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ESTIMATION OF RUNOFF PEAK RATES
AND
VOLUMES FROM FLATWOODS WATERSHEDS

Final Report

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Submitted to

South Florida Water Management District
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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Figure

ABSTRACT

Several methods of estimating stormwater runoff total volume and peak discharge are evaluated as to their performance on watersheds of Florida's Flatwoods Resource Area. Characteristics of these watersheds include extremely flat relief, sandy soils, dynamic water tables, and scattered wetlands. Data collected by the U.S. Geological Survey and South Florida Water Management District (SFWMD) from five small (20-3600 acres), agricultural watersheds (improved and unimproved pasture) served as the basis of evaluation. All total volume estimation techniques examined rely upon the SCS runoff equation. Best results were achieved with methods which included antecedent depth to the water table as a measure of watershed storage potential. A simplified water table dynamics model is also developed and compared to measured data. Runoff peak rate estimation techniques ranged in approach from empirical formulas to an overland flow simulation model. For the original methods examined, standard errors of estimate were inversely proportional to model sophistication. Two peak rate estimation methods, the CREAMS hydrologic model equation and the SCS unit hydrograph method, were modified to better reflect observed data.

CHAPTER I

INTRODUCTION

Many techniques have been developed to estimate stormwater total runoff volume and peak discharge rates from small watersheds. However problems arise when these methods are applied to the unusual hydrologic conditions found in Florida's Flatwoods Resource Area. Watersheds of this area typically have very flat slopes, extremely permeable sandy soils, high water tables, and wetlands scattered throughout their basins. Such characteristics are unlike those of the watersheds which served as the models for the development of most runoff prediction methods. The problems introduced by these atypical watershed conditions are often compounded when the methods are called upon to predict runoff resulting from rainfall events for which they were not intended i.e., frequent, instead of extreme (design), events.

Studies which document the accuracy of standard runoff prediction techniques as applied to Florida's flatwoods watersheds under a range of rainfall events are not currently available. Hydrologists, engineers, and water resource managers are therefore forced to make decisions based upon runoff estimates resulting from methods which, although generally accepted, are not necessarily accurate under these particular watershed conditions. The users often appreciate the errors and limitations associated with their runoff estimates, but do not have sufficient information with which to offer improvements. The research described in this report represents an effort to help fill the existing information gap.

Objectives

The purpose of this study is to evaluate and offer improvements to runoff estimation techniques as applied to small watersheds in Florida's Flatwoods Resource Area and was initiated with the following objectives:

A. Evaluate runoff peak rate estimation methods currently in use for flat, high-water-table watersheds using observed data collected as part of the Kissimmee Coordinating Council Upland Detention Demonstration Project (SFWMD, 1980),

B. Modify an existing peak rate estimation method, if necessary, to improve its predictive ability under the above watershed conditions,

C. Test the modified peak rate estimation methods using adequate observed data to demonstrate their improved performance, and

D. Re-examine total volume estimation techniques as analyzed and modified by Konyha et al. (1982) using data not previously available.

CHAPTER II

LITERATURE REVIEW

Hydrologic Modeling Approaches

Today, hundreds of models are being used to aid in the solution of hydrologic problems. Overviews of various hydrologic computer models, their capabilities, approaches, and limitations have begun to appear in an attempt to help potential users wade through this avalanche of model development. Sources of such summaries are Fleming (1975), Renard et al. (1982), Huber and Heaney (1982), El-Kadi and van der Heijde (1983), and OTA (1982). Many of the hydrologic models differ in their scope i.e., definition of the system. One reason for the number of models in existence today is the wide range of objectives which need to be met. Many models are site specific, containing simplifications and assumptions which preclude their universal use (Renard et al., 1982).

Among these many models are certain similarities, common approaches and general hydrologic process considerations. Of the seventy-five computer models surveyed by Renard et al., 67% contained components to address the process of surface runoff. Surface runoff can be defined as that portion of rainfall excess which, during and immediately following a storm event, ultimately appears as flowing water in the drainage network of a watershed. This flowing water may arrive either by overland or subsurface routes (Huggins and Burney, 1982).

Certain elements are basic to the modeling of the rainfall-runoff process. Among these processes are precipitation, infiltration, evapotranspiration, and surface and subsurface flow routing. The degree to which each is accounted for depends upon the type of model or submodel being used. The following sections present basic elements of runoff models and several specific modeling techniques.

Elements of the Rainfall-Runoff Process

Viessman et al. (1977) represent the hydrologic balance for a watershed with the following equation:

$$P - R - G - E - T = \Delta S \qquad [1]$$

where P = precipitation input,
R = net surface outflow,
G = net groundwater outflow,
E = evaporation losses,
T = transpiration losses, and
ΔS = change in watershed storage.

If no surface or groundwater inflows are assumed and these two outflow terms are combined (RO) as are evaporation and transpiration (ET), then Equation 1 becomes:

$$P - ET - \Delta S = RO \qquad [2]$$

For the Florida flatwoods watershed, this is a reasonable mass balance model. Knisel et al. (1978) point out that groundwater deep percolation is small for the watersheds of the Tayor Creek/Nubbin Slough Basin. Geologic assessment indicated that the Hawthorne Formation serves as a floor for the unconfined groundwater of the area. Furthermore, when the water budget is considered on a single 24-hour rainfall event basis, deep percolation as well as ET become negligible. In general, an effective rainfall-runoff model should contain elements to address the terms of this mass balance equation: rainfall (P), evapotranspiration (ET), infiltration and surface storage (ΔS), and elements of surface and subsurface routing to describe the time-distribution of runoff (RO).

Precipitation

Precipitation, rainfall in this case, is the basic input to most runoff models (Osborne et al., 1982). Important factors are the rainfall magnitude and its time-space distribution. The modeling of these factors is generally approached stochastically, but can be handled deterministically. Echternacht (1982) points out that much more research will be required before physically based meso-scale meteorological models for South Florida are realized. General approaches to the stochastic modeling of rainfall have been surveyed by Osborne et al. (1982).

To address the magnitude of rainfall events, the Soil Conservation Service (SCS) has published a depth-duration-frequency atlas for the Southeast (USDA-SCS, 1979). SFWMD (1981) reports similar information developed more recently and specific to Central and South Florida. Table 1 presents information for the Lower Kissimmee River Basin derived from both these sources.

Rainfall time-depth distributions have been developed for design rainfall events in Central and South Florida (SFWMD, 1983). The SCS also reports a similar distribution specifically intended for Florida. Figure 1 is a graphical representation of the two distributions and the SCS Type II distribution (USDA-SCS, 1972b). The SCS-Florida curve presented here was developed by plotting decreasing 30-minute duration intensity ratios symmetrically about noon. These ratios are the accumulated rainfall total for a given time divided by the 24-hour total depth for the Florida interim distribution (USDA-SCS, 1980).

Table 1. Point rainfall depths, in inches, for storms of given duration and return periods for the Lower Kissimmee River Basin area from A (USDA-SCS, 1979) and B (SFWMD, 1981).

Duration	Rainfall Event Return Period, in Years						
	1	2	5	10	25	50	100
A							
30-Minute	1.5	1.7	2.1	2.3	2.7	2.9	3.3
1-Hour	1.9	2.2	2.7	2.9	3.3	3.7	4.0
2-Hour	2.3	2.7	3.3	3.8	4.3	4.8	5.3
3-Hour	2.5	2.9	3.7	4.3	4.8	5.4	5.9
6-Hour	2.9	3.5	4.5	5.2	5.9	6.7	7.5
12-Hour	3.4	4.2	5.3	6.3	7.5	8.5	9.0
24-Hour	3.8	4.7	6.3	7.5	8.5	9.5	10.5
B							
1-Day	3.6	-	4.5	5.0	6.0	7.0	8.0
2-Day	4.5	-	5.6	6.6	8.0	9.0	11.0
3-Day	5.3	-	6.5	7.5	9.0	10.0	11.5
5-Day	5.7	-	7.0	8.5	10.0	11.0	12.5

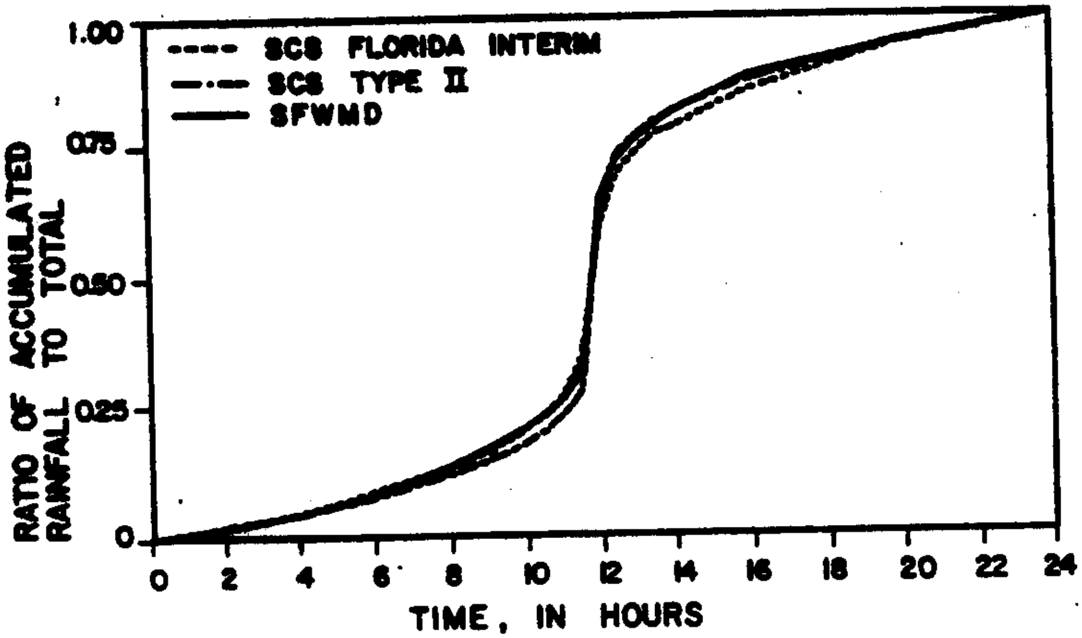


Figure 1. Time-depth distributions for 24-hour design rainfall events.

The depth-area distribution over a watershed can also be an important factor in estimating runoff, particularly for short-lived thunderstorms of limited areal extent (Osborne et al., 1980). Such thunderstorms are representative of rainfall patterns in Florida. Osborne and others (1980) presented criteria for establishing raingage networks and accounting for this spatial variability in areas where thunderstorms are prevalent. When measured rainfall from gaging locations is used as input to a rainfall-runoff model, techniques such as Theissen weighting or isohyetal mapping are often employed (Viessman et al., 1977). However for small watersheds, uniform areal rainfall depth is often assumed.

Infiltration

Infiltration is that process which will determine the amount and time-distribution of rainfall excess that is available for runoff and surface storage. The same properties which control infiltration will govern subsurface movement of water after infiltration (Skaggs and Khaleel, 1982).

The physical phenomenon of infiltration is described by Richard's Equation which combines Darcy's basic porous media flow equation with the conservation of mass principle as applied to the soil water system. Approximations of the infiltration process as quantified by Richard's Equation are numerous and include Holtan's Equation, Horton's Method, Philip's Equation, and the Green and Ampt Model. Another method used to account for infiltration, as well as all other abstractions in the rainfall-runoff process, is the SCS runoff equation (USDA-SCS, 1972b).

Significant characteristics influencing infiltration in Florida's flatwoods soils are their discontinuous hydraulic properties i.e., layered soils, high permeability of sandy surface layers, and high water tables. Recently several studies have been reported where infiltration models were applied to situations with one or more of these characteristics.

Bruce and Thomas (1983) report results of applying Richard's Equation and the Green and Ampt Model to layered soils. Shiromohamedi and Skaggs (1983a&b) have conducted infiltration experiments on sandy soils with high water tables. Included in their research were investigations into the effect of surface condition and entrapped air. The first study concluded that infiltration capacity increased with density of surface vegetation (bare, soybean, and grass) as a result of the interaction of vegetation and air movement. The second study involved modification of the Green and Ampt Model to account for air trapped between an advancing wetting front and a water table. Skaggs (1978a) pointed out that entrapped air is an important factor in sandy soils with high water tables and results in variable drainable porosity values. Research related to the modeling of water tables is described by King and Lambert (1976), Knoch et al. (1983), Decoursey et al. (1983) and Xue et al. (1983). These works address the additional problems of predicting water table movements and drainage.

Only limited studies have been conducted specifically on Florida flatwoods water tables and their effect upon infiltration and available ground storage. As part of a Taylor Creek Basin hydrologic study, Speir et al.

(1969) published curves describing changes in water table depths due to water losses and gains (see Figure 2). Also presented as part of this study is a regression equation describing water table response to rainfall at depths between 2.5 and 4.0 feet. The South Florida Water Management District (1983) published a general curve which determines available soil profile storage as a direct function of depth to the water table (see Figure 3). Observations by Parker (1982) noted that the degree of flooding resulting from tropical storms is highly dependent upon the water table depth preceding the event. He cites the low water table (6 feet) as the reason for the lack of a devastating flood which could have resulted from the 18+ inches which Hurricane Dennis dumped upon south Dade County in 1980.

The recent research into applications of Richard's Equation and the Green and Ampt Model is important to models which consider the infiltration process in detail. However, El-Kadi and van der Heijde (1983) pointed out that the technique most often used in general watershed hydrologic models to account for infiltration is the SCS runoff equation and curve number method (USDA-SCS, 1972b).

Among the reasons why the SCS approach is so widely used is its simplicity and range of application. Rainfall-runoff models based upon infiltration equations require components to account for other abstractions in the process i.e., interception and surface storage. All these abstraction terms contribute to the storage term of Viessman's model (ΔS). However the SCS runoff equation (described later) lumps all abstractions in a different model formulation.

Brakensiek and Rawls (1982) present an excellent qualitative comparison between the infiltration approach to runoff modeling and the curve number approach. Recently, an attempt has been made at combining elements of these two approaches into a new model (Chu and Engman, 1983). Huddleson et al. (1983) also reformulate the SCS equation into an intensity-dependent function and further into a Darcy-type infiltration equation. Such models will require further research before general application.

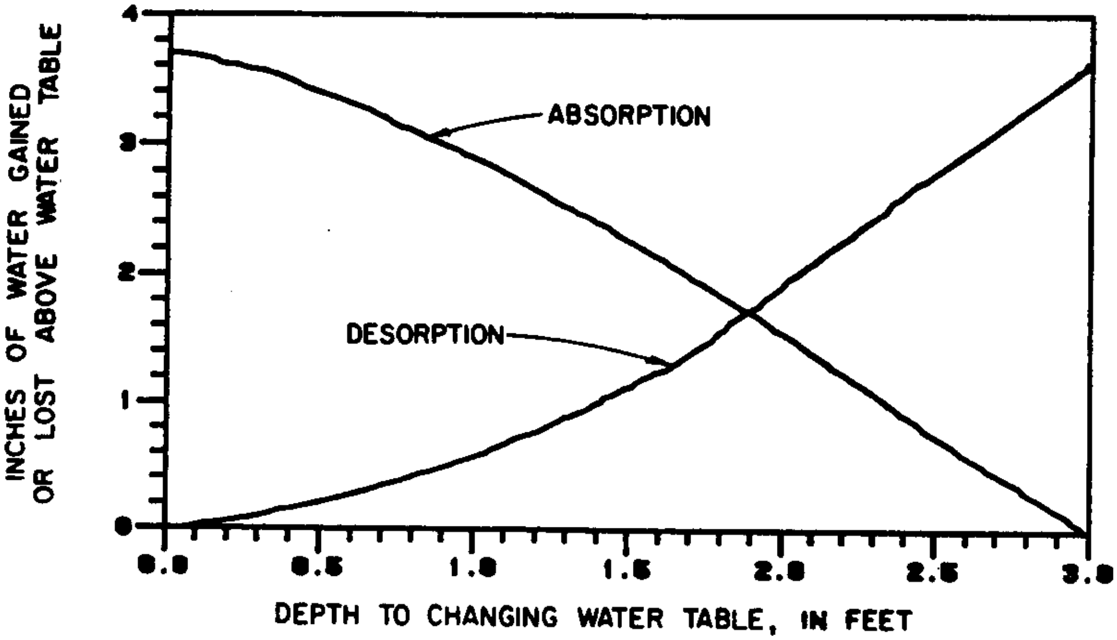
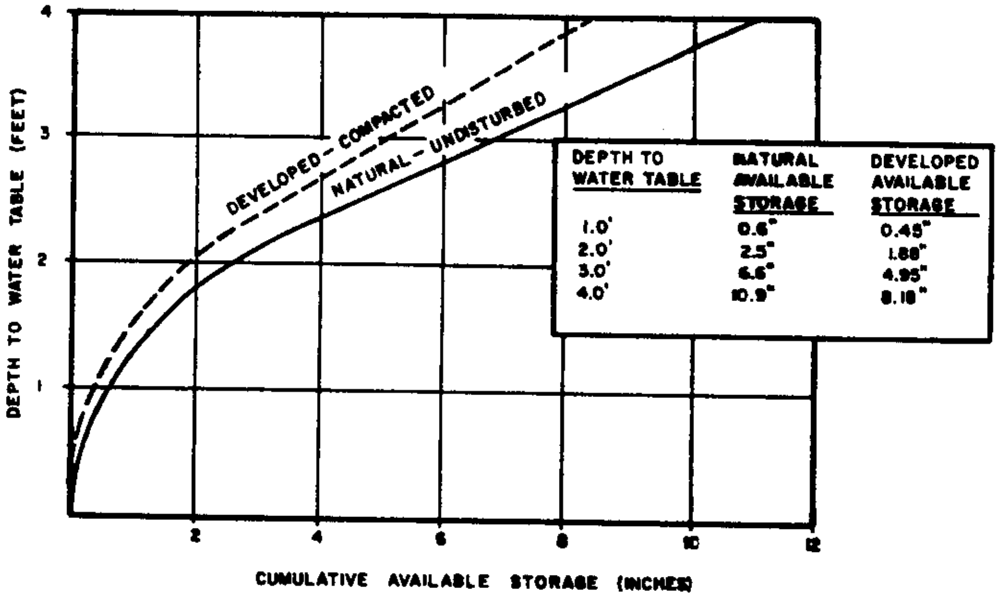


Figure 2. Absorption/desorption characteristics for sandy soils of the Taylor Creek area (Speir et al., 1969).



DEPTH TO WATER TABLE	NATURAL AVAILABLE STORAGE	DEVELOPED AVAILABLE STORAGE
1.0'	0.6"	0.45"
2.0'	2.5"	1.88"
3.0'	6.6"	4.95"
4.0'	10.9"	8.18"

Figure 3. Available soil profile storage for natural and developed watersheds of South Florida (SFWMD, 1983).

Evapotranspiration

Evapotranspiration (ET) is the process which returns water to the atmosphere either by evaporation from free water and ground surfaces or by transpiration from plants. Several techniques are available for the estimation of potential ET (that ET rate which would occur given sufficient water availability). Such techniques include the Penman, Thornthwaite, Blaney-Criddle, Jensen-Haise, Stephens-Steward, and radiation methods (Clark and Smajstrla, 1984). All are parametric models based upon various combinations and formulations of ET driving factors (radiation, temperature, wind speed, and vapor pressure). Shih et al. (1983) reported that for South Florida a modified Penman technique gave good results. Alternatively, potential ET may be determined from measured pan evaporation. Smajstrla et al. (1983) published an ET summary for Florida based upon such data.

Potential ET derived from models or data must be further modified to arrive at estimates of the actual ET rate. Physically based models and empirical techniques are available which limit the potential rate with moisture and crop condition factors (Burman et al., 1982). Skaggs (1978b) reports on a computer model, DRAINMOD, capable of using soil properties and water table depths in the determination of actual ET.

Allen (1982) observed that in Florida, ET is the most constant component of the annual hydrologic cycle, but that it does vary depending upon rainfall. For the Kissimmee River Basin and South Florida in general, a minimum of 30-35 and 35-40 annual rainfall inches, respectively are required before appreciable runoff will be observed (Huber, 1982). Heaney (1982) and Allen (1982) report 32-34 and 32-35 inch threshold values for the Lower Kissimmee River and Taylor Creek Basins respectively. Speir et al. (1969) presents a curve which indirectly describes the ET demand upon water stored in the Taylor Creek Basin. The curves in Figure 4 present the general water table recession characteristics for flatwoods soils of the area. Given Speir and others observations of the aquiclude underlying the groundwater and a lack of runoff with water table depths greater than 2.5 feet, this deeper recession can be attributed to ET extraction.

Surface Runoff Routing

The estimation of surface runoff, as defined earlier, requires the consideration of overland, subsurface and channel flow regimes. The flow characteristics of these transport mechanisms will dictate both the quantity of excess rainfall which appears at the watershed outlet and its time distribution with respect to that point. Following is a review of methods used to quantify surface runoff which consider the processes individually and in lumped or approximate fashions.

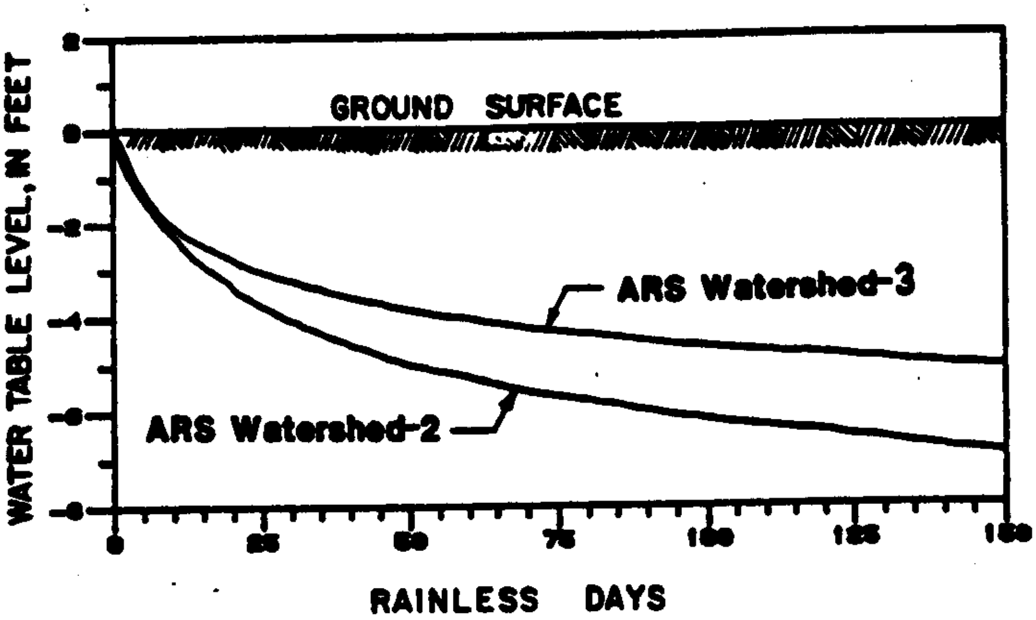


Figure 4. Water table recession curves (Speir et al., 1969).

Process Techniques

Overland. Huggins and Burney (1982) emphasize the importance of overland flow in models used for small watersheds. They explain that as the size of the watersheds decreases, the dominant flow regime shifts from channel to overland.

The basis of routing methods are the concepts of conservation of mass and conservation of momentum, commonly referred to as the St. Venant Equations. Because the implementation is complicated, requiring much data and computing capacity, the kinematic approximation is often applied (Fleming, 1975). Such an approach neglects dynamic terms, like backwater effects, in the momentum equation and reduces the overland solution to the form:

$$q = a\, y^{m} \qquad [3]$$

where q = discharge rate,
m = 1.67 (for turbulent flow using Manning's Equation),
y = flow depth, and
a = a flow parameter.

The other component of the model, continuity, involves accounting for watershed inflows, outflows, and storage characteristics. Various levels of implementation of these submodels have been used. Usually they vary in the degree of discretion. Heatwole et al. (1982) report an application of the FESHM (Finite Element Storm Hydrograph Model) model which uses finite element techniques to introduce areal distribution of the flow parameter and calculates continuity on a small elemental basis. Other models approximate the watershed as a single flow plane utilizing lumped flow parameters and continuity calculated on a watershed scale (Huggins, 1976). Still other methods use overland theory as the basis for arriving at watershed time parameters for use in more simple routing approximations (Gregory, 1982).

Subsurface. Given the high soil permeability, low relief, and high water tables of flatwoods watersheds, subsurface flow may be the mechanism by which significant quantities of runoff arrive in the drainage system. Huber et al. (1976) pointed out that data are not available to allow the partitioning of observed runoff quantity between the overland and subsurface regimes. For the Southeast Coastal Plain with which the Florida flatwoods are generally associated, Knisel (1980) estimates that 80% of streamflow has at one time been subsurface flow. Models designed to aid in answering such questions of flow paths are available (Skaggs, 1978b), however studies describing their application to flatwoods soils have not been conducted. Subsurface flow models are based upon Darcy's equation and given a drainage system, the Van Schilfgaarde equation (1974).

Channel. On small flatwoods watersheds, channel flow can be a nebulous term. Channels are often wide, shallow, heavily vegetated, and have very little slope. They often operate under backwater conditions and at velocities less than 0.1 feet per second (Mierau, 1981). Because of these conditions, it is difficult to apply the standard techniques of channel routing.

Approximate Techniques

Because of the difficulty in separating and describing flow regimes and the effort and expense involved with defining parameters and implementing their routing techniques, simpler, more approximate routing methods are often employed. Such techniques include parametric regression equations, unit hydrograph methods, and linear reservoir models.

Linear Reservoir. The concept of linear reservoir routing is based upon a direct relationship between storage and discharge. This concept can be extended to "n" linear reservoirs, each discharging into the next, and the outflow hydrograph of the last being observed. This scheme is often referred to as the Nash model. The flexibility introduced by the variable "n" allows its application to small urban areas as well as large flat agricultural watersheds (Huggins and Burney, 1982).

Regression Equations. Parametric routing equations typically yield a peak discharge as the function of rainfall excess and watershed characteristics. Examples of these are the Cypress Creek Formula (Speir et al., 1969) and an equation contained in the CREAMS hydrologic model (Knisel, 1980). Both equations will be discussed in detail later. Huggins and Burney (1982) explain that small agricultural watersheds tend to have "noisy" hydrographs (multiple peaks). They conclude that the smaller watersheds are not amenable to transfer function analysis due to their lack of modulating influences (reservoir effects). However, for the flat, high-water-table watersheds of Florida, this may not be a reasonable conclusion.

Bridges (1982) describes three regionalized peak discharge equations for Florida and reports a standard error of estimate of about 50% overall. The equation applicable to Central and South Florida is:

$$Q_T = C(DA^{B1})(SL^{B2})(LK+3)^{B3} \qquad [4]$$

where Q_T = peak discharge for a runoff event of T return period,

DA = drainage area,
SL = channel slope,
LK = percent lakes, and
C, B1, B2, B3 are regression values for the runoff event of T return period.

The formulation of this model is "black box" ie., no physical relationships are implied by the equation formulation. In fact, rainfall is only an indirect input related to the return period of the storm. The parameters and regression values are those values which Bridges found most accounted for the variability in the available data base. Parametric equations of the stochastic variety, like this one, are simple and can be useful for certain applications (Haan, 1977).

Unit Hydrographs. The basis of unit hydrograph theory is that for a given duration of rainfall excess and constant land use and watershed conditions, response to a unit rainfall input will be constant. The validity of its application is also dependent upon the assumption of watershed linearity ie., that the superposition principle is valid for runoff. Overton and Meadows (1976) cite numerous studies which concluded that this is not the case. Still, unit hydrograph theory and application remain a basic tool in estimating runoff. Unit hydrographs represent the discharge pattern resulting from a rainfall excess of one depth unit applied uniformly to the watershed over a given time span. Synthetic unit hydrograph curves are typically approximated by a single triangle (SCS approach) or multiple triangles (TVA approach). The multiple triangle scheme has the advantage of accounting for differing initial and delayed response characteristics (Overton and Meadows, 1976). Both approaches require the estimation of time parameters which dictate the shape and relative peak of the hydrograph. Studies into such topics are presented by Duru (1980), Welle et al. (1980), and the U.S. Army Corps of Engineers (1955).

Specific Rainfall-Runoff Techniques

As shown in the preceding dicussion, a variety of approaches and levels of implementation are available to estimate stormwater runoff volumes and rates on an event basis. Following are descriptions of specific techniques commonly employed to perform this task.

Storm Runoff Volume

As an alternative to the infiltration-based models, the Soil Conservation Service developed a method which lumps all significant losses in the rainfall-runoff process into a single equation. The basis of this method developed by Mockus (USDA-SCS, 1972b) is the relationship:

$$\frac{F}{S'} = \frac{Q}{P} \qquad [5]$$

where F = actual retention (rainfall not appearing as runoff),
S' = potential maximum storage,
Q = actual runoff, and
P = potential maximum runoff (total rainfall).

If actual retention, F, is expressed as:

$$F = P - Q \qquad [6]$$

and the concept of initial abstraction is introduced via the following substitutions:

$$P = P - I_a \qquad [7]$$

$$S' = S \qquad [8]$$

where I_a = initial abstraction and
S = a storage parameter which accounts for initial abstraction ie., $S = S'+I_a$ qualitatively, but not explicitly,

then the Mockus relationship can be written as:

$$\frac{(P - I_a) - Q}{S} = \frac{Q}{(P - I_a)} \qquad [9]$$

McGurk (1982) presents a useful graphical interpretation of this relationship. Equation 9 can be rearranged into the form:

$$Q = \frac{(P - I_a)^2}{(P - I_a + S)} \qquad [10]$$

Runoff is therefore reduced to a function of three quantities: rainfall, a watershed storage parameter, and initial abstraction. Mockus defines initial abstraction as including interception, surface storage, and infiltration occurring prior to the initiation of runoff. To further simplify the runoff method, data from experimental plots throughout the U.S. were analyzed and a relationship was generalized to:

$$I_a = 0.2(S) \qquad [11]$$

Mockus accounts for the observed scatter (see Figure 5) as error associated with data collection and subsequent estimates of I_a and S. Alternatives to the SCS I_a-S relationship are presented by Aron et al. (1979) and Golding (1979). Substitution of the initial abstraction relation into equation 10 yields the familiar SCS runoff formula:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad [12]$$

where P = 24-hour rainfall depth and
S = watershed storage parameter.

This SCS runoff equation will serve as the basis for the total volume estimation methods examined in this report and analyzed by Konyha et al. (1982). Each of the following methods uses a distinct technique for arriving at the storage parameter, S.

NEH-4. The SCS National Engineering Handbook-Section 4, Hydrology (USDA-SCS,1972b), outlines a procedure for the determination of a watershed storage parameter. In this technique, S is calculated as a function of runoff curve number (CN), where:

$$S = \frac{1000}{CN} - 10 \qquad [13]$$

This CN parameter varies between 0 and 100 as a function of several watershed factors, namely: 1) the predominant soil types, 2) the soils' infiltration properties, 3) the vegetative cover condition of the soil, 4) the antecedent moisture condition of the soil, and 5) land use and practices. Changes in watershed conditions impact runoff volume through changes in these characteristics. Guidelines for the determination of the runoff curve number are documented in Appendix I.

The number resulting from this procedure is said to apply to "average" watershed antecedent moisture condition (AMC=II). NEH-4 provides criteria for varying this AMC based upon the cumulative rainfall occurring during the five days prior to the rainfall event being examined. Table 2 presents criteria for the discrete partitioning between wet conditions with high runoff potential (AMC=III) and dry conditions with low runoff potential (AMC=I). Thus the NEH-4 method provides estimates for maximum, minimum and median runoff volume. Figure 6 presents the solution to the combination of equations 12 and 13.

SCS-Florida. The AMC partitioning method was developed for clay or loamy soils which expand upon wetting thus reducing infiltration. For the sandy conditions of Florida the NEH-4 AMC method was not felt to be a reliable indicator of watershed wetness. The SCS, in their Florida Interim Procedure report (USDA-SCS, 1980), therefore recommended that AMC=II be used for all storm events. Given this single curve number value, the SCS-Florida method does not account for varying watershed wetness conditions, but simply gives an estimate of a median runoff volume determined using the storage equation:

$$S = \frac{1000}{CN_{II}} - 10 \qquad [14]$$

where CN_{II} = SCS curve number at AMC=II.

Efforts to refine the curve number method are in progress in other regions of the United States. Hailey and McGill (1983) report that, for Texas, a climatic index served as a good curve number index of variation.

DRM. The South Florida Water Management District in their District Regulatory Manual (SFWMD, 1983) outline a procedure whereby the storage parameter, S, is a direct function of the depth to the water table (DWT) as shown in Figure 3. This approach cites the water table depth as the primary factor controlling runoff. While the curve number attempts to account for infiltration rate, the DRM approach considers only the total available storage capacity of the soil. Latitude for assessing the influence of development activity is available through the presentation of two available

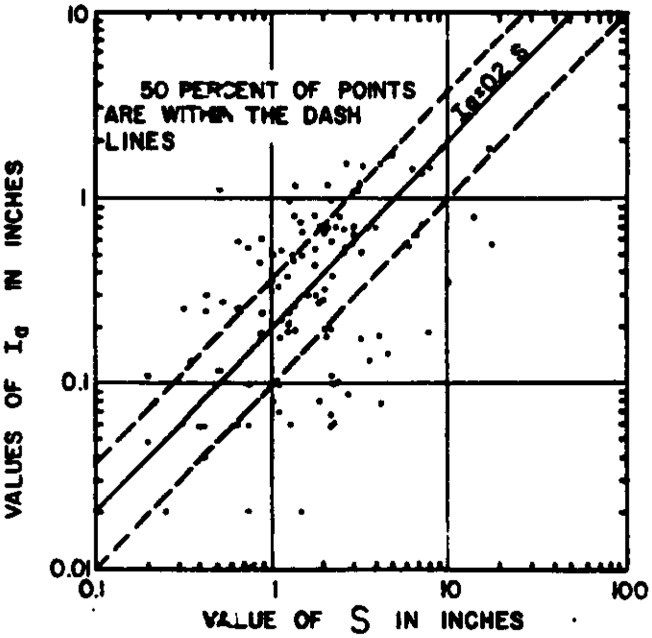


Figure 5. Initial abstraction versus the SCS watershed storage parameter (USDA-SCS, 1972b).

Table 2. Seasonal rainfall limits for determining AMC (antecedent moisture condition) in the NEH-4 method (USDA-SCS, 1972b).

AMC Group	Total 5-Day Antecedent Rainfall	
	Dormant Season	Growing Season
	Inches	Inches
I	Less than 0.5	Less than 1.4
II	0.5 to 1.1	1.4 to 2.1
III	Over 1.1	Over 2.1

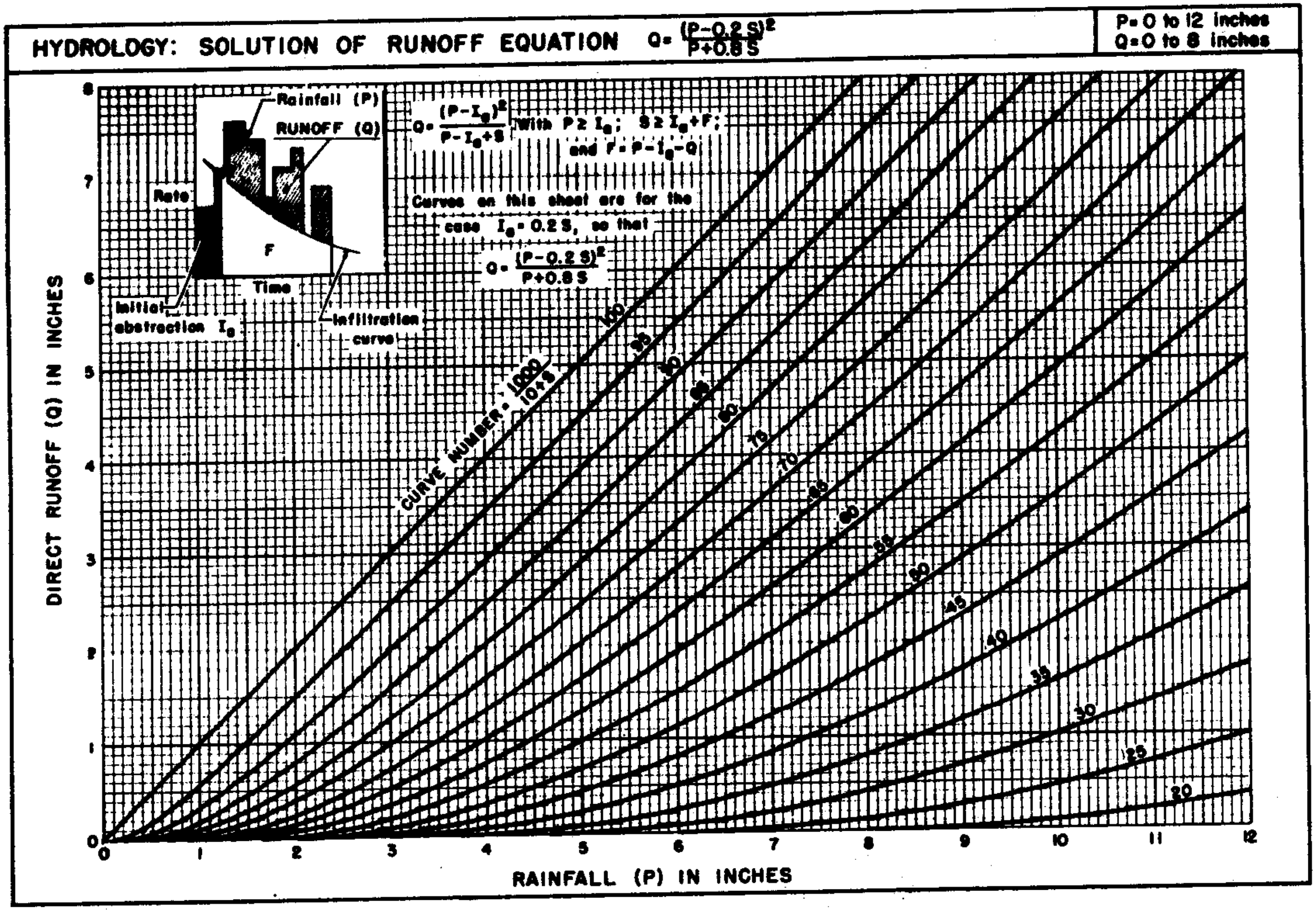


Figure 6. Solution of the SCS runoff equation (USDA-SCS, 1972b).

soil storage curves; one representing natural soil conditions and another representing the impact of development activities upon soil compaction. Further modifications can be introduced by weighting the overall watershed parameter with the watershed percent impervious area. The DRM method calculates S as:

$$S = S_{DRM}(1 - IMP) \qquad [15]$$

where S_{DRM} = overall watershed soil storage as a function of depth to the water table, from Figure 3 and
IMP = fraction of watershed covered by impervious surfaces.

This method is tailored specifically for use on watersheds within the District. It does not allow for the assessment of impacts upon runoff due to crop cover, hydrologic condition, or agricultural management practices.

CR-1. The CR-1 method as developed by Konyha et al. (1982), employs a watershed storage parameter weighting function extracted from the CREAMS hydrologic simulation model. CREAMS (A Field Scale Model for Chemicals, Runoff, and Erosion from Agricultural Management Systems) as described in Knisel (1980), contains the algorithm:

$$S = S_{max}\left(\frac{UL - SM}{UL}\right) \qquad [16]$$

where UL = upper limit of soil water storage,
SM = soil moisture content, and
S_{max} = maximum value of S;

$$S_{max} = \left(\frac{1000}{CN_I}\right) - 10 \qquad [18]$$

where CN_I = SCS curve number at AMC=I.

The numerator of equation 16 represents an available soil moisture term and the denominator a maximum storage term. For the Florida flatwoods watersheds, Konyha assumed an upper limit soil water storage value of 5.0 inches. Two reasons were cited for this choice of effective storage limit, despite the SFWMD curve's indication of additional available storage at low water table conditions. First, the flatwoods soils generally have an impeding layer at a depth of 2 to 3 feet below the surface which can decrease infiltration capacity. Secondly, entrapped air can further slow infiltration. The 5.0 inch limit introduces infiltration rate rather than total available storage as the limiting storage factor when high rainfall events fall upon dry (low water table) conditions. The equation used by the CR-1 method is:

$$S = S_{max}\left(\frac{S_{DRM}}{5.0}\right) \qquad [18]$$

where $S_{DRM} < 5.0$ as determined from Figure 3.

This method combines the depth to the water table soil storage function with the flexibility of the curve number approach, thus accounting for the influence of watershed wetness and agricultural management practices on runoff volume.

<u>CR-2.</u> Like CR-1, the CR-2 technique is based upon the CREAMS weighting algorithm (equation 16). However, instead of using a depth to the water table function, Konyha et al. (1982) employed a simplified soil moisture accounting model. The CR-2 version of the CREAMS storage parameter equation is:

$$S = S_{max}\left(\frac{5.0 - I_t}{5.0}\right) \qquad [19]$$

where I_t = soil moisture, in inches.

The I_t term is determined using the storage depletion model developed by Stephens and Mills (1965) in a statistical study of southern Florida flatwoods watersheds. The soil moisture status at any time can be determined by:

$$I_t = I_o(K^t) \qquad [20]$$

where I_o = water initially in storage,
K = a recession factor (0.96 in dormant season, 0.94 otherwise), and
t = rainless days.

In this procedure, soil moisture is assigned a maximum effective value of 5.0 inches at 30 days prior to the event being examined. The decay model then determines the moisture status at the time of the next rainfall. This rainfall raises the soil moisture to a new value not to exceed

5.0 inches. The accounting procedure (shown graphically in Figure 7) continues until the date of the rainfall event being examined.

The CR-2 method does not require an assumed or measured water table depth as does the CR-1 technique. Instead, an assumed or measured rainfall history is used to arrive at an estimate of watershed wetness at the time of a storm event.

CR-WT. Both of the previously described methods employ the storage parameter weighting function as extracted from the CREAMS hydrologic model. The CR-WT method also uses equation 16, but in the context of the entire simulation model i.e., the full model determines soil moisture status.

Heatwole et al. (1984) describe a version of the CREAMS model adapted to account for a fluctuating water table. The effect of these modifications was to prevent deep percolation out of the soil profile. Soil moisture depletions are modeled as interflow, ET, and slight amounts of slow, lateral drainage. Implementation of the CREAMS-WT (Option I) model requires inputs of daily rainfall and temperature, monthly radiation, and land use parameters. Results can include water quality as well as stormwater runoff volumes.

Storm Runoff Peak Rate

As described earlier, a variety of approaches are available for the routing of stormwater to arrive at peak discharge rates. Several techniques representing a range of complexity levels have been applied to the Florida flatwoods. They are presented as follows beginning with the very empirical and progressing through to the more theoretical approaches.

Cypress Creek Formula. The Cypress Creek Formula was developed to aid in the design of drainage systems for small agricultural watersheds (Stephens and Mills, 1965). This formula does not predict the instantaneous peak of a stormwater hydrograph, but estimates a maximum 24-hour-average discharge using the following equation:

$$q = C(M^{5/6}) \qquad [21]$$

where q = maximum 24-hour-average discharge rate in cfs,
M = watershed area in mi^2, and
C = a coefficient based upon topography and rainfall.

Speir et al. (1969) analyzed this formula as applied to the Taylor Creek Basin and arrived at:

$$C = 16.39 + 14.75(R_e) \qquad [22]$$

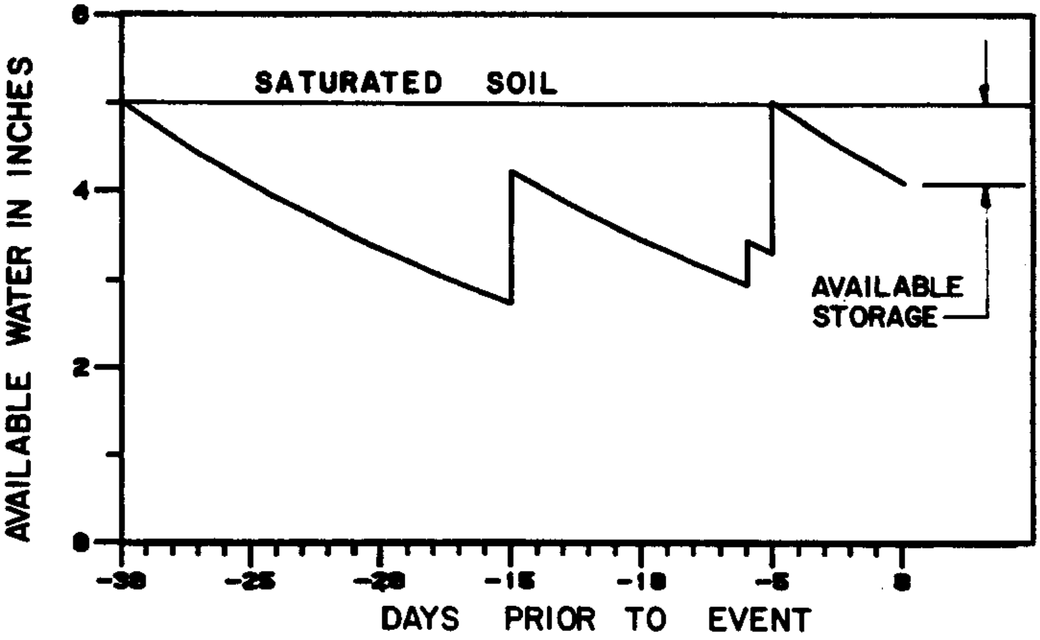


Figure 7. Example of CR-2 soil moisture accounting method application (Konyha et al., 1982).

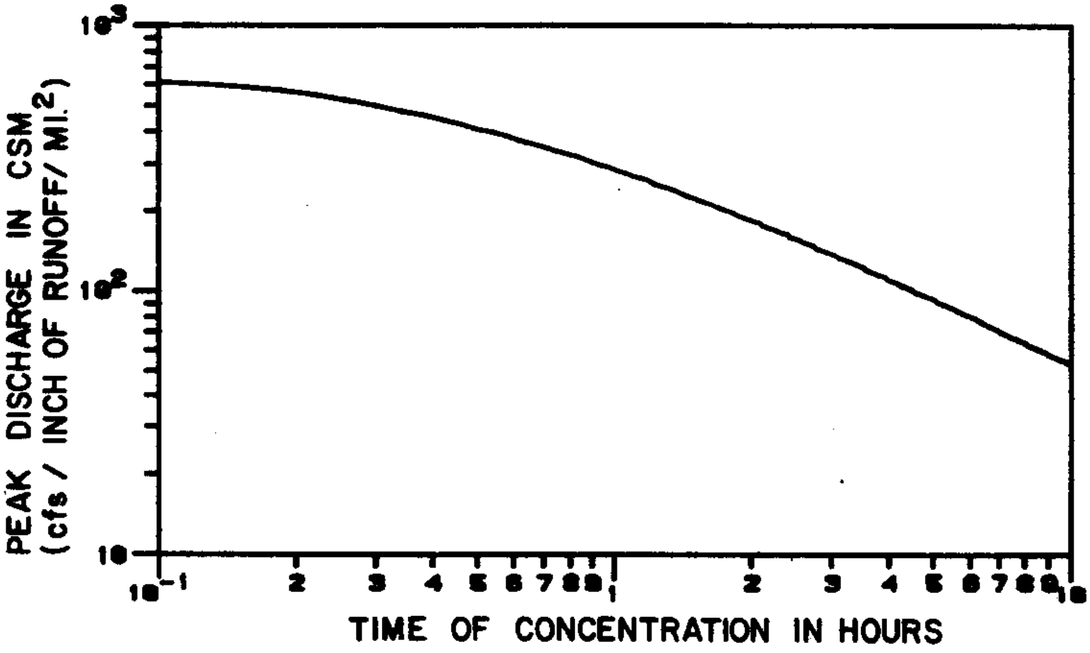


Figure 8. Design peak rate curve for use in the SCS Graphical Method (USDA-SCS, 1980).

where R_e = 24-hour rainfall excess in inches.

Given equations 21 and 22, the Cypress Creek Formula allows only rainfall excess and watershed area as variable factors. The resulting peak should underestimate the instantaneous peak rate since the 24-hour-average will be equal to or less than any instantaneous rate within the same time period. Stephens and Mills (1965) present a curve relating instantaneous peak to the average 24-hour rate as a function of drainage area. An equivalent expression is:

$$r = 2.0 - 0.43(\log M) \qquad [23]$$

where r = instantaneous peak/maximum 24-hour-average rate.

<u>CREAMS Equation</u>. The algorithm used in the CREAMS hydrologic model (Knisel, 1980) to estimate peak daily flows is:

$$q_p = 200(DA^{0.70})(CS^{0.159})(LW^{-0.187})(Q^{0.917(DA^{0.0166})}) \qquad [24]$$

where q_p = peak runoff rate in cfs,
DA = drainage area in mi^2,
CS = main channel slope in ft/mi,
LW = watershed length to width ratio, and
Q = daily runoff volume in inches.

This empirical formula was developed with data from 304 storms occurring on 56 watersheds in 14 states (none in Florida) (Smith and Williams, 1980). Its formulation is similar to that of the Cypress Creek Formula, but has channel slope and length to width ratio as added independent variables.

<u>SCS Graphical Method</u>. The Soil Conservation Service (USDA-SCS, 1980) published an interim peak discharge curve for Florida (Figure 8). Associated with this graph is the polynomial equation:

$$\log q_p = 2.45337 - 0.58595(\log T_c) - 0.20265(\log T_c)^2 + 0.05437(\log T_c)^3 \qquad [25]$$

where q_p = peak discharge in csm (cfs per mi^2 per inch of runoff) and
T_c = time of concentration in hours.

Figure 8 and equation 25 differ from that published in TR-55 (USDA-SCS, 1975) due to the rainfall time-distribution used to generate each (see Figure 1). SCS hydrologists observed that using the standard SCS Type II rainfall distribution resulted in unrealistically high runoff peak estimates and, therefore, the Florida interim rainfall distribution was developed to more accurately reflect rainfall patterns in South Florida. Both the TR-55 and Florida curves represent simplified results from execution of the SCS TR-20 computer model. This graphical approach is applicable for watersheds where channel routing is not required and the watershed is homogeneous.

TR-55 presents two techniques for estimating time of concentration (T_c) which is a hydraulic wave's travel time through a watershed. NEH-4 approximates T_c as that time required for runoff to travel from the hydraulically most remote part of the watershed to the point of reference. The simpler of the two techniques relates time of concentration to a watershed time lag parameter:

$$T_c = 1.67(L) \qquad [26]$$

where L = watershed lag (time from rainfall excess center of mass to peak rate of runoff);

$$L = \frac{l^{0.8}\,(S+1)^{0.7}}{1900\,Y^{0.5}} \qquad [27]$$

where l = hydraulic length of watershed in feet,
S = SCS watershed storage parameter from equation 14, and
Y = average watershed land slope in percent.

The alternate SCS method for estimating T_c relies upon the calculation of watershed total travel time. For a natural watershed this includes overland and channel flow times. Estimates of flow velocity for each regime are first made and then combined with the respective flow lengths to arrive at total travel time. Figure 9 offers estimates of overland flow velocity for various surface conditions and slopes. The recommended procedure for estimating channel velocity is the Manning equation applied to bank-full conditions. However, as discussed earlier, Mierau (1981) pointed out that this is not always an easy task for flatwoods watersheds.

Compared to the two empirical relations described previously, the SCS Graphical Method represents a slightly higher level approach to runoff peak rate estimation.

SCS Chart Method. The SCS Chart Method is comparable to the graphical method. However, instead of calculating the watershed lag directly, general slope and length considerations are internal to the chart. For the Florida flatwoods conditions the appropriate chart is that for the Florida

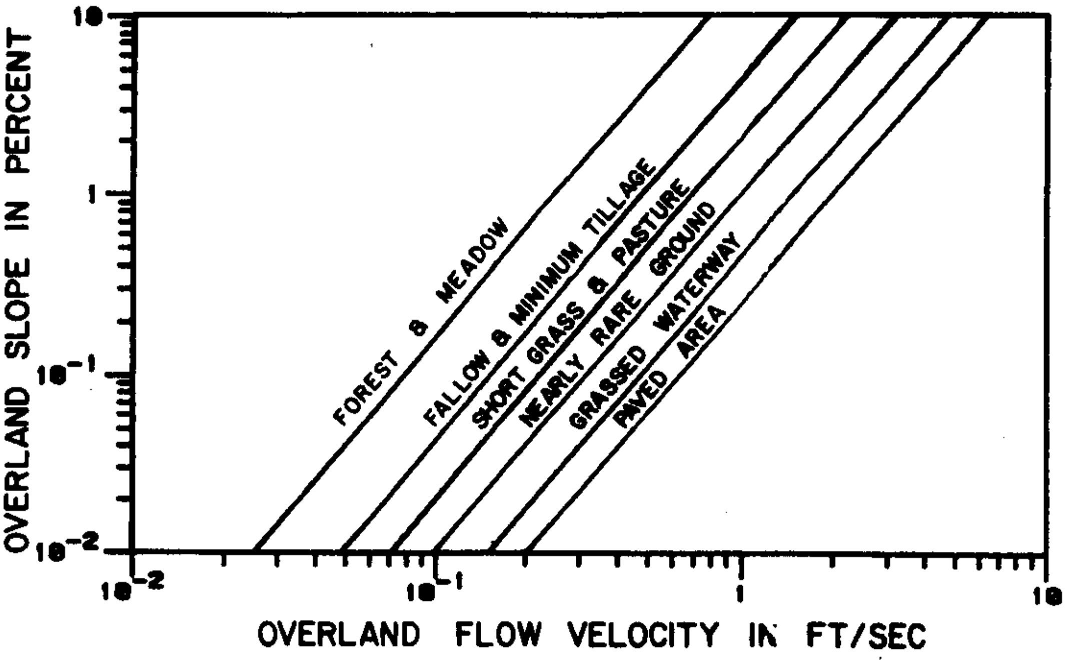


Figure 9. Overland flow velocity estimation curves for use in the SCS Graphical Method (USDA-SCS, 1972b).

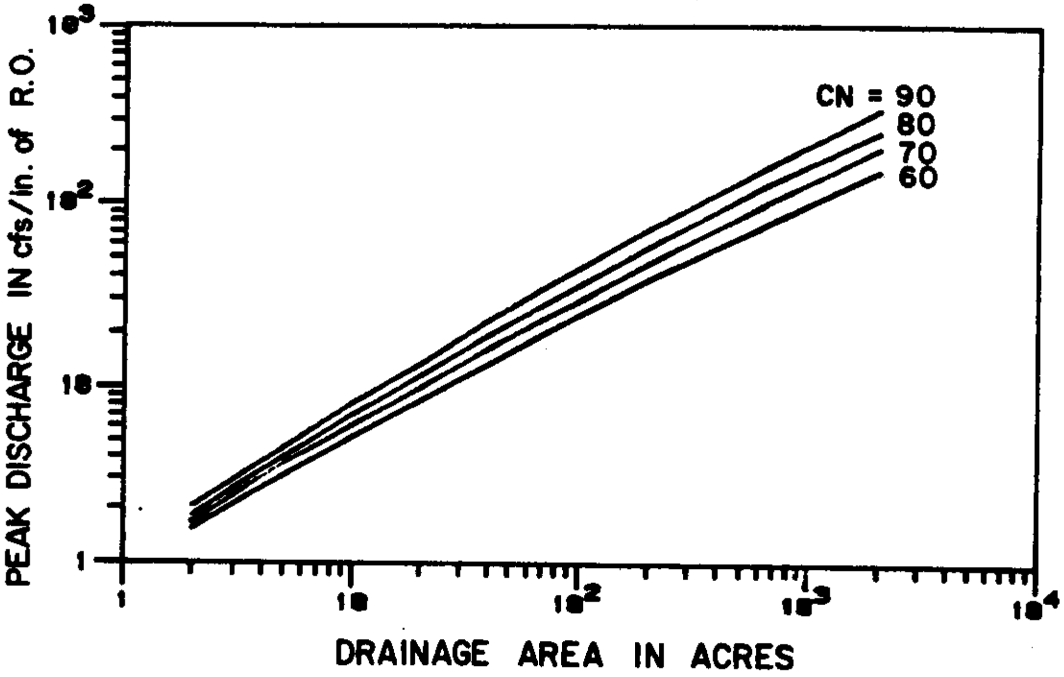


Figure 10. Design peak rate curves for use in the SCS Chart Method (USDA-SCS, 1980).

Table 3. Peak rate adjustment factors for swamps and ponds (spread throughout the watershed) for use in the SCS Chart Method (USDA-SCS, 1972). Values corresponding to frequencies less than 1-year were determined by extrapolation.

% Swamps and Ponds	Storm Frequency in Years 0.1	0.5	1.0	2.0	5.0	10.	25.	50.	100.
0.2	0.90	0.93	0.93	0.94	0.95	0.96	0.97	0.98	0.99
0.5	0.85	0.86	0.87	0.88	0.89	0.90	0.91	0.92	0.94
1.0	0.80	0.82	0.82	0.83	0.84	0.86	0.87	0.88	0.90
2.0	0.75	0.76	0.77	0.78	0.79	0.81	0.83	0.85	0.87
2.5	0.69	0.71	0.72	0.73	0.74	0.76	0.78	0.81	0.84
3.3	0.65	0.67	0.68	0.69	0.70	0.71	0.74	0.77	0.81
5.0	0.61	0.63	0.64	0.65	0.66	0.68	0.72	0.75	0.78
6.7	0.58	0.60	0.61	0.62	0.63	0.65	0.69	0.72	0.75
10.0	0.54	0.56	0.57	0.58	0.59	0.61	0.65	0.68	0.71
20.0	0.50	0.52	0.52	0.53	0.54	0.56	0.60	0.63	0.68
25.0	0.47	0.48	0.49	0.50	0.51	0.53	0.57	0.61	0.66

Table 4. Peak rate adjustment factors for watershed slope for use in the SCS Chart Method (USDA-SCS, 1975). Values corresponding to slopes less than 0.1% were determined by extrapolation.

Watershed % Slope	Drainage Area in Acres 10	20	50	100	200	500	1000	2000
0.01	0.24	0.22	0.20	0.19	0.18	0.18	0.17	0.17
0.02	0.30	0.28	0.25	0.24	0.23	0.23	0.23	0.22
0.03	0.34	0.32	0.29	0.28	0.27	0.27	0.26	0.25
0.04	0.37	0.35	0.32	0.31	0.30	0.30	0.29	0.28
0.05	0.40	0.37	0.34	0.33	0.32	0.32	0.32	0.31
0.06	0.42	0.40	0.37	0.36	0.35	0.35	0.34	0.33
0.07	0.44	0.42	0.39	0.38	0.38	0.37	0.36	0.35
0.08	0.46	0.43	0.41	0.40	0.39	0.39	0.38	0.37
0.09	0.47	0.45	0.42	0.41	0.40	0.40	0.39	0.38
0.10	0.49	0.47	0.44	0.43	0.42	0.41	0.41	0.40
0.30	0.69	0.67	0.65	0.64	0.63	0.62	0.62	0.61
0.50	0.82	0.80	0.78	0.77	0.77	0.76	0.76	0.76
0.70	0.90	0.89	0.88	0.87	0.87	0.87	0.87	0.87
1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1.50	1.13	1.14	1.14	1.15	1.16	1.17	1.17	1.17
2.00	1.21	1.24	1.24	1.28	1.29	1.30	1.31	1.31

interim rainfall distribution and flat watershed slopes in Figure 10 (SCS, 1980). Given the watershed curve number and drainage area, an initial peak discharge estimate is first determined. This quantity is then modified with adjustment factors for specific watershed slope (Table 3) and the distribution and extent of swamps and ponds within the drainage basin (Table 4).

The SCS chart method can be summarized as:

$$q_p = q_p'(F_s)(F_p) \qquad [28]$$

where q_p = peak discharge in cfs per inch of runoff,
q_p' = peak discharge from Figure 10,
F_s = slope adjustment factor from Table 3, and
F_p = swamps and ponds adjustment factor from Table 4.

SCS Unit Hydrograph Method. The SCS unit hydrograph approach to estimating stormwater peak discharges utilizes a triangular approximation of a runoff unit hydrograph (Figure 11). Synthetic unit hydrographs of this shape can be created using watershed and storm characteristics to estimate time parameters of the triangular hydrograph. The basic relationship of the triangle relates the geometry of its shape:

$$Q = \frac{(q_p)(T_b)}{2} \qquad [29]$$

where Q = unit runoff volume (L^3),
q_p = peak discharge rate (L^3/T), and
T_b = hydrograph time base (T).

noting that: $T_b = T_p + T_r$ and rearranging yields:

$$q_p = \frac{2Q}{T_p + T_r} \qquad [30]$$

where T_p = time to peak (T) and
T_r = recession time (T).

If the relationship between the time parameters is lumped into a single factor, K, such that:

$$K = \frac{2}{1 + T_r/T_p} \qquad [31]$$

then equation 30 can be written as:

$$q_p = \frac{(K)(Q)}{T_p} \qquad [32]$$

If specific units are introduced for these quantities, then the triangular hydrograph function becomes:

$$q_p = \frac{645.33(K)(A)(Q)}{T_p} \qquad [33]$$

where q_p = peak runoff rate in cfs,
A = Area in mi^2,
Q = rainfall excess depth in inches,
T_p = time to peak in hours,
K = hydrograph shape factor, and
645.33 = unit conversion factor.

By lumping the shape and unit conversion factors into a single quantity, K', the SCS triangular unit hydrograph equation simplifies to:

$$q_p = \frac{(K')(A)(Q)}{T_p} \qquad [34]$$

Therefore, synthesis of an SCS unit hydrograph requires the estimation of two time parameters. The standard estimate for K' (484) describes a hydrograph whose recession is 1.67 times as long as its time to peak. Mockus (USDA-SCS, 1972b) notes that this K' value has been known to vary from 600 in steep terrain to 300 in flat swampy country. For the Delmarva peninsula, which includes Delaware and parts of Maryland, Welle et al. (1980) concluded that a value of 256 is more appropriate. The watersheds examined were small with sandy soils and slopes in the range of 2%. The U.S. Army Corps of Engineers (1955) studied records from several large

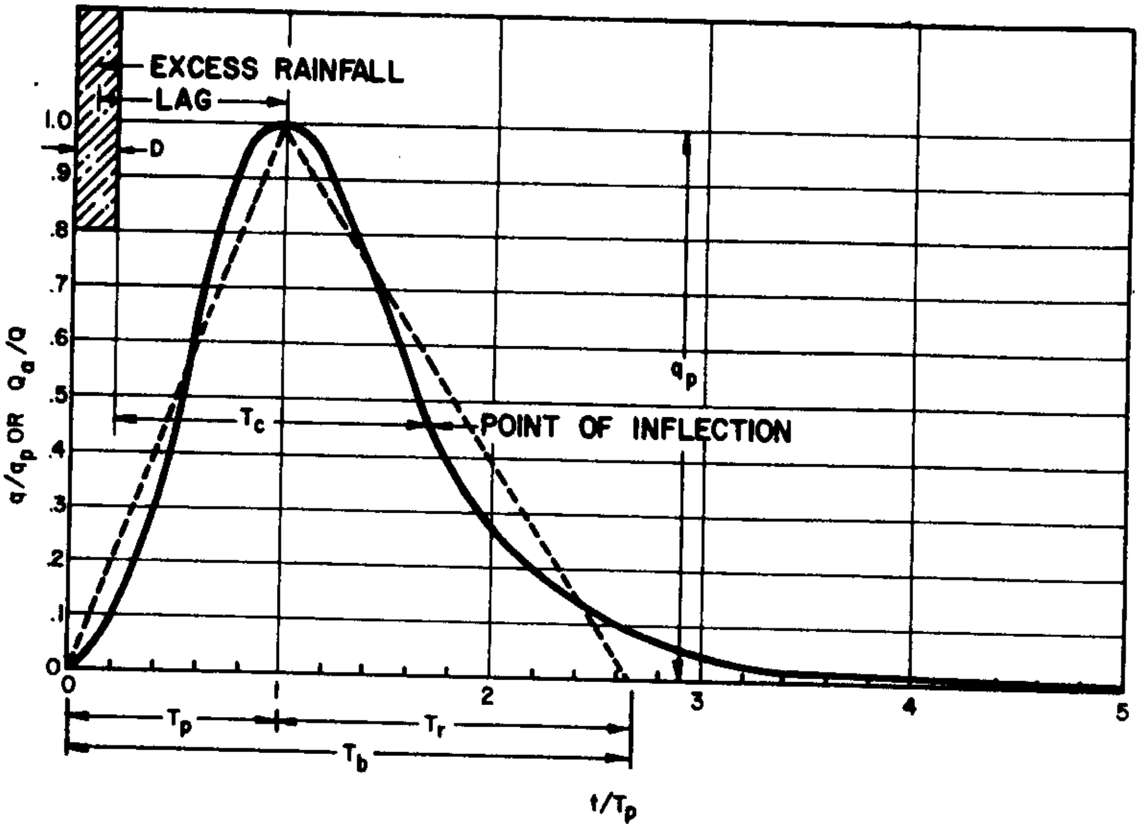


Figure 11. SCS triangular unit hydrograph approximation and time parameter interpretations (USDA-SCS, 1972b).

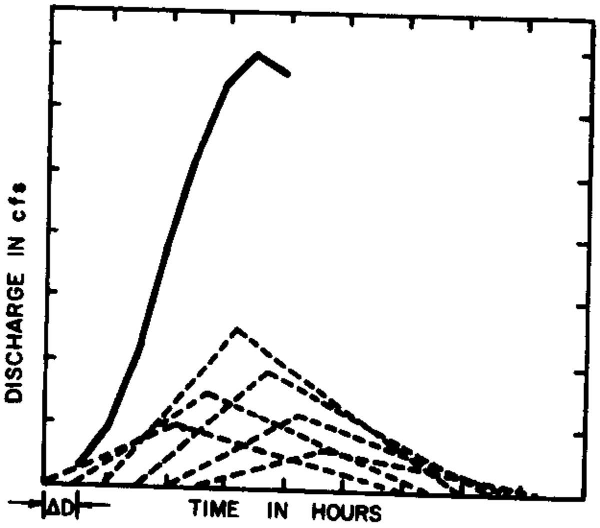


Figure 12. Generation of a composite discharge hydrograph by the superposition of incremental unit hydrographs (Kent, 1973).

watersheds in Central and South Florida (the entire Kissimmee River Basin being one) and determined an appropriate time factor for use in a similar peak discharge equation. Miller and Einhouse (1984) translated this factor into the SCS form, arriving at a value of 284 for K'.

The other time parameter in equation 34, T_p, is defined as:

$$T_p = L + \frac{\Delta D}{2} \qquad [35]$$

where L = watershed time lag and
ΔD = rainfall excess duration.

The SCS recommends using a duration not exceeding 20% of the time to peak. Lag can be calculated with equation 27 or alternatively can be determined using a total travel time estimate and equation 26.

Given a triangular unit hydrograph tailored to a specific watershed and rainfall excess duration, a composite storm hydrograph can be developed. Kent (1973) describes such a procedure. First, the rainfall mass curve (Figure 1) is discretized into equal increments of ΔD. The rainfall excess for each increment is then calculated with equation 13 and an individual hydrograph developed for each. Superposition is applied to the series of hydrographs resulting in a composite storm discharge hydrograph (Figure 12). An estimate of peak discharge can be extracted from this composite hydrograph.

SFWMD Model. The South Florida Water Management District (SFWMD) uses a graphically-based technique to determine peak discharges for watersheds within its jurisdiction. The graphs in the District Regulatory Manual IV (SFWMD, 1983) originate from output of an overland flow computer model as constructed by Higgins (1976) and implemented by SFWMD (1979).

This program employs Manning's form of the overland flow momentum equation (equation 3) combined with an assumed retention depth:

$$q = W \left(\frac{1.49}{n}\right) (D-D_r)^{1.67} S^{0.5} \qquad [36]$$

where q = watershed outflow in cfs,
W = watershed width in ft,
n = Manning's roughness coefficient,
D = surface water depth in ft,
S = watershed groundslope in ft/ft, and
D_r = watershed retention depth in ft.

The watershed is modeled as a single uniform inclined plane with continuity calculated using the following scheme:

$$D_i = D_t + R\ \Delta t - f\ \Delta t \qquad [37]$$

$$D_{t+1} = D_i - \left(\frac{q\ (D_i)\ \Delta t(3600)}{A}\right) \qquad [38]$$

where D_i = intermediary water depth in ft
D_t = initial water depth in ft,
Δt = simulation time increment in hours,
R = rainfall rate in ft/hr,
f = infiltration rate, in ft/hr,
D_{t+1} = final water depth in ft,
$q(D_i)$ = outflow rate calculated at D_i in cfs, and
A = watershed area in ft^2.

Watershed outflow rate calculation begins when D_i exceeds D_r (2.0 inches) and continues for each time increment until D_i again approaches D_r.

The two components of the continuity procedure other than outflow are rainfall and infiltration. Rainfall is assumed to follow the SFWMD distribution shown in Figure 1. Infiltration is calculated using Horton's equation with an initial rate of 3.1 in/hr and a final rate of 0.01 in/hr. In Horton's method, infiltration rate decays exponentially with time. Higgins (1976) made the exponent of this decay function dependent upon the available ground storage. However, once this available ground storage is filled, infiltration continues to approach its final rate.

Peak discharge can be determined from the hydrograph produced by this simulation. Runoff volume is calculated only after runoff rate and is the integral of the discharge hydrograph. This simulation of overland flow and infiltration represents a more theoretical approach to stormwater modeling, but still includes many approximations of the real watershed system.

CHAPTER III

SITE AND DATA DESCRIPTION

Approximately one third of Florida is classified as having flatwoods soils. These are of the Spodosol order, meaning amorphous materials (organic matter, aluminum and iron oxides) in subsurface horizons. The specific suborder in Florida is Aquods, common to areas which are seasonally saturated with water, gently rolling range or woodland and, where drained, can support citrus and other special crops (Brady, 1974). Three general geographic classifications of flatwoods occur in Florida: the Gulf Coast and Atlantic Coast Flatwoods (thermic zone) and the Southern Florida Flatwoods (hyperthermic zone) as shown in Figure 13.

Data collection sites for this study are within the Lower Kissimmee River and Taylor Creek-Nubbin Slough Basins (see Figure 14). The predominant soil associations for both basins are Myakka-Immokalee-Waveland and Wabasso-Felda-Pompano (Caldwell and Johnson, 1982). Despite the high hydraulic conductivities of these soils (>16 cm/hr), drainage is poor unless augmented by extensive ditching. Hydrologic classification is A/D or B/D, the exact class determined by the effectiveness of drainage improvements at lowering the water table.

Land use capability is classified as IVw over 70-80% of the two basins, describing lands of limited productivity due to water related problems. Approximately 10-20% of the basins are rated as Class IIIw, requiring extensive treatment for cultivation (Huber et al., 1976; Speir et al., 1969). Natural vegetation consists primarily of wet and dry prairie grasslands and pine-palmetto forests. In the depressional areas, wetlands species predominate and include maidencane, cordgrass, St. Johnswort, pond pine, and various hardwoods. Land use in the two basins is dominated by improved and unimproved pasture, claiming about 75% of the total area in 1980 (Huber et al., 1976; Allen et al., 1982).

The means of transformation from a natural marsh and slough system to agricultural use has been drainage improvement achieved through ditching. Extensive channel networks combined with extremely low watershed slopes (<0.5%) make delineation of watershed boundaries a difficult task in some cases. Drainage patterns can, in fact, shift depending upon rainfall patterns and runoff magnitude.

The climate of South Florida is sub-tropical with an average temperature of 73-degrees Fahrenheit and average annual rainfall of 56 inches. The majority of rainfall occurs in the summer, however high-intensity storms can occur throughout the year (Speir et al., 1969). Figures 15, 16, and 17 present typical rainfall, temperature, and radiation patterns for South Central Florida.

Hydrologic data from five watersheds located within the Lower Kissimmee River and Taylor Creek-Nubbin Slough Basins were collected between 1979 and 1983 in conjunction with the Upland Detention/Retention Demonstration Project conducted by the Kissimmee Coordinating Council and the South Florida Water Management District. Primary emphasis of the project was to assess the water quality characteristics of the area and potential benefits from the creation of artificial impoundments/wetlands.

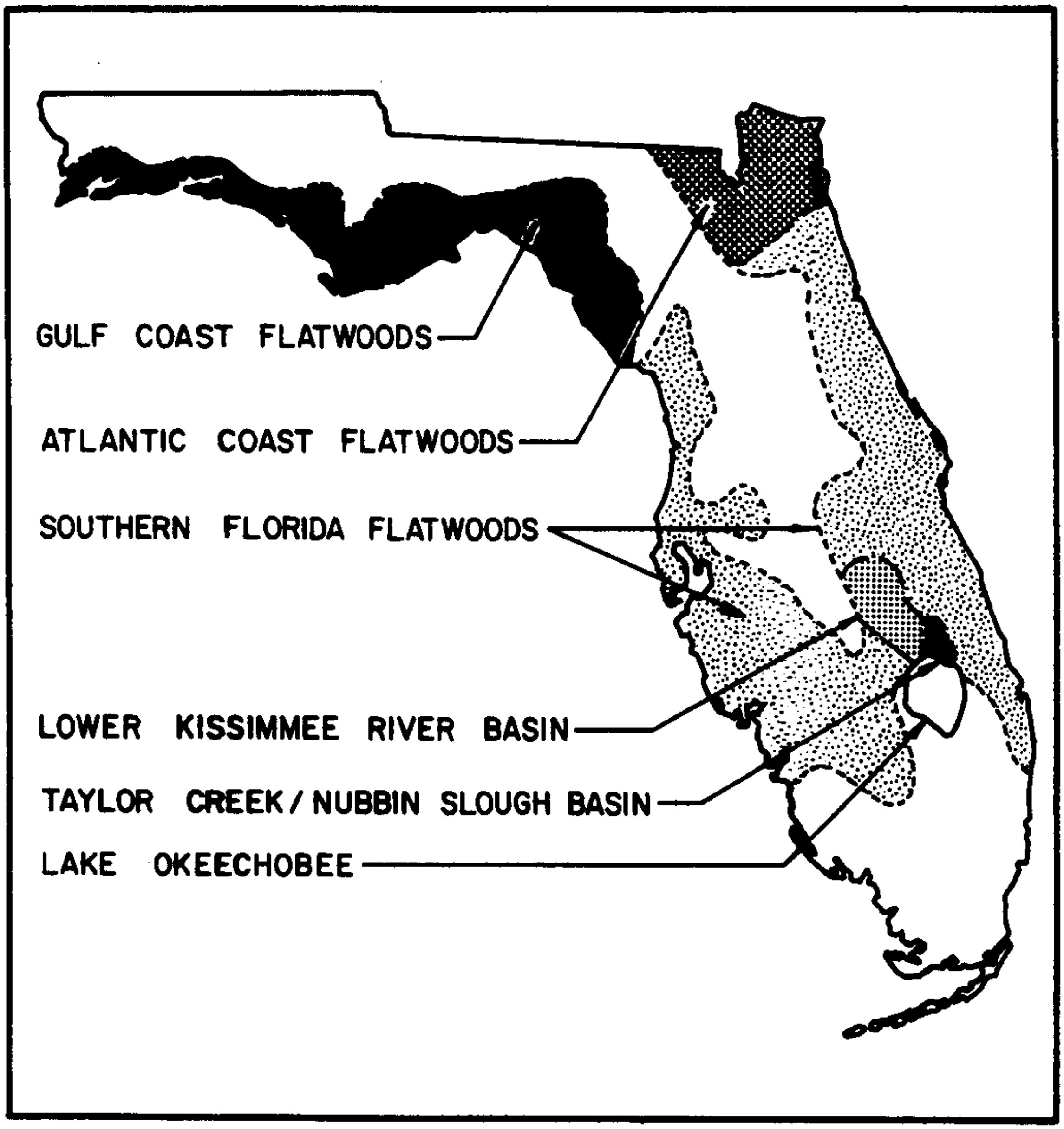


Figure 13. General Classification and distribution of flatwoods soils in Florida (Brady, 1974) and study area location.

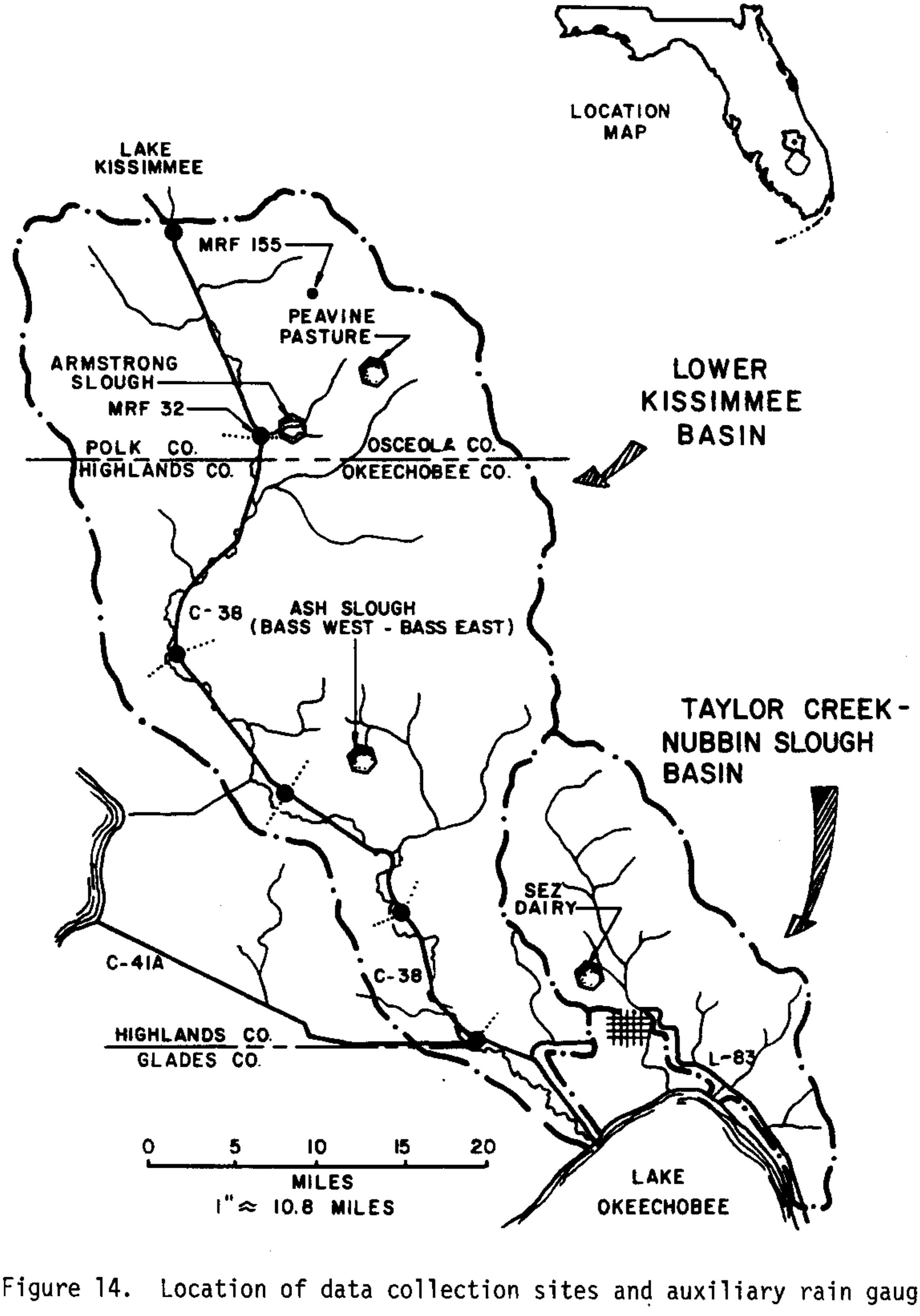


Figure 14. Location of data collection sites and auxiliary rain gauges.

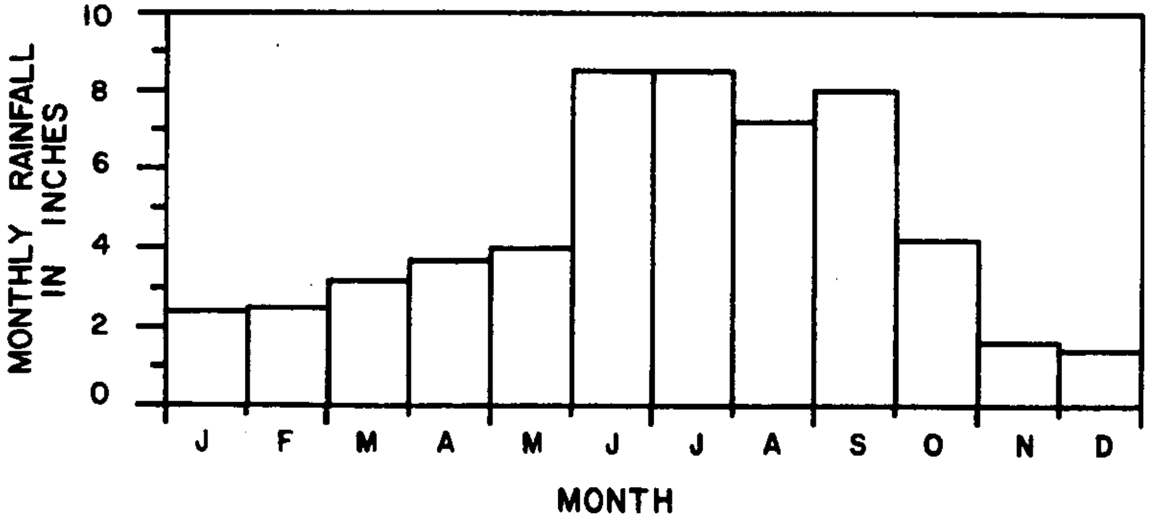


Figure 15. Study area monthly rainfall (Huber et al., 1976).

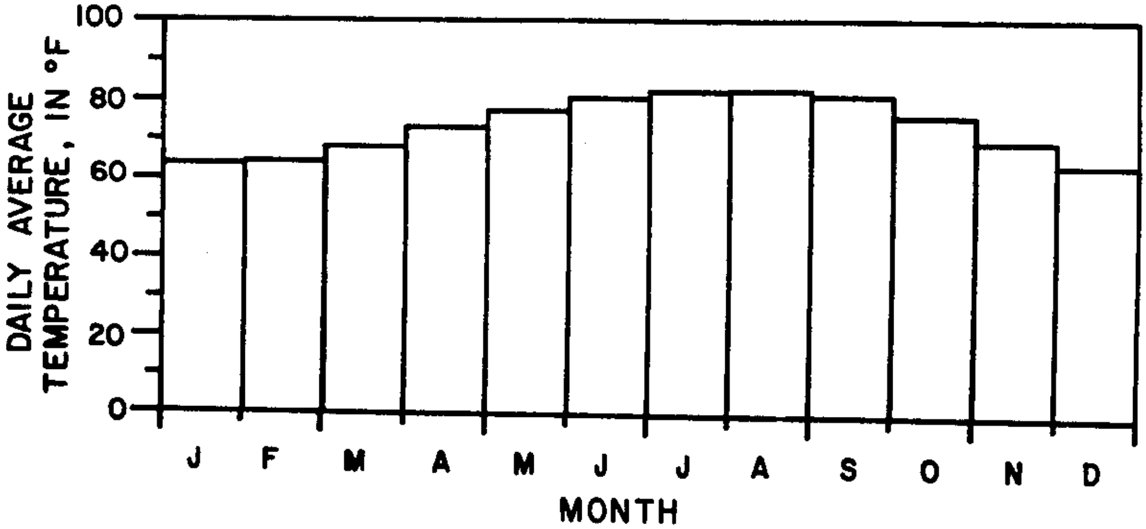


Figure 16. Study area average temperatures (USDC-NOAA, 1972-82).

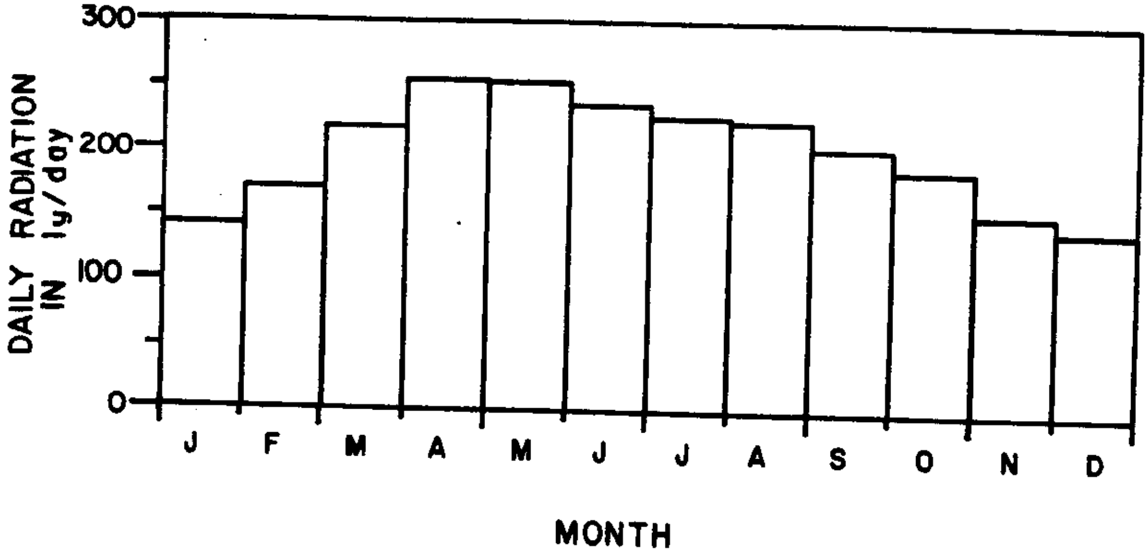


Figure 17. Study area average daily radiation (USDC-NOAA, 1972-82).

The U.S. Geological Survey installed and maintained instrumentation for the acquisition of rainfall and water table data. Measurement was on a continuous basis using automatic-feed strip chart recorders. Water table elevations were measured in shallow wells (<10 ft deep) equipped with float devices. Rainfall was recorded as weighing bucket traces recorded between monthly service intervals.

The South Florida Water Management District maintained responsibility for gathering discharge data. Instrumentation consisted primarily of stage recorders located upstream and downstream from critical depth flumes and drop inlet culverts. Readings were taken at 30-minute intervals at each site. Mierau (1981) describes the criteria governing the design of these structures and a District report (SFWMD, 1980) provides construction specifications for each structure. Operational schematics of the two runoff measurement systems are shown in Figures 18 and 19.

Critical depth flumes were selected as primary flow measurement devices as they provide accurate measurement over a wide range of operating conditions. Flow through these structures can be reliably calculated from physical dimensions, thus removing the need for empirical calibration (SFWMD, 1980). Another benefit of critical depth flumes is the small head differential required between upstream and downstream water levels for accurate flow measurement during high runoff events. The stage and relative differences between the upstream and downstream water surfaces dictate which flow condition (free flow or submerged) is in effect. Given free flowing conditions, the upstream elevation is sufficient for determination of flow rate. Under submerged conditions, both water surface elevations are required to estimate discharge.

Following is a brief description of each study site, its physical characteristics, instrumentation, and any significant observations associated with each. Table 5, which follows these sections, provides a summary of general watershed characteristics.

Armstrong Slough

A 3600 acre subbasin of the 12,000-acre Armstrong Slough watershed served as the largest site examined in this study (Figure 20). Runoff measurements were from a flume located at the watershed's outflow point into an artificial detention/wetland area. Offsite, but near the detention area, was the primary raingage used for this watershed. Due to the length of the drainage basin (5 miles), records from this gage were supplemented with data from raingages at the Peavine Pasture site and the S-65A Kissimmee River control structure (MRF32). Also offsite was the groundwater well used for estimating water table depths on Armstrong. Konyha et al. (1982) reported that this well was possibly influenced by water levels in the controlled detention area. Based upon later aerial inspection, the observation well farthest from the impoundment was judged an adequate representation of the general basin water table conditions.

The main channel servicing this basin is blocked at its far end, approximately four miles upstream of the flume. The upland boundaries are

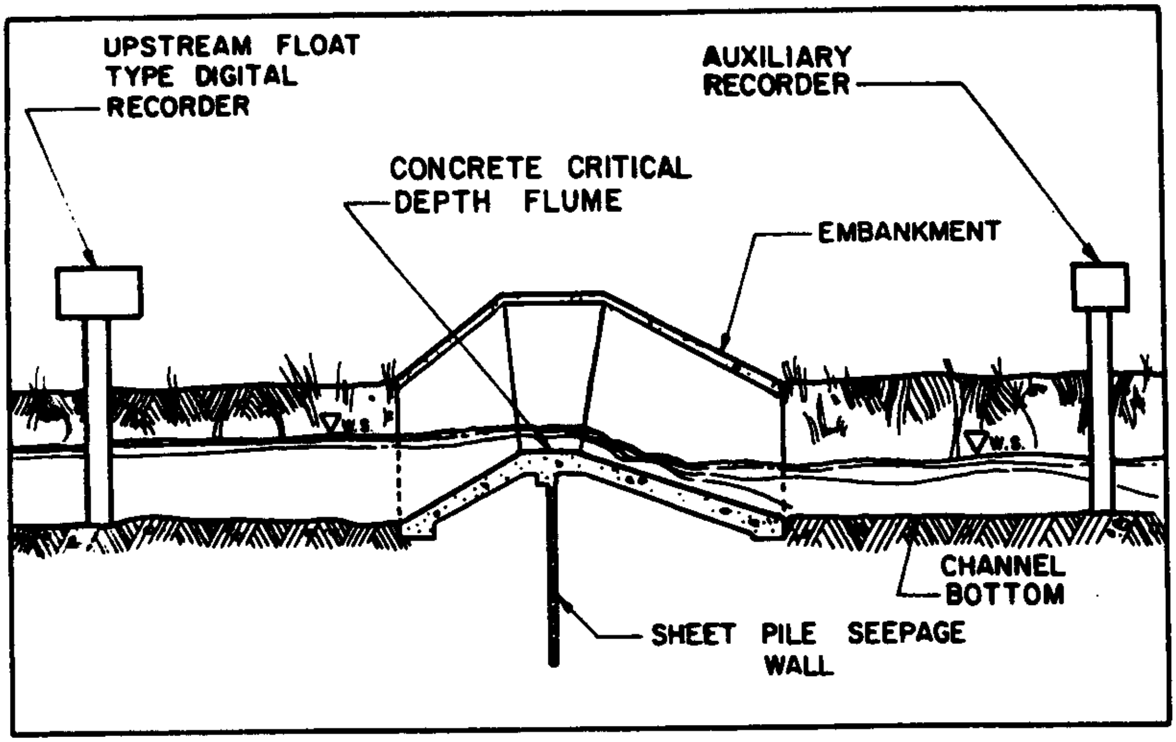


Figure 18. Critical depth flume discharge measurement system.

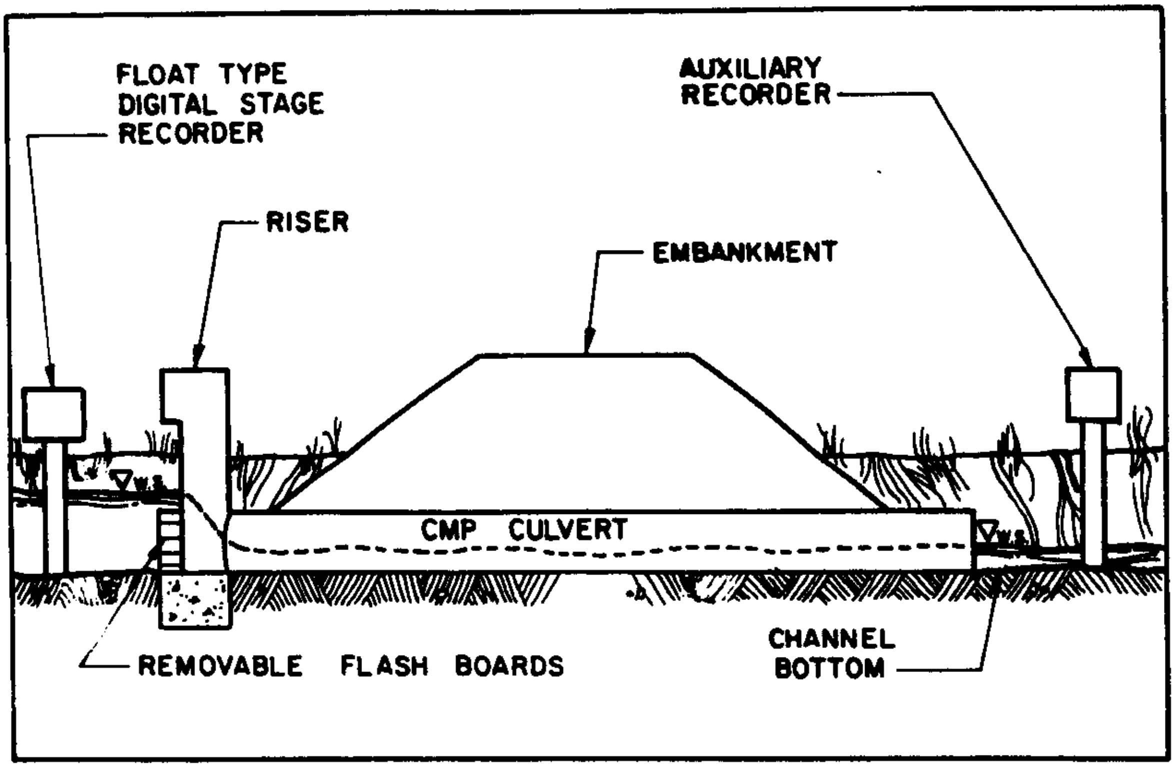


Figure 19. Culvert and riser discharge measurement system.

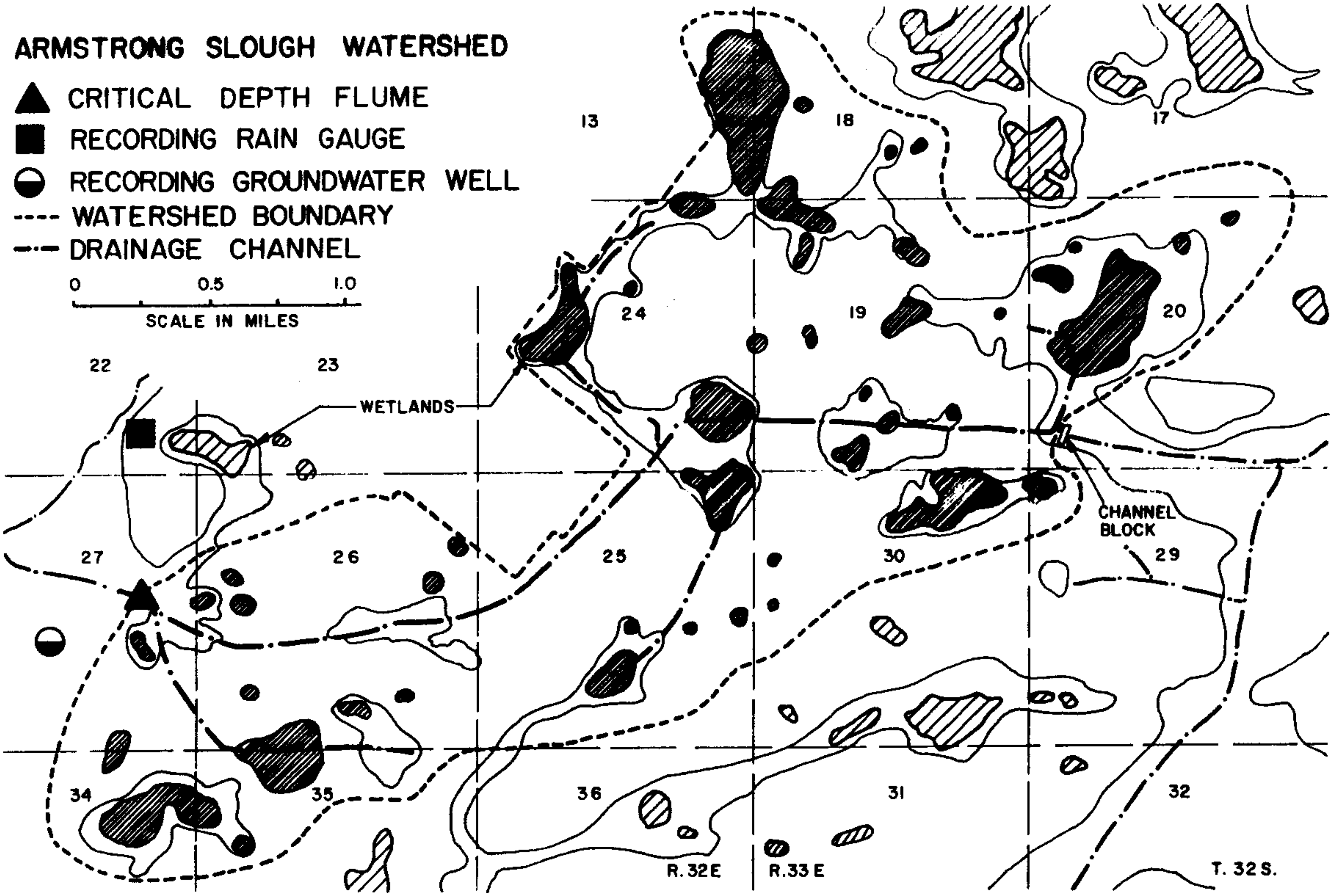


Figure 20. Armstrong Slough sub-watershed data collection site.

poorly defined due to little variation in relief. Basin delineations were, therefore, subjectively based upon drainage patterns as interpreted from aerial photographs and USGS topographic maps.

Armstrong Slough is described as a natural watershed consisting primarily of unimproved pasture and approximately 13% wetlands. The predominant soil type is Smyrna fine sand (41%) with Malabar, Pompano, Eaugallie, and Oldsmar combined accounting for an additional 46% of the watershed.

Periods of data records are: rainfall, April 1979 to February 1983; water table, January 1980 to October 1983; runoff, August 1979 to February 1983. Runoff records between September 1979 and March 1980 are coded as "estimated" due to a partial failure of the flume structure caused by heavy runoff associated with Hurricane David.

Peavine Pasture

The drainage area contributing to flow at the Peavine Pasture flume varied depending upon runoff event magnitude. Under normal conditions (when runoff was confined to the ditch) an artificial channel block limited the contributing area to 775 acres. During flood flows, however, overland flow dominated and the watershed reverted back to its natural drainage area of approximately 1800 acres (see Figure 21).

One raingage and observation well are located within the smaller basin. For the large runoff events, rainfall records from a SFWMD rainfall network site, MRF155, supplemented the USGS data.

Peavine Pasture is a relatively natural site consisting of improved pasture and 21-23% wetlands. Eaugallie fine sand accounts for 33% of the watershed's soil with Smyrna, Myakka, Malabar, and Pompano combined representing an additional 45%.

Periods of data records are: rainfall, April 1979 to August 1982; water table, January 1980 to September 1982; runoff, June 1979 to February 1983. Because of the additional area contributing to flow during larger runoff events, the flume was described as 90% submerged during most significant runoff events (±30% accuracy). By necessity, these events were included in the analysis.

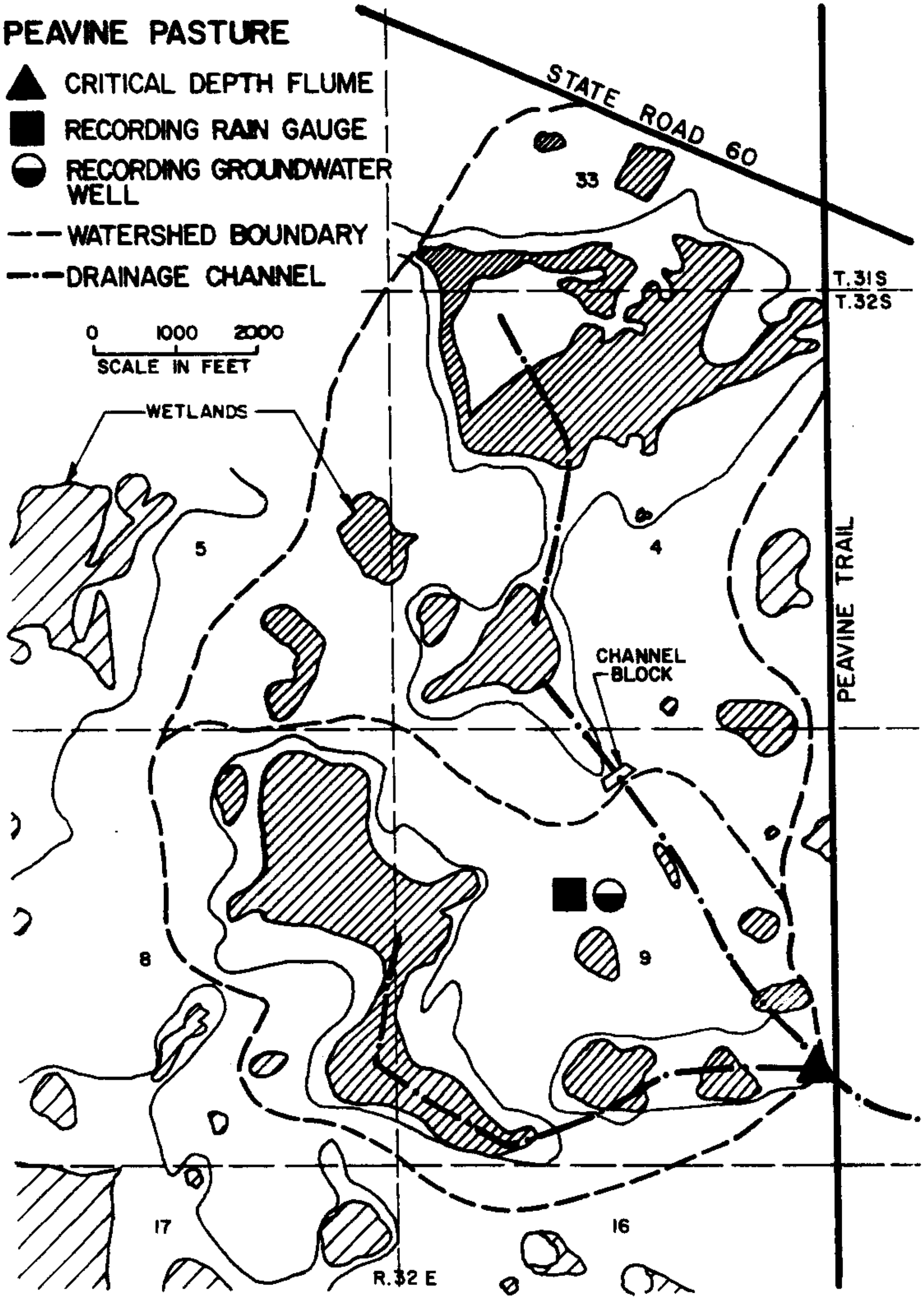


Figure 21. Peavine Pasture watershed data collection site.

SEZ DAIRY

SEZ Dairy is an elongated 710-acre watershed located in the Taylor Creek Basin. A well-defined perimeter ditch drains this improved pasture and dairy operation (see Figure 22). Discharge measurement is by a small culvert and riser located at the dairy's outflow point into a deep canal. This measurement reflects stormwater runoff and lagoon effluent reaching the drainage system. Discharge from a lagoon system used to treat dairy barn wash-water is measured with a flume as it flows onto the pasture's seepage field.

The site has east and west observation wells and one rain gage at the western well location. The buildings associated with the dairy operation are situated at the western end of the two-mile long watershed. The remainder of the land is devoted to improved pasture with 7% occupied by wetlands. Immokalee fine sand is the dominant soil type (55%) with Myakka, Parkwood, Charlotte, and Bass/Placid Complex combined accounting for an additional 26%.

Periods of data records are: rainfall, May 1979 to February 1983; water table, May 1980 to August 1982; runoff, November 1979 to February 1983. The culvert and riser outflow control from SEZ Dairy has a maximum capacity of 14 cfs. This limit was reached several times during the period of record, preventing the occurrence of natural flood peaks. Many of the significant runoff events were therefore eliminated from this analysis. Considering the deep perimeter ditch, it is unknown whether significant subsurface contributions may have been introduced from outside the diked watershed.

Bass West Pasture

The Bass West pasture site is the larger of two basins contributing to the Ash Slough impoundment/wetlands area and consists of 160 well-drained acres (see Figure 23). A well-defined (3-4 ft) perimeter ditch accepts flow from a network of shallow (2 ft) ditches. Outflow is measured at a flume where runoff enters the impoundment area. A water table observation well and raingage are located onsite. Land use is entirely improved pasture with no significant wetlands. Soil type is uniformly Myakka fine sand.

Periods of data records are: rainfall, May 1979 to February 1983; water table, January 1980 to July 1982; runoff, August 1979 to January 1983. As with SEZ Dairy, the perimeter ditch may have introduced some subsurface flow from outside the diked area. In late 1983 numerous breaches resulting from livestock traffic were observed in the low levee surrounding the pasture. These breaches may have resulted in unknown amounts of inflow to the pasture as well as flow bypassing the flume. Since it is not known when the size of the breaches became significant and which caused inflow and which caused outflow, no attempt was made to quantify these possible errors.

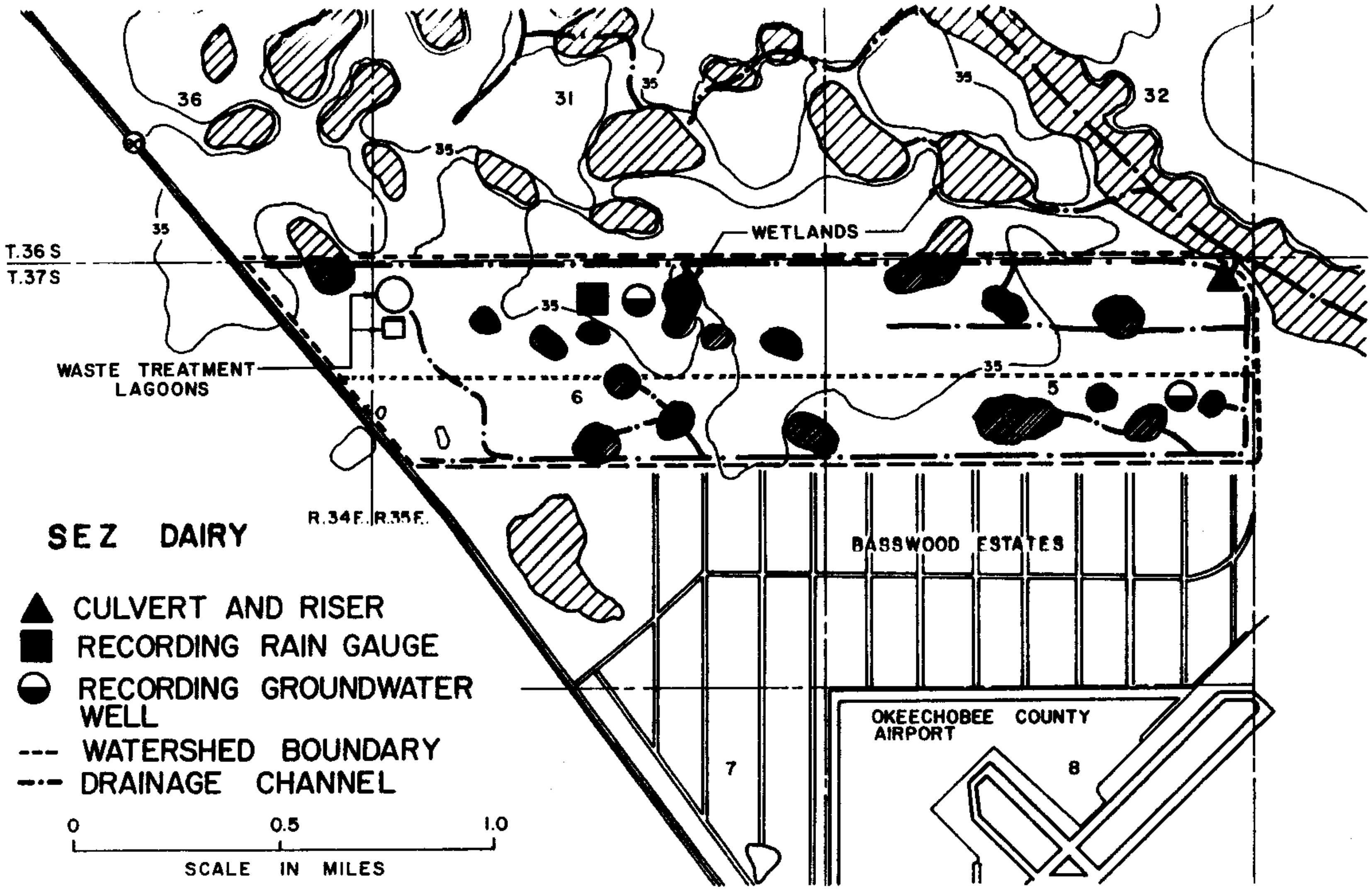


Figure 22. SEZ Dairy watershed data collection site.

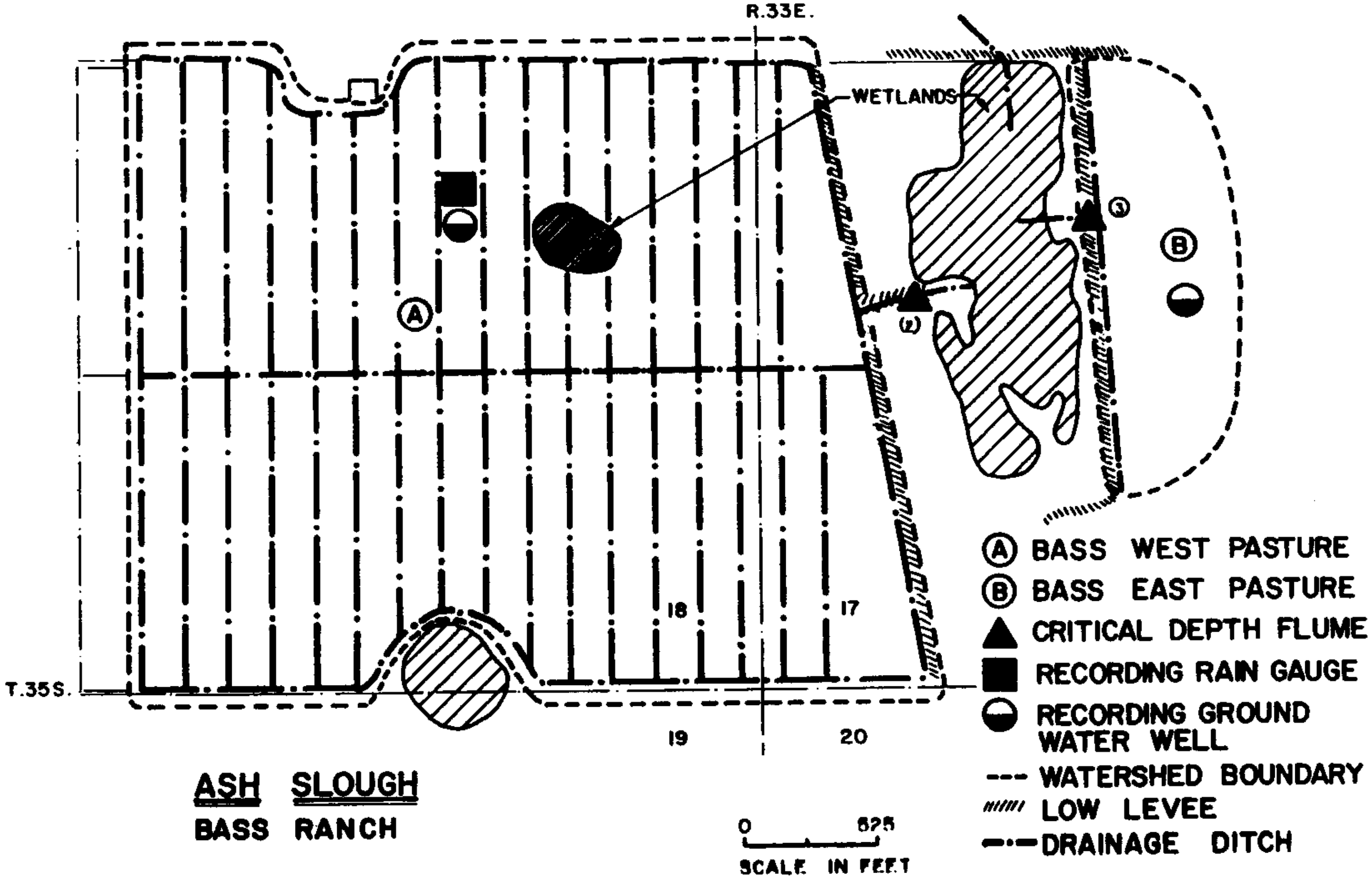


Figure 23. Ash Slough (Bass East and Bass West) data collection sites.

Bass East Pasture

The Bass East pasture is across the Ash Slough impoundment area from the Bass West site (see Figure 23). Flow recorded at its small flume is estimated to be contributed by 20 acres of a much larger pasture area. Runoff is collected in a single shallow ditch oriented perpendicular to the flume flow direction. A water table observation well is located onsite. Rainfall data are taken from the Bass West gage located approximately one-quarter mile offsite. Like the west site, land use is entirely improved pasture with no significant wetlands. Soil is uniformly Myakka fine sand.

Periods of data records are: rainfall, May 1979 to February 1983; January 1980 to July 1982; runoff, August 1979 to January 1983. A low levee directs the standing water in the ditch toward the flume. Again, like the west site, livestock traffic breached this embankment resulting in unmeasured outflow. Therefore runoff data taken after the introduction of this error (early 1981) were not included in this study.

Table 5. Watershed Characteristics.

Characteristic	Site					
	ARMS.	PEAV.	PEAV-S.	SEZ	ASH-W.	ASH-E.
Drainage Area (acres)	3600	1800	775	710	160	20
Channel Slope (ft/mi)	1.6	0.8	1.1	1.1	5.3	1.1
Overland Slope (ft/mi)	7.9	7.9	9.0	4.2	6.3	5.3
Drainage Density (mi/mi^2)	1.06	1.58	1.58	6.86	38.02	20.56
Ponds and Marsh (%)	13	23	21	7	0	0
Length/Width Ratio	3.5	2.1	2.3	4.8	1.1	0.3

CHAPTER IV

METHODS

Basic Hydrologic Analysis

The evaluation of stormwater runoff estimation techniques requires an accurate data base as a foundation. Interpretation and manipulation of raw data into forms usable for such an analysis can be a time consuming task requiring the application of several basic hydrologic tools and techniques. Among these are: data collation and editing, water budget calculations, and hydrograph analysis. Methods used to establish the data base for later analyses are described below.

Data Interpretation

Watersheds in the South Florida area are not conducive to precise hydrologic measurement and, therefore, data collected from them must be examined carefully. Acceptable data for the purposes of testing current and proposed total volume and peak rate estimation techniques include the following: 1) an accurate record of rainfall representative of that experienced by the entire watershed, preferably with a time distribution of the rainfall event, 2) a reasonable estimation of the contributing area for each runoff event, 3) an accurate record of discharge rates from the watershed, derived from a measurement system which does not significantly alter the discharge rate, 4) documentation of the watershed's antecedent moisture condition, including records of water table elevations and recent rainfall events, and 5) knowledge of other watershed physical characteristics, such as drainage improvements, topography, soil types, and land use patterns.

Table 6 is a summary of all storm events used in this study. Daily rainfall records were examined and all events equal to or exceeding 0.70 inches were recorded. Based upon the existence and quality of the corresponding runoff and water table records, the combined data were either included or excluded from the event data base used in the evaluation of the various runoff methods. Many of the 189 daily events listed in Table 6 occurred back-to-back and were, therefore, combined into multiple-day events. The actual number of separate events was approximately 160. The exact number used varied depending upon requirements of the specific method being evaluated.

Rainfall. Daily summaries and continuous strip chart records were supplied by the USGS for each raingage site. SFWMD gages (MRF32 and MRF155) reported rainfall on a daily total basis only. Armstrong and Peavine areal average rainfall depths were calculated for each event using a Theissen weighting technique. Since these events were identified from the USGS continuous records and SFWMD data are reported as 24-hour totals (not beginning and ending at midnight), the calculation of average watershed rainfall in some cases necessitated that SFWMD records be shifted by one day. This was done to accomodate records which, although representing the same rainfall event, were reported on different days at each site.

Table 6. Event Data Summary.

Data Parameter	Site					Totals
	Arms	Peav	SEZ	BassW	BassE	
Total Events[a]	91	53	85	75	75	379
Events Used[d]	34	42	33	45	35	189
> 1-Year Event[b]	2	2	1	1	1	7
> 5-Year Event[c]	0	1	0	0	0	1
With No Runoff	20	19	11	15	17	82
With Runoff	14	23	22	30	18	107
With >1 Inch Runoff	4	10	2	14	8	38
Events Not Used	57	11	52	30	40	190
No Runoff Data	0	2	17	0	9	28
No Water Table Data	3	3	7	10	6	29
Rain Data Problems	9	0	0	0	0	9
Runoff Data Problems	26[e]	4[f]	21[g]	16[h]	21[i]	88
Complex Hydrographs[j]	19	2	7	4	4	36

[a] Daily rainfall total equal or exceeding 0.70 inches.

[b] Daily rainfall equal to or exceeding 3.6 inches, based on SFWMD reported frequencies.

[c] Daily rainfal equal to or exceeding 4.5 inches, based on SFWMD reported frequencies.

[d] Many of these events will be combined into multiple day events.

[e] Mainly data coded as "estimated" and "*" (undetermined gage codes).

[f] Event data coded as "*" (undetermined gage codes).

[g] Mainly runoff events where culvert capacity (14 cfs) was reached.

[h] Mainly data coded as "*" (undetermined gage codes).

[i] Mainly due to flow bypassing flume through dike breach.

[j] Many complex runoff events were separated and used, however the number listed here could not be separated with confidence.

The USGS strip chart records were digitized into breakpoint files. A computer routine converted these files into equal increment rainfall records of 15, 30, 60 and 120-minute intervals. Equal increment rainfall distributions were then adjusted by a factor to reflect the Theissen weighted rainfall total as calculated for the specific event. Adjustment was not necessary for the SEZ and Bass sites as single gages represented rainfall on each. Incremental forms of the SFWMD 1- and 3-day distributions replaced measured distributions for events where only daily rainfall totals were available or usable. Figures 24 and 25 show example storms in breakpoint and 60-minute incremental form for both measured and assumed cases, respectively.

Using the data from the four USGS gages. rainfall frequency histograms were developed for the period of record of each. The histograms, shown in Figure 26, were used to determine the number of rainfall events of a given return period occurring during the period of record.

Comparison of USDA-SCS (1979) and SFWMD (1981) published rainfall return periods for the Lower Kissimmee River Basin showed significant disagreement between the two i.e., for a given return period, the SFWMD reported a much lower rainfall depth than did the SCS. Values given in Table 6 reflect rainfall frequencies reported by SFWMD. Only one 24-hour, 1-year return period storm was recorded but not included in the final event data base. This was a 3.72-inch rain at SEZ Dairy which occurred in 1979 prior to the initiation of runoff data collection.

Runoff. Continuous and event plots were generated from 30-minute discharge data for the determination of runoff total volumes and peak rates. Mierau (1981) describes the design criteria for runoff measurement structures at all sites as being primarily the accurate measurement of low-flow events with acceptable sacrifices in the measurement of rare high runoff events. Low-flow events are most critical for the calculation of nutrient loading which was the original thrust of SFWMD's runoff data collection effort. Based upon a modified flow duration analysis conducted on 20 years of data from a stream in the study area, Mierau concluded that less than 10% of runoff would occur at a rate in excess of 0.75 inch/day. This analysis served as the basis for design of the runoff measurement structures at these sites. Calculations to compare measured runoff against Mierau's estimate showed that 48% of runoff from Bass West occurred at a rate greater than 0.75 inch/day as did 22% of runoff from Bass East. These percentages are highly dependent upon accurate knowledge of the area contributing runoff to each structure. Therefore, inaccurate estimations of contributing area may be the cause of the apparent disagreement between observed runoff and the flow duration analysis results. The culvert at SEZ Dairy has a capacity of only 0.47 inch/day and the Peavine structure begins to submerge at a discharge rate of 0.15 inch/day (due to the increased contributing area). Neither of these sites ever recorded runoff greater than 0.75 inch/day. Armstrong site recorded only 5.5% of runoff above this threshold value, well within Mierau's estimate.

Water Table. Daily maximum water table elevations for the six locations in the study area (2 wells for SEZ Dairy, 1 for each of the other 4 sites) were plotted for visual inspection. Information was separated into recession and rising data bases for further analysis of water table dynam-

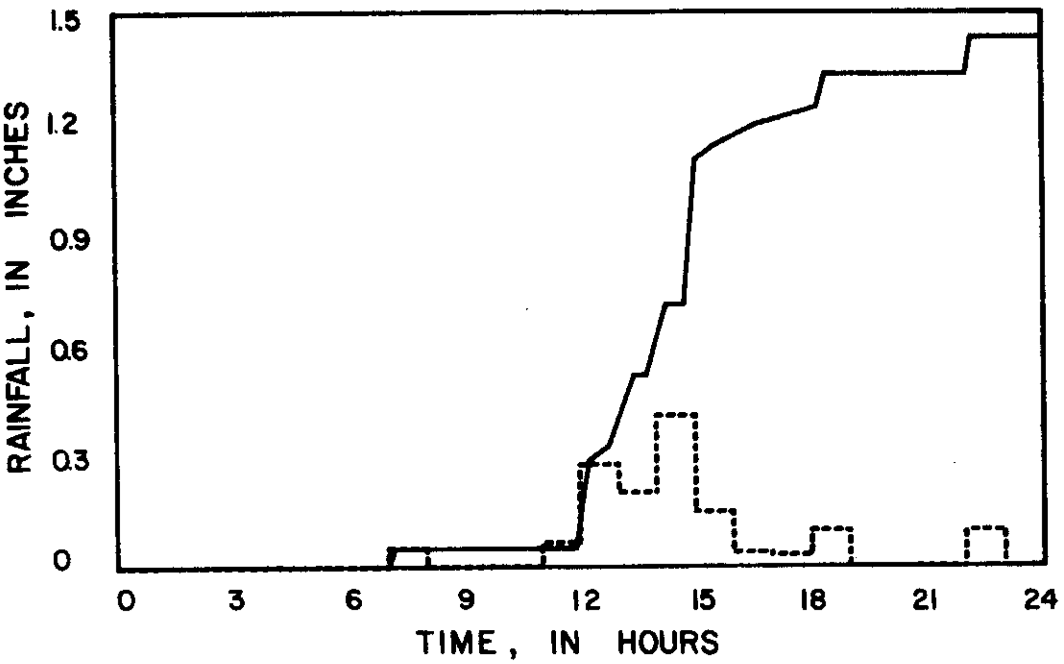


Figure 24. Example measured breakpoint and 1-hour incremental rainfall time-distributions.

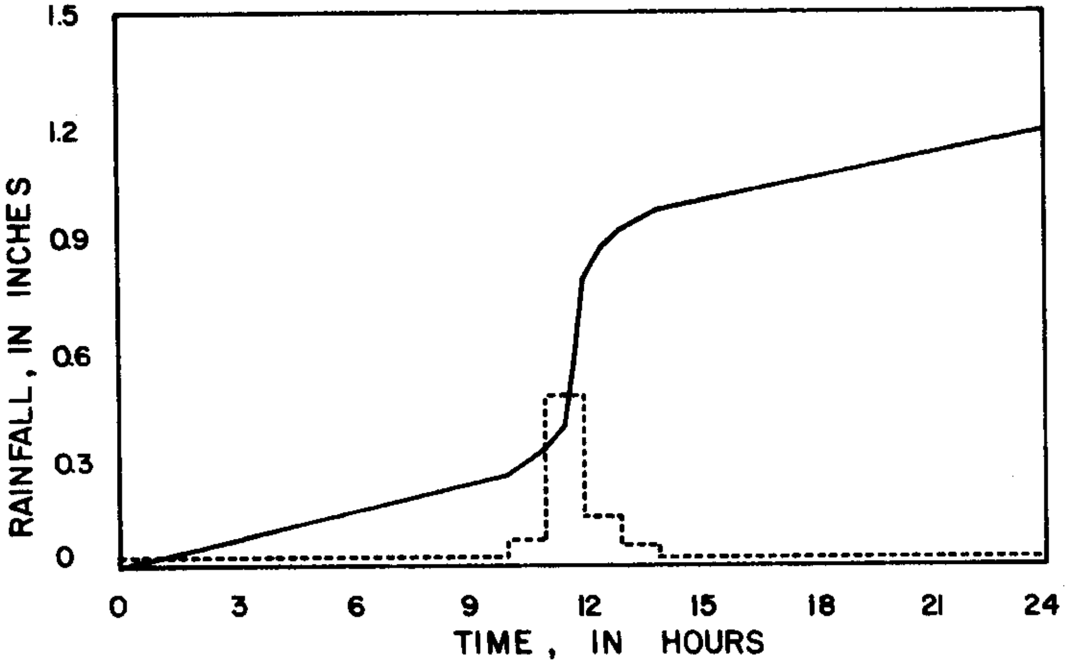


Figure 25. Example SFWMD assumed accumulated and 1-hour incremental rainfall time-distributions.

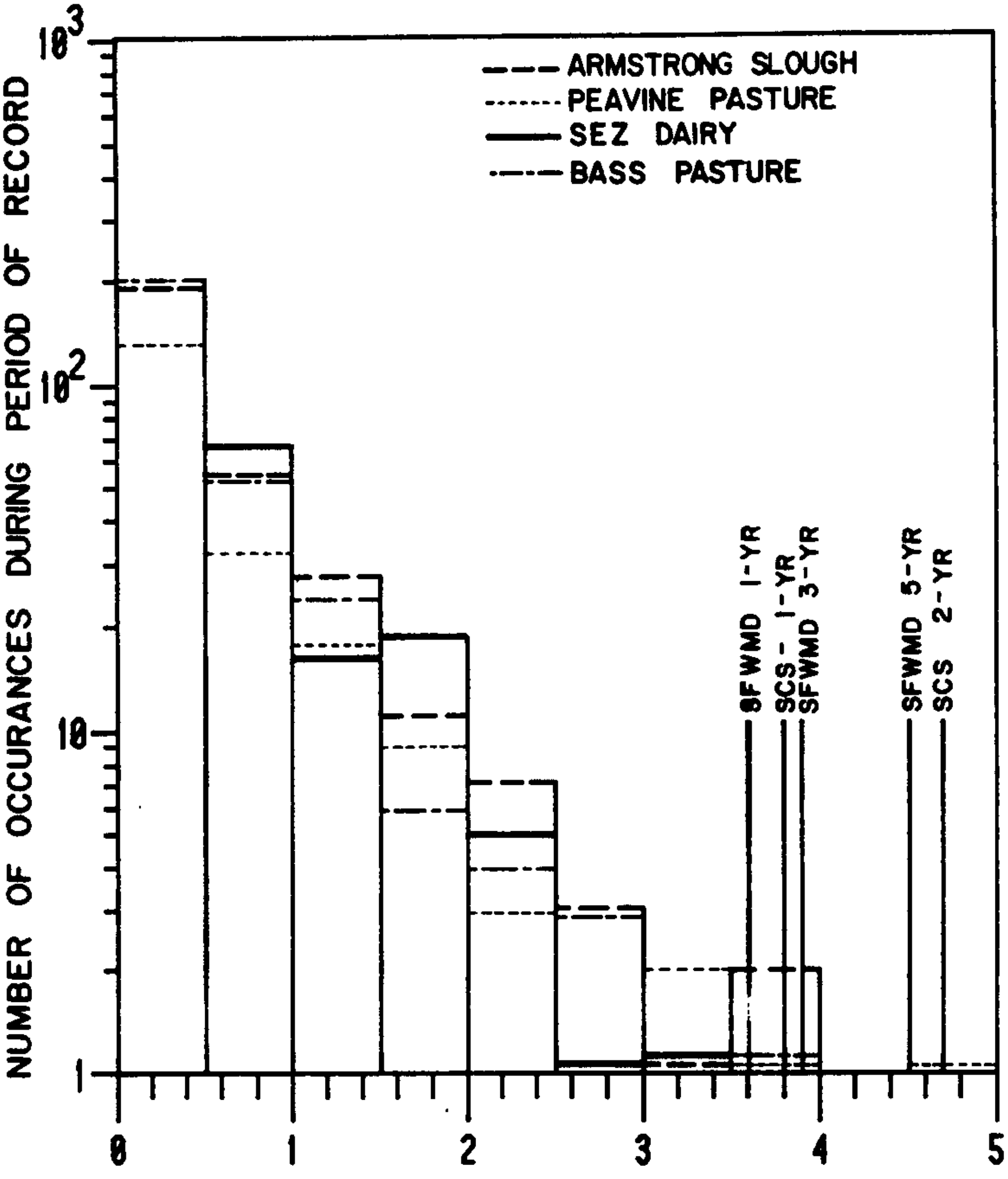


Figure 26. Rainfall record histograms for data collection sites.

ics and comparisons to results described by Speir et al. (1969). Continuous strip chart records, although available, were not copied and digitized because resolution beyond daily maximum water table elevation was not necessary for this study.

After graphical inspection of water table trends for the period of record and consultation with USGS staff, a correction of -0.88 ft. was applied to 1980 and 1981 water table elevations recorded at the Bass East well location. A similar, but unofficial, adjustment of -0.23 ft. was applied to all water table elevations for the SEZ Dairy west well location. Recorded water table elevations indicate that surface ponding of 3 inches occurred for periods in excess of two weeks. Such prolonged ponding, assuming that it did occur, does not accurately represent overall watershed conditions. For the purposes of representing general watershed water table depth, the two SEZ well readings were averaged when both were available.

Pan Evaporation. Daily and monthly total evaporation from standard U.S. Weather Bureau Class A pans at Lake Alfred AREC and Belle Glade AREC (Agricultural Research and Education Center) were compiled as available from NOAA Climatological Data Summaries, Florida, (USDC-NOAA, 1972-82). Average daily potential evapotranspiration (PET) rates in inches per day were calculated for each month based upon an 11-year average of the monthly total pan evaporation recorded at the two sites and application of an assumed coefficient of 0.70 i.e. PET = 0.7 Pan Evap. (see Figure 27). Shown in Figure 28 are comparisons between the monthly AREC average daily PET for the years of this study and the 11-year average. Daily evapotranspiration estimates were also calculated based on an average of the pan evaporation for the two sites for the period of record concurrent with that of the runoff events being examined.

The daily estimates of ET based upon concurrent periods of record proved to be unreliable for use at the study sites. Daily pan evaporation showed a high degree of variablity between the two AREC locations. Evaporation is dependent upon local weather conditions i.e., radiation, cloud cover, rainfall, and wind speed. On a daily basis these factors vary considerably from location to location. Due to this variability, the 11-year average daily ET estimates for the appropriate month were used in water budget calculations instead of the ET estimates derived from recorded daily evaporation amounts at Lake Alfred and Belle Glade.

Deep Seepage. Bimonthly water levels of 3 Floridan Aquifer wells in the Lower Kissimmee River Basin, near the study sites were compiled from USGS Water-Data Reports FL-79-2B, FL-80-2B, and FL-81-2B, (USDI-USGS, 1979-81). Average annual depths to the piezometric surface of the Floridan Aquifer beneath the study area were calculated. This information was used to estimate the significance of deep seepage based upon the gradient between the water table and the Floridan Aquifer. The annual average gradient was approximately 2.5 feet downward during 1979 and 1980. Therefore, the deep seepage component of the water budget was estimated to be negligible for the purposes of this study as was also the conclusion of Speir et al. (1969) in their hydrologic study of Taylor Creek watersheds.

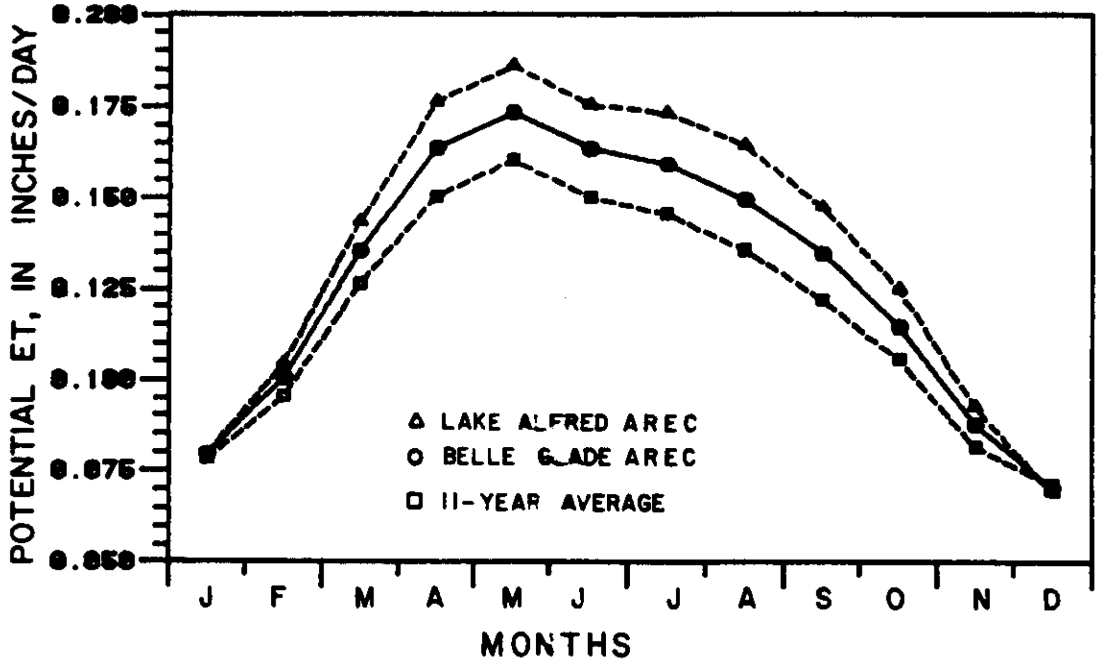


Figure 27. Potential ET rates for the study area based upon an 11-year average of data from two locations (USDC-NOAA, 1972-82).

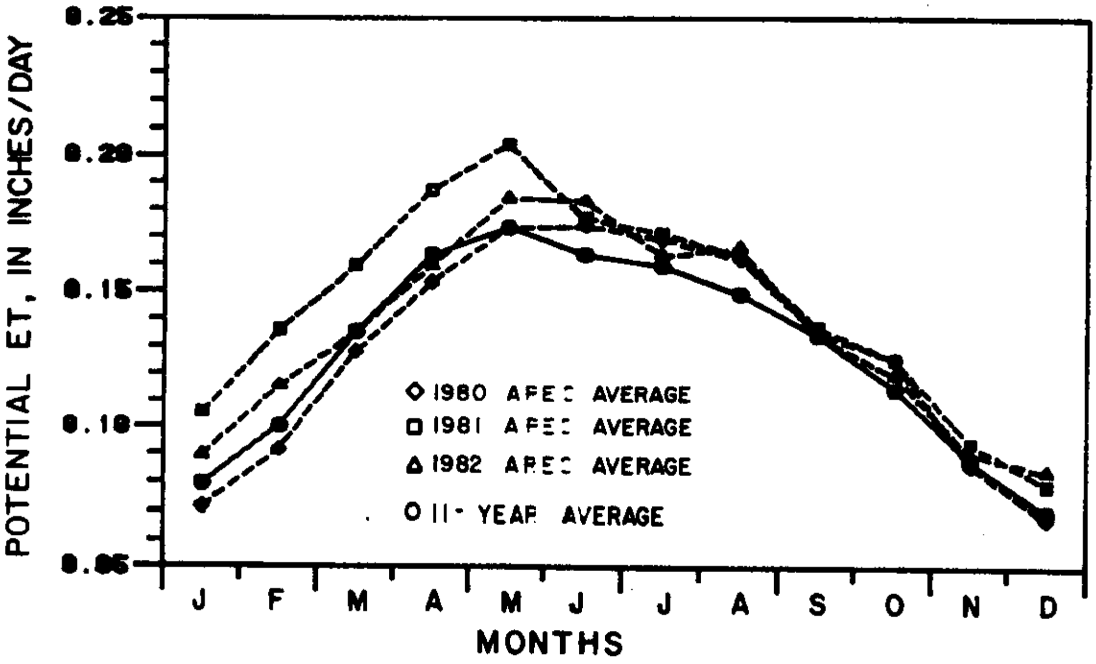


Figure 28. Study area potential ET rates between 1980 and 1982 compared to the 11-year average (USDC-NOAA, 1972-1982).

Soils. Spatial distribution of soils on each of the six watersheds was assessed from USDA SCS soil surveys of Osceola County (USDA-SCS, 1974) and Okeechobee County (USDA-SCS, 1972a). Soil types and distributions on each watershed were documented and digitized. Fractional area of watershed occupied by each soil type aided in the determination of SCS runoff curve numbers as described in Appendix I.

Hydrologic classifications of soils were found for each watershed in the SCS National Engineering Handbook - Section 4, Hydrology, (USDA-SCS, 1972b). Hydrologic classification ranges were combined with drainage characteristics for each watershed to arrive at specific hydrologic classes. All but a few of the soils occurring on the watershed sites had published hydrologic classifications. Reasonable assumptions were made for those soil types not documented in SCS NEH-4.

Topography / Land Use. Land surface and channel elevations for each watershed were determined from USGS Topographic Maps and actual site inspection. Approximate overland and channel slopes were calculated from the five-foot contour maps. Drainage basins were delineated based on topography and drainage patterns as apparent on the USGS maps. These data were supplemented with limited surveys conducted at Bass East watershed. Accuracy of these interpretations is limited by the resolution of the contour maps, therefore, slopes and delineations are subjective and somewhat approximate.

Basin delineations were refined based upon drainage patterns as apparent from Mark Hurd aerial photographs. Ponds, marshes, and swamps were delineated and digitized to determine percent wetlands for each watershed. Land use patterns combined with soils data helped determine the effective curve numbers for each location.

Water Budget

A basic water budget helped answer questions regarding data accuracy, drainage basin delineations, and rainfall variability. Components included were measured rainfall, runoff, and depth to water table (change in storage) and estimates for evapotranspiration. Deep seepage to the Floridan Aquifer was estimated to be negligible. The analysis also neglected possible external subsurface flow into perimeter ditches.

Long term water balance results are shown in Table 7. The only two complete years of record are 1980 and 1981 which, as pointed out by Huber (1982), were part of a severe drought of approximately an 30 to 50 year return period. Generalizations based upon this balance are therefore, inconclusive. However certain observations can be made regarding the sites.

Annual rainfall at Peavine Pasture is significantly lower (≈10 inches) than that recorded at Armstrong Slough only five miles away. Evapotranspiration, calculated as the remainder of the water balance, was correspondingly lower than the other sites except Bass West. Initial water balance

Table 7. Water balance summary by calander year, all values in inches.

Year	Site	P[a]	-	RO[b]	-	S[c]	=	ET[d]
1980	Armstrong	37.8		1.65		0.86		35.3
	Peavine	25.7		1.62		-0.58		24.7
	SEZ Dairy	36.2		4.19[e]		0.29		31.7
	Bass West	43.8		23.9		0.00		19.0
	Bass East	43.8		12.4		0.00		31.4
1981	Armstrong	36.5		5.58		-1.73		32.7
	Peavine	28.4		6.28		0.29		21.8
	SEZ Dairy	33.2		5.09[e]		-0.86		29.0
	Bass West	25.0		0.20		-1.15		26.0
	Bass East	25.0		0.15		-1.15		26.0
1982	Armstrong	55.0		30.2		-		24.8
	Peavine	-		22.0		-		-
	SEZ Dairy	51.0		24.4[e]		-		26.6
	Bass West	-		35.1		-		-
	Bass East	-		14.3		-		-

[a] Rainfall measured by USGS gage.

[b] Calculated with SFWMD reported discharge and estimated drainage areas.

[c] Change in storage as reflected in water table elevations as measured at USGS observation wells and storage change from Figure 2.

[d] Estimated as remainder of the mass balance accounting.

[e] Runoff measured at drop inlet culvert minus discharge from lagoon.

- Indicates insufficient data for determination.

calculations at Peavine indicated an area adjustment was necessary. These calculations, combined with hydrograph analysis, led to the drainage area adjustment mentioned in the site description.

Runoff volume measured at the Bass West flume was significantly higher than the east site as well as all other sites for 1980 and 1982. On an event basis, several Bass West storms were deleted from the analysis because measured runoff exceeded measured rainfall. Considering the size of the watershed (160 acres), the central location of the raingage on the watershed, and annual totals comparable to the other sites, rainfall is not a likely source of error. The Bass West site does have the most extensive drainage network of all sites and also a 3-4 ft. perimeter ditch, factors which might contribute to the high volume of observed runoff. Breaches in the surrounding levee may also have allowed inflow during some large runoff events.

Hydrograph Analysis

The SCS runoff equation is only intended to predict direct runoff. Therefore, the significance of base flow for the study watersheds must be ascertained. Given base flow, this portion of the discharge hydrograph must be separated before the calculation of runoff volume. The standard terms of overland flow, interflow, and base flow are difficult to apply to flatwoods watersheds. Accepted interpretations associate these terms with particular flow processes and specific time intervals on discharge hydrographs. Although the same flow processes occur on flatwoods watersheds, they are often difficult to separate and not associated with the same hydrograph time interpretations. Direct runoff is typically defined as including overland flow and interflow. Base flow is defined as being nearly constant and originating from groundwater. The rule of thumb usually employed to separate direct runoff from base flow given by Linsley et al. (1975) is:

$$N = A^{0.20} \qquad [39]$$

where N = number of days after the hydrograph peak at which time direct runoff is considered to have terminated and
A = drainage area in square miles.

For the five watersheds included in the Upland Detention Project, this N value ranges from 0.5 to 1.5 days. However, data collected as part of the project, indicate that this definition of direct runoff would remove a significant amount of what appears to be direct runoff (see Figure 29). A separation technique reported by Istok et al. (1983) yields very similar results. Much of the runoff these methods would define as base flow is probably groundwater by flowpath, but is directly associated with the recent rainfall event and is still distinctly receding. More appropriate terms to describe runoff from flatwoods watersheds are rapid, intermediate, and slow flow. These terms do not attempt to explain process, but refer only to flow rate (Speir et al., 1969).

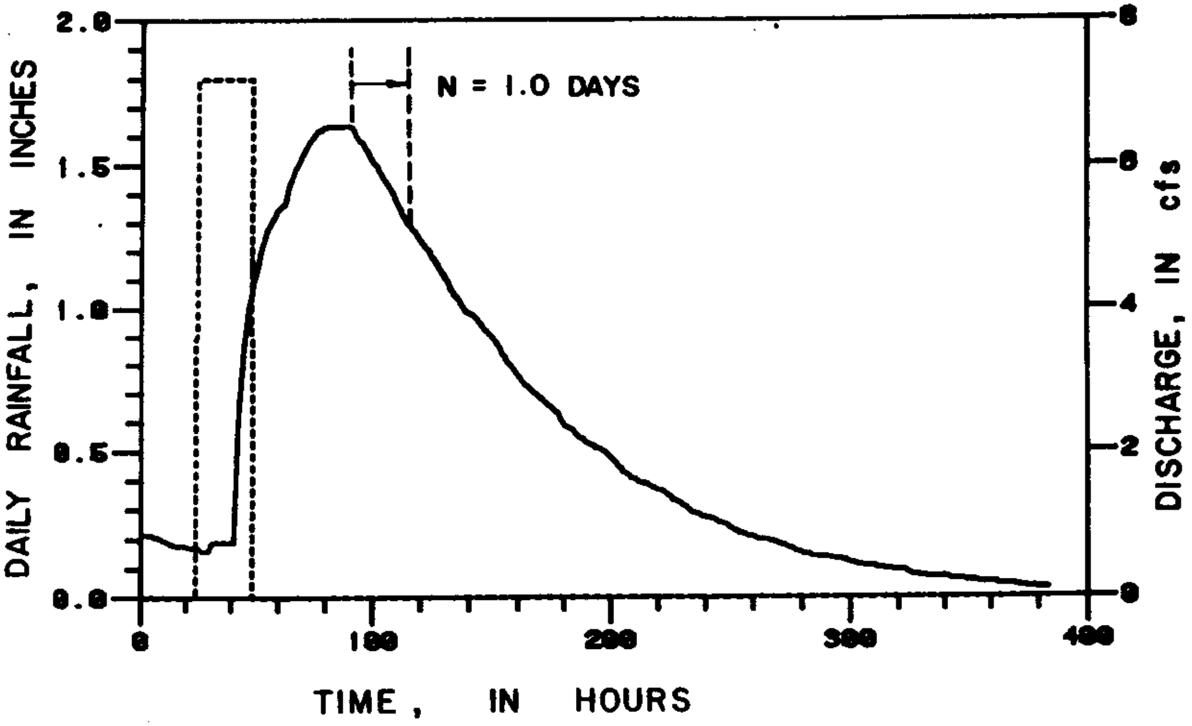


Figure 29. Recorded rainfall and runoff for Peavine Pasture beginning 4/25/82. The standard estimate of the end of direct runoff (1 day) for this 775 acre watershed is shown.

Speir et al. also analyzed the recession characteristics of Taylor Creek watershed hydrographs and reported K values for rapid, intermediate and slow flow. This K ranged from 2 to 4 for rapid, 6 to 9 for intermediate, and 10 to 15 for slow flow and represents the time in days for flow rate to decrease by one log cycle. Analysis of representative hydrographs from each study site yielded rapid K values of about 7 for watersheds with greater than 20% wetlands (Peavine), 4 for watersheds with 7-13% wetlands (Armstrong and SEZ), and 2 for watersheds with less than 1% wetlands (Bass East and West). SEZ Dairy lies within the ARS watershed W-2. The ARS study reported a rapid K of 4.11 for W-2 which agrees with the value of 4.16 determined for SEZ.

Semi-log plots used to arrive at these K values showed that base flow was discernable on only a few hydrographs and for Peavine was nonexistent, i.e., an extremely constant K characterized Peavine's hydrograph recessions. This can be explained by the watershed's significant quantity of wetlands from which flow recedes in a manner described by a linear reservoir model. The absence of base flow from watersheds with high percentages of wetlands agrees with the conclusions of Carter et al. (1978). This phenomenon is explained as resulting from high soil storage capacity combined with open water surfaces and dense vegetation. These features tend to consume available water via ET at the expense of base flow.

Based upon examination of hydrographs, base flow was not considered to be significant for these watersheds when examined on an event basis. Therefore, determination of direct runoff consisted primarily of the separation of complex hydrographs where reasonable (see Figure 30). Because of slow response characteristics many hydrographs tended to merge together making separation impossible. Such cases were not included in the event data base used in this study (see Figure 31).

Hydrograph analysis also supported the conclusion of a variable contributing area for Peavine Pasture. Runoff data documentation indicated that at a discharge of about 8 cfs, upstream water surface elevation was approximately equal to that of the surrounding land, implying bank-full conditions and overland flow. Initial basin delineation was based upon the effectiveness of the artificial block in one of two contributing channels. However when the watershed shifted from a channel runoff mode to a sheetflow mode, this block is believed to have been ineffective. An estimate of watershed boundaries, ignoring the channel block, suggested a contributing area of 1800 acres.

Water Table

Konyha et al. (1982) reported best estimates for total runoff volume were achieved by the DRM method which uses measured water table depths. The CR-2 method attempted to model soil moisture as a substitute for water table measurements, but did not perform as well. A simplified computer model was developed as part of this study in an effort to predict water table levels for use in runoff volume estimation techniques. The model also serves to point out differences in the water table response characteristics attributable to site specific soil and drainage characteristics,

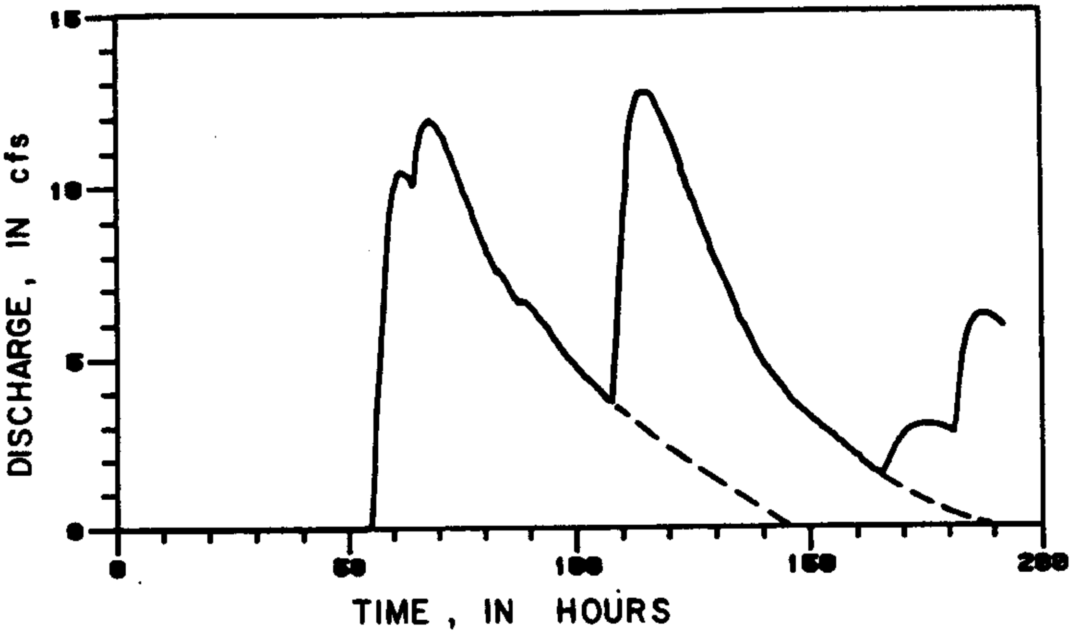


Figure 30. Runoff recorded at Bass West Pasture beginning 6/18/82 and showing a typical hydrograph separation.

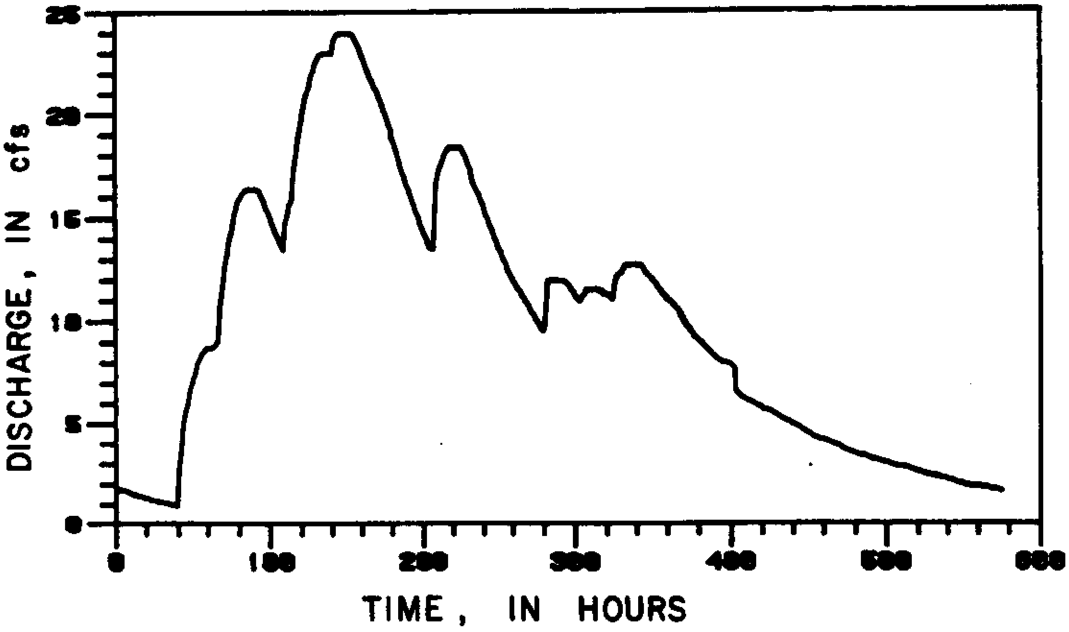


Figure 31. Runoff recorded at Peavine Pasture beginning 9/5/81 and showing a case where complex hydrographs could not reasonably be separated.

independent of rainfall variability. Water table movements were divided into two submodels, rise and recession. Simulation was conducted on a daily time step.

Rise is simulated as a filling of available storage equal to the rainfall depth while soil profile available storage is calculated as a function of the state variable DWT (depth to the water table). Figure 32 shows the relationship between DWT and storage. This curve was interpreted from information published by Speir et al. (1969) shown in Figure 2. An average of the original ARS absorption curves was extended to a depth of six feet and fitted with four linear segments:

$$\begin{aligned}
AS &= 0.50(DWT) & 0.5 > DWT > 0.0 \\
AS &= 0.26 + 1.09(DWT - 0.5) & 1.0 > DWT > 0.5 \\
AS &= 0.81 + 1.26(DWT - 1.0) & 1.5 > DWT > 1.0 \\
AS &= 1.44 + 1.50(DWT - 1.5) & 6.0 > DWT > 1.5
\end{aligned} \qquad [40]$$

where AS = soil profile available moisture storage in inches and
DWT = depth to water table in feet.

The storage relationship does not account for differing soil types, but simply represents an area median.

Water table recession is modeled as only a time-dependent function as shown in Figure 4 from the ARS study. An equation was fitted to this recession curve and has the form:

$$DWT = M(DAY)^X \qquad [41]$$

where DWT = depth to the water table and
DAY = rainless days.

The water table depth for each subsequent rainless day can be expressed as:

$$DWT_{t+1} = M\left[\left(\frac{DWT_t}{M}\right)^{(1/X)} + 1\right]^X \qquad [42]$$

where DWT_t = initial depth to water table,
DWT_{t+1} = next day's depth to water table,
M = 0.918, and
X = 0.355.

To more realistically represent recession characteristics, the model holds water table depth constant the day following a rainfall event. This is intended to reflect the lack of ET extraction from the soil due to interception moisture availability. The slow runoff characteristics of these watersheds also tends to maintain soil saturation for a day following a large rainfall. No attempt was made in this model to account for seasonal variability, drainage improvements or site specific soil properties.

Storm Runoff Total Volume

The event data base documented in Table 6 was subjected to the stormwater volume techniques described in Chapter II. The NEH-4, SCS-Florida, and CR-2 methods were applied unchanged from the Konyha et al., (1982) analysis. All methods were translated into Fortran code for ease of execution. Techniques which were introduced or modified are outlined below.

ARS

The technique herein referred to as the ARS method represents a synthesis of the DRM method and an available storage relationship (equation 40) adapted from the ARS absorption curve (Figure 2). Runoff volume is calculated using the SCS runoff formula (equation 12) and the storage as determined by:

$$S = S_{ARS} \qquad [43]$$

where S = SCS watershed storage parameter and
S_{ARS} = available storage from equation 40 or Figure 32.

The ARS-based storage curve was chosen as a substitute for the SFWMD curve since it agreed more closely with maximum available storage observations made by Allen (1982). The relationship between runoff volume and rainfall as quantified by the SCS runoff equation and ARS storage curve is expressed graphically in Figure 34.

CR-1

The CR-1 method was modified to use the ARS DWT-storage relationship as opposed to the SFWMD relationship as was used in the original version of this method. The modified form of equation 18 is:

$$S = S_{max}\left(\frac{S_{ARS}}{5.0}\right) \qquad [44]$$

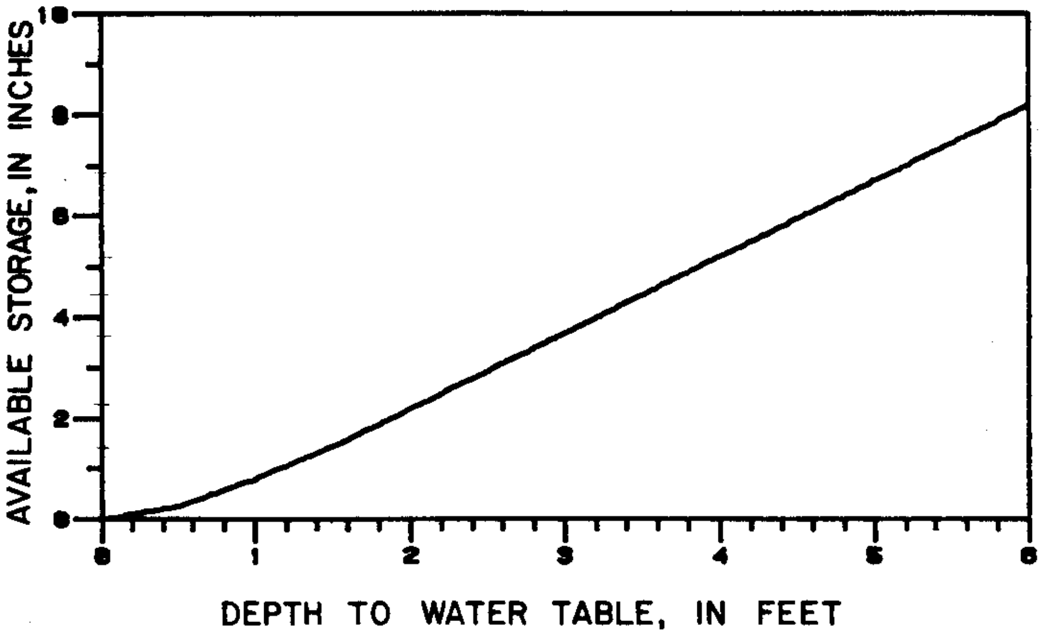


Figure 32. Soil profile moisture storage capacity as a function of depth to the water table for use in the ARS Method.

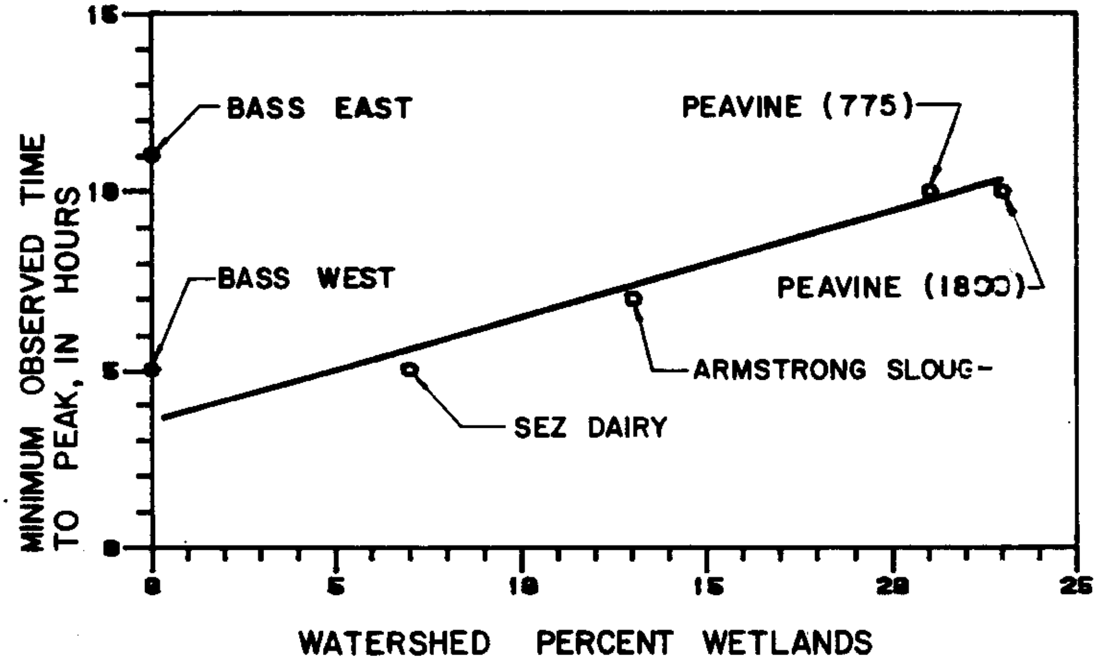


Figure 33. Correlation between watershed minimum observed time to peak and watershed percent wetlands which served as the basis for the modified lag estimation equation.

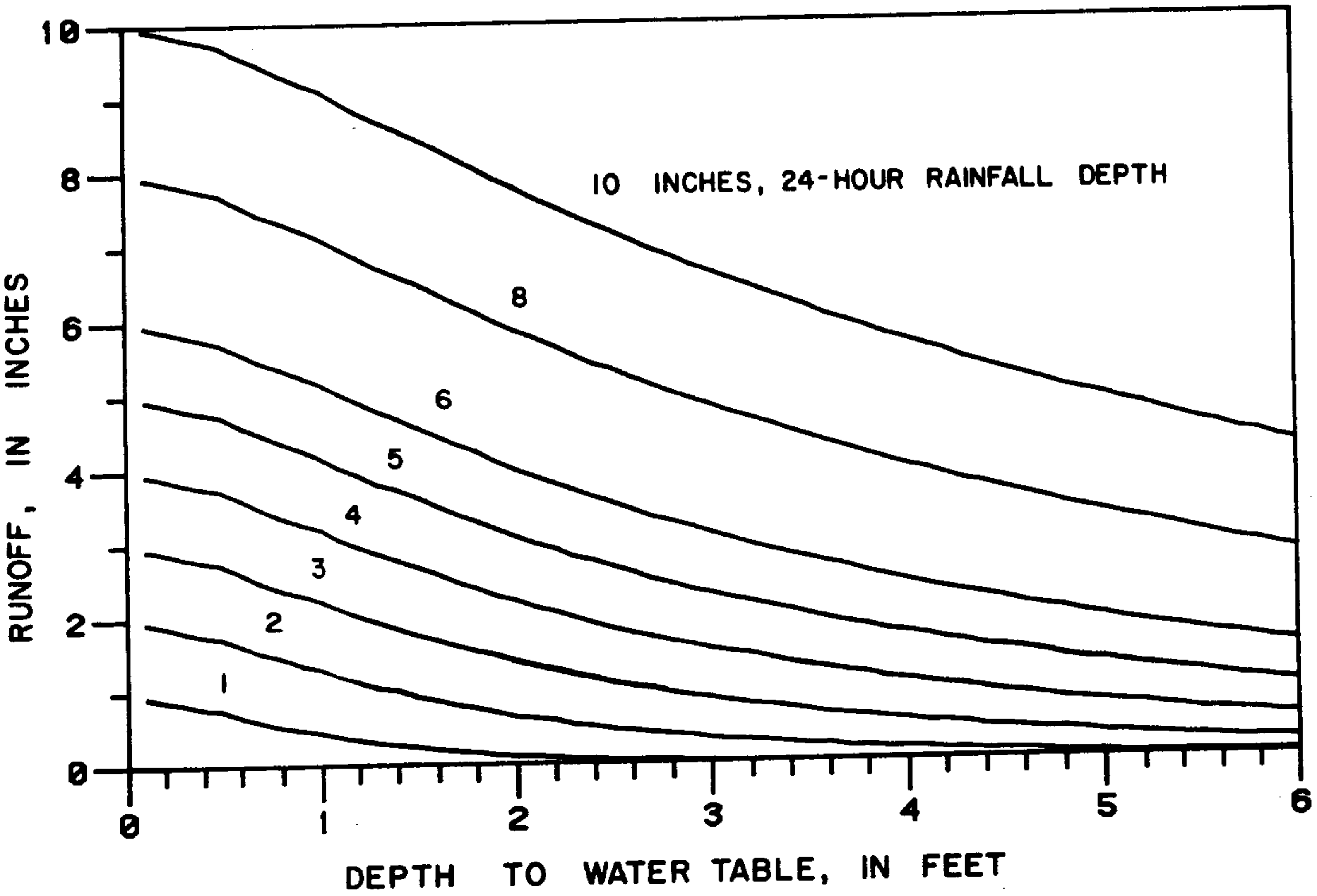


Figure 34. Solution of the SCS runoff equation using watershed storage parameters as determined from Figure 32 (ARS Method).

where S = SCS watershed storage parameter and
S_{max} = maximum value of storage parameter from equation 17.

CR-WT

Implementation of the CREAMS-WT model requires weather data inputs of daily rainfall and temperature plus monthly radiation. Temperature and radiation data were not available at the study sites, therefore, long-term average data from the Fort Pierce AREC and Belle Glade AREC were used for all years at all sites (see Figures 16 and 17). Generalized soil porosity data were also used, with Myakka fine sand considered representative of all sites.

Storm Runoff Peak Rate

To avoid compounding errors, all methods for estimating stormwater peak discharge rates were evaluated on the basis of measured runoff volume. Details of implementation and modifications of specific techniques outlined in Chapter II are described in the following sections.

Cypress Creek Formula

The Cypress Creek Formula (equation 21) was applied to each watershed and its associated rainfall. The formula is very simple and requires only that drainage area and rainfall excess be estimated. Rainfall excess was taken to be the measured runoff volume for each storm event and drainage areas are listed in Table 5. Equation 23 was used to transform the maximum 24-hour rate calculated by the Cypress Creek Formula into an instantaneous peak rate.

CREAMS Equation

Procedures for evaluating the CREAMS peak rate equation included: (1), the implementation of the equation in its original form (equation 24), (2), a regression to determine best-fit coefficients for the study data base, and (3), the introduction and substitution of additional independent variables into the model.

The Statistical Analysis System (SAS) package performed the regression and arrived at estimates for the two factors and five exponents used in the CREAMS equation. The regression was conducted by the SAS nonlinear procedure (NLIN) solved with the multivariate secant method (DUD). This method estimates model partial derivatives from the history of iterations rather than being supplied analytically (Goodnight and Sarle, 1982). Analytically derived partial derivatives were determined, but a regression based upon them failed to converge. The original CREAMS coefficients served as the initial guesses required by the DUD solution procedure.

Efforts at introducing additional independent variables of possible significance on flatwoods watersheds (percent wetlands, drainage density, and overland slope), were inconclusive. Since only five sites or degrees of freedom exist for each watershed parameter, regression against many independent variables was not reasonable. Furthermore, some watershed parameters showed high degrees of cross-correlation, particularly percent wetlands, channel slope, overland slope and drainage density. Given this cross-correlation little improvement in peak rate estimates were seen when more than one of these independent variables were introduced into the model.

SCS Graphical Method

The SCS Graphical Method was not applied to each storm event of the study data base. Instead, estimates for time of concentration, T_c, were determined by hydrograph analysis and plotted against measured peak discharges for runoff events exceeding 0.50 inches. The resulting pattern for each site was delineated and compared against the SCS curve. Since T_c estimates were extremely subjective, estimates from the lag method (equations 26 and 27) served as plotting positions for the average observed csm value for each site. Given the unusual characteristics of the flatwoods watersheds, an attempt was made to better estimate T_c. Minimum observed hydrograph times to peak were plotted against watershed percent wetlands (Figure 33). All sites except Bass East appeared to follow a trend. Hydrographs from the Bass East site displayed unusually long times to peak for its size. The apparent correlation between T_p and percent wetlands served as the basis for the equation:

$$L = 3.0 + 0.34(A^{0.11})(W+1)^{0.71} \qquad [45]$$

where L = watershed lag in hours,
A = drainage area in acres, and
W = percent wetlands.

T_c estimates, determined from this equation and the SCS relationship between lag and T_c (equation 26), also served as alternate plotting positions for the average observed csm values.

SCS Chart Method

Application of the SCS Chart Method yields peak discharge estimates in terms of cfs per inch of runoff. Using the SCS-Florida conclusion that CN should only be calculated at AMC=II, a watershed will have a unique cfs/in factor. Evaluation of the chart technique required first the estimation of the watershed factor and then application of that factor to each storm event in the study data base.

The SCS Chart Method could not be applied directly to the sites. Determination of the watershed factor requires adjustment factors for slope and percent swamps and ponds (see equation 28). Standard SCS tables used for identifying the appropriate slope factor (F_s) do not include the extremely small slopes observed on the study watersheds. Therefore, factors reported in Table 3 corresponding to slopes less than 0.10% represent extrapolated values. Extrapolation involved plotting the adjustment factors versus percent slope on log-log paper. Lines fitted to these plots were then extended down to 0.01%.

The SCS swamps and ponds factor (F_p) table (Table 4) also required extrapolation before being applied to the observed data. SCS techniques are intended for application to large design storm events with recurrence intervals exceeding 2 years. The data base of this study included very few storms of this magnitude (see Figure 26). The SCS table applicable to the Armstrong, SEZ Dairy, and Peavine sites is that for watersheds with swamps and ponds spread throughout the basin, as opposed to being located either near the outlet or only in the remote portions of the basin. Values from this table were plotted on log-log paper with storm frequency, in years, as the abscissa and adjustment factor as the ordinate. The linear fit to this plotting was not as good as for the slope curve, but was better than attempted linear fits on linear, semi-log, and probability paper. The extrapolated adjustment factors corresponding to a storm frequency of 0.1 years differed from SCS values by only about 5%. Therefore, a uniform F_p taken at 1.0 years was applied to all events.

SCS Unit Hydrograph Method

Application of the SCS unit hydrograph technique was conducted first by applying the method as presented in the SCS NEH-4 and second by modifying the technique to better fit the observed data. Analysis of all the storm events required that several computer programs be employed to compute and combine the incremental hydrographs.

The SCS Unit Hydrograph Method herein described as "standard" includes a change in the implementation described by NEH-4 (USDA-SCS, 1972b). Instead of applying the SCS runoff volume equation to each increment of rainfall, all rainfall not appearing as runoff was extracted from the beginning of the storm. This was judged more appropriate since total soil storage, not infiltration rate, appears to be the limiting factor for runoff. It was also the only feasible implementation which would allow separation of the model's routing component from its volume component.

Equation 34, describing the triangular unit hydrograph relationship, contains two measured quantities (drainage area, A, and runoff volume, Q) and two estimated parameters (hydrograph factor, K', and time to peak, T_p). The SCS estimates for K' (484 normally and 300 for flat, swampy areas) and time to peak from the lag method were used to evaluate the standard approach. The watershed storage factor included in the lag equation was calculated by two approaches. Storage determined at CN_{II} gave a fixed watershed lag, while S determined by S_{ARS} (equation 43) allowed lag to vary with watershed wetness.

Rainfall time-distributions used by the computer program included both the SFWMD assumed and that measured by the USGS. For the larger watersheds

(Armstrong, SEZ Dairy, and Peavine) a time increment (ΔD) of 60-minutes was sufficient. This value appeared reasonable after test runs on selected events showed little difference between peak estimates derived from the 15, 30, and 60-minute time increments. However, for the smaller watersheds (Bass West and East), a 15-minute time increment was necessary to avoid missing the hydrograph peak.

In modifying the unit hydrograph method, best-fit hydrograph factors (K') and revised time to peak estimates (T_p) were substituted for the standard SCS values. Time increments identical to those described above served as the time step for the iterative computer search program which was applied to each storm event and yielded best-fit K' values. For each storm, both the assumed and measured rainfall time-distributions plus various time to peak estimates were considered in the K' optimization program. T_p estimates for each watershed were found by five methods. Two, as described above, are fixed and variable estimates from the lag method. A third set was derived from the minimum observed times to peak as shown in Figure 33. The fourth set of estimates is from equation 45 and the fifth from a variable form of the same equation:

$$L = 3.0 + 0.34(A^{0.11})(W+1)^{0.71}(S+1)^{0.50} \qquad [46]$$

where S = SCS watershed storage parameter.

The last term of this equation allows lag to vary over the same range as does the SCS variable lag (1-3 times the fixed estimate). No attempt was made to base the variability of the lag quantity on observed hydrographs. The variable term used by the SCS was simply added to the modified lag equation.

A computer solution scheme applied to all these parameter combinations permitted observation of changes in best-fit K' attributable to time to peak as well as rainfall distribution estimates.

SFWMD Model

To evaluate stormwater routing calculations independent of the total volume calculations required that portions of the SFWMD overland flow model be shut down or modified. Other factors relating to the model's rainfall excess prediction method also made this action necessary.

First, as implemented, Horton's equation allows infiltration to continue after the defined soil storage has been filled. Although the final rate is only 0.01 inch/hour, this can be significant over the long timebases of the observed discharge hydrographs (>100 hours). This does not significantly impact the runoff volume occurring up to the time of maximum discharge rate (<24 hours), but does make evaluation of error attributable to routing difficult to differentiate from error due to runoff volume.

Secondly, the high surface retention depth (2 inches) requires that at least two inches of rainfall, over and above infiltration extraction, be available before any runoff will be observed. Therefore, given the particular infiltration scheme employed and the surface retention depth, runoff volume errors on the order of 4 inches can be expected if, for example, a rainfall event falls upon a saturated soil profile and runoff continues for a period of 200 hours. Such a model might be reasonable for use on storms with rainfall depths on the order of 10 inches, but is not acceptable for the data base of this study.

To force the overland flow model to route off the measured water volume, the retention depth was reduced to 0.25 inches. The infiltration procedure was also modified to halt when the remaining rainfall equaled measured runoff volume, after accounting for the retention depth and interception losses.

CHAPTER V

RESULTS

Storm Runoff Total Volume

Seven techniques for estimating stormwater runoff volume were applied and evaluated on an event basis. The events which served as the data base are summarized in Table 6 and detailed in Appendix II. These selected storms measured 0.70 or more inches of rainfall in 24 hours and may or may not have produced measurable runoff.

The seven runoff volume estimation methods applied to the data set were: NEH-4, SCS-Florida, DRM, ARS, CR-1, CR-2, and CR-WT as described in Chapter IV. These fall into three general groups all of which use the basic SCS runoff equation, but differ in their determination of the watershed storage parameter. The first two, NEH-4 and SCS-Florida, employ variations of the curve number approach. The second two, DRM and ARS, simply use measured depth to the water table and storage curves. The last three, CR-1, CR-2, and CR-WT, use variations of the CREAMS weighting method. Figures 35 through 41 show how results from each prediction method compared against measured data. Results are presented for individual watershed location as well as for all storm events combined.

Tables 8 through 10 present the same results as standard error of estimates determined with the following equation:

$$\varepsilon = \left[\frac{\sum_{i=1}^{n} (Q_i' - Q_i)^2}{n - 1}\right]^{0.5} \qquad [47]$$

where ε = standard error of estimate in inches,
Q_i' = predicted runoff volume for event i in inches,
Q_i = measured runoff volume for event i in inches, and
n = total number of storm events.

Each table represents technique performance as applied to selected classes of events: all of the rainfall events from Table 6, the subset which produced measurable runoff, and a smaller subset where measured runoff equaled or exceeded 0.50 inches.

Standard errors corresponding to "all" sites do not weight each watershed equally. Instead, the overall method standard error of estimate is most heavily weighted toward the sites which had more usable events. The ranking corresponding to "all" sites was determined by comparing the sum of the methods' performance ranking for each site and, therefore, weights performance on each watershed equally.

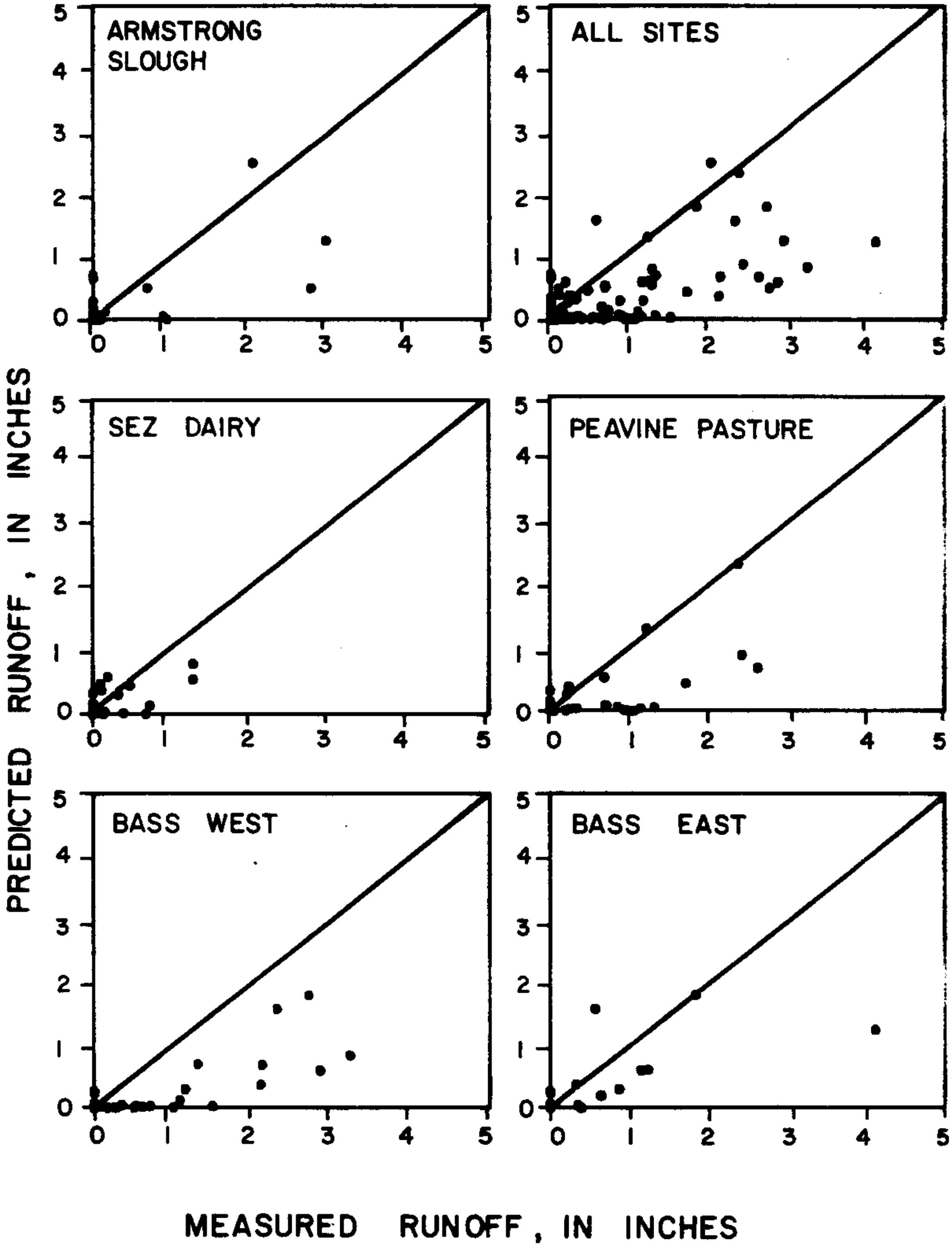


Figure 35. Comparisons of measured runoff volumes and estimates from the NEH-4 Method.

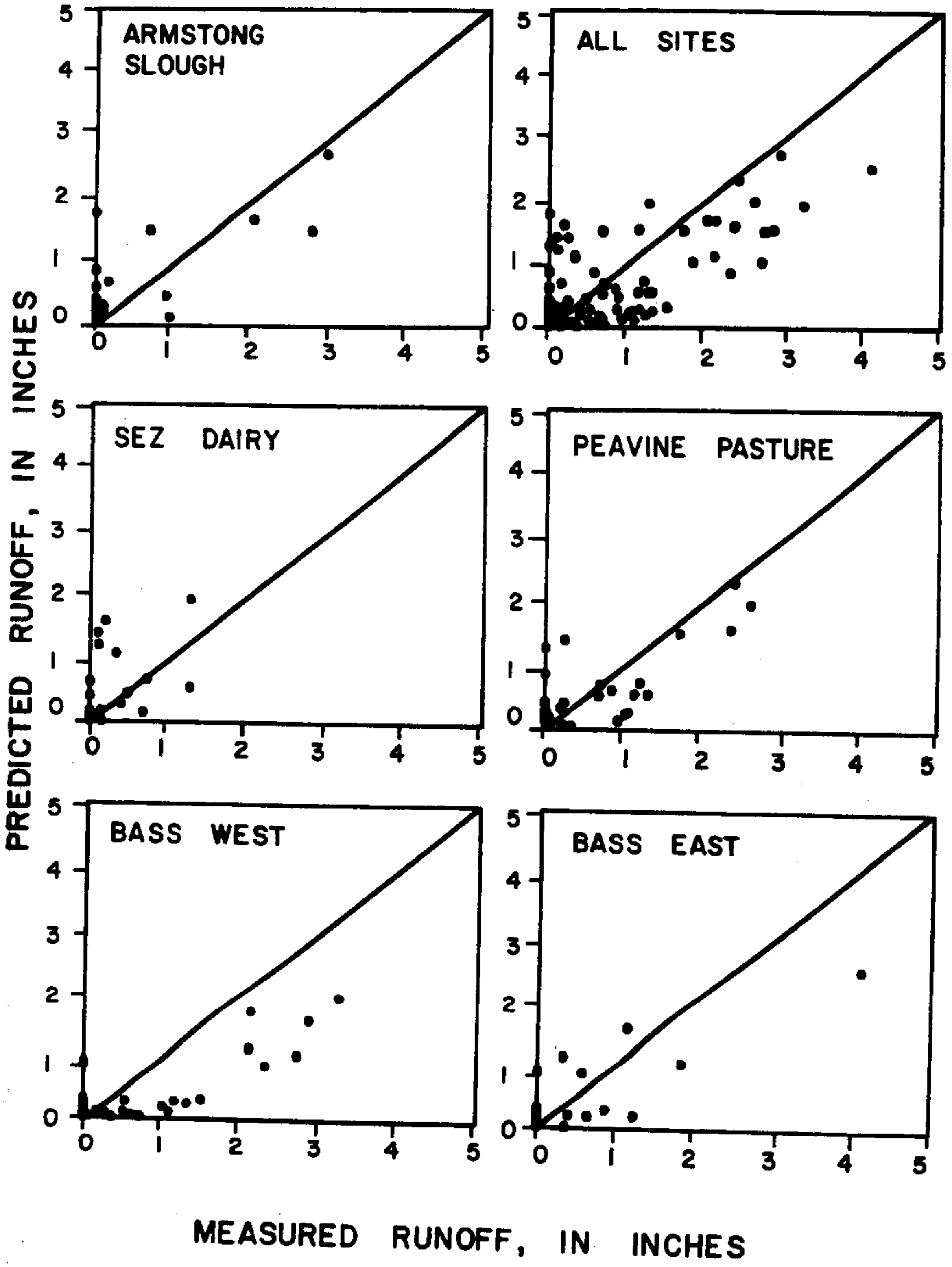


Figure 36. Comparisons of measured runoff volumes and estimates from the SCS-Florida Method.

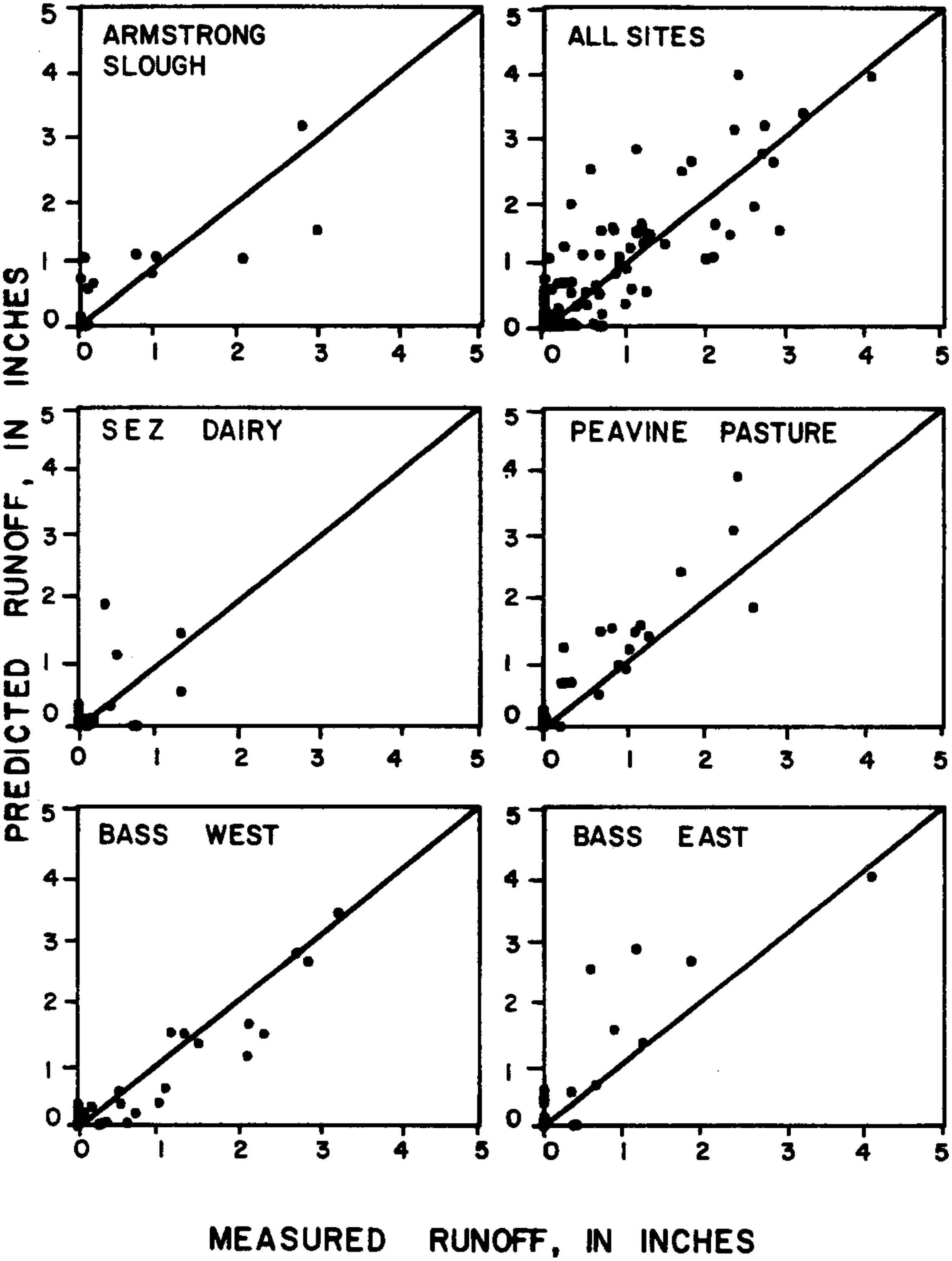


Figure 37. Comparisons of measured runoff volumes and estimates from the DRM Method.

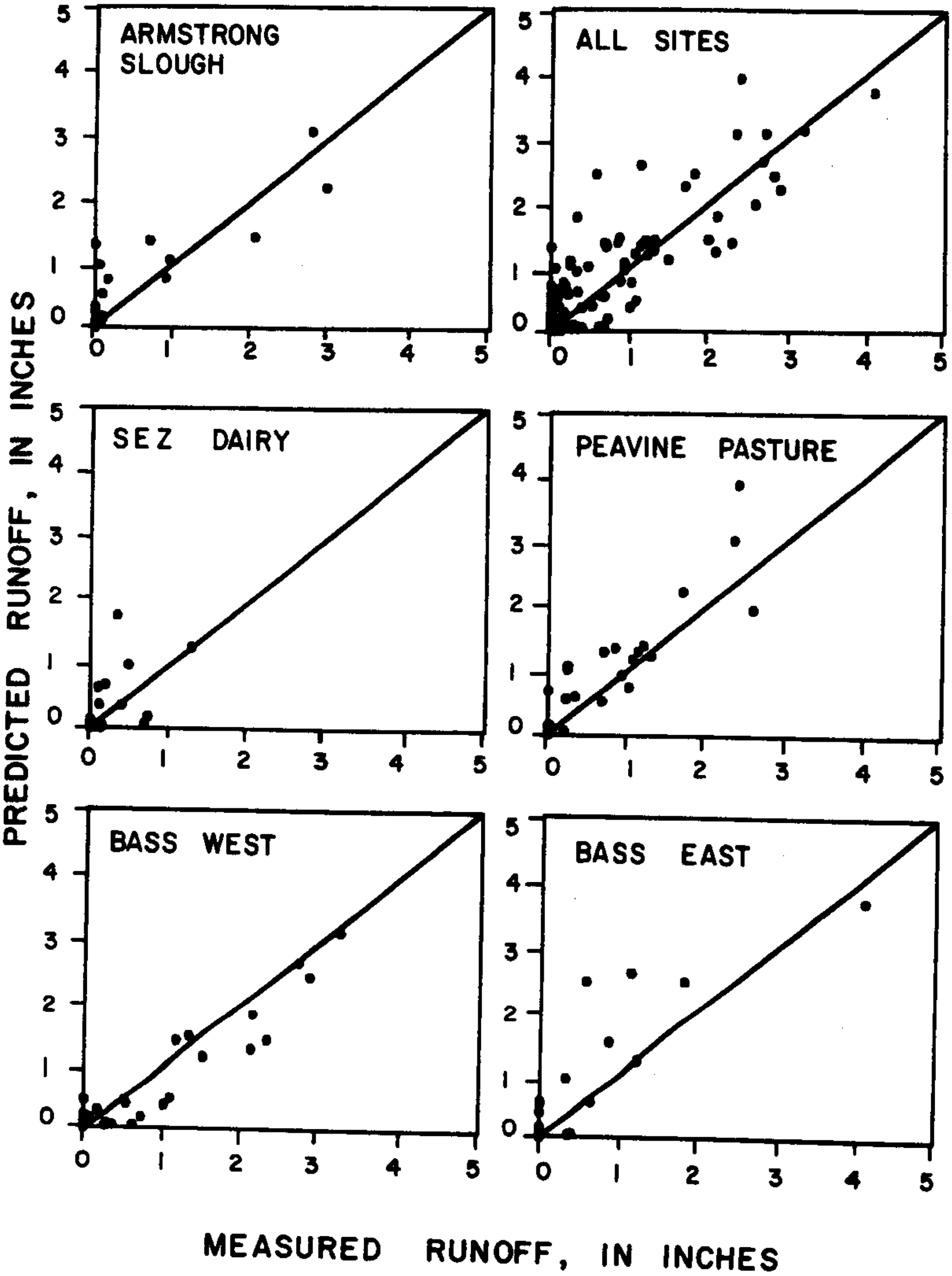


Figure 38. Comparisons of measured runoff volumes and estimates from the ARS Method.

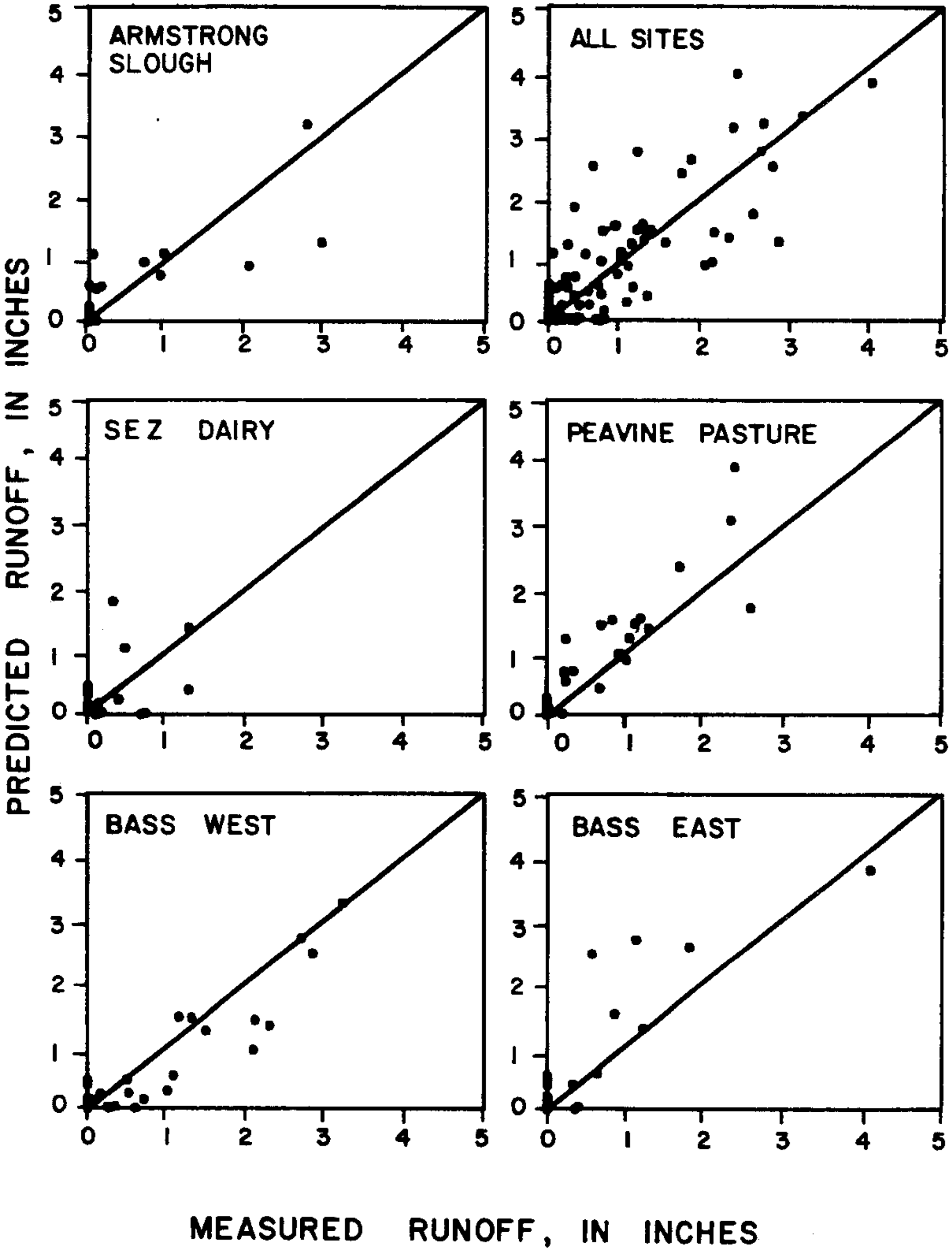


Figure 39. Comparisons of measured runoff volumes and estimates from the CR-1 Method.

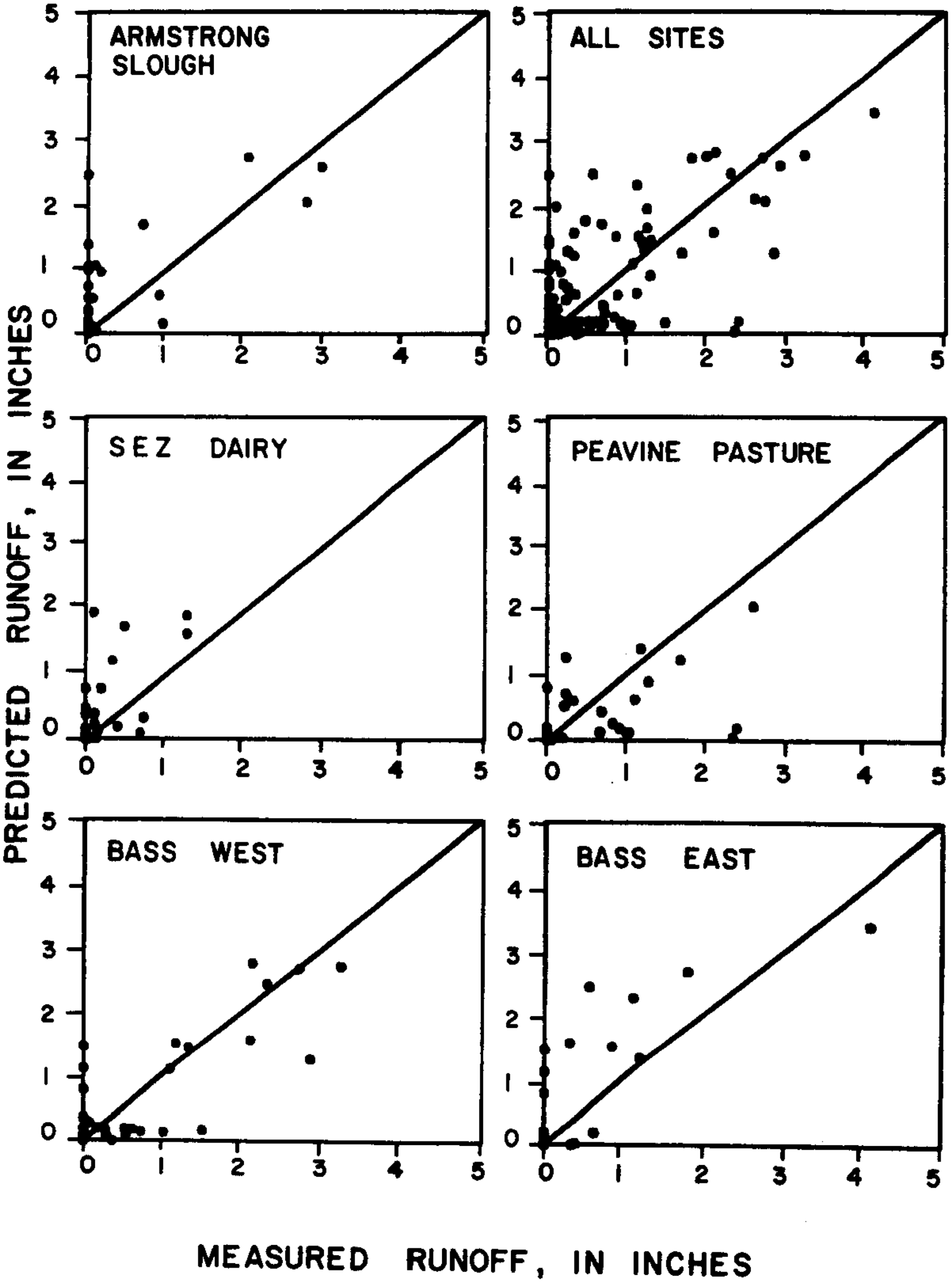


Figure 40. Comparisons of measured runoff volumes and estimates from the CR-2 Method.

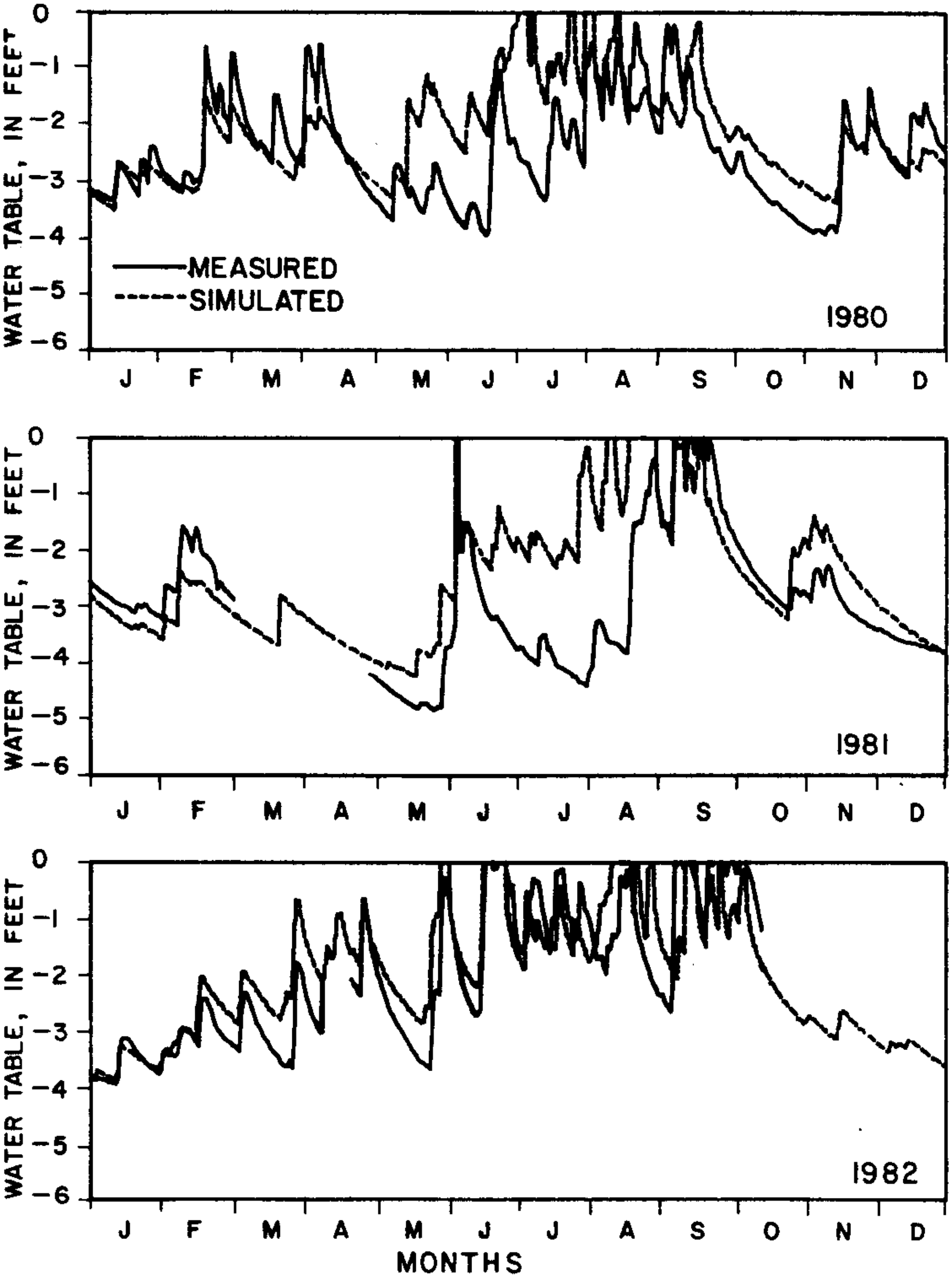


Figure 42. Simulated and measured water table levels for the Armstrong Slough observation well.

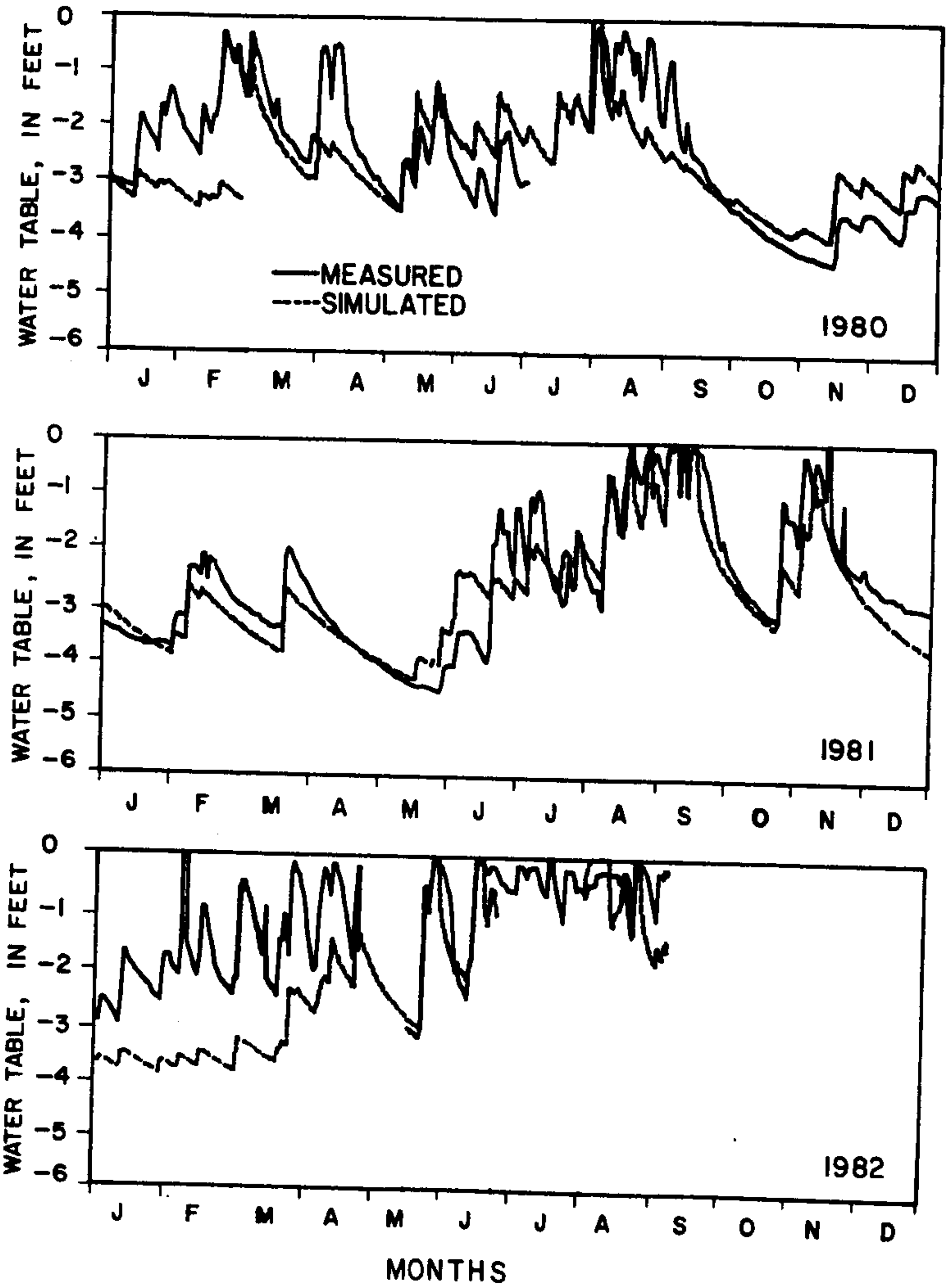


Figure 43. Simulated and measured water table levels for the Peavine Pasture observation well.

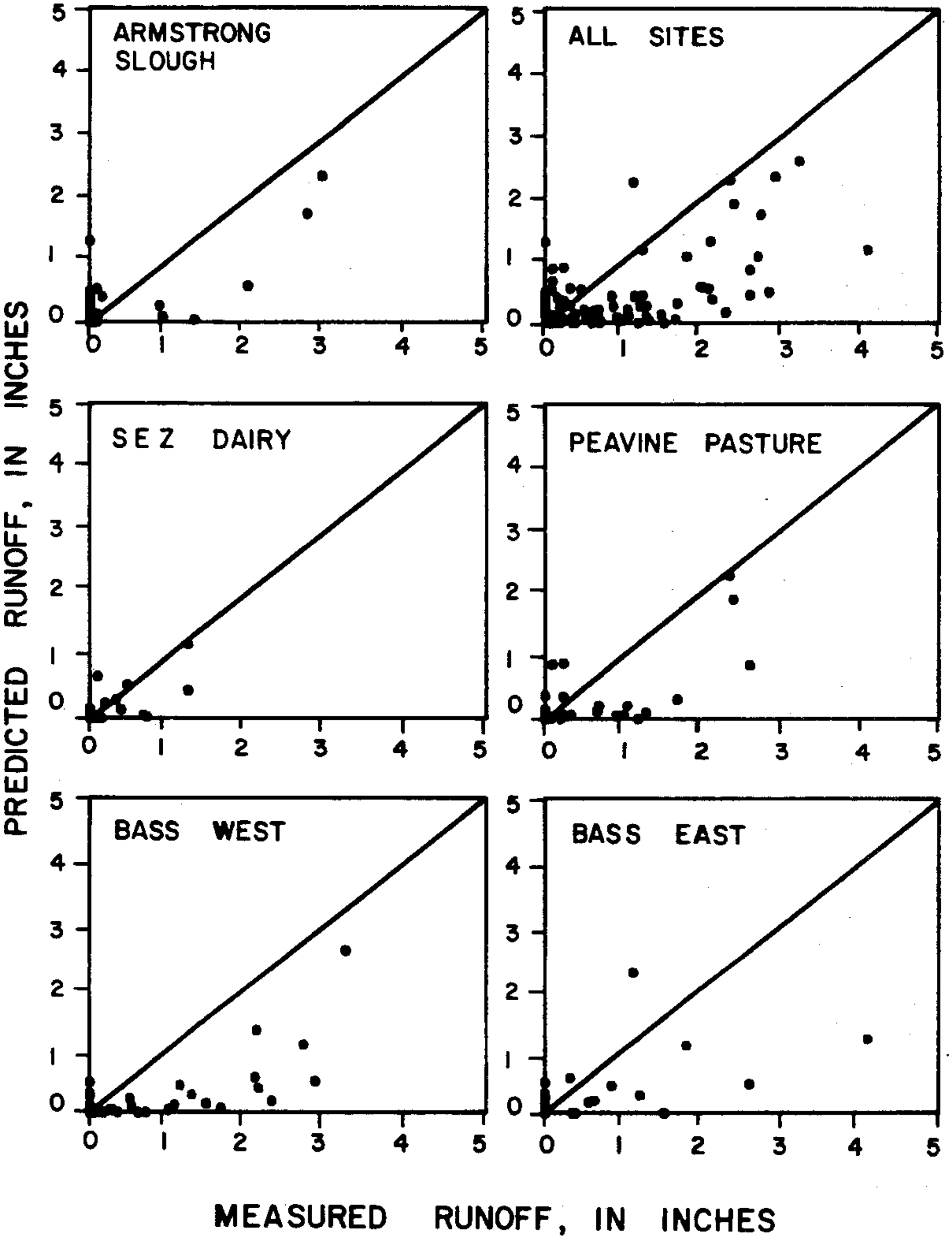


Figure 41. Comparisons of measured runoff volumes and estimates from the CR-WT Method.

Table 8. Standard errors of runoff volume estimates, in inches, for all events.

Site	Method						
	NEH-4	SCS-FL	DRM	ARS	CR-1	CR-2	CR-WT
Armstrong	0.57	0.52	0.40	0.38	0.44	0.70	0.52
Peavine	0.66	0.57	0.45	0.45	0.45	0.70	0.61
SEZ Dairy	0.31	0.57	0.52	0.40	0.46	0.61	0.30
Bass West	1.12	0.86	0.50	0.46	0.53	0.64	1.11
Bass East	0.64	0.54	0.57	0.55	0.57	0.69	0.84
All	0.73	0.63	0.47	0.44	0.48	0.66	0.74
Site Ranking	(6)	(4)	(2)	(1)	(3)	(7)	(5)

Table 9. Standard errors of runoff volume estimates, in inches, for events with measured runoff.

Site	Method						
	NEH-4	SCS-FL	DRM	ARS	CR-1	CR-2	CR-WT
Armstrong	0.86	0.53	0.61	0.47	0.66	0.60	0.71
Peavine	0.85	0.61	0.57	0.55	0.56	0.88	0.78
SEZ Dairy	0.39	0.70	0.57	0.51	0.55	0.73	0.38
Bass West	1.45	1.07	0.63	0.59	0.67	0.71	1.43
Bass East	1.09	0.80	0.92	0.89	0.90	0.97	1.32
All	1.00	0.77	0.62	0.58	0.64	0.76	1.01
Site Ranking	(7)	(4)	(3)	(1)	(2)	(6)	(5)

Table 10. Standard errors of runoff volume estimates, in inches, for events with measured runoff equal to or exceeding 0.50 inches.

Site	Method						
	NEH-4	SCS-FL	DRM	ARS	CR-1	CR-2	CR-WT
Armstrong	1.38	0.79	0.81	0.54	0.93	0.75	1.12
Peavine	1.09	0.58	0.65	0.59	0.65	1.07	1.05
SEZ Dairy	0.71	0.64	0.71	0.47	0.76	0.60	0.72
Bass West	1.85	1.37	0.78	0.72	0.83	0.88	1.62
Bass East	1.31	0.91	1.01	1.04	1.08	1.06	1.53
All	1.36	0.93	0.76	0.67	0.79	0.90	1.32
Site Ranking	(7)	(2)	(5)	(1)	(4)	(3)	(6)

Generalizations can be drawn from these rankings regarding technique overall performance and trends through changing runoff volume. The ARS method consistently performed better than all other methods. The SCS-Florida method demonstrated improved accuracy as runoff volume increased, as would be expected of a method intended for design applications. The CR-1 method performed very well on the smaller events, but not as well on the larger events. The DRM method gave results very similar to ARS and CR-1 for small events and also demonstrated decreased accuracy when applied to the larger runoff events. The CR-2 method performed poorly on the small events, but improved somewhat on larger events. Both the NEH-4 and CR-WT methods produced consistently inaccurate estimations of runoff volume.

Water Table

The total volume results demonstrate that water table levels can be used as a good indicator of stormwater runoff volume. However techniques which attempted to model the water table or soil moisture did not yield good estimates of runoff volume.

Results of the simplified water table model described in Chapter IV are shown in Figures 42 through 47. Periodic gaps in rainfall data are the cause of gaps in the water table depth simulation traces. Following such missing data, the model would be reinitialized i.e., the simulated water table level would again be set equal to the measured value. The model also tended to re-synchronize with measured levels during wet periods when the soil profile became completely saturated. Because of this, the model performed better during the wet year of 1982 than during the drought years of 1980 and 1981.

When the water table dropped to extremely low levels, the model would allocate any rainfall directly to the water table and not properly account for available storage in the dry profile above. This error tended to be seasonal, occurring during the early summer which coincides with the beginning of the rainy season. Another seasonal trend was the model's tendency to underpredict water table responses to rainfall during the winter and early spring months.

When examined site by site, differences in water table response characteristics became apparent. Since the model is driven only by rainfall, differences in error trends can be associated with variations in soil properties and drainage conditions.

Peavine's water table trends were closer to model response characteristics during the drought of 1981. However during 1982, the model deviated from Peavine measurements more so than from measurements at other sites. During the early months of 1982 and much of 1980, Peavine's water table reacted more so than the model indicated it would. This may indicate a possible error in rainfall measurement which is suspected based upon the water budget analysis.

The two SEZ observation wells demonstrated a distinct change in recession rate at a depth of 2.5 feet. This is due to a spodic horizon which

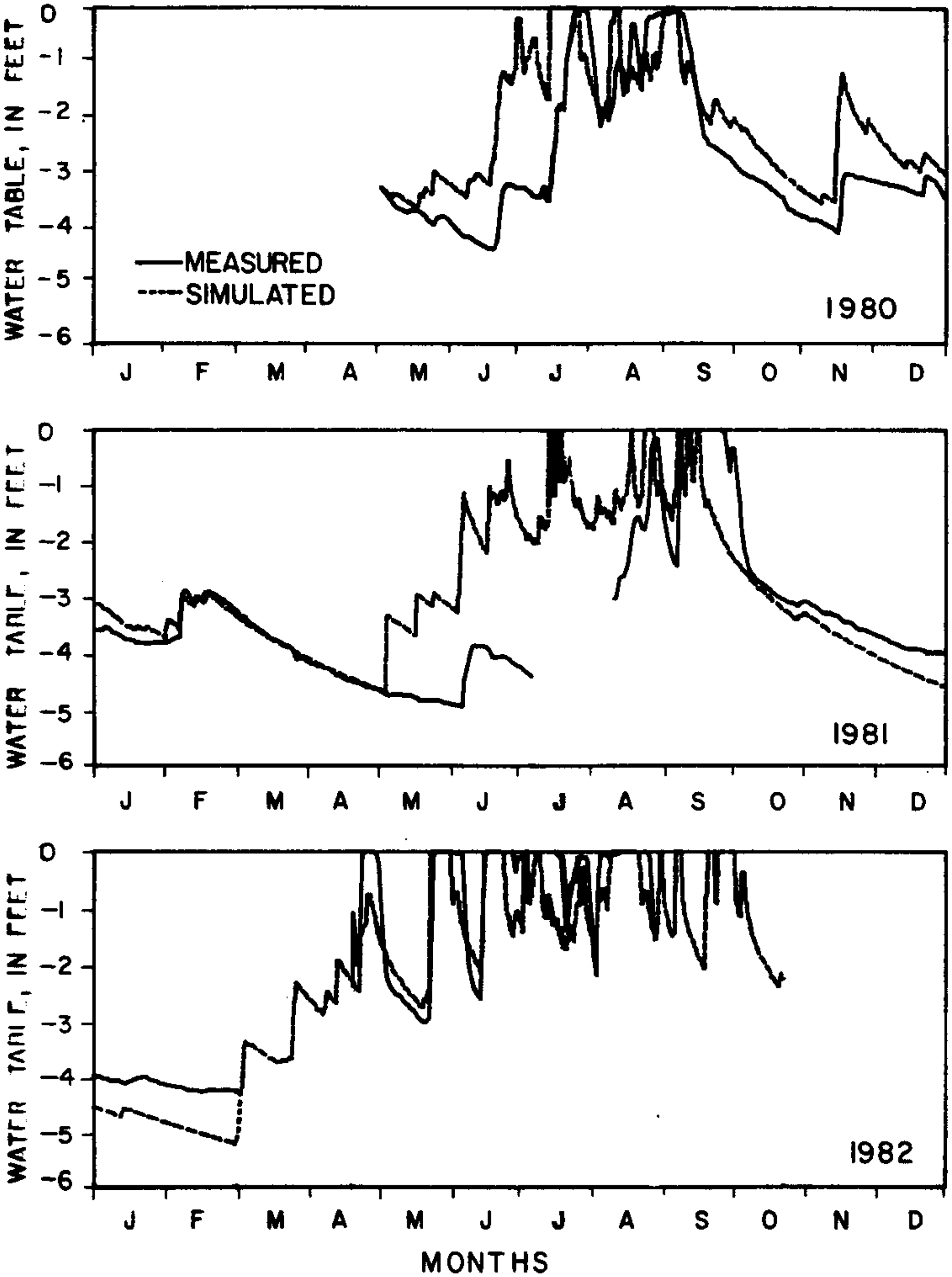


Figure 44. Simulated and measured water table levels for the SEZ Dairy east observation well.

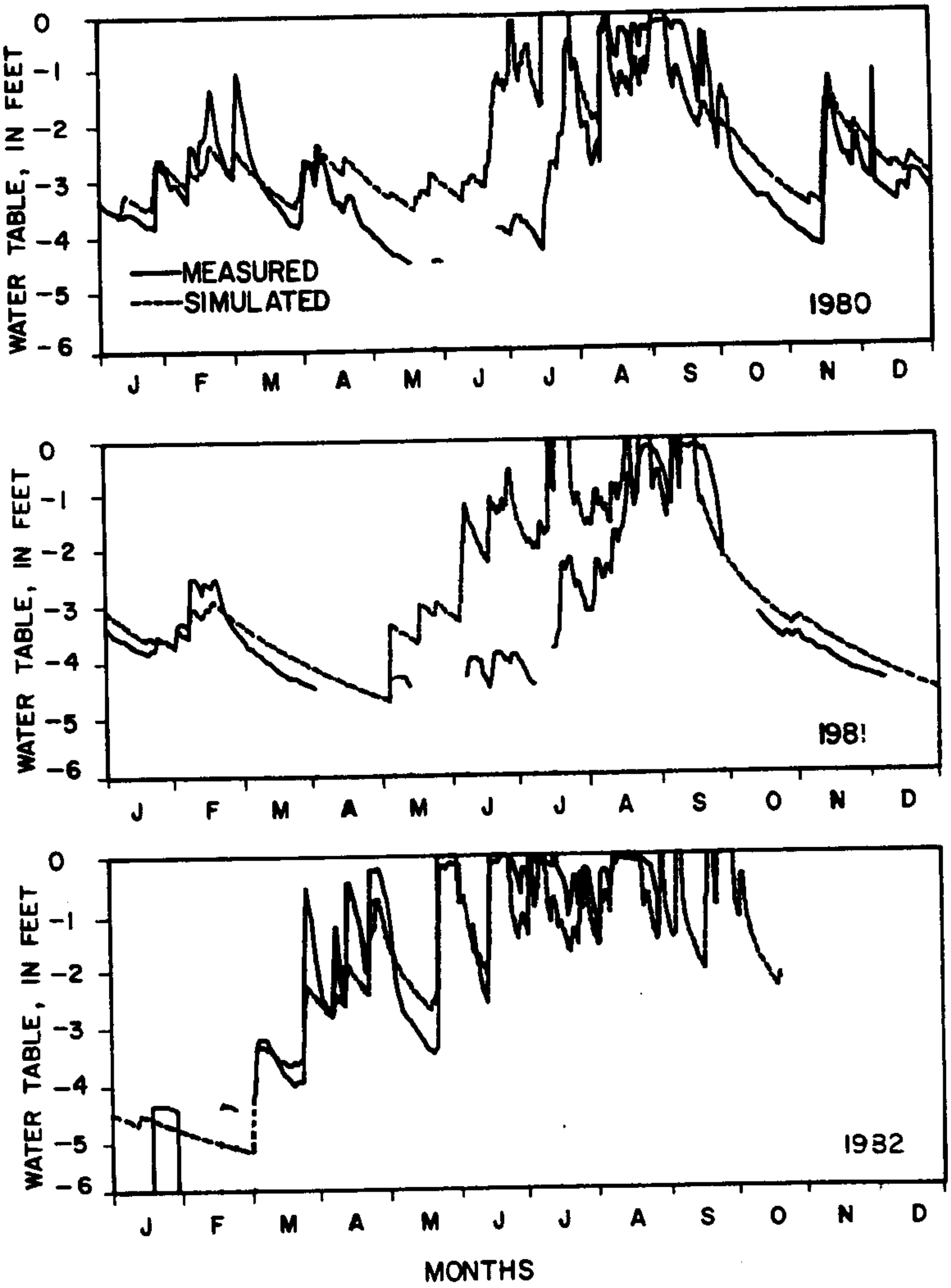


Figure 45. Simulated and measured water table levels for the SEZ Dairy west observation well.

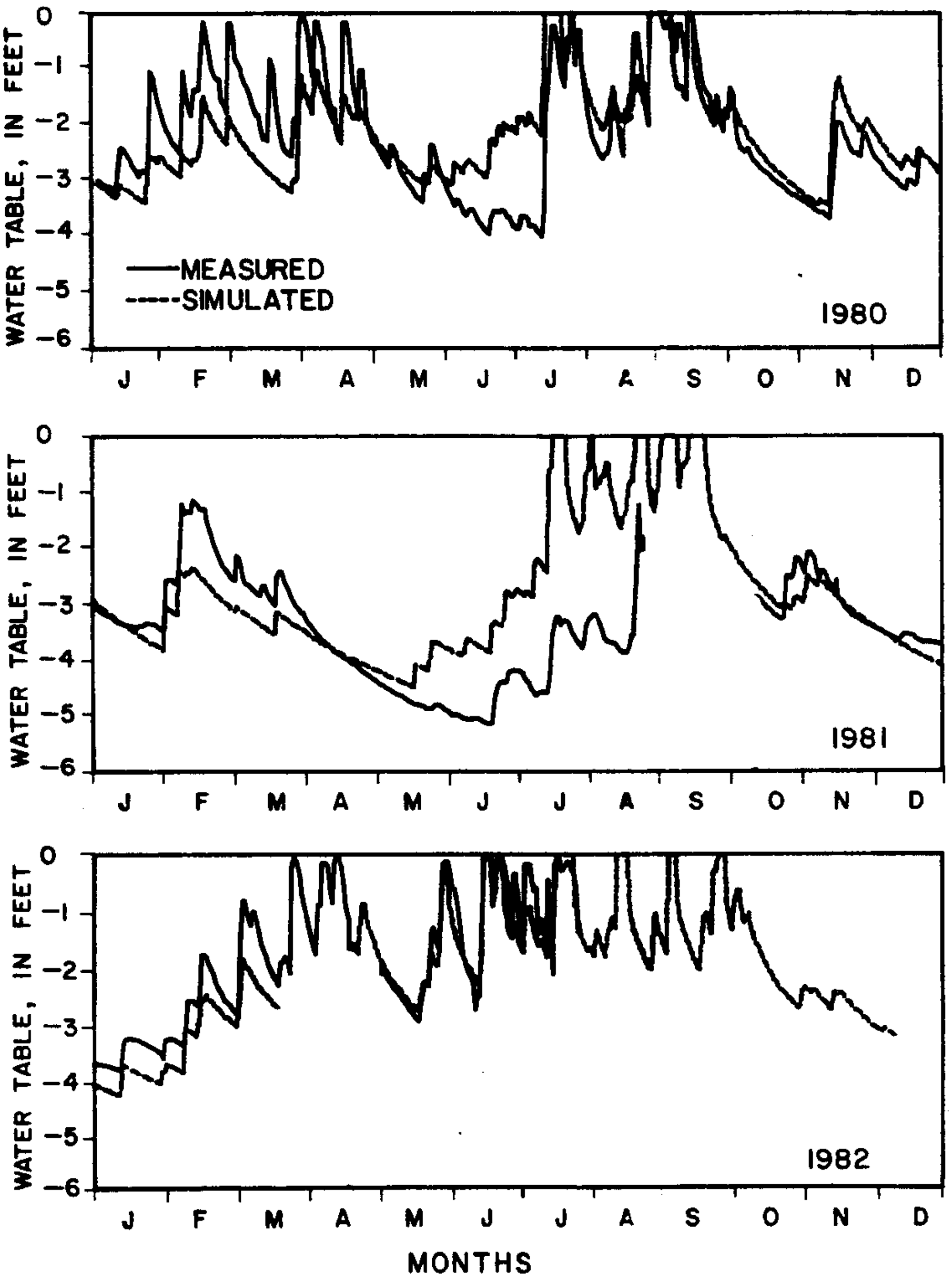


Figure 46. Simulated and measured water table levels for the Bass West observation well.

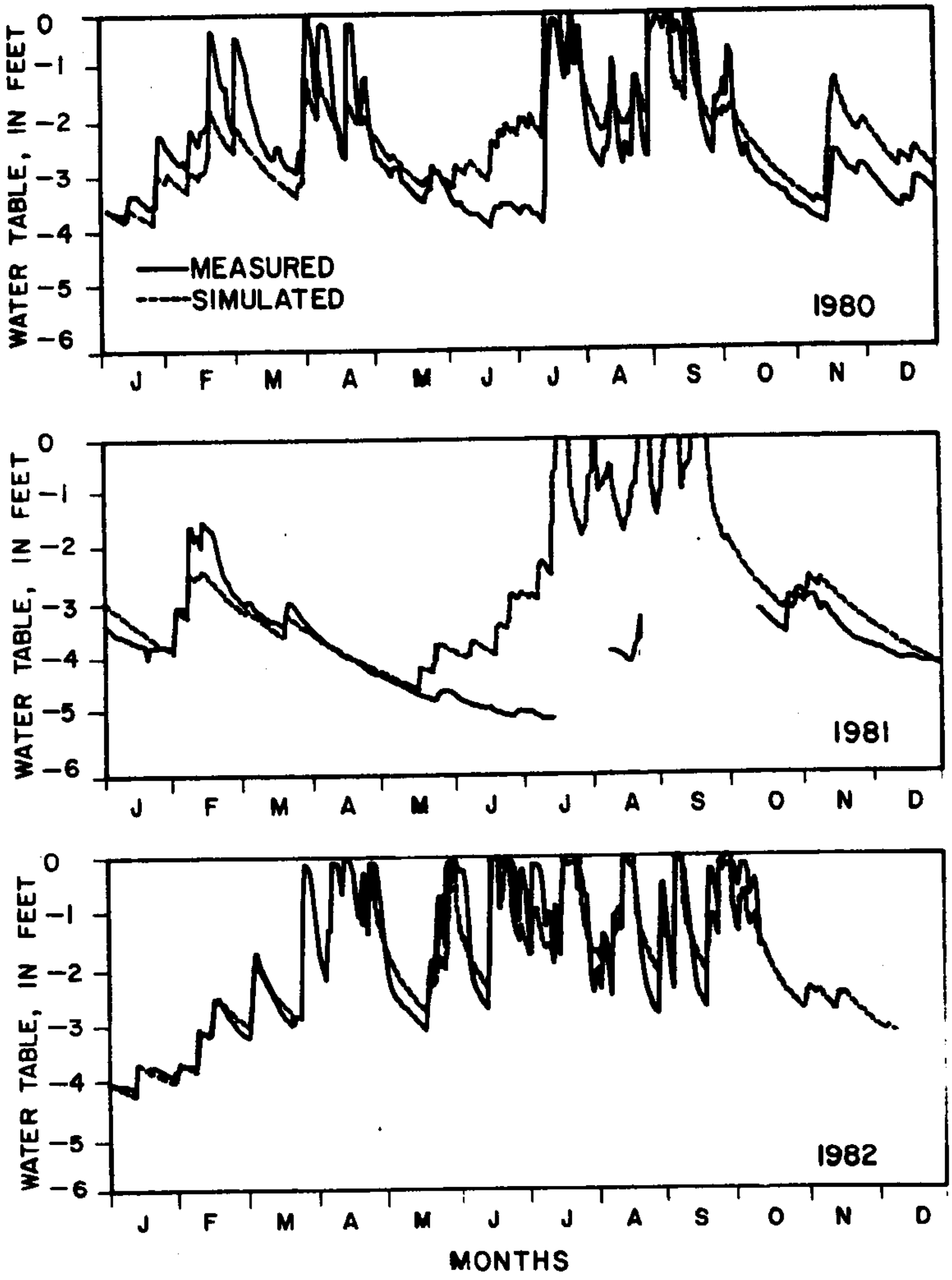


Figure 47. Simulated and measured water table levels for the Bass East observation well.

the SCS soil survey describes as being present. SEZ also tended to maintain higher water tables immediately following a storm than did the other sites, an effect also attributable to the impeding horizon.

The two Bass sites demonstrated almost identical response characteristics. Such agreement indicates that for a given soil profile type, drainage density, and rainfall conditions, the water table should be reasonably uniform over an area.

Armstrong Slough observation well demonstrated fairly well-drained soil conditions (relative to the other sites) and, overall, most agreed with model results. SCS soil survey indicates very similar soil profiles for the Armstrong and Peavine watersheds. The observed tendency for Peavine to be more sluggish in drainage may therefore be indicative of their differences in wetlands percentage.

Storm Runoff Peak Rate

The following sections present results of the peak rate estimation techniques as applied to each watershed and all runoff events. Performance is quantified as standard error of estimate, in percent, and average error of estimate, in percent:

$$\varepsilon = 100 \left[\frac{\sum_{i=1}^{n} \left(\frac{q_i' - q_i}{q_i}\right)^2}{n - 1}\right]^{0.5} \qquad [48]$$

$$\xi = 100 \left[\frac{\sum_{i=1}^{n} \left(\frac{q_i' - q_i}{q_i}\right)}{n}\right] \qquad [49]$$

where
ε = standard error of estimate in percent,
ξ = average error of estimate in percent,
q_i' = predicted peak rate for event i in cfs,
q_i = measured peak rate for event i in cfs,
n = total number of runoff events.

The average error for each site describes a method's tendency to overpredict or underpredict while the standard error quantifies error absolute magnitude. Rainfall events greater than 0.70 inches and having measurable runoff were included in the data base for this analysis, as detailed in Appendix II. Results described as applying to "all" sites are biased toward the sites having more usable runoff events.

Runoff events measuring less than 0.50 inches tended to produce erratic results. Because estimation errors are expressed as a percent of measured peak, very small events are prone to produce large errors of estimate. Another problem was that with small quantities of measured runoff, ground water discharge becomes more significant and produces atypical hydrographs. For these reasons, emphasis is placed on peak rates predicted for runoff events equal to or exceeding 0.50 inches.

In most of the following evaluations, no differentiation is made between the two drainage area conditions of Peavine Pasture. Most results corresponding to Peavine represent both runoff conditions; however, in some cases, distinction is made between the two conditions.

Cypress Creek Formula

Predictions from the Cypress Creek Formula are compared against measured peaks in Table 11. Summaries are tabulated for all runoff events and the subset exceeding 0.50 inches. As previously described, prediction errors associated with small events are extremely large.

The standard and average errors are comparable in magnitude and average errors are all positive. Thus, the method consistently resulted in large overpredictions of measured peak discharge. Standard errors ranged from 200% for Armstrong (the largest watershed) to 1000% for Bass East (the smallest watershed). Even when the effect of transforming a 24-hour maximum rate into an instantaneous rate was removed (failing to apply equation 23), the method still overpredicted.

CREAMS Equation

The standard CREAMS equation (equation 24) performed worse than any other method examined in this study. It consistently overpredicted by an order of magnitude or more (see Table 12). The results when examined graphically (see Figure 48) indicated that the estimation error was fairly consistent for all sites.

A regression of the CREAMS model formulation against measured data yielded a modified version of equation 24:

$$q_p = 4.52(DA^{1.06})(CS^{0.77})(LW^{0.389})(Q^{0.87(DA^{-0.20})}) \qquad [50]$$

where q_p = peak runoff rate in cfs,
DA = drainage area in mi^2,
CS = main channel slope in ft/mi,
LW = watershed length to width ratio, and
Q = daily runoff volume in inches.

Table 11. Percent error of peak discharge estimates from the Cypress Creek Formula.

Site	Events with Runoff		Events with Runoff >0.50 In.	
	Std. Error	Avg. Error	Std. Error	Avg. Error
Armstrong	2062.	1279.	256.	229.
Peavine	3936.	2133.	946.	877.
SEZ Dairy	6067.	3652.	656.	548.
Bass West	654.	509.	363.	324.
Bass East	1159.	1000.	1050.	905.
All	3279.	1665.	715.	600.

Table 12. Percent error of peak discharge estimates from the CREAMS equation for events with measured runoff equal to or exceeding 0.50 inches.

Site	Original Equation		Revised Equation	
	Std. Error	Avg. Error	Std. Error	Avg. Error
Armstrong	1002.	887.	21.0	-1.3
Peavine	2770.	2568.	33.0	10.6
SEZ Dairy	1764.	1443.	34.4	-5.3
Bass West	2069.	1975.	20.8	-7.4
Bass East	7166.	5943.	85.5	-1.8
All	3511.	2692.	42.2	-0.2

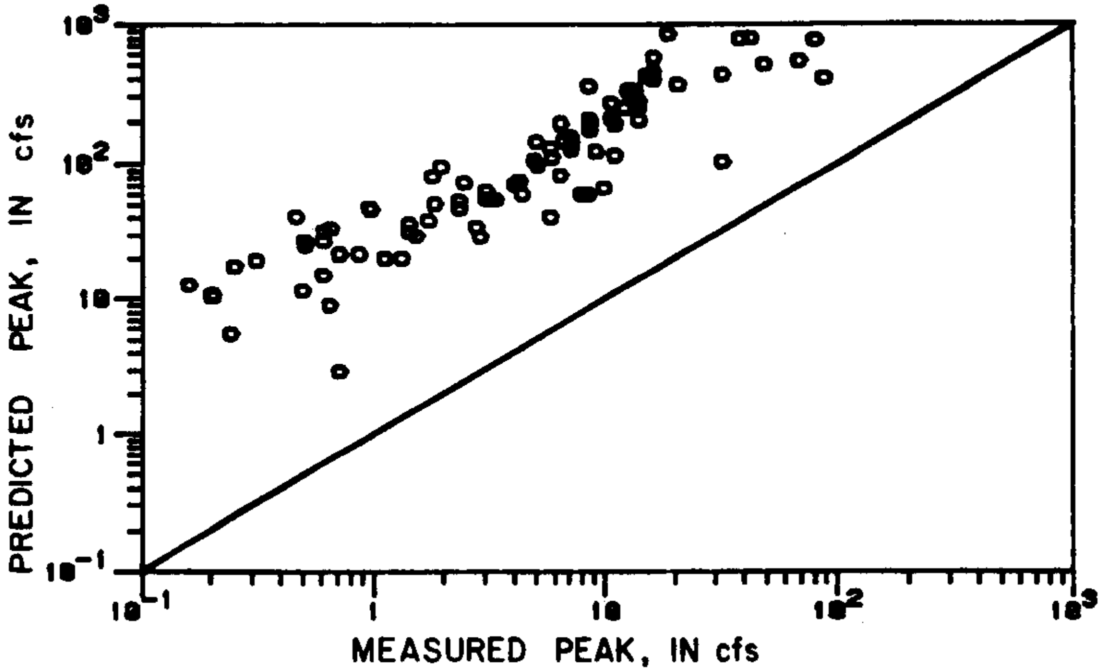


Figure 48. Comparison of measured peak discharge rates to estimates from the original CREAMS equation.

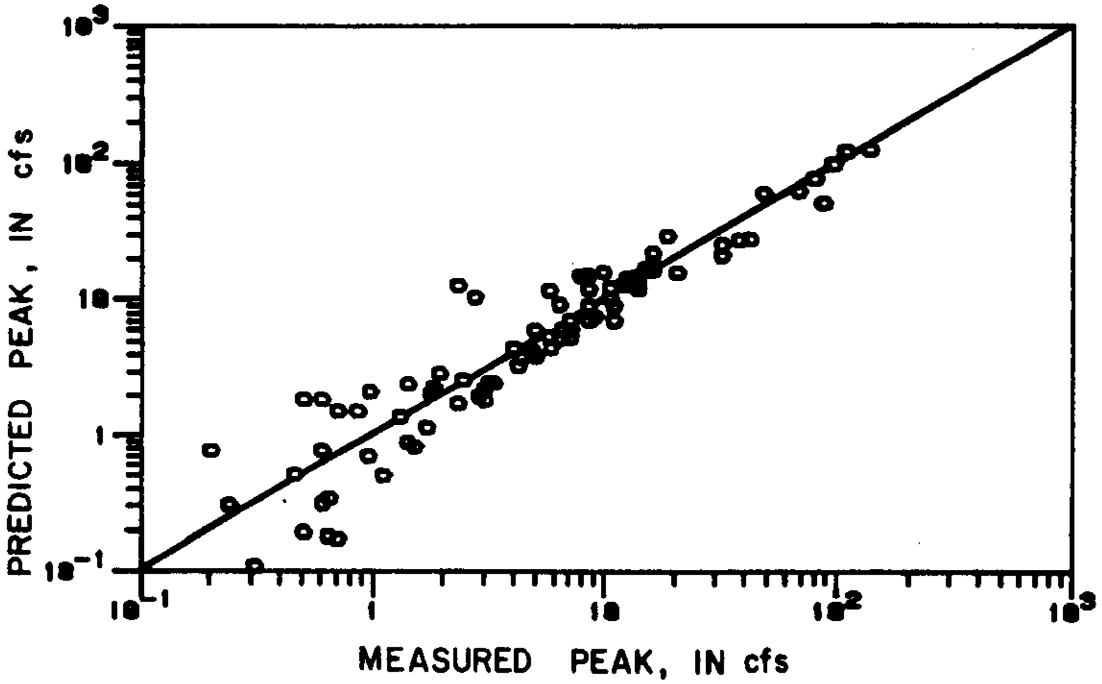


Figure 49. Comparison of measured peak discharge rates to estimates from the modified CREAMS equation.

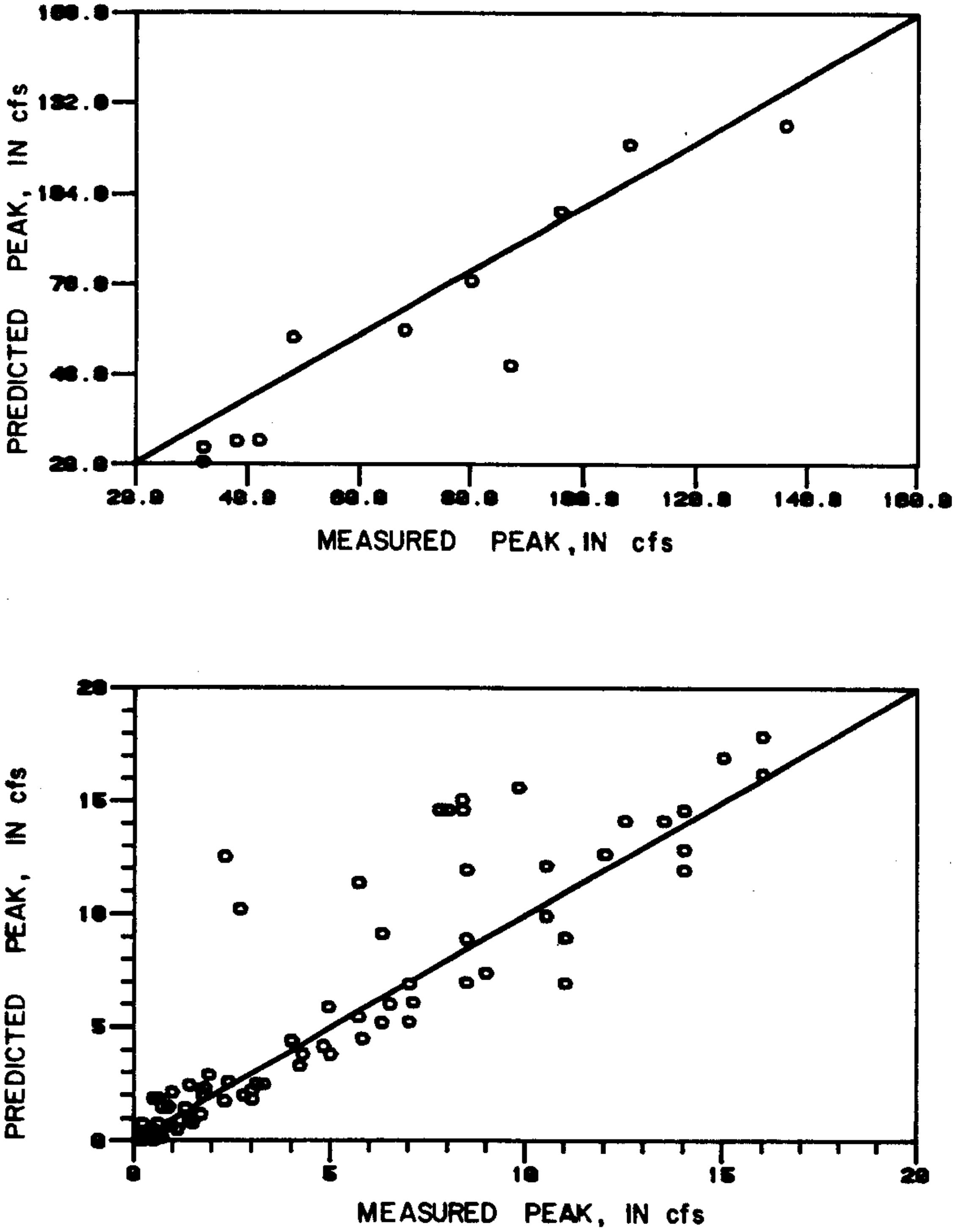


Figure 50. Linear scale comparisons of measured peak discharge rates to estimates from the modified CREAMS equation.

When reapplied to the data base, performance of this equation was good. Table 12 and Figures 49 and 50 do not represent independent evaluations of the modified CREAMS eqaution, but simply measure the regression fit to the data. This equation resulted in an R^2 of 0.96. The non-linear regression was also conducted fitting only the multiplicative factor for the equation while maintaining the original exponent values. The original factor of 200 was replaced with 14.3. The R^2 measured 0.67 indicating that, although the original multiplicative factor accounts for most of the data variability, modification of the other model parameters was also significant in accounting for much of the data variance.

The standard error of estimate associated with equation 48 ranged from 20% for Bass West to 85% for Bass East.

SCS Graphical Method

Figure 51 presents the standard design peak estimation curve developed specifically for Florida by the SCS. Also shown are the patterns produced by plotting measured peak discharges, in csm (cfs/inch of runoff/square mile) against hydrograph analysis estimates for time of concentration for runoff events exceeding 0.50 inches. Most observed hydrograph time of concentrations were out of the SCS curve range. Where they were in range, significant overpredictions were apparent. Plots of the data exhibit considerable scatter and do not show correlation between measured peaks and time of concentration. The upper limits of a pattern (highest observed csm value) did not always coincide with the largest observed runoff peak for a site.

Figure 52 shows an average csm value for each pattern plotted against the SCS estimates of time of concentration from the lag method equation 45. This attempt showed limited improvement in generating a trend in the graph, but also resulted in more severe overestimates of peak discharge. The same csm values were also plotted against modified estimates of T_c from equation 45. This plot produced a definite trend as shown by the subjectively fitted curve. Apparently, employing the modified time of concentration estimation method forced the observed peak data into a general order. The trend is parallel to the SCS curve but is translated downward. The revised curve may be useful for predicting peaks from typical runoff events, however for design applications, the upper extremes of scatter should be noted.

SCS Chart Method

Performance comparisons for the SCS Chart Method are presented in Table 13. Unlike the Cypress Creek Formula, large events produced results similar to those for all events. This method also tended to overpredict peak discharge. Maximum and minimum standard errors of estimate were 1000% and 50%, corresponding to Bass East and Armstrong Slough, respectively.

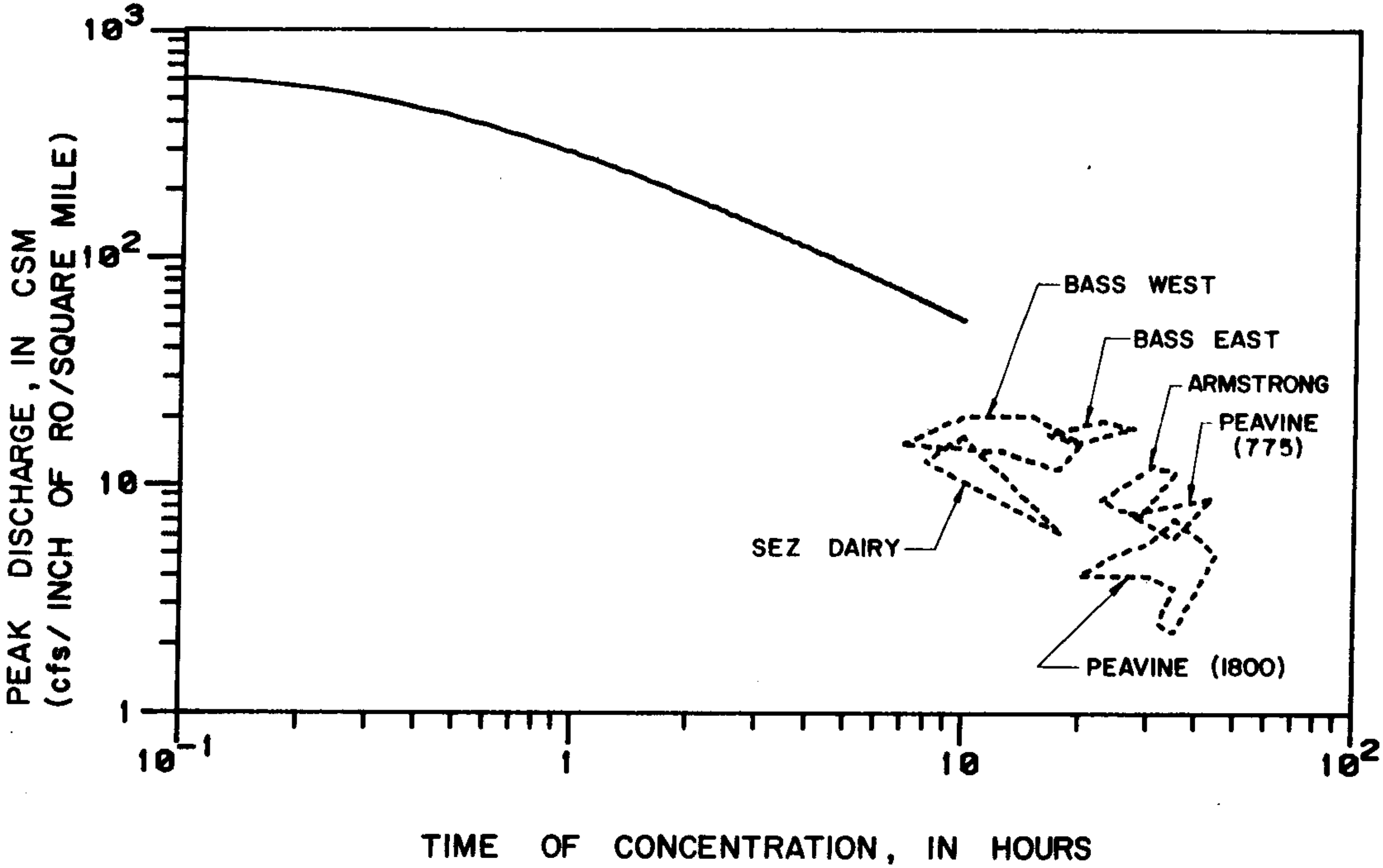


Figure 51. Comparison of observed discharge hydrograph time of concentration and peak rate to the SCS Graphical Method design curve. The polygons shown represent the regions generated by the runoff events for each site measuring 0.5 inches or more and for which time of concentration estimates could be made.

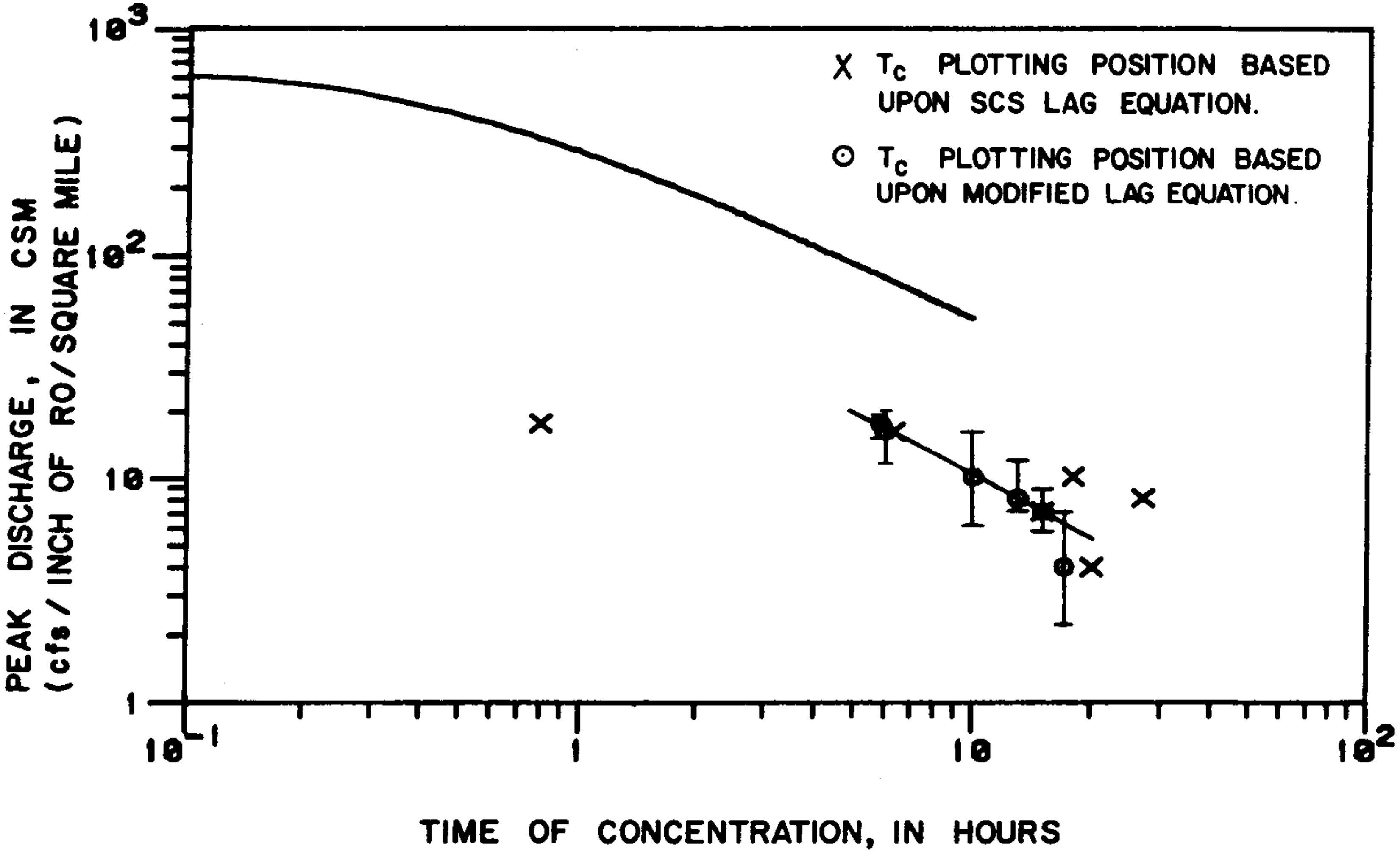


Figure 52. Comparison of the SCS Graphical Method design curve to plottings of the average of peaks for each site, as shown in Figure 51 against estimates for time of concentration. These estimates are based on equation 26 and watershed lags calculated by the SCS standard method (equation 27) and by the modified method (equation 45).

Table 13. Percent error of peak discharge estimates from the SCS Chart method.

Site	Events with Runoff		Events with Runoff >0.50 In.	
	Std. Error	Avg. Error	Std. Error	Avg. Error
Armstrong	49.	7.7	46.	22.
Peavine	142.	129.	142.	102.
SEZ Dairy	204.	146.	117.	80.
Bass West	186.	172.	201.	189.
Bass East	1021.	857.	1075.	887.
All	393.	221.	461.	261.

Table 14. Results summary for SCS unit hydrograph method using SCS-fixed lag estimates (equations 27 and 14). Errors are reported in percent.

Site	K' = 300		K' = 484	
	Std. Error	Avg. Error	Std. Error	Avg. Error
Armstrong	106.	88.	230.	200.
Peavine	430.	380.	724.	646.
SEZ Dairy	192.	142.	352.	272.
Bass West	188.	175.	337.	316.
Bass East	44.	28.	112.	94.
All	254.	195.	439.	354.

SCS Unit Hydrograph Method

Evaluation of the unit hydrograph method was conducted in three steps: (1), evaluation of standard SCS methodology; (2), evaluation and modification of certain aspects of the method; and (3), re-evaluation of the unit hydrograph method implementing various modifications.

Tables 14 and 15 present results from application of the recommended K' factors (300 and 484) and two methods of calculating time to peak (fixed and variable lag estimates from equations 27 and 35). SFWMD assumed rainfall distributions were used for all events. Best peak estimates were obtained using a K' of 300 and the fixed lag estimates for each watershed. However this method still tended to overpredict discharge peaks by about 200%.

The second step of the evaluation was to determine best-fit K' factors using various estimates of time to peak, both measured and assumed rainfall time-distributions, and different classes of runoff events. Table 16 is a summary of the K' optimization analysis. The $\bar{K}$ values reported represent an average of the best-fit K' values for each site. The best-fit K' for a given site is itself an average taken over all events within the site and runoff event class. The s (standard deviation) quantifies the variability of the factor over all sites. It is the standard deviation associated with the calculation of $\bar{K}$.

Generalizations can be drawn from the information presented in Table 16. The most definite conclusion is the insignificance of the rainfall time-distribution employed in the calculation of composite hydrographs. The SFWMD assumed rainfall distribution yielded almost identical results to those derived from measured distributions. The user should realize, however, that when working with more "responsive" watersheds, i.e. developed flatwoods watersheds or natural watersheds with K values greater than 300, time distribution of rainfall becomes a significant factor in peak runoff rates.

Differences in best-fit K' values were apparent when comparing the two runoff event classes. Factors for events less than 0.50 inches were 20-30% higher than factors for the larger class of events. The scatter among K' values for each site is also greater for smaller events. Focusing in on the K' solved using assumed rainfall distributions and events greater than 0.50 inches, all were computed to be less than 100. Even when using the SCS fixed lag estimates, results were significantly lower than the 300 value recommended for flat, swampy areas. Values were also lower than the 256 found to be appropriate for the Delmarva Peninsula (Welle et al., 1980). The Delmarva area, although having similar soil types, still has slopes (2%) which are steeper than the flatwoods watersheds examined in this study.

Having isolated a range of factors between 75 and 100, the next step was to isolate a technique for estimating lag which would result in a fairly consistent K' for all sites. For both the modified and standard lag estimation techniques, no benefit (in terms of factor uniformity) was de-

Table 15. Results summary for SCS unit hydrograph method using SCS-variable lag estimates (equations 27 and 43). Errors are reported in percent.

Site	K' = 300		K' = 484	
	Std. Error	Avg. Error	Std. Error	Avg. Error
Armstrong	201.	142.	367.	281.
Peavine	602.	554.	985.	907.
SEZ Dairy	257.	139.	444.	272.
Bass West	288.	264.	479.	442.
Bass East	1125.	982.	1694.	1454.
All	574.	447.	898.	717.

Table 16. Results summary for SCS unit hydrograph K' factor optimization. $\bar{R}$ values represent an average for all sites.

Lag Method	Assumed Distributions				Measured Distributions			
	RO > 0.50"		RO < 0.50"		RO > 0.50"		RO < 0.50"	
	$\bar{R}$	s	$\bar{R}$	s	$\bar{R}$	s	$\bar{R}$	s
MOD Fixed[a]	72.	13.	85.	29.	71.	14.	84.	28.
MES Fixed[b]	95.	57.	122.	64.	98.	59.	118.	57.
SCS Fixed[c]	87.	53.	107.	80.	89.	52.	110.	77.
MOD Variable[d]	85.	30.	122.	42.	82.	31.	133.	34.
SCS Variable[e]	83.	62.	112.	94.	79.	61.	124.	86.

[a] as calculated from equation 45.

[b] minimum observed times to peak from hydrograph analysis.

[c] as calculated from equations 27 and 14.

[d] as calculated from equations 46 and 43.

[e] as calculated from equations 27 and 43.

Table 17. Results summary for SCS unit hydrograph K' factor optimization using SFWMD assumed rainfall distribution, runoff events equal to or exceeding 0.50 inches, and modified-fixed lag estimates. $\bar{K}$ values represent an average of all events for a site.

Site	n Events	Maximum[a]	Minimum[b]	$\bar{K}$	s	David[c]
Armstrong	5	119.	62.	83.	22.	88.
Peavine (1800)	10	74.	27.	50.	15.	66.
Peavine (775)	3	76.	53.	66.	12.	-
SEZ Dairy	4	101.	39.	70.	26.	-
Bass West	16	107.	59.	88.	12.	118.
Bass East	7	84.	66.	72.	8.	70.
Average				72.	13.	

[a] Maximum observed optimized event K' factor.

[b] Minimum observed optimized event K' factor.

[c] Based on available data for rainfall associated with Hurricane David (9-3-79).

Table 18. Results summary for SCS unit hydrograph K' factor optimization using SFWMD assumed rainfall distribution, runoff events equal to or exceeding 0.50 inches, and SCS-fixed lag estimation equation. $\bar{K}$ values represent an average of all events for a site.

Site	n Events	Maximum[a]	Minimum[b]	$\bar{K}$	s	David[c]
Armstrong	5	228.	121.	161.	41.	170.
Peavine (1800)	10	90.	31.	59.	19.	79.
Peavine (775)	3	74.	53.	65.	11.	-
SEZ Dairy	4	185.	72.	129.	46.	-
Bass West	16	113.	62.	93.	13.	187.
Bass East	7	14.	12.	14.	1.3	14.
Average				87.	53.	

[a] Maximum observed optimized event K' factor.

[b] Minimum observed optimized event K' factor.

[c] Based on available data for rainfall associated with Hurricane David (9-3-79).

rived from the use of a variable lag equation. For the fixed lag estimates, the modified method (equation 45) produced the most consistent optimized K' for all sites.

Tables 17 and 18 show the site variability of best-fit factors for the SCS and modified fixed lag cases. Here the average K' is calculated over n events for each site and s is the standard deviation associated with that average. The trend among sites was for an increasing K' value with decreasing watershed percent wetlands. Results reported for Peavine Pasture differentiate between the large and small drainage area conditions. The smaller factor associated with the 1800 acre watershed may be due to channel block effects.

Also included in Tables 17 and 18 are the best-fit factors for the large runoff events associated with Hurricane David. Runoff from this event ranged between 2.5 and 5.2 inches, depending upon the specific site. Armstrong data for this period are documented as being "estimated." Questions also exist regarding the actual contributing area for this event as well as other events at other sites. Examination of the SCS unit hydrograph equation shows that when measured runoff data are used, the influence of errors in drainage area estimates upon peak rate calculations is confined to the T_p term. The multiplication of the depth and area terms in the numerator of equation 34 yields volume. This is the inverse of the calculation used to estimate runoff depth (measured volume divided by estimated contributing area) and thus negates the influence of drainage area estimates.

Table 19 presents the optimized K' results for events of short duration and high intensity. For a given lag method, K' factors tended to be consistent among the different groups of rainfall events ie., all large events, large, short-duration events, and very large events (Hurricane David).

Incorporating results from the K' analysis, the incremental unit hydrograph method was re-applied to the data base. Like the initial evaluation, only SFWMD assumed rainfall distributions and runoff events greater than 0.50 inches were examined. Tables 20 through 23 present results using revised K' factors. Two combinations of discrete factors were used: 75 and 100 for the fixed lag estimates and 85 and 100 for the variable lag estimates (which had higher optimized lag values). These two sets of factors were tested using various lag estimation techniques. Best results were achieved with the modified lag equation and a K' of 75.

SFWMD

The SFWMD overland flow computer model (as modified) was applied to runoff events exceeding 0.50 inches. Results are summarized in Table 24. The model underpredicted on most sites. However, where it did overpredict, the percent error was high (Bass East). Best results were associated with the Bass West and Peavine watersheds. The large Peavine Pasture watershed is believed to respond in a sheetflow manner and should be described well

Table 19. Results summary for SCS unit hydrograph K' factor optimization using runoff events equal to or exceeding 0.50 inches with short durations and measured rainfall distributions. K values represent a site average.

Site	MOD-Fixed	MES-Fixed	SCS-Fixed	MOD-Var.	SCS-Var.
Armstrong	89.	79.	172.	105.	75.
Peavine (1800)	74.	74.	90.	45.	50.
Peavine (775)	72.	80.	71.	46.	68.
SEZ Dairy	64.	56.	121.	176.	118.
Bass West	84.	133.	88.	75.	120.
Bass East	77.	223.	14.	12.	82.
Average	77.	108.	93.	77.	86.
Std. Dev.	9.	62.	53.	58.	28.

Table 20. Results summary for SCS unit hydrograph method using SCS-fixed lag estimates (equations 27 and 14). Errors are reported in percent.

Site	K' = 75		K' = 100	
	Std. Error	Avg. Error	Std. Error	Avg. Error
Armstrong	57	-50.	41.	-34.
Peavine	55.	32.	96.	73.
SEZ Dairy	46.	-33.	37.	-12.
Bass West	21.	-16.	18.	7.
Bass East	69.	-64.	58.	-53.
All	46.	-16.	58.	9.

Table 21. Results summary for SCS unit hydrograph method using SCS-variable lag estimates (equations 27 and 43). Errors are reported in percent.

Site	K' = 85		K' = 100	
	Std. Error	Avg. Error	Std. Error	Avg. Error
Armstrong	49.	-25.	48.	-13.
Peavine	126.	110.	162.	144.
SEZ Dairy	71.	-26.	76.	-14.
Bass West	40.	26.	57.	44.
Bass East	387.	337.	448.	391.
All	169.	94.	200.	122.

Table 22. Results summary for SCS unit hydrograph method using modified-fixed lag estimates (equations 45 and 43). Errors are reported in percent.

Site	K' = 75		K' = 100	
	Std. Error	Avg. Error	Std. Error	Avg. Error
Armstrong	21.	-5.	38.	24.
Peavine	77.	51.	126.	98.
SEZ Dairy	51.	19.	87.	55.
Bass West	18.	-12.	21.	12.
Bass East	26.	-4.	40.	22.
All	45.	11.	73.	43.

Table 23. Results summary for SCS unit hydrograph method using modified-variable lag estimates (equation 46 and 43). Errors are reported in percent.

Site	K' = 85		K' = 100	
	Std. Error	Avg. Error	Std. Error	Avg. Error
Armstrong	43.	7.	56.	24.
Peavine	118.	98.	152.	130.
SEZ Dairy	69.	-3.	81.	12.
Bass West	263.	247.	319.	300.
Bass East	33.	-2.	40.	13.
All	166.	114.	203.	147.

Table 24. Results summary from SFWMD overland flow computer model as applied to runoff events with measured runoff equal to or exceeding 0.50 inches. Errors are reported in percent.

Site	Standard Error	Average Error
Armstrong	67.	-60.
Peavine (1800)	59.	10.
Peavine (775)	50.	-35.
SEZ Dairy	64.	-46.
Bass West	29.	-8.
Bass East	479.	410.
All	181.	48.

by an overland flow model. For the large watersheds with significant channel effects (Armstrong Slough and SEZ Dairy), the model underpredicted as would be expected. However for the smallest watershed (Bass East), where the overland flow approximation would appear most applicable, results were not good. The observed overprediction is probably due to the length-to-width ratio of the pasture. It is wide, about 1700 feet, and only 500 feet long. This shape would simulate as a very high peak-producing watershed.

Summary

When compared with one another (see Table 25), the methods demonstrate magnitudes of error inversely proportional to their degree of complexity. With decreasing overall standard error of estimate, the original methods line up as: CREAMS, 3500%; Cypress Creek Formula, 700%; SCS Chart, 400%, SCS unit hydrograph, 250%; SFWMD, 180%. The CREAMS and Cypress Creek Formula should be reversed based upon complexity level, however the CREAMS equation was not developed using Florida data, while the Cypress Creek Formula is described as being applicable to the Florida flatwoods.

Modifications to the CREAMS equation and the SCS unit hydrograph approach significantly improved the performance of both methods. Each achieved between 40% and 45% standard error of estimate.

Table 25. Results summary for peak rate estimation techniques as applied to events with measured runoff equal to or exceeding 0.50 inches. Results are reported as standard error of estimate, in percent.

Site	Peak Rate Estimation Technique						
	Cypress	CREAMS	SCS-Chart	SCS-UH	SFWMD	CR-Mod	UH-Mod
Armstrong	256.	1002.	46.	106.	67.	21.	21.
Peavine	946.	2770.	142.	430.	55.	33.	77.
SEZ Dairy	656.	1764.	117.	192.	64.	34.	51.
Bass West	363.	2069.	201.	188.	29.	21.	18.
Bass East	1050.	7166.	1075.	44.	479.	86.	26.
All	715.	3511.	461.	254.	181.	42.	45.

CHAPTER VI

SUMMARY AND DISCUSSION

Study Overview

Objectives of this study were to evaluate the performance of stormwater runoff volume and peak rate estimation techniques as applied to Florida's flatwoods watersheds. Characteristics of these watersheds include extremely flat relief, sandy soils, highly dynamic water tables, and scattered wetlands.

The U.S. Geological Survey and South Florida Water Management District collected rainfall, water table, and runoff data from five agricultural (improved and unimproved pasture) watersheds in the Lower Kissimmee River and Taylor Creek/Nubbin Slough Basins. Drainage areas for the watersheds ranged from 20 to 3600 acres. The data collection process began in late 1979, continued through early 1983, and included two severe drought years, 1980 and 1981.

Data interpretation and analysis produced an event data base which included all 24-hour rainfall events equal to or exceeding 0.70 inches and having reliable concurrent runoff and water table data. The total number of events meeting these criteria were about 160 events, ranging from 15 to 30 events per site.

Seven methods of estimating stormwater runoff volume, all relying upon the SCS runoff equation, were applied to the event data base and their performances evaluated. Building upon research conducted by the USDA-ARS, a simplified water table model was developed in an effort to simulate observed water table fluctuations useful in estimating runoff volume.

Six distinct methods of estimating stormwater peak discharge rates were applied to the subset of the data base having measured runoff (approximately 80 events). Each method's performance was documented and two were selected for modifications designed to improve estimate reliability. A regression of the CREAMS equation formulation against the observed data yielded an improved algorithm. The SCS unit hydrograph method was also adjusted to incorporate triangular unit hydrograph relationships as observed for flatwoods watersheds.

Data Summary

As discussed earlier, accurate data from flatwoods watersheds can be very elusive. Rainfall events associated with this study were generally small. Runoff from only one 24-hour, 5-year return period rainfall event was recorded. However within the data set were several large runoff producing events, including rainfall associated with Hurricane David.

The measurement of very high runoff events is made difficult by the total watershed submergence which occurs during these periods. Backwater effects complicate runoff measurement since structures or modifications designed for measurements under such conditions can often induce changes in drainage patterns or runoff time-distributions. As a result, precise data collection from flatwoods sites is very expensive and may simply not be feasible for very large "design" events.

The high areal variability of thunderstorm activity area also makes average watershed rainfall difficult to quantify. Where this problem was considered significant, data from additional raingages supplemented onsite measurements. Collection of water table elevations is less complicated and subsequently, greater confidence can be assigned to these data.

In general, the data demonstrated hydrologic response characteristics which are distinct from those typical of most small watersheds in the United States.

Total Volume Evaluation

The SCS runoff equation was developed for application to large design events occurring on small watersheds. However, in many instances it has been applied to smaller rainfall events with little or no consideration given to accuracy implications. Specific techniques employed to determine the watershed storage parameter, an input to the SCS runoff equation, have not been sufficiently evaluated as to their suitability for atypical watershed conditions.

Evaluations of the SCS equation and specific methods for determining its inputs demonstrate that large errors can be associated with runoff estimates for smaller events. For the seven methods examined, overall standard error of estimates ranged from several hundred to fifty percent. For both the larger and smaller events, best estimates of runoff volume resulted from techniques which incorporated antecedent water table conditions.

Three of the methods (DRM, ARS, and CR-1) relied upon measured water table elevations and performed similarly on small events. The ARS method consistently performed best on all event classes. The CR-1 method incorporates the ARS storage relationship, but has the added advantage of accounting for factors other than water table depth via the SCS curve number. This offers latitude useful in evaluating changes in runoff volume resulting from alternative land use patterns and agricultural practices. The CR-2 method has the same advantages, but rather than water table history or assumptions, a rainfall history is required. This method did not perform as well as CR-1. The SCS-Florida method considers strictly land use and soils, ignoring variations in watershed wetness. Therefore its use could lead to significant runoff estimation errors when applied to large events falling upon saturated watersheds. Neither the NEH-4 nor CR-WT methods should be used for runoff estimation on an event basis.

Estimates of runoff volume and the evaluation of prediction methods are more sensitive to errors in data collection and drainage area determination than are peak rates. However results demonstrate that techniques which incorporate water table levels (total available soil storage) can be expected to yield more accurate estimates of runoff volume for flatwoods watersheds.

Water Table Model

Since water table dynamics are such a critical factor in the hydrologic responses of flatwoods watersheds, an effort was made to quantify this phenomenon. The simplified model developed in this study performed reasonably well during the wet years. However during the end of prolonged dry periods, significant errors were apparent. With added sophistication, a reliable water table model appears feasible.

The model developed herein was not linked to a runoff prediction method since the usefulness of water table elevations in runoff estimates has already been demonstrated. Reapplication of model results to runoff estimates would simply be another measure of error in water table simulation.

Peak Rate Evaluation

Results of this study demonstrate that more accurate estimates of runoff peak rates can be expected as models progress from the empirical to the more physically based. However when empirical models are tailored to specific watershed conditions, results may be comparable to those from more complex models. As watershed conditions change or changes are anticipated, physically based models again become more reliable than empirical techniques.

The two extremes of empirical and physical models are represented by the CREAMS equation and the SFWMD overland flow computer program. The overland flow model performed best of all the original methods examined, however, it still demonstrated considerable overall error.

With modifications, estimation error was significantly reduced in the CREAMS equation and SCS unit hydrograph method. For the CREAMS equation, overall standard error of estimate was reduced from 3500% to 42%. For the unit hydrograph method, modifications reduced the overall standard error estimate from 250% to 45%. Between the two modified methods, the SCS method is more versatile and should be more transportable to other flatwoods watersheds. The SCS technique is capable of handling multiple-day (complex) events, whereas the CREAMS equation does not allow superposition.

Evaluation of the SCS unit hydrograph method demonstrated the need for improved algorithms for estimating time to peak. A simplified algorithm based upon data from the five study sites is presented, but its transportability has not been verified. Significant unit hydrograph results also indicate that the SCS recommended triangular hydrograph factor, 300, is too high. Analysis indicates that a value less than 100 is more appropriate for Florida's flatwoods watersheds. Also noteworthy were the almost identical peak rate estimates derived from measured and assumed rainfall time-distributions. Discharge hydrographs from flatwoods watersheds are much more attenuated and produce much lower peaks than most other small watersheds of the United States.

Future Research

Conclusions regarding current techniques and modifications to existing runoff volume and peak rate estimation methods are based only upon the currently available data. These conclusions and modifications should be verified by independent data collection as well as further studies into the specific hydrologic processes which characterize flatwoods watersheds. Effort at acquiring data from larger rainfall-runoff events should be pursued. However, given the expense and difficulty associated with such data collection, this may not be feasible.

Water table data collected by the USGS are good and upon further examination should serve as the basis for productive research into water table processes. Models of possible application include DRAINMOD (Skaggs, 1978) and the Green and Ampt, sandy soil infiltration models currently being developed by Shiromahedi and Skaggs (1983a&b). These models should not only help quantify the hydrologic processes controlling runoff volume, but should also help quantify subsurface and surface flow processes.

Models, like those noted, approach hydrology from a more physical perspective, an approach which has been shown to produce more accurate results. Physical models for infiltration, interflow and drainage, if combined with overland flow models as used by the South Florida Water Management District should do a good job of simulating runoff processes on flatwoods watersheds. Such a model should ideally include wetland and channel effects plus sufficient distribution to permit its application to a range of watershed sizes, land uses, and cultural practices. Another modeling approach which may be suited for handling the effects of watershed percent wetlands upon discharge hydrographs is the cascaded linear reservoir (Nash) model.

This degree of sophistication is neither necessary nor reasonable for all applications, yet would be useful for the design and operation of water control structures, the evaluation of crop management strategies (controlled water table conditions), the development of water quality loading models, the evaluation of wetlands alteration scenarios, and the development of regional land use/runoff models.

APPENDIX I

CURVE NUMBER SELECTION PROCEDURE

SCS NEH-4, Chapter 9 (USDA-SCS, 1972b) presents a standard procedure for determining runoff curve numbers. The following outline describes the SCS procedure as adapted for and applied to the flatwoods watersheds of this study.

I. Information and Equipment
   - A. SCS soil survey map of watershed.
   - B. Soil types and hydrologic classifications (NEH-4 Table 7.1)
   - C. Watershed crop cover and soil condition information.
   - D. Runoff curve numbers for hydrologic soil-cover complex (NEH-4 Table 9.1).
   - E. Curve number adjustment table for AMC (NEH-4 Table 10.1).
   - F. A planimeter, digitizer, or grid overlay.

II. Curve Number Estimation Procedure
   - A. Document each soil type occurring on the watershed.
   - B. Document hydrologic class (or classes) for each soil type from Table N of SCS NEH-4.
   - C. Estimate hydrologic class based upon effectiveness of drainage improvements. For a soil classed as A/D, assign A if very well drained and D if drainage is not sufficient to maintain the watertable well below the surface. Aerial photographs and USGS topographic maps are useful in determining the extent of drainage improvements.
   - D. Estimate hydrologic condition as judged from site inspection and Table 26.
   - E. Determine land use patterns over watershed from aerial photographs, USGS topographic maps or site inspection.
   - F. Determine appropriate curve number for each cover-soil complex (soil class, condition and land use combination) from Table 27 or SCS NEH-4 Table 9.1.
   - G. Determine fractional area occupied by each cover-soil complex.
   - H. Calculate overall watershed curve number (CN_{II})

III. Antecedent Moisture Condition Adjustment (as required by the specific method being implemented).
   - A. NEH-4: CN_I, CN_{II}, or CN_{III} as determined from Table 2.
   - B. SCS-Florida: CN_{II}.
   - C. CR-1, CR-2, or CR-WT: CN_I.

This procedure was used to estimate the runoff curve numbers for each of the study watersheds summarized in Table 28. Figure 53 shows the SCS soil survey delineation of the soil types occurring on SEZ Dairy. The soil types were lumped into four general groupings based upon similar soil types hydrologic classification range and land use. These subareas were digitized and a specific hydrologic classification was assigned to each as shown in Table 29. For the purposes of this study, wetlands were assigned a curve number of 100 at AMC=II and III and the curve number of the surrounding pasture at AMC=I.

Table 26. Criteria for determination of hydrologic condition from Table 8.1 of NEH-4 (USDA-SCS, 1972b).

Vegetative Condition	Hydrologic Condition
Heavily Grazed (plant cover <50%)	Poor
Not Heavily Grazed (50%< plant cover <75%)	Fair
Lightly Grazed (plant cover >75%)	Good

Table 27. SCS runoff curve numbers at AMC=II for selected land uses from Table 9.1 of NEH-4 (USDA-SCS, 1972b).

Land Use	Hydrologic Condition	Hydrologic Soil Group			
		A	B	C	D
Range or Pasture (with no mechanical treatment)	Poor	68	79	86	89
Same	Fair	49	69	79	84
Same	Good	39	61	74	80

Table 28. Runoff curve numbers determined for study watersheds.

Watershed	Antecedent Moisture Condition		
	I	II	III
Armstrong	63	82	91
Peavine (775)	63	84	93
Peavine (1800)	63	84	93
SEZ Dairy	63	81	92
Bass West	63	80	91
Bass East	63	80	91

Table 29. Curve number determination for SEZ Dairy.

Group	Class	Soil Series	Land Use	% of Area	Cond.	Group	CN_{II}
1	A/D A/D A/D A/D	Myakka Parkwood Charlotte Bass/Placid	Pasture	25.7	G	D	80
2	B/D	Immokalee	Pasture	53.2	G	D	80
3	A/D	Bass/Pompano	Pasture	14.1	G	D	80
4	A/D	Bass/Pompano	Wetlands	7.0			100
		SEZ Dairy Effective Curve Number					81

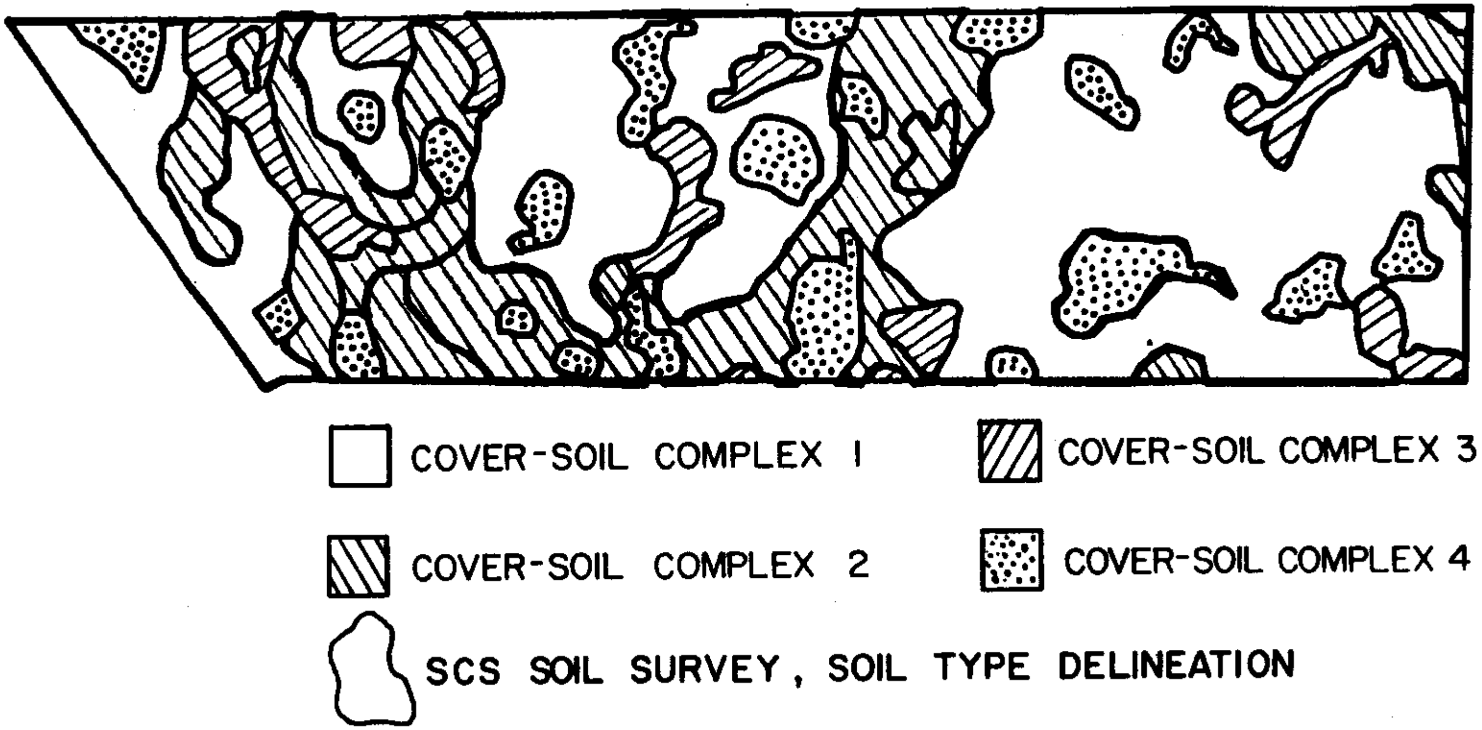


Figure 53. SEZ Dairy soil-cover complex delineation for use in determining the SCS runoff curve number. Soil types and distributions were identified from SCS soil survey (USDA-SCS, 1972a).

APPENDIX II

EVENT DATA AND EVALUATION RESULTS

Following are three sets of tables. The first set (Tables 30-34) is a summary of information for each of the events included in the evaluation data base. In the second set (Tables 35-39) are documentation of each runoff volume estimation technique performance. The third set (Tables 40-44) includes results of runoff peak rate estimation techniques as applied to the event data base.

Not every event was subjected to a given estimation technique. Some had insufficient data for implementation of certain estimation methods while others were judged to contain unacceptable measurement error.

The numbering system for rainfall-runoff events began at "01" for the runoff event with the highest observed peak for each site. The large Peavine Pasture events begin at "21" to distinguish them from runoff events considered to have resulted from the smaller contributing area. Events designated "51" or greater are rainfall events for which no runoff was recorded.

Table 30. Armstrong Slough events.

EVENT	DATE	JDAY	DAYS	RAIN IN	ROVOL IN	PEAK CFS	DWT FT	TP HR	TC HR
AS-01	6-17-82	168	2	4.59	2.920	136.00	2.63	20.0	24.0
AS-02	9-12-82	255	2	3.15	2.730	108.00	0.11	12.7	26.0
AS-03	5-30-82	149	2	3.36	2.000	96.00	2.35	23.0	28.0
AS-04	9- 7-82	250	1	3.15	0.680	87.00	2.20	7.0	
AS-05	4-15-82	105	2	1.14	1.340	80.00	0.00	29.7	29.0
AS-06	9- 9-82	252	1	1.08	0.930	68.00	0.03	10.0	32.0
AS-07	7- 6-82	187	1	1.74	0.880	48.00	1.40	23.0	28.0
AS-08	9- 7-81	249	2	2.06	0.160	32.00	1.88	7.0	
AS-09	2- 8-81	39	1	1.43	0.100	9.80	2.70	13.0	
AS-10	4- 1-80	92	1	1.26	0.087	8.40	2.40	7.2	
AS-11	3-22-81	81	1	1.42	0.095	8.00	2.59	8.0	
AS-12	8-13-80	226	1	1.31	0.089	7.80	1.28	12.0	30.0
AS-13	7-20-82	201	1	1.08	0.059	5.70	0.17	11.0	
AS-14	2-16-82	47	2	1.29	0.055	2.70	3.30		
AS-15	3- 6-82	65	2	1.48	0.070	2.30	3.35		
AS-51	7-31-80	213	1	3.48	0.000	0.00	2.72		
AS-52	5-26-82	146	1	2.33	0.000	0.00	3.65		
AS-53	5-30-81	150	1	1.94	0.000	0.00	4.74		
AS-54	6- 6-81	157	1	1.92	0.000	0.00	3.26		
AS-55	7-29-81	210	1	1.64	0.000	0.00	4.29		
AS-56	5-15-80	136	1	1.49	0.000	0.00	3.27		
AS-57	8-20-81	232	1	1.39	0.000	0.00	3.30		
AS-58	8-10-81	222	1	1.24	0.000	0.00	3.50		
AS-59	8-19-81	231	1	1.24	0.000	0.00	3.81		
AS-60	6-20-80	172	1	1.22	0.000	0.00	3.89		
AS-61	10-26-81	299	1	0.96	0.000	0.00	3.03		
AS-62	6-10-80	162	1	0.87	0.000	0.00	3.83		
AS-63	7- 8-81	189	1	0.86	0.000	0.00	3.84		
AS-64	10-27-81	300	1	0.84	0.000	0.00	2.66		
AS-65	11-28-80	333	1	0.80	0.000	0.00	2.40		
AS-66	7-15-80	197	1	0.82	0.000	0.00	3.22		
AS-67	7- 8-80	190	1	0.78	0.000	0.00	2.92		
AS-68	6-24-81	175	1	0.75	0.000	0.00	3.14		
AS-69	8-11-81	223	1	0.74	0.000	0.00	3.59		
AS-70	8- 8-81	220	1	0.71	0.000	0.00	3.27		

Table 31. Peavine Pasture events.

EVENT	DATE	JDAY	DAYS	RAIN IN	ROVOL IN	PEAK CFS	DWT FT	TP HR	TC HR
PV-01	7-19-82	200	2	1.81	0.840	8.50	0.66	20.0	42.0
PV-02	4-26-82	116	2	1.93	0.700	6.50	0.86	38.0	27.0
PV-03	7-29-82	210	1	0.00	0.620	5.70	0.21	40.0	
PV-04	3-28-82	87	2	1.67	0.680	4.95	1.76	34.0	35.0
PV-05	8-29-81	241	1	0.71	0.340	4.20	0.18	48.0	38.0
PV-06	8-20-81	232	2	1.49	0.240	3.30	0.64	30.0	22.0
PV-07	7-31-80	213	1	2.86	0.240	3.10	2.42	10.0	20.0
PV-08	11- 5-81	309	1	1.43	0.240	2.40	1.22	15.0	10.0
PV-09	8- 2-80	215	1	0.74	0.220	1.80	0.32	10.0	18.0
PV-10	5-26-82	146	1	3.03	0.220	1.00	3.00	18.0	
PV-11	10-26-81	299	2	1.33	0.200	0.96	3.12		
PV-12	3- 1-80	61	1	0.78	0.070	0.70	1.41	24.0	38.0
PV-13	3- 6-82	65	1	1.05	0.060	0.60	2.19	34.0	30.0
PV-14	4- 1-80	92	1	1.23	0.020	0.24	2.25	30.0	
PV-21	9- 3-79	246	2	3.93	2.400	42.00		24.0	
PV-22	9-14-79	257	1	3.08	2.350	38.00		10.0	
PV-23	9- 9-79	252	2	1.22	1.060	20.50		26.0	
PV-24	6-17-82	168	2	3.54	2.600	18.50	1.80	47.0	34.0
PV-25	8-27-82	239	2	3.00	1.700	16.00	0.95	58.0	39.0
PV-26	9- 6-81	249	2	1.72	1.300	16.00	0.70	40.0	20.0
PV-27	7- 4-82	185	2	1.71	1.130	16.00	0.61	28.0	42.0
PV-28	5-29-82	149	3	1.98	1.200	15.00	0.80	50.0	30.0
PV-29	9-21-82	264			0.930	13.50		40.0	32.0
PV-30	8-10-79	222	2	0.96	0.930	12.50		24.0	
PV-31	9-26-82	269			0.750	10.50		30.0	30.0
PV-32	4-15-82	105	1	1.18	1.020	8.37	0.69	27.0	35.0
PV-51	8-10-81	222	3	2.71	0.000	0.00	2.95		
PV-52	11-15-80	320	3	2.18	0.000	0.00	4.35		
PV-53	3-22-81	81	1	1.52	0.000	0.00	3.32		
PV-54	5-15-80	135	1	1.49	0.000	0.00	3.04		
PV-55	6-20-80	172	1	1.47	0.000	0.00	3.49		
PV-56	2- 8-81	39	1	1.42	0.000	0.00	3.16		
PV-57	5- 9-80	130	1	1.27	0.000	0.00	3.46		
PV-58	6-10-80	162	1	1.20	0.000	0.00	3.36		
PV-59	6- 6-81	157	1	1.19	0.000	0.00	3.99		
PV-60	7- 8-81	189	1	1.17	0.000	0.00	2.29		
PV-61	5-22-80	143	2	1.15	0.000	0.00	2.60		
PV-62	7-29-81	210	1	1.12	0.000	0.00	2.54		
PV-63	6- 1-81	151	1	1.10	0.000	0.00	4.36		
PV-64	12-16-80	351	1	0.99	0.000	0.00	3.91		

Table 32. SEZ Dairy events.

EVENT	DATE	JDAY	DAYS	RAIN IN	ROVOL IN	PEAK CFS	BWT FT	TP HR	TC HR
SD-01	5-26-82	146	1	3.78	1.260	14.00	3.19	11.5	10.0
SD-02	4-16-82	106	1	1.14	0.670	11.00	2.65	11.0	10.0
SD-03	3-28-82	87	2	2.10	0.720	9.00	4.00	14.0	9.0
SD-04	8-27-81	239	3	1.92	1.260	8.50	0.89	20.0	18.0
SD-05	7-25-80	207	1	1.74	0.470	6.30	1.06	12.8	22.0
SD-06	8-10-80	223	1	2.70	0.330	4.30	1.16	19.0	23.0
SD-07	4-26-82	116	1	1.44	0.390	4.00	1.86	12.5	
SD-08	8- 5-82	217	1	0.72	0.150	2.80	1.85	4.7	
SD-09	11-15-80	320	3	3.36	0.190	1.40	4.10	40.0	
SD-10	7-15-80	197	1	3.12	0.100	1.30	3.85	20.0	
SD-11	8-13-81	225	1	0.84	0.110	0.85	2.72	30.0	
SD-12	3- 5-82	64	3	2.88	0.110	0.70	4.55	20.0	
SD-13	1-26-80	26	2	1.20	0.140	0.60	3.80	25.0	
SD-14	2- 8-81	39	1	0.84	0.140	0.50	3.51	40.0	
SD-15	4- 6-80	98	1	0.84	0.050	0.20	3.02	10.0	
SD-51	6-22-80	174	2	2.04	0.000	0.00	4.29		
SD-52	5- 7-81	127	1	2.04	0.000	0.00	4.73		
SD-53	6-20-81	171	1	1.68	0.000	0.00	4.16		
SD-54	7-19-81	200	1	1.20	0.000	0.00	3.75		
SD-55	6- 8-81	159	1	1.08	0.000	0.00	4.89		
SD-56	5-20-81	140	1	1.02	0.000	0.00	4.74		
SD-57	6- 7-81	158	1	0.96	0.000	0.00	4.88		
SD-58	6- 9-81	160	1	0.90	0.000	0.00	4.90		
SD-59	7- 1-80	183	1	0.90	0.000	0.00	3.59		
SD-60	6-28-81	179	1	0.72	0.000	0.00	4.00		

Table 33. Bass West Pasture events.

EVENT	DATE	JDAY	DAYS	RAIN IN	ROVOL IN	PEAK CFS	DWT FT	TP HR	TC HR
BW-01	9- 3-79	246	2	5.20	5.200	32.00	0.00		
BW-02	9-15-80	259	2	3.84	3.220	14.00	0.88	6.0	11.0
BW-03	8-31-80	244	2	2.70	2.700	14.00	0.08	13.7	9.0
BW-04	6-18-82	168	2	3.36	2.840	12.00	1.13	11.0	15.0
BW-05	8-15-82	227	1	3.54	2.110	11.00	2.03	7.5	15.0
BW-06	7-16-80	198	3	2.46	2.300	10.50	1.39		
BW-07	8-30-80	243	1	2.82	2.090	8.50	2.00	7.8	19.0
BW-08	2-18-80	49	2	1.56	1.490	7.10	0.68	10.9	
BW-09	9- 2-80	246	1	1.44	1.310	7.00	0.00	10.0	
BW-10	7-20-82	201	1	1.02	1.670	7.00	0.00	10.0	
BW-11	9- 7-82	250	1	1.98	2.140	6.30	3.24	10.0	10.0
BW-12	9- 7-80	251	1	1.50	1.150	5.80	0.25	9.0	14.0
BW-13	5-31-82	151	1	1.32	1.000	5.00	1.65	7.2	17.0
BW-14	9- 8-82	251	2	1.08	1.080	4.80	1.01	8.0	12.0
BW-15	3-31-80	91	1	1.50	0.520	3.00	1.88	7.0	21.0
BW-16	7- 6-82	187	1	0.84	0.710	2.40	1.40	13.0	
BW-17	4- 7-80	98	1	1.08	0.500	2.30	1.10	14.1	
BW-18	7-19-82	200	1	0.90	0.610	3.00	2.15	12.0	
BW-19	3- 1-80	61	1	0.72	0.350	1.70	1.75	16.4	
BW-20	9- 1-82	244	1	1.02	0.260	1.50	2.87	7.5	
BW-21	4-19-80	110	1	1.02	0.280	1.40	2.30	11.7	14.0
BW-22	8-23-80	236	1	1.08	0.170	1.10	1.50	7.0	
BW-23	10- 5-82	278	2	0.84	0.070	0.64	1.43	18.0	
BW-51	7-15-80	197	1	2.52	0.000	0.00	3.48		
BW-52	7-17-81	198	1	2.46	0.000	0.00	4.57		
BW-53	8-24-81	236	1	1.56	0.000	0.00	3.55		
BW-54	7-14-80	196	1	1.38	0.000	0.00	4.00		
BW-55	2-10-82	41	1	1.20	0.000	0.00	3.41		
BW-56	2- 2-81	33	1	1.20	0.000	0.00	3.46		
BW-57	2- 8-81	39	1	1.20	0.000	0.00	2.64		
BW-58	8- 2-81	214	1	1.02	0.000	0.00	5.13		
BW-59	1-26-80	26	1	0.96	0.000	0.00	3.42		
BW-60	11-15-80	320	1	0.96	0.000	0.00	3.67		
BW-61	3- 7-82	66	1	0.90	0.000	0.00	1.45		
BW-62	7-11-81	192	1	0.84	0.000	0.00	4.57		
BW-63	2-17-82	48	1	0.84	0.000	0.00	2.60		
BW-64	9-21-82	264	1	0.84	0.000	0.00	2.50		
BW-65	6-28-81	179	1	0.72	0.000	0.00	4.38		

Table 34. Bass East Pasture events.

EVENT	DATE	JDAY	DAYS	RAIN IN	ROVOL IN	PEAK CFS	DWT FT	TP HR	TC HR
BE-01	9- 3-79	246	3	4.56	4.090	1.90	1.00	26.0	
BE-02	2- 8-80	39	2	1.68	3.340	1.75	1.98	11.0	
BE-03	8-31-80	244	2	2.70	1.820	0.95	0.41	11.0	18.0
BE-04	9- 2-80	246	1	1.32	1.220	0.65	0.23	11.0	22.0
BE-05	9-15-80	259	1	3.36	1.140	0.61	0.96	16.0	19.0
BE-06	9- 7-80	251	1	1.50	0.860	0.50	0.08	15.0	25.0
BE-07	3- 1-80	61	1	0.72	0.350	0.49	2.06	12.0	
BE-08	8-30-79	242	2	0.96	1.530	0.46	0.00	35.0	
BE-09	9-21-79	264	3	2.16	2.600	0.42	0.00	50.0	
BE-10	4- 1-80	92	1	1.26	0.640	0.31	1.15	18.0	
BE-11	7-16-80	198	3	2.46	0.570	0.25	0.00	50.0	
BE-12	8-30-80	243	1	2.82	0.330	0.20	2.58	16.0	
BE-13	1-26-80	26	2	1.32	0.400	0.16	3.38		
BE-51	7-15-80	197	1	2.52	0.000	0.00	3.21		
BE-52	7-17-81	198	1	2.46	0.000	0.00	5.00		
BE-53	8-24-81	236	1	1.56	0.000	0.00	3.69		
BE-54	3-31-80	91	1	1.50	0.000	0.00	1.50		
BE-55	7-14-80	196	1	1.38	0.000	0.00	3.73		
BE-56	2- 2-81	33	1	1.20	0.000	0.00	3.43		
BE-57	2- 8-81	39	1	1.20	0.000	0.00	3.01		
BE-58	2-10-82	41	1	1.20	0.000	0.00	3.86		
BE-59	4- 7-80	98	1	1.08	0.000	0.00	1.21		
BE-60	8-23-80	236	1	1.08	0.000	0.00	2.53		
BE-61	4-19-80	110	1	1.02	0.000	0.00	2.55		
BE-62	11-15-80	320	1	0.96	0.000	0.00	3.78		
BE-63	6-21-80	173	1	0.90	0.000	0.00	3.76		
BE-64	3- 7-82	66	1	0.90	0.000	0.00	3.14		
BE-65	2-16-82	48	1	0.84	0.000	0.00	3.22		
BE-66	7-11-81	192	1	0.84	0.000	0.00	4.96		
BE-67	6-27-81	179	1	0.72	0.000	0.00	4.97		

Table 35. Armstrong Slough total volume evaluation results.

EVENT	RAIN IN	DWT FT	AMC	CNEFF%	RO IN	NEH4 IN	SCSFL IN	DRM IN	ARS IN	CR-1 IN	CR-2 IN	CR-WT IN
AS-01	4.59	2.63	1	84.00	2.92	1.26	2.72	1.50	2.21	1.25	2.58	2.32
AS-02	3.15	0.11	1	96.00	2.73	0.50	1.50	3.14	3.08	3.14	2.03	1.72
AS-03	3.36	2.35	3	86.00	2.00	2.50	1.67	1.05	1.43	0.88	2.72	0.57
AS-06	1.08	0.03	1	99.00	0.93	0.00	0.14	1.08	1.06	1.08	0.15	0.08
AS-07	1.74	1.40	1	90.00	0.88	0.05	0.48	0.82	0.78	0.73	0.60	0.26
AS-08	2.06	1.88	1	65.00	0.16	0.12	0.69	0.66	0.75	0.55	0.97	0.40
AS-09	1.43	2.70	1	72.00	0.10	0.01	0.31	0.02	0.15	0.01	0.05	0.06
AS-10	1.26	2.40	1	75.00	0.09	0.00	0.22	0.05	0.14	0.02	0.06	0.12
AS-11	1.42	2.59	1	72.00	0.09	0.01	0.30	0.04	0.17	0.01	0.03	0.00
AS-12	1.31	1.28	1	74.00	0.09	0.00	0.25	0.58	0.52	0.51	1.06	0.52
AS-13	1.08	0.17	1	76.00	0.06	0.00	0.14	1.06	0.98	1.06	0.53	0.07
AS-14	1.29	3.30	1	71.00	0.05	0.00	0.24	0.01	0.05	0.04	0.05	0.00
AS-15	1.48	3.35	1	69.00	0.07	0.02	0.33	0.00	0.08	0.02	0.10	0.05
AS-51	3.48	2.72	1	36.00	0.00	0.65	1.77	0.74	1.31	0.57	2.45	1.29
AS-52	2.33	3.65	1	46.00	0.00	0.19	0.88	0.02	0.32	0.00	0.56	0.22
AS-53	1.94	4.74	1	51.00	0.00	0.09	0.61	0.06	0.07	0.12	0.19	0.07
AS-54	1.92	3.26	1	51.00	0.00	0.08	0.60	0.02	0.24	0.00	0.33	0.38
AS-55	1.64	4.29	1	55.00	0.00	0.03	0.42	0.05	0.04	0.11	0.35	0.17
AS-56	1.49	3.27	1	57.00	0.00	0.02	0.34	0.00	0.09	0.01	0.07	0.29
AS-57	1.39	3.30	3	59.00	0.00	0.71	0.29	0.00	0.07	0.02	1.39	0.47
AS-58	1.24	3.50	1	62.00	0.00	0.00	0.21	0.03	0.03	0.07	0.99	0.12
AS-59	1.24	3.81	1	62.00	0.00	0.00	0.21	0.06	0.01	0.12	1.05	0.48
AS-60	1.22	3.89	1	62.00	0.00	0.00	0.20	0.08	0.01	0.14	0.15	0.00
AS-61	0.96	3.03	1	68.00	0.00	0.01	0.10	0.02	0.01	0.05	0.00	0.04
AS-62	0.87	3.83	1	70.00	0.00	0.02	0.07	0.15	0.00	0.22	0.02	0.00
AS-63	0.86	3.84	1	70.00	0.00	0.02	0.07	0.15	0.00	0.23	0.06	0.00
AS-64	0.84	2.66	3	70.00	0.00	0.29	0.06	0.01	0.01	0.02	0.02	0.00
AS-65	0.80	2.40	1	71.00	0.00	0.03	0.05	0.00	0.02	0.00	0.00	0.00
AS-66	0.82	3.22	1	71.00	0.00	0.02	0.06	0.07	0.00	0.11	0.17	0.00
AS-67	0.78	2.92	1	72.00	0.00	0.03	0.05	0.04	0.00	0.07	0.37	0.03
AS-68	0.75	3.14	1	73.00	0.00	0.03	0.04	0.07	0.00	0.12	0.06	0.03
AS-69	0.74	3.59	3	73.00	0.00	0.22	0.04	0.15	0.01	0.21	0.74	0.06
AS-70	0.71	3.27	1	74.00	0.00	0.04	0.03	0.10	0.00	0.15	0.12	0.00

Table 36. Peavine Pasture total volume evaluation results.

EVENT	RAIN IN	DWT FT	ANC	CNEFF%	RO IN	NEH4 IN	SCSFL IN	DRM IN	ARS IN	CR-1 IN	CR-2 IN	CR-WT IN
PV-02	1.93	0.86	1	84.00	0.70	0.09	0.69	1.49	1.32	1.42	0.44	0.21
PV-04	1.67	1.76	2	87.00	0.68	0.52	0.52	0.50	0.54	0.41	0.12	0.12
PV-05	0.71	0.18	1	95.00	0.34	0.04	0.05	0.69	0.61	0.69	0.61	0.07
PV-06	1.49	0.64	1	78.00	0.24	0.02	0.41	1.24	1.09	1.20	0.71	0.04
PV-07	2.86	2.42	1	58.00	0.24	0.38	1.40	0.68	1.03	0.53	1.28	0.88
PV-08	1.43	1.22	1	96.00	1.00	0.01	0.37	0.72	0.64	0.64	0.12	0.35
PV-09	0.74	0.32	3	92.00	0.22	0.26	0.06	0.68	0.57	0.67	0.52	0.07
PV-10	3.03	3.00	1	50.00	0.10	0.45	1.54	0.35	0.88	0.24	0.81	0.86
PV-11	1.33	3.12	1	80.00	0.20	0.00	0.32	0.00	0.07	0.01	0.03	0.00
PV-12	0.78	1.41	1	77.00	0.01	0.03	0.07	0.16	0.14	0.12	0.00	0.00
PV-13	1.05	2.19	1	77.00	0.06	0.00	0.17	0.05	0.10	0.02	0.00	0.00
PV-14	1.23	2.25	1	69.00	0.02	0.00	0.26	0.08	0.16	0.04	0.01	0.00
PV-21	3.93	0.01	1	85.00	2.40	0.88	2.31	3.93	3.92	3.93	0.19	1.89
PV-22	3.08	0.01	3	93.00	2.35	2.33	1.58	3.08	3.07	3.08	0.04	2.27
PV-23	1.22	0.01	1	99.00	1.06	0.00	0.26	1.22	1.21	1.22	0.12	0.21
PV-24	3.54	1.80	1	91.00	2.60	0.68	1.97	1.87	1.98	1.69	2.07	0.85
PV-25	3.00	0.95	1	86.00	1.70	0.43	1.52	2.43	2.26	2.35	1.24	0.31
PV-26	1.72	0.70	1	96.00	1.30	0.05	0.55	1.42	1.26	1.37	0.90	0.10
PV-28	1.98	0.80	3	92.00	1.20	1.30	0.73	1.59	1.42	1.53	1.42	0.00
PV-30	0.96	0.01	1	100.00	0.93	0.01	0.13	0.96	0.95	0.96	0.19	0.05
PV-32	1.18	0.69	1	99.00	1.02	0.00	0.24	0.90	0.76	0.86	0.12	0.06
PV-51	2.71	2.95	1	42.00	0.00	0.32	1.28	0.26	0.70	0.17	0.81	0.02
PV-52	2.18	4.35	1	48.00	0.00	0.15	0.87	0.01	0.16	0.04	0.19	0.00
PV-53	1.52	3.32	1	57.00	0.00	0.02	0.43	0.00	0.10	0.01	0.04	0.09
PV-54	1.49	3.04	1	57.00	0.00	0.02	0.41	0.00	0.12	0.00	0.10	0.37
PV-55	1.47	3.49	1	58.00	0.00	0.01	0.40	0.01	0.07	0.03	0.16	0.15
PV-56	1.42	3.16	1	58.00	0.00	0.01	0.37	0.00	0.09	0.01	0.04	0.11
PV-57	1.27	3.46	1	61.00	0.00	0.00	0.28	0.02	0.03	0.06	0.01	0.03
PV-58	1.20	3.36	1	63.00	0.00	0.00	0.25	0.02	0.03	0.06	0.08	0.00
PV-59	1.19	3.99	1	63.00	0.00	0.00	0.24	0.10	0.00	0.16	0.05	0.05
PV-60	1.17	2.29	1	63.00	0.00	0.00	0.23	0.05	0.13	0.03	0.05	0.02
PV-61	1.15	2.60	1	63.00	0.00	0.00	0.22	0.01	0.08	0.00	0.12	0.02
PV-62	1.12	2.54	1	64.00	0.00	0.00	0.21	0.01	0.08	0.00	0.09	0.00
PV-63	1.10	4.36	1	65.00	0.00	0.00	0.20	0.17	0.00	0.26	0.00	0.00
PV-64	0.99	3.91	1	67.00	0.00	0.01	0.15	0.13	0.00	0.20	0.00	0.02

Table 37. SEZ Dairy total volume evaluation results.

EVENT	RAIN IN	DWT FT	AMC	CNEFF%	RO IN	NEH4 IN	SCSFL IN	DRM IN	ARS IN	CR-1 IN	CR-2 IN	CR-WT IN
SD-01	3.78	3.19	1	71.00	1.26	0.80	1.94	0.54	1.28	0.39	1.63	1.17
SD-02	1.14	2.65	1	95.00	0.67	0.00	0.15	0.00	0.07	0.00	0.08	0.05
SD-03	2.10	4.00	1	82.00	0.72	0.13	0.67	0.00	0.18	0.02	0.32	0.02
SD-04	1.92	0.89	2	93.00	1.26	0.55	0.55	1.45	1.29	1.38	1.92	0.44
SD-05	1.74	1.06	2	82.00	0.47	0.45	0.45	1.12	1.00	1.05	1.74	0.52
SD-06	2.70	1.16	1	64.00	0.33	0.31	1.09	1.91	1.78	1.80	1.21	0.28
SD-07	1.44	1.86	1	84.00	0.39	0.01	0.28	0.31	0.36	0.24	0.19	0.13
SD-08	0.72	1.85	1	90.00	0.15	0.04	0.02	0.03	0.05	0.01	0.12	0.00
SD-09	3.36	4.10	1	51.00	0.19	0.59	1.60	0.10	0.69	0.03	0.78	0.23
SD-10	3.12	3.85	1	49.00	0.10	0.48	1.41	0.10	0.64	0.04	1.96	0.66
SD-11	0.84	2.72	1	85.00	0.11	0.02	0.05	0.01	0.01	0.03	0.23	0.00
SD-12	2.88	4.55	1	52.00	0.11	0.38	1.22	0.00	0.37	0.00	0.39	0.00
SD-13	1.20	3.80	1	79.00	0.14	0.00	0.17	0.07	0.01	0.13	0.12	0.00
SD-14	0.84	3.51	1	87.00	0.14	0.02	0.05	0.11	0.00	0.17	0.00	0.00
SD-15	0.84	3.02	1	81.00	0.05	0.02	0.05	0.04	0.00	0.07	0.00	0.00
SD-51	2.04	4.29	1	50.00	0.00	0.11	0.63	0.01	0.13	0.05	0.37	0.01
SD-52	2.04	4.73	1	50.00	0.00	0.11	0.63	0.04	0.09	0.10	0.12	0.13
SD-53	1.68	4.16	1	54.00	0.00	0.04	0.41	0.04	0.06	0.09	0.43	0.15
SD-54	1.20	3.75	2	63.00	0.00	0.17	0.17	0.06	0.01	0.12	0.78	0.14
SD-55	1.08	4.89	1	65.00	0.00	0.00	0.13	0.27	0.01	0.38	0.07	0.04
SD-56	1.02	4.74	1	66.00	0.00	0.00	0.10	0.26	0.01	0.37	0.01	0.00
SD-57	0.96	4.88	1	68.00	0.00	0.01	0.08	0.31	0.02	0.42	0.00	0.00
SD-58	0.90	4.90	3	69.00	0.00	0.33	0.07	0.33	0.03	0.45	0.15	0.04
SD-59	0.90	3.59	1	69.00	0.00	0.01	0.07	0.10	0.00	0.16	0.40	0.00
SD-60	0.72	4.00	1	74.00	0.00	0.04	0.02	0.23	0.02	0.31	0.47	0.00

Table 38. Bass West Pasture total volume evaluation results.

EVENT	RAIN IN	DWT FT	AMC	CNEFF%	RO IN	NEH4 IN	SCSFL IN	DRM IN	ARS IN	CR-1 IN	CR-2 IN	CR-WT IN
BW-01	5.20	0.01	1	100.00	5.20	1.64	3.07	5.20	5.19	5.20	5.00	1.18
BW-02	3.84	0.88	1	94.00	3.22	0.83	1.91	3.33	3.13	3.26	2.74	2.57
BW-03	2.70	0.08	3	100.00	2.70	1.79	1.03	2.70	2.65	2.70	2.70	1.06
BW-04	3.36	1.13	1	95.00	2.84	0.59	1.53	2.57	2.42	2.46	1.24	0.49
BW-05	3.54	2.03	1	86.00	2.11	0.68	1.67	1.59	1.80	1.40	2.79	1.30
BW-06	2.46	1.39	3	99.00	2.30	1.57	0.86	1.43	1.38	1.32	2.46	0.17
BW-07	2.82	2.00	1	93.00	2.09	0.36	1.12	1.08	1.24	0.93	1.56	0.55
BW-08	1.56	0.68	1	99.00	1.49	0.02	0.32	1.28	1.12	1.24	0.17	0.13
BW-09	1.44	0.01	3	99.00	1.31	0.69	0.26	1.44	1.43	1.44	1.44	0.27
BW-12	1.50	0.25	2	97.00	1.15	0.29	0.29	1.46	1.36	1.45	1.50	0.42
BW-13	1.32	1.65	1	97.00	1.00	0.00	0.20	0.35	0.37	0.28	0.14	0.04
BW-14	1.08	1.01	2	100.00	1.08	0.11	0.11	0.58	0.48	0.53	1.08	0.11
BW-15	1.50	1.88	1	87.00	0.52	0.02	0.29	0.33	0.39	0.25	0.08	0.11
BW-16	0.84	1.40	1	99.00	0.71	0.02	0.04	0.19	0.18	0.15	0.15	0.00
BW-17	1.08	1.10	1	93.00	0.50	0.00	0.11	0.52	0.44	0.46	0.18	0.21
BW-18	0.90	2.15	1	97.00	0.61	0.01	0.06	0.03	0.06	0.01	0.19	0.00
BW-19	0.72	1.75	1	95.00	0.35	0.04	0.02	0.05	0.06	0.03	0.00	0.00
BW-20	1.02	2.87	1	88.00	0.26	0.00	0.09	0.01	0.03	0.02	0.19	0.03
BW-21	1.02	2.30	1	88.00	0.28	0.00	0.09	0.02	0.08	0.01	0.08	0.04
BW-22	1.08	1.50	1	83.00	0.17	0.00	0.11	0.29	0.28	0.23	0.20	0.00
BW-23	0.84	1.43	1	83.00	0.07	0.02	0.04	0.18	0.17	0.14	0.29	0.00
BW-51	2.52	3.48	1	44.00	0.00	0.25	0.90	0.07	0.44	0.02	1.45	0.48
BW-52	2.46	4.57	1	45.00	0.00	0.23	0.86	0.00	0.21	0.03	0.77	0.25
BW-53	1.56	3.55	1	56.00	0.00	0.02	0.32	0.01	0.08	0.03	1.09	0.07
BW-54	1.38	4.00	1	59.00	0.00	0.01	0.23	0.06	0.02	0.12	0.18	0.00
BW-55	1.20	3.41	1	63.00	0.00	0.00	0.15	0.03	0.02	0.06	0.01	0.00
BW-56	1.20	3.46	1	63.00	0.00	0.00	0.15	0.03	0.02	0.07	0.00	0.03
BW-57	1.20	2.64	1	63.00	0.00	0.00	0.15	0.01	0.09	0.00	0.02	0.10
BW-58	1.02	5.13	1	66.00	0.00	0.00	0.09	0.33	0.02	0.45	0.36	0.00
BW-59	0.96	3.42	1	68.00	0.00	0.01	0.07	0.07	0.00	0.12	0.00	0.00
BW-60	0.96	3.67	1	68.00	0.00	0.01	0.07	0.10	0.00	0.16	0.00	0.30
BW-61	0.90	1.45	2	69.00	0.00	0.06	0.06	0.21	0.19	0.16	0.03	0.00
BW-62	0.84	4.57	1	70.00	0.00	0.02	0.04	0.29	0.02	0.39	0.01	0.00
BW-63	0.84	2.60	1	70.00	0.00	0.02	0.04	0.00	0.01	0.02	0.00	0.00
BW-64	0.84	2.50	1	70.00	0.00	0.02	0.04	0.00	0.02	0.01	0.07	0.00
BW-65	0.72	4.38	1	74.00	0.00	0.04	0.02	0.30	0.04	0.40	0.00	0.00

Table 39. Bass East Pasture total volume evaluation results.

EVENT	RAIN IN	DWT FT	ANC	CNEFF%	RO IN	NEH4 IN	SCSFL IN	DRM IN	ARS IN	CR-1 IN	CR-2 IN	CR-WT IN
BE-01	4.56	1.00	1	96.00	4.09	1.24	2.51	3.91	3.72	3.80	3.42	1.17
BE-03	2.70	0.41	3	91.00	1.82	1.79	1.03	2.59	2.46	2.57	2.70	1.06
BE-04	1.32	0.23	3	99.00	1.22	0.60	0.20	1.29	1.19	1.28	1.32	0.27
BE-05	3.36	0.96	1	74.00	1.14	0.59	1.53	2.77	2.59	2.69	2.29	2.23
BE-06	1.50	0.08	2	93.00	0.86	0.29	0.29	1.50	1.45	1.50	1.50	0.42
BE-07	0.72	2.06	1	95.00	0.35	0.04	0.02	0.01	0.03	0.00	0.00	0.00
BE-10	1.26	1.15	2	93.00	0.64	0.18	0.18	0.63	0.55	0.56	0.18	0.19
BE-11	2.46	0.01	3	74.00	0.57	1.57	0.86	2.46	2.45	2.46	2.46	0.16
BE-12	2.82	2.58	1	62.00	0.33	0.36	1.12	0.52	0.93	0.39	1.56	0.55
BE-13	1.32	3.38	1	87.00	0.40	0.00	0.20	0.01	0.05	0.04	0.02	0.00
BE-51	2.52	3.21	1	44.00	0.00	0.25	0.90	0.12	0.52	0.06	1.45	0.48
BE-52	2.46	5.00	1	45.00	0.00	0.23	0.86	0.02	0.16	0.07	0.77	0.25
BE-53	1.56	3.69	1	56.00	0.00	0.02	0.32	0.01	0.07	0.04	1.09	0.07
BE-54	1.50	1.50	1	57.00	0.00	0.02	0.29	0.56	0.55	0.48	0.08	0.11
BE-55	1.38	3.73	1	59.00	0.00	0.01	0.23	0.04	0.03	0.08	0.18	0.00
BE-56	1.20	3.43	1	63.00	0.00	0.00	0.15	0.03	0.02	0.07	0.00	0.03
BE-57	1.20	3.01	1	63.00	0.00	0.00	0.15	0.00	0.05	0.02	0.02	0.10
BE-58	1.20	3.86	1	63.00	0.00	0.00	0.15	0.08	0.01	0.14	0.01	0.00
BE-59	1.08	1.21	1	65.00	0.00	0.00	0.11	0.45	0.39	0.39	0.18	0.21
BE-60	1.08	2.53	1	65.00	0.00	0.00	0.11	0.01	0.07	0.00	0.20	0.00
BE-61	1.02	2.55	1	66.00	0.00	0.00	0.09	0.00	0.05	0.00	0.08	0.04
BE-62	0.96	3.78	1	68.00	0.00	0.01	0.07	0.12	0.00	0.18	0.00	0.30
BE-63	0.90	3.76	1	69.00	0.00	0.01	0.06	0.13	0.00	0.20	0.00	0.00
BE-64	0.90	3.14	2	69.00	0.00	0.06	0.06	0.04	0.00	0.08	0.03	0.00
BE-65	0.84	3.22	1	70.00	0.00	0.02	0.04	0.07	0.00	0.11	0.00	0.00
BE-66	0.84	4.96	1	70.00	0.00	0.02	0.04	0.36	0.04	0.48	0.01	0.00
BE-67	0.72	4.97	1	74.00	0.00	0.04	0.02	0.41	0.06	0.53	0.00	0.00

Table 40. Armstrong Slough peak rate evaluation results.

EVENT	PEAK	CYPRESS	CHART	OLDCREAMS	NEWCREAMS	UNIT-SCS	UNIT-MOD	WSHS
AS-01	136.00	420.70	201.48	1584.57	125.42	274.68	139.63	81.00
AS-02	108.00	400.87	188.37	1472.81	119.57	259.18	130.31	73.00
AS-03	96.00	324.69	138.00	1098.07	98.72	189.02	95.20	46.00
AS-04	87.00	186.93	46.92	396.74	50.79	64.57	33.17	29.00
AS-05	80.00	255.81	92.46	752.49	77.14			19.00
AS-06	68.00	213.02	64.17	533.11	61.59	88.29	44.40	12.00
AS-07	48.00	207.80	60.72	506.02	59.53	82.24	42.15	15.00
AS-08	32.00	132.66	11.04	101.28	20.83	15.89	8.00	
AS-09	9.80	126.40	6.90	65.00	15.59	9.97	5.01	
AS-10	8.40	125.05	6.00	58.85	14.61	8.68	4.36	
AS-11	8.00	125.88	6.56	58.85	14.61	9.48	4.76	
AS-12	7.80	125.25	6.14	58.85	14.61	8.88	4.46	
AS-13	5.70	122.12	4.07	40.14	11.38	5.93	2.97	
AS-14	2.70	121.71	3.80	33.79	10.18	5.56	2.78	
AS-15	2.30	123.27	4.83	46.42	12.52	7.06	3.53	

Table 41. SEZ Dairy peak rate evaluation results.

EVENT	PEAK	CYPRESS	CHART	OLDCREAMS	NEWCREAMS	UNIT-SCS	UNIT-MOD	WSHS
SD-01	14.00	75.56	23.94	201.44	11.91	31.97	8.41	10.00
SD-02	11.00	56.76	12.73	112.77	6.95	17.07	16.00	3.00
SD-03	9.00	58.35	13.68	120.47	7.39	18.25	9.14	2.00
SD-04	8.50	75.56	23.94	201.44	11.91	32.54	15.70	8.00
SD-05	6.30	50.39	8.93	81.42	5.14	12.03	6.01	
SD-06	4.30	45.93	6.27	58.84	3.80	8.90	4.35	
SD-07	4.00	47.84	7.41	68.60	4.38	9.98	4.99	
SD-08	2.80	40.19	2.85	28.52	1.94	3.90	1.94	
SD-09	1.40	41.46	3.61	35.43	2.38	5.08	2.44	
SD-10	1.30	38.60	1.90	19.65	1.38	2.88	1.36	
SD-11	0.85	38.92	2.09	21.45	1.49	2.95	1.45	
SD-12	0.70	38.92	2.09	21.45	1.49	2.94	1.40	
SD-13	0.60	39.87	2.66	26.77	1.83	3.79	1.85	
SD-14	0.50	39.87	2.66	26.77	1.83	3.70	1.82	
SD-15	0.20	37.00	0.95	10.40	0.76	1.40	0.67	

Table 42. Peavine Pasture peak rate evaluation results.

EVENT	PEAK	CYPRESS	CHART	OLDCREAMS	NEWCREAMS	UNIT-SCS	UNIT-MOD	WSHS
PV-01	8.50	66.27	25.20	169.15	6.96	27.34	7.78	5.00
PV-02	6.50	61.51	21.00	143.03	5.97	22.82	6.51	2.00
PV-03	5.70	58.80	18.60	127.92	5.39			4.00
PV-04	4.95	60.83	20.40	139.26	5.83	22.11	6.31	5.00
PV-05	4.20	49.29	10.20	73.61	3.26	11.07	3.15	
PV-06	3.30	45.89	7.20	53.43	2.44	8.28	2.27	
PV-07	3.10	45.89	7.20	53.43	2.44	8.74	2.31	
PV-09	1.80	45.21	6.60	49.32	2.26	7.23	2.05	
PV-11	0.96	44.53	6.00	45.18	2.09			
PV-12	0.70		0.30	2.87	0.17	4.47	1.23	
PV-13	0.60	39.78	1.80	14.93	0.76	2.24	0.58	
PV-14	0.24	38.42	0.60	5.43	0.30	0.78	0.20	
PV-21	42.00	221.38	45.60	783.69	27.59	144.23	46.78	45.00
PV-22	38.00	218.23	44.65	768.45	27.19	142.88	45.78	42.00
PV-23	20.50	136.89	20.14	365.65	15.48	64.64	20.63	10.00
PV-24	18.50	233.99	49.40	844.45	29.20	157.76	50.65	49.00
PV-25	16.00	177.25	32.30	568.12	21.62	101.86	33.14	26.00
PV-26	16.00	152.03	24.70	442.34	17.88	79.00	25.32	18.00
PV-27	16.00	141.31	21.47	388.13	16.19	68.18	22.02	15.00
PV-28	15.00	145.72	22.80	410.51	16.90	72.87	23.28	16.00
PV-29	13.50	128.70	17.67	323.64	14.11			9.00
PV-30	12.50	128.70	17.67	323.64	14.11	56.26	18.04	9.00
PV-31	10.50	117.35	14.25	264.79	12.12			6.00
PV-32	8.37	134.37	19.38	352.76	15.06	62.21	19.85	10.00

Table 43. Bass West Pasture peak rate evaluation results.

EVENT	PEAK	CYPRESS	CHART	OLDCREAMS	NEWCREAMS	UNIT-SCS	UNIT-MOD	WSHS
BW-01	32.00	66.23	70.60	425.25	25.29	68.49	21.45	39.00
BW-02	14.00	45.45	41.86	276.77	14.59	44.77	13.46	19.00
BW-03	14.00	40.00	35.10	250.43	12.84	35.56	11.14	14.00
BW-04	12.00	41.47	36.92	247.31	12.63	32.59	11.23	16.00
BW-05	11.00	33.81	27.43	189.50	8.98	27.21	8.85	11.00
BW-06	10.50	35.80	29.90	204.72	9.91	23.86	8.60	10.00
BW-07	8.50	33.60	27.17	187.89	8.88	28.54	8.76	10.00
BW-08	7.10	27.30	19.37	138.74	6.02	20.24	6.18	5.00
BW-09	7.00	25.41	17.03	153.67	6.87	18.11	5.46	4.00
BW-10	7.00	29.19	21.71	123.62	5.20			6.00
BW-11	6.30	34.12	27.82	191.91	9.13			9.00
BW-12	5.80	23.73	14.95	110.00	4.47	15.81	4.82	4.00
BW-13	5.00	22.16	13.00	97.05	3.81	13.72	4.19	3.00
BW-14	4.80	23.00	14.04	103.98	4.16	14.23	4.45	3.00
BW-15	3.00	17.12	6.76	62.32	2.16	6.04	2.22	2.00
BW-16	2.40	19.11	9.23	71.40	2.57	9.87	2.97	2.00
BW-17	2.30	16.91	6.50	52.15	1.72	6.04	2.11	2.00
BW-18	3.00	18.06	7.93		1.80	8.14	2.56	2.00
BW-19	1.70	15.33	4.55	37.88	1.14	4.28	1.47	
BW-20	1.50	14.39	3.38	29.02	0.81	3.06	1.13	
BW-21	1.40	14.60	3.64	31.02	0.88	3.27	1.21	
BW-22	1.10	13.45	2.21	19.83	0.50	2.23	0.76	
BW-23	0.64	12.40	0.91	8.95	0.18	1.06	0.32	

Table 44. Bass East Pasture peak rate evaluation results.

EVENT	PEAK	CYPRESS	CHART	OLDCREAMS	NEWCREAMS	UNIT-SCS	UNIT-MOD	WSHS
BE-01	1.90	11.24	15.95	94.88	2.87	2.59	1.99	13.00
BE-02	1.75	9.62	13.03	79.63	2.01			15.00
BE-03	0.95	6.33	7.10	47.09	0.69	1.30	0.97	6.00
BE-04	0.65	5.04	4.76	33.32	0.34	0.88	0.64	3.00
BE-05	0.61	4.86	4.45	31.42	0.31	0.81	0.62	2.00
BE-06	0.50	4.26	3.35	24.62	0.19	0.61	0.46	2.00
BE-07	0.49	3.16		11.31	0.04	0.25	0.19	
BE-08	0.46	5.71	5.97	40.53	0.51			
BE-10	0.31	3.78	2.50	19.07	0.11	0.45	0.34	1.00
BE-11	0.25	3.63	2.22	17.25	0.09	0.40	0.31	1.00

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