FLOOD PROTECTION PLANNING STUDY FOR THE FREDERICKSBURG AREA

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CITY OF FREDERICKSBURG
Gillespie County, Texas

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1.0 INTRODUCTION

1.1 STUDY OVERVIEW

This regional Flood Protection Planning Study has been undertaken to provide an evaluation of existing flooding conditions and needed drainage improvements and flood control measures within the City of Fredericksburg and adjacent areas of Gillespie County. The study has focused on localized solutions to existing and projected flooding problems, as well as, regional control measures such as stormwater detention facilities. The costs associated with implementing various flood protection options for different portions of the planning area also have been examined. A flood protection and drainage improvement plan has been formulated that identifies and prioritizes the most important projects to be implemented. As part of this overall planning effort, a number of hydrologic and hydraulic analytical tools have been developed that will be useful for continuing to evaluate the effects of future development on stormwater runoff, streamflows and flooding levels throughout the City.

1.2 STUDY PARTICIPANTS

This regional Flood Protection Planning Study for the City of Fredericksburg and the surrounding area has been prepared for the City of Fredericksburg under contract to the Texas Water Development Board with funding assistance through its Research and Planning Grant program. The applicant for funding for this study and the contractor with the Texas Water Development Board has been the City of Fredericksburg. Gillespie County has served as a participating political subdivision.

1.3 STUDY BACKGROUND

The City of Fredericksburg has grown steadily during the past several decades from a population of about 4,000 in 1950 to almost 7,000 in 1990. Today, it is estimated that there are over 8,000 people living within the City, with growth in and around the City continuing at an accelerated pace. The attraction of Fredericksburg's clean, small-town setting in the Hill Country of Texas, coupled with its increasing importance as a center for tourism, has played a major role in this recent growth of the City.

With this growth in population, residential and commercial development, and redevelopment, of land within the City and the surrounding area naturally has taken place. Major residential subdivisions comprised of single-family housing have been constructed and, presently, there are over a thousand residential lots being planned for development. Extensive expansion of the downtown retail area also has occurred in response to the need for basic services and the increased interest in tourism. Commercial developments and some light manufacturing facilities also have been located around the City.

With these changes in land use to more developed and densely-populated conditions, corresponding changes in the characteristics of the watersheds that drain the City also have occurred. With more streets, parking lots and roof tops, the imperviousness of the land surface has increased, thereby causing infiltration of rainfall to be reduced and rates and volumes of stormwater runoff to be increased. Basically, today there is more stormwater generated within the City by the same amount of rainfall than there was just five or ten years ago, and the extent to which existing watercourses and drainage facilities can handle these higher amounts of runoff under the more extreme rainfall conditions has been of concern to City officials.

While there are areas within the City that have experienced some shallow water flooding and street blockage during intense rainfall events, no major flooding of entire blocks or subdivisions, with floodwaters in homes or businesses, has been experienced. However, the actual severity of past storm events with respect to normally-accepted design flood conditions and/or typical levels of regulatory flood protection is not known. Some of the larger storms possibly could cause such flooding, particularly now that a greater portion of the watersheds both within and upstream of the City have been and are being developed. Investigations of the floodwater-carrying capacity of existing watercourses and drainage facilities have been needed to establish the degree of risk associated with flooding by storm events of varying magnitudes.

The City and Gillespie County both participate in the National Flood Insurance Program (NFIP), and, as such, they both have floodplain management ordinances in effect that regulate development within the existing 100-year floodplains along the major creeks within and just outside the City's corporate boundaries. Current flood insurance rate maps for the City indicate that specific base flood elevation information and the associated floodplain boundary delineations have been determined for portions of Barons Creek, Town Creek and an unnamed tributary of Barons Creek located in the extreme northeastern part of the City referred to as Stream FB-1. The flood related information shown on currently-effective flood insurance rate maps for the City are based on studies conducted by the Flood Insurance Administration (FIA), now the

Federal Emergency Management Agency (FEMA), during the late 1970's, and they have not been updated since. Basically, the current floodplain maps for the City reflect watershed and creek channel conditions as they existed almost twenty years ago. It is important that any effort to examine current and future flooding conditions within the City as affected by recent and ongoing development should include a review and reevaluation of flood levels and floodplains along the major creeks through the City as originally studied by FIA. If conditions have changed significantly or if conditions are expected to change due to continued land development and/or proposed drainage and flood control improvements, it is important that revised floodplain boundary maps and associated documents be prepared and submitted to FEMA so that the existing flood insurance maps can be updated and republished.

As flooding problems are identified, improvements in the existing watercourses and drainage facilities may be warranted in order to provide an acceptable level of flood protection for City residents and visitors and properties within the City. Such improvements may consist of widening and deepening of existing watercourses and channels within and downstream of developed areas, installing new drainageways, pipes or conduits to convey excess stormwater from the City's streets to the major creeks, and/or constructing runoff detention pond systems to reduce stormwater flow rates. It is important to determine now the extent to which such drainage improvements and flood control measures need to be implemented, and what it will cost, so that City officials can effectively evaluate if, how and when such projects might be incorporated into the Capital Improvements Program.

Future development within the City's jurisdiction also needs to take place so as not to exacerbate any existing flooding problems or to cause the design floodwater-carrying capacity of existing and/or improved watercourses and drainage facilities to be exceeded. One way to accomplish this is for the City to decide to limit the rates of runoff from the watersheds that drain to and through the City to present levels so that the existing floodwater conveyance system does not have to be expanded in order to handle the higher stormwater flows associated with increased development. Such a stormwater detention program could be implemented either by the City undertaking the construction of major regional runoff detention facilities and allocating the costs among those that benefit and/or new development projects, or by the City adopting ordinances requiring all new development projects to install appropriate onsite runoff detention ponds. It is important for these options, and others for controlling future stormwater runoff, to be examined and evaluated now so that informed decisions can be made.

Finally, it is important that any new stormwater conveyance facilities or related drainage systems be uniformly designed and sized in accordance with accepted engineering practice and design standards. The City needs to adopt a set of drainage design criteria, with which all new drainage facilities and development projects must comply. Such criteria need to be relatively straightforward and easy to check with regard to compliance by City staff doing project reviews. Such drainage design criteria manuals have been developed by other small communities like Fredericksburg and are being used as a means to effectively assure that new drainage facilities are adequately sized and properly designed and constructed.

1.4 PLANNING AREA

The planning area for this Flood Protection Planning Study encompasses all of the Barons Creek watershed, extending from its mouth at the Pedernales River northwestward through the City of Fredericksburg to its headwaters, a distance of about fourteen miles. This watershed, which also includes Town Creek and a major unnamed tributary referred to as Stream FB-1, covers about 33 square miles and drains practically all of the City of Fredericksburg. A small portion of the southwestern part of the City in the vicinity of the High School lies outside of this watershed and drains directly to the Pedernales River. This outside area, which encompasses about one square mile, also is included in the planning area. All of the planning area is within Gillespie County. The map of Gillespie County in Figure 1-1 shows the boundaries of the planning area for this Flood Protection Planning Study.

The planning area for this Flood Protection Planning Study has been delineated based primarily on drainage area boundaries, particularly for the watershed that drains the vast majority of the City of Fredericksburg. This is the area of concern with regard to existing and future drainage and flooding problems and the potential impacts of new development on existing drainage and flooding conditions. The entire planning area is within the watershed of the Pedernales River. The Pedernales River is a tributary of the Colorado River, which flows directly into the Gulf of Mexico.

The City and Gillespie County have jurisdiction over the entire planning area with regard to drainage and flood control issues.

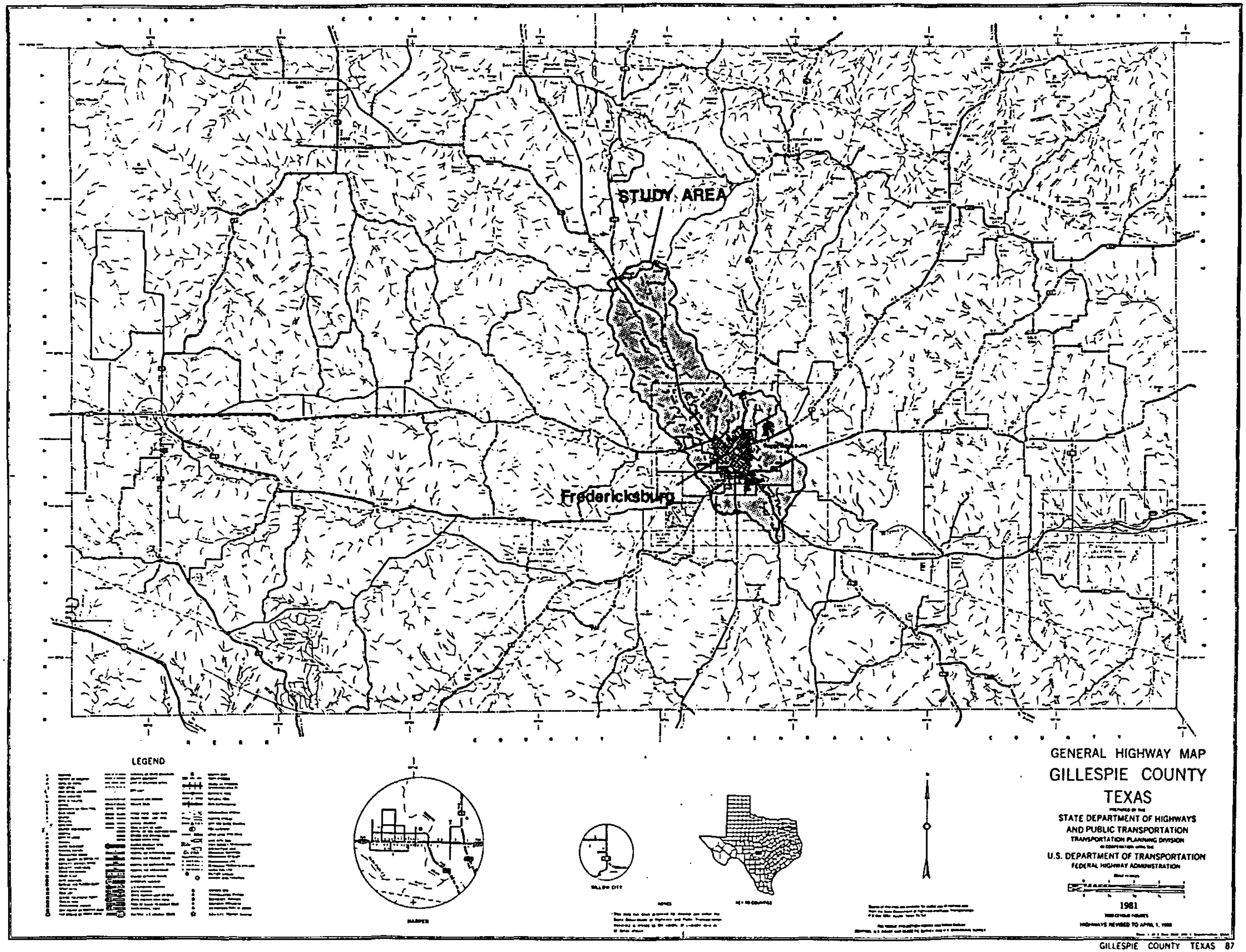

FIGURE 1-1 FLOOD PROTECTION PLANNING AREA

2.0 DATA AND INFORMATION

2.1 EXISTING SOURCES

Considerable data and information have been compiled and analyzed for purposes of this Flood Protection Planning Study. Much of this data and information has been obtained from existing sources. Following is a list of the various items that have been assembled from existing sources and used in this study.

- Topographic maps of the planning area (1"=2,000', 10' contours) as published by the U. S. Geological Survey.
- Topographic maps of the planning area (1"=800', 5' contours) and the associated aerial photography as provided by the Engineering Department of the City of Fredericksburg.
- Roadway and stream maps of Gillespie County as published by the Texas Department of Transportation.
- Street and stream maps of the planning area (1"=800') from the Engineering Department of the City of Fredericksburg.
- 1994 aerial photographs of the Fredericksburg area from the Engineering Department of the City of Fredericksburg as provided by the Gillespie County Tax Assessor/Collector's Office.
- Existing land use map (May 2, 1996) from the Comprehensive Plan for the City of Fredericksburg as prepared by Hankamer Consulting.
- Future land use map (May 15, 1996) from the Comprehensive Plan for the City of Fredericksburg as prepared by Hankamer Consulting.
- "City of Fredericksburg, Texas Comprehensive Plan '96"; prepared for the City of Fredericksburg by Hankamer Consulting; Austin, Texas; November, 1996.
- "Fredericksburg Comprehensive Plan, 1985"; prepared for the City of Fredericksburg by Bovay Engineers; 1985.
- Current zoning map (1996) from the Engineering Department of the City of Fredericksburg.
- "Storm Drainage System Study for North Sector"; prepared for the City of Fredericksburg by Hogan & Rasor, Inc.; Austin, Texas; March, 1982.

- Effective Flood Insurance Maps (May 19, 1991) and Flood Insurance Study (November 19, 1980) for the City of Fredericksburg.

- Flood Insurance Work Maps for the City of Fredericksburg (1980).

- HEC-2 Backwater Models for Barons Creek Town Creek and Stream FB-1 corresponding to the Effective Flood Insurance Maps (May 19, 1991) for the City of Fredericksburg.

- Revised Flood Insurance Maps and and supporting documentation for Letter of Map Revision (February 7, 1995) for a 60-acre tract in the southwest part of the City of Fredericksburg.

- Effective Flood Insurance Maps (May 10, 1977) for Gillespie County.

- "Flood Insurance Study Guidelines and Specifications for Study Contractors"; FEMA 37; Federal Emergency Management Agency; Washington, D. C.; January, 1995.

- Article 3.700, Flood Damage Prevention, of Chapter 3: Building and Construction of the City of Fredericksburg's Code of Ordinances.

- Chapter 9: Subdivisions of the 1996 Subdivision Ordinance of the City of Fredericksburg's Code of Ordinances.

- Subdivision Ordinance for City of Fredericksburg; April, 1984 Edition; Chapter 19.

- Article 11.800, Drainage Utility, of the City of Fredericksburg's Code of Ordinances.

- Zoning Ordinance for City of Fredericksburg; November, 1991 Edition; and Revisions dated 10/26/92, 1/10/94, and 8/22/94.

- Preliminary drainage plans, analyses, and calculations for proposed Stone Ridge Subdivision.

- Preliminary drainage plans, analyses, and calculations for proposed Cross Mountain Subdivision.

- Preliminary drainage plans, analyses, and calculations for proposed Heritage Park Subdivision.

- Preliminary drainage plans, analyses, and calculations for proposed Highland Oaks Apartments

- "Report on Heritage Park Development, A Residential Development in Fredericksburg, Texas"; Grape Creek Ranch Family Ltd. Partnership.

2.2 FIELD SURVEYS

To obtain site specific information regarding ground topography, channel geometry, and drainage facilities features, field surveys were performed at numerous sites throughout the planning area. Field surveys were preformed to provide information on potential localized flooding problems, as well as, major stream channels. A preliminary identification of problem areas first was made by reviewing existing topographic maps (scale: 1" = 800' and five-foot contours) and visiting locations identified as problem areas by City personnel and through citizen complaints. Key features of the potential problem areas were surveyed or measured as necessary for further analysis of hydraulic conditions. Surveyed or measured features included curb heights, roadway widths and crown elevations, distances to and elevations of nearby structures, culvert sizes and flowline elevations, and swale and channel section geometry. The field surveying also included verification of drainage subarea boundaries and flow paths needed to calculate runoff to the potential localized problem areas.

Presented in Table 2-1 is a listing of all of the sites where field surveying has been performed during this study and a general description of the types of information obtained. Work maps are available that indicate the specific location of each of these survey sites.

2.3 GILLESPIE COUNTY FLOOD INSURANCE STUDY

During the course of this Flood Protection Planning Study, the Fort Worth District Office of the U. S. Army Corps of Engineers (Corps) initiated a study of portions of Gillespie County pursuant to the National Flood Insurance Program. Under contract to the Federal Emergency Management Agency (FEMA), the Corps has performed hydraulic analyses, including HEC-2 backwater modeling, of all or parts of several creeks and

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
BARONS CREEK	
Fort Martin Scott and 290	Surveyed new x-section between COE Sections A and 119+00
Wastewater Treatment Plant	Surveyed six sections upstream of WWTP discharge point.
Goehmann Road	Surveyed new x-section between COE Sections B and Goehmann Road
F.M. 1631	Surveyed two new x-sections between Goehmann Road and F.M. 1631 at approximately 190+00 and 210+00
Creek Street	Surveyed four sections at low water crossing.
Creek St. at W. Elk St.	Surveyed new x-section across creek, as if extended from Creek St., at approximately 282+00.
Creek Street	Surveyed across channel from filled area on left bank at approximately 283+50
Washington St.	Surveyed new x-section upstream of Washington St. at fill site on right bank at approximately 296+40.
Prop. Walk Bridge at Llano St.	Surveyed creek at site of proposed walk bridge at end of Llano St.
Walk Bridge at Orange St.	Surveyed newly constructed walk bridge at Orange St.
S. Bowie Street	Surveyed new x-section across creek, as if extended from S. Bowie St. at approximate station 362+75
Peach Street	Surveyed new x-section across creek from Peach St. at approximately 376+00

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
TOWN CREEK	
Elk Street	Surveyed old low water crossing under TxDOT bridge.
Lincoln and Schubert	Surveyed five sections where channel improvements had been made.
Crockett Street	Surveyed new pond structure downstream of Crockett, and associated channel improvements Surveyed CMP's under Crockett, including all wingwalls upstream and downstream Surveyed road profile along Crockett Street.
Orange Street	Surveyed all culverts under Orange Street. Surveyed edge of concrete on upstream and downstream face of Crockett. Surveyed road profile along Orange Street.
D/S Cherry & Morse St.	Surveyed section 50 feet downstream of confluence of tributaries from Cherry St. and Morse St.
N. Cherry Street	Surveyed section 85 feet downstream of Cherry St. culvert. Surveyed section 53 feet downstream of Cherry St. culvert. Surveyed old tank car "culvert" under roadway. Surveyed road profile along N. Cherry Street.
W. Morse Street	Surveyed old tank car "culvert" under roadway. Surveyed road profile along W. Morse Street.
Town Creek Extensions	Surveyed section 16 feet upstream of Morse St. culvert headwall. Surveyed section 400 feet upstream of Morse St. culvert headwall. Surveyed section 900 feet upstream of Morse St. culvert headwall. Surveyed section 1360 feet upstream of Morse St. culvert headwall.

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
DRY CREEK TRIBUTARY	
Bob Moritz Drive	Surveyed low water crossing at Bob Moritz Drive. This is the downstream crossing, approximately 2,500 feet from the U.S. 87 / U.S. 290 intersection.
Gold Road	Measured road crossings, culverts and channel at Gold Road
U.S. 87	Measured culverts at Dry Creek and U.S. 87. Field checked TxDOT design plans. Measured culverts at U.S. 87. Field checked TxDOT design plans.
TRIBUTARY FTB-1	
F.M. 1631	Surveyed new x-sections approximately 60 and 600 feet upstream of low water crossing upstream of confluence near F.M. 1631
Tanglewood Dr.	Surveyed new x-section at approximately 125+00 to pick up effects of wooden retaining walls.
Ridgewood and Glenwood	Surveyed new x-section at approximately 134+10 at site of old bridge.
Briarwood Circle	Surveyed new x-section at approximately 157+00

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
STONERIDGE TRIBUTARY	
D/S Ridgewood St.	Surveyed section upstream of confluence with FTB-1. Surveyed section 35 feet downstream of Ridgewood.
Ridgewood St.	Surveyed road profile and culverts under Ridgewood. Surveyed section 20 feet upstream of culverts. Surveyed section 100 feet upstream of culverts.
FRIENDSHIP LANE	
South Creek Subdivision near Dow Street	Measured channel section.
Friendship Lane	Surveyed road and swales near South Creek Street. Surveyed road and swales downstream of Washington.
Washington	Measured box culvert.
Friendship Lane	Surveyed road and swales upstream of Washington. Surveyed road and swales downstream of channel.
Channel	Surveyed channel 50 feet upstream of Friendship Lane. Surveyed channel 100 feet upstream of Friendship Lane. Surveyed channel 150 feet upstream of Friendship Lane.
South Adams Street	Measured curb and curb cut.

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
CROSS MOUNTAIN-MILAM	
North Milam	Surveyed section @ 604 Milam. Surveyed section @ 705 Milam.
Pecan Street	Measured section downstream of West College Street.
West College Street	Surveyed section between Pecan and Edison.
Edison Street	Surveyed section upstream of West College. Surveyed section downstream of Centre.
Centre Street	Surveyed section just east of Edison. Surveyed section just west of Pecan. Surveyed section just east of Pecan.
Channel south of Burbank	Measured channel downstream of Burbank near Avenue A.
Burbank	Measured curb cut on Burbank near Avenue A.
Avenue D	Surveyed and measured channel at end of Burbank.
Cross Mountain	Surveyed and measured channel at intersection with Avenue D. Surveyed street section.
North Milam	Measured swales and culverts @ Broadmoor.

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
BURBANK-LLANO	
Llano	Surveyed street section @ 905 Llano.
Burbank	Measured street section just west of Llano.
NORTH LINCOLN	
North Lincoln	Surveyed section between Centre and College.
COLLEGE-LLANO	
College	Surveyed street section downstream (east) of Llano.

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
COLLEGE-TRAVIS	
Sycamore	Surveyed street section south of College.
Washington	Surveyed channel upstream of culverts. Surveyed culvert flowlines.
Orchard	Surveyed channel section downsteam of Orchard.
North Pine	Surveyed street section.
East Travis	Surveyed street section at 414 E. Travis. Surveyed street section between Elk St. intersections. Measured channel and street section west of N. Lee St.
North Lee	Measured culvert at City Cemetery. Surveyed channel downstream of N. Lee in City Cemetery.
TRAILMOOR	
Trailmoor	Measured street section. Measured culvert inlet configuration.
Morning Glory	Measured culvert.

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
MORNING GLORY-LLANO	
Llano	Measured box culverts.
Lower Crabapple	Measured channel section next to road.
Morning Glory	Measured box culverts.
CARRIAGE HILLS	
Edgewood	Surveyed channel section west side. Surveyed channel section east side.
Driftwood	Surveyed street section @ 206 Driftwood. Surveyed street section @ 204 Driftwood. Surveyed intersection @ Ridgewood. Surveyed street section @ 114 Driftwood. Surveyed street section @ 112 Driftwood.
North Adams	Surveyed street section east of Driftwood. Surveyed street section just west of Crestwood. Surveyed channel south of Adams.
Frederick	Measured channel section.
Tanglewood	Measured channel section.

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
WEST CREEK STREET	
South Bowie	Field located flat street section.
West San Antonio	Field located flat street section.
OLD HARPER POND	
Armory Road	Field verified low water crossing.
Basse Lane	Field verified low water crossing. Measured swale along road. Measured culvert under Duderstadt (Private Drive)
Duderstadt	Measured swale along road.
South Bowie Street	Measured box culvert.
WINFRIED CREEK	
South Milam	Measured bridge.
Post Oak Blvd.	Measured bridge/culvert. Measured culvert.
Smith Road	Measured culvert.
Live Oak	Measured culvert and upstream wall. Measured culvert.

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
FIVE POINTS	
South Lincoln	Surveyed street and swale section 200 feet north of intersection. Surveyed street and swale section 300 feet north of intersection.
East Live Oak	Surveyed street and swale section 75 feet east of intersection. Surveyed street and swale section 200 feet east of intersection. Surveyed street and swale section 300 feet east of intersection.
South Live Oak & Park St.	Surveyed street and swale section 100 feet west of intersection. Surveyed street and swale section 200 feet west of intersection.
Five Points Intersection	Surveyed flowlines of culverts and measured culvert sites.
East Ufer Street	Measured culvert just north of street (downstream).
East Live Oak	Field verified swale location and condition near Granite Avenue.
Granite Avenue	Measured inlet and culvert sizes @ Granite near Ufer.
SOUTH ADAMS	
Friendship Lane	Measured culverts. Measured channel sections.

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
HIGHWAY-APPLE	
Highway St.	Field verified flow paths.
Apple St.	Field verified flow paths.
South Eagle Street	Field verified low water crossing.
Crenwelge	Measured channel section upstream of Crenwelge culverts. Measured culverts. Estimated channel section downstream of Crenwelge culverts.

streams in the immediate vicinity of the City of Fredericksburg, and now has prepared work maps showing either newly established or revised floodplain boundaries and flood elevations for the 100-year and 500-year floods. Some of the watercourses studied by the Corps are extensions of stream segments that lie within the City of Fredericksburg and, consequently, relate to the flooding analyses performed in this Flood Protection Planning Study. For this reason, portions of this Flood Protection Planning Study have been undertaken within a timeframe that has allowed results from the Corps' Gillespie County investigations to be fully utilized and incorporated. In the early stages of the Corps' Gillespie County flood insurance studies, it was agreed that results from this Flood Protection Planning Study relating to flood flows for the various creeks and streams in the planning area would be provided to the Corps in exchange for hydraulic results and HEC-2 models for the various stream segments analyzed by the Corps. In addition, arrangements also were made to purchase certain detailed and digitized topographic information from the Corps for specific stream reaches within the planning area.

The specific stream segments for which HEC-2 backwater models have been developed by the Corps pursuant to its Gillespie County flood insurance studies and provided to this Flood Protection Planning Study are identified on the map of the Fredericksburg area in Figure 2-1. Basically, the Corps developed HEC-2 models for a portion of Barons Creek extending from near the City's wastewater treatment plant south of downtown upstream to the U. S. Highway 290 bridge and for all of Stream FB-1 from its confluence with Barons Creek upstream to above Lower Crabapple Road. Except for a reach of Stream FB-1 within the Carriage Hills subdivision in the northwestern portion of the City, all of the stream segments modeled by the Corps lie outside the corporate boundaries of the City.

In developing its HEC-2 backwater models, the Corps utilized digitized topographic information to establish channel cross-section geometry. The Corps also made field surveys to obtain dimensions and flowline elevations for bridges and culverts along each of the modeled stream segments. The peak flood flows used by the Corps for specific flood events were agreed upon through discussions with FEMA representatives after hydrologic results from this Flood Protection Planning Study were available for Barons Creek and Stream FB-1. In essence, it was determined that peak flood flows for streams within and in the vicinity of the City under current land use and watershed conditions are not appreciably different from those flows used in the previous flood

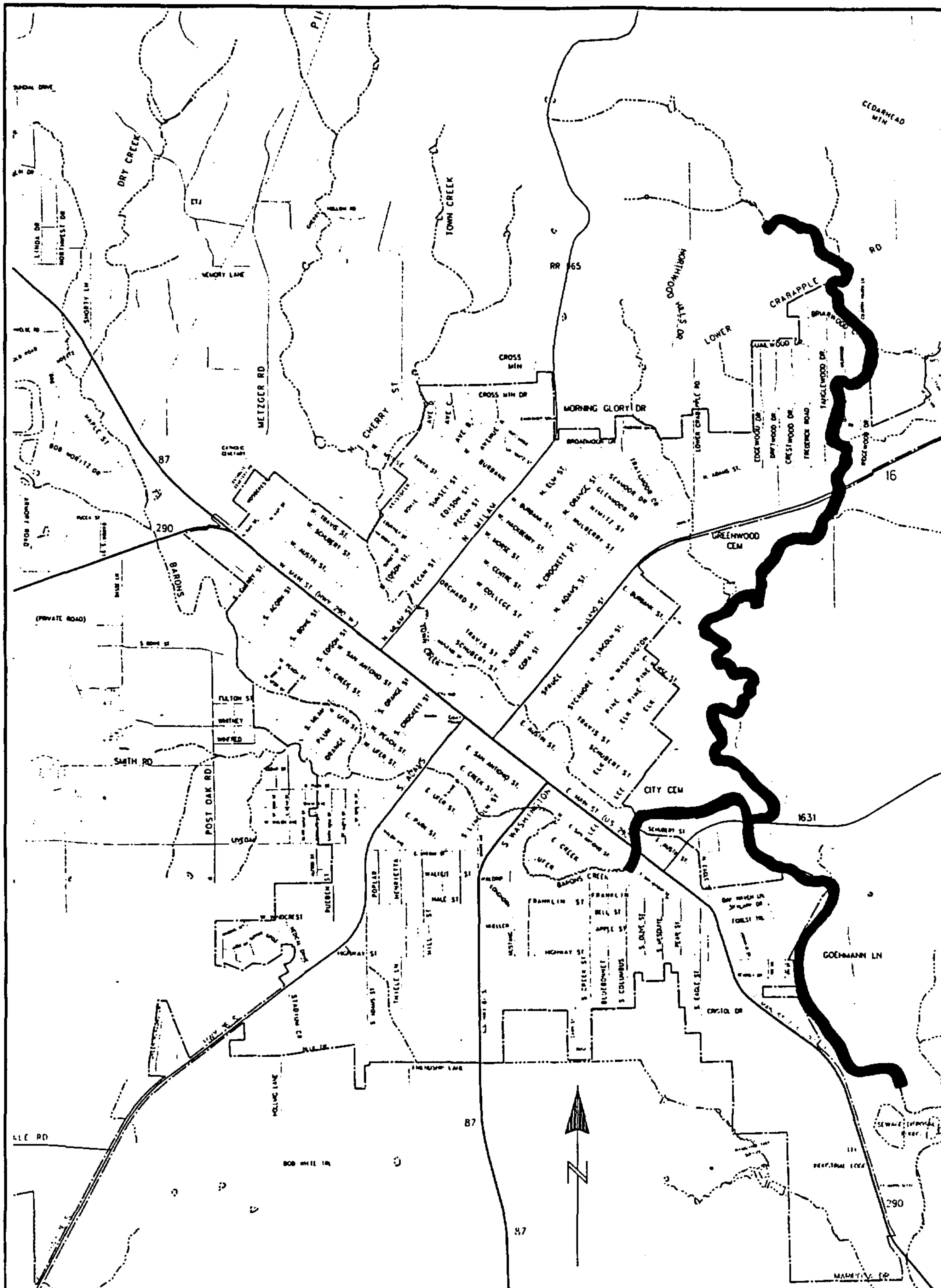

FIGURE 2-1 CORPS OF ENGINEERS GILLESPIE COUNTY HEC-2 MODELS

insurance study for the City that form the basis for the currently-effective flood insurance maps. Hence, in accordance with FEMA's general guidelines for conducting flood insurance studies, it was agreed that the original peak flood flows used in the effective flood insurance study would be utilized by the Corps in its Gillespie County flood insurance studies and also in this Flood Protection Planning Study for the City.

3.0 FLOOD FLOW CONDITIONS

3.1 PREVIOUS FLOOD INSURANCE STUDY

In 1980, Albert H. Halff & Associates completed the Flood Insurance Study (FIS) that provides the basis for the current floodplain boundaries and flood elevations indicated on the effective flood insurance maps of the City of Fredericksburg, which are dated May 19, 1981. As part of this previous investigation, peak flood flows for various creeks and streams within the planning area for this Flood Protection Planning Study were determined for the 10-, 50-, 100- and 500-year flood events. Since the quantities of flood flows occurring at different locations on the creeks and streams within the planning area are fundamental to this analysis of flooding problems and, more importantly, to the development of effective solution measures, the FIS flood flows have been examined and evaluated with respect to corresponding results from this study. Requests were made to the Federal Emergency Management Agency (FEMA) for the original FIS flood flows and backwater models, and these materials were provided.

The specific stream reaches for which hydrologic and hydraulic analyses were performed during the previous FIS for the City of Fredericksburg are identified on the map of the Fredericksburg area in Figure 3-1. Basically, these include portions of Barons Creek, Town Creek and Stream FB-1 in the vicinity of the City. In 1995, a formal Letter of Map Revision (LOMR) was approved by FEMA at the request of the City. This LOMR added a portion of another tributary of Barons Creek, referred to as Stream FB-2, to the effective flood insurance maps for the City. Stream FB-2 enters Barons Creek in the extreme southern portion of the City near U. S. Highway 290.

The peak flood flows from the previous FIS and LOMR for the City are summarized in Table 3-1. Values for the 10-, 50-, 100- and 500-year flood events are presented at several locations along each of the streams included on the effective flood insurance maps for the City. These flood flows will be referred to later in this report.

3.2 HEC-1 HYDROLOGIC ANALYSES

For purposes of examining existing flooding problems and evaluating the effectiveness of alternative flood control and drainage improvement measures in this Flood Protection Planning Study, it has been necessary to develop a computer simulation model capable of describing the hydrologic behavior and response of the several watersheds that encompass the City and the planning area. For this model, the U. S. Army Corps of Engineers <u>HEC-1 Flood Hydrograph Package</u> (September 1990) has

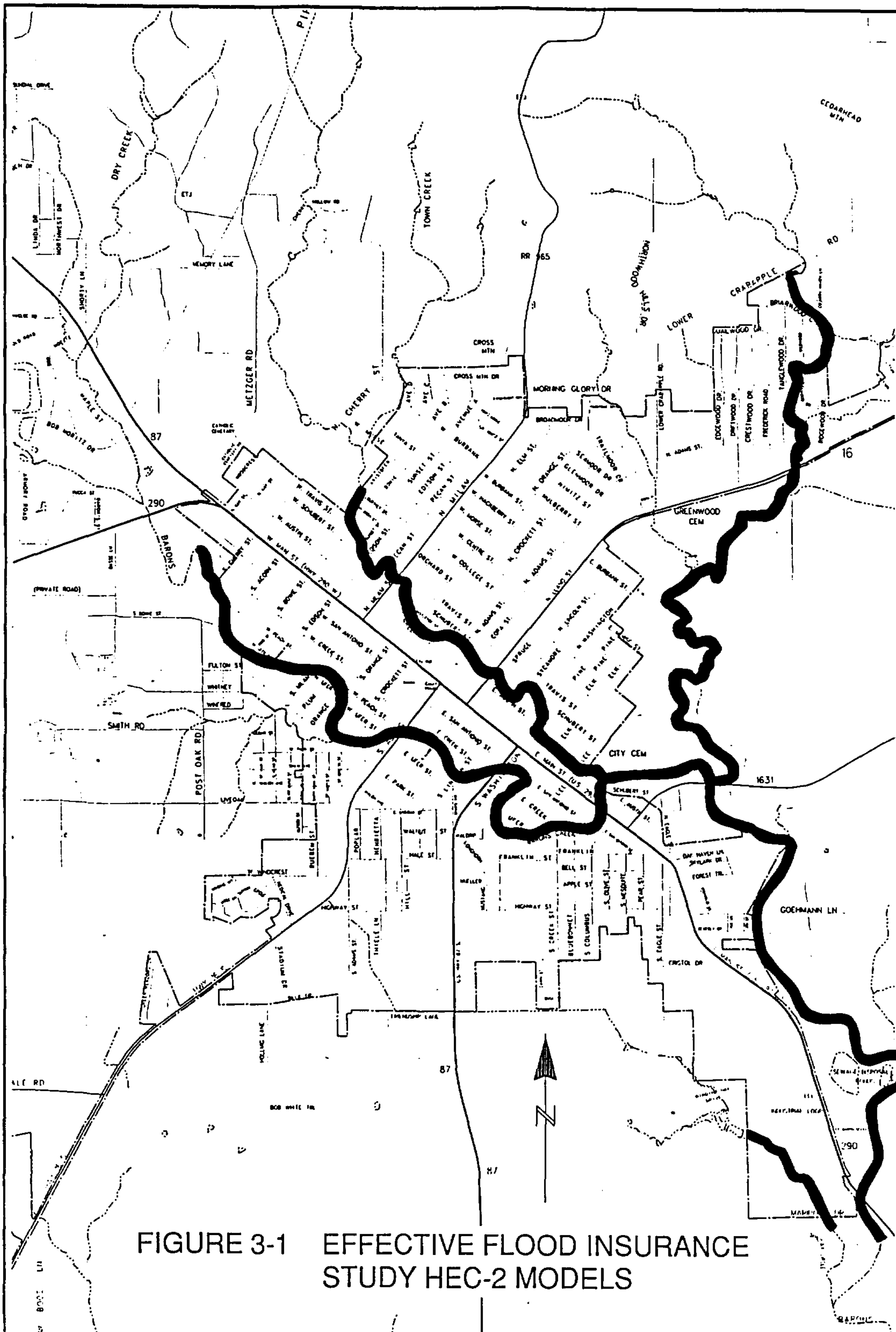

FIGURE 3-1 EFFECTIVE FLOOD INSURANCE STUDY HEC-2 MODELS

TABLE 3-1
EFFECTIVE FIS PEAK FLOOD FLOWS

SITE / CROSSING	10-YEAR FLOOD FLOW cfs	50-YEAR FLOOD FLOW cfs	100-YEAR FLOOD FLOW cfs	500-YEAR FLOOD FLOW cfs
BARONS CREEK				
S. Bowie Street	5,440	9,550	11,800	18,000
S. Adams Street	5,630	9,760	12,000	18,000
S. Llano Street	5,790	9,920	12,100	18,000
Washington Street	5,860	10,100	12,300	18,200
Upstream of Town Creek Confluence	6,690	10,900	13,200	19,000
FM 1631 Upstream Stream FB-1 Confl.	7,540	12,300	14,900	21,000
FM 1631 Downstream Stream FB-1 Confl.	8,250	13,700	16,600	24,000
U/S Wastewater Treatment Plant	8,590	14,200	17,100	24,600
D/S Wastewater Treatment Plant	9,070	14,800	17,900	25,500
Confluence with Stream FB-2	8,840	14,600	17,600	25,500
TOWN CREEK				
Confl. below N. Cherry St. and W. Morse St	1,490	2,620	3,240	4,900
N. Milam Street	1,840	3,090	3,800	5,650
N. Adams Street	1,960	3,270	4,000	5,870
N. Washington Street	2,040	3,370	4,120	5,950
Immediately U/S Confl. with Barons Creek	2,080	3,410	4,160	6,000
STREAM FB-1				
Lower Crabapple Road	860	1,540	1,930	2,950
N. Llano Street	1,520	2,590	3,190	4,680
Carriage Hills Runoff and Stream FB-1	1,990	3,400	4,230	6,300
Immediately D/S Cemetery	2,530	4,310	5,350	7,900
Immediately U/S Confl. with Barons Creek	2,270	3,790	4,650	6,900
STREAM FB-2				
Stock Pond at Camp	1,210	2,022	2,446	4,158
Immediately U/S Confl. with Barons Creek	1,210	2,022	2,446	4,158

been utilized and applied to the various watersheds draining to Barons Creek, Town Creek and Stream FB-1, down to the confluence with the Pedernales River south of the City of Fredericksburg. As stated in the HEC-1 User's Manual,

> *The HEC-1 model is designed to simulate the surface runoff response of a river basin to precipitation by representing the basin as an interconnected system of hydrologic and hydraulic components. Each component models an aspect of the precipitation-runoff process within a portion of the basin, commonly referred to as a subbasin. A component may represent a surface runoff entity, a stream channel, or a reservoir. Representation of a component requires a set of parameters which specify the particular characteristics of the component and mathematical relations which describe the physical processes. The result of the modeling process is the computation of streamflow hydrographs at desired locations in the river basin.*

3.2.1 HEC-1 Model Application

For applying the HEC-1 model to the Barons Creek system, the entire 33-square mile watershed has been divided into forty-one subbasins, or subwatersheds, with each corresponding to a smaller creek or group of creeks, to a change in watershed runoff conditions, and/or to a potential site for a flood control facility such as a detention pond. The boundaries of the model subwatersheds have been determined by examining the hydrologic features depicted on U. S. Geological Survey topographic maps of the region. These boundaries are delineated on the map of the Barons Creek watershed in Plate 3-1. They also are listed in Table 3-2 along with their respective drainage areas. As indicated, most of the subareas in the vicinity of the City are smaller in size than a few hundred acres. The largest subwatershed in the model, Subwatershed BC-12, covers about 13.8 square miles in the extreme upper portion of the Barons Creek watershed that is predominantly undeveloped and expected to remain so in the foreseeable future.

In the process of developing the HEC-1 model of the Barons Creek watershed, a number of different hydrologic parameters that are required for the runoff calculations have been determined. This includes the time of concentration for each of the subwatersheds. The time of concentration is defined as the average time it takes for a particle of water (stormwater runoff) to travel from the farthest upstream point of a subwatershed down to the point of discharge from the subwatershed. This route

TABLE 3-2
HYDROLOGIC PARAMETERS FOR HEC-1 MODEL SUBWATERSHEDS

WATERSHED SUBAREA ID	DRAINAGE AREA ACRES	SQ MILES	TIME OF CONCENTRATION MINUTES	SCS LAG TIME HOURS	ROUTING TIME HOURS	SCS CURVE NUMBERS EXISTING	FUTURE
BARONS CREEK							
BC 01	274.7	0.429	65	0.653	-	77	82
BC 02	338.8	0.529	71	0.709	0.282	76	77
BC 03	294.7	0.460	88	0.881	0.226	82	86
BC 04	392.8	0.614	28	0.284	0.170	82	84
BC 05	456.9	0.714	63	0.632	0.139	75	76
BC 06	159.4	0.249	18	0.184	0.111	82	89
BC 07	310.3	0.485	39	0.393	0.190	80	86
BC 08	175.1	0.274	51	0.510	0.132	79	80
BC 09	354.5	0.554	28	0.284	0.159	80	82
BC 10	287.5	0.449	63	0.628	0.233	81	89
BC 11	1,016.3	1.588	58	0.577	0.167	84	84
BC 12	8,840.3	13.813	170	1.697	0.289	87	87
TOWN CREEK							
TC 01	239.1	0.374	94	0.943	-	83	85
TC 02	330.2	0.516	61	0.612	0.217	77	79
TC 03	346.3	0.541	33	0.326	0.072	84	84
TC 04	327.0	0.511	33	0.332	0.317	86	90
TC 05A	430.6	0.673	47	0.473	0.133	85	83
TC 05B	111.4	0.174	20	0.203	0.178	79	73
STREAM FB-1							
FB1-1	520.7	0.814	45	0.454	-	70	72
FB1-2	269.4	0.421	54	0.536	0.257	73	80
FB1-3	190.7	0.298	31	0.312	0.300	85	85
FB1-4	312.2	0.488	38	0.385	-	69	72
FB1-5A	119.4	0.187	20	0.197	0.409	82	84
FB1-5B	55.1	0.086	46	0.459	0.128	74	75
FB1-6	206.6	0.323	29	0.288	0.084	75	77
FB1-7	697.0	1.089	46	0.462	0.158	83	85
FB1-8	207.5	0.324	27	0.268	0.063	72	76
FB1-9	39.5	0.062	33	0.325	0.168	68	70
BARONS CREEK TRIBUTARIES							
BCT-1A	274.6	0.429	59	0.594	-	74	80
BCT-1B	517.9	0.809	77	0.771	0.209	80	83
BCT-1C	119.7	0.187	51	0.510	0.189	77	75
BCT-1D	175.1	0.274	55	0.546	0.106	74	87
BCT-1E	59.5	0.093	30	0.295	0.173	82	86
BCT-2	387.1	0.605	46	0.460	-	75	76
BCT-3	499.6	0.781	56	0.560	0.183	64	68
BCT-4	193.8	0.303	22	0.219	0.239	82	82
BCT-5	552.2	0.863	36	0.356	-	76	80
BCT-6	172.3	0.269	47	0.467	-	76	83
BCT-7	276.1	0.431	41	0.410	-	86	87
DRY CREEK							
DC-1	662.7	1.036	54	0.545	0.200	87	87
DC-2	98.7	0.154	31	0.308	0.222	84	84

TOTAL AREA OF WATERSHEC 33.271

typically includes some overland sheet flow in the upper reaches of a subwatershed, some shallow concentrated flow through small drainageways, and, finally, some channelized or conduit (pipe) flow through the lower reaches of the subwatershed. For describing the travel times through these different types of flow conditions, standard methods and procedures developed by the U. S. Soil Conservation Service (SCS) have been employed. These methods apply to both undeveloped areas without significant drainage improvements and developed areas where stormwater runoff may sheet flow across a parking lot, flow down a paved street, or be conveyed in a storm drain or concrete lined channel. The procedures that have been applied are described in the SCS Technical Release No. TR-55, Urban Hydrology for Small Watersheds (1986). The resulting times of concentration for each of the subwatersheds corresponding to existing land use and development conditions are summarized in Table 3-2. For future land use and development conditions, the times of concentrations have been reduced by 20 percent to reflect the effects of increased imperviousness of the land surface and future drainage improvements. Other hydrologic parameters such as the SCS lag time and the channel routing time for each subwatershed also are listed in the table. These parameters are required specifically by the HEC-1 model for simulating runoff hydrographs in response to specified rainfall events.

Another parameter that plays a key role in determining how much rainfall on a given area actually flows from the land surface as runoff, as opposed to infiltrating or being lost to evapotranspiration, is referred to as the SCS curve number. The curve number is a numerical quantity ranging between zero and 100 that describes the relative amount of runoff produced by a specified amount of rainfall on a particular type of watershed. A value of 100 reflects complete imperviousness, meaning that all rainfall occurs as runoff. Generalized values of curve numbers have been established by the SCS that relate to specific types of soils, vegetative cover, land use and surface imperviousness. These relationships are summarized in various tables and graphs that also are contained in the SCS Technical Release No. TR-55.

For purposes determining curve numbers for this Flood Protection Planning Study, the hydrologic condition of the land surface of each of the subwatersheds included in the HEC-1 model of the Barons Creek basin has been examined and characterized in terms of the relative areas of the different types of soils, vegetative cover, land use and surface imperviousness. These analyses have been undertaken for both existing land use conditions and future land use conditions, and the corresponding curve number calculations have been performed and summarized in spreadsheets similar to that shown in Table 3-3. For describing the hydrologic characteristics of the soils within the basin, the hydrologic group classifications (A, B, C or D) presented in the SCS Soil Survey of Gillespie County, Texas (1975) have been used. For vegetative cover and land use characteristics within each of the subwatersheds, 1994 aerial photographs of the planning area have been examined. The land use maps depicting existing and future conditions that have been recently prepared as part of the City's Comprehensive Plan '96 have been used to establish land use acreages for each of the subwatersheds in the HEC-1 model. To relate the land use types delineated on the City's land use maps to specific curve number values established by the SCS, the assignments summarized in Table 3-4 have been used for existing land use conditions and those in Table 3-5 have been used for future land use conditions.

The resulting curve number values that have been determined for each of the subwatersheds in the HEC-1 model are listed in Table 3-2. Values for both existing and future land use conditions are presented.

3.2.2 Rainfall Statistics

Because of the enormous expense often involved in providing fail-safe protection from flooding with guaranteed certainty, it is common practice to design and construct flood control and drainage facilities with some acceptable risk of failure incorporated into their operating capacities. For example, the National Flood Insurance Program that is administered by the Federal Emergency Management Agency uses the 100-year flood event as the standard for which an acceptable degree of flood protection is to be provided along streams and rivers. For some types of flood control works such as levees where failure could mean catastrophic losses of life and property, higher standards often are used as the basis for design. For example, many levee designs, particularly with regard to height, are based on the probable maximum flood. For other drainage facilities such as roadway culverts and storm drains, flood flows exceeding their design capacities might be considered more of an inconvenience, rather than a life-threatening occurrence with significant flood damages. For these types of facilities, designs often are based on smaller, more frequent storm events such as the 10-year or the 25-year flood.

Because of the wide range of failure risks inherent in the design standards for drainage

TABLE 3-3
EXAMPLE SOIL CONSERVATION SERVICE CURVE NUMBER CALCULATIONS

PROJECT: FREDERICKSBURG MASTER DRAINAGE STUDY
EXISTING CONDITIONS

DATE: 12/23/96

SUBAREA ID: TC-2

BY: WRY

LAND USE DESCRIPTION	FRACTION		SOIL TYPE A	B	C	D	SCS CN	FRACTION SCS CN
Single Family		% AREA	0.30	0.20	0.50	0.00		
	0.57	CN	61	75	83	87	74.8	42.6
Vacant Undeveloped		% AREA	0.00	0.70	0.30	0.00		
	0.29	CN	68	79	86	89	81.1	23.5
Public/Institutional		% AREA	0.00	0.50	0.50	0.00		
cemetery, park grass cover 5(	0.01	CN	49	69	79	84	74.0	0.7
Commercial/Retail		% AREA	0.40	0.20	0.40	0.00		
	0.02	CN	89	92	94	95	91.6	1.8
Manufactured Home		% AREA	0.00	0.50	0.50	0.00		
	0.02	CN	77	85	90	92	87.5	1.8
Agricultural		% AREA	0.50	0.50	0.00	0.00		
	0.02	CN	49	69	79	84	59.0	1.2
Vacant Developed		% AREA	0.10	0.40	0.50	0.00		
	0.07	CN	49	69	79	84	72.0	5.0

TOTAL OF PRODUCT = 76.7

USE CN = 77

TABLE 3-4
GENERALIZED LAND USE AND CURVE NUMBER ASSIGNMENTS FOR EXISTING CONDITIONS WATERSHED

HANKAMER CONSULTING	SCS TR-55	SCS CURVE NO.			
LAND USE	CORRESPONDING LAND USE	A	B	C	D
RESIDENTIAL	RESIDENTIAL				
Single Family	1/4 acre	61	75	83	87
Duplex	1/8 acre or less	77	85	90	92
Multi-Family	1/8 acre or less	77	85	90	92
Manufactured Home	1/8 acre or less	77	85	90	92
COMMERCIAL	COMMERCIAL				
Retail	Commercial and Business	89	92	94	95
Office/Professional	Commercial and Business	89	92	94	95
INDUSTRIAL	INDUSTRIAL				
Light Industry	Industry	81	88	91	93
Heavy Industrial	Industry	92	94	96	97
Heavy Commercial	Industry	92	94	96	97
INSTITUTIONAL	INSTITUTIONAL	based on facility, i.e., park or office			
STREET R O W	STREET R O W	98	98	98	98
OPEN SPACE	OPEN SPACE				
Park/Recreation	Open, Good condition	39	61	74	80
Agriculture	Pasture, Fair Condition	49	69	79	84
Vacant Developed	Open, Fair condition	49	69	79	84
Vacant Undeveloped	Pasture, Poor Condition	68	79	86	89

TABLE 3-5
GENERALIZED LAND USE AND CURVE NUMBER ASSIGNMENTS FOR FUTURE CONDITIONS WATERSHED

HANKAMER CONSULTING LAND USE	SCS TR-55 CORRESPONDING LAND USE	SCS CURVE NO. A	B	C	D
RESIDENTIAL	RESIDENTIAL				
Low Density	1/4 acre	61	75	83	87
Medium Density	1/8 acre or less	77	85	90	92
Multi-Family	1/8 acre or less	77	85	90	92
COMMERCIAL	COMMERCIAL				
Central Business District	Commercial and Business	95	96	97	98
Office/Commercial	Commercial and Business	89	92	94	95
INDUSTRIAL	INDUSTRIAL				
Industrial / Heavy Commercial	Industry (90% Imp. Cover)	92	94	96	97
INSTITUTIONAL	INSTITUTIONAL	based on facility, i.e., park or office			
STREET R O W	STREET R O W	98	98	98	98
OPEN SPACE	OPEN SPACE				
Park/Open Space	Pasture, Fair Condition	49	69	79	84
Greenbelt, urban	Residential, 1/4 acre	61	75	83	87
Greenbelt, rural	Pasture, Poor Condition	68	79	86	89
Agriculture	Pasture, Poor Condition	68	79	86	89

and flood control facilities, it is necessary to be able to establish peak flood flows that correspond to a similar wide range of probabilities of occurrence. For this purpose, rainfall statistics often are used as the basis for establishing the frequencies associated with the occurrence of certain flood events. For purposes of this Flood Protection Planning Study for the Fredericksburg area, such rainfall statistics have been compiled from the following existing publications of the U. S. Department of Commerce.

> Hershfield, D. M.; 1961; "Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years"; U. S. Department of Commerce, Weather Bureau; Technical Paper No. 40; Washington, D.C.
>
> Miller, J. F.; 1964; "Two- to Ten-Day Precipitation for Return Periods from 2 to 100 Years in the Contiguous United States"; U. S. Department of Commerce, Weather Bureau; Technical Paper No. 49.; Washington, D.C.

Using rainfall information from these publications specifically for the Fredericksburg area, rainfall amounts for specific frequencies of occurrence and specific storm durations have been compiled and analyzed. These results are presented in Table 3-6 in terms of total rainfall amounts and rainfall intensities. Corresponding rainfall duration-intensity curves are plotted in Figure 3-2.

3.2.3 Critical Storm Duration

During the occurrence of a storm event on a given watershed, rainfall infiltrates the soil initially and then gradually begins to accumulate on and runoff from the land surface. Depending on drainage area size and shape, soil conditions, vegetative cover, imperviousness, surface depressions and other features of the watershed, the rate of runoff varies with time. Typically, the variation of the rate of runoff with time after the beginning of a rainfall event produces a bell-shaped flow hydrograph with a flattened and elongated falling limb. The shape and peak of the flow hydrograph for a given rainfall amount on a given watershed varies as a function of storm duration. Short duration, high intensity rainfall events sometimes do not last long enough to allow the entire drainage area of a particular watershed to contribute runoff to the peak flow rate at the discharge point. On the other hand, long duration storms often are characterized by low rainfall rates and, therefore, do not produce a high rate of peak runoff.

TABLE 3-6
RAINFALL DEPTHS AND INTENSITIES FOR FREDERICKSBURG, TEXAS

PEAK RAINFALL DEPTHS IN INCHES

DURATION		EVENT							
HR	MIN	1-year	2-year	5-year	10-year	25-year	50-year	100-year	500-year*
0.5	30	1.15	1.45	1.90	2.22	2.55	2.90	3.25	3.70
1	60	1.45	1.75	2.40	2.80	3.25	3.70	4.12	4.70
2	120	1.70	2.15	2.88	3.40	4.00	4.50	5.10	5.75
3	180	1.90	2.35	3.20	3.75	4.40	5.00	5.70	6.50
6	360	2.25	2.85	3.80	4.60	5.40	6.00	6.80	7.75
12	720	2.60	3.30	4.55	5.40	6.40	7.30	8.25	9.20
24	1440	3.00	3.80	5.25	6.25	7.50	8.45	9.50	12.00
48	2880	-	4.40	5.75	6.95	8.40	9.50	11.00	13.00

* Extrapolated from 25-, 50-, and 100-year data.

PEAK RAINFALL INTENSITIES IN INCHES/HOUR

DURATION		EVENT							
HR	MIN	1-year	2-year	5-year	10-year	25-year	50-year	100-year	500-year
0.5	30	2.30	2.90	3.80	4.44	5.10	5.80	6.50	7.40
1	60	1.45	1.75	2.40	2.80	3.25	3.70	4.12	4.70
2	120	0.85	1.08	1.44	1.70	2.00	2.25	2.55	2.88
3	180	0.63	0.78	1.07	1.25	1.47	1.67	1.90	2.17
6	360	0.38	0.48	0.63	0.77	0.90	1.00	1.13	1.29
12	720	0.22	0.28	0.38	0.45	0.53	0.61	0.69	0.77
24	1440	0.13	0.16	0.22	0.26	0.31	0.35	0.40	0.50
48	2880	-	0.09	0.12	0.14	0.18	0.20	0.23	0.27

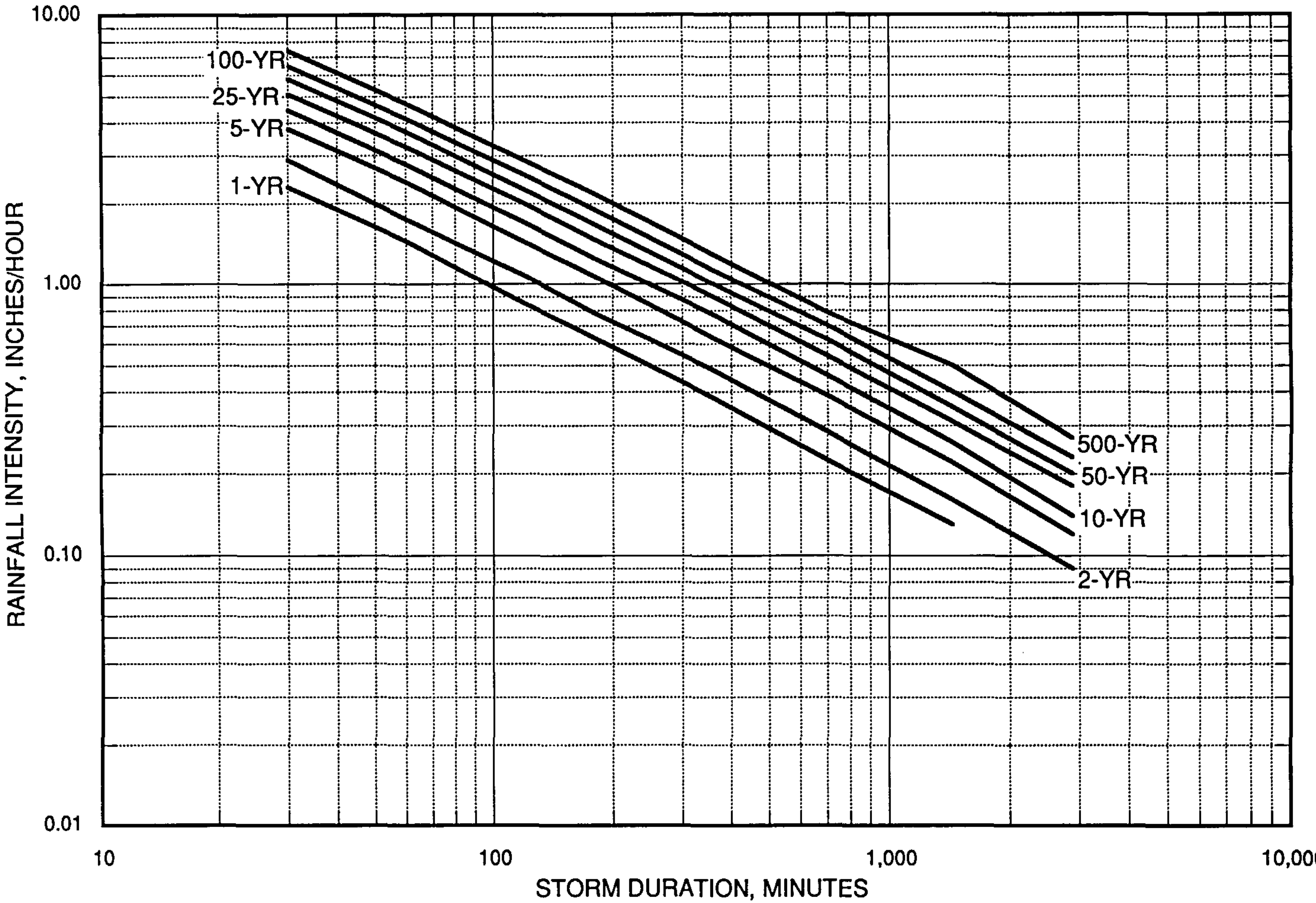

FIGURE 3-2 RAINFALL DURATION-INTENSITY CURVE FOR FREDERICKSBURG

When performing flood studies, it is important to determine the optimum duration of storm event that produces the maximum peak rate of runoff for a given amount of rainfall on a given watershed so that the most critical flooding conditions can be considered. Such analyses have been performed for the various watersheds within the planning area. The HEC-1 model of the Barons Creek basin has been operated for the 100-year rainfall event assuming different storm durations ranging from the two-hour storm up to the 24-hour storm. From these simulations, the peak runoff rates for the various subwatersheds have been examined to determine storm durations producing the maximum flood flows. These results are summarized in Table 3-7 for all of the storm durations analyzed and for both existing and future land use conditions. Peak flow rates are listed for different locations along each of the principal streams in the planning area, and the maximum flow rate at each location for a particular storm duration is identified with a box.

As illustrated by the maximum peak flow rates in Table 3-7, the six-hour storm generally produces the highest peak rates of runoff along the upper and middle reaches of Barons Creek, and, as would be expected, the longer duration 12-hour storm generates the highest peak flow rates along the lower portion of the stream because of the longer travel time from the upper watershed to the mouth. For the other smaller watersheds such as Town Creek and Stream FB-1, the three-hour storm duration appears to be most critical as it generally results in the highest peak flow rates.

Since most of the existing flooding problems within the planning area occur in the smaller watersheds and not necessarily along Barons Creek, the three-hour storm duration has been adopted as the critical storm event for purposes of this Flood Protection Planning Study. As such, the three-hour storm has been used in analyzing flood flows and associated flooding problems.

3.2.4 Peak Flood Flows

Using the rainfall amounts for the three-hour storm events as listed in Table 3-6, the HEC-1 model has been operated to generate peak flood flows along the principal streams throughout the planning area. Simulations have been made for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year rainfall events. The peak flows from the 10-, 50-, 100- and 500-year simulations are listed in Table 3-8 for both existing and future land use conditions.

TABLE 3-7
HEC-1 MODEL 100-YEAR FLOOD FLOWS FOR DIFFERENT STORM DURATIONS

SITE / CROSSING	2-HOUR STORM		3-HOUR STORM		6-HOUR STORM		12-HOUR STORM		24-HOUR STORM	
	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE
BARONS CREEK										
National Guard Armory	12,889	12,889	13,865	13,865	13,807	13,807	13,555	13,555	11,659	11,659
290 west of Town	13,427	13,375	14,963	14,923	14,966	14,965	14,677	14,669	12,811	12,791
S. Bowie Street	13,373	13,317	14,902	14,844	15,111	15,109	14,814	14,807	12,958	12,936
Washington Street	13,308	13,239	14,897	14,787	15,597	15,600	15,286	15,282	13,409	13,385
Upstream Town Creek Confluence	13,273	13,204	14,856	14,746	15,641	15,647	15,333	15,331	13,463	13,436
Downstream Town Creek Confluence	13,499	13,298	15,662	15,314	16,841	16,781	16,463	16,399	14,637	14,473
FM 1631 Upstream FB-1 Confluence	13,510	13,287	15,742	15,350	17,067	16,993	16,684	16,605	14,881	14,683
FM 1631 Downstream FB-1 Confluence	16,449	17,063	17,004	17,354	18,553	18,467	18,330	18,680	17,238	17,167
Goehmann Road	16,592	17,350	17,059	17,649	18,726	18,634	18,574	18,913	17,425	17,374
Downstream Wastewater Treatment Plant	17,777	18,805	18,341	19,112	19,373	19,450	19,829	20,289	18,448	17,503
Confluence with Stream FB-2	19,280	20,607	19,967	21,013	20,326	21,269	21,448	22,098	19,795	19,901
Confluence with Pedernales River	19,194	20,465	20,069	21,035	20,408	21,275	21,515	22,134	19,904	20,100
TOWN CREEK										
West Fork Town Creek	2,077	2,384	2,104	2,379	2,024	2,241	1,740	1,852	1,153	1,193
East Fork Town Creek / Cross Mountain W	1,312	1,393	1,325	1,406	1,258	1,319	1,051	1,063	695	687
Confl. below N. Cherry St. and W. Morse S	3,528	3,848	3,563	3,889	3,420	3,679	2,972	3,099	2,005	2,022
N. Milam Street	4,049	4,439	4,073	4,443	3,950	4,265	3,534	3,706	2,450	2,494
Immediately U/S Confl. with Barons Creek	4,371	4,829	4,433	4,872	4,323	4,698	3,907	4,138	2,779	2,849
STREAM FB-1										
Lower Crabapple Road	2,016	2,444	2,042	2,433	1,957	2,260	1,650	1,792	1,096	1,143
Ridgewood Drive in Carriage Hills	2,286	2,706	2,328	2,731	2,268	2,594	2,007	2,189	1,372	1,435
N. Llano Street	2,677	3,160	2,705	3,192	2,685	3,102	2,442	2,681	1,687	1,778
West Carriage Hills Runoff below N. Llano S	431	484	436	488	426	469	369	393	248	257
Morning Glory / Trailmoor Watershed	1,172	1,431	1,198	1,460	1,149	1,359	986	1,112	668	724
Immediately D/S Cemetery	4,510	5,289	4,619	5,410	4,606	5,294	4,220	4,661	2,963	3,147
Immediately U/S Confl. with Barons Creek	5,025	5,799	5,097	5,861	5,113	5,784	4,834	5,306	3,554	3,777

TABLE 3-7
HEC-1 MODEL 100-YEAR FLOOD FLOWS FOR DIFFERENT STORM DURATIONS

SITE / CROSSING	2-HOUR STORM		3-HOUR STORM		6-HOUR STORM		12-HOUR STORM		24-HOUR STORM	
	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE
STREAM FB-2										
Channel Near High School	212	268	213	267	194	226	150	165	95	100
South Creek Street	517	834	528	842	527	765	471	612	325	392
Friendship Road Low Water Crossing	767	1,077	785	1,096	778	1,026	704	848	492	557
Stock Pond at Camp	1,756	2,302	1,786	2,328	1,785	2,253	1,666	1,964	1,221	1,345
Immediately U/S Confl. with Barons Creek	2,128	2,751	2,165	2,774	2,175	2,696	2,068	2,440	1,559	1,734
DRY CREEK										
Upper Watershed	1,987	1,987	1,987	1,987	1,892	1,892	1,611	1,611	1,085	1,085
D/S U.S. 87	2,120	2,120	2,111	2,111	2,016	2,016	1,772	1,772	1,228	1,228
Confluence w/ West Fork	2,814	2,832	2,818	2,833	2,717	2,732	2,422	2,432	1,673	1,678
U/S U.S. 290 W (Immed. U/S of Barons Cr	3,038	3,080	3,048	3,082	2,961	2,988	2,686	2,708	1,900	1,928

TABLE 3-8
HEC-1 MODEL FLOOD FLOWS FOR EXISTING AND FUTURE WATERSHED CONDITIONS

SITE / CROSSING	10-YEAR		50-YEAR		100-YEAR		500-YEAR	
	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE
BARONS CREEK								
National Guard Armory	7,863	7,863	11,683	11,683	13,865	13,865	16,380	16,380
290 west of Town	8,503	8,483	12,607	12,577	14,963	14,923	17,682	17,630
S. Bowie Street	8,466	8,444	12,563	12,518	14,902	14,844	17,598	17,528
S. Milam Street	8,458	8,417	12,570	12,494	14,919	14,824	17,628	17,510
Washington Street	8,447	8,400	12,548	12,460	14,897	14,787	17,607	17,471
Upstream Town Creek Confluence	8,419	8,373	12,514	12,426	14,856	14,746	17,556	17,421
Downstream Town Creek Confluence	8,852	8,678	13,184	12,897	15,662	15,314	18,523	18,103
FM 1631 Upstream FB-1 Confluence	8,892	8,693	13,249	12,927	15,742	15,350	18,620	18,146
FM 1631 Downstream FB-1 Confluence	9,548	9,337	14,281	14,330	17,004	17,354	20,149	20,900
Goehmann Road	9,536	9,418	14,275	14,598	17,059	17,649	20,503	21,196
Upstream Wastewater Treatment Plant	9,608	9,623	14,417	14,873	17,395	17,954	20,900	21,548
Downstream Wastewater Treatment Plant	9,812	10,176	15,159	15,797	18,341	19,112	22,091	23,003
Confluence with Stream FB-2	10,564	11,161	16,463	17,355	19,967	21,013	24,073	25,271
Confluence with Pedernales River	10,649	11,216	16,550	17,387	20,069	21,035	24,182	25,281
TOWN CREEK								
West Fork Town Creek	1,136	1,339	1,751	1,998	2,104	2,379	2,512	2,817
East Fork Town Creek / Cross Mountain West	716	737	1,103	1,161	1,325	1,406	1,581	1,690
Confl. below N. Cherry St. and W. Morse St.	1,927	2,116	2,966	3,243	3,563	3,889	4,252	4,634
N. Milam Street	2,173	2,398	3,378	3,697	4,073	4,443	4,879	5,321
Immediately U/S Confl. with Barons Creek	2,352	2,621	3,670	4,049	4,433	4,872	5,318	5,824
STREAM FB-1								
Lower Crabapple Road	1,071	1,321	1,686	2,028	2,042	2,433	2,455	2,899
Ridgewood Drive in Carriage Hills	1,199	1,454	1,913	2,264	2,328	2,731	2,812	3,270
N. Llano Street	1,376	1,666	2,209	2,623	2,705	3,192	3,295	3,852
West Carriage Hills Runoff below N. Llano St.	210	241	352	396	436	488	535	596
Carriage Hills Runoff and FB-1 mainstem	1,712	2,054	2,816	3,286	3,467	4,009	4,248	4,883
Morning Glory / Trailmoor Watershed	595	757	974	1,202	1,198	1,460	1,460	1,759
Immediately D/S Cemetery	2,262	2,737	3,734	4,423	4,619	5,410	5,658	6,560
Immediately U/S Confl. with Barons Creek	2,489	2,970	4,126	4,780	5,097	5,861	6,273	7,149

TABLE 3-8
HEC-1 MODEL FLOOD FLOWS FOR EXISTING AND FUTURE WATERSHED CONDITIONS

SITE / CROSSING	10-YEAR		50-YEAR		100-YEAR		500-YEAR	
	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE
STREAM FB-2								
Channel Near High School	110	146	175	223	213	267	258	318
South Creek Street	250	464	423	705	528	842	654	1,000
Friendship Road Low Water Crossing	372	578	628	906	785	1,096	970	1,316
Stock Pond at Camp	883	1,233	1,450	1,933	1,786	2,328	2,183	2,808
Immediately U/S Confl. with Barons Creek	1,062	1,465	1,748	2,294	2,165	2,774	2,655	3,331
DRY CREEK								
Upper Watershed	1,112	1,112	1,670	1,670	1,987	1,987	2,353	2,353
D/S U.S. 87	1,184	1,184	1,775	1,775	2,111	2,111	2,507	2,507
Confl. with West Fork of Dry Creek	1,572	1,586	2,366	2,380	2,818	2,833	3,344	3,363
U/S U.S. 290 W (Immed. U/S of Barons Creek)	1,680	1,721	2,550	2,586	3,048	3,082	3,627	3,662

Comparisons of the peak flow rates for the 100-year flood as simulated with the HEC-1 model with those previously used in the effective flood insurance study for the City of Fredericksburg as listed in Table 3-1 indicate that the current HEC-1 results generally are slightly higher by about five to fifteen percent. These levels of increase in the peak flood flows of the more urbanized streams, i. e., Town Creek and Stream FB-1, during the last fifteen years are not surprising considering the growth and expansion of the City that has occurred over this same timeframe. However, such increases in the peak flow rates for the upper and middle reaches of Barons Creek probably are due more to differences in engineering judgment and the particular analytical methods employed rather than any changes in these portions of the watershed that have produced additional runoff.

As part of this Flood Protection Planning Study, the peak flow results from the current HEC-1 modeling have been discussed with representatives from FEMA and the Fort Worth District of the Corps of Engineers, and the slight increases above the flood flows used in the original FIS have been noted. Considering FEMA's guidelines for allowing changes in flood flows previously used in determining effective flood insurance base flood elevations and floodplain boundaries, it was jointly agreed that the peak flood flows used in the previous FIS for the City of Fredericksburg would be used to reflect current watershed conditions for all issues related to flood insurance in both this Flood Protection Planning Study and in the Gillespie County flood insurance studies being conducted by the Corps. For all other analyses in this Flood Protection Planning Study, however, the peak flood flows simulated with the HEC-1 model for both existing and future water conditions have been used. This includes the analysis of existing flooding problems and the design of drainage improvements and flood control measures.

3.3 LOCALIZED RUNOFF ANALYSES

During the course of this Flood Protection Planning Study, a number of localized flooding problem areas have been identified and investigated. These are described and discussed in Section 5.0 of this report. As part of the flood investigations for each of these localized flooding problem areas, it has been necessary to estimate the peak rates of runoff from the various subwatersheds and subareas that contribute flood waters to the various problem areas. These flood flows have been used in evaluating the flooding depths associated with storms of different magnitudes and in developing the appropriate drainage improvements and flood control measures needed to mitigate the flooding problems. In some cases, it has been necessary to determine peak flood flows for several different subareas within the total drainage area that contributes stormwater to a particular problem area. The subwatersheds corresponding to each of the designated localized flooding problem areas and their individual subareas are delineated on the map of the City in Plate 3-2.

Typically, the contributing subwatersheds, and the associated subareas, for the localized flooding problem areas are less than a few hundred acres in size; therefore, the determination of peak flood flows has been made using a procedure known as the Rational Formula. With this method, the peak flow rate from a given watershed (Q) is estimated as the product of a runoff coefficient (C), ranging in magnitude from zero to one depending on watershed conditions, times the drainage area (A) expressed in acres times the appropriate rainfall intensity (i) expressed in inches per hour, i. e., $Q = C\ i\ A$. To maximize the peak flow rate, the rainfall intensity usually is taken as the value corresponding to a storm duration that is equal to the time of concentration for a given watershed.

For all of the identified localized problem areas, the Rational Formula was used to calculate the peak flood flows produced by the 2-, 5-, 10-, 25- and 100-year rainfall events. The contributing drainage areas, and various subareas thereof, were determined using the existing five-foot contour topographic maps as provided by the City, along with some field verification of drainage divides. The same maps also were used to determine runoff flow paths for each of the subareas within a particular problem subwatershed. The flow paths were field verified, as necessary. Based on the flow paths, the times of concentration for the various subareas were determined using the SCS procedures as described in Technical Release No. TR-55 and as discussed previously for the HEC-1 modeling in Section 3.2.1. Critical rainfall intensities for each storm frequency were established for durations corresponding to the times of concentration for each of the subareas.

Runoff coefficients for each subarea were estimated for each storm frequency using standard runoff coefficients from the City of Austin's <u>Drainage Criteria Manual</u> (1996). Runoff coefficients corresponding to developed watershed conditions were estimated by using the "fair grass (2-7% slope)" runoff coefficient for pervious areas and the average of the "asphaltic" and "concrete/roof" values for impervious areas. For planning purposes, fully-developed watershed conditions, with an average of 35-percent impervious cover, have been assumed for establishing the appropriate runoff coefficients. The impervious and pervious runoff coefficients for the different storm frequencies and and the resulting fully-developed watershed runoff coefficients as used for the peak flood flow determinations are summarized below.

WATERSHED CONDITION	RUNOFF COEFFICIENTS FOR DIFFERENT STORM FREQUENCIES 2-Year	5-Year	10-Year	25-Year	100-Year
Impervious	0.74	0.78	0.82	0.87	0.96
Pervious	0.33	0.36	0.38	0.42	0.49
Fully-Developed	0.47	0.51	0.53	0.58	0.65

Results from the peak runoff calculations for various subareas within the different localized flooding problem areas are summarized in Table 3-9. For each subarea within the problem area subwatersheds, the drainage area size in acres and the time of concentration in minutes are indicated. Then, for each of the storm frequencies analyzed, the runoff coefficient, the rainfall intensity corresponding to the indicated time of concentration, and the resulting peak runoff rate are presented for each subarea. The names of the localized flooding problem areas listed in the table and the associated subarea names are the same as the identifiers used in Sections 5.0 and 6.0 of this report to reference the various problem areas and subareas when discussing flooding conditions and potential drainage improvements and flood control measures. The names of the localized flooding problem areas and their respective subareas also are noted on the map in Plate 3-2. These names generally correspond to the street names nearest to the problem sites or nearest the subarea discharge locations.

TABLE 3-9
LOCALIZED AREA FLOODING ANALYSIS

LOCALIZED FLOODING PROBLEM AREA / DRAINAGE SUBAREA	AREA	TIME CONC.	2-YEAR EVENT			5-YEAR EVENT			10-YEAR EVENT			25-YEAR EVENT			100-YEAR EVENT		
			C2	i2	Q2	C5	i5	Q5	C10	i10	Q10	C25	i25	Q25	C100	i100	Q100
	acres	min		in/hr	cfs		in/hr	cfs		in/hr	cfs		in/hr	cfs		in/hr	cfs
Friendship Lane																	
Schneider Hill	42.13	17.6	0.47	3.76	74.5	0.51	4.99	107.3	0.53	5.89	131.5	0.58	6.80	166.2	0.65	8.58	235.0
Schubert																	
Schubert	27.50	31.2	0.47	2.71	35.0	0.51	3.61	50.6	0.53	4.25	62.0	0.58	4.91	78.3	0.65	6.23	111.3
Cross Mountain - Milam																	
Cross Mt.	8.08	35.1	0.47	2.51	9.5	0.51	3.35	13.8	0.53	3.95	16.9	0.58	4.56	21.4	0.65	5.79	30.4
Ave D	13.55	27.8	0.47	2.90	18.5	0.51	3.86	26.7	0.53	4.56	32.7	0.58	5.26	41.3	0.65	6.66	58.7
Ave. A	29.38	17.6	0.47	3.76	51.9	0.51	4.99	74.8	0.53	5.89	91.7	0.58	6.80	115.8	0.65	8.58	163.8
Pecan	82.93	43.5	0.47	2.19	85.3	0.51	2.93	123.8	0.53	3.45	151.6	0.58	3.98	191.7	0.65	5.06	273.0
Milam U/S (N)	11.84	27.3	0.47	2.93	38.9	0.51	3.90	56.2	0.53	4.60	68.9	0.58	5.32	87.1	0.65	6.73	123.6
Milam U/S (N & M)	28.24	32.6	0.47	2.63	53.2	0.51	3.51	77.0	0.53	4.13	94.3	0.58	4.77	119.2	0.65	6.06	169.4
Milam U/S (S)	21.17	34.2	0.47	2.55	25.4	0.51	3.40	36.8	0.53	4.01	45.0	0.58	4.64	56.9	0.65	5.88	80.9
Milam D/S & Milam U/S (S)	64.21	54.2	0.47	1.89	56.9	0.51	2.53	82.9	0.53	2.98	101.6	0.58	3.45	128.3	0.65	4.39	183.0
Burbank - Llano																	
Burbank - Llano	47.75	40.5	0.47	2.29	51.4	0.51	3.06	74.6	0.53	3.61	91.4	0.58	4.17	115.5	0.65	5.30	164.4
North Lincoln																	
N. Lincoln & Burbank	99.47	58.9	0.47	1.78	83.3	0.51	2.40	121.5	0.53	2.82	148.8	0.58	3.26	188.1	0.65	4.15	268.3
College - Llano																	
College - Llano	147.51	68.3	0.47	1.61	111.5	0.51	2.17	163.0	0.53	2.55	199.6	0.58	2.95	252.2	0.65	3.76	360.2
College - Travis																	
College & N. Lincoln	275.22	88.7	0.47	1.34	172.9	0.51	1.81	253.6	0.53	2.13	310.5	0.58	2.46	392.4	0.65	3.14	561.1
Travis	341.48	107.0	0.47	1.17	187.3	0.51	1.58	275.7	0.53	1.86	337.4	0.58	2.15	426.5	0.65	2.75	610.3
Trailmoor																	
Trailmoor	84.48	44.5	0.47	2.15	85.5	0.51	2.88	124.3	0.53	3.40	152.2	0.58	3.93	192.4	0.65	4.99	274.0
Morning Glory - Llano																	
Morning Glory	185.12	40.9	0.47	2.27	197.9	0.51	3.04	287.2	0.53	3.59	351.8	0.58	4.14	444.7	0.65	5.26	633.1
Lower Crabapple - Llano	277.98	49.8	0.47	2.00	260.9	0.51	2.68	379.6	0.53	3.16	464.9	0.58	3.64	587.6	0.65	4.64	837.5

TABLE 3-9
LOCALIZED AREA FLOODING ANALYSIS

LOCALIZED FLOODING PROBLEM AREA DRAINAGE SUBAREA	AREA acres	TIME CONC. min	2-YEAR EVENT C2	i2 in/hr	Q2 cfs	5-YEAR EVENT C5	i5 in/hr	Q5 cfs	10-YEAR EVENT C10	i10 in/hr	Q10 cfs	25-YEAR EVENT C25	i25 in/hr	Q25 cfs	100-YEAR EVENT C100	i100 in/hr	Q100 cfs
Carriage Hills																	
Edgewood	42.02	18.4	0.47	3.67	72.5	0.51	4.88	104.5	0.53	5.75	128.1	0.58	6.64	161.9	0.65	8.38	228.9
Driftwood N. & Edgewood	89.73	29.0	0.47	2.83	119.2	0.51	3.77	172.4	0.53	4.44	211.2	0.58	5.13	267.0	0.65	6.50	379.1
Driftwood S.	96.73	36.4	0.47	2.45	111.5	0.51	3.28	161.6	0.53	3.86	198.0	0.58	4.46	250.2	0.65	5.66	355.9
Adams	29.38	35.8	0.47	2.48	34.2	0.51	3.31	49.6	0.53	3.90	60.7	0.58	4.51	76.8	0.65	5.72	109.2
Adams & Driftwood	126.11	36.4	0.47	2.45	145.4	0.51	3.28	210.8	0.53	3.86	258.2	0.58	4.46	326.4	0.65	5.66	464.2
Crestwoods	35.53	37.6	0.47	2.40	40.2	0.51	3.21	58.2	0.53	3.79	71.3	0.58	4.37	90.1	0.65	5.55	128.2
Adams & Crestwoods	161.63	37.6	0.47	2.40	182.7	0.51	3.21	264.8	0.53	3.79	324.4	0.58	4.37	410.0	0.65	5.55	583.4
N. Llano & Adams	181.79	40.6	0.47	2.29	195.3	0.51	3.06	283.4	0.53	3.60	347.1	0.58	4.16	438.7	0.65	5.29	624.5
Frederick	10.08	34.9	0.47	2.52	11.9	0.51	3.37	17.3	0.53	3.97	21.2	0.58	4.58	26.8	0.65	5.82	38.1
Tanglewood & Frederick	13.32	35.7	0.47	2.49	15.6	0.51	3.32	22.6	0.53	3.91	27.6	0.58	4.52	34.9	0.65	5.74	49.7
West Creek St.																	
S. Bowie	24.77	21.2	0.47	3.40	39.6	0.51	4.51	57.0	0.53	5.32	69.9	0.58	6.15	88.3	0.65	7.77	125.1
S. Bowie S. & S. Bowie	34.28	24.2	0.47	3.15	50.8	0.51	4.19	73.2	0.53	4.94	89.8	0.58	5.71	113.4	0.65	7.22	160.9
Edison N.	21.47	49.2	0.47	2.02	20.3	0.51	2.70	29.6	0.53	3.18	36.2	0.58	3.68	45.8	0.65	4.68	65.3
Edison S. & Edison N.	27.90	51.0	0.47	1.97	25.8	0.51	2.64	37.5	0.53	3.11	45.9	0.58	3.59	58.1	0.65	4.56	82.8
W. Creek W.	7.92	28.5	0.47	2.86	10.6	0.51	3.81	15.4	0.53	4.49	18.8	0.58	5.19	23.8	0.65	6.57	33.8
W. Creek E.	9.18	21.8	0.47	3.34	14.4	0.51	4.44	20.8	0.53	5.24	25.5	0.58	6.05	32.2	0.65	7.65	45.7
W. Creek @ S. Milam	17.10	28.5	0.47	2.86	23.0	0.51	3.81	33.2	0.53	4.49	40.7	0.58	5.18	51.4	0.65	6.57	73.0
Old Harper Rd.																	
Armory Rd. E.	77.66	32.0	0.47	2.66	97.1	0.51	3.55	140.5	0.53	4.18	172.1	0.58	4.83	217.6	0.65	6.12	309.2
Highway 290 S.	50.44	19.8	0.47	3.53	83.6	0.51	4.68	120.5	0.53	5.52	147.7	0.58	6.38	186.6	0.65	8.06	264.2
Old Harper Rd. W.	44.01	24.3	0.47	3.14	64.9	0.51	4.18	93.7	0.53	4.92	114.8	0.58	5.69	145.1	0.65	7.19	205.8
Old Harper Rd. Middle	62.74	17.0	0.47	3.83	112.9	0.51	5.08	162.5	0.53	5.99	199.2	0.58	6.92	251.8	0.65	8.73	355.8
Old Harper Rd. E.	61.94	28.0	0.47	2.89	84.0	0.51	3.84	121.4	0.53	4.53	148.8	0.58	5.24	188.1	0.65	6.63	267.0

TABLE 3-9
LOCALIZED AREA FLOODING ANALYSIS

LOCALIZED FLOODING	AREA	TIME	2-YEAR EVENT			5-YEAR EVENT			10-YEAR EVENT			25-YEAR EVENT			100-YEAR EVENT		
PROBLEM AREA		CONC.	C2	i2	Q2	C5	i5	Q5	C10	i10	Q10	C25	i25	Q25	C100	i100	Q100
DRAINAGE SUBAREA	acres	min		in/hr	cfs		in/hr	cfs		in/hr	cfs		in/hr	cfs		in/hr	cfs
Winfried Creek																	
Winfried Creek (WC1)	108.17	28.1	0.47	2.89	146.7	0.51	3.84	212.0	0.53	4.53	259.8	0.58	5.23	328.3	0.65	6.63	466.0
Soutwest Trib. (WC3)	49.88	24.6	0.47	3.12	73.2	0.51	4.15	105.6	0.53	4.90	129.5	0.58	5.66	163.6	0.65	7.16	232.0
SW Trib. (WC2 & WC3)	145.29	28.2	0.47	2.87	196.3	0.51	3.83	283.7	0.53	4.51	347.7	0.58	5.21	439.4	0.65	6.61	623.8
WC2 N, WC2 & WC3	148.03	29.2	0.47	2.82	196.0	0.51	3.75	283.3	0.53	4.43	347.2	0.58	5.11	438.8	0.65	6.48	623.1
Winfried Cr. (WC1 & WC2)	256.19	29.2	0.47	2.82	339.1	0.51	3.75	490.2	0.53	4.42	600.6	0.58	5.11	759.1	0.65	6.47	1078.0
South Trib. (WC4-S)	125.03	29.9	0.47	2.77	163.0	0.51	3.70	235.7	0.53	4.36	288.7	0.58	5.03	364.9	0.65	6.38	518.3
Winfried Creek @ S. Milam	469.82	33.8	0.47	2.57	567.6	0.51	3.43	821.9	0.53	4.04	1006.9	0.58	4.67	1272.6	0.65	5.92	1809.2
Five Points																	
S. Adams	42.70	38.9	0.47	2.35	47.2	0.51	3.14	68.4	0.53	3.70	83.8	0.58	4.28	105.9	0.65	5.43	150.8
Ufer	27.04	30.7	0.47	2.73	34.7	0.51	3.64	50.2	0.53	4.30	61.6	0.58	4.96	77.8	0.65	6.29	110.5
Park St.	11.39	39.4	0.47	2.33	12.5	0.51	3.12	18.1	0.53	3.67	22.2	0.58	4.24	28.0	0.65	5.39	39.9
Live Oak	18.62	22.1	0.47	3.32	29.0	0.51	4.41	41.9	0.53	5.20	51.3	0.58	6.01	64.9	0.65	7.59	91.9
South Lincoln	25.10	34.9	0.47	2.52	29.7	0.51	3.36	43.1	0.53	3.97	52.8	0.58	4.58	66.7	0.65	5.81	94.8
Five Points Intersection	43.72	34.9	0.47	2.52	51.8	0.51	3.37	75.1	0.53	3.97	92.0	0.58	4.58	116.3	0.65	5.82	165.3
Granite	20.55	29.0	0.47	2.83	27.3	0.51	3.77	39.5	0.53	4.44	48.4	0.58	5.13	61.2	0.65	6.50	86.8
Granite @ E. Live Oak	64.28	40.0	0.47	2.31	69.8	0.51	3.09	101.2	0.53	3.64	124.0	0.58	4.20	156.7	0.65	5.34	223.1
South Adams																	
South Adams South	59.78	34.1	0.47	2.56	71.9	0.51	3.41	104.1	0.53	4.03	127.5	0.58	4.65	161.2	0.65	5.90	229.2

TABLE 3-9
LOCALIZED AREA FLOODING ANALYSIS

LOCALIZED FLOODING PROBLEM AREA DRAINAGE SUBAREA	AREA acres	TIME CONC. min	2-YEAR EVENT C2	i2 in/hr	Q2 cfs	5-YEAR EVENT C5	i5 in/hr	Q5 cfs	10-YEAR EVENT C10	i10 in/hr	Q10 cfs	25-YEAR EVENT C25	i25 in/hr	Q25 cfs	100-YEAR EVENT C100	i100 in/hr	Q100 cfs
Highway - Apple																	
Highway St. W.	33.76	45.0	0.47	2.14	33.9	0.51	2.86	49.3	0.53	3.37	60.4	0.58	3.90	76.3	0.65	4.95	108.7
Highway St. N.	19.02	30.0	0.47	2.77	24.8	0.51	3.69	35.8	0.53	4.35	43.9	0.58	5.03	55.4	0.65	6.37	78.7
Highway St. W. & N.	52.78	45.0	0.47	2.14	53.0	0.51	2.86	77.0	0.53	3.37	94.4	0.58	3.90	119.3	0.65	4.95	169.9
Highway St. E., W. & N	75.55	65.9	0.47	1.65	58.6	0.51	2.22	85.5	0.53	2.62	104.7	0.58	3.02	132.4	0.65	3.85	189.0
Eagle St. & Highway St.	112.75	65.9	0.47	1.65	87.4	0.51	2.22	127.6	0.53	2.62	156.3	0.58	3.02	197.5	0.65	3.85	282.0
Franklin W.	4.38	38.7	0.47	2.36	4.9	0.51	3.15	7.0	0.53	3.72	8.6	0.58	4.29	10.9	0.65	5.45	15.5
Franklin E. & W.	7.91	45.2	0.47	2.13	7.9	0.51	2.86	11.5	0.53	3.37	14.1	0.58	3.89	17.8	0.65	4.94	25.4
Apple St. W.	26.48	41.1	0.47	2.27	28.2	0.51	3.03	41.0	0.53	3.58	50.2	0.58	4.13	63.4	0.65	5.25	90.3
Apple St. E. &. W.	32.34	44.4	0.47	2.16	32.8	0.51	2.89	47.6	0.53	3.40	58.3	0.58	3.93	73.7	0.65	4.99	105.0
HW 290 @ Apple	3.99	5.0	0.47	6.32	11.8	0.51	8.36	17.0	0.53	9.87	20.8	0.58	11.40	26.3	0.65	14.20	36.8
Crenweldge D/S Apple	49.36	45.4	0.47	2.13	49.3	0.51	2.85	71.7	0.53	3.36	87.8	0.58	3.88	110.9	0.65	4.93	158.1

Notes:
1. TIME CONC. is the Time of Concentration.
2. C2 is the Runoff Coefficient for the 2-year flood event used in the Rational Formula.
3. i2 is the Rainfall Intensity for the 2-year flood event used in the Rational Formula.
4. Q2 is the runoff for the 2-year flood event calculated by the Rational Formula.

4.0 STREAM HYDRAULIC ANALYSES

4.1 STREAM MODEL DEVELOPMENT

As discussed in the previous section, the currently-effective Flood Insurance Study (FIS) for the City of Fredericksburg was completed in 1980. As part of this earlier study, computerized hydraulic models of portions of several of the principal streams within the City were developed for purposes of establishing flood levels and floodplain boundaries as required by the National Flood Insurance Program. These original FIS hydraulic models were developed using the U. S. Army Corps of Engineers' HEC-2 Water Surface Profiles program. The specific streams modeled in the original FIS included portions of Barons Creek, Town Creek and Stream FB-1, a tributary of Barons Creek that extends through the extreme northeast portion of the City. The modeled reaches of these streams previously have been identified on the map of the area in Figure 3-1, along with the reach of Stream FB-2, another tributary of Barons Creek located south of downtown Fredericksburg, that was modeled pursuant to a 1995 Letter of Map Revision (LOMR) issued by the Federal Emergency Management Agency (FEMA).

For purposes of this Flood Protection Planning Study, copies of the original FIS HEC-2 computer models of Barons Creek, Town Creek and Stream FB-1 were obtained from FEMA. To a large extent, the original FIS models for Barons Creek and Town Creek have formed the basis for the revised models that have been developed as part of this study. Both of these models have been updated with current channel and bridge information through the downtown area. For Stream FB-1, the model recently developed (1996) by the Fort Worth District of the Corps of Engineers as part of the ongoing Gillespie County flood insurance studies has been acquired and used in this Flood Protection Planning Study, with minor modifications. Use of the Corps' model of Stream FB-1 assures consistency between the results from this planning effort and those developed by the Corps in the Gillespie County flood insurance studies. For the same reason, the Corps model of the reach of Barons Creek extending from near the City's wastewater treatment plant south of the downtown area upstream to the U. S. Highway 290 bridge also has been incorporated into the overall HEC-2 model of Barons Creek for purposes of this Flood Protection Planning Study. In addition, the FIS hydraulic models for Barons Creek and Town Creek have been extended upstream of the City in this Flood Protection Planning Study using data and information acquired in the field and from available topographic maps. The Town Creek HEC-2 model also has been extended through the new Cross Mountain subdivision using information provided to the City by the subdivision engineer.

The various reaches of the principal streams in the vicinity of the City of Fredericksburg for which revised and updated HEC-2 hydraulic models now have been developed are identified on the map of the area in Figure 4-1. These are the models that have been used in this Flood Protection Planning Study for the analyses of flood levels corresponding to various storm events, watershed conditions and alternative flood control measures and drainage improvements.

As noted previously, all of the stream hydraulic models are based on the Corps' HEC-2 Water Surface Profiles program (September 1990). Predecessor versions of this program have been widely used for performing backwater calculations in streams and rivers for almost thirty years. As stated in the HEC-2 User's Manual,

> *The program is intended for calculating water surface profiles for steady gradually varied flow in natural or man-made channels. Both subcritical and supercritical flow profiles can be calculated. The effects of various obstructions such as bridges, culverts, weirs, and structures in the flood plain may be considered in the computations. The computational procedure is based on the solution of the one-dimensional energy equation with energy loss due to friction evaluated with Manning's equation. The computational procedure is generally known as the standard step method. The program is also designed for application in flood plain management and flood insurance studies to evaluate floodway encroachments. Also, capabilities are available for assessing the effects of channel improvements and levees on water surface profiles.*

4.2 BARONS CREEK HEC-2 ANALYSIS

The original FIS version of the HEC-2 model of Barons Creek extended from a section below the U. S. Highway 290 crossing approximately two and one half miles southeast of downtown Fredericksburg upstream to a section located near the intersection of U. S. Highway 290 and U. S. Highway 87 on the northwest side of the City. To update this original model to reflect existing channel conditions, 21 cross sections on the mainstem were field surveyed. Seventeen of these cross sections were incorporated into the FIS model to reduce the distance between existing computational sections or to provide descriptions of channel geometry where modifications such as fill placement has occurred. In addition, four of the new surveyed channel cross sections were incorporated into the model to describe conditions at the new low water crossing at

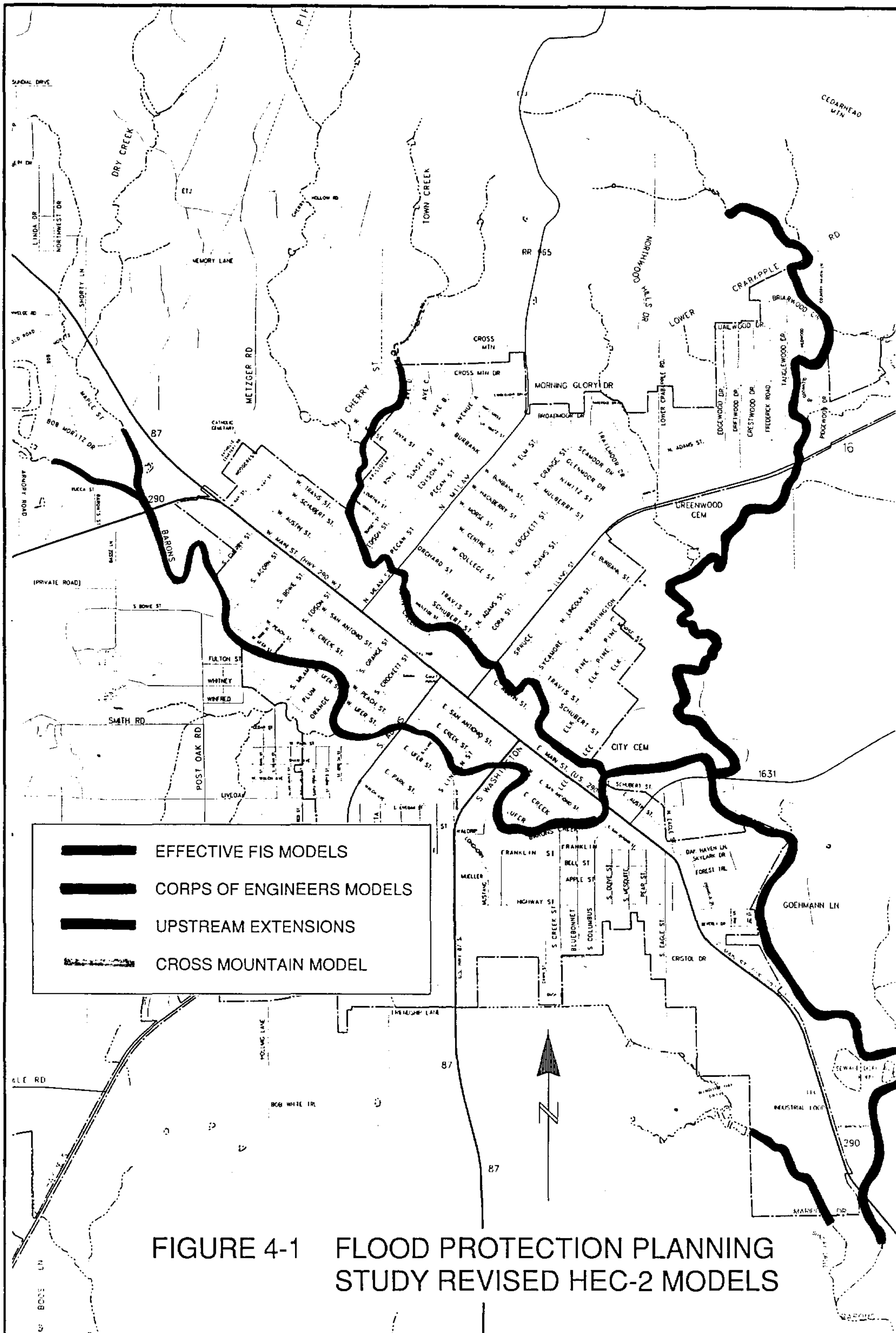

FIGURE 4-1 FLOOD PROTECTION PLANNING STUDY REVISED HEC-2 MODELS

Creek Street. Other computational sections were added to the model to describe the bridge improvements at Adams Street as shown on design plans from the Texas Department of Transportation (TxDOT). The HEC-2 model also was extended upstream of South Bowie Street to above U. S. Highway 290 using TxDOT design plans for the U. S. Highway 290 crossing and information from the City's existing five-foot contour topographic maps for sections along Dry Creek and the mainstem of Barons Creek upstream of U. S. Highway 290.

Between Section 142+89, which is adjacent to the City's wastewater treatment plant southeast of downtown, and Section 252+13 just upstream of Main Street, a channel distance of about two miles, the updated FIS model of Barons Creek was replaced with the Corps' current HEC-2 model of Barons Creek as developed in the Gillespie County flood insurance studies. As explained earlier, this modification was made primarily to assure consistency between the hydraulic results from this Flood Protection Planning Study and those developed by the Corps in the Gillespie County flood insurance studies. In this segment of the Barons Creek model, the Corps section numbering system has been retained, even though it is not compatible with the section numbers in the original FIS model. The section numbers in the model do not affect the hydraulic calculations.

The revised model of Barons Creek, with all of the additional field-surveyed computational sections incorporated and with the Corps' Gillespie County model included, has been operated to simulate water surface profiles along the stream for the 10-, 50-, 100- and 500-year flood events. Two sets of simulations have been made based on flood flows from the original FIS corresponding to existing watershed conditions (Table 3-1) and from the HEC-1 model developed in this study corresponding to future developed watershed conditions (Table 3-8). Results from these simulations in terms of water surface elevations for the 100-year flood are presented in Table 4-1. For comparison purposes, the corresponding 100-year flood water surface elevations from the original FIS also are presented, as are the minimum flowline elevations of the Barons Creek channel at each computational section. Profile plots of these same 100-year flood levels along the length of Barons Creek are presented in Figures 4-2 and 4-3 for the lower and the upper segments of the creek, respectively.

As expected, the 100-year flood water levels corresponding to future watershed

TABLE 4-1
BARONS CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
	9302	1594.04	9302	1594.04	1594.75
	9372	1594.31	9372	1594.31	1594.98
U.S. 290	9382	1594.20	9382	1594.20	1594.80
	9424	1594.74	9424	1594.74	1595.52
	9434	1594.61	9434	1594.61	1595.49
	9550	1596.41	9550	1596.41	1597.24
	9800	1597.47	9800	1597.47	1598.23
	11900	1605.09	11900	1605.09	1605.80
	13400	1611.26	13400	1611.26	1611.57
BEGIN COE SECTIONS	-	-	0	1614.59	1614.91
	-	-	194	1614.77	1615.07
	-	-	379	1614.94	1615.25
	-	-	763	1616.02	1616.32
	-	-	1182	1616.55	1616.86
	-	-	1609	1617.81	1618.10
	16120	1617.11	-	-	-
	-	-	1922	1618.29	1618.54
	-	-	2379	1619.41	1619.67
	-	-	2828	1620.72	1621.00
	-	-	-	-	-
	-	-	3137	1621.49	1621.77
	-	-	3441	1623.22	1623.52
	-	-	3776	1624.32	1624.63
	18000	1621.75	-	-	-
	-	-	3853	1625.37	1625.69
	18035	1621.61	-	-	-
GOEHMANN RD.	18045	1622.69	3872	1625.43	1625.75
	18055	1623.27	-	-	-
	18065	1622.82	3892	1625.88	1626.22
	-	-	3904	1625.85	1626.19
	18100	1624.86	-	-	-
	-	-	3959	1625.89	1626.23
	-	-	4170	1626.63	1626.96
	-	-	4421	1628.06	1628.38
	-	-	4654	1628.66	1629.00
	-	-	-	-	-
	-	-	5097	1629.59	1629.93
	-	-	5551	1630.73	1631.06
	20180	1632.81	-	-	-

TABLE 4-1
BARONS CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
	-	-	6009	1632.32	1632.65
	-	-	6557	1633.56	1633.89
	-	-	-	-	-
	-	-	7022	1634.83	1635.17
	-	-	7483	1636.42	1636.76
	-	-	7867	1637.44	1637.76
	22020	1639.45	-	-	-
	-	-	7979	1637.96	1638.27
	22064	1639.64	-	-	-
F.M. 1631	22074	1640.81	-	-	-
	-	-	8000	1640.48	1641.03
	22086	1640.98	-	-	-
	22096	1641.23	-	-	-
	-	-	8030	1641.25	1641.86
	22120	1641.23	-	-	-
	22155	1641.12	-	-	-
	-	-	8101	1641.33	1641.92
	-	-	8412	1641.67	1642.19
	22600	1641.82	-	-	-
	-	-	8704	1642.30	1642.72
	-	-	8952	1643.47	1643.77
	23400	1643.40	-	-	-
	-	-	9418	1644.57	1644.77
	-	-	10001	1646.16	1646.27
	24400	1647.06	-	-	-
	-	-	10517	1648.38	1648.43
	-	-	10839	1649.46	1649.39
	-	-	10988	1649.52	1649.48
	25015	1649.24	-	-	-
	25057	1649.63	-	-	-
MAIN ST.	25067	1650.17	11110	1649.69	1649.70
	25113	1650.46	-	-	-
	25123	1650.33	-	-	-
	25165	1650.63	-	-	-
	-	-	11228	1651.62	1652.19
END COE SECTIONS	-	-	11262	1651.76	1652.36
	25700	1652.74	25700	1652.46	1653.06
	26250	1655.46	26250	1655.66	1656.44
	26284	1655.69	-	-	-

TABLE 4-1
BARONS CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
CREEK ST.	-	-	26285	1655.30	1656.07
	26294	1655.75	-	-	-
	26306	1655.77	-	-	-
	26316	1655.55	26316	1655.27	1656.05
	26350	1655.35	26350	1657.66	1656.45
	27100	1657.57	27100	1657.70	1658.47
	27700	1659.16	27700	1659.23	1660.18
	-	-	28200	1661.12	1662.13
	-	-	28350	1661.60	1662.59
	29275	1665.12	29275	1665.22	1666.28
	29317	1665.21	29317	1665.30	1666.33
WASHINGTON ST.	29327	1665.17	29327	1665.27	1666.19
	29373	1665.93	29373	1666.05	1667.25
	29383	1666.23	29383	1666.37	1667.95
	29425	1666.37	29425	1666.50	1668.05
	-	-	29640	1666.62	1668.04
	30250	1668.16	30250	1669.05	1670.37
	30270	1668.16	30270	1669.03	1670.34
LINCOLN ST.	30280	1668.01	30280	1668.90	1670.12
	30320	1668.33	30320	1669.17	1670.59
	30330	1668.26	30330	1669.14	1670.75
	30350	1669.04	30350	1669.79	1671.49
	31000	1670.78	31000	1671.61	1673.02
	31625	1673.45	31625	1674.11	1675.33
	-	-	31661	1674.15	1675.37
ADAMS ST.	31663	1673.53	-	-	-
	31673	1673.53	-	-	-
	31727	1673.76	-	-	-
	31737	1674.17	-	-	-
	-	-	31740	1674.35	1675.58
	31775	1674.18	31775	1674.63	1675.89
	32900	1675.62	32900	1675.62	1676.94
	32900	1677.00	32900	1677.00	1678.30
	34068	1683.35	34068	1683.35	1684.62
	34093	1683.41	34093	1683.41	1684.68
ORANGE ST.	34099	1683.37	34099	1683.37	1684.59
	34101	1683.39	34101	1683.39	1684.60
	34107	1683.44	34107	1683.44	1684.71
	34132	1683.47	34132	1683.47	1684.74

TABLE 4-1
BARONS CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
	34750	1684.94	34750	1684.94	1686.12
	34778	1685.16	34778	1685.16	1686.37
MILAM ST.	34788	1685.14	34788	1685.14	1686.13
	34812	1685.58	34812	1685.50	1686.80
	34822	1686.45	34822	1686.45	1688.65
	34850	1686.52	34850	1686.52	1688.66
	35500	1687.62	35500	1687.62	1689.50
	-	-	36275	1690.20	1691.63
	36900	1691.41	36900	1692.39	1693.77
	36928	1691.37	36928	1692.38	1693.77
BOWIE ST.	36943	1692.61	36943	1693.26	1694.69
	36957	1692.63	36957	1693.28	1694.71
	36977	1692.09	36977	1692.86	1694.28
	37000	1692.70	37000	1693.21	1694.55
	-	-	37600	1695.23	1696.45
END FIS SECTIONS	38400	1698.16	38400	1698.66	1699.85
	-	-	41062	1707.69	1709.02
U.S. 290 W	-	-	41162	1707.54	1708.68
	-	-	41189	1708.31	1709.72
	-	-	41239	1710.09	1712.13
	-	-	41770	1713.56	1714.63
	-	-	42230	1718.79	1719.41
	-	-	43020	1723.71	1724.64
	-	-	43990	1727.22	1728.01

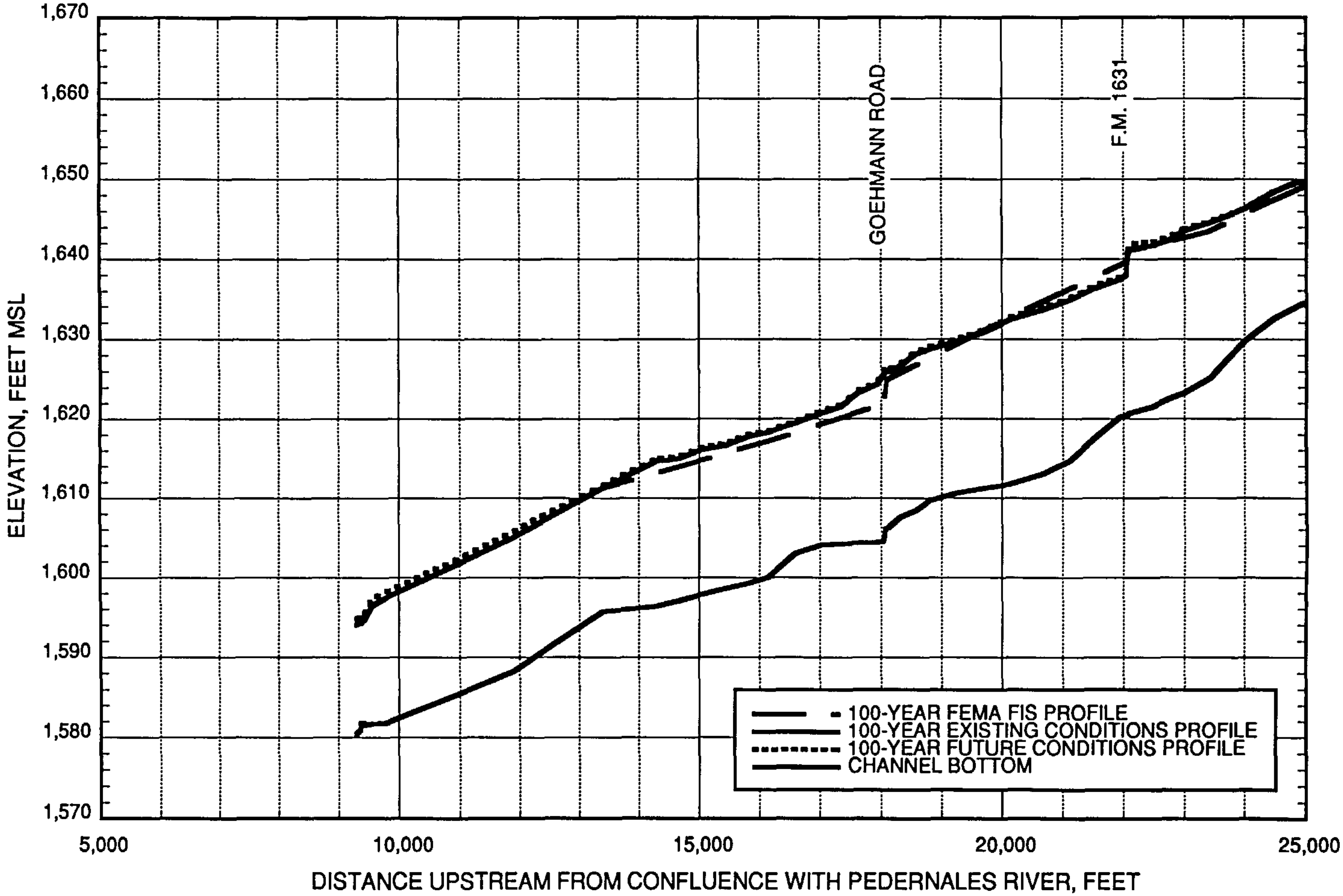

FIGURE 4-2 BARONS CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE PROFILES

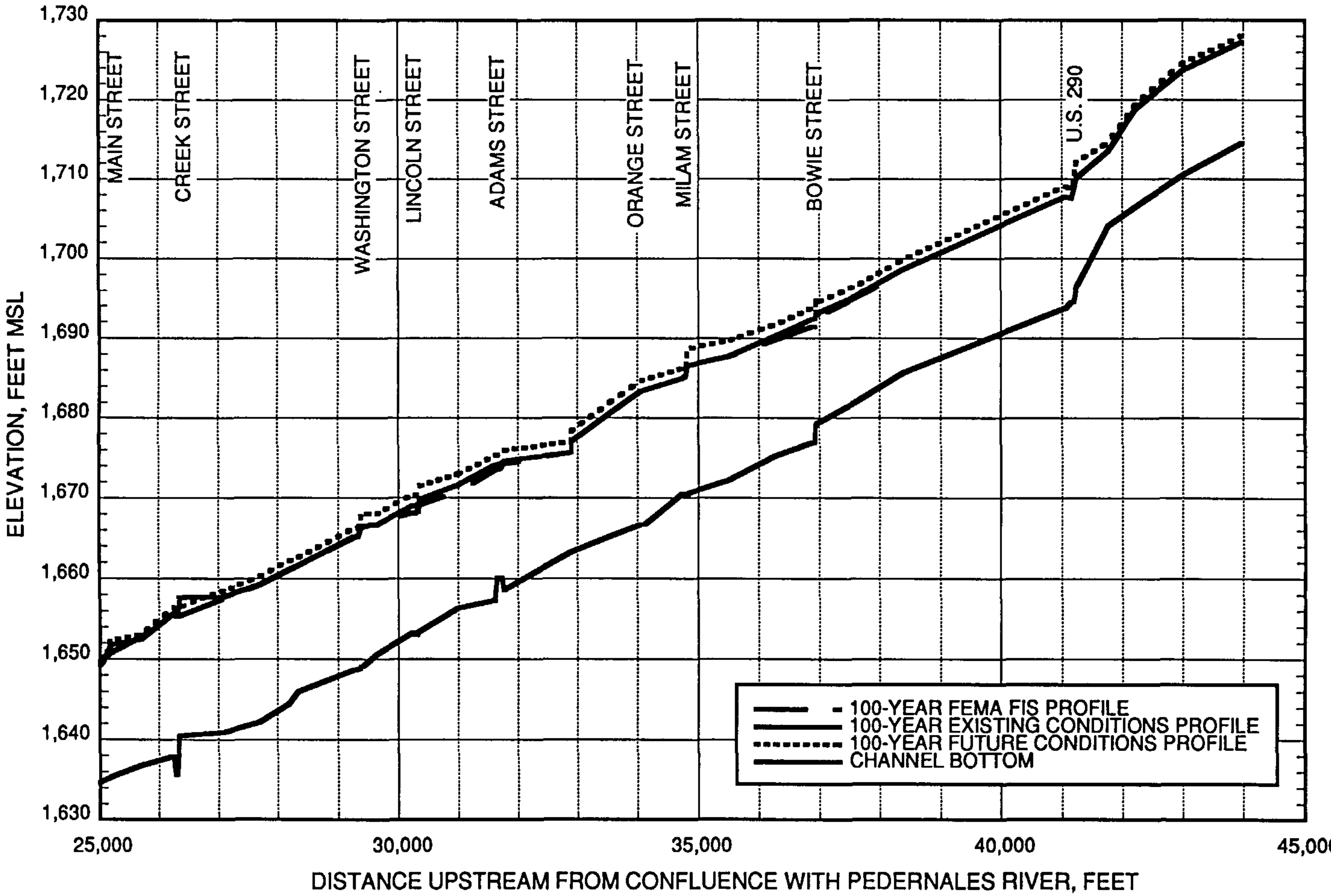

FIGURE 4-3 BARONS CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE PROFILES

conditions are somewhat higher than those for existing watershed conditions. Downstream of Main Street (U. S. Highway 290), the increase in flood levels averages about 0.4 feet, while upstream of Main Street the effect of future development in the watershed is to increase flood levels an average of about 1.2 feet. The maximum increase in flood levels due to the projected future development of the watershed is on the order of 2.2 feet, which occurs upstream of Milam Street.

There are also several reaches along Barons Creek where the 100-year flood levels for existing watershed conditions as simulated with the revised HEC-2 model developed during this Flood Protection Planning Study differ significantly from those determined during the original FIS. In the reach downstream of Goehmann Road, the higher water levels from the revised HEC-2 model appear to be the result of the increased accuracy provided by the new computational sections that have been added to the revised model. The FIS model has only three computational sections to describe the channel geometry from near the City's wastewater treatment plant to Goehmann Road, and the revised model has 13 computational sections for this same reach.

Approximately 1,000 feet upstream of the F. M. 1631 bridge, the flood levels simulated with the revised model for existing watershed conditions exceed those from the original FIS model by about 1.4 feet. Again, this difference in flood levels is due to the improved descriptions of channel geometry through this reach of the updated model. At the Creek Street crossing, increased flood levels in the revised model are the result of including the new low-water bridge in the revised model. The 100-year flood levels immediately upstream of this new bridge as simulated with the revised HEC-2 model are about 2.3 feet higher than those from the FIS model.

The only other significant differences in flood water levels between the results from the revised HEC-2 model and the FIS model occur along the reach from Lincoln Street to Adams Street and near South Bowie Street. These increases also are attributable to the improved accuracy of the revised model reflected in the additional computational sections that have been incorporated to describe existing channel conditions.

4.3 TOWN CREEK HEC-2 ANALYSIS

The HEC-2 model for Town Creek from the original FIS extended from the mouth of the creek at its confluence with Barons Creek upstream to a point near the intersection of

Travis and Bowie Streets in the northwestern part of the City. To update this model to reflect existing conditions, 26 cross sections were surveyed at different locations along the creek to obtain information on various channel and floodplain modifications. Five of these new cross sections were used to describe fill that had been placed in the floodplain of the creek, ten were used to describe modified road crossings at Elk, Crockett and Orange Streets, and ten of the new sections were used to extend the model upstream across Morse Street and up to the new Cross Mountain subdivision. New computational sections were incorporated into the model to reflect these modified conditions. The HEC-2 model of the reach of Town Creek through the new Cross Mountain subdivision, which was developed by the subdivision engineer, also was added to the overall Town Creek model.

Listings of the 100-year flood water surface elevations as simulated with the revised model of Town Creek are presented in Table 4-2 based on flood flows from the HEC-1 model corresponding to existing and future watershed and land use conditions. Also included in the table for comparison purposes are the corresponding 100-year flood levels from the original FIS for the City. Although HEC-2 simulations for the 10-, 50-, and 500-year floods have been made, the resulting flood levels have not been tabulated for this report.

Profile plots of the 100-year flood levels along Town Creek as simulated with the revised HEC-2 model and from the original FIS are presented in Figures 4-4 and 4-5 for the lower and the upper segments of the creek, respectively. Because significant portions of the Town Creek watershed are projected to develop in the future, the flood levels for future watershed conditions in the plots are somewhat higher than those simulated for existing conditions. Increases in 100-year flood levels due to future watershed development on the order of 0.4 to 0.7 feet occur from Elk Street to Adams Street, and upstream of Adams Street, the increases vary between zero and 0.8 feet.

Of most significance are the apparent differences in 100-year flood levels between those from the original FIS and those simulated with the revised HEC-2 model. As shown by the profile plots, the flood levels immediately upstream of Elk Street as simulated with the revised model are as much as 3.5 feet higher than those from the effective FIS. This water level difference apparently is caused by an old bridge structure beneath the new bridge that has never been removed and now obstructs flood flows passing down the creek. From Adams Street to Crockett Street, the revised-

TABLE 4-2
TOWN CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
	230	1645.04	230	1645.04	1648.43
	600	1648.35	600	1648.35	1649.38
	1210	1654.35	1210	1654.36	1655.04
	-	-	1287	1655.09	1655.77
ELK ST.	1298	1655.32	-	-	-
	1332	1655.32	-	-	-
	-	-	1333	1657.57	1658.14
	1430	1657.24	1430	1660.03	1660.79
LOW WATER	1641	1658.96	1641	1660.61	1661.39
CROSSING	1651	1658.73	1651	1660.29	1661.09
	1669	1660.67	1669	1660.75	1661.48
	1689	1661.50	1689	1661.53	1662.27
	1890	1661.65	1890	1661.69	1662.43
	1933	1661.69	1933	1661.72	1662.46
AUSTIN ST.	1943	1661.40	1943	1661.44	1662.12
	1957	1661.52	1957	1661.55	1662.27
	1967	1660.71	1967	1660.71	1661.54
	2000	1663.67	2000	1663.61	1664.52
	2195	1665.91	2195	1665.78	1666.81
	2249	1665.97	2249	1665.84	1666.88
WASHINGTON ST.	2259	1665.68	2259	1665.57	1666.55
	2281	1665.98	2281	1665.88	1666.91
	2291	1666.85	2291	1666.76	1667.98
	2320	1666.87	2320	1666.78	1668.00
	-	-	2600	1667.05	1668.22
	-	-	2850	1667.55	1668.65
	-	-	3100	1668.25	1668.65
	3300	1668.47	-	-	-
	3300	1669.36	-	-	-
	-	-	3250	1669.71	1670.58
	-	-	3450	1670.51	1671.28
	3910	1675.37	3910	1674.29	1674.90
	3982	1676.11	3982	1675.72	1676.34
LLANO ST.	3992	1675.92	3992	1675.54	1676.07
	4028	1676.94	4028	1676.74	1677.53
	4038	1677.03	4038	1676.84	1677.62
	4110	1677.52	4110	1677.36	1678.15
	4635	1680.28	4635	1680.43	1681.20

TABLE 4-2
TOWN CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
	4690	1680.68	4690	1680.82	1681.58
ADAMS ST.	4700	1681.79	4700	1681.85	1682.57
	4720	1682.05	4720	1682.11	1682.94
	4730	1681.26	4730	1681.37	1682.35
	4760	1682.49	4760	1682.49	1683.66
	-	-	4970	1685.86	1685.83
	-	-	5230	1687.92	1688.45
	-	-	5403	1688.18	1688.70
	-	-	5428	1688.58	1689.17
	-	-	5439	1688.55	1689.12
	-	-	5440	1688.33	1688.87
	-	-	5441	1688.33	1688.87
	-	-	5443	1688.76	1689.38
	-	-	5462	1688.71	1689.31
	5470	1688.16	-	-	-
			5494	1688.66	1689.24
CROCKETT ST.	5496	1688.26	-	-	-
	5539	1689.66	-	-	-
	-	-	5541	1692.41	1692.65
	-	-	5561	1692.25	1692.42
	5595	1691.70	5595	1693.09	1693.51
	6250	1693.22	6252	1693.69	1694.13
	6272	1692.92	6272	1693.65	1694.08
ORANGE ST.	6282	1697.70	6282	1697.70	1697.80
	6318	1698.90	6318	1698.54	1698.82
	6328	1698.90	-	-	-
	-	-	6340	1698.34	1698.50
	6350	1698.96	-	-	-
	6810	1699.57	6810	1699.86	1700.34
SCHUBERT ST.	6834	1699.48	6834	1699.83	1700.35
	6886	1699.86	6886	1700.08	1700.57
	6910	1699.57	6910	1699.80	1700.18
	7210	1701.32	7210	1701.31	1701.86
	7245	1702.28	7245	1702.28	1702.20
MILAM ST.	7255	1702.56	7255	1702.58	1702.79
	7285	1703.08	7285	1703.09	1703.60
	7295	1702.95	7295	1702.96	1703.32
	7320	1703.32	7320	1703.33	1703.78

TABLE 4-2
TOWN CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
	7775	1704.17	7775	1705.15	1705.66
	7812	1705.63	7812	1705.62	1706.42
SCHUBERT ST.	7820	1708.25	7820	1708.24	1708.93
	7830	1708.26	7830	1708.25	1708.94
	7838	1707.28	7838	1707.29	1707.95
	7875	1708.85	7875	1708.82	1709.55
	8240	1709.80	8240	1709.80	1710.30
	8273	1709.98	8273	1709.98	1710.47
EDISON ST.	8283	1710.20	8283	1710.20	1710.66
	8297	1710.22	8297	1710.22	1710.68
	8307	1709.87	8307	1709.87	1710.33
	8340	1709.94	8340	1709.94	1710.39
	8710	1711.35	8710	1711.58	1711.99
	8773	1712.71	8773	1712.63	1712.96
TRAVIS ST.	8783	1712.71	8783	1712.64	1712.97
	8797	1712.73	8797	1712.65	1712.98
	8807	1712.76	8807	1712.69	1713.02
	8910	1712.76	8910	1712.70	1713.03
	9700	1715.39	9700	1715.41	1716.17
END FIS	10250	1721.80	10250	1721.47	1722.26
BEGIN EXTENSION	-	-	10635	1723.19	1724.03
	-	-	10810	1724.36	1725.19
MORSE ST.	-	-	10863	1727.85	1728.26
	-	-	10895	1728.83	1729.16
	-	-	10911	1729.77	1730.17
	-	-	11300	1730.81	1731.31
	-	-	11800	1735.24	1735.85
	-	-	12260	1740.38	1741.11
	-	-	12440	1742.51	1743.40
BEGIN CROSS MTN	-	-	12698	1745.25	1745.49
	-	-	12842	1745.42	1745.64
	-	-	12956	1746.89	1747.15
	-	-	13272	1751.79	1751.86
	-	-	13652	1757.08	1757.20
	-	-	14013	1760.85	1761.01
	-	-	14294	1767.19	1767.18
	-	-	14687	1770.99	1771.07
	-	-	14824	1772.45	1772.49

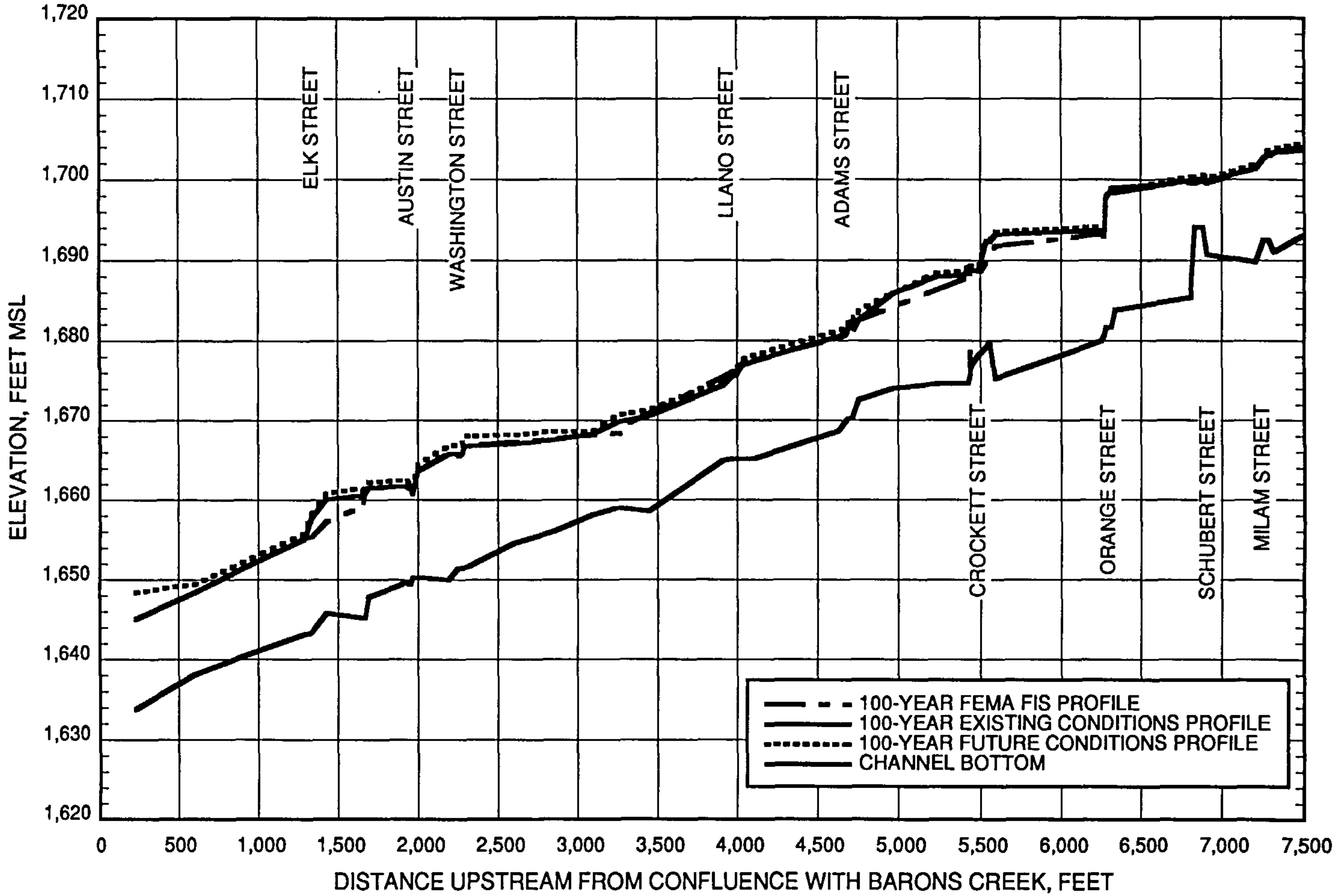

FIGURE 4-4 TOWN CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE PROFILES

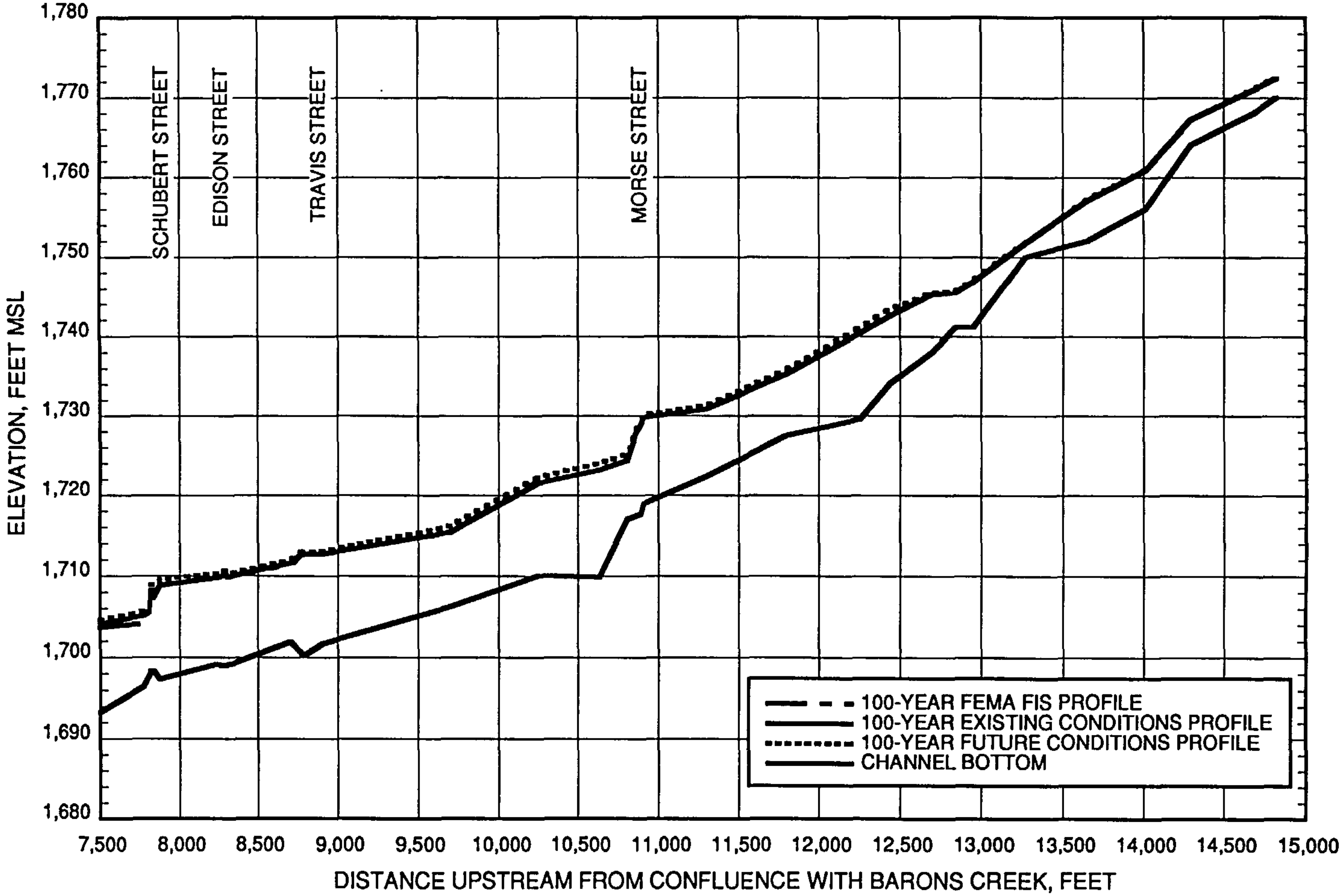

FIGURE 4-5 TOWN CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE PROFILES

model flood levels are up to 1.1 feet above the FIS water surface elevations, which is likely the result of fill material and other channel modifications along this reach of the creek. Immediately upstream of Crockett Street, the increase in flood levels is approximately 3.0 feet, which apparently has been caused by bridge and culvert modifications at this crossing. Upstream of Orange Street, there is very little difference between the revised-model results and those from the original FIS.

4.4 STREAM FB-1 HEC-2 ANALYSIS

For this tributary of Barons Creek, the HEC-2 model from the original FIS, which extended from the mouth of the creek near F. M. 1631 upstream to above Briarwood Circle in the Carriage Hills subdivision, has been replaced entirely with the revised HEC-2 model developed by the Corps of Engineers in the Gillespie County flood insurance study. The revised model now extends up to Lower Crabapple Road, almost 3,000 feet beyond the end of the original FIS model. The revised model incorporates considerably more detail with regard to describing channel geometry. It includes 95 computational sections from the confluence at Barons Creek to Lower Crabapple Road, whereas the original FIS model included only 19 computational sections.

The revised model of Stream FB-1, with all of the additional computational sections incorporated in accordance with the Corps' Gillespie County model, also has been operated to simulate water surface profiles along the stream for the 10-, 50-, 100- and 500-year flood events. Again, simulations have been made using flood flows for existing watershed conditions (Table 3-1) and future developed watershed conditions (Table 3-8). Results from these simulations in terms of water surface elevations for the 100-year flood are presented in Table 4-3. For comparison purposes, the corresponding 100-year flood water surface elevations from the original FIS also are presented. Profile plots of these same 100-year flood levels along the length of Stream FB-1 are presented in Figures 4-6 and 4-7 for the lower and the upper segments of the watercourse, respectively.

Examination of the flood profiles indicates that development of the watershed will likely cause 100-year flood levels to increase on the order of 0.6 to 1.2 feet along Stream FB-1 from near its mouth up to about the Llano Highway (State Highway 16). These flood level increases are not expected to dramatically affect floodplain boundaries.

TABLE 4-3
STREAM FB-1 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
BARONS CREEK	-	-	0	1632.53	1641.03
CONCRETE CHANNEL	-	-	31	1634.94	1640.61
	-	-	52	1637.33	1641.03
	100	1638.69	-	-	-
	-	-	76	1337.73	1641.26
	-	-	113	1638.97	1641.21
LOW WATER CROSS.	-	-	171	1639.50	1641.07
	200	1640.17	-	-	-
	-	-	231	1640.92	1641.65
	-	-	412	1641.18	1641.95
	-	-	561	1642.34	1643.14
	-	-	942	1643.93	1644.82
	-	-	1192	1645.38	1646.41
	-	-	1441	1646.62	1647.62
	-	-	1597	1647.80	1648.80
	-	-	1791	1648.51	1649.49
	-	-	1949	1649.19	1650.12
	-	-	2167	1651.03	1651.85
	-	-	2341	1652.44	1653.26
	-	-	2514	1654.02	1654.98
	-	-	2820	1655.51	1656.50
	2250	1647.96	-	-	-
	2251	1649.43	-	-	-
	-	-	3009	1655.99	1656.92
	-	-	3201	1655.74	1656.88
	-	-	3276	1658.74	1659.94
	-	-	3363	1659.78	1661.11
	-	-	3524	1660.16	1661.40
	3120	1656.49	-	-	-
	-	-	3855	1661.94	1663.13
	-	-	3963	1662.72	1663.63
	-	-	4051	1663.32	1664.34
	-	-	4167	1666.50	1667.69
	-	-	4280	1667.18	1668.26
	-	-	4408	1667.89	1668.94
	-	-	4506	1669.13	1669.80
	-	-	4656	1670.38	1671.44

TABLE 4-3
STREAM FB-1 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
	-	-	4792	1670.76	1671.76
	-	-	5139	1671.25	1672.19
	-	-	5366	1671.79	1672.65
	-	-	5458	1672.24	1673.10
	-	-	5549	1672.58	1673.47
	-	-	5725	1674.39	1675.40
	-	-	5820	1674.44	1675.45
	5400	1672.39	-	-	-
	-	-	6047	1675.09	1675.99
	-	-	6350	1677.02	1677.67
	-	-	6510	1678.33	1679.05
	-	-	6698	1678.71	1679.42
	-	-	6911	1679.24	1679.88
	7000	1679.57	-	-	-
	-	-	7143	1680.44	1681.02
	-	-	7334	1681.63	1681.87
	-	-	7618	1683.08	1683.16
	-	-	7899	1684.52	1684.41
	7820	1681.93	-	-	-
	-	-	8204	1685.37	1685.25
	-	-	8436	1685.79	1685.67
	-	-	8631	1686.32	1686.20
	-	-	8890	1687.36	1687.23
	-	-	9075	1688.70	1688.58
	-	-	9360	1691.18	1691.04
	9280	1691.04	-	-	-
	-	-	9548	1692.80	1692.69
	-	-	9768	1694.65	1694.51
	-	-	9959	1695.89	1695.71
	-	-	10107	1696.48	1696.30
	-	-	10270	1697.50	1697.35
	-	-	10375	1698.86	1698.69
	-	-	10581	1700.42	1700.23
	-	-	10781	1702.28	1702.10
	10600	1699.08	-	-	-
	-	-	11170	1704.37	1704.22
	-	-	11353	1705.71	1705.55

TABLE 4-3
STREAM FB-1 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
	11360	1703.68	-	-	-
	11380	1703.61	-	-	-
LLANO HWY	-	-	11540	1708.34	1708.12
	11444	1708.46	11600	1710.27	1710.30
	11494	1708.50	-	-	-
	-	-	11727	1710.68	1710.70
	-	-	12039	1710.80	1710.81
	-	-	12450	1711.87	1711.88
	-	-	12646	1712.62	1712.62
	12500	1711.90	-	-	-
	-	-	12876	1713.56	1713.57
	-	-	12970	1715.70	1715.70
	-	-	13050	1716.44	1716.45
	-	-	13244	1718.50	1718.50
	-	-	13355	1720.16	1720.16
	-	-	13452	1720.96	1720.96
	-	-	13563	1721.35	1721.35
	13410	1718.18	-	-	-
RIDGEWOOD DR.	-	-	13642	1721.28	1721.45
	-	-	13800	1722.88	1722.56
	-	-	13930	1723.65	1723.27
	-	-	14102	1724.85	1724.50
	-	-	14262	1726.34	1726.02
	-	-	14429	1728.85	1728.29
	-	-	14525	1730.69	1730.26
	14400	1726.01	-	-	-
	-	-	14756	1732.92	1732.62
	14800	1729.22	-	-	-
	-	-	15100	1734.31	1733.94
	-	-	15303	1735.00	1734.61
	-	-	15455	1736.52	1736.12
	-	-	15588	1738.47	1738.10
	-	-	15661	1739.53	1739.12
	-	-	15740	1740.01	1739.62
	-	-	15900	1741.98	1741.60
	-	-	16073	1744.21	1743.69
	-	-	16302	1746.01	1745.61

TABLE 4-3
STREAM FB-1 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
	-	-	16532	1748.22	1747.87
	16400	1748.18	-	-	-
	-	-	16676	1749.27	1748.69
	-	-	16791	1752.17	1751.70
	-	-	16874	1753.56	1753.11
	-	-	17053	1756.02	1755.54
	-	-	17186	1756.22	1755.76
LOWER CRABAPPLE	-	-	17334	1756.85	1756.73
	-	-	17362	1757.28	1756.99
	-	-	17460	1757.91	1758.17
	-	-	17810	1759.50	1759.89
	-	-	18092	1761.72	1762.13
	-	-	18480	1766.53	1767.07
	-	-	19304	1779.48	1779.90

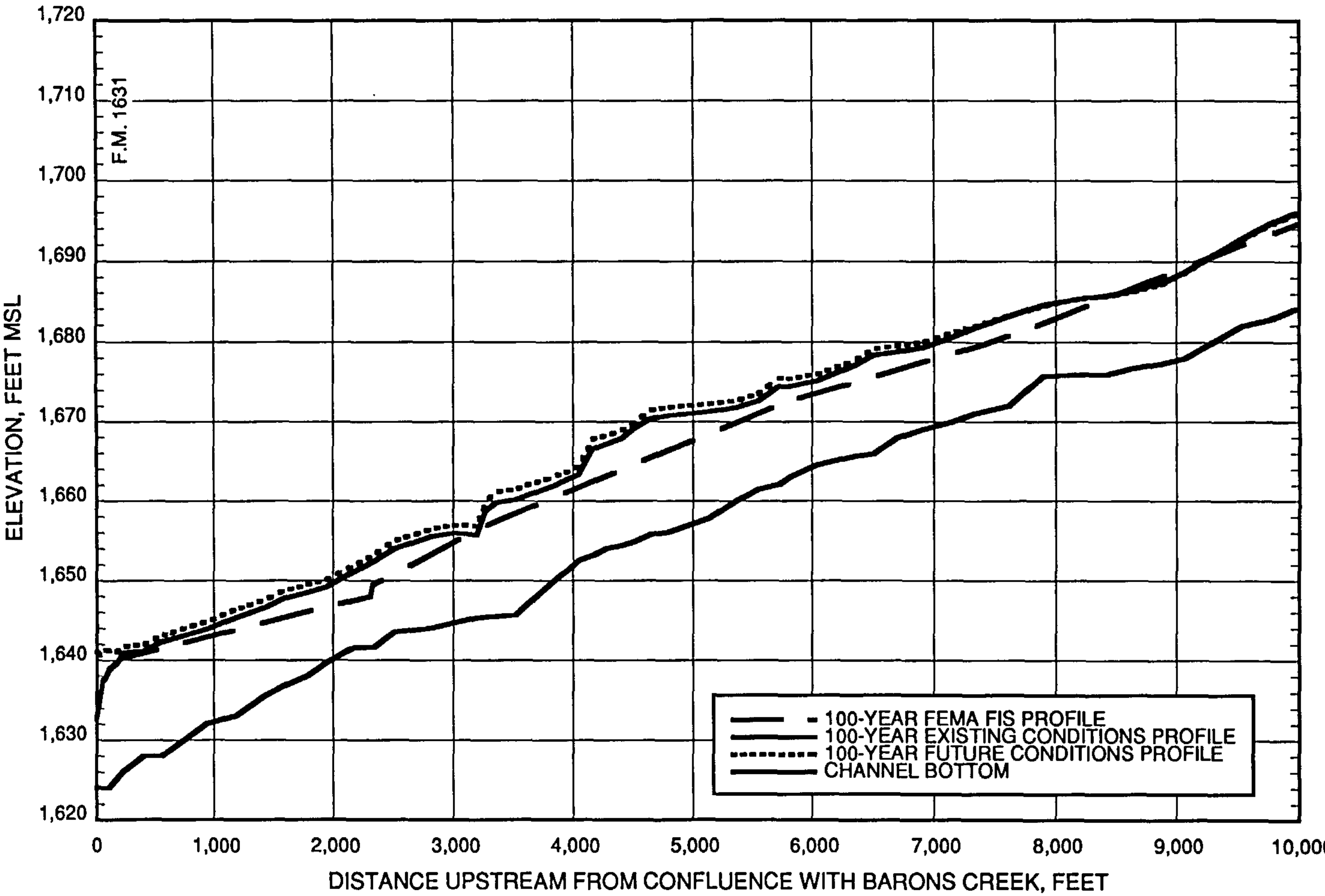

FIGURE 4-6 STREAM FB-1 100-YEAR FLOOD HEC-2 WATER SURFACE PROFILES

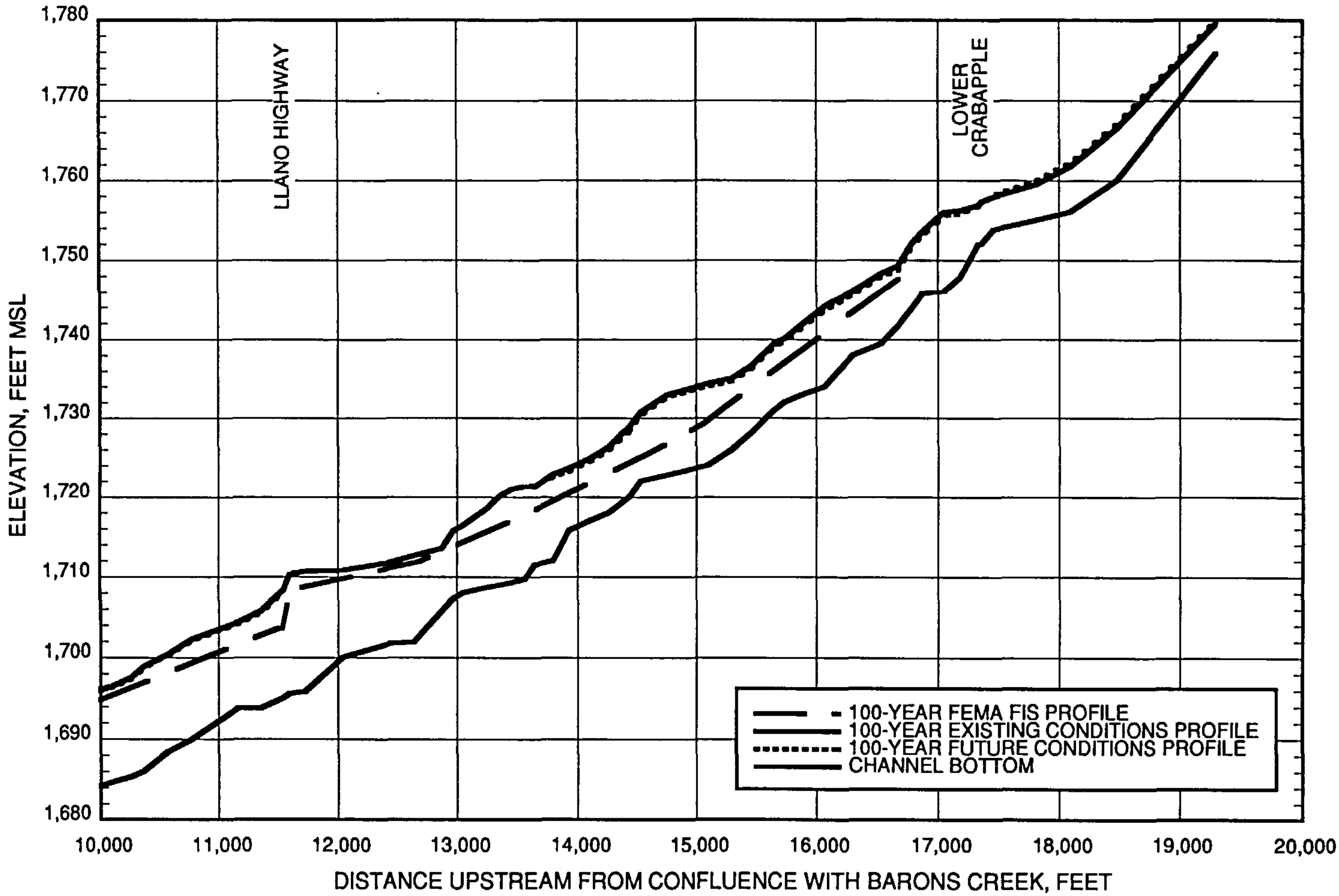

FIGURE 4-7 STREAM FB-1 100-YEAR FLOOD HEC-2 WATER SURFACE PROFILES

Of more concern are the 100-year flood level increases indicated from the original FIS results to the water surface elevations simulated with the revised HEC-2 model. Between the confluence of the stream and the Llano Highway, the flood levels from the revised model exceed those from the original FIS by as much as 7.5 feet, and typically are on the order of 3.5 feet. Fortunately, the existing land use along this reach of the stream is primarily agricultural, so it does not appear that there are any residential structures affected by the increased flood levels. Also, comparisons of the floodplain top widths simulated with the two hydraulic models do not indicate significant discrepancies, and the simulated depths of flow also are similar. Hence, it appears that differences in the topography and channel geometry used in developing the models are the primary causes of the flood level deviations. With the revised model having been developed based on current and much more detailed topographic information as compiled by the Corps, the revised model should be more accurate than the original FIS model.

Another reach of the stream where significant increases in flood levels are indicated from the original FIS results to the water surface elevations simulated with the revised HEC-2 model is through the Carriage Hills subdivision between the Llano Highway and Lower Crabapple Road. Again, maximum increases in 100-year flood levels are on the order of 7.0 feet. Certainly, this would appear significant, but when the top widths of the respective floodplains are examined, the revised HEC-2 simulation actually results in a decrease in the extent of effective FIS 100-year floodplain.

5.0 EXISTING FLOODING PROBLEMS

5.1 LOCALIZED FLOODING

Extensive efforts have been undertaken to identify existing localized flooding problems throughout the planning area. Through numerous meetings with City personnel and officials and extensive field inspections and surveys of known flooding sites, a list of specific localized areas believed to encompass the most severe existing flooding problems or those with the greatest potential for flooding has been compiled. The localized flooding problem areas previously have been identified on the vicinity map of the City of Fredericksburg in Plate 3-1. Specific flooding problem sites within the various localized flooding problem areas are identified on the map of the City in Plate 5-1, and they are listed and generally described in Table 5-1.

It should be noted that flooding in the localized problem areas generally is limited in depth to a few feet and typically is caused by either the lack of drainage facilities or inadequately sized drainage facilities. Often, this type of flooding is more of a nuisance, than it is life threatening. Still, such flooding can cause considerable property damage and can result in considerable disruption of community activities. Generally, it is primarily the stormwater runoff from the immediate drainage area of these various localized flooding problem areas that produces the excessive floodwater quantities and depths. Solutions to these types of flooding problems often involve installation of larger-capacity drainage facilities or possibly combinations of localized drainage improvements that can benefit several flooding areas. Hence, these types of flooding problems are somewhat different from those normally associated with the major creeks and streams that flow through the City where flooding may be more extensive and often requires implementation of major drainage improvements and more regional-type flood control facilities in order to achieve significant flood damage reductions.

In this Flood Protection Planning Study, sites of known or suspected localized flooding have been evaluated with respect to flooding severity (water depths) and frequency. This evaluation generally has been accomplished by performing hydraulic calculations using surveyed or measured topographic data with estimates of localized runoff quantities for the 10-year storm event. This magnitude of storm has been selected for the analyses because it is considered to be a reasonable storm event for which flood protection might be provided in many of the flood prone area of the City that are already substantially developed. The runoff quantities for the 10-year storm event, expressed as peak flow rates, associated with specific subareas within each of the identified localized flooding problem areas previously have been presented in Table 3-9 in

TABLE 5-1
LOCALIZED FLOODING PROBLEM SITES

PROBLEM SITE DESIGNATION	LOCALIZED FLOODING PROBLEM SITE	TYPE OF PROBLEM	PROBLEM LOCATION
L1	Friendship Lane	Roadway Overtopping	Low Water Crossing
L2	Friendship Lane	Street/House Flooding	South Creek Subdivision
L3	Friendship Lane	Limited Swale Capacity	Friendship Lane from S. Creek St. to S. Washington
L4	Friendship Lane	Roadway Overtopping	S. Washington
L5	Friendship Lane	Limited Swale Capacity	Friendship Lane from Channel to S. Washington
L6	Friendship Lane	Street Flooding	W. Highway and S. Adams
L7	Schubert Street	Street/Property Ponding House Flooding	Between Bowie and Acorn
L8	Cross Mountain-Milam	Street/House Flooding	N. Milam from W. Centre St. to Travis St.
L9	Cross Mountain-Milam	Street/House Flooding	W. College St. and Pecan St.
L10	Cross Mountain-Milam	Street/House Flooding	W. Centre and Edison
L11	Cross Mountain-Milam	Street/House Flooding	W. Burbank @ Avenue A
L12	Cross Mountain-Milam	Street/House Flooding	W. Burbank @ Avenue D
L13	Cross Mountain-Milam	Overflow to College-Llano Area	W. Milam from W. Burbank to Glenmoor
L14	Cross Mountain-Milam	Overflow to Trailmoor Subarea	N. Milam @ Broadmoor

TABLE 5-1

LOCALIZED FLOODING PROBLEM SITES

PROBLEM SITE DESIGNATION	LOCALIZED FLOODING PROBLEM SITE	TYPE OF PROBLEM	PROBLEM LOCATION
L15	Burbank-Llano	Street/Structure Flooding	N. Llano from Burbank to Hackberry and E. Burbank @ N. Llano
L16	North Lincoln	Street Flooding/ Possible House Flooding	N. Lincoln from Morse to W. College
L17	College-Llano	Major Street Flooding/ Potential Structure Flooding	College @ Llano
L18	College-Travis	Major Street Flooding	Sycamore
L19	College-Travis	House Flooding	Channel Upstream of Washington
L20	College-Travis	Road Overtopping	Washington and Orchard
L21	College-Travis	Potential House Flooding	Channel between Orchard and N. Pine
L22	College-Travis	Street/House Flooding	N. Pine and Travis
L23	College-Travis	Channel Erosion	North of Travis, Upstream of N. Lee
L24	College-Travis	Minor Channel Erosion	Cemetery Channel Downstream of N. Lee
L25	Trailmoor	Street Ponding	Trailmoor Upstream of Llano
L26	Trailmoor	Roadway Overtopping	Llano

TABLE 5-1

LOCALIZED FLOODING PROBLEM SITES

PROBLEM SITE DESIGNATION	LOCALIZED FLOODING PROBLEM SITE	TYPE OF PROBLEM	PROBLEM LOCATION
L27	Morning Glory-Llano	Roadway Overtopping for 100-yr Storm	Llano @ Lower Crabapple
L28	Carriage Hills	Street Flooding/High Velocities	Edgewood @ Channel
L29	Carriage Hills	Street Flooding/Curb Overtopping/House Flooding	204 & 206 Driftwood
L30	Carriage Hills	Street/House Flooding	112 & 114 Driftwood
L31	Carriage Hills	Street Flooding	N. Adams from Driftwood to just East of Crestwood
L32	Carriage Hills	Street Ponding	Frederick @ Channel Inlet
L33	Carriage Hills	Street Ponding	Tanglewood @ Channel Inlet
L34	Carriage Hills	Street Overtopping	Ridgewood @ Stone Ridge Tributary
L35	West Creek St.	Street Ponding Potential House Flooding	S. Bowie & San Antonio to Edison & W. Creek St.
L36	Old Harper Road	Street Overtopping	Low Water Crossing on Armory Road
L37	Old Harper Road	Street Overtopping	Low Water Crossing on Basse Lane
L38	Old Harper Road	Street Flooding Future Conditions	Swales along Basse Lane from Duderstadt at Low Water Crossing

TABLE 5-1
LOCALIZED FLOODING PROBLEM SITES

PROBLEM SITE DESIGNATION	LOCALIZED FLOODING PROBLEM SITE	TYPE OF PROBLEM	PROBLEM LOCATION
L39	Old Harper Road	Street Overtopping	Culverts under Duderstadt at Basse Lane
L40	Old Harper Road	Street Flooding Future Conditions	Swale along South Side of S. Bowie approximately 900 feet west of Culverts
L41	Old Harper Road	Street Overtopping Future Conditions	Box Culvert under S. Bowie 200 feet west of Post Oak Road
L42	Winfried Creek	Street Overtopping 100-yr Future Conditions	S. Milam Bridge near Whitney
L43	Winfried Creek	Erosion	Downstream of Box Culvert on post Oak Blvd. just north of Smith Road
L44	Five Points	Street Overtopping 5-yr Storm	Culverts @ Intersection of Park, Live Oak and S. Lincoln
L45	Five Points	Street Ponding	West of Five Points on Park St.
L46	Five Points	Street Ponding	E. Ufer 100 to 300 feet west of S. Lincoln
L47	Five Points	Street & Building Flooding	East of Five Points on E. Live Oak
L48	Five Points	Street Ponding and Building Flooding	Channel betwee Granite and E. Live Oak
L49	Five Points	Street Overtopping 25-yr Storm	Culvert under Granite and Ufer

TABLE 5-1

LOCALIZED FLOODING PROBLEM SITES

PROBLEM SITE DESIGNATION	LOCALIZED FLOODING PROBLEM SITE	TYPE OF PROBLEM	PROBLEM LOCATION
L50	South Adams	Potential House Flooding Future 25-yr Storm	Channel Downstream of Friendship Lane
L51	Highway-Apple	Street Flooding	Highway St. from Mesquite to S. Eagle
L52	Highway-Apple	Street Flooding and Potential House Flooding	Apple St. and Pearl St. and area between Mesquite and S. Eagle
L53	Highway-Apple	Street Overtopping	Eagle Street Low Water Crossing
L54	Dry Creek	Erosion/Backwater	Downstream of Hwy 87 @ Old Road Culverts
L55	Dry Creek	Street Overtopping/ Building Flooding	Dry Creek Tributary at Crenwelge near Gold Road

Section 3.3 of this report. For most of the specific flooding problem sites, the depth of flooding has been quantified by determining the "normal" depth of flow for the 10-year storm event. For this purpose, the Manning's uniform flow equation has been applied to specific channel or street cross sections within each of the identified localized flooding problem areas. Field surveys were conducted to measure the geometry of these channel and street cross sections. The specific sections where field surveys were performed are delineated on the map of the City in Plate 5-2. Ground and street slopes were derived from the field survey data or from the available five-foot contour topographic maps of the City.

Results from the hydraulic calculations for selected channel and street cross sections within the identified localized flooding problem areas are summarized in Table 5-2. The specific locations of these cross sections are the same as the survey cross sections identified on the map of the City in Plate 5-2, and they are referenced by the same section designations. In Table 5-2, a number of pertinent flood-related parameters are provided for each of the cross sections analyzed. These are defined below:

Localized Flooding Problem Site - Specific site identified on the map in Plate 5-1 where flooding problems occur.

Cross Section Designation - Specific section identified on the map in Plate 5-2 where field surveying has been performed to obtain geometry and elevation data.

Drainage Subarea - Specific watershed area delineated on map in Plate 3-1 that contributes Flood Flow to the Cross Section.

Conveyance Slope - Longitudinal slope of the street, channel, swale, ditch or other conveyance facility carrying the stormwater runoff.

10-Year Flood Flow - Peak flow rate for the 10-year storm event.

Height of Curb, Bank or Edge of Pavement - Vertical distance from street low point or channel flowline to the top of curb, top of channel bank or edge of pavement, channel flowline above which floodwater overflows and area flooding occur.

TABLE 5-2
STREET AND CHANNEL FLOODING DEPTHS

LOCALIZED FLOODING PROBLEM SITE (PLATE 5-1)	CROSS SECTION DESIGNATION (PLATE 5-2)	DRAINAGE SUBAREA (PLATE 3-1)	CONVEY. SLOPE	10-YR FLOOD FLOW (cfs)	HEIGHT OF CURB, BANK OR EDGE OF PAVEMENT (feet)	10-YR FLOOD DEPTH (feet)	AVERAGE VELOCITY (fps)	STREET OR CHANNEL WIDTH (feet)	FLOW TOP WIDTH (feet)
L2	LX01	Friendship	0.014	464.0	0.70	1.36	13.21	20	34
L3	LX02	Friendship	0.012	305.0	2.93	2.91	5.30	59	59
L3	LX03	Friendship	0.012	305.0	2.54	2.53	5.53	53	53
L5	LX05	Friendship	0.009	225.0	0.86	1.58	5.20	58	58
-	LX07	Friendship	0.015	146.0	3.51	2.67	9.26	8	8
L8	LX11	Milam D/S	0.007	101.6	0.87	1.39	1.34	64	158
L8	LX12	Milam D/S	0.007	101.6	0.66	0.73	2.59	64	110
L9	LX13	Pecan	0.007	151.6	0.00	0.86	6.08	28	38
L9	LX14	Pecan	0.005	151.6	0.46	0.78	4.50	67	67
L9	LX15	Pecan	0.005	151.6	0.42	0.91	4.27	87	87
L10	LX16	Pecan	0.017	151.7	0.32	0.73	7.16	65	66
L10	LX17	Pecan	0.004	15.0	0.71	0.41	2.46	48	35
L10	LX18	Pecan	0.004	15.0	0.59	0.53	2.33	48	42
L10	LX19	Pecan	0.004	30.0	0.79	0.63	2.06	48	75
L11	LX20	Avenue A	0.024	91.7	0.75	0.98	4.82	18	38
L11	LX21	Avenue A	0.024	91.7	0.75	0.94	7.49	7	38
L12	LX22	Avenue D	0.008	32.7	1.53	0.72	6.91	9	9
-	LX23	Cross Mtn.	0.028	16.9	0.98	0.25	7.79	9	9
-	LX24	Cross Mtn.	0.031	16.9	0.65	0.31	6.27	39	17
L13	LX25	Milam U/S	0.013	94.3	2.50	1.45	5.77	42	24
L15	LX26	Llano	0.004	91.4	0.42	0.72	3.68	74	74
L15	LX27	Llano	0.006	91.4	0.92	0.57	5.30	42	42

TABLE 5-2

STREET AND CHANNEL FLOODING DEPTHS

LOCALIZED FLOODING PROBLEM SITE (PLATE 5-1)	CROSS SECTION DESIGNATION (PLATE 5-2)	DRAINAGE SUBAREA (PLATE 3-1)	CONVEY. SLOPE	10-YR FLOOD FLOW (cfs)	HEIGHT OF CURB, BANK OR EDGE OF PAVEMENT (feet)	10-YR FLOOD DEPTH (feet)	AVERAGE VELOCITY (fps)	STREET OR CHANNEL WIDTH (feet)	FLOW TOP WIDTH (feet)
L16	LX28	N. Lincoln	0.008	148.8	0.67	0.60	7.19	39	39
L17	LX29	College	0.006	203.6	1.00	1.00	6.55	51	53
L18	LX30	Travis	0.005	310.5	1.61	1.31	9.01	31	31
L19	LX31	Travis	0.004	320.0	2.20	3.73	3.74	24	68
L21	LX33	Travis	0.004	320.0	2.40	3.05	5.35	24	27
L22	LX34	Travis	0.004	337.4	1.55	1.46	6.73	43	43
L22	LX35	Travis	0.008	337.4	0.47	1.34	6.77	81	81
L22	LX36	Travis	0.008	337.4	0.83	1.23	7.33	84	84
L24	LX39	Travis	0.019	337.4	3.10	2.56	7.18	24	28
L25	LX40	Trailmoor	0.010	152.2	0.83	0.72	8.30	40	33
L27	LX44	Morn. Glory-Llano	0.014	464.9	3.00	1.74	18.25	21	17
L28	LX46	Edgewood	0.015	128.1	1.40	0.74	11.63	15	15
L28	LX47	Edgewood	0.015	128.1	1.38	0.75	11.63	15	15
L29	LX48	Driftwood N.	0.012	211.2	0.83	0.62	9.31	39	39
L29	LX49	Driftwood N.	0.009	211.2	0.90	0.67	8.62	38	38
-	LX50	Driftwood N.	0.009	211.2	0.70	0.71	6.76	43	72
L30	LX51	Driftwood S.	0.003	198.0	0.75	1.23	4.34	39	73
L30	LX52	Driftwood S.	0.003	198.0	0.68	1.15	4.46	38	70
L31	LX53	N. Adams	0.001	258.2	0.63	1.64	2.99	39	90
L31	LX54	N. Adams	0.001	258.2	0.68	1.64	3.12	39	84
L31	LX55	N. Adams	0.011	347.1	1.70	1.38	11.39	40	34
L32	LX56	Frederick	0.014	21.2	0.83	0.30	6.24	11	11

TABLE 5-2

STREET AND CHANNEL FLOODING DEPTHS

LOCALIZED FLOODING PROBLEM SITE (PLATE 5-1)	CROSS SECTION DESIGNATION (PLATE 5-2)	DRAINAGE SUBAREA (PLATE 3-1)	CONVEY. SLOPE	10-YR FLOOD FLOW (cfs)	HEIGHT OF CURB, BANK OR EDGE OF PAVEMENT (feet)	10-YR FLOOD DEPTH (feet)	AVERAGE VELOCITY (fps)	STREET OR CHANNEL WIDTH (feet)	FLOW TOP WIDTH (feet)
L33	LX57	Tanglewood	0.014	27.6	0.83	0.35	6.87	11	11
L38	LX62	Old Harper	0.013	199.2	1.00	1.10	3.96	22	22
L42	LX66	Winfried	0.017	1006.9	5.00	4.71	9.79	35	33
L44	LX72	Five Points	0.190	22.2	NA	0.39	4.42	32	32
L44	LX73	Five Points	0.019	22.2	0.78	0.46	4.78	27	27
L47	LX74	Five Points	0.009	92.0	0.00	1.00	3.65	37	37
L47	LX75	Five Points	0.009	92.0	0.50	1.63	3.60	38	38
L47	LX76	Five Points	0.009	92.0	0.83	0.91	4.44	49	49
L45	LX77	Five Points	0.014	22.2	2.88	1.32	4.30	8	8
L45	LX77	Five Points	0.014	51.3	1.46	0.82	6.25	20	20
L45	LX78	Five Points	0.014	22.2	0.90	0.79	2.87	23	23
L45	LX78	Five Points	0.014	51.3	0.42	0.83	3.82	29	29
L48	LX81	Five Points	0.019	124.0	2.30	1.94	2.52	60	51
L50	LX84	S. Adams	0.007	127.5	2.00	1.15	4.00	42	34
-	LX88	Apple	0.013	87.8	2.50	1.03	4.57	48	26
-	LX90	Apple	0.013	87.8	2.50	1.50	5.52	20	16

Notes: 1) D/S - Downstream
2) U/S - Upstream
3) Ch. - Channel
4) 1+00 - Survey Station
5) Numbers before street names are adresses
6) NA - Not applicable

10-Year Flood Depth - Depth of floodwater above street low point or channel flowline.

Average Velocity - Average velocity of floodwater flowing in street or channel.

Street or Channel Width - Width of street or channel conveying floodwater.

Flow Top Width - Width of floodwater surface within or outside of street or channel.

The extent of flooding at each cross section has been evaluated by comparing the "10-Year Flood Depth" to the "Height of Curb, Bank or Edge of Pavement" to determine if floodwater overflows out of a conveying street or channel occur and, thereby, cause potential flooding of adjacent properties. Also, if the calculated "Flow Top Width" at a particular section significantly exceeds the available "Street or Channel Width", it also is likely that potential flooding of adjacent properties is occurring. The "Average Velocity" of the flowing floodwater has been examined at each section to assess whether or not the momentum of the flowing floodwater might cause street curbs and channel banks at corners and bends to be overtopped and, thereby, contribute to the potential flooding of adjacent properties.

In some cases, other hydraulic and hydrologic calculations have been performed, including additional HEC-1 runoff simulations, to provide additional information when necessary. Also, some of the localized flooding problem areas have streets with nearly flat or negative slopes which preclude the performance of meaningful uniform flow hydraulic calculations. In these cases, the severity of the flooding problems has been subjectively examined based on such factors as the relative elevations of threatened structures and flood conveyance systems, the general volume of traffic that might be disrupted during flooding events, and/or the quantity of runoff flowing through a potential flooding problem site. Where necessary, the hydraulic capacity of roadway culverts has been analyzed using standard culvert hydraulics procedures similar to those described in the Texas Highway Department's (now Texas Department of Transportation) Drainage Manual (1985).

Following is a discussion of flooding conditions within each of the localized flooding problem areas. Where appropriate, the specific flooding problem sites are referenced in accordance with the site designations listed in Table 5-1. These sites also are identified on the map of the City in Plate 5-1.

5.1.1 Friendship Lane Drainage

One of the most significant localized flooding problems is along Friendship Lane in the southern part of the City. The watershed that contributes stormwater runoff to this area originates in the vicinity of Schneider Hill southwest of the downtown area and generally extends eastward along Friendship Lane. Runoff from the watershed tends to concentrate east of U. S. Highway 87 (South Washington Street) and flow along much of Friendship Lane. For most storms, Friendship Lane becomes impassable at the low water crossing between South Creek Street and South Eagle Street (Site L1).

Based on results from the HEC-1 runoff model of the Barons Creek basin, the peak flow rate for the 10-year flood at the low water crossing (Site L1) has been determined to be 578 cubic feet per second (cfs). East of South Washington Street, the swale along the north side of the Friendship Lane roadway (Site L3) has very limited floodwater-carrying capacity, and stormwater tends to spill northward into a natural low area. This stormwater then must flow through the South Creek subdivision through a shallow (8.5 inches deep), relatively narrow (16 feet wide) trapezoidal channel. The floodwater-carrying capacity of this channel is less than the peak flow rate of the 2-year storm, and during the occurrence of larger storms (5- and 10-year rainfall events), floodwaters threaten the adjacent houses and cause streets within the subdivision to be impassable (Site L2). Additionally, although the drainage swales on both sides of Friendship Lane upstream of the South Creek subdivision have sufficient capacity to convey about the 10-year flood flow, the numerous driveway crossings have undersized culverts that force the water out of the swales and over the road or onto the adjacent land.

The box culvert (4' x 4') under U. S. Highway 87 (South Washington Street) also is undersized, with capacity for conveying floodwaters less than that produced by the 10-year storm. Larger storms cause floodwaters to flow over the highway and become impounded upstream (Site L4). Near the upstream end of the watershed, at West Highway Street just west of South Adams Street, a large culvert discharges stormwater onto West Highway Street from Schneider Hill and State Highway 16 (10-Year Flow = 127 cfs). This concentrated flow crosses both Highway Street and South Adams, posing a significant traffic hazard (Site L6), and then discharges into a channel leading southeastward toward Friendship Lane, where there is limited floodwater-carrying capacity (Site L5).

Overall, the Friendship Lane is characterized as a significant localized flooding problem area, even for storms as small as the 2-year event.

5.1.2 Schubert Street Ponding

A natural low-lying area and a surrounding depression exists on Schubert Street between Bowie and Acorn Streets northwest of the downtown area (Site L7). The main portion of the depression is located south of Schubert Street on two vacant town lots (1/2-acre each). It has been reported that historically a natural pond existed at this location and that it was filled as the area developed for residential use. The existing depression collects and stores stormwater runoff from the surrounding watershed, which encompasses about 28 acres. Preliminary calculations indicate that the existing low-lying area naturally (predeveloped watershed) would have flooded up to about elevation 1,732.4 feet msl (above mean sea level) during the occurrence of a 100-year storm with a 12-hour duration. Ponding of stormwater in this area now has been partially alleviated by an 18-inch storm drain and inlets that were installed by the City. However, frequent ponding of stormwater still occurs since the discharge capacity of this storm drain is only about 9 cfs, and the peak flow rate of the two-year storm is on the order of 35 cfs. With the existing storm drain, the 100-year, 12-hour storm causes stormwater runoff to pond in the depression area to an elevation just over 1,731 feet msl. This elevation would be close to the finished-floor elevations of adjacent residential structures, and would result in up to two feet of floodwater over the Schubert Street roadway.

Concerns have been expressed by the owners of the remaining vacant lots in the depression area at Schubert Street that the current ponding of stormwater runoff prevents the construction of buildings on these lots. Of course, construction of buildings on these lots would require filling of the depression, which, in turn, would increase the flooding levels on both the currently vacant lots and the adjacent lots with existing houses. Increased flood damages very likely would result.

The Schubert Street ponding is considered to be a significant localized flooding problem area; although, the problem involves primarily the existing vacant lots.

5.1.3 Cross Mountain - Milam Drainage

This area is generally bounded by North Milam Street to the east, Town Creek to the south and west, and Cross Mountain to the north. The streets, including Cross Mountain Drive, Avenue D, Avenue A, Pecan Street and Milam Street, are the primary drainageways for conveying stormwater runoff in this area. Because of limited street floodwater-carrying capacities, relatively large drainage areas and flat ground slopes, there is some interaction and cross-over of floodwater flows between adjacent streets. Significant street and some house flooding occurs in the vicinity of the lower segments of Milam and Pecan Streets near their intersections with College and Centre Streets (Sites L8 & L9). Relatively large drainage areas for both Milam and Pecan Streets contribute runoff to these low, flat areas (64 and 82 acres, respectively). At 604 Milam (Site L8), the 10-year flood flow has been determined to be approximately 102 cfs, which produces a water depth on the order of 1.4 feet. At Pecan and West College Streets (Site L9), the 10-year flood flow is about 152 cfs. There is no curb on the east side of Pecan Street at this location and there is significant potential for flooding of the residences. Even with a curb, there would not be sufficient floodwater-carrying capacity in the street. On West College Street, the depth of the 10-year flood flow exceeds the curb height. Another problem occurs at the intersection of Edison and Centre Streets (Site L10). There is no curb on the east side of Edison just south of Centre and the 10-year flood depths are on the order of 0.7 to 0.9 feet. Some stormwater flow spills over to Milam Street down Centre Street at this location.

Stormwater runoff from a portion of the Cross Mountain residential area flows down Avenue A to Burbank Street (Site L11). At this point, the natural slope of the land generally takes stormwater flows south to the existing flooding problem areas along Pecan Street. There is a small curb-cut on the south side of Burbank Street that allows these flows to proceed southward down a grassed channel. The estimated 10-year flood depth in Burbank just upstream of the curb-cut is approximately 1.6 feet, and the corresponding depth in the downstream channel is on the order of one foot. This depth exceeds the curb height at the edge of the channel. Because of the depth of flow in Burbank Street and the inlet control limitation on flow through the curb-cut, some of the stormwater flows down Burbank Street to the northwest toward Avenue D.

A potential flooding problem exists at the concrete channel into Town Creek at the western end of Burbank Street near Avenue D (Site L12). This channel section is about nine feet wide and about one foot deep. Although the channel itself can carry the 10-year flood flow, the inlet to the channel limits the inflow and, thereby, forces stormwater to flow over the street curb to reach the channel. The overflows of stormwater from the Avenue A drainage area, flowing down Burbank Street, make this condition worse. This could result in flooding of the adjacent homes, especially during larger storms, i. e., greater than the 10-year event. Another similar channel into Town Creek exists at the end of Cross Mountain Drive where it intersects Avenue D. However, because of a smaller drainage area, less stormwater runoff flows to this point, and with the inlet to this channel being approximately 13 feet wide, there does not appear to be a potential problem at this location, even for the 100-year storm.

Some stormwater runoff from the area between Cross Mountain and North Milam Street normally flows down Milam all the way to Town Creek. However, for higher intensity storms, some of this stormwater spills over to the east and contributes to flooding problems in the College-Llano drainage area. These spill-overs generally occur along Milam Street from Burbank Street north to Glenmoor Street (Site L13), with some additional spill-overs at the intersection of Burbank and Milam Streets (Site L14). These spill-over waters eventually flow to the Trailmoor Drive area and contribute to the existing flooding problems there.

5.1.4 Burbank - Llano Drainage

This drainage area includes approximately 40 acres west of North Llano Street and north of Hackberry Street and an additional 18 acres east of North Llano Street, including the drainage to North Lincoln Street upstream and north of College Street. The primary flooding problem site within this area is the portion of North Llano Street between Burbank and Hackberry Streets (Site L15), where all of the stormwater runoff from the western 40 acres is concentrated within the street section and sometimes overtops the curb. The 10-year flood flow at this location (about 90 cfs) produces water depths that overtop the curb along North Llano by about 0.3 feet, and the associated velocity is nearly four feet per second (fps). These conditions are especially dangerous where the floodwaters cross North Llano Street and flow to the east. At the entrance to Burbank Street, the flow has a velocity over five feet per second, and as the stormwater turns to flow down Hackberry Street, the depth of the flow is about one foot. These conditions produce a dangerous situation for a major roadway, and they also pose a flooding threat to adjacent houses and businesses.

5.1.5 North Lincoln Drainage

Downstream of the Burbank-Llano flooding problem area is the North Lincoln problem area. This area encompasses an additional 52 acres of watershed. Along North Lincoln Street, the 10-year flood flow is nearly 150 cfs. Although uniform flow calculations indicate that this quantity of flow is just barely conveyed within the existing street section, irregularities in ground slopes and section geometry along the street probably result in overtopping of the curb a some locations (Site L16). The 10-year flood depth of 0.6 feet in the street, with a velocity of over seven feet per second, represents a relatively hazardous situation and would make crossing the street in a vehicle difficult, at best. For storms greater than the 10-year event, some homes along the street also would be threatened with flooding.

5.1.6 College - Llano Drainage

The College-Llano flooding problem area encompasses about 148 acres of contributing watershed that produces a concentrated 10-year flood flow of about 200 cfs that discharges across Llano Street at its intersection with College Street (Site L17). The depth associated with this flow is on the order of one foot, and the velocity is about 6.5 fps. This depth of flow is just at the curb height along College Street. Because of the rapid expansion and contraction of the flow as it crosses Llano Street, the actual depths may reach as much as 1.7 feet at some points including along the eastside curb of Llano Street. The 25-year flood flow produces depths well above (>0.5 feet) the curb that could cause floodwaters to reach the adjacent residential and commercial structures.

5.1.7 College - Travis Drainage

This area is downstream of the College-Llano, Burbank-Llano and North Lincoln flooding problem areas; therefore, it receives very high inflows of stormwater runoff that must be conveyed primarily through the streets and some shallow grass/earth channels. The total drainage area contributing runoff encompasses about 340 acres, including the North Milam area that very likely contributes floodwater spill-overs. The 10-year flood flows range from 310 to 340 cfs from the intersection of East College and

North Lincoln Streets to the eastern end of Travis Street. Water depths produced by the 10-year storm in Sycamore Street south of College Street (Site L18) are on the order of 1.3 feet, with velocities about nine feet per second. This flow is contained within the street, however, by the existing high curbs, which are approximately 1.6 feet in height. The stormwater flowing down Sycamore Street enters a grass-lined channel that traverses in the direction of the intersection of Washington and Orchard Streets. The 10-year flow depth in this channel is estimated to be on the order of 3.7 feet, which exceeds of the banks of the channel (Site L19).

Water from the channel discharges through three culverts under Washington Street. The combined capacity of these culverts is equivalent to about the two-year flood flow; consequently, the 10-year flood flow would overtop Washington Street by more than 0.5 feet. The limited conveyance capacity of the channel and culverts in this area creates the potential for flooding of nearby homes by storms slightly greater than the 10-year event (Site L20).

The stormwater discharges from the culverts under Washington Street flow across Orchard Street into a channel with tree-lined banks. The 10-year flood flow in this channel produces depths on the order of three feet, which is about 0.6 feet above the top of the channel banks (Site L21). These floodwaters then discharge into North Pine Street, where they are contained within the existing high curbs, similar to those along Sycamore Street. From North Pine Street, the floodwaters discharge into East Travis Street, where they flow down a channel-like depression along the north side of the street, but within the curb. The 10-year flood flow overtops the curb along this street and reaches to within 0.5 feet (elevation) of the adjacent houses (Site L22). Downstream of Elk Street, the Travis Street floodwaters discharge into a grass/earth channel. Just upstream of North Lee Street, this channel is significantly eroded due to the high flood flows and velocities caused by the runoff from the upper watersheds (Site L23). Floodwaters in the channel pass beneath a bridge/culvert at North Lee Street and then, finally, into a grass/earth channel through the City Cemetery to Stream FB-1. Some erosion is occurring within the channel through the cemetery (Site L24).

Stormwater discharges on the order of 300 cfs through and across residential streets with water depths greater than one foot are considered a major flooding problem. Most of these streets are impassable with the occurrence of less than the one-year storm event, and there is potential for flooding of residences by storms greater than about the

10-year event.

5.1.8 Trailmoor Drainage

The Trailmoor Drainage flooding problem area encompasses about 85 acres and lies primarily west of Trailmoor Street, east of North Milam Street and north of Nimitz Street. Stormwater runoff from several streets is concentrated in Trailmoor Street at its intersection with North Adams Street. The specific flooding problem site is along about 200 feet of a flat section of Trailmoor just northwest of North Llano Street (Site L25). Flood backwater conditions along this segment of Trailmoor Street are caused by the flow restriction created by the inlet to the existing culverts under North Llano Street. Even the two-year storm event produces flood backwater conditions on Trailmoor that result in overtopping of the North Llano roadway (Site L26).

5.1.9 Morning Glory - Llano Drainage

The concrete-lined channel adjacent to Lower Crabapple Road and the culverts under North Llano Street at Lower Crabapple Road have been analyzed to evaluate their floodwater-carrying capacities. While the channel is capable of conveying the 100-year flood flow, inlet restrictions to the box culvert under North Llano Street cause overtopping of the roadway during the 100-year storm (Site L27).

5.1.10 Carriage Hills Drainage

Significant localized flooding problems exist in the drainage area that lies generally north of the Llano Highway (Highway 16) and south and east of Lower Crabapple Road. The greatest number of reported drainage problems are located along Edgewood Drive and Driftwood Drive in the Carriage Hills subdivision. A concrete-lined channel conveys stormwater runoff through this subdivision from the currently undeveloped area west of Edgewood Drive to Driftwood Drive. Although this channel has sufficient capacity for conveying the 10-year flood flows (fully-developed watershed conditions), flooding problems occur at the inlets and outlets of the channel segments (Sites L28 & L29). Channelized flood flows from the west discharge at over 11 feet per second into Edgewood Drive. Because of the abrupt change in section geometry at this location, the inlet to the channel on the opposite side of the street appears to control the flow, which forces some of the stormwater over the curb (Site L28).

Similar conditions occur where the channel discharges into Driftwood Drive, at which point the additional stormwater runoff flowing down Driftwood Drive from the north causes substantial overtopping of the curb and flooding of adjacent houses (Site L29). Although uniform flow calculations indicate that the 10-year flood flow could be contained within the street section, the unsteady nature and high energy of the flow are sufficient to push the water over a half a foot above the curb.

Downstream (south) on Driftwood Drive, additional drainage problems exist through a flat section of the roadway (Site L30). Because of the flat slope (0.003 feet per foot), the 10-year flood depths exceed the curb height by about 0.5 feet, and the flow spreads to the adjacent houses on the east side of the street. Stormwater flows produced by storms equal to or greater than the 10-year event will cause some flooding of residential structures.

Additional runoff flowing into the intersection of Driftwood Drive and North Adams Street causes the 10-year flood depths to exceed 1.6 feet along North Adams Street (Site L31) and to pond to about 2.5 feet at the inlet to the existing grass channel between North Adams Street and the Llano Highway (Highway 16). The grass channel appears to have sufficient floodwater-carrying capacity for the 10-year storm, except for the flow limitations at the inlet.

Other localized flooding problems in this area occur along the existing 11-foot wide concrete curb channel (10-inch curb height) that conveys stormwater flows from Frederick Road to Tanglewood Drive and thence to Stream FB-1. Inlet control conditions limit the inflows into these channels. This may cause some ponding at the inlets on both of these streets (Sites L32 & L33).

Another localized drainage problem in this area relates to the culverts under Ridgewood Drive where the tributary from the Stone Ridge development crosses in route to Stream FB-1. The three existing 30-inch pipes are not capable of conveying the 10-year flood flow (fully-developed watershed conditions) without causing overtopping of the roadway. The 100-year flood flow would overtop the roadway by approximately 1.5 feet, with most of the flow passing over the road. The limited channel capacity of this tributary through the Carriage Hills subdivision also is of concern with regard to flooding of adjacent houses.

5.1.11 West Creek Street Drainage

This flooding problem area generally encompasses a subwatershed bounded by West Main Street on the north, Peach Street on the south, Orange Street on the east, and Acorn Street on the west. The basic flooding problem in this area is street ponding caused by the extremely flat ground slopes. Specific flooding problem sites occur along South Bowie Street from San Antonio Street to West Creek Street and along San Antonio Street from South Bowie Street to South Edison Street (Site L35). Curb overflows of stormwater along both South Bowie Street and San Antonio Street could impact houses in the block bounded by to South Bowie, San Antonio, Edison and West Creek Streets.

5.1.12 Old Harper Road Drainage

This area lies generally southwest of Barons Creek and south of Old Harper Road (also known Basse Road and South Bowie Street) and Armory Road. Currently, this area is undeveloped, and stormwater flows drain northward across both roads at several low water crossings. Also, the existing swales along Old Harper Road have the capacity to convey close to the 10-year flood flow (fully-developed conditions). There is a single 24-inch corrugated metal pipe under a private drive marked as Duderstadt Lane, and a 4'x2' box culvert under the South Bowie portion of Old Harper Road near Post Oak Road. These pipes and culverts appear to be undersized for handling future flood flow conditions (Sites L36 through L41). Depending on the extent of upstream development and the types of drainage facilities constructed, future flood flows are projected to be as much as 40 percent greater than existing flows.

5.1.13 Winfried Creek Drainage

This area encompasses a large, well defined watershed south of Barons Creek. The drainage area covers nearly 470 acres of relatively steep terrain above the bridge at South Milam Street. Currently, most of this area is undeveloped. Most of the creek crossings have sufficient capacity under existing conditions and also generally would convey the 10-year flood flows under fully-developed watershed conditions. One concern is the bridge at South Milam Street (Site L42). For existing watershed conditions, the 100-year flood flow passes through the bridge without overtopping.

However, for fully-developed watershed conditions, there would be some overtopping. This does not appear to threaten any houses; although, the increase in future flood flows is dependent on the level of development that occurs and the types of drainage facilities constructed in the area. Some erosion around other bridges and culverts also has been observed (Site L43).

5.1.14 Five Points Area

There is a significant existing drainage problem in the vicinity of the Five Points intersection. This area is located at the intersection of Park Street, South Lincoln Street and East Liveoak Street. The 10-year flood flow entering this intersection is approximately 114 cfs. For conveyance of stormwater through this intersection, there are two sets of culverts (two storm drains from Park Street and one box culvert from Liveoak Street) with a combined capacity equal to approximately the five-year flood flow (Site L44). This limitation forces water over the roadways and causes ponding on Park Street (Site L45). Floodwaters from Park Street overflow into the park area to the north and flow toward Ufer Street through a grass swale. Fairly significant ponding of floodwaters occurs on Ufer Street at an existing low point (Site L46), in part because of an undersized culvert on private property just north of the street. Flow that does pass through the box culvert at the Five Points intersection discharges into a swale downstream along Liveoak Street. In this swale, the depth of the 10-year flood flow exceeds the elevation of the building to the northwest of Live Oak (Site L47). These floodwaters combine with runoff from the street to the south of Live Oak (Walnut Street) and then flow through a small swale northward toward Granite Avenue. This swale has approximately a 10-year flood flow capacity (Site L48). This limitation, combined with the close proximity of the adjacent buildings, results in frequent flooding of area properties. Stormwater discharges from the swale area then enter a culvert under the Granite and Ufer intersection (Site L49). This culvert discharges into Barons Creek. The inlet capacity of this culvert is sufficient to handle approximately the 10-year flood flow from the upstream drainage area. The flooding in the Five Points area is considered a significant problem with respect to streets and structures.

5.1.15 South Adams Drainage

This area lies south of Schneider Hill and is generally located south of Highway 16, west of South Adams Street, east of Stadium Drive and north of Billie Drive. Although this area technically is outside the planning area, its drainage conditions have been evaluated since part of the Schneider Hill subwatershed, which is in the planning area, would contribute stormwater runoff to this subwatershed under natural conditions. However, due to various drainage pipes and channels that are in place today, stormwater from the Schneider Hill subwatershed now discharges to the Friendship Lane drainage area. Runoff from the South Adams subwatershed discharges through several culverts under the west end of Friendship Lane into an grass-lined channel (Site L50). This channel has sufficient capacity for conveying the 10-year flood flow, with about one foot of freeboard. The proximity of a house just east of the channel on Friendship Lane raises some concern with respect to flooding at higher flow conditions.

5.1.16 Highway - Apple Drainage

The area lies north of Highway Street and west of U. S. Highway 290. The Highway Street and Apple Street drainage areas are fairly long and end along very flat street sections near South Eagle and Pear Streets. Highway Street has a drainage area of about 75 acres, with a 10-year flood flow of 105 cfs. At high flows, some of this water spills out of the roadway and flows southward into the Friendship Lane drainage area either through the South Creek-Bluebonnet-Columbus Streets system or through a small drainageway that discharges into South Eagle Street. The primary areas with street flooding problems are along Apple Street (Site L52) and Highway Street (Site L51) from South Mesquite Street to South Eagle Street. There also is some potential for flooding of residential structures along Peach Street between Apple and Highway Streets. The floodwater spill-overs from Highway Street cause additional flooding problems along South Eagle Street at the low water crossing just south of Highway Street (Site L53). Runoff from the Apple Street drainage area discharges under U. S. Highway 290 through a box culvert into a grass-lined channel and through another set of culverts under Crenwelge Drive. The floodwater-carrying capacities of this channel and the associated culverts are well in excess of the 10-year flood flow.

5.1.17 Dry Creek Drainage

This area encompasses a well defined watershed with two major tributary channels. It is located northwest of the City near U. S. Highway 87 and Bob Moritz Drive. The main channel does not appear to have any significant flooding problems; however, there is a old bridge just downstream of U. S. Highway 87 (Site L54) that is causing significant channel erosion and may cause some backwater problems for the culverts under U. S. Highway 87. On the western tributary, there is an existing culvert under South Crenwelge Road near its intersection with Gold Road (Site L55). The 10-year flood flow causes overtopping of this road, which could result in flooding of adjacent businesses.

5.2 STREAM FLOODING

Areas of potential stream flooding have been analyzed by first identifying reaches where significant increases in flood levels are indicated based on comparisons of the simulated 100-year flood results from the revised HEC-2 models developed in this study with those previously determined during the original Flood Insurance Study (FIS) for the City of Fredericksburg. Floodplain widths and boundaries based on the HEC-2 modeling results have been examined for these reaches to determine if the indicated flood level rises translate into meaningful floodplain changes. In this process, the effective FIS 100-year floodplain boundaries have been plotted on base maps of the City of Fredericksburg. The revised floodplain boundaries based on the revised HEC-2 results also have been added to these maps to delineate areas of increased or decreased flooding.

5.2.1 Barons Creek

As discussed in Section 4.2, there are several reaches along Barons Creek where the 100-year flood profile plots (Figures 4-2 and 4-3) indicate significant increases in the flood levels from the updated HEC-2 model with respect to those previously determined in the original FIS. These areas of potentially increased flooding are discussed in the following paragraphs.

5.2.1.1 Wastewater Treatment Plant to Goehmann Road

Presented in Figure 5-1 is a map of this reach of Barons Creek with the 100-year floodplains delineated based on the effective FIS and based on the results from the revised HEC-2 model of this portion of the creek. For the revised floodplain, only those boundaries that are different from the effective FIS floodplain boundaries are plotted. Both sets of floodplain boundaries generally reflect flood flows corresponding to existing watershed and land use conditions. As illustrated, even with the higher flood

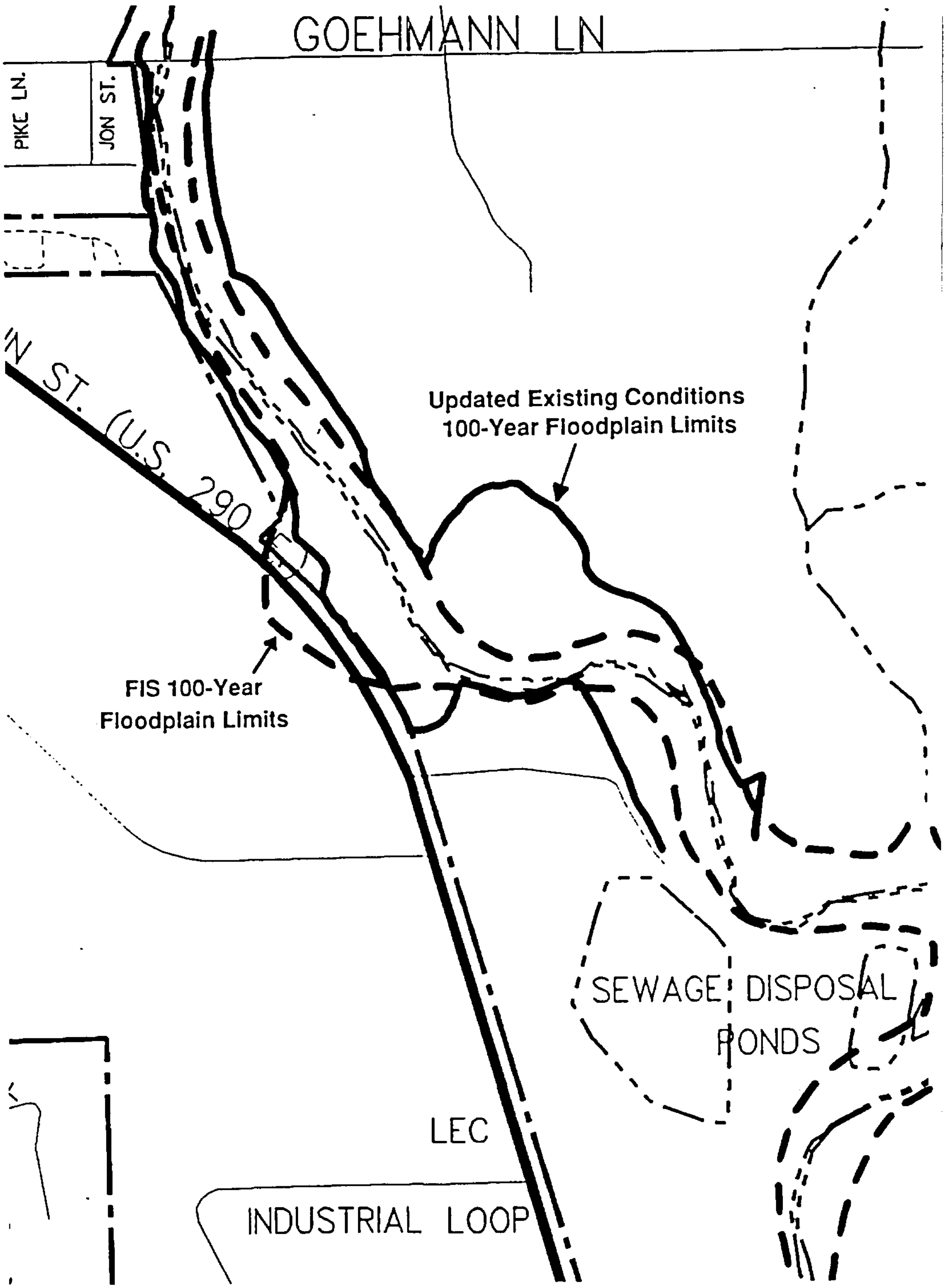

FIGURE 5-1 BARONS CREEK 100-YEAR FLOODPLAIN BOUNDARIES UPSTREAM OF CITY WASTEWATER TREATMENT PLANT

levels simulated with the revised HEC-2 model, the revised floodplain boundaries along the west bank are not significantly different from the FIS floodplain boundaries. It is interesting to note that the revised 100-year floodplain does not encompass the U. S. Highway 290 roadway, as does the effective FIS floodplain. There are some additional areas included in the revised floodplain along the left (east) bank of the creek that are not contained within the FIS floodplain. The inclusion of these areas results primarily from better definition of the overbank topography in the revised HEC-2 model and the availability of more detailed topographic information from the Corps of Engineers' Gillespie County flood insurance study for establishing the floodplain boundaries. Based on examination of 1994 aerial photographs of this reach of Barons Creek, none of these modifications in the floodplain boundaries appear to impact any structures along the creek.

5.2.1.2 Upstream of F. M. 1631

Based on an analysis of the actual locations of the 100-year flood level increases in this area as discussed previously, i. e., 1.4 feet of increase in the revised HEC-2 model results compared to those from the effective FIS, and examination of 1994 aerial photographs, it has been determined that there are no apparent flooding impacts on structures along this reach of the creek. In the vicinity of the one house that has been identified as being potentially impacted, the increase in the revised 100-year flood level is only about one-half foot, and this is not enough to cause any flooding of the structure.

5.2.1.3 Lincoln to Adams Reach

Results from the HEC-2 hydraulic modeling for the reach of Barons Creek from just downstream of Lincoln Street upstream to Adams Street indicate an increase in the 100-year flood level of about 0.8 feet from the effective FIS flood elevation to the levels simulated with the revised model under existing watershed and land use conditions. Despite these increased flood levels, the width of the floodplain changes very little from that depicted on the effective flood insurance maps. This is due primarily to the steep banks that characterize the channel and floodplain through this reach. There is one section about 700 feet upstream of Lincoln Street which does indicate an increase in the floodplain width of about 22 feet. Because of the proposed construction of a walk bridge across the creek at Llano Street, this section was resurveyed in 1996 as part of this study. The resurveyed section has been used to replace an existing section in the original FIS model; hence, it is not surprising that a change in the 100-year flood levels and floodplain boundaries in this vicinity has occurred. The changes in flood levels and topography through this reach are reflected in some small amounts of additional floodplain area on the west bank of Barons Creek.

Based on an analysis of 1994 aerial photographs of this reach of Barons Creek, the increased flood levels simulated with the revised HEC-2 model may have the effect of bringing one additional residential structure into the 100-year floodplain. This structure would join five other residential structures that presently are included within the effective FIS floodplain in this immediate vicinity. Without field surveying the actual ground elevations in the vicinity of these structures, however, it is not possible to determine with certainty whether or not they should be included in the revised floodplain. The simulated floodplain widths appear to be greater than those expected based on an analysis of the City's five-foot contour maps, and the elevations of the banks of the creek through this area based on the topographic maps appear to be higher than the revised 100-year flood levels. In essence, based on information shown on the City's five-foot contour maps, a rise in the 100-year flood levels on the order of 0.8 feet would appear to have no impact on the existing structures along this each of the creek. Field surveying of the finished-floor elevations of these structures would be necessary to confirm this observation.

5.2.1.4 South Bowie Street

The inclusion of a new surveyed section downstream of South Bowie Street in the revised HEC-2 model of Barons Creek has caused water levels to rise approximately 0.9 feet above those previously determined in the effective FIS. The reach in question extends over a distance of about 700 feet upstream along the creek from the new section, which is located approximately 650 feet downstream of the South Bowie Street bridge. The new section added to the revised HEC-2 model provides for a more accurate, but also a more constricted, definition of the channel in this area than was accounted for in the original FIS model.

The resulting increases in the revised 100-year flood levels from the revised HEC-2 model produce corresponding increases in the width of the effective FIS floodplain along this reach of Barons Creek on the order of 10 to 40 feet. Width increases of approximately 30 to 40 feet occur at the Bowie Street low water crossing, whereas, upstream of the road crossing, the floodplain increases are on the order of 10 feet. Based on examination of 1994 aerial photographs, no existing residential structures are impacted by these increased levels of flooding. All of the homes adjacent to the South Bowie Street low water crossing are a considerable distance from the revised 100-year floodplain boundaries. There are two structures presently within the effective FIS floodplain at the end of West Peach Street, and these structures could experience an additional 0.5 feet of floodwater. According to the City's five-foot contour maps, a flood level rise of 0.5 feet should have no impact on the houses at the end of West Peach Street because the revised 100-year flood level appears to be below the existing bank elevations.

5.2.2 Town Creek

Four areas previously have been identified from the 100-year flood water surface profile plots (Figures 4-4 and 4-5) as having significantly higher flood levels based on results from the revised HEC-2 model of Town Creek than those determined in the effective FIS. These areas include short reaches of the creek upstream of Elk Street, Crockett Street, Orange Street, and Edison-Schubert Streets.

5.2.2.1 Elk Street

The existing obstruction within the bridge at Elk Street, i. e., the old bridge structure, causes 100-year flood levels to increase as much as 2.8 feet above the effective FIS levels. However, it does not appear that even this amount flood level increase results in significant widening of the floodplain above Elk Street. Field surveying conducted during this study has provided more accurate channel and floodplain descriptions in the updated HEC-2 model. The map of the area upstream of Elk Street in Figure 5-2 shows only two minor reaches where the revised floodplain boundaries are slightly wider than those from the effective FIS. The greatest change in the floodplain boundary occurs at a driveway approximately 300 feet upstream of the Elk Street bridge, where the floodplain is widened by about 18 feet. This increase is not expected to impact the structure adjacent to the driveway.

5.2.2.2 Crockett Street

Results from the revised HEC-2 model of Town Creek indicate a flood level increase of

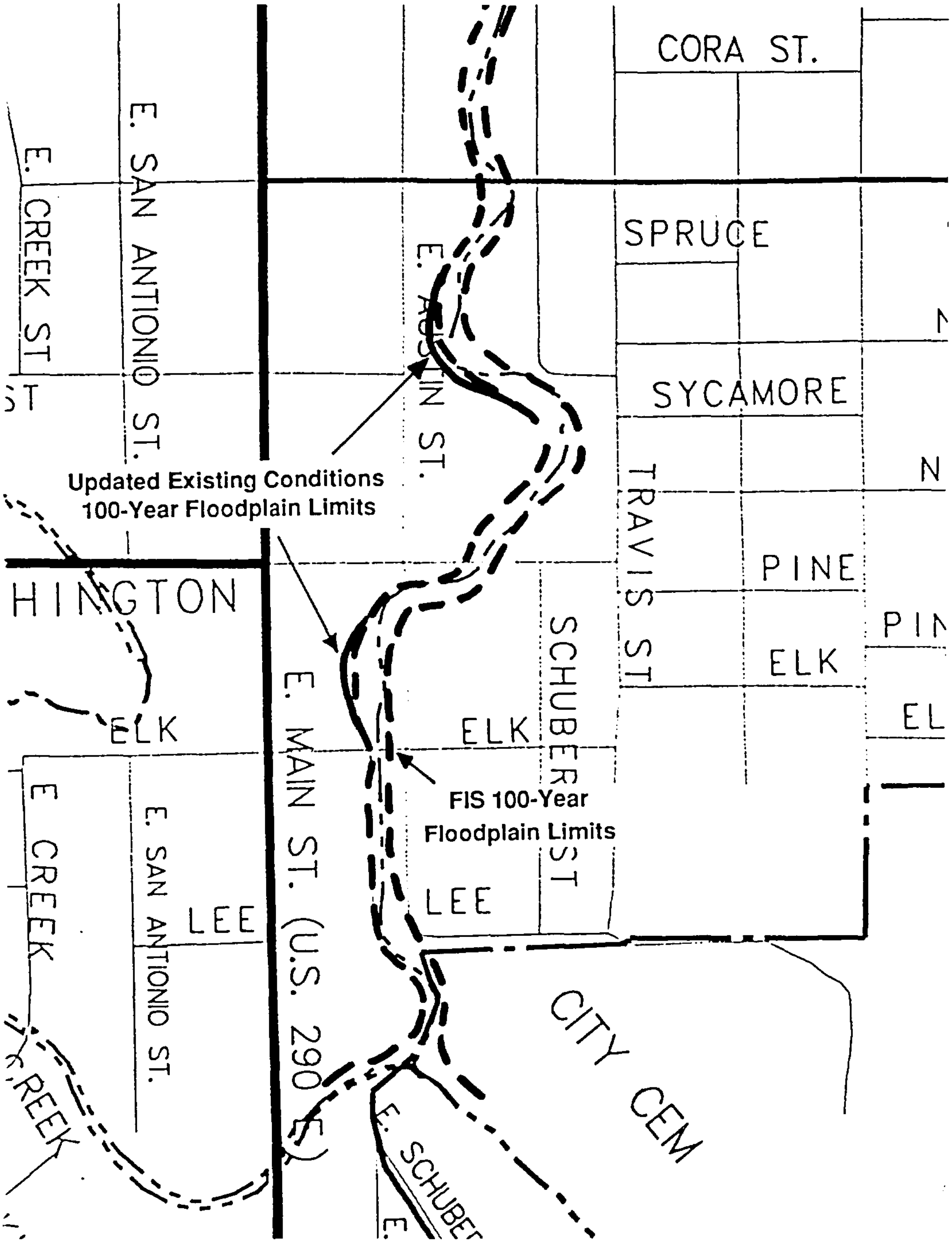

FIGURE 5-2 TOWN CREEK 100-YEAR FLOODPLAIN BOUNDARIES UPSTREAM OF ELK STREET

about 2.8 feet upstream of Crockett Street with respect to the effective FIS base flood elevations. Much of this increase can be attributed to channel modifications in the floodplain. This flood level increase translates to about an additional 175 feet of floodplain width. Figure 5-3 presents a map of the reach of the creek upstream of Crockett Street with the effective FIS floodplain boundaries delineated and the revised portions of the floodplain based on the revised HEC-2 model results also shown. As indicated, the major area of additional floodplain is located immediately upstream of Crockett Street. Based on an examination of 1994 aerial photographs, it appears that this additional flooding encompasses two residences along Crockett and Mistletoe Streets and two small commercial buildings along Crockett Street on the west bank. These structures are in addition to three residential structures at Crockett and Mistletoe Streets, one large commercial site at Crockett and Austin Streets, and three residences along Austin Street that already included in the effective FIS 100-year floodplain.

5.2.2.3 Orange Street

Orange Street is the next road crossing on Town Creek upstream of Crockett Street. There is one area immediately downstream of Orange Street where the revised 100-year flood levels exceed those from the effective FIS, and these flood level increases cause the width of the floodplain to be increased by about 12 feet beyond the effective FIS floodplain width. This increase in width does not impact any additional structures. There are, however, seven residential structures and one commercial building in the effective FIS 100-year floodplain of Town Creek between Orange Street and Milam Street, and there are an additional seven houses in the effective FIS floodplain between Milam and Edison Streets.

5.2.2.4 Edison-Schubert Streets

Flood flow hydraulics and flooding conditions along Town Creek within this overall area are quite complicated. The Town Creek channel makes a series of turns and bends as it crosses three streets with bridges over a linear distance of approximately 700 feet. Town Creek actually turns back on itself twice through this S-curve traverse before continuing downstream to cross Milam Street. Within this reach, there is an increase in the revised 100-year flood level on the order of 0.23 feet upstream of Edison Street. This increase does not significantly alter the floodplain such that the number of structures in the floodplain changes. There are two houses within the effective FIS

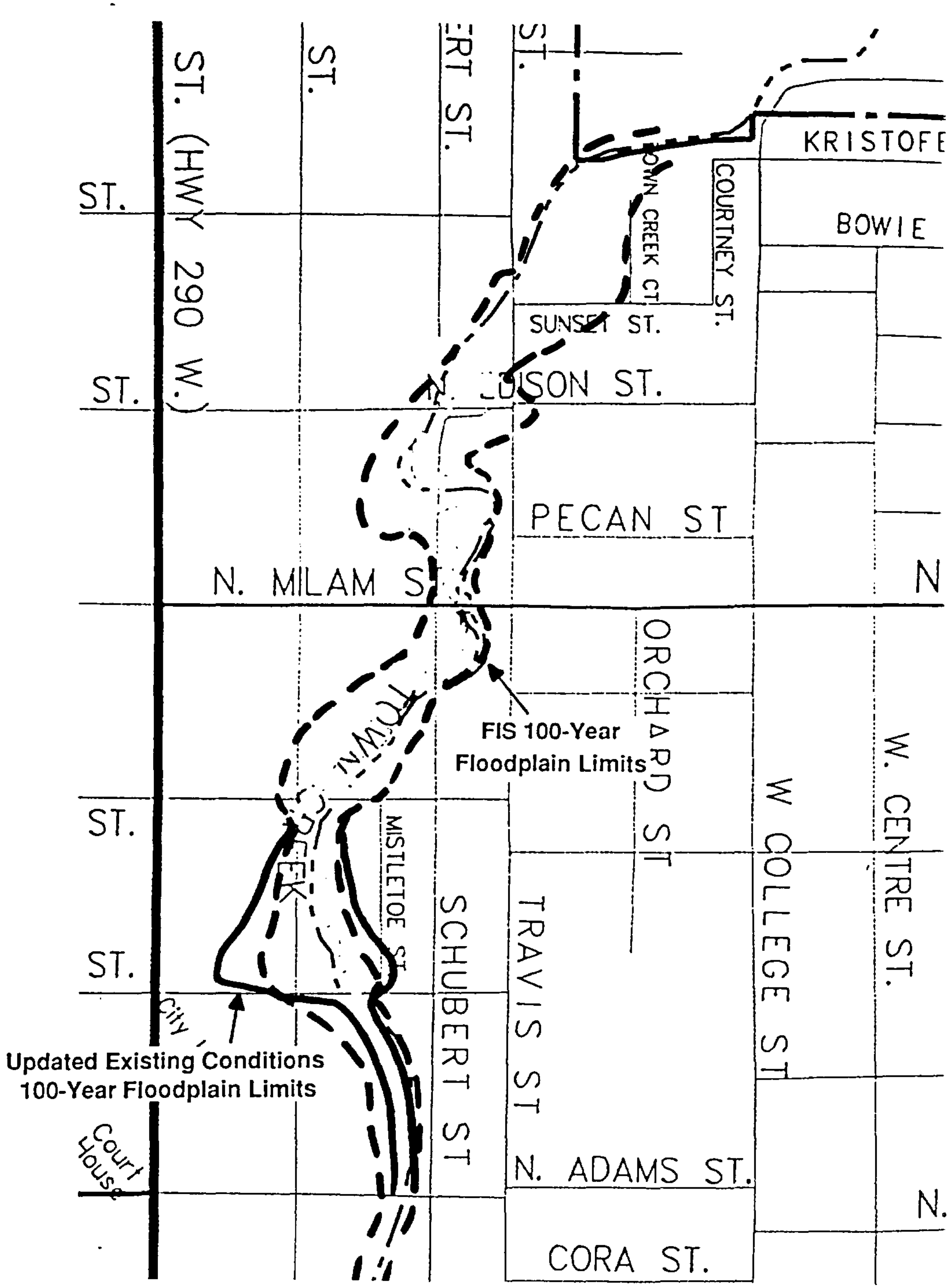

FIGURE 5-3 TOWN CREEK 100-YEAR FLOODPLAIN BOUNDARIES UPSTREAM OF CROCKETT STREET

floodplain between Edison Street and Travis Street, and one house at Sunset Street. For analyzing existing structures within the floodplain along this reach of the creek, the City's 1994 aerial photographs have been used.

5.2.3 Stream FB-1

As described previously, there are two principal reaches of Stream FB-1 where the revised 100-year flood levels from the revised HEC-2 model are significantly different from those determined in the effective FIS. The lower reach is midway between the mouth of the creek at is confluence with Barons Creek and the Llano Highway (Highway 16). Since the present day land use in this area is primarily agricultural, no residential structures are affected by the increased flood levels. These differences can be up to 7.5 feet, but are usually on the order of 3.5 feet. Comparisons of the floodplain widths simulated with revised HEC-2 model with those from the effective FIS do not indicate significant discrepancies, and the depths of flow also are similar. Hence, it appears that differences in the topography and channel geometry used in developing the models are the primary causes of the flood level deviations. With the revised model having been developed based on current and much more detailed topographic information, the revised model should be more accurate than the original FIS results.

The second reach of the Stream FB-1 where significant increases in flood levels are indicated with respect to the original FIS results is through the Carriage Hills subdivision between the Llano Highway and Lower Crabapple Road. Again, maximum increases in 100-year flood levels are on the order of 7.0 feet. Certainly, this would appear significant, but when the top widths of the respective floodplains are examined, the revised HEC-2 simulation actually results in a decrease in the 100-year floodplain. Based on 1994 aerial photography of this reach of the creek, the effective FIS 100-year floodplain encompasses 18 homes in the Carriage Hills subdivision. Twelve of these homes are upstream of Ridgewood Drive, and six are downstream. Based on the improved topography along Stream FB-1 and the revised model results, it appears that 13 of these homes actually are outside the 100-year floodplain. All five of the remaining homes are upstream of Ridgewood Drive. Figure 5-4 presents a map of this area and shows the differences between the effective FIS floodplain boundaries and the revised 100-year floodplain boundaries developed in this study.

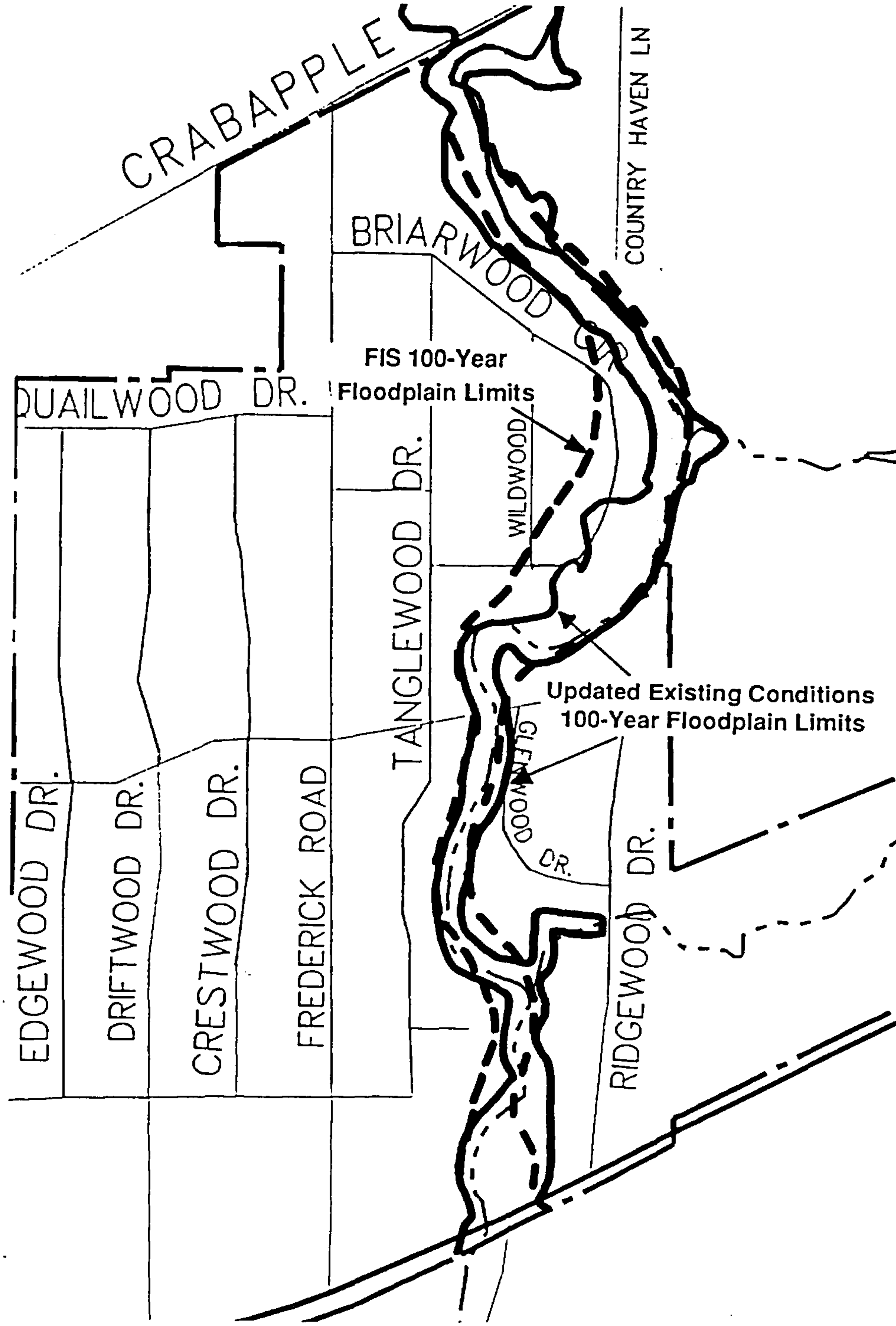

FIGURE 5-4 STREAM FB-1 100-YEAR FLOODPLAIN BOUNDARIES THROUGH CARRIAGE HILLS SUBDIVISION

5.3 ROADWAY FLOODING

Simulated flood levels from the revised HEC-2 models of Barons Creek, Town Creek and Stream FB-1 have been examined to assess overtopping conditions at major street and road crossings on these watercourses. These results are summarized in Table 5-3 for the 10-, 50- and 100-year flood events. The simulated flood levels immediately upstream of each of the crossings are listed. Also presented in the table are the minimum roadway elevations of the various streets and roads. Comparison of these roadway elevations with the different flood levels provides an indication of the extent and frequency of overtopping of the various streets and roads by floodwaters.

TABLE 5-3
LIST OF ROAD CROSSINGS AND ASSOCIATED FLOODWATER ELEVATIONS

SECTION LOCATION	HEC-2 SECTION NUMBER (U/S FACE)	MINIMUM ROADWAY ELEVATION feet msl	FLOODWATER ELEVATIONS 10-YEAR FLOOD feet msl	50-YEAR FLOOD feet msl	100-YEAR FLOOD feet msl
BARONS CREEK					
U.S. 290	9424	1600.00	1592.82	1594.66	1595.52
GOEHMANN LWC	3892	1611.50	1622.04	1624.92	1626.22
F.M. 1631	8000	1641.00	1634.68	1638.94	1641.03
MAIN ST.	11110	-	1646.96	1648.90	1649.70
CREEK ST. LWC	26316	1644.94	1652.27	1654.78	1656.05
WASHINGTON	29373	1669.31	1663.75	1666.12	1667.25
LINCOLN	30320	1671.97	1666.89	1669.26	1670.59
ADAMS ST.	31740	1681.25	1672.31	1674.50	1675.58
ORANGE ST. BRIDGE	34101	1681.19	1681.65	1683.61	1684.60
MILAM ST.	34812	1687.70	1683.63	1685.81	1686.80
BOWIE ST. LWC	36957	1681.76	1691.50	1693.62	1694.71
U.S. 290 W	41189	1718.80	1706.66	1708.66	1709.72
TOWN CREEK					
ELK ST.	1333	1662.89	1656.04	1657.58	1658.14
DRIVEWAY	1651	1651.31	1658.37	1660.16	1661.09
AUSTIN ST.	1957	1663.67	1659.80	1661.44	1662.27
WASHINGTON	2281	1668.40	1663.18	1665.69	1666.91
LLANO ST.	4028	1681.20	1674.80	1676.60	1677.53
ADAMS ST.	4720	1685.70	1679.97	1681.91	1682.94
CROCKETT ST.	5541	1691.43	1691.69	1692.36	1692.65
ORANGE ST.	6318	1695.20	1697.88	1698.47	1698.82
SCHUBERT ST. LWC	6886	1694.10	1698.86	1699.97	1700.57
MILAM ST.	7285	1702.40	1702.57	1702.85	1703.60
SCHUBERT ST.	7830	1700.80	1706.22	1708.14	1708.94
EDISON ST.	8297	1700.30	1708.68	1710.14	1710.68
TRAVIS ST.	8797	1705.80	1711.48	1712.66	1712.98
MORSE ST.	10895	1726.00	1728.19	1728.84	1729.16
STREAM FB-1					
LOW WATER CROSS.	171	1635.70	1638.42	1639.56	1641.07
LLANO HWY	11600	1707.50	1706.15	1709.26	1710.30
LOWER CRABAPPLE	17362	1755.00	1756.59	1756.85	1756.99

6.0 DRAINAGE IMPROVEMENT AND FLOOD PROTECTION ALTERNATIVES

6.1 LOCALIZED FLOODING

Areas identified as having significant localized flooding problems in Section 6.1 have been further evaluated to develop alternative measures to eliminate or to reduce the severity of the existing flooding conditions. Various alternatives that have been determined to be effective and that appear to be technically feasible are listed in Table 6-1, and they are identified by location on the map of the area in Plate 6-1.

The alternatives evaluation generally has been accomplished using techniques similar to those applied for the initial evaluation of the flooding problem areas. This includes performing hydraulic calculations for the proposed channel, storm drain and culvert improvements with estimates of localized runoff for different design storm events under fully-developed watershed conditions. For proposed channels, the "normal" depth of flow has been determined using Manning's uniform flow equation for specific levels of storm protection. Preliminary design slopes have been estimated using available information from field surveys and topographic maps as compiled during this study. Trial culvert sizes have been analyzed using standard culvert hydraulic procedures similar to those described in the Texas Highway Department's (now Texas Department of Transportation) Drainage Manual (1985).

In some cases, other hydraulic and hydrologic calculations have been performed, including additional HEC-1 runoff simulations, to provide additional information when necessary. Various hydrologic analyses have been undertaken to evaluate alternatives that modify runoff from or divert runoff away from problem drainage areas, thus reducing downstream flood flows. Also, alternatives that involve stormwater detention have necessitated the use of the HEC-1 runoff routing model to determine preliminary pond sizes and outlet configurations, as well as, to determine the general effectiveness of various ponds for reducing downstream flood flows.

A preliminary review of potential detention pond sites was made using available topographic maps and general knowledge regarding the location of existing flooding problems. Over 40 pond sites were reviewed with regard to their potential effectiveness for improving both localized and stream flooding problems. After initial screening, field reconnaissance surveys were made of the most promising detention pond sites and recent (1994) aerial photographs were reviewed. For pond sites that generally appeared to be technically feasible, inflow hydrographs were developed using the

TABLE 6-1

ALTERNATIVES FOR LOCALIZED DRAINAGE IMPROVEMENTS AND FLOOD CONTROL MEASURES

PROBLEM SITE DESIGNATION	PROBLEM SITE	LOCATION	DESCRIPTION	EFFECTIVENESS
A1a	L1	Friendship Lane Low Water Crossing	13 - 36" x 56" CGMP 450 feet D/S Channelization Roadway Weir Overflow Section	10-Year Capacity 100-Year - 1 feet over top of road
A1b	L1	Friendship Lane Low Water Crossing	4 - 36" x 56" CGMP Downstream Channelization Roadway Weir Overflow Section	10-Year Capacity with Regional Detention 100-Year < 0.5 feet over top of road
A1c	L1	Friendship Lane Low Water Crossing	7 - 36" x 56" CGMP Downstream Channelization Associated Roadway Work	100-Year Capacity with Regional Detention
A2	L1 - L5	Friendship Ln. Drainage West of Washington	Regional Detention Pond Area - 8.6 acres Max. Depth - 6' 100-Year Volume 26 acre-feet 18" Outlet Pipe	Reduces 100-Year Storm flow 96% at the site. Reduces 10-Year and 100-Year flood flows 20% at the South Creek Subdivision.
A3	L1 - L3	Friendship Ln. Drainage Just U/S of South Creek Subdivision	Regional Detention Pond Area - 9.3 acres Max. Depth - 7.6' 100-Year Volume - 48 acre-feet 24" Outlet Pipe 800 feet U/S Channel	Combined with A2 Pond, Reduces 100-Year flow at South Creek Subdivision to 42 cfs and reduces the peak flow at the Friendship Lane low water crossing by 72%.
A4	L2	South Creek Subdivision	460 feet- 54" RCP 1025 feet D/S Channelization	Carries 33% of 10-Year flood runoff.

TABLE 6-1

ALTERNATIVES FOR LOCALIZED DRAINAGE IMPROVEMENTS AND FLOOD CONTROL MEASURES

PROBLEM SITE DESIGNATION	PROBLEM SITE	LOCATION	DESCRIPTION	EFFECTIVENESS
A5	L3	Friendship Lane from Low Water Crossing to Washington	2,800' Grass Lined Trapezoidal Channel 35' Top Width (North Side) 2,800' Grass Lined Trapezoidal Channel 18' Top Width (South Side) Replace Creek St. culverts with 3 - 2.5' x 8' box culverts Replace driveways with box culverts	10-Year Capacity
A6a	L4	Washington @ Friendship	Add 2 - 4' x 4' box culverts	10-Year Capacity
A6b	L4	Washington @ Friendship	Add 1 - 4' x 4' box culverts	10-Year Capacity with U/S Detention
A7	L5	Friendship Lane U/S of Washington	1,100' Grass Lined Trapezoidal Channel 30' Top Width on North Side	10-Year Capacity
A8	L6	Highway St. & S. Adams	450 feet - 48" RCP 500 feet of D/S Channelization	10-Year Capacity
A9	L7	Schubert St.	1,100 feet - 42" RCP 11 inlets & 800 feet stormsewer	100-Year Capacity
A10a	L7	Schubert St.	Purchase 2 - 0.5 acre vacant lots Regrading	25-Year Protection for Houses Reduces Street Flooding
A10b	L7	Schubert St.	Purchase 2 - 0.5 acre vacant lots Excavation - 3.6 acre-feet pond 1,100 feet - 24" RCP	Eliminates House/Street Flooding for 100-Year Storm
A11	L8	N. Milam St.	1,900feet - 48" RCP 10 inlets	10-Year Capacity Eliminates House Flooding for 100-Year Storm

TABLE 6-1

ALTERNATIVES FOR LOCALIZED DRAINAGE IMPROVEMENTS AND FLOOD CONTROL MEASURES

PROBLEM SITE DESIGNATION	PROBLEM SITE	LOCATION	DESCRIPTION	EFFECTIVENESS
A12	L8 - L10	N. Milam St.	600 feet - 48" RCP 1,900 feet - 60" RCP 20 inlets 500 feet of curb	10-Year Capacity Eliminates House Flooding on Milam Eliminates House Flooding on W. Centre & W. College with additional upstream improvements (A13)
A13	L8 - L12	W. Burbank	1,050 feet - 48" RCP 11 inlets	10-Year Capacity Eliminates House Flooding on Burbank Eliminates House Flooding D/S with Alternative A12
A14	L15, L16, L18 - L24	E. Burbank	2,200 feet - 48" RCP 9 inlets Minor Channelization Drainage Easement Acquisition	10-Year Capacity Eliminates Structure Flooding near Llano & W. Burbank Reduces downstream problems
A15	L16	N. Lincoln	250 feet of Berm	10-Year Capacity within Street Eliminates House Flooding Potential with Alternative A14
A16	L17 - L24	N. Llano	1,300 feet - 60" RCP (Llano) 1,000 feet other Stormsewer 20 inlets	10-Year Capacity Reduces D/S flows 50 - 60% Eliminates House Flooding except near Travis for 50-Year to 100-Year events. Reduces D/S Erosion
A17	L16 - L22	College & Travis	2,500 feet - 72" RCP 500 feet - 60" RCP 1,000 feet Other Stormsewer 30 inlets D/S Energy Dissipation & Erosion Control	10-Year Capacity Eliminates House Flooding except for events near 100-Year floods.

TABLE 6-1

ALTERNATIVES FOR LOCALIZED DRAINAGE IMPROVEMENTS AND FLOOD CONTROL MEASURES

PROBLEM SITE DESIGNATION	PROBLEM SITE	LOCATION	DESCRIPTION	EFFECTIVENESS
A18	L23, L24	Travis	400 feet of Erosion Control 200 feet Minor Grading & Channelization	Reduces existing erosion problem
A19	L25, L26	Trailmoor & Llano	800 feet - 36" RCP 300 feet other Stormsewer 15 inlets	10-Year Capacity Reduces Llano overtopping
A20	L25, L26	Morning Glory & Broadmoor	2,000 feet - 24" & 36" RCP 12 inlets	10-Year Capacity Provides 100-Year Capacity @ Trailmoor and Llano with Alternative A19
A21	L13, L14 L25 - L27	North of Morning Glory	Regional Detention Pond Area - 6 acres Max. Depth - 8.5 feet 100-Year Volume - 37 acre-feet 3' x 5' Box Culvert Outlet 1,100 feet U/S Channelization 2 - 36" x 58" CGMP	Eliminates 100-Year overtopping of Llano Offsets additional discharge from A20 Reduces street flooding and spillovers on N. Milam
A22	L28 - L31	West of Edgewood	Regional Detention Pond Area - 5 acres Max. Depth - 5' 100-Year Volume 15 acre-feet 18" RCP outlet	Reduces 100-Yr flow at the discharge point by 94% Eliminates problems on Driftwood north of Ridgewood 10-Year protection downstream Reduces downstream street flooding
A23	L30, L31	Driftwood & Adams	600 feet - 48" RCP 400 feet - 54" RCP 20 inlets 700 feet Grass Lined Channel Top Width - 35 feet	5-Year Capacity with Detention Approximately 100-Yr protection from house flooding Reduces street flooding

TABLE 6-1

ALTERNATIVES FOR LOCALIZED DRAINAGE IMPROVEMENTS AND FLOOD CONTROL MEASURES

PROBLEM SITE DESIGNATION	PROBLEM SITE	LOCATION	DESCRIPTION	EFFECTIVENESS
A24	L34	South Ridge Subdivision	Regional Detention Pond Area - 11 acres Max. Depth - 13 feet 100-Year Volume 19 acre-feet 3.5' x 4' Box Culvert Outlet	Reduces 100-Yr flow at the discharge point by 66% Reduces 10-Year flood to pass through existing culverts Reduces overflows and house flooding potential
A25	L34	Ridgewood	2 - 48" RCP U/S & D/S Channel Grading	10-Year Capacity 50-Year Capacity with 1 feet of overtopping 100-Year Capacity with upstream detention (A24)
A26	L35	South Bowie	600 feet - 36" RCP 7 inlets	10-Year Capacity
A27	L35	South Edison	750 feet - 30" RCP 4 inlets	10-Year Capacity
A28	L36	Armory Road	4 - 36" x 58" Arch CGMP 400 feet of D/S Channel	10-Year Capacity
A29	L37	Basse Lane	4 - 36" x 58" Arch CGMP 250 feet of D/S Channel	10-Year Capacity
A30	L38, L39	Basse Lane	4 - 36" x 58" Arch CGMP 850 feey Grass Lined Channel Top Width - 30 feet	10-Year Capacity
A31	L40	South Bowie	3 - 36" x 58" Arch CGMP	10-Year Capacity
A32	L44 - L49	Park Street	1,150 feet - 42" RCP 300 feet - 36" RCP 200 feet - 18" RCP 14 inlets	10-Year Capacity Approximately 100-Year protection for buildings

TABLE 6-1

ALTERNATIVES FOR LOCALIZED DRAINAGE IMPROVEMENTS AND FLOOD CONTROL MEASURES

PROBLEM SITE DESIGNATION	PROBLEM SITE	LOCATION	DESCRIPTION	EFFECTIVENESS
A33	L46	Ufer Street	600 feet - 24" RCP 4 inlets	5-Year Stormsewer Capacity 10-Year Capacity at low point of street
A34	L51, L53	Highway Street & South Creek Street	1,400 feet - 36" RCP 1,300 feet - 30" RCP 9 inlets	10-Year Capacity Reduces flooding duration near Highway St. & Eagle Reduces spillover to Friendship Lane Drainage
A35	L51, L53	Highway Street South Eagle	1,800 feet Grass Lined Trap. Channel Top Width Approx. 30 feet 3 - 36" x 58" Arch CGMP	Eliminates street flooding along Highway Street (L52) Provides 10-Year Capacity at Eagle Street
A36	L52	Apple Street	1,150 feet - 36" RCP 6 inlets	10-Year Capacity Reduces house flooding potential
A37	L54	U. S. Highway 87	Remove old road bridge Revegetation	Reduces Erosion Eliminates backwater from structure
A38	L55	Crenwelge Road	Add box to culvert Approximately 300 feet of channel improvements	10-Year Capacity Reduces structure flooding potential

HEC-1 model, and preliminary pond grading plans and outlet designs were established based on spreadsheet hydrologic analyses of the hydrographs. Additional HEC-1 simulations then were performed to evaluate the effectiveness of the selected ponds for reducing downstream flood flows and to refine and revise the outlet designs and pond configurations.

The level of flood protection considered in developing alternative drainage improvements and flood control measures has varied depending on the severity and nature of the flooding problems examined. Problems involving combinations of the flooding of residential structures and significant street flooding have been considered to be the most significant, and where it has appeared to be feasible, alternatives providing flood protection for the 25-year and/or 100-year storm event have been evaluated. Overtopping of major streets and roadways by floodwaters also has been considered to be a serious problem because of the danger to motorists and pedestrians and the potential for loss of life. Protection from overtopping has been evaluated for the major streets and roadways considering the 25-year and 100-year storm events, with 10-year capacity without overtopping considered to be the minimum design standard.

Solution alternatives for problem areas with some street flooding and some potential for flooding of residential structures have been evaluated considering primarily the 10-year storm event, since conveyance of at least the 10-year flood flow would significantly reduce flooding risks. Furthermore, stormwater control facilities that are designed for the 10-year storm event in the Fredericksburg area also will provide sufficient conveyance to handle about 55 percent of the 100-year flood flows. Because of the expense and difficulty of implementing the higher levels of protection and because of the greater benefits of providing protection for more area for the more frequent storms, the 10-year storm event, under fully-developed watershed conditions, has been adopted and used as the primary design standard for most of the solution alternatives evaluated.

Although the conversion of land in the Fredericksburg area from a natural, undeveloped state to a fully-developed condition theoretically can result in a 40- to 50-percent increase in the 10-year flood flow for moderate intensity development, many of the existing localized flooding problem areas are within watersheds that already are approaching full development intensity. Hence, under these circumstances, on-site detention of stormwater runoff is not considered to provide an effective means for reducing floodwater discharges, except for very localized drainage situations immediately downstream of development projects. For this reason, on-site stormwater detention has not been specifically considered as a solution alternative for each individual flooding problem site.

However, in some areas with significant projected growth involving intensely-developed land uses, such as commercial, office and industrial projects, a much higher increase in peak flood flows can be expected under fully-developed conditions. In these watersheds with higher-intensity development, on-site stormwater detention obviously is a more significant alternative that should be given strong consideration. Conversely, watershed areas with low intensity development, such as parks or low-density residential subdivisions, will have much less of an increase in peak flow flows between existing and fully developed conditions, and stormwater detention may not be required.

It should be noted that the facility sizes and capacities developed in this Flood Protection Planning Study as part of the solutions for existing flooding problems are considered to be preliminary and will need to be verified and refined through detailed, site-specific design studies. The facility designs described herein are approximate and conceptual, but are considered to fully adequate for planning purposes. Detailed surveys and additional, more detailed hydraulic analyses will required for final facility designs. Some additional hydrologic analyses also may be desirable to develop more cost-effective final designs. It should be noted that the fully-developed flows used for these analyses are only estimates based on projected land use and may vary significantly depending on the level of ultimate development and the types of stormwater conveyance and control facilities that ultimately are constructed.

Specific drainage improvements and flood control measures, to the extent they are required, are discussed in the following sections for each of the previously identified flooding problem areas.

6.1.1 Friendship Lane Drainage

One of the major flooding problems regarding this area is that the Friendship Lane readily becomes impassable at the low water crossing during the occurrence of even small storm events. The peak flood flow for the 10-year storm at the low water crossing is 578 cfs, assuming fully-developed watershed conditions upstream, while the corresponding 100-year flood flow at this location is 1,096 cfs. Although a higher level of protection may be justified for this location due to traffic volumes, site limitations with regard to existing ground and roadway elevations necessitate using no more than the 10-year storm event as the standard for developing a practical and feasible culvert design.

To convey the 10-year flood flow beneath the road will require some channelization work downstream for approximately 450 feet in order to lower the flowline enough to place drain pipes under the roadway. For conveying the 10-year flood flow without overtopping of the roadway, thirteen 36" by 58" corrugated metal (GCMP) arch pipes are required. Using this size pipe still will require the roadway to be raised about 1.5 feet at the low point, which will involve road work over a distance of nearly 400 feet. The roadway surface could be designed so as to serve as an overflow weir for passing flood flows produced by storms greater than the 10-year flood. A flat section of concrete-capped roadway 200 feet long, in conjunction with the thirteen 36" x 58" pipes, would be capable of passing the 100-year flood flow with a maximum depth over the roadway of about one foot. Additional detailed hydraulic analyses will need to be performed to ensure that this type of culvert facility will not raise the water surface along the upstream channel. Alternatively, appropriate easements can be acquired to accommodate the effects of any increases in upstream flood levels. Downstream easements also will be required to allow the necessary channelization work. Significant flood flows and ponding of stormwater runoff already occurs along the watercourse; hence, there should be some incentive for adjacent land owners to assist with implementation of the proposed culvert project. As a minimum, construction of the proposed culvert could be coordinated with drainage work required by future development projects.

The number of pipes required for conveying the 10-year flood flow could be reduced to as few as four 36" by 58" CGMP arch pipes provided that the two regional stormwater detention ponds described below are constructed upstream within the Friendship Lane drainage area. With the regional stormwater detention and the four pipes described above, the 100-year flood flow would overtop the roadway less than 0.5 feet. Alternatively, conveyance of the entire 100-year flood flow under the roadway could be accomplished with seven pipes of this same size if the upstream stormwater detention is implemented.

Because of the many significant flooding problems and their associated site constraints within the Friendship Lane drainage area, it would appear that one feasible alternative is to provide regional stormwater detention facilities at one or more sites within the watershed. Since there are major problems throughout this watershed, the prime detention sites necessarily must be located farther upstream in the watershed. For this purpose, two detention sites have been evaluated in detail. One site (A2) is located west of South Washington Street (U. S. Highway 87) and along and just east of the channel running southeastward from South Adams Street, and the other site (A3) is located just west and upstream of the South Creek subdivision. Except for the street flooding at the intersection of Highway Street and South Adams Street (Site L6), which is upstream of these pond sites, detention ponds at these locations potentially would be effective in significantly reducing or eliminating all the identified flooding problems within the Friendship Lane drainage area.

Based on preliminary hydrologic analyses, the A2 detention pond site appears to be effective for improving flooding conditions because of its location near the headwaters of the Friendship Lane drainage and because it is upstream of the most significant problem sites. A pond at this site could be designed to detain nearly all of the 100-year flood flow from the upstream watershed and then to slowly release this water after passage of the storm when downstream flooding has subsided. The effectiveness of the pond also can be improved by routing additional stormwater into the pond from the end of Sunco Avenue. For full retention of the 100-year flood, the pond facility would cover approximately 8.6 acres with a maximum depth of six feet and a required total volume of approximately 26 acre-feet. The required outlet is an 18-inch reinforced concrete pipe. This pond configuration would reduce the 100-year flood peak flow from 382 cfs to 17 cfs, a 96-percent reduction in the flow rate. This large flow reduction is necessary in order to effectively reduce downstream flood flows at the individual flooding problem sites since there still is a significant downstream contribution of stormwater runoff that is not being detained. This pond would reduce the 100-year flood peak flow at the South Creek subdivision from approximately 842 cfs to 675 cfs, a 20 percent decrease in flow. For the 10-year storm, the flood flow at the South Creek subdivision would be reduced from 464 cfs to 375 cfs. More significant flow reductions would be achieved at South Washington Street and immediately upstream since runoff from most of the upper drainage area would be detained and controlled. This pond would eliminate the need for additional channel work upstream (west) of South

Washington Street, and it would significantly reduce the size of drainage improvements needed at and downstream of South Washington along Friendship Lane.

The A3 pond site also is a very effective detention site since it is located just upstream of the major flooding problem area (L2) in the South Creek subdivision. In combination with the A2 detention pond, the A3 pond also could be designed to detain nearly all the 100-year flood flows from the upstream watershed and then to slowly release these flows at or below the minimum conveyance capacity of the downstream channels. To provide for full 100-year flood flow retention (along with the A2 pond), the A3 pond facility would cover approximately 9.3 acres with a maximum depth of 7.6 feet and a required total volume of approximately 48 acre-feet. The required outlet is a 24-inch reinforced concrete pipe. This would reduce the upstream peak 100-year flood flow from 675 cfs to 42 cfs, a 94-percent reduction. The combined detention effects of the two ponds would be sufficient to allow the 100-year flood flows to safely pass through the South Creek subdivision, and they would reduce the 100-year flood flows at the Friendship Lane low water crossing by 72 percent. The two ponds would eliminate the need for additional drainage improvements through the South Creek subdivision and, as noted above, would significantly reduce the number of culverts required at the Friendship Lane low water crossing crossing.

Without the upstream detention ponds, some form of improved floodwater conveyance through the South Creek subdivision area is needed. Alternative A4 involves installation of a storm drain through the subdivision. A 54" reinforced concrete pipe would carry approximately 150 cfs, which is about one-third of the total stormwater flow of the Friendship Lane drainage. To install the pipe, channelization would be required downstream of Creek Street all the way to the existing low water crossing on Friendship Lane. Also, an inlet sump would be needed just west (upstream) of the South Creek subdivision. Although these facilities, by themselves, would not eliminate flooding within the South Creek subdivision, a 10-year flood protection level (or more) could be achieved in combination with other alternatives, including some upstream detention and drainage improvements along Friendship Lane.

The limited floodwater-carrying capacity of the swale along Friendship Lane causes flooding of adjacent properties and forces much of the stormwater from upstream to spill northward and flow through the South Creek subdivision. Several methods for improving conveyance have been considered to keep the stormwater flows off the roadway and in the road right-of-way. A 60" reinforced concrete pipe installed along Friendship Lane would be capable of carrying about two-thirds of the 10-year flood flow produced at Friendship Lane and South Washington Street, and it would provide less than half of the total discharge capacity needed to convey floodwaters beyond the South Creek subdivision. Using concrete-lined channels along Friendship Lane would require one channel 18-feet wide (top width) on the north side of the roadway and one channel 12-feet wide on the south side of the roadway. Although these channels would carry the 10-year flood flow, they would require replacement of all the driveways and the Creek Street culverts with small bridges in order to prevent any obstruction of the stormwater flows in the channels. Velocities in the channels would be on the order of 11 feet per second. A more practical alternative (A5) involves the construction of grass-lined trapezoidal channels along the current alignments of the existing swales adjacent to the roadway. For conveying the 10-year flood flow, a trapezoidal channel with a top width of 35 feet would be required on the north side of the roadway and a channel with a top width of 18 feet would be required on the south side of the roadway. This channel work would require replacement of the the north side driveways and South Creek Street culverts with three 2.5' (high) by 8.0' (wide) box culverts and replacement of the south side driveway culverts with one 2.5' (high) by 8.0' (wide) box culvert. This channel configuration in combination with Alternative A4 (54" storm drain through the South Creek subdivision) would provide 10-year flood protection along much of Friendship Land and through the South Creek subdivision. Of course, these drainage improvements would not prevent flooding of the roadway and residential structures by flood flows produced by larger storm events, i. e., greater than the 10-year flood.

The existing box culvert at South Creek Street and Friendship Lane is undersized for the 10-year flood event. Two additional 4' by 4' box culverts are needed at this location to convey the 10-year flood flow. If the upstream detention project is implemented as described for Alternative A2, only one additional culvert would be required.

Upstream of South Washington Street, a grass-lined trapezoidal channel with a top width of 30 feet is needed along the north side of Friendship Lane to safely convey floodwaters downstream. This channel is not needed if the upstream detention pond (Alternative A2) is constructed.

Since the Friendship Lane watershed is partially undeveloped, another alternative to consider is to require on-site detention for new developments. Although on-site detention would not be as effective as the regional stormwater detention alternatives, it would significantly reduce the sizes of other required improvements. The 10-year flood flow at the South Creek subdivision based on existing watershed conditions is 250 cfs, whereas the corresponding flow under fully-developed watershed conditions is 464 cfs. This represents an 85-percent increase in peak flow rate. A 55-percent increase in peak flow rate is projected for the 10-year flood flow at the Friendship Lane low water crossing. These are the highest projected increases in flood flows for any watershed within the Fredericksburg Flood Protection Planning area, and they are attributable to the significant increases expected in intense land uses, including commercial, industrial, heavy commercial and medium-density residential. On-site detention is not specifically described as an alternative for this drainage area; however, on-site detention would be effective for partially mitigating the projected increases in the peak flood flows associated with the conversion from undeveloped to fully-developed watershed conditions. For the Friendship Lane drainage, on-site detention is considered a secondary alternative to regional detention.

At Highway and Adams Streets, 450 feet of 48-inch reinforced concrete pipe is needed to provide conveyance capacity for the 10-year flood flow. Installation of this storm drain will require downstream channelization work for about 500 feet.

6.1.2 Schubert Street Ponding

Because this is a closed drainage basin (one with no natural outlet) and since there is major street flooding and likely some flooding of residential structures during the larger storm events, this area should be considered for 100-year flood protection. Approximately 1,100 feet of 42-inch reinforced concrete pipe would be needed to provide this level of protection and to allow building on the currently-vacant lots in the depression area. This alternative (A9) would also require approximately 11 inlets and 800 feet of storm drains to collect the 100-year flood runoff. Certainly, this level of project would represent a major undertaking with regard to costs.

Converting the vacant lots into a City-owned and operated detention pond and grading the area to provide additional detention storage capacity is a more reasonable and cost-effective approach for resolving the existing drainage and flooding problems than installing additional storm drains and inlets. With minor regrading of the vacant lots and continuing to use the existing 18-inch storm drain as the outlet, it appears that 25-year flood protection could be provided to the adjacent homes. Protection for the 100-year storm would require excavation of these lots and installation of a 24-inch storm drain at a steeper grade and lower upstream flowline. The purchase of the lots and minor regrading could serve as an interim solution until the other more extensive improvements could be made.

6.1.3 Cross Mountain - Milam Drainage

Along the downstream portion of North Milam, from Town Creek upstream to Morse Street, a 48-inch storm drain and approximately 10 inlets are needed to provide capacity for conveying the 10-year flood flow. With the present overflow capacity of the street, this alternative (A11) would provide nearly 100-year flood protection to the adjacent houses along North Milam Street. A variation of this alternative, Alternative A12, involves oversizing this pipe to 60 inches to allow conveyance of runoff from the Pecan Street and Edison Street areas. For this alternative, an additional 600 feet of 48-inch storm drain would be required along West Centre Street from North Milam Street to Edison Street, as well as, 10 additional inlets. About 500 feet of curb also would be needed to reduce the potential for flooding of residential structures along West Centre and West College Streets. To achieve 100-year flood protection for houses in the vicinity of the Centre-Edison Streets intersection and the College-Pecan Streets intersection, however, additional upstream drainage improvements along Burbank Street would be necessary.

Stormwater runoff that creates a flooding problem (Site L11) near Burbank Street and Avenue A also contributes to the downstream flooding problems along West Centre and West College Streets near Pecan Street and Edison Street. Because of the flow limitations created by the existing curb-cut on Burbank Street and by the capacity of the grass swale downstream of Burbank, some stormwater is diverted westward down Burbank Street to the existing flooding problem site at Avenue D (Site L12). It is not recommended that the curb-cut be enlarged or that improvements be made to the existing grass swale because these modifications could increase the contribution of runoff to the downstream problem sites (Sites L8, L9 and L10). One possible solution would be to install a storm drain northward from Town Creek near Pecan Street up through the natural flow path to Burbank Street near Avenue A. However, this would require about 3,800 feet of 48- and 54-inch pipes. A more practical alternative (A13) is to install a 48-inch storm drain westward down Burbank Street from Avenue A to Town

Creek west of Avenue D. This would provide conveyance for the 10-year flood flow and virtually would eliminate the flooding problems along Burbank Street. The existing curb-cut and grass swale would have to be maintained for conveyance of flood flows from larger storms, as would the channel at the western end of Burbank Street at Avenue D. This alternative would also sufficiently reduce the downstream flood flow contributions from the Burbank Street area such that Alternative A12 would provide 100-year flood protection for houses in the vicinity of the Centre-Edison Streets intersection and the College-Pecan Streets intersection.

Although there is some spill-over of stormwater from the upper end of North Milam Street to the east, the relatively minor nature of the associated flooding problems (Sites L13 & L14) do not appear to warrant the additional 3,000 feet of storm drain that would be required for mitigation. It should be noted that the storm drain described in Alternative A11 is not sized for any future extension up North Milam Street past Burbank Street.

6.1.4 Burbank - Llano Drainage

The only feasible alternative (A14) to correct flooding problems in this area (Site L15) is to install a storm drain eastward along Burbank Street from North Adams Street to just east of North Washington Street. This would require about 2,220 feet of 48-inch reinforced concrete pipe, along with nine inlets. Some minor channel work also would be necessary at the outfall, and drainage easements would need to be obtained down to Stream FB-1. This alternative would also provide significant downstream benefits, especially along North Lincoln Street.

6.1.5 North Lincoln Drainage

The most attractive alternative for alleviating this flooding problem (Site L16) is the alternative described above for the Burbank-Llano area. That alternative would reduce flood flows in North Lincoln Street by about 35 percent. Additional storm drain improvements for this area do not appear to be justified. However, to contain the runoff from larger storms within the street section, a berm could be constructed along the east side of North Lincoln Street from East Centre Street to East College Street and a short distance eastward along Centre Street from North Lincoln. These improvements should only be installed, however, if the upstream storm drain project along Burbank

Street (A14) is implemented, since without the upstream improvements, the berm would increase street flooding and possibly pose a flooding threat to the houses on the west side of North Lincoln Street.

6.1.6 College - Llano Drainage

To collect flood flows near the College-Llano intersection and convey them southward to Town Creek (along and beneath Llano Street) would require installation of a 60-inch reinforced concrete pipe (A16). Because this alignment crosses the drainage divide between the College-Llano drainage and Town Creek, the depth of the 60" storm drain would reach a maximum of about 23 feet just south of Orchard Street. However, the very hazardous flooding conditions at the College-Llano intersection and the significant flooding problems downstream justify the relatively large pipe size and extensive depth of cut. This alternative would provide conveyance capacity for the 10-year flood flows at the College-Llano intersection, eliminate structure flooding in this area, and reduce street and house flooding downstream. With this alternative, the street flooding associated with the 10-year storm would be reduced to the level that normally occurs every two years or so under existing drainage conditions. This alternative would also require approximately 1,000 feet of other storm drains and 20 inlets in order to collect the upstream stormwater runoff.

6.1.7 College - Travis Drainage

An optional alternative to running the 60-inch storm drain down North Llano Street from College Street (A16) is placing a storm drain along the entire existing flow path from the College-Llano intersection to just west of the City Cemetery. This alternative (A17) appears to be cost prohibitive since it requires 3,000 feet of 60-inch and 72-inch pipes along with 30 inlets and 1,400 feet of additional collector storm drains. A significant negative impact of this alternative is the increased erosion that might result downstream of the storm drain outfall near the City Cemetery, where erosion problems already exist. However, this alternative would provide 10-year flood-flow capacity throughout the College-Travis drainage and would almost eliminate the potential for flooding of residential houses in this area.

The existing erosion problems (Sites L23 & L24) near the downstream end of this subwatershed require some remediation work in order to prevent a worsening of the problem and possible undermining of roadways or drainage structures. Some minor grading, channelization and re-vegetation or armoring is needed (A18).

6.1.8 Trailmoor Drainage

The significant ponding of stormwater on Trailmoor Drive and the associated overtopping of North Llano Street by flood flows for the two-year storm event could be eliminated for storms up to the 10-year event by removing the single drop inlet to the culverts under North Llano and installing a stormwater collection system along Trailmoor up to the intersection with North Adams Street (A19). This would require approximately 800 feet of 36-inch storm drain with 300 feet of smaller pipes and 15 inlets.

Additional upstream improvements are required to eliminate overtopping of North Llano Street and ponding of stormwater on Trailmoor Drive for storms greater than the 10-year event. These improvements would involve installing storm drains along Broadmoor Drive and Morning Glory Drive to the small tributary of Stream FB-1 that passes under North Llano Street just west of Lower Crabapple Road. This alternative (A20) would require approximately 2,000 feet of 24- and 36-inch pipes and 12 inlets. An added benefit of this alternative would be reduced street flows and depths along the entire length of Trailmoor Drive. However, it would also discharge additional stormwater to the Morning Glory - Llano drainage.

On-site detention would have a moderate benefit in this drainage area, particularly at the upper end of the watershed.

6.1.9 Morning Glory - Llano Drainage

A good regional detention pond site is located within this drainage area just north of Morning Glory Road. Although there are no major flooding problems within this drainage area, the regional detention pond would eliminate overtopping of North Llano Street at Lower Crabapple Road for the 100-year storm. The pond could also offset the additional flood flows from the Trailmoor drainage that would be discharged under Alternative A20. A third benefit of the pond is that runoff from the upper end of North Milam Street could be routed to the pond along an existing ditch. It is likely that some flood flows may already spill into this ditch for larger storm events. The proposed pond site provides 37 acre-feet of detention storage, with a maximum pond depth of 8.5 feet and an area of approximately 6 acres. With this detention pond in operation and with a 3' by 5' box culvert outlet, the 100-year flood flow corresponding to fully-developed watershed conditions can be reduced by 76 percent.

On-site detention would have a significant effect on flows within this drainage area. The primary benefits would be to prevent overtopping of North Llano Street during the 100-year storm event and to maintain the existing conveyance capacity for future diversions of flood flows from the Trailmoor drainage.

6.1.10 Carriage Hills Drainage

A prime detention site is located just upstream of the major localized flooding problem sites in the Carriage Hills subdivision. This pond site (Alternative A22) is just west of Edgewood Drive and just upstream of the channel that causes flooding problems as it discharges onto Driftwood Drive. This pond site would essentially eliminate the flooding problems upstream of Ridgewood Drive on both Edgewood Drive and Driftwood Drive. It would also provide 10-year flood protection relative to downstream flooding problems and significantly reduce potential flood damages and street flooding up to the 100-year storm. The proposed pond size is 15 acre-feet, has a maximum depth of five feet and has a surface area of approximately five acres. A 94-percent reduction in the 100-year flood flow (from 267 cfs to 16 cfs) can be achieved with this pond size and an 18-inch pipe outlet.

Although preliminary consideration has been given to installing a storm drain from the north part of Driftwood Drive (Site L29) to Stream FB-1, this alternative does not appear to be cost-effective because it would require over 2,200 feet of 66-inch and 72-inch pipes to provide conveyance capacity for the 10-year flood flow. A more practical alternative involves combining the upstream detention pond (A22) with storm drains at the lower (south) end of Driftwood Drive and along North Adams Street (A23). Because of the significant flow reductions provided for larger storms by the proposed upstream detention pond, the design storm for these storm drains can be limited to the five-year flood event. Even with this level of design protection, this alternative still would require installation of 600 feet of 48-inch reinforced concrete pipe (up Driftwood), 400 feet of 54-inch reinforced concrete pipe (along North Adams), 20 inlets and 700 feet of grass-lined channel. The flood flow reductions provided by the upstream detention pond, combined with the five-year flood storm drain capacity and available street flow conveyance, overall would provide flooding protection for the residential structures along Driftwood Drive and North Adams Street for approximately the 100-year flood event.

On-site detention would have a benefit within this drainage area in that it would prevent increases in flood flows that are already are causing flooding problems. Although not as effective as the proposed regional detention pond, on-site detention would provide benefits to other locations that are not downstream of the proposed pond. The primary benefit of on-site detention in this watershed would be to reduce design flows for the storm drain alternatives.

The conveyance capacity of the culverts under Ridgewood Drive where the tributary of Stream FB-1 from the Stone Ridge subdivision crosses is considerably less than that required to pass the 10-year flood flow. A potential regional detention pond site is located just upstream of this crossing and downstream of the existing Stone Ridge temporary detention pond. This regional pond could be constructed to reduce flood flows so that the Ridgewood culverts would have at least 10-year flood flow capacity without overtopping the roadway. This pond could also achieve a 60-percent reduction in the 100-year flood flow at this location. This level of reduction would reduce the amount of roadway overtopping and also reduce the flooding threat to adjacent residential structures. This regional detention pond facility would cover approximately 11 acres and have a storage capacity of 19 acre-feet, with a maximum depth of 13 feet. The outlet required to achieve the stated flow reductions is a 4.5-feet wide by 3-feet high box culvert.

It would also be possible to improve the Ridgewood culverts to provide additional floodwater conveyance capacity. With some additional channel grading upstream and downstream of the roadway, two 48-inch reinforced concrete pipes would provide sufficient capacity for conveying the 10-year flood flow, without overtopping. Allowing for one foot of overflow would provide capacity for the 50-year flood event. This alternative, combined with the regional detention alternative described above, would allow the passage of the 100-year flood flow through the expanded culverts.

On-site detention would prevent increased overtopping and flooding at the Ridgewood crossing. Without on-site detention, there is a projected 40 percent increase in flows for the 10-year storm event, from 228 cfs to 318 cfs.

6.1.11 West Creek Street Drainage

Since there are significant stormwater ponding problems in this area, a new storm drain designed for conveyance of 10-year flood flows appears to be justified. For South Bowie Street, this requires approximately 600 feet of 36-inch reinforced concrete pipe and seven inlets. An additional 750 feet of 30-inch reinforced concrete pipe with 4 inlets is required for South Edison Street and west of West San Antonio Street.

6.1.12 Old Harper Road Drainage

For passing the 10-year flood flow under fully-developed watershed conditions, the low water crossing on Armory Road will require four 36-inch by 58-inch corrugated metal pipes. Installation of these pipes with the current road elevation will require construction of a grass-lined trapezoidal channel downstream for approximately 400 feet. The required top width of the channel is about 40 feet.

At the low water crossing on Basse Lane (Site L37), three 36-inch by 58-inch corrugated metal pipes are needed for conveying the 10-year flood flow. The existing swale would need to be deepened and graded to form a triangular channel for about 250 feet downstream of the culverts.

To accommodate future conditions, it appears to be desirable to reroute the stormwater runoff underneath Basse Lane at Duderstadt Drive instead of allowing it to continue to flow northward along the roadside swale (Site L38) toward the low water crossing. This flow rerouting would reduce the size of the culverts required at the Basse Lane low water crossing, and it would eliminate the need to improve the swale running north along Basse Lane. However, this alternative would require construction of a 30-foot wide (top width) trapezoidal channel north and east of Basse Lane and acquisition of a drainage easement for the channel. For floodwater conveyance underneath Basse Lane, four 36-inch by 58-inch corrugated metal arch pipes would be needed.

Along South Bowie Street between Basse Lane and Postoak Road, a set of culverts is needed to safely convey stormwater that normally spills over the roadway. This would involve installing three 36-inch by 58-inch corrugated metal arch pipes at a point approximately 800 feet north of Basse Lane. Some minor grading would also be required upstream and downstream of the road.

No regional detention pond sites have been identified within this drainage; however, some regional ponds could be developed depending on eventual development patterns. On-site detention would be beneficial since the 10-year flood flows could increase considerably with the conversion from existing watershed conditions to fully-developed watershed conditions. On-site detention would reduce the required sizes and/or capacities of drainage facilities by about one-third of those described above. It should be noted that the fully-developed flood flows projected for this drainage are only estimates and may vary significantly depending on the level of ultimate development and the type of conveyance facilities that are constructed.

6.1.13 Winfried Creek Drainage

No specific alternatives have been identified for this drainage area since there are no major flooding problems. On-site detention would be beneficial in that potential future problems with erosion and overtopping of some bridge crossings could be reduced. Several good regional detention pond sites are available in the area if stormwater detetion is deemed necessary in the future. Some monitoring of erosion problems around bridges and culverts also is recommended.

6.1.14 Five Points Area

Alleviation of flooding in this area would require installation of a 42-inch storm drain northward from the Five Points intersection to Barons Creek. There are several potential storm drain routes; however, the most attractive appears to be through the park and the proposed bus terminal area. This is the natural flow path for stormwater runoff, and it would result in the least disruption of traffic. Approximately 1,150 feet of 42-inch reinforced concrete pipe are needed, which includes 200 feet of pipe running east along Park Street to the Five Points intersection. This alternative (A32) would also require 300 feet of 36-inch reinforced concrete pipe, 200 feet of 18-inch reinforced concrete pipe, and 14 inlets. An enhancement (A33) to this alternative would be to include storm drains and inlets in Ufer Street. This enhancement would add 600 feet of 24-inch reinforced concrete pipe and 4 inlets to provide 5-year floodwater conveyance capacity in the street and 10-year floodwater conveyance capacity at the low point on

Ufer.

Alternative A32 should also alleviate most of the flooding north of the Five Points intersection along Liveoak Street and at the channel to Granite Street (Sites L47, L48 and L49).

6.1.15 South Adams Drainage

No drainage improvements have been identified for this area.

6.1.16 Highway - Apple Drainage

One alternative is to intercept stormwater flows on the upstream end of Highway Street at Creek Street to reduce spills into the Friendship Lane drainage and to reduce the amount of flow at the Highway Street and South Eagle Street flooding problem areas (Sites L51 & L53). This would require routing the flow through 1,400 feet of 36-inch reinforced concrete pipe along South Creek Street to Barons Creek. An additional 1,300 feet of 30-inch reinforced concrete pipe, along with approximately nine inlets, would also be required. These improvements would not significantly affect the peak flows at South Eagle Street, although they would reduce the flood duration. Implementation of this alternative is more critical if the regional detention alternative is not used for the Friendship Lane drainage.

At the downstream end of Highway Street near South Eagle Street, the best alternative would be to construct a grass-lined channel south of Highway Street and extending through the natural low area and natural flow path. Three 36-inch by 58-inch corrugated metal arch pipe would be needed to convey the flow under South Eagle Street at the current location of the low water crossing.

Problems along Apple Street could be remediated by installing a 36-inch reinforced concrete pipe along with six inlets to provide nearly 10-year floodwater conveyance capacity.

6.1.17 Dry Creek Drainage

The old road bridge just downstream of U. S. Highway 87 should be removed to reduce erosion and to prevent backwater problems for the culverts under U. S. Highway 87. Some re-vegetation of the channel banks is needed.

The culvert under South Crenwelge Road near its intersection with Gold Road needs to be expanded to include an additional box to provide 10-year floodwater conveyance capacity. Some channel work upstream and downstream of this location is also necessary.

6.2 STREAM FLOODING

The extent of existing flooding problems along the principal creeks and streams flowing through the City have been discussed in Section 5.2. The effective flood insurance maps of the City delineate existing 100-year floodplains along Barons Creek, Town Creek and Stream FB-1. Based on results from revised and updated HEC-2 hydraulic models that have been developed in this Flood Protection Planning Study for these same watercourses, it does not appear that the recent growth and development of the City have yet to significantly change floodplain areas and flooding conditions along the major creeks and streams. As described in Section 6.1, most of the present flooding problems within the City generally are considered to be localized in nature and typically caused by inadequate drainage facilities, or the lack of drainage facilities.

Still, there are some areas along the major creeks and streams where flooding of adjacent properties can occur, particularly during larger storm events such as the 100-year flood. There are also some areas along the major watercourses where the present 100-year flood levels, as determined in this study using the refined HEC-2 hydraulic models, appear to be somewhat higher than those previously determined in the effective Flood Insurance Study for the City. There are also some areas where certain modifications in existing channels, bridges or other drainage structures should be made in order to improve floodwater conveyance or to reduce the potential for upstream flooding. Several of these situations are discussed below.

6.2.1 Town Creek

Perhaps one of the most obvious flood control measures that could be undertaken to improve the hydraulic efficiency of Town Creek is to remove the old low water crossing from under the Elk Street bridge. Based on simulations with the revised HEC-2 model of Town Creek, it appears that 100-year flood levels upstream of Elk Street would be lowered by about 2.5 feet if the existing bridge obstruction is removed and the existing bridge abutments are restructured to a 45-degree slope (Alternative A39). These modifications would increase the bottom width of the channel under the bridge from 16 feet to 51 feet. The 2.5 feet of drop in upstream flood levels due to removal of the obstruction would occur over the first 100 feet of channel immediately upstream of the bridge. At the low water crossing upstream of Elk Street, the resulting drop in the 100-year flood level would be about 1.8 feet, and since this low water crossing causes the flow in the creek to pass through critical depth, no additional benefits of the Elk Street bridge improvements are realized upstream of this crossing. As mentioned previously in Section 5.2.2.1, the structure adjacent to the low water crossing presently is not within the 100-year floodplain; consequently, the removal of the old bridge obstruction at Elk Street and the associated reductions in upstream 100-year flood levels are not likely achieve any significant immediate reductions in the potential flooding of adjacent properties. Still, from the standpoint of improving floodwater conveyance, it is important that removal of the Elk Bridge bridge obstruction be given serious consideration (Alternative A39).

Results from the revised HEC-2 hydraulic model of Town Creek, which now extends upstream through the new Cross Mountain West subdivision, indicate that the roadway at Morse Street is overtopped by the 10-year flood flow. At this location, an old railroad tank car presently serves as the culvert under Morse Street. Replacement of this existing culvert with four 8' x 8' concrete boxes (Alternative A40) and raising the road surface from its existing elevation of 1726.0 feet msl up to 1727.5 feet msl would provide sufficient conveyance capacity to handle flood flows produced by the 100-year storm (Alternative A40).

6.2.2 Stream FB-1

Simulated flood levels from the revised HEC-2 hydraulic model of Stream FB-1 indicate that the roadway at the Lower Crabapple Road crossing is inundated by floodwaters during the 10-year flood event. The culverts at this crossing consist of two 24-inch drain pipes. Aside from these pipes being severely undersized for effectively conveying floodwaters from the upstream watershed, it appears that some of the roadway overtopping problem is caused by high tailwater on the culverts as a result of the narrow channel downstream of the road crossing. Essentially, backwater from the downstream channel is reducing the hydraulic capacity of the existing culverts. Before installing larger culverts to improve the floodwater conveyance under the roadway, the constricted flow conditions downstream would need to be improved.

Options for widening and lowering of the downstream channel to provide additional conveyance capacity and to lower flood levels downstream of the Lower Crabapple Road crossing have been investigated using the revised HEC-2 hydraulic model of the stream. A trapezoidal channel with a bottom width of 25-feet, 4:1 side slopes and a flattened bottom slope of about 0.01 feet per foot has been incorporated into the model from the road crossing downstream for a distance of about 700 feet. This length of channel improvement extends through the most constricted section of the existing channel. With this modified and flattened channel, the flowline of the channel at the existing culverts would be lowered from 1752.00 feet msl to 1747.75 feet msl, which would allow larger pipes to be installed under the road without raising the road surface above its present elevation of 1755.00 feet msl.

With the improved channel downstream and with four 53" by 85" corrugated metal arch pipes replacing the existing 24" culverts under the roadway (Alternative A41), the revised HEC-2 model has been operated to evaluate flooding conditions in the vicinity of the crossing. These results indicate flood flows up to and including those produced by the 50-year storm event would be conveyed through the larger pipes without overtopping of the roadway. With the benefits of a regional detention pond upstream, as is described in the next section, the four 53" by 85" corrugated metal arch pipes also would be capable of passing the 100-year flood flows without overtopping the roadway.

6.3 REGIONAL DETENTION PONDS

6.3.1 Town Creek

The feasibility of regional stormwater detention ponds has been investigated within the Town Creek watershed. Such regional detention ponds have been considered as a means for reducing the existing flooding threat to structures along the creek, for reducing floodwater overtopping of roadways, for offsetting the potential increases in peak flood flows caused by future watershed development, and for possibly accommodating any increased discharges resulting from certain localized drainage improvement or flood control alternatives.

Seven sites have been reviewed for their potential effectiveness at improving both localized and downstream flooding conditions. After initial screening, the prime sites were visited and recent (1994) aerial photographs of the areas were examined. For two of the best sites that appeared to be feasible, the inflow hydrographs for the 100-year flood were developed using the HEC-1 model developed in this study, and preliminary grading plans and outlet sizes were established based on spreadsheet analyses of the hydrographs. Additional HEC-1 simulations then were performed to evaluate the effectiveness of the ponds for reducing downstream flood flows and flood levels and to refine the outlet and pond designs. Although such detailed analyses have been performed for only the two pond sites, at least three other sites also appear to be feasible and could be used as alternate pond sites, if necessary.

The primary detention site for the Town Creek watershed is located upstream of North Cherry Street on the western tributary to Town Creek (Alternative A42). The proposed pond has a storage capacity of 105 acre-feet, with a maximum depth of about 11 feet. This stormwater detention facility would cover approximately 19 acres, and it would have an outlet consisting of four 3' by 5' box culverts. Some additional considerations may be needed with regard to the existing stock pond that is located just downstream of this detention pond site. With this configuration, the pond would reduce the 100-year flood flow at the outlet by over 1,350 cfs, for a 57-percent reduction. Because of the lagging effect of the pond on the outflow hydrograph relative to the times of concentration for other subwatersheds, the reduction in flood flow actually increases to about 1,450 cfs at the confluence of Town Creek with Barons Creek. This represents a significant reduction in flood flows that correspondingly results in reduced water surface elevations throughout the mainstem of Town Creek downstream of West Morse Street. One of the side benefits of the reductions in flood flows from this project would be that it allows additional discharges of stormwater into the creek downstream from some of the localized drainage improvement alternatives. For example, this would include Alternative A16, which would divert the College-Llano drainage to Town Creek, instead of allowing it to continue to flow to Stream FB-1. The design discharge for Alternative A16 (for the 10-year storm) is approximately 200 cfs. The pond configuration described above (Alternative A42) would more than offset the increased flow associated with Alternative A16. It also would be feasible to downsize the pond at this site, if the goal is only to offset the effects of Alternative A16.

The second detention pond site evaluated in detail is on the mainstem of Town Creek upstream of Morse Road and just upstream of the new box culverts in the Cross Mountain West subdivision (Alternative A43). This alternative would involve modifying the upstream drop structure to serve as the outlet for the pond excavated upstream. The volume of this pond at the 100-year peak stage is 11 acre-feet, and it has a maximum depth of just over 10 feet. The area of the pond is about six acres. The outlet would consist of four 54-inch reinforced concrete pipes constructed through the existing drop structure with a 94-foot weir section located along the current flowline at the top of the drop structure. The effectiveness of this site is somewhat limited by the elevation of the adjacent platted lots; however, this detention pond does provide a reduction of 130 cfs in the 100-year flood flow, which is equal to about nine percent of the total flow. The peak discharge rate from the pond for the 100-year flood is approximately 50 cfs less than the peak flow under existing watershed conditions, and this appears to be enough to offset the additional flow that would be discharged to this branch of Town Creek under localized flooding improvement Alternative A13. This regional pond is particularly effective with respect to reducing flood flows over Morse Road. The overtopping of Morse Road is reduced by 0.5 feet (to less than two feet) for the 10-year flow. The projected downstream reduction in flood flows associated with this pond appears to be sufficient to prevent any downstream impacts from the diversions associated with Alternative A16.

The combined reduction in flood flows for the 100-year storm by the two detention ponds results in the lowering of water surface elevations throughout Town Creek (downstream of Morse Road) by about two feet, with a maximum water level decrease of about three feet upstream of Washington Street. This effectively eliminates the threat of flooding along Town Creek with respect to existing residential structures and commercial buildings. This also lowers the depth of flow over the roadway structures that are overtopped and provides 10-year flood flow capacity at Crockett and Milam Streets, which are overtopped by the 10-year storm under existing flood flow conditions. This is a particularly important benefit since all the roadway crossings on Town Creek on the west side of the City are overtopped for storms more frequent than the 10-year event.

6.3.2 Barons Creek

A preliminary investigation of the feasibility of regional detention also has been performed for Barons Creek. Because of the limited number of problem areas along

Barons Creek, there is little need for regional detention. Additionally, Barons Creek has a relatively long (approximately three hours) time of concentration due to the large portion of the watershed upstream of the City. Regional detention within or near the City could actually increase flows in Barons Creek by lagging the relatively quick local watershed discharges to be in phase with the later peak flows from the upper Barons Creek drainage area. Several potential regional pond sites have been identified in the Barons Creek watershed upstream of the City; however, no detailed analyses have been performed because of the apparent lack of need for flow reductions along Barons Creek through the City.

It should be noted that, in general, the same principle of regional detention ponds applies to on-site detention with respect to Barons Creek. However, on-site detention may still be required for control of localized flooding. If safe conveyance is available or provided to Barons Creek, on-site detention would not be necessary. For cases where on-site detention is necessary for localized problems, the detention time used to determine storage volumes should be less than one hour.

6.3.3 Stream FB-1

Eight potential regional pond sites have been identified for Stream FB-1. Three of these have been analyzed in detail with respect to localized flooding problems. One additional pond site just upstream of Lower Crabapple Road (Alternative A44) has been analyzed in detail specifically as an alternative for reducing downstream flooding. With a detention pond covering about 8.5 acres, a 100-year storage volume of 36 acre-feet and a maximum depth of about 8.5 feet, this site provides a reduction in the 100-year flood flow of about 570 cfs. This represents 23 percent of the peak flood flow just upstream of the flooding problem area in the Carriage Hills subdivision. This level of flood flow reduction also extends downstream to the Llano Highway crossing. The effect of this reduction is to lower the 100-year flood water surface elevation by 0.6 to 1.0 feet along the stream where five homes are located within the floodplain. For the 10-year storm, the detention pond would also reduce the flows sufficiently to prevent overtopping of the Llano Highway.

The detention pond site in the Stone Ridge subdivision that was analyzed as a localized flooding improvement alternative (A24) was also evaluated for its effectiveness with regard to stream flooding along Stream FB-1. A minimal reduction in flood flow (five percent) was achieved at the Llano Highway crossing with this pond, although there is a significant reduction of peak flow from the pond site. This site also discharges downstream of the primary stream flooding problems in the Carriage Hills subdivision. When considered with the proposed detention pond upstream of Lower Crabapple Road, this site provides no additional reduction in flood flows relative to that achieved by the other site alone. Therefore, this site is not considered to be generally effective with regard to reducing downstream flooding.

The regional detention pond site upstream of Lower Crabapple Road (Alternative A44) also was considered in conjunction with two other regional ponds in this watershed, Alternatives A21 and A22 as previously described in the localized flooding analysis in Section 6.1. The combination of these three ponds reduces the 100-year flood flows in Stream FB-1 by 23 percent at its confluence with Barons Creek. However, this is not a significant benefit since no current stream flooding problems have been identified south (downstream) of the Llano Highway.

7.0 DRAINAGE AND FLOOD PROTECTION ORDINANCES

As part of this Flood Protection Planning Study, consideration has been given to the possibility of the City implementing certain ordinances that would help to alleviate future flooding and drainage problems associated with and caused by the continued growth and development of the City. One particularly attractive option for such authority is a stormwater detention ordinance that would require all future development projects, with some noted exceptions, to implement drainage control measures to assure that existing rates of runoff are not being increased. This would tend to cap existing flood flows at their present levels.

Without stormwater detention, peak flood flows would increase because of increased stormwater runoff volumes caused by the additional impervious cover created by new development projects and because of faster rates of conveyance across or through new driveways, streets, parking lots, storm drains and channels. The conversion of land in the Fredericksburg area from a natural, undeveloped state to a moderately-developed condition (35-percent impervious cover) can result in a 40- to 50-percent increase in peak flood flows. However, more intense development for commercial, office, retail and/or medium density residential uses would result in greater increases in flood flows. Results from HEC-1 analyses performed as part of this Flood Protection Planning Study indicate that the 10-year flood flow from some subwatersheds could double if the projected future land use conditions occur. Conversely, low-density development, such as large-lot single family residential subdivisions, may not increase peak flood flows at all.

Stormwater detention provided by an individual land owner or developer as part of a specific new development project is referred to as on-site detention. This type of detention typically is provided on or immediately downstream of the development site by creating a stormwater storage pond. Such detention ponds usually are constructed by excavation within a drainageway, with berms or embankments installed around the excavated area. At the bottom of the detention storage pond, a small or restricted drainage outlet is provided to drain the pond. The outlet pipe or weir is designed to slowly release stormwater during a storm event so as to reduce the rate of runoff from the developed site to no more than that which occurred under predeveloped conditions, with the excess stormwater detained in the pond. Other typical features of on-site detention ponds include an emergency spillway to pass stormwater flows greater than the design discharge rate of the pond, an inlet flume or pipe to convey stormwater runoff into the pond without causing erosion, and various types of erosion protection works and velocity dissipators downstream of the pond outlet.

On-site stormwater detention is an effective means for preventing increased flooding problems by controlling the increased rates of runoff usually associated with watershed development. For this purpose, a draft stormwater detention ordinance has been prepared and presented to the City for review and consideration. This document now is under review by the City. Following is the text of the draft stormwater detention ordinance as it currently is being considered by the City.

DRAFT

City of Fredericksburg, Texas

STORMWATER DETENTION ORDINANCE

October 24, 1996

1.0 Purpose and Applicability

a) The growth in and around the City of Fredericksburg and the associated development and construction of buildings, paved surfaces, roads and other improvements has altered in the past and continues to alter the natural flow of surface waters on the land, which together with the construction of gutters, culverts, drains and channels for the carrying off of surface waters has both increased the quantity of stormwater and amplified the peak flow rates of runoff, thus leading to present and potential flooding of property and homes, dangerous flows within and over public roadways and streets, and soil and channel erosion.

b) It is the intention of the City Council to protect the health and safety of the citizens and visitors of the community and to prevent damage to private property and public facilities through the proper design and construction of both on-site and regional stormwater detention facilities that prevent or adequately reduce increases in peak flow rates of runoff that may otherwise increase the risk of flooding and the associated risk of public endangerment, property damage and erosion.

c) It is the intention of the City Council, through this Ordinance, to establish a regional stormwater detention pond program for the design and construction of regional stormwater detention facilities so that, where practical, the most cost-effective protection from flooding may be accomplished.

d) It is the intention of the Council to protect the health and safety of the citizens and visitors of the community and to prevent damage to private property and public facilities through the installation and use of temporary and permanent erosion control practices that prevent or adequately reduce increases in erosion and siltation that may otherwise increase the risk of flooding and the associated risk of public endangerment and property damage by clogging and/or partial filling of constructed or natural drainageways as well as drainage structures and detention ponds.

e) This Ordinance shall apply to all property within the planning jurisdiction of the City unless otherwise stated.

f) This Ordinance shall not apply to single family or duplex residential lots of subdivisions approved prior to the adoption of this Ordinance, unless specifically required by prior agreement between the City and the owners or developers of such subdivisions, or to new one- or two-lot subdivisions for single family or duplex residential lots, and this Ordinance is intended to be implemented for entire subdivisions at the time of platting and construction of street and drainage improvements and not on an individual lot basis for single family and duplex residential subdivisions.

2.0 Standards and Requirements for Stormwater Detention

a) No final subdivision plat, subdivision construction plan, site plan or building permit shall be approved by the City unless it can be demonstrated by the owner or developer of such property that the proposed development will not result in the additional identifiable adverse flooding of other property or public facilities, including roadways.

b) The above requirement shall be accomplished through one of the following means:

1) Design and construction of an on-site stormwater detention facility, or facilities, by the land owner or developer which limits the peak flood flows from the proposed development to the existing peak flood flows from the subject tract.

2) Participation by the land owner or developer in the Regional Stormwater Detention Pond Program in a manner sufficient to accomplish the goal stated in Item 2.a above. This may be accomplished though the contribution of funds and/or land to the Regional Stormwater Detention Pond Fund, as established in Section 3.0 below.

3) Construction of, or participation in the construction of, off-site drainage improvements, such as storm inlets, storm sewers, culverts, channel modifications, land filling, and/or other drainage facilities such that the peak flood flows for fully-developed watershed conditions from the watershed area in which the proposed development is located will be sufficiently and safely passed without flooding of downstream property and roadways.

4) Design and construction of the development utilizing limited impervious cover, infiltration of runoff from impervious cover via flow through pervious areas, and/or grass-lined swales or channels such that these measures result in a minimal increase in peak flood flows from the development.

c) Acceptance of requests from the land owner or developer to meet the stormwater detention requirements through measures listed in Items 2.b.2 through 2.b.4 above is solely at the discretion of the City.

d) Acceptance by the City of on-site stormwater detention plans will be based on the suitability and adequacy of the engineering and technical design of the proposed stormwater detention facility, as described in Section 5.0 below.

3.0 Regional Detention Pond Program

a) The City hereby establishes the Regional Stormwater Detention Pond Program whereby the City will design and direct construction of or otherwise facilitate construction of regional stormwater detention ponds in order to prevent increases in and, if practicable, to reduce peak flows of stormwater runoff.

b) The City hereby establishes, as the funding mechanism for the Regional Stormwater Detention Pond Program, the Regional Stormwater Detention Pond Fund, a dedicated fund into which the contributions by land owners and developers are deposited in lieu of construction of on-site stormwater detention facilities and from which funds are allocated for the design and construction of regional stormwater detention ponds and/or other off-site stormwater management and control facilities.

c) It is the intention of the Council to allow contributions to the Regional Stormwater Detention Pond Fund by land owners and developers in lieu of construction of on-site stormwater detention facilities for the purpose of the design and construction off-site improvements, which may include, either singly or in combination, regional stormwater detention ponds, storm sewers, culverts, inlets, gutters, swales and improved channels, in order to prevent or reduce downstream flooding problems.

d) The contributions to the Regional Stormwater Detention Pond Fund are non-refundable and are intended to be dedicated solely to implementation of drainage improvements and stormwater management and control facilities.

e) The level of contribution required to participate in the Regional Stormwater Detention Pond Program shall be based on the increase in volume and peak flow of the stormwater runoff from a proposed development and the potential for adverse downstream flooding impacts; therefore, the level of contribution will generally increase with increasing size of development, amount of impervious cover, and extent of on-site drainage conveyance modifications.

4.0 Standards and Requirements for Erosion/Sedimentation Controls

a) No final subdivision plat, subdivision construction plan, site plan or building permit shall be approved by the City unless the plans for the proposed development include temporary and permanent erosion and sedimentation control measures such that siltation of downstream drainageways are minimized.

b) The above requirement shall be accomplished through a combination of the following practices:

1) Installation of silt fences and rock berms before and during construction in order to reduce on-site soil erosion and provide temporary capture of sediment.

2) Temporary and/or permanent revegetation of bare ground in order to stabilize disturbed soil at the earliest practicable date.

3) Construction of on-site stormwater detention facilities by the land owner or developer in a manner such that detention ponds function as temporary sedimentation basins until permanent revegetation of the subject tract is accomplished.

4) Other measures which may be necessary to control erosion and sedimentation on a site by site basis.

5.0 Additional Standards for Approval

a) A Registered Professional Engineer, licensed in the State of Texas and qualified and experienced in the design and operation of stormwater detention ponds and related stormwater management facilities, shall perform the hydraulic and structural design of stormwater detention ponds and related stormwater management facilities, including the development of engineering and technical information required for evaluation by the City.

b) All design and technical information necessary to thoroughly evaluate the suitability and adequacy of the engineering and technical design of proposed on-site stormwater detention facilities and, if proposed, off-site facilities shall be provided to the City for review. All detention and runoff calculations, including computer model simulations, if used, shall be provided.

c) All on-site stormwater detention facilities shall be designed to adequately and safely pass all stormwater inflows, including flood flows and runoff from upstream and adjacent properties that have natural and/or existing overland flows toward and onto the subject tract. The on-site stormwater detention facilities should not impound stormwater onto or cause backwater to inundate any upstream or adjacent properties in excess of existing conditions.

d) On-site stormwater detention facilities shall not be placed such that they encroach into the regulatory 100-year floodplain as established by the City, Gillespie County, and/or the Federal Emergency Management Agency, unless it can be satisfactorily demonstrated to the City through the use of hydraulic modeling that such encroachment will not cause any rise in the 100-year flood level on other off-site properties or that the increase in the 100-year flood level caused by such encroachment will occur entirely onsite on the owner's or developer's property.

e) Additional engineering and technical rules and guidance with respect to the application and review of the stormwater detention requirements of this Ordinance may be provided by the City within a Drainage Criteria Manual.

f) Additional rules, guidance and requirements with respect to the application and review of requests for participation in the Regional Stormwater Detention Pond Program, off-site drainage improvements and other alternatives to on-site stormwater detention as listed in Items 2.b.2 through 2.b.4 above may be provided by the City within a Drainage Criteria Manual.

g) All design and technical information necessary to thoroughly evaluate the suitability and adequacy of proposed erosion and sedimentation control measures shall be provided to the City for review.

h) Additional rules, guidance and requirements with respect to the review and acceptance of temporary and permanent erosion and sedimentation control plans may be provided by the City within a Drainage Criteria Manual.

8.0 DRAINAGE IMPROVEMENT AND FLOOD PROTECTION PLAN

8.1 LOCALIZED FLOODING PLAN

The various alternatives for addressing localized flooding problems throughout the planning area as developed in Section 6.1 and as listed in Table 6-1 have been evaluated in general terms with respect to their relative feasibility, constructability, cost and effectiveness. A preliminary estimate of implementation costs has been prepared for the prime alternatives, i. e., those demonstrating the greatest effectiveness for reducing flooding in areas with the most critical problems. Some additional preliminary cost estimates also have been prepared for a few secondary alternatives to allow comparison with the primary alternatives. Based on these additional evaluations and cost comparisons, a list of thirteen recommended alternatives has been developed. These are listed and generally described in Table 8-1. The locations of the recommended alternatives are shown on the map of the area in Plate 8-1. Although other effective and feasible alternatives exist, these recommended alternatives appear to be the best suited for improving the most critical drainage and flooding conditions in the Fredericksburg area. The recommended alternatives are listed Table 8-1 in the general order of priority for implementation based on the same factors identified above that were considered in developing the list.

Considering that the recommended alternatives provide effective solutions for existing localized flooding problems and that the potential damages and loses, including loss of life, caused by this flooding could be a substantial burden for the citizens of Fredericksburg, it is important for the City to give strong consideration to implementing the recommended alternatives as soon as economically feasible. These recommended alternatives should be considered to represent the initial implementation phase of the overall master drainage plan for the City. Other effective, but more long-term, alternatives should be implemented as practical and as opportunities arise. These more long-term alternatives are listed and generally described in Table 8-2. These long-term alternatives are grouped in two levels of implementation priorities. The first group is referred to as Phase II (with Phase I being the recommended alternatives). These Phase II alternatives are considered to be relatively effective and efficient for reducing localized flooding problems, but they are not considered to be as critical as the recommended Phase I alternatives, particularly with regard to reducing flooding of structures and major street and road crossings. The second group of long-term alternatives is referred to as "Future" alternatives and generally, these have either a longer-term implication with respect to drainage and flood control planning or they are considered to be desirable drainage enhancements. Any specific alternative in either group of the long-term alternatives may be implemented as opportunities arise. Some

TABLE 8-1
LOCALIZED FLOODING RECOMMENDED ALTERNATIVES

PRIORITY	ALTERNATIVE DESIGNATION	PROBLEM SITE DESIGNATION	LOCATION	DESCRIPTION	EFFECTIVENESS	IMPLEMENTATION COST
1	A2	L1 - L5	Friendship Lane Drainage West of Washington	Regional Detention Pond Area - 8.6 acres Max. Depth - 6 feet 100-Year Volume - 26 acre-feet 18" Outlet Pipe	Reduces 100-Yeear Storm flow 96% at the site Reduces 10-Year and 100-Year flows 20% at the South Creek Subdivision	$340,000
2	A12	L8 - L10	N. Milam Street	600 feet - 48" RCP 1,900 feet - 60" RCP 20 inlets 500 feet of curb	10-Year Capacity Eliminates House Flooding on Milam Eliminates House Flooding on W. Centre & W. College with additional upstream improvements (A13)	$670,000
3	A22	L28 - L31	West of Edgewood	Regional Detention Pond Area - 5 acres Max. Depth - 5 feet 100-Year Volume - 15 acre-feet 18" RCP outlet	Reduces 100-Year flow at the discharge point by 94% Eliminates problems on Driftwood north of Ridgewood 10-Year protection downstream Reduces downstream street flooding	$305,000
4	A32	L44 - L49	Park Street	1,150 feet - 42" RCP 300 feet - 36" RCP 200 feet - 18" RCP 14 inlets	10-Year Capacity Approximately 100-Year protection for buildings	$260,000
5	A33	L46	Ufer Street	600 feet - 24" RCP 4 inlets	5-Year Stormsewer Capacity 10-Year Capacity at low p of streetoint	$65,000
6	A16	L17 - L24	N. Llano	1,300 feet - 60" RCP 1,000 feet other Stormsewer 20 inlets	10-Year Capacity Reduces D/S flows 50 - 60% Eliminates House Flooding except near Travis for 50-Year to 100-Year events. Reduces D/S Erosion	$535,000
7	A27	L35	South Edison	750 feet - 30" RCP 4 inlets	10-Year Capacity	$70,000
8	A1c	L1	Friendship Lane Low Water Crossing	7 - 36" x 56" CGMP Downstream Channelization Associated Roadway Work	100-Year Capacity with Regional Detention	$80,000

TABLE 8-1
LOCALIZED FLOODING RECOMMENDED ALTERNATIVES

PRIORITY	ALTERNATIVE DESIGNATION	PROBLEM SITE DESIGNATION	LOCATION	DESCRIPTION	EFFECTIVENESS	IMPLEMENTATION COST
9	A3	L1 - L3	Friendship Lane Drainage Just upstream of South Creek Subdivision	Regional Detention Pond Area - 9.3 acres Max. Depth - 7.6 feet 100-Year Volume - 48 acre-feet 24" Outlet Pipe 800 feet U/S Channel	Combined with A2 Pond, Reduces 100-Year flow at South Creek Subdivision to 42 cfs and reduces the flow at Friendship Lane low water crossing by 94%	$465,000
10	A23	L30 - L31	Driftwood and Adams	600 feet - 48" RCP 400 feet - 54" RCP 20 inlets 700 feet Grass Lined Channel	5-Year Capacity with Detention Approximately 100-Yr protection from house flooding Reduces street flooding	$301,000
11	A10a	L7	Schubert St.	Purchase 2 - 0.5 acre vacant lots Regrading	25-Year Flood Protection for Houses Reduced Street Flooding	$30,000
12	A19	L25 - L26	Trailmoor and Llano	800 feet - 36" RCP 300 feet other Stormsewer 15 inlets	10-Year Capacity Reduces Llano overtopping	$160,000
13	A36	L52	Apple Street	1,150 feet - 36" RCP 6 inlets	10-Year Capacity Reduces house flooding potential	$155,000
					TOTAL	$3,436,000

TABLE 8-2

LONG-TERM DRAINAGE IMPROVEMENT ALTERNATIVES

ALTERNATIVE DESIGNATION	PROBLEM SITE DESIGNATION	LOCATION	DESCRIPTION	EFFECTIVENESS
PHASE II				
A6b	L4	Washington @ Friendship Ln.	Add 1 - 4' x 4' box culvert	10-Year Capacity with U/S Detention
A13	L8 - L12	W. Burbank	1,050 feet - 48" RCP 11 inlets	10-Year Capacity Eliminates House Flooding on Burbank Eliminates House Flooding D/S with Alternative A12
A14	L15, L16, L18 - L24	E. Burbank	2,200 feet - 48" RCP 9 inlets Minor Channelization Drainage Easement Acquisition	10-Year Capacity Eliminates Structure Flooding near Llano & W. Burbank Reduces downstream problems
A15	L16	N. Lincoln	250 feet of Berm	10-Year Capacity within Street Eliminates House Flooding Potential with Alternative A14
A18	L23 - L24	Travis	400 feet of Erosion Control 200 feet Minor Grading & Channelization	Reduces existing erosion problem
A25	L34	Ridgewood	2 - 48" RCP U/S & D/S Channel Grading	10-Year Capacity 50-Year Capacity with 1' of overtopping 100-Year Capacity with upstream detention (A24)
A26	L35	South Bowie	600 feet - 36" RCP 7 inlets	10-Year Capacity
A35	L51 and L53	Highway Street South Eagle	1,800 feet Grass Lined Trap. Channel Top Width - Approximately 30 feet 3 - 36" x 58" Arch CGMP	Eliminates Street Flooding along Highway Street (L52) Provides 10-Year Capacity at Eagle Street

TABLE 8-2
LONG-TERM DRAINAGE IMPROVEMENT ALTERNATIVES

ALTERNATIVE DESIGNATION	PROBLEM SITE DESIGNATION	LOCATION	DESCRIPTION	EFFECTIVENESS
PHASE II				
A37	L54	U. S. Highway 87	Remove old road bridge Revegetation	Reduces Erosion Eliminates backwater from structure
A38	L55	Crenwelge Road	Add box to culvert Approximately 300 feet of channel improvements	10-Year Capacity Reduces structure flooding potential
FUTURE				
A10b	L7	Schubert St.	Excavation - 3.6 acre-feet pond 1,100 feet - 24" RCP	Eliminates House/Street Flooding for 100-Year Storm
A20	L25 - L26	Morning Glory & Broadmoor	2,000 feet - 24" & 36" RCP 12 inlets	10-Year Capacity Provides 100-Year Capacity at Trailmoor and Llano with Alternative A19
A21	L13, L14, L25 - L27	North of Morning Glory	Regional Detention Pond Area - 6 acres Max. Depth - 8.5 feet 100-Yr Volume 37 acre-feet 3' x 5' Box Culvert Outlet 1,100 feet U/S Channelization 2 - 36" x 58" CGMP	Eliminates 100-Year overtopping of Llano Offsets additional discharge from A20 Reduces Street Flooding and Spillovers on N. Milam
A28	L36	Armory Road	4 - 36" x 58" Arch CGMP 400 feet of D/S Channel	10-Year Capacity
A29	L37	Basse Lane	4 - 36" x 58" Arch CGMP 250 feet of D/S Channel	10-Year Capacity

TABLE 8-2
LONG-TERM DRAINAGE IMPROVEMENT ALTERNATIVES

ALTERNATIVE DESIGNATION	PROBLEM SITE DESIGNATION	LOCATION	DESCRIPTION	EFFECTIVENESS
FUTURE				
A30	L38 and L39	Basse Lane	4 - 36" x 58" Arch CGMP 850 feet Grass Lined Trap. Channel Top Width - 30 feet	10-Year Capacity
A31	L40	South Bowie	3 - 36" x 58" Arch CGMP	10-Year Capacity

examples of these opportunities include the installation of storm drains when streets are repaved or other utilities are installed, the installation of drainage channels as part of new subdivision developments, and the installation of drainage improvements in conjunction with highway projects. Although funding restrictions may preclude implementation of many of the long-term alternatives, they are included here for general guidance purposes with respect to long-range planning by the City.

The implementation cost estimates presented in Table 8-1 for each of the recommended Phase I alternatives are preliminary and should be considered approximate. These estimates will need to be refined during the preliminary engineering design of the alternatives as they are selected for implementation by the City. The estimates account for all of the significant cost factors associated with implementing each alternative and are reasonable for the purposes of cost comparisons and planning. Work sheets itemizing the cost details for each of the alternatives are available. These work sheets present the basis for estimating the total costs for the alternatives, and they include costs for earth work, material hauling, concrete facilities construction, drain pipes and culverts, engineering and surveying, land acquisition, and contingencies.

The total estimated cost for implementing the thirteen recommended localized flooding alternatives is approximately 3.5 million dollars. This level of investment in the City's drainage system provides substantial flood protection benefits for most of the significant flooding problem sites located the City. Since many of the most serious flooding problem sites experience some degree of flooding during the occurrence of storms much smaller than the 10-year event, the adoption of the 10-year flood design capacity for most of the recommended storm drains and the 100-year flood design for detention ponds provides major improvements with regard to flooding potential and existing flooding hazards.

It should be noted that two of the recommended alternatives (A-12, North Milam Street storm drains, and A-16, North Llano Street storm drains) involve drainage improvements along State highways. While the total costs for implementing these projects are relatively high compared to those for other recommended alternatives (they represent over 35 percent of the total Phase I costs), there is some potential for cost sharing on these projects with the Texas Department of Transportation (TxDOT), since a substantial portion of the benefits to be derived from these projects relates to reduced flood flows on or across the State roadways.

Additional details regarding the various Phase I, Phase II and Future alternatives for drainage improvements and flood control measures is provided in the following sections for each of the localized flooding problem areas.

8.1.1 Friendship Lane Drainage

The combination of alternatives involving regional stormwater detention (A1c, A2, A3 & A6b) provides the most cost-effective solution in this drainage area. Combinations of alternatives involving channels and/or storm drains (A1a, A4, A5a, A6a & A7 or A1a, A4, A5b, A6a & A7) generally afford protection for storms less than the 10-year event, with implementation costs that typically are 10 to 25 percent higher than those for the regional detention alternatives. The recommended regional detention ponds also provide significant flow reductions and flood benefits for floods ranging up to the 100-year event. The significant flood reduction benefits resulting from construction of the regional ponds translate downstream without implementation of other drainage improvements, whereas channelization and storm drain projects typically need to be implemented from downstream to upstream within a given watershed to avoid creating additional flooding problems. Alternatives involving the construction of drainage channels (A5a & A7) along Friendship Lane have the additional adverse impact of consuming right-of-way that may be needed for future widening of this important roadway. Therefore, additional right-of-way purchase for the channel alternatives was included in the overall cost for comparison purposes.

8.1.2 Schubert Street Ponding

The recommended alternative (A10a) for initial implementation involves purchasing the two vacant lots, performing some minor regrading to enhance the existing detention characteristics of the depression area, and installing some additional inlets. The cost of these improvements would be less than one-sixth of that required to install adequate storm drain capacity to make the vacant lots buildable, i. e., Alternative A9, with a total cost of $185,000. In the future, these lots could be excavated to create a larger detention pond (Alternative A10b) that would provide nearly 100-year flood protection for about 80 to 85 percent of the cost of the large storm drain alternative (A9).

8.1.3 Cross Mountain - Milam Drainage

Alternative A12 is recommended to improve flooding conditions in the lower end of this drainage area. Although the total costs associated with this project are significant ($670,000), they are about seven percent less than those required to install storm drains up both Milam and Pecan Streets.

8.1.4 Burbank - Llano Drainage

The alternative for this localized flooding problem area is included in the Phase II implementation list since it is relatively expensive with respect to the amount of benefits provided.

8.1.5 North Lincoln Drainage

The berm alternative for this localized flooding problem area (Alternative A15) should be installed in Phase II at the same time the Burbank-Llano storm drain project (Alternative A14) is constructed.

8.1.6 College - Llano Drainage

Alternative A16 is recommended even though the total cost of this project is relatively high due to the large pipe size and the extensive depth of the trenching required. Even with consideration of the extra costs associated with the deep trenches, this alternative still is less than 50 percent of the cost of installing storm drains down College Street (Alternative A17) to discharge stormwater into Stream FB-1 at the eastern end of Travis Street. However, with Alternative A16, some type of stormwater detention facility located on Town Creek upstream of Llano Street would be necessary to offset the increased flood flows in the lower portion of Town Creek caused by the stormwater diversions associated with this alternative. Either of the regional detention pond alternatives (described in Section 6.3) would be sufficient to offset the flood flow increases in Town Creek associated with this alternative. If the incremental cost of the upstream regional detention required to offset the increased flood flows in lower Town Creek is assigned to the cost of this alternative, the total cost still would be less than that of Alternative A17 by about $400,000.

8.1.7 College - Travis Drainage

The College-Llano storm drain (Alternative A16) will provide significant benefits for all the localized flooding problem sites in this area.

8.1.8 Trailmoor Drainage

Alternative A19 is recommended for this drainage area; although, existing flooding problems are not particularly hazardous.

8.1.9 Morning Glory - Llano Drainage

Although the regional stormwater detention site in this drainage area is effective for reducing flood flows, it has been categorized as a Future long-term alternative. Changes in projected land use within this area or other watershed modifications may increase the implementation priority of this alternative at a later date.

8.1.10 Carriage Hills Drainage

The regional detention pond (Alternative A22) is very effective for reducing the street and structure flooding problems in this area. This alternative and Alternative A23 (storm drains along Driftwood Drive and North Adams Street) are recommended for implementation.

8.1.11 West Creek Street Drainage

The storm drain along South Edison Street (Alternative A27) is recommended since it helps to alleviate the significant floodwater ponding problem along West San Antonio Street, just west of Edison Street.

8.1.12 Old Harper Road Drainage

The alternatives for this area are all considered to be Future alternatives since the need for these drainage improvements is somewhat dependent upon the manner in which development occurs. The alternatives identified for this area serve as a general guide for future drainage improvements; therefore, plans for specific development projects in the area may necessitate some adjustments and modifications in the alignments and capacities of the proposed drainage improvements.

8.1.13 Winfried Creek Drainage

No specific alternatives have been identified for this drainage area since no major flooding problems exist.

8.1.14 Five Points Area

Two drainage improvement projects (Alternatives A32 & A33) are recommended for this area because of the significant amount of flooding and the relatively high volume of traffic that occurs through this problem area. The final alignment of the 42-inch storm drain is somewhat dependent on acquisition of easements; however, the overall cost should not vary significantly.

8.1.15 South Adams Drainage

No specific drainage improvement alternatives have been identified for this area since no major flooding problems exist.

8.1.16 Highway - Apple Drainage

The most significant flooding problem sites within this area appear to be along Apple and Pear Streets (Site L52). Most of the flooding problems can be eliminated through implementation of the recommended alternative (A36).

8.1.17 Dry Creek Drainage

The potential flooding conditions in this area do not represent a significant immediate problem. However, the identified alternatives should be considered as part of the Phase II implementation program.

8.2 STREAM FLOODING PLAN

Three of the drainage and flood improvement alternatives previously identified and discussed in Section 6.2 are recommended. These alternatives include two regional detention ponds, one each on Town Creek and Stream FB-1, and a culvert replacement project on Stream FB-1 at Lower Crabapple Road, and they are listed and generally described, with estimated implementation costs, in Table 8-3. These alternatives provide effective benefits with regard to the most significant stream flooding problem sites, and they should be included as part of the initial Phase I of the overall master drainage plan for the City. As indicated in Table 8-3, the total estimated cost of the three recommended stream flood protection alternatives is nearly two million dollars. Additional stream flood protection alternatives that are considered to be less critical and, therefore, more long-term projects are listed and generally described in Table 8-4.

Further discussion of the various alternatives available for drainage and flood improvements along the principal watercourses in the planning area is presented in the following sections.

8.2.1 Town Creek

The most cost-effective means for reducing flooding along Town Creek is construction of the large regional detention pond on the western tributary to upper Town Creek (Alternative A42). This detention facility will reduce the 100-year flood water surface along most of Town Creek by nearly two feet. Based on hydraulic analyses performed with the revised HEC-2 model of Town Creek, this alternative would produce lower flood levels at most locations along Town Creek than would result if several of the roadway crossings were replaced with larger bridges, the effects of which typically would occur only over very short reaches (less than 2,000 feet) upstream of the bridges. Furthermore, the cost of this regional detention pond alternative ($1,170,000) would be approximately equal to the cost of replacing two roadway crossings with bridges. Therefore, this regional detention pond can provide more flood level reduction benefits for more of Town Creek than replacement of any two roadway structures on Town Creek. Also, with this regional detention pond in place, Alternative A16 (storm drains) could be implemented to reduce the flooding problems at and downstream of Llano and College Streets. For these reasons, Alternative 42 is recommended for implementation as a Phase I project.

The regional detention pond located near Cross Mountain West (Alternative A43) is not

TABLE 8-3
RECOMMENDED STREAM FLOOD PROTECTION ALTERNATIVES

ALTERNATIVE DESIGNATION	LOCATION	DESCRIPTION	EFFECTIVENESS	IMPLEMENTATION COST
A42	Town Creek - West Tributary U/S of N. Cherry	Regional Detention Pond Area - 19 acres Max. Depth - 10.4 feet 100-Year Volume - 105 acre-feet Outlet 4 - 3' x 5' Box Culverts	Reduces 100-Year Storm flow 57% at the site Reduces 100-Year flows by 1,450 - 1,600 cfs in Town Cr. Eliminates overtopping of Milam and Crockett for storms smaller than the 10-Year storm Reduces 100-Year water surface elevation by 0.5 - 3.0 feet from just below Morse Road to Barons Creek confluence	$1,170,000
A44	Stream FB-1 Upstream of Lower Crabapple Road and Carriage Hills	Regional Detention Pond Area - 9 acres Max. Depth - 8.5 feet 100-Year Volume - 36 acre-feet Outlet 7 - 4' x 6' Box Culverts	Reduces 100-Year Storm flow 23% at the site Reduces 100-Year flood elevation by 0.5 - 1.0 feet through the Carriage Hills problem site Eliminates Llano overtopping for the 10-Year storm	$665,000
A41	Stream FB-1 Lower Crabapple Road	4 - 53" x 85" Arch CGMP 700 feet D/S Channelization	50-Year Capacity without overtopping. 100-Year Capacity with upstream detention (A44)	$110,000
			TOTAL	$1,945,000

TABLE 8-4
LONG-TERM STREAM FLOOD PROTECTION ALTERNATIVES

ALTERNATIVE DESIGNATION	LOCATION	DESCRIPTION	EFFECTIVENESS
PHASE II			
A40	Morse St. Town Creek	4 - 8' x 8' Box Culvert	100-Year Capacity without overtopping.
FUTURE			
A43	Town Creek At Cross Mountain West	Regional Detention Pond Area - 6 acres Max. Depth - 10 feet 100-Year Volume - 11 acre-feet 4 - 54" RCP Low Flow Outlet Concrete Weir Length - 94 feet	Reduces 100-Yr Storm flow 9% at the site Offsets increase from existing to fully-developed cond. Reduces the amount and frequency of overtopping at West Morse Eliminates overtopping of Milam and Crockett for the 10-year storm with Alternative A42

recommended for implementation in Phase I because it is not nearly as effective for reducing flood levels downstream along Town Creek as Alternative 42. The cost per unit flow reduction of Alternative A43 is over six times more expensive than that of the western tributary regional detention pond (Alternative A42). It also has a minimal effect on flood levels along most of Town Creek, although it does produce some significant flood level reductions in the short reach just downstream of the pond site and upstream of the confluence with the western tributary. It does not reduce the 10-year flood flow sufficiently to eliminate overtopping of Morse Road. The cost of the Alternative A43 detention pond is significantly more expensive than the cost of replacing the existing Morse Road tank car culvert with a set of concrete boxes (4 - 8' x 8') that can pass the 100-year flood flow without overtopping. Furthermore, implementation of the Cross Mountain West pond (Alternative A43) is not critical since it is not immediately needed to offset the flood flows that would be diverted into Town Creek by Alternative A13 (West Burbank Street storm drain) since Alternative A13 is not included as part of the recommended alternatives for Phase I. Therefore, this alternative is not recommended, at least for immediate implementation.

Although the Morse Road culvert replacement project (Alternative A40) is a cost-effective measure to eliminate road overtopping, it is not recommended at this time, but should be considered for implementation as part of the Phase II program.

8.2.2 Stream FB-1

The regional detention pond on Stream FB-1 upstream of Lower Crabapple Road (Alternative A44) is very effective for reducing stream flooding problems downstream through the Carriage Hills subdivision, and it is recommended for installation as part of Phase I. The culvert replacement for Lower Crabapple Road (Alternative A41) is also recommended along with the associated downstream channel improvements. The combination of these drainage improvement projects will prevent overtopping of Lower Crabapple Road for floods up to the 100-year flood event.

PLATES

FLOOD PROTECTION PLANNING STUDY
FOR THE FREDERICKSBURG AREA
Contract No. 96-483-161

The following maps are not attached to this report. Due to their size, they could not be copied. They are located in the official file and may be copied upon request.

Plate 3-1 Subwatersheds
Plate 3-2 Flood Problem Areas
Plate 5-1 Localized Flooding Problem Sites
Plate 5-2 Map Of Localized Flooding Problem Survey Locations
Plate 6-1 Map Drainage And Flood , Protection Alternatives
Plate 8-1 Map Of Drainage And Flood Protection Plan

Please contact Research and Planning Fund Grants Management Division at (512) 463-7926 for copies.

FLOOD PROTECTION PLANNING STUDY FOR THE FREDERICKSBURG AREA

TEXAS WATER DEVELOPMENT BOARD
Research and Planning Fund
TWDB Contract No. 96-483-161

August 1997

prepared for

CITY OF FREDERICKSBURG
Gillespie County, Texas

prepared by

R. J. BRANDES COMPANY
Austin, Texas

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1.0 INTRODUCTION

1.1 STUDY OVERVIEW

This regional Flood Protection Planning Study has been undertaken to provide an evaluation of existing flooding conditions and needed drainage improvements and flood control measures within the City of Fredericksburg and adjacent areas of Gillespie County. The study has focused on localized solutions to existing and projected flooding problems, as well as, regional control measures such as stormwater detention facilities. The costs associated with implementing various flood protection options for different portions of the planning area also have been examined. A flood protection and drainage improvement plan has been formulated that identifies and prioritizes the most important projects to be implemented. As part of this overall planning effort, a number of hydrologic and hydraulic analytical tools have been developed that will be useful for continuing to evaluate the effects of future development on stormwater runoff, streamflows and flooding levels throughout the City.

1.2 STUDY PARTICIPANTS

This regional Flood Protection Planning Study for the City of Fredericksburg and the surrounding area has been prepared for the City of Fredericksburg under contract to the Texas Water Development Board with funding assistance through its Research and Planning Grant program. The applicant for funding for this study and the contractor with the Texas Water Development Board has been the City of Fredericksburg. Gillespie County has served as a participating political subdivision.

1.3 STUDY BACKGROUND

The City of Fredericksburg has grown steadily during the past several decades from a population of about 4,000 in 1950 to almost 7,000 in 1990. Today, it is estimated that there are over 8,000 people living within the City, with growth in and around the City continuing at an accelerated pace. The attraction of Fredericksburg's clean, small-town setting in the Hill Country of Texas, coupled with its increasing importance as a center for tourism, has played a major role in this recent growth of the City.

With this growth in population, residential and commercial development, and redevelopment, of land within the City and the surrounding area naturally has taken place. Major residential subdivisions comprised of single-family housing have been constructed and, presently, there are over a thousand residential lots being planned for development. Extensive expansion of the downtown retail area also has occurred in response to the need for basic services and the increased interest in tourism. Commercial developments and some light manufacturing facilities also have been located around the City.

With these changes in land use to more developed and densely-populated conditions, corresponding changes in the characteristics of the watersheds that drain the City also have occurred. With more streets, parking lots and roof tops, the imperviousness of the land surface has increased, thereby causing infiltration of rainfall to be reduced and rates and volumes of stormwater runoff to be increased. Basically, today there is more stormwater generated within the City by the same amount of rainfall than there was just five or ten years ago, and the extent to which existing watercourses and drainage facilities can handle these higher amounts of runoff under the more extreme rainfall conditions has been of concern to City officials.

While there are areas within the City that have experienced some shallow water flooding and street blockage during intense rainfall events, no major flooding of entire blocks or subdivisions, with floodwaters in homes or businesses, has been experienced. However, the actual severity of past storm events with respect to normally-accepted design flood conditions and/or typical levels of regulatory flood protection is not known. Some of the larger storms possibly could cause such flooding, particularly now that a greater portion of the watersheds both within and upstream of the City have been and are being developed. Investigations of the floodwater-carrying capacity of existing watercourses and drainage facilities have been needed to establish the degree of risk associated with flooding by storm events of varying magnitudes.

The City and Gillespie County both participate in the National Flood Insurance Program (NFIP), and, as such, they both have floodplain management ordinances in effect that regulate development within the existing 100-year floodplains along the major creeks within and just outside the City's corporate boundaries. Current flood insurance rate maps for the City indicate that specific base flood elevation information and the associated floodplain boundary delineations have been determined for portions of Barons Creek, Town Creek and an unnamed tributary of Barons Creek located in the extreme northeastern part of the City referred to as Stream FB-1. The flood related information shown on currently-effective flood insurance rate maps for the City are based on studies conducted by the Flood Insurance Administration (FIA), now the

Federal Emergency Management Agency (FEMA), during the late 1970's, and they have not been updated since. Basically, the current floodplain maps for the City reflect watershed and creek channel conditions as they existed almost twenty years ago. It is important that any effort to examine current and future flooding conditions within the City as affected by recent and ongoing development should include a review and reevaluation of flood levels and floodplains along the major creeks through the City as originally studied by FIA. If conditions have changed significantly or if conditions are expected to change due to continued land development and/or proposed drainage and flood control improvements, it is important that revised floodplain boundary maps and associated documents be prepared and submitted to FEMA so that the existing flood insurance maps can be updated and republished.

As flooding problems are identified, improvements in the existing watercourses and drainage facilities may be warranted in order to provide an acceptable level of flood protection for City residents and visitors and properties within the City. Such improvements may consist of widening and deepening of existing watercourses and channels within and downstream of developed areas, installing new drainageways, pipes or conduits to convey excess stormwater from the City's streets to the major creeks, and/or constructing runoff detention pond systems to reduce stormwater flow rates. It is important to determine now the extent to which such drainage improvements and flood control measures need to be implemented, and what it will cost, so that City officials can effectively evaluate if, how and when such projects might be incorporated into the Capital Improvements Program.

Future development within the City's jurisdiction also needs to take place so as not to exacerbate any existing flooding problems or to cause the design floodwater-carrying capacity of existing and/or improved watercourses and drainage facilities to be exceeded. One way to accomplish this is for the City to decide to limit the rates of runoff from the watersheds that drain to and through the City to present levels so that the existing floodwater conveyance system does not have to be expanded in order to handle the higher stormwater flows associated with increased development. Such a stormwater detention program could be implemented either by the City undertaking the construction of major regional runoff detention facilities and allocating the costs among those that benefit and/or new development projects, or by the City adopting ordinances requiring all new development projects to install appropriate onsite runoff detention ponds. It is important for these options, and others for controlling future stormwater runoff, to be examined and evaluated now so that informed decisions can be made.

Finally, it is important that any new stormwater conveyance facilities or related drainage systems be uniformly designed and sized in accordance with accepted engineering practice and design standards. The City needs to adopt a set of drainage design criteria, with which all new drainage facilities and development projects must comply. Such criteria need to be relatively straightforward and easy to check with regard to compliance by City staff doing project reviews. Such drainage design criteria manuals have been developed by other small communities like Fredericksburg and are being used as a means to effectively assure that new drainage facilities are adequately sized and properly designed and constructed.

1.4 PLANNING AREA

The planning area for this Flood Protection Planning Study encompasses all of the Barons Creek watershed, extending from its mouth at the Pedernales River northwestward through the City of Fredericksburg to its headwaters, a distance of about fourteen miles. This watershed, which also includes Town Creek and a major unnamed tributary referred to as Stream FB-1, covers about 33 square miles and drains practically all of the City of Fredericksburg. A small portion of the southwestern part of the City in the vicinity of the High School lies outside of this watershed and drains directly to the Pedernales River. This outside area, which encompasses about one square mile, also is included in the planning area. All of the planning area is within Gillespie County. The map of Gillespie County in Figure 1-1 shows the boundaries of the planning area for this Flood Protection Planning Study.

The planning area for this Flood Protection Planning Study has been delineated based primarily on drainage area boundaries, particularly for the watershed that drains the vast majority of the City of Fredericksburg. This is the area of concern with regard to existing and future drainage and flooding problems and the potential impacts of new development on existing drainage and flooding conditions. The entire planning area is within the watershed of the Pedernales River. The Pedernales River is a tributary of the Colorado River, which flows directly into the Gulf of Mexico.

The City and Gillespie County have jurisdiction over the entire planning area with regard to drainage and flood control issues.

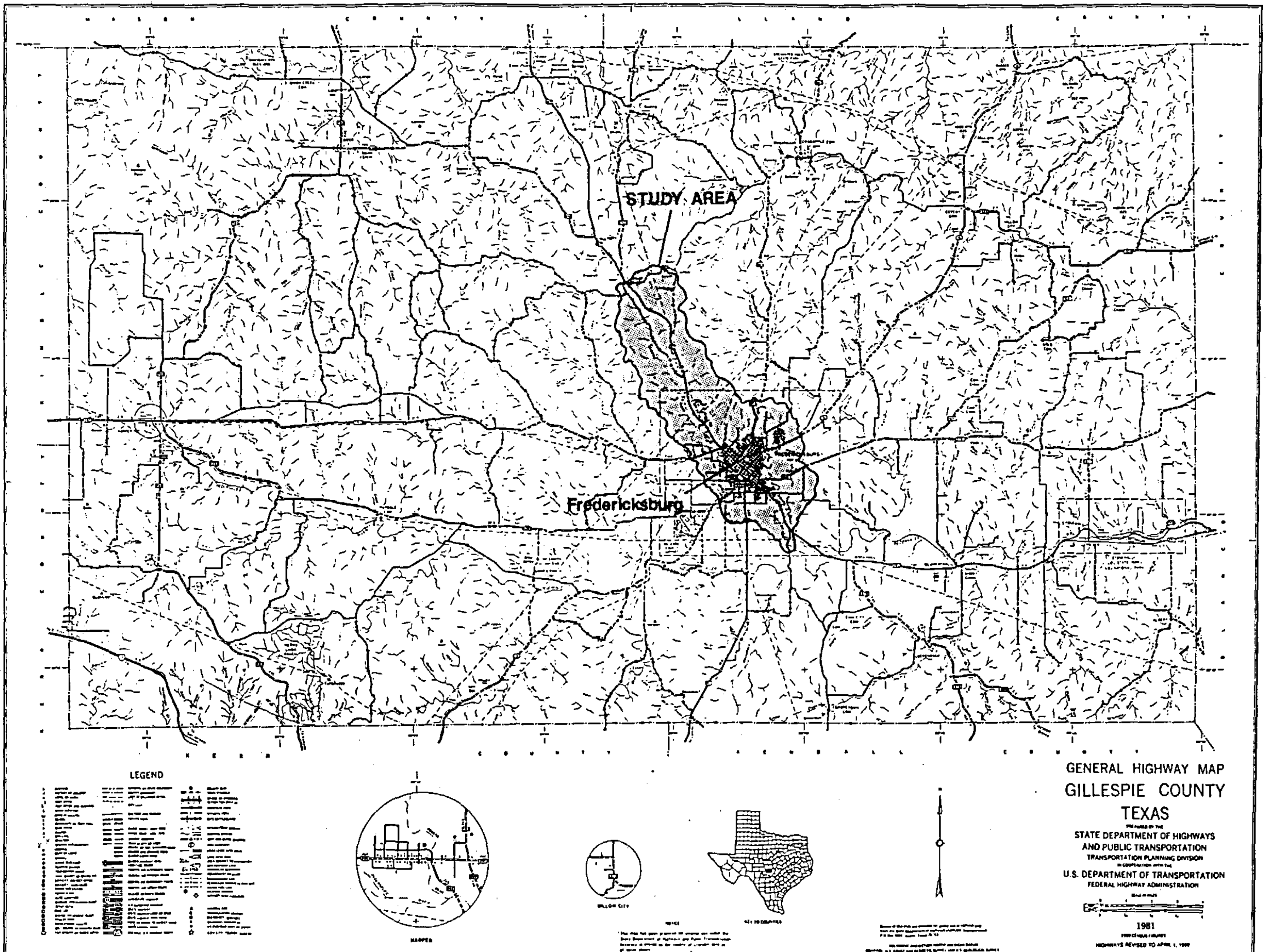

FIGURE 1-1 FLOOD PROTECTION PLANNING AREA

2.0 DATA AND INFORMATION

2.1 EXISTING SOURCES

Considerable data and information have been compiled and analyzed for purposes of this Flood Protection Planning Study. Much of this data and information has been obtained from existing sources. Following is a list of the various items that have been assembled from existing sources and used in this study.

- Topographic maps of the planning area (1"=2,000', 10' contours) as published by the U. S. Geological Survey.
- Topographic maps of the planning area (1"=800', 5' contours) and the associated aerial photography as provided by the Engineering Department of the City of Fredericksburg.
- Roadway and stream maps of Gillespie County as published by the Texas Department of Transportation.
- Street and stream maps of the planning area (1"=800') from the Engineering Department of the City of Fredericksburg.
- 1994 aerial photographs of the Fredericksburg area from the Engineering Department of the City of Fredericksburg as provided by the Gillespie County Tax Assessor/Collector's Office.
- Existing land use map (May 2, 1996) from the Comprehensive Plan for the City of Fredericksburg as prepared by Hankamer Consulting.
- Future land use map (May 15, 1996) from the Comprehensive Plan for the City of Fredericksburg as prepared by Hankamer Consulting.
- "City of Fredericksburg, Texas Comprehensive Plan '96"; prepared for the City of Fredericksburg by Hankamer Consulting; Austin, Texas; November, 1996.
- "Fredericksburg Comprehensive Plan, 1985"; prepared for the City of Fredericksburg by Bovay Engineers; 1985.
- Current zoning map (1996) from the Engineering Department of the City of Fredericksburg.
- "Storm Drainage System Study for North Sector"; prepared for the City of Fredericksburg by Hogan & Rasor, Inc.; Austin, Texas; March, 1982.

- Effective Flood Insurance Maps (May 19, 1991) and Flood Insurance Study (November 19, 1980) for the City of Fredericksburg.
- Flood Insurance Work Maps for the City of Fredericksburg (1980).
- HEC-2 Backwater Models for Barons Creek Town Creek and Stream FB-1 corresponding to the Effective Flood Insurance Maps (May 19, 1991) for the City of Fredericksburg.
- Revised Flood Insurance Maps and and supporting documentation for Letter of Map Revision (February 7, 1995) for a 60-acre tract in the southwest part of the City of Fredericksburg.
- Effective Flood Insurance Maps (May 10, 1977) for Gillespie County.
- "Flood Insurance Study Guidelines and Specifications for Study Contractors"; FEMA 37; Federal Emergency Management Agency; Washington, D. C.; January, 1995.
- Article 3.700, Flood Damage Prevention, of Chapter 3: Building and Construction of the City of Fredericksburg's Code of Ordinances.
- Chapter 9: Subdivisions of the 1996 Subdivision Ordinance of the City of Fredericksburg's Code of Ordinances.
- Subdivision Ordinance for City of Fredericksburg; April, 1984 Edition; Chapter 19.
- Article 11.800, Drainage Utility, of the City of Fredericksburg's Code of Ordinances.
- Zoning Ordinance for City of Fredericksburg; November, 1991 Edition; and Revisions dated 10/26/92, 1/10/94, and 8/22/94.
- Preliminary drainage plans, analyses, and calculations for proposed Stone Ridge Subdivision.
- Preliminary drainage plans, analyses, and calculations for proposed Cross Mountain Subdivision.

- Preliminary drainage plans, analyses, and calculations for proposed Heritage Park Subdivision.

- Preliminary drainage plans, analyses, and calculations for proposed Highland Oaks Apartments

- "Report on Heritage Park Development, A Residential Development in Fredericksburg, Texas"; Grape Creek Ranch Family Ltd. Partnership.

2.2 FIELD SURVEYS

To obtain site specific information regarding ground topography, channel geometry, and drainage facilities features, field surveys were performed at numerous sites throughout the planning area. Field surveys were preformed to provide information on potential localized flooding problems, as well as, major stream channels. A preliminary identification of problem areas first was made by reviewing existing topographic maps (scale: 1" = 800' and five-foot contours) and visiting locations identified as problem areas by City personnel and through citizen complaints. Key features of the potential problem areas were surveyed or measured as necessary for further analysis of hydraulic conditions. Surveyed or measured features included curb heights, roadway widths and crown elevations, distances to and elevations of nearby structures, culvert sizes and flowline elevations, and swale and channel section geometry. The field surveying also included verification of drainage subarea boundaries and flow paths needed to calculate runoff to the potential localized problem areas.

Presented in Table 2-1 is a listing of all of the sites where field surveying has been performed during this study and a general description of the types of information obtained. Work maps are available that indicate the specific location of each of these survey sites.

2.3 GILLESPIE COUNTY FLOOD INSURANCE STUDY

During the course of this Flood Protection Planning Study, the Fort Worth District Office of the U. S. Army Corps of Engineers (Corps) initiated a study of portions of Gillespie County pursuant to the National Flood Insurance Program. Under contract to the Federal Emergency Management Agency (FEMA), the Corps has performed hydraulic analyses, including HEC-2 backwater modeling, of all or parts of several creeks and

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
BARONS CREEK	
Fort Martin Scott and 290	Surveyed new x-section between COE Sections A and 119+00
Wastewater Treatment Plant	Surveyed six sections upstream of WWTP discharge point.
Goehmann Road	Surveyed new x-section between COE Sections B and Goehmann Road
F.M. 1631	Surveyed two new x-sections between Goehmann Road and F.M. 1631 at approximately 190+00 and 210+00
Creek Street	Surveyed four sections at low water crossing.
Creek St. at W. Elk St.	Surveyed new x-section across creek, as if extended from Creek St., at approximately 282+00.
Creek Street	Surveyed across channel from filled area on left bank at approximately 283+50
Washington St.	Surveyed new x-section upstream of Washington St. at fill site on right bank at approximately 296+40.
Prop. Walk Bridge at Llano St.	Surveyed creek at site of proposed walk bridge at end of Llano St.
Walk Bridge at Orange St.	Surveyed newly constructed walk bridge at Orange St.
S. Bowie Street	Surveyed new x-section across creek, as if extended from S. Bowie St. at approximate station 362+75
Peach Street	Surveyed new x-section across creek from Peach St. at approximately 376+00

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
TOWN CREEK	
Elk Street	Surveyed old low water crossing under TxDOT bridge.
Lincoln and Schubert	Surveyed five sections where channel improvements had been made.
Crockett Street	Surveyed new pond structure downstream of Crockett, and associated channel improvements Surveyed CMP's under Crockett, including all wingwalls upstream and downstream Surveyed road profile along Crockett Street.
Orange Street	Surveyed all culverts under Orange Street. Surveyed edge of concrete on upstream and downstream face of Crockett. Surveyed road profile along Orange Street.
D/S Cherry & Morse St.	Surveyed section 50 feet downstream of confluence of tributaries from Cherry St. and Morse St.
N. Cherry Street	Surveyed section 85 feet downstream of Cherry St. culvert. Surveyed section 53 feet downstream of Cherry St. culvert. Surveyed old tank car "culvert" under roadway. Surveyed road profile along N. Cherry Street.
W. Morse Street	Surveyed old tank car "culvert" under roadway. Surveyed road profile along W. Morse Street.
Town Creek Extensions	Surveyed section 16 feet upstream of Morse St. culvert headwall. Surveyed section 400 feet upstream of Morse St. culvert headwall. Surveyed section 900 feet upstream of Morse St. culvert headwall. Surveyed section 1360 feet upstream of Morse St. culvert headwall.

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
DRY CREEK TRIBUTARY	
Bob Moritz Drive	Surveyed low water crossing at Bob Moritz Drive. This is the downstream crossing, approximately 2,500 feet from the U.S. 87 / U.S. 290 intersection.
Gold Road	Measured road crossings, culverts and channel at Gold Road
U.S. 87	Measured culverts at Dry Creek and U.S. 87. Field checked TxDOT design plans. Measured culverts at U.S. 87. Field checked TxDOT design plans.
TRIBUTARY FTB-1	
F.M. 1631	Surveyed new x-sections approximately 60 and 600 feet upstream of low water crossing upstream of confluence near F.M. 1631
Tanglewood Dr.	Surveyed new x-section at approximately 125+00 to pick up effects of wooden retaining walls.
Ridgewood and Glenwood	Surveyed new x-section at approximately 134+10 at site of old bridge.
Briarwood Circle	Surveyed new x-section at approximately 157+00

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
STONERIDGE TRIBUTARY	
D/S Ridgewood St.	Surveyed section upstream of confluence with FTB-1. Surveyed section 35 feet downstream of Ridgewood.
Ridgewood St.	Surveyed road profile and culverts under Ridgewood. Surveyed section 20 feet upstream of culverts. Surveyed section 100 feet upstream of culverts.
FRIENDSHIP LANE	
South Creek Subdivision near Dow Street	Measured channel section.
Friendship Lane	Surveyed road and swales near South Creek Street. Surveyed road and swales downstream of Washington.
Washington	Measured box culvert.
Friendship Lane	Surveyed road and swales upstream of Washington. Surveyed road and swales downstream of channel.
Channel	Surveyed channel 50 feet upstream of Friendship Lane. Surveyed channel 100 feet upstream of Friendship Lane. Surveyed channel 150 feet upstream of Friendship Lane.
South Adams Street	Measured curb and curb cut.

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
CROSS MOUNTAIN-MILAM	
North Milam	Surveyed section @ 604 Milam. Surveyed section @ 705 Milam.
Pecan Street	Measured section downstream of West College Street.
West College Street	Surveyed section between Pecan and Edison.
Edison Street	Surveyed section upstream of West College. Surveyed section downstream of Centre.
Centre Street	Surveyed section just east of Edison. Surveyed section just west of Pecan. Surveyed section just east of Pecan.
Channel south of Burbank	Measured channel downstream of Burbank near Avenue A.
Burbank	Measured curb cut on Burbank near Avenue A.
Avenue D	Surveyed and measured channel at end of Burbank.
Cross Mountain	Surveyed and measured channel at intersection with Avenue D. Surveyed street section.
North Milam	Measured swales and culverts @ Broadmoor.

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
BURBANK-LLANO	
Llano	Surveyed street section @ 905 Llano.
Burbank	Measured street section just west of Llano.
NORTH LINCOLN	
North Lincoln	Surveyed section between Centre and College.
COLLEGE-LLANO	
College	Surveyed street section downstream (east) of Llano.

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
COLLEGE-TRAVIS	
Sycamore	Surveyed street section south of College.
Washington	Surveyed channel upstream of culverts. Surveyed culvert flowlines.
Orchard	Surveyed channel section downsteam of Orchard.
North Pine	Surveyed street section.
East Travis	Surveyed street section at 414 E. Travis. Surveyed street section between Elk St. intersections. Measured channel and street section west of N. Lee St.
North Lee	Measured culvert at City Cemetery. Surveyed channel downstream of N. Lee in City Cemetery.
TRAILMOOR	
Trailmoor	Measured street section. Measured culvert inlet configuration.
Morning Glory	Measured culvert.

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
MORNING GLORY-LLANO	
Llano	Measured box culverts.
Lower Crabapple	Measured channel section next to road.
Morning Glory	Measured box culverts.
CARRIAGE HILLS	
Edgewood	Surveyed channel section west side. Surveyed channel section east side.
Driftwood	Surveyed street section @ 206 Driftwood. Surveyed street section @ 204 Driftwood. Surveyed intersection @ Ridgewood. Surveyed street section @ 114 Driftwood. Surveyed street section @ 112 Driftwood.
North Adams	Surveyed street section east of Driftwood. Surveyed street section just west of Crestwood. Surveyed channel south of Adams.
Frederick	Measured channel section.
Tanglewood	Measured channel section.

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
WEST CREEK STREET	
South Bowie	Field located flat street section.
West San Antonio	Field located flat street section.
OLD HARPER POND	
Armory Road	Field verified low water crossing.
Basse Lane	Field verified low water crossing. Measured swale along road. Measured culvert under Duderstadt (Private Drive)
Duderstadt	Measured swale along road.
South Bowie Street	Measured box culvert.
WINFRIED CREEK	
South Milam	Measured bridge.
Post Oak Blvd.	Measured bridge/culvert. Measured culvert.
Smith Road	Measured culvert.
Live Oak	Measured culvert and upstream wall. Measured culvert.

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
FIVE POINTS	
South Lincoln	Surveyed street and swale section 200 feet north of intersection. Surveyed street and swale section 300 feet north of intersection.
East Live Oak	Surveyed street and swale section 75 feet east of intersection. Surveyed street and swale section 200 feet east of intersection. Surveyed street and swale section 300 feet east of intersection.
South Live Oak & Park St.	Surveyed street and swale section 100 feet west of intersection. Surveyed street and swale section 200 feet west of intersection.
Five Points Intersection	Surveyed flowlines of culverts and measured culvert sites.
East Ufer Street	Measured culvert just north of street (downstream).
East Live Oak	Field verified swale location and condition near Granite Avenue.
Granite Avenue	Measured inlet and culvert sizes @ Granite near Ufer.
SOUTH ADAMS	
Friendship Lane	Measured culverts. Measured channel sections.

TABLE 2-1

INVENTORY OF FIELD SURVEYING SITES

WATERSHED STREET LOCATION	SURVEY DESCRIPTION
HIGHWAY-APPLE	
Highway St.	Field verified flow paths.
Apple St.	Field verified flow paths.
South Eagle Street	Field verified low water crossing.
Crenwelge	Measured channel section upstream of Crenwelge culverts. Measured culverts. Estimated channel section downstream of Crenwelge culverts.

streams in the immediate vicinity of the City of Fredericksburg, and now has prepared work maps showing either newly established or revised floodplain boundaries and flood elevations for the 100-year and 500-year floods. Some of the watercourses studied by the Corps are extensions of stream segments that lie within the City of Fredericksburg and, consequently, relate to the flooding analyses performed in this Flood Protection Planning Study. For this reason, portions of this Flood Protection Planning Study have been undertaken within a timeframe that has allowed results from the Corps' Gillespie County investigations to be fully utilized and incorporated. In the early stages of the Corps' Gillespie County flood insurance studies, it was agreed that results from this Flood Protection Planning Study relating to flood flows for the various creeks and streams in the planning area would be provided to the Corps in exchange for hydraulic results and HEC-2 models for the various stream segments analyzed by the Corps. In addition, arrangements also were made to purchase certain detailed and digitized topographic information from the Corps for specific stream reaches within the planning area.

The specific stream segments for which HEC-2 backwater models have been developed by the Corps pursuant to its Gillespie County flood insurance studies and provided to this Flood Protection Planning Study are identified on the map of the Fredericksburg area in Figure 2-1. Basically, the Corps developed HEC-2 models for a portion of Barons Creek extending from near the City's wastewater treatment plant south of downtown upstream to the U. S. Highway 290 bridge and for all of Stream FB-1 from its confluence with Barons Creek upstream to above Lower Crabapple Road. Except for a reach of Stream FB-1 within the Carriage Hills subdivision in the northwestern portion of the City, all of the stream segments modeled by the Corps lie outside the corporate boundaries of the City.

In developing its HEC-2 backwater models, the Corps utilized digitized topographic information to establish channel cross-section geometry. The Corps also made field surveys to obtain dimensions and flowline elevations for bridges and culverts along each of the modeled stream segments. The peak flood flows used by the Corps for specific flood events were agreed upon through discussions with FEMA representatives after hydrologic results from this Flood Protection Planning Study were available for Barons Creek and Stream FB-1. In essence, it was determined that peak flood flows for streams within and in the vicinity of the City under current land use and watershed conditions are not appreciably different from those flows used in the previous flood

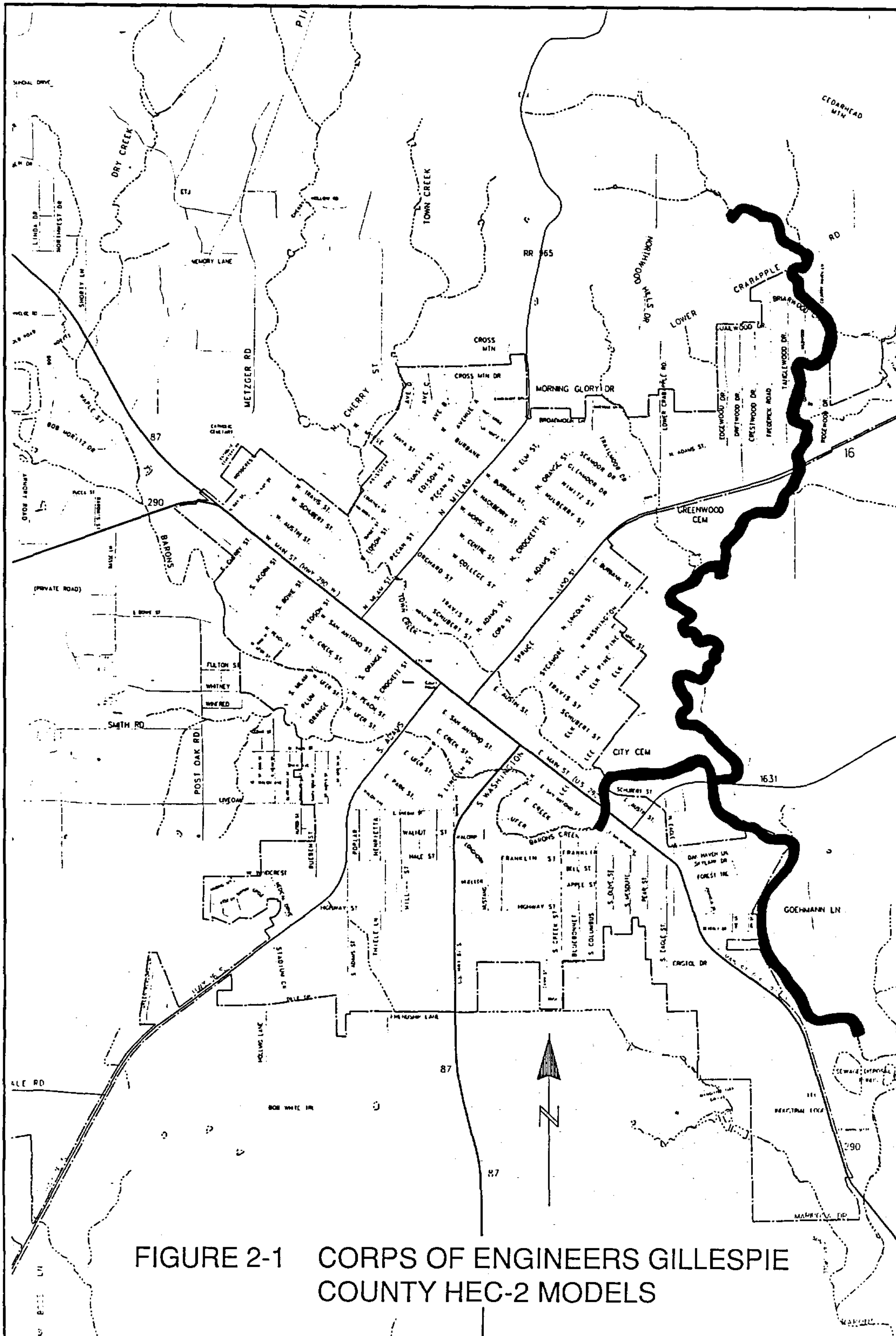

FIGURE 2-1 CORPS OF ENGINEERS GILLESPIE COUNTY HEC-2 MODELS

insurance study for the City that form the basis for the currently-effective flood insurance maps. Hence, in accordance with FEMA's general guidelines for conducting flood insurance studies, it was agreed that the original peak flood flows used in the effective flood insurance study would be utilized by the Corps in its Gillespie County flood insurance studies and also in this Flood Protection Planning Study for the City.

3.0 FLOOD FLOW CONDITIONS

3.1 PREVIOUS FLOOD INSURANCE STUDY

In 1980, Albert H. Halff & Associates completed the Flood Insurance Study (FIS) that provides the basis for the current floodplain boundaries and flood elevations indicated on the effective flood insurance maps of the City of Fredericksburg, which are dated May 19, 1981. As part of this previous investigation, peak flood flows for various creeks and streams within the planning area for this Flood Protection Planning Study were determined for the 10-, 50-, 100- and 500-year flood events. Since the quantities of flood flows occurring at different locations on the creeks and streams within the planning area are fundamental to this analysis of flooding problems and, more importantly, to the development of effective solution measures, the FIS flood flows have been examined and evaluated with respect to corresponding results from this study. Requests were made to the Federal Emergency Management Agency (FEMA) for the original FIS flood flows and backwater models, and these materials were provided.

The specific stream reaches for which hydrologic and hydraulic analyses were performed during the previous FIS for the City of Fredericksburg are identified on the map of the Fredericksburg area in Figure 3-1. Basically, these include portions of Barons Creek, Town Creek and Stream FB-1 in the vicinity of the City. In 1995, a formal Letter of Map Revision (LOMR) was approved by FEMA at the request of the City. This LOMR added a portion of another tributary of Barons Creek, referred to as Stream FB-2, to the effective flood insurance maps for the City. Stream FB-2 enters Barons Creek in the extreme southern portion of the City near U. S. Highway 290.

The peak flood flows from the previous FIS and LOMR for the City are summarized in Table 3-1. Values for the 10-, 50-, 100- and 500-year flood events are presented at several locations along each of the streams included on the effective flood insurance maps for the City. These flood flows will be referred to later in this report.

3.2 HEC-1 HYDROLOGIC ANALYSES

For purposes of examining existing flooding problems and evaluating the effectiveness of alternative flood control and drainage improvement measures in this Flood Protection Planning Study, it has been necessary to develop a computer simulation model capable of describing the hydrologic behavior and response of the several watersheds that encompass the City and the planning area. For this model, the U. S. Army Corps of Engineers HEC-1 Flood Hydrograph Package (September 1990) has

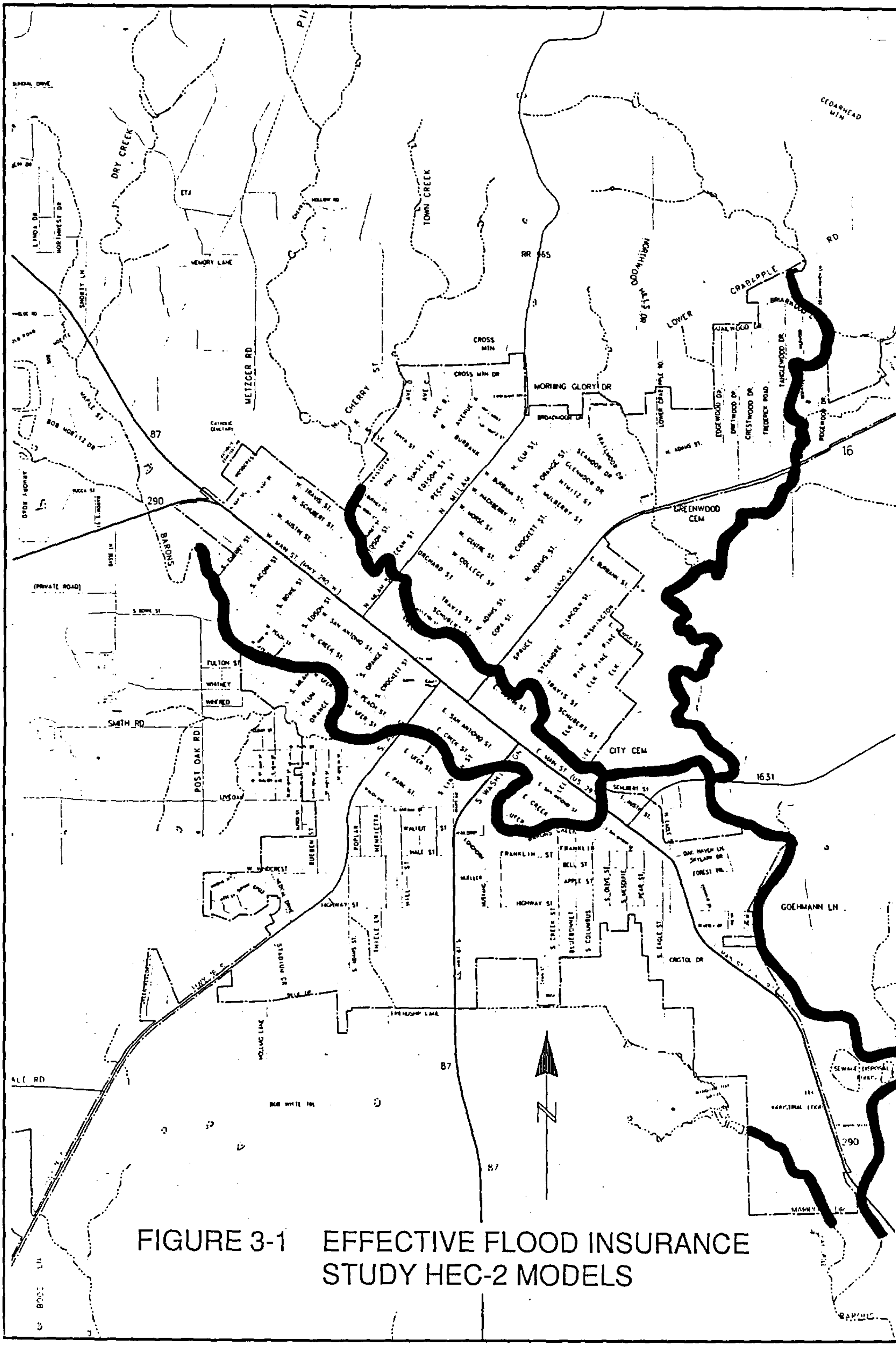

FIGURE 3-1 EFFECTIVE FLOOD INSURANCE STUDY HEC-2 MODELS

TABLE 3-1
EFFECTIVE FIS PEAK FLOOD FLOWS

SITE / CROSSING	10-YEAR FLOOD FLOW cfs	50-YEAR FLOOD FLOW cfs	100-YEAR FLOOD FLOW cfs	500-YEAR FLOOD FLOW cfs
BARONS CREEK				
S. Bowie Street	5,440	9,550	11,800	18,000
S. Adams Street	5,630	9,760	12,000	18,000
S. Llano Street	5,790	9,920	12,100	18,000
Washington Street	5,860	10,100	12,300	18,200
Upstream of Town Creek Confluence	6,690	10,900	13,200	19,000
FM 1631 Upstream Stream FB-1 Confl.	7,540	12,300	14,900	21,000
FM 1631 Downstream Stream FB-1 Confl.	8,250	13,700	16,600	24,000
U/S Wastewater Treatment Plant	8,590	14,200	17,100	24,600
D/S Wastewater Treatment Plant	9,070	14,800	17,900	25,500
Confluence with Stream FB-2	8,840	14,600	17,600	25,500
TOWN CREEK				
Confl. below N. Cherry St. and W. Morse St	1,490	2,620	3,240	4,900
N. Milam Street	1,840	3,090	3,800	5,650
N. Adams Street	1,960	3,270	4,000	5,870
N. Washington Street	2,040	3,370	4,120	5,950
Immediately U/S Confl. with Barons Creek	2,080	3,410	4,160	6,000
STREAM FB-1				
Lower Crabapple Road	860	1,540	1,930	2,950
N. Llano Street	1,520	2,590	3,190	4,680
Carriage Hills Runoff and Stream FB-1	1,990	3,400	4,230	6,300
Immediately D/S Cemetery	2,530	4,310	5,350	7,900
Immediately U/S Confl. with Barons Creek	2,270	3,790	4,650	6,900
STREAM FB-2				
Stock Pond at Camp	1,210	2,022	2,446	4,158
Immediately U/S Confl. with Barons Creek	1,210	2,022	2,446	4,158

been utilized and applied to the various watersheds draining to Barons Creek, Town Creek and Stream FB-1, down to the confluence with the Pedernales River south of the City of Fredericksburg. As stated in the HEC-1 User's Manual,

> *The HEC-1 model is designed to simulate the surface runoff response of a river basin to precipitation by representing the basin as an interconnected system of hydrologic and hydraulic components. Each component models an aspect of the precipitation-runoff process within a portion of the basin, commonly referred to as a subbasin. A component may represent a surface runoff entity, a stream channel, or a reservoir. Representation of a component requires a set of parameters which specify the particular characteristics of the component and mathematical relations which describe the physical processes. The result of the modeling process is the computation of streamflow hydrographs at desired locations in the river basin.*

3.2.1 HEC-1 Model Application

For applying the HEC-1 model to the Barons Creek system, the entire 33-square mile watershed has been divided into forty-one subbasins, or subwatersheds, with each corresponding to a smaller creek or group of creeks, to a change in watershed runoff conditions, and/or to a potential site for a flood control facility such as a detention pond. The boundaries of the model subwatersheds have been determined by examining the hydrologic features depicted on U. S. Geological Survey topographic maps of the region. These boundaries are delineated on the map of the Barons Creek watershed in Plate 3-1. They also are listed in Table 3-2 along with their respective drainage areas. As indicated, most of the subareas in the vicinity of the City are smaller in size than a few hundred acres. The largest subwatershed in the model, Subwatershed BC-12, covers about 13.8 square miles in the extreme upper portion of the Barons Creek watershed that is predominantly undeveloped and expected to remain so in the foreseeable future.

In the process of developing the HEC-1 model of the Barons Creek watershed, a number of different hydrologic parameters that are required for the runoff calculations have been determined. This includes the time of concentration for each of the subwatersheds. The time of concentration is defined as the average time it takes for a particle of water (stormwater runoff) to travel from the farthest upstream point of a subwatershed down to the point of discharge from the subwatershed. This route

TABLE 3-2
HYDROLOGIC PARAMETERS FOR HEC-1 MODEL SUBWATERSHEDS

WATERSHED SUBAREA ID	DRAINAGE AREA ACRES	SQ MILES	TIME OF CONCENTRATION MINUTES	SCS LAG TIME HOURS	ROUTING TIME HOURS	SCS CURVE NUMBERS EXISTING	FUTURE
BARONS CREEK							
BC 01	274.7	0.429	65	0.653	-	77	82
BC 02	338.8	0.529	71	0.709	0.282	76	77
BC 03	294.7	0.460	88	0.881	0.226	82	86
BC 04	392.8	0.614	28	0.284	0.170	82	84
BC 05	456.9	0.714	63	0.632	0.139	75	76
BC 06	159.4	0.249	18	0.184	0.111	82	89
BC 07	310.3	0.485	39	0.393	0.190	80	86
BC 08	175.1	0.274	51	0.510	0.132	79	80
BC 09	354.5	0.554	28	0.284	0.159	80	82
BC 10	287.5	0.449	63	0.628	0.233	81	89
BC 11	1,016.3	1.588	58	0.577	0.167	84	84
BC 12	8,840.3	13.813	170	1.697	0.289	87	87
TOWN CREEK							
TC 01	239.1	0.374	94	0.943	-	83	85
TC 02	330.2	0.516	61	0.612	0.217	77	79
TC 03	346.3	0.541	33	0.326	0.072	84	84
TC 04	327.0	0.511	33	0.332	0.317	86	90
TC 05A	430.6	0.673	47	0.473	0.133	85	83
TC 05B	111.4	0.174	20	0.203	0.178	79	73
STREAM FB-1							
FB1-1	520.7	0.814	45	0.454	-	70	72
FB1-2	269.4	0.421	54	0.536	0.257	73	80
FB1-3	190.7	0.298	31	0.312	0.300	85	85
FB1-4	312.2	0.488	38	0.385	-	69	72
FB1-5A	119.4	0.187	20	0.197	0.409	82	84
FB1-5B	55.1	0.086	46	0.459	0.128	74	75
FB1-6	206.6	0.323	29	0.288	0.084	75	77
FB1-7	697.0	1.089	46	0.462	0.158	83	85
FB1-8	207.5	0.324	27	0.268	0.063	72	76
FB1-9	39.5	0.062	33	0.325	0.168	68	70
BARONS CREEK TRIBUTARIES							
BCT-1A	274.6	0.429	59	0.594	-	74	80
BCT-1B	517.9	0.809	77	0.771	0.209	80	83
BCT-1C	119.7	0.187	51	0.510	0.189	77	75
BCT-1D	175.1	0.274	55	0.546	0.106	74	87
BCT-1E	59.5	0.093	30	0.295	0.173	82	86
BCT-2	387.1	0.605	46	0.460	-	75	76
BCT-3	499.6	0.781	56	0.560	0.183	64	68
BCT-4	193.8	0.303	22	0.219	0.239	82	82
BCT-5	552.2	0.863	36	0.356	-	76	80
BCT-6	172.3	0.269	47	0.467	-	76	83
BCT-7	276.1	0.431	41	0.410	-	86	87
DRY CREEK							
DC-1	662.7	1.036	54	0.545	0.200	87	87
DC-2	98.7	0.154	31	0.308	0.222	84	84

TOTAL AREA OF WATERSHEC 33.271

typically includes some overland sheet flow in the upper reaches of a subwatershed, some shallow concentrated flow through small drainageways, and, finally, some channelized or conduit (pipe) flow through the lower reaches of the subwatershed. For describing the travel times through these different types of flow conditions, standard methods and procedures developed by the U. S. Soil Conservation Service (SCS) have been employed. These methods apply to both undeveloped areas without significant drainage improvements and developed areas where stormwater runoff may sheet flow across a parking lot, flow down a paved street, or be conveyed in a storm drain or concrete lined channel. The procedures that have been applied are described in the SCS Technical Release No. TR-55, Urban Hydrology for Small Watersheds (1986). The resulting times of concentration for each of the subwatersheds corresponding to existing land use and development conditions are summarized in Table 3-2. For future land use and development conditions, the times of concentrations have been reduced by 20 percent to reflect the effects of increased imperviousness of the land surface and future drainage improvements. Other hydrologic parameters such as the SCS lag time and the channel routing time for each subwatershed also are listed in the table. These parameters are required specifically by the HEC-1 model for simulating runoff hydrographs in response to specified rainfall events.

Another parameter that plays a key role in determining how much rainfall on a given area actually flows from the land surface as runoff, as opposed to infiltrating or being lost to evapotranspiration, is referred to as the SCS curve number. The curve number is a numerical quantity ranging between zero and 100 that describes the relative amount of runoff produced by a specified amount of rainfall on a particular type of watershed. A value of 100 reflects complete imperviousness, meaning that all rainfall occurs as runoff. Generalized values of curve numbers have been established by the SCS that relate to specific types of soils, vegetative cover, land use and surface imperviousness. These relationships are summarized in various tables and graphs that also are contained in the SCS Technical Release No. TR-55.

For purposes determining curve numbers for this Flood Protection Planning Study, the hydrologic condition of the land surface of each of the subwatersheds included in the HEC-1 model of the Barons Creek basin has been examined and characterized in terms of the relative areas of the different types of soils, vegetative cover, land use and surface imperviousness. These analyses have been undertaken for both existing land use conditions and future land use conditions, and the corresponding curve number calculations have been performed and summarized in spreadsheets similar to that shown in Table 3-3. For describing the hydrologic characteristics of the soils within the basin, the hydrologic group classifications (A, B, C or D) presented in the SCS Soil Survey of Gillespie County, Texas (1975) have been used. For vegetative cover and land use characteristics within each of the subwatersheds, 1994 aerial photographs of the planning area have been examined. The land use maps depicting existing and future conditions that have been recently prepared as part of the City's Comprehensive Plan '96 have been used to establish land use acreages for each of the subwatersheds in the HEC-1 model. To relate the land use types delineated on the City's land use maps to specific curve number values established by the SCS, the assignments summarized in Table 3-4 have been used for existing land use conditions and those in Table 3-5 have been used for future land use conditions.

The resulting curve number values that have been determined for each of the subwatersheds in the HEC-1 model are listed in Table 3-2. Values for both existing and future land use conditions are presented.

3.2.2 Rainfall Statistics

Because of the enormous expense often involved in providing fail-safe protection from flooding with guaranteed certainty, it is common practice to design and construct flood control and drainage facilities with some acceptable risk of failure incorporated into their operating capacities. For example, the National Flood Insurance Program that is administered by the Federal Emergency Management Agency uses the 100-year flood event as the standard for which an acceptable degree of flood protection is to be provided along streams and rivers. For some types of flood control works such as levees where failure could mean catastrophic losses of life and property, higher standards often are used as the basis for design. For example, many levee designs, particularly with regard to height, are based on the probable maximum flood. For other drainage facilities such as roadway culverts and storm drains, flood flows exceeding their design capacities might be considered more of an inconvenience, rather than a life-threatening occurrence with significant flood damages. For these types of facilities, designs often are based on smaller, more frequent storm events such as the 10-year or the 25-year flood.

Because of the wide range of failure risks inherent in the design standards for drainage

TABLE 3-3
EXAMPLE SOIL CONSERVATION SERVICE CURVE NUMBER CALCULATIONS

PROJECT: FREDERICKSBURG MASTER DRAINAGE STUDY
EXISTING CONDITIONS
DATE: 12/23/96

SUBAREA ID: TC-2
BY: WRY

LAND USE DESCRIPTION	FRACTION		SOIL TYPE				SCS CN	FRACTION SCS CN
			A	B	C	D		
Single Family		% AREA	0.30	0.20	0.50	0.00		
	0.57	CN	61	75	83	87	74.8	42.6
Vacant Undeveloped		% AREA	0.00	0.70	0.30	0.00		
	0.29	CN	68	79	86	89	81.1	23.5
Public/Institutional		% AREA	0.00	0.50	0.50	0.00		
cemetery, park grass cover 5(	0.01	CN	49	69	79	84	74.0	0.7
Commercial/Retail		% AREA	0.40	0.20	0.40	0.00		
	0.02	CN	89	92	94	95	91.6	1.8
Manufactured Home		% AREA	0.00	0.50	0.50	0.00		
	0.02	CN	77	85	90	92	87.5	1.8
Agricultural		% AREA	0.50	0.50	0.00	0.00		
	0.02	CN	49	69	79	84	59.0	1.2
Vacant Developed		% AREA	0.10	0.40	0.50	0.00		
	0.07	CN	49	69	79	84	72.0	5.0

TOTAL OF PRODUCT = 76.7

USE CN = 77

TABLE 3-4
GENERALIZED LAND USE AND CURVE NUMBER ASSIGNMENTS FOR EXISTING CONDITIONS WATERSHED

HANKAMER CONSULTING LAND USE	SCS TR-55 CORRESPONDING LAND USE	SCS CURVE NO. A	B	C	D
RESIDENTIAL	RESIDENTIAL				
Single Family	1/4 acre	61	75	83	87
Duplex	1/8 acre or less	77	85	90	92
Multi-Family	1/8 acre or less	77	85	90	92
Manufactured Home	1/8 acre or less	77	85	90	92
COMMERCIAL	COMMERCIAL				
Retail	Commercial and Business	89	92	94	95
Office/Professional	Commercial and Business	89	92	94	95
INDUSTRIAL	INDUSTRIAL				
Light Industry	Industry	81	88	91	93
Heavy Industrial	Industry	92	94	96	97
Heavy Commercial	Industry	92	94	96	97
INSTITUTIONAL	INSTITUTIONAL	based on facility, i.e., park or office			
STREET R O W	STREET R O W	98	98	98	98
OPEN SPACE	OPEN SPACE				
Park/Recreation	Open, Good condition	39	61	74	80
Agriculture	Pasture, Fair Condition	49	69	79	84
Vacant Developed	Open, Fair condition	49	69	79	84
Vacant Undeveloped	Pasture, Poor Condition	68	79	86	89

TABLE 3-5
GENERALIZED LAND USE AND CURVE NUMBER ASSIGNMENTS FOR FUTURE CONDITIONS WATERSHED

HANKAMER CONSULTING LAND USE	SCS TR-55 CORRESPONDING LAND USE	SCS CURVE NO. A	B	C	D
RESIDENTIAL	RESIDENTIAL				
Low Density	1/4 acre	61	75	83	87
Medium Density	1/8 acre or less	77	85	90	92
Multi-Family	1/8 acre or less	77	85	90	92
COMMERCIAL	COMMERCIAL				
Central Business District	Commercial and Business	95	96	97	98
Office/Commercial	Commercial and Business	89	92	94	95
INDUSTRIAL	INDUSTRIAL				
Industrial / Heavy Commercial	Industry (90% Imp. Cover)	92	94	96	97
INSTITUTIONAL	INSTITUTIONAL	based on facility, i.e., park or office			
STREET R O W	STREET R O W	98	98	98	98
OPEN SPACE	OPEN SPACE				
Park/Open Space	Pasture, Fair Condition	49	69	79	84
Greenbelt, urban	Residential, 1/4 acre	61	75	83	87
Greenbelt, rural	Pasture, Poor Condition	68	79	86	89
Agriculture	Pasture, Poor Condition	68	79	86	89

and flood control facilities, it is necessary to be able to establish peak flood flows that correspond to a similar wide range of probabilities of occurrence. For this purpose, rainfall statistics often are used as the basis for establishing the frequencies associated with the occurrence of certain flood events. For purposes of this Flood Protection Planning Study for the Fredericksburg area, such rainfall statistics have been compiled from the following existing publications of the U. S. Department of Commerce.

> Hershfield, D. M.; 1961; "Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years"; U. S. Department of Commerce, Weather Bureau; Technical Paper No. 40; Washington, D.C.
>
> Miller, J. F.; 1964; "Two- to Ten-Day Precipitation for Return Periods from 2 to 100 Years in the Contiguous United States"; U. S. Department of Commerce, Weather Bureau; Technical Paper No. 49.; Washington, D.C.

Using rainfall information from these publications specifically for the Fredericksburg area, rainfall amounts for specific frequencies of occurrence and specific storm durations have been compiled and analyzed. These results are presented in Table 3-6 in terms of total rainfall amounts and rainfall intensities. Corresponding rainfall duration-intensity curves are plotted in Figure 3-2.

3.2.3 Critical Storm Duration

During the occurrence of a storm event on a given watershed, rainfall infiltrates the soil initially and then gradually begins to accumulate on and runoff from the land surface. Depending on drainage area size and shape, soil conditions, vegetative cover, imperviousness, surface depressions and other features of the watershed, the rate of runoff varies with time. Typically, the variation of the rate of runoff with time after the beginning of a rainfall event produces a bell-shaped flow hydrograph with a flattened and elongated falling limb. The shape and peak of the flow hydrograph for a given rainfall amount on a given watershed varies as a function of storm duration. Short duration, high intensity rainfall events sometimes do not last long enough to allow the entire drainage area of a particular watershed to contribute runoff to the peak flow rate at the discharge point. On the other hand, long duration storms often are characterized by low rainfall rates and, therefore, do not produce a high rate of peak runoff.

TABLE 3-6
RAINFALL DEPTHS AND INTENSITIES FOR FREDERICKSBURG, TEXAS

PEAK RAINFALL DEPTHS IN INCHES

DURATION		EVENT							
HR	MIN	1-year	2-year	5-year	10-year	25-year	50-year	100-year	500-year*
0.5	30	1.15	1.45	1.90	2.22	2.55	2.90	3.25	3.70
1	60	1.45	1.75	2.40	2.80	3.25	3.70	4.12	4.70
2	120	1.70	2.15	2.88	3.40	4.00	4.50	5.10	5.75
3	180	1.90	2.35	3.20	3.75	4.40	5.00	5.70	6.50
6	360	2.25	2.85	3.80	4.60	5.40	6.00	6.80	7.75
12	720	2.60	3.30	4.55	5.40	6.40	7.30	8.25	9.20
24	1440	3.00	3.80	5.25	6.25	7.50	8.45	9.50	12.00
48	2880	-	4.40	5.75	6.95	8.40	9.50	11.00	13.00

* Extrapolated from 25-, 50-, and 100-year data.

PEAK RAINFALL INTENSITIES IN INCHES/HOUR

DURATION		EVENT							
HR	MIN	1-year	2-year	5-year	10-year	25-year	50-year	100-year	500-year
0.5	30	2.30	2.90	3.80	4.44	5.10	5.80	6.50	7.40
1	60	1.45	1.75	2.40	2.80	3.25	3.70	4.12	4.70
2	120	0.85	1.08	1.44	1.70	2.00	2.25	2.55	2.88
3	180	0.63	0.78	1.07	1.25	1.47	1.67	1.90	2.17
6	360	0.38	0.48	0.63	0.77	0.90	1.00	1.13	1.29
12	720	0.22	0.28	0.38	0.45	0.53	0.61	0.69	0.77
24	1440	0.13	0.16	0.22	0.26	0.31	0.35	0.40	0.50
48	2880	-	0.09	0.12	0.14	0.18	0.20	0.23	0.27

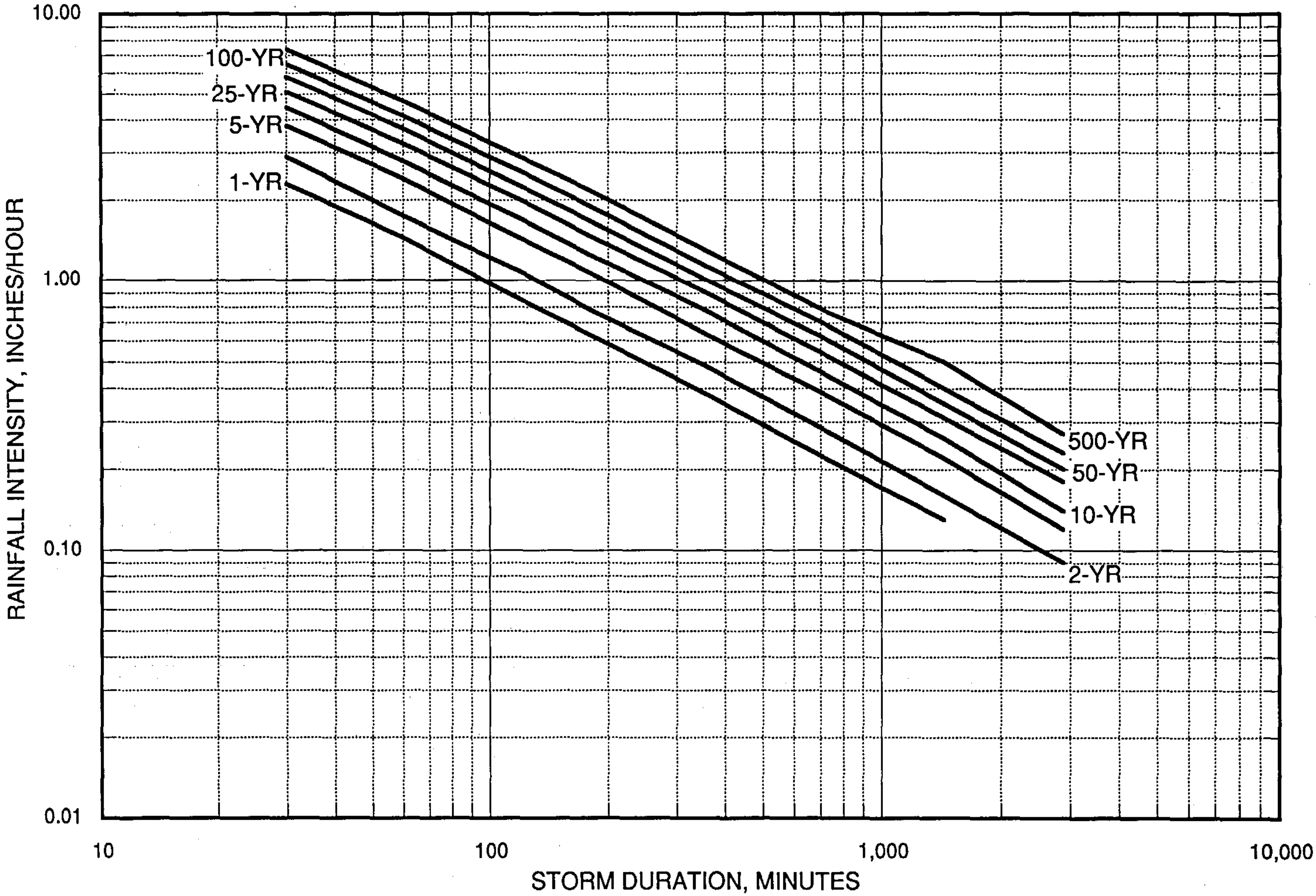

FIGURE 3-2 RAINFALL DURATION-INTENSITY CURVE FOR FREDERICKSBURG

When performing flood studies, it is important to determine the optimum duration of storm event that produces the maximum peak rate of runoff for a given amount of rainfall on a given watershed so that the most critical flooding conditions can be considered. Such analyses have been performed for the various watersheds within the planning area. The HEC-1 model of the Barons Creek basin has been operated for the 100-year rainfall event assuming different storm durations ranging from the two-hour storm up to the 24-hour storm. From these simulations, the peak runoff rates for the various subwatersheds have been examined to determine storm durations producing the maximum flood flows. These results are summarized in Table 3-7 for all of the storm durations analyzed and for both existing and future land use conditions. Peak flow rates are listed for different locations along each of the principal streams in the planning area, and the maximum flow rate at each location for a particular storm duration is identified with a box.

As illustrated by the maximum peak flow rates in Table 3-7, the six-hour storm generally produces the highest peak rates of runoff along the upper and middle reaches of Barons Creek, and, as would be expected, the longer duration 12-hour storm generates the highest peak flow rates along the lower portion of the stream because of the longer travel time from the upper watershed to the mouth. For the other smaller watersheds such as Town Creek and Stream FB-1, the three-hour storm duration appears to be most critical as it generally results in the highest peak flow rates.

Since most of the existing flooding problems within the planning area occur in the smaller watersheds and not necessarily along Barons Creek, the three-hour storm duration has been adopted as the critical storm event for purposes of this Flood Protection Planning Study. As such, the three-hour storm has been used in analyzing flood flows and associated flooding problems.

3.2.4 Peak Flood Flows

Using the rainfall amounts for the three-hour storm events as listed in Table 3-6, the HEC-1 model has been operated to generate peak flood flows along the principal streams throughout the planning area. Simulations have been made for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year rainfall events. The peak flows from the 10-, 50-, 100- and 500-year simulations are listed in Table 3-8 for both existing and future land use conditions.

TABLE 3-7
HEC-1 MODEL 100-YEAR FLOOD FLOWS FOR DIFFERENT STORM DURATIONS

SITE / CROSSING	2-HOUR STORM		3-HOUR STORM		6-HOUR STORM		12-HOUR STORM		24-HOUR STORM	
	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE
BARONS CREEK										
National Guard Armory	12,889	12,889	13,865	13,865	13,807	13,807	13,555	13,555	11,659	11,659
290 west of Town	13,427	13,375	14,963	14,923	14,966	14,965	14,677	14,669	12,811	12,791
S. Bowie Street	13,373	13,317	14,902	14,844	15,111	15,109	14,814	14,807	12,958	12,936
Washington Street	13,308	13,239	14,897	14,787	15,597	15,600	15,286	15,282	13,409	13,385
Upstream Town Creek Confluence	13,273	13,204	14,856	14,746	15,641	15,647	15,333	15,331	13,463	13,436
Downstream Town Creek Confluence	13,499	13,298	15,662	15,314	16,841	16,781	16,463	16,399	14,637	14,473
FM 1631 Upstream FB-1 Confluence	13,510	13,287	15,742	15,350	17,067	16,993	16,684	16,605	14,881	14,683
FM 1631 Downstream FB-1 Confluence	16,449	17,063	17,004	17,354	18,553	18,467	18,330	18,680	17,238	17,167
Goehmann Road	16,592	17,350	17,059	17,649	18,726	18,634	18,574	18,913	17,425	17,374
Downstream Wastewater Treatment Plant	17,777	18,805	18,341	19,112	19,373	19,450	19,829	20,289	18,448	17,503
Confluence with Stream FB-2	19,280	20,607	19,967	21,013	20,326	21,269	21,448	22,098	19,795	19,901
Confluence with Pedernales River	19,194	20,465	20,069	21,035	20,408	21,275	21,515	22,134	19,904	20,100
TOWN CREEK										
West Fork Town Creek	2,077	2,384	2,104	2,379	2,024	2,241	1,740	1,852	1,153	1,193
East Fork Town Creek / Cross Mountain W	1,312	1,393	1,325	1,406	1,258	1,319	1,051	1,063	695	687
Confl. below N. Cherry St. and W. Morse S	3,528	3,848	3,563	3,889	3,420	3,679	2,972	3,099	2,005	2,022
N. Milam Street	4,049	4,439	4,073	4,443	3,950	4,265	3,534	3,706	2,450	2,494
Immediately U/S Confl. with Barons Creek	4,371	4,829	4,433	4,872	4,323	4,698	3,907	4,138	2,779	2,849
STREAM FB-1										
Lower Crabapple Road	2,016	2,444	2,042	2,433	1,957	2,260	1,650	1,792	1,096	1,143
Ridgewood Drive in Carriage Hills	2,286	2,706	2,328	2,731	2,268	2,594	2,007	2,189	1,372	1,435
N. Llano Street	2,677	3,160	2,705	3,192	2,685	3,102	2,442	2,681	1,687	1,778
West Carriage Hills Runoff below N. Llano S	431	484	436	488	426	469	369	393	248	257
Morning Glory / Trailmoor Watershed	1,172	1,431	1,198	1,460	1,149	1,359	986	1,112	668	724
Immediately D/S Cemetery	4,510	5,289	4,619	5,410	4,606	5,294	4,220	4,661	2,963	3,147
Immediately U/S Confl. with Barons Creek	5,025	5,799	5,097	5,861	5,113	5,784	4,834	5,306	3,554	3,777

TABLE 3-7
HEC-1 MODEL 100-YEAR FLOOD FLOWS FOR DIFFERENT STORM DURATIONS

SITE / CROSSING	2-HOUR STORM		3-HOUR STORM		6-HOUR STORM		12-HOUR STORM		24-HOUR STORM	
	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE
STREAM FB-2										
Channel Near High School	212	268	213	267	194	226	150	165	95	100
South Creek Street	517	834	528	842	527	765	471	612	325	392
Friendship Road Low Water Crossing	767	1,077	785	1,096	778	1,026	704	848	492	557
Stock Pond at Camp	1,756	2,302	1,786	2,328	1,785	2,253	1,666	1,964	1,221	1,345
Immediately U/S Confl. with Barons Creek	2,128	2,751	2,165	2,774	2,175	2,696	2,068	2,440	1,559	1,734
DRY CREEK										
Upper Watershed	1,987	1,987	1,987	1,987	1,892	1,892	1,611	1,611	1,085	1,085
D/S U.S. 87	2,120	2,120	2,111	2,111	2,016	2,016	1,772	1,772	1,228	1,228
Confluence w/ West Fork	2,814	2,832	2,818	2,833	2,717	2,732	2,422	2,432	1,673	1,678
U/S U.S. 290 W (Immed. U/S of Barons Cr	3,038	3,080	3,048	3,082	2,961	2,988	2,686	2,708	1,900	1,928

TABLE 3-8
HEC-1 MODEL FLOOD FLOWS FOR EXISTING AND FUTURE WATERSHED CONDITIONS

SITE / CROSSING	10-YEAR		50-YEAR		100-YEAR		500-YEAR	
	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE
BARONS CREEK								
National Guard Armory	7,863	7,863	11,683	11,683	13,865	13,865	16,380	16,380
290 west of Town	8,503	8,483	12,607	12,577	14,963	14,923	17,682	17,630
S. Bowie Street	8,466	8,444	12,563	12,518	14,902	14,844	17,598	17,528
S. Milam Street	8,458	8,417	12,570	12,494	14,919	14,824	17,628	17,510
Washington Street	8,447	8,400	12,548	12,460	14,897	14,787	17,607	17,471
Upstream Town Creek Confluence	8,419	8,373	12,514	12,426	14,856	14,746	17,556	17,421
Downstream Town Creek Confluence	8,852	8,678	13,184	12,897	15,662	15,314	18,523	18,103
FM 1631 Upstream FB-1 Confluence	8,892	8,693	13,249	12,927	15,742	15,350	18,620	18,146
FM 1631 Downstream FB-1 Confluence	9,548	9,337	14,281	14,330	17,004	17,354	20,149	20,900
Goehmann Road	9,536	9,418	14,275	14,598	17,059	17,649	20,503	21,196
Upstream Wastewater Treatment Plant	9,608	9,623	14,417	14,873	17,395	17,954	20,900	21,548
Downstream Wastewater Treatment Plant	9,812	10,176	15,159	15,797	18,341	19,112	22,091	23,003
Confluence with Stream FB-2	10,564	11,161	16,463	17,355	19,967	21,013	24,073	25,271
Confluence with Pedernales River	10,649	11,216	16,550	17,387	20,069	21,035	24,182	25,281
TOWN CREEK								
West Fork Town Creek	1,136	1,339	1,751	1,998	2,104	2,379	2,512	2,817
East Fork Town Creek / Cross Mountain West	716	737	1,103	1,161	1,325	1,406	1,581	1,690
Confl. below N. Cherry St. and W. Morse St.	1,927	2,116	2,966	3,243	3,563	3,889	4,252	4,634
N. Milam Street	2,173	2,398	3,378	3,697	4,073	4,443	4,879	5,321
Immediately U/S Confl. with Barons Creek	2,352	2,621	3,670	4,049	4,433	4,872	5,318	5,824
STREAM FB-1								
Lower Crabapple Road	1,071	1,321	1,686	2,028	2,042	2,433	2,455	2,899
Ridgewood Drive in Carriage Hills	1,199	1,454	1,913	2,264	2,328	2,731	2,812	3,270
N. Llano Street	1,376	1,666	2,209	2,623	2,705	3,192	3,295	3,852
West Carriage Hills Runoff below N. Llano St.	210	241	352	396	436	488	535	596
Carriage Hills Runoff and FB-1 mainstem	1,712	2,054	2,816	3,286	3,467	4,009	4,248	4,883
Morning Glory / Trailmoor Watershed	595	757	974	1,202	1,198	1,460	1,460	1,759
Immediately D/S Cemetery	2,262	2,737	3,734	4,423	4,619	5,410	5,658	6,560
Immediately U/S Confl. with Barons Creek	2,489	2,970	4,126	4,780	5,097	5,861	6,273	7,149

TABLE 3-8
HEC-1 MODEL FLOOD FLOWS FOR EXISTING AND FUTURE WATERSHED CONDITIONS

SITE / CROSSING	10-YEAR		50-YEAR		100-YEAR		500-YEAR	
	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE	EXISTING	FUTURE
STREAM FB-2								
Channel Near High School	110	146	175	223	213	267	258	318
South Creek Street	250	464	423	705	528	842	654	1,000
Friendship Road Low Water Crossing	372	578	628	906	785	1,096	970	1,316
Stock Pond at Camp	883	1,233	1,450	1,933	1,786	2,328	2,183	2,808
Immediately U/S Confl. with Barons Creek	1,062	1,465	1,748	2,294	2,165	2,774	2,655	3,331
DRY CREEK								
Upper Watershed	1,112	1,112	1,670	1,670	1,987	1,987	2,353	2,353
D/S U.S. 87	1,184	1,184	1,775	1,775	2,111	2,111	2,507	2,507
Confl. with West Fork of Dry Creek	1,572	1,586	2,366	2,380	2,818	2,833	3,344	3,363
U/S U.S. 290 W (Immed. U/S of Barons Creek)	1,680	1,721	2,550	2,586	3,048	3,082	3,627	3,662

Comparisons of the peak flow rates for the 100-year flood as simulated with the HEC-1 model with those previously used in the effective flood insurance study for the City of Fredericksburg as listed in Table 3-1 indicate that the current HEC-1 results generally are slightly higher by about five to fifteen percent. These levels of increase in the peak flood flows of the more urbanized streams, i. e., Town Creek and Stream FB-1, during the last fifteen years are not surprising considering the growth and expansion of the City that has occurred over this same timeframe. However, such increases in the peak flow rates for the upper and middle reaches of Barons Creek probably are due more to differences in engineering judgment and the particular analytical methods employed rather than any changes in these portions of the watershed that have produced additional runoff.

As part of this Flood Protection Planning Study, the peak flow results from the current HEC-1 modeling have been discussed with representatives from FEMA and the Fort Worth District of the Corps of Engineers, and the slight increases above the flood flows used in the original FIS have been noted. Considering FEMA's guidelines for allowing changes in flood flows previously used in determining effective flood insurance base flood elevations and floodplain boundaries, it was jointly agreed that the peak flood flows used in the previous FIS for the City of Fredericksburg would be used to reflect current watershed conditions for all issues related to flood insurance in both this Flood Protection Planning Study and in the Gillespie County flood insurance studies being conducted by the Corps. For all other analyses in this Flood Protection Planning Study, however, the peak flood flows simulated with the HEC-1 model for both existing and future water conditions have been used. This includes the analysis of existing flooding problems and the design of drainage improvements and flood control measures.

3.3 LOCALIZED RUNOFF ANALYSES

During the course of this Flood Protection Planning Study, a number of localized flooding problem areas have been identified and investigated. These are described and discussed in Section 5.0 of this report. As part of the flood investigations for each of these localized flooding problem areas, it has been necessary to estimate the peak rates of runoff from the various subwatersheds and subareas that contribute flood waters to the various problem areas. These flood flows have been used in evaluating the flooding depths associated with storms of different magnitudes and in developing the appropriate drainage improvements and flood control measures needed to mitigate the flooding problems. In some cases, it has been necessary to determine peak flood flows for several different subareas within the total drainage area that contributes stormwater to a particular problem area. The subwatersheds corresponding to each of the designated localized flooding problem areas and their individual subareas are delineated on the map of the City in Plate 3-2.

Typically, the contributing subwatersheds, and the associated subareas, for the localized flooding problem areas are less than a few hundred acres in size; therefore, the determination of peak flood flows has been made using a procedure known as the Rational Formula. With this method, the peak flow rate from a given watershed (Q) is estimated as the product of a runoff coefficient (C), ranging in magnitude from zero to one depending on watershed conditions, times the drainage area (A) expressed in acres times the appropriate rainfall intensity (i) expressed in inches per hour, i. e., $Q = C\ i\ A$. To maximize the peak flow rate, the rainfall intensity usually is taken as the value corresponding to a storm duration that is equal to the time of concentration for a given watershed.

For all of the identified localized problem areas, the Rational Formula was used to calculate the peak flood flows produced by the 2-, 5-, 10-, 25- and 100-year rainfall events. The contributing drainage areas, and various subareas thereof, were determined using the existing five-foot contour topographic maps as provided by the City, along with some field verification of drainage divides. The same maps also were used to determine runoff flow paths for each of the subareas within a particular problem subwatershed. The flow paths were field verified, as necessary. Based on the flow paths, the times of concentration for the various subareas were determined using the SCS procedures as described in Technical Release No. TR-55 and as discussed previously for the HEC-1 modeling in Section 3.2.1. Critical rainfall intensities for each storm frequency were established for durations corresponding to the times of concentration for each of the subareas.

Runoff coefficients for each subarea were estimated for each storm frequency using standard runoff coefficients from the City of Austin's Drainage Criteria Manual (1996). Runoff coefficients corresponding to developed watershed conditions were estimated by using the "fair grass (2-7% slope)" runoff coefficient for pervious areas and the average of the "asphaltic" and "concrete/roof" values for impervious areas. For planning purposes, fully-developed watershed conditions, with an average of 35-percent impervious cover, have been assumed for establishing the appropriate runoff coefficients. The impervious and pervious runoff coefficients for the different storm frequencies and and the resulting fully-developed watershed runoff coefficients as used for the peak flood flow determinations are summarized below.

WATERSHED CONDITION	RUNOFF COEFFICIENTS FOR DIFFERENT STORM FREQUENCIES				
	2-Year	5-Year	10-Year	25-Year	100-Year
Impervious	0.74	0.78	0.82	0.87	0.96
Pervious	0.33	0.36	0.38	0.42	0.49
Fully-Developed	0.47	0.51	0.53	0.58	0.65

Results from the peak runoff calculations for various subareas within the different localized flooding problem areas are summarized in Table 3-9. For each subarea within the problem area subwatersheds, the drainage area size in acres and the time of concentration in minutes are indicated. Then, for each of the storm frequencies analyzed, the runoff coefficient, the rainfall intensity corresponding to the indicated time of concentration, and the resulting peak runoff rate are presented for each subarea. The names of the localized flooding problem areas listed in the table and the associated subarea names are the same as the identifiers used in Sections 5.0 and 6.0 of this report to reference the various problem areas and subareas when discussing flooding conditions and potential drainage improvements and flood control measures. The names of the localized flooding problem areas and their respective subareas also are noted on the map in Plate 3-2. These names generally correspond to the street names nearest to the problem sites or nearest the subarea discharge locations.

TABLE 3-9
LOCALIZED AREA FLOODING ANALYSIS

LOCALIZED FLOODING PROBLEM AREA DRAINAGE SUBAREA	AREA acres	TIME CONC. min	2-YEAR EVENT C2	i2 in/hr	Q2 cfs	5-YEAR EVENT C5	i5 in/hr	Q5 cfs	10-YEAR EVENT C10	i10 in/hr	Q10 cfs	25-YEAR EVENT C25	i25 in/hr	Q25 cfs	100-YEAR EVENT C100	i100 in/hr	Q100 cfs
Friendship Lane																	
Schneider Hill	42.13	17.6	0.47	3.76	74.5	0.51	4.99	107.3	0.53	5.89	131.5	0.58	6.80	166.2	0.65	8.58	235.0
Schubert																	
Schubert	27.50	31.2	0.47	2.71	35.0	0.51	3.61	50.6	0.53	4.25	62.0	0.58	4.91	78.3	0.65	6.23	111.3
Cross Mountain - Milam																	
Cross Mt.	8.08	35.1	0.47	2.51	9.5	0.51	3.35	13.8	0.53	3.95	16.9	0.58	4.56	21.4	0.65	5.79	30.4
Ave D	13.55	27.8	0.47	2.90	18.5	0.51	3.86	26.7	0.53	4.56	32.7	0.58	5.26	41.3	0.65	6.66	58.7
Ave. A	29.38	17.6	0.47	3.76	51.9	0.51	4.99	74.8	0.53	5.89	91.7	0.58	6.80	115.8	0.65	8.58	163.8
Pecan	82.93	43.5	0.47	2.19	85.3	0.51	2.93	123.8	0.53	3.45	151.6	0.58	3.98	191.7	0.65	5.06	273.0
Milam U/S (N)	11.84	27.3	0.47	2.93	38.9	0.51	3.90	56.2	0.53	4.60	68.9	0.58	5.32	87.1	0.65	6.73	123.6
Milam U/S (N & M)	28.24	32.6	0.47	2.63	53.2	0.51	3.51	77.0	0.53	4.13	94.3	0.58	4.77	119.2	0.65	6.06	169.4
Milam U/S (S)	21.17	34.2	0.47	2.55	25.4	0.51	3.40	36.8	0.53	4.01	45.0	0.58	4.64	56.9	0.65	5.88	80.9
Milam D/S & Milam U/S (S)	64.21	54.2	0.47	1.89	56.9	0.51	2.53	82.9	0.53	2.98	101.6	0.58	3.45	128.3	0.65	4.39	183.0
Burbank - Llano																	
Burbank - Llano	47.75	40.5	0.47	2.29	51.4	0.51	3.06	74.6	0.53	3.61	91.4	0.58	4.17	115.5	0.65	5.30	164.4
North Lincoln																	
N. Lincoln & Burbank	99.47	58.9	0.47	1.78	83.3	0.51	2.40	121.5	0.53	2.82	148.8	0.58	3.26	188.1	0.65	4.15	268.3
College - Llano																	
College - Llano	147.51	68.3	0.47	1.61	111.5	0.51	2.17	163.0	0.53	2.55	199.6	0.58	2.95	252.2	0.65	3.76	360.2
College - Travis																	
College & N. Lincoln	275.22	88.7	0.47	1.34	172.9	0.51	1.81	253.6	0.53	2.13	310.5	0.58	2.46	392.4	0.65	3.14	561.1
Travis	341.48	107.0	0.47	1.17	187.3	0.51	1.58	275.7	0.53	1.86	337.4	0.58	2.15	426.5	0.65	2.75	610.3
Trailmoor																	
Trailmoor	84.48	44.5	0.47	2.15	85.5	0.51	2.88	124.3	0.53	3.40	152.2	0.58	3.93	192.4	0.65	4.99	274.0
Morning Glory - Llano																	
Morning Glory	185.12	40.9	0.47	2.27	197.9	0.51	3.04	287.2	0.53	3.59	351.8	0.58	4.14	444.7	0.65	5.26	633.1
Lower Crabapple - Llano	277.98	49.8	0.47	2.00	260.9	0.51	2.68	379.6	0.53	3.16	464.9	0.58	3.64	587.6	0.65	4.64	837.5

TABLE 3-9
LOCALIZED AREA FLOODING ANALYSIS

LOCALIZED FLOODING	AREA	TIME	2-YEAR EVENT			5-YEAR EVENT			10-YEAR EVENT			25-YEAR EVENT			100-YEAR EVENT		
PROBLEM AREA		CONC.	C2	i2	Q2	C5	i5	Q5	C10	i10	Q10	C25	i25	Q25	C100	i100	Q100
DRAINAGE SUBAREA	acres	min		in/hr	cfs		in/hr	cfs		in/hr	cfs		in/hr	cfs		in/hr	cfs
Carriage Hills																	
Edgewood	42.02	18.4	0.47	3.67	72.5	0.51	4.88	104.5	0.53	5.75	128.1	0.58	6.64	161.9	0.65	8.38	228.9
Driftwood N. & Edgewood	89.73	29.0	0.47	2.83	119.2	0.51	3.77	172.4	0.53	4.44	211.2	0.58	5.13	267.0	0.65	6.50	379.1
Driftwood S.	96.73	36.4	0.47	2.45	111.5	0.51	3.28	161.6	0.53	3.86	198.0	0.58	4.46	250.2	0.65	5.66	355.9
Adams	29.38	35.8	0.47	2.48	34.2	0.51	3.31	49.6	0.53	3.90	60.7	0.58	4.51	76.8	0.65	5.72	109.2
Adams & Driftwood	126.11	36.4	0.47	2.45	145.4	0.51	3.28	210.8	0.53	3.86	258.2	0.58	4.46	326.4	0.65	5.66	464.2
Crestwoods	35.53	37.6	0.47	2.40	40.2	0.51	3.21	58.2	0.53	3.79	71.3	0.58	4.37	90.1	0.65	5.55	128.2
Adams & Crestwoods	161.63	37.6	0.47	2.40	182.7	0.51	3.21	264.8	0.53	3.79	324.4	0.58	4.37	410.0	0.65	5.55	583.4
N. Llano & Adams	181.79	40.6	0.47	2.29	195.3	0.51	3.06	283.4	0.53	3.60	347.1	0.58	4.16	438.7	0.65	5.29	624.5
Frederick	10.08	34.9	0.47	2.52	11.9	0.51	3.37	17.3	0.53	3.97	21.2	0.58	4.58	26.8	0.65	5.82	38.1
Tanglewood & Frederick	13.32	35.7	0.47	2.49	15.6	0.51	3.32	22.6	0.53	3.91	27.6	0.58	4.52	34.9	0.65	5.74	49.7
West Creek St.																	
S. Bowie	24.77	21.2	0.47	3.40	39.6	0.51	4.51	57.0	0.53	5.32	69.9	0.58	6.15	88.3	0.65	7.77	125.1
S. Bowie S. & S. Bowie	34.28	24.2	0.47	3.15	50.8	0.51	4.19	73.2	0.53	4.94	89.8	0.58	5.71	113.4	0.65	7.22	160.9
Edison N.	21.47	49.2	0.47	2.02	20.3	0.51	2.70	29.6	0.53	3.18	36.2	0.58	3.68	45.8	0.65	4.68	65.3
Edison S. & Edison N.	27.90	51.0	0.47	1.97	25.8	0.51	2.64	37.5	0.53	3.11	45.9	0.58	3.59	58.1	0.65	4.56	82.8
W. Creek W.	7.92	28.5	0.47	2.86	10.6	0.51	3.81	15.4	0.53	4.49	18.8	0.58	5.19	23.8	0.65	6.57	33.8
W. Creek E.	9.18	21.8	0.47	3.34	14.4	0.51	4.44	20.8	0.53	5.24	25.5	0.58	6.05	32.2	0.65	7.65	45.7
W. Creek @ S. Milam	17.10	28.5	0.47	2.86	23.0	0.51	3.81	33.2	0.53	4.49	40.7	0.58	5.18	51.4	0.65	6.57	73.0
Old Harper Rd.																	
Armory Rd. E.	77.66	32.0	0.47	2.66	97.1	0.51	3.55	140.5	0.53	4.18	172.1	0.58	4.83	217.6	0.65	6.12	309.2
Highway 290 S.	50.44	19.8	0.47	3.53	83.6	0.51	4.68	120.5	0.53	5.52	147.7	0.58	6.38	186.6	0.65	8.06	264.2
Old Harper Rd. W.	44.01	24.3	0.47	3.14	64.9	0.51	4.18	93.7	0.53	4.92	114.8	0.58	5.69	145.1	0.65	7.19	205.8
Old Harper Rd. Middle	62.74	17.0	0.47	3.83	112.9	0.51	5.08	162.5	0.53	5.99	199.2	0.58	6.92	251.8	0.65	8.73	355.8
Old Harper Rd. E.	61.94	28.0	0.47	2.89	84.0	0.51	3.84	121.4	0.53	4.53	148.8	0.58	5.24	188.1	0.65	6.63	267.0

TABLE 3-9
LOCALIZED AREA FLOODING ANALYSIS

LOCALIZED FLOODING PROBLEM AREA DRAINAGE SUBAREA	AREA acres	TIME CONC. min	2-YEAR EVENT C2	i2 in/hr	Q2 cfs	5-YEAR EVENT C5	i5 in/hr	Q5 cfs	10-YEAR EVENT C10	i10 in/hr	Q10 cfs	25-YEAR EVENT C25	i25 in/hr	Q25 cfs	100-YEAR EVENT C100	i100 in/hr	Q100 cfs
Winfried Creek																	
Winfried Creek (WC1)	108.17	28.1	0.47	2.89	146.7	0.51	3.84	212.0	0.53	4.53	259.8	0.58	5.23	328.3	0.65	6.63	466.0
Soutwest Trib. (WC3)	49.88	24.6	0.47	3.12	73.2	0.51	4.15	105.6	0.53	4.90	129.5	0.58	5.66	163.6	0.65	7.16	232.0
SW Trib. (WC2 & WC3)	145.29	28.2	0.47	2.87	196.3	0.51	3.83	283.7	0.53	4.51	347.7	0.58	5.21	439.4	0.65	6.61	623.8
WC2 N, WC2 & WC3	148.03	29.2	0.47	2.82	196.0	0.51	3.75	283.3	0.53	4.43	347.2	0.58	5.11	438.8	0.65	6.48	623.1
Winfried Cr. (WC1 & WC2)	256.19	29.2	0.47	2.82	339.1	0.51	3.75	490.2	0.53	4.42	600.6	0.58	5.11	759.1	0.65	6.47	1078.0
South Trib. (WC4-S)	125.03	29.9	0.47	2.77	163.0	0.51	3.70	235.7	0.53	4.36	288.7	0.58	5.03	364.9	0.65	6.38	518.3
Winfried Creek @ S. Milam	469.82	33.8	0.47	2.57	567.6	0.51	3.43	821.9	0.53	4.04	1006.9	0.58	4.67	1272.6	0.65	5.92	1809.2
Five Points																	
S. Adams	42.70	38.9	0.47	2.35	47.2	0.51	3.14	68.4	0.53	3.70	83.8	0.58	4.28	105.9	0.65	5.43	150.8
Ufer	27.04	30.7	0.47	2.73	34.7	0.51	3.64	50.2	0.53	4.30	61.6	0.58	4.96	77.8	0.65	6.29	110.5
Park St.	11.39	39.4	0.47	2.33	12.5	0.51	3.12	18.1	0.53	3.67	22.2	0.58	4.24	28.0	0.65	5.39	39.9
Live Oak	18.62	22.1	0.47	3.32	29.0	0.51	4.41	41.9	0.53	5.20	51.3	0.58	6.01	64.9	0.65	7.59	91.9
South Lincoln	25.10	34.9	0.47	2.52	29.7	0.51	3.36	43.1	0.53	3.97	52.8	0.58	4.58	66.7	0.65	5.81	94.8
Five Points Intersection	43.72	34.9	0.47	2.52	51.8	0.51	3.37	75.1	0.53	3.97	92.0	0.58	4.58	116.3	0.65	5.82	165.3
Granite	20.55	29.0	0.47	2.83	27.3	0.51	3.77	39.5	0.53	4.44	48.4	0.58	5.13	61.2	0.65	6.50	86.8
Granite @ E. Live Oak	64.28	40.0	0.47	2.31	69.8	0.51	3.09	101.2	0.53	3.64	124.0	0.58	4.20	156.7	0.65	5.34	223.1
South Adams																	
South Adams South	59.78	34.1	0.47	2.56	71.9	0.51	3.41	104.1	0.53	4.03	127.5	0.58	4.65	161.2	0.65	5.90	229.2

TABLE 3-9
LOCALIZED AREA FLOODING ANALYSIS

LOCALIZED FLOODING	AREA	TIME	2-YEAR EVENT			5-YEAR EVENT			10-YEAR EVENT			25-YEAR EVENT			100-YEAR EVENT		
PROBLEM AREA		CONC.	C2	i2	Q2	C5	i5	Q5	C10	i10	Q10	C25	i25	Q25	C100	i100	Q100
DRAINAGE SUBAREA	acres	min		in/hr	cfs		in/hr	cfs		in/hr	cfs		in/hr	cfs		in/hr	cfs
Highway - Apple																	
Highway St. W.	33.76	45.0	0.47	2.14	33.9	0.51	2.86	49.3	0.53	3.37	60.4	0.58	3.90	76.3	0.65	4.95	108.7
Highway St. N.	19.02	30.0	0.47	2.77	24.8	0.51	3.69	35.8	0.53	4.35	43.9	0.58	5.03	55.4	0.65	6.37	78.7
Highway St. W. & N.	52.78	45.0	0.47	2.14	53.0	0.51	2.86	77.0	0.53	3.37	94.4	0.58	3.90	119.3	0.65	4.95	169.9
Highway St. E., W. & N	75.55	65.9	0.47	1.65	58.6	0.51	2.22	85.5	0.53	2.62	104.7	0.58	3.02	132.4	0.65	3.85	189.0
Eagle St. & Highway St.	112.75	65.9	0.47	1.65	87.4	0.51	2.22	127.6	0.53	2.62	156.3	0.58	3.02	197.5	0.65	3.85	282.0
Franklin W.	4.38	38.7	0.47	2.36	4.9	0.51	3.15	7.0	0.53	3.72	8.6	0.58	4.29	10.9	0.65	5.45	15.5
Franklin E. & W.	7.91	45.2	0.47	2.13	7.9	0.51	2.86	11.5	0.53	3.37	14.1	0.58	3.89	17.8	0.65	4.94	25.4
Apple St. W.	26.48	41.1	0.47	2.27	28.2	0.51	3.03	41.0	0.53	3.58	50.2	0.58	4.13	63.4	0.65	5.25	90.3
Apple St. E. &. W.	32.34	44.4	0.47	2.16	32.8	0.51	2.89	47.6	0.53	3.40	58.3	0.58	3.93	73.7	0.65	4.99	105.0
HW 290 @ Apple	3.99	5.0	0.47	6.32	11.8	0.51	8.36	17.0	0.53	9.87	20.8	0.58	11.40	26.3	0.65	14.20	36.8
Crenweldge D/S Apple	49.36	45.4	0.47	2.13	49.3	0.51	2.85	71.7	0.53	3.36	87.8	0.58	3.88	110.9	0.65	4.93	158.1

Notes:
1. TIME CONC. is the Time of Concentration.
2. C2 is the Runoff Coefficient for the 2-year flood event used in the Rational Formula.
3. i2 is the Rainfall Intensity for the 2-year flood event used in the Rational Formula.
4. Q2 is the runoff for the 2-year flood event calculated by the Rational Formula.

4.0 STREAM HYDRAULIC ANALYSES

4.1 STREAM MODEL DEVELOPMENT

As discussed in the previous section, the currently-effective Flood Insurance Study (FIS) for the City of Fredericksburg was completed in 1980. As part of this earlier study, computerized hydraulic models of portions of several of the principal streams within the City were developed for purposes of establishing flood levels and floodplain boundaries as required by the National Flood Insurance Program. These original FIS hydraulic models were developed using the U. S. Army Corps of Engineers' HEC-2 Water Surface Profiles program. The specific streams modeled in the original FIS included portions of Barons Creek, Town Creek and Stream FB-1, a tributary of Barons Creek that extends through the extreme northeast portion of the City. The modeled reaches of these streams previously have been identified on the map of the area in Figure 3-1, along with the reach of Stream FB-2, another tributary of Barons Creek located south of downtown Fredericksburg, that was modeled pursuant to a 1995 Letter of Map Revision (LOMR) issued by the Federal Emergency Management Agency (FEMA).

For purposes of this Flood Protection Planning Study, copies of the original FIS HEC-2 computer models of Barons Creek, Town Creek and Stream FB-1 were obtained from FEMA. To a large extent, the original FIS models for Barons Creek and Town Creek have formed the basis for the revised models that have been developed as part of this study. Both of these models have been updated with current channel and bridge information through the downtown area. For Stream FB-1, the model recently developed (1996) by the Fort Worth District of the Corps of Engineers as part of the ongoing Gillespie County flood insurance studies has been acquired and used in this Flood Protection Planning Study, with minor modifications. Use of the Corps' model of Stream FB-1 assures consistency between the results from this planning effort and those developed by the Corps in the Gillespie County flood insurance studies. For the same reason, the Corps model of the reach of Barons Creek extending from near the City's wastewater treatment plant south of the downtown area upstream to the U. S. Highway 290 bridge also has been incorporated into the overall HEC-2 model of Barons Creek for purposes of this Flood Protection Planning Study. In addition, the FIS hydraulic models for Barons Creek and Town Creek have been extended upstream of the City in this Flood Protection Planning Study using data and information acquired in the field and from available topographic maps. The Town Creek HEC-2 model also has been extended through the new Cross Mountain subdivision using information provided to the City by the subdivision engineer.

The various reaches of the principal streams in the vicinity of the City of Fredericksburg for which revised and updated HEC-2 hydraulic models now have been developed are identified on the map of the area in Figure 4-1. These are the models that have been used in this Flood Protection Planning Study for the analyses of flood levels corresponding to various storm events, watershed conditions and alternative flood control measures and drainage improvements.

As noted previously, all of the stream hydraulic models are based on the Corps' HEC-2 Water Surface Profiles program (September 1990). Predecessor versions of this program have been widely used for performing backwater calculations in streams and rivers for almost thirty years. As stated in the HEC-2 User's Manual,

> *The program is intended for calculating water surface profiles for steady gradually varied flow in natural or man-made channels. Both subcritical and supercritical flow profiles can be calculated. The effects of various obstructions such as bridges, culverts, weirs, and structures in the flood plain may be considered in the computations. The computational procedure is based on the solution of the one-dimensional energy equation with energy loss due to friction evaluated with Manning's equation. The computational procedure is generally known as the standard step method. The program is also designed for application in flood plain management and flood insurance studies to evaluate floodway encroachments. Also, capabilities are available for assessing the effects of channel improvements and levees on water surface profiles.*

4.2 BARONS CREEK HEC-2 ANALYSIS

The original FIS version of the HEC-2 model of Barons Creek extended from a section below the U. S. Highway 290 crossing approximately two and one half miles southeast of downtown Fredericksburg upstream to a section located near the intersection of U. S. Highway 290 and U. S. Highway 87 on the northwest side of the City. To update this original model to reflect existing channel conditions, 21 cross sections on the mainstem were field surveyed. Seventeen of these cross sections were incorporated into the FIS model to reduce the distance between existing computational sections or to provide descriptions of channel geometry where modifications such as fill placement has occurred. In addition, four of the new surveyed channel cross sections were incorporated into the model to describe conditions at the new low water crossing at

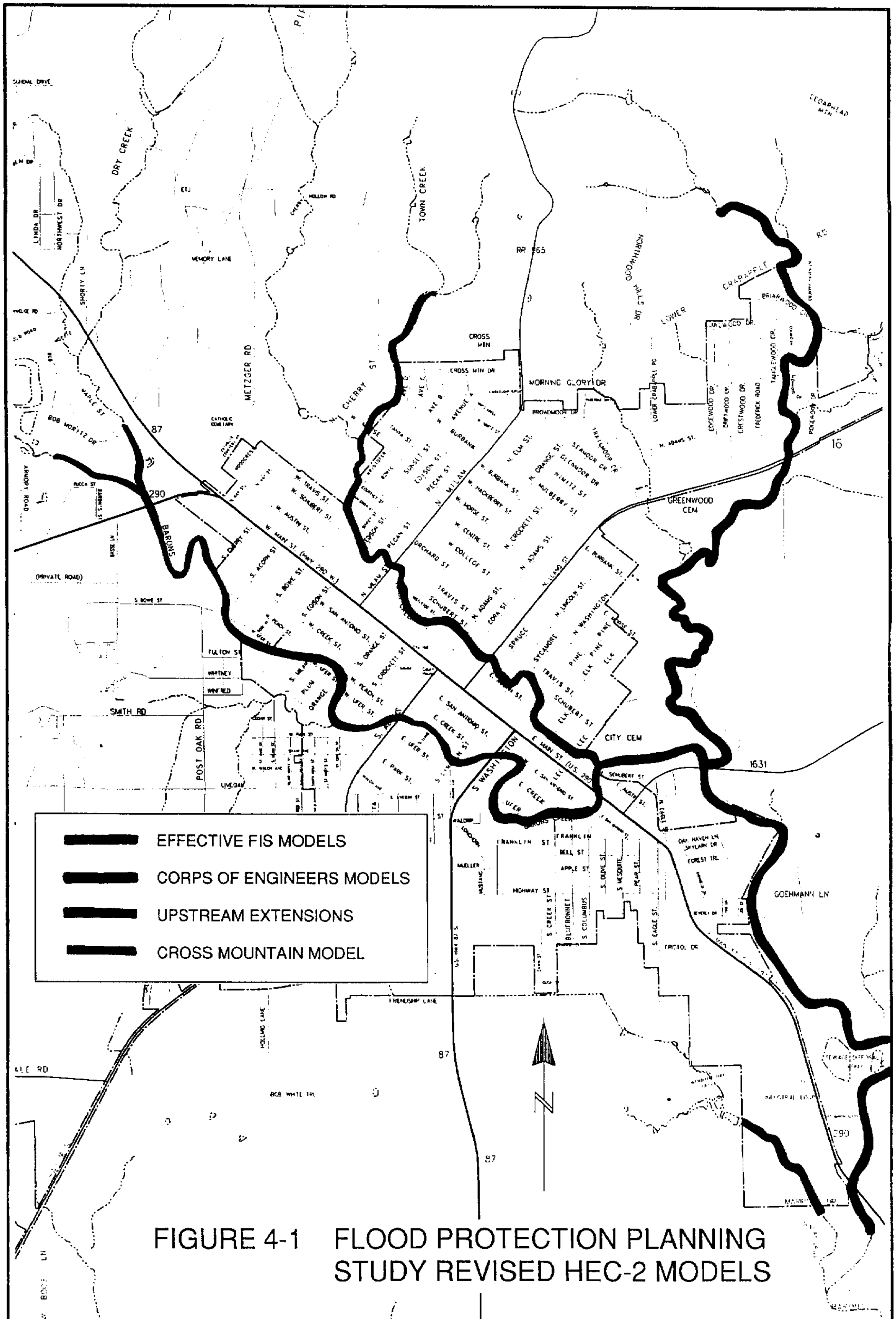

FIGURE 4-1 FLOOD PROTECTION PLANNING STUDY REVISED HEC-2 MODELS

Creek Street. Other computational sections were added to the model to describe the bridge improvements at Adams Street as shown on design plans from the Texas Department of Transportation (TxDOT). The HEC-2 model also was extended upstream of South Bowie Street to above U. S. Highway 290 using TxDOT design plans for the U. S. Highway 290 crossing and information from the City's existing five-foot contour topographic maps for sections along Dry Creek and the mainstem of Barons Creek upstream of U. S. Highway 290.

Between Section 142+89, which is adjacent to the City's wastewater treatment plant southeast of downtown, and Section 252+13 just upstream of Main Street, a channel distance of about two miles, the updated FIS model of Barons Creek was replaced with the Corps' current HEC-2 model of Barons Creek as developed in the Gillespie County flood insurance studies. As explained earlier, this modification was made primarily to assure consistency between the hydraulic results from this Flood Protection Planning Study and those developed by the Corps in the Gillespie County flood insurance studies. In this segment of the Barons Creek model, the Corps section numbering system has been retained, even though it is not compatible with the section numbers in the original FIS model. The section numbers in the model do not affect the hydraulic calculations.

The revised model of Barons Creek, with all of the additional field-surveyed computational sections incorporated and with the Corps' Gillespie County model included, has been operated to simulate water surface profiles along the stream for the 10-, 50-, 100- and 500-year flood events. Two sets of simulations have been made based on flood flows from the original FIS corresponding to existing watershed conditions (Table 3-1) and from the HEC-1 model developed in this study corresponding to future developed watershed conditions (Table 3-8). Results from these simulations in terms of water surface elevations for the 100-year flood are presented in Table 4-1. For comparison purposes, the corresponding 100-year flood water surface elevations from the original FIS also are presented, as are the minimum flowline elevations of the Barons Creek channel at each computational section. Profile plots of these same 100-year flood levels along the length of Barons Creek are presented in Figures 4-2 and 4-3 for the lower and the upper segments of the creek, respectively.

As expected, the 100-year flood water levels corresponding to future watershed

TABLE 4-1
BARONS CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
	9302	1594.04	9302	1594.04	1594.75
	9372	1594.31	9372	1594.31	1594.98
U.S. 290	9382	1594.20	9382	1594.20	1594.80
	9424	1594.74	9424	1594.74	1595.52
	9434	1594.61	9434	1594.61	1595.49
	9550	1596.41	9550	1596.41	1597.24
	9800	1597.47	9800	1597.47	1598.23
	11900	1605.09	11900	1605.09	1605.80
	13400	1611.26	13400	1611.26	1611.57
BEGIN COE SECTIONS	-	-	0	1614.59	1614.91
	-	-	194	1614.77	1615.07
	-	-	379	1614.94	1615.25
	-	-	763	1616.02	1616.32
	-	-	1182	1616.55	1616.86
	-	-	1609	1617.81	1618.10
	16120	1617.11	-	-	-
	-	-	1922	1618.29	1618.54
	-	-	2379	1619.41	1619.67
	-	-	2828	1620.72	1621.00
	-	-	-	-	-
	-	-	3137	1621.49	1621.77
	-	-	3441	1623.22	1623.52
	-	-	3776	1624.32	1624.63
	18000	1621.75	-	-	-
	-	-	3853	1625.37	1625.69
	18035	1621.61	-	-	-
GOEHMANN RD.	18045	1622.69	3872	1625.43	1625.75
	18055	1623.27	-	-	-
	18065	1622.82	3892	1625.88	1626.22
	-	-	3904	1625.85	1626.19
	18100	1624.86	-	-	-
	-	-	3959	1625.89	1626.23
	-	-	4170	1626.63	1626.96
	-	-	4421	1628.06	1628.38
	-	-	4654	1628.66	1629.00
	-	-	-	-	-
	-	-	5097	1629.59	1629.93
	-	-	5551	1630.73	1631.06
	20180	1632.81	-	-	-

TABLE 4-1
BARONS CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
	-	-	6009	1632.32	1632.65
	-	-	6557	1633.56	1633.89
	-	-	-	-	-
	-	-	7022	1634.83	1635.17
	-	-	7483	1636.42	1636.76
	-	-	7867	1637.44	1637.76
	22020	1639.45	-	-	-
	-	-	7979	1637.96	1638.27
	22064	1639.64	-	-	-
F.M. 1631	22074	1640.81	-	-	-
	-	-	8000	1640.48	1641.03
	22086	1640.98	-	-	-
	22096	1641.23	-	-	-
	-	-	8030	1641.25	1641.86
	22120	1641.23	-	-	-
	22155	1641.12	-	-	-
	-	-	8101	1641.33	1641.92
	-	-	8412	1641.67	1642.19
	22600	1641.82	-	-	-
	-	-	8704	1642.30	1642.72
	-	-	8952	1643.47	1643.77
	23400	1643.40	-	-	-
	-	-	9418	1644.57	1644.77
	-	-	10001	1646.16	1646.27
	24400	1647.06	-	-	-
	-	-	10517	1648.38	1648.43
	-	-	10839	1649.46	1649.39
	-	-	10988	1649.52	1649.48
	25015	1649.24	-	-	-
	25057	1649.63	-	-	-
MAIN ST.	25067	1650.17	11110	1649.69	1649.70
	25113	1650.46	-	-	-
	25123	1650.33	-	-	-
	25165	1650.63	-	-	-
	-	-	11228	1651.62	1652.19
END COE SECTIONS	-	-	11262	1651.76	1652.36
	25700	1652.74	25700	1652.46	1653.06
	26250	1655.46	26250	1655.66	1656.44
	26284	1655.69	-	-	-

TABLE 4-1
BARONS CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
CREEK ST.	-	-	26285	1655.30	1656.07
	26294	1655.75	-	-	-
	26306	1655.77	-	-	-
	26316	1655.55	26316	1655.27	1656.05
	26350	1655.35	26350	1657.66	1656.45
	27100	1657.57	27100	1657.70	1658.47
	27700	1659.16	27700	1659.23	1660.18
	-	-	28200	1661.12	1662.13
	-	-	28350	1661.60	1662.59
	29275	1665.12	29275	1665.22	1666.28
	29317	1665.21	29317	1665.30	1666.33
WASHINGTON ST.	29327	1665.17	29327	1665.27	1666.19
	29373	1665.93	29373	1666.05	1667.25
	29383	1666.23	29383	1666.37	1667.95
	29425	1666.37	29425	1666.50	1668.05
	-	-	29640	1666.62	1668.04
	30250	1668.16	30250	1669.05	1670.37
	30270	1668.16	30270	1669.03	1670.34
LINCOLN ST.	30280	1668.01	30280	1668.90	1670.12
	30320	1668.33	30320	1669.17	1670.59
	30330	1668.26	30330	1669.14	1670.75
	30350	1669.04	30350	1669.79	1671.49
	31000	1670.78	31000	1671.61	1673.02
	31625	1673.45	31625	1674.11	1675.33
	-	-	31661	1674.15	1675.37
ADAMS ST.	31663	1673.53	-	-	-
	31673	1673.53	-	-	-
	31727	1673.76	-	-	-
	31737	1674.17	-	-	-
	-	-	31740	1674.35	1675.58
	31775	1674.18	31775	1674.63	1675.89
	32900	1675.62	32900	1675.62	1676.94
	32900	1677.00	32900	1677.00	1678.30
	34068	1683.35	34068	1683.35	1684.62
	34093	1683.41	34093	1683.41	1684.68
ORANGE ST.	34099	1683.37	34099	1683.37	1684.59
	34101	1683.39	34101	1683.39	1684.60
	34107	1683.44	34107	1683.44	1684.71
	34132	1683.47	34132	1683.47	1684.74

TABLE 4-1
BARONS CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
	34750	1684.94	34750	1684.94	1686.12
	34778	1685.16	34778	1685.16	1686.37
MILAM ST.	34788	1685.14	34788	1685.14	1686.13
	34812	1685.58	34812	1685.50	1686.80
	34822	1686.45	34822	1686.45	1688.65
	34850	1686.52	34850	1686.52	1688.66
	35500	1687.62	35500	1687.62	1689.50
	-	-	36275	1690.20	1691.63
	36900	1691.41	36900	1692.39	1693.77
	36928	1691.37	36928	1692.38	1693.77
BOWIE ST.	36943	1692.61	36943	1693.26	1694.69
	36957	1692.63	36957	1693.28	1694.71
	36977	1692.09	36977	1692.86	1694.28
	37000	1692.70	37000	1693.21	1694.55
	-	-	37600	1695.23	1696.45
END FIS SECTIONS	38400	1698.16	38400	1698.66	1699.85
	-	-	41062	1707.69	1709.02
U.S. 290 W	-	-	41162	1707.54	1708.68
	-	-	41189	1708.31	1709.72
	-	-	41239	1710.09	1712.13
	-	-	41770	1713.56	1714.63
	-	-	42230	1718.79	1719.41
	-	-	43020	1723.71	1724.64
	-	-	43990	1727.22	1728.01

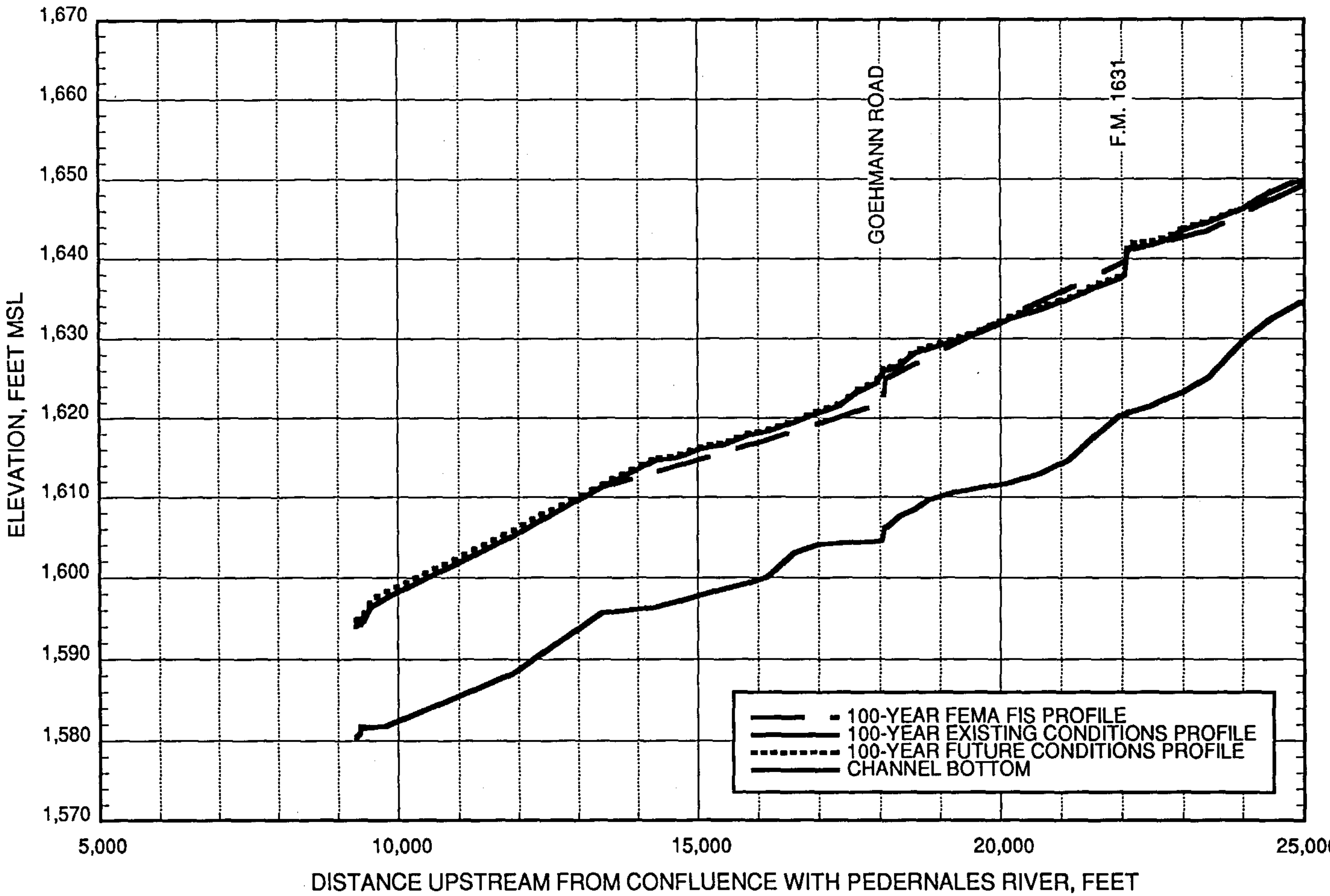

FIGURE 4-2 BARONS CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE PROFILES

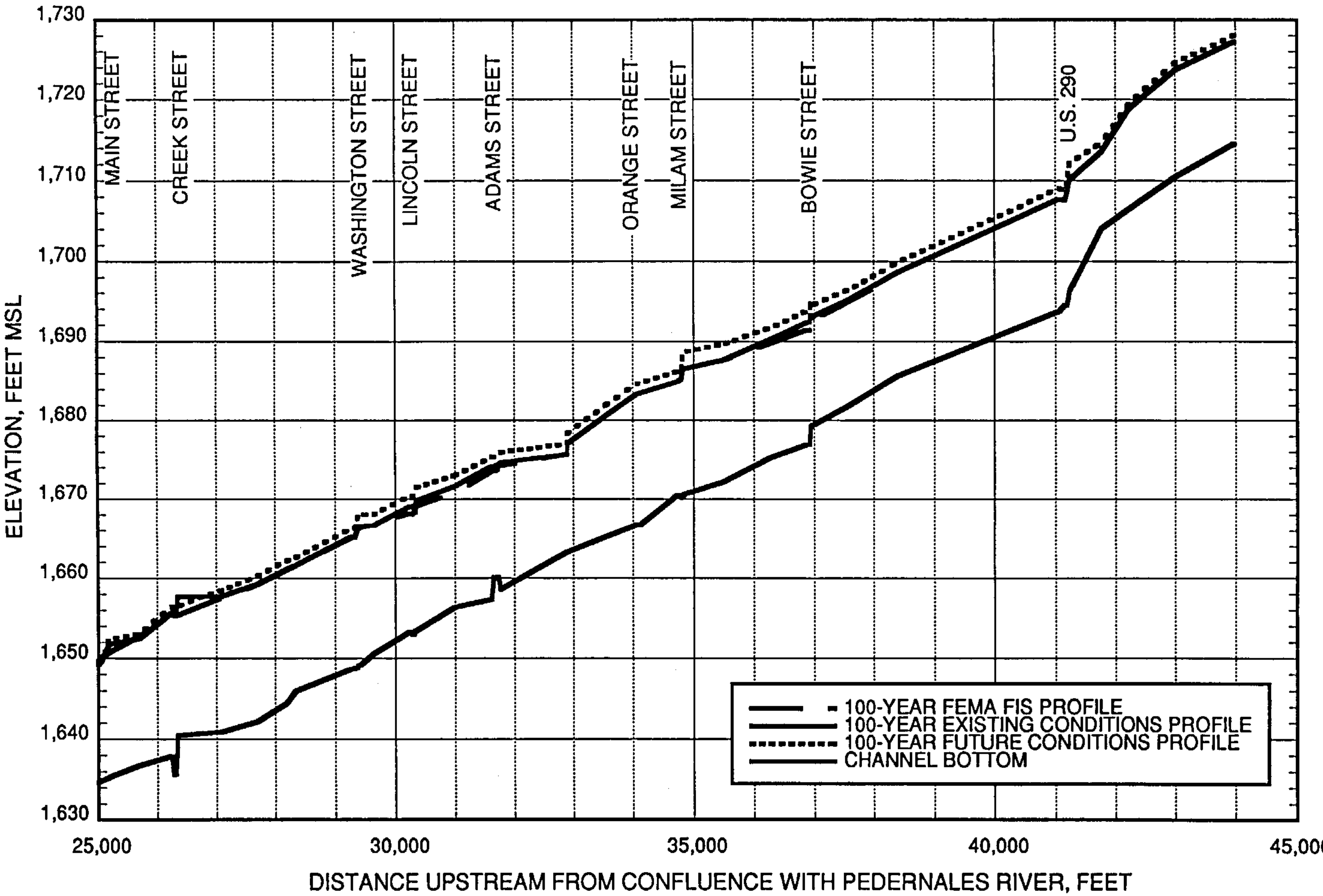

FIGURE 4-3 BARONS CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE PROFILES

conditions are somewhat higher than those for existing watershed conditions. Downstream of Main Street (U. S. Highway 290), the increase in flood levels averages about 0.4 feet, while upstream of Main Street the effect of future development in the watershed is to increase flood levels an average of about 1.2 feet. The maximum increase in flood levels due to the projected future development of the watershed is on the order of 2.2 feet, which occurs upstream of Milam Street.

There are also several reaches along Barons Creek where the 100-year flood levels for existing watershed conditions as simulated with the revised HEC-2 model developed during this Flood Protection Planning Study differ significantly from those determined during the original FIS. In the reach downstream of Goehmann Road, the higher water levels from the revised HEC-2 model appear to be the result of the increased accuracy provided by the new computational sections that have been added to the revised model. The FIS model has only three computational sections to describe the channel geometry from near the City's wastewater treatment plant to Goehmann Road, and the revised model has 13 computational sections for this same reach.

Approximately 1,000 feet upstream of the F. M. 1631 bridge, the flood levels simulated with the revised model for existing watershed conditions exceed those from the original FIS model by about 1.4 feet. Again, this difference in flood levels is due to the improved descriptions of channel geometry through this reach of the updated model. At the Creek Street crossing, increased flood levels in the revised model are the result of including the new low-water bridge in the revised model. The 100-year flood levels immediately upstream of this new bridge as simulated with the revised HEC-2 model are about 2.3 feet higher than those from the FIS model.

The only other significant differences in flood water levels between the results from the revised HEC-2 model and the FIS model occur along the reach from Lincoln Street to Adams Street and near South Bowie Street. These increases also are attributable to the improved accuracy of the revised model reflected in the additional computational sections that have been incorporated to describe existing channel conditions.

4.3 TOWN CREEK HEC-2 ANALYSIS

The HEC-2 model for Town Creek from the original FIS extended from the mouth of the creek at its confluence with Barons Creek upstream to a point near the intersection of

Travis and Bowie Streets in the northwestern part of the City. To update this model to reflect existing conditions, 26 cross sections were surveyed at different locations along the creek to obtain information on various channel and floodplain modifications. Five of these new cross sections were used to describe fill that had been placed in the floodplain of the creek, ten were used to describe modified road crossings at Elk, Crockett and Orange Streets, and ten of the new sections were used to extend the model upstream across Morse Street and up to the new Cross Mountain subdivision. New computational sections were incorporated into the model to reflect these modified conditions. The HEC-2 model of the reach of Town Creek through the new Cross Mountain subdivision, which was developed by the subdivision engineer, also was added to the overall Town Creek model.

Listings of the 100-year flood water surface elevations as simulated with the revised model of Town Creek are presented in Table 4-2 based on flood flows from the HEC-1 model corresponding to existing and future watershed and land use conditions. Also included in the table for comparison purposes are the corresponding 100-year flood levels from the original FIS for the City. Although HEC-2 simulations for the 10-, 50-, and 500-year floods have been made, the resulting flood levels have not been tabulated for this report.

Profile plots of the 100-year flood levels along Town Creek as simulated with the revised HEC-2 model and from the original FIS are presented in Figures 4-4 and 4-5 for the lower and the upper segments of the creek, respectively. Because significant portions of the Town Creek watershed are projected to develop in the future, the flood levels for future watershed conditions in the plots are somewhat higher than those simulated for existing conditions. Increases in 100-year flood levels due to future watershed development on the order of 0.4 to 0.7 feet occur from Elk Street to Adams Street, and upstream of Adams Street, the increases vary between zero and 0.8 feet.

Of most significance are the apparent differences in 100-year flood levels between those from the original FIS and those simulated with the revised HEC-2 model. As shown by the profile plots, the flood levels immediately upstream of Elk Street as simulated with the revised model are as much as 3.5 feet higher than those from the effective FIS. This water level difference apparently is caused by an old bridge structure beneath the new bridge that has never been removed and now obstructs flood flows passing down the creek. From Adams Street to Crockett Street, the revised-

TABLE 4-2
TOWN CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
	230	1645.04	230	1645.04	1648.43
	600	1648.35	600	1648.35	1649.38
	1210	1654.35	1210	1654.36	1655.04
	-	-	1287	1655.09	1655.77
ELK ST.	1298	1655.32	-	-	-
	1332	1655.32	-	-	-
	-	-	1333	1657.57	1658.14
	1430	1657.24	1430	1660.03	1660.79
LOW WATER	1641	1658.96	1641	1660.61	1661.39
CROSSING	1651	1658.73	1651	1660.29	1661.09
	1669	1660.67	1669	1660.75	1661.48
	1689	1661.50	1689	1661.53	1662.27
	1890	1661.65	1890	1661.69	1662.43
	1933	1661.69	1933	1661.72	1662.46
AUSTIN ST.	1943	1661.40	1943	1661.44	1662.12
	1957	1661.52	1957	1661.55	1662.27
	1967	1660.71	1967	1660.71	1661.54
	2000	1663.67	2000	1663.61	1664.52
	2195	1665.91	2195	1665.78	1666.81
	2249	1665.97	2249	1665.84	1666.88
WASHINGTON ST.	2259	1665.68	2259	1665.57	1666.55
	2281	1665.98	2281	1665.88	1666.91
	2291	1666.85	2291	1666.76	1667.98
	2320	1666.87	2320	1666.78	1668.00
	-	-	2600	1667.05	1668.22
	-	-	2850	1667.55	1668.65
	-	-	3100	1668.25	1668.65
	3300	1668.47	-	-	-
	3300	1669.36	-	-	-
	-	-	3250	1669.71	1670.58
	-	-	3450	1670.51	1671.28
	3910	1675.37	3910	1674.29	1674.90
	3982	1676.11	3982	1675.72	1676.34
LLANO ST.	3992	1675.92	3992	1675.54	1676.07
	4028	1676.94	4028	1676.74	1677.53
	4038	1677.03	4038	1676.84	1677.62
	4110	1677.52	4110	1677.36	1678.15
	4635	1680.28	4635	1680.43	1681.20

TABLE 4-2
TOWN CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
	4690	1680.68	4690	1680.82	1681.58
ADAMS ST.	4700	1681.79	4700	1681.85	1682.57
	4720	1682.05	4720	1682.11	1682.94
	4730	1681.26	4730	1681.37	1682.35
	4760	1682.49	4760	1682.49	1683.66
	-	-	4970	1685.86	1685.83
	-	-	5230	1687.92	1688.45
	-	-	5403	1688.18	1688.70
	-	-	5428	1688.58	1689.17
	-	-	5439	1688.55	1689.12
	-	-	5440	1688.33	1688.87
	-	-	5441	1688.33	1688.87
	-	-	5443	1688.76	1689.38
	-	-	5462	1688.71	1689.31
	5470	1688.16	-	-	-
			5494	1688.66	1689.24
CROCKETT ST.	5496	1688.26	-	-	-
	5539	1689.66	-	-	-
	-	-	5541	1692.41	1692.65
	-	-	5561	1692.25	1692.42
	5595	1691.70	5595	1693.09	1693.51
	6250	1693.22	6252	1693.69	1694.13
	6272	1692.92	6272	1693.65	1694.08
ORANGE ST.	6282	1697.70	6282	1697.70	1697.80
	6318	1698.90	6318	1698.54	1698.82
	6328	1698.90	-	-	-
	-	-	6340	1698.34	1698.50
	6350	1698.96	-	-	-
	6810	1699.57	6810	1699.86	1700.34
SCHUBERT ST.	6834	1699.48	6834	1699.83	1700.35
	6886	1699.86	6886	1700.08	1700.57
	6910	1699.57	6910	1699.80	1700.18
	7210	1701.32	7210	1701.31	1701.86
	7245	1702.28	7245	1702.28	1702.20
MILAM ST.	7255	1702.56	7255	1702.58	1702.79
	7285	1703.08	7285	1703.09	1703.60
	7295	1702.95	7295	1702.96	1703.32
	7320	1703.32	7320	1703.33	1703.78

TABLE 4-2
TOWN CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
	7775	1704.17	7775	1705.15	1705.66
	7812	1705.63	7812	1705.62	1706.42
SCHUBERT ST.	7820	1708.25	7820	1708.24	1708.93
	7830	1708.26	7830	1708.25	1708.94
	7838	1707.28	7838	1707.29	1707.95
	7875	1708.85	7875	1708.82	1709.55
	8240	1709.80	8240	1709.80	1710.30
	8273	1709.98	8273	1709.98	1710.47
EDISON ST.	8283	1710.20	8283	1710.20	1710.66
	8297	1710.22	8297	1710.22	1710.68
	8307	1709.87	8307	1709.87	1710.33
	8340	1709.94	8340	1709.94	1710.39
	8710	1711.35	8710	1711.58	1711.99
	8773	1712.71	8773	1712.63	1712.96
TRAVIS ST.	8783	1712.71	8783	1712.64	1712.97
	8797	1712.73	8797	1712.65	1712.98
	8807	1712.76	8807	1712.69	1713.02
	8910	1712.76	8910	1712.70	1713.03
	9700	1715.39	9700	1715.41	1716.17
END FIS	10250	1721.80	10250	1721.47	1722.26
BEGIN EXTENSION	-	-	10635	1723.19	1724.03
	-	-	10810	1724.36	1725.19
MORSE ST.	-	-	10863	1727.85	1728.26
	-	-	10895	1728.83	1729.16
	-	-	10911	1729.77	1730.17
	-	-	11300	1730.81	1731.31
	-	-	11800	1735.24	1735.85
	-	-	12260	1740.38	1741.11
	-	-	12440	1742.51	1743.40
BEGIN CROSS MTN	-	-	12698	1745.25	1745.49
	-	-	12842	1745.42	1745.64
	-	-	12956	1746.89	1747.15
	-	-	13272	1751.79	1751.86
	-	-	13652	1757.08	1757.20
	-	-	14013	1760.85	1761.01
	-	-	14294	1767.19	1767.18
	-	-	14687	1770.99	1771.07
	-	-	14824	1772.45	1772.49

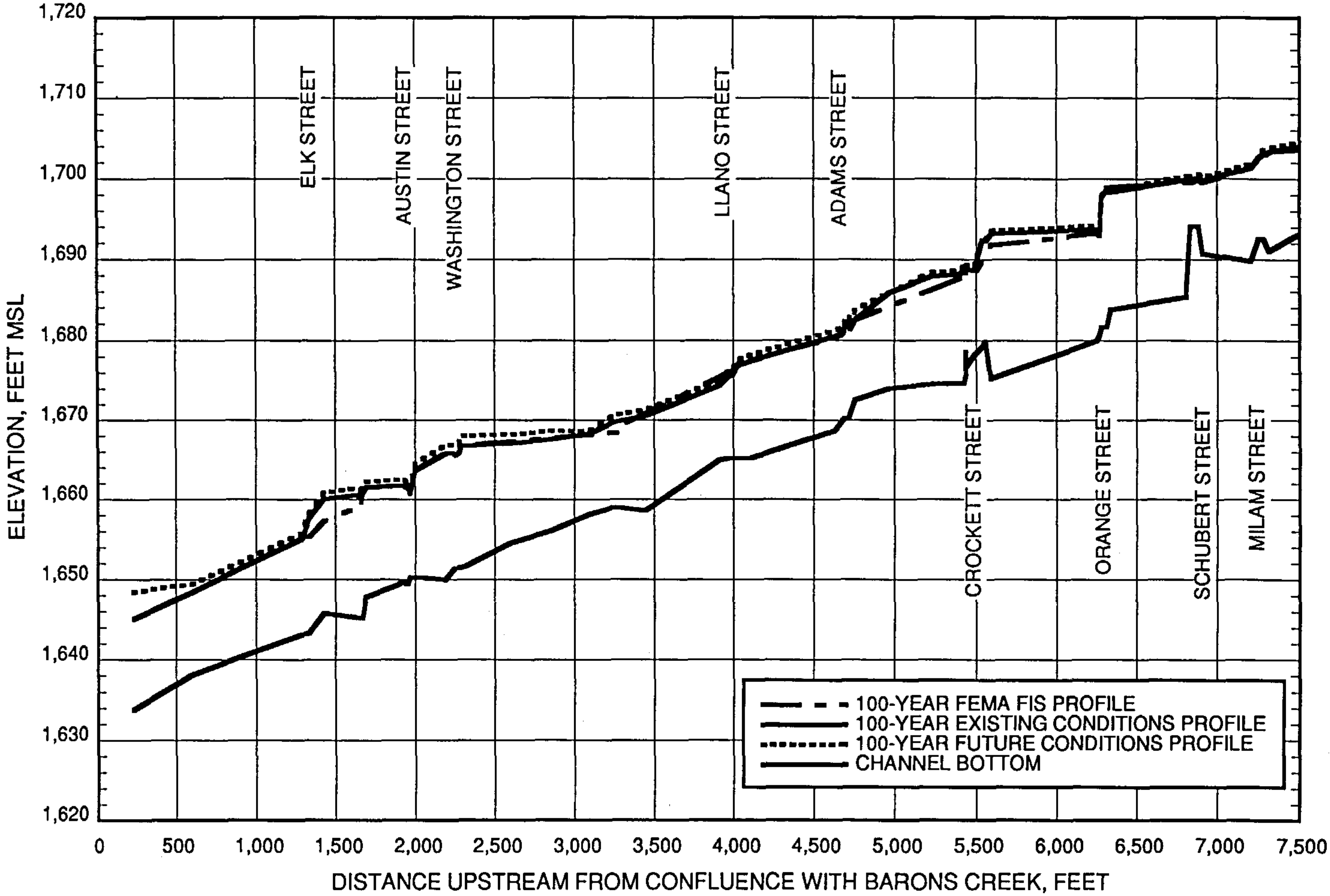

FIGURE 4-4 TOWN CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE PROFILES

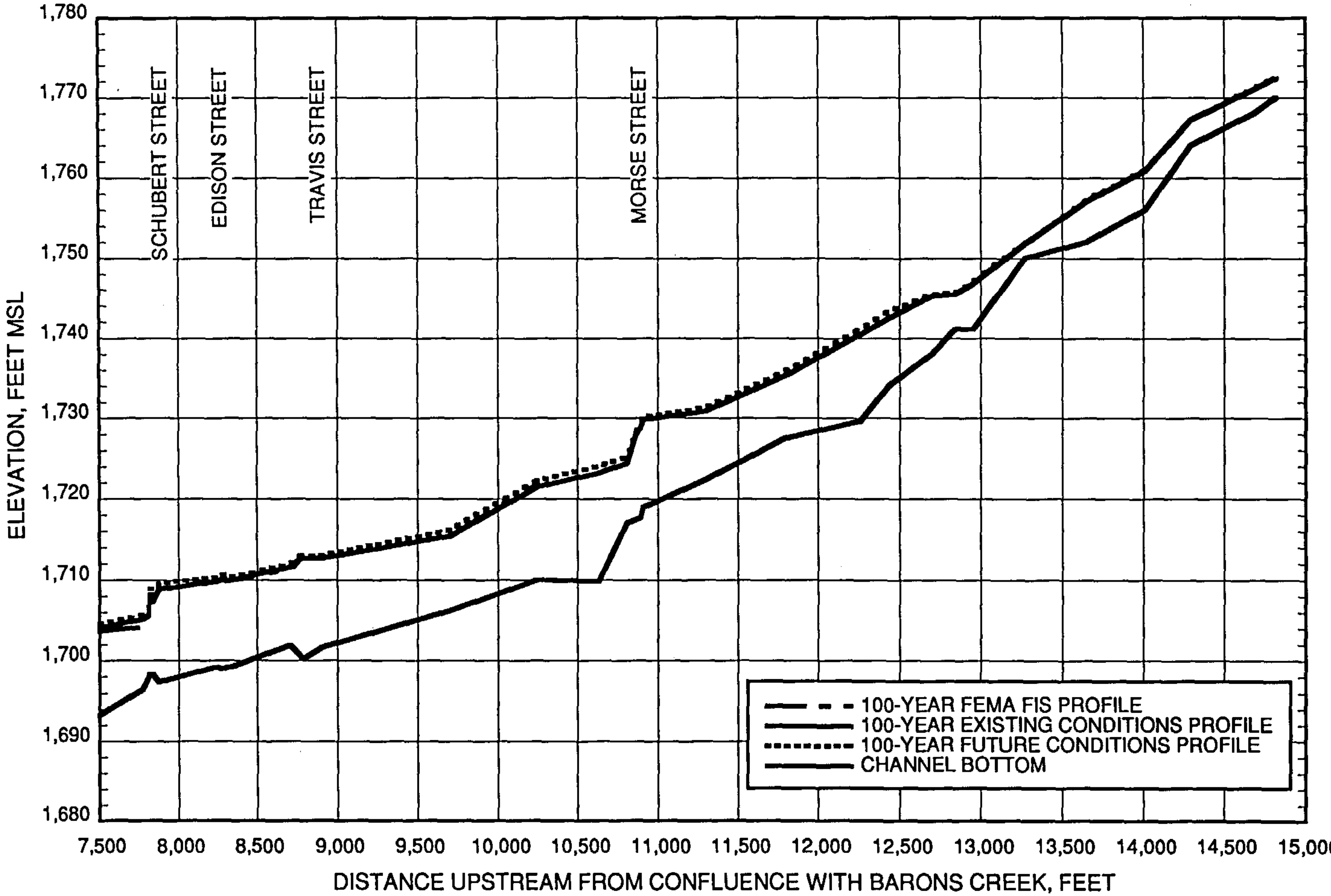

FIGURE 4-5 TOWN CREEK 100-YEAR FLOOD HEC-2 WATER SURFACE PROFILES

model flood levels are up to 1.1 feet above the FIS water surface elevations, which is likely the result of fill material and other channel modifications along this reach of the creek. Immediately upstream of Crockett Street, the increase in flood levels is approximately 3.0 feet, which apparently has been caused by bridge and culvert modifications at this crossing. Upstream of Orange Street, there is very little difference between the revised-model results and those from the original FIS.

4.4 STREAM FB-1 HEC-2 ANALYSIS

For this tributary of Barons Creek, the HEC-2 model from the original FIS, which extended from the mouth of the creek near F. M. 1631 upstream to above Briarwood Circle in the Carriage Hills subdivision, has been replaced entirely with the revised HEC-2 model developed by the Corps of Engineers in the Gillespie County flood insurance study. The revised model now extends up to Lower Crabapple Road, almost 3,000 feet beyond the end of the original FIS model. The revised model incorporates considerably more detail with regard to describing channel geometry. It includes 95 computational sections from the confluence at Barons Creek to Lower Crabapple Road, whereas the original FIS model included only 19 computational sections.

The revised model of Stream FB-1, with all of the additional computational sections incorporated in accordance with the Corps' Gillespie County model, also has been operated to simulate water surface profiles along the stream for the 10-, 50-, 100- and 500-year flood events. Again, simulations have been made using flood flows for existing watershed conditions (Table 3-1) and future developed watershed conditions (Table 3-8). Results from these simulations in terms of water surface elevations for the 100-year flood are presented in Table 4-3. For comparison purposes, the corresponding 100-year flood water surface elevations from the original FIS also are presented. Profile plots of these same 100-year flood levels along the length of Stream FB-1 are presented in Figures 4-6 and 4-7 for the lower and the upper segments of the watercourse, respectively.

Examination of the flood profiles indicates that development of the watershed will likely cause 100-year flood levels to increase on the order of 0.6 to 1.2 feet along Stream FB-1 from near its mouth up to about the Llano Highway (State Highway 16). These flood level increases are not expected to dramatically affect floodplain boundaries.

TABLE 4-3
STREAM FB-1 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
BARONS CREEK	-	-	0	1632.53	1641.03
CONCRETE CHANNEL	-	-	31	1634.94	1640.61
	-	-	52	1637.33	1641.03
	100	1638.69	-	-	-
	-	-	76	1337.73	1641.26
	-	-	113	1638.97	1641.21
LOW WATER CROSS.	-	-	171	1639.50	1641.07
	200	1640.17	-	-	-
	-	-	231	1640.92	1641.65
	-	-	412	1641.18	1641.95
	-	-	561	1642.34	1643.14
	-	-	942	1643.93	1644.82
	-	-	1192	1645.38	1646.41
	-	-	1441	1646.62	1647.62
	-	-	1597	1647.80	1648.80
	-	-	1791	1648.51	1649.49
	-	-	1949	1649.19	1650.12
	-	-	2167	1651.03	1651.85
	-	-	2341	1652.44	1653.26
	-	-	2514	1654.02	1654.98
	-	-	2820	1655.51	1656.50
	2250	1647.96	-	-	-
	2251	1649.43	-	-	-
	-	-	3009	1655.99	1656.92
	-	-	3201	1655.74	1656.88
	-	-	3276	1658.74	1659.94
	-	-	3363	1659.78	1661.11
	-	-	3524	1660.16	1661.40
	3120	1656.49	-	-	-
	-	-	3855	1661.94	1663.13
	-	-	3963	1662.72	1663.63
	-	-	4051	1663.32	1664.34
	-	-	4167	1666.50	1667.69
	-	-	4280	1667.18	1668.26
	-	-	4408	1667.89	1668.94
	-	-	4506	1669.13	1669.80
	-	-	4656	1670.38	1671.44

TABLE 4-3
STREAM FB-1 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
	-	-	4792	1670.76	1671.76
	-	-	5139	1671.25	1672.19
	-	-	5366	1671.79	1672.65
	-	-	5458	1672.24	1673.10
	-	-	5549	1672.58	1673.47
	-	-	5725	1674.39	1675.40
	-	-	5820	1674.44	1675.45
	5400	1672.39	-	-	-
	-	-	6047	1675.09	1675.99
	-	-	6350	1677.02	1677.67
	-	-	6510	1678.33	1679.05
	-	-	6698	1678.71	1679.42
	-	-	6911	1679.24	1679.88
	7000	1679.57	-	-	-
	-	-	7143	1680.44	1681.02
	-	-	7334	1681.63	1681.87
	-	-	7618	1683.08	1683.16
	-	-	7899	1684.52	1684.41
	7820	1681.93	-	-	-
	-	-	8204	1685.37	1685.25
	-	-	8436	1685.79	1685.67
	-	-	8631	1686.32	1686.20
	-	-	8890	1687.36	1687.23
	-	-	9075	1688.70	1688.58
	-	-	9360	1691.18	1691.04
	9280	1691.04	-	-	-
	-	-	9548	1692.80	1692.69
	-	-	9768	1694.65	1694.51
	-	-	9959	1695.89	1695.71
	-	-	10107	1696.48	1696.30
	-	-	10270	1697.50	1697.35
	-	-	10375	1698.86	1698.69
	-	-	10581	1700.42	1700.23
	-	-	10781	1702.28	1702.10
	10600	1699.08	-	-	-
	-	-	11170	1704.37	1704.22
	-	-	11353	1705.71	1705.55

TABLE 4-3
STREAM FB-1 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
	11360	1703.68	-	-	-
	11380	1703.61	-	-	-
LLANO HWY	-	-	11540	1708.34	1708.12
	11444	1708.46	11600	1710.27	1710.30
	11494	1708.50	-	-	-
	-	-	11727	1710.68	1710.70
	-	-	12039	1710.80	1710.81
	-	-	12450	1711.87	1711.88
	-	-	12646	1712.62	1712.62
	12500	1711.90	-	-	-
	-	-	12876	1713.56	1713.57
	-	-	12970	1715.70	1715.70
	-	-	13050	1716.44	1716.45
	-	-	13244	1718.50	1718.50
	-	-	13355	1720.16	1720.16
	-	-	13452	1720.96	1720.96
	-	-	13563	1721.35	1721.35
	13410	1718.18	-	-	-
RIDGEWOOD DR.	-	-	13642	1721.28	1721.45
	-	-	13800	1722.88	1722.56
	-	-	13930	1723.65	1723.27
	-	-	14102	1724.85	1724.50
	-	-	14262	1726.34	1726.02
	-	-	14429	1728.85	1728.29
	-	-	14525	1730.69	1730.26
	14400	1726.01	-	-	-
	-	-	14756	1732.92	1732.62
	14800	1729.22	-	-	-
	-	-	15100	1734.31	1733.94
	-	-	15303	1735.00	1734.61
	-	-	15455	1736.52	1736.12
	-	-	15588	1738.47	1738.10
	-	-	15661	1739.53	1739.12
	-	-	15740	1740.01	1739.62
	-	-	15900	1741.98	1741.60
	-	-	16073	1744.21	1743.69
	-	-	16302	1746.01	1745.61

TABLE 4-3
STREAM FB-1 100-YEAR FLOOD HEC-2 WATER SURFACE ELEVATIONS

SECTION LOCATION (D/S FACE)	FEMA FIS HEC-2 SECTION NUMBER	FEMA FIS 100-YEAR WATER SURFACE ELEVATION FT MSL	UPDATED HEC-2 SECTION NUMBER	UPDATED EXISTING 100-YEAR WATER SURFACE ELEVATION FT MSL	FUTURE CONDITIONS 100-YEAR WATER SURFACE ELEVATION FT MSL
	-	-	16532	1748.22	1747.87
	16400	1748.18	-	-	-
	-	-	16676	1749.27	1748.69
	-	-	16791	1752.17	1751.70
	-	-	16874	1753.56	1753.11
	-	-	17053	1756.02	1755.54
	-	-	17186	1756.22	1755.76
LOWER CRABAPPLE	-	-	17334	1756.85	1756.73
	-	-	17362	1757.28	1756.99
	-	-	17460	1757.91	1758.17
	-	-	17810	1759.50	1759.89
	-	-	18092	1761.72	1762.13
	-	-	18480	1766.53	1767.07
	-	-	19304	1779.48	1779.90

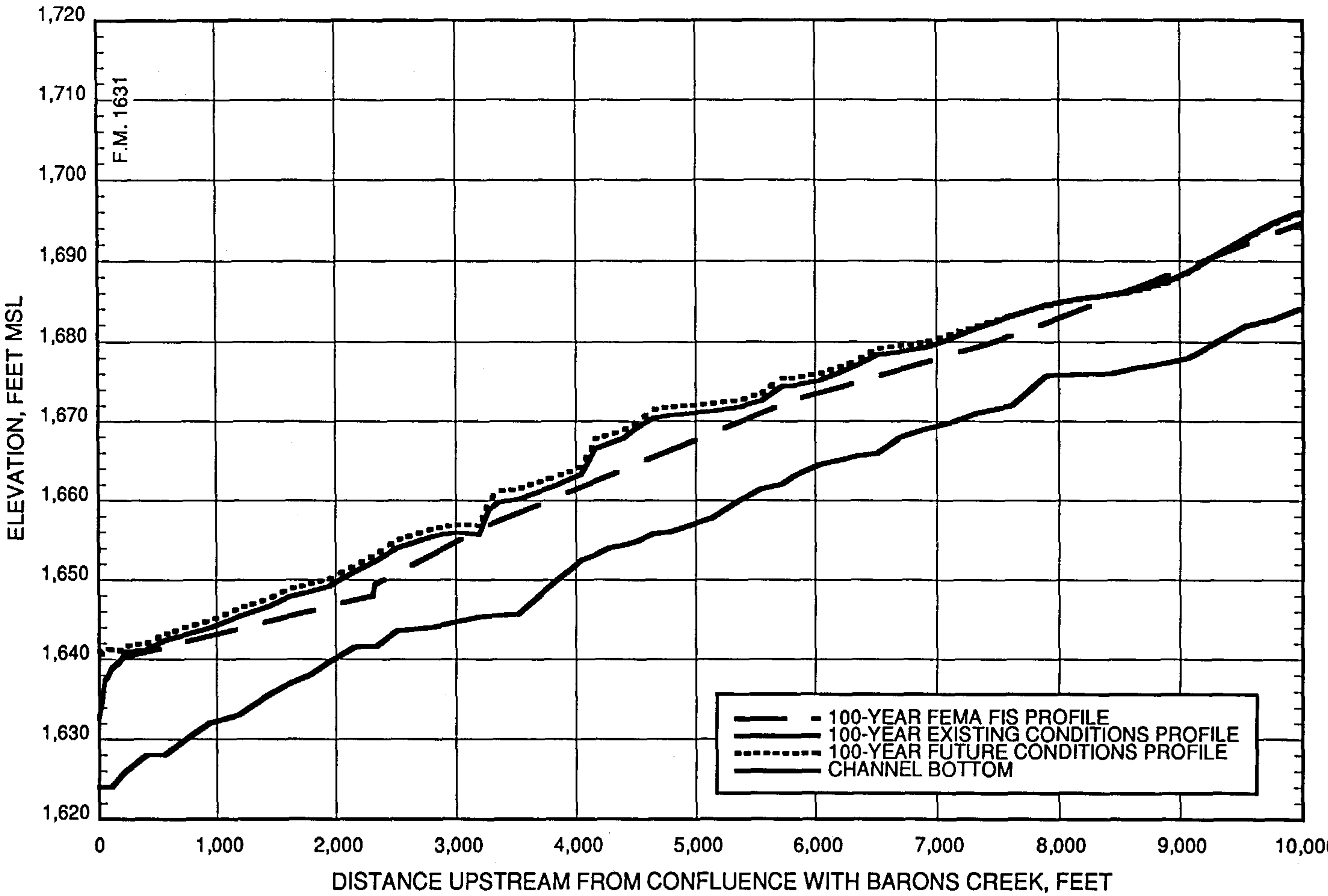

FIGURE 4-6 STREAM FB-1 100-YEAR FLOOD HEC-2 WATER SURFACE PROFILES

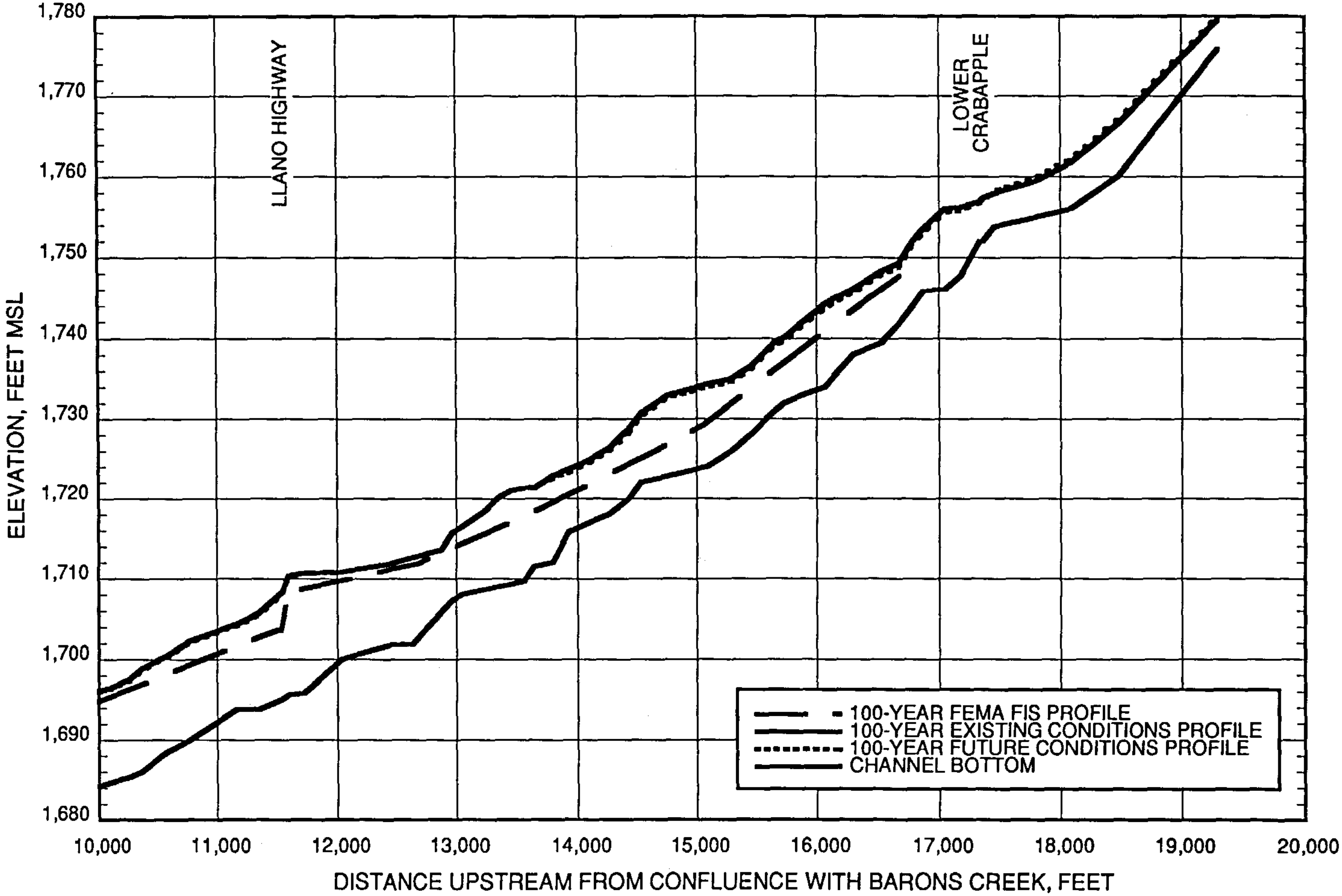

FIGURE 4-7 STREAM FB-1 100-YEAR FLOOD HEC-2 WATER SURFACE PROFILES

Of more concern are the 100-year flood level increases indicated from the original FIS results to the water surface elevations simulated with the revised HEC-2 model. Between the confluence of the stream and the Llano Highway, the flood levels from the revised model exceed those from the original FIS by as much as 7.5 feet, and typically are on the order of 3.5 feet. Fortunately, the existing land use along this reach of the stream is primarily agricultural, so it does not appear that there are any residential structures affected by the increased flood levels. Also, comparisons of the floodplain top widths simulated with the two hydraulic models do not indicate significant discrepancies, and the simulated depths of flow also are similar. Hence, it appears that differences in the topography and channel geometry used in developing the models are the primary causes of the flood level deviations. With the revised model having been developed based on current and much more detailed topographic information as compiled by the Corps, the revised model should be more accurate than the original FIS model.

Another reach of the stream where significant increases in flood levels are indicated from the original FIS results to the water surface elevations simulated with the revised HEC-2 model is through the Carriage Hills subdivision between the Llano Highway and Lower Crabapple Road. Again, maximum increases in 100-year flood levels are on the order of 7.0 feet. Certainly, this would appear significant, but when the top widths of the respective floodplains are examined, the revised HEC-2 simulation actually results in a decrease in the extent of effective FIS 100-year floodplain.

5.0 EXISTING FLOODING PROBLEMS

5.1 LOCALIZED FLOODING

Extensive efforts have been undertaken to identify existing localized flooding problems throughout the planning area. Through numerous meetings with City personnel and officials and extensive field inspections and surveys of known flooding sites, a list of specific localized areas believed to encompass the most severe existing flooding problems or those with the greatest potential for flooding has been compiled. The localized flooding problem areas previously have been identified on the vicinity map of the City of Fredericksburg in Plate 3-1. Specific flooding problem sites within the various localized flooding problem areas are identified on the map of the City in Plate 5-1, and they are listed and generally described in Table 5-1.

It should be noted that flooding in the localized problem areas generally is limited in depth to a few feet and typically is caused by either the lack of drainage facilities or inadequately sized drainage facilities. Often, this type of flooding is more of a nuisance, than it is life threatening. Still, such flooding can cause considerable property damage and can result in considerable disruption of community activities. Generally, it is primarily the stormwater runoff from the immediate drainage area of these various localized flooding problem areas that produces the excessive floodwater quantities and depths. Solutions to these types of flooding problems often involve installation of larger-capacity drainage facilities or possibly combinations of localized drainage improvements that can benefit several flooding areas. Hence, these types of flooding problems are somewhat different from those normally associated with the major creeks and streams that flow through the City where flooding may be more extensive and often requires implementation of major drainage improvements and more regional-type flood control facilities in order to achieve significant flood damage reductions.

In this Flood Protection Planning Study, sites of known or suspected localized flooding have been evaluated with respect to flooding severity (water depths) and frequency. This evaluation generally has been accomplished by performing hydraulic calculations using surveyed or measured topographic data with estimates of localized runoff quantities for the 10-year storm event. This magnitude of storm has been selected for the analyses because it is considered to be a reasonable storm event for which flood protection might be provided in many of the flood prone area of the City that are already substantially developed. The runoff quantities for the 10-year storm event, expressed as peak flow rates, associated with specific subareas within each of the identified localized flooding problem areas previously have been presented in Table 3-9 in

TABLE 5-1

LOCALIZED FLOODING PROBLEM SITES

PROBLEM SITE DESIGNATION	LOCALIZED FLOODING PROBLEM SITE	TYPE OF PROBLEM	PROBLEM LOCATION
L1	Friendship Lane	Roadway Overtopping	Low Water Crossing
L2	Friendship Lane	Street/House Flooding	South Creek Subdivision
L3	Friendship Lane	Limited Swale Capacity	Friendship Lane from S. Creek St. to S. Washington
L4	Friendship Lane	Roadway Overtopping	S. Washington
L5	Friendship Lane	Limited Swale Capacity	Friendship Lane from Channel to S. Washington
L6	Friendship Lane	Street Flooding	W. Highway and S. Adams
L7	Schubert Street	Street/Property Ponding House Flooding	Between Bowie and Acorn
L8	Cross Mountain-Milam	Street/House Flooding	N. Milam from W. Centre St. to Travis St.
L9	Cross Mountain-Milam	Street/House Flooding	W. College St. and Pecan St.
L10	Cross Mountain-Milam	Street/House Flooding	W. Centre and Edison
L11	Cross Mountain-Milam	Street/House Flooding	W. Burbank @ Avenue A
L12	Cross Mountain-Milam	Street/House Flooding	W. Burbank @ Avenue D
L13	Cross Mountain-Milam	Overflow to College-Llano Area	W. Milam from W. Burbank to Glenmoor
L14	Cross Mountain-Milam	Overflow to Trailmoor Subarea	N. Milam @ Broadmoor

TABLE 5-1

LOCALIZED FLOODING PROBLEM SITES

PROBLEM SITE DESIGNATION	LOCALIZED FLOODING PROBLEM SITE	TYPE OF PROBLEM	PROBLEM LOCATION
L15	Burbank-Llano	Street/Structure Flooding	N. Llano from Burbank to Hackberry and E. Burbank @ N. Llano
L16	North Lincoln	Street Flooding/ Possible House Flooding	N. Lincoln from Morse to W. College
L17	College-Llano	Major Street Flooding/ Potential Structure Flooding	College @ Llano
L18	College-Travis	Major Street Flooding	Sycamore
L19	College-Travis	House Flooding	Channel Upstream of Washington
L20	College-Travis	Road Overtopping	Washington and Orchard
L21	College-Travis	Potential House Flooding	Channel between Orchard and N. Pine
L22	College-Travis	Street/House Flooding	N. Pine and Travis
L23	College-Travis	Channel Erosion	North of Travis, Upstream of N. Lee
L24	College-Travis	Minor Channel Erosion	Cemetery Channel Downstream of N. Lee
L25	Trailmoor	Street Ponding	Trailmoor Upstream of Llano
L26	Trailmoor	Roadway Overtopping	Llano

TABLE 5-1
LOCALIZED FLOODING PROBLEM SITES

PROBLEM SITE DESIGNATION	LOCALIZED FLOODING PROBLEM SITE	TYPE OF PROBLEM	PROBLEM LOCATION
L27	Morning Glory-Llano	Roadway Overtopping for 100-yr Storm	Llano @ Lower Crabapple
L28	Carriage Hills	Street Flooding/High Velocities	Edgewood @ Channel
L29	Carriage Hills	Street Flooding/Curb Overtopping/House Flooding	204 & 206 Driftwood
L30	Carriage Hills	Street/House Flooding	112 & 114 Driftwood
L31	Carriage Hills	Street Flooding	N. Adams from Driftwood to just East of Crestwood
L32	Carriage Hills	Street Ponding	Frederick @ Channel Inlet
L33	Carriage Hills	Street Ponding	Tanglewood @ Channel Inlet
L34	Carriage Hills	Street Overtopping	Ridgewood @ Stone Ridge Tributary
L35	West Creek St.	Street Ponding Potential House Flooding	S. Bowie & San Antonio to Edison & W. Creek St.
L36	Old Harper Road	Street Overtopping	Low Water Crossing on Armory Road
L37	Old Harper Road	Street Overtopping	Low Water Crossing on Basse Lane
L38	Old Harper Road	Street Flooding Future Conditions	Swales along Basse Lane from Duderstadt at Low Water Crossing

TABLE 5-1

LOCALIZED FLOODING PROBLEM SITES

PROBLEM SITE DESIGNATION	LOCALIZED FLOODING PROBLEM SITE	TYPE OF PROBLEM	PROBLEM LOCATION
L39	Old Harper Road	Street Overtopping	Culverts under Duderstadt at Basse Lane
L40	Old Harper Road	Street Flooding Future Conditions	Swale along South Side of S. Bowie approximately 900 feet west of Culverts
L41	Old Harper Road	Street Overtopping Future Conditions	Box Culvert under S. Bowie 200 feet west of Post Oak Road
L42	Winfried Creek	Street Overtopping 100-yr Future Conditions	S. Milam Bridge near Whitney
L43	Winfried Creek	Erosion	Downstream of Box Culvert on post Oak Blvd. just north of Smith Road
L44	Five Points	Street Overtopping 5-yr Storm	Culverts @ Intersection of Park, Live Oak and S. Lincoln
L45	Five Points	Street Ponding	West of Five Points on Park St.
L46	Five Points	Street Ponding	E. Ufer 100 to 300 feet west of S. Lincoln
L47	Five Points	Street & Building Flooding	East of Five Points on E. Live Oak
L48	Five Points	Street Ponding and Building Flooding	Channel betwee Granite and E. Live Oak
L49	Five Points	Street Overtopping 25-yr Storm	Culvert under Granite and Ufer

TABLE 5-1
LOCALIZED FLOODING PROBLEM SITES

PROBLEM SITE DESIGNATION	LOCALIZED FLOODING PROBLEM SITE	TYPE OF PROBLEM	PROBLEM LOCATION
L50	South Adams	Potential House Flooding Future 25-yr Storm	Channel Downstream of Friendship Lane
L51	Highway-Apple	Street Flooding	Highway St. from Mesquite to S. Eagle
L52	Highway-Apple	Street Flooding and Potential House Flooding	Apple St. and Pearl St. and area between Mesquite and S. Eagle
L53	Highway-Apple	Street Overtopping	Eagle Street Low Water Crossing
L54	Dry Creek	Erosion/Backwater	Downstream of Hwy 87 @ Old Road Culverts
L55	Dry Creek	Street Overtopping/ Building Flooding	Dry Creek Tributary at Crenwelge near Gold Road

Section 3.3 of this report. For most of the specific flooding problem sites, the depth of flooding has been quantified by determining the "normal" depth of flow for the 10-year storm event. For this purpose, the Manning's uniform flow equation has been applied to specific channel or street cross sections within each of the identified localized flooding problem areas. Field surveys were conducted to measure the geometry of these channel and street cross sections. The specific sections where field surveys were performed are delineated on the map of the City in Plate 5-2. Ground and street slopes were derived from the field survey data or from the available five-foot contour topographic maps of the City.

Results from the hydraulic calculations for selected channel and street cross sections within the identified localized flooding problem areas are summarized in Table 5-2. The specific locations of these cross sections are the same as the survey cross sections identified on the map of the City in Plate 5-2, and they are referenced by the same section designations. In Table 5-2, a number of pertinent flood-related parameters are provided for each of the cross sections analyzed. These are defined below:

Localized Flooding Problem Site - Specific site identified on the map in Plate 5-1 where flooding problems occur.

Cross Section Designation - Specific section identified on the map in Plate 5-2 where field surveying has been performed to obtain geometry and elevation data.

Drainage Subarea - Specific watershed area delineated on map in Plate 3-1 that contributes Flood Flow to the Cross Section.

Conveyance Slope - Longitudinal slope of the street, channel, swale, ditch or other conveyance facility carrying the stormwater runoff.

10-Year Flood Flow - Peak flow rate for the 10-year storm event.

Height of Curb, Bank or Edge of Pavement - Vertical distance from street low point or channel flowline to the top of curb, top of channel bank or edge of pavement, channel flowline above which floodwater overflows and area flooding occur.

TABLE 5-2
STREET AND CHANNEL FLOODING DEPTHS

LOCALIZED FLOODING PROBLEM SITE (PLATE 5-1)	CROSS SECTION DESIGNATION (PLATE 5-2)	DRAINAGE SUBAREA (PLATE 3-1)	CONVEY. SLOPE	10-YR FLOOD FLOW (cfs)	HEIGHT OF CURB, BANK OR EDGE OF PAVEMENT (feet)	10-YR FLOOD DEPTH (feet)	AVERAGE VELOCITY (fps)	STREET OR CHANNEL WIDTH (feet)	FLOW TOP WIDTH (feet)
L2	LX01	Friendship	0.014	464.0	0.70	1.36	13.21	20	34
L3	LX02	Friendship	0.012	305.0	2.93	2.91	5.30	59	59
L3	LX03	Friendship	0.012	305.0	2.54	2.53	5.53	53	53
L5	LX05	Friendship	0.009	225.0	0.86	1.58	5.20	58	58
-	LX07	Friendship	0.015	146.0	3.51	2.67	9.26	8	8
L8	LX11	Milam D/S	0.007	101.6	0.87	1.39	1.34	64	158
L8	LX12	Milam D/S	0.007	101.6	0.66	0.73	2.59	64	110
L9	LX13	Pecan	0.007	151.6	0.00	0.86	6.08	28	38
L9	LX14	Pecan	0.005	151.6	0.46	0.78	4.50	67	67
L9	LX15	Pecan	0.005	151.6	0.42	0.91	4.27	87	87
L10	LX16	Pecan	0.017	151.7	0.32	0.73	7.16	65	66
L10	LX17	Pecan	0.004	15.0	0.71	0.41	2.46	48	35
L10	LX18	Pecan	0.004	15.0	0.59	0.53	2.33	48	42
L10	LX19	Pecan	0.004	30.0	0.79	0.63	2.06	48	75
L11	LX20	Avenue A	0.024	91.7	0.75	0.98	4.82	18	38
L11	LX21	Avenue A	0.024	91.7	0.75	0.94	7.49	7	38
L12	LX22	Avenue D	0.008	32.7	1.53	0.72	6.91	9	9
-	LX23	Cross Mtn.	0.028	16.9	0.98	0.25	7.79	9	9
-	LX24	Cross Mtn.	0.031	16.9	0.65	0.31	6.27	39	17
L13	LX25	Milam U/S	0.013	94.3	2.50	1.45	5.77	42	24
L15	LX26	Llano	0.004	91.4	0.42	0.72	3.68	74	74
L15	LX27	Llano	0.006	91.4	0.92	0.57	5.30	42	42

TABLE 5-2
STREET AND CHANNEL FLOODING DEPTHS

LOCALIZED FLOODING PROBLEM SITE (PLATE 5-1)	CROSS SECTION DESIGNATION (PLATE 5-2)	DRAINAGE SUBAREA (PLATE 3-1)	CONVEY. SLOPE	10-YR FLOOD FLOW (cfs)	HEIGHT OF CURB, BANK OR EDGE OF PAVEMENT (feet)	10-YR FLOOD DEPTH (feet)	AVERAGE VELOCITY (fps)	STREET OR CHANNEL WIDTH (feet)	FLOW TOP WIDTH (feet)
L16	LX28	N. Lincoln	0.008	148.8	0.67	0.60	7.19	39	39
L17	LX29	College	0.006	203.6	1.00	1.00	6.55	51	53
L18	LX30	Travis	0.005	310.5	1.61	1.31	9.01	31	31
L19	LX31	Travis	0.004	320.0	2.20	3.73	3.74	24	68
L21	LX33	Travis	0.004	320.0	2.40	3.05	5.35	24	27
L22	LX34	Travis	0.004	337.4	1.55	1.46	6.73	43	43
L22	LX35	Travis	0.008	337.4	0.47	1.34	6.77	81	81
L22	LX36	Travis	0.008	337.4	0.83	1.23	7.33	84	84
L24	LX39	Travis	0.019	337.4	3.10	2.56	7.18	24	28
L25	LX40	Trailmoor	0.010	152.2	0.83	0.72	8.30	40	33
L27	LX44	Morn. Glory-Llano	0.014	464.9	3.00	1.74	18.25	21	17
L28	LX46	Edgewood	0.015	128.1	1.40	0.74	11.63	15	15
L28	LX47	Edgewood	0.015	128.1	1.38	0.75	11.63	15	15
L29	LX48	Driftwood N.	0.012	211.2	0.83	0.62	9.31	39	39
L29	LX49	Driftwood N.	0.009	211.2	0.90	0.67	8.62	38	38
-	LX50	Driftwood N.	0.009	211.2	0.70	0.71	6.76	43	72
L30	LX51	Driftwood S.	0.003	198.0	0.75	1.23	4.34	39	73
L30	LX52	Driftwood S.	0.003	198.0	0.68	1.15	4.46	38	70
L31	LX53	N. Adams	0.001	258.2	0.63	1.64	2.99	39	90
L31	LX54	N. Adams	0.001	258.2	0.68	1.64	3.12	39	84
L31	LX55	N. Adams	0.011	347.1	1.70	1.38	11.39	40	34
L32	LX56	Frederick	0.014	21.2	0.83	0.30	6.24	11	11

TABLE 5-2
STREET AND CHANNEL FLOODING DEPTHS

LOCALIZED FLOODING PROBLEM SITE (PLATE 5-1)	CROSS SECTION DESIGNATION (PLATE 5-2)	DRAINAGE SUBAREA (PLATE 3-1)	CONVEY. SLOPE	10-YR FLOOD FLOW (cfs)	HEIGHT OF CURB, BANK OR EDGE OF PAVEMENT (feet)	10-YR FLOOD DEPTH (feet)	AVERAGE VELOCITY (fps)	STREET OR CHANNEL WIDTH (feet)	FLOW TOP WIDTH (feet)
L33	LX57	Tanglewood	0.014	27.6	0.83	0.35	6.87	11	11
L38	LX62	Old Harper	0.013	199.2	1.00	1.10	3.96	22	22
L42	LX66	Winfried	0.017	1006.9	5.00	4.71	9.79	35	33
L44	LX72	Five Points	0.190	22.2	NA	0.39	4.42	32	32
L44	LX73	Five Points	0.019	22.2	0.78	0.46	4.78	27	27
L47	LX74	Five Points	0.009	92.0	0.00	1.00	3.65	37	37
L47	LX75	Five Points	0.009	92.0	0.50	1.63	3.60	38	38
L47	LX76	Five Points	0.009	92.0	0.83	0.91	4.44	49	49
L45	LX77	Five Points	0.014	22.2	2.88	1.32	4.30	8	8
L45	LX77	Five Points	0.014	51.3	1.46	0.82	6.25	20	20
L45	LX78	Five Points	0.014	22.2	0.90	0.79	2.87	23	23
L45	LX78	Five Points	0.014	51.3	0.42	0.83	3.82	29	29
L48	LX81	Five Points	0.019	124.0	2.30	1.94	2.52	60	51
L50	LX84	S. Adams	0.007	127.5	2.00	1.15	4.00	42	34
-	LX88	Apple	0.013	87.8	2.50	1.03	4.57	48	26
-	LX90	Apple	0.013	87.8	2.50	1.50	5.52	20	16

Notes: 1) D/S - Downstream
2) U/S - Upstream
3) Ch. - Channel
4) 1+00 - Survey Station
5) Numbers before street names are adresses
6) NA - Not applicable

10-Year Flood Depth - Depth of floodwater above street low point or channel flowline.

Average Velocity - Average velocity of floodwater flowing in street or channel.

Street or Channel Width - Width of street or channel conveying floodwater.

Flow Top Width - Width of floodwater surface within or outside of street or channel.

The extent of flooding at each cross section has been evaluated by comparing the "10-Year Flood Depth" to the "Height of Curb, Bank or Edge of Pavement" to determine if floodwater overflows out of a conveying street or channel occur and, thereby, cause potential flooding of adjacent properties. Also, if the calculated "Flow Top Width" at a particular section significantly exceeds the available "Street or Channel Width", it also is likely that potential flooding of adjacent properties is occurring. The "Average Velocity" of the flowing floodwater has been examined at each section to assess whether or not the momentum of the flowing floodwater might cause street curbs and channel banks at corners and bends to be overtopped and, thereby, contribute to the potential flooding of adjacent properties.

In some cases, other hydraulic and hydrologic calculations have been performed, including additional HEC-1 runoff simulations, to provide additional information when necessary. Also, some of the localized flooding problem areas have streets with nearly flat or negative slopes which preclude the performance of meaningful uniform flow hydraulic calculations. In these cases, the severity of the flooding problems has been subjectively examined based on such factors as the relative elevations of threatened structures and flood conveyance systems, the general volume of traffic that might be disrupted during flooding events, and/or the quantity of runoff flowing through a potential flooding problem site. Where necessary, the hydraulic capacity of roadway culverts has been analyzed using standard culvert hydraulics procedures similar to those described in the Texas Highway Department's (now Texas Department of Transportation) Drainage Manual (1985).

Following is a discussion of flooding conditions within each of the localized flooding problem areas. Where appropriate, the specific flooding problem sites are referenced in accordance with the site designations listed in Table 5-1. These sites also are identified on the map of the City in Plate 5-1.

5.1.1 Friendship Lane Drainage

One of the most significant localized flooding problems is along Friendship Lane in the southern part of the City. The watershed that contributes stormwater runoff to this area originates in the vicinity of Schneider Hill southwest of the downtown area and generally extends eastward along Friendship Lane. Runoff from the watershed tends to concentrate east of U. S. Highway 87 (South Washington Street) and flow along much of Friendship Lane. For most storms, Friendship Lane becomes impassable at the low water crossing between South Creek Street and South Eagle Street (Site L1).

Based on results from the HEC-1 runoff model of the Barons Creek basin, the peak flow rate for the 10-year flood at the low water crossing (Site L1) has been determined to be 578 cubic feet per second (cfs). East of South Washington Street, the swale along the north side of the Friendship Lane roadway (Site L3) has very limited floodwater-carrying capacity, and stormwater tends to spill northward into a natural low area. This stormwater then must flow through the South Creek subdivision through a shallow (8.5 inches deep), relatively narrow (16 feet wide) trapezoidal channel. The floodwater-carrying capacity of this channel is less than the peak flow rate of the 2-year storm, and during the occurrence of larger storms (5- and 10-year rainfall events), floodwaters threaten the adjacent houses and cause streets within the subdivision to be impassable (Site L2). Additionally, although the drainage swales on both sides of Friendship Lane upstream of the South Creek subdivision have sufficient capacity to convey about the 10-year flood flow, the numerous driveway crossings have undersized culverts that force the water out of the swales and over the road or onto the adjacent land.

The box culvert (4' x 4') under U. S. Highway 87 (South Washington Street) also is undersized, with capacity for conveying floodwaters less than that produced by the 10-year storm. Larger storms cause floodwaters to flow over the highway and become impounded upstream (Site L4). Near the upstream end of the watershed, at West Highway Street just west of South Adams Street, a large culvert discharges stormwater onto West Highway Street from Schneider Hill and State Highway 16 (10-Year Flow = 127 cfs). This concentrated flow crosses both Highway Street and South Adams, posing a significant traffic hazard (Site L6), and then discharges into a channel leading southeastward toward Friendship Lane, where there is limited floodwater-carrying capacity (Site L5).

Overall, the Friendship Lane is characterized as a significant localized flooding problem area, even for storms as small as the 2-year event.

5.1.2 Schubert Street Ponding

A natural low-lying area and a surrounding depression exists on Schubert Street between Bowie and Acorn Streets northwest of the downtown area (Site L7). The main portion of the depression is located south of Schubert Street on two vacant town lots (1/2-acre each). It has been reported that historically a natural pond existed at this location and that it was filled as the area developed for residential use. The existing depression collects and stores stormwater runoff from the surrounding watershed, which encompasses about 28 acres. Preliminary calculations indicate that the existing low-lying area naturally (predeveloped watershed) would have flooded up to about elevation 1,732.4 feet msl (above mean sea level) during the occurrence of a 100-year storm with a 12-hour duration. Ponding of stormwater in this area now has been partially alleviated by an 18-inch storm drain and inlets that were installed by the City. However, frequent ponding of stormwater still occurs since the discharge capacity of this storm drain is only about 9 cfs, and the peak flow rate of the two-year storm is on the order of 35 cfs. With the existing storm drain, the 100-year, 12-hour storm causes stormwater runoff to pond in the depression area to an elevation just over 1,731 feet msl. This elevation would be close to the finished-floor elevations of adjacent residential structures, and would result in up to two feet of floodwater over the Schubert Street roadway.

Concerns have been expressed by the owners of the remaining vacant lots in the depression area at Schubert Street that the current ponding of stormwater runoff prevents the construction of buildings on these lots. Of course, construction of buildings on these lots would require filling of the depression, which, in turn, would increase the flooding levels on both the currently vacant lots and the adjacent lots with existing houses. Increased flood damages very likely would result.

The Schubert Street ponding is considered to be a significant localized flooding problem area; although, the problem involves primarily the existing vacant lots.

5.1.3 Cross Mountain - Milam Drainage

This area is generally bounded by North Milam Street to the east, Town Creek to the south and west, and Cross Mountain to the north. The streets, including Cross Mountain Drive, Avenue D, Avenue A, Pecan Street and Milam Street, are the primary drainageways for conveying stormwater runoff in this area. Because of limited street floodwater-carrying capacities, relatively large drainage areas and flat ground slopes, there is some interaction and cross-over of floodwater flows between adjacent streets. Significant street and some house flooding occurs in the vicinity of the lower segments of Milam and Pecan Streets near their intersections with College and Centre Streets (Sites L8 & L9). Relatively large drainage areas for both Milam and Pecan Streets contribute runoff to these low, flat areas (64 and 82 acres, respectively). At 604 Milam (Site L8), the 10-year flood flow has been determined to be approximately 102 cfs, which produces a water depth on the order of 1.4 feet. At Pecan and West College Streets (Site L9), the 10-year flood flow is about 152 cfs. There is no curb on the east side of Pecan Street at this location and there is significant potential for flooding of the residences. Even with a curb, there would not be sufficient floodwater-carrying capacity in the street. On West College Street, the depth of the 10-year flood flow exceeds the curb height. Another problem occurs at the intersection of Edison and Centre Streets (Site L10). There is no curb on the east side of Edison just south of Centre and the 10-year flood depths are on the order of 0.7 to 0.9 feet. Some stormwater flow spills over to Milam Street down Centre Street at this location.

Stormwater runoff from a portion of the Cross Mountain residential area flows down Avenue A to Burbank Street (Site L11). At this point, the natural slope of the land generally takes stormwater flows south to the existing flooding problem areas along Pecan Street. There is a small curb-cut on the south side of Burbank Street that allows these flows to proceed southward down a grassed channel. The estimated 10-year flood depth in Burbank just upstream of the curb-cut is approximately 1.6 feet, and the corresponding depth in the downstream channel is on the order of one foot. This depth exceeds the curb height at the edge of the channel. Because of the depth of flow in Burbank Street and the inlet control limitation on flow through the curb-cut, some of the stormwater flows down Burbank Street to the northwest toward Avenue D.

A potential flooding problem exists at the concrete channel into Town Creek at the western end of Burbank Street near Avenue D (Site L12). This channel section is about nine feet wide and about one foot deep. Although the channel itself can carry the 10-year flood flow, the inlet to the channel limits the inflow and, thereby, forces stormwater to flow over the street curb to reach the channel. The overflows of stormwater from the Avenue A drainage area, flowing down Burbank Street, make this condition worse. This could result in flooding of the adjacent homes, especially during larger storms, i. e., greater than the 10-year event. Another similar channel into Town Creek exists at the end of Cross Mountain Drive where it intersects Avenue D. However, because of a smaller drainage area, less stormwater runoff flows to this point, and with the inlet to this channel being approximately 13 feet wide, there does not appear to be a potential problem at this location, even for the 100-year storm.

Some stormwater runoff from the area between Cross Mountain and North Milam Street normally flows down Milam all the way to Town Creek. However, for higher intensity storms, some of this stormwater spills over to the east and contributes to flooding problems in the College-Llano drainage area. These spill-overs generally occur along Milam Street from Burbank Street north to Glenmoor Street (Site L13), with some additional spill-overs at the intersection of Burbank and Milam Streets (Site L14). These spill-over waters eventually flow to the Trailmoor Drive area and contribute to the existing flooding problems there.

5.1.4 Burbank - Llano Drainage

This drainage area includes approximately 40 acres west of North Llano Street and north of Hackberry Street and an additional 18 acres east of North Llano Street, including the drainage to North Lincoln Street upstream and north of College Street. The primary flooding problem site within this area is the portion of North Llano Street between Burbank and Hackberry Streets (Site L15), where all of the stormwater runoff from the western 40 acres is concentrated within the street section and sometimes overtops the curb. The 10-year flood flow at this location (about 90 cfs) produces water depths that overtop the curb along North Llano by about 0.3 feet, and the associated velocity is nearly four feet per second (fps). These conditions are especially dangerous where the floodwaters cross North Llano Street and flow to the east. At the entrance to Burbank Street, the flow has a velocity over five feet per second, and as the stormwater turns to flow down Hackberry Street, the depth of the flow is about one foot. These conditions produce a dangerous situation for a major roadway, and they also pose a flooding threat to adjacent houses and businesses.

5.1.5 North Lincoln Drainage

Downstream of the Burbank-Llano flooding problem area is the North Lincoln problem area. This area encompasses an additional 52 acres of watershed. Along North Lincoln Street, the 10-year flood flow is nearly 150 cfs. Although uniform flow calculations indicate that this quantity of flow is just barely conveyed within the existing street section, irregularities in ground slopes and section geometry along the street probably result in overtopping of the curb a some locations (Site L16). The 10-year flood depth of 0.6 feet in the street, with a velocity of over seven feet per second, represents a relatively hazardous situation and would make crossing the street in a vehicle difficult, at best. For storms greater than the 10-year event, some homes along the street also would be threatened with flooding.

5.1.6 College - Llano Drainage

The College-Llano flooding problem area encompasses about 148 acres of contributing watershed that produces a concentrated 10-year flood flow of about 200 cfs that discharges across Llano Street at its intersection with College Street (Site L17). The depth associated with this flow is on the order of one foot, and the velocity is about 6.5 fps. This depth of flow is just at the curb height along College Street. Because of the rapid expansion and contraction of the flow as it crosses Llano Street, the actual depths may reach as much as 1.7 feet at some points including along the eastside curb of Llano Street. The 25-year flood flow produces depths well above (>0.5 feet) the curb that could cause floodwaters to reach the adjacent residential and commercial structures.

5.1.7 College - Travis Drainage

This area is downstream of the College-Llano, Burbank-Llano and North Lincoln flooding problem areas; therefore, it receives very high inflows of stormwater runoff that must be conveyed primarily through the streets and some shallow grass/earth channels. The total drainage area contributing runoff encompasses about 340 acres, including the North Milam area that very likely contributes floodwater spill-overs. The 10-year flood flows range from 310 to 340 cfs from the intersection of East College and

North Lincoln Streets to the eastern end of Travis Street. Water depths produced by the 10-year storm in Sycamore Street south of College Street (Site L18) are on the order of 1.3 feet, with velocities about nine feet per second. This flow is contained within the street, however, by the existing high curbs, which are approximately 1.6 feet in height. The stormwater flowing down Sycamore Street enters a grass-lined channel that traverses in the direction of the intersection of Washington and Orchard Streets. The 10-year flow depth in this channel is estimated to be on the order of 3.7 feet, which exceeds of the banks of the channel (Site L19).

Water from the channel discharges through three culverts under Washington Street. The combined capacity of these culverts is equivalent to about the two-year flood flow; consequently, the 10-year flood flow would overtop Washington Street by more than 0.5 feet. The limited conveyance capacity of the channel and culverts in this area creates the potential for flooding of nearby homes by storms slightly greater than the 10-year event (Site L20).

The stormwater discharges from the culverts under Washington Street flow across Orchard Street into a channel with tree-lined banks. The 10-year flood flow in this channel produces depths on the order of three feet, which is about 0.6 feet above the top of the channel banks (Site L21). These floodwaters then discharge into North Pine Street, where they are contained within the existing high curbs, similar to those along Sycamore Street. From North Pine Street, the floodwaters discharge into East Travis Street, where they flow down a channel-like depression along the north side of the street, but within the curb. The 10-year flood flow overtops the curb along this street and reaches to within 0.5 feet (elevation) of the adjacent houses (Site L22). Downstream of Elk Street, the Travis Street floodwaters discharge into a grass/earth channel. Just upstream of North Lee Street, this channel is significantly eroded due to the high flood flows and velocities caused by the runoff from the upper watersheds (Site L23). Floodwaters in the channel pass beneath a bridge/culvert at North Lee Street and then, finally, into a grass/earth channel through the City Cemetery to Stream FB-1. Some erosion is occurring within the channel through the cemetery (Site L24).

Stormwater discharges on the order of 300 cfs through and across residential streets with water depths greater than one foot are considered a major flooding problem. Most of these streets are impassable with the occurrence of less than the one-year storm event, and there is potential for flooding of residences by storms greater than about the

10-year event.

5.1.8 Trailmoor Drainage

The Trailmoor Drainage flooding problem area encompasses about 85 acres and lies primarily west of Trailmoor Street, east of North Milam Street and north of Nimitz Street. Stormwater runoff from several streets is concentrated in Trailmoor Street at its intersection with North Adams Street. The specific flooding problem site is along about 200 feet of a flat section of Trailmoor just northwest of North Llano Street (Site L25). Flood backwater conditions along this segment of Trailmoor Street are caused by the flow restriction created by the inlet to the existing culverts under North Llano Street. Even the two-year storm event produces flood backwater conditions on Trailmoor that result in overtopping of the North Llano roadway (Site L26).

5.1.9 Morning Glory - Llano Drainage

The concrete-lined channel adjacent to Lower Crabapple Road and the culverts under North Llano Street at Lower Crabapple Road have been analyzed to evaluate their floodwater-carrying capacities. While the channel is capable of conveying the 100-year flood flow, inlet restrictions to the box culvert under North Llano Street cause overtopping of the roadway during the 100-year storm (Site L27).

5.1.10 Carriage Hills Drainage

Significant localized flooding problems exist in the drainage area that lies generally north of the Llano Highway (Highway 16) and south and east of Lower Crabapple Road. The greatest number of reported drainage problems are located along Edgewood Drive and Driftwood Drive in the Carriage Hills subdivision. A concrete-lined channel conveys stormwater runoff through this subdivision from the currently undeveloped area west of Edgewood Drive to Driftwood Drive. Although this channel has sufficient capacity for conveying the 10-year flood flows (fully-developed watershed conditions), flooding problems occur at the inlets and outlets of the channel segments (Sites L28 & L29). Channelized flood flows from the west discharge at over 11 feet per second into Edgewood Drive. Because of the abrupt change in section geometry at this location, the inlet to the channel on the opposite side of the street appears to control the flow, which forces some of the stormwater over the curb (Site L28).

Similar conditions occur where the channel discharges into Driftwood Drive, at which point the additional stormwater runoff flowing down Driftwood Drive from the north causes substantial overtopping of the curb and flooding of adjacent houses (Site L29). Although uniform flow calculations indicate that the 10-year flood flow could be contained within the street section, the unsteady nature and high energy of the flow are sufficient to push the water over a half a foot above the curb.

Downstream (south) on Driftwood Drive, additional drainage problems exist through a flat section of the roadway (Site L30). Because of the flat slope (0.003 feet per foot), the 10-year flood depths exceed the curb height by about 0.5 feet, and the flow spreads to the adjacent houses on the east side of the street. Stormwater flows produced by storms equal to or greater than the 10-year event will cause some flooding of residential structures.

Additional runoff flowing into the intersection of Driftwood Drive and North Adams Street causes the 10-year flood depths to exceed 1.6 feet along North Adams Street (Site L31) and to pond to about 2.5 feet at the inlet to the existing grass channel between North Adams Street and the Llano Highway (Highway 16). The grass channel appears to have sufficient floodwater-carrying capacity for the 10-year storm, except for the flow limitations at the inlet.

Other localized flooding problems in this area occur along the existing 11-foot wide concrete curb channel (10-inch curb height) that conveys stormwater flows from Frederick Road to Tanglewood Drive and thence to Stream FB-1. Inlet control conditions limit the inflows into these channels. This may cause some ponding at the inlets on both of these streets (Sites L32 & L33).

Another localized drainage problem in this area relates to the culverts under Ridgewood Drive where the tributary from the Stone Ridge development crosses in route to Stream FB-1. The three existing 30-inch pipes are not capable of conveying the 10-year flood flow (fully-developed watershed conditions) without causing overtopping of the roadway. The 100-year flood flow would overtop the roadway by approximately 1.5 feet, with most of the flow passing over the road. The limited channel capacity of this tributary through the Carriage Hills subdivision also is of concern with regard to flooding of adjacent houses.

5.1.11 West Creek Street Drainage

This flooding problem area generally encompasses a subwatershed bounded by West Main Street on the north, Peach Street on the south, Orange Street on the east, and Acorn Street on the west. The basic flooding problem in this area is street ponding caused by the extremely flat ground slopes. Specific flooding problem sites occur along South Bowie Street from San Antonio Street to West Creek Street and along San Antonio Street from South Bowie Street to South Edison Street (Site L35). Curb overflows of stormwater along both South Bowie Street and San Antonio Street could impact houses in the block bounded by to South Bowie, San Antonio, Edison and West Creek Streets.

5.1.12 Old Harper Road Drainage

This area lies generally southwest of Barons Creek and south of Old Harper Road (also known Basse Road and South Bowie Street) and Armory Road. Currently, this area is undeveloped, and stormwater flows drain northward across both roads at several low water crossings. Also, the existing swales along Old Harper Road have the capacity to convey close to the 10-year flood flow (fully-developed conditions). There is a single 24-inch corrugated metal pipe under a private drive marked as Duderstadt Lane, and a 4'x2' box culvert under the South Bowie portion of Old Harper Road near Post Oak Road. These pipes and culverts appear to be undersized for handling future flood flow conditions (Sites L36 through L41). Depending on the extent of upstream development and the types of drainage facilities constructed, future flood flows are projected to be as much as 40 percent greater than existing flows.

5.1.13 Winfried Creek Drainage

This area encompasses a large, well defined watershed south of Barons Creek. The drainage area covers nearly 470 acres of relatively steep terrain above the bridge at South Milam Street. Currently, most of this area is undeveloped. Most of the creek crossings have sufficient capacity under existing conditions and also generally would convey the 10-year flood flows under fully-developed watershed conditions. One concern is the bridge at South Milam Street (Site L42). For existing watershed conditions, the 100-year flood flow passes through the bridge without overtopping.

However, for fully-developed watershed conditions, there would be some overtopping. This does not appear to threaten any houses; although, the increase in future flood flows is dependent on the level of development that occurs and the types of drainage facilities constructed in the area. Some erosion around other bridges and culverts also has been observed (Site L43).

5.1.14 Five Points Area

There is a significant existing drainage problem in the vicinity of the Five Points intersection. This area is located at the intersection of Park Street, South Lincoln Street and East Liveoak Street. The 10-year flood flow entering this intersection is approximately 114 cfs. For conveyance of stormwater through this intersection, there are two sets of culverts (two storm drains from Park Street and one box culvert from Liveoak Street) with a combined capacity equal to approximately the five-year flood flow (Site L44). This limitation forces water over the roadways and causes ponding on Park Street (Site L45). Floodwaters from Park Street overflow into the park area to the north and flow toward Ufer Street through a grass swale. Fairly significant ponding of floodwaters occurs on Ufer Street at an existing low point (Site L46), in part because of an undersized culvert on private property just north of the street. Flow that does pass through the box culvert at the Five Points intersection discharges into a swale downstream along Liveoak Street. In this swale, the depth of the 10-year flood flow exceeds the elevation of the building to the northwest of Live Oak (Site L47). These floodwaters combine with runoff from the street to the south of Live Oak (Walnut Street) and then flow through a small swale northward toward Granite Avenue. This swale has approximately a 10-year flood flow capacity (Site L48). This limitation, combined with the close proximity of the adjacent buildings, results in frequent flooding of area properties. Stormwater discharges from the swale area then enter a culvert under the Granite and Ufer intersection (Site L49). This culvert discharges into Barons Creek. The inlet capacity of this culvert is sufficient to handle approximately the 10-year flood flow from the upstream drainage area. The flooding in the Five Points area is considered a significant problem with respect to streets and structures.

5.1.15 South Adams Drainage

This area lies south of Schneider Hill and is generally located south of Highway 16, west of South Adams Street, east of Stadium Drive and north of Billie Drive. Although this area technically is outside the planning area, its drainage conditions have been evaluated since part of the Schneider Hill subwatershed, which is in the planning area, would contribute stormwater runoff to this subwatershed under natural conditions. However, due to various drainage pipes and channels that are in place today, stormwater from the Schneider Hill subwatershed now discharges to the Friendship Lane drainage area. Runoff from the South Adams subwatershed discharges through several culverts under the west end of Friendship Lane into an grass-lined channel (Site L50). This channel has sufficient capacity for conveying the 10-year flood flow, with about one foot of freeboard. The proximity of a house just east of the channel on Friendship Lane raises some concern with respect to flooding at higher flow conditions.

5.1.16 Highway - Apple Drainage

The area lies north of Highway Street and west of U. S. Highway 290. The Highway Street and Apple Street drainage areas are fairly long and end along very flat street sections near South Eagle and Pear Streets. Highway Street has a drainage area of about 75 acres, with a 10-year flood flow of 105 cfs. At high flows, some of this water spills out of the roadway and flows southward into the Friendship Lane drainage area either through the South Creek-Bluebonnet-Columbus Streets system or through a small drainageway that discharges into South Eagle Street. The primary areas with street flooding problems are along Apple Street (Site L52) and Highway Street (Site L51) from South Mesquite Street to South Eagle Street. There also is some potential for flooding of residential structures along Peach Street between Apple and Highway Streets. The floodwater spill-overs from Highway Street cause additional flooding problems along South Eagle Street at the low water crossing just south of Highway Street (Site L53). Runoff from the Apple Street drainage area discharges under U. S. Highway 290 through a box culvert into a grass-lined channel and through another set of culverts under Crenwelge Drive. The floodwater-carrying capacities of this channel and the associated culverts are well in excess of the 10-year flood flow.

5.1.17 Dry Creek Drainage

This area encompasses a well defined watershed with two major tributary channels. It is located northwest of the City near U. S. Highway 87 and Bob Moritz Drive. The main channel does not appear to have any significant flooding problems; however, there is a old bridge just downstream of U. S. Highway 87 (Site L54) that is causing significant channel erosion and may cause some backwater problems for the culverts under U. S. Highway 87. On the western tributary, there is an existing culvert under South Crenwelge Road near its intersection with Gold Road (Site L55). The 10-year flood flow causes overtopping of this road, which could result in flooding of adjacent businesses.

5.2 STREAM FLOODING

Areas of potential stream flooding have been analyzed by first identifying reaches where significant increases in flood levels are indicated based on comparisons of the simulated 100-year flood results from the revised HEC-2 models developed in this study with those previously determined during the original Flood Insurance Study (FIS) for the City of Fredericksburg. Floodplain widths and boundaries based on the HEC-2 modeling results have been examined for these reaches to determine if the indicated flood level rises translate into meaningful floodplain changes. In this process, the effective FIS 100-year floodplain boundaries have been plotted on base maps of the City of Fredericksburg. The revised floodplain boundaries based on the revised HEC-2 results also have been added to these maps to delineate areas of increased or decreased flooding.

5.2.1 Barons Creek

As discussed in Section 4.2, there are several reaches along Barons Creek where the 100-year flood profile plots (Figures 4-2 and 4-3) indicate significant increases in the flood levels from the updated HEC-2 model with respect to those previously determined in the original FIS. These areas of potentially increased flooding are discussed in the following paragraphs.

5.2.1.1 Wastewater Treatment Plant to Goehmann Road

Presented in Figure 5-1 is a map of this reach of Barons Creek with the 100-year floodplains delineated based on the effective FIS and based on the results from the revised HEC-2 model of this portion of the creek. For the revised floodplain, only those boundaries that are different from the effective FIS floodplain boundaries are plotted. Both sets of floodplain boundaries generally reflect flood flows corresponding to existing watershed and land use conditions. As illustrated, even with the higher flood

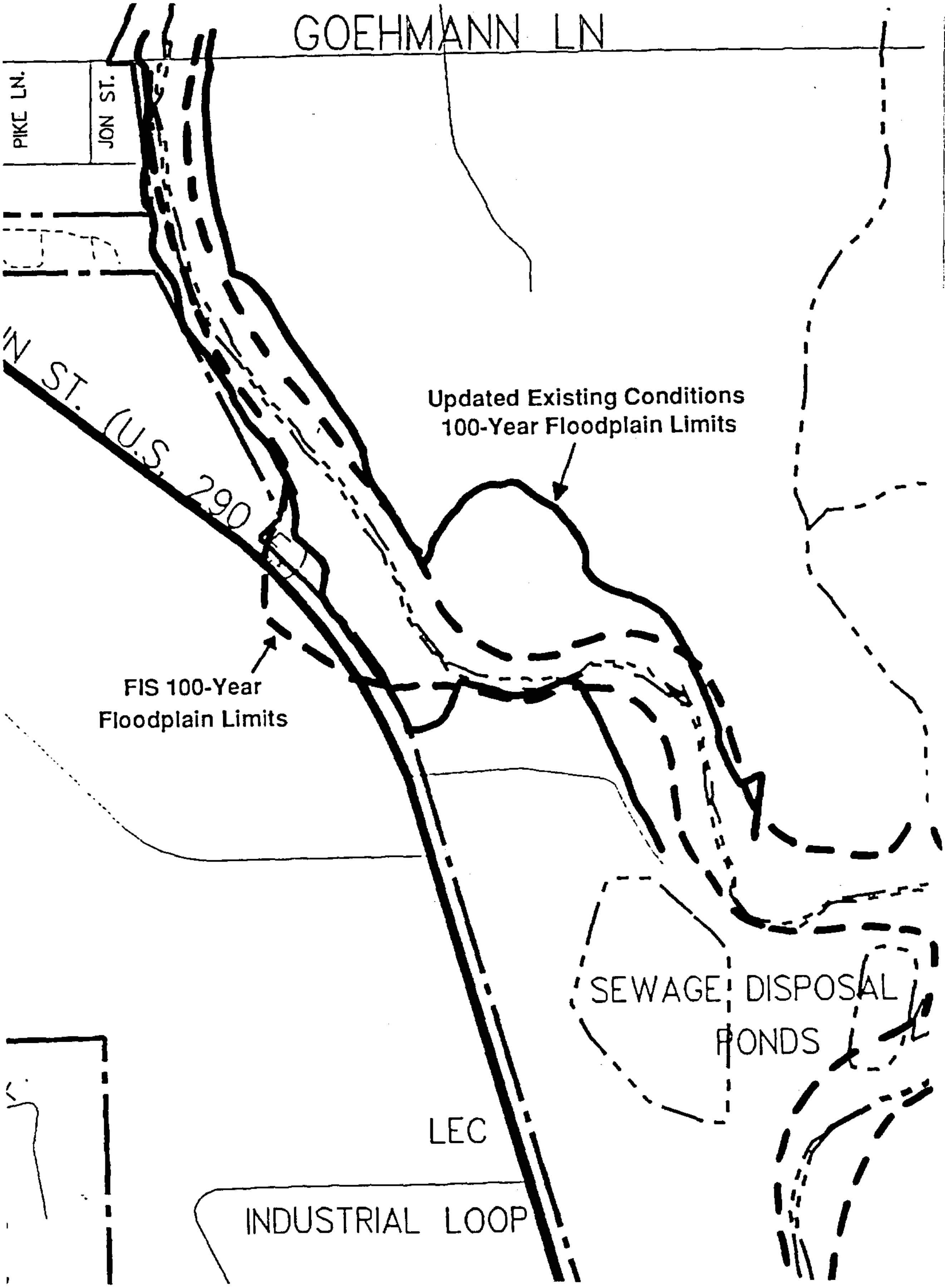

FIGURE 5-1 BARONS CREEK 100-YEAR FLOODPLAIN BOUNDARIES UPSTREAM OF CITY WASTEWATER TREATMENT PLANT

levels simulated with the revised HEC-2 model, the revised floodplain boundaries along the west bank are not significantly different from the FIS floodplain boundaries. It is interesting to note that the revised 100-year floodplain does not encompass the U. S. Highway 290 roadway, as does the effective FIS floodplain. There are some additional areas included in the revised floodplain along the left (east) bank of the creek that are not contained within the FIS floodplain. The inclusion of these areas results primarily from better definition of the overbank topography in the revised HEC-2 model and the availability of more detailed topographic information from the Corps of Engineers' Gillespie County flood insurance study for establishing the floodplain boundaries. Based on examination of 1994 aerial photographs of this reach of Barons Creek, none of these modifications in the floodplain boundaries appear to impact any structures along the creek.

5.2.1.2 Upstream of F. M. 1631

Based on an analysis of the actual locations of the 100-year flood level increases in this area as discussed previously, i. e., 1.4 feet of increase in the revised HEC-2 model results compared to those from the effective FIS, and examination of 1994 aerial photographs, it has been determined that there are no apparent flooding impacts on structures along this reach of the creek. In the vicinity of the one house that has been identified as being potentially impacted, the increase in the revised 100-year flood level is only about one-half foot, and this is not enough to cause any flooding of the structure.

5.2.1.3 Lincoln to Adams Reach

Results from the HEC-2 hydraulic modeling for the reach of Barons Creek from just downstream of Lincoln Street upstream to Adams Street indicate an increase in the 100-year flood level of about 0.8 feet from the effective FIS flood elevation to the levels simulated with the revised model under existing watershed and land use conditions. Despite these increased flood levels, the width of the floodplain changes very little from that depicted on the effective flood insurance maps. This is due primarily to the steep banks that characterize the channel and floodplain through this reach. There is one section about 700 feet upstream of Lincoln Street which does indicate an increase in the floodplain width of about 22 feet. Because of the proposed construction of a walk bridge across the creek at Llano Street, this section was resurveyed in 1996 as part of this study. The resurveyed section has been used to replace an existing section in the original FIS model; hence, it is not surprising that a change in the 100-year flood levels and floodplain boundaries in this vicinity has occurred. The changes in flood levels and topography through this reach are reflected in some small amounts of additional floodplain area on the west bank of Barons Creek.

Based on an analysis of 1994 aerial photographs of this reach of Barons Creek, the increased flood levels simulated with the revised HEC-2 model may have the effect of bringing one additional residential structure into the 100-year floodplain. This structure would join five other residential structures that presently are included within the effective FIS floodplain in this immediate vicinity. Without field surveying the actual ground elevations in the vicinity of these structures, however, it is not possible to determine with certainty whether or not they should be included in the revised floodplain. The simulated floodplain widths appear to be greater than those expected based on an analysis of the City's five-foot contour maps, and the elevations of the banks of the creek through this area based on the topographic maps appear to be higher than the revised 100-year flood levels. In essence, based on information shown on the City's five-foot contour maps, a rise in the 100-year flood levels on the order of 0.8 feet would appear to have no impact on the existing structures along this each of the creek. Field surveying of the finished-floor elevations of these structures would be necessary to confirm this observation.

5.2.1.4 South Bowie Street

The inclusion of a new surveyed section downstream of South Bowie Street in the revised HEC-2 model of Barons Creek has caused water levels to rise approximately 0.9 feet above those previously determined in the effective FIS. The reach in question extends over a distance of about 700 feet upstream along the creek from the new section, which is located approximately 650 feet downstream of the South Bowie Street bridge. The new section added to the revised HEC-2 model provides for a more accurate, but also a more constricted, definition of the channel in this area than was accounted for in the original FIS model.

The resulting increases in the revised 100-year flood levels from the revised HEC-2 model produce corresponding increases in the width of the effective FIS floodplain along this reach of Barons Creek on the order of 10 to 40 feet. Width increases of approximately 30 to 40 feet occur at the Bowie Street low water crossing, whereas, upstream of the road crossing, the floodplain increases are on the order of 10 feet. Based on examination of 1994 aerial photographs, no existing residential structures are impacted by these increased levels of flooding. All of the homes adjacent to the South Bowie Street low water crossing are a considerable distance from the revised 100-year floodplain boundaries. There are two structures presently within the effective FIS floodplain at the end of West Peach Street, and these structures could experience an additional 0.5 feet of floodwater. According to the City's five-foot contour maps, a flood level rise of 0.5 feet should have no impact on the houses at the end of West Peach Street because the revised 100-year flood level appears to be below the existing bank elevations.

5.2.2 Town Creek

Four areas previously have been identified from the 100-year flood water surface profile plots (Figures 4-4 and 4-5) as having significantly higher flood levels based on results from the revised HEC-2 model of Town Creek than those determined in the effective FIS. These areas include short reaches of the creek upstream of Elk Street, Crockett Street, Orange Street, and Edison-Schubert Streets.

5.2.2.1 Elk Street

The existing obstruction within the bridge at Elk Street, i. e., the old bridge structure, causes 100-year flood levels to increase as much as 2.8 feet above the effective FIS levels. However, it does not appear that even this amount flood level increase results in significant widening of the floodplain above Elk Street. Field surveying conducted during this study has provided more accurate channel and floodplain descriptions in the updated HEC-2 model. The map of the area upstream of Elk Street in Figure 5-2 shows only two minor reaches where the revised floodplain boundaries are slightly wider than those from the effective FIS. The greatest change in the floodplain boundary occurs at a driveway approximately 300 feet upstream of the Elk Street bridge, where the floodplain is widened by about 18 feet. This increase is not expected to impact the structure adjacent to the driveway.

5.2.2.2 Crockett Street

Results from the revised HEC-2 model of Town Creek indicate a flood level increase of

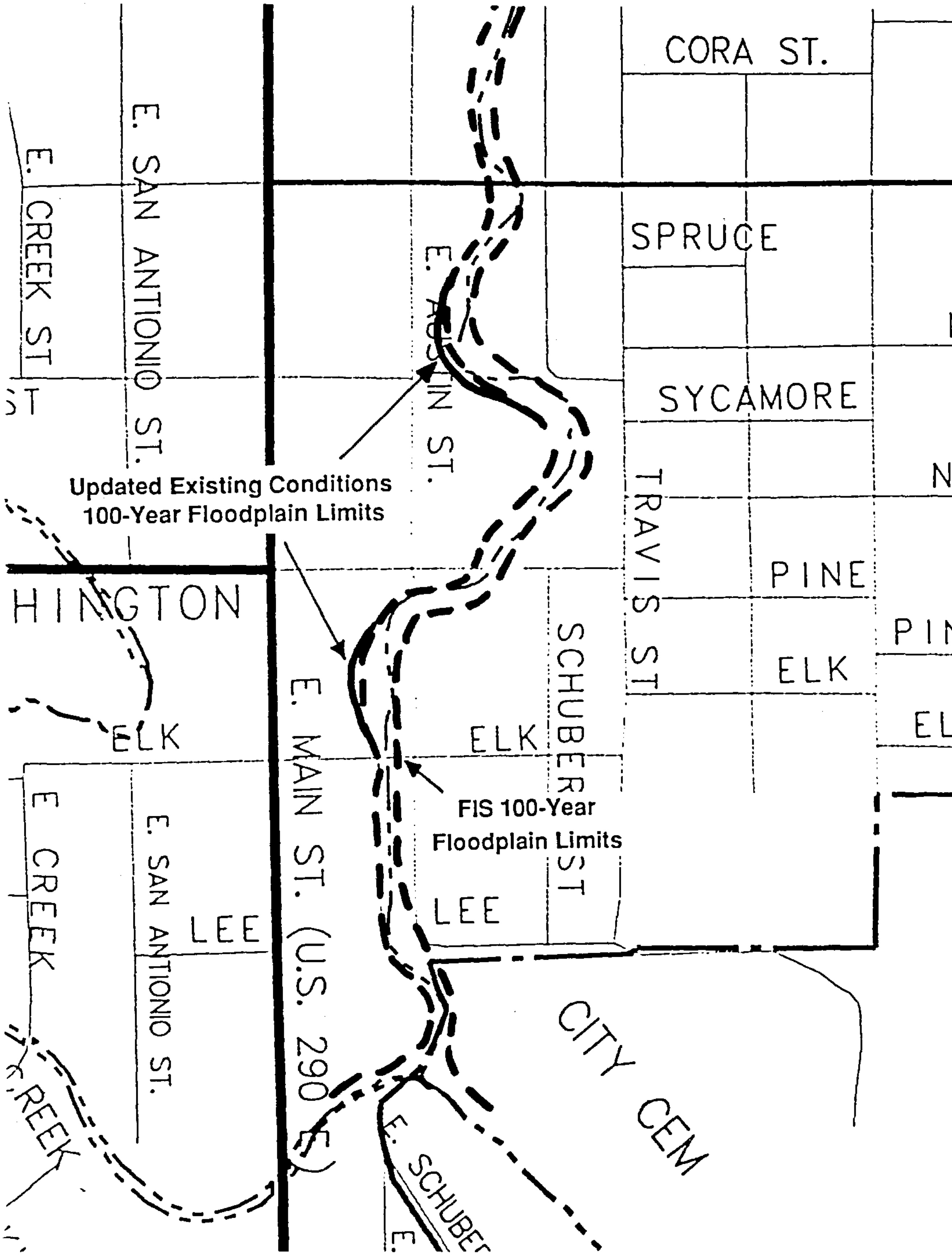

FIGURE 5-2 TOWN CREEK 100-YEAR FLOODPLAIN BOUNDARIES UPSTREAM OF ELK STREET

about 2.8 feet upstream of Crockett Street with respect to the effective FIS base flood elevations. Much of this increase can be attributed to channel modifications in the floodplain. This flood level increase translates to about an additional 175 feet of floodplain width. Figure 5-3 presents a map of the reach of the creek upstream of Crockett Street with the effective FIS floodplain boundaries delineated and the revised portions of the floodplain based on the revised HEC-2 model results also shown. As indicated, the major area of additional floodplain is located immediately upstream of Crockett Street. Based on an examination of 1994 aerial photographs, it appears that this additional flooding encompasses two residences along Crockett and Mistletoe Streets and two small commercial buildings along Crockett Street on the west bank. These structures are in addition to three residential structures at Crockett and Mistletoe Streets, one large commercial site at Crockett and Austin Streets, and three residences along Austin Street that already included in the effective FIS 100-year floodplain.

5.2.2.3 Orange Street

Orange Street is the next road crossing on Town Creek upstream of Crockett Street. There is one area immediately downstream of Orange Street where the revised 100-year flood levels exceed those from the effective FIS, and these flood level increases cause the width of the floodplain to be increased by about 12 feet beyond the effective FIS floodplain width. This increase in width does not impact any additional structures. There are, however, seven residential structures and one commercial building in the effective FIS 100-year floodplain of Town Creek between Orange Street and Milam Street, and there are an additional seven houses in the effective FIS floodplain between Milam and Edison Streets.

5.2.2.4 Edison-Schubert Streets

Flood flow hydraulics and flooding conditions along Town Creek within this overall area are quite complicated. The Town Creek channel makes a series of turns and bends as it crosses three streets with bridges over a linear distance of approximately 700 feet. Town Creek actually turns back on itself twice through this S-curve traverse before continuing downstream to cross Milam Street. Within this reach, there is an increase in the revised 100-year flood level on the order of 0.23 feet upstream of Edison Street. This increase does not significantly alter the floodplain such that the number of structures in the floodplain changes. There are two houses within the effective FIS

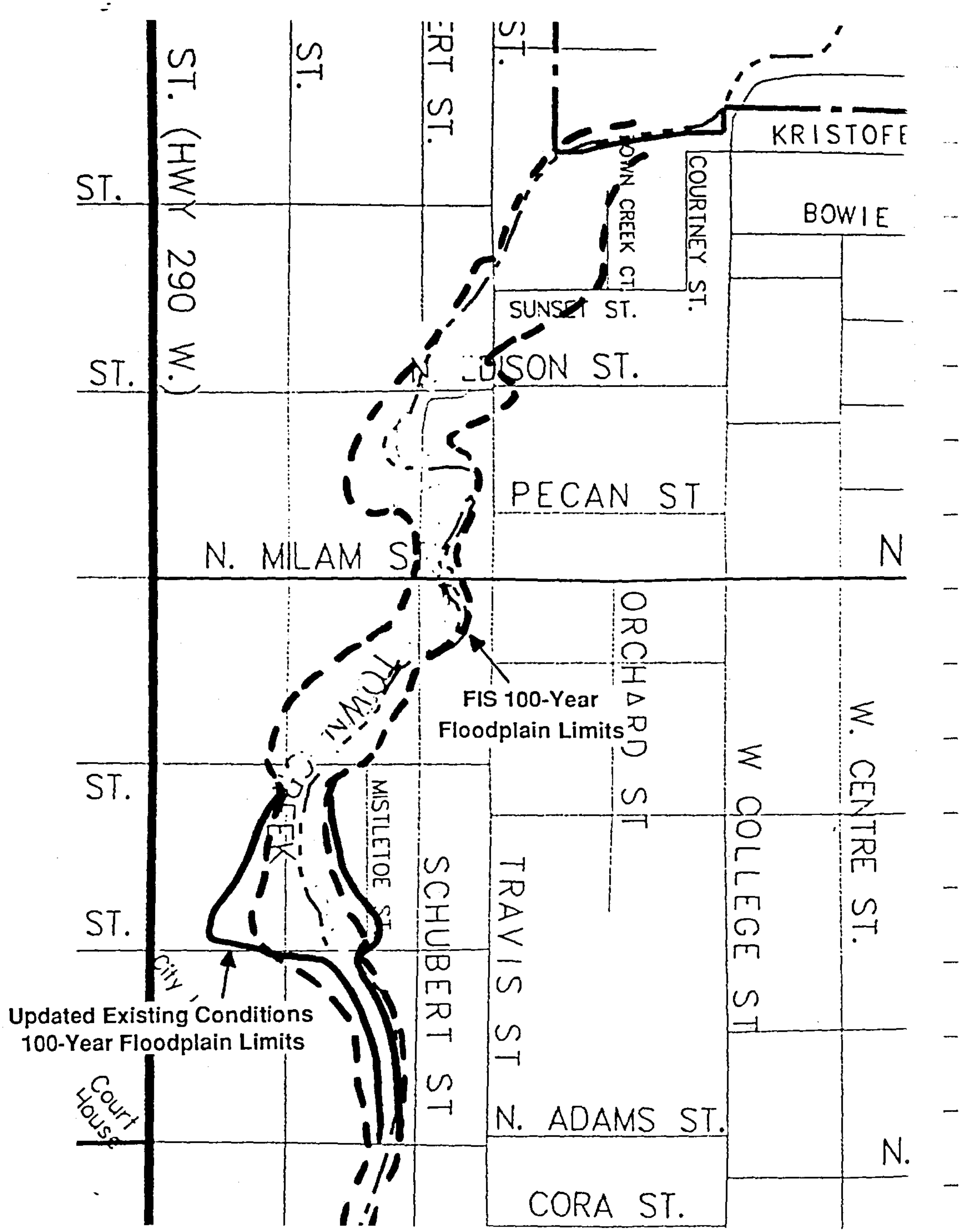

FIGURE 5-3 TOWN CREEK 100-YEAR FLOODPLAIN BOUNDARIES UPSTREAM OF CROCKETT STREET

floodplain between Edison Street and Travis Street, and one house at Sunset Street. For analyzing existing structures within the floodplain along this reach of the creek, the City's 1994 aerial photographs have been used.

5.2.3 Stream FB-1

As described previously, there are two principal reaches of Stream FB-1 where the revised 100-year flood levels from the revised HEC-2 model are significantly different from those determined in the effective FIS. The lower reach is midway between the mouth of the creek at is confluence with Barons Creek and the Llano Highway (Highway 16). Since the present day land use in this area is primarily agricultural, no residential structures are affected by the increased flood levels. These differences can be up to 7.5 feet, but are usually on the order of 3.5 feet. Comparisons of the floodplain widths simulated with revised HEC-2 model with those from the effective FIS do not indicate significant discrepancies, and the depths of flow also are similar. Hence, it appears that differences in the topography and channel geometry used in developing the models are the primary causes of the flood level deviations. With the revised model having been developed based on current and much more detailed topographic information, the revised model should be more accurate than the original FIS results.

The second reach of the Stream FB-1 where significant increases in flood levels are indicated with respect to the original FIS results is through the Carriage Hills subdivision between the Llano Highway and Lower Crabapple Road. Again, maximum increases in 100-year flood levels are on the order of 7.0 feet. Certainly, this would appear significant, but when the top widths of the respective floodplains are examined, the revised HEC-2 simulation actually results in a decrease in the 100-year floodplain. Based on 1994 aerial photography of this reach of the creek, the effective FIS 100-year floodplain encompasses 18 homes in the Carriage Hills subdivision. Twelve of these homes are upstream of Ridgewood Drive, and six are downstream. Based on the improved topography along Stream FB-1 and the revised model results, it appears that 13 of these homes actually are outside the 100-year floodplain. All five of the remaining homes are upstream of Ridgewood Drive. Figure 5-4 presents a map of this area and shows the differences between the effective FIS floodplain boundaries and the revised 100-year floodplain boundaries developed in this study.

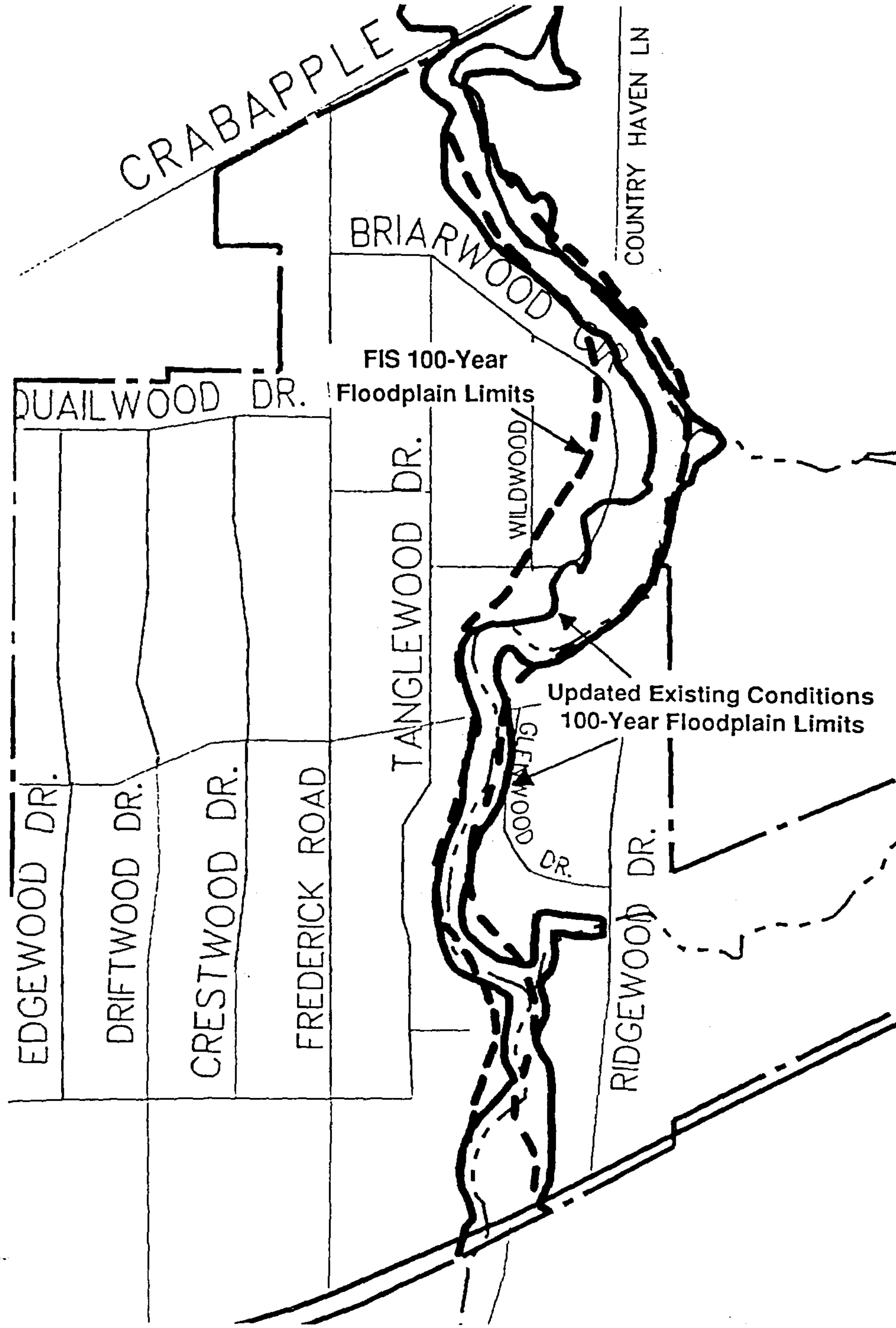

FIGURE 5-4 STREAM FB-1 100-YEAR FLOODPLAIN BOUNDARIES THROUGH CARRIAGE HILLS SUBDIVISION

5.3 ROADWAY FLOODING

Simulated flood levels from the revised HEC-2 models of Barons Creek, Town Creek and Stream FB-1 have been examined to assess overtopping conditions at major street and road crossings on these watercourses. These results are summarized in Table 5-3 for the 10-, 50- and 100-year flood events. The simulated flood levels immediately upstream of each of the crossings are listed. Also presented in the table are the minimum roadway elevations of the various streets and roads. Comparison of these roadway elevations with the different flood levels provides an indication of the extent and frequency of overtopping of the various streets and roads by floodwaters.

TABLE 5-3
LIST OF ROAD CROSSINGS AND ASSOCIATED FLOODWATER ELEVATIONS

SECTION LOCATION	HEC-2 SECTION NUMBER (U/S FACE)	MINIMUM ROADWAY ELEVATION feet msl	FLOODWATER ELEVATIONS		
			10-YEAR FLOOD feet msl	50-YEAR FLOOD feet msl	100-YEAR FLOOD feet msl
BARONS CREEK					
U.S. 290	9424	1600.00	1592.82	1594.66	1595.52
GOEHMANN LWC	3892	1611.50	1622.04	1624.92	1626.22
F.M. 1631	8000	1641.00	1634.68	1638.94	1641.03
MAIN ST.	11110	-	1646.96	1648.90	1649.70
CREEK ST. LWC	26316	1644.94	1652.27	1654.78	1656.05
WASHINGTON	29373	1669.31	1663.75	1666.12	1667.25
LINCOLN	30320	1671.97	1666.89	1669.26	1670.59
ADAMS ST.	31740	1681.25	1672.31	1674.50	1675.58
ORANGE ST. BRIDGE	34101	1681.19	1681.65	1683.61	1684.60
MILAM ST.	34812	1687.70	1683.63	1685.81	1686.80
BOWIE ST. LWC	36957	1681.76	1691.50	1693.62	1694.71
U.S. 290 W	41189	1718.80	1706.66	1708.66	1709.72
TOWN CREEK					
ELK ST.	1333	1662.89	1656.04	1657.58	1658.14
DRIVEWAY	1651	1651.31	1658.37	1660.16	1661.09
AUSTIN ST.	1957	1663.67	1659.80	1661.44	1662.27
WASHINGTON	2281	1668.40	1663.18	1665.69	1666.91
LLANO ST.	4028	1681.20	1674.80	1676.60	1677.53
ADAMS ST.	4720	1685.70	1679.97	1681.91	1682.94
CROCKETT ST.	5541	1691.43	1691.69	1692.36	1692.65
ORANGE ST.	6318	1695.20	1697.88	1698.47	1698.82
SCHUBERT ST. LWC	6886	1694.10	1698.86	1699.97	1700.57
MILAM ST.	7285	1702.40	1702.57	1702.85	1703.60
SCHUBERT ST.	7830	1700.80	1706.22	1708.14	1708.94
EDISON ST.	8297	1700.30	1708.68	1710.14	1710.68
TRAVIS ST.	8797	1705.80	1711.48	1712.66	1712.98
MORSE ST.	10895	1726.00	1728.19	1728.84	1729.16
STREAM FB-1					
LOW WATER CROSS.	171	1635.70	1638.42	1639.56	1641.07
LLANO HWY	11600	1707.50	1706.15	1709.26	1710.30
LOWER CRABAPPLE	17362	1755.00	1756.59	1756.85	1756.99

6.0 DRAINAGE IMPROVEMENT AND FLOOD PROTECTION ALTERNATIVES

6.1 LOCALIZED FLOODING

Areas identified as having significant localized flooding problems in Section 6.1 have been further evaluated to develop alternative measures to eliminate or to reduce the severity of the existing flooding conditions. Various alternatives that have been determined to be effective and that appear to be technically feasible are listed in Table 6-1, and they are identified by location on the map of the area in Plate 6-1.

The alternatives evaluation generally has been accomplished using techniques similar to those applied for the initial evaluation of the flooding problem areas. This includes performing hydraulic calculations for the proposed channel, storm drain and culvert improvements with estimates of localized runoff for different design storm events under fully-developed watershed conditions. For proposed channels, the "normal" depth of flow has been determined using Manning's uniform flow equation for specific levels of storm protection. Preliminary design slopes have been estimated using available information from field surveys and topographic maps as compiled during this study. Trial culvert sizes have been analyzed using standard culvert hydraulic procedures similar to those described in the Texas Highway Department's (now Texas Department of Transportation) Drainage Manual (1985).

In some cases, other hydraulic and hydrologic calculations have been performed, including additional HEC-1 runoff simulations, to provide additional information when necessary. Various hydrologic analyses have been undertaken to evaluate alternatives that modify runoff from or divert runoff away from problem drainage areas, thus reducing downstream flood flows. Also, alternatives that involve stormwater detention have necessitated the use of the HEC-1 runoff routing model to determine preliminary pond sizes and outlet configurations, as well as, to determine the general effectiveness of various ponds for reducing downstream flood flows.

A preliminary review of potential detention pond sites was made using available topographic maps and general knowledge regarding the location of existing flooding problems. Over 40 pond sites were reviewed with regard to their potential effectiveness for improving both localized and stream flooding problems. After initial screening, field reconnaissance surveys were made of the most promising detention pond sites and recent (1994) aerial photographs were reviewed. For pond sites that generally appeared to be technically feasible, inflow hydrographs were developed using the

TABLE 6-1

ALTERNATIVES FOR LOCALIZED DRAINAGE IMPROVEMENTS AND FLOOD CONTROL MEASURES

PROBLEM SITE DESIGNATION	PROBLEM SITE	LOCATION	DESCRIPTION	EFFECTIVENESS
A1a	L1	Friendship Lane Low Water Crossing	13 - 36" x 56" CGMP 450 feet D/S Channelization Roadway Weir Overflow Section	10-Year Capacity 100-Year - 1 feet over top of road
A1b	L1	Friendship Lane Low Water Crossing	4 - 36" x 56" CGMP Downstream Channelization Roadway Weir Overflow Section	10-Year Capacity with Regional Detention 100-Year < 0.5 feet over top of road
A1c	L1	Friendship Lane Low Water Crossing	7 - 36" x 56" CGMP Downstream Channelization Associated Roadway Work	100-Year Capacity with Regional Detention
A2	L1 - L5	Friendship Ln. Drainage West of Washington	Regional Detention Pond Area - 8.6 acres Max. Depth - 6' 100-Year Volume 26 acre-feet 18" Outlet Pipe	Reduces 100-Year Storm flow 96% at the site. Reduces 10-Year and 100-Year flood flows 20% at the South Creek Subdivision.
A3	L1 - L3	Friendship Ln. Drainage Just U/S of South Creek Subdivision	Regional Detention Pond Area - 9.3 acres Max. Depth - 7.6' 100-Year Volume - 48 acre-feet 24" Outlet Pipe 800 feet U/S Channel	Combined with A2 Pond, Reduces 100-Year flow at South Creek Subdivision to 42 cfs and reduces the peak flow at the Friendship Lane low water crossing by 72%.
A4	L2	South Creek Subdivision	460 feet- 54" RCP 1025 feet D/S Channelization	Carries 33% of 10-Year flood runoff.

TABLE 6-1

ALTERNATIVES FOR LOCALIZED DRAINAGE IMPROVEMENTS AND FLOOD CONTROL MEASURES

PROBLEM SITE DESIGNATION	PROBLEM SITE	LOCATION	DESCRIPTION	EFFECTIVENESS
A5	L3	Friendship Lane from Low Water Crossing to Washington	2,800' Grass Lined Trapezoidal Channel 35' Top Width (North Side) 2,800' Grass Lined Trapezoidal Channel 18' Top Width (South Side) Replace Creek St. culverts with 3 - 2.5' x 8' box culverts Replace driveways with box culverts	10-Year Capacity
A6a	L4	Washington @ Friendship	Add 2 - 4' x 4' box culverts	10-Year Capacity
A6b	L4	Washington @ Friendship	Add 1 - 4' x 4' box culverts	10-Year Capacity with U/S Detention
A7	L5	Friendship Lane U/S of Washington	1,100' Grass Lined Trapezoidal Channel 30' Top Width on North Side	10-Year Capacity
A8	L6	Highway St. & S. Adams	450 feet - 48" RCP 500 feet of D/S Channelization	10-Year Capacity
A9	L7	Schubert St.	1,100 feet - 42" RCP 11 inlets & 800 feet stormsewer	100-Year Capacity
A10a	L7	Schubert St.	Purchase 2 - 0.5 acre vacant lots Regrading	25-Year Protection for Houses Reduces Street Flooding
A10b	L7	Schubert St.	Purchase 2 - 0.5 acre vacant lots Excavation - 3.6 acre-feet pond 1,100 feet - 24" RCP	Eliminates House/Street Flooding for 100-Year Storm
A11	L8	N. Milam St.	1,900feet - 48" RCP 10 inlets	10-Year Capacity Eliminates House Flooding for 100-Year Storm

TABLE 6-1

ALTERNATIVES FOR LOCALIZED DRAINAGE IMPROVEMENTS AND FLOOD CONTROL MEASURES

PROBLEM SITE DESIGNATION	PROBLEM SITE	LOCATION	DESCRIPTION	EFFECTIVENESS
A12	L8 - L10	N. Milam St.	600 feet - 48" RCP 1,900 feet - 60" RCP 20 inlets 500 feet of curb	10-Year Capacity Eliminates House Flooding on Milam Eliminates House Flooding on W. Centre & W. College with additional upstream improvements (A13)
A13	L8 - L12	W. Burbank	1,050 feet - 48" RCP 11 inlets	10-Year Capacity Eliminates House Flooding on Burbank Eliminates House Flooding D/S with Alternative A12
A14	L15, L16, L18 - L24	E. Burbank	2,200 feet - 48" RCP 9 inlets Minor Channelization Drainage Easement Acquisition	10-Year Capacity Eliminates Structure Flooding near Llano & W. Burbank Reduces downstream problems
A15	L16	N. Lincoln	250 feet of Berm	10-Year Capacity within Street Eliminates House Flooding Potential with Alternative A14
A16	L17 - L24	N. Llano	1,300 feet - 60" RCP (Llano) 1,000 feet other Stormsewer 20 inlets	10-Year Capacity Reduces D/S flows 50 - 60% Eliminates House Flooding except near Travis for 50-Year to 100-Year events. Reduces D/S Erosion
A17	L16 - L22	College & Travis	2,500 feet - 72" RCP 500 feet - 60" RCP 1,000 feet Other Stormsewer 30 inlets D/S Energy Dissipation & Erosion Control	10-Year Capacity Eliminates House Flooding except for events near 100-Year floods.

TABLE 6-1

ALTERNATIVES FOR LOCALIZED DRAINAGE IMPROVEMENTS AND FLOOD CONTROL MEASURES

PROBLEM SITE DESIGNATION	PROBLEM SITE	LOCATION	DESCRIPTION	EFFECTIVENESS
A18	L23, L24	Travis	400 feet of Erosion Control 200 feet Minor Grading & Channelization	Reduces existing erosion problem
A19	L25, L26	Trailmoor & Llano	800 feet - 36" RCP 300 feet other Stormsewer 15 inlets	10-Year Capacity Reduces Llano overtopping
A20	L25, L26	Morning Glory & Broadmoor	2,000 feet - 24" & 36" RCP 12 inlets	10-Year Capacity Provides 100-Year Capacity @ Trailmoor and Llano with Alternative A19
A21	L13, L14 L25 - L27	North of Morning Glory	Regional Detention Pond Area - 6 acres Max. Depth - 8.5 feet 100-Year Volume - 37 acre-feet 3' x 5' Box Culvert Outlet 1,100 feet U/S Channelization 2 - 36" x 58" CGMP	Eliminates 100-Year overtopping of Llano Offsets additional discharge from A20 Reduces street flooding and spillovers on N. Milam
A22	L28 - L31	West of Edgewood	Regional Detention Pond Area - 5 acres Max. Depth - 5' 100-Year Volume 15 acre-feet 18" RCP outlet	Reduces 100-Yr flow at the discharge point by 94% Eliminates problems on Driftwood north of Ridgewood 10-Year protection downstream Reduces downstream street flooding
A23	L30, L31	Driftwood & Adams	600 feet - 48" RCP 400 feet - 54" RCP 20 inlets 700 feet Grass Lined Channel Top Width - 35 feet	5-Year Capacity with Detention Approximately 100-Yr protection from house flooding Reduces street flooding

TABLE 6-1

ALTERNATIVES FOR LOCALIZED DRAINAGE IMPROVEMENTS AND FLOOD CONTROL MEASURES

PROBLEM SITE DESIGNATION	PROBLEM SITE	LOCATION	DESCRIPTION	EFFECTIVENESS
A24	L34	South Ridge Subdivision	Regional Detention Pond Area - 11 acres Max. Depth - 13 feet 100-Year Volume 19 acre-feet 3.5' x 4' Box Culvert Outlet	Reduces 100-Yr flow at the discharge point by 66% Reduces 10-Year flood to pass through existing culverts Reduces overflows and house flooding potential
A25	L34	Ridgewood	2 - 48" RCP U/S & D/S Channel Grading	10-Year Capacity 50-Year Capacity with 1 feet of overtopping 100-Year Capacity with upstream detention (A24)
A26	L35	South Bowie	600 feet - 36" RCP 7 inlets	10-Year Capacity
A27	L35	South Edison	750 feet - 30" RCP 4 inlets	10-Year Capacity
A28	L36	Armory Road	4 - 36" x 58" Arch CGMP 400 feet of D/S Channel	10-Year Capacity
A29	L37	Basse Lane	4 - 36" x 58" Arch CGMP 250 feet of D/S Channel	10-Year Capacity
A30	L38, L39	Basse Lane	4 - 36" x 58" Arch CGMP 850 feey Grass Lined Channel Top Width - 30 feet	10-Year Capacity
A31	L40	South Bowie	3 - 36" x 58" Arch CGMP	10-Year Capacity
A32	L44 - L49	Park Street	1,150 feet - 42" RCP 300 feet - 36" RCP 200 feet - 18" RCP 14 inlets	10-Year Capacity Approximately 100-Year protection for buildings

TABLE 6-1

ALTERNATIVES FOR LOCALIZED DRAINAGE IMPROVEMENTS AND FLOOD CONTROL MEASURES

PROBLEM SITE DESIGNATION	PROBLEM SITE	LOCATION	DESCRIPTION	EFFECTIVENESS
A33	L46	Ufer Street	600 feet - 24" RCP 4 inlets	5-Year Stormsewer Capacity 10-Year Capacity at low point of street
A34	L51, L53	Highway Street & South Creek Street	1,400 feet - 36" RCP 1,300 feet - 30" RCP 9 inlets	10-Year Capacity Reduces flooding duration near Highway St. & Eagle Reduces spillover to Friendship Lane Drainage
A35	L51, L53	Highway Street South Eagle	1,800 feet Grass Lined Trap. Channel Top Width Approx. 30 feet 3 - 36" x 58" Arch CGMP	Eliminates street flooding along Highway Street (L52) Provides 10-Year Capacity at Eagle Street
A36	L52	Apple Street	1,150 feet - 36" RCP 6 inlets	10-Year Capacity Reduces house flooding potential
A37	L54	U. S. Highway 87	Remove old road bridge Revegetation	Reduces Erosion Eliminates backwater from structure
A38	L55	Crenwelge Road	Add box to culvert Approximately 300 feet of channel improvements	10-Year Capacity Reduces structure flooding potential

HEC-1 model, and preliminary pond grading plans and outlet designs were established based on spreadsheet hydrologic analyses of the hydrographs. Additional HEC-1 simulations then were performed to evaluate the effectiveness of the selected ponds for reducing downstream flood flows and to refine and revise the outlet designs and pond configurations.

The level of flood protection considered in developing alternative drainage improvements and flood control measures has varied depending on the severity and nature of the flooding problems examined. Problems involving combinations of the flooding of residential structures and significant street flooding have been considered to be the most significant, and where it has appeared to be feasible, alternatives providing flood protection for the 25-year and/or 100-year storm event have been evaluated. Overtopping of major streets and roadways by floodwaters also has been considered to be a serious problem because of the danger to motorists and pedestrians and the potential for loss of life. Protection from overtopping has been evaluated for the major streets and roadways considering the 25-year and 100-year storm events, with 10-year capacity without overtopping considered to be the minimum design standard.

Solution alternatives for problem areas with some street flooding and some potential for flooding of residential structures have been evaluated considering primarily the 10-year storm event, since conveyance of at least the 10-year flood flow would significantly reduce flooding risks. Furthermore, stormwater control facilities that are designed for the 10-year storm event in the Fredericksburg area also will provide sufficient conveyance to handle about 55 percent of the 100-year flood flows. Because of the expense and difficulty of implementing the higher levels of protection and because of the greater benefits of providing protection for more area for the more frequent storms, the 10-year storm event, under fully-developed watershed conditions, has been adopted and used as the primary design standard for most of the solution alternatives evaluated.

Although the conversion of land in the Fredericksburg area from a natural, undeveloped state to a fully-developed condition theoretically can result in a 40- to 50-percent increase in the 10-year flood flow for moderate intensity development, many of the existing localized flooding problem areas are within watersheds that already are approaching full development intensity. Hence, under these circumstances, on-site detention of stormwater runoff is not considered to provide an effective means for reducing floodwater discharges, except for very localized drainage situations immediately downstream of development projects. For this reason, on-site stormwater detention has not been specifically considered as a solution alternative for each individual flooding problem site.

However, in some areas with significant projected growth involving intensely-developed land uses, such as commercial, office and industrial projects, a much higher increase in peak flood flows can be expected under fully-developed conditions. In these watersheds with higher-intensity development, on-site stormwater detention obviously is a more significant alternative that should be given strong consideration. Conversely, watershed areas with low intensity development, such as parks or low-density residential subdivisions, will have much less of an increase in peak flow flows between existing and fully developed conditions, and stormwater detention may not be required.

It should be noted that the facility sizes and capacities developed in this Flood Protection Planning Study as part of the solutions for existing flooding problems are considered to be preliminary and will need to be verified and refined through detailed, site-specific design studies. The facility designs described herein are approximate and conceptual, but are considered to fully adequate for planning purposes. Detailed surveys and additional, more detailed hydraulic analyses will required for final facility designs. Some additional hydrologic analyses also may be desirable to develop more cost-effective final designs. It should be noted that the fully-developed flows used for these analyses are only estimates based on projected land use and may vary significantly depending on the level of ultimate development and the types of stormwater conveyance and control facilities that ultimately are constructed.

Specific drainage improvements and flood control measures, to the extent they are required, are discussed in the following sections for each of the previously identified flooding problem areas.

6.1.1 Friendship Lane Drainage

One of the major flooding problems regarding this area is that the Friendship Lane readily becomes impassable at the low water crossing during the occurrence of even small storm events. The peak flood flow for the 10-year storm at the low water crossing is 578 cfs, assuming fully-developed watershed conditions upstream, while the corresponding 100-year flood flow at this location is 1,096 cfs. Although a higher level of protection may be justified for this location due to traffic volumes, site limitations with regard to existing ground and roadway elevations necessitate using no more than the 10-year storm event as the standard for developing a practical and feasible culvert design.

To convey the 10-year flood flow beneath the road will require some channelization work downstream for approximately 450 feet in order to lower the flowline enough to place drain pipes under the roadway. For conveying the 10-year flood flow without overtopping of the roadway, thirteen 36" by 58" corrugated metal (GCMP) arch pipes are required. Using this size pipe still will require the roadway to be raised about 1.5 feet at the low point, which will involve road work over a distance of nearly 400 feet. The roadway surface could be designed so as to serve as an overflow weir for passing flood flows produced by storms greater than the 10-year flood. A flat section of concrete-capped roadway 200 feet long, in conjunction with the thirteen 36" x 58" pipes, would be capable of passing the 100-year flood flow with a maximum depth over the roadway of about one foot. Additional detailed hydraulic analyses will need to be performed to ensure that this type of culvert facility will not raise the water surface along the upstream channel. Alternatively, appropriate easements can be acquired to accommodate the effects of any increases in upstream flood levels. Downstream easements also will be required to allow the necessary channelization work. Significant flood flows and ponding of stormwater runoff already occurs along the watercourse; hence, there should be some incentive for adjacent land owners to assist with implementation of the proposed culvert project. As a minimum, construction of the proposed culvert could be coordinated with drainage work required by future development projects.

The number of pipes required for conveying the 10-year flood flow could be reduced to as few as four 36" by 58" CGMP arch pipes provided that the two regional stormwater detention ponds described below are constructed upstream within the Friendship Lane drainage area. With the regional stormwater detention and the four pipes described above, the 100-year flood flow would overtop the roadway less than 0.5 feet. Alternatively, conveyance of the entire 100-year flood flow under the roadway could be accomplished with seven pipes of this same size if the upstream stormwater detention is implemented.

Because of the many significant flooding problems and their associated site constraints within the Friendship Lane drainage area, it would appear that one feasible alternative is to provide regional stormwater detention facilities at one or more sites within the watershed. Since there are major problems throughout this watershed, the prime detention sites necessarily must be located farther upstream in the watershed. For this purpose, two detention sites have been evaluated in detail. One site (A2) is located west of South Washington Street (U. S. Highway 87) and along and just east of the channel running southeastward from South Adams Street, and the other site (A3) is located just west and upstream of the South Creek subdivision. Except for the street flooding at the intersection of Highway Street and South Adams Street (Site L6), which is upstream of these pond sites, detention ponds at these locations potentially would be effective in significantly reducing or eliminating all the identified flooding problems within the Friendship Lane drainage area.

Based on preliminary hydrologic analyses, the A2 detention pond site appears to be effective for improving flooding conditions because of its location near the headwaters of the Friendship Lane drainage and because it is upstream of the most significant problem sites. A pond at this site could be designed to detain nearly all of the 100-year flood flow from the upstream watershed and then to slowly release this water after passage of the storm when downstream flooding has subsided. The effectiveness of the pond also can be improved by routing additional stormwater into the pond from the end of Sunco Avenue. For full retention of the 100-year flood, the pond facility would cover approximately 8.6 acres with a maximum depth of six feet and a required total volume of approximately 26 acre-feet. The required outlet is an 18-inch reinforced concrete pipe. This pond configuration would reduce the 100-year flood peak flow from 382 cfs to 17 cfs, a 96-percent reduction in the flow rate. This large flow reduction is necessary in order to effectively reduce downstream flood flows at the individual flooding problem sites since there still is a significant downstream contribution of stormwater runoff that is not being detained. This pond would reduce the 100-year flood peak flow at the South Creek subdivision from approximately 842 cfs to 675 cfs, a 20 percent decrease in flow. For the 10-year storm, the flood flow at the South Creek subdivision would be reduced from 464 cfs to 375 cfs. More significant flow reductions would be achieved at South Washington Street and immediately upstream since runoff from most of the upper drainage area would be detained and controlled. This pond would eliminate the need for additional channel work upstream (west) of South

Washington Street, and it would significantly reduce the size of drainage improvements needed at and downstream of South Washington along Friendship Lane.

The A3 pond site also is a very effective detention site since it is located just upstream of the major flooding problem area (L2) in the South Creek subdivision. In combination with the A2 detention pond, the A3 pond also could be designed to detain nearly all the 100-year flood flows from the upstream watershed and then to slowly release these flows at or below the minimum conveyance capacity of the downstream channels. To provide for full 100-year flood flow retention (along with the A2 pond), the A3 pond facility would cover approximately 9.3 acres with a maximum depth of 7.6 feet and a required total volume of approximately 48 acre-feet. The required outlet is a 24-inch reinforced concrete pipe. This would reduce the upstream peak 100-year flood flow from 675 cfs to 42 cfs, a 94-percent reduction. The combined detention effects of the two ponds would be sufficient to allow the 100-year flood flows to safely pass through the South Creek subdivision, and they would reduce the 100-year flood flows at the Friendship Lane low water crossing by 72 percent. The two ponds would eliminate the need for additional drainage improvements through the South Creek subdivision and, as noted above, would significantly reduce the number of culverts required at the Friendship Lane low water crossing crossing.

Without the upstream detention ponds, some form of improved floodwater conveyance through the South Creek subdivision area is needed. Alternative A4 involves installation of a storm drain through the subdivision. A 54" reinforced concrete pipe would carry approximately 150 cfs, which is about one-third of the total stormwater flow of the Friendship Lane drainage. To install the pipe, channelization would be required downstream of Creek Street all the way to the existing low water crossing on Friendship Lane. Also, an inlet sump would be needed just west (upstream) of the South Creek subdivision. Although these facilities, by themselves, would not eliminate flooding within the South Creek subdivision, a 10-year flood protection level (or more) could be achieved in combination with other alternatives, including some upstream detention and drainage improvements along Friendship Lane.

The limited floodwater-carrying capacity of the swale along Friendship Lane causes flooding of adjacent properties and forces much of the stormwater from upstream to spill northward and flow through the South Creek subdivision. Several methods for improving conveyance have been considered to keep the stormwater flows off the roadway and in the road right-of-way. A 60" reinforced concrete pipe installed along Friendship Lane would be capable of carrying about two-thirds of the 10-year flood flow produced at Friendship Lane and South Washington Street, and it would provide less than half of the total discharge capacity needed to convey floodwaters beyond the South Creek subdivision. Using concrete-lined channels along Friendship Lane would require one channel 18-feet wide (top width) on the north side of the roadway and one channel 12-feet wide on the south side of the roadway. Although these channels would carry the 10-year flood flow, they would require replacement of all the driveways and the Creek Street culverts with small bridges in order to prevent any obstruction of the stormwater flows in the channels. Velocities in the channels would be on the order of 11 feet per second. A more practical alternative (A5) involves the construction of grass-lined trapezoidal channels along the current alignments of the existing swales adjacent to the roadway. For conveying the 10-year flood flow, a trapezoidal channel with a top width of 35 feet would be required on the north side of the roadway and a channel with a top width of 18 feet would be required on the south side of the roadway. This channel work would require replacement of the the north side driveways and South Creek Street culverts with three 2.5' (high) by 8.0' (wide) box culverts and replacement of the south side driveway culverts with one 2.5' (high) by 8.0' (wide) box culvert. This channel configuration in combination with Alternative A4 (54" storm drain through the South Creek subdivision) would provide 10-year flood protection along much of Friendship Land and through the South Creek subdivision. Of course, these drainage improvements would not prevent flooding of the roadway and residential structures by flood flows produced by larger storm events, i. e., greater than the 10-year flood.

The existing box culvert at South Creek Street and Friendship Lane is undersized for the 10-year flood event. Two additional 4' by 4' box culverts are needed at this location to convey the 10-year flood flow. If the upstream detention project is implemented as described for Alternative A2, only one additional culvert would be required.

Upstream of South Washington Street, a grass-lined trapezoidal channel with a top width of 30 feet is needed along the north side of Friendship Lane to safely convey floodwaters downstream. This channel is not needed if the upstream detention pond (Alternative A2) is constructed.

Since the Friendship Lane watershed is partially undeveloped, another alternative to consider is to require on-site detention for new developments. Although on-site detention would not be as effective as the regional stormwater detention alternatives, it would significantly reduce the sizes of other required improvements. The 10-year flood flow at the South Creek subdivision based on existing watershed conditions is 250 cfs, whereas the corresponding flow under fully-developed watershed conditions is 464 cfs. This represents an 85-percent increase in peak flow rate. A 55-percent increase in peak flow rate is projected for the 10-year flood flow at the Friendship Lane low water crossing. These are the highest projected increases in flood flows for any watershed within the Fredericksburg Flood Protection Planning area, and they are attributable to the significant increases expected in intense land uses, including commercial, industrial, heavy commercial and medium-density residential. On-site detention is not specifically described as an alternative for this drainage area; however, on-site detention would be effective for partially mitigating the projected increases in the peak flood flows associated with the conversion from undeveloped to fully-developed watershed conditions. For the Friendship Lane drainage, on-site detention is considered a secondary alternative to regional detention.

At Highway and Adams Streets, 450 feet of 48-inch reinforced concrete pipe is needed to provide conveyance capacity for the 10-year flood flow. Installation of this storm drain will require downstream channelization work for about 500 feet.

6.1.2 Schubert Street Ponding

Because this is a closed drainage basin (one with no natural outlet) and since there is major street flooding and likely some flooding of residential structures during the larger storm events, this area should be considered for 100-year flood protection. Approximately 1,100 feet of 42-inch reinforced concrete pipe would be needed to provide this level of protection and to allow building on the currently-vacant lots in the depression area. This alternative (A9) would also require approximately 11 inlets and 800 feet of storm drains to collect the 100-year flood runoff. Certainly, this level of project would represent a major undertaking with regard to costs.

Converting the vacant lots into a City-owned and operated detention pond and grading the area to provide additional detention storage capacity is a more reasonable and cost-effective approach for resolving the existing drainage and flooding problems than installing additional storm drains and inlets. With minor regrading of the vacant lots and continuing to use the existing 18-inch storm drain as the outlet, it appears that 25-year flood protection could be provided to the adjacent homes. Protection for the 100-year storm would require excavation of these lots and installation of a 24-inch storm drain at a steeper grade and lower upstream flowline. The purchase of the lots and minor regrading could serve as an interim solution until the other more extensive improvements could be made.

6.1.3 Cross Mountain - Milam Drainage

Along the downstream portion of North Milam, from Town Creek upstream to Morse Street, a 48-inch storm drain and approximately 10 inlets are needed to provide capacity for conveying the 10-year flood flow. With the present overflow capacity of the street, this alternative (A11) would provide nearly 100-year flood protection to the adjacent houses along North Milam Street. A variation of this alternative, Alternative A12, involves oversizing this pipe to 60 inches to allow conveyance of runoff from the Pecan Street and Edison Street areas. For this alternative, an additional 600 feet of 48-inch storm drain would be required along West Centre Street from North Milam Street to Edison Street, as well as, 10 additional inlets. About 500 feet of curb also would be needed to reduce the potential for flooding of residential structures along West Centre and West College Streets. To achieve 100-year flood protection for houses in the vicinity of the Centre-Edison Streets intersection and the College-Pecan Streets intersection, however, additional upstream drainage improvements along Burbank Street would be necessary.

Stormwater runoff that creates a flooding problem (Site L11) near Burbank Street and Avenue A also contributes to the downstream flooding problems along West Centre and West College Streets near Pecan Street and Edison Street. Because of the flow limitations created by the existing curb-cut on Burbank Street and by the capacity of the grass swale downstream of Burbank, some stormwater is diverted westward down Burbank Street to the existing flooding problem site at Avenue D (Site L12). It is not recommended that the curb-cut be enlarged or that improvements be made to the existing grass swale because these modifications could increase the contribution of runoff to the downstream problem sites (Sites L8, L9 and L10). One possible solution would be to install a storm drain northward from Town Creek near Pecan Street up through the natural flow path to Burbank Street near Avenue A. However, this would require about 3,800 feet of 48- and 54-inch pipes. A more practical alternative (A13) is to install a 48-inch storm drain westward down Burbank Street from Avenue A to Town

Creek west of Avenue D. This would provide conveyance for the 10-year flood flow and virtually would eliminate the flooding problems along Burbank Street. The existing curb-cut and grass swale would have to be maintained for conveyance of flood flows from larger storms, as would the channel at the western end of Burbank Street at Avenue D. This alternative would also sufficiently reduce the downstream flood flow contributions from the Burbank Street area such that Alternative A12 would provide 100-year flood protection for houses in the vicinity of the Centre-Edison Streets intersection and the College-Pecan Streets intersection.

Although there is some spill-over of stormwater from the upper end of North Milam Street to the east, the relatively minor nature of the associated flooding problems (Sites L13 & L14) do not appear to warrant the additional 3,000 feet of storm drain that would be required for mitigation. It should be noted that the storm drain described in Alternative A11 is not sized for any future extension up North Milam Street past Burbank Street.

6.1.4 Burbank - Llano Drainage

The only feasible alternative (A14) to correct flooding problems in this area (Site L15) is to install a storm drain eastward along Burbank Street from North Adams Street to just east of North Washington Street. This would require about 2,220 feet of 48-inch reinforced concrete pipe, along with nine inlets. Some minor channel work also would be necessary at the outfall, and drainage easements would need to be obtained down to Stream FB-1. This alternative would also provide significant downstream benefits, especially along North Lincoln Street.

6.1.5 North Lincoln Drainage

The most attractive alternative for alleviating this flooding problem (Site L16) is the alternative described above for the Burbank-Llano area. That alternative would reduce flood flows in North Lincoln Street by about 35 percent. Additional storm drain improvements for this area do not appear to be justified. However, to contain the runoff from larger storms within the street section, a berm could be constructed along the east side of North Lincoln Street from East Centre Street to East College Street and a short distance eastward along Centre Street from North Lincoln. These improvements should only be installed, however, if the upstream storm drain project along Burbank

Street (A14) is implemented, since without the upstream improvements, the berm would increase street flooding and possibly pose a flooding threat to the houses on the west side of North Lincoln Street.

6.1.6 College - Llano Drainage

To collect flood flows near the College-Llano intersection and convey them southward to Town Creek (along and beneath Llano Street) would require installation of a 60-inch reinforced concrete pipe (A16). Because this alignment crosses the drainage divide between the College-Llano drainage and Town Creek, the depth of the 60" storm drain would reach a maximum of about 23 feet just south of Orchard Street. However, the very hazardous flooding conditions at the College-Llano intersection and the significant flooding problems downstream justify the relatively large pipe size and extensive depth of cut. This alternative would provide conveyance capacity for the 10-year flood flows at the College-Llano intersection, eliminate structure flooding in this area, and reduce street and house flooding downstream. With this alternative, the street flooding associated with the 10-year storm would be reduced to the level that normally occurs every two years or so under existing drainage conditions. This alternative would also require approximately 1,000 feet of other storm drains and 20 inlets in order to collect the upstream stormwater runoff.

6.1.7 College - Travis Drainage

An optional alternative to running the 60-inch storm drain down North Llano Street from College Street (A16) is placing a storm drain along the entire existing flow path from the College-Llano intersection to just west of the City Cemetery. This alternative (A17) appears to be cost prohibitive since it requires 3,000 feet of 60-inch and 72-inch pipes along with 30 inlets and 1,400 feet of additional collector storm drains. A significant negative impact of this alternative is the increased erosion that might result downstream of the storm drain outfall near the City Cemetery, where erosion problems already exist. However, this alternative would provide 10-year flood-flow capacity throughout the College-Travis drainage and would almost eliminate the potential for flooding of residential houses in this area.

The existing erosion problems (Sites L23 & L24) near the downstream end of this subwatershed require some remediation work in order to prevent a worsening of the problem and possible undermining of roadways or drainage structures. Some minor grading, channelization and re-vegetation or armoring is needed (A18).

6.1.8 Trailmoor Drainage

The significant ponding of stormwater on Trailmoor Drive and the associated overtopping of North Llano Street by flood flows for the two-year storm event could be eliminated for storms up to the 10-year event by removing the single drop inlet to the culverts under North Llano and installing a stormwater collection system along Trailmoor up to the intersection with North Adams Street (A19). This would require approximately 800 feet of 36-inch storm drain with 300 feet of smaller pipes and 15 inlets.

Additional upstream improvements are required to eliminate overtopping of North Llano Street and ponding of stormwater on Trailmoor Drive for storms greater than the 10-year event. These improvements would involve installing storm drains along Broadmoor Drive and Morning Glory Drive to the small tributary of Stream FB-1 that passes under North Llano Street just west of Lower Crabapple Road. This alternative (A20) would require approximately 2,000 feet of 24- and 36-inch pipes and 12 inlets. An added benefit of this alternative would be reduced street flows and depths along the entire length of Trailmoor Drive. However, it would also discharge additional stormwater to the Morning Glory - Llano drainage.

On-site detention would have a moderate benefit in this drainage area, particularly at the upper end of the watershed.

6.1.9 Morning Glory - Llano Drainage

A good regional detention pond site is located within this drainage area just north of Morning Glory Road. Although there are no major flooding problems within this drainage area, the regional detention pond would eliminate overtopping of North Llano Street at Lower Crabapple Road for the 100-year storm. The pond could also offset the additional flood flows from the Trailmoor drainage that would be discharged under Alternative A20. A third benefit of the pond is that runoff from the upper end of North Milam Street could be routed to the pond along an existing ditch. It is likely that some flood flows may already spill into this ditch for larger storm events. The proposed pond site provides 37 acre-feet of detention storage, with a maximum pond depth of 8.5 feet and an area of approximately 6 acres. With this detention pond in operation and with a 3' by 5' box culvert outlet, the 100-year flood flow corresponding to fully-developed watershed conditions can be reduced by 76 percent.

On-site detention would have a significant effect on flows within this drainage area. The primary benefits would be to prevent overtopping of North Llano Street during the 100-year storm event and to maintain the existing conveyance capacity for future diversions of flood flows from the Trailmoor drainage.

6.1.10 Carriage Hills Drainage

A prime detention site is located just upstream of the major localized flooding problem sites in the Carriage Hills subdivision. This pond site (Alternative A22) is just west of Edgewood Drive and just upstream of the channel that causes flooding problems as it discharges onto Driftwood Drive. This pond site would essentially eliminate the flooding problems upstream of Ridgewood Drive on both Edgewood Drive and Driftwood Drive. It would also provide 10-year flood protection relative to downstream flooding problems and significantly reduce potential flood damages and street flooding up to the 100-year storm. The proposed pond size is 15 acre-feet, has a maximum depth of five feet and has a surface area of approximately five acres. A 94-percent reduction in the 100-year flood flow (from 267 cfs to 16 cfs) can be achieved with this pond size and an 18-inch pipe outlet.

Although preliminary consideration has been given to installing a storm drain from the north part of Driftwood Drive (Site L29) to Stream FB-1, this alternative does not appear to be cost-effective because it would require over 2,200 feet of 66-inch and 72-inch pipes to provide conveyance capacity for the 10-year flood flow. A more practical alternative involves combining the upstream detention pond (A22) with storm drains at the lower (south) end of Driftwood Drive and along North Adams Street (A23). Because of the significant flow reductions provided for larger storms by the proposed upstream detention pond, the design storm for these storm drains can be limited to the five-year flood event. Even with this level of design protection, this alternative still would require installation of 600 feet of 48-inch reinforced concrete pipe (up Driftwood), 400 feet of 54-inch reinforced concrete pipe (along North Adams), 20 inlets and 700 feet of grass-lined channel. The flood flow reductions provided by the upstream detention pond, combined with the five-year flood storm drain capacity and available street flow conveyance, overall would provide flooding protection for the residential structures along Driftwood Drive and North Adams Street for approximately the 100-year flood event.

On-site detention would have a benefit within this drainage area in that it would prevent increases in flood flows that are already are causing flooding problems. Although not as effective as the proposed regional detention pond, on-site detention would provide benefits to other locations that are not downstream of the proposed pond. The primary benefit of on-site detention in this watershed would be to reduce design flows for the storm drain alternatives.

The conveyance capacity of the culverts under Ridgewood Drive where the tributary of Stream FB-1 from the Stone Ridge subdivision crosses is considerably less than that required to pass the 10-year flood flow. A potential regional detention pond site is located just upstream of this crossing and downstream of the existing Stone Ridge temporary detention pond. This regional pond could be constructed to reduce flood flows so that the Ridgewood culverts would have at least 10-year flood flow capacity without overtopping the roadway. This pond could also achieve a 60-percent reduction in the 100-year flood flow at this location. This level of reduction would reduce the amount of roadway overtopping and also reduce the flooding threat to adjacent residential structures. This regional detention pond facility would cover approximately 11 acres and have a storage capacity of 19 acre-feet, with a maximum depth of 13 feet. The outlet required to achieve the stated flow reductions is a 4.5-feet wide by 3-feet high box culvert.

It would also be possible to improve the Ridgewood culverts to provide additional floodwater conveyance capacity. With some additional channel grading upstream and downstream of the roadway, two 48-inch reinforced concrete pipes would provide sufficient capacity for conveying the 10-year flood flow, without overtopping. Allowing for one foot of overflow would provide capacity for the 50-year flood event. This alternative, combined with the regional detention alternative described above, would allow the passage of the 100-year flood flow through the expanded culverts.

On-site detention would prevent increased overtopping and flooding at the Ridgewood crossing. Without on-site detention, there is a projected 40 percent increase in flows for the 10-year storm event, from 228 cfs to 318 cfs.

6.1.11 West Creek Street Drainage

Since there are significant stormwater ponding problems in this area, a new storm drain designed for conveyance of 10-year flood flows appears to be justified. For South Bowie Street, this requires approximately 600 feet of 36-inch reinforced concrete pipe and seven inlets. An additional 750 feet of 30-inch reinforced concrete pipe with 4 inlets is required for South Edison Street and west of West San Antonio Street.

6.1.12 Old Harper Road Drainage

For passing the 10-year flood flow under fully-developed watershed conditions, the low water crossing on Armory Road will require four 36-inch by 58-inch corrugated metal pipes. Installation of these pipes with the current road elevation will require construction of a grass-lined trapezoidal channel downstream for approximately 400 feet. The required top width of the channel is about 40 feet.

At the low water crossing on Basse Lane (Site L37), three 36-inch by 58-inch corrugated metal pipes are needed for conveying the 10-year flood flow. The existing swale would need to be deepened and graded to form a triangular channel for about 250 feet downstream of the culverts.

To accommodate future conditions, it appears to be desirable to reroute the stormwater runoff underneath Basse Lane at Duderstadt Drive instead of allowing it to continue to flow northward along the roadside swale (Site L38) toward the low water crossing. This flow rerouting would reduce the size of the culverts required at the Basse Lane low water crossing, and it would eliminate the need to improve the swale running north along Basse Lane. However, this alternative would require construction of a 30-foot wide (top width) trapezoidal channel north and east of Basse Lane and acquisition of a drainage easement for the channel. For floodwater conveyance underneath Basse Lane, four 36-inch by 58-inch corrugated metal arch pipes would be needed.

Along South Bowie Street between Basse Lane and Postoak Road, a set of culverts is needed to safely convey stormwater that normally spills over the roadway. This would involve installing three 36-inch by 58-inch corrugated metal arch pipes at a point approximately 800 feet north of Basse Lane. Some minor grading would also be required upstream and downstream of the road.

No regional detention pond sites have been identified within this drainage; however, some regional ponds could be developed depending on eventual development patterns. On-site detention would be beneficial since the 10-year flood flows could increase considerably with the conversion from existing watershed conditions to fully-developed watershed conditions. On-site detention would reduce the required sizes and/or capacities of drainage facilities by about one-third of those described above. It should be noted that the fully-developed flood flows projected for this drainage are only estimates and may vary significantly depending on the level of ultimate development and the type of conveyance facilities that are constructed.

6.1.13 Winfried Creek Drainage

No specific alternatives have been identified for this drainage area since there are no major flooding problems. On-site detention would be beneficial in that potential future problems with erosion and overtopping of some bridge crossings could be reduced. Several good regional detention pond sites are available in the area if stormwater detetion is deemed necessary in the future. Some monitoring of erosion problems around bridges and culverts also is recommended.

6.1.14 Five Points Area

Alleviation of flooding in this area would require installation of a 42-inch storm drain northward from the Five Points intersection to Barons Creek. There are several potential storm drain routes; however, the most attractive appears to be through the park and the proposed bus terminal area. This is the natural flow path for stormwater runoff, and it would result in the least disruption of traffic. Approximately 1,150 feet of 42-inch reinforced concrete pipe are needed, which includes 200 feet of pipe running east along Park Street to the Five Points intersection. This alternative (A32) would also require 300 feet of 36-inch reinforced concrete pipe, 200 feet of 18-inch reinforced concrete pipe, and 14 inlets. An enhancement (A33) to this alternative would be to include storm drains and inlets in Ufer Street. This enhancement would add 600 feet of 24-inch reinforced concrete pipe and 4 inlets to provide 5-year floodwater conveyance capacity in the street and 10-year floodwater conveyance capacity at the low point on

Ufer.

Alternative A32 should also alleviate most of the flooding north of the Five Points intersection along Liveoak Street and at the channel to Granite Street (Sites L47, L48 and L49).

6.1.15 South Adams Drainage

No drainage improvements have been identified for this area.

6.1.16 Highway - Apple Drainage

One alternative is to intercept stormwater flows on the upstream end of Highway Street at Creek Street to reduce spills into the Friendship Lane drainage and to reduce the amount of flow at the Highway Street and South Eagle Street flooding problem areas (Sites L51 & L53). This would require routing the flow through 1,400 feet of 36-inch reinforced concrete pipe along South Creek Street to Barons Creek. An additional 1,300 feet of 30-inch reinforced concrete pipe, along with approximately nine inlets, would also be required. These improvements would not significantly affect the peak flows at South Eagle Street, although they would reduce the flood duration. Implementation of this alternative is more critical if the regional detention alternative is not used for the Friendship Lane drainage.

At the downstream end of Highway Street near South Eagle Street, the best alternative would be to construct a grass-lined channel south of Highway Street and extending through the natural low area and natural flow path. Three 36-inch by 58-inch corrugated metal arch pipe would be needed to convey the flow under South Eagle Street at the current location of the low water crossing.

Problems along Apple Street could be remediated by installing a 36-inch reinforced concrete pipe along with six inlets to provide nearly 10-year floodwater conveyance capacity.

6.1.17 Dry Creek Drainage

The old road bridge just downstream of U. S. Highway 87 should be removed to reduce erosion and to prevent backwater problems for the culverts under U. S. Highway 87. Some re-vegetation of the channel banks is needed.

The culvert under South Crenwelge Road near its intersection with Gold Road needs to be expanded to include an additional box to provide 10-year floodwater conveyance capacity. Some channel work upstream and downstream of this location is also necessary.

6.2 STREAM FLOODING

The extent of existing flooding problems along the principal creeks and streams flowing through the City have been discussed in Section 5.2. The effective flood insurance maps of the City delineate existing 100-year floodplains along Barons Creek, Town Creek and Stream FB-1. Based on results from revised and updated HEC-2 hydraulic models that have been developed in this Flood Protection Planning Study for these same watercourses, it does not appear that the recent growth and development of the City have yet to significantly change floodplain areas and flooding conditions along the major creeks and streams. As described in Section 6.1, most of the present flooding problems within the City generally are considered to be localized in nature and typically caused by inadequate drainage facilities, or the lack of drainage facilities.

Still, there are some areas along the major creeks and streams where flooding of adjacent properties can occur, particularly during larger storm events such as the 100-year flood. There are also some areas along the major watercourses where the present 100-year flood levels, as determined in this study using the refined HEC-2 hydraulic models, appear to be somewhat higher than those previously determined in the effective Flood Insurance Study for the City. There are also some areas where certain modifications in existing channels, bridges or other drainage structures should be made in order to improve floodwater conveyance or to reduce the potential for upstream flooding. Several of these situations are discussed below.

6.2.1 Town Creek

Perhaps one of the most obvious flood control measures that could be undertaken to improve the hydraulic efficiency of Town Creek is to remove the old low water crossing from under the Elk Street bridge. Based on simulations with the revised HEC-2 model of Town Creek, it appears that 100-year flood levels upstream of Elk Street would be lowered by about 2.5 feet if the existing bridge obstruction is removed and the existing bridge abutments are restructured to a 45-degree slope (Alternative A39). These modifications would increase the bottom width of the channel under the bridge from 16 feet to 51 feet. The 2.5 feet of drop in upstream flood levels due to removal of the obstruction would occur over the first 100 feet of channel immediately upstream of the bridge. At the low water crossing upstream of Elk Street, the resulting drop in the 100-year flood level would be about 1.8 feet, and since this low water crossing causes the flow in the creek to pass through critical depth, no additional benefits of the Elk Street bridge improvements are realized upstream of this crossing. As mentioned previously in Section 5.2.2.1, the structure adjacent to the low water crossing presently is not within the 100-year floodplain; consequently, the removal of the old bridge obstruction at Elk Street and the associated reductions in upstream 100-year flood levels are not likely achieve any significant immediate reductions in the potential flooding of adjacent properties. Still, from the standpoint of improving floodwater conveyance, it is important that removal of the Elk Bridge bridge obstruction be given serious consideration (Alternative A39).

Results from the revised HEC-2 hydraulic model of Town Creek, which now extends upstream through the new Cross Mountain West subdivision, indicate that the roadway at Morse Street is overtopped by the 10-year flood flow. At this location, an old railroad tank car presently serves as the culvert under Morse Street. Replacement of this existing culvert with four 8' x 8' concrete boxes (Alternative A40) and raising the road surface from its existing elevation of 1726.0 feet msl up to 1727.5 feet msl would provide sufficient conveyance capacity to handle flood flows produced by the 100-year storm (Alternative A40).

6.2.2 Stream FB-1

Simulated flood levels from the revised HEC-2 hydraulic model of Stream FB-1 indicate that the roadway at the Lower Crabapple Road crossing is inundated by floodwaters during the 10-year flood event. The culverts at this crossing consist of two 24-inch drain pipes. Aside from these pipes being severely undersized for effectively conveying floodwaters from the upstream watershed, it appears that some of the roadway overtopping problem is caused by high tailwater on the culverts as a result of the narrow channel downstream of the road crossing. Essentially, backwater from the downstream channel is reducing the hydraulic capacity of the existing culverts. Before installing larger culverts to improve the floodwater conveyance under the roadway, the constricted flow conditions downstream would need to be improved.

Options for widening and lowering of the downstream channel to provide additional conveyance capacity and to lower flood levels downstream of the Lower Crabapple Road crossing have been investigated using the revised HEC-2 hydraulic model of the stream. A trapezoidal channel with a bottom width of 25-feet, 4:1 side slopes and a flattened bottom slope of about 0.01 feet per foot has been incorporated into the model from the road crossing downstream for a distance of about 700 feet. This length of channel improvement extends through the most constricted section of the existing channel. With this modified and flattened channel, the flowline of the channel at the existing culverts would be lowered from 1752.00 feet msl to 1747.75 feet msl, which would allow larger pipes to be installed under the road without raising the road surface above its present elevation of 1755.00 feet msl.

With the improved channel downstream and with four 53" by 85" corrugated metal arch pipes replacing the existing 24" culverts under the roadway (Alternative A41), the revised HEC-2 model has been operated to evaluate flooding conditions in the vicinity of the crossing. These results indicate flood flows up to and including those produced by the 50-year storm event would be conveyed through the larger pipes without overtopping of the roadway. With the benefits of a regional detention pond upstream, as is described in the next section, the four 53" by 85" corrugated metal arch pipes also would be capable of passing the 100-year flood flows without overtopping the roadway.

6.3 REGIONAL DETENTION PONDS

6.3.1 Town Creek

The feasibility of regional stormwater detention ponds has been investigated within the Town Creek watershed. Such regional detention ponds have been considered as a means for reducing the existing flooding threat to structures along the creek, for reducing floodwater overtopping of roadways, for offsetting the potential increases in peak flood flows caused by future watershed development, and for possibly accommodating any increased discharges resulting from certain localized drainage improvement or flood control alternatives.

Seven sites have been reviewed for their potential effectiveness at improving both localized and downstream flooding conditions. After initial screening, the prime sites were visited and recent (1994) aerial photographs of the areas were examined. For two of the best sites that appeared to be feasible, the inflow hydrographs for the 100-year flood were developed using the HEC-1 model developed in this study, and preliminary grading plans and outlet sizes were established based on spreadsheet analyses of the hydrographs. Additional HEC-1 simulations then were performed to evaluate the effectiveness of the ponds for reducing downstream flood flows and flood levels and to refine the outlet and pond designs. Although such detailed analyses have been performed for only the two pond sites, at least three other sites also appear to be feasible and could be used as alternate pond sites, if necessary.

The primary detention site for the Town Creek watershed is located upstream of North Cherry Street on the western tributary to Town Creek (Alternative A42). The proposed pond has a storage capacity of 105 acre-feet, with a maximum depth of about 11 feet. This stormwater detention facility would cover approximately 19 acres, and it would have an outlet consisting of four 3' by 5' box culverts. Some additional considerations may be needed with regard to the existing stock pond that is located just downstream of this detention pond site. With this configuration, the pond would reduce the 100-year flood flow at the outlet by over 1,350 cfs, for a 57-percent reduction. Because of the lagging effect of the pond on the outflow hydrograph relative to the times of concentration for other subwatersheds, the reduction in flood flow actually increases to about 1,450 cfs at the confluence of Town Creek with Barons Creek. This represents a significant reduction in flood flows that correspondingly results in reduced water surface elevations throughout the mainstem of Town Creek downstream of West Morse Street. One of the side benefits of the reductions in flood flows from this project would be that it allows additional discharges of stormwater into the creek downstream from some of the localized drainage improvement alternatives. For example, this would include Alternative A16, which would divert the College-Llano drainage to Town Creek, instead of allowing it to continue to flow to Stream FB-1. The design discharge for Alternative A16 (for the 10-year storm) is approximately 200 cfs. The pond configuration described above (Alternative A42) would more than offset the increased flow associated with Alternative A16. It also would be feasible to downsize the pond at this site, if the goal is only to offset the effects of Alternative A16.

The second detention pond site evaluated in detail is on the mainstem of Town Creek upstream of Morse Road and just upstream of the new box culverts in the Cross Mountain West subdivision (Alternative A43). This alternative would involve modifying the upstream drop structure to serve as the outlet for the pond excavated upstream. The volume of this pond at the 100-year peak stage is 11 acre-feet, and it has a maximum depth of just over 10 feet. The area of the pond is about six acres. The outlet would consist of four 54-inch reinforced concrete pipes constructed through the existing drop structure with a 94-foot weir section located along the current flowline at the top of the drop structure. The effectiveness of this site is somewhat limited by the elevation of the adjacent platted lots; however, this detention pond does provide a reduction of 130 cfs in the 100-year flood flow, which is equal to about nine percent of the total flow. The peak discharge rate from the pond for the 100-year flood is approximately 50 cfs less than the peak flow under existing watershed conditions, and this appears to be enough to offset the additional flow that would be discharged to this branch of Town Creek under localized flooding improvement Alternative A13. This regional pond is particularly effective with respect to reducing flood flows over Morse Road. The overtopping of Morse Road is reduced by 0.5 feet (to less than two feet) for the 10-year flow. The projected downstream reduction in flood flows associated with this pond appears to be sufficient to prevent any downstream impacts from the diversions associated with Alternative A16.

The combined reduction in flood flows for the 100-year storm by the two detention ponds results in the lowering of water surface elevations throughout Town Creek (downstream of Morse Road) by about two feet, with a maximum water level decrease of about three feet upstream of Washington Street. This effectively eliminates the threat of flooding along Town Creek with respect to existing residential structures and commercial buildings. This also lowers the depth of flow over the roadway structures that are overtopped and provides 10-year flood flow capacity at Crockett and Milam Streets, which are overtopped by the 10-year storm under existing flood flow conditions. This is a particularly important benefit since all the roadway crossings on Town Creek on the west side of the City are overtopped for storms more frequent than the 10-year event.

6.3.2 Barons Creek

A preliminary investigation of the feasibility of regional detention also has been performed for Barons Creek. Because of the limited number of problem areas along

Barons Creek, there is little need for regional detention. Additionally, Barons Creek has a relatively long (approximately three hours) time of concentration due to the large portion of the watershed upstream of the City. Regional detention within or near the City could actually increase flows in Barons Creek by lagging the relatively quick local watershed discharges to be in phase with the later peak flows from the upper Barons Creek drainage area. Several potential regional pond sites have been identified in the Barons Creek watershed upstream of the City; however, no detailed analyses have been performed because of the apparent lack of need for flow reductions along Barons Creek through the City.

It should be noted that, in general, the same principle of regional detention ponds applies to on-site detention with respect to Barons Creek. However, on-site detention may still be required for control of localized flooding. If safe conveyance is available or provided to Barons Creek, on-site detention would not be necessary. For cases where on-site detention is necessary for localized problems, the detention time used to determine storage volumes should be less than one hour.

6.3.3 Stream FB-1

Eight potential regional pond sites have been identified for Stream FB-1. Three of these have been analyzed in detail with respect to localized flooding problems. One additional pond site just upstream of Lower Crabapple Road (Alternative A44) has been analyzed in detail specifically as an alternative for reducing downstream flooding. With a detention pond covering about 8.5 acres, a 100-year storage volume of 36 acre-feet and a maximum depth of about 8.5 feet, this site provides a reduction in the 100-year flood flow of about 570 cfs. This represents 23 percent of the peak flood flow just upstream of the flooding problem area in the Carriage Hills subdivision. This level of flood flow reduction also extends downstream to the Llano Highway crossing. The effect of this reduction is to lower the 100-year flood water surface elevation by 0.6 to 1.0 feet along the stream where five homes are located within the floodplain. For the 10-year storm, the detention pond would also reduce the flows sufficiently to prevent overtopping of the Llano Highway.

The detention pond site in the Stone Ridge subdivision that was analyzed as a localized flooding improvement alternative (A24) was also evaluated for its effectiveness with regard to stream flooding along Stream FB-1. A minimal reduction in flood flow (five percent) was achieved at the Llano Highway crossing with this pond, although there is a significant reduction of peak flow from the pond site. This site also discharges downstream of the primary stream flooding problems in the Carriage Hills subdivision. When considered with the proposed detention pond upstream of Lower Crabapple Road, this site provides no additional reduction in flood flows relative to that achieved by the other site alone. Therefore, this site is not considered to be generally effective with regard to reducing downstream flooding.

The regional detention pond site upstream of Lower Crabapple Road (Alternative A44) also was considered in conjunction with two other regional ponds in this watershed, Alternatives A21 and A22 as previously described in the localized flooding analysis in Section 6.1. The combination of these three ponds reduces the 100-year flood flows in Stream FB-1 by 23 percent at its confluence with Barons Creek. However, this is not a significant benefit since no current stream flooding problems have been identified south (downstream) of the Llano Highway.

7.0 DRAINAGE AND FLOOD PROTECTION ORDINANCES

As part of this Flood Protection Planning Study, consideration has been given to the possibility of the City implementing certain ordinances that would help to alleviate future flooding and drainage problems associated with and caused by the continued growth and development of the City. One particularly attractive option for such authority is a stormwater detention ordinance that would require all future development projects, with some noted exceptions, to implement drainage control measures to assure that existing rates of runoff are not being increased. This would tend to cap existing flood flows at their present levels.

Without stormwater detention, peak flood flows would increase because of increased stormwater runoff volumes caused by the additional impervious cover created by new development projects and because of faster rates of conveyance across or through new driveways, streets, parking lots, storm drains and channels. The conversion of land in the Fredericksburg area from a natural, undeveloped state to a moderately-developed condition (35-percent impervious cover) can result in a 40- to 50-percent increase in peak flood flows. However, more intense development for commercial, office, retail and/or medium density residential uses would result in greater increases in flood flows. Results from HEC-1 analyses performed as part of this Flood Protection Planning Study indicate that the 10-year flood flow from some subwatersheds could double if the projected future land use conditions occur. Conversely, low-density development, such as large-lot single family residential subdivisions, may not increase peak flood flows at all.

Stormwater detention provided by an individual land owner or developer as part of a specific new development project is referred to as on-site detention. This type of detention typically is provided on or immediately downstream of the development site by creating a stormwater storage pond. Such detention ponds usually are constructed by excavation within a drainageway, with berms or embankments installed around the excavated area. At the bottom of the detention storage pond, a small or restricted drainage outlet is provided to drain the pond. The outlet pipe or weir is designed to slowly release stormwater during a storm event so as to reduce the rate of runoff from the developed site to no more than that which occurred under predeveloped conditions, with the excess stormwater detained in the pond. Other typical features of on-site detention ponds include an emergency spillway to pass stormwater flows greater than the design discharge rate of the pond, an inlet flume or pipe to convey stormwater runoff into the pond without causing erosion, and various types of erosion protection works and velocity dissipators downstream of the pond outlet.

On-site stormwater detention is an effective means for preventing increased flooding problems by controlling the increased rates of runoff usually associated with watershed development. For this purpose, a draft stormwater detention ordinance has been prepared and presented to the City for review and consideration. This document now is under review by the City. Following is the text of the draft stormwater detention ordinance as it currently is being considered by the City.

DRAFT

City of Fredericksburg, Texas

STORMWATER DETENTION ORDINANCE

October 24, 1996

1.0 Purpose and Applicability

a) The growth in and around the City of Fredericksburg and the associated development and construction of buildings, paved surfaces, roads and other improvements has altered in the past and continues to alter the natural flow of surface waters on the land, which together with the construction of gutters, culverts, drains and channels for the carrying off of surface waters has both increased the quantity of stormwater and amplified the peak flow rates of runoff, thus leading to present and potential flooding of property and homes, dangerous flows within and over public roadways and streets, and soil and channel erosion.

b) It is the intention of the City Council to protect the health and safety of the citizens and visitors of the community and to prevent damage to private property and public facilities through the proper design and construction of both on-site and regional stormwater detention facilities that prevent or adequately reduce increases in peak flow rates of runoff that may otherwise increase the risk of flooding and the associated risk of public endangerment, property damage and erosion.

c) It is the intention of the City Council, through this Ordinance, to establish a regional stormwater detention pond program for the design and construction of regional stormwater detention facilities so that, where practical, the most cost-effective protection from flooding may be accomplished.

d) It is the intention of the Council to protect the health and safety of the citizens and visitors of the community and to prevent damage to private property and public facilities through the installation and use of temporary and permanent erosion control practices that prevent or adequately reduce increases in erosion and siltation that may otherwise increase the risk of flooding and the associated risk of public endangerment and property damage by clogging and/or partial filling of constructed or natural drainageways as well as drainage structures and detention ponds.

e) This Ordinance shall apply to all property within the planning jurisdiction of the City unless otherwise stated.

f) This Ordinance shall not apply to single family or duplex residential lots of subdivisions approved prior to the adoption of this Ordinance, unless specifically required by prior agreement between the City and the owners or developers of such subdivisions, or to new one- or two-lot subdivisions for single family or duplex residential lots, and this Ordinance is intended to be implemented for entire subdivisions at the time of platting and construction of street and drainage improvements and not on an individual lot basis for single family and duplex residential subdivisions.

2.0 Standards and Requirements for Stormwater Detention

a) No final subdivision plat, subdivision construction plan, site plan or building permit shall be approved by the City unless it can be demonstrated by the owner or developer of such property that the proposed development will not result in the additional identifiable adverse flooding of other property or public facilities, including roadways.

b) The above requirement shall be accomplished through one of the following means:

1) Design and construction of an on-site stormwater detention facility, or facilities, by the land owner or developer which limits the peak flood flows from the proposed development to the existing peak flood flows from the subject tract.

2) Participation by the land owner or developer in the Regional Stormwater Detention Pond Program in a manner sufficient to accomplish the goal stated in Item 2.a above. This may be accomplished though the contribution of funds and/or land to the Regional Stormwater Detention Pond Fund, as established in Section 3.0 below.

3) Construction of, or participation in the construction of, off-site drainage improvements, such as storm inlets, storm sewers, culverts, channel modifications, land filling, and/or other drainage facilities such that the peak flood flows for fully-developed watershed conditions from the watershed area in which the proposed development is located will be sufficiently and safely passed without flooding of downstream property and roadways.

4) Design and construction of the development utilizing limited impervious cover, infiltration of runoff from impervious cover via flow through pervious areas, and/or grass-lined swales or channels such that these measures result in a minimal increase in peak flood flows from the development.

c) Acceptance of requests from the land owner or developer to meet the stormwater detention requirements through measures listed in Items 2.b.2 through 2.b.4 above is solely at the discretion of the City.

d) Acceptance by the City of on-site stormwater detention plans will be based on the suitability and adequacy of the engineering and technical design of the proposed stormwater detention facility, as described in Section 5.0 below.

3.0 Regional Detention Pond Program

a) The City hereby establishes the Regional Stormwater Detention Pond Program whereby the City will design and direct construction of or otherwise facilitate construction of regional stormwater detention ponds in order to prevent increases in and, if practicable, to reduce peak flows of stormwater runoff.

b) The City hereby establishes, as the funding mechanism for the Regional Stormwater Detention Pond Program, the Regional Stormwater Detention Pond Fund, a dedicated fund into which the contributions by land owners and developers are deposited in lieu of construction of on-site stormwater detention facilities and from which funds are allocated for the design and construction of regional stormwater detention ponds and/or other off-site stormwater management and control facilities.

c) It is the intention of the Council to allow contributions to the Regional Stormwater Detention Pond Fund by land owners and developers in lieu of construction of on-site stormwater detention facilities for the purpose of the design and construction off-site improvements, which may include, either singly or in combination, regional stormwater detention ponds, storm sewers, culverts, inlets, gutters, swales and improved channels, in order to prevent or reduce downstream flooding problems.

d) The contributions to the Regional Stormwater Detention Pond Fund are non-refundable and are intended to be dedicated solely to implementation of drainage improvements and stormwater management and control facilities.

e) The level of contribution required to participate in the Regional Stormwater Detention Pond Program shall be based on the increase in volume and peak flow of the stormwater runoff from a proposed development and the potential for adverse downstream flooding impacts; therefore, the level of contribution will generally increase with increasing size of development, amount of impervious cover, and extent of on-site drainage conveyance modifications.

4.0 Standards and Requirements for Erosion/Sedimentation Controls

a) No final subdivision plat, subdivision construction plan, site plan or building permit shall be approved by the City unless the plans for the proposed development include temporary and permanent erosion and sedimentation control measures such that siltation of downstream drainageways are minimized.

b) The above requirement shall be accomplished through a combination of the following practices:

1) Installation of silt fences and rock berms before and during construction in order to reduce on-site soil erosion and provide temporary capture of sediment.

2) Temporary and/or permanent revegetation of bare ground in order to stabilize disturbed soil at the earliest practicable date.

3) Construction of on-site stormwater detention facilities by the land owner or developer in a manner such that detention ponds function as temporary sedimentation basins until permanent revegetation of the subject tract is accomplished.

4) Other measures which may be necessary to control erosion and sedimentation on a site by site basis.

5.0 Additional Standards for Approval

a) A Registered Professional Engineer, licensed in the State of Texas and qualified and experienced in the design and operation of stormwater detention ponds and related stormwater management facilities, shall perform the hydraulic and structural design of stormwater detention ponds and related stormwater management facilities, including the development of engineering and technical information required for evaluation by the City.

b) All design and technical information necessary to thoroughly evaluate the suitability and adequacy of the engineering and technical design of proposed on-site stormwater detention facilities and, if proposed, off-site facilities shall be provided to the City for review. All detention and runoff calculations, including computer model simulations, if used, shall be provided.

c) All on-site stormwater detention facilities shall be designed to adequately and safely pass all stormwater inflows, including flood flows and runoff from upstream and adjacent properties that have natural and/or existing overland flows toward and onto the subject tract. The on-site stormwater detention facilities should not impound stormwater onto or cause backwater to inundate any upstream or adjacent properties in excess of existing conditions.

d) On-site stormwater detention facilities shall not be placed such that they encroach into the regulatory 100-year floodplain as established by the City, Gillespie County, and/or the Federal Emergency Management Agency, unless it can be satisfactorily demonstrated to the City through the use of hydraulic modeling that such encroachment will not cause any rise in the 100-year flood level on other off-site properties or that the increase in the 100-year flood level caused by such encroachment will occur entirely onsite on the owner's or developer's property.

e) Additional engineering and technical rules and guidance with respect to the application and review of the stormwater detention requirements of this Ordinance may be provided by the City within a Drainage Criteria Manual.

f) Additional rules, guidance and requirements with respect to the application and review of requests for participation in the Regional Stormwater Detention Pond Program, off-site drainage improvements and other alternatives to on-site stormwater detention as listed in Items 2.b.2 through 2.b.4 above may be provided by the City within a Drainage Criteria Manual.

g) All design and technical information necessary to thoroughly evaluate the suitability and adequacy of proposed erosion and sedimentation control measures shall be provided to the City for review.

h) Additional rules, guidance and requirements with respect to the review and acceptance of temporary and permanent erosion and sedimentation control plans may be provided by the City within a Drainage Criteria Manual.

8.0 DRAINAGE IMPROVEMENT AND FLOOD PROTECTION PLAN

8.1 LOCALIZED FLOODING PLAN

The various alternatives for addressing localized flooding problems throughout the planning area as developed in Section 6.1 and as listed in Table 6-1 have been evaluated in general terms with respect to their relative feasibility, constructability, cost and effectiveness. A preliminary estimate of implementation costs has been prepared for the prime alternatives, i. e., those demonstrating the greatest effectiveness for reducing flooding in areas with the most critical problems. Some additional preliminary cost estimates also have been prepared for a few secondary alternatives to allow comparison with the primary alternatives. Based on these additional evaluations and cost comparisons, a list of thirteen recommended alternatives has been developed. These are listed and generally described in Table 8-1. The locations of the recommended alternatives are shown on the map of the area in Plate 8-1. Although other effective and feasible alternatives exist, these recommended alternatives appear to be the best suited for improving the most critical drainage and flooding conditions in the Fredericksburg area. The recommended alternatives are listed Table 8-1 in the general order of priority for implementation based on the same factors identified above that were considered in developing the list.

Considering that the recommended alternatives provide effective solutions for existing localized flooding problems and that the potential damages and loses, including loss of life, caused by this flooding could be a substantial burden for the citizens of Fredericksburg, it is important for the City to give strong consideration to implementing the recommended alternatives as soon as economically feasible. These recommended alternatives should be considered to represent the initial implementation phase of the overall master drainage plan for the City. Other effective, but more long-term, alternatives should be implemented as practical and as opportunities arise. These more long-term alternatives are listed and generally described in Table 8-2. These long-term alternatives are grouped in two levels of implementation priorities. The first group is referred to as Phase II (with Phase I being the recommended alternatives). These Phase II alternatives are considered to be relatively effective and efficient for reducing localized flooding problems, but they are not considered to be as critical as the recommended Phase I alternatives, particularly with regard to reducing flooding of structures and major street and road crossings. The second group of long-term alternatives is referred to as "Future" alternatives and generally, these have either a longer-term implication with respect to drainage and flood control planning or they are considered to be desirable drainage enhancements. Any specific alternative in either group of the long-term alternatives may be implemented as opportunities arise. Some

TABLE 8-1
LOCALIZED FLOODING RECOMMENDED ALTERNATIVES

PRIORITY	ALTERNATIVE DESIGNATION	PROBLEM SITE DESIGNATION	LOCATION	DESCRIPTION	EFFECTIVENESS	IMPLEMENTATION COST
1	A2	L1 - L5	Friendship Lane Drainage West of Washington	Regional Detention Pond Area - 8.6 acres Max. Depth - 6 feet 100-Year Volume - 26 acre-feet 18" Outlet Pipe	Reduces 100-Yeear Storm flow 96% at the site Reduces 10-Year and 100-Year flows 20% at the South Creek Subdivision	$340,000
2	A12	L8 - L10	N. Milam Street	600 feet - 48" RCP 1,900 feet - 60" RCP 20 inlets 500 feet of curb	10-Year Capacity Eliminates House Flooding on Milam Eliminates House Flooding on W. Centre & W. College with additional upstream improvements (A13)	$670,000
3	A22	L28 - L31	West of Edgewood	Regional Detention Pond Area - 5 acres Max. Depth - 5 feet 100-Year Volume - 15 acre-feet 18" RCP outlet	Reduces 100-Year flow at the discharge point by 94% Eliminates problems on Driftwood north of Ridgewood 10-Year protection downstream Reduces downstream street flooding	$305,000
4	A32	L44 - L49	Park Street	1,150 feet - 42" RCP 300 feet - 36" RCP 200 feet - 18" RCP 14 inlets	10-Year Capacity Approximately 100-Year protection for buildings	$260,000
5	A33	L46	Ufer Street	600 feet - 24" RCP 4 inlets	5-Year Stormsewer Capacity 10-Year Capacity at low p of streetoint	$65,000
6	A16	L17 - L24	N. Llano	1,300 feet - 60" RCP 1,000 feet other Stormsewer 20 inlets	10-Year Capacity Reduces D/S flows 50 - 60% Eliminates House Flooding except near Travis for 50-Year to 100-Year events. Reduces D/S Erosion	$535,000
7	A27	L35	South Edison	750 feet - 30" RCP 4 inlets	10-Year Capacity	$70,000
8	A1c	L1	Friendship Lane Low Water Crossing	7 - 36" x 56" CGMP Downstream Channelization Associated Roadway Work	100-Year Capacity with Regional Detention	$80,000

TABLE 8-1
LOCALIZED FLOODING RECOMMENDED ALTERNATIVES

PRIORITY	ALTERNATIVE DESIGNATION	PROBLEM SITE DESIGNATION	LOCATION	DESCRIPTION	EFFECTIVENESS	IMPLEMENTATION COST
9	A3	L1 - L3	Friendship Lane Drainage Just upstream of South Creek Subdivision	Regional Detention Pond Area - 9.3 acres Max. Depth - 7.6 feet 100-Year Volume - 48 acre-feet 24" Outlet Pipe 800 feet U/S Channel	Combined with A2 Pond, Reduces 100-Year flow at South Creek Subdivision to 42 cfs and reduces the flow at Friendship Lane low water crossing by 94%	$465,000
10	A23	L30 - L31	Driftwood and Adams	600 feet - 48" RCP 400 feet - 54" RCP 20 inlets 700 feet Grass Lined Channel	5-Year Capacity with Detention Approximately 100-Yr protection from house flooding Reduces street flooding	$301,000
11	A10a	L7	Schubert St.	Purchase 2 - 0.5 acre vacant lots Regrading	25-Year Flood Protection for Houses Reduced Street Flooding	$30,000
12	A19	L25 - L26	Trailmoor and Llano	800 feet - 36" RCP 300 feet other Stormsewer 15 inlets	10-Year Capacity Reduces Llano overtopping	$160,000
13	A36	L52	Apple Street	1,150 feet - 36" RCP 6 inlets	10-Year Capacity Reduces house flooding potential	$155,000
					TOTAL	$3,436,000

TABLE 8-2
LONG-TERM DRAINAGE IMPROVEMENT ALTERNATIVES

ALTERNATIVE DESIGNATION	PROBLEM SITE DESIGNATION	LOCATION	DESCRIPTION	EFFECTIVENESS
PHASE II				
A6b	L4	Washington @ Friendship Ln.	Add 1 - 4' x 4' box culvert	10-Year Capacity with U/S Detention
A13	L8 - L12	W. Burbank	1,050 feet - 48" RCP 11 inlets	10-Year Capacity Eliminates House Flooding on Burbank Eliminates House Flooding D/S with Alternative A12
A14	L15, L16, L18 - L24	E. Burbank	2,200 feet - 48" RCP 9 inlets Minor Channelization Drainage Easement Acquisition	10-Year Capacity Eliminates Structure Flooding near Llano & W. Burbank Reduces downstream problems
A15	L16	N. Lincoln	250 feet of Berm	10-Year Capacity within Street Eliminates House Flooding Potential with Alternative A14
A18	L23 - L24	Travis	400 feet of Erosion Control 200 feet Minor Grading & Channelization	Reduces existing erosion problem
A25	L34	Ridgewood	2 - 48" RCP U/S & D/S Channel Grading	10-Year Capacity 50-Year Capacity with 1' of overtopping 100-Year Capacity with upstream detention (A24)
A26	L35	South Bowie	600 feet - 36" RCP 7 inlets	10-Year Capacity
A35	L51 and L53	Highway Street South Eagle	1,800 feet Grass Lined Trap. Channel Top Width - Approximately 30 feet 3 - 36" x 58" Arch CGMP	Eliminates Street Flooding along Highway Street (L52) Provides 10-Year Capacity at Eagle Street

TABLE 8-2
LONG-TERM DRAINAGE IMPROVEMENT ALTERNATIVES

ALTERNATIVE DESIGNATION	PROBLEM SITE DESIGNATION	LOCATION	DESCRIPTION	EFFECTIVENESS
PHASE II				
A37	L54	U. S. Highway 87	Remove old road bridge Revegetation	Reduces Erosion Eliminates backwater from structure
A38	L55	Crenwelge Road	Add box to culvert Approximately 300 feet of channel improvements	10-Year Capacity Reduces structure flooding potential
FUTURE				
A10b	L7	Schubert St.	Excavation - 3.6 acre-feet pond 1,100 feet - 24" RCP	Eliminates House/Street Flooding for 100-Year Storm
A20	L25 - L26	Morning Glory & Broadmoor	2,000 feet - 24" & 36" RCP 12 inlets	10-Year Capacity Provides 100-Year Capacity at Trailmoor and Llano with Alternative A19
A21	L13, L14, L25 - L27	North of Morning Glory	Regional Detention Pond Area - 6 acres Max. Depth - 8.5 feet 100-Yr Volume 37 acre-feet 3' x 5' Box Culvert Outlet 1,100 feet U/S Channelization 2 - 36" x 58" CGMP	Eliminates 100-Year overtopping of Llano Offsets additional discharge from A20 Reduces Street Flooding and Spillovers on N. Milam
A28	L36	Armory Road	4 - 36" x 58" Arch CGMP 400 feet of D/S Channel	10-Year Capacity
A29	L37	Basse Lane	4 - 36" x 58" Arch CGMP 250 feet of D/S Channel	10-Year Capacity

TABLE 8-2
LONG-TERM DRAINAGE IMPROVEMENT ALTERNATIVES

ALTERNATIVE DESIGNATION	PROBLEM SITE DESIGNATION	LOCATION	DESCRIPTION	EFFECTIVENESS
FUTURE				
A30	L38 and L39	Basse Lane	4 - 36" x 58" Arch CGMP 850 feet Grass Lined Trap. Channel Top Width - 30 feet	10-Year Capacity
A31	L40	South Bowie	3 - 36" x 58" Arch CGMP	10-Year Capacity

examples of these opportunities include the installation of storm drains when streets are repaved or other utilities are installed, the installation of drainage channels as part of new subdivision developments, and the installation of drainage improvements in conjunction with highway projects. Although funding restrictions may preclude implementation of many of the long-term alternatives, they are included here for general guidance purposes with respect to long-range planning by the City.

The implementation cost estimates presented in Table 8-1 for each of the recommended Phase I alternatives are preliminary and should be considered approximate. These estimates will need to be refined during the preliminary engineering design of the alternatives as they are selected for implementation by the City. The estimates account for all of the significant cost factors associated with implementing each alternative and are reasonable for the purposes of cost comparisons and planning. Work sheets itemizing the cost details for each of the alternatives are available. These work sheets present the basis for estimating the total costs for the alternatives, and they include costs for earth work, material hauling, concrete facilities construction, drain pipes and culverts, engineering and surveying, land acquisition, and contingencies.

The total estimated cost for implementing the thirteen recommended localized flooding alternatives is approximately 3.5 million dollars. This level of investment in the City's drainage system provides substantial flood protection benefits for most of the significant flooding problem sites located the City. Since many of the most serious flooding problem sites experience some degree of flooding during the occurrence of storms much smaller than the 10-year event, the adoption of the 10-year flood design capacity for most of the recommended storm drains and the 100-year flood design for detention ponds provides major improvements with regard to flooding potential and existing flooding hazards.

It should be noted that two of the recommended alternatives (A-12, North Milam Street storm drains, and A-16, North Llano Street storm drains) involve drainage improvements along State highways. While the total costs for implementing these projects are relatively high compared to those for other recommended alternatives (they represent over 35 percent of the total Phase I costs), there is some potential for cost sharing on these projects with the Texas Department of Transportation (TxDOT), since a substantial portion of the benefits to be derived from these projects relates to reduced flood flows on or across the State roadways.

Additional details regarding the various Phase I, Phase II and Future alternatives for drainage improvements and flood control measures is provided in the following sections for each of the localized flooding problem areas.

8.1.1 Friendship Lane Drainage

The combination of alternatives involving regional stormwater detention (A1c, A2, A3 & A6b) provides the most cost-effective solution in this drainage area. Combinations of alternatives involving channels and/or storm drains (A1a, A4, A5a, A6a & A7 or A1a, A4, A5b, A6a & A7) generally afford protection for storms less than the 10-year event, with implementation costs that typically are 10 to 25 percent higher than those for the regional detention alternatives. The recommended regional detention ponds also provide significant flow reductions and flood benefits for floods ranging up to the 100-year event. The significant flood reduction benefits resulting from construction of the regional ponds translate downstream without implementation of other drainage improvements, whereas channelization and storm drain projects typically need to be implemented from downstream to upstream within a given watershed to avoid creating additional flooding problems. Alternatives involving the construction of drainage channels (A5a & A7) along Friendship Lane have the additional adverse impact of consuming right-of-way that may be needed for future widening of this important roadway. Therefore, additional right-of-way purchase for the channel alternatives was included in the overall cost for comparison purposes.

8.1.2 Schubert Street Ponding

The recommended alternative (A10a) for initial implementation involves purchasing the two vacant lots, performing some minor regrading to enhance the existing detention characteristics of the depression area, and installing some additional inlets. The cost of these improvements would be less than one-sixth of that required to install adequate storm drain capacity to make the vacant lots buildable, i. e., Alternative A9, with a total cost of $185,000. In the future, these lots could be excavated to create a larger detention pond (Alternative A10b) that would provide nearly 100-year flood protection for about 80 to 85 percent of the cost of the large storm drain alternative (A9).

8.1.3 Cross Mountain - Milam Drainage

Alternative A12 is recommended to improve flooding conditions in the lower end of this drainage area. Although the total costs associated with this project are significant ($670,000), they are about seven percent less than those required to install storm drains up both Milam and Pecan Streets.

8.1.4 Burbank - Llano Drainage

The alternative for this localized flooding problem area is included in the Phase II implementation list since it is relatively expensive with respect to the amount of benefits provided.

8.1.5 North Lincoln Drainage

The berm alternative for this localized flooding problem area (Alternative A15) should be installed in Phase II at the same time the Burbank-Llano storm drain project (Alternative A14) is constructed.

8.1.6 College - Llano Drainage

Alternative A16 is recommended even though the total cost of this project is relatively high due to the large pipe size and the extensive depth of the trenching required. Even with consideration of the extra costs associated with the deep trenches, this alternative still is less than 50 percent of the cost of installing storm drains down College Street (Alternative A17) to discharge stormwater into Stream FB-1 at the eastern end of Travis Street. However, with Alternative A16, some type of stormwater detention facility located on Town Creek upstream of Llano Street would be necessary to offset the increased flood flows in the lower portion of Town Creek caused by the stormwater diversions associated with this alternative. Either of the regional detention pond alternatives (described in Section 6.3) would be sufficient to offset the flood flow increases in Town Creek associated with this alternative. If the incremental cost of the upstream regional detention required to offset the increased flood flows in lower Town Creek is assigned to the cost of this alternative, the total cost still would be less than that of Alternative A17 by about $400,000.

8.1.7 College - Travis Drainage

The College-Llano storm drain (Alternative A16) will provide significant benefits for all the localized flooding problem sites in this area.

8.1.8 Trailmoor Drainage

Alternative A19 is recommended for this drainage area; although, existing flooding problems are not particularly hazardous.

8.1.9 Morning Glory - Llano Drainage

Although the regional stormwater detention site in this drainage area is effective for reducing flood flows, it has been categorized as a Future long-term alternative. Changes in projected land use within this area or other watershed modifications may increase the implementation priority of this alternative at a later date.

8.1.10 Carriage Hills Drainage

The regional detention pond (Alternative A22) is very effective for reducing the street and structure flooding problems in this area. This alternative and Alternative A23 (storm drains along Driftwood Drive and North Adams Street) are recommended for implementation.

8.1.11 West Creek Street Drainage

The storm drain along South Edison Street (Alternative A27) is recommended since it helps to alleviate the significant floodwater ponding problem along West San Antonio Street, just west of Edison Street.

8.1.12 Old Harper Road Drainage

The alternatives for this area are all considered to be Future alternatives since the need for these drainage improvements is somewhat dependent upon the manner in which development occurs. The alternatives identified for this area serve as a general guide for future drainage improvements; therefore, plans for specific development projects in the area may necessitate some adjustments and modifications in the alignments and capacities of the proposed drainage improvements.

8.1.13 Winfried Creek Drainage

No specific alternatives have been identified for this drainage area since no major flooding problems exist.

8.1.14 Five Points Area

Two drainage improvement projects (Alternatives A32 & A33) are recommended for this area because of the significant amount of flooding and the relatively high volume of traffic that occurs through this problem area. The final alignment of the 42-inch storm drain is somewhat dependent on acquisition of easements; however, the overall cost should not vary significantly.

8.1.15 South Adams Drainage

No specific drainage improvement alternatives have been identified for this area since no major flooding problems exist.

8.1.16 Highway - Apple Drainage

The most significant flooding problem sites within this area appear to be along Apple and Pear Streets (Site L52). Most of the flooding problems can be eliminated through implementation of the recommended alternative (A36).

8.1.17 Dry Creek Drainage

The potential flooding conditions in this area do not represent a significant immediate problem. However, the identified alternatives should be considered as part of the Phase II implementation program.

8.2 STREAM FLOODING PLAN

Three of the drainage and flood improvement alternatives previously identified and discussed in Section 6.2 are recommended. These alternatives include two regional detention ponds, one each on Town Creek and Stream FB-1, and a culvert replacement project on Stream FB-1 at Lower Crabapple Road, and they are listed and generally described, with estimated implementation costs, in Table 8-3. These alternatives provide effective benefits with regard to the most significant stream flooding problem sites, and they should be included as part of the initial Phase I of the overall master drainage plan for the City. As indicated in Table 8-3, the total estimated cost of the three recommended stream flood protection alternatives is nearly two million dollars. Additional stream flood protection alternatives that are considered to be less critical and, therefore, more long-term projects are listed and generally described in Table 8-4.

Further discussion of the various alternatives available for drainage and flood improvements along the principal watercourses in the planning area is presented in the following sections.

8.2.1 Town Creek

The most cost-effective means for reducing flooding along Town Creek is construction of the large regional detention pond on the western tributary to upper Town Creek (Alternative A42). This detention facility will reduce the 100-year flood water surface along most of Town Creek by nearly two feet. Based on hydraulic analyses performed with the revised HEC-2 model of Town Creek, this alternative would produce lower flood levels at most locations along Town Creek than would result if several of the roadway crossings were replaced with larger bridges, the effects of which typically would occur only over very short reaches (less than 2,000 feet) upstream of the bridges. Furthermore, the cost of this regional detention pond alternative ($1,170,000) would be approximately equal to the cost of replacing two roadway crossings with bridges. Therefore, this regional detention pond can provide more flood level reduction benefits for more of Town Creek than replacement of any two roadway structures on Town Creek. Also, with this regional detention pond in place, Alternative A16 (storm drains) could be implemented to reduce the flooding problems at and downstream of Llano and College Streets. For these reasons, Alternative 42 is recommended for implementation as a Phase I project.

The regional detention pond located near Cross Mountain West (Alternative A43) is not

TABLE 8-3

RECOMMENDED STREAM FLOOD PROTECTION ALTERNATIVES

ALTERNATIVE DESIGNATION	LOCATION	DESCRIPTION	EFFECTIVENESS	IMPLEMENTATION COST
A42	Town Creek - West Tributary U/S of N. Cherry	Regional Detention Pond Area - 19 acres Max. Depth - 10.4 feet 100-Year Volume - 105 acre-feet Outlet 4 - 3' x 5' Box Culverts	Reduces 100-Year Storm flow 57% at the site Reduces 100-Year flows by 1,450 - 1,600 cfs in Town Cr. Eliminates overtopping of Milam and Crockett for storms smaller than the 10-Year storm Reduces 100-Year water surface elevation by 0.5 - 3.0 feet from just below Morse Road to Barons Creek confluence	$1,170,000
A44	Stream FB-1 Upstream of Lower Crabapple Road and Carriage Hills	Regional Detention Pond Area - 9 acres Max. Depth - 8.5 feet 100-Year Volume - 36 acre-feet Outlet 7 - 4' x 6' Box Culverts	Reduces 100-Year Storm flow 23% at the site Reduces 100-Year flood elevation by 0.5 - 1.0 feet through the Carriage Hills problem site Eliminates Llano overtopping for the 10-Year storm	$665,000
A41	Stream FB-1 Lower Crabapple Road	4 - 53" x 85" Arch CGMP 700 feet D/S Channelization	50-Year Capacity without overtopping. 100-Year Capacity with upstream detention (A44)	$110,000
			TOTAL	$1,945,000

TABLE 8-4
LONG-TERM STREAM FLOOD PROTECTION ALTERNATIVES

ALTERNATIVE DESIGNATION	LOCATION	DESCRIPTION	EFFECTIVENESS
PHASE II			
A40	Morse St. Town Creek	4 - 8' x 8' Box Culvert	100-Year Capacity without overtopping.
FUTURE			
A43	Town Creek At Cross Mountain West	Regional Detention Pond Area - 6 acres Max. Depth - 10 feet 100-Year Volume - 11 acre-feet 4 - 54" RCP Low Flow Outlet Concrete Weir Length - 94 feet	Reduces 100-Yr Storm flow 9% at the site Offsets increase from existing to fully-developed cond. Reduces the amount and frequency of overtopping at West Morse Eliminates overtopping of Milam and Crockett for the 10-year storm with Alternative A42

recommended for implementation in Phase I because it is not nearly as effective for reducing flood levels downstream along Town Creek as Alternative 42. The cost per unit flow reduction of Alternative A43 is over six times more expensive than that of the western tributary regional detention pond (Alternative A42). It also has a minimal effect on flood levels along most of Town Creek, although it does produce some significant flood level reductions in the short reach just downstream of the pond site and upstream of the confluence with the western tributary. It does not reduce the 10-year flood flow sufficiently to eliminate overtopping of Morse Road. The cost of the Alternative A43 detention pond is significantly more expensive than the cost of replacing the existing Morse Road tank car culvert with a set of concrete boxes (4 - 8' x 8') that can pass the 100-year flood flow without overtopping. Furthermore, implementation of the Cross Mountain West pond (Alternative A43) is not critical since it is not immediately needed to offset the flood flows that would be diverted into Town Creek by Alternative A13 (West Burbank Street storm drain) since Alternative A13 is not included as part of the recommended alternatives for Phase I. Therefore, this alternative is not recommended, at least for immediate implementation.

Although the Morse Road culvert replacement project (Alternative A40) is a cost-effective measure to eliminate road overtopping, it is not recommended at this time, but should be considered for implementation as part of the Phase II program.

8.2.2 Stream FB-1

The regional detention pond on Stream FB-1 upstream of Lower Crabapple Road (Alternative A44) is very effective for reducing stream flooding problems downstream through the Carriage Hills subdivision, and it is recommended for installation as part of Phase I. The culvert replacement for Lower Crabapple Road (Alternative A41) is also recommended along with the associated downstream channel improvements. The combination of these drainage improvement projects will prevent overtopping of Lower Crabapple Road for floods up to the 100-year flood event.

PLATES