model and the steady state ground water levels of the Tucson Basin of 1940, determined actual recharge per mile of channel per year by assuming inflow to the basin equaled outflow at that time. Matlock (1965) performed flume studies with streambed samples from washes in the Tucson area to evaluate infiltration as affected by sediment load and flow velocity. Infiltration losses in the flume study correlated well with observed losses in washes of the Tucson Basin. Many studies, such as those done by Babcock and Cushing (1942), Keppel and Renard (1962), and Sharp and Saxton (1962), evaluated transmission losses of flood events between gauging stations to determine recharge or empirical relationships between recharge and flow. Models of alluvial aquifers generally require that the actual recharge volume or rate be given them (Qazi and Danielson 1974; Neuman and Witherspoon 1970) and simply distribute this known amount. A study done by Besbes, Delhomme, and de Marsily (1978) in Tunisia used observation wells to develop a model for determination of recharge. The authors related the impulse of a flood event to the response of the observation wells. Model calibration and measured recharge from an actual flood event were both used to determine this relationship.

On the major drainages of the Black Mesa, none of the above methods of recharge estimation could be used because the lack of ground water and streamflow data prevented either calibration or direct measurement. It was also desired that the operation of the stream-aquifer system be simulated in time instead of simply obtaining average values over long periods of time for recharge, ground water supply, runoff events, etc. In this manner, characteristics of the system and hydrologic relationships can more easily be observed. These requirements made modeling of the actual processes involved a logical alternative.

The random nature of the convective, runoff-producing storms in the Southwest lends itself to stochastic modeling. Since the areas covered by convective storms are considerably smaller than the watersheds to be modeled, spatial thunderstorm characteristics were considered. Relationships describing storm center maximums, developed by Fogel and Duckstein (1969), were used to describe rainfall events. Time between events and storm center maximums are generated stochastically and all other characteristics of precipitation are treated deterministically.

Watershed data on the Black Mesa is absent, as well as many places on the Colorado Plateau. Therefore, a watershed model which only needs parameters of basin geometry was desired. Such a model is HYMO, developed by Williams and Hann (1973) of the Agricultural

Research Service. This watershed model only requires parameters derivable from a topographic map, cross-sectional surveys of the stream channel, and Soil Conservation Service curve numbers to estimate infiltration.

The infiltration-recharge model considers depth of flow and aquifer characteristics such as permeability, storage coefficient, and soil moisture curves. The ground water model uses only Darcy's Law and the continuity equation, and consequently needs only permeability, specific yield, porosity, and values describing the initial conditions and aquifer configuration.

The following pages relate the structure and use of program ALAMO. ALAMO is an acronym for Alluvial Aquifer Model.

CHAPTER 2

MODEL COMPONENTS AND OPERATION

This chapter describes the actual equations, procedures, and algorithms used to model the various components of program ALAMO. The actual program is listed in Appendix B. The reader should continually refer to Appendix B while following the narrative description in Chapter 2.

Model Control and General Operation

The generalized flowchart in Figure 2 illustrates the sequence of component execution. After all parameters are read and variables initialized, the first summer season begins and control is given to subroutine PRECIP. PRECIP determines interarrival times and then calls GRDWTR to evaluate head changes in the aquifer during these periods. At the end of the summer season, the 242 day winter season is simply treated as an extended interarrival time. GRDWTR then returns control to PRECIP for calculation of precipitation depth and duration and a check for the capacity of the event to produce runoff. If runoff is possible, control is returned to the main program, ALAMO, to determine which subwatersheds produce runoff and to call the subroutines CMPHYD, ROUTE, ADHYD, and INFIL in the correct order to determine the hydrographs and recharge at known cross-sections. These cross-sections are the points where flow depths have been determined for given discharges and are assumed to represent reaches of the stream aquifer system.

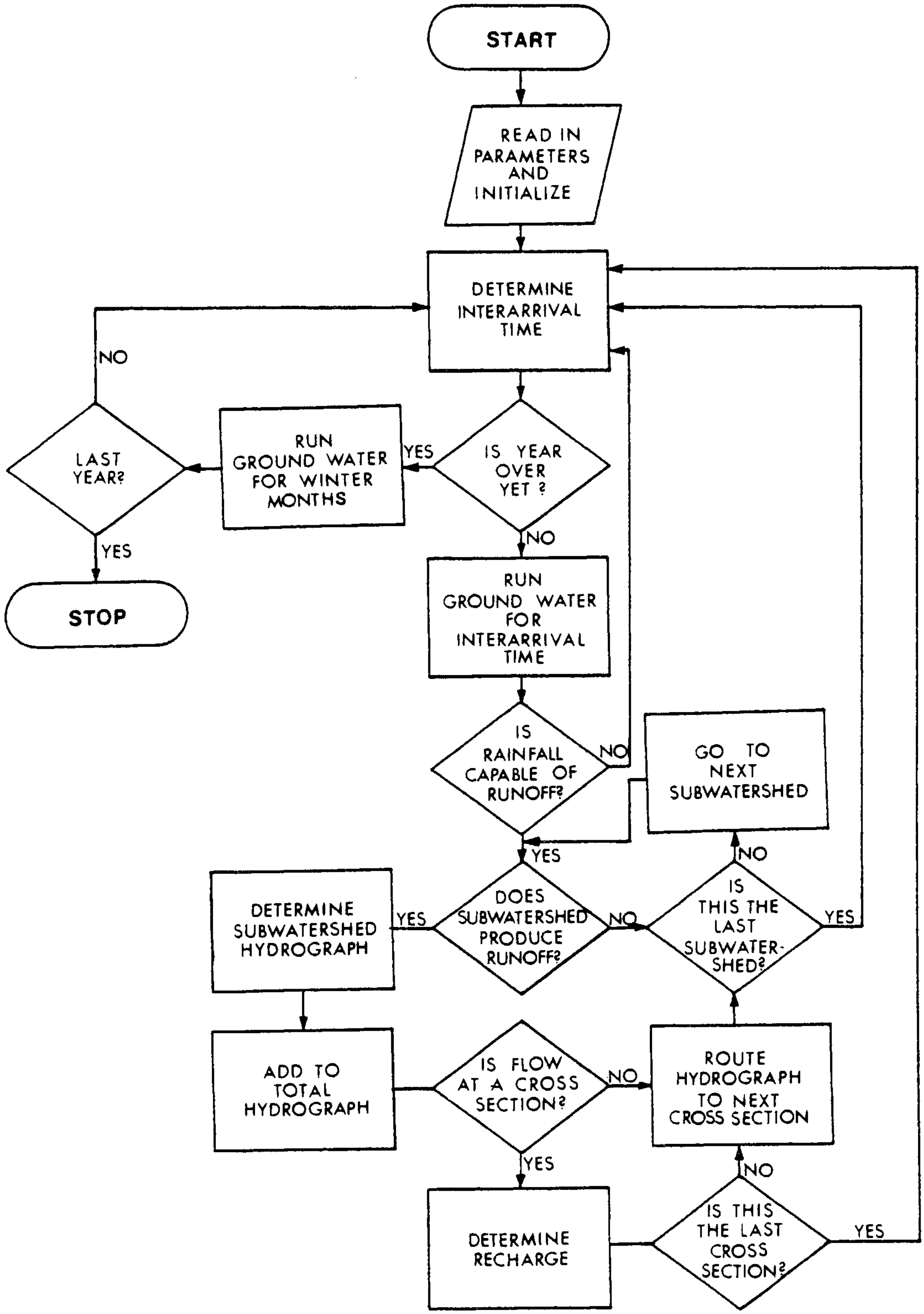

Figure 2. Generalized flowchart of program ALAMO.

Once it has been determined that runoff occurs on a given subwatershed, the hydrograph is determined and recharge will occur at all cross-sections below it. The original hydrograph is routed downstream to the next cross-section and any hydrographs from subwatersheds along the way are added to it. Once the hydrograph and recharge at the lowest cross-section have been determined, control is returned to PRECIP to begin the entire sequence again with the new heads determined by subroutine INFIL.

The check in PRECIP for storm center maximums which could not possibly cause runoff on any subwatershed prevents unnecessary hydrograph computation. This depth can be determined by taking the lowest curve number on the watershed and computing the soil moisture deficit by the SCS formula:

$$.2[(1000/CN) - 10] = SMD \qquad (1)$$

where CN = curve number, and

SMD = soil moisture deficit in inches.

If the storm center maximum is below this required amount, the next interarrival time and rainfall amount is determined without return of control to the main program. When a runoff event is possible and control is returned to the main program, checks are again provided to see if runoff will occur on each individual subwatershed. If no runoff is possible on the subwatershed in question, runoff is set to 0 and considerable computer time is saved in hydrograph computation.

Total flow at a cross-section is determined by adding and routing flows from the subwatersheds in their correct order. These functions are carried out in the ADHYD and ROUTE subroutines. ADHYD checks to see if flow is occurring above the portion of aquifer being modeled. If flow does occur here, the upper storage reservoir (see Ground Water section of this chapter), representing all indurated alluvium above this point, is filled.

Program ALAMO reads in all parameters and data needed for all of the other subroutines. These parameters and data requirements are discussed individually in the section on model components and the section on parameter estimation. ALAMO is set up to receive any number of data cards for the ground water routine. To indicate the end of junction data and channel data, cards with 251 and 351 in the far left are inserted after each set of data, respectively, to indicate the last data card of the set (see Ground Water section of this chapter for definitions of "channels" and "junctions").

Model Components

Precipitation

Precipitation is generated stochastically as input into the rest of the model. Virtually all runoff events on the Black Mesa occur as a result of intense summer convective storms. Frontal storms which occur in the winter months are usually of long duration and consequently all precipitation is absorbed by the soil. This is somewhat borne out by the winter of 1977-78, when an unusual amount of rain produced little, if any, runoff on the larger watersheds of the Black Mesa. Consequently, only summer storms are simulated.

Work done by Fogel and Duckstein (1969, 1970) dealing with convective storms describes many of the characteristics of storm center maximums and areal distributions of convective storms in the Southwest. Several distributions were obtained from the highly instrumented Atterbury Watershed near Tucson, Arizona. Since mean annual rainfall and precipitation characteristics at the Black Mesa and Tucson appear to be similar, and because there is no data describing areal distributions of precipitation on or near the Black Mesa, most of the distributions of Fogel and Duckstein have been used. These distributions are discussed below.

An event based approach is used to simulate precipitation. Interarrival times are described by a geometric distribution. This distribution is generally used to describe the probability of a given number of Bernoulli trials before a "success" occurs. A Bernoulli trial has only two possible outcomes ("success" and "failure") with the probabilities of each being fixed for all such trials, and the outcome of any trial is completely independent of another. In this case, each day is a Bernoulli trial with a success occurring when a storm is centered on or near the watershed. Consequently, it is required that an event on any day in no way affects the probability of an event occurring on the next day. Since atmospheric conditions are somewhat cyclic, the precipitation events of the summer season on the Black Mesa are slightly bunched. Although this slight bunching does not allow complete independence of rainfall events, probabilities conditioned on the previous day's outcome appear to be close enough to unconditioned probabilities to assume independence of events for the purposes of this model. A Kolmogorov-Smirnov test for goodness of fit between point rainfall data from Betatakin, Arizona (a nearby station at a similar elevation) and the geometric distribution indicates a slight underestimation of the number of one and two day, as well as very long, interarrival times. These slight deviations will tend to average out and not significantly affect the amount of recharge available to the alluvial aquifer, which is the purpose of this simulation. Actual fitting of data to a distribution is not possible since storm centers are being considered and not rainfall at a point. More than one storm per day is not allowed.

After the interarrival time has been determined, GRDWTR is called to calculate head fluctuations for that amount of time. Control is then returned to PRECIP and the location of a storm center is determined. A storm is randomly centered on any point of a grid placed on and several miles around the basin. The grid is in units of one mile and defines a coordinate system.

Next, the depth of precipitation at the storm center is determined stochastically using a two parameter Gumbel distribution, which was also fit to the Tucson data. At this point is the check for runoff potential, described in the section of this chapter on Model Control and General Operation.

The depth of rainfall at the center of each subwatershed is obtained using a deterministic relationship between depth and distance from storm center. First, the distance from storm center is determined from the coordinates of the storm center and the subwatershed center in question. This distance is then used in the equation below (Fogel and Duckstein 1969) to determine depth:

$$DEPTH = CMAX(e^{-\pi DIS^2 b})$$

where $b = .27e^{(-.65CMAX)}$,

DIS = distance from storm center to a subwatershed,

CMAX = maximum center depth, and

DEPTH = rainfall depth at water of a subwatershed.

The above relationship assumes a circular rainfall pattern, with rainfall decreasing toward the outside as shown in Figure 3.

The duration of rainfall is also obtained deterministically from an equation (Fogel and Duckstein 1970), which is a product of least squares analysis done on the Atterbury data:

$$R = 2.42\log D + 1.89$$

where R = rainfall depth in inches, and

D = duration of 90 percent of the total storm depth measured from the mass center of the storm.

The remaining 10 percent of the rainfall depth is assumed to occur in a 10 minute interval at the first of the storm. The duration is then broken up into 10 minute intervals. The rainfall intensities for each interval are equal, but they must be determined because of the requirements for CMPHYD.

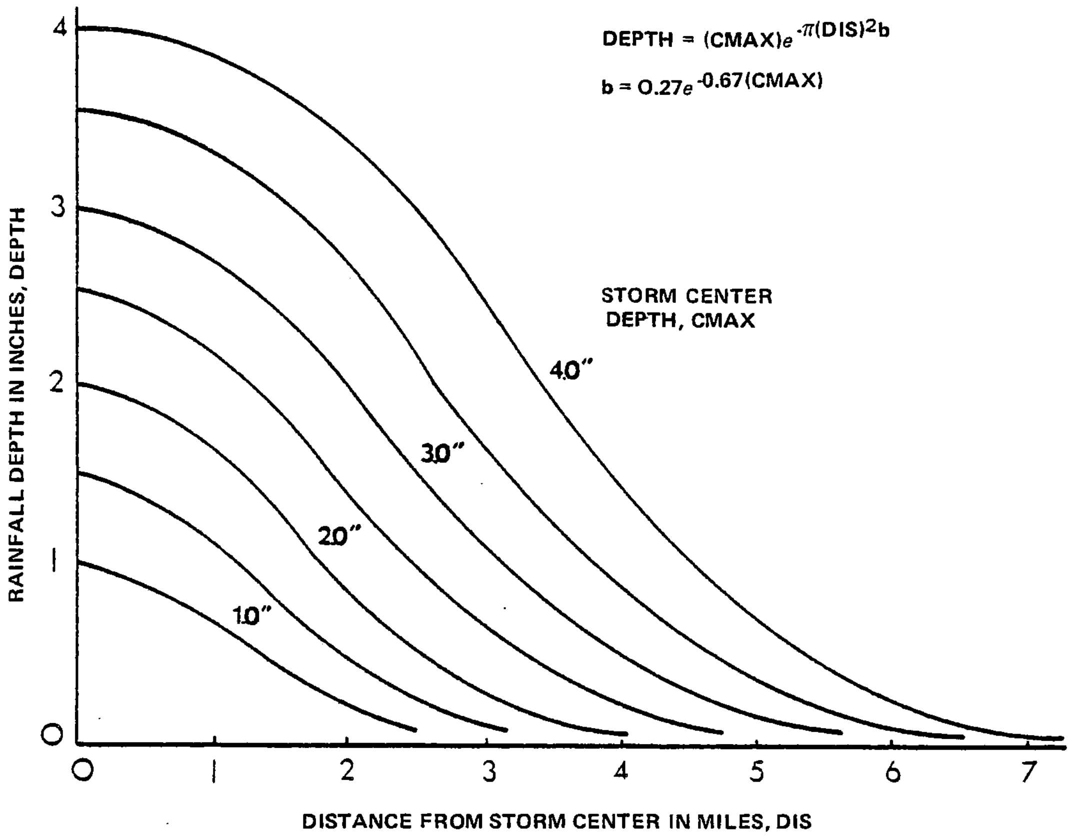

Figure 3. Spatial distribution model of convective storm rainfall. -- (Fogel and Duckstein 1969)

At the end of PRECIP the maximum rainfall depth is determined for each storm from all the subwatershed centers. If this maximum would not produce runoff on the subwatershed with the highest curve number as expressed in Equation (1), the next interarrival time and depths are determined without returning control to the main program.

Runoff

Runoff simulation is accomplished through use of selected routines in the HYMO watershed model. For detailed information on the CMPHYD, ADHYD, and ROUTE subroutines, one is referred to the HYMO user's manual (Williams and Hann 1973). HYMO was chosen because it requires no historical flow record (although such records could be used), it was believed to have acceptable accuracy, and it was readily adaptable to the reception of spotty rainfall, typical of the arid Southwest. Since in the summer, convective storms commonly cover only 10 square miles, it is impossible to extrapolate point rainfall over large watersheds. For this reason, large watersheds are subdivided into subwatersheds, which are small enough to assume an even areal distribution of rainfall in depth and time. Hydrographs are computed from all of the subwatersheds and then routed and added in the correct sequence to obtain the final hydrograph from the whole watershed.

Hydrograph Computation. A unit hydrograph is computed for each subwatershed from basin geometry. Unit graphs are divided into three parts for computation (Figure 4). From the beginning of rise to the inflection point, t_0, the hydrograph is computed by the two parameter gamma distribution equation:

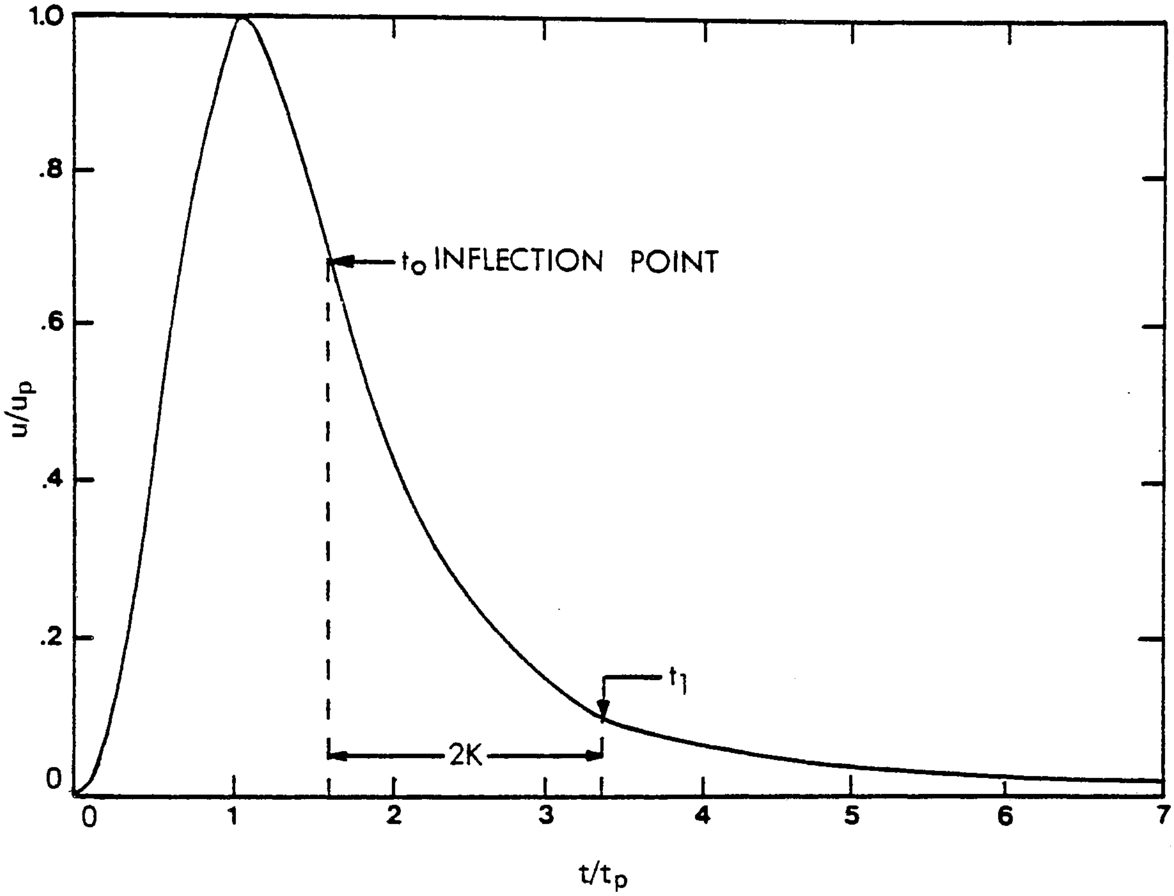

Figure 4. Dimensionless unit hydrograph used in subroutine CMPHYD. -- (Williams and Hann 1973)

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

where q = flow rate in cubic feet per second at time t,

q_p = peak flow rate in cubic feet per second,

t_p = time to peak in hours, and

n = dimensionless parameter.

From t_0 to t_1 ($t_1 = t_0 + 2K$) the hydrograph is computed by the recession depletion equation:

$$q = q_0 e^{\frac{t_0 - t}{K}}$$

where q_0 = flow rate at the inflection point,

t_0 = time at the inflection point, and

K = recession constant in hours.

From t_1 to ∞ the recession depletion equation becomes:

$$q = q_1 e^{\frac{t_0 - t}{K_1}}$$

where q_1 = flow rate at t_1, and

$K_1 = 3K$ = second recession constant.

The dimensionless shape parameter, n, is a function of K/t_p, as shown in Figure 5. In the actual program, n is computed through an iterative technique if the variable XNSUB is given a negative value through a call statement parameter. Since this parameter is a constant

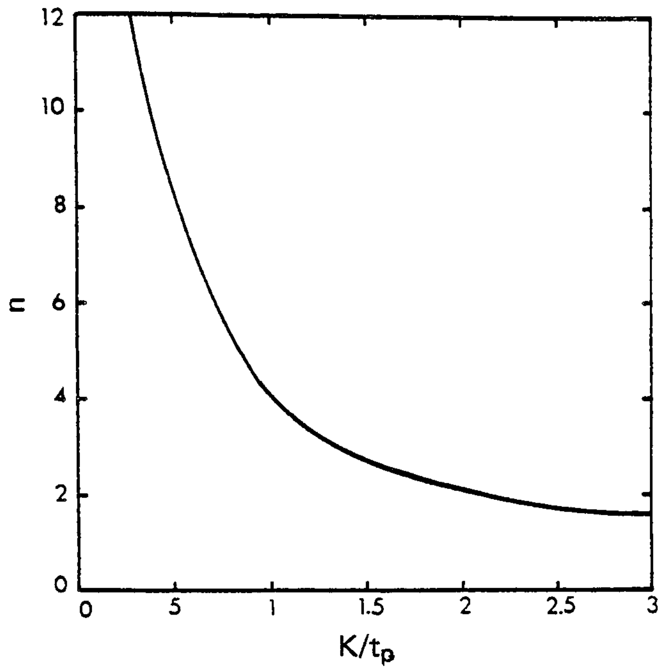

Figure 5. Relationship between dimensionless shape parameter (n) and recession constant (k)/time peak (t_p). -- (Williams and Hann 1973)

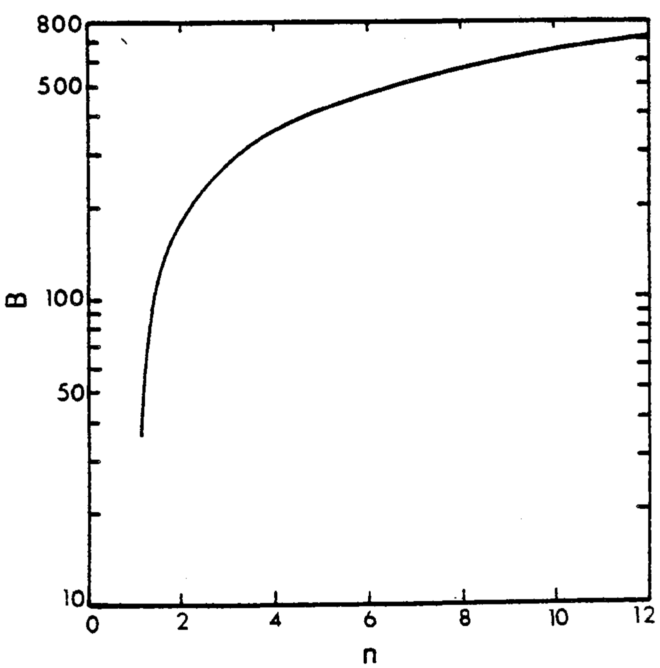

Figure 6. Relationship between dimensionless shape parameter (n) and watershed parameter (B). -- (Williams and Hann 1973)

for the watershed, it need not be computed more than once. Once computed, n is substituted into the parameter list of the call statement for the XNSUB value instead of the negative value. CMPHYD will then use this value of n instead of going through the iterative calculation which requires considerable computer time. The peak flow rate is computed by the equation:

$$q_p = \frac{BAQ}{t_p}$$

where B = a watershed parameter, a function of n as shown in Figure 6,

A = watershed area in square miles, and

Q = volume of runoff in inches.

Therefore, the entire unit hydrograph can be computed if K and t_p are known. K and t_p can be determined by hydrograph analysis (Williams 1969). For ungauged watersheds, however, K and t_p are computed using the following equations:

$$K = 27.0A^{0.231}SLP^{-0.777}(L/W)^{0.124}$$

$$tp = 4.63A^{0.422}SLP^{-0.46}(L/W)^{0.133}$$

where SLP = difference in elevation in feet, divided by flood-plain distance in miles, between watershed outlet and most distant point on the watershed, and

L/W = watershed length-width ratio.

Excess precipitation is calculated using the Soil Conservation Service rainfall-runoff relationship, which is expressed in the form of "curve numbers." Curve numbers range from 0-100 and determine the retention capacity of the watershed by the familiar formula:

$$R1 = 1000/CN-10$$

where R1 = potential maximum retention in inches.

Before any runoff occurs, initial abstractions (infiltration, interception, and surface storage) must be subtracted and are a fraction of the potential retention which has been empirically approximated by:

$$B1 = .2R1.$$

Runoff volume is then calculated by:

$$Q2 = \frac{(RAIN - B1)}{RAIN + .8(R1)} \quad . \qquad (2)$$

RAIN is supplied to Equation (2) as a cumulative value after a known amount of equal time increments have occurred. Therefore, for each time increment, the previous cumulative runoff volume is subtracted to get an incremental runoff value. These incremental runoff values are multiplied times the unit graph and then added to determine the complete hydrograph. The SCS National Engineering Handbook (United States Department of Agriculture 1969) should be used to determine appropriate curve numbers.

<u>Routing</u>. Since runoff is calculated on subwatersheds, consequent hydrographs have to be routed and added to obtain the final

outflow. The routing procedure also comes from the HYMO watershed model and has been called the "Variable Storage Coefficient (VSC) Flood Routing Method" (Williams 1969; Williams and Hann 1973). This method was chosen because it requires little computer time, is free of convergence problems, and is sufficiently accurate. The VSC routing equations are:

$$O_2 = C_2[I_a + (\frac{1}{C_1} - 1)O_1] \tag{3}$$

$$C_2 = \frac{2\Delta t}{2T_2 + \Delta t} \tag{4}$$

$$C_1 = \frac{2\Delta t}{2T_1 + \Delta t} \tag{5}$$

$$T_1 = [\frac{L}{1800(V_{I_1} + V_{O_1})}] \times [\frac{LxSLP_O}{LxSLP_O + D_{I_1} - D_{O_1}}]^{1/2} \tag{6}$$

$$T_2 = [\frac{L}{1800(V_{I_2} + V_{O_2})}] \times [\frac{L+SLP_O}{LxSLP_O + D_{I_2} - D_{O_2}}]^{1/2} \tag{7}$$

In these equations, subscripts 1 and 2 refer to the beginning and end of the time interval Δt and the units are cubic feet/second for flow, hours for time, feet/second for velocity, and feet for length and depth. The symbols are defined as follows:

I = inflow rate,

O = outflow rate,

$I_a = \frac{I_1+I_2}{2}$ = average inflow rate,

C = storage coefficient,

T = travel time through the reach,

L = reach length,

V = velocity,

SLP_0 = normal slope, and

D = depth.

For a rigorous derivation of these equations, one is referred to Williams (1969).

Since T_2 and C_2 are dependent upon O_2, an iterative technique is required to solve the routing equations. In Equation (3), I_a and O_1 are known, and C_1 can be computed from Equation (5). This leaves only O_2 and C_2 as unknowns. O_1 can be used as a first approximation of O_2. The normal depth and velocity for the approximate value of O_2 are entered into Equation (7) for computing T_2. Then Equation (4) is used to compute C_2. The second approximation of O_2 is then obtained from Equation (3). This iterative process continues until the difference between successive O_2 values is acceptable. ROUTE is set to accept differences of 0.1 percent or less. Usually about four iterations are required.

Because the VSC method takes into account changing travel times between cross-sections with changing discharge (which is the most uniquely desirable characteristic of the VSC method), it requires computation of travel time tables. A separate program for generation of this table is included in Appendix C. Travel time, T, is defined by the following relationship:

$$T = \frac{S}{q}$$

where S = storage volume in the reach, and

q = discharge rate.

S and q are determined from rating curves in the reach. Another separate routine in Appendix C computes the stage-discharge-cross-sectional area relationship using the Manning Equation and points on the cross-section. Both programs mentioned above are modified versions of subroutines found in the HYMO watershed model (Williams and Hann 1973) and are described therein.

Aquifer Recharge

In the model it is assumed that all recharge is from seepage through the channel bottom during a flow event, and that any lateral contribution of ground water from the watershed is negligible. This assumption is likely valid, since the high evapotranspiration demands and infrequent rainfall in northeastern Arizona keep soil moisture deficits so high that very little deep percolation can occur from a rainstorm.

Aquifer recharge is accomplished in subroutine INFIL. Seepage into the aquifer is a two step process. First, water infiltrates through the unsaturated portion of aquifer underneath the streambed until the wetting front meets the water table. At this point further recharge occurs through "spreading" of the ground water ridge below the channel (Figure 7).

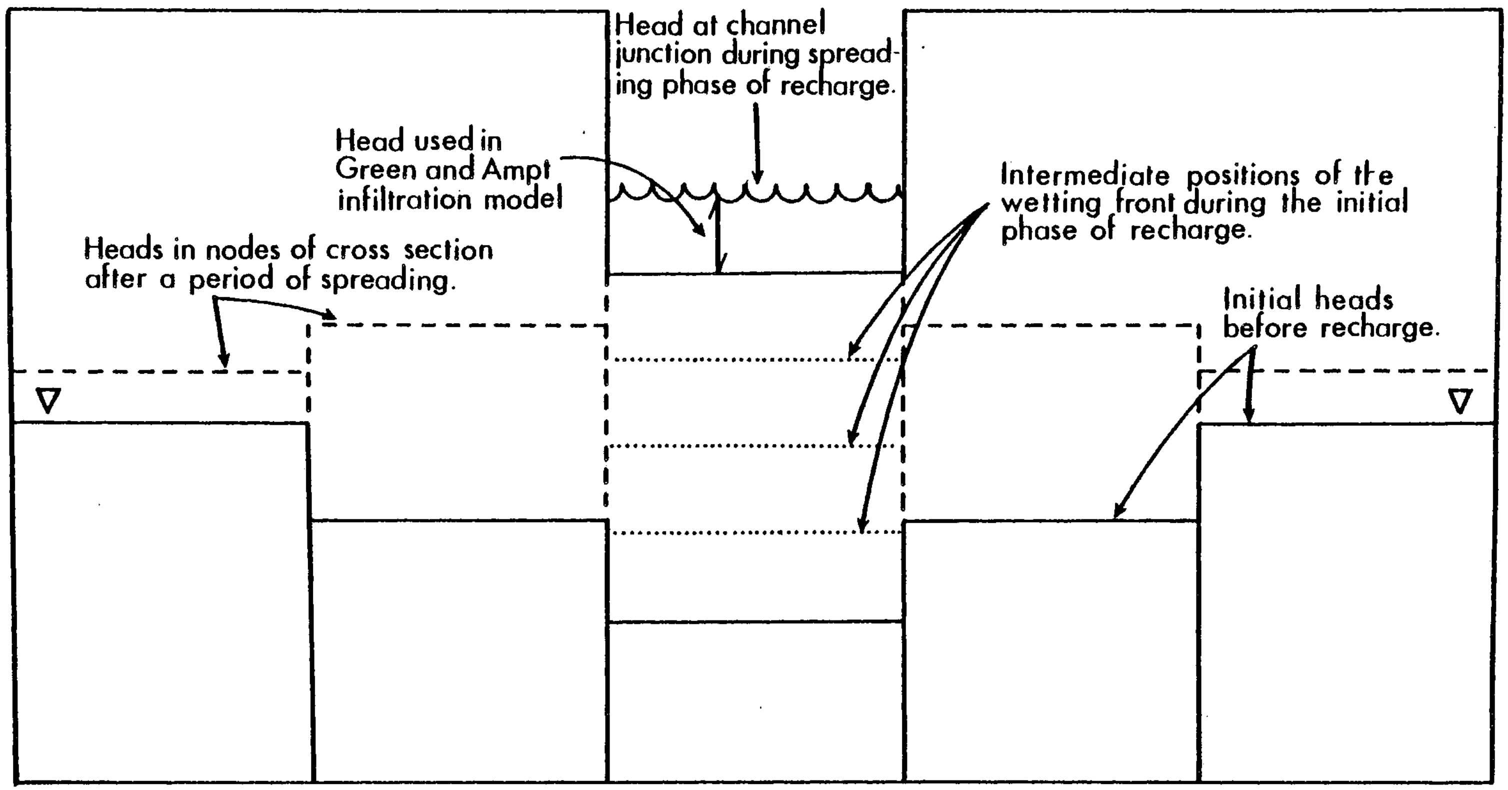

Figure 7. Computer conception of a typical cross-section of aquifer during a recharge event.

The initial infiltration is described by the Green and Ampt equation:

$$i = ECOND\left(\frac{FLOWDP + H_f}{WFRONT}\right) + 1$$

where i = infiltration rate,

ECOND = effective conductivity of the transmission zone,

FLOWDP = head at the ground surface,

H_f = suction head at the wetting front (a positive value), and

WFRONT = distance from ground surface to the wetting front.

This model has the advantage of being able to account for positive heads (depth of flow) at the ground surface (channel bottom). The major difficulty in the Green and Ampt model is determination of pressure head at the wetting front. For the purposes of the recharge model, it is always assumed that head in the unsaturated portion of the aquifer is always in equilibrium with the water table. The difference in pressure head between the channel bottom and the wetting front is the driving force for infiltration. Effective hydraulic conductivity of the transmission zone above the wetting front is estimated as 85 percent of the saturated hydraulic conductivity of the aquifer. In actual program operation, the unsaturated zone is divided into 0.1 foot intervals and the time needed to fill these intervals is calculated. In this manner, changing infiltration rates, flow depths, time to complete saturation, and unfilled pore volume can be measured. From soil moisture curves for channel bottom material, percent saturation at a known suction can be determined. This amount determines how much of the depth increment in question is to be filled, and therefore partially controls the time required to fill it. As soil characteristics vary radically between events and within short segments of the stream channel, a typical soil moisture curve for sand was chosen (Figure 8) to represent the channel sediments.

During the water spreading stage a finite difference, one dimensional form of the nonsteady state ground water equation (Walton 1970) was used:

$$\frac{\partial^2 h}{\partial x^2} + \cancelto{0}{\frac{\partial^2 h}{\partial y^2}} + \cancelto{0}{\frac{\partial^2 h}{\partial z^2}} = \frac{S}{T}\frac{\partial h}{\partial t}$$

or in simplified finite difference form:

$$\Delta h = (T\Delta t)\,\frac{(h_1 - h_2) - (h_2 - h_3)}{\Delta x^2 (S)}$$

where h = head,

S = storage coefficient,

T = transmissivity,

Δt = time interval, and

Δx = distance interval.

During recharge, ground water flow downstream is assumed negligible compared to the component normal to the channel. This assumption allows a much simpler, one dimensional computation of ground water spreading during recharge. Once the recharge event is over, the two dimensional ground water model resumes operation. In INFIL, Δt equals

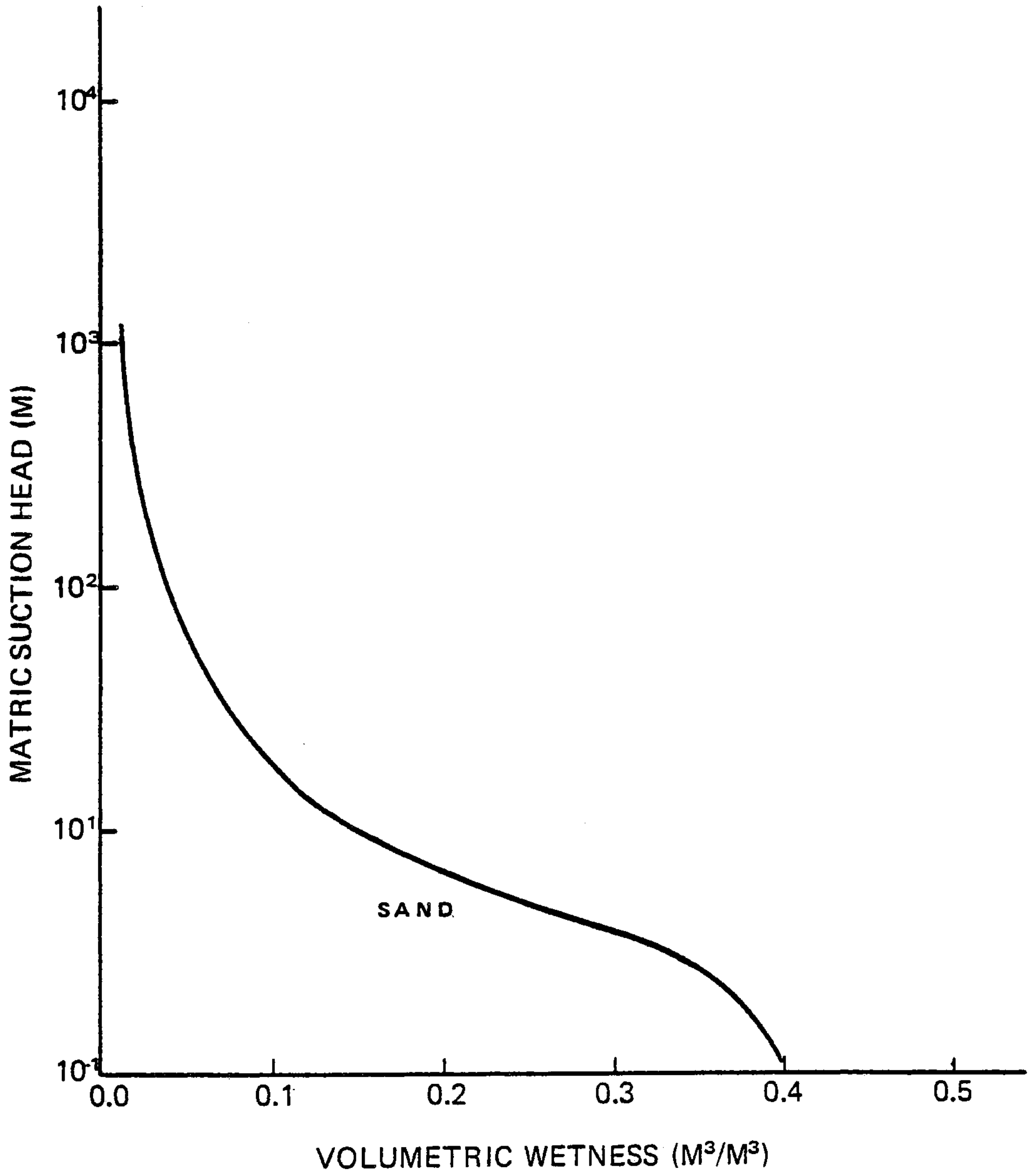

Figure 8. Typical soil moisture curve for sand used in subroutine INFIL. -- (Hillel 1977)

the time increment used in the hydrograph from which flow depth is being calculated.

Recharge is calculated at the several valley cross-sections where rating curves have been calculated. The point in the cross-section from which recharge occurs can be placed at any one of the node points (aquifer junctions) in the cross-section. Although the aquifer junctions used in the ground water model are of flexible size, the cross-sections should be approximately even in spacing because the cross-sections define the location of aquifer junctions. The channel is assumed to be exactly one junction wide (DX). Therefore, it is recommended that DX be determined by:

$$DX = W_{AC}(S)$$

where DX = cross-sectional distance interval for GRDWTR and INFIL,

W_{AC} = average channel width, and

S = sinuosity of stream (channel length/floodplain length).

Water leaving streamflow and entering the groundwater system is not subtracted from outflow hydrographs. Transmission losses are assumed negligible.

Ground Water

A ground water model for nonartesian systems has been developed which uses only Darcy's Law and the continuity equation (Crim 1972).

The aquifer is sectioned off into rectangles called "junctions" which are connected by imaginary "channels." Water flows through the channels according to Darcy's Law and all inflows and outflows to a junction are summed and head at a junction is calculated according to the continuity equation:

$$\frac{dV}{dt} = \Sigma Q \text{ in} - \Sigma Q \text{ out}$$

or in finite difference form:

$$\Delta H_t = \frac{(\Sigma Q \text{ in} - \Sigma Q \text{ out})\ \Delta t}{A_s}$$

where $V = H(A_s)$ = water volume in a junction,

ΔH_t = change in head,

A_s = "surface area" of a junction, and

Δt = time increment.

Than a new head at a junction is calculated:

$$H_t = H_{t-1} + \Delta H_t \quad .$$

Certain characteristics are defined as belonging to junctions and others belonging to channels:

Junction characteristics:

head,

"surface area," and

porosity;

Channel characteristics:

flow,

cross-sectional area,

permeability,

depth,

length, and

width.

Of all the variables above, only surface area, channel length, and width are constant in time. The term "surface area" of a junction has been set off in quotes above because it is not the actual surface area of a junction. The concept of junctions and channels is illustrated in Figures 9 and 10. It should be noted that channels overlap junctions and overlap each other. The "surface area" of a junction, then, is determined by computing all the areas of overlap of the junction and its individual contributing channels and summing these areas.

A master network chart illustrating the relative positions of all channels and junctions is very useful in preparation of data. Such a chart is illustrated in Figure 10. All such charts should be numbered in a fashion similar to Figure 10 as execution of the model is impossible if the numbering sequence of channels and junctions is not consistent.

Since many data cards are required to define the network and mistakes are hard to find, a modification of a routine used by Crim (1972) to check the compatibility and consistency of connecting channels and junctions is provided in Appendix C.

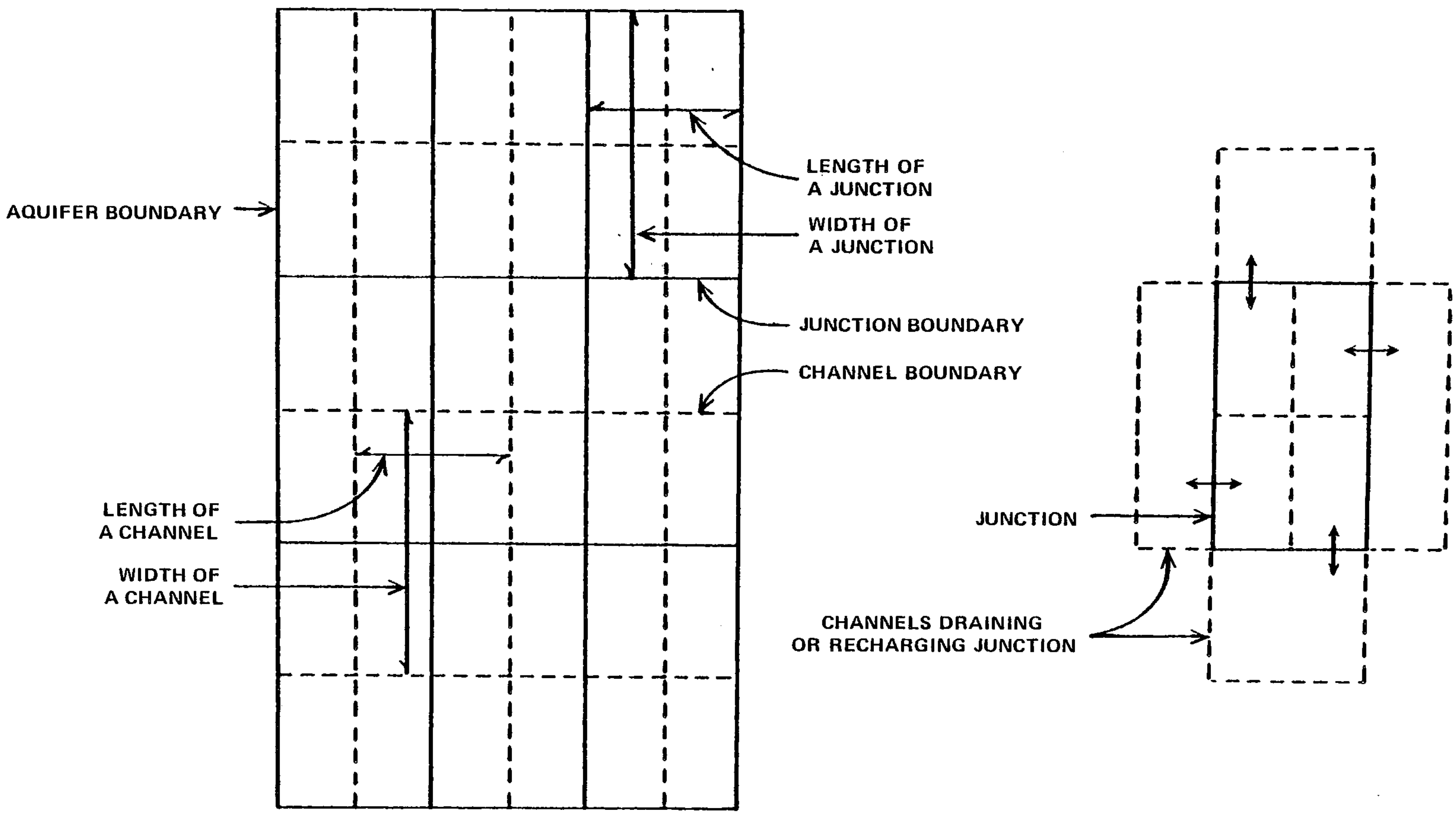

Figure 9. Illustration of a single interior junction and its four contributing channels, and a whole network of channels overlaying junctions as viewed from the top.

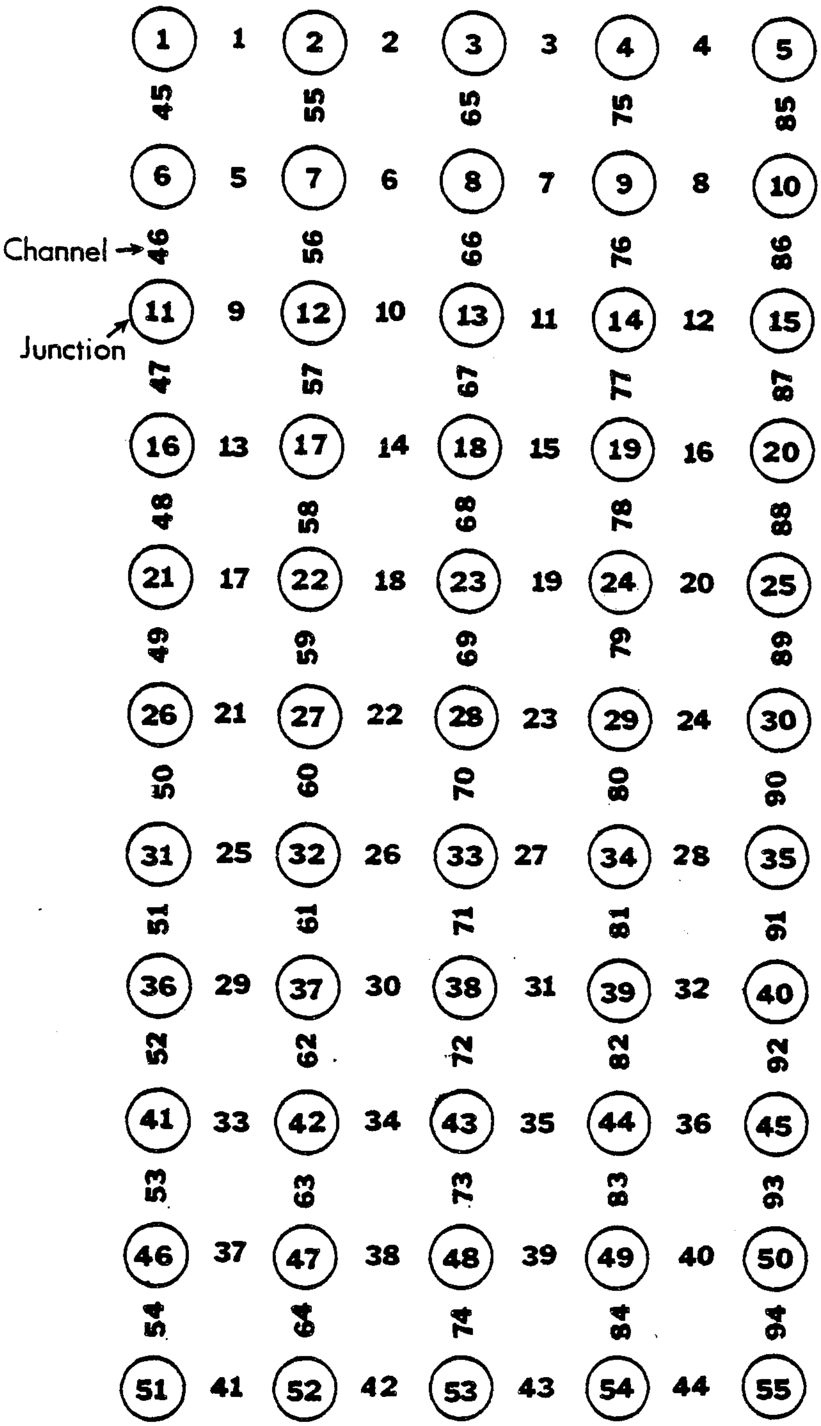

Figure 10. Master network used in subroutine GRDWTR showing required form for layout of channels and junctions.

The variables listed above that are not constant in time vary according to the following algorithm:

Start:

1. Compute half-step velocities and flows for all channels.
2. Compute half-step values of heads for all junctions.
3. Compute half-step cross-sections for all channels.
4. Compute full-step velocities and flows for each channel.
5. Compute full-step heads at each junction.
6. Compute full-step cross sections and depths in each channel.
7. Compute full-step surface areas at each junction.

Certain comments can be made for clarification:

1. Depth and surface area are computed only on the full-step.

2. A trap in the program sets the velocity and flow of a channel to zero when its depth becomes less than 0.5 feet.

3. The run will abort if any velocity exceeds 20 feet per second in any channel.

4. The full-step value of time is presently 24 hours. Additional experience may call for more or less time as the problem may require.

5. Certain heads may be held constant by assigning a negative value to the junction number at that point. Computation for these is bypassed.

It should be known that positive flow and velocity in a channel is defined as flow from a low numbered junction to a higher numbered junction. Negative flow is the opposite.

Instability may occur if a large time step is used on a small network. Conversely, excessive computer time may be required when using a small step on a large network.

For any ground water model boundary conditions must be determined. The side boundaries are simply assumed to be impermeable, but the upper and lower boundaries require some discussion. Above the aquifer segment being modeled are more, much less extensive alluvial deposits. These deposits do store and supply some water to the aquifer segment modeled. To account for this water, another ground water reservoir is attached to the upper end (see Figure 11). This upper reservoir is subsequently referred to as the imaginary aquifer segment since it in no way describes the shape of the upper aquifer sediments. This reservoir completely fills after a flow event in the upper reaches of the wash. This imaginary segment is also large enough in size and high enough above the real aquifer segment that it can go dry from drainage and evapotranspiration. In the program the size of the upper alluvial reservoir is determined by going a far enough distance upstream from the real segment so that the bottom of the alluvium in the imaginary segment is higher than the top of the indurable sediments in the real aquifer. At this point, the imaginary aquifer thickness is estimated and the width will be the same as at all cross-sections in the real aquifer. The length of imaginary upper reservoir is determined by optimization techniques or estimation of the total volume of the imaginary aquifer. The width x thickness x length of this upper reservoir should approximate the volume of all indurable alluvial

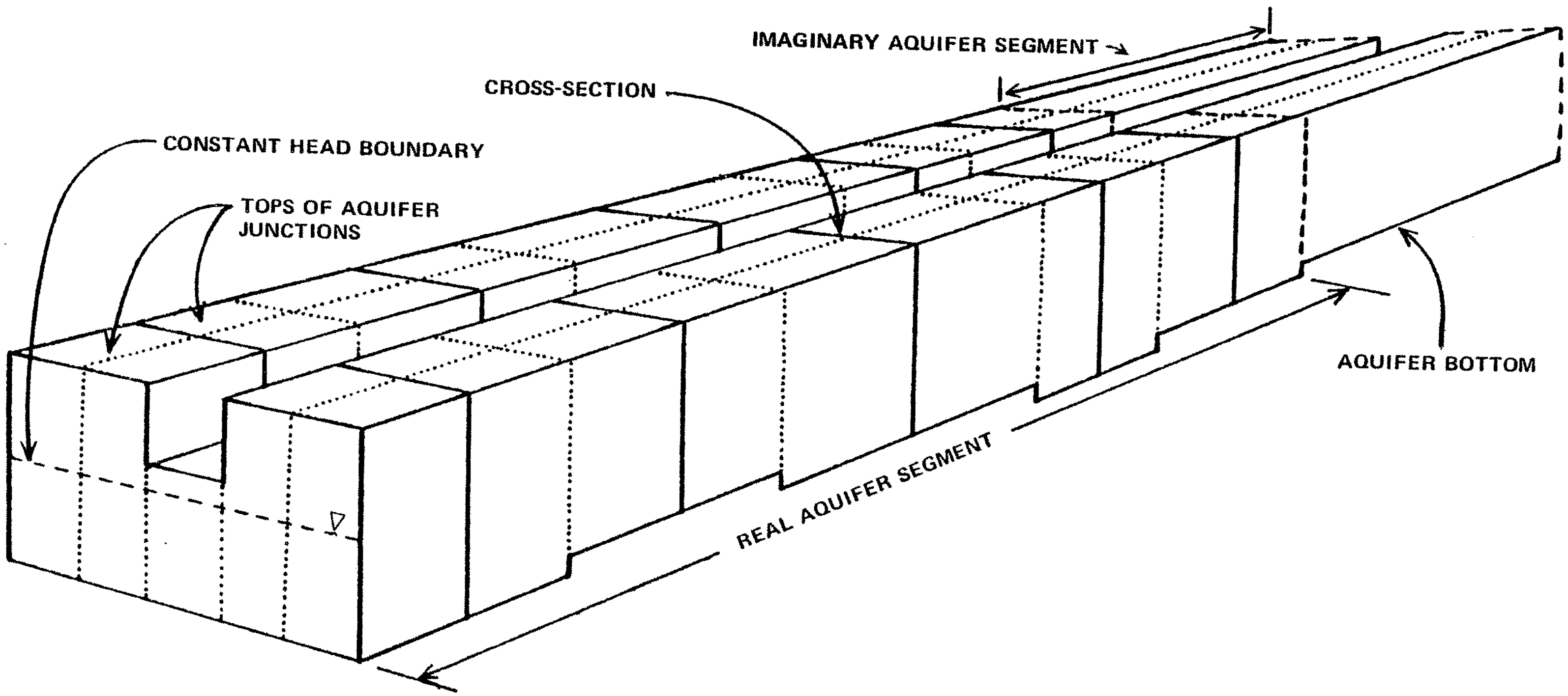

Figure 11. Illustration of "computerized" aquifer.

sediments above the real aquifer segment. The boundary above the upper reservoir is then assumed impermeable.

The lower boundary is assumed to have a constant head, since this point is near the confluence of two major alluvial aquifers. This head is below the bottom of the aquifer at the next cross-section upstream to allow for the possibility of the aquifer going dry above the constant head boundary.

To say the least, aquifer configurations are extremely variable from point to point in the stream and in time. To perfectly determine this configuration would be impossible without expensive geophysical exploration. Therefore, to provide detailed data on aquifer shape is useless. However, the program will accept values for aquifer thickness, aquifer bottom elevations, and elevations of the channel bottom at the cross-sections. The computer "conception" of an aquifer cross-section is depicted in Figure 7. No flows are allowed to overtop the banks of the channel.

Evapotranspiration losses are subtracted from junctions of the aquifer underlying the stream channel only. Since areas of the floodplain are several meters above the water table, and deep-rooted plants are absent, evapotranspiration losses are negligible. However, in the stream channel the water table is generally within a few feet of the surface. For this reason, evapotranspiration losses are set equal to the potential evapotranspiration rate. Two PET rates are used to coincide with two six-month seasons. In sandy soils, evaporation losses rapidly decrease after the water table drops below a certain level. GRDWTR checks to see if the water table is 5 or more feet below the surface before ET losses are subtracted. Below this depth, ET is assumed negligible and is set to 0.

To precisely determine evapotranspiration, a more sophisticated soil moisture model would be needed. These models typically consume much computer time, and in light of the changing channel conditions in both time and space, the increased accuracy was not considered worth the increased computer costs. However, actual operation of the model indicated the importance of evapotranspiration losses (discussed in Chapter 3).

The ground water routine contains a check for intersection of the channel bottom and the water table. This commonly occurs in some sections where the alluvium thins over a rise in bedrock and a small amount of water appears as channel flow. In this case, any excess ground water is simply added to the next channel junction downstream. A check for dry junctions is also provided, and if a channel depth drops below .5 feet, no flow is allowed through that channel. A message is printed if a junction goes dry.

CHAPTER 3

RESULTS AND DISCUSSION

The following pages detail runs made for the Black Mesa and should serve as an example for the ALAMO user. Also, an analysis of the results and a few notes about the model are included.

Parameter Estimation

A list of parameters and variables is given in Appendix A. Many are self-explanatory and can be obtained easily with a topographic map or from commonly available data. However, some parameters, particularly ground water parameters, require some data collection from the watershed in question. Since very little data is available on the larger natural watersheds of the Black Mesa, calibration was impossible. Following, then, are some of the methods used to obtain values for several parameters. Most values used are given in the text. All values used are listed in Appendix A.

Precipitation Parameters

The only parameter needed for determination of interarrival times is the probability, XPROB, of a storm being centered somewhere on the grid over the watershed. Very little data are available on storm center depths and probabilities of occurrence. However, on the Atterbury Watershed, near Tucson, Arizona, an average of 5.33 storm centers per summer season fell on a 20 square mile area. If it is assumed that for smaller areas the number of storm centers increases linearly with area, then an average of 58.6 storm centers per year would occur on the 220 square mile grid area around Coal Mine Wash. This amount translates into a value of 0.47 for XPROB. However, four years of point rainfall data gathered at the Black Mesa indicate that interarrival times at Atterbury are somewhat longer than those at the Black Mesa. For this reason, XPROB was reduced from 0.47 to 0.35, which is a reduction proportional to that indicated by the point rainfall records.

Two parameters are needed when using the Gumbel distribution for depth of the center maximum. They are the standard deviation (SD) and mean (GMEAN) storm center depth. Data from the Atterbury Watershed indicate values of 0.65 inches and 1.41 inches. Again, four years of point rainfall data from the Black Mesa indicate that rainfall depths are somewhat smaller than those found at Atterbury, and the mean storm center depth was reduced from 1.41 to 1.30 inches. The standard deviation was left at 0.65 inches.

Watershed Parameters

First, a delineation of subwatersheds is made. Their areas should not be so large that a point rainfall value at the middle of the subwatershed would not be fairly indicative of rainfall over the whole subwatershed for a convective storm. The 44 square miles of the Coal Mine Watershed was divided into 14 subwatersheds with the largest being 7.6 square miles (see Figure 1). It should also be kept in mind that some of the subwatershed outlets on the main stem are the locations for cross-sections used in computing travel times, routing, infiltration and recharge, and junctions for the ground water routine.

The parameter hardest to determine is the curve number for each subwatershed. Curve numbers can be obtained from the SCS National Engineering Handbook (United States Department of Agriculture 1969). This source, along with the results from several runs of a rainfall simulator, indicate values of 55-60 for the natural sandy soils of the Black Mesa and 78-80 for reclaimed spoils.

Ground Water Parameters

The hardest ground water parameter to estimate is permeability. If data were available, an optimization procedure would probably be the best way to determine an average permeability, as alluvial aquifers are notoriously nonhomogeneous. However, to estimate the hydraulic conductivity in the absence of data, several infiltration tests were run in the wash. The final infiltration capacities were used to approximate the saturated conductivity. A value of .02 cm/sec was used.

In nonartesian systems, the storage coefficient is equal to the specific yield of the medium. Average values for unsorted sand and gravel of .28 and .40 (Walton 1970) were used for the storage coefficient (SC) and porosity.

Parameters describing aquifer size and configuration were obtained from topographic maps and logs from a few observation wells drilled in the alluvium to begin data collection in a ground water monitoring program. Also, small stretches of extended flow in the wash were taken to indicate thinner aquifer thickness. Since a totally accurate picture of aquifer size and configuration is impossible at this time, an averaged, idealized aquifer is assumed.

A soil moisture curve (SMC) is required to determine the head under which infiltration is pulled from underneath. Since channel bottoms have extremely variable soil moisture characteristics, a typical soil moisture curve for sand was used (see Figure 8).

The cross-sectional distance interval of the aquifer (DX) represents two things: (1) fineness of aquifer grid desired or meaningful, and (2) the length of cross-section in which infiltration takes place. For the latter reason, DX was equal to 60 feet, which is the product of average channel width and stream sinuosity.

Evapotranspiration rates were determined from evaporation data collected at Many Farms, Arizona, near the Black Mesa. Limited evaporation data collected during the summer months on the Black Mesa do not themselves imply any correction for the elevation difference of approximately 1500 feet. Using a pan coefficient of .7, the average potential evapotranspiration rate in the summer was .5264 cm/day and .168 cm/day in the winter.

System Characteristics and the Implications of Mining

In this section the results of three runs of ALAMO are compared to determine the hydrologic impacts of strip mining on the Coal Mine Wash Watershed. The first run simulated the undisturbed conditions for 20 years. The second run simulated conditions for 12 years after the planned mining operation is completed, and the third run simulated conditions for 12 years after a more extensive hypothetical operation.

At the time of this writing, plans call for the stripping and reclaiming of approximately 7 square miles of the Coal Mine Wash Watershed and no overturning of the associated alluvial aquifers. In this case the only major change was significantly higher curve numbers on the subwatersheds included in the 7 square miles to be mined. Curve numbers were raised around 25 points. The hypothetical situation allowed for 18 square miles of stripped and reclaimed land and 7.5 miles of overturned aquifer. Consequently, the permeability of disturbed portions of the aquifer was reduced from .02 to .002 cm/sec, which is an average figure for the hydraulic conductivity of mine spoils on the Black Mesa.

One of the more significant characteristics of the ground water system brought out by ALAMO was that little water is lost from the aquifer through drainage downstream. Evidently, the aquifer slope is too low to allow significant downstream drainage with the existing permeability. The factor causing over 80 percent of the ground water depletion, then, is evapotranspiration. Since the ground water system resembles a reservoir more than a subsurface stream, mining across it would not significantly reduce the recharge and water levels in the aquifer segment downstream from the mining operation. The above conclusion assumes that no dams or structures prevent flood events from reaching and recharging the undisturbed portions of aquifer below. One important difference between the ground water system and a reservoir on the surface, however, is the automatic termination of ET losses from the system once the water table drops below a certain level. Therefore, these alluvial aquifers are very fine storage reservoirs. When more data are available and further study is warranted, a sensitivity analysis should be done to indicate model components having the largest effects on ground water levels. However, after these initial runs it appears that evapotranspiration losses have the major role in water level control.

Flow events in Coal Mine Wash are infrequent occurrences. In 20 years of simulation, there were only 43 runoff events. One year of simulation failed to produce any runoff. This characteristic is borne out by 4 years of peak flows recorded by the United States Geological Survey on Coal Mine and Yellow Water Washes, in which the average number of events was between 2 and 3. The high infiltration capacity and consequent low curve numbers (55-59) of the sandy soils of the Black Mesa require an exceptional storm to produce runoff. A curve number of 55 would absorb 1.63 inches of rain without producing runoff.

Obviously, the reaches farther downstream receive more frequent and longer recharge events and therefore generally keep those portions of the aquifer full. In fact, it is only in the thinner sections of the lower reaches where extended flow between runoff events occurs. At the upper end, however, junctions in the imaginary aquifer segment (see Ground Water section of Chapter 2) went dry for 234 days (during the 20 year run) due to lack of recharge.

A characteristic of the 2-stage recharge process was the short period of time required to fill the unsaturated portion of aquifer in the channel junctions. Generally, less than 15 minutes were required to move the wetting front to the water table under undisturbed conditions. This was due to the consistently high water tables and consequent lack of airspace in the unsaturated alluvium above it. The recharge process, then, is dominated by the second stage, ground water spreading.

Now that the above characteristics have been noted, a few comparisons to post-mining conditions and the hypothetically mined conditions can be made. The effect of the actual mining was predictable. Since no aquifer disturbances are planned, the only effect was increased runoff volume and events. In 12 years, under the same precipitation sequence, the number of runoff events under the undisturbed, post-mined, and hypothetically mined conditions was 33, 66, and 81 respectively. The total runoff volume was 0.28, 0.72, and 0.84 watershed inches. It is interesting to note that mining 1/6 of the watershed resulted in a three-fold increase in runoff volume.

This increased potential for ground water recharge was best exemplified in the run simulating actual mining plans. Water tables in these undisturbed aquifers were consistently higher than levels produced by either natural or the hypothetically mined watershed. Where the aquifer is overturned, the lower conductivity reduces recharge in both stages of infiltration and ground water spreading. The dominant recharge process is still the spreading phase with infiltration lasting less than 20 minutes. Reduced permeability also gives rise to steeper head gradients in the cross-sections. This factor would be significant when producing wells were desired, as their capacity would be severely impaired.

In both simulations with mined land, the upper portions of the watershed were left undisturbed, and upper reaches of the aquifer had values of head nearly identical to those in the run for undisturbed conditions.

Limitations and Adaptability

This model has been designed for use in areas where data are limited. Consequently, many averaged values were used. It would be particularly useful to have storm center parameters, distributions, and relationships derived from data in northeastern Arizona. As data are gathered, calibration and validation procedures can be performed and parameters better estimated. However, even though the model cannot be quantitatively validated at this time, the model did seem to reproduce some observable characteristics of the stream-aquifer system such as infrequent flow events, channel flow between events during the summer season, and slow ground water depletion from downstream flow.

For application in other areas, some parts of the model or subroutines could be replaced by more appropriate models. For instance, in an area where precipitation-runoff data are available, a different watershed model might take better advantage of this information. In areas where frontal activity is a more crucial factor, a completely different approach to rainfall would be appropriate.

This model is limited to areas where all aquifer recharge occurs from infiltration through stream bottoms. No deep seepage from the watershed at large is allowed. The applicability to areas in which alluvial aquifers are large and have several contributing streams is untried and questionable. The most limiting factor in this case would probably be computer time.

CHAPTER 4

CONCLUSIONS

This study has integrated individual watershed, stochastic precipitation, ground water and recharge models into a single model for evaluation of the behavior of ground water in alluvial deposits associated with ephemeral washes. The individual models were chosen in such a manner to allow the larger model to operate on watersheds of 5 to 100 square miles, in regions of data deficiencies, and regions where spotty, convective precipitation is the input. Although no long-term, systematic data were available to validate or calibrate the model, it does reproduce observable characteristics of the stream aquifer system. The model is also amenable to hydrologic changes such as those caused by surface mining. By making such changes, the model could be a valuable tool for making decisions regarding reclamation practices and aquifer disturbance.

Analysis of three runs with differing degrees of disturbance indicates minimal harm to the stream-aquifer system if overturning of the aquifer itself is limited to a small short segment. Stripping of upland areas, while changing soil, vegetation, and possibly other characteristics, only serves to provide more water for recharge to the alluvial aquifer. Less water is lost to evapotranspiration and an actual savings may by realized.

It should also be noted that if precipitation falling on reclaimed lands is allowed to flow offsite, the increase in flow size and number will likely have an adverse effect on structures in the floodplain downstream. The danger of flash flooding will increase and the morphology of the alluvial system may change to accommodate the new set of flow requirements.

Overturning small portions of aquifer does not affect ground water supplies significantly either upstream or downstream. However, any portions of aquifer overturned will be permanently destroyed and the value of those portions as storage reservoirs will be lost. If irrigation was to be developed and practiced (possibly to revegetate mine spoils) and the aquifer is left undisturbed, the whole system could function as a kind of giant water harvesting system, with the mined areas acting as collectors and the aquifer as the storage reservoir.

This model contains several components and parameters which have not been evaluated in the Black Mesa region, and as data are gathered, continued study and refinement are certainly warranted. However, ALAMO is a model that is workable in regions where data are scarce, and since it has reproduced observable system characteristics, it is felt that it has produced usable results in its present state.

APPENDIX A

VARIABLES USED IN ALAMO

Listed below are the significant variables used in ALAMO. Where appropriate, the initial values for the natural watershed and units of the variables are given. Such things as counters, assignment variables, and variables used for temporary storage are not listed. Arrays are indicated by an asterisk.

	Variable	Definition	Value and Units
*	A	Cross-sectional area of a channel.	$feet^2$
*	AK	Permeability of a channel.	.02 cm/sec (units are changed by the program)
*	AS	"Surface area" of a junction.	$feet^2$
	ASAJ	Porosity at current water table.	
*	ASK	Change in porosity per foot.	0.0
*	AT	Cross-sectional areas of an aquifer chanel at time T+DELT.	$feet^2$
	AVIN	Average of two points of an inflow hydrograph which is being routed through a reach.	cfs
*	B	Aquifer channel width.	feet
	B1	In CMPHYD B1 is initial abstractions. In PRECIP B1 is a dispersion parameter dependent on CMAX.	inches

*	BOT	Elevation of bottom of aquifer junctions.	feet
*	BOTT	Elevations of bottom of aquifer junctions in the stream channel.	feet
*	C	Travel time table. Length of time water is in transit between cross-sections for given discharges.	
*	CFS	Dimensionless unit hydrograph used in HYMO.	
	CMAX	Depth of precipitation at storm center.	inches
*	CN1	Curve numbers for the subwatershed.	
	COEF	Routing coefficient as described in the VSC flood routing method (see references).	
*	COORD	Coordinates of the centroids of the subwatersheds.	miles
*	DA	Areas of the subwatersheds.	$miles^2$
*	DATA	A working array used for various purposes.	
*	DEEP	Depth of flows for given discharges in rating curves.	feet
	DELT	A time increment used in calculation of the dimensionless unit hydrograph.	hours
	DELT1	Time step used in GRDWTR.	86,400 seconds
	DELT2	1/2 of DELT1.	seconds
	DEPTH	Depth of rainfall at a subwatershed.	inches
	DI2	Depth of inflow to a reach during routing.	feet

	DIS	Distance from subwatershed centroid to storm center.	
	DIST	Difference in elevation at the entrance and exit of a reach (used in ROUTE).	feet
	DIST1	Depth increment used in filling unsaturated portion of aquifer below the stream channel junction.	meters
*	DH	Head change after 1 time increment in the junction of a cross-section during water table spreading.	meters
	DO2	Depth of outflow from a reach after routing.	feet
*	DP	Depth of flow associated with given discharge and travel time in the travel time table.	feet
	DURAT	Ninety percent of the storm duration.	hours
	DX	Distance between junctions in a cross-section.	meters
	ECOND	Effective conductivity used in Green and Ampt infiltration.	m/sec
	FLAG	Flag used to tell ALAMO whether or not ADHYD, ROUTE, and INFIL can be skipped due to no flow.	
	FLOWDP	Depth calculated for a known flow.	meters
	GMEAN	Mean storm center depth used in Gumbel distribution.	1.3 inches
*	H	Heads in junctions after a full time step.	feet
*	HL	Heads of the junctions to the left of the stream channel junction.	meters

*	HR	Heads of the junctions to the right of the stream channel junction.	meters
*	HT	Heads in junctions after a 1/2 time step.	feet
	HTT	Difference between highest and lowest elevations in the sub-watershed.	feet
	IA	Interarrival time.	days
	ID	Identification number of hydrographs to be added, routed, etc.	1-6
	ID1 and ID2	Identification numbers of hydrographs to be added.	1-6
	IDH	Identification number of hydrograph to be routed.	1-6
	IDAY	Days since beginning of run.	days
	IDRC	Identification number of rating curves and associated cross-sections (increasing with progression downstream).	1-9
	IEND	Number of time increments in the longest of 2 hydrographs to be added.	
	IFLAG	Flag signals CMPHYD whether to compute a hydrograph or set hydrograph equal to 0 due to insufficient rainfall.	
	IPRT	First cycle of groundwater fluctuations to be printed.	
	J	Sometimes a counter, but often refers to junction number.	
	K	Number of depth increments between surface and water table in subroutine INFIL.	
	KPRT	Next cycle of groundwater fluctuations to be printed.	

*	LEN	Length of aquifer channel.	
*	LST	Junctions with fixed heads are given a value of 1 in this array.	
	NC	Total number of aquifer channels.	94
*	NCHAN	The channels associated with a junction.	
*	NCH	Stream channel junctions.	8, 13, 18, 23, 28 33, 38, 43, 48
	NCYPER	Number of groundwater cycles per time step.	
	NCYC	The number of time steps (cycles) GRDWTR is to run.	
	NHD	Subwatershed number.	
	NIDRC	Total number of rating curves and associated cross-sections.	9
	NJ	Total number of aquifer junctions.	55
	NJUNC	The 2 junctions on either end of a channel.	
	NN	The number of junctions in the cross-section.	5
	NNL	Number of junctions to the left of the stream channel junction.	2
	NNR	Number of junctions to the right of the stream channel junction.	2
	NPRT	Number of groundwater cycles between output printings.	10
	NPTSMC	Number of points on the soil moisture curve.	9
	NT	Number of travel time tables.	11
*	NUMB	Number of time increments required by a storm on the subwatershed.	

	NWS	Number of subwatersheds for which rainfall is to be calculated.	14
	NX	Abscissa of the storm center.	miles
	NY	Ordinate of the storm center.	miles
	NYRS	Number of years to be simulated.	20
*	OCFS	Ordinates of hydrographs.	
	O2	Outflow of a reach at end of time increment after routing.	cFs
	PERIOD	Number of hours in a day.	24
	PERM1	If this value is negative, its absolute value is the permeability given a node.	
	PET1	Potential evapotranspiration in the summer months.	.000000257 cm/sec
	PET2	Potential evapotranspiration in the winter months.	.000000082 cm/sec
*	Q	Flow in the aquifer channels.	
*	QD	The given discharges in the rating curves.	cFs
*	QIN	Pumping rate at a junction (not used).	0.0 cFs
	QP	Peak flow of dimensionless unit hydrograph used in CMPHYD.	
*	QOU	Artificial recharge at a junction (not used).	0.0 cFs
*	R	Aquifer channel depths.	feet
	R1	Potential maximum retention of subwatershed as determined by curve numbers.	inches
*	RAIN	Cumulative rainfall values for a subwatershed.	inches
*	RINT	Rainfall intensity (amount/number of time increments).	

	RO	Runoff volume.	watershed inches
	S	Channel storage during routing.	
	SATCON	Default value for permeability.	.02 cm/sec
*	SCFS	The given discharges in the travel time tables.	cFs
	SD	Standard deviation of the storm center depths.	inches
	SLOPE	The channel slope of a reach for ROUTE.	
*	SMC	The soil moisture curve.	
	SPOR	Default value for porosity.	.40
	SUCTN	Suction at a given depth increment above the water table.	meters
	SUMQ	The sum of all the flows from all channels, pumping, or recharge to a junction.	ft^3/sec
	T	Time counter in GRDWTR.	seconds
	THETA	Soil moisture of a given depth increment above the water table.	
*	THICK	Aquifer thickness.	meters
	TINF	Time of inflection point in the dimensionless unit graph used in CMPHYD.	
	TIME	Day of the year.	days
	TMAX	Highest rainfall depth on any subwatershed for a given storm.	inches
	TI2	Travel time of inflow at current time increment.	hours
	TINFIL	Amount of void space available in an unsaturated depth increment to be filled by infiltration.	

	TO2	Travel time of outflow through a reach during routing.	hours
	TOTINF	Total amount of water needed to fill unsaturated portion of aquifer below the stream channel.	meters
	TP	Time to peak of dimensionless unit graph.	
	TRANSM	Transmissivity between two junctions of a cross-section during the spreading phase of recharge.	m^2/sec
	TREC1	Time at which second recession constant takes over in dimensionless unit graph.	
	TT	Time required to fill an unsaturated depth increment during Green and Ampt infiltration.	seconds
	TTT	Cumulative time counter used to determine which time increment of hydrograph is to be used for depth calculation.	seconds
	TXJ	Time increment of hydrographs.	.16667 hrs
	TZERO	Beginning time value.	0.0
*	V	Velocity of flows through aquifer channels.	feet/day
*	VOID	Porosity of a junction if default value is to not be used.	.40
*	VT	Velocity of flow in aquifer channels at the half time step.	feet/day
	WFRONT	Distance from surface to wetting front during Green and Ampt recharge.	meters
	WTNEW	Height of water table above the aquifer bottom at the stream channel junction of a cross-section.	meters

XFLAG	Tells GRDWTR to reinitialize aquifer channel values and print current heads of a recharge event has just been completed.	0.0
XL	In CMPHYD, XL is the length of the longest drainage in the subwatershed.	CMPHYD-miles
	In ROUTE, XL is the length of the reach through which a hydrograph is being routed.	ROUTE-feet
XN	Dimensionless subwatershed shape parameter.	
XPROB	The probability of a storm being centered somewhere on the grid.	.35
YR	Current year.	years

APPENDIX B

PROGRAM LISTING OF ALAMO

```
      PROGRAM ALAMO(INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
C
      COMMON CFS(300),OCFS(300,6),IEND(6),DATA(310),DA(6),DP(20,16),NHD
     1,SCFS(20,16),C(20,16),QD(20,11),RAIN(200),DEEP(20,11),IFLAG,FLAG,
     2XPROB,DIST(6),TXJ,HL(9),DH(9),GMEAN,DIST1,YR,PI,XFLAG,SATCON,DX,
     3TIME,ROIN,NHYD(6),SMC(2,10),HR(9),BOT(11),QX(9),R(210),BOTT(121),
     4NWS,NPTSMC,SPOR,NUMB(16),RINT(16),COORD(15,2),NYRS,SD,DELT1,NCH(10
     5),THICK(11),NJ,NCYC,NCHAN(121,5),LST(121),VOID(121),NTEMP(5),H(121
     6),HT(121),HN(121),AS(121),ASK(121),QIN(121),QOU(121),LEN(210),NC,
     7 B(210),A(210),AT(210),V(210),VT(210),Q(210),AK(210),NJUNC(210,2),
     8PERIOD,PERM1,TZERO,DELT,DELT2,CN1(15),NPRT,IPRT,KPRT,NETCYC,NCYPER
     9,PET,PET1,PET2
      REAL LEN
C                            WATERSHED DATA
      READ 40,(CN1(I),I=1,15)
      READ (5,41) SPOR,NPTSMC,DIST1,PET1,PET2
      READ (5,42)((SMC(I,J), J=1,NPTSMC), I=1,2)
      READ 45, XPROB, SD, GMEAN, NWS, NYPS, TXJ
      READ 50,NIDRC,NT,NNR,NN,SC
      READ 55,((COORD(I,J),I=1,NWS), J=1,2)
      READ 60,((QD(I,J),I=1,20),J=1,NIDRC)
      READ 70,((DEEP(I,J),I=1,20),J=1,NIDRC)
      READ 70, ((DP(I,J),I=1,20),J=1,NT)
      READ 80,((SCFS(I,J),I=1,20),J=1,NT)
      READ 90,((C(I,J),I=1,20),J=1,NT)
      READ 35, (THICK(I),I=1,NIDRC),(BOT(J),J=1,NIDRC)
35    FORMAT( 9F8.1)
      DO99 I5=1,11
      NI=THICK(I5)/DIST1+1
99    THICK(I5)=NI*DIST1
      READ 103,(NCH(I3),I3=1,10)
103   FORMAT(10I5)
C                          GROUND WATER CONTROL DATA
      READ(5,105)DELT1,TZERO,KCYC,NPRT,IPRT,PERIOD,SATCON
105   FORMAT(5X,2F10.0,3I5,2F10.3)
      DO 9003 I=1,121
      H(I)=0.0
      LST(I)=0
      DO 9003 J=1,5
9003  NCHAN(I,J)=0
      DO 9004 I=1,210
      V(I)=Q(I)=0.0
      DO 9004 J=1,2
9004  NJUNC(I,J)=0
      NJ=0
      DO 121 I=1,250
C                              JUNCTION DATA
      READ(5,120) J,HEAD,SURF,SLOPE,QF1,QF2,(NTEMP(K),K=1,5),VOYD
120   FORMAT(I5,F10.0,E10.0,F10.0,2F5.0,5I5,F5.0)
      IF(J.GT.250 ) GOTO 1122
      IF (J.GT.0) GOTO 122
      J=-J
      LST(J)=1
122   IF(J.GT.NJ) NJ=J
      H(J)=HEAD
      AS(J)=SURF
      ASK(J)=SLOPE
      QIN(J)=QF1
      QOU(J)=QF2
      VOID(J)=VOYD
      DO 2101 M=1,5
      NCHAN(J,M)=NTEMP(M)
2101  CONTINUE
121   CONTINUE
```

```
1122  CONTINUE
      READ 95,(BOTT(I),I=1,NJ)
95    FORMAT(5F10.0)
C                                    CHANNEL DATA
      NC=0
      DO 131 I=1,350
      READ(5,130) N,ALEN,WIDE,AREA,PERM,VEL,DEP1,(NTEMP(K),K=1,2)
130   FORMAT(I5,6F10.0,2I5)
      IF(N.GT.350) GOTO1132
      IF(N.GT.NC) NC=N
      LEN(N)=ALEN
      B(N)=WIDE
      A(N)=AREA
      R(N)=DEP1
      AK(N)=SATCON
      IF(PERM .LT. 0.0) AK(N)=-1.0*PERM
      V(N)=VEL
      NJUNC(N,1)=MIN0(NTEMP(1),NTEMP(2))
      NJUNC(N,2)=MAX0(NTEMP(1),NTEMP(2))
131   CONTINUE
1132  CONTINUE
C                                         INITIALIZE
      DX=LEN(1)*.3048
      DELT2=DELT1/2.0
      TZERO=TZERO*3600.0
      PERIOD=PERIOD*3600.0
      NCYPER=(PERIOD/DELT1)+.5
      NETCYC=NCYC-NCYPER
      T=TZERO
      KPRT=IPRT
      PET=PET1
      DO 186J=1,NJ
      AS(J)=AS(J)*VOID(J)
      HT(J)=H(J)
      HN(J)=H(J)
186   CONTINUE
      CHANGE=1.0/(2.54*12.0)
      DO 190 N=1,NC
      AK(N)=AK(N)*CHANGE
      AT(N)=A(N)
190   CONTINUE
      YR=0.0
      PI=3.141
      XFLAG=1.0
C                BEGIN HYDROGRAPH AND RECHARGE CALL SEQUENCE
1     CALL PRECIP
      FLAG=0.0
      IF(RINT(1)*NUMB(1) .LT. 200/CN1(1)-2) IFLAG=0
      CALL CMPHYD(2,301,7.65,CN1(1),1248.0,4.96,5.419)
2     IF(FLAG .EQ. 0.0) GOTO 3
      CALL ROUTE(1 ,1,101,2,9000.,.014)
3     IF(RINT(2)*NUMB(2) .LT. 200/CN1(2)-2) IFLAG=0
      CALL CMPHYD(3,302,5.55,CN1(2),1199.0,5.303,4.872)
4     IF(RINT(3)*NUMB(3) .LT. 200/CN1(3)-2) IFLAG=0
      CALL CMPHYD(5,303,5.62,CN1(3),1140.0,4.85,4.945)
5     IF(FLAG .EQ. 0.0) GOTO 6
      CALL ADHYD(6,1,5,3,NN)
      CALL ROUTE(2,4,102,6,6000.,.013)
      CALL ADHYD(2,103,4,1,NN)
6     IF(RINT(4)*NUMB(4) .LT. 200/CN1(4)-2) IFLAG=0
      CALL CMPHYD(1,304,2.47,CN1(4),830.0,2.65,4.568)
7     IF(FLAG .EQ. 0.0) GOTO 8
      CALL ADHYD(3,2,2,1,NN)
      CALL INFIL(1,SC,NNR,NN,3)
      CALL ROUTE(3,1,104,3,6600.,.0112)
8     IF(RINT(5)*NUMB(5) .LT. 200/CN1(5)-2) IFLAG=0
```

```
      CALL CMPHYD(2,305,4.82,CN1(5),660.0,4.61,4.018)
9     IF(FLAG .EQ. 0.0) GOTO 10
      CALL ADHYD(5,3,3,2,NN)
      CALL INFIL(2,SC,NNR,NN,5)
      CALL ROUTE(4,1,105,5,7000.,.007)
10    IF(RINT(6)*NUMB(6) .LT. 200/CN1(6)-2) IFLAG=0
      CALL CMPHYD(4,306,3.34,CN1(6),521.0,3.01,3.957)
11    IF(FLAG .EQ. 0.0) GOTO 14
      CALL ADHYD(5,4,1,4,NN)
      CALL INFIL(3,SC,NNR,NN,5)
      CALL ROUTE(5,2,119,5,4400.,.008)
14    IF(RINT(7)*NUMB(7) .LT. 200/CN1(7)-2) IFLAG=0
      CALL CMPHYD(1,307,1.52,CN1(7),556.0,1.44,4.401)
15    IF(FLAG .EQ. 0.0) GOTO 16
      CALL ADHYD(3,26,1,2,NN)
16    IF(RINT(8)*NUMB(8) .LT. 200/CN1(8)-2) IFLAG=0
      CALL CMPHYD(1,308,1.65,CN1(8),489.0,2.235,3.718)
17    IF(FLAG .EQ. 0.0) GOTO 18
      CALL ADHYD(2,27,1,3,NN)
      CALL INFIL(4,SC,NNR,NN,2)
      CALL ROUTE(6,1,121,2,5600.,.008)
18    IF(RINT(9 )*NUMB(9 ) .LT. 200/CN1(9)-2) IFLAG=0
      CALL CMPHYD(2,309,1.94,CN1(9),571.0,2.424,3.936)
19    IF(FLAG .EQ. 0.0) GOTO 20
      CALL ADHYD(3,28,1,2,NN)
      CALL INFIL(5,SC,NNR,NN,3)
      CALL ROUTE(7,1,126,3,4600.,.0119)
20    IF(RINT(10)*NUMB(10) .LT. 200/CN1(10)-2) IFLAG=0
      CALL CMPHYD(2,310,1.52,CN1(10),438.0,2.35,3.480)
21    IF(FLAG .EQ. 0.0) GOTO 22
      CALL ADHYD(5,29,1,2,NN)
22    IF(RINT(11)*NUMB(11) .LT. 200/CN1(11)-2) IFLAG=0
      CALL CMPHYD(1,311,1.18,CN1(11),413.0,2.613,3.166)
23    IF(FLAG .EQ. 0.0) GOTO 24
      CALL ADHYD(3,32,1,5,NN)
      CALL INFIL(6,SC,NNR,NN,3)
      CALL ROUTE(8,1,127,3,7800.,.0105)
24    IF(RINT(12)*NUMB(12) .LT. 200/CN1(12)-2) IFLAG=0
      CALL CMPHYD(2,312, 2.66,CN1(12),407.0,2.99,3.503)
25    IF(FLAG .EQ. 0.0) GOTO 26
      CALL ADHYD(6,33,1,2,NN)
      CALL INFIL(7,SC,NNR,NN,6)
      CALL ROUTE(9,2,128,6,6400.,.0075)
26    IF(RINT(13)*NUMB(13) .LT. 200/CN1(13)-2) IFLAG=0
      CALL CMPHYD(1,313,2.67,CN1(13),317.0,1.82,3.763)
27    IF(FLAG .EQ. 0.0) GOTO 28
      CALL ADHYD(3,36,1,2,NN)
      CALL INFIL(8,SC,NNR,NN,3)
      CALL ROUTE(10,1,131,3,8000.,.0075)
28    IF(RINT(14)*NUMB(14) .LT. 200/CN1(14)-2) IFLAG=0
      CALL CMPHYD(2,314,1.46,CN1(14),315.0,2.12,3.213)
29    IF(FLAG .EQ. 0.0) GOTO 1
      CALL ADHYD(3,37,1,2,NN)
      CALL INFIL(9,SC,NNR,NN,3)
      GOTO 1
40    FORMAT(15F5.1)
41    FORMAT(F6.4,I6,F7.5,2F12.10)
42    FORMAT(10F7.3)
45    FORMAT(F10.8,2F5.3,2I5, F7.5)
50    FORMAT(4I5,F5.2)
55    FORMAT(14F5.2)
60    FORMAT(10F8.1)
70    FORMAT(10F8.2)
80    FORMAT(10F8.0)
90    FORMAT(10F8.3)
100   STOP
```

```
      END

      SUBROUTINE PRECIP
C  THIS SUBROUTINE DETERMINES PRECIPITATION ON ALL OF THE SUBWATERSHEDS
      COMMON CFS(300),OCFS(300,6),IEND(6),DATA(310),DA(6),DP(20,16),NHD
     1,SCFS(20,16),C(20,16),QD(20,11),RAIN(200),DEEP(20,11),IFLAG,FLAG,
     2XPROB,DIST(6),TXJ,HL(9),DH(9),GMEAN,DIST1,YR,PI,XFLAG,SATCON,DX,
     3TIME,RDIN,NHYD(6),SMC(2,10),HR(9),BOT(11),QX(9),R(210),BOTT(121),
     4NWS,NPTSMC,SPOR,NUMB(16),RINT(16),COORD(15,2),NYRS,SD,DELT1,NCH(10
     5),THICK(11),NJ,NCYC,NCHAN(121,5),LST(121),VOID(121),NTEMP(5),H(121
     6),HT(121),HN(121),AS(121),ASK(121),QIN(121),QOU(121),LEN(210),NC,
     7 B(210),A(210),AT(210),V(210),VT(210),Q(210),AK(210),NJUNC(210,2),
     8PERIOD,PERM1,TZERO,DELT,DELT2,CN1(15),NPRT,IPRT,KPRT,NETCYC,NCYPER
     9,PET,PET1,PET2
      GOTO 18
C  CHECK AND RESET TIME VARIABLES
16    IA=IA+242
      YR=YR+1.0
      PRINT 71, YR
      TIME=IA-242-(123-TIME)
      GOTO 40
C  INTERARRIVAL TIME
18    X=RANF(1.0)
      IA=INT(1.0+ALOG(X)/ALOG(1.0-XPROB))
      PRINT 35, IA
35    FORMAT(1H ,///,1H ,20H  INTERARRIVAL TIME=, I2)
      TIME=TIME+IA
      IF(TIME .GT. 92) PET=PET2
      IF(TIME .GT. 123) GOTO 16
40    NCYC=IA*86400/DELT1
      CALL GROWTR(5)
      IF(IA .GT. 125 .AND. YR .GE. NYRS) GOTO 80
      XFLAG=1.0
C  LOCATE STORM CENTER
      XX=RANF(7.0)
      XY=RANF(7.0)
      NY=INT(XY*20+.5)
      NX=INT(XX*11+.5)
C  CENTER MAXIMUM
      XG=RANF(7.0)
      YQ=-1.0*ALOG(-1.0*ALOG(XG))
      CMAX=.7797*SD*YQ-.45*SD+GMEAN
      PRINT 55, NX, NY, CMAX
55    FORMAT(1H ,///,1H ,10H ABSCISSA=,I2,14H     ORDINATE=,I2,29H      M
     1AX CENTER PRECIP DEPTH=,F4.2,/)
C  CHECK FOR RUNOFF POTENTIAL
      IF(CMAX .LT. .8) GOTO 18
      TMAX=0.0
C  DETERMINE PRECIP AMOUNTS FOR SUBWATERSHEDS
```

```
      DO 60 K=1,NWS
      DIS =SQRT((COORD(K,1)-NX)**2+(COORD(K,2)-NY)**2)+.05
      IF(DIS  .GT. 3.75) GOTO 57
      B1=.27*2.7183**(-1.0*.67*CMAX)
      DEPTH=CMAX*2.7183**(-1.0*PI*DIS **2.0*B1)
      DURAT=10.0**((DEPTH*.9-1.89)/2.42)
      GOTO 58
57    DEPTH=DURAT=0.0
58    TMAX=AMAX1(TMAX,DEPTH)
      IF(DEPTH .NE. 0.00) PRINT 70,K,DEPTH,DURAT

C  DETERMINE RAINFALL INTENSITIES FOR TEN MINUTE INTERVALS.

      NUMB(K)=INT(DURAT/TXJ+1.5)
      RINT(K)=DEPTH/NUMB(K)
60    CONTINUE

C  CHECK RUNOFF POTENTIAL

65    IF(TMAX .LT. .50) GOTO 18
70    FORMAT(1H ,16HSUBWATERSHED NO=,I2,24H        RAINFALL DEPTH=,F4.2
     1,19H          DURATION=,F5.3)
71    FORMAT(1X,//,30X,*---------------YEAR=*,F4.0,*---------------*,//)
      RETURN
80    STOP
      END



      SUBROUTINE GROWTR(NN)

C  THIS SUBROUTINE CALCULATES THE GROUND WATER FLUCTUATIONS DURING INTERARRIVALS

      COMMON CFS(300),QCFS(300,6),IEND(6),DATA(310),DA(6),DP(20,16),NHD
     1,SCFS(20,16),C(20,16),QD(20,11),RAIN(200),DEEP(20,11),IFLAG,FLAG,
     2XPROB,DIST(6),TXJ,HL(9),DH(9),GMEAN,DIST1,YR,PI,XFLAG,SATCON,DX,
     3TIME,ROIN,NHYD(6),SMC(2,10),HR(9),BOT(11),QX(9),P(210),BOTT(121),
     4NWS,NPTSMC,SPOR,NUMB(16),RINT(16),COORD(15,2),NYRS,SD,DELT1,NCH(10
     5),THICK(11),NJ,NCYC,NCHAN(121,5),LST(121),VOID(121),NTEMP(5),H(121
     6),HT(121),HN(121),AS(121),ASK(121),QIN(121),QOU(121),LEN(210),NC,
     7 B(210),A(210),AT(210),V(210),VT(210),Q(210),AK(210),NJUNC(210,2),
     8PERIOD,PERM1,TZERO,DELT,DELT2,CN1(15),NPRT,IPRT,KPRT,NETCYC,NCYPER
     9,PET,PET1,PET2
      REAL LEN
      KPRT=NPRT
C             AFTER A FLOW,(XFLAG NE 1.0), NEW DEPTHS, CROSS SECTIONAL AREAS,
C                    AND FLOW VELOCITIES ARE CALCULATED AND NEW HEADS WRITTEN.
      IF(XFLAG .EQ. 1.0) GOTO 199
      WRITE(6,321) ((J,H(J), J+1,H(J+1), J+2,H(J+2), J+3,H(J+3), J+4,H(J+4))
     1, J=1,55,NN)
      DO 274 I1=1,NC
      R(I1)=((H(NJUNC(I1,1))-BOTT(NJUNC(I1,1)))+(H(NJUNC(I1,2))-BOTT(NJU
     1NC(I1,2))))/2
203   A(I1)=R(I1)*B(I1)
274   V(I1)=-AK(I1)*((H(NJUNC(I1,2))-H(NJUNC(I1,1)))/LEN(I1))
199   IF(NCYC .LT. 80) GOTO 1
C               IF ENTERING INTO WINTER SEASON, TIME STEP IS 3 TIMES LARGER.
      JR=MOD(NCYC,3)
      NCYC=NCYC/3
      DELT1=DELT1*3
      DELT2=DELT2*3
C                 MAIN LOOP.
1     DO 285 ICYC=1,NCYC
      T2=T+DELT2
      T=T+DELT1
```

```
      IDAY=T/86400.0
      IF(MOD(IDAY,365) .GE. 274) PET=PET1
C                             VELOCITIES AND FLOWS AT T+DELT/2
      DO 204 N=1,NC
      IF(NJUNC(N,1) .LE. 0) GOTO 204
C                          CHECK FOR DRY CHANNEL
      IF(R(N) .GT. .5) GOTO 503
      VT(N)=Q(N)=0.0
      GOTO 204
503   NL=NJUNC(N,1)
      NH=NJUNC(N,2)
      VT(N)=-AK(N)*((H(NH)-H(NL))/LEN(N))
      Q(N)=VT(N)*A(N)
204   CONTINUE
C                                        HEADS AT T+DELT/2
      DO 225 J=1,NJ
      IF(NCHAN(J,1) .LE. 0) GOTO 225
      IF(LST(J) .NE. 0) GOTO 225
      SUMQ=0.0
      DO 220 K=1,5
      IF(NCHAN(J,K) .LE. 0) GOTO224
      N=NCHAN(J,K)
      IF(J .NE. NJUNC(N,1)) GOTO 215
      SUMQ=SUMQ+Q(N)
      GOTO 220
215   SUMQ=SUMQ-Q(N)
220   CONTINUE
224   HT(J)=H(J)-DELT2*SUMQ/AS(J)
225   CONTINUE
C                    AREAS,VELOCITIES AND FLOWS AT T+DELT
      DO 230 N=1,NC
      IF(NJUNC(N,1).LE. 0) GOTO 230
      NL=NJUNC(N,1)
      NH=NJUNC(N,2)
      DELH=(HT(NH)-H(NH)+HT(NL)-H(NL))/2.0
      RNT=R(N)+DELH
      AT(N)=A(N)+B(N)*DELH
C                            CHECK FOR DRY CHANNELS
      IF(RNT .GT. .5) GOTO 501
      V(N)=Q(N)=0.0
      GOTO 230
501   CONTINUE
      V(N)=-AK(N)*((HT(NH)-HT(NL))/LEN(N))
      Q(N)=(Q(N)+V(N)*AT(N))/2.0
230   CONTINUE
C                                HEADS AT T+DELT
      DO255 J=1.NJ
      IF(NCHAN(J,1) .LE. 0) GOTO 255
      IF(LST(J).NE. 0) GOTO 255
      ASAJ=AS(J)+ASK(J)*(HT(J)-H(J))
      SUMQ=0.0
      DO 250 K=1,5
      IF(NCHAN(J,K) .LE. 0) GOTO 254
      N=NCHAN(J,K)
      IF(J .NE. NJUNC(N,1)) GOTO 245
      SUMQ=SUMQ+Q(N)
      GOTO 250
245   SUMQ=SUMQ-Q(N)
250   CONTINUE
254   HN(J)=H(J)-DELT1*SUMQ/ASAJ
255   CONTINUE
C                             DEPTH AND AREAS AT T+DELT
      DO 257 N=1,NC
      IF(NJUNC(N,1) .LE. 0)  GOTO 257
      NL=NJUNC(N,1)
      NH=NJUNC(N,2)
```

```
      DELH=.5*(HN(NH)-H(NH)+HN(NL)-H(NL))
      R(N)=R(N)+DELH
      A(N)=A(N)+R(N)*DELH
257   CONTINUE
C                            COMPUTE NEW SURFACE AREAS
C                            SHIFT HEADS TO H ARRAY FOR NEW CYCLE
      J1=1
      DO 258 J=1,NJ
      IF(LST(J) .NE. 0) GOTO 258
      AS(J)=AS(J)+ASK(J)*(HN(J)-H(J))
      H(J)=HN(J)
C                              CHECK FOR DRY JUNCTION.
      IF(H(J) .LT. BOTT(J)) PRINT 259,J
C       CHECK FOR INTERSECTION OF CHANNEL BOTTOM AND WATER TABLE AND SUBTRACT ET
      IF(J .NE. NCH(J1)) GOTO 258
-     IF(H(J) .GT. BOT(J1)+THICK(J1)/.3048-5.0) H(J)=H(J)-PET*DELT1
      IF(H(J) .LT. BOT(J1)+THICK(J1)/.3048) GOTO 271
      H(NCH(J1+1))=(H(J)-(BOT(J1)+THICK(J1)/.3048))+H(NCH(J1+1))
      PRINT 256,J
256   FORMAT(5X,*STREAMFLOW OCCURS AS WATER TABLE INTERSECTS CHANNEL BOT
     1TOM AT NODE*,I3,*.  NEXT AQUIFER SEGMENT IS RECHARGED BY EXCESS.*)
      H(J)=BOT(J1)+THICK(J1)/.3048
271   J1=J1+1
258   CONTINUE
      H(NCH(1)-NN)=H(NCH(1)-NN)-PET*DELT1
259   FORMAT(10X, *NODE *,I3,* IS DRY*)
C                                 COMPUTE AVERAGE H, Q, V
C                                 CHECK VELOCITIES FOR EXCESS
      DO 275 N=1,NC
      IF(NJUNC(N,1) .LE. 0)  GOTO 275
      IF(ABS(V(N)) .LE. 20.0) GOTO 275
      WRITE (6,270)ICYC,N, Q(N),R(N),V(N)
270   FORMAT(1H ,33H VELOCITY EXCEEDS 20FPD IN CYCLE ,I4, 8H CHANNEL ,I4
     1,4H Q = ,1PE12.4,8H DEPTH = ,F7.3,4H V = ,1PE12.4)
      IF(ICYC .GT.30) GOTO 312
275   CONTINUE
C                          PRINT OUTPUT
      IF(ICYC .EQ. NCYC) GOTO 319
      IF(ICYC .NE. KPRT) GOTO 285
      KPRT=KPRT+NPRT
319   HOUR=(T-86400.0*FLOAT(IDAY))/3600.0
      WRITE(6,320)ICYC,IDAY,HOUR
320   FORMAT(1H ,//,9X,31H HYDRAULIC CONDITIONS FOR CYCLE ,I4,4H DAY ,I5
     1,5H HOUR,F6.2,/)
      WRITE(6,329)
329   FORMAT(1H , 5(11H JUNC  HEAD ,2X,)/,1H , 5(7X,4H(FT),2X)/)
      WRITE(6,321) ((J,H(J),J+1,H(J+1),J+2,H(J+2),J+3,H(J+3),J+4,H(J+4))
     1, J=1,55,NN)
321   FORMAT( 5(I5,F7.2,1X))
C                                 END MAIN LOOP
285   CONTINUE
      IF(JR .EQ. 0 .AND. NCYC .LE. 79) GOTO 312
      NCYC=JR
      JR=0
      DELT1=DELT1/3
      DELT2=DELT2/3
      IF(NCYC .GE. 1) GOTO 1
312   RETURN
      END



      SUBROUTINE INFIL(IDRC,SC,NNR,NN,ID)

C     THIS SUBROUTINE CALCULATES INFILTRATION TO THE WATER TABLE AND CONSEQUENT
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C     WATER TABLE SPREADING.

      COMMON CFS(300),OCFS(300,6),IEND(6),DATA(310),DA(6),DP(20,16),NHD
     1,SCFS(20,16),C(20,16),OD(20,11),RAIN(200),DEEP(20,11),IFLAG,FLAG,
     2XPROB,DIST(6),TXJ,HL(9),DH(9),GMEAN,DIST1,YR,PI,XFLAG,SATCON,DX,
     3TIME,ROIN,NHYD(6),SMC(2,10),HR(9),BOT(11),QX(9),R(210),BOTT(121),
     4NWS,NPTSMC,SPOR,NUMB(16),RINT(16),COORD(15,2),NYRS,SD,DELT1,NCH(10
     5),THICK(11),NJ,NCYC,NCHAN(121,5),LST(121),VOID(121),NTEMP(5),H(121
     6),HT(121),HN(121),AS(121),ASK(121),OIN(121),QOU(121),LEN(210),NC,
     7 B(210),A(210),AT(210),V(210),VT(210),Q(210),AK(210),NJUNC(210,2),
     8PERIOD,PERM1,TZERO,DELT,DELT2,CN1(15),NPRT,IPRT,KPRT,NETCYC,NCYPER
     9,PET,PET1,PET2

C  INITIALIZE

      ECOND=.85*AK(IDRC*(NN-1)-1)*.3048
      WTNEW=(H((IDRC+1)*NN-NNR)-BOT(IDRC))*.3048
      I=0
      TOTINF=0.0
      WFRONT=-.5*DIST1
      DO 3 II=1,300
3     IF(OCFS(II,ID) .GT. 0.0) GOTO 4
4     DO 10 I1=II,300
      IF(OCFS(I1,ID) .LE. 0.2 .AND. I1 .GT.20) GOTO 490

C  DETERMINE FLOW DEPTH AT CURRENT TIME AND HYDROGRAPH.

      DO 20 J1=1,20
20    IF(OCFS(I1,ID) .LT.OD(J1,IDRC)) GOTO 30
30    FLOWDP=(DEEP(J1,IDRC)-((OD(J1,IDRC)-OCFS(I1,ID))/(OD(J1,IDRC)-OD(J
     11-1,IDRC)))*(DEEP(J1,IDRC)-DEEP(J1-1,IDRC)))*.3048
      TTT=0.0

C  DETERMINE NUMBER OF DEPTH INCREMENTS IN UNSATURATED PORTION.

      K=(THICK(IDRC)-WTNEW)/DIST1
      IF(K .LT. 1) GOTO 211
200   I=I+1

C  FIND VOLUME OF DEPTH INCREMENT AVAILABLE FOR INFILTRATING WATER FROM SOIL
C           MOISTURE CURVE AND HEIGHT OF POINT ABOVE THE WATER TABLE.

      SUCTN=(K+.5-I)*DIST1
      DO 202 J=1,NPTSMC
202   IF(SUCTN .LT. SMC(1,J))GOTO 203
203   THETA=SMC(2,J-1)-(((SUCTN-SMC(1,J-1))/(SMC(1,J)-SMC(1,J-1)))*(SMC(
     12,J-1)-SMC(2,J)))
      TINFIL=(SPOR-THETA)*DIST1
      WFRONT=WFRONT+DIST1

C  CALCULATE TIME NEEDED FOR INFILTRATING WATER TO FILL DEPTH INCREMENT

      TT=TINFIL/((FLOWDP+((THICK(IDRC)-WTNEW)-WFRONT))*ECOND/WFRONT+ECON
     1D)
      TTT=TT+TTT
      TOTINF=TOTINF+TINFIL
      IF(TTT .GT. TXJ*3600 .AND. I .LT. K) GOTO 10

C  CHECK FOR SATURATION OF PPOFILE OR END OF FLOW EVENT

      IF(I .LT. K) GOTO 200
      GOTO 211
10    CONTINUE
211   PRINT 230, TTT,ID,K,I1,IDRC,DX,ECOND
230   FORMAT(1X,*WFRONT HAS REACHED THE WATER TABLE. TTT=*,F6.2,*   ID=*
     1,I1,*   NO. OF .1 INCH DEPTH INCREMENTS=*,I3,*   TIME INTERVAL WHE
```

```
     2N SAT. OCCURS=*,I1,/,1X,*WATER TABLE HEIGHTS AT CROSS SECTION *,I2
     3,*.  POINTS ARE*,F6.2,* METERS APART.  ECOND=*,F11.9)
499   WTNEW=THICK(IDRC)

C     SPREADING OF INFILTRATING WATER VIA FINITE DIFFERENCE METHOD.

      NNL=NN-NNR-1

C  INITIALIZE WATER TABLE.

      DO 305 JJ=1,NN
      IF(NNL+2-JJ .LE. 0) GOTO 306
      HL(NNL+2-JJ)=(H(IDRC*NN+JJ)-BOT(IDRC))*.3048
      GOTO 305
306   HR(JJ-MAXO(NNL,NNR))=(H((IDRC+1)*NN-(NN-JJ))-BOT(IDRC))*.3048
305   CONTINUE
      HR(1)=HL(1)
      DO 450 I2=I1,300
310   IF(OCFS(I2,ID) .LT. .1 .AND. I2 .GT. 20) GOTO 500
      DO 307 J1=1,20
307   IF(OCFS(I2,ID) .LT.QD(J1,IDRC)) GOTO 308
308   FLOWDP=(DEEP(J1,IDRC)-((QD(J1,IDRC)-OCFS(I2,ID))/(QD(J1,IDRC)-QD(J
     11-1,IDRC)))*(DEEP(J1,IDRC)-DEEP(J1-1,IDRC)))*.3048
      HR(1)=THICK(IDRC)+FLOWDP
      HR(NNR+2)=HR(NNR)
      NNR1=NNR+1

C  CALCULATE NEW HEADS TO THE RIGHT OF THE CHANNEL JUNCTION.

      DO 320 I=2,NNR1
      TRANSM=AK(IDRC*(NN-1)-1)*((HR(I-1)+HR(I))/2.0)*.3048
320   DH(I)=((HR(I-1)-HR(I))-(HR(I)-HR(I+1)))*TRANSM*(TXJ*3600-TTT)/(DX*
     1*2*SC)
      DO 340 I=2,NNR1
340   HR(I)=HR(I)+DH(I)
      HL(1)=HR(1)

C  CALCULATE NEW HEADS TO THE LEFT OF THE CHANNEL JUNCTION.

      HL(NNL+2)=HL(NNL)
      NNL1=NNL+1
      DO 360 I=2,NNL1
      TRANSM=AK(IDRC*(NN-1)-1)*((HL(I-1)+HL(I))/2.0)*.3048
      DH(I)=((HL(I-1)-HL(I))-(HL(I)-HL(I+1)))*TRANSM*(TXJ*3600-TTT)/(DX*
     1*2*SC)
360   CONTINUE
      DO 380 I=2,NNL1
380   HL(I)=HL(I)+DH(I)
      TTT=0.0
450   CONTINUE
      PRINT 475
475   FORMAT(1H ,*FLOW LASTS TOO LONG*)
490   PRINT 495
      H(IDRC*NN+NNL+1)=(WTNEW+WFRONT)*3.281+BOT(IDRC)
      PRINT 497, H(IDRC*NN+NNL+1)
      RETURN
495   FORMAT(1H ,//,1H , *WETTING FRONT DID NOT REACH THE WATER TABLE*)
496   FORMAT(4F6.3,4I6)
497   FORMAT(1H ,*WTNEW=*, F6.2)
500   DO 392 I6=1,NN
      I5=NNL+2-I6
      IF(I5 .LE. 0) GOTO 389
      H(IDRC*NN+I6)=HL(I5)*3.281+BOT(IDRC)
      GOTO 392
389   H(IDRC*NN+I6)=HR(-I5+2)*3.281+BOT(IDRC)
392   CONTINUE
```

```
      RETURN
      END


      SUBROUTINE CMPHYD(ID,NHYDR,AREA,CN,HTT,XL,XNSUB)
C     THIS SUBROUTINE COMPUTES THE HYDROGRAPHS FROM THE SUBWATERSHEDS
      COMMON CFS(300),OCFS(300,6),IEND(6),DATA(310),DA(6),DP(20,16),NHD
     1,SCFS(20,16),C(20,16),QD(20,11),RAIN(200),DEEP(20,11),IFLAG,FLAG,
     2XPROB,DIST(6),TXJ,HL(9),DH(9),GMEAN,DIST1,YR,PI,XFLAG,SATCON,DX,
     3TIME,ROIN,NHYD(6),SMC(2,10),HR(9),BOT(11),QX(9),R(210),BOTT(121),
     4NWS,NPTSMC,SPOR,NUMB(16),RINT(16),COORD(15,2),NYPS,SD,DELT1,NCH(10
     5),THICK(11),NJ,NCYC,NCHAN(121,5),LST(121),VOID(121),NTEMP(5),H(121
     6),HT(121),HN(121),AS(121),ASK(121),QIN(121),QOU(121),LEN(210),NC,
     7 B(210),A(210),AT(210),V(210),VT(210),Q(210),AK(210),NJUNC(210,2),
     8PERIOD,PERM1,TZERO,DELT,DELT2,CN1(15),NPRT,IPRT,KPRT,NETCYC,NCYPER
     9,PET,PET1,PET2
C     ZERO OUT HYDROGRAPH
      DO 1 I=1,300
    1 OCFS(I,ID)=0.
      XN=XNSUB
C     CHECK RUNOFF POTENTIAL TO SEE IF THE REST OF CMPHYD CAN BE SKIPPED
      IF(IFLAG .EQ. 0) GOTO 35
C     INITIALIZE
      FLAG=1.0
      XFLAG=0.0
      NHD=NHYDR
      DA(ID)=AREA
      SLOPE=HTT/XL
      XLDW=(XL**2.)/DA(ID)
      XK=27.0*(DA(ID)**.231)*(SLOPE**(-.777))*(XLDW**.124)
      TP=4.63*(DA(ID)**.422)*(SLOPE**(-.46))*(XLDW**.133)
C     IF XN NOT GREATER THAN 0.0, COMPUTE XN BY ITERATION
      XKTP=XK/TP
      IF(XN .GT. 0.0) GOTO 2
      XN=5.0
      DO 6 I=1,50
      TINF=1.+SQRT(1./(XN-1.))
      XN1=.05/(XKTP*(ALOG(TINF/(TINF+.05))+.05))+1.
      DIFF=ABS(XN1-XN)
      IF (DIFF-.001) 7,7,5
    5 XN=XN1
    6 CONTINUE
      WRITE (6,29)
      GOTO 35
    2 TINF=1.+SQRT(1./(XN-1.))
C     DETERMINE C1.
    7 DELT=TINF/100.
      TC1=0.
      XN1P=XN-1.
      XN1M=1.-XN
      DO 8 I=2,101
      TC1=TC1+DELT
```

```
8      CFS(I)=(TC1**XN1P)*EXP(XN1M*(TC1-1.))
       SUM=CFS(101)/2.
       DO 9 I=2,100
9      SUM=SUM+CFS(I)
       C1=SUM*DELT
       CFSII=CFS(101)
       TTINF=TINF*TP
       TREC1=TTINF+2.*XK
       EEE=EXP((TTINF-TREC1)/XK)
       XK1=3.*XK

C      COMPUTE B9, QP, AND CFSI.

       B9=645.333/(C1+CFSII*(XKTP*(1.-EEE)+EEE*(XK1/TP)))
       QP=(B9*DA(ID))/TP
       CFSI=QP*CFS(101)
       CFSR1=CFSI*EEE

C      DETERMINE INCREMENTAL RUNOFF.

       R1=1000./CN-10.
       B1=.2*R1
       NWSN=NUMB(NHD-300)
       DO 11 I=1,NWSN
11     RAIN(I)=RINT(NHD-300)*I
13     DO 15 I=1,NWSN
       IF (RAIN(I)-B1) 33,33,14
33     DATA (I)=0.
       Q1=0.
       GO TO 15
14     Q2=((RAIN(I)-B1)**2.)/(RAIN(I)+.8*R1)
       DATA (I)=Q2-Q1
       Q1=Q2
15     CONTINUE

C      COMPUTE UNIT HYDROGRAPH.

       T2=0.
       CFS(1)=0.
       DO 20 I=2,300
       T2=T2+TXJ
       IF (T2-TTINF) 16,16,17
16     CFS(I)=QP*((T2/TP)**XN1P)*EXP(XN1M*(T2/TP-1.))
       GO TO 20
17     IF (T2-TREC1) 18,18,19
18     CFS(I)=CFSI*EXP((TTINF-T2)/XK)
       GO TO 20
19     CFS(I)=CFSR1*EXP((TREC1-T2)/XK1)
       IF (CFS(I)-1.) 21,21,20
20     CONTINUE
       I=300
21     ICND=I

C      COMPUTE STORM HYDROGRAPH.

       DO 24 J=2,NWSN
       N=J+ICND-2
       IF (N-300) 23,23,22
22     N=300
23     KK=J
       I=2
       DO 24 K=KK,N
       OCFS(K,ID)=OCFS(K,ID)+DATA(J)*CFS(I)
24     I=I+1
       M=N-1
       RO=0.
```

```
      DO 26 I=2,M
26    RO=RO+OCFS(I,ID)
      RO=(RO*TXJ)/(DA(ID)*645.333)
      I=M
      IEND(ID)=I
      M=I
      PRINT 98, NHYDR-300, RO
C
29    FORMAT("N DID NOT CONVERGE AFTER 50 ITERATIONS.")
35    IFLAG=1
      RETURN
98    FORMAT(1X,//,4X,*HYDROGRAPH FROM WATERSHED *,I2,*.  RUNOFF VOLUME
     1=*,F5.2,* INCHES*,/)
      END



      SUBROUTINE ADHYD(ID,NHYDG,ID1,ID2,NN)

C     THIS SUBROUTINE ADDS TWO HYDROGRAPHS.

      COMMON CFS(300),OCFS(300,6),IEND(6),DATA(310),DA(6),DP(20,16),NHD
     1,SCFS(20,16),C(20,16),QD(20,11),RAIN(200),DEEP(20,11),IFLAG,FLAG,
     2XPROB,DIST(6),TXJ,HL(9),DH(9),GMEAN,DIST1,YR,PI,XFLAG,SATCON,DX,
     3TIME,ROIN,NHYD(6),SMC(2,10),HR(9),BOT(11),QX(9),R(210),BOTT(121),
     4NWS,NPTSMC,SPOR,NUMB(16),RINT(16),COORD(15,2),NYRS,SD,DELT1,NCH(10
     5),THICK(11),NJ,NCYC,NCHAN(121,5),LST(121),VOID(121),NTEMP(5),H(121
     6),HT(121),HN(121),AS(121),ASK(121),QIN(121),QOU(121),LEN(210),NC,
     7 B(210),A(210),AT(210),V(210),VT(210),Q(210),AK(210),NJUNC(210,2),
     8PERIOD,PERM1,TZERO,DELT,DELT2,CN1(15),NPRT,IPRT,KPRT,NETCYC,NCYPER
     9,PET,PET1,PET2

C     CHECK FOR RECHARGE TO IMAGINARY AQUIFER

      IF(NHYDG .NE. 2) GOTO 3
      DO 2 IJ=1,NN
2     H(IJ)=664.0

C     FIND NUMBER OF TIME INCREMENTS OF LONGEST HYDROGRAPH

3     IF (IEND(ID1)-IEND(ID2)) 4,4,5
4     M3=IEND(ID1)
      K1=ID2
      IEND(ID)=IEND(ID2)
      GO TO 18
5     M3=IEND(ID2)
      K1=ID1
      IEND(ID)=IEND(ID1)
18    M=IEND(ID)

C     ADD HYDROGRAPHS

      DO 20 I=1,M3
20    OCFS(I,ID)=OCFS(I,ID1)+OCFS(I,ID2)
22    IF (M-M3) 25,25,23
23    M3=M3+1
      DO 24 I=M3,M
24    OCFS(I,ID)=OCFS(I,K1)
25    M=M+1
      DO 26 I=M,300
26    OCFS(I,ID)=0.0
      PRINT 98, NHYDG
27    RETURN
98    FORMAT(1X,//,4X,*ADDED HYDROGRAPH NO. *,I3,/)
      END
```

```
      SUBROUTINE ROUTE(NT,ID,NHYDG,IDH,XL,SLOPE)

C     THIS SUBROUTINE ROUTES A HYDROGRAPH THROUGH A REACH WITH THE
C     NEW VSC METHOD OF FLOOD ROUTING. THIS METHOD ACCOUNTS FOR THE
C     VARIATION IN WATER SURFACE SLOPE.

      COMMON CFS(300),OCFS(300,6),IEND(6),DATA(310),DA(6),DP(20,16).NHD
     1,SCFS(20,16),C(20,16),QD(20,11),RAIN(200),DEEP(20,11),IFLAG,FLAG,
     2XPROB,DIST(6),TXJ,HL(9),DH(9),GMEAN,DIST1,YR,PI,XFLAG,SATCON,DX,
     3TIME,ROIN,NHYD(6),SMC(2,10),HR(9),BOT(11),QX(9),R(210),BOTT(121),
     4NWS,NPTSMC,SPOR,NUMB(16),RINT(16),COORD(15,2),NYRS,SD,DELT1,NCH(10
     5),THICK(11),NJ,NCYC,NCHAN(121,5),LST(121),VOID(121),NTEMP(5),H(121
     6),HT(121),HN(121),AS(121),ASK(121),QIN(121),QOU(121),LEN(210),NC,
     7 B(210),A(210),AT(210),V(210),VT(210),Q(210),AK(210),NJUNC(210,2),
     8PERIOD,PERM1,TZERO,DELT,DELT2,CN1(15),NPRT,IPRT,KPRT,NETCYC,NCYPER
     9,PET,PET1,PET2

C     INITIALIZE

      NHD=NHYDG
      DIST(ID)=SLOPE*XL
      DA(ID)=DA(IDH)
      M=IEND(IDH)
3     NERRT=0
      N=19
      OCFS(1,ID)=0.
      S=0.
      T1=C(1)
      J=1
      GUES=1.
      CFS(1)=0.

C     IF INFLOW IS ZERO, SO IS OUTFLOW

15    DO 16 L=2,M
      IF (OCFS(L,IDH)) 16,16,49
16    OCFS(L,ID)=0.
C     ROUTE
49    DATA (L-1)=0.
      DO 42 I=L,300
      IF (I-M) 18,18,17
17    OCFS(I,IDH)=OCFS(I-1,IDH)*.9
18    AVIN=(OCFS(I,IDH)+OCFS(I-1,IDH))/2.
      SIA=S+AVIN
      J=1

C     DETERMINE DEPTH AND TRAVEL TIME OF INFLOW

      IF (OCFS(I,IDH)-SCFS(1,NT)) 19,23,20
19    DI2=(OCFS(I,IDH)/SCFS(1,NT))*DP(1,NT)
      T12=C(1,NT)
      GO TO 25
20    DO 21 J=2,N
      IF (OCFS(I,IDH)-SCFS(J,NT)) 24,23,21
21    CONTINUE
      IF (NERRT) 22,22,36
22    WRITE (6,46)
      NERRT=1
      GO TO 36
23    DI2=DP(J,NT)
      TI2=C(J,NT)
      GO TO 25
```

```
24     RATIO=(OCFS(I,IDH)-SCFS(J-1,NT))/(SCFS(J,NT)-SCFS(J-1,NT))
       DI2=DP(J-1,NT)+RATIO*(DP(J,NT)-DP(J-1,NT))
       TI2=C(J-1,NT)+RATIO*(C(J,NT)-C(J-1,NT))
25     DO 35 IT=1,10
       J=1

C      DETERMINE DEPTH AND TRAVEL TIME OF OUTFLOW

       IF (GUES-SCFS(1,NT)) 26,29,27
26     DO2=(GUES/SCFS(1,NT))*DP(1)
       TO2=C(1,NT)
       GO TO 31
27     DO 28 J=2,N
       IF (GUES-SCFS(J,NT)) 30,29,28
28     CONTINUE
       J=N
29     DO2=DP(J,NT)
       TO2=C(J,NT)
       GO TO 31
30     RATIO=(GUES-SCFS(J-1,NT))/(SCFS(J,NT)-SCFS(J-1,NT))
       DO2=DP(J-1,NT)+RATIO*(DP(J,NT)-DP(J-1,NT))
       TO2=C(J-1,NT)+RATIO*(C(J,NT)-C(J-1,NT))

C      FIND WATER SURFACE SLOPE

31     DDD=DIST(ID)/(DIST(ID)+DI2-DO2)
       IF (DDD-.01) 32,32,33
32     GUES=OCFS(I-1,IDH)
       GO TO 35
33     T2=.5*(TI2+TO2)
       T2=T2*SQRT(DDD)
       T=T1+T2

C      COMPUTE ROUTING COEFFICIENT

       COEF=(2.*TXJ)/(T+TXJ)
       O2=COEF*SIA
       TRY1=GUES
       RATIO=O2/(GUES+.1E-20)
       DIFF=ABS(1.-RATIO)

C      TEST FOR CONVERGENCE

       IF (DIFF-.001) 37,37,34
34     GUES=O2
35     CONTINUE
       OCFS(I,ID)=DATA(I-1)*SIA
       DATA (I)=DATA(I-1)
       IF(OCFS(I,ID) .GT. 0.0) WRITE(6,47) I, OCFS(I,ID)
       GO TO 38
36     OCFS(I,ID)=DATA(I-1)*SIA
       DATA (I)=DATA(I-1)
       GO TO 38
37     OCFS(I,ID)=O2
       DATA (I)=COEF

C      COMPUTE NEW STORAGE

38     S=SIA-OCFS(I,ID)
       T1=T2
       IF (OCFS(I,ID)-OCFS(I-1,ID)) 39,42,42
39     IF (OCFS(I,ID)-1.) 43,43,42
42     CONTINUE
       I=300
43     IEND(ID)=I
       PRINT 98, NHYDG
```

```
45     RETURN
C
46     FORMAT(1H0,"TRAVEL TIME TABLE EXCEEDED")
47     FORMAT(T10,"PROBLEM FAILED TO CONVERGE AFTER10 ITERATIONS. CONVERG
      1ENCE WAS FORCED."/T20,"OUTFLOW NUMBER = ",I4,"RATE =",F10.2)
98     FORMAT(1X,//,4X,*ROUTED HYDROGRAPH NO. *,I3,/)
       END
```

APPENDIX C

SUPPLEMENTARY DATA SYNTHESIS PROGRAMS

Some data requirements for program ALAMO require some calculations from the raw data gathered in the field and topographic maps. Programs CMPRC and CMPTT compute and punch the rating curves and travel time tables needed in ALAMO. Program CHKNET simply checks the aquifer network of channels and junctions for inconsistencies and prints the GRDWTR input data for inspection.

Program CHKNET

CHKNET reads all the junction and channel data and puts it all in the same arrays as in ALAMO. The channels entering each node are then checked against the two junctions at the end of each channel. If an inconsistency is detected, a message displaying the junction and channel numbers in question is printed. If no inconsistencies are detected, a printout of all the data needed for GRDWTR is provided for inspection. Using CHKNET can save the user much time and frustration by eliminating mistakes in keypunched data before it is used in the more expensive ALAMO program. Following is the program listing for CHKNET.

```
      PROGRAM CHKNET(INPUT,OUTPUT,TAPE5=INPUT,TAPE3=OUTPUT)
      DIMENSION NJUNC(350,2),NCHAN(250,5),NTEMP(5),LST(250),H(250),AS(25
     10),ASK(250),QIN(250),QOU(250),VOID(250),LEN(350),B(350),A(350),R(3
     250),AK(350),V(350)
      REAL LEN
C                              JUNCTION DATA
      DO 121 I=1,250
      READ(5,120) J,HEAD,SURF,SLOPE,QF1,QF2,(NTEMP(K),K=1,5),VOYD
120   FORMAT(1X,I4,F10.1,E10.4,F10.1,2F5.2,5I5,F5.2)
      IF(J.GT.250 ) GOTO 1122
      IF (J.GT.0) GOTO 122
      J=-J
      LST(J)=1
122   IF(J.GT.NJ) NJ=J
      H(J)=HEAD
      AS(J)=SURF
      ASK(J)=SLOPE
      QIN(J)=QF1
      QOU(J)=QF2
      VOID(J)=VOYD
      DO 2101 M=1,5
      NCHAN(J,M)=NTEMP(M)
2101  CONTINUE
121   CONTINUE
1122  CONTINUE
C                              CHANNEL DATA
      NC=0
      DO 131 I=1,350
      READ(5,130) N,ALEN,WIDE,AREA,PERM,VEL,DEP1,(NTEMP(K),K=1,2)
130   FORMAT(I5,6F10.0,2I5)
      IF(N.GT.350) GOTO1132
      IF(N.GT.NC) NC=N
      LEN(N)=ALEN
      B(N)=WIDE
      A(N)=AREA
      R(N)=DEP1
      AK(N)=SATCON
      V(N)=VEL
      NJUNC(N,1)=MINO(NTEMP(1),NTEMP(2))
      NJUNC(N,2)=MAXO(NTEMP(1),NTEMP(2))
131   CONTINUE
1132  CONTINUE
C                              CHECK NETWORK
      NEXIT=0
      DO 150 N=1,NC
      DO 150 I=1,2
      IF(NJUNC(N,I).LE.0) GOTO 150
      J=NJUNC(N,I)
      DO 140 K=1,5
      IF(N.EQ. NCHAN(J,K)) GOTO 150
140   CONTINUE
      NEXIT=NEXIT+1
      WRITE(3,145) N,J
145   FORMAT(1H0, 28H COMPATIBILITY CHECK CHANNEL ,I4,9H JUNCTION ,I4)
150   CONTINUE
      DO 170 J=1,NJ
      DO 165 K=1,5
```

```
      IF(NCHAN(J,K).LE.0) GOTO 170
      N=NCHAN(J,K)
      DO 160 I=1,2
      IF(J.EQ.NJUNC(N,I)) GOTO 165
160   CONTINUE
      NEXIT=NEXIT+1
      WRITE(3,145) N,J
165   CONTINUE
170   CONTINUE
      IF(NEXIT.NE.0) CALLEXIT
C                            WRITE OUTPUT INFORMATION
      WRITE(3,123)   NJ
123   FORMAT(1H ,//////////,1H ,24H MAXIMUM JUNCTION NUMBEL   ,I5)
      WRITE(3,1024)
1024  FORMAT(1H ,8HJUNCTION,7X,7HINITIAL,5X,7HSURFACE,4X,8HPOROSITY,11X,
     16HINFLOW,4X,7HOUTFLOW,7X,26HCHANNELS ENTERING JUNCTION,3X,
     A 8HPOROSITY,5X,4HCODE,
     2 /,18X,4HHEAD,8X,4HAREA,6X,5HSLOPE,13X,5H(CFS),5X,5H(CFS),49X,
     3   7H1=FIXED,/,18X,4H(FT),7X,6H(SQFT),87X,6H0=FREE)
      DO 125 J=1,NJ
      IF(NCHAN(J,1).LE.0) GOTO 125
      WRITE(3,124) J,H(J),AS(J),ASK(J),QIN(J), QOU(J),(NCHAN(J,K),K=1,5),
     1  VOID(J), LST(J)
124   FORMAT(1H ,I7,F13.2,2(3X,1PE12.5),2X,0P2F10.1,4X,5I6,F10.5,5X,I5)
125   CONTINUE
      WRITE(3,133)   NC
133   FORMAT(1H ,//////////1H ,23H MAXIMUM CHANNEL NUMBER   ,I5)
      WRITE(3,1034)
1034  FORMAT(1H ,7HCHANNEL,3X,6HLENGTH,3X,5HWIDTH,5X,4HAREA,3X,5HPERM.,
     17X,7HINITIAL,6X,
     17HINITIAL,9X,17HJUNCTIONS AT ENDS,/,49X,8HVELOCITY,6X,5HDEPTH,/,
     211X,4H(FT),5X,4H(FT),4X,6H(SQFT),3X,8H(CM/SEC),4X,8H(FT/SEC)
     3,6X,4H(FT))
      DO 135 N=1,NC
      IF(NJUNC(N,1).LE.0) GOTO 135
      WRITE(3,134) N,LEN(N),B(N),A(N),AK(N),V(N),R(N),(NJUNC(N,K),K=1,2)
134   FORMAT(1H ,I5,F11.0,F8.0,F10.0,1PE10.2,0PE10.2,F13.1,10X,2I6)
135   CONTINUE
      STOP
      END
```

Program CMPRC

CMPRC computes rating curves from cross-sections and Manning's n. At least three values of Manning's n must be provided for each cross-section. One is for the channel and the other two are for either side in the floodplain. The channel value for n is designated by placing a minus sign in front of it. The input data required for CMPRC are defined below.

ID	Any number between 1 and 6.
NSEG	The number of segments in the cross-section to which a different Manning's n is to be assigned.
VS	Any number. This number is simply for identification of cross-sections.
ELO	Lowest elevation in cross-section.
EMAX	Highest elevation in cross-section.
SLOPE1	Channel slope.
SLOPE2	Floodplain slope.
DATA	The first 2xNSEG elements of the DATA array contain the Manning's n values, each followed by the segment boundary point as measured from one end of the cross-section.
DATA	The elements of the DATA array between 2xNSEG and NP are the horizontal and vertical positions of the points describing the cross-section. The points alternate with a horizontal value (as measured from one end of the cross-section) being first.
NP	The highest element to be filled in the DATA array. This value, then, is 2xNSEG + 2x (the number of elevation points in the cross-section).

The input data should be punched according to formats 26 and 27.

CMPRC punches the rating curve data on cards in a form acceptable to ALAMO and CMPTT. The output variables are defined below.

C	Water surface elevation in feet.
Q	Flow rate in cfs.
DEEP	Depth of water at matching Q.
A	Cross-sectional area of flow at matching Q.

Following is the program listing, sample input and output for CMPRC.

```
      PROGRAM CMPRC(INPUT,OUTPUT,PUNCH)
C     THIS PROGRAM COMPUTES RATING CURVES FROM CROSS SECTIONAL DATA.
      DIMENSION A(20,6),Q(20,6),DATA(50),DIST(6),ISG(6),SEGN(3),C(20),DE
     1EP(20,6)
      READ 26,ID,NP,NSEG,VS,ELO,EMAX,SLOPE1,SLOPE2,(DATA(I),I=1,6)
      PRINT 28
      PRINT 26,ID,NP,NSEG,VS,ELO,EMAX,SLOPE1,SLOPE2,(DATA(I),I=1,6)
C     READ IN ELEVATIONS AND DISTANCES FROM END OF CROSS SECTION.
      NSEG2=2*NSEG+1
      READ 27,(DATA(I),I=NSEG2,NP)
      PRINT 29,(DATA(I),I=NSEG2,NP)
      DIF=(EMAX-ELO)/19.
      C(1)=ELO
      DO 1 I=2,20
1     C(I)=C(I-1)+DIF
C     SET AREA AND DISCHARGE ARRAYS = 0.
      DO 2 I=1,20
      A(I,ID)=0.
2     Q(I,ID)=0.
      J=1
      PRINT 24,VS
      DO 3 I=1,NSEG
      SEGN(I)=DATA(J)
      DIST(I)=DATA(J+1)
3     J=J+2
C     REMAINING DATA ITEMS ARE DISTANCES AND ELEVATIONS.
      JJJ=J
      DO 6 I=1,NSEG
4     J=J+2
      IF (DATA(J)-DIST(I)) 4,5,5
5     ISG(I)=J+1
6     CONTINUE
C     COMPUTE DISCHARGES AND END AREAS FOR EACH SEGMENT.
      DO 22 K=1,NSEG
      J=JJJ
      JJJ1=JJJ+1
      IF (SEGN(K)) 7,7,8
7     SLOPE=SLOPE1
      SEGN(K)=-SEGN(K)
      GO TO 9
8     SLOPE=SLOPE2
9     SLPN=1.486*SLOPE**.5
C     COMPUTE AREA AND DISCHARGE FOR SEGMENT.
      DO 21 I=2,20
      AA=0.
      P=0.
      J=JJJ-1
      DEP2=0.
10    J=J+2
      IF (J-ISG(K)) 12,12,11
```

```
11     IF (AA-.001) 21,21,20
12     IF (DATA(J)-C(I)) 13,10,10
13     DEP1=C(I)-DATA(J)
       IF (J-JJJ1) 16,16,14
14     XL=DATA(J-1)-DATA(J-3)
       DEP3=ABS(DATA(J-2)-DATA(J))
       XL=XL*DEP1/DEP3
15     AA=AA+XL*(DEP1+DEP2)/2.
       P=P+SQRT((DEP1-DEP2)**2+XL**2)
16     DEP2=DEP1
       J=J+2
       IF (J-ISG(K)) 17,17,20
17     IF (DATA(J)-C(I)) 18,18,19
18     DEP1=C(I)-DATA(J)
       XL=DATA(J-1)-DATA(J-3)
       GO TO 15
19     DEP1=0.
       XL=DATA(J-1)-DATA(J-3)
       DEP3=ABS(DATA(J-2)-DATA(J))
       XL=XL*DEP2/DEP3
       AA=AA+XL*(DEP1+DEP2)/2.
       P=P+SQRT((DEP1-DEP2)**2+XL**2)
       DEP2=0.
       GO TO 10
20     R=AA/P
       SGN=SEGN(K)-.0025*R

C      ADD DISCHARGES AND AREAS FOR ALL SEGMENTS TO OBTAIN TOTALS FOR
C      VALLEY SECTION.

       Q(I,ID)=Q(I,ID)+AA*R**.66667*SLPN/SGN
       A(I,ID)=A(I,ID)+AA
21     CONTINUE
       JJJ=J-3
22     CONTINUE
       DO 23 I=1,20
       DEEP(I,ID)=C(I)-ELO
       PRINT 25, C(I),A(I,ID),Q(I,ID),DEEP(I,ID)
23     CONTINUE
       PUNCH 30,(Q(I,ID),I=1,20)
       PUNCH 30,(A(I,ID),I=1,20)
       PUNCH 40,(DEEP(I,ID),I=1,20)
C
24     FORMAT(1H0,T48,"RATING CURVE VALLEY SECTION",F5.1/T46,"WATER",T56,
      1"FLOW",T66,"FLOW",T76,*DEPTH* /T45,"SURFACE",T56,"AREA",T66,"RATE"
      2/T46,*ELEV*,T56,*SQ FT*,T66,*CFS*,T77,*FT*)
25     FORMAT (40X,F10.2,2F10.1,F10.2)
26     FORMAT(1X,I2,2I3,F4.1,2F6.0,2F6.4,2F6.2,4F7.2)
27     FORMAT(8F10.1)
28     FORMAT(1X,////,5X,*INPUT DATA*,/)
29     FORMAT(1X,8F10.1)
30     FORMAT(10F8.1)
40     FORMAT(10F8.2)
       STOP
       END
```

INPUT DATA

```
2 24  3 1.0 6670. 6680. .0021 .0071   .04 90.00   -.03 150.00    .04 200.00
    0.0     6680.0      35.0     6678.0       70.0    6675.0      91.0    6673.0
  110.0     6670.0     145.0     6672.0      160.0    6673.0     189.0    6676.0
  200.0     6680.0
```

RATING CURVE VALLEY SECTION 1.0

WATER SURFACE ELEV	FLOW AREA SQ FT	FLOW RATE CFS	DEPTH FT
6670.00	0.0	0.0	0.00
6670.53	3.3	3.8	.53
6671.05	13.2	24.7	1.05
6671.58	29.7	74.8	1.58
6672.11	52.8	166.4	2.11
6672.63	82.0	312.7	2.63
6673.16	117.1	542.7	3.16
6673.68	157.9	917.4	3.68
6674.21	204.3	1428.5	4.21
6674.74	256.3	2110.3	4.74
6675.26	313.8	3004.0	5.26
6675.79	377.3	4164.4	5.79
6676.32	446.3	5679.0	6.32
6676.84	519.5	7643.8	6.84
6677.37	596.6	10194.0	7.37
6677.89	677.8	13552.3	7.89
6678.42	763.5	18048.4	8.42
6678.95	854.7	24379.7	8.95
6679.47	951.5	33892.5	9.47
6680.00	1054.0	49764.1	10.00

Program CMPTT

Program CMPTT computes the travel times required for flow through a reach at given discharges. The main input data are at least one rating curve in the reach which is either known or has been calculated and punched by CMPRC. The input variables used for CMPTT are defined in the section of this Appendix entitled Program CMPRC as output variables. The remaining input variables for CMPTT are defined below.

ID	Any number between 1 and 6.
NOVS	The number of valley sections in the reach being considered.
REACH	Reach number is for identification only and can be any number between 1 and 999.9.
XL	Length of the reach in feet.
SLOPE	Either channel or floodplain slope or a weighted average of the two.

The output variables are punched and printed in a form acceptable to ALAMO and are defined below.

SCFS	Flow rate in cfs.
DP	Depth at a matching flow rate.
C	Travel time at a matching flow rate.

Following is a program listing, sample input, and output for program CMPTT.

```
      PROGRAM CMPTT(INPUT,OUTPUT,PUNCH)
C      THIS PROGRAM COMPUTES THE TRAVEL TIME AT GIVEN DISCHARGE RATES.
      DIMENSION DATA(310),Q(20,6),SCFS(20),A(20,6),CFS(300),DEEP(20,6),
     1DP(20),C(20),DIST(6)
      READ 18,ID,NOVS,REACH,XL,SLOPE

C     READ IN END AREAS, DISCHARGES, AND FLOWDEPTHS.

      READ 20,((A(I,J),I=1,20),J=1,NOVS)
      READ 20,((Q(I,J),I=1,20),J=1,NOVS)
      READ 30,((DEEP(I,J),I=1,20),J=1,NOVS)
      PRINT 22
      PRINT 23,ID,NOVS,REACH,XL,SLOPE
      PRINT 24,((A(I,J),I=1,20),J=1,NOVS)
      PRINT 24,((Q(I,J),I=1,20),J=1,NOVS)
      PRINT 25,((DEEP(I,J),I=1,20),J=1,NOVS)
      DIST(ID)=SLOPE*XL
      XLD36=XL/3600.

C     ZERO ARRAYS

      DO 1 J=1,20
      DATA (J)=0.
1     CFS(J)=0.
      ID1=1

C     FIND RATING CURVE WITH SMALLEST MAXIMUM FLOW RATE

2     QMIN=Q(20,ID1)
      MIN=ID1
      GO TO 4
3     ID1=ID1+1
      IF (QMIN-Q(20,ID1)) 4,4,2
4     IF (ID1-NOVS) 3,5,5
5     I=1

C     SET SCFS ARRAY EQUAL TO Q ARRAY OF LOWEST RATING CURVE

      DO 6 J=2,20
      SCFS(I)=Q(J,MIN)
6     I=I+1

C     COMPUTE END AREA AND DEPTH

      DO 9 ID1=1,NOVS
      DO 9 J=1,19
      DO 7 I=2,20
      IF (Q(I,ID1)-SCFS(J)) 7,17,8
7     CONTINUE
17    DATA (J)=A(I,ID1)+DATA(J)
      CFS(J)=DEEP(I,ID1)+CFS(J)
      GO TO 9
8     XY=(SCFS(J)-Q(I-1,ID1))/(Q(I,ID1)-Q(I-1,ID1))
      DATA (J)=A(I-1,ID1)+XY*(A(I,ID1)-A(I-1,ID1))+DATA(J)
      CFS(J)=DEEP(I-1,ID1)+XY*(DEEP(I,ID1)-DEEP(I-1,ID1))+CFS(J)
9     CONTINUE
      XNOVS=NOVS
11    PRINT 15,  ID,REACH,XL,SLOPE,NOVS
      PRINT 13, REACH

C     COMPUTE TRAVEL TIME

      DO 10 I=1,19
```

```
      AVAREA=DATA(I)/XNOVS
      DP(I)=CFS(I)/XNOVS
      S=AVAREA*XLD36
      C(I)=S/SCFS(I)
      PRINT 14, DP(I),SCFS(I),C(I)
10    CONTINUE
C     PRINT AND PUNCH OUTPUT.
      PUNCH 16,(DP(I),I=1,19)
      PUNCH 19,(SCFS(I),I=1,19)
      PUNCH 21,(C(I),I=1,19)
C
13    FORMAT(1H0,T50,"TRAVEL TIME TABLE"/T54,"REACH",F5.1//T46,"WATER",T
     156,"FLOW",T66,"TRAVEL"/T46,"DEPTH",T56,"RATE",T66,"TIME"/T46,"FEET
     2",T56,"CFS",T66,"HRS")
14    FORMAT (40X,F10.2,F10.0,F10.4)
15    FORMAT(1X,/////////,5X,*ID=*,I1,5X,*REACH NO.=*,F4.1,5X,*LENGTH=*,F
     18.1,5X,*SLOPE=*,F8.5,5X,*NO. OF VALLEY SECTIONS IN REACH=*,I1,//)
16    FORMAT(10F8.2)
18    FORMAT(2I5,F5.1,2F10.3)
19    FORMAT(10F8.0)
20    FORMAT(10F8.1)
21    FORMAT(10F8.3)
22    FORMAT(1X,/////,1X,*INPUT DATA*,/)
23    FORMAT(1X,2I5,F5.1,2F10.3)
24    FORMAT(1X,10F8.1)
25    FORMAT(1X,10F8.2)
30    FORMAT(10F8.2)
      STOP
      END
```

INPUT DATA

1	2	1.0	1000.000	.007					
0.0	3.3	13.2	29.7	52.8	82.0	117.1	157.9	204.3	256.3
313.8	377.3	446.3	519.5	596.6	677.8	763.5	854.7	951.5	1054.0
0.0	5.7	23.0	47.8	77.6	112.1	151.1	196.5	249.0	308.5
375.0	449.2	530.7	618.3	711.9	811.6	917.9	1031.3	1152.0	1280.0
0.0	3.8	24.7	74.8	166.4	312.7	542.7	917.4	1428.5	2110.3
3004.0	4164.4	5679.0	7643.8	10194.0	13552.3	18048.4	24379.7	33892.5	49764.1
0.0	10.0	66.1	202.6	424.8	766.2	1240.3	1910.4	2780.9	3892.9
5291.0	7035.8	9218.1	11926.6	15270.7	19420.2	24577.7	31125.6	39628.3	51013.2
0.00	.53	1.05	1.58	2.11	2.63	3.16	3.68	4.21	4.74
5.26	5.79	6.32	6.84	7.37	7.89	8.42	8.95	9.47	10.00
0.00	.53	1.05	1.58	2.11	2.63	3.16	3.68	4.21	4.74
5.26	5.79	6.32	6.84	7.37	7.89	8.42	8.95	9.47	10.00

ID=1 REACH NO.= 1.0 LENGTH= 1000.0 SLOPE= .00700

TRAVEL TIME TABLE
REACH 1.0

WATER DEPTH FEET	FLOW RATE CFS	TRAVEL TIME HRS
.37	4.	.1998
.86	25.	.1318
1.33	75.	.1008
1.77	166.	.0785
2.24	313.	.0642
2.72	543.	.0529
3.24	917.	.0426
3.76	1429.	.0358
4.27	2110.	.0306
4.79	3004.	.0266
5.32	4164.	.0233
5.85	5679.	.0205
6.39	7644.	.0180
6.94	10194.	.0158
7.49	13552.	.0137
8.07	18048.	.0119
8.67	24380.	.0101
9.29	33893.	.0083
9.97	49764.	.0065

REFERENCES

Anderson, T. W. 1972. Electrical analog analysis of the hydrologic system. U. S. Geological Survey Water Supply Paper 1939-C, 34 pages.

Babcock, H. M., and E. M. Cushing. 1942. Recharge to ground water from floods in a typical desert wash. Transactions of the American Geophysical Union, Vol. 23, pp. 49-56.

Besbes, M., J. P. Delhomme, and G. de Marsily. 1978. Estimating recharge from ephemeral streams in arid regions: A case study at Kairorian, Tunisia. Water Resources Research 14(2): 281-290.

Burkham, D. E. 1970. Depletion of streamflow by infiltration in the main chanels of the Tucson Basin, southeastern Arizona. U. S. Geological Survey Water Supply Paper 1939-B, 36 pages.

Crim, R. L. 1972. Numerical method for ground water hydraulics. Environmental Protection Agency, Annapolis, Md., Water Quality Office, EPA Technical Report 52, 29 pages.

Fogel, M. M., and L. Duckstein. 1969. Point rainfall frequencies in convective storms. Water Resources Research 5(6):1229-1237.

Fogel, M. M., and L. Duckstein. 1970. Prediction of convective storm runoff in semiarid regions. Proceedings, Symposium on the Results of Reach on Representative and Experimental Basins. Wellington, New Zealand, pp. 465-478.

Hillel, D. 1977. Computer simulation of soil-water dynamics: A compendium of recent work. International Development Research Centre, 81 pages.

Keppel, R. V., and K. G. Renard. 1962. Transmission losses in ephemeral sand-gravel bed streams of the Southwest. Proceedings ASCE, Journal of the Hydraulics Division, 88(HY3): 38-43.

Matlock, W. G. 1965. The effect of silt-laden water on infiltration in alluvial channels. Ph.D. Dissertation, The University of Arizona, Tucson, 102 pages.

Neuman, S. P., and P. A. Witherspoon. 1970. Finite element method of analyzing steady seepage with a free surface. Water Resources Research 6(3):889-897.

Qazi, R., and J. Danielson. 1974. Analytical model for management of alluvial aquifers. Proceedings ASCE, Journal of the Irrigation and Drainage Division, 100(IR2):143-152.

Sharp, A. L., and K. E. Saxton. 1962. Transmission losses in natural stream valleys. Proceedings ASCE, Journal of the Hydraulic Division, 88(HY5):121-142.

United States Department of Agriculture. 1969. Soil Conservation Service, National Engineering Handbook, Hydrology, Section 4:10.1-17.48.

Walton, W. C. 1970. Ground Water Resource Evaluation. McGraw-Hill Book Company, 127 pages.

Williams, J. R. 1969. Flood routing with variable travel time or variable storage coefficients. Trans. American Society for Agricultural Engineers 12(1):100-103.

Williams, J. R., and R. W. Hann, Jr. 1973. HYMO: Problem-oriented computer language for hydrologic modeling user's manual. U. S. Department of Agriculture, Agricultural Research Service, Southern Region, ARS-S-9, 76 pages.