RICE UNIVERSITY

HYDROLOGIC SIMULATION OF STORMWATER DETENTION STORAGE IN AN URBANIZING FLOODPLAIN

by

DAVID PAXTON SMITH, JR.

A THESIS SUBMITTED
IN PARTIAL FULFILLMENT OF THE
REQUIREMENTS FOR THE DEGREE

MASTER OF SCIENCE

APPROVED, THESIS COMMITTEE:

Philip B Bedient
Dr. Philip B. Bedient
Chairman

William G. Characklis
Dr. William G. Characklis

Peter G. Rowe
Mr. Peter G. Rowe

HOUSTON, TEXAS

MAY, 1980

ABSTRACT

Emerging concepts of urban flood control consider the use of storage detention, especially where channel capacities are being overtaxed by urban runoff. Particular problems exist where high rainfall intensities and low topographic relief combine with rapid urban development to produce potential flooding. Traditional approaches to flood control emphasize channelization of main streams and laterals to speed urban runoff out of developed areas. However, in low relief areas where the effect of urban drainage may be to greatly increase the peak flow rate and decrease the time to peak, flood control solutions of the 1950's cannot handle the increasing development of the 1970's. This has been experienced in rapidly growing coastal cities such as Houston, Texas.

The purpose of the present study is to analyze the effect of detention storage placement and design on downstream flood flows in an urbanizing watershed. Effects of rainfall frequency, land use condition, and storage policy are directly considered in the methodology. The approach can be applied to any urban watershed in which historical rainfall data and stream-flow data as well as land use information is available. The U.S. Army Corps of Engineers HEC-1 Model forms the basic tool for analysis of flood flows. A storage detention model is used in conjunction with empirical unit hydrographs which are derived as functions of land use. Storage detention is tested in both existing urban areas as well as projected future developments to discover effects on flood frequency flows.

It is concluded that the ability to reduce the flooding potential of a rapidly urbanizing watershed with detention storage is limited by topography, remaining open space, and the presence of downstream development.

ACKNOWLEDGEMENTS

I would like to express my appreciation to my Committee Chairman, Dr. Philip Bedient, for his ideas, guidance, and especially for his understanding throughout my graduate career. I would also like to thank my other committee members, Dr. William Characklis and Mr. Peter Rowe for their criticisms and comments.

The assistance of the City of Houston Public Health Engineering Department and the U.S. Geological Survey, who furnished the basic data, is deeply appreciated. Espey, Huston and Associates, Inc., Dallas, Texas, also deserves my sincere gratitude for their help in typing and drafting all text and figures in this document.

Finally, I would like to thank my parents for their steadfast support and Ms. Susan Hoffman for her love and patience.

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

LIST OF FIGURES

LIST OF TABLES

1.0 INTRODUCTION

The traditional philosophy of urban drainage design is based upon the idea of collecting and discharging stormwater as quickly as possible to the downstream receiving system. However, as urbanization of a rural watershed takes place, increases in impervious surfaces and drainage densities result in decreased times of concentration and higher peak discharges. The additional runoff can overtax the existing drainage system and produce greater frequency floods in channels which can no longer carry the floods for which they were designed. The flooding problem is further compounded by the fact that existing downstream development lies in the path of an encroaching floodplain.

New methods of urban runoff control consider the temporary detention of stormwater as a possible alternative to transporting ever increasing peak flow rates. A common approach is that which permits "zero increases in stormwater runoff (Theil, 1977)." In practice, this goal is achieved by storing the higher rates of runoff so that the peak discharge does not exceed that of the undeveloped condition.

The effect of stormwater detention can be demonstrated by applying a dynamic hydrologic model to the watershed. Hydrologic modeling has utility not only in predicting the changing hydrologic response of an urbanizing basin but also in simulating the additional storage needed to offset the development process. Storage capacities that prevent downstream flooding can be determined for a given rainfall event.

The main objective of this research effort is to investigate the effect of rainfall frequency, detention policy, and development scenario upon the flows at the basin outlet in order to minimize the potential for flooding. The adopted procedure consists of applying suitable hydrologic models at both the watershed level and at the catchment (sub-watershed) level. The catchment level simulation consists of a detention pond which collects and stores runoff from developed areas. Storage in the catchments can then be compared to the storage lost during the urbanization process. The model chosen to simulate the overall watershed hydrology, HEC-1 (U.S. Army Corps of Engineers, 1973), requires a significant amount of calibration to historic rainfall and streamflow records. This calibration procedure results in estimated values for rainfall loss rates and flood routing coefficients. Chapter 3 discusses the methods utilized in simulating the hydrologic response of each catchment and in routing the flows through open channels and storage reservoirs.

Chapter 4 describes the application of this procedure to a test site, Brays Bayou, a rapidly developing watershed in southwest Houston. All pertinent physical data are presented including land use, topography, and hydraulic conditions.

The calibration results for flows at the watershed outlet and at two tributary gages are included in Chapter 5. Also presented in this section is a detailed analysis of the model's sensitivity to the adjustment of key input parameters.

Rainfall hyetographs for a range of return periods are required to design the optimal detention reservoir for a given storage criteria and land use. By examining feasible combinations of rainfall frequency and storage policy, a matrix of the necessary storage volumes to minimize downstream flooding can be obtained.

2.0 LITERATURE REVIEW

2.1 HYDROLOGIC RESPONSE

The effect of the process of urbanization upon the hydrologic character of natural watercourses has been the subject of extensive research during the past decade. As a result of the increases in artificially drained and impervious areas within a watershed, urban streams often carry quantitatively higher stormwater runoff volumes with substantially greater peak flowrates than in the undeveloped condition (Leopold, 1968; Tholin and Keifer, 1969; Van Sickle, 1969).

Although the increase in runoff volume diminishes for larger storm events, the major effect of development on urban hydrology is to accelerate the peak discharge. A review of several studies (Chow, 1976) confirmed that among the major effects of urban development upon the hydrologic response of a watershed were:

1. Increased runoff volumes
2. Decreased baseflows
3. Faster runoff responses
4. Higher peak flowrates

When a tributary draining an urbanized area flows into a river which drains a predominantly undeveloped area, the same review noted that:

1. Total runoff volumes and variations in magnitude and timing of flood peaks will in part be a function of the relative amount of urban area.
2. Stream hydrographs may become double-peaked due to the faster response of the developed areas.
3. Backwater effects may increase the flooding upstream of the confluence or even reverse the direction of flow of the river.

Urbanization occuring upstream of an already existing development has been shown to increase the frequency of flooding downstream by enlarging the extent of the floodplain (Van Sickle, 1969; Lovell and Smith, 1979).

It is generally believed that several hydrologic factors determine the duration and extent of storm hydrographs. These causative factors include:

1. Watershed size and configuration
2. Slope
3. Land use
4. Soil type
5. Impervious area
6. Drainage density
7. Available storage
8. Rainfall duration and intensity
9. Channel hydraulics

Variations in the runoff hydrograph based on various watershed parameters and land uses have been predicted using empirical relations. Snyder (1938) and Carter (1961) reported that the time-to-peak discharge (measured from the centroid of rainfall) is a function of total channel length, average channel slope, and the percentage of developed area. To account for variations in the shape of a watershed, many equations for time-to-peak include the length along the main channel to the centroid of the watershed as an additional factor (Johnson and Sayre, 1973; Rodman, 1977). More recently, the drainage density parameter has been shown to describe variations in time-to-peak due to increased urbanization (Halff et al., 1978) or agricultural drainage (Bedient et al., 1977).

2.2 SURFACE RUNOFF PREDICTION

In an effort to extend the usefulness of existing data as well as analyze existing relationships, stormwater simulation models have been developed and applied. A hydrologic segment is usually included in such models to solve the equations of flow and continuity. From functional relationships between watershed factors and runoff quantity, hydrographs can be predicted for any rainfall event.

Much of the early work in runoff modeling was prompted by information about the mechanism of soil infiltration (Horton, 1935; Holtan, 1961). Some of the earliest hydrologic models utilize concepts of soil storage and infiltration to predict groundwater flow, interflow, and surface runoff (James, 1965; Crawford and Linsley, 1966). With the advent of digital computer models, a continuous computation of the water balance is possible if all of the physical inputs and system dynamics are known. The best documented of the available techniques include the Environmental Protection Agency Storm Water Management Model (EPA, 1971), the Corps of Engineers STORM (1976) and HEC-1 (1973) models, and the Soil Conservation Service TR-20 (1965) program.

The SWMM is a highly sophisticated computer simulation program capable of predicting continuous hydrographs and pollutographs for various locations in the drainage network. Storm runoff, dry weather flow, and infiltration are generated as functions of input data such as land use, antecedent conditions, and topography for a given rainfall event. Available runoff is routed through overland and channel flow using the kinematic wave method calculated for each step in time and location.

The SWMM has been tested by many research groups and found to be relatively accurate and helpful in the analysis and control of urban storm runoff (Diniz and Characklis, 1976). However, the model suffers a disadvantage of not being a continuous simulation of both storm and low flow records, but is limited to the simulation of single runoff events. The stability requirement for small time steps places a limitation on the simulation of long-term seasonal effects and storm spacing.

Both STORM and TR-20 utilize the Soil Conservation Service curve number technique to calculate the volume of runoff as a function of rainfall volume and curve number. Land use, soil type, and antecedent conditions are all defined by a single curve number. Although less sophisticated than SWMM, STORM and TR-20 require little field data and are reasonably accurate for catchment sizes as large as 25 square miles.

The main disadvantages of the STORM model are the one-hour time step limitation and the lack of a channel or reservoir routing procedure. Peak flowrates with response times less than one hour are underestimated by STORM; therefore, the model's accuracy is diminished when used in small, heavily developed catchments. TR-20, however, does provide for channel routing using the convex method and has a variable time step capability.

HEC-1 was developed in 1958 for use by the Corps of Engineers, initially for use in large rural watersheds. It uses the Snyder unit hydrograph procedure (Snyder, 1938) to calculate flows for single storm events. Losses are calculated by abstracting an initial loss and a continuous loss for each catchment. The remaining water then flows into the receiving stream and is routed to the watershed outlet.

HEC-1 is an empirical model which requires catchments small enough (10 square miles or less) so that the unit hydrograph approximation is valid. But the differences between such a simplistic model and more conceptual models at the large watershed level are small (U.S. Corps of Engineers, 1960). HEC-1 has been utilized at Rice University for the Brays Bayou Study funded by the City of Houston Public Health Engineering Department (Bedient, et al., 1978). Intended as a stormwater monitoring and modeling effort, HEC-1 was chosen because of the minimal amount of watershed data required to obtain accurate and consistent results for all types of land use. Calibration using observed and computed hydrographs yield superior results considering the low costs and wide applicability of the program to urban planning and flood control. Many storage and channel routing methods are available as well as a variable computation interval. Key parameters can be optimized using available rainfall and streamflow records.

2.3 STORMWATER MANAGEMENT

In an attempt to replace the natural storage of runoff which is lost when development occurs, man-made storage can be provided in urban watersheds. As early as 1960, local governments began to adopt policies requiring no increases in runoff rates as a result of development (Kamedulski and McCuen, 1979; Erie, 1979). A proper balance between channel transport and "on-site" storage is recommended when the effects of urbanization within a watershed are to be controlled. Channel storage can be substantial depending upon the condition of the watercourse; this fact should not be overlooked when one considers the additional storage necessary in a basin.

Temporary storage of runoff is a very innovative concept; consequently, there exist a multitude of different techniques depending upon the scope of the flooding problem (SCS, 1975). Rooftop storage for large buildings is an inexpensive non-structural solution if the site is designed to carry an equivalent snow load. The collection of rainwater by cisterns is an ancient practice which serves as an alternative to rooftop storage at the residential level. Swale drainage, the use of greenbelts, and other means of decreasing overland flow velocities represent secondary methods of stormwater management when combined with on-site measures such as parking lot or rooftop detention.

Structural solutions such as man-made detention ponds require the use of large amounts of land which limits their applicability in totally developed areas to already existing flood control easements. However, analysis of single detention reservoirs show that they effectively reduce and retard the peak discharge and can be implemented throughout a developing watershed (Rice, 1971; Poertner, 1974).

Several authors have investigated the proper sizing and location of detention reservoirs. As the number of such reservoirs increase, the possibility of localized flooding becomes greater due to backwater effects (Lumb, 1974). The storage necessary for an individual detention basin increases with the distance downstream (Abt and Grigg, 1978). Erie (1979) compares the effect of several storage basin sizes and storm frequencies upon the reductions of peak discharge for the outflow hydrograph.

In addition to many graphical and empirical techniques for the design of detention basins, digital computer solutions to the equations of continuity and outlet discharge have been obtained (Curtis and McCuen, 1977; Amandes and Bedient, 1980).

Not only are the peak flowrates downstream from a developed site decreased by storage, but they are also delayed considerably. In downstream catchments of a developing watershed; however, runoff should not be detained in order to preserve channel capacity for the delayed flows from upstream catchments (Mills, 1977).

3.0 METHODOLOGY

3.1 OVERVIEW

The overall method for this research consists of first generating the runoff response of the catchments using existing topography, drainage patterns, and land use. Next, the required detention storage is calculated for a typical developed site. Finally, the combined effects of present and future development, detention policy, and storm frequency are evaluated with respect to the possibility of flooding at the outlet of the watershed.

HEC-1 simulates the hydrologic response from each catchment using unit hydrograph theory. Regression equations based upon regional data are used to estimate the peak unit discharge and the time-to-peak as functions of the length and slope of the main channel, the drainage area, and the percentage of impervious area. These unit hydrograph parameters are then compared with those obtained from calibration events and are adjusted accordingly.

Rainfall loss rates are assumed to be constant throughout the storm after an initial loss has been satisfied. Values for loss rates are obtained by calculating the infiltration indices for observed storms (Cook, 1946).

The model routes the computed runoff hydrograph downstream using the Muskingum method. This technique assumes channel storage to be a weighted function of both the inflow and outflow to a reach. A comparison with the exact solution of the equations of continuity and momemtum for a flood wave verifies the accuracy of the Muskingum routing method.

Hydrographs from a typical 200-acre development are routed through a hypothetical detention reservoir for several storm frequencies. The required volume of storage is the value that results in no increase from the undeveloped peak flowrate for a particular design storm. Storage vs. peak discharge relationships for a given detention policy are then applied in an additive manner throughout the catchment in order to simulate the effect of stormwater management upon both existing and future development. Finally, the value of man-made storage as a substitute for natural storage is determined by noting the differences in flows at sites within the watershed and at the outlet.

3.2 UNIT HYDROGRAPH DEVELOPMENT

A unit hydrograph describes the runoff response from a specific catchment due to one inch of excess rainfall. Moreover, excess rainfall is defined as that portion of the total precipitation which results in direct surface runoff. The unit hydrograph must define not only the baseline hydrology of the catchment but also that which occurs following urbanization.

Figure 1 shows two unit hydrographs and their associated parameters. Runoff from the undeveloped condition is more evenly distributed than that from the developed state. Leopold (1968) stated that peak unit discharges may increase from two to five times the amount which occurs prior to urbanization. The time-to-peak is reduced significantly by development as

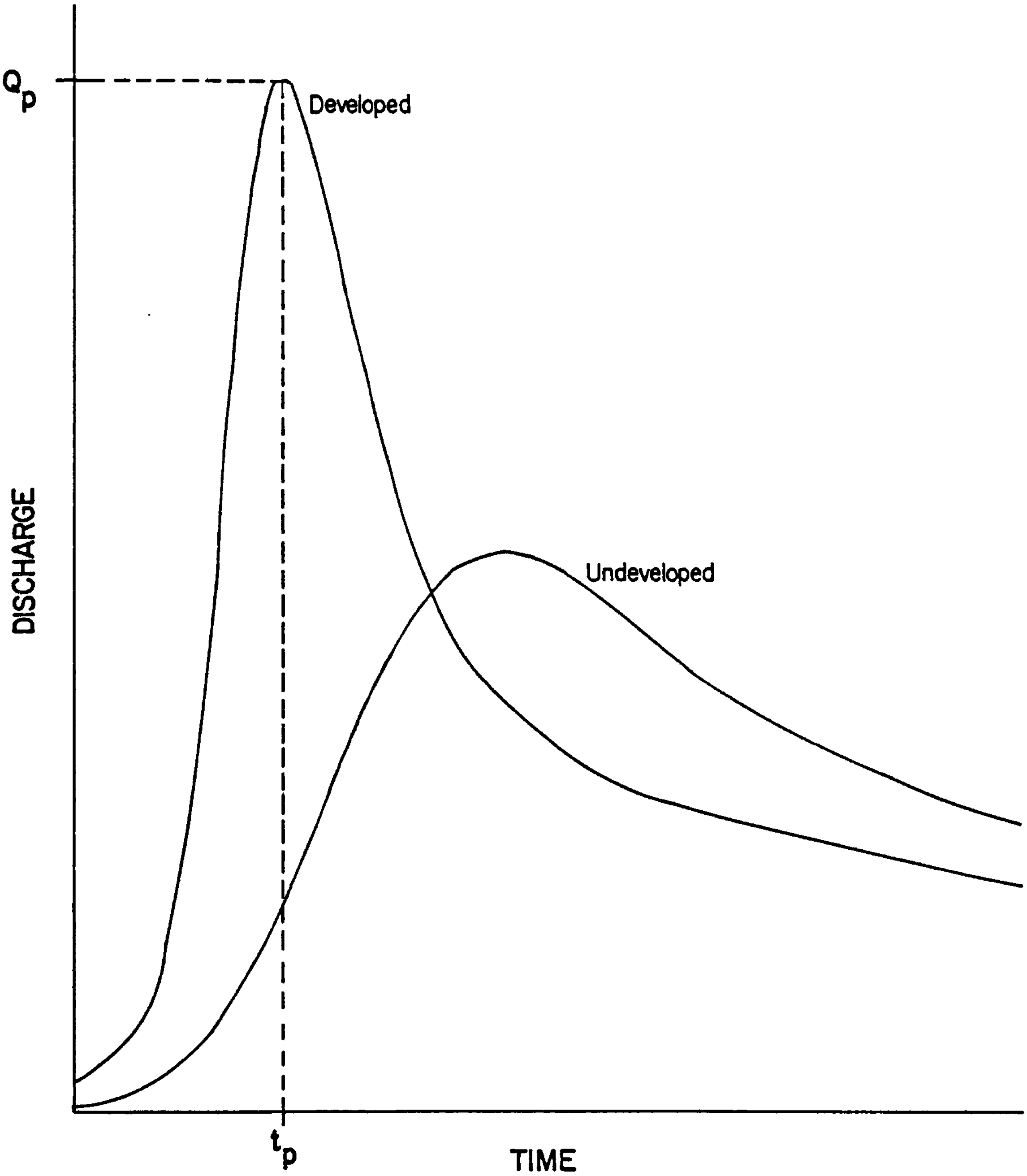


Figure 1

EFFECT OF URBAN DEVELOPMENT UPON UNIT HYDROGRAPH PARAMETERS more efficient channels are created to transport runoff. Thus, the timing of the unit response is related to the length and slope of the major drainage channel and the extent of impervious surfaces within a catchment.

Espey and Winslow (1968) studied hydrologic data from 22 gaged watersheds in the Houston area of varying soil type with drainage areas ranging from 0.5 to 213 square miles and impervious areas varying from 0 to 35 percent. Empirical equations from their work for the unit hydrograph parameters of interest to this research were determined to be:

$$t_p = (16.4\,\Phi\,L^{0.316}\,S^{-0.0488}\,I^{-0.49}) - 15 \quad (1)$$

$$Q_p = 3.54 \times 10^4\,(t_p + 15)^{-1.10}\,A \quad (2)$$

where t_p is the time-to-peak (minutes) of the 30-minute duration unit hydrograph, Φ is a dimensionless conveyance factor, L is the length of the main channel (feet), S is the unit slope of the main channel, I is the percentage of impervious area, Q_p is the peak unit discharge (cubic feet per second), and A is the drainage area (square miles). The conveyance factor, Φ, can be attributed to the hydraulic resistance of the channel as shown in Table 1.

Therefore, measuring L, S, A, and I and choosing an appropriate value for Φ allows one to estimate the unit hydrograph of an ungaged catchment for a 30-minute duration of one inch of rainfall excess. Gaged tributaries, however, provide the means for calibrating these equations with recorded data by adjusting the value of Φ. Chapter 5 describes the method used in determining Φ for tributaries within the test watershed, Brays Bayou.

TABLE 1

Φ CLASSIFICATION

	Φ_1 Classification
0.6	Extensive channel improvement and storm sewer system, closed conduit channel system.
0.8	Some channel improvement and storm sewers; mainly cleaning and enlargement of existing channel.
1.0	Natural channel conditions.

	Φ_2 Classification
0.0	No channel vegetation.
0.1	Light channel vegetation.
0.2	Moderate channel vegetation.
0.3	Heavy channel vegetation.

$$\Phi_1 + \Phi_2 = \Phi$$

Source: Espey and Winslow, 1968.

The model chosen to predict the runoff hydrograph for the entire watershed, HEC-1, is a digital model which performs all calculations using a specified computational time step. Due to the duration of the Espey and Winslow unit hydrograph and the response time of Brays Bayou, a 30-minute time step is utilized in all simulations.

Rainfall increments are also supplied to the model at 30-minute intervals. Excess rainfall for each time period is computed by subtracting a constant loss rate after an initial abstraction is satisfied. Constant loss is analogous to infiltration, while initial loss corresponds to interception and depression storage.

Once the rainfall excess is computed for each time step, the values are convoluted with the unit hydrograph ordinants to produce a composite hydrograph. Additional requirements for calculating surface runoff include the shape of the hydrograph recession curve, the flowrate at which direct runoff ceases and the recession begins, and the initial baseflow at the start of the simulation. The direct runoff hydrograph is superimposed upon the baseflow to produce the total storm hydrograph.

The relative simplicity of the model causes several limitations to this approach. First, equations (1) and (2) are derived from regression analysis; hence, they are only the equations of best fit and thus contain a small amount of error. Table 2 lists the multivariate correlation coefficients and the standard errors of estimate for the equations for t_p and Q_p. However, calibration of these equations to fit historical streamflow records minimizes

TABLE 2

STATISTICAL PARAMETERS FOR UNIT HYDROGRAPH EQUATIONS

Equation	Dependent Variable	r	S_e	D (%)
(1)	Time to peak, t_p	.901	101 minutes	61
(2)	Peak unit discharge, Q_p	.907	154 cfs	70

r: is the correlation coefficient

S_e: is the standard error of estimate,

D: is the percentage of the data for which the value predicted by the equation is within $\pm$ 30 percent of the actual value.

Source: Espey and Winslow, 1968.

the error included in the regression technique and, in effect, regionalizes the model.

Secondly, the unit hydrograph technique is only valid for drainage areas on the order of 1-10 square miles and for rainfall intensities of 2-3 inches per hour or less. The design rainfall developed in Appendix A exceeds 3 inches per hour at the peak rate of rainfall; therefore, some inaccuracy exists in the peak discharge values for the catchments in the model.

Finally, it is assumed that a runoff is able to reach the main channel and that backwater in the secondary network is minimal. This assumption fails if the downstream areas do not drain quickly enough before the upstream peak arrives. However, the effect of storage upon the relative decrease in peak flows at the outlet can be determined in spite of these limitations. Runoff either reaches the drainage network or else ponds in the streets and houses because the secondary network becomes surcharged. A decrease in the peak discharge of the main channel will mitigate this surcharge condition.

3.3 HYDROGRAPH ROUTING

3.3.1 Overview

Channel routing may be considered as the propagation of a flood wave through the channel reach. Depending upon the dimensions and flow resistance of the channel cross-section, a substantial amount of storage occurs between the beginning and the end of the stream segment. In some cases this storage effect is great enough to eliminate the need for on-site detention or other stormwater controls. Natural channels with unrestricted floodplains provide a good example of channel storage. Both the travel time and the attenuation of the flood wave are increased by additional channel storage. In order to calculate the effect of flood routing upon the inflow hydrograph, both exact and approximate numerical methods are utilized. Hydraulic routing methods solve the equations of continuity and momentum while hydrologic methods assume a storage vs. discharge relationship and solve only the equation of continuity.

3.3.2 Method of Characteristics

The method of characteristics (Harbaugh, 1967) simultaneously solves continuity and momentum equations (3) and (4) at discrete intervals in time and in distance along the reach.

$$y \frac{\partial V}{\partial x} + V \frac{\partial y}{\partial x} + \frac{\partial y}{\partial t} = 0 \qquad (3)$$

$$\frac{\partial V}{\partial t} + V \frac{\partial V}{\partial x} + g \frac{\partial y}{\partial x} + g(S-S_f) = 0 \qquad (4)$$

In the previous equations, y is the depth of the flood wave, V is the average flow velocity, x is the distance along the reach, t is time, g is gravitational acceleration, S is the channel slope, S_f is the slope of the water surface and ∂ is the partial derivative operator. However, the computations must occur at values of ∂x and ∂t small enough to avoid unstable solutions. When considering high peak flowrates, short response times, and large frictional losses, the computational elements become exceedingly small resulting in solutions which are difficult to calculate even by digital computer.

3.3.3 Muskingum Approximation

An HEC-1 subroutine optimizes the values for the Muskingum hydrologic routing parameters given the inflow hydrograph and the method of characteristics outflow. The Muskingum routing method (McCarthy, 1938) consists of equating storage with the inflow and outflow according to:

$$S = K (X I + (1-X) O) \tag{5}$$

where S is the average storage within the reach, I and O are the inflow and outflow, respectively, K is an assumed travel time, and X is a dimensionless storage coefficient. The storage is computed for every time step using the known inflow and the outflow calculated at the end of the previous time step.

Although the Muskingum method is only a linear approximation of a non-linear flood routing process, optimized values of K and X result in routed hydrographs which vary little from the exact solution with respect to peak discharge and travel time. Chapter 5 presents the results of this comparison along with the K and X values chosen for the HEC-1 model.

3.4 DETENTION STORAGE DESIGN

3.4.1 On-Site Level

On-site storage of runoff is only one of many techniques available for reducing the effect of urbanization upon the hydrologic response of a catchment. The advantage of simulating detention storage lies in the fact that the results can be compared directly between different catchments. Graphical and empirical methods exist to design on-site detention facilities; however, the accuracy of these methods limits their usefulness to small drainage areas. Numerical techniques involving the use of digital computers have both a wider range of applicability and less uncertainity in the results.

A reservoir storage model developed by Amandes and Bedient (1980) simulates the effect of localized runoff detention at the on-site level. 200 acres represents the size of the unit development drained by a single detention pond. The model assumes a 3-foot deep basin with an outlet structure consisting of a 2-foot diameter pipe and a 50-foot emergency spillway. The 3-foot depth represents an upper limit for impoundments in the Houston area using gravity drainage, due to the flat topography. Discharge is calculated by Manning's equation for the pipe and by an equation for broad crested weirs for the spillway (Brater and King, 1963).

Reservoir routing is calculated using a first-order Runge-Kutta technique and a one-minute time step to reduce the error in integration. To design the reservoir, the model successively increases the surface area of the basin until a given design peak outflow is attained. In this manner, the storage necessary to reduce the peak discharge to the undeveloped level can be determined. Moreover, the effect of varying the design storage or the design frequency storm is revealed by applying the storage model.

Rainfall for the 5-, 10-, 25-, and 100-year events are obtained from U.S. Weather Service data for the 24-hour duration storm (Hershfield, 1961). The time-of-concentrations for both undeveloped and developed

200-acre parcels are proposed based upon typical subdivisions within the Brays Bayou watershed. Using the SCS curve number method, design hydrographs for both development conditions and for all frequency storms are produced which serve as the inflow to the detention reservoir model (see Appendix A).

3.4.2 Catchment Level

Future development within each catchment is considered as a multiple of the 200-acre unit size. Once the storage vs. discharge relationship for a single reservoir of a given design storage is known, then the total storage vs. discharge curve for the entire watershed is simply that same multiple times the unit curve. This technique is applied to simulate the overall effect of detention storage using the Modified Puls storage routing subroutine contained in HEC-1. For a given level of development and design storage policy, HEC-1 requires a unique storage vs. discharge curve. The new development is treated as a distinct catchment, and is separated from the remaining area of the original catchment.

Levels of development considered include present land use and both 1995 and projected ultimate land use. Future conditions are predicted based upon a Rice University School of Architecture investigation (1978) which predicts population gains and probable land uses in the upper Brays Bayou watershed.

An effort is made to separate the developed contribution from the undeveloped portion of the catchment runoff. The SCS 200-acre unit hydrograph is lagged until the peak of the resulting hydrograph coincides with the occurrence of the original peak. The lagged hydrograph is justified by the assumption that development within a catchment determines the overall time to peak. Subtraction of the lagged SCS curve from the Espey and Winslow curve produces the undeveloped contribution.

A sample calcuclation using the composite hydrograph method is shown in Table 3 for catchment 6. There currently exists 2.9 square miles of urban development within this catchment with 2.5 square miles remaining. Therefore, the SCS-derived unit hydrograph corresponding to 2.9 square miles is lagged by an additional 1.5 hours to produce a t_p equal to 2.5 hours. This coincides with the Espey and Winslow t_p of 2.63 hours. The positive difference between the Espey and Winslow unit hydrograph and the SCS unit hydrograph equals the estimated undeveloped contribution. Negative differences are ignored. This example shows that 92 percent of the total catchment runoff volume originates from 54 percent of the total catchment area.

TABLE 3

SAMPLE CALCULATIONS

COMPOSITE UNIT HYDROGRAPH METHOD FOR CATCHMENT 6

Unit Hydrograph	Drainage Area (mi^2)	Rainfall (in)[a]	Runoff Volume (in)	Runoff Volume (ac-ft)[b]	Q_p (cfs)	Peak Discharge (cfs)[c]
Espey and Winslow	5.4	8.40	4.20	1,210	660	2,770
SCS	2.9	8.40	7.20	1,110	421	3,030
Undeveloped	2.5	8.40	0.75	100	d	960

a 10-year, 24-hour rainfall

b runoff volume (ac-ft) = runoff volume (in) x drainage area (mi^2) x 53.3

c peak discharge = Q_p x drainage area

d not calculated

4.0 APPLICATION TO BRAYS BAYOU

4.1 OVERVIEW

The Brays Bayou Watershed provides an interesting test case for this study. The upper half of the basin is undergoing very rapid urbanization, mostly in the form of residential subdivisions. On the other hand, the lower half is essentially fully developed. In fact, existing development such as the multi-billion dollar Texas Medical Center is located partially within the FIA 100-year floodplain for Brays Bayou. Residents in certain areas of the lower basin routinely experience backwater conditions caused by bankfull flows in Brays Bayou; consequently, streets, basements, and other low-lying areas are inundated. The June 15, 1976 flood caused over twenty million dollars in damages and is included as a calibration event in this research.

Another reason for studying Brays Bayou is the abundance of hydrologic, hydraulic, topographic, and land use data that is readily available. Rainfall is gaged by the U.S. Weather Service at up to eight recording stations and several other total rainfall stations, both within or adjacent to the Brays Bayou watershed. Streamflow is gaged by U.S. Geological Survey (USGS) not only at the watershed outlet (Brays Bayou at South Main Street) but also at two tributary locations (Keegans Bayou at Roark Road and Bintliff Ditch at Bissonnet Street). Twelve years of flood hydrograph records exist for all three gages (USGS, 1977). A 1968 drainage study (Turner, Collie, and Braden, Inc., 1968) serves as the source for all topographic data and channel specifications within the basin. Aerial photographs at a scale of 1:12,000 taken in January, 1978 are used to verify and update information obtained from the drainage study. Finally, accurate land use information is available at Rice University (Bedient et al., 1978) which categorizes the current development within the watershed. Future growth until 1995 is predicted by a land use allocation model. Each source of data contributes to the reliability of the modeling effort.

4.2 SITE DESCRIPTION

Located approximately 50 miles from the Gulf of Mexico, the topography of the basin is extremely flat and can be described as a coastal prairie. With an average watershed slope of only 0.04 percent, hydraulic conditions are sluggish without artificial drainage. The annual average rainfall of 43 inches occurs as relatively intense thunderstorms during the summer, while drizzle patterns dominate during the winter (U.S. Department of Commerce, 1973). Thus, the combination of the wet climate and the flat terrain necessitates the construction of extensive storm sewer networks wherever development occurs.

Figure 2 shows the location of the study site in relation to the Houston metropolitan area. Brays Bayou, like most other streams in this area, flows from west to east; thus, new developments contribute runoff upstream of the older urban center. The 86.8-square mile study area is gaged at South Main Street, located far enough upstream of the Houston Ship Channel to avoid any tidal influences. Channel capacity at the South Main Street site is approximately 29,000 cfs.

The Brays Bayou Watershed is shown in greater detail in Figure 3. For modeling purposes, the basin is divided into eleven separate catchments

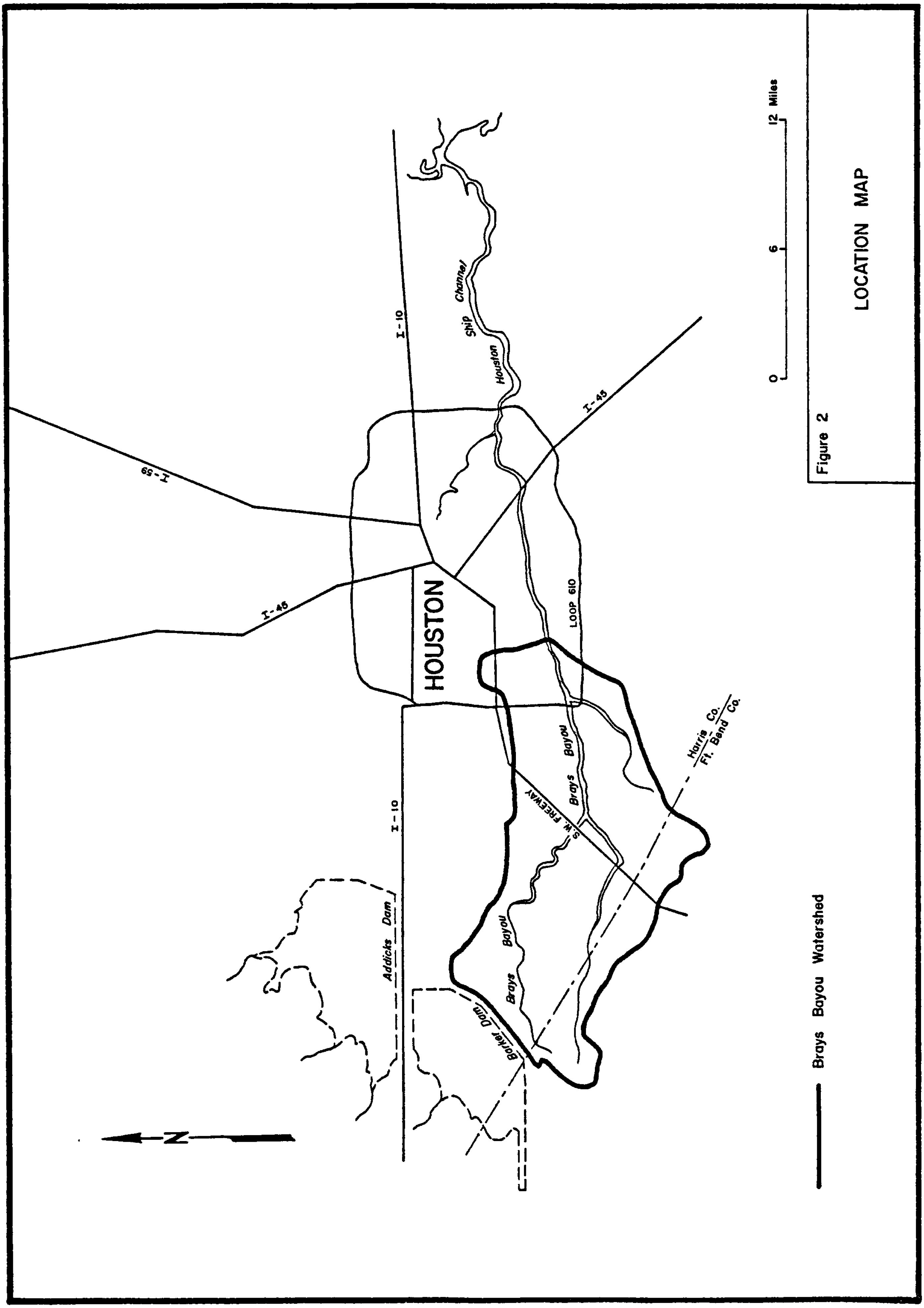


Figure 2 LOCATION MAP

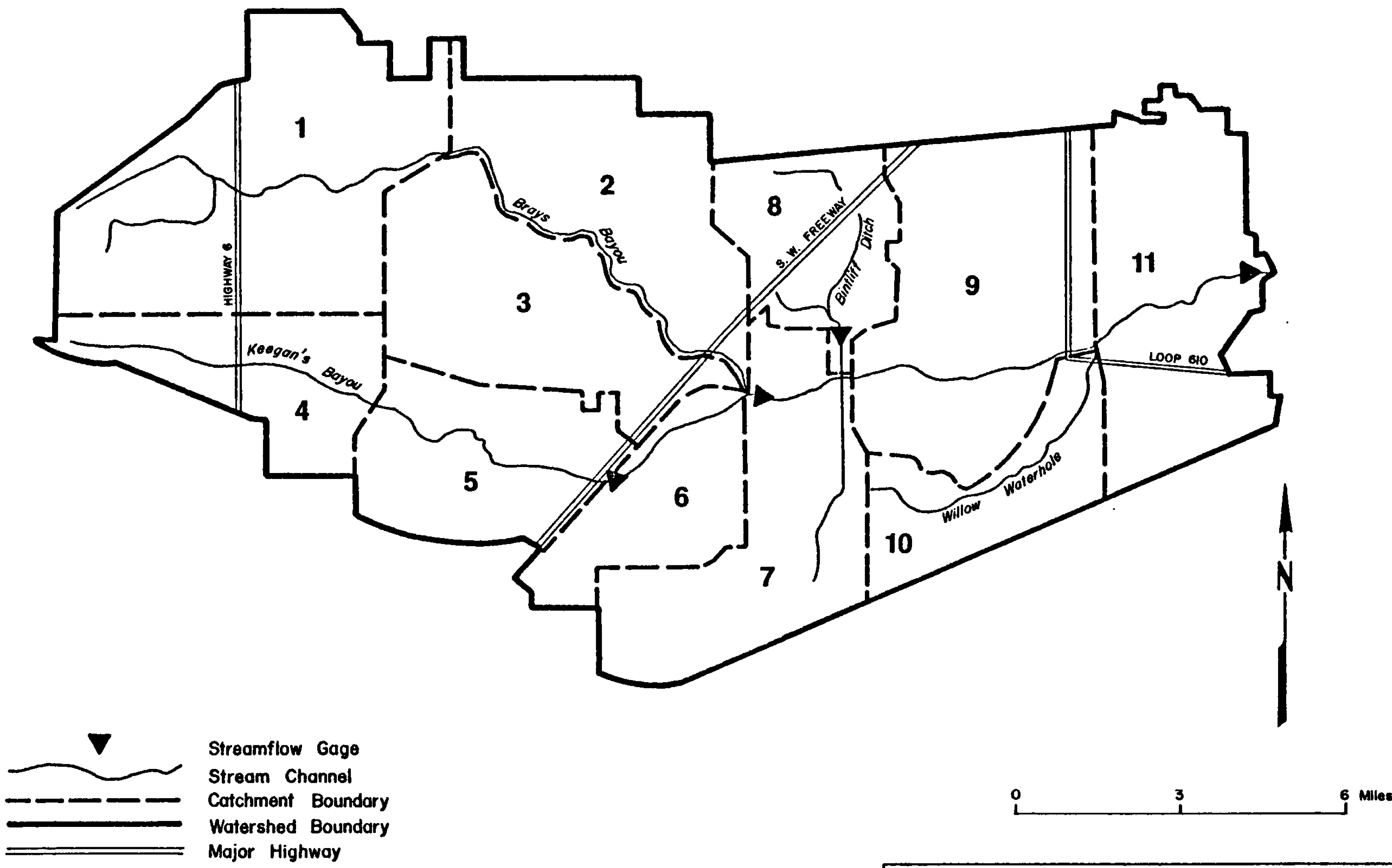


Figure 3

STREAMFLOW GAGES AND CATCHMENTS
WITHIN BRAYS BAYOU WATERSHEDS

which have fairly homogenous land uses and drainage patterns. Catchments 1-6 are developing from open pasture into residential and commercial land uses. Catchments 7-11, however, are considered to be fully urbanized; hence, stormwater control measures are not considered in these lower catchments. Almost 8 miles of Brays Bayou up to the Southwest Freeway (U.S. 59) bridge were channelized by the Corps of Engineers in the early 1950's to accommodate the 100-year flood. The remaining channel length at present is rectified but not concrete-lined.

A land use inventory adopted for catchments 7-11 is presented in Table 4. The percentage of impervious area for each land use type was derived from an earlier study (Characklis, et al., 1976). Land use information for catchments 1-6 is based on 1978 conditions while data for catchments 7-11 originate from 1975 Houston census figures. This discrepancy should not effect the validity of the model since the lower catchments have been relatively stable over this period of time.

TABLE 4

LAND USE INVENTORY

Subwatershed (% Imp.)	Streams	Single-Family Dwelling	Multi-Family Dwelling	Commercial	Industrial	Educational	Undeveloped	Total Acres	Total Square Miles
		(46%)	(50%)	(95%)	(95%)	(50%)	(5%)		
1 (8.3)	Brays	255	4	80	129	6	8,543	9,017	14.09
2 (22.5)	Brays	2,510	602	566	524	286	9,297	13,785	21.54
3 (6.8)	Keegans	101	--	8	26	--	3,933	4,068	6.36
4 (18.2)	Keegans	1,006	28	34	47	22	2,703	3,840	6.00
5 (21.1)	Keegans	718	15	75	210	--	2,439	3,457	5.40
6 (37.4)	Brays	955	66	87	4	46	458	1,616	2.53
7 (43.6)	Bintliff	950	220	444	161	128	921	2,824	4.41
8 (47.5)	Brays	2,777	395	829	146	111	1,021	5,281	8.25
9 (19.3)	Willow Waterhole	987	21	73	188	8	3,281	4,558	7.12
10 (36.9)	Willow Waterhole	911	84	157	84	33	749	2,018	3.15
11 (43.4)	Brays	3,000	323	506	114	36	1,103	5,082	7.94
TOTAL		14,170	1,758	2,859	1,673	676	34,448	55,544	86.79
PERCENT OF TOTAL		25.5	3.2	5.2	2.9	1.2	62.0	100.0	

Source: Bedient, et al., 1978.

5.0 HYDROLOGIC MODEL CALIBRATION

5.1 OVERVIEW

In order to simulate the existing hydrology of the Brays Bayou Watershed two recent flood events are chosen to calibrate the HEC-1 model. A flow chart in Figure 4 depicts the process followed in fitting the model.

Precipitation patterns for each catchment are derived from a Thiessen grid of the eight recording gages. Total storm precipitation is determined by also considering the non-recording gages in the grid network.

Rainfall loss rates used in the model are constant for each catchment. Values for these loss rates are extrapolated from data obtained from the Keegans Bayou and Bintliff Ditch gages (see Figure 3). The infiltration index (obtained by comparing rainfall and streamflow records) equals the loss rate for a catchment. A constant loss is subtracted from the rainfall increment to yield precipitation excess, or runoff, during that particular increment.

Unit hydrographs are developed for every catchment based upon equations by Espey and Winslow. Physiographic data are related by equations (1) and (2) to the shape of the unit hydrograph. After comparing the predicted and observed flows for the two tributary gages, the variable in equation (1) is adjusted to compensate for specific hydraulic conditions found in Brays Bayou and its tributaries.

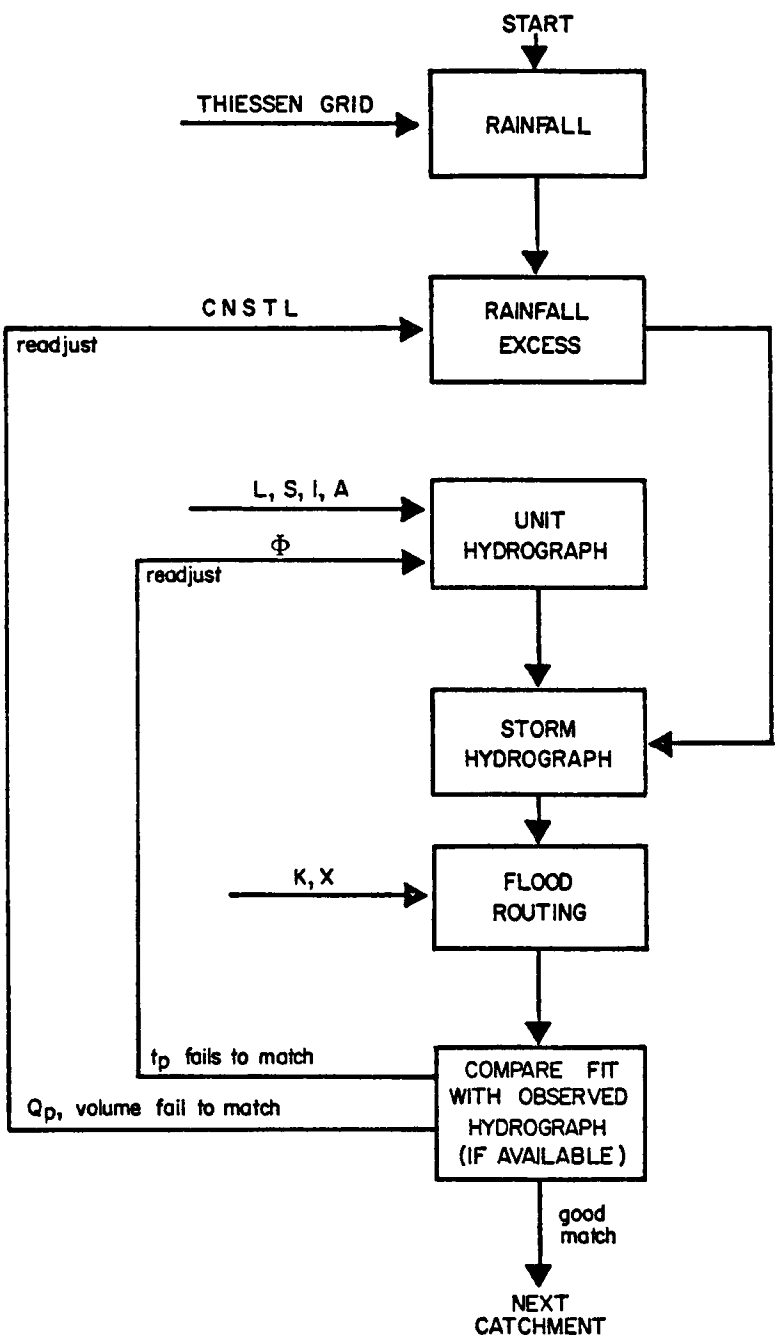


Figure 4

FLOW CHART FOR
HEC-I MODEL CALIBRATION

Observed and predicted streamflow hydrographs are calibrated not only at the basin outlet but also at the two tributary gages. Loss rate values are altered on an individual basis until the best match of all three hydrographs is achieved.

HEC-1 utilizes the Muskingum technique to route floods through the channel reaches. The method of characteristics solution of the equations of motion provides the exact solution to the flood routing problem. An HEC-1 subroutine then optimizes the Muskingum routing variables to reproduce the exact solution. Included in this chapter is a discussion of the accuracy of the Muskingum method.

Finally, the detention storage model is applied to the typical 200-acre unit development. Runoff hydrographs obtained by the SCS method (see Appendix A) are routed through storage basins so that the developed peak flow rate does not exceed the undeveloped value. Two sizes of detention basis are selected: the 5-year design and the 10-year design. The storage effectiveness of each size of detention pond is characterized as a function of storm frequency.

5.2 RAINFALL EVENTS

Two historical storm events are chosen for calibration purposes: the June 9, 1975, and the June 15, 1976 event. Each flood is the largest for that particular water-year and each storm is documented by superior rainfall and streamflow records (USGS,1979). Unfortunately, each storm is also heaviest over the lower basin and tends to bias the model to some degree. However, earlier storm events of sufficient size to merit study do not reflect the current (1978) land use in the watershed.

Tables 5 and 6 present total gaged rainfall and the total catchment rainfall as a weighted average of the adjacent gaged values, respectively. These weighted averages are the result of a Thiessen polygon grid constructed among the rainfall gages. Figure 5 shows the grid network for the June 15, 1976 event. Both total rainfall and rainfall intensity are averaged using the Thiessen grid.

5.3 RUNOFF DETERMINATION

A constant infiltration rate is chosen to separate the surface runoff from the precipitation for each increment of time. No runoff is produced until after an initial storage (STRTL) has been satisfied. The value for the constant loss rate (CNSTL) is derived from the infiltration index for either the Keegans Bayou or Bintliff Ditch site. Values for STRTL are estimated by the assumption that undeveloped areas retain 1/4-inch while developed areas store only 1/8-inch of rainfall. Initial estimates for CNSTL are adjusted so that observed and predicted runoff volumes are in agreement for the two gaged catchments.

The calculated precipitation excess is distributed over the storm duration using unit hydrograph equations (1) and (2). Table 7 lists the physiographic variables necessary to determine t_p and Q_p. A value of Φ, the

TABLE 5

TOTAL RAINFALL IN INCHES RECORDED BY USGS GAGES IN THE VICINITY OF BRAYS BAYOU WATERSHED

Gage No.*	Calibration Storm	
	June 9, 1975	June 15, 1976
0-S	5.20	N
5-S	2.50	N
31-R	4.00	N
32-R	2.50	2.88
34-S	1.30	N
35-S	8.00	10.12
39-R	1.10	5.51
301-R	2.60	N
303-R	N	3.90
304-R	N	7.20
308-R	N	10.47
4780-R	N	3.50
4800-R	4.13	2.82
4950-R	N	3.42

* R: recording
S: non-recording
N: no record

TABLE 6

TOTAL RAINFALL IN INCHES APPLIED TO CATCHMENTS WITHIN BRAYS BAYOU WATERSHED

	Calibration Storm	
Catchment	June 9, 1975	June 15, 1976
1	2.04	3.29
2	2.48	3.36
3	3.15	3.07
4	2.36	3.81
5	3.51	3.44
6	4.10	2.72
7	6.00	6.20
8	2.50	4.85
9	5.80	8.93
10	8.00	10.12
11	5.20	10.47

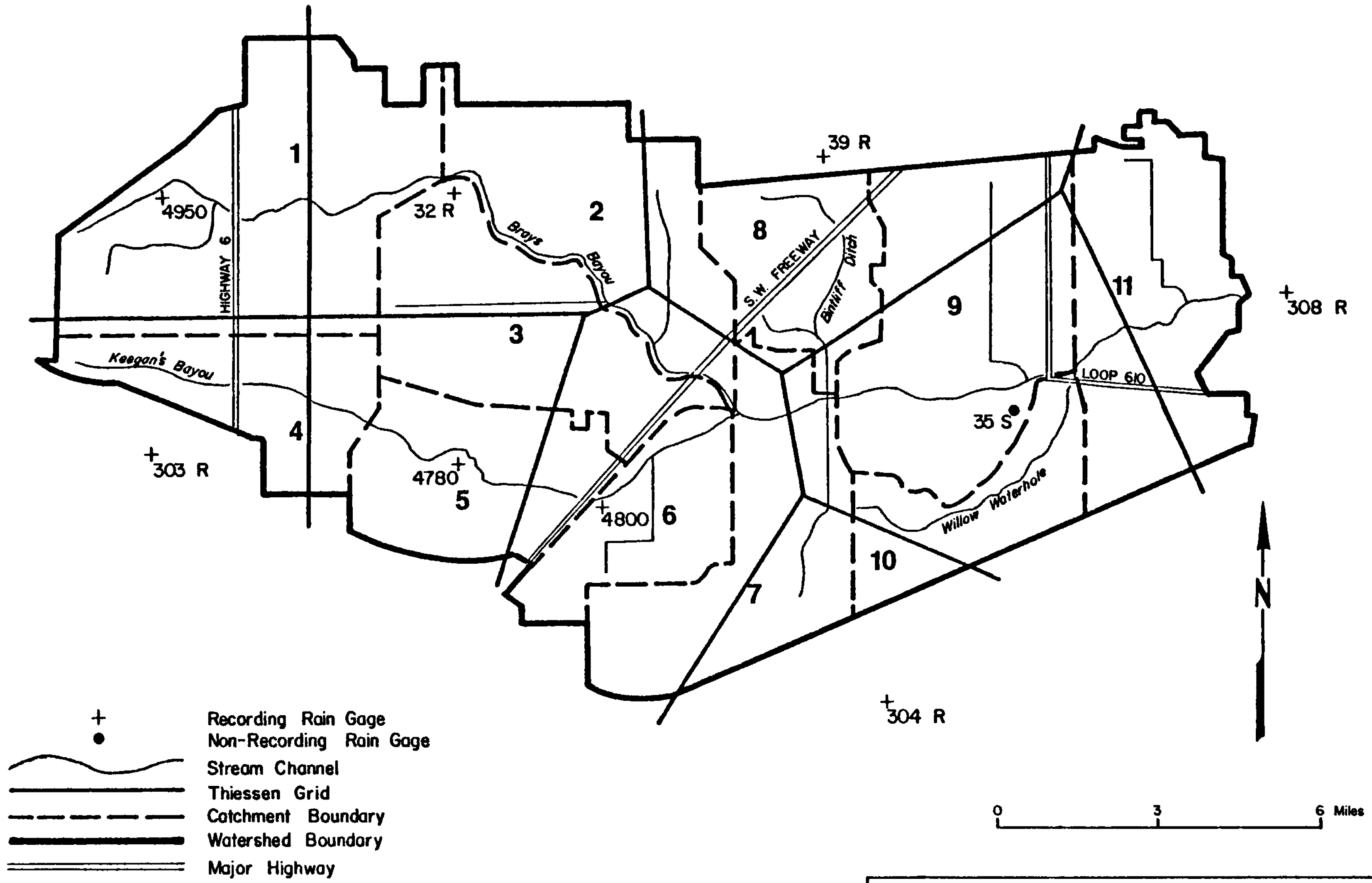


Figure 5

THIESSEN GRID NETWORK FOR THE JUNE 15, 1976 EVENT

TABLE 7

PHYSIOGRAPHIC PARAMETERS UTILIZED IN UNIT HYDROGRAPH DETERMINATIONS

Catchment	Area (mi^2)	L (ft x 10^4)	S (x 10^{-4})	I (%)	Φ
1	13.4	3.05	8.1	5.9	1.33
2	10.9	2.24	9.8	21.0	0.97
3	11.3	3.88	7.5	15.2	1.33
4	6.4	2.40	2.5	3.3	1.80
5	6.0	2.05	8.3	18.1	1.33
6	5.4	2.66	6.0	21.8	1.33
7	8.2	2.06	8.4	43.6	0.85
8	4.4	2.40	13.3	24.1	1.33
9	8.5	1.35	13.8	47.5	0.85
10	4.4	2.50	6.9	36.9	1.09
11	7.9	2.23	18.9	43.4	0.85

channel conveyance factor, is chosen from Table 8 depending upon the type of channel encountered. These values are altered from those in Table 1 in order to adapt equations (1) and (2) for use in Brays Bayou watershed.

The value of L in Table 7 represents the length of main channel within a catchment. Such a channel is chosen by selecting the storm sewer or stream which drains the most area within a given catchment. The average slope, S, is measured from 10 percent to 85 percent of the channel length, beginning at the channel outlet.

Table 9 presents the adopted unit hydrograph variables based upon the physiographic data. Because of the negligible development in catchment 4, the unit hydrograph from this catchment is generated using equations (6) and (7) which were derived from rural Houston watersheds (Espey and Winslow, 1968). The values of t_p and Q_p for catchment 8 are obtained by the HEC-1 optimization subroutine using historical records as input. The reason for this procedure is that the Bintliff Ditch gage lies downstream of a bridge culvert which constricts the flow. The bridge stores additional runoff upstream as backwater and decreases the expected peak flow at this site.

$$t_p = (2.68\, L^{0.223}\, S^{-0.302}) - 15 \qquad (6)$$

$$Q_p = 8.25 \times 10^4\; A^{0.988} (t_p + 15)^{-1.26} \qquad (7)$$

5.4 ROUTING PARAMETERS

The Muskingum routing parameters K and X are obtained by solving the equations of motion for a typical flood wave using the method of

TABLE 8

ADOPTED Φ CLASSIFICATION FOR BRAYS BAYOU

	Φ_1 Classification
0.85	Extensive channel improvement and storm sewer system, closed conduit channel system.
1.13	Some channel improvement and storm sewers; mainly cleaning and enlargement of existing channel.
1.50	Natural channel conditions.

	Φ_2 Classification
0.0	No channel vegetation.
0.1	Light channel vegetation.
0.2	Moderate channel vegetation.
0.3	Heavy channel vegetation.

$$\Phi_1 + \Phi_2 = \Phi$$

TABLE 9
30-MINUTE UNIT HYDROGRAPH VARIABLES
BRAYS BAYOU WATERSHED

Catchment	t_p (hours)	Q_p (cfs)
1	5.33	791
2	2.07	2,100
3	3.02	965
4	10.17[a]	190[a]
5	2.62	738
6	2.63	660
7	2.30	1,190
8	0.84[b]	774[b]
9	0.72	3,320
10	1.53	914
11	0.92	2,610

[a] Derived from rural equations (6) and (7).

[b] Derived from streamflow records using HEC-1 optimization subroutine.

characteristics. Figure 6 shows the schematic representation of six routing reaches which the model considers. Reach D is simply an extension of Reach E because of the short travel time encountered in the concrete-lined channel.

Although the reaches all have trapezoidal cross-sections, the method of characteristics computer solution available (Viessman, 1977) can accept only rectangular cross- sections as input. The difference in results can be minimized by assuming a slightly wider channel than actually exists. This fictitious increase in width corresponds to the width of a rectangular channel which has the same bankfull capacity as the actual channel.

Figure 7 presents an example of one of the methods of characteristics solution. The inflow hydrograph to the reach is generated by a gamma distribution in order to simulate the expected runoff from an upstream catchment. HEC-1 then optimizes values of K and X until the best fit of the two computed outflow hydrographs is achieved.

Results for all six reaches indicate that while X depends mostly upon the type of channel lining, K appears to be almost equal to the length of the reach divided by the peak average velocity of flow. Table 10 lists the adopted values for K and X obtained from the two calibration storms. Although the value of K depends upon the magnitude of the flood event, peak average velocities did not differ significantly between the two storms.

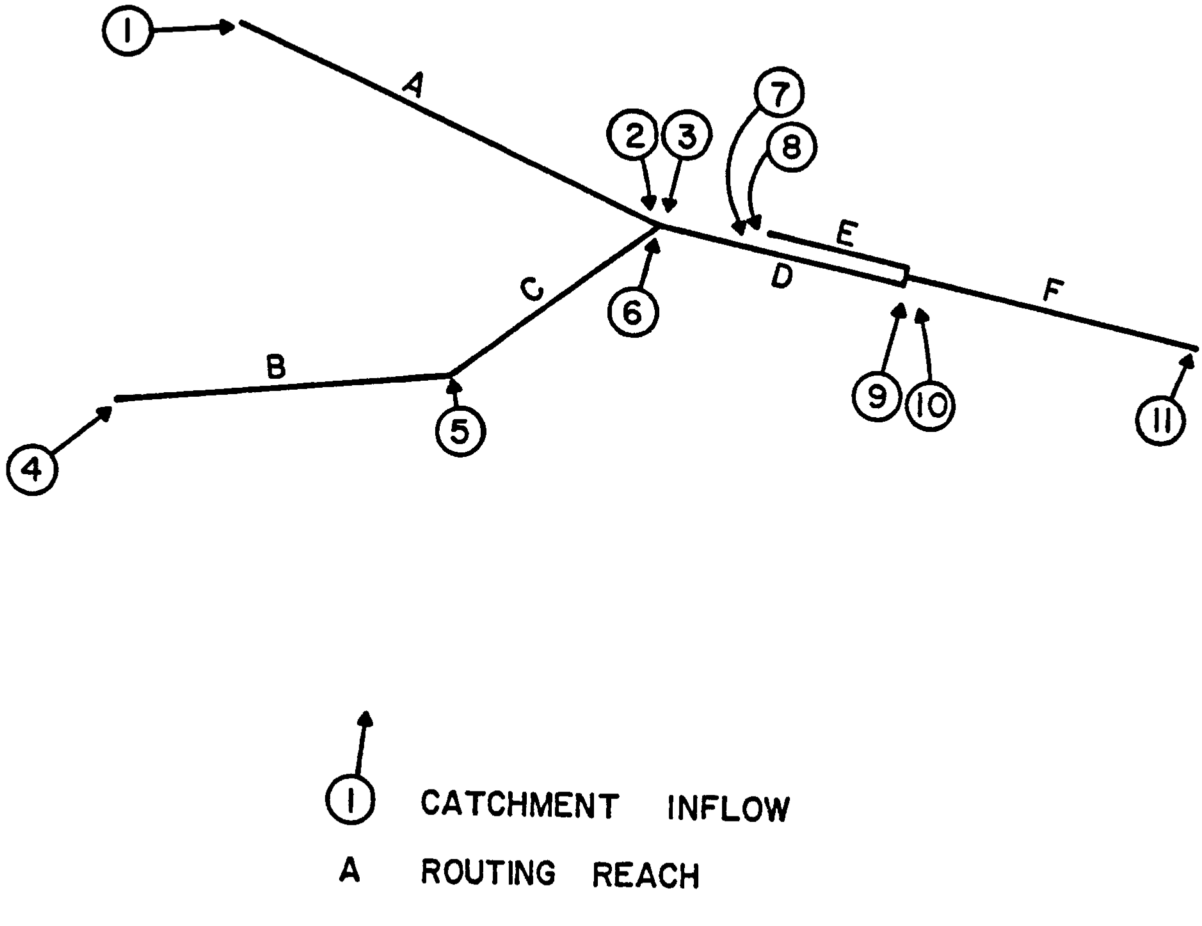


Figure 6

FLOOD ROUTING
SCHEMATIC DIAGRAM
BRAYS BAYOU WATERSHED

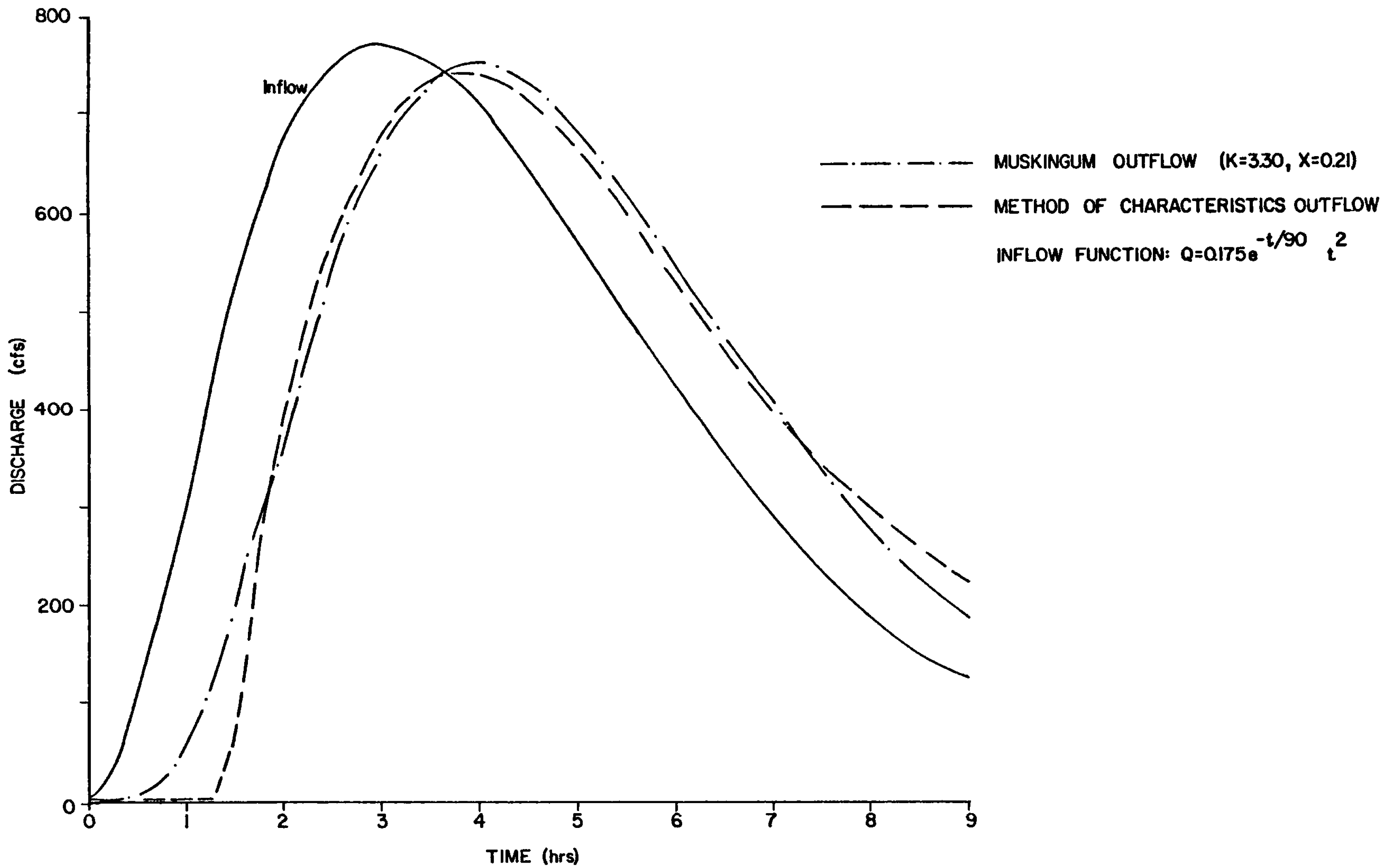


Figure 7

METHOD OF CHARACTERISTICS

vs.

MUSKINGUM ROUTING METHOD

TABLE 10

CHANNEL ROUTING PARAMETERS

Reach	Length (ft x 10^3)	Channel Slope (x 10^{-4})	Manning's "n" Coefficient	Channel Bottom Width (ft)	Adopted Muskingum Coefficients K (hrs) June 9, 1975	Adopted Muskingum Coefficients K (hrs) June 15, 1976	X
A	37.2	4.8	.030	60	3.55	3.30	.21
B	20.5	8.3	.030	12	2.10	1.86	.21
C	13.0	6.2	.030	20	0.94	1.00	.21
D	26.0	5.2	.015	63	0.80	0.86	.24
E	17.9	6.0	.015	63	0.56	0.54	.24
F	16.0	6.7	.015	75	0.34	0.32	.24

5.5 FINAL CALIBRATION

After adjusting values for CNSTL throughout the basin in order to simulate the storage lost to development, the final predicted peak discharges are listed in Table 11. Calibration was attempted at all three gages for both storm events and the associated hydrographs are included in Appendix B. A comparison of both observed and predicted peak discharge and runoff volume for the three gaged sites are shown in Table 12.

Although the South Main Street gage shows the poorest agreement of the observed and predicted flows, most of this error is due to the backwater effect. As the stage rises in the lower reaches the hydraulic gradient in the drainage laterals decreases to a point where the discharge is greatly reduced. Runoff in the lower basin "backs up" and is stored outside the main channel. HEC-1 does not consider the backwater phenomenon in its calculations; however, this condition would exist whenever the stage in Brays Bayou reaches a certain height.

Differences in the antecedent conditions prior to the two storms explain the higher values of CNSTL for the June 9, 1975 storm. Using one inch of rainfall as the criterion, nine days without rain preceeded the 1975 event while only three days preceeded the 1976 storm. Considering the spatial non-uniformity of the rainfall, the overall calibration of the model is considered sufficient for the purpose of this study.

TABLE 11

RAINFALL LOSS RATES AND PEAK DISCHARGES

FOR CATCHMENTS WITHIN BRAYS BAYOU

		CNSTL (in/hr)		Peak Discharge (cfs)	
Catchment	STRTL (in)	June 9, 1975	June 15, 1976	June 9, 1975	June 15, 1976
1	.24	.40	.40	956	1,110
2	.22	.25	.30	2,290	1,990
3	.23	.30	.35	2,040	1,150
4	.25	1.00	.60	153	273
5[a]	.23	.70	.40	965	791
6	.22	.30	.30	1,510	976
7	.22	.25	.30	4,340	4,510
8[b]	.20	.17	.28	826	1,410
9	.19	.15	.20	5,360	9,290
10	.20	.20	.30	3,650	5,140
11	.20	.15	.20	3,950	11,500
WATERHSED TOTAL				18,400	28,500

[a] Keegans Bayou at Roark Road gage

[b] Bintliff Ditch at Bissonnet Street gage

TABLE 12

OBSERVED AND PREDICTED PEAK DISCHARGES

AND RUNOFF VOLUMES FOR GAGES WITHIN BRAYS BAYOU

Storm	Peak Discharge (cfs)			Runoff Volume (inches)		
	Predicted	Observed	% Error	Predicted	Observed	% Error
June 9, 1975:						
Keegans Bayou	965	966	0.1	0.78	0.95	17.9
Bintliff Ditch	826	864	4.4	1.58	1.62	2.5
Brays Bayou at South Main Street	18,400	18,000	2.2	2.45	2.60	5.8
June 15, 1976:						
Keegans Bayou	791	768	3.0	1.17	1.05	11.4
Bintliff Ditch	1,406	1,170	20.5	3.11	3.13	0.6
Brays Bayou at South Main Street	28,500	29,000	1.7	3.37	4.07	17.2

5.6 SENSITIVITY ANALYSIS

Error inherent in a hydrologic model is extremely difficult to quantify because of the multiplicity of sources of error. The inaccuracy of the rainfall data, loss estimates, unit hydrograph theory, and routing methods all contribute to error in the final result.

Therefore, several important inputs to HEC-1 were altered by as much as + 50 percent and the relative error in the outflow peak discharge is noted. Figure 8 depicts the range of answers that intentional error produces in the model. According to the results, the total rainfall parameter is the most influential followed by Q_p and t_p, respectively. In fact, relative errors in total rainfall ($\pm$ 20 percent) are amplified by the model to produce even larger relative errors in the discharge ($\pm$ 25 percent) at South Main Street.

5.7 STORAGE DESIGN

The feasibility of detention storage in the upper basin is examined relative to storm frequency and storage capacity. The storage volume necessary to contain the 5, 10, and 25-year frequency storm are determined as shown in Table 13. These calculations are based upon the SCS curve number procedure as discussed in Appendix A. Also included in Table 13 are the peak discharges and times-to-peak of the outflow hydrograph for a matrix of storm return periods and storage designs. The results of Table 13 are graphically shown in Figure 9. In this figure the peak outfow discharge is related to storm return period for a given storage design. These peak discharge vs. frequency

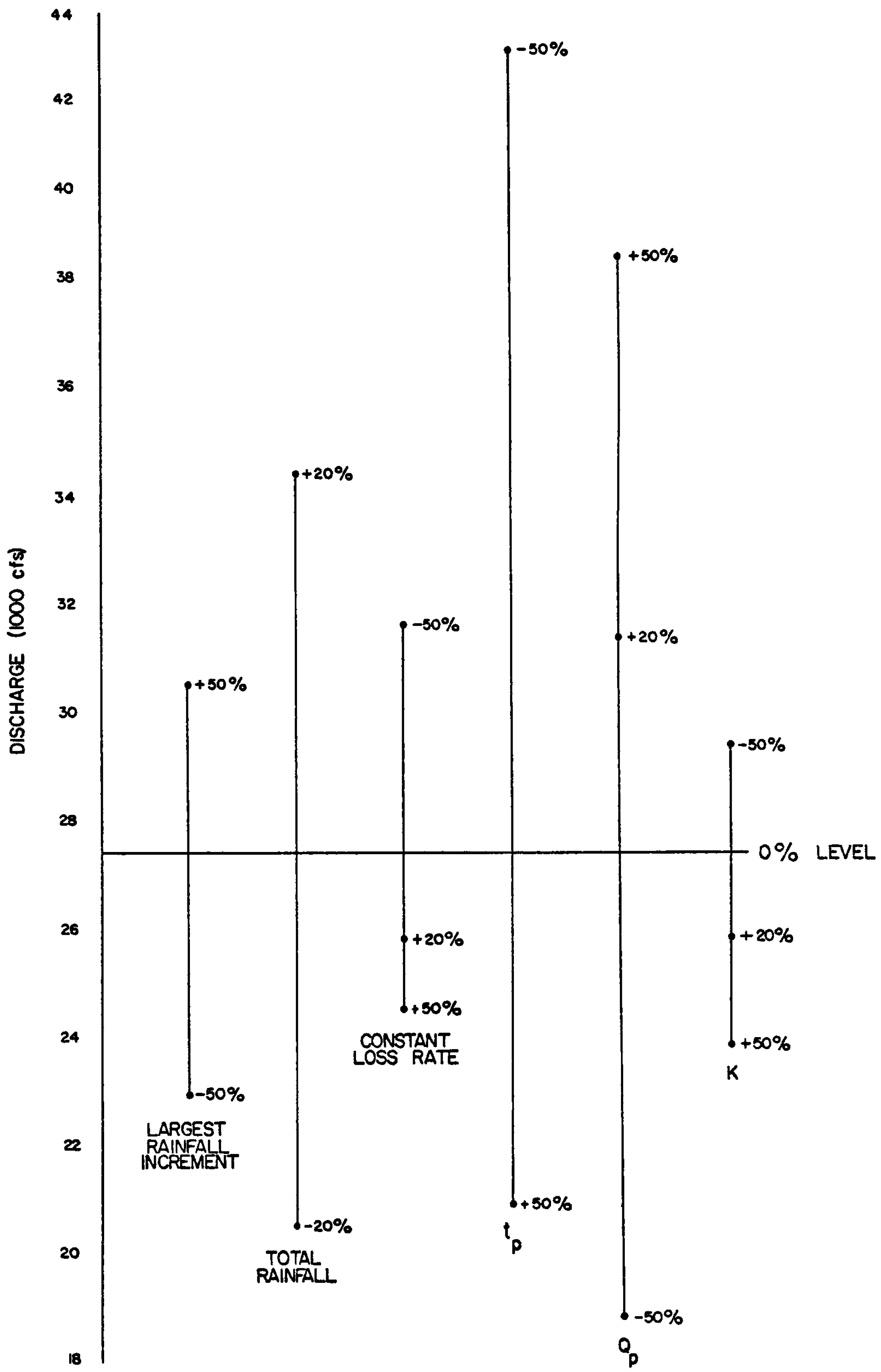


Figure 8

COMPARISON OF PEAK FLOWS AT SOUTH MAIN STREET FROM SENSITIVITY STUDY

TABLE 13

EFFECTS OF VARIOUS STORAGE VOLUMES ON 200-ACRE HYDROGRAPH PEAK FLOWS AND TIMES TO PEAK FLOW

Land Use	24 Hour Storm Frequencies and Rainfalls			
	5 year 6.8 inches	10 year 8.6 inches	25 year 9.7 inches	100 year 12.7 inches
Undeveloped	253 cfs[a] 124 min	336 cfs 124 min	405 cfs 124 min	570 cfs 124 min
Developed without storage controls	556 cfs 34 min	711 cfs 34 min	837 cfs 34 min	1,132 cfs 34 min
Developed with 5 year storage controls (29 ac-ft)	248 cfs 81 min	390 cfs 70 min	510 cfs 63 min	794 cfs 57 min
Developed with 10 year storage controls (35 ac-ft)	172 cfs 100 min	316 cfs 81 min	424 cfs 74 min	691 cfs 63 min
Developed with 25 year storage controls (37 ac-ft)	154 cfs 107 min	284 cfs 89 min	400 cfs 77 min	659 cfs 66 min

[a] Upper figure represents peak outflow, lower figure represents time to peak outflow.

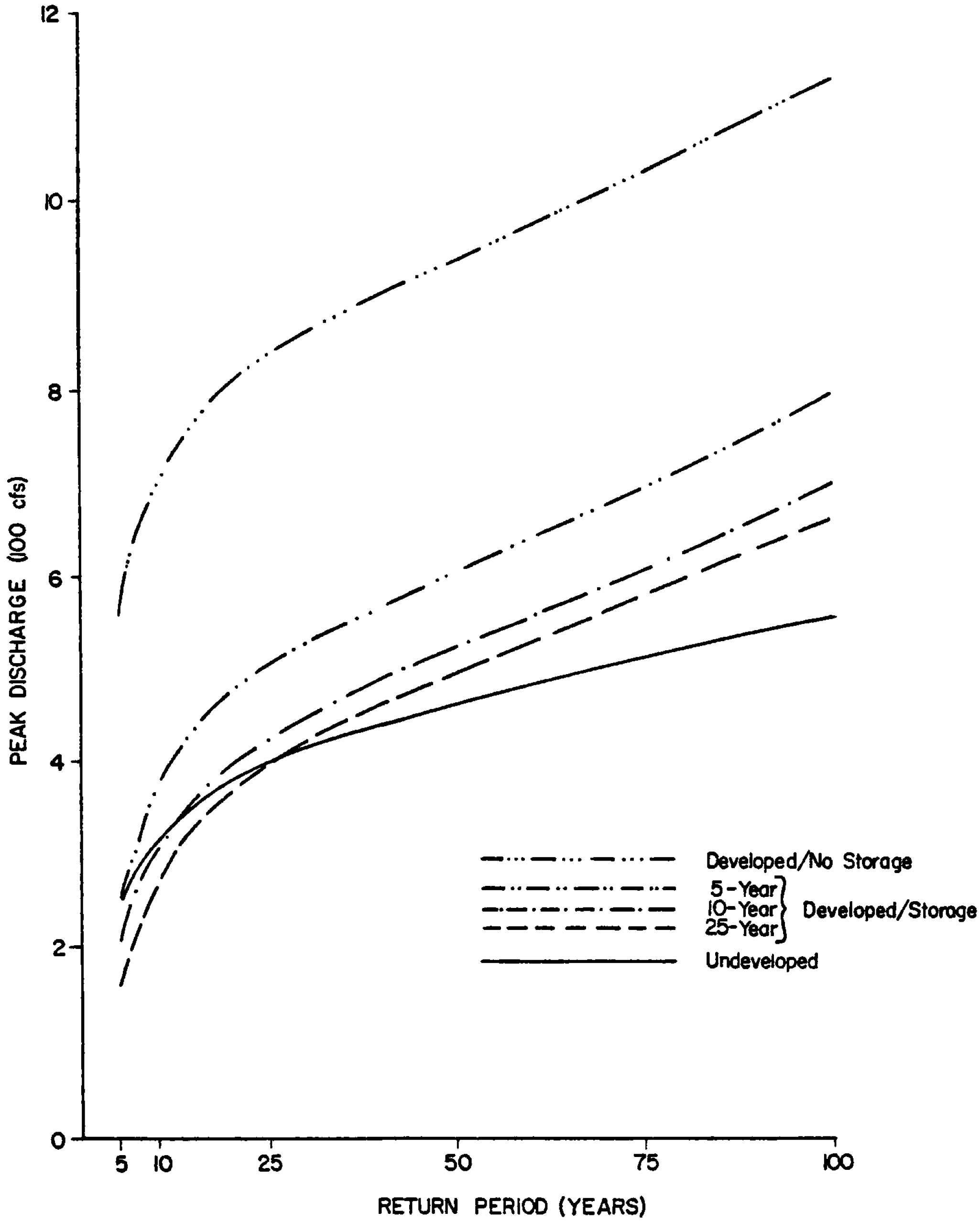


Figure 9

RETURN PERIOD vs. PEAK DISCHARGE FOR VARIOUS DETENTION STORAGE DESIGNS — 200 ACRE DEVELOPMENT curves can be compared with the curves representing the undeveloped condition as well as those describing the developed condition without detention storage.

Storage vs. discharge relationships for each detention policy are presented in Figure 10. These results are multiplied in a linear fashion to simulate the effect of many 200-acre developments provided with detention storage. It was felt that because of the lack of topography a 10-year storage policy would be the largest feasible design. Therefore, only the 5 and 10-year designs are considered as stormwater management options.

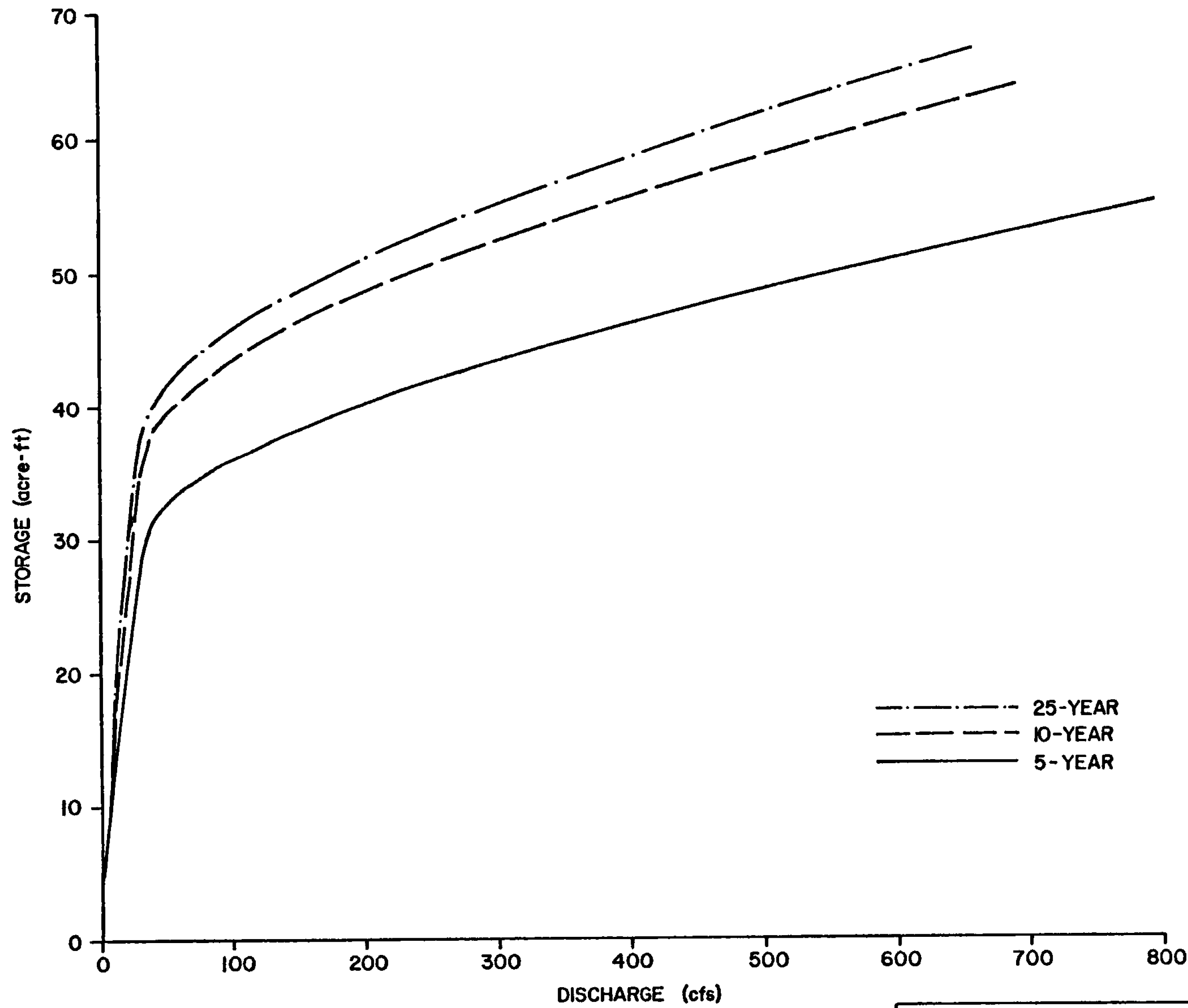


Figure 10

STORAGE-DISCHARGE CURVES
FOR VARIOUS DETENTION STORAGE DESIGNS
200-ACRE DEVELOPMENT

6.0 RESULTS

6.1 RAINFALL FREQUENCY ANALYSIS

The SCS Type-II 24-hour precipitation pattern provides the design rainfall for the basin model in addition to the detention storage model. Peak rainfall intensity occurs 11.88 hours into the storm as 38 percent of the total rainfall amount falls between 11.5 and 12.0 hours. This rainfall distribution is common to all of the continental United States except the West Coast (SCS, 1975). Figure A-1 presents cumulative rainfall fractions for the Type II storm.

The 24-hour rainfall amounts are obtained for return periods ranging from 5 to 100 years (Hershfield, 1961). Total amounts are as follows: 5-year, 6.8 inches; 10-year, 8.4 inches; 25-year, 9.7 inches; 100-year, 11.7 inches. These amounts are applied uniformly across each catchment and throughout the watershed. Results for each frequency storm are expressed as a function of land use (1978 and 1995) detention policy (5-year and 10-year), and the amount of development influenced by storage (present, future, and present plus future development). Results considering baseline conditions (1978 land use with no storage) are presented in Table 14 and plotted on log-normal paper in Figure 11. Extrapolation of these results indicate that the 2.33-year flood (mean annual flood) is approximately 14,800 cfs. A linear regression analysis of the logarithm of the peak discharge vs. the standardized variate of the normal probability distribution yields an adjusted correlation coefficient equal to 0.995 and a 90 percent error range of $\pm$ 6.8 percent.

TABLE 14

PEAK DISCHARGES FOR 1978 LAND USE WITH NO STORAGE

BRAYS BAYOU AT SOUTH MAIN STREET

Storm Frequency	Peak Discharge (cfs)
5-yr	20,900
10-yr	27,300
25-yr	32,900
100-yr	46,800

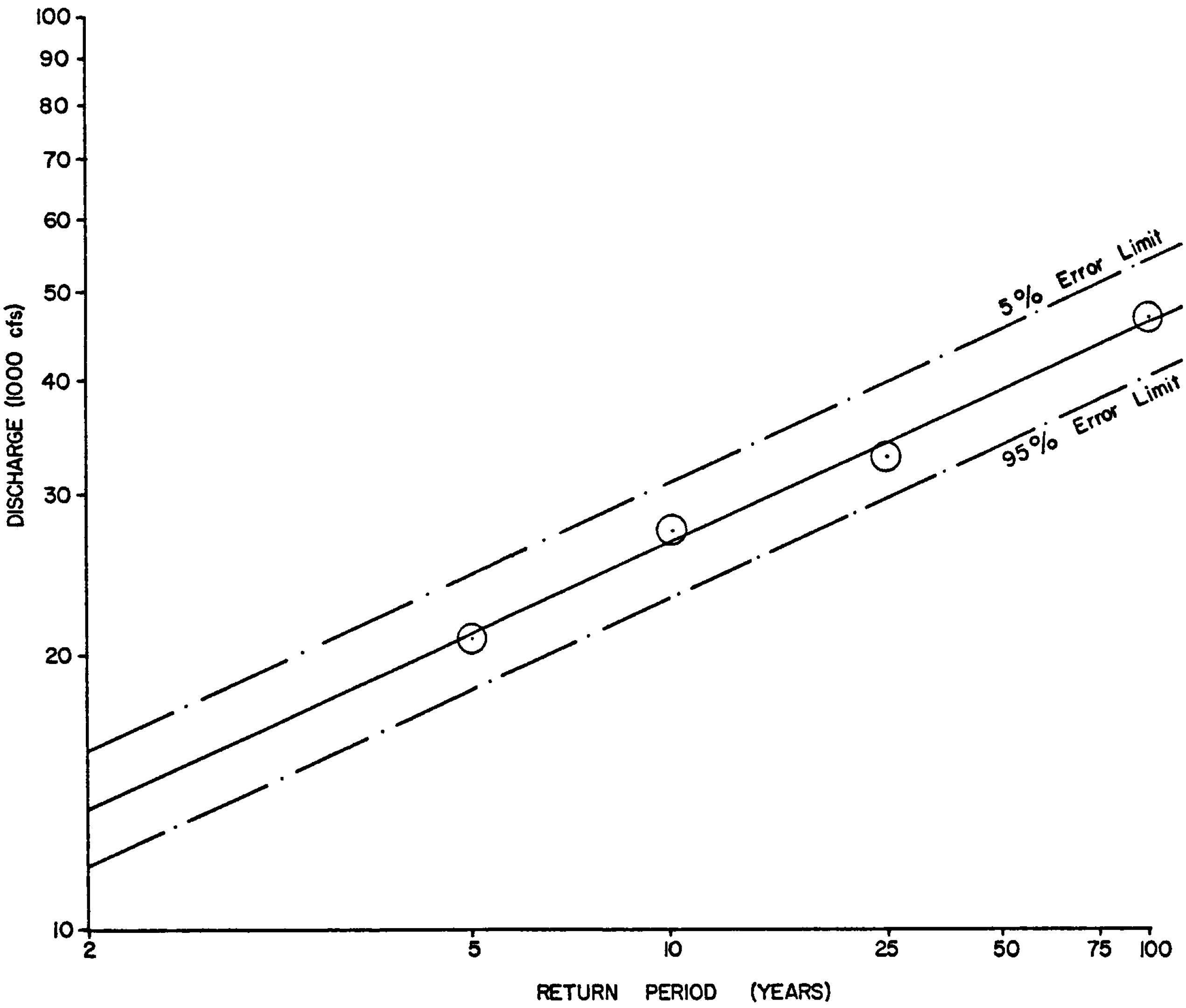


Figure 11

BRAYS BAYOU AT SOUTH MAIN STREET
RETURN PERIOD vs. PEAK DISCHARGE
NO STORAGE
1978 LAND USE

6.2 LAND USE ANALYSIS

The model considers two levels of development: 1978 and 1995 conditions. Table 15 presents the additional area predicted to undergo development between 1978 and 1995. All types of new development are considered to have similar impacts upon the hydrologic regime.

This additional development is then expressed as multiples of 200-acre tracts. The effect of additional development is predicted by adjusting the developed hydrograph of a catchment by the percentage increase in developed area while reducing the undeveloped hydrograph by the percentage decrease in open space.

6.3 EFFECT OF STORAGE DESIGN

Table 16 lists the peak discharge at the basin outlet for the storm frequencies and storage policies mentioned previously for new development only. Land use ranges from 1978 to ultimate land use. Without storage, only the 5- and 10-year floods remain within the channel under present conditions; under 1995 land use, however, only the 5-year event remains below channel capacity (29,000 cfs). Even though detention storage reduces the peak discharge, its effectiveness diminishes with higher storm return periods. This is due to the fact that when the peak rainfall for the larger storms occurs, the detention ponds are already full (Erie, 1978).

Note also that the peak discharge for the 10-year storm with 10-year storage design and 1995 land use (Table 16) is significantly higher than

TABLE 15

NEW DEVELOPMENT BETWEEN 1978 AND 1995

BRAYS BAYOU WATERSHED

Catchment	Development (acres)
1	250
2	2,000
3	1,500
4	50
5	775
6	650

Source: Rice School of Architecture, 1978.

TABLE 16

PEAK DISCHARGES IN CFS FOR 1995 LAND USE

STORAGE ON NEW DEVELOPMENT ONLY

BRAYS BAYOU AT SOUTH MAIN STREET

	Storage Policy		
Storm Frequency	No Storage	5-yr	10-yr
5-yr	28,500	24,100	22,700
10-yr	36,400	32,700	31,600
25-yr	42,900	39,500	38,600
100-yr	58,300	56,000	54,600

for existing land use (Table 14). The reason for this apparent inconsistency is that although detention storage produces the undeveloped peak discharge, it does not duplicate the undeveloped time-to-peak (see Table 13). Higher flows are sustained for longer periods of time than in the undeveloped case (McCuen, 1979). This timing difference is important in a watershed with such a fast response time as Brays Bayou. In summary, one inch of man-made detention storage is not an adequate substitute for one inch of natural storage.

Although a detention pond is sized in Chapter 5 to store the 25-year event, this design is not considered in this study because the results obtained from 25-year storage would not be significantly better than those using 10-year storage to justify the additional land required. If the ponds are built deeper than three feet, however, the 25-year storage design may become feasible. This design would require a pumping system in order to release the contents from flood storage after a storm.

6.4 EFFECT OF STORAGE IMPLEMENTATION TIMING

Since there is a sizable amount of development already existing in the upper basin, how soon detention requirements are actually put into effect is as important as how much storage is implemented. To test this theory, detention storage is assigned to all present development in the upper six catchments. This retrofit technique assumes that land is available to accomplish on-site storage under present conditions. The development which occurs between 1978 and 1995 also must provide adequate stormwater retention. These results can then be compared with those in Section 6.3 to recognize the importance of regulating flows from present as well as future development.

Table 17 includes the results for both the present and 1995 retrofit storage analysis. The 10-year frequency storm is selected as the design event because its flows are confined within the channel at present but not for 1995 land use under the two storage policies proposed. A 10-year event, therefore, represents an upper limit on the present carrying capacity of Brays Bayou.

Although the 1978 retrofit scenario provides some peak discharge reduction, the greatest effect results from the combination of existing and future storage for 1995 conditions. A peak flow reduction of almost 28 percent occurs using 10-year retrofit storage under 1995 land use. Thus, both the 5-year and the 10-year retrofit storage proves sufficient to prevent overbank flows by 1995. If the post-1995 development also provides 10-year storage, then the peak discharge from the 10-year storm should always remain within the channel.

The storm hydrographs for the 10-year storm results in Tables 16 and 17 are displayed in Figures 12, 13, and 14. These hydrographs show a double-peaked phenomenon for Brays Bayou at the South Main Street site. The first peak, after about 13 hours, represents the contribution from the developed lower basin. The arrival of the second peak, which varies depending upon the storage policy from 15 to 16.5 hours, corresponds to the routed flows from the upper basin. Although detention storage in the upper basin attenuates and lags the second peak, the initial peak remains unchanged. For the 10-year storm, a lower limit of discharge is reached at 25,200 cfs due to the static response of the lower basin.

TABLE 17

10-YEAR STORM WITH RETROFIT STORAGE*

PEAK DISCHARGES FOR BRAYS BAYOU AT SOUTH MAIN STREET

Land Use	Storage Policy	Peak Discharge (cfs)
1978	No storage	28,700
	5-yr	25,200
	10-yr	25,200
1995	No storage	34,900
	5-yr	27,000
	10-yr	25,200

* Detention storage applied to all existing development in catchments 1-6.

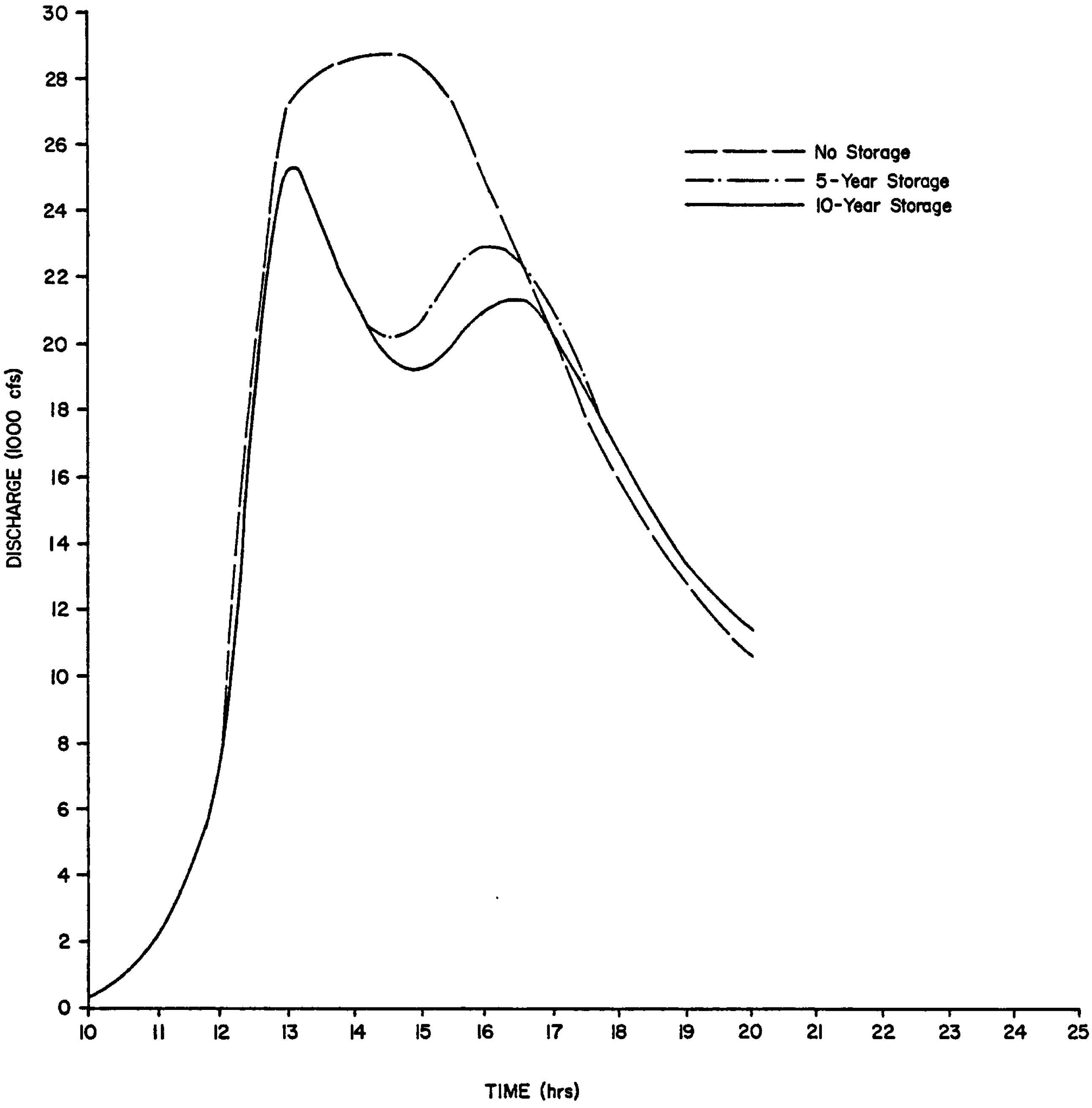


Figure 12

BRAYS BAYOU AT SOUTH MAIN STREET
10-YEAR STORM
RETROFIT STORAGE
1978 LAND USE

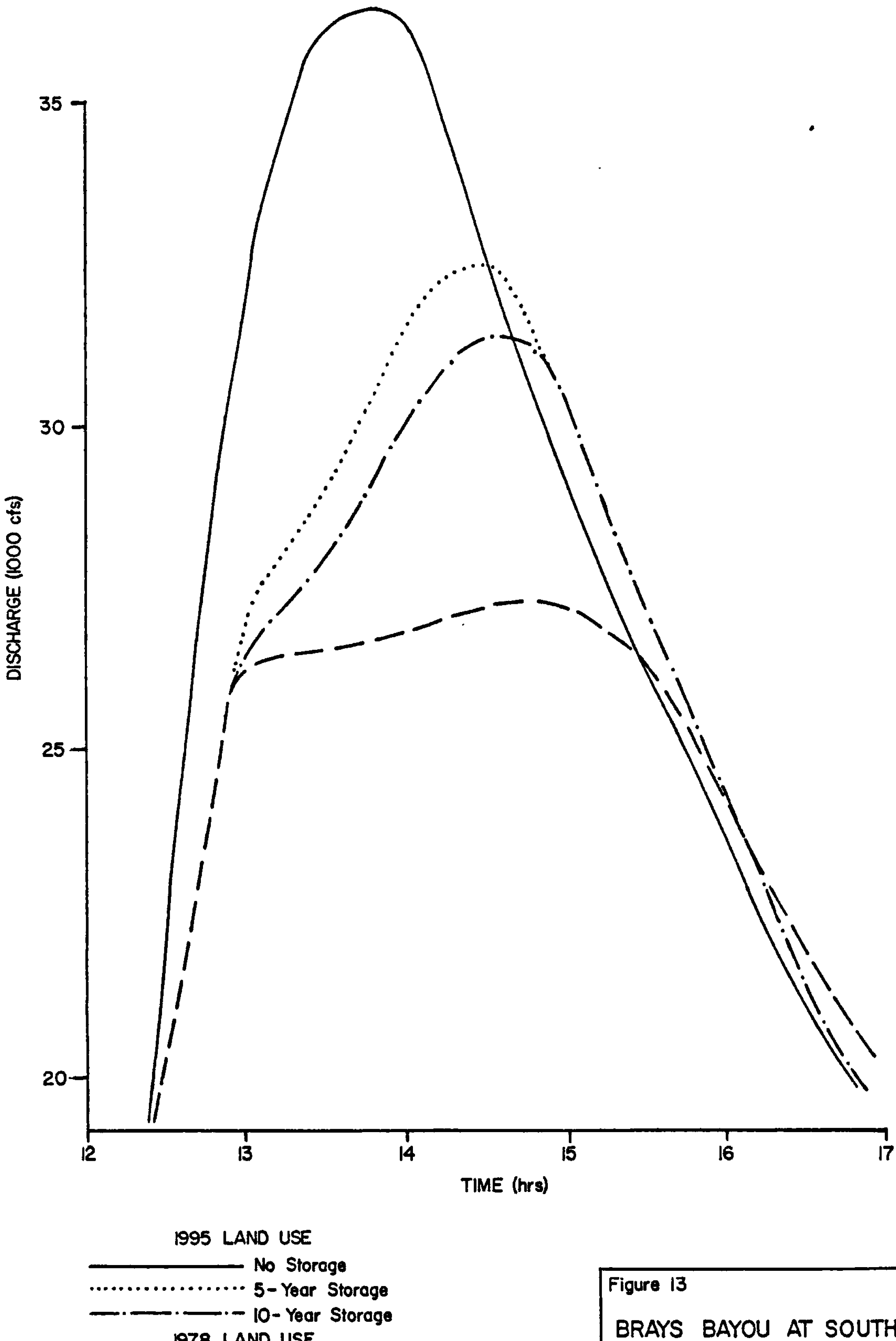


Figure 13

BRAYS BAYOU AT SOUTH MAIN STREET
10-YEAR STORM
STORAGE ON NEW DEVELOPMENT ONLY

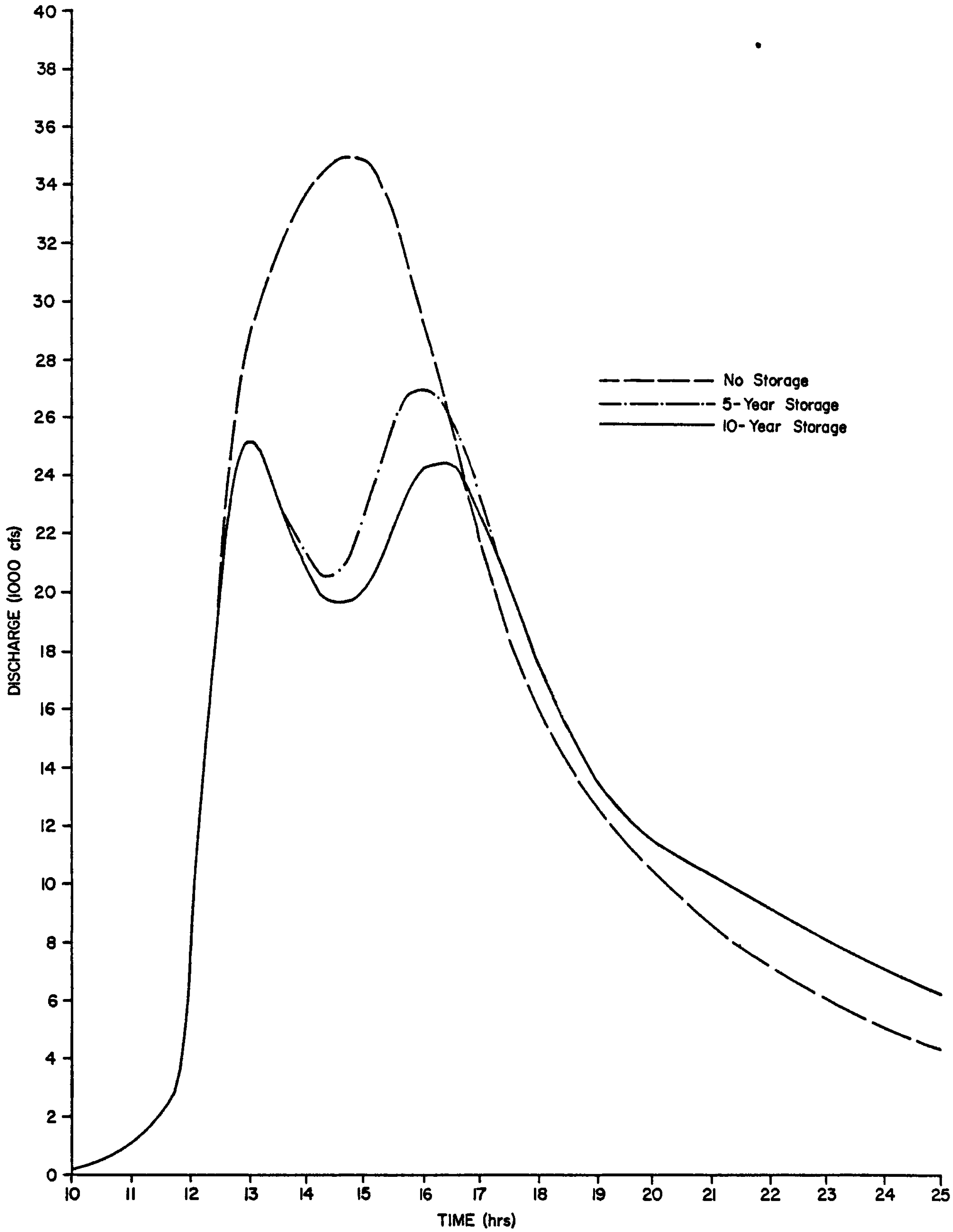


Figure 14

BRAYS BAYOU AT SOUTH MAIN STREET
10-YEAR STORM
RETROFIT AND FUTURE STORAGE
1995 LAND USE

One final result should be mentioned. In this study two completely different methods are selected to generate unit hydrographs: the Espey and Winslow regression equations and the SCS curve number technique. Because of the range of data used to derive the regression equations, it is appropriate to select a more detailed approach such as the SCS procedure when considering drainage areas as small as 200 acres. The SCS method provides the inflow to the detention basins from urbanized areas and is outlined in Appendix A. While the model is being calibrated, only the Espey and Winslow approach is used. However, the results in this chapter rely upon a combination of both unit hydrograph methods to predict the runoff from developed and undeveloped areas. A check upon the variance in the computed flows using each procedure can be seen by comparing the results for 1978 land use with no storage in Table 17 with its counterpart for the 10-year storm in Table 14. The 5 percent difference in peak flow (27,300 vs. 28,700 cfs) is small when compared with the magnitude of other errors in the model.

7.0 CONCLUSIONS

Full scale development of Upper Brays Bayou threatens to reduce the capacity at the watershed outlet from the 100-year storm as originally designed to less than the 10-year storm. This analysis neglects the backwater condition created in the secondary drainage system by bankfull flows in Brays Bayou. In all probability, a storm at the 10-year level would not overtop its banks within the basin; however, since runoff volume must be conserved, backwater flooding will cause widespread damages in the lower basin.

In order to prevent a continued decrease in the theoretical channel capacity to the 5-year event, stormwater controls on existing development in the upper five catchments should be considered by developers as an alternative to existing drainage policies when enough undeveloped land is available to provide storage. Appropriate city and county agenices should also mandate the implementation of detention storage, where feasible, for new developments. To retain the present 10-year capacity at the outlet, the minimum storage design required is the 10-year design. Assuming a three-foot depth for the detention basin, the 10-year design would require almost 6 percent (or 11 2/3 acres per 200 acres) of the total area of development. Modifications of the design of the basin itself could further reduce the percentage of impounded surface area necessary for storage.

An added benefit of detention storage is the delay in the arrival of the upstream peak at the basin outlet. This delay (up to two hours for a 10-year storage policy and a 10-year storm event) would allow the downstream tributaries to discharge runoff more effectively and reduce the extent of the backwater flooding. It is also suggested that no additional channelization take place along either Brays or Keegans Bayous so that the travel time to the outlet is not further reduced. A more viable alternative to further structural improvement of the channel itself is the temporary on-site detention of runoff in the Upper Brays Basin.

However, in light of the inherent inaccuracies in the input data and the hydrologic calculations, the effects of detention storage are not significantly greater than the margin of error expected in the model. A major conclusion is that detention storage may not be effective enough as a runoff control measure by itself. Other means of stormwater controls may be necessary in addition to detention storage due to the dwindling percentage of open space in the upper watershed. Perhaps the most salient fact resulting from this research is that flood control benefits derived from implementing detention storage in a rapidly urbanizing basin are severely constrained by the extent and the location of urban land use within that basin.

8.0 LITERATURE CITED

Abt, S.R. and N.S. Grigg. 1978. An Approximate Method for Sizing Detention Reservoirs, Water Resources Bulletin, American Water Resources Association, Vol. 14, No. 4, Aug. 1978, pp. 956-965.

Amandes, C.B. and P.B. Bedient. 1980. Stormwater Detention in Developing Watersheds, Journal of the Environmental Engineering Division, American Society of Civil Engineers, Vol. 106, No. EE2, April 1980, pp. 403-419.

Bedient, P.B., et al. 1977. Environmental Model fo the Kissimmee River Basin, Journal of the Water Resources Planning and Management Division, American Society of Civil Engineers, Proceeding Paper 13326, Nov. 1977.

_____. 1978. Automatic Water Quality Monitoring and Data Management System, City of Houston Public Health Department, Houston, Texas, Oct. 1978.

Brater, E.F. and H.W. King. 1976 Handbook of Hydraulics, 6th ed., McGraw-Hill, Inc., New York, New York.

Carter, R.W. 1961. Magnitude and Frequency of Floods in Suburban Areas, U.S. Geological Survey Professional Paper 424-B, 1961.

Characklis, W.G., et al. 1976. Maximum Utilization of Water Resources in a Planned Community, EPA Research, Grant #802433, Department of Environmental Science and Engineering, Rice University, Houston, Texas. Dec. 1976.

Chow, V.T. and B.C. Yen. 1976. Urban Storm Water Runoff: Determination of Volumes and Flowrates, EPA Report 600-2-76-116, May 1976.

Cook, H.L. 1946. The Infiltration Approach to the Calculation of Surface Runoff, Transactions American Geophysical Union, Vol. 27, Oct. 1946. pp. 726-747.

Crawford, N.H. and R.K. Linsley. 1966. Digital Simulation in Hydrology, Stanford Watershed Model IV, Technical Report 39, Department of Civil Engineering, Stanford University, Stanford, California, July 1966.

Curtis, D.C. and R.H. McCuen. 1977. Design Efficiency of Stormwater Detention Basins, Journal of the Water Resources Planning and Management Division, American Society of Civil Engineers, Vol. 103, No. WR1, May 1977, pp. 125-140.

Diniz, E.V. and W.G. Characklis. 1976. Modeling Urban Runoff from a Planned Community, presented at the April 1976 EPA Conference on Environmental Modeling and Simulation, Cincinnati, Ohio.

EPA. 1971. Storm Water Management Model, Vol. I-IV, Environmental Protection Agency, Washington, D.C., July 1971.

Erie, L.J. 1979. Stormwater Retention Criterion for Urban Drainage Basin Management, presented at the February 1979 American Society of Civil Engineers Water Systems 1979 Specialty Conference, Houston, Texas.

Espey, W.H. and D.E. Winslow. 1968. The Effect of Urbanization on Unit Hydrographs for Small Watersheds, Tracor Document No. 68-975-U, Office of Water Resources Research, Washington, D.C., Sept. 1968.

Halff, A.H., et al. 1978. Effect of Urbanization and Other Factors in the Calculation of Time to Peak of Synthetic Unit Hydrographs, Rice University Studies, Vol. 65, No. 1, Urban Watershed Management: Flooding and Water Quality, Houston, Texas, pp. 17-28.

Harbaugh, T.E. 1967. Numerical Techniques for Spatially Varied Unsteady Flow, University of Missouri Water Resources Center, Report No. 3.

Hershfield, D.M. 1961. Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years, Technical Paper No. 40, Cooperative Studies Section, U.S. Weather Bureau.

Holtan, N.H. 1961. A Concept of Infiltration Estimates in Watershed Engineering, ARS 41-51, U.S. Department of Agriculture, Washington, D.C.

Horton. 1935. Surface Runoff Phenomena, Edwards Brothers, Inc., Ann Arbor, Michigan.

James, L.D. 1965. Using a Computer to Estiamte the Effects of Urban Development on Flood Peaks, Water Resources Research, Vol. 1, No. 2, 1965.

Johnson, S.L. and D.M. Sayre. 1973. Effects of Urbanization on Floods in the Houston, Texas Metropolitan Area, Water Resources Investigations 3-73, U.S. Geological Survey, Houston, Texas, April 1973.

Kamedulski, G.E. and R.H. McCuen. 1979. Evaluation of Alternative Stormwater Detention Policies, Journal of the Water Resources Planning and Management Division, American Society of Civil Engineers, Vol. 105, No. WR2, Sept. 1979, pp. 171-186.

Lovell, T.L. and M.D. Smith. 1979. Findings of the Rowlett Creek Basin Expanded Floodplain Information Study, Rice University Studies, Vol. 65, No. 1, Urban Watershed Management: Flooding and Water Quality, Houston, Texas, pp. 137-152.

Leopold, L.B. 1968. Hydrology for Urban Land Planning - Guidebook on the Hydrologic Effects of Urban Land Use, U.S. Geological Survey, Circular S54, 1968.

Lumb, A.M., et al. 1974. Analysis of Urban Land Treatment Measures for Flood Peak Reduction, Office of Water Resources Research, Washington, D.C., 1974.

McCarthy, G.T. 1938. The Unit Hydrographs and Flood Routing, unpublished manuscript. U.S. Army Corps of Engineers, North Atlantic Division.

McCuen, R.H. 1979 Downstream Effects of Stormwater Management Basins, Journal of the Hydraulics Division, American Society of Civil Engineers, Vol. 105, No. HY11, Nov. 1979, pp. 1343-1356.

Mills, G.H. 1977. Introduction, Modern Concepts in Urban Drainage, Conference Proceedings No. 5, Ontario Ministry of the Environment, Toronto, Ontario, Canada, March 1977, pp. 1-12.

Poertner, H.G. 1974. Practices in Detention of Urban Stormwater Runoff, Special Report No. 43, American Public Works Association, Chicago, Illinois. June 1974.

Rice, L. 1971. Reduction of Urban Runoff Peak Flows by Ponding, presented at the January 1971 American Society of Civil Engineers National Water Resources Engineering Meeting, Phoenix, Arizona.

Rice University School of Architecture. 1978. Investigations in Urban Form and Process, unpublished monograph, Houston, Texas, May 1978.

Rodman, P.K. 1977. Effects of Urbanization on Various Frequency Peak Discharges, U.S. Army Corps of Engineers, Ft. Worth, Texas, Oct. 1977.

SCS. 1965. Technical Release No. 20, Supplement No. 1, Computer Program for Project Formulation, Hydrology, U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C., May 1965.

_____. 1972. National Engineering Handbook, Section 4, Hydrology. U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C., August 1972.

_____. 1973. A Method for Estimating Volume and Rate of Runoff in Small Watersheds, Technical Paper, U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C., April 1973.

_____. 1975. Urban Hydrology for Small Watersheds, Technical Release No. 55, U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C., Jan. 1975.

_____. 1976. Soil Survey of Harris County, Texas, U.S. Department of Agriculture, Soil Conservation Service, Temple, Texas, August 1976.

Snyder, F.F. 1938. Synthetic Unit Hydrographs, Transactions American Geophysical Union, Vol. 19, Part 1, pp. 447-454.

Theil, P.E. 1977. Urban Drainage Design for New Development, Modern Concepts in Urban Drainage, Conference Proceedings No. 5, Ontario Ministry of the Environment, Toronto, Ontario, Canada, March 1977, pp. 261-299.

Tholin, A.L., and C.J. Keifer. 1959. The Hydrology of Urban Runoff, Journal of the Sanitary Engineering Division, American Society of Civil Engineers, Vol. 85, March 1959, pp. 47-106.

Turner, Collie, and Braden, Inc. 1971. Comprehensive Study of Drainage for Metropolitan Houston, Section VI, Brays Bayou Watershed, Turner, Collis, and Brader, Inc., Consulting Engineers, Houston, Texas.

U.S. Army Corps of Engineers. 1960. Routing of Floods Through River Channels, Engineering and Design, EM 1110-2-1408.

_____. 1973. HEC-1 Flood Hydrograph Package, users manual on generalized computer program. Prepared by the Hydrologic Engineering Center, Davis, California, Jan. 1973.

_____. 1976. STORM Model, users manual on generalized computer program. Prepared by the Hydrologic Engineering Center, Davis, California, July 1976.

USGS. 1979. Hydrologic Data for Urban Studies in the Houston, Texas Metropolitan Area, 1977, U.S. Geological Survey, OFR/WRD, June 1979.

Van Sickle, D. 1969. Experiences with the Evaluation of Urban Effects for Drainage Design, presented at the October 1969 Water Resources Symposium on the Effect of Watershed Changes in Streamflow, University of Texas, Austin, Texas.

Viessman, W., et al. 1977. Introduction to Hydrology, Dun-Donnelley, Inc., New York, New York.

APPENDIX A

Unit hydrographs for typical developed areas are estimated using standard SCS procedures outlined the publication TP-55, Urban Hydrology for Small Watersheds (SCS, 1975). Hydrographs for both developed and undeveloped conditions are derived based upon the SCS dimensionless unit hydrograph and the SCS Type II storm distribution shown in Figure A-1. The dimensionless unit hydrograph is expressed in terms of the ratio between elapsed time and the time of concentration, t_c, of the catchment. Total runoff volumes are determined by selecting runoff curve numbers, CN, which reflect native soil types and both before- and after-development land uses. All estimates assume a constant contributing drainage area of 200 acres.

Time of concentration is the time of travel from the most distant watershed divide to the watershed outlet. For the undeveloped 200-acre parcel shown in Table A-1, the most distant length would be 3300 feet assuming that each half of the catchment contributes runoff independently of the other half because of a drainage divide. If the average slope is approximately 0.2 percent and the land cover consists of shortgrass pasture then the overland flow velocity would be about 0.3 feet per second (SCS, 1975). These estimates result in a value for the undeveloped t_c of three hours.

Soil types in the urbanizing portions of the watershed are mostly Lake Charles clays and Bernard clay loams that are very slowly permeable and poorly drained (SCS, 1976). These soils belong to Hydrologic Soil Group D in the SCS classification system and have a CN equal to 80 when they occur in combination with pasture land use in good hydrologic condition (SCS, 1972).

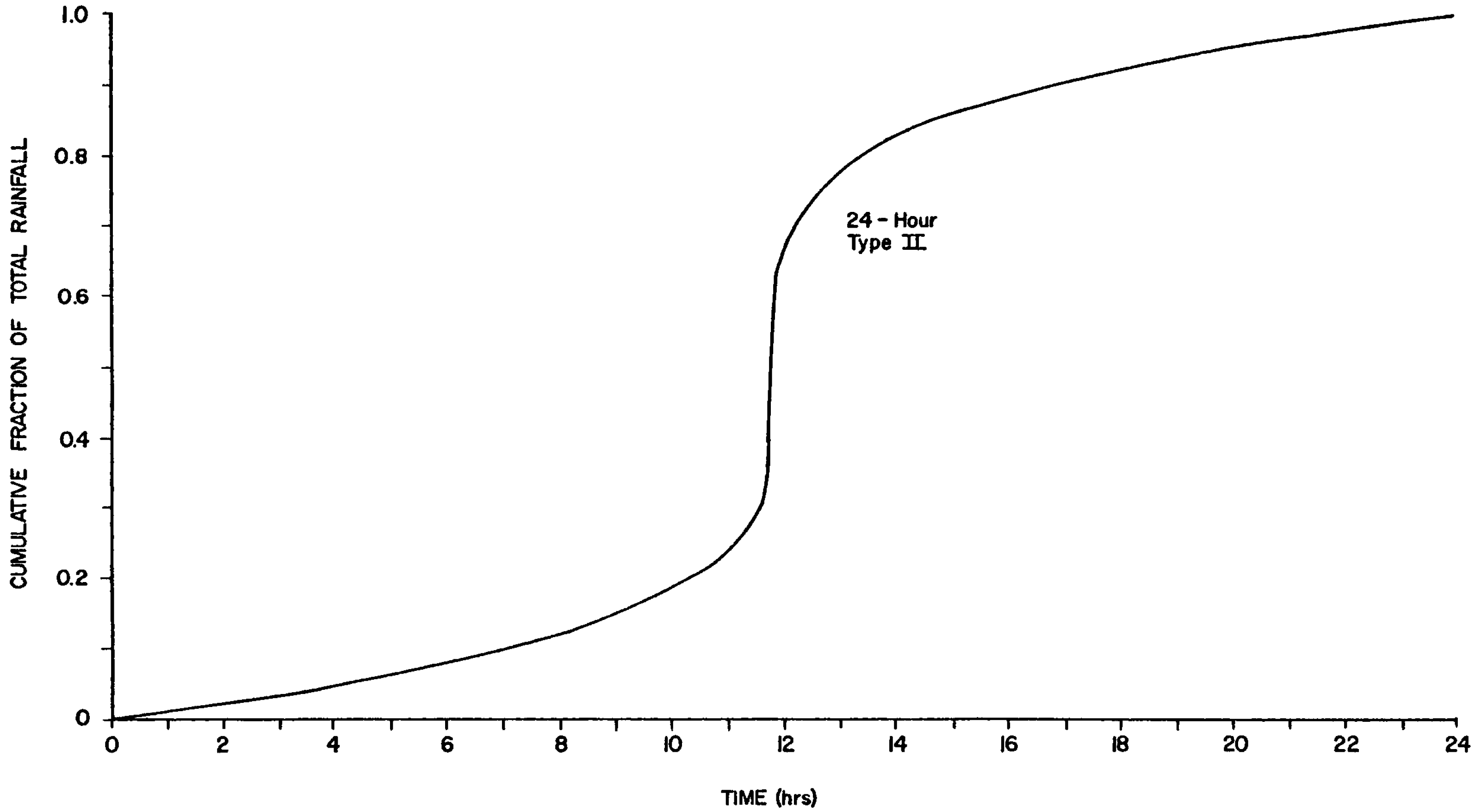


Figure A-1

SCS STANDARD RAINFALL DISTRIBUTION

TABLE A-1

TIME OF CONCENTRATION CALCULATION

200-ACRE PARCEL

Flow Path	Length (ft)	Slope (%)	Channel Type	Flow Velocity (fps)	t_c (minutes)
Undeveloped					
1	3,300	0.17	Overland Flow	0.3	183.6
TOTAL UNDEVELOPED					183.6 (3.06 hours)
Developed					
1			Rooftop Detention		10.0
2	60	1.0	Overland Flow Grassed Lawn	0.7	1.4
3	1,500	0.1	Paved Gutter	0.6	41.7
4	700	0.1	Concrete Pipe 24-inch diameter n = 0.015	2.0	5.9
5	700	0.1	Concrete Pipe 36-inch diameter n = 0.015	2.6	4.5
6	700	0.1	Concrete Pipe 48-inch diameter n = 0.015	3.1	3.7
7	700	0.1	Concrete Pipe 60-inch diameter n = 0.015	3.7	3.2
8	1,500	0.05	Earthen Channel Trapezoidal cross section 3:1 side slopes 10-foot depth, 20-foot bottom width n = 0.02	5.5	4.6
TOTAL DEVELOPED					75.0 (1.25 hours)

Infiltration rates of 0.06 inches per hour or less would be expected for such soil types.

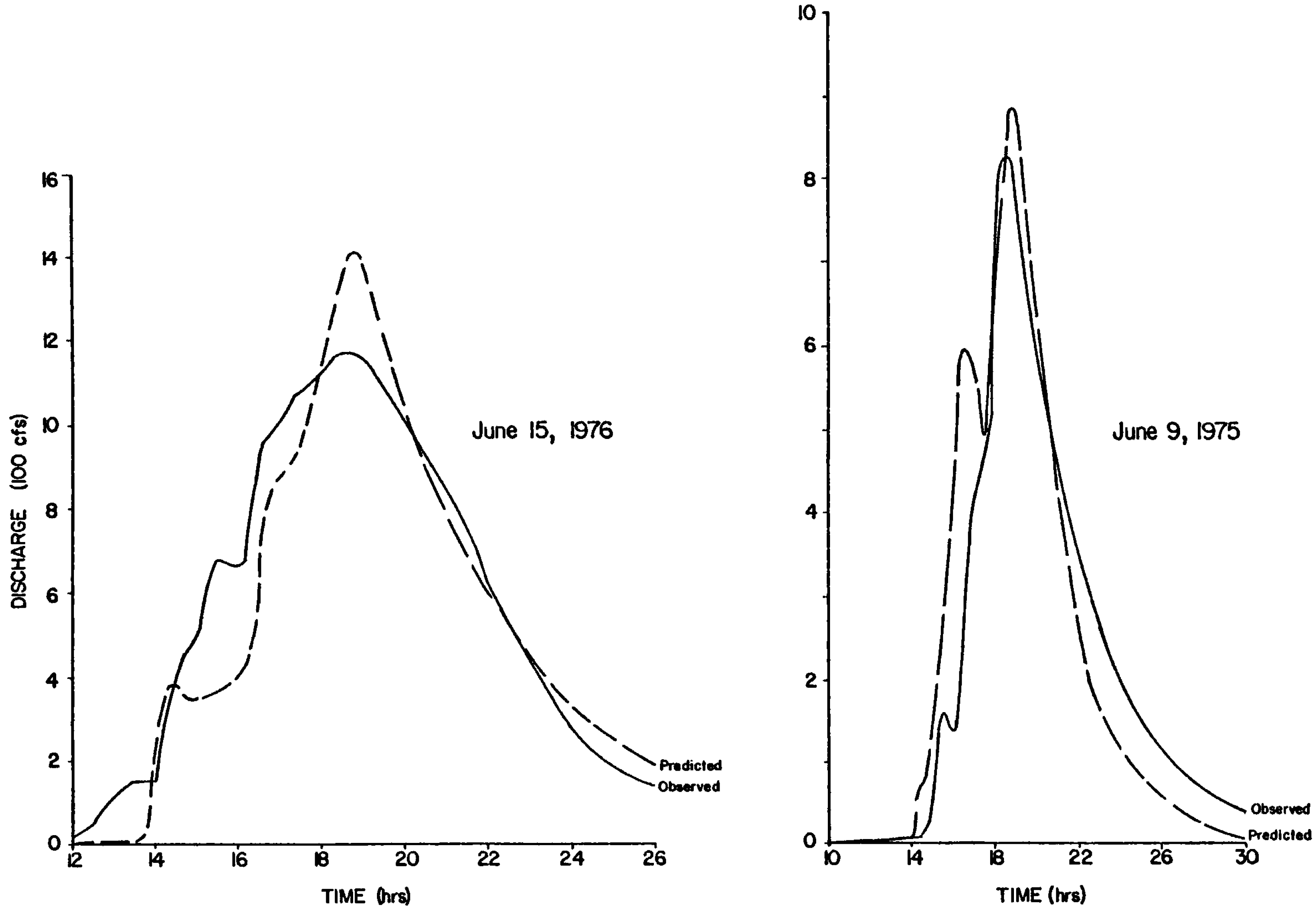
Developed hydrographs are snythesized according to the plan in Table A-1. Because of regrading and the installation of a storm sewer network the parcel no longer has the same t_c as before. The new longest flow path consists of an impervious roof and downspout followed by a graded lawn surface, street gutter, storm sewer, and drainage lateral. Although the flow length is almost twice as long (5,860 feet versus 3,300 feet), flow velocities range from 0.6 feet per second for paved street flow to 5.5 feet per second for channel flow in an earthen ditch (SCS, 1975). Therefore, the developed t_c is decreased to about one hour. A developed parcel corresponding to 50 percent impervious area has a composite CN equal to 90; therefore, additional runoff is produced along with the changes in the timing of the unit hydrograph.

Figure 1 displays unit hydrographs obtained from the SCS procedures for both types of land use, with Q_p and t_p equal to 99 cfs and 34 minutes for the developed case compared to 56 cfs and 124 minutes for the undeveloped condition. Time to peak discharge, t_p, is measured from the centroid of rainfall, which occurs 11.88 hours into the storm, to the peak discharge of the unit hydrograph, Q_p. The difference in magnitude between the developed and undeveloped peak discharges for the actual storm hydrographs is greater than for the unit hydrographs in Figure 1 since the developed parcel produces more runoff than the undeveloped parcel.

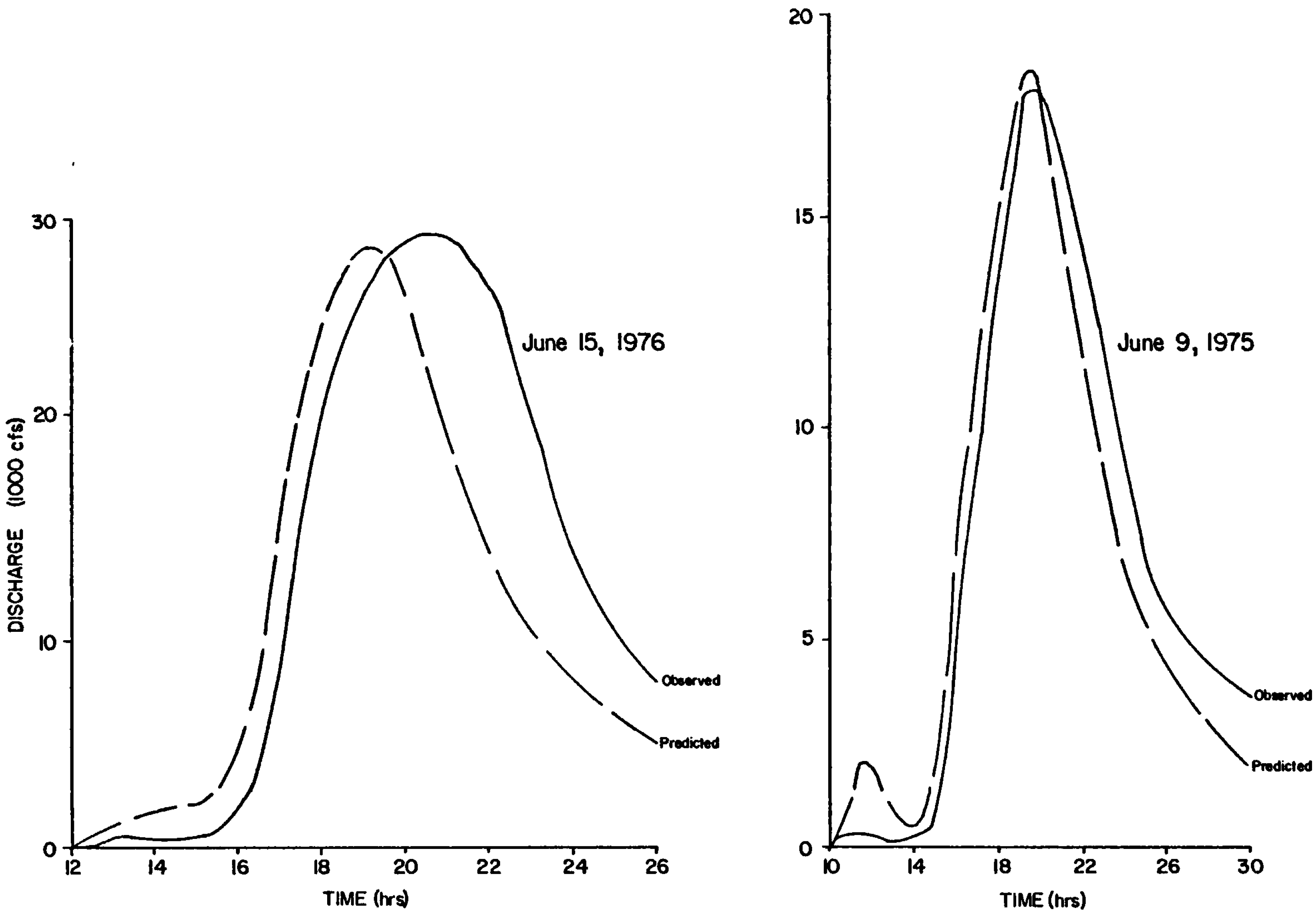
APPENDIX B

CALIBRATION STORM HYDROGRAPHS

BINTLIFF DITCH AT BISSONNET



BRAYS AT MAIN



KEEGANS AT ROARK