HARRIS COUNTY FLOOD CONTROL DISTRICT
HARRIS-GALVESTON COASTAL SUBSIDENCE DISTRICT
CITY OF HOUSTON
FORT BEND COUNTY DRAINAGE DISTRICT

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

DECEMBER 1986

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

Prepared for

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TEXT

December 1986

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Mr. James B. Green, Director
Harris County Flood Control District
9900 Northwest Freeway, Suite 300
Houston, Texas 77092

Dear Mr. Green:

The accompanying report, entitled "A Study of the Relationship Between Subsidence and Flooding," presents the results of the initial study into the effects of subsidence on inland flooding.

The purpose of this study is to specifically define the impacts of subsidence on the components of the inland drainage and flood control systems in the greater Houston area in order to assist the study sponsors: the Fort Bend County Drainage District, the Harris-Galveston Coastal Subsidence District, the City of Houston, and the Harris County Flood Control District in the performance of their duties and responsibilities in project planning and implementation or regulatory review and approval associated with stormwater management, water supply, and subsidence control.

The Summary which immediately precedes Chapter I, outlines the essential conclusions of this report. A brief history of subsidence for the area and a discussion of the arrangement of the report appears in Chapter I, the introductory section. Chapters II, III, and IV include the basic information used to perform the analyses, impacts on flooding, and recommendations for the three major components of drainage and flood control systems: Riverine Drainage Systems, Localized Drainage Systems, and the Addicks and Barker Flood Control Reservoir System. Immediately following the tables and exhibits section, a technical appendix is included to complete the documentation of the basic data used for the Riverine Flooding Analysis (Chapter II).

We certainly appreciate this opportunity to be of service, and gratefully acknowledge the cooperation and support the study sponsors have given us during the entire study.

Respectfully submitted,

TURNER COLLIE & BRADEN INC.

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TABLE OF CONTENTS

SUMMARY

RIVERINE FLOODING ANALYSIS
LOCALIZED DRAINAGE ANALYSIS
RESERVOIR CAPACITY ANALYSIS
RECOMMENDATIONS

CHAPTER I - INTRODUCTION

PURPOSE OF STUDY
SUBSIDENCE HISTORY
GENERAL SCOPE OF STUDY
STUDY SPONSORS
ARRANGEMENT OF REPORT

CHAPTER II - RIVERINE FLOODING ANALYSIS

INTRODUCTION
Overview
Primary Objective
Study Area
Buffalo Bayou
Brays Bayou
Sims Bayou
Addicks Reservoir Major Tributaries
Barker Reservoir Major Tributaries
TECHNICAL APPROACH
Overview
Simulation of Subsidence
Procedure
Storm Events Analyzed

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

Application of Hydraulic Models
Application of Hydrologic Models
ANALYSIS OF RIVERINE SUBSIDENCE
Overview
Selection of Subsidence Cases
Subsidence and Its Effect on Storm Flows
Subsidence and Its Effect on Depth of Flooding
Subsidence and Its Effect on Flood Plain Area
A Procedure for Estimating the Effect on Flood Plain Area
Subsidence and Its Effect on the Brays Bayou Watershed
Flood Plain Analysis
Flood Damage Analysis
CONCLUSIONS AND RECOMMENDATIONS

CHAPTER III - LOCALIZED DRAINAGE ANALYSIS

INTRODUCTION
Overview
Primary Objectives
Study Area
TECHNICAL APPROACH
Overview
Selection of Computer Model
Development of Input Parameters
Calibration of Model
ANALYSIS OF LOCAL EFFECTS OF SUBSIDENCE
Overview
Selection of Subsidence Cases
Localized Well Field Subsidence Cone
Regional Subsidence Gradient
Analysis of Localized Subsidence

Subsidence Pattern vs. Depth of Ponding at Street Inlets and Change in Depth of Ponding
Subsidence Pattern vs. Timing of Inlet Ponding
Evaluation of Street Ponding Impacts
Subsidence Pattern vs. Change in Water Depth in Bintliff Ditch
Subsidence Pattern vs. Percent Change in Peak Flow in Bintliff Ditch
Evaluation of Channel Impacts
CONCLUSIONS AND RECOMMENDATIONS

CHAPTER IV - RESERVOIR CAPACITY ANALYSIS

INTRODUCTION
Overview
Primary Objective
Technical Approach
Study Area
Description of Reservoirs
Watershed Characteristics
Relationship to Historic Subsidence
ANALYSIS OF RESERVOIR CAPACITY
Development of Base Models
Subsidence Cases
Modification of Base Models
Evaluation of Revised Capacities
Case 1R
Case 2R
Case 3R
Case 4R
Capacity Evaluation Considering Proposed Embankment Modifications
CONCLUSIONS AND RECOMMENDATIONS

CHAPTER V - REFERENCES

CHAPTER I - INTRODUCTION
CHAPTER II - RIVERINE FLOODING ANALYSIS
CHAPTER III - LOCALIZED DRAINAGE ANALYSIS
CHAPTER IV - RESERVOIR CAPACITY ANALYSIS

LIST OF TABLES

Number	Title
CHAPTER II - RIVERINE FLOODING ANALYSIS	
II-1	RIVERINE POINT RAINFALL AMOUNTS
II-2	LOSS RATES FOR STORM EVENTS
II-3	CHANGES IN UNITGRAPH COEFFICIENTS DUE TO CHANGE IN CHANNEL GRADIENT (25 PERCENT FLATTER)
II-4	CHANGE IN PEAK FLOW FROM SUB-AREAS ON WILLOW FORK - (25 PERCENT FLATTER)
II-5	WILLOW FORK EXISTING STORAGE-DISCHARGE INFORMATION
II-6	WILLOW FORK STORAGE-DISCHARGE RELATED TO SUBSIDENCE (25 PERCENT FLATTER)
II-7	TYPICAL FLOW CHANGES ON WILLOW FORK ATTRIBUTED TO CHANGES IN UNITGRAPH COEFFICIENTS AND REVISED STORAGE-DISCHARGE RELATIONSHIPS CAUSED BY SUBSIDENCE (25 PERCENT FLATTER)
II-8	CONSTANT SLOPE ANALYSIS REACHES
II-9	DESCRIPTION OF SUBSIDENCE CASES SELECTED FOR ANALYSIS
II-10	SUBSIDENCE CASE MATRIX
II-11	MAGNITUDE OF SUBSIDENCE ALONG CHANNEL CENTERLINE
II-12	FLOOD DEPTH CHANGE
II-13	RESULTS FROM PREDICTIVE PROCEDURE FOR ESTIMATING CHANGE IN FLOOD PLAIN AREA (FPA)
II-14	BRAYS BAYOU ECONOMIC FLOOD DAMAGE DATA
CHAPTER III - LOCALIZED DRAINAGE ANALYSIS	
III-1	BINTLIFF DITCH HISTORICAL STORMS
III-2	AVAILABLE COMPUTER MODELS FOR STORM SEWER ANALYSIS
III-3	CALIBRATED INPUT PARAMETER RANGES
III-4	SUMMARY OF PEAK FLOWS IN BINTLIFF DITCH AT BISSONNET (GAGE SITE)

Number	Title
III-5	SUMMARY OF PEAK FLOWS IN SELECTED STORM SEWERS
III-6	SIX DETAILED SUBBASIN DRAINAGE AREA CHARACTERISTICS
III-7	MAXIMUM INLET PONDING DEPTHS (FEET) FOR BASE CONDITION (NO SUBSIDENCE)
III-8	CHANGE IN INLET PONDING DEPTHS (FEET) FROM THE BASE CONDITION DUE TO SUBSIDENCE CASES
III-9	INITIAL TIMING OF INLET PONDING EQUAL TO SIX INCHES FOR BASE CONDITION
III-10	DURATION OF INLET PONDING GREATER THAN OR EQUAL TO SIX INCHES FOR BASE CONDITION

CHAPTER IV - RESERVOIR CAPACITY ANALYSIS

Number	Title
IV-1	RESERVOIR DATA
IV-2	EXISTING STORAGE-CAPACITY COMPARISON
IV-3	REVISED STORAGE-CAPACITY CURVES, ADDICKS RESERVOIR
IV-4	REVISED STORAGE-CAPACITY CURVES, BARKER RESERVOIR
IV-5	SUMMARY OF RESERVOIR STORAGE CHANGES
IV-6	FREEBOARD COMPARISON, ADDICKS RESERVOIR
IV-7	FREEBOARD COMPARISON, BARKER RESERVOIR
IV-8	RESERVOIR EMBANKMENT ELEVATION COMPARISON, PRE-PROJECT VS. POST-PROJECT
IV-9	FREEBOARD COMPARISON WITH PROPOSED MODIFICATIONS
IV-10	SPILLWAY DESIGN FLOOD FREEBOARD COMPARISION, ADDICKS RESERVOIR
IV-11	SPILLWAY DESIGN FLOOD FREEBOARD COMPARISION, BARKER RESERVOIR

LIST OF EXHIBITS

Number	Title

CHAPTER I - INTRODUCTION

Number	Title
I-1	HISTORIC LAND-SURFACE SUBSIDENCE 1906-1978

CHAPTER II - RIVERINE FLOODING ANALYSIS

Number	Title
II-1	RIVERINE ANALYSIS VICINITY MAP
II-2	AREAL SUBSIDENCE OVERLAY
II-3	RIVERINE AREAL SUBSIDENCE CONTOUR MAP-HGCSD PLAN
II-4	SCHEMATIC OF STREAM GRADIENT ADJUSTMENTS
II-5	BUFFALO, BRAYS, & SIMS BAYOUS - WATERSHED BOUNDARIES & SUB-WATERSHED BOUNDARIES
II-6	ADDICKS & BARKER TRIBUTARIES-WATERSHED BOUNDARIES & SUB-WATERSHED BOUNDARIES
II-7	RIVERINE AREAL SUBSIDENCE CONTOUR BASE MAP
II-8	PERCENT CHANGE IN AVERAGE PEAK DISCHARGE VS. PERCENT CHANGE IN GRADIENT
II-9	PERCENT CHANGE IN HYDRAULIC CARRYING CAPACITY VS. PERCENT CHANGE IN GRADIENT
II-10	DRAINAGE AREA VS. PERCENT CHANGE IN AVERAGE PEAK DISCHARGE FOR CHANGES IN GRADIENT
II-11	AVERAGE CHANGE IN DEPTH OF FLOODING VS. PERCENT CHANGE IN GRADIENT
II-12	MAXIMUM CHANGE IN DEPTH OF FLOODING VS. SUBSIDENCE (100-YEAR EVENT)
II-13	PERCENT CHANGE IN FLOOD PLAIN AREA VS. PERCENT CHANGE IN GRADIENT
II-14	PROCEDURE FOR ESTIMATION OF MAXIMUM CHANGE IN FLOOD PLAIN AREA (FPA)
II-15	PREDICTIVE PROCEDURE SIMULATION - BUFFALO BAYOU CASE 3
II-16	BRAYS BAYOU 100-YEAR STATION-DISCHARGE CURVES

Number	Title
II-17	BRAYS BAYOU IMPACTS OF SUBSIDENCE
II-18 & II-19	BRAYS BAYOU FLOOD PLAIN, 100-YEAR FLOOD
II-20 THRU II-25	BRAYS BAYOU STREAM PROFILES
II-26	BRAYS BAYOU ECONOMIC REACH DELINEATIONS

CHAPTER III - LOCALIZED DRAINAGE ANALYSIS

Number	Title
III-1	BINTLIFF DITCH WATERSHED MAP
III-2	LOCATION OF SUBBASINS
III-3	EXISTING AND REDESIGNED DRAINAGE SYSTEM
III-4	DRAINAGE SYSTEM PROFILES
III-5	MODEL CALIBRATION TO HISTORICAL STORMS
III-6	LOCALIZED WELL FIELD SUBSIDENCE CONES
III-7	REGIONAL SUBSIDENCE GRADIENTS
III-8	TYPICAL SUBBASIN DRAINAGE AREA AND STREET PONDING
III-9	SUBSIDENCE VS. CHANNEL DEPTH CASES 1a AND 1b
III-10	SUBSIDENCE VS. CHANNEL DEPTH CASES 1c AND 1d
III-11	SUBSIDENCE VS. CHANNEL DEPTH CASES 2a AND 2b
III-12	SUBSIDENCE VS. CHANNEL DEPTH CASES 2c AND 2d
III-13	SUBSIDENCE VS. CHANNEL FLOW CASES 1a AND 1b
III-14	SUBSIDENCE VS. CHANNEL FLOW CASES 1c AND 1d
III-15	SUBSIDENCE VS. CHANNEL FLOW CASES 2a AND 2b
III-16	SUBSIDENCE VS. CHANNEL FLOW CASES 2c AND 2d

CHAPTER IV - RESERVOIR CAPACITY ANALYSIS

Number	Title
IV-1	RESERVOIR ANALYSIS VICINITY MAP
IV-2	ADDICKS RESERVOIR EXISTING RESERVOIR CONTOUR MAP
IV-3	BARKER RESERVOIR EXISTING RESERVOIR CONTOUR MAP
IV-4	ADDICKS RESERVOIR SUBSIDENCE CASES
IV-5	BARKER RESERVOIR SUBSIDENCE CASES
IV-6	ADDICKS RESERVOIR DEPTH CAPACITY CURVES
IV-7	BARKER RESERVOIR DEPTH CAPACITY CURVES
IV-8	ADDICKS RESERVOIR 100-YEAR FLOOD POOLS
IV-9	BARKER RESERVOIR 100-YEAR FLOOD POOLS
IV-10	ADDICKS RESERVOIR CONTOUR CHANGES (CASE 3R)
IV-11	BARKER RESERVOIR CONTOUR CHANGES (CASE 4R)

TECHNICAL APPENDIX

Number	Title
A-1	EXISTING BASIN CHARACTERISTICS AND UNITGRAPH COEFFICIENTS
A-2	EXISTING CONDITION STORAGE-DISCHARGE INFORMATION
A-3	CHANNEL CHARACTERISTICS, WILLOW FORK (100-YEAR DESIGN CHANNEL)
A-4	STORAGE-DISCHARGE INFORMATION, WILLOW FORK (100-YEAR DESIGN CHANNEL)
A-5	EXISTING STORM FLOWS
A-6 THRU A-15	REVISED STORAGE-DISCHARGE RELATIONSHIPS
A-16 THRU A-26	REVISED STORM FLOWS
A-27 THRU A-36	HYDRAULIC CARRYING CAPACITY (FLOODED SECTION FACTOR)
A-37 THRU A-46	FLOOD PLAIN AREA

SUMMARY

SUMMARY

Subsidence of the land surface and its effect on tidal flooding has long been recognized as a major problem facing the greater Houston area. Considerable efforts to convert from groundwater use to surface water have been made since the mid to late 1970's in eastern Harris County where maximum historical subsidence has occurred. These efforts have resulted in dramatic reductions in the rate of subsidence in eastern Harris County and a virtual halt in land surface subsidence in areas affected by tidal conditions.

At the same time the conversion to surface water was occurring in eastern Harris County, continual increases in groundwater pumpage were occurring in the western and northern metropolitan areas not affected by tidal flooding. The result has been the gradual shifting of the regional concentration of subsidence westward. The relationship between subsidence and inland flooding, however, is not as clear as in coastal areas where one foot of subsidence corresponds to a one-foot increase in depth of flooding and varying opinions exist as to the effects that subsidence has on inland flooding conditions. This report presents the results of a study to specifically evaluate the impact of subsidence on inland flooding. The study focused on the impacts of subsidence on three major areas of the inland drainage system: riverine flooding on major watersheds, localized drainage in small watersheds, and the Addicks and Barker flood control reservoir system.

The study was conducted through the cooperative efforts of the four local entities with primary responsibility for water supply and flood control in the Houston metropolitan area: the Fort Bend County Drainage District, the Harris County Flood Control District, the Harris-Galveston Coastal Subsidence District (HGCSD), and the City of Houston. Each of these entities, through project planning and implementation or regulatory review and approval, will play a significant role in the ultimate solution of the Houston metropolitan area's problems associated with storm-water management, water supply, and subsidence.

RIVERINE FLOODING ANALYSIS

The riverine flooding analysis quantifies, through detailed hydrologic and hydraulic analyses of a number of stream systems, the impacts of subsidence on major open channel systems. It demonstrates that in contrast to coastal areas, where a foot of subsidence results in a foot of additional flooding, an average of the conditions tested indicates that in areas where increased flooding occurred, the average increase in flooding depth was 1/10 of the related subsidence with the maximum increase at any specific location not exceeding 1/3 of the related subsidence. A similar magnitude of impact was found to occur in conditions when flood levels decreased.

The subsidence-riverine flooding analysis was performed on nine channels in five major watersheds in the Houston area. The five watersheds studied are Brays Bayou, Sims Bayou, Buffalo Bayou, Addicks Reservoir tributaries, and Barker Reservoir tributaries. A total of 48 subsidence simulations were analyzed on these watersheds to identify trends and relationships between subsidence and flood plain/flood flow parameters. The overall trend in impacts of subsidence observed from stream system to stream system were very consistent.

Where a cone of subsidence is located within a stream system, decreased flood levels occur upstream of the center of the cone, or point of maximum subsidence, and increased flood levels occur downstream. The extent of increase or decrease in flooding is dependent on the magnitude of subsidence as well as localized topographic features. As the stream gradient is steepened, the carrying capacity of the channel is increased as is the peak discharge rate. However, since the carrying capacity increases at a faster rate, a net decrease in water surface elevation is realized. The converse is also true.

Although generally consistent trends were observed from stream to stream, a generalized predictive methodology cannot be used in lieu of detailed hydrologic and hydraulic analyses for projecting actual flood impacts. Localized conditions produce anomalies which cannot be predicted in a generalized methodology and thus limit the potential applications of generalized procedures, particularly with respect to the evaluations of any mitigating flood control projects. A general methodology was developed; however, its use should be restricted to screening of watersheds for more detailed analysis given a predefined subsidence case. Final plan evaluations should be based on detailed watershed modeling which can be accomplished using currently available hydrologic and hydraulic models. The procedures must provide for coordinated evaluation of flood control impacts predicted for a given subsidence case considering the compensating effect of flood control projects.

An effectiveness of this coordinated approach is exemplified by an analysis of the Brays Bayou watershed which addresses both the effects of subsidence and flood control improvements in response to the overall flooding problem. Two cases of subsidence were analyzed. One was the projected subsidence pattern assuming limited future surface water conversions including only the expansion of the City of Houston's East Water Purification Plant presently under construction. The second case was the projected subsidence pattern assuming the much more extensive conversion required by the Harris-Galveston Coastal Subsidence District's Groundwater Management Plan (HGCSD's Plan). The damages associated with each of these two subsidence conditions was in turn evaluated assuming the structural improvements proposed by the Brays and Sims Bayou Regional Flood Control Plan were in place.

Some increase in flood damages along Brays Bayou are projected to occur over the 34-year analysis period for both cases of subsidence. However, a significant reduction in flood damage increases is accomplished with the HGCSD's Plan. For the 100-year storm event, the increase in flood damages over the analysis period is decreased from $58 million to $24 million and for the 10-year storm event the increase in flood damages is decreased from $129 million to $40 million. The fact that flood damages still increase even with the HGCSD's aggressive plan of conversion from groundwater to surface water shows the difficulty of addressing the problem through the control of groundwater pumpage alone. Imposing the Brays Bayou and Sims Bayou Regional Flood Control Plan essentially eliminates flood damages for the 10-year storm event and minimal residual flood damages remain for the 100-year storm. This indicates the ability of flood control system improvements to mitigate subsidence impacts and the need for joint planning so that, with full implementation, all goals are fully achieved.

The riverine flooding analysis points out that although the flooding depth increase is relatively small in inland areas as compared to coastal areas, the potential for increased damages may be high in certain areas. To fully mitigate these impacts, increased conversions to surface water will be required as proposed in the HGCSD's Plan and regional flood control programs may be designed to address any residual flooding impacts.

LOCALIZED DRAINAGE ANALYSIS

The localized drainage analysis defines the impacts of area-wide subsidence, localized well placements, and storm sewer design criteria on a typical small drainage system in the Houston area. Street ponding, although not a design feature of urban drainage systems, is a regular occurrence in the metropolitan Houston area and was specifically analyzed as a part of the local drainage analysis. It was found that for all subsidence conditions analyzed, the effects on street ponding due to subsidence were negligible.

The Localized Drainage Analysis results indicate that general subsidence patterns do not significantly affect street ponding. While the change in design criteria adopted by the City of Houston has reduced the impact of ponding, neither system designed in accordance with the "old" or "new" criteria showed a significant response to regional subsidence patterns. The results further indicate that optimum placement of well fields should be near the drainage divide of small watersheds or on similar areas of high topography within these watersheds. While localized street ponding is not significantly affected, the placement of major well fields adjacent to the primary outfall channels of small basins will tend to increase local flooding of the channel near the well placement. If unavoidable, placement of well fields adjacent to primary outfall channels or sewers should be combined with drainage system modifications to provide an increase in system capacity based on the predictable effects of subsidence on the channel or sewer gradient.

In general, drainage system design criteria are of much greater importance in controlling localized flooding than is the placement of local well fields or regional subsidence gradients. Although system surcharge and street ponding are common in the Houston area due to the practical constraints of designing a system to handle the intense rainfall events and the extremely flat topography, the use of current storm sewer design criteria results in significantly reduced ponding levels and duration of ponding than previous criteria. The current design criteria also address street ponding and require methods to control ponding to prevent flood damage to structures.

RESERVOIR CAPACITY ANALYSIS

The reservoir capacity analysis addresses how subsidence impacts can potentially affect the ability of the Addicks and Barker reservoirs to regulate storm flows consistent with their design concepts. It was concluded from the analysis that subsidence resulting from groundwater withdrawals consistent with the HGCSD's Plan, projecting the lowering of land surface elevations generally in a northwest to southeast direction, would have an insignificant effect on the functions of Barker or Addicks Reservoirs. This general pattern of subsidence would increase storage capacity in the reservoirs and reduce inundation of private lands. The level of subsidence projected by the HGCSD's Plan has minimal impact on embankment freeboards.

Subsidence from northeast to southwest would result in reductions in storage in Barker Reservoir, increased inundation of private lands upstream of the Reservoir, and the potential for inundation of private lands adjacent to the Reservoir. Subsidence from southwest to northeast would result in similar impacts on Addicks Reservoir. Based on this analysis, trends in subsidence in the northeast to southwest direction or the southwest to northeast direction should be guarded against. Either will result in reservoir storage reductions and increased inundation of private lands upstream and adjacent to the reservoirs.

CONCLUSIONS AND RECOMMENDATIONS

The results of this initial study into the effects of subsidence on inland flooding indicate that subsidence is not a problem affecting just coastal areas. A cone of subsidence located within an inland watershed will result in increased flooding, although not to the degree observed in coastal areas. The study defined the potential impacts of subsidence on riverine drainage systems, localized drainage systems, and two flood detention reservoirs as a result of the development of the westerly cone.

In the riverine systems, subsidence causing steeper channel gradients generally reduced flood impacts while flattening of channel gradients increased flood impacts. The magnitude of the impacts were controlled by specific channel and watershed conditions though it was found that the average increase in flooding depth was one tenth of the related subsidence for a 100-year storm event. Regional subsidence patterns were found to have little impact on local drainage systems, but the localized subsidence created by placement of wells within a small drainage system could have an adverse impact on the system unless placed properly. It was found that the least impact on local drainage systems occurred when wells were located near the drainage system divide. Current subsidence trends were found to have no adverse impacts on the function of the Addicks and Barker Reservoir system. This system could be adversely impacted, however, if other subsidence patterns developed.

Implementation of the HGCSD's Plan will significantly reduce future increases in flooding levels along these riverine systems due to land surface subsidence but will not eliminate them. The HGCSD's Plan for conversion from groundwater to surface water is admittedly agressive and would be costly, if not impossible, to accelerate in response to the projected residual increases in flooding levels. It is recommended that the joint planning effort which produced this study be continued with the goal of developing a plan which considers the management of groundwater, stormwater, and subsidence to effect a least cost solution to these three related problems.

Finally, due to the potential for changes in subsidence trends resulting from changing groundwater pumpage patterns, it is recommended that the HGCSD and the Fort Bend County Drainage District explore the financial and technical feasibility of an ongoing interagency agreement which would allow an exchange of information on groundwater utilization and resulting subsidence patterns. This exchange would allow the HGCSD to better define subsidence within its jurisdiction, predict changes in subsidence trends which could affect the Addicks and Barker Reservoirs, and to provide the Fort Bend County Drainage District with needed information on subsidence to better plan and regulate drainage and flood control.

CHAPTER I

INTRODUCTION

CHAPTER I INTRODUCTION

PURPOSE OF STUDY

Since the turn of the century, increased quantities of groundwater have been withdrawn from local aquifers to keep pace with the greater Houston area's growth. As a result, significant declines in groundwater levels have occurred causing a depressurization of the clay lenses. The depressurized clays have compacted from the burden of the soils above causing a sinking of the land surface, or subsidence. In coastal areas, the increase of tidal flooding can be directly related to subsidence since, while the land surface has lowered, the sea level remains relatively constant. The relationship between inland flooding and the impact of subsidence on streams, storm sewer systems, and reservoir systems, however, is not as apparent. This report presents the results of a study to determine the relationship between subsidence and inland flooding. It also presents relationships developed during the study which can be used to assess the potential impact of future subsidence cases on inland flooding.

The additional withdrawal of groundwater for water supply will generally result in some degree of subsidence, and this study is intended to provide an understanding of the impacts of subsidence on flooding. Policy decisions relating to the permitted level of subsidence or the definition of programs to mitigate resulting impacts are not within the scope of this study. However, this study is intended to allow such decisions to be made with greater understanding of what impacts may result.

SUBSIDENCE HISTORY

As early as 1918, land-surface subsidence due to the withdrawal of oil and gas was noted in the Baytown, Texas area. Also during this time, substantial groundwater withdrawals were occurring in the Baytown area from large-capacity industrial wells with resulting reduced aquifer pressures and associated land-surface subsidence. By 1925, these withdrawals had caused as much as 3.25 feet of subsidence near the Goose Creek Field. While surface water was being introduced as an alternative source through the construction of the City of Houston's East Water Purification Plant in 1954, pumpage continued to increase throughout the greater Houston area such that by 1973 the levels of subsidence exceeded 9.0 feet in the Pasadena area, 8.0 feet on the western side of Baytown area, and a localized center of at least 9.0 feet on the southeastern side of Baytown in the Goose Creek area. This subsidence resulted in permanent flooding of some land adjacent to the coast and substantially increased flooding in areas subject to tidal surges associated with tropical storms.

In the early 1970's, community leaders saw the need to reduce pumpage. To respond to this need, the City of Houston sharply curtailed groundwater withdrawal in the southeastern portion of the City and, in mutual cooperation with industrial leaders, created the Coastal Industrial Water Authority (now Coastal Water Authority, CWA) to transport surface water from the Trinity River to the eastern and southeastern metropolitan area for municipal and industrial use. In 1975 the Harris-Galveston Coastal Subsidence District (HGCSD) was created by the Texas Legislature to plan and regulate groundwater withdrawal. The District's primary function was to develop and implement a plan to regulate groundwater withdrawal to control subsidence. In 1976 and 1977 conversion to surface water began in the heavily industrial area east of downtown Houston along the ship channel where the maximum amount of subsidence had occurred. As a result, the rate of subsidence in this area has declined. The areas west of downtown Houston, however, have experienced continued growth and increased groundwater withdrawal. While the western area experienced about 4.0 feet of subsidence between 1906 and 1978, it is currently experiencing subsidence at a rate of about one foot every seven years (Exhibit I-1 presents the historic land-surface subsidence map of the Houston-Galveston area).

The impact on flooding from inland subsidence was not fully defined. The need for more definitive information became evident as the local entities moved forward in planning for water supply, drainage and flood control, and groundwater regulation. To respond to the need for better information, this study was undertaken by the local entities primarily responsible for water supply, control of subsidence, and flood control in the Houston metropolitan area.

GENERAL SCOPE OF STUDY

The scope of this study includes three major components of the drainage and flood control systems in the greater Houston area: Riverine Drainage systems, Localized Drainage systems, and the Addicks and Barker flood control reservoir system. The Riverine Flooding Analysis portion of the study (Chapter II) is an evaluation of flooding that may result from potential subsidence along main drainage channels with the objective of determining if a relationship exists between gradient change caused by subsidence and storm flows or flood plain area. The Localized Drainage Analysis (Chapter III) is an evaluation of the impacts of regional subsidence and well field placements on localized drainage systems and the resultant effects on minor drainage channels, storm sewer systems, and street ponding. The Reservoir Capacity Analysis (Chapter IV) addresses the effects that subsidence-caused gradient changes have on the maximum flood storage capacities and 100-year pool levels (100-year flood plain elevations) of the Addicks and Barker flood control reservoirs.

STUDY SPONSORS

Management of subsidence, or the effects of subsidence, will require the joint efforts of a number of local political subdivisions. This study was conducted through the cooperating efforts of four such entities: the Fort Bend County Drainage District, the Harris County Flood Control District, the Harris-Galveston Coastal Subsidence District, and the City of Houston. Each of these entities, through project planning and implementation or regulatory review and approval, will play a significant role in the ultimate solution of the Houston metropolitan area's problems associated with subsidence.

ARRANGEMENT OF REPORT

This report is comprised of three major components: text, tables and exhibits, and a technical appendix. The text portion of the report references all tables and exhibits contained in the bound sections on the right-hand side.

The text report is divided into five chapters as follows:

I. Introduction
II. Riverine Flooding Analysis
III. Localized Drainage Analysis
IV. Reservoir Capacity Analysis
V. References

Each section is complete in describing the technical approach used, analysis of test results achieved, and conclusions and recommendations pertaining to the specific aspects of drainage that was investigated. The tables and exhibits in the right-hand bound volume are arranged accordingly to permit the reader ease in reviewing the tabulated information while reading the text.

The number of subsidence cases tested combined with the number of channel systems evaluated in the riverine flooding analysis resulted in very long, voluminous tables of data. To avoid confusing the reader with the full extent of data when reviewing the report, the text refers to, and the tables reflect, only selected cases on selected channels. The complete documentation of all the test data for the riverine analysis is contained in a technical appendix in the back of the tables and exhibits volume.

CHAPTER II

CHAPTER II RIVERINE FLOODING ANALYSIS

INTRODUCTION

Overview

Houston's early development occurred in the eastern portion of the City and Harris County along the Houston Ship Channel. As stated in Chapter I, this early residential and industrial development, served primarily by groundwater, created demands on groundwater resources that resulted in localized subsidence of as much as 8.0 to 9.0 feet in the Baytown and Pasadena areas from 1906 to 1978. The decreases in ground elevations occurred in areas that, because of their low elevation relative to sea level, were subjected to increased flooding due to tropical storm surges. As ground elevations in the coastal zone decreased, the area impacted by these tidal and storm surges increased.

During this same period the subsidence in the western portions of the City and County ranged between 1.0 to 4.0 feet. This differential in subsidence across the City created for the most part relatively minor increases in stream gradients for the major drainage channels in Houston which primarily flow from west to east. Although not evaluated in detail in previous studies, it was assumed that the impact of subsidence on flooding of the inland areas was not significant, and the major emphasis of concern continued to be on the coastal areas. In 1976, with the availability of water from Lake Livingston, a much more intensive conversion from groundwater to surface water began in the eastern portion of the metropolitan area to serve the heavy industrial demands of the area. As a result of this conversion, a significant decline in the subsidence rate in this eastern area has occurred, limiting the worsening impacts of tidal storm surges.

With the reductions in groundwater pumpage in the eastern metropolitan area and rapid growth to the west and southwest, served exclusively by groundwater, the area of most rapid subsidence has moved westward. This movement of the center of the cone of subsidence requires an understanding of the impacts of subsidence on inland or riverine flooding conditions so that appropriate steps can be taken by the sponsors in the management of this problem.

Subsidence in coastal areas can be directly correlated to an increase in tidal flooding. While the land surface is lowered, the sea level and storm surge levels remain constant. Thus, each foot of subsidence results in an increase depth of flooding of one foot. However, in the areas that are not subject to tidal influence, the relationship between subsidence and flooding is not so evident. In riverine flooding, the channel capacity and rate of flow, rather than the tidal elevation, are the controlling factors. Channel capacity is primarily a function of the geometry of the channel cross-section and the slope of the energy gradient for a given flow. Of these two parameters, only the slope of the energy grade line is significantly impacted by land subsidence. Unless extremely severe differential subsidence occurs, the changes in any channel cross-section is so insignificant that no discernible impact on a cross-section can be reflected in an analysis. The energy grade line, however, extends for the entire length of the channel and is directly related to the ground elevations. Therefore, relatively minor changes in ground slope, when extended for the length of a stream channel, can have significant impacts on the slope of the energy gradient and, thus, the channel capacity.

Flow rate in a channel is a function of several factors including the time of concentration (how quickly water gets to a stream and travels down the stream) and the quantity of stormwater that is within the channel and its adjacent flood plain (generally referred to as storage). Since the time of concentration can be affected by ground and channel slope and since the storage is dependent upon the configuration or geometry of a channel cross section and the depth of flooding, it becomes evident that flow rate and channel capacity are not independent.

Increased slopes will result in increased channel capacity. As a result, subsidence which steepens the slope of a channel would be expected to decrease the elevations of flooding for a given discharge. This lower flood elevation is accompanied by decreased storage since storage is related to water surface elevation. The net result of these changes is an increase in the quantity of stormwater which will reach downstream channel sections. This increased flow will result in an increase in downstream flood elevations unless the increased channel capacity (resulting from increased channel slope) is sufficient to offset the effect of the greater flow.

A decreased channel slope results in decreased channel capacity; therefore, subsidence which decreases slope would be expected to increase the elevation of flooding. But, as discussed above, the storage is also affected by this change in elevation (increased for increased elevations), and the rate of flow would then be decreased in downstream sections of the channel. Because of the interrelationship among these factors, the net impact of subsidence on flooding was unknown, although it has been generally assumed that increased slopes would result in decreased flood elevations and decreased slopes would result in increased flood elevations.

A third situation potentially exists, where a subsidence pattern results in an increased slope on one portion of a channel and a decreased slope on another portion. The net result of changing slopes on channel capacity, time of concentration, and storage for this situation is even less clear than the simpler cases described above. It is important, however, that these impacts also be understood since the current subsidence patterns are increasing the possibility of this situation occurring.

Primary Objective

The Riverine Flooding Analysis phase of the overall study of the relationships between subsidence and flooding has been performed to evaluate the effects of subsidence on inland drainage systems not influenced by tidal conditions. This analysis includes an investigation of numerous simulated subsidence conditions imposed on various channels in the Houston area riverine system, and an evaluation of the specific impacts resulting from each set of assumed subsidence conditions.

The primary objectives of the Riverine Flooding Analysis are as follows.

(1) Quantify the impacts of the simulated subsidence conditions on the channels analyzed in terms of change in storm discharges, flood plain area, and depth of flooding.

(2) Define the relationships which exist between the subsidence imposed and the resultant impacts, and determine if a generalized procedure can be developed to predict the potential impact of subsidence on any watershed.

(3) Develop guidelines for future analysis of these systems.

(4) Quantify the impact of the simulated subsidence on Brays Bayou in terms of dollars of flood damage incurred under existing conditions as opposed to the subsided condition.

Study Area

Diversified watersheds were selected for study in the Riverine Flooding Analysis to develop a comprehensive base of data on the impacts of subsidence on flooding and to compare impacts between channels with varying characteristics. Two groups of watersheds were selected for analysis: watersheds downstream of Addicks and Barker Reservoirs which are nearing full development with significant existing flooding problems, and watersheds upstream of the reservoirs which are currently much less developed than the downstream watersheds and with much less significant flooding conditions.

The following specific watershed systems were included in this study.

- Buffalo Bayou (Ship Channel Turning Basin to Barker Dam)
- Brays Bayou (Including Keegans Bayou)
- Sims Bayou
- Addicks Reservoir Major Tributaries (Horsepen Creek, Langham Creek, Bear Creek, and South Mayde Creek)
- Barker Reservoir Major Tributaries (Mason Creek and Willow Fork of Buffalo Bayou)

Exhibit II-1 presents a vicinity map of the selected watershed systems. A detailed description of each watershed follows.

Buffalo Bayou

The Buffalo Bayou watershed downstream of Addicks and Barker reservoirs drains an area of approximately 101.4 square miles, excluding the White Oak Bayou watershed, and spans the central core of the City of Houston from the Ship Channel turning basin west to Barker Reservoir. Most of the drainage area is located within Harris County with the exception of the Clodine Ditch subwatershed, which is located in Fort Bend County south of Barker Reservoir. The bayou meanders naturally from the Ship Channel to the confluence of Rummel Creek. Upstream of Rummel Creek to Barker Reservoir it has been straightened and realigned. The studied portion of the channel has a length of 32 stream miles. Historically, as presented on Exhibit I-1, the watershed experienced subsidence ranging from about 6 feet in the vicinity of the Houston Ship Channel to about 2 feet near Barker Dam.

The majority of the watershed, except the Clodine Ditch subwatershed, lies within the Houston city limits and is nearly fully urbanized. The eastern portion of the watershed consists mostly of commercial, office, and light industrial development and is drained by an extensive network of storm sewers. The western portion of the watershed is largely single-family and multifamily residential development with open-channel drainage and storm sewer laterals.

Even though the Buffalo Bayou watershed is nearing full development, significant channel improvements on the mainstem have been limited and have not kept pace with the rapid urbanization of this watershed, resulting in a substantial residual flood plain area. The tributaries of Buffalo Bayou have all been improved to some degree to provide for improved drainage into the mainstem.

Brays Bayou

The Brays Bayou watershed, an area of approximately 129.5 square miles, is situated in the southwestern portion of Harris County and the northeastern portion of Fort Bend County, Texas. The channel is approximately 26 miles in length, extending easterly from Fort Bend County to its confluence with the Houston Ship Channel just downstream from the City of Houston's central business district. The Brays Bayou watershed is similar to the Buffalo Bayou watershed in development pattern and density. Subsidence within the watershed has ranged from about 6 feet near its confluence with Buffalo Bayou to less than 2 feet in the upper reaches near Barker Dam.

Extensive channel rectification has occurred in the Brays Bayou watershed in an attempt to accommodate increased flood flows resulting from urbanization. Except for the uppermost segment of upper Brays Bayou and a portion of Keegans Bayou, the mainstem and most of its tributaries have been straightened, widened, and deepened. Upstream of the Keegans-Brays confluence, the mainstem is a trapezoidal earthen channel; downstream of this location, a major portion of the channel is concrete-lined and represents one of the most significantly improved major flood control systems in Harris County. However, these improvements still do not provide adequate carrying capacity for the highly urbanized area's run-off and, as a result, flooding potential still exists along many of the channels.

Sims Bayou

The Sims Bayou watershed is located in south central Harris County and drains an area of approximately 92.5 square miles. The watershed is approximately 25 miles in length and extends easterly from just west of the Fort Bend-Harris County line to its confluence with Buffalo Bayou east of IH-610 East. Subsidence within the watershed has ranged from a little over 6 feet near its confluence with the Houston Ship Channel to about 2 feet near the Harris County line. The Sims Bayou watershed downstream of IH-45 is intensely developed, with single-family and multifamily residential dwellings the predominate land use, although some heavy industrial use also exists. Upstream of IH-45 the watershed is moderately developed as residential with scattered commercial sites.

The mainstem of Sims Bayou has been rectified to various degrees throughout its length, but less than necessary to provide adequate carrying capacity for flood flows. As a result, flooding potential exists throughout much of the length of the main channel. The tributaries have been improved to better accommodate the urban storm flows. A major channel rectification project is underway on the lower end of Sims Bayou downstream of IH-45 which will eliminate out-of-bank flooding in this reach for storms up to and including the 100-year storm. Plans are underway to extend these improvements upstream as funds become available. All channels are trapezoidal earthen channels, although some portions may be concrete-lined for erosion protection at confluences.

Addicks Reservoir Major Tributaries

The Addicks Reservoir watershed, located west of Addicks Dam, comprises approximately 136 square miles of drainage area. Runoff is collected by five subwatersheds within the reservoir drainage area: South Mayde, Bear, Horsepen, Langham, and Turkey creeks. Of these, only South Mayde, Bear, Horsepen, and Langham creeks were studied in detail. The collected runoff is discharged into Buffalo Bayou and outfalls 32 miles downstream into the Houston Ship Channel. Subsidence within the watershed has ranged from about 2 feet near the reservoir outfall to less than 1 foot in the upper areas. Approximate subwatershed drainage areas follow:

Subwatershed	Drainage Area
South Mayde Creek	40 square miles
Bear Creek	31 square miles
Horsepen Creek	18 square miles
Langham Creek	37 square miles
Turkey Creek	10 square miles

Most land development in the watershed is single-family residential and has progressed upstream with the City's westward growth. The upper reaches of the watershed remain primarily in agricultural use. The smaller tributaries in the upper reaches of Addicks Reservoir watershed are relatively unimproved. Most of the major creeks located adjacent to developed areas have been modified to accommodate flow due to increased runoff. Channel modifications have transformed natural creeks to earthen trapezoidal channels. Flooding of developed areas which has occurred has been in the vicinity of the reservoir boundary as a result of inadequate improvements. This problem is being resolved, and rectification projects are planned or underway on each major tributary to eliminate these flooding conditions.

Barker Reservoir Major Tributaries

The Barker Reservoir watershed is in the western portion of the Houston metropolitan area draining portions of Harris, Fort Bend, and Waller counties. The contributing drainage area of the watershed is approximately 130 square miles. Runoff is collected by two major subwatersheds within the watershed: Willow Fork of Buffalo Bayou (111 square miles) and Mason Creek (19 square miles). Mason Creek has been constructed as a trapezoidal section, whereas Willow Fork has not been improved since the 1950s, when it was designed to accommodate agricultural runoff. Additional channel improvements are currently under construction in the lower reaches to accommodate drainage from planned urban development. The existing watershed is almost entirely undeveloped, with the exception of some areas along the reservoir boundaries and the Katy Freeway (IH-10). Subsidence within the Barker Reservoir watershed has ranged from about 2 feet near Highway 6 to less than 1 foot in its upper reaches.

TECHNICAL APPROACH

Overview

Limited investigations have previously been accomplished that address the effects of subsidence on riverine drainage systems in the Houston area. Hydraulic studies that incorporate ground elevation changes due to subsidence have generally been on such a localized scale that no area-wide study has been possible. In 1984, the Harris County Flood Control District (HCFCD) developed a series of hydrologic and hydraulic models of twenty-one watersheds in the area that reflected watershed conditions at a common baseline date. These models, developed as part of the Harris County Flood Hazard Study, defined the characteristics of land use and channel rectification as of the year 1982. More importantly, the hydraulic models were all based on field surveyed cross-sections with the common datum of the 1973 adjustment to the National Geodetic Vertical Datum of 1929 (NGVD, 1973 adjustment). As a result, the channel profiles established by these models could all be varied to simulate the effects of subsidence on an area-wide basis and still be at a common datum, thereby permitting comparison between the watersheds.

This riverine flooding analysis was performed on nine channels in five major watersheds integral to drainage in the Houston area. The five watersheds studied, shown on Exhibit II-1, are Brays Bayou, Sims Bayou, Buffalo Bayou, Addicks Reservoir tributaries (Langham Creek, Bear Creek, South Mayde Creek, and Horsepen Creek), and Barker Reservoir tributaries (Mason Creek and Willow Fork of Buffalo Bayou).

With the exception of the models on Willow Fork of Buffalo Bayou (Willow Fork) and Brays Bayou upstream of Gessner Road, the hydrologic (HEC-1) and hydraulic (HEC-2) models used in this analysis were provided by the Harris County Flood Control District. The models furnished were originally developed for the Harris County Flood Hazard Study and reflect the existing (1982) conditions in the watershed as previously described. Similar hydrologic and hydraulic models for Willow Fork were provided by the Fort Bend County Drainage District. The base condition models for Brays Bayou upstream of Gessner Road were updated as part of this study.

The hydrologic models utilize the Corps of Engineers' computer program HEC-1, "Flood Hydrograph Package." Storm flows are computed using the Clark's unitgraph method and the modified-Puls routing technique. The Clark's unitgraph coefficients were computed using the procedure described in the Harris County Flood Hazard Study, September 1984. The procedure developed in that study uses the physical parameters of watershed slope, channel gradient, channel conveyance, watershed shape, watershed ponding, and urbanization to compute the time of concentration (TC) and storage (R) coefficients for the Clark's unitgraph. Of these six watershed parameters, only changes in channel gradient due to subsidence were considered in the determination of the Clark's unitgraph coefficients.

Changes in system storage due to changes in gradient were accounted for through the flood routing techniques provided in the HEC-1 model. The modified-Puls flood routing technique selected for use assumes an invariable relationship exists between channel storage (flooded area) and channel flow. The Puls method of routing was initially designed for reservoir routing and adapted for channel use. This flood wave routing technique is accomplished using the hydraulic models as a means of computing the volume of water storage within a specified reach for a given channel flow which is then input to the HEC-1 model. The HEC-1 model uses the continuity relationship between the discharge rate of the flood wave entering the channel reach over a time increment and the volume of water within the reach to produce an outflow rate from the channel reach over the same time period.

The hydraulic models utilize the Corps of Engineers' computer program HEC-2, "Water Surface Profiles." Water surface profiles for stream channels are computed considering varying channel cross-sections and hydraulic structures such as bridges or culverts. Normal depth was used as the starting water surface condition for backwater computations on all streams analyzed including those channels outfalling to the Houston Ship Channel subject to tidal fluctuations.

Simulation of Subsidence

Two basic subsidence simulations were provided by the Harris-Galveston Coastal Subsidence District (HGCSD) for analysis of the impacts on the riverine systems. The subsidence simulation shown in Exhibit II-2 (contained in a pocket in the tables and exhibits section) was the primary condition of areal subsidence to be used in this analysis. It represents the projected subsidence pattern over 35 to 40 years assuming limited future surface-water conversions including the City of Houston's East Water Purification Plant and its current expansion and indicates subsidence of approximately ten (10) feet at the cone of maximum subsidence and zero to two (2) feet of subsidence at the outer fringes.

The second subsidence simulation utilized only for the more detailed analysis of Brays Bayou in the riverine analysis is representative of the projected subsidence from 1986 to 2020 resulting from implementation of the HGCSD's Plan. This subsidence simulation is illustrated in Exhibit II-3. The HGCSD's Plan was adjusted in the analysis phase to represent subsidence from 1973 to the year 2020 to allow comparison with other subsidence conditions and for mapping of the impacts of the plan. The maximum subsidence over the 1973 to 2020 period for the HGCSD's Plan is approximately five (5) feet.

Rather than attempt to limit the scope of analysis to subsidence reflecting limited conditions of groundwater withdrawal, or conversely, to evaluate the impact of several subsidence simulations of unrelated conditions of magnitude or location, the procedure used by this study followed a controlled variation of the primary condition of areal subsidence that would permit an evaluation of the sensitivity of magnitude and location of subsidence. The subsidence simulation provided was varied both in location and magnitude to derive additional cases for investigation on the Brays, Buffalo, and Sims Bayou watersheds. The slope of these subsidence contours was then used for selection of the channel gradient changes to be modeled in the upper watersheds of Addicks and Barker Reservoirs.

Procedure

With the models reflecting existing watershed conditions provided, and the subsidence simulations to be analyzed defined, the procedure used to obtain the results of the analysis is as follows:

- STEP 1. The cross-section data in the HEC-2 models were modified to reflect the change in elevation corresponding to the subsidence case being simulated.

- STEP 2. The HEC-2 models were executed over a range of storm flows to determine the storage-discharge relationship for each channel reach for use in the modified-Puls channel routing technique.

- STEP 3. The subsidence information was used to change the channel slopes in the HEC-1 sub-watersheds and new Clark unitgraph coefficients were defined. (After initial investigations, this step was eliminated because the change in unitgraph coefficients proved insignificant to the results.)

- STEP 4. The revised storage-discharge relationships were input to the HEC-1 model to reflect the subsided conditions, and new storm flows were computed.

- STEP 5. The storm flows computed in HEC-1 were input into the revised HEC-2 models and new water surface profiles computed.

- STEP 6. In mapping the flood plain and flood profile of the example stream (Brays Bayou), the flood profiles from HEC-2 were adjusted to reflect a datum of 1973 adjustment, NGVD.

The results obtained from the cases studied, using this procedure, were then evaluated to identify trends and relationships between subsidence and flood plain-flood flow parameters.

Storm Events Analyzed

The 10-year and 100-year frequency storm events have been analyzed for each subsidence condition and compared to the base condition for each stream. The rainfall amounts used in modeling are the 10-year and 100-year 24-hour point values taken from National Weather Service publication TP-40, "Rainfall Frequency Atlas of the United States." Point values were adjusted for drainage area using Figure 15 of that same publication. The rainfall has been distributed using the Corps of Engineers' critical pattern of alternating intensities before and following the peak value. The point rainfall values used for the riverine watersheds are presented in Table II-1.

The empirical exponential loss rate method, which relates loss rate to rainfall intensity and accumulated losses, was used in the HEC-1 models and was determined from storm verification runs performed as part of the Harris County Flood Hazard Study. The exponential loss rate parameters used are summarized in Table II-2.

Application of Hydraulic Models

For areal subsidence case analysis, appropriate cross-section elevation changes were determined by linear interpolation between the locations where the areal contour lines crossed each stream channel. The HEC-2 models were modified using the elevation adjustment option located on the section geometry specification cards. All bridge section data was remodeled to agree with the modified cross-sections. Cross-sections of channels were subsided uniformly so that the impacts of skewed channel characteristics were not introduced into the analysis.

Modification of the channel slopes for Buffalo Bayou, Brays Bayou, and Sims Bayou involved locating the cone of subsidence at various locations along the channel length. Exhibit II-4 shows typical stream gradient adjustments for the cases modeled. Modification of reach slopes on the major tributaries of the Addicks and Barker Reservoirs comprised a constant adjustment of the slopes to form flatter or steeper stream gradients. For flatter gradients, the downstream ends of the HEC-2 models were assumed to remain unchanged and for steeper gradients, the upstream ends were assumed to remain unchanged. Specific cross-section elevation changes were interpolated along the slopes, and modifications to the HEC-2 models were performed as indicated for the areal subsidence cases.

For mapping of the flood plain area in the Brays Bayou watershed, the City of Houston monumentation maps were used where available. The subsidence condition adjusted water surface profiles computed in the HEC-2 analyses were readjusted to 1973 datum for mapping purposes. Throughout the riverine analysis, flood plain areas were used for comparison of the impact of subsidence on various channels. These flood plain areas were derived from the output of the HEC-2 computations for top width area of flooding based on depth of flow and channel sections.

The flood damage analysis was performed utilizing stage-damage curves from the U.S. Army Corps of Engineers standard project flood analysis of Brays Bayou in conjunction with the flood stages determined for this study.

Application of Hydrologic Models

The HEC-1 models comprise numerous subwatersheds describing the drainage patterns within the entire watershed. Many of the subwatersheds, as indicated on Exhibits II-5 and II-6, are associated with tributary channels of the main stream. The study approach initially concentrated on the impact that changes in channel gradient would have on storm flows. As previously indicated, these changes in storm flows could be related to the changes in Clark's unitgraph coefficients TC and R as well as the storage-discharge relationship defined in the modified-Puls flood routing technique.

According to the hydrologic method established in the Harris County Flood Hazard Study and as used in this analysis, both the unitgraph coefficient relating to time of concentration (TC) and storage (R) are inversely proportional to the square root of the channel gradient. Consequently, an increase in the channel gradient should result in lower values for both parameters. Conversely, a lessening of the channel gradient should result in an increase in both parameter values. Table II-3 summarizes the typical range of change experienced by several unitgraph coefficients representing subwatersheds in the Willow Fork watershed for a 25 percent flatter channel gradient. Table II-4 shows the corresponding change in peak storm discharge from each subarea. Table II-5 presents the existing condition storage-discharge relationships for Willow Fork and Table II-6 presents the storage-discharge relationships resulting from the change in gradient. Table II-7 indicates the change in the Willow Fork channel flow resulting from changes in just the storage-discharge relationships and with both changes in the storage-discharge relationships and unitgraph coefficients. By comparing the resultant change in storm flows due to the change in unitgraph in unitgraph coefficients and changes in storage-discharge relationships versus the change due to just the storage-discharge relationships, it was concluded that the changes in unitgraph coefficients were insignificant to the study results. As a consequence, no further changes to the unitgraph coefficients were included in the analysis of the remaining subsidence cases.

ANALYSIS OF RIVERINE SUBSIDENCE

Overview

The impacts of subsidence on the various watersheds have been quantified by changes from the base condition for storm flows, flood plain area, and depth of flooding. The results are shown graphically as a series of trends summarizing the analysis performed on all watersheds. In addition, specific data is presented for Brays Bayou through flood plain mapping, stream and water surface profile plots, and a tabulation of monetary damages projected based on relationships of stream stage versus monetary damage developed by the U.S. Army Corps of Engineers. Reaches of relatively constant bottom slope were identified for each length of channel studied. The analysis of change in peak discharge and flood plain area was performed by evaluating the average change in channel gradient in each reach resulting from the subsidence cases studied. A total of 28 reaches were identified and are summarized in Table II-8.

Selection of Subsidence Cases

The technical approach used in this analysis was composed of two parts. The first part was an evaluation of idealized situations targeted at identifying and quantifying what parameters are affected by gradient change. To perform this portion of the analysis, the watersheds upstream of the Addicks and Barker reservoir system were selected for investigation because they exhibit a relatively wide range in characteristics of slope, urbanization, and channel rectification. At the same time, they are not nearly as urbanized as watersheds downstream of the reservoirs and do not exhibit as many obstructions to flow such as bridges or pipeline crossings. By avoiding the complex flow patterns created by obstructions, the response of the individual physical channel characteristics to gradient change could be more easily identified and described.

A total of 30 cases of gradient change were evaluated on the six study channels upstream of the reservoirs. These cases included increasing the existing channel slopes by steepening as much as 25 percent or decreasing the slope as much as 50 percent. This range of change was selected to represent the maximum slope of the subsidence cone realized by the condition represented by Exhibit II-2 and comparing this slope of subsidence to channel gradients in the Houston area. The maximum differential slope caused by subsidence depicted in Exhibit II-2 is approximately 0.00017 foot per foot (0.9 foot per mile). Channel gradients typically range from .0005 to .003 foot per foot. As a result, the range of 25 percent increase to 50 percent decrease was representative of the maximum change that could be anticipated.

Of the 30 cases studied, 20 were used to simulate a uniform change in gradient in a single direction to characterize a subsidence condition centered outside of the watershed boundaries. Combinations of steepening and flattening on the same channel were also evaluated to simulate a condition of a subsidence cone located along the channel lengths in the South Mayde Creek and Willow Fork basins. Exhibit II-4 graphically depicts the modifications to channel gradients resulting from a typical subsidence situation. In addition to examining the impact on the existing channel, Willow Fork was simulated as a trapezoidal section constructed according to current HCFCD design criteria. This "redesigned" channel was then evaluated against the same cases as the existing Willow Fork channel in an attempt to define the impact of subsidence on channelized reaches compared to natural reaches.

The second portion of the analysis involved investigations of the more complex watersheds downstream of the reservoirs and their response to more complex conditions of gradient changes. The watersheds analyzed were Brays Bayou (including the subsidence-related impacts from Keegans Bayou), Sims Bayou, and Buffalo Bayou. The evaluation of the downstream channels was accomplished using the projection of a subsidence cone across the watershed rather than a uniform gradient change, as used in the upper watersheds. The cone simulation reflects the areal distribution of subsidence and as a result is considerably more complex to evaluate.

The location and magnitude of the primary case of subsidence provided by HGCSD was varied so that 17 additional investigations were performed. For ten of the investigations, the primary subsidence condition was shifted to varying points within each watershed to identify the channel response if the largest magnitude of subsidence was to occur in the upper, middle, or lower portion of the watershed. By considering the same subsidence case in each situation, the significance of the location of subsidence to watershed response could be evaluated. An additional seven cases were investigated by varying the magnitude of subsidence at common locations in an effort to identify any relationship between magnitude of subsidence and increase in flood levels.

Exhibit II-7 indicates the location of the center of the subsidence cones used in Cases 1, 2, 3, and 4, and described in Table II-9. Each case simulation can be visualized by shifting the areal subsidence overlay (Exhibit II-2) to the center location on Exhibit II-7 and keeping the north arrows parallel.

An eighteenth investigation was also performed on Brays Bayou for the specific purpose of identifying the potential impact of subsidence on flood damages in a watershed. For this investigation, the HGCSD's Plan for subsidence through the period of 1986-2020, as shown in Exhibit II-3, was applied. The plan was extrapolated to incorporate historic subsidence from 1973 through 1986 since the base hydraulic models used to simulate the watersheds were all referenced to 1973 datum.

By using this two-step approach, the relationships identified using the simplified conditions upstream of the reservoirs could be used to provide explanations to some of the phenomenon that occur downstream of the reservoirs. Table II-10 summarizes the cases that were analyzed for each of the ten study channels. It should be noted that only the mainstem of each watershed was analyzed. Potential changes in tributary slopes, drainage areas, and watershed divides that may be created as a result of subsidence were not included in this analysis.

<u>Subsidence and Its Effect on Storm Flows</u>

The 10-year and 100-year frequency peak storm flows were computed for each case of subsidence that was analyzed. Data were evaluated in terms of absolute values and the percent change in values. Exhibit II-8 summarizes the relationship between the percent change in channel gradient and the percent change in average peak storm flows along the channel. Tables summarizing the storm flows resulting from each analysis case are included in the Technical Appendix. In general, increases in gradient resulted in increases in discharge capacity along the channel and, conversely, decreases in gradient caused decreases in discharge.

From Mannings Equation, channel discharge capacity (Q) is related to the channel slope by the equation

$$Q = 1.49\ AR^{2/3}\ S^{1/2}/n \qquad \text{EQ.(1)}$$

or Q is a function of slope when the geometry of the section and flow depth remain unchanged

$$Q = f\ (S^{1/2}) \qquad \text{EQ.(2)}$$

Accordingly, changes in channel slope should correspond to a change in channel discharge capacity by the relationship

$$Q_2 = f\ [Q_1\ (S_2/S_1)^{1/2}] \qquad \text{EQ.(3)}$$

Exhibit II-8 shows a consistency with EQ.(3) in that increases in channel slope resulted in increases in discharge capacity. Correspondingly decreases in slope were accompanied by decreases in discharge capacity. In all cases, however, the magnitude of change in discharge capacity was significantly less than theorized by EQ.(3). The difference between the observed data and theoretical equation can be attributed to the change in channel storage and conveyance associated with a change in channel slope.

Exhibit II-8 indicates that channels with steeper slopes conform more closely to the theoretical relationship than do those with flatter slopes. This can be associated with the fact that steeper slopes are often associated with narrower flood plains (less storage) and require less section conveyance to pass a given channel flow than do channels with flatter slopes. Exhibit II-8 indicates that a relationship may exist due to the condition of the channel (either rectified or not). It was found, however, that the three streams reflected natural flooding conditions better than rectified conditions, though only one stream is a natural channel. This trend is generally due to the inadequacies of the existing channel improvements.

Exhibit II-9 confirms the conclusion that channel storage and section conveyance accounts for the differential between observed and theoretical changes in discharge capacity. A rearrangement of Mannings equation, EQ.(1), will give

$$Q/S^{1/2} = 1.49\ \frac{AR^{2/3}}{n} \qquad \text{EQ.(4)}$$

The right hand side of EQ.(4) is frequently referred to as hydraulic conveyance. For purposes of this study, the left-hand side of the equation is referred to as the flooded section factor required to pass a given storm flow. The flooded section factor conforms to the same relationship noted in EQ.(3). Exhibit II-9 describes the increase (or decrease) in the flooded section factor associated with a change in slope. Similar to that witnessed in the change in discharge, the change in flooded section factor is less than would be predicted by the theoretical equation. This indicates the shared participation in discharge and channel storage, or flooded section factor, in responding to channel slope changes created by subsidence.

The size of the contributing drainage area also had an impact on the degree of change in discharges caused by changes in stream gradients for both 10-year and 100-year storms. Generally, the larger the contributing drainage area, the greater the extent of the change in discharges for each gradient change condition as shown on Exhibit II-10. For flattened gradients, discharges generally decrease and the magnitude of decrease is greater for larger drainage areas. Similarly, for steepened gradient, discharges generally increase and the magnitude of the increase is greater for larger drainage areas. The relationship between magnitude of increase or decrease and drainage area is a result of the cumulative effects of changes in channel storage along the stream. Generally, the larger the drainage area, the longer the length of channel involved and the greater the cumulative effects of gradient changes on channel storage.

Subsidence and Its Effect on Depth of Flooding

Unlike tidal areas, where a unit change in ground elevation is accompanied by a unit increase in flooding depth, subsidence effects on riverine flooding depths does not appear as significant. Exhibit II-11 shows the average change in depth of flooding compared to the change in channel gradient. In general, increases in gradient resulted in decreases in the depth of flooding and conversely decreases in gradient caused increases in the depth of flooding. This can be related to the fact that steeper slopes require less section conveyance (less depth of flow) to pass a given flow, and flatter slopes require more section conveyance (more depth of flow). No general trends could be identified between the existing channel gradients, or channel type, and the resultant change in depth of flooding because of the anomalies produced by localized conditions of the streams. However, the maximum increase in depth of flooding was observed to occur downstream from the center of subsidence cone in all cases.

For the 48 cases studied, the maximum increase in flooding depth was found to be less than 1/3 of the related ground subsidence. An average of the conditions tested indicates that in areas where increased flooding occurred, the increase in flooding depth was 1/10 of the related subsidence. A similar magnitude of impact was found to occur in conditions when flood levels decreased. Of these 48 cases, 26 were of the simplified condition of either steepening or flattening the channel slopes and 22 were of the more complex cone simulations. The subsidence simulated for the 26 simplified cases ranged from 1.4 to 39.2 feet with a maximum increase in depth of flooding ranging from -1.7 to 4.0 feet for the 100-year event. The subsidence simulated for the 22 cone cases ranged from 2.6 to 12.4 feet with a maximum increase in depth of flooding ranging from 0.0 to 2.3 feet for the 100-year event. Table II-11 presents the magnitude of subsidence along each watershed for the cases analyzed. Table II-12 summarizes the maximum subsidence within each watershed and the resulting maximum increase in depth of flooding for the 100-year event. Exhibit II-12 presents this data graphically.

Subsidence and Its Effect on Flood Plain Area

The comparison of flood plain areas due to various subsidence conditions is of key interest in the Riverine Flooding Analysis. Changes in the flood plain areas are related to changes in depth of flow in the streams and the local topography in the reaches of depth changes. The changes in flood plain areas resulting from subsidence are due to changes in flows and changes in capacity of the streams to accommodate the flows. Exhibits II-8 through II-11 indicate that as stream gradients steepen, flows increase. At the same time, however, depths of flow and the flooded section factor decrease, indicating that flooded section factor decreases more rapidly than the discharge increases. As a result, the flood plain area generally is reduced. Conversely, flattening of the stream gradients will generally result in increases in flood plain areas, and a decrease in discharge rates. This increased flood plain reflects the combination of reduced ability of the channel to convey water and subsequent increased flood plain storage. Exhibit II-13 demonstrates the impact of gradient changes on flood plain area for the 10-year and 100-year storm events for all the watersheds. In all cases, flood plain area increased when the channel slope was decreased or flattened and flood plain area generally decreased for increased channel slopes.

The relative slopes of the lines shown on Exhibit II-13 indicate that the rate of change in flood plain area increases faster for a given decrease in channel gradient than does the rate of decrease in flood plain area for a given channel gradient increase. The difference in these rates are reflected in the relative impact of the location of subsidence on the magnitude of flooding. Exhibit II-13 indicates that the further upstream the center of subsidence is located, the greater the extent of potential flood plain increase due to the longer reach of channel being flattened downstream.

A Procedure for Estimating the Effect on Flood Plain Area

Using the data and trends developed in this investigation, a procedure was developed which estimates the magnitude and location of the maximum change in the flood plain area (FPA) anticipated by a given condition of subsidence. The data used to develop this procedure was derived from the analysis conducted and data collected on the upstream watersheds. The procedure was then tested using the results obtained from subsidence case studies conducted on Brays Bayou, Sims Bayou, Buffalo Bayou, South Mayde Creek, and Willow Fork. The procedure utilizes the change in channel gradient created by a condition of subsidence combined with the location of the center of the cone of subsidence, and determines the centroid of the positive and negative slope changes to find the location and magnitude of the largest change in flood plain width.

The procedure, presented graphically on Exhibits II-14 and II-15, is described by the following steps:

° Step 1. Divide the stream into constant slope reaches and determine the weighted average slope of the stream.

° Step 2. Select a subsidence case and establish the center of subsidence along the stream. Determine the slope of the selected subsidence case upstream and downstream of the center.

° Step 3. Determine the percent change of the constant slope reaches, caused by the selected subsidence case, by dividing the slope of the subsidence by the slope of the constant slope reach.

° Step 4. Enter Exhibit II-14, Figure 1, with the weighted average slope of the stream to determine, for positive and negative changes, the correct slope of line to be used from Exhibit II-13.

- Step 5. Enter Exhibit II-13 with the percent change for each constant slope reach, along the line determined in Step 4, and determine the change in FPA.

- Step 6. Construct a diagram similar to the one presented on Exhibit II-14, Figure 2, depending on the number of constant slope reaches on either side of the center of subsidence. For one constant slope reach on either side of the center, plot the change in FPA at the center of the constant slope reach length and draw a line from zero change, at either the upstream or downstream end depending on the direction of change, through the change in FPA to the station at the center of subsidence. The value at this point is the contribution, from the respective direction established, to the total change in FPA. For two or more constant slope reaches on either side of the center, a weighted average change in FPA must be determined over the length concerned. This value is plotted at the center of mass of the constant slope reach areas and a line is drawn from zero through this point to the station at the center of subsidence.

- Step 7. The magnitude of the maximum change in FPA is determined as the sum of the absolute values of the negative and positive changes.

- Step 8. To predict the location of the maximum change in FPA, determine the location of center of mass of the two triangles constructed in Step 6. If the center of the subsidence is located at the center of the stream, take the moment of the areas around the center. If the center of subsidence is located upstream of the center of the stream, first find the areas (both upstream and downstream) for just the length equal to the distance from the center of subsidence to the upstream end, and then take the moment around the upstream end of stream. If the center of subsidence is located downstream of the center of the stream, do the reverse. The location of the maximum change is the resulting center of mass location.

- Step 9. An estimate of the change in flood plain area can be obtained from the areas computed under Step 8.

The test results, summarized in Table II-13, indicate the procedure estimated with relative success both the magnitude and the location of the point of Sims Bayou, Buffalo Bayou, South Mayde Creek, and Willow Fork experiencing the maximum change in flood plain width. In each case it is noted that the channel reach experiencing the largest increase in flood plain width was located downstream of the center of the subsidence cone. It was also noted that the further the center of the cone was downstream, the larger the magnitude of increase in flood plain width. Exhibit II-15 reflects the simulation of flood plain area on Buffalo Bayou for Case 3 comparing the results of the predictive procedure to that obtained by HEC-2 analysis. The exhibit shows a good comparison of the length of reach affected by increase and decrease in flood plain width.

The predictive procedure shows a smooth general trend of the average increase in cumulative flood plain area that can be anticipated, however, it does not reflect the section by section variations identified by HEC-2. These variations are attributed to bridge sections or locations where the flood flows may jump from being in channel to out of bank, and thereby have a significant percentage of increase in flood plain width. Although the change of these individual locations appear large, they are relatively insignificant to the impact on the overall channel and are considered an impact more of a structural constraint, rather than an impact of subsidence.

The procedure also reasonably predicted the length of channel reach affected and the total change in flood plain area. The differences on Brays Bayou appear to indicate that the technique has limitations when addressing basins where bridge losses or major tributary channels may cause large fluctuations in flood widths. The procedure is successful in predicting the location and magnitude of the largest increase in flood plain area but it cannot be used to identify the economic significance of these changes or the exact locations of flood plain changes caused by the impact of structures or major tributaries specific to a given watershed. The application of this procedure will best be used to screen alternatives to limit the number requiring detailed investigation.

Subsidence and Its Affect on the Brays Bayou Watershed

Flood Plain Analysis

The Brays Bayou watershed was the subject of a separate detailed investigation in the Riverine Flooding Analysis. A total of seven subsidence cases were analyzed on Brays Bayou, including three variations in magnitude of subsidence at the Case 2 center of subsidence, three variations in the magnitude of subsidence at the Case 3 center point, and a single case of subsidence at the Case 1 center point. These seven subsidence cases were compared to the base condition defined by the models provided by HCFCD. The District Plan of the Harris-Galveston Coastal Subsidence District was analyzed on Brays Bayou as one of the subsidence cases at the Case 3 center point and is labeled Case 3a in this study. The subsidence impacts on Keegans Bayou were also modeled and considered in the Brays Bayou analyses.

Station-discharge curves for the 100-year frequency storm were developed for the seven subsidence cases and base case of Brays Bayou and are shown on Exhibit II-16. As discussed earlier, steepened channel gradients generally produce increases in discharges and flattened gradients produce decreases in discharge levels. Subsidence cases centered at the upstream portion of the watershed are labeled Cases 3, 3a, and 3b. These cases result in steeper gradients and slightly increased peak discharges upstream of the center point. Conversely, the channel gradient was flattened and peak discharges lower downstream of this center point. The response of discharges to subsidence cases centered at the midstream of the watershed (Cases 2, 2a, and 2b) is similar to those cases at the upstream location--peak discharges are greater than the base condition upstream of the center point of subsidence where the gradient is steeper than the base condition, and peak discharges are less than the base condition downstream of the center point where the gradient is flatter than the base condition.

The response of peak discharges to gradient changes in the cases described above is consistent with the relationship of changes in discharge versus changes in channel gradient presented earlier in this section and shown in Exhibit II-8. The Case 1 subsidence condition located at the most downstream portion of the watershed also results in greater discharge levels upstream of the center point where the channel gradient has been steepened. However, unlike the responses developed for the watersheds above Addicks and Barker reservoirs, the discharges downstream of the Case 1 center point, where the channel gradient has been flattened, are greater than the base condition discharges. This response for the downstream subsidence case also occurs in the similar cases imposed on Buffalo and Sims Bayous. This result indicates the increased discharges from the lengthy channel upstream of the center point are of such a magnitude that they override the channel flow response downstream of the center point. The relatively short channel reach downstream of the center point with a flattened gradient and resultant reduced carrying capacity must, in this subsidence case, accommodate the significantly increased flows from the upstream channel reaches.

A review of the subsidence impacts on Brays Bayou, shown on Exhibit II-17, shows that, of all cases studied, Case 1 created the greatest increases in flood plain width downstream of the center of subsidence, but Case 3 resulted in the greatest cumulative increase in flood plain area over the entire channel. The flood plain area increase in Case 1 is the product of imposing significantly higher discharges on the downstream channel reach while dramatically reducing its carrying capacity through flattening of the channel gradient in a short reach of the channel. Though the Case 1 response to discharges in Brays Bayou does appear to be anomalous to the other subsidence case responses, its occurrence can be anticipated based on the analysis of the changes in channel flow characteristics for the downstream portion of the watershed.

The base condition and subsidence Cases 3 and 3a (HGCSD's Plan) for Brays Bayou were also analyzed in detail with regard to flood plain mapping, water surface profiles, and changes in monetary flood damages. Exhibits II-18 and II-19 show the base condition 100-year flood plain on Brays Bayou as defined by the HEC-2 analysis and mapping performed in this study. The City of Houston monumentation maps were used for flood plain delineations to take into account the most detailed topographic information available. Since this mapping source was not used in the Harris County Flood Hazard Study, the resulting flood plain delineation will not coincide in all cases with the Federal Insurance Rate Maps for Brays Bayou.

The flood plain exhibits also indicate the locations where the flood elevations produced in the analysis of subsidence Case 3 and Case 3a differed from the base condition by 0.25 foot or more. As shown, neither of these subsidence cases results in a substantial variation in 100-year flood plain from the base condition. Resultant water surface profiles for the base condition, subsidence Case 3, and subsidence Case 3a for both the 10-year and 100-year storm events are shown on Exhibits II-20 through II-25. Elevation differences between the base condition and the subsidence cases of less than 0.25 foot are not shown for reasons of clarity only. These profiles show that, for the 100-year storm event, a relatively small portion of the entire bayou length is impacted by changes in water surface elevation of more than 0.25 foot from the base condition for either subsidence case. However, for the 10-year storm event, a significant number of reaches are impacted. The dramatic increase in water surface rise for the 10-year storm event results because the existing channel carries the 10-year flow at or slightly below the top of the channel bank. As the channel subsides, the rise in water surface is considerably faster since the channel storage is relatively small, resulting in minimal attenuation of peak discharges. Conversely for the 100-year storm event, an extensive overbank flood plain exists which provides considerable channel storage and peak discharge attenuation. As a result, the subsided condition does not cause as large a flood depth increase. In many cases, the rise in water surface for the 10-year storm event results in a residual flood plain, whereas the existing water surface is confined within channel banks. These results are further reflected in the flood damage analysis.

Flood Damage Analysis

The Brays Bayou analysis also included a comparison of economic flood damage data for the base condition, Case 3 and Case 3a. Flood stage data were computed for the index station of each economic reach, and economic flood damages were then compiled using stage-damage curves developed by the U.S. Army Corps of Engineers. The economic reaches are shown on Exhibit II-26. The 10-year and 100-year frequency storms were analyzed. The base condition total damages were $36.1 million and $461.6 million for the 10-year and 100-year event, respectively. Table II-14 shows the incremental damages by reach as well as the total damages for the two storm events and three channel conditions included in this analysis.

For the subsidence cases analyzed, the 10-year storm event results in relatively dramatic increases in damages when compared to the base condition. Case 3 damages for this storm event are $165.4 million (458 percent of base), and Case 3a damages are $75.7 million (210 percent of base). The 100-year storm results in damages of $520.3 million for Case 3 (113 percent of base) and $486.2 million for Case 3a (105 percent of base). The large increases in damages in the 10-year storm flood plain compared to the base condition flood plain indicate that the base condition 10-year storm is at or near flooding levels in various segments of the channel and slight increases in water surface elevations produce significant widening of the flood plain and increase flood damages.

This analysis shows that the nature of the existing flow and localized topographic conditions is a significant component of the impact of subsidence on flood damages. Stream conditions that approach a critical plateau in flood damages may find that plateau exceeded and damage levels increased although the subsidence rate is drastically limited. Therefore, flood mitigation efforts that reduce the critical nature of the existing condition flood damage level can also reduce the potential impact of subsidence on flood damages. In that regard, a regional plan of improvements for Brays and Sims Bayous was developed in September, 1985 and adopted by the Harris County Commissioners Court. The Harris County Flood Control District is proceeding with implementation of this flood control plan. Two large regional detention areas recommended in the plan have been purchased and the design of an initial phase of improvements is underway. Construction of these improvements could begin in 1987.

The impact of subsidence with implementation of this plan was reviewed to gain an understanding of the potential for the mitigation of the effects of subsidence through flood control improvement programs. An analysis of this regional plan, with the HGCSD's Plan subsidence case imposed over it, showed a significant decrease in the amount of potential flood damages. Total potential damages in the 10-year storm event were essentially eliminated. The total damage impact of the 100-year storm is reduced from its existing level of $461.6 million ($486.2 million with the HGCSD's Plan) to $20.2 million for the period of 1973 to 2020 if the ultimate regional flood control plan is implemented. Implementation of flood control measures which reduce the potential for flooding will provide for some mitigation of the effects of future subsidence. It is probable that modifications in the flood control plan could further reduce or eliminate residual damage levels associated with projected subsidence.

CONCLUSIONS AND RECOMMENDATIONS

This chapter addresses the impacts of subsidence on riverine systems and generally focuses on changes in flood discharge, flood plain area, and depth of flooding. Subsidence which results in a flattening of existing channel gradients, or slopes, will cause increases in the depth of flooding. In no case, however, did the subsidence result in an equal or like change in the depth of flooding as is realized in coastal flooding resulting from tidal conditions. In fact, the maximum increase in flooding depth was found to be less than 1/3 of the related subsidence and the average of the conditions tested indicated increases of only 1/10 of the related subsidence.

The data developed during the course of this study shows that the impact of subsidence on the riverine system follows generally consistent patterns. Subsidence that creates steeper stream gradients will result in increased channel conveyance, increased peak flows, decreased channel storage, and decreased flood impacts. Conversely, flatter stream gradients due to subsidence will result in decreased channel capacity, increased flood storage, decreased peak flows, and increased flood levels. When a cone of subsidence is located within a stream system, decreased flood levels occurred upstream of the center of subsidence and increased flood levels occurred downstream. The distance of increase or decrease was dependent on the magnitude of subsidence.

A specific objective of this study was to determine if generalized relationships could be developed which would facilitate the prediction of the specific effects of subsidence on any watershed without the necessity of detailed hydrologic and hydraulic modeling. Such a generalized methodology was developed which projects the location of maximum flooding increase and the percent change in flood plain area resulting from any subsidence case. When compared to the results of detailed hydrologic and hydraulic modeling, the generalized methodology adequately predicted the general location where the maximum increase was observed as well as the percent increase in flood plain area. Where localized conditions produced anomalies such as multiple peaks in the flood level, the accuracy of the generalized methodology decreased.

It was concluded that the generalized predicted methodology should be used only for rough screening of alternative subsidence cases and detailed hydrologic and hydraulic modeling should be used for any final plan evaluation. Where hydrologic and hydraulic models are available, the modeling procedures developed during this study will allow a detailed evaluation of subsidence through watershed modeling with a level of effort not significantly greater than that required for use of the generalized predictive methodology but with significantly higher level of accuracy.

An evaluation of specific subsidence cases on the Brays Bayou watershed, including the HGCSD's Plan, reveals relatively small increases in the 100-year flood level and flood plain for the period 1973 to 2020, as well as a relatively minor increase in total flood damages over the same period. However, for the 10-year storm event, which is very near the current channel capacity of the system under existing conditions, projected increases in the 10-year flood plain and flood damages are substantial. The imposition of the HGCSD's Plan reduces significantly the magnitude of this increase, but does not eliminate these increased flood damages due to subsidence.

With implementation of the current flood control plan of improvements for Brays Bayou, which assumes full watershed development but makes no allowance for subsidence, flooding is essentially eliminated for both the 10-year and 100-year storm events. Imposing the future subsidence projected to occur with implementation of the HGCSD's Plan results in no increased flooding for the 10-year storm event. For the 100-year storm, increased flooding does occur with damage increases comparable to those increases which were projected to occur prior to imposition of the flood control plan. Thus, implementation of the current flood control plan for Brays Bayou could result in the elimination of any flooding impacts from future subsidence projected in the HGCSD's Plan for a 10-year storm event and, with some modifications of the flood control plan, could likewise eliminate increased flooding for the 100-year storm.

In summary, the conclusions of the Riverine Flooding Analysis are as follows:

- A foot of inland subsidence results in significantly less than a foot increase in riverine flooding depth.
- Impacts follow consistent patterns with decreased flood levels upstream of the center of subsidence and increased flood levels downstream.
- Channel characteristics and localized conditions preclude the ability to adequately predict flooding impacts with generalized predictive relationships.
- The planning methodology presented in this study facilitates specific analysis of potential subsidence conditions.
- Future subsidence impacts should be considered in development of flood control programs.

The current HGCSD's Plan for conversion from groundwater to surface water restricts significantly the future use of groundwater in an effort to control subsidence which can contribute to increased flooding. The HGCSD's Plan also recognizes the time required for such conversion by establishing varying time schedules for area conversions. The analysis of Brays Bayou presented herein focuses on the difficulty of eliminating increases in flooding resulting from subsidence through groundwater controls alone. However, flood control system improvements designed with consideration of anticipated future subsidence can mitigate the effects of subsidence on flooding.

The current HGCSD's Plan was developed with a focus on conversion from groundwater to surface water which would minimize future subsidence consistent with the City of Houston's long-term plan for constructing major surface water treatment and transmission facilities to serve the area. While the plan takes into account the financial and time constraints involved in the conversion process, adequate data was not available to evaluate the impacts on the flood control systems. This study has developed data to begin to evaluate those impacts. It is recommended that the joint planning effort be continued to define the specific impacts on flood control projects of the HGCSD's Plan as well as possible alternative groundwater withdrawal cases with the purpose of defining a subsidence plan which addresses both groundwater production and flood control improvements in plan development. The effort should review selected regional flood control projects and define the required modifications to fully mitigate impacts resulting from projected future subsidence cases and the cost for such modifications. Additionally, the costs of surface water conversion facilities for each case evaluated will provide the final data necessary for a determination of a cost-effective plan for addressing the long-term impacts of subsidence.

CHAPTER III

CHAPTER III

LOCALIZED DRAINAGE ANALYSIS

INTRODUCTION

Overview

As previously described, Houston's early residential and industrial development created heavy demands on groundwater resources which eventually resulted in widespread land-surface subsidence. The impacts of such widespread regional subsidence patterns on localized street or drainage system flooding were generally assumed to be insignificant with respect to coastal flooding, although specific data on the relationship between regional subsidence gradients and localized flooding was virtually non-existent. Similarly, while subsidence patterns were observed occurring around local groundwater well fields, no direct link between these local subsidence cones and surrounding localized flooding patterns had been studied or defined. With the increased conversion to surface water in eastern coastal areas and the shift of the regional subsidence pattern to the inland areas of the western metropolitan area, concern for an understanding of the relationship between subsidence and localized flooding patterns has been redirected. The impacts of regional subsidence and local well fields on street and local drainage system flooding must be defined in order for appropriate steps to be taken by the study sponsors in the ongoing management of potential future problems.

In evaluating flooding characteristics of a small watershed with respect to subsidence patterns, critical analysis parameters are similar to those found when studying a large riverine system. Channel slope is altered by subsidence patterns, which in turn affects channel water velocity and peak discharge. But the secondary storm sewer system and lateral channels may be opposingly aligned and thus complicate the assumed effects of subsidence on the more complex drainage system of the small urban basin. More specifically, the effects of subsidence on the design conditions of the secondary system rapidly become complicated by the physical factors of internal system backwater, storm sewer surcharge and pressurized flow, unsteady flow conditions, street ponding and related inlet flooding, etc. By far the most critical design parameter during flooding conditions is the capacity of the secondary system.

The severity of flooding due to the exceedance of the secondary storm sewer system capacity in Houston is witnessed throughout the metropolitan area during heavy rainfall. Studies dealing with the interrelationship of storm sewer systems and street flooding have concluded that the capacity of the storm sewer system to convey runoff into the respective outfall channels has a direct correlation to the subsequent level of street flooding. Secondary drainage system surcharge is, therefore, considered to be the critical analysis parameter when determining localized effects of subsidence on flooding and street ponding.

By computer modeling of the secondary drainage system under subsidence conditions, the effects of both regional subsidence patterns and local well field subsidence patterns on localized street flooding can be characterized. The effect of system design condition changes on flooding can be quantified by determining the magnitude of the subsidence-flooding relationship. In this way, an optimum placement of well fields in an urban area such as the Houston metropolitan complex may be recommended, and local problems associated with predictable regional subsidence patterns may be mitigated.

Primary Objectives

The localized drainage analysis phase of the overall study of the effects of subsidence on flooding in the Houston metropolitan area was initiated with a specific goal of producing well-defined guidelines for the optimum positioning of major water well sites with respect to local drainage patterns based on the impacts of subsidence created by these wells on the local drainage systems. As a result of preliminary analyses from all three phases of the project, this goal was expanded to include projections of localized flooding characteristics resulting from regional subsidence cases as well as those resulting from local well fields. Therefore, the primary objectives of the Localized Drainage Analysis phase of the project are defined as follows:

(1) Define the impacts that specific major water well fields may have on small watershed drainage systems;

(2) Develop generalized drainage standards for locating water wells based on (1) above;

(3) Determine the significance of projected area-wide subsidence on localized drainage;

(4) Present a comparison of current City of Houston design criteria vs. pre-1970's design criteria with respect to their effects on street ponding.

These goals were used to determine the project organization and to develop a procedural outline which would allow for each objective to be evaluated independently upon completion of the analysis.

Study Area

A watershed of approximately five (5) square miles was selected as representative of a typical small Houston drainage area and was evaluated on both the regional (macro) and local (micro) level. This watershed, known as the Bintliff Ditch drainage basin, has been monitored by the United States Geological Survey (U.S.G.S.) for over twenty years with a stream gage at Bissonnet Street. This gage provided data for calibration of the model to historical storms as recorded in the annual U.S.G.S. publication entitled "Hydrologic Data for Urban Studies in the Houston, Texas, Metropolitan Area."

Bintliff Ditch is located in southwest Houston and drains a watershed encompassing approximately 4.9 square miles between Bissonnet Street and the Southern Pacific Railroad at Westpark Drive. The U.S.G.S. topographic series map of the area, surveyed in 1970 and updated in 1982, was used as a base for delineating the watershed boundaries, as shown on Exhibit III-1. Several previously published reports were also used as guides in determining the boundaries, including the "Comprehensive Study of Drainage for Metropolitan Houston" and the U.S.G.S. "Hydrologic Data for Urban Studies in the Houston, Texas, Metropolitan Area."

The entire watershed area is naturally divided into two subwatersheds, with the 2,077-acre area to the north and east draining into Bintliff Ditch and the remaining 1,041-acre area to the west and south draining into Country Club Ditch. After delineating the watershed boundaries, the area was subdivided into 61 smaller drainage basins ranging in size from 5 acres to 157 acres as presented on Exhibit III-2. The location and size of each of these minor subbasins was determined by the variability of basin characteristics and the drainage pattern of the storm sewers and ditches in the basin. A total of 37 of the minor subbasins drain into Bintliff Ditch and the remaining 24 into Country Club Ditch. Each minor subbasin represents an area of the watershed which is drained by a street/storm sewer network with a single outfall into Bintliff Ditch or Country Club Ditch.

The Bintliff Ditch watershed was selected for analysis in this study for several reasons. First, its size was amenable to a detailed flooding evaluation of both its primary and secondary drainage systems with computer modeling. Second, it has been monitored for many years by a U.S.G.S. rainfall-streamflow gage and, therefore, has extensive historical data for aid in calibrating the model (Table III-1). Third, the watershed contains both earthen and concrete-lined channel sections which provide variations for data analysis.

TECHNICAL APPROACH

Overview

In order to accomplish the aforementioned objectives, a technical approach to the problem under consideration was developed from existing data. A study area of approximately five square miles was initially determined to be an adequate size basin which could be modeled for subsidence effects on the microscopic level. Detailed drainage system responses could be traced and evaluated on this level while widespread basin responses could be extrapolated macroscopically to be representative of typical small drainage basins in the Houston area. Critical concerns for the hydrologic and hydraulic evaluation of this size basin were identified with the most prominent critical factor being the ability to evaluate the response of secondary storm sewer systems during surcharged conditions.

The watershed response to subsidence was evaluated for two conditions. The first was the response of the existing drainage system when subjected to subsidence. The drainage system in this watershed has, for the most part, been in place for well over ten years and was designed and constructed prior to the adoption of the current City of Houston design criteria. The second evaluation was a representation of the drainage system for the watershed constructed according to current design criteria. The redesigned system was subjected to the same form of subsidence as the existing drainage system, therefore, affording a comparison of the effectiveness of the design criteria.

Selection of Computer Model

Several existing computer programs have the capability to model detailed storm sewer networks under surcharged and pressurized flow conditions. Reviews of urban stormwater models as presented by Dendrou (1982), Diniz and Suarez (1979), and Williams (1980) were read and considered in the model selection process. Table III-2 is a comparison of characteristics of stormwater runoff models as developed by other investigators. Only available storm sewer routing models are included. After comparing the capability of the various models available, the Environmental Protection Agency's (EPA) Storm Water Management Model (SWMM) was selected for application in this study because of its ability to address sheet flow and ponding as well as storm sewer surcharged flow.

This model performs overland flow computations and gutter, storm sewer, and channel routing based on rainfall hyetographs, antecedent conditions, land use, topography, channel and storm sewer parameters, and receiving water conditions. SWMM3 (the most recent version of the SWMM package) is also capable of considering backwater conditions, outfall and receiving water levels, and inlet ponding. This versatility and attention to detail made SWMM3 a primary choice for application to the analysis of localized subsidence effects on drainage.

Development of Input Parameters

In addition to a description of the size and shape of each subbasin, operation of the SWMM3 model requires data input for precipitation, infiltration and other water losses, ground slope, land use, and flood routing parameters.

Precipitation data for historic storms over the project area were derived from the U.S.G.S. "Hydrologic Data for Urban Studies." Rainfall amounts were computed for each 15 or 30 minute interval during the storm event intervals. Six historical storms were simulated during a calibration process while three design storms were simulated in order to evaluate the drainage systems' response.

Overland flow routing parameters were derived from the previous reports and maps supplemented by field surveys and design drawings obtained from the City of Houston. Land-use information was obtained from aerial photographs. Infiltration data was estimated using the Horton equation considering season, precipitation intensity, and antecedent conditions.

In the model representation of the drainage system, conduits and junctions are idealized as links and nodes, respectively. Links transmit flow from node to node and have properties of roughness, length, cross-sectional area, hydraulic radius, and surface width. Nodes correspond to manholes or pipe junctions in the physical system and have variables of volume, head, and surface area associated with them. Inflows, such as inlet hydrographs and outflows, such as weir diversions, take place at the nodes. The volume of the node at any time is equivalent to the water volume in the halfpipe lengths connected to any one node. The change in nodal volume during a given time step forms the basis for discharge calculations in the links. Model output is described in terms of hydraulic gradient flows and velocities in conduits, and maximum depths of water at conduit ends.

As noted previously, the watershed was evaluated in its existing condition and with a drainage system simulated using current drainage criteria. Both drainage systems are shown on Exhibit III-3. While the link-node representation is similar for both systems, differences in pipe sizes, node invert elevations, and ditch parameters made it necessary to establish a separate model for each system.

The existing drainage system in the Bintliff Ditch watershed has essentially been in place for well over ten years. Most of the residential subdivisions in the watershed were developed by the mid-1960's with Houston Baptist University, Sharpstown Country Club, Sharpstown Shopping Center, and other large land uses constructed by 1970. As a result, storm sewers, inlets, outfalls, and receiving ditches were all designed prior to the adoption of the current City of Houston design criteria and, therefore, perform with varying degrees of efficiency. Generally, inlets are fewer, storm sewers are smaller and steeper, outfalls are higher and ditches are shallower and narrower than would currently be designed for areas with similar development.

Exhibit III-4 shows the profile of Bintliff Ditch and two typical subbasin storm sewer systems under both the existing and "redesigned" conditions. Bintliff Ditch under existing conditions varies from an earthen channel at the upper end, to large underground box culverts at several locations in the mid-reach, to a uniform concrete-lined channel at the lower end. Bridges were placed at elevations convenient to feeder roads. As a result, most sewer outfalls function under surcharged conditions during most storms, and significant backwater occurs upstream of most bridge crossings.

For the "redesigned" condition, the channel was, for the most part, simulate as an earthen section with 3:1 side slopes irrespective of right-of-way requirements. Bridges were elevated so as to pass the anticipated 100-year flow without obstruction. Exhibit III-4 shows the profile of the redesigned Bintliff Ditch as a uniform earthen channel with bridges placed well above the projected water surface elevation and sewer outfalls conforming to current City of Houston design criteria. Storm sewers were redesigned to comply with current City of Houston criteria. Accordingly, storm sewers were designed to convey flows from the City of Houston storm sewer design curves. The Bintliff Ditch channel was redesigned to convey the 25-year design flow at an elevation at or below the tailwater elevation used to design the storm sewer system.

The simulation of the drainage system required the following assumptions in construction of the computer model:

1. Street system area in each subbasin is simulated by a single trapezoidal open channel with a length approximately equal to one-half the subbasin length in the plane of the major direction of flow and a width equal to the actual street surface area divided by the representative length.

2. Overland flow between minor subbasins and between the Country Club Ditch major basin and the Bintliff Ditch major basin is represented by trapezoidal open conduits leading from the street node with higher ground elevation to the street node with lower ground elevation. Interflow conduits of this type lead to and from nodes at either the upper or lower ends of the street conduit depending on the particular flow characteristics of the two subbasins involved. Interflow conduits were set at elevations three-tenths of a foot above the invert elevations of the subbasin street conduit at each node to simulate crests which must be topped before interflow can occur in the street and overland flow systems of the two basins.

3. All runoff was assumed to be contained within the Bintliff Ditch Watershed proper so that the Bintliff Ditch-Country Club Ditch drainage area could be represented as a self-contained system for modeling purposes. No interbasin transfer of runoff was simulated into or out of the watershed. Flow into the watershed from the west was assumed to be equivalent to flow exiting the watershed to the east.

4. Storm sewers are modeled as representative circular conduits leading from the representative trapezoidal open street conduit to the outfall at the receiving channel. The largest diameter storm sewer in each subbasin drainage system as shown on Exhibit III-3 was used as the simulated storm sewer conduit.

5. The impacts of regional subsidence on the receiving stream of Bintliff Ditch were not considered in this analysis. Bintliff Ditch was assumed to outfall at its normal depth condition for all cases in the localized drainage analysis.

A secondary objective of the Localized Drainage Analysis phase of the project was to compare a recently designed drainage basin to one that has been constructed for at least ten years. As described previously, this approach would allow for evaluation of the impact of subsidence on drainage systems constructed using different design criteria. To this end, the existing Bintliff Ditch Basin as described above represents an "old" drainage basin and the system defined using current design criteria represents a "new" drainage basin.

By establishing parallel computer models of the "old" and "new" drainage systems as described, the objective of evaluating the effectiveness of current drainage criteria was accomplished. The objective of defining the impacts of water wells was accomplished by varying the location and degree of simulated subsidence situations.

Calibration of Model

Three steps were completed in the process of calibrating the RUNOFF and EXTRAN data sets of SWMM3 before actual modeling of the subsidence cases could be performed. First the assumptions made in discretizing the watershed and in representing the subbasin and routing characteristics with link-node input parameters were verified. Second, a sensitivity analysis was completed on the input parameters described above so that the overall computer model for the existing drainage system produced results comparable to historically measured data for the watershed. Third, both the "new" and "old" drainage systems were evaluated with standard design storms to verify the consistency of the model between the two systems. The following paragraphs describe the application of each of these calibration processes to the study project.

In order to apply SWMM3 to the Bintliff Ditch drainage system, several assumptions were required as described previously. These assumptions must necessarily be verified before proceeding to calibrate the model by numerically altering the input parameters. Application of the model must therefore be made to a smaller, well-documented drainage system where each detail of the simulated network can be evaluated independently.

The Lazybrook storm sewer watershed is a 0.13-square-mile watershed located in northwest Houston in the White Oak Bayou basin. The system has been monitored by a U.S.G.S. flood-hydrograph gage and rainfall recorder since 1978 and, therefore, is represented by excellent historical flow records for calibration purposes for at least seven years. The watershed was simulated by 13 sub-areas each encompassing a cul-de-sac or otherwise isolated street segment and also as a single aggregated subbasin.

The two models of the Lazybrook system were then subjected to several storm events. The outfall hydrographs for each event were compared in terms of timing, peakflow, and runoff volume. The largest variation between the two models for any event was less than 10 percent for any of the 3 hydrograph parameters examined. As a result, it was concluded that the aggregated system and related assumptions used in the Bintliff Ditch model would provide adequate simulation of the complex drainage system.

Six storms were selected from the U.S.G.S. "Hydrologic Data for Urban Studies" for use in calibrating the model. The intensities of each storm are shown in Table III-1 and represent a large range of durations and runoff volumes for analysis of parameter sensitivity to different storm characteristics. Calibration was achieved by adjusting the model input parameters relating to water losses (i.e., infiltration, evaporation, and depression storage), and fine-tuning theoretical representation of the physical drainage systems. Values for these parameters were selected to provide the best overall comparison to the six storm events selected. Table III-3 summarizes the adopted values for the hydrologic model input parameters.

Upon calibration of both the Country Club Ditch subwatershed and the Bintliff Ditch subwatershed individually and in combination, the Bintliff Ditch subwatershed was selected for modeling analyses for the remainder of the project. This simplification of the model retained interbasin flows from the Country Club Ditch subwatershed but served to greatly reduce computer simulation times for the remainder of the project. Exhibit III-5 shows the observed hydrograph for Bintliff Ditch vs. the SWMM3 predicted runoff hydrograph for two of the six storm calibration events using the parameters defined in Table III-3. The two storm hydrographs shown in Exhibit III-5 are for the two most recent events and, therefore, match the existing condition parameters most closely.

With the completion of the calibration phase of the SWMM3 application to Bintliff Ditch, the model was tested for its response to three design storms: the 3-year rainfall, the 10-year rainfall, and the 100-year rainfall. To verify the model, the SWMM3 model response to the design rainfall associated with the City of Houston storm sewer design criteria was determined and the peak flow rates compared to the flow rates used for the system design, as described below. The 10-year and 100-year design rainfalls were then applied to both the existing and redesigned drainage system models to establish the base response for these frequencies. The model response to the design rainfall associated with the City of Houston storm sewer design criteria was determined by making the assumption that this design rainfall is equivalent to a storm with a 3-year frequency of return. Rainfall distributions from the Weather Bureau Technical Paper No. 40

were used for the 3-year, 10-year, and 100-year frequency storm events for application to the model. Parameters relating to water losses were taken from the calibrated model.

The resultant storm flows for both the existing and redesigned drainage system models are summarized in Table III-4. The storm flows are compared at the Bissonnet gage. For each of the three frequencies analyzed, the existing system model flows are significantly lower than that of the redesigned system. Additionally, each of the modeled flows closely parallel the channel capacity. In the case of the existing system, the model predicts a 100-year discharge of 1,470 cfs while the actual channel capacity is 1,300 cfs. Likewise in the redesigned system model, the 100-year discharge was predicted to be 3,560 cfs whereas the channel capacity, if designed according to HCFCD criteria (using the HCFCD curve developed for Brays Bayou), would have a capacity of 3,200 cfs.

The small variation in "new" system flows between the higher frequency events is attributable to street ponding in the basin. The same explanation holds for the only slight variation in flows for all three events on the "old" existing system. This small variation in peak flows for the existing drainage system is consistent with historical records from the Bissonnet gage monitored by the U.S.G.S. The six maximum flows for the Bintliff Ditch watershed recorded by the U.S.G.S. between 1968 and 1979 vary by only twenty percent from a mean value of 1,100 cfs.

Both the 10-year and 100-year frequency events cause considerable ponding in the streets of the existing and redesigned drainage system, and even the 3-year event causes significant ponding in the streets of the existing system. This ponding occurs because the storm sewer system of the "new" system was designed for the City of Houston design storm, which is approximately equivalent to a 3-year storm event, while the "old" system was constructed before adoption of the current City of Houston design criteria and does not meet the 3-year standard. The adequacy of the current criteria is shown by the decreased ponding evidenced in the application of the 3-year storm to the "new" drainage system.

The SWMM3 analysis indicated that the storm sewer system will convey a flow much greater than the expected design flow during the 3-year event (see Table III-5). This increased conveyance is possibly due to two reasons. First, the 3-year storm developed from TP-40 is somewhat higher than the storm event used to generate the City of Houston design curves. Second, conditions experienced in the sewer system under this 3-year storm event probably differ from those predicted under the design assumptions. For example, storm sewers are designed with the hydraulic grade line below the gutter line. As soon as ponding occurs at the inlet, pressurized conditions are experienced in the system and gradients in the initial storm sewers are greater than the design assumptions. In addition, the 25-year outfall channel elevation assumed to occur under design assumptions does not occur at all during the 3-year event or occurs after the time of maximum inflow to the inlets. Generally, for the "new" system, the storm sewer discharges for the 3-year event are considerably higher than that of the typical City of Houston design value of 1.5 cfs per acre due to ponding and increased head occurring at the inlets. For the TP-40 3-year storm, the existing "old" system discharges averaged 1.74 cfs per acre while the redesigned "new" system discharges averaged 3.27 cfs per acre.

ANALYSIS OF LOCAL EFFECTS OF SUBSIDENCE

Overview

A primary objective of the study was to evaluate the effects that subsidence may have on the ability of secondary drainage systems to convey runoff and control street ponding. Six discrete subbasins were selected from the thirty-one subbasins draining directly into Bintliff Ditch for evaluation of how subsidence affects a typical storm sewer system.

The six subbasins selected for detailed analysis were labeled and distinguished from other subbasins by adding a "9" prefix to the existing number, i.e., 906, 907, 910, 918, 924, and 929. The six subbasins are shown on the map in Exhibit III-2. The characteristics of each of the individual drainage areas are considered to be typical of the variety of discrete runoff patterns and collection systems in the Houston metropolitan area. Subbasin 906, for example, is representative of a single-family/multifamily mixed residential area on a major thoroughfare, while subbasin 924 is representative of a single-family residential block encompassed by a residential neighborhood. Table III-6 presents the drainage characteristics particular to each of the six subbasins. Each of these discrete subbasins was analyzed in detail for the subsidence cases described below.

Selection of Subsidence Cases

The localized effects of subsidence on the Bintliff Ditch watershed were evaluated for two general types of subsidence. The first involves a subsidence cone generated from a specific water well field due to hypothetical draw-down of the potentiometric surface in the local area. The second involves a general subsidence gradient resulting from groundwater withdrawal across the Houston metropolitan area over a period of many years. Both the well field subsidence cone and the regional subsidence gradient changes in elevation were provided by the Harris-Galveston Coastal Subsidence District.

The well field subsidence cone is a "worst-case" scenario generated by the Subsidence District specifically for use in this study. The simulated subsidence cone is the result of modeling three water wells located in close proximity and withdrawing water in unison at a high rate. This case causes highly localized subsidence and is predicted to be highly unlikely to occur in reality. The regional subsidence gradient was developed from the simulation shown in Exhibit II-2 and discussed in the Riverine Flooding Analysis. A maximum change in elevation of 1.6 feet across the local watershed was implemented in this phase of the study. In each case, the location and direction of the subsidence gradient relative to the watershed was varied in an attempt to identify the most critical cases.

For the purposes of this analysis, all modeling performed on the local well field subsidence cone was considered Case 1, while all modeling concerning the regional subsidence gradient was referred to as Case 2. Each subsidence case was modeled in four different ways (differentiated as a, b, c, and d) as described below.

Localized Well Field Subsidence Cone

Exhibit III-6 presents the localized well field subsidence cone (Case 1) as it was modeled on the Bintliff Ditch drainage area by SWMM3. The four placements of the subsidence cone were selected to reflect the drainage system response to maximum subsidence at critical points in the watershed. Case 1a represents placement of the well field near the upper end of Bintliff Ditch. At this location, the subsidence contours have the general effect of decreasing the overall slope of the main channel while the major storm sewer system slopes are increased in the upper end of the watershed and remain nearly constant in the mid to lower portions of the watershed.

Case 1b represents placement of the well field near the center of Bintliff Ditch. The overall effects of the subsidence contours at this location are to steepen the upstream slope and flatten the downstream slope of Bintliff Ditch. The slopes of the storm sewers in the upper and lower portions of the watershed remain relatively unchanged, while those near the center of the watershed are increased.

Case 1c represents placement of the well field away from the main channel and near the highest topographical point in the watershed. At this location, the subsidence contours have the least effect on the overall slope of channel and have only a moderate flattening effect on the storm sewer gradients in the immediate vicinity of the cone epicenter.

Case 1d represents the placement of the well field at the lower end of Bintliff Ditch near the mouth of the watershed. The subsidence contours have the overall effect at this location of increasing the slope of Bintliff Ditch and of the storm sewers in close proximity to the epicenter. The remaining storm sewer systems reflect little or no effect from the subsidence cone.

Regional Subsidence Gradient

The regional subsidence gradient options are shown on Exhibit III-7. Case 2a represents an increasing subsidence gradient from north to south across the watershed corresponding to the major direction of channel flow in the drainage area. The general effect of the subsidence gradient in this direction is to steepen the slope of Bintliff Ditch while having no effect on the majority of storm sewer systems. In the upper northwest section of the watershed the main channel is unaffected by the subsidence gradients while the slopes of storm sewers south of the channel are flattened and the slopes of storm sewers north of the channel are steepened. Similar but opposite effects are caused by the placement of the subsidence gradients increasing from south to north as represented by Case 2b. The slope of the main channel is flattened on all but the northwest section, which reflects no change, and the slopes of the majority of sewer systems remain unchanged, except for those in the northwest portion of the basin. The latter reflect opposite changes to those described for Case 2a. Specifically, those storm sewer systems south of Bintliff Ditch steepen in gradient while those systems north of the ditch flatten in gradient.

Cases 2c and 2d represent the placement of the subsidence gradients in the east-west direction perpendicular to Cases 2a and 2b. As shown on Exhibit III-7.

Case 2c simulates the regional subsidence gradient as increasing from west to east across the watershed. This case affects Bintliff Ditch by increasing the slope on the upper two-thirds of the channel while having little or no impact on the lower third of the channel. In addition, the slopes of the storm sewers in the northwest third of the watershed are relatively unaffected by this placement of the gradients, while those in the rest of the watershed are flattened if they are positioned east of Bintliff Ditch and steepened if they are west of the ditch.

Case 2d represents the placement of the subsidence gradients opposite to those represented by Case 2c. The subsidence gradient increases from east to west and flattens the upper two-thirds of Bintliff Ditch while having little or no effect on the lower third. As with Case 2c, the slopes of the storm sewers in the northwest portion of the watershed are relatively unaffected by this subsidence case; however, the remaining storm sewer system slopes are steepened for those systems east of the ditch and flattened for those systems to the west.

The SWMM3 computer model was applied to each of the eight subsidence cases described above as well as to the base condition representing no subsidence. The nine cases were modeled for the 3-year, 10-year, and 100-year frequency storm events for each of the represented drainage systems, i.e., existing ("old") and redesigned ("new"). A total of fifty-four computer simulations were, therefore, performed and evaluated during the study.

Each subsidence case was integrated into the respective drainage system designs by subtracting the appropriate change in elevation noted on Exhibits III-6 and III-7 from the elevation data in the base condition data file. In this way, both the ground surface elevations and the invert elevations for each pipe conduit and street in the drainage system were altered to reflect the eight subsidence cases.

Analysis of Localized Subsidence

The results of the computer simulations of the nine subsidence cases for the 3-year, 10-year, and 100-year storm events were evaluated in a number of different ways in order to meet the four objectives presented at the beginning of this chapter. Two levels of analysis were developed, with specific evaluation criteria determined for each. The goal of these evaluation criteria was to assist in the ranking of the different well field locations with respect to pertinent flooding effects such as residential and commercial damages and restrictive vehicular access. In addition, similar rankings were desired for the various regional subsidence patterns to guide in placement of wells across the Houston metropolitan area.

The two levels of analysis designed to organize the modeling results were: (1) evaluate the effects of subsidence on street ponding, structure flooding, and vehicular access patterns internally in the subbasins; and (2) evaluate the effects of subsidence on peak flows, timing of peaks, and depth of water in the major drainage channel (Bintliff Ditch). For each level of analysis, comparisons were made between the existing drainage system (old) and the redesigned (new) drainage system, as well as for differences between the three frequencies of design storms. The modeling results are presented below in relationship to the selected evaluation criteria.

Subsidence Pattern vs. Depth of Ponding at Street Inlets and Change in Depth of Ponding

The base condition (no subsidence) and eight subsidence cases were modeled on each of the existing and redesigned drainage systems for the 3-year, 10-year, and 100-year design storms and were evaluated with respect to depth of ponding in the street system. This analysis was performed by determining the maximum volume of water stored in the representative street conduit for each of the six subbasins studied in detail. This volume was calculated from the maximum depths of water simulated by SWMM3 at each end of the street conduit. The volume was then compared to a stage-storage curve developed from the actual topography for each subbasin, and a maximum depth of ponding at the lowest inlet was determined.

Exhibit III-8 shows a typical subbasin in the Bintliff Ditch watershed. This subbasin happens to be a commercial/industrial area on a cul-de-sac with the street system designed to convey most of the storm water runoff. The existing drainage system 3-year and 100-year flood plain limits are shown in blue dashed and solid lines, respectively. The 3-year and 100-year flood plain limits corresponding to the redesigned drainage system are outlined respectively in red dashed and solid lines. The differences between flood plains for the existing and redesigned drainage systems for the same frequency of storm are typical for subbasins throughout the watershed. Table III-7 presents the maximum ponding depths for the base condition (no subsidence) in each of the six detailed subbasins used in the localized analysis.

Less significant differences in depth are apparent between subsidence cases for any particular subbasin. Table III-8 lists the change in depth due to subsidence from the base condition for each subbasin for each storm event. These relative changes in depth due to varying subsidence cases seem also to be typical across the entire watershed.

Subsidence Pattern vs. Timing of Inlet Ponding

The fifty-four computer simulations were also evaluated with respect to the timing and duration of flooding in the street system. The timing and duration of street flooding is important in terms of limited vehicular access and disruption of traffic patterns.

In order to analyze the effects of subsidence on these flooding aspects, a specific limiting depth had to be determined, and the timing before flooding reached that depth and the duration the flood waters were at or above that depth had to be discerned from the computer modeling results. A critical depth of six inches at the inlet was chosen as a depth which would initiate traffic disruptions and restrict or prohibit vehicular access to the street system in the subbasin.

Table III-9 lists the initial time at which inlet ponding was equal to six inches for the base condition at each subbasin. This time is presented in clock time from the beginning of the rainfall. The increase in timing for the 10-year and 100-year frequency values over the 3-year frequency values occurs due to the duration of the storm events. The 3-year storm was simulated over three hours while the 10- and 100-year storms were simulated over six hours. As indicated by the results summarized in Table III-9 a significant difference between the existing drainage system and redesigned drainage system initial times can be noted for several of the subbasins. However, no change in this initial time was evident between the base condition and the various subsidence scenarios for any of the subbasins.

Table III-10 lists the duration of ponding at a depth greater than or equal to six inches for each of the subbasins under base conditions. The duration is presented in clock time beginning when the ponding depth initially equaled six inches. Again, there is a significant difference in duration of flooding between the existing drainage system and the redesigned drainage system for many of the subbasins. As with the initial timing to inlet ponding results discussed above, no apparent changes from base conditions for any of the subsidence cases were noted for the duration of ponding results.

Evaluation of Street Ponding Impacts

Maximum ponding depths at subbasin inlets logically increase with less frequent storm events for both the existing and redesigned drainage systems. From the data presented in Table III-7, a significant reduction in street ponding depths is evidenced by upgrading the storm sewer system and main drainage channel to current design criteria. Even for subbasins with adequately designed sewer systems under existing conditions, such as subbasins 907 and 929, improved outfall and ditch designs reduced ponding depths. The beneficial impact of the redesigned drainage system is reduced as storm frequencies decrease, supporting the contention that once the secondary drainage system is inundated by large, intense rainfalls, interflow between subbasins becomes more widespread and ponding occurs generally throughout the watershed.

While most of the changes in ponding depths presented in Table III-8 are statistically insignificant, several changes in ponding depths varied from the base condition by one-tenth of a foot or more and may show general data trends for specific subsidence cases. One such trend is presented by the results produced evaluating Case 1c on subbasin 924 under the existing drainage system. The maximum ponding depth at the subbasin inlet increased by 6 percent for the 3-year storm, by 11 percent for the 10-year storm, and by 16 percent for the 100-year storm. These latter increases correspond closely with the increases in depths associated from the 3-year to the 10-year and from the 10-year to the 100-year events for the base condition. The consistent increase in depth associated with Case 1c indicates increased ponding due to flattening of the subbasin secondary drainage system by the well field subsidence case; however, a much larger increase in ponding is caused by the increasing runoff from larger storm events. Increases in ponding depths of over a foot were evident between the 3-year and 100-year frequency events (Table III-7), while the maximum change in ponding due to a subsidence case was less than half a foot (Table III-8). The evidence of a surcharged sewer system and increased street ponding with larger storm events support the general conclusion that the effects of subsidence on depth of flooding and time of street ponding are overwhelmed by changes in ponding caused by inadequate drainage system flow capacities. The increased capacity of the redesigned drainage system generally produces lower ponding depths than those resulting under the existing system.

The perturbations and apparent anomalies noted in Table III-8 and in later data tables may be attributed to fluctuations in the model simulations caused by pressurized flow conditions in the secondary drainage system. As yet, no computer model is readily available which models pressurized sewer system flows accurately on a consistent basis; therefore, when drawing conclusions from data produced by SWMM3 or any other model with limited capabilities, care must be taken to evolve general trends and patterns from the data and not rely on specific numerical data points. For this reason, the data presented in Table III-8 generally seem to indicate that little, if any, significant changes in maximum inlet ponding depths from subsidence can be substantiated. General ponding depth increases of less than a tenth of a foot cannot be adequately represented on monumentation maps and are considered insignificant in terms of increased structure flooding.

Data evaluated with respect to timing and duration of street ponding also supports this conclusion. Not only were no effects of subsidence on initial timing of inlet ponding noted from the data, but the results presented in Table III-9 show only a slight reduction of the initial timing due to the increased capacity of the drainage system on several of the basins. When ponding did occur, the initial time period required to reach six inches seemed fairly constant irrespective of drainage system design. However, the duration of ponding at a depth equal to or greater than six inches did vary with drainage system design. Table III-10 clearly shows that the drainage system designed under current design criteria significantly reduces the period of critical street ponding. No changes in the effects on duration of ponding were noted for the subsidence cases, however.

Subsidence Pattern vs. Change in Water Depth in Bintliff Ditch

The second level of analysis of the modeling results involved evaluating the effects of the subsidence cases on the hydraulic and hydrologic properties of Bintliff Ditch. Initially, this was performed by noting the maximum depth of water at specific points along the channel for both the existing and redesigned systems during the 3-, 10-, and 100-year storm events.

Exhibits III-9, III-10, III-11, and III-12 summarize the change in water depth along the length of the channel with respect to the zero-line base condition for the extremes of the 3-year and 100-year storm events. Subsidence Cases 1a and 1b are shown on Exhibit III-9, Cases 1c and 1d are on Exhibit III-10, Cases 2a and 2b are on Exhibit III-11, and Cases 2c and 2d are on Exhibit III-12. The change in the channel flow line due to the respective subsidence case is also referenced below each change in depth graph. Some anomalies and inconsistent points can be seen on the graphs; however, the points selected were chosen to minimize error introduced by channel constrictions (bridges and culverts) and are representative of general data trends which are discussed in detail later in this chapter.

Subsidence Pattern vs. Percent Change in Peak Flow in Bintliff Ditch

The second level of the analysis on the effects on the subsidence cases on Bintliff Ditch was concerned with evaluating the changes in flow characteristics along the ditch during the subsidence simulations. While no consistent data emerged relating the timing of peak flows with particular subsidence cases, significant changes in the magnitude of peak flows along the channel did result from the cases modeled.

Exhibits III-13 through III-16 display the changes in channel flow resulting from the different subsidence cases. The subsidence cases were evaluated for the 3-, 10-, and 100-year frequency storm events on both the existing and redesigned drainage system. Cases 1a and 1b are shown on Exhibit III-13 for the 3-year and 100-year events, Cases 1c and 1d are on Exhibit III-14, Cases 2a and 2b are on Exhibit III-15, and Cases 2c and 2d are on Exhibit III-15. The changes in channel flow were combined with the changes in depth described previously to determine the overall effects of the localized well field cones and regional subsidence gradients on the hydraulics of the main drainage channel for typical Houston watersheds.

Evaluation of Channel Impacts

Results corresponding to the channel evaluation criteria were presented for the two event extremes corresponding to the 3-year and 100-year recurrence frequencies. The 10-year event was difficult to evaluate graphically due to the response of the drainage system to this size storm event. This frequency event surcharges the storm sewer network but is adequately conveyed by the channel, causing flow conditions in the channel to oscillate between reactions consistent with the lower frequency 3-year storm and higher frequency 100-year storm. In order to be adequately explained, these oscillations required a level of detailed analysis inconsistent with the time frame or scope of this report and served only to confuse the data presentations contained herein. Therefore, the 3-year and 100-year event results were analyzed and presented for flow conditions in Bintliff Ditch and support the general data trends discussed below.

The effects of both local well field and regional subsidence on the flow characteristics in the main drainage artery, Bintliff Ditch, are presented as Exhibits III-9 through III-16. Several general comments can be made about the data results prior to the discussion of specific conclusions for each subsidence case. First, data spikes on the exhibit graphs near the upper end of Bintliff Ditch at Station 210+00 reflect the instability of the water surface in the existing drainage system at the upstream entrance to the Harwin Drive culvert. The contraction of flow in the ditch due to the long underground culvert causes significant backwater effects and flow oscillations at the upstream point of constriction. These unstable data points on the graphs presenting the modeling results were, therefore, considered independently from the general data trends discussed below. A second general comment derived from the data results is that for the redesigned drainage system, subsidence cases generally have a smaller impact on the channel due to the increased overall conveyance capacity of the channel drainage system. The improved design of the drainage system due to the current design standards is, therefore, not as significantly affected by an increase or decrease in slope caused by subsidence.

The eight subsidence cases each impact the water depth in Bintliff Ditch in two ways - changes in the gradient of the drainage system or changes in discharge caused by changes in the slope of the basin. In many cases, both of these impacts occur simultaneously and may be difficult to separate or identify in the graphical representation of the subsidence simulation results. Conclusions relating these impacts to each subsidence case are discussed below.

Case 1a, which reflects the placement of the local well field subsidence cone near the upper end of the drainage area and a decrease in channel slope, causes the largest overall increase in flooding depth along the channel. The maximum increase in depth of nearly 1 foot occurs for the existing drainage system at several points in the upstream portion of the channel. Increased flows from the storm sewer networks in the vicinity of the cone combined with the flattening of the upper end of the main channel and related decreased channel conveyance produce the increased water depth. The redesigned drainage system follows similar increased depth conditions of nearly 0.5 foot near the cone epicenter but shows almost no change in channel flows. Therefore, less effect from either increased slopes on storm sewers or decreased slopes along the channel are evidenced on the "new" drainage system. However, the increased storage near the epicenter results in a slight decrease in flow and depth at the downstream end of the ditch for the existing system.

A major oscillation in the graphs for the existing system occurs at about station 120+00, as shown by existing drainage system data spikes at this station for depths and flows for both Cases 1a and 1b. The relatively unchanged slopes downstream of this point combine with the upstream subsidence impacts and cause the water conditions to surge at this station. The channel conditions at the location of the spike correspond to an existing channel expansion directly downstream of the 5,000-foot box culvert which passes underground from north of the Southwest Freeway to south of Bellaire Boulevard.

Case 1b, which reflects the placement of the local well field subsidence cone near the center of the drainage area, causes little change in either depth or flow at either end of the channel, but causes a significant increase in depth adjacent to the cone epicenter of nearly 0.5 foot for the redesigned system and over 1 foot for the existing system. Flows are increased between 10 and 15 percent near the epicenter for the existing drainage system, but show negligible increases under the redesigned drainage system. Generally, the effects of the subsidence cone on channel depths and flows for Cases 1b, 1c and 1d are highly localized and are diminished at the outer reaches of the cone influence.

Case 1c, which reflects placement of the local well field cone near the highest topographic elevation to the west away from the ditch, shows an overall decrease in depth averaging about 0.5 foot and a decrease in flow of less than 20 percent in the channel due to increased storage in the system due to the decrease in channel slope and decreased flows from the large sewer systems in the vicinity of the cone. Case 1d, which reflects placement of the local well field cone near the downstream end of the ditch, produces results of the same direction and magnitude as Case 1c. In this case, overall channel depths and flows are decreased due to increased channel slope. The existing drainage system produces an increase in depth of less than 0.5 foot for the 3-year event and an increase in flow of almost 20 percent for the 100-year event in the immediate vicinity of the epicenter. However, very little overall change in depth or flow is produced by the redesigned drainage system for either the 3-year or 100-year event for both Cases 1c and 1d.

The four regional subsidence cases produce results consistent with those for the local well field subsidence cone, but are more generalized and widespread across the watershed. Case 2a, which reflects a regional subsidence gradient increasing from north to south across the watershed, produces an overall increase in channel slope with a maximum of 1.6 feet and a resulting decrease in water depth of up to 1 foot along the majority of the ditch. A consistent increase in channel flow of around 10 percent is reflected by both the 3-year and 100-year storm events on the redesigned drainage system. A general decrease in flow (after removal of spikes at Station 210+00 as described previously) in the upper end of the watershed for the existing drainage system indicates a significant reduction in the existing storm sewer system flow capacities for those sewers located to the south of the upper reach of Bintliff Ditch. This reduction in total flow reaching the ditch is overshadowed by the increase in flows resulting from the increased slope in the lower portions of the ditch.

Conversely, Case 2b, which reflects the opposite regional gradient to that of 2a, produces an overall decrease in channel slope resulting in an increase in water depth along the majority of the channel with a maximum increase of approximately 1.7 feet for the existing drainage system 3-year event. Flows are generally decreased due to the decreased channel conveyance capabilities for all but the 3-year, redesigned drainage system case. The flows for this case are higher by as much as 30 percent in the upper end of the channel probably due to the increased flows from the steepened storm sewers to the south of the upstream portion of Bintliff Ditch. These storm sewers can adequately convey the 3-year event under redesigned conditions and are increased in slope (and, therefore, conveyance) during this subsidence case. Both frequency storm events on the existing system and the 100-year event on the redesigned system, however, cause significant surcharging and ponding of the secondary drainage system throughout the watershed under base conditions, and flows cannot significantly increase in the surcharged pipes even with the slight steepening in slope resulting from Case 2b.

Case 2c, which reflects the regional subsidence gradient increasing from west to east, produces a slight increase in slope along the main channel and a greater steepening of storm sewer slopes located to the west of the ditch. This case causes a slight increase in depth of up to 0.4 foot for the 3-year events on both the existing and redesigned drainage systems since more runoff from the large drainage areas to the west of Bintliff Ditch can reach the channel due to increased conveyance of the storm sewer pipes in these areas. This is not true for the 100-year event on both systems, however, since large-scale surcharging and ponding occurs for this event under base conditions and is not significantly improved by storm sewer gradient changes on the magnitude of the regional subsidence gradient considered in this study. More effect on the 100-year event is caused by the slight steepening of the slope of the channel, which decreases the depth of water in the channel over 1 foot in some places. An increase in channel flows averaging around 10 percent is evidenced by the redesigned system due to the steeper slope, but the inadequate capacity of the existing channel reflects little or no improvement to the conveyance capabilities. In fact, flows in the existing channel decrease by more than 10 percent in the upper half of the basin, possibly due to overland flow increasing from northwest to southeast during this subsidence case. Normal interbasin flows in this direction due to natural topography are increased due to the subsidence gradient. A portion of the runoff which is ponded in the streets can then enter the lower subbasins instead of remaining in the upper subbasins as in the base condition.

Case 2d, which reflects an east-to-west subsidence gradient opposite to that in Case 2c, produces the least impact due to subsidence than any of the other regional cases. The slight flattening of the upper end of the channel and of the storm sewer gradients to the west of the main portion of the channel causes an overall increase in water depth of less than 0.5 foot in Bintliff Ditch. Slight changes in flows of less than 10 percent for both the "old" and "new" systems from those resulting from the base condition are also caused by this case and reflect the combined effects of changes in channel slope, storm sewer slopes, and overland flow slope. Less effects are evidenced by this case than by Case 2c since the gradient change is opposite to the natural ground slope and some impact from the subsidence is therefore negated.

CONCLUSIONS AND RECOMMENDATIONS

As discussed in the body of this section, it should be noted that the well field subsidence cone simulated in this study was a "worst case" scenario highly unlikely to occur in the Houston area. Placement of such a well field on the banks of a main drainage channel (as in Cases 1a, 1b, and 1d) would be very unusual. The flooding effects resulting from the well field subsidence cone are highly localized and occur mainly in the immediate region of influence of the cone.

The significance of the results discussed above can be summarized in general terms. Placement of the local well field adjacent to the channel at either the upper or mid-portion of the drainage basin (Case 1a and 1b) will result in subsidence patterns which will cause an overall increase in channel water depths and an overall decrease in peak channel flows. Placement of the well field adjacent to the downstream end of the channel (Case 1d) will result in a subsidence pattern which will cause an overall decrease in water depths and peak flows in the channel. Placement of the local well field away from the main channel at a higher elevation than the majority of the drainage basin (Case 1c) will produce a subsidence pattern which will decrease both water depths and flows in the channel. Very little significant changes in depths or peak flows in the redesigned drainage system were caused by Case 1c. In fact, the placement of the local well field in each case as described above had almost no effect on peak flows in the redesigned system. Generally, therefore, the effect of the local subsidence cone on channel depths and flows for both drainage systems is highly localized and is diminished at the outer reaches of the cone influences. In addition, changes in slopes caused by the subsidence cone have less impact on depths and flows in the redesigned Bintliff Ditch channel than in the existing channel.

For the subsidence cases derived from the placement of the regional subsidence gradient, maximum increases in water depths in the channel occurred when the channel slope was significantly flattened, as in Case 2b. Maximum decreases in channel depths resulted from major steepening of the channel slope, as in Case 2a. Slight steepening of the channel slope and a larger slope increase for major storm sewer systems in the watershed (Case 2c) caused an increase in water depth for the 3-year storm event and a decrease in water depth for the 100-year event in this case. Decreasing the overland slope, main channel slope, and majority of storm sewer slopes (Cases 2d) causes on overall increase in channel depths. Peak flows in the channel were decreased in the upstream end and increased in the downstream end for the existing drainage system under Cases 2a, 2c, and 2d. Peak flows were increased overall in the redesigned drainage system for Cases 2a and 2c and remained relatively unchanged for Case 2d. For Case 2b, existing drainage system flows were generally decreased along the channel, while the 3-year flows were increased and the 100-year flows were decreased in the redesigned drainage system.

The localized drainage analysis phase of the study of the effects of subsidence on flooding in the Houston metropolitan area produced results which lead to the following conclusions concerning the localized impacts of subsidence. General guidelines for locating proposed well fields are derived from these conclusions and can be used in preliminary site analyses for proposed well fields. Each individual site should be evaluated with respect to specific local drainage patterns before final site selection can occur. In general, however, the results presented herein support the following conclusions:

- Adequate drainage system design has a much greater impact on localized flooding than does the placement of local well fields or regional subsidence gradients.

- Use of current design criteria results in significantly reduced ponding levels and duration when compared to ponding resulting from previous criteria.

- Results indicate that while placement of a well field away from the main drainage channel at an elevation higher than that of the majority of the watershed such as near the drainage divide (Case 1c) may increase ponding depths a small amount within the immediate vicinity of the cone epicenter, water depths and flows in the channel are slightly decreased and an overall positive effect on flooding in the watershed may be realized.

- An orientation of the regional gradient with increasing subsidence from upstream to downstream along the primary drainage system (Case 2a) may cause a small increase in peak flows in the main channel but may actually decrease the water depth along the majority of the channel length by increasing the slope of the channel. The placement of a regional subsidence gradient in the opposite direction (Case 2b) causes the largest increases in the channel water depths and decreased flow.

In summary, the effects of localized or regional subsidence on storm sewer systems and street ponding were found to be negligible in this analysis.

However, well placement and the localized subsidence which results can affect flooding along the primary outfall drainage channel of a small drainage area depending on location of the well field. It is recommended that all wells be located at or near the drainage divide of small watersheds. Such placement will result in minimal impact on watershed flooding. The location of wells adjacent to the primary outfall channel or sewer should be avoided or, if unavoidable, should be combined with system modifications to provide an increase in primary outfall system capacity to offset predictable reductions in channel or sewer gradient. Future predicted regional subsidence patterns which are oriented opposite to the drainage channel flows in small watersheds should be planned for in advance by compensating for potential increased flooding with additional channel freeboard or other design modifications.

CHAPTER IV

RESERVOIR CAPACITY ANALYSIS

CHAPTER IV

RESERVOIR CAPACITY ANALYSIS

INTRODUCTION

Overview

Addicks and Barker Dams were designed and constructed for the specific purpose of protecting the highly populated reaches of Buffalo Bayou within the City of Houston from flooding during intense rainstorms. Since their construction by the U.S. Army Corps of Engineers in 1948 and 1945, respectively, there has been a relatively constant monitoring of the dams' performance and operating procedures. More recent investigations by the Corps of Engineers addressed the concern that the changing urban character of the upstream watersheds may affect the safety and functional reliability of the reservoirs in protecting the developed areas downstream of the dams. This concern, along with more severe storm and freeboard criteria, has led to several proposed modifications to the dams.

As stated in the previous chapters of the report, another factor associated with urbanization is an increase in the demand for water, usually obtained from groundwater, and subsequently a potential for additional subsidence. The Reservoir Capacity Analysis portion of this study addresses how subsidence can potentially affect the ability of the reservoir system to store floodwaters.

Primary Objective

The primary objectives of the capacity analysis on the Addicks and Barker Reservoir System were to:

(1) Identify the effects that possible subsidence would have on maximum flood control storage capacities;

(2) Determine the changes in 100-year pool levels and the extent of land affected resulting from a variety of subsidence cases; and

(3) Recommend possible solutions or future studies to be made if major impacts are identified.

Technical Approach

The analysis performed on the Addicks and Barker Reservoir System adhered to the following methodology.

- Determination of storage-elevation data was accomplished using existing topographic maps obtained from the Corps of Engineers. The resulting storage-elevation relationships were compared to published capacity tables and any discrepancies resolved.

- Projection of revised capacity curves were then established by adjustment of the base level station-elevation data to reflect the gradient changes provided by the Harris-Galveston Coastal Subsidence District for possible subsidence cases. An evaluation was made of the impacts on the maximum flood control storage capacity and the extent of land flooded. This evaluation was limited to changes resulting from gradient change adjustments based on potential subsidence cases. No analysis of the operation of the reservoir system was made. It was assumed that the storage required for a 100-year event as shown in Table IV-1 was unchanged by the impacts of subsidence.

Study Area

Description of Reservoirs

The dams are located in the upper Buffalo Bayou watershed of the San Jacinto River Basin approximately 18 miles west of downtown Houston. Exhibit IV-1 presents a vicinity map of the reservoir areas. The dams are similar structures consisting of long earthen embankments, each having five gated conduits to discharge flood waters into downstream channels. The operating procedure for the reservoir system permits the gated conduits to discharge a maximum of 2,000 cfs downstream during periods of minor rainfall. However, if the rainfall rate reaches or exceeds 1 inch in 24 hours on the downstream area, the gates are closed and no discharges are permitted until downstream flood levels recede or critical reservoir levels are reached.

The design of the reservoirs is such that the majority of 100-year frequency floodwaters are contained within government-owned property. The reservoir embankments are constructed substantially higher than the 100-year pool level, since they were designed to protect against failure from floods approaching probable maximum intensities. Table IV-1 presents pertinent reservoir data, obtained from Corps of Engineers publications, concerning each dam. All existing elevations are referenced to the 1929 National Geodetic Vertical Datum (NGVD), 1973 adjustment. For the purposes of this report, the level at which flood waters begin to spill around the low reservoir ends will be identified as the Maximum Flood Control Pool Level.

Watershed Characteristics

The Buffalo Bayou watershed above the Addicks and Barker dams, as was presented on Exhibit II-1, comprises approximately 266 square miles of drainage area. The Addicks Reservoir watershed comprises approximately 136 square miles of this area and is roughly 17 miles long, 10 miles wide, and varies in elevation from 170 feet in the upper reaches to 75 feet near the dam. The Barker Reservoir watershed comprises the remaining 130 square miles of the total drainage area and is roughly 27 miles long, 6 miles wide, and varies in elevation from 200 feet in the upper reaches to 75 feet near the dam.

Runoff is collected by eight subwatersheds within the reservoir system: South Mayde, Bear, Langham, Horsepen, and Turkey creeks within the Addicks Reservoir watershed and Willow Fork, Mason Creek, and an unnamed tributary designated as T103-00-00 within the Barker Reservoir watershed. Most development in these watersheds is single-family residential and is generally located along the lower reaches of each creek. Land use in the upper reaches of the watersheds is primarily agricultural. Most of the major creeks located adjacent to developed areas have been modified to earthen trapezoidal channels to provide adequate drainage for these areas. The smaller tributaries in the upper reaches of the watersheds are relatively natural or have been rectified only to a level necessary for agricultural drainage.

Relationship to Historic Subsidence

The Addicks and Barker reservoir areas experienced approximately two feet of subsidence between 1906 and 1978. However, current figures from the Addicks extensometer now indicate rates of subsidence of approximately one foot every seven years because of accelerated groundwater development and continuing declines in aquifer pressures.

Subsidence can affect the reservoirs in one of two ways. First, if the land upstream of the government boundaries subsides at a more rapid rate than the dams, then the 100-year frequency pool level would inundate additional private lands outside of the boundaries. Secondly, if subsidence occurs at the dams more rapidly than the land upstream of the dams, the operation of the reservoirs may be affected. This study addresses the extent of impact that can be anticipated if the reservoirs are subjected to various subsidence gradients.

ANALYSIS OF RESERVOIR CAPACITY

Development of Base Models

In order to determine any change in storage capacity due to subsidence, it was first necessary to develop a computer model to simulate the published storage-capacity data for each reservoir. The elevation-capacity curves contained in the U.S. Army Corps of Engineers' August 1977 report "Hydrology, Addicks and Barker Reservoirs" were selected as the benchmark data. These curves were computed by digitizing detailed one-foot contour interval topographic maps developed for this purpose. To aid in reconstruction, copies of the topographic maps of the reservoir areas were obtained from the Galveston District Corps of Engineers. Exhibits IV-2 and IV-3 present smoothed contour maps derived from the detailed Corps' data.

Elevation data was digitized into a computer data base in the form of station-elevation data for use in the Hydrologic Engineering Center's computer program entitled "HEC-2, Water Surface Profiles." A cross-section interval of 200 feet between sections was used to provide reasonably accurate computations. A series of HEC-2 storage-discharge runs were made using a minimal discharge of 0.1 cfs and varying starting water surface elevations, ranging from natural ground near the reservoir control structures to the maximum flood control elevation at the ends of the dam, to obtain elevation-capacity data.

The resulting storage-capacity curves were compared against the benchmark Corps of Engineers curves to assure that the new curves were consistent with the Corps' curves. Table IV-2 presents the existing and reconstructed storage-capacity curves for Addicks and Barker reservoirs, respectively. Based on the small differences between the new curves and the benchmark data, the computer models were adopted for the base condition.

Subsidence Cases

Four cases of subsidence were examined. Case 1R is an extension of the initial areal subsidence case studied in the Riverine Flooding Analysis with the Brays Bayou Case 2 centering. It should be noted that although the HGCSD's Plan was not specifically analyzed in the reservoir analysis, Case 1R has similar, but more severe impacts on the reservoirs. The HGCSD's Plan results are about one-half the subsidence simulated by Case 1R. Thus, Case 1R is indicative of the impacts of the HGCSD's Plan on the reservoir system. The other three cases all simulated gradient changes equal to the maximum gradient that might be expected to occur from reasonable subsidence cases with the gradients sloped toward various critical locations on the reservoir embankments. Case 2R assumed an anticipated gradient change of 0.017 percent (0.9 foot per mile) for Addicks Reservoir and 0.023 percent (1.2 feet per mile) for Barker Reservoir occurring between the ends of dam and the southeastern reservoir embankment. Cases 3R and 4R assumed the 0.017 percent (0.9 foot per mile) anticipated gradient change occurring along a line connecting the ends of the dam, Case 3R towards the northeast and Case 4R towards the southwest. Exhibits IV-4 and IV-5 present the lines of equal subsidence for each case for Addicks and Barker reservoirs, respectively.

Modification of Base Models

The modification of the base condition storage relationship developed for this study was accomplished using a computer technique which adjusted the topography of the reservoir according to the specific case of subsidence evaluated. This technique consisted of first digitizing each of the gradient change cases into a data base and then adjusting the station-elevation data of each HEC-2 cross-section according to the lines of equal subsidence. The HEC-2 program was then executed to determine modified storage-capacity curves for each case. Revised contour maps of the basins were developed by interpolation between subsidence contour lines and station-elevation data to facilitate the drawing of elevation contours.

Evaluation of Revised Capacities

Using the modified storage-capacity curves presented in Tables IV-3 and IV-4 and shown on Exhibits IV-6 and IV-7, the change in volume and freeboard above the maximum flood control pool level was determined for each case. Case 1R indicated differing results for the two reservoirs, decreasing the maximum storage capacity on one and increasing the capacity on the other. Cases 2R, 3R, and 4R produced similar results on each reservoir. In general, Case 2R increased the maximum storage capacities and Cases 3R and 4R decreased the maximum storage capacities. A case-by-case evaluation follows. Again, it should be noted that the revised 100-year pool elevations are based on a volume of storage equal to that currently specified as the 100-year storage by the Corps of Engineers and, that for the purposes of this report, the level at which flood waters begin to spill around the low reservoir ends will be identified as the Maximum Flood Control Pool Level. Although elevations are referenced to a specific datum, the significance of a higher flood elevation versus a lower elevation is not immediately apparent because subsidence will result in a general lowering of the land. It is possible, therefore, that a lower flood elevation could be more critical because surrounding ground elevations would also be lower. Rather than attempt to determine the significance of each flood elevation, the study emphasizes instead the impact on flooded area and the safety factor or freeboard of the dam. Table IV-5 summarizes the resultant flooded acreage and reservoir storage capacity for each case studied.

Case 1R

Case 1R applied to Addicks Reservoir resulted in a simulated subsidence with land elevation decreases, ranging from less than 5 feet in the western areas to about 7 feet along the eastern embankment with the north end of the dam experiencing approximately 1.5 feet more subsidence than the southwestern end. The maximum flood control storage capacity of the reservoir was decreased approximately 6,540 acre-feet (-3.3 percent) by the tilting effect of this case. As a result, the freeboard above the maximum flood control pool was reduced approximately 0.6 foot along the eastern embankment (from 4.0 feet to 3.4 feet north of the Clay Road crossing and from 8.0 feet to 7.4 feet south of Clay Road). The resultant 100-year flooded area was reduced from the existing flood area of 11,470 acres to 11,313 acres. Inundation of private lands increases from 170 acres to 209 acres along the north boundary near Turkey Creek.

Applying Case 1R to Barker Reservoir resulted in simulated subsidence ranging from about 5 feet at the ends of the dam to a little over 7 feet along the southeastern embankment. Unlike Addicks Reservoir, only 0.4 feet of differential subsidence was observed between the ends of the dam. The maximum flood control storage-capacity of the reservoir was increased by approximately 5,630 acre-feet (2.7 percent) due to the gradient change. As a result, the freeboard above the maximum flood control pool along the southeastern embankment of the reservoir was reduced from 4 feet to 2.2 feet. The resultant 100-year total flooded area was reduced from the existing flood area of 12,681 acres to 11,934 acres. Inundation of private lands decreases from 776 acres to 255 acres along the reservoir's western boundary.

Case 2R

Applying Case 2R to Addicks Reservoir resulted in simulated subsidence of approximately 3 feet from the ends of the dam to along the southeastern embankment. The maximum flood control storage-capacity of the reservoir was increased by approximately 13,250 acre-feet (6.6 percent) by the tilting of the reservoir to the southeast. As a result, the freeboard above the maximum flood control pool was reduced approximately 3.2 feet at the southeastern corner of the dam (from 9.5 feet to 6.3 feet). The resultant 100-year total flooded area was reduced from 11,470 acres to 10,974 acres. Inundation of private lands decreases from 170 acres to 144 acres with the reduction located along the north boundary near Turkey Creek.

Simulated subsidence of approximately 4.5 feet from the ends of the dam to along the southeastern embankment occurred on Barker Reservoir. The maximum flood control storage capacity of the reservoir was increased by approximately 18,970 acre-feet (9.1 percent) by the reservoir tilting. As a result, the freeboard above the maximum flood control pool was reduced from 4.0 feet to zero freeboard along the southeastern embankment from the Barker-Clodine Road crossing to just south of the Noble Road crossing. Just south of Beeler Road the embankment would be approximately 0.5 foot lower than the ends of the dam. The resultant 100-year total flooded area was reduced from 12,681 acres to 11,521 acres. Inundation of private lands decreases from 776 acres to 180 acres along the reservoir's western boundary.

Case 3R

Simulated subsidence of approximately 7.5 feet from the southwestern end of the dam to the north end of the dam occurred as a result of applying Case 3R to Addicks Reservoir. The maximum flood control storage capacity of the reservoir was decreased by approximately 45,330 acre-feet (-22.6 percent) by the tilting to the northeast. As a result, the freeboard above the maximum flood control pool along the southwestern embankment was increased from 5.0 feet to 10.9 feet, but remained unchanged along the northeastern embankments. The resultant 100-year total flooded area was reduced from 11,470 acres to 10,973 acres. Inundation of private land increases from 170 acres to 370 acres along the north boundary near Turkey Creek.

Applying Case 3R to Barker Reservoir resulted in a simulated subsidence of approximately 8.0 feet from the southwestern end of the dam to the northeastern embankment corner, with subsidence to the north end of the dam of approximately 6.5 feet. The maximum flood control storage-capacity of the reservoir decreased by approximately 34,140 acre-feet (-16.3 percent) by the tilting to the northeast. As a result, the freeboard above the maximum flood control pool at the northeastern embankment was reduced from 6.5 feet to 5 feet. The resultant 100-year total flooded area was reduced from 12,681 acres to 11,398 acres. Flooding on private land decreases from 776 acres to 95 acres along the western boundary near Willow Fork.

Case 4R

Addicks Reservoir experienced a simulated subsidence of approximately 7.0 feet from the north end of the dam to the southwestern end of the dam by application of Case 4R. The maximum flood control storage capacity of the reservoir decreased by approximately 46,200 acre-feet (-23.0 percent) by the tilting to the southwest. As a result, the freeboard above the maximum flood control pool was increased from 4.0 feet to 9.9 feet along the northeastern embankment, but remained unchanged along the southwestern embankment. The resultant 100-year total flooded area was increased from 11,470 acres to 11,510 acres. Inundation of private land decreases from 170 acres to 2 acres along the north boundary near Turkey Creek.

Applying Case 4R to Barker Reservoir resulted in subsidence of approximately 6.5 feet from the north end of the dam to the southwestern end of the dam. The maximum flood control storage capacity of the reservoir decreased by approximately 65,960 acre-feet (-31.6 percent) by the tilting to the southwest. As a result, the freeboard above the maximum flood control pool was increased along the northeastern embankment from 4.0 feet to 10.5 feet. The resultant 100-year total flooded area was increased from 12,681 acres to 13,210 acres. Inundation of private land increases from 776 acres to 1,453 acres along the western boundary near Willow Fork.

Exhibits IV-8 and IV-9 present the resultant pool limits for the 100-year flood for each case. Tables IV-6 and IV-7 present a case-by-case comparison of freeboard depths for both the maximum flood control storage and the 100-year storage for each subsidence case. Specific detailed examples of contour adjustments and changes in the 100-year storage are presented on Exhibits IV-10 and IV-11 for the most critical changes to each reservoir. The most severe impacts occurred in Cases 3R and 4R. The simulated subsidence from Case 3R resulted in a large increase in inundation of private lands for Addicks Reservoir. Application of Case 4R to Barker Reservoir resulted both in a large loss in storage capacity and in a large increase in flooding on private lands.

Capacity Evaluation Considering Proposed Embankment Modifications

The Corps of Engineers has proposed a plan of improvement designed to upgrade the integrity of the dams by providing additional freeboard for occurrence of the Spillway Design Flood (SDF). The SDF for these dams corresponds to the Probable Maximum Flood, defined as the flood caused by the theoretically greatest depth of precipitation for a given duration reasonably possible over a particular drainage area. The proposed plan consists of raising the main embankment on portions of Addicks Dam approximately 0.5 to 3 feet and raising the main embankment on Barker Dam approximately 2 to 5 feet. This design provides the existing dams with a minimum 3-foot freeboard plus additional freeboard for wave run-up in certain reaches for the SDF. Current 100-year freeboard varies, but is a minimum of 8 feet. In addition, the ends of the reservoir embankments are proposed to be armor-plated to prevent erosion.

The proposed modifications do not alter the existing maximum flood control storage of the reservoirs; therefore, the storage-capacity curves presented in Tables IV-3 and IV-4 can be used to evaluate subsidence on the reservoirs with these modifications. In general, maximum flood control storage capacities for each subsidence case would be identical to the modified curves previously discussed, except for Barker Reservoir under the Case 2R condition. Because of the proposed increase in the embankment elevation along the southeastern levee of 3.5 feet under Case 2R, the maximum flood control storage capacity of Barker Reservoir would be increased to 235,980 acre-feet, with a reservoir elevation at the ends of the dam of 106 feet. This represents an increase of 12.9 percent in the total storage capacity of the reservoir. Table IV-8 presents a comparison of the pre-project embankment elevations with the post-project embankment elevations along each reservoir for the subsidence cases studied. Freeboard for the reservoirs would remain the same as shown on Tables IV-6 and IV-7 for the low reservoir embankments near the ends of dam. Changes along the affected high reservoir embankments are presented in Table IV-9 for each reservoir. The resultant 100-year flood pool levels would not be affected.

In order to evaluate the total effect on the Corps' proposed plan, it was necessary to tabulate freeboard data referencing SDF pool levels. The reference SDF pool levels determined by the Corps of Engineers for existing conditions are 118.1 feet and 110.3 feet, respectively, for Addicks and Barker reservoirs. Tables IV-10 and IV-11 present the SDF freeboard data for pre- and post-project conditions for all subsidence cases. As can be seen by the tables, the post-project SDF freeboard is reduced along the high embankments during most of the subsidence cases but does indicate an improvement over pre-project conditions for this event.

CONCLUSIONS AND RECOMMENDATIONS

The four cases of simulated subsidence imposed on the Addicks and Barker reservoir system demonstrate the potential impacts that subsidence could have on the reservoir capacities, freeboard depths, and inundated land. The study also resulted in a base model which can be used by the Sponsors or others to evaluate the impact of other subsidence conditions that affect the reservoir system.

Several general conclusions can be drawn from the analysis performed. Subsidence which tilts the reservoir toward the dam outlet structures is likely to have the least effect on the reservoir operation. The HGCSD's Plan produces this type of effect on the reservoirs, although the degree or magnitude of tilt would be approximately one-half of that analyzed. Storage capacity is increased and private land inundated is decreased. This pattern of subsidence does reduce the existing freeboard of the reservoirs, however, and if not addressed in future plans could subject downstream areas to a higher risk during extreme storm events.

On the other hand, subsidence which results in lowering of the ends of the embankment structure relative to the remainder of the structure and reservoir property has generally negative impacts with storage volume decreasing in all cases. Inundation of private land outside the government-owned property for the resulting 100-year ponding elevation may be increased or decreased depending on the specific pattern. Tilting Barker Reservoir toward the southwest (FM 1093) resulted in significant increases in inundation of private land while tilting the reservoir toward the northeast (IH-10) resulted in decreased privately-owned land inundation. Similar results were noted for Addicks; tilting toward the southwest (IH-10) decreased inundation of private land while tilting toward the north increased such inundation.

Based on the response to the patterns of subsidence evaluated, it can be concluded that subsidence projected in the HGCSD's Plan will have minimal impact on the reservoir operation or 100-year pool elevation relative to surrounding property. Other patterns which result in a tilting toward the ends of the dams can have significant impacts, however, and as a result it is recommended that all future revisions to the HGCSD's Plan include an evaluation of the resulting subsidence patterns on Barker and Addicks reservoirs. Additionally, it is recommended that the HGCSD and the Fort Bend County Drainage District explore the financial and technical feasibility of an ongoing interagency agreement which would allow an exchange of information on groundwater utilization and resulting subsidence patterns. This exchange would allow the HGCSD to better define subsidence within its jurisdiction, to monitor for changes in subsidence trends which would affect Addicks and Barker Reservoirs, and provide the Fort Bend County Drainage District with needed information on subsidence to better plan and regulate drainage and flood control.

CHAPTER V

REFERENCES

CHAPTER V

REFERENCES

CHAPTER I - INTRODUCTION

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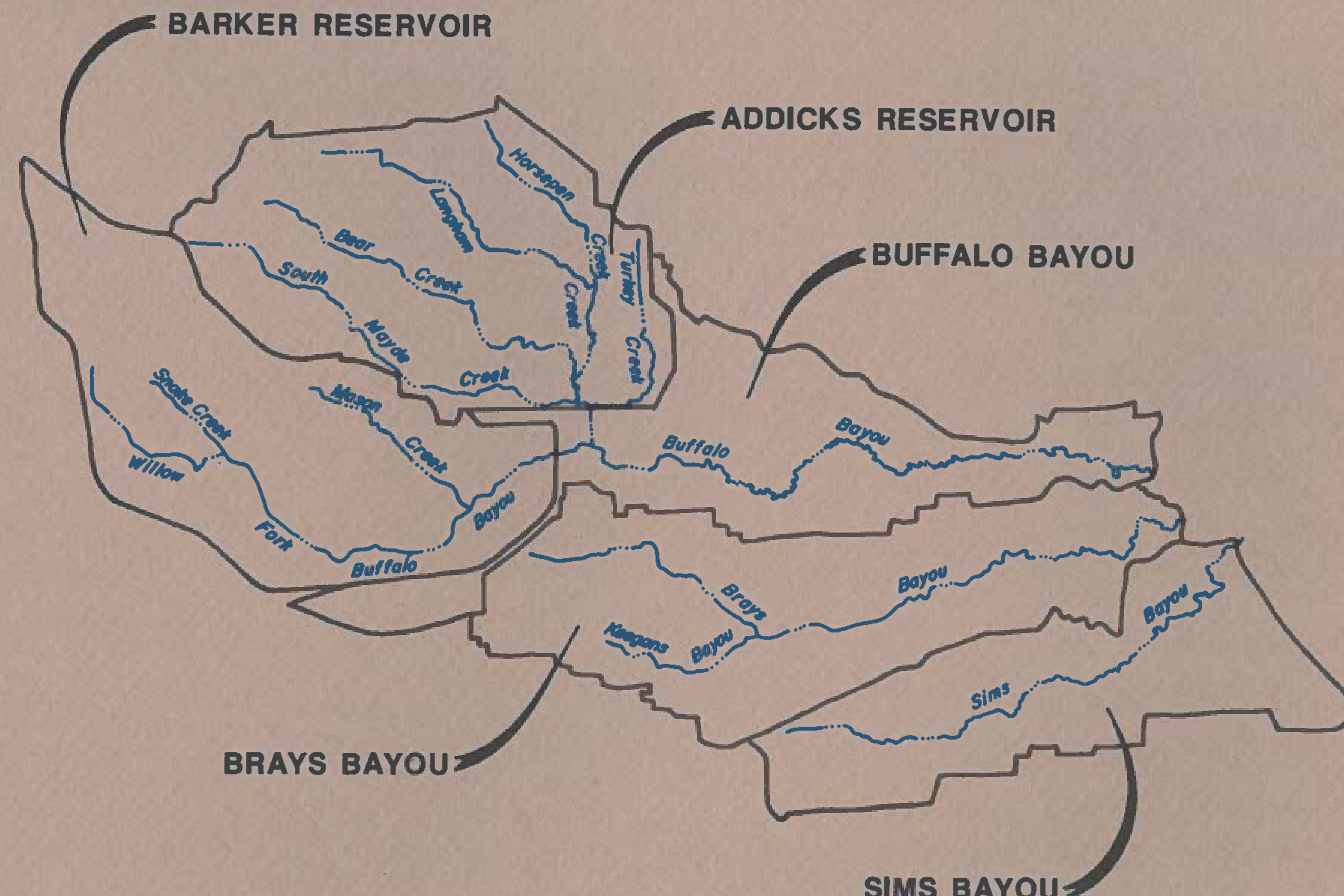
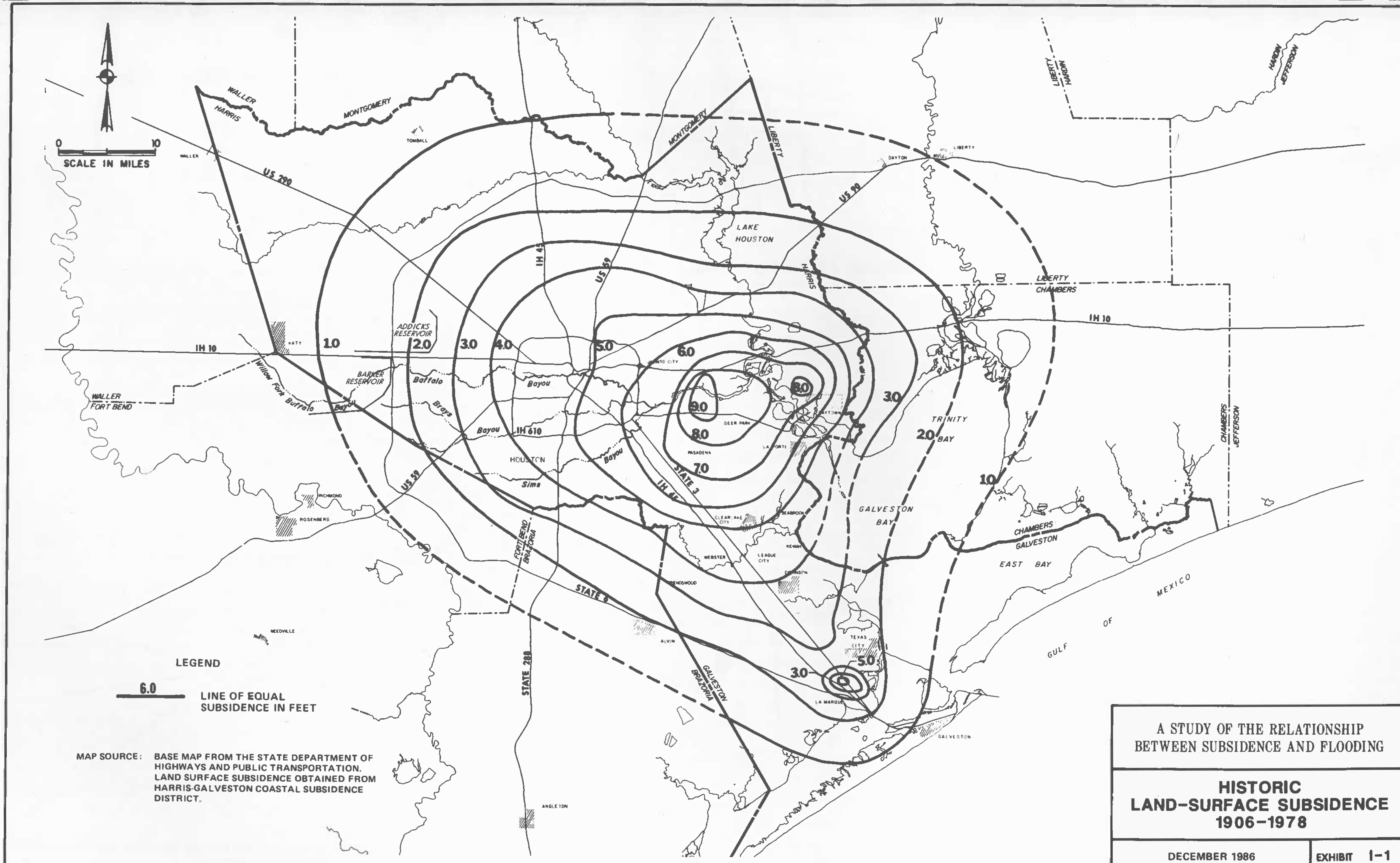
TABLES AND EXHIBITS





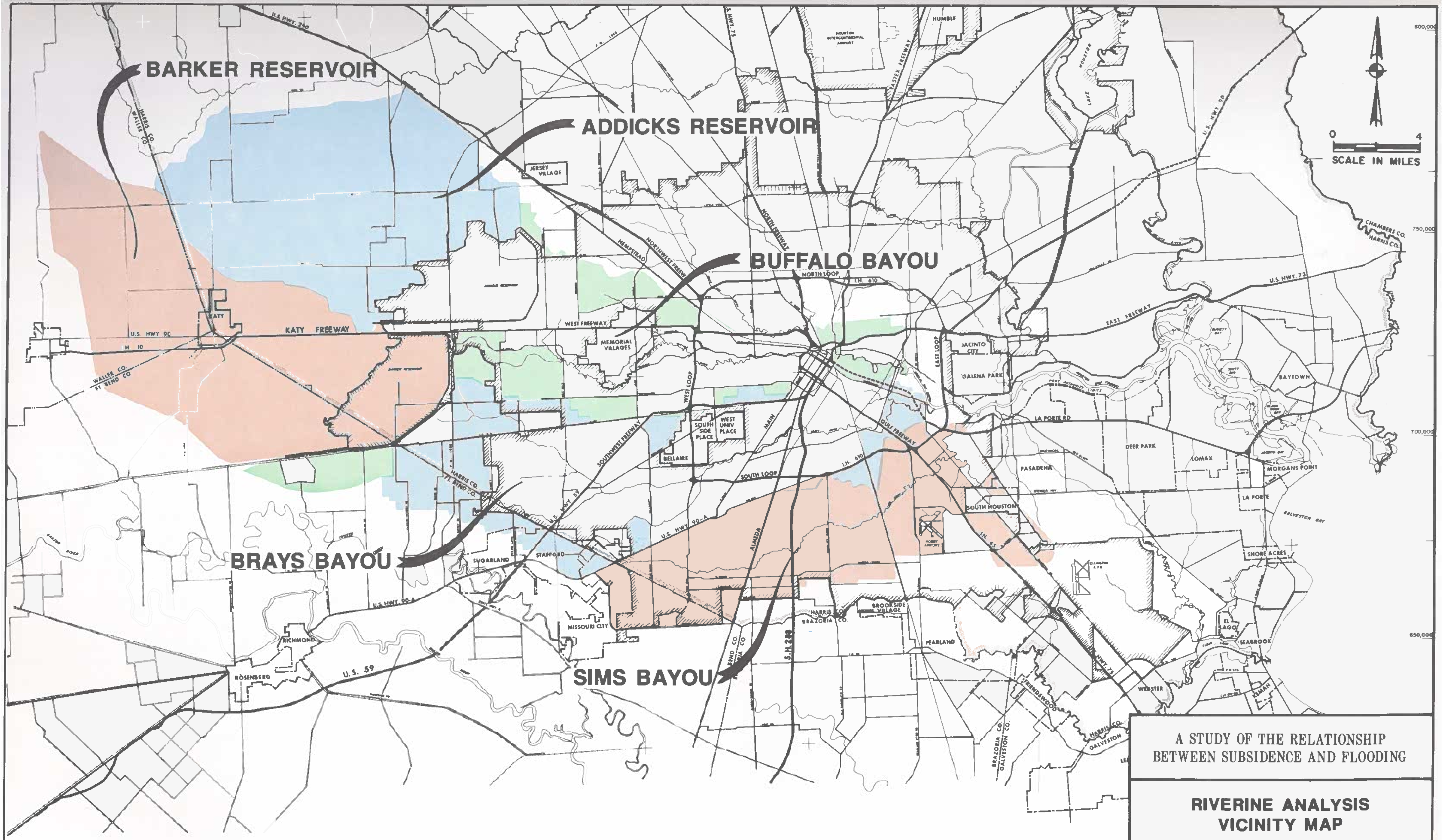
MAP SOURCE: BASE MAP FROM THE STATE DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION. LAND SURFACE SUBSIDENCE OBTAINED FROM HARRIS-GALVESTON COASTAL SUBSIDENCE DISTRICT.

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

HISTORIC LAND-SURFACE SUBSIDENCE 1906-1978

DECEMBER 1986

EXHIBIT I-1



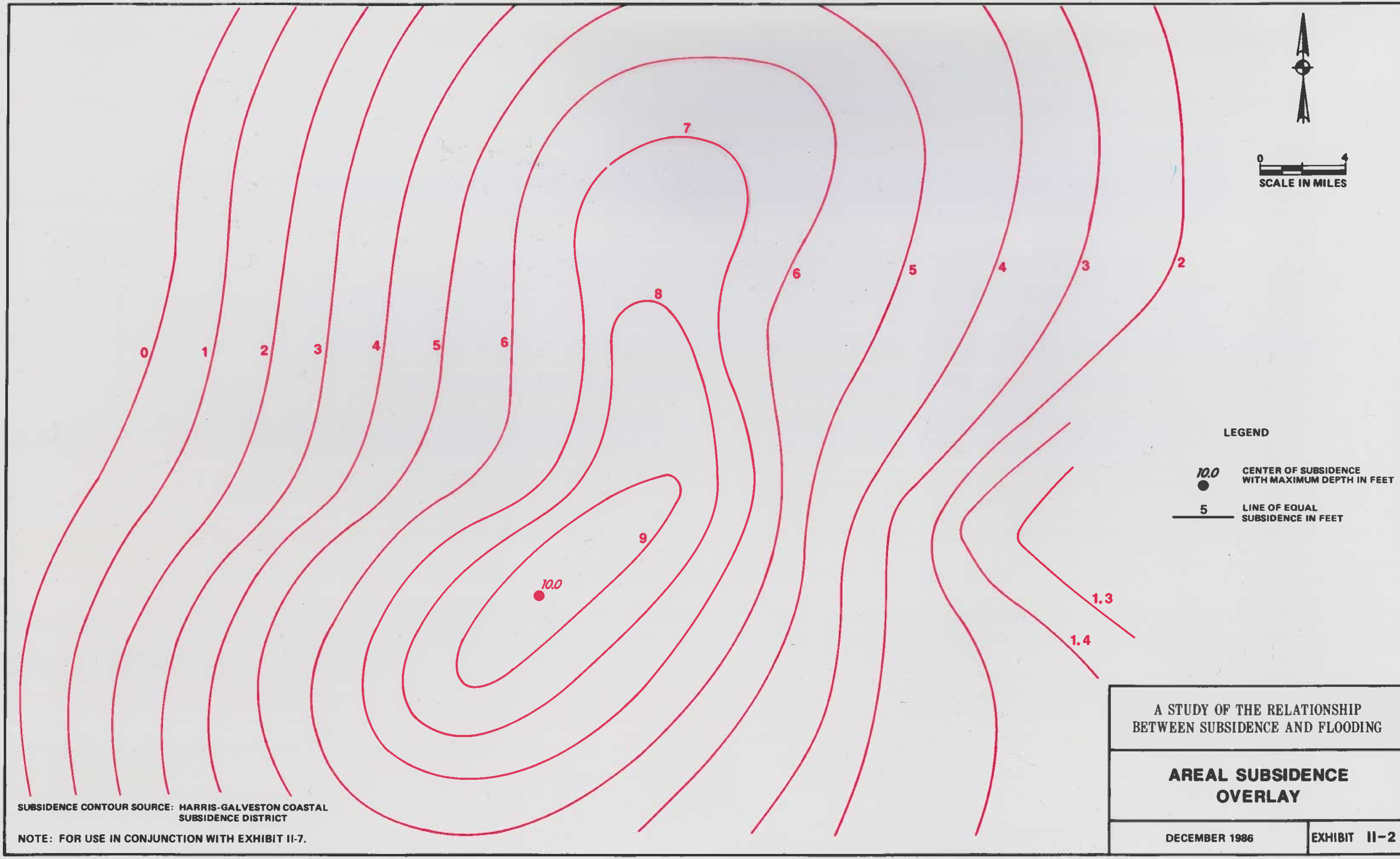
MAP SOURCE: CITY OF HOUSTON COMPREHENSIVE SURVEY, MONUMENTATION AND MAPPING PROGRAM 1976

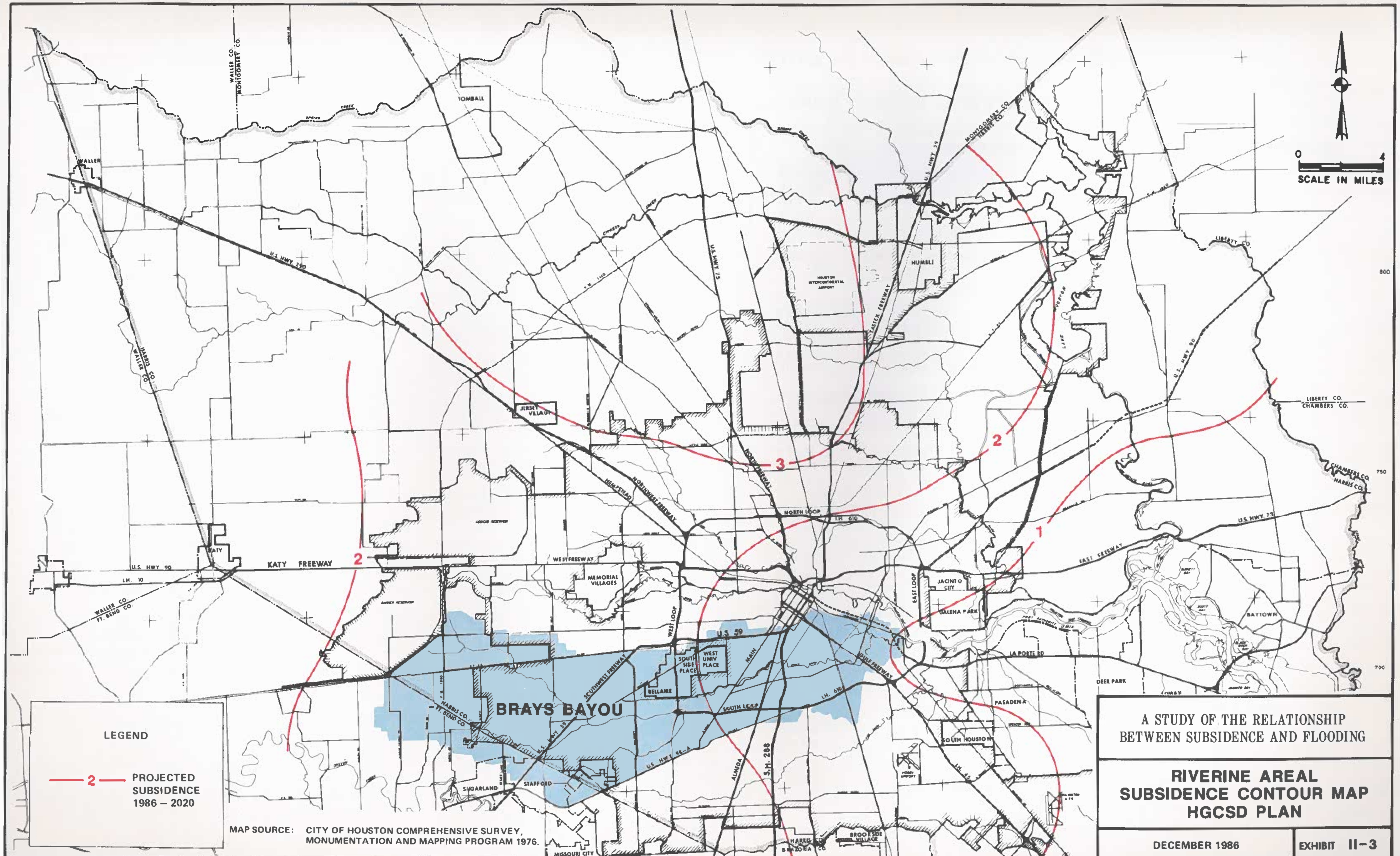
A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

RIVERINE ANALYSIS VICINITY MAP

DECEMBER 1986

EXHIBIT II-1


SCALE IN MILES
0
4
LEGEND
10.0
CENTER OF SUBSIDENCE WITH MAXIMUM DEPTH IN FEET
5
LINE OF EQUAL SUBSIDENCE IN FEET
0
1
2
3
4
5
6
7
8
9
10.0
6
5
4
3
2
1.3
1.4
SUBSIDENCE CONTOUR SOURCE: HARRIS-GALVESTON COASTAL SUBSIDENCE DISTRICT
NOTE: FOR USE IN CONJUNCTION WITH EXHIBIT II-7.
A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING
AREAL SUBSIDENCE OVERLAY
DECEMBER 1986
EXHIBIT II-2



A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

RIVERINE AREAL SUBSIDENCE CONTOUR MAP HGCSD PLAN

DECEMBER 1986

EXHIBIT II–3

TABLE II-1 - RIVERINE POINT RAINFALL AMOUNTS

Stream	Storm Frequency (Years)	Rainfall (Inches)
Addicks and Barker Reservoir Major Tributaries	10	8.2
	100	12.5
Buffalo Bayou & Brays Bayou	10	8.4
	100	12.6
Sims Bayou	10	8.6
	100	12.8

TABLE II-2 - LOSS RATES FOR STORM EVENTS

ADDICKS RESERVOIR MAJOR TRIBUTARIES

	Langham & Horsepen Creeks	Bear Creek	South Mayde Creek
STRKR	0.3	0.3	0.3
DLTKR	1.0	0.0	1.0
RTIOL	4.0	4.0	4.0
ERAIN	0.55	0.55	0.55
QRCSN	0.25 x Peak Q	0.10 x Peak Q	0.10 x Peak Q

BARKER RESERVOIR MAJOR TRIBUTARIES

	Willow Fork	Mason Creek
STRKR	0.3	0.3
DLTKR	1.0	1.0
RTIOL	4.0	4.0
ERAIN	0.55	0.55
QRCSN	0.15 x Peak Q	0.10 x Peak Q

BUFFALO BAYOU

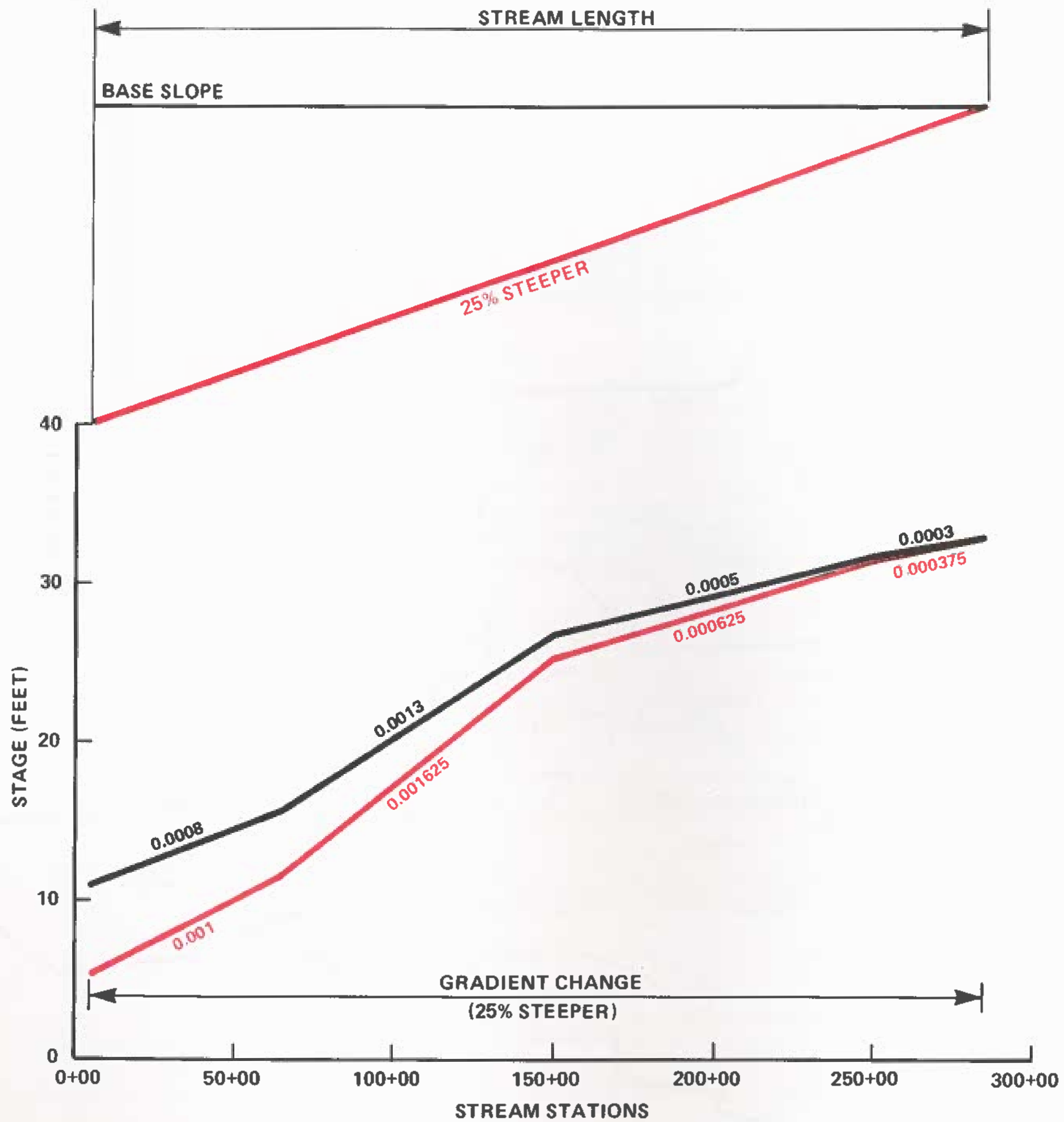
STRKR	0.1
DLTKR	2.0
RTIOL	1.0
ERAIN	0.55
QRCSN	0.03 x Peak Q

BRAYS BAYOU

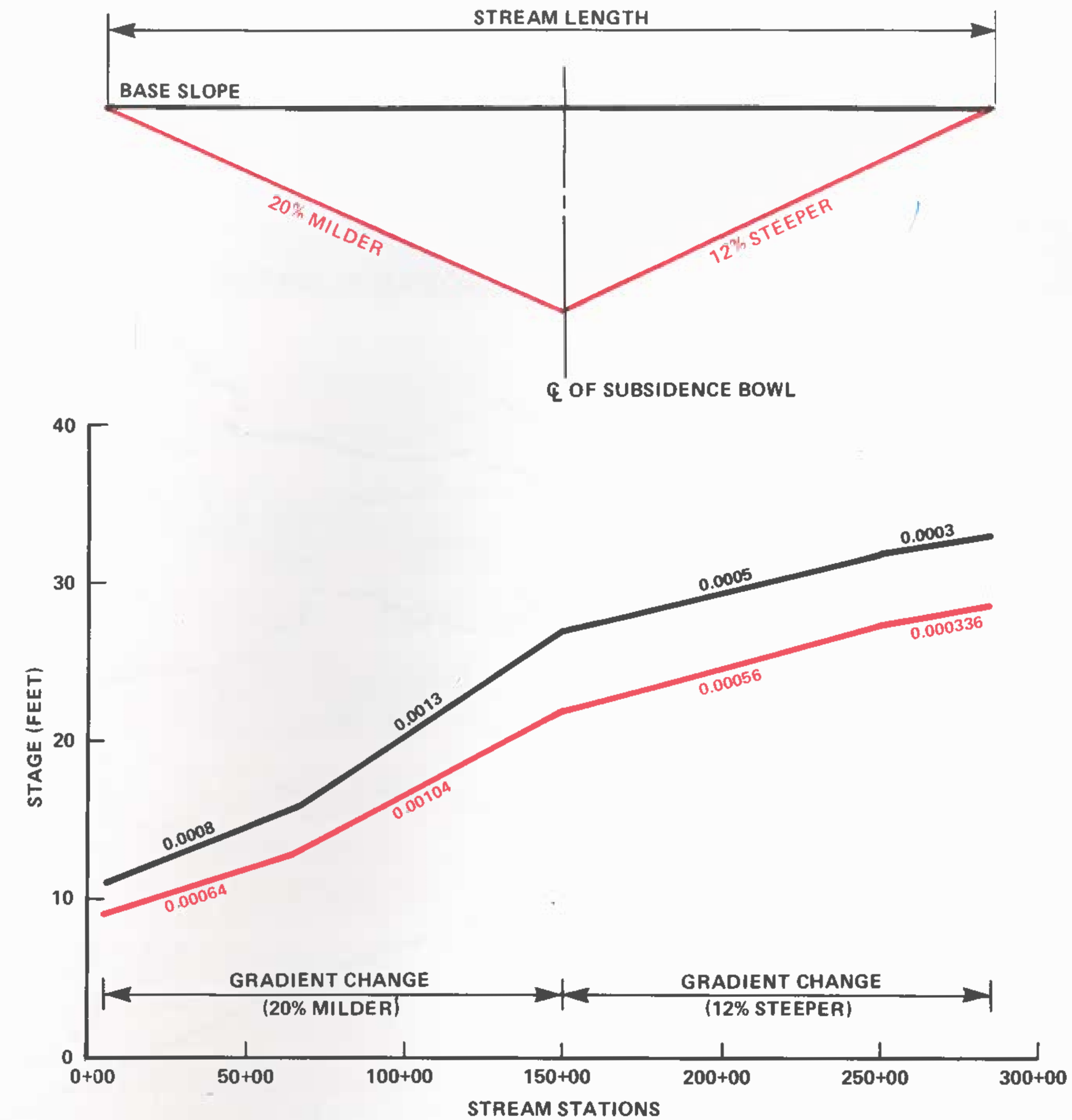
	Upper Brays Bayou	Keegans Bayou	Lower Brays Bayou
STRKR	0.3	0.35	0.08
DLTKR	2.0	2.0	2.0
RTIOL	2.8	2.8	1.0
ERAIN	0.54	0.54	0.54
QRCSN	0.07 x Peak Q	0.40 x Peak Q	0.10 x Peak Q

SIMS BAYOU

	Upper Sims Bayou	Lower Sims Bayou	Berry Bayou
STRKR	0.3	0.1	0.2
DLTKR	1.0	1.0	2.0
RTIOL	1.3	1.3	4.0
ERAIN	0.55	0.55	0.70
QRCSN	0.20 x Peak Q	0.20 x Peak Q	0.20 x Peak Q



EXAMPLE I – SUBSIDENCE BOWL OUTSIDE OF BASIN



EXAMPLE II – SUBSIDENCE BOWL WITHIN BASIN

LEGEND

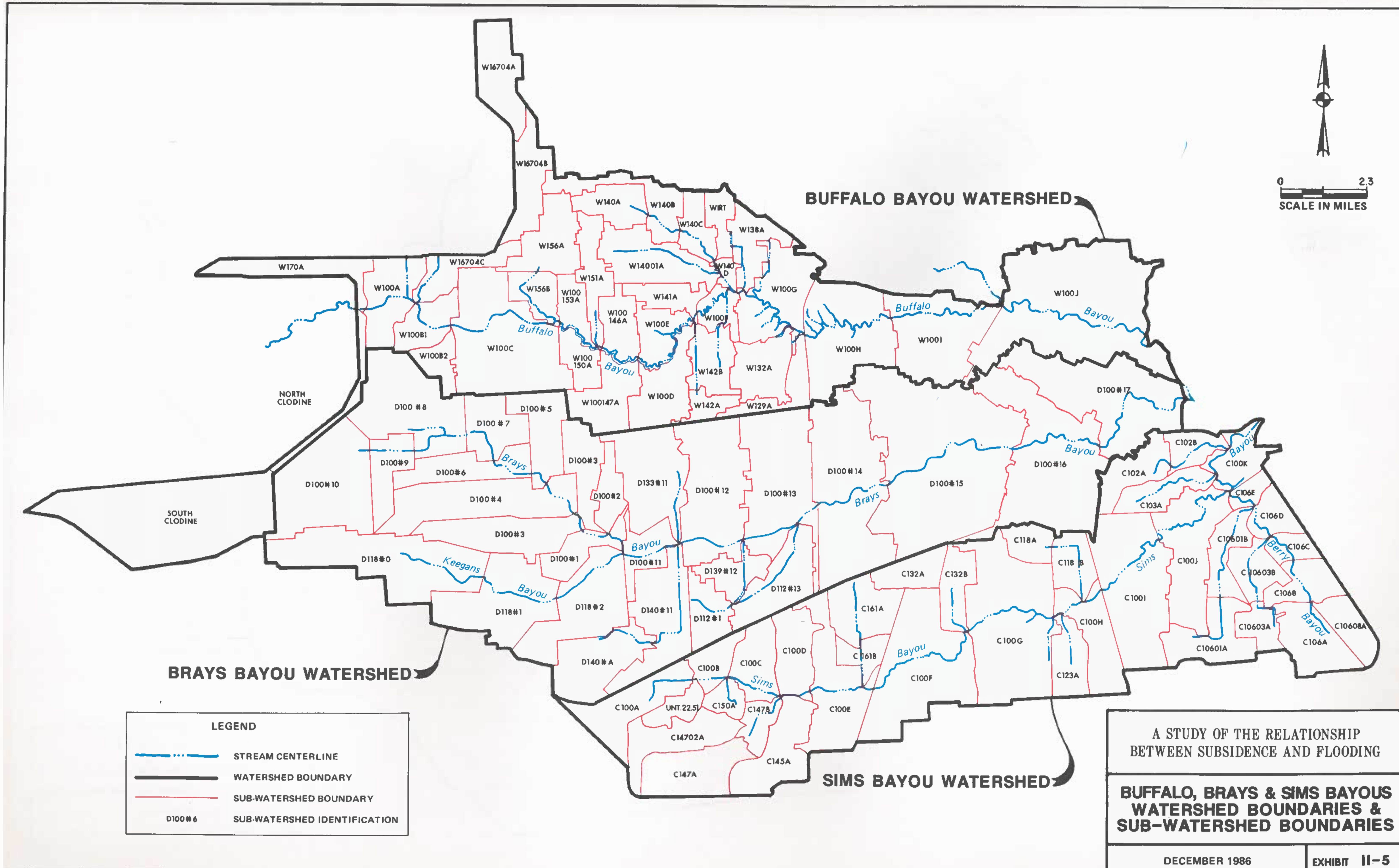
0.0008 — EXISTING STREAM GRADIENT WITH SLOPE IN FOOT PER FOOT

0.001 — ADJUST STREAM GRADIENT WITH SLOPE IN FOOT PER FOOT

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

SCHEMATIC OF STREAM GRADIENT ADJUSTMENTS

DECEMBER 1986 | EXHIBIT II–4

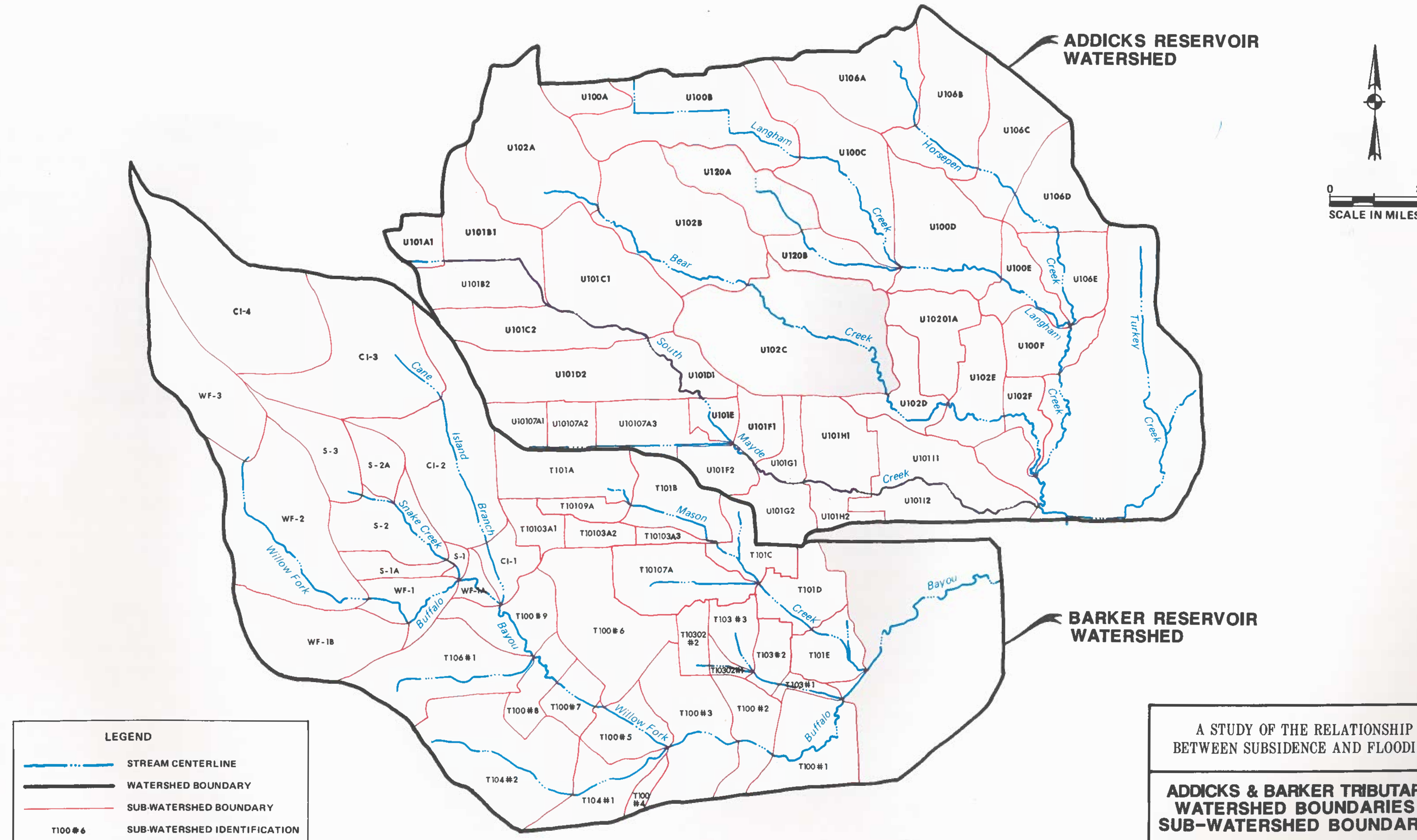


A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

BUFFALO, BRAYS & SIMS BAYOUS WATERSHED BOUNDARIES & SUB-WATERSHED BOUNDARIES

DECEMBER 1986

EXHIBIT II-5



A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

ADDICKS & BARKER TRIBUTARIES WATERSHED BOUNDARIES & SUB-WATERSHED BOUNDARIES

DECEMBER 1986 | EXHIBIT II-6

TABLE II-3 - CHANGES IN UNITGRAPH COEFFICIENTS DUE TO CHANGE IN CHANNEL GRADIENT (25 PERCENT FLATTER)

	Channel Gradient (ft/mi)		Existing Condition Unitgraph Coefficients				Subsided Condition Unitgraph Coefficients			
			10-Year		100-Year		10-Year		100-Year	
Subwatershed	Existing	Subsided	TC	R	TC	R	TC	R	TC	R
Willow Fork										
T100#1	2.20	1.87	3.23	19.37	3.23	15.27	3.53	20.29	3.53	16.00
T100#2	2.37	1.61	1.02	12.01	1.02	9.29	1.25	13.57	1.25	10.50
T100#3	1.55	1.09	3.02	24.85	3.02	18.51	3.64	27.57	3.64	20.54
T100#4	4.00	4.00	1.14	11.16	1.14	8.40	1.14	11.16	1.14	8.40
T100#5	2.50	1.93	1.53	15.88	1.53	12.10	1.76	17.21	1.76	13.11
T104#1	4.32	3.91	1.89	6.01	1.89	4.58	2.00	6.15	2.00	4.68
T104#2	4.87	2.90	1.84	28.14	1.84	19.84	2.42	33.14	2.42	23.36
T100#6	7.25	6.30	2.05	19.90	2.05	14.57	2.20	20.76	2.20	15.21
T100#7	8.78	8.14	0.35	8.49	0.35	6.27	0.36	8.71	0.36	6.43
T100#8	5.22	4.45	0.41	13.99	0.41	11.03	0.45	14.77	0.45	11.65
T100#9	10.00	10.00	0.72	18.54	0.72	13.86	0.72	18.54	0.72	13.86
T106#1	6.73	5.01	1.86	26.89	1.86	18.82	2.17	29.50	2.17	20.64
CI-1	15.10	15.10	0.50	8.55	0.50	8.55	0.50	8.55	0.50	8.55
WF-1A	14.35	13.53	0.35	3.65	0.35	3.65	0.36	3.72	0.36	3.72
S-1	17.08	16.47	0.23	2.47	0.23	2.47	0.24	2.50	0.24	2.50
S-1A	6.47	5.00	1.37	7.57	1.37	7.16	1.56	8.18	1.56	7.73
WF-1	11.96	11.05	0.83	6.16	0.83	6.16	0.86	6.32	0.86	6.32
WF-1B	6.61	5.97	2.03	20.45	2.03	15.72	2.14	21.11	2.14	16.22
CI-2	8.12	8.44	2.09	17.90	2.09	15.42	2.05	17.68	2.05	15.23
S-2	11.48	10.25	1.13	5.57	1.13	5.57	1.20	5.78	1.20	5.78
WF-2	3.96	3.96	3.63	20.68	3.63	17.82	3.63	20.68	3.63	17.82
S-3	4.19	4.19	1.97	23.21	1.97	17.64	1.97	23.21	1.97	17.64
CI-3	3.68	3.68	4.75	20.10	4.75	15.92	4.75	20.10	4.75	15.92
S-2A	6.04	6.04	0.99	5.87	0.99	5.87	0.99	5.87	0.99	5.87
CI-4	5.18	5.18	2.79	25.70	2.79	20.83	2.79	25.70	2.79	20.83
WF-3	3.92	3.92	1.86	25.82	1.86	20.66	1.86	25.82	1.86	20.66

TABLE II-4 - CHANGE IN PEAK FLOW FROM SUB-AREAS ON WILLOW FORK - (25 PERCENT FLATTER)

	Existing Condition Peak Discharges (cfs)		Subsided Condition Peak Discharges (cfs)	
Subwatershed	10-Year	100-Year	10-Year	100-Year
T100#1	512	1,028	491	988
T100#2	447	888	404	808
T100#3	494	1,052	450	961
T100#4	96	193	96	193
T100#5	320	654	299	613
T104#1	933	1,749	919	1,718
T104#2	630	1,413	543	1,228
T100#6	667	1,423	643	1,374
T100#7	338	682	331	669
T100#8	268	526	256	504
T100#9	341	710	341	710
T106#1	420	943	386	872
CI-1	336	534	336	534
WF-1A	282	432	277	426
S-1	220	329	218	327
S-1A	288	479	271	453
WF-1	709	1,118	694	1,097
WF-1B	664	1,326	627	1,292
CI-2	1,061	1,970	1,072	1,990
S-2	1,209	1,899	1,175	1,847
WF-2	1,058	1,982	1,058	1,982
S-3	292	609	292	609
CI-3	1,045	2,084	1,045	2,084
S-2A	506	797	506	797
CI-4	1,030	2,047	1,030	2,047
WF-3	533	1,073	533	1,073

TABLE II-5 - WILLOW FORK EXISTING STORAGE-DISCHARGE INFORMATION

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Volume (ac-ft)	Travel Time (hrs.)
Willow Fork (T100-00-00)				
WF-2	WF-1	1,840	663	3.35
		3,900	1,620	
		7,450	2,963	
		11,800	4,310	
		16,000	5,498	
		19,200	6,343	
WF-1	WF-1A	1,840	149	0.95
		3,900	522	
		7,450	958	
		11,800	1,388	
		16,000	1,743	
		19,200	1,982	
17.36	15.56	2,260	198	1.10
		4,750	651	
		9,300	1,325	
		14,500	2,026	
		19,600	2,668	
		23,520	3,150	
15.56	14.36	2,290	102	0.99
		4,800	438	
		9,400	1,225	
		14,700	1,950	
		19,800	2,535	
		23,760	2,931	
14.36	13.29	2,400	93	0.33
		5,100	172	
		9,900	463	
		15,400	926	
		20,800	1,327	
		24,960	1,623	
13.29	11.50	2,450	157	0.60
		5,200	324	
		10,100	1,273	
		15,700	2,159	
		21,200	2,946	
		25,440	3,782	
11.50	9.75	2,520	442	1.57
		5,330	1,396	
		10,400	2,799	
		16,200	4,066	
		21,900	5,124	
		26,280	5,853	
9.75	8.95	2,580	300	0.85
		5,400	751	
		10,500	1,257	
		16,500	1,737	
		22,300	2,140	
		26,760	2,415	
8.95	5.90	2,640	2,496	2.39
		5,550	4,450	
		10,800	6,842	
		16,900	9,032	
		22,900	10,868	
		27,480	12,123	

TABLE II-6 - WILLOW FORK STORAGE-DISCHARGE RELATED TO SUBSIDENCE (25 PERCENT FLATTER)

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Volume (ac-ft)	Travel Time (hrs.)
Willow Fork (T100-00-00)				
WF-2	WF-1	1,840	846	3.97
		3,900	1,914	
		7,450	3,491	
		11,800	5,134	
		16,000	6,527	
		19,200	7,514	
WF-1	WF-1A	1,840	300	1.15
		3,900	707	
		7,450	1,203	
		11,800	1,701	
		16,000	2,112	
		19,200	2,398	
17.36	15.56	2,260	241	1.51
		4,750	872	
		9,300	1,981	
		14,500	3,080	
		19,600	4,050	
		23,520	4,707	
15.56	14.36	2,290	105	1.14
		4,800	525	
		9,400	1,596	
		14,700	2,509	
		19,800	3,188	
		23,760	3,641	
14.36	13.29	2,400	82	0.31
		5,100	167	
		9,900	482	
		15,400	994	
		20,800	1,424	
		24,960	1,743	
13.29	11.50	2,450	148	0.75
		5,200	367	
		10,100	1,640	
		15,700	2,858	
		21,200	3,866	
		25,440	4,608	
11.50	9.75	2,520	428	1.57
		5,330	828	
		10,400	2,630	
		16,200	4,276	
		21,900	5,562	
		26,280	6,431	
9.75	8.95	2,580	250	1.01
		5,400	657	
		10,500	1,413	
		16,500	2,090	
		22,300	2,633	
		26,760	2,995	
8.95	5.90	2,640	1,418	3.89
		5,550	2,899	
		10,800	5,469	
		16,900	7,726	
		22,900	9,608	
		27,480	10,913	

TABLE II-7 - TYPICAL FLOW CHANGES ON WILLOW FORK ATTRIBUTED TO CHANGES IN UNITGRAPH COEFFICIENTS AND REVISED STORAGE-DISCHARGE RELATIONSHIPS CAUSED BY SUBSIDENCE - (25 PERCENT FLATTER)

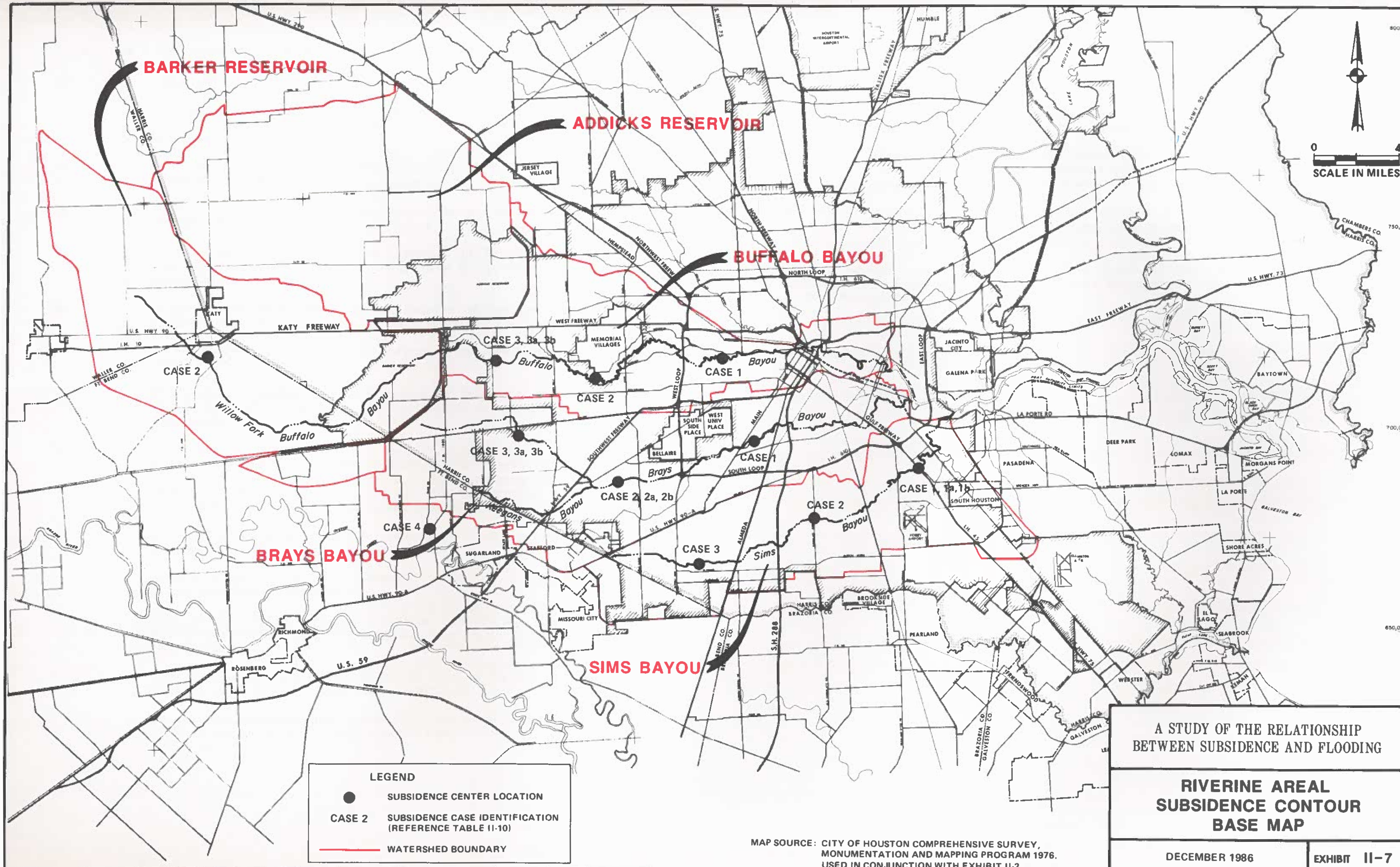
Drainage Area (Mi^2)	Peak Discharges (cfs): I - Existing Conditions 10-Year	I - Existing Conditions 100-Year	II - Existing Unitgraph Coef. and Revised Storage-Discharge Relationships 10-Year	II - Existing Unitgraph Coef. and Revised Storage-Discharge Relationships 100-Year
20.34	2060	3670	2000	3630
28.71	3370	5720	3130	5530
29.24	3350*	5730	3100*	5530
55.38	5620	9060	5490	8890
61.80	6040	9950	5860	9660
63.26	6070	10070	5890	9740
64.50	6130	10140	5920	9780
71.49	6500	10870	6260	10400
80.83	7050	12000	6690	11260
87.55	7340	12550	6910	11650
91.34	7130*	12330*	7010	11790

Drainage Area (Mi^2)	Peak Discharges (cfs): III - Adjusted Unitgraph Coef. and Revised Storage-Discharge Relationships 10-Year	III - Adjusted Unitgraph Coef. and Revised Storage-Discharge Relationships 100-Year	Difference Between Conditions II and III (Percent) 10-Year	Difference Between Conditions II and III (Percent) 100-Year
20.34	1990	3610	0.50	0.55
28.71	3120	5520	0.32	0.18
29.24	3100*	5520	0.00	0.18
55.38	5490	8890	0.00	0.00
61.80	5850	9640	0.17	0.21
63.26	5870	9720	0.68	0.21
64.50	5910	9760	0.17	0.20
71.49	6240	10390	0.32	0.10
80.83	6650	11200	0.60	0.54
87.55	6870	11610	0.58	0.34
91.34	6970	11750	0.57	0.34

*Inconsistencies are attributable to apparent instabilities in stream routings.

TABLE II-8 - CONSTANT SLOPE ANALYSIS REACHES

Stream	Reach Stations Upst.	Reach Stations Dwnst.	Constant Slope (ft/ft)	Channel Condition
Bear Creek	91349	67079	0.00118	Natural
(U102-00-00)	67079	19694	0.00076	Improved
Langham Creek	90755	76200	0.00052	Improved
(U100-00-00)	76200	52300	0.00096	Improved
	52300	49000	0.00280	Improved
	49000	17899	0.00063	Improved
Horsepen Creek	41346	23512	0.00128	Natural
(U106-00-00)	23512	528	0.00085	Improved
South Mayde	99975	55232	0.00115	Natural
(U101-00-00)	55232	19219	0.00077	Improved
Mason Creek	40281	27647	0.00059	Improved
(T101-00-00)	27647	18100	0.00070	Improved
	18100	7000	0.00131	Improved
	7000	528	0.00077	Improved
Willow Fork	114259	98894	0.00087	Natural
(T100-00-00)	98894	70646	0.00088	Natural
	70646	30994	0.00032	Natural
Willow Fork	114259	98894	0.00080	Improved
(100-year	98894	70646	0.00125	Improved
design channel)	70646	30994	0.00035	Improved
Brays Bayou	157000	132000	0.00039	Improved
(D100-00-00)	132000	116200	0.00082	Improved
	116200	93000	0.00048	Improved
	93000	0	0.00064	Improved
Buffalo Bayou	250800	114800	0.00041	Natural
(W100-00-00)	114800	83200	0.00033	Natural
Sims Bayou	127000	105000	0.00121	Improved
(C100-00-00)	105000	0	0.00054	Improved



A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

RIVERINE AREAL SUBSIDENCE CONTOUR BASE MAP

DECEMBER 1986

EXHIBIT II-7

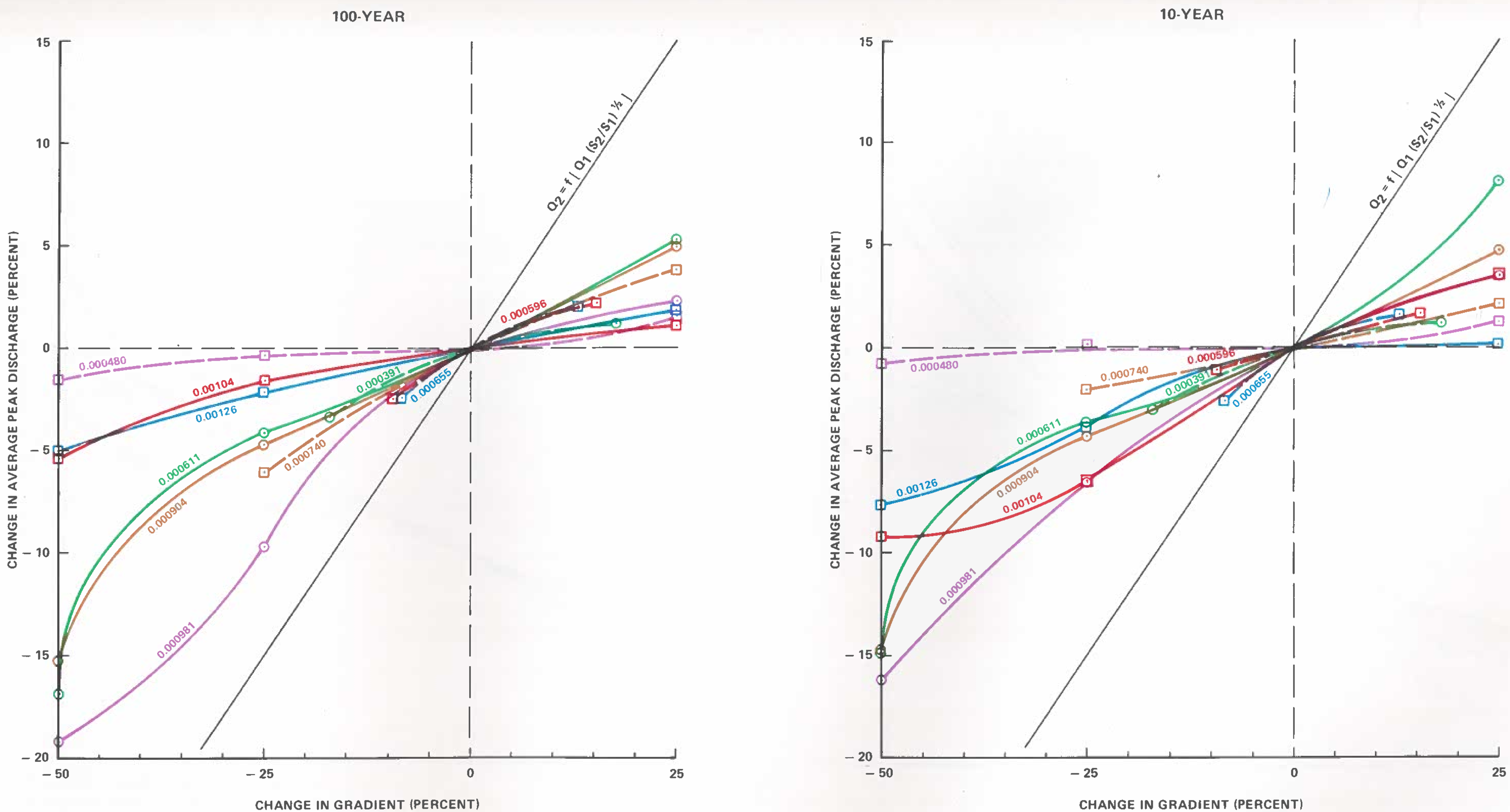
TABLE II-9 - DESCRIPTION OF SUBSIDENCE CASES SELECTED FOR ANALYSIS

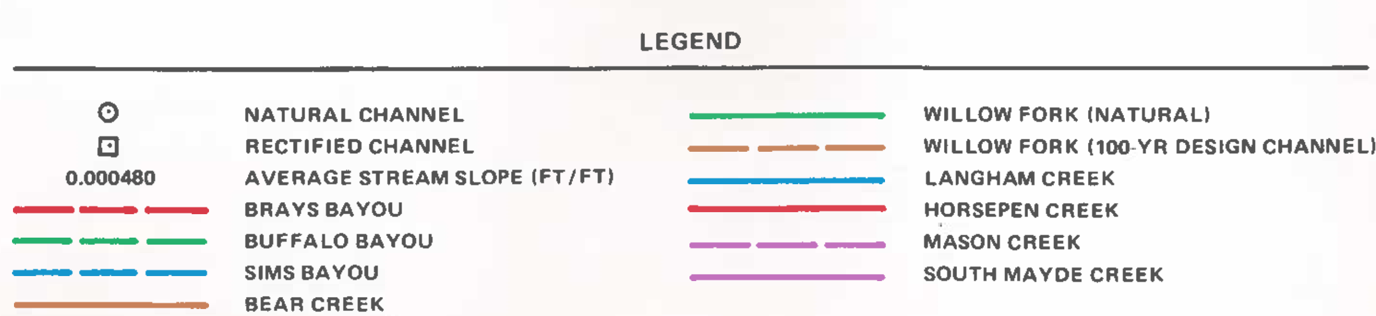
Case*	Description
1	The primary subsidence condition presented in Exhibit II-2 located such that 80 percent of the basin drainage area is upstream of the center of subsidence case.
1a	A 50 percent reduction in the magnitude of the primary subsidence condition located as indicated by Case 1.
1b	A 50 percent increase in the magnitude of the primary subsidence condition located as indicated by Case 1.
2	The primary subsidence condition presented in Exhibit II-2 located such that 50 percent of the basin drainage area is upstream of the center of subsidence case.
2a	A 50 percent reduction in the magnitude of the primary subsidence condition located as indicated by Case 2.
2b	A 50 percent increase in the magnitude of the primary subsidence condition located as described in Case 2.
3	The primary subsidence condition presented in Exhibit II-2 located such that 20 percent of the basin drainage area is upstream of the center of subsidence case.
3a	A 50 percent reduction in the magnitude of the primary subsidence condition located as indicated in Case 3. On Brays Bayou, this case closely approximates the HGCSD's Plan for subsidence extrapolated from 1973 thru 2020. As a result, the HGCSD's Plan studied on Brays Bayou is referred to as Case 3a throughout this report.
3b	A 50 percent increase in the magnitude of the primary subsidence condition located as indicated in Case 3.
4	Primary subsidence condition presented in Exhibit II-2 located upstream of the Sims Bayou drainage area.
5	This is the same subsidence condition analyzed for Case 2 of Brays Bayou but applied only to the watersheds above Addicks and Barker reservoirs.
6	Center of subsidence cone located the same as Case 5 with a gradient increase of 25 percent above base conditions.
7	Center of subsidence cone located upstream of the subject basin with a gradient reduction of 25 percent below base conditions.
8	Center of subsidence cone located upstream of the subject basin with a gradient change of 50 percent below base conditions.
9	Center of subsidence cone located at 25 percent of the length upstream from the mouth with a gradient change 20 percent flatter downstream from the center of subsidence.
10	Center of subsidence cone located at 50 percent of the length upstream from the mouth with a gradient change 25 percent flatter downstream from the center of subsidence.

*Cases 1 through 4 generally refer to Buffalo, Sims, and Brays Bayous as presented on Exhibit II-7. Cases 5 through 10 refer to the channels upstream of Addicks and Barker Reservoirs.

TABLE II-10 - SUBSIDENCE CASE MATRIX

Watershed	Cases 1	1a	1b	2	2a	2b	3	3a	3b	4	5	6	7	8	9	10
Brays Bayou	X			X	X	X	X	X	X							
Sims Bayou	X	X	X	X			X			X						
Buffalo Bayou	X			X			X	X	X							
Langham Creek											X	X	X	X		
Horsepen Creek											X	X	X	X		
Bear Creek											X	X	X	X		
South Mayde Creek											X	X	X	X	X	X
Mason Creek											X	X	X	X		
Willow Fork (Natural Channel)				X							X	X	X	X		
Willow Fork (100-year Channel)				X								X	X			

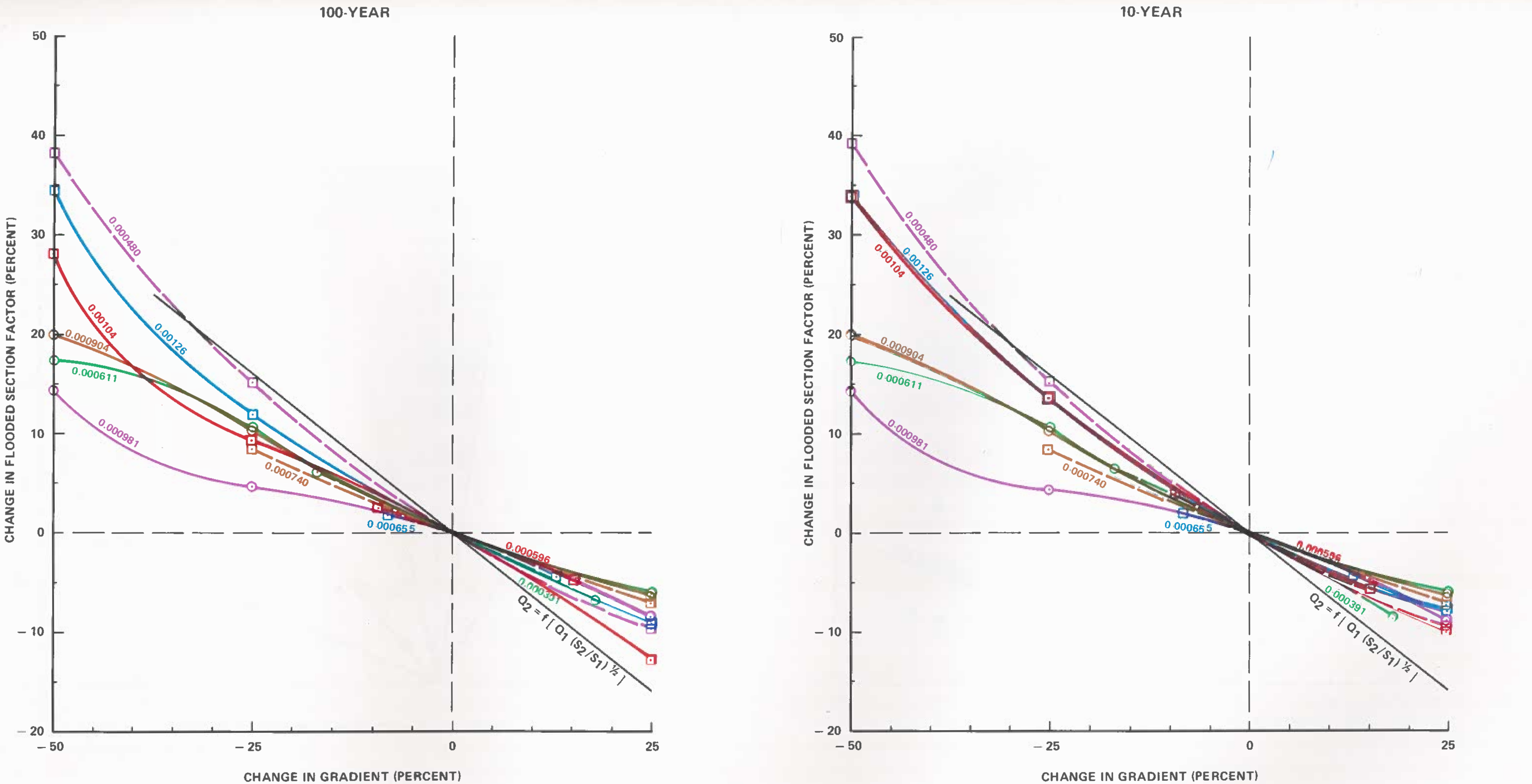


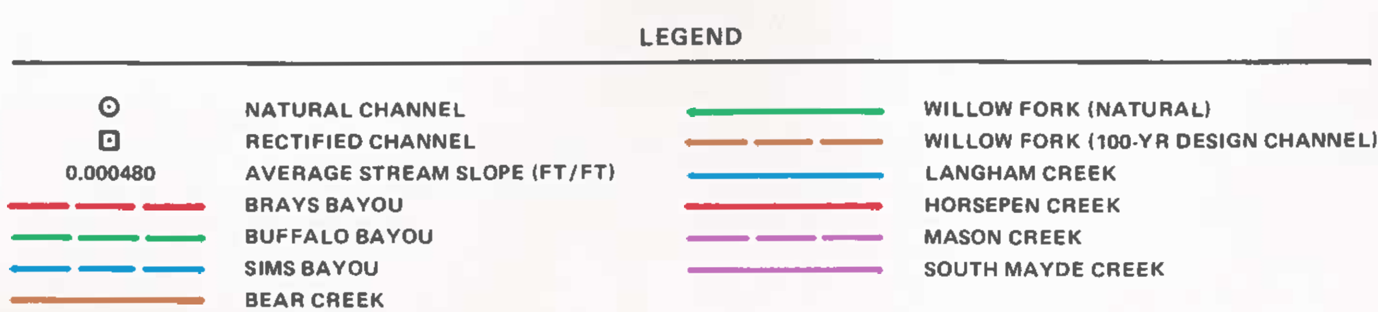


A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

PERCENT CHANGE IN AVERAGE PEAK DISCHARGE VS. PERCENT CHANGE IN GRADIENT

DECEMBER 1986 | EXHIBIT II-8



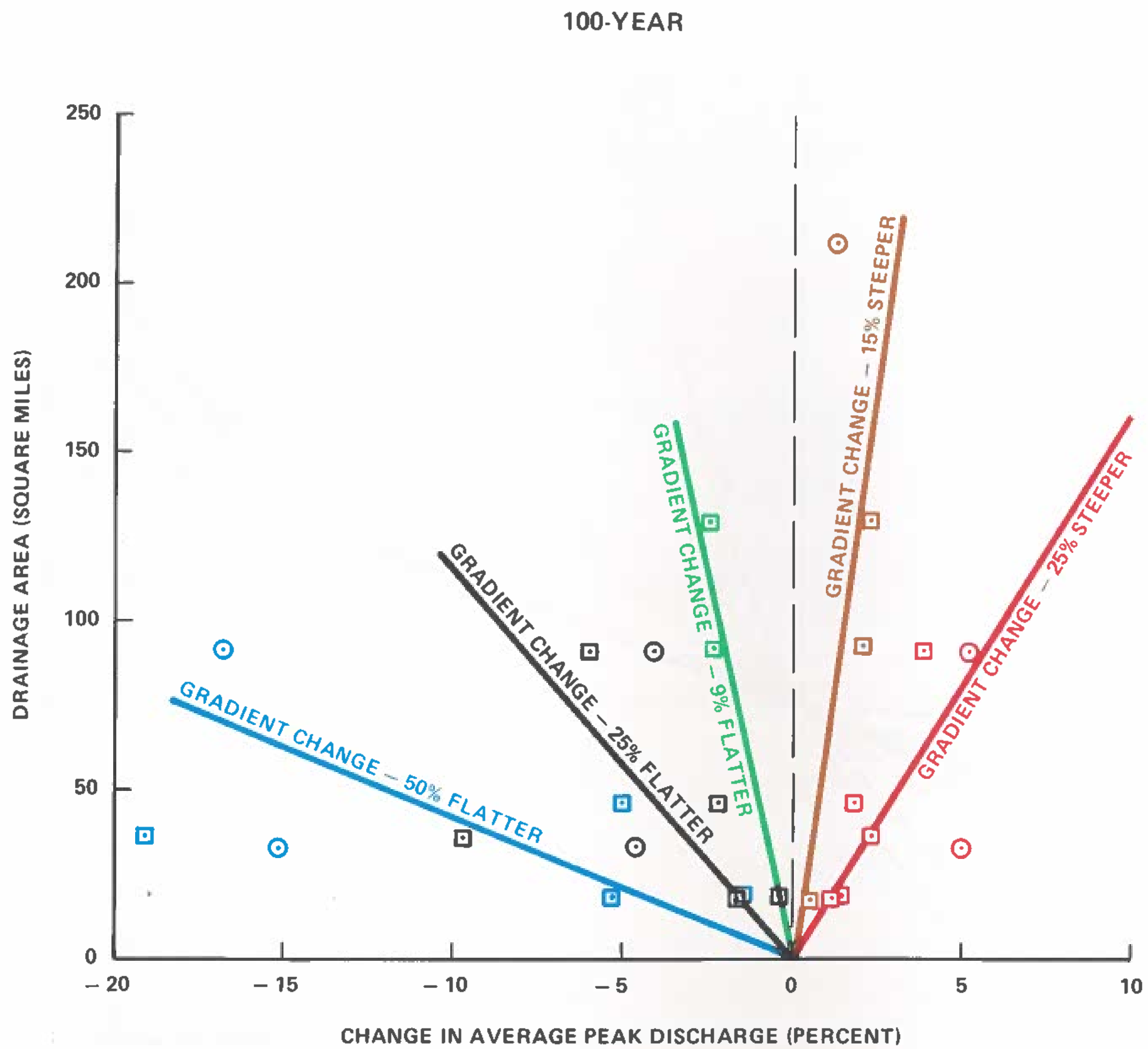


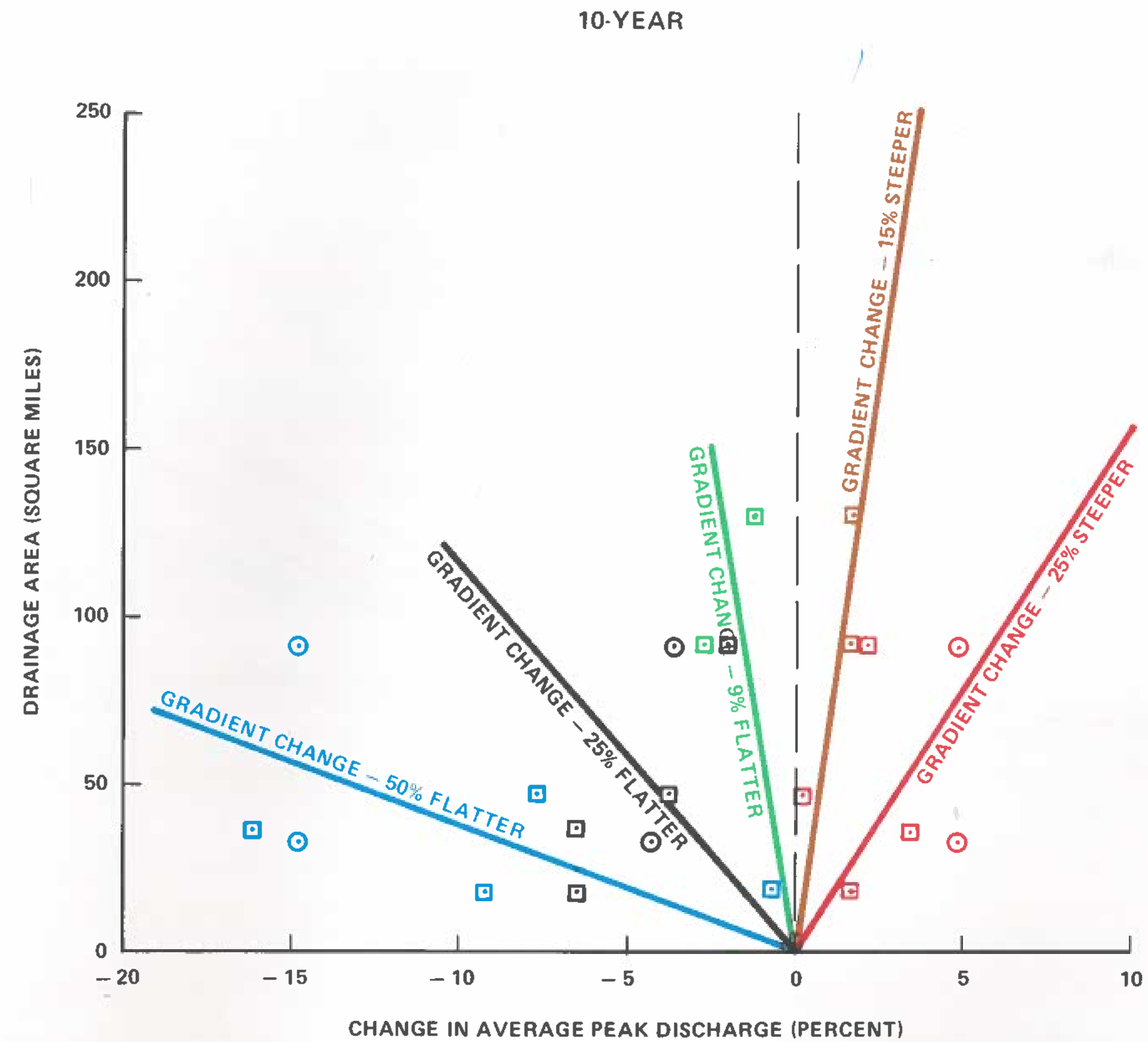
A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

PERCENT CHANGE IN HYDRAULIC CARRYING CAPACITY VS. PERCENT CHANGE IN GRADIENT

DECEMBER 1986

EXHIBIT II–9





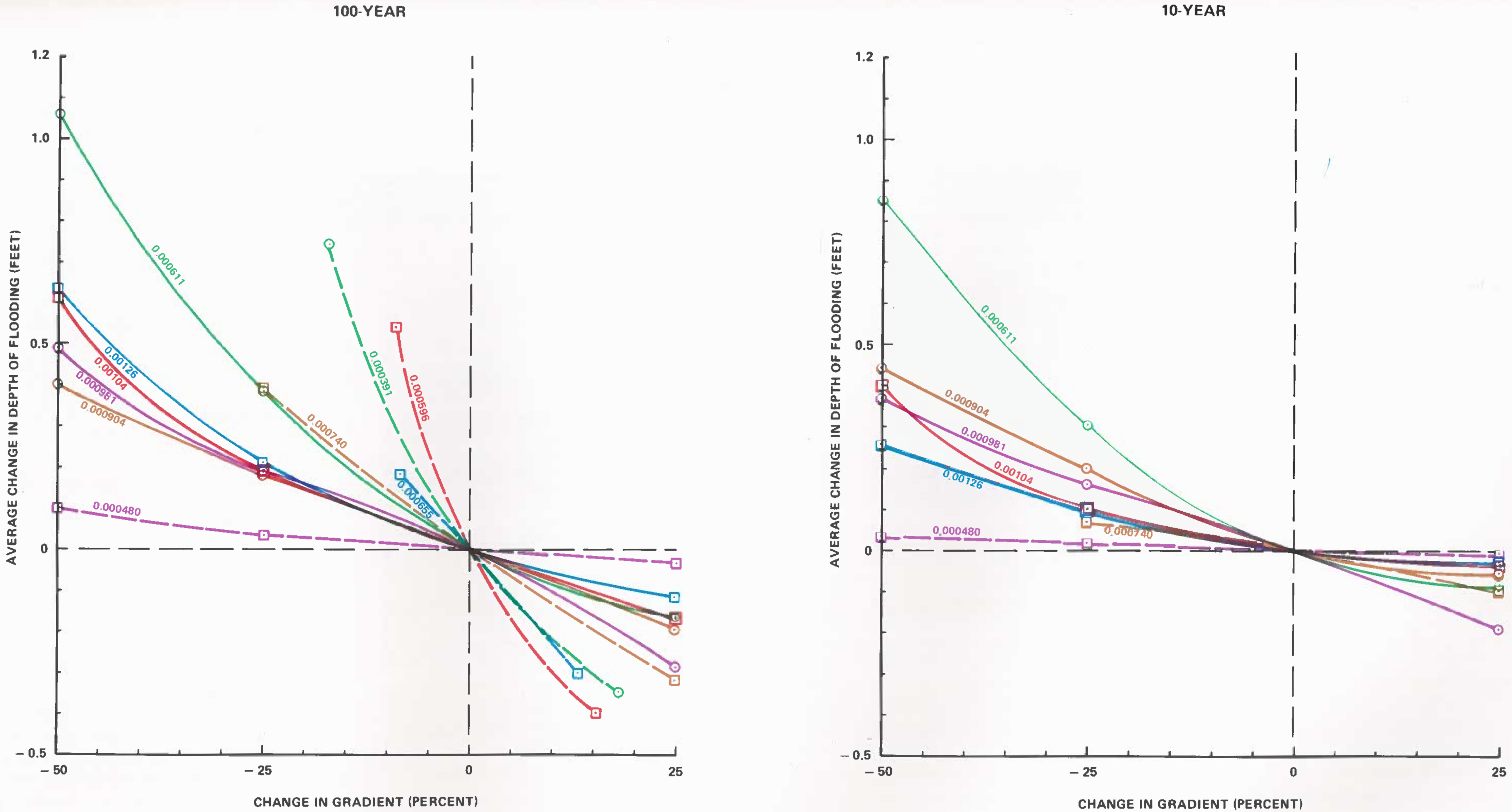
LEGEND

⊡ RECTIFIED CHANNEL
⊙ NATURAL CHANNEL

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

DRAINAGE AREA VS. PERCENT CHANGE IN AVERAGE PEAK DISCHARGE FOR CHANGES IN GRADIENT

DECEMBER 1986 | EXHIBIT II–10



LEGEND

⊙	NATURAL CHANNEL	———	WILLOW FORK (NATURAL)
⊡	RECTIFIED CHANNEL	— — —	WILLOW FORK (100-YR DESIGN CHANNEL)
0.000480	AVERAGE STREAM SLOPE (FT/FT)	———	LANGHAM CREEK
— — —	BRAYS BAYOU	———	HORSEPEN CREEK
— — —	BUFFALO BAYOU	— — —	MASON CREEK
— — —	SIMS BAYOU	———	SOUTH MAYDE CREEK
———	BEAR CREEK		

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

AVERAGE CHANGE IN DEPTH OF FLOODING VS. PERCENT CHANGE IN GRADIENT

DECEMBER 1986 | EXHIBIT II–11

TABLE II-11 - MAGNITUDE OF SUBSIDENCE ALONG CHANNEL CENTERLINE

Watershed	Subsidence Case	Downstream Limit of Study Station (ft.)	Downstream Limit of Study Subsidence (ft.)	Center of Cone Station (ft.)	Center of Cone Subsidence (ft.)	Upstream Limit of Study Station (ft.)	Upstream Limit of Study Subsidence (ft.)
Brays Bayou	1	0	-6.9	53300	-9.8	157000	-1.7
	2	0	-3.8	93000	-9.8	157000	-4.8
	2a	0	-1.9	93000	-4.9	157000	-2.4
	2b	0	-5.7	93000	-14.7	157000	-7.2
	3	0	-1.5	124600	-9.8	157000	-7.7
	3a	0	-0.8	124600	-4.9	157000	-3.9
	3b	0	-2.3	124600	-14.7	157000	-11.6
Sims Bayou	1	0	-8.6	27600	-9.7	127000	-1.6
	1a	0	-4.3	27600	-4.9	127000	-0.8
	1b	0	-12.9	27600	-14.7	127000	-2.4
	2	0	-6.2	64900	-9.8	127000	-4.1
	3	0	-4.4	103000	-9.8	127000	-7.6
	4	0	-1.3	--	--	127000	-6.6
Buffalo Bayou	1	83200	-5.5	135700	-9.8	250920	-3.9
	2	83200	-2.4	200500	-9.8	250920	-6.3
	3	83200	-1.3	237300	-9.8	250920	-8.5
	3a	83200	-0.6	237300	-4.9	250920	-4.3
	3b	83200	-1.9	237300	-14.7	250920	-12.8
Langham Creek	5	17800	-6.0	--	--	90760	-2.6
	6	17899	-14.9	--	--	90755	0.0
	7	17899	0.0	--	--	90755	-14.9
	8	17899	0.0	--	--	90755	-29.5
Horsepen Creek	5	500	-6.1	--	--	41400	-4.7
	6	528	-10.5	--	--	41346	0.0
	7	528	0.0	--	--	41346	-10.5
	8	528	0.0	--	--	41346	-21.1
Bear Creek	5	19690	-5.6	--	--	91350	-2.1
	6	19694	-16.2	--	--	91349	0.0
	7	19694	0.0	--	--	91349	-16.2
	8	19694	0.0	--	--	91349	-32.4

TABLE II-11 (Cont'd)

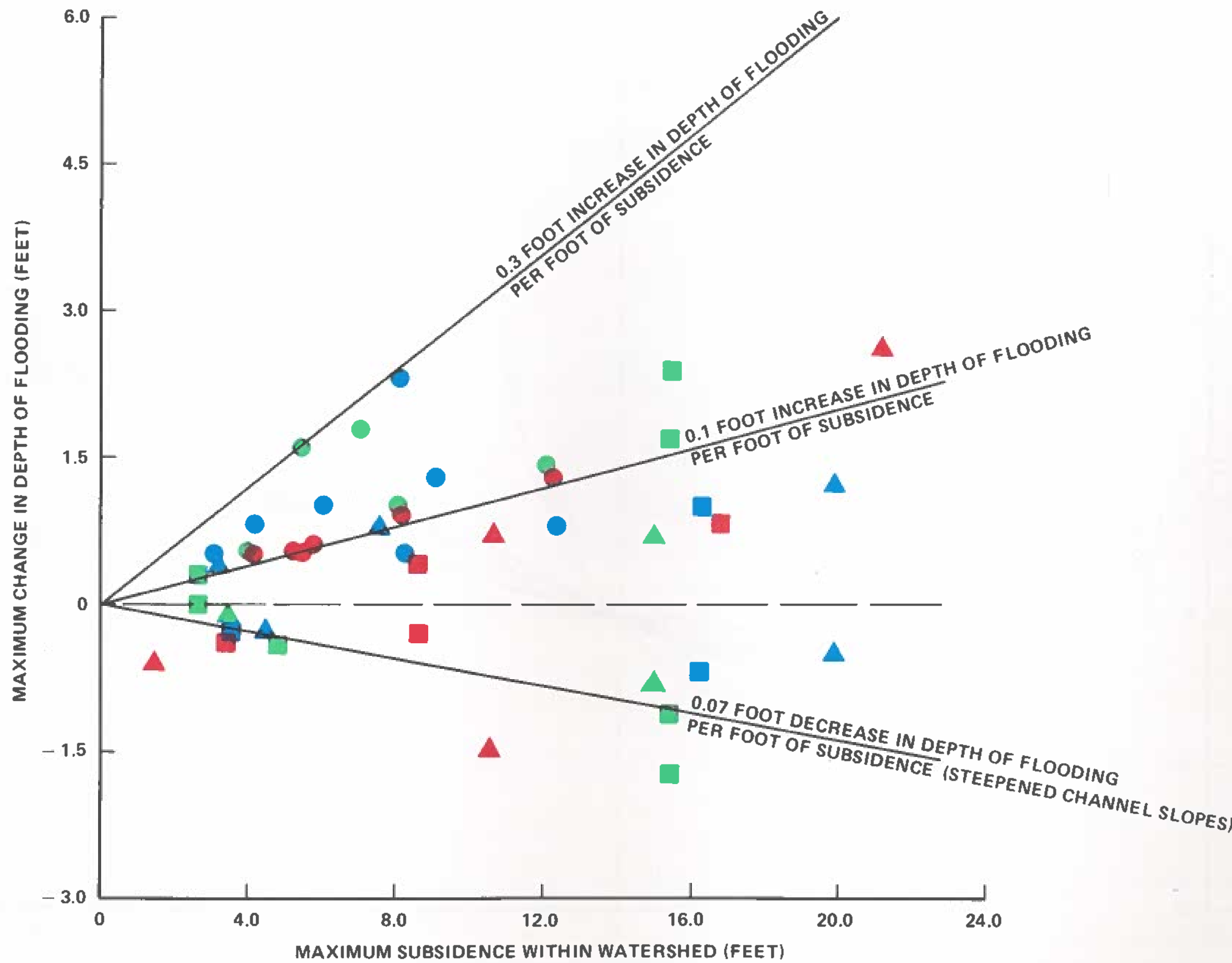
Watershed	Subsidence Case	Downstream Limit of Study Station (ft.)	Downstream Limit of Study Subsidence (ft.)	Center of Cone Station (ft.)	Center of Cone Subsidence (ft.)	Upstream Limit of Study Station (ft.)	Upstream Limit of Study Subsidence (ft.)
South Mayde Creek	5	19200	-5.4	--	--	99975	-1.0
	6	19219	-19.8	--	--	99974	0.0
	7	19219	0.0	--	--	99974	-19.8
	8	19219	0.0	--	--	99974	-39.2
	9	19219	0.0	39600	-3.1	99974	0.0
	10	19219	0.0	60000	-7.5	99974	0.0
Mason Creek	5	500	-6.2	--	--	40290	-2.8
	6	528	-8.5	--	--	40281	0.0
	7	528	0.0	--	--	40281	-8.5
	8	528	0.0	--	--	40281	-16.7
Willow Fork (Natural Channel)	2	39000	-7.6	92800	-10.0	142000	-7.4
	5	30994	-6.2	--	--	114259	-1.5
	6	30994	-15.3	--	--	114259	0.0
	7	30994	0.0	--	--	114259	-15.3
	8	30994	0.0	--	--	114259	-30.6
(100-Year Design Channel)	2	39000	-7.6	92800	-10.0	142000	-7.4
	6	30994	-15.3	--	--	114259	0.0
	7	30994	0.0	--	--	114259	-15.3

TABLE II-12 - FLOOD DEPTH CHANGE

Watershed	Subsidence Case	Maximum Subsidence (ft.)[1]	Maximum Change in 100-Year Flood Depth (ft.)[2]
Brays Bayou	1	8.1	2.3
	2	6.0	1.0
	2a	3.0	0.5
	2b	9.0	1.3
	3	8.3	0.6
	3a	4.1	0.6
	3b	12.4	0.8
Sims Bayou	1	8.1	0.9
	1a	4.1	0.5
	1b	12.2	1.3
	2	5.7	0.6
	3	5.4	0.5
	4	5.3	0.5
Buffalo Bayou	1	5.4	1.6
	2	6.9	1.8
	3	8.0	1.0
	3a	4.1	0.5
	3b	12.1	1.4
Langham Creek	5	3.4	-0.1
	6	14.9	-0.8
	7	14.9	-0.7
	8	29.5	2.1
Horsepen Creek	5	1.4	-0.6
	6	10.5	-1.5
	7	10.5	0.7
	8	21.1	2.6
Bear Creek	5	3.5	-0.3
	6	16.2	-0.7
	7	16.2	1.0
	8	32.4	2.1
South Mayde Creek	5	4.4	-0.3
	6	19.8	-0.5
	7	19.8	1.2
	8	39.2	1.3
	9	3.1	0.4
	10	7.5	0.8
Mason Creek	5	3.4	-0.4
	6	8.5	-0.3
	7	8.5	0.4
	8	16.7	0.8
Willow Fork			
(Natural Channel)	2	2.6	0.3
	5	4.7	-0.4
	6	15.3	-1.1
	7	15.3	1.7
	8	30.6	4.0
(100-Year Design Channel)	2	2.6	0.0
	6	15.3	-1.7
	7	15.3	2.4

Notes: [1] Maximum Subsidence within Limit of Study. Determined as the maximum differential from the data on Table II-11.

[2] Spikes due to anomalies produced by bridges or low banks were omitted.



LEGEND

- SIMS BAYOU
- BUFFALO BAYOU
- BRAYS BAYOU
- HORSEPEN CREEK
- LANGHAM CREEK
- SOUTH MAYDE CREEK
- BEAR CREEK
- MASON CREEK
- WILLOW FORK

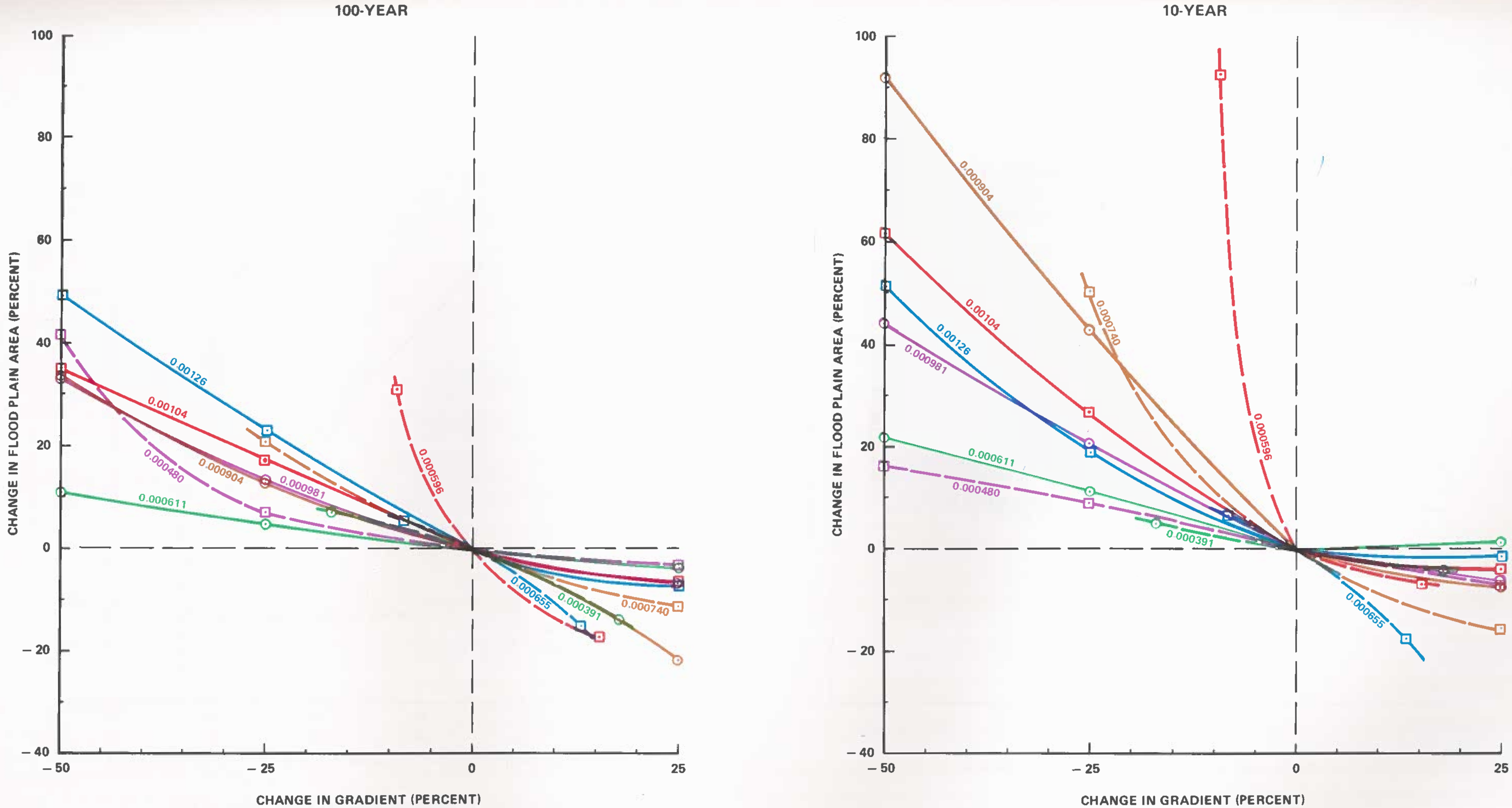
NOTE:
1. THE SUBSIDENCE VALUES WERE DETERMINED AS THE MAXIMUM DIFFERENTIAL WITHIN THE LIMITS OF STUDY FOR EACH WATERSHED. DECREASES IN DEPTH OF FLOODING RESULTED FROM THE SIMPLIFIED CASES OF STEEPENING CHANNEL SLOPE.
2. REFERENCE TABLE II-12 FOR DEFINITION OF PLOTTING POINTS BY SUBSIDENCE CASE.

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

MAXIMUM CHANGE IN DEPTH OF FLOODING VS. SUBSIDENCE (100–YEAR EVENT)

DECEMBER 1986

EXHIBIT II–12



A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

PERCENT CHANGE IN FLOOD PLAIN AREA VS. PERCENT CHANGE IN GRADIENT

DECEMBER 1986

EXHIBIT II–13

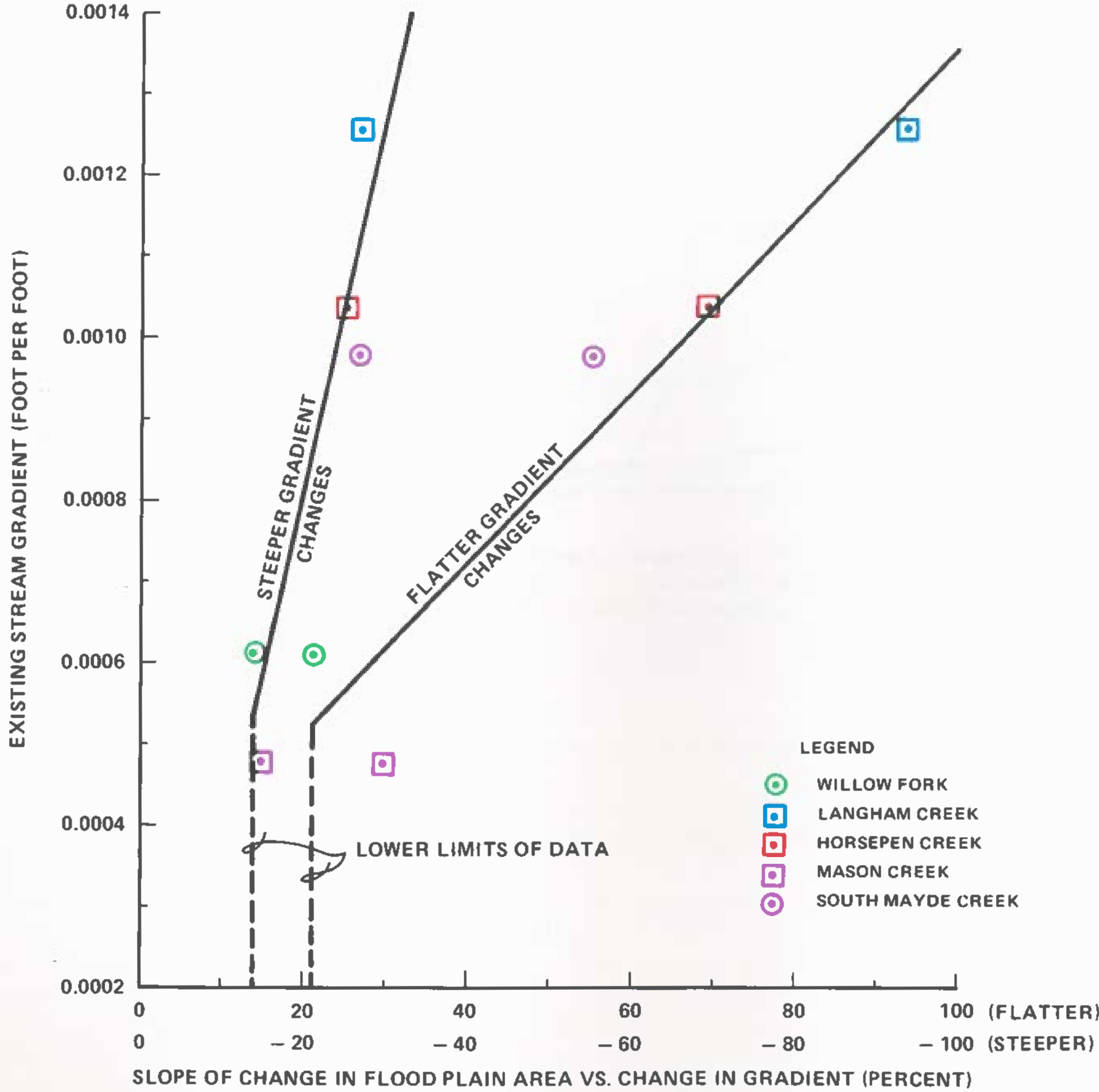


FIGURE 1
FOR SELECTION OF CHANGE IN FLOOD PLAIN AREA
VS. CHANGE IN GRADIENT RELATIONSHIP
(USED WITH EXHIBIT II-13)

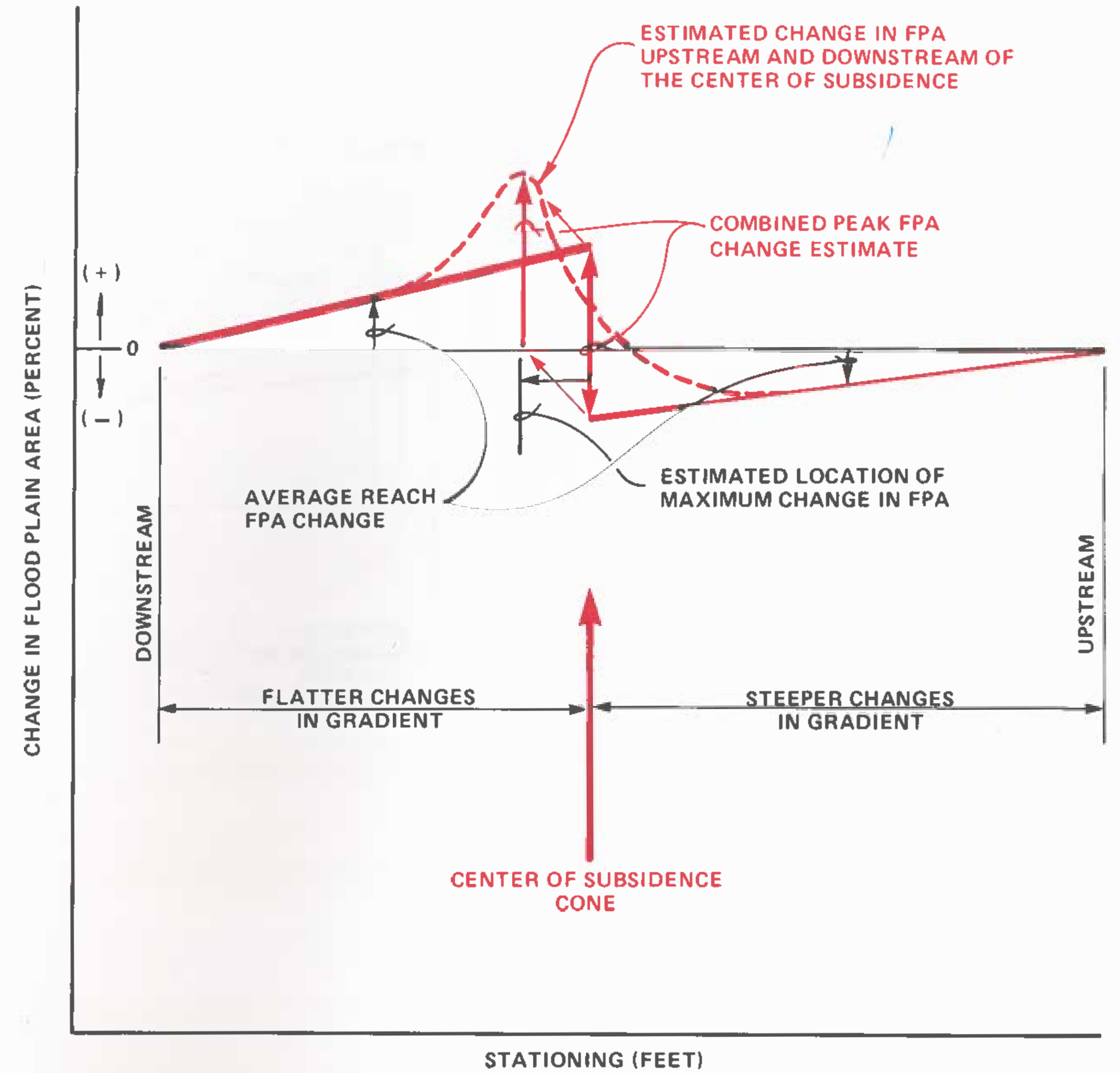


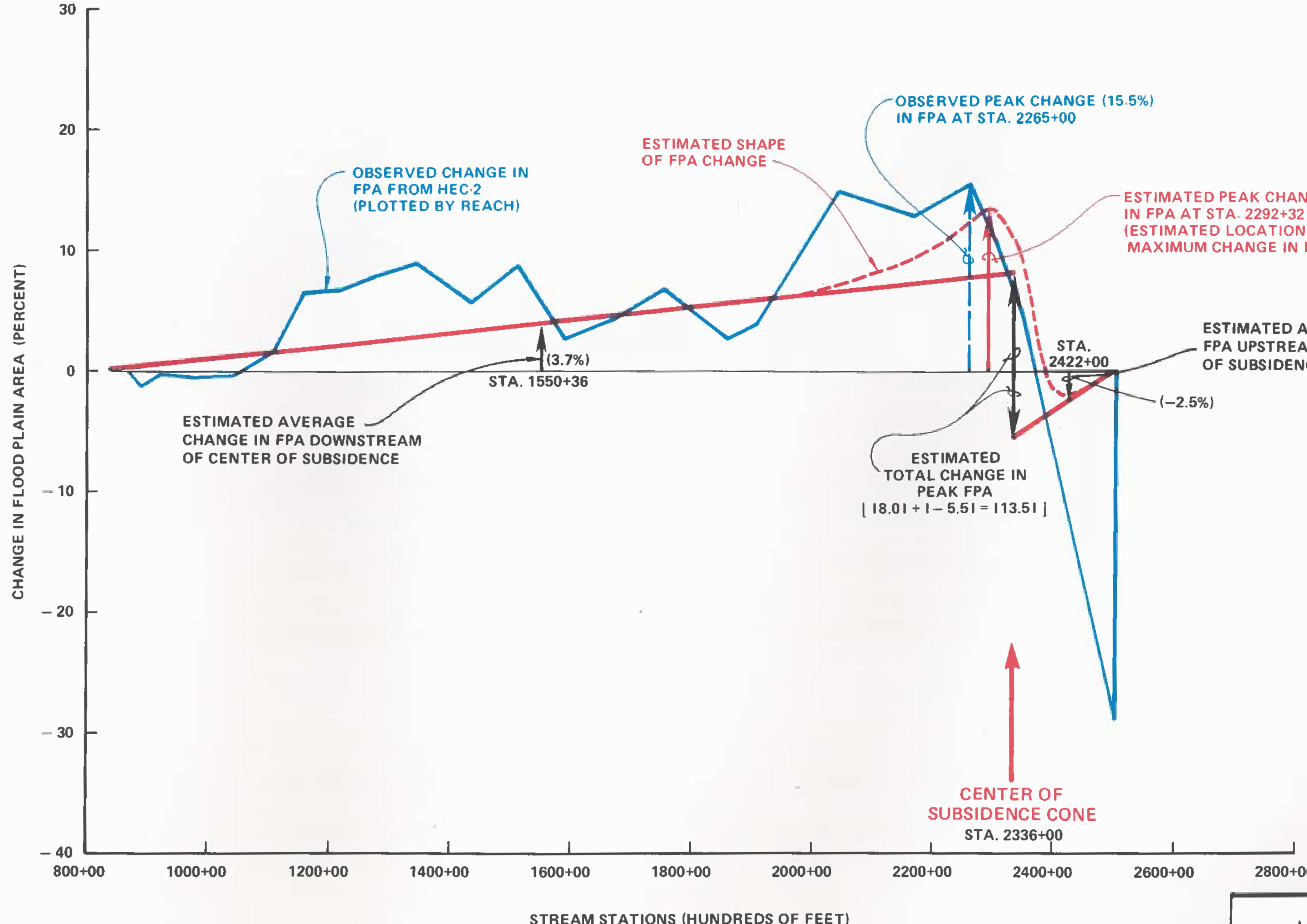
FIGURE 2
FOR ESTIMATION OF MAXIMUM CHANGE IN FPA

NOTE: DATA DEVELOPED FROM THE ANALYSIS CONDUCTED ON THE UPSTREAM WATERSHEDS FOR CHANGES IN THE 100-YEAR FPA.

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

PROCEDURE FOR ESTIMATION OF MAXIMUM CHANGE IN FLOOD PLAIN AREA (FPA)

DECEMBER 1986 | EXHIBIT II–14



A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

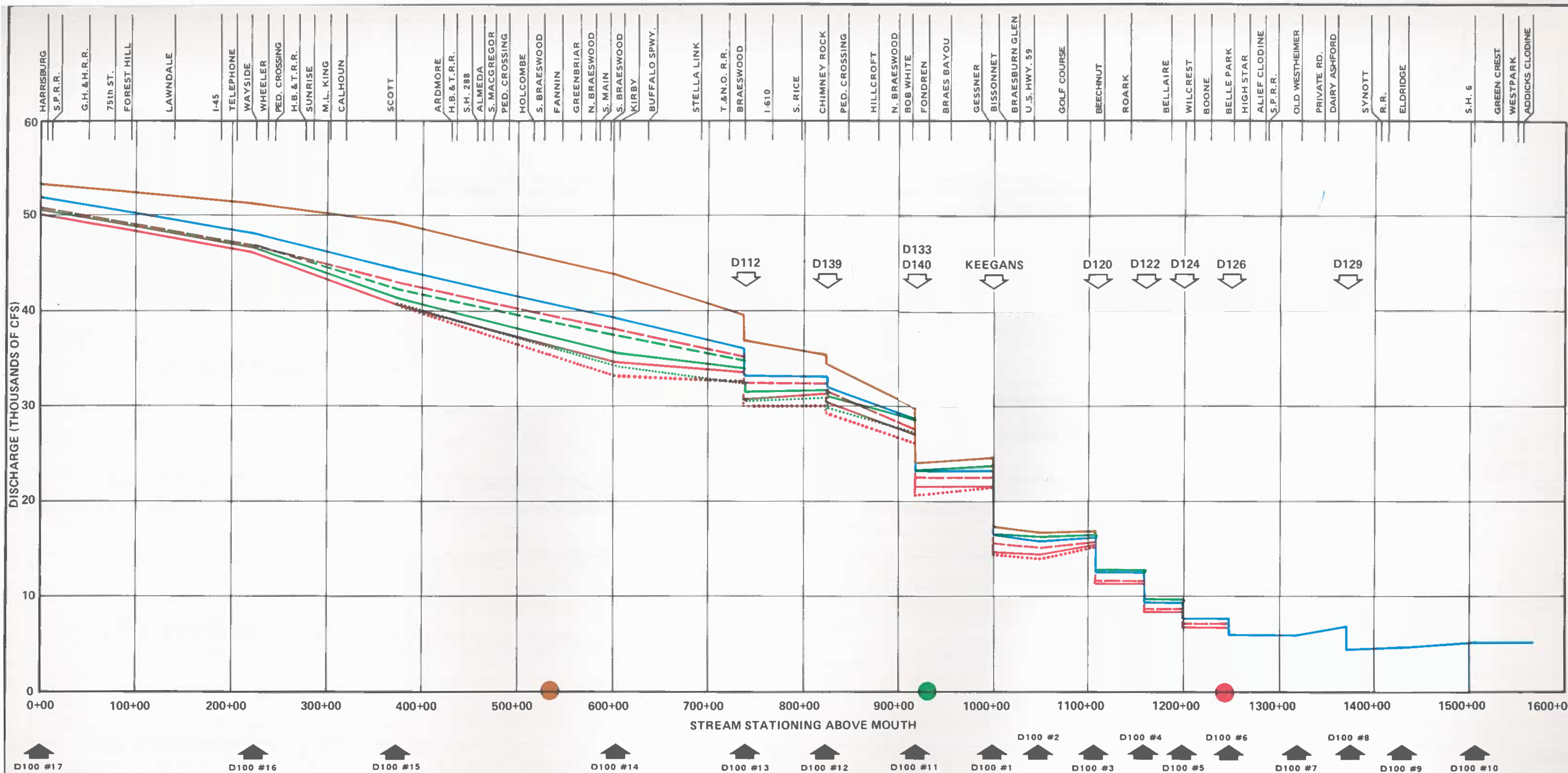
PREDICTIVE PROCEDURE SIMULATION – BUFFALO BAYOU CASE 3

DECEMBER 1986

EXHIBIT II–15

TABLE II-13 - RESULTS FROM PREDICTIVE PROCEDURE FOR ESTIMATING CHANGE IN FLOOD PLAIN AREA (FPA)

Watershed	Case	Magnitude of 100-Year Max. FPA Change (Percent)		Location of 100-Year Max. FPA Change (Stationing)	
		Observed	Predicted	Observed	Predicted
Brays Bayou	1	162.0	10.5	45000	61240
	2	95.0	11.8	90000	84650
	3	95.0	12.0	120500	117610
Sims Bayou	1	12.6	11.0	36500	31410
	2	15.0	12.2	65600	60990
Buffalo Bayou	1	19.0	14.1	159000	139390
	2	14.5	13.0	186500	191270
	3	15.5	13.5	226500	229232
South Mayde Creek	9	28.0	29.0	38500	35220
	10	48.0	42.0	44000	51450
Willow Fork					
(Natural Channel)	2	11.0	13.2	74180	78810
(100-Year Design Channel)	2	18.5	9.0	83420	81050



LEGEND

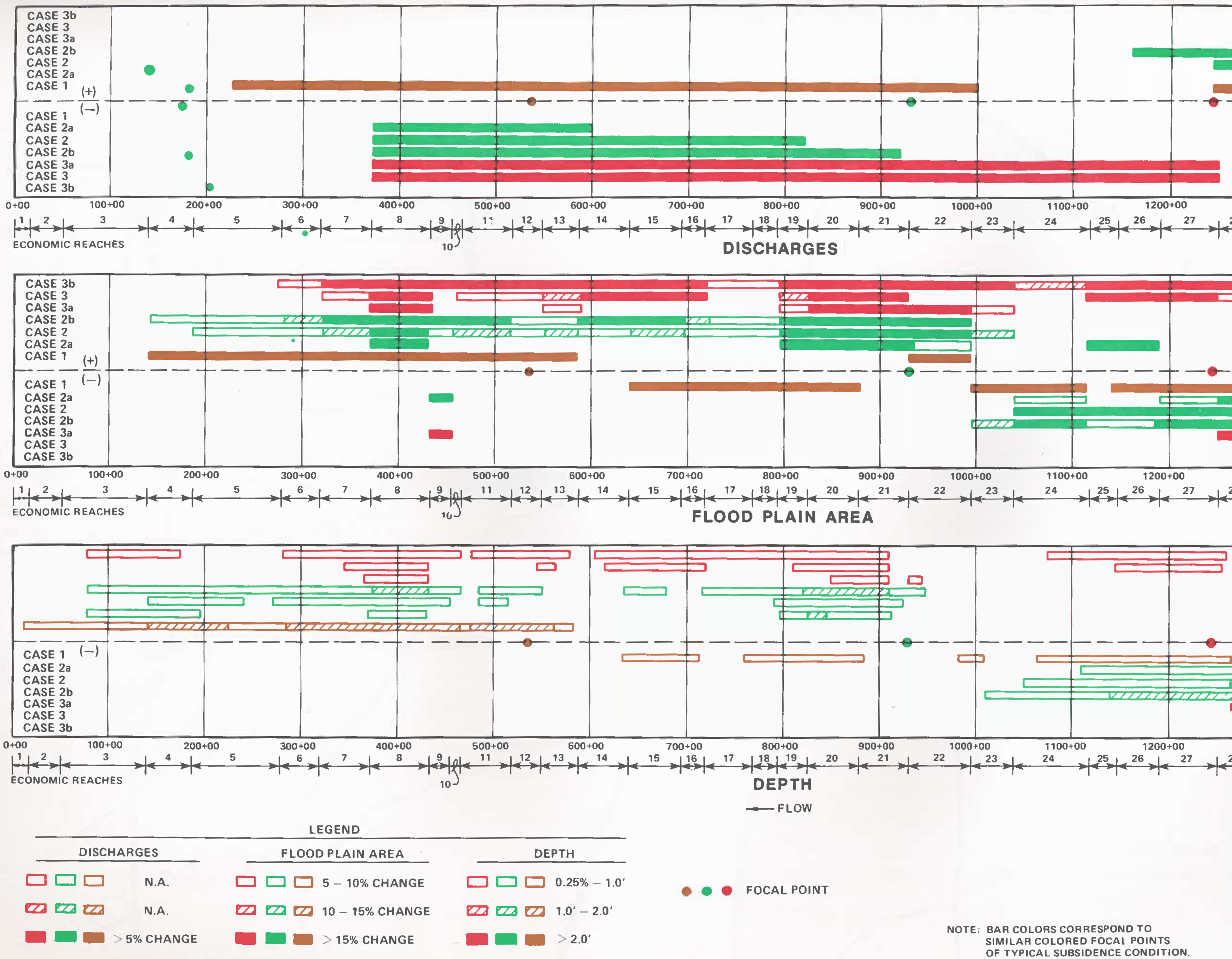
SUBSIDENCE FOCAL POINT CASE 1	BASE CONDITION	CASE 2b
SUBSIDENCE FOCAL POINT CASE 2, 2a, 2b	CASE 1	CASE 3
SUBSIDENCE FOCAL POINT CASE 3, 3a, 3b	CASE 2	CASE 3a
HEC-1 ANALYSIS POINT	CASE 2a	CASE 3b
LOCATION OF TRIBUTARY CHANNELS		

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

BRAYS BAYOU 100-YEAR STATION-DISCHARGE CURVES

DECEMBER 1986

EXHIBIT II-16

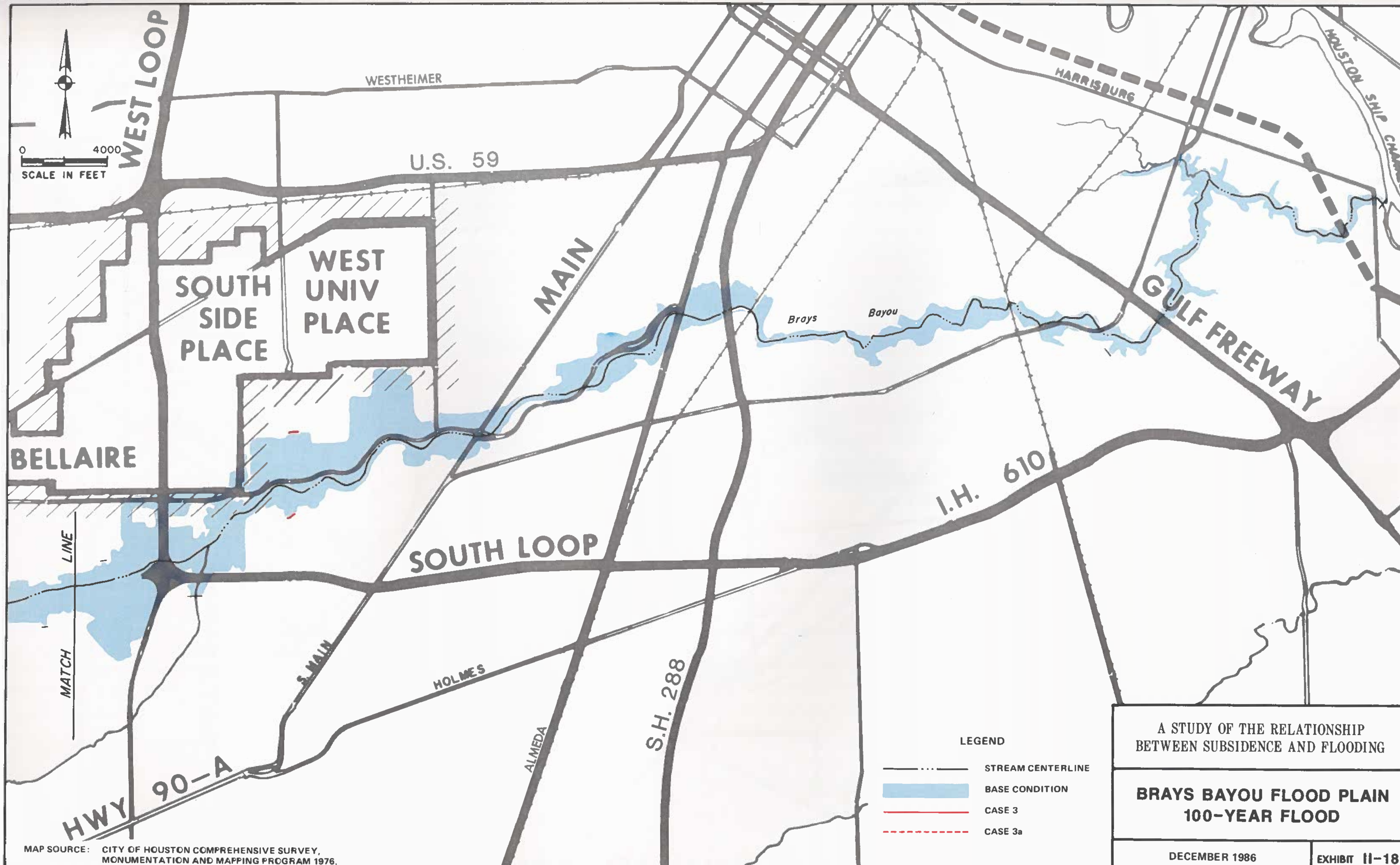


A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

BRAYS BAYOU IMPACTS OF SUBSIDENCE

DECEMBER 1986

EXHIBIT II-17

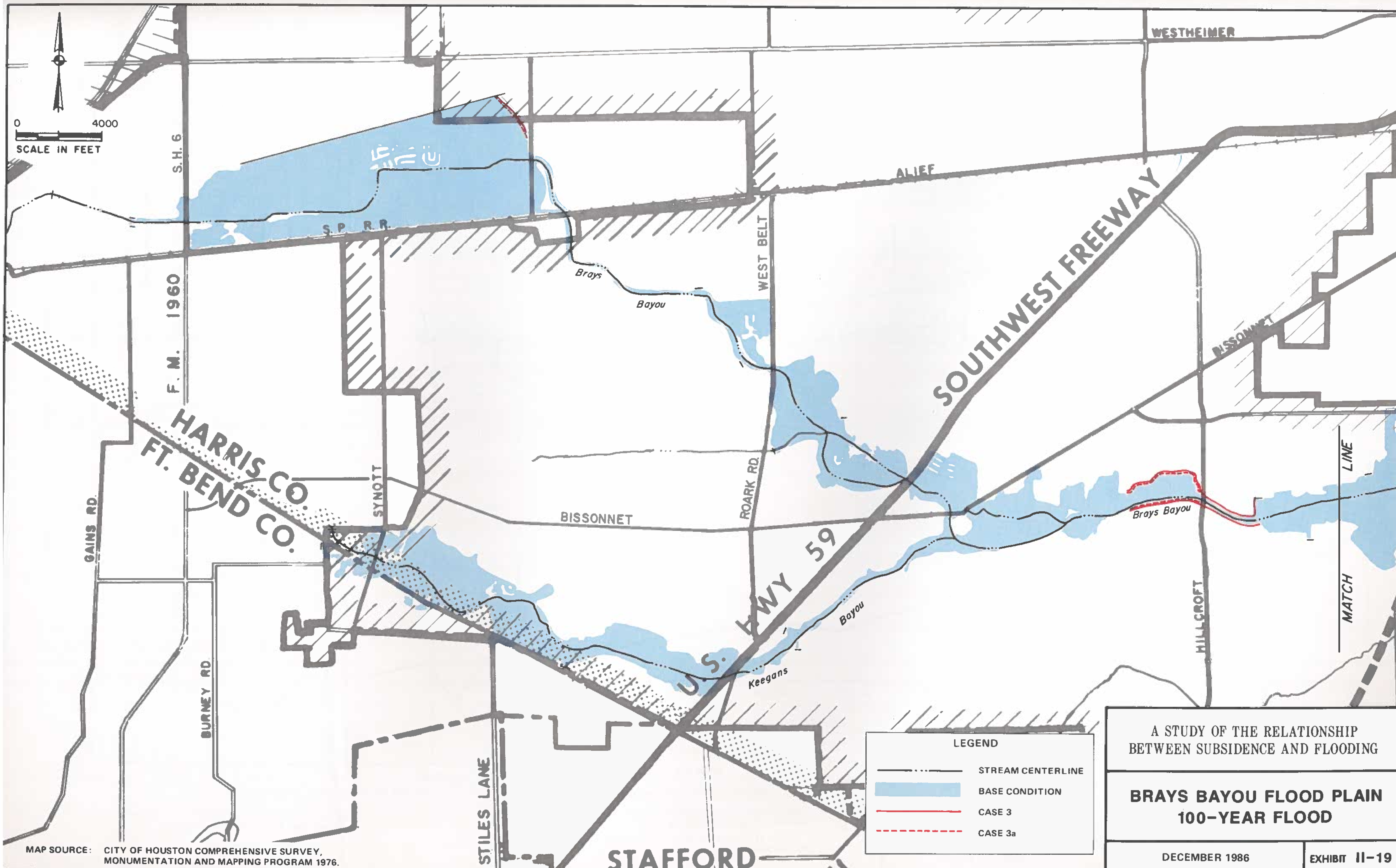


A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

BRAYS BAYOU FLOOD PLAIN 100-YEAR FLOOD

DECEMBER 1986

EXHIBIT II-18

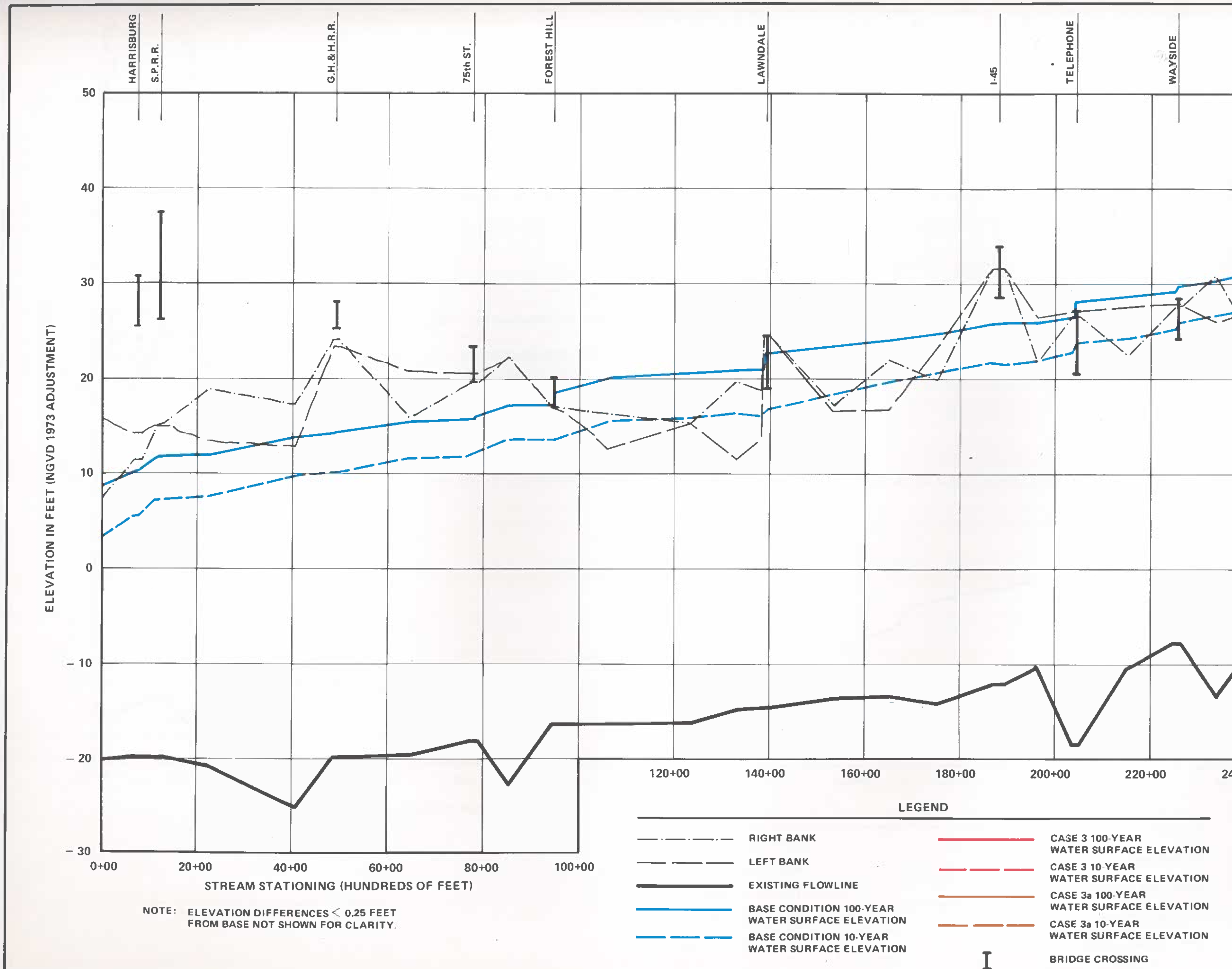


A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

BRAYS BAYOU FLOOD PLAIN 100-YEAR FLOOD

DECEMBER 1986

EXHIBIT II-19

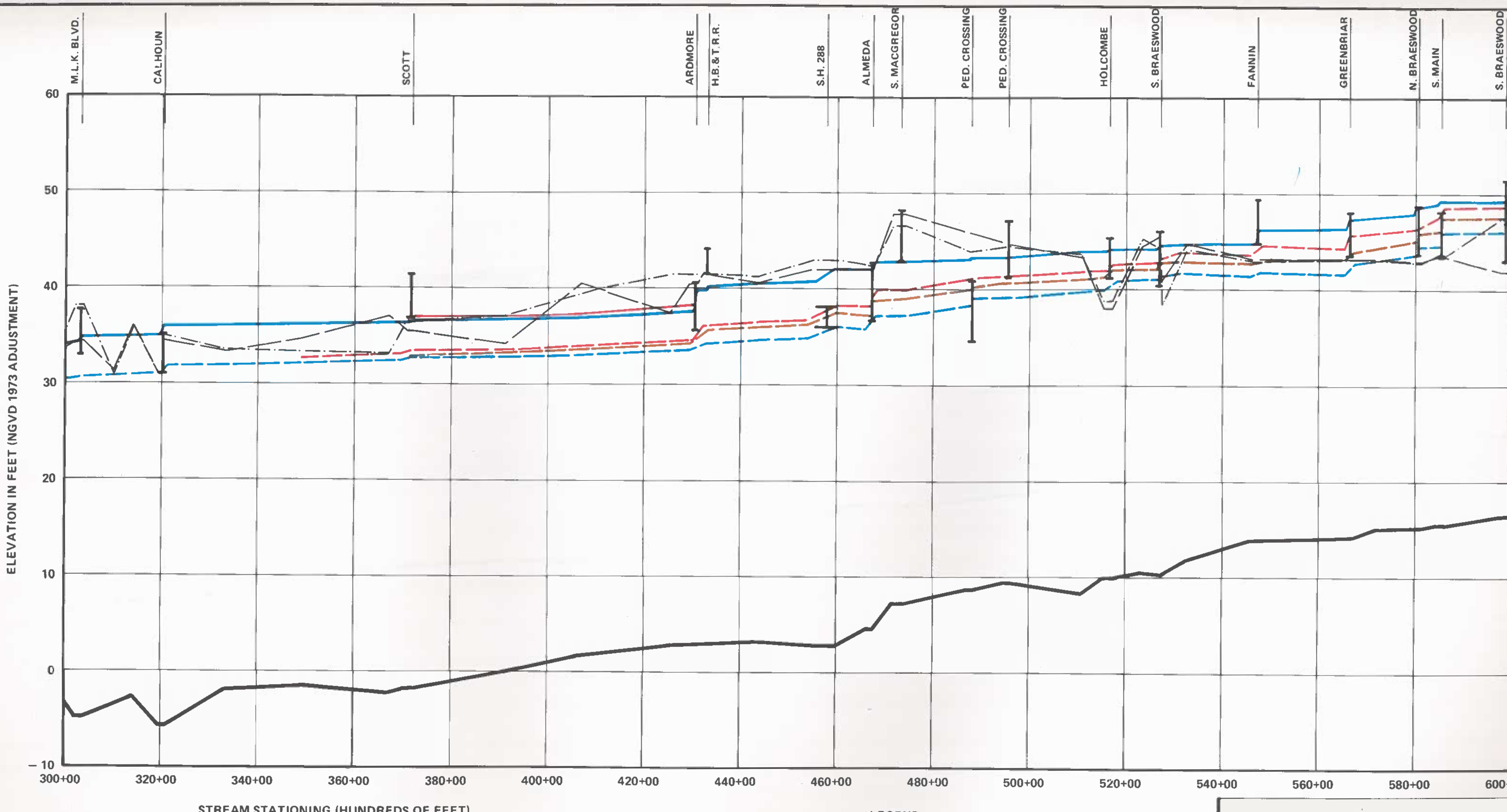


A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

BRAYS BAYOU STREAM PROFILE STA. 0+00 TO STA. 300+00

DECEMBER 1986

EXHIBIT II-20



LEGEND

RIGHT BANK
LEFT BANK
EXISTING FLOWLINE
BASE CONDITION 100-YEAR WATER SURFACE ELEVATION
BASE CONDITION 10-YEAR WATER SURFACE ELEVATION
CASE 3 100-YEAR WATER SURFACE ELEVATION
CASE 3 10-YEAR WATER SURFACE ELEVATION
CASF 3a 100-YEAR WATER SURFACE ELEVATION
CASE 3a 10-YEAR WATER SURFACE ELEVATION
I BRIDGE CROSSING

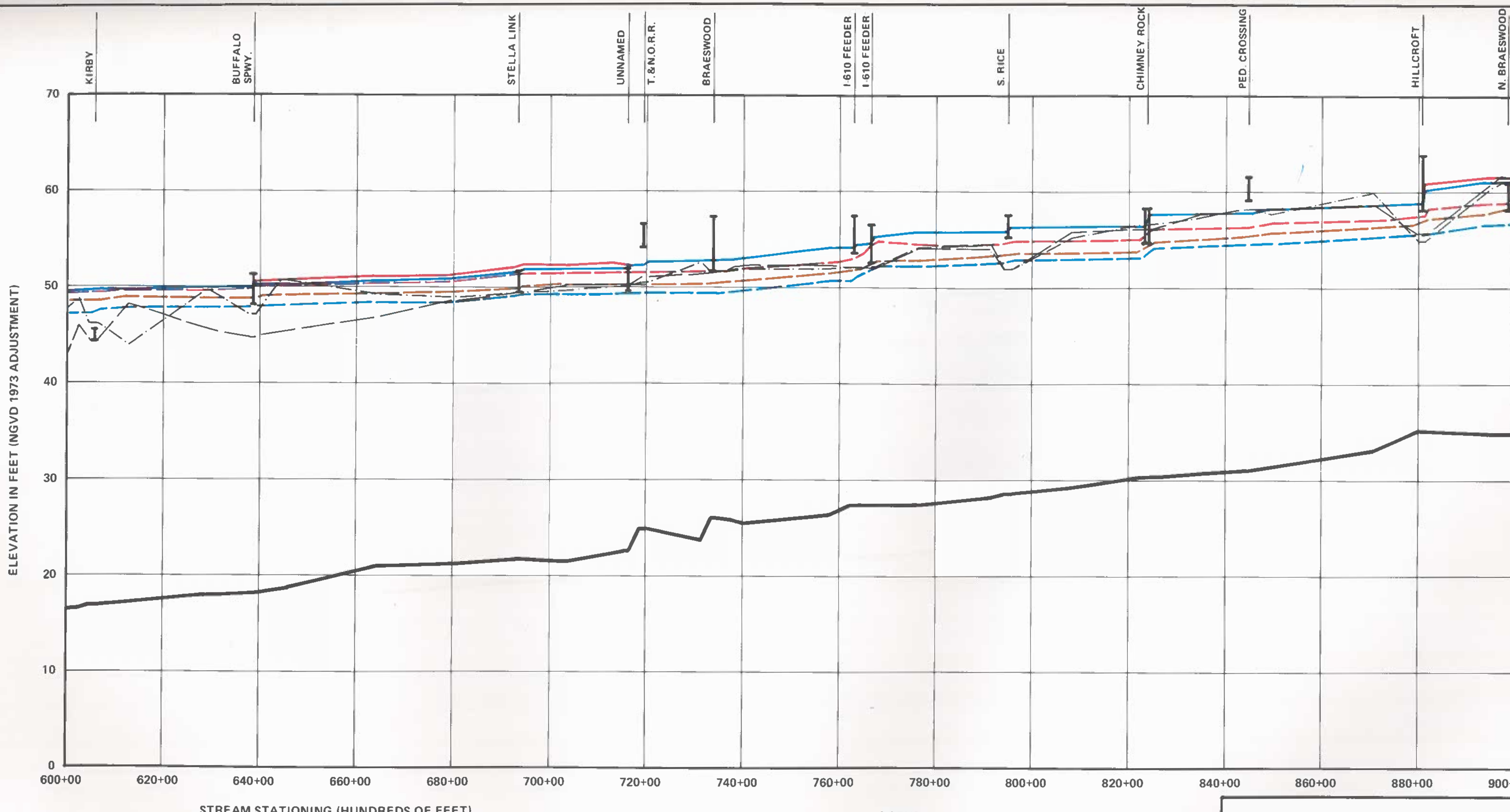
NOTE: ELEVATION DIFFERENCES < 0.25 FEET FROM BASE NOT SHOWN FOR CLARITY.

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

BRAYS BAYOU
STREAM PROFILE
STA. 300+00 TO STA. 600+00

DECEMBER 1986

EXHIBIT II–21



NOTE: ELEVATION DIFFERENCES < 0.25 FEET FROM BASE NOT SHOWN FOR CLARITY.

LEGEND

RIGHT BANK

LEFT BANK

EXISTING FLOWLINE

BASE CONDITION 100-YEAR WATER SURFACE ELEVATION

BASE CONDITION 10-YEAR WATER SURFACE ELEVATION

CASE 3 100-YEAR WATER SURFACE ELEVATION

CASE 3 10-YEAR WATER SURFACE ELEVATION

CASE 3a 100-YEAR WATER SURFACE ELEVATION

CASE 3a 10-YEAR WATER SURFACE ELEVATION

BRIDGE CROSSING

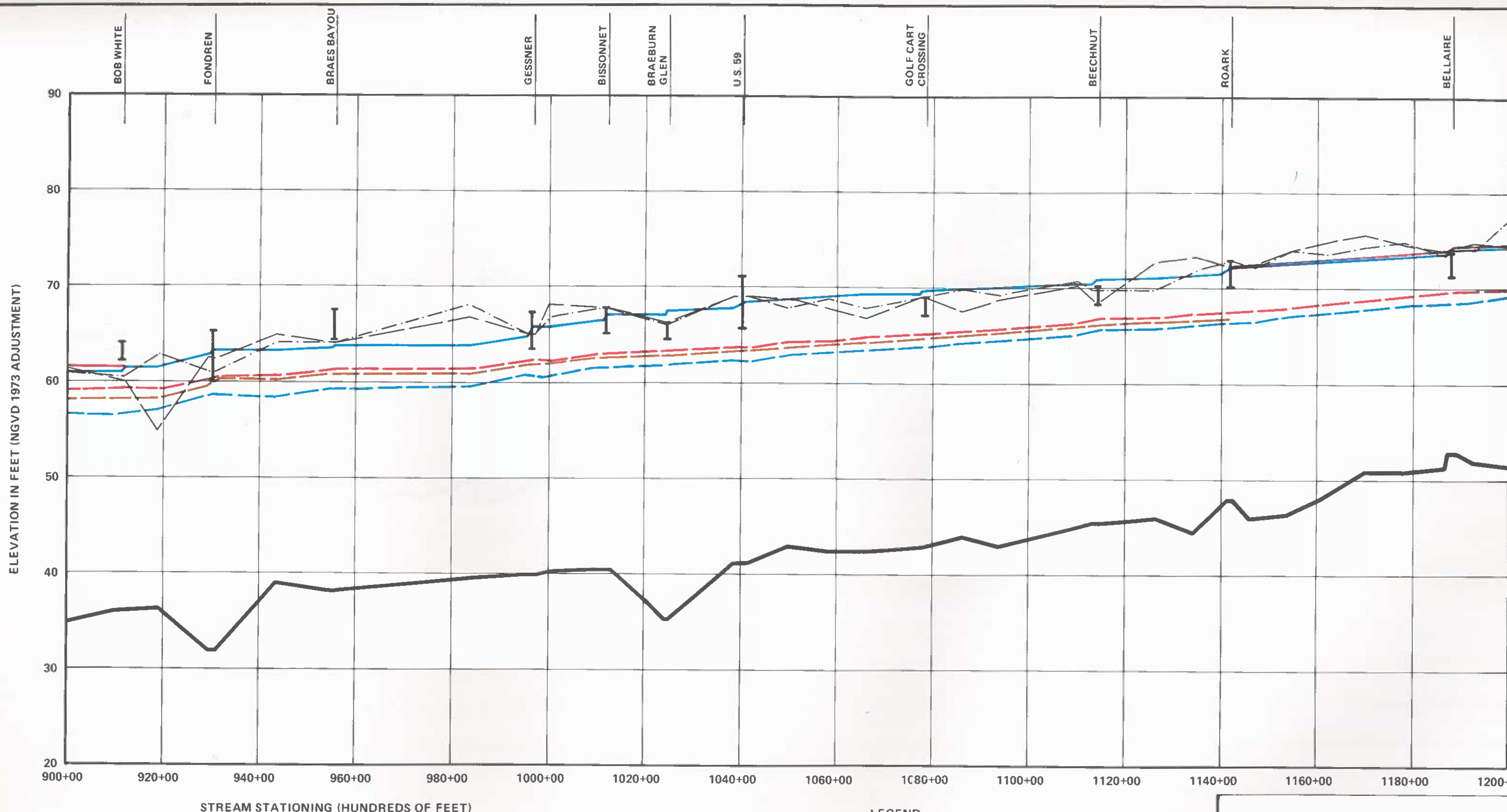
A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

BRAYS BAYOU STREAM PROFILE

STA. 600+00 TO STA. 900+00

DECEMBER 1986

EXHIBIT II-22



LEGEND

- RIGHT BANK
- LEFT BANK
- EXISTING FLOWLINE
- BASE CONDITION 100 YEAR WATER SURFACE ELEVATION
- BASE CONDITION 10 YEAR WATER SURFACE ELEVATION
- CASE 3 100-YEAR WATER SURFACE ELEVATION
- CASE 3 10-YEAR WATER SURFACE ELEVATION
- CASE 3a 100-YEAR WATER SURFACE ELEVATION
- CASE 3a 10-YEAR WATER SURFACE ELEVATION
- BRIDGE CROSSING

NOTE: ELEVATION DIFFERENCES < 0.25 FEET FROM BASE NOT SHOWN FOR CLARITY.

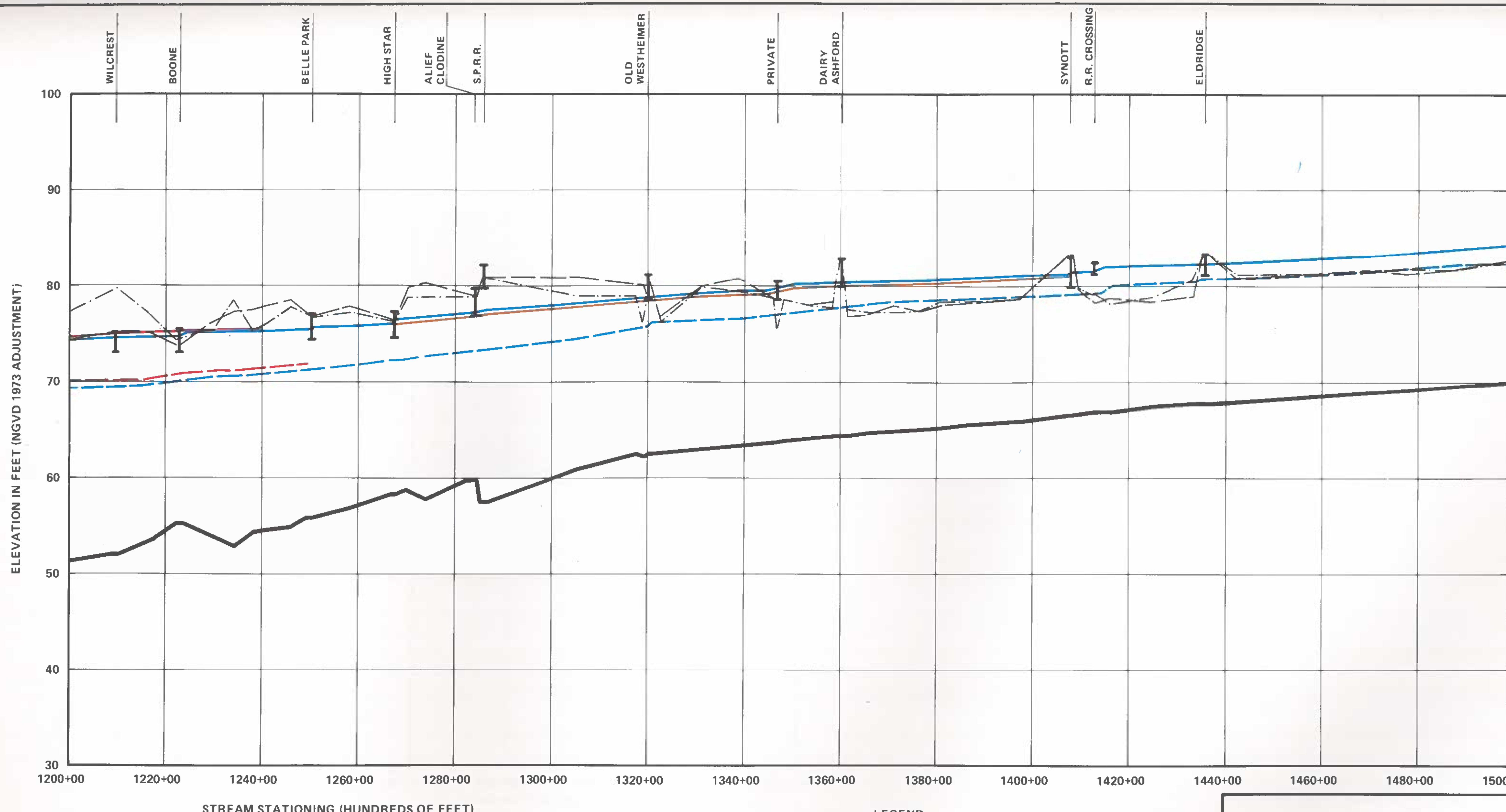
A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

BRAYS BAYOU STREAM PROFILE

STA. 900+00 TO STA. 1200+00

DECEMBER 1986

EXHIBIT II-23



NOTE: ELEVATION DIFFERENCES < 0.25 FEET FROM BASE NOT SHOWN FOR CLARITY.

LEGEND

- RIGHT BANK
- LEFT BANK
- EXISTING FLOWLINE
- BASE CONDITION 100-YEAR WATER SURFACE ELEVATION
- BASE CONDITION 10-YEAR WATER SURFACE ELEVATION
- CASE 3 100-YEAR WATER SURFACE ELEVATION
- CASE 3 10-YEAR WATER SURFACE ELEVATION
- CASE 3a 100-YEAR WATER SURFACE ELEVATION
- CASE 3a 10-YEAR WATER SURFACE ELEVATION
- I BRIDGE CROSSING

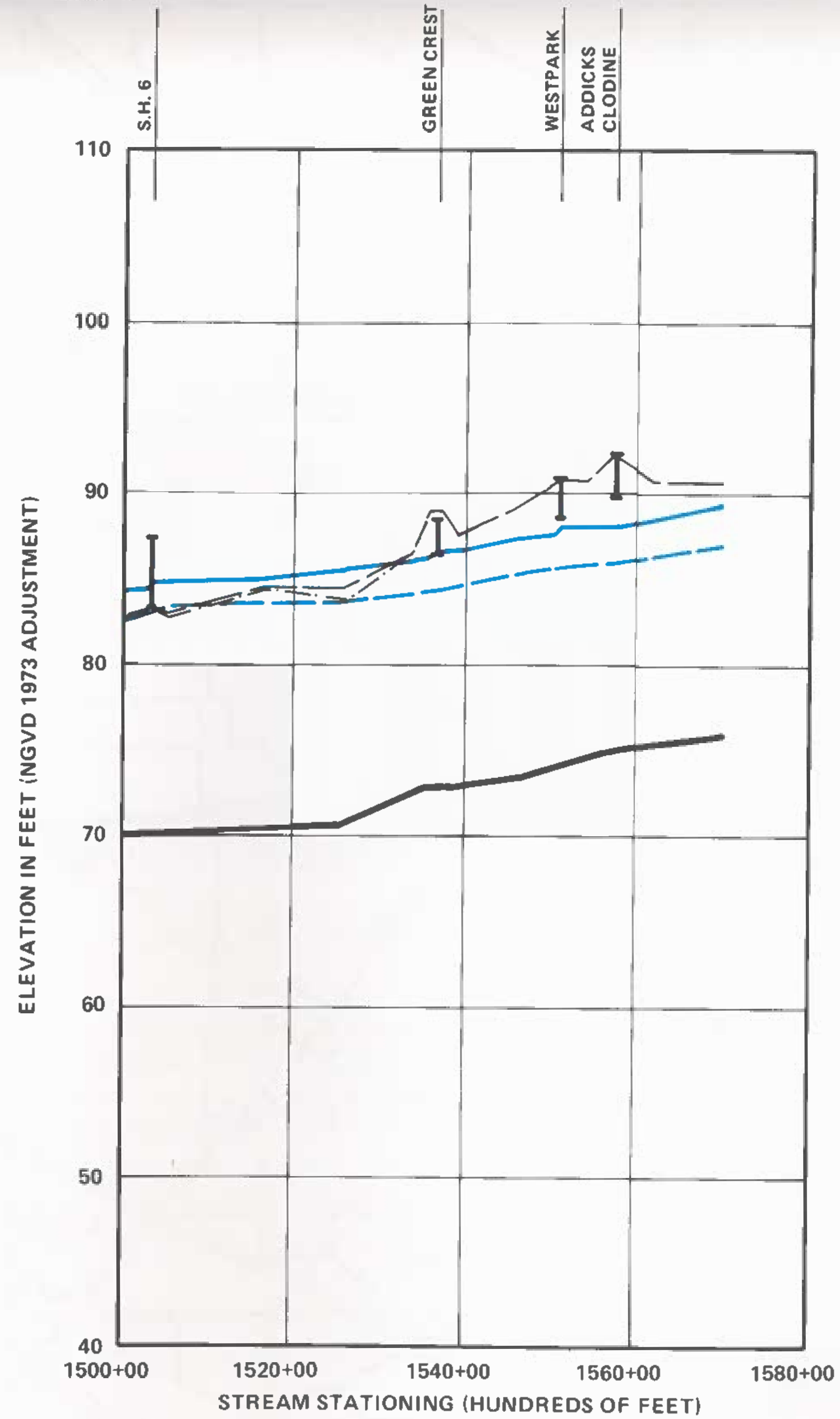
A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

BRAYS BAYOU STREAM PROFILE

STA. 1200+00 TO STA. 1500+00

DECEMBER 1986

EXHIBIT II-24



LEGEND

RIGHT BANK	CASE 3 100-YEAR WATER SURFACE ELEVATION
LEFT BANK	CASE 3 10-YEAR WATER SURFACE ELEVATION
EXISTING FLOWLINE	CASE 3a 100-YEAR WATER SURFACE ELEVATION
BASE CONDITION 100 YEAR WATER SURFACE ELEVATION	CASE 3a 10-YEAR WATER SURFACE ELEVATION
BASE CONDITION 10 YEAR WATER SURFACE ELEVATION	BRIDGE CROSSING

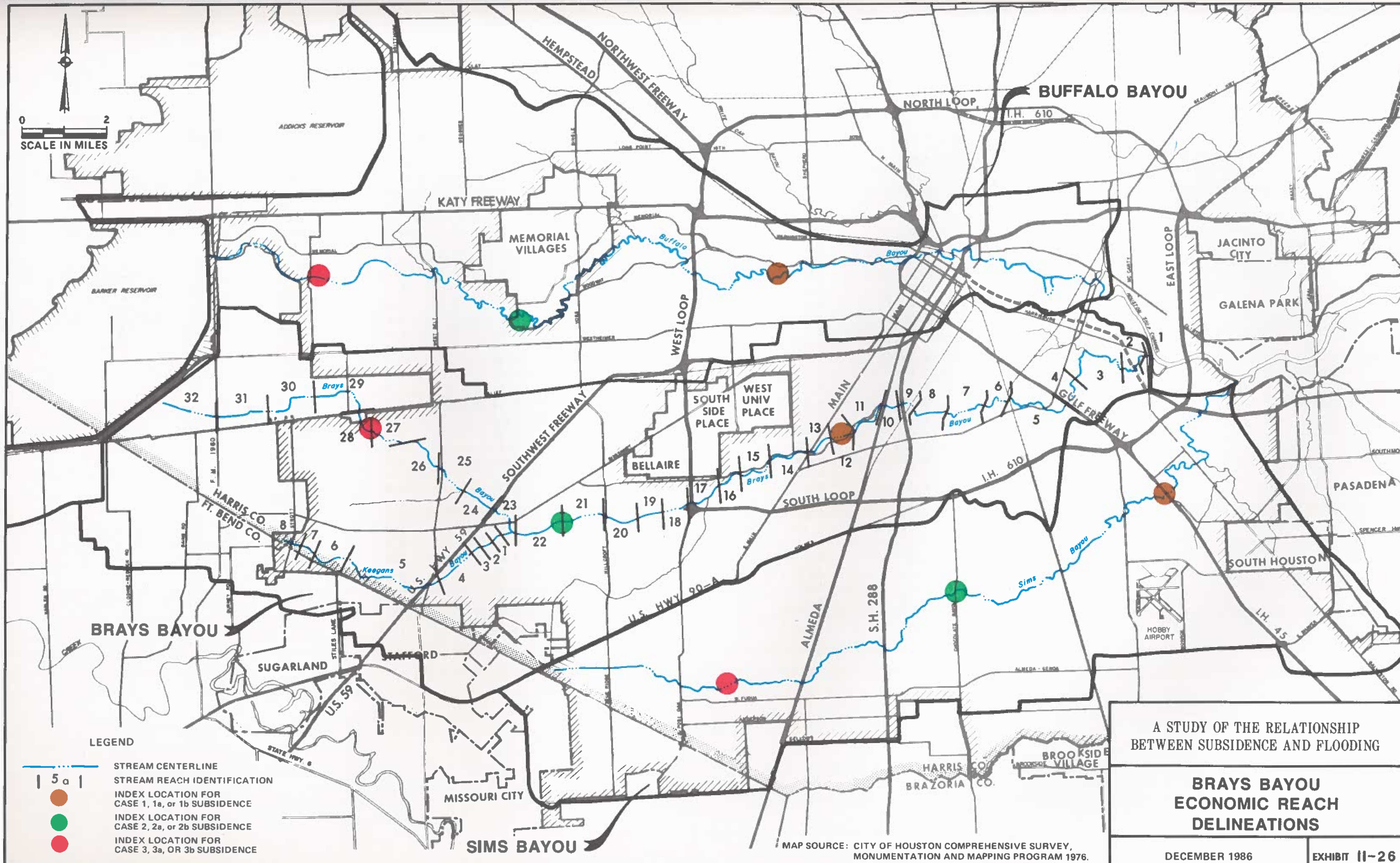
NOTE: ELEVATION DIFFERENCES < 0.25 FEET FROM BASE NOT SHOWN FOR CLARITY.

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

BRAYS BAYOU
STREAM PROFILE
STA. 1500+00 TO STA. 1569+13

DECEMBER 1986

EXHIBIT II-25



A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

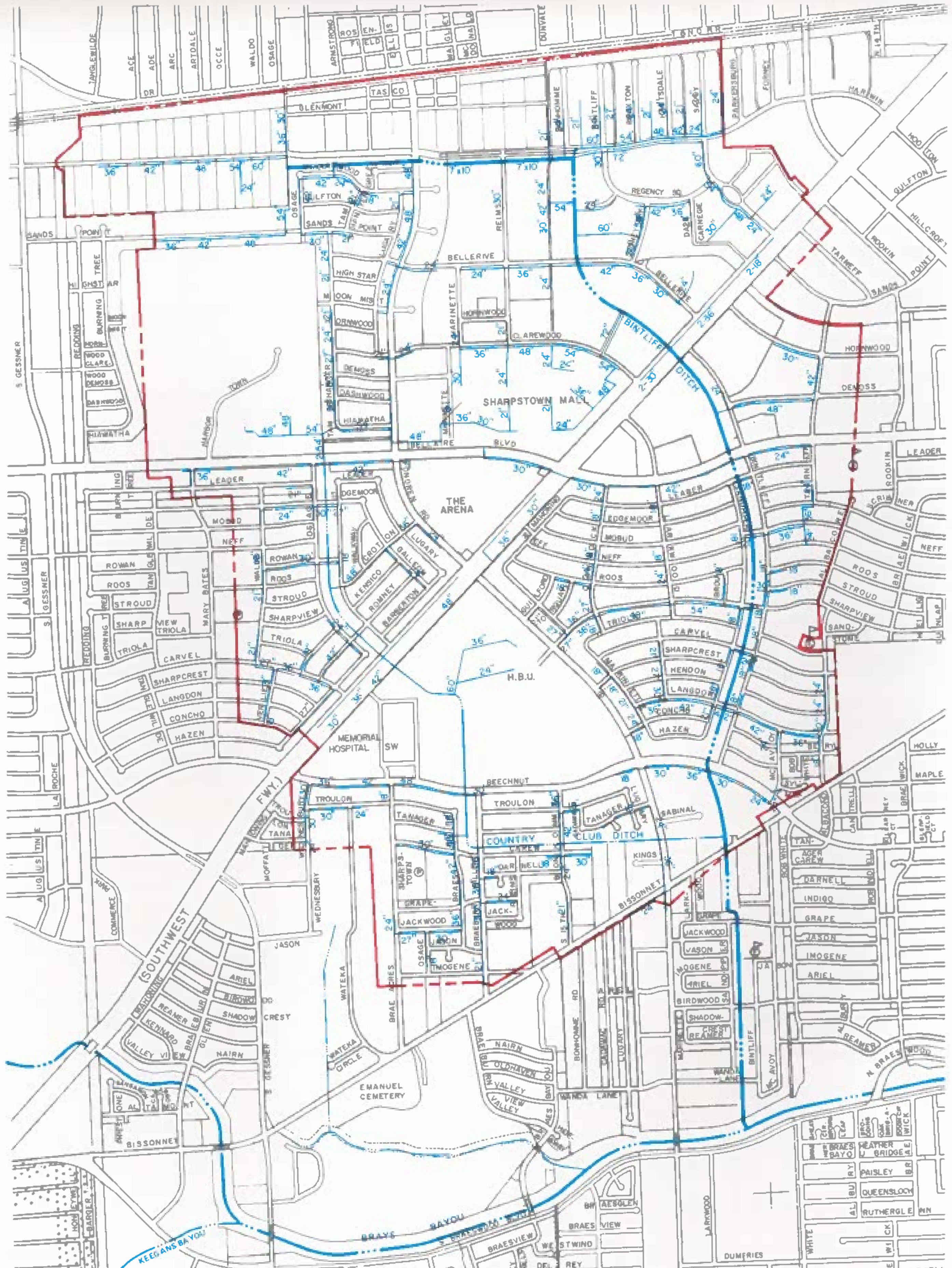
BRAYS BAYOU ECONOMIC REACH DELINEATIONS

DECEMBER 1986

EXHIBIT II-26

TABLE II-14 - BRAYS BAYOU ECONOMIC FLOOD DAMAGE DATA

Reach	Index Station	10-Year Flood Damages (Millions of Dollars) Existing Increment	Case 3 Increment	Case 3a (HGCSD Plan) Increment	100-Year Flood Damages (Millions of Dollars) Existing Increment	Case 3 Increment	Case 3a (HGCSD Plan) Increment
1	789	0.0	0.0	0.0	0.0	0.0	0.0
2	2315	0.0	0.0	0.0	0.1	0.1	0.1
3	9558	0.0	0.0	0.0	0.2	0.2	0.2
4	16570	0.0	0.0	0.0	0.6	0.6	0.7
5	22482	0.1	0.1	0.1	0.5	1.4	1.5
6	30262	0.0	0.0	0.0	0.0	0.0	0.0
7	36754	0.0	0.0	0.0	0.5	0.6	0.6
8	40708	0.0	0.0	0.0	0.4	0.5	0.4
9	44405	0.0	0.0	0.0	0.2	0.2	0.2
10	46077	0.0	0.0	0.0	0.3	0.3	0.3
11	51128	0.0	0.0	0.0	0.0	0.0	0.0
12	54646	0.0	0.0	0.0	0.1	0.1	0.1
13	56647	0.0	0.0	0.0	0.0	0.1	0.0
14	60741	0.5	44.0	16.0	62.0	70.0	62.0
15	63990	7.0	36.0	21.0	38.5	50.0	41.0
16	71691	0.0	1.0	0.0	2.0	5.0	2.5
17	74128	0.1	7.0	2.0	15.5	15.3	15.0
18	79262	0.1	11.0	1.8	25.9	31.0	29.5
19	82330	1.0	13.5	2.5	26.5	36.0	29.5
20	83429	9.0	29.5	14.5	56.0	75.5	69.5
21	89436	0.1	3.0	0.2	22.5	34.0	34.0
22	98436	0.1	8.0	2.5	46.0	52.0	52.0
23	102572	0.0	0.0	0.0	34.3	34.2	36.8
24	110282	0.0	0.0	0.0	1.2	1.6	1.5
25	114233	0.0	0.0	0.0	4.0	7.0	5.0
26	118152	0.0	0.0	0.0	4.8	10.4	4.4
27	122317	0.0	0.1	0.0	11.0	14.1	11.4
28	127972	0.0	0.0	0.0	0.2	0.1	0.0
29	131877	0.0	0.0	0.0	1.3	1.0	1.0
30	139302	0.5	0.1	0.3	25.2	14.0	16.5
31	145067	3.6	2.8	2.8	30.8	23.0	24.5
32	152757	14.0	9.3	12.0	51.0	42.0	46.0
TOTAL		36.1	165.4	75.7	461.6	520.3	486.2



2000 1000 0 2000 4000

SCALE IN FEET

LEGEND

EXISTING DRAINGE SYSTEM

BINTLIFF DITCH WATERSHED BOUNDARY

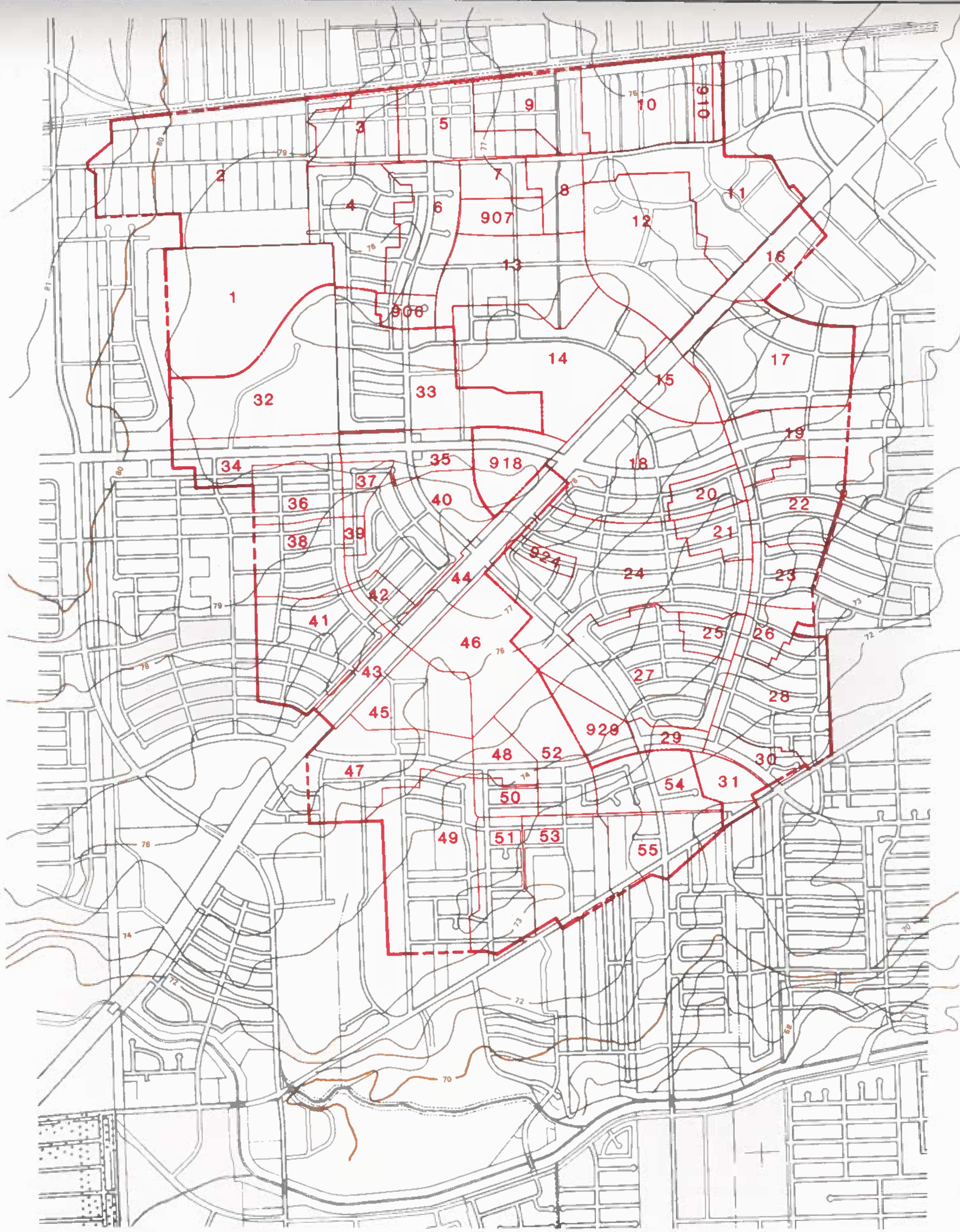
BASE MAP FROM HARRIS COUNTY FLOOD CONTROL DISTRICT

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

BINTLIFF DITCH WATERSHED MAP

DECEMBER 1986

EXHIBIT III-1



2000 1000 0 2000 4000

SCALE IN FEET

LEGEND

- BINTLIFF DITCH WATERSHED BOUNDARY
- BINTLIFF DITCH/COUNTRY CLUB DITCH WATERSHED DIVIDE
- SUBBASIN BOUNDARIES
- 21 SUBBASIN NUMBERS
- GENERALIZED CONTOURS (1.0' INTERVAL)

BASE MAP FROM HARRIS COUNTY FLOOD CONTROL DISTRICT

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

LOCATION OF SUBBASINS

DECEMBER 1986

EXHIBIT III-2

TABLE III-1 - BINTLIFF DITCH HISTORICAL STORMS

Date of Storm	85% Duration (hours)	Total Rainfall (inches)	Max. Rainfall Intensity (in/hr)	Recorded Runoff (inches)	Maximum Discharge (cfs)
July 20, 1969	1.0	1.00	1.00	0.25	256
June 11-12, 1973	46.5	9.08	1.43	6.75	1,130
June 9, 1975	17.0	5.21	2.41	3.47	1,050
June 15, 1976	5.0	4.89	1.27	3.13	1,170
June 7, 1978	1.0	1.93	1.74	1.20	1,020
Jan. 20-22, 1980	42.5	4.44	0.98	3.67	1,030

TABLE III-2 - AVAILABLE COMPUTER MODELS FOR STORM SEWER ANALYSIS

Model Characteristics	(1) RRL	(2) SWMM	(3) CURM	(4) UI	(5) UWMM	(6) CFS	(7) CHM	(8) ILLUDAS
Multiple Catchment Inflows	x	x	x	x	x	x	x	x
Dry Weather Flow	x	x		x	x	x	x	x
Input of Several Hyetographs	x	x			x	x	x	x
Snowmelt		x		x		x		
Impervious Area Runoff		x	x	x	x	x	x	x
Water Balance between Storms						x	x	x
Flow Routing in Sewers	x	x	x	x	x	x	x	x
Up and Downstream Flow Controls		x		x				
Surcharging and Pressure Flows		x			x			
Diversions		x		x	x			
Pumping Stations		x		x				
Storage		x		x	x	x		x
Prints Stage		x		x	x	x		
Prints Velocities		x		x	x			
Continuous Simulation		x				x		
Choice of Time Interval	x	x	x	x	x	x	x	x
Design Computation		x		x	x		x	
Real-Time					x			

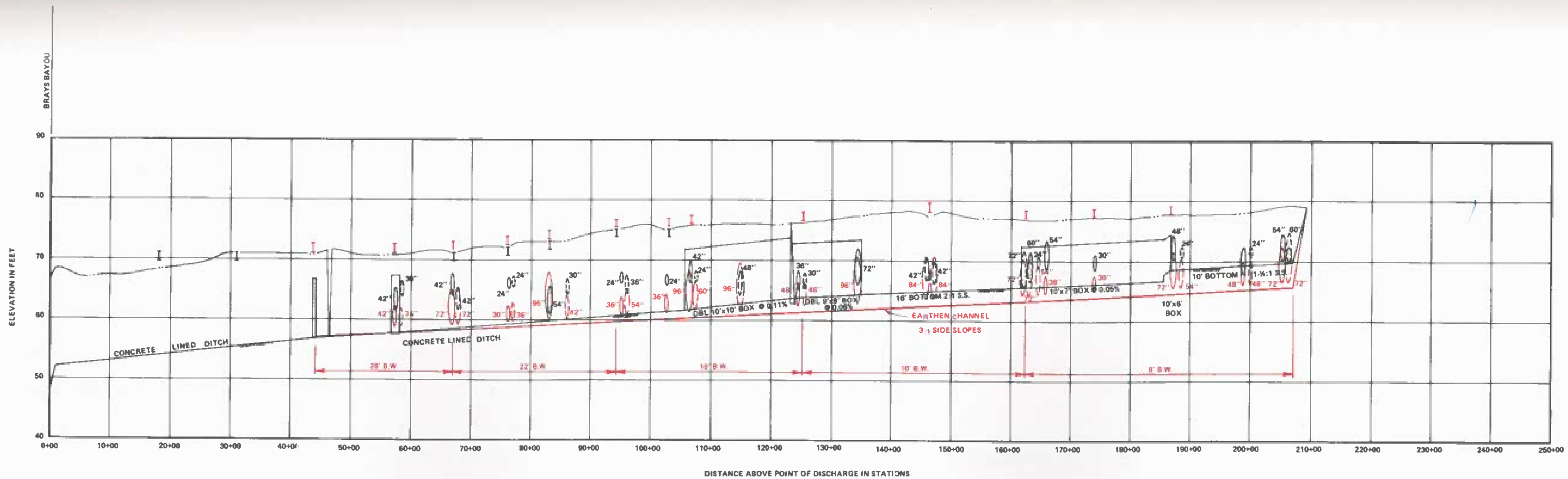
(1) British Road Research Laboratory
(2) Environmental Protection Agency and variants (Storm Water Management Model)
(3) University of Cincinnati (Cincinnati Urban Runoff Model)
(4) University of Illinois
(5) Batelle Memorial Institute (Urban Wastewater Management Model)
(6) Chicago Flow Simulation
(7) Chicago Hydrograph Method
(8) Illinois Urban Drainage Area Simulator

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

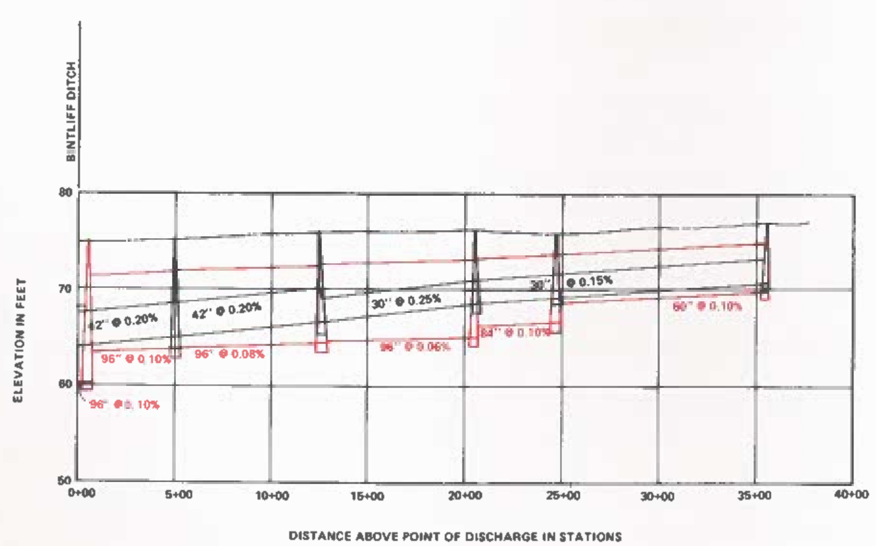
EXISTING AND REDESIGNED DRAINAGE SYSTEM

DECEMBER 1986

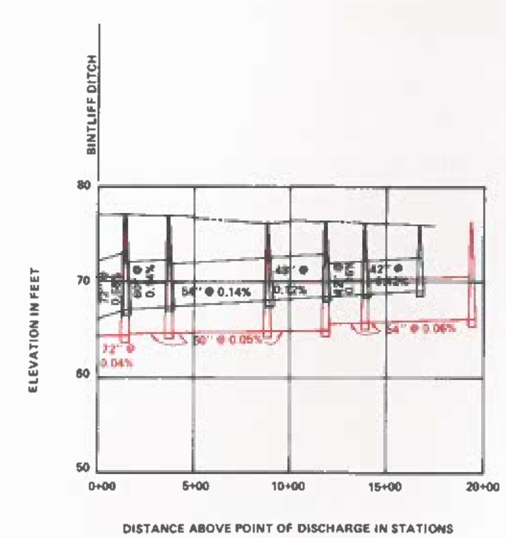
EXHIBIT III-3



BINTLIFF DITCH



BASIN 918 STORM SEWER



BASIN 910 STORM SEWER

LEGEND

- EXISTING GROUND & TOP OF CHANNEL BANK
- FLOWLINE EXISTING CHANNEL W/ BOTTOM WIDTH & SIDE SLOPES
- FLOWLINE REDESIGNED CHANNEL W/ BOTTOM WIDTH
- EXISTING STORM SEWER PROFILE WITH SIZE & SLOPE
- REDESIGNED STORM SEWER PROFILE W/ SIZE & SLOPE
- EXISTING BRIDGE
- REDESIGNED BRIDGE CROSSING

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

DRAINAGE SYSTEM PROFILES

DECEMBER 1986 | EXHIBIT III-4

TABLE III-3 - CALIBRATED INPUT PARAMETER RANGES

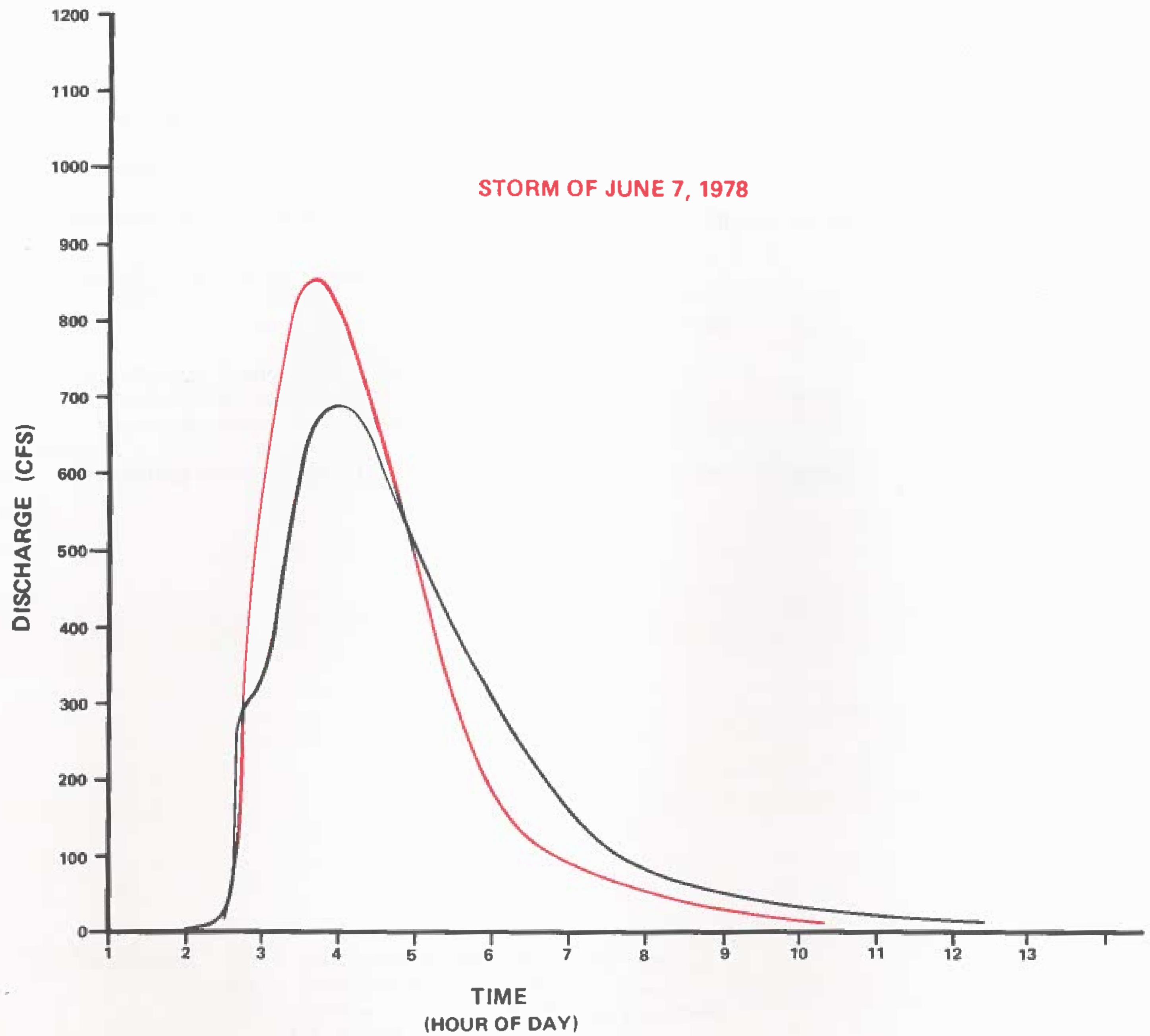
	Minimum	Maximum	Constant
Evaporation Rate (in./day)	-	-	0.1
Percent of Impervious Area with Zero Detention	-	-	1.0
Subbasin Area (acres)	7	157	
Subbasin Percent Imperviousness	2	95	
Subbasin Ground Slope (ft/ft)	0.0003	0.0020	
Impervious Area Roughness Factor (n)	0.10	0.15	
Pervious Area Roughness Factor (n)	0.25	0.40	
Impervious Area Depression Storage (in.)	-	-	0.2
Pervious Area Depression Storage (in.)	-	-	0.5
Infiltration Rate (in./hr.)	0.05	1.50	
Horton's Infiltration Decay Rate (1/sec.)	-	-	0.00115

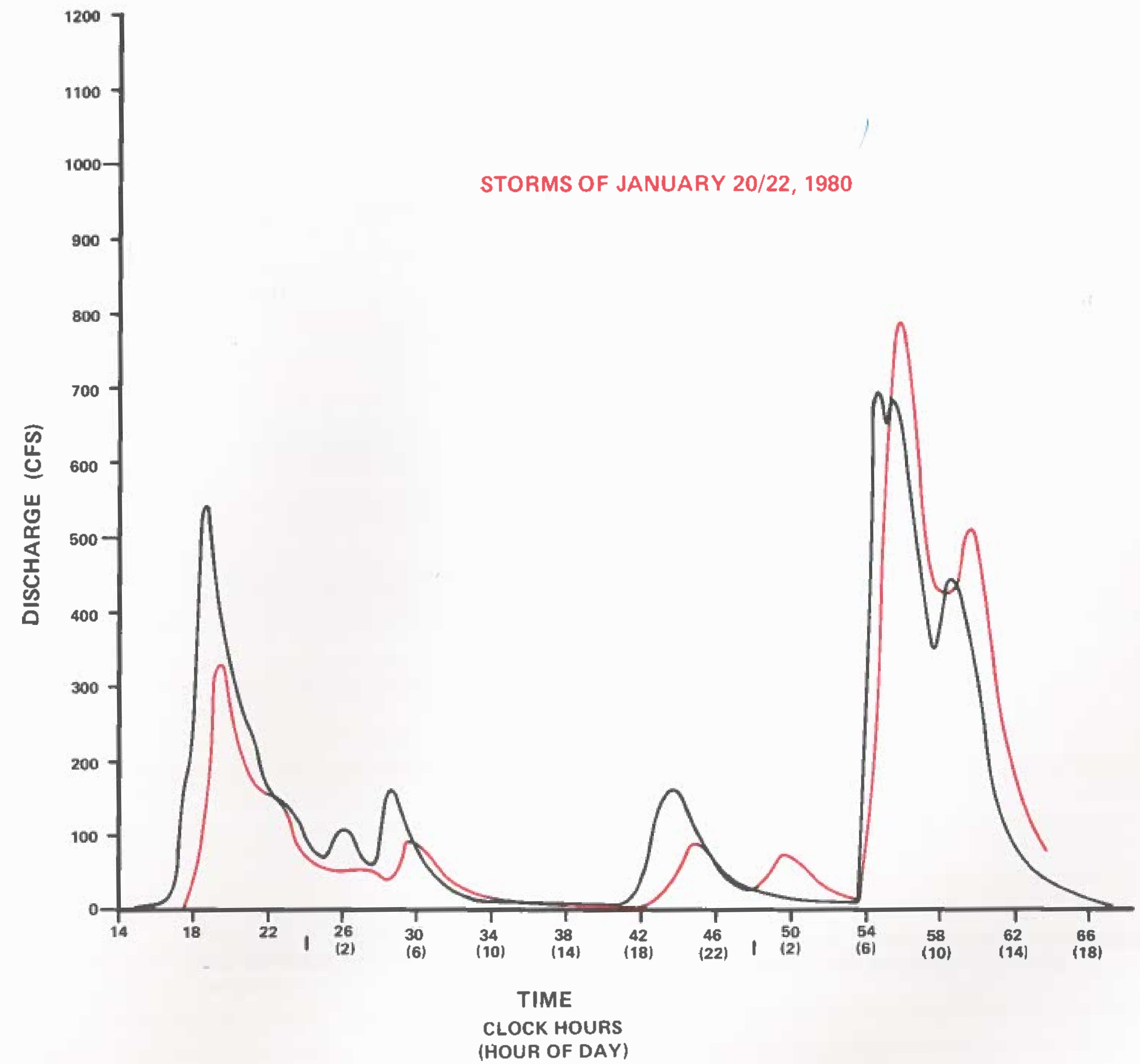
TABLE III-4 - SUMMARY OF PEAK FLOWS IN BINTLIFF DITCH AT BISSONNET (GAGE SITE)

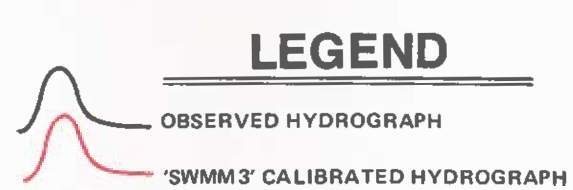
Drainage System	Channel Capacity (cfs)		Modeled Flow (cfs)		
	Existing	Design	3-yr.	10-yr.	100-yr.
Existing	1,300		1,440	1,460	1,470
Redesigned		3,200	2,090	3,270	3,560

TABLE III-5 - SUMMARY OF PEAK FLOWS IN SELECTED STORM SEWERS

Subbasin	Drainage System	Pipe Size	COH Design Flow (cfs)	Modeled 3-Year Flow (cfs)
3	Existing	24	10	28
	Redesigned	48	65	96
10	Existing	24	9	13
	Redesigned	30	15	22
918	Existing	30	16	30
	Redesigned	84	200	242







A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

MODEL CALIBRATION TO HISTORICAL STORMS

DECEMBER 1986

EXHIBIT III-5

TABLE III-6 - SIX DETAILED SUBBASIN DRAINAGE AREA CHARACTERISTICS

Subbasin No.:	906	907	910	918	924	929
Area (acres)	12	18	10	32	7	17
Land Use Classification*	RS/RM	GR	C2	C1	RS	C1/GR
Percent Imperviousness	62	2	90	95	35	10
Ground Slope (ft/ft)	0.0005	0.0005	0.0004	0.0010	0.0007	0.0013
Roughness Factor (n):						
Impervious Area	0.15	0.15	0.15	0.10	0.15	0.15
Pervious Area	0.25	0.40	0.35	0.32	0.25	0.32
Depression Storage (in.):						
Impervious Area	0.20	0.20	0.20	0.20	0.20	0.20
Pervious Area	0.50	0.50	0.50	0.50	0.50	0.50
Overland Flow Width (ft)	2056	1570	1735	9220	2033	3272

*Symbols: RS - Residential (Single-family homes)
RM - Residential (Multi-family homes)
C1 - Commercial (Buildings surrounded by paved lots)
C2 - Commercial/Industrial - Buildings surrounded by pavement and lawns or fields.
GR - Grasslands

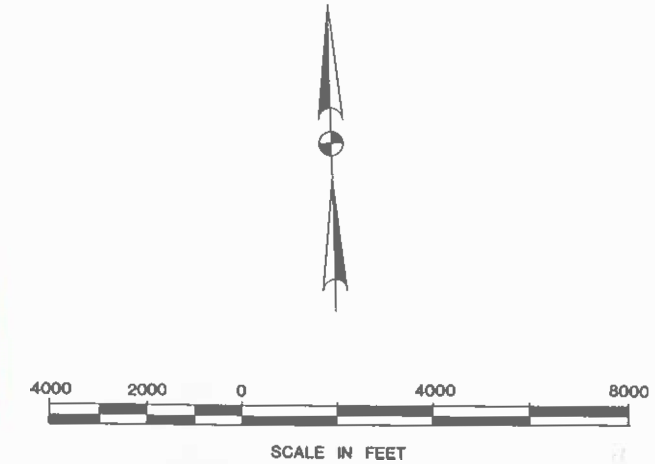
TABLE III-7 - MAXIMUM INLET PONDING DEPTHS (FEET) FOR BASE CONDITION (NO SUBSIDENCE)

Storm Frequency	Drainage System	Subbasin Number 906	907	910	918	924	929
3-year	Existing	1.83	1.49	1.63	1.14	1.04	1.33
3-year	Redesigned	1.83	0.00	1.24	0.00	0.82	0.74
10-year	Existing	2.13	2.37	1.94	1.24	1.12	1.55
10-year	Redesigned	1.95	2.02	1.77	1.17	1.04	1.26
100-year	Existing	2.38	2.64	2.39	1.40	1.37	1.74
100-year	Redesigned	2.52*	2.53	2.18	1.40	1.18	1.84*

*"Apparent inconsistencies during the 100-year storm event are due to modeling instabilities attributable to excessive surcharging and pressurized flow conditions."

CASE 1a

CASE 1b



CASE 1c

CASE 1d

LEGEND

EXISTING DRAINAGE SYSTEM

0.5 SUBSIDENCE CONE CONTOURS (0.1' INTERVAL)

WATERSHED DIVIDE

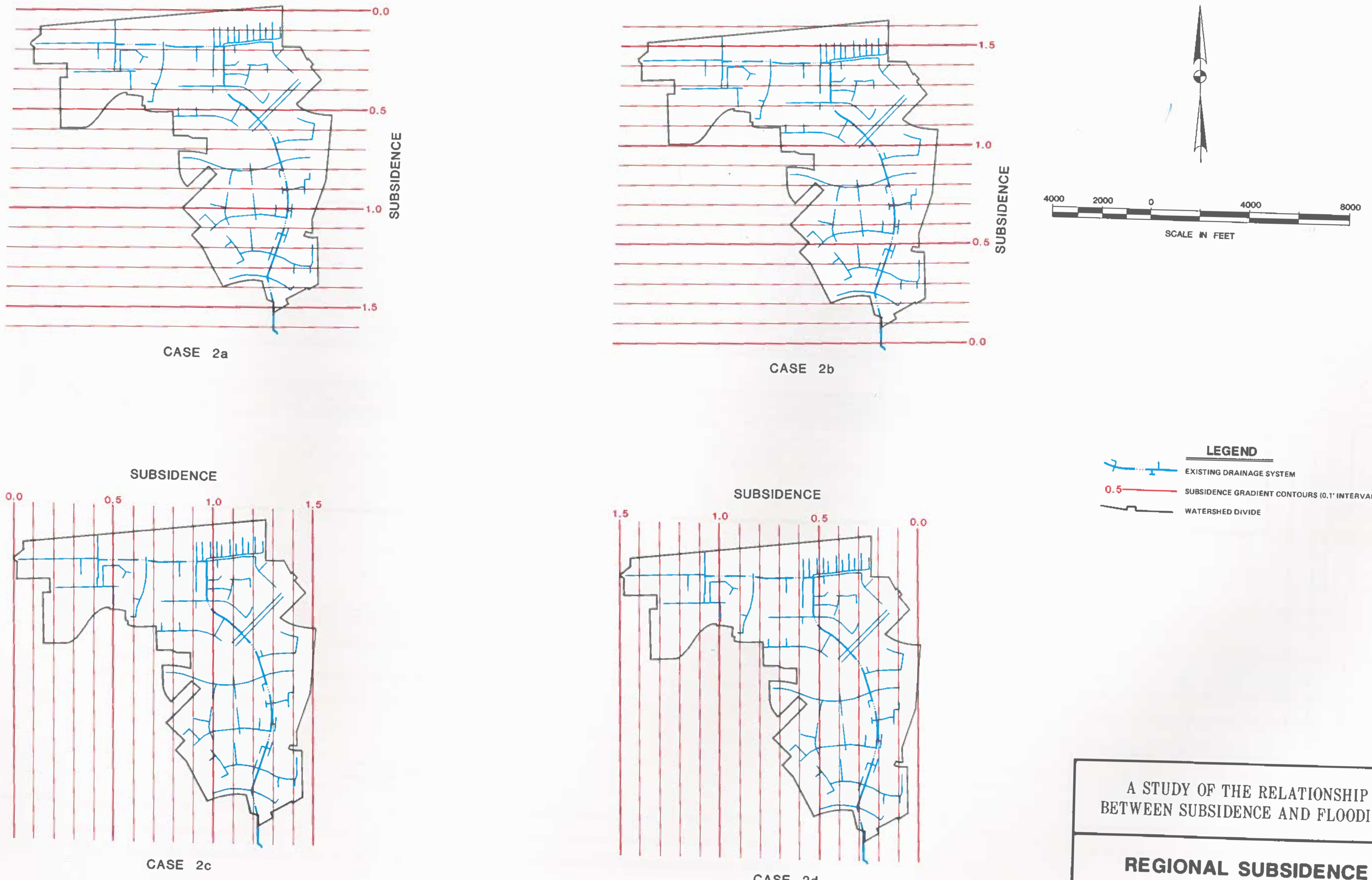
WELL FIELD EPICENTER

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

LOCALIZED WELL FIELD SUBSIDENCE CONES

DECEMBER 1986

EXHIBIT III-6

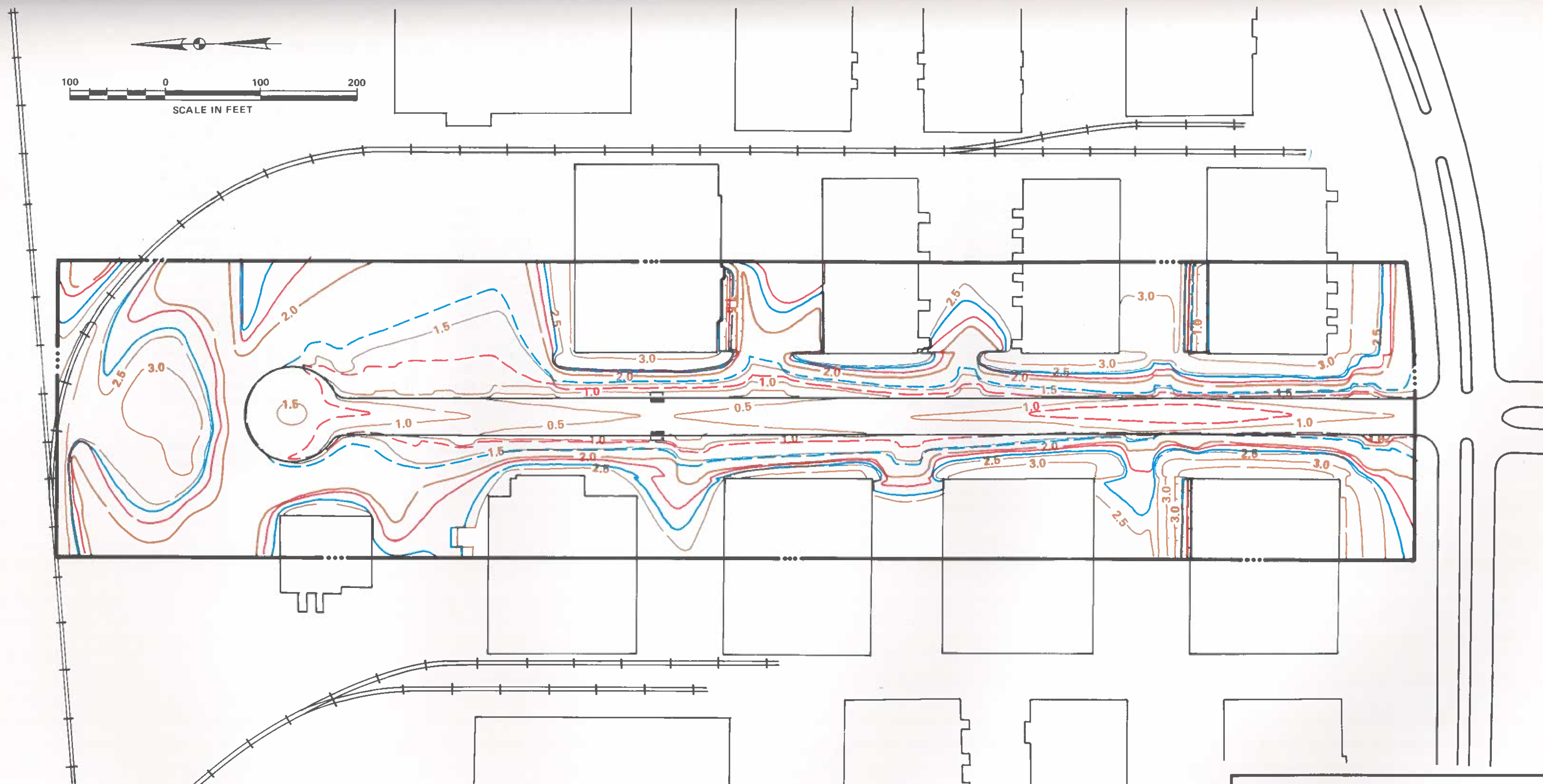


A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

REGIONAL SUBSIDENCE GRADIENTS

DECEMBER 1986

EXHIBIT III-7



LEGEND

SUBBASIN BOUNDARY (BASIN 910)

1.5 GROUND CONTOURS (0.5' INTERVAL ABOVE INLET)

3-YR. FLOOD PLAIN, EXISTING SYSTEM, CASE 2d (1.6')

100-YR. FLOOD PLAIN, EXISTING SYSTEM, CASE 2d (2.4')

3-YR. FLOOD PLAIN, REDESIGNED SYSTEM, CASE 2d (1.2')

100-YR. FLOOD PLAIN, REDESIGNED SYSTEM, CASE 2d (2.2')

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

TYPICAL SUBBASIN DRAINAGE AREA AND STREET PONDING

DECEMBER 1986

EXHIBIT III-8

TABLE III-8 - CHANGE IN INLET PONDING DEPTHS (FEET) FROM BASE CONDITION DUE TO SUBSIDENCE CASES

Drainage System	Subsidence Case	3-year Inlet Depths Subbasin Number						10-year Inlet Depths Subbasin Number						100-year Inlet Depths Subbasin Number					
		906	907	910	918	924	929	906	907	910	918	924	929	906	907	910	918	924	929
Existing	1a	-.09	+.45	+.05	-.01	-.01	-.03	+.04	+.05	+.05	.00	+.01	.00	.00	+.02	+.03	.00	+.07	+.01
Existing	1b	-.01	+.06	+.04	-.03	.00	-.04	+.08	+.04	+.02	-.02	.00	.00	+.01	+.01	+.05	-.02	+.03	+.02
Existing	1c	.00	+.09	-.03	+.02	+.06	-.02	+.02	+.02	-.02	+.02	.12	+.01	-.01	.00	+.02	+.01	+.22	+.02
Existing	1d	-.01	+.05	-.02	.00	+.02	-.04	+.01	+.02	-.02	+.01	-.01	-.01	-.02	.00	+.01	+.01	-.04	-.08
Existing	2a	+.05	+.10	-.04	+.01	-.01	-.09	+.03	-.04	+.11	+.02	-.01	-.04	-.03	-.03	-.02	+.01	-.07	-.03
Existing	2b	-.05	+.36	+.07	-.02	+.01	+.01	+.08	+.07	+.09	-.01	+.01	+.04	.00	+.02	+.09	-.01	+.08	+.07
Existing	2c	+.03	-.04	+.02	-.02	-.01	-.04	-.11	+.01	+.03	.00	-.01	-.02	-.07	+.01	+.03	.00	-.08	-.02
Existing	2d	+.11	+.51	.00	.00	+.02	-.01	+.13	+.01	-.01	+.01	+.02	+.03	+.04	-.02	-.01	.00	+.12	+.06
Redesigned	1a	-.05	.00	-.02	.00	+.18	+.24	-.07	-.01	-.10	+.06	+.01	+.02	+.02	+.02	.00	-.01	+.01	-.01
Redesigned	1b	.00	.00	-.01	.00	-.04	-.01	.00	-.08	-.10	-.04	+.01	.00	+.02	.00	.00	-.01	+.05	-.01
Redesigned	1c	-.01	.00	-.02	.00	-.33	+.01	-.01	-.05	-.10	+.03	+.08	+.02	-.01	-.01	-.01	+.01	+.07	+.08
Redesigned	1d	-.01	.00	-.02	.00	-.02	-.13	.00	-.01	+.05	+.01	.00	-.01	+.03	+.01	-.01	+.01	-.02	-.27
Redesigned	2a	+.01	.00	-.06	.00	-.02	-.02	+.04	-.08	-.11	+.08	+.02	.00	+.04	-.01	+.03	+.01	.00	-.06
Redesigned	2b	-.02	.00	+.15	.00	-.03	-.40	-.04	+.09	.00	.00	+.02	+.16	+.02	+.03	+.07	-.01	+.38	+.12
Redesigned	2c	-.03	.00	+.08	.00	-.37	-.07	-.02	+.08	-.05	-.02	-.01	-.01	-.03	+.03	+.04	.00	-.02	-.24
Redesigned	2d	+.03	.00	-.05	.00	-.02	+.01	+.02	+.02	-.09	+.04	+.02	+.03	+.08	-.02	-.02	-.02	+.02	+.27

+ indicates an increase from the base condition.
- indicates a decrease from the base condition.

TABLE III-9 - INITIAL TIMING OF INLET PONDING EQUAL TO SIX INCHES FOR BASE CONDITION

Storm Frequency	Drainage System	Subbasin Number 906	907	910	918	924	929
3-year	Existing	2:30*	2:45	2:30	2:15	2:30	2:30
3-year	Redesigned	2:30	(1)	2:30	(1)	2:45	2:45
10-year	Existing	4:30	4:30	4:15	3:30	4:30	4:30
10-year	Redesigned	4:15	4:30	4:30	4:30	4:30	4:30
100-year	Existing	3:45	4:15	3:45	3:00	4:00	4:00
100-year	Redesigned	4:00	4:15	4:15	4:15	4:15	4:15

*Note: Timing is in clock hours and represents the period from the initiation of rainfall until the depth at the subbasin inlet is approximately equal to six inches.

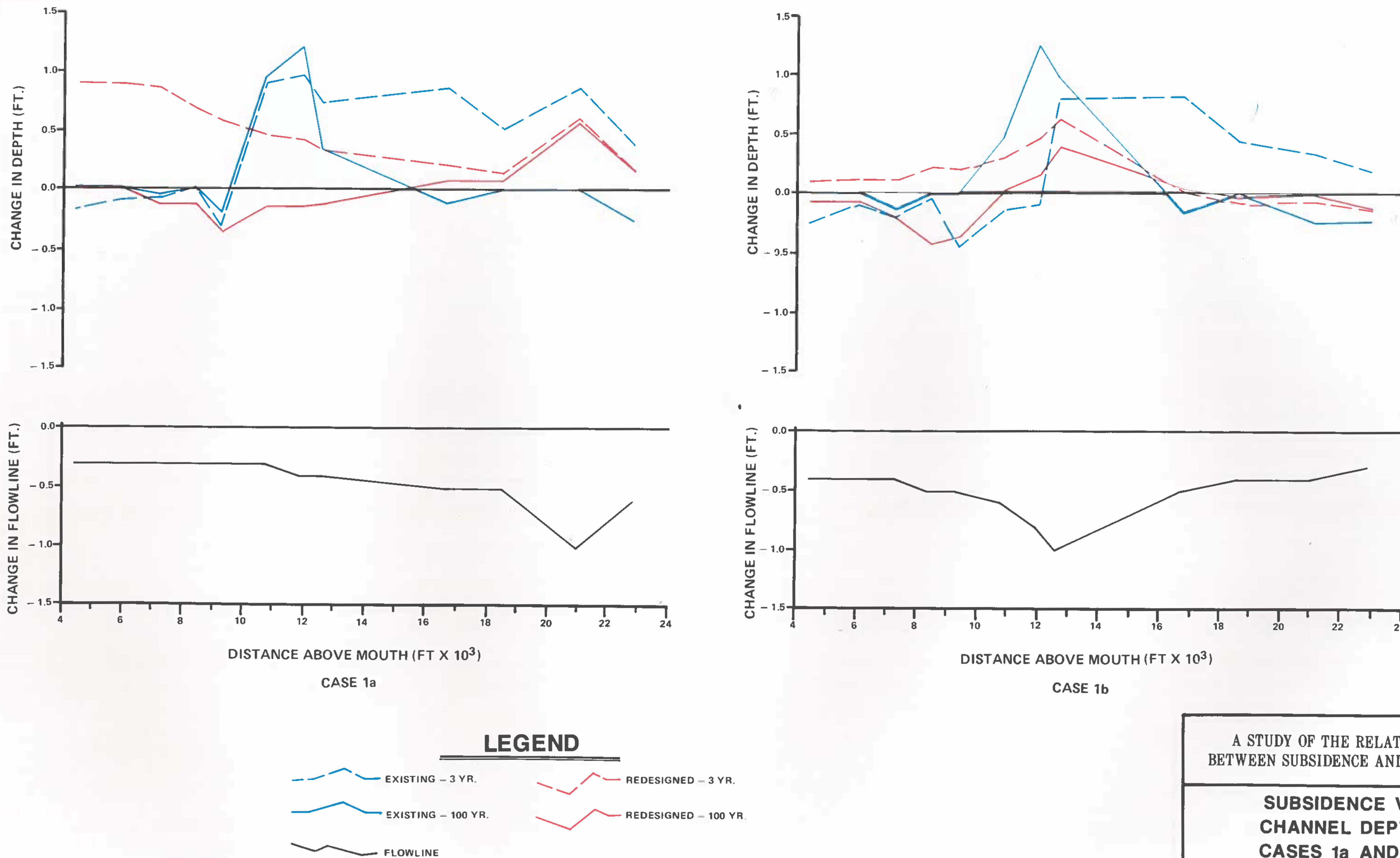
(1)Inlet ponding does not reach six inches for the base condition.

TABLE III-10 - DURATION OF INLET PONDING GREATER THAN OR EQUAL TO SIX INCHES FOR BASE CONDITION

Storm Frequency	Drainage System	Subbasin Number 906	907	910	918	924	929
3-year	Existing	3:30*	2:30	3:30	3:45[a]	1:15	2:45
3-year	Redesigned	3:45	0	1:00	0	0:15	<0:15
10-year	Existing	5:00	4:30	5:15	5:45	1:45	3:30
10-year	Redesigned	4:15	3:00	2:45	1:30	0:30	1:00
100-year	Existing	6:15[a]	5:45[a]	6:15[a]	7:00[a]	2:30	4:30
100-year	Redesigned	5:30	5:15	4:15	2:30	1:15	2:15

*Note: Duration is in clock hours and represents the period from the first time the inlet ponding depth is approximately equal to six inches until the last time the inlet ponding depth is approximately equal to six inches.

[a]Note: Duration periods with this superscript exceeded the simulation period of the computer run and are either greater than or nearly equal to the value indicated.

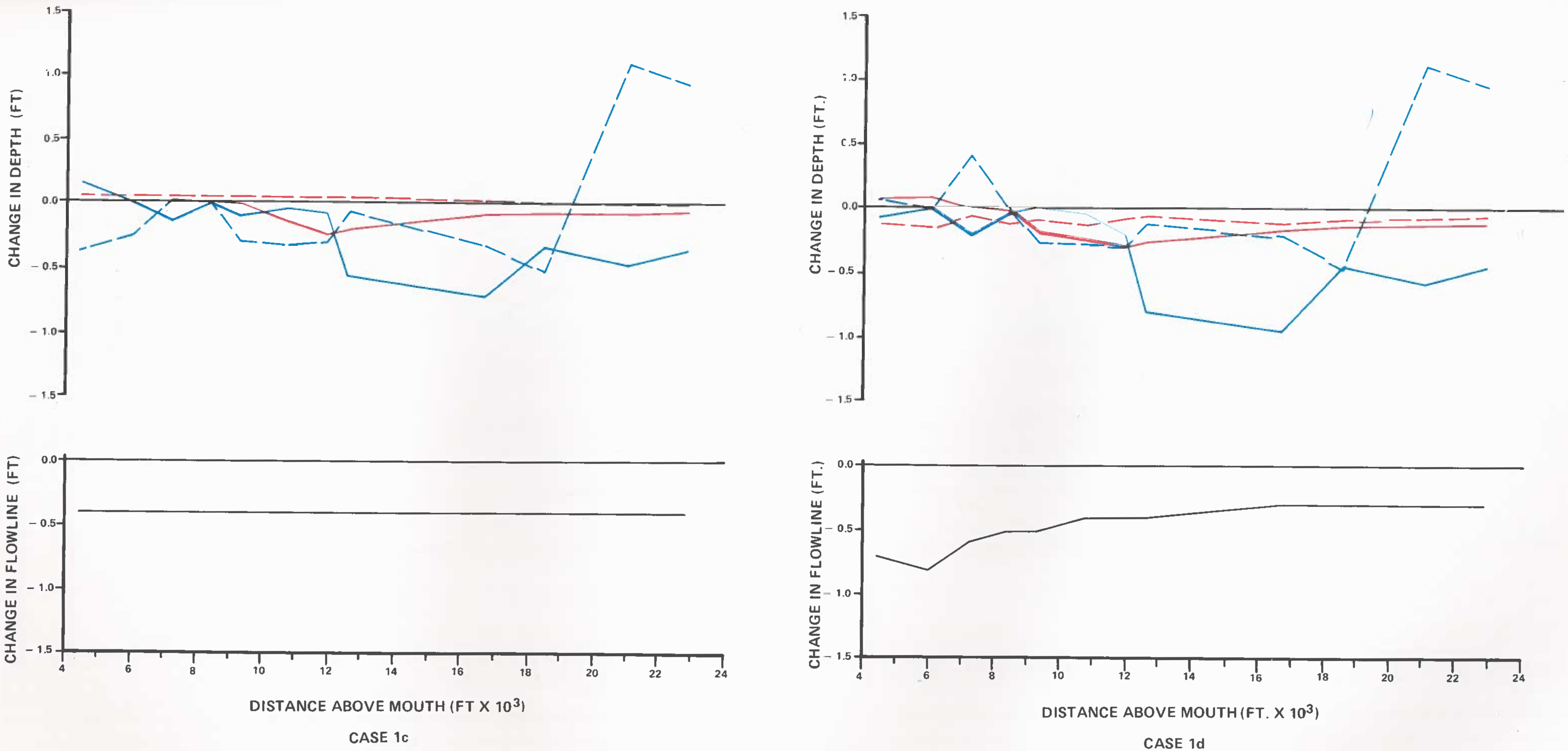


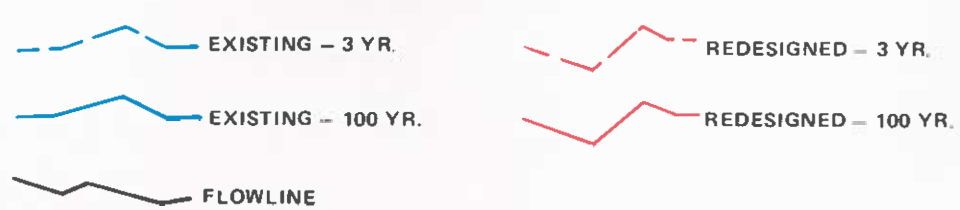
A STUDY OF THE RELATIONSHIP
BETWEEN SUBSIDENCE AND FLOODING

SUBSIDENCE VS. CHANNEL DEPTH CASES 1a AND 1b

DECEMBER 1986

EXHIBIT III–9



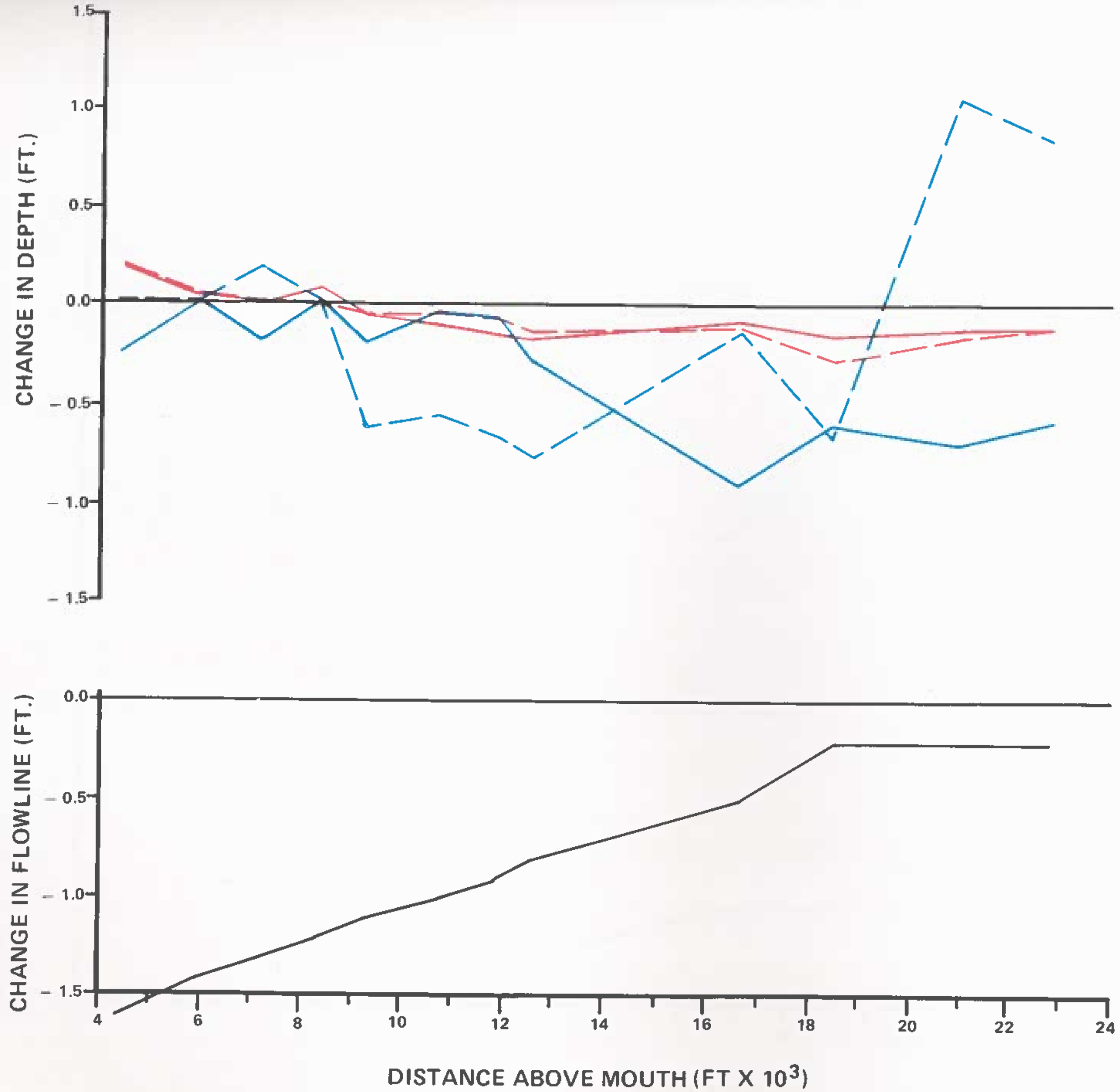


A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

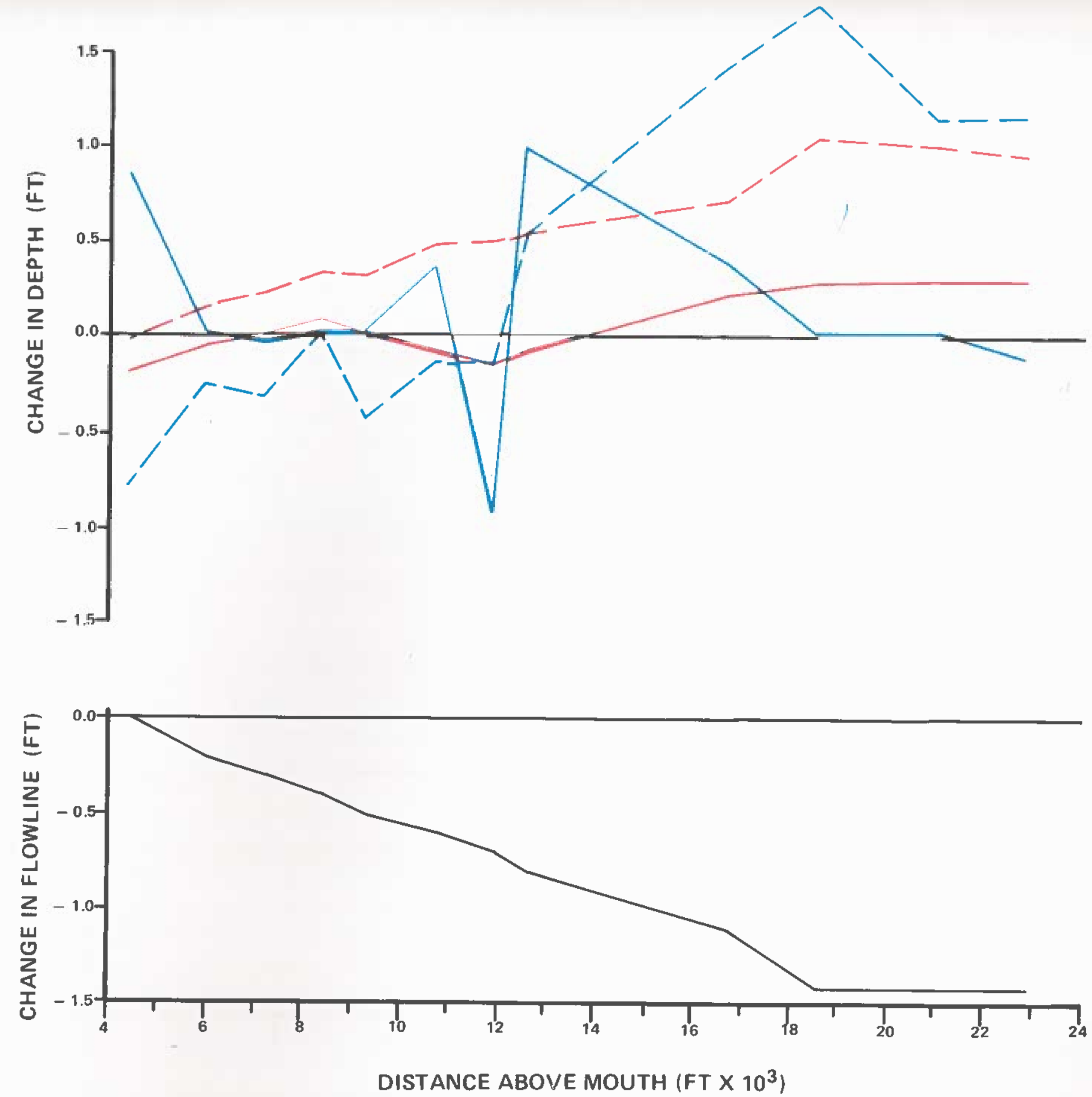
SUBSIDENCE VS. CHANNEL DEPTH CASES 1c AND 1d

DECEMBER 1986

EXHIBIT III–10

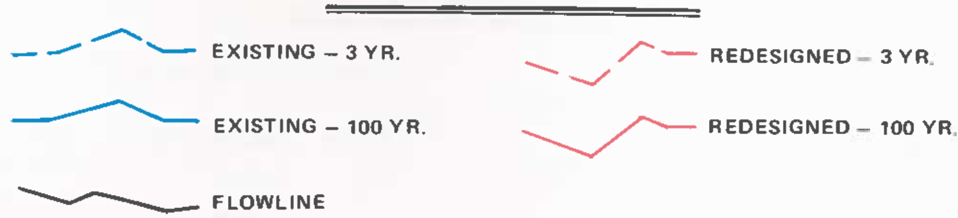


CASE 2a



CASE 2b

LEGEND

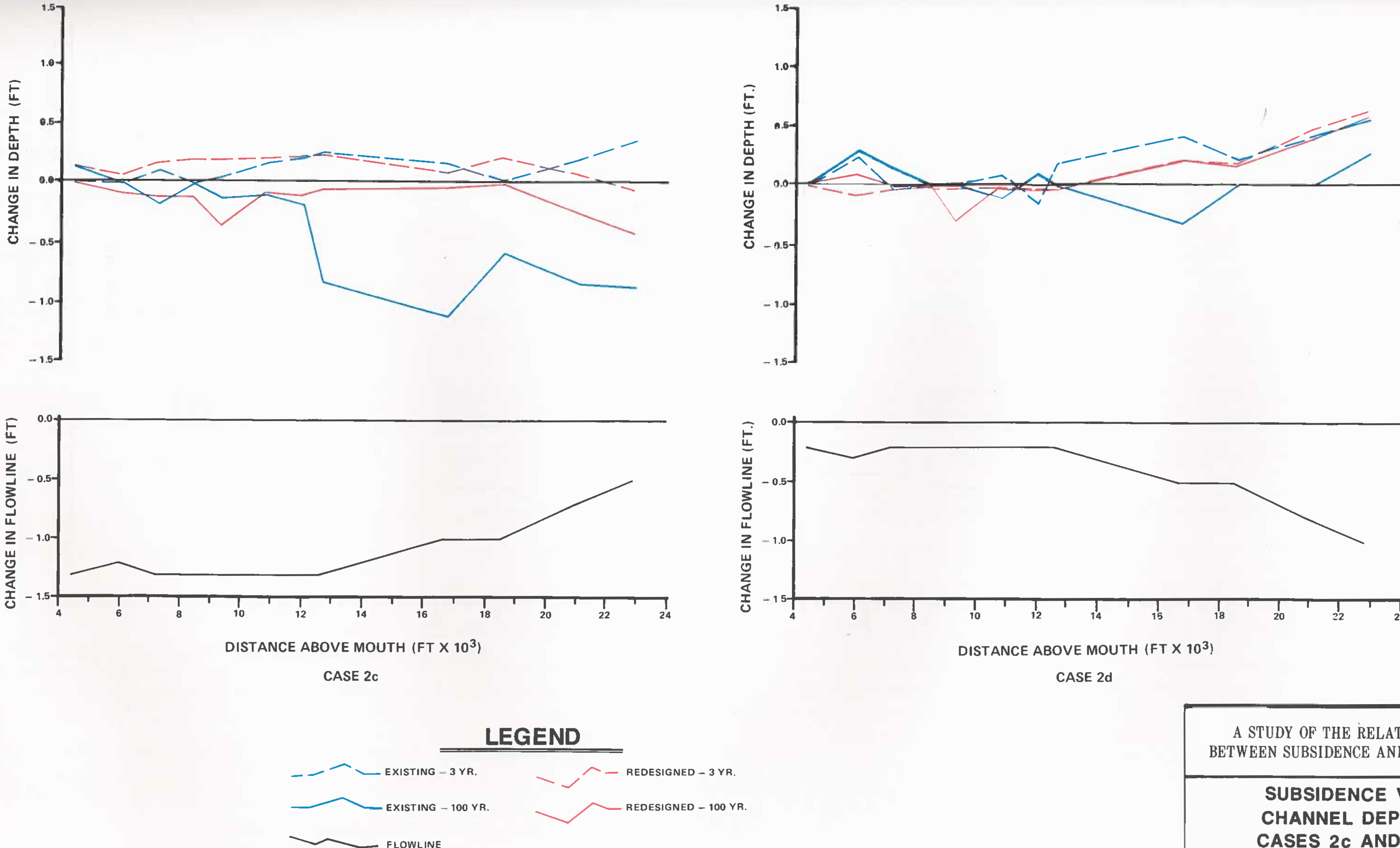


A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

SUBSIDENCE VS. CHANNEL DEPTH CASES 2a AND 2b

DECEMBER 1986

EXHIBIT III–11

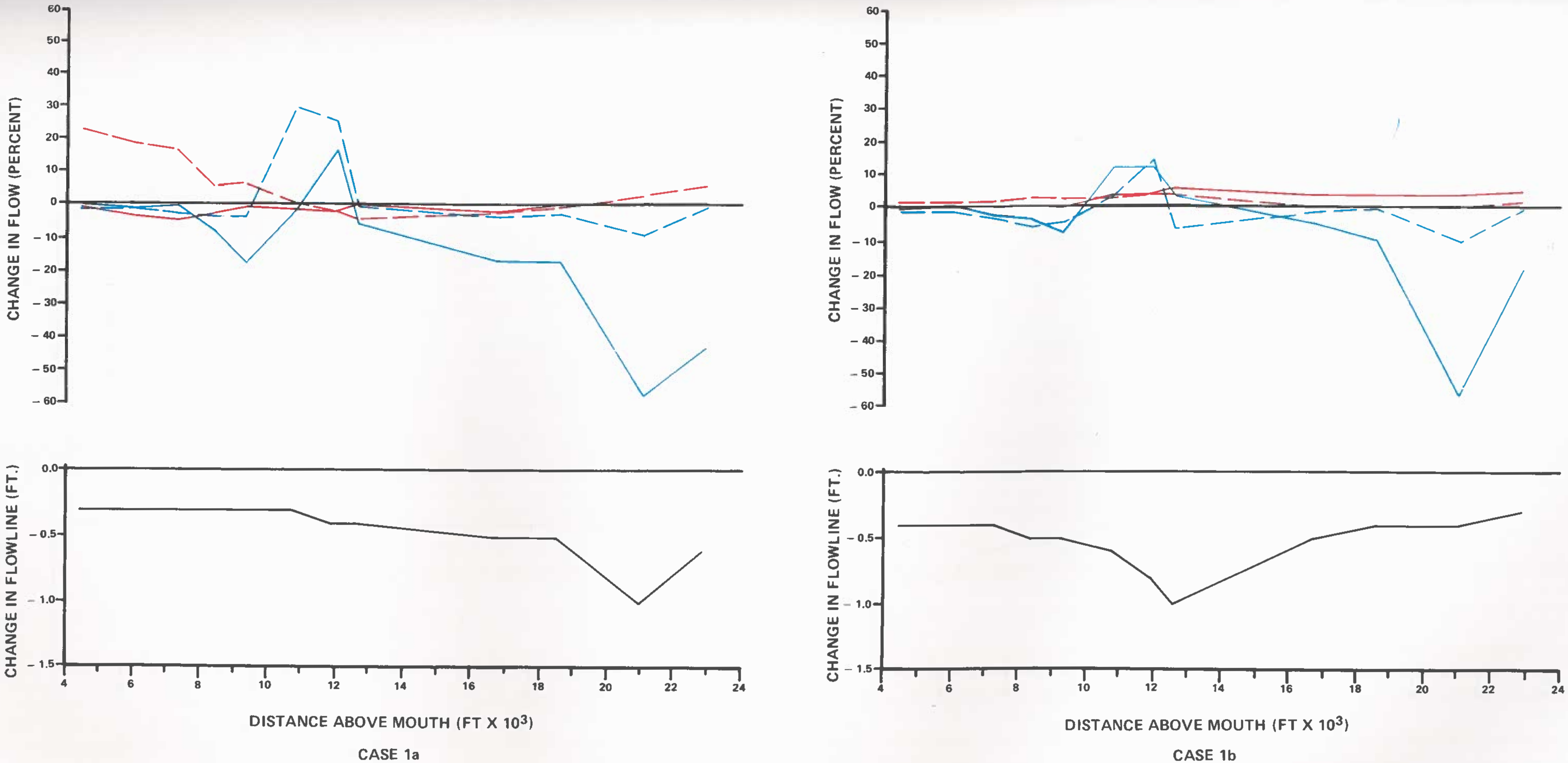


A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

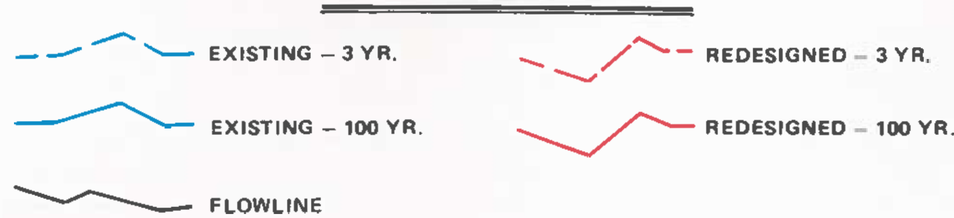
SUBSIDENCE VS. CHANNEL DEPTH CASES 2c AND 2d

DECEMBER 1986

EXHIBIT III–12



LEGEND

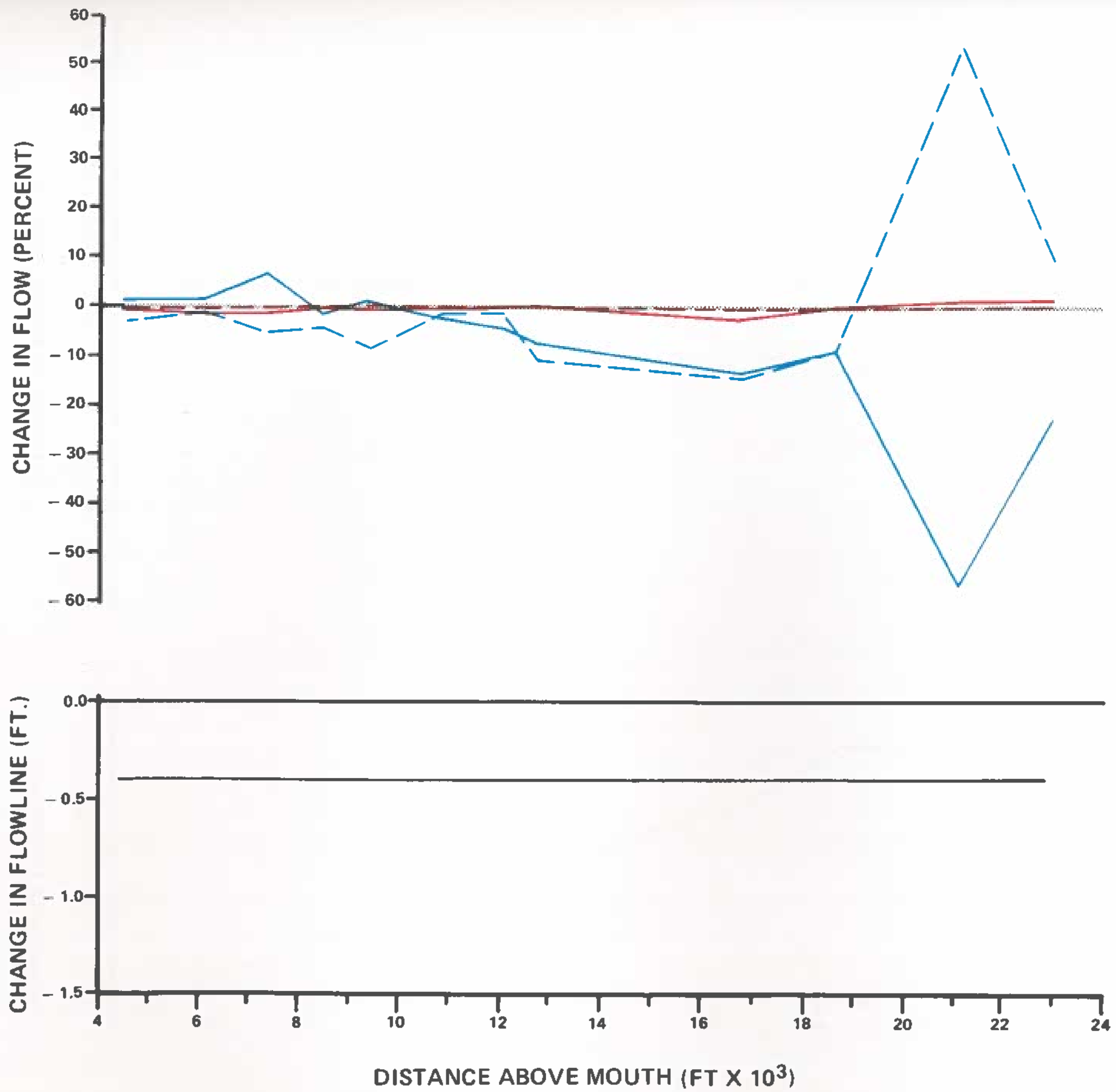


A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

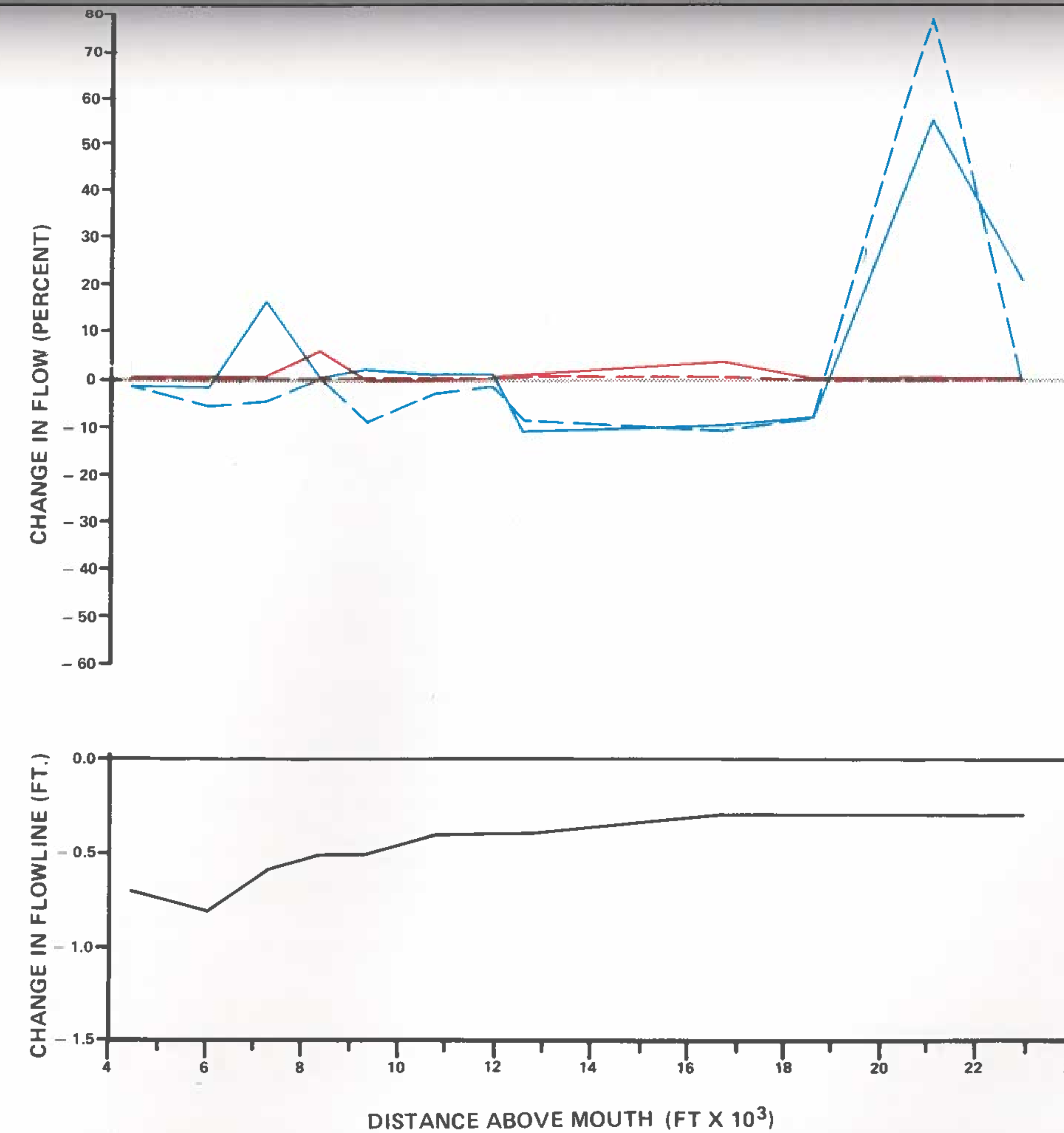
SUBSIDENCE VS. CHANNEL FLOW CASES 1a AND 1b

DECEMBER 1986

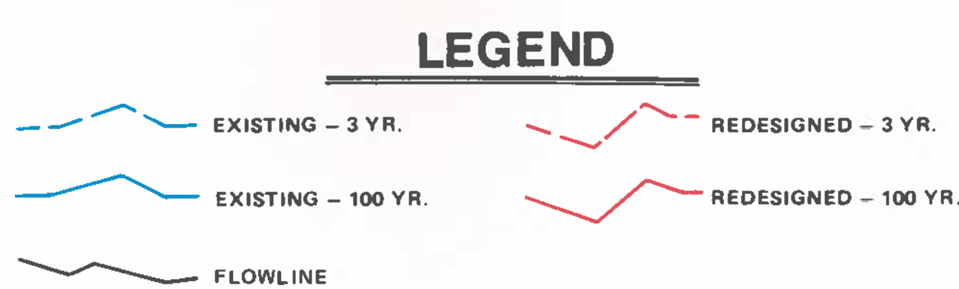
EXHIBIT III–13



CASE 1c



CASE 1d

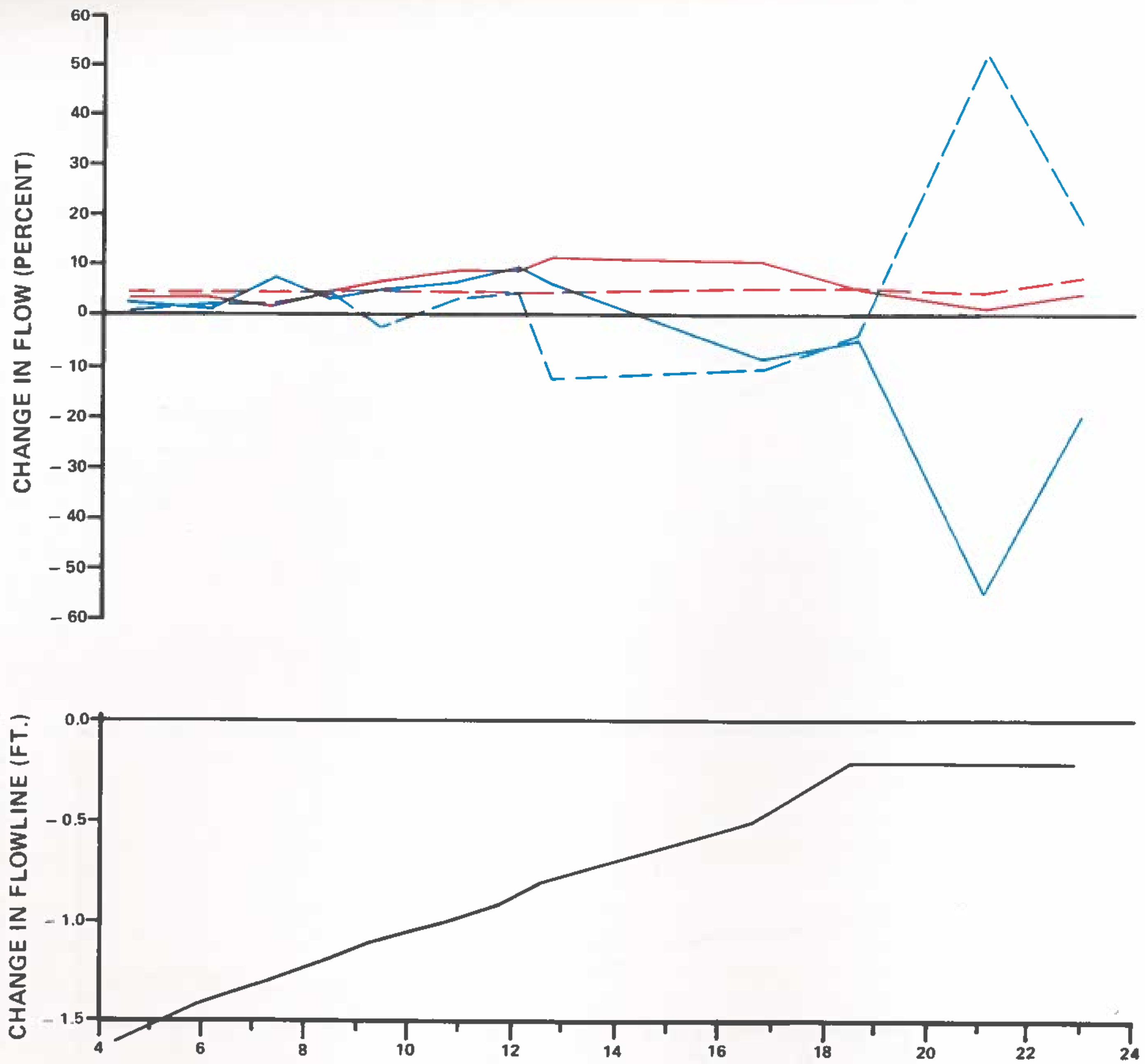


A STUDY OF THE RELATIONSHIP
BETWEEN SUBSIDENCE AND FLOODING

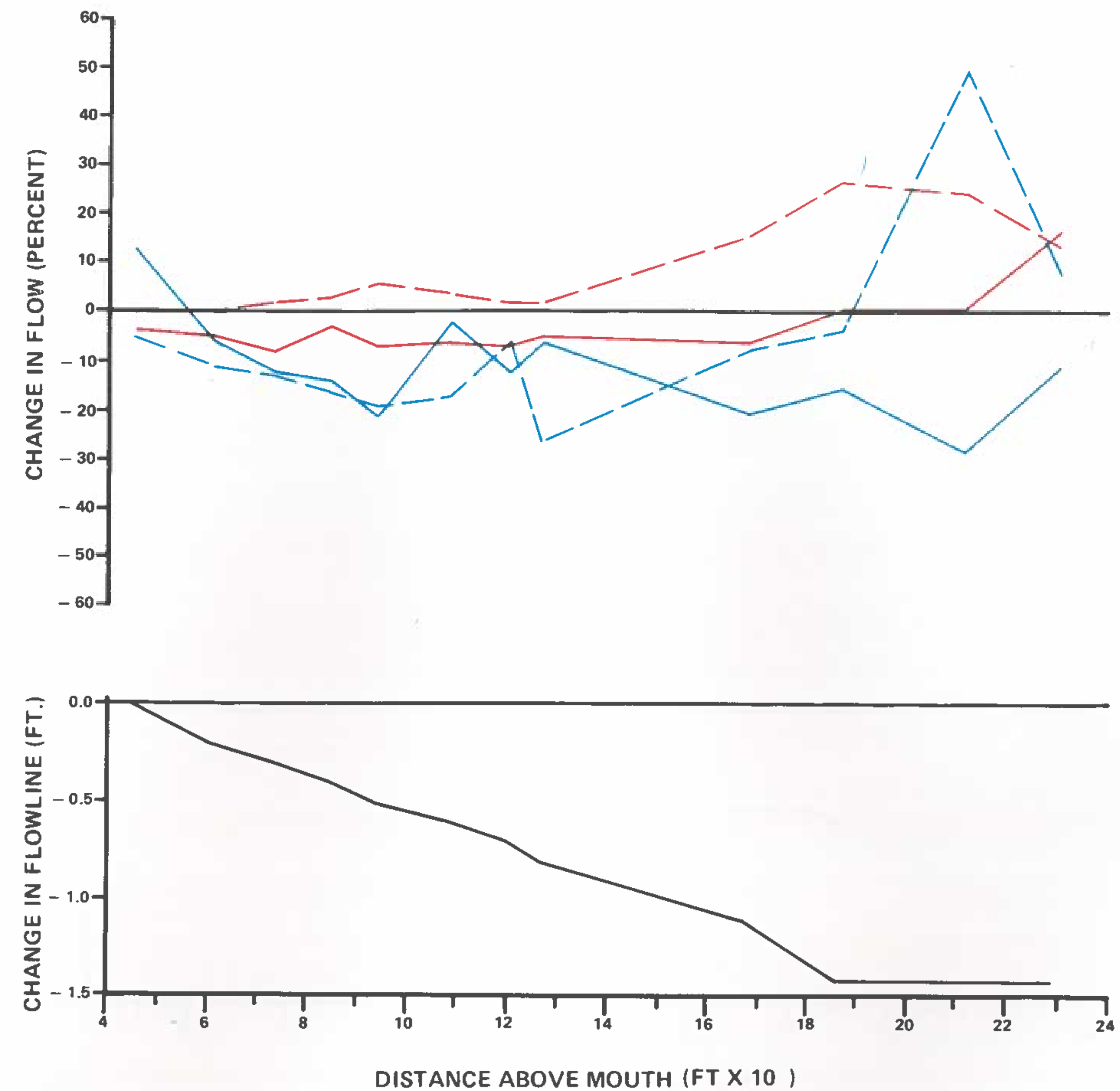
**SUBSIDENCE VS.
CHANNEL FLOW
CASES 1c AND 1d**

DECEMBER 1986

EXHIBIT III–14



CASE 2a



CASE 2b

LEGEND

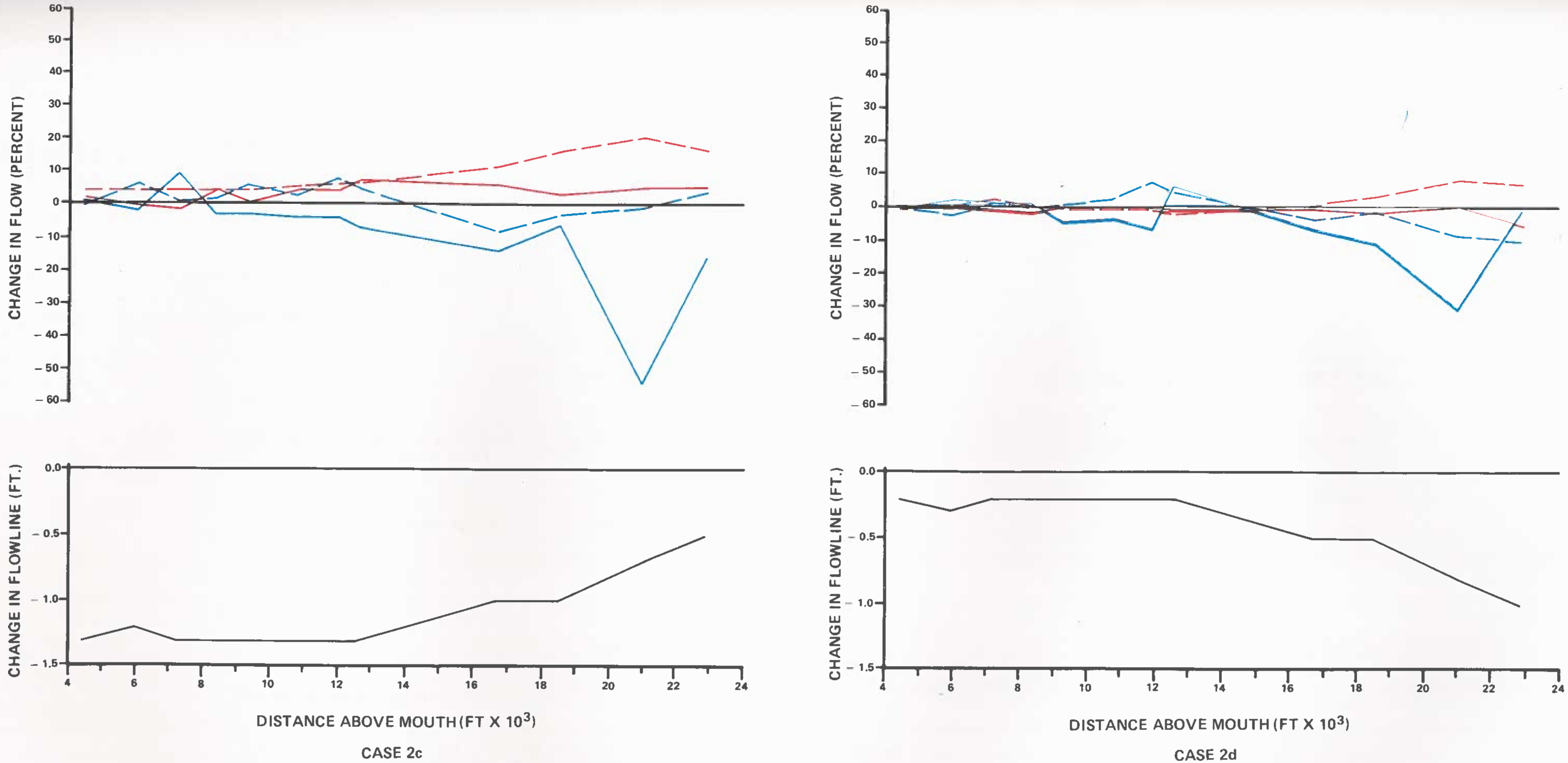
EXISTING – 3 YR.
EXISTING – 100 YR.
FLOWLINE
REDESIGNED – 3 YR.
REDESIGNED – 100 YR.

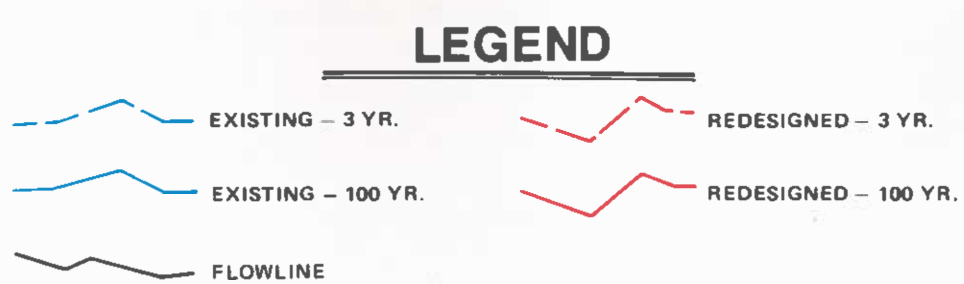
A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

SUBSIDENCE VS. CHANNEL FLOW CASES 2a AND 2b

DECEMBER 1986

EXHIBIT III–15





A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

SUBSIDENCE VS. CHANNEL FLOW CASES 2c AND 2d

DECEMBER 1986

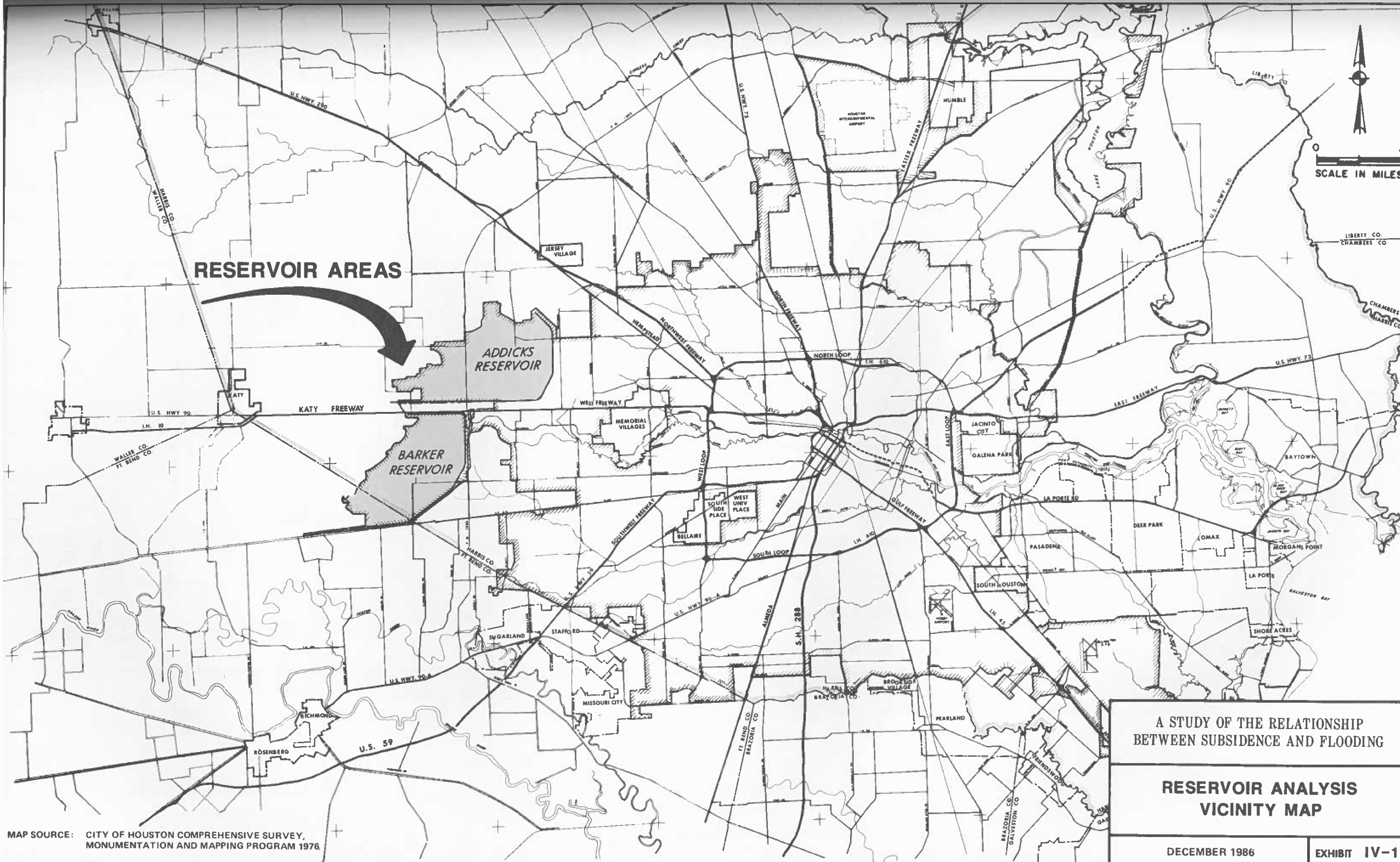
EXHIBIT III–16

TABLE IV-1 - RESERVOIR DATA[1]

Data Item	Addicks Reservoir		Barker Reservoir	
Location	On South Mayde Creek about 1 mile above its confluence with Buffalo Bayou, Harris County, Texas.		At Mile 49.8 of Buffalo Bayou about 1.5 miles above its confluence with South Mayde Creek, Harris County, Texas.	
Drainage Area	136 square miles		130 square miles	
Dam				
Type	Rolled Earth Embankment		Rolled Earth Embankment	
Length	61,166 feet		71,900 feet	
Height (Above Stream Bed)	48.5 feet		36.5 feet	
Reservoir				
	Elevation (feet)[2]	Storage Capacity (acre-feet)	Elevation (feet)[2]	Storage Capacity (acre-feet)
Top of Dam	121.6	--	112.5	--
Natural Ground at Ends of Dam (Maximum Flood Control Pool)	112.0	200,840	106.0	209,010
100-Year Flood Pool	104.0	91,450	97.8	89,500
Government-Owned Real Estate Limit	106.1	116,300	97.3	83,400
Conduit Invert	71.1	0	73.2	0
Control Structure				
Conduits	5 conduits (8 feet wide x 6 feet high x 252 feet long each)		5 conduits (9 feet wide x 7 feet high x 190.5 feet long each)	
Number of Gated Conduits	5		5	

(1) Source: "Hydrology, Addicks and Barker Reservoirs," U.S. Army Corps of Engineers, Galveston District, dated August 1977.

(2) "National Geodetic Vertical Datum of 1929," 1973 Adjustment.



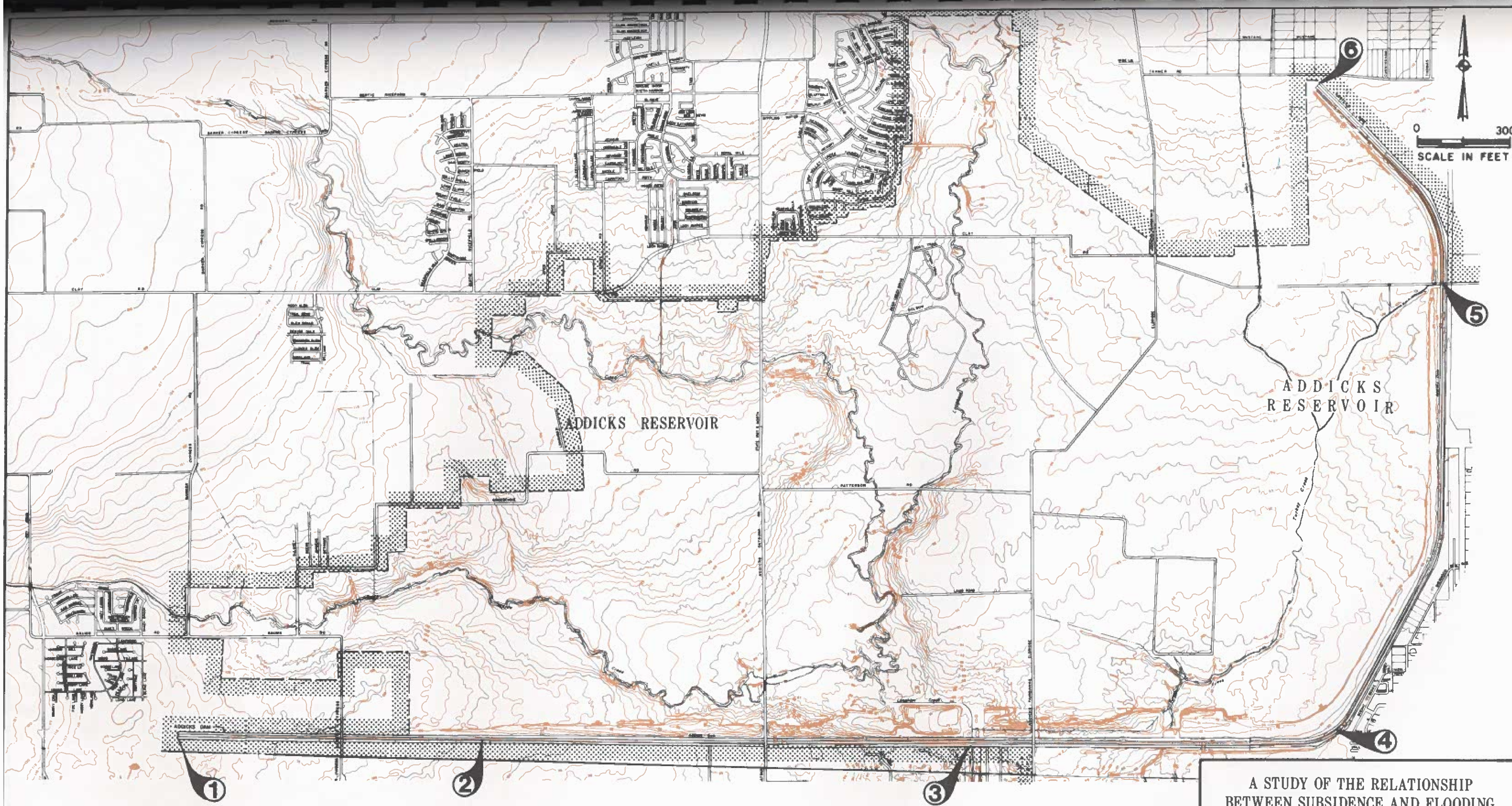
MAP SOURCE: CITY OF HOUSTON COMPREHENSIVE SURVEY, MONUMENTATION AND MAPPING PROGRAM 1976.

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

RESERVOIR ANALYSIS VICINITY MAP

DECEMBER 1986

EXHIBIT IV-1



A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

ADDICKS RESERVOIR EXISTING RESERVOIR CONTOUR MAP

DECEMBER 1986

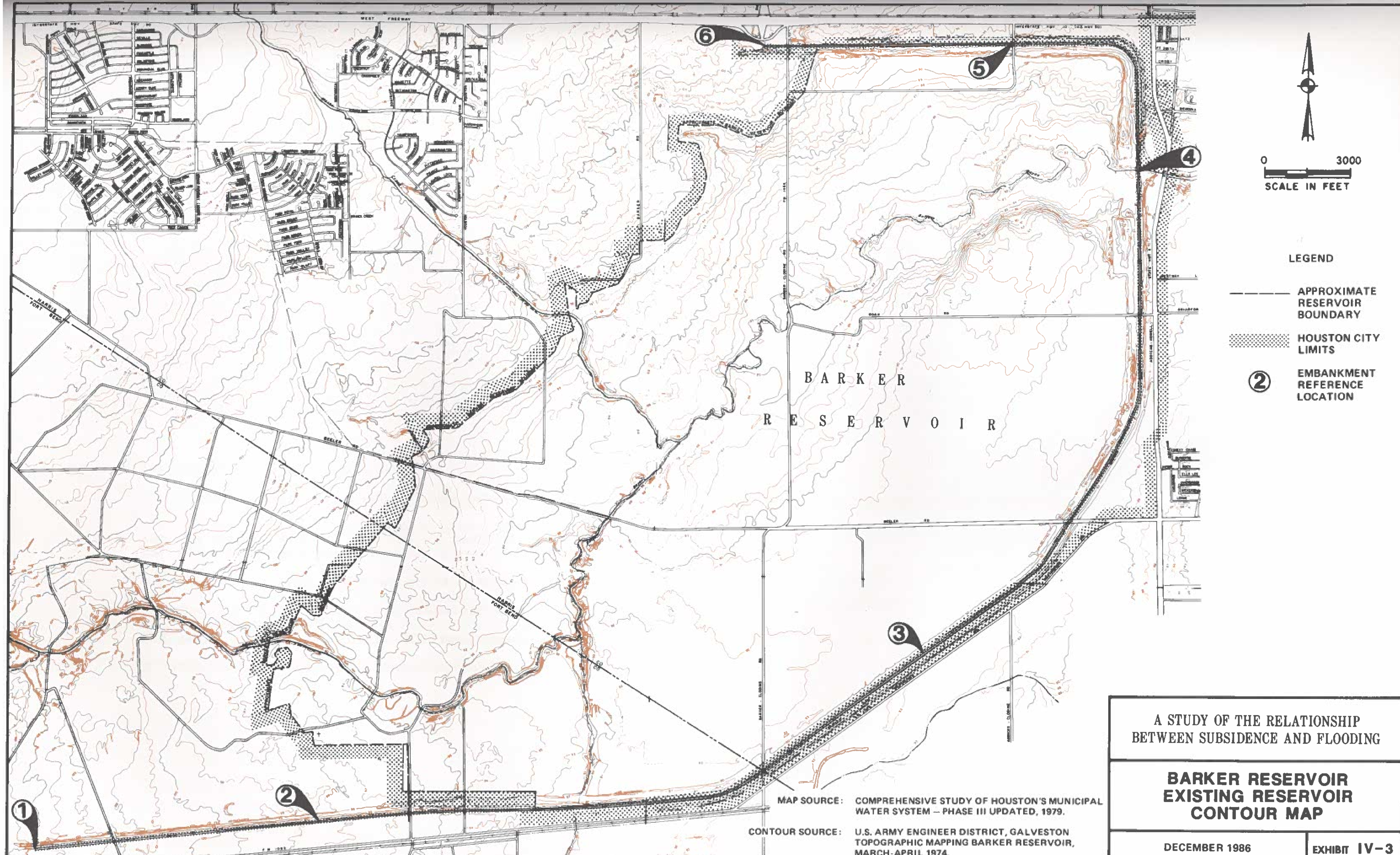
EXHIBIT IV-2

MAP SOURCE: COMPREHENSIVE STUDY OF HOUSTON'S MUNICIPAL WATER SYSTEM – PHASE III UPDATED, 1979.

CONTOUR SOURCE: U.S. ARMY ENGINEER DISTRICT, GALVESTON TOPOGRAPHIC MAPPING ADDICKS RESERVOIR, MARCH-APRIL 1974.

LEGEND

- APPROXIMATE RESERVOIR BOUNDARY
- HOUSTON CITY LIMITS
- ② EMBANKMENT REFERENCE LOCATION



A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

BARKER RESERVOIR EXISTING RESERVOIR CONTOUR MAP

DECEMBER 1986

EXHIBIT IV–3

TABLE IV-2 - EXISTING STORAGE-CAPACITY COMPARISON

Elevation (feet)[1]	Corps of Engineers' Storage-Capacity (acre-feet)	Base Condition Storage-Capacity Reconstruction (acre-feet)
Addicks Reservoir		
71	0	0
77	47	--[2]
81	295	--[2]
87	2,315	1,747
88	3,190	2,534
90	5,707	4,711
92	9,926	8,353
94	16,704	14,066
96	26,256	24,873
98	38,461	37,291
100	53,182	52,140
102	70,712	69,875
104[3]	91,454	90,733
106	115,020	114,712
108	140,962	140,124
110	169,449	168,871
112	200,840	199,996[4]

Elevation (feet)[1]	Corps of Engineers' Storage-Capacity (acre-feet)	Base Condition Storage-Capacity Reconstruction (acre-feet)
Barker Reservoir		
74	0	0
80	36	--[2]
86	3,979	3,786
87	6,005	5,686
89	11,756	11,050
91	20,533	20,477
93	36,200	37,348
95	56,989	58,025
97	79,813	80,937
97.8[5]	89,498	--[2]
99	104,726	105,866
101	132,078	133,117
103	161,252	162,101
105	192,544	192,888
106	209,013	208,833[6]

Notes: (1)"National Geodetic Vertical Datum of 1929," 1973 adjustment.

(2)Storage-capacities were not computed at these levels.

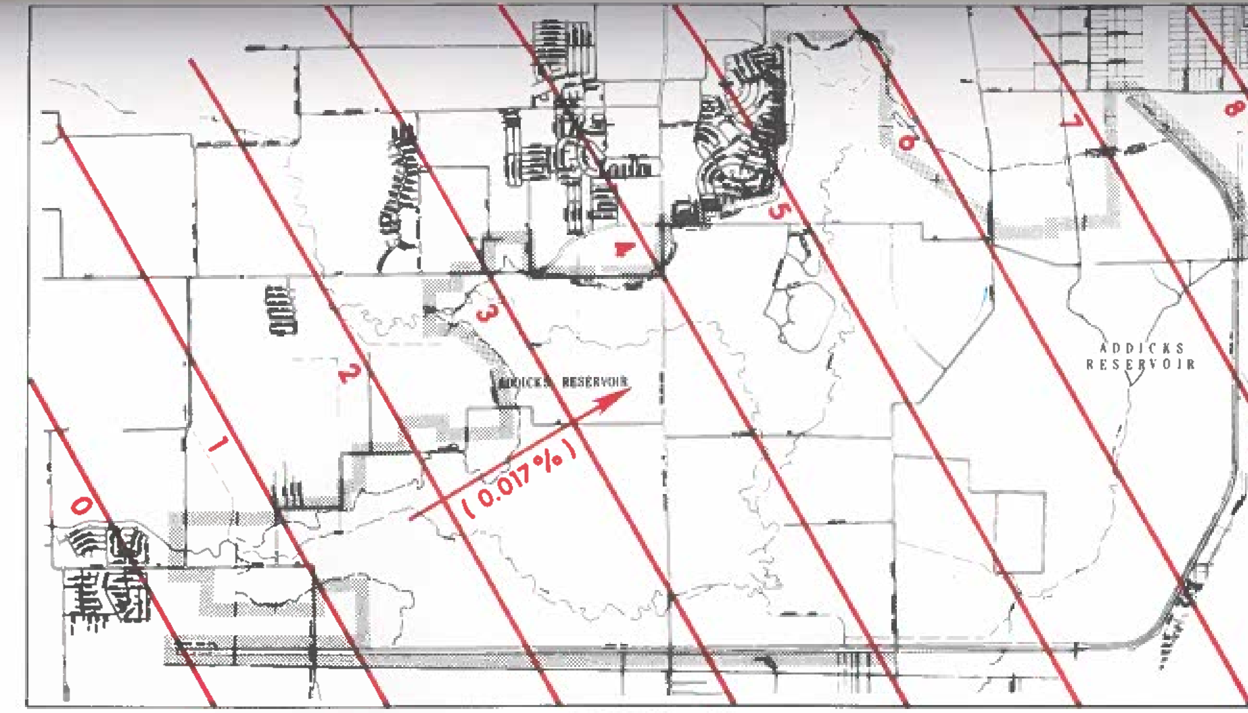
(3)Elevation 104 feet is the 100-year pool level for Addicks Reservoir. The 100-year capacity of 91,454 acre-feet will result in an elevation 104.1 feet on the reconstructed curve (0.1 foot increase).

(4)A difference in maximum storage-capacity of -0.42 percent.

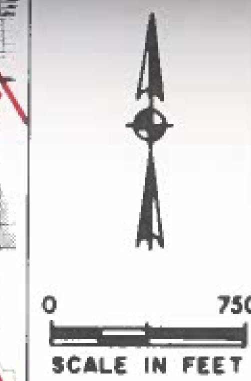
(5)Elevation 97.8 feet is the 100-year pool level for Barker Reservoir. The 100- year capacity of 89,498 acre-feet will result in an elevation of 97.7 feet on the reconstructed curve (0.1 foot decrease).

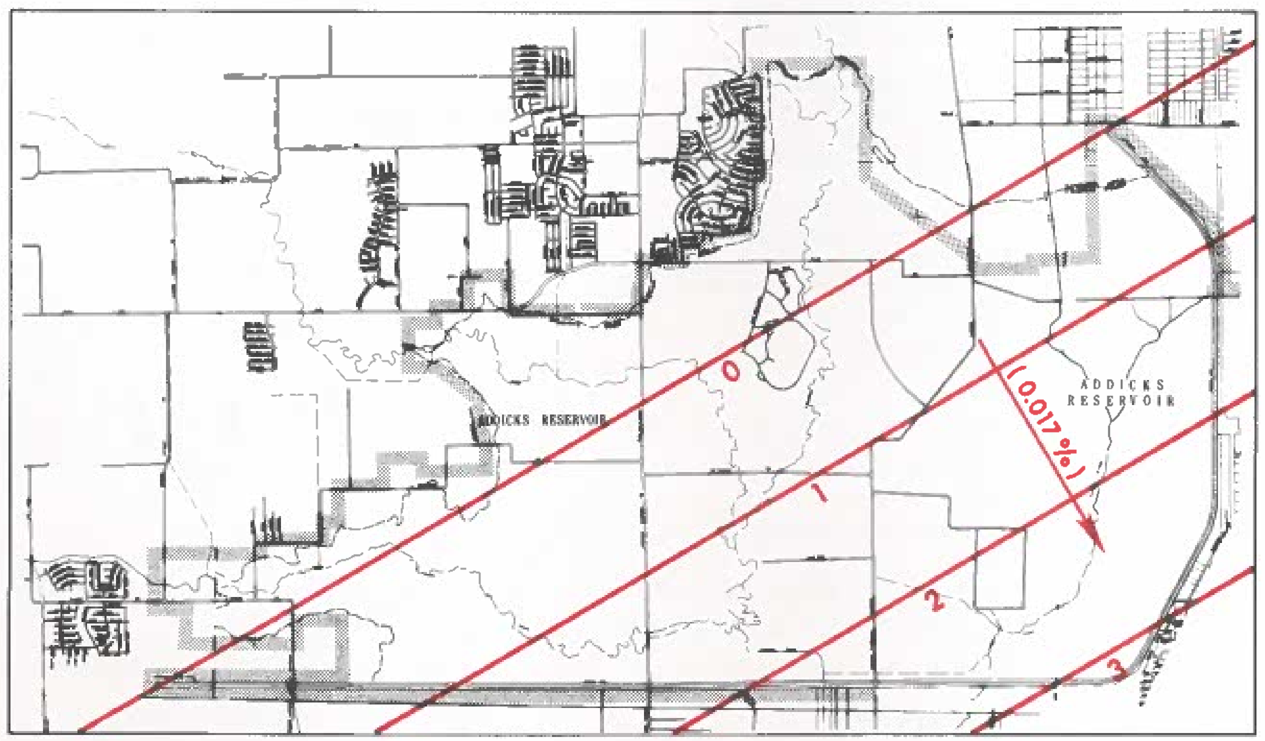
(6)A difference in maximum storage-capacity of -0.09 percent.

CASE 1R

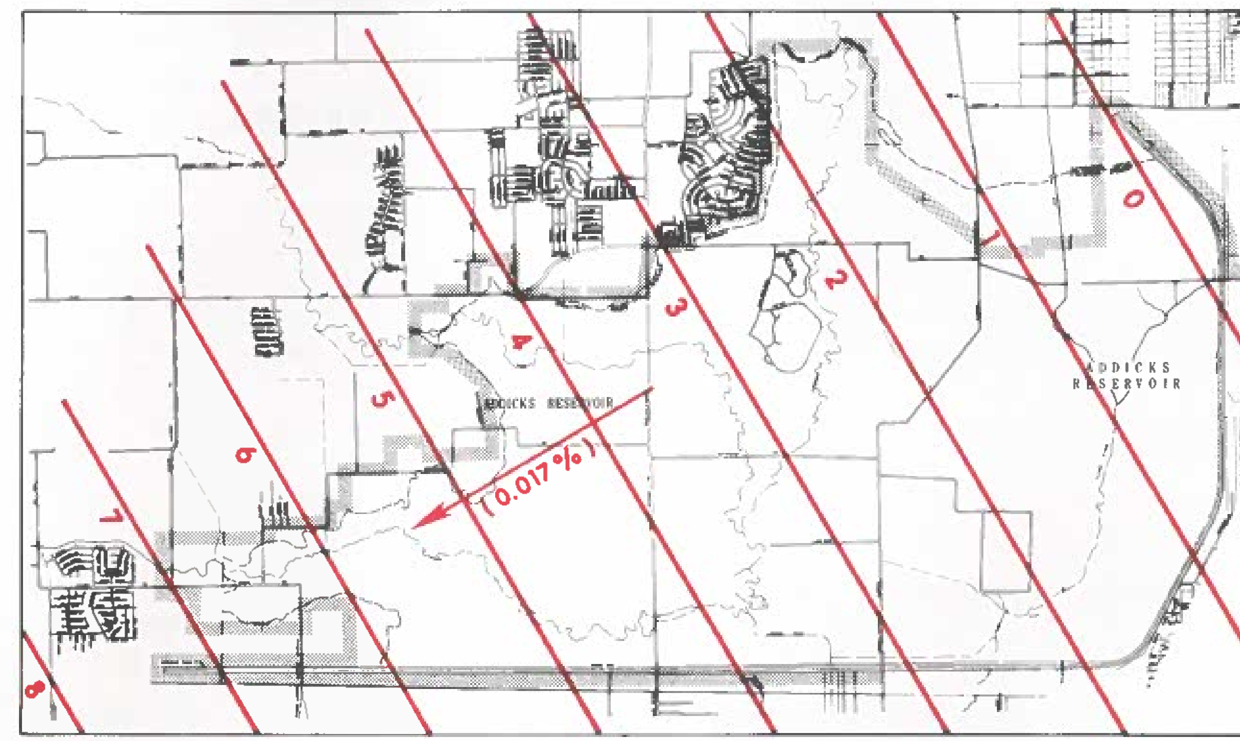


CASE 3R





CASE 2R



CASE 4R

LEGEND

LINE OF EQUAL SUBSIDENCE

5 AMOUNT OF SUBSIDENCE IN FEET

(0.017 %) GRADIENT CHANGE (SLOPE)

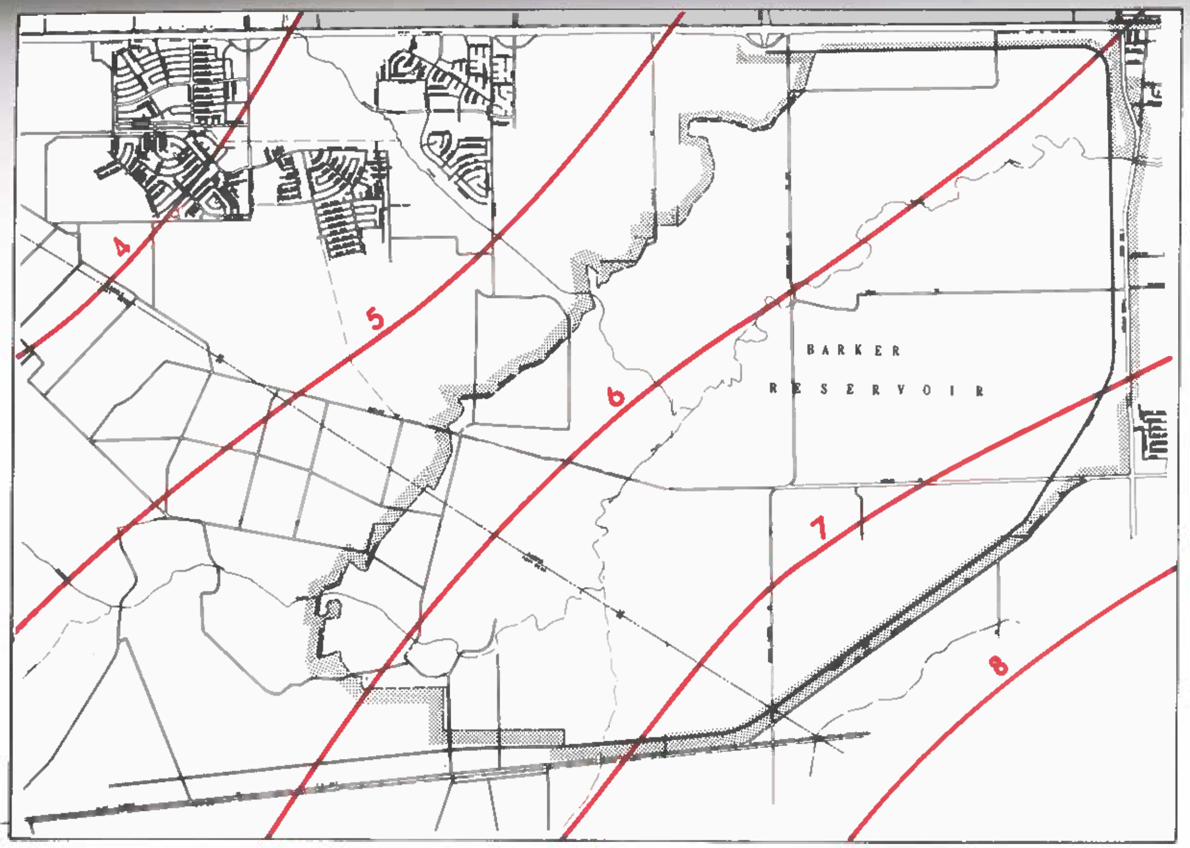
MAP SOURCE: COMPREHENSIVE STUDY OF HOUSTON'S MUNICIPAL WATER SYSTEM – PHASE III UPDATED, 1979.

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

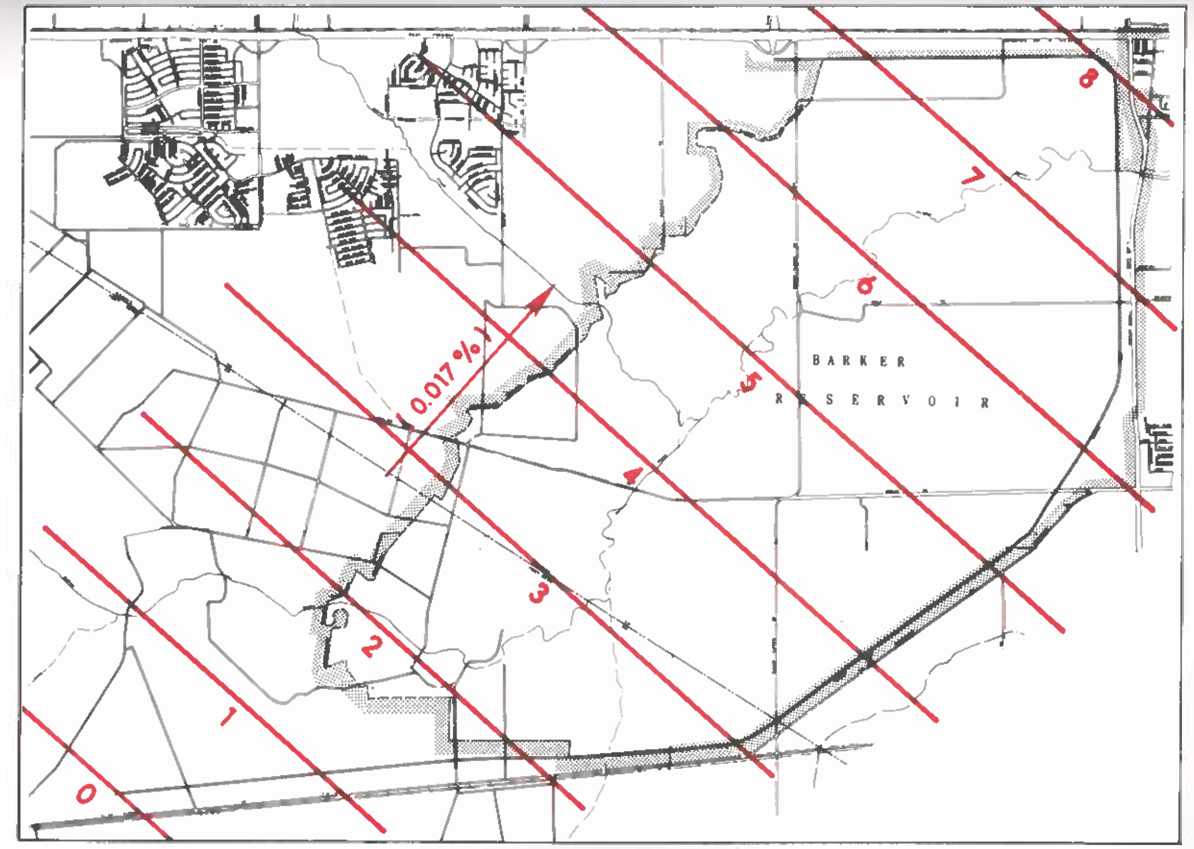
ADDICKS RESERVOIR SUBSIDENCE CASES

DECEMBER 1986

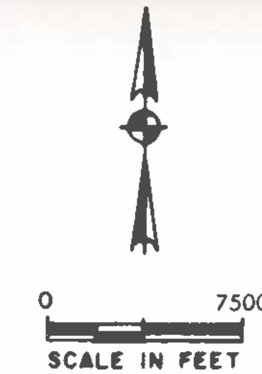
EXHIBIT IV–4



CASE 1R



CASE 3R

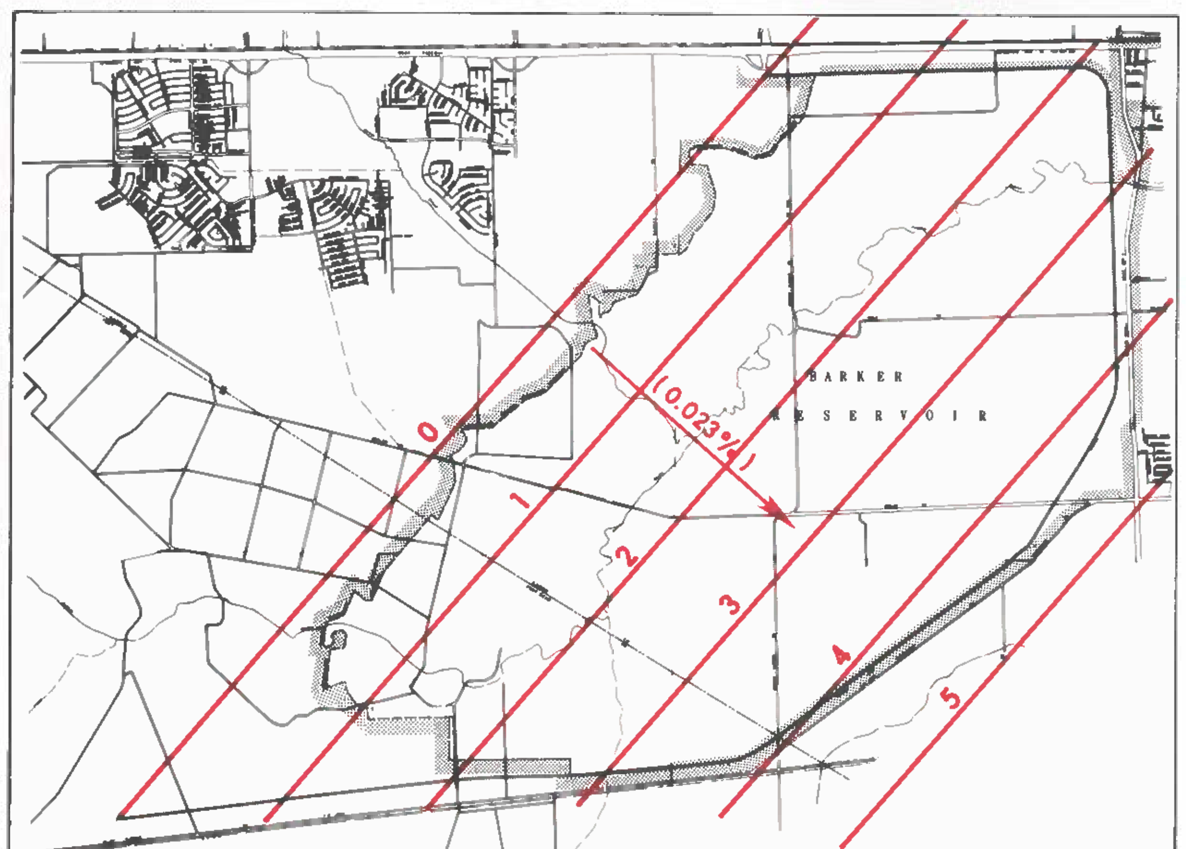


LEGEND

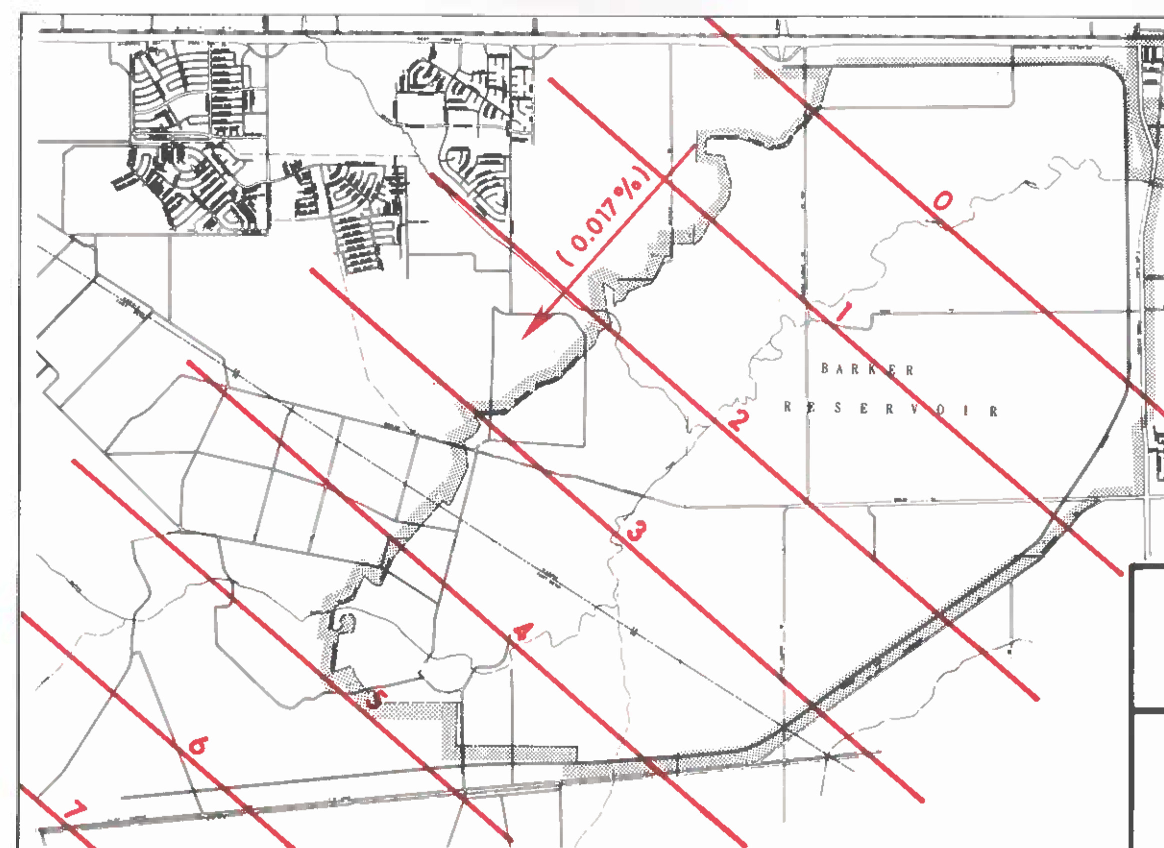
LINE OF EQUAL SUBSIDENCE

5 — AMOUNT OF SUBSIDENCE IN FEET

(0.017%) — GRADIENT CHANGE (SLOPE)



CASE 2R



CASE 4R

MAP SOURCE: COMPREHENSIVE STUDY OF HOUSTON'S MUNICIPAL WATER SYSTEM – PHASE III UPDATED, 1979.

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

BARKER RESERVOIR SUBSIDENCE CASES

DECEMBER 1986

EXHIBIT IV–5

TABLE IV-3 - REVISED STORAGE-CAPACITY CURVES, ADDICKS RESERVOIR

Subsidence Case 1R

Post-Subsidence Elevation (feet)[1]	Revised Storage-Capacity (acre-feet)
83	3,837
85	7,073
87	12,284
89	21,354
91	32,399
93	46,171
95	62,600
97	81,906
99	104,162
101	128,324
103	154,312
105	183,531
105.7[2]	194,303

Subsidence Case 2R

Post-Subsidence Elevation (feet)[1]	Revised Storage-Capacity (acre-feet)
87	3,432
88	4,657
90	7,358
92	13,884
94	23,075
96	33,983
98	47,514
100	63,560
102	82,365
104	104,235
106	132,072
108	153,754
110[3]	182,804
112	214,089

Subsidence Case 3R

Post-Subsidence Elevation (feet)[1]	Revised Storage-Capacity (acre-feet)
85	3,755
86	5,269
88	9,885
90	18,201
92	28,723
94	42,440
96	59,122
98	78,188
100	99,906
102	123,374
104.5[2]	155,510

Subsidence Case 4R

Post-Subsidence Elevation (feet)[1]	Revised Storage-Capacity (acre-feet)
86	2,547
88	4,254
90	10,788
92	18,471
94	29,792
96	43,116
98	58,893
100	77,347
102	99,823
104[4]	126,346
106	154,640

Notes:

[1] "National Geodetic Vertical Datum of 1929," 1973 adjustment and specific adjustments for the assumed subsidence cases.

[2] Location of Flood Control Crest - North end of dam.

[3] Location of Flood Control Crest - North and South ends of dam.

[4] Location of Flood Control Crest - Southwestern end of dam.

TABLE IV-4 - REVISED STORAGE-CAPACITY CURVES, BARKER RESERVOIR

Subsidence Case 1R

Post-Subsidence Elevation (feet)[1]	Revised Storage-Capacity (acre-feet)
80	3,762
81	5,225
83	10,369
85	21,283
87	38,941
89	59,488
91	82,171
93	106,805
95	133,582
97	162,195
99	192,497
100.4[2]	214,639

Subsidence Case 2R

Post-Subsidence Elevation (feet)[1]	Revised Storage-Capacity (acre-feet)
83	2,381
85	5,661
87	11,685
89	23,652
91	41,290
93	61,427
95	83,365
97	107,070
99	132,542
101	160,089
103	189,200
105	220,033
105.5[3]	227,978

Subsidence Case 3R

Post-Subsidence Elevation (feet)[1]	Revised Storage-Capacity (acre-feet)
78	2,052
79	3,180
81	6,550
83	11,403
85	18,690
87	31,304
89	48,300
91	68,562
93	90,335
95	114,199
97	139,972
99.5[4]	174,874

Subsidence Case 4R

Post-Subsidence Elevation (feet)[1]	Revised Storage-Capacity (acre-feet)
86	5,268
87	8,201
89	17,259
91	34,435
93	56,167
95	80,964
97	108,064
99.4[2]	143,046

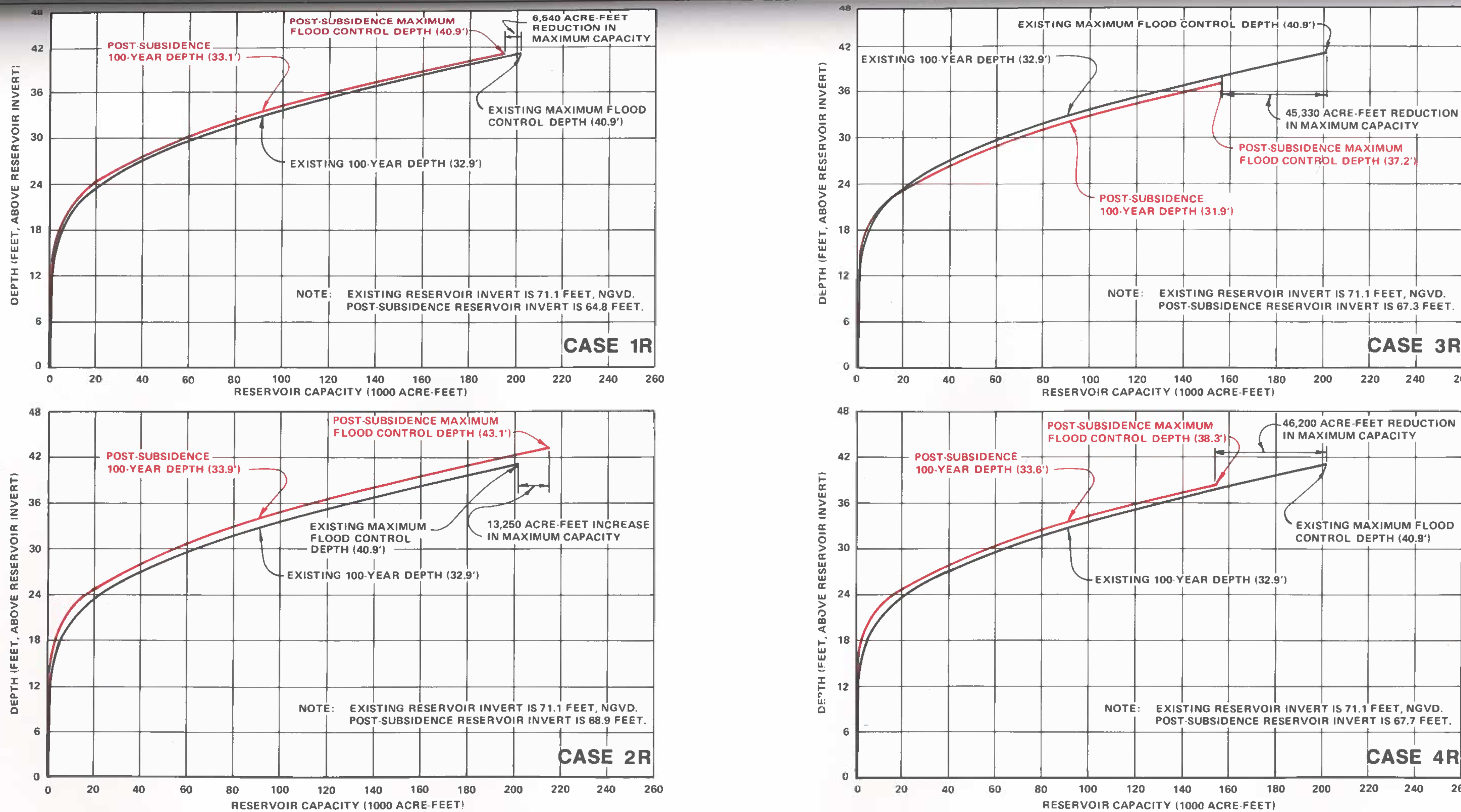
Notes:

[1] "National Geodetic Vertical Datum of 1929," 1973 adjustment and specific adjustments for the assumed subsidence cases.

[2] Location of Flood Control Crest - Southwestern end of dam.

[3] Location of Flood Control Crest - along Southeastern embankment.

[4] Location of Flood Control Crest - North end of dam.

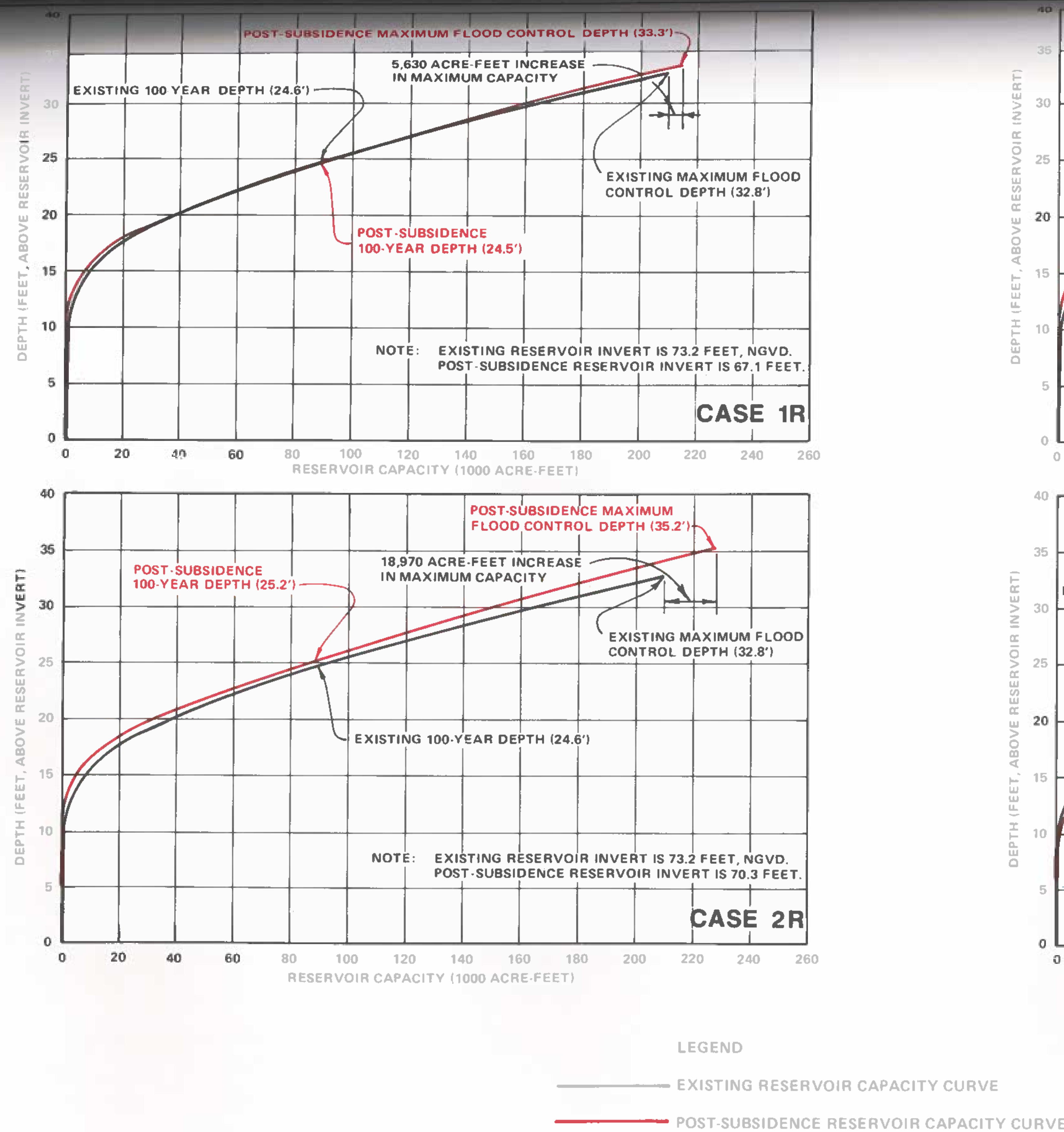


A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

ADDICKS RESERVOIR DEPTH CAPACITY CURVES

DECEMBER 1986

EXHIBIT IV-6



A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

BARKER RESERVOIR DEPTH CAPACITY CURVES

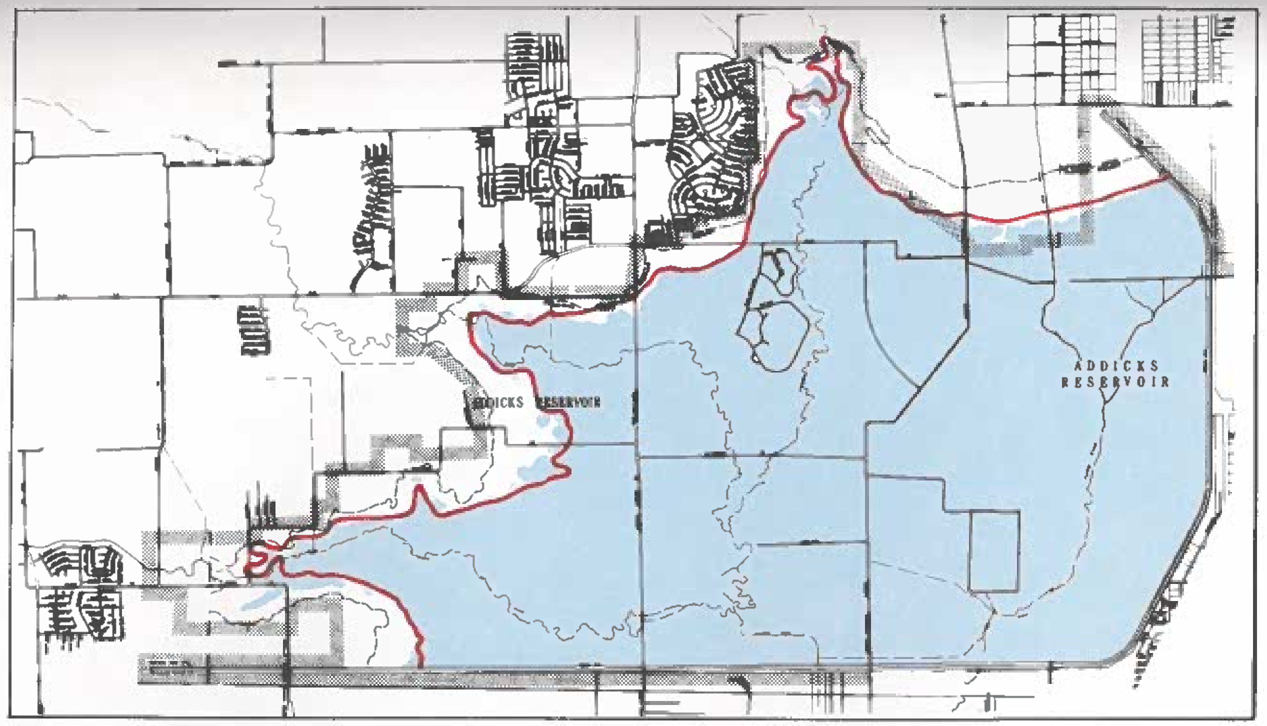
DECEMBER 1986

EXHIBIT IV-7

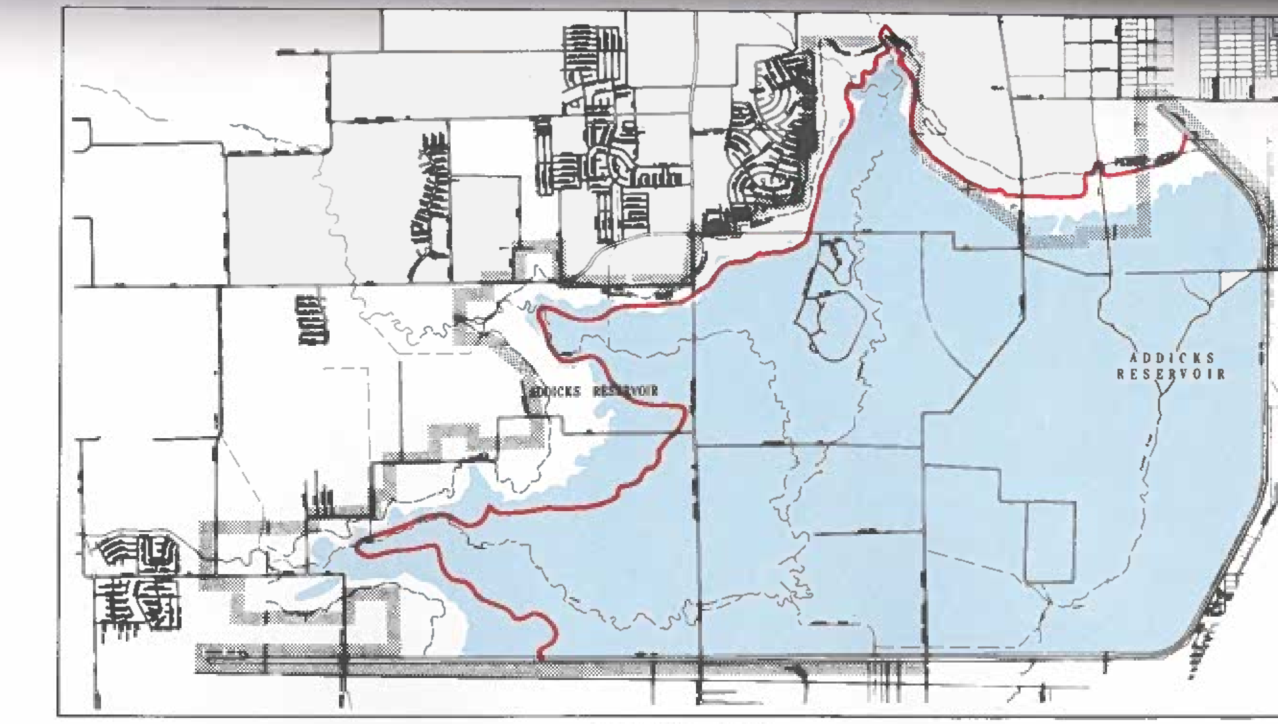
TABLE IV-5 - SUMMARY OF RESERVOIR STORAGE CHANGES

Condition	Maximum Storage (Acre-Feet)	Change in Maximum Storage (Acre-Feet)	Change in Maximum Storage (Percent)	Flooded Area (Acres)	Change in Flooded Area (Acres)	Change in Flooded Area (Percent)	Private Lands Flooded (Acres)	Change in Private Lands Flooded (Acres)	Change in Private Lands Flooded (Percent)
ADDICKS RESERVOIR									
Existing	200,840	0	0	11,470(1)	0	0	170	0	0
Case 1R	194,300	-6,540	-3.3	11,313	-157	-1.4	209	39(2)	22.9
Case 2R	214,090	13,250	6.6	10,974	-496	-4.3	144	-26(2)	-15.3
Case 3R	155,510	-45,330	-22.6	10,973	-497	-4.3	370	200(2)	117.6
Case 4R	154,640	-46,200	-23.0	11,510	40	0.4	2	-168(2)	-98.8
BARKER RESERVOIR									
Existing	209,010	0	0	12,681(3)	0	0	776	0	0
Case 1R	214,640	5,630	2.7	11,934	-747	-5.9	255	-521(4)	-67.1
Case 2R	227,980	18,970	9.1	11,521	-1,160	-9.2	180	-596(4)	-76.8
Case 3R	174,780	-34,140	-16.3	11,398	-1,283	-10.1	95	-681(4)	-87.8
Case 4R	143,050	-65,960	-31.6	13,210	529	4.2	1,453	677(4)	87.2

Notes:
(1) Government-owned Real Estate above Addicks Dam is approximately 12,972 acres.
(2) Located along the north boundary near Turkey Creek.
(3) Government-owned Real Estate above Barker Dam is approximately 11,886 acres.
(4) Located along the western boundary near Willow Fork Buffalo Bayou.

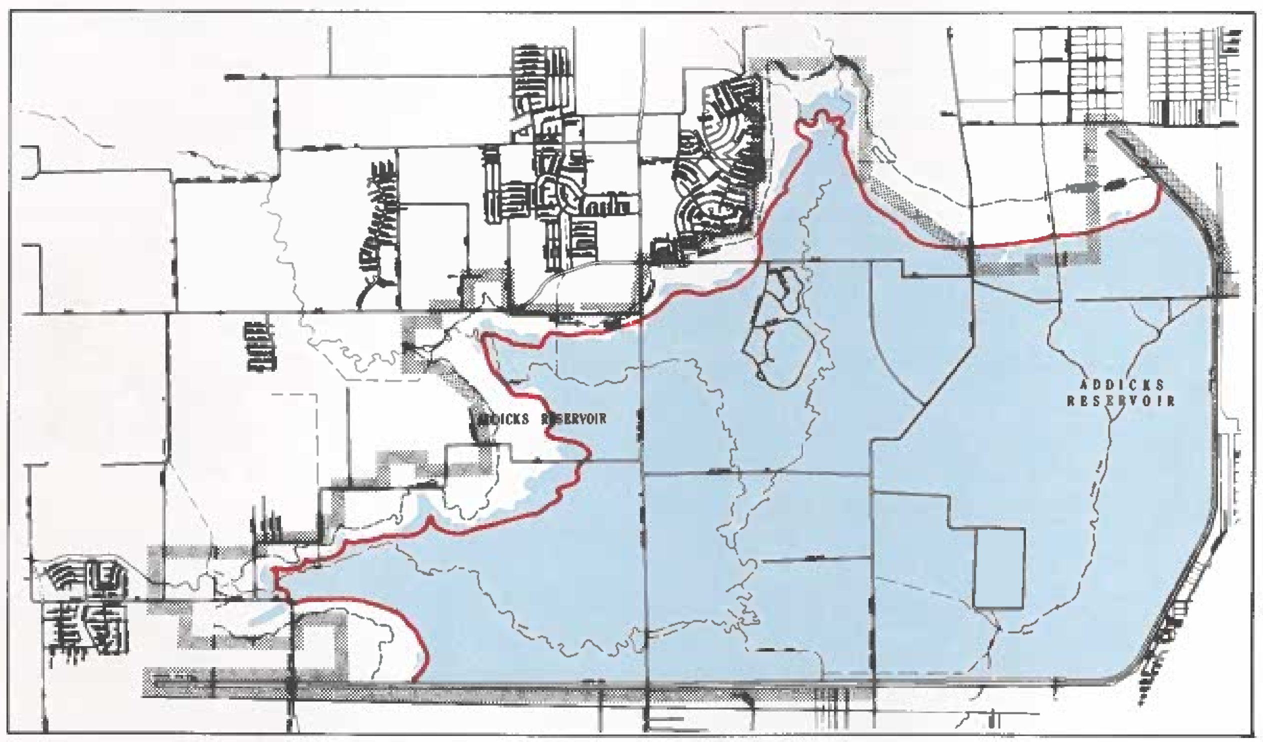


CASE 1R

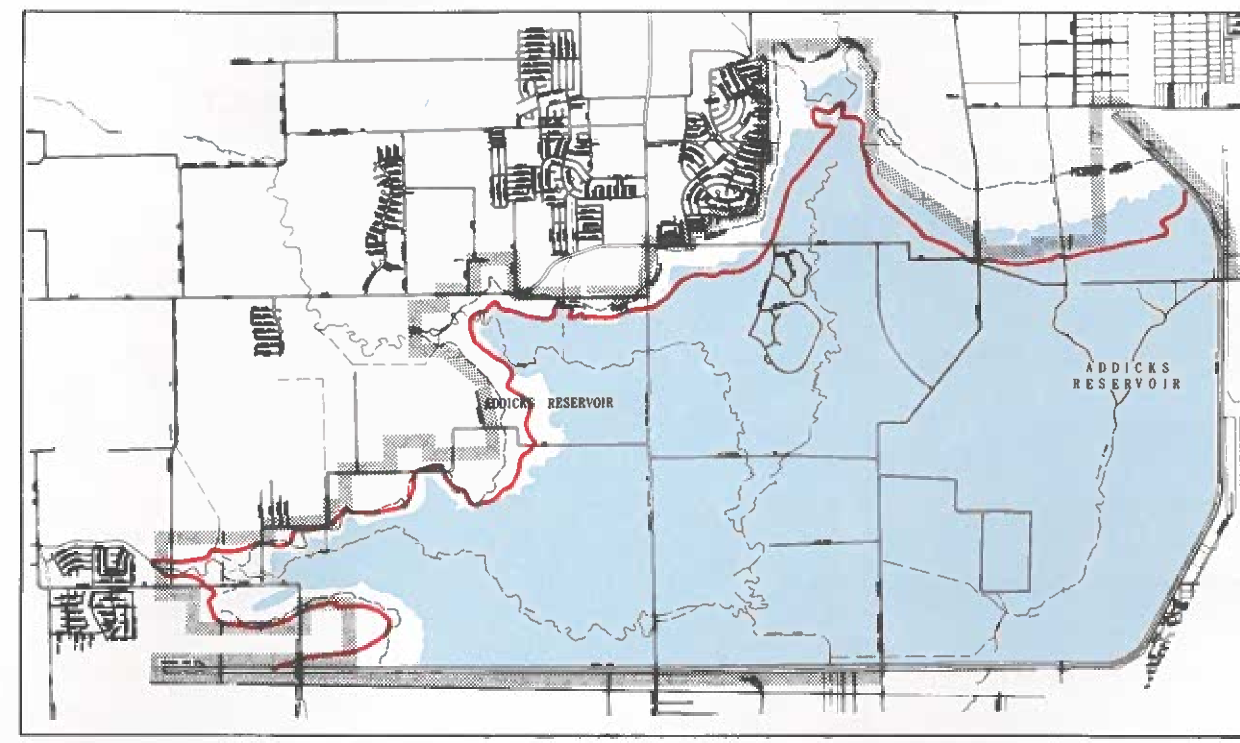


CASE 3R

0 7500
SCALE IN FEET



CASE 2R



CASE 4R

LEGEND

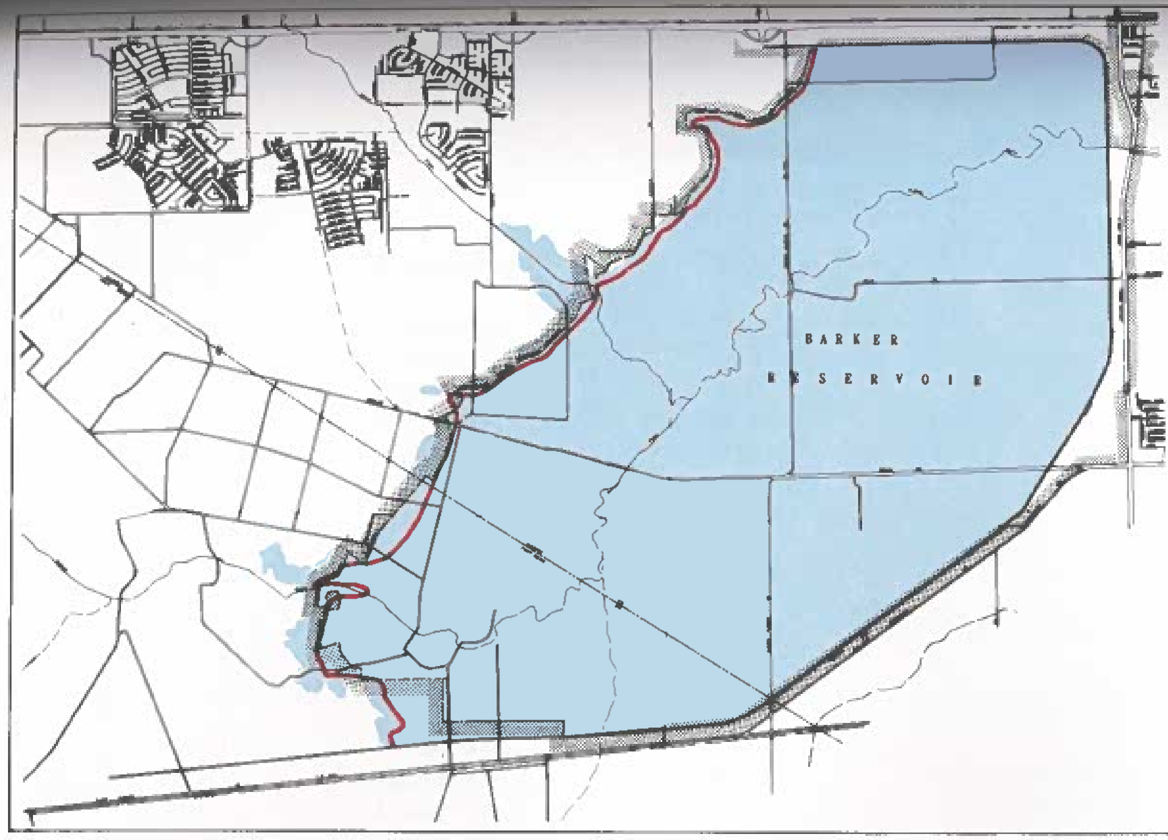
EXISTING 100-YEAR FLOOD POOL

POST SUBSIDENCE 100-YEAR FLOOD POOL

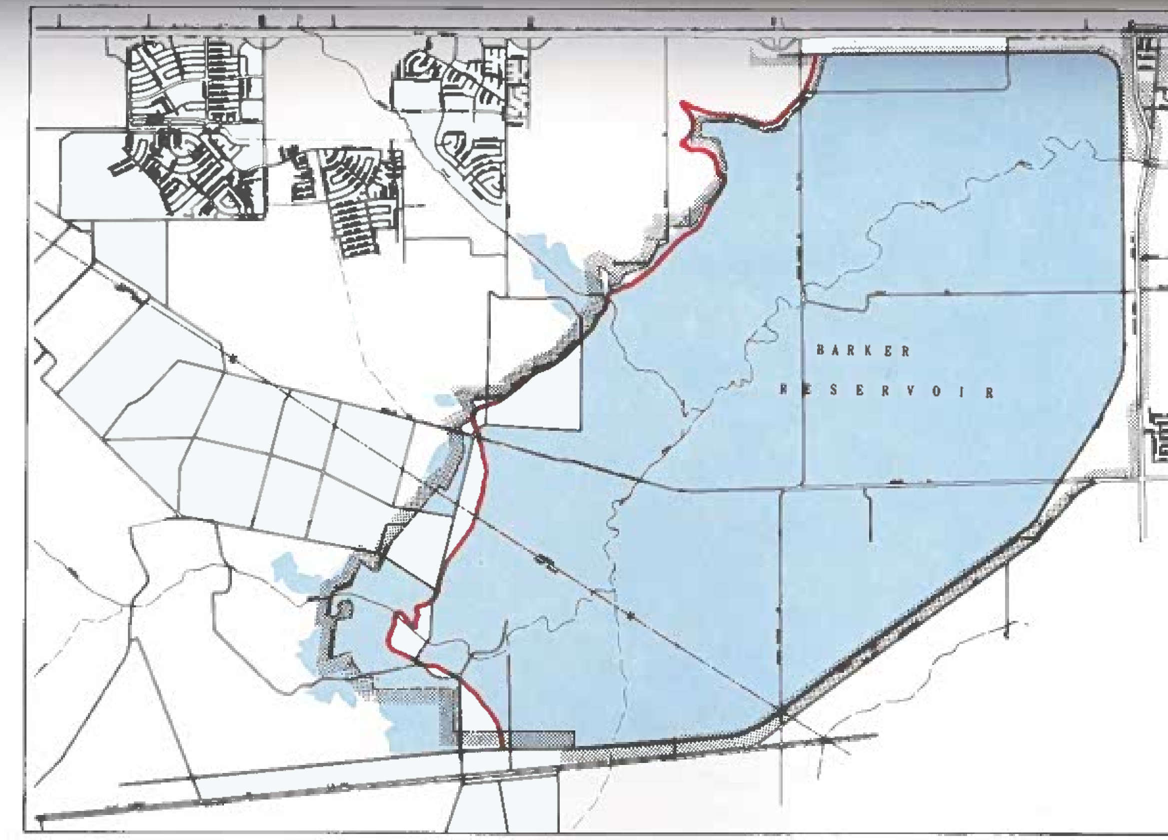
RESERVOIR BOUNDARY

MAP SOURCE: COMPREHENSIVE STUDY OF HOUSTON'S MUNICIPAL WATER SYSTEM – PHASE III UPDATED, 1979.

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING	
ADDICKS RESERVOIR 100-YEAR FLOOD POOLS	
DECEMBER 1986	EXHIBIT IV-8



CASE 1R



CASE 3R

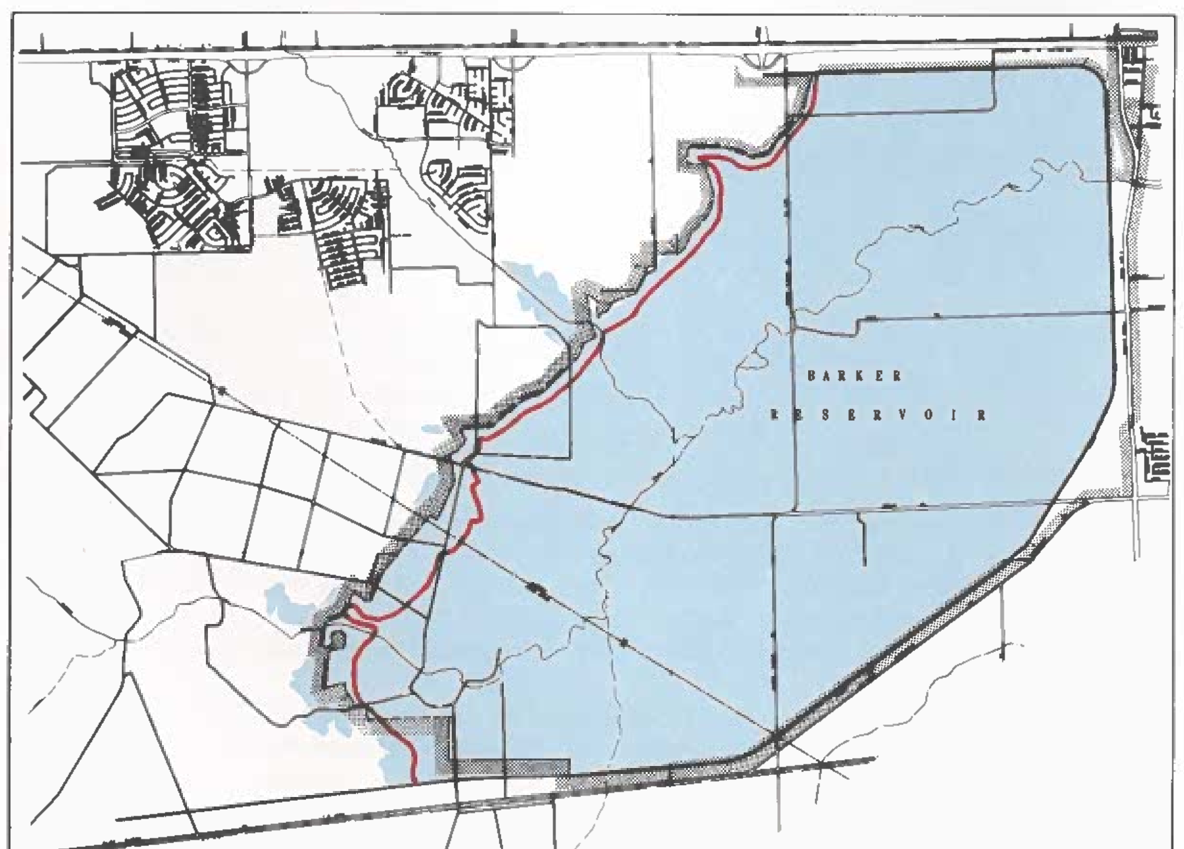
0 7500
SCALE IN FEET

LEGEND

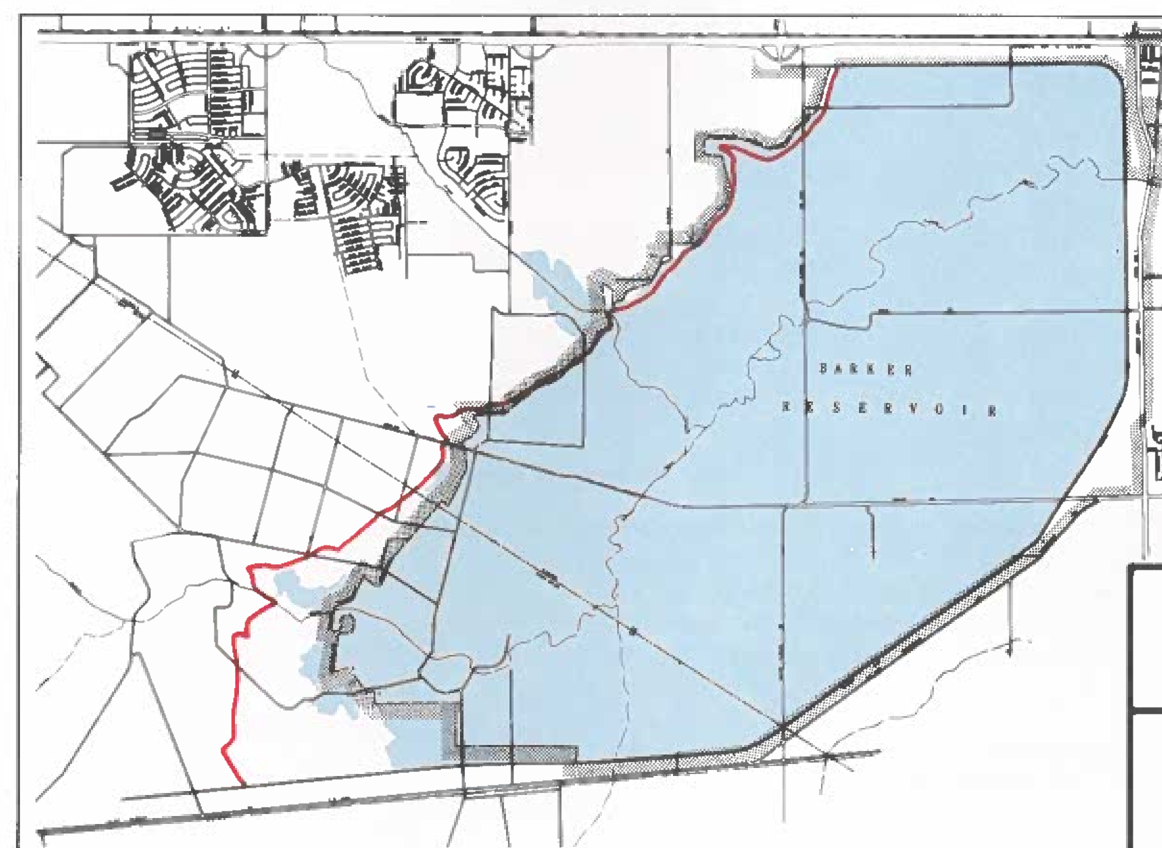
EXISTING 100-YEAR FLOOD POOL

POST SUBSIDENCE 100-YEAR FLOOD POOL

RESERVOIR BOUNDARY



CASE 2R



CASE 4R

MAP SOURCE: COMPREHENSIVE STUDY OF HOUSTON'S MUNICIPAL WATER SYSTEM – PHASE III UPDATED, 1979.

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

BARKER RESERVOIR 100-YEAR FLOOD POOLS

DECEMBER 1986

EXHIBIT IV-9

TABLE IV-6 - FREEBOARD COMPARISON, ADDICKS RESERVOIR

Embankment Location(1)	Condition	Embankment Freeboard (Feet above referenced storage level) Maximum Flood Control Storage Level(2)	100-Year Storage Level
1	Existing	1.0	9.0
	Case 1R	2.5	10.3
	Case 2R	1.0	10.2
	Case 3R	8.5	13.8
	Case 4R	0.0	4.7
2	Existing	5.0	13.0
	Case 1R	5.9	13.7
	Case 2R	4.2	13.4
	Case 3R	10.9	16.2
	Case 4R	5.1	9.8
3	Existing	8.0	16.0
	Case 1R	8.0	15.8
	Case 2R	5.8	15.0
	Case 3R	11.7	17.0
	Case 4R	10.6	15.3
4	Existing	9.5	17.5
	Case 1R	8.5	16.3
	Case 2R	6.3	15.5
	Case 3R	11.4	16.7
	Case 4R	13.8	18.5
5	Existing	4.0	12.0
	Case 1R	3.4	11.2
	Case 2R	2.6	11.8
	Case 3R	4.1	9.4
	Case 4R	9.9	14.6
6	Existing	0.0	8.0
	Case 1R	0.0	7.8
	Case 2R	0.0	9.2
	Case 3R	0.0	5.3
	Case 4R	6.0	10.7

Notes: (1) Reference locations are presented on Exhibit IV-10 and are described as follows: 1 - Southwestern end of dam, 2 - About 10,000 feet east of location 1 on low overflow embankment, 3 - Reservoir control structure, 4 - High overflow embankment about 10,000 feet east of Eldridge Road, 5 - Near Clay Road, 6 - North end of dam.

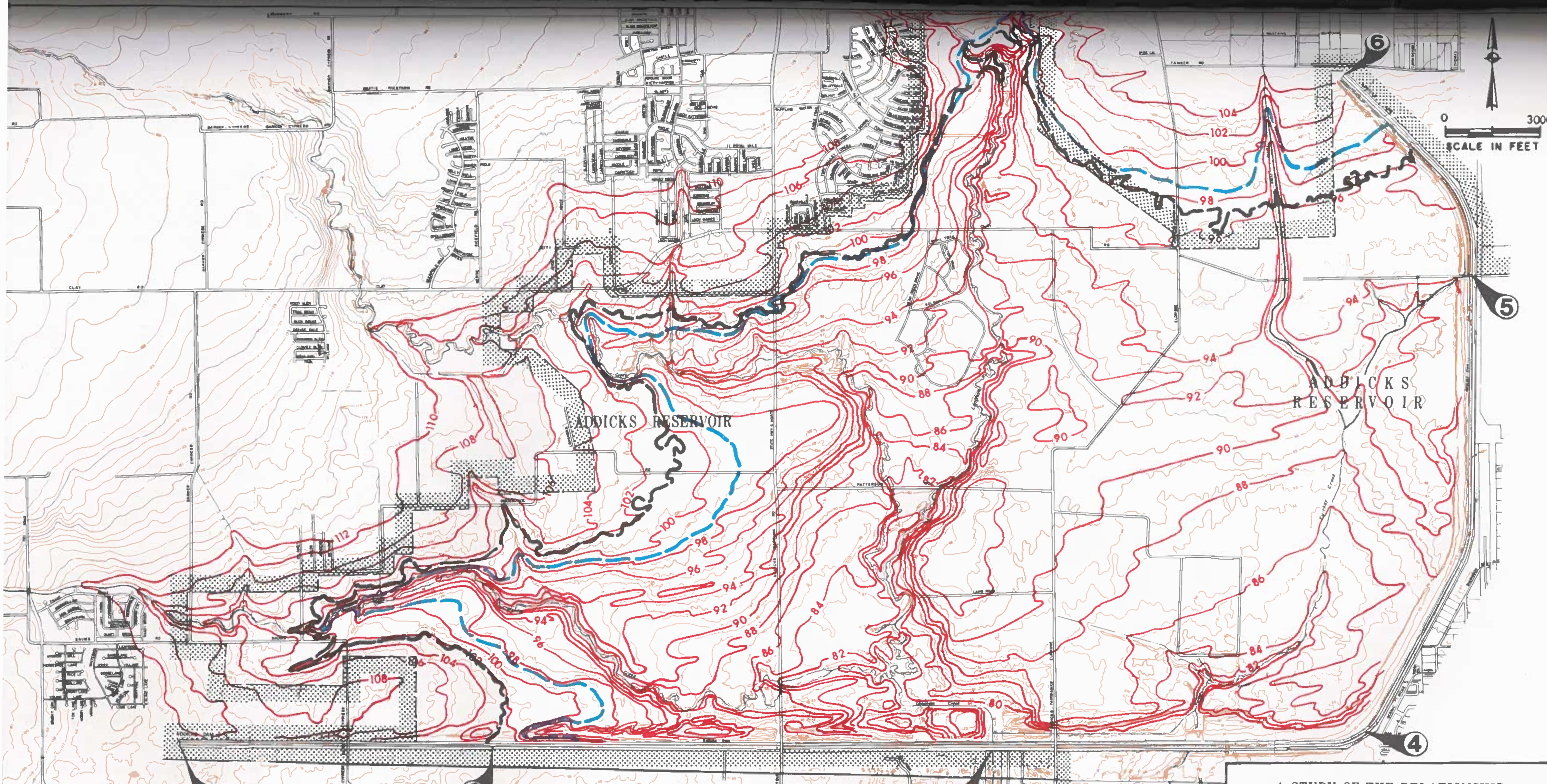
(2) The level at which flood waters begin to spill around the low reservoir ends is identified as the Maximum Flood Control Storage Level.

TABLE IV-7 - FREEBOARD COMPARISON, BARKER RESERVOIR

Embankment Location(1)	Condition	Embankment Freeboard (Feet above referenced storage level) Maximum Flood Control Storage Level(2)	100-Year Storage Level
1	Existing	0.0	8.2
	Case 1R	0.0	8.8
	Case 2R	0.5	10.5
	Case 3R	6.5	13.2
	Case 4R	0.0	3.8
2	Existing	2.0	10.2
	Case 1R	1.4	10.2
	Case 2R	1.0	11.0
	Case 3R	7.3	14.0
	Case 4R	3.4	7.2
3	Existing	4.0	12.2
	Case 1R	2.2	11.0
	Case 2R	0.0	10.0
	Case 3R	6.0	12.7
	Case 4R	8.6	12.4
4	Existing	4.0	12.2
	Case 1R	3.5	12.3
	Case 2R	1.6	11.6
	Case 3R	3.0	9.7
	Case 4R	10.5	14.3
5	Existing	4.0	12.2
	Case 1R	4.0	12.8
	Case 2R	3.0	13.0
	Case 3R	2.8	9.5
	Case 4R	10.6	14.4
6	Existing	0.0	8.2
	Case 1R	0.4	9.2
	Case 2R	0.5	10.5
	Case 3R	0.0	6.7
	Case 4R	6.6	10.4

Notes: (1) Reference locations are presented on Exhibit IV-11 and are described as follows: 1 - Southwestern End of Dam, 2 - About 10,000 feet east of location 1 on low overflow embankment, 3 - About 7,000 feet northeast of Barker-Clodine Road, 4 - Reservoir Control Structure, 5 - Road crossing about 8,000 feet east of Addicks-Clodine Road, 6 - North End of Dam.

(2) The level at which flood waters begin to spill around the low reservoir ends in identified as the Maximum Flood Control Storage Level.



LEGEND

HOUSTON CITY LIMITS

APPROXIMATE RESERVOIR BOUNDARY

100 EXISTING CONTOUR LINE

96 POST-SUBSIDENCE CONTOUR LINE

EXISTING LIMIT OF 100-YEAR RESERVOIR POOL (104 FT., NGVD)

POST SUBSIDENCE LIMIT OF 100-YEAR RESERVOIR POOL (99.2 FT., CASE 3R)

2 EMBANKMENT REFERENCE LOCATION

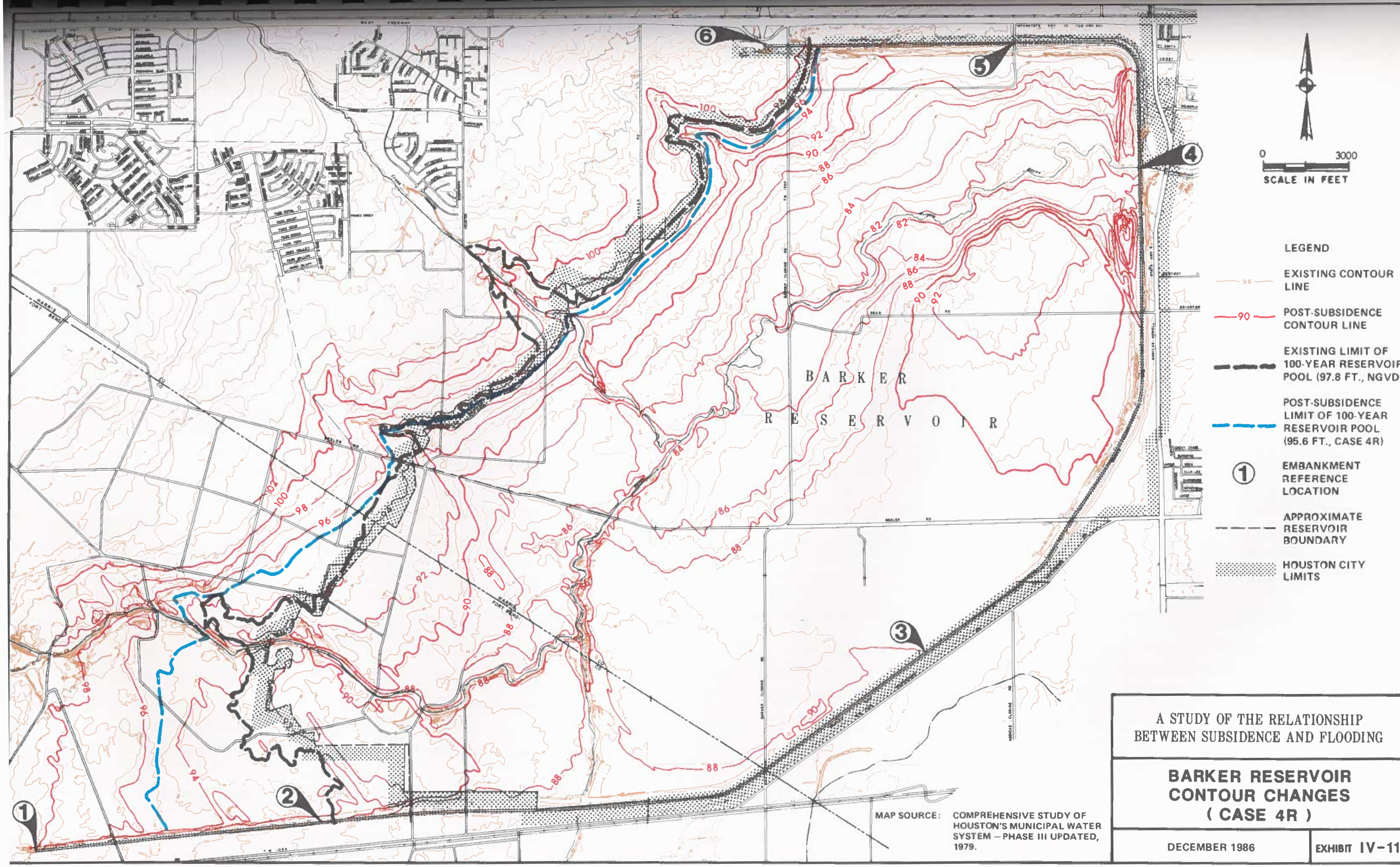
MAP SOURCE: COMPREHENSIVE STUDY OF HOUSTON'S MUNICIPAL WATER SYSTEM – PHASE III UPDATED, 1979.

A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING

ADDICKS RESERVOIR CONTOUR CHANGES (CASE 3R)

DECEMBER 1986

EXHIBIT IV–10



BARKER RESERVOIR
SCALE IN FEET
0
3000
LEGEND
EXISTING CONTOUR LINE
POST-SUBSIDENCE CONTOUR LINE
EXISTING LIMIT OF 100-YEAR RESERVOIR POOL (97.8 FT., NGVD)
POST-SUBSIDENCE LIMIT OF 100-YEAR RESERVOIR POOL (95.6 FT., CASE 4R)
EMBANKMENT REFERENCE LOCATION
APPROXIMATE RESERVOIR BOUNDARY
HOUSTON CITY LIMITS
MAP SOURCE: COMPREHENSIVE STUDY OF HOUSTON'S MUNICIPAL WATER SYSTEM – PHASE III UPDATED, 1979.
A STUDY OF THE RELATIONSHIP BETWEEN SUBSIDENCE AND FLOODING
BARKER RESERVOIR CONTOUR CHANGES (CASE 4R)
DECEMBER 1986
EXHIBIT IV-11

TABLE IV-8 – RESERVOIR EMBANKMENT ELEVATION COMPARISON, PRE-PROJECT VS. POST-PROJECT

Embankment Location	Condition	Pre-Project Embankment Elevation (feet) (1)	Post-Project Embankment Elevation (feet) (1)
Addicks Reservoir (2)			
1	Existing	113.0	113.0 (4)
	Case 1R	108.2	108.2
	Case 2R	113.0	113.0
	Case 3R	113.0	113.0
	Case 4R	106.0	106.0
2	Existing	117.0	117.0
	Case 1R	111.6	111.6
	Case 2R	116.2	116.2
	Case 3R	115.4	115.4
	Case 4R	111.1	111.1
3	Existing	120.0	122.7
	Case 1R	113.7	116.4
	Case 2R	117.8	120.5
	Case 3R	116.2	118.9
	Case 4R	116.6	119.3
4	Existing	121.5	121.5
	Case 1R	114.2	114.2
	Case 2R	118.3	118.3
	Case 3R	115.9	115.9
	Case 4R	119.8	119.8
5	Existing	116.0	116.0
	Case 1R	109.1	109.1
	Case 2R	114.6	114.6
	Case 3R	108.6	108.6
	Case 4R	115.9	115.9
6	Existing	112.0	112.0 (4)
	Case 1R	105.7	105.7
	Case 2R	112.0	112.0
	Case 3R	104.5	104.5
	Case 4R	112.0	112.0

Embankment Location	Condition	Pre-Project Embankment Elevation (feet) (1)	Post-Project Embankment Elevation (feet) (1)
Barker Reservoir (3)			
1	Existing	106.0	106.0 (4)
	Case 1R	100.4	100.4
	Case 2R	106.0	106.0
	Case 3R	106.0	106.0
	Case 4R	99.4	99.4
2	Existing	108.0	108.0 (4)
	Case 1R	101.8	101.8
	Case 2R	106.5	106.5
	Case 3R	106.8	106.8
	Case 4R	102.8	102.8
3	Existing	110.0	113.5
	Case 1R	102.6	106.1
	Case 2R	105.5	109.0
	Case 3R	105.5	109.0
	Case 4R	108.0	111.5
4	Existing	110.0	113.8
	Case 1R	103.9	107.7
	Case 2R	107.1	110.9
	Case 3R	102.5	106.3
	Case 4R	109.9	113.7
5	Existing	110.0	113.3
	Case 1R	104.4	107.7
	Case 2R	108.5	111.8
	Case 3R	102.3	105.6
	Case 4R	110.0	113.3
6	Existing	106.0	106.0 (4)
	Case 1R	100.8	100.8
	Case 2R	106.0	106.0
	Case 3R	99.5	99.5
	Case 4R	106.0	106.0

Notes: (1) "National Geodetic Vertical Datum of 1929," 1973 adjustment and specific adjustments for the assumed subsidence cases.

(2) Reference Table IV-6 for description of embankment locations for Addicks Reservoir.

(3) Reference Table IV-7 for description of embankment locations for Barker Reservoir.

(4) Embankments for all cases are armor-plated.

TABLE IV-9 - FREEBOARD COMPARISON WITH PROPOSED MODIFICATIONS

Embankment Location	Condition	Post-Project Embankment Freeboard (Feet Above Referenced Storage Level): Maximum Flood Control Storage level	100-Year Storage Level
Addicks Dam - 3(1)	Existing	10.7	18.7
	Case 1R	10.7	18.5
	Case 2R	8.5	17.7
	Case 3R	14.4	19.7
	Case 4R	13.3	18.0
Barker Dam - 3(2)	Existing	7.5	15.7
	Case 1R	5.7	14.5
	Case 2R	3.0	13.5
	Case 3R	9.5	16.2
	Case 4R	12.1	15.9
4(2)	Existing	7.8	16.0
	Case 1R	7.3	16.1
	Case 2R	4.9	15.4
	Case 3R	6.8	13.5
	Case 4R	14.3	18.1
5(2)	Existing	7.3	15.5
	Case 1R	7.3	16.1
	Case 2R	5.8	16.3
	Case 3R	6.1	12.8
	Case 4R	13.9	17.7

Notes: (1) Reference Table IV-6 for location description.

(2) Reference Table IV-7 for location description.

TABLE IV-10 - SPILLWAY DESIGN FLOOD FREEBOARD COMPARISON, ADDICKS RESERVOIR

Embankment Location*	Condition	Spillway Design Flood Freeboard: Pre-Project (feet)	Post-Project (feet)
1	Existing	-5.1	-5.1
	Case 1R	-4.2	-4.2
	Case 2R	-4.4	-4.4
	Case 3R	-2.6	-2.6
	Case 4R	-10.1	-10.1
2	Existing	-1.1	-1.1
	Case 1R	-0.8	-0.8
	Case 2R	-1.2	-1.2
	Case 3R	-0.2	-0.2
	Case 4R	-5.0	-5.0
3	Existing	1.9	4.6
	Case 1R	1.3	4.0
	Case 2R	0.4	3.1
	Case 3R	0.6	3.3
	Case 4R	0.5	3.2
4	Existing	3.4	3.4
	Case 1R	1.8	1.8
	Case 2R	0.9	0.9
	Case 3R	0.3	0.3
	Case 4R	3.7	3.7
5	Existing	-2.1	-2.1
	Case 1R	-3.3	-3.3
	Case 2R	-2.8	-2.8
	Case 3R	-7.0	-7.0
	Case 4R	-0.2	-0.2
6	Existing	-6.1	-6.1
	Case 1R	-6.7	-6.7
	Case 2R	-5.4	-5.4
	Case 3R	-11.1	-11.1
	Case 4R	-4.1	-4.1

*Reference Table IV-6 for description of embankment locations.

TABLE IV-11 - SPILLWAY DESIGN FLOOD FREEBOARD COMPARISON, BARKER RESERVOIR

Embankment Location*	Condition	Spillway Design Flood Freeboard Pre-Project (feet)	Spillway Design Flood Freeboard Post-Project (feet)
1	Existing	-4.3	-4.3
	Case 1R	-4.1	-4.1
	Case 2R	-2.8	-2.8
	Case 3R	-1.0	-1.0
	Case 4R	-9.4	-9.4
2	Existing	-2.3	-2.3
	Case 1R	-2.7	-2.7
	Case 2R	-2.3	-2.3
	Case 3R	-0.2	-0.2
	Case 4R	-6.0	-6.0
3	Existing	-0.3	3.2
	Case 1R	-1.9	1.6
	Case 2R	-3.3	0.2
	Case 3R	-1.5	2.0
	Case 4R	-0.8	2.7
4	Existing	-0.3	3.5
	Case 1R	-0.6	3.2
	Case 2R	-1.7	2.1
	Case 3R	-4.5	-0.7
	Case 4R	1.1	4.9
5	Existing	-0.3	3.0
	Case 1R	-0.1	3.2
	Case 2R	-0.3	3.0
	Case 3R	-4.7	-1.4
	Case 4R	1.2	4.5
6	Existing	-4.3	-4.3
	Case 1R	-3.7	-3.7
	Case 2R	-2.8	-2.8
	Case 3R	-7.5	-7.5
	Case 4R	-2.8	-2.8

*Reference IV-7 for description of embankment locations.

TECHNICAL APPENDIX

TECHNICAL APPENDIX

This technical appendix is included to complete the documentation of the basic data used for the riverine flooding analysis (Chapter II) and to present the detailed results of the analysis which are not presented in the text. As mentioned in the text, the number of subsidence cases tested combined with the number of channel systems evaluated resulted in very long, voluminous tables of data. To avoid confusing the reader, the tables referenced by the text reflected only selected cases on selected channels and the full extent of the data is included in this technical appendix for reference.

A brief description of each table follows:

- Table A-1 presents the existing condition basin characteristics and unitgraph coefficients used in this analysis (discussed in the Technical Approach section of Chapter II).

- Table A-2 presents the existing condition storage-discharge information for each watershed.

- Table A-3 presents the characteristics of the simulated channel on Willow Fork according to HCFCD design criteria (discussed in the Analysis of Riverine Subsidence section under Selection of Subsidence Cases).

- Table A-4 presents the storage-discharge information for the 100-year design channel on Willow Fork.

- Table A-5 presents the existing condition storm flows for each watershed.

- Tables A-6 through A-15 present the revised storage-discharge relations for each watershed.

- Tables A-16 through A-26 present the revised storm flows for each watershed (discussed in the Analysis of Riverine Subsidence section under Subsidence and its Effect on Storm Flows).

- Tables A-27 through A-36 present the hydraulic carrying capacity for each watershed (discussed in the Analysis of Riverine Subsidence section under Subsidence and its Effect on Storm Flows).

- Tables A-37 through A-46 present the flood plain area data for each watershed (discussed in the Analysis of Riverine Subsidence section under Subsidence and its Effect on Flood Plain Area).

TABLE A-1 - EXISTING BASIN CHARACTERISTICS AND UNITGRAPH COEFFICIENTS

Sub-Watershed	Drainage Area (mi^2)	Length (mi)	Length to Centroid (mi)	Channel Slope (ft/mi)	Overland Slope (ft/mi)	Ponding Area (%)	Urban Development (%)	Channel Conveyance (%)	Channel Improvement (%)	Unitgraph Coefficients 10-Yr. Tc	R	100-Yr. Tc	R
Langham Creek													
U100A	1.01	1.99	0.97	3.02	10.0	85	0	100	100	0.75	22.40	0.75	15.72
U100B	6.33	5.23	2.61	3.59	10.0	85	0	17	17	3.20	36.09	3.20	25.34
U100C	6.23	5.85	2.84	6.52	10.0	75	4	85	85	1.72	34.15	1.72	24.17
U120A	3.38	3.86	1.76	5.68	10.0	75	5	62	62	1.29	26.90	1.29	19.04
U120B	1.87	3.10	1.62	10.80	10.0	6	6	90	90	0.70	11.45	0.70	9.60
U100D	5.21	4.38	2.16	3.09	10.0	0	18	100	100	1.63	12.18	1.63	12.18
U100E	1.77	2.84	1.48	3.33	10.0	0	32	77	77	1.19	6.26	1.19	6.26
U100F	2.24	2.19	1.08	3.23	10.0	0	45	25	25	1.04	15.69	1.04	15.69
Horsepen Creek													
U106A	4.27	3.58	1.59	3.62	10.0	85	0	20	20	1.86	29.35	1.86	20.60
U106B	4.89	4.38	1.19	7.06	10.0	84	2	25	25	0.93	29.02	0.93	20.38
U106C	3.71	3.41	1.25	12.47	10.0	0	20	100	100	0.43	5.29	0.43	5.29
U106D	2.95	3.64	2.22	4.32	10.0	0	46	100	100	1.27	3.61	1.27	3.61
U106E	2.41	2.50	1.22	3.88	10.0	0	27	100	100	1.23	5.12	1.23	5.12
Bear Creek													
U102A	6.88	4.18	1.93	2.91	10.0	85	0	43	43	2.28	35.23	2.28	24.73
U102B	6.96	4.32	2.16	5.26	10.0	85	4	55	55	1.74	29.73	1.74	20.87
U102C	10.03	5.63	2.95	4.38	10.0	29	3	52	52	2.72	29.67	2.72	22.38
U102D	1.99	3.30	1.76	5.81	10.0	0	32	40	40	1.35	11.48	1.35	11.48
U10201A	2.77	3.35	1.48	8.33	10.0	0	28	100	100	0.62	4.51	0.62	4.51
U102E	2.71	2.84	1.25	1.36	10.0	0	39	25	25	2.17	24.50	2.17	24.50
S. Mayde Creek													
U101A1	1.07	1.42	0.68	6.03	10.0	85	0	100	0	0.60	13.30	0.60	9.30
U101B1	3.15	2.59	0.80	2.48	10.0	85	0	100	0	1.20	28.20	1.20	19.80
U101B2	2.30	2.64	1.36	5.89	10.0	85	0	100	0	1.30	19.70	1.30	13.80
U101C1	5.17	2.41	1.01	4.84	10.0	85	3	100	0	1.10	20.60	1.10	14.50
U101C2	2.87	3.78	1.65	4.92	10.0	85	0	100	0	1.80	27.20	1.80	19.10
U101D1	1.16	1.02	0.51	9.18	10.0	85	0	100	0	0.40	9.20	0.40	6.50
U101D2	4.66	5.28	2.51	5.07	10.0	85	1	100	0	2.80	32.50	2.80	22.80
U101E1	0.89	0.85	0.40	18.02	10.0	0	5	100	0	0.20	2.10	0.20	2.10
U10107A1	0.88	1.45	0.67	5.96	10.0	85	0	100	0	0.60	13.60	0.60	9.60
U10107A2	1.07	1.05	0.48	5.61	10.0	85	0	100	0	0.50	11.30	0.50	7.90
U10107A3	2.34	1.00	0.53	4.84	10.0	85	0	100	0	1.50	22.30	1.50	15.70
U101F1	1.21	1.36	0.68	9.90	10.0	25	3	100	0	0.50	8.60	0.50	6.50
U101F2	1.37	1.89	1.05	5.28	10.0	5	0	100	50	0.80	9.70	0.80	8.20
U101G1	0.96	1.85	0.77	9.15	10.0	0	6	100	0	0.60	4.60	0.60	4.60
U101G2	1.83	1.31	0.56	3.42	10.0	0	37	100	50	0.50	2.90	0.50	2.90

TABLE A-1 (Cont'd)

Sub-Watershed	Drainage Area (mi^2)	Length (mi)	Length to Centroid (mi)	Channel Slope (ft/mi)	Overland Slope (ft/mi)	Ponding Area (%)	Urban Development (%)	Channel Conveyance (%)	Channel Improvement (%)	Unitgraph Coefficients 10-Yr. Tc	Unitgraph Coefficients 10-Yr. R	Unitgraph Coefficients 100-Yr. Tc	Unitgraph Coefficients 100-Yr. R
S. Mayde Creek (Cont'd)													
U101H1	2.80	2.01	1.02	5.28	10.0	0	15	100	50	0.80	5.80	0.80	5.80
U101H2	0.97	0.89	0.42	3.02	10.0	0	33	100	50	0.40	2.50	0.40	2.50
U101I1	3.25	2.61	2.05	2.70	10.0	0	0	100	0	3.10	6.90	3.10	6.90
U101I2	2.16	3.30	2.36	5.87	10.0	0	3	100	0	2.40	6.60	2.40	6.60
Mason Creek													
T101A	3.26	3.83	1.52	3.8	10.0	50	18	100	0	1.83	26.38	1.83	19.19
T10109A	0.87	2.27	1.14	4.6	10.0	0	25	100	100	0.65	5.19	0.65	5.19
T101B	2.87	2.27	1.44	3.1	10.0	30	26	100	100	1.03	13.91	1.03	10.47
T10103A1	1.17	2.00	1.15	5.8	10.0	0	74	100	21	0.86	9.83	0.86	9.83
T10103A2	0.91	1.48	0.81	2.5	10.0	0	0	100	0	1.18	5.61	1.18	5.61
T10103A3	0.40	1.59	0.89	3.3	10.0	0	0	100	0	1.16	5.46	1.16	5.46
T101C	1.40	2.31	1.13	2.7	10.0	0	23	100	100	0.87	6.84	0.87	6.84
T10107A	3.46	3.47	1.86	5.0	10.0	0	48	100	100	0.97	3.99	0.97	3.99
T101D	2.81	1.95	1.04	2.7	10.0	0	27	100	100	0.79	5.25	0.79	5.25
T101E	1.77	1.74	0.91	3.8	10.0	0	0	100	0	1.09	5.58	1.09	5.58
Willow Fork													
T100#1	3.79	3.05	1.92	2.20	10.0	15	0	100	0	3.23	19.37	3.23	15.27
T100#2	2.19	1.23	0.67	2.37	10.0	20	0	100	0	1.02	12.01	1.02	9.29
T100#3	4.53	2.72	1.51	1.55	10.0	35	0	100	0	3.02	24.85	3.02	18.51
T100#4	0.43	1.51	1.00	4.00	10.0	30	19	100	0	1.14	11.16	1.14	8.40
T100#5	1.98	1.84	1.01	2.50	10.0	25	0	100	0	1.53	15.88	1.53	12.10
T104#1	2.53	2.98	1.79	4.32	10.0	25	53	100	0	1.89	6.01	1.89	4.58
T104#2	6.38	4.00	1.69	4.87	10.0	80	4	100	0	1.84	28.14	1.84	19.84
T104#6	5.01	2.85	2.27	7.25	10.0	45	2	100	0	2.05	19.90	2.05	14.57
T100#7	1.24	1.07	0.47	8.78	10.0	40	0	100	0	0.35	8.49	0.35	6.27
T100#8	1.46	2.08	0.43	5.22	10.0	15	11	100	0	0.41	13.99	0.41	11.03
T100#9	2.36	3.60	1.02	10.00	10.0	33	13	100	0	0.72	18.54	0.72	13.86
T106#1	4.06	4.36	2.01	6.73	10.0	89	5	100	0	1.86	26.89	1.86	18.82
CI-1	1.20	2.08	0.91	15.10	10.0	0	29	35	0	0.50	8.55	0.50	8.55
WF-1A	0.53	1.63	0.61	14.35	10.0	0	5	21	0	0.35	3.65	0.35	3.65
S-1	0.31	1.02	0.45	17.08	10.0	0	7	100	0	0.23	2.47	0.23	2.47
S-1A	0.97	2.56	1.46	6.47	10.0	1	0	100	0	1.37	7.57	1.37	7.16
WF-1	2.02	3.28	1.24	11.96	10.0	9	2	49	0	0.83	6.16	0.83	6.16
WF-1B	4.97	4.44	2.14	6.61	10.0	22	0	100	0	2.03	20.45	2.03	15.72
CI-2	7.20	6.31	2.50	8.12	10.0	4	12	8	0	2.09	17.90	2.09	15.42
S-2	3.21	3.03	1.63	11.48	10.0	0	2	20	0	1.13	5.57	1.13	5.57
WF-2	8.32	6.05	2.88	3.96	10.0	4	1	21	0	3.63	20.68	3.63	17.82
S-3	2.49	3.91	1.67	4.19	10.0	26	4	10	0	1.97	23.21	1.97	17.64
CI-3	8.05	4.89	3.60	3.68	10.0	14	5	4	0	4.75	20.10	4.75	15.92
S-2A	1.39	2.27	1.04	6.04	10.0	0	0	100	0	0.99	5.87	0.99	5.87
CI-4	9.69	6.67	2.58	5.18	10.0	10	4	2	0	2.79	25.70	2.79	20.83
WF-3	5.03	5.11	1.52	3.92	10.0	12	0	10	0	1.86	25.82	1.86	20.66

TABLE A-1 (Cont'd)

Sub-Watershed	Drainage Area (mi^2)	Length (mi)	Length to Centroid (mi)	Channel Slope (ft/mi)	Overland Slope (ft/mi)	Ponding Area (%)	Urban Development (%)	Channel Conveyance (%)	Channel Improvement (%)	Unitgraph Coefficients 10-Yr. Tc	10-Yr. R	100-Yr. Tc	100-Yr. R
Brays Bayou													
D100#10	6.64	4.05	2.27	3.02	10.0	0	62	97	88	1.640	4.060	1.640	4.060
D100#9	1.76	2.42	1.29	2.35	10.0	0	14	94	100	1.110	9.220	1.110	9.220
D100#8	6.11	4.70	2.56	3.31	10.0	0	39	95	100	1.740	6.820	1.740	6.820
D100#7	2.11	2.08	0.83	3.22	10.0	0	28	100	100	0.560	5.240	0.560	5.240
D100#6	2.48	3.90	1.78	5.54	10.0	0	73	100	100	0.790	3.110	0.790	3.110
D100#5	3.16	3.22	1.70	3.37	10.0	0	39	100	100	1.120	5.080	1.120	5.080
D100#4	5.27	5.38	2.59	5.41	10.0	0	55	100	100	1.470	4.500	1.470	4.500
D100#3W	3.80	4.77	2.54	5.68	10.0	0	53	100	100	1.230	4.299	1.230	4.299
D100#3N	2.30	2.88	1.67	3.78	10.0	0	88	100	100	0.840	2.330	0.840	2.330
D100#2	1.33	2.46	1.14	7.95	10.0	0	96	100	100	0.365	1.692	0.365	1.692
D100#1	1.42	3.41	1.95	4.29	10.0	0	82	98	100	0.950	2.700	0.950	2.700
D100#11	1.77	2.54	0.91	5.12	10.0	0	100	100	100	1.356	2.034	1.356	2.034
D100#12	7.74	4.70	2.35	4.91	10.0	0	100	100	100	0.995	2.751	0.995	2.751
D100#13	5.46	4.67	2.77	4.28	10.0	0	94	94	100	1.310	3.020	1.310	3.020
D100#14	9.07	5.42	1.52	4.21	10.0	0	82	96	100	0.740	4.460	0.740	4.460
D100#15	15.83	6.59	2.61	5.46	10.0	0	100	100	100	1.051	3.529	1.051	3.529
D100#16	9.52	4.84	1.63	9.86	10.0	0	100	100	100	0.467	2.523	0.467	2.523
D100#17	7.67	7.23	2.99	6.03	10.0	0	100	100	100	1.152	3.570	1.152	3.570
D118#0	8.18	6.50	2.89	2.35	10.0	0	40	100	100	2.368	9.000	2.368	9.000
D118#1	4.00	3.35	1.66	4.30	10.0	0	84	100	100	0.800	2.680	0.800	2.680
D118#2	5.30	4.96	2.23	3.30	10.0	0	60	80	80	1.655	6.199	1.655	6.199
D140#A*	3.79	--		--			--						
D140#11	4.14	5.34	2.46	2.64	10.0	0	92	100	100	1.510	3.890	1.510	3.890
D133#11	4.53	4.56	2.23	3.83	10.0	0	100	80	100	1.070	3.890	1.070	3.890
D139#12	1.36	2.55	1.21	6.18	10.0	0	100	100	100	0.436	1.807	0.436	1.807
D112#1	1.83	2.55	0.81	3.98	10.0	0	100	100	100	0.360	2.523	0.360	2.253
D112#13	2.93	3.90	1.82	3.98	10.0	0	82	100	100	0.920	3.120	0.920	3.120

*Kinematic Wave Routing - Length = 18400 feet - Channel Slope = 0.00063.

TABLE A-1 (Cont'd)

Sub-Watershed	Drainage Area (mi^2)	Length (mi)	Length to Centroid (mi)	Channel Slope (ft/mi)	Overland Slope (ft/mi)	Ponding Area (%)	Urban Development (%)	Channel Conveyance (%)	Channel Improvement (%)	Unitgraph Coefficients 10-Yr.		Unitgraph Coefficients 100-Yr.	
										Tc	R	Tc	R
Sims Bayou													
C100A	2.67	5.00	2.42	4.7	10.0	4.5	32	90	90	1.52	13.83	1.52	11.82
UNT 22.51	0.72	2.30	1.30	3.4	10.0	5.0	26	100	100	0.88	9.77	0.88	8.29
C100B	1.39	1.56	0.54	8.9	10.0	4.5	15	80	80	0.25	7.50	0.25	6.41
C150A	0.97	1.65	0.80	5.9	10.0	4.8	48	67	67	0.47	6.27	0.47	5.34
C100C	1.69	2.50	1.26	3.2	10.0	4.5	41	90	90	0.90	8.20	0.90	7.01
C147A	2.19	3.83	1.75	6.5	10.0	5.0	37	100	100	0.82	8.71	0.82	7.39
C14702A	3.31	4.38	1.90	6.5	10.0	4.8	56	100	100	0.83	6.91	0.83	5.88
C147B	1.08	2.20	1.10	6.5	10.0	4.8	56	50	50	0.69	8.86	0.69	7.54
C145A	3.17	3.80	2.05	4.9	10.0	4.6	59	100	100	1.04	6.21	1.04	5.30
C100D	2.36	2.77	1.59	4.9	10.0	4.5	49	80	80	0.97	7.29	0.97	6.23
C100E	4.83	4.10	2.14	4.8	10.0	7.0	48	90	90	1.20	8.09	1.20	6.71
C161A	2.06	2.95	1.80	4.2	10.0	8.0	19	90	90	1.19	15.56	1.19	12.80
C137A	1.87	3.94	2.07	4.4	10.0	8.2	30	90	90	1.34	14.14	1.34	11.61
C100F	9.15	5.03	3.15	2.6	10.0	7.2	19	90	90	2.87	26.28	2.87	21.77
C132A	1.57	2.61	1.73	10.9	10.0	8.6	23	57	57	0.88	15.38	0.88	12.59
C132B	1.65	2.85	1.80	10.9	10.0	8.0	41	85	85	0.71	6.87	0.71	5.65
C100G	8.88	5.53	2.88	3.0	10.0	7.0	74	90	90	1.99	9.05	1.99	7.51
C123A	2.54	2.30	1.58	11.6	10.0	7.0	11	100	100	0.59	9.23	0.59	7.66
C118A	0.78	1.80	0.69	3.1	10.0	7.2	100	60	60	0.50	6.01	0.50	4.98
C118B	1.70	2.70	1.30	8.9	10.0	7.4	83	60	60	0.60	6.15	0.60	5.08
C100H	2.85	3.15	1.43	2.7	10.0	7.0	24	100	100	1.10	15.23	1.10	12.64
C100I	7.09	4.76	2.99	1.8	10.0	7.0	73	100	100	2.48	7.92	2.48	6.57
C100J	4.03	5.10	3.40	4.4	10.0	2.0	100	100	100	1.57	3.75	1.57	3.38
C106A	3.34	3.41	1.40	15.3	10.0	0.0	64	37	37	0.60	6.48	0.60	6.48
C10608A	1.80	3.13	1.57	5.7	10.0	0.0	75	55	55	1.00	4.78	1.00	4.78
C106B	0.98	1.74	1.44	8.1	10.0	0.0	100	35	35	0.79	3.50	0.79	3.50
C106C	0.74	1.75	1.05	14.4	10.0	0.0	100	100	100	0.24	1.04	0.24	1.04
C10603A	1.30	2.20	0.89	4.3	10.0	0.0	100	30	30	0.69	6.67	0.69	6.67
C10603B	1.29	1.97	1.06	11.8	10.0	0.0	100	30	30	0.48	4.28	0.48	4.28
C106D	1.40	2.05	1.20	0.1	10.0	0.0	100	100	100	3.84	4.39	3.84	4.39
C10601A	3.03	2.80	1.30	6.4	10.0	0.0	88	30	30	0.86	7.41	0.86	7.41
C10601B	1.87	2.80	1.50	8.5	10.0	0.0	100	30	30	0.83	6.05	0.83	6.03
C106E	0.71	1.40	0.80	4.0	10.0	0.0	100	100	100	0.35	1.36	0.35	1.36
C103A	1.67	3.10	1.70	12.8	10.0	0.0	100	100	100	0.43	1.56	0.43	1.56
C102A	2.23	2.75	1.25	8.4	10.0	0.0	97	96	96	0.41	1.84	0.41	1.84
C102B	1.20	2.50	1.10	11.2	10.0	0.0	100	76	76	0.37	1.97	0.37	1.97
C100K	1.78	3.05	1.35	3.6	10.0	2.0	100	100	100	0.66	3.56	0.66	3.21

TABLE A-1 (Cont'd)

Sub-Watershed	Drainage Area (mi^2)	Length (mi)	Length to Centroid (mi)	Channel Slope (ft/mi)	Overland Slope (ft/mi)	Ponding Area (%)	Urban Development (%)	Channel Conveyance (%)	Channel Improvement (%)	Unitgraph Coefficients 10-Yr. Tc	10-Yr. R	100-Yr. Tc	100-Yr. R
Buffalo Bayou													
S. Clodine	8.95	6.82	3.79	3.7	10.0	80	8	100	60	3.66	43.05	3.66	30.34
N. Clodine	0.43	5.30	2.65	2.2	10.0	0	0	90	100	2.58	15.24	2.58	15.24
W170A	2.58	4.92	2.46	5.4	10.0	0	42	70	20	2.18	7.29	2.18	7.29
W100A	2.85	1.52	0.66	2.8	10.0	0	85	90	100	0.37	2.17	0.37	2.17
W16704A	3.73	3.26	1.95	3.9	10.0	0	32	35	35	1.91	16.80	1.91	16.80
W16704B	2.39	3.00	1.84	3.9	10.0	0	65	75	75	1.26	4.00	1.26	4.00
W16704C	1.85	3.13	2.18	3.1	10.0	0	100	100	100	1.17	2.10	1.17	2.10
W100B1	1.77	1.57	0.85	3.5	10.0	0	47	90	100	0.51	3.09	0.51	3.09
W100B2	1.10	1.78	1.00	6.2	10.0	0	47	50	100	0.45	5.23	0.45	5.23
W100C	8.02	3.47	1.89	2.8	10.0	0	96	90	100	1.08	3.09	1.08	3.09
W156A	2.77	2.69	1.33	8.5	10.0	0	96	30	0	0.88	6.00	0.88	6.00
W156B	1.36	3.09	1.33	11.5	10.0	0	97	50	50	0.84	3.28	0.84	3.28
W100153A	1.29	1.89	1.08	15.8	10.0	0	100	80	40	0.40	1.23	0.40	1.23
W100150A	1.34	1.36	0.51	3.2	10.0	0	95	50	70	0.34	3.34	0.34	3.34
W151A	1.82	4.07	2.18	5.0	10.0	0	100	70	90	1.01	3.72	1.01	3.72
W100146A	1.74	1.67	0.95	6.8	10.0	0	95	90	100	0.33	1.52	0.33	1.52
W100147A	2.35	2.46	1.57	12.1	10.0	0	95	100	50	0.64	1.15	0.64	1.15
W100D	2.79	5.07	2.74	2.2	10.0	0	93	90	35	3.18	2.92	3.18	2.92
W100E	1.82	4.73	1.14	2.2	10.0	0	93	90	35	1.26	4.51	1.26	4.51
W142A	1.00	1.89	0.53	2.2	10.0	0	100	70	70	0.42	3.26	0.42	3.26
W142B	1.94	1.70	1.00	5.7	10.0	0	100	100	0	0.80	0.94	0.80	0.94
W141A	1.38	2.84	1.51	20.6	10.0	0	90	60	20	0.57	2.22	0.57	2.22
W100F	1.17	2.75	0.93	2.2	10.0	0	100	100	0	1.22	2.17	1.22	2.17
W140A	1.75	1.82	0.94	6.9	10.0	0	100	50	100	0.32	3.02	0.32	3.02
W140B	2.51	4.98	2.27	5.3	10.0	0	95	80	100	0.94	3.91	0.94	3.91
W140C	1.81	4.36	2.36	6.4	10.0	0	100	60	20	1.66	3.62	1.66	3.62
WIRT	1.05	2.05	1.27	7.7	10.0	0	100	60	100	0.41	2.51	0.41	2.51
W14001A	3.64	3.79	2.37	14.1	10.0	0	100	90	75	0.74	1.72	0.74	1.72
W138A	2.04	2.97	1.45	6.6	10.0	0	95	90	80	0.64	2.16	0.64	2.16
W100G	4.19	2.84	1.52	2.6	10.0	0	98	100	100	0.39	2.43	0.89	2.43
W132A	2.14	2.99	2.05	8.8	10.0	0	95	50	50	1.01	3.48	1.01	3.48
W129A	1.45	4.55	2.94	2.3	10.0	0	100	70	100	1.89	4.83	1.89	4.83
W100H	6.38	3.98	2.10	2.2	10.0	0	71	100	40	2.47	3.08	2.47	3.08
W100I	5.63	3.35	2.01	2.8	10.0	0	94	100	100	1.17	2.57	1.17	2.57
W100J	12.36	4.75	2.06	2.2	10.0	0	100	100	100	1.32	3.65	1.32	3.65

TABLE A-2 - EXISTING CONDITION STORAGE-DISCHARGE INFORMATION

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Volume (ac-ft)	Travel Time (hrs.)
Langham Creek U100-00-00				
U100#6	U100#5	3,500	546	1.44
		7,800	1,008	
		12,100	1,386	
		16,400	1,710	
		20,700	2,001	
		25,000	2,278	
U100#5	U100#4	2,300	471	1.05
		5,100	9,242	
		7,900	1,321	
		10,700	1,693	
		13,500	2,059	
		16,500	2,437	
U100#4	U100#3	2,100	237	0.97
		4,600	480	
		7,100	795	
		9,600	1,183	
		12,100	1,550	
		14,500	1,966	
U100#3	U100#2	1,600	547	3.47
		3,400	1,271	
		5,200	2,029	
		7,000	2,753	
		8,800	3,419	
		10,500	4,039	
U100#2	U100#1	1,600	1,418	6.14
		3,400	2,508	
		5,200	3,396	
		7,000	4,150	
		8,800	4,819	
		10,500	5,398	
Horsepen Creek U106-00-00				
U106#5	U106#4	3,000	477	1.11
			547	
			617	
			686	
			758	
			839	
U106#4	U106#3	3,000	157	0.68
		4,000	190	
		5,000	225	
		6,000	271	
		7,000	317	
		8,000	370	
U106#3	U106#2	2,500	94	0.43
		3,500	128	
		4,500	165	
		5,500	222	
		6,500	265	
		7,500	299	
U106#2	U106#1	2,500	865	3.24
		3,500	1,172	
		4,500	1,469	
		5,500	1,738	
		6,500	1,985	
		7,500	2,227	
Bear Creek U102-00-00				
U102#5	U102#4	2,500	289	0.99
		5,700	614	
		8,900	897	
		12,100	1,151	
		15,300	1,382	
		18,500	1,594	
U102#4	U102#3	2,500	362	0.98
		5,700	863	
		8,900	1,284	
		12,100	1,651	
		15,300	1,998	
		18,500	2,321	
U102#3	U102#2	2,200	721	3.47
		4,800	1,875	
		7,400	3,361	
		10,000	5,218	
		12,600	6,026	
		15,000	6,977	

TABLE A-2 (Cont'd)

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Volume (ac-ft)	Travel Time (hrs.)
U102#2	U102#1	1,800	700	3.94
		3,800	1,942	
		5,900	3,126	
		8,000	4,100	
		10,000	4,876	
		12,000	5,574	
South Mayde Creek U101-00-00				
18.26	15.55	1,600	1,118	5.42
		3,400	2,102	
		5,200	2,830	
		7,000	3,476	
		8,800	4,037	
		10,500	4,513	
15.55	12.87	1,600	614	3.26
		3,400	1,531	
		5,200	2,132	
		7,000	2,724	
		8,800	3,282	
		10,500	3,754	
12.87	10.74	2,000	283	1.56
		4,300	756	
		6,600	1,217	
		8,900	1,633	
		11,200	2,045	
		13,500	2,405	
10.74	9.38	2,000	110	.42
		4,300	197	
		6,600	266	
		8,900	375	
		11,200	543	
		13,500	731	
9.38	8.58	2,500	73	.22
		5,700	127	
		8,900	225	
		12,100	383	
		15,300	536	
		18,500	684	
8.58	7.22	2,500	118	.39
		5,700	224	
		8,900	470	
		12,100	795	
		15,300	1,103	
		18,500	1,394	
7.22	5.38	2,500	323	.96
		5,700	875	
		8,900	1,351	
		12,100	1,808	
		15,300	2,248	
		18,500	2,652	
5.38	0.00	2,700	526	1.35
		6,200	1,145	
		9,700	1,526	
		13,200	1,850	
		16,700	2,142	
		20,000	2,399	
Mason Creek T101-00-00				
6.997	4.72	1,500	179	1.05
		2,800	246	
		4,100	425	
		5,400	786	
		6,700	1,163	
		8,000	1,563	
4.72	3.35	2,000	142	0.62
		3,800	243	
		5,600	340	
		7,400	461	
		9,200	590	
		11,000	722	
3.35	1.84	2,700	110	0.28
		5,300	167	
		7,900	213	
		10,500	279	
		13,100	512	
		15,000	777	

TABLE A-2 (Cont'd)

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Volume (ac-ft)	Travel Time (hrs.)
1.84	0	2,700	321	0.85
		5,300	522	
		7,900	752	
		10,500	1,105	
		13,100	1,373	
		15,000	1,591	
Willow Fork (Natural Channel) T100-00-00				
WF-2	WF-1	1,840	663	3.35
		3,900	1,620	
		7,450	2,963	
		11,800	4,310	
		16,000	5,498	
		19,200	6,343	
WF-1	WF-1A	1,840	149	0.95
		3,900	522	
		7,450	958	
		11,800	1,388	
		16,000	1,743	
		19,200	1,982	
17.36	15.56	2,260	198	1.10
		4,750	651	
		9,300	1,325	
		14,500	2,026	
		19,600	2,668	
		23,520	3,150	
15.56	14.36	2,290	102	0.99
		4,800	438	
		9,400	1,225	
		14,700	1,950	
		19,800	2,535	
		23,760	2,931	
14.36	13.29	2,400	93	0.33
		5,100	172	
		9,900	463	
		15,400	926	
		20,800	1,327	
		24,960	1,623	
13.29	11.50	2,450	157	0.60
		5,200	324	
		10,100	1,273	
		15,700	2,159	
		21,200	2,946	
		25,440	3,782	
11.50	9.75	2,520	442	1.57
		5,330	1,396	
		10,400	2,799	
		16,200	4,066	
		21,900	5,124	
		26,280	5,853	
9.75	8.95	2,580	300	0.85
		5,400	751	
		10,500	1,257	
		16,500	1,737	
		22,300	2,140	
		26,760	2,415	
8.95	5.90	2,640	2,496	2.39
		5,550	4,450	
		10,800	6,842	
		16,900	9,032	
		22,900	10,868	
		27,480	12,123	
Brays Bayou D100-00-00				
D100#10	D100#9	2,056	118	0.78
		3,084	230	
		4,112	655	
		5,140	961	
		6,168	1,272	
D100#9	D100#8	2,420	94	0.61
		3,630	140	
		4,840	450	
		6,050	819	
		7,260	1,119	

TABLE A-2 (Cont'd)

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Volume (ac-ft)	Travel Time (hrs.)
D100#8	D100#7	2,420	111	0.97
		3,630	149	
		4,840	268	
		6,050	1,613	
		7,260	2,460	
D100#7	D100#6	3,040	133	0.54
		4,560	185	
		6,080	248	
		7,600	547	
		9,120	838	
D100#6	D100#5	3,704	116	0.37
		5,556	160	
		7,408	211	
		9,260	352	
		11,112	512	
D100#5	D100#4	4,896	102	0.25
		7,344	139	
		9,792	181	
		12,240	323	
		14,688	476	
D100#4	D100#3	6,132	147	0.28
		9,198	199	
		12,264	261	
		15,330	536	
		18,396	1,000	
D100#3	D100#2	6,132	221	0.41
		9,198	299	
		12,264	408	
		15,300	815	
		18,396	1,628	
D100#2	D100#1	6,132	99	0.21
		9,198	144	
		12,264	216	
		15,330	368	
		18,396	747	
D100#1	D100#11	8,944	214	0.30
		13,416	307	
		17,888	518	
		22,360	923	
		26,832	2,426	

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Volume (ac-ft)	Travel Time (hrs.)
D100#11	D100#12	12,816	308	0.31
		19,224	421	
		25,632	771	
		32,040	1,068	
		38,448	5,717	
D100#12	D100#13	13,432	209	0.22
		20,148	290	
		26,864	995	
		33,580	2,285	
		40,296	7,796	
D100#13	D100#14	16,048	568	0.45
		24,072	785	
		32,096	2,184	
		40,120	2,887	
		48,144	5,710	
D100#14	D100#15	17,952	848	0.59
		26,928	1,203	
		35,904	2,093	
		44,880	2,975	
		53,856	4,408	
D100#15	D100#16	18,924	925	0.54
		28,386	1,277	
		37,848	1,751	
		47,310	2,427	
		56,772	3,171	
D100#16	D100#17	20,020	1,672	0.80
		20,030	2,072	
		40,040	2,494	
		50,050	3,183	
		60,060	3,989	
Sims Bayou C100-00-00				
C100#1	C100#2	400	28	0.55
		1,200	58	
		2,000	98	
		3,400	287	
		4,000	385	
		6,100	815	
		9,800	1,438	

TABLE A-2 (Cont'd)

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Volume (ac-ft)	Travel Time (hrs.)
C100#2	C100#3	700	70	1.01
		1,800	131	
		2,600	197	
		4,000	461	
		4,600	626	
		8,000	1,598	
		13,000	3,839	
C100#3	C100#4	1,500	36	0.43
		3,000	63	
		4,000	114	
		5,000	238	
		5,500	289	
		8,000	573	
		13,000	1,383	
C100#4	C100#5	1,500	36	0.31
		3,000	66	
		4,000	95	
		5,000	168	
		5,500	211	
		8,000	411	
		13,000	881	
C100#5	C100#6	1,700	149	1.06
		3,900	276	
		5,000	356	
		6,000	709	
		6,600	915	
		10,000	2,087	
		16,800	4,491	
C100#6	C100#6A	2,100	154	0.76
		4,800	281	
		6,000	334	
		7,600	434	
		8,100	471	
		10,000	809	
		16,800	2,817	
C100#6A	C100#7	2,200	241	1.15
		5,200	460	
		6,600	546	
		8,500	785	
		8,900	907	
		10,000	1,642	
		16,800	5,009	
C100#7	C100#8	2,800	372	1.41
		6,200	656	
		7,400	792	
		9,900	1,355	
		10,500	1,696	
		14,000	3,779	
		24,000	8,934	
C100#8	C100#9	3,000	134	0.46
		6,400	213	
		8,100	258	
		10,400	410	
		11,100	552	
		15,800	1,309	
		26,500	3,532	
C100#9	C100#10	3,100	117	0.36
		6,500	185	
		8,300	223	
		10,700	292	
		11,700	371	
		15,800	905	
		26,500	2,789	
C100#10	C100#11	3,400	416	1.09
		6,700	647	
		9,000	791	
		11,400	1,107	
		12,500	1,309	
		17,000	2,237	
		29,000	6,972	
C100#11	C100#12	3,900	658	1.65
		7,500	1,094	
		9,800	1,343	
		12,400	1,674	
		13,400	1,826	
		17,000	2,549	
		30,500	6,415	
C100#12	C100#13	5,400	190	0.23
		9,200	244	
		12,800	287	
		16,000	325	
		17,300	340	
		29,000	462	
		45,000	688	

TABLE A-2 (Cont'd)

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Volume (ac-ft)	Travel Time (hrs.)
C100#13	C100#14	5,400	531	0.71
		9,200	662	
		12,800	765	
		16,000	856	
		17,300	896	
		29,000	1,219	
		45,000	1,947	
Buffalo Bayou W100-00-00				
W100#1	W100#2	4,900	227	0.46
		6,700	400	
		8,500	706	
		10,300	1,029	
		12,100	1,395	
		13,900	1,792	
W100#2	W100#3	5,650	574	1.54
		7,800	1,418	
		9,950	2,512	
		12,150	3,567	
		14,300	4,621	
		16,450	5,719	
W100#3	W100#4	7,700	1,050	1.90
		10,800	1,973	
		13,900	3,925	
		17,000	5,934	
		20,100	7,940	
		23,200	9,909	
W100#4	W100#5	8,300	1,103	1.29
		12,400	1,567	
		16,600	2,264	
		20,700	3,218	
		24,900	4,474	
		29,000	5,881	
W100#5	W100#6	8,100	460	0.64
		11,750	715	
		15,450	1,015	
		19,100	1,468	
		22,800	2,006	
		26,450	2,563	

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Volume (ac-ft)	Travel Time (hrs.)
W100#6	W100#7	8,300	1,615	1.79
		12,400	2,411	
		16,600	3,321	
		20,700	4,467	
		24,900	5,811	
		29,000	7,156	
W100#7	W100#8	11,400	2,175	1.96
		16,400	3,068	
		21,400	4,206	
		26,500	5,804	
		31,500	7,764	
		36,500	9,717	
W100#8	W100#9	13,000	1,815	1.31
		19,000	2,565	
		25,000	3,541	
		31,000	4,693	
		37,000	6,071	
		43,000	7,383	
W100#9	W100#10	13,000	3,182	2.48
		19,000	4,672	
		25,000	6,690	
		31,000	8,754	
		37,000	11,077	
		43,000	13,143	
W100#10	W100#11	13,700	3,377	2.97
		20,000	5,104	
		26,200	8,149	
		32,500	11,647	
		38,700	15,302	
		45,000	18,434	
W100#11	W100#12	34,000	5,234	1.55
		43,000	6,960	
		58,000	9,724	
		74,000	13,349	
		90,000	17,564	
		105,000	21,518	

TABLE A-3 - CHANNEL CHARACTERISTICS, WILLOW FORK
(100-YEAR DESIGN CHANNEL)

Location: Downstream Cross-Section (feet)	Location: Upstream Cross-Section (feet)	Bottom Width (ft.)	Average Depth (ft.)	Slope (ft/ft)
30994	40656	220	11.4	0.00020
40656	52536	220	14.5	0.00035
52536	59200	220	14.6	0.00030
59200	60720	160	15.3	0.00040
60720	61500	145	16.7	0.00040
61500	67200	100	18.1	0.00040
67200	70170	85	20.1	0.00040
70170	71500	75	21.4	0.00040
71500	75820	85	20.5	0.00040
75820	82160	95	19.0	0.00040
82160	87000	95	17.4	0.00045
87000	92870	90	18.6	0.00050
92870	98870	45	19.0	0.00065
98870	107800	30	15.9	0.00080
107800	114259	30	15.6	0.00070
114259	117430	25	17.8	0.00080
117430	122620	25	14.2	0.00065
122620	134890	25	13.5	0.00060
134890	140567	25	13.3	0.00070

TABLE A-4 - STORAGE-DISCHARGE INFORMATION, WILLOW FORK
(100-YEAR DESIGN CHANNEL)

HEC-1 Analysis Point Upst.	HEC-1 Analysis Point Dnst.	Discharge (cfs)	Volume (ac-ft)	Travel Time (hrs.)
WF-2	WF-1	1,840	195	1.47
		3,900	428	
		7,450	1,484	
		11,800	2,833	
		16,000	4,015	
		19,200	4,910	
WF-1	WF-1A	1,840	69	0.39
		3,900	117	
		7,450	187	
		11,800	608	
		16,000	1,002	
		19,200	1,263	
17.36	15.56	2,260	148	0.61
		4,750	242	
		9,300	386	

TABLE A-4 (Cont'd)

HEC-1 Analysis Point Upst.	HEC-1 Analysis Point Dnst.	Discharge (cfs)	Volume (ac-ft)	Travel Time (hrs.)
		14,500	618	
		19,600	1,216	
		23,520	1,690	
15.56	14.36	2,290	130	0.51
		4,800	217	
		9,400	347	
		14,700	499	
		19,800	908	
		23,760	1,296	
14.36	13.29	2,400	108	0.38
		5,100	174	
		9,900	271	
		15,400	367	
		20,800	525	
		24,960	672	
13.29	11.50	2,450	183	0.59
		5,200	289	
		10,100	438	
		15,700	577	
		21,200	1,528	
		25,440	2,562	
11.50	9.75	2,520	234	0.70
		5,330	378	
		10,400	563	
		16,200	805	
		21,900	1,562	
		26,280	2,517	
9.75	8.95	2,580	128	0.42
		5,400	208	
		10,500	295	
		16,500	781	
		22,300	1,721	
		26,760	2,137	
8.95	5.90	2,640	481	1.64
		5,550	779	
		10,800	1,899	
		16,900	4,097	
		22,900	6,206	
		27,480	7,315	

TABLE A-5 - EXISTING STORM FLOWS

Channel	Stream Station (ft.)	Existing Peak Discharges (CFS) 10-Year	100-Year
Langham Creek (U100-00-00)	91080	120	270
	69010	530	1,150
	47942	960	2,130
	--	1,540	3,260
	34267	2,270	4,480
	24763	2,500	4,880
	--	5,540	9,900
	17899	5,800	10,380
Horsepen Creek (U106-00-00)	35270	405	915
	24658	820	1,790
	19536	1,890	3,360
	11827	3,280	5,550
	0	3,670	6,730
Bear Creek (U102-00-00)	87226	560	1,260
	64944	1,120	2,420
	38650	1,840	3,810
	26664	2,020	4,120
	--	2,220	4,470
	19694	2,470	4,860
South Mayde Creek (U101-00-00)	96413	200	440
	82104	670	1,430
	67954	1,430	3,060
	56707	1,930	4,000
	49526	1,980	4,090
	--	2,450	5,130
	45302	2,750	5,630
	38122	3,210	6,670
	28400	3,970	7,750
	0	5,030	9,520
Mason Creek (T101-00-00)	36944	350	760
	--	670	1,260
	24943	1,160	2,230
	--	1,890	3,390
	17688	2,200	3,950
	--	3,410	5,920
	9715	4,290	7,370
	0	4,720	8,000

Channel	Stream Station (ft.)	Existing Peak Discharges (CFS) 10-Year	100-Year
Willow Fork (Natural Channel) (T100-00-00)	140567	530	1,070
	114259	1,360	2,220
	99729	2,060	3,670
	98303	3,370	5,720
	94238	3,350*	5,730
	92443	5,620	9,060
	83415	6,040	9,950
	75817	6,070	10,070
	70646	6,130	10,140
	60721	6,500	10,870
	60351	7,050	12,000
	40657	7,340	12,550
	30994	7,130*	12,330*
(100-Year Design Channel)	140567	530	1,070
	114259	1,550	2,990
	99797	2,500	4,630
	98303	4,150	7,390
	94238	4,250	7,570
	92443	6,100	10,500
	83415	6,690	11,790
	75817	6,880	12,140
	70646	7,070	12,470
	60721	7,820	14,040
	60351	8,860	16,110
	40657	9,540	17,230
	30994	9,780	17,380
Brays Bayou (D100-00-00)	150537	3,280	5,007
	143307	3,397	4,307*
	137537	5,137	6,699
	131965	5,375	5,953*
	124702	6,027	7,467
	120102	6,876	9,137
	116302	8,668	12,271
	111102	10,936	16,271
	105092	11,265	15,722*
	100059	16,486	23,158
	91977	20,093	28,349
	82473	23,891	33,019
	74088	27,112	35,926
	60320	28,901	39,155
	37276	34,754	44,426
	22512	38,218	47,607
	0	40,963	51,724

TABLE A-5 (Cont'd)

Channel	Stream Station (ft.)	Existing Peak Discharges (CFS) 10-Year	100-Year
Sims Bayou (C100-00-00)	119717	753	1,218
	113969	1,562	2,447
	106692	4,069	5,834
	104632	4,983	6,919
	102463	5,438	7,604
	94310	6,890	8,798
	85982	7,051	8,927
	73500	9,013	11,758
	58386	11,294	14,289
	53702	11,510	14,721
	49900	11,915	15,202
	36251	13,110	16,699
	16377	17,088	23,195
	13103	17,351	23,888
	6774	18,341	26,606
Buffalo Bayou (W100-00-00)	250920	1,654	2,633
	246074	5,216	8,019
	237188	6,276	7,885*
	220513	10,656	15,973
	208679	11,501	16,917
	202918	13,075	18,886
	186563	13,533	19,436
	164268	15,096	22,046
	148093	15,763	24,226
	127899	16,200	24,951
	109975	16,754	25,513
	84384	46,466	65,500

*Inconsistencies are attributable to apparent instabilities in stream routings.

TABLE A-6 - REVISED STORAGE-DISCHARGE RELATIONSHIPS - LANGHAM CREEK (U100-00-00)

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Case 5 Volume (ac. ft.)	Case 6 Volume (ac. ft.)	Case 7 Volume (ac. ft.)	Case 8 Volume (ac. ft.)
U100#1	U100#2	1600	1346	1451	1735	2937
		3400	2395	2454	2986	3860
		5200	3251	3263	3971	4945
		7000	3983	3882	4795	5879
		8800	4630	4493	5526	6709
		10500	5191	5020	6164	7422
U100#2	U100#3	1600	526	461	692	949
		3400	1233	1101	1554	2233
		5200	2006	1781	2526	3511
		7000	2728	2444	3373	4740
		8800	3410	3063	4228	5970
		10500	4055	3646	5022	7118
U100#3	U100#4	2100	256	217	282	306
		4600	456	398	654	603
		7100	802	682	1231	1159
		9600	1225	1052	1669	1636
		12100	1650	1419	2142	2232
		14500	2139	1822	2679	2859
U100#4	U100#5	2300	378	372	482	596
		5100	721	684	879	1033
		7900	993	320	1224	1432
		10700	1289	1253	1593	1870
		13500	1543	1527	1930	2239
		16500	1804	1813	2259	2601
U100#5	U100#6	3500	545	499	606	675
		7800	1007	942	1092	1185
		12100	1387	1304	1489	1600
		16400	1707	1612	1824	1949
		20700	2000	1893	2128	2267
		25000	2275	2148	2428	2604

TABLE A-7 - REVISED STORAGE-DISCHARGE RELATIONSHIPS - HORSEPEN CREEK (U106-00-00)

HEC-1 Analysis Point Upst.	HEC-1 Analysis Point Dnst.	Discharge (cfs)	Case 5 Volume (ac. ft.)	Case 6 Volume (ac. ft.)	Case 7 Volume (ac. ft.)	Case 8 Volume (ac. ft.)
U106#1	U106#2	2500	858	766	1045	1428
		3500	1154	1032	1407	1898
		4500	1436	1287	1752	2344
		5500	1699	1523	2061	2737
		6500	1944	1745	2354	3090
		7500	2175	1959	2622	3415
U106#2	U106#3	2500	94	89	106	125
		3500	130	123	146	167
		4500	167	158	197	210
		5500	223	214	247	271
		6500	260	256	284	320
		7500	299	284	330	405
U106#3	U106#4	3000	154	142	179	218
		4000	186	173	216	265
		5000	221	204	261	320
		6000	264	240	312	385
		7000	307	281	369	465
		8000	359	320	436	573
U106#4	U106#5	3000	452	422	494	548
		4000	518	485	568	635
		5000	584	544	644	729
		6000	651	604	726	830
		7000	724	668	815	942
		8000	804	735	916	1066

TABLE A-8 - REVISED STORAGE-DISCHARGE RELATIONSHIPS - BEAR CREEK (U102-00-00)

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Case 5 Volume (ac. ft.)	Case 6 Volume (ac. ft.)	Case 7 Volume (ac. ft.)	Case 8 Volume (ac. ft.)
U102#1	102#2	1800	599	517	847	1431
		3800	1725	1489	2323	3545
		5900	2873	2561	3676	5349
		8000	3861	3481	4860	6791
		10000	4766	4314	5984	8106
		12000	5658	5178	6876	9330
U102#2	U102#3	2200	694	622	868	1117
		4800	1798	1637	2192	2777
		7400	3033	2745	3965	4614
		10000	4734	4321	5647	6988
		12600	5910	5438	7110	8677
		15000	7052	6476	8433	10324
U102#3	U102#4	2500	320	285	399	670
		5700	786	724	916	1373
		8900	1236	1155	1406	1965
		12100	1639	1544	1850	2507
		15300	2032	1918	2280	3019
		18500	2413	2288	2697	3504
U102#4	U102#5	2500	280	260	318	360
		5700	586	550	656	733
		8900	848	795	951	1059
		12100	1087	1022	1208	1328
		15300	1302	1229	1436	1567
		18500	1501	1419	1647	1789

TABLE A-9 - REVISED STORAGE-DISCHARGE RELATIONSHIPS - SOUTH MAYDE CREEK (U101-00-00)

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Case 5 Volume (ac. ft.)	Case 6 Volume (ac. ft.)	Case 7 Volume (ac. ft.)	Case 8 Volume (ac. ft.)	Case 9 Volume (ac. ft.)	Case 10 Volume (ac. ft.)
18.26	15.55	1600	1145	1047	1461	1905	1155	1088
		3400	2063	1919	2482	3139	2085	1982
		5200	2813	2628	3350	4140	2843	2709
		7000	3499	3280	4110	4994	3536	3376
		8800	4125	3890	4811	5765	4168	3998
		10500	4693	4420	5432	6447	4736	4541
15.55	12.87	1600	669	578	892	1293	672	616
		3400	1545	1398	1961	2812	1578	1462
		5200	2305	2100	2879	3994	2314	2186
		7000	2939	2690	3630	5016	2949	2817
		8800	3512	3230	4308	5984	3546	3350
		10500	4026	3684	4917	6874	4041	3808
12.87	10.74	2000	300	244	425	645	298	292
		4300	851	728	1132	1638	847	817
		6600	1403	1217	1796	2509	1396	1354
		8900	1877	1629	2379	3139	1866	1800
		11200	2350	2069	2899	3713	2339	2263
		13500	2841	2490	3455	4223	2828	2708
10.74	9.38	2000	96	89	111	133	96	110
		4300	184	172	208	236	184	207
		6600	264	248	296	367	262	293
		8900	388	355	476	607	386	469
		11200	540	485	668	848	536	660
		13500	705	627	871	1085	700	861
9.38	8.58	2500	65	61	76	91	66	76
		5700	115	108	129	152	115	129
		8900	168	152	214	282	168	213
		12100	269	232	340	435	268	338
		15300	381	342	470	586	380	467
		18500	493	448	597	732	491	593
8.58	7.22	2500	129	121	153	181	141	152
		5700	242	225	296	380	264	294
		8900	479	419	639	830	523	635
		12100	843	760	1042	1258	898	1037
		15300	1190	1090	1427	1694	1257	1420
		18500	1536	1417	1819	2140	1619	1811

TABLE A-9 (Cont'd)

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Case 5 Volume (ac. ft.)	Case 6 Volume (ac. ft.)	Case 7 Volume (ac. ft.)	Case 8 Volume (ac. ft.)	Case 9 Volume (ac. ft.)	Case 10 Volume (ac. ft.)
7.22	5.38	2500	314	288	410	563	390	407
		5700	776	697	1016	1348	967	1005
		8900	1293	1173	1642	2112	1579	1632
		12100	1818	1668	2245	2785	2168	2232
		15300	2326	2150	2817	3413	2732	2802
		18500	2809	2614	3341	3998	3247	3325
5.38	0.00	2700	487	472	570	673	556	568
		6200	989	946	1124	1278	1110	1120
		9700	1398	1337	1562	1736	1534	1558
		13200	1739	1672	1921	2120	1891	1916
		16700	2038	1961	2244	2472	2209	2239
		20000	2298	2213	2533	2790	2493	2527

TABLE A-10 - REVISED STORAGE-DISCHARGE RELATIONSHIPS - MASON CREEK (T101-00-00)

HEC-1 Analysis Point Upst.	HEC-1 Analysis Point Dnst.	Discharge (cfs)	Case 5 Volume (ac. ft.)	Case 6 Volume (ac. ft.)	Case 7 Volume (ac. ft.)	Case 8 Volume (ac. ft.)
6.997	4.72	1500	169	159	205	236
		2800	233	222	280	403
		4100	377	316	647	938
		5400	721	630	1026	1453
		6700	1051	917	1563	2084
		8000	1521	1320	2154	2702
4.72	3.35	2000	123	119	130	136
		3800	220	214	235	249
		5600	311	303	333	359
		7400	430	415	472	523
		9200	537	504	601	673
		11000	653	608	734	839
3.35	1.84	2700	100	95	114	128
		5300	158	150	176	195
		7900	206	196	231	269
		10500	271	242	392	628
		13100	531	411	685	1111
		15000	723	593	973	1290
1.84	0	2700	308	268	309	337
		5300	442	398	468	514
		7900	566	555	614	676
		10500	786	836	869	1031
		13100	910	855	1239	1286
		15000	1221	1197	1403	1569

TABLE A-11 - REVISED STORAGE-DISCHARGE RELATIONSHIPS - WILLOW FORK (T100-00-00)

HEC-1 Analysis Point Upst.	HEC-1 Analysis Point Dnst.	Discharge (cfs)	Case 2 Volume (ac. ft.)	Case 5 Volume (ac. ft.)	Case 6 Volume (ac. ft.)	Case 7 Volume (ac. ft.)	Case 8 Volume (ac. ft.)
WF-2	WF-1	1840	623	647	517	846	1195
		3900	1534	1549	1390	1915	2585
		7450	2848	2875	2608	3491	4767
		11800	4209	4263	3882	5134	7131
		16000	5379	5475	4975	6527	9875
		19200	6229	6314	5742	7514	10713
WF-1	WF-1A	1840	156	143	76	300	612
		3900	519	556	448	707	1174
		7450	945	898	796	1203	2224
		11800	1373	1307	1179	1700	3100
		16000	1729	1687	1534	2112	3735
		19200	1973	1923	1756	2398	4132
17.36	15.56	2260	171	120	111	241	493
		4750	652	636	479	871	1803
		9300	1350	1250	1039	1981	4211
		14500	2111	1916	1604	3081	5952
		19600	2791	2564	2106	4050	7151
		23520	3302	3040	2485	4707	7888
15.56	14.36	2290	91	87	81	104	190
		4800	357	270	222	526	950
		9400	1163	1023	804	1597	2426
		14700	1932	1738	1437	2509	3483
		19800	2541	2326	1953	3189	4264
		23760	2945	2793	2392	3641	4753
14.36	13.29	2400	76	66	63	82	91
		5100	154	133	128	166	191
		9900	403	341	331	482	611
		15400	891	751	739	993	1172
		20800	1302	1256	1120	1424	1630
		24960	1599	1721	1489	1744	1979
13.29	11.50	2450	145	139	140	149	158
		5200	344	393	372	368	451
		10100	1603	1454	1440	1640	1730
		15700	2809	3767	3735	2858	3030
		21200	3800	3932	3950	3865	4073
		25440	4522	4552	4670	4607	4830
11.50	9.75	2520	407	246	243	427	471
		5330	758	929	895	828	1000
		10400	2476	2695	2630	2630	2938
		16200	4107	5025	4966	4277	4641
		21900	5364	5309	5257	5562	5988
		26280	6214	5911	5855	6432	6900

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Case 2 Volume (ac. ft.)	Case 5 Volume (ac. ft.)	Case 6 Volume (ac. ft.)	Case 7 Volume (ac. ft.)	Case 8 Volume (ac. ft.)
9.75	8.95	2580	233	216	208	251	329
		5400	593	513	487	656	826
		10500	1318	1184	1155	1412	1616
		16500	1975	1812	1776	2090	2335
		22300	2504	2324	2280	2633	2901
		26760	2857	2668	2619	2994	3280
8.95	5.90	2640	1301	1237	1119	1418	1798
		5550	2563	2403	2041	2900	3607
		10800	5053	4814	4326	5470	6305
		16900	7281	7028	6466	7726	8648
		22900	9116	8843	8219	9609	10603
		27480	10384	10105	9421	10914	11959

[illegible]
WILLOW FORK WITH 100-YEAR DESIGN CHANNEL
(T100-00-00)

HEC-1 Analysis Point Upst.	HEC-1 Analysis Point Dnst.	Discharge (cfs)	Case 2 Volume (ac. ft.)	Case 6 Volume (ac. ft.)	Case 7 Volume (ac. ft.)
WF-2	WF-1	1840	182	192	203
		3900	327	361	597
		7450	591	1200	1958
		11800	1781	2373	3513
		16000	3023	3469	4867
		19200	3881	4299	5920
WF-1	WF-1A	1840	69	67	76
		3900	118	111	130
		7450	192	175	391
		11800	474	387	949
		16000	857	737	1384
		19200	1126	980	1680
17.36	15.56	2260	168	142	169
		4750	274	226	282
		9300	436	353	455
		14500	744	482	1258
		19600	1372	737	2215
		23520	1857	1139	2944
15.56	14.36	2290	136	121	148
		4800	227	196	246
		9400	361	315	390
		14700	596	435	809
		19800	1143	574	1453
		23760	1538	827	2003
14.36	13.29	2400	104	104	113
		5100	170	166	184
		9900	265	258	284
		15400	357	349	388
		20800	565	463	590
		24960	805	569	821
13.29	11.50	2450	172	177	194
		5200	276	279	306
		10100	421	422	459
		15700	565	558	607
		21200	1475	990	1882
		25440	2710	2199	3009
11.50	9.75	2520	234	211	269
		5330	372	242	424
		10400	546	518	598
		16200	729	659	1189
		21900	1606	1207	2269
		26280	2522	1989	3015
9.75	8.95	2580	189	125	133
		5400	311	201	214
		10500	484	289	450
		16500	1030	512	1483
		22300	1594	1447	2069
		26760	2001	1801	2416
8.95	5.90	2640	642	459	506
		5550	1132	730	900
		10800	2529	1706	2320
		16900	4837	3546	4761
		22900	6931	5460	6731
		27480	8295	6886	8050

TABLE A-13 - REVISED STORAGE-DISCHARGE RELATIONSHIPS - BRAYS BAYOU (D100-00-00)

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Case 1 Volume (ac. ft.)	Case 2 Volume (ac. ft.)	Case 2a Volume (ac. ft.)	Case 2b Volume (ac. ft.)	Case 3 Volume (ac. ft.)	Case 3a Volume (ac. ft.)	Case 3b Volume (ac. ft.)
D100#10	D100#9	2056	114	112	115	110	113	116	111
		3084	191	179	200	159	190	211	181
		4112	511	487	578	447	565	606	542
		5140	846	812	871	756	824	890	791
		6168	1104	1063	1148	987	1098	1181	1049
D100#9	D100#8	2420	90	89	92	87	91	93	90
		3630	128	128	133	123	134	138	131
		4840	315	312	391	279	454	446	449
		6050	656	649	730	599	728	774	696
		7260	924	907	1006	815	1003	1066	954
D100#8	D100#7	2420	106	106	108	103	108	109	106
		3630	141	141	145	137	145	147	144
		4840	186	185	205	177	387	286	441
		6050	1110	1111	1343	888	1399	1466	1340
		7260	2086	2050	2226	1740	2244	2388	2168
D100#7	D100#6	3040	126	126	129	123	134	132	136
		4560	174	174	179	170	190	184	193
		6080	221	224	234	217	293	260	322
		7600	412	426	481	374	582	535	612
		9120	702	705	763	627	883	823	915
D100#6	D100#5	3704	109	109	113	107	123	117	127
		5556	150	151	155	147	171	162	177
		7408	189	193	203	188	256	225	288
		9260	287	297	320	277	432	367	484
		11112	428	434	472	402	605	528	658
D100#5	D100#4	4896	96	97	99	95	109	103	112
		7344	132	133	136	129	148	141	153
		9792	165	170	176	166	209	192	235
		12240	236	256	286	232	405	337	453
		14688	408	420	447	389	559	488	597
100#4	D100#3	6132	139	139	143	136	155	149	160
		9198	188	189	193	184	211	203	218
		12264	236	249	255	243	317	281	364
		15330	366	422	462	385	764	607	932
		18396	813	845	905	750	1221	1081	1337
D100#3	D100#2	6132	209	211	216	206	234	227	241
		9198	282	287	292	281	317	306	327
		12264	359	396	402	390	487	438	553
		15330	557	694	730	654	1164	929	1421
		18396	1304	1461	1485	1319	1966	1759	2125

TABLE A-13 (Cont'd)

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Case 1 Volume (ac. ft.)	Case 2 Volume (ac. ft.)	Case 2a Volume (ac. ft.)	Case 2b Volume (ac. ft.)	Case 3 Volume (ac. ft.)	Case 3a Volume (ac. ft.)	Case 3b Volume (ac. ft.)
D100#2	D100#1	6132	93	95	97	93	105	101	109
		9198	134	140	142	138	154	149	159
		12264	186	214	216	213	250	231	266
		15330	277	358	356	365	486	406	570
		18396	598	699	718	679	878	800	946
D100#1	D100#11	8944	202	215	214	215	223	219	229
		13416	289	310	308	313	321	313	329
		17888	420	619	567	676	617	558	678
		22360	718	1165	1007	1400	1183	1015	1511
		26832	1672	2957	2643	3378	3149	2732	3643
D100#11	D100#12	12816	294	324	315	335	322	313	330
		19224	398	447	434	464	443	429	456
		25632	542	1093	930	1346	1045	854	1233
		32040	942	2693	2092	3798	2615	1876	3525
		38448	3246	9007	7387	10886	8442	6539	9553
D100#12	D100#13	13432	199	220	213	227	218	210	223
		20148	273	306	298	315	304	294	312
		26864	557	1493	1233	1860	1450	1089	1728
		33580	1230	3344	2819	4650	3523	2519	4793
		40296	4258	10596	9283	11887	10142	8486	10544
D100#13	D100#14	16048	544	589	577	600	599	575	616
		24072	749	814	799	829	827	795	852
		32096	1785	2636	2409	2813	2839	2346	3208
		40120	2233	3503	3171	4106	4068	3061	5136
		48144	3990	8652	6940	10254	9937	6643	12337
D100#14	D100#15	17952	881	889	867	914	899	867	928
		26929	1258	1269	1234	1308	1282	1233	1329
		35904	2315	2374	2227	2524	2429	2228	2607
		44880	3391	3470	3225	3733	3571	3216	3978
		53856	5985	6474	5110	7654	7011	5315	8912
D100#15	D100#16	18924	954	969	946	994	971	954	984
		28386	1313	1331	1304	1360	1329	1309	1349
		37848	1820	1864	1802	1936	1860	1813	1909
		47310	2555	2607	2508	2702	2604	2519	2676
		56772	3361	3454	3309	3623	3427	3319	3568
D100#16	D100#17	20020	1702	1715	1693	1738	1806	1804	1739
		30030	2105	2121	2096	2146	2199	2192	2146
		40040	2532	2558	2522	2591	2632	2616	2590
		50050	3257	3300	3238	3353	3380	3348	3349
		60060	4075	4128	4060	4198	4204	4159	4184

TABLE A-14 - REVISED STORAGE-DISCHARGE RELATIONSHIPS - SIMS BAYOU (C100-00-00)

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Case 1 Volume (ac. ft.)	Case 1a Volume (ac. ft.)	Case 1b Volume (ac. ft.)	Case 2 Volume (ac. ft.)	Case 3 Volume (ac. ft.)	Case 4 Volume (ac. ft.)
C100#1	C100#2	400	26	27	26	26	26	28
		1200	55	56	54	55	55	61
		2000	89	93	85	86	88	111
		3400	258	286	258	259	262	316
		4000	353	366	335	336	338	432
		6100	761	782	730	733	738	873
		9800	1357	1389	1311	1316	1322	1535
C100#2	C100#3	700	65	67	63	64	68	74
		1800	122	126	118	121	130	139
		2600	165	178	156	162	197	230
		4000	358	404	332	353	433	574
		4600	483	564	442	474	596	751
		8000	1334	1464	1249	1307	1488	1848
		13000	3169	3498	2945	3101	3578	4442
C100#3	C100#4	1500	34	35	33	34	36	37
		3000	58	60	56	58	64	66
		4000	81	94	73	80	129	144
		5000	174	202	154	174	249	278
		5500	228	258	201	227	305	331
		8000	465	510	424	462	592	658
		13000	1159	1260	1062	1149	1415	1549
C100#4	C100#5	1500	35	36	34	35	38	38
		3000	62	64	61	62	70	69
		4000	80	86	74	79	115	111
		5000	132	147	120	131	206	202
		5500	163	186	147	163	256	243
		8000	353	377	325	350	466	459
		13000	739	798	677	728	1009	989
C100#5	C100#6	1700	139	143	135	138	157	154
		3900	259	267	252	259	294	288
		5000	316	332	305	318	420	397
		6000	478	563	433	484	963	868
		6600	623	764	554	633	1317	1118
		10000	1663	1854	1482	1728	2468	2336
		16800	3959	4215	3767	3935	5083	4854
C100#6	C100#6A	2100	145	149	141	146	162	160
		4800	263	271	255	266	300	294
		6000	310	321	301	316	361	352
		7600	383	404	367	394	499	474
		8100	406	433	388	418	602	534
		10000	573	681	503	587	1038	957
		16800	2408	2585	2238	2426	3366	3161

TABLE A-14 (Cont'd)

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Case 1 Volume (ac. ft.)	Case 1a Volume (ac. ft.)	Case 1b Volume (ac. ft.)	Case 2 Volume (ac. ft.)	Case 3 Volume (ac. ft.)	Case 4 Volume (ac. ft.)
C100#6A	C100#7	2200	222	231	214	230	261	255
		5200	423	440	408	444	496	486
		6600	500	521	481	533	596	583
		8500	638	700	600	725	1010	930
		8900	703	783	652	821	1226	1115
		10000	1188	1427	1037	1401	2150	1992
		16800	3914	4406	3534	4319	6239	5877
C100#7	C100#8	2800	334	351	319	396	404	398
		6200	597	624	573	693	705	696
		7400	712	748	681	877	888	862
		9900	978	1121	882	1676	1783	1703
		10500	1179	1378	1023	2157	2296	2186
		14000	2729	3269	2310	4489	4746	4604
		24000	6999	7847	6311	10063	10483	10417
C100#8	C100#9	3000	122	128	117	144	144	140
		6400	198	205	190	227	227	221
		8100	239	248	230	283	280	271
		10400	308	351	285	543	536	486
		11100	382	448	333	728	720	657
		15800	978	1118	853	1623	1611	1501
		26500	2873	3188	2584	3960	3937	3875
C100#9	C100#10	3100	109	113	105	125	124	122
		6500	174	179	169	195	194	192
		8300	211	217	205	234	234	230
		10700	260	272	248	337	336	319
		11700	296	316	278	456	452	427
		15800	707	794	628	1051	1049	1006
		26500	2367	2555	2167	3007	3005	3026
C100#10	C100#11	3400	398	407	390	435	434	428
		6700	620	634	608	675	673	665
		9000	756	774	742	830	827	816
		11400	1003	1053	963	1226	1117	1183
		12500	1172	1237	1120	1462	1454	1405
		17000	1999	2116	1900	2484	2470	2386
		29000	6060	6507	5663	7930	7885	7586
C100#11	C100#12	3900	664	661	669	677	672	676
		7500	1180	1098	1105	1119	1114	1114
		9800	1351	1349	1359	1376	1368	1368
		12400	1678	1679	1687	1732	1817	1714
		13400	1828	1831	1838	1893	1877	1873
		17000	2550	2556	2565	2656	2628	2622
		30500	6284	6454	6367	6771	6808	6773

[illegible]

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Case 1 Volume (ac. ft.)	Case 1a Volume (ac. ft.)	Case 1b Volume (ac. ft.)	Case 2 Volume (ac. ft.)	Case 3 Volume (ac. ft.)	Case 4 Volume (ac. ft.)
C100#12	C100#13	5400	196	192	199	198	195	203
		9200	269	247	253	253	249	250
		12800	294	290	307	297	293	293
		16000	332	329	336	335	331	331
		17300	348	344	352	351	347	345
		29000	471	467	475	474	470	467
		45000	663	693	671	666	697	693
C100#13	C100#14	5400	548	540	557	554	546	567
		9200	679	671	688	686	677	681
		12800	782	774	782	789	780	779
		16000	875	866	887	884	874	871
		17300	916	906	928	925	915	912
		29000	1243	1231	1258	1255	1242	1231
		45000	1830	1956	1857	1839	1968	1956

TABLE A-15 - REVISED STORAGE DISCHARGE RELATIONSHIPS - BUFFALO BAYOU (W100-00-00)

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Case 1 Volume (ac. ft.)	Case 2 Volume (ac. ft.)	Case 3 Volume (ac. ft.)	Case 3a Volume (ac. ft.)	Case 3b Volume (ac. ft.)
W100#1	W100#2	4900	212	630	222	224	220
		6700	323	765	363	382	348
		8500	582	1019	651	680	632
		10300	861	1299	981	1007	978
		12100	1170	1610	1377	1384	1423
		13900	1512	1965	1853	1810	1981
W100#2	W100#3	5650	534	696	590	577	606
		7800	1156	1305	1455	1442	1496
		9950	2174	2315	2566	2545	2652
		12150	3162	3307	3705	3636	3873
		14300	4142	4287	4897	4741	5232
		16450	5142	5308	6248	5931	6852
W100#3	W100#4	7700	1050	1000	986	1146	1091
		10800	1973	1764	1734	2441	2174
		13900	3925	3442	3379	4820	4317
		17000	5934	5279	5252	7224	6501
		20100	7940	7071	7127	9697	8695
		23200	9909	8852	9140	12303	10933
W100#4	W100#5	8300	1081	1090	1236	1181	1300
		12400	1543	1583	1793	1698	1910
		16600	2109	2212	2679	2439	2982
		20700	2934	3180	4005	3557	4582
		24900	4036	4528	5660	4997	6500
		29000	5302	6151	7546	6598	8699
W100#5	W100#6	8100	441	503	500	478	528
		11750	674	780	789	748	837
		15450	949	1113	1186	1089	1302
		19100	1351	1626	1764	1596	1965
		22800	1844	2233	2417	2186	2698
		26450	2365	2905	3136	2808	3541
W100#6	W100#7	8300	1557	1666	1726	1668	1812
		12400	2305	2531	2625	2509	2766
		16600	3169	3560	3716	3496	3993
		20700	4223	4924	5155	4775	5634
		24900	5477	6453	6737	6237	7348
		29000	6782	8012	8273	7673	8979
W100#7	W100#8	11400	2097	2346	2340	2252	2453
		16400	2989	3400	3352	3193	3543
		21400	4087	4829	4788	4472	5173
		26500	5591	6925	6825	6273	7505
		31500	7505	9225	9118	8406	9938
		36500	9507	11447	11265	10447	12166

TABLE A-15 (Cont'd)

HEC-1 Analysis Point Upst.	Dnst.	Discharge (cfs)	Case 1 Volume (ac. ft.)	Case 2 Volume (ac. ft.)	Case 3 Volume (ac. ft.)	Case 3a Volume (ac. ft.)	Case 3b Volume (ac. ft.)
W100#8	W100#9	13000	1869	2086	1983	1893	2099
		19000	2694	3004	2827	2681	3001
		25000	3742	4232	3992	3751	4259
		31000	4980	5730	5401	5028	5842
		37000	6400	7273	6955	6497	7455
		43000	7814	8748	8365	7854	8896
W100#9	W100#10	13000	3670	3844	3632	3400	3925
		19000	5402	5628	5265	4962	5664
		25000	7566	7906	7536	7110	8037
		31000	9909	10420	10013	9364	10733
		37000	12322	12852	12523	11786	13310
		43000	14634	15198	14750	13935	15578
W100#10	W100#11	13700	4014	3863	3651	3512	3848
		20000	6192	5958	5511	5311	5815
		26200	9603	9297	8829	8499	9218
		32500	13337	13023	12611	12136	13152
		38700	17054	16641	16388	15847	16952
		45000	20409	19990	19579	19010	20143
W100#11	W100#12	34000	5525	5478	5256	5259	5279
		43000	7489	7353	6993	7001	7031
		58000	10763	10549	9782	9795	9868
		74000	14822	14500	13461	13465	13579
		90000	19243	18865	17706	17694	17832
		105000	23350	22920	21674	21656	21803

Frequency	Stream Station (ft)(1)	Existing Peak Discharge (cfs)	Revised Peak Discharges(cfs) Case 5	Case 6	Case 7	Case 8
100-Year	17899	10390	10580	10730	10080	9840
	49000	2130	2180	2160	2080	1980
	52300	1927	1975	1951	1886	1801
10-Year	17899	5840	5796	5850	5546	5298
	49000	965	983	970	945	911
	52300	875	893	877	853	825

TABLE A-17 - REVISED STORM FLOWS - HORSEPEN CREEK (U106-00-00)

Frequency	Stream Station (ft)(1)	Existing Peak Discharge (cfs)	Revised Peak Discharges(cfs) Case 5	Case 6	Case 7	Case 8
100-Year	528	6240	6290	6310	6130	5910
	23512	2110	2130	2130	2080	2000
10-Year	528	3720	3441	3510	3381	3283
	23512	1059	1045	1220	1025	977

TABLE A-18 - REVISED STORM FLOWS - BEAR CREEK (U102-00-00)

Frequency	Stream Station (ft)(1)	Existing Peak Discharge (cfs)	Revised Peak Discharges(cfs) Case 5	Case 6	Case 7	Case 8
100-Year	19694	4860	5050	5160	4590	4060
	67079	2310	2390	2370	2250	2020
10-Year	19694	2470	2560	2630	2330	2046
	67079	1065	1094	1084	1048	957

Note: (1)Downstream point of constant slope reach.

TABLE A-19 - REVISED STORM FLOWS - SOUTH MAYDE CREEK (U101-00-00)

Frequency	Stream Station (ft)[1]	Existing Peak Discharge (cfs)	Revised Peak Discharges (cfs) Case 5	Case 6	Case 7	Case 8	Case 9	Case 10
100-Year	19219	8320	8322	8610	7630	6830	8100	8010
	39600	6330	--	--	--	--	6230	6240
	55232	4000	4000	4050	3570	3200	3930	4000
	60000	3620	--	--	--	--	3540	3610
10-Year	19219	4692	4923	4860	4740	4650	4280	4200
	39600	3030	--	--	--	--	3010	2990
	55232	1930	1925	1950	1780	1570	1900	1930
	60000	1730	--	--	--	--	1700	1735

TABLE A-20 - REVISED STORM FLOWS - MASON CREEK (T101-00-00)

Frequency	Stream Station (ft)[1]	Existing Peak Discharge (cfs)	Revised Peak Discharges(cfs) Case 5	Case 6	Case 7	Case 8
100-Year	528	7970	8330	8180	8030	7860
	7000	7550	7770	7670	7520	7350
	18100	3920	3990	3960	3890	3860
	27647	2010	2070	2030	2000	2000
10-Year	528	4692	4923	4860	4740	4650
	7000	4411	4590	4530	4428	4345
	18100	2204	2250	2240	2211	2196
	27647	1046	1075	1055	1042	1046

TABLE A-21 - REVISED STORM FLOWS - WILLOW FORK (T100-00-00)

Frequency	Stream Station (ft)[1]	Existing Peak Discharge (cfs)	Revised Peak Discharges (cfs) Case 2	Case 5	Case 6	Case 7	Case 8
100-Year	30994	12330	12699	12240	13160	11750	9820
	70646	10140	10094	10360	10550	9760	8740
	98200	5733	5604	--	--	--	--
	98303	5720	--	5880	5910	5520	4940
10-Year	30994	7130	7378	7310	7910	6970	6080
	70646	6130	6055	6220	6420	5910	5380
	92800	3345	3268	--	--	--	--
	98303	3370	--	3430	3610	3120	2690

Notes: [1]Downstream point of constant slope reach.

Frequency	Stream Station (ft)(1)	Existing Peak Discharge (cfs)	Revised Peak Discharges (cfs) Case 6	Case 7
100-Year	30994	16970	17800	15730
	70646	11910	12290	11320
	98894	5750	5900	5500
10-Year	30994	9616	9893	9302
	70696	6848	6953	6799
	98894	3254	3298	3223

Notes: (1) Downstream point of constant slope reach.
(2) Model considers only the length of stream below the Fort Bend-Waller County Line.

Frequency	Stream Station (ft)(1)	Existing Peak Discharge (cfs)	Revised Peak Discharges (cfs) Case 2
100-Year	30994	17380	17320
	70646	12470	12270
	92800	10500	10310
10-Year	30994	--	--
	70646	--	--
	92800	--	--

Notes: (1) Downstream point of constant slope reach.
(2) Model considers the total length of stream to the headwaters in Waller County.

TABLE A-24 - REVISED STORM FLOWS - BRAYS BAYOU (D100-00-00)

Frequency	Stream Station (ft)(1)	Existing Peak Discharge (cfs)	Revised Peak Discharges (cfs) Case 1	Case 2	Case 2a	Case 2b	Case 3	Case 3a	Case 3b
100-Year	0	51724	53634	50575	51088	50132	50258	51019	50010
	53300	39155	44069	35503	37402	34149	34914	38117	33568
	93000	23039	23923	23072	23154	22900	21633	22593	20742
	116200	12271	12821	12786	12616	12884	11724	11947	11558
	124600	7467	7909	7897	7782	7963	6851	7303	6715
	132000	5953	6089	6091	6060	6366	5890	5913	5954
10-Year	0	40963	40965	40499	40782	40193	40274	40730	40039
	53000	28901	30369	28316	28553	28200	28134	28645	28116
	93000	16738	17416	16860	16784	16821	16288	16595	15976
	116200	8668	9296	9296	9052	9407	8530	8568	8502
	124600	6027	6477	6486	6351	6544	5663	5963	5495
	132000	5375	5546	5552	5503	5575	5249	5368	5095

Notes: (1) Downstream point of constant slope reach.

Frequency	Stream Station (ft)[1]	Existing Peak Discharge (cfs)	Revised Peak Discharges (cfs) Case 1	Case 1a	Case 1b	Case 2	Case 3	Case 4
100-Year	0	26606	27207	26772	27107	26272	25970	26424
	27600	16699	18107	17502	18720	15897	15542	15772
	64900	11758	12188	11951	12331	12047	11125	11301
	105000	6919	6923	6871	6992	6927	6895	6821
10-Year	0	18341	18911	18526	19193	18079	17888	16954
	27600	13110	14026	13659	14361	12733	12714	12264
	64900	9013	9392	9261	9499	9312	8770	8161
	105000	4983	5086	5031	5110	5090	4924	4400

Notes: [1]Downstream point of constant slope reach.

TABLE A-26 - REVISED STORM FLOWS - BUFFALO BAYOU (W100-00-00)

Frequency	Stream Station (ft)[1]	Existing Peak Discharge (cfs)	Revised Peak Discharges(cfs) Case 1	Case 2	Case 3	Case 3a	Case 3b
100-Year	83200	65500	64793	63793	64793	65293	64293
	114800	24951	24969	23581	23476	24139	22766
	135800	24226	24486	22980	22898	23495	22388
	200200	18886	19402	18791	17764	18396	17070
	233600	7885	8414	8105	7944	7898	7901
10-Year	83200	46466	46138	45398	45418	45910	44279
	114800	16200	16317	15432	15337	15720	14798
	135800	15763	16105	15143	14628	15195	14235
	200200	13075	13293	12788	12465	12746	12128
	233600	6276	6351	5868	6315	6304	6300

Notes: [1]Downstream point of constant slope reach.

Frequency	Stream Station (ft)[1]	Hydraulic Carrying Capacity Existing	Case 5	Case 6	Case 7	Case 8
100-Year	17899	413948	409352	381756	464956	550073
	49000	40253	40979	36511	45389	52918
	52300	63259	62929	57158	71554	83984
	76200	37275	36734	33340	43548	52715
10-Year	17899	232671	224254	208398	255005	298508
	49000	18237	18478	16396	20622	24347
	52300	28241	28453	25317	31789	37656

TABLE A-28 - HYDRAULIC CARRYING CAPACITY (FLOODED SECTION FACTOR) - HORSEPEN CREEK (U106-00-00)

Frequency	Stream Station (ft)[1]	Hydraulic Carrying Capacity Existing	Case 5	Case 6	Case 7	Case 8
100-Year	528	230837	207974	195653	244286	287417
	23512	50032	48442	45250	56804	66408
10-Year	528	127595	113774	107657	133855	159249
	23512	29600	23766	30500	33082	39410

TABLE A-29 - HYDRAULIC CARRYING CAPACITY (FLOODED SECTION FACTOR) - BEAR CREEK (U102-00-00)

Frequency	Stream Station (ft)[1]	Hydraulic Carrying Capacity Existing	Case 5	Case 6	Case 7	Case 8
100-Year	19694	175829	177879	166974	191750	207728
	67079	67304	67871	67304	76012	83233
10-Year	19694	89361	90172	85105	97337	104682
	67079	31030	31067	28244	35248	39432

Note: [1]Downstream point of constant slope reach.

Frequency	Stream Station (ft)[1]	Hydraulic Carrying Capacity Existing	Case 5	Case 6	Case 7	Case 8	Case 9	Case 10
100-Year	19219	299832	292293	277887	316819	345850	326338	330840
	39600	228117	--	--	--	--	217273	257733
	55232	118248	114704	107517	123100	134118	113347	128686
	60000	106748	--	--	--	--	102099	98705
10-Year	19219	169088	172910	156611	197158	236986	172435	173474
	39600	109194	--	--	--	--	104985	123497
	55232	56913	55201	51423	60592	65474	54802	62091
	60000	51015	--	--	--	--	49034	47439

TABLE A-31 - HYDRAULIC CARRYING CAPACITY (FLOODED SECTION FACTOR) - MASON CREEK (T101-00-00)

Frequency	Stream Station (ft)[1]	Hydraulic Carrying Capacity Existing	Case 5	Case 6	Case 7	Case 8
100-Year	528	286858	280804	265299	339329	398007
	7216	208046	207395	188903	239259	285709
	18059	223755	216625	202260	254542	307350
	27647	86044	83689	76830	98683	120089
10-Year	528	169088	165954	156611	197158	236986
	7000	121871	122515	111929	75726	38835
	18100	83303	122157	75726	96496	117381
	27647	43063	43462	38835	49507	60900

Note: [1]Downstream point of constant slope reach.

Frequency	Stream Station (ft)[1]	Hydraulic Carrying Capacity Existing	Case 2	Case 5	Case 6	Case 7	Case 8
100-Year	30994	717329	721255	676605	672291	764000	785410
	70646	289914	291777	283773	264071	320308	350000
	92800	162154	153370	--	--	--	--
	98303	202940	--	199786	184677	225761	247500
10-Year	30994	381115	419042	393101	377954	429789	459605
	70646	173383	175026	170224	162389	192968	215200
	92800	94611	89438	--	--	--	--
	98303	119147	--	116506	114158	127373	134500

TABLE A-33 - HYDRAULIC CARRYING CAPACITY (FLOODED SECTION FACTOR) - WILLOW FORK (100-YEAR DESIGN CHANNEL) (T100-00-00)

Frequency	Stream Station (ft)[1]	Hydraulic Carrying Capacity Existing	Case 2	Case 6	Case 7
100-Year	30994	907085	983710	850517	969953
	70646	336866	354676	310865	369611
	92800	296985	282163	--	--
	98303	243598	--	223573	268627
10-Year	30994	513997	--	472706	573586
	70646	193691	--	175870	221995
	98894	115046	--	104292	131578

Note: [1]Downstream point of constant slope each.

TABLE A-34 - HYDRAULIC CARRYING CAPACITY (FLOODED SECTION FACTOR) - BRAYS BAYOU (D100-00-00)

Frequency	Stream Station (ft)[1]	Hydraulic Carrying Capacity Existing	Case 1	Case 2	Case 2a	Case 2b	Case 3	Case 3a	Case 3b
100-Year	0	2041383	2211814	2105411	2126767	2086969	2095949	2127685	2085606
	53300	1545325	1635584	1477971	1557026	1421605	1456046	1589623	1399913
	93000	1053779	1009145	978462	981948	971176	1067097	1114452	1023147
	116200	427740	425708	425959	420300	429228	426396	434506	420358
	124600	260283	262610	263084	259256	265286	229902	245070	225338
	132000	300672	279087	280954	279527	293642	275526	276600	278518
10-Year	0	1616681	1689368	1685995	1697777	1673256	1679542	1698558	1669742
	53300	1140631	1127099	1178811	1188677	1173981	1173269	1194579	1172602
	93000	765578	734650	715023	711800	713369	803428	818571	788038
	116200	302147	308669	309695	301566	313393	310233	311615	309215
	124600	210088	215065	216080	211582	218012	190038	200105	184400
	132000	271479	254200	256095	253834	257156	245538	251104	238334

TABLE A-35 - HYDRAULIC CARRYING CAPACITY (FLOODED SECTION FACTOR) - SIMS BAYOU (C100-00-00)

Frequency	Stream Station (ft)[1]	Hydraulic Carrying Capacity Existing	Case 1	Case 1a	Case 1b	Case 2	Case 3	Case 4
100-Year	0	1147066	1219142	1199650	1214661	1195264	1176946	1182907
	27600	719945	724283	700083	748803	723246	704305	706055
	64900	506923	487522	478042	493242	481820	504179	505905
	105000	198743	193504	192051	195433	191108	186727	200355
10-Year	0	790737	847422	830170	860059	822623	810583	758965
	27600	565212	561640	546360	574440	579372	576126	549012
	64900	388578	375680	370440	379960	372480	397407	365337
	105000	143133	152158	140621	142829	140417	133276	129244

Note: [1]Downstream point of constant slope reach.

TABLE A-36 - HYDRAULIC CARRYING CAPACITY (FLOODED SECTION FACTOR) - BUFFALO BAYOU (W100-00-00)

Frequency	Stream Station (ft)[1]	Hydraulic Carrying Capacity Existing	Case 1	Case 2	Case 3	Case 3a	Case 3b
100-Year	83200	3594780	3957933	3896847	3957933	3988476	3927390
	114800	1239825	1352127	1276964	1271278	1307181	1232830
	135800	1203800	1121126	1244419	1239978	1272307	1212360
	200200	938453	888348	860373	961960	996185	924379
	233600	391809	385247	371099	363727	361621	361759
10-Year	83200	2550153	2818327	2773124	2774346	2804400	2704770
	114800	804984	883615	835690	830545	851286	801357
	135800	783270	737397	820039	792151	822855	770868
	200200	649702	608645	585522	675018	690235	656768
	233600	311857	290792	268677	289144	288640	288457

Note: [1] Downstream point of constant slope reach.

TABLE A-37 - FLOOD PLAIN AREA (LANGHAM CREEK)

Frequency	Downstream Station (ft)	Upstream Station (ft)	Flood Plain Area (Acres)				
			Existing	Case 5	Case 6	Case 7	Case 8
100-Year	17899	49000	817	818	761	948	1033
	49000	52300	8	8	7	7	26
	52300	76200	784	872	712	986	1231
	76200	90755	508	554	494	672	880
10-Year	17899	49000	607	606	587	636	685
	49000	52300	6	6	5	6	7
	52300	90755	320	336	300	373	558

TABLE A-38 - FLOOD PLAIN AREA (HORSEPEN CREEK)

Frequency	Downstream Station (ft)	Upstream Station (ft)	Flood Plain Area (Acres)				
			Existing	Case 5	Case 6	Case 7	Case 8
100-Year	528	23512	248	233	206	271	323
	23512	41346	1247	1282	1196	1484	1704
10-Year	528	23512	175	170	164	185	206
	23512	41356	710	755	686	940	1227

TABLE A-39 - FLOOD PLAIN AREA (BEAR CREEK)

Frequency	Downstream Station (ft)	Upstream Station (ft)	Flood Plain Area (Acres)				
			Existing	Case 5	Case 6	Case 7	Case 8
100-Year	19694	67079	1227	1251	1172	1405	1482
	67079	91349	1448	1177	925	1620	2106
10-Year	19694	67079	487	498	467	592	675
	67079	91349	210	278	179	404	724

TABLE A-40 - FLOOD PLAIN AREA (SOUTH MAYDE CREEK)

Frequency	Downstream Station (ft)	Upstream Station (ft)	Flood Plain Area (Acres) Existing	Case 5	Case 6	Case 7	Case 8	Case 9	Case 10
100-Year	19219	39600	1138	1108	1058	1204	1321	1133	1340
	39600	55232	2728	--	--	--	--	2650	--
	55232	60000	2649	2704	2476	3109	3740	--	--
	60000	99974	2567	--	--	--	--	--	2403
10-Year	19219	55232	745	735	698	806	891	--	--
	55232	99974	1278	1389	1200	1641	2030	--	--

TABLE A-41 - FLOOD PLAIN AREA (MASON CREEK)

Frequency	Downstream Station (ft)	Upstream Station (ft)	Flood Plain Area (Acres) Existing	Case 5	Case 6	Case 7	Case 8
100-Year	528	7216	321	313	310	345	367
	7216	18059	35	35	35	36	40
	18059	27647	110	111	105	120	230
	27647	40281	26	25	24	28	62
10-Year	528	7216	254	241	231	281	301
	7216	18059	31	31	30	32	33
	18059	27647	57	58	57	61	62
	27647	40281	22	22	21	24	28

TABLE A-42 - FLOOD PLAIN AREA (WILLOW FORK - NATURAL CHANNEL)

Frequency	Downstream Station (ft)	Upstream Station (ft)	Flood Plain Area (Acres) Existing	Case 2	Case 5	Case 6	Case 7	Case 8
100-Year	30994	70646	6136	6136	6063	6072	6174	6119
	70646	98894	1444	1443	1401	1252	1768	2140
	98894	114259	913	911	917	866	1009	1207
10-Year	30994	70646	4250	4250	4232	4452	4651	4696
	70646	98894	975	975	957	873	1107	1506
	98894	114259	576	577	641	594	726	894

TABLE A-43 - FLOOD PLAIN AREA (WILLOW FORK - 100-YEAR DESIGN CHANNEL)

Frequency	Downstream Station (ft)	Upstream Station (ft)	Flood Plain Area (Acres) Existing	Case 2	Case 6	Case 7
100-Year	30994	70646	2890	3306	2717	3216
	70646	98894	125	120	113	387
	98894	114259	270	50	164	372
10-Year	30994	70646	1013	--	839	1586
	70646	98894	99	--	97	104
	98894	114259	42	--	41	45

TABLE A-44 - FLOOD PLAIN AREA (BRAYS BAYOU)

Frequency	Downstream Station (ft)	Upstream Station (ft)	Flood Plain Area (Acres) Existing	Case 1	Case 2	Case 2a	Case 2b	Case 3	Case 3a	Case 3b
100-Year	0	53300	878	1370	962	938	1096	932	867	1140
	53300	93000	2421	2175	2912	2655	3101	2975	2628	3140
	93000	116200	658	620	678	669	572	734	752	920
	116200	124600	59	44	43	61	42	97	60	209
	124600	132000	326	194	191	240	157	261	266	320
	132000	157000	1414	1112	1096	1166	1029	1168	1218	1122
10-Year	0	53300	349	442	487	381	478	421	384	490
	53300	93000	471	689	1452	836	1728	1459	783	1824
	93000	116200	85	84	140	98	133	107	104	141
	116200	124600	36	35	35	36	36	37	36	38
	124600	132000	30	30	30	30	29	29	30	30
	132000	157000	412	295	339	320	243	343	322	279

TABLE A-45 - FLOOD PLAIN AREA (SIMS BAYOU)

Frequency	Downstream Station (ft)	Upstream Station (ft)	Flood Plain Area (Acres) Existing	Case 1	Case 1a	Case 1b	Case 2	Case 3	Case 4
100-Year	0	27600	285	300	291	308	285	281	284
	27600	64900	2317	2183	2267	2110	2514	2328	2313
	64900	105000	2559	2027	2306	1862	2296	2766	2791
	105000	127000	318	229	274	194	211	256	412
10-Year	0	27600	237	247	242	252	239	236	256
	27600	64900	1255	1190	1217	1069	1427	1367	1100
	64900	105000	1062	711	866	600	923	1409	989
	105000	127000	50	35	43	33	34	40	47

TABLE A-46 - FLOOD PLAIN AREA (BUFFALO BAYOU)

Frequency	Downstream Station (ft)	Upstream Station (ft)	Flood Plain Area (Acres) Existing	Case 1	Case 2	Case 3	Case 3a	Case 3b
100-Year	83200	114800	440	462	447	441	443	442
	114800	135800	429	478	455	458	448	473
	135800	200200	1013	1017	1104	1071	1040	851
	200200	233600	1249	1172	1110	1416	1314	1754
	233600	250800	511	371	279	447	429	456
10-Year	83200	114800	298	309	308	296	296	294
	114800	135800	277	302	304	288	282	291
	135800	200200	710	711	736	727	718	735
	200200	233600	324	319	273	384	349	421
	233600	250800	105	84	65	118	107	123