SIMULATION OF RUNOFF FROM AN AIR FORCE
BASE USING A PROGRAMMABLE CALCULATOR

James W. Luginbyhl, 2Lt, USAF

LSSR 60-81

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In this thesis the AFIT Runoff Model was developed and tested against other stormwater models and actual runoff data. The AFIT Runoff Model simulates the stormwater runoff from an Air Force Base. It was based on the Air Force Runoff Model (AFRUM), but it is used on a Texas Instruments TI-59 programmable calculator. The inputs to the model are land use characteristics of the watershed and an estimated Curve Number from the SCS Curve Number Model. The land use characteristics are percent impervious, percent forested, percent agricultural, percent denuded and surface area of the watershed.

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SIMULATION OF RUNOFF FROM AN AIR FORCE BASE USING A PROGRAMMABLE CALCULATOR

A Thesis

Presented to the Faculty of the School of Systems and Logistics

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In Partial Fulfillment of the Requirements for the

Degree of Master of Science in Engineering Management

By

James W. Luginbyhl, BSCE
Second Lieutenant, USAF

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This thesis, written by

Second Lieutenant James W. Luginbyhl

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Chapter 1

INTRODUCTION

Every year flash floods, produced by heavy rainfall, take lives and damage properties in many parts of the world. It has been estimated that in the twenty years preceding 1970, floods in Canada have caused a total of $100 million in damages with the cost to the federal government of $40 million (6:1). In addition to the direct costs associated with a flood are the loss of life, injury, inconvenience and other indirect losses. Properly designed drainage structures could greatly reduce the amount of damage produced by a flood.

A stormwater drainage structure conducts runoff from places where it is not wanted to the nearest acceptable discharge point or stores the runoff until it can be safely released (1:41). Examples of these drainage structures are culverts, storm sewers, drainage ditches and retention basins.

The most important parameter in the design of a stormwater drainage structure is an accurate estimate of the rate of flow of the stormwater runoff the structure will be expected to handle. Stormwater runoff is that portion of the precipitation which flows over the ground surface during, and for a short time after a storm (1:41).

There is a large cost penalty associated with the inaccurate estimation of the runoff rate. An estimate that is too low will result in a structure that cannot transport or store the amount of runoff that could result from an extreme rainfall event. Damages would result from flooding from an underdesigned structure. The American Water Works Association has reported that out of 293 dam failures in the United States and other countries since 1799, about twenty percent of the failures were due to underestimation of the volume of water the spillway would need to transport (6:1).

Costs resulting from overestimation of the rate of flow of runoff are reflected in the increased costs of construction for a larger structure. Drainage structures are expensive. Approximately $500 million is spent annually for highway culverts and small bridges in the United States (2:1). That figure represents fifteen percent of the total annual cost of interstate and state highways for construction and maintainance (2:1). Larger structures cost much more than smaller ones; therefore, each structure must be designed so that it can safely carry the maximum amount of runoff that is expected without having any excess capacity.

There are many models available for use when estimating the runoff from a watershed. Some are simple enough for hand calculations, while others are so complex that they require a computer. Although the computer models are generally much more accurate, the hand calculated methods are used more often (1:42). The more commonly used models are described in Chapter 2.

The Air Force Runoff Model, AFRUM, is a computerized model that has been developed to simulate runoff from an Air Force Base. However, this model has not been used extensively. Captain George W. Schlossnagle, the project officer for the development of AFRUM, believes that the reason the model has not been utilized to its fullest extent is a result of the difficulty involved in getting the program processed (14). Currently, an engineer who wants to use AFRUM must send all the necessary data to the Air Force Engineering and Services Center at Tyndall Air Force Base, Florida. Captain Schlossnagle said that in his opinion an engineer would rather use a hand calculated method, that is less accurate, because it takes less time and is easier than attempting to obtain results from a computerized model (14).

PURPOSE

The purpose of this thesis was to develop a storm-water runoff model, based on AFRUM, that could simulate run-off without the use of a large computer. The model that was developed is called the AFIT Runoff Model and operates on a programmable calculator. The AFIT Runoff Model eliminates the need for a large computer, while retaining the simulation methods found in AFRUM.

The AFIT Runoff Model was written for the Texas Instruments TI-59 was chosen because it is one of the most commonly used calculators that has the capacity to handle a program as complex as the AFIT Runoff Model. This calculator has up to 960 program steps available when no memories are used or up to 100 memories when only ten steps are used. The AFIT Runoff Model uses about 500 steps and 50 memories. The TI-59 has the ability to record the program on magnetic cards for easy reloading. Optional steps will be included for use when the calculator is locked on the PC-100A thermal printer, a TI-59 accessory that prints a hard copy.

The AFIT Runoff Model utilizes land use, soil type and hydrologic condition of the watershed to simulate runoff. The land use characteristics used are percent of the watershed that is impervious, percent forested, percent denuded and the surface drainage area in square miles. Soil type and hydrologic condition are input using the Curve Number of the Soil Conservation Service.

LIMITATIONS

The storms that can be used in the AFIT Runoff Model must be continuous, that is they cannot stop then start again. Another limitation to the model's use is the rate of rainfall cannot vary widely from one time period to another. These limitations must be made because of the method used in calculating the excess precipitation. Excess precipitation is the portion of the rain that contributes to the runoff. These two limitations would not be critical when using a design storm. The design storms, which the model was principally designed for, are not affected by these limitations. Most real storms can also be used with the AFIT Runoff Model; however, if a non-continuous storm or a storm whose rate of rainfall varies widely is encountered, AFRUM can be used instead.

VERIFICATION

The AFIT Runoff Model was tested by simulating the hydrographs for storms using the actual rainfall and runoff data. Hydrographs for the same storm events were also predicted by four other methods, the Environmental Protection Agency Stormwater Management Model (SWMM), the Army Corps of Engineers Urban Stormwater Runoff Model (STORM) and the Rational Method. The predicted hydrographs were statistically compared to the observed hydrograph and to each other. A goodness-of-fit test indicated the assumption of normal distributions was justified; therefore, parametric tests were used.

The rainfall and runoff data that were used came from three watersheds on Grissom Air Force Base in Indiana. The characteristics that were compared include: (1) peak runoff, (2) time to peak, and (3) total volume of the runoff.

HYPOTHESES TESTED

For each of the three characteristics, two hypotheses were tested. The first test compared the mean of the characteristics predicted by the AFIT Runoff Model to the mean of the characteristics of the observed hydrograph to determine if they were statistically the same.

The second hypothesis tested whether the parameters predicted by each of the five models were significatly different.

RESEARCH QUESTIONS

The specific research questions addressed in this thesis were:

1. Can the AFIT Runoff Model accurately predict the runoff from an Air Force base?

2. Is the AFIT Runoff Model better than other commonly used stormwater runoff models?

Chapter 2

BACKGROUND

The estimation of runoff has been a problem for engineers for many years. Because of the potential dangers involved and the high cost of drainage structures, many methods have been developed for predicting the amount of runoff those structures will have to handle. However, few of those models have been widely accepted. In the report "Estimating Runoff Rates From Ungaged Small Rural Watersheds", a research report sponsered by the American Association of State Highway Officials, the authors stated:

> One of the classical hydrologic problems yet unsolved is that of estimating floods of various frequencies from ungaged small rural watersheds.... Many design engineers and hydrologists consider present methods as inadequate for estimating peak flow rates from ungaged small rural drainage basins. As a result there is no generally accepted design method. The plethora of methods being used throughout the United States and within individual states have produced inconsistent estimates of magnitude of floods of various frequencies [2:1].

This chapter will explain how stormwater models are compared and then briefly describe some of the major methods that are currently being used for the estimation of stormwater runoff. The advantages and disadvantages of each method will be discussed along with how AFRUM compares to each method.

STORMWATER MODELS

The purpose of a stormwater model is to mathematically recreate a real world situation (7:129). Overton and Meadows classified modeling in three approaches: (1) deterministic, (2) parametric and (3) stochastic. In a deterministic system the output can be predicted for a given input, there is no element of chance involved. A stochastic model, on the other hand, has probability associated with the output. The difference between deterministic and parametric is a matter of degree. Overton and Meadows wrote:

> ... the parametric approach strives for the definition of the functional relations between hydrologic and geometric and land use characteristics of a catchment [11:159].

Stormwater models attempt to predict, at least, two major factors. The peak flow and the shape of the hydrograph are the most important parameters. Peak flow is the maximum rate of flow the drainage structure will be required to hold. For most purposes this value is all that is needed in the design of the drainage structure. The idea is that if the structure can handle the peak flow its capacity will not be exceeded. However, sometimes the peak will be maintained for only a short time; therefore, if some minor ponding can be allowed, the engineer may be able to design a smaller drainage structure. Before the engineer can make this kind of decision, or if the total volume of the runoff is needed the storm hydrograph is required.

HYDROGRAPH

A hydrograph is a chart plotting discharge against time (8:219). A typical hydrograph for a storm event is shown in Figure 1. The ordinate of a hydrograph is in cubic feet per second, or cubic meters per second and the abscissa is in units of time, hours for small basins or even days for large watersheds (17:112). The total volume of runoff is determined by the area under the curve.

The hydrograph usually has three general parts: (1) the rising limb or concentration curve, (2) the crest segment, and (3) the recession or falling limb (17:112). Each of these parts has certain inherent properties which, within limits, fix its shape (8:390). These sections are shown on Figure 1 along with the following three definitions. Lag time, is the time interval from the center of mass of the rainfall excess to the peak of the resulting hydrograph. Tp is the time to peak, which is the time interval from the start of rainfall excess, to the peak of the hydrograph. The final time interval shown on the hydrograph is the time of concentration, Tc, which is the time from the end of the rainfall excess to the point on the falling limb where the recession curve begins or the point of inflection (17:112).

RATIONAL METHOD

The most frequently used method for calculating peak runoff is the Rational Method (17:109). More than

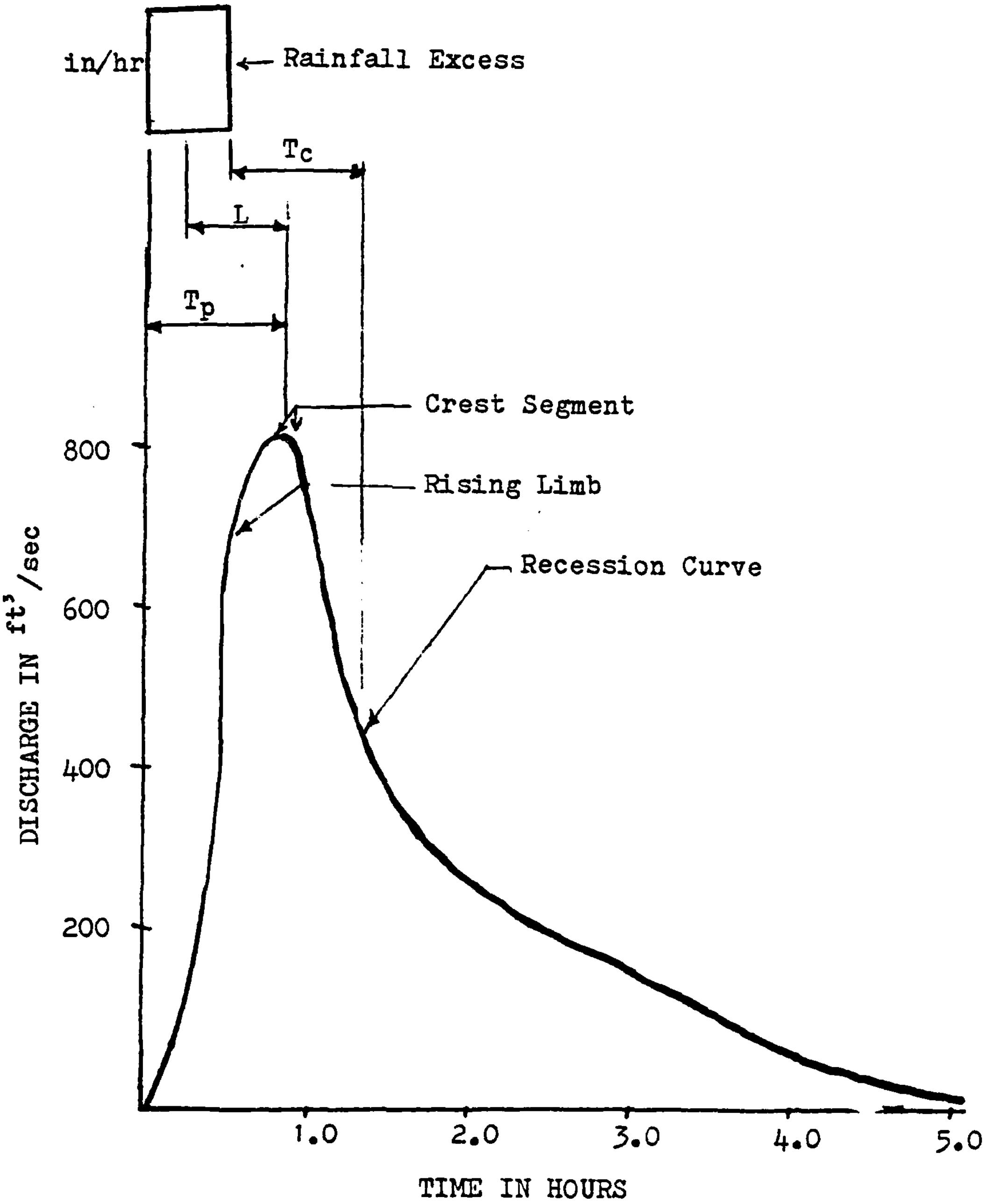

Figure 1. Typical Hydrograph (17:113)

ninety percent of the engineering offices throughout the United States answering a survey in 1956 on storm sewer design practice used the rational method (1:42).

At the first Engineering Research Conference on Urban Hydrology Research in 1965, it was pointed out that the rational method still is widely used. Because of its widespread use, the rational method is generally considered current practice (1:42).

The rational method is a simple model that relates runoff to rainfall intensity by the formula:

$$Q = CiA$$

in which Q is the peak runoff rate in cubic feet per second, C is a runoff coefficient that depends on the characteristics of the basin, i is the average rainfall intensity in inches per hour, and A is the drainage area in acres (1:42).

The critical factor in this formula is the runoff coefficient, C. It is usually estimated on the basis of previous experience with similar areas and watersheds, since it must represent many elements in runoff. It has to serve for the following modifications: (1) infiltration losses, (2) equilization of flow caused by surface detention, (3) equilization of flow caused by valley and channel storage and (4) the effects of the various physical factors of the watershed on flow (3:343).

The peak flow computed by the rational method is actually the peak of a equilateral triangle (17:119). Figure 2 shows the hydrograph predicted by the rational

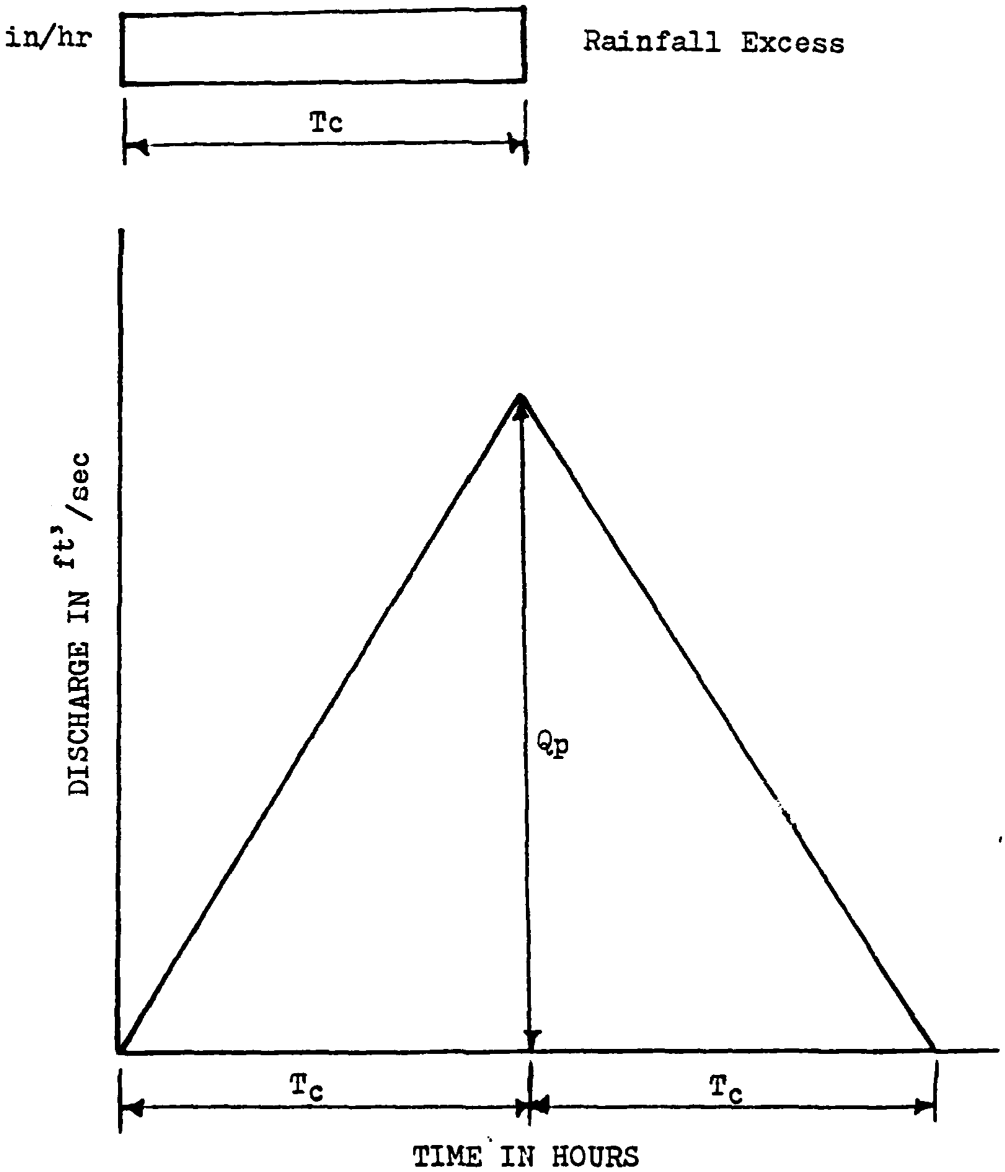

Figure 2. Hydrograph From Rational Method (17:119)

method. The volume predicted by this hydrograph compared to the volume predicted by the typical hydrograph shape in Figure 1 indicates that the rational method is a conservative design procedure (17:120).

The advantages of computing a hydrograph by the rational method hydrograph procedure are ease and simplicity. The obvious disadvantage is inaccuracy. However, where small watersheds, ten acres or less, are involved, this method will produce satisfactory results (17:160). Edgar Foster said:

> It is evident that successful use of this rational formula depends entirely upon the skill and judgement of the engineer in estimating suitable coefficients. It has been largely superseeded in estimation of flows for airport drainage by the more recent methods [3:343].

UNIT HYDROGRAPH METHOD

The next method for predicting the hydrograph from rainfall is the unit hydrograph. The method was first presented by Leroy K. Sherman in 932 and has been improved and supplemented many times since then (10:514). This concept has been called one of the most important contributions ever made to the science of hydrology (18:247). The unit hydrograph method is based on the idea that the physical characteristics of the basin, such as shape, size and slope are constant, Therefore, similarity in the shape of hydrographs can be expected from storms of similar rainfall characteristics (9:235).

A unit hydrograph is defined as the hydrograph produced by one inch of direct runoff from a storm of specified duration (9:238). The basic premise of the unit hydrograph method is that hydrograph produced by storms other than one inch of runoff are proportional in discharge throughout their length, and that when properly arranged with respect to time, the ordinates of several individual hydrographs can be added to give ordinates representing the total storm discharge (17:124).

The principles of this method are not rigorously true for all channels. Channel storage varies with stage; consquently, the unit hydrographs of large flows will differ from those of small flows (17:125). Commonly used unit hydrograph procedures have the tendency to compute peak flows that are higher than the actual runoff (17:147).

COMPUTERIZED MODELS

The computer has had a pronounced effect on storm-water modeling. Timothy Lazaro wrote:

> For some time now, the rational method and the unit hydrograph have been applied to estimate water quantity flows within the urban watershed. These procedures may be easily computed by hand. With the introduction of high speed analog and digital computers, a door has been opened into the use of formerly time-consuming mathematical methods. These methods allow significantly closer approximations of the physical processes of rainfall and runoff [7:154].

There are literally hundreds of computerized models that were developed to predict runoff. In the research report "Estimating Peak Runoff Rates From Small Ungaged Rural Watersheds", eighty-four sets of prediction equations (2:19), including the methods used for highway construction by thirty-one states were compared with differing results (2:11).

Some models are highly specific in their application. They are only valid for one type of watershed, one region or even one watershed. The two most widely used computer models are the Envoronmental Protection Agency Stormwater Management Model, SWMM, and the United States Army Corps of Engineers Urban Stormwater Runoff Model, STORM.

COMPARISON OF STORMWATER MODELS

A two year study was performed at Grissom Air Force Base, Indiana to determine the effectiveness of the Air Force Runoff Model (15:1). In the study actual hydrographs from storms were determined for three watersheds. Also the hydrographs for those storms were simulated using AFRUM, SWMM, and STORM models. The predicted hydrographs from each model were compared with each other by how close they were to the observed hydrograph for peak discharge, time to peak, the volume of direct runoff and the shape of the hydrograph.

In comparing the three models it should be understood that every model is developed to fulfill specific objectives. These three models were not developed to fulfill the same objectives. Their structures and application procedures are different (12:139). Table 1 shows a comparison of the structural differences of the three models.

TABLE 1

SUMMARY OF MODEL COMPARISONS (12:157)

Model	Structure	Runoff Response	Parameter Prediction
AFRUM	Parametric	Nonlinear	Yes
STORM	Deterministic	Nonlinear	Default Values
SWMM	Parametric	Linear	Default Values

Because the STORM users manual did not include a prediction method for the model parameters, all input parameters were estimated by considering the examples in the manual. Error in these estimates might have contributed to the simulated peak discharge and volumes being higher than on the observed hydrographs and the shapes of the simulated hydrographs being only fair (12:129).

SWMM is a deterministic model; therefore, when input parameters are known with a high degree of cetainty, storm simulations should be modeled accurately. Unfortunately, the input parameters were not known. Because default values suggested in the users manual were used, the SWMM simulations were very poor (12:130).

AFRUM appeared to have done the best job of predicting the storm hydrographs in the Grissom study. AFRUM simulated high rainfall volume storms with a fair degree of accuracy but had trouble with the multi-burst and low intensity, short duration storms (12:137). It should be noted that for design purposes it is the storms that AFRUM simulated well, the high volume storms, that are used. These storms produce the highest peak flows, and if a structure will fail, it will do so during this type of storm.

Chapter 3

DESCRIPTION OF AFRUM

The Air Force Runoff Model is a parametric stormwater model that considers land use, soil type and hydrologic condition to predict runoff. Besides having a simulation phase, AFRUM also contains an analysis phase (12:139). AFRUM simulates direct runoff volume and rates using the United States Soil Conservation Service's Curve Number Model.

The parameters supplied as input to the model concerning land use are the percentage of the watershed that is impervious, percent denuded, percent forested and the total area of the drainage basin. All loses except evapotranspiration are lumped into a single initial abstraction.

DEVELOPMENT OF AFRUM

AFRUM was developed in the course of analysing 410 storms observed on 36 watersheds. The watersheds in the studies included Air Force Bases, agricultural lands, urban areas, forested and areas that were being strip mined for coal (13:2).

The model resulted from three separate studies that evaluated the effects of specialized land use on stormwater runoff. The United States Department of Energy was studying the effect of coal strip mining on runoff, while the Department of Interior Office of Water Resources Technology was investigating the effects of urbanization. The last study contributing to AFRUM was conducted by the Air Force on the runoff from Air Force bases (13:1).

AFRUM was developed for the Air Force by the Department of Civil Engineering at the University of Tennessee, Knoxville. The model was extensively modified by the Air Force Engineering and Services Center (13:2).

DOUBLE TRIANGLE MODEL

AFRUM uses a Unit Response Function, URF, that was coupled with the Curve Number Model to form the TVA double triangle model. The quadrilateral URF that was formed, is based on the concept that intial response from a watershed comes from the riparian areas, or areas in and near the water channel, and as the other areas become saturated they too begin to contribute to runoff in the form of a delayed response.

It is assumed that the two responses can be represented by two separate triangular response functions. Both triangles begin at time zero but have different slopes. When the two triangles are added together, they form the quadrilateral unit response function.

The quadrilateral unit response function is shown in Figure 3. The symbols being used in the Figure are listed below.

I is the precipitation excess intensity in inches per hour. I = 1/DT

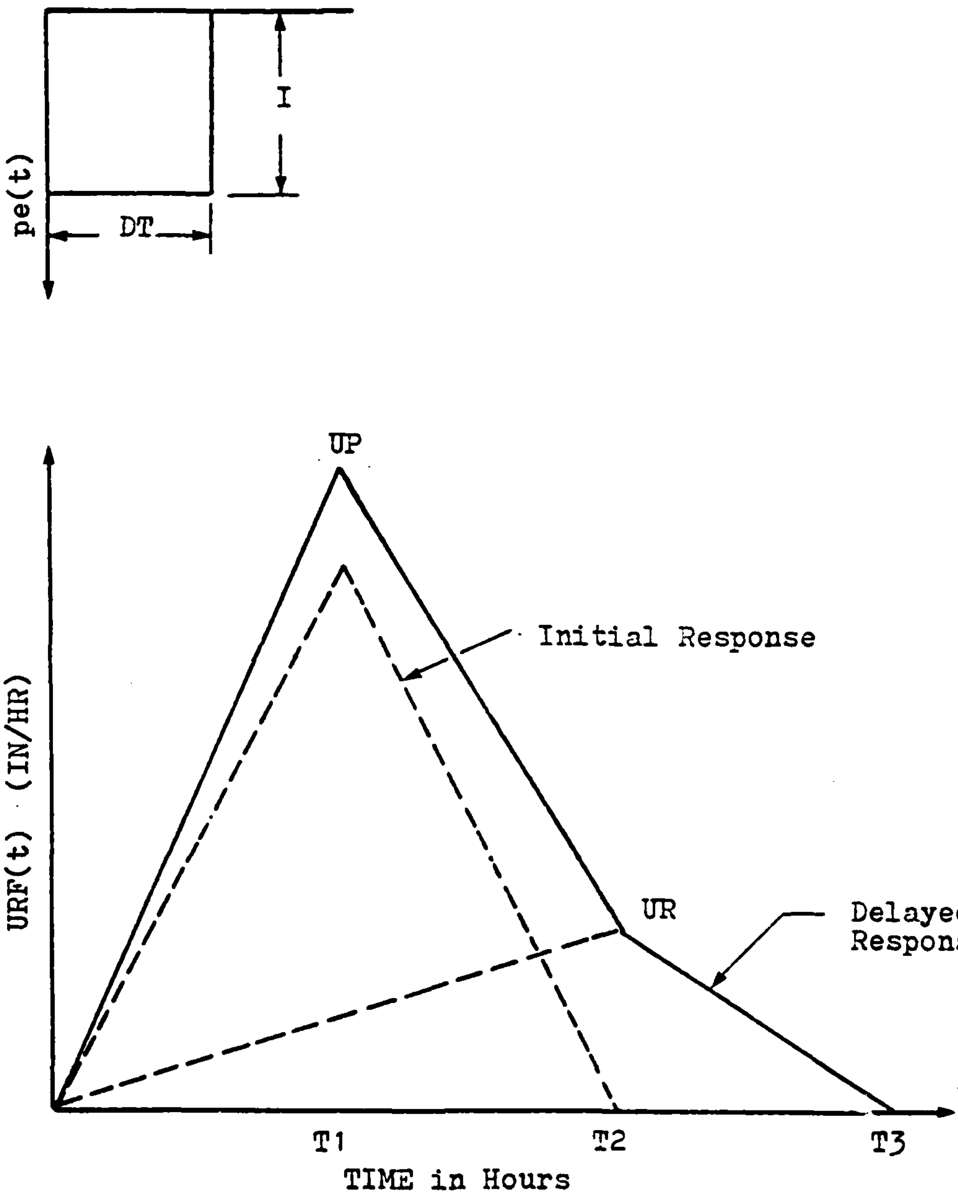

Figure 3. Double Triangle Model (13:11).

DT is the time interval used in abstracting rainfall and discharge records in hours.

UP is the peak of the unit response function.

UR is the peak of the delayed response function.

T1 is the time to peak of the initial response.

T2 is the time base of the initial response and equal to the time to peak of the delayed response.

T3 is the time to the end of the delayed response.

pe(t) is the precipitation excess as a function of time, t, in inches per hour.

URF(t) is the unit response function ordinate as a function of time in inches per hour.

In deriving the URF, the peak of the delayed response was assumed to occur at the end of the initial response, and the time bases of both responses and the time to peak of the initial response are integer multiples of DT. The relative volumes in the initial and delayed responses and the relative magnitudes of the peaks of the individual responses were not fixed.

The double triangle URF is defined by the five parameters UP, UR, T1, T2, and T3. T3 is found by the equation:

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

where NOBS is the number of storm hydrograph ordinates in multiples of DT and NRAIN is the number of rainfall increments in multiples of DT. By maintaining a unit volume, UR is calculated from:

$$UR = (2 - (UP * T2) / (T3 - T1))$$

Therefore, defining a storm URF involves determining vaues of UP, T1 and T2 (13:13).

The parameters UP, T1 and T2 are optimized using the pattern search technique. Since all five parameters describing the model are allowed to vary from storm to storm, the model is considered nonlinear (14:4).

NORMALIZED UNIT RESPONSE FUNCTION

The variability of the URF between storms within a watershed was explained by normalizing the time and discharge scale by the associated URF lag time, TL, where TL is equal to the time lapse between occurences of fifty percent of the rainfall excess block and fifty percent of the URF volume. The Normalized URF's are called NURF's (13:13).

The NURF for each land category, strip mined, 100 percent forested, urban without extensive storm sewers, urban with storm sewers, and agricultural land, has been determined empirically. The NURF's for each land use category is shown in Figure 4 along with the NURF observed for sheet surface runoff from a plane to provide a reference (13:13).

All of the NURF's in Figure 4 can be placed in the context of an initial response, IR, and delayed response, DR. The highest IR is from sheet surface runoff, and the lowest IR would be from a completely forested watershed. The initial

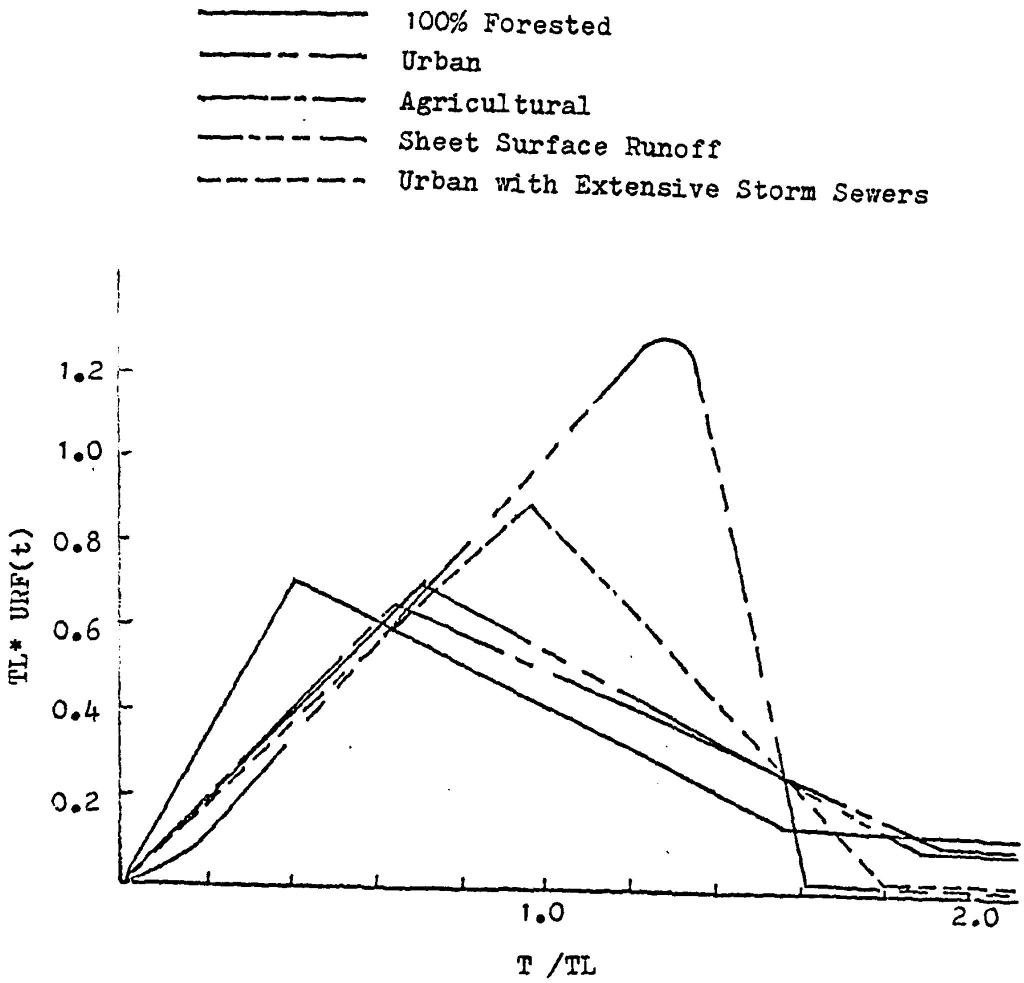

Figure 4. NURF for Various Land Use Conditions (13:14)

and delayed responses are shown in Table 2 (11:74). The effect that storm sewers have on runoff is demonstrated in Table 2. Urban areas with extensive storm sewers have a higher percentage of the runoff in the initial response, which also means higher and quicker peak flows.

The AFIT Runoff Model was based on the Air Force Runoff Model; therefore, the theories discussed in this chapter apply to the AFIT Runoff Model as well.

TABLE 2

INITIAL AND DELAYED RESPONSE (13:78)

Land Use	IR (Percent)	DR (Percent)
Sheet Surface Runoff	97	3
Urban Storm Sewers	86	14
Urban	65	35
Agricultural	62	38
Contour Strip Mining	48	52
Forested	46	54

Chapter 4

METHODOLOGY

The AFIT Runoff Model is based on the Air Force Runoff Model. AFRUM is a highly complex program consisting of ten subroutines and over 1200 lines of input. The AFIT Runoff Model is a simpler model developed for the TI-59 calculator. The TI-59 is a programmable calculator that can hold a maximum of 960 steps. A program step is much smaller than a line of computer input. One line from AFRUM could require four to forty program steps to achieve the same results.

Therefore, it is evident that some assumptions and simplifications were needed to write the AFIT Runoff Model. The assumptions made about the excess precipitation resulted in the limitations described in Chapter 1, continuous and fairly uniform rainfall.

REDUCING THE MODEL

Even with these assumptions, the model is too long to fit on a programmable calculator. Therefore, it must be reduced by performing some of the functions manually. Several one time calculations near the beginning and middle of the program were selected for hand calculation, because they were simple and quick. If they were calculated using the program, they could use as much as twenty percent of the program steps.

In the model, there are several sets of empirical constants that depend on the characteristics of the watershed. These constants vary depending on whether the drainage area is: (1) urban or rural, (2) if it is agricultural, forested, or denuded and (3) if the watershed contains storm sewers. Rather than having these constants in the program, they will be set up in tables for the user to enter into the program as input.

PROGRAMMING THE CALCULATOR

A calculator program must be written differently than a program for a computer. In a computer program, all the values in each array can be calculated and stored at one time, then recalled as needed. A calculator program, on the other hand, cannot do this because of the limited storage space. The calculator program must go completely through to the end in one pass. Each time the calculator makes a pass through the program it will calculate another ordinate on the hydrograph.

The program was written following the procedures set forth in the TI59 user's manual. The program is divided into two major sections. The first section calculates the excess precipitation from the rainfall data. Some intermediate values are then calculated by hand and entered back into the program. The second section of the program calculates the hydrograph ordinates so they can be plotted with multiples of DT, the time interval between ordinates, along the abscissa.

TESTING THE MODEL

To determine the accuracy and range of application of the AFIT Runoff Model, the model was tested against the observed hydrographs and hydrographs predicted by four other models. Seventeen hydrographs from three watersheds at Grissom Air Force Base were used for the comparisons. The rainfall and runoff data for the hydrographs were collected in 1978 and 1979. The three hydrograph parameters used in the tests were peak flow, time to peak, and volume.

The first set of tests compared the hydrographs predicted by the AFIT Runoff Model to the observed hydrographs. The test used for the comparison was the matched pair t-test. This test assumes both populations to be distributed normally. The hypothesis tested whether the mean of the observed population was significantly different from the mean of the predicted population.

$$H_0: \mu_{AFIT} = \mu_{observed}$$

$$H_1: \mu_{AFIT} \neq \mu_{observed}$$

The second set of tests compared the hydrograph parameters predicted by the five models, the AFIT Runoff Model, AFRUM, SWMM, STORM, and the Rational Method. The mean of the parameters predicted by these models were compared to each other and to the mean of the population of the observed parameters using a randomized complete block design. This test also assumes normality. It compares all the means together at one time in the following manner:

H_0: $\mu_{AFIT} = \mu_{AFRUM} = \mu_{RM} = \mu_{STORM} = \mu_{SWMM} = \mu_{observed}$

H_1: At least one inequality

If the null hypothesis, H_0, was rejected, another test was performed. Duncan's multiple-range test determined which of the six means were significantly different and which ones were not.

The final statistical test used was the Lilliefors goodness-of-fit test. This test determined that the assumption of normality made in the other statistical tests was justified.

Chapter 5

USING THE AFIT RUNOFF MODEL

This chapter explains the procedures and hand calculations that are necessary to use the AFIT Runoff Model. This chapter can serve as a user's guide for individuals who wish to use the model to design a stormwater drainage structure for an Air Force base.

Before these procedures can be used, the program listed in Appendix A must be loaded into the calculator. To load the program, the calculator is placed in the learn mode by pressing the LRN key. After the program has been entered, the LRN key must be pressed again to exit the learn mode.

EXCESS PRECIPITATION

When rain strikes the earth's surface it can either infiltrate into the soil, evaporate back to the atmosphere, be retained in surface storage or flow over the surface. Excess precipitation is the portion of the rainfall that flows over the surface to become runoff. The first part of the AFIT Runoff Model's program was designed to calculate the excess precipitation based on the Soil Conservation Service Curve Number Model. Soils are divided into four hydrologic soils groups: A, B, C, and D. Group A soils have a high infiltration rate even when throughly wet.

When thoroughly wet, group B soils have a moderate infiltration rate, group C soils have a slow infiltration, group D soils have a very slow infiltration rate (16:181). More than 9,000 soils and their hydrologic group are listed in reference 16.

Rain that infiltrated into the soil from a previous storm and is still present in the soil is called antecedent moisture. In the Curve Number Model, Antecedent Moisture Condition II is an average condition while Condition III indicates that soils in the watershed are practically saturated from antecedent rain. Condition III has the highest runoff potential.

The curve number estimated from Table 3 was used in the AFIT Runoff Model to determine a surface retention factor by the formula:

$$S = (1000.0 / CN) - 10.0$$

where S = the effective surface retention

CN = the curve number

This factor was hand calculated and entered into the program by pressing the user key labled B.

LAG TIME

Lag time, TL, for a storm was simulated in the AFIT Runoff Model using the concept that varies inversely with the rainfall excess intensity. The lag time was calculated within the program using a factor called the lag modulus. The lag modulus, U, is empirically related to

TABLE 3

RUNOFF CURVE NUMBERS FOR ANTECEDENT MOISTURE CONDITIONS II AND III

Zoning Classification	II Hydrologic Soil A	II B	II C	II D	III Hydrologic Soil A	III B	III C	III D
Business, Industrial or Commercial	82	88	90	91	92	95	96	97
Apartment Houses	78	85	88	90	90	94	95	96
Schools	68	78	84	87	84	90	93	95
Urban Residential Lots ± 10,000 ft²	65	77	83	86	82	89	93	94
Suburban Residential Lots ± 12,000 ft²	62	76	82	85	80	89	92	94
Suburban Residential Lots ± 17,000 ft²	60	74	81	84	78	88	92	93
Suburban Residential	58	72	80	84	77	86	91	93
Parks and Cemetaries	55	71	79	83	74	86	91	93
Unimproved Areas	53	70	78	92	73	85	90	92
Lawns	45	65	75	80	66	82	88	91
Woods	36	60	73	79	*	*	*	*
Meadow (permanent)	30	58	71	78	*	*	*	*
Pasture or Range	49	69	79	84	*	*	*	*

*Data Unavailable

watershed characteristics.

For rural watersheds, the lag modulus is calculated by the formula:

$$U = 0.060 * SQMI + 0.0203 * PF + 1.16$$

where SQMI = the area of the watershed in square miles

PF = the percent of the watershed that is forested

The lag modulus for urban watersheds can be calculated from the formula:

$$U = 3.24 * (SQMI/PI)^{0.6}$$

where PI = the percent of the watershed that is impervious

After calculating the lag modulus, it was entered into the program through user key A.

The time interval that was used to abstract the hydrograph, DT, was entered through the user key C. DT can be any time interval that is desired, generally it is in tenths of a hour or fourths of an hour.

The cumulative rainfall for each DT was entered, one at a time into user key D until all of the rainfall data were entered. Lag time of the watershed was then calculated by the program after the user key E was pressed.

DOUBLE TRIANGLE PARAMETERS

With the lag time of the watershed known, the parameters of the TVA Double Triangle Model, UP, UR, T1, T2, and T3, can be calculated. Empirically derived values were used to find the parameters. These values depend on the land use characteristics of the watershed. The values for watersheds that are urban without extensive storm sewers, urban with storm sewers, denuded, agricultural and completely forested are listed in Table 4. The formulas used to calculate the double triangle parameters are:

$$UP' = KUP/TL$$

$$T1 = KT1 * TL$$

$$T2 = KT2 * TL$$

$$UR' = KUR/TL$$

$$T3 = T1 + ((2 - T2 * UP') / UR')$$

where KUP, KT1 KT2 and KUR are found in Table 4.

Before using those parameters, sveral adjustments were made to them. The first adjustment was to round off T1, T2 and T3 to the nearest multiple of DT. This was one of the basic assumptions of the TVA Double Triangle Model, it assures that the peak and inflection points occur at one of the calculated hydrograph ordinates.

The next correction was to make sure that T2 was at least one DT greater than T1and that T3 was greater than T2 by the same amount. This correction keeps the parameters in their relative order. To keep T3 from being an extremely large number, it was limited to fifteen times the value of TL.

The final correction makes sure that the area of the unit response function, URF, was equal to one. This is accomplished by first calculating the area of the initial URF from the formula:

TABLE 4

DOUBLE TRIANGLE PARAMETER FACTORS

Land Use	KUP	KT1	KT2	KUR
Urban	0.663	0.632	1.88	0.12
Urban with storm sewers	0.900	0.956	1.80	0.035
Coal Strip Mined	0.740	0.253	1.085	0.21
Agricultural	0.705	0.695	1.87	0.13
Forested	0.716	0.394	1.57	0.10

$$AREA = (0.5 * T1 * UP') + (0.5 * (UP' + UR') * (T2 - T1)) + (0.5 * UR' * (T3 - T2))$$

The area calculated was used to adjust UP and UR by the formulas:

$$UP = (1.0/AREA) * UP'$$

$$UR = (1.0/AREA) * UR'$$

CALCULATING HYDROGRAPH ORDINATES

With the double triangle parameters known, the hydrograph ordinates can be calculated. However, these values were not entered directly into the program. The number of ordinates in each section of the hydrograph were calculated by the formulas:

$$N1 = T1/DT$$

$$N2 = T2/DT$$

$$N3 = T3/DT$$

where N1, N2 and N3 are the number of ordinates. It should be noted that these three numbers should be whole numbers due to the fact that T1, T2 and T3 were multiples of DT. These numbers were entered into the program by pressing user key B' while N1 is in the calculator display, then N2 and N3 were entered by pressing the run/stop key, R/S, for each in order.

The next values that were entered into the calculator were calculated by the formulas:

$$S1 = UP/T1$$

$$S2 = (UR - UP)/(T2 - T1)$$

$$S3 = (O - UR)/(T3 - T2)$$

These values were entered by pressing user key C' with S1 in the calculator display, then entering S2 and S3 through the R/S key.

One final entry was made before the program was started. The surface area of the drainage basin in square miles was entered through the user key A'.

After the program was finally loaded, the hydrograph ordinates were calculated by pressing user key D'. Because a PC-100A printer was used for this thesis, each ordinate was printed out without the program stopping. However, if a printer was not available, the program would have stopped after calculating each ordinate for the user to record. The program would have to be restarted by pressing the R/S key.

After all the ordinates were calculated, the hydrograph was plotted by hand. Seventeen hydrographs predicted by the AFIT Runoff Model are shown in Appendix G. The observed hydrographs and the hydrographs predicted by AFRUM, STORM and SWMM, in Appendix G, were obtained from reference 12. The peak of the hydrograph predicted by the Rational Method occurs at the end of the rainfall.

SUMMARY

This section summarizes all the formulas used in the program and how the parameters are entered into the calculator. The formulas were:

$$S = (1000.0/CN) - 10.0$$

$$U = 3.24 * (SQMI/PI)^{0.6} \quad \text{for urban watersheds or}$$

$$U = 0.060 * SQMI + 0.0203 * PF + 1.16 \quad \text{for rural watersheds}$$

$$UP' = KUP/TL$$

$$T1 = KT1 * TL$$

$$T2 = KT2 * TL$$

$$UR' = KUR/TL$$

$$T3 = T1 + ((2 - T2 * UP') / UR')$$

T1, T2 and T3 must be corrected as specified in this chapter.

$$AREA = (0.5 * T1 * UP') + (0.5 * (UP' + UR') * (T2 - T1)) + (0.5 * UR' * (T3 - T2))$$

$$UP = (1.0 / AREA) * UP'$$

$$UR = (1.0 / AREA) * UR'$$

$$N1 = T1/DT$$

$$N2 = T2/DT$$

$$N3 = T3/DT$$

$$S1 = UP/T1$$

$$S2 = (UR - UP) / (T2 - T1)$$

$$S3 = (0 - UR) / T3 - T2)$$

The data were entered by the following procedures:

Value in the Calculator Display	Calculator Key Pressed	Value Calculated by Program
S	B	
U	A	
DT	C	
Cumulative Rainfall	D	

Value in the Calculator Display	Calculator Key Pressed	Value Calculated by Program
Cumulative Rainfall	D	
•	•	
•	•	
•	•	
0	E	TL
N1	2nd B'	
N2	R/S	
N3	R/S	
S1	2nd C'	
S2	R/S	
S3	R/S	
SQMI	2nd A'	
0	2nd D'	Hydrograph Ordinate
	R/S	Hydrograph Ordinate
	R/S	Hydrograph Ordinate
	•	•
	•	•
	•	•

Chapter 6

STATISTICAL TESTING AND ANALYSIS

This chapter will detail the statistical tests used to attempt to verify the research questions proposed at the end of Chapter 1. Those questions again, were as follows:

1. Can the AFIT Runoff Model accurately predict the runoff from an Air Force base?

2. Is the AFIT Runoff Model better than other commonly used stormwater models?

The simulation results for each model, shown in Appendix B, were used for the comparisons.

STATISTICAL TESTING

Throughout this chapter, several sets of hypotheses will be presented in the general form:

H_0: Something will happen

H_1: Something will not happen

alpha = 0.05

The key to the test is the null hypotheis, H_0. The objective of the test is to either reject H_0 or fail to reject H_0. Notice that the acceptance of H_0 is not an alternative. If a test is performed and H_0 is rejected, then the alternative hypothesis, H_1, is the choice to be selected. If a test is performed and H_0 is not rejected, then the null hypothesis is the alternative selected. The fact that H_0 is not rejected does not in itself provide proof of the validity of H_0. It merely means that there is not enough statistical evidence available to reject H_0 (4:264).

The decision to reject or fail to reject H_0 is based on probabilities and not on certainty. Hence, there are chances of error in making a decicion. The value of alpha indicates the importance that is attached to the consequences associated with rejecting H_0 when, in fact, H_0 should not have been rejected. An alpha level of .05 means that a five percent chance of being wrong when H_0 is rejected can be accepted (5:199).

NORMAL DISTRIBUTIONS

Many statistical tests, including the tests used in this thesis, require the assumption that the samples being tested came from a normally distributed population. A goodness-of-fit test should be used to determine whether the assumption is justified or not justified. The test used in this thesis had the following hypotheses and alpha risk:

H_0: the probability distribution is Normal

H_1: the probability distribution is not Normal

alpha = .05

In this thesis, the Lilliefors goodnes-of-fit test was used. The test, shown in Appendix C, failed to reject H_0; therefore, the assumption of normality was justified.

RESEARCH QUESTION NUMBER 1

Can the AFIT Runoff Model accurately predict the runoff from an Air Force Base?

To test this question, the hydrograph predicted by the AFIT Runoff Model was compared to the observed hydrograph. Three hydrograph parameters were used in the comparison, peak runoff, time to peak and volume of runoff. Each parameter was tested using the matched pair t-test. This test uses the differences of the population means to make the inferences. The following equation is used in this test:

$$\mu_D = \mu_O - \mu_A$$

where μ_D = the mean of the population of the differences

μ_O = the mean of the population of the observed parameters

μ_A = the mean of the population of the parameter predicted by the AFIT Runoff Model

If the parameters predicted by the AFIT Runoff Model are from the same population as the observed parameters, then the two means will be equal and the mean of the differences will be zero. Thus, the hypotheses are:

$$H_o: \mu_D = 0$$

$$H_1: \mu_D \neq 0$$

$$\text{alpha} = .05$$

The results of the matched pair t-test for peak flow, time to peak and volume of runoff are shown in Appendix D.

The tests for peak flow and volume of runoff failed to reject H_o; therefore, it can be concluded that there is no significant difference between the peak flow and volume predicted by the AFIT Runoff Model and the actual peak flow and volume. In the test for the time to peak parameter, the null hypothesis was rejected at an alpha level of 0.05.

RESEARCH QUESTION NUMBER 2

Is the AFIT Runoff Model better than other commonly used stormwater models?

Four other models were compared to the AFIT Runoff Model. The peak flow, time to peak and volume predicted by the five models were all compared to the observed parameters using the randomized complete block design analogue to the paired t-test. The equation for this test is:

$$y = \mu + t + \beta + e$$

where y = the individual parameter

μ = the overall mean

t = a treatment, or model, effect

β = a block, or watershed, effect

e = a random error

If the parameter prediction from all of the stormwater models and the observed parameter are statistically the same, the treatment effect, t, would be equal to zero. Hence, the hypothesis:

H_0: $t_1 = t_2 = t_3 = t_4 = t_5 = t_6 = 0$

H_1: At least one inequality

alpha = 0.05

The results of these tests, shown in Appedix E, were the same for all three parameters, reject H_0. This means that at least one of the models was significantly different from the others.

To determine which models are statistically related, Duncan's multiple-range test was used. The results of this series of tests are in Appendix F. No significant difference was found between the AFIT Runoff Model and the Air Force Runoff Model in the prediction of the peak flow. This can be expected seeing as the AFIT Runoff Model was based on AFRUM. All the models, with the exception of SWMM, produced peak flows that were statistically the same as the actual peak flow.

In predicting the volume of the runoff, SWMM was the only model, again, that predicted values that were statistically different from the observed values. The AFIT Runoff Model, AFRUM, STORM and the Rational Method were all related to the observed volume.

Only the Rational Method predicted time to peak values that were significantly different from the observed values. All other models, the AFIT Runoff Model, AFRUM, STORM and SWMM simulated values that were statistically the same as the observed time to peak values according to the Duncan's multiple range test. This is in contrast to the results obtained from the matched pair t-test, which concluded that the time to peak values predicted by the AFIT Runoff Model were not the same as the observed values. The reason for the discrepancy is that statistics is not an exact science. There was a five percent chance of error when rejecting H_0. A random error caused one test to conclude one thing and another to contradict it.

Chapter 7

RECOMMENDATIONS AND CONCLUSIONS

The objective of this thesis was to develop a stormwater runoff model that can accurately simulate the runoff from an Air Force base without the aid of a computer. The AFIT Runoff Model does not require a computer; all that is needed to use this model is a Texas Instruments TI-59 programmable calculator.

ACCURACY

The accuracy of the model was taken into consideration in the first research question which stated: Can the AFIT Runoff Model accurately predict the runoff from an Air Force base? The statistical evidence tended to support the conclusion that the AFIT Runoff Model can predict the runoff accurately. The peak flow and volume predicted by the AFIT Runoff Model were statistically the same as the observed values. The AFIT Runoff Model did not do as well predicting the time to peak, according to the matched pair t-test.

The peak flow is all that is usually needed from the model for design purposes. The time to peak and volume parameters were tested because they are characteristics of the hydrograph shape. The hydrograph is needed only when designing a retention or catch basin. All other structures only need the peak flow for design.

COMPARISON TO OTHER MODELS

The second research question stated: Is the AFIT Runoff Model better than other commonly used stormwater models? The AFIT Runoff Model was compared to four other commonly used models. The Rational Method is a non-computerized method which is the most commonly used model. The Environmental Protection Agency Stormwater Management Model (SWMM), the Corps of Engineers Urban Stormwater Runoff Model (STORM) and the Air Force Runoff Model (AFRUM) are computerized models. All of the models were tested against the AFIT Runoff Model and the observed hydrograph.

There was not enough evidence in this study to prove that the AFIT Runoff Model is better at predicting peak flow than the Rational Method. However, the Rational Method has the tendency to overestimate the peak runoff from a high intensity, short duration storm, such as in Appendix G-3.

The hydrographs produced by the AFIT Runoff Model were very close to those predicted by AFRUM. The AFIT Runoff Model tended to peak lower and later than the hydrographs predicted by AFRUM, but it was very similar in shape. AFRUM has the advantage of automatically plotting the hydrograph and also predicting the quality of the runoff. The

AFIT Runoff Model, on the other hand, has the advantages of being as accurate as AFRUM while not requiring a computer. The base level Civil Engineers can use the AFIT Runoff Model at their own base, a characteristic not found in AFRUM.

RECOMMENDATIONS

The AFIT Runoff Model was tested using three watersheds on the same Air Force base. Future research could center on testing the model at other bases in other regions of the United States or in other countries concentrating primarily on comparing the AFIT Runoff Model with the Rational Method. Attempts to improve the accuracy of the model should concentrate on improving the accuracy of the models that the AFIT Runoff Model is based on. The accuracy of the AFIT Runoff Model is directly related to the accuracy of the curve numbers from the SCS Curve Number Model and AFRUM.

The AFIT Runoff Model works on a Texas Instruments calculator; however, this model could be easily modified to operate on other programmable calculators. The Hewlett and Packard company manufactures a line of popular programmable calculators that could handle the AFIT Runoff Model. The conversion of the model to one of the other calculators would be as simple as changing a Fortran program to Basic.

SAFETY FACTOR

The final recommendation is directed to the engineer using the AFIT Runoff Model. Generally the safety factors used in the design of stormwater drainage structures are taken into account in the selection of the return period of the design storm. For example, a structure designed for a 100-year storm will be larger than a structure based on a twenty-five year storm. However, care should be taken in using the AFIT Runoff Model, if the engineer has been using a more conservative model such as the Rational Method. The Rational Method adds another safety factor by virtue of the conservative nature of the model. Because the AFIT Runoff Model simulates the runoff more closely to the actual runoff, the engineer may want to chose a design storm with a longer return period, such as using a 100-year storm rather than a 50-year storm. This may be more important if the area being drained by the structure can be severely damaged by the water if the structure fails.

CONCLUSION

The AFIT Runoff Model can be a useful tool for the Air Force engineer. It is simple to use yet as accurate as the more complicated computer models. The AFIT Runoff Model combines some of the advantages of a computer model with the advantages of the non-computerized methods. The Rational Method is easy to use and it does not require a computer, but it may not always be as accurate as the user would like. Computer models, like AFRUM, are usu-ly more accurate, but they require a computer to run. The AFIT Runoff Model is easy to use, does not require a computer and has simulation power similar to AFRUM.

The determination as to when the AFIT Runoff Model should be used depends largely on the judgement of the engineer. He may want to use the AFIT Runoff Model if the size of the watershed indicates that a longer duration storm would produce more runoff. The AFIT Runoff Model should also be used if a hydrograph is desired. In all other cases, the Rational Method could be used.

APPENDICES

APPENDIX A

LISTING OF THE AFIT RUNOFF MODEL

LISTING OF THE AFIT RUNOFF MODEL

Code No.	Key	Comments	Code No.	Key	Comments
00	2nd LBL	ENTER U	21	STO	
01	A		22	02	RAIN (I)
02	STO		23	-	
03	08	U	24	RCL	
04	R/S		25	03	RAIN (I-1)
05	2nd LBL	ENTER S	26	=	
06	B		27	STO	
07	STO		28	04	RF (I)
08	01	S	29	RCL	
09	R/S		30	02	RAIN (I)
10	2nd LBL	ENTER DT	31	STO	
11	C		32	03	RAIN (I-1)
12	STO		33	CLR	
13	00	DT	34	X⇋ t	t = 0
14	20		35	RCL	
16	STO		36	04	RF (I)
17	18		37	2ndX≥t	
18	R/S		38	47	
19	2nd LBL	ENTER RAINFALL	40	RCL	
20	D		41	10	

42	STO	
43	09	
44	GTO	
45	55	
47	RCL	
48	10	
49	+	
50	RCL	
51	04	
52	=	
53	STO	
54	09	ARF
55	RCL	
56	01	S
57	x	
58	0.2	
60	=	
61	X ⇌ t	t = .2 S
62	RCL	
63	09	ARF (I)
64	X ≥ t	
65	74	
67	RCL	
68	12	SRO (I-1)
69	STO	
70	11	SRO (I)
71	GTO	
72	98	
74	(	
75	RCL	
76	09	ARF (I)
77	-	
78	0.2	
80	x	
81	RCL	
82	01	S
83	)	
84	x^2	
85	÷	
86	(	
87	RCL	
88	09	ARF (I)
89	+	
90	0.8	
92	x	
93	RCL	
94	01	S
95	=	
96	STO	
97	11	SRO (I)
98	-	
99	RCL	

100	12	
101	=	PE (I)
102	STO	
103	10	
104	SBR	
105	CE	
106	RCL	
107	10	
108	÷	
109	RCL	
110	00	
111	=	PEI (I)
112	2nd SUM	
113	06	SUMPE
114	X^2	
115	2nd SUM	
116	07	SSQPE
117	RCL	
118	09	
119	STO	
120	10	
121	RCL	
122	11	
123	STO	
124	12	
125	1	
126	2nd SUM	
127	05	
128	RCL	
129	05	
130	R/S	
131	2nd LBL	Calculate TL
132	E	
133	RCL	
134	07	SSQPE
135	÷	
136	RCL	
137	06	SUMPE
138	=	REI
139	y^x	
140	0.4	
142	=	
143	1/x	
144	x	
145	RCL	
146	08	U
147	=	TL
148	2nd PRT	
149	R/S	
150	2nd LBL	
151	CE	
152	CLR	

153	X ⇌ t	t = 0
154	RCL	
155	10	
156	X = t	
157	193	
159	2nd SUM	
160	19	
161	1	
162	2nd SUM	
163	12	
164	4	*
165	X ⇌ t	
166	RCL	
167	12	
168	X ≥ t	
169	174	
171	GTO	
172	196	
174	RCL	
175	19	
176	+	
177	4	*
178	=	
179	STO IND	
180	18	
181	CLR	
182	1	
183	2nd SUM	
184	18	
185	CLR	
186	STO	
187	12	
188	STO	
189	19	
190	GTO	
191	196	
193	1	
194	2nd SUM	
195	17	
196	INV SBR	
197	2nd LBL	ENTER SQMI
198	2nd A'	
199	STO	
200	02	
201	20	
203	STO	
204	18	
205	R/S	
206	2nd LBL	ENTER N!
207	2nd B'	
208	STO	
209	06	

210	R/S	ENTER N2
211	STO	
212	07	
213	R/S	ENTER N3
214	STO	
215	08	
216	R/S	
217	2nd LBL	ENTER S1
218	2nd C'	
219	STO	
220	02	
221	R/S	ENTER S2
222	STO	
223	03	
224	R/S	ENTER S3
225	STO	
226	04	
227	R/S	
228	2nd LBL	Calculate Hydrograph
229	2nd D'	
230	2	
231	STO	
232	16	
233	1	
234	2nd SUM	
235	14	
236	RCL	
237	14	
238	-	
239	1	
240	=	
241	STO	
242	05	
243	2nd DSZ	
244	05	
245	250	
247	GTO	
248	350	
250	RCL	
251	14	
252	$X \rightleftarrows t$	
253	RCL	
254	06	
255	$X \geq t$	
256	322	
258	RCL	
259	07	
260	$X \geq t$	
261	297	
263	RCL	
264	14	
265	-	

266	RCL
267	06
268	-
269	RCL
270	07
271	=
272	x
273	RCL
274	04
275	+
276	RCL
277	07
278	x
279	RCL
280	03
281	+
282	RCL
283	06
284	x
285	RCL
286	02
287	=
288	x
289	RCL
290	00
291	=
292	STO
293	09
294	GTO
295	333
297	RCL
298	14
299	-
300	RCL
301	06
302	=
303	x
304	RCL
305	03
306	+
307	RCL
308	06
309	x
310	RCL
311	02
312	=
313	x
314	RCL
315	00
316	=
317	STO
318	09

319	GTO
320	333
322	RCL
323	14
324	x
325	RCL
326	00
327	x
328	RCL
329	02
330	=
331	STO
332	09
333	SBR
334	1/x
335	RCL
336	09
337	x
338	RCL
339	10
340	=
341	+
342	RCL
343	15
344	=
345	STO
346	15
347	GTO
348	233
350	RCL
351	11
352	x
353	640
356	x
357	43560
362	÷
363	(
364	3600
368	x
369	12
371	=
372	x
373	RCL
374	15
375	=
376	R/S or PRT if using a printer
377	2nd D'
378	2nd LBL
379	1/x
380	RCL
381	17
382	+

383	1	
384	=	
385	X ⇌ t	
386	RCL	
387	16	
388	X ≥ t	
389	397	
391	CLR	
392	STO	
393	10	
394	GTO	
395	424	
397	4	*
398	STO	
399	12	
400	RCL	
401	12	
402	X ⇌ t	
403	RCL	
404	16	
405	X ≥ t	
406	415	
408	RCL IND	
409	18	
410	STO	
411	10	
412	GTO	
413	423	
415	4	*
416	2nd SUM	
417	12	
418	1	
419	2nd SUM	
420	18	
421	GTO	
422	400	
424	20	
426	STO	
427	18	
428	INV SBR	

* For extra long storms, these four numbers can be changed from four to five to avoid exceeding the calculators storage space.

APPENDIX B

SIMULATION RESULTS

APPENDIX B-1

PEAK FLOW SIMULATION RESULTS FOR MCDOWELL DITCH

Storm Date	Observed (cfs)	AFIT (cfs)	AFRUM (cfs)	STORM (cfs)	SWMM (cfs)	Rational Method (cfs)
3 Aug 78	15.0	14.9	18.6	58.2	8.1	17.0
27 Aug 78	6.7	2.6	2.7	7.2	10.5	3.8
15 Sept 78	6.6	4.1	4.7	7.6	55.4	91.0
14 Nov 78	13.0	8.1	9.0	30.0	74.6	11.8
3 Dec 78	22.0	25.1	42.2	55.8	74.1	13.7
11 Apr 79	0.9	1.3	2.4	6.3	12.9	5.4
8 Jul 79	9.2	12.1	17.5	66.4	225.0	14.2

APPENDIX B-2

PEAK FLOW SIMULATION RESULTS FOR EAST DITCH

Storm Date	Observed (cfs)	AFIT (cfs)	AFRUM (cfs)	STORM (cfs)	SWMM (cfs)	Rational Method (cfs)
14 Nov 78	1.4	0.6	0.6	4.8	5.3	2.6
17 Nov 78	0.6	0.6	0.6	1.0	1.7	1.9
3 Dec 78	4.0	4.3	5.0	8.6	2.0	3.8
7 Dec 78	0.7	0.4	0.5	0.7	23.5	1.0
11 Apr 79	1.1	1.1	0.9	2.5	4.5	1.5
8 Jul 79	2.1	3.5	5.9	19.9	12.9	3.9

APPENDIX B-3

PEAK FLOW SIMULATION RESULTS FOR CLINE DITCH

Storm Date	Observed (cfs)	AFIT (cfs)	AFRUM (cfs)	STORM (cfs)	SWMM (cfs)	Rational Method (cfs)
2 Aug 78	2.0	0.9	0.9	18.6	23.3	5.9
27 Aug 78	1.7	1.2	1.2	6.2	14.5	2.3
14 Sept 78	1.9	1.1	1.3	3.2	7.1	31.9
3 Dec 78	1.9	2.1	2.1	6.0	5.4	2.8

APPENDIX B-4

VOLUME SIMULATION RESULTS FOR MCDOWELL DITCH

Storm Date	Observed (in)	AFIT (in)	AFRUM (in)	STORM (in)	SWMM (in)	Rational Method (in)
3 Aug 78	0.166	0.159	0.178	0.451	0.110	0.458
27 Aug 78	0.066	0.065	0.067	0.059	0.080	0.165
15 Sept 78	0.052	0.049	0.054	0.058	0.150	0.163
14 Nov 78	0.309	0.313	0.320	0.485	0.440	0.488
3 Dec 78	0.581	0.564	0.568	0.484	0.510	0.502
11 Apr 79	0.112	0.110	0.112	0.180	0.275	0.349
8 Jul 79	0.568	0.571	0.568	0.950	4.410	0.918

APPENDIX B-5

VOLUME SIMULATION RESULTS FOR EAST DITCH

Storm Date	Observed (in)	AFIT (in)	AFRUM (in)	STORM (in)	SWMM (in)	Rational Method (in)
14 Nov 78	0.130	0.124	0.127	0.220	0.450	0.442
17 Nov 78	0.063	0.059	0.060	0.045	0.060	0.148
3 Dec 78	0.264	0.265	0.226	0.341	0.070	0.431
7 Dec 78	0.058	0.051	0.055	0.045	0.350	0.283
11 Apr 79	0.277	0.281	0.277	0.200	0.357	0.425
8 Jul 79	0.834	0.836	0.834	0.850	1.700	0.994

APPENDIX B-6

VOLUME SIMULATION RESULTS FOR CLINE DITCH

Storm Date	Observed (in)	AFIT (in)	AFRUM (in)	STORM (in)	SWMM (in)	Rational Method (in)
2 Aug 78	0.014	0.015	0.015	0.525	0.190	0.539
27 Aug 78	0.013	0.011	0.013	0.269	0.180	0.465
14 Sept 78	0.011	0.011	0.011	0.098	0.070	0.208
3 Dec 78	0.021	0.021	0.021	0.198	0.110	0.438

APPENDIX B-7

TIME TO PEAK SIMULATION RESULTS FOR MCDOWELL DITCH

Storm Date	Observed (hrs)	AFIT (hrs)	AFRUM (hrs)	STORM (hrs)	SWMM (hrs)	Rational Method (hrs)
3 Aug 78	2.00	2.70	2.50	2.00	7.00	7.5
27 Aug 78	11.50	10.25	9.25	7.00	4.25	12.00
15 Sept 78	2.25	2.00	1.75	2.00	0.50	0.50
14 Nov 78	10.75	10.25	10.50	8.00	8.50	11.50
3 Dec 78	6.50	7.25	6.50	6.00	8.50	10.20
11 Apr 79	5.00	8.25	9.00	3.00	3.00	18.00
8 Jul 79	2.00	5.00	4.00	3.00	3.50	18.00

APPENDIX B-8

TIME TO PEAK SIMULATION RESULTS FOR EAST DITCH

Storm Date	Observed (hrs)	AFIT (hrs)	AFRUM (hrs)	STORM (hrs)	SWMM (hrs)	Rational Method (hrs)
14 Nov 78	10.50	11.25	11.00	9.00	8.75	12.00
17 Nov 78	7.50	7.75	7.50	6.00	4.50	5.50
3 Dec 78	4.75	4.75	4.75	7.00	3.25	8.00
7 Dec 78	7.25	9.25	9.00	4.00	6.25	14.50
11 Apr 79	3.00	13.50	12.00	5.00	3.00	20.00
8 Jul 79	3.00	6.50	5.00	3.00	1.50	18.00

APPENDIX B-9

TIME TO PEAK SIMULATION RESULTS FOR CLINE DITCH

Storm Date	Observed (hrs)	AFIT (hrs)	AFRUM (hrs)	STORM (hrs)	SWMM (hrs)	Rational Method (hrs)
2 Aug 78	0.75	1.50	1.25	2.00	1.00	7.00
27 Aug 78	2.25	2.50	2.25	2.00	0.75	15.5
14 Sept 78	1.25	1.25	1.00	2.00	1.00	0.50
3 Dec 78	5.75	6.00	5.50	4.00	4.00	12.00

APPENDIX C

LILLIEFORS GOODNESS-OF-FIT TEST

LILLIEFORS TEST

H_0: f(x) is Normal

H_1: f(x) is not Normal

alpha = .05

$\bar{x}$ = 10.49

s = 6.854

x	$Z = \frac{x - 10.49}{6.854}$		S_z	F_z	Max $\lvert F(z)-S(z) \rvert$
0.9	-1.396	0	0.143	0.0823	0.0607
6.6	-0.568	0.143	0.286	0.2843	0.1413
6.7	-0.553	0.286	0.428	0.2912	0.1368
9.2	-0.188	0.421	0.571	0.4247	0.1463 **
13.0	0.366	0.571	0.714	0.6443	0.0733
15.0	0.658	0.714	0.857	0.7454	0.1116
22.0	1.679	0.857	1.00	0.9535	0.0965

0.1463 < 0.300

FAIL TO REJECT H_0

Assumption of Normality is Justified

APPENDIX D

MATCHED PAIR T-TEST

APPENDIX D-1

PEAK FLOW T-TEST

$H_0: \mu_d = 0$

$H_1: \mu_d \neq 0$

Observed	AFIT Runoff Model	Difference
15.0	14.9	0.1
6.7	2.6	4.1
6.6	4.1	2.5
13.0	8.1	4.9
22.0	25.1	-3.1
0.9	1.3	-0.4
9.2	12.1	-2.9
1.38	0.61	0.77
0.59	0.58	0.01
4.03	3.71	0.32
0.65	0.40	0.25
1.05	1.11	-0.06
2.10	3.52	-1.42
2.02	0.88	1.14
1.70	1.18	0.52
1.90	1.07	0.83
1.90	2.07	-0.17
		7.41

$\bar{x} = 0.43588$

$S_x = 2.04756$

$$t = \frac{\bar{x} - u_0}{S_x / \sqrt{n}} = \frac{0.43588 - 0}{2.04756 / \sqrt{17}} = 0.8777$$

$1.746 > 0.8777$

FAIL TO REJECT H_0

There is no significant difference between the actual peak flow and the peak flow predicted by the AFIT Runoff Model.

APPENDIX D-2

VOLUME T-TEST

H_0: $\mu_D = 0$

H_1: $\mu_D \neq 0$

alpha = .05

Observed	AFIT Runoff Model	Difference
0.166	0.159	0.007
0.066	0.065	0.001
0.052	0.049	0.003
0.309	0.313	-0.004
0.581	0.564	0.017
0.112	0.110	0.002
0.568	0.571	-0.003
0.130	0.124	0.006
0.063	0.059	0.004
0.264	0.265	-0.001
0.058	0.051	0.007
0.277	0.281	-0.004
0.834	0.836	-0.002
0.014	0.015	-0.001
0.013	0.011	0.002
0.011	0.011	0
0.021	0.021	0

$S_x = 0.00522$

$\bar{X} = 0.002$

$$t = \frac{\bar{X} - \mu_o}{S_x / \sqrt{n}} = \frac{0.002 - 0}{0.00522 / \sqrt{17}} = 1.5797$$

$1.746 > 1.5797$

FAIL TO REJECT H_o

There is no significant difference between the observed volume and the volume predicted by the AFIT Runoff Model.

APPENDIX D-3

TIME TO PEAK T-TEST

H_o: $\mu_D = 0$

H : $\mu_D \neq 0$

alpha = .05

Observed	AFIT Runoff Model	Difference
2.00	2.70	-0.70
11.50	10.25	1.25
2.25	2.00	0.25
10.75	10.25	0.50
6.50	7.25	-0.75
5.00	8.25	-3.25
2.00	5.00	-3.00
10.50	11.25	-3.00
7.50	7.75	-0.25
4.75	4.75	0
7.25	9.25	-2.0
3.00	13.5	-10.50
3.00	6.50	-3.50
0.75	1.50	0.75
2.25	2.50	-0.25
1.25	1.25	0
5.75	6.00	-0.25

$S_x = 2.74608$

$\bar{x} = -1.3206$

$$t = \frac{-1.3206 - 0}{2.74608 / \sqrt{17}} = -1.983$$

$$-1.983 < -1.746$$

REJECT H_0

At an alpha level of .05, there is a difference between the observed time to peak and the time to peak predicted by the AFIT Runoff Model; however, at an alpha level of .01 the null hypothesis could not be rejected.

APPENDIX E

RANDOMIZED COMPLETE BLOCK DESIGN TEST

APPENDIX E-1

RANDOMIZED COMPLETE BLOCK DESIGN FOR PEAK FLOW

H_0: $t_1 = t_2 = t_3 = t_4 = t_5 = t_6 = 0$

H_1: At least one inequality

Observed	AFIT	AFRUM	RM	STORM	SWMM	Rk	$\bar{Y}$
15.0	14.9	18.6	17.0	58.2	8.1	88.8	14.8
6.7	2.6	2.7	3.8	7.2	10.5	33.5	5.6
6.6	4.1	4.7	91.0	7.6	55.4	169.4	28.2
13.0	8.1	9.0	11.8	30.0	74.6	146.5	24.4
22.0	25.1	42.2	13.7	55.8	74.1	232.9	38.8
0.9	1.3	2.4	5.4	6.3	12.9	29.2	4.8
9.2	12.1	17.5	14.2	66.4	225.0	344.9	57.4
1.4	0.6	0.6	2.6	4.8	23.5	33.5	5.6
0.6	0.6	0.6	1.9	1.0	1.6	6.4	1.1
4.3	3.7	3.6	3.8	8.6	2.0	25.8	4.3
0.7	0.4	0.5	1.0	0.7	23.5	26.7	4.5
1.1	1.1	0.9	1.5	2.5	4.5	11.6	1.9
2.1	3.5	5.9	3.9	19.9	12.9	48.2	8.1
2.0	0.9	0.9	5.9	18.6	23.3	51.6	8.6
1.7	1.2	1.2	2.3	6.2	14.5	27.1	4.5
1.9	1.1	1.0	31.9	3.2	7.1	46.2	7.7
1.9	2.1	2.1	2.8	6.0	5.4	20.3	3.4
90.6	83.3	114.5	214.5	303.0	578.9	1342.1	
5.33	4.9	6.7	12.6	17.8	34.1		

AD-A109 875 AIR FORCE INST OF TECH WRIGHT-PATTERSON AFB OH SCHOOL--ETC F/G 8/8
SIMULATION OF RUNOFF FROM AN AIR FORCE BASE USING A PROGRAMMABL--ETC(U)
SEP 81 J W LUGINBYHL
UNCLASSIFIED AFIT-LSSR-60-81 NL

2 OF 2

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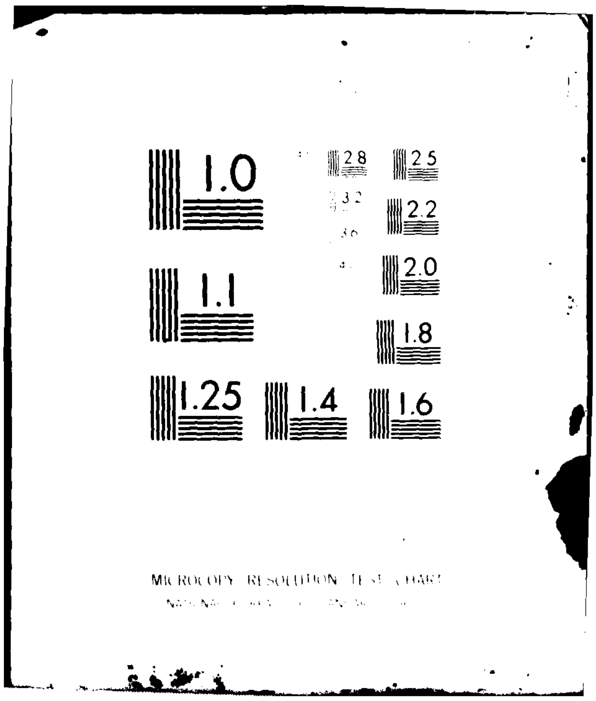
1.0
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MICROCOPY RESOLUTION TEST CHART

$$CF = \frac{(\Sigma\Sigma\Sigma Y_{ijk})^2}{JKN} = \frac{(1342.08)^2}{12 * 6 * 1} = 17658$$

$$SST = \Sigma\Sigma\Sigma\ Y^2_{ijk} - CF = 91787 - 17658 = 74128$$

$$SSC = \frac{\Sigma\ C_j^2}{KN} - CF = 11829$$

$$SSR = \frac{\Sigma\ Rk^2}{JN} - CF = 22978$$

$$SSE = SST - SSC - SSR = 39321$$

ANOVA TABLE

Source	SS	d.f.	MS	F = MS/MSE
Columns (Models)	11829	5	2365.8	4.81
Rows (Storms)	22978	16	1436.1	2.92
Error	39321	80	491.5	
Total	74128	101		

$F_{.05,5,80} = 2.33$

$2.33 < 4.81$

REJECT H_0

The models are different.

APPENDIX E-2

RANDOMIZED COMPLETE BLOCK DESIGN FOR VOLUME

H_0: $t_1 = t_2 = t_3 = t_4 = t_5 = t_6 = 0$

H_1: At least one inequality

Observed	AFIT	AFRUM	RM	STORM	SWMM	Rk	$\bar{Y}$
0.17	0.16	0.18	0.46	0.45	0.11	1.52	0.254
0.07	0.07	0.07	0.16	0.06	0.08	0.50	0.084
0.05	0.05	0.05	0.16	0.06	0.15	0.53	0.088
0.31	0.31	0.32	0.49	0.49	0.44	2.35	0.393
0.58	0.56	0.57	0.50	0.49	0.51	3.21	0.535
0.11	0.11	0.11	0.35	0.18	0.28	1.14	0.190
0.57	0.57	0.57	0.92	0.95	4.41	7.99	1.331
0.13	0.12	0.13	0.44	0.22	0.45	1.49	0.249
0.6	0.6	0.6	0.15	0.05	0.06	0.44	0.073
0.26	0.26	0.27	0.43	0.34	0.07	1.64	0.273
0.06	0.05	0.06	0.28	0.04	0.35	0.84	0.140
0.28	0.28	0.28	0.42	0.20	0.35	1.81	0.302
0.83	0.84	0.83	0.99	0.85	1.70	6.05	1.008
0.01	0.02	0.02	0.54	0.53	0.19	0.82	0.136
0.01	0.01	0.01	0.46	0.27	0.18	0.95	0.158
0.01	0.01	0.01	0.21	0.10	0.07	0.41	0.068
0.02	0.02	0.02	0.44	0.20	0.11	0.81	0.135
3.55	3.51	3.55	7.42	5.46	9.51	32.48	
0.21	0.21	0.21	0.44	0.32	0.56		

CF = 10.343

SST = 24.869

SSC = 2.158

SSR = 9.8997

SSE = 12.8091

ANOVA TABLE

Source	SS	d.f.	MS	F = MS/MSE
Columns	2.1582	5	0.43164	2.696
Rows	9.8997	16	0.61873	3.864
Error	12.8091	80	0.16011	
Total	24.867	101		

$F_{.05,5,80} = 2.33$

$2.33 < 2.696$

REJECT H_0

All models are not the same

APPENDIX E-3

RANDOMIZED COMPLETE BLOCK DESIGN FOR TIME TO PEAK

H_0: $t_1 = t_2 = t_3 = t_4 = t_5 = t_6 = 0$

H_1: At least one inequality

Observed	AFIT	AFRUM	RM	STORM	SWMM	Rk	$\bar{Y}$
2.00	2.75	2.50	7.5	2.0	7.00	23.75	3.95
11.50	10.25	9.25	12.00	7.00	4.25	54.25	9.04
2.25	2.00	1.75	0.50	2.00	0.50	9.00	1.50
10.75	10.25	10.50	11.50	8.00	8.50	59.50	9.92
6.50	7.25	6.50	10.20	6.00	8.50	44.95	7.49
5.00	8.25	9.00	18.00	3.00	3.00	46.25	7.71
2.00	5.00	4.00	18.00	3.00	3.50	35.50	5.92
10.50	11.25	11.00	12.00	9.00	8.75	62.50	10.42
7.50	7.75	7.50	5.50	6.00	4.50	38.75	6.46
4.75	4.75	4.75	8.00	7.00	3.25	32.50	5.42
7.25	9.25	9.00	14.50	4.00	6.25	50.25	8.38
3.00	13.5	12.00	20.00	5.00	3.00	56.50	9.42
3.00	6.50	5.00	18.00	3.00	1.50	37.00	6.17
0.75	1.5	1.25	7.00	2.00	1.00	13.50	2.25
2.25	2.50	2.25	15.50	2.00	0.75	25.25	4.21
1.25	1.25	1.00	0.50	2.00	1.00	7.00	1.17
5.75	6.00	5.50	12.00	4.00	4.00	37.25	6.21
86.00	110.00	102.75	190.70	75.00	69.25	633.70	
5.06	6.47	6.04	11.22	4.41	4.07		

CF = 3937

SST = 5920

SSC = 2537

SSR = 2743

SSE = 640

ANOVA TABLE

Source	SS	d.f.	MS	F = MS/MSE
Columns	2537	5	507	63.4
Rows	2743	16	171	21.4
Error	640	80	8	
Total	5920	101		

$F_{.05,\ 5,\ 80} = 2.33$

$2.33 < 63.4$

REJECT H_0

All models are not the same.

APPENDIX F

DUNCAN'S MULTIPLE RANGE TEST

APPENDIX F-1

DUNCAN'S MULTIPLE RANGE FOR PEAK FLOW

Model	$\bar{x}$
AFIT	4.90
Observed	5.33
AFRUM	6.74
RM	12.62
STORM	17.82
SWMM	34.06

MSE = 491.5   d.f. = 80

$$S_{\bar{x}} = \sqrt{\frac{MSE}{n}} = \sqrt{\frac{491.5}{17}}$$

$S_{\bar{x}} = 5.377$

p =	2	3	4	5	6
Table Range	2.819	2.966	3.063	3.134	3.189
Least Significant Range	15.16	15.95	16.47	16.85	17.15

SWMM vs. AFIT	34.06 - 4.9 = 29.16 > 17.15
SWMM vs. Observed	34.06 - 5.33 = 28.73 > 16.85
SWMM vs. AFRUM	34.06 - 6.74 = 27.33 > 16.47
SWMM vs. RM	34.06 - 12.62 = 21.44 > 15.95
SWMM vs. STORM	34.06 - 17.82 = 16.24 > 15.16

STORM vs. AFIT	17.82	-	4.90	=	12.92	<	16.85
STORM vs. Observed	17.82	-	5.33	=	12.49	<	16.47
STORM vs. AFRUM	17.82	-	6.75	=	11.08	<	15.95
STORM vs. RM	17.82	-	12.62	=	5.30	<	15.16
RM vs. AFIT	12.62	-	4.90	=	7.72	<	16.47
RM vs. Observed	12.62	-	5.33	=	7.29	<	15.95
RM vs. AFRUM	12.62	-	6.74	=	5.89	<	15.16
AFRUM vs. AFIT	6.74	-	4.90	=	1.84	<	15.95
AFRUM vs. Observed	6.74	-	5.33	=	1.41	<	15.16
Obseved vs. AFIT	5.33	-	4.90	=	0.43	<	15.16

AFIT   Observed   AFRUM   RM   STORM   SWMM

All models underlined by the same line are not significantly different. All models, with the exception of SWMM are statistically the same.

APPENDIX F-2

DUNCAN'S MULTIPLE RANGE TEST FOR VOLUME

Model	$\bar{\bar{x}}$
AFIT	0.2026
Observed	0.2082
AFRUM	0.2086
STORM	0.3211
RM	0.4362
SWMM	0.5592

MSE = 0.16011

$S_{\bar{x}}$ = 0.09705

p =	2	3	4	5	6
Table Range	2.819	2.966	3.063	3.134	3.189
LSR	0.274	0.288	0.297	0.304	0.309

SWMM vs. AFIT	0.5529 - 0.2062 = 0.353 < 0.
SWMM vs. Observed	0.5529 - 0.2082 = 0.351 > 0.
SWMM vs. AFRUM	0.5529 - 0.2086 = 0.351 > 0.
SWMM vs. STORM	0.5529 - 0.3211 = 0.238 > 0.
SWMM vs. RM	0.5529 - 0.4362 = 0.123 > 0.

RM vs. AFIT	0.4362 - 0.2062	≈	0.230	< 0.304
RM vs. Observed	0.4362 - 0.2082	≈	0.228	< 0.297
RM vs. AFRUM	0.4362 - 0.2086	≈	0.228	< 0.288
RM vs. STORM	0.4362 - 0.3211	≈	0.115	< 0.274
STORM vs. AFIT	0.3211 - 0.2062	≈	0.115	< 0.297
STORM vs. Observed	0.321 - 0.2082	≈	0.113	< 0.288
STORM vs. AFRUM	0.3211 - 0.2086	≈	0.113	< 0.274
AFRUM vs. AFIT	0.2086 - 0.2062	≈	0.002	< 0.288
AFRUM vs. Observed	0.2086 - 0.2082	≈	0.0004	< 0.274
Observed vs. AFIT	0.2082 - 0.2062	≈	0.002	< 0.274

SWMM RM STORM AFRUM Observed AFIT

The underlined models are statistically the same. SWMM is statistically the same as the Rational Method, STORM and AFRUM but not the AFIT Runoff Model or the Observed hydrograph. All other models are related.

APPENDIX F-3

DUNCAN'S MULTIPLE RANGE TEST FOR TIME TO PEAK

Model	$\bar{x}$
SWMM	4.07
STORM	4.41
Observed	5.06
AFRUM	6.04
AFIT	6.47
Rational Method	11.22

MSE = 8

$S_{\bar{x}}$ = 0.686

p =	2	3	4	5	6
Table Range	2.819	2.966	3.063	3.134	3.189
LSR	1.934	2.035	2.101	2.150	2.188

RM vs. SWMM	11.22 - 4.07 = 7.15 > 2.188
RM vs. STORM	11.22 - 4.41 = 6.81 > 2.150
RM vs. Observed	11.22 - 5.06 = 6.16 > 2.101
RM vs. AFRUM	11.22 - 6.04 = 5.18 > 2.035
RM vs. AFIT	11.22 - 6.47 = 4.75 > 1.934

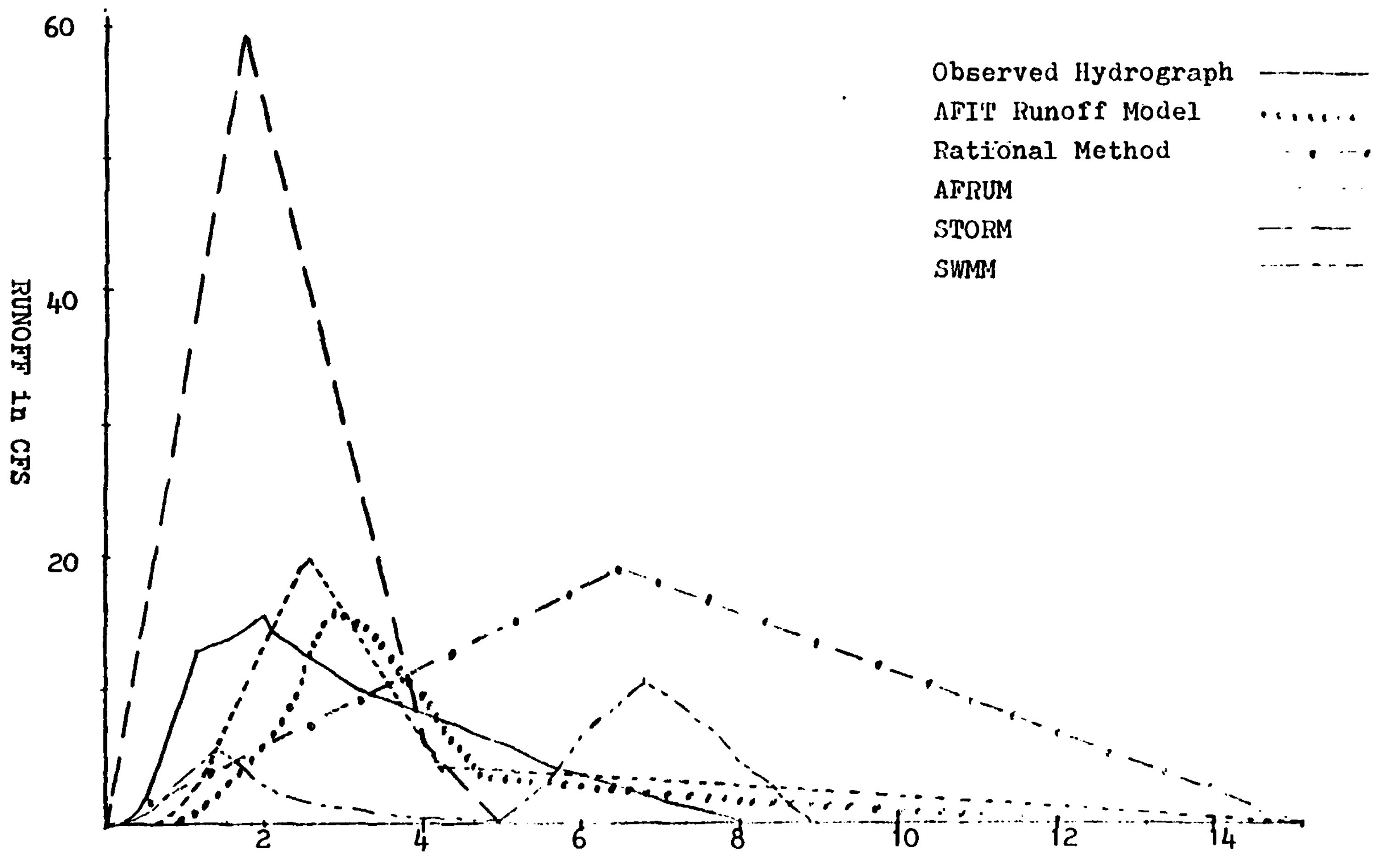
AFIT vs. SWMM	$6.47 - 4.07 = 2.40 > 2.150$
AFIT vs. STORM	$6.47 - 4.41 = 2.06 < 2.101$
AFIT vs. Observed	$6.47 - 5.06 = 1.41 < 2.035$
AFIT vs. AFRUM	$6.47 - 6.04 = 0.98 < 1.934$
AFRUM vs. SWMM	$6.04 - 4.07 = 1.97 < 2.101$
AFRUM vs. STORM	$6.04 - 4.41 = 1.63 < 2.035$
AFRUM vs. Observed	$6.04 - 5.06 = 0.98 < 1.934$
Observed vs. STORM	$5.06 - 4.07 = 0.99 < 2.035$
Observed vs. SWMM	$5.06 - 4.41 = 0.65 < 1.934$
STORM vs. SWMM	$4.41 - 4.07 = 0.34 < 1.934$

RM AFIT AFRUM OBSERVED STORM SWMM

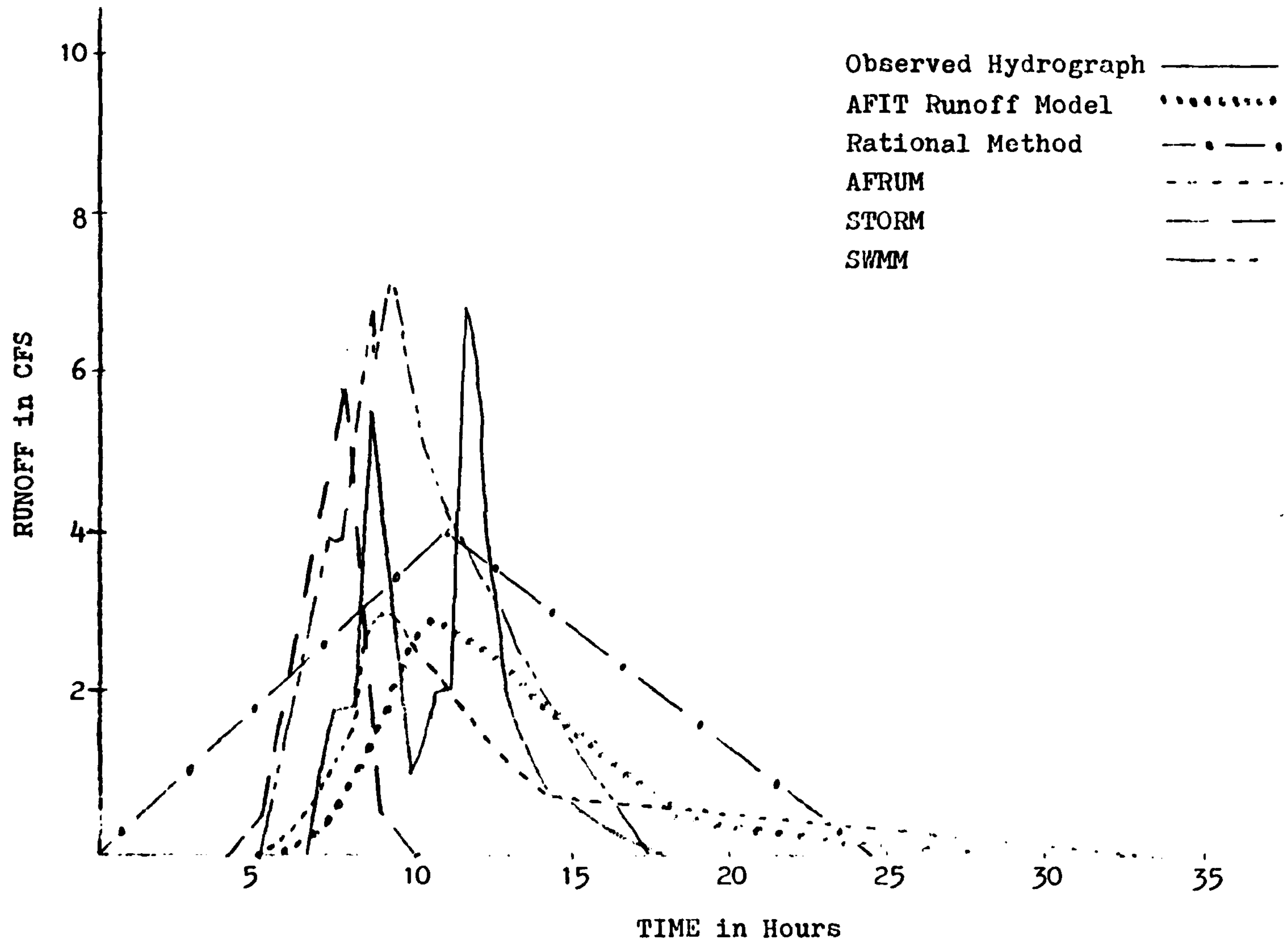
Underlined models are the same. Only the Rational Method predicted values that were not the same as the observed values.

APPENDIX G

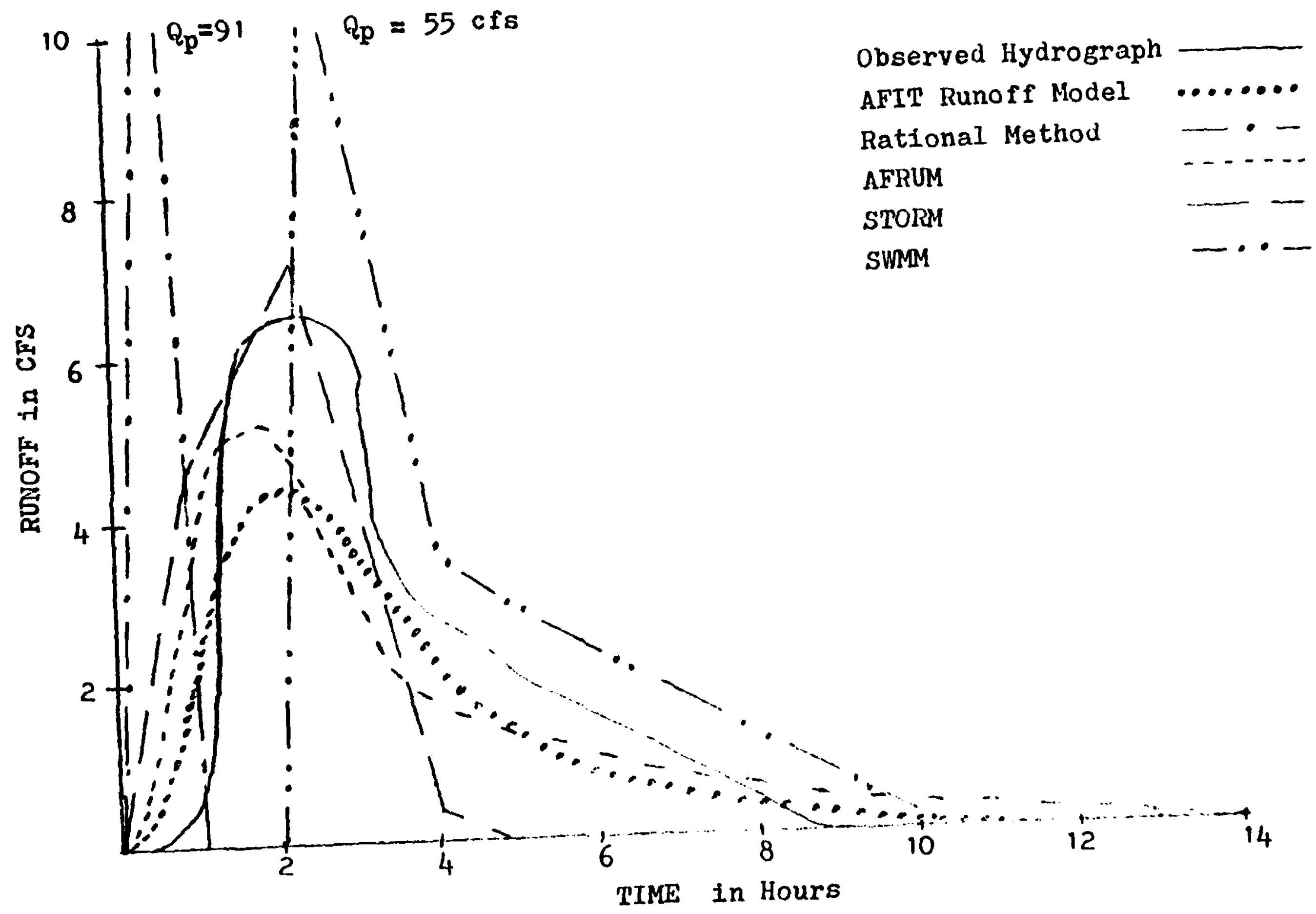
SIMULATED HYDROGRAPHS



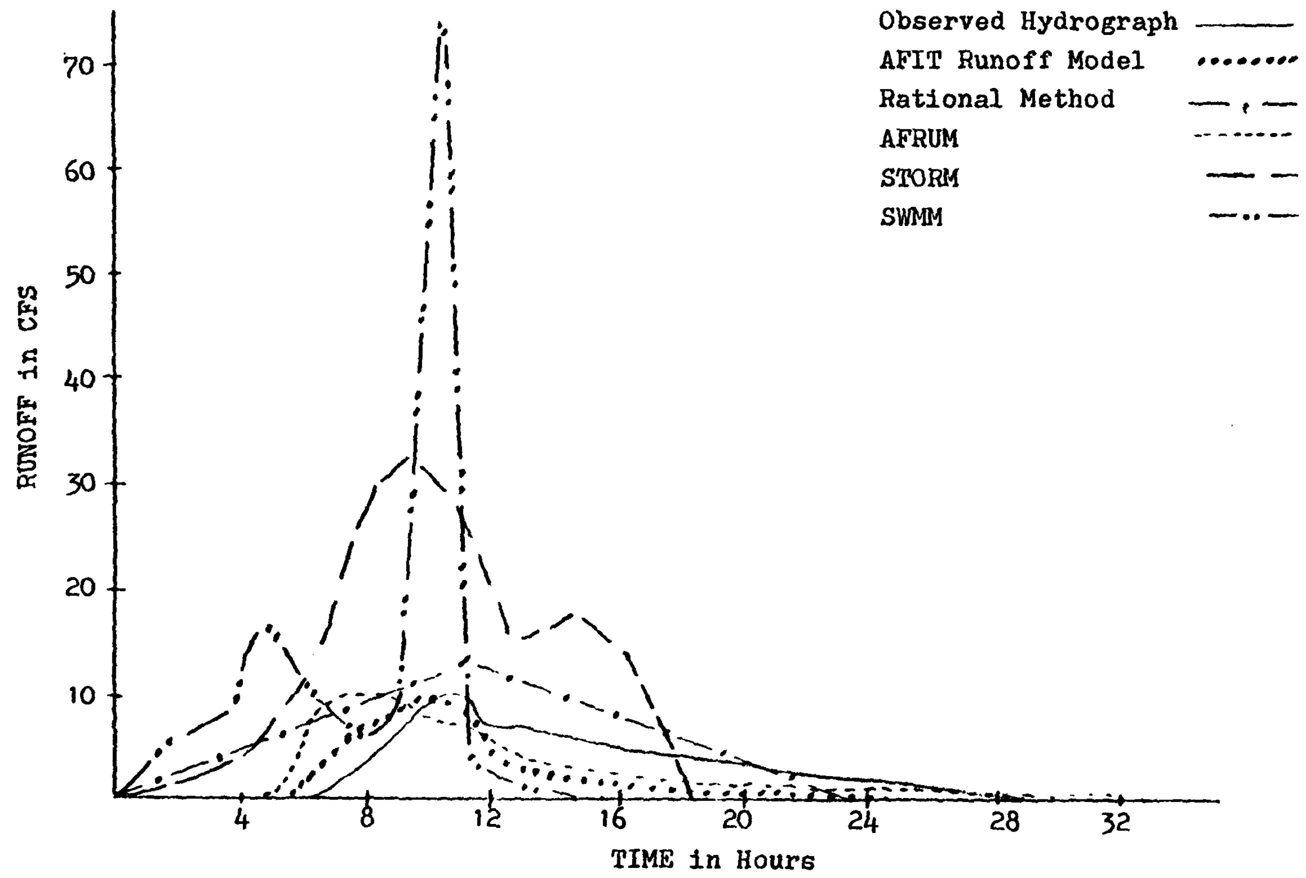
G-1 MCDOWELL DITCH 3 August 1978



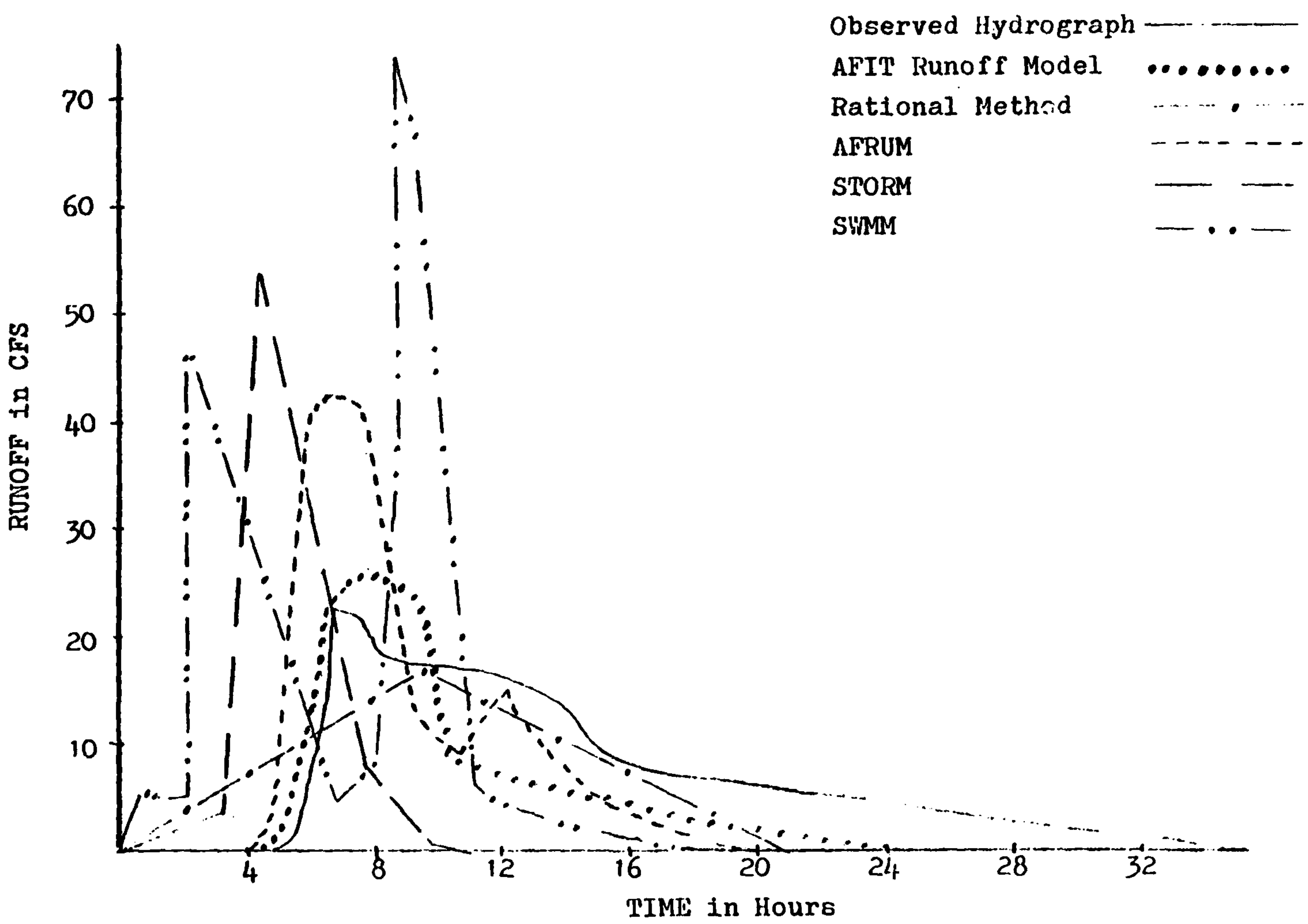
G-2 MCDOWELL DITCH 27 August 1978



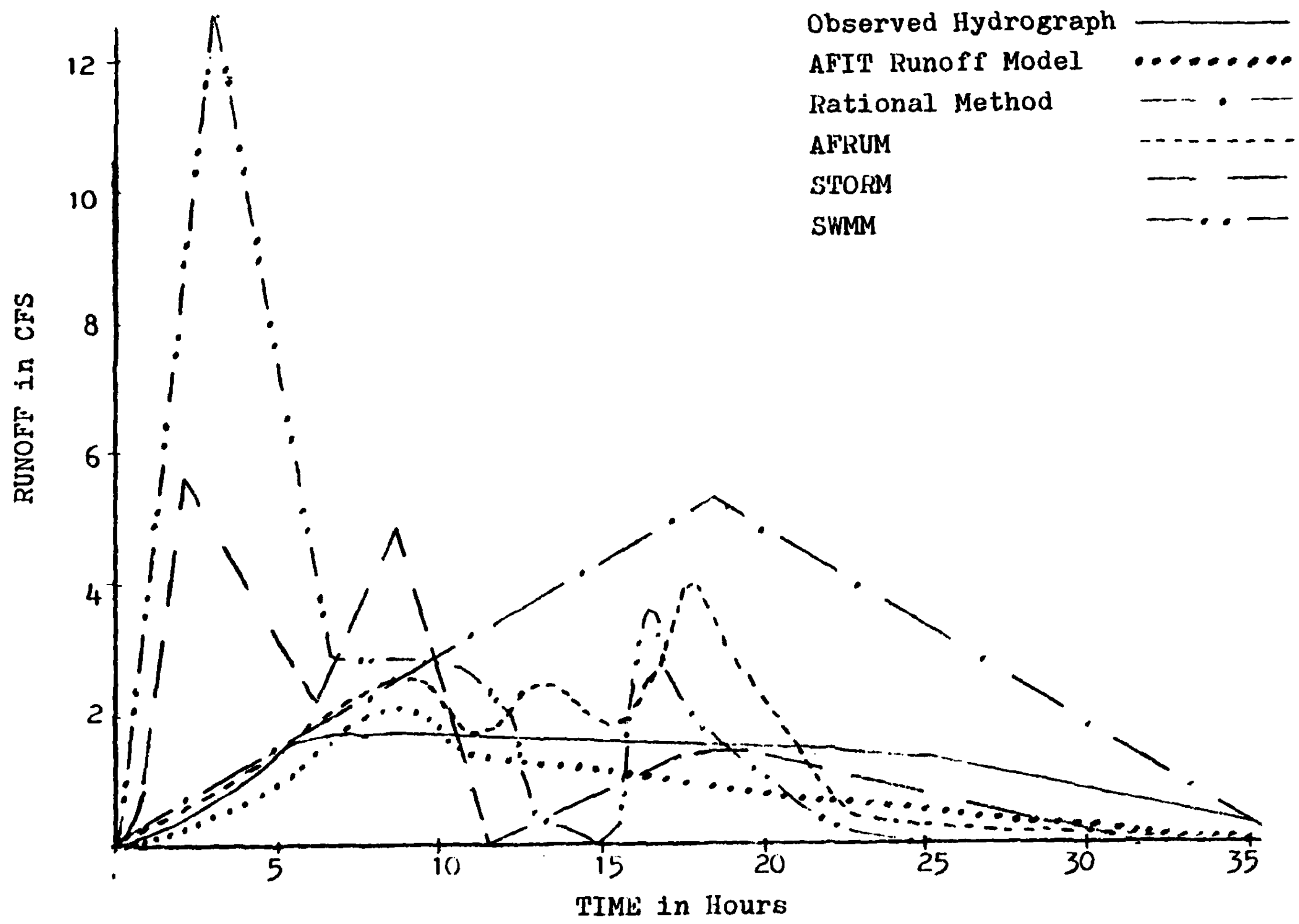
G-3 MCDOWELL DITCH 15 September 1978



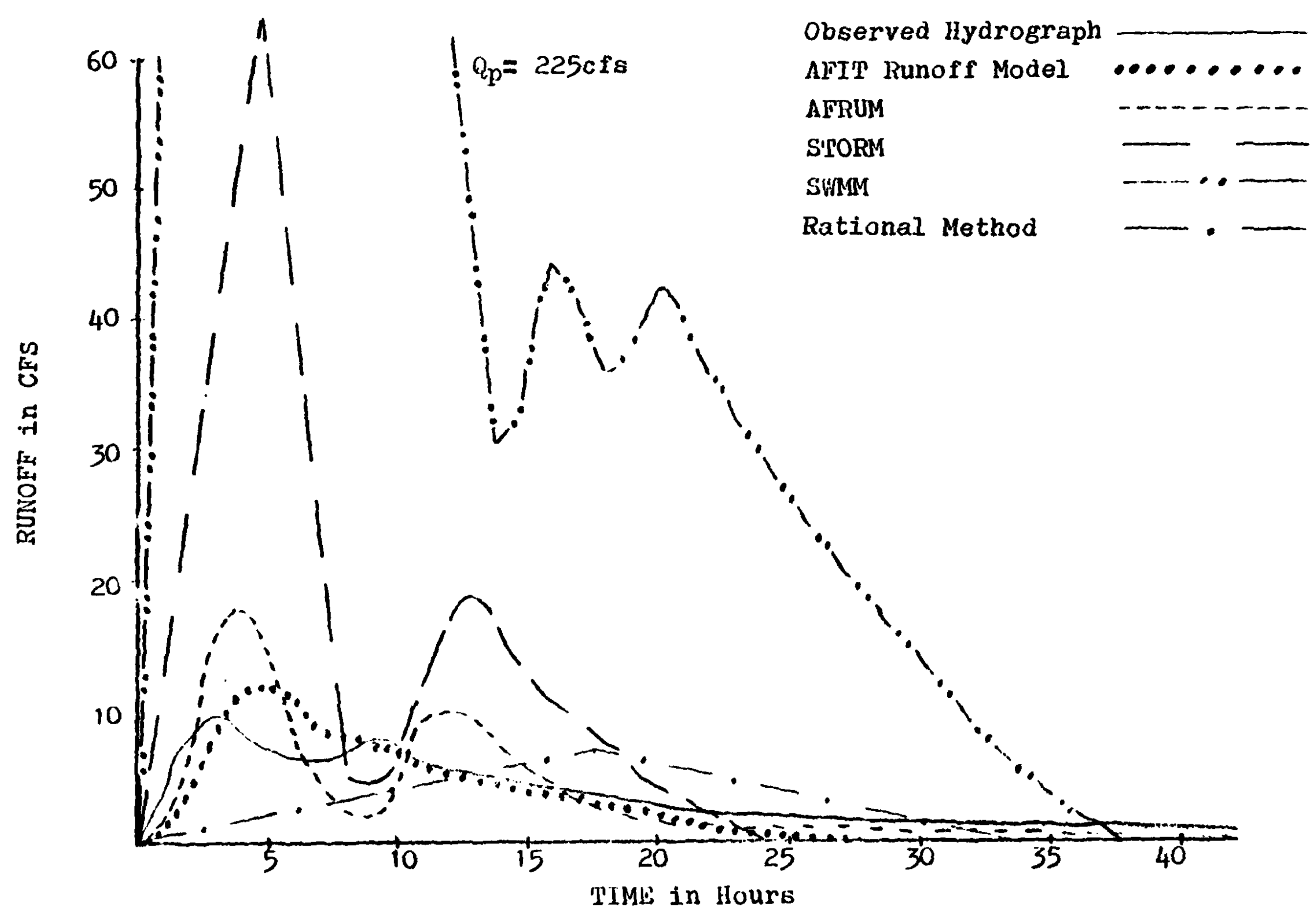
G-4 MCDOWELL DITCH 14 November 1978



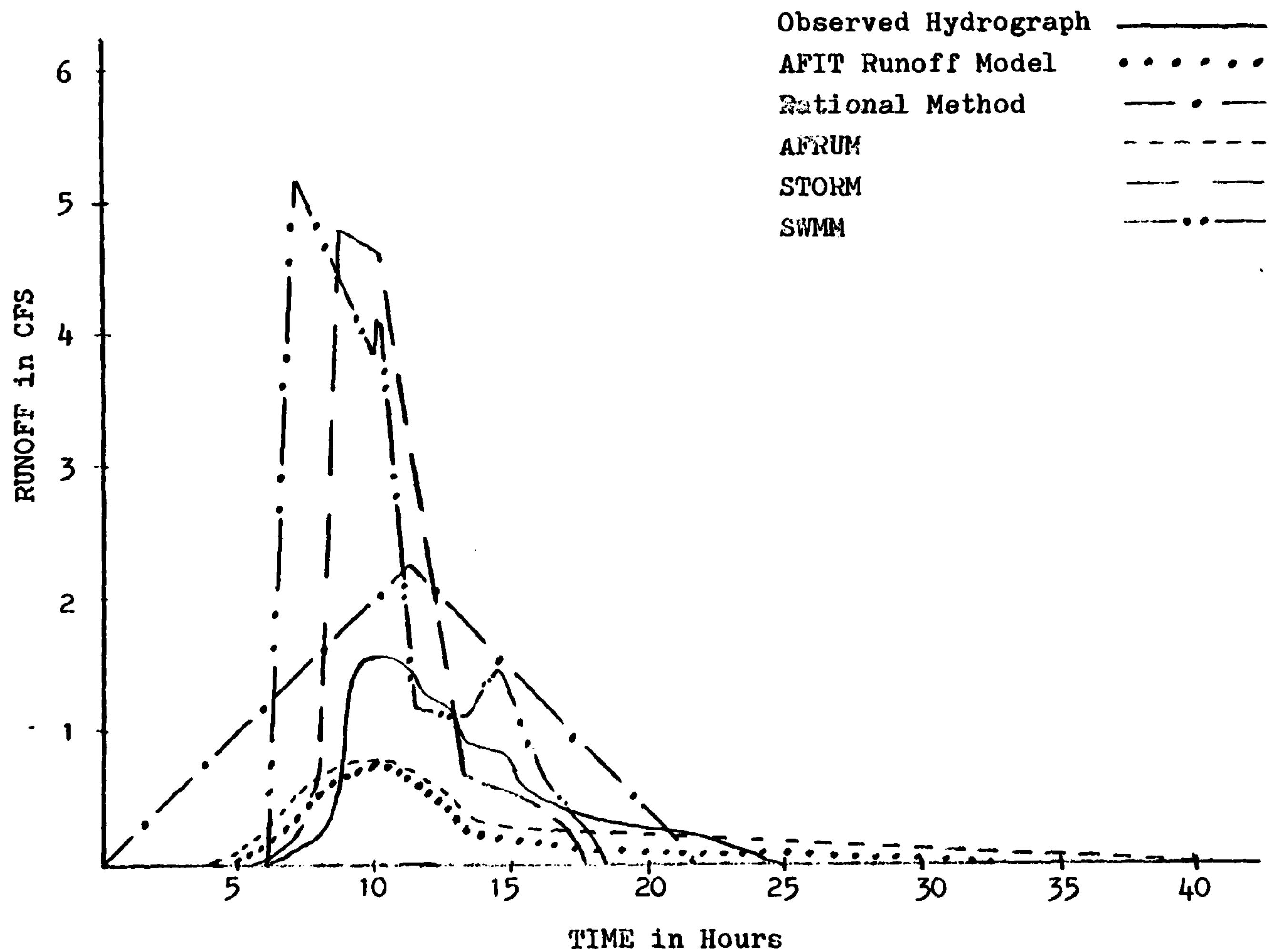
G-5 MCDOWELL DITCH 3 December 1978



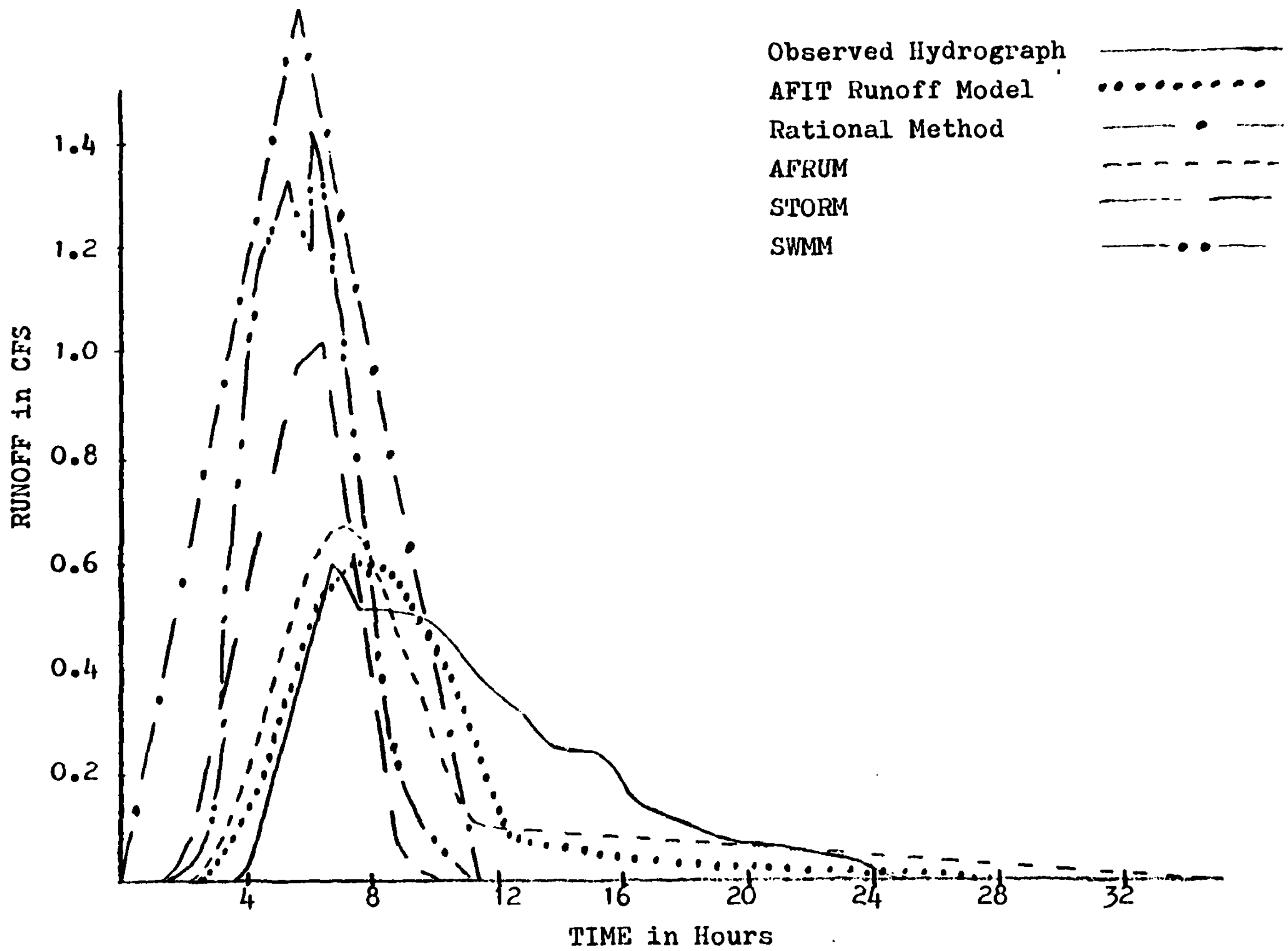
G-6 MCDOWELL DITCH 11 April 1979



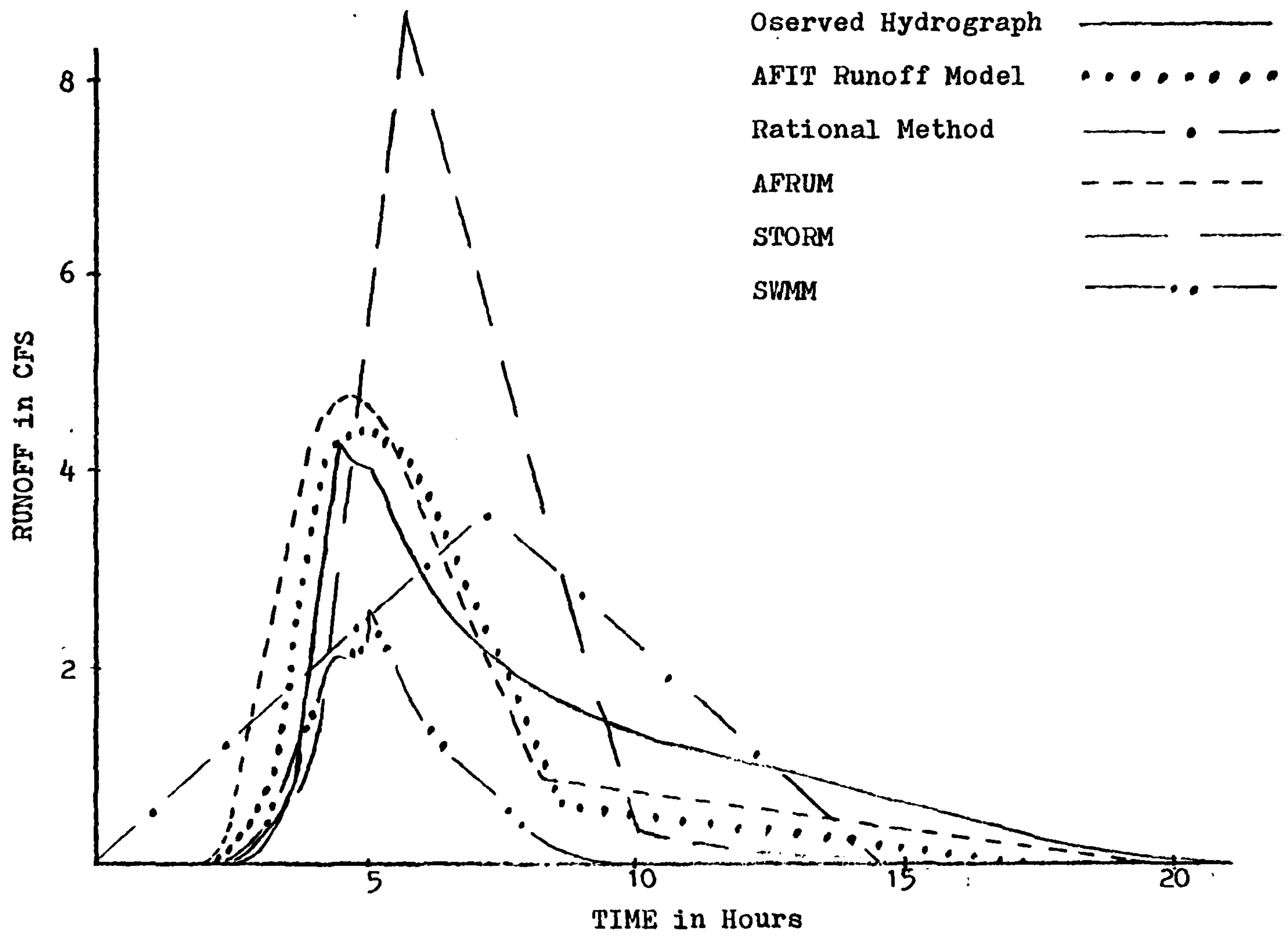
G-7 MCDOWELL DITCH 8 July 1979



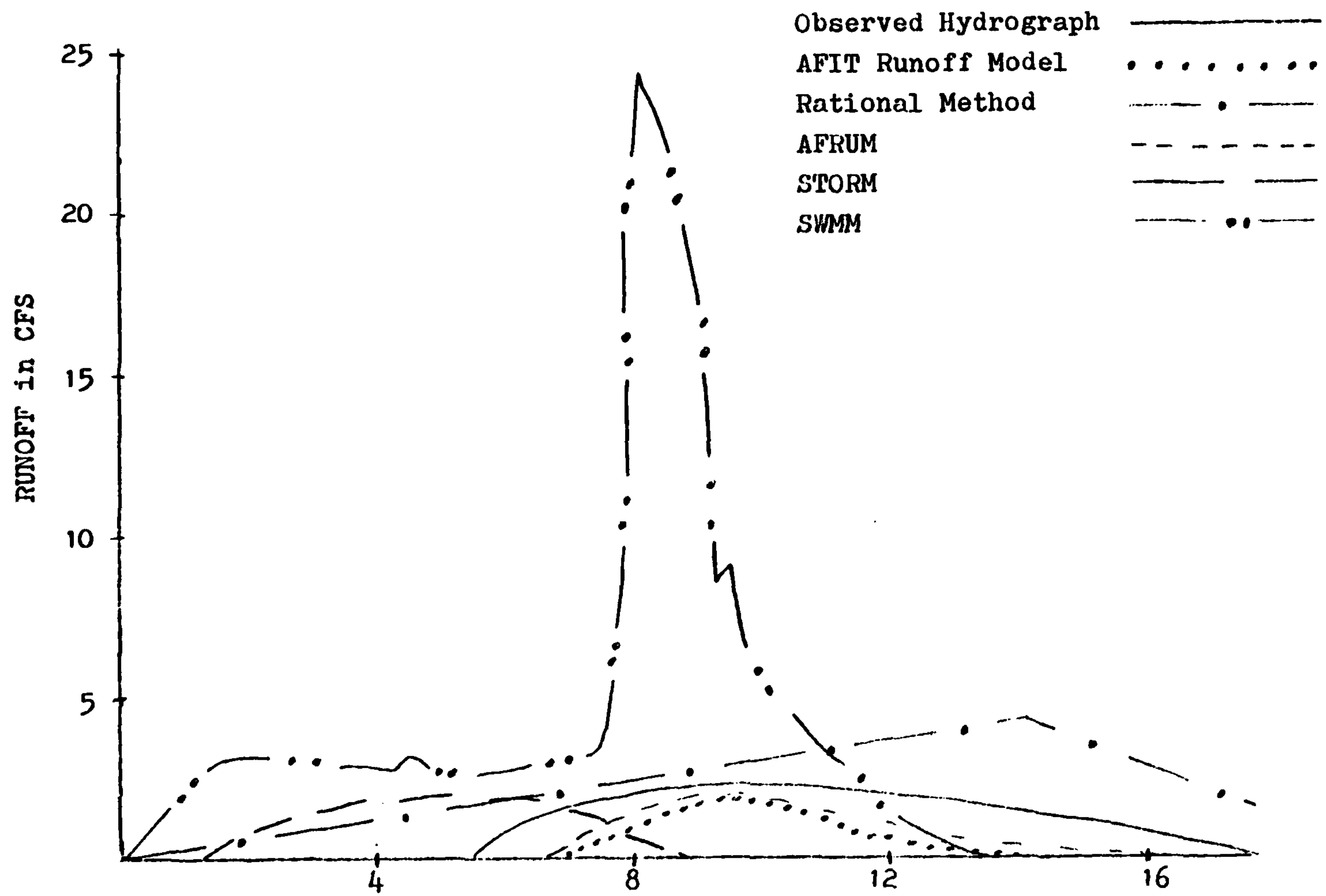
G-8 EAST DITCH 14 November 1978



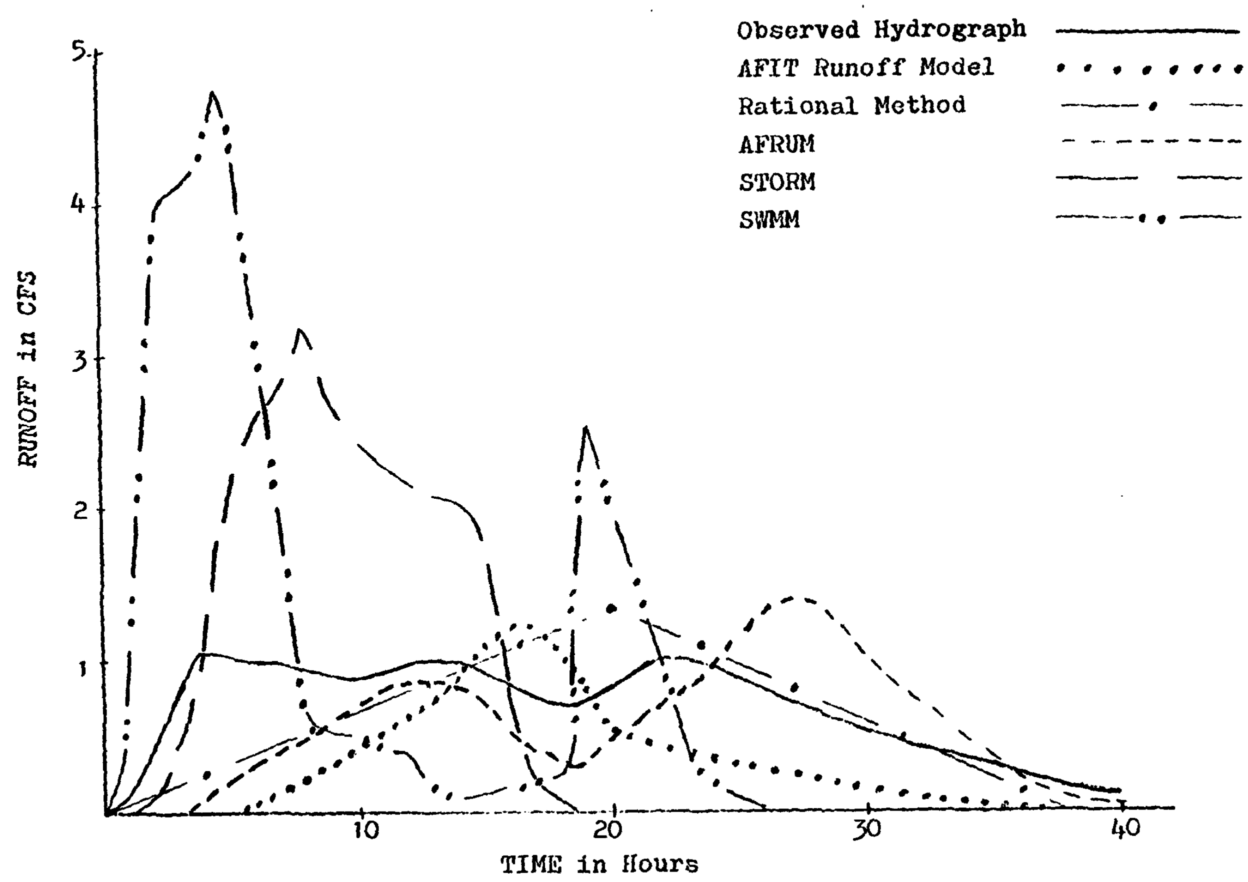
G-9 EAST DITCH 17 November 1978



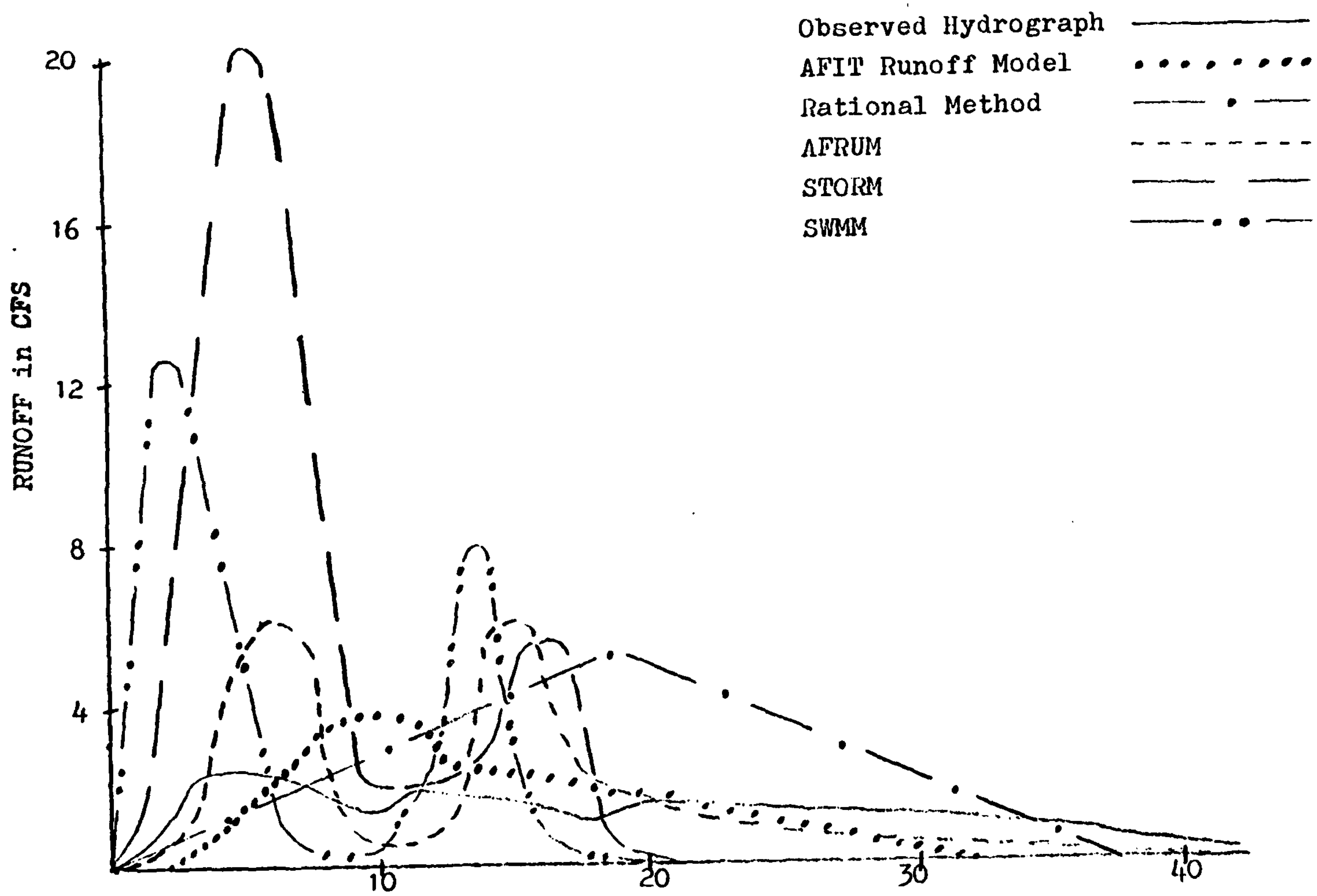
G-10 EAST DITCH 3 December 1978



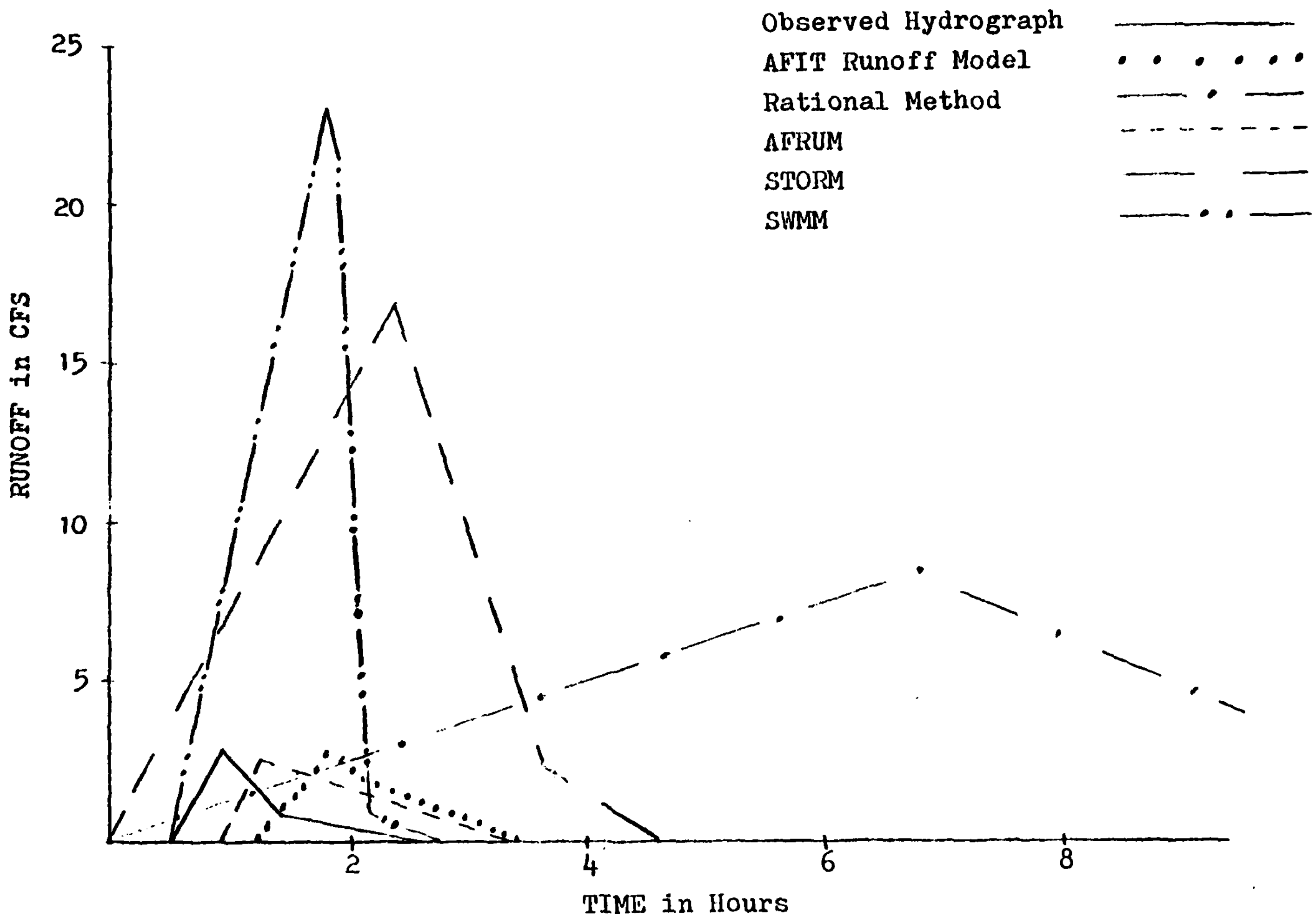
G-11 EAST DITCH 7 December 1978



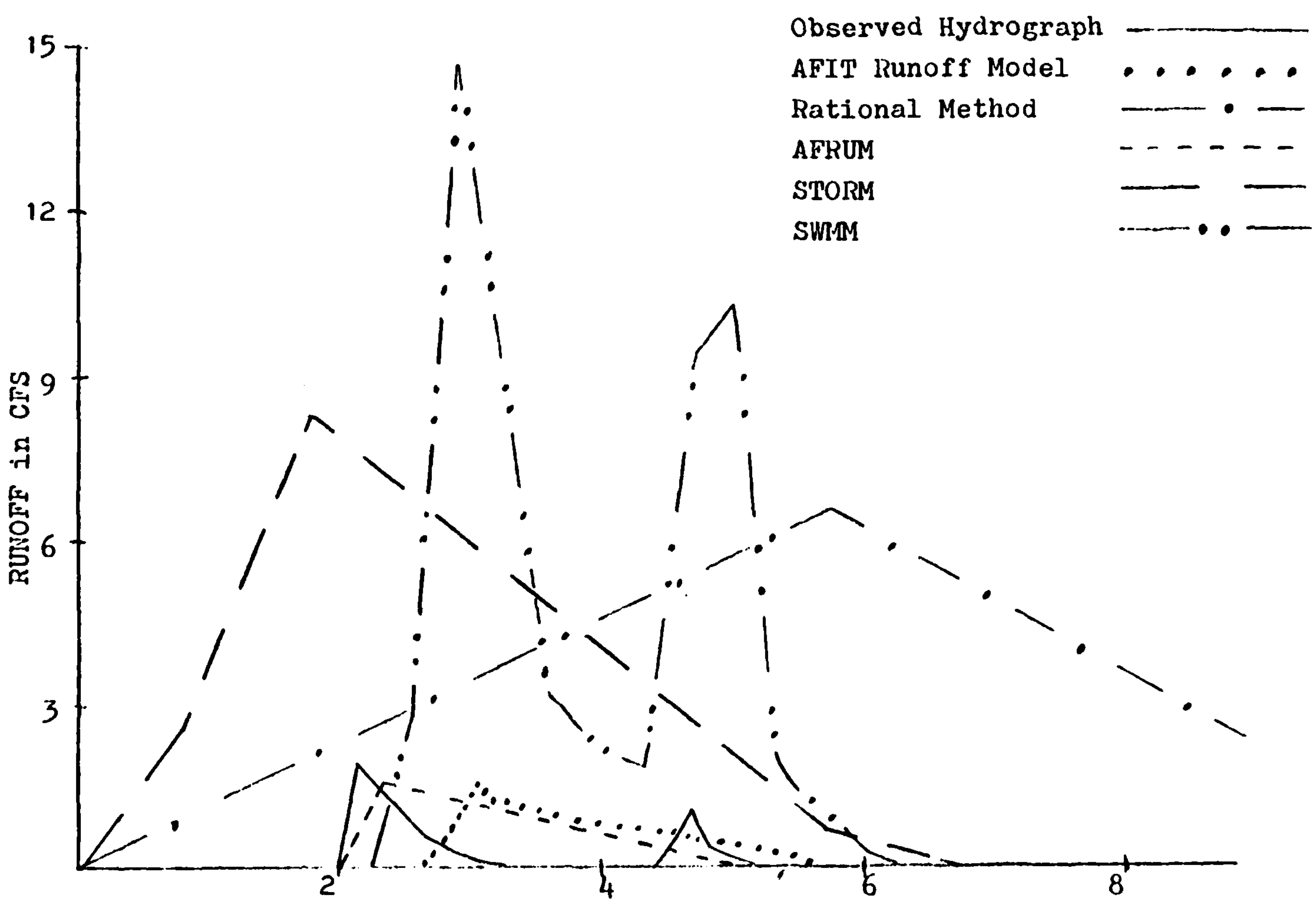
G-12 EAST DITCH 11 April 1979



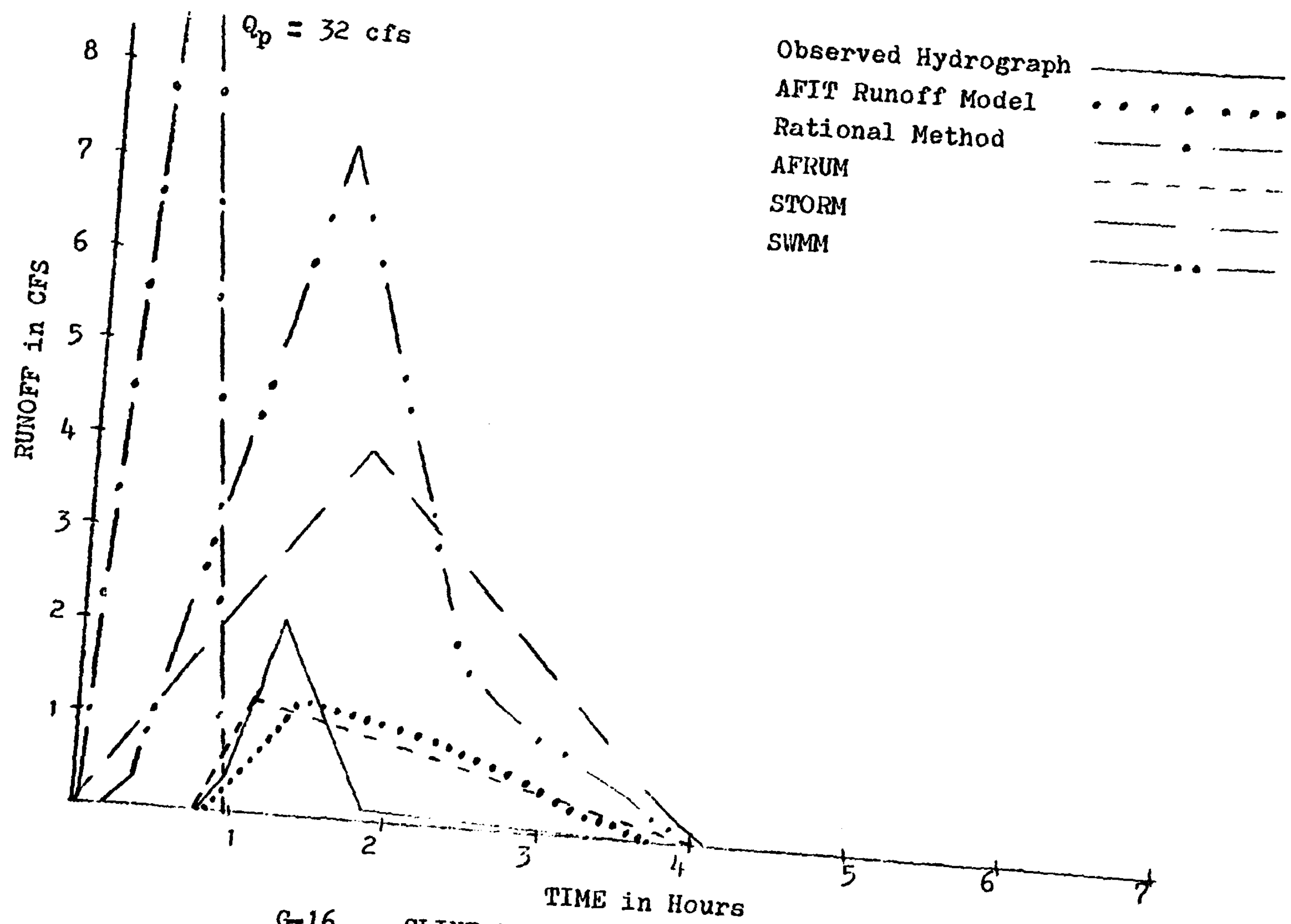
G-13 EAST DITCH 8 July 1979



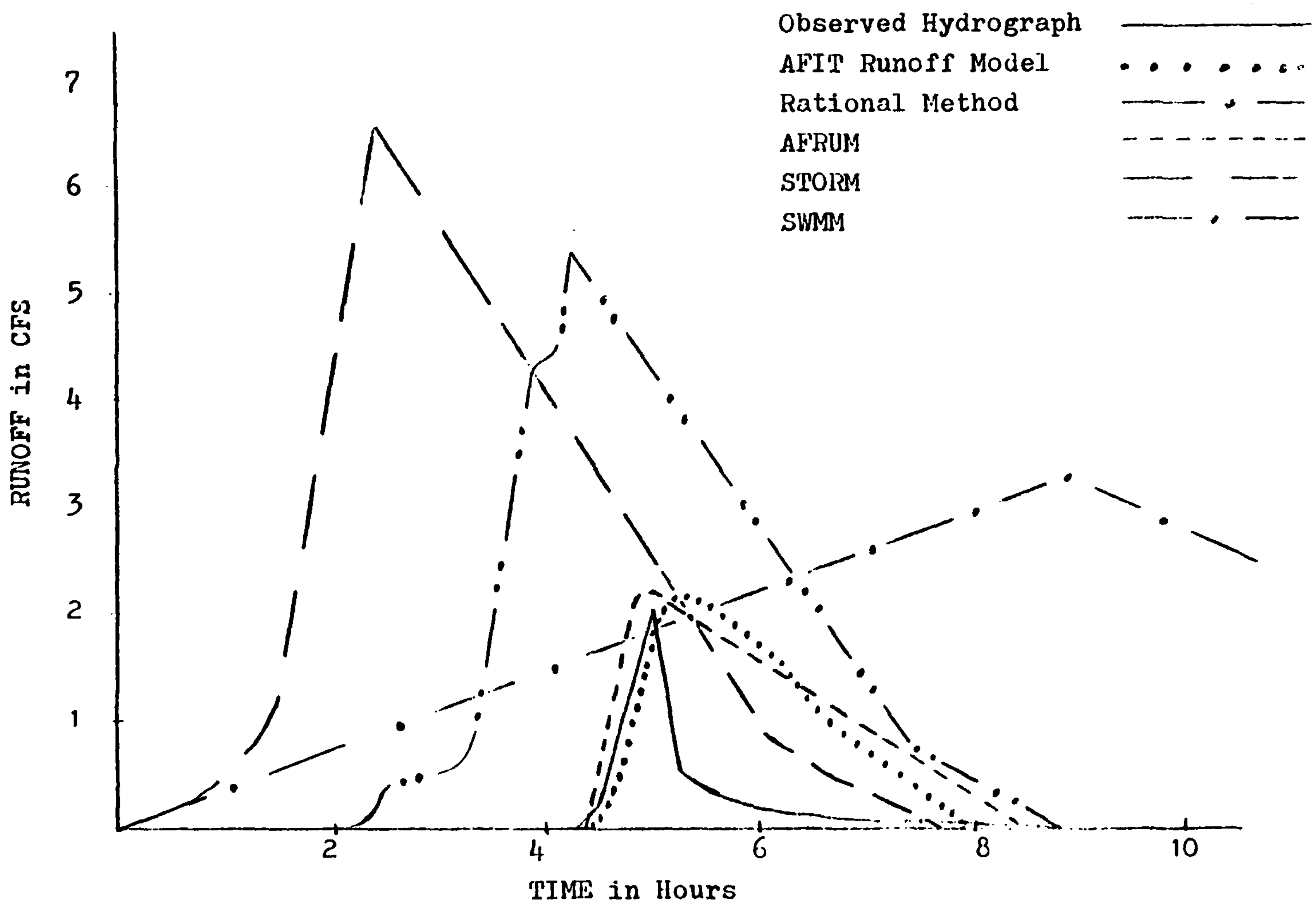
G-14 CLINE DITCH 2 August 1978



G-15 CLINE DITCH 27 August 1978



G-16 CLINE DITCH 14 September 1978



G-17 CLINE DITCH 3 December 1978

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