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# A comparison of three urban hydrology models

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A COMPARISON OF THREE URBAN HYDROLOGY MODELS

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This thesis has been prepared in the style utilized by the Journal of the Hydraulics Division, American Society of Civil Engineers. Appendix III has been added for purposes normal to thesis writing.

# CIVIL ENGINEERING ABSTRACT

A Comparison of Three Urban Hydrology Models, by Richard F. Astrack. Three models, the British Road Research Laboratory Hydrograph Method, the Hydrocomp Simulation Program, and the HEC-1 Flood Hydrograph Package, are applied to the Boneyard Creek basin. Comparing the ability of these models to reproduce the observed peak discharge, time of the peak, and the first runoff volume determines the accuracy of these models.

# A COMPARISON OF THREE URBAN HYDROLOGY MODELS

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KEY WORDS: Comparison: Computer; Hydraulics; Mathematical models; Overland flow; Runoff; Simulation; Urban hydrology.

ABSTRACT: A comparison of three methods used for the calculation of urban stormwater runoff is presented. The Hydrocomp Simulation Program, the British Road Research Laboratory Hydrograph Method, and the HEC-1 Flood Hydrograph Package are applied to the Boneyard Creek basin located in Champaign-Urbana, Illinois. Simulation results are based on the accuracy of these models to reproduce observed peak discharges, time of the peak, and the direct runoff volume. This comparison demonstrates that all three methods will satisfactorily simulate urban runoff within certain stated limitations.

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# A COMPARISON OF THREE URBAN HYDROLOGY MODELS

by Richard F. Astrack,[1] A.M. ASCE

## INTRODUCTION

In the past decade, the major emphasis of hydrology has shifted to the urban scene. This change can be illustrated by the changing mission of the U. S. Army Corps of Engineers. Ten years ago, the Corps had under consideration only a few single-purpose flood control studies located in urban areas. Now, the Corps' urban studies are comprehensive regional water resource studies which generally include consideration of wastewater management, flood control, flood plain management, water quality management, recreation, water supply, and environmental enhancement. With such a major shift in emphasis, the hydrologist needs methods available for determining the hydrologic response of urban watersheds.

The important characteristics of any runoff hydrograph are its peak discharge, timing of the peak, and direct runoff volume. Depending on the situation, one, two, or all three hydrograph characteristics may have to be determined. The peak discharge, generally the most important hydrograph parameter, determines the magnitude of flooding or the size of most hydraulic structures. For large basins or basins with highly variable characteristics, the timing of the peak discharge is important in determining how much in phase the subarea hydrographs are as they are routed and combined.

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Volume of direct runoff is an important hydrograph characteristic in the design of storage facilities. The volume determines the size of the reservoir and its outlet facilities for both large and small detention structures. Therefore, depending on the situation, the determination of any or all of these hydrograph characteristics may be critical.

This application deals with three runoff simulation models, each of which has been available for at least five years. The three hydrograph simulation models are applied to a gaged, urbanized watershed, Boneyard Creek, located in Champaign-Urbana, Illinois. The models are evaluated according to their ability to reproduce the observed hydrograph in peak discharge, timing of the peak, and direct runoff volume. The British Road Research Laboratory Hydrograph Method (10)[2] was developed exclusively for design of urban runoff water systems. A general model, the Hydrocomp Simulation Program (7) was developed from the Stanford Watershed Model and is copyrighted by Hydrocomp, Inc. The final method used, the HEC-1 Flood Hydrograph Package (5), developed by the U. S. Army Corps of Engineers, is a general model based on the unit hydrograph theory. The following material presents each hydrograph simulation model and discusses their applicability and limitations in urban hydrology.

## THE BRITISH ROAD RESEARCH LABORATORY HYDROGRAPH METHOD

The British Road Research Laboratory Hydrograph Method (RRL method) was developed as the result of a comprehensive research program of the

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[2]Numerals in parentheses refer to corresponding items in the Appendix I. - References.

British Road Research Laboratory (13). The initial RRL method was introduced to the United States in 1969 by Stall and Terstriep (11) and later revised and applied to 10 basins located throughout the United States (10).

The revised RRL method has several features which make it desirable for use as either a storm drain design method or a research tool. It can be used to analyze existing systems or to design new ones. The model can provide an entire runoff hydrograph for a simple or complex storm. The following material describes the necessary data preparation to apply the RRL method to an urban watershed.

*Development of Basin Characteristics.* First, the basin is divided into sub-basins, i.e., areas of the basin contributing to one or a set of inlets. For each sub-basin, the impervious area directly connected to the storm drainage system is determined. If roof outlets either enter the drainage system by direct underground connection or empty onto an impervious surface (driveway or sidewalk) which is directly connected to the street gutter, then this additional impervious roof area is added to the total impervious portion of the sub-basin.

Travel times are computed for the gutter flow by assuming a design flow of 0.5 cubic feet per second (0.014 $m^3$) per acre of impervious area and using Manning's equation as modified by Izzard (8) to compute flow velocities. The equation is as follows:

$$Q = 0.56 \left(\frac{Z}{n}\right) S^{1/2} y^{8/3} \quad \ldots\ldots\ldots\ldots \quad (1)$$

in which

$Q$ = discharge in cfs ($m^3$/sec)

$y$ = maximum depth in feet (m)

$Z$ = ratio of width to $Y$

$n$ = Manning's roughness coefficient

$S$ = average channel slope in ft/ft (m/m)

Using the above equation, the flow depth is determined. Assuming uniform flow, then the flow velocity and thus travel time are computed for the gutter flow.

With the travel time calculated, isochrons of equal travel time are plotted. The area in each isochron is measured and a time-area curve prepared. Thus, the impervious area contributing during each time interval is known.

*Hydrograph Computation*. The runoff hydrograph for each sub-basin is computed by applying the effective rainfall rate to the time-area curve. The effective rainfall is total rainfall minus losses. The losses, on the order of magnitude of 0.1 inches (.254 cm) as selected by the user, represent initial wetting and depression storage. If the areas for each time interval of the time-area relationship are designated $A_1$, $A_2$, $A_3 \ldots$, $A_n$ and if the effective rainfall intensity of each interval ($I_1$, $I_2, \ldots$, $I_n$) that is applied to $A_1$ is designated $I_{1-1}$, $I_{1-2}, \ldots$, $I_{1-n}$ and that is applied to $A_2$ is designated $I_{2-1}$, $I_{2-2}, \ldots$, $I_{2-n}$ and so on, then the runoff hydrograph ordinates ($Q_1$, $Q_2$, $Q_3, \ldots$, $Q_n$) are computed as:

$$Q_0 = 0. \qquad (2)$$

$$Q_1 = I_{1-1} A_1. \qquad (3)$$

$$Q_2 = I_{1-2} A_2 + I_{2-1} A_1. \qquad (4)$$

$$Q_3 = I_{1-3} A_3 + I_{2-2} A_2 + I_{2-1} A_1. \qquad (5)$$

$$Q_n = I_{(1-n)} A_{(n-1)} + \ldots + I_{(n-1)} A_1. \qquad (6)$$

*Hydrograph Routing*. With all of the sub-basin hydrographs computed, the furthest upstream sub-basin hydrograph is routed downstream to the next inlet. This hydrograph is then combined with the second sub-basin hydrograph and this new hydrograph routed to the next inlet. This procedure of routing and combining hydrographs is continued through the basin keeping the timing of the sub-basin hydrographs correct.

The sub-basin hydrographs are routed from inlet to inlet using a simple storage routing technique (4). The technique requires that a relationship must be determined between discharge and storage. A stage-discharge curve is developed by using Manning's equation. Then, by assuming uniform flow and knowing the channel geometry for the reach, the discharge-storage relationship is computed for the reach. The discharge out of the reach for the first time interval is computed by the following equation:

$$1/2\ I\ t = S_1 + 1/2\ O\ t \qquad (7)$$

$I$ = discharge into the reach in cfs ($m^3/s$)

$O$ = discharge out of the reach in cfs ($m^3/s$)

$S_1$= storage during the first time interval in cu ft ($m^3$)

$t$ = time interval in seconds

For the time interval from $t$ to $2t$

$$(I_1 + I_2) = \frac{2(S_2 - S_1)}{t} + (O_1 + O_2) \qquad (8)$$

Since $S_2$ is determined from the discharge-storage relationship and all the other parameters except $O_2$ are known, then the discharge out of the reach at the end of the second time interval is computed. Using this step-by-step procedure, all ordinates of the routed hydrograph may be determined.

## THE HYDROCOMP SIMULATION PROGRAM

The Hydrocomp Simulation Program (HSP) does the complete set of computations needed to predict storm runoff from rainfall. Moisture accounting procedures keep track of the amount of water going into and out of each component of the hydrologic cycle. Changes in soil moisture storage are established as the model continuously computes evapotranspiration losses, percolation to groundwater, interflow discharge, and groundwater discharge to the stream. Continuous operation of HSP results in a moisture balance maintained throughout the basin such that basin moisture conditions are consistent with the antecedent rainfall. A complete time history of runoff, as opposed to runoff for a single event as with the RRL method, is available.

Rainfall is first applied to the simulation model. Then the model continuously models the interception, infiltration, interflow, upper zone storage, lower zone storage, groundwater storage, evapotranspiration, the resulting depth of overland flow, and stream flow. The following material describes how the above components are modeled by HSP.

*Description of Modeling the Components.* The first loss encountered by the rainfall is interception. This is rain that is retained on vegetation, and this loss must be filled to capacity before any other factors may act on the rainfall. Water is removed from the interception storage by evaporation. Typical values for the interception storage capacity are: 0.10 (.254 cm) inches for grassland; 0.15 (.381 cm) inches for light forest cover; and, 0.20 (.508 cm) inches for heavy forest cover. In determining runoff from impervious areas of the basin, interception is the only loss applied to the rainfall.

Infiltration is the movement of water through the soil surface into the soil profile. Since infiltration is dependent on the basin characteristics, soil type, percolation, slope, cover, and soil moisture content, continuous simulation must first represent a mean infiltration rate for a relatively uniform portion of the basin, and secondly, represent the areal variation in infiltration. The first requirement is satisfied by using the following empirical infiltration equation:

$$f\,F^2 = \text{constant} \quad \ldots\ldots\ldots\ldots \quad (9)$$

where $f$ = infiltration rate in in/hr (cm/hr)

$F$ = cumulative infiltration in inches (cm)

By assuming that the cumulative infiltration capacity for the basin is a linear relationship, the second requirement, the areal variation of infiltration, is modeled.

Interflow, that portion of infiltration which becomes runoff after moving laterally in the soil, is defined by the empirical equation:

$$c = \text{INTERFLOW} \cdot 2^{(LZS/\text{LZSN})} \quad \ldots\ldots\ldots\ldots \quad (10)$$

where $c$ = interflow

$LZS$ = current soil moisture storage level

LZSN = index level of soil moisture storage

INTERFLOW = variable, determined only through calibration runs

The depth of rainfall remaining after subtraction of interception, interflow and infiltration will either enter upper zone storage or contribute to overland flow. The upper zone models depression storage and storage in highly permeable surface soils. The upper zone storage is represented by a series of empirical expressions of which the major feature is the upper zone storage ratio (*UZS*/UZSN) where *UZS* is the upper zone storage and UZSN is the normal capacity of the upper zone storage. Various values of UZSN based on the watershed topography are presented in the HSP Operations Manual (7). Percolation will occur from the upper zone to the groundwater and lower zone storages if the upper zone storage ratio (*UZS*/UZSN) is greater than the soil moisture content ratio (*LZS*/LZSN).

Evapotranspiration is continuously subtracted from the upper zone storage. Similarily to the modeling of the areal variation of infiltration, continuous estimation of the actual evapotranspiration is made by a linear approximation of the process.

Infiltration and percolation from the upper zone storage enter the lower zone or soil moisture storage and groundwater storage. As the total water available from the upper zone is known, the soil moisture storage is determined by subtraction of the groundwater storage from the total. The percent of infiltration entering groundwater storage is defined as an S-shaped curve when related to the soil moisture content ratio. If the value of *LZS*/LZSN is 1.0, fifty percent of the infiltration enters the groundwater storage. When *LZS*/LZSN is greater than 2.3, all of the infiltration enters the groundwater storage.

*Overland Flow*. The overland flow depth is the total of effective rainfall depth, outflow depth from interflow, and outflow depth from groundwater storage. The outflow from interflow is computed by an equation relating the interflow $c$ and a daily recession constant of interflow. The recession constant is simply the ratio of the present interflow discharge to the interflow discharge one day earlier. The amount of outflow from groundwater storage is evaluated by an equation based on the groundwater storage, the slope of the groundwater surface, and a recession constant of groundwater flow.

Overland flow is treated as a turbulent flow process. The following equation is an empirical (7) relationship between outflow depth and detention storage.

$$y = D/L\ (1.0 + 0.6\ (D/De)^3) \qquad (11)$$

where $D$ = ratio of detention depth at any instant

$L$ = length of the flow plain in feet (m)

$De$ = surface detention storage at equilibrium in cu ft/ft ($m^3/m$)

Substituting the above relationship into the Chezy-Manning equation yields:

$$q = \frac{1.486}{n}\ S^{1/2}\ (D/L)^{5/3}\ (1.0 + 0.6\ (D/De)^3)^{5/3} \qquad (12)$$

where $q$ = discharge in cfs ($m^3/s$)

$n$ = Manning's roughness coefficient

$S$ = average slope of the ground surface in ft/ft (m/m)

The following continuity equation is explicitly solved by HSP to obtain depth at selected time intervals.

$$\mathcal{D}_2 = \mathcal{D}_1 + \Delta\mathcal{D} - \overline{q}\,\Delta t \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (13)$$

where $\mathcal{D}_2$ = surface detention at end of current time interval in cu ft ($m^3$)

$\mathcal{D}_1$ = surface detention at end of previous time interval in cu ft ($m^3$)

$\Delta\mathcal{D}$ = increment added to surface detention during the time interval in cu ft ($m^3$)

$\overline{q}$ = overland flow into the stream channel during the time interval

$\Delta t$ = time interval in seconds

Usually, calculations of discharge from overland flow are made on a 15-minute interval which is sufficiently small so that the value of discharge in any time interval is a small fraction of the volume of surface detention. However, for small basins, a smaller computational interval must be used so the above difference relationships between discharge and surface storage approximate the true solution of the equations.

*Flow Routing.* The overland flow is routed through the basin using the "Kinematic wave" (6) procedure. The channel system is divided into reaches with the channel dimensions, lengths, cross sections, and Manning's $n$ being determined. The tributary area to each reach is also determined.

The local inflow for each channel reach is computed from the overland flow depth and the tributary area. The channel routing proceeds from the upstream reach downstream in order to the outlet. Each reach receives flow from its tributary area and from upstream reaches. Continuous stage, flow velocities, and discharges are computed using Manning's equation:

$$Q = (\textit{area of flow}) \cdot \frac{1.49}{n} R^{2/3} S_o^{1/2} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \quad (14)$$

where $Q$ = discharge in cfs ($m^3/s$)

$n$ = Manning's roughness coefficient

$R$ = hydraulic radius in ft (m)

$S_o$ = average ground slope in ft/ft (m/m)

## HEC-1 FLOOD HYDROGRAPH PACKAGE

The HEC-1 Flood Hydrograph Package (HEC-1) was developed in 1967 by the Hydrologic Engineering Center, U. S. Army Corps of Engineers. HEC-1, a combination and expansion of a number of computer programs, does hydrologic calculations of the following processes: unit hydrograph and rainfall loss rate optimization; basin rainfall and snowmelt; unit hydrographs and hydrographs; streamflow routing optimization; hydrograph combining and routing; and, balanced hydrographs.

All ordinary flood hydrograph computations associated with a single recorded or hypothetical storm are accomplished by HEC-1 (4). The best fit unit hydrograph and rainfall loss rate coefficients are derived automatically. The following material describes the computational mechanisms used by HEC-1.

*Hydrograph Computation.* Processes internal to HEC-1 for hydrograph computation use the Clark Instantaneous Unit Hydrograph (2), a hydrograph that would result from one unit of rainfall excess occurring over the basin in a specified areal pattern and zero time. Thus, the instantaneous unit hydrograph can be used to compute a unit hydrograph for any unit duration equal to or greater than the time interval used in the computations. Two parameters, the time of concentration (TC) and hydrograph storage coefficient (R), along with a time-area relationship, are required to compute the Clark Unit Hydrograph.

The time of concentration is defined as the travel time of water from the furthest upstream point (timewise) in the basin to the outflow location. On a hydrograph, TC is approximately the time from the end of rainfall excess to the point of inflection on the recession limb of the hydrograph. The hydrograph storage coefficient is defined by the following equation:

$$R = -Q_I/S_I \qquad (15)$$

where R = hydrograph storage coefficient

$Q_I$ = discharge at the point of inflection on the recession limb of the hydrograph in cfs ($m^3/s$)

$S_I$ = slope of a line tangent to the point of inflection on the recession limb of the hydrograph

The time-area relationship is developed by laying out isochrones representing equal travel time using the distance traveled per unit of time to establish the locations of the lines. The areas between the isochrones are then measured and tabulated with the corresponding travel time for each incremental area. It should be noted that a synthetic time-area curve based on the following equations will be used by the program unless one is developed for the basin as described above.

$$AI = T^{1.5}/.707 \text{ for } (0<T<.5) \quad (16)$$

$$1-AI = (1-T)^{1.5}/.707 \text{ for } (.5<T<1) \quad (17)$$

where $AI$ = percent of the total basin area contributing at time T

$T$ = ratio of time T to time of concentration

*Computation of Effective Rainfall.* As in the previous methods, the effective rainfall must be determined. In HEC-1, rainfall loss is computed using the exponential rainfall loss curve as defined in Figure 1 or by selecting appropriate initial and uniform loss rates. The exponential rainfall loss curve is defined by the following equations:

$$ALOSS = (AK + DLTK)(RAIN)^{ERAIN} \quad (18)$$

$$AK = STRKR/(RTIOL)^{.1\ CUML} \quad (19)$$

$$DLTK = 0.2\ DLTKR\ (1-(CUML/DLTKR))^2 \quad (20)$$

for $(CUML/DLTKR) <1$; otherwise zero

where $ALOSS$ = loss rate in inches/hour (cm/hr)

$AK$ = basic loss coefficient

$DLTK$ = incremental loss coefficient

$RAIN$ = rainfall in inches/hour (cm/hr)

$CUML$ = accumulated loss in inches (cm)

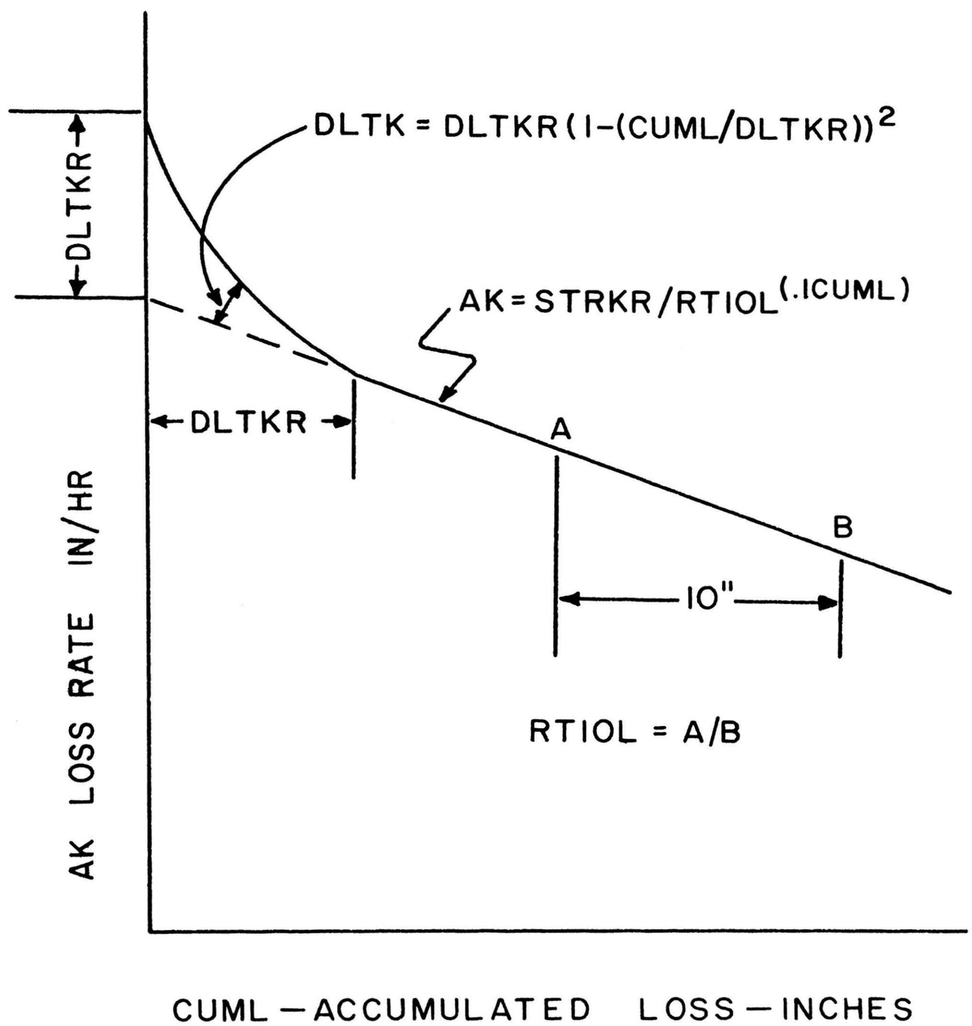
$ERAIN$ = variable relative to how storms occur over the basin

$RTIOL$ = ratio of $AK$ to that $AK$ after an additional 10 inches of accumulated loss occurs

$STRKR$ = loss index for the start of the storm in inches/hours (cm/hr)

$DLTKR$ = incremental loss index

FIGURE 1. - HEC-1 GENERAL LOSS RATE FUNCTION


DLTK = DLTKR(1-(CUML/DLTKR))^2
DLTKR
AK=STRKR/RTIOL^(.1CUML)
DLTKR
A
B
10"
RTIOL = A/B
AK LOSS RATE IN/HR
CUML — ACCUMULATED LOSS — INCHES

It should be noted that *ERAIN* and *RTIOL* are regional parameters, and that *STRKR* and *DLTKR* are related directly to the antecedent moisture conditions.

*Calibration.* The available streamflow and corresponding rainfall data are obtained for the basin under consideration. Approximately five to ten rainfall-runoff events are required to calibrate the HEC-1 model. The rainfall and runoff data, plus the drainage area of the basin are input in HEC-1 and six variables, two unit hydrograph and four rainfall loss rate, are optimized for each storm event using the univariate gradient search procedure (1)(9). The best or optimum reconstitution is considered to be the one which minimizes the weighted squared deviations between the observed and a reconstituted hydrograph. With the variables optimized for each storm event, the best single value for each variable is determined for all the storm events.

## THE HEC-1 APPLICATION TO THE BONEYARD CREEK BASIN

In 1969, the RRL method (11) was applied to the Boneyard Creek watershed. The HSP model was applied to the Boneyard Creek basin in 1971 (3) as part of an urban runoff simulation study. With these studies complete and detailed rainfall and streamflow data available, the HEC-1 model was applied to this same basin. Thus, comparisons could be made between these three simulation models.

*Watershed Description.* The Boneyard Creek watershed is an entirely urbanized area with about 44 percent of the 3.58 square mile (9.27 $km^2$) drainage basin having impervious cover. The watershed contains a portion of the University of Illinois, old and new residential areas, and a sizeable commercial area. A major portion of the basin is served by a storm drainage system that discharges into approximately three miles (4.8 km) of open channels. One recording U. S. Geological Survey stream gage, and five recording rain gages provided the basic data available for calibrating the simulation models.

*Model Calibration.* The first step in calibration of the HEC-1 model was the utilization of available rainfall and runoff data (12). These actual recorded data and the unit hydrograph and rainfall loss rate optimization option of the HEC-1 model, were used to automatically determine the optimum value for each of the unit hydrograph variables, TC and R, and each of the rainfall loss rate variables, *ERAIN*, *RTIOL*, *DLTKR* and *STRKR* for each storm event. The first set of values in Table 1 presents the results of a "wide open" optimization run, with all unit hydrograph and loss rate variables optimized.

The best value of TC and R for the basin can thus be determined. This is accomplished through a series of trial runs where the value of TC and R is incrementally changed. The same value of TC and R is used for each storm event, with just the four rainfall loss rate variables being optimized. The first value of TC and R is determined as the approximate average of all the values optimized in the first "wide open" runs. The first incremental change in TC is a 100 percent increase of the initial average value. Thereafter, as seen in Table 1, the incremental change in TC and R is reduced from trial to trial as the best fit value is approached.

As the incremental change of both TC and R may be positive or negative, the results of the changes in TC and R are evaluated by a visual comparison of plots of the recorded and computed hydrographs for each

TABLE 1. - OPTIMIZATION BY HEC-1 MODEL

| STORM | FROZEN VARIABLE | UNIT HYDROGRAPH COEFFICIENTS TC | R | RAINFALL LOSS COEFFICIENTS STRKR | ERAIN | DLTKR | RTIOL |
|---|---|---|---|---|---|---|---|
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) |
| 19 Jul 63 | None | .35 | .95 | .49 | .77 | 1.09 | 1.75 |
| 28 Aug 63 | | .12 | 1.19 | .50 | .30 | 1.02 | 1.24 |
| 19 Apr 64 | | .19 | 1.56 | .57 | .69 | .09 | 1.0 |
| 25 May 65 | | .51 | 1.09 | .48 | .44 | 1.03 | 1.36 |
| 2 Jul 65 | | .11 | 1.29 | .75 | .46 | 1.47 | 1.0 |
| 19 Jul 63 | TC, R | .20 | 1.0 | .53 | .99 | 1.0 | 2.74 |
| 28 Aug 63 | | | | .45 | .32 | 1.10 | 1.0 |
| 19 Apr 64 | | | | .68 | 1.0 | .24 | 1.0 |
| 25 May 65 | | | | .60 | 1.0 | .81 | 4.48 |
| 2 Jul 65 | | | | .87 | .47 | 1.21 | 1.0 |
| 19 Jul 63 | TC, R | .40 | .80 | .51 | .79 | 1.07 | 2.01 |
| 28 Aug 63 | | | | .57 | .30 | .94 | 1.0 |
| 19 Apr 64 | | | | .61 | .87 | .25 | 1.0 |
| 25 May 65 | | | | .58 | 1.0 | .81 | 3.05 |
| 2 Jul 65 | | | | .83 | .45 | 1.43 | 1.0 |
| 19 Jul 63 | TC, R | .50 | .70 | .66 | .50 | .95 | 1.66 |
| 28 Aug 63 | | | | .61 | .19 | .97 | 1.0 |
| 19 Apr 64 | | | | .55 | .59 | .14 | 1.0 |
| 25 May 65 | | | | .57 | .84 | .88 | 2.75 |
| 2 Jul 65 | | | | .83 | .45 | 1.43 | 1.0 |
| 19 Jul 63 | TC, R | .60 | .70 | .66 | .31 | 1.04 | 2.53 |
| 28 Aug 63 | | | | .60 | .14 | 1.03 | 1.0 |
| 19 Apr 64 | | | | .62 | .88 | .14 | 1.0 |
| 25 May 65 | | | | .57 | .48 | .94 | 2.24 |
| 2 Jul 65 | | | | .82 | .46 | 1.41 | 1.0 |
| 19 Jul 63 | TC, R | .50 | .90 | .50 | .40 | 1.18 | 1.76 |
| 28 Aug 63 | | | | .50 | .12 | 1.13 | 1.0 |
| 19 Apr 64 | | | | .61 | .83 | .18 | 1.0 |
| 25 May 65 | | | | .59 | .41 | .93 | 2.96 |
| 2 Jul 65 | | | | .84 | .40 | 1.45 | 1.0 |
| 19 Jul 63 | TC, R | .50 | 1.0 | .51 | .17 | 1.21 | 1.0 |
| 28 Aug 63 | | | | .51 | .12 | 1.12 | 1.0 |
| 19 Apr 64 | | | | .60 | .80 | .16 | 1.0 |
| 25 May 65 | | | | .61 | .35 | .91 | 2.11 |
| 2 Jul 65 | | | | .91 | .37 | 1.34 | 1.0 |
| 19 Jul 63 | TC, R | .50 | 1.0 | .51 | .40 | 1.16 | 2.0 |
| 28 Aug 63 | ERAIN | | | .52 | | .99 | |
| 19 Apr 64 | RTIOL | | | .50 | | .04 | |
| 25 May 65 | | | | .69 | | .72 | |
| 2 Jul 65 | | | | .98 | | 1.32 | |

storm. The present computed hydrograph is compared to the previous computed hydrograph. If this latest change in TC or R improved the fit of the computed hydrograph, then the value is further incremented. This procedure continues until the fit of the computed hydrograph is no longer improved, thereby selecting the previous value of TC or R as the best fit value.

The second through seventh series of trial runs in Table 1 presents the results of the trial runs as the best fit values of TC, 0.50, and R, 1.0, are determined. It is seen that the value of TC was increased as the plotted hydrographs were compared. Each increase up to a TC value of 0.5 resulted in a better hydrograph fit. When a TC of 0.6 was tried, the computed hydrograph did not fit the observed hydrograph as well as for the last previous value of TC. Thus, the optimum value of TC was determined. Likewise, the value of R was incrementally changed arriving at the best value, even though results indicate the initial estimate of R equal to 1.0 was the best fit value.

The rainfall loss rate variables were then calibrated to the basin. Since two variables, *ERAIN* and *RTIOL*, are regional; the other two variables, *STRKR* and *DLTKR*, are the only variables dependent on basin characteristics and particularly antecedent soil moisture conditions. As a result of other studies completed in this region, values of *ERAIN* and *RTIOL* of 0.4 and 2.0, respectively, were utilized in the last optimization run.

The final HEC-1 calibration run was made using the best fit unit hydrograph values and the regional values for *ERAIN* and *RTIOL*. Only the two loss rate variables, *DLTKR* and *STRKR*, were optimized. The last series of values in Table 1 shows the results of this final calibration run. The results indicate that the minimum value for *STRKR* is 0.50 for these storm events. This value increases to about 1.0 as the antecedent soil moisture content decreases. The range of values for *DLTKR* is 0.4 to 1.32. Low values of *DLTKR* indicate high antecedent soil moisture.

Figure 2 is a plot comparing the final computed and observed hydrographs for the 19 April 1964 storm event. The representative plot indicates the accuracy of the hydrograph fitting process and shows that multi-peaked hydrographs are satisfactorily reconstituted by HEC-1.

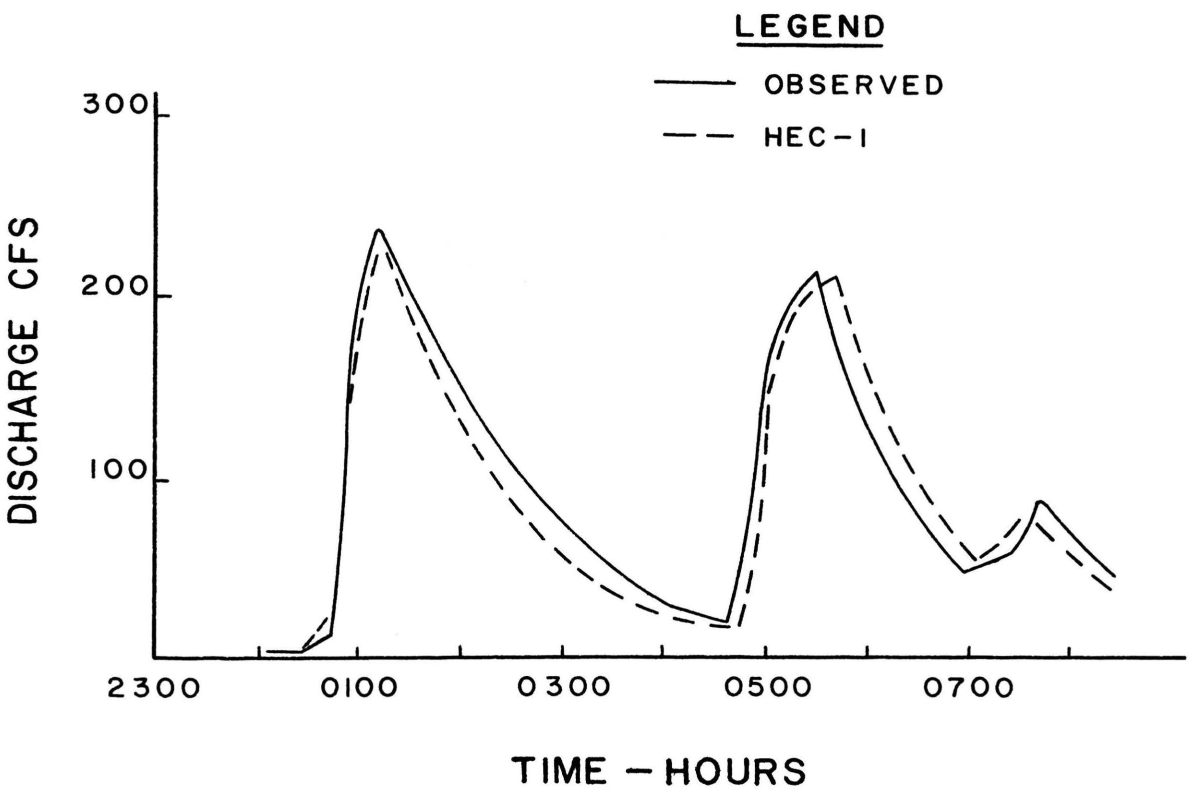
*Hydrograph Computation.* The data required to compute a runoff hydrograph for an individual storm event includes synthetic or observed rainfall, derived best fit unit hydrograph, and rainfall loss rate variables, and a time-area curve for the basin.

For this application, observed rainfall data were used. Though data were available for five rain gages, only one gage, located adjacent to the stream gage, was used to apply rainfall to the basin. This reduced the amount of rainfall data required by a factor of five.

The best fit values of TC and R, 0.5 and 1.0, respectively, were derived as explained above. Regional values of *ERAIN*, 0.4, and *RTIOL*, 2.0, were used. For high antecedent soil moisture conditions, only hours or a few days since the last rainfall, low values of *DLTKR* and *STRKR* were selected. Conversely, as the soil moisture decreases, longer time since the previous rainfall, high values of *DKTKR* and *STRKR* were selected.

Again, to simplify data requirements, the synthetic time-area curve available in HEC-1 was used rather than developing a specific time-area curve for the Boneyard Creek basin.

FIGURE 2. - FINAL OPTIMIZATION HEC-1 19 APR 1964


LEGEND
OBSERVED
HEC-1
300
200
100
DISCHARGE CFS
2300
0100
0300
0500
0700
TIME — HOURS

## COMPARISON OF RESULTS

A total of 22 storm events were analyzed and simulated by the three runoff models. Eleven storm events were simulated by the RRL method; six events were simulated by the HEC-1 model; and five were simulated by the HSP model. Comparisons between the RRL method and the other models were made, because the RRL simulation method was applied to the same storm event as the other two models. The simulation results of the HSP model as applied to the Boneyard Creek basin were unavailable at the time of the HEC-1 model application; thus, there are no storm events for which both of these models have been applied. However, comparisons between simulations by the HEC-1 and HSP models are considered valid since both were applied to the same basin for the same time frame, early and mid 1960s.

*Comparison of RRL and HEC-1 Models.* A comparison of computed hydrograph characteristics for six storm events using the revised RRL and HEC-1 methods is presented in Table 2. Table 2 includes the observed hydrograph values for peak discharge, time of the peak and runoff volume, and the corresponding computed values by the RRL and HEC-1 methods with the percent error for each of the computed values. Figures 3 and 4 present the observed hydrograph and the simulated hydrographs by the RRL and HEC-1 models for two storm events.

Considering peak discharge, the mean error in simulation by the revised RRL method is 67.3%. The high error in comparing the observed and simulated peak discharges by the RRL method is graphically seen in Figure 5. Using the HEC-1 model, the mean error of the simulated peak is 9.2%. Figure 6 presents a comparison of observed and simulated

TABLE 2. - COMPARISON OF RESULTS OF RRL AND HEC-1 METHODS

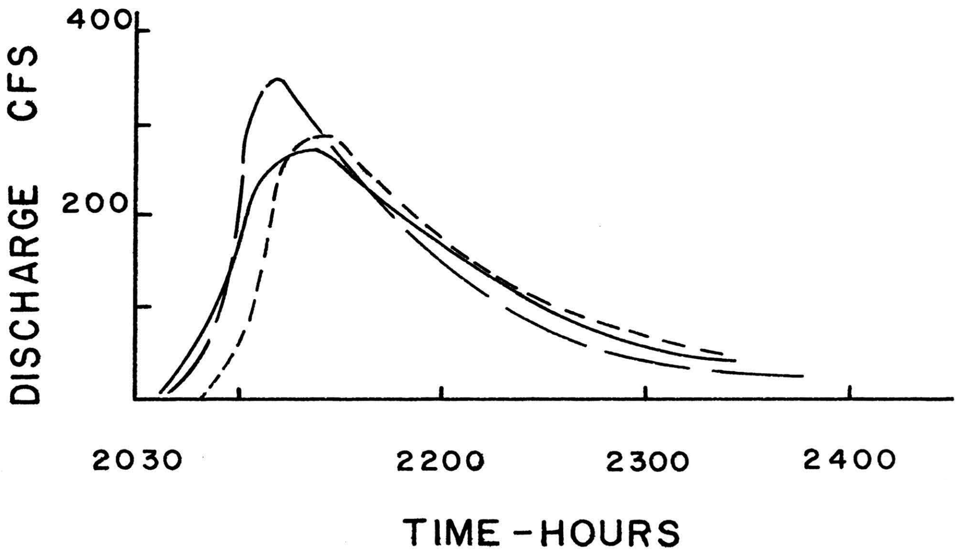
| STORM EVENT | OBSERVED HYDROGRAPH PEAK in cubic feet per second | RRL METHOD | | HEC-1 METHOD | |
|---|---|---|---|---|---|
| | | PEAK in cubic feet per second | PERCENT ERROR | PEAK in cubic feet per second | PERCENT ERROR |
| (1) | (2) | (3) | (4) | (5) | (6) |
| 15 Nov 1960 | 223. | 262. | 17.5 | 223. | 0.0 |
| 6 Jun 1961 | 479. | 1154. | 140.9 | 592. | 23.6 |
| 13 Jul 1962 | 274. | 423. | 54.4 | 305. | 11.3 |
| 19 Jul 1963 | 388. | 734. | 89.2 | 393. | 1.3 |
| 14 Sep 1965 | 270. | 350. | 29.6 | 291. | 7.8 |
| 20 Apr 1966 | 308. | 531. | 72.4 | 343. | 11.4 |

| STORM EVENT | OBSERVED HYDROGRAPH TIME to PEAK in minutes | RRL METHOD | | HEC-1 METHOD | |
|---|---|---|---|---|---|
| | | TIME to PEAK in minutes | PERCENT ERROR | TIME to PEAK in minutes | PERCENT ERROR |
| (1) | (2) | (3) | (4) | (5) | (6) |
| 15 Nov 1960 | 162. | 144. | -11.1 | 165. | 1.9 |
| 6 Jun 1961 | 96. | 90. | - 6.2 | 90. | - 6.2 |
| 13 Jul 1962 | 84. | 66. | -21.4 | 72. | -14.3 |
| 19 Jul 1963 | 84. | 60. | -28.6 | 72. | -14.3 |
| 14 Sep 1965 | 60. | 42. | -30.0 | 54. | -10.0 |
| 20 Apr 1966 | 96. | 90. | - 6.3 | 108. | 12.5 |

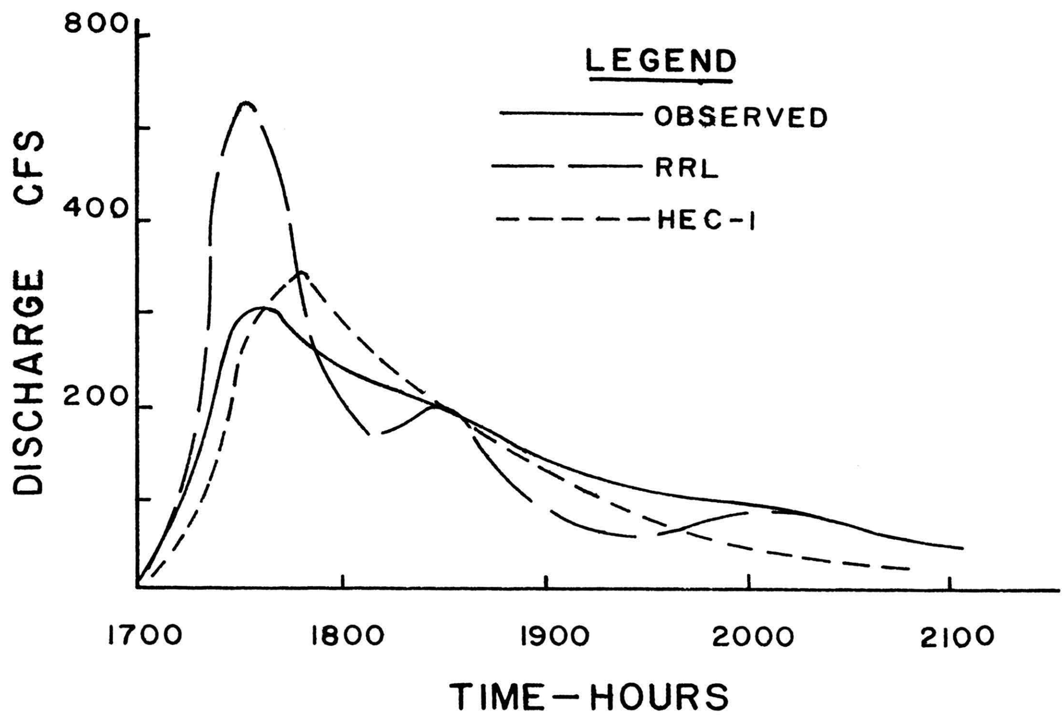
| STORM EVENT | OBSERVED HYDROGRAPH VOLUME in acre feet | RRL METHOD | | HEC-1 METHOD | |
|---|---|---|---|---|---|
| | | VOLUME in acre feet | PERCENT ERROR | VOLUME in acre feet | PERCENT ERROR |
| (1) | (2) | (3) | (4) | (5) | (6) |
| 15 Nov 1960 | 33.1 | 37.1 | 12.1 | 35.8 | 8.2 |
| 6 Jun 1961 | 85.0 | 84.9 | 0.1 | 66.1 | -22.2 |
| 13 Jul 1962 | 42.8 | 30.4 | -29.0 | 40.3 | - 5.8 |
| 19 Jul 1963 | 48.2 | 48.1 | 0.2 | 47.6 | - 1.2 |
| 14 Sep 1965 | 35.0 | 33.0 | - 5.7 | 31.8 | - 9.1 |
| 20 Apr 1966 | 52.2 | 51.8 | - 0.8 | 44.4 | -14.9 |

FIGURE 3. - COMPARISON OF RRL AND HEC-1 HYDROGRAPHS FOR 14 SEP 1965

FIGURE 4. - COMPARISON OF RRL AND HEC-1 HYDROGRAPHS FOR 20 APR 1966



400
200
DISCHARGE CFS
2030
2200
2300
2400
TIME-HOURS

800
400
200
DISCHARGE CFS
LEGEND
OBSERVED
RRL
HEC-I
1700
1800
1900
2000
2100
TIME-HOURS

FIGURE 5. - RRL VS. OBSERVED PEAK DISCHARGES

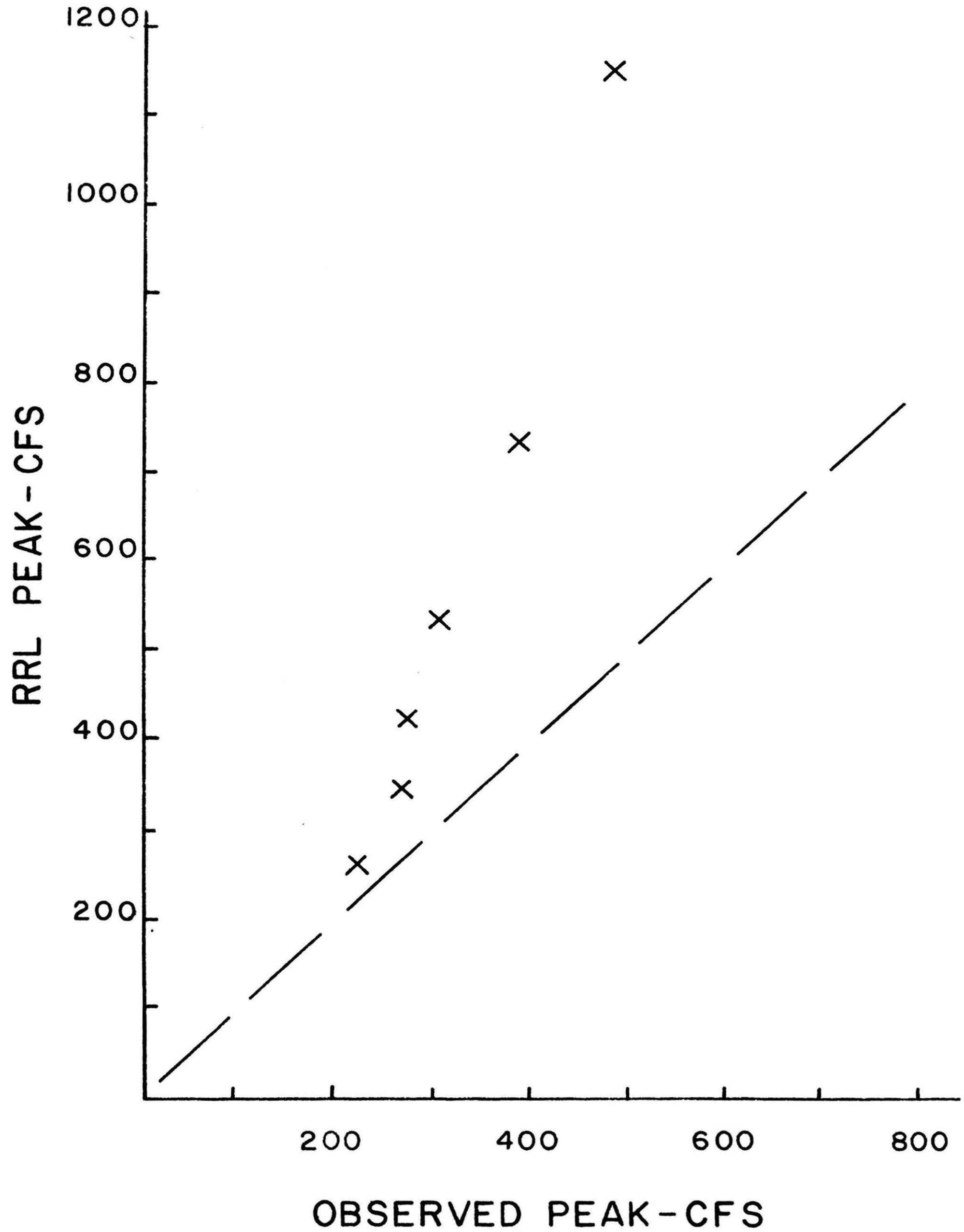
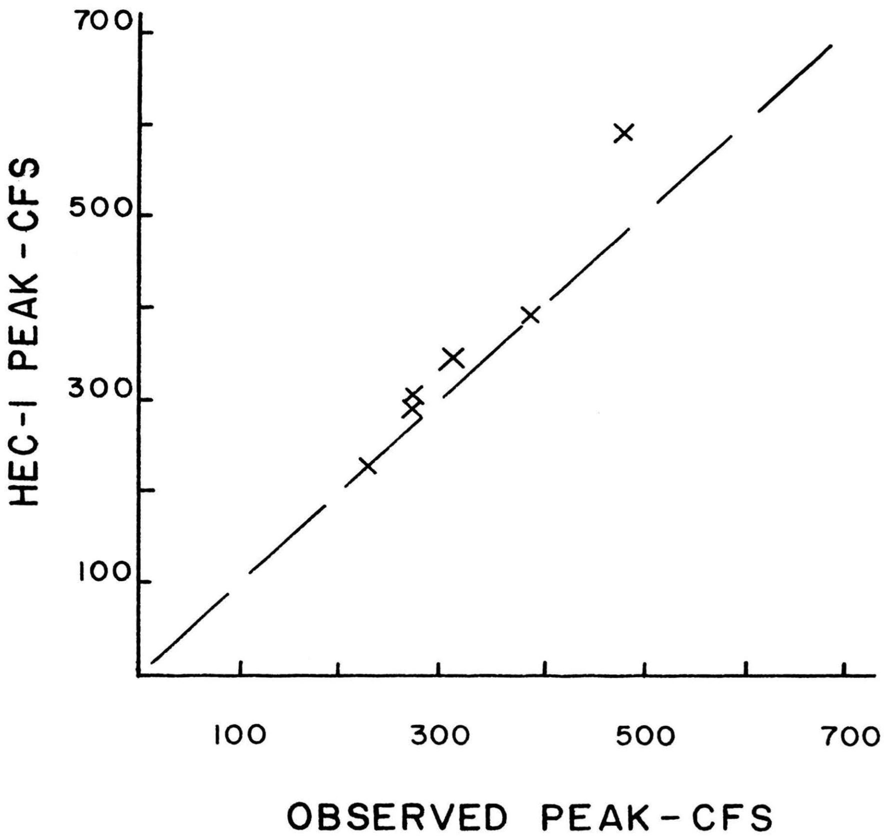

1200
1000
800
600
400
200
RRL PEAK-CFS
200
400
600
800
OBSERVED PEAK-CFS

FIGURE 6. - HEC-1 VS. OBSERVED PEAK DISCHARGES


700
500
300
100
HEC-1 PEAK - CFS
100
300
500
700
OBSERVED PEAK - CFS

peaks by the HEC-1 model. The accuracy of the HEC-1 simulation model is clearly seen.

Along with the higher computed peak, the RRL simulated peak discharge occurs an average of 17.3% earlier than the observed peak. On the average, the HEC-1 simulated peak occurs 5.1% ahead of the observed peak as seen in Table 2.

Generally, both models accurately reproduce the total direct runoff volume. The RRL method simulates the observed runoff volume 4% low and the HEC-1 model simulates 7% low.

*Comparison of RRL and HSP Models.* Table 3 presents a comparison of the observed and simulated hydrograph parameters as computed by the RRL and HSP models. As before, two storm events showing the observed hydrograph and the RRL and HSP simulated hydrograph are presented in Figures 7 and 8. With a range of 69.6% to 118.5%, the revised RRL method computed that peak discharge to be high by an average of 88.6%. Using the revised RRL results, overdesign by a factor of almost two, as seen in Figure 9, would occur. The HSP simulation model, on the other hand, underestimates the observed peak discharge by 11.8%. Figure 10 graphically shows a comparison of the observed and HSP simulated peak discharges. Considering the timing of the peak, the mean error by the RRL and HSP models is 15.1% early and 5.1% late, respectively.

As seen in Figures 7 and 8, the complete hydrograph was not simulated in these HSP computer runs. Even though the HSP model simulates runoff continuously, only discharge values greater than 200 cfs (5.6 $m^3$/s) were printed by the computer for this application. Since only high flows are of interest in this application, this cutoff of lower discharges significantly reduced the data management. Because

TABLE 3. - COMPARISON OF RESULTS OF RRL AND HSP METHODS

| STORM EVENT (1) | OBSERVED HYDROGRAPH PEAK in cubic feet per second (2) | RRL METHOD PEAK in cubic feet per second (3) | RRL METHOD PERCENT ERROR (4) | HEC-1 METHOD PEAK in cubic feet per second (5) | HEC-1 METHOD PERCENT ERROR (6) |
|---|---|---|---|---|---|
| 25 May 1965 | 378. | 704. | 86.2 | 314. | -16.9 |
| 2 Jul 1965 | 579. | 1265. | 118.5 | 494. | -14.7 |
| 25 Aug 1965 | 605. | 1190. | 96.7 | 572. | - 5.4 |
| 27 Jun 1966 | 224. | 380. | 69.6 | 225. | 4.5 |
| 18 Aug 1966 | 418. | 718. | 71.8 | 308. | -26.3 |

| STORM EVENT (1) | OBSERVED HYDROGRAPH TIME to PEAK in minutes (2) | RRL METHOD TIME to PEAK in minutes (3) | RRL METHOD PERCENT ERROR (4) | HEC-1 METHOD TIME to PEAK in minutes (5) | HEC-1 METHOD PERCENT ERROR (6) |
|---|---|---|---|---|---|
| 25 May 1965 | 114. | 90. | -20.1 | 90. | -20.1 |
| 2 Jul 1965 | 90. | 78. | -13.3 | 84. | - 6.7 |
| 25 Aug 1965 | 42. | 30. | -28.6 | 45. | 7.1 |
| 27 Jun 1966 | 132. | 114. | -13.6 | 192. | 45.4 |
| 18 Aug 1966 | 102. | 102. | 0.0 | 105. | 2.9 |

| STORM EVENT (1) | OBSERVED HYDROGRAPH VOLUME in acre feet (2) | RRL METHOD VOLUME in acre feet (3) | RRL METHOD PERCENT ERROR (4) | HEC-1 METHOD VOLUME in acre feet (5) | HEC-1 METHOD PERCENT ERROR (6) |
|---|---|---|---|---|---|
| 25 May 1965 | 16.0 | 22.2 | 38.8 | 16.4 | 2.5 |
| 2 Jul 1965 | 72.8 | 79.2 | 8.8 | 69.7 | - 4.2 |
| 25 Aug 1965 | 72.4 | 52.4 | -27.6 | 80.0 | 10.5 |
| 27 Jun 1966 | 5.4 | 2.7 | -50.0 | 17.5 | 224.1 |
| 18 Aug 1966 | 19.7 | 29.2 | 48.2 | 27.2 | 38.1 |

FIGURE 7. - COMPARISON OF RRL AND HSP HYDROGRAPHS FOR 2 JUL 1965

FIGURE 8. - COMPARISON OF RRL AND HSP HYDROGRAPHS FOR 25 AUG 1965

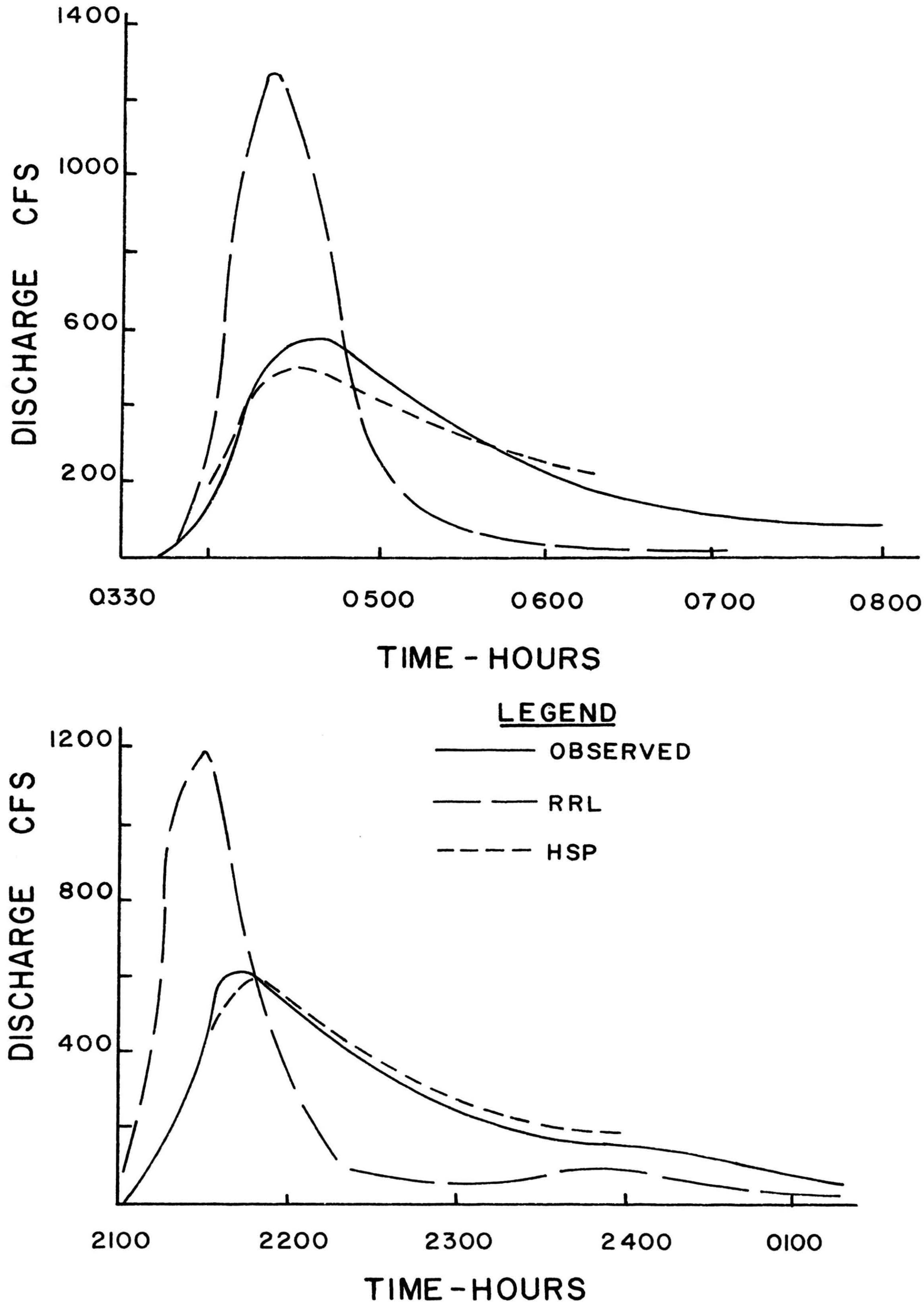

1400
1000
600
200
DISCHARGE CFS
0330
0500
0600
0700
0800
TIME - HOURS
LEGEND
OBSERVED
RRL
HSP
1200
800
400
DISCHARGE CFS
2100
2200
2300
2400
0100
TIME-HOURS

FIGURE 9. - RRL VS. OBSERVED PEAK DISCHARGES

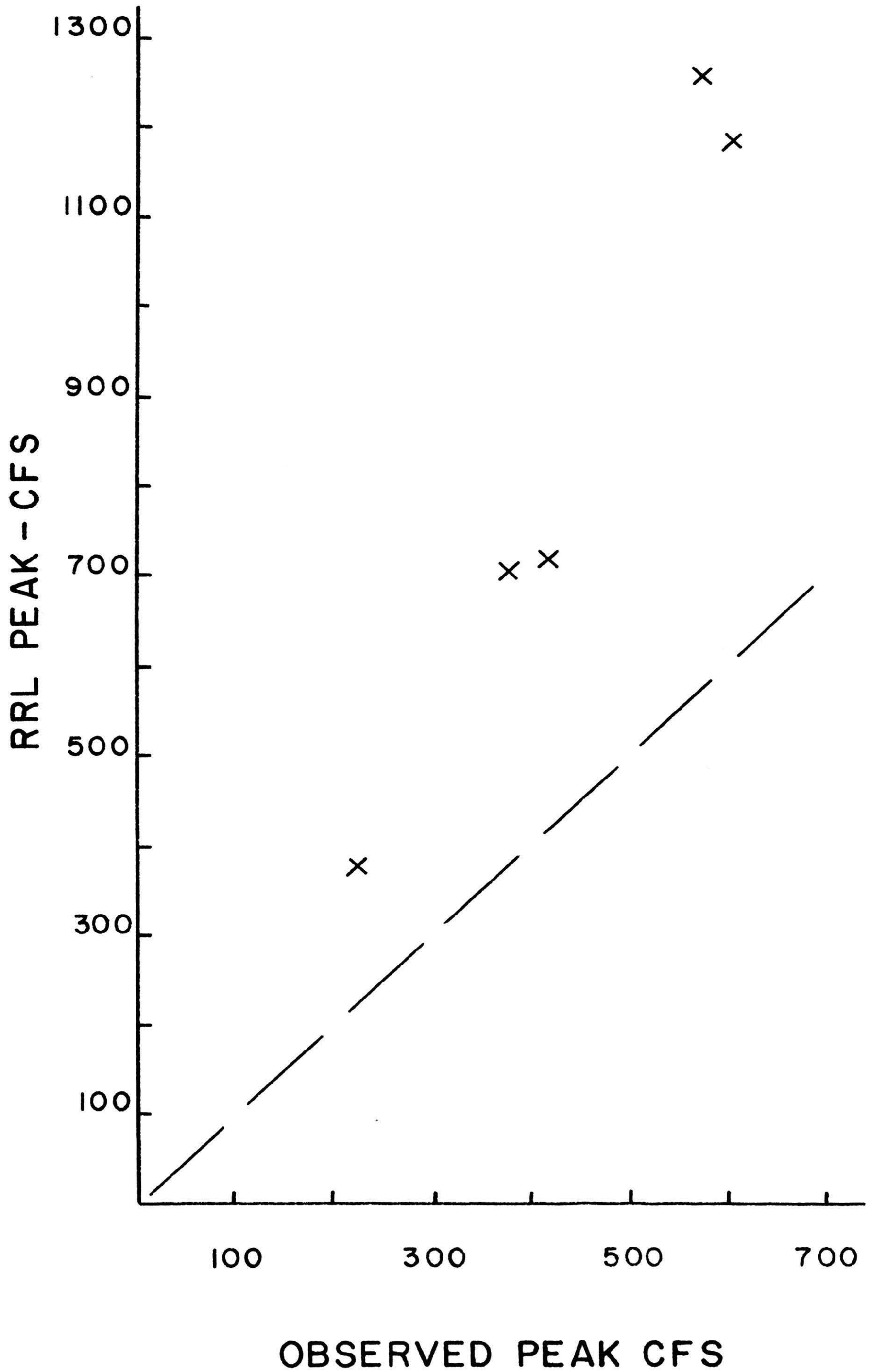
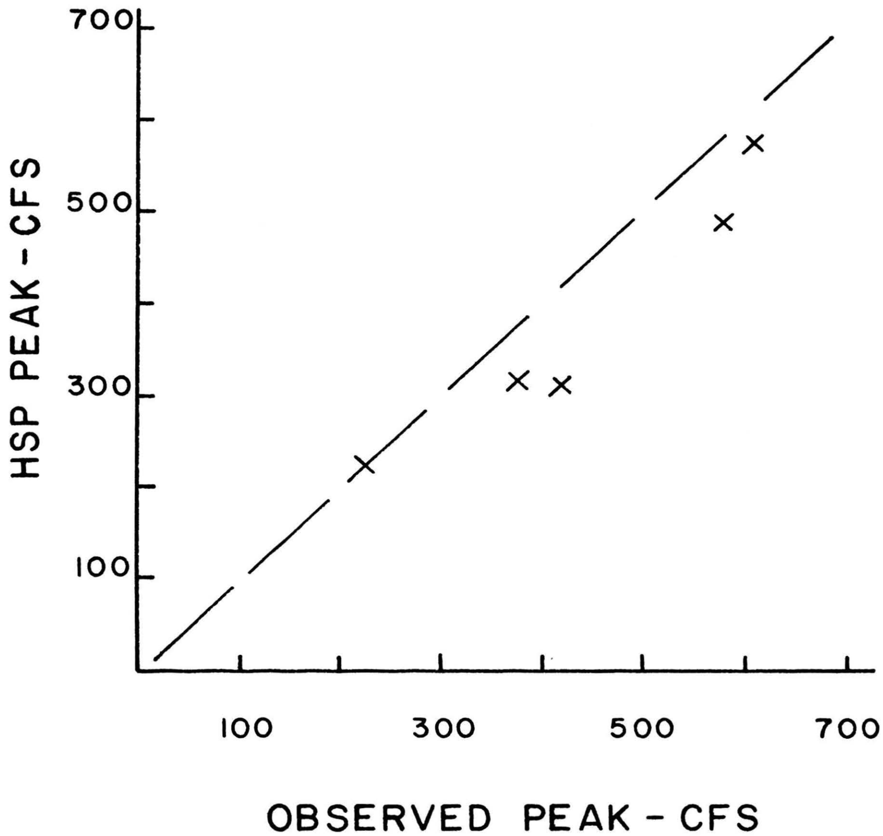

1300
1100
900
700
500
300
100
RRL PEAK - CFS
100
300
500
700
OBSERVED PEAK CFS

FIGURE 10. - HSP VS. OBSERVED PEAK DISCHARGES


700
500
300
100
HSP PEAK - CFS
100
300
500
700
OBSERVED PEAK - CFS

of this, the volume comparison between the observed and the RRL and the HSP simulated hydrographs is determined only for the base time period when data are computed by the HSP model (i.e., above 200 cfs).

The range of error of the volume simulation by the RRL method is -50 to 48.2% with an average of 3.6%. Though the average error is small, the high range indicates that the volume computed by this method for one storm event could probably be significantly in error. The HSP model computes hydrographs whose volume is in error by a range of -4.2 to 224.1%. The average error is 54.2%. Ignoring the storm event of 27 June 1966, the average error reduces to 11.7%. As seen in Table 3, the large error in volume by the 27 June 1966 storm event is a result of the timing of the peak occurring 60 minutes later than the observed peak. If these hydrograph peaks weren't so out of phase, the volumes would have compared favorably.

In general, both the HEC-1 and HSP simulated hydrographs compare well with the observed hydrograph. However, the revised RRL method computed hydrograph peaks considerably higher than the corresponding observed hydrograph peak discharge.

These results for the RRL method, obtained from Stall and Terstriep, vary considerably from the results presented in their 1969 ASCE publication (4). Their original paper showed a range of error of -15.2% to +17.7% for these same eleven storm events. The difference in the results is due to changes in the RRL model. In the original model, the discharge-storage relationship is only approximated, and a simplified one-step storage-routing procedure is used for the entire basin. Therefore, the detail required for application of the earlier model is very rough to be consistent with the procedures used. However, the improved version of the RRL model requires that the discharge-storage relationship be defined more accurately. As this relationship was not accurately defined for the application presented in this paper, the computed hydrographs are in error. A detailed field survey of the Boneyard Creek basin by Stall and Terstriep indicated many clogged and partially blocked sewers formerly unknown and also that a higher roughness factor should be used for the open channels in the basin. With these changes made, Stall indicated to the author that the revised RRL method is performing very satisfactorily again for the Boneyard Creek basin.

*Sensitivity of HEC-1 to Unit Hydrograph Characteristics.* The basic element of the HEC-1 method is the unit hydrograph, defined by the time of concentration (TC) and the hydrograph storage coefficient (R). The time of concentration is determined as the time from the end of rainfall excess to the point of inflection on the recession side of the hydrograph. Table 4 shows the sensitivity of simulated peak discharge to changes in TC only. The initial optimization run for TC = .20 and R = 1.0 gives a range of error of -16.2% to +49.4%. The mean error for this run is 12.0%. The final run for T = .50 and R = 1.0 shows a range of -5.1% to +38.0% with an average error of 10.0%. Thus, it is seen that a large percentage change in TC results in only a small change in the peak discharge.

The hydrograph storage coefficient (R) of the unit hydrograph is defined as the slope of a tangent to the recession limb of a hydrograph at its point of inflection. Table 5 shows the sensitivity of peak discharge to changes in R only. The optimization run for TC = 0.50% and R = 0.70 shows a range of error of -5.1% to 55.8% with an average error of 27.0%. The final optimization run for TC = 0.50 and R = 1.0 has an average error of 9.8%.

TABLE 4. - SENSITIVITY OF PEAK DISCHARGE TO CHANGES IN TIME OF CONCENTRATION

| STORM EVENT | OBSERVED HYDROGRAPH PEAK in cubic feet per second | TC = .20 R =1.0 PEAK in cubic feet per second | TC = .20 R =1.0 PERCENT ERROR[a] | TC = .50 R =1.0 PEAK in cubic feet per second | TC = .50 R =1.0 PERCENT ERROR[b] |
|---|---|---|---|---|---|
| (1) | (2) | (3) | (4) | (5) | (6) |
| 19 Jul 63 | 388. | 425. | 9.5 | 396. | 2.1 |
| 28 Aug 63 | 410. | 467. | 13.9 | 440. | 7.3 |
| 19 Apr 64 | 234. | 196. | -16.2 | 222. | - 5.1 |
| 25 May 65 | 378. | 390. | 3.2 | 403. | 6.6 |
| 2 Jul 65 | 579. | 865. | 49.4 | 799. | 38.0 |

[a] Mean percent error, 12.0.

[b] Mean percent error, 10.0.

TABLE 5. - SENSITIVITY OF PEAK DISCHARGE TO CHANGES IN ATTENUATION

| STORM EVENT | OBSERVED HYDROGRAPH PEAK in cubic feet per second | TC = .50 R = .70 PEAK in cubic feet per second | TC = .50 R = .70 PERCENT ERROR[a] | TC = .50 R =1.0 PEAK in cubic feet per second | TC = .50 R =1.0 PERCENT ERROR[b] |
|---|---|---|---|---|---|
| (1) | (2) | (3) | (4) | (5) | (6) |
| 19 Jul 63 | 388 | 460. | 18.6 | 396. | 2.1 |
| 28 Aug 63 | 410. | 558. | 36.1 | 440. | 7.3 |
| 19 Apr 64 | 234. | 286. | - 5.1 | 222. | - 5.1 |
| 25 May 65 | 378. | 490. | 29.6 | 403. | 6.6 |
| 2 Jul 65 | 579. | 902. | 55.8 | 799. | 38.0 |

[a] Mean percent error, 27.0.

[b] Mean percent error, 9.8.

Thus, the definition of the storage relationship of the Boneyard Creek basin is of prime importance. As seen in the HEC-1 application, the results are very sensitive to changes in the hydrograph storage coefficient. Likewise, with the storage roughly defined, poor results were derived using the RRL method. When the storage relationship was adequately defined, the RRL method computed satisfactorily runoff hydrographs.

## CONCLUSIONS

As a result of this study, the following statements are made about the capabilities of these urban runoff simulation studies. These three models are compared as to their capability in reproducing peak discharges, time to the peak, and volume of the storm hydrograph.

1. *Simulation of Peak Discharge.* Analysis of these data presented in this study indicates that the HEC-1 and the HSP models simulate the magnitude of the peak ordinate of the discharge hydrograph more accurately than does the revised RRL method.

For the storm events evaluated, the HEC-1 model computed peak discharges that were, on the average, 10% higher than those observed. The HSP model reproduced peak discharges that were, on the average, 12% lower than those observed. The hydrograph peaks predicted by the revised RRL method were much larger than those measured. This method computed peak discharges that averaged 78% larger than observed.

Thus, using the HEC-1 model, simulated peak discharges would be expected to be slightly conservative. On the other hand, the HSP model will compute peak discharges which can be expected to be exceeded occasionally. Significant overdesign would occur if the results of the revised RRL method were used.

2. *Simulation of Timing of the Peak.* As with the simulation of peak discharges, the HEC-1 and the HSP models simulate the timing of the peak discharge more accurately than does the revised RRL method.

The HEC-1 model simulates the peak discharge to occur, on the average, 5% earlier than the observed peak discharge. For the five storm events considered, the HSP model computed the peak to occur an average of 5% later than the observed peak. The predicted timing of the peak by the revised RRL method was computed to occur 17% before those peaks measured.

Therefore, the timing of the peak discharge can be expected to be satisfactorily simulated by the HEC-1 and HSP models. However, the error in using the revised RRL method can be important.

3. *Simulation of the Direct Runoff Volume.* The analysis of these data indicate that the revised RRL and HEC-1 models simulate the volume of direct runoff more accurately than the HSP model.

For the eleven storm events evaluated, the revised RRL method computes a runoff volume that is higher than the measured volume by an average of 4%. The volume computed by the HEC-1 model is, on the average, 7% higher than the observed volume. The HSP model simulated runoff volumes that are significantly higher than the measured hydrograph volumes. This method computed runoff volumes that averaged 54% larger than observed.

Thus, the revised RRL and HEC-1 models would be expected to compute runoff volumes that are slightly conservative. Using the HSP model to simulate the runoff volume would result in a significant overdesign.

4. The revised RRL method simulates direct runoff volume that is slightly conservative. However, a major overdesign occurs in attempting to compute peak discharges and the timing of the peak by this method.

5. The HSP model computes peak discharges that are somewhat lower and later than the measured discharges. This model computes volumes of runoff that would result in a large overdesign of detention structures.

6. The HEC-1 model simulates peak discharges and volumes of direct runoff that are a little conservative. The simulated time of the peak is computed to occur before the actual peak by this model. Overall, the HEC-1 model simulated best the observed hydrographs.

## ACKNOWLEDGEMENTS

The research described herein was carried out by the author under the general direction of T. E. Harbaugh, Professor of Civil Engineering at the University of Missouri-Rolla. The author gratefully acknowledges the assistance of John B. Stall and Michael L. Terstriep of the Illinois State Water Survey who provided the basic rainfall-streamflow data and the unpublished results of the revised RRL method application to the Boneyard Creek basin. Also acknowledged is the assistance of Norman H.

Crawford, Ray K. Lindsay, and Delbert D. Franz of Hydrocomp International, Inc. who provided the unpublished results of the HSP application to the Boneyard Creek basin.

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## APPENDIX II. - NOTATION

The following symbols are used in this paper:

$A_1, A_2, \ldots, A_n$ = Areas of the time-area curve for corresponding increments of time

$AI$ = Percent of the total basin area contributing at time T

$AK$ = Basic rainfall loss coefficient

$ALOSS$ = Rainfall loss rate in inches per hour (cm/hr)

$c$ = Interflow

$CUML$ = Accumulated rainfall loss in inches (cm)

$D$ = Ratio of detention depth

$D_e$ = Surface detention storage at equilibrium in cu ft/ft ($m^3/m$)

$DLTK$ = Incremental rainfall loss coefficient

$DLTKR$ = Incremental rainfall loss index

$D_1$ = Surface detention at the end of the previous time interval in cu ft ($m^3$)

$D_2$ = Surface detention at the end of the current time interval in cu ft ($m^3$)

$\Delta D$ = Increment of surface detention in cu ft ($m^3$)

$ERAIN$ = Exponent of rainfall

$f$ = Infiltration rate in in/hr (cm/hr)

$F$ = Cumulative infiltration in inches (cm)

$I$ = Discharge into a reach in cfs ($m^3/s$)

$I_1, I_2, \ldots, I_n$ = Rainfall intensity for equal durations

$L$ = Length of the flow plane in feet (m)

$LZS$ = Current storage level

$LZSN$ = Index level for moisture storage

$n$ = Manning's roughness coefficient

$O$ = Discharge out of a reach in cfs ($m^3/s$)

$q$ = Discharge in cfs ($m^3/s$)

$\bar{q}$ = Overland flow during a time interval

$Q$ = Discharge in cfs (m$^3$/s)

$Q_I$ = Discharge at the point of inflection on the recession limb of the hydrograph in cfs (m$^3$/s)

$Q_1, Q_2, \ldots, Q_n$ = Runoff hydrograph ordinates

R = Hydrograph storage coefficient

$R$ = Hydraulic radius in feet (m)

$RAIN$ = Rainfall in inches per hour (cm/hr)

$RTIOL$ = Ratio of $AK$ to that $AK$ after 10 inches more of accumulated loss occurs

$S$ = Channel slope or average ground slope in ft/ft (m/m)

$S_I$ = Slope of line tangent to the point of inflection on the recession limb of the hydrograph

$S_1$ = Storage during first time interval in cu ft (m$^3$)

$S_o$ = Slope of the energy gradient in ft/ft (m/m)

$STRKR$ = Basic rainfall loss index in in/hr (cm/hr)

$t$ = Time in seconds

$\Delta t$ = Time interval in seconds

$T$ = Ratio of time T to time of concentration

$UZS$ = Upper zone storage

$UZSN$ = Upper zone nominal capacity

$y$ = Depth in feet (m)

$Y$ = Maximum depth in feet (m)

$Z$ = Ratio of flow width to $Y$

# APPENDIX III. - VITA

## Richard Frederick Astrack

Mr. Astrack was born on August 3, 1945 in St. Louis, Missouri. He grew up and attended elementary and high school in St. Louis. In June, 1964, Mr. Astrack graduated from Cleveland High School located in south St. Louis. He completed the requirements for the BSCE Degree in May 1968 at the University of Missour-Rolla. Immediately after graduation, he entered graduate school at night to work toward his MSCE Degree. Mr. Astrack accepted employment with the U. S. Army Corps of Engineers, St. Louis District, on June 3, 1968 and was assigned to the Hydraulics Branch of the Engineering Division in May, 1969. Presently he is assigned to the St. Louis Metropolitan Area Study, a comprehensive regional water resources study of the St. Louis area. Mr. Astrack was married on June 27, 1970 and has no children.