A METHODOLOGY TO ASSESS THE IMPACT OF A CHANGING FLOOD PLAIN DETERMINATION ON AN UNGAGED URBAN BASIN

By

ROBERT LOUIS TORTORELLI

Bachelor of Science
University of Illinois
at Urbana-Champaign
Urbana, Illinois
1967

Master of Science
University of Illinois
at Urbana-Champaign
Urbana, Illionis
1969

Submitted to the Faculty of the Graduate College
of the Oklahoma State University
in partial fulfillment of the requirements
for the Degree of
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Thesis Approved:

Richard N. DeVries
Thesis Adviser

Don F Kincannon

Marcia A. Bates

Arthur K. Tyagi

Douglas C. Kent

Norman N Durham
Dean of the Graduate College

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LIST OF SYMBOLS

A	contributing drainage area of a watershed, mi^2
C	channel routing coefficient in convex routing method
C_d	direct damage for a particular flood event, dollars
CN	curve number, or hydrologic soil-cover complex number
d	depth of flooding, ft
D	duration of rainfall storm, hr
D_t	total damage to structure and contents, dollars
F	infiltration occurring after runoff begins, in.
F_c	fraction of contents damaged
F_s	fraction of structure damaged
I	inflow rate, ft^3/s
I_a	initial abstraction, in.
K	constant for triangular hydrograph, 484
K_d	marginal flood damage per unit of depth, ft^{-1}
M_s	market value of inundated structure, dollars
n	Manning's coefficient of roughness
O	outflow rate, ft^3/s
P	total storm rainfall, in.
P_a	mean annual precipitation, in.
P_e	potential runoff, or effective storm runoff, storm rainfall minus initial abstraction, in.
P_r	probability, in this study the probability of a specific floor event occurring in any one year

q_p peak rate of discharge, ft^3/s

Q direct runoff, in.

Q_T peak rate of discharge, ft^3/s

R_L urban adjustment factor, ratio of the mean annual flood under urban conditions to rural conditions

S storage volume, ft^3

S_a potential abstraction, in.

S_o main channel bottom slope, determined from elevation at points 10 and 85 percent of distance along the channel from the gaging station to drainage divide, ft/mi

t time elapsed, hr

t_b base time, total duration time of runoff hydrograph, hr

t_c time of concentration, the time it takes water to flow from the hydraulically most remote point on a watershed to the watershed outlet, hr

t_p time to peak of a runoff hydrograph, hr

t_L lag time, the time from the center of mass of effective rainfall to the center of mass of the runoff hydrograph, hr

T recurrence interval or return period, average time interval between occurrence of a hydrological event of a given or greater magnitude, yr

V velocity, ft/s

$\forall$ volume of water, in both effective rainfall and runoff hydrograph, in.

V_c market value of structure, dollars

V_s market value of contents, dollars

CHAPTER I

INTRODUCTION

Flood plains have been and continue to be under pressure for development to more intensive uses, and today they comprise a disproportionate amount of urbanized land in many sections of the nation. Pressure to intensify flood plain utilization is increasing as accessible undeveloped lands near urban regions are becoming more scarce (4).

In recent years, the federal government has spent many billions of dollars to indemnify flood victims for property losses. Since 1936, more than $7 billion have been spent to construct flood protection works, (4, 11). Yet annual flood losses exceed $1 billion and are continuing to increase, mainly as a consequence of the improper use of the nation's flood plains (4, 11, 62, 90).

The Stillwater, Oklahoma, metropolitan area is no exception. Stillwater, Boomer, Cow, and Duck Creeks flood frequently, causing thousands of dollars in property damage. Duck Creek has been flood-prone for years; at least once a year it overtops its banks and threatens to flood the residences that adjoin the creek (70). The October, 1959 flood is the maximum flood of record, causing about $79,000 in flood damages in Duck Creek (95). In the most recent flood (May, 1975) over four inches of rain fell in less than two hours. McFarland Street was transformed into "McFarland River" with water flowing over three feet deep in the street (27).

Recognizing that the nation can no longer tolerate the losses of lives and property that result from the improper and unrestrained use of our flood plains, the Congress enacted the Flood Disaster Protection Act of 1973 (43). Every flood-prone community in the nation is required to manage new development in areas subject to flooding in order to minimize flood damage. In addition, property owners in flood-prone areas must purchase flood insurance as a prerequisite for any form of federal or federally-related financial assistance for acquisition or construction of buildings in designated special flood hazard areas (11, 43).

A vital step in meeting the goal of a nationwide program of proper flood plain management measures is an evaluation of a community's existing flood damage potential. This evaluation, the Flood Insurance Study, is also an important prerequisite in the community's continued participation in the National Flood Insurance Program. Basically, detailed engineering (field) studies and backwater analyses are made that result in the determination of the 10-[1], 50-, 100-, and 500-year flood profiles and that provide data necessary for floodway determination. From this information, flood insurance rate maps are made that divide the study area into zones that are used to establish actuarial insurance rates (44).

The major flaw in the Flood Insurance Study is that only existing conditions are studied. Only a minor concession is made for future conditions: "Flood hazard determinations should be based on conditions that

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[1] The 10-year flood has the probability of occurring once in ten years. The probability of a specific flood occurring in any one year is $P_r = 1/T$; where P is the probability and T is the return period or frequency. Thus, the probability of a 10-year flood occurring in any one year is: $P_r = 1/10$ or 10 percent.

will exist in the community 12 months following completion of the draft report" (44, pp. 2-4).

Urban development of the watershed basically affects drainage characteristics in two ways: (1) reduction in infiltration losses because of covering the permeable soils with streets, parking lots, roofs, etc.; and (2) provision of more hydraulically efficient drainage systems (storm sewers, improved channels, etc.). These changes generally result in an overall increase in storm runoff volume because of reduced infiltration losses and higher peak runoff rates because of shorter concentration time in the more efficient dranage systems (55). Therefore, hydrologic analyses should include not only estimates of flows under existing conditions, but also estimates of how flows for various frequencies would be affected by watershed changes (67).

For example, take a hypothetical homeowner who takes the precaution of checking the existing flood hazard maps and whose home is clearly out of any flood-prone area. Then in the future, say ten years later, a new set of flood hazard maps is produced and his home <u>is</u> in a flood-prone area due to watershed development. What is the impact of the change in the designated flood-prone area?

Study Objective

The objective of this study is to develop a methodology to assess the impact of a changing flood plain determination on an ungaged urban basin. Duck Creek, Stillwater, Oklahoma, is used as the test basin in this investigation.

Duck Creek is a small tributary of Stillwater Creek with its drainage basin located in the northwest portion of Stillwater, Oklahoma

(Figure 1). In this study, four basin development conditions are investigated:

1. Present basin development (October, 1978) with present urbanization and present channel.

2. Present urbanization with a planned channel improvement project (100) simulated between the mouth of Duck Creek and 6th Avenue.

3. Future urbanization simulated on the basin with no channel improvement.

4. Future urbanization and a planned channel improvement simulated on the basin.

Hydrographs, peak discharges, flood profiles, and flood hazard maps which correspond to each basin development alternative are determined. The results for the present basin development should correlate approximately with the present flood hazard maps (26). However, there has been construction of more efficient bridge structures on Sherwood Avenue, Arrowhead Drive, and 12th Avenue which make present conditions different from the previous conditions under which the existing flood hazard maps were developed.

Finally, the 100-year flood direct damages are determined for each basin development alternative to provide a relative comparison of the impact if new areas in the Duck Creek watershed are included in the designated flood-prone area when the new flood profiles and flood hazard maps are developed.

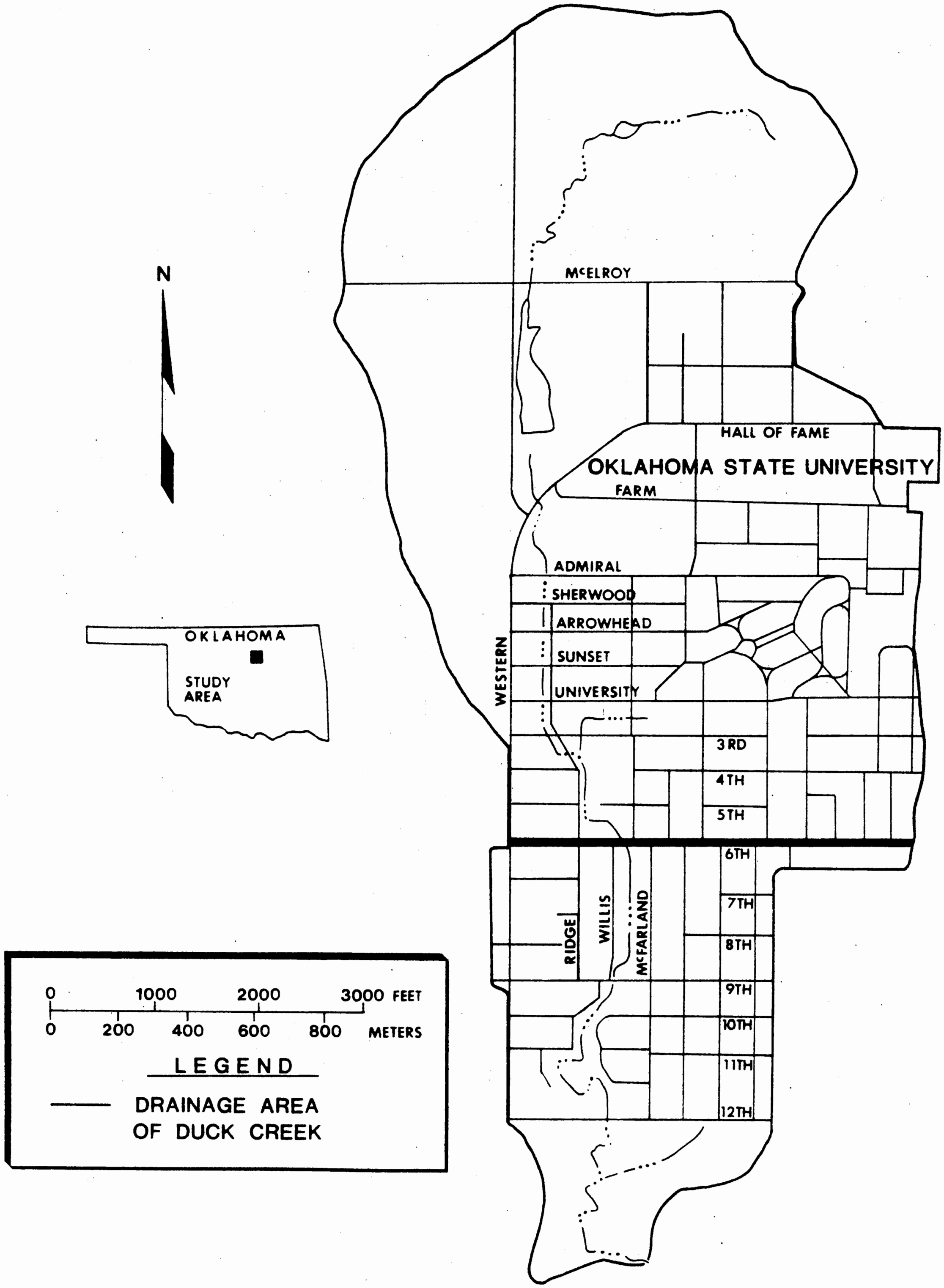


Figure 1. Duck Creek Basin

CHAPTER II

LITERATURE REVIEW

National Flood Insurance Program

Introduction

Today flood insurance for a home and its contents is available in many more areas of the nation than ever before--and it is affordable. The National Flood Insurance Program (NFIP) is responsible for making it available (37).

Over the past 40 years the United States government has been unable to stop the annual increase in flood losses by structural flood-control measures (75). A feasibility study requested by Congress found that in addition to increasing pressure for development in flood-prone property, many people were seriously uninformed about flood risks, were overoptimistic about the chances that their property would not be flooded, or expected the government to assist them after a flood disaster (11).

Congress accepted the study's recommendation for sound land use and control measures when it enacted the National Flood Insurance Act of 1968 (82). However, the low enrollment in the program made it clear that the voluntary nature of the program was its major defect and that without mandatory requirements to promote sound flood plain management, no real progress could be made toward decreasing flood losses (11, 62). Therefore, Congress passed the Flood Disaster Protection Act of 1973 (43).

This Act and its amendments expanded the 1968 Flood Insurance Program by:

1. Requiring insurance on all federal or federally assisted financing of development in flood-prone areas.

2. Creating incentives for flood-prone communities to participate in the program and thus make insurance available to their citizens.

3. Accelerating the completion of Flood Insurance Studies for flood-prone communities.

4. Establishing detailed procedures for technical appeals of floor elevation determinations (11, p. 11).

Now residents may make their location decisions with full knowledge of the flood risk through premiums paid for flood insurance. The NFIP has subsidized rates (11, 17, 105), but it should result in more information and better location decisions.

Insurance companies have published easy-to-understand articles explaining flood insurance (37) and the federal government has prepared a pamphlet which explains the NFIP in clear, layman terms (85). In addition, flood plain management guidelines for federal agencies (4, 46) have been adapted so that federal agencies may "lead the Nation by exemplary demonstration of a comprehensive approach to floodplain management" (46, p. 1).

The cornerstone of the NFIP is the Flood Insurance Study (FIS). Evaluation of special flood hazard areas is accomplished in this study for a flood-prone community and portrayed in Flood Insurance Rate Maps. These are detailed maps which show the elevations and boundaries of the 100-year (Zones A and V) and 500-year flood plains (46).

The Floodway

One of the important components of the FIS is the inclusion of a designated "floodway" for a watercourse. For flood plain management purposes, no construction of buildings or any development that would obstruct the flood flow of the watercourse is allowed within the boundaries of the designated floodway.

The concept of a floodway is more easily grasped with the help of a diagram (Figure 2). Guidelines and Specifications for Study Contractors defines the floodway as follows:

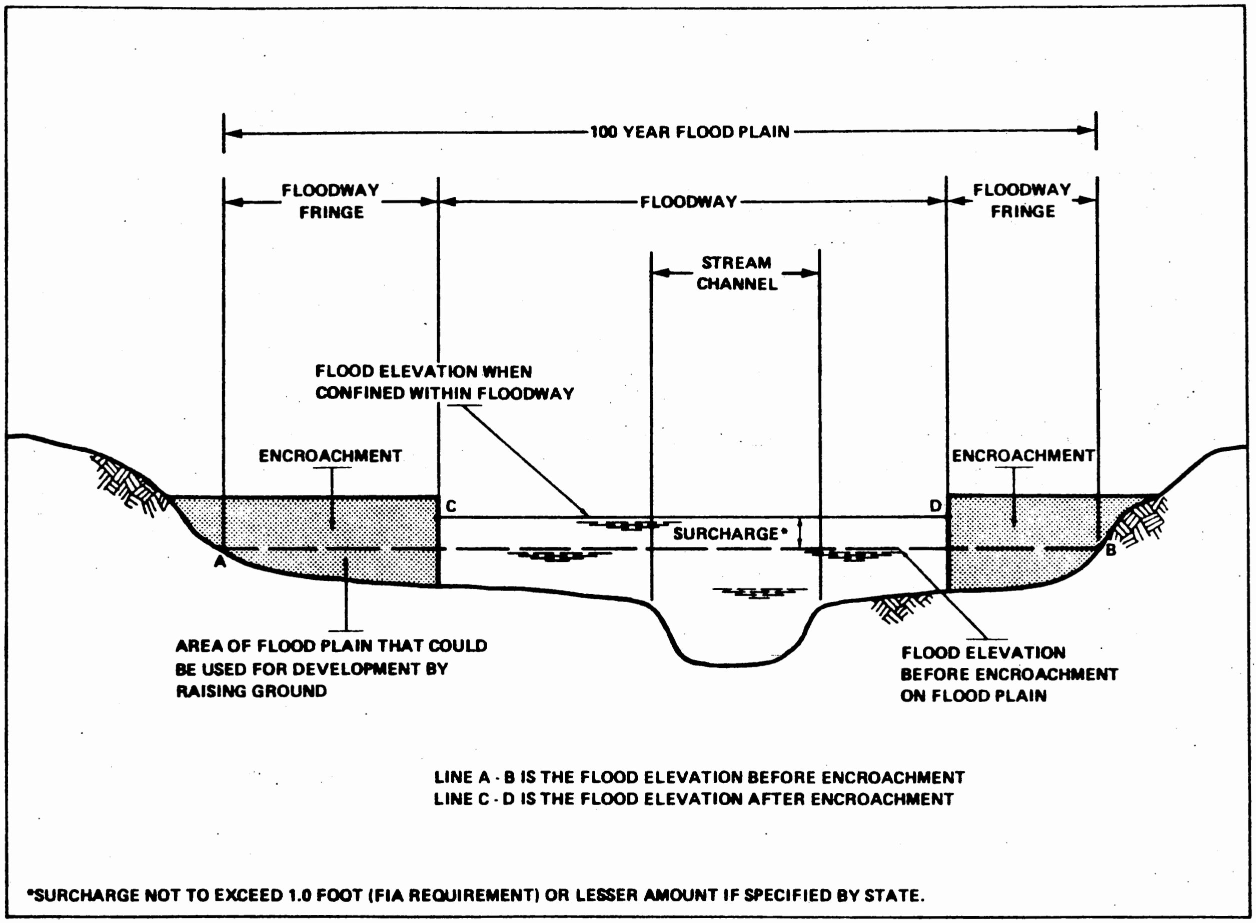
1. A floodway is the channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a designated height.

2. Normally the floodway will include the stream channel and that portion of the adjacent land areas required to pass the 100-year frequency flood discharge without cumulatively increasing the water surface elevation at any point more than one (1) foot above that of the pre-floodway condition (44, pp. 2-13).

Conflicting Flood Plain Determinations

Since the passage of the NFIP, the use of detailed studies for regulatory purposes where a small variation in flood elevations affects a large amount of property has led to numerous conflicting studies being prepared on behalf of various interests. Disputes have arisen as to which of several conflicting flood plain determinations should be utilized as the basis for local regulations and flood insurance rates (5).

The present state of the practice of hydrology is as much an art as a science; therefore, the detailed studies made by any of the several private consultants or government agencies engaged in this type of work are subject to wide variation. The factors causing this variation can be classified by the four basic portions of the study analyses (5): (1) geometric data, (2) hydrologic analysis, (3) hydraulic analysis, and (4) mapping.



Source: U.S. Federal Insurance Administration (44), pp. 4-19

Figure 2. Floodway Schematic

The accuracy of the geometric or cross-sectional data can have a significant effect on flood plain determination. Stream channel cross sections are usually obtained by using a large scale topographic map, field surveys, or aerial photography. Besides the error inherent in each method, the cross sections may not fully represent channel geometry due to improvements constructed since the cross sections were determined. Due to the alternate methods of determining cross-section geometry, a difference in cross-section area and flow capacity can easily occur (5).

Another source of variation may be the method of hydrologic analysis and the application of this method. The methods generally used in hydrologic analyses are: (1) hydrograph analyses, (2) statistical analyses, and (3) regional discharge studies. Usually the largest variations in the hydrologic analyses result from differences in assumptions based on engineering judgment, and the amount of detail used in the application of a particular method (5).

Sources of variation in hydraulic analyses are: (1) the method of computation, (2) the alignment and spacing of cross sections, and (3) the roughness coefficients, or Manning's "n" values. The step-backwater method is the accepted method to be used for detailed studies (44), and involves a detailed solution of the Bernoulli equation for steady, gradually varied flow. However, the computer program used for the calculations can affect the results (80). In the step-backwater method, the spacing, location, and alignment of cross sections are important factors in the computation of the water surface profiles--the selection of which is based on engineering judgment. The roughness coefficients are usually determined by an initial estimate based on references such as Chow (20) and engineering judgment. Calibration of "n" values may be possible if recorded elevations and discharges are available (5).

Large differences in the areal coverage of a particular flood event may be the result of either the contour interval or the relative accuracy of maps used for flood plain determinations. Topographic maps have an expected accuracy of one-half contour interval. Also variation between the datum of mapping and the datum of the geometric data may result in differing flood plains (5).

However, as can be seen from an overview of the sources of variation in conflicting flood plain determinations, the skill and judgment of the analyst are the most important components of a detailed study (5, 34, 48, 66).

Effect of Urbanization

Introduction

The United States has become a metropolitan nation, with only about one-twentieth of the land occupied by over two-thirds of its population. If projections based on historical growth and trends are valid, the amount of urbanized land will double in the next 30 years (89). The development of an urban area within a watershed is a significant change of land use and it has major effects on the hydrologic response of the watershed during flood conditions (61).

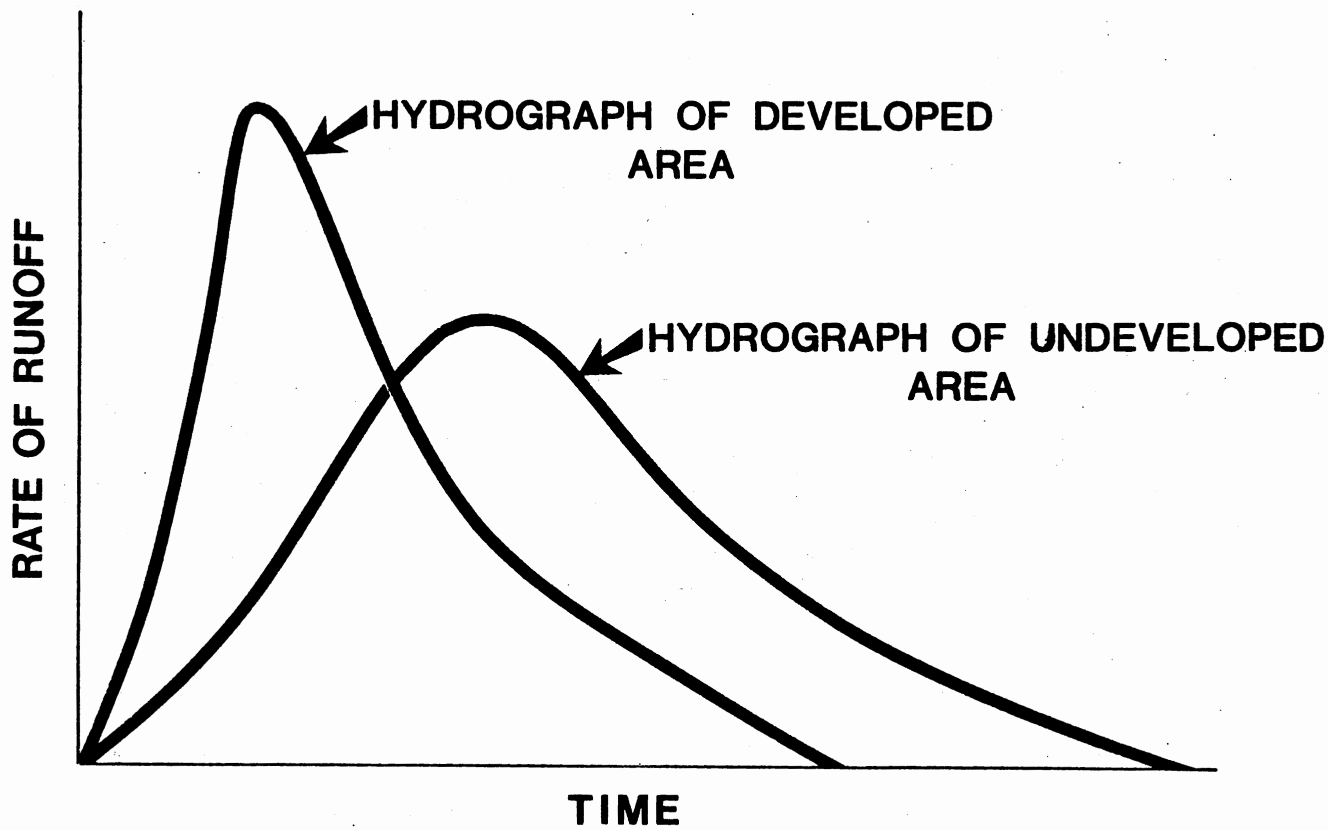
The urbanization process affects the drainage characteristics of a watershed in two basic ways: (1) rendering a large portion of the land impervious by covering the natural ground with roofs, streets, parking lots, driveways, etc., and (2) providing more hydraulically efficient channels for storm runoff. These factors result in an increase in storm runoff volume due to reduced infiltration and storage and higher peak runoff rates because of shorter concentration time (lag time) in the more efficient drainage systems (3, 7, 12, 14, 31, 34, 38, 55, 67, 74, 76, 77, 86, 89, 104, 108). Generally, the most significant effect of urban development is to produce flood hydrographs of increased magnitude that are quicker to rise and recede than those for natural runoff (77) (Figure 3).

It has not been difficult to determine the general effects of urban development, but it has been very difficult to develop relationships which accurately define the extent of these changes. Chow (22) has a comprehensive table of the general hydrologic effects of urbanization. Two task committee reports of the American Society of Civil Engineers (38, 39) have selected bibliographies of literature related to specific urbanization effects. The following is a selected review of the attempts to quantify the extent of urbanization effects.

Impact on Peak Rates

The effects of increased imperviousness and improved drainage systems are numerous. The precipitation cannot infiltrate through an impervious surface as readily so the volume of runoff increases. More hydraulically efficient surfaces and drainage systems cause the runoff to occur faster. In addition, less natural storage in the basin further increases the rate of runoff. This results in generally higher peak flows (34).

Anderson (3) found that on small, steep basins, drainage improvements alone may triple average flood sizes and complete development of stream channels and the basin surface may increase average flood peak magnitudes by a factor of eight. Bras and Perkins (14) observed peak increases from 7% to 200%. Based on analyses by Dempster (36), changing a rural basin

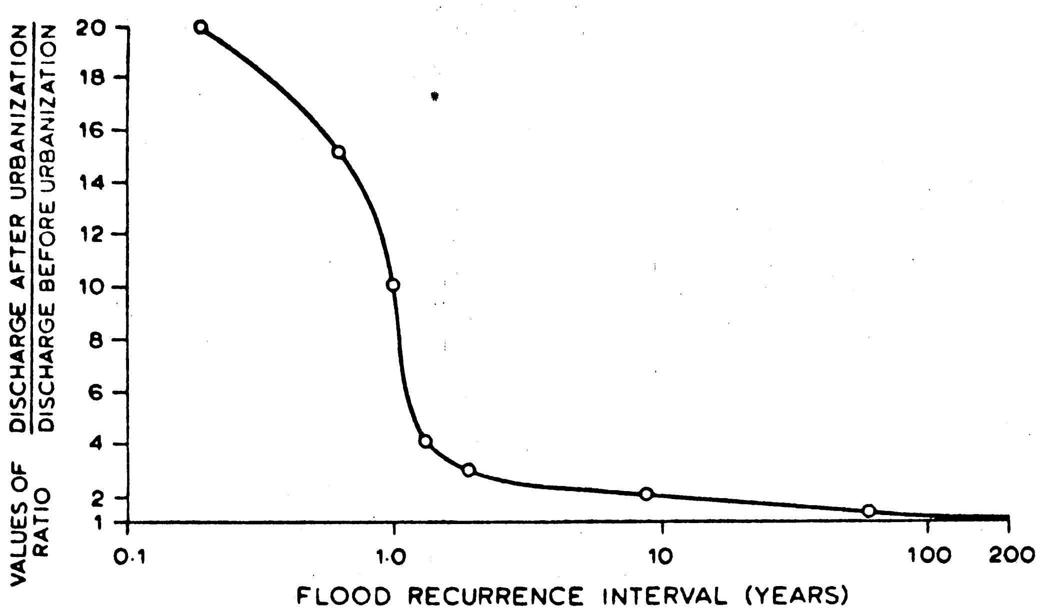


Source: Hare (55).

Figure 3. Effects of Watershed Development

to a fully developed residential urban basin will increase the flood peak at the 2-year recurrence interval by about 1.4 times, at the 10-year recurrence interval by about 1.2 times, and at the 50-year recurrence interval by 1.2 times. Esprey and Winslow (39) found that the flood peak discharge due to urbanization is significantly increased on two Texas creeks, ranging from no increase to about a 200% increase. In the Houston area, Hare (55) demonstrated that urbanization of an area will increase peak discharge rates for a given storm by a factor of from two to three. Work by Hollis (61) suggests that: (1) small floods may be increased by a factor of 10 or more depending on the amount of urbanization, and (2) 100-year floods may be doubled by the complete urbanization of a basin if at least 30% paving occurs. Simulations by Walesch and Videkovich (107) indicated that 100-year peak discharges at different locations in a watershed may be expected to increase by factors of 1.4 to 6.4 with a median value of 1.9.

For more severe storms, the effects of urbanization on a watershed can be expected to be less pronounced. After the initial infiltration loss and surface storage, a watershed begins to respond in a similar manner, whether the basin is urban or rural (39, 86). An analysis by Hollis (61) showed that the relative increase in flood peak discharge caused by urbanization declines as recurrence intervals increase (Figure 4). This relationship was also demonstrated by Croley and Barnard (30), who found that most of the urbanization impact appears as changes in the low recurrence interval flows. Anderson (3, p. 20) stated, "A completely impervious surface increases the average-sized flood by a factor of 2½, but an impervious surface has a decreasing effect upon larger floods and has an insignificant effect upon the 100-year flood."



Source: Hollis (61), p. 434.

Figure 4. Effect on Flood Magnitude of Paving 20% of a Basin

Impact on Lag Time

An impervious surface is much smoother than natural ground and thus more hydraulically efficient so that runoff occurs faster. The collector channels replace the natural channels with storm sewers or channel improvements that convey flow efficiently. Therefore, another net effect on a watershed that has considerable urban development as compared to its natural condition is that of increased speed of runoff or reduced lag time (34).

Anderson (3) found that lag time was the basin characteristic that was most affected by urbanization. Streams studied in northern Virginia showed that the lag time for a completely storm-sewered system is about one-eighth that of a comparable natural system. Bras and Perkins (14) showed urbanization reduces time to peak from 8% to 40% in Puerto Rico. The lag time of a basin in Charlotte, North Carolina, was found to decrease from 57% to 15% of the natural basin lag time as urbanization increased by Cruise and Contractor (31). McCuen (76) demonstrated that time-to-peak changed very little on a developed watershed in Maryland as compared to the natural basin.

Flood Hydrograph and Peak Flow Frequency Techniques

Introduction

The accurate prediction of streamflows is vital to the planning of water resources systems (41). This is especially true as concerned engineers, planners, and other professionals grapple with the consequences of land development and use on the quantity, and quality, of water in the surface water system of entire basins. Decisions on future urbanization may now be made with the benefit of prior evaluation of the probable effects of that urbanization on the surface water system. Numerous urban flood hydrograph and peak flow frequency techniques, primarily digital computer models of varying complexity, are now available to predict the impact of urban development and provide data for land-use planning in flood-prone areas (107).

A report by Rawls, Stricker and Wilson (86) provides a literature review of 128 papers (1962 to 1979) on urban flood flow frequency techniques. A concise overview of all categories of flood flow frequency procedures with descriptions of the more common models can be found in Feldman's report (41). Chen (19) and Narayana et al. (81) present well-documented reviews of the development of urban runoff models and Yen (112) provides a comprehensive review of existing urban storm runoff models.

The following sections will review the classification, comparison, and determination of use of flood hydrograph and peak flow frequency techniques.

Classification

Numerous classification schemes have been proposed for flood flow frequency estimation procedures (10, 41, 48, 86, 112). One of the most logical classification schemes is that presented by Feldman (41), which proposes that the techniques be separated into the following categories: (1) empirical formulae, (2) frequency analysis of historical streamflows, (3) statistical equations, (4) single event watershed models, and (5) continuous watershed models. In general, the first three categories predict only peak flow, while the second two categories predict the whole hydrograph or series of hydrographs, including a peak flow, by simulating the rainfall-runoff process.

Empirical formulae estimate a flood discharge of a given frequency as a function of watershed, climatic, and urban (where applicable) characteristics. The most famous and long lasting of these equations is the Rational formula, which is included in a comprehensive summary of various methods which utilize hydrologic variables in the design of small drainage structures by Chow (21). However, use of these equations is inconsistent and requires a great deal of engineering judgment at best.

Frequency analysis of historical streamflows utilizes streamflow records to directly estimate peak discharges at various frequencies. If adequate records exist and the watershed has not changed during that period of record, then this method may produce a good estimate of a watershed's flood responses in its present condition (41). The Water Resources Council's guidelines (52) describe the currently recommended techniques for utilizing the Log Pearson Type III distribution with numerous refinements and special situations. A basic understanding of the technique can be found in Beard (6) or Hjelmfelt and Cassidy (59). However, this method cannot be used directly to predict the magnitude-frequency of streamflows under some future watershed condition or if the basin has undergone significant changes during the period of record (41).

Statistical flood peak estimation procedures predict instantaneous peak flows of designated frequencies through a regression analysis of drainage basin and meteorologic variables affecting the storm runoff. A basic discussion of the method and the geographic variables that can be used to predict streamflows is presented in Beard (6). The most common examples of this technique are the U.S. Geological Survey's statewide regional analysis multiple regression equations. Thomas and Corley's report for Oklahoma (103) is an example. Adjustments for urban basins are based on percentage of the area impervious and served by storm sewers adapted from Leopold (72) (Figure 5). This method is subject to statistical error.

When it is necessary to use a watershed model instead of the simplified empirical or statistical methods? The watershed models are usually required when: (1) an entire hydrograph is desired; (2) analyzing complex areas; or (3) the proposed future watershed response characteristics are changing. Watershed models are particularly desirable when analyzing the effect of various water management plans (41).

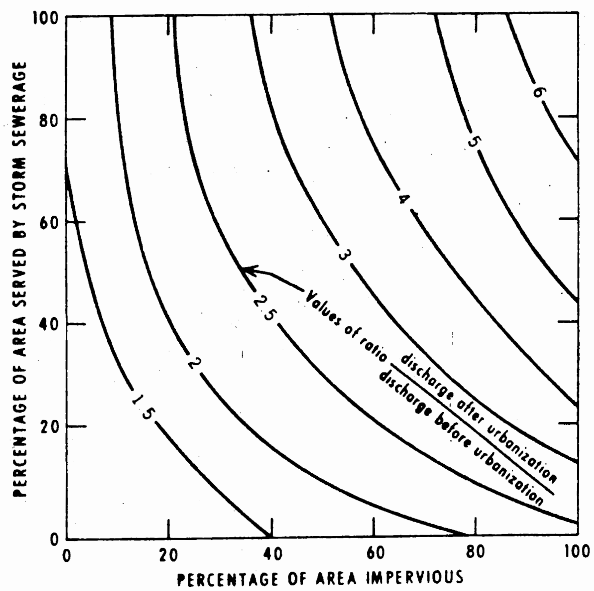
A single event model is used mainly for individual storm events. Two factors usually limit its use to single events: (1) the continuity of soil moisture (loss rates) is not simulated, and/or (2) the model is so detailed and requires so much computation time that it is not economical to run over long periods (41). Some of the most widely used single event models are:

1. HEC-1: Flood Hydrograph Package (56).

2. TR-20: Computer Program for Project Formulation Hydrology (28, (29).

3. U.S. Geological Survey (USGS) Rainfall-Runoff Simulator (35).

The current tendency in watershed modeling is to incorporate parameters that relate to the physical process and can be determined directly from easily available geographic data. As single event models become more geographically based and capable of readily predicting initial conditions, the less necessary continuous models may be. Then the single event model could be started before each significant event and a statistical analysis



Source: Leopold (72), p. 5.

Figure 5. Effect of Urbanization on Mean Annual Flood for 1-mi^2 Drainage Area

of the peak flows could be performed to make predictions for design purposes (41).

Most of today's continuous watershed models are derived from the Stanford Watershed Model. One of the most widely known of these derivations is the National Weather Service's NWSRFS model (84). The U.S. Army Corps of Engineers' STORM (99) is one of the simplest and most economical continuous watershed models. In these models the continuity of soil moisture (loss rates) is simulated and a long-record precipitation series is synthetically generated. This type of model is often criticized for its enormous data requirements. Usually the cost of assembling the necessary data often prohibits the use of these models in all but the most comprehensive studies (10, 41).

Urban Model Comparisons

There are now a multitude of urban runoff mathematical models that differ greatly in their scope, reliability, intended use, data requirements, and output. The continuous development of model refinements, and the large number of models available have hampered efforts to develop an acceptable criterion for systematic evaluation of model performance (2). However, there have been several efforts to categorize and compare their capabilities.

Brandstetter (13) made a comprehensive analysis of 18 urban stormwater models which compared catchment hydrology, sewer hydraulics, wastewater quality, and miscellaneous characteristics. Wanielista (109) reviewed 16 mathematical models relating details on input/output and computer hardware requirements. Chow and Yen (23) compared and evaluated 8 urban stormwater prediction methods. Six models, plus two variants of one and a variant of another were tested by Abbott (2), who made a preliminary evaluation of their relative capabilities, accuracies, and ease of application. Six single event urban rainfall-runoff quantity models commonly used by federal agencies were compared by Williams (111) and categorized by engineering uses, model use costs, and model resource needs. Rawls (86) reviewed 12 articles containing comparisons of urban flood flow frequency procedures.

There is an interesting program presently underway by the U.S. Water Resources Council (WRC). The WRC is testing procedures for estimating flood magnitude and frequency for ungaged watersheds (41, 102). In the first stage of the test, several people will estimate flood-frequency curves (2-, 10-, and 100-year peak discharges) at 65 watersheds in northwestern and central United States using ten different estimating techniques. The ten different techniques, to be tested, including some models, are:

1. U.S. Geological Survey (USGS) statewide regression equations.
2. Federal Highway Administration (FHWA) regression equations.
3. Regression equations developed by Brian Reich (Flood-Plain Manager, Pima County Highway Department, Tucson, Arizona).
4. U.S. Army Corps of Engineers (USCE) snowmelt runoff equations.
5. USGS Index Flood Method.
6. Rational Formula.
7. Procedure in Soil Conservation Service (SCS), Chapter 4.
8. Procedure in SCS TR-55, Chapter 5.
9. SCS TR-20 unit-hydrograph computer model.
10. USCE HEC-1 unit-hydrograph computer model (102, p. 88).

The ten methods will be applied at each of the 65 gaging station sites as if no data existed. The estimated flood-frequency curves will be compared to the station flood-frequency curves and the following criteria evaluated: accuracy, reproducibility, and practicality (102). The second phase will include a similar application in the southwestern and southeastern United States. A later phase of the studies will include urban studies (41).

Determination of Model Use

After reviewing the comparisons between urban runoff mathematical models, there remains the problem of determining which model to use. In selecting a model or models to use for a particular study, a tradeoff always exists between model simplicity and model accuracy (10, 48, 67). A more sophisticated model generally increases the accuracy of estimates, but requires more extensive data and increases the study cost.

General considerations in urban model selection are suggested by Beard (10): (1) data and time requirement for calibration and/or application, (2) computation requirement for application, (3) suitability for evaluating impact of urbanization, and (4) computer equipment required. In addition, another important consideration for many studies, including this one, is suitability for use on an ungaged watershed.

Hydrology

Introduction

An urban area is usually a hydrologically complex area with many factors contributing to the rainfall-runoff relationship. Therefore, it is desirable to employ an urban flood flow frequency method which develops the entire hydrograph instead of just the peak flow.

The shape of a watershed's hydrograph is a function of two main groups of factors that must be accounted for in a method (12): (1) hydraulic characteristics of the watershed, and (2) storm characteristics.

The hydraulic characteristics can be divided further into two major groups (12): (1) surface properties such as topography, stream density, channel storage, and percentage of impervious cover; and (2) watershed geometry such as area, length, shape, and slope. Storm characteristics include (106): (1) frequency, (2) duration, (3) amount, (4) temporal distribution, and (5) spatial distribution.

In developing design runoff hydrographs, there are generally four major tasks: (1) estimating a design storm rainfall; (2) estimating abstractions from rainfall; (3) developing a hydrograph from rainfall excess; and (4) routing the hydrograph through stream channels and reservoirs.

Design Storm

The starting point for most urban water resources studies is the consideration of storm rainfall. Rainfall data are much more readily available than streamflow data and less affected by urbanization (54). It is necessary to compute design floods from rainfall where conditions in the watershed change from historical conditions or where runoff records are not available (7).

The factors that must be considered in a design storm are (106): (1) frequency, (2) duration, (3) amount, (4) temporal distribution, and (5) spatial distribution.

In general, it is desirable to express the magnitude of peak flow for a specified frequency of recurrence. There are two general classes of rainfall-based prediction techniques: (1) runoff frequency is assumed to be equal to rainfall frequency, and (2) runoff frequency is calculated independently of rainfall frequency (41). The first assumption is often used (7, 10, 41, 55) because it simplifies the required analysis and because the second technique requires runoff records to develop.

Procedures for the computation of frequency curves of station precipitation are generally identical to those for streamflow analysis (6, 22, 23). However, instantaneous peak intensities are not usually analyzed, but linked with amounts for specific durations to obtain depth-duration-frequency curves. An extensive compilation of these curves for the United States can be found in U.S. Weather Bureau Technical Paper 40 (USWB TP40) (58) with instructions of how to apply data to specific locations. Some single event modelers have devised their own rainfall frequency analysis procedures to use in thier models (41).

One of the problems with a single storm is that a particular sequence of precipitation events may also cause a critical flood situation. This would make the use of continuous models attractive. However, the construction of a long-period precipitation series is also a difficult task due to the scarcity of data (41).

In urban stormwater studies, relatively short duration but high intensity rainfalls are the most important (54). For small watersheds, durations of approximately 6 hours or less are satisfactory for design storms (106). Kent (71) states the effective storm period that contributes to an instantaneous peak rate of discharge for most watersheds smaller than 2,000 acres is less than 6 hours. Design storms of 30 minutes to 14 hours have been used for urban basins (1, 19, 23, 53, 54, 66, 101).

Rainfall amount or depth is obtained using a network of both recording and non-recording gages. The National Weather Service (NWS, formerly USWB) maintains the largest network in the United States with many organizations and individuals contributing data. The recording gages provide a complete time-intensity history of rainfall events with the non-recording gages used primarily for 24-hour rainfall amounts. As previously mentioned, the data are used to compute depth-duration-frequency curves.

Next to the degree of watershed imperviousness, the storm pattern used in a study is the most important factor. Runoff changes significantly with temporal rainfall distribution (1, 15).

The difficulty of estimating a rainstorm pattern given a return period has led to the development of synthetic storms. These synthetic storms have the advantage of a consistent basis for design (54). In general, a single time pattern for any given storm frequency is satisfactory, if the depth-duration relationship represents an average of all storms of that frequency (8). A "balanced" storm rainfall pattern is constructed from the depth-duration-frequency curves consisting of a typical time sequence with intensities or depth for each duration corresponding to the specified recurrence frequency for that duration (9, 10). In other words, for a given frequency the 30-minute depth is in the peak 30 minutes of the synthetic pattern, the 1-hour depth is contained in the peak 1 hour of the curve, etc.

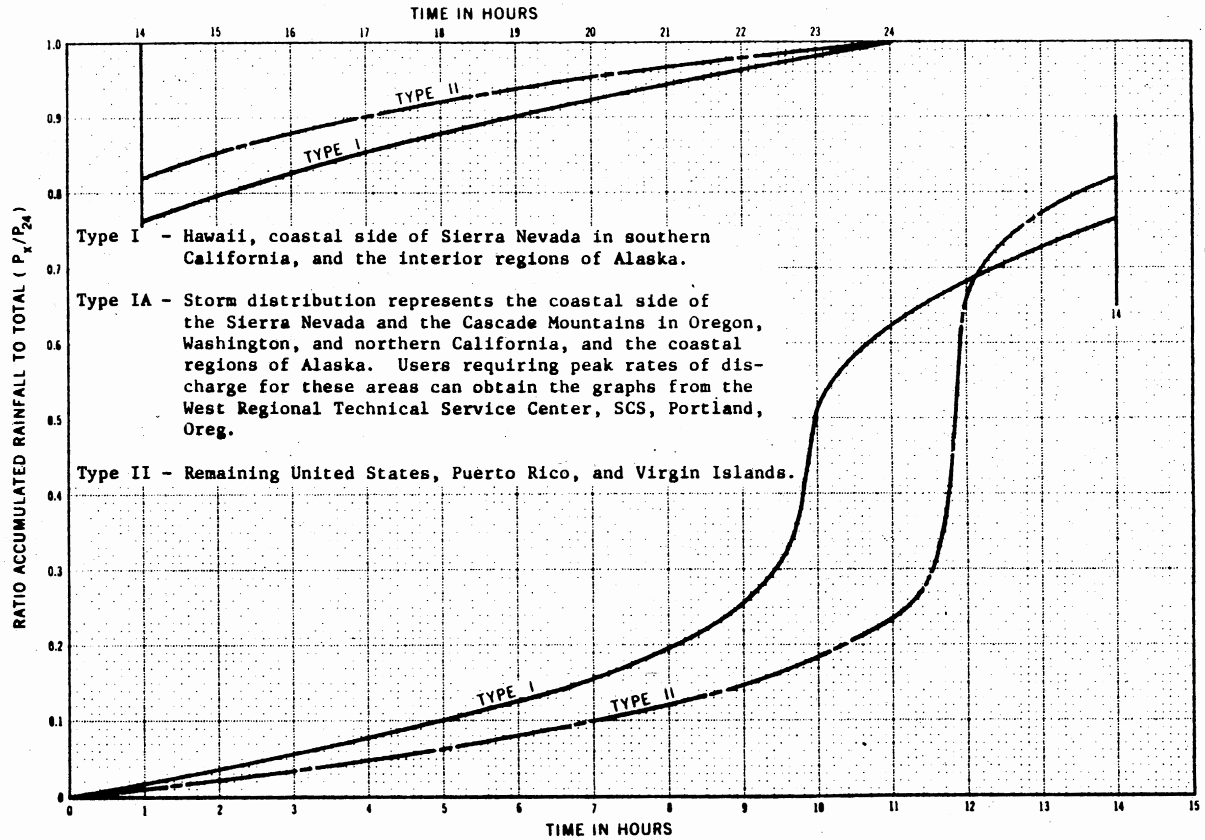
How the incremental volumes should be arranged to form a typical pattern has been the subject of much research. Huff (63) studied the time distribution of heavy rainfalls from small central Illinois watersheds with a duration of 3 to 48 hours. He divided the storms into four groups depending on the time quartile in which the majority of the rainfall volume occurred. In each quartile, nine curves were constructed from 10% to 90% probability, which indicate a storm has that time distribution or one above it. Of the total number of storms, 66% were in the first or second quartiles. The 50% or median curve is recommended for most applications (106).

The SCS has developed two 24-hour storm patterns called Type I and Type II (Figure 6) (65, 71). The curve used depends on the part of the United States that is being studied. These mass curves were derived so that for the selected frequency, the depth-duration curve based on the curves would be close to the depth-duration curve developed from the USWB TP 40 (54, 58). A synthetic storm of a given frequency for a given duration, two hours for example, would be constructed by using the most intense two hours of the curve. These two hours are then incremented and the 24-hour rainfall amount is multiplied by the incremental curve values (54).

The SCS has also developed a 6-hour design storm distribution used in developing emergency spillway and freeboard hydrographs (58). This curve is very similar to Huff's 50% (median) second quartile curve (63, 106).

Precipitation depths often vary from point to point during a storm. This spatial or areal variation can have a significant impact on runoff hydrographs (15, 101).

Rainfall depth-duration-frequency data are developed from point rainfall information. When the data are applied to large watersheds, reduction factors must be applied as given in USWB TP 40 (58). The correction is much greater for short duration storms which might generally be thunderstorms (54).



Source: Kent (71), p. 2

Figure 6. Soil Conservation Service Twenty-Four-Hour Rainfall Distribution

In small watersheds, areal variation in design storm depth is normally disregarded (106).

Abstractions

Abstractions from precipitation are "losses" that do not show up as storm runoff. Therefore, the volume of stormwater runoff is equal to the volume of effective rainfall or rainfall excess, precipitation minus abstractions. Abstractions include evaporation, transpiration, interception, detention storage, and infiltration.

Evaporation is the process by which water is transferred from the land and water masses of a watershed to the atmosphere. Transpiration is the process by which water is evaporated from the pores in plant leaves. The total evaporation from an area, combined evaporation and transpiration, is called evapotranspiration. During storm periods, evapotranspiration is usually not significant (106). There are discussions of these factors and estimation techniques in Chow (22), Hjelmfelt and Cassidy (59), and Viessman et al. (106). Only complex models, especially continuous event models, account for these factors.

Interception is the part of storm precipitation which is intercepted by vegetation and other forms of cover on the watershed. Detention storage is the part of precipitation that is trapped in numerous small depressions on the surface of the watershed. Reservoir storage is usually treated in hydrograph routing, as explained in a later section. These factors are generally included in initial abstraction which includes all the storm rainfall occurring before surface runoff starts (65, 104). Detailed discussions of these components with numerous account methods can be found in hydrology references (22, 65, 106).

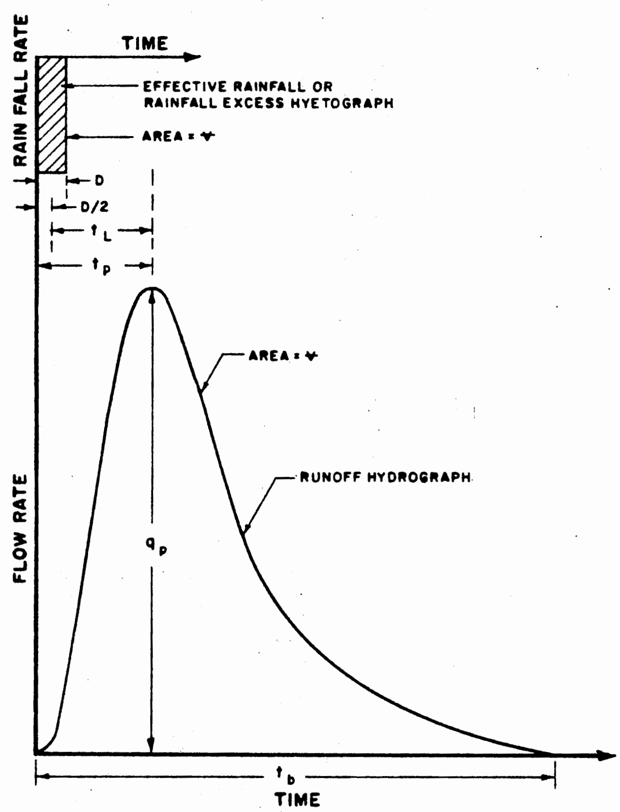
Infiltration is the flow of water into the ground through the earth's crust. The rate at which it occurs is highly dependent upon the type and condition of the watershed's surface. Infiltration is a very important factor because it not only affects the timing, but also the distribution and magnitude of surface runoff (54, 106). Therefore, any hydrologic model must include a reliable method of estimating infiltration, which is a major abstraction from precipitation. All models of moderate to high complexity generally employ some nonlinear relationship that indicates as rainfall supply exceeds infiltration capacity, infiltration rate tends to decrease in an exponential manner. Discussion of specific infiltration functions may be found in hydrology references (22, 59, 65, 106) and in each model's description (19, 23, 28, 33, 35, 40, 48, 53, 54, 56, 60, 71, 74, 76, 79, 84, 87, 92, 101, 104).

Hydrograph Development

The unit hydrograph method is the most versatile approach to hydrograph synthesis of excess runoff (40) and this method is utilized in most hydrologic models (93). The following discussion is a brief overview of hydrograph nomenclature, unit hydrograph concept, and the synthetic unit hydrograph method.

Most of the nomenclature used in discussing runoff hydrographs is shown in Figure 7 (54). The rainfall excess hyetograph is depicted as a single block of rainfall with duration D in the upper portion of the diagram. The runoff hydrograph comprises the lower portion of the figure. The area enclosed by the hydrograph and hyetograph depicts the same volume of water.

The maximum flow rate on the hydrograph is the peak flow q_p; the time



Source: Hann (54), p. 359.

Figure 7. Hydrograph Terminology

from the start of the hydrograph to q_p is the time to peak t_p. The total duration time of the hydrograph is known as the base time t_b. The lag time t_L is defined here as the time from the center of mass of the effective rainfall to the peak of the runoff hydrograph. Using that definition:

$$t_p = t_L + D/2 \tag{2.1}$$

However, some define lag time as the time from the center of mass of effective rainfall to the center of mass of the runoff hydrograph (54).

A time parameter not displayed in Figure 7 is the time of concentration, t_c. The time of concentration is the time it takes water to flow from the hydraulically most remote point on a watershed to the watershed outlet.

Next the unit hydrograph concept will be discussed. A unit hydrograph is defined as the direct runoff hydrograph due to one inch of effective rainfall falling uniformly over the watershed during a storm of a specified duration (12). The method of constructing a unit hydrograph from an observed runoff hydrograph with a given rainfall excess is described in detail in hydrology books (22, 59, 106). Basically, after finding the storm of the desired duration, and separating the baseflow from direct runoff on the chosen stream flow hydrograph, each of the runoff time coordinates is divided by the average depth of rainfall excess to find the unit hydrograph ordinates. Usually many are constructed and an average or representative unit hydrograph is used (59).

To construct a hydrograph resulting from a storm with the same duration as the unit hydrograph, but with rainfall excess different than one inch, just multiply the ordinates by inches of excess rainfall, keeping the time coordinate unchanged.

Then to construct a hydrograph representing a storm duration different than that of a unit hydrograph, a method is used as shown in Figure 8. The rainfall excess is divided into increments, with each increment having a duration equal to that of the unit hydrograph. Then the unit hydrograph is applied to each increment and a composite hydrograph is constructed (59).

However, the number of streams which have gaging stations is very small compared to the total of streams and rivers, especially urban streams. Therefore, it is usually necessary to synthesize a unit hydrograph for a stream of interest.

Many methods have been developed for obtaining synthetic unit hydrographs which are presented in hydrology references (22, 59, 65, 106) and in each hydrologic model's description (19, 23, 28, 33, 35, 40, 48, 53, 54, 56, 71, 74, 76, 79, 84, 87, 92, 101, 104).

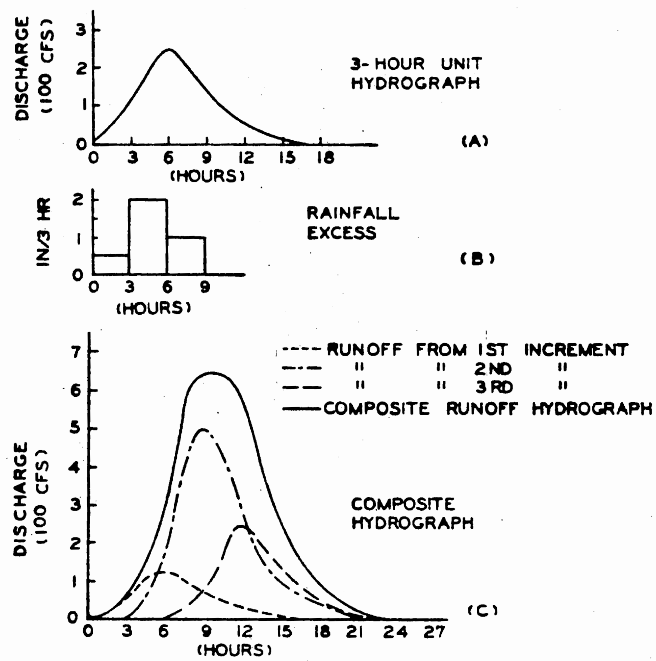
The general procedure for predicting the hydrologic characteristics of a watershed by synthetic unit hydrograph includes the following:

1. Choosing a number of hydraulic watershed parameters, such as percentage of impervious cover and area, that seem likely to influence the unit hydrograph.

2. Selecting a number of gaged watersheds possessing these parameters in a varying degree.

3. Looking for correlations between these parameters and characteristics of the observed unit hydrographs such as peak discharge, and time to peak.

4. Expressing the most significant correlations either graphically or mathematically in such a form that they can be used to predict the unit hydrographs of either other ungaged watersheds, or gaged watersheds where a change, such as an increase in urbanization, has taken place (12, p. 150).



Source: Hjelmfelt, Cassidy (59), p. 107.

Figure 8. Application of the Unit Hydrograph

Hydrograph Routing

The method for computing runoff hydrographs that is most complete would be to route the rainfall excess as overland flow to established channels and as channel flow to the watershed outlet or control point. This would take into account the storage characteristics of the watershed that would delay and decrease the peak of the runoff hydrograph (54).

Any procedure would rely on the momentum equation and continuity equations (known as the St. Venant equations) as set forth by Yen (112), as well as flow relationships between various hydraulic factors such as slope, roughness, channel shape, and hydraulic radius. Various simplifications have been developed to give approximate solutions to the two complex equations (112).

Routing models using only the continuity equation, often rewritten in the form:

$$I - O = dS/dt \tag{2.2}$$

where

I = rate of inflow into the control volume considered;

O = rate of outflow from the control volume considered;

S = storage within the control volume considered; and

t = time elapsed.

are known as hydrology routings (112). The hydrologic routing methods, including the various coefficient routing procedures such as the Muskinghum technique, and the reservoir routing, can be found in standard reference books (20, 22, 59, 65, 106).

Many models also include the option for routing through reservoir storage which would have similar effects on the runoff hydrograph.

Hydraulics

Introduction

Once the flood discharges are computed by a hydrologic model, the water surface elevations along a stream must then be determined for a FIS. Flood elevations are normally calculated using step-backwater computer models (44, 93). These models utilize an iterative procedure, the standard step method, which attempts to solve Bernoulli's energy equation in a stream reach defined by two cross sections at the two ends of the reach. Chow (20) presents a detailed discussion of the theory behind the method and a comprehensive example of the step-by-step procedure. The calculations are very laborious, and as an iterative method quite suited to digital computer application.

Three commonly used step-backwater computer programs are (80): (1) HEC-2, developed by the USCE Hydrologic Engineering Center (57); (2) E-431, developed by the USGS (91); and (3) WSP-2, developed by the SCS. The models differ in how they compute head losses and conveyance, or the measure of the carrying capacity of the channel.

Model Differences

The head losses usually accounted for are friction head losses, head loss through a bridge structure, and minor losses such as expansion and contraction losses. In all the models, the friction head loss in the reach is computed by Manning's equation, but head loss through a bridge is computed differently in each model by using different hydraulic equations (57, 80, 91). HEC-2 requires the input of both expansion and contraction coefficients, which are then used to multiply the difference in the velocity heads of the two cross sections to obtain the expansion and contraction head losses in the reach. E-431 assumes the expansion loss to be one-half the difference in the velocity heads, and no contraction loss. WSP-2 does not have any provision for minor losses in the model (80).

A cross-section can be divided into subsections to calculate hydraulic properties in all the models. HEC-2 computes overbank conveyances station by station, while E-431 computes a total wetted perimeter and hydraulic radius and then conveyance for each overbank section. Therefore, HEC-2 will use a smaller wetted perimeter and larger conveyance for overbank sections. The average conveyance of the stream reach are computed differently in the models: HEC-2 takes the arithmetic mean of the conveyances at the two cross sections; E-431 uses the geometric mean; and WSP-2 assumes the conveyance of the upstream section as the average conveyance (80).

Motayed and Dawdy (80) conducted a comparison of the three models on a stream reach and found differences in water surface elevation computation due to minor loss and conveyance calculation differences alone. E-431 gave the highest elevations, WSP-2 the least, and HEC-2 produced an intermediate water surface profile. Bridge computations should also contribute to differences in water surface elevation profiles. Therefore, before utilizing a particular model, the user must understand the problem studied, and the assumptions and validity of the model results.

Urban Flood Damages

Introduction

In any flood plain management plan, methods are needed to estimate flood damage to assess flood control measures. There are five empirical categories of flood damage (51, 67, 68, 75):

1. Direct damages to inundated property such as structures, and public facilities such as roads and utilities.

2. Indirect damages caused when a flood interrupts business and services, and the cost of the alleviation of hardship and health safeguards.

3. Secondary damages resulting from losses to those depending on the use of or output from the interrupted services or damaged property.

4. Intangible damages such as hardship, grief, loss of life and health, environmental quality, social well being, and aesthetic values or other items that are difficult to evaluate in monetary terms.

5. Uncertainty damages accruing to the occupants of a flood plain because of the uncertainty with regard to when the next flood will occur and how severe it will be.

Indirect damages have been estimated as a percentage of direct damages depending on land use (51, 67, 107). Generally, the secondary damages are offset by secondary benefits and are not included in damage estimates (51). Intangible benefits are very hard to quantify (51). Therefore, the next sections will explore methods of estimating direct damages and uncertainty damages, followed by a synopsis of the factors that affect flood damages.

Direct Damages

There are various methods used to calculate direct damages. Using the classification proposed by Grigg and Helweg (51), there are three categories: (1) aggregate formulas, (2) historical damage curves, and (3) empirical depth-damage curves.

James and Lee (68) have an example of the aggregate formula approach. They suggest for estimation of single event urban damages:

$$C_d = K_d M_s d \tag{2.3}$$

where

C_d = direct damage for a particular flood event, in dollars;

K_d = marginal flood damage per unit of depth, in feet^{-1} (ft^{-1});

M_s = market value of inundated structures, in dollars; and

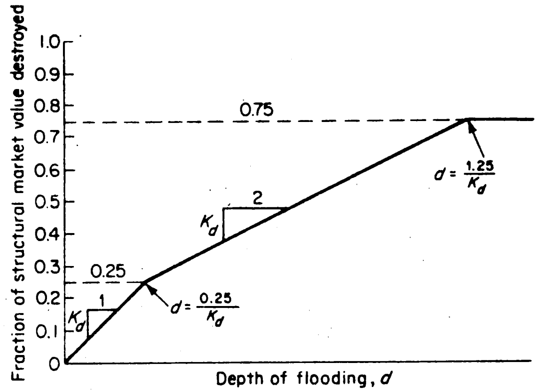
d = depth of flooding, in feet (ft).

For shallow flooding one value of K_d is used and for deeper flooding the marginal flood damage per unit of depth can be expected to decrease approximately as shown in Figure 9 (68).

The historical damage curve method results in the historical damages of floods plotted against flood stage; an example is shown in Figure 10. For valid current use, the damage costs must be corrected to present values by including additional development of the flood plain and the correction of inflation (51).

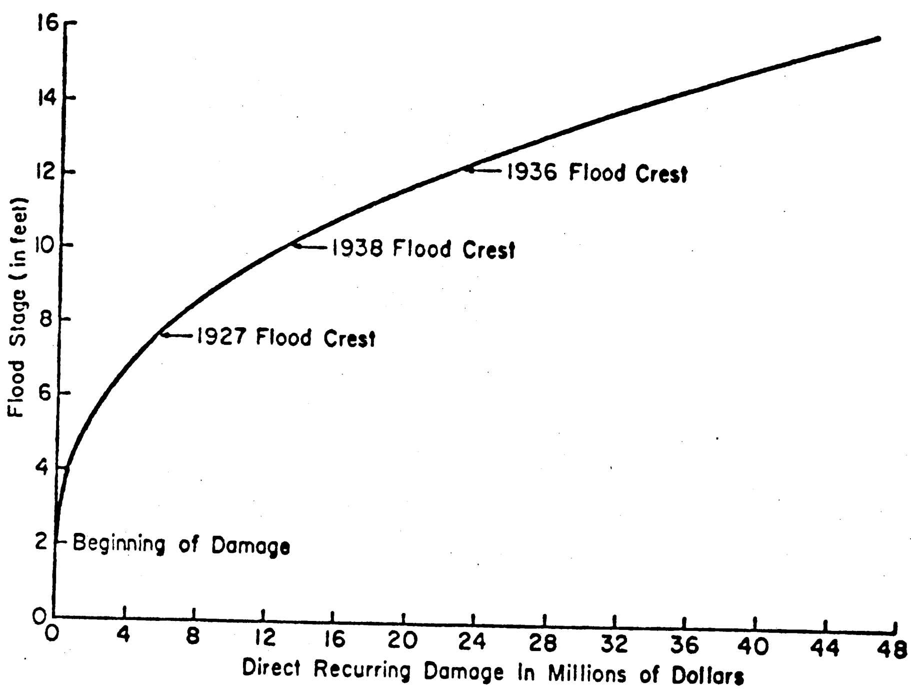
The third damage estimation method requires a property survey of the flood plain and either an individual or composite estimate of depth versus damage curves for the structures on the flood plain (51).

The value of property on the flood plain can be obtained in many ways. The most common method is to obtain the market value of property from local property tax records (67). There has been some work to utilize statistical techniques to obtain land valuation (50, 90, 94). The two major problems with utilizing regression equations or any other statistical technique has been low predictability, since it is unlikely that



Source: James (68), p. 252

Figure 9. Flood-Damage-Depth Curve for Urban Property



Source: Grigg (51), p. 383

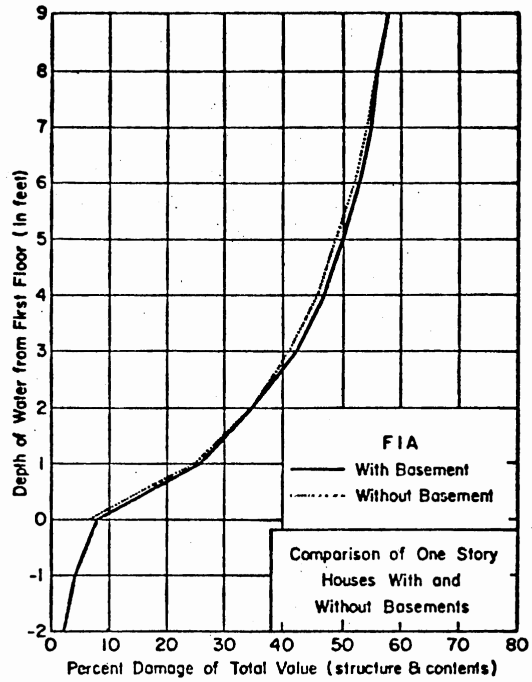
Figure 10. Historical Depth-Damage Curve

a single set of relevant independent variables will be applicable across different urban areas, and the requirement for extensive data (50).

The depth-damage curves are commonly developed as depth versus percent damage tables. The Federal Insurance Agency (FIA, now Insurance and Mitigation Division of the Federal Emergency Management Agency) has constructed many depth versus percent-damage tables from extensive data for residential structures (42). Grigg and Helweg (51) compared these tables with other available data and found them reasonable.

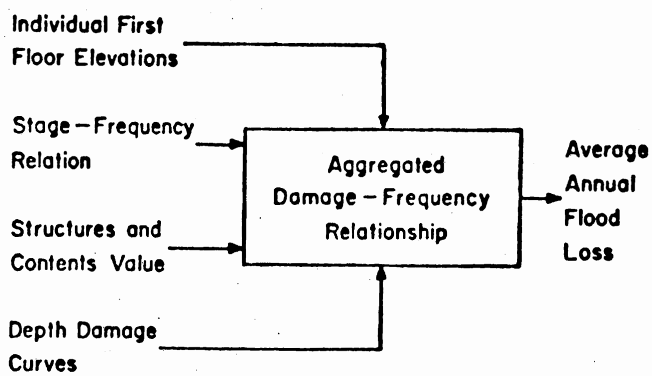
One problem in using the damage tables is that percent damage is applied separately to value of structure and then to value of contents. Due to the difficulty of obtaining content value data, often a percentage of structure value is used. Values for this percentage range from 20% to 60% (51, 67, 69). Many private insurance companies use 50%. After a percentage is chosen, many authors use a composite depth versus percentage damage curve, as shown in Figure 11 (51, 69).

Therefore, depth-damage curves, in conjunction with flood elevations, may be used to estimate single event direct damages or used to estimate a term called expected annual average flood loss, as depicted in Figure 12. The stage-discharge relationships and discharge-frequency data for a stream are used to obtain a stage-frequency relationship. Then this is combined with the single event damages for various floods to obtain an aggregated damage-frequency curve. The area under this curve yields the expected average annual flood loss. Lovell and Smith (73) describe a computer program called DAMAL which utilizes an extensive data base on a watershed to compute these economic data.



Source: Grigg (51), p. 388

Figure 11. Comparative Depth-Damage Curve



Source: Grigg (51), p. 383

Figure 12. Computational Procedure for Average Annual Flood Loss

Uncertainty Damages

The uncertainty damage cost may be computed as the amount in excess of the expected value of the damages that flood plain residents are willing to pay to avoid a flood loss (51). Also, it has been shown that some individuals are willing to pay an amount greater than the expected monetary value of a loss in insurance premiums to escape uncertainty (105). Therefore, annual insurance premiums would be a reasonable indicator of uncertainty damage costs.

Vaut (105) has a comprehensive discussion of the economic theory of flood insurance, including individual behavior under uncertainty.

Flood Damage Factors

Although the general depth-damage curves will give a good estimate in most cases, it must be noted that damages are affected by many variables besides depth. McCrory, James and Jones (75) present a summary of these factors:

1. Flood depth. Flood depth determines the elevation to which property is wetted, the magnitude of hydrostatic pressure, and whether or not escape transportation is cut off.

2. Flow velocity. High velocity flows create hydrodynamic forces that add pressure on walls, scour around foundations, and transport debris that can batter structures.

3. Flood duration. Prolonged wetting lengthens the decay period and adds to the damage of most materials, and prolonged periods of inundation add to the seriousness of human displacements.

4. Advanced warning. Longer advanced warnings provide greater opportunity for emergency flood proofing and moving transportable items to safety.

5. Sediment content. Sediment increases abrasive action, adds to the work and cost of cleanup, and accelerates deterioration by slowing the drying process.

6. Wave action. Waves increase flood depths and add to hydrodynamic forces.

7. Season. Recreational activities are most vulnerable to damage in the summer, and crops are most vulnerable immediately before harvest.

8. Time between floods. People tend to forget the risk and unwisely develop the flood plain during long flood free periods. After very short periods between floods, previous damages may not be sufficiently repaired for much more harm to occur.

9. Type of structure. Certain building materials and layouts are more subject to flood damage than others.

10. Placement of contents. Flood damages are reduced as more of the building's contents are located at higher elevations (75, p. 199).

CHAPTER III

BASIC DATA

Introduction

Before any water resources investigation is undertaken, a large data base must be compiled. Many federal, state, and local government agencies have to be contacted to obtain every data source possible. The following sections are compilations of the basic data--the maps, photographs, and cross-section data--used in this study.

Maps

Maps are an invaluable aide to the investigator to obtain watershed characteristics such as area, topography, and drainage systems. An accurate, current, large scale map, with a scale of 1 inch = 400 feet or larger and contour intervals of 5 feet or less, is desirable for urban studies.

The following maps were utilized in this investigation:

1. City of Stillwater planimetric maps with a scale of 1 inch = 200 feet and a contour interval of 5 feet (25).

2. City of Stillwater drainage area map of 6th Avenue with a scale of 1 inch = 200 feet.

3. City of Stillwater drainage area map of Western Road and Hall of Fame Avenue with a scale of 1 inch = 600 feet.

4. Oklahoma State University planning map showing the university buildings and storm sewer system with a scale of 1 inch = 200 feet.

5. USGS topographic maps with a 10 foot contour interval and a scale of 1 inch = 2000 feet (97, 98).

6. A base map of the Duck Creek Study Area with a scale of 1 inch = 400 feet was made by enlarging a composite of the USGS topographic maps above.

Photographs

Since maps usually do not portray a current picture of a watershed's characteristics, aerial photographs and field photographs are a necessity to obtain an up-to-date assessment of such factors as urban development, watershed cover, and stream channel condition (49, 108). Current aerial photographs of a scale 1 inch = 400 feet or larger are desirable for urban studies.

The photographs used in this investigation are:

1. City of Stillwater aerial photographs with a scale of 1 inch = 200 feet (24).

2. SCS soil survey field sheets of Payne County with a scale of 1 inch = 1320 feet.

3. Duck Creek channel field photographs by author showing channel condition (Figure 13).

4. A base aerial photograph of the Duck Creek Study Area with a scale of 1 inch = 400 feet was made by reducing a composite of the City of Stillwater aerial photographs above.

a. Looking Upstream Near Mouth of Duck Creek

b. Looking Upstream From 9th Avenue Culvert

Figure 13. Examples of Channel Photographs

Cross-Section Information

Cross-section information is necessary for hydrologic analysis if a moderate to complex model is used and for the hydraulic analysis of flood-prone area determination. Next to the hydrologic and hydraulic analyses, obtaining these basic data is often the most expensive part of a study if no previous data are available.

In addition to utilizing maps, photographs, and field inspection, the following sources were used for cross-section geometry:

1. USCE surveyed cross-section information used in the Stillwater Flood Plain Information Report (45).

2. Hudgins, Thompson, Ball and Associates preliminary construction plans for Duck Creek channel improvement (100).

3. SCS as-built construction drawings of Dam 30 (96).

CHAPTER IV

HYDROLOGY

Introduction

One of the major decisions in a flood plain management study is which method to use in the hydrologic analysis. The SCS method (65, 104), utilizing the TR-20 computer program (28, 29), was chosen because it is well suited to flood plain management studies and is moderate in model use costs and resource needs (111).

The SCS's curve number technique has received increased interest and usage (19, 32, 41, 48, 53, 76, 79, 102) due to the current strong interest in relating watershed model parameters to geographic characteristics. This method is the only one in which both the precipitation loss rate and the excess precipitation to runoff transformation (unit hydrograph) can be determined from readily available geographic data (41). The advantage of a model that has input parameters defined in terms of land use or land cover is that the investigator can experiment with alternate conditions of land development and assess the impact the changes might have (87).

Although it is desirable to calibrate any model with observed runoff data, the TR-20 model should give reasonable estimates on ungaged watersheds, as shown by studies such as those conducted by Danushkodi (32) and Williams (111).

Thomas and Corley's regression equations (103) are also used to provide an estimate of peak discharge rates in order to compare the TR-20 results with a method that has low model use costs and resource needs.

In the following sections, first the two methods of peak discharge computation will be discussed and then the preparation of the input parameters for the TR-20 model. Finally, the resulting design peak discharges will be presented.

U.S. Geological Survey Regression Equations

Introduction

Thomas and Corley (103) contains the USGS's statewide regression equations for estimating flood discharges for Oklahoma streams with drainage areas under 2,500 square miles. Equations and graphs for obtaining estimates of the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year flood peak discharges are presented.

To obtain peak discharges for an ungaged urban basin requires two steps: (1) calculate the 2-year peak discharge and other recurrence interval floods as desired for an ungaged rural site, and (2) calculate the urban peak discharges with the ungaged urban site equations.

Ungaged Rural Site

In the first step, the following equations for an ungaged rural site were used for the Duck Creek basin (103):

$$Q_2 = 0.111\ A^{0.66}\ S_o^{0.23}\ P_a^{1.92} \qquad (4.1)$$

$$Q_{10} = 2.99\ A^{0.68}\ S_o^{0.28}\ P_a^{1.22} \qquad (4.2)$$

$$Q_{50} = 20.0\ A^{0.69}\ S_o^{0.31}\ P_a^{0.81} \qquad (4.3)$$

$$Q_{100} = 38.6\ A^{0.70}\ S_o^{0.32}\ P_a^{0.67} \qquad (4.4)$$

$$Q_{500} = 140\ A^{0.71}\ S_o^{0.33}\ P_a^{0.40} \qquad (4.5)$$

where

Q_T = peak discharge for recurrence interval T, in cubic feet per second (ft^3/s);

A = contributing drainage area of the basin, in square miles (mi^2);

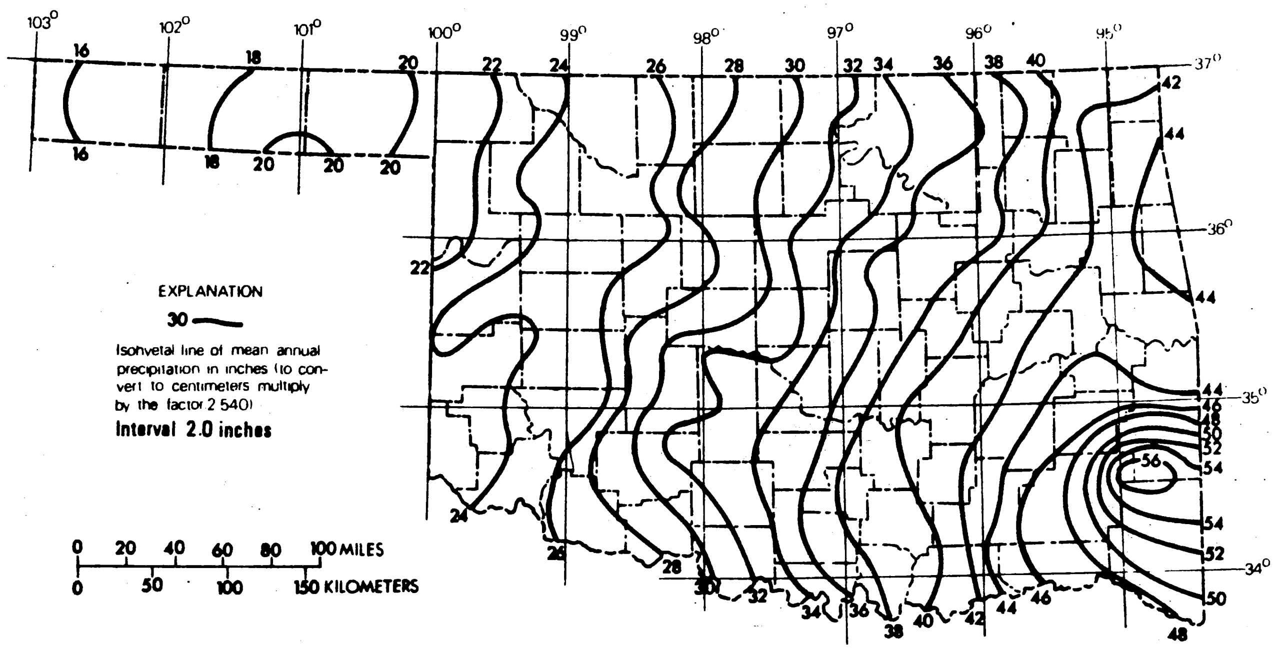
S_o = main-channel bottom slope, determined from elevations at points 10 and 85 percent of the distance along the channel from the gaging station (control point) to drainage divide, in feet per mile (ft/mi); and

P_a = mean annual precipitation for the basin during the period 1931-1960 (Figure 14), in inches (in.).

Ungaged Urban Site

In the second step, the percentage of the basin impervious and served by storm sewers is required in addition to the variables required for the rural site equations. The percentage of the basin impervious was determined from the curve number analysis, which will be explained in a later section. Thomas and Corley (103) state that the percentage of the watershed served by storm sewers "should be determined from the best available storm sewer and drainage map" (p. 14). Since the streets serve almost as efficiently as storm sewers in high recurrence interval flood events, it was assumed that all of the urban area was storm sewered and a value of 100% was used within that area. These values must be weighted by area when open spaces and rural areas are included with the urban area.

After determining the percentage of the basin impervious and served



Source: Thomas (103), p. 5

Figure 14. Mean Annual Precipitation for the Period 1931-60

by storm sewers, R_L, the urban adjustment factor, is obtained from Figure 5. The urban adjustment factor is the ratio of the mean annual flood under urban conditions to rural conditions. The following equations can then be used to adjust estimates from equations in step one to urban conditions:

$$Q_{2(u)} = R_L \, Q_2 \tag{4.6}$$

$$Q_{10(u)} = 1.87 \, (R_L - 1) \, Q_2 + 0.167 \, (7 - R_L) \, Q_{10} \tag{4.7}$$

$$Q_{50(u)} = 2.46 \, (R_L - 1) \, Q_2 + 0.167 \, (7 - R_L) \, Q_{50} \tag{4.8}$$

$$Q_{100(u)} = 2.72 \, (R_L - 1) \, Q_2 + 0.167 \, (7 - R_L) \, Q_{100} \tag{4.9}$$

$$Q_{500(u)} = 3.30 \, (R_L - 1) \, Q_2 + 0.167 \, (7 - R_L) \, Q_{500}. \tag{4.10}$$

Soil Conservation Service Method

Introduction

The SCS developed its computer program TR-20 (28, 29) for storm water runoff in 1965, originally intended as a design method for flood retention structures on agricultural basins. In 1975, a procedure for implementing the model on urban basins was introduced (104). The program is a single event model that calculates a complete hydrograph for surface runoff from any synthetic or natural storm rainfall event. It can account for watershed conditions affecting runoff and will route the hydrograph through stream channels and reservoirs. The model can combine the routed hydrographs with those from other tributaries (basin subareas) and print out the resultant hydrograph, and the water surface elevations corresponding with the hydrograph coordinates, at any designated cross section or structure (control points).

The method can be found in detail in SCS literature (65, 71, 104) and consists of two basic steps: (1) solving a runoff equation to estimate direct runoff from precipitation, and (2) transforming this runoff into a hydrograph. Channel and reservoir routing may be performed if the basin is divided into subareas and/or if a reservoir is present in the study watershed.

Rainfall-Runoff Equation

Figure 15 shows the schematic curves of accumulated storm runoff P, direct runoff Q, and infiltration plus initial abstraction $(F + I_a)$ used in developing the rainfall-runoff equation. Assume:

$$\frac{F}{S_a} = \frac{Q}{P_e} \qquad (4.11)$$

where

F = infiltration occurring after runoff begins, in inches (in.);

S_a = potential abstraction, in inches (in.);

Q = direct runoff, in inches (in.); and

P_e = potential runoff or effective storm runoff, storm rainfall minus initial abstraction, in inches (in.).

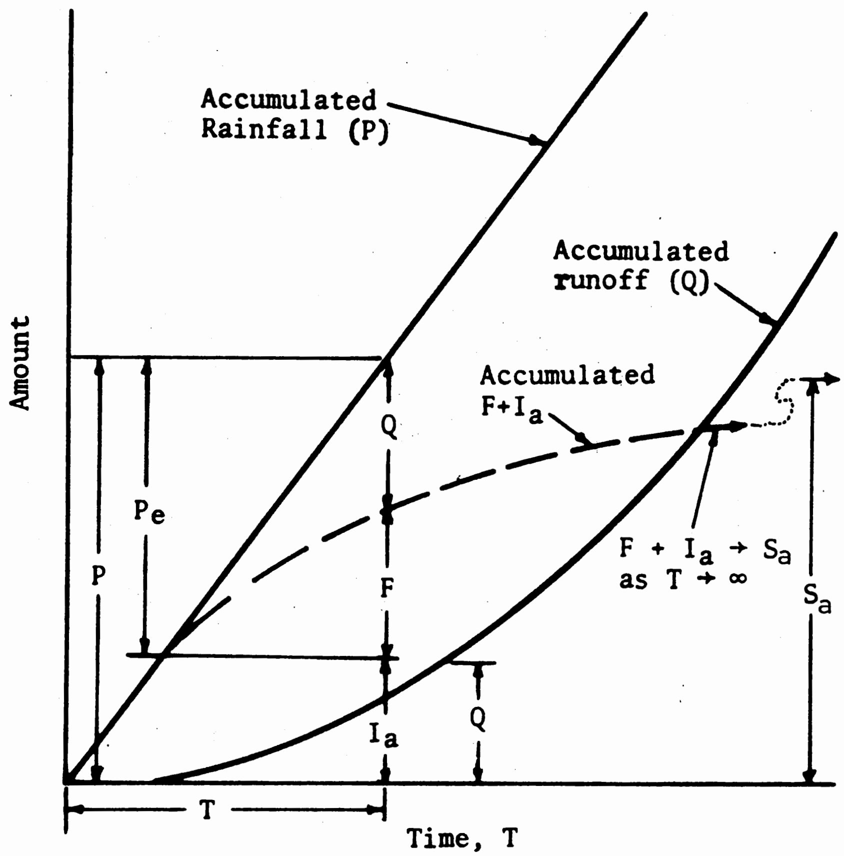
Since

$$F = P_e - Q \qquad (4.12)$$

Equation (4.11) can be rewritten as:

$$Q = \frac{P_e^2}{P_e + S_a} \qquad (4.13)$$

An empirical relation based on data from small watersheds gives an



Source: U.S. Soil Conservation Service, p. 2-2

Figure 15. Schematic Curves of Accumulated Rainfall (P), Runoff (Q), and Infiltration Plus Initial Abstraction ($F+I_a$) Showing the Relation Expressed by Equation (4.16)

estimate of the initial abstraction:

$$I_a = 0.2\ S_a \tag{4.14}$$

where I_a is the initial abstraction, in inches (in.). Therefore:

$$P_e = P - I_a = P - 0.2\ S_a \tag{4.15}$$

where P is the total storm rainfall, in inches (in.). Substituting Equation (4.15) in Equation (4.13) gives the rainfall-runoff equation:

$$Q = \frac{(P - 0.2\ S_a)^2}{P + 0.8\ S_a} \tag{4.16}$$

Potential abstraction S_a is related to the cover conditions and soil conditions of a watershed. The SCS had developed a parameter, CN, called the runoff "curve number," or hydrologic soil-cover complex number, which is related to a watershed's hydrologic soil types, vegetative cover, percent imprevious cover, and antecedent soil moisture. Tables and procedures outlining the estimation of this parameter for a soil-cover complex are covered in References (65), (71), and (104). The CN is related to potential abstraction by:

$$CN = \frac{1000}{S_a + 10} \tag{4.17}$$

from which

$$S_a = \frac{1000}{CN} - 10 \tag{4.18}$$

Thus all rainfall losses may be expressed in terms of one parameter, the curve number. If runoff records are also available, the model can be calibrated by solving for CN in Equations (4.16), (4.17), and (4.18).

Triangular Hydrograph Equation

After the rainfall-runoff relation is developed, the next step is to transform the excess precipitation into a hydrograph. The SCS has developed a triangular hydrograph equation to represent excess runoff with only one rise, one peak, and one recession (65, 71, 104).

The following equation will estimate the peak rate of discharge:

$$q_p = (K\,A\,Q)/t_p \qquad (4.19)$$

where q_p is the peak rate of discharge, in cubic feet per second (ft^3/s); and K is a constant, 484 for units used here. Time to peak is expressed as:

$$t_p = \frac{D}{2} + t_L \qquad (2.1)$$

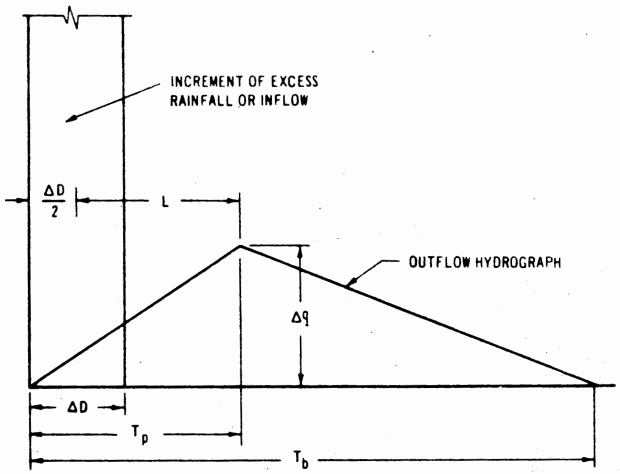
The following empirical relationship between lag time and time of concentration is used when the entire hydrograph is developed:

$$t_L = 0.6\ t_c \qquad (4.20)$$

To use Equation (4.19) for other than uniform storm rainfall, it is required to divide the rainfall into increments of duration (ΔD) and compute the corresponding increments of runoff (ΔQ) (Figure 16). The peak discharge equation for an increment of runoff becomes:

$$\Delta q_p = \frac{484\ A\ (\Delta Q)}{\frac{\Delta D}{2} + t_L} \qquad (4.21)$$

The ordinates of the individual triangular hydrographs for each Δq_p are then added to develop a composite hydrograph (Figure 17). Note that each incremental hydrograph is displaced one ΔD to the right for each subsequent time increment.



$$\Delta q = \frac{484\ A\,(\Delta Q)}{\frac{\Delta D}{2} + L} \text{ in C.F.S.}$$

Where:

ΔD = INCREMENT OF STORM PERIOD IN HOURS

ΔQ = RUNOFF IN INCHES DURING PERIOD ΔD

Δq = PEAK DISCHARGE IN C.F.S. FOR AN INCREMENT OF RUNOFF

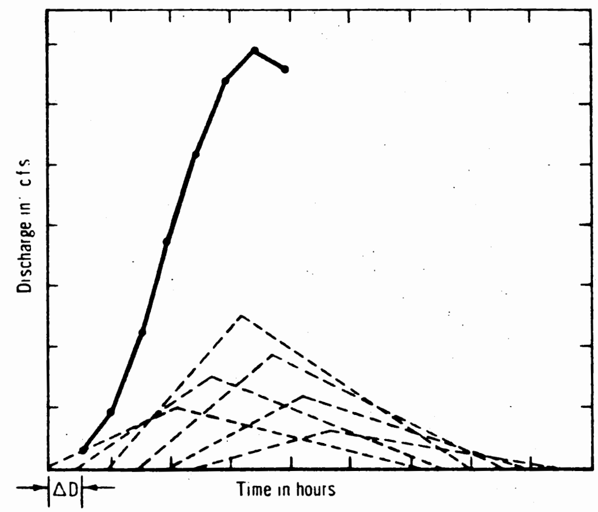
A = DRAINAGE AREA IN SQUARE MILES

T_p = TIME TO PEAK $\left(= \frac{\Delta D}{2} + L\right)$ IN HOURS

T_b = TIME OF BASE $(= 2.67\ T_p)$ IN HOURS

Source: Kent (71), p. 7

Figure 16. Triangular Hydrograph Relationships



Source: Kent (71), p. 13

Figure 17. Composite Hydrograph from Hydrographs for Storm Increments ΔD

In TR-20, a dimensionless curvilinear unit hydrograph (Figure 18) is utilized that has the same properties as the triangular unit hydrograph.

Input Parameters

A detailed description of the capabilities, input, and output of TR-20 is presented in the computer program user's manual (28, 29). The input parameters required for a hydrograph are:

1. The cumulative rainfall mass curve.
2. The watershed's surface area.
3. The watershed's curve number.
4. The watershed's time of concentration.
5. The watershed's dimensionless unit hydrograph shape.

If the basin is subdivided into subareas to give a better estimate of a complex watershed, the above parameters must be input for each subarea.

In addition, if channel routing is utilized, usually in a subdivided watershed, the following parameters are required:

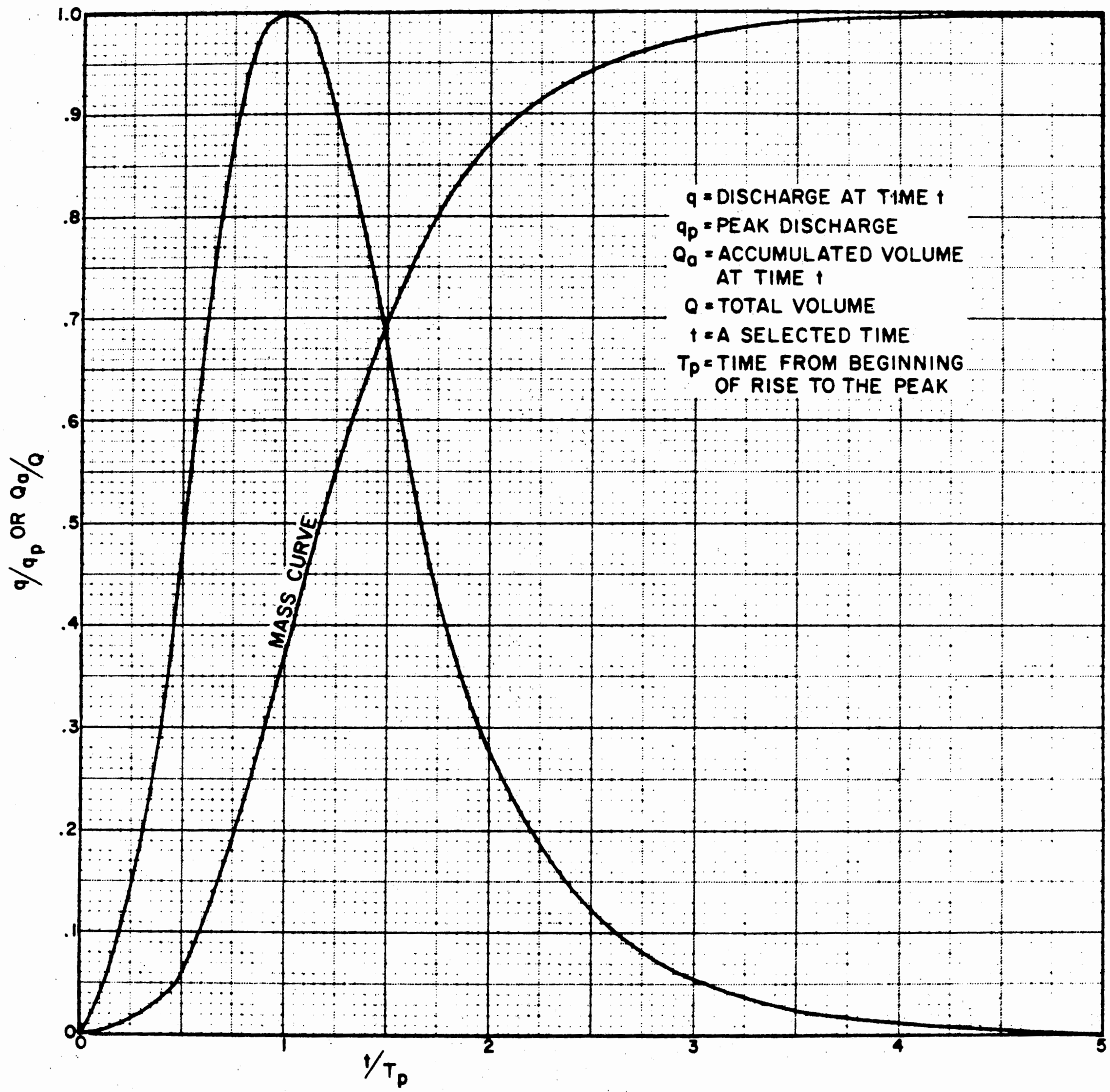
1. Cross-section rating curves.
2. Stream reach length between cross sections.

If there is reservoir routing to be performed, the following structure data are also required: (1) storage curve, and (2) outlet works rating curve.

Design Storm

Frequency

In this study, the frequency of the storm rainfall is assumed to be the same as the flood peak discharge frequency. Therefore, the frequencies



Source: U.S. Soil Conservation Service (65), 16.3

Figure 18. Dimensionless Unit Hydrograph and Mass Curve

utilized are the same as mandated for flood insurance studies (44): (1) 10-year, (2) 50-year, (3) 100-year, and (4) 500-year storms.

Duration

Duck Creek has a small drainage area and has a history of effective storm peak rainfall occurring in a few hours (27). Therefore, the 24-hour storm was not used. Since the effective storm rainfall duration for small watersheds has been proposed to be 6 hours or less (71, 106), the 1-, 3-, and 6-hour storm were utilized in a preliminary discharge run. The results will be presented in a later section.

There is another reason that the 6-hour storm was the maximum duration used in modeling the Duck Creek basin. The maximum number of hydrograph points in TR-20 is 300. Therefore, Δt or main time increment should be: (1) small enough to adequately define the hydrograph and large enough so that most of the hydrograph will fit into 300 elements (28); and (2) small enough to get good hydrograph definition encountered in the subareas with small t_c--the smallest t_c was 0.10 hour. Kent (71) and Williams (110) recommend if possible:

$$\Delta t < t_c/4 \qquad (4.22)$$

Therefore, the main time increment was 0.02 hour for the 1- and 3-hour storms, and 0.03 hour for 6-hour storms.

Amount

The rainfall amount or depth used in the study design storms was taken from Meyer's report (78). Since local rainfall data were available, it was not necessary to utilize the data in the NWS TP 40 (58).

The 500-year storm data were not available in Meyer (78). Therefore, the available data were plotted on log-probability paper to obtain depth-duration-frequency curves, and the 500-year storm depths were extrapolated from the graphs (Figure 19).

Distribution

The SCS emergency spillway design storm distribution (Figure 20) was utilized as the temporal storm pattern for all durations and frequencies. For the 1- and 3-hour storms, it was input as a dimensionless pattern and then assigned the appropriate duration. A check revealed that this pattern is a "balanced storm" pattern for the Stillwater data. For example, the 5-, 10-, 15-, 30-minute, and 1-, 3-hour amounts were in the peak 5-, 10-, 15-, 30-minute, and 1-, 3-hour peak increments of the 6-hour distribution.

Adjustment of the rainfall data with respect to area, or spatial, distribution is not necessary because the drainage area of Duck Creek is small (71).

Basin Subareas

Since the Duck Creek watershed, as many urban basins, has a complex hydrologic response, it was subdivided into basin subareas in order to provide a better estimate of peak discharges. Subarea drainage divides were determined by using topographic maps, storm sewer drainage maps, and field inspection. Two sets of subarea configurations were developed in order to compare the estimates from a simple and a complex pattern.

The simple subarea configuration (Figure 21) utilized 9 subareas and 9 stream control points. The complex subarea configuration (Figure 22) used 22 subareas and 12 stream control points.

The subareas were outlined on the 1 inch = 400 feet base map and

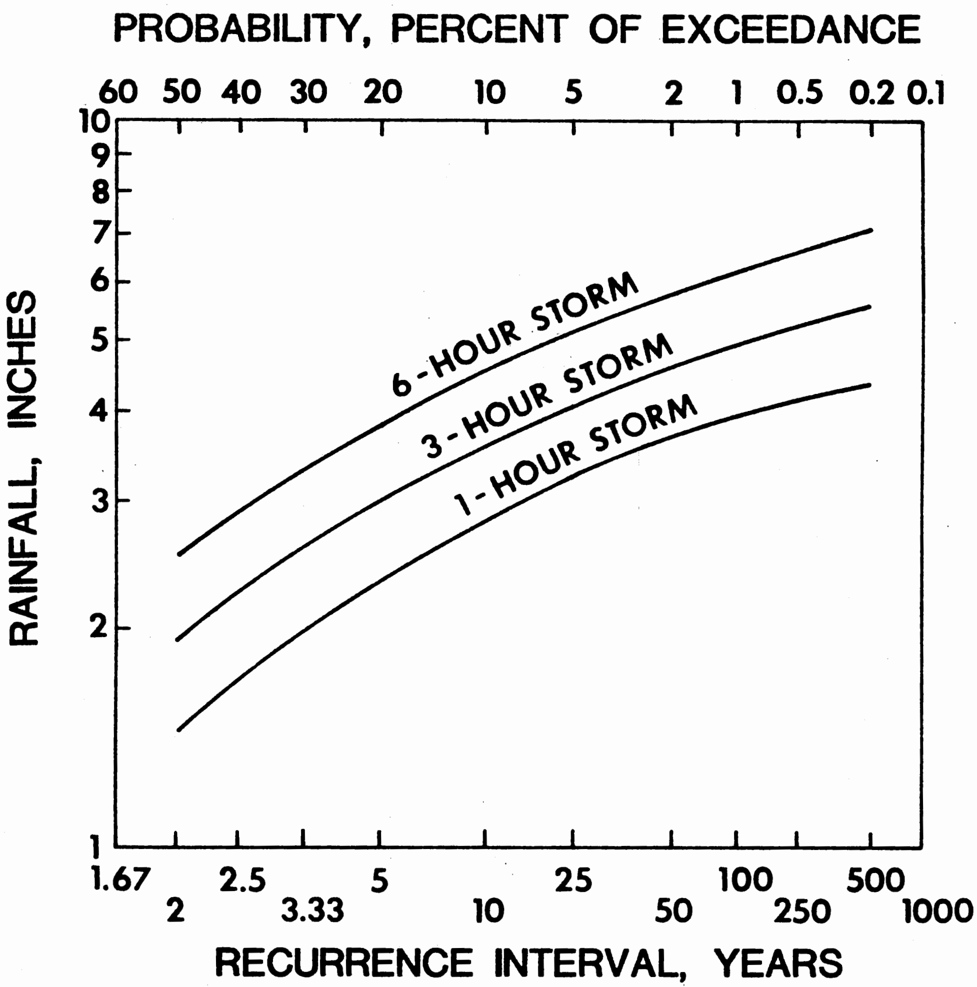
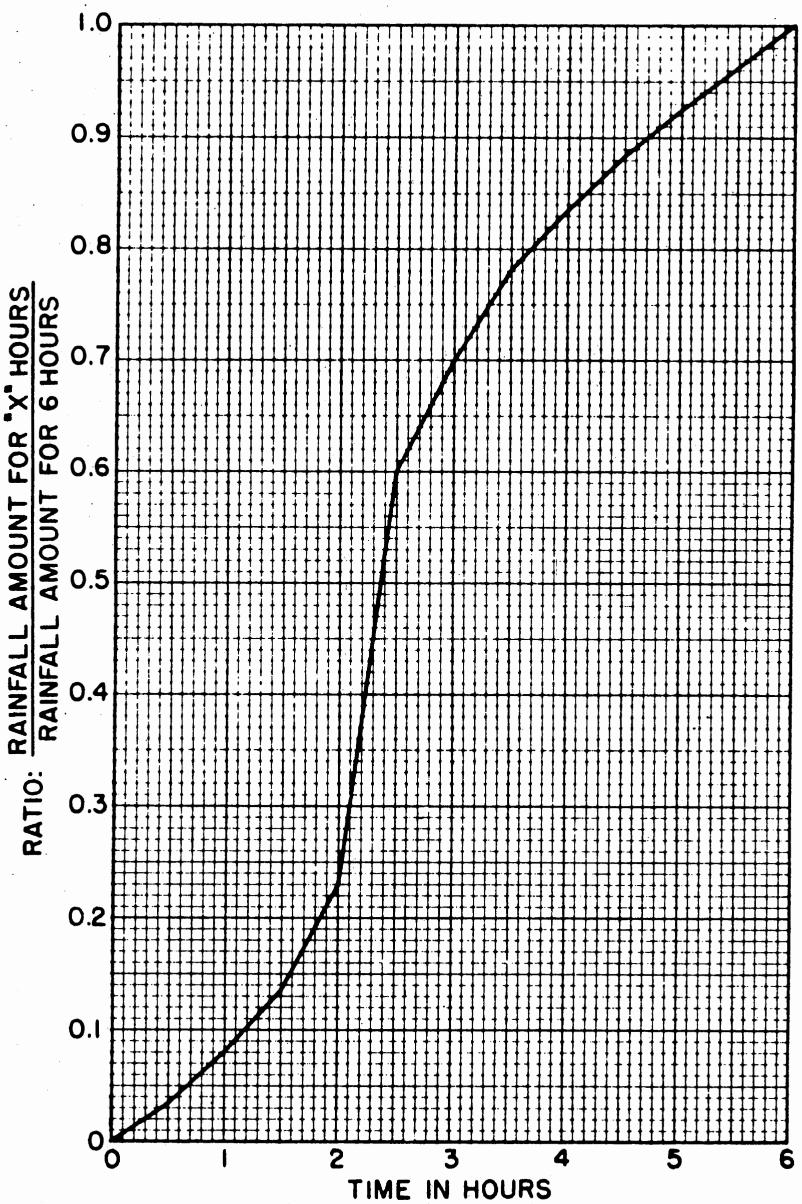


Figure 19. Rainfall Depth-Duration-Frequency Curves for Duck Creek



Source: U.S. Soil Conservation Service, p. 21.81

Figure 20. Six-Hour Design Storm Distribution

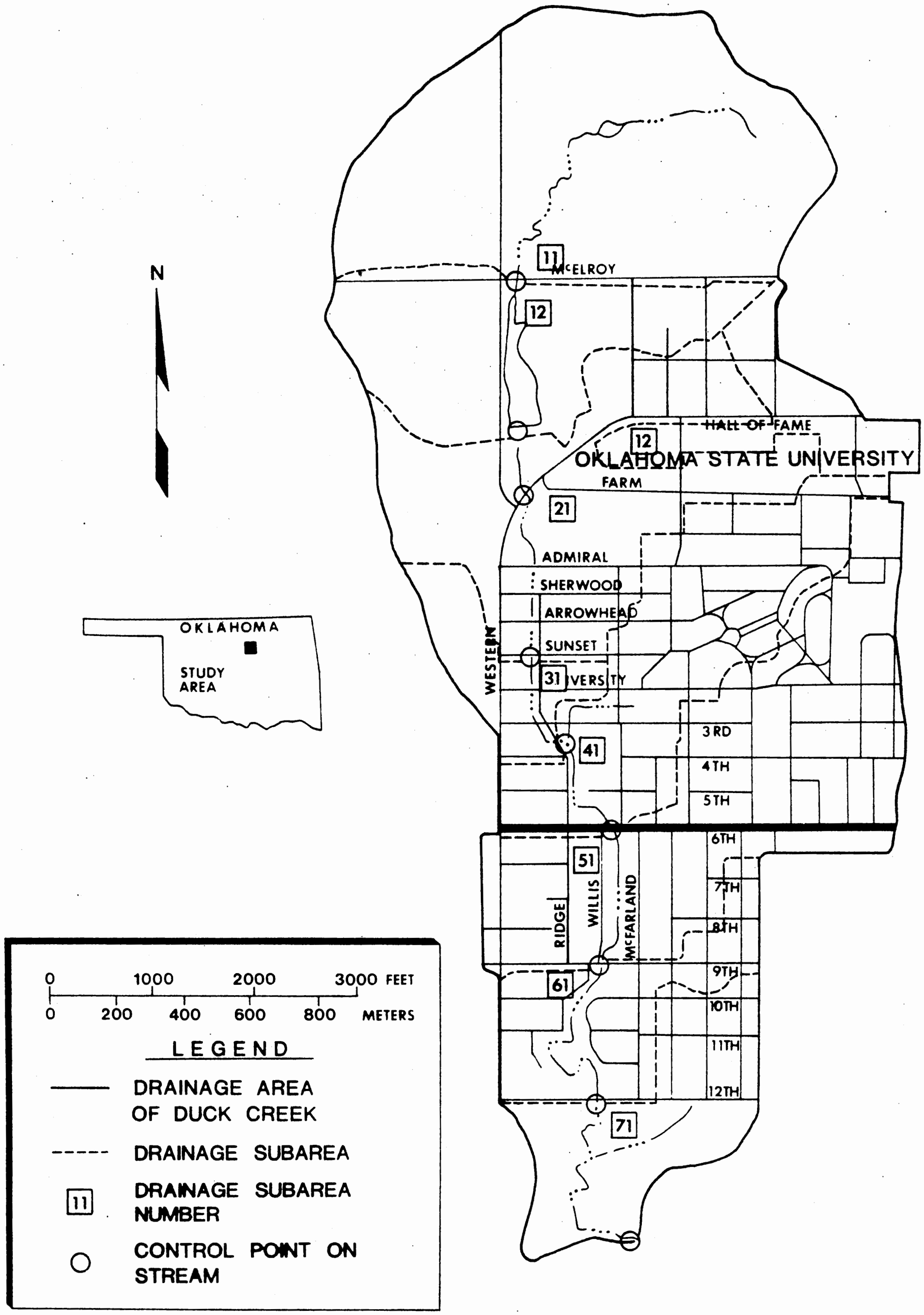


Figure 21. Simple Subarea Configuration

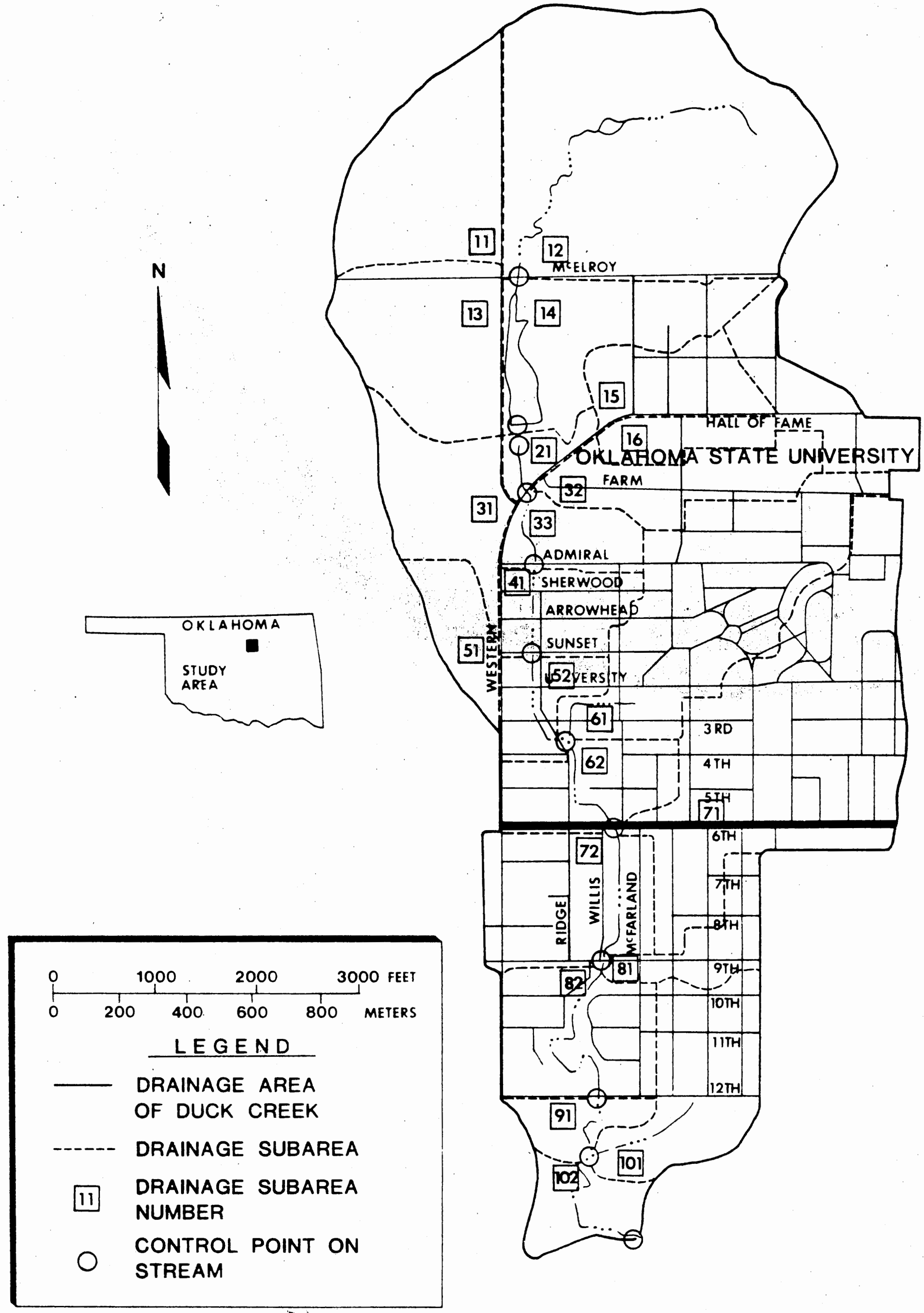


Figure 22. Complex Subarea Configuration

drainage areas were determined with a Dietzgen digital readout planimeter.

Abstractions

Hydrologic Soil Groups

Soil properties influence the rainfall-runoff process and must be considered in runoff estimation. The SCS has provided tables (65, 104) which list soil names and their hydrologic classification, A, B, C, D, which is an indicator of the minimum rate of infiltration obtained for a bare soil after prolonged wetting. By using the hydrologic soil classification and associated land use, curve numbers can be computed.

The hydrologic soil groups, as defined by the SCS, are:

A. (Low runoff potential). Soils having a high infiltration rate even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels.

B. Soils having a moderate infiltration rate when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse texture.

C. Soils having a slow infiltration rate when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water or soils with moderately fine to fine texture.

D. (High runoff potential). Soils having a very slow infiltration rate when thoroughly wetted and consisting chiefly of clayey soils with a high swelling potential, soils with a permanent high water table, soils with a claypan, or clay layer at or near the surface, and shallow soils over nearly impervious material (104, p. B-1).

Soil descriptions and soil survey field sheets (aerial photographs with soil series overprinted on them) for the Duck Creek area were obtained from the Payne County SCS office. The hydrologic soil groups (Figure 23) determined from this information were outlined on the base map to assist in curve number determination.

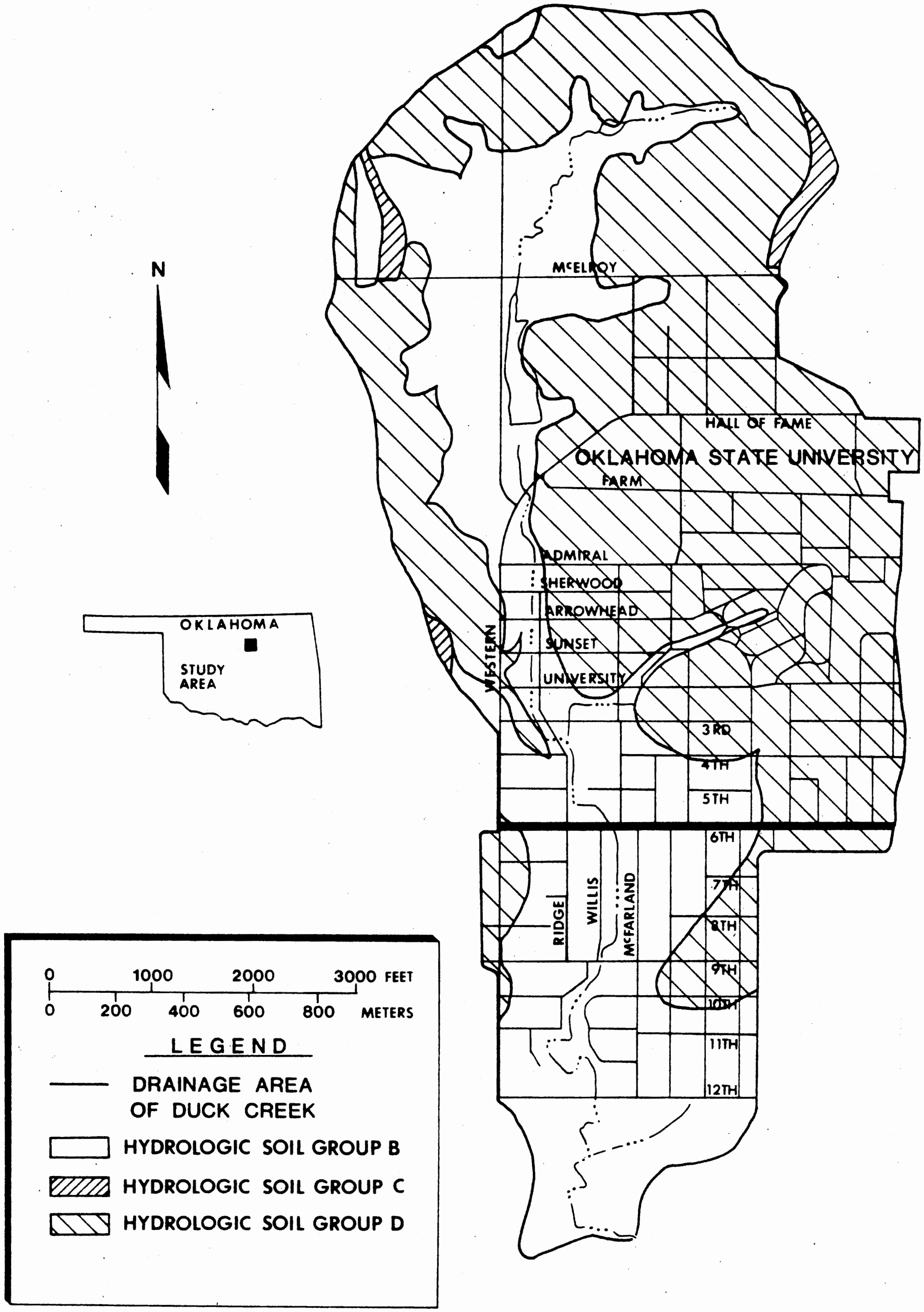


Figure 23. Hydrologic Soil Groups on Duck Creek

Urbanization

To analyze the difference in future hydrologic response of the watershed's soil-cover complex, a future urbanization condition was imposed on the basin.

Areas of future probable development in the basin for the following types of development were identified (Figure 24): (1) Oklahoma State University, (2) commercial, and (3) residential.

Curve Numbers

A weighted curve number, CN, was computed for each complex subarea in the present basin condition as outlined in Chapter 2, Reference (104). The CNs were selected from Table I using aerial photographs (24) as a guide for cover condition and the hydrologic soil groups as previously determined.

The weighted CNs were then computed for the simple subareas in the present basin condition by compositing the above information.

The process was repeated for determining the CNs with the basin in the future urbanization condition by adjusting the curve numbers in the appropriate subareas. The resulting CNs are presented in Table II.

Antecedent Soil Moisture

A succession of storms, such as one a day for a week, decreases the magnitude of S_a each day because the limiting factor, whether it is the infiltration rate at the soil surface, or the transmission rate of the soil profile, or the water capacity of the soil profile, does not have a chance to completely recover.

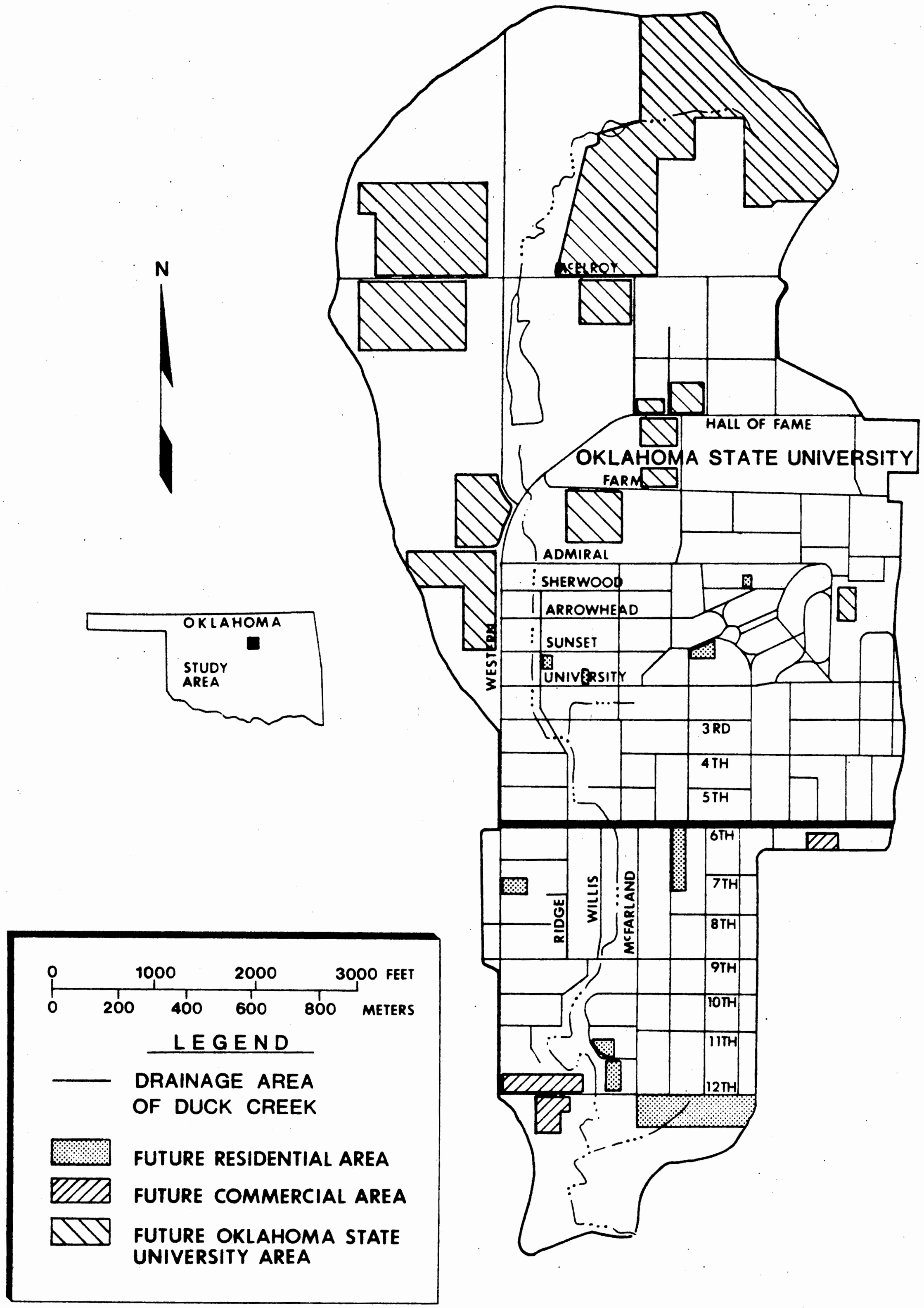


Figure 24. Future Urbanization on Duck Creek

TABLE I

RUNOFF CURVE NUMBERS FOR SELECTED AGRICULTURAL, SUBURBAN, AND URBAN LAND USE

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.

Source: U.S. Soil Conservation Service (104), p. 2-5

TABLE II

COMPARISON OF SCS CURVE NUMBERS

LOCATION	SUBAREA NUMBER: SUBAREA CONFIGURATION: SIMPLE	SUBAREA NUMBER: SUBAREA CONFIGURATION: COMPLEX	DRAINAGE AREA (SQUARE MILES): SUBAREA CONFIGURATION: SIMPLE	DRAINAGE AREA (SQUARE MILES): SUBAREA CONFIGURATION: COMPLEX	DRAINAGE AREA (SQUARE MILES): TOTAL: BASIN	DRAINAGE AREA (SQUARE MILES): TOTAL: BELOW DAM	SCS CURVE NUMBER: URBANIZATION PRESENT: SIMPLE	SCS CURVE NUMBER: URBANIZATION PRESENT: COMPLEX	SCS CURVE NUMBER: URBANIZATION FUTURE: SIMPLE	SCS CURVE NUMBER: URBANIZATION FUTURE: COMPLEX
		11		0.09				71		79
		12		0.26				76		83
	11		0.35				75		82	
MCELROY					0.35	0.00				
		13		0.08				73		79
		14		0.08				83		85
		16		0.07				87		88
	12		0.23				80		84	
SCS DAM 30					0.58	0.00				
		21		0.01				75		75
HALL OF FAME					0.60	0.01				
		15		0.04				88		89
		31		0.06				78		83
		32		0.05				84		86
		33		0.03				81		84
ADMIRAL AVENUE					0.78	0.19				
		41		0.04				85		85
	21		0.23				82		85	
SUNSET DRIVE					0.81	0.23				
		51		0.03				79		83
		52		0.03				79		79
	31		0.06				79		81	
RIDGE ROAD					0.87	0.29				

TABLE II (Continued)

LOCATION	SUBAREA NUMBER: SUBAREA CONFIGURATION: SIMPLE	SUBAREA NUMBER: SUBAREA CONFIGURATION: COMPLEX	DRAINAGE AREA (SQUARE MILES): SUBAREA CONFIGURATION: SIMPLE	DRAINAGE AREA (SQUARE MILES): SUBAREA CONFIGURATION: COMPLEX	DRAINAGE AREA (SQUARE MILES): TOTAL: BASIN	DRAINAGE AREA (SQUARE MILES): TOTAL: BELOW DAM	SCS CURVE NUMBER: URBANIZATION PRESENT: SIMPLE	SCS CURVE NUMBER: URBANIZATION PRESENT: COMPLEX	SCS CURVE NUMBER: URBANIZATION FUTURE: SIMPLE	SCS CURVE NUMBER: URBANIZATION FUTURE: COMPLEX
RIDGE ROAD					0.87	0.29				
		61		0.13				88		88
		62		0.05				76		76
	41		0.18				85		85	
SIXTH AVENUE					1.05	0.47				
		71		0.21				87		88
		72		0.08				81		81
	51		0.29				85		86	
NINTH AVENUE					1.34	0.75				
		81		0.03				86		86
		82		0.07				72		75
	61		0.10				76		78	
TWELVETH AVENUE					1.43	0.85				
		91		0.02				64		70
BELOW 12TH AVE					1.45	0.87				
		101		0.07				76		77
		102		0.03				60		60
	71		0.12				70		72	
CONFLUENCE					1.55	0.97				

In the SCS method the change in S_a (related by CN) is based on antecedent moisture condition determined by the total rainfall in the 5-day period before a storm. Three levels of antecedent moisture condition (AMC) are used (65):

1. I is the lower limit of moisture or the upper limit of S_a.
2. II is the average for which CNs of Table I apply.
3. III is the upper limit of moisture or lower limit of S_a.

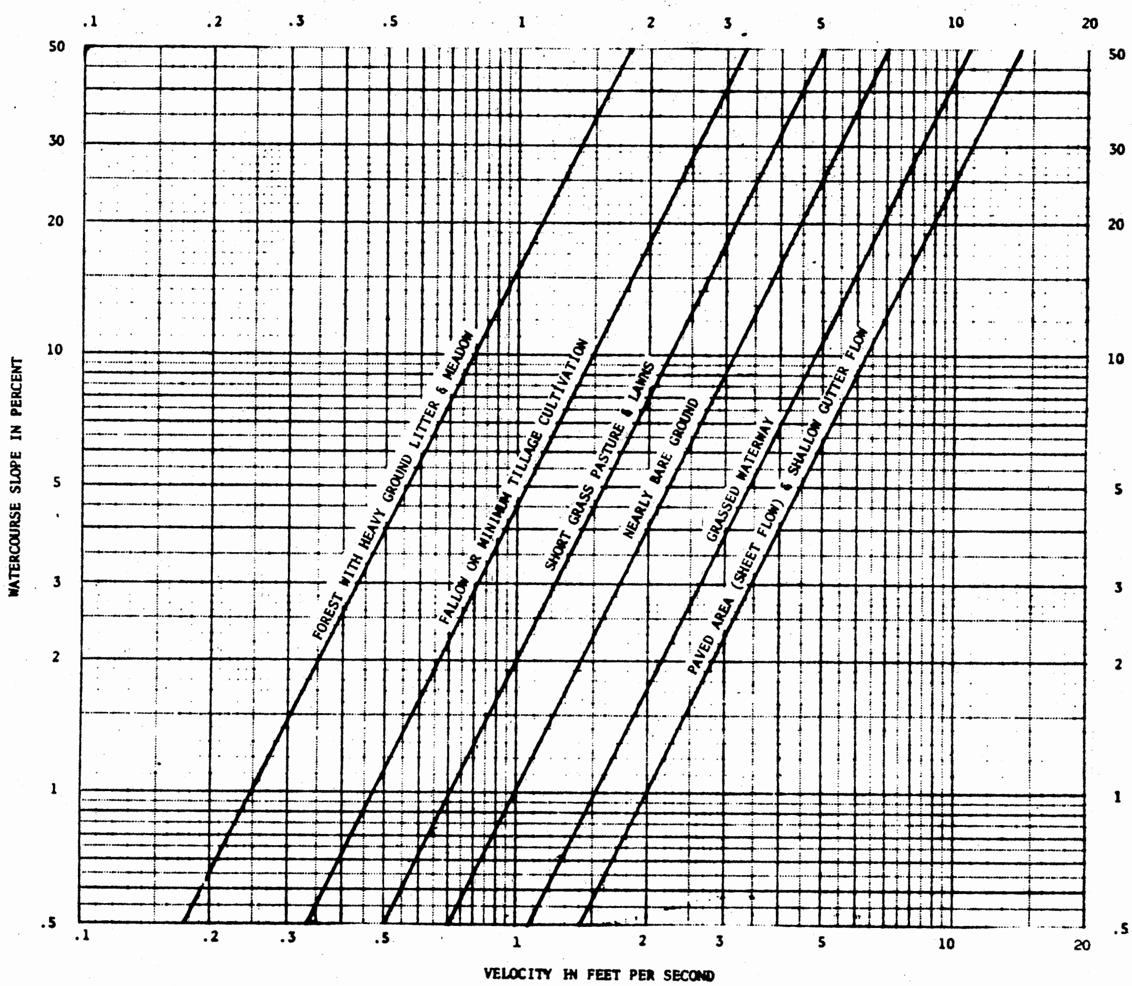
There are conversion tables for obtaining CNs for other antecedent moisture conditions than II (65, 71). TR-20 will automatically convert CNs if I or III are indicated in the input.

Time of Concentration

Time of concentration, t_c, is an important input parameter for the TR-20 model. The travel times for overland flow, storm sewers, and small tributaries are usually lumped in this term.

The t_c was determined for each complex subarea as outlined in Chapter 3, Reference (104). Basically, travel time is computed for the various flow conditions by dividing the length of flow by velocity. Time of concentration is the sum of the travel times for the longest flow path of a basin.

Figure 25 was utilized to determine overland velocities, including paved and shallow gutter flow. Manning's equation was used with available storm sewer data to determine pipe full velocities for storm sewers. In many cases a storm event will generally cause both storm sewer and gutter flow. In this situation an arithmetic mean velocity was used to compute a mean travel time.



Source: U.S. Soil Conservation Service (104), p. 3-2

Figure 25. Average Velocities for Estimating Travel Time for Overland Flow

To compute velocity across ponds or lakes in the flow path, the wave velocity formula was used (65).

Channel Routing

Convex Routing Method

The convex method of routing a hydrograph through stream channels is used by the TR-20 model to account for bank storage (28, 29). A detailed discussion of the procedure is presented in Reference (65). The working equation is:

$$O_2 = (1 - C)\ O_1 + C\ I_1 \tag{4.23}$$

where

I_1 = inflow rate at time increment 1, in cubic feet per second (ft^3/s);

O_1 = outflow rate at time increment 1, in cubic feet per second (ft^3/s);

O_2 = outflow rate at time increment 2, in cubic feet per second (ft^3/s); and

C = routing coefficient.

The routing coefficient is estimated by:

$$C = \frac{V}{V + 1.7} \tag{4.24}$$

where V is the steady-flow water velocity related to the reach travel time for steady-flow discharge, in feet per second (ft/s).

TR-20 contains a routing coefficient table related to increments of V. Reach length and rating curves for cross-sections are input to estimate V.

Local inflows and transmission losses may be incorporated into the routing procedure, but this unnecessarily complicates the working equation (65). It is common practice to add local inflows either to inflow hydrograph or to the routed outflow hydrograph to get total outflows. In this study, local inflows were added to the inflow hydrograph.

Cross-Section Rating Curves

Cross-section rating curves are used with reach lengths in the TR-20 model to perform channel routing. The cross-section information is used to obtain steady-flow velocities for the routing reach. The USCE HEC-2 step-backwater model was used to estimate the cross-section rating curves.

Cross-section geometry was coded in HEC-2 format using the information sources mentioned in the basic data chapter. First the basin was modeled with the present channel. Streets and buildings were included in the overbank geometry, as shown in the example cross-section (Figure 26) using aerial photographs as a guide (24).

The improved channel was then superimposed on the cross-section data, as shown in the example cross-section (Figure 27) using the construction plans as a guide (100).

The basic changes in the stream model for the channel improvement are:

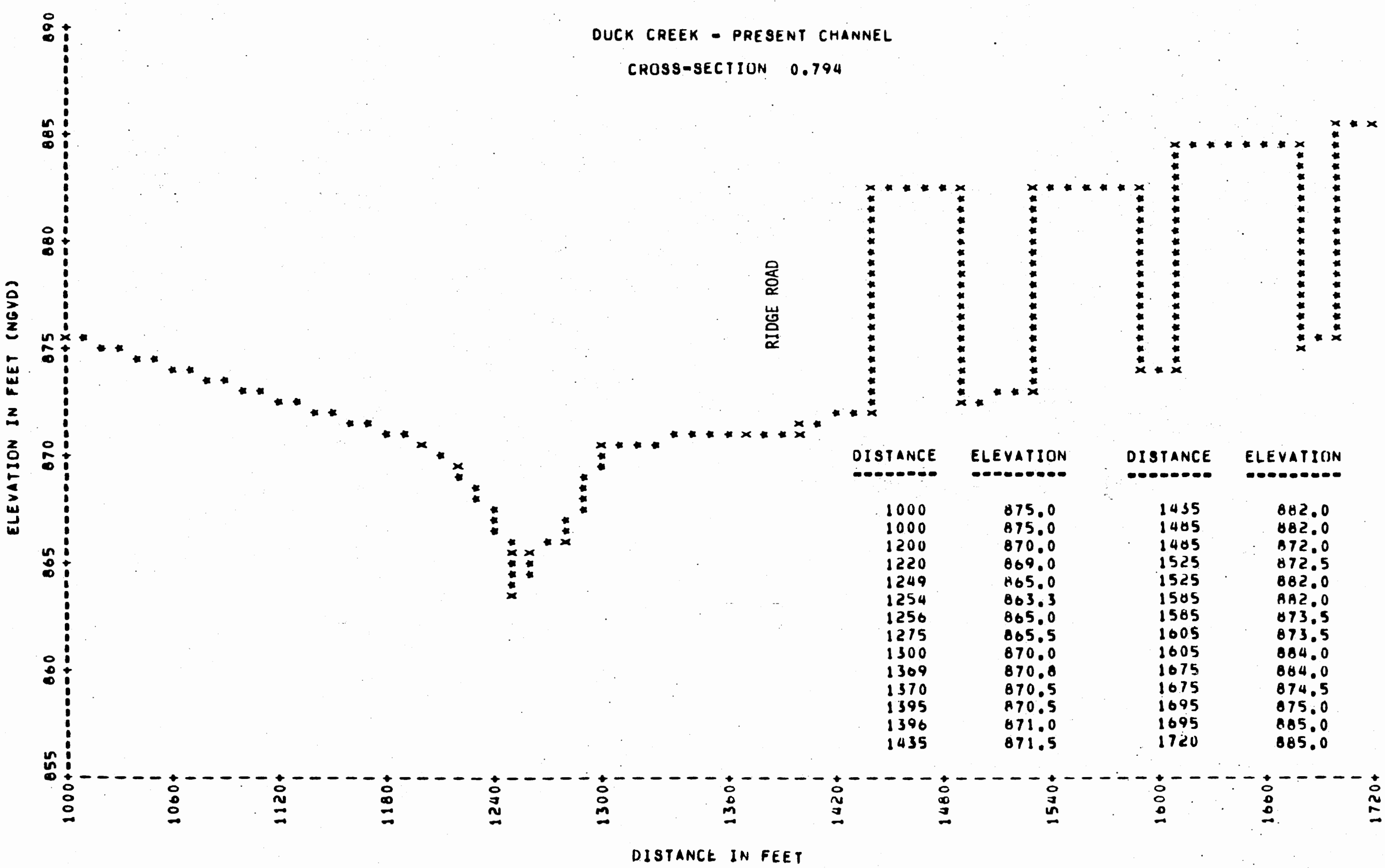
1. Earth channel improvement from the mouth of Duck Creek to 9th Avenue.

2. Concrete channel improvement from 9th Avenue to 6th Avenue.

3. New box culvert at oxbow near 11th Avenue.

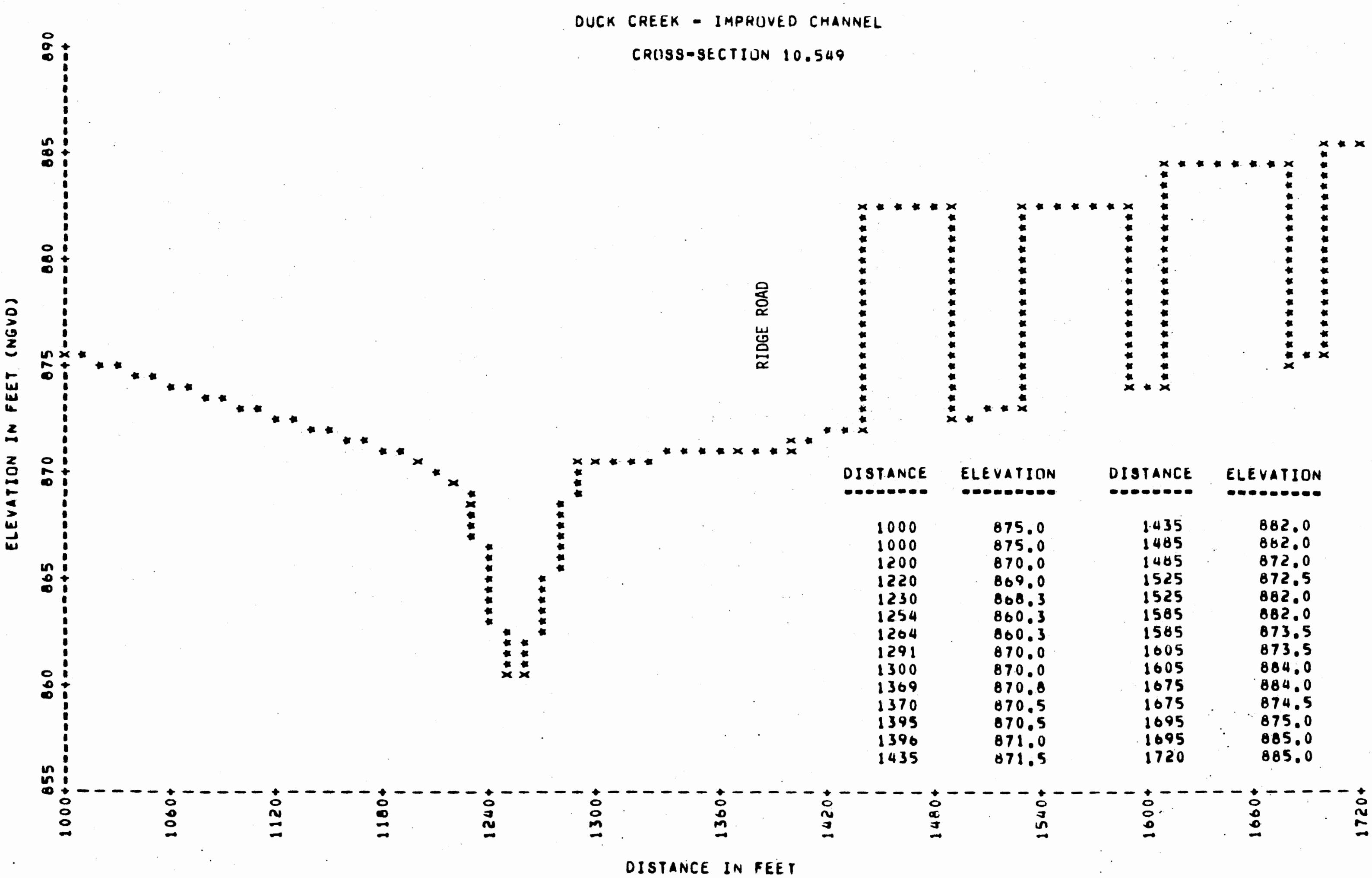
4. New box culvert at 9th Avenue.

5. Channel cleared above 9th Avenue.



DISTANCE	ELEVATION	DISTANCE	ELEVATION
1000	875.0	1435	882.0
1000	875.0	1485	882.0
1200	870.0	1485	872.0
1220	869.0	1525	872.5
1249	865.0	1525	882.0
1254	863.3	1585	882.0
1256	865.0	1585	873.5
1275	865.5	1605	873.5
1300	870.0	1605	884.0
1369	870.8	1675	884.0
1370	870.5	1675	874.5
1395	870.5	1695	875.0
1396	871.0	1695	885.0
1435	871.5	1720	885.0

Figure 26. Example Cross Section for Present Channel



DISTANCE	ELEVATION	DISTANCE	ELEVATION
1000	875.0	1435	882.0
1000	875.0	1485	882.0
1200	870.0	1485	872.0
1220	869.0	1525	872.5
1230	868.3	1525	882.0
1254	860.3	1585	882.0
1264	860.3	1585	873.5
1291	870.0	1605	873.5
1300	870.0	1605	884.0
1369	870.8	1675	884.0
1370	870.5	1675	874.5
1395	870.5	1695	875.0
1396	871.0	1695	885.0
1435	871.5	1720	885.0

Figure 27. Example Cross Section for Improved Channel

The HEC-2 computer program was utilized to route step-backwater water surface profiles up the stream for 14 discharges, 50 ft^3/s to 3,000 ft^3/s. These data were used to compute rating curves for use at control points along the stream. Where the control point is at a street, the first channel cross section downstream of the bridge, or exit section, was used as a rated cross section to help define the routing reach.

Reservoir Routing

Storage-Indication Method

The storage-indication method of routing a hydrograph through a reservoir is used by TR-20 to account for reservoir storage (28, 29). A detailed discussion of the procedure is presented in References (29), (59), (65), and (106). The method uses the continuity equation in the form:

$$\frac{(I_1 + I_2)}{2} + \frac{S_1}{\Delta t} - \frac{O_1}{2} = \frac{S_2}{\Delta t} + \frac{O_2}{2} \qquad (4.25)$$

where

I_2 = inflow rate at time increment 2, in cubic feet per second (ft^3/s);

S_1 = storage volume at time increment 1, in cubic feet (ft^3); and

S_2 = storage volume at time increment 2, in cubic feet (ft^3).

Duck Creek has a floodwater retention structure, SCS Dam 30, just upstream from Hall of Fame Avenue. Therefore, any hydrologic analysis must include reservoir routing through that structure. In this study, the dam was treated as two structures in series, since McElroy Avenue cuts through the storage pool at the upper end and effectively acts as another dam.

Storage Curves

One requirement for reservoir routing is a storage curve (elevation-storage). The storage curve for the entire pond was found in the construction plans (96).

The storage curve for above McElroy was developed by: (1) locating cross sections on the base map, (2) utilizing the cross-section properties feature of the USGS computer program E-431 to obtain cross-section areas, and (3) computing storage volumes using the average end area method. Therefore, the storage curve for the lower portion of the flood retention pool was computed by subtracting the above results from the entire storage curve.

Outlet Works Rating Curves

The second requirement for reservoir routing is the outlet works rating curves.

The rating curve for McElroy Avenue was developed by using the HEC-2 program to route surface water profiles from 9 discharges through the small box culvert and over the road to a point 50 feet upstream of the road.

The main structure was rated by combining the principle spillway (pipe with drop inlet) and emergency spillway data. The rating curve of the principle spillway was constructed using the submerged orifice equation (16, 88). For emergency spillway data, the HEC-2 program was used to route water surface profiles from 12 discharges starting at the top of the emergency spillway, assuming critical flow, to a point 100 feet upstream of the dam.

Watershed Schematic Diagrams

A schematic diagram of the watershed is an important tool for both compiling input data and ensuring that proper hydrologic routing is performed by the model.

The location of structures and cross sections that depict routing-reach terminals are shown numbered in proper sequence. Reach lengths are noted, and for each subarea the drainage area, curve number, and time of concentration are indicated.

A watershed schematic diagram for each of the following alternatives, both simple and complex subarea configuration, was drawn:

1. Present channel-present urbanization
2. Improved channel-present urbanization
3. Present channel-future urbanization
4. Improved channel-future urbanization.

For the first alternative, Figure 28 is the simple subarea configuration watershed schematic diagram, and Figure 29 is the complex subarea configuration watershed schematic diagram. The other channel/urbanization alternative schematic diagrams are shown in Appendix A.

Peak Discharges--Preliminary Run

A preliminary TR-20 run was made for the present channel, present urbanization alternative with complex subarea configuration using the following variables:

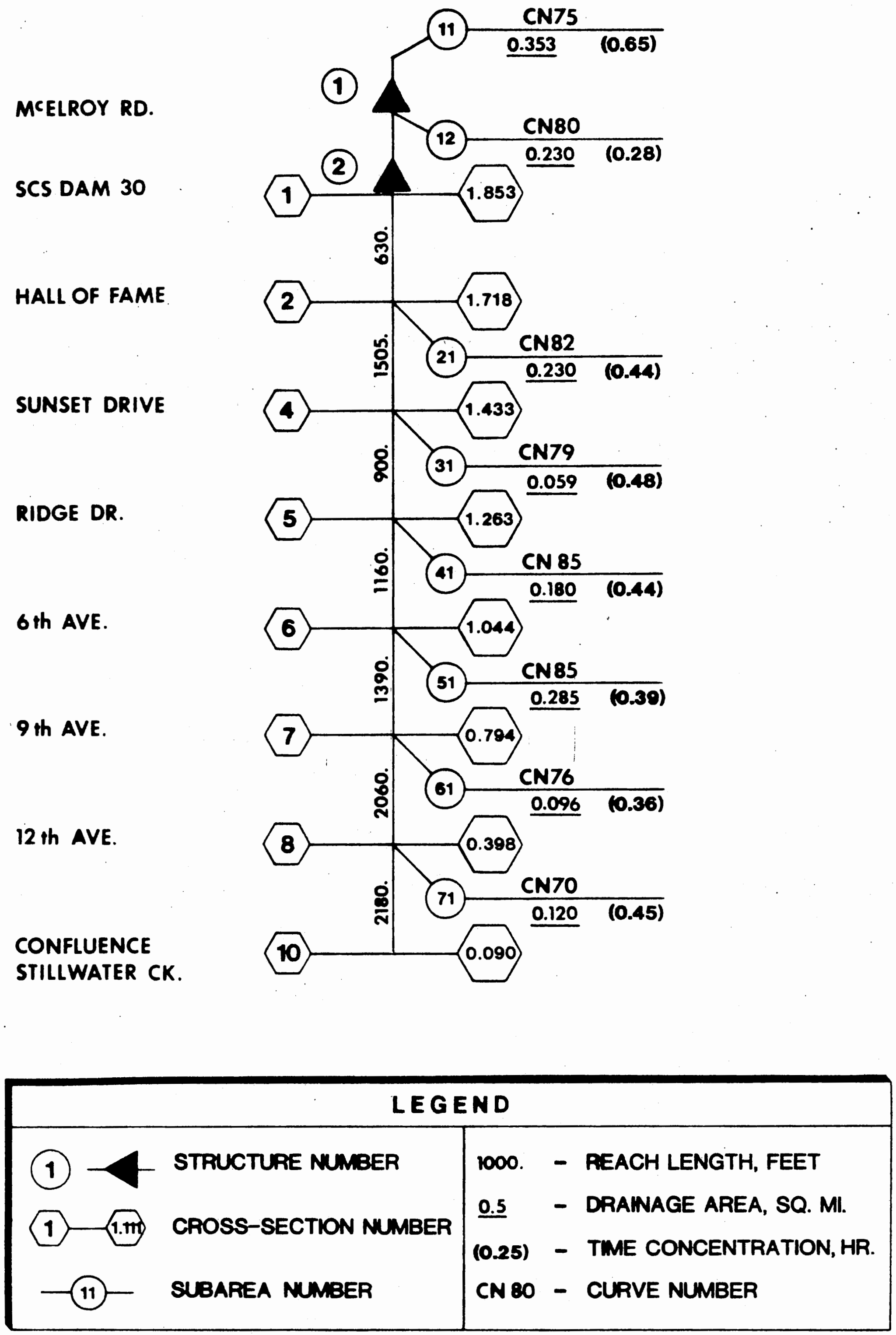


Figure 28. Watershed Schematic Diagram for Present Channel-Present Urbanization, Simple Subarea Configuration

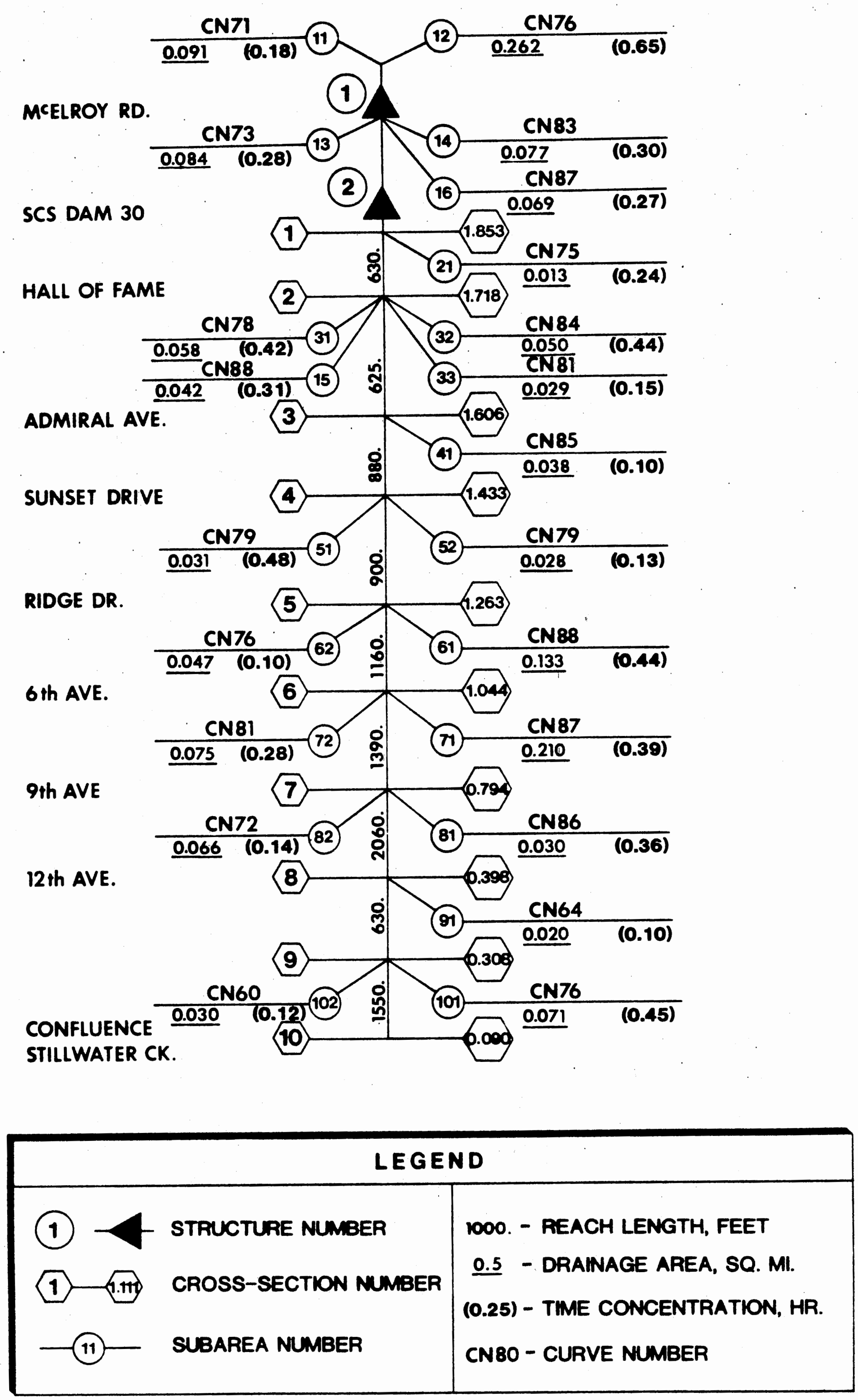


Figure 29. Watershed Schematic Diagram for Present Channel-Present Urbanization, Complex Subarea Configuration

1. Storm frequency
   a. 10-year
   b. 50-year
   c. 100-year
   d. 500-year
2. Antecedent soil moisture condition
   a. II designated as SMC-2
   b. III designated as SMC-3
3. Design storm duration
   a. 1-hour
   b. 3-hour
   c. 6-hour
4. Residential imperviousness (for CN determination)
   a. Assumed to be 20%
   b. Taken from Table I.

The USGS regression equations (103) were also run for comparison purposes. It was assumed that there was no contributing drainage area above Dam 30 for the analysis.

The resultant peak discharges are shown in Table III. The flows are in hydrologic routing order; each discharge represents the flow from just downstream of the streets indicated to just downstream of the next location.

The results were as expected for the AMC variables. The AMC III produced higher peak discharges than AMC II in all cases.

However, the storm duration variables did not produce exactly what was to be expected. It was expected that the shorter storm durations would always produce larger peak discharges than the longer storm

TABLE III

PRELIMINARY DISCHARGE DETERMINATION, PRESENT CHANNEL-PRESENT URBANIZATION

LOCATION	10-YEAR FLOOD (CFS) USGS	10-YEAR SCS TR-20 SMC-2 STORM DURATION 1-HR	10-YEAR SCS TR-20 SMC-2 STORM DURATION 3-HR	10-YEAR SCS TR-20 SMC-2 STORM DURATION 6-HR	10-YEAR SCS TR-20 SMC-3 STORM DURATION 1-HR	10-YEAR SCS TR-20 SMC-3 STORM DURATION 3-HR	10-YEAR SCS TR-20 SMC-3 STORM DURATION 6-HR	50-YEAR FLOOD (CFS) USGS	50-YEAR SCS TR-20 SMC-2 STORM DURATION 1-HR	50-YEAR SCS TR-20 SMC-2 STORM DURATION 3-HR	50-YEAR SCS TR-20 SMC-2 STORM DURATION 6-HR	50-YEAR SCS TR-20 SMC-3 STORM DURATION 1-HR	50-YEAR SCS TR-20 SMC-3 STORM DURATION 3-HR	50-YEAR SCS TR-20 SMC-3 STORM DURATION 6-HR
RESIDENTIAL IMPERVIOUSNESS ASSUMED TO BE 20%														
ADMIRAL AVE	374	281	272	297	474	435	416	569	435	410	422	659	593	542
RIDGE DR	621	539	517	552	897	822	787	920	836	779	795	1244	1113	1036
SIXTH AVE	869	851	826	886	1467	1340	1265	1284	1346	1271	1287	2066	1834	1669
NINTH AVE	901	899	865	927	1565	1424	1344	1357	1426	1341	1357	2219	1961	1792
CONFLUENCE	954	916	886	947	1636	1481	1406	1414	1481	1385	1403	2326	2054	1890
RESIDENTIAL IMPERVIOUSNESS TAKEN FROM TR-55 TABLES														
ADMIRAL AVE	397	289	278	305	479	440	419	597	444	418	431	665	600	546
RIDGE DR	672	577	549	578	919	840	801	980	878	820	827	1271	1132	1049
SIXTH AVE	939	965	914	954	1542	1391	1295	1366	1469	1373	1361	2145	1886	1698
NINTH AVE	973	1023	963	1004	1654	1484	1382	1412	1568	1461	1445	2314	2024	1830
CONFLUENCE	1006	1051	988	1031	1730	1546	1449	1476	1634	1511	1500	2429	2127	1934

TABLE III (Continued)

LOCATION	100-YEAR FLOOD (CFS)							500-YEAR FLOOD (CFS)						
	USGS	SCS TR-20						USGS	SCS TR-20					
		SMC-2			SMC-3				SMC-2			SMC-3		
		STORM DURATION			STORM DURATION				STORM DURATION			STORM DURATION		
		1-HR	3-HR	6-HR	1-HR	3-HR	6-HR		1-HR	3-HR	6-HR	1-HR	3-HR	6-HR
RESIDENTIAL IMPERVIOUSNESS ASSUMED TO BE 20%														
ADMIRAL AVE	662	506	467	470	741	657	589	890	589	559	564	834	755	684
RIDGE DR	1062	970	892	887	1399	1231	1128	1409	1126	1068	1069	1579	1411	1313
SIXTH AVE	1483	1575	1458	1439	2331	2038	1820	1967	1841	1764	1741	2632	2347	2121
NINTH AVE	1532	1671	1543	1522	2508	2184	1960	2030	1959	1878	1852	2837	2519	2299
CONFLUENCE	1637	1742	1598	1579	2635	2292	2073	2178	2053	1952	1933	2980	2649	2437
RESIDENTIAL IMPERVIOUSNESS TAKEN FROM TR-55 TABLES														
ADMIRAL AVE	690	516	477	478	747	664	593	921	599	569	573	840	762	687
RIDGE DR	1126	1014	934	919	1427	1250	1141	1478	1173	1108	1105	1606	1430	1325
SIXTH AVE	1569	1701	1564	1514	2412	2091	1848	2059	1974	1859	1817	2715	2398	2148
NINTH AVE	1622	1824	1669	1612	2604	2245	1998	2127	2126	1991	1947	2936	2580	2335
CONFLUENCE	1701	1906	1731	1678	2735	2360	2116	2248	2225	2074	2038	3079	2716	2479

durations, since the rainfall intensity, depth per hour, is greater for the shorter duration storms. This is exactly what happened in all cases for the AMC III, where the watershed storage characteristics are essentially eliminated due to the high antecedent moisture in the soil-cover complex.

However, for AMC II, the smaller frequency storms, 10- and 50-year, did not follow this trend. The 3-hour storm frequently produced a lower peak than the 6-hour storm, and the magnitudes were very similar. The 100-year and 500-year peak discharges generally followed the expected pattern, but again the 3- and 6-hour magnitudes were very close. This probably is because at the 10- and 50-year storms the storage characteristics of the soil-cover complex were still able to cope with differing intensities and total amount was still overriding the intensity differences between the 3- and 6-hour storms. At the higher frequency storms, the watershed was receiving so much rain that the storage characteristics of the soil-cover complex was "overwhelmed" and intensity differences had more of an effect.

The results were as expected for assuming 20% residential imperviousness and Table I values. The Table I values yielded a higher peak discharge in all cases, both in the USGS and TR-20 methods. Since these values were used for the residential lot only, and streets were computed separately, there was not a large difference between the resultant discharges. Therefore, the 20% value could be used for a quick estimate if necessary.

The variables chosen for the final peak discharge analysis were:

1. AMC II;
2. Storm duration of 6 hours; and

3. Residential imperviousness values taken from Table I.

Peak Discharges--Final Run

The final peak discharge determination TR-20 run was made for the previously noted frequencies for the following alternatives:

1. Watershed urbanization
   a. Present
   b. Future
2. Channel condition
   a. Present
   b. Improved
3. Subarea configuration
   a. Simple
   b. Complex.

The final USGS regression equation calculations compared for the two urbanization conditions: (1) present, and (2) future.

The resultant peak discharges are shown in Table IV for present urbanization, and Table V for future urbanization. Again, the flows are listed in hydrologic routing order. A few observations are readily apparent from these two tables.

The USGS method does not have the capacity for reservoir routing; therefore, it was assumed that there was no contributing drainage area above Dam 30, not a bad assumption for this study since the highest flow released by the dam was less than 60 ft^3/s. Second, that method does not have the capacity to assess channel improvements; therefore, there is no real comparison with the TR-20 channel improvement alternatives. However, comparing the present channel results, it can be seen that the USGS method

TABLE IV

PEAK DISCHARGE DETERMINATION, PRESENT URBANIZATION

LOCATION	10-YEAR FLOOD (CFS)					50-YEAR FLOOD (CFS)				
	USGS	SCS TR-20				USGS	SCS TR-20			
		CHANNEL CONDITION					CHANNEL CONDITION			
		PRESENT		IMPROVED			PRESENT		IMPROVED	
		SUBAREA CONFIGURATION					SUBAREA CONFIGURATION			
		SIMPLE	COMPLEX	SIMPLE	COMPLEX		SIMPLE	COMPLEX	SIMPLE	COMPLEX
ABOVE MCELROY	****	261	247	261	247	****	400	381	400	381
MCELROY AVE	****	157	156	157	156	****	219	217	219	217
ABOVE DAM 30	****	352	378	352	378	****	515	547	515	547
BELOW DAM 30	30	37	39	37	39	52	41	48	41	48
HALL OF FAME	342	300	262	300	262	522	419	367	419	367
ADMIRAL AVE	397	****	304	****	305	597	****	431	****	432
SUNSET DR	426	355	359	355	361	649	502	513	503	516
RIDGE ROAD	672	564	577	566	580	980	802	822	806	828
SIXTH AVE	939	882	949	899	965	1366	1269	1355	1263	1351
NINTH AVE	973	939	1000	980	1040	1412	1367	1438	1384	1461
TWELVETH AVE	979	974	984	1050	1041	1421	1432	1721	1498	1464
BELOW 12TH AVE	****	****	1029	****	1108	****	****	1496	****	1566
CONFLUENCE	1006	970	1026	1046	1104	1476	1427	1492	1492	1563

NOTE: **** INDICATES DISCHARGE VALUE NOT COMPUTED AT THIS LOCATION

TABLE IV (Continued)

LOCATION	100-YEAR FLOOD (CFS)					500-YEAR FLOOD (CFS)				
	USGS	SCS TR-20				USGS	SCS TR-20			
		CHANNEL CONDITION					CHANNEL CONDITION			
		PRESENT		IMPROVED			PRESENT		IMPROVED	
		SUBAREA CONFIGURATION					SUBAREA CONFIGURATION			
		SIMPLE	COMPLEX	SIMPLE	COMPLEX		SIMPLE	COMPLEX	SIMPLE	COMPLEX
ABOVE MCELROY	****	455	434	455	434	****	566	546	566	546
MCELROY AVE	****	230	229	230	229	****	256	255	256	255
ABOVE DAM 30	****	576	612	576	612	****	702	744	702	744
BELOW DAM 30	61	43	51	43	51	85	45	58	45	58
HALL OF FAME	607	465	406	465	406	815	556	485	555	485
ADMIRAL AVE	690	****	483	****	479	921	****	572	****	573
SUNSET DR	755	557	579	558	572	1016	667	683	668	686
RIDGE ROAD	1126	889	930	894	920	1478	1062	1095	1069	1104
SIXTH AVE	1569	1411	1526	1398	1496	2059	1692	1806	1666	1788
NINTH AVE	1622	1525	1623	1534	1619	2127	1840	1933	1832	1933
TWELVETH AVE	1631	1604	1606	1666	1624	2139	1947	1915	2000	1941
BELOW TWELVETH	****	****	1695	****	1740	9999	****	2029	****	2087
CONFLUENCE	1701	1599	1690	1660	1737	2248	1941	2024	1994	2084

NOTE: **** INDICATES DISCHARGE VALUE NOT COMPUTED AT THIS LOCATION

TABLE V

PEAK DISCHARGE DETERMINATION, FUTURE URBANIZATION

LOCATION	10-YEAR FLOOD (CFS)					50-YEAR FLOOD (CFS)				
	USGS	SCS TR-20				USGS	SCS TR-20			
		CHANNEL CONDITION					CHANNEL CONDITION			
		PRESENT		IMPROVED			PRESENT		IMPROVED	
		SUBAREA CONFIGURATION					SUBAREA CONFIGURATION			
		SIMPLE	COMPLEX	SIMPLE	COMPLEX		SIMPLE	COMPLEX	SIMPLE	COMPLEX
ABOVE MCELROY	****	462	468	462	468	****	656	662	656	662
MCELROY AVE	****	214	215	214	215	****	251	251	251	251
ABOVE DAM 30	****	501	509	501	509	****	683	686	683	686
BELOW DAM 30	30	40	42	40	42	52	43	50	43	50
HALL OF FAME	395	334	299	334	299	584	456	407	456	407
ADMIRAL AVE	443	****	349	****	350	651	****	481	****	479
SUNSET DR	492	383	419	383	421	727	525	588	528	581
RIDGE ROAD	706	610	646	614	651	1020	847	916	852	908
SIXTH AVE	1032	964	1037	975	1046	1474	1350	1469	1343	1445
NINTH AVE	1070	1029	1091	1065	1127	1526	1461	1562	1476	1563
TWELVETH AVE	1077	1062	1074	1142	1129	1535	1527	1545	1599	1568
BELOW 12TH AVE	****	****	1119	****	1197	****	****	1620	****	1673
CONFLUENCE	1085	1059	1115	1137	1194	1568	1521	1615	1593	1670

NOTE: **** INDICATES DISCHARGE VALUE NOT COMPUTED AT THIS LOCATION

TABLE V (Continued)

LOCATION	100-YEAR FLOOD (CFS)					500-YEAR FLOOD (CFS)				
	USGS	SCS TR-20				USGS	SCS TR-20			
		CHANNEL CONDITION					CHANNEL CONDITION			
		PRESENT		IMPROVED			PRESENT		IMPROVED	
		SUBAREA CONFIGURATION					SUBAREA CONFIGURATION			
		SIMPLE	COMPLEX	SIMPLE	COMPLEX		SIMPLE	COMPLEX	SIMPLE	COMPLEX
ABOVE MCELROY	****	731	737	731	737	****	880	890	880	890
MCELROY AVE	****	266	266	266	266	****	295	295	295	295
ABOVE DAM 30	****	747	745	747	745	****	865	862	865	862
BELOW DAM 30	61	44	53	44	53	85	47	60	47	60
HALL OF FAME	672	502	448	502	448	887	595	530	595	530
ADMIRAL AVE	748	****	527	****	527	983	****	625	****	625
SUNSET DR	837	575	639	579	641	1105	678	758	683	762
RIDGE ROAD	1168	933	997	939	1003	1524	1103	1187	1113	1195
SIXTH AVE	1683	1491	1607	1478	1595	2183	1771	1913	1752	1897
NINTH AVE	1741	1620	1713	1627	1726	2257	1938	2074	1931	2052
TWELVETH AVE	1751	1698	1696	1769	1732	2270	2044	2053	2110	2061
BELOW 12TH AVE	****	****	1782	****	1852	****	****	2173	****	2211
CONFLUENCE	1799	1693	1777	1763	1849	2353	2039	2167	2104	2206

NOTE: **** INDICATES DISCHARGE VALUE NOT COMPUTED AT THIS LOCATION

also differs significantly with the TR-20 estimates when the watershed shape is not "uniform"; i.e., when the subareas have a t_c greater than the main channel. Until 6th Avenue, the USGS method's estimates are significantly higher than the TR-20 estimates. At 6th Avenue and to the confluence, there is a remarkable similarity in the discharge estimates, except for the 500-year flood, as the watershed shape becomes more uniform.

Comparing the simple with the complex subarea configuration estimates, it appears that the simple subarea configuration estimates were lower, but relatively close to the estimates obtained from the complex configuration estimates.

It can be seen that the structure on the upper end of the watershed effectively negates the effect of the future urbanization above it. This is a good example of the value of having a reservoir routing option available in the hydrologic model.

Some classic effects of channel improvement can be seen in the hydrograph plots taken at the mouth of Duck Creek plotted taking all four channel/urbanization alternatives for each frequency flood (Figures 30 through 33). The channel improvement reduces the lag time and increases the peak flow for a given urbanization alternative. Also note the large secondary peak that appears on the recession side of the hydrograph indicating a nonuniformity in the watershed shape. This is probably due to the large subareas coming in at Ridge Road and 6th Avenue.

Hydrograph plots plotted taking all four frequency floods for a given channel/urbanization alternative are presented in Appendix B.

The peak flood flows used in the hydraulic phase of this study are presented in Table VI. Note the flows are now listed in hydraulic routing

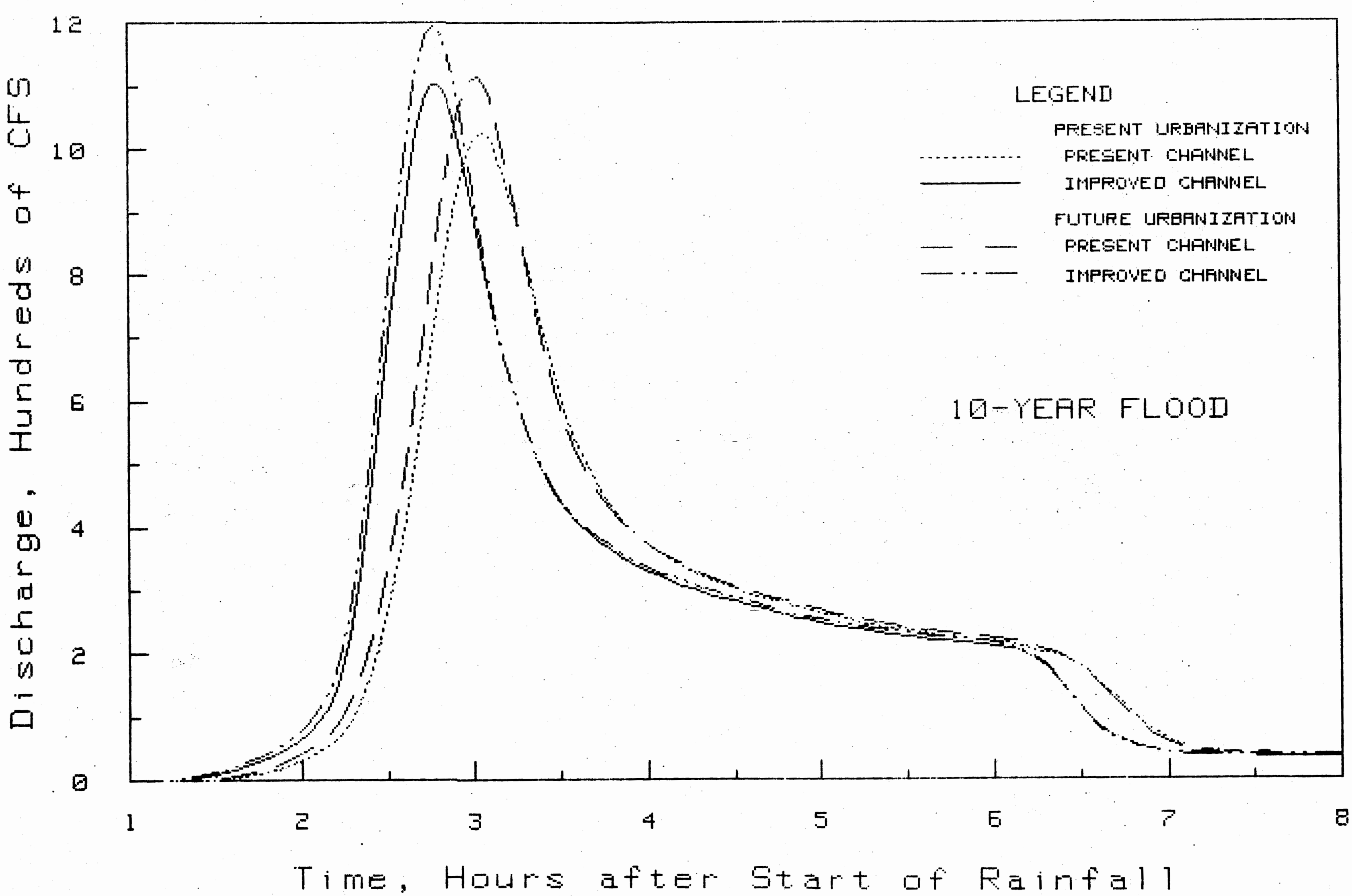


Figure 30. 10-Year Flood Hydrographs at the Mouth of Duck Creek

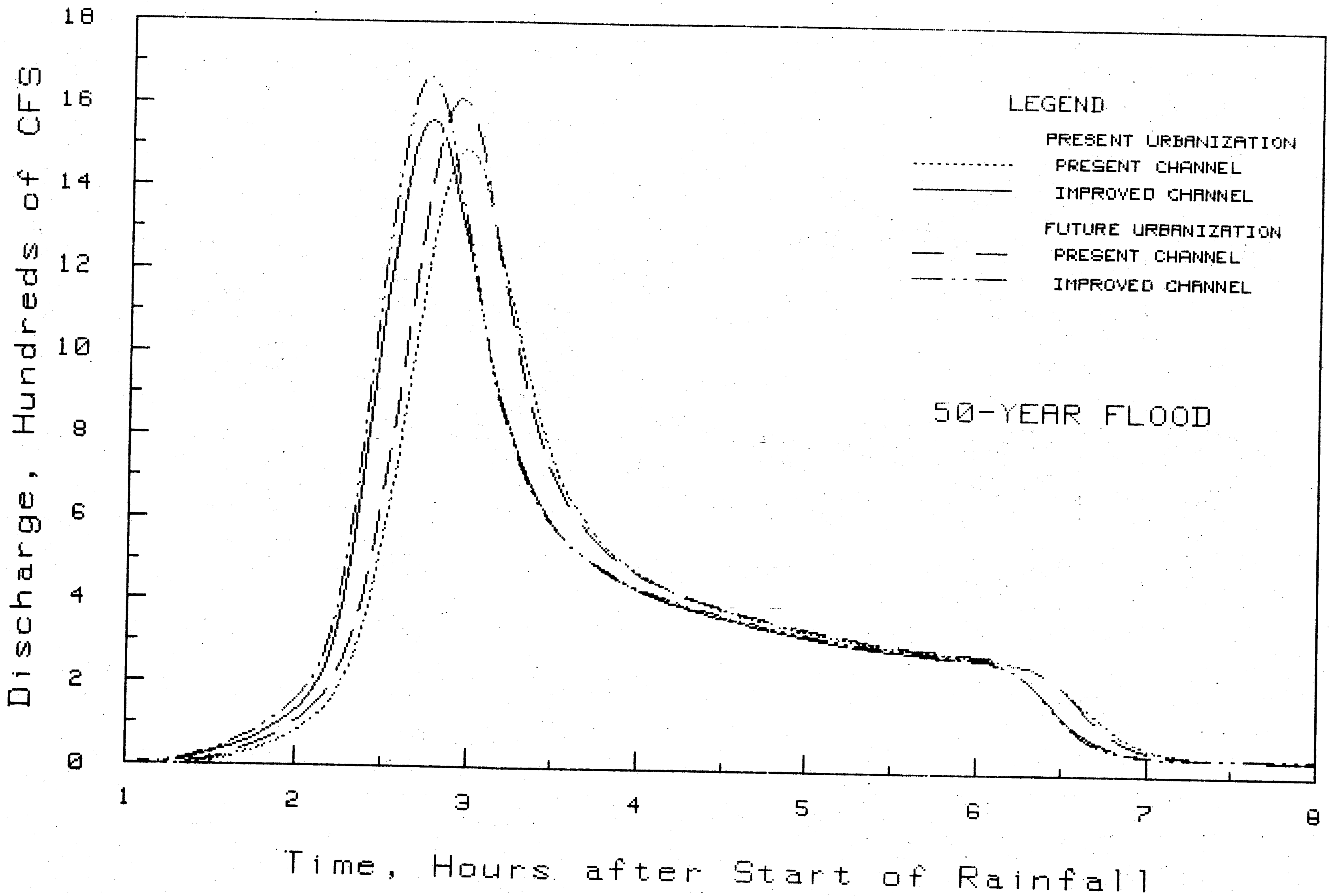


Figure 31. 50-Year Flood Hydrographs at the Mouth of Duck Creek

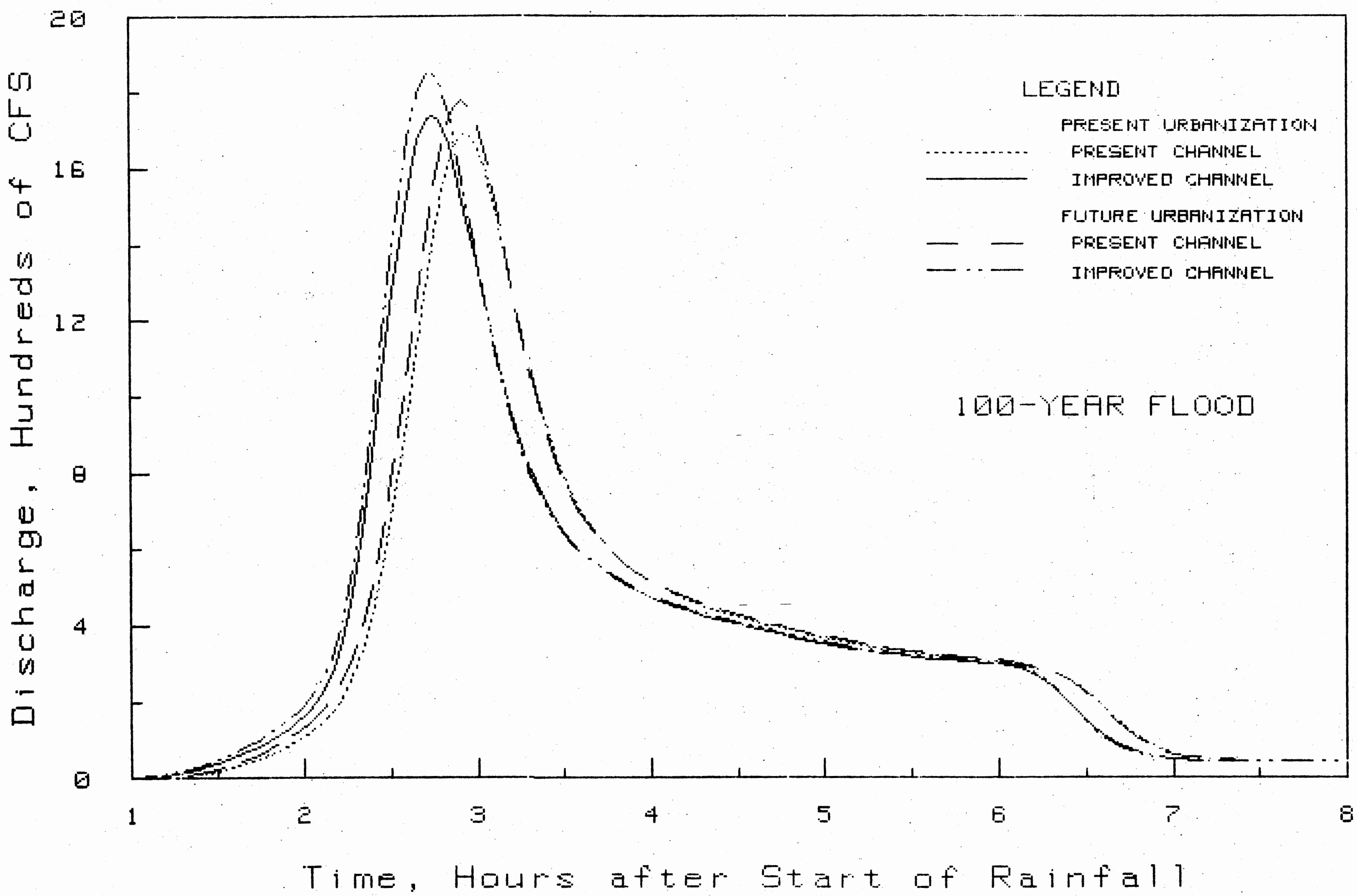


Figure 32. 100-Year Flood Hydrographs at the Mouth of Duck Creek

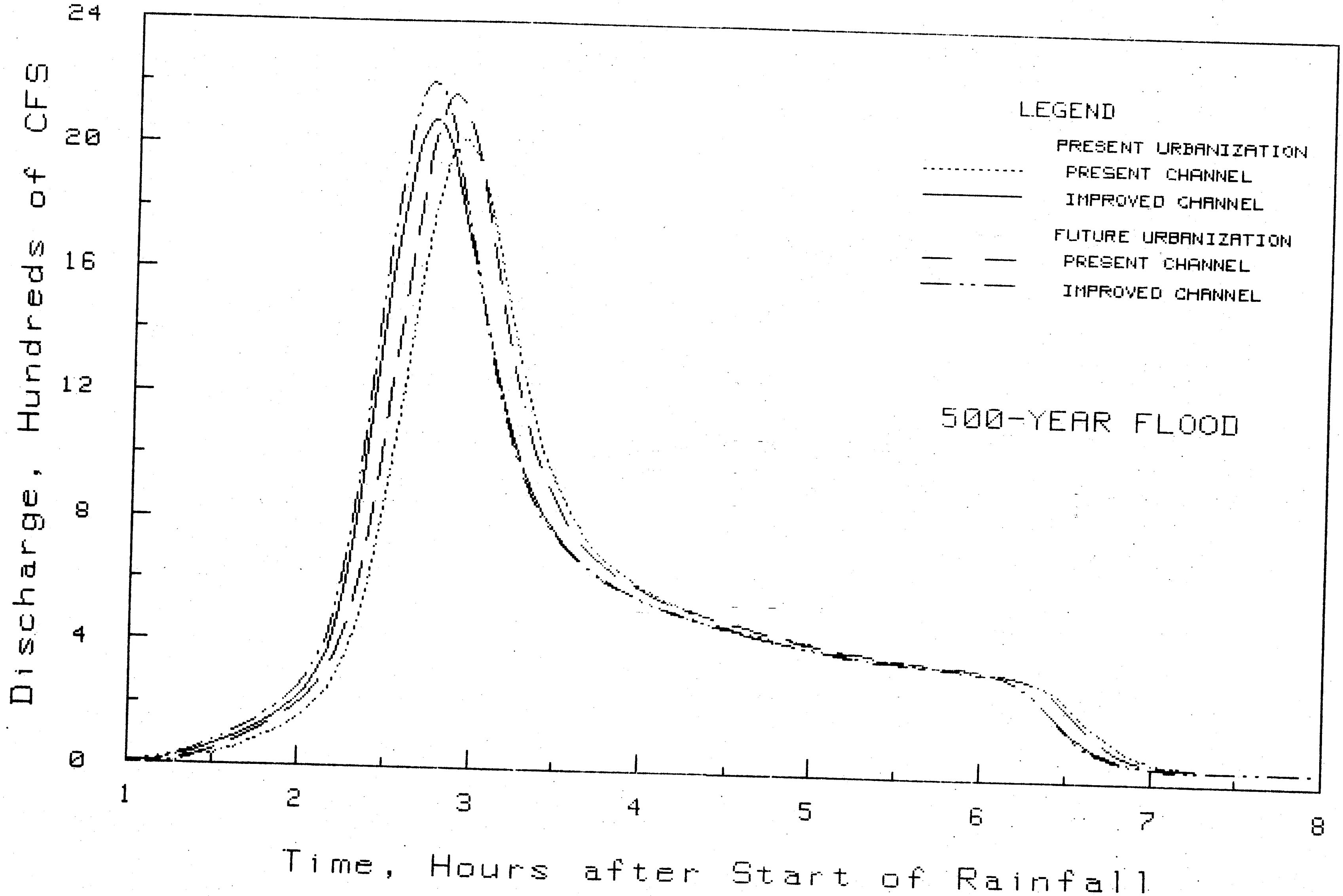


Figure 33. 500-Year Flood Hydrographs at the Mouth of Duck Creek

TABLE VI

PEAK DISCHARGE UTILIZED FOR FLOOD STUDY ANALYSIS

LOCATION	10-YEAR FLOOD (CFS)				50-YEAR FLOOD (CFS)			
	URBANIZATION: PRESENT		URBANIZATION: FUTURE		URBANIZATION: PRESENT		URBANIZATION: FUTURE	
	CHANNEL CONDITION: PRESENT	IMPROVED	PRESENT	IMPROVED	PRESENT	IMPROVED	PRESENT	IMPROVED
CONFLUENCE	1030	1110	1120	1195	1495	1565	1620	1675
BELOW 12TH AVE	1000	1040	1090	1130	1440	1465	1560	1570
NINTH AVE	950	965	1035	1045	1355	1350	1470	1445
SIXTH AVE	575	580	645	650	820	830	915	910
ABOVE RIDGE RD	360	360	420	420	515	515	590	580
SUNSET DR	305	305	350	350	430	430	480	480
ADMIRAL AVE	260	260	300	300	365	365	405	405
HALL OF FAME	40	40	40	40	50	50	50	50

TABLE VI (Continued)

LOCATION	100-YEAR FLOOD (CFS)				500-YEAR FLOOD (CFS)			
	URBANIZATION				URBANIZATION			
	PRESENT		FUTURE		PRESENT		FUTURE	
	CHANNEL CONDITION				CHANNEL CONDITION			
	PRESENT	IMPROVED	PRESENT	IMPROVED	PRESENT	IMPROVED	PRESENT	IMPROVED
CONFLUENCE	1695	1740	1780	1850	2030	2085	2175	2210
BELOW 12TH AVE	1620	1625	1715	1730	1935	1940	2075	2060
NINTH AVE	1525	1495	1605	1595	1805	1790	1915	1895
SIXTH AVE	930	920	995	1005	1095	1105	1190	1195
ABOVE RIDGE RD	580	570	640	640	685	685	760	760
SUNSET DR	485	480	525	525	570	575	625	625
ADMIRAL AVE	405	405	450	450	485	485	530	530
HALL OF FAME	50	50	55	55	60	60	60	60

order so that a flow starts just downstream of a street location to just downstream of the next upstream location. These are the complex subarea configurations of the TR-20 results rounded to the nearest five ft^3/s.

CHAPTER V

HYDRAULICS

Introduction

The next phase of the investigation was to perform the hydraulic analyses which are normally required in a flood insurance study (44). Once the flood discharges were determined, the following analyses were completed: (1) flood elevation determination, (2) floodway determination, and (3) flood hazard determination.

Flood Elevation Determination

The USCE computer program HEC-2, Water Surface Profiles, was the step-backwater model used for the investigation (57). Utilizing the flood peak discharges in Table VI, flood elevations of the 10-, 50-, 100-, and 500-year floods were determined in the Duck Creek basin for the following alternatives:

1. Present channel-present urbanization
2. Improved channel-present urbanization
3. Present channel-future urbanization
4. Improved channel-future urbanization.

All cross sections where the streets and buildings were parallel to the flow path were coded as in the example cross sections, Figures 26 and 27, to give a better definition of conveyance and flood boundaries on the overbank areas than just an average "n" value would provide. Profile stationing of the cross sections was obtained by using the apparent centroid of flow along an inundated flood plain. Profile distances were measured along this line. The starting elevation for each flood flow was determined by the slope-area method option in HEC-2 (57), since it is highly improbable that coincident floods on Duck Creek and Stillwater Creek are likely. The starting elevations were checked using the slope-conveyance method recommended for the USGS's E-431 model (91).

The major differences in the Duck Creek stream model used for the channel improvement versus the present channel were: (1) two new bridge structures, (2) channel slope, (3) channel roughness values, and (4) reach lengths between cross sections.

The concrete channel improvement reach between 6th Avenue and 9th Avenue was determined to be in the supercritical flow regime by using the Section Factor method in Chow (20) to determine critical slope and comparing the results with the proposed slope of that reach. However, both subcritical and supercritical water surface profiles for four test discharges (500, 1000, 1500, and 2500 ft^3/s) were run to see which flow regime produced the most reasonable results. Profile plots of the water surface elevations and energy grade lines were drawn and it was decided that the subcritical run was the most reasonable due to a large adverse slope portion in the middle of the reach for the supercritical water surface profiles. Therefore, the subcritical flow regime was used in all rating curves and flood elevation determinations.

All flood profiles were smoothed where dips in the water surface profiles occurred, generally at the approach section to bridges. Elevations from the upstream side of the bridge were input at the approach section and the profile restarted.

The final water surface profiles are presented in two forms, with bridge elevations omitted for clarity and brevity: (1) profile plots, and (2) summary tables. Both forms of data presentation are organized in two ways: (1) comparison of the 10-, 50-, 100-, and 500-year flood elevations for a given channel/urbanization alternative; and (2) comparison of the four channel/urbanization alternatives for a given recurrence interval flood.

The water surface profile plots comparing the four floods for: (1) present channel, present urbanization; and (2) improved channel, present urbanization are presented in Appendix C. The future urbanization profile plots have been omitted for the sake of brevity since those plots are very similar to the present urbanization alternatives. The profile summary tables for all four channel/urbanization alternatives are presented in Appendix D.

The water surface profile plots comparing the four channel/urbanization alternatives for the 10-year flood are presented in Figure 34, and for the 100-year flood are presented in Figure 35. The water surface elevation comparison tables for each flood are presented in Appendix E.

Note that although there is no coincident flooding from Stillwater Creek, there is backwater flooding that must be considered in flood boundary determination, flood hazard determinations, and flood damages. Only the flood damage chapter will consider these elevations and these elevations are not shown in flood profile plots or tables except in Figures 34 and 35.

The backwater flood elevations, in the National Geodetic Vertical Datum (NGVD) of 1929 (formerly called the Sea Level Datum of 1929), from Stillwater Creek are (18):

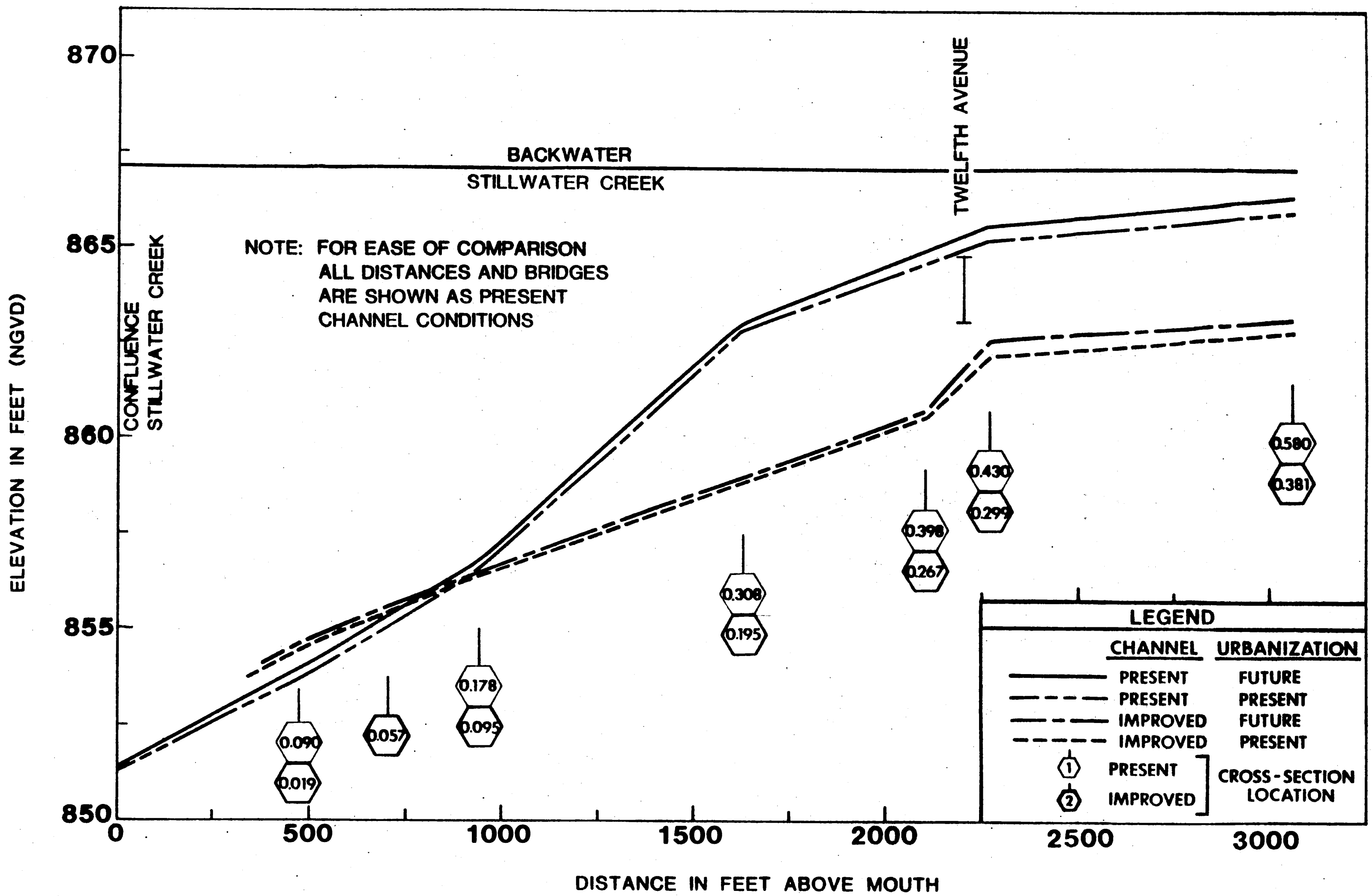


Figure 34. Comparison of 10-Year Flood Water Surface Profiles

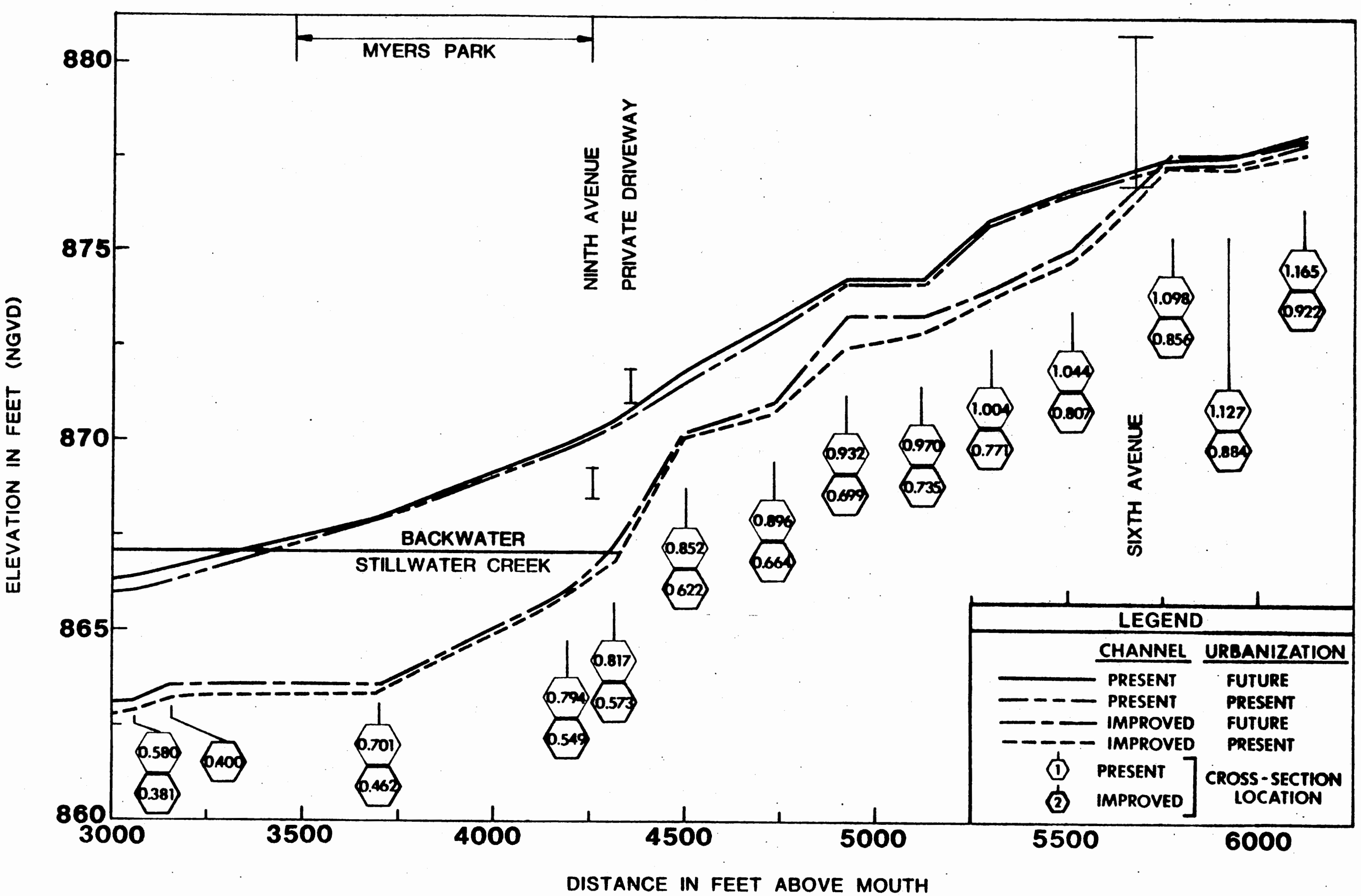


Figure 34. (Continued)

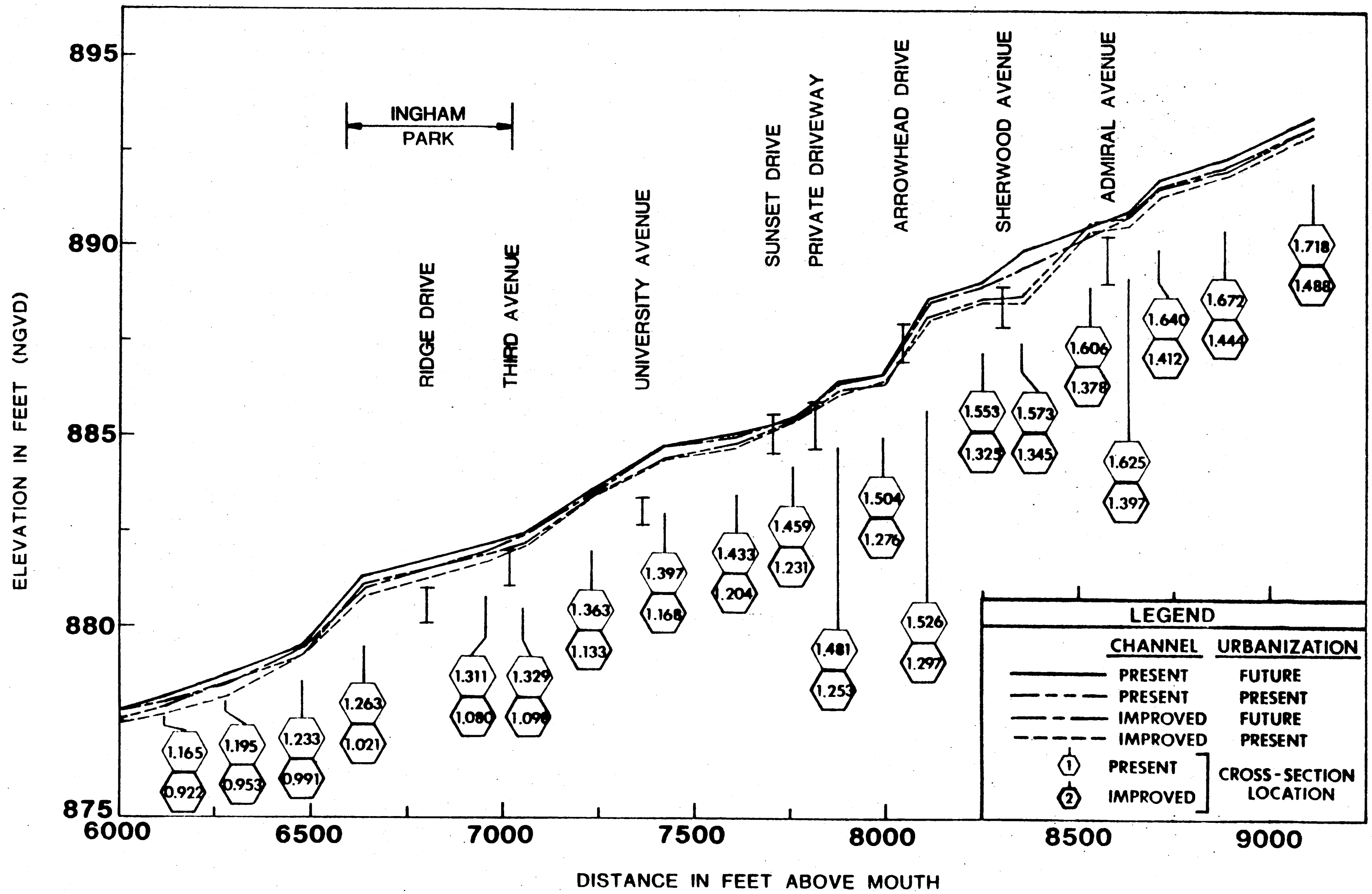


Figure 34. (Continued)

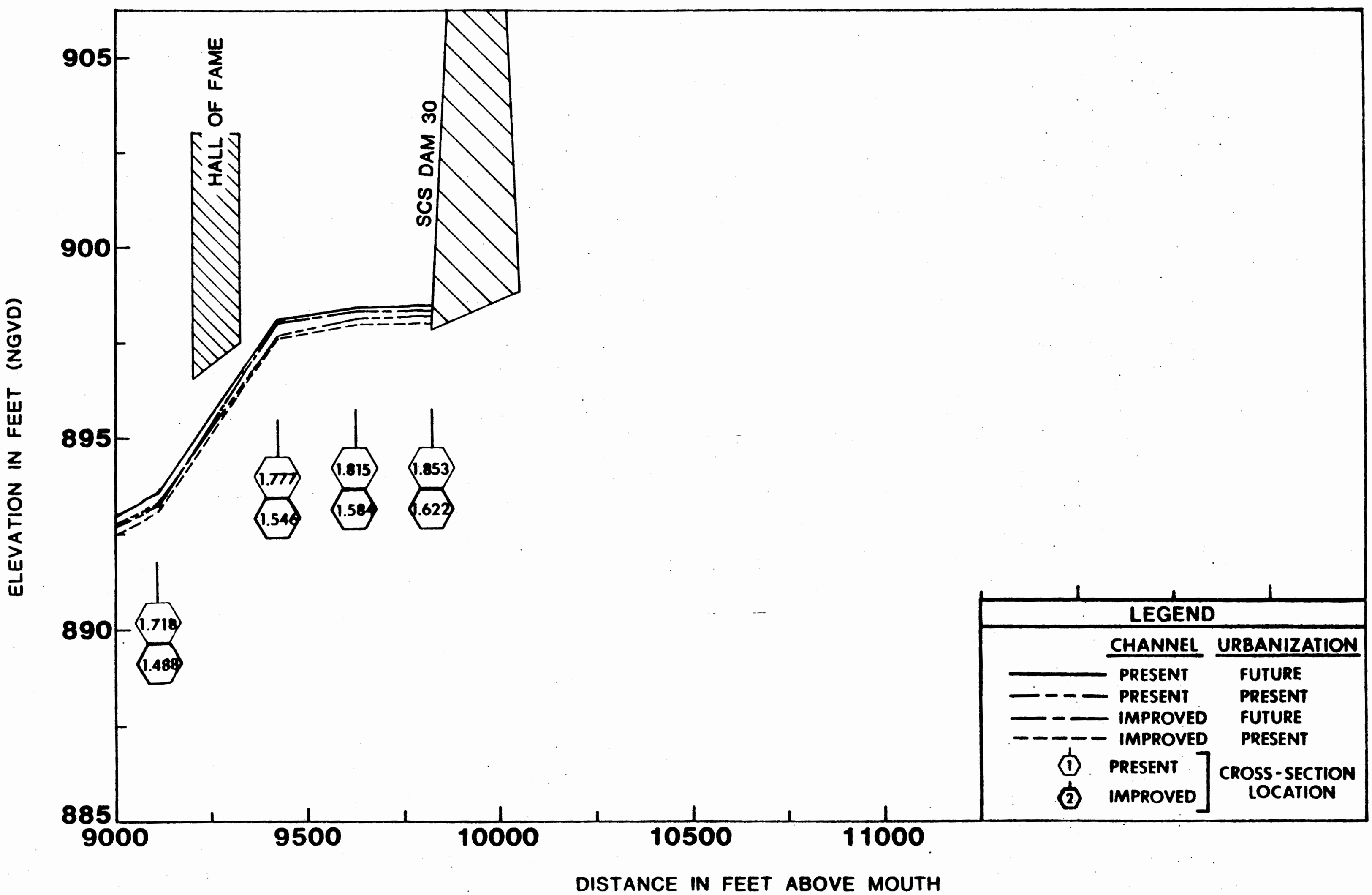


Figure 34. (Continued)

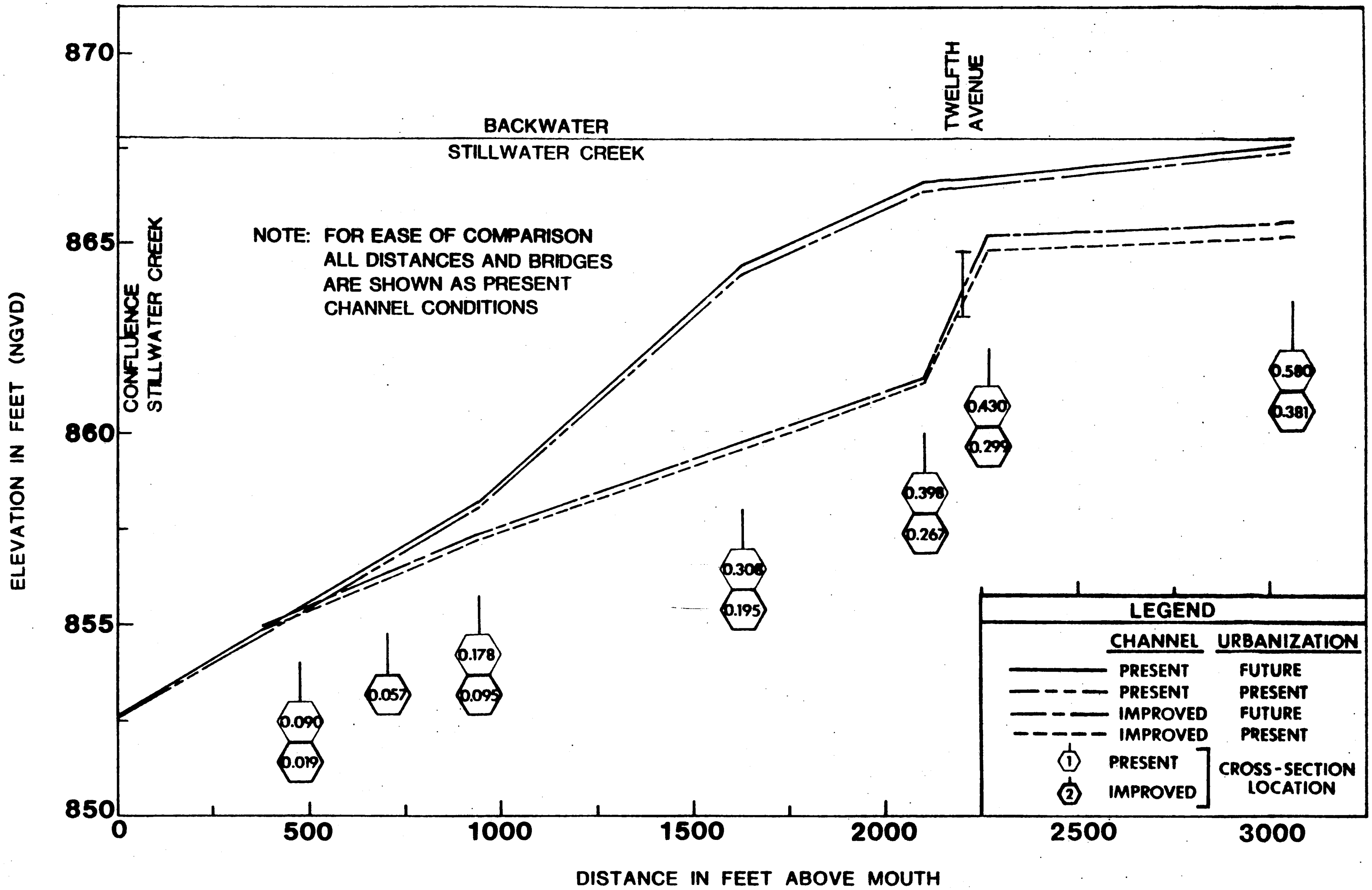


Figure 35. Comparison of 100-Year Flood Water Surface Profiles

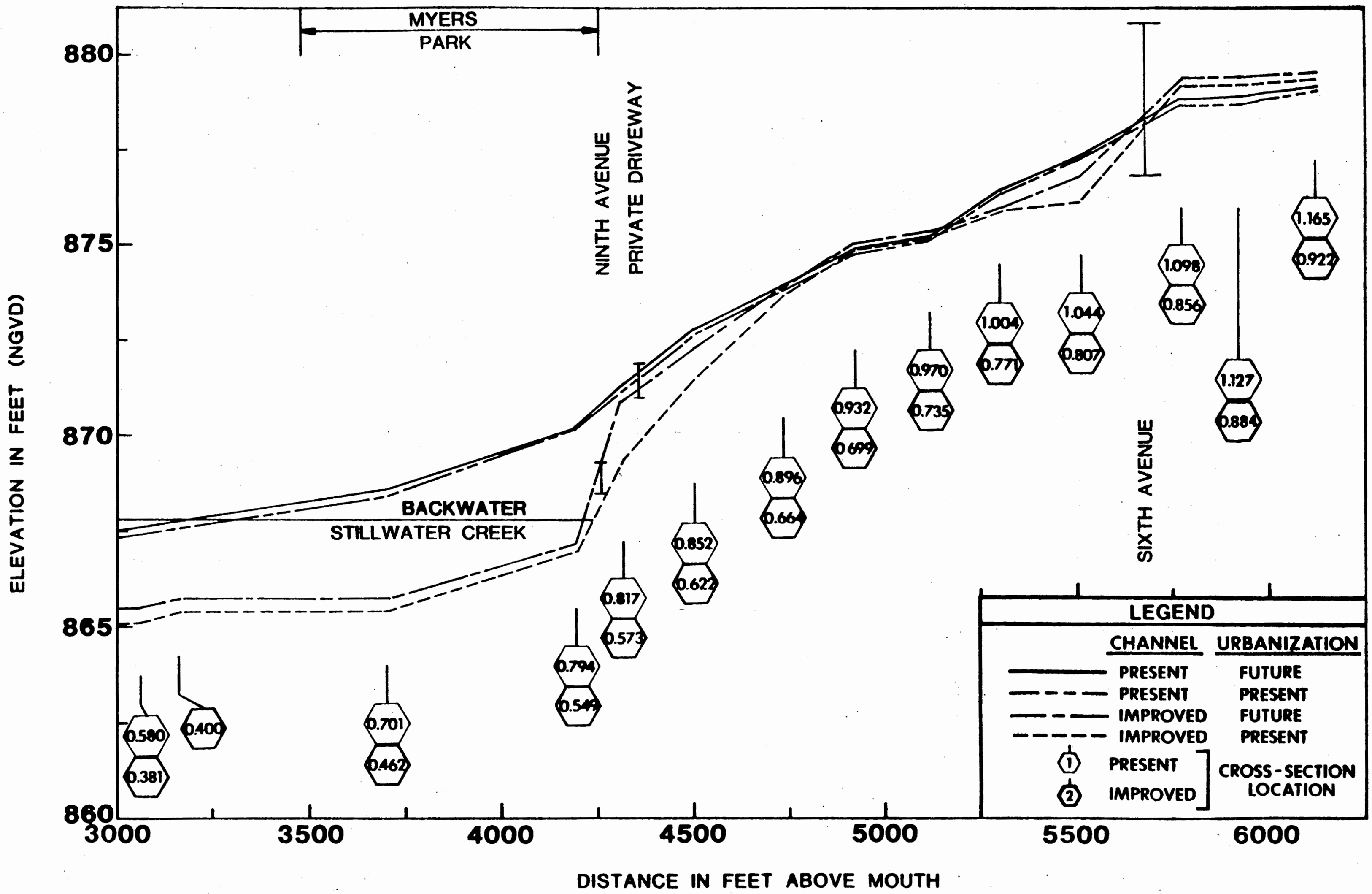


Figure 35. (Continued)

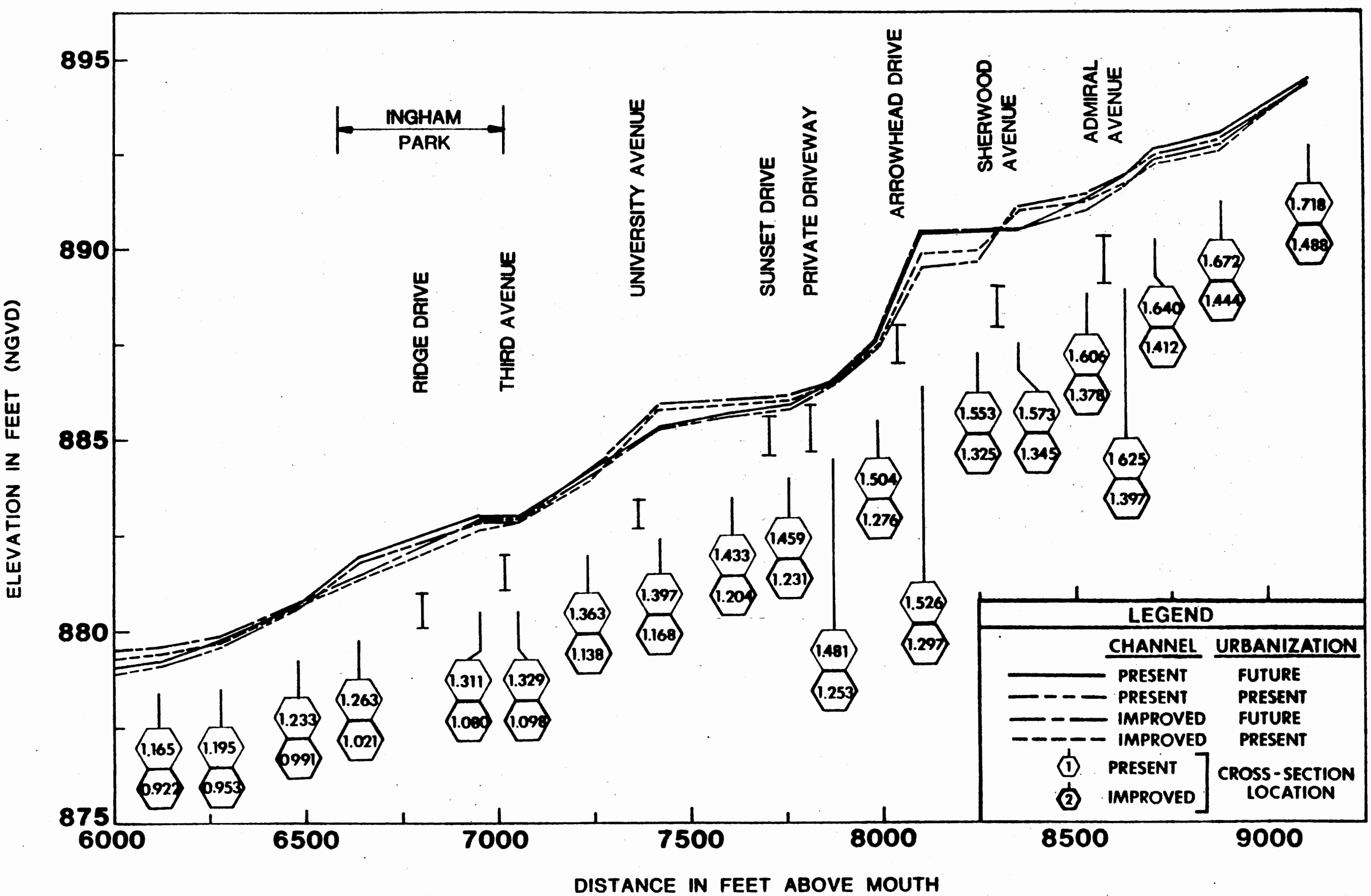


Figure 35. (Continued)

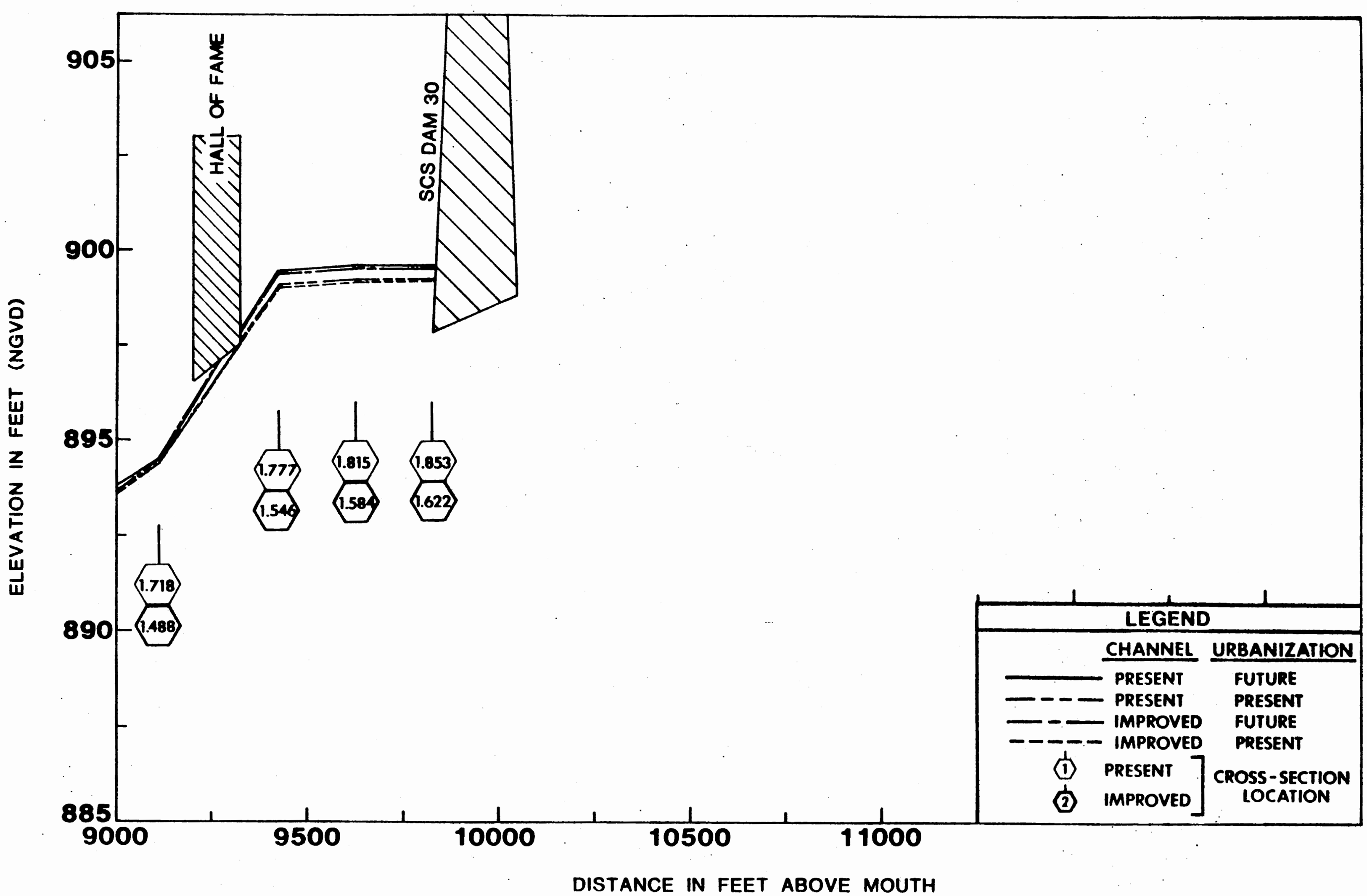


Figure 35. (Continued)

1. 10-year flood, 867.10, feet (NGVD)
2. 50-year flood, 867.59, feet (NGVD)
3. 100-year flood, 867.77, feet (NGVD)
4. 500-year flood, 868.16 feet (NGVD).

The flood elevation determinations were also used to map the flood boundaries of the 100- and 500-year floods on the base map. The tabular presentation of these boundaries can be seen in Appendix D. Although the adjustments are not shown in the profile summary tables, the backwater flood elevations and boundaries from Stillwater Creek had to be utilized downstream of 9th Avenue.

Examination of the water surface profile plots comparing the 10- and 100-year floods, Figures 34 and 35, indicates that the channel improvement does reduce the flood elevations, especially in the smaller floods. However, the benefit of the improved channel below 9th Avenue is almost completely negated by the backwater from Stillwater Creek.

Floodway Determination

A 100-year floodway was determined for each of the four channel/urbanization study alternatives by using HEC-2. Detailed procedures can be found in References (47) and (57).

Generally, the procedure involves making a first trial by one method and then subsequent trials by another method using the first trial as a guide. The first trial was performed using Method 4, which models encroachment on the stream by reducing conveyance on each overbank until the target increase in water surface elevation is obtained (47, 57). The targets used were: (1) 0.6 foot, (2) 0.8 foot, and (3) 1.0 foot. Subsequent trials were then performed using Method 1, which models encroachment by reading in the desired stationing directly.

The final designated floodways were determined by the following conditions in order of priority:

1. Encroachment was not allowed to go into the channel, beyond the stream banks.

2. The water surface was not allowed to rise more than 1.04 feet.

3. All existing structures were kept out of the floodway, if possible.

4. Encroachment was stopped if excessive velocities were developed.

5. The floodway boundaries were uniform, i.e., no excessive constrictions.

These floodways were drawn on the base maps with the 100- and 500-year flood boundaries. Floodway data are included in Appendix D.

The results were as expected. The future urbanization alternatives caused wider floodways than the present urbanization alternatives, due to an increase in discharge. The channel improvement alternatives produced generally more uniform and narrower floodways than the present channel alternatives below 6th Avenue.

Flood Hazard Determination

The flood hazard determination requires two steps: (1) determine Flood Hazard Factors (FHF), and (2) assign a Flood Insurance Rate Zone based on the FHF. The HEC-2 program option to determine these parameters was utilized for flood hazard determination. For a detailed discussion of procedures and definitions, see References (44) and (57).

The definition of the FHF is:

1. The Flood Hazard Factor (FHF) is used to correlate flood-frequency information directly into insurance rate tables. The FHF is a three-digit code which defines the difference in elevation between the 10-year and 100-year flood. FIA has correlated property damage from floods with FHF and has established a set of actuarial rate insurance premium tables (by building type) based on the FHF from 0.5 foot to 20 feet.

2. The FHF code expresses the differences between the 10- and 100-year flood elevations to the nearest one-half foot below FHF 100 and to the nearest one foot above FHF 100. For example, for a difference of 1.2 feet, the FHF is 010; for a difference of 1.4 feet, the FHF is 015; and for a difference of 5.0 feet, the FHF is 050 (44, pp. 2-15).

The FHFs are basically determined by a weighting procedure using the 100-10 year flood elevation differences within a reach. Then they are assigned a zone designation to assist insurance agents in determining actuarial flood insurance rates for specific properties. Areas within the 100-year flood boundary are called Special Hazard Areas, Zone A, and assigned numbers if detailed methods were used to determine flood elevations according to FHFs.

A comparison of the flood insurance zones for all channel/urbanization alternatives is shown in Table VII. Zone A2 indicates a one-foot difference between the 10- and 100-year floods while Zone A4 indicates a two-foot difference between the two floods. Again the backwater from Stillwater Creek would actually be used to determine the zone (A2) for below 9th Avenue.

The results are as to be expected. The Flood Insurance Zones for the channel improvement are greater than those of the present channel alternatives. This is because the improved channel is very efficient in conveying the smaller floods and produces a relatively much lower water surface elevation for the 10-year flood than the present channel. On the

TABLE VII

COMPARISON OF FLOOD INSURANCE ZONES

LOCATION	CHANNEL CROSS SECTION NUMBER		FLOOD INSURANCE ZONE			
			CHANNEL CONDITION			
			PRESENT		IMPROVED	
			URBANIZATION		URBANIZATION	
	PRESENT	IMPROVED	PRESENT	FUTURE	PRESENT	FUTURE
CONFLUENCE	0.090	10.019	A2	A2	A2	A2
	******	10.057	****	****	A2	A2
	0.178	10.095	A2	A2	A2	A2
	0.308	10.195	A2	A2	A2	A2
TWELVETH AVENUE	0.398	10.267	A2	A2	A2	A2
	0.430	10.299	A2	A2	A4	A4
OXBOW-WILLIS AVE	0.580	10.381	A2	A2	A4	A4
	******	10.400	****	****	A4	A4
MEYERS PARK	0.701	10.462	A2	A2	A4	A4
NINTH AVENUE	0.794	10.549	A2	A2	A4	A4
	0.817	10.573	A2	A2	A4	A4
	0.852	10.622	A2	A2	A4	A4
	0.896	10.664	A2	A2	A4	A4
	0.932	10.699	A2	A2	A4	A4
	0.970	10.735	A2	A2	A4	A4
	1.004	10.771	A2	A2	A4	A4
SIXTH AVENUE	1.044	10.807	A2	A2	A4	A4
	1.098	10.856	A2	A2	A2	A2
	1.127	10.884	A2	A2	A2	A2
	1.165	10.922	A2	A2	A2	A2

NOTE: ****** INDICATES CROSS SECTION NOT USED AT THAT LOCATION

TABLE VII (Continued)

LOCATION	CHANNEL CROSS SECTION NUMBER		FLOOD INSURANCE ZONE			
			CHANNEL CONDITION			
			PRESENT		IMPROVED	
			URBANIZATION		URBANIZATION	
	PRESENT	IMPROVED	PRESENT	FUTURE	PRESENT	FUTURE
	1.195	10.953	A2	A2	A2	A2
	1.233	10.991	A2	A2	A2	A2
RIDGE RD-INGHAM PK	1.263	11.021	A2	A2	A2	A2
THIRD AVENUE	1.311	11.080	A2	A2	A2	A2
	1.329	11.098	A2	A2	A2	A2
UNIVERSITY AVENUE	1.363	11.133	A2	A2	A2	A2
	1.397	11.168	A2	A2	A2	A2
SUNSET DRIVE	1.433	11.204	A2	A2	A2	A2
	1.459	11.231	A2	A2	A2	A2
	1.481	11.253	A2	A2	A2	A2
ARROWHEAD DRIVE	1.504	11.276	A2	A2	A2	A2
	1.526	11.297	A2	A2	A2	A2
ADMIRAL AVENUE	1.606	11.378	A2	A2	A2	A2
	1.625	11.397	A2	A2	A2	A2
	1.640	11.412	A2	A2	A2	A2
	1.672	11.444	A2	A2	A2	A2
HALL OF FAME	1.718	11.488	A2	A2	A2	A2
	1.777	11.546	A2	A2	A2	A2
	1.815	11.584	A2	A2	A2	A2
TOE OF SCS DAM 30	1.853	11.622	A2	A2	A2	A2

other hand, the improved channel is not as relatively efficient for the larger floods. Therefore, a greater difference between the 10- and 100-year flood elevations for the improved channel results.

CHAPTER VI

FLOOD DAMAGES

Introduction

Of the five empirical categories of flood damages, two quantitative costs were contemplated to compare the economic impact of the four channel/urbanization alternatives: (1) direct cost-flood damages, and (2) uncertainty cost--flood insurance premiums.

The flood insurance premiums cost was dropped from consideration after checking the Rate Tables (83). All zones from A1 to A7 have the same premium cost, so there is no discernible difference in costs between Zone A2 (present channel) and Zone A4 (improved channel) on that basis.

Therefore, it was decided that a relative comparison of economic costs between the study alternatives could be made by estimating the 100-year flood direct damages.

100-Year Flood Direct Damage Cost

The procedure for estimating the 100-year flood damages was:

1. Determination of which structures are in the 100-year flood boundary.

2. Determination of the first floor elevations of those structures.

3. Determination of the 100-year flood elevations at the identified structures.

4. Determination of property values of the identified structures.

5. Selection of damage curves.

6. Calculation of flood damages.

The determination of which structures were in the 100-year flood boundary was a relatively easy task utilizing the results of the investigation thus far. Positive transparencies of the 1 inch = 400 feet composite aerial photograph of the Duck Creek Study Area were made. Then the 100-year flood boundaries were traced onto these transparencies from the flood boundaries drawn on the base map. The structures within the flood boundaries, or very close to them, were identified and assigned code numbers.

Next, the first floor elevations of the identified structures were estimated from cross-section plots similar to those in Figures 26 and 27. Elevations for the structures between the cross sections were interpolated. Then, the 100-year flood elevations were determined at the cross sections for all study alternatives and interpolated between the cross sections. The results are presented in Appendix F.

The property values of the identified structures, in 1980 dollars, were estimated by consulting a local real estate broker. This method was used to get the most current values possible.

The 1970 depth-damage curves compiled by the FIA provide reasonable estimates of damage (42, 51, 69). Although a 1974 set of data has been compiled, the 1970 depth-percent damage relationships were used because the data use total value based on replacement cost. The more recent data represent a downward revision of the 1970 data, due mainly to the deduction of depreciation from the costs. Therefore, the 1970 data represent an upper bound on total damage for comparison purposes (69).

Johnson (69) modified the 1970 data slightly at and below the first floor elevation to reflect detailed distribution of damage found in unpublished USCE data for along the Ohio River. These modified data were used to plot curves and construct a depth-percent damage table with 0.1 foot increments (Appendix G).

Since the table is constructed for separate structure and contents costs, a value of 35% of structure value was utilized for contents value. The 100-year damage cost for each identified structure for each of the study alternatives was calculated as follows:

$$D_t = F_s V_s + F_c V_c \qquad (6.1)$$

where

D_t = total damage to the structure and contents, in dollars;

F_s = fraction of the structure damaged;

F_c = fraction of the contents damaged;

V_s = market value of the structure, in dollars; and

V_c = market value of the contents, in dollars.

The results for each identified structure are presented in Table VIII. The comparison of the total 100-year flood damages for the four basin alternatives is shown in Table IX.

The upper portion of Table IX represents the three major segments of Duck Creek: (1) between the mouth of Duck Creek and 9th Avenue, the earth improvement portion of the improved channel alternative; (2) between 9th Avenue and 6th Avenue, the concrete improvement portion of the improved channel alternative; and (3) between 6th Avenue and the end of the study, the cleared channel portion of the improved channel alternative.

In the first segment of Duck Creek, the improved channel alternatives had only slightly lower 100-year flood damages than the present channel

TABLE VIII

INDIVIDUAL BUILDING DAMAGE COMPARISONS

BLDG CODE NO.	1980 VALUE (DOLLARS, THOUSANDS): BUILDING	1980 VALUE (DOLLARS, THOUSANDS): CONTENTS (35% BLDG)	DEPTH-DAMAGE CURVE NUMBER	100-YEAR FLOOD DAMAGE (1980 DOLLARS, THOUSANDS), CHANNEL CONDITION: PRESENT, URBANIZATION: PRESENT	PRESENT, URBANIZATION: FUTURE	IMPROVED, URBANIZATION: PRESENT	IMPROVED, URBANIZATION: FUTURE
1	56.0	19.6	1	1.1	1.8	0.2	0.2
2	63.0	22.0	1	1.6	2.5	0.3	0.3
3	64.0	22.4	1	6.2	11.6	0.3	0.3
4	48.5	17.0	1	0.3	0.3	0.3	0.3
5	50.0	17.5	1	17.1	17.1	17.1	17.1
6	49.5	17.3	1	0.3	0.3	0.3	0.3
7	52.0	18.2	1	17.8	17.8	17.8	17.8
8	52.0	18.2	1	24.1	24.1	24.1	24.1
9	51.5	18.0	1	21.3	21.3	21.3	21.3
10	49.0	17.1	1	22.7	23.7	20.3	20.3
11	51.0	17.8	1	13.4	13.0	10.6	11.7
12	51.0	17.8	1	7.6	7.6	7.6	10.6
13	50.0	17.5	1	2.0	2.0	2.0	7.5
14	50.5	17.7	1	9.2	10.5	1.0	4.9
15	48.0	16.8	1	17.9	17.9	8.7	15.4
16	49.0	17.1	1	21.4	22.1	11.2	18.9
17	48.5	17.0	1	10.1	12.3	0.0	7.3
18	50.0	17.5	1	16.0	18.0	0.2	13.8
19	49.0	17.1	1	13.5	15.7	0.0	11.2
20	50.0	17.5	1	26.8	27.2	20.7	25.2
21	51.0	17.8	1	23.0	23.6	18.4	21.7
22	52.0	18.2	1	1.4	1.7	0.6	1.4
23	52.0	18.2	1	7.8	9.4	7.8	9.4
24	51.5	18.0	1	15.4	16.5	15.4	16.5
25	51.5	18.0	1	19.9	20.7	20.7	21.3
26	53.5	18.7	1	14.7	14.7	14.7	17.1
27	51.5	18.0	1	7.7	7.7	7.7	10.7
28	51.0	17.8	1	7.6	9.2	5.0	7.6

TABLE VIII (Continued)

BLDG CODE NO.	1980 VALUE (DOLLARS, THOUSANDS): BUILDING	1980 VALUE (DOLLARS, THOUSANDS): CONTENTS (35% BLDG)	DEPTH-DAMAGE CURVE NUMBER	100-YEAR FLOOD DAMAGE (1980 DOLLARS, THOUSANDS), CHANNEL CONDITION: PRESENT, URBANIZATION: PRESENT	PRESENT CHANNEL, URBANIZATION: FUTURE	IMPROVED CHANNEL, URBANIZATION: PRESENT	IMPROVED CHANNEL, URBANIZATION: FUTURE
29	50.5	17.7	1	7.6	7.6	1.6	2.0
30	52.5	18.4	1	7.9	7.9	1.0	1.4
31	52.0	18.2	1	1.4	1.7	0.1	0.4
32	37.0	12.9	1	3.6	3.6	0.2	1.0
33	69.0	24.1	1	12.5	14.3	0.3	1.8
34	46.5	16.3	1	13.9	14.9	4.5	9.6
35	46.5	16.3	1	18.7	18.7	14.9	15.9
36	47.0	16.4	1	18.2	18.2	16.1	16.9
37	47.5	16.6	1	17.1	17.7	16.3	17.1
38	43.0	15.0	1	14.7	14.7	14.7	16.0
39	41.5	14.5	1	18.7	18.7	18.7	19.7
40	38.0	13.3	2	11.4	11.7	11.7	12.0
41	38.0	13.3	2	10.9	11.2	11.2	11.4
42	37.5	13.1	2	9.3	9.7	9.3	9.7
43	38.0	13.3	1	14.7	15.3	14.7	15.3
44	37.0	12.9	2	5.4	5.8	4.2	5.4
45	44.0	15.4	1	18.2	18.8	14.1	17.0
46	42.0	14.7	1	19.9	20.3	12.6	17.9
47	39.0	13.6	1	12.5	14.1	0.2	10.7
48	42.5	14.9	1	13.6	15.3	0.2	11.7
49	39.5	13.8	1	1.3	3.9	0.0	0.8
50	41.0	14.3	1	22.0	22.3	17.0	20.6
51	42.0	14.7	1	19.0	19.0	13.4	17.4
52	39.0	13.6	1	0.5	0.6	0.0	0.2
53	47.5	16.6	1	9.9	10.9	13.1	15.2
54	60.0	21.0	2	0.6	0.8	0.6	1.2
55	60.0	21.0	2	5.4	6.1	5.4	6.1
56	56.0	19.6	1	1.5	1.8	0.3	0.7

TABLE VIII (Continued)

BLDG CODE NO.	1980 VALUE (DOLLARS, THOUSANDS) BUILDING	CONTENTS (35% BLDG)	DEPTH-DAMAGE CURVE NUMBER	100-YEAR FLOOD DAMAGE (1980 DOLLARS, THOUSANDS) CHANNEL CONDITION: PRESENT — URBANIZATION PRESENT	PRESENT — URBANIZATION FUTURE	IMPROVED — URBANIZATION PRESENT	IMPROVED — URBANIZATION FUTURE
57	58.5	20.5	1	27.1	27.8	24.9	27.1
58	61.5	21.5	1	33.0	33.5	31.4	33.0
59	64.0	22.4	1	11.6	13.3	1.7	2.6
60	48.5	17.0	1	4.7	7.3	4.7	7.3
61	52.5	18.4	1	10.9	12.0	10.9	12.0
62	51.0	17.8	1	16.3	17.5	16.3	17.5
63	61.9	21.7	1	15.7	15.7	15.7	18.6
64	62.5	21.9	1	0.4	0.5	1.6	2.5
65	66.5	23.3	1	12.1	13.8	19.9	22.8
66	53.0	18.5	1	0.4	0.6	1.1	1.7
67	50.0	17.5	1	17.1	18.0	18.7	19.3
68	53.0	18.5	1	20.5	21.3	21.9	22.6
69	54.0	18.9	1	1.4	1.7	2.2	5.3
70	49.5	17.3	1	9.0	18.5	12.6	18.5
71	53.0	18.5	1	12.1	21.3	15.9	21.3
72	52.0	18.2	1	9.4	19.4	13.2	19.4
73	49.0	17.1	1	0.3	7.3	0.8	7.3
74	51.0	17.8	1	0.6	9.2	1.0	9.2
75	51.0	17.8	1	1.0	11.7	1.6	11.7
76	52.5	18.4	1	12.0	20.3	14.5	20.3
77	44.0	15.4	1	0.0	1.1	0.2	1.1
78	38.0	13.3	1	7.9	1.2	7.9	1.2
79	39.9	14.0	1	16.0	12.8	16.0	12.8
80	41.5	14.5	1	1.3	1.1	0.8	0.5
81	42.5	14.9	1	1.7	1.7	1.1	1.1
82	43.0	15.0	1	0.5	0.5	0.3	0.3

TABLE IX

TOTAL DAMAGE COMPARISONS, FOUR BASIN ALTERNATIVES

LOCATION	TOTAL 100-YEAR FLOOD DIRECT DAMAGE (1980 DOLLARS, THOUSANDS)			
	CHANNEL CONDITION			
	PRESENT		IMPROVED	
	URBANIZATION		URBANIZATION	
	PRESENT	FUTURE	PRESENT	FUTURE
BETWEEN MOUTH OF DUCK CREEK AND 9TH AVE	112.6	120.5	101.9	101.9
BETWEEN 9TH AVE AND 6TH AVE	528.5	555.6	349.5	484.3
BETWEEN 6TH AVE AND END OF STUDY	260.4	328.7	276.4	340.0
TOTAL	901.5	1004.8	727.8	926.2

alternatives, since the backwater from Stillwater Creek eliminates most of the benefit from the channel improvement. In the present channel alternatives, the future urbanization alternative resulted in higher 100-year flood damages.

In the second segment, the major differences between the four channel/urbanization alternatives appear. The present channel and future urbanization alternatives produce higher 100-year flood damages than the improved channel and present urbanization alternatives, respectively. Note in the improved channel, the small increase in discharge for the future urbanization alternative, 100 ft^3/s, resulted in much higher flood damages than the present urbanization alternative.

In the third segment of the stream, all the alternatives had similar damages. The improved channel alternatives had slightly higher damages than the present channel alternatives. This was due to the water surface profiles "crossing" in this segment, a phenomenon that often occurs when there is only a small difference in the discharges and roughness coefficients.

The improved channel alternatives do result in lower total 100-year flood damages than the present channel alternatives, but at a relatively smaller degree for the future urbanization alternative. The future urbanization alternatives result in higher total flood damages than the present urbanization alternatives.

CHAPTER VII

SUMMARY AND CONCLUSIONS

Summary

In an effort to develop a methodology to assess the impact of a changing flood plain determination on an ungaged urban watershed, a flood insurance type study for the Duck Creek basin in northwest Stillwater, Oklahoma, was conducted for each of the four following channel/urbanization alternatives:

1. Present channel-present urbanization
2. Improved channel-present urbanization
3. Present channel-future urbanization
4. Improved channel-future urbanization.

Preliminary hydrologic analyses were performed for the present channel, present urbanization alternative. Using the SCS TR-20 model, the following variables were compared: (1) two AMCs: II and III; (2) three design storm durations: 1-, 3-, and 6-hours; and (3) two residential imperviousness estimates: 20% and Table I values. The USGS regression equation method was used to compute discharges for the two residential imperviousness estimates.

Next, final hydrologic analyses were performed utilizing the AMC II, 6-hour storm duration, and the Table I residential imperviousness estimates for all four channel-urbanization alternatives comparing: (1) two hydrologic models: TR-20, and USGS regression equations; and (2) two subarea configurations for the TR-20 model: simple and complex.

Then the water surface profiles for the 10-, 50-, 100-, and 500-year floods, the floodways, and the Flood Insurance Zones were determined using the HEC-2 model and the complex subarea peak flood discharges from the TR-20 hydrologic model. The results were used with a depth-percent damage relationship to determine the 100-year flood direct damages in order to provide a relative comparison of the impact of changing flood plain determinations.

The findings in the hydrology, hydraulics, and flood damages phases of the investigation are summarized in the following sections.

Hydrology

In the preliminary peak discharge run, the AMC III produced higher peak discharges than the AMC II. Within the AMC III, the shorter the rainfall storm duration, the higher the peak discharge for the 1-, 3-, and 6-hour design storm durations. Within the AMC II, the 100-year and 500-year flood peak discharges generally followed the expected pattern as in above. However, the 10-year and the 50-year frequency discharges did not follow the trend; the 3-hour storm often produced a lower peak discharge than the 6-hour storm.

The 20% residential imperviousness assumption yielded lower peak discharge values than the Table I estimates, but there was not a large difference between the resultant discharges.

In the final peak discharge run, it was found in comparing the urbanization alternatives for the present channel that the USGS regression equation method differs significantly from the TR-20 method estimates when the watershed shape is not "uniform." As the watershed shape becomes more uniform, the two methods give similar discharge estimates. The USGS method does not have the capability to assess channel improvements; therefore, no comparison is possible with the improved channel alternatives.

The simple subarea configuration in the TR-20 method produces generally lower peak discharge estimates than the complex subarea configuration, but the estimates are not significantly different.

Hydrograph plots at the mouth of Duck Creek reveal some of the classic effects of channel improvement. Lag time is reduced and peak flows are larger for the improved channel than the present channel.

The future urbanization alternatives do produce higher peak discharges for Duck Creek. However, most of the future urbanization would take place in the upper end of the watershed where a SCS flood retention stucture eliminates discharge increases from that portion of the basin.

Hydraulics

The channel improvement does reduce flood elevations, especially in the smaller floods. The reductions of flood elevations for the 100-, and 500-year floods are minimal. Also the benefit of the improved channel below 9th Avenue is almost completely eliminated by backwater from Stillwater Creek. The future urbanization alternatives did generally result in higher flood elevations than the present urbanization alternatives.

The floodway results were as expected. The improved channel allowed the use of generally more uniform and narrow floodways than the present channel below 6th Avenue. The future urbanization alternatives required wider floodways than the present urbanization alternatives.

The Flood Insurance Zones for the channel improvement alternatives are greater than those for the present channel alternatives between 12th Avenue and 6th Avenue. However, there is not an increase in flood insurance rates, since both zones fall within the same flood insurance rate increment.

Flood Damages

The improved channel alternatives do result in lower 100-year flood direct damages than the present channel alternatives for Duck Creek. Most of the reduction occurred in the concrete channel improvement segment between 9th Avenue and 6th Avenus. The future urbanization alternatives did result in higher 100-year flood direct damages than the present urbanization alternatives.

Conclusions

The objective of this study was to develop a methodology to assess the impact of a changing flood plain determination on an ungaged urban basin. It was found on Duck Creek, Stillwater, Oklahoma, that both channel improvement and future urbanization of a watershed can significantly affect the estimated 100-year flood direct damages.

One of the most difficult steps in a flood plain management study on an ungaged urban basin is to estimate the peak flood flows. Both the USGS regression equation method and the SCS TR-20 model were utilized to determine estimates of the peak flows.

The USGS regression equation method cannot be used to assess the effects of a flood retention reservoir or a channel improvement on a basin. However, where the basin changes are expected only from urbanization and the basin shape is uniform, i.e., the subareas do not have a t_c greater than the main channel, the USGS method produces peak flow estimates similar to the more complex SCS TR-20 model.

In the TR-20 model, the single family residential lot imperviousness can be assumed to be 20 percent to yield peak discharge estimates reasonably comparable to assessing each residential lot using Table I values. A simple subarea configuration can produce peak discharge estimates reasonably comparable to a complex subarea configuration if care is exercised in choosing subarea boundaries.

The TR-20 hydrologic model is relatively easy to use and requires a data base that is moderately easy to obtain and relate to the physical characteristics of the watershed. The model can assess the effects of reservoirs, channel improvements and future urbanization.

The best flood plain management tool for a community would be a series of flood insurance type studies for some reasonable channel/urbanization alternatives to obtain a good indication of the effects of future planning decisions. However, due to present regulations, only one flood insurance study at a time is allowed and then restudy requests may be made in the future after significant watershed changes have occurred.

One solution would be to make use of the flood insurance study data to construct rating curves and plot water surface profiles for the new discharges based on these rating curves as proposed by Huntzinger (64). Although one must realize the limitations of extending rating curves, this should yield a fair estimate if there are no changes in channel condition in the basin studied.

Another solution for a community would be to obtain the flood insurance study data and make its own analyses for various channel/urbanization alternatives utilizing the methodology proposed in this investigation.

Flood plain determinations are highly susceptible to changes in channel condition and watershed urbanization. The SCS TR-20 model and the methodology presented in this study can assist a community with ungaged urban basins make intelligent flood plain management decisions.

CHAPTER VIII

SUGGESTIONS FOR FUTURE STUDY

These suggestions for future study would be helpful for the City of Stillwater, Oklahoma, to determine their floodplain management policies:

1. Collect rainfall-runoff data on Duck Creek and other metropolitan streams. Compare the results with the TR-20 model and the U.S. Geological Survey regression equation method to help assess the reliability and accuracy of the methods.

2. Compare rating curve extensions with complete hydraulic analyses to assess the reliability and accuracy of the rating curve extension method.

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APPENDIX A

WATERSHED SCHEMATIC DIAGRAMS

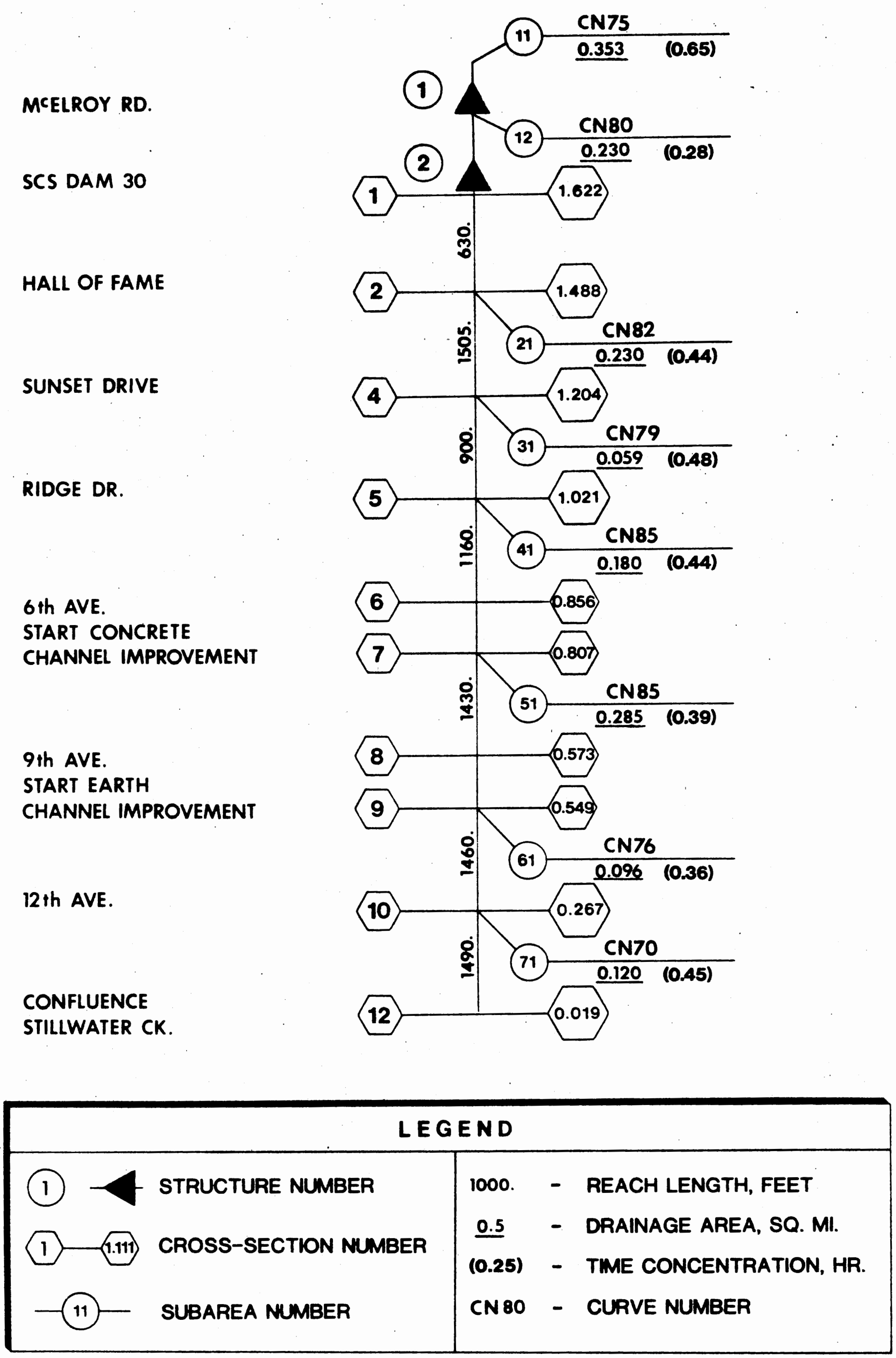


Figure 36. Watershed Schematic Diagram for Improved Channel-Present Urbanization, Simple Subarea Configuration

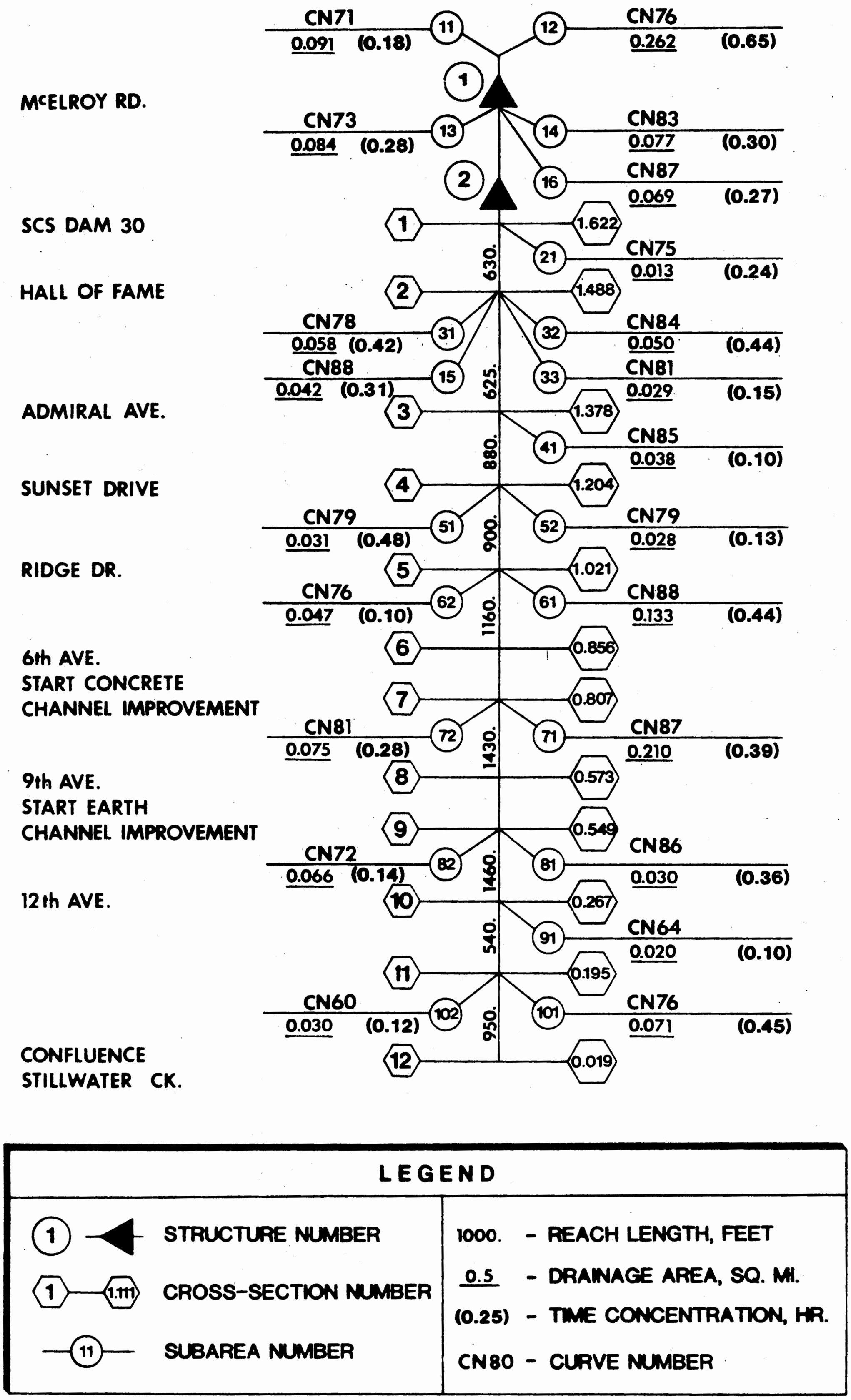
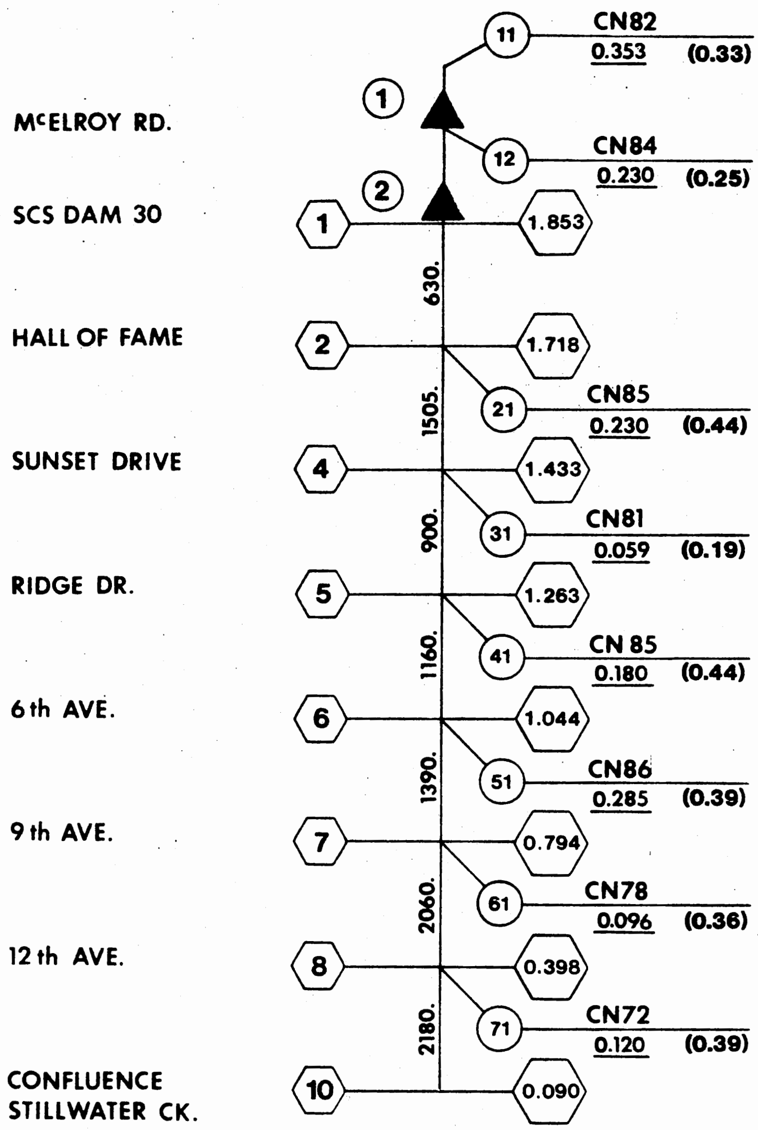
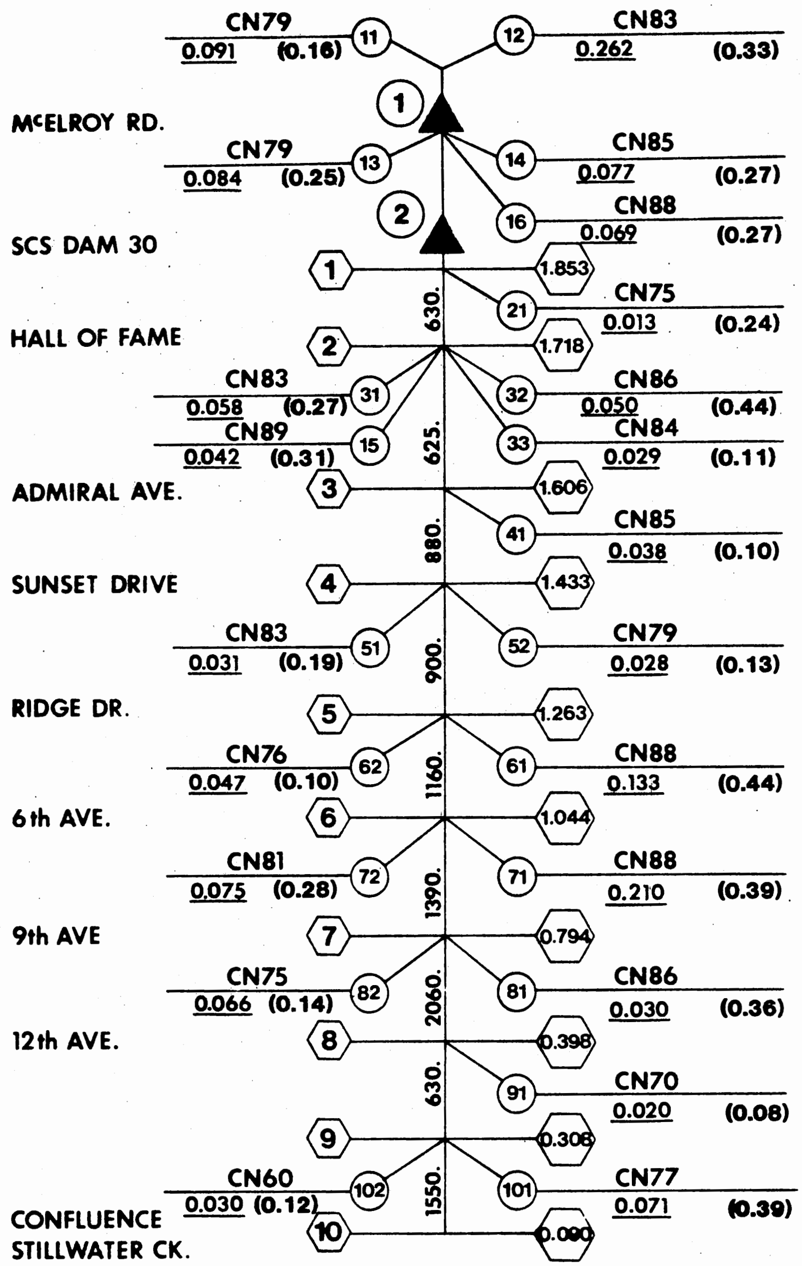


Figure 37. Watershed Schematic Diagram for Improved Channel-Present Urbanization, Complex Subarea Configuration



LEGEND	
(1) ◄ STRUCTURE NUMBER	1000. – REACH LENGTH, FEET
(1)—(1.111) CROSS-SECTION NUMBER	0.5 – DRAINAGE AREA, SQ. MI.
—(11)— SUBAREA NUMBER	(0.25) – TIME CONCENTRATION, HR.
	CN 80 – CURVE NUMBER

Figure 38. Watershed Schematic Diagram for Present Channel-Future Urbanization, Simple Subarea Configuration



LEGEND	
(1) ◄ STRUCTURE NUMBER	1000. - REACH LENGTH, FEET
⟨1⟩—⟨1.111⟩ CROSS-SECTION NUMBER	0.5 - DRAINAGE AREA, SQ. MI.
—(11)— SUBAREA NUMBER	(0.25) - TIME CONCENTRATION, HR.
	CN80 - CURVE NUMBER

Figure 39. Watershed Schematic Diagram for Present Channel-Future Urbanization, Complex Subarea Configuration

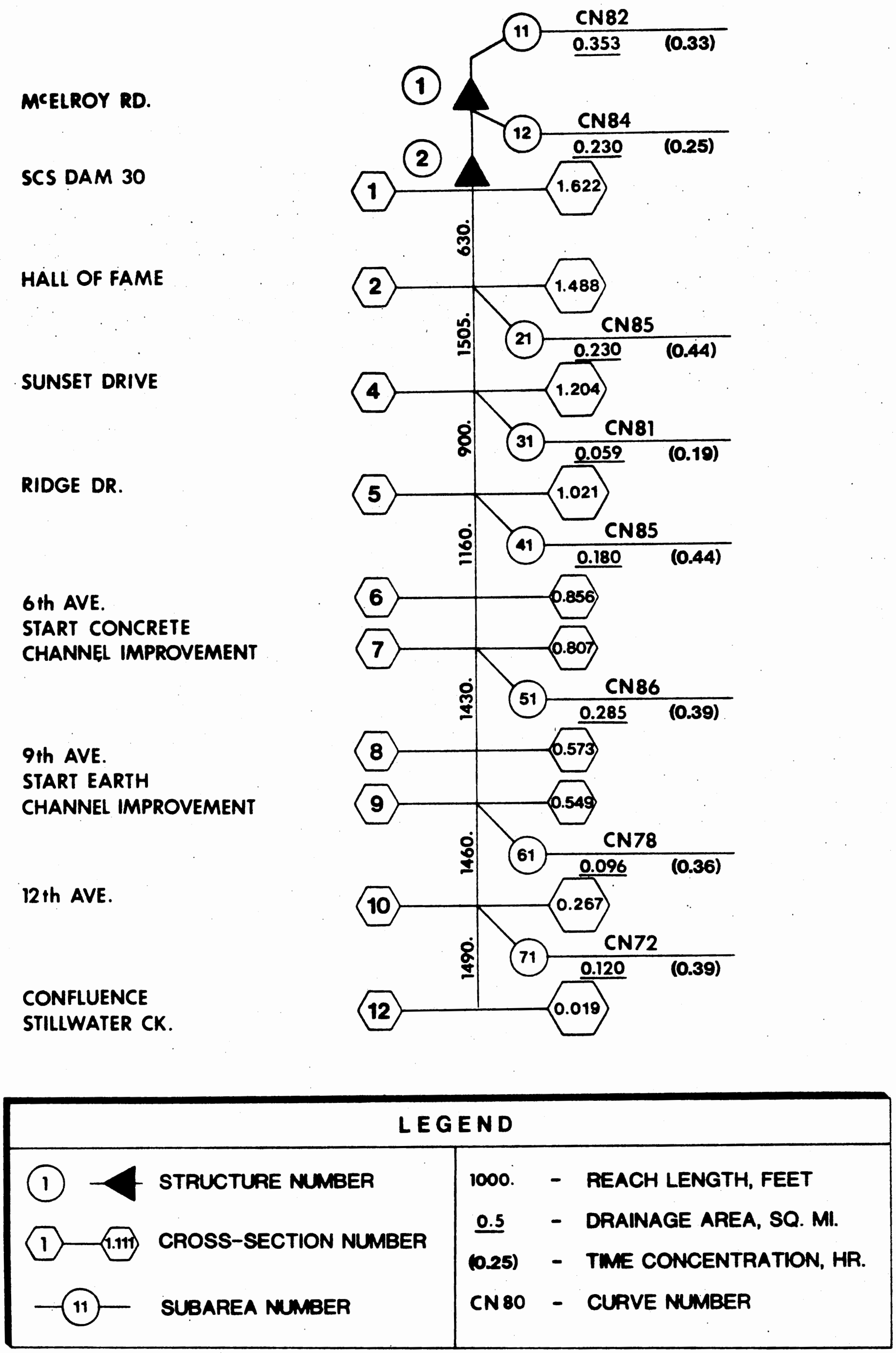


Figure 40. Watershed Schematic Diagram for Improved Channel-Future Urbanization, Simple Subarea Configuration

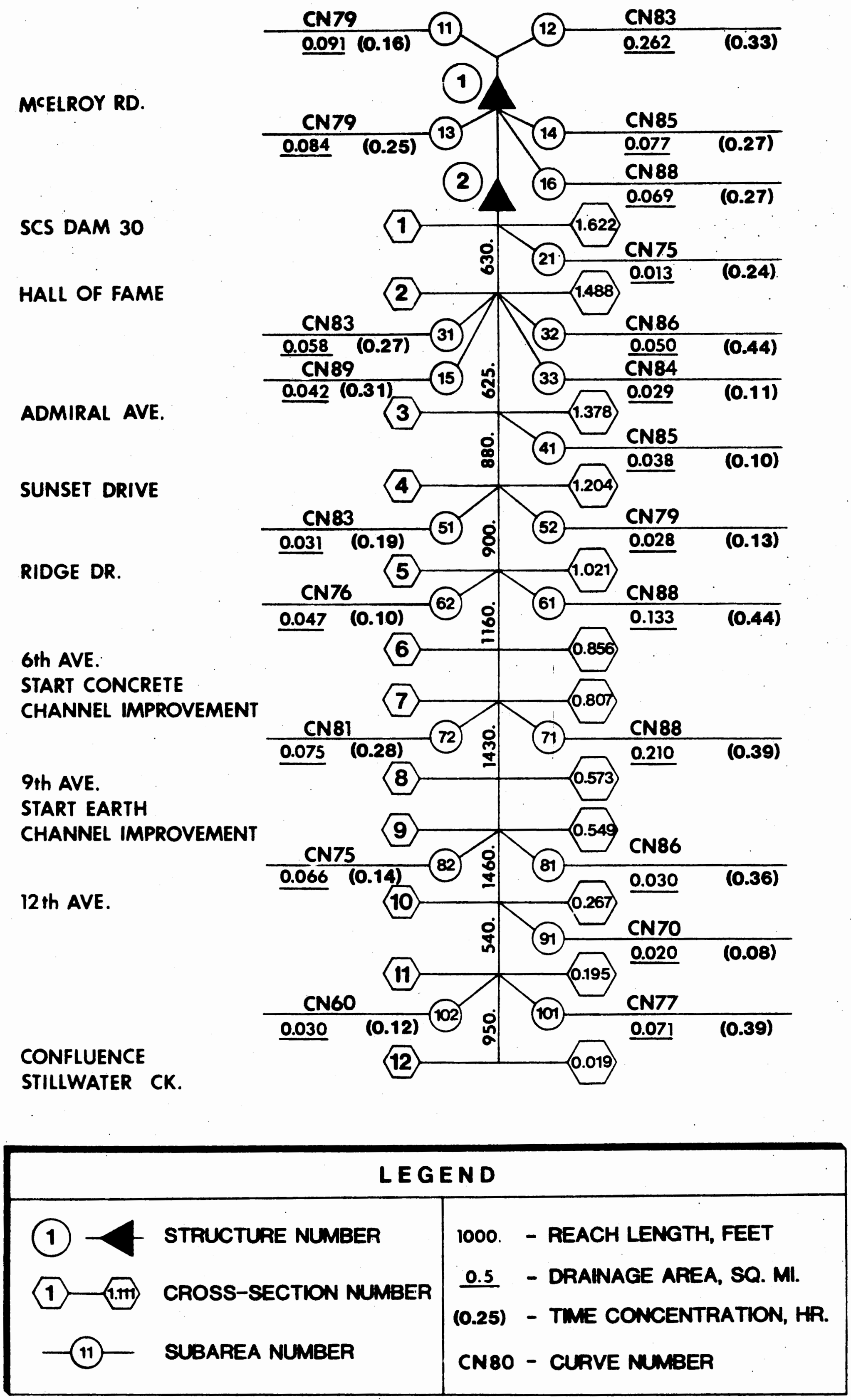


Figure 41. Watershed Schematic Diagram for Improved Channel-Future Urbanization, Complex Subarea Configuration

APPENDIX B

HYDROGRAPH PLOTS

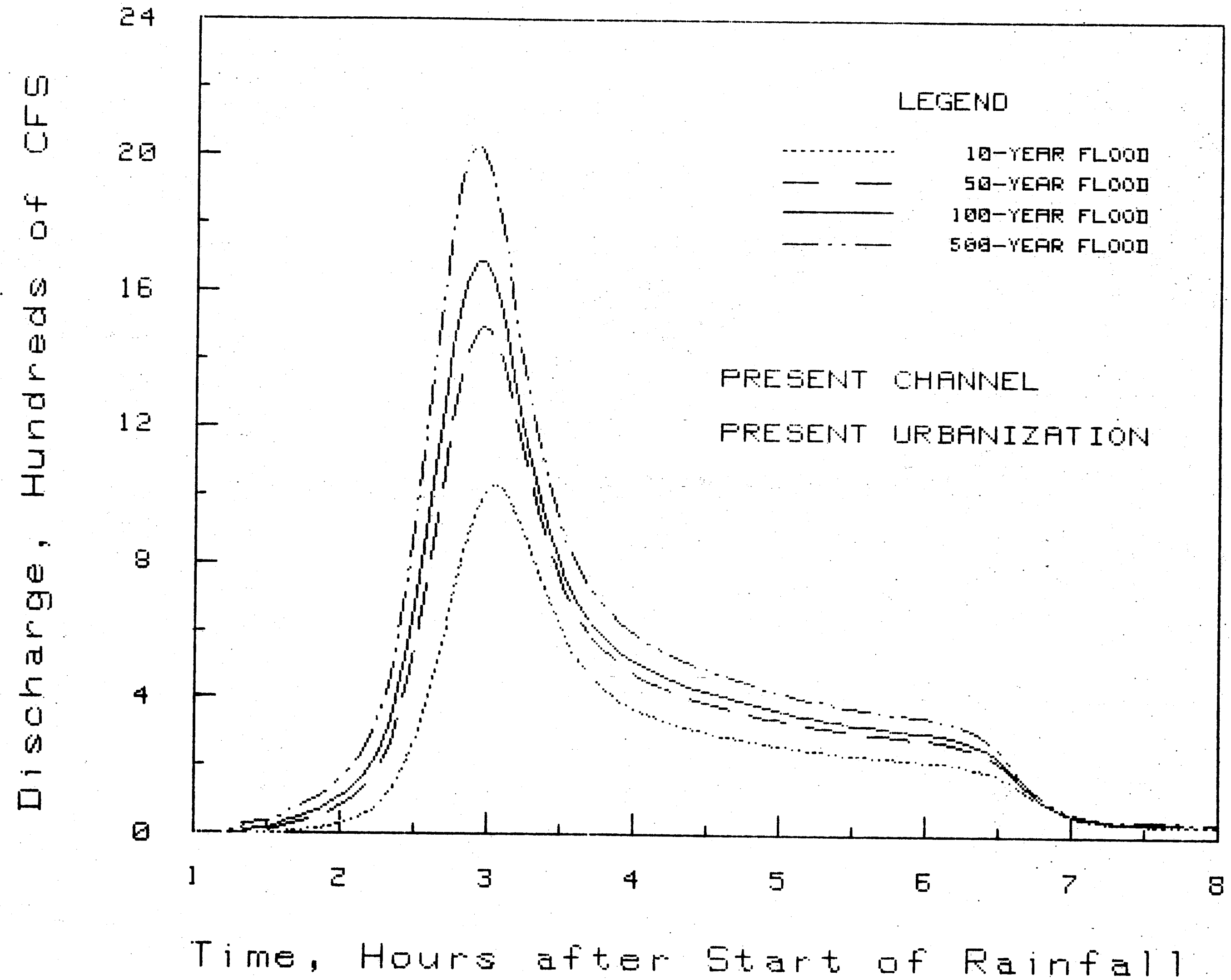


Figure 42. Flood Hydrographs at the Mouth of Duck Creek for Present Channel-Present Urbanization

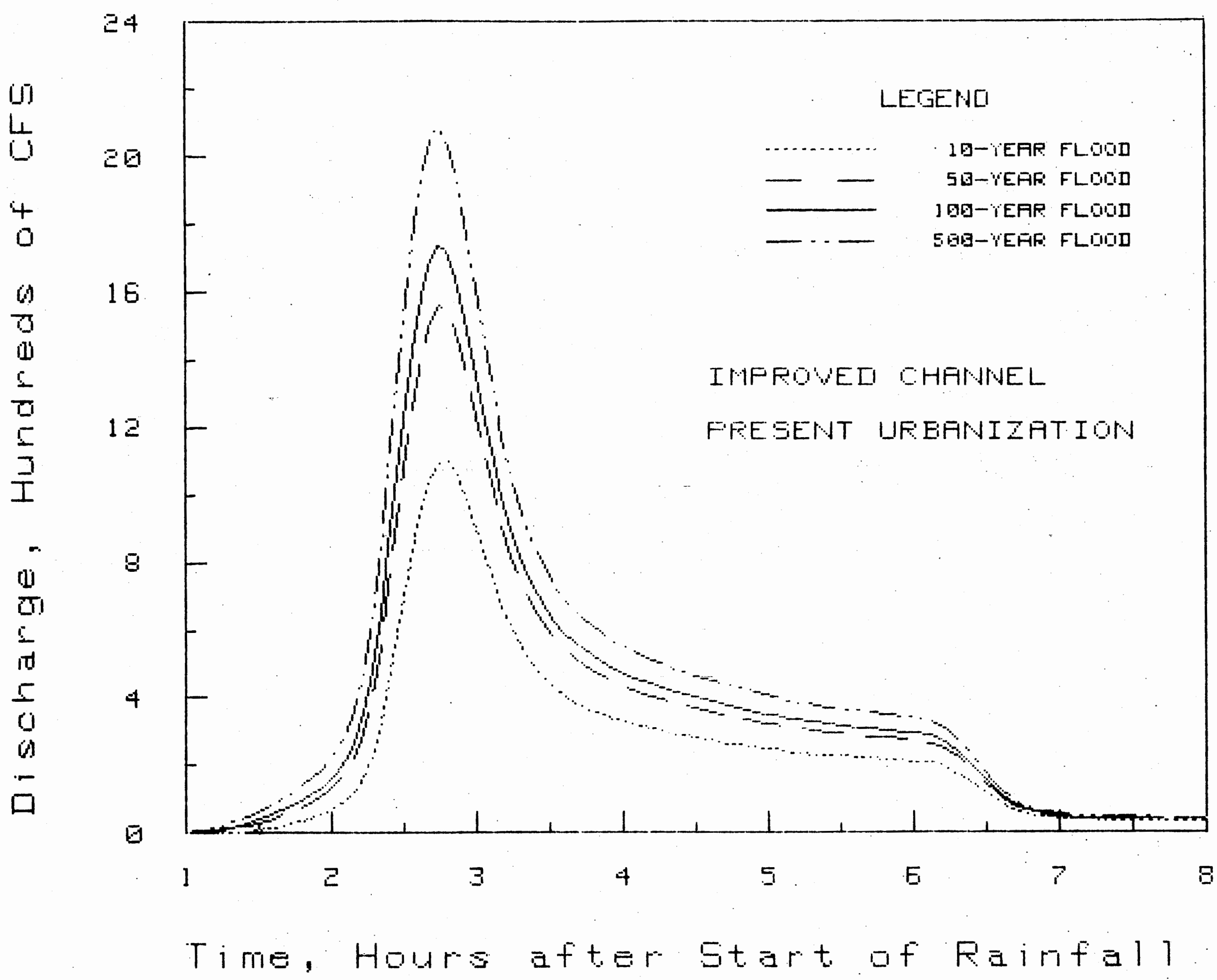


Figure 43. Flood Hydrographs at the Mouth of Duck Creek for Improved Channel-Present Urbanization

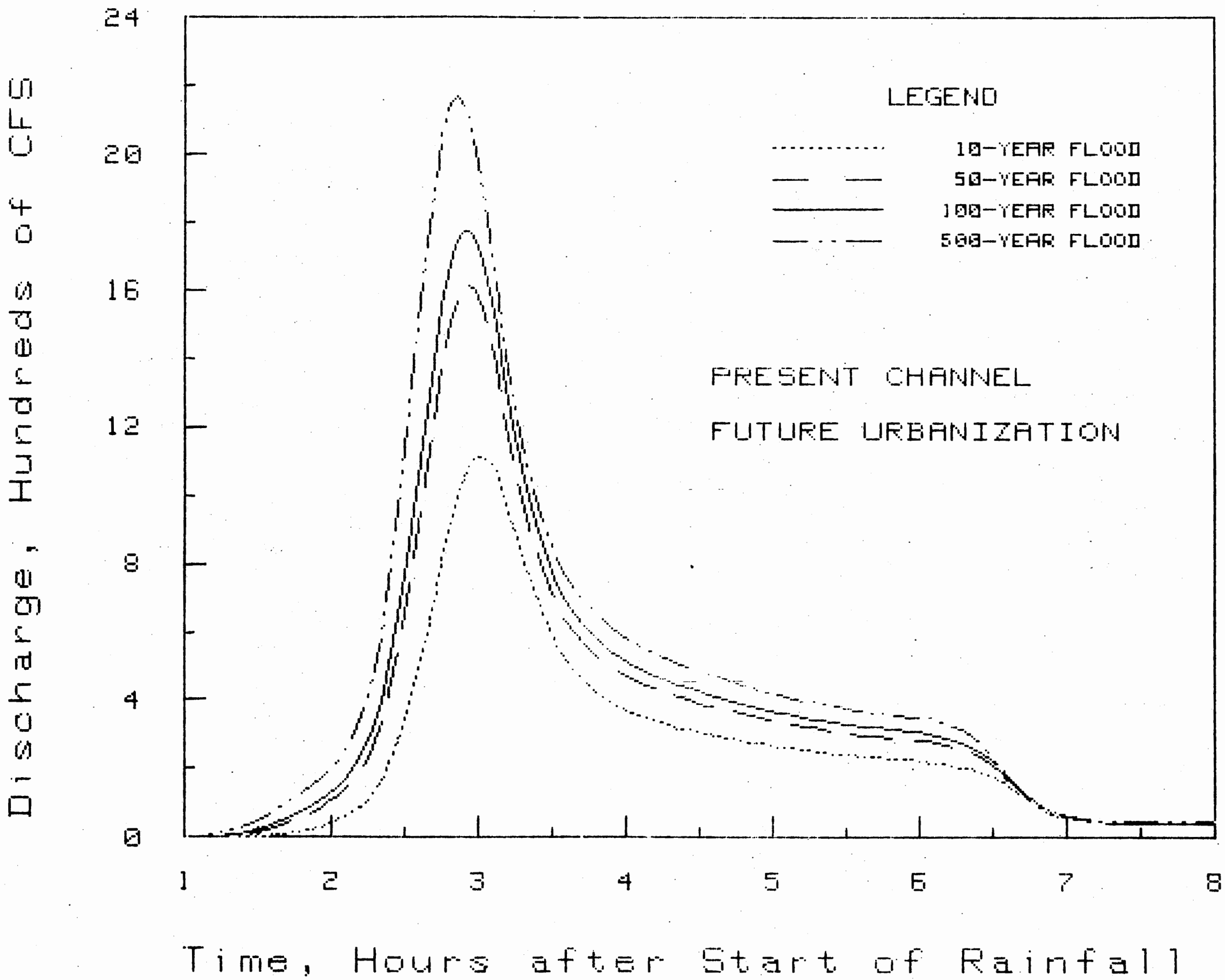


Figure 44. Flood Hydrographs at the Mouth of Duck Creek for Present Channel-Future Urbanization

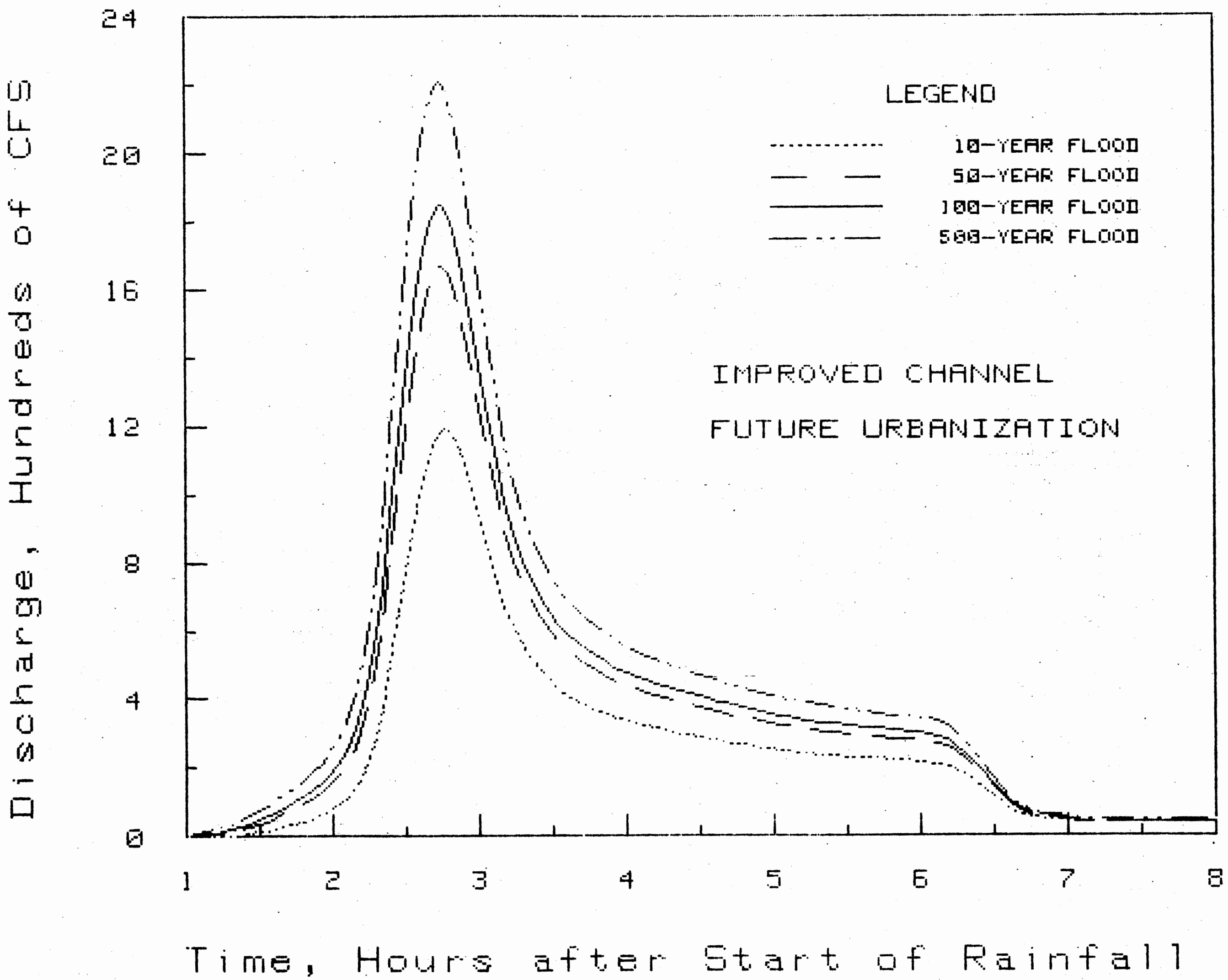


Figure 45. Flood Hydrographs at the Mouth of Duck Creek for Improved Channel-Future Urbanization

APPENDIX C

WATER SURFACE PROFILE PLOTS

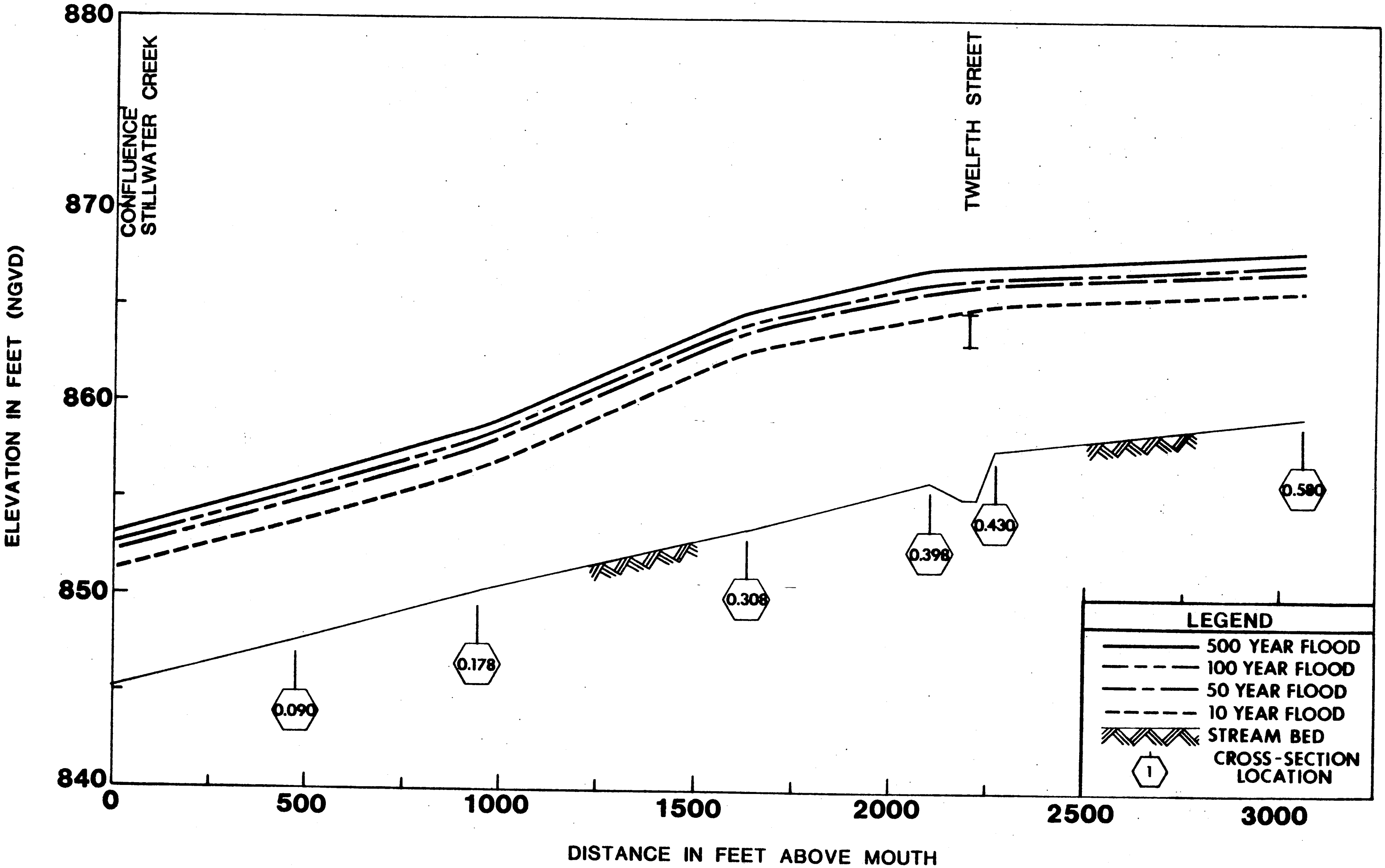


Figure 46. Water Surface Profile Plots for Present Channel-Present Urbanization

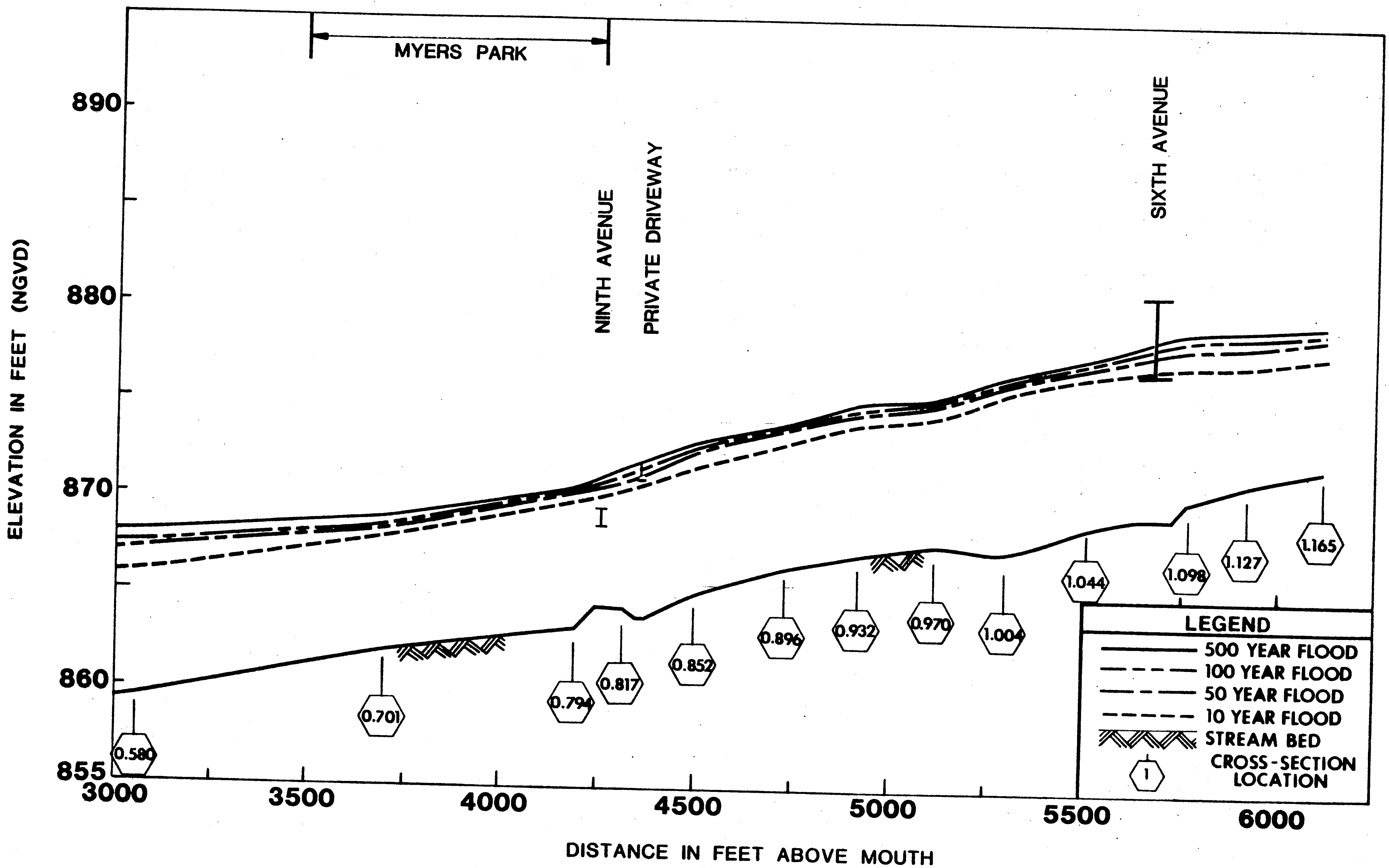


Figure 46. (Continued)

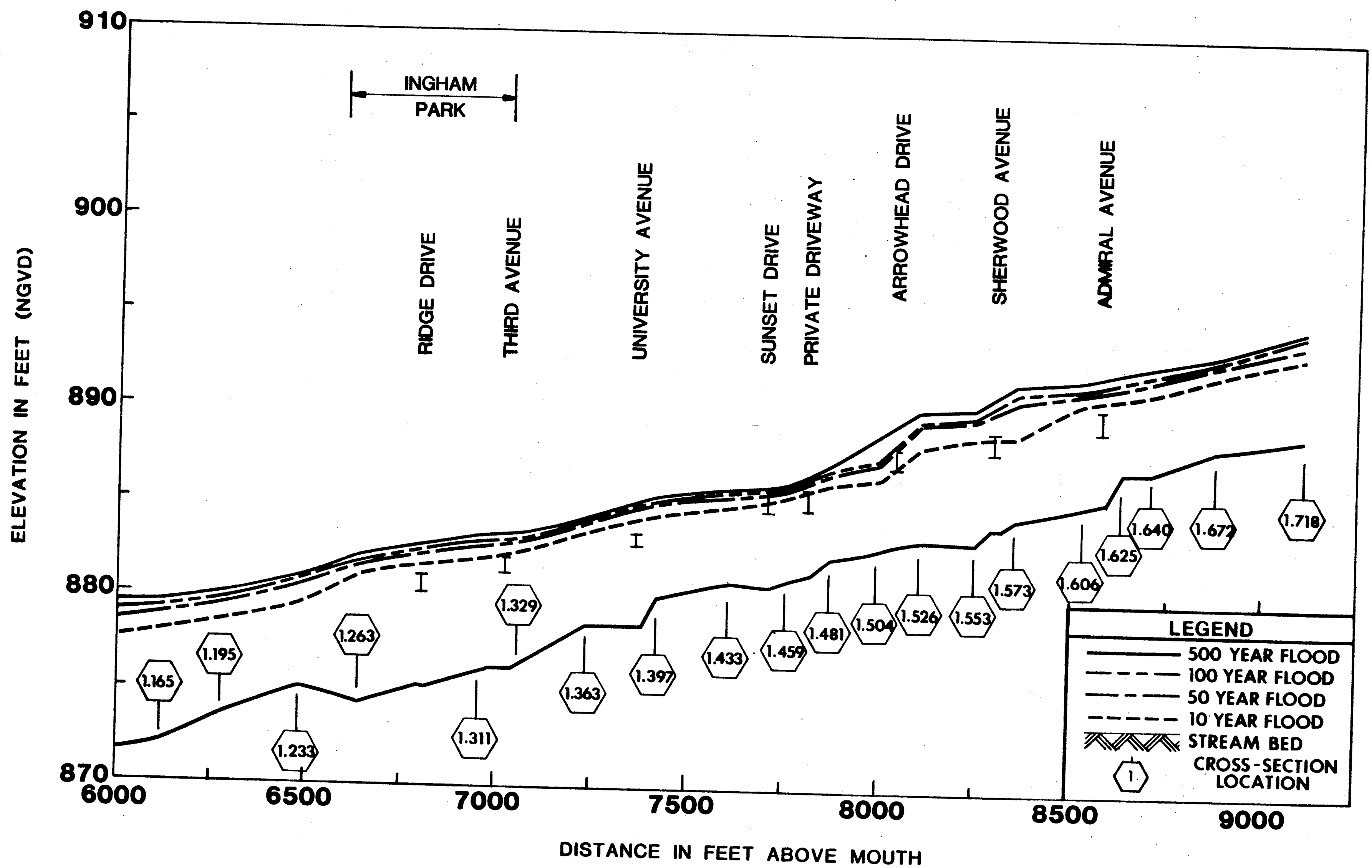


Figure 46. (Continued)

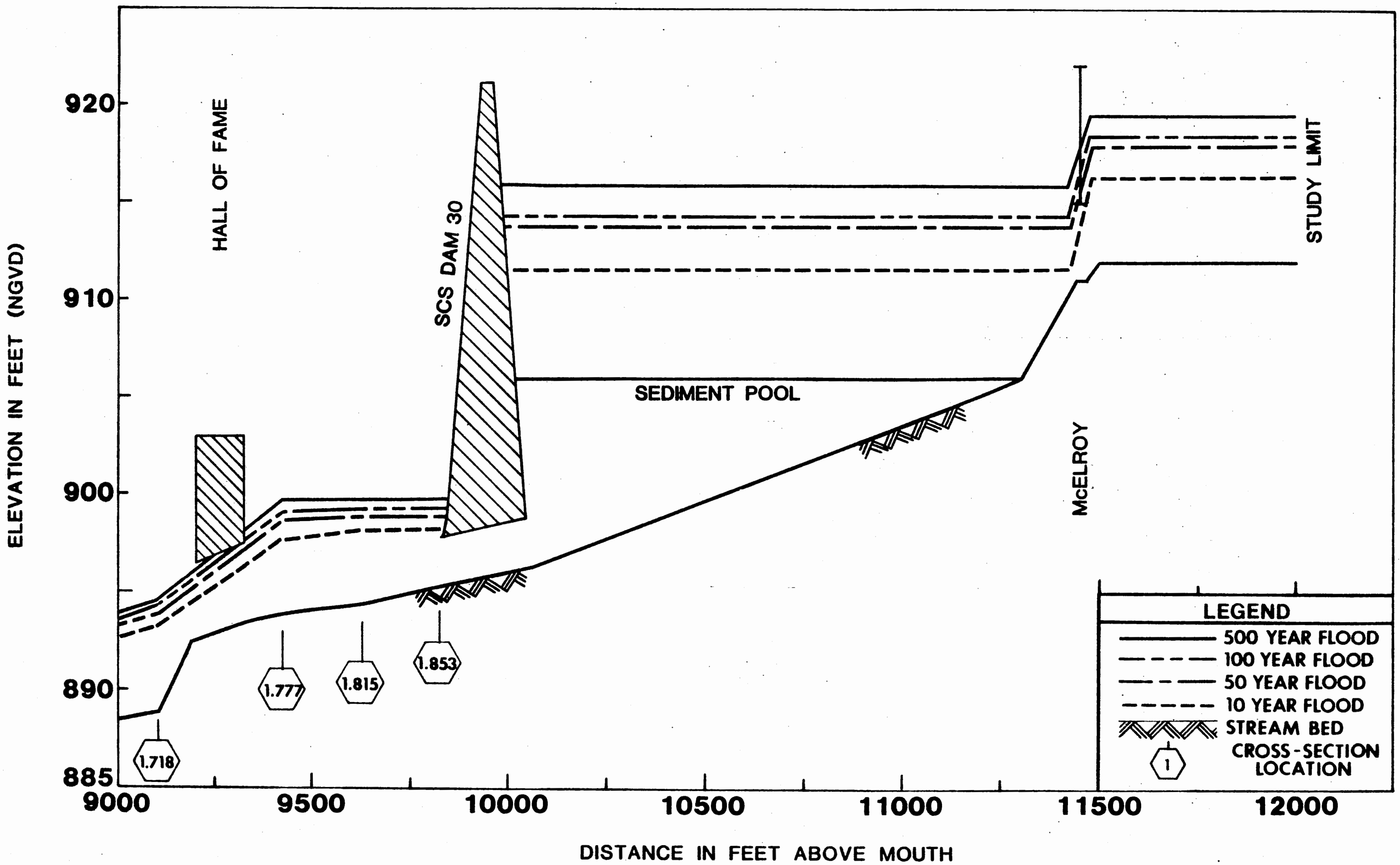


Figure 46. (Continued)

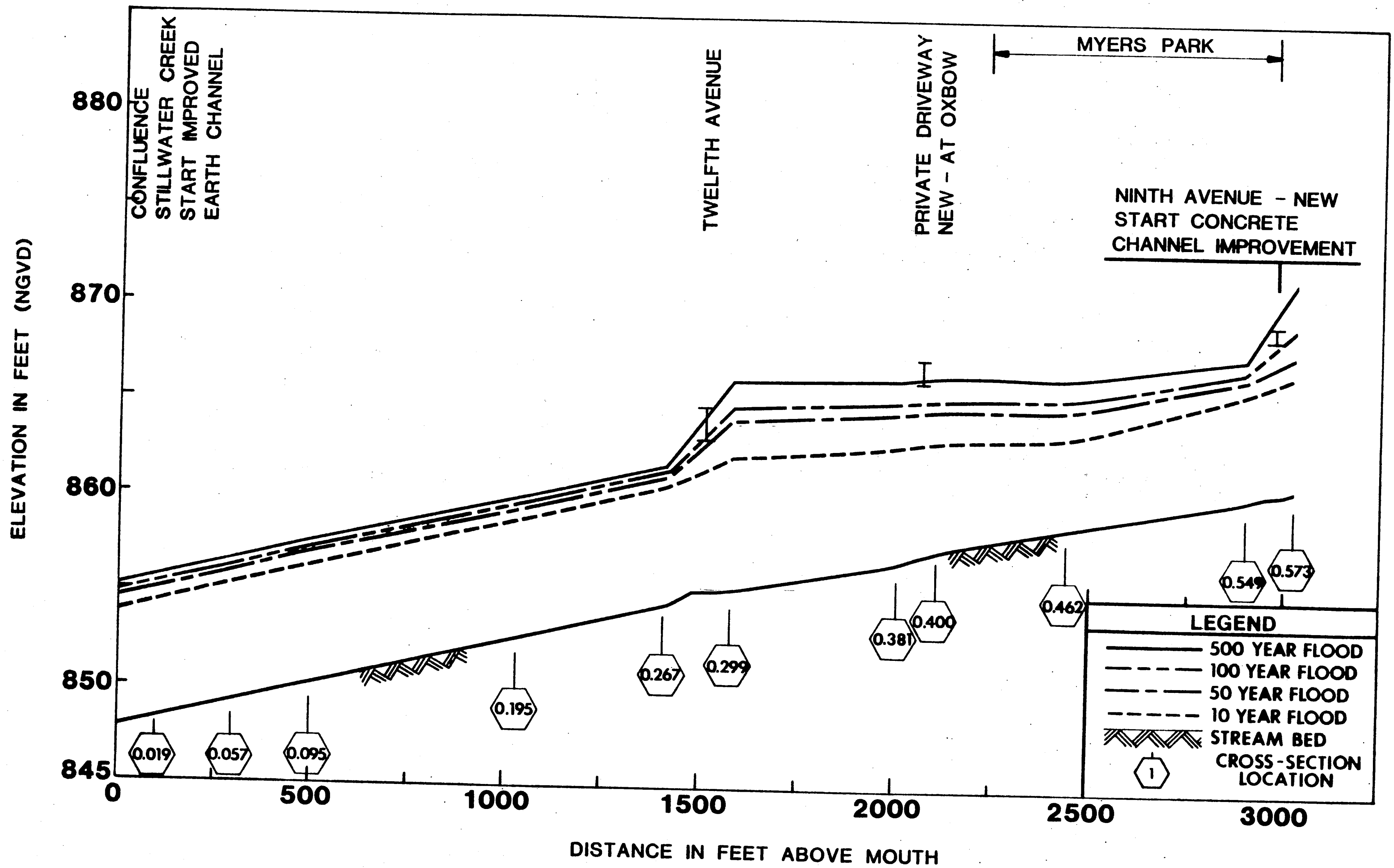


Figure 47. Water Surface Profile Plots for Improved Channel-Present Urbanization

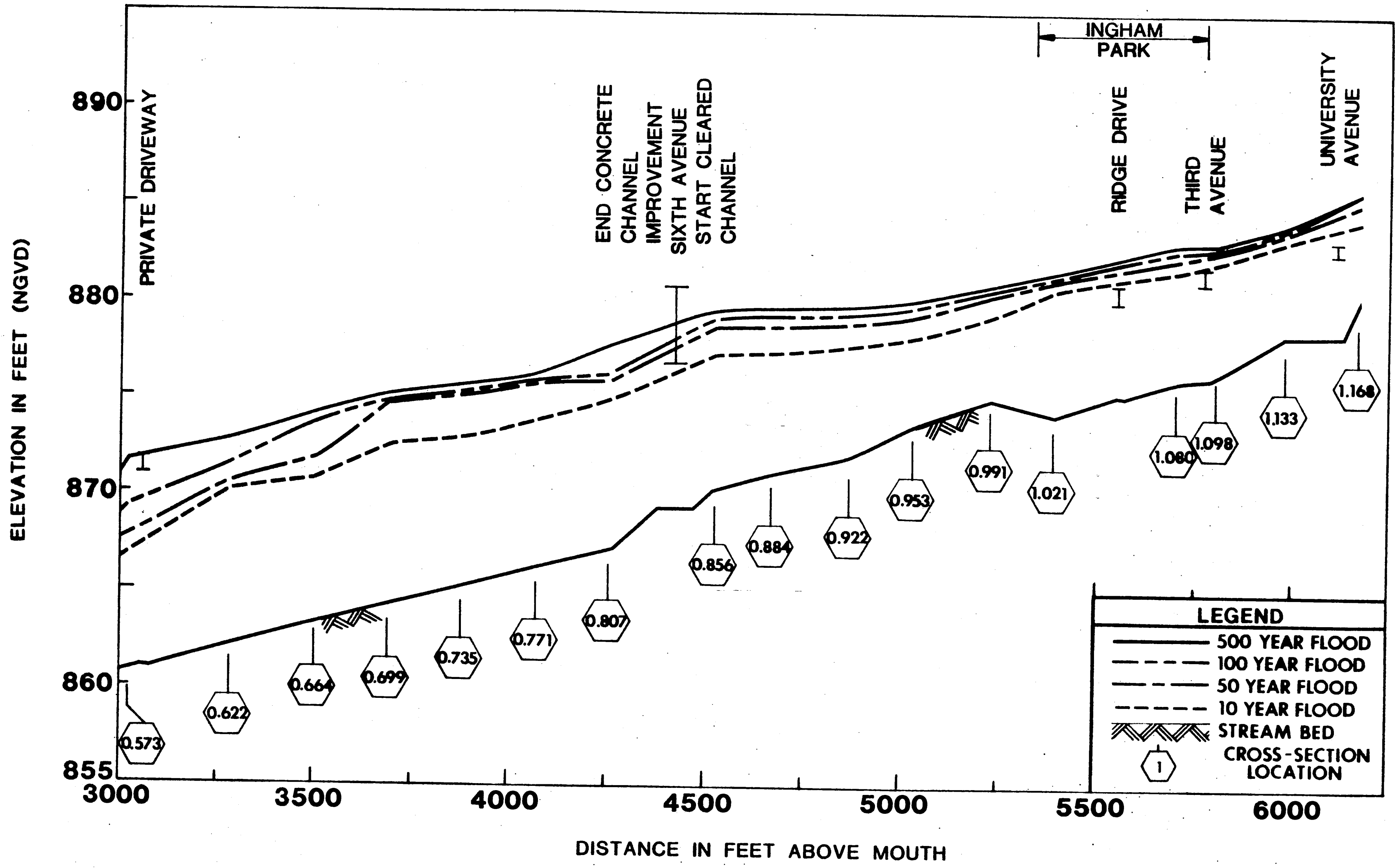


Figure 47. (Continued)

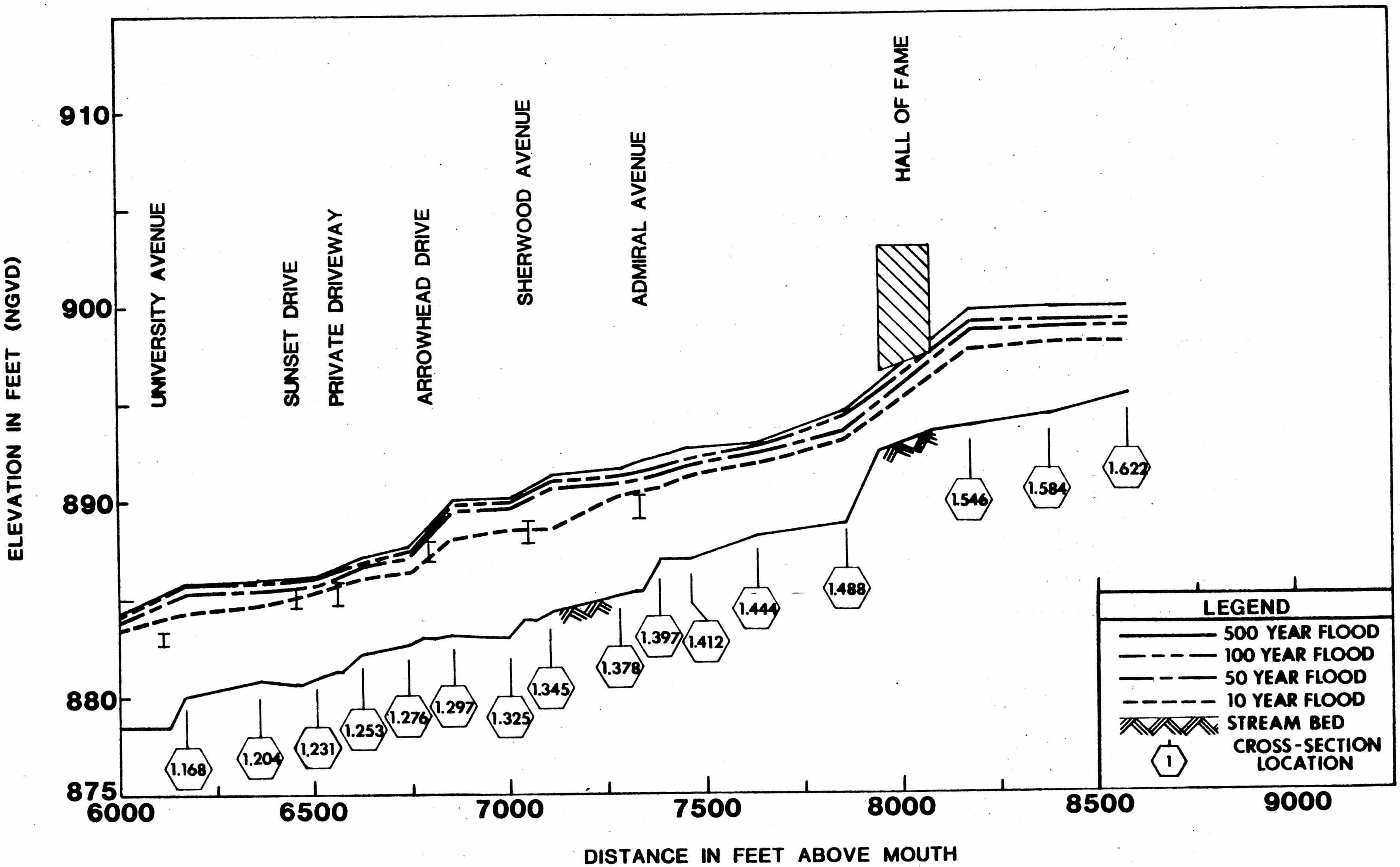


Figure 47. (Continued)

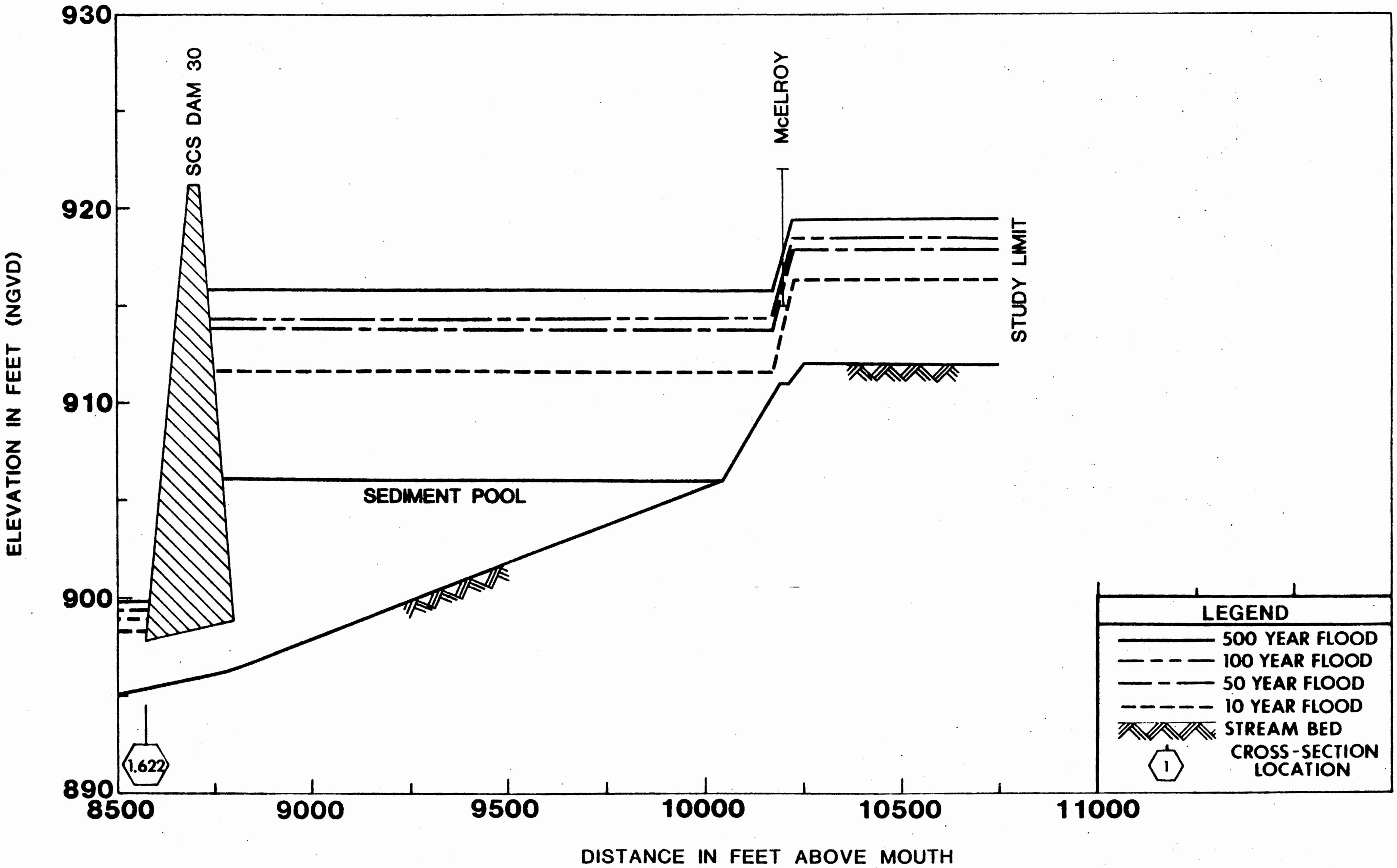


Figure 47. (Continued)

APPENDIX D

PROFILE SUMMARY TABLES

TABLE X

PROFILE SUMMARY TABLE FOR PRESENT CHANNEL, PRESENT URBANIZATION

CROSS SECTION NUMBER	DIST. (FT)	10-YR. FLOOD	50-YR. FLOOD	100-YEAR FLOOD				500-YEAR FLOOD				FLOODWAY						
		ELEV (FT)	ELEV (FT)	ELEV (FT)	CHVEL (FPS)	LEW (FT)	REW (FT)	ELEV (FT)	CHVEL (FPS)	LEW (FT)	REW (FT)	ELEV (FT)	CHVEL (FPS)	AREA (SQFT)	SRCH (FT)	LEW (FT)	REW (FT)	WIDTH (FT)
0.090	475.	853.68	854.85	855.24	7.64	1125.	1171.	855.80	8.15	1124.	1175.	856.24	6.33	268.	1.0	1125.	1171.	46.
0.178	940.	856.52	857.63	858.03	7.67	1239.	1299.	858.67	7.76	1234.	1301.	858.00	7.74	219.	0.0	1239.	1299.	60.
0.308	1630.	862.82	863.87	864.26	6.99	1470.	1529.	864.85	7.22	1466.	1530.	864.22	7.08	229.	0.0	1471.	1528.	57.
0.398	2100.	864.61	865.90	866.35	3.31	1470.	1580.	867.04	3.44	1296.	1595.	866.23	3.43	473.	0.0	1470.	1550.	80.
0.430	2270.	865.24	866.35	866.55	2.95	1371.	1551.	867.21	2.87	1234.	1564.	867.04	2.94	552.	0.5	1423.	1520.	97.
0.580	3060.	865.99	867.16	867.44	2.37	1205.	1825.	868.05	2.45	1125.	1825.	867.68	2.33	696.	0.2	1711.	1825.	114.
0.701	3700.	867.95	868.23	868.42	6.27	1228.	1435.	868.92	5.19	1219.	1435.	868.33	6.78	232.	0.1	1229.	1396.	167.
0.794	4190.	869.77	870.31	870.42	5.12	1183.	1335.	870.47	6.00	1181.	1341.	870.55	4.95	339.	0.1	1200.	1300.	100.
0.817	4315.	870.41	870.58	871.20	8.09	1408.	1525.	871.55	8.06	1407.	1590.	871.06	8.65	182.	0.0	1409.	1486.	77.
0.852	4500.	871.58	872.56	872.68	1.66	1159.	1830.	872.96	1.75	1155.	1839.	873.11	3.24	694.	0.4	1290.	1440.	150.
0.896	4730.	872.95	873.63	873.76	7.69	1094.	1330.	873.98	7.90	1093.	1330.	873.65	9.59	236.	0.0	1095.	1195.	100.
0.932	4920.	874.18	874.65	874.81	2.70	1364.	1736.	875.04	2.86	1364.	1761.	875.59	4.25	407.	0.8	1365.	1445.	80.
0.970	5120.	874.20	874.89	875.16	7.09	1245.	1743.	875.29	7.63	1245.	1746.	876.16	6.04	371.	1.0	1245.	1345.	100.
1.004	5300.	875.81	876.30	876.37	3.89	1243.	1490.	876.58	3.98	1242.	1536.	877.17	5.42	332.	0.8	1243.	1345.	102.
1.044	5510.	876.73	877.11	877.29	7.21	1145.	1366.	877.55	7.57	1145.	1410.	878.32	5.93	286.	1.0	1150.	1220.	70.
1.098	5770.	877.35	878.35	878.68	2.35	1275.	1348.	879.14	2.55	1274.	1410.	879.39	2.08	447.	0.7	1275.	1348.	73.
1.127	5920.	877.36	878.38	878.72	3.54	1272.	1521.	879.20	3.33	1269.	1555.	879.41	3.10	322.	0.7	1272.	1362.	90.
1.165	6120.	877.91	878.80	879.09	3.46	1434.	1629.	879.50	3.49	1386.	1630.	879.64	3.10	301.	0.6	1549.	1629.	80.
1.195	6280.	878.49	879.32	879.59	3.28	1255.	1785.	879.97	3.19	1254.	1786.	880.00	2.89	322.	0.4	1664.	1759.	95.
1.233	6480.	879.28	880.41	880.57	6.89	1806.	1941.	880.78	7.01	1794.	1949.	880.51	7.26	127.	0.1	1855.	1920.	65.

TABLE X (Continued)

CROSS SECTION NUMBER	DIST. (FT)	10-YR. FLOOD ELEV (FT)	50-YR. FLOOD ELEV (FT)	100-YEAR FLOOD ELEV (FT)	100-YEAR FLOOD CHVEL (FPS)	100-YEAR FLOOD LEW (FT)	100-YEAR FLOOD REW (FT)	500-YEAR FLOOD ELEV (FT)	500-YEAR FLOOD CHVEL (FPS)	500-YEAR FLOOD LEW (FT)	500-YEAR FLOOD REW (FT)	FLOODWAY ELEV (FT)	FLOODWAY CHVEL (FPS)	FLOODWAY AREA (SQFT)	FLOODWAY SRCH (FT)	FLOODWAY LEW (FT)	FLOODWAY REW (FT)	FLOODWAY WIDTH (FT)
1.263	6640.	881.10	881.62	881.79	6.40	1910.	2076.	882.03	6.88	1898.	2077.	881.91	6.39	153.	0.1	1925.	1985.	60.
1.311	6955.	881.87	882.60	882.85	2.03	1468.	1980.	883.16	2.04	1465.	1984.	883.11	2.40	596.	0.3	1670.	1980.	310.
1.329	7055.	882.23	882.63	882.87	6.94	1785.	1918.	883.19	6.65	1641.	1920.	883.14	6.36	136.	0.3	1863.	1918.	55.
1.363	7235.	883.46	883.89	884.00	4.02	1690.	1763.	884.18	4.25	1621.	1763.	884.18	3.80	134.	0.2	1708.	1763.	55.
1.397	7420.	884.46	885.12	885.27	2.62	1692.	1883.	885.29	3.06	1691.	1883.	885.36	2.68	189.	0.1	1709.	1765.	76.
1.433	7610.	884.84	885.42	885.59	3.65	1364.	1560.	885.73	3.78	1363.	1560.	885.51	4.63	110.	0.0	1472.	1532.	60.
1.459	7755.	885.43	885.66	885.80	2.75	1275.	1455.	885.94	2.94	1275.	1455.	886.28	2.30	162.	0.5	1375.	1440.	65.
1.481	7870.	886.19	886.78	886.90	2.76	1053.	1370.	887.06	2.77	1051.	1373.	887.34	4.03	155.	0.4	1200.	1300.	100.
1.504	7990.	886.40	887.13	887.46	10.10	1260.	1276.	888.58	7.11	1223.	1278.	887.48	10.03	48.	0.0	1260.	1276.	16.
1.526	8105.	888.12	889.45	889.50	3.75	1375.	1550.	890.01	3.23	1300.	1620.	889.83	3.38	142.	0.3	1479.	1530.	51.
1.553	8250.	888.67	889.60	889.68	2.53	1309.	1445.	890.12	2.27	1280.	1525.	889.96	2.55	158.	0.3	1384.	1436.	52.
1.573	8355.	888.75	890.64	891.05	3.18	1275.	1429.	891.51	3.02	1275.	1455.	890.98	3.99	146.	0.0	1379.	1429.	50.
1.606	8530.	890.67	891.09	891.41	4.81	1320.	1358.	891.81	4.99	1320.	1360.	891.44	4.79	101.	0.0	1328.	1358.	30.
1.625	8630.	890.91	891.51	891.85	5.64	1430.	1459.	892.28	5.76	1430.	1465.	891.66	6.12	66.	0.0	1432.	1459.	27.
1.640	8710.	891.62	892.21	892.47	4.04	1387.	1498.	892.90	3.84	1374.	1532.	892.39	4.68	87.	0.1	1432.	1460.	28.
1.672	8880.	892.17	892.71	892.89	5.13	1416.	1445.	893.17	5.55	1415.	1445.	893.00	4.95	82.	0.1	1416.	1445.	29.
1.718	9110.	893.25	893.92	894.40	5.42	1319.	1468.	894.55	5.46	1312.	1471.	894.14	7.87	52.	0.0	1405.	1430.	25.
1.777	9425.	897.69	898.73	899.08	2.40	1057.	1207.	899.74	1.93	1048.	1249.	899.04	3.16	129.	0.0	1075.	1115.	40.
1.815	9625.	898.16	898.95	899.24	0.42	1156.	1243.	899.83	0.37	1140.	1260.	899.32	0.44	114.	0.1	1175.	1215.	40.
1.853	9825.	898.19	898.96	899.25	0.66	1126.	1167.	899.84	0.60	1121.	1172.	899.33	0.63	79.	0.1	1126.	1165.	39.

TABLE XI

PROFILE SUMMARY TABLE FOR IMPROVED CHANNEL, PRESENT URBANIZATION

CROSS SECTION NUMBER	DIST. (FT)	10-YR. FLOOD	50-YR. FLOOD	100-YEAR FLOOD				500-YEAR FLOOD				FLOODWAY						
		ELEV (FT)	ELEV (FT)	ELEV (FT)	CHVEL (FPS)	LEW (FT)	REW (FT)	ELEV (FT)	CHVEL (FPS)	LEW (FT)	REW (FT)	ELEV (FT)	CHVEL (FPS)	AREA (SQFT)	SRCH (FT)	LEW (FT)	REW (FT)	WIDTH (FT)
10,019	100.	854.46	855.04	855.25	7.01	1214.	1296.	855.65	7.41	1213.	1297.	856.25	5.27	330.	1.0	1214.	1296.	82.
10,057	300.	855.42	856.01	856.21	7.11	1149.	1231.	856.60	7.55	1148.	1232.	856.62	6.24	279.	0.4	1149.	1231.	82.
10,095	500.	856.41	857.01	857.22	6.86	1039.	1130.	857.60	7.30	1038.	1130.	857.29	6.70	260.	0.1	1039.	1121.	82.
10,195	1030.	858.81	859.40	859.61	6.64	1454.	1536.	860.00	7.01	1453.	1537.	859.58	6.72	242.	0.0	1454.	1536.	82.
10,267	1410.	860.56	861.13	861.33	6.73	1445.	1529.	861.69	7.11	1443.	1531.	861.33	6.73	242.	0.0	1445.	1529.	84.
10,299	1580.	862.17	864.13	864.80	3.41	1416.	1518.	866.13	3.09	1373.	1543.	864.80	3.41	477.	0.0	1416.	1518.	102.
10,381	2010.	862.81	864.53	865.14	3.01	1243.	1805.	866.37	2.75	1181.	1805.	865.14	3.01	769.	0.0	1435.	1800.	365.
10,400	2110.	863.25	864.81	865.39	3.10	1439.	1825.	866.51	2.92	1205.	1825.	865.39	3.10	749.	0.0	1439.	1820.	381.
10,462	2440.	863.33	864.81	865.39	6.89	1193.	1277.	866.51	6.37	1190.	1300.	865.39	6.89	239.	0.0	1193.	1277.	84.
10,549	2900.	865.88	866.66	866.99	8.46	1234.	1283.	867.67	8.56	1232.	1284.	866.99	8.46	192.	0.0	1234.	1283.	49.
10,573	3025.	866.86	867.81	869.35	10.46	1419.	1446.	871.63	7.48	1409.	1590.	868.36	12.46	120.	0.0	1423.	1442.	19.
10,622	3285.	870.14	870.58	871.51	5.19	1185.	1750.	872.76	3.38	1156.	1833.	870.76	9.81	288.	0.0	1290.	1405.	115.
10,664	3505.	870.77	871.85	873.70	8.97	1095.	1330.	874.15	8.96	1093.	1418.	872.98	11.35	183.	0.0	1098.	1195.	97.
10,699	3690.	872.51	874.71	874.85	4.08	1365.	1740.	875.18	4.22	1364.	1767.	874.86	6.33	323.	0.0	1365.	1440.	75.
10,735	3880.	872.83	875.06	875.22	8.62	1245.	1744.	875.63	8.42	1245.	1775.	874.86	10.53	226.	0.0	1246.	1315.	69.
10,771	4070.	873.80	875.83	875.91	8.01	1245.	1485.	876.15	8.47	1244.	1490.	875.69	10.53	164.	0.0	1246.	1315.	69.
10,807	4260.	874.78	875.97	876.22	11.37	1161.	1197.	877.72	8.56	1145.	1410.	876.43	10.75	139.	0.2	1161.	1197.	36.
10,856	4520.	877.31	878.71	879.21	2.11	1273.	1443.	879.59	2.33	1269.	1695.	879.13	2.14	429.	0.0	1275.	1350.	75.
10,884	4670.	877.31	878.72	879.24	2.80	1269.	1555.	879.63	2.85	1267.	1640.	879.13	3.49	270.	0.0	1270.	1344.	74.
10,922	4870.	877.71	878.93	879.37	3.11	1400.	1630.	879.75	3.26	1355.	1648.	879.38	3.30	279.	0.0	1550.	1630.	80.
10,953	5030.	878.18	879.24	879.64	3.18	1255.	1785.	880.03	3.17	1253.	1787.	879.68	3.16	291.	0.0	1664.	1758.	94.

TABLE XI (Continued)

CROSS SECTION NUMBER	DIST. (FT)	10-YR. FLOOD	50-YR. FLOOD	100-YEAR FLOOD				500-YEAR FLOOD				FLOODWAY						
		ELEV (FT)	ELEV (FT)	ELEV (FT)	CHVEL (FPS)	LEW (FT)	REW (FT)	ELEV (FT)	CHVEL (FPS)	LEW (FT)	REW (FT)	ELEV (FT)	CHVEL (FPS)	AREA (SQFT)	SRCH (FT)	LEW (FT)	REW (FT)	WIDTH (FT)
10,991	5230.	879.26	880.37	880.60	6.91	1804.	1943.	880.84	7.14	1791.	1952.	880.41	7.85	120.	0.0	1855.	1920.	65.
11,021	5390.	880.77	881.23	881.33	7.18	1956.	1985.	881.60	8.11	1920.	2075.	881.45	7.00	131.	0.1	1956.	1985.	29.
11,080	5705.	881.70	882.43	882.65	2.33	1471.	1977.	883.13	2.27	1465.	1984.	882.88	2.79	535.	0.2	1670.	1975.	305.
11,098	5805.	882.19	882.68	882.86	7.18	1785.	1918.	883.17	7.18	1641.	1918.	882.93	7.21	124.	0.1	1863.	1918.	55.
11,133	5985.	883.46	883.83	883.93	4.37	1690.	1763.	884.15	4.64	1623.	1763.	884.15	3.94	132.	0.2	1708.	1763.	55.
11,168	6170.	884.43	885.36	885.78	2.14	1685.	1885.	885.79	2.56	1685.	1885.	885.36	2.71	189.	0.0	1709.	1785.	76.
11,204	6360.	884.76	885.52	885.89	3.15	1361.	1560.	885.94	3.63	1360.	1560.	885.43	5.06	99.	0.0	1479.	1532.	53.
11,231	6505.	885.42	885.72	886.01	2.73	1275.	1455.	886.11	3.10	1275.	1455.	886.30	2.62	163.	0.3	1375.	1440.	65.
11,253	6620.	886.10	886.77	886.88	2.86	1054.	1370.	887.06	2.87	1051.	1373.	887.27	4.31	148.	0.4	1200.	1300.	100.
11,276	6740.	886.41	887.13	887.43	10.08	1261.	1276.	888.63	7.07	1224.	1276.	887.39	10.23	47.	0.0	1261.	1276.	15.
11,297	6855.	888.09	889.58	889.83	3.15	1309.	1620.	890.03	3.38	1300.	1620.	889.76	3.52	138.	0.0	1479.	1530.	51.
11,325	7000.	888.58	889.70	889.93	2.58	1287.	1456.	890.14	2.76	1280.	1525.	889.89	2.73	155.	0.0	1384.	1436.	52.
11,345	7105.	888.58	890.66	891.02	3.56	1275.	1429.	891.41	3.58	1275.	1430.	890.88	4.42	141.	0.0	1379.	1429.	50.
11,378	7280.	890.38	890.93	891.24	5.04	1320.	1358.	891.60	5.37	1320.	1359.	891.21	5.09	94.	0.0	1328.	1358.	30.
11,397	7380.	890.64	891.38	891.71	5.98	1430.	1459.	892.15	6.05	1430.	1465.	891.49	6.57	62.	0.0	1432.	1458.	26.
11,412	7460.	891.36	891.96	892.22	4.68	1395.	1477.	892.66	4.55	1381.	1513.	892.11	5.13	79.	0.0	1432.	1460.	28.
11,444	7630.	891.87	892.40	892.59	5.75	1417.	1444.	892.90	6.13	1416.	1445.	892.64	5.66	72.	0.0	1417.	1444.	27.
11,488	7860.	893.03	893.55	894.42	5.63	1318.	1468.	894.54	5.95	1312.	1471.	893.76	9.13	44.	0.0	1406.	1423.	17.
11,546	8175.	897.61	898.66	899.02	2.63	1058.	1204.	899.71	2.18	1048.	1247.	898.97	3.23	126.	0.0	1075.	1115.	40.
11,584	8375.	898.00	898.86	899.18	0.44	1158.	1241.	899.80	0.40	1141.	1259.	899.22	0.46	110.	0.0	1175.	1215.	40.
11,622	8575.	898.02	898.87	899.18	0.68	1127.	1165.	899.80	0.61	1121.	1171.	899.22	0.67	75.	0.0	1127.	1165.	38.

TABLE XII

PROFILE SUMMARY TABLE FOR PRESENT CHANNEL, FUTURE URBANIZATION

CROSS SECTION NUMBER	DIST. (FT)	10-YR. FLOOD	50-YR. FLOOD	100-YEAR FLOOD				500-YEAR FLOOD				FLOODWAY						
		ELEV (FT)	ELEV (FT)	ELEV (FT)	CHVEL (FPS)	LEW (FT)	REW (FT)	ELEV (FT)	CHVEL (FPS)	LEW (FT)	REW (FT)	ELEV (FT)	CHVEL (FPS)	AREA (SQFT)	SRCH (FT)	LEW (FT)	REW (FT)	WIDTH (FT)
0.090	475.	853.94	855.10	855.37	7.81	1125.	1172.	856.01	8.37	1124.	1176.	856.37	6.47	275.	1.0	1125.	1172.	47.
0.178	940.	856.75	857.89	858.21	7.71	1238.	1300.	858.94	7.77	1232.	1302.	858.19	7.70	231.	0.0	1238.	1300.	62.
0.308	1630.	863.05	864.12	864.44	7.07	1469.	1529.	865.00	7.48	1465.	1530.	864.43	7.09	242.	0.0	1469.	1529.	60.
0.398	2100.	864.90	866.20	866.57	3.36	1469.	1584.	867.29	3.50	1271.	1600.	866.46	3.50	491.	0.0	1470.	1550.	80.
0.430	2270.	865.66	866.47	866.74	2.96	1371.	1555.	867.47	2.84	1234.	1570.	867.24	3.00	571.	0.5	1423.	1520.	97.
0.580	3060.	866.37	867.33	867.62	2.41	1125.	1825.	868.28	2.46	1125.	1825.	867.89	2.38	720.	0.3	1710.	1825.	115.
0.701	3700.	867.94	868.33	868.59	5.83	1225.	1435.	869.10	5.02	1216.	1535.	868.55	6.04	271.	0.0	1225.	1396.	171.
0.794	4190.	869.96	870.41	870.41	5.43	1184.	1335.	870.56	6.26	1178.	1395.	870.51	5.30	336.	0.1	1200.	1300.	100.
0.817	4315.	870.54	871.11	871.32	8.04	1408.	1525.	871.61	8.30	1407.	1590.	871.10	8.96	185.	0.0	1408.	1486.	78.
0.852	4500.	871.87	872.62	872.76	1.69	1156.	1833.	873.07	1.78	1155.	1865.	873.32	3.25	725.	0.6	1290.	1440.	150.
0.896	4730.	873.19	873.72	873.82	7.77	1094.	1330.	874.05	7.98	1093.	1390.	873.81	9.53	252.	0.0	1095.	1195.	100.
0.932	4920.	874.31	874.76	874.88	2.74	1364.	1745.	875.12	2.91	1364.	1765.	875.60	3.74	472.	0.7	1364.	1505.	141.
0.970	5120.	874.31	875.13	875.21	7.18	1245.	1744.	875.41	7.35	1245.	1748.	876.05	6.59	359.	0.8	1245.	1345.	100.
1.004	5300.	875.96	876.33	876.43	3.92	1243.	1490.	876.61	4.15	1242.	1538.	877.25	5.41	351.	0.8	1243.	1350.	107.
1.044	5510.	876.75	877.24	877.36	7.36	1145.	1368.	877.65	7.63	1145.	1410.	878.40	6.02	306.	1.0	1150.	1230.	80.
1.098	5770.	877.58	878.59	878.83	2.45	1275.	1349.	879.33	2.67	1272.	1495.	879.53	2.17	459.	0.7	1275.	1349.	74.
1.127	5920.	877.59	878.63	878.88	3.51	1271.	1540.	879.40	3.31	1268.	1555.	879.56	3.18	334.	0.7	1271.	1361.	90.
1.165	6120.	878.16	879.02	879.23	3.50	1417.	1629.	879.68	3.54	1364.	1644.	879.79	3.19	314.	0.6	1549.	1629.	80.
1.195	6280.	878.74	879.54	879.74	3.27	1255.	1785.	880.15	3.11	1250.	1790.	880.15	2.96	337.	0.4	1660.	1759.	99.
1.233	6480.	879.50	880.55	880.66	6.94	1801.	1945.	880.91	6.95	1787.	1954.	880.57	7.53	130.	0.0	1855.	1920.	65.

TABLE XII (Continued)

CROSS SECTION NUMBER	DIST. (FT)	10-YR. FLOOD	50-YR. FLOOD	100-YEAR FLOOD				500-YEAR FLOOD				FLOODWAY						
		ELEV (FT)	ELEV (FT)	ELEV (FT)	CHVEL (FPS)	LEW (FT)	REW (FT)	ELEV (FT)	CHVEL (FPS)	LEW (FT)	REW (FT)	ELEV (FT)	CHVEL (FPS)	AREA (SQFT)	SRCH (FT)	LEW (FT)	REW (FT)	WIDTH (FT)
1.263	6640.	881.36	881.77	881.89	6.61	1905.	2076.	882.14	7.17	1893.	2077.	882.07	6.58	163.	0.2	1925.	1985.	60.
1.311	6955.	882.16	882.81	882.98	2.03	1465.	1982.	883.33	2.04	1465.	1986.	883.29	2.35	644.	0.3	1670.	1980.	310.
1.329	7055.	882.41	882.84	883.01	6.77	1643.	1918.	883.36	6.53	1640.	1922.	883.32	6.21	146.	0.3	1863.	1918.	55.
1.363	7235.	883.55	883.98	884.06	4.26	1627.	1763.	884.27	4.45	1616.	1763.	884.30	3.93	140.	0.2	1708.	1763.	55.
1.397	7420.	884.77	885.27	885.29	2.85	1691.	1883.	885.30	3.37	1691.	1883.	885.56	2.70	203.	0.3	1709.	1785.	76.
1.433	7610.	885.10	885.60	885.67	3.72	1363.	1560.	885.84	3.88	1362.	1560.	885.68	4.57	121.	0.0	1472.	1532.	60.
1.459	7755.	885.45	885.81	885.88	2.82	1275.	1455.	886.05	3.00	1275.	1455.	886.36	2.39	167.	0.5	1375.	1440.	65.
1.481	7870.	886.45	886.89	886.98	2.76	1052.	1372.	887.16	2.77	1050.	1375.	887.43	4.12	163.	0.5	1200.	1300.	100.
1.504	7990.	886.66	887.43	887.63	10.34	1260.	1276.	888.67	7.36	1224.	1279.	887.67	10.22	51.	0.0	1260.	1276.	16.
1.526	8105.	888.70	889.99	890.40	2.40	1300.	1620.	890.41	2.84	1300.	1620.	890.29	3.00	165.	0.0	1479.	1530.	51.
1.553	8250.	889.10	890.07	890.46	1.72	1280.	1525.	890.49	2.00	1280.	1525.	890.37	2.33	180.	0.0	1384.	1436.	52.
1.573	8355.	889.93	890.43	890.50	4.64	1275.	1429.	890.55	5.37	1275.	1429.	890.83	4.58	139.	0.3	1379.	1429.	50.
1.606	8530.	890.63	891.13	891.26	5.47	1320.	1358.	891.51	6.01	1320.	1359.	891.44	5.19	101.	0.2	1328.	1358.	30.
1.625	8630.	890.97	891.64	891.82	6.34	1430.	1459.	892.20	6.47	1430.	1465.	891.70	6.70	67.	0.0	1432.	1459.	27.
1.640	8710.	891.83	892.39	892.61	4.17	1383.	1509.	892.99	4.00	1371.	1539.	892.56	4.93	91.	0.0	1432.	1460.	28.
1.672	8880.	892.39	892.87	893.03	5.44	1416.	1445.	893.28	5.86	1415.	1446.	893.18	5.17	87.	0.2	1416.	1445.	29.
1.718	9110.	893.52	894.39	894.46	5.57	1316.	1469.	894.73	4.69	1302.	1475.	894.26	8.34	55.	0.0	1405.	1430.	25.
1.777	9425.	898.09	899.08	899.46	2.12	1052.	1231.	900.02	1.81	1044.	1265.	899.41	3.15	144.	0.0	1075.	1115.	40.
1.815	9625.	898.46	899.24	899.57	0.39	1147.	1252.	900.09	0.32	1133.	1266.	899.68	0.43	129.	0.1	1175.	1215.	40.
1.853	9825.	898.48	899.25	899.58	0.62	1124.	1169.	900.09	0.54	1118.	1175.	899.68	0.59	93.	0.1	1125.	1165.	40.

TABLE XIII

PROFILE SUMMARY TABLE FOR IMPROVED CHANNEL, FUTURE URBANIZATION

CROSS SECTION NUMBER	DIST. (FT)	10-YR. FLOOD	50-YR. FLOOD	100-YEAR FLOOD				500-YEAR FLOOD				FLOODWAY						
		ELEV (FT)	ELEV (FT)	ELEV (FT)	CHVEL (FPS)	LEW (FT)	REW (FT)	ELEV (FT)	CHVEL (FPS)	LEW (FT)	REW (FT)	ELEV (FT)	CHVEL (FPS)	AREA (SQFT)	SRCH (FT)	LEW (FT)	REW (FT)	WIDTH (FT)
10,019	100.	854,57	855,18	855,38	7.16	1214.	1296.	855,79	7.56	1213.	1297.	856.38	5.43	341.	1.0	1214.	1296.	82.
10,057	300.	855,54	856,14	856,34	7.26	1149.	1231.	856,73	7.69	1148.	1232.	856.76	6.38	290.	0.4	1149.	1231.	82.
10,095	500.	856,53	857,14	857,34	7.01	1039.	1130.	857,73	7.45	1037.	1130.	857.42	6.84	270.	0.1	1039.	1122.	83.
10,195	1030.	858,93	859,53	859,74	6.78	1454.	1536.	860,13	7.15	1453.	1537.	859.71	6.86	252.	0.0	1454.	1536.	82.
10,267	1410.	860,69	861.26	861.46	6.86	1444.	1530.	861,83	7.25	1443.	1532.	861.46	6.65	253.	0.0	1444.	1530.	86.
10,299	1580.	862,57	864,58	865,22	3.32	1415.	1524.	866,59	2.98	1371.	1552.	865.23	3.32	521.	0.0	1415.	1520.	105.
10,381	2010.	863,15	864,94	865,53	2.94	1223.	1805.	866,61	2.66	1159.	1805.	865.54	2.94	833.	0.0	1433.	1800.	367.
10,400	2110.	863,56	865,19	865.76	3.04	1438.	1825.	866.93	2.86	1205.	1825.	865.76	3.05	809.	0.0	1438.	1820.	382.
10,462	2440.	863.60	865.19	865.76	6.73	1192.	1277.	866.93	6.11	1188.	1320.	865.76	6.73	260.	0.0	1192.	1277.	85.
10,549	2900.	866.03	866.88	867,21	8.53	1234.	1283.	867,90	8.62	1231.	1285.	867.20	8.54	203.	0.0	1234.	1283.	49.
10,573	3025.	867,20	868,79	871,00	7.96	1412.	1486.	872,07	6.91	1407.	1590.	867.72	14.63	109.	0.0	1425.	1441.	16.
10,622	3285.	870,19	871,20	872,30	3.69	1173.	1819.	872,97	3.29	1155.	1839.	871.61	7.54	407.	0.0	1290.	1440.	150.
10,664	3505.	871,00	873,58	873,82	9.12	1094.	1330.	874,26	9.09	1092.	1447.	873.25	11.38	210.	0.0	1096.	1195.	99.
10,699	3690.	873,28	874,79	874,98	4.10	1364.	1757.	875,29	4.26	1364.	1772.	875.10	6.38	342.	0.1	1364.	1440.	76.
10,735	3880.	873,14	875,26	875,35	8.67	1245.	1747.	875,64	8.85	1245.	1775.	875.10	10.62	242.	0.0	1246.	1315.	69.
10,771	4070.	874,03	875,86	875,99	8.16	1245.	1490.	876,23	8.60	1244.	1490.	875.90	10.59	178.	0.0	1245.	1315.	70.
10,807	4260.	875,11	876,19	876,80	10.38	1145.	1199.	877,80	8.77	1145.	1410.	876.58	11.02	145.	0.0	1160.	1198.	38.
10,856	4520.	877,62	879,03	879,42	2.21	1271.	1535.	879,82	2.35	1267.	1696.	879.48	2.21	455.	0.1	1275.	1350.	75.
10,884	4670.	877,63	879,06	879,46	2.79	1268.	1555.	879,85	2.80	1266.	1641.	879.48	3.49	294.	0.0	1268.	1343.	75.
10,922	4870.	878,02	879,22	879,58	3.15	1375.	1639.	879,96	3.26	1330.	1658.	879.70	3.29	305.	0.1	1550.	1630.	80.
10,953	5030.	878,47	879,52	879,85	3.15	1254.	1786.	880,23	3.05	1247.	1792.	879.99	3.13	321.	0.1	1660.	1759.	99.

TABLE XIII (Continued)

CROSS SECTION NUMBER	DIST. (FT)	10-YR. FLOOD	50-YR. FLOOD	100-YEAR FLOOD				500-YEAR FLOOD				FLOODWAY						
		ELEV (FT)	ELEV (FT)	ELEV (FT)	CHVEL (FPS)	LEW (FT)	REW (FT)	ELEV (FT)	CHVEL (FPS)	LEW (FT)	REW (FT)	ELEV (FT)	CHVEL (FPS)	AREA (SQFT)	SRCH (FT)	LEW (FT)	REW (FT)	WIDTH (FT)
10.991	5230.	879.44	880.58	880.72	7.02	1798.	1947.	880.95	7.21	1784.	1956.	880.56	7.84	130.	0.0	1855.	1920.	65.
11.021	5390.	881.02	881.32	881.46	7.63	1956.	1985.	881.72	8.49	1914.	2076.	881.53	7.52	134.	0.1	1956.	1985.	29.
11.080	5705.	881.97	882.63	882.88	2.31	1467.	1980.	883.34	2.25	1465.	1987.	883.08	2.79	587.	0.2	1670.	1975.	305.
11.098	5805.	882.38	882.84	882.92	7.55	1785.	1918.	883.38	6.93	1640.	1922.	883.13	7.08	135.	0.2	1863.	1918.	55.
11.133	5985.	883.52	883.91	884.08	4.49	1626.	1763.	884.20	4.94	1620.	1763.	884.29	4.11	140.	0.2	1708.	1763.	55.
11.168	6170.	884.78	885.94	885.95	2.22	1685.	1885.	885.96	2.62	1685.	1685.	885.60	2.74	206.	0.0	1709.	1785.	76.
11.204	6360.	885.03	886.03	886.06	3.17	1360.	1560.	886.11	3.62	1360.	1560.	885.63	4.99	110.	0.0	1479.	1532.	53.
11.231	6505.	885.51	886.13	886.18	2.73	1275.	1455.	886.27	3.10	1275.	1455.	886.44	2.70	172.	0.3	1375.	1440.	65.
11.253	6620.	886.40	886.88	886.97	2.85	1052.	1372.	887.15	2.89	1050.	1374.	887.39	4.36	159.	0.4	1200.	1300.	100.
11.276	6740.	886.68	887.43	887.63	10.35	1260.	1276.	888.68	7.41	1224.	1279.	887.67	10.23	51.	0.0	1260.	1276.	16.
11.297	6855.	888.62	889.83	890.42	2.53	1300.	1620.	890.43	3.00	1300.	1620.	890.31	3.04	166.	0.0	1479.	1530.	51.
11.325	7000.	889.02	889.93	890.47	2.15	1280.	1525.	890.50	2.52	1280.	1595.	890.39	2.45	181.	0.0	1384.	1436.	52.
11.345	7105.	889.50	890.30	890.51	5.04	1275.	1429.	890.56	5.84	1275.	1429.	890.74	5.09	134.	0.2	1379.	1429.	50.
11.378	7280.	890.35	890.82	890.99	5.97	1328.	1357.	891.19	6.66	1320.	1358.	891.17	5.64	93.	0.2	1328.	1357.	29.
11.397	7380.	890.80	891.44	891.66	6.78	1430.	1459.	892.04	6.88	1430.	1465.	891.61	6.94	65.	0.0	1432.	1459.	27.
11.412	7460.	891.59	892.13	892.35	4.90	1391.	1488.	892.74	4.78	1379.	1519.	892.29	5.36	84.	0.0	1432.	1460.	28.
11.444	7630.	892.09	892.56	892.73	6.07	1417.	1444.	892.99	6.49	1416.	1445.	892.82	5.87	77.	0.1	1417.	1444.	27.
11.488	7860.	893.25	894.33	894.50	5.79	1314.	1470.	894.58	6.23	1310.	1472.	894.07	9.00	50.	0.0	1405.	1430.	25.
11.546	8175.	898.02	899.02	899.41	2.36	1052.	1228.	899.98	2.07	1044.	1264.	899.35	3.21	141.	0.0	1075.	1115.	40.
11.584	8375.	898.33	899.18	899.53	0.41	1148.	1251.	900.06	0.35	1134.	1266.	899.58	0.44	125.	0.1	1175.	1215.	40.
11.622	8575.	898.35	899.18	899.53	0.63	1125.	1168.	900.06	0.55	1118.	1175.	899.58	0.62	89.	0.1	1125.	1165.	40.

APPENDIX E

COMPARISON OF FLOOD WATER SURFACE ELEVATIONS

TABLE XIV

COMPARISON OF 10-YEAR WATER SURFACE ELEVATIONS

LOCATION	CHANNEL CROSS SECTION NUMBER		WATER SURFACE ELEVATION (FEET,NGVD)			
			CHANNEL CONDITION			
			PRESENT		IMPROVED	
			URBANIZATION		URBANIZATION	
	PRESENT	IMPROVED	PRESENT	FUTURE	PRESENT	FUTURE
CONFLUENCE	0.090	10.019	853.68	853.94	854.46	854.57
	******	10.057	******	******	855.42	855.54
	0.178	10.095	856.52	856.75	856.41	856.53
	0.308	10.195	862.82	863.05	858.81	858.93
TWELVETH AVENUE	0.398	10.267	864.61	864.90	860.56	860.69
	0.430	10.299	865.24	865.66	862.17	862.57
OXBOW-WILLIS AVE	0.580	10.381	865.99	866.37	862.81	863.15
	******	10.400	******	******	863.25	863.56
MEYERS PARK	0.701	10.462	867.95	867.94	863.33	863.60
NINTH AVENUE	0.794	10.549	869.77	869.96	865.88	866.03
	0.817	10.573	870.41	870.54	866.86	867.20
	0.852	10.622	871.58	871.87	870.14	870.19
	0.896	10.664	872.95	873.19	870.77	871.00
	0.932	10.699	874.18	874.31	872.51	873.28
	0.970	10.735	874.20	874.31	872.83	873.26
	1.004	10.771	875.81	875.96	873.80	874.03
SIXTH AVENUE	1.044	10.807	876.63	876.75	874.78	875.11
	1.098	10.856	877.35	877.58	877.31	877.62
	1.127	10.884	877.36	877.59	877.31	877.63
	1.165	10.922	877.91	878.16	877.71	878.02

NOTE: ****** INDICATES CROSS SECTION NOT USED AT THAT LOCATION

TABLE XIV (Continued)

LOCATION	CHANNEL CROSS SECTION NUMBER		WATER SURFACE ELEVATION (FEET,NGVD)			
			CHANNEL CONDITION			
			PRESENT		IMPROVED	
			URBANIZATION		URBANIZATION	
	PRESENT	IMPROVED	PRESENT	FUTURE	PRESENT	FUTURE
	1.195	10.953	878.49	878.74	878.18	878.47
	1.233	10.991	879.28	879.50	879.26	879.44
RIDGE RD-INGHAM PK	1.263	11.021	881.10	881.36	880.77	881.02
THIRD AVENUE	1.311	11.080	881.87	882.16	881.70	881.97
	1.329	11.098	882.23	882.41	882.19	882.38
UNIVERSITY AVENUE	1.363	11.133	883.46	883.55	883.46	883.52
	1.397	11.168	884.46	884.77	884.43	884.78
SUNSET DRIVE	1.433	11.204	884.84	885.10	884.76	885.03
	1.459	11.231	885.43	885.45	885.42	885.51
	1.481	11.253	886.19	886.45	886.10	886.40
ARROWHEAD DRIVE	1.504	11.276	886.40	886.66	886.41	886.68
	1.526	11.297	888.12	888.70	888.09	888.62
SHERWOOD AVENUE	1.553	11.325	888.67	889.10	888.58	889.02
	1.573	11.345	888.75	889.93	888.58	889.50
ADMIRAL AVENUE	1.606	11.378	890.67	890.63	890.38	890.35
	1.625	11.397	890.91	890.97	890.64	890.80
	1.640	11.412	891.62	891.83	891.36	891.59
	1.672	11.444	892.17	892.39	891.87	892.09
HALL OF FAME	1.718	11.488	893.25	893.52	893.03	893.25
	1.777	11.546	897.69	898.09	897.61	898.02
	1.815	11.584	898.16	898.46	898.00	898.33
TOE OF SCS DAM 30	1.853	11.622	898.19	898.48	898.02	898.35

TABLE XV

COMPARISON OF 50-YEAR WATER SURFACE ELEVATIONS

LOCATION	CHANNEL CROSS SECTION NUMBER		WATER SURFACE ELEVATION (FEET,NGVD)			
			CHANNEL CONDITION			
			PRESENT		IMPROVED	
			URBANIZATION		URBANIZATION	
	PRESENT	IMPROVED	PRESENT	FUTURE	PRESENT	FUTURE
CONFLUENCE	0.090	10.019	854.85	855.10	855.04	855.18
	******	10.057	******	******	856.01	856.14
	0.178	10.095	857.63	857.89	857.01	857.14
	0.308	10.195	863.87	864.12	859.40	859.53
TWELVETH AVENUE	0.398	10.267	865.90	866.20	861.13	861.26
	0.430	10.299	866.35	866.47	864.13	864.58
OXBOW-WILLIS AVE	0.580	10.381	867.16	867.33	864.53	864.94
	******	10.400	******	******	864.81	865.19
MEYERS PARK	0.701	10.462	868.23	868.33	864.81	865.19
NINTH AVENUE	0.794	10.549	870.31	870.41	866.66	866.88
	0.817	10.573	870.58	871.11	867.81	868.79
	0.852	10.622	872.56	872.62	870.58	871.20
	0.896	10.664	873.63	873.72	871.85	873.58
	0.932	10.699	874.65	874.76	874.71	874.79
	0.970	10.735	874.89	875.13	875.06	875.26
	1.004	10.771	876.30	876.33	875.83	875.86
SIXTH AVENUE	1.044	10.807	877.11	877.24	875.97	876.19
	1.098	10.856	878.35	878.59	878.71	879.03
	1.127	10.884	878.38	878.63	878.72	879.06
	1.165	10.922	878.80	879.02	878.93	879.22

NOTE: ****** INDICATES CROSS SECTION NOT USED AT THAT LOCATION

TABLE XV (Continued)

LOCATION	CHANNEL CROSS SECTION NUMBER		WATER SURFACE ELEVATION (FEET,NGVD)			
			CHANNEL CONDITION			
			PRESENT		IMPROVED	
			URBANIZATION		URBANIZATION	
	PRESENT	IMPROVED	PRESENT	FUTURE	PRESENT	FUTURE
	1.195	10.953	879.32	879.54	879.24	879.52
	1.233	10.991	880.41	880.55	880.37	880.58
RIDGE RD-INGHAM PK	1.263	11.021	881.62	881.77	881.23	881.32
THIRD AVENUE	1.311	11.080	882.60	882.81	882.43	882.63
	1.329	11.098	882.63	882.84	882.68	882.84
UNIVERSITY AVENUE	1.363	11.133	883.89	883.98	883.83	883.91
	1.397	11.168	885.12	885.27	885.36	885.94
SUNSET DRIVE	1.433	11.204	885.42	885.60	885.52	886.03
	1.459	11.231	885.66	885.81	885.72	886.13
	1.481	11.253	886.78	886.89	886.77	886.88
ARROWHEAD DRIVE	1.504	11.276	887.13	887.43	887.13	887.43
	1.526	11.297	889.45	889.99	889.58	889.83
SHERWOOD AVENUE	1.553	11.325	889.60	890.07	889.70	889.93
	1.573	11.345	890.64	890.43	890.66	890.30
ADMIRAL AVENUE	1.606	11.378	891.09	891.13	890.93	890.82
	1.625	11.397	891.51	891.64	891.38	891.44
	1.640	11.412	892.21	892.39	891.96	892.13
	1.672	11.444	892.71	892.87	892.40	892.56
HALL OF FAME	1.718	11.488	893.92	894.39	893.55	894.33
	1.777	11.546	898.73	899.08	898.66	899.02
	1.815	11.584	898.95	899.24	898.86	899.18
TOE OF SCS DAM 30	1.853	11.622	898.96	899.25	898.87	899.18

TABLE XVI

COMPARISON OF 100-YEAR WATER SURFACE ELEVATIONS

LOCATION	CHANNEL CROSS SECTION NUMBER		WATER SURFACE ELEVATION (FEET,NGVD) CHANNEL CONDITION			
			PRESENT		IMPROVED	
			URBANIZATION		URBANIZATION	
	PRESENT	IMPROVED	PRESENT	FUTURE	PRESENT	FUTURE
CONFLUENCE	0.090	10.019	855.24	855.37	855.25	855.38
	******	10.057	******	******	856.21	856.34
	0.178	10.095	858.03	858.21	857.22	857.34
	0.308	10.195	864.26	864.44	859.61	859.74
TWELVETH AVENUE	0.398	10.267	866.35	866.57	861.33	861.46
	0.430	10.299	866.55	866.74	864.80	865.22
OXBOW-WILLIS AVE	0.580	10.381	867.44	867.62	865.14	865.53
	******	10.400	******	******	865.39	865.76
MEYERS PARK	0.701	10.462	868.42	868.59	865.39	865.76
NINTH AVENUE	0.794	10.549	870.42	870.41	866.99	867.21
	0.817	10.573	871.20	871.32	869.35	871.00
	0.852	10.622	872.68	872.76	871.51	872.30
	0.896	10.664	873.76	873.82	873.70	873.82
	0.932	10.699	874.81	874.88	874.85	874.96
	0.970	10.735	875.16	875.21	875.22	875.35
	1.004	10.771	876.37	876.43	875.91	875.99
SIXTH AVENUE	1.044	10.807	877.29	877.36	876.22	876.80
	1.098	10.856	878.68	878.83	879.21	879.42
	1.127	10.884	878.72	878.88	879.24	879.46
	1.165	10.922	879.09	879.23	879.37	879.58

NOTE: ****** INDICATES CROSS SECTION NOT USED AT THAT LOCATION

TABLE XVI (Continued)

LOCATION	CHANNEL CROSS SECTION NUMBER		WATER SURFACE ELEVATION (FEET,NGVD)			
			CHANNEL CONDITION			
			PRESENT		IMPROVED	
			URBANIZATION		URBANIZATION	
	PRESENT	IMPROVED	PRESENT	FUTURE	PRESENT	FUTURE
	1,195	10,953	879.59	879.74	879.64	879.85
	1,233	10,991	880.57	880.66	880.60	880.72
RIDGE RD-INGHAM PK	1,263	11,021	881.79	881.89	881.33	881.46
THIRD AVENUE	1,311	11,080	882.85	882.98	882.65	882.88
	1,329	11,098	882.87	883.01	882.86	882.92
UNIVERSITY AVENUE	1,363	11,133	884.00	884.06	883.93	884.08
	1,397	11,168	885.27	885.29	885.78	885.95
SUNSET DRIVE	1,433	11,204	885.59	885.67	885.89	886.06
	1,459	11,231	885.80	885.88	886.01	886.18
	1,481	11,253	886.90	886.98	886.88	886.97
ARROWHEAD DRIVE	1,504	11,276	887.46	887.63	887.43	887.63
	1,526	11,297	889.50	890.40	889.83	890.42
SHERWOOD AVENUE	1,553	11,325	889.68	890.46	889.93	890.47
	1,573	11,345	891.05	890.50	891.02	890.51
ADMIRAL AVENUE	1,606	11,378	891.41	891.26	891.24	890.99
	1,625	11,397	891.85	891.82	891.71	891.66
	1,640	11,412	892.47	892.61	892.22	892.35
	1,672	11,444	892.89	893.03	892.59	892.73
HALL OF FAME	1,718	11,488	894.40	894.46	894.42	894.50
	1,777	11,546	899.08	899.46	899.02	899.41
	1,815	11,584	899.24	899.57	899.18	899.53
TOE OF SCS DAM 30	1,853	11,622	899.25	899.58	899.18	899.53

TABLE XVII

COMPARISON OF 500-YEAR WATER SURFACE ELEVATIONS

LOCATION	CHANNEL CROSS SECTION NUMBER		WATER SURFACE ELEVATION (FEET,NGVD)			
			CHANNEL CONDITION			
			PRESENT		IMPROVED	
			URBANIZATION		URBANIZATION	
	PRESENT	IMPROVED	PRESENT	FUTURE	PRESENT	FUTURE
CONFLUENCE	0.090	10.019	855.80	856.01	855.65	855.79
	******	10.057	******	******	856.60	856.73
	0.178	10.095	858.67	858.94	857.60	857.73
	0.308	10.195	864.85	865.00	860.00	860.13
TWELVETH AVENUE	0.398	10.267	867.04	867.29	861.69	861.83
	0.430	10.299	867.21	867.47	866.13	866.59
OXBOW-WILLIS AVE	0.580	10.381	868.05	868.28	866.37	866.81
	******	10.400	******	******	866.51	866.93
MEYERS PARK	0.701	10.462	868.92	869.10	866.51	866.93
NINTH AVENUE	0.794	10.549	870.47	870.56	867.67	867.90
	0.817	10.573	871.55	871.61	871.63	872.07
	0.852	10.622	872.96	873.07	872.76	872.97
	0.896	10.664	873.98	874.05	874.15	874.26
	0.932	10.699	875.04	875.12	875.18	875.29
	0.970	10.735	875.29	875.41	875.63	875.64
	1.004	10.771	876.58	876.61	876.15	876.23
SIXTH AVENUE	1.044	10.807	877.55	877.65	877.72	877.80
	1.098	10.856	879.14	879.33	879.59	879.82
	1.127	10.884	879.20	879.40	879.63	879.85
	1.165	10.922	879.50	879.68	879.75	879.96

NOTE: ****** INDICATES CROSS SECTION NOT USED AT THAT LOCATION

TABLE XVII (CONTINUED)

LOCATION	CHANNEL CROSS SECTION NUMBER		WATER SURFACE ELEVATION (FEET, NGVD)			
			CHANNEL CONDITION			
			PRESENT		IMPROVED	
			URBANIZATION		URBANIZATION	
	PRESENT	IMPROVED	PRESENT	FUTURE	PRESENT	FUTURE
	1.195	10.953	879.97	880.15	880.03	880.23
	1.233	10.991	880.78	880.91	880.84	880.95
RIDGE RD-INGHAM PK	1.263	11.021	882.03	882.14	881.60	881.72
THIRD AVENUE	1.311	11.080	883.16	883.33	883.13	883.34
	1.329	11.098	883.19	883.36	883.17	883.38
UNIVERSITY AVENUE	1.363	11.133	884.18	884.27	884.15	884.20
	1.397	11.168	885.29	885.30	885.79	885.96
SUNSET DRIVE	1.433	11.204	885.73	885.84	885.94	886.11
	1.459	11.231	885.94	886.05	886.11	886.27
	1.481	11.253	887.06	887.16	887.06	887.15
ARROWHEAD DRIVE	1.504	11.276	888.58	888.67	888.63	888.68
	1.526	11.297	890.01	890.41	890.03	890.43
SHERWOOD AVENUE	1.553	11.325	890.12	890.49	890.14	890.50
	1.573	11.345	891.51	890.55	891.41	890.56
ADMIRAL AVENUE	1.606	11.378	891.61	891.51	891.60	891.19
	1.625	11.397	892.28	892.20	892.15	892.04
	1.640	11.412	892.90	892.99	892.66	892.74
	1.672	11.444	893.19	893.28	892.90	892.99
HALL OF FAME	1.718	11.488	894.55	894.73	894.54	894.58
	1.777	11.546	899.74	900.02	899.71	899.98
	1.815	11.584	899.83	900.09	899.80	900.06
TOE OF SCS DAM 30	1.853	11.622	899.84	900.09	899.80	900.06

APPENDIX F

FLOOD DAMAGE WATER SURFACE ELEVATIONS

TABLE XVIII

FLOOD DAMAGE WATER SURFACE ELEVATIONS

BLDG CODE NO.	FIRST FLOOR ELEVATION (FEET,NGVD)	100-YEAR FLOOD WATER SURFACE ELEVATION (FEET,NGVD)			
		CHANNEL CONDITION			
		PRESENT		IMPROVED	
		URBANIZATION		URBANIZATION	
		PRESENT	FUTURE	PRESENT	FUTURE
1	868.6	868.31	868.48	867.77	867.77
2	868.6	868.42	868.59	867.77	867.77
3	868.6	868.71	868.85	867.77	867.77
4	868.5	867.77	867.77	867.77	867.77
5	866.8	867.77	867.77	867.77	867.77
6	868.5	867.77	867.77	867.77	867.77
7	866.8	867.77	867.77	867.77	867.77
8	865.9	867.77	867.77	867.77	867.77
9	866.3	867.77	867.77	867.77	867.77
10	866.3	868.19	868.36	867.77	867.77
11	875.0	875.50	875.55	875.41	875.53
12	875.0	875.16	875.21	875.22	875.35
13	875.0	874.98	875.04	875.04	875.16
14	872.9	873.20	873.27	872.56	873.03
15	871.8	872.96	873.04	872.08	872.70
16	871.0	872.68	872.76	871.51	872.30
17	871.0	871.44	871.55	869.70	871.21
18	870.5	871.44	871.55	869.70	871.21
19	870.7	871.44	871.55	869.70	871.21
20	870.0	872.68	872.76	871.51	872.30
21	871.3	873.10	873.17	872.37	872.89
22	873.7	873.52	873.60	873.22	873.49
23	873.7	873.93	873.99	873.88	874.00
24	873.6	874.42	874.49	874.43	874.55
25	873.5	874.84	874.91	874.89	875.02
26	874.3	874.98	875.04	875.04	875.16
27	875.0	875.16	875.21	875.22	875.35
28	875.3	875.50	875.55	875.41	875.53

TABLE XVIII (Continued)

BLDG CODE NO.	FIRST FLOOR ELEVATION (FEET,NGVD)	100-YEAR FLOOD WATER SURFACE ELEVATION (FEET,NGVD)			
		CHANNEL CONDITION			
		PRESENT		IMPROVED	
		URBANIZATION		URBANIZATION	
		PRESENT	FUTURE	PRESENT	FUTURE
29	875.8	875.97	876.02	875.68	875.78
30	876.2	876.37	876.43	875.91	875.99
31	877.0	876.83	876.90	876.06	876.40
32	877.5	877.56	877.64	876.80	877.30
33	877.0	877.29	877.36	876.22	876.80
34	876.0	876.83	876.90	876.06	876.40
35	875.0	876.37	876.43	875.91	875.99
36	874.7	875.97	876.02	875.68	875.78
37	874.4	875.50	875.55	875.41	875.53
38	874.2	875.16	875.21	875.22	875.35
39	873.2	874.98	875.04	875.04	875.16
40	872.2	874.84	874.91	874.89	875.02
41	872.2	874.64	874.71	874.69	874.80
42	872.4	874.28	874.35	874.28	874.40
43	872.6	873.93	873.99	873.88	874.00
44	872.6	873.52	873.60	873.22	873.49
45	871.7	873.20	873.27	872.56	873.03
46	870.7	872.68	872.76	871.51	872.30
47	870.5	871.44	871.55	869.70	871.21
48	870.5	871.44	871.55	869.70	871.21
49	871.5	871.44	871.55	869.70	871.21
50	870.0	872.68	872.76	871.51	872.30
51	871.2	872.96	873.04	872.08	872.70
52	873.7	873.20	873.27	872.56	873.03
53	878.7	879.09	879.23	879.37	879.58
54	880.0	879.79	879.92	879.83	880.02
55	880.2	880.57	880.66	880.60	880.72
56	882.0	881.79	881.89	881.33	881.46

TABLE XVIII (Continued)

BLDG CODE NO.	FIRST FLOOR ELEVATION (FEET,NGVD)	100-YEAR FLOOD WATER SURFACE ELEVATION (FEET,NGVD)			
		CHANNEL CONDITION			
		PRESENT		IMPROVED	
		URBANIZATION		URBANIZATION	
		PRESENT	FUTURE	PRESENT	FUTURE
57	881.0	882.85	882.98	882.65	882.88
58	880.2	882.85	882.98	882.65	882.88
59	881.5	881.79	881.89	881.33	881.46
60	883.0	883.12	883.24	883.10	883.18
61	882.7	883.12	883.24	883.10	883.18
62	882.2	883.12	883.24	883.10	883.18
63	883.7	884.27	884.33	884.33	884.48
64	886.0	885.34	885.37	885.80	885.97
65	885.0	885.34	885.37	885.80	885.97
66	886.2	885.59	885.67	885.89	886.06
67	885.0	885.99	886.07	886.16	886.32
68	884.7	885.99	886.07	886.16	886.32
69	886.2	885.99	886.07	886.16	886.32
70	889.2	889.50	890.40	889.83	890.42
71	889.0	889.50	890.40	889.83	890.42
72	889.2	889.50	890.40	889.83	890.42
73	890.2	889.50	890.40	889.83	890.42
74	890.2	889.68	890.46	889.93	890.47
75	890.0	889.68	890.46	889.93	890.47
76	889.2	889.68	890.46	889.93	890.47
77	890.7	889.68	890.46	889.93	890.47
78	890.7	891.09	890.59	891.05	890.56
79	889.7	891.09	890.59	891.05	890.56
80	891.5	891.41	891.26	891.24	890.99
81	892.0	892.00	892.02	891.84	891.83
82	892.5	892.00	892.02	891.84	891.83

APPENDIX G

DEPTH-DAMAGE DATA

TABLE XIX

1970 FIA DEPTH-DAMAGE DATA TABLE
(MODIFIED SET A)

DEPTH (FEET)	DAMAGE (PERCENT OF STRUCTURE VALUE)			
	ONE STORY RESIDENCE NO BASEMENT		TWO STORY RESIDENCE NO BASEMENT	
	STRUCTURE	CONTENTS	STRUCTURE	CONTENTS
-1.0	0.0	0.0	0.0	0.0
-0.9	0.2	0.0	0.0	0.0
-0.8	0.4	0.0	0.0	0.0
-0.7	0.6	0.0	0.1	0.0
-0.6	0.8	0.0	0.2	0.0
-0.5	1.2	0.0	0.3	0.0
-0.4	1.6	0.0	0.4	0.0
-0.3	2.0	0.0	0.7	0.0
-0.2	2.6	0.0	1.0	0.0
-0.1	3.2	0.0	1.3	0.0
0.0	4.0	0.0	2.0	0.0
0.1	8.0	5.0	4.0	5.0
0.2	12.0	8.5	4.8	6.2
0.3	14.0	11.8	5.5	7.5
0.4	15.5	15.0	6.0	8.5
0.5	16.5	18.2	6.8	9.8
0.6	17.8	21.8	7.5	11.0
0.7	18.8	25.0	8.2	12.2
0.8	20.0	28.5	8.8	13.5
0.9	20.8	32.0	9.5	14.8
1.0	22.0	35.0	10.0	16.0
1.1	22.8	37.8	10.8	17.0
1.2	23.5	39.5	11.5	18.2
1.3	24.2	41.2	12.0	19.5
1.4	25.2	42.8	12.8	20.8
1.5	26.0	44.0	13.2	21.8
1.6	26.8	45.2	13.8	23.0
1.7	27.5	46.5	14.2	24.2
1.8	28.5	47.5	15.0	25.5
1.9	29.2	48.8	15.5	26.8
2.0	30.0	50.0	16.0	28.0

TABLE XIX (Continued)

DEPTH (FEET)	DAMAGE (PERCENT OF STRUCTURE VALUE)			
	ONE STORY RESIDENCE NO BASEMENT		TWO STORY RESIDENCE NO BASEMENT	
	STRUCTURE	CONTENTS	STRUCTURE	CONTENTS
2.0	30.0	50.0	16.0	28.0
2.1	30.5	50.8	16.2	29.0
2.2	31.0	51.8	16.8	30.0
2.3	31.8	53.0	17.2	31.0
2.4	32.2	54.0	17.5	32.0
2.5	32.8	54.8	18.0	33.0
2.6	33.2	55.8	18.2	33.8
2.7	33.8	56.8	18.8	34.5
2.8	34.2	57.8	19.2	35.5
2.9	34.8	58.8	19.5	36.2
3.0	35.0	60.0	20.0	37.0
3.1	35.5	61.0	20.5	37.5
3.2	36.0	61.8	20.8	38.2
3.3	36.5	62.8	21.2	39.0
3.4	36.8	63.5	21.8	39.5
3.5	37.2	64.2	22.0	40.2
3.6	37.5	65.2	22.5	40.8
3.7	38.0	66.0	22.8	41.5
3.8	38.2	66.8	23.2	42.0
3.9	38.8	67.2	23.5	42.5
4.0	39.0	68.0	24.0	43.0
4.1	39.2	68.8	24.2	43.5
4.2	39.5	69.2	24.7	44.0
4.3	39.8	70.0	24.9	44.4
4.4	40.0	70.5	25.3	44.9
4.5	40.2	71.2	25.6	45.3
4.6	40.5	71.8	25.8	45.7
4.7	40.7	72.2	26.2	46.1
4.8	40.8	73.0	26.4	46.4
4.9	40.9	73.5	26.7	46.7
5.0	41.0	74.0	27.0	47.0

2

VITA

Robert Louis Tortorelli

Candidate for the Degree of

Doctor of Philosophy

Thesis: A METHODOLOGY TO ASSESS THE IMPACT OF A CHANGING FLOOD PLAIN DETERMINATION ON AN UNGAGED URBAN BASIN

Major Field: Civil Engineering

Biographical:

Personal Data: Born in December 6, 1945, in Chicago, Illinois, the son of Louis A. and Alice H. Tortorelli. Married to Phung T. Tortorelli, and father of Michael, John, and Richard.

Education: Graduated from Lane Technical High School, Chicago, Illinois, in June, 1963; received the Bachelor of Science degree in Forestry with high honors from the University of Illinois, Urbana, Illinois, in June, 1967; completed the requirements for the Master of Science degree in Forest Hydrology from the University of Illinois, Urbana, Illinois, in June, 1969; completed the requirements for the Doctor of Philosophy degree in Civil Engineering at Oklahoma State University, Stillwater, Oklahoma, in May, 1981.

Professional Experience: Graduate research/teaching assistant, Forestry, University of Illinois, September, 1967, to June, 1969; U.S. Army Officer, Corps of Engineers Branch, Tulsa U.S. Army Engineer District, Vietnam, Fort Riley Kansas, October, 1969, to September, 1975; controller/umpire, engineer team, 95th Division Maneuver Training Command, U.S. Army Reserves, October, 1975, to present; graduate teaching assistant, Civil Engineering, Oklahoma State University, September, 1976, to August, 1978; surface water hydrologist for U.S. Geological Survey, September, 1978 to present.

Membership in Professional Societies: Associate member of the American Society of Civil Engineers; member of Sigma Xi; studdent member of American Water Works Association; member of Reserve Officers Association.

Membership in Honorary Societies: Chi Epsilon; Phi Kappa Phi; Gamma Sigma Delta; Alpha Zeta; Xi Sigma Pi.