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I am submitting herewith a thesis written by Gene C. Koonce, Jr., entitled "Simulation of Stormwater Response-Grissom Air Force Base, Indiana." I recommend that it to be accepted in partial fulfillment of the requirements for the degree of Master of Science with a major in Environmental Engineering.

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SIMULATION OF STORMWATER RESPONSE-GRISSOM AIR FORCE BASE, INDIANA

A Thesis

Presented for the

Master of Science

Degree

The University of Tennessee, Knoxville

Gene C. Koonce, Jr.

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ABSTRACT

This study was an attempt to evaluate three stormwater simulation models: The University of Tennessee Model (TENN-I), the Storage, Treatment, Overflow, and Runoff Model (STORM), and the Stormwater Management Model (SWMM). Two parametric models (TENN-I and STORM) and one deterministic model (SWMM) are evaluated by comparing real storm hydrographs which occurred at Grissom Air Force Base with the simulated hydrographs produced by the models. Rainfall and runoff data collected at three watersheds within Grissom Air Force Base were used in the evaluations.

Four characteristics of the simulated hydrographs were used to make the comparisons: peak discharge, time to peak, volume of the direct storm runoff, and hydrograph shape. Sensitivity analysis was performed on each model to determine which parameters produced the most and least sensitive effect on the simulated output.

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CHAPTER I

INTRODUCTION

General

Public concern of stormwater management and pollution control have made the development of sophisticated computer simulation models necessary. The need for such models is especially great for small, urban watersheds where quick stormwater response due to high percentages of imperviousness generate high concentrations of pollutants. Therefore, accurate predictions of storm hydrographs and pollutant loads are required in order to assess the effects of land use changes on the runoff regimes.

Unfortunately, as basin size decreases, adequate rainfall and runoff data becomes, at best, scarce. Overton and Meadows (1) have suggested five years of high quality rainfall and runoff data are needed to obtain a wide variation in storm events. For this reason, it has made the task of developing modeling techniques which accurately define the rainfall-runoff process most difficult.

The approaches used to simulate stormwater runoff are varied. The first developments began in the late 19th Century as engineers used empirical formulas to simulate design discharges for storm sewers. One of those methods, the Rational Formula, is still used today. In 1932, Sherman (2) published the Unit Hydrograph Method. He defined a unit hydrograph as "the hydrograph of runoff from a given area due to a one-inch runoff depth applied in one day or in any other convenient unit of time." Many researchers, such as

Dooge (3), Minshall (4), Singh (5), Overton (6), Nash (7), Prasad (8), Mockus (9), and others, have made numerous refinements to this theory. Kinematic wave theory, a more deterministic approach to modeling stormwater runoff, was proposed by Lighthill and Whitman (10) in 1955. This approach has been used to simulate overland and groundwater flow for several geometric shapes by Henderson and Wooding (11), Overton and Brakensiek (12), and Singh (13). The mathematics of this approach are somewhat more complex and difficult than the mathematics of the parametric modeling approach.

Numerous mathematical models have evolved from these modeling techniques. Brandstetter (14), Fromuth (15), and Linsley (16) have published evaluations of mathematical models for urban watersheds. In addition, some investigators, such as Papadakis and Preul Marsalek, Dick, Winser, and Clarke (18), and Abbott (19), have made comparisons among selected models and the model's ability to simulate observed data.

Objective of Study

The primary objective of this study was to evaluate the effectiveness of three computer stormwater models in simulating storm hydrographs on three small basins located on Grissom AFB, Indiana. This comparison was based on the peak discharge simulated, the time to peak simulated, as well as the entire shape of each of the storm hydrographs simulated by these models. The three models which were evaluated were: the Storage, Treatment, Overflow, Runoff Model (STORM) developed by the United States Army Corp of Engineers, the Stormwater Management Model (SWMM) developed for the United States

Environmental Protection Agency, and The University of Tennessee Stormwater Runoff Model (TENN-I) developed at The University of Tennessee.

A secondary objective of this study was to make recommendations on the most reliable and the least expensive way one can characterize and predict stormwater hydrographs at U.S. Air Force Bases by using one of the three models used in this study.

CHAPTER II

LITERATURE REVIEW

The main purpose of this literature review is to describe stormwater modeling concepts and selected available computer stormwater hydrograph models. It is necessary for one to be able to understand how the computer models synthesize the storm hydrographs in order to obtain more accurate solutions. In addition, sensitivity analysis would be easier since the modeler would be familiar with the parameters used in the model.

Modeling Concepts

Two conceptual approaches have been used in developing stormwater models, deterministic and parametric modeling (1). Deterministic modeling is based upon physical laws and theory; and, when the initial boundary conditions are known, the output should be known with a high degree of certainty. The input, though, rarely describes these initial and boundary conditions completely because of the complexity of the stormwater process. Therefore, simplifications and approximations are usually made about the input conditions, and this makes calibration of the model necessary.

The second approach modelers use to model the stormwater process is called parametric modeling (1). This modeling process is simplified and somewhat less developed. The coefficients are usually based upon parameters derived from watershed characteristics, and sometimes are fitted by some optimization technique to the hydrologic data.

Parametric Modeling Concepts

The first modeling developments in predicting stormwater runoff were parametric. At the turn of the 20th century engineers began developing empirical formulas to determine discharges for storm sewers. The Rational Method was developed during this time. This formula is expressed as $Q = CiA$, where Q is the peak discharge in cfs, C is the runoff coefficient depending on the watershed characteristics, i is the uniform rate of rainfall intensity, in inches per hour for a duration equal to the time of concentration, and A is the watershed area in acres. The time of concentration is defined as the time it takes water to travel from the most hydraulically distant point in the watershed to its outlet (20). Gregory and Arnold (21) in 1932 made modifications to the Rational Method by taking into consideration the effects of shapes and slopes of the catchments, elements of channel flow, and the pattern of the drainage system.

Also, in 1932, Sherman (2) published the Unit Hydrograph Method. His definition of the unit hydrograph was included in the introduction of this report. The basic concept behind this method is that once a unit hydrograph has been determined for one watershed, a unit hydrograph can be derived for any similar watershed, regardless of the size. Sherman's (2) study watersheds were relatively large, thus causing the stormwater response to act linearly. Prasad (8) showed that the Unit Hydrograph Theory is based upon three basic assumptions that hold true for large watersheds, but Overton (12), Nash (7), Amorocho and Orlob (22), and others have questioned the validity of the assumptions for small urban watersheds.

Using Sherman's (2) technique, a unit hydrograph may be found for any gaged watershed. Unfortunately, most small watersheds are ungaged, therefore rendering the Unit Hydrograph Method useless. To be able to apply this method to those ungaged basins, synthetic unit hydrograph techniques have been developed based upon watershed characteristics such as the length, slope, area, width and other features of the watershed.

In 1938, Snyder (23) published a technique to synthesize a unit hydrograph for an ungaged watershed. Snyder (23) developed empirical equations to solve for the basin lag time, peak discharge, and the time base of the unit hydrograph. Snyder (23) defined lag time as the "time from the center of mass of rainfall to peak of runoff." Another definition exists for lag time: the time from the center of the rainfall excess to the center of mass of the resulting hydrograph (24). Snyder derived his set of formulas for the Appalachian mountains region. Other researchers, most notably Linsley (25) and Eagleson (26), derived similar sets of equations for other parts of the country.

The Soil Conservation Service (SCS) also modified the unit hydrograph method to synthesize stormwater hydrographs. In 1947, Mockus (9) published the SCS-Curve Number (CN) technique to solve for direct runoff from rainfall. The runoff-rainfall equation was developed from storm data which included rainfall totals for each calendar day, but was not distributed over time (9). The equation is:

$$Q = \frac{(P - Ia)^2}{(P - Ia) + S} \qquad (1)$$

where Q = accumulated direct runoff

P = accumulated rainfall (potential maximum runoff)

Ia = initial abstraction including surface storage, interception, and infiltration prior to runoff.

S = potential maximum retention

A relationship between Ia and S was developed from experimental watershed data. The empirical equation used is:

$$Ia = 0.2\ S \quad . \tag{2}$$

Replacing Ia with 0.2 S in equation (1), the equation follows:

$$Q = \frac{(P - 0.2\ S)^2}{(P + 0.8\ S)} \quad . \tag{3}$$

The runoff-rainfall relationship can be shown graphically when the S values are transformed into curve numbers (CN) by the use of the following equation:

$$CN = \frac{1000}{10 + S} \ . \tag{4}$$

The curve numbers were associated with various hydrologic soil-cover complexes and land uses in order to solve for accumulated direct runoff given accumulated precipitation.

The Soil Conservation Service (20) also developed a technique to simulate an entire stormwater hydrograph. The SCS used triangular hydrograph relationships to describe a unit hydrograph. Three parameters, time to peak, peak discharge, and the time of base, define a single triangle that may be convoluted to give the entire stormwater hydrograph. The equations the SCS developed are:

$$T_p = L + D/2 \tag{5}$$

$$T_b = 2.67\ T_p \tag{6}$$

$$Q_p = (KAQ)/T_p \tag{7}$$

where T_p = the time to peak

T_b = the time of base

Q_p = peak rate of discharge

L = watershed lag

D = storm duration

K = a constant

A = drainage area.

Kent (20) defined watershed lag as a weighted time of concentration and expressed it as:

$$L = \frac{\ell^{0.8} (S + 1)^{0.7}}{1900\ Y^{0.5}} \tag{8}$$

where L = lag in hours

ℓ = length of mainstream to farthest divide in feet

Y = average slope in watershed in percent

$S = \frac{1000}{CN'} - 10$

CN' = a retardance factor approximated by the curve number representing the watershed's hydrologic soil-cover complex.

Once the unit hydrograph is defined, the storm can be divided into increments of duration, and by plotting and summing the unit hydrograph ordinates, a storm hydrograph can be obtained. The advantage of the SCS technique is that the process to develop the storm hydrograph is simplified significantly, but this concept is based upon linearity of the watershed response. As previously pointed out, overland and channel flow are nonlinear systems.

Another modification was made to the unit hydrograph method; investigators began thinking that a hydrograph produced by an instantaneous storm (i.e., the duration of the storm approaches zero) could give an indication of the drainage-basin characteristics (27). In 1945, Clark (28) coupled this ideal with the unit hydrograph theory and formulated the instantaneous unit hydrograph (IUH).

Some conceptual models of the IUH are based upon linear reservoirs, linear channels, or time-area diagrams, which have been developed to determine outflow discharges. Nash (7) suggested that the IUH could be derived by routing the instantaneous rainfall through successive linear reservoirs. Dooge (3) derived an instantaneous unit hydrograph using time-area curves, or isoclones, from the assumption that the reservoir action in a catchment can be separated from the translation. Singh (5) developed a routing model of two unequal reservoirs, one representing overland flow and one representing channel flow. These routing models are linear, and as previously pointed out, watershed response is distinctly nonlinear. Even though this flaw exists in routing models, Overton (29) and others have shown that routine techniques are mathematically simpler than convoluting and still obtain reasonable solutions.

Investigators began presenting evidence demonstrating the nonlinearity of the watershed response. Minshall (4), from data collected from small watersheds up to 290 acres, showed that the peak ordinate and time to peak of the unit hydrographs varied with rainfall excess. His general results indicated that as rainfall excess increased, the peak ordinate increased, and the time to peak decreased. Overton (30), using data provided by the Corp of Engineers, also demonstrated

the nonlinearity of stormwater response by showing the variation of lagtime to the excess rain rate.

Because of evidence presented by Minshall (9), Overton (30), Amorocho (22), and others, investigators began developing the nonlinear unit hydrograph theory. Most attempts utilized either the routing concept based upon Nash's (7) and Dooge's (3) work or the instantaneous unit hydrograph concept based upon Clark's (28) work.

In 1964, Singh (5) published a nonlinear instantaneous unit hydrograph based upon data from six large watersheds. Using three parameters, two being factors of proportionality and the third being the time of concentration, Singh (5) suggested an equation to solve for the direct surface runoff hydrograph. His results were similar to Minshall's (9); he found the peak ordinate varied directly and time to peak varied inversely with the weighted rainfall excess.

Another nonlinear hydrologic system response model was presented by Prasad (8) in 1967. He derived his model from linear routing models proposed by Nash (7) and Dooge (3). Prasad (8) utilized three parameters in his model, two constants (K_1 and K_2) and an exponential term (N). He varied these three parameters one at a time to show the effect on outflow. Prasad (8) also applied his model to actual basins located in east central Illinois. Using two important basin characteristics, peak discharge (Q_p) and the time to peak (t_p), he correlated the parameters to the actual data from the Illinois watersheds and derived best-fit equations for K_1, K_2, T, Q_p, and T_p.

Another approach to the nonlinear unit hydrograph was taken by Ardis (31) in 1972. He derived a quadrilateral shaped unit response function (urf), or unit hydrograph (31). Ardis (31) decided there were

two basic responses of the watershed during the runoff process, an initial response and delayed response. This response function was derived based on the following assumptions: (1) the area under the response function must be unity, (2) the peak of the delayed response must occur at the end of the initial response, and (3) the time bases of both triangles must be multiples of the time interval used in abstracting rainfall and runoff data. Since no assumption was made about the relative sizes of the two responses, the unit response function is able to assume an infinite number of shapes.

Five parameters are needed to define the double triangle unit response function: UP, the peak of initial response; UR, the peak of the delayed response; T1, the time to peak of the initial response; T2, the time to peak of the delayed response; and T3, the time to the end of the delayed response. T3 can be mathematically defined as:

$$T3 = (NOBS - NRAIN + 1) * DT \qquad (9)$$

where NOBS = number of storm hydrograph ordinates in multiples of Dt

NRAIN = number of rainfall increments in multiples of DT.

Since the area under the urf must be unity, UR can be defined as:

$$UR = 2 - (UP * T2)/(T3 - T1) \; . \qquad (10)$$

Once values for UP, T1, and T2 are determined, the unit response function can be defined.

Ardis (31) and Betson (32) have regionalized the urf from data collected in the Tennessee Valley. Both investigators optimized the double triangle parameters by a pattern search method developed by

Green (33). Ardis (31) used 140 storms from eleven watersheds in his analysis, while Betson (32) used 349 storms from twenty-four watersheds in his analysis. The watersheds ranged from highly urbanized to 100 percent forested; all were located within the Tennessee Valley and the Cumberland Valley.

The predicting-parameter equation used was the same for both Ardis (31) and Betson (32):

$$PAR_i = a\, X_j^{b}\, X_2^{c} \ldots X_n^{k} \qquad (11)$$

where PAR_i = double-triangle parameter

X_j = watershed characteristic

a-k = coefficients.

Betson (32) included eight watershed characteristics in his regionalization scheme: drainage area, a dimensionless measure of the shape, main channel slope, a measure of the drainage density, a measure of the sinuosity, percent of the watershed forested, SCS-curve number, and a measure of the storm sewer improvement (32). In addition to the eight physical watershed characteristics, two additional variables, the volume of stormwater runoff and number of periods of precipitation excess, were included. These variables were allowed to vary from storm to storm, and this feature introduced nonlinear variation in the linear unit-hydrograph method.

Overton and Troxler (34) further defined the double-triangle unit function by regionalizing lag time and lag modulus in terms of the drainage area, percent impervious, and percent forest. Normalized unit response functions (i.e., unit response functions in which the time scale is divided by the associated lag time and the ordinates

are multiplied by the associated lag time) were then grouped into three land uses: 100 percent forest, agriculture, and urban. Therefore, given relatively little information about the watershed, one can define a unit response function to simulate the storm hydrograph.

Deterministic Modeling Concepts

Kinematic wave theory, a deterministic approach to stormwater modeling, has also proven to be a useful tool for simulating stormwater flows. First published by Lighthill and Whitham (10) in 1955, this method has been used to determine runoff given a steady rainfall of finite duration. Kinematic wave theory was first used on a single sloping plane (11), but researchers have published models for use on two rectangular planes plus a channel (a V-shaped watershed) (12), a converging section of a cone (13), and a cascade of planes contributing to channels (13).

Kinematic Wave Theory is based upon two equations, the equation of continuity and the equation of momentum. The two equations may be written, respectively, as:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 9 \qquad (12)$$

$$Q = Q_n \quad \overline{1 - \frac{1}{S}\left\{\frac{\partial y}{\partial x} + \frac{V}{g}\frac{\partial V}{\partial x} + \frac{1}{g}\frac{\partial V}{\partial t} + \frac{Vg}{Sy}\right\}} \qquad (13)$$

where Q = discharge

y = mean depth of flow

V = mean velocity of flow

g = gravitation acceleration

x = space coordinates

t = time coordinate

S = slope of the channel

q = lateral inflow

Q_n = normal flow.

The equation of momentum may be simplified if the sum of the terms to the right of the minus sign is much smaller than one. Discharge may then be set approximately equal to normal flow, or:

$$Q \simeq Q_n \quad . \tag{14}$$

Normal flow may be expressed by either the Manning equation or the Chezy equation.

Woolhiser and Liggett (35) derived a parameter to determine if the dynamic terms in equation (13) may be neglected, called kinematic flow number. The kinematic flow number may be expressed as:

$$k = \frac{SL}{Y_o F_o^2} \tag{15}$$

where L = length of the plane

S = slope of the plane

Y_o = depth of flow at equilibrium at the end of the plane

F_o = Froude number of equilibrium at the end of the plane.

For values above 10, Woolhiser and Liggett (35) found the kinematic wave equations yield good approximations of overland flow. If the kinematic flow number is large, then discharge may be found by extrapolating a normal flow formula to an unsteady nonuniform flow. Two equations, Manning's formula and Chezy's formula, have been used for this purpose. Manning's formula may be written in the form:

$$Q = \frac{1.49}{n} AR^{2/3} S_o^{1/2} \qquad (16)$$

where Q = mean discharge in cfs

A = area in ft^2

R = hydraulic radius in ft

S_o = slope of the channel

n = Manning's resistance coefficient in $ft^{1/6}$.

For overland flow, Manning's formula becomes:

$$Q = \frac{1.49}{n} w\, y^{5/3} S_o^{1/2} \qquad (17)$$

where w = width of channel

y = mean depth of flow;

since the hydraulic radius is approximately equal to depth for a wide rectangular channel. Documentation of Manning's resistance cofficients has been reported in the literature for various channel conditions (36). Because of the simplicity of the equation and satisfactory results, Manning's formula is one of the most widely used equations for open channel computations.

The second equation, Chezy's formula, utilizes Newton's Laws of Motion in the derivation (37). The final form of the equation may be written as:

$$Q = C\, A \sqrt{R\, S_o} \qquad (18)$$

where Q = mean discharge

A = cross-sectional area

R = hydraulic radius

S_o = slope of the channel

C = factor of flow resistance

Equation (17) reduces to

$$Q = C\ w\ y^{3/2}\ \sqrt{S_o} \tag{19}$$

where w = width of channel

y = mean depth of flow

for wide rectangular channels. The determination of the factor of flow resistance has been documented in the literature (36).

A generalized form of the normal flow equation may be written as:

$$Q = a\ y^{m} \tag{20}$$

where a and m are functions of the resistance formula being used. Placing the depth of flow in terms of rainfall excess rate and time, equation (20) may be put in the form:

$$Q = a\ [(q/w)t]^{m} \quad . \tag{21}$$

Discharge is equal to qL at the time of equilibrium, t_e, therefore, substituting the terms into equation (21), t_e may be placed in the form:

$$t_e = (\frac{Lq}{a})^{1/m} * \frac{w}{q} \quad . \tag{22}$$

In order to make comparisons of rising hydrographs, a dimensionless solution to the kinematic flow approximation was made (38) by dividing equation (22) into equation (21). The resulting equation is:

$$(\frac{Q}{qL}) = (\frac{t}{t_e})^{m} \quad . \tag{23}$$

Unfortunately, Overton (38) discovered that the shape of the rising hydrograph was effected by small changes in the chosen value of the time to equilibrium. Therefore, he derived a more stable parameter, lagtime, that could be more easily computed with little error. The derivation has been described by Overton (38). The final form for the equation of lagtime may be written as:

$$TL = \left(\frac{m}{m+1}\right)\left(\frac{qL}{a}\right)^{1/m}\left(\frac{w}{q}\right). \qquad (24)$$

In a study by Overton (38), 214 overland flow hydrographs were normalized by dividing the discharge scale by the rain rate and the time scale by the corresponding lagtime. By superimposing all normalized hydrographs on transparent paper, it was clear that a single dimensionless rising hydrograph could represent all normalized hydrographs within a small error. This dimensionless rising hydrograph was referred to by Overton (38) as the observed rising hydrograph.

Using equation (20), Overton (38) calculated the rising hydrograph for Manning's equation, Chezy's equation, and a laminar solution. The appropriate "a" and "m" values were used in the equation.

The three kinematic models were also normalized and compared to the observed rising hydorgraph. The models appeared to fit the data reasonably well. The Manning-kinematic solution produced a 15 percent standard error in fitting the data. The Chezy and laminar solutions produced a 19 percent standard error (1).

Henderson and Wooding (11) derived kinematic solutions for the following hydrograph. There are two cases for solving the falling hydrograph: (1) when the rising hydrograph is at equilibrium, and (2) when the rising hydrograph is at a flow less than equilibrium.

In both cases, time is placed in terms of depth of flow and the characteristics of the plane. Depth of flow is determined, and Manning's equation is used to find the discharge hydrograph.

An approximate two-zone analytical solution for a V-shaped watershed was derived for channel inflow and the rising hydrograph of the channel (12). In zone one, the rate is less than the time to equilibrium of overland flow, t_{eo}. In zone two, the rain rate was greater than t_{eo} and less than the time to equilibrium for the entire watershed, t_{eq}. Recession can be calculated following the same procedure used for a single plane. Overland flow solutions for a converging surface and a cascade of planes are placed in the form of characteristic equations. Numerical solutions are necessary in order to solve these equations.

Stormwater Runoff Models

Since mathematical modeling has become the main instrument for transferring hydrological information from one basin to another, numerous computer models have been developed. These models may be broken down into three broad categories: planning models, design models, and operational models (39). The objectives are different for the three types of models, therefore, this produces models with different characteristics. Some models tend to overlap and may be used for more than one objective.

Planning models can be typified by large time steps (hours) and long simulation periods (months and years). They are used for an overall assessment of the urban runoff problem and sometimes show tradeoffs for design features (treatment vs. storage). The mathematical

complexity is relatively low and data requirements are minimal for this type of model. The Battelle Urban Wastewater Management Model, the Chicago Flow Simulation Program, and the Storage, Treatment, Overflow, Runoff Model (STORM) are examples of planning models.

The Battelle Urban Wastewater Management Model (40) was developed for sewer system components such as trunk and interceptor sewers, storage facilities, and treatment plants. The primary objective of this model is to provide a means to evaluate existing and planned sewerage systems (14). This model is limited to single event simulations.

Rainfall excess is determined separately for impervious and pervious surfaces, then combined to obtain the total volume of water available for runoff. Losses from pervious areas are determined by a modified Holtan's infiltration equation while losses from impervious areas are determined from available depression storage.

To obtain the discharge hydrograph, rainfall excess is convoluted with a unit hydrograph. The unit hydrograph is determined by a method developed by the U.S. Soil Conservation Service. This technique was chosen for simplification purposes (14).

Additional features included in the Battelle Model include water quality analysis, dry-weather flow, and a scheme to estimate least cost sizes for overflow storage and treatment facilities. This is the only model at present to include least cost design optimization for stormwater runoff (14).

The Chicago Flow Simulation Program (41) is designed for urban and nonurban watersheds. The model uses large time steps (hours) and can be used for continuous simulation. Water quality, real-time control, and storage and treatment design features are not included within the model.

Runoff is computed differently for sewered and nonsewered areas. For sewered areas, all rainfall on impervious areas is assumed to become runoff and a constant fraction of rainfall on pervious areas becomes runoff. For nonsewered areas, all rainfall becomes runoff on impervious areas and linear storage routing based on overland flow storage, watershed shape, and watershed slope is used to determine runoff from rainfall (14).

The Chicago Model uses a storage routing technique combined with Manning's equation to determine discharge hydrographs for open channel flow and circular sewers. This model does not consider upstream and downstream flow controls, flow reversals, and backwater effects for flow routing. A water storage feature is included in the model which does not require geometric data on the reservoir or outlet.

If a more detailed study is necessary, design models should be employed. These models are typified by short time steps (minutes) and short simulation periods (hours) for a single runoff event. The mathematical complexity of this type of model is high. Accurate predictions of water quantity and quality may be made with design models and least cost design features are sometimes included. The Environmental Protection Agency's (EPA) Stormwater Management Model (SWMM), the British Road Research Laboratory Model (RRLM), Water Resources Engineers Stormwater Management Model, and Hydrocomp Simulation Package are examples of design models.

In 1969, Terstriep and Stall (42) published the British Road Research Laboratory Model. This model is limited to urban watersheds; it simulates time-varying runoff in combined sewerage systems. The RRLM is limited to single runoff events and does not include water

quality analysis or treatment and storage design features.

The RRLM computes losses from rainfall according to a function of time described in the input data. This model does not consider runoff from pervious areas, therefore, infiltration schemes are ignored.

A storage routing technique is used to simulate the combined flow from each inlet which uses average travel times computed from Manning's Equation. Brandstetter (14) reported that this technique seems satisfactory for low intensity storms, but for high intensity storms or storms with large flow variations, an error is introduced.

According to Stall and Terstriep (42), the RRLM simulates adequately for watersheds which are less than five square miles and comprised of more than 15 percent of impervious areas, and the frequency of the storm is less than 20 years. These limitations are imposed upon the models because pervious runoff is ignored and approximations in overland and sewer flow produce unacceptable errors.

The Water Resources Engineers (WRE) Stormwater Management Model (43) is a modified version of the Environmental Protection Agency's (EPA) Stormwater Management Model (SWMM). It simulates combined storm and sanitary runoff and quality for a sewer and open channel network. This model is limited to simulations of single runoff events.

Runoff is computed separately for pervious and impervious areas and begins when all available depression storage is filled. Potential infiltration for pervious areas is calculated using Horton's equation. Losses by other means are not considered.

Dynamic wave equations combined with an explicit finite difference solution developed by WRE defines the runoff hydrograph. This model is

capable of handling downstream and upstream flow control, backwater, and flow reversal and several geometric shapes of sewers in the solution technique. Hydraulic equations are also used in the model to simulate flow control through a diversion structure.

The WRE Stormwater Management Model is useful where backwater, flow control, flow reversal and diversion structures should be considered. If this phenomenon is not expected to occur, then simpler models should be used. This model has been verified with data from Baltimore, Maryland, and San Francisco, California. Both water quantity and quality were predicted within the right orders of magnitude.

One of the most comprehensive mathematical models developed to date is the Hydrocomp Simulation Package. Runoff computations are patterned after the Stanford Watershed Model (16), but is considered an improvement to the original Stanford Watershed Model with respect to mathematical formulations and data handling capabilities (14).

The Hydrocomp model considers interception, infiltration, and groundwater contributions to surface flow. Depression storage is calculated using empirical equations which requires calibration with observed data to obtain adequate results. Flow routing in open channels and sewers is computed by kinematic wave theory and the use of Manning's equation.

Stormwater quality computations used by the Hydrocomp model are based upon formulations first used by EPA's Stormwater Management Model. Special Empirical functions are used for water quality balance between runoff events for different land uses, street cleaning practices, and pollutant accumulations. Calibration of the parameters used in these equations with measured data is required.

The Hydrocomp Simulation Package was developed originally for rural watersheds, but recent modifications and testing have shown this model may be applied to urban watersheds. Unfortunately, the model is very complex to operate, therefore, Hydrocomp, Incorporated, requires users to complete a training program prior to application.

When actual control decisions are required during a storm event, operational models may be used. These models are usually derived from design models, but are developed and applied for a particular system. Operational models employ telemetered stations which record rainfall, then input the data during the storm event into the model. The model is able to predict the system response shortly after runoff begins. Control options, such as storage systems, diversions, or regulator setting, may be put into effect. Examples of operational models are the Minneapolis-St. Paul Urban Runoff Model and the Seattle Computer Augmented Treatment and Disposal System.

The Minneapolis-St. Paul Urban Runoff Model (44) was developed for real-time forecasting of flows for the Minneapolis-St. Paul combined sewerage system. The purpose of the model is to reduce untreated stormwater runoff into the Mississippi River. The model predicts flows for several catchments, and depending on the volume of runoff, diversion systems are employed to route the water to the treatment plant. It is used specifically for real-time control of stormwater runoff and is not designed for the assessment of existing or planned sewerage systems because the coefficients used by the model have been calibrated with measured data.

Excess precipitation is determined separately for pervious and impervious areas. Runoff from impervious areas occurs once all

depression storage has been filled. A modified Holtan's equation is used to determine runoff from pervious areas.

Runoff from both areas are combined and convoluted using a unit hydrograph to obtain the discharge hydrograph. The unit hydrograph is defined from basin characteristics by a method developed by the U.S. Soil Conservation Service. This method was chosen for two reasons: (1) the discharge hydrograph can be obtained rapidly, and (2) the basins in the Minneapolis-St. Paul area are large, therefore the unit hydrograph procedure may be used with a higher degree of accuracy.

The Minneapolis-St. Paul model is intended for real-time control of flows in the Minneapolis-St. Paul sewerage network. Since this model has been developed and calibrated specifically for this system, use elsewhere may produce errors in the flow computations. Simplifications in the flow routing techniques make this model unsuitable for applications in planned or existing sewerage systems.

The Seattle Computer Augmented Treatment and Disposal System (45) is another real-time control model to prevent untreated overflows from the sewerage system to escape treatment. This model does not include a mathematical formulation to predict runoff, but it is capable of automatic control of diversion structures and pumping stations by use of a small computer.

The Seattle model employs the use of rule curves, curves which represent sewer storage capacities as a function of time for the sewer upstream of each controlled regulator and pumping station (14), to determine the operation of the diversion structures and pumping stations. The rule curves were developed from historical rainfall data.

This system has proven effective in reducing untreated combined overflows at Seattle. It has been suggested that the rule curve be changed to a more sophisticated method of computing flows (14), but this system works for this area. It would be difficult to transfer the Seattle model to other parts of the country.

To try to present all available models in this study would be impossible. Some investigators, such as Brandstetter (14), Fromuth (15), and Lindsey (16), have published reports which focus on the evaluation of selected models.

Brandstetter (14) reported an assessment of mathematical models for storm and combined sewers. Twenty-five models were discussed; seven models were tested by computer runs with observed data, eleven models were evaluated from published information and from personal communication with the model builders, and the remaining seven models were listed and described briefly. Brandstetter (14) includes a summary, features, computer requirements, and an evaluation of each of the models.

Fromuth (15) evaluated eighteen models which have been reported in the literature. The evaluations were based on data requirements, availability of the model, nature of output, representation of the runoff process, calibration requirements, applicability of the model to both rural and urban areas, and the inclusion of land use variables. No evaluations were made with computer simulations; the models were analyzed strictly on the basis of published information.

Linsley (16) also reviewed major urban stormwater models and modeling techniques. Summaries describing the major points of the models are given along with limited computer simulations. Linsley (16) also

compared the models using 16 factors pertaining to where the model may be applied, the type of modeling technique used, and the availability of water quality analysis.

Many mathematical models have been developed since the development of the digital computer. Linsley (16) reported in 1971 that there was over a hundred stormwater models available at that time. Some water resource engineers have suggested that the models already available should be further refined before new models are developed. Since no two objectives are exactly the same, though, then it should be assumed that no two models will be exactly the same. Therefore, a trade-off must be made by the user between utilizing currently available modeling techniques and "tailoring" the model to the watershed and purpose of the study (1).

CHAPTER III

METHOD AND PROCEDURE

Description of Study Basins

Three watersheds located at Grissom Air Force Base were used as the study basins. Grissom Air Force Base is located in northcentral Indiana and within the Wabash River watershed. Land use on the watersheds remained constant during the study period.

Land use varied from watershed to watershed. McDowell Ditch watershed is the largest of the three and contains the main runway, aircraft and maintenance buildings, and land used for pasture. Cline Ditch watershed contains a large residential area, a wooded area, and commercial buildings. East Ditch watershed contains apron space for aircraft, commercial buildings, and land used for agricultural activities. Table 1 lists the general physical characteristics of the study watersheds. Figure 1 shows the topographical map of Grissom Air Force Base and the surrounding areas, while Figures 2 and 3 show the storm sewer locations within Grissom Air Force Base.

Two types of soil exist at Grissom Air Force Base, Brookston silty clay loam and Fincastle silt loam. Brookston silty clay loam is a deep, poorly drained soil with a high available water capacity and organic matter content. This soil is usually located in valleys or on broad flats. Fincastle silt loam is a deep, somewhat poorly drained soil with a high available water capacity and a moderate organic content. It is usually located on gentle slopes. Both types of soil have low permeability.

TABLE 1

PHYSICAL CHARACTERISTICS OF STUDY WATERSHEDS

Watershed	Area (acres)	PI [1]	PF [2]	Slope (ft/ft)	Width of Overland Flow (ft)
McDowell Ditch	276	20	0	0.0014	3200
East Ditch	70	25	0	0.0043	1800
Cline Ditch	76	21	5	0.0190	1800

[1] Percent Imperviousness

[2] Percent Forest

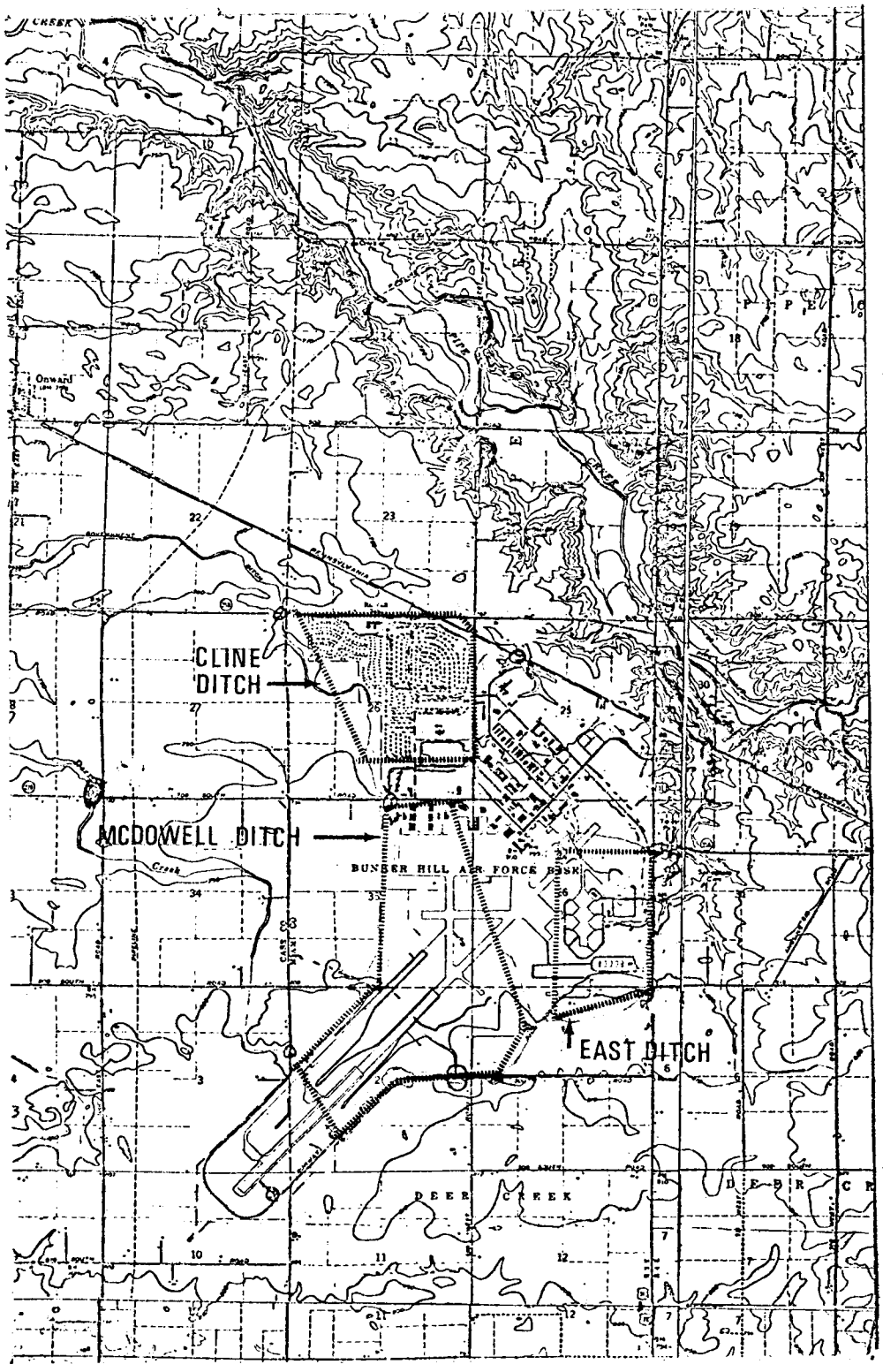

Figure 1. Topographical Map of Grissom Air Force Base, Indiana.

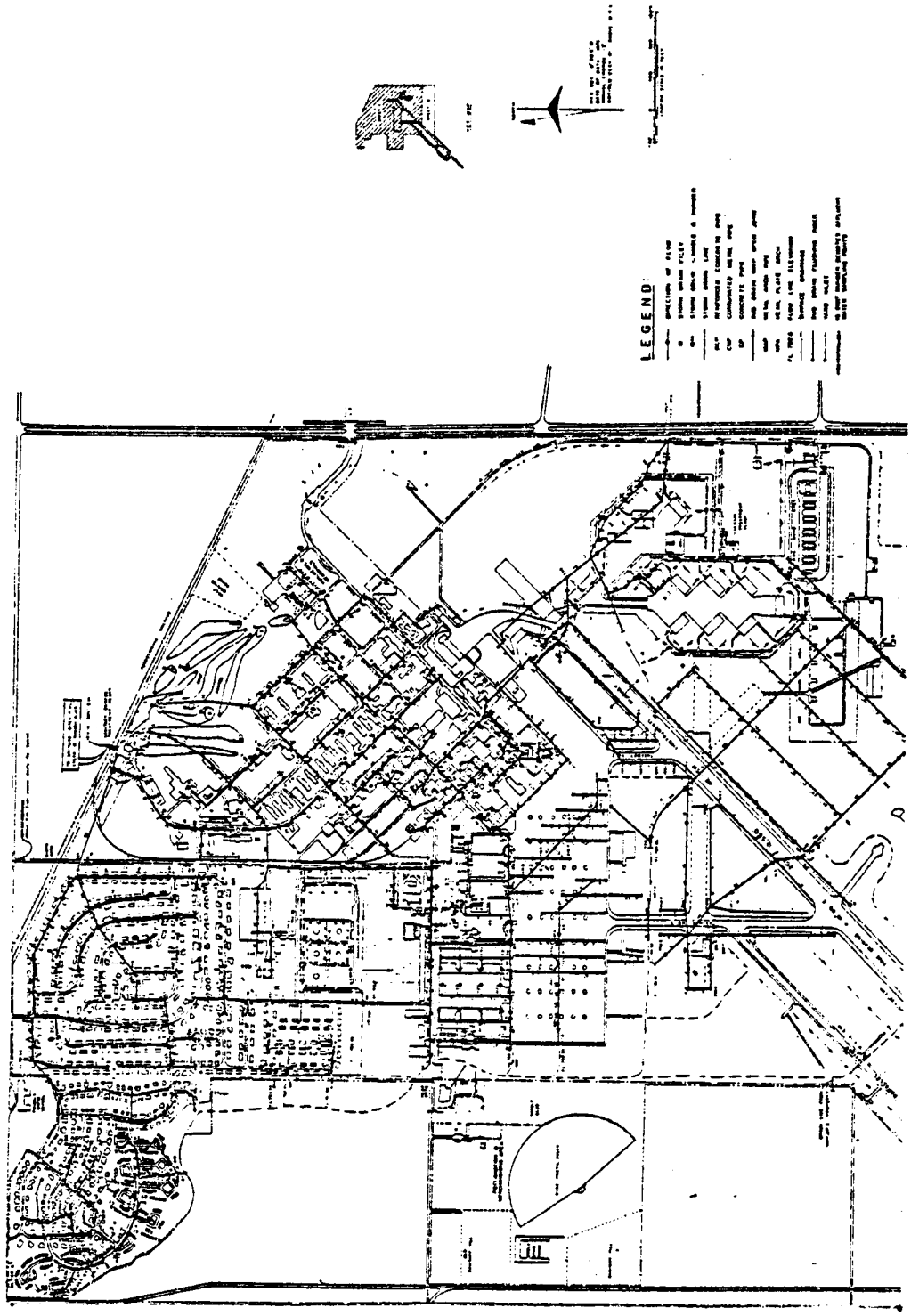

Figure 2. Storm Sewer Map (Part 1) of Grissom Air Force Base, Indiana.

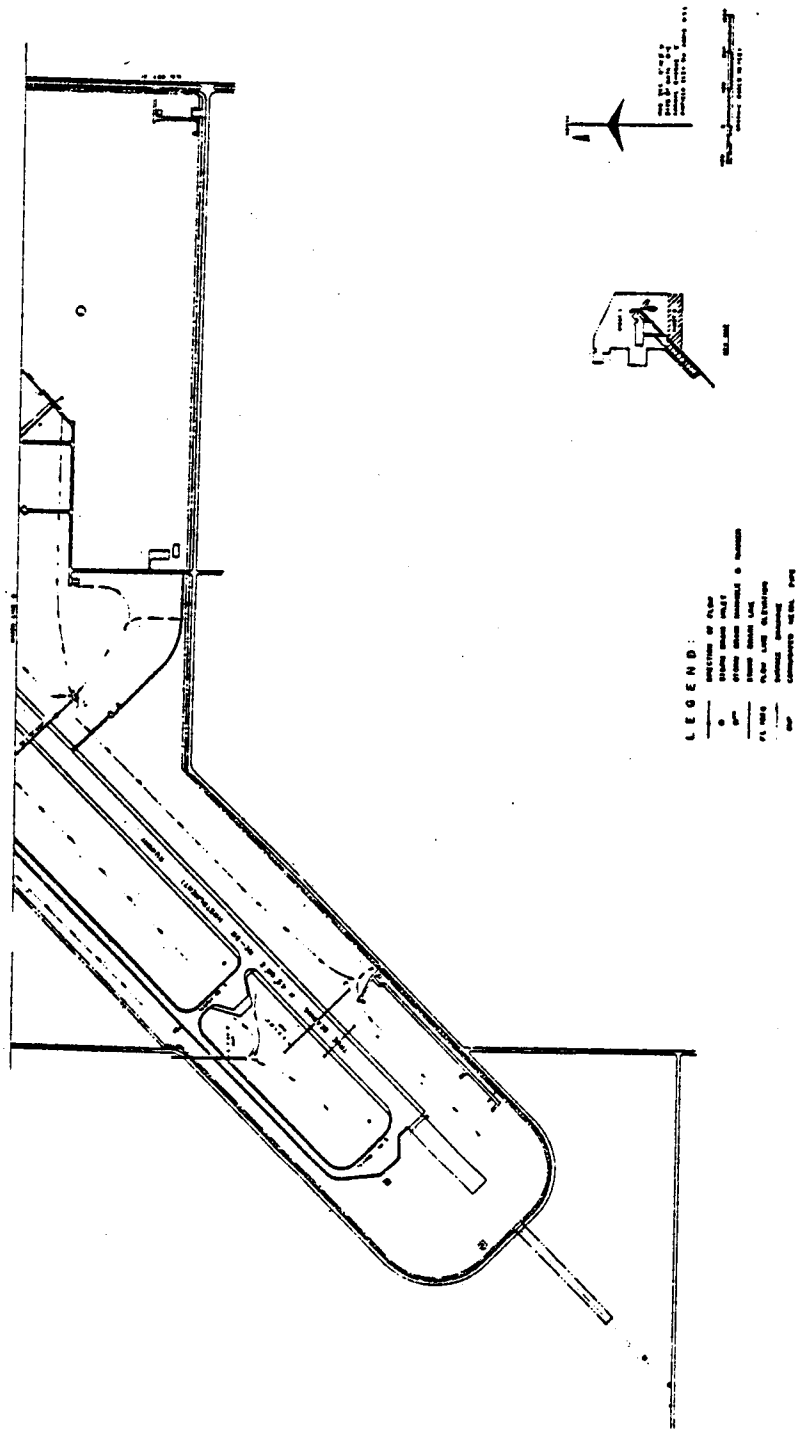

Figure 3. Storm Sewer Map (Part 2) of Grissom Air Force Base, Indiana.

Climate at Grissom Air Force Base is moderate. The average temperature is 60°F and the mean annual rainfall is 35 inches. The majority of the precipitation occurs in late spring and early summer. Historically, the maximum rainfall to occur in 24 hours was 4.5 inches, which is comparable to a 25 year return period storm. Snowfall, which occurs the heaviest in January and February, averages 28 inches annually.

Data Acquisition

Streamflow gauges were located at the outlet of each study basin, and a rainfall gauge was located within each study basin. Continuous rainfall records were obtained from early May, 1978, to middle December, 1978, at each gauge. Due to mechanical failures, some rainfall records were not obtained. In such cases, rainfall data from the remaining rainfall gauges were utilized to fill in the missing records.

The streamflow gauges were set to trip at a predetermined water level, thereby recording only the larger stormwater events. Five useable storm events were recorded at McDowell Ditch watershed, and four useable storm events were recorded at both East and Cline Ditch watersheds.

Rainfall and streamflow records were abstracted in fifteen minute time intervals, thus establishing the basic time step for the input data into the simulation programs. Streamflow data were obtained by the use of storage-discharge relationships developed for each gauge. Rainfall and runoff characteristics of the storm events used at Grissom Air Force Base are shown in Table 2.

TABLE 2

STORM CHARACTERISTICS,
GRISSOM AFB, BUNKER HILL, INDIANA

Watershed	Date	Rainfall (in)	Runoff (in)	Rainfall Duration (hrs)
McDowell Ditch	8-2-78	1.40	0.166	1.50
	8-27-78	0.50	0.066	2.00
	9-14-78	0.50	0.052	0.50
	11-14-78	1.49	0.309	11.50
	12-3-78	1.53	0.581	10.25
East Ditch	11-14-78	1.22	0.130	7.0
	11-17-78	0.42	0.063	5.5
	12-3-78	1.20	0.264	8.0
	12-7-78	0.60	0.058	14.5
Cline Ditch	8-2-78	1.30	0.014	1.0
	8-27-78	1.10	0.013	15.5
	9-14-78	0.50	0.011	0.5
	12-3-78	1.05	0.021	12.0

Most of the storms that occurred during data collection were not extreme events. The storm of August 2, 1978, was found to have characterized a return period of two years (46). The remaining rainfall events used in the simulations had return periods of less than one year.

To establish the stage-discharge relationship at each streamflow gauging station, rating curves were developed. Based on the assumption of steady-uniform flow, Manning's flow equation was used to determine discharges given depth of flow. Since the McDowell and East Ditch gauging stations were located at the outlet of rectangular box culverts and the Cline Ditch gauging station was located at the outlet of a large, circular storm sewer, physical data, such as the length, width, slope, and diameter, could easily be obtained. Inputing this information and flow depth into Manning's Equation, the rating curves were derived for each gauging station. These rating curves are shown in the Appendix section of this report. Table 3 lists the physical characteristics of the outlets of each watershed.

TENN-I Model

As previously stated, the main objective of this study was to evaluate the stormwater hydrograph simulation capabilities of three computer models. The first model tested was TENN-I, developed at The University of Tennessee. TENN-I utilizes the U.S. Soil Conservation Services (SCS)-Curve Number method for simulating rainfall excess (9) and the TVA double triangle unit hydrograph, URF, for distributing the runoff over time at the watershed outfall.

TABLE 3

PHYSICAL CHARACTERISTICS OF WATERSHED CULVERTS,
GRISSOM AIR FORCE BASE, INDIANA

Watershed	Width (ft)	Length (ft)	Slope (ft/ft)	Manning's in	Diameter (ft)
McDowell Ditch*	11.25	4.75	0.00138	0.012	
East Ditch*	5.65	3.75	0.0043	0.012	
Cline Pipe[1]			0.0021	0.012	3.5

*rectangular box culverts

[1]circular storm sewer pipe

Rainfall data was inputed at 15 minute time intervals (DT), and this DT was used in the model to simulate the stormwater hydrographs. The SCS curve number model can be expressed as:

$$SRO_i = (ARF_i - 0.2 * S)^2/(ARF_i + 0.8\ S) \qquad (25)$$

where SRO_i = total storm runoff to time i, inches

ARF_i = accumulated rainfall to time i, inches

S = maximum potential retention which is related to the SCS curve number CN, by definition, as CN = 1000/(10+S). Rainfall excess, PE, for each time interval is expressed as:

$$PE_i = SRO_i - SRO_{i-1}\ . \qquad (26)$$

To simulate the stormwater hydrograph, the double triangle model (31) URF was convoluted with rainfall excess. The URF represents a sum of two watershed responses, an initial and delayed response. The double triangle model URF is shown graphically in Figure 4. Symbols used in this figure are:

I = 1/DT

DT = time interval (min)

T1 = time to peak of initial response (hrs)

T2 = time to peak of delayed response and equal to the time of peak delayed response (hrs)

T3 = time to end of delayed response (hrs)

UP = peak of unit response function at T1

UR = peak of delayed response function at T2

pe(t) = precipitation excess as a function of time, t (in/hr).

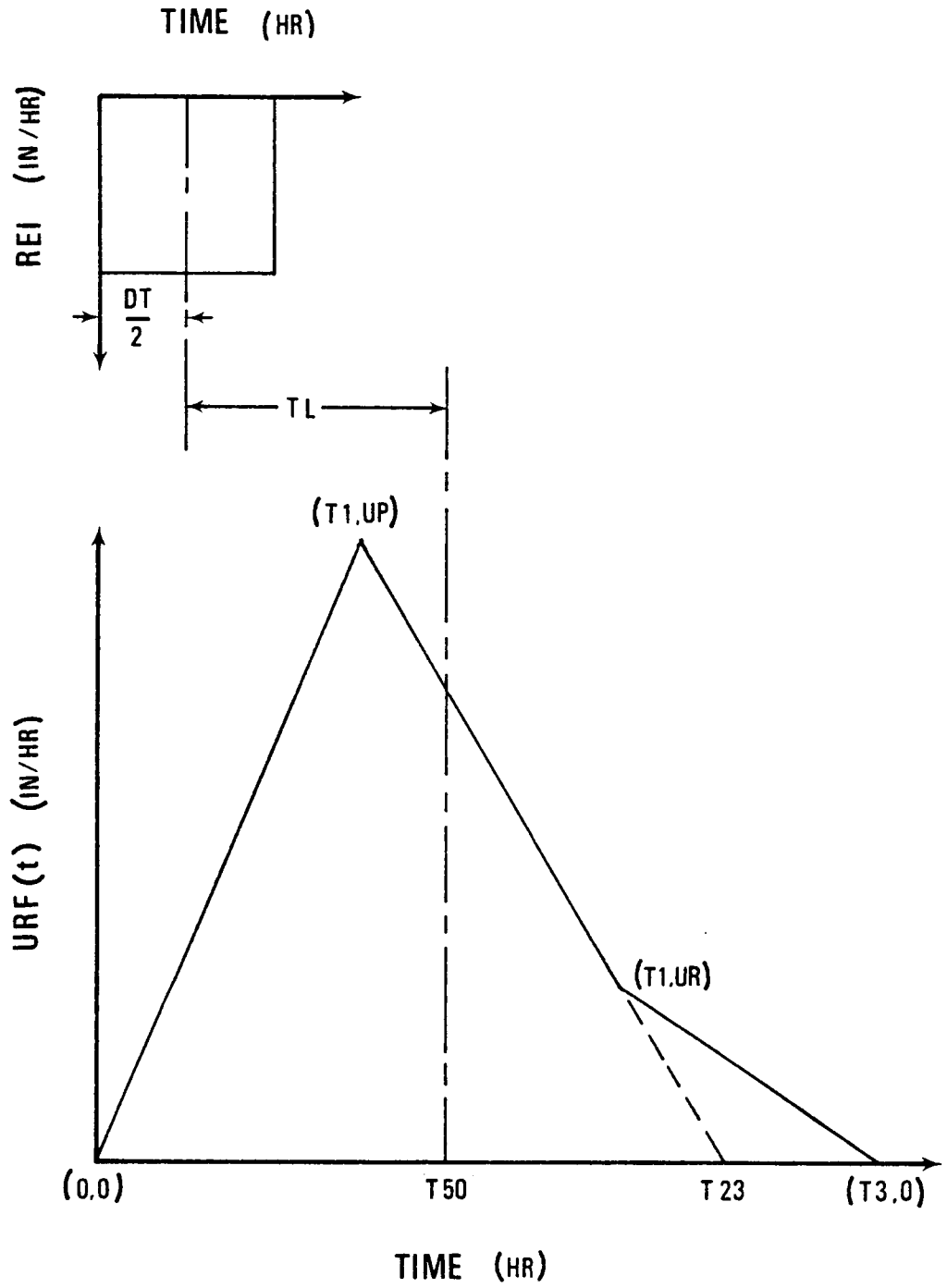

Figure 4. TVA Double Triangle Unit Response Function.

In order to obtain the shape of the URF for the study watersheds for each storm, coefficients relating the coordinates of the URF to lag time had to be defined. These coefficients, KT1, KT2, KT23, and KUP, are coordinates of the normalized unit function (NURF) which was defined in Chapter II. Once these coefficients of the watershed NURF are defined the storm URF can be defined by multiplying the time scale and dividing the ordinate scale by the associated lag time.

The NURF was defined using two approaches. For McDowell and East Ditch watersheds, the land use was considered basically rural, therefore, these coordinates were defined from the work of Overton and Troxler (34). For the Cline Ditch watershed, the NURF was derived using the analysis phase of the TVA double triangle model. To date no watershed had been analyzed which had extensive storm sewers.

The four optimized NURFs for each associated storm on Cline Ditch watershed were used to define the watershed NURF. First, lag time was computed for each storm by the method outlined by Troxler (24). Then, using the TVA double triangle mdoel, the storms were analyzed and T1, T2, T3, UP, UR were optimized for each storm. The NURFs for each storm were derived by dividing the ordinate scale and multiplying the time scale by the associated lag time. The results are shown in Tables 4 and 5 and in Figures 5 and 6.

In order to obtain the URF for the watershed, lag time must be determined. This value was computed by first solving for the lag modulus for the watershed. For rural watersheds, Overton and Troxler (34) found that lag modulus could be related to watershed characteristics as:

TABLE 4

CLINE DITCH OPTIMIZED URF's

Storm Date	TI	T2	T3	UP	UR	TL
8-2-78	0.940	3.008	61.00	0.524	0.0071	1.374
8-27-78	1.992	3.180	50.00	0.457	0.0114	1.969
9-15-78	3.040	4.332	38.00	0.462	0.00005	2.442
12-3-78	2.900	6.880	48.00	0.259	0.00489	3.288

TABLE 5

CLINE DITCH NURF's

Storm Date	KT1	KT2	KT3	KUP	KUR
8-2-78	0.684	2.189	44.41	0.719	0.00969
8-27-78	1.011	1.615	25.39	0.900	0.0224
9-14-78	1.245	1.302	15.56	1.126	0.000122
12-3-78	0.882	2.039	14.59	0.852	0.0161
Mean	0.956	1.80	25.0	0.900	0.0121

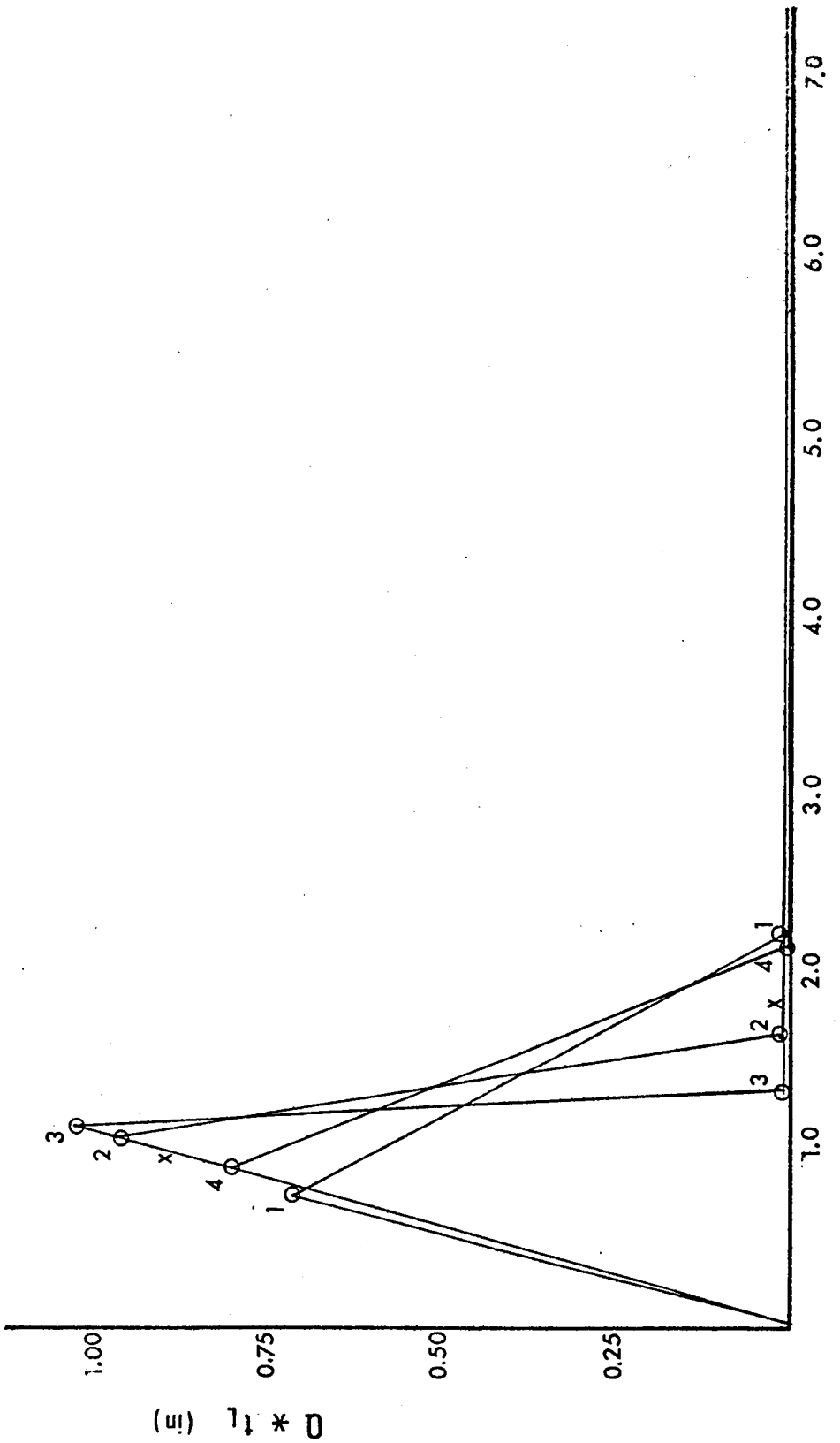

Figure 5. Cline Ditch Nomalized Unit Response Functions.

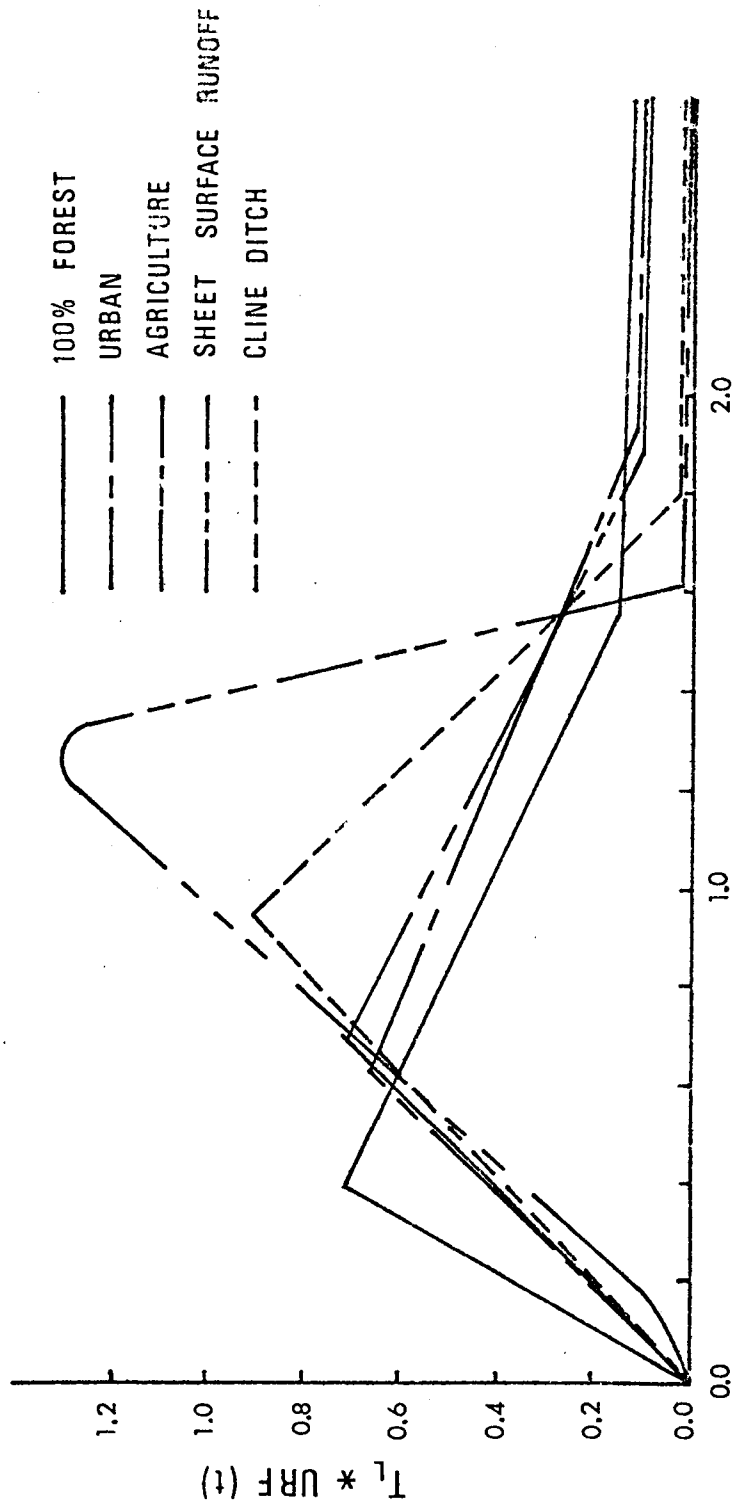

Figure 6. Normalized Unit Response Functions for Different Land Uses.

$$\mu \text{ (hrs)} = 0.0600 * \text{AREA} + 0.0203 \text{ PF} + 1.16 \qquad (27)$$

where AREA = surface drainage area, sq.mi.,

PF = percent forest.

For urban watersheds, lag modulus was related to AREA and the percent imperviousness, PI, as:

$$\mu \text{ (hrs)} = 3.24[\text{AREA}/\text{PI}]^{0.6}. \qquad (28)$$

Overton (29), using kinematic wave theory, related lag time to lag modulus and the weighted rainfall excess, or:

$$t_L = \mu/\text{REI}^{0.4} \qquad (29)$$

where t_L = lag time in minutes

μ = lag modulus in minutes

REI = weighted rainfall excess in inches/hour.

Weighted rainfall excess was defined as:

$$\text{REI} = \sum_{j=1}^{N} (i_{e_j}{}^2) / \sum_{j=1}^{N} (i_{e_j}) \qquad (30)$$

where i_e = rainfall excess rate

N = number of time increments of i_e.

Hence, once the NURFs and lag times are defined for each watershed, the URF may be simulated. Rainfall excess for a storm is convoluted with the URF in TENN-I which produces the simulated storm runoff hydrograph.

Storage, Treatment, Overflow, and Runoff Model (STORM)

The next computer model tested was the Storage, Treatment, Overflow, and Runoff Model (STORM) developed by Water Resources Engineers, Inc. for the U.S. Army Corp of Engineers (47). This model uses three methods to simulate precipitation excess: a runoff coefficient, a SCS-CN method, and the combination of the first two methods. The resulting direct runoff is convoluted with the SCS single triangle URF to produce the simulated storm hydrograph.

STORM is a continuous simulation package capable of predicting flows for long durations. Therefore, precipitation data is inputed on hourly time steps for any time period to be modeled. For this study, all recorded rainfall data, which covers approximately six continuous months, was inputed into STORM.

STORM simulates rainfall excess by one of three methods: the coefficient method, the SCS Curve Number technique, or a combination of the two (47). The coefficient method assumes a certain fraction of rainfall becomes runoff. The SCS-Curve Number technique is based upon a curvilinear relationship between rainfall and runoff. The third method uses the coefficient method on impervious areas and the SCS Curve Number on pervious areas, weighting the sum according to the percent impervious of the entire watershed.

In order to make comparisons between TENN-I and STORM, the SCS-CN method was used in this study. Also, parameters used in this method have, to some degree, been documented in the literature. No guidelines were given to define the parameters used in the coefficient method in STORM, hence, this option was not used.

The SCS-CN method is based upon the relationship previously described in Chapter I, and may be written as equation (3). Since STORM is a continuous simulation model, a procedure was developed to account for losses such as evapotranspiration, infiltration, and percolation during periods of no rain. The model computes soil moisture capacity (deficit) at the beginning of each time increment by the following equation:

$$S_t = S_{t-1} - IN * \Delta t + A * EV * \Delta t + B * MP * \Delta t \qquad (31)$$

where $A = 0.7\ ((SM-S_{t-1})/SM))^v$

$B = ((SM-S_{t-1})/SM)^p$

S = soil moisture capacity for storage of water in inches

IN = maximum infiltration rate from initial abstraction in inches/hour

EV = pan evaporation rate in inches/hour

MP = maximum soil percolation rate in inches/hour

SM = maximum soil capacity for storage of water in inches

v = exponent regulating evapotranspiration

p = exponent regulating percolation.

Table 6 lists values for the parameters used in Equation (31). These input values were suggested by the builders of STORM for different land uses in sample runs previously made to test the model (47).

STORM requires that each watershed be delineated into different land uses in order to account for rainfall losses to the soil. Soil properties and percent imperviousness for each land use are used by the model to simulate total rainfall losses. Table 7 shows how each

TABLE 6

INPUT VALUES FOR PARAMETERS USED BY STORM

Land Use	IN (in/hr)	SM (in)	MP (in/hr)	V	P
Pasture	0.03	3.3	0.02	1.0	2.0
Commercial	0.01	1.1	0.03	1.0	2.0
Unimproved	0.02	1.8	0.02	1.0	2.0
Single	0.02	1.8	0.02	1.0	2.0
Multiple	0.01	1.1	0.03	1.0	2.0

TABLE 7

LAND USES FOR THE STUDY WATERSHEDS

	Land Use	Percent Area	Percent Imperviousness
McDowell Ditch	Range	70	2
	Comercial	25	80
	Unimproved	5	0
East Ditch:	Range	65	2
	Commercial	25	75
	Multiple	10	25
Cline Ditch:	Park	5	0
	Single	70	25
	Multiple	25	15

of the watersheds were divided by land use and the values inputed to STORM. Infiltration values for soil properties suggested by the model builders for different land uses in this study were inputed into STORM. Pan evaporation rates were obtained from the National Weather Service,

The triangular shaped unit hydrograph, developed by the SCS (20), shown in Figure 7, is used in STORM to route rainfall excess to the outfall. Only two additional variables are required to use this procedure: (1) the time of concentration of the basin and (2) the ratio of time to recession to time of peak of the unit hydrograph. The equations which describe the characteristics of the unit hydrograph are:

$$T_p = 0.5 + 0.6\ T_c \tag{32}$$

$$K = 2/(1 + T_r/T_p) \tag{33}$$

$$Q_p = (1.00833\ (KAQ))/T_p \tag{34}$$

where T_p = time to peak of the unit hydrograph in hours

T_c = time of concentration of the subbasin in hours

T_r = time of recession of the unit hydrograph in hours

A = drainage area in area

Q = one inch of surface runoff

Q_p = unit hydrograph peak in cubic feet per second.

For this study, the value inputed into STORM for the ratio of time to recession to time of peak was 1.67; the value inputed for the time of concentration was 1.5. These values are suggested by the builders

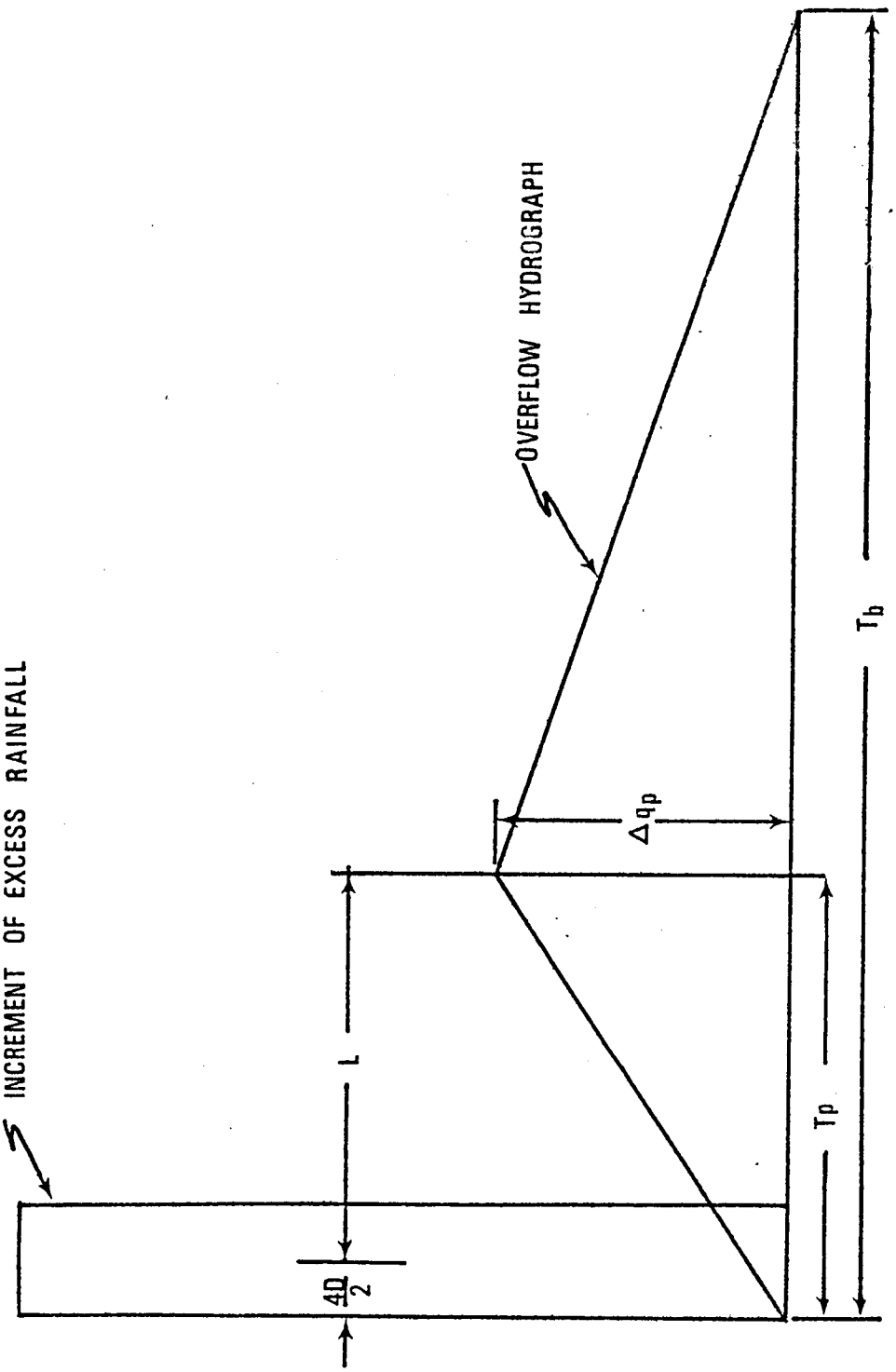

Figure 7. SCS Unit Response Function.

of STORM when unit hydrograph characteristics of the watershed are unknown.

Once the SCS unit hydrograph is defined, rainfall excess is convoluted at one hour time steps to obtain the simulated stormwater hydrograph.

Storm Water Management Model (SWMM)

The Storm Water Management Model (SWMM) is the most technically complex and deterministic model evaluated in this report. This model is divided into five separate subroutines or "blocks." Each block performs some function for water quantity and quality analysis. Since the objective of this study was to simulate stormwater hydrographs, only the first block, called Runoff, was utilized.

The rainfall hyetograph (inches/hour) was inputed into SWMM. In order to simulate infiltration and direct runoff, rainfall is converted to water depths at the watershed surface (39). The depth of flow may be expressed as:

$$D_1 = D_t + R\Delta t \tag{35}$$

where D_1 = water depth after rainfall

D_t = water depth of the subcatchment at time, t

R_t = intensity of rainfall in time interval, Δt.

Infiltration is simulated by Horton's exponential function, which may be written as

$$I_t = f_0 + (f_i - f_0)\, e^{-\alpha t} \tag{36}$$

where I_t = infiltration at time t

f_o, f_i, α = coefficients in Horton's model.

Excess runoff, or depth of flow, is computed by the following equation:

$$D_2 = D_1 - I_t \Delta t \tag{37}$$

where D_2 = water depth after accounting for infiltration

D_1 = water depth before accounting for infiltration.

SWMM has the capability of dividing a watershed into subcatchments and computing runoff from each of the subcatchments. Overland, open channel, and pipe flows are simulated by Manning's equation, or:

$$V = \frac{1.49}{n} (D_t)^{2/3} S^{1/2} \tag{38}$$

and

$$Q_W = V W D_t \tag{39}$$

where V = velocity of flow

n = Manning's resistance coefficient

S = ground slope

W = width

Q_W = outflow rate.

Inflow to a gutter or main channel is computed by summing all outflow rates from upstream subcatchments and flow rate of immediate upstream gutters. This flow becomes the discharge hydrograph at the outlet of the watershed. Figure 8 describes the drainage system defined by SWMM.

Two major steps are involved in simulating storm hydrographs using SWMM: (1) determining the geometric representation of the

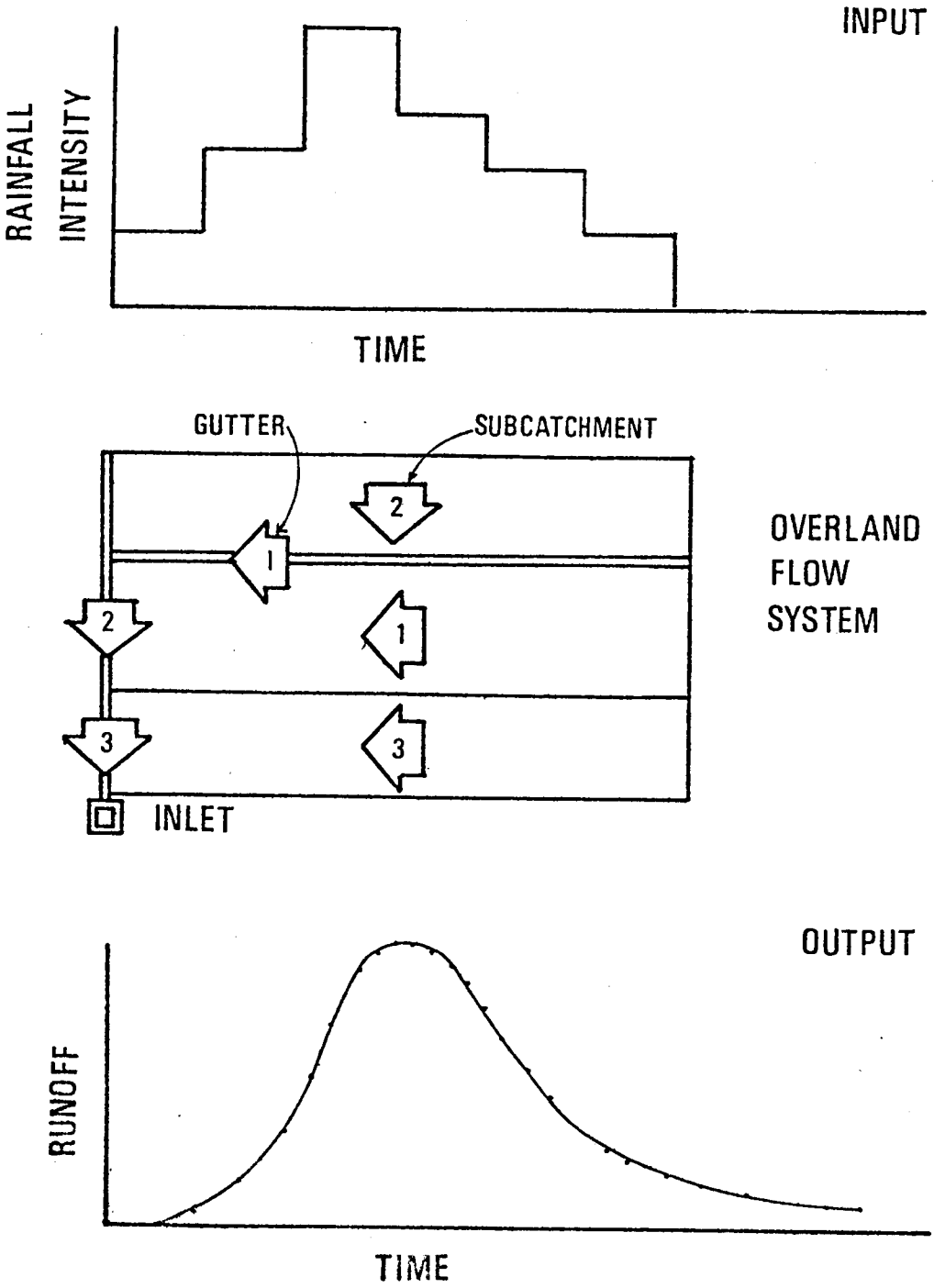

Figure 8. SWMM Hydrograph Simulation Technique.

drainage basin and (2) estimating the coefficients used in the infiltration and overland flow equations.

The method of discretization is used to describe the physical watershed characteristics. SWMM is capable of representing the watershed as a network of hydraulic elements such as subcatchments, gutters, and pipes. Watershed geometry may be described by one of two approaches: (1) a "coarse" or (2) "fine" discretization (45). A "fine" discretization may result in the drainage area being broken down into several subcatchments containing all major transport systems within the watershed. A "coarse" discretization is a more unrestrained approach for describing the watershed, thereby reducing the number of transport systems used in simulation.

Since the builders of SWMM suggested only a "coarse" discretization for predicting flows at one outlet (39), this approach was chosen for representing the characteristics of the study watersheds. Since McDowell and East Ditch watersheds are basically rural and limited storm sewer networks, the "coarse" discretization may be justified. Cline Ditch watershed is comprised of a large residential section that is heavily sewered. Since all lateral sewers feed into one main trunk pipe, and the area is uniform in physical characteristics, this watershed was treated as one hydrologic unit. For this reason, a "course" discretization was used for Cline Ditch watershed.

Required data for "coarse" discretizations are width of overland flow, area, percent imperviousness, ground slope, Manning's resistance factors for pervious and impervious areas, and maximum and minimum infiltration rates for each subcatchment in the drainage basin.

Default values recommended by the builders of SWMM were used for Manning's resistance factors and Horton's infiltration coefficients. The SWMM User's Guide (39) outlines the procedure for estimating the width of overland flow of the subcatchment, while the remaining parameters used in SWMM were estimated from maps of the study basins. Table 8 tabulates the input values of the parameters used by SWMM.

TABLE 8

INPUT VALUES FOR PARAMETERS USED BY SWMM

Watershed	Width (ft)[1]	PII[2]	PI	Slope (ft/ft)
McDowell Ditch	3200	10	20	0.001
East Ditch	1800	11	24	0.0004
Cline Ditch	1900	10	15	0.002

[1]Refers to width of overland flow

[2]Refers to the percent impervious with zero detention

CHAPTER IV

DISCUSSION OF RESULTS

Storm Hydrograph Simulations

The major objective of this report is to evaluate the ability of three stormwater models to simulate real storm events at Grissom AFB, Indiana. Tables 9, 10, 11 and 12 and Figures 9, 10 and 11 show the results of the evaluations. The remaining storm simulations are shown in the Appendix section of this study. In addition, sensitivity analysis was performed on each of the models in order to determine the parameters which chiefly affect the simulated hydrographs.

Three characteristics of the simulated hydrographs were used to evaluate the models: peak discharge, time to peak, and the volume of direct runoff. Shape, a more qualitative characteristic, was also used to judge the model's ability to simulate.

TENN-I Simulations

In order to determine how well TENN-I simulated peak discharge, time to peak, and shape of the storm hydrograph, the CN's were optimized to allow the volumes of the observed and simulated hydrographs to be equal. As a test of the runoff volume simulation phase of TENN-I the curve numbers were estimated using the SCS Hydrology Handbook (9). The anecedent moisture condition (AMC) is divided into three categories: (1) AMC-I is the lower limit of moisture or the upper limit of S, (2) AMC-II is the average moisture content, and (3) AMC-III is the upper limit of moisture or the lower limit of S.

TABLE 9

MCDOWELL DITCH SIMULATION RESULTS

Storm Date	Qp Observed (cfs)	tp Observed (hrs)	TENN-I Qp (cfs)	TENN-I tp (hrs)	STORM Qp (cfs)	STORM tp (hrs)	SWMM Qp (cfs)	SWMM tp (hrs)
8-3-78	15.0	2.00	18.6	2.50	58.2	2.00	8.07	7.00
8-27-78	6.7	11.50	2.7	9.25	7.2	7.00	10.45	4.25
9-15-78	6.6	2.25	4.7	1.75	7.6	2.00	55.42	0.50
11-14-78	13.0	10.75	9.0	10.50	30.0	8.00	74.61	8.50
12-3-78	22.0	6.50	42.21	6.50	55.8	6.00	74.05	8.50

TABLE 10

EAST DITCH SIMULATION RESULTS

Storm Date	Qp Observed (cfs)	tp Observed (hrs)	TENN-I Qp (cfs)	TENN-I tp (hrs)	STORM Qp (cfs)	STORM tp (hrs)	SWMM Qp (cfs)	SWMM tp (hrs)
11-14-78	1.38	10.50	0.64	11.00	4.8	9.00	32.28	8.75
11-17-78	0.59	7.50	0.63	7.50	1.0	6.00	1.69	4.50
12-3-78	4.03	4.75	3.63	4.75	8.6	7.00	2.04	3.25
12-7-78	0.65	7.25	0.45	9.00	0.7	4.00	23.51	6.25

TABLE 11

CLINE DITCH SIMULATION RESULTS

Storm Date	Qp Observed (cfs)	tp Observed (hrs)	TENN-I Qp (cfs)	TENN-I tp (hrs)	STORM Qp (cfs)	STORM tp (hrs)	SWMM Qp (cfs)	SWMM tp (hrs)
8-2-78	2.02	0.75	0.88	1.25	18.6	2.00	23.33	1.00
8-27-78	1.70	2.25	1.20	2.25	6.2	2.00	14.48	0.75
9-14-78	1.90	1.25	1.04	1.00	3.2	2.00	7.11	1.00
12-3-78	1.90	5.75	2.12	5.50	6.0	4.00	5.41	4.00

TABLE 12

VOLUME SIMULATION RESULTS

Watershed	Storm Date	Rainfall(in)	Observed Runoff (in)	TENN-I Runoff (in)	STORM Runoff (in)	SWMM Runoff (in)
McDowell Ditch	8-3-78	1.4	0.166	0.178	0.451	0.110
	8-27-78	0.5	0.066	0.067	0.059	0.080
	9-15-78	0.5	0.052	0.054	0.058	0.150
	11-14-78	1.49	0.309	0.320	0.485	0.440
	12-3-78	1.53	0.581	0.568	0.484	0.510
East Ditch	11-14-78	1.22	0.130	0.127	0.220	0.450
	11-17-78	0.42	0.063	0.060	0.045	0.060
	12-3-78	1.20	0.264	0.266	0.341	0.070
	12-7-78	0.60	0.058	0.055	0.045	0.350
Cline Ditch	8-2-78	1.30	0.014	0.015	0.525	0.190
	8-27-78	1.10	0.013	0.013	0.269	0.180
	9-14-78	0.50	0.011	0.011	0.098	0.070
	12-3-78	1.05	0.021	0.021	0.198	0.110

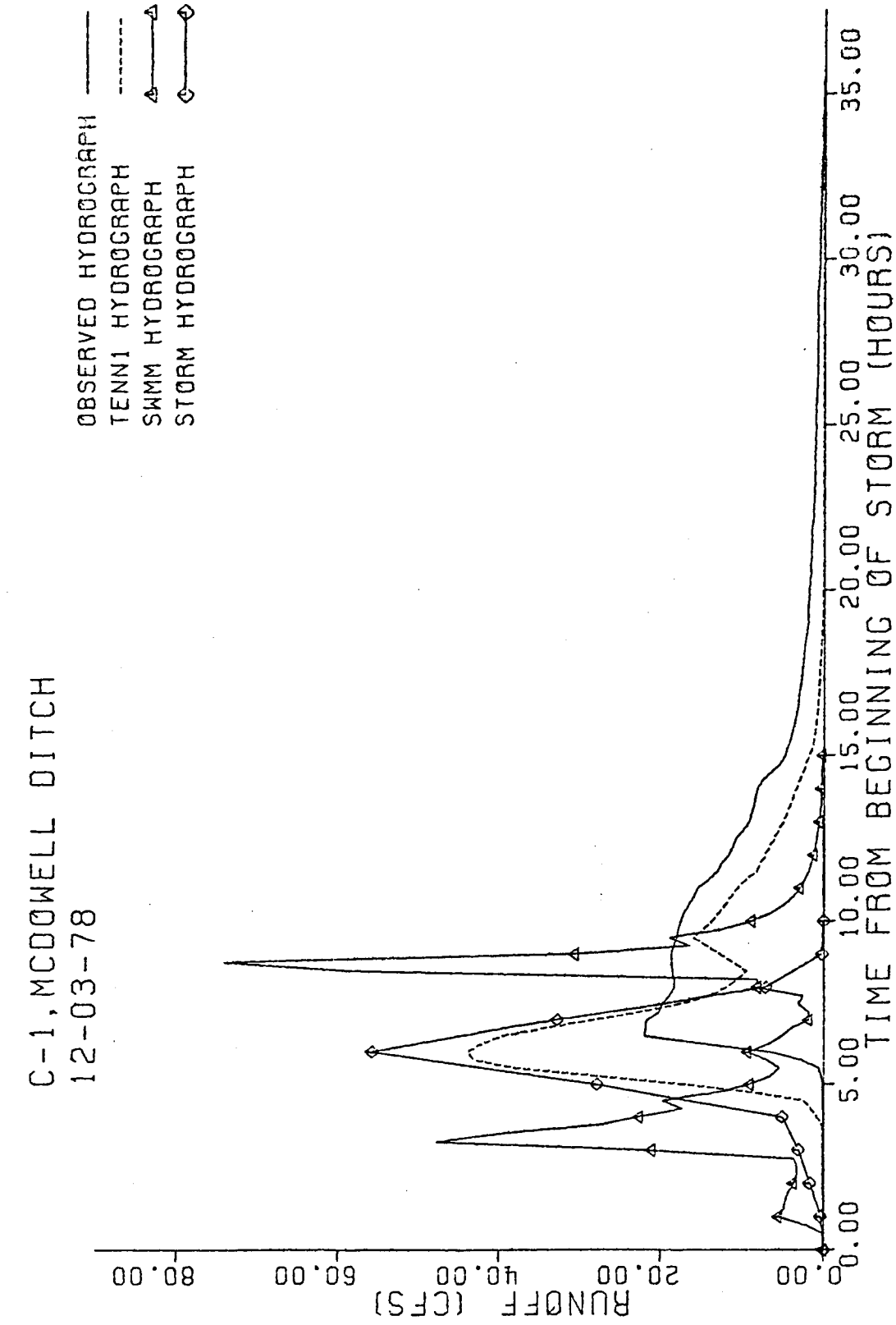

Figure 9. Example of McDowell Ditch Hydrograph Simulation Results.

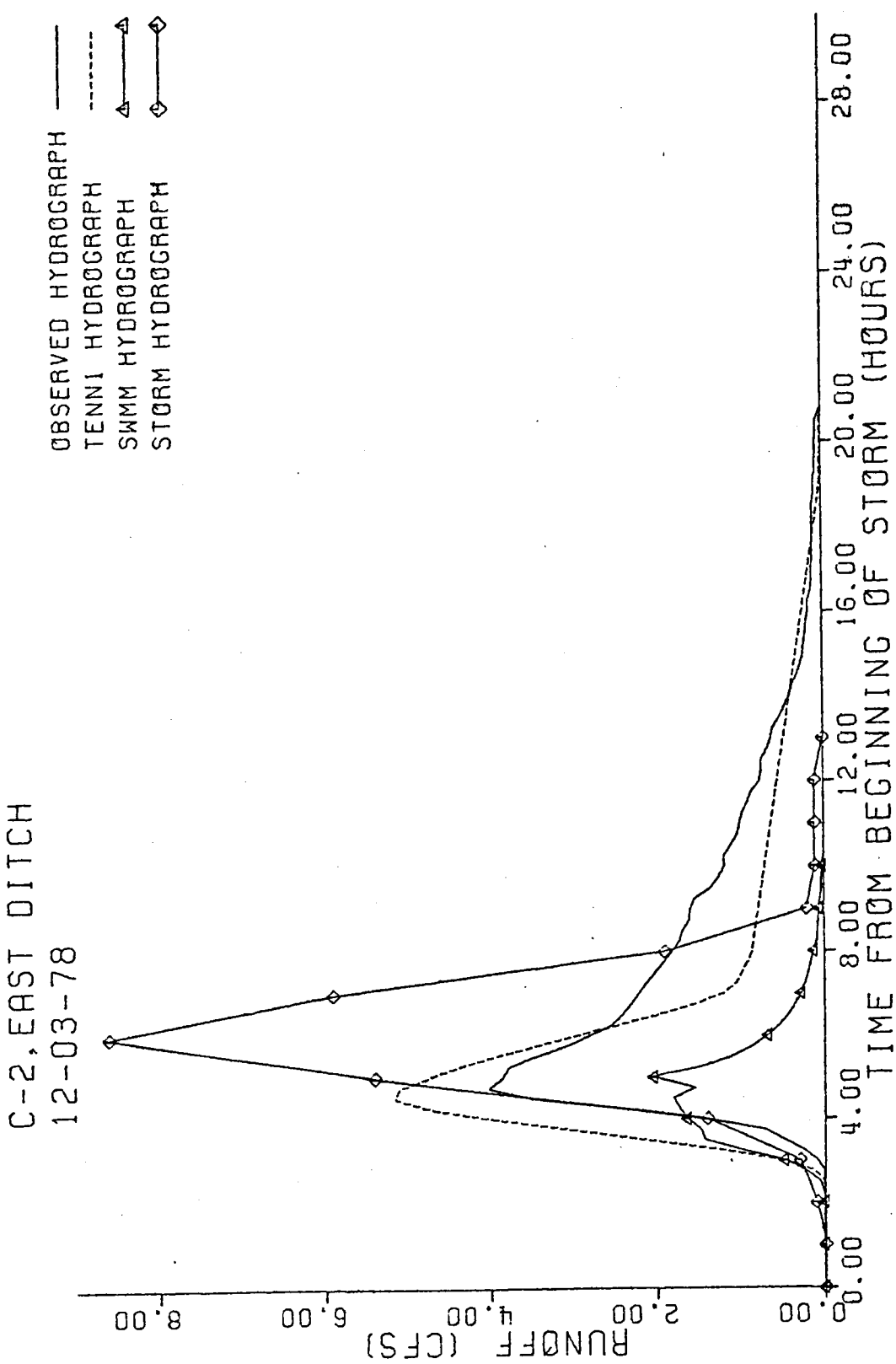

Figure 10. Example of East Ditch Hydrograph Simulation Results.

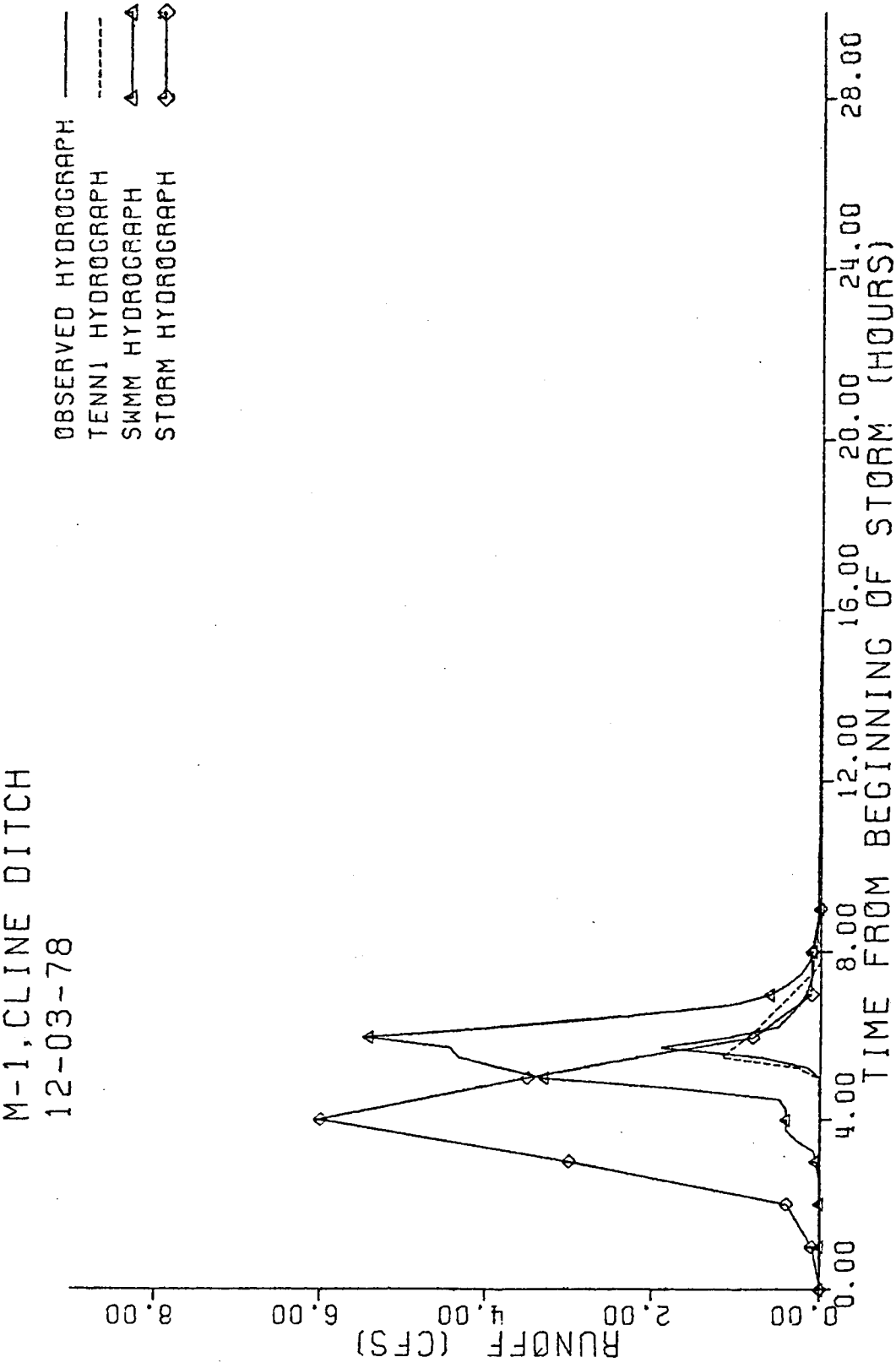

Figure 11. Example of Cline Ditch Hydrograph Simulation Results.

The AMC was determined by the total amount of rainfall which occurred in the five day period preceding a storm. The results are tabulated in Table 13.

For an average AMC (i.e., AMC-II), there is close agreement between the optimized and estimated CNs. Otherwise, estimated CNs were lower than optimized CNs for storms with AMC-I and higher than optimized CNs for storms with AMC-III. Equation (1), used in TENN-I, assumes an AMC-II when simulating rainfall excess. Therefore, the CNs were optimized for AMC-II, which may explain the variation in the estimated CNs and optimized CNs for AMC-I and AMC-III.

According to Troxler (24), TENN-I was developed based upon three assumptions: the watershed response to rainfall is lumped, time-invariant, and linear. Since the study watersheds are small and large portions of these watersheds were comprised of one type of land use, these assumptions could be made with a fair degree of conficence. This is evident by comparing the simulated peak discharges with the associated observed peak discharges.

For high rainfall volume storms, TENN-I simulated hydrographs were in close agreement with the observed hydrographs. For multi-peak, long duration storms, though, TENN-I simulations were not in good agreement with the observed stormwater hydrographs. The storm of August 27, 1978, at McDowell Ditch had two definite peaks, but the TENN-I hydrograph simulated only one. Fortunately, the high rainfall volume storms are considered most important to hydrologic studies or for design, therefore TENN-I is able to simulate storms of hydrologic significance accurately.

TABLE 13

OPTIMIZED CN's VERSUS ESTIMATED CN's

Location	Storm Date	Rainfall (in)	Runoff (in)	AMC	Opt CN	Est. CN	Est. Runoff (in)	Percent Error(%)
McDowell Ditch	8-3-78	1.60	0.180	III	77	89	0.55	205
	8-27-78	0.50	0.067	I	91	70	0.01	85
	9-15-78	0.50	0.054	I	90	67	0.01	81
	11-14-78	1.49	0.32	II	81	83	0.33	3.1
	12-3-78	1.53	0.568	I	87	73	0.10	82
East Ditch	11-14-78	1.22	0.127	I	78	60	0.01	92
	11-17-78	0.42	0.060	II	93	88	0.05	16
	12-3-78	1.20	0.266	I	85	70	0.03	88
	12-7-78	0.60	0.055	II	87	85	0.03	45
Cline Ditch	8-2-78	1.30	0.014	I	68	45	0.01	28
	8-27-78	1.10	0.012	III	70	85	0.20	1566
	9-14-78	0.50	0.011	III	85	95	0.18	1536
	12-3-78	1.05	0.019	II	72	70	0.01	47

In addition, the NURF used to simulate the McDowell and East Ditch storms were optimized for storms in the Tennessee Valley. This may explain why some of the TENN-I simulated hydrograph shapes were poor. Had NURFs been derived for these watersheds as in Cline Ditch, the shapes may have been closer to the observed shapes.

STORM Simulations

STORM and TENN-I have certain similarities. They both utilize the same rainfall excess simulation and both utilize a URF to convolute the rainfall excess. However, STORM specifies a constant URF for a watershed, whereas TENN-I specifies a NURF for a watershed. Given rainfall excess, a storm URF is simulated. TENN-I, therefore, considers watershed response to be linear within a storm, but nonlinear from storm to storm, thereby producing a family of URFs for each watershed. Unlike TENN-I, however, STORM is a continuous model which performs a soil moisture accounting between rainfall events.

All input parameters to STORM used in the simulations were estimated by considering the examples displayed in the user's manual. This was necessary because the STORM user's manual did not include a prediction scheme for the model parameters. Hence, it would have been fortuitous for STORM to have produced accurate simulations at Grissom AFB. For example, all STORM simulated peak dischargers and volumes for the Cline runoff events were considerably higher than the observed peak discharges and volumes. This could be due in part to errors in predicting the input parameters.

In addition, the lack of a nonlinear URF in STORM (i.e., the SCS model) the shapes of the simulated storm hydrographs were only fair.

Some storm events, such as the one at McDowell Ditch on August 8, 1978, were poor while others, such as the one at McDowell Ditch on September 9, 1978, were good. STORM also seemed to simulate well for short, intense storms, but poor for storms of longer duration.

STORM was not effective in simulating the time to peak. The time increment (DT) used by STORM to input rainfall and to convolute a hydrograph is fixed at one hour. For this reason, peak discharges are limited to hourly increments from the beginning of the storm, and this may create errors when simulating the time to peak. Also, errors in simulating time to peak may be induced by the technique used in STORM to continually account for the soil moisture capacity value in Equation (31). For example, for the storm of August 27, 1978, at McDowell Ditch, STORM simulated the peak discharge to be seven hours from the beginning of the storm which was four and a half hours earlier than the observed peak discharge. Since no large storms had occurred preceding this storm, the AMC for this storm event may have been condition I, thereby causing the simulated hydrograph to peak earlier than the observed.

SWMM Simulations

SWMM is a deterministic model, therefore, when input parameters are known with a high degree of certainty, storm simulations should be modeled accurately. Unfortunately, little work has been reported for defining the input parameters. Since "default" values suggested in the user's manual were used in this study, SWMM simulations for the watersheds at Grissom Air Force Base were very poor. As with STORM,

it would have been fortuitous for SWMM to have produced accurate simulations.

The peak discharges simulated by SWMM were consistently higher than the observed for all three study watersheds. This is true especially for the storms simulated at Cline Ditch. In addition, the accuracy of SWMM might increase as the watershed is divided into subcatchments. Also, since no predictive scheme for the parameters was provided in the user's manual, "default" values would have to be used for all subcatchments.

The alternatives to physically describe the watershed by SWMM are infinite. They range from a detailed or "fine" discretization of the basin to a simplified or "coarse" discretization of the basin. Therefore, a "fine and "coarse" discretization plan for each study watershed was inputed into SWMM to evaluate the effects on the stormwater hydrographs.

The first set of simulations for the watershed utilized a "coarse" plan. Each basin was represented by one subcatchment with no gutter or pipe network. The second set of simulations for the watersheds utilized a "fine" plan. Cline Ditch was represented by three subcatchments with each subcatchment containing a pipe. East Ditch was represented by three subcatchments with each subcatchment containing a ditch.

The SWMM stormwater hydrograph simulations show little variation between the two discretization methods in terms of peak discharge,

time to peak, and shape. Table 14 tabulates the results of the simulations.

Generally, SWMM simulated peak discharges higher than the observed peak discharges, but for some storms the shape of the simulated hydrographs paralleled the observed observed hydrographs. The Cline Ditch storm of August 27, 1978, shows SWMM overpredicting, but it simulated both peaks and the associated times to peak correctly. For the remaining storms, though, the shapes of the simulated hydrographs were poor with regard to the observed hydrographs. For example, consider the storm of November 14, 1978, at McDowell Ditch. The simulated peak discharge occurred two hours earlier than the observed peak discharge and was considerably higher than the observed. This may be due to a large burst of rainfall occurring at a time when Horton's infiltration model had simulated saturation conditions in the soil. Therefore, this large burst of rainfall may have contributed heavily to the SWMM simulation of rainfall excess.

The discharges simulated by SWMM tended to be "flashy." Sharp rises in the hydrograph were predicted after intense bursts of rainfall. This may have been due to the percent imperviousness with zero detention parameter used by SWMM. The parameter value used in this study may have been considerably higher than the actual basin characteristics indicated.

Sensitivity Analysis

In order to determine the input parameters which should be measured with the greatest care, sensitivity analysis was performed on the three stormwater models. This was done by perturbating one parameter and

TABLE 14

SWMM SIMULATIONS: "FINE" DISCRETIZATION VERSUS "COARSE" DISCRETIZATION

Watershed	Storm Date	"Fine" Discretization		"Coarse" Discretization	
		Q_p (cfs)	t_p (hrs)	Q_p (cfs)	t_p (hrs)
East Ditch	11-14-78	5.8	3.5	23.2	8.75
	11-17-78	1.8	4.5	1.7	4.5
	12-03-78	1.8	3.0	2.0	3.25
	12-07-78	2.0	6.5	23.5	6.25
Cline Ditch	08-02-78	24.4	1.0	23.33	1.0
	08-27-78	15.8	1.0	14.50	0.75
	09-15-78	7.2	1.0	7.11	1.00
	12-03-78	5.8	4.0	5.4	4.00

holding the remainder constant for a simulation. The storm used for the sensitivity analysis was the McDowell Ditch storm of December 3, 1978.

The input parameters of STORM describing the watershed characteristics were all individually perturbated $\pm$ 10 percent of their true values for the purpose of evaluating their sensitivity on peak discharge and the associated time to peak. The simulations showed that the parameters which are used to simulate soil moisture capacity in Equation (31) have a negligible effect on the peak discharge or its associated timing. The results of the sensitivity analysis for STORM are shown in Table 15.

As seen in this table, the two parameters which determine the shape of the URF, time of concentration and ratio of time or recession to time to peak, have a sensitive effect on the peak discharge, but not on the timing of the peak discharge.

The land use parameters in STORM were also varied to evaluate the effect or the peak discharge and time to peak. The percent of range land use and commercial land use were perturbated and showed very little effect on the stormwater characteristics.

The input parameters used in SWMM to simulate the storm hydrograph may be divided into three categories: (1) parameters used to describe the watershed characteristics, (2) parameters used in Manning's equation, and (3) parameters used in Horton's infiltration equation. All parameters were perturbated $\pm$ 10 percent of their "true" values. The sensitivity analysis showed that no parameter effected the time to peak characteristic. The results of the sensitivity analysis for SWMM are shown in Table 16.

TABLE 15

SENSITIVITY ANALYSIS RESULTS - STORM,
STORM OF DECEMBER 3, 1978, MCDOWELL DITCH

Parameter	True Parameter Value	Perturbated Value [1]	True Discharge	Perturbated Discharge	Simulation Error %
TSUB	1.50		55.8		
		1.65		53.0	-5.0
		1.35		58.3	+4.4
TRTP	1.67		55.8		
		1.51		58.4	+4.6
		1.84		53.2	-4.6
EERC	2.0		55.8		
		1.0		54.9	-1.6
		3.0		56.2	+0.7
EPRC	1.0		55.8		
		2.0		61.4	+10.0
		0.5		54.8	-1.8
DEPR	0.20		55.8		
		0.22		55.2	-1.1
		0.18		56.5	+1.3
SACT	1.50		55.8		
		1.35		55.2	-1.1
		1.65		55.2	-1.1
SMAX	3.30		55.8		
		2.97		57.3	+2.7
		3.63		53.3	-4.5
FIMP*	2.0		55.8		
		1.0		55.8	0
		3.0		55.8	0

*fraction imperviousness of range land use

[1] $\pm$ 10% of "True" Parameter Value

TABLE 16

SENSITIVITY ANALYSIS RESULTS - SWMM,
STORM OF DECEMBER 3, 1978, MCDOWELL DITCH

Parameter	"True" Parameter Value	Perturbated Value	Correct Discharge	Perturbated Discharge	Simulation Error (%)
Infiltration Rate (Min)	0.52		74.1		
		0.57		74.1	0
		0.46		74.1	0
Infiltration Rate (Max)	3.00		74.1		
		3.30		74.1	0
		2.70		74.3	+0.2
Surface Storage Imperviousness	0.062		74.1		
		0.068		74.1	0
		0.055		74.1	0
Surface Storage Perviousness	0.184		74.1		
		0.165		74.1	0
		0.202		74.1	
Percent Imperviousness	20.0		74.1		
		22.0		79.3	+7.0
		18.0		68.5	-7.6
Manning's n-IMP	0.012		74.1		
		0.013		75.6	+2.0
		0.011		72.5	-2.2
Manning's n-PER	0.25		74.1		
		0.23		74.1	0
		0.27		74.1	0
Width of flow	3200		74.1		
		3520		75.9	+2.4
		2880		71.8	-3.1
Slope	0.001		74.1		
		0.002		79.9	+7.8
		0.005		73.2	-1.2

1] $\pm$ 10% of "true" parameter value

Varying the parameters used in Horton's infiltration equation had little effect on the simulated peak discharge. Percent imperviousness had a substantial effect on the peak discharge. Also, parameters used in Manning's equation produced a sensitive effect on peak discharge. The parameters which produce the most sensitive effects on peak discharge, in order of their importance, were: percent imperviousness, slope, width of overland flow, and Manning n-value.

Sensitivity analysis of the TENN-I model was performed in two parts. First, the NURF parameters used were perturbated $\pm$ 10 percent to evaluate their sensitivity on peak discharge. Second, the watershed characteristics were perturbated to evaluate their sensitivity on peak discharge. Table 17 tabulates these results.

The results of all sensitivity analyses performed must be qualified in the context of the $\pm$ 10 percent perturbations performed. It is widely known than an "excellent" measurement of discharge will be within $\pm$ 10 percent of the true value. Hence, a mathematical simulation which is within $\pm$ 10 percent of the measured value should be judged as excellent also. Therefore, if a $\pm$ 10 percent error in predicting model parameters generates an error in simulation greater than $\pm$ 10 percent, it follows that the model parameters must be predicted more accurate than $\pm$ 10 percent if there is any hope of making an "excellent" simulation--a task which is virtually impossible. In all of the sensitivity perturbations, only one simulation error was greater than $\pm$ 10 percent (KUP in TENN-I). Hence, all parameters in STORM and SWMM, and all but one in TENN-I appear not to have a very sensitive effect upon peak discharge. The sensitivity effect appears to be distributed among several parameters in each associated model.

TABLE T7

SENSITIVITY ANALYSIS RESULTS - TENN-I,
STORM OF DECEMBER 3, 1978, MCDOWELL DITCH

Parameter	"True" Parameter Value	Perturbated Value	Correct Discharge (cfs)	Perturbated Discharge (cfs)	Simulation Error %
KUP	0.677		42.2		
		0.744		48.7	+15.4
		0.69		40.3	- 4.5
KT1	0.390		42.2		
		0.440		44.7	+ 5.9
		0.340		44.9	- 6.4
KT2	1.469		42.2		
		1.616		44.0	+ 4.3
		1.322		44.3	+ 5.0
PF	5.0		42.2		
		6.0		41.8	- 0.9
		4.0		41.8	- 0.9
PI	20.0		42.2		
		18.0		41.8	- 0.9
		22.0		41.8	- 0.9

1] $\pm$ 10% of "true" parameter value

CHAPTER V

SUMMARY AND CONCLUSIONS

The objective of this study was to evaluate the simulations of the stormwater response of three stormwater models with observed rainfall-runoff events which occurred at Grissom Air Force Base. Given the rainfall and watershed characteristics of three study watersheds, each model was tested to determine its ability to accurately simulate stormwater hydrographs.

The three models chosen for this evaluation were The University of Tennessee Model (TENN-I), the Storage, Treatment, Overflow, and Runoff Model (STORM), and the Stormwater Management Model (SWMM). These models were considered a good cross-section of available stormwater models since they represented models from both parametric and deterministic approaches.

Six months of continuous rainfall data along with major runoff events within that time span were recorded at each study basin. Watershed characteristics needed by the three stormwater models as input data were obtained from storm sewer and topographical maps of Grissom Air Force Base. This data, for the most part, was considered reliable and accurate.

The simulated hydrographs from the three models, which have been presented within this study, show the results of this study. TENN-I simulated high rainfall volume storms with a fair degree of accuracy, but for multi-burst and low intensity, short duration storms the simulations were poor. STORM, for most cases, overpredicted peak discharges

and simulated the hydrographs poorly in terms of shape. This could be due to the fact that default values were used for several input parameters. Therefore, it would have been fortuitous for STORM to produce accurate simulations. SWMM also failed to simulate the storm events accurately. This model generally overpredicted peak discharges and simulated the shapes poorly. Since default values were also used for several input parameters, this may have caused the poor simulations.

The results of SWMM (Table 14, page 70) also show that a "coarse" discretization of a watershed simulates the stormwater hydrograph as well as a "fine" discretization. SWMM simulates the stormwater hydrograph by kinematic flow, therefore, two reasons may explain these results. First, since the storms used in the simulations had relatively low rainfall volumes, the pipes and gutters used in "fine" discretizations had no appreciable backwater effects. Second, the pipe and gutter network made a negligible contribution to the stormwater hydrograph in terms of direct stormwater volume and timing. This can be seen by comparing the lag modulus for overland flow to the lag modulus for channel flow. Overton and Meadows (1) defined lag modulus as the surface runoff response in terms of geometry and roughness for a unit rainfall excess intensity. The lag modulus for overland flow at Cline Ditch was computed to be 186 minutes while the lag modulus for pipe flow at Cline Ditch was computed to be 21 minutes. The pipe flows may attenuate the stormwater hydrograph peak, but should not make a major contribution relative to overland flow.

A sensitivity analysis was performed on each model to determine the parameter which should be accurately defined. The results of this

that all parameters in STORM and SWMM, and all but one in TENN-I, appear not to have a very sensitive effect upon peak discharge. The sensitivity effect appears to be distributed among several parameters in each associated model.

The decision of where to apply TENN-I, STORM, and SWMM should be based on the objectives the models were originally designed to fulfill. STORM, a parametric model, is able to give a general overview of the water quality and quantity parameters of direct stormwater runoff. SWMM, a deterministic model, is capable of breaking down the watershed into subcatchments, but should only be applied to areas where sheet surface runoff occurs. These models were basically designed for urban watersheds. TENN-I, a parametric model, treats the watershed response as a lumped system, therefore allowing only a general overview, but acts nonlinearily from storm to storm. It has also been shown the model may be applied to wide range of land uses.

CHAPTER VI

RECOMMENDATIONS

In order to use the three models presented in this study more effectively, it is necessary to calibrate the input parameters in the models. TENN-I has both an analysis and simulation phase, therefore, if rainfall-runoff data is available, the calibration of this mdoel would be relatively simple. Calibration of TENN-I is exemplified by the procedure used to develop a NURF diagram for the Cline Ditch watershed. This same procedure may be applied to any watershed.

To calibrate STORM or SWMM for a particular watershed, a trial and error procedure must be used. Unfortunately, this procedure may prove costly in terms of time and money. It would also be difficult to apply any physical significance to a calibrated set of coefficients, since an infinite set of coefficients would produce essentially the same fit. Therefore, this calibration procedure is not recommended unless the resources are available.

Another effective use of these models may be the combined use of parametric and deterministic models. The parametric model, such as TENN-I and STORM, is capable in assessing major land use effects on direct stormwater runoff. The deterministic model, such as SWMM, is capable of simulating runoff throughout the entire system. Therefore, the parametric model can be used to simulate hydrographs on a regional basis and the deterministic model can be used to simulate the transport mechanism at various outlets within the watershed. Each type of model may be used as feedback to the other in order to determine where additional study is needed.

Further study involving SWMM may be useful. As previously stated, a more detailed representation of the watershed may yield better results. In addition, further work to define input parameters used in STORM and SWMM rather than relying on default values given by the model builders may also improve simulations.

Any recommendations for the use of the stormwater simulation models depends upon the objectives of the study. The modeler must decide how well the system is to be represented. If a highly complex model, such as SWMM, is desired, then the risk of not representing the system will be minimized but the difficulty in obtaining a solution will increase. Much data, programming effort, time, and money will be required. These constraints should be considered when attempting to solve an hydrologic problem. Conversely, if a parametric model is chosen, such as TENN-I or STORM, the data requirements will be minimized but the risk of not representing the system will increase (1). The model should be tailored to the objectives of the study.

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APPENDIX

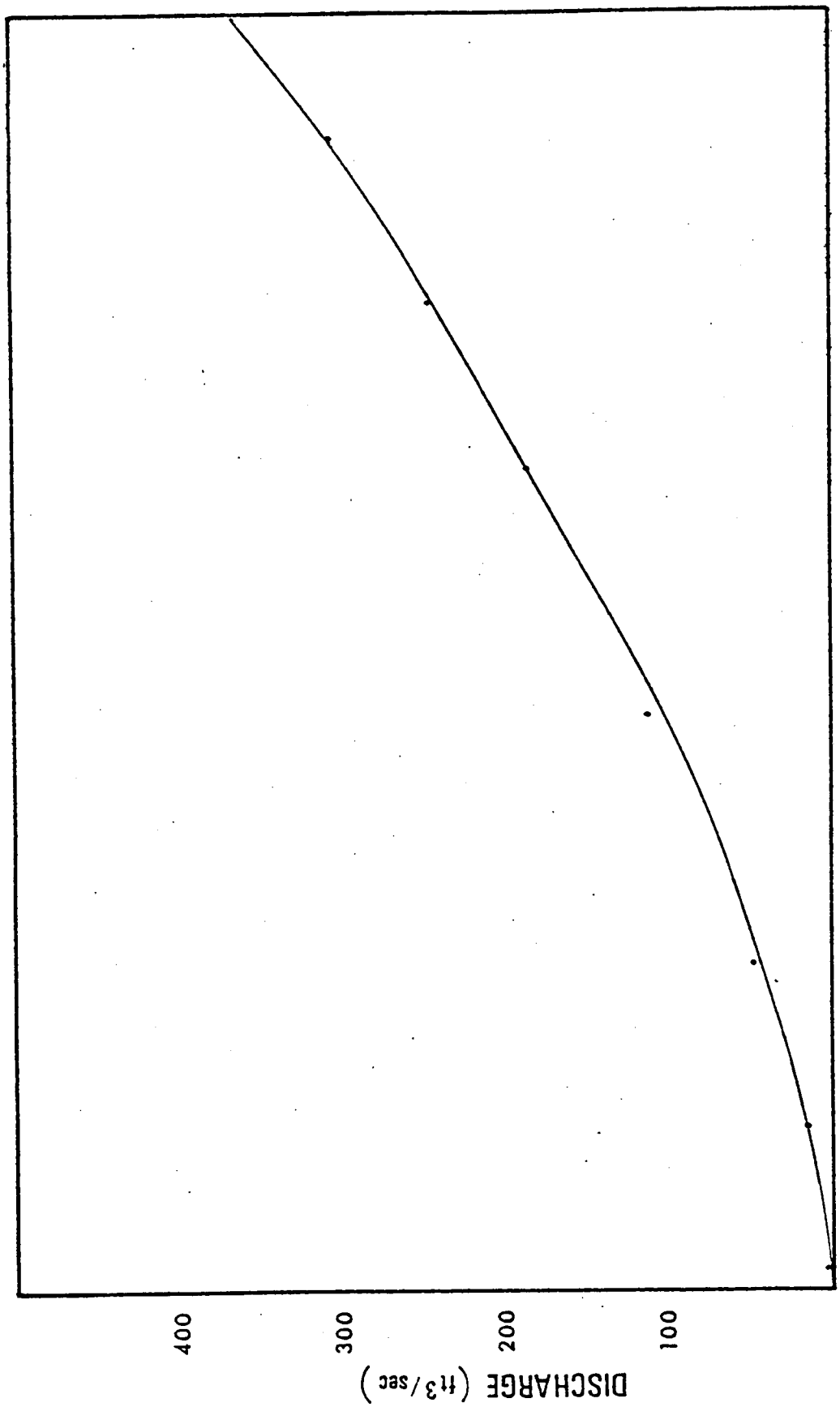

Figure 12. McDowell Ditch Rating Curve.

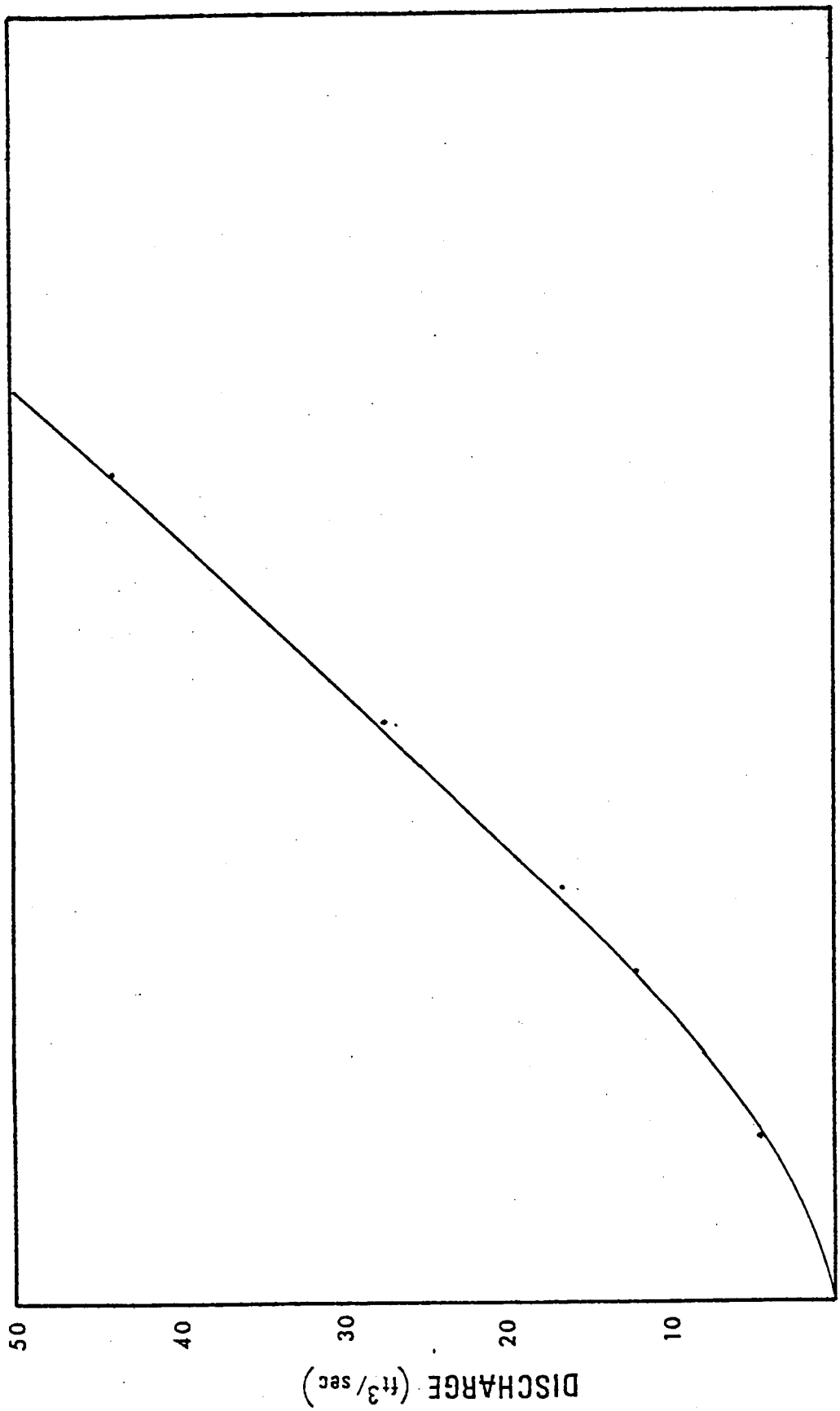

Figure 13. East Ditch Rating Curve.

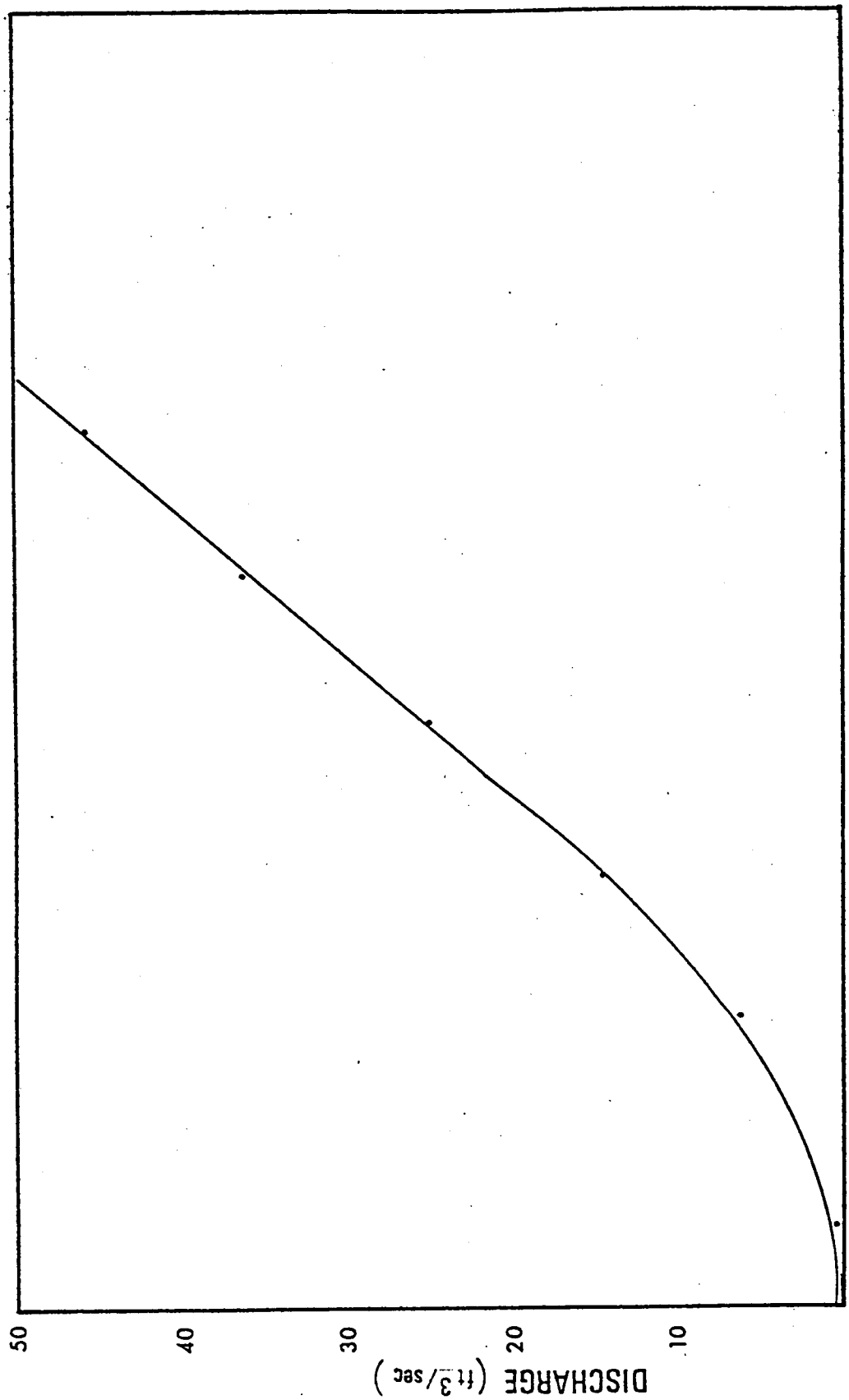

Figure 14. Cline Ditch Rating Curve.

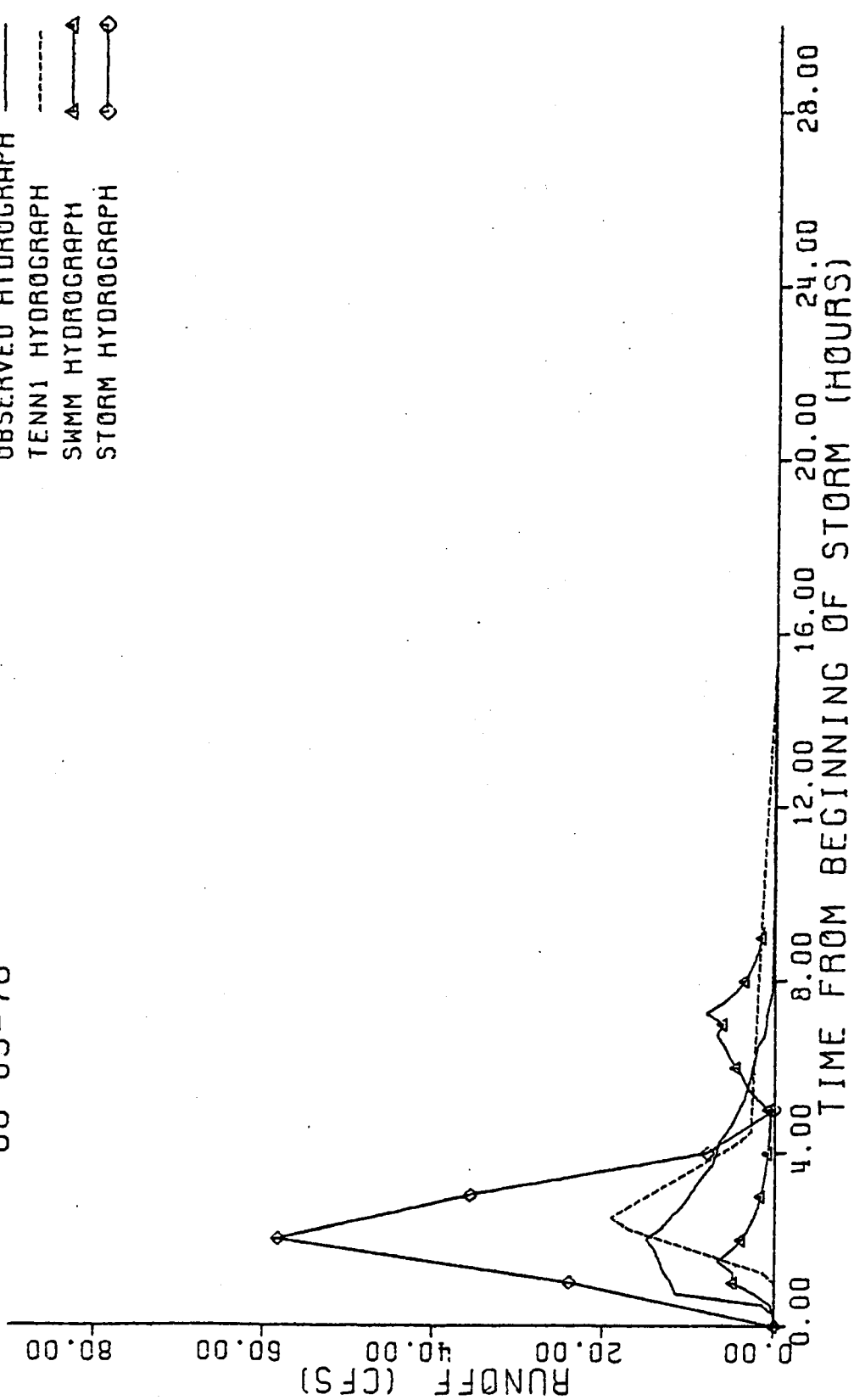

Figure 15. Hydrograph Simulations for the Storm of August 3, 1978, at McDowell Ditch.

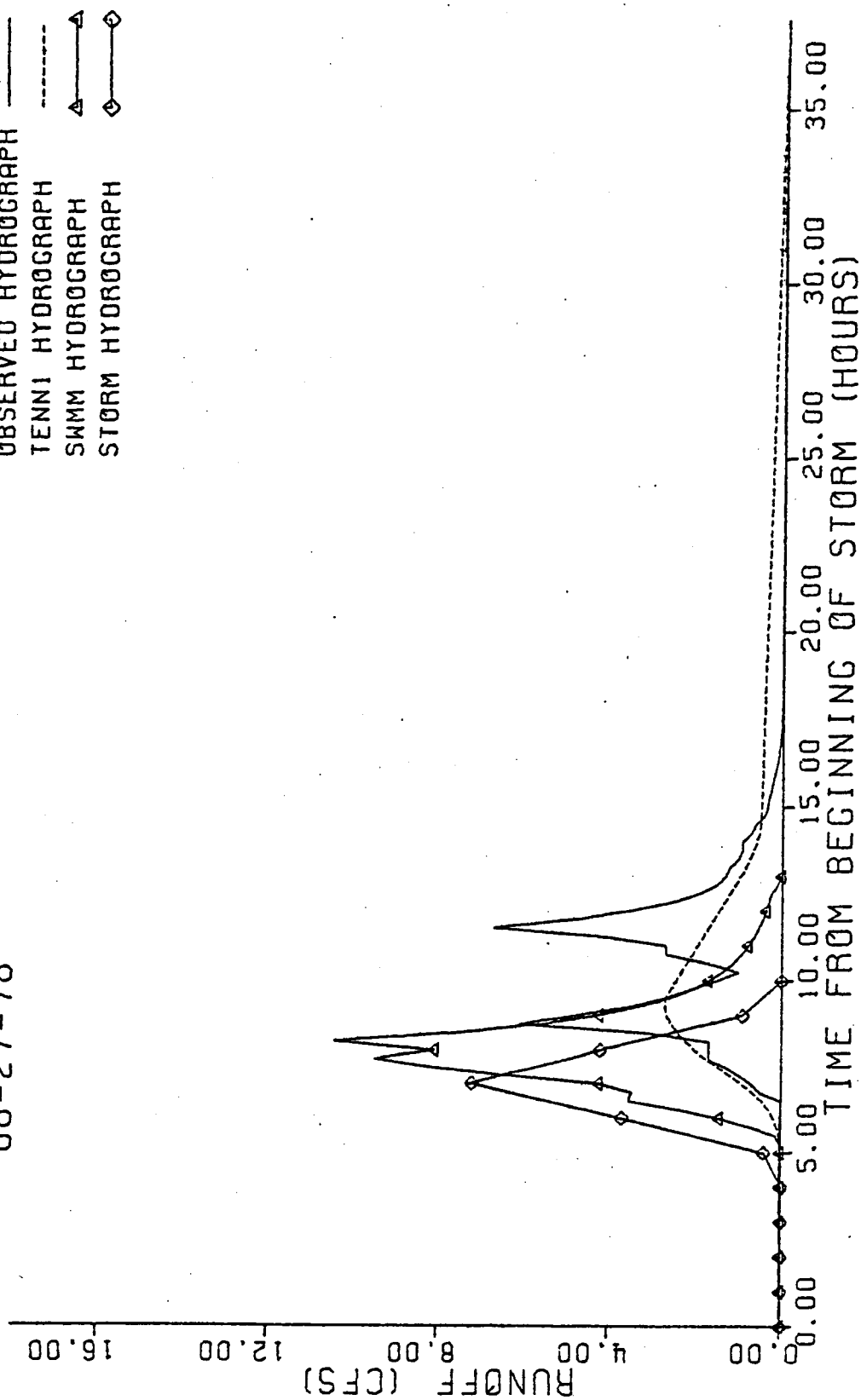

Figure 16. Hydrograph Simulations for the Storm of August 27, 1978, at McDowell Ditch.

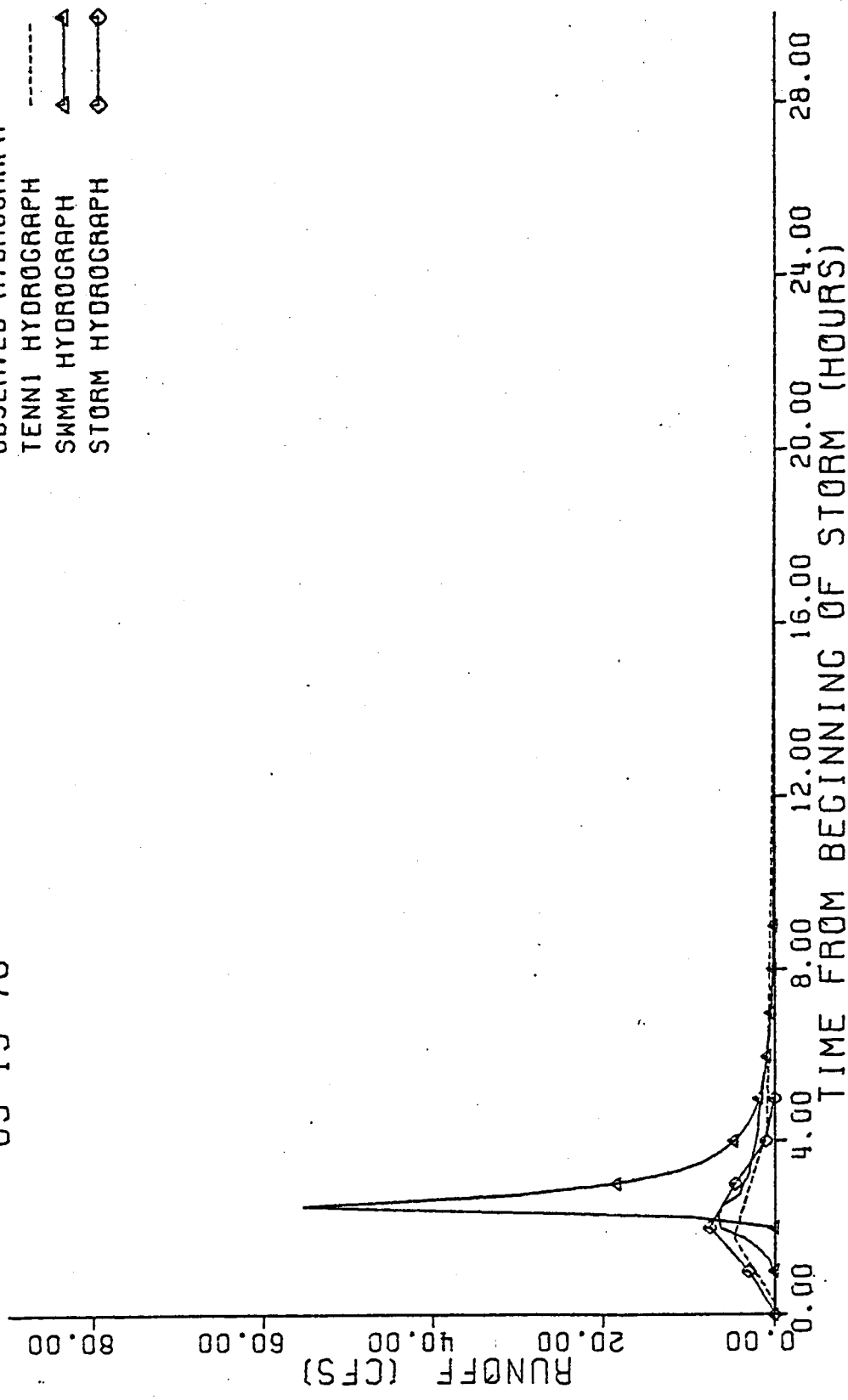

Figure 17. Hydrograph Simulations for the Storm of September 15, 1978, at McDowell Ditch.

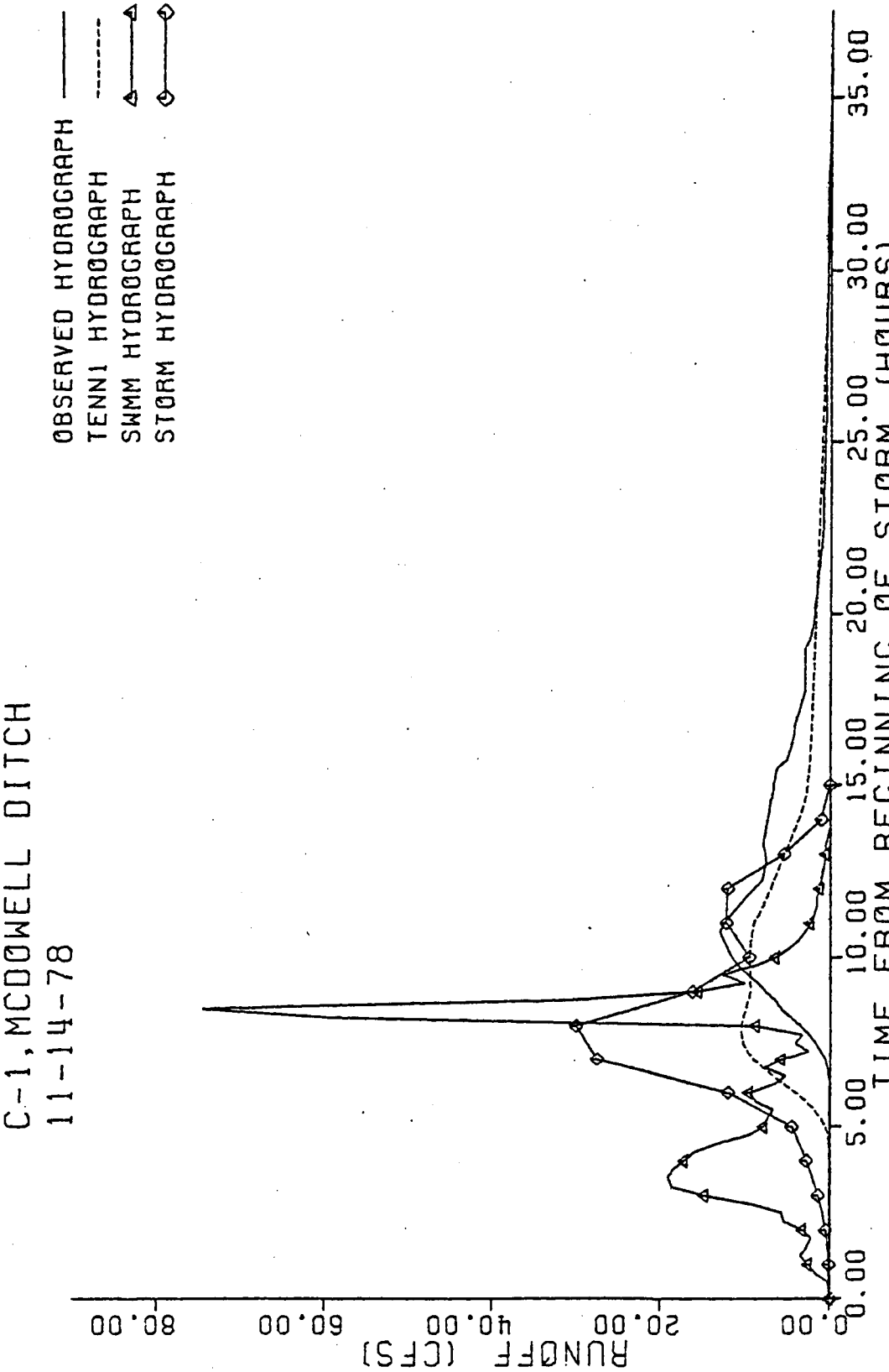

Figure 18. Hydrograph Simulations for the Storm of November 14, 1978, at McDowell Ditch.

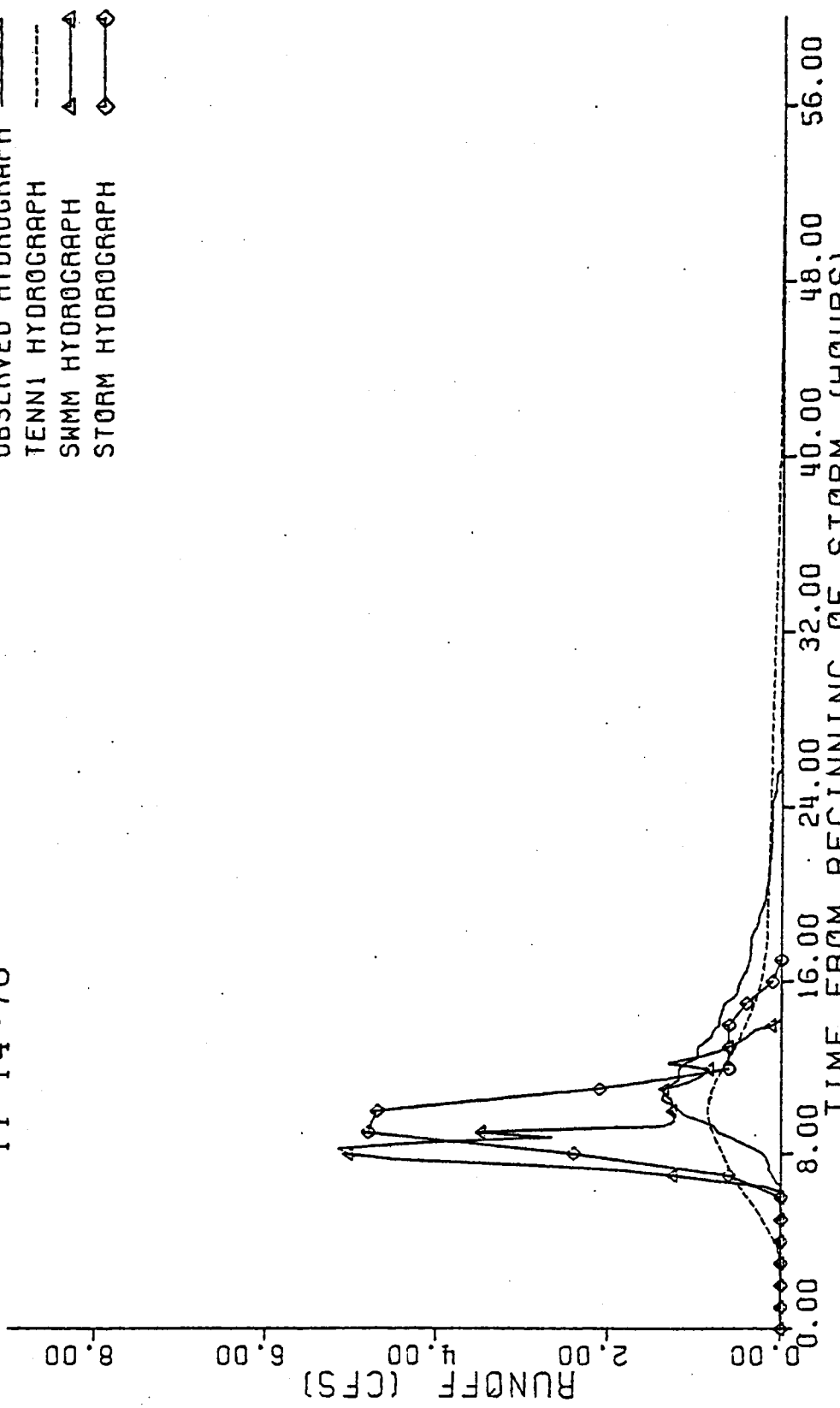

Figure 19. Hydrograph Simulations for the Storm of November 14, 1978, at East Ditch.

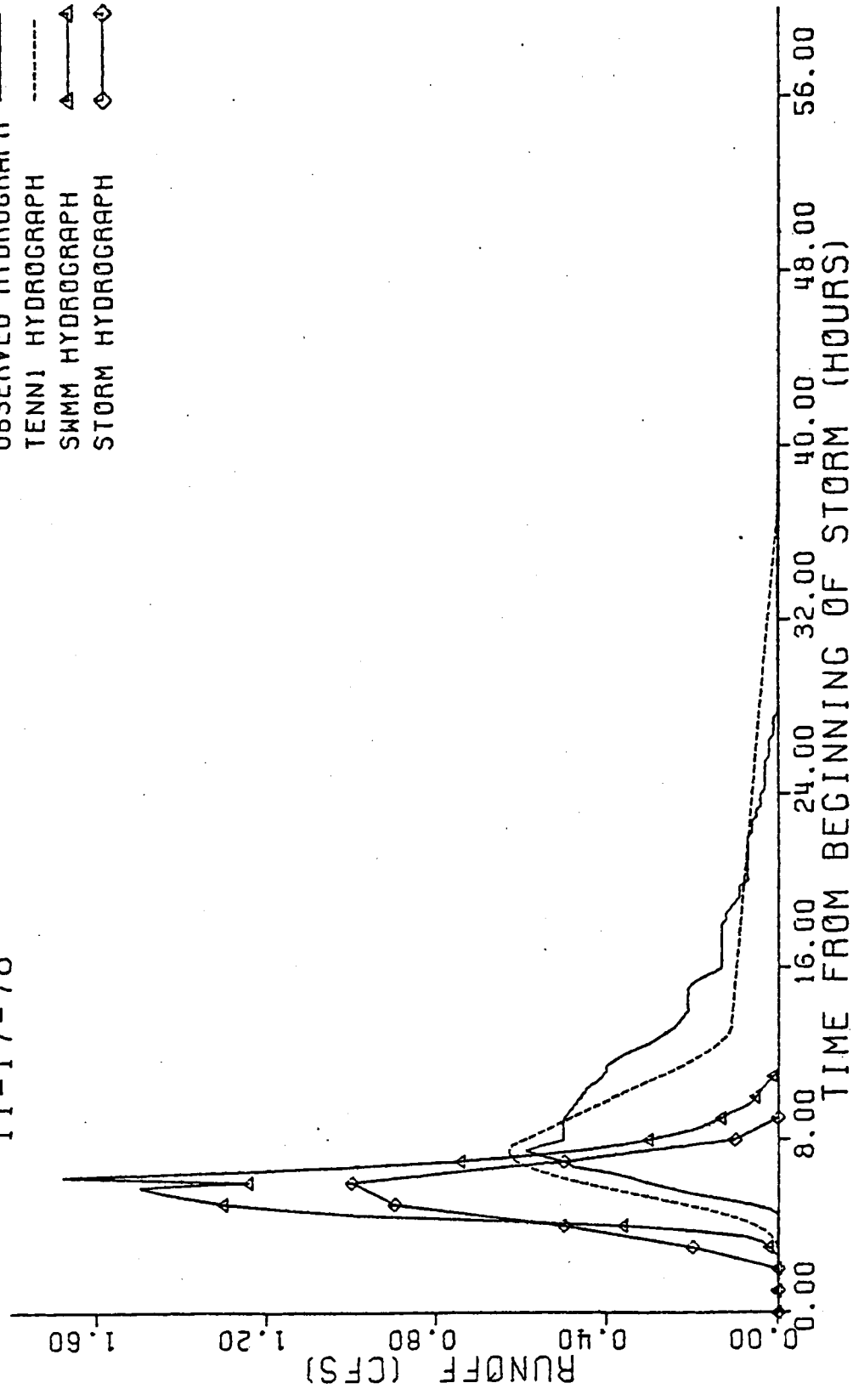

Figure 20. Hydrograph Simulations for the Storm of November 17, 1978, at East Ditch.

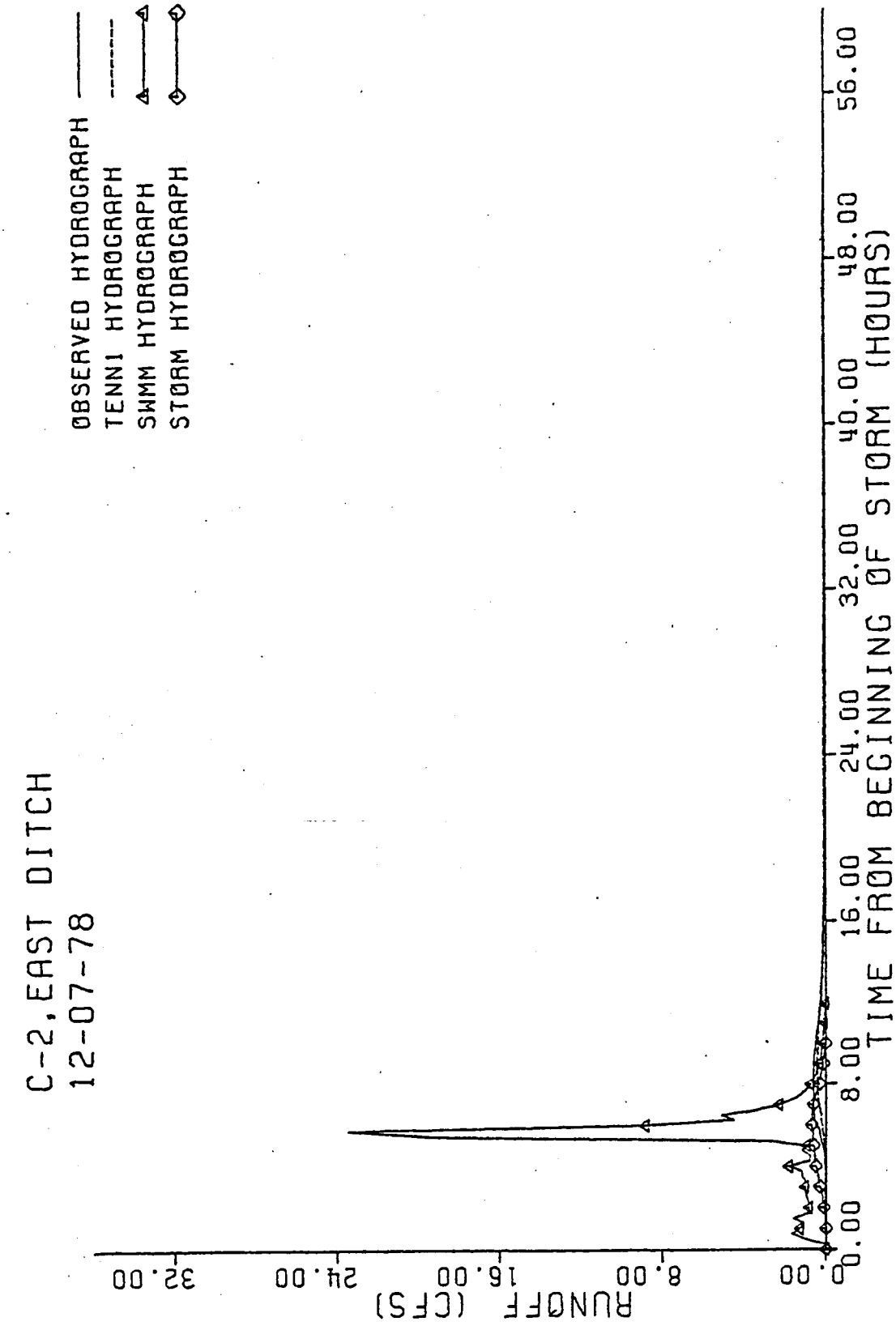

Figure 21. Hydrograph Simulations for the Storm of December 7, 1978, at East Ditch.

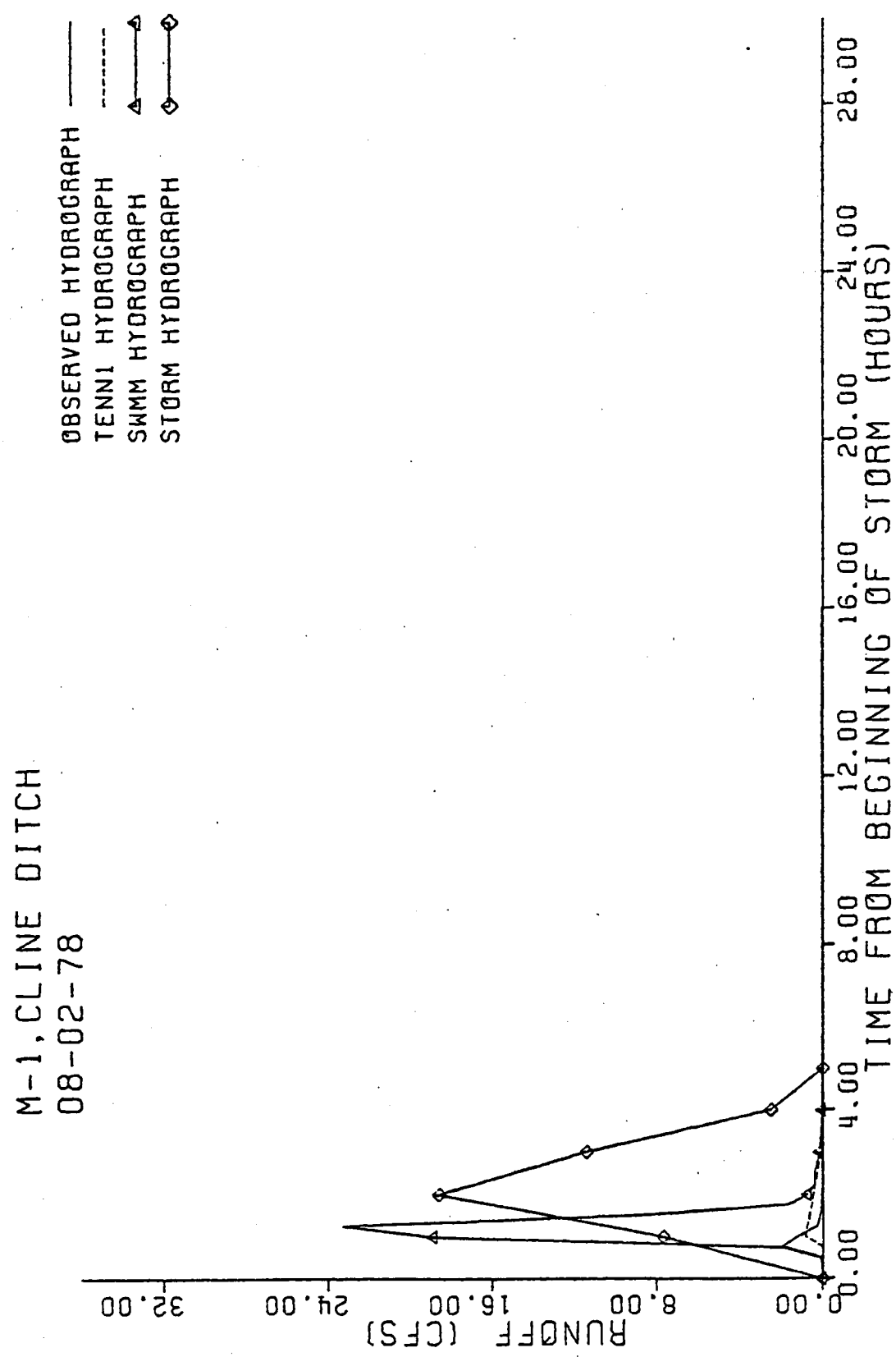

Figure 22. Hydrograph Simulations for the Storm of August 2, 1978, at Cline Ditch.

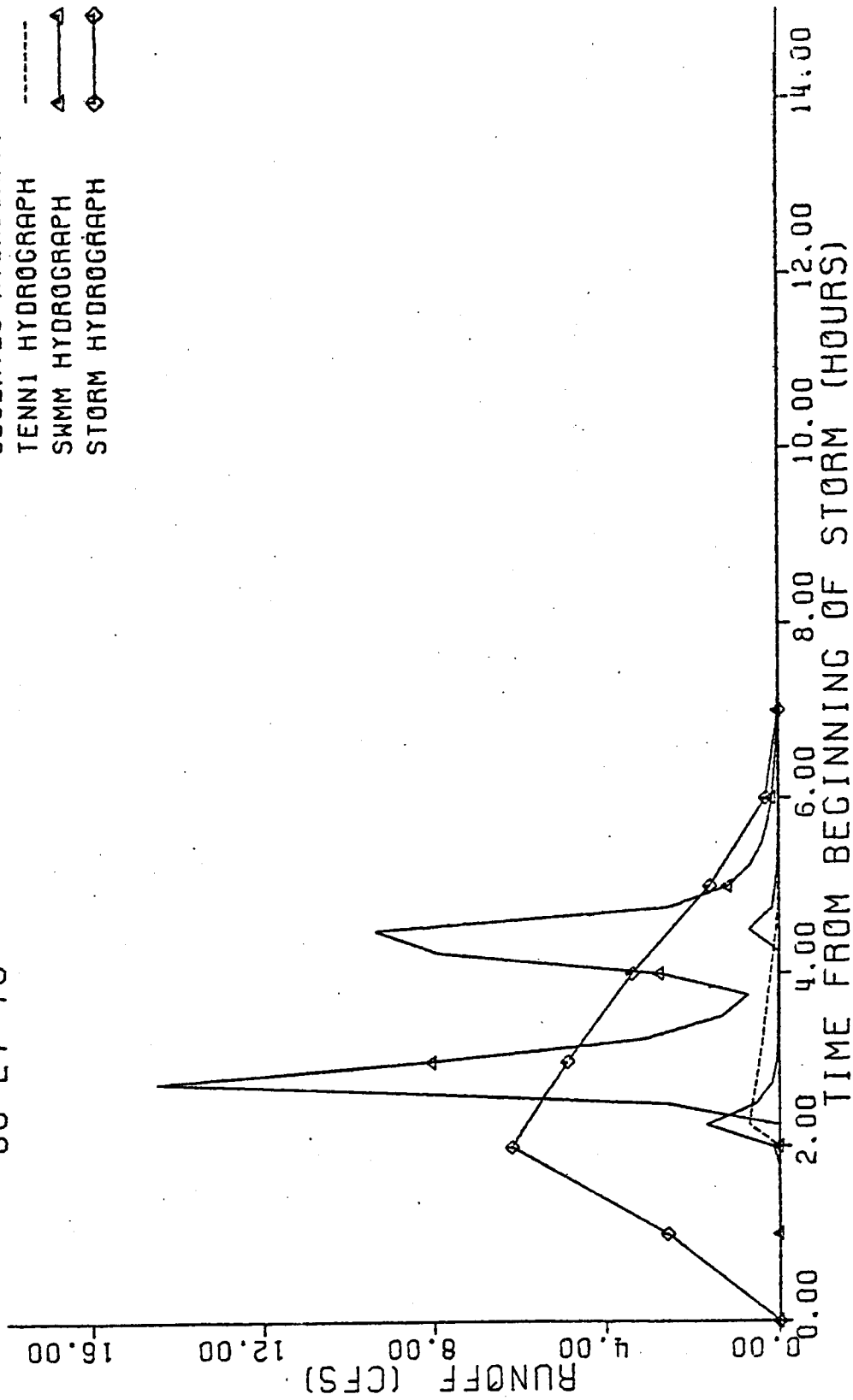

Figure 23. Hydrograph Simulations for the Storm of August 27, 1978, at Cline Ditch.

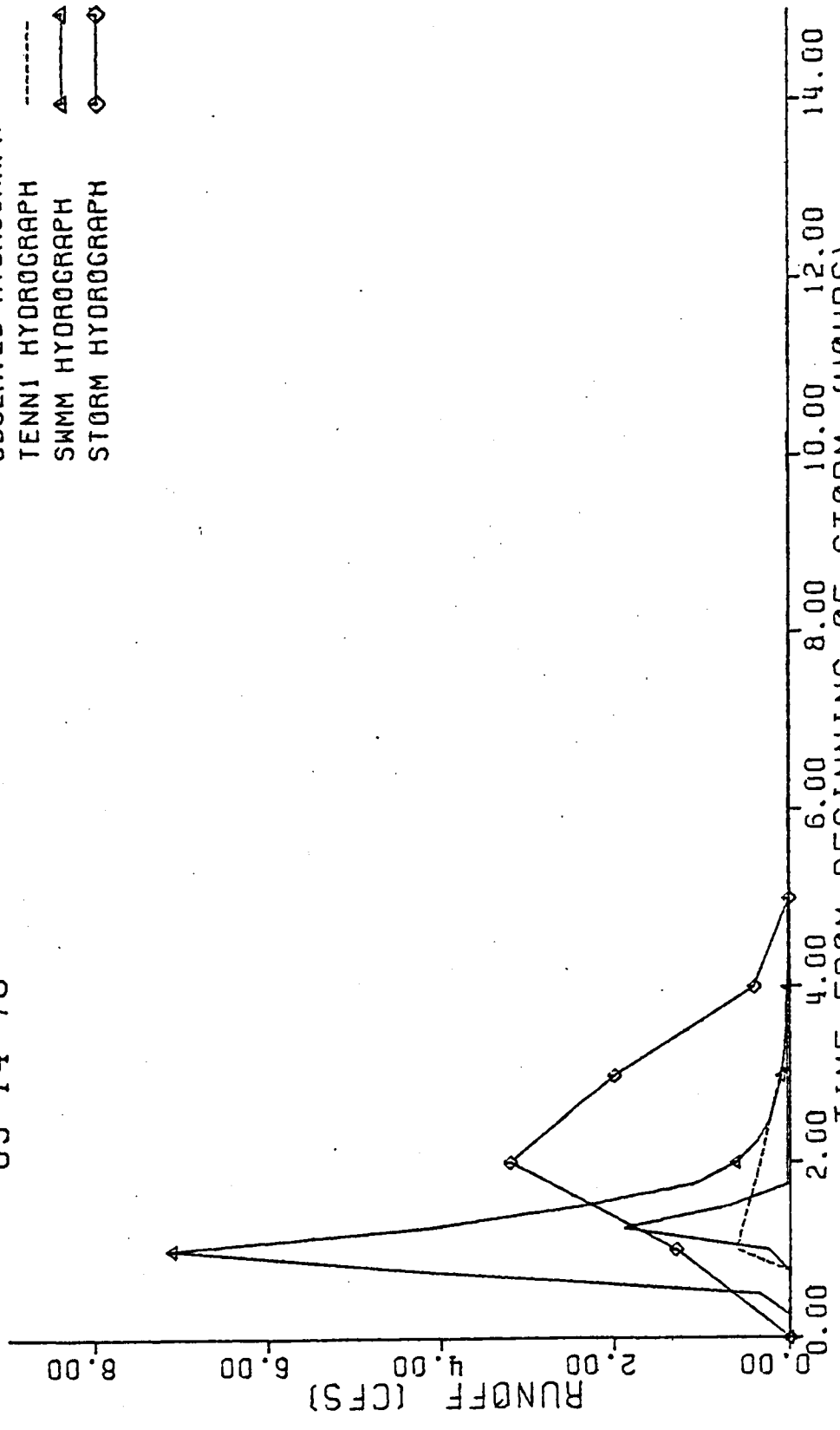

Figure 24. Hydrograph Simulations for the Storm of September 14, 1978, at Cline Ditch.

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