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A MATHEMATICAL MODEL AND INTEGRATED SIMULATION FOR URBAN FLOOD AND LAND USE ANALYSIS

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THE UNIVERSITY OF OKLAHOMA

GRADUATE COLLEGE

A MATHEMATICAL MODEL AND INTEGRATED SIMULATION FOR URBAN FLOOD AND LAND USE ANALYSIS

A DISSERTATION

SUBMITTED TO THE GRADUATE FACULTY

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BY

SUE J. LIN LEWIS

Norman, Oklahoma

1983

A MATHEMATICAL MODEL AND INTEGRATED SIMULATION

FOR URBAN FLOOD AND LAND USE ANALYSIS

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DEDICATION

This work is dedicated to my parents who said: "Continue to learn and grow, wherever you are." Without their unselfish love and encouragement, this goal would have been impossible to reach.

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ABSTRACT

A MATHEMATICAL MODEL AND INTEGRATED SIMULATION
FOR URBAN FLOOD AND LAND USE ANALYSIS

BY: SUE J. LIN LEWIS

MAJOR PROFESSOR: JIMMY F. HARP

A new approach for estimating, analyzing, and evaluating urban floods and flood damages was formulated through integrated modeling and simulation.

A mathematical model for estimating potential urban flood damages was developed. Model parameters and variables were determined and verified by statistical analyses. HEC-1 and HEC-2 programs were used to simulate the hydrologic responses and hydraulic characteristics of flooding sources for a test watershed under five propositions, each varied with the degree of urbanization. The economic analysis involved computations of economic criteria and a "trade-off" strategy for selection of the most "promising" plan for future land use. Cost benefit analysis was then applied to evaluate several feasible flood control alternatives for the selected plan.

This study indicates that land use control plays a significant role in urban flood management. The scheme of this technique can be applied to other watersheds with modification of basin parameters to form a general guide in evaluating land use proposals and flood alleviation projects.

Chapter I

INTRODUCTION

Flood damage has been one of the major watershed problems since urbanization began. In 1975, the annual nataional flood loss was estimated by the U.S. Water Resources Council to be greater than $3,500,000,000. Approximately 20,000 U.S. communities were identified to have flood hazards (Leman Powell Associates, Inc., 1980)[37]. Today, the problem is more intense than ever before.

In general, a flood may be defined as a body of water which overflows the bank of a channel and proceeds to inundate the adjacent floodplain (Ward, 1978)[62]. These high flows are usually caused by natural forces, such as rain or snow, but mankind's activities also compound flooding problems. Nixon (1966)[45] pointed out that man's invasion of flood plains is the basic cause of flood disasters. Floodplains are generally socially desirable and economically viable areas in which to live (James, 1971)[34]. The problem we are facing is a dilemma: the desire for occupying flood plains, and the fear of suffering the consequences. Man has attemped enormous efforts to alleviate flood hazards throughout history; on

* superscript after parentheses refers to Bibliography

the other hand, we have continued to occupy floodplains and to place massive life-risking investments on flood-prone areas. However, it is an impossible goal to totally eliminate flood problems by evacuating the floodplains. Man can only adjust to this problem by minimizing the risk of flooding.

A question may arise: "What are the restrictions and what amount of development should be imposed and allowed in order to maximize land use and minimize the increased risk of flooding?" Much work has been devoted to techniques of analyzing the pre-urban hydrological system, which is then taken as a reference situation for comparison of results from studies in urbanized areas. However, little work has been conducted in assessing the existing and potential flood damages due to floodplain developments. The historical records of flood damages are usually not representative for future conditions, especially with changes of land use in time and space. Therefore, it is very important to reach the position that the consequences and the potential flood damage of a planned urban activity can be predicted in a rational manner prior to the implementation of plans.

The purpose of this study is to: (1) develop a mathematical model for assessing potential urban flood damages, (2) formulate an integrated methodology to simulate hydrologic, hydraulic responses, and to assess flood damages on urban floodplains, and (3) provide a quantitative means for evaluating the economic feasibility of a range of urban activities and flood alleviation projects. The scheme of this research begins with a literature review, addressed in chapter II, followed by model development in chapter III, continued with the formulation of methodology in chapter IV, demonstrated by the case study in chapter V, and summarized in chapter VI.

Chapter II

LITERATURE REVIEW

A literature survey was conducted for the following areas:

a) hydrologic impacts of urbanization,

b) urban runoff computation and simulation models, and

c) flood damage assessment.

2.1 URBAN STUDY

Many studies have been made to evaluate the effects of urbanization during the last two decades. Leopold (1969)[38] summarized the results of several urban studies from Carter and Wiltala (1961)[67], Wilson (1966)[66], Espey; (1966)[18], and Anderson (1968)[4] and generated a series of curves and tables to illustrate the following result: urbanization induces significant increased peak flow and runoff volume. Still, Leopold cautiously stated that those curves and tables should be used with extreme caution for different drainage areas and different flow frequencies.

As many computer programs have become available since the last decade, numerous research work has been devoted to this area based on mathematical models. For example, Dempster(1974)[17] and McCorkle (1979)[41] presented an urban study for

Dallas, Texas; Beard and Chang (1979)[8] studied the urbanization impact for Tulsa, Oklahoma; Amandes and Bedient (1980)[3] presented a study for Houston, Texas. The general findings of these studies include the folllowing effects of urbanization:

a) change in total runoff,

b) change in distribution of total runoff: higher peak flow rate and less base flow,

c) change in time of concentration,

d) change in sediment content of stream.

However, the results of these studies are not likely to be applied to other watersheds because of the heterogeneous characteristics of watersheds and the lack of a good index to measure the degree of urbanization and changes of land use with time.

2.2 COMPUTATION AND SIMULATION TECHNIQUES

Many techniques have been developed throughout the centuries to define the hydrologic process, to assess the hydrologic and meteorologic data, and to quantify the outcome of these complex physical processes. As early as 1851, Mulvaney proposed the well-known Rational method which is still in extensive use (Gregory, 1932)[25]. Within two decades, the rapid growth of computer technology has offered a "boost" for the development of mathematical methods relating to this subject. Many methods have been devised with increased capacity and speed to deal with rising complicated urban flood problems.

In order to select the technique best-suited for this study, several were reviewed. Generally, techniques available for computing urban runoff can be classified into the following catagories (Feldman 1979)[21]:

a) Empirical formulae,

b) Statistical equations,

c) Single-event watershed models, and

d) Continuous watershed models.

The Rational method, as mentioned earlier, represents an empirical formula. This formula is as follows:

$$Q = CIA \qquad \text{(Eq. 2.1)}$$

where Q,C,I and A represent peak flow rate, runoff coefficient, rainfall intensity, and drainage area, respectively. This method is still widely used among engineers. However, the major drawback with this method is that it provides only the peak flow rate, not the runoff, and the coefficient "C" cannot account for the effects of flood attenuation or storage on the flood plain, both of which become important for the flood hydrograph in larger basins.

The U.S. Geological Survey "State Regression Equations" demonstrates a statistical technique for computing runoff magnitude. Sauer (1974)[50], Thomas and Corley (1977)[55] developed this sequence of equations to estimate the flood peak discharges of several recurrent intervals for Oklahoma streams. Peak flow was found to be a function of drainage area (A), channel slope (S), and mean annual precipitation (P). The general equation can be expressed as below (Sauer, 1974)[50]:

$$Q_x = k \cdot A^a \cdot S^b \cdot P^c \qquad \text{(Eq. 2.2)}$$

where Qx represents the peak flow for recurrence interval X years; K, a, b, and c represent regression coefficients, and the other terms are defined as above. The peak flow was then adjusted by using an urban factor, Rf, which was investigated and developed by Leopold (1969)[38] to account for the effect of urbanization. Also, the Federal Highway Administration has developed a similar procedure to estimate stream discharges based on the relationships of discharge, drainage area, and elevation differences in the watershed (Trent, 1978)[56] Most of the statistical techniques are based on regression analysis to relate the peak flow of a known frequency in a hydrologic region to basin characteristics without performing the rainfall-runoff analysis. It is more difficult to apply these relationships in urbanized or urbanizing basins because the rainfall-runoff relationship changes as urbanization occurs. Therefore, additional parameters must be included to account for those variations.

Many watershed simulation techniques with various complexities have been developed. The application of simulation techniques depends greatly upon the purpose, scope, and constraints of the studies. The following discussion will emphasize some of the practical state- of-the-art methods for single-event and continuous models.

In general, a single-event model simulates a single individual storm event without consideration of infiltration loss-rate recovery during periods of zero precipitation (Abbott,1977)[1]. Some of the most widely used single-event models include:

a) HEC-1 : Flood Hydrograph Package,

b) TR-20: Computer Program for Project Formulation Hydrology,

c) SWMM: Storm Water Management,

d) MITCAT: MIT Catchment Model.

The HEC-1 program was originally developed by Leo R. Beard by assembling several earlier separate hydrologic component models in 1967. After several revisions (1969,1970,1973, and 1981), the present up-to- date version has been equipped with many powerful options and additional capabilities (HEC-1 users manual, 1981)[31]. The major functions of HEC-1 include the simulation of rainfall-runoff relationships, the generation of flood hydrographs, routing and combining operations of stream networks, the evaluation for multiflood-multiplan events, flood damage analysis, automatic calibration for model parameters and alternative sizing optimization and other additional capabilities as described in the HEC-1 manual. Simulations of infiltration and routing of basins and channels are based on hydrologic and/or hydraulic concepts with several computational options. This program requires a minimum of data with easy application for computation routines.

The TR-20 program was designed by the Soil Conservation Service for storm runoff computation primarily for agricultural basins (S.C.S., 1965)[51]. Similar to HEC-1, it has the capability of developing runoff hydrographs, routing and combining separate hydrographs in a watershed, and evaluating various alternatives. It uses SCS curve numbers to account for the infiltration process. Basin and channel routings are computed on a hydraulic basis which requires geometric information of the studied basin and channel. This program also requires a relatively small amount of input data. However, it does not have the option to compute flood damage which is necessary for this study.

The SWMM model was designed by the Federal Environmental Protection Agency (EPA, 1975)[61] to simulate storm events on the basis of rainfall and basin characteristics. This program was formulated to predict the storm runoff, water quality, and treatment of the receiving stream. The infiltration process is based on Holtan's equation. Routing routines are computed based on kinematic wave theory. This program represents a highly comprehensive and complicated model with the requirements of detailed data on subbasin, channel and water qualities (Abbott, 1978)[1].

The MIT model is a relatively complex model. It concentrates primarily on the routing process for urban systems (Resource Analysis, Inc., 1975)[48]. The bases, functions, and requirement of data are similar to those of the SWMM model, but it has no capability to compute runoff quality. In general, those single-event models differ in complexity and capability. However, they share a general character; that is, these models generate a storm event without accounting for antecedent soil moisture.

On the contrary, continuous event models simulate a continuous series of storm events and account for the antecedent soil moisture. Some of the more widely-used continuous models are: the Stanford Watershed Model, HSP model, SSARR model, USGS G-824, and the STORM model which will be described briefly as follows.

The Stanford Watershed Model (Crawford, 1966)[16] may be regarded as the "pioneer" among the continuous models. It is a digital program to simulate all hydrologic processes in watersheds with a total of 21 parameters. Contemporaneously, the SSARR model was designed for the Corps of Engineers with less complexity compared to the Stanford model (Rockwood, 1964)[49]. It uses relatively simple concepts for infiltration and the routing process to compute runoff. The HSP model (Hydrocomp, 1976)[27] represents the most advanced modification of the Stanford model with improved data handling and a channel routing process.

The G-824 program, developed by the U.S.G.S., uses 7 parameters to calibrate a rainfall-runoff model with a relatively simple concept for basin and channel routing (Carrigan, 1977)[11]. The STORM program represents the simplest and most economical continuous model. It has the capacity to evaluate storm runoff and treatment required for receiving streams (HEC, 1976)[30].

Comparisons among some of the single event models and continuous models have been made by Brandstetter (1976)[9], Abbott (1978)[1], and Williams (1979)[65]. In general, the continuous models require an enormous amount of rainfall and stream flow data to simulate a sequence of continuous storms. Usually, such data is limited and inadequate in time and space. The extensive cost and time consumption to assemble the required data by the continuous model usually make it unjustified for its major advantage: accounting for antecedent soil moisture. Therefore, the single-event model appears more practical than the continous model. Also, the Rational and Statistical Equations methods are ruled out for this study since these methods lack mathematical foundations and the capacity to evaluate the rainfall-runoff relationship, which is an important segment in urban floods.

Among the single event models, the HEC-1 program is selected to be used in this study based on the following reasons:

a) minimum data requirement, high flexibility, and easy application for computation routines,

b) algorithms accepted widely and being extensively used by the profession,

c) automatic calibration capability for parameters and sizing optimizations,

d) ability to generate runoff hydrographs, stream networks, and flood damage computations for multifloods and multiplans in a single computer run with very economical costs.

2.3 FLOOD DAMAGE ASSESSMENT

The importance of flood damage assissment has increased since the implementation of the Flood Insurance Act (1968)[23] and the Flood Disaster Protection Act (1973)[24]. It is a vital segment in the following areas: (1) operations of flood emergency plans, (2) design criteria for engineering feasibility tests, (3) guidelines for land planning and development policy.

Surprisingly, little research has been done in this field compared to other aspects of flooding (Ackermann[2], 1968). The state-of-the-art is still in a crude stage involving a lot of "guess work". The seriousness of the lack in urban flood damage data sets was described by Ackermann (1968)[2]

> ".... The contemporary absence of a satisfactory body of economic field data on urban floods constitutes a liability of monumental proportions in

the assessment of these floods and their associated damages....."

White (1964)[64] was one of the first to relate the stage with flood damages; he developed several depth-damage curves for eight different establishments at La Follette, Tennesse. James and Lee (1971)[34] established a generalized depth-damage function for residential flood damage estimation. The formula can be expressed as:

$$C_d = K_d \cdot M_s \cdot d \qquad \text{(Eq. 2.3)}$$

where Cd represents the flood damage in dollars, Ms is the market value of the inundated structure, d is the depth of flooding, and Kd is a damage factor dependent upon the structure. The Federal Insurance Administration (FIA) has been active in collecting depth-damage data. A series of depth-damage curves were derived for residential and small business structures (FIA, 1975)[19]. Flood damage was estimated according to depth of flooding and type of structure. However, besides depth of flooding , there are several other factors, such as magnitude of flow, flood frequency, velocity, and duration of flooding that may influence the outcome of flood damages. Still, the state-of-the-art in flood damage estimation lacks a more consistent method to quantify the above factors. To predict the future potential flood damage due to urbanization, the changes in land use and the resulting flood characteristics, and the variation of costs with time must be considered and included in flood damage estimation. Therefore, the efforts of this study are directed to develop a mathematical model for estimating potential flood damages due to urbanization and a methodology for demonstrating the application of this information to future land planning and development in a watershed.

Chapter III

MODEL DEVELOPMENT AND FORMULATION

The purpose of this chapter is to evaluate urban flood problems, identify the constraints which directly affect flood damages, formulate a mathematical model for estimating the potential urban flood damage, and provide methods to determine and verify variables and parameters for the model.

According to Grigg (1975)[26], flood damages can be classified into five categories:

a) Direct damages, which affect floodplain properties and their contents, infrastructures, such as roads and public utilities, and agricultural lands and spoil crops,

b) Indirect damages, which include the economic loss of business and service, the cost of safeguarding health, rerouting traffic, delays, etc.,

c) Secondary damages, which may occur when the economic loss by flooding extends further than the immediate area of flooding,

d) Intangible damages, which include the loss of life, the reduction in environmental quality and aesthetic values, and

e) Uncertainty damages, which describe hardships on floodplain occupants because of the uncertainties of flooding.

The scope of flood damage analyses in this study is limited to the category of direct damages since indirect damages are usually taken as percentages of direct damages (Kates, 1965)[36]; secondary and uncertainty damages tend to be offset by secondary benefits (Grigg, 1975)[26], and intangible damages are not feasibly measured in monetary terms though they should be included for project justification. Also, the main emphasis will be focused on floods generated by climatological events in river basins.

3.1 PROBLEM STATEMENT

Some flood damages occur when man makes use of floodplains which are susceptible to inundation. The reasons for using floodplains are well remarked by James et al. (1975)[35]:

> "...Historically, development on the floodplains along major rivers has held locational advantages for many types of industry and commerce and the constraints of low incomes and slow transportation have caused people to live near their jobs. The use of rivers for transportation and power and their attractiveness to industry and commerce as sources of water and as depositories for wastes are the important factors which made industrial and commercial development least expensive near rivers, job opportunities migrated toward riverfront cities, and residential development followed on nearby flood plains..."

Therefore, it may be concluded that flood plain invasion is the primary cause of the flood problem as shown in Figure 1. However, this problem has been compounded by the increasing rate of urbanization, and the over-reliance on some of ineffective corrective measures which may induce a false sense of security and encourage unwise new development and economic investment on floodplains.

In fact, the flood problem must be reviewed from an interdisiplinary background so that the problem and its environment can be recognized clearly, and the constraints which effect the consequences of flood damages may be identified. Figure 2 demonstrates the logic and framework which will be addressed in this chapter. By using the systematic approach as shown in this diagram, the flood problem and its environment will be treated as a whole because of their interactive aspects rather than deal with some fragmented aspect in an isolated context.

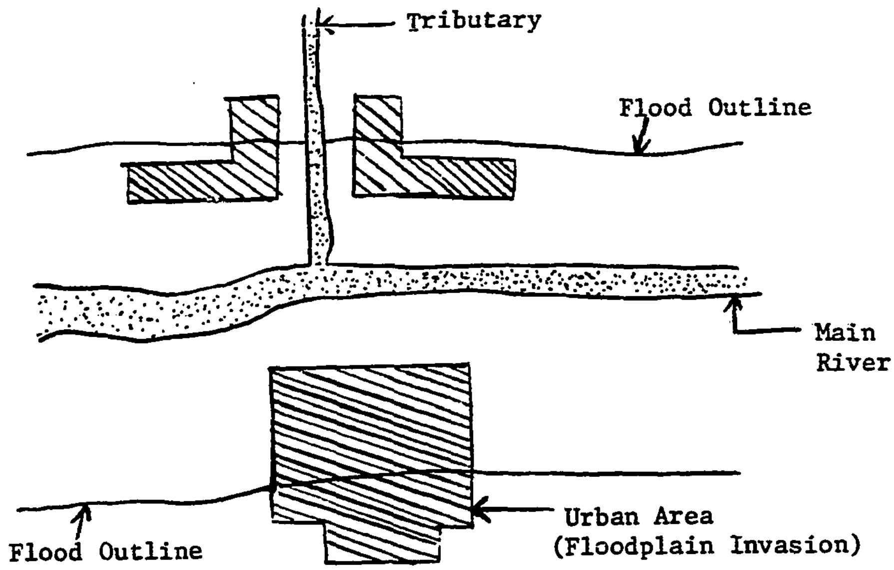
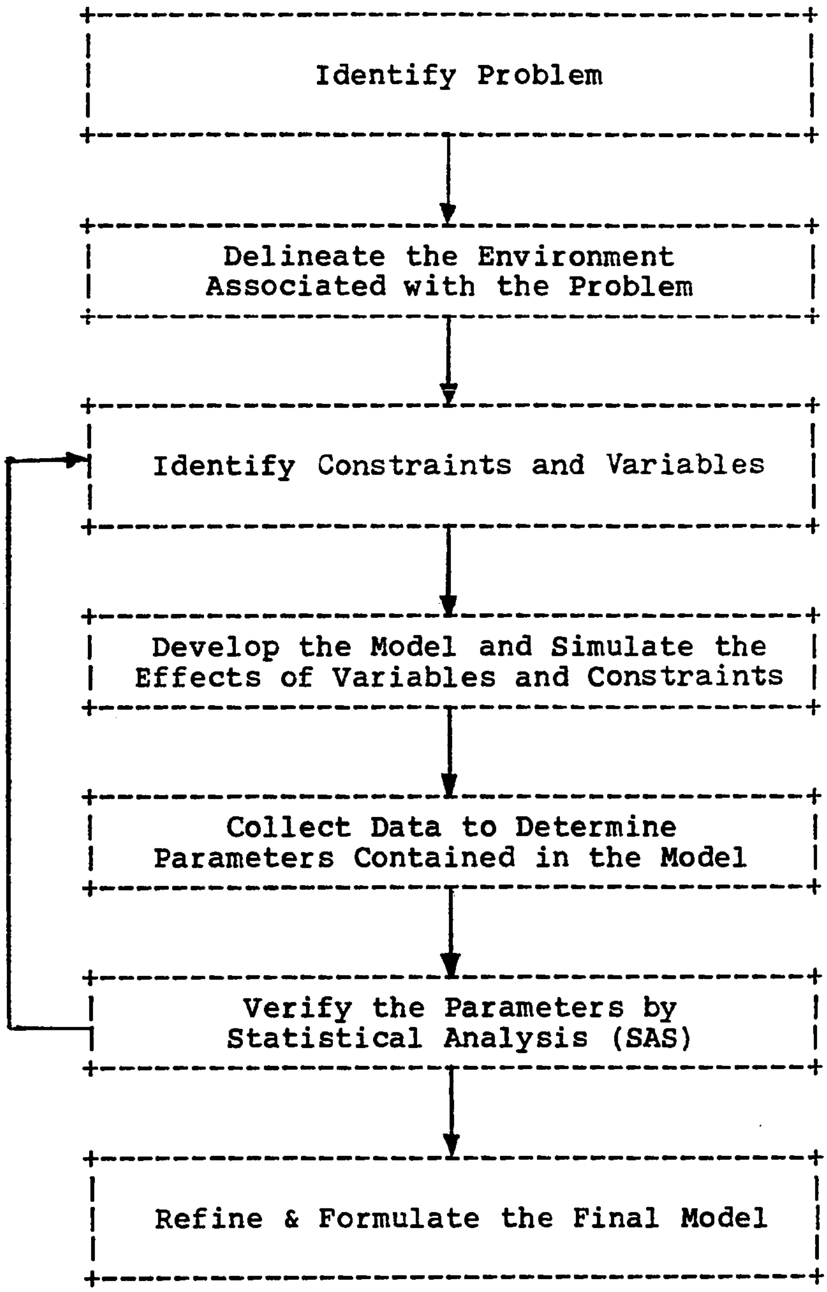


Figure 1. BASIC FLOOD PROBLEM
After: Nixon, 1966

FIGURE 2: FRAMEWORK OF MODEL DEVELOPMENT



3.2 ENVIRONMENT AND STRUCTURE

As mentioned earlier, the flood problem has been compounded by urbanization and other aspects; it needs a system to describe the environment and simulate the problem as closely as possible. Figure 3 is a structural model of a urban environment including five components that affect the flood problem. These components are described as below:

a) the physical component, which involves the physical characteristics of basin, channel, and network, including the aspects of soil, geology, topography, and land cover.

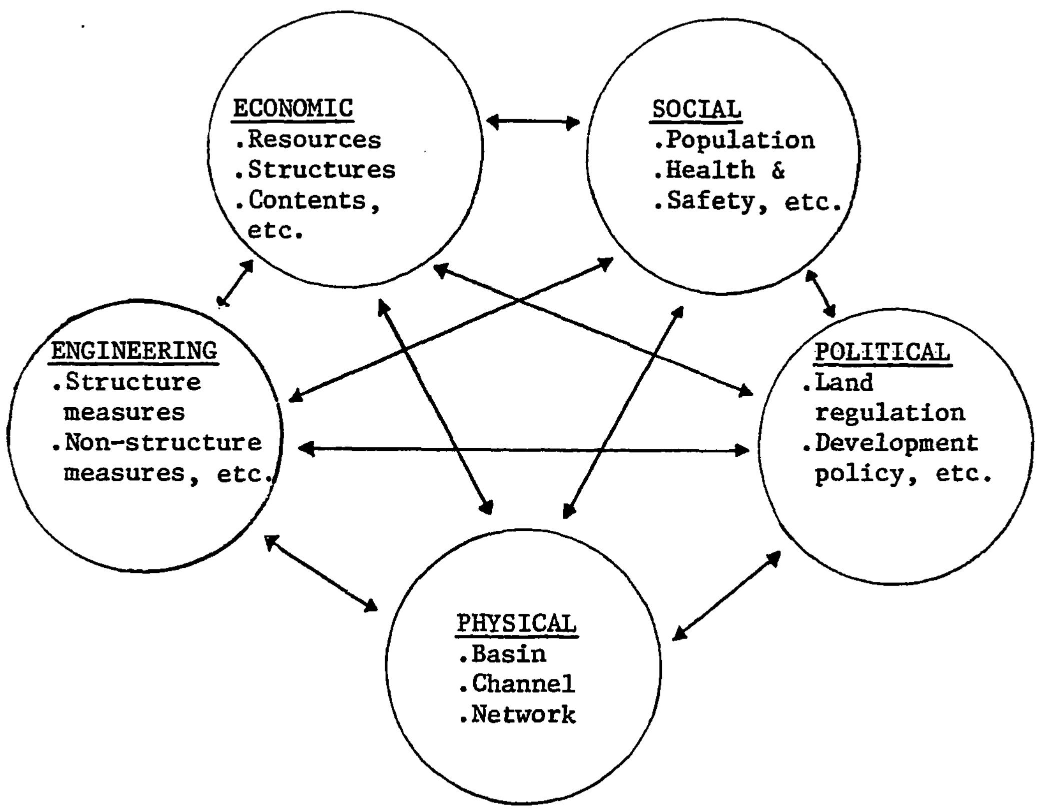
b) the engineering component, which represents the engineering works that modify the nature of basin, channel, and drainage; such as changes in surface storage, channel alteration, etc..

c) the economic component, which comprises the resources, capital investment, structures, and their contents located on floodplains.

d) the social component, which contains the human dimensions, such as population distribution on flood plains, the health and safety of flood plain occupants, and the social behaviours toward flood problems.

e) the political component, which involves the legal and administrative aspects, such as land regulations and development policies on a specific flood plain.

Figure 3. A SCHEMATIC MODEL OF THE FLOOD PROBLEM ENVIRONMENT



INPUT (Stimulus)
.Climatological event
.Time, etc.

OUTPUT (Response)
.Runoff
.Overflow
.Flood Damage

Three things are noteworthy in Figure 3. First, the INPUT represents the external parameters, such as precipitation, storm, and time, which are beyond the control of the environment. Secondly, the OUTPUT represents the flood runoff and flood damages which are affected by the input and the status of those five components. Thirdly, these five components, which include a group of factors individually, are all interrelated, interactive, and contain conflicts implicit in the problem environment. This depicts the problem that exists in reality.

In the system, each component responds to a stimulus (input) according to its intrinsic nature, but the actural stimulation it receives and its subsequent actual impact to the system is conditioned by the interaction of other components. For example, the behaviours and impacts from social and political components are influenced by the combination effects of economic, engineering and physical aspects. Similarly, the responses of economic, engineering, and physical components in the system are significantly affected and accomodated by other components. Therefore, the output (flood damage) is a dynamic, composite result from all the ingredients comprised in the environment as shown in Figure 3.

As urbanization continues on floodplains, more and more properties are placed on inundated areas with risk. It is very important that the consequences and potential flood damage can be predicted and analyzed prior to the implementation of future urban activities in order to provide a sound base for land planning and quantitative information for flood control alternative evaluation. Normally, historical damage data is applied to inundated areas where land use remains unchanged. However, as urbanization continues, the changes in land pattern and intensity with time prevent the direct use of historical damage records. Therefore, a need exists to develop a mathematical model to simulate the outcome of future potential flood damage.

3.3 VARIABLE/PARAMETER IDENTIFICATION AND MODEL FORMULATION

Recognizing the complexity of the flood problem, it is extremely difficult to measure flood damages, particularly the intangible and uncertainty damages, in a accurate and comprehensive manner. However, the scope of this study is limited to the measurement of direct flood damages. These losses are further classified into residential, commercial /industrial, amd agricultural losses according to the pattern of land-use. After carefully examining the flood problem, exploring the environment, and analyzing the inter-relationships among these five components, six factors are identified and extracted from this complex system as the constraints which have direct and signigicant impacts on future flood damages. These include:

a) Property value(Pv), which comprises the economic value measured in dollars per acre of land, structures, and their contents located on flood plains.

b) Damage cost factor(Kc), which represents the damage cost in percentages of property cost per foot of depth of flooding.

c) Intensity of urbanization(Iu), which represents the percentages of area used by urban activities on flood plains.

d) Cost index (Ci), which represents the changes in percentages of flood damages cost according to inflation.

e) Flood severity (Fs), which reprsent the severity of flooding measured by the difference of total runoff volume (Vt) and the volume of channel conveyance (Vc) in acre-feet.

f) Ratio of inundated land (Ri), which represents the fraction of area on floodplains that are inundated by excess flood waters.

Based on equation 2.2, developed by James and Lee (1971)[34], these six factors described above are assumed to have a linear relationship with the direct potential flood damage (FD). This can be expressed as:

$$FD = P_v \cdot K_c \cdot C_i \cdot I_u \cdot (V_t - V_c) \cdot R_i \qquad \text{(Eq. 3.1)}$$

where all the terms are as defined previously. The dimensions of each term are listed as :

FD = direct potential flood damage in dollars,

Pv = property value in dollars per acre,

KC = damage cost factor in % of property cost per foot of flooding water,

Ci = cost index in percentage (%), unitless,

Iu = intensity of urbanization in percentage (%), unitless,

Vt-Vc = excess volume of floodwater in acre-foot,

Ri = ratio of inundated land, unitless.

Check the consistency of units applied in equation 3.1 as follows:

(\$) = (\$/acre)(%/ft)(unitless)(unitless)(acre-ft)(unitless)

This proves that the dimensions on the left side are the same as that on the right side which indicates that this equation holds logically and consistently with related subjects. The individual parameters and variables in this equation will be further elaborated and verified in the next section.

3.4 METHODS TO DETERMINE PARAMETERS AND VARIABLES

This section describes the effects of the factors, parameters or variables, included in the model developed previously, and the procedures used to determine and verify those factors.

3.4.1 Property Value (Pv)

The total property value of a urban lot or tract varies with the size, type of structure, content of structure, and location of the lot. In general, Pv can be estimated as:

$$P_v = L + S + C \qquad \text{(Eq 3.2)}$$

where L, S, and C represent the market value for land, its structure and contents of the structure, respectively. The land value is normally computed on a front-foot basis for urban area or on acreage basis in open country (Chapin, 1965). Since urban activities are the main concern, the land value is estimated in order to combine the structure value on a square-foot basis.

Types of structures are classified into two major categories: residential and commercial/industrial. The residential catagory is further divided into four groups: (1) single family, (2) duplex, (3) multi-family unit, and (4) mobile home, whereas commercial/industrial category includes four major types of structures as classified according to the construction permits authorized by the Community Development Department of Oklahoma City. These types are shopping centers, offices, and other commercial and industrial structures. The maket values of the major type structures as mentioned above can be estimated by conducting a survey through the following sources: Census of Population and Housing, Research and Economic Development Division, urban construction contractors, and realtors. Then, the average total structure value for both categories is estimated based on the square-foot price by multipling the number of structures of each type per acre by the market value of each type surveyed from samples and summing up the different types of structures by using a weight factor. This relationship is shown as:

$$S_t = \Sigma W_i \cdot S_i \cdot N_i \;/\; \Sigma W_i \qquad \text{(Eq. 3.3)}$$

where, St = the average total structure value in dollars per acre,

Wi = the relative weight for structure type i in percentages,

Si = the average structure value of type i in dollars per unit,

Ni = the maximum dwelling units of structure type i per acre of urban land.

For the building content value, it is usually taken as a percentage of the structure value. For example, a Federal agency used an assumption of 32% of the structure value to compute the content value in conducting a flood study (Grigg, 1975)[26]. Herein, 30% of the structure value is assumed to compute the content value for residential, commercial, and office buildings. However, the content value of industrial buildings may vary significantly with types of industry. A survey census was conducted for manufacturing and industries, and county business patterns (U.S. Department of Commerce, 1972, 1977, 1980) to identify the major groups of industry. Then, the total content value was calculated as the weighted average of measurements of the major industrial groups surveyed from samples. This can be computed as:

$$CI_t = \Sigma W_i' I_i / \Sigma W_i' \qquad \text{(Eq. 3.4)}$$

where, CI = the average total industrial content value.

Ii = the content value of industry type i.

Wi' = the relative weight for industry type i.

Tables 1, 2, and 3 represent the estimated results for residential and industrial/commercial market values surveyed in the Oklahoma County area.

Table 1.

ESTIMATION OF RESIDENTIAL PROPERTY VALUE

Type of Structure	Ni (unit/acre)	Si ($/unit)	Wi (%)
Single Family	4	60,000	7o
Duplex	10	46,000	15
Multi-Family	15	40,000	10
Mobile Home	10	32,000	5

Resulting Estimations: St = 313,000 $/acre

Pv = (1+30%)St = 407,000 $/acre

Note: Ni, Si, Wi, and St were defined in eq 3.3

Table 2.

ESTIMATION OF INDUSTRIAL CONTENT VALUE

Type of Industry	Ii (k $/unit)	Wi (%)
General Service	300	31
Retail	260	26
Whole Sale Trade	500	10
Contract Construction	400	10
Manufacturing	1540	6
Machinery	500	5
Transportation	2000	3
Furniture Stores	300	5
Mining	1500	5

Resulting Estimations: CIt = 548,000 $/unit industry

* CIt'= 27,400/acre

* averaging of 15314 industry units established in Oklahoma County

Note: Ii, Wi, and CIt were defined in eq 3.4

Table 3.

ESTIMATION OF COMMERCIAL/INDUSTRIAL PROPERTY VALUE

Type of Structure	Ni (#/acre)	Si (K $/#)	Wi (%)	Ci (vary)
Shopping Center	0.50	805	15	30% of Si
Office	1.00	420	37	30% of Si
Other Commertials	1.00	189	28	30% of Si
Industry	0.20	241	20	table 2

Resulting Estimation: Pv = 386,000 $/acre

Note: Ni, Si, Wi, and Ci were defined in eq 3.3 & 3.4

3.4.2 Damage Cost Factor

Several federal agencies have proposed a series of depth vs. damage curves (TVA, 1969;[54] USACE, 1970;[59] FIA, 1970;[20] USDA, 1970)[57]. These results are scattered due to diverse sources of data. However, the revised depth-damage relationship developed by the Federal Insurance Administration (FIA) appears to be most resonable because the results have been based on a substantial data base (Grigg, 1975)[26]. As a result, damage is exhibited as percentages of market value according to the classification of structure type. Damage increases as the depth of flooding increases for each type of structure. In order to find the relationship between percentages of damage and depth of flooding, a proposed linear model is:

$$Y = a + K_c \cdot X \qquad \text{(Eq. 3.5)}$$

where, Y = damage in percentages of structure value,

X = depth of flooding in feet,

a = interceptor,

Kc= percentages of damage per foot of flooding.

A regression analysis was performed by using the statistical analysis system program (Barr, 1976)[5]. The results are shown in Table 4. Except for curve 10, which represents mobile homes, the other types of structures exhibit high values of R-square (R^2), and very low values of the significance probability for Model F (PR > F). This indicates a very good fit for the model proposed by equation 3.5. Kc values are fairly consistent among all types of structures except for mobile home. Therefore, a general model is attempted for all types of structures excluding mobile homes. The results from regression analysis yield:

$$R^2 = 0.857, \ PR>F = 0.0001, \text{ and } Kc = 2.91$$

Statistically, R^2 ranges from 0 to 1. The higher the R^2 value, the better the fit of the model. A value of 0.857 indicates a fairly good fit for the proposed model. Also, the F value and PR>F value from SAS output are equivalent to the results of a t-test for testing the hypothesis that the regression parameter equals zero. A very small value of PR>F, such as 0.0001 in this case, implies that the parameter (Kc) is not likely to be zero, and the independent variable (X, depth of flooding) contributes significantly to the model. Therefore,the Kc value resulting from the general linear model is again verified to be acceptable for all types of structures except for the moble home which was determined separately as shown in Table 4.

Table 4.

RESULTS OF DEPTH-DAMAGE REGRESSION ANALYSIS

Structure	R^2	F value	PR > F	Kc (%)
curve #1	0.873	89.80	0.0001	3.14
curve #3	0.969	539.85	0.0001	2.55
curve #5	0.911	143.62	0.0001	3.03
curve #13	0.967	380.29	0.0001	3.66
curve #18	0.949	321.36	0.0001	2.86
curve #23	0.961	364.91	0.0001	3.30
curve #10	0.750	74.96	0.0001	11.07
average except #10	0.857	591.57	0.0001	2.91

where:

curve #1 = one story, no basement

curve #3 = two or more stories, no basement

curve #5 = split level, no basement

curve #10 = mobile home with foundation

curve #13 = one story with basement

curve #18 = two or more stories with basement

curve #23 = split level with basement

R^2 = ratio of the sum of squares fit the model divided by the sum of corrected squares

F = ratio of the mean square of model divided by the mean square of error

PR>F = the significance probability when the parameter equals zero

Kc = damage cost factor

3.4.3 Cost Index (Ci)

Inflation occurs when the dollar value shrinks with time. The question may be raised as to how inflation affects potential direct flood damages. It is anticipated that the changes of land-use pattern and intensity with time may change the values of land and structures. It is not a straight forward issue because it involves several variables and complex interactions between variables. In general, the land value intends to appreciate as time goes on, while the structure value intends to depreciate with time. If these two effects are assumed to be offset by each other, the changes in service costs with time, such as repairment for flood damages, will still rise as a result of inflation. In order to account for the effect of inflation with time, the cost index (Ci) is included in the flood damage model (Eq. 3.1) to accomodate the changes of cost for potential flood damages.

The consumer price index (CPI) for 1967 to 1982 was selected as the data base to compute Ci, since the CPI was surveyed and compiled based on diverse sources and substantial data. The CPI has been revised and converted to the reference base for the year 1967 equivalent to 100 in compliance with recommendations of U.S. office of Management and Budget (Bureau of Labor Statistics, 1982)[10]. For the purpose of finding the correlation between the cost index with time, several hypotheses were proposed, and the regression analysis was performed. Finally, a geometrical progression relationship between CPI and time (in years) was tested as:

$$CPI = b \cdot (1+r)^{\Delta t} \qquad \text{(Eq 3.6)}$$

This can be converted to a linear relation through a logarithmic transformation as:

$$\mathrm{Log}(CPI) = \mathrm{Log}\, b + (\Delta t) \cdot \mathrm{Log}(1+r) \qquad \text{(Eq 3.7)}$$

where, Δt = year difference from 1967

r = inflation rate per year

b = a correlation constant

The regression analysis yields the following results:

$r = 0.08$, $b = 0.980$, and $PR>F = 0.0001$

These features strongly support that the cost index increases as a geometrical progression rate with time, and the inflation rate yearly has been computed to be around 8%.

These findings can be used to project the future cost index as:

$$CI_{t12} = \frac{CPI_{t2}}{CPI_{t1}} = (1+r)^{t2-t1} \qquad \text{(Eq 3.8)}$$

where subscripts 1 & 2 represent the current year t1 and future year t2; CI_{t2} is the cost index for future year t2 based on current year t1 price. For example, the cost index for year 2000 can be projected as:

$$CI_{2000-1983} = (1+r)^{2000-1983} = 19.6$$

Therefore, the future potential flood damage can be estimated to account for the effect of inflation with time.

3.4.4 Intensity of Urbanization (Iu)

Urbanization is a characteristic of our time. It may be regarded as the conversion of rural areas to cities and suburban communities resulting in human activies involving changes in land occupancy and use (Chapin, 1965)[12]. According to the United Nation Water Conference (Lindh, 1977)[39]:

> "....On a world-wide scale, the total growth during this century has been accompanied by a continuous increase in the ratio of urban to rural dwellers, and it is expected that by the year 2000, half of the world's population will be urban. Among the obvious effects of the migration to urban areas are increased population density, and increased density of residential, industrial and commercial buildings. Paradoxically, the land occupied by urban population is only a small fraction, often less than 5%, of the total land area."

Indeed, the concentration of population and urban activities in small areas intensifies urban land use. As urbanization continues on floodplains, there is more property invesment with risk of flooding which will result in the increase of potential flood damages. Therefore, the intensity of urbanization (Iu) is selected as one of the variables which effect the outcome of potential flood damages.

Iu can be computed by taking the ratio of the space devoted to infrastructural and urbanal activities to the total land area for a particular tract of land. To provide an advantage of convenience for future land planning and development, the relationship between the intensity of urbanization and population density (Pd) are further investigated based on census of population (1970-1980) and land areas surveyed according to census tracts within the Oklahoma City area (Research and Economic Development Division, OKC, 1982).[46] The correlation between the intensity of urbanization and population density for 37 tracts located in Oklahoma City is tested by the regression analysis. The results from the SAS program yield the following relationship with a R^2 value of 0.95 and PR>F of 0.0001.

$$I_u = 0.011(P_d) \qquad \text{(Eq. 3.9)}$$

where Iu is the percentage of devoloped urban land, and Pd denotes the population per square mile.

Though the relationship between Iu and Pd was only tested locally, a similar function may be established for other areas as:

$$I_u = k \cdot P_d \qquad \text{(Eq. 3.10)}$$

where k represents a correlation constant. This relationship provides a quantitative manner to estimate the urban land which may be developed in the future based on population projections.

3.4.5 Flood Severity (Fs)

An essential step in estimating flood damages is estimating the severity of the flooding produced by a given flood. Before proceeding further, it is important to define this subject first. Different people view flooding differently because floods are complex phenomena. According to Chow (1956)[13], "A flood is a relatively high flow which overtaxes the natural channel provided for the runoff." A more general definition was provided by Ward (1978)[62] as "A flood is a body of water which rises to overflow land which is not normally surbmerged.". On the whole, floods always imply damages on inundated land. Floods can result from a number of basic causes, such as climatological events, coastal storm surges, streamflow and tidal interaction, earthquakes, landslides, and other phenomena (Ward, 1978)[62]. It can occur on riverine areas as well as coastal areas. The main emphasis herein will be on river floods generated by climatological events, predominantly rainfall.

A flood hydrograph as depicted in Figure 4 provides a good perception of a given flood. This diagram presents an example of a flood hydrograph which traces the magnitude of discharge against time during a flood event. It contains three major parts: rising limb, crest segment, and falling limb. Td, Tc, and Tb denote time of storm duration, time of concentration, and base time of the given flood respectively. P signifies the peak discharge. The area covered by the section APB yields the total runoff volume; whereas the remaining area contributes to the baseflow. The shape of the flood hydrograph, which provides a good insight for flood severity, may be modified by the climatological input as well as the variations of basin and channel characteristics. For example, the hydrograph may have a low peak corresponding to a prolonged-time base for a sluggish stream; in contrast, a flashy stream may have a high peak and a short-time base resulted from a same flood event as shown in Figure 4.

Several variables can be used to measure the severity of flooding. Parker, et al, (1972)[47] suggested that stage, frequency, discharge rate, duration, and velocity have major effects on the severity of flooding. Among them, stage and discharge rate are most commmonly used as indices to measure flood severity.

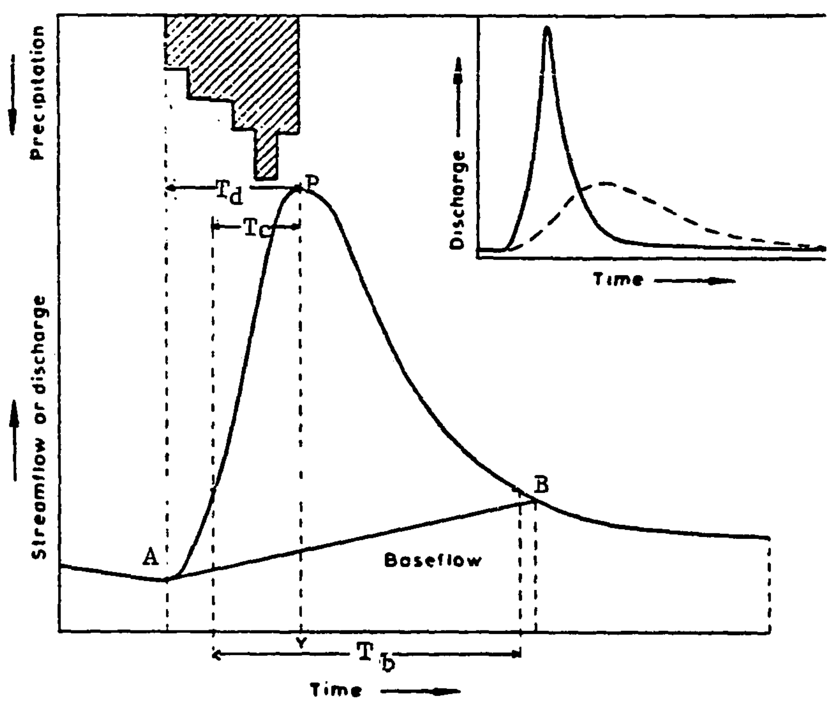


Figure 4. RIVER FLOOD HYDROGRAPH: Inset Hydrograph on flashy and sluggish streams

* Source: Hoyt and Langbein, 1955

Water stage has the advantage that it may be measured directly; while discharge rate can provide a basis to predict the flood magnitude. However, the total severity depends on the areal extent of flooding to each depth which varies with the topography of floodplains, as well as duration and velocity. The discharge rate reflects velocity implicitly, but it does not include flood duration. In fact, a more comprehensive way to measure flood severity would be the quantitative difference between the total runoff volume (Vt) and the channel conveyance (Vc). The former, Vt, reflects the magnitude of flood, velocity, and duration since it is computed by integrating the direct runoff discharge rate with flood duration. The channel conveyance (Vc) denotes the carrying capacity of a stream channel for a period of traveling time. It reflects the volume and the velocity of floodwater in a channel. The difference between Vt and Vc is the actual volume of flood water which overtops the banks of a channel and inundates adjacent land.

In summary, the severity of flooding can be measured as:

$$V_e = V_t - V_c \qquad \text{(Eq. 3.11)}$$

Where Ve is the excess volume of floodwater overflowing land in acre-foot; Vt and Vc are the same as previously defined. Besides the merit of taking account of the effects of depth, magnitude, velocity, and duration, this way of measuring flood severity actually allows two variables, Vt and Vc, to be modified simultaneously. For example, the flood severity of a given flood can be measured for a stream with the natural channel condition compared with other conditions of alternatives in which Vt and Vc can be altered with the changes of surface storage or channel capacity. Therefore, this method has the advantage to provide a sound basis for alternative evaluation and selection concerning the severity of flooding. The HEC-1 and the HEC-2 (Water Surface profile,1982)[33] programs can be used to compute Vt and Vc, respectively. The detail procedures will be described in chapter IV.

3.4.6 Ratio of Inundated Land (Ri)

The nature of the flood is closely related to the physical characterisics of basin, channel, and channel network in the following aspects: topography, soil, geology, and land cover. As illustrated in Figure 5, some characteristics are relatively stable and others are very comparatively variable. The effects of variable characteristics are very complex due to the interactions between climate, soil, geology, vegetation cover and man's influence. However, the interactions of these complex variables can be modeled and treated by using HEC-1, and the outcome, as expected, may have some effects on peak discharge, time of concentration, total runoff volume, and/or channel conveyance.

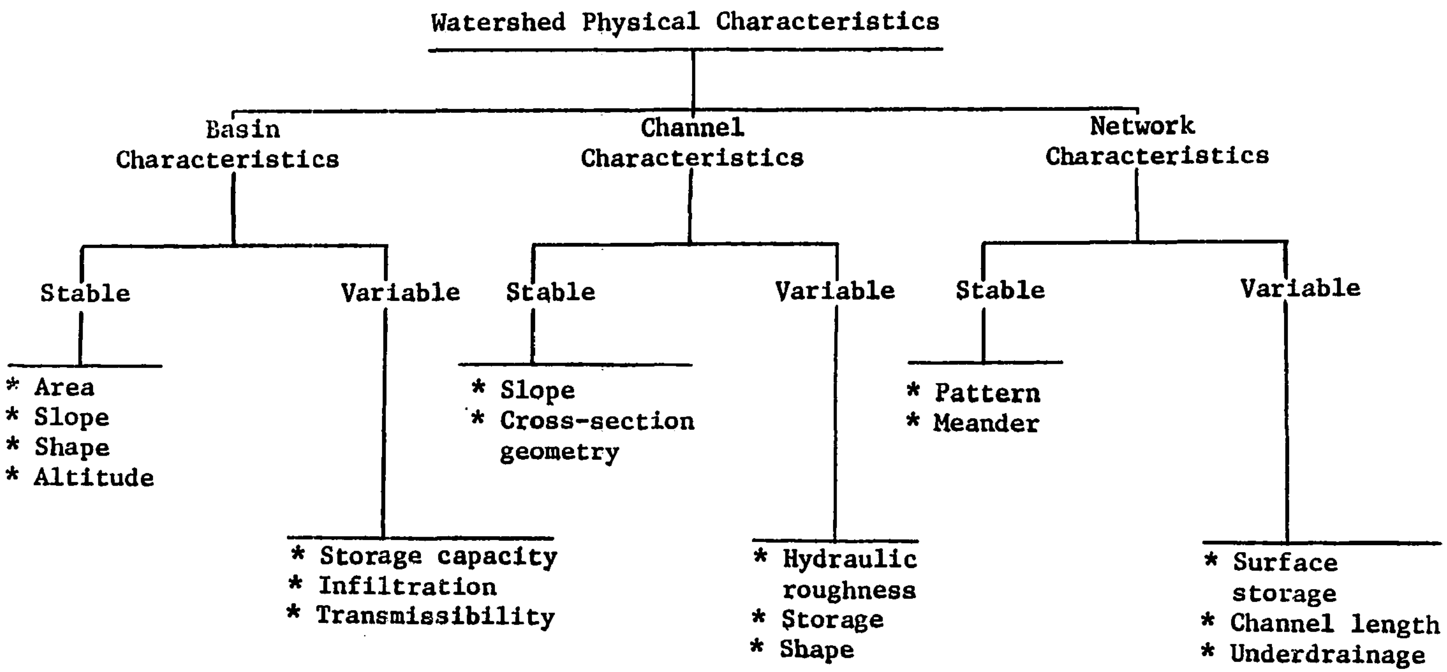


Figure 5. PHYSICAL CHARACTERISTICS OF A WATERSHED

Source: After Ward, 1978

The stable characteristics are considered relatively unchanged. They are more related to the aspect of topography, such as area, slope, length, width, and shape of a basin. Among them, area is probably the most important element because it affects the time of concentration and the total volume of runoff generated by a given flood event. As has been noted, not all the total volume of runoff overflows floodplains since the channel carries a portion of floodwater. The fraction of inundated area (Ri) during a flood event depends on the width and the length of path where the floodwater travels. In order to identify and determine Ri, the investigation starts with channel cross section and floodplain outline as depicted in Figure 6.

First, the conventions used in this diagram are defined as follows:

Ti = the top width of floodplain at cross section i,

Bi = the channel bank width at cross section i,

Yi = the depth difference between the flood surface elevation and the channel bank surface level at cross-section i,

Sl/Sr = the slopes of left/right side of overbank which are equivalent to the values of 1/Zl and 1/Zr,

ΔAli/ΔAlr = the portions of cross-sectional area on left/right sides of overbank land at cross-section inundated by a given flood,

Ai' = the channel cross section area at cross section i,

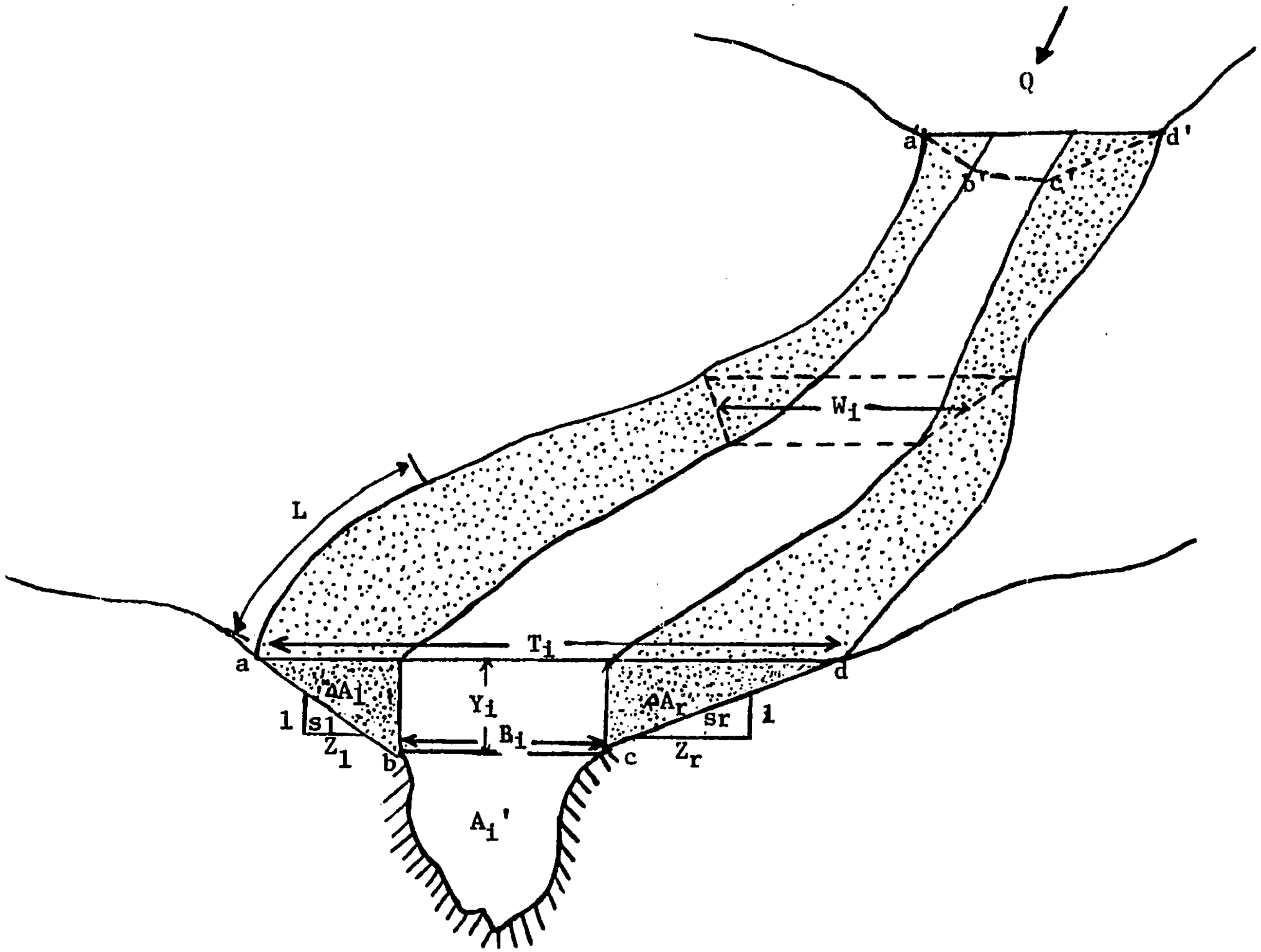


Figure 6. CHANNEL AND FLOODPLAIN CROSS SECTION PROFILE

A_i = the trapezoidal area bound by denotions a, b, c, and d,

ΔL_i = the channel length between cross section i-1, and cross section i.

W_i = the average width of T_i and B_i.

Furthermore, the following conditions are assumed herein:

a) Channel has a rigid boundary.

b) The geometry of channel and the slope of overbank land remains fairly consistent between cross sections.

c) The channel encroachment is "squeezed" to the boundary of channel banks which is the maximum allowance according to the FIA floodway regulation.

According to Figure 6, the following relationships exist for any cross-section i in the channel.

$$A_i = (0.5)\cdot(T_i + B_i)\cdot Y_i \qquad \text{(Eq. 3.12)}$$

$$\Delta A_{li} + \Delta A_{ri} = A_i - (B_i)(Y_i) = 0.5(T_i - B_i)Y_i \qquad \text{(Eq. 3.13)}$$

Also, Ti can be computed as:

$$T_i = B_i + (Z_l + Z_r)\cdot Y_i \qquad \text{(Eq. 3.14)}$$

Which yields,

$$Y_i = (T_i - B_i) / (Z_l + Z_r) \qquad \text{(Eq. 3.15)}$$

Combining the above equations results in:

$$\frac{\Delta A_{li} + \Delta A_{ri}}{A_i} = \frac{T_i - B_i}{T_i + B_i} \qquad \text{(Eq. 3.16)}$$

The excess volume of floodwater (Ve) for a given flood event can be determined by taking triple integral of the depth (Y), the width (W), and the length (L). It can be denoted as:

$$V_e = \iiint f(W \cdot Y \cdot L) dW \cdot dY \cdot dL \qquad \text{(Eq. 3.17)}$$

If the channel is assumed to be evenly subdivided to m cross-sections, then Ve can be found by substituting and extending the above equation to the case of discrete segments.

$$V_e = \Sigma W_i \cdot Y_i \cdot \Delta L_i = \Sigma 0.5(T_i + B_i) \cdot Y_i \cdot \Delta L_i \qquad \text{(Eq. 3.18)}$$

Combining with equation 3.12, yield the following relationship:

$$V_e = \Sigma A_i \cdot \Delta L_i \qquad \text{(Eq. 3.19)}$$

By the same token, the volume of floodwater actually inundates floodplain (Vi) can be computed as:

$$V_i = \Sigma(\Delta A_{li} + \Delta A_{ri}) \cdot (\Delta L_i) \qquad \text{(Eq. 3.20)}$$

The ratio of Vi to Ve yields Ri as:

$$R_i = \frac{V_i}{V_e} = \frac{\Sigma(\Delta A_{li} + \Delta A_{ri}) \Delta L_i}{\Sigma \; (\Delta A_i)(\Delta L_i)} \qquad \text{(Eq. 3.21)}$$

Since the cross-sections are assummed to be evenly divided, the above equation can be simplified to:

$$R_i = \frac{\Sigma(\Delta A_{li} + \Delta A_{ri})}{\Sigma \; A_i} \qquad \text{(Eq. 3.22)}$$

Combining with equation 3.16,

$$R_i = \frac{\Sigma(T_i - B_i)}{\Sigma(T_i + B_i)}, \quad \text{or} \quad R_i = \frac{\bar{T}_i - \bar{B}_i}{\bar{T}_i + \bar{B}_i} \qquad \text{(Eq. 3.23)}$$

Where $\bar{T}i$ and $\bar{B}i$ represent the average weighted values of Ti and Bi, respectively. Bi or $\bar{B}i$ can be obtained from field surveys, while Ti or $\bar{T}i$ can be determined by using the HEC-2 program to compute water surface profiles. The detail will be elaborated in chapter IV.

As assumed earlier, the maximum encroachment for urban land is the boundaries of the channel banks. This implies that the floodway is delineated evenly with channel banks which yields a maximum area for urban use on floodplains. In general cases, the Bi value represents the width of the floodway at cross-section i, and it can be determined by using the HEC-2 program as it will be discussed in the next chapter.

Chapter IV

METHODOLOGY

This chapter describes the methodology and integrated programs involved in the application of the flood damage model which was developed in the previous chapter. It uses the HEC-1 program to simulate basin hydrologic processes, employs the HEC-2 program to model channel hydraulic characteristics and to compute water surface profiles, and ultilizes the expected annual damage computation, featured in HEC-1, as the basis for flood damage economic analysis. This integrated approach can be very useful in evaluating scenarios and alternatives concerning future land use and flood control projects. In fact, this chapter delineates the following aspects: (1) basin hydrologic simulation with HEC-1, (2) water surface profiles and floodway encroachment using HEC-2, (3) expected annual damage computation, and (4) summary of the methodology.

4.1 BASIN HYDROLOGIC SIMULATION WITH HEC-1

A powerful feature of HEC-1 is its capability to model the flood runoff from a single storm event for complex river basins. The modeling includes describing the topographic structure of the basin, organizing the logic network between subbasins and stream channels, defining the parameters, and simulating the rainfall-runoff response of the basin. Rainfall is computed based on lumped basin parameters including loss rate, unit hydrograph, and routing criteria. The algorithm to compute the runoff volume (Vt) and flood hydrographs using HEC-1 is illustrated in Figure 7, in which the runoff is generated by transforming the rainfall excess to a unit hydrograph through the processes of routing and combining local subbasins to the basin outlet.

```
+---------------------------------------+
|      Basin Topographic Structure      |
+---------------------------------------+
|     *subbasin areas                   |
|     *drainage boundary                |
|     *channel layout                   |
+---------------------------------------+

+---------------------------------------+
|         Basin Average Rainfall        |
+---------------------------------------+
|     *historical events                |
|     *hypothetical events              |
|        ~specific frequency            |
|        ~stand project storm (SPS)     |
|        ~probable maximum storm (PMS)  |
+---------------------------------------+

+---------------------------------------+
|         Rainfall Average Losses       |
+---------------------------------------+
|     *infiltration                     |
|     * base flow                       |
+---------------------------------------+

+---------------------------------------+
|             Rainfall Excess           |
+---------------------------------------+
|     *rainfall minus losses            |
+---------------------------------------+

+---------------------------------------+
|              Unit Hydgraph            |
+---------------------------------------+
|     *given unit graphs                |
|     *synthesized by Clark, Snyder, or |
|      SCS method                       |
+---------------------------------------+

+---------------------------------------+
|        Direct Runoff Hydrograph       |
+---------------------------------------+
|     *routing process                  |
|     *combining local inflows          |
+---------------------------------------+
```

FIGURE 7: ALGORITHM OF RAINFALL-RUNOFF MODEL USING HEC-1

4.1.1 Modeling Processes

The essential processes involved in basin hydrologic modeling are described briefly as below (HEC-1 manual, 1981)[31].

4.1.1.1 Topography

The topographic structure of the basin is modeled in the program by defining the channel network and routing reaches as shown in Figure 8. Hydrographs are computed, routed, and combined in accordance with the data sequence provided. In this manner any complex basin comprising large number of subbasins and reaches can be simulated rationally and accurately.

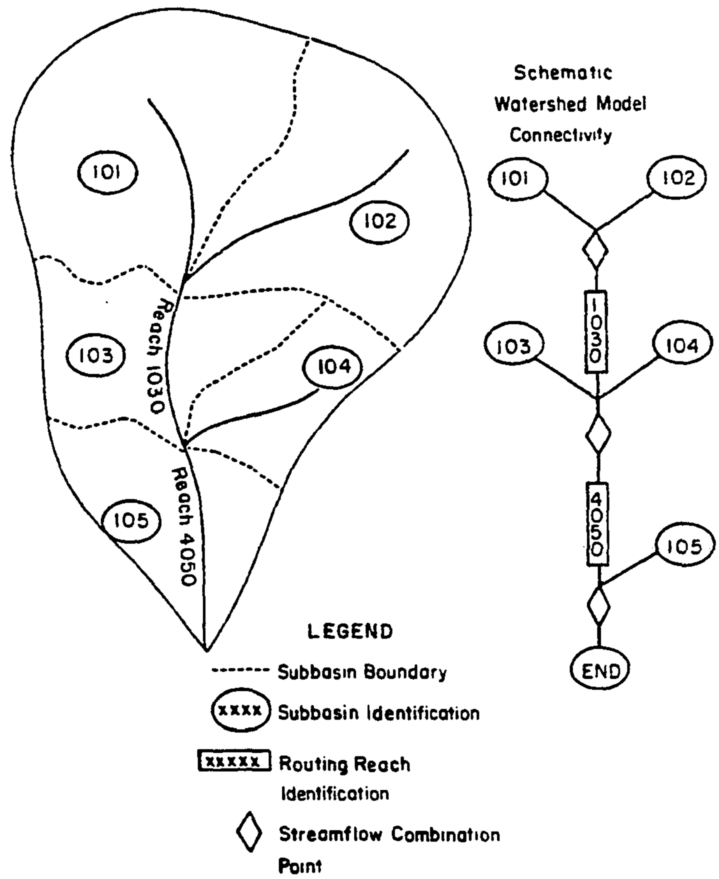


Figure 8. <u>TYPICAL HEC-1 WATERSHED MODEL COMPONENTS</u>

* Source: After Feldman, 1981

4.1.1.2 Precipitation

There are several forms of precipitation. Herein, rainfall is considered to be predominant and the terminology is used throughout this study to refer to precipitation. In general, two types of rainfall data can be used as input data for HEC-1: historical events and hypothetical events. The historical storm data may be obtained from weather stations and local goverment agencies; the subbasin total precipitation can be computed according to weights provided from each station or specified as an average total precipitation with a temporal pattern for distributing the total precipitation. For hypothetical storms, the program can compute automatically the Standard Project Storm (SPS) using the criteria developed by the Corps of Engineering (1952)[58], the Probable Maximum Precipitation (PMP) using criteria developed by the National Weather Service (NWS, 1956)[42], and synthetic storms for specific frequencies using data developed by the NWS (Hydro-35, 1977[44]; TP-40, 1961)[43]. Thus, precipitation data, historical or hypothetic, can be supplied directly or computed in the program to simulate various storms.

4.1.1.3 Loss Rate

There are four techniques available in HEC-1 to compute precipitation loss rate, namely: (1) initial and constant method, (2) HEC exponential method, (3) SCS method, and (4)

Holtan method. In this study the SCS method (S.C.S., 1975)[53] is selected because it has the advantage to signify the combined effects of soil type, vegetation, land use, and antecedent soil moisture conditions.

The curve number (CN) values can be estimated from Table 5 developed by the Soil Conservation Service (SCS) based on extensive experiments. As illustrated in that table, CN values vary with soil group, land use, and land treatment. Also, the CN value is affected by the antecedent moisture condition as shown in Table 6. This can be used to adjust the CN values according to the moisture condition specified. The CN values for urban areas, anticipating future developments, also can be estimated and modified based on categories of land use and degrees of imperviousness as shown in Figure 9. To compute the composite runoff CN value for each subbasin, a weighted average method is used as:

$$CN = CN_i \cdot W_i \ / \Sigma \ W_i \qquad \text{(Eq. 4.1)}$$

where, CN = weighted composite CN value in a subbasin,

CNi = CN value for certain land use type i,

Wi = relative weight of area in percentages for land use i,

Thus, changes of rainfall loss due to changes of land use in either pattern or intensity can be accounted through modifying CN values.

Table 5. RUNOFF CURVE NUMBER OF HYDROLOGIC SOIL COVER COMPLEX

LAND USE DESCRIPTION		HYDROLOGIC SOIL GROUP			
		A	B	C	D
Cultivated land[1/]: without conservation treatment		72	81	88	91
: with conservation treatment		62	71	78	81
Pasture or range land: poor condition		68	79	86	89
good condition		39	61	74	80
Meadow: good condition		30	58	71	78
Wood or Forest land: thin stand, poor cover, no mulch		45	66	77	83
good cover[2/]		25	55	70	77
Open Spaces, lawns, parks, golf courses, cemeteries, etc.					
good condition: grass cover on 75% or more of the area		39	61	74	80
fair condition: grass cover on 50% to 75% of the area		49	69	79	84
Commercial and business areas (85% impervious)		89	92	94	95
Industrial districts (72% impervious).		81	88	91	93
Residential:[3/]					
Average lot size	Average % Impervious[4/]				
1/8 acre or less	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
Paved parking lots, roofs, driveways, etc.[5/]		98	98	98	98
Streets and roads:					
paved with curbs and storm sewers[5/]		98	98	98	98
gravel		76	85	89	91
dirt		72	82	87	89

[1/] For a more detailed description of agricultural land use curve numbers refer to National Engineering Handbook, Section 4, Hydrology, Chapter 9, Aug. 1972.

[2/] Good cover is protected from grazing and litter and brush cover soil.

[3/] Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.

[4/] The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

[5/] In some warmer climates of the country a curve number of 95 may be used.

From: National Engineering Handbook, Section, Hydrology, 1972

Table 6.

CN VALUES MODIFIED WITH ANTECEDENT MOISTURE CONDITION

CN for Condition II	Corresponding CN for Conditions I	III
100	100	100
95	87	99
90	78	98
85	70	97
80	63	94
75	57	91
70	51	87
65	45	83
60	40	79
55	35	75
50	31	70
45	27	65
40	23	60
35	19	55
30	15	50
25	12	45
20	9	39
15	4	33
10	2	26
5	2	17
0	0	0

where:

Condition I: soils are dry but not to wilting points; satisfactory cultivation has taken place.

Condition II: average conditions.

Condition III: heavy rainfall, or light rainfall and low temperatures have occurred within the last 5 days; saturated soil.

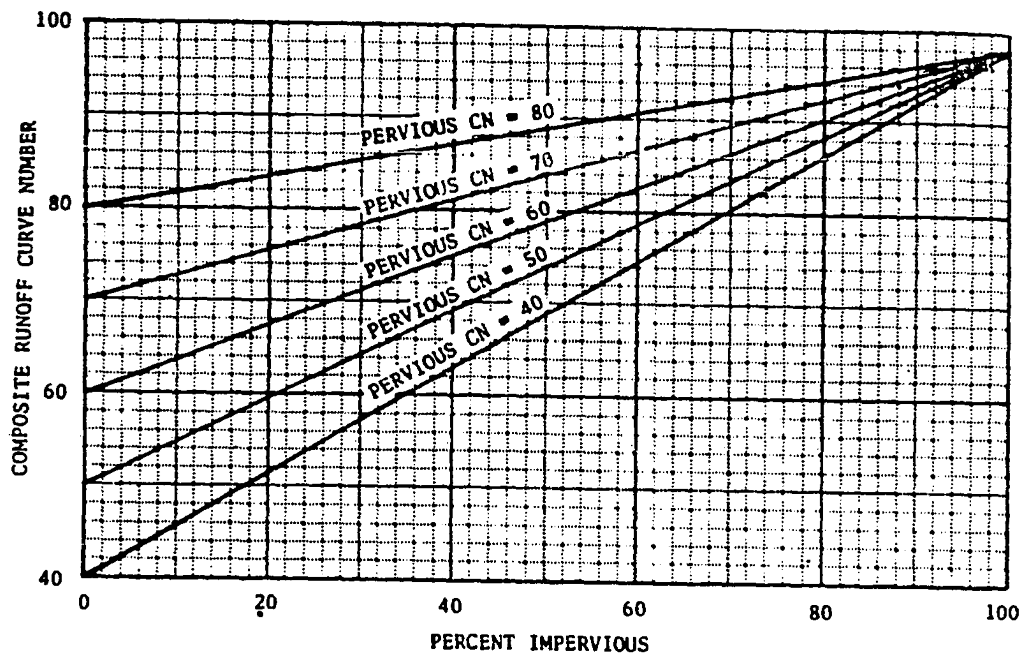


Figure 9. PERCENTAGE OF IMPERVIOUS AREAS VS. COMPOSITE CN's FOR GIVEN PERVIOUS AREA CN's

Source: National Engineering Handbook, Section 4, 1972

4.1.1.4 Unit Hydrograph

A unit hydrograph is defined as the hydrograph that results from one inch of excess rainfall during a particular duration of storm as shown in Figure 10. It can be input directly to the program or synthesized from supplied parameters. Three techniques are available to synthesize the unit hydrograph: Clark, Snyder, and SCS methods.

The Clark method has advantages of directly computing unit hydrographs and a means of adjusting for changes in drainage patterns through the modification of parameters. There are two parameters required in using the Clark method: (1) the time of concentration (Tc), known as the travel time of water from the most upstream point to the downstream outflow location, and (2) the storage coefficient (R), described as the subbasin natural attenuation characteristics. The Clark method translates incremental instantaneous runoff from subbasin to the basin outlet according to the travel time of overlands, then routes the runoff through a linear reservoir to account for the basin storges. The instantaneous routed runoffs are then averaged to produce the unit hydrograph. Both Tc and R can be estimated by using charts, graphs, or empirical formulae (Linsley, 1975;[40] Thomas 1975)[55]. However, HEC-1 has the capability to optimize Tc and R, and determines the "best-fit" unit hydrograph by automatic calibration of parameters.

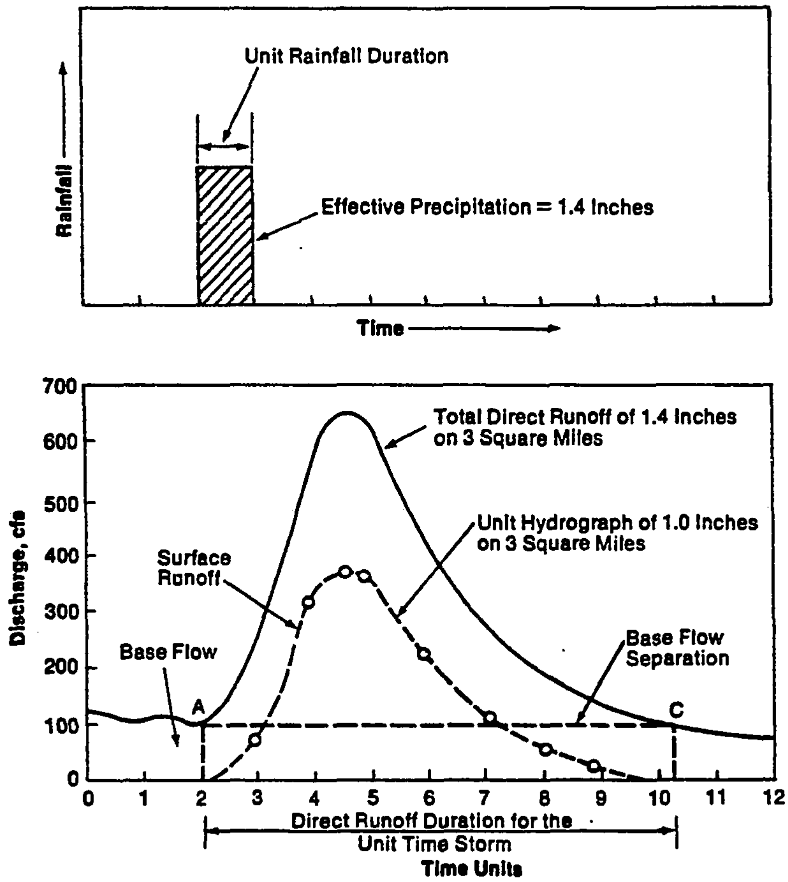


Figure 10. ILLUSTRATION OF THE DERIVATION OF A UNIT HYDROGRAPH FROM AN ISOLATED STORM

* Source: After Clark, 1971

4.1.1.5 Base Flow

Conventionally, runoff can be divided into two major parts: direct surface runoff and base flow as depicted in Figure 11. Base flow is normally contributed by the portion of water which percolates through soil layers until it reaches the water table (Linsley, 1975)[40]. HEC-1 uses a logarithmic decay function to simulate the base flow. The program requires three parameters to describe the base flow and the recession. They are: (1) starting flow (Qs), (2) recession flow (Qr) and (3) recession ratio (RR). By this means, the direct surface hydrograph can be separated from the baseflow.

4.1.1.6 Routing

There are several expedient techniques available in HEC-1 to simulate channel and/or reservoir routings. These include the methods of Muskingum, Modified Puls, Kinematic,Straddle-Stagger, Working R & D, Normal- Depth, and Tatum method. The bases of these techniques are well described in HEC-1 and other supplemental documents (HEC, 1960;[28] Feldman, 1981)[22]. Herein, the Muskingum and Modified Puls methods, which will be applied in the case study in chapter V, are briefly mentioned as follows.

The Muskingum method, generally applied to channel routing, determines reach outflows based on inflows and coefficients which describe the reach travel time and the storage characteristics. The relation among storage (S), and inflow (I), and outflow (O) can be expressed as:

$$S = K[x \cdot I + (1-x)O] \qquad \text{(Eq. 4.2)}$$

where K and x denote the reach travel time and storage coefficient respectively. Both K and x, required as parameters in the program, can be derived from charts or graphs or optimized by HEC-1. This technique is still widely used because of its simplicity and effectiveness in application.

The Modified Puls method (Chow,1964)[15] uses a storage-discharge relation based on a solution of the continuity equation as:

$$O_2/2 = (I_1 + I_2)/2 - (S_2 - S1)/\Delta t - O_1/2 \qquad \text{(Eq. 4.3)}$$

where Δt denotes the routing period, subscripts 1 & 2 represent the beginning and end of the routing period; O, I, and S were previously defined. This method is appropriate for both channel and reservoir routings. Two parameters are required: storage volume (SV) and discharge (SQ) which can be obtained from the output of a water surface profile (HEC-2) computer run. This approach has several advantages over others: (1) channel characteristics can be simulated as closely as possible, (2) the overbank storage is taken into consideration, and (3) the continuity is maintained between the surface profiles of different reaches.

4.1.1.7 Stream Network

The basin modeling in HEC-1 must begin at the uppermost subbasin of a stream branch and proceed downstream by converging tree-like network as shown in Figure 8. The sequence of the elements (subbasins and reaches) supplied in the data deck has to represent the logical drainage pattern of the modeled basin. For example, as illustrated in Figure 8, the runoff from subbasins 101 to 102 must be computed and combined before routing through reach 1030 and computing the runoff from subbasin 103. After the routing is specified and performed, the local runoffs at subbasins 103 and 104 are computed, routed and combined with the local inflow at subbasin 105 at which the job is complete.

4.1.2 Assumptions and Limitations of HEC-1

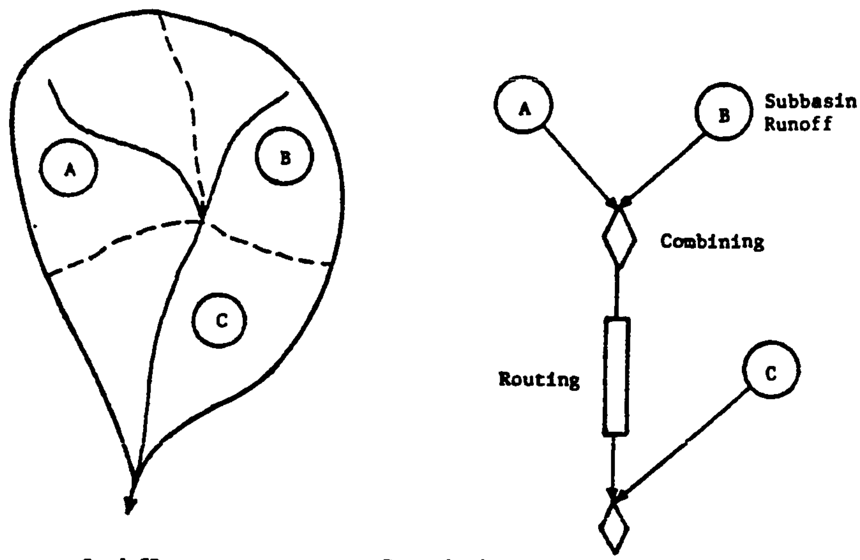
The program assumes that the basin hydrologic processes can be simulated by lumped parameters which reflect the average temporal and spatial conditions of a subbasin. Therefore, care must be taken for selecting the rainfall duration-time interval and size of basin component so that the average parameters will represent the subbasin characteristics without creating significant deviations from real conditions. As noted earlier in chapter 2, the major limitation of HEC-1 is that this program only simulates a single storm event which does not account for the soil moisture recovery during periods of zero precipitaion. However, the advantages of HEC-1 as discussed previously outweights this limitation. Besides, the rainfall loss rate may be adjusted for the antecedent moisture condition by using SCS curve number technique. Regarding the parameter optimization, the program uses a univariate gradient procedure (Beard,1966)[7] which does not guarantee a global optimization; therefore, a reasonable estimate must be initiated to insure that the procedure does not arrive at a local optimum. Furthermore, basin and channel routing are majorly performed by hydrologic methods which take less consideration of overbank storage and the attenuation of floodplains. However, HEC-1 has the option to use kinematic theory which employs hydraulic elements including channel shape, length, slope and roughness to simulate overland and channel routings. In this study, HEC-2 is used to obtain storage-outflow data. Those data are then used to join with the HEC-1 routing process to account for the effects of channel storage and floodplain attenuation.

4.1.3 Data Requirements and Input Structures

The basin hydrologic simulation requires parameters to describe the loss rate, baseflow, and routings occurring in each subbasin and reach. The general structure of input data is illustrated in Figure 12. Column 1 & 2 in the first field are designated by two characters to specify the desired function. For example, BA denotes basin area; LS is for loss rate by SCS method. The program proceeds from upstream to downstream until a "ZZ" card is encountered which ends the job.

4.1.4 Program Overview and Output Format

HEC-1 is designed to accept input in card or record formats. Figure 13 outlines the overviews of program operations. As mentioned earlier, it is vital to input the data deck in the correct order and sequence to represent the topographic structure of the basin and the network between subbasins and channel reaches. Regarding the output format, the "ID" card can be used to specify the degree of detail. In general, computations for rainfall losses, excess, runoff volume, discharge rate, optimized parameters are tabulated upon request. A summary table, which includes peak flows,



	Card ID	Description
	ID	Title
	IT	Time interval and beginning time
	IØ	Output control option for whole job
	KK	Subbasin A
Runoff from Subbasin A	BA	Area
	BF	Base flow
	P_	Select one precipitation method, use IN if necessary
	L_	Select one loss rate method
	U_	Select one rainfall excess transformation method
Subbasin B runoff	KK, BA, BF, P_,L_,U_	Similar to above for Subbasin A
Combine A + B	KK	Station name
	KM	Combine runoff from A and B (message option)
	HC	Indicate 2 hydrographs are to be combined
Route A + B to C	KK	Station name
	RL	Channel loss optional
	R_	Select one routing method
Subbasin C runoff	KK, BA, BF, P_,L_,U_	Similar to above for Subbasin A
Combine routed + C	KK	Station name
	HC	Indicate 2 hydrographs are to be combined
	KK, IN, QØ	Compare computed and observed flows
	ZZ	End

Figure 12. Example Input Data Organization for a River Basin

From: HEC-1 user,s manual, 1981

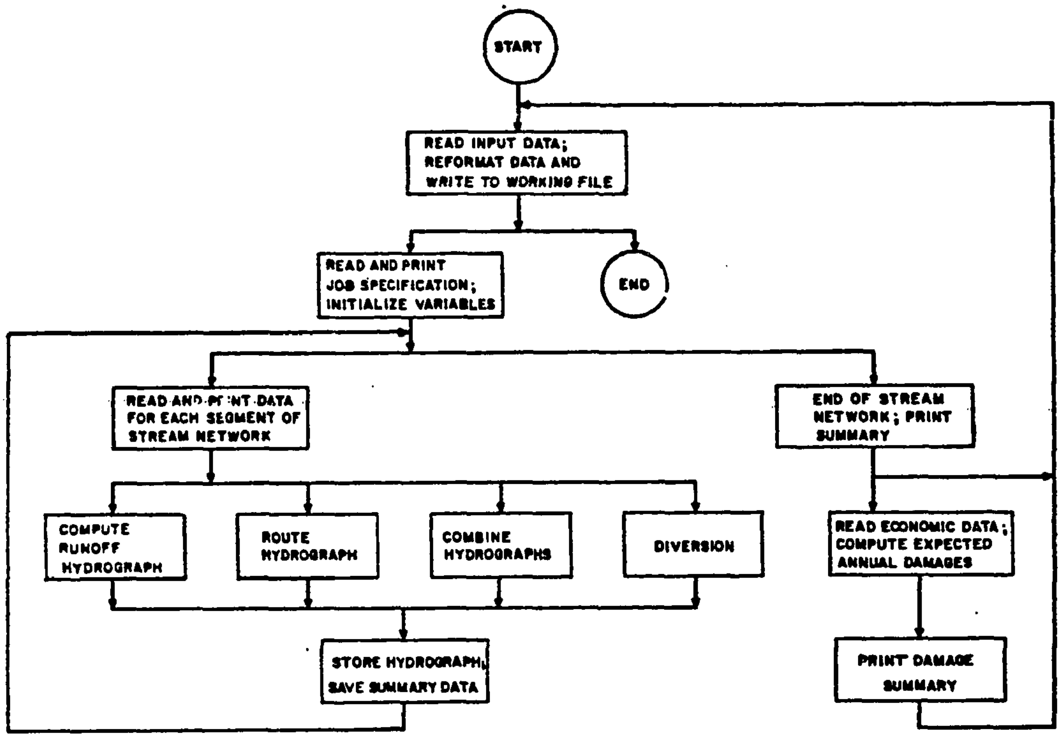


Figure 13. HEC-1 PROGRAM OPERATION OVERVIEW

* Source: HEC-1 programer's manual, 1981

accumulated drainage area, peak time, etc., is usually provided. Also, the scheme of the channel network can be traced by including a "DIAGRAM" card. In addition, hydrographs can be plotted in tables and/or graphs with time and sequence number for each ordinate. In short, HEC-1 is versatile and effective in providing a great variety of output format.

4.1.5 Computer Requirements

HEC-1 requires a FORTRAN IV compiler and up to 16 input/output files. These can be stored on tape or disk. HEC-1 was originally developed and tested on UNIVAC 1108 and CDC 7600 (HEC-1 programmers manual,1981)[32]. It has been installed and modified to use the IBM-3081 at the University of Oklahoma. Table 7 lists the compiler time and memory required for a few tested computers.

Table 7. HEC-1 COMPUTER MEMORY AND TIME REQUIREMENTS

	UNIVAC 1108	CDC 7600	CDC Cyber175	HARRIS 500	IBM 3081
MEMORY (K WORDS)	39	55-small 49-large	116	525	288 (K BITES)
CPU (sec)	30	17.49	39.28	570	7.47

4.2 HYDRAULIC ANALYSIS WITH HEC-2

HEC-2 is now considered to be the most acceptable and widely used program for channel hydraulic analysis. It was originally developed by Bill S. Eichert in 1967 and has been modified with up-to-date information and increased capability and ease of use. The program is designed to simulate the hydraulic characteristics of channels, bridges, culverts, and weirs. It performs the steady, gradually varied flow computations for river channels of any cross-section under either sub- or supercritical flow condition. The effects of channel improvements, levees, and floodways on water surface profiles can be also simulated, computed, and assessed. The results of water profile and encroachment computations with

HEC-2 yield important data, such as: (1) chnnnel capacity (Vc) which can be applied in the flood damage model and for channel design, (2) storage-outflow relation to join with HEC-1 for stream routing, and (3) flood areas and widths of given storms which can be applied to the flood damage model in computing the inundated ratio (Ri) as described in Chapter III.

Not all of the features built in the program are used in this study. This section concentrates on the following areas: (1) water profile and encroachment, (2) assumptions and limitations, (3) input structure and data requirement, (4) output format, and (5) computer requirements.

4.2.1 Profile Computations

The profile computation with HEC-2 is based on the solution of the one-dimensional Bernoulli equation with other losses equations. In fact, the follwing two equations are used and solved in HEC-2 by an iterative procedure to compute an unknown water surface elevation at cross-section 2 as depicted in Figure 14 .

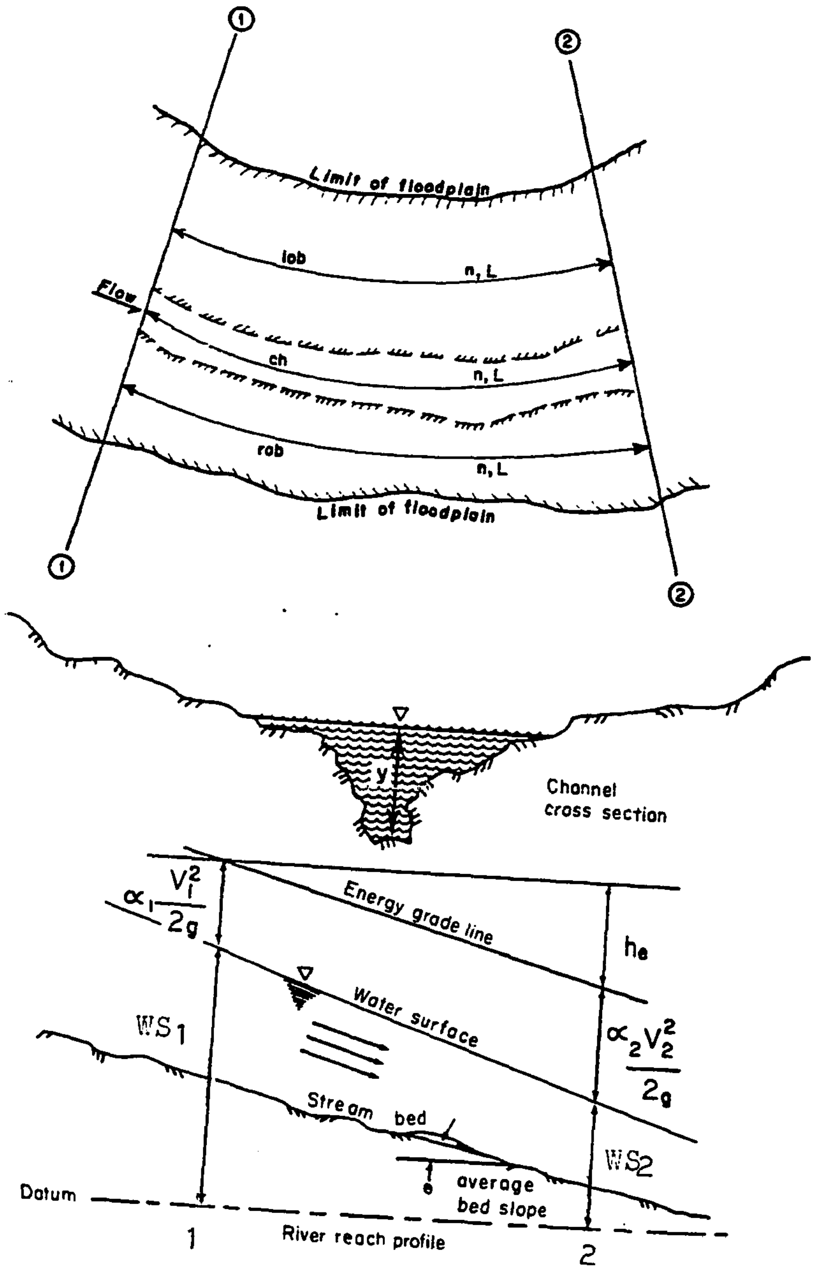


Figure 14. RIVER CROSS SECTION & PROFILE SHOWING COMPONENTS OF ENERGY EQUATION

Souce: After Feldman, 1981

$$WS_2 + \frac{\alpha_2 V_2^2}{2g} = WS_1 + \frac{\alpha_1 V_1^2}{2g} + H_e \qquad \text{(Eq. 4.4)}$$

$$H_e = LS_f + c\left|\frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 V^2}{2g}\right| \qquad \text{(Eq. 4.5)}$$

where, WS1, WS2 = water surface elevations (ft) at cross-sections 1 & 2,

V1, V2 = mean velocities (ft/sec) at cross-sections 1 & 2,

$\alpha 1$, $\alpha 2$ = velocity coefficients for flows at cross-section 1 & 2,

g = acceleration of gravity (ft/sec^2),

He = energy head loss (ft),

L = discharge-weighted reach length (ft),

Sf = representive friction slope for reach,

C = expansion or contraction loss coefficient.

HEC-2 uses the standard-step procedure to compute water elevation at a specific location (see Figure 14) by solving equations 4.4 and 4.5 iteratively. The procedures are summarized as follows (HEC-2 manual,1981)[33]:

a) assume water surface elevation at cross-section 2: WS2,

b) compute total conveyance (K2) and velocity head (α2V2/2g) based on geometry and Manning equation,

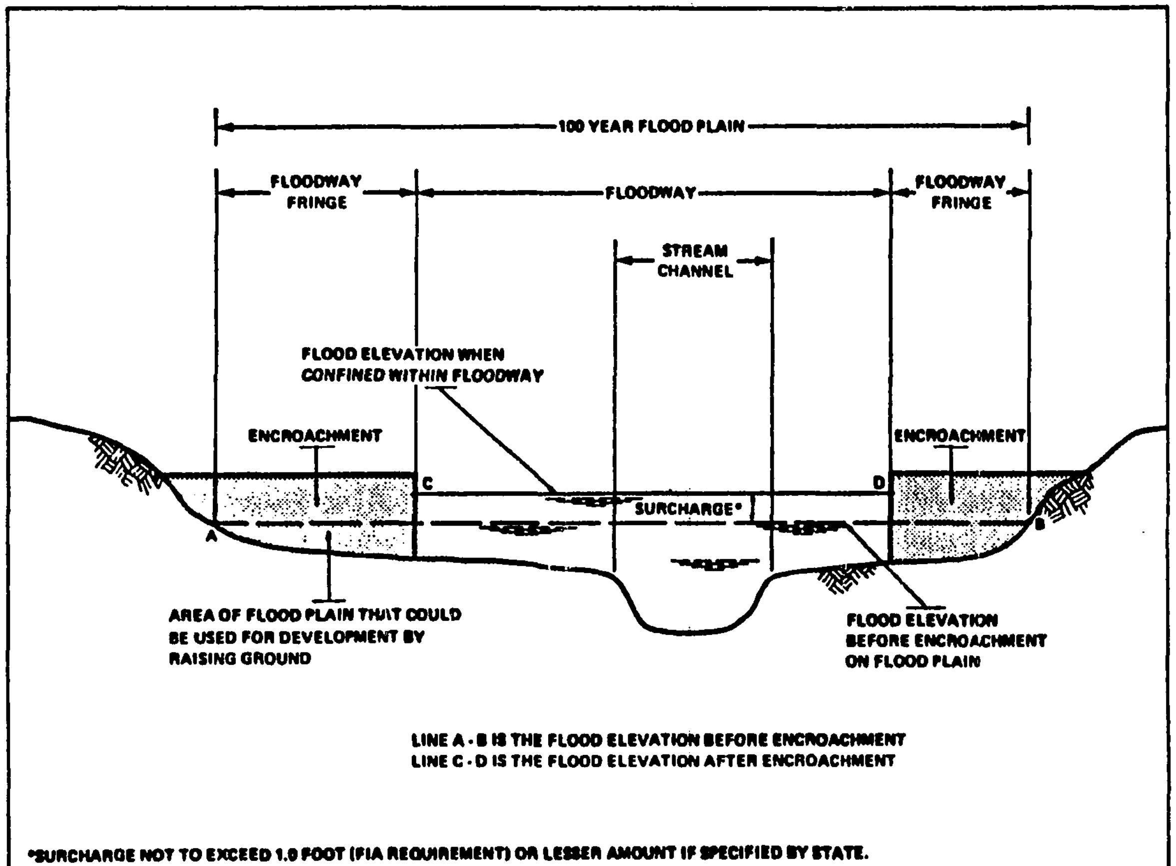
c) compute Sf with the options available in HEC-2, compute WS2 by combining eq 4.4 and 4.5

d) compare the computed WS2 with the assumed WS2; repeat until the error is within 0.01 ft.

4.2.2 floodway encroachment

The concept of floodway and the impact of encroachment on water surface profiles are important aspects for planners, land developers, and engineers in balancing the economic gain from floodplain development aginst the resulting increase in flood hazards. The current policy regarding floodway determination is based upon a one-percent exceedance frequency flood of the existing condition of a floodplain. As shown in Figure 15, a 100 year floodplain is divided into two parts: a floodway and a floodway fringe. The floodway is designated to be kept free of encroachment in order to carry the selected 100-year flood discharge without raising the water surface more than one foot above that of the natural floodway; while the floodway fringe is assumed to be filled with solid for development (U.S. Corps of Engineers, 1972)[60].

There are six option methods avalable in HEC-2 to specify encroachments for floodway determinations. The variations and detail procedures for these options are well described (HEC-2 manual, 1981)[33].



* Source: U.S. Water Resource Council, 1978

Figure 15. FLOODWAY SCHEMATIC

In this study, the floodway is computed so that the conveyance with the encroached cross-sections (at some higher elevation) is kept equal to that of the natural cross-section at the natural water level as shown in Figure 16 (method 4 of encroachment options). The encroachment stations are programmed so that an equal loss of increased conveyance due to higher water elevation is eliminated on each side of channel, if possible, to carry the original selected discharge. When half reduction of conveyance cannot be obtained in one overbank, the other overbank makes up the difference so that encroached stations will not fall within the main channel. Table 8 illustrates the format of data input for the encroachment computer run. The overall procedures for floodway and encroachment determinations include the following steps:

a) The water surface elevation and the conveyance are computed for the natural condition (without encroachment) as the first profile of a multiple profile run using HEC-2.

b) The water elevation is delineated with an increment (one foot for example) and this increased value and the selected method is assigned in the INQ field of the ET card (see table 8). For example, a value of 10.4 is assigned, where "10" denotes the tenths of a foot allowed for increase in water elevation and "4" designates the method 4 of encroachment options.

c) The resulting conveyance and encroached stations are computed in the subsequent profiles. Additional computer runs may be made to meet the criteria of the designated floodway.

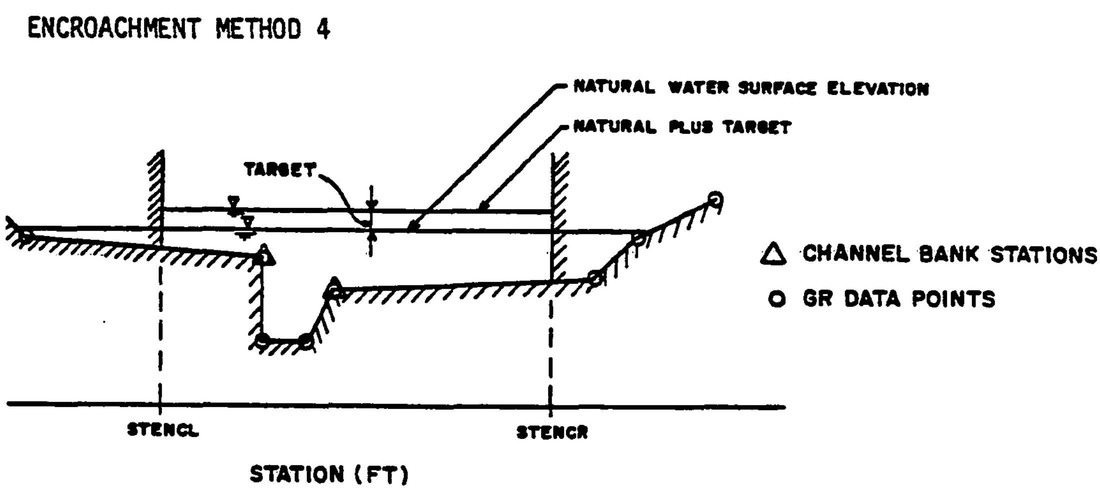


Figure 16. A Scheme of Encroachment Method 4
(After HEC-2 User's Manual, 1981)

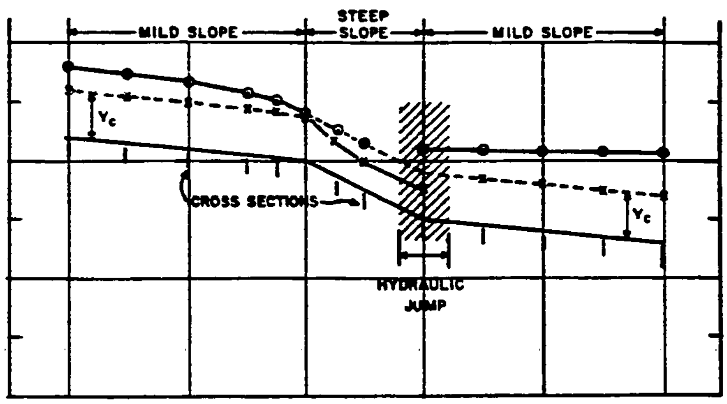
Table 8 Encroachment Data Organization

ENCROACHMENT DATA ORGANIZATION

CARD	VALUES	COMMENTS
T1 - T3		Title information (natural profile)
J1	INQ(J1.2=2) WSEL(J1.9)	Read 2nd field of ET and QT card. Starting water surface elevation is specified here.
J2	ITRACE(J2.10=15)	Request flow distribution for natural profile.
J3	IVAR(J3.1=110), IVAR(J3.2=200)	Summary table 110 and 200 will be requested for summary printout.
NC QT		
ET	ENCFP(ET.2=0) ENCFP(ET.3=8.4) ENCFP(ET.4=10.#)	1st profile is natural profile. 2nd profile is Method 4 with .8 foot rise. 3rd profile is Method 4 with one foot rise.
X1 GR X1 GR		
ET	ENCFP(ET.2=0) ENCFP(ET.3=7.4) ENCFP(ET.4=5.41)	1st profile is natural profile (no change). 2nd profile is changed to 7.4. 3rd profile is changed to 5.41. Bridge encroachment stations (for the BT cards) will be the same as the downstream encroachments.
X1 GR		
SB		
ET	ENCFP(ET.2=0) ENCFP(ET.3=7.11) (ET.7=STENCL) (ET.8=STENCR) ENCFP(ET.4=0)	1st profile is natural profile (no change). 2nd profile is changed to Method 1 for bridge. Bridge encroachments (for both BT and GR cards) are specified in the 7th and 8th fields of the ET card. Continue previous encroachment instructions.
X1 X2 BT		
ET	ENCFP(ET.2=0) ENCFP(ET.3=15.3) ENCFP(ET.4=10.5)	1st profile is natural profile (no change). 2nd profile is changed to Method 3. 3rd profile is changed to Method 5.
X1 GR X1 GR		
EJ		End of data.
T1 - T3		Title information (Method 4 encroachment).
J1	INQ(J1.2=3) STRT(J1.5=0) WSEL(J1.9)	Read 3rd fields of ET and QT card. Slope area method of starting should not be used for encroachment profile. Starting water surface elevation specified here.
J2	NPROF(J2.1=2)	2nd profile.
T1 - T3		Title information (Method 4 encroachment).
J1	INQ(J1.2=4) STRT(J1.5=0) WSEL(J1.9)	Read 4th fields of ET and QT card. Slope area method should not be used. Starting water surface elevation specified area.
J2	NPROF(J2.1=15)	Last profile, request summary printout.
3 blank cards		
ER		End of run.

4.2.3 Computation Controls

HEC-2 has the automatic ability to "balance" the unknown water surface elevation by an iterative procedure as mentioned earlied. In addition, the critical depth, known as the depth of critical flow at which the specific energy is a minimum value for a given discharge, can be computed by the program as a criteria to verify the flow regime and to assure the computed elevation is on the "right" side. For instance,the water elevation for a subcritical flow regime is expected to be higher than the critical elevation for a given cross-section; while the elevation is expected to be below than the critical elevation for flow in the supercritical regime. HEC-2 assumes the computed profile is either all subcritical or all supercritical. Either flow condition can be processed but must be done separately. If a change of flow regime is identified in the computation as shown in Figure 17, it prints out messages which may imply that either a local phenomenon (eg. hydraulic jump) occurs or a problem ("red-flag") exists in the assumed flow regime. A different flow regime to restart the computation or additional cross-sections inserted in between the problem area to simulate the stream more detail is strongly suggested (HEC, Vol#6, 1975)[29] to ascertain the flow condition.



(⊙) subcritical run, (×) supercritical run: (dashed line) critical; (solid line) actual.

Figure 17. TRANSITION FROM SUBCRITICAL TO SUPERCRITICAL FLOW (After Feldman, 1981)

4.2.4 Assumptions of HEC-2

The following assumptions are made in the profile and encroachment computations (HEC-2 manual,1981)[33]:

a) Flow is steady, which means the time variation with flow is not included in the energy equation.

b) Flow is gradually varied, which is based in the premise that the hydrostatic pressure distribution domains at each cross-section.

c) Flow is one-dimensional, the direction of the predominant velocity is parallel to the flow.

d) Manning's equation, developed for uniform flow, is applied to evaluate the conveyance and friction slope in a gradually varied flow.

e) Channels have relative small slopes so that the pressure head can be represented by the water depth measured vertically.

f) The flow regime is assumed to be either subcritical proceeding downstream to upstream, or supercritical proceeding upstream to downstream. The program only processes one flow regime at a time.

4.2.5 Data Requirements and Input Structures

The data required by HEC-2 to perform water surface profile computations includes: specified flow regime (sub- or supercritical), starting water elevation, discharge flow, loss coefficients, cross-section geometry, reach lengths, and the configuration of local obstructions such as bridges and culverts. The general input structure for a multiple water surface profile computation is listed in Table 9.

4.2.6 Program Overview and Output Format

The program has been revised to ease data handling and to increase manipulation capability. Many options and routines are available to simulate and manipulate the cross-section with skewing factor, raising or lowering the geometry as desired. Figure 18 illustrates the overall operation of HEC-2 to compute the water surface profile. As noted, this program is comprised of a number of large subroutines with specific functions for each component.

Regarding the output format, HEC-2 is featured with a large selection of output control options. The simplest and most efficient output includes a list of input data and specified summary tables. The detail computations of cross-sections, flow distribution in three subdivisions (left overbank, main channel, and right overbank) of each cross section, and some trace variables, such as critical depth can be requested for checking and debugging purposes. Also, the storage-outflow data for each cross-section can be provided for conjunction with HEC-1 channel routings. In addiition, plots of any profile and/or any cross-section can be performed at any scale as desired.

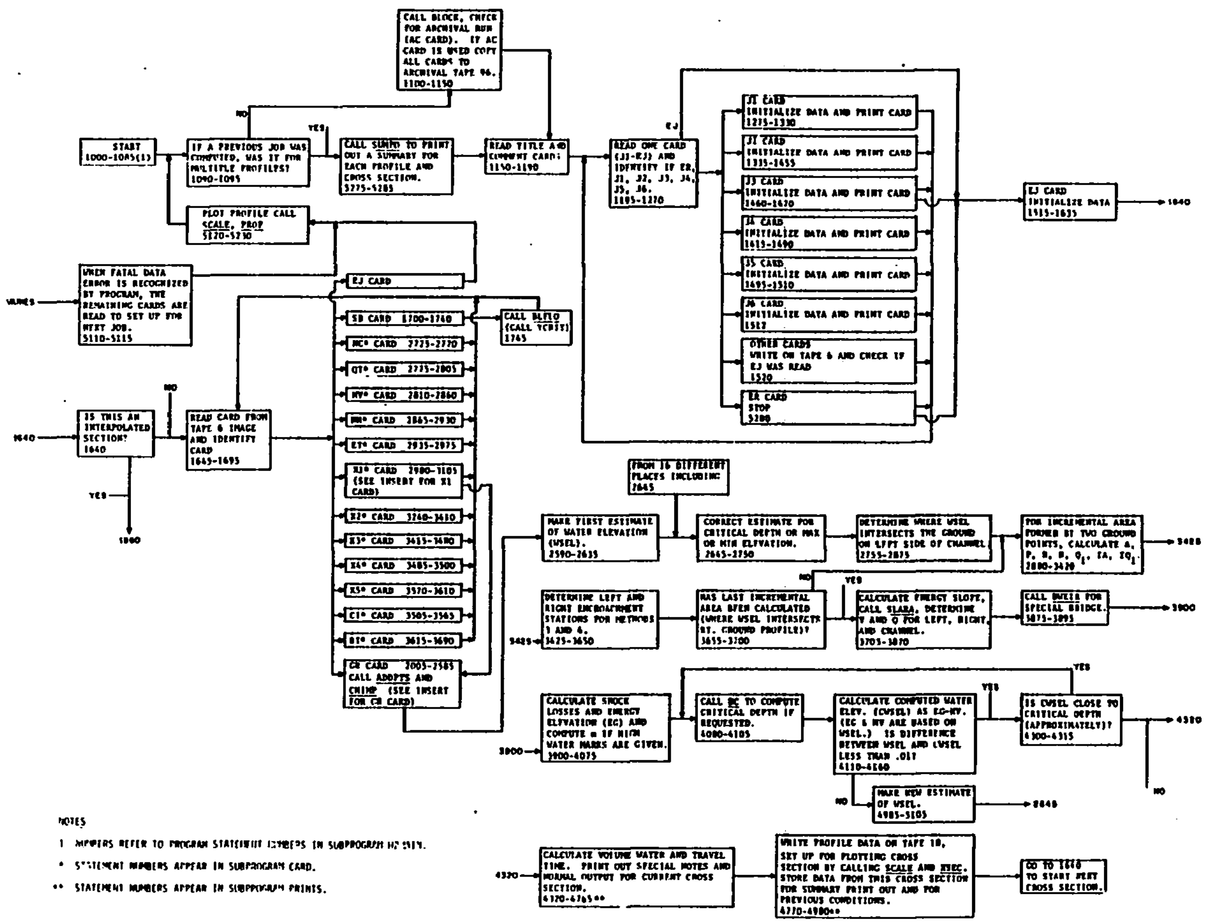
TABLE 9
TYPICAL HEC-2 DATA ORGANIZATION

(Multiple Profile Run)

CARD TYPE	CARD IDENTIFICATION	APPLICATION
Documentation	AC, C	All profiles
Documentation	T1* - T3*	1st profile
Job Control	J1*, J2	
Job Control	J3 - J6	All profiles
Change	NC*, NH, NV, QT, ET	
Cross Section	X1*, CI, X2, X3, X4, X5, GR*	
Bridge (Special Bridge)	SB*	
Cross Section	X1*, X2*, X3, X4, X5, BT, GR	
Change	NC, NH, NV, QT, ET	
Cross Section	X1*, CI, X2, X3, X4, X5, GR	
Cross Section	X1*, CI, X2, X3, X4, X5, GR	
Job Control	EJ*	
Documentation	T1* - T3*	2nd profile
Job Control	J1*, J2*	
Documentation	T1* - T3*	Last profile
Job Control	J1*, J2*	
Job Control	3 blank cards*	Terminate run
Job Control	ER*	

* From HEC-2 manual, 1981

Figure 18. GENERAL FLOW CHART OF HEC-2 COMPUTER PROGRAM



NOTES

1 NUMBERS REFER TO PROGRAM STATEMENT NUMBERS IN SUBPROGRAM [illegible].

* STATEMENT NUMBERS APPEAR IN SUBPROGRAM CARD.

** STATEMENT NUMBERS APPEAR IN SUBPROGRAM PRINTS.

Table 10 summarizes output control options; and detail information is found available from HEC-2 manual (1981)[33].

Table 10. CONTROL OF HEC-2 PROGRAM OUTPUT

OUTPUT	CONTROL (CARDS)
Commentary	C
Input Data Listing	J1.1
Detailed Output by Cross Section	J5
Flow Distribution	J2.10, X2.10
Traces	J2.10, X2.10
Summary Table	J2.1, J3, J5
Profile Plots	J2.3
Cross Section Plots	J2.2, X1.10
Archival Tapes	AC
Program Storge Tapes	J6
Punched Cards	J4

4.2.7 Computer requirement

The HEC-2 program was originally developed for use on the CDC 6600. It has been adapted to several other computers including the IBM-3081. Table 11 lists the approximate memory and time requirements of several tested computers.

TABLE 11

HEC-2 COMPUTER MEMORY AND TIME REQUIREMENTS

COMPUTER	MEMORY (WORDS)	CPU TIME (SEC)
CDC 7600	32000	3.9
CDC 6600	32000	24.1
UNIVAC 1108	32000	35.0
IBM 370/168	248 K BYTES	121.4
HONEYWELL CS6058	46000	59.6
HERRIS S120	96000	402
IBM 3081	288 K BYTES	7.5

4.3 EXPECTED ANNUAL DAMAGE COMPUTATION

Economic efficiency is one of the main criteria in evaluating future land development and flood control alternatives. Therefore, the expected annual flood damage is taken as a economic base for comparison. The damage reduction benefits due to a project or a future land use scenario can be computed as the difference between damage values, occurring in a river basin, with and without the project or scenario.

As illustrated in Figure 19, the expected annual damage is computed from integrating the damage-frequency curve which is obtained by combining and transforming the flow-frequency and the flow-damage curve for a damage reach in a basin. HEC-1 has the capability to compute EAD provided with flow (or stage)-frequency relations and flow (or stage)-damage relations. It uses a Guassian quadrature procedure to establish the damage-frequency function and integrates the resulting damage-frequency function for multiplan (eg. existing condition and alternatives) analyses.

4.3.1 Flow-Frequency curve

The flow-frequency data are usually non-linear. HEC-1 uses a cubic-spline fit for interpolation as shown in Figure 20 to construct the flow-frequency curve from the multiple flood analysis based on ratio of precipitation or runoff to the base event (eg. 25-year storm).

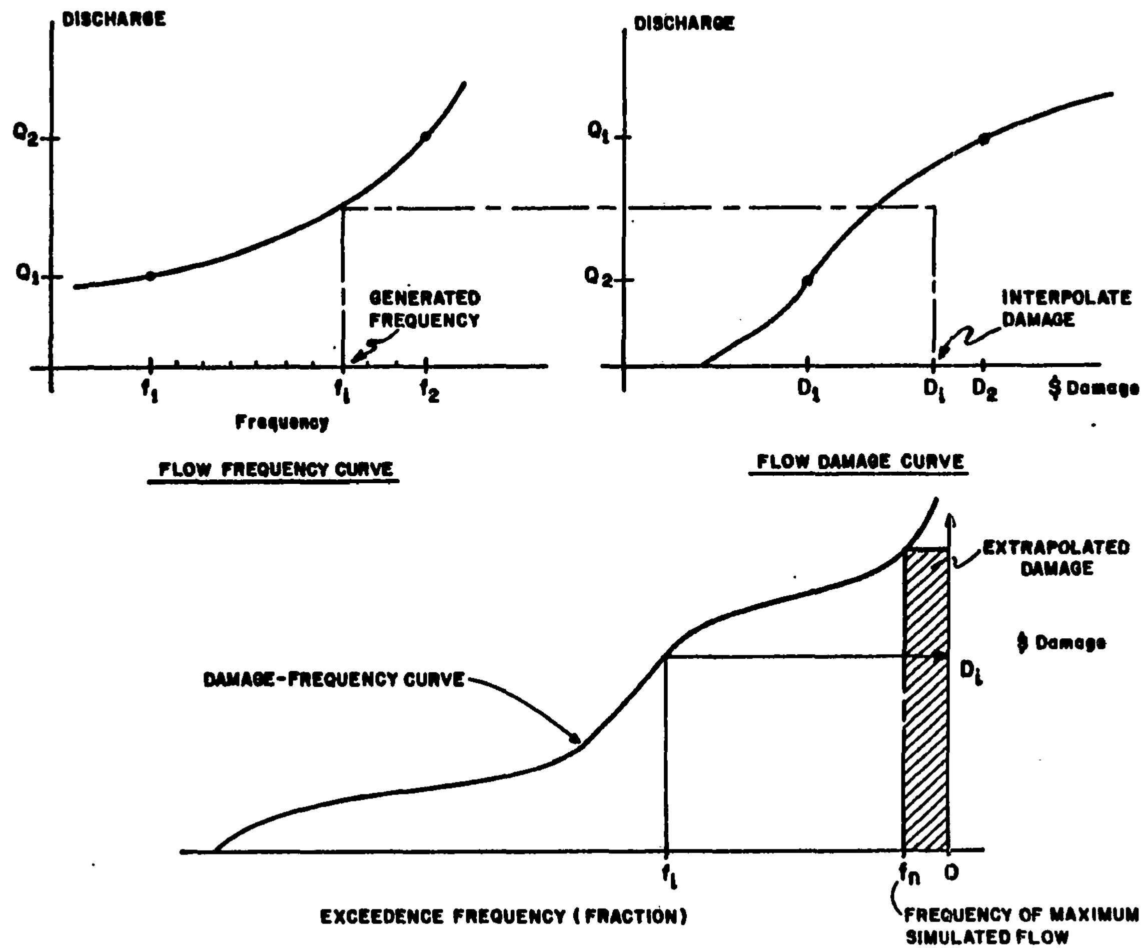


Figure 19. Damage Frequency Curve (After HEC-1 manual, 1981)

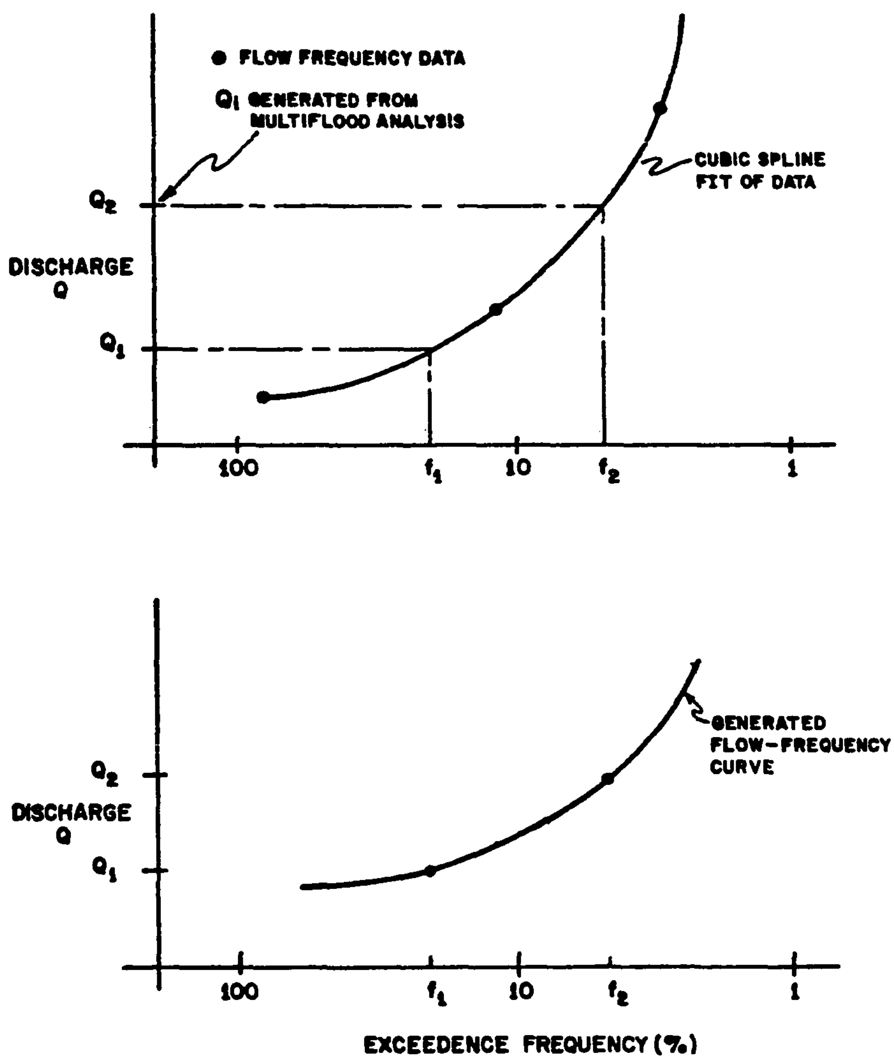


Figure 20. Flow Frequency Curve

From: HEC-1 user's manual, 1981

Also, the program is featured to model multiple plans in one single computer run; thus the flow-frequency relationship corresponding to each plan can be established and modified automatically according to the changes in discharge flows resulted from each plan (see Figure 21). The modified flow-frequcncy curves can be carried out and used to compute the EAD for modified conditions in a multiflood, multiplan analysis.

4.3.2 Computation of EAD

As mentioned earlier, the EAD computation is performed by integrating the damage-frequency curve which results from transforming the flow-frequency and flow-damage curves. Besides the flow-frequency curves, the flow-damage data must be provided in order to carry out the EAD computation. In this study, the flow-damage relation is calculated based on the flood damage model, whereas the flow-frequency curve can be obtained from a multiflood analysis using HEC-1 basin simulation. Usually, several damage reaches are specified, as shown in Figure 22, according to the designated index locations which are selected to represent the average damage condition for each damage reach. A damage reach is selected as a length of river with consistent profiles for the range of specified discharges which are significant for the EAD computation. The changes in damage due to alternatives or future scenarios can be computed through the modified flow-frequency curve or the modified flow-damage function.

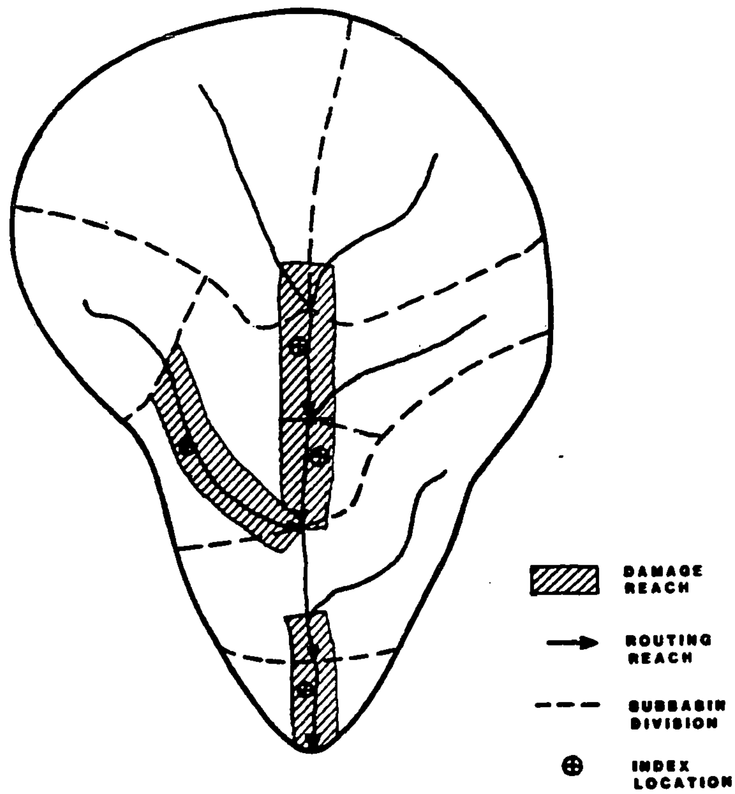


Figure 22. Flood Damage Reduction Model
(From: HEC-2 user's manual, 1981)

4.3.3 Assumptions of EAD

The following assumptions are made in the EAD computation:

a) Same time-pattern of precipitation is applied to all ratios in the multiflood computation, unless the ratios are developed from separate computer runs with specified temporal distribution of precipitation to each flood.

b) In modifying the flow-frequency curve for multiplans as shown in Figure 20, the frequency of each ratio remains the same; only the peak flows change with plans. Time variation of EAD is not taken into account in HEC-1.

In spite of these limitations, the EAD computation using HEC-1 can provide a quantitative measure in evaluating alternatives and scenarios regarding flood control and future land use. The computational efficiency and avalability of this method have been outweighted its constrains. Furthermore, by using the flood damage model, the time variation of EAD values can be included by cost index (Ci) to account for the effect of inflation as time goes on.

4.3.4 Data Requirements and Input Structures

The data required for the EAD include: (1) damage area for each damage reach according to the classification of damages which can be estimated base on the flood damage model, (2) frequency data which can be in the form of stage-frequency or flow-frequency relations.

Table 12 summarizes the input structure for an EAD computation. If damage data change with plans, then different sets of QD and DG cards must be provided for each plan and located after EP card until all the plans are evaluated.

TABLE 12

INPUT DATA ORGANIZATION FOR EAD COMPUTATION

Card Identification	Card Type & Function
EC	Job Indentification for EAD
CN	Damage Category Identification
FR	Frequency Data
QF	Discharge corresponding to FR
PN	Plan Name
QD	Discharge corresponding to Damage
DG	Damage Data
EP	End of Plan

4.3.5 Output Format

A summary table for EAD computations usually includes a cross table tabulated by stream station, damage reach, damage category, and EAD values computed for each plan as shown in an example computation illustrated in Table 13. The damage is summed for each plan, and the damage benefits are computed by taking the difference of damage values between alternatives and the base condition.

Table 13. EXPECTED ANNUAL FLOOD DAMAGE SUMMARY

STREAM STATION	DAMAGE REACH	WATERSHED	TOWNSHIP	*	DAMAGE CATEGORY	EXPECTED ANNUAL DAMAGE PLAN 1	PLAN 2	PLAN 3
RCH1	1			*	1 RESID	0.00	0.00	0.00
				*	2 IND/COM	0.00	0.00	0.00
				*	3 AGRIC	129.22	6.18	6.27
				*				
				*	TOTAL	129.22	6.18	6.27
			DAMAGE CHANGE (BENEFITS)			BASE	123.04	122.95
RCH2	2			*	1 RESID	1099.86	139.58	375.40
				*	2 IND/COM	20.21	1.97	5.29
				*	3 AGRIC	0.00	0.00	0.00
				*				
				*	TOTAL	1120.06	141.55	380.69
			DAMAGE CHANGE (BENEFITS)			BASE	978.52	739.38
BASIN TOTAL				*	1 RESID	1099.86	139.58	375.40
				*	2 IND/COM	20.21	1.97	5.29
				*	3 AGRIC	129.22	6.18	6.27
				*				
				*	TOTAL	1249.28	147.73	386.96
			DAMAGE CHANGE (BENEFITS)			BASE	1101.56	862.33

*** NORMAL END OF HEC-1 ***

* Source: HEC-1 manual, 1981

4.3.6 Computer Requirement

Since the EAD computation is a feature built in the HEC-1 program, the computer requirement is the same as described and summarized in Table 11.

4.4 SUMMARY OF THE METHODOLOGY

The procedure to apply the flood damage model for evaluating the hydrologic impact, hydraulic efficiency, and economic feasibility of land use and flood control alternatives as described in previous sections can be summarized into three phases with detail steps listed as follows:

PHASE I: HYDROLOGIC ANALYSIS

a) Obtain detailed topographic, soil, and land maps to identify the basin characteristics.
b) Define the drainage boundary, locate index stations and stream network configuration, and divide the basin and stream into subbasins and reaches.
c) Collect rainfall, streamflow data for gaged stations, or adapt hypothetical storms from TP-40 and HYDRO-35 for ungaged stations.
d) Determine the CN valules and other parameters including area (A), time of concentration (Tc), storage constant (R), and routing criteria (K or SQ-SV) for each subbasin and each reach.

e) Structure the input deck according to the topographic layout of the basin and apply to HEC-1 to generate design storm hydrographs (eg. 2-, 10-, 25-, 50-, and 100- year storms).

f) Construct the flow-frequency curve based on HEC-1 results and compare it with the one developed based on recorded flows; adjust parameters until the two coincide.

g) Perform a multiflood-multiplan HEC-1 run including the existing and future conditions.

PHASE II: HYDRAULIC ANALYSIS

a) Determine channel parameters, including design storm flow rates, Manning's roughness values, cross-sectional elevataion and station data, reach length, starting water surface elevation and flow regime.

b) Obtain configurations of culverts, bridges, or other obstructions located along the stream.

c) Compute the existing water surface profiles and floodway encroachment for selected storms (eg. 10-, 50-, 100-, and 500- years storms).

d) Determine the channel capacity and outflow-storge relationships for each reach by profile computations

e) Estimate the profiles and flodway encroachments of slected storms for future conditions by changing discharge rates and/or other parameters.

f) Delineate the floodway and channel encroachment of 100-year base flood for existing and future conditions.

PHASE III: ECONOMIC ANALYSIS

a) Specify index locations and damage reaches.

b) Determine the parameters, comprised in the flood damage model, including Pv, Kc, Ci, Iu; and obtain Ve, and Ri from output of HEC-1 and HEC-2 runs.

c) Construct the flow-damage data of existing and future conditions for a range of selected frequency events for each damage reach.

d) Perform EAD computations for the existing and future conditions in a multifood-multiplan computer run, modify the flow-damage data for each plan if damages change with plans.

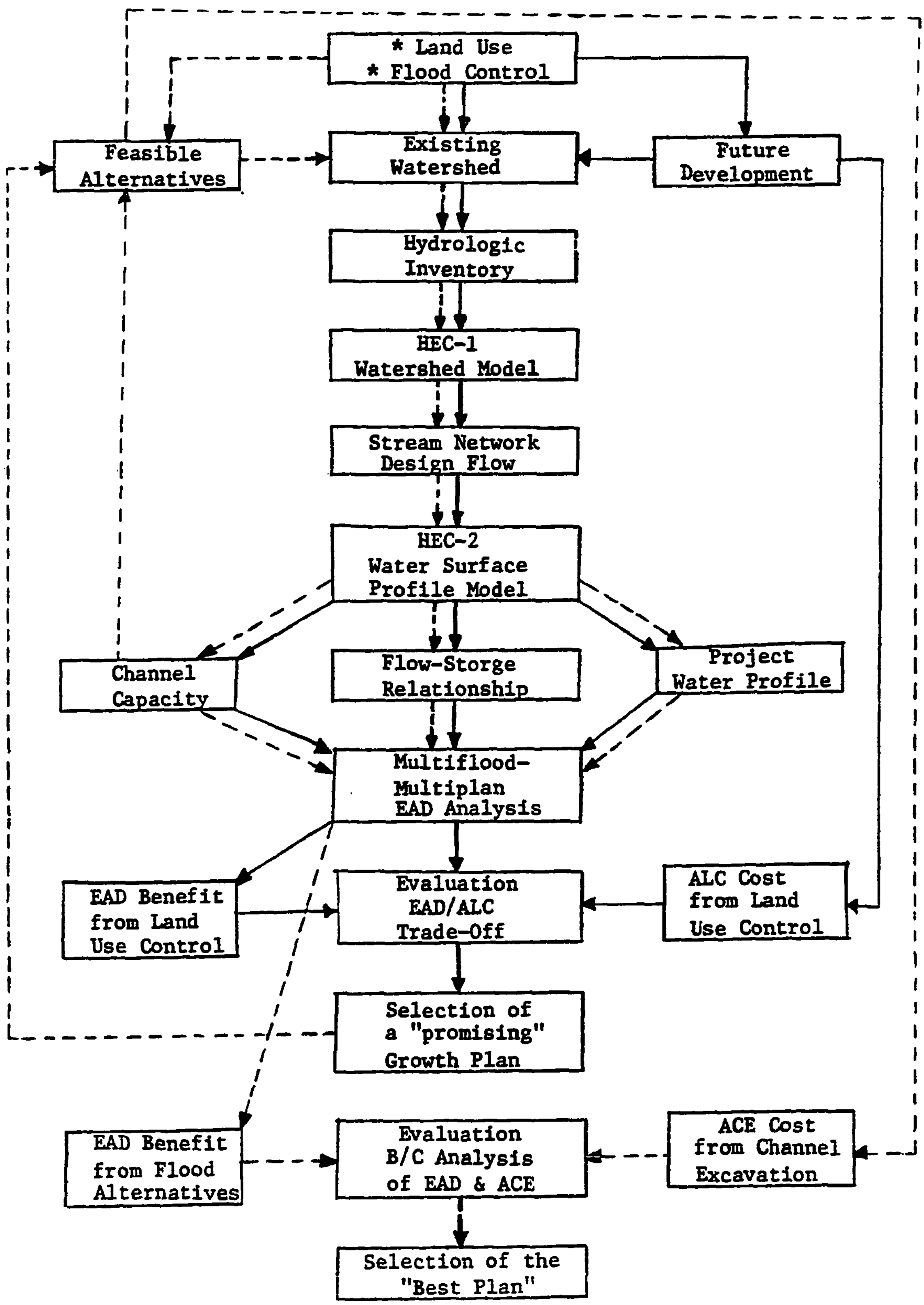
e) Evaluate the economic impacts of existing and future conditions by trading-off the values among the resulting EAD values and the estimated cost due to land use control.

f) Repeat steps (a) to (d) for a selected future land use condition based on result (e), with and without various degree of flood protection.

g) Evaluate the economic efficiency of proposed alternatives; select a "best-plan" by using the cost-benefit analysis.

Summing up, Figure 23 structures the framework and the inter-relationships among the components of this integrated methodology.

FIGURE 23 FLOW DIAGRAM OF PROPOSED STUDY



Chapter V

CASE STUDY

5.1 OBJECTIVES OF CASE STUDY

The concept of the model and the structure of the integrated methodology have been decribed previously. The case study herein is conducted to achieve the following objectives through testing on a selected watershed.

* To demonstrate HOW the model and the methodology works.
* To highlight WHAT the major effects are of land use on flood characteristics, as well as floods on land use.
* To show WHY this technique can provide a quantitative economic basis in guiding land development as well as evaluating flood control alternatives.

To accomplish these objectives, a number of assertions are made:

a) The constraints of HEC-1 and HEC-2 are validated and retained in modeling the hydrologic and hydraulic characteristics of the testing watershed.

b) Land use is classified into three major categories: agricultural, residential, and commercial/industrial zones; the residential and commercial/industrial areas are further divided into 4 groups respectively as described in chapter III.

c) The degree of land development is signified by the intensity of urbanization (Iu) which is defined as the ratio of space presumably projected for urban growth and infrastructural activities to the total land area.

d) The economic feasibility of land-use is determined solely by trading off the cost of preventing urban development and the benefit of flood damage reduction from land-use control. Political, social, and jurisdictional influences are not considered.

e) The economic loss due to preventing land-use is computed based on estimating the maximum economic gain when land is in its highest use.

f) The flood damages are categorized into three groups according to the type of land-use classified; damage reduction benefits are computed as the difference between the expected annual damage (EAD) values, with and without proposed land uses or control projects.

g) Channel improvement is chosen as the example alternative to demonstrate the ability of this technique for analyzing and evaluating the economic efficiency of flood alleviation projects; other measures, structural or non-structural, can be similarly applied, but not repeated herein.

It is important to choose a basin with realistic size so that the model's computation and data requirements do not become prohibitive. The Cow Creek basin is selected because of the realistic size of the basin, the expection of substantial development in the near future, and the avalability of hydrologic and hydraulic data.

5.2 WATERSHED DESCRIPTION

The Cow Creek basin, as diagrammed in Figure 24, encompasses an approximate drainage area of 20 square miles. It is fan-shaped with a maximum width of 6 miles in the east-west direction and a maximum length of 5 miles along the south-north direction.

Cow Creek, which is a tributary of the Canadian River, composes a main stream and three branches with a total stream length of 21 miles. The headwater starts in the area of MacArthur Boulevard and S. 74th street of Oklahoma City, Oklahoma. It joins the Canadian River at approximately one-half mile south of S. 149th street and one-half mile east of Rockwell Avenue. Most channels are earth-made with cross-sections varying from trapezoidal to triangle in shape. Streams are mostly narrow and meandering with low banks. In general, the drainage pattern follows an easterly and southerly direction.

The climate of the study area is typically continental in type. The average annual precipitation is approximately 31.8 inches with a maximum rainfall of 52.0 inches in 1908. Temperature ranges from a low of 28°F to a high of 50°F in winter, and 70°F to 98°F in summer.

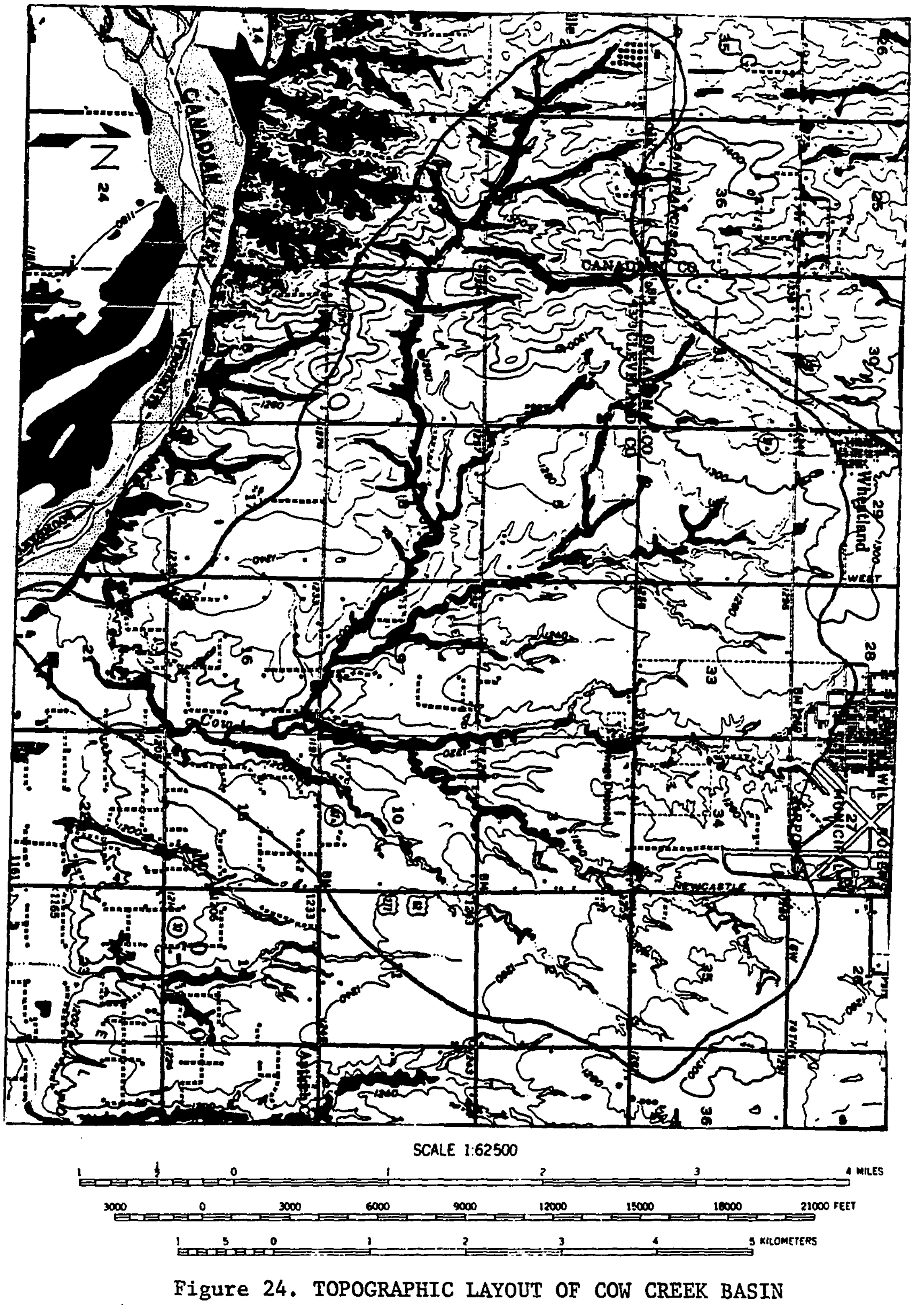


Figure 24. TOPOGRAPHIC LAYOUT OF COW CREEK BASIN

The topography of this basin shows a moderate rolling profile with an average slope of approximately 25 feet per mile (Figure 24). The elevation ranges from 1200 feet to 1350 feet. The geologic formation of the Cow Creek watershed is majorly underlain by silt stones and sand stones, commonly known as the Bison formation. The remaining area is underlain by terrace deposits of sand, silt, and clay with a low permeability (SCS, soil survey, 1974)[52]. Most floodplain areas along the streams are open grass and farmland except the riparian areas are densely wooded.

The current land use in the watershed is primarily open space and agriculture with scattered low density residential and light commercial usage. Oklahoma City is experiencing rapid growth and expanding urbanization at the present time. The City has a growing plan for the eastern and southern portions of the basin to become medium to heavy industrial and commercial zones. In the past, flood problems were generally confined to local tributaries. The increase of flood problems is expected with anticipating land-uses and developments in the future.

5.3 DATA ACQUISITION AND INPUT DEVELOPMENT

The basic data required for this study include maps (land, soil, and topography), basin characteristics, meteorologic and streamflow records, channel configurations, and economic data for the Cow Creek basin.

Maps, basin, and channel information were obtained from the U.S. Geologic Survey. Precipitation records for Oklahoma City, considered typical of the basin, were available from year 1891 to the present. There was no stream gaging station within the basin. However, streamflow records were available at two nearby stations: one on the Worley Creek near Tuttle (station no. 7228930), the other on the Canadian River near New Castle (station no. 7228960). The records of these two streams with similar size and comparable hydrologic characteristics were used by statistical analysis (Beard, 1962)[6] to calibrate the parameters of the basin. The economic data were not easy to obtain. However, substantial data were sampled and collected from various sources. Then, the data were consolidated by statistical analysis as described in chapter III. Estimations of property value for different land-use categories were tabulated in Table 1, 2, and 3.

To apply the model, certain parameters and input need to be developed and transformed into the right format prior to the performance of hydrologic, hydraulic, and economic analyses. These include basin and stream characteristics, loss rate, unit hydrograph parameters, channel routing criteria, design storm pattern, flow-frequency relationship, and flow-damage relations. The bases of developing these input data were described in chapter IV. They were constructed and summarized in the following manner.

5.3.1 Basin Charcteristics

The Cow Creek basin was divided into 13 subbasins and 7 reaches as diagramed in Figure 25. Tables 14 & 15 summarize the physical characteristics of each of the subbasins and reaches. Subbasin areas were measured with a planimeter. The average subbasin land slope was determined by averaging several representative land slopes, each was measured by dividing the elevation difference of two contour lines into the normal distance between these lines depicted on the topographic map. The length of overland was determined by dividing the subbasin area into the length of the reach receiving the subbasin's overland flow. Stream length was measured by a stadiometer along a map representative of the main channel from the outlet to the basin divide. Stream slopes were measured by dividing the difference of elevations between upstream and downstream locations by the stream length. Bottom widths were calculated by averaging the bottom widths of cross-sections within the respective reach. The drainage network among subbasins and stream reaches was illustrated in Figure 26.

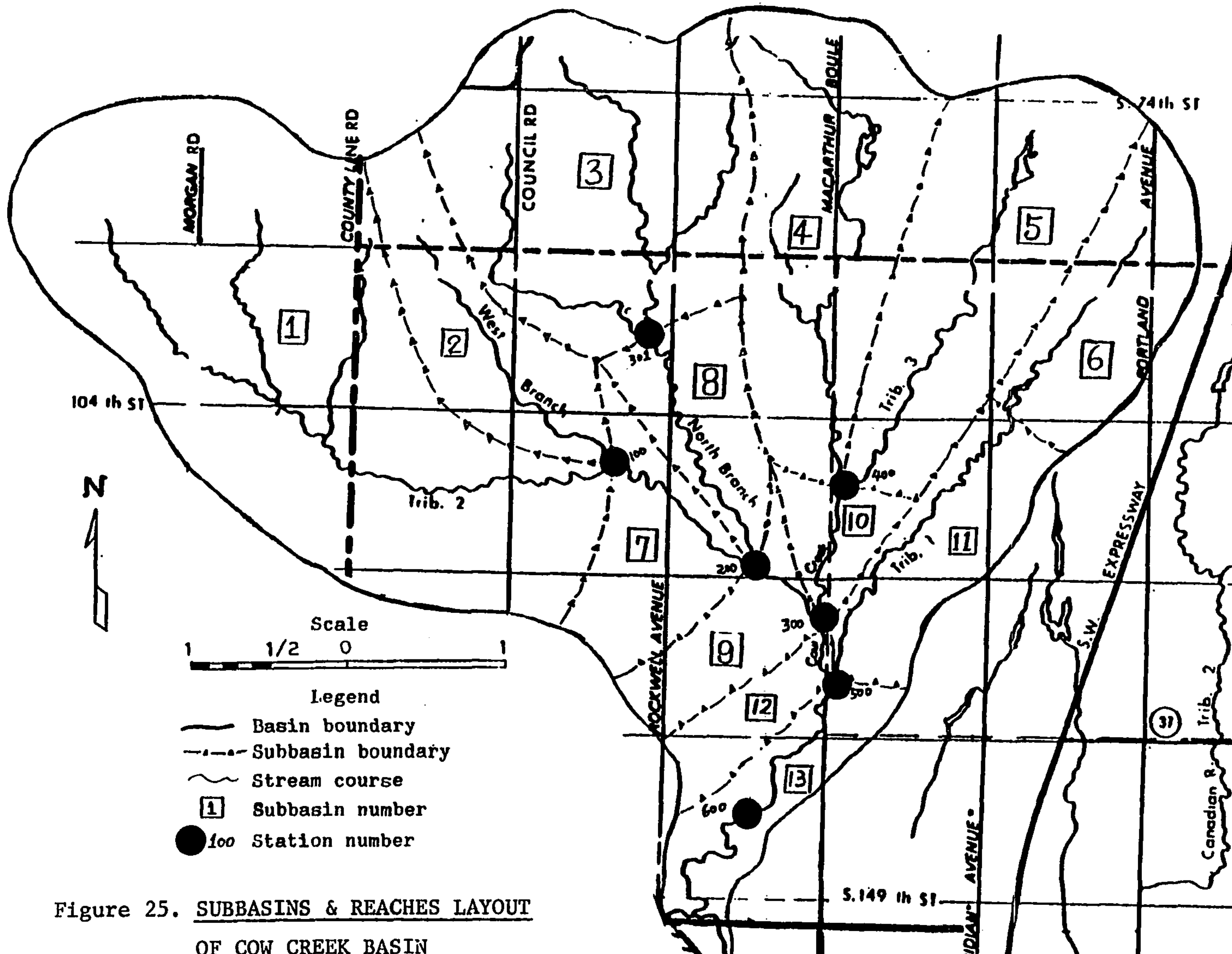


Figure 25. SUBBASINS & REACHES LAYOUT OF COW CREEK BASIN

TABLE 14

PHYSICAL CHARACTERISTICS OF COW CREEK BASIN

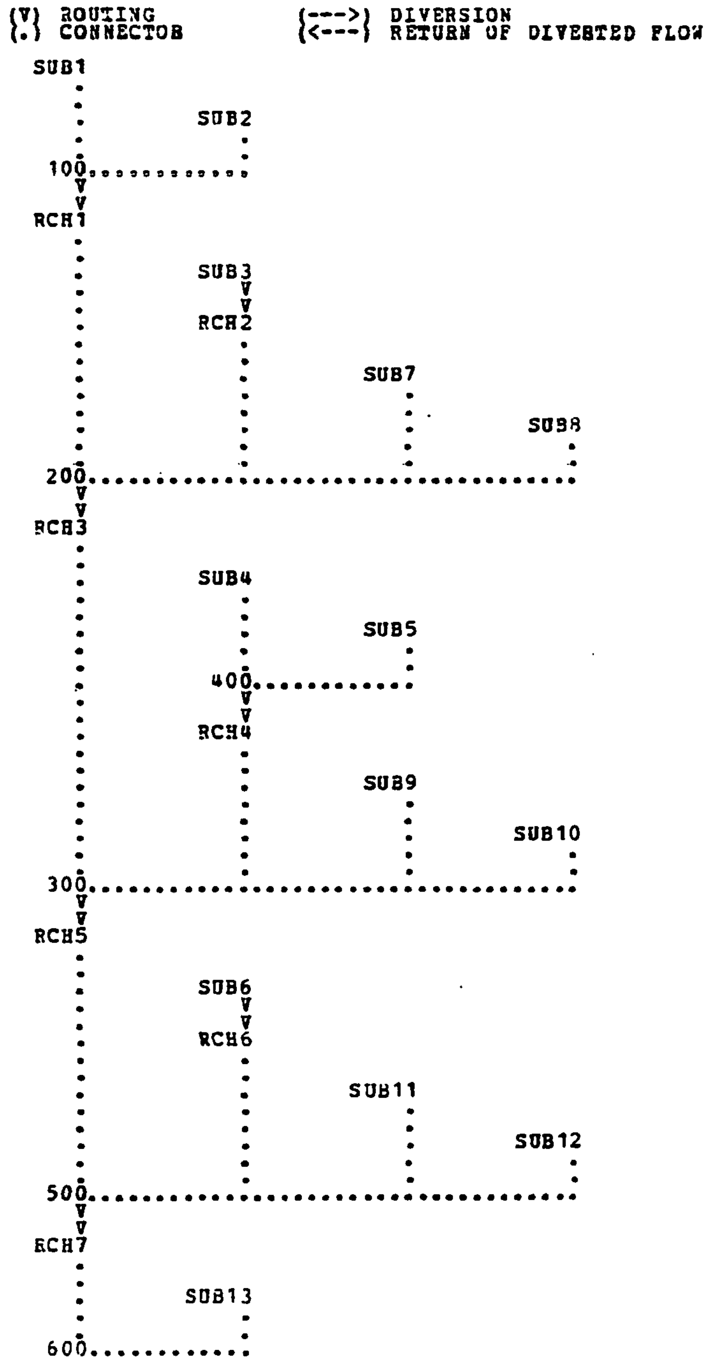
Subbasin	Area (sm)	Length (mi)	Slope (ft/mi)
1	3.5	3.2	21.8
2	1.5	0.9	33.8
3	3.0	1.1	36.4
4	3.2	2.8	18.7
5	2.2	2.0	20.8
6	1.0	1.1	26.0
7	0.9	1.5	16.1
8	1.0	1.9	16.1
9	0.8	0.5	23.4
10	0.8	1.0	13.5
11	1.2	2.2	18.7
12	0.6	0.4	14.6
13	1.5	1.5	10.0

TABLE 15

PHYSICAL CHARACTERISTICS OF COW CREEK REACHES

Reach	Length (ft)	Flowing slope (%)	Side slope (H/V)	Bottom Width (ft)
1	7890	0.10	2	5
2	9950	0.30	2	6
3	2630	0.10	2	8
4	5230	0.05	2	7
5	2100	0.50	2	9
6	11500	0.60	4	3
7	7850	0.10	2	8

Figure 26. A SCHEMATIC DIAGRAM OF STREAM NETWORK



5.3.2 Imperviousness and Loss Rate

The subbasin percentage of impervious cover was determined by superimposing a grid on basin areal photographs, averaging the number of grid points with roof tops, streets, and other infrastructural activities, and dividing by the total number of grid points on the subbasin (Thomas & Corley, 1977)[55].

The loss rates were computed by the SCS method as described in chapter III. The average moisture condition (see Table 6) was assumed as the antecedent moisture condition throughout the testing for this basin. The CN values, modified and adjusted according to changes of urbanization intensity are tabulated in Table 16.

TABLE 16

CN VALUES FOR VARIOUS EXTENT OF URBANIZATION

Subbasin	EC*	Iu* =25%	Iu=50%	Iu=75%	Iu=100%
1	77	83	88	92	98
2	74	81	86	90	97
3	76	83	87	91	98
4	75	82	87	91	98
5	76	83	87	91	98
6	75	82	87	91	97
7	78	84	88	92	98
8	77	83	88	92	98
9	77	83	88	92	98
10	75	82	87	91	97
11	76	83	87	91	98
12	77	83	88	92	98
13	77	83	88	92	98

* EC denotes the existing condition.

* Iu is the intensity of urbanization as defined in eq 3.1.

5.3.3 Unit Hydrograph Parameters

The Clark method was selected to develop unit hydrographs. The parameters, time of concentration (Tc) and storage coefficient (R), were initially estimated by equations developed by the U.S. Geologic Survey using a regression analysis (Thomas & Corley, 1977)[55].

$$T_c = 0.388 L^{0.440} S^{-0.345} I^{2.20} \qquad \text{(Eq. 5.1)}$$

$$R = 0.093\ A^{.229}\ S^{-.345}\ I^{2.92} \qquad \text{(Eq. 5.2)}$$

where, A is the drainage area in square miles, S is main-channel slope in feet per mile, L is the main-channel length in miles, and I denotes the 2-year, 24-hour rainfall in inches.

HEC-1 automatically calibrated these two parameters in "trial runs", and determined the "best-fit" unit hydrographs for selected design storms. Table 17 summarizes results of Tc and R for each basin.

TABLE 17

UNIT HYDROGRAPH PARAMETERS (Existing Condition)

Subbasin	Tc (hr)	R (hr)
1	2.9	2.0
2	1.4	1.4
3	2.0	1.6
4	2.8	2.1
5	2.4	1.5
6	1.7	2.0
7	2.4	1.7
8	2.8	1.8
9	1.3	1.0
10	2.1	1.6
11	2.7	1.6
12	1.4	1.4
13	3.3	2.4

Further, Tc and R were optimized and converted to Tp (basin leg time) and Cp (Snyder's coefficient). These two Snyder's parameters were adjusted by the SCS method, and then used in the multiplan analysis to account for the effects of the increased imperviousness due to additional extent of land use and decreased lag time resulting from channel hydraulic modefication. Table 18 summarizes these adjustments.

TABLE 18

BASIN LAG TIME (Tp) MODIFIED WITH THE EXTENT OF URBANIZATION

Subbasin	* EC Tp (hr)	Iu=25% Tp (hr)	Iu=50% Tp (hr)	Iu=75% Tp (hr)	Iu=100% Tp (hr)
1	2.46	2.19	1.94	1.89	1.87
2	1.31	1.15	1.01	0.98	0.97
3	1.74	1.53	1.36	1.32	1.31
4	2.41	2.12	1.88	1.81	1.78
5	2.03	1.79	1.58	1.52	1.50
6	1.45	1.28	1.13	1.10	1.09
7	1.99	1.77	1.57	1.53	1.51
8	2.36	2.10	1.86	1.82	1.79
9	1.10	0.98	0.87	0.85	0.84
10	1.82	1.60	1.42	1.37	1.35
11	2.19	1.95	1.71	1.66	1.64
12	1.31	1.17	1.03	1.01	1.00
13	2.85	2.54	2.25	2.19	2.17

* EC denotes the existing condition

5.3.4 Routing criteria

First, the Muskingum K's were estimated by the folloing empirical formula:

$$K = L / v \qquad \text{(Eq. 5.3)}$$

where, K =the reach travel time in hours,

L = the reach length in miles,

v = the average reach velocity in miles per hour.

These initial K's values for each routing reach were used in a number of HEC-1 computer runs to model basin hydrologic and hydraulic responses, and to generate peak discharges for selected storm events. The final routing criteria for multiflood-multiplan computations were obtained from HEC-2 output of storge (SV) and outflow (SQ) data resulting from channel capacity runs.

5.3.5 Design Storm Pattern

Since the historical streamflow will not repeat exactly in the future, especially with anticipated changes in land-uses, hypothetical streamflow for selected storms (eg. 10, 25, 50, and 100-year recurrence) were constructed by procedures established in the U.S. Water Council Guidelines for determining flood flow frequency (WRC, 1976).[63] The average point depths were taken from the isopluvial maps (NWS: TP-40,1961;[43] HYDRO-35, 1977)[44] for the study area for return periods from 2 to 100 years and for durations from 5 minutes to 24 hours as tabulated in Table 19.

HEC-1 automatically adjusted the point rainfall to area rainfall and distributed the storm patterns for each subbasin according to the depth-duration-area data specified in input.

TABLE 19

POINT RAINFALL FOR SYNTHETIC STORMS

Storm Duration	2 yr (in)	5 yr (in)	10 yr (in)	25 yr (in)	50 yr (in)	100 yr (in)
5 min	0.64	0.56	0.63	0.72	0.80	0.87
10 min	0.78	1.01	1.10	1.24	1.35	1.45
15 min	1.01	1.33	1.43	1.60	1.73	1.86
30 min	1.40	1.80	2.01	2.34	2.58	2.84
1 hr	1.82	2.29	2.62	3.10	3.46	3.85
2 hr	2.01	2.57	2.95	3.50	3.93	4.36
3 hr	2.23	2.86	3.29	3.91	4.38	4.87
6 hr	2.78	3.59	4.15	4.95	5.57	6.18
12 hr	3.23	4.24	4.91	5.86	6.60	7.34
24 hr	3.75	4.89	5.68	6.78	7.65	8.50

5.3.6 Flow-Frequency Relation

A discharge-frequency statistical analysis has been made for statewide gaging stations located in Oklahoma State by the U.S. Geologic Survey using regression equations to associate the flow rates with basin and climatic characteristics (Thomas & Corley, 1977)[55].

Two flow-frequency curves were conducted based on HEC-1 rainfall- runoff modeling and recorded flow rates for stations on Worley Creek and Canadian River Tributary; the results were found parallel with the ones estimated by U.S. Geological Survey.

Then, the synthetic streamflows for selected recurrence intervals from 2 to 100 years for the Cow Creek basin were developed by HEC-1 program using parameters (Tc and R) estimated from regression equations (Eq 5.1 & 5.2). Additional computer runs were made to adjust parameters by calibration on a regional basis. Table 20 summarizes the recommended peak discharge rates at selected index points.

TABLE 20

RECOMMENDED PEAK FLOW (Qp) AT SELECTED INDEX POINTS

Station	2-yr (cfs)	5-yr (cfs)	10-yr (cfs)	25-yr (cfs)	50-yr (cfs)	100-yr (cfs)
100	570	950	1260	1700	2080	2440
200	1140	1890	2510	3400	4150	4860
400	560	940	1260	1720	2120	2500
300	400	680	910	1240	1520	1780
500	2180	3620	4780	6400	7600	8610
600	2340	3710	4880	6590	7690	9010

* see Figure 26 for station location

5.3.7 Flow Damage Relationship

The methods of determining the parameters comprised in the flood damage model for economic analysis were described in chapter III. Table 21 summarizes the estimations of these parmeters. The flow-damage data were constructed for a range of selected frequency events (eg. 2-, 5-, 10-, 25-, 50-, 100-year intervals) based on the outcome of the flood damage model, and the relationship of flow-frequency established earlier.

TABLE 21. ESTIMATION OF PROPERTY FLOOD DAMAGE COST

Parameter	Value	Estimating Method
Pv	Table 1, 2, and 3	Pv=L+S+C (eq 3.2) projected from tax assessment records and local real estates, and data compiled by Census of Population and Housing (1980-1982)
Kc	0.030	regression analysis based on depth-damage relationship (table 4)
Ci	vary with t years	regression analysis based on consumer cost index where the yearly inflation rate was computed to be 8% (eq 3.7 & 3.8)
Iu	ranging from 0 to 1	estimated by taking the ratio of the space occupied by urban activities to the total land area, Iu of 1.0 representing full development without zoning
Ve	=(Vt-Vc) vary with Vt and Vc	Vt, total runoff volume, resulted from HEC-1 computation; Vc, channel capacity, obtained from HEC-2 run.
Ri	vary	computed by eq 3.23

5.3.8 Channel Configurations

The data required to perform water surface profiles and floodway encroachments were available from the U.S. Geologic Survey.

The starting water elevations at downstream points for each design storm were computed by using the slope-area method available in HEC-2. The discharge rates for each selected design storm were obtained from the output of HEC-1 discharge-frequency analysis. The Manning's n values, used to signify the channel and floodplain roughness characteristics, were assigned during the field reconnaissance. They were made based on engineering judgment and the methodology described in "Open Channel Hydraulics" (Chow, 1959)[14]. Table 22 presents a summary of roughness factors used in the streams.

TABLE 22

ROUGHNESS FACTORS USED FOR COW CREEK BASIN

Stream	Channel From	n Value To	Overbank From	n Value To
Main Stream	0.035	0.100	0.040	0.110
Tributary #1	0.030	0.070	0.035	0.075
Tributary #2	0.040	0.090	0.045	0.130
Tributary W2	0.040	0.100	0.040	0.110
Tributary N2	0.035	0.080	0.035	0.095
Tributary #3	0.040	0.080	0.035	0.095

* Tributary W2 & N2 denote the west and north branches of Tributary #2

5.3.9 Cost of Land-Use Control

As James stated (1971)[34] "....The economic loss caused by outside forces, such as floodplain zoning , to prevent the realization of the full potential income from the land would equal to the difference between the potential and the actual income..." In other words, the economic loss due to land-use control or floodplain regulation would be computed as the difference between the potential land income (Lp) that would be expected when land was in its maximum use and the actural land income (La) would be experienced when land-use was under control.

The average annual land cost due to land-use control would be determined as:

$$ALC = (F_{cr})\left[(F_{pf})(L_p) - L_a\right] \qquad \text{(Eq. 5.4)}$$

where, ALC denotes the annual cost of preventing full land development on floodplain based on projected land market value. Lp and La are previously defined. Fcr represents the capital recovery factor based on projected discount rate i in t years, and Fpf is the factor to convert the future value of Lp to present value based on interest rate j for t years. Table 23 summarizes the estimations of annual land-use control cost developed for the studied basin.

TABLE 23

ESTIMATION OF ANNUAL COST OF LAND-USE CONTROL

Parameter	Value	Estimation Method
Lp (\$/acre)	vary (Iu)	Estimated from projected land use and value of land equvalent to t years later
La (\$/acre)	vary (Iu)	projected from tax assessment records,local real estate investments
Fcr (A/P,i%,t)	0.1359	capital recovery factor for 6% and 10 years
Fpf (P/F,j%,t)	0.4632	Convert future value factor to present value based on 8% & 10 yr

5.3.10 Cost of Channel Excavation

The amortized annual cost for channel excavation was estimated by the following equation.

$$ACE_i = (C_u)(V_{di})(F_{cr}) \qquad \text{(Eq. 5.5)}$$

where, ACEi = the amortized annual cost for channel excavation for bottom width of i feet.

Cu = unit cost of excavation in $/cfs.

Vdi = volume of dirt need to be excavated for bottom width of i feet.

Fcr = capital recovery factor, same as in eq 5.4.

From channel geometry, by taking cross-sectional areas and length of channel to be excavated into consideration as shown in Figure 27, Vdi was derived and transformed into the following relationships:

For triangle cross-sections:

$$V_{di} = (b_i)(L)(Y) \qquad \text{(Eq. 5.6)}$$

For trapezoid cross-sections:

$$V_{di} = (b_i - b_o)(L)(Y) \qquad \text{(Eq. 5.7)}$$

where, Ai = trapezoidal cross-sectional area (sq-ft) after excavation with bottom width i feet and side slope to be paralled to the original cross-sections,

Ao = original cross-sectional area in sq-ft,

L = length of channel to be excavated in ft,

bi = bottom width after excavation in ft,

Y = elevation difference between bank and channel bottom in ft.

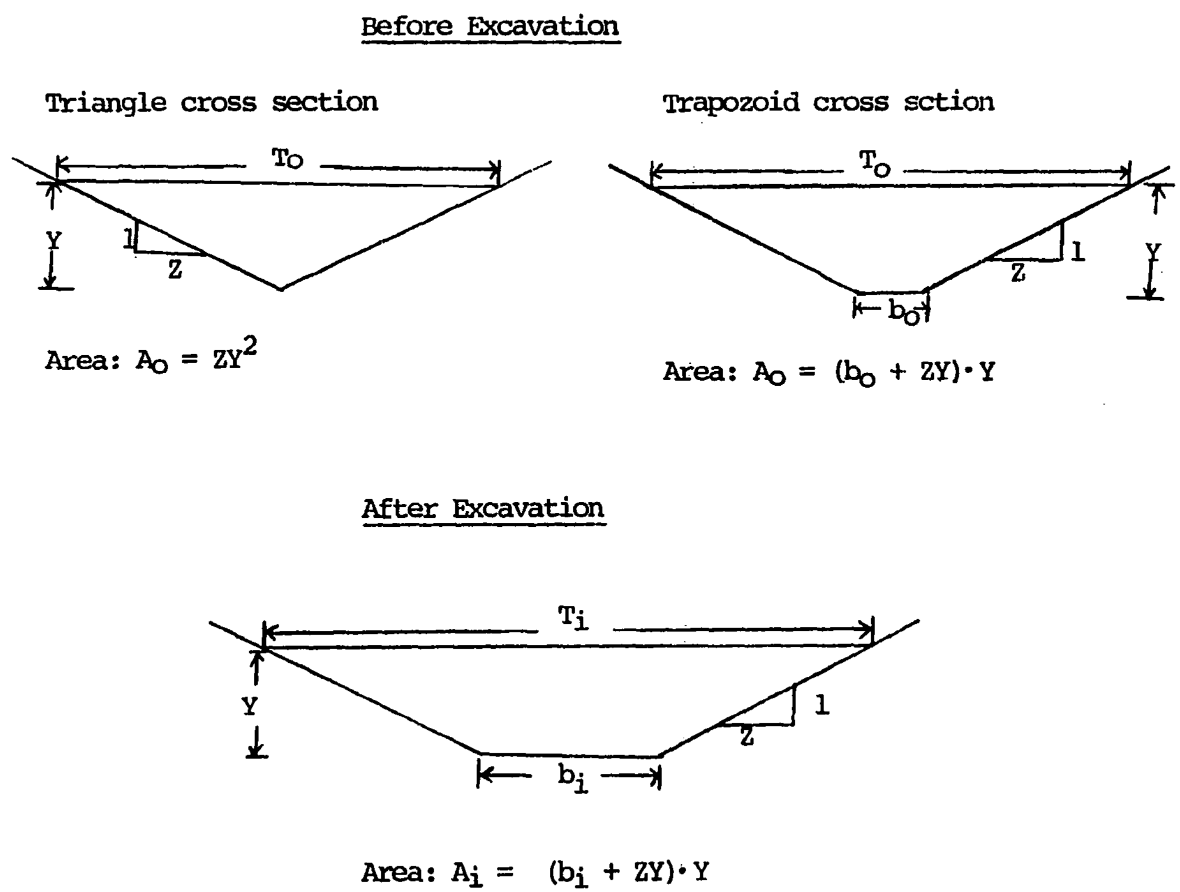


Figure 27. GEOMETRIC ELEMENTS OF CHANNEL SECTIONS

Combining equtions 5.5, 5.6, and 5.7 yields the following relationships:

$$ACE_i = (b_i)(K) \quad \text{or} \quad ACE_i = (b_i - b_o)K \qquad \text{(Eq. 5.8)}$$

where, K denotes the product of Cu, Fcr, L, and Y, which is non-varying with cuts of channel excavation. Therefore, the ratio of ACE's for different cuts can be expressed in a simplified manner as shown in Table 24 which summarizes the excavation cost ratios of channel improvements performed by altering bottom widths for studied streams.

TABLE 24

EXCAVATION COST RATIO FOR VARIOUS BOTTOM WIDTH (bi)

Reach	bi=10ft bo *	bi=20ft bo *	bi=40ft bo *	bi=60ft bo *	bi=80ft bo *
1	1.0	3.0	7.0	11.0	15.0
2	1.0	3.5	8.5	13.5	18.5
3	1.0	6.0	16.0	26.0	36.0
4	1.0	4.3	11.0	17.7	24.3
5	1.0	11.0	31.0	51.0	71.0
6	1.0	2.4	5.3	8.1	11.0
7	1.0	6.0	16.0	26.0	36.0

* bo: refer to table 15 for original channel bottom width

5.4 ANALYSES AND OUTPUT RESULTS

The hydrologic, hydraulic, and economic analyses as outlined in chapter IV, were performed for the following propositions:

a) existing condition (plan#1): agriculture and open space,

b) future development plans including 4 incremental levels of urbanization: Iu=25% (plan#2), Iu=50% (plan#3), Iu=75% (plan#4), Iu=100% (plan#5); with no target of protection,

c) various degrees of channel improvement proposed for a reasonably future development plan.

5.4.1 Hydrologic study and effects of land-uses on floods

After developing the rainfall-runoff parameters, the input was structured according to the topographic layout of the basin and applied to HEC-1 to generate design storm hydrographs.

Additional computer runs were made to construct the flow-frequency relationships for selected flood intervals. Then, a multiflood-multiplan computation was made in one single computer run to determine the flood characteristics for the existing and proposed future plans. Tables 25, 26 and 27 highlight the major effects of land-use on flood peak discharges, total runoff volume, and time of concentration.

These results show that peak discharge (Qp) and runoff volume (Vt) increase with the intensity of urbanization. The more frequent storms appear to be mainly affected by changes of land use. For example, a magnitude of 1 to 4 in Qp was changed at station 300 for 2-year storm, compared to a magnitude of 1 to 2 for 100-year storm under same conditions (Table 25). Conversely, the time of concentration

(Tc) decreases with the increase of urbanization as shown in Table 27.

TABLE 25

PEAK DISCHARGE (Qp) RESULTING FROM MULTIFLOOD-MULTIPLAN ANALYSIS

Flood Qp Sta.	Plan	2-yr cfs	5-yr cfs	10-yr cfs	25-yr cfs	50-yr cfs	100-yr cfs
100	P#1	573	950	1260	1700	2080	2436
	P#2	1020	1500	1880	2410	2850	3260
	P#3	1530	2110	2570	3180	3690	4160
	P#4	1900	2540	3020	3660	4180	4660
	P#5	2150	2800	3280	3930	4450	4933
200	P#1	1140	1890	2510	3400	4150	4860
	P#2	2030	2980	3750	4790	5630	6430
	P#3	2970	4115	4989	6131	7099	7987
	P#4	3722	4944	5930	7054	8044	8987
	P#5	4201	5427	6342	7575	8588	9405
300	P#1	1862	3123	4140	5544	6742	7733
	P#2	3372	4935	6120	7628	8806	10310
	P#3	4950	6690	7910	9880	11310	12460
	P#4	6144	7864	9368	11277	12547	14638
	P#5	6810	8530	10260	12030	13900	15690
400	P#1	555	942	1266	1728	2124	2500
	P#2	1054	1566	1974	2539	3013	3458
	P#3	1631	2263	2752	3413	3958	4464
	P#4	2086	2778	3301	4002	4574	5102
	P#5	2374	3086	3620	4332	4911	5445
500	P#1	2180	3620	4790	6400	7610	8610
	P#2	3880	5710	6960	8460	9620	10950
	P#3	5700	7440	8620	10340	11810	13100
	P#4	6930	8570	9870	11750	13170	14630
	P#5	7580	9200	10670	12510	13980	15550
600	P#1	2340	3720	4890	6580	7690	9020
	P#2	3950	5790	7120	8740	10190	11622
	P#3	5710	7450	8910	10920	12510	13890
	P#4	6920	8810	10390	12440	13970	15500
	P#5	7530	9620	11260	13250	14810	16440

TABLE 26

TOTAL RUNOFF (Vt) RESULTS FROM MULTIFLOOD-MULTIPLAN ANALYSIS

Flood Vt Plan	2-yr ac-ft	5-yr ac-ft	10-yr ac-ft	25-yr ac-ft	50-yr ac-ft	100-yr ac-ft
RCH 1 P#1	183	300	397	534	650	760
P#2	308	450	562	717	846	967
P#3	427	588	713	881	1020	1149
P#4	527	699	830	1001	1147	1279
P#5	590	766	899	1075	1219	1351
RCH 2 P#1	111	184	243	327	398	465
P#2	190	278	347	443	522	597
P#3	257	355	430	532	616	694
P#4	318	422	501	607	693	773
P#5	358	464	544	651	738	818
RCH 3 P#1	366	601	794	1068	1300	1520
P#2	615	897	1120	1426	1682	1922
P#3	847	1166	1411	1744	2018	2272
P#4	1038	1377	1634	1978	2259	2518
P#5	1165	1513	1774	2122	2406	2667
RCH 4 P#1	184	307	410	555	678	795
P#2	318	468	586	751	888	1017
P#3	456	630	763	944	1093	1232
P#4	566	752	893	1081	1235	1377
P#5	636	826	969	1160	1315	1458
RCH 5 P#1	598	986	1306	1758	2141	2505
P#2	1017	1478	1859	2372	2800	3200
P#3	1420	1956	2369	2928	3389	3817
P#4	1751	2324	2758	3340	3814	4253
P#5	1964	2552	2991	3578	4056	4497
RCH 6 P#1	37	62	81	109	133	156
P#2	62	90	113	144	170	194
P#3	85	117	142	175	203	228
P#4	105	139	165	200	228	255
P#5	117	152	179	214	243	269
RCH#7 P#1	687	1119	1481	1993	2428	2840
P#2	1152	1684	2105	2686	3170	3628
P#3	1607	2213	2681	3315	3836	4321
P#4	1985	2635	3127	3787	4325	4823
P#5	2230	2895	3395	4062	4605	5105

TABLE 27

TIME OF CONCENTRATION (Tc) VARIATION WITH EXTENT OF URBANIZATION

Subbasin	EC Tc (hr)	Iu=25% Tc (hr)	Iu=50% Tc (hr)	Iu=75% Tc (hr)	Iu=100% Tc (hr)
1	2.88	2.63	2.31	2.17	2.17
2	1.43	1.30	1.23	1.19	1.18
3	2.00	1.91	1.69	1.50	1.48
4	2.79	2.57	2.17	2.10	2.06
5	2.39	1.99	1.83	1.82	1.82
6	1.67	1.35	1.24	1.23	1.22
7	2.38	2.03	1.91	1.88	1.88
8	2.77	2.61	2.14	2.12	2.12
9	1.30	1.22	0.94	0.92	0.90
10	2.12	1.97	1.66	1.57	1.54
11	2.71	2.39	2.08	2.06	2.06
12	1.43	1.31	1.23	1.21	1.21
13	3.34	3.03	2.65	2.63	2.63

* EC denotes the existing condition

5.4.2 Hydraulic study and effects of floods on land use

The existing water surface profiles and flood boundaries for selected storms were computed. The channel capacity and the outflow-storage relationship for each reach were determined by a multi-profiles computation using a series of sequential flow rates. Then, the output was used to join the channel routing in the final multiflood-multiplan computation.

Next, the water profiles and floodway encroachments were made by changing peak discharges, resulting form HEC-1 multiflood-multiplan runs, for future conditions. The results were compared with the ones generated from existing condition.

Table 28 summarizes the effects of flood characteristics (i.e. peak flow) of existing and future conditions on floodplain zoning and water surface profile. The flood hazard factor (FHF) was defined by FIA as the average weighted difference between the 10-year and 100-year flood water surface elevations expressed to the nearest one-half foot, and shown as a three-digit code. The FHZ, flood hazard zone, was designated as A1 through A30 according to the corresponding FHF value (HEC-2 manual, 1981)[33]. These two factors were usually used in flood studies as flood information to correlate flood insurance rates; the higher the FHF and FHZ, the higher the insurance premium rate. Also, they can be ultilized as good indices for floodplain zoning.

Table 28. CHANGES OF WATERPROFILES AND FLOOD ZONES DUE TO URBANIZATION

		Plan #1			Plan #2			Plan #3			Plan #4			Plan #5		
Reach	ELV_{10}	WED	FHF	FHZ	WED	FHF	FHZ	WED	FHF	FHZ	WED	FHF	FHZ	WED	FHF	FHZ
#1	1205.20	2.68	025	A5	3.40	035	A7	4.23	040	A7	6.72	065	A13	7.40	075	A15
#2	1209.15	1.49	015	A3	4.05	040	A8	6.37	065	A13	8.08	080	A16	8.70	085	A17
#3	1194.95	3.95	040	A8	4.45	045	A9	4.82	050	A10	5.31	055	A11	5.47	055	A11
#4	1196.11	2.02	020	A4	2.47	025	A5	4.01	040	A8	4.59	045	A9	4.66	045	A9
#5	1190.51	1.90	020	A4	3.58	035	A7	4.15	040	A8	4.67	045	A9	5.27	055	A11
#6	1211.72	1.82	020	A4	1.96	020	A4	2.51	025	A5	3.15	030	A6	5.11	050	A10
#7	1183.89	1.96	020	A4	3.25	035	A7	3.65	035	A7	4.31	045	A9	4.83	050	A10

Notes:

ELV_{10} -- the weighted water surface elevation of 10-year flood of existing hydrologic condition

WED --- weghted average elevation difference between the 100-year flood of respective plans and the 10-year flood of exixting condition

FHF --- flood hazard factor defined by FIA (1977)

FHZ --- flood hazard zone designated according the respective FHF value (FIA, 1977)

As shown in this table, the increased FHF and FHZ signify the increasing severity of floods on the land as urbanization intensity increases. In other words, changes of flood characteristics (peak flow per se) due to intensified urbanization have significant impacts on flood zoning which should be taken into consideration in future land use and the development policy.

5.4.3 Economic Evaluation

The benefits of flood damage reduction were computed as the difference between the annual damage value (EAD) of the studied basin, with and without changes in land-use or flood control projects.

The EAD computation was performed by a multiflood-multiplan damage analysis to include the existing and future conditions. The results are presented in Table 29.

Meanwhile, the average annual land cost (ALC) due to land-use control with different levels of urbanization were estimated. The results were compared with EAD values as summarized in table 30. The most "promising" land-use pattern, level of urbanization per se, was selected as the one that yielded the maximum net benefit between EAD and ALC values. In this case, a 50% of land development was recommended as the result of trading-off between the economic scales of land-use and flood damage control.

* Table 29. Expected Annual Damage Summary For Various Extent Of Urbanization

STREAM STATION	DAMAGE REACH	TOWNSHIP	*	DAMAGE CATEGORY	EXPECTED ANNUAL DAMAGE PLAN 1	PLAN 2	PLAN 3	PLAN 4	PLAN 5
RCH1	1		*	1 AGRIC	4.59	0.0	0.0	0.0	0.0
			*	2 RESID	0.0	24.59	70.95	313.16	575.80
			*	3 INC/COM	0.0	65.96	199.82	861.59	1453.60
			*	TOTAL	4.59	90.55	270.77	1174.75	2029.40
		DAMAGE CHANGE (BENEFITS)			BASE	-85.96	-266.18	-1170.16	-2024.81
RCH2	2		*	1 AGRIC	2.93	0.0	0.0	0.0	0.0
			*	2 RESID	0.0	28.85	111.98	1546.55	400.21
			*	3 INC/COM	0.0	79.09	308.31	855.41	1100.62
			*	TOTAL	2.93	107.94	420.29	2401.96	1500.83
		DAMAGE CHANGE (BENEFITS)			BASE	-105.00	-417.35	-2399.03	-1497.89
RCH3	3		*	1 AGRIC	3.61	0.0	0.0	0.0	0.0
			*	2 RESID	0.0	43.12	173.45	545.06	979.53
			*	3 INC/COM	0.0	121.02	464.31	1498.70	2693.74
			*	TOTAL	3.61	164.14	637.76	2043.76	3673.27
		DAMAGE CHANGE (BENEFITS)			BASE	-160.53	-634.16	-2040.15	-3669.67
RCH4	4		*	1 AGRIC	6.10	0.0	0.0	0.0	0.0
			*	2 RESID	0.0	57.22	271.59	582.43	834.86
			*	3 INC/COM	0.0	157.55	747.41	1406.13	2295.79
			*	TOTAL	6.10	214.78	1018.99	1988.56	3130.65
		DAMAGE CHANGE (BENEFITS)			BASE	-208.67	-1012.89	-1982.46	-3124.55
RCH5	5		*	1 AGRIC	11.72	0.0	0.0	0.0	0.0
			*	2 RESID	0.0	132.94	725.13	1666.73	1697.66
			*	3 INC/COM	0.0	365.68	2061.45	4583.64	4668.91
			*	TOTAL	11.72	498.61	2786.58	6250.38	6366.57
		DAMAGE CHANGE (BENEFITS)			BASE	-486.89	-2774.85	-6238.65	-6354.84
RCH6	6		*	1 AGRIC	0.45	0.0	0.0	0.0	0.0
			*	2 RESID	0.0	2.14	11.67	36.18	67.40
			*	3 INC/COM	0.0	5.90	31.62	99.95	184.93
			*	TOTAL	0.45	8.05	43.29	136.13	252.33
		DAMAGE CHANGE (BENEFITS)			BASE	-7.60	-42.84	-135.68	-251.88
RCH7	7		*	1 AGRIC	3.34	0.0	0.0	0.0	0.0
			*	2 RESID	0.0	78.67	391.77	998.04	1651.59
			*	3 INC/COM	0.0	216.38	1077.24	2744.87	4541.92
			*	TOTAL	3.34	295.05	1469.01	3742.91	6193.51
		DAMAGE CHANGE (BENEFITS)			BASE	-291.71	-1465.67	-3739.57	-6190.16
BASIN TOTAL			*	1 AGRIC	32.74	0.0	0.0	0.0	0.0
			*	2 RESID	0.0	367.54	1756.54	5688.16	6207.04
			*	3 INC/COM	0.0	1011.58	4890.14	12050.27	16939.50
			*	TOTAL	32.74	1379.12	6646.67	17738.43	23146.54
		DAMAGE CHANGE (BENEFITS)			BASE	-1346.37	-6613.93	-17705.68	-23113.80

* See Appendix-A for Program Input

TABLE 30

TRADE-OFF BETWEEN EAD AND ALC VALUES

Value	Plan #1 EC	Plan #2 Iu=25%	Plan #3 Iu=50%	Plan #4 Iu=75%	Plan #5 Iu=100%
EAD	Base 0	-1346k	-6614k	-17706k	-23114k
ALC	Base 0	+3070k	+9890k	+16523k	+22848k
Net	0	+1724k	*+3276K	-1182 k	-265 k

EAD--changes of expected annual flood damages resulting from Table 29

ALC--annual land cost computed by eq.5.4 and table 23

"*"--the maximum net benefit resulting from EAD and ALC trade-off

Next, channel improvement was analyzed and evaluated as the example alternative to show the sensitivity of this economic analysis. The protection target was set for the

50-year, 6-hour duration storm. By altering channel bottom width (i.e. bi = 10, 20, 40, 60, and 80 feet), the channel improvement computation, available in HEC-2, was conducted for the selected future growth plan (50% of urbanization) to assess channel capacities for these various degrees of channel excavation. Also, the output of storge-outflow were used to join with HEC-1 in multiflood, multiplan evaluation.

Further, EAD computations were performed to analyze the potential outcome of flood damage reductions with and without various level of channel improvements. The results are presented in Table 31.

The amortized annual channel excavation cost (ACE) were estimated in the manner presented in Table 24. To select the "best plan" for this example, the cost-benefit analysis was applied to evaluate the economic feasibility among these selected alternatives. The results were summarized in Table 32. The alternative with sufficient channel capacity and yielding the highest B/C ratio as the result of cost-benefit analysis was selected as the "best plan".

DAMAGE REACH	TOWNSHIP	*	DAMAGE CATEGORY	EXPECTED ANNUAL DAMAGE PLAN 1	PLAN 2	PLAN 3	PLAN 4	PLAN 5
RCH4		*	1 AGRIC	0.0	0.0	0.0	0.0	0.0
		*	2 RESID	280.21	152.53	34.60	0.28	0.0
		*	3 IND/COM	771.30	419.95	95.48	0.77	0.0
		*						
		*	TOTAL	1051.50	572.47	130.08	1.04	0.0
	DAMAGE CHANGE (BENEFITS)			BASE	479.03	921.42	1050.46	1051.50
RCH5		*	1 AGRIC	0.0	0.0	0.0	0.0	0.0
		*	2 RESID	801.89	586.96	359.57	33.23	6.80
		*	3 IND/COM	2276.40	1721.05	1027.08	97.90	20.39
		*						
		*	TOTAL	3078 30	2308.01	1386.65	131.13	27.19
	DAMAGE CHANGE (BENEFITS)			BASE	770.29	1691 65	2947.17	3051.10
RCH7		*	1 AGRIC	0.0	0.0	0.0	0.0	0.0
		*	2 RESID	419.07	35.02	7.90	0.0	0.0
		*	3 IND/COM	1152.46	96.34	21.70	0.0	0.0
		*						
		*	TOTAL	1571.52	131.36	29.59	0.0	0.0
	DAMAGE CHANGE (BENEFITS)			BASE	1440.16	1541.93	1571.52	1571.52
BASIN TOTAL		*	1 AGRIC	0.0	0.0	0.0	0.0	0.0
		*	2 RESID	1501.17	774.51	402.07	33.51	6.80
		*	3 IND/COM	4200.16	2237.34	1144.26	98.67	20.39
		*						
		*	TOTAL	5701.32	3011.85	1546.33	132.17	27.19
	DAMAGE CHANGE (BENEFITS)			BASE	2689.47	4154.99	5569.14	5674.12

* Table 31 <u>Expected Annual Damage Summary For Alternatives With Various Extent Of Channel Excavation</u>

* See Appendix-B for Program Input

TABLE 32

COST-BENEFIT ANALYSIS FOR ALTERNATIVES BASED ON A 50% DEVELOPMENT PLAN WITH/WITHOUT CHANNEL IMPROVEMENT

Parameter	ALT #1 bi=bo Iu=50%	ALT #2 bi=10ft Iu=50%	ALT #3 bi=20ft Iu=50%	ALT #4 bi=40ft Iu=50%	ALT #5 bi=60ft Iu=50%	ALT #6 bi=80ft Iu=50%
R4/Channel Capacity	-	-	+	+	+	
EAD net ACE ratio	base /	479K 1	921K 4.3	1050K 11.0	1052K 17.7	1053K 24.3
B/C ratio	NF	NF	* 214	94	59	43
R5/Channel Capacity	-	-	+	+	+	
EAD net ACE ratio	base /	770k 1	1692K 2.4	2947K 5.3	3051K 8.1	3054K 11.0
B/C ratio	NF	NF	* 705	556	377	278
R7/Channel Capacity	-	-	-	+	+	
EAD net ACE ratio	base /	1440K 1	1542K 6.0	1572K 16.0	1572K 26.0	1573K 36.0
B/C ratio	NF	NF	NF	* 98	60	44

Notes:
NF denotes non-feasible alternative
"-" represents insufficient channel capacity
"+" represents sufficient channel capacity
"*" signifies the selected alternative with the highest B/C ratio and with a sufficient capacity

Chapter VI

SUMMARY AND CONCLUSION

This study represents a new approach for estimating, analyzing, and evaluating the reducible flood damages in urban areas associated with land use and flood alleviation projects.

The model for estimating potential urban flood damages was developed and formulated using an interdisciplinary approach (Figures 2 & 3). The model parameters and variables were determined and verified by statistical analyses as addressed in chapter III.

The methodology, schematically shown in Figure 23, integrates the HEC-1 and HEC-2 programs in simulating the hydrologic responses and channel hydraulic characteristics for the Cow Creek basin under five propositions, including the existing and future plans, each varied with the degree of urbanization as elaborated in chapters IV and V.

The economic analysis involves computations of the following criteria: (1) the expected annual flood damage (EAD), (2) the annual land cost (ALC), and (3) the amortized annual channel excavation cost (ACE). The flood damage reduction benifits were computed, then the trade-off between the benifits of reducible EAD due to land use control and costs of

ALC from preventing urban development was made in order to select the most "promising" plan for future land use. Furthermore, channel improvement, chosen as the example alternative to demonstrate this technique, was tested by hydrologic and hydraulic analyses for various channel bottom widths. The cost benefit analysis was then applied to the hydraulic efficient ones for evaluating the economic feasibility of these alternatives. Tables 29-32 summarize these results.

This study resulted in several important conclusions. First, the flood model provides a quantitative means for estimating potential urban flood damages, especially since historical damage data is no longer valid due to rapid urbanization on floodplains.

Second, flood characteristics are significantly affected by land use. Hydrologic analysis for the Cow Creek basin shows that peak flow (Qp) and runoff volume (Vt) increase with the intensity of urbanization (tables 25 & 26). The more frequent storms appear to be more influenced by changes of land use. Conversely, the time of concentration (Tc) decreases with the increase of urbanization as summarized in table 27.

Third, besides land use, Qp and Vt are also affected by the frequency and the duration of storm events. Both increase as the increase in duration and/or frequency of storms, while Tc remains the same for changes of duration and frequency.

Fourth, changes of flood characteristics induce changes in flood land zoning. Hydraulic analysis shows that the increase of peak flow due to urbanization projected for the future has significant impacts on water surface profiles and floodway zoning (see Table 28). These effects should be taken into account in floodplain delineation and design of flood control projects to accomodate changes in future development. Conventionally, flood studies for watersheds were conducted based on present hydrologic conditions, which is not truely representative for the future, especially with changes in land use. It is recommended that a floodplain be zoned to coordinate with the forcasted outcome due to the anticipated future land use.

Fifth, the sensitivity of the economic scale with changes of land use and channel improvement has been demonstrated and results of this study (Tables 29 to 32) yield the most allowable amount of urbanization and selection of the "best" plan for flood alternatives. This technique provides a sound, quantitative economic basis for evaluating land use and flood alternatives.

Sixth, changes of land use has vital impacts on the watershed with regard to the following aspects: hydrologic responses of the basin, hydraulic characteristics of streams and changes of flood zones, and economic impacts in terms of potential increasing flood damages and land use profits. Therefore, land use control plays a significant role in urban flood management.

The uniqueness of this integrated approach includes the following:

a) The flood model provides a quantitative means for estimating future urban flood damages.

b) Minimum data and reasonable cost are required for model computation; with the capability and availability of the HEC-1 and HEC-2 programs, many complex propositions can be simulated in a single computer run.

c) This technique offers dual consideration between land use and floods; the effects of land use on floods and the effects of floods on land use can be explored by hydrologic and hydraulic analyses as outlined in chaper IV.

d) The scheme of this technique can be applied to other watersheds with modification of basin parameters and land use patterns to fomulate a general guide for evaluating land use proposals and flood alleviation projects.

Practically, as conducted in this study, the test basin can be first divided into discrete units in space and time. These lumped parameters are used to represent the average hydrologic and economic conditions for the model computation and to form a guideline in planning stage. For use in final design and implementation of selected plans, these lumped units must be subdivided as small as possible to represent the actual individual components. Due to the heterogenous characteristics of watersheds, each separate entity of floodplains requires an individual treatment for best management. Also, time variations demand continuous updating of analysis and periodic adjustment of the plan.

In addition to the technical and economic aspects dealt with in this study, the social, institutional, and environmental aspects, which are beyond the scope of this research, must be carefully weighted, and integrated in floodplain management to reach the balance of the pursuit of mankind's benefits and maintaining harmony with the river.

It is hoped that this study will be beneficial for the developer, urban planner, consulting engineering, and policy decision- maker in seeking flood problem solutions and effective planning for future land use.

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APPENDIX A

Program Input of a Multiflood-Multiplan Analysis for Various Extent of Urbanization

HEC-1 INPUT

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

   1          ID        EFFECT OF URBANIZATION ON RUNOFF, TIME OF CONCENTRATION & FLOOD DAMAGE
   2          ID        INVESTIGATION BY SUE LIN LEWIS
   3          ID        FLOOD DAMAGE ANALYSIS  (EXPECTED ANNUAL DAMAGE)
   4          ID        MULTIFLOOD, MULTIPLAN ANALYSIS
   5          ID        STREAM NETWORK COMPUTATION
   6          ID         25 YR,  6 HR DURATION
   7          ID        COW CREEK BASIN, OKLAHOMA CITY, OK
              *DIAGRAM
   8          IT       5                     100
   9          IO       5
  10          JP       5
  11          JR    PREC    0.56    0.72    0.84    1.00    1.13    1.25

  12          KK    SUB1
  13          KM        RUNOFF COMPUTATION
  14          BA     3.5
  15          BF       0    -0.1     1.2
  16          PH       4       0    0.63    1.44    2.88    3.29    3.75    4.80
  17          LS       0      77
  18          US    2.46    0.66
  19          KP       2
  20          LS       0      81      20
  21          US    2.19    0.66
  22          KP       3
  23          LS       0      85      40
  24          US    1.94    0.66
  25          KP       4
  26          LS       0      90      60
  27          US    1.89    0.66
  28          KP       5
  29          LS       0      94      80
  30          US    1.87    0.66

  31          KK    SUB2
  32          KM        RUNOFF COMPUTATION
  33          BA     1.5
  34          LS       0      74
  35          US    1.31    0.58
  36          KP       2
  37          LS       0      78      20
  38          US    1.15    0.58
  39          KP       3
  40          LS       0      83      40
  41          US    1.15    0.58
  42          KP       4
  43          LS       0      88      60
  44          US    0.98    0.58
  45          KP       5
  46          LS       0      93      80
  47          US    0.97    0.58

  48          KK     100
  49          KM        COMBINE
  50          KO       0       2       0       0
  51          HC       2

  52          KK    RCH1
  53          KM        ROUTING
  54          FS       1    FLOW      -1
  55          SV      20      33      47      60      74      91     107     124     140     200
  56          SV     231     276     326     340     350     365     397     429
  57          SQ     500    1000    1500    2000    2500    3000    3500    4000    4500    5000
  58          SQ    5500    6000    6500    7000    7500    8000    9000   10000

  59          KK    SUB3
  60          KM        RUNOFF COMPUTATION
  61          BA     3.0
  62          LS       0      76
  63          US    1.74    0.63
  64          KP       2
  65          LS       0      80      20
  66          US    1.53    0.63
  67          KP       3
  68          LS       0      84      40
  69          US    1.36    0.63
  70          KP       4
  71          LS       0      89      60
  72          US    1.32    0.63
```

```
KP      5
LS      0      94      80
US   1.31    0.63

KK   RCH2
KM        ROUTING
RS      1    FLOW      -1
SV     36      69     102     137     171     203     235     266     296     338
SV    369     419     472     505     514     540     591     631
SQ    500    1000    1500    2000    2500    3000    3500    4000    4500    5000
SQ   5500    6000    6500    7000    7500    8000    9000   10000

KK   SUB7

KM        RUNOFF COMPUTATION
BA    0.9
LS      0      79
US   1.99    0.63
KP      2
LS      0      82      20
US   1.77    0.63
KP      3
LS      0      86      40
US   1.57    0.63
KP      4
LS      0      90      60
US   1.53    0.63
KP      5
LS      0      94      80
US   1.51    0.63

KK   SUB8
KM        RUNOFF COMPUTATION
BA    1.0
LS      0      77
US   2.36    0.68
KP      2
LS      0      81      20
US   2.10    0.68
KP      3
LS      0      85      40
US   1.86    0.68
KP      4
LS      0      90      60
US   1.82    0.68
KP      5
LS      0      94      80
US   1.79    0.68

KK    200
KO      0       2       0       0
KM        COMBINE
HC      4

KK   RCH3
KM        ROUTING
RS      1    FLOW      -1
SV      6      11      14      18      22      28      37      46      55      63
SV     74      88     111     131     166     170     175     188
SQ    500    1000    1500    2000    2500    3000    3500    4000    4500    5000
SQ   5500    6000    6500    7000    7500    8000    9000   10000

KK   SUB4
KM        RUNOFF COMPUTATION
BA    3.2
LS      0      75
US   2.41    0.64
KP      2
LS      0      79      20
US   2.12    0.64
KP      3
LS      0      84      40
US   1.88    0.64
KP      4
LS      0      89      60
US   1.81    0.64
KP      5
LS      0      93      80
US   1.78    0.64
```

```
KK    SUB8
KM        RUNOFF COMPUTATION
BA     2.2
LS       0       75
US    2.03     0.59
KP       2
LS       0       79       20
US    1.79     0.59
KP       3
LS       0       84       40
US    1.58     0.59
KP       4
LS       0       89       60
US    1.52     0.59
KP       5
LS       0       94       80
US    1.50     0.59

KK     400
KO       0        2        0        0
KM        COMBINE
HC       2

KK    RCH4
KM        ROUTING
RS       1     FLOW       -1
SV      12       20       32       51      110      120      131      152      189      195
SV     208      215      238      257      283      306      317      347
SQ     500     1000     1500     2000     2500     3000     3500     4000     4500     5000
SQ    5500     6000     6500     7000     7500     8000     9000    10000

KK    SUB9

KM        RUNOFF COMPUTATION
BA     0.8
LS       0       77
US    1.10     0.60
KP       2
LS       0       81       20
US    0.98     0.60
KP       3
LS       0       85       40
US    0.87     0.60
KP       4
LS       0       90       60
US    0.85     0.60
KP       5
LS       0       94       80
US    0.84     0.60
KK   SUB10
KM        RUNOFF COMPUTATION
BA     0.8
LS       0       75
US    1.82     0.65
KP       2
LS       0       79       20
US    1.60     0.65
KP       3
LS       0       84       40
US    1.42     0.65
KP       4
LS       0       89       60
US    1.37     0.65
KP       5
US    1.35     0.65
LS       0       93       80

KK     300
KO       0        2        0        0
KM        COMBINE
HC       4

KK    RCH5
KM        ROUTING
RS       1     FLOW       -1
SV       6       11       14       18       23       28       37       46       55       61
SV      71       85      107      124      158
SQ     500     1000     1500     2000     2500     3000     3500     4000     4500     5000
SQ    5500     6000     6500     7000     7500
```

```
KK    SUB6
KM        RUNOFF COMPUTATION
BA      1.0
LS        0       76
US     1.45     0.55
KP        2
LS        0       80       20
US     1.28     0.55
KP        3
LS        0       84       40
US     1.13     0.55
KP        4
LS        0       89       60
US     1.10     0.55
KP        5
LS        0       93       80
US     1.09     0.55
KK     RCH6
KM        ROUTING
RM        1      1.4      0.1

KK    SUB11
KM        RUNOFF COMPUTATION
BA      1.2
LS        0       76
US     2.19     0.69
KP        2
LS        0       80       20
US     1.95     0.69
KP        3
LS        0       84       40
US     1.71     0.69
KP        4
LS        0       89       60
US     1.66     0.69
KP        5
LS        0       94       80
US     1.64     0.69

KK    SUB12
KM        RUNOFF COMPUTATION
BA      0.6
LS        0       77
US     1.31     0.58
KP        2
LS        0       81       20
US     1.17     0.58
KP        3
LS        0       85       40
US     1.03     0.58
KP        4
LS        0       90       60
US     1.01     0.58
KP        5
LS        0       94       80
US     1.00     0.58

KK      500
KO        0        2        0        0
KM        COMBINE
HC        4

KK     RCH7
KM        ROUTING
RS        1     FLOW       -1
SV       13       21       28       34       39       49       80      107      132      154
SV      174      192      230      267      305      322      348      372
SQ      500     1000     1500     2000     2500     3000     3500     4000     4500     5 0
SQ     5500     6000     6500     7000     7500     8000     9000    10000
KK    SUB13
KM        RUNOFF COMPUTATION
BA      1.5
LS        0       77
US     2.85     0.66
KP        2
LS        0       81       20
US     2.54     0.66
KP        3
LS        0       85       40
US     2.25     0.66
KP        4
LS        0       90       60
```

```
US    2.19    0.66
KP       5
LS       0      94      80
US    2.17    0.66

KK     600
KO       0       2       0       0
KM        COMBINE
HC       2

EC

KK    RCH1
CN       3   AGRIC   RESID IND/COM
FR               8     400     100      50      20      10       4       2       1
QF                     220     300     560     930    1230    1672    2040    2378
PN       1         EXISTING LAND USE: AGRICULTURE & OPEN GRASSLAND
QD               8     220     300     560     930    1230    1672    2040    2378
DG             101       0       0       0       8      15      25      33      41
EP
PN       2         FUTURE DEVELOPMENT: 25 % URBANIZATION
QD               8     340     590     997    1464    1842    2354    2760    3161
DG             201       0       0       0       0       0       0       0       0
DG             202       0       0       0      20      33     207     326     426
DG             203       0       0       0      45      91     571     898    1171
EP
PN       3         FUTURE DEVELOPMENT: 50 % URBANIZATION
QD               8     530     880    1474    2047    2480    3042    3523    3960
DG             302       0       0       0      47     199     393    1011    1245
DG             303       0       0       0     135     593    1082    2780    3425
EP
PN       4         FUTURE DEVELOPMENT: 75 % URBANIZATION
QD               8     660    1110    1850    2455    2889    3499    3980    4450
DG             402       0      38      90     497     970    1487    1864    3391
DG             403       0     105     248    1367    2670    4090    5128    9326
EP
PN       5         FUTURE DEVELOPMENT: 100 % URBANIZATION
QD               8     749    1250    2083    2685    3139    3747    4246    4586
DG             502       0     163     413    1078    1440    3027    2751    3297
DG             503       0     450     999    2964    3961    5324    7565    9067
KK    RCH2
CN       3   AGRIC   RESID IND/COM
FR               8     400     100      50      20      10       4       2       1
QF                      90     180     500     570     755    1025    1250    1470
PN       1         EXISTING LAND USE: AGRICULTURE & OPEN GRASSLAND
QD               8      90     180     500     570     755    1025    1250    1470
DG             101       0       0       1       6      10      16      22      26
EP
PN       2         FUTURE DEVELOPMENT: 25 % URBANIZATION
QD               8     220     360     606     895    1124    1442    1699    1941
DG             201       0       0       0       0       0       0       0       0
DG             202       0       0       0      44      85     168     232     304
DG             203       0       0       0     121     230     463     640     835
EP
PN       3         FUTURE DEVELOPMENT: 50 % URBANIZATION
QD               8     317     528     881    1222    1485    1831    2120    2391
DG             302       0       0      65     194     324     494     663     786
DG             303       0       0     179     535     892    1358    1822    2162
EP
PN       4         FUTURE DEVELOPMENT: 75 % URBANIZATION
QD               8     500     880    1108    1475    1744    2109    2411    2697
DG             402       0      52     791     491     676     977    1141    1377
DG             403       0     145     611    2176    1859    2687    3139    3786
EP
PN       5         FUTURE DEVELOPMENT: 100 % URBANIZATION
QD               8     575     924    1257    1627    1900    2272    2579    2868
DG             502       0     205     403     759    1052    1437    1715    1987
DG             503       0     550    1108    2088    2892    3953    4715    5464

KK    RCH3
CN       3   AGRIC   RESID IND/COM
FR               8     400     100      50      20      10       4       2       1
QF                     500     750    1140    1890    2510    3370    4110    4820
PN       1         EXISTING LAND USE: AGRICULTURE & OPEN GRASSLAND
QD               8     500     750    1140    1890    2510    3370    4110    4820
DG             101       0       0       0       0      11      31      47      63
EP
PN       2         FUTURE DEVELOPMENT: 25 % URBANIZATION
QD               8     742    1212    2023    2965    3701    4736    5537    6218
DG             201       0       0       0       0       0       D       0       0
DG             202       0       0       0       0     120     208     795    1003
DG             203       0       0       0       0     360     573    2187    2759
EP
```

```
PN    3        FUTURE DEVELOPMENT: 50 % URBANIZATION
QD          8   1074   1790   2975   4050   4920   5966   6752   7411
DG        302      0      0      0      0    409   1707   2436   2882
DG        303      0      0      0      0    999   4659   6699   7826
EP
PN    4        FUTURE DEVELOPMENT: 75 % URBANIZATION
QD          8   1319   2200   3665   4875   5691   6704   7444   8924
DG        402      0      0      0    587   2277   3222   4290   5145
DG        403      0      0      0   1613   6261   8862  11798  14148
EP
PN    5        FUTURE DEVELOPMENT: 100 % URBANIZATION
QD          8   1480   2478   4130   5320   6112   7113   8417   9316
DG        502      0      0      0   1744   3515   4817   6452   7402
DG        503      0      0      0   4795   9669  13246  17744  20356

KK  RCH4
CN    3    AGRIC   RESID IND/COM
FR          8    400    100     50     20     10      4      2      1
QF               150    200    500    933   1240   1664   2011   2226
PN    1        EXISTING LAND USE: AGRICULTURE & OPEN GRASSLAND
QD          8    150    200    500    933   1240   1664   2011   2226
DG        101      0      0      0     12     20     30     39     47
EP
PN    2        FUTURE DEVELOPMENT: 25 % URBANIZATION
QD          8    200    300   1034   1510   1870   2220   2580   3200
DG        201      0      0      0      0      0      0      0      0
DG        202      0      0            81    160    369    542    625
DG        203      0      0           224    440   1014   1490   1720
EP
PN    3        FUTURE DEVELOPMENT: 50 % URBANIZATION
QD          8    350    620   1558   2051   2320   3080   3640   4050
DG        302      0      0    169    532    840   1197   1417   1666
DG        303      0      0    467   1464   2311   3291   3896   4583
EP
PN    4        FUTURE DEVELOPMENT: 75 % URBANIZATION
QD          8    700   1167   1946   2324   2908   3660   4103   4461
DG        402      0    164    446   1704   1613   2098   2502   2867
DG        403      0    450   1228   3407   4437   4999   6880   7885
EP
PN    5        FUTURE DEVELOPMENT: 100 % URBANIZATION
QD          8    756   1260   2099   2534   3302   3942   4325   5024
DG        502      0    443   1017   1989   2434   3073   3632   4179
DG        503      0   1220   2796   5471   6693   8450   9989  11492

KK  RCH5
CN    3    AGRIC   RESID IND/COM
FR          8    400    100     50     20     10      4      2      1
QF               750   1150   1870   3100   4105   5495   6552   7403
PN    1        EXISTING LAND USE: AGRICULTURE & OPEN GRASSLAND
QD          8    750   1150   1870   3100   4105   5495   6552   7403
DG        101      0      0      0     16     39     71     98    124
EP
PN    2        FUTURE DEVELOPMENT: 25 % URBANIZATION
QD          8   1200   2000   3339   4908   6002   7285   8804  10247
DG        201      0      0      0      0      0      0      0      0
DG        202      0      0      0     24     87   1428   2386   2776
DG        203      0      0      0     66    240   3928   6562   7633
EP
PN    3        FUTURE DEVELOPMENT: 50 % URBANIZATION
QD          8   1770   2949   4915   6451   7463   9794  11247  12410
DG        302      0      0     40   1204   2593   4684   4772   6345
DG        303      0      0    109   3664   7150  12880  13125  17448
EP
PN    4        FUTURE DEVELOPMENT: 75 % URBANIZATION
QD          8   2160   3600   5999   7416   9296  11210  12500  14300
DG        402      0     54    654   4016   5437   7026   8319   8327
DG        403      0    150   1799  11045  14950  19320  22878  22900
EP

PN    5        FUTURE DEVELOPMENT: 100 % URBANIZATION
QD          8   2860   3940   6568   8565  10176  11960  13630  14590
DG        502      0    461   1191   4330   6075   7675   8979  10184
DG        503      0   1270   3275  11909  16708  21107  24694  28005

KK  RCH6
CN    3    AGRIC   RESID IND/COM
FR          8    400    100     50     20     10      4      2      1
QF                50     80    120    200    250    350    455    495
PN    1        EXISTING LAND USE: AGRICULTURE & OPEN GRASSLAND
QD          8     50     80    120    200    250    350    455    495
DG        101      0      0      0      0      2      4      5      7
EP
```

```
PN     2        FUTURE DEVELOPMENT: 25 % URBANIZATION
QD           8      70     120     195     288     361     463     548     640
DG         201       0       0       0       0       0       0       0       0
DG         202       0       0       0       0       5      14      35      49
DG         203       0       0       0       0      14      38      96     136
EP
PN     3        FUTURE DEVELOPMENT: 50 % URBANIZATION
QD           8     108     180     285     395     478     594     686     773
DG         302       0       0       0      14      44      78     105     137
DG         303       0       0       0      36     121     214     290     378
EP
PN     4        FUTURE DEVELOPMENT: 75 % URBANIZATION
QD           8     108     210     354     470     559     677     773     865
DG         402       0       0      17      64     107     155     206     257
DG         403       0       0      47     178     294     426     567     706
EP
PN     5        FUTURE DEVELOPMENT: 100 % URBANIZATION
QD           8     140     240     396     515     604     724     821     910
DG         502       0      16      48     116     186     240     314     375
DG         503       0      45     131     324     511     659     864    1032

KK  BCH7
CN     3  AGRIC   RESID IND/COM
FR           8     400     100      50      20      10       4       2       1
QF                 900    1450    2192    3476    4576    6140    7200    8435
PN     1        EXISTING LAND USE: AGRICULTURE & OPEN GRASSLAND
QD           8     900    1450    2192    3476    4576    6140    7200    8435
DG         101       0       0       0       0      10      19      50      79
EP
PN     2        FUTURE DEVELOPMENT: 25 % URBANIZATION
QD           8     999    2340    3690    5417    6565    8192    9939   11643
DG         201       0       0       0       0       0       0       0       0
DG         202       0       0       0       0     261     683    1055    1577
DG         203       0       0       0       0     717    1880    2902    4338
EP
PN     3        FUTURE DEVELOPMENT: 50 % URBANIZATION
QD           8    1930    3200    5320    6988    8355   10958   12742   14183
DG         302       0       0       0     668    1360    2354    3422    4205
DG         303       0       0       0    1836    3740    6474    9412   11565
EP
PN     4        FUTURE DEVELOPMENT: 75 % URBANIZATION
QD           8    2332    3885    6472    8267    9999   12686   14286   16193
DG         402       0       0     530    1942    3014    5266    6636    7999
DG         403       0       0    1458    5341    8290   14480   18249   21997
EP
PN     5        FUTURE DEVELOPMENT: 100 % URBANIZATION
QD           8    3450    6250    7073    9281   11390   13579   15376   17362
DG         502       0     454    1402    3326    5418    7576    9916   11634
DG         503       0    1250    3856    9146   14900   20834   27269   31995
ZZ
```

APPENDIX B

Program Input of a Multiflood-Multiplan Analysis for Various Extent of Channel Modification

HEC-1 INPUT

```
LINE   ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

       ID        EFFECTS OF CHANNEL MODIFICATIONS WITH 50 % FUTURE URBANIZATION
       ID        INVESTIGATION BY SUE LIN LEWIS
       ID        FLOOD DAMAGE ANALYSIS  (EXPECTED ANNUAL DAMAGE)
       ID        MULTIFLOOD, MULTIPLAN ANALYSIS
       ID        STREAM NETWORK COMPUTATION
       ID        50  YR,  6 HR DURATION
       ID        COW CREEK BASIN, OKLAHOMA CITY, OK
       *DIAGRAM
       IT      5                      100
       IO      4
       JP      5
       JR   PREC    0.56    0.72    0.84    1.00    1.13    1.25

       KK   SUB1
       KM        RUNOFF COMPUTATION
       BA    3.5
       BF      0    -0.1     1.2
       PH      4       0    0.63    1.44    2.88    3.29    3.75    4.80
       LS      0      85      40
       US   1.94    0.66

       KK   SUB2
       KM        RUNOFF COMPUTATION
       BA    1.5
       LS      0      83      40
       US   1.15    0.58

       KK    100
       KM         COMBINE
       HC      2

       KK   RCH1
       KM        ROUTING
       RS      1    FLOW      -1
       SV     20      33      47      60      74      91     107     124     140     200
       SV    231     276     326     340     350     365     397     429
       SQ    500    1000    1500    2000    2500    3000    3500    4000    4500    5000
       SQ   5500    6000    6500    7000    7500    8000    9000   10000

       KK   SUB3
       KM        RUNOFF COMPUTATION
       BA    3.0
       LS      0      84      40
       US   1.36    0.63

       KK   RCH2
       KM        ROUTING
       RS      1    FLOW      -1
       SV     36      69     102     137     171     203     235     266     296     338
       SV    369     419     472     505     514     540     591     631
       SQ    500    1000    1500    2000    2500    3000    3500    4000    4500    5000
       SQ   5500    6000    6500    7000    7500    8000    9000   10000
       KK   SUB7
       KM        RUNOFF COMPUTATION
       BA    0.9
       LS      0      86      40
       US   1.57    0.63

       KK   SUB8
       KM        RUNOFF COMPUTATION
       BA    1.0
       LS      0      85      40
       US   1.86    0.68

       KK    200
       KM        COMBINE
       HC      4

       KK   RCH3
       KM        ROUTING
       RS      1    FLOW      -1
       SV      6      11      14      18      22      28      37      46      55      63
       SV     74      88     111     131     166     170     175     188
       SQ    500    1000    1500    2000    2500    3000    3500    4000    4500    5000
       SQ   5500    6000    6500    7000    7500    8000    9000   10000
```

```
KK    SUB4
KM        RUNOFF COMPUTATION
BA     3.2
LS       0       84      40
US    1.88     0.64

KK    SUB5
KM        RUNOFF COMPUTATION
BA     2.2
LS       0       84      40
US    1.58     0.59

KK     400
KM        COMBINE
HC       2

KK    RCH4
KM        ROUTING
RS       1     FLOW      -1
SV      12       20      32      51     110     120     131     152     189     195
SV     208      215     238     257     283     306     317     347
SE  1186.5   1188.7  1190.5  1192.5  1195.3  1195.6  1195.9  1196.5  1196.7  1196.9
SE  1197.1   1197.4  1197.6  1197.7  1197.9  1198.0  1198.2  1198.4
SQ     500     1000    1500    2000    2500    3000    3500    4000    4500    5000
SQ    5500     6000    6500    7000    7500    8000    9000   10000
KP       2
RS       1     FLOW      -1
SV      12       20      31      49     105     114     122     140     172     177
SV     187      193     214     230     243     260     272     288
SE  1184.8   1186.4  1188.4  1190.8  1192.9  1195.4  1195.6  1195.8  1195.9  1196.0
SE  1196.4   1196.9  1197.2  1197.4  1197.6  1197.8  1198.0  1198.2
SQ     500     1000    1500    2000    2500    3000    3500    4000    4500    5000
SQ    5500     6000    6500    7000    7500    8000    9000   10000
KP       3
RS       1     FLOW      -1
SV      12       19      28      45     100     107     112     125     152     160
SV     165      173     189     200     210     222     232     239
SE  1184.0   1186.0  1187.8  1189.6  1190.1  1190.5  1191.3  1192.1  1192.9  1194.4
SE  1195.8   1196.0  1196.1  1196.6  1197.0  1197.3  1197.7  1198.1
SQ     500     1000    1500    2000    2500    3000    3500    4000    4500    5000
SQ    5500     6000    6500    7000    7500    8000    9000   10000
KP       4
RS       1     FLOW      -1
SV      12       19      30      45      99     105     111     124     146     150
SV     156      165     170     174     178     180     182     190
SE  1183.6   1184.1  1185.0  1186.0  1186.8  1187.2  1188.3  1189.0  1189.6  1190.2
SE  1190.8   1191.3  1191.8  1192.3  1195.8  1196.0  1196.2  1196.3
SQ     500     1000    1500    2000    2500    3000    3500    4000    4500    5000
SQ    5500     6000    6500    7000    7500    8000    9000   10000
KP       5
RS       1     FLOW      -1
SV      12       19      29      40      93      98     104     116     138     140
SV     150      159     164     170     174     176     178     182
SE  1182.8   1183.5  1184.1  1184.6  1185.1  1185.8  1186.4  1187.0  1187.5  1188.1
SE  1188.6   1189.1  1189.5  1190.0  1190.4  1190.8  1191.6  1192.3
SQ     500     1000    1500    2000    2500    3000    3500    4000    4500    5000
SQ    5500     6000    6500    7000    7500    8000    9000   10000

KK    SUB9
KM        RUNOFF COMPUTATION
BA     0.8
LS       0       85      40
US    0.87     0.60

KK    SUB10
KM        RUNOFF COMPUTATION
BA     0.8
LS       0       84      40
US    1.42     0.65
KP       2
LS       0       84      40
US    1.05     0.65
KP       3
LS       0       84      40
US    1.05     0.65
KP       4
LS       0       84      40
US    1.05     0.65
KP       5
LS       0       84      40
US    1.05     0.65
```

```
KK    300
KM       COMBINE
HC     4

KK    RCH5
KM       ROUTING
RS     1      FLOW      -1
SV     6       11       14       18       23       28       37       46       55       61
SV    71       85      107      124      158
SE  1185.2   1187.4   1189.1   1190.6   1191.8   1192.6   1193.2   1193.7   1194.1   1194.5
SE  1195.2   1195.9   1196.2   1196.5   1196.7
SQ   500     1000     1500     2000     2500     3000     3500     4000     4500     5000
SQ  5500     6000     6500     7000     7500
KP     2
RS     1      FLOW      -1
SV     6       11       14       17       21       26       34       42       51       55
SV    64       78       99      114      144
SE  1184.9   1187.0   1188.5   1189.8   1190.9   1191.9   1192.7   1193.5   1194.3   1194.9
SE  1195.4   1195.8   1196.1   1196.2   1196.3
SQ   500     1000     1500     2000     2500     3000     3500     4000     4500     5000
SQ  5500     6000     6500     7000     7500
KP     3
RS     1      FLOW      -1
SV     6       11       14       16       20       25       33       41       48       52
SV    60       80       87       98      121
SE  1184.2   1186.0   1187.4   1188.5   1189.5   1190.4   1191.2   1191.9   1192.6   1193.2
SE  1193.8   1194.4   1194.9   1195.3   1195.6
SQ   500     1000     1500     2000     2500     3000     3500     4000     4500     5000
SQ  5500     6000     6500     7000     7500
KP     4
RS     1      FLOW      -1
SV     6       10       13       15       18       23       29       35       40       43
SV    52       58       65       78       90
SE  1182.8   1184.0   1185.1   1186.0   1186.8   1187.5   1188.2   1188.9   1189.5   1190.1
SE  1190.6   1191.1   1191.6   1192.1   1192.5
SQ   500     1000     1500     2000     2500     3000     3500     4000     4500     5000
SQ  5500     6000     6500     7000     7500
KP     5
RS     1      FLOW      -1
SV     6       10       13       15       18       23       27       33       37       39
SV    47       52       57       68       78
SE  1182.1   1183.0   1183.8   1184.5   1185.1   1185.8   1186.3   1186.9   1187.4   1187.9
SE  1188.4   1188.9   1189.3   1189.7   1190.2
SQ   500     1000     1500     2000     2500     3000     3500     4000     4500     5000
SQ  5500     6000     6500     7000     7500

KK    SUB6
KM       RUNOFF COMPUTATION
BA    1.0
LS     0       84       40
US   1.13     0.55
KK    RCH6
KM       ROUTING
RM     1      1.4      0.1

KK    SUB11
KM       RUNOFF COMPUTATION
BA    1.2
LS     0       84       40
US   1.71     0.69
KP     2
LS     0       84       40
US   1.53     0.69
KP     3
LS     0       84       40
US   1.53     0.69
KP     4
LS     0       84       40
US   1.53     0.69
KP     5
LS     0       84       40
US   1.53     0.69

KK    SUB12
KM       RUNOFF COMPUTATION
BA    0.6
LS     0       85       40
US   1.03     0.58
```

```
216          KP       2
217          LS       0      85      40
218          US    0.52    0.58
219          KP       3
220          LS       0      85      40
221          US    0.52    0.58
222          KP       4
223          LS       0      85      40
224          US    0.52    0.58
225          KP       5
226          LS       0      85      40
227          US    0.52    0.58

228          KK     500
229          KM        COMBINE
230          HC       4

231          KK    RCH7
232          KM        ROUTING
233          RS       1    FLOW      -1
234          SV      13      21      28      34      39      49      80     107     132     154
235          SV     174     192     230     267     305     322     348     372
236          SE  1174.0  1176.5  1178.3  1179.8  1181.0  1182.4  1183.3  1183.8  1184.2  1184.6
237          SE  1184.9  1185.1  1185.4  1185.6  1185.9  1186.1  1186.3  1186.4
238          SQ     500    1000    1500    2000    2500    3000    3500    4000    4500     5 0
239          SQ    5500    6000    6500    7000    7500    8000    9000   10000
240          KP       2
241          RS       1    FLOW      -1
242          SV      12      19      25      29      34      38      42      49      59      63
243          SV      69      79      94     114     151     192     257     298
244          SE  1171.8  1173.5  1174.7  1175.8  1176.3  1177.5  1178.2  1178.9  1179.5  1180.1
245          SE  1180.6  1181.1  1181.7  1182.1  1182.6  1183.0  1183.7  1184.4
246          SQ     500    1000    1500    2000    2500    3000    3500    4000    4500    5000
247          SQ    5500    6000    6500    7000    7500    8000    9000   10000
248          KP       3
249          RS       1    FLOW      -1
250          SV      12      19      25      30      34      39      43      47      51      58
251          SV      62      66      70      76      85     116     152     224
252          SE  1171.1  1172.9  1174.1  1175.0  1175.9  1176.7  1177.4  1178.0  1178.6  1179.2
253          SE  1179.7  1180.3  1180.7  1181.2  1181.7  1182.1  1182.9  1183.6
254          SQ     500    1000    1500    2000    2500    3000    3500    4000    4500    5000
255          SQ    5500    6000    6500    7000    7500    8000    9000   10000
256          KP       4
257          RS       1    FLOW      -1
258          SV      13      19      27      32      37      42      46      50      54      61
259          SV      65      69      72      76      79      87     100     160
260          SE  1170.6  1172.1  1173.1  1174.0  1174.8  1175.5  1176.2  1176.8  1177.4  1177.9
261          SE  1178.4  1178.9  1179.4  1179.8  1180.2  1180.6  1181.4  1182.2
262          SQ     500    1000    1500    2000    2500    3000    3500    4000    4500    5000
263          SQ    5500    6000    6500    7000    7500    8000    9000   10000
264          KP       5
265          RS       1    FLOW      -1
266          SV      15      23      29      35      40      45      49      54      58      66
267          SV      70      74      77      81      85      92      99     121
268          SE  1170.3  1171.6  1172.6  1173.4  1174.2  1174.9  1175.5  1176.1  1176.6  1177.1
269          SE  1177.6  1178.0  1178.5  1178.9  1179.3  1179.7  1180.4  1181.1
270          SQ     500    1000    1500    2000    2500    3000    3500    4000    4500    5000
271          SQ    5500    6000    6500    7000    7500    8000    9000   10000

272          KK   SUB13
273          KM        RUNOFF COMPUTATION
274          BA     1.5
275          LS       0      85      40
276          US    2.25    0.66
277          KP       2
278          LS       0      85      40
279          US    1.13    0.66
280          KP       3
281          LS       0      85      40
282          US    1.13    0.66
283          KP       4
284          LS       0      85      40
285          US    1.13    0.66
286          KP       5
287          LS       0      85      40
288          US    1.13    0.66
289          KK     600
```

```
KM      COMBINE
HC    2
EN

KK   RCH4
CN    3    AGRIC    RESID IND/COM
FR              8     400     100      50      20      10       4       2       1
SF                 1187.4  1189.0  1190.7  1192.8  1194.3  1195.7  1196.1  1196.5
SD              8  1197.4  1189.0  1190.7  1192.8  1194.3  1195.7  1196.1  1196.5
DG            101       0       0       0       0       0       0       0       0
DG            102       0       0     169     532     840    1197    1417    1663
DG            103       0       0     467    1464    2311    3291    3896    4582

KK   RCH5
CN    3    AGRIC    RESID IND/COM
FR              8     400     100      50      20      10       4       2       1
SF                 1189.9  1192.2  1194.4  1196.2  1196.7  1197.3  1197.8  1198.3
SD              8  1189.9  1192.2  1194.4  1196.2  1196.7  1197.3  1197.8  1198.3
DG            101       0       0       0       0       0       0       0       0
DG            102       0       0      40    1204    2593    4684    4772    6345
DG            103       0       0     109    3664    7130   12880   13125   17448

KK   RCH7
CN    3    AGRIC    RESID IND/COM
FR              8     400     100      50      20      10       4       2       1
SF                 1179.6  1182.8  1184.8  1185.6  1186.2  1186.4  1186.6  1186.7
SD              8  1179.6  1182.8  1184.8  1185.6  1186.2  1186.4  1186.6  1186.7
DG            101       0       0       0       0       0       0       0       0
DG            102                               668    1360    2351    3422    4205
DG            103                              1836    3740    6470    9412   11565
ZZ
```