HYDRAULIC STUDY REPORT

NOTRE DAME BOULEVARD OVER LITTLE CHICO CREEK,

MP NORTHFORK, LLC

CITY OF CHICO, CA

CENTRAL VALLEY FLOOD PROTECTION BOARD ENCROACHMENT PERMIT APPLICATION

NORTHSTAR

...Designing Solutions

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REPORT PREPARER AND ENGINEER'S STATEMENT

This report has been prepared under the direction of the following Registered Civil Engineer. The Registered Civil Engineer attests to the technical information contained herein, and the engineering data upon which recommendations, conclusion, and decisions are based.

Prepared By:

Prepared By: Kirk Koester

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NorthStar

Date June 30, 2021

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This bridge hydraulic analysis has been prepared for the distinct purpose of meeting the requirements of 23 CCR § 128, 23 CFR 650 Subpart A, Section 650.11(b)(c)(d), 23 CFR §650.115 and §650.118 dealing with bridges, structures and hydraulics. Although potentially useful for other purposes, this analysis has not been prepared for any other purpose and reuse for any other purpose is not endorsed nor encouraged by the author. Said reuse of this work is at the sole risk of the entity reusing the information contained herein.

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EXECUTIVE SUMMARY

Note: All elevations here within this report are based on the topography survey datum of NGVD29. This report doesn't account for any falsework.

Project Context/ Purpose:	Install a new bridge over Little Chico Creek with pedestrian undercrossing
Scope of Study:	CVFCB Encroachment Permit application
Project Funding Program:	Private
Design Discharges:	**Base Flood Discharge = 2,400cfs (CVHS Q_{200})** FEMA (Q_{200}) = 2,400 cfs FEMA (Q_{100}) = 2,200 cfs FEMA (Q_{50}) = 2,000 cfs **Scour Analysis Discharge = 2,400cfs (CVHS Q_{200}), Q_{500} = 2,500 cfs**
Freeboard/ Drift Requirements:	**3.0 feet at Q_{200} (CVFPB [23 CCR § 128 (a)(10)(A)** 2.0 feet at Q_{50} (Caltrans Highway Design Manual, Chapter 820) 0.0 feet at Q_{100} (Caltrans Highway Design Manual, Chapter 820)
Design Exceptions:	None noted for hydraulic design/ modeling.

HYDROLOGIC CONDITION

The Little Chico Diversion Structure project southeast of Chico, CA, constructed by the Corps of Engineers in the 1950s, diverts peak discharges from Little Chico Creek to the south into the Little Chico Creek Diversion Channel/ Butte Creek by bifurcating flows at the Little Chico Creek Diversion Channel (O&M Manual SAC516). This diversion structure is comprised of two hydraulic structures; four parallel sluice gates that maintain base flow in Little Chico Creek and an ungated sharp crested weir that bypasses flood discharges south to the Little Chico Creek Diversion Channel which flows to Butte Creek near Highway 99. Original hydraulic design flows for the portion of Little Chico Creek downstream of the weirs are identified in the USACE Design Memorandum (USACE, 1960) for this structure as being 2,200 cfs, which was the limiting flow for the creek (CVHS, 2014). Additional investigation was performed to better determine the 200-year discharge at the project site. The bridge project is located within the City of Chico urban area and is within the State Plan of Flood Control. Compliance with the FloodSAFE legislation requires the evaluation of the 200-year return of frequency storm at the project site. The 200-year discharge was determined by using the best available data for this system, including FEMA analysis and Central Valley Hydrology Study (CVHS). Additionally, this bridge is required to meet the minimum freeboard requirements set forth by the Federal Highway Administration (FHWA) CalTrans. The design criteria are the 2% probability flood (Q_{50}) with 2 feet of freeboard and the 1% probability flood (Q_{100}) without freeboard (without causing objectionable backwater, excessive flow velocities or encroaching on through traffic lanes). The 50 and 100-year discharges were determined from the FEMA analysis conducted on Little Chico Creek.
This report doesn't account for any falsework.

KEY FINDINGS

Proposed Bridge Description:	100 feet long, 56 feet wide, 3-span reinforced concrete slab, with 2.0% Cambered 20" thick concrete deck, supported on multiple pile bents and vertical reinforced concrete wall abutments

Proposed Bottom Soffit Elevation =	255.39’ (center of roadway); 253.13’ (upstream controlling bridge member) 253.19’ (downstream controlling bridge member)
Proposed Floodplain Water Surface Elev. =	FEMA Q_{50}: 249.24’ (upstream), 248.88’ (downstream) FEMA Q_{100}: 249.47’ (upstream), 249.09’ (downstream) **Base Flood (CVHS Q_{200}): 249.69’ (upstream), 249.28’ (downstream)**
Proposed WSE/Soffit Clearance =	$Q_{(50)}$ = 3.89’ (FEMA [2,000 cfs]) $Q_{(100)}$ = 3.66’ (FEMA [2,200 cfs]) **$Q_{(200)}$ = 3.44’ (Base Flood [2,400 cfs])(3.0' min per CVFPB reqs)**
Existing Floodplain Water Surface Elev. =	$Q_{(50)}$ = 249.25’ (FEMA [2,000 cfs]) $Q_{(100)}$ = 249.50’ (FEMA/CVFPB [2,200 cfs]) **$Q_{(200)}$ = 249.73’ (Base Flood [2,400 cfs]))**

1 INTRODUCTION

NorthStar was retained to provide technical civil design and hydraulic analysis for a new bridge project across Little Chico Creek on Notre Dame Blvd located between Humboldt Road and East 20th Street in Chico.

To better estimate the hydraulic conveyance capacity for both the existing and proposed condition, a one-dimensional backwater HEC-RAS numerical hydraulic model was developed. This report serves to summarize the methodology and results of the model to aid in the review of the permitting process and support technical design recommendations. Additionally, the function of this report provides historical context, a brief project description, and channel hydrology for this portion of Little Chico Creek.

1.1 STUDY PURPOSE

This hydraulic study serves to accompany an encroachment permit to the Central Valley Flood Protection Board for a new bridge across Little Chico Creek on Notre Dame Blvd, Chico, Butte County California. More specifically this bridge will be a critical link to area wide circulation, this bridge will provide significant relief for of the nearby arterials, Bruce Road and Forest Avenue. The proposed bridge will accommodate travel lanes, bike lanes, and sidewalks.

LOCATION HYDRAULIC STUDY

The purpose of this hydraulic study is to evaluate the project's impacts to the prevailing hydraulic conditions and the associated risk. If needed, this study serves to accompany the NEPA environmental review for the proposed project pursuant to 23 CFR 771. Referred to as the Location Hydraulic Study for this purpose, this study serves to:

- Determine if this project will encroach into the FEMA designated floodplain.
- Evaluate and discuss potential alternatives to any longitudinal encroachments.
- For the proposed structure that contains an encroachment and for those actions which would support base floodplain development, a discussion of the following items (commensurate with the significance of the risk or environmental impact):
    - The risks associated with implementation of the action,
    - The impacts on natural and beneficial floodplain values,
    - The support of probable incompatible floodplain development,
    - The measures to minimize flood-plain impacts associated with the action, and
    - The measures to restore and preserve the natural and beneficial floodplain values impacted by the action.
- Evaluation and discussion of the ***practicability*** of alternatives to any significant encroachments or any support of incompatible floodplain development.
- Consultation with local, state, and federal water resources and floodplain management agencies to determine if the proposed roadway action is consistent with existing watershed and floodplain management programs.
- Discuss emergency access data, availability of detours, as applicable.

The above items are pursuant to 23 CFR 650, Subpart A, Sections 650.111 (c) and (d) must be summarized in environmental review documents prepared pursuant to 23 CFR 771.

NFIP ENCROACHMENT REVIEW

Similar to the above, this analysis is also expected to facilitate NFIP Encroachment Review by the local Floodplain administrator. The methods and findings here within provide floodplain administrator with information to assist in this review process.

DESIGN SUPPORT

In addition to providing the above items to the hydraulic condition in the project environmental review, this analysis also functions to provide project designers with pertinent estimates of the hydraulic and scour conditions for various discharge events.

CENTRAL VALLEY FLOOD PROTECTION BOARD

Similarly, this hydraulics analysis will also be applied with and supplement a Central Valley Flood Protection Board encroachment permit and/or maintenance letter to provide additional insight into hydraulics associated with this structure.

1.1.1 APPLICABLE CRITERIA

This is a local-funded project and is administered by the City of Chico following protocols provided by City of Chico Ordinance and Central Valley Flood Protection Board (CVFPB). Additional references are cited within the text of the document.

The hydraulic design and methodology discussed within this report follow industry standards, appropriate model reference documents, and criteria set forth by the CVFPB in 23 CCR § 128 (a)(10)(A), (policies and procedures for the location and hydraulic design of highway encroachments), Caltrans Highway Design Manual, Chapter 800, 820 and 840, Local Assistance Procedures Manual (Chapter 11), Federal Emergency Management Agency (FEMA) National Flood Insurance Program (NFIP) regulations (44 CFR Parts 59 and 60), along with any local drainage and hydraulic design or modeling criteria provided. Additional references are cited within the text of this document. Hydraulic design criteria for channel crossings is specifically addressed in the Caltrans Highway Design Manual, Chapter 820 “Cross Drainage” (Caltrans, 2020), where two conveyance criteria for bridges are presented:

1. design flood, (by definition the roadway will not be inundated from the stage of the design flood) for bridges is based on the 50-year discharge event with 2 feet of drift/freeboard, and
2. the flood or tide having a one percent (1%) chance of being exceeded in any given year (100- year flood) without freeboard.

Despite reference to FEMA regulatory floodplains in 23 CFR 650, the base flood is not necessarily the FEMA determined 100-year flood. Additionally, City of Chico’s storm drainage ordinance (Chapter 4.3, *Storm Drainage*) considers the 200-year discharge as the Base Flood and 3-feet of freeboard shall be added to the corresponding water surface elevation(s) when evaluating encroachments. This matches the FloodSAFE and CVFPB requirements.

This structure is within the State Plan of Flood Control, and as such is subject to the criteria set forth in 23 CCR § 128 (a)(10)(A) administered by the CVFPB. Criteria is noted to maintain a three (3) foot freeboard above the design flood water surface elevation.

The design flood for this study is a 200-year return frequency discharge based on recent CVHS, CVFPB O&M Manual, and FEMA analysis. The 100-year return frequency discharge was based on the Flood Insurance Study (FIS) for Butte County conducted by FEMA in 2011.

All elevations here within this report are based on the topography survey datum of NAVD 88.

1.2 Project Location and Context

The Notre Dame Blvd Bridge is located approximately 320 feet north of Emerson Way and 2,000 feet south of Humboldt Road. This structure will be located between two existing crossings of Little Chico Creek, downstream at Forest Ave and upstream at Bruce Road, both crossings are within half a mile of the proposed bridge. The proposed crossing is the second most upstream (eastern) crossing of Little Chico Creek within the city of Chico, CA. (Latitude: 39°44'05.88"N, Longitude: 121°47'43.86"W [WGS 84]). The project site and corresponding channel reach is situated in an urban, primarily residential, with natural bank features setting. The study reach spans from 375 feet downstream to 325 feet upstream of the Notre Dame Bridge. The Little Chico Creek Diversion Structure is located approximately 6,900 feet upstream of the project site.

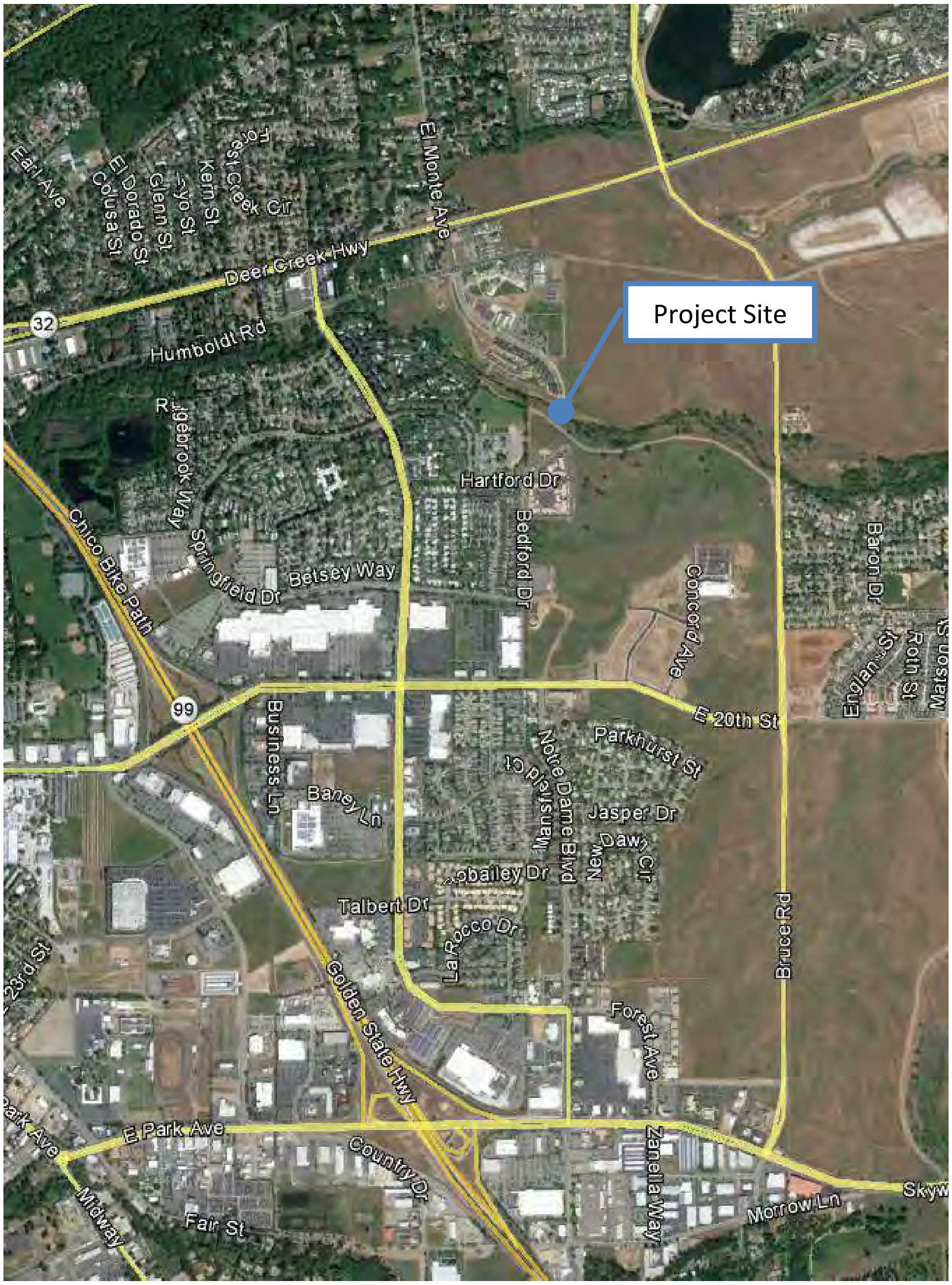

Figure 1: Project Location/ Vicinity Map

Upstream of the bridge, channel cross-sectional geometry, planform, and gradient are more consistent with pool-riffle channel morphology. There is a moderately sized floodplain bench on river-left[1] upstream of the bridge that inundates during large storm events. Downstream of the bridge, the channel gradient is relatively uniform, and planform is straight with no pool-riffle or other geomorphic characteristics which suggests anthropogenic conditions. Upstream of the bridge, on both channel banks is vacant land covered mostly with grasses and thin to medium sized trees, the channel banks are moderately to heavily vegetated with medium to large

[1] Channel orientation reference throughout this document is based on viewing downstream

diameter trees and short riparian grasses. The upstream left bank slope is gentle, 3:1 or greater(H:V), and the upstream right bank slope is moderate, 2:1 (H:V). Downstream of the bridge, on both channel banks is vacant land covered mostly with grasses and thin to medium sized trees, the channel banks are moderately vegetated with medium to large diameter trees and short riparian grasses, with moderate, 2:1 (H:V), bank slopes.

1.2.1 EXISTING CONDITION

There is no existing bridge (vehicle or pedestrian) at the project location. The existing site topography, at the proposed bridge location, consists of vacant land covered mostly with riparian grass and a couple medium to large diameter trees. The left (looking downstream) areas are inundated during larger (10+/-) storm events, with bank slopes of approximately 3:1. The upstream, right bank slopes were observed to be approximately 2:1, with no evidence of overbank/floodplain inundation. The City of Chico bike path is located approximately 150' south of the creek centerline. Notre Dame Blvd, on the north, currently terminates approximately 135 feet prior to the creek, and terminated approximately 320 feet prior. Notre Dame Blvd terminates approximately 135 feet northly and 320 feet southerly of the creek centerline.

Figure 2: Existing Conditions (2020). Looking North.

1.2.2 PROPOSED CONDITION

The proposed structure is 100 feet long three span, 56 feet wide, reinforced concrete column with reinforced concrete decking. The proposed bridge is designed to accommodate travel lanes, bike lanes, sidewalks, and appropriate safety barriers. The bridge will be skewed to efficiently match the existing channel configuration. There will be a Class I bike path crossing under the proposed structure along the south bank of Little Chico Creek. The proposed structure length is designed to optimally span the creek and bike path. The proposed structure deck, elevation, and profile are vertically aligned to match existing roadway profile north of the creek. The structure width is offset to the upstream side of the proposed roadway alignment to accommodate the multi-use sidewalk / bikeway. This report doesn't account for any falsework.

Work elements within the channel prism involved in the replacement project include:

- Removal of two existing trees and several medium sized bushes, and mitigation per the requirements of the California Environmental Quality Act (CEQA);
- Removal and relocation of existing City of Chico bike path to cross under proposed bridge;
- Revegetation of disturbed areas (cut/fill areas);

- Installation of new foundation and abutments, two bents, reinforced concrete column with reinforced concrete decking, temporary falsework to facilitate bridge assembly and temporary staging of utilities across the channel;
- Grading of the channel bank slopes and installation of RSP within the proposed bridge outline.

1.2.3 Watershed and Proximity within the State Plan of Flood Control

The Little Chico Creek watershed is approximately 132.8 square miles in size, and drains portions of the Sierra foothill area east of Chico, and flows westerly from Chico to the Sacramento River. Floodwaters are managed in Little Chico Creek by the Little Chico Creek Diversion Structure which functions to divert channel discharges above the design discharge to the Little Chico Creek Diversion Channel connecting to Butte Creek. The diversion structures are maintained by and the Little Chico Creek Channel is maintained by DWR- Sutter Maintenance Yard (DWR 2016).

1.2.3.1 Channel Morphology

The study reach of Little Chico Creek can be considered straight upstream of the bridge for approximately 200 feet and then bends slightly to the right (looking downstream), downstream can be considered straight, except for a minor bend to the left (looking downstream) approximately 230 feet downstream of the bridge. Upstream of the bridge, channel morphology can be described as a relatively incised (major flows contained well within the channel prism), with two separate low-flow channels which rejoin approximately 200 feet upstream of the bridge. Approximately 200 feet upstream of the bridge the channel morphology transforms towards a regular, trapezoidal channel (major flows contained within the channel banks). Downstream of the bridge, the channel morphology can be described as a regular, trapezoidal channel (major flows contained within the channel banks). The downstream extents of the study reach terminates approximately 80 feet downstream of the slight left bend in the channel.

1.2.3.2 Streambed Materials

Streambed substrate observed in the channel bed was primarily coarse gravel (3/4” < 3” diameter) to cobbles (3” < 12” diameter). Pebble counts or other quantification were not performed with this effort. However, observations on the relative bed roughness heights were made during low flows in January of 2021 to facilitate appropriate roughness coefficients.

2 HYDROLOGIC CONDITION

2.1 LITTLE CHICO CREEK DIVERSION STRUCTURE

As part of the State Plan of Flood Control, discharge within Little Chico Creek at the project reach is regulated by the Little Chico Creek Diversion Structure approximately 6,900 feet upstream of the project site. Constructed in the 1950s, two flood diversion structures bifurcates Little Chico Creek into two branches – Little Chico Creek and Little Chico Creek Diversion Channel. Based on the design rating curve for the structure (USACE, 1957), the Little Chico Creek Diversion Channel conveys approximately 65% of the 200-year return frequency storm discharge. The hydraulic structure that controls discharges to Little Chico Creek (to the project site) is a series of four uncontrolled narrow sluice gates.

The Little Chico Creek sluice gates function to both limit flood discharge and maintain low (non-flood) discharge down Little Chico Creek (to the project site). As channel discharge and stage increase, the hydraulic capacity of the sluice gates become limited and flood discharge is diverted over the sharp crested, ogee weir into the Little Chico Creek Diversion Channel and ultimately Butte Creek.

Figure 3: Little Chico Creek diversion structure; sluice gates (near) and Little Chico Creek Diversion Channel diversion weir (distance).

Figure 4: Photograph of Little Chico Creek Sluice Gates, looking downstream.

Figure 5: Looking upstream at the Little Chico Creek Diversion Channel diversion weir

Table 1 demonstrates how the flows above the diversion structure are split between Little Chico Creek and Little Chico Creek Diversion Structure.

Table 1: Little Chico Creek Inflow Diversion Function, taken from CVHS (USACE, 2014).

Inflow (cfs) (1)	Diversion to Little Chico Creek Diversion Channel (cfs)	Diversion to Little Chico Creek (cfs)	% Discharge to Little Chico Creek Diversion Channel
0	0	0	0%
888	0	888	0%
1,167	107	1060	9%
1,568	327	1241	21%
2,468	1,018	1450	41%
3,046	1,472	1574	48%
3,445	1,780	1665	52%
3,714	1,990	1724	54%
4,424	2,562	1862	58%
5,005	3,040	1965	61%
5,158	3,168	1990	61%
5,900	3,790	2110	64%
6,585	4,360	2225	66%
6,712	4,465	2247	67%
6,750	4,497	2253	67%

1. Inflow diversion table based on Little Chico Creek Diversion Structure discharge rating curve

2.2 Flood Discharges

As noted previously, this analysis serves to address multiple regulatory functions which vary most in return frequency or specific flood discharges, and drift/ freeboard criteria. These different discharges are discussed in more detail below.

Location Hydraulic Study Design Flood

Per the Caltrans Highway Design Manual:

Design Flood. The peak discharge (when appropriate, the volume, stage, or wave crest elevation) of the flood associated with the probability of exceedance selected for the design of a highway encroachment. By definition, the roadway will not be inundated by the design flood. In a FEMA floodplain, see 23 CFR, Part 650, Subpart A, for definitions of "overtopping flood" and "base flood."

CVFPB Design Flood Discharge

This hydraulic study was developed to accompany an encroachment permit for bridge replacement activities within the Adopted State Plan of Flood Control, and as such it is important to recognize the California Code of Regulations regulating this system 23 CCR § 4 (j) “Definitions” that describes the “Design Flood” as the following:

> *“(j) Design Flood. “Design flood” means the flood against which protection is provided or may eventually be provided by means of flood protection or control works, or that flood which the board otherwise determines to be compatible with future developments.”*

Additionally,

"(k) Design Flood Plane. "Design flood plane" means the water surface elevation at design flow as determined by the Army Corps of Engineers, the Board, or Federal Emergency Management Agency, or other higher elevations based upon best available information, as determined by the board."

FEMA

This project is within a NFIP designated Zone AE which is by definition regulated by the 100-year return frequency (or 0.01 exceedance probability). Discharge for this discharge condition was taken from the published FIS for this channel (FEMA 2011)

Urban Level of Protection

The project site is located within the Chico Urban Area, and in conformance with State FloodSAFE legislation, improvements within the system are required to consider and pass the 200-year return frequency (0.5% exceedance probability) discharge as the "design flood". No streamflow gauges were available on Little Chico Creek to perform statistical analysis of streamflow. Estimation of peak channel discharges for the purposes of this report utilized the Central Valley Hydrology Study and FEMA's Flood Insurance Study (FIS) for this basin.

The CVHS evaluated the hydrologic relationship of the Butte Creek system including Little Chico Creek (CVHS, 2014). The CVHS developed and calibrated HEC-HMS models to better estimate peak discharges in the Butte Creek system. Peak stream flows were evaluated at the Little Chico Creek Diversion Structure ("Little Chico Creek Diversion", approximately 6,900 feet upstream of the project bridge [Figure 6]) and are presented in Table 2 below. The flow into Little Chico Creek is the difference between the inflow and diversion to Butte Creek.

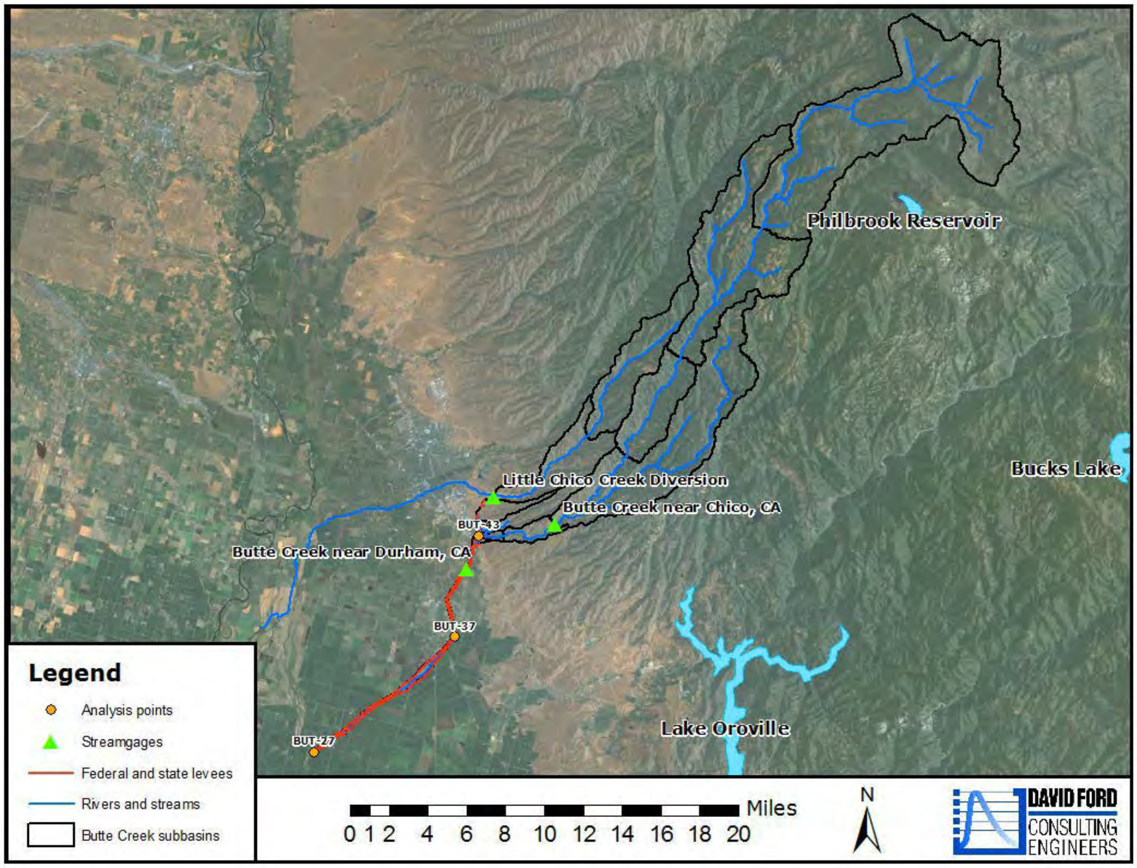

Figure 6 Aerial Map of Little Chico Creek Watershed, taken from CVHS (USACE, 2014)

Table 2: Little Chico Creek Inflow Diversion Function, taken from CVHS (USACE, 2014).

Inflow (cfs)[1]	Diversion to Butte Creek (cfs)	Diversion to Little Chico Creek (cfs)
0	0	0
888	0	888
1,167	107	1060
1,568	327	1241
2,468	1,018	1450
3,046	1,472	1574
3,445	1,780	1665
3,714	1,990	1724
4,424	2,562	1862
5,005	3,040	1965
5,158	3,168	1990
5,900	3,790	2110
6,585	4,360	2225
6,712	4,465	2247
6,750	4,497	2253
1. Inflow diversion table based on Little Chico Creek Diversion Structure discharge rating curve		

FEMA's FIS for Little Chico Creek (FEMA, 2011) utilized a HEC-1 modeling to estimate peak discharges for the basin and Little Chico Creek. The estimated peak discharges published in the FIS are provided below in Table 3. Reviewing the flows listed in the table for the row titled 'Below Diversion Structure', we think this is an error and should be title as 'Above Diversion Structure' based on the data available from the CVHS and other sources.

Table 3: FEMA FIS Peak Discharge for Little Chico Creek. Taken from FEMA, 2011, Table 3, pg. 13.

		Peak Discharges (cfs)			
Flooding Source and Location	Drainage Area (sq mi)	10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
LITTLE CHICO CREEK					
Below Diversion Structure	*	2,300	4,400	5,600	7,800
At Forest Avenue	*	1,500	2,000	2,200	2,500
At State Highway 99	*	2,100	3,400	3,700	*
Approximately 100 feet above Bruce Street	*	2,100	3,400	3,500	3,700
At Bruce Street	*	2,200	3,100	3,100	3,100
At Mills Street	*	2,200	2,800	2,800	2,800

The CVHS reviewed other previous hydrologic analysis to evaluate their findings. In particular, two USACE studies on the minor tributaries of the Sacramento River (including Little Chico Creek) from 1957 and a study of Little Chico Creek from 1963. Both studies found the creek downstream of the diversion structure is limited in capacity to 2,200 cfs. With this information, we determined the flows from the FEMA FIS 'below the diversion structure' to be an error and should instead be 'above the diversion structure'. This conclusion is appropriate since at the location 'Forest Avenue' which is approximately 10,000 feet downstream of the diversion structure (and approximately 2,600 feet downstream of the Bruce Road crossing), the flows are significantly lower. There are no diversions off Little Chico Creek between the diversion structure and Forest Avenue to observe such significant reduction of storm flows. Additionally, the 1 percent (100-year) flows at Forest Avenue of 2,200 cfs are similar to those described in the CVHS study for Little Chico Creek at the Little Chico Creek Diversion Structure.

In summary, review and analysis of the CVHS and FEMA studies on the Little Chico Creek, the 200-year peak streamflow at the site is estimated at 2,400 cfs.

CVFPB Design Discharge

The published design discharge for Little Chico Creek is 2,200 cfs.

FEMA

The published 100-year exceedance probability is 2,200 cfs (Q_{100}) and the 50-year exceedance probability is 2,000 cfs (Q_{50}).

3 HYDRAULIC ANALYSES

3.1 MODELING OVERVIEW

The Army Corps of Engineers' Hydrologic Engineering Center's (HEC) River Analysis System (HEC-RAS) software performs one-dimensional hydraulic backwater calculations and other hydraulic analysis. Version 5.0.7 of HEC-RAS was used for the hydraulic analysis of both the existing condition and design condition. The pre-model development understanding that flows remained contained within the channel and exhibited gradually varied flow conditions, the approximations associated with a one-dimensional backwater model deemed appropriate for hydraulic analysis of this channel. Additionally, this particular one-dimensional software was selected for use due to the long history and industry acceptance of use, robust computational performance and flexibility associated in evaluating bridge design configurations.

3.1.1 MODELING APPROACH

The methodology of modeling closely followed the guidance provided by the User's Manual and Hydraulic Reference Manual associated with this software version. Several model input parameters exist for both the existing and design conditions. These common input parameters of note are discussed further below in subsequent sections.

3.1.2 VERTICAL DATUM

Elevations referenced in this study are based on the National Geodetic Vertical Datum of 1929 (NGVD 29). Topography Survey provided by the Owner (and performed by NorthStar), is on the City's local datum. Elevations referenced in this study were adjusted by +0.53' to get to the NGVD 29 datum.

3.2 CROSS-SECTIONAL GEOMETRIC DATA

Cross-sectional data was obtained from field topographic survey data, and to capture bathymetric data, which was completed in the winter of 2020. All cross-sectional information used in this model was developed from observations and physically surveyed data.

3.2.1 BRIDGE ORIENTATION

The proposed bridge follows a similar alignment as to optimally connect the existing roadway on either side of Little Chico Creek. The proposed structure is designed to connect the existing travel lanes and pedestrian paths on Notre Dame Blvd on either side of the creek. This will accommodate an upgraded (and safer) travel corridor that has designated pedestrian and bicycle facilities. The footprint is offset to the, referenced from the roadway centerline, upstream side of the proposed structure. The bridge will be skewed to efficiently match the existing channel configuration. See Appendix A for more information. Since there is no existing structure, to appropriately model the existing condition unique cross sections were developed for both the existing and proposed conditions, with respect to the upstream, downstream, and bridge cross-sections in accordance with modeling best practices and aforementioned user manual. A direct comparison under the proposed structure is not possible due to the differences in cross-section layout between the existing and proposed conditions. A conscience decision was made to develop more accurate cross sections rather than promote potentially less accurate results. Figures 7 and 8 illustrate the *approximate* relative difference between the spatially closest existing and proposed cross sections; note the cross section are not sampling the exact same location. Please refer to Appendix A for a more detailed cross section comparison for this crossing.

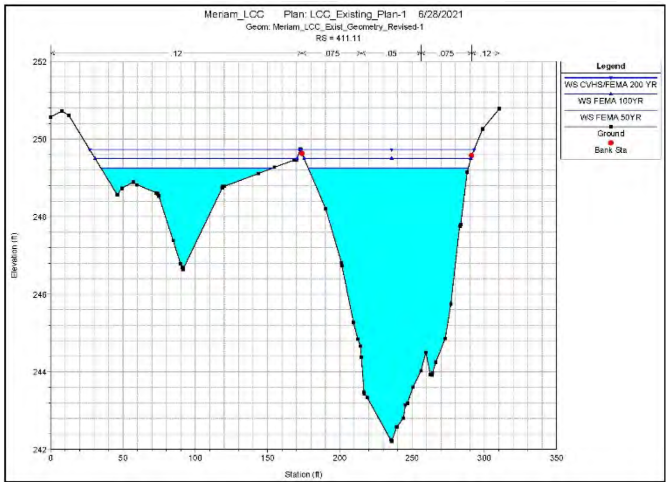

Figure 7: Existing condition cross section; taken from HEC-RAS geometry editor.

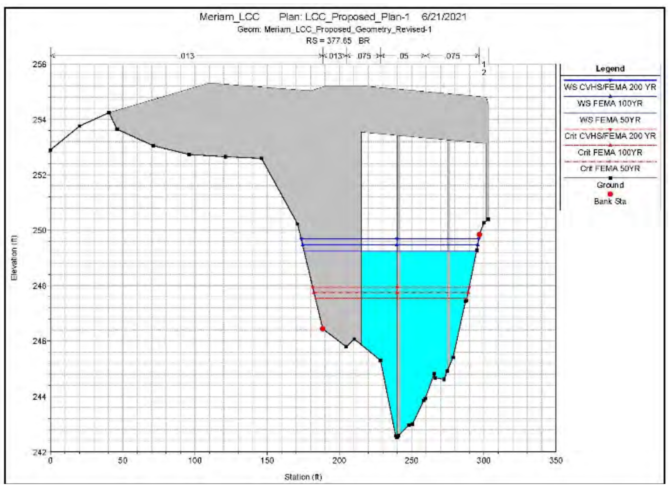

Figure 8: Proposed condition cross-section; taken from HEC-RAS geometry editor.

3.2.2 ROUGHNESS VALUES

HEC-RAS utilizes Manning's n roughness coefficient values to account for total roughness. Roughness values were estimated by observation, professional opinion and reference to the Robust Prediction of Hydraulic Roughness and associated HYDROCAL spreadsheet (USACE, 2011) and as described in related HEC reference manuals. Based on a distinct difference between grain and vegetation roughness elements observed in the project reach, roughness values were spatially segregated to represent these areas. Roughness values were consistent between the existing and proposed models as the channel bed material will remain. Higher roughness values were developed for the channel overbanks to account for strong influence of vegetation. Refer to Appendix A for exhibit illustrating extents of roughness areas, and Appendix C for site photographs. Vertical variations in roughness values were not established for this model.

Edit Manning's n or k Values

River: LCC | Edit Interpolated XS's | Channel n Values have a light green background
Reach: RAS | All Regions

Selected Area Edit Options: Add Constant ... | Multiply Factor ... | Set Values ... | Replace ... | Reduce to L Ch R ...

	ver Statio	rctn (n/K)	n #1	n #2	n #3	n #4	n #5	n #6	n #7	n #8	n #9
1	693.07	n	0.12	0.013	0.12	0.075	0.05	0.075	0.05	0.075	0.12
2	671.79	n	0.12	0.013	0.12	0.075	0.05	0.075	0.05	0.075	0.12
3	653.79	n	0.12	0.013	0.12	0.075	0.05	0.075	0.05	0.075	0.12
4	637.96	n	0.12	0.013	0.12	0.013	0.12	0.075	0.05	0.075	0.12
5	611.2	n	0.12	0.013	0.12	0.013	0.12	0.075	0.05	0.075	0.12
6	552.6	n	0.12	0.013	0.12	0.013	0.12	0.075	0.05	0.075	0.12
7	501.91	n	0.013	0.12	0.075	0.013	0.075	0.05	0.075	0.12	
8	428.29	n	0.013	0.013	0.075	0.05	0.075	0.12			
9	377.85	Bridge									
10	326.31	n	0.12	0.013	0.075	0.05	0.075	0.12			
11	286.88	n	0.12	0.013	0.12	0.075	0.05	0.075	0.012		
12	266.95	n	0.12	0.013	0.12	0.075	0.05	0.075	0.12		
13	229.39	n	0.12	0.013	0.12	0.075	0.05	0.075	0.12		
14	190.29	n	0.12	0.075	0.05	0.075	0.12				
15	151.66	n	0.12	0.075	0.05	0.075	0.12				
16	126.11	n	0.12	0.075	0.05	0.075	0.12				
17	82.75	n	0.12	0.075	0.05	0.075	0.12				
18	41.88	n	0.12	0.075	0.05	0.075	0.12				
19	0.5	n	0.12	0.075	0.05	0.075	0.12				

Figure 9: Table of roughness values; existing and proposed conditions. Taken from HEC-RAS Geometry Editor.

3.2.2.1 Channel Bed

Visual inspection of the channel bed suggests bed material is coarse gravel and cobble, and bed topography is uniform and void of vegetation. The channel bed was delineated by bank stations within each cross section, and a conservative Manning's 'n' roughness values of 0.05 were set for the channel bed. The relative roughness height of bed roughness compared to the flow depth at the design discharge, a lower roughness value could be applicable. However, the higher range of roughness was used to account for variability in sediment transport and debris associated with higher discharge events.

3.2.2.2 Bank Areas

Upstream of the bridge to the upper extents of the study reach, both banks are fairly similar with unmanaged riparian grass and medium to large diameter trees. A short distance upstream of the bridge, the left overbank (looking downstream) areas are inundated during larger (+/- 10 year) storm events, with bank slopes of approximately 3:1. The upstream, right bank slopes were observed to be approximately 2:1, with the lack of overbank/floodplain inundation potential.

Downstream of the bridge to the lowest extents of the study reach, both banks are fairly similar with unmanaged riparian grasses and medium to large diameter trees. Bank slopes downstream of the bridge were observed to be approximately 2:1, with the lack of overbank/floodplain inundation potential. This consistent and steep channel geometry, along with the linear planform suggest anthropogenic channel conditions downstream of the bridge. Additionally, variable amounts of underbrush was observed on both channel bank areas upstream of the bridge to the upper extents of the survey reach.

The overbank areas along the study reach exhibit denser grasses and trees with additional underbrush growth compared to the channel banks. There is higher density of large woody vegetation upstream of the bridge.

For both channel banks, a Manning’s ‘n’ roughness value of 0.075 was used to represent the increase in roughness of the vegetation. For the overbank areas and areas of the bank where the vegetation changes, a Manning’s ‘n’ roughness value of 0.12 was used to represent the high density of vegetation ‘grain’ roughness. For the proposed realignment of the bike path, a Manning's 'n' roughness value of 0.013 was used to represent the decrease in roughness of the concrete path surface.

3.2.2.3 Proposed Condition Vegetation

As part of the bridge replacement project is the removal of existing trees to allow for the construction of and the ultimate footprint of the new structure. Mitigation and revegetation will follow the conditions described in the CEQA permitting documentation.

3.2.3 Ineffective Flow Areas

Ineffective flow within the channel was not considered due to the prismatic and uniform geometry of the channel. However, minor ineffective flow areas were estimated at the vertical bridge abutments. Abutments are vertical and set adjacent to the channel banks. Minor protrusions due to the vertical nature are considered to generate minor ineffective flow areas at both abutments. Ineffective flow areas are incorporated into the model, but are only expected to have a minor impact to hydraulic conditions.

3.2.4 Expansion and Contraction Coefficients

Typical expansion and contraction coefficients of 0.3 and 0.1, respectively, were used to represent variations in channel uniformity. For the proposed structure expansion and contraction coefficients at the structure were maintained to 0.1 and 0.3 to account for the maintaining existing channel geometry and that the proposed abutments are outside of the primary flow area, and removal of the vegetation immediately adjacent to the structure.

3.3 Boundary Conditions

The downstream boundary condition assumes normal flow depth with a channel gradient of 0.009 ft/ft. The downstream boundary condition was determined from the analysis of the collected topographic survey and multiple site visits. More specifically, the slope was developed from the water surface slope between the two cross sections located at the lower most extents of the study reach. The state of flow is subcritical within the study reach, and as such an upstream boundary condition was not needed.

3.4 Flow Data

A steady flow condition was utilized for this analysis. No temporal variation in discharge was required to adequately determine hydraulic conditions.

The estimated 200-year discharge of 2,400 cfs at the project site was used for both the existing and proposed condition design discharge. The 100-year (2,200 cfs) and 50-year (2,000 cfs) discharges were also modeled for both conditions to verify the criteria of the Highway Design Manual were satisfied.

3.5 MODEL VALIDATION

Model validation is adjusting various inputs and verifying the models adjusts as it would be expected to. For example, decreasing Manning's *n* values along the plan reach one would expect the water surface level to decrease. For this hydraulic model the roughness, contraction and expansion, and pier widths were varied to see how the model responded. As Manning's *n* values were increased in Little Chico Creek the water surface elevation rose in an expected manner. Decreases in pier widths resulted in lowering of the water surface elevation. Additionally, as the contraction and expansion coefficients were increased, the water surface elevation increased as expected.

3.6 RESULTS

This hydraulic analysis was performed to identify and document the existing hydraulic condition as a baseline reference for which to compare the proposed design condition. This section discusses the more salient findings for both conditions; comprehensive model results and output are provided in Appendix E and F for the existing and proposed conditions respectively.

3.6.1 EXISTING CONDITION INPUTS

The existing condition is a natural, unmodified, creek channel. At the location of the proposed bridge, the existing condition model follows the cross-sections generated by the topographic survey and field visits. Below is a summary of key input parameters.

Table 4: Summary of Select Bridge Inputs: Existing Condition

Typical Channel Roughness	Channel- 0.05, Bank L/R- 0.075/0.075, Overbank L/R: 0.12
Expansion/Contraction Coefficients	0.3 / 0.1;
Selected Bridge Modeling Methods	Energy and Momentum (low flow), Energy (high flow)

3.6.2 EXISTING CONDITION RESULTS

Review of modeled profile suggests a clear M1 (mild sloped, subcritical backwater) flow profile. Figure 10 provides the existing condition flow profile.

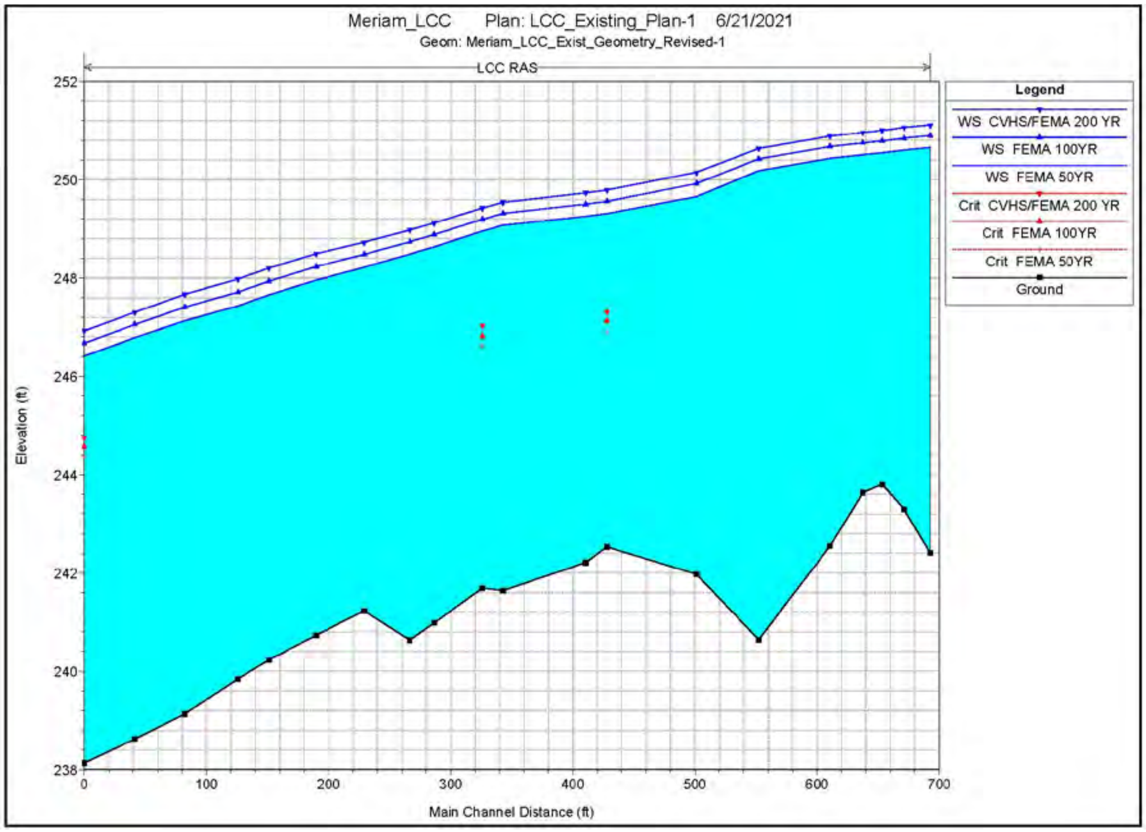

Figure 10: Study Reach Profile; Existing Conditions, 2400cfs, taken from HEC-RAS.

The flattening of the water surface and energy grade line slopes illustrates the influence of the relatively mild slope provided at the downstream boundary condition.

As discussed in the previous hydrology section in this report, discharge is limited in this channel by upstream weirs. Discharges of the CVHS study (200 year), CVFPB O&M Manual (100- year), FEMA 100-year and FEMA 50-year flows were modeled. A summary of the results are provided below in Table 5.

Table 5: Summary of Key Results; Existing Condition-

Flood Event					
Name	Recurrence Interval (year)	Discharge (cfs)	Existing Water Surface Elevation [1] (WSE)	Clearance to Bottom Soffit (ft)	Cross-Section Average Velocity[2] (ft/s)
Project Design Flood Discharge	200	2,400	249.73	N/A	4.30
FEMA/CVHS Estimated Discharge	100	2,200	249.50	N/A	4.17
CVFPB O&M Manual	100	2,200	249.50	N/A	4.17
FEMA Estimated Discharge	50	2,000	249.25	N/A	4.05

1. At approximate upstream proposed bridge section
2. At approximate downstream proposed bridge section.

3.6.3 Proposed Condition Inputs

Modeling of the Proposed Condition is based on the understood proposed bridge configurations. As discussed previously, the proposed structure has a similar alignment as the existing roadway alignment with an offset (relative to the roadway centerline) to the upstream side. The bridge will be skewed to efficiently match the existing channel configuration. Proposed abutments and piers are set away from the channel centerline as not to encumber freeboard conditions. A summary of the bridge components are summarized below in Table 6.

Table 6: Summary of Select Bridge Inputs; Proposed Condition

Recommended Bridge Candidate Configuration	
Bridge Description:	3-span reinforced concrete slab, with Cambered concrete deck, supported on multiple pile bents and reinforced concrete wall abutments
Structure Skew:	30°
Structure Length:	100 feet
Left Abutment:	Vertical
Right Abutment:	Vertical
Structure Width:	56 feet, 29.5’ upstream of roadway alignment, 26.5' downstream of roadway alignment
Deck Thickness	20” reinforced concrete deck
Soffit Elevations:	253.13 (upstream lowest bridge member) 253.19 (downstream lowest bridge member)
Wingwalls:	Wingwalls are proposed.
Rock Slope Protection:	Yes, assumed d_{50} = 450 mm
Channel Bed:	Assumed to remain as existing condition
Expansion/Contraction Coefficients:	0.3 / 0.1
Selected Bridge Modeling Methods	Energy and Momentum (low flow), , Energy (high flow)
Other:	Design configuration holds existing roadway alignment

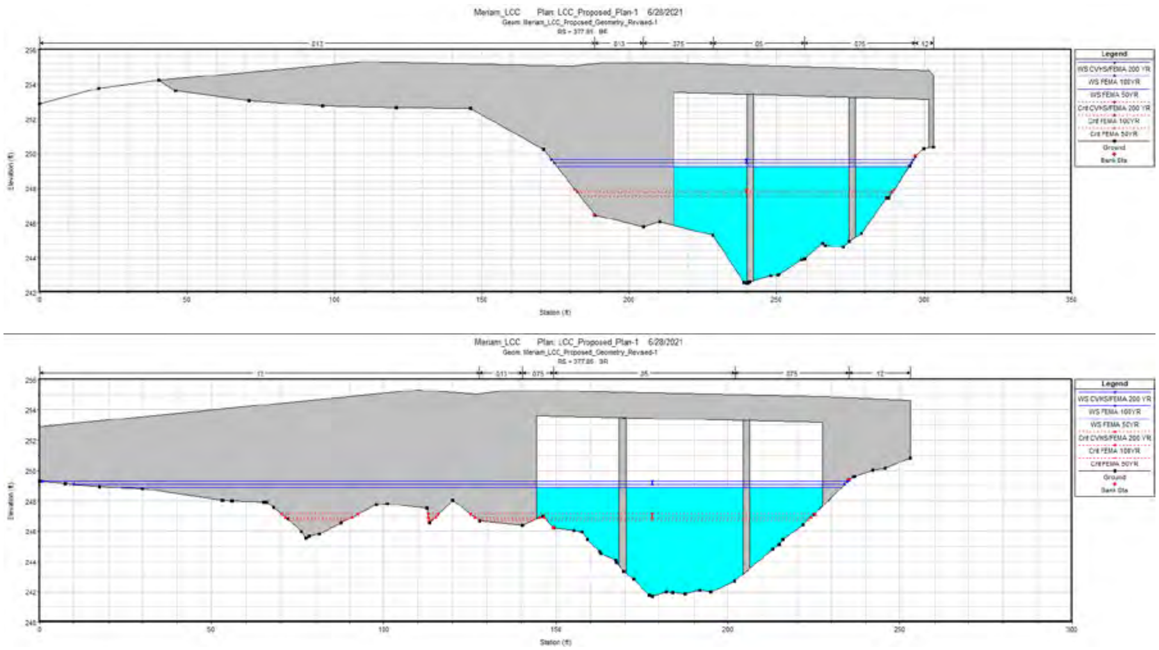

Figure 11: Bridge Cross-Sections- Proposed condition, taken from HEC-RAS

3.6.4 Proposed Condition Results

The proposed condition profile, as shown in Figure 12, continues the expected M1 flow profile with the downstream boundary normal flow boundary condition influencing the backwater hydraulics of the study reach.

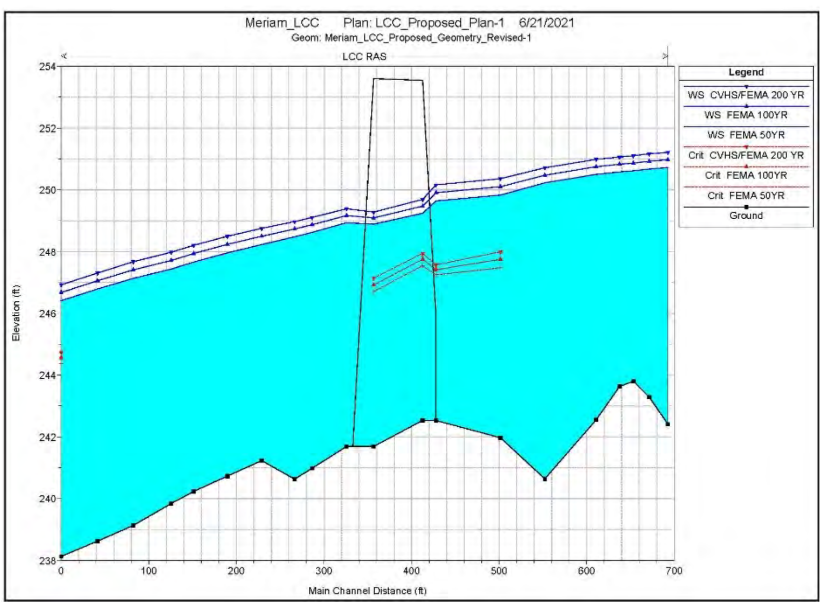

Figure 12: Study Reach Profile; Proposed Conditions, taken from HEC-RAS.

As in the existing conditions model runs, the Design Flood Discharge, CVFPB O&M Manual, and the FEMA flow events were modeled. A summary of results are provided below in Table 7.

Table 7: Summary of Key Results; Proposed Condition

Flood Event								
Name	Recurrence Interval (year)	Discharge (cfs)	Water Surface Elevation [1] (WSE)		Clearance to Bottom Soffit (ft)		Cross-Section Average Velocity[2] (ft/s)	
Project Design Flood Discharge	200	2,400	249.69[1]	249.28[2]	3.44[1]	3.91[2]	6.62[1]	6.01[2]
FEMA Estimated Discharge	100	2,200	249.47[1]	249.09[2]	3.66[1]	4.10[2]	6.36[1]	5.72[2]
CVFPB O&M Manual	100	2,200	249.47[1]	249.09[2]	3.66[1]	4.10[2]	6.36[1]	5.72[2]
FEMA Estimated Discharge	50	2,000	249.24[1]	248.88[2]	3.89[1]	4.31[2]	6.10[1]	5.42[2]

1. At upstream bridge section
2. At downstream bridge section.
See summary table in Section 5 for comparison results

4 SCOUR ANALYSIS

For this project, scour analysis was performed to provide the structural (bridge) designers insight to the potential for scour and hydraulic conditions at this structure.

Scour can result from a variety of influences, from local hydraulic conditions to landscape-scale changes. Changes to the channel geometry can include aggradation (filling), degradation (incision), or longitudinal widening or narrowing. These changes can be driven by changes to either temporal or spatial changes to streamflow, sediment, or by local geometric changes. In any case, evaluation of scour potential is a critical component to inform project designers of estimated minimum foundation elevations or design of scour countermeasures.

4.1 METHODOLOGY

In addition to evaluating channel hydraulics, HEC-RAS software was utilized to estimate channel bed scour of the proposed bridge in the proposed channel condition. Using the calculated channel hydraulic conditions, HEC-RAS uses a variety of empirical and theoretical equations to estimate the depth of scour. Scour analysis of the design condition was calculated following the applicable criteria and standard methodology presented in HEC-18 and related HEC-RAS manuals.

The evaluation of scour is compartmentalized into three (Local Scour) categories of Contraction, Pier, and Abutment Scour. Calculations are made independently for each evaluation type, and then combined to provide a total scour depth and corresponding elevation.

The key parameters required to complete scour calculations include average (50% grain size fraction) channel bed sediment size, water temperature, type of abutment (vertical, spill through) and confirmation of model-calculated geometric parameters, such as the toe station of the roadway embankment and abutment. Sediment size is utilized for estimation of incipient motion in the Laursen 1963 contraction scour equation. Water temperature is used in Laursen's 1960 live-bed scour equation; a temperature of 55˚F was estimated for this project. High flow events expected to provide hydraulic conditions best suited to induce significant bed scour would be during winter and spring rainfall hydrologic events which exhibit this approximate temperature. Pier scour was estimated using Colorado State University (CSU) Equation.

4.2 RESULTS

Scour calculations were based on the base flood discharge (Q_{200}) and 500-year (Q_{500}) storm events. Scour calculations were completed on a higher than base flood discharge to account for uncertainty and risk associated with designing scour related issues. The scour analysis assumed a d_{50} of the channel banks and inside edge of the bents to 450 mm (~18 inch) diameter RSP, which is the planned design for the protection of the channel and bents. Scour calculations assume unconsolidated (sand) material is present; which can overestimate the depth of scour. However, these values provide a conservative perspective of the potential scour for the given conditions.

Figure 13 provides a cross section view and Table 8 provides a summary of scour related findings for Bents 2 and 3 for the base flood and "check" discharge. Refer to Appendix F for additional scour results.

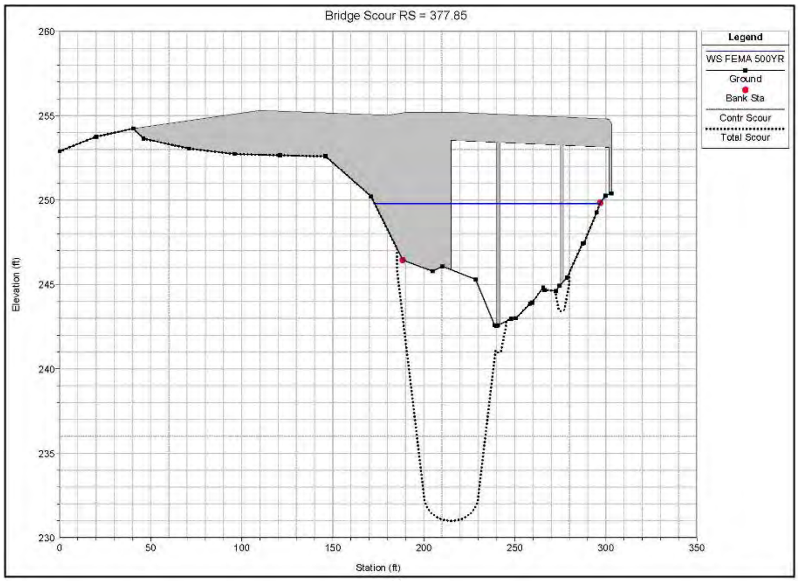

Figure 13: Cross Section of Estimate Scour at Bridge, 2,500 cfs, Proposed Condition.

Table 8: Summary of Scour Related Output for Bridge,

		Bent #2 (Pier #1)				Bent #3 (Pier #2)			
Discharge Event Name	**Discharge (cfs)**	**Velocity[1] (ft/s)**	**Bed Shear Stress[1,] (lb/ft^2)**	**Flow depth[1] (ft)**	**Total Scour Depth[2] (ft)**	**Velocity[1] (ft/s)**	**Bed Shear Stress[1] (lb/ft^2)**	**Flow depth[1] (ft)**	**Total Scour Depth[2] (ft)**
Base Flood (CVHS-200yr)	2,400	6.62	2.71	4.69	14.59	6.62	2.71	4.69	1.62
Check (Q_{500})	2,500	6.74	2.81	4.977	14.88	6.74	2.81	4.77	1.63

1. The higher value from either cross-section, channel portion output [u/s bridge section], or scour calculation output [d/s bridge section].
2. As reported from scour calculation output (Abutment + Contraction + Pier).

Note:
Estimates of potential scour presented here are developed using empirical and theoretical relationships presented in HEC-18 that assume non-cohesive or fully-erodible substrate. These assumptions may be inappropriate if the geotechnical investigation reports different substrate or channel bed maternal.

The model results show a need for scour countermeasures at both bens. However, the existing site conditions do not substantiate the model results. The creek bed is lined with gravel and cobble with very dense / hard silty clay / clayey silt underneath. The proposed structure will utilize RSP on the northern slope, the proposed concrete bike path at the southern abutment, maintaining the current channel bed, and will be designed using the calculated scour analysis. Taking into consideration the existing conditions and the proposed improvements, the potential for breaching the RSP and concrete bike path and producing scour as the model predicts is highly unlikely. Additionally, the existing bridge at Bruce Road (upstream of Notre Dame Blvd) crossing Little Chico Creek has experienced minimal (less than a couple inches) scour at both the piers and abutments since its construction in 1969.

5 SUMMARY AND CONCLUSIONS

The HEC-RAS model runs prepared for the reach of Little Chico Creek estimates that there is a negligible rise of the proposed design floodplain compared to the existing condition. Since the existing site conditions are a natural creek channel compared to the proposed bridge structure, comparison of the water surface elevations utilized common cross sections slightly upstream of the proposed structure. The cross section common between the existing and proposed conditions, and closest to the proposed structure, is Station 4+28.29. At Station 4+28.29 the existing conditions model WSE is 249.79', and the WSE of the proposed is 250.16, an increase of 0.37'. This increase is attributed to installation of the proposed structure and the proposed structure slightly decreases the channel width. As shown by the continuity equation where the flow is equal to the product of channel velocity and channel area (Q=v*A). Since the flow remains constant as the channel velocity increases and the channel width decreases, the water surface (i.e. height) must increase.

See Table 9 below for a summary comparison of the existing and proposed bridge structures.

Table 9: Summary of Comparison Results

		Flood Event			
	Name	Project Design Flood Discharge	FEMA Estimated Discharge	CVFPB O&M Manual	FEMA Estimated Discharge
	Return Interval (year)	200	100	100	50
Structure	**Discharge (cfs)**	2,400	2,200	2,200	2,000
Existing	**Water Surface Elevation[1] (WSE)**	249.73	249.50	249.50	249.25
	Clearance to Bottom Soffit (ft)	N/A	N/A	N/A	N/A
	Cross-Section Average Velocity[2] (ft/s)	4.30	4.17	4.17	4.05
Proposed	**Water Surface Elevation[1] (WSE)**	249.69	249.47	249.47	249.24
	Clearance to Bottom Soffit (ft)	3.44	3.66	3.66	3.89
	Cross-Section Average Velocity[2] (ft/s)	6.01	5.72	5.72	5.42

1. At upstream bridge section
2. At downstream bridge section.

Additionally, the proposed bridge structure is estimated to meet the freeboard requirement of three (3) feet above the design floodplain at an estimated 200-year discharge. This satisfies the CVFPB, City of Chico, and CalTrans freeboard requirements for bridge structures.

Hydraulic modeling of the subcritical and mild sloped reach upstream of Little Chico Creek indicated a sensitivity to both channel roughness parameters and proposed slopes under the new structure. The modeling approach included distinction between roughness values of the channel and channel banks. Roughness values were spatially varied to best approximate the change in channel width through the study reach. The downstream boundary was developed through topographic survey and multiple visual inspections of the site.

Scour analysis was also modeled using the base flood discharge in Little Chico Creek combined with the proposed channel bank grading and RSP sizing under the proposed bridge structure. The scour modeling indicated sensitivity to the d_{50} of the RSP placed on the channel banks and around the bents. Due to the nature of the streambed (coarse gravel and cobble) and proposed RSP on the channel banks it is reasonable to estimate there will be minimal long-term scour around the bents and abutments.

5.1 FLOOD RISK

The modeled results estimate a minor increase in water surface elevations in the proposed condition; therefore, the flood risk is presumed to be similar or slightly higher with the proposed condition. Additionally, the offset of abutments and bents as well as maintained freeboard condition, passing of potential drift is expected to be suitable for the project site.

6 REFERENCES

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APPENDIX A

HYDRAULIC MODEL EXHIBIT

PROPOSED BRIDGE CHANNEL CROSS SECTIONS

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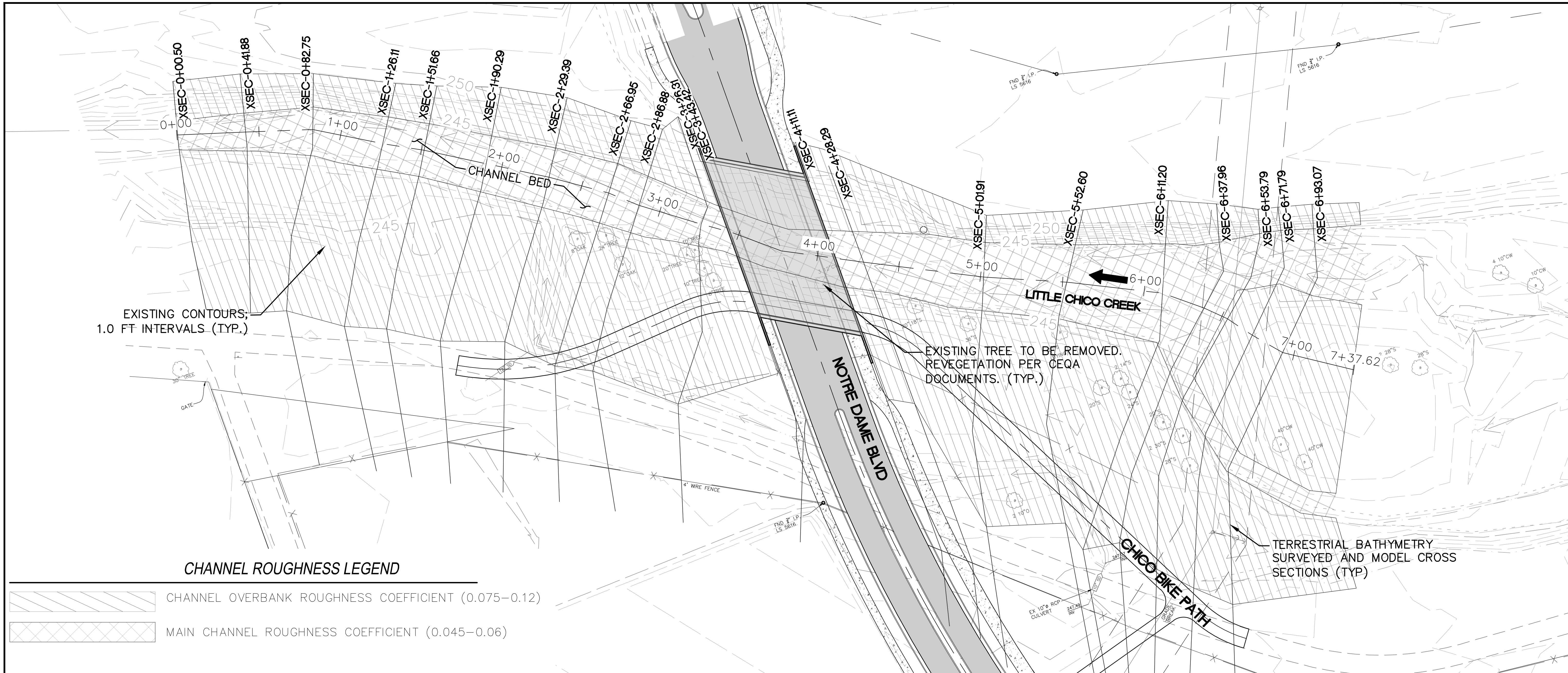


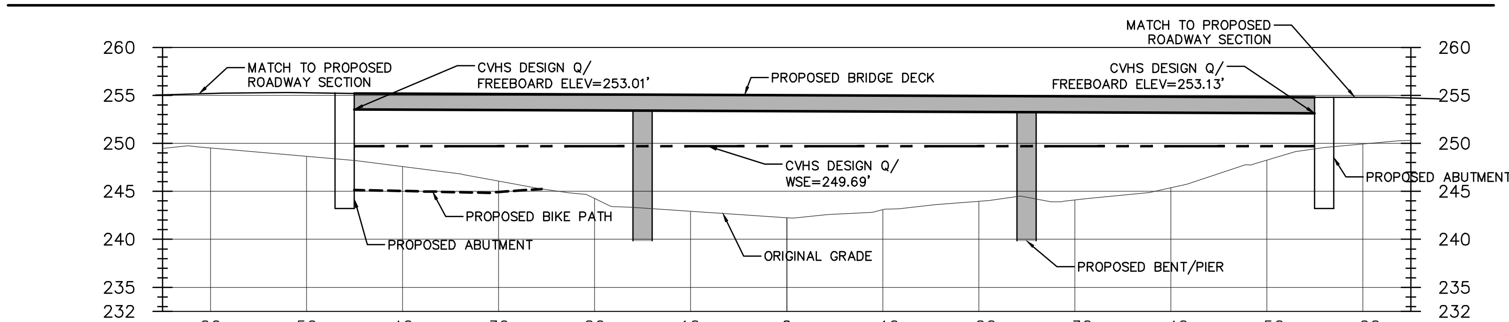
CHANNEL ROUGHNESS LEGEND

CHANNEL OVERBANK ROUGHNESS COEFFICIENT (0.075–0.12)

MAIN CHANNEL ROUGHNESS COEFFICIENT (0.045–0.06)

NOTE: ELEVATIONS ARE BASED ON THE NGVD29 DATUM

UPSTREAM BRIDGE CROSS SECTION; 4+11.11 (1"=20,1:1,H;V)



LICENSED PROFESSIONAL ENGINEER
KIRK JOHN KOESTER
No. 88513
CIVIL
STATE OF CALIFORNIA

Designed: KJK	NORTHSTAR ... Designing Solutions		MP NORTHFORK, LLC	HEC-RAS MODEL EXHIBIT
Drawn By: KJK	111 MISSION RANCH BLVD. SUITE 100, CHICO, CA 95926		1262 HUMBOLDT ROAD	MERIAM PARK - NOTRE DAME BRIDGE
Approved:	PHONE: (530) 893-1600 www.northstareng.com		CHICO, CALIFORNIA	APN Number; Job Number 15-235; Scale 1"= 60' Horz. N/A Vert.; Sheet 1 Of 1
Date: JUNE 2021				

APPENDIX B

PROJECT CONSTRUCTION GENERAL AND FOUNDATION PLANS

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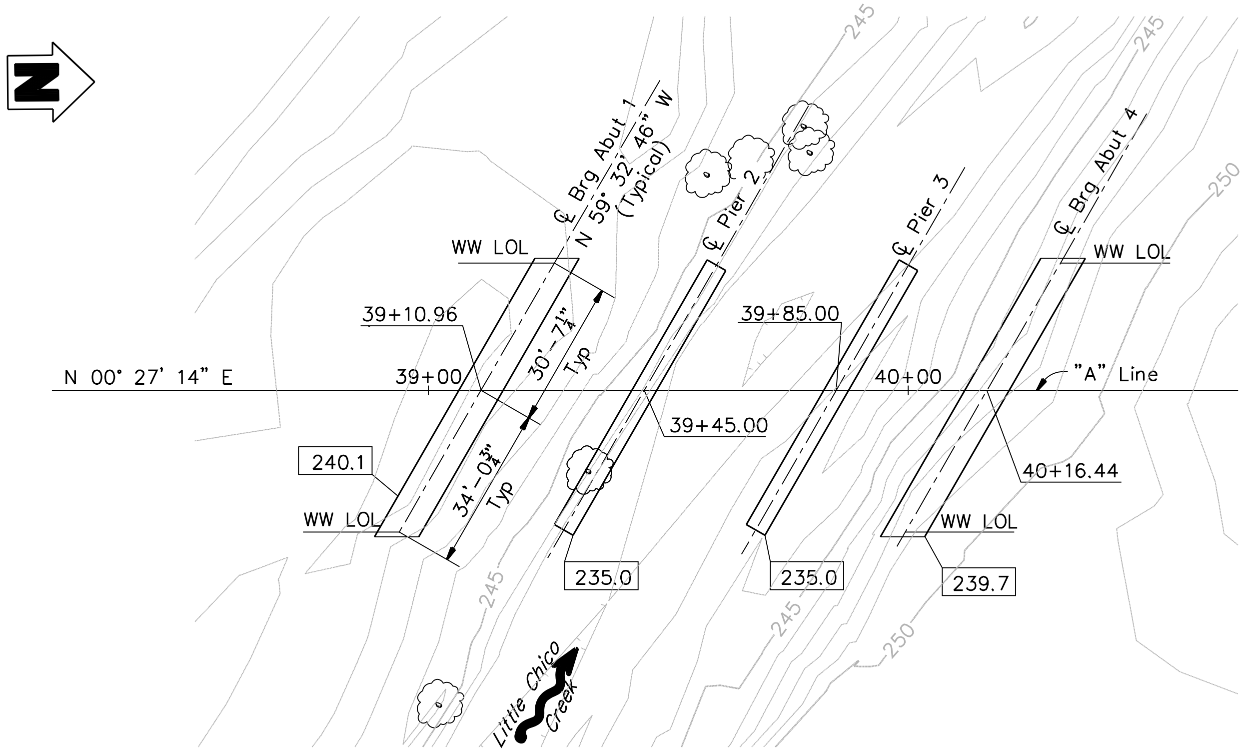


FOUNDATION PLAN

1' = 20'-0"

Notes:

XXX.XX — Indicates bottom of footing elevation

Contour data based on topographic maps prepared by Northstar Dated 12/04/20.

Vertical Datum: Based on City of Chico Benchmark #5C at the top Northwest wingwall of the bridge at Humboldt Road and Dead Horse Slough. Elev = 23.601 (Monument destroyed in bridge widening project).

Horizontal Datum: Control based on a local Meridian Park coordinate system.

Basis of Bearing: The centerline of Notre Dame Blvd. at the northeastern corner of parcel 4 as shown per Map Book 145, at pages 79-83. The bearing of said line being S 00° 27' 14" W.

HYDROLOGIC SUMMARY

DRAINAGE AREA 133 SQ MI

	DESIGN FLOOD	BASE FLOOD	CVHS 200 YEAR BASE FLOOD	RECORD FLOOD
FREQUENCY, YEARS	50	100	200	N/A
DISCHARGE CUBIC FT/SEC.	2,000	2,200	2,400	N/A
WATER SURFACE ELEVATION AT BRIDGE	248.54	248.78	249.01	N/A

LOCATION HYDRAULIC STUDY PREPARED BY NORTHSTAR.
FLOOD PLAIN DATA ARE BASED UPON INFORMATION AVAILABLE WHEN THE PLANS WERE PREPARED AND ARE SHOWN TO MEET FEDERAL REQUIREMENTS. THE ACCURACY OF SAID INFORMATION IS NOT WARRANTED. INTERESTED OR AFFECTED PARTIES SHOULD MAKE THEIR OWN INVESTIGATION.

SCOUR DATA TABLE

Support	Long Term (Degradation and Contraction) Scour Depth (ft)	Short Term (Local) Scour Depth (ft)
Pier 2	0	1.6
Pier 3	0	1.6

PILE DATA – STEEL HP PILES

Location	Pile Type	Nominal Resistance Compression	Nominal Resistance Tension	Design Tip Elev	Specified Tip Elev	Req'd Nominal Driving Resistance
Abut 1	HP10x57	280 Kips	140 Kips	X.X(1)	--	280 Kips
Pier 2	HP10x57	280 Kips	140 Kips	X.X(1)	--	280 Kips
Pier 3	HP10x57	280 Kips	140 Kips	X.X(1)	--	280 Kips
Abut 4	HP10x57	280 Kips	140 Kips	X.X(1)	--	280 Kips

Design tip elevation is controlled by the following conditions:
(1) Axial Compression.
(2) The specified tip shall not be raised.

GENERAL NOTES
LOAD RESISTANCE FACTOR DESIGN

DESIGN: AASHTO LRFD Bridge Design Specifications, 6th Edition W/ California Amendments.

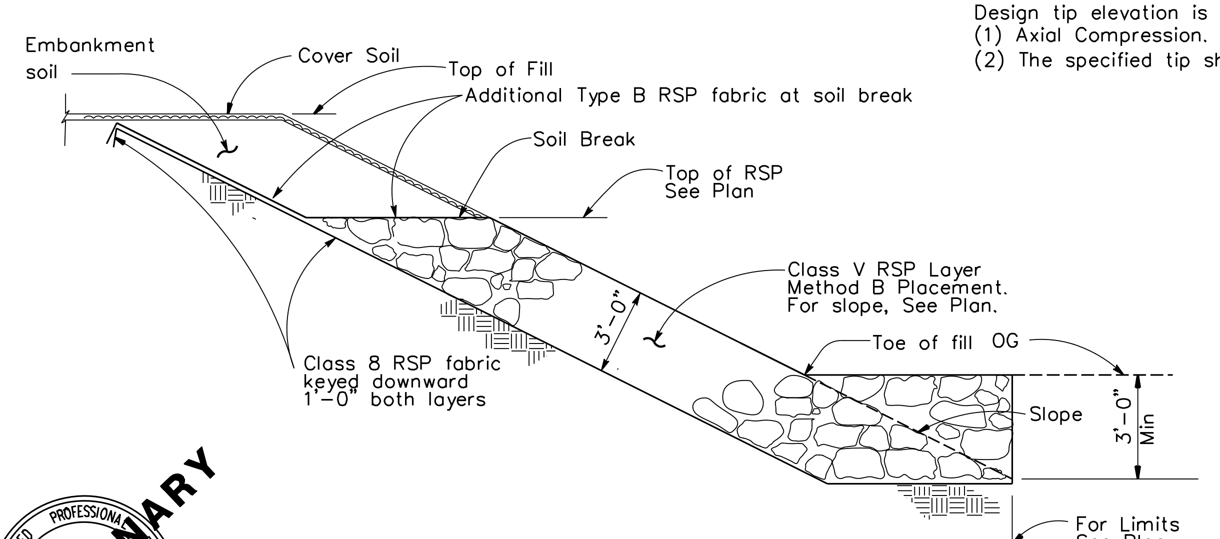
SEISMIC DESIGN: Caltrans Seismic Design Criteria (SDC), Version 1.7 dated April 2013

DEAD LOAD: Includes 35 psf for future wearing surface.

LIVE LOADING: HL93 W/ "Low-Boy" and Permit Design Vehicle

SEISMIC LOADING: Soil Profile: Type D
Moment Magnitude: X.X
Peak Ground Acceleration = 0.XXg

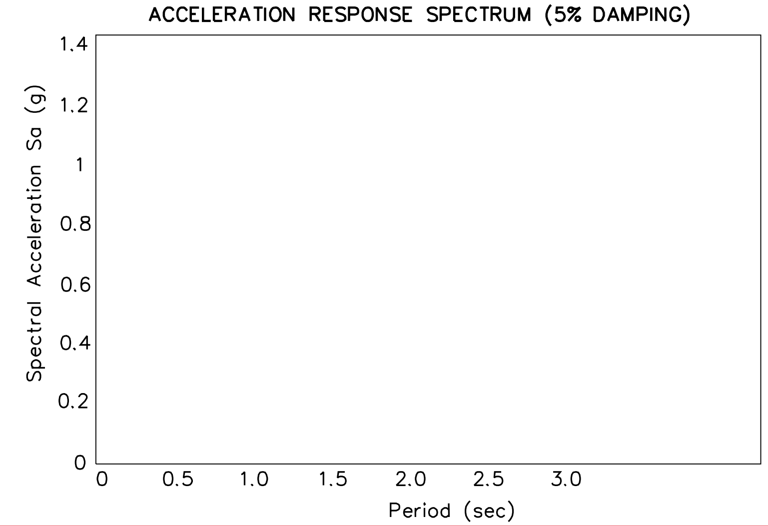
REINFORCED CONCRETE: fy = 60 ksi
f'c = 4.0 ksi (unless noted)



ROCK SLOPE PROTECTION

No Scale

ACCELERATION RESPONSE SPECTRUM (5% DAMPING)



PRELIMINARY

REGISTERED PROFESSIONAL ENGINEER Robert Morrison Jr. No. S5577 Exp. 3-31-23 STRUCTURAL STATE OF CALIFORNIA

Note: Elevations on the following bridge plans reference the City of Chico's datum. To adjust the elevations to the NGVD 29 datum, add 0.53'.

Designed: R.Morrison, Jr	Approved:	Revision	Date	By
Drawn By: J.Gallino				
Checked:	Date: 4/26/21			

CITY OF CHICO
PUBLIC WORKS DEPARTMENT

MORRISON STRUCTURES
1890 PARK MARINA DR. SUITE 104
REDDING, CA 96001
PHONE: (530) 246-8628 MORRISONSTRUCTURES.COM

LITTLE CHICO CREEK BRIDGE ON NOTRE DAME BLVD.
FOUNDATION PLAN

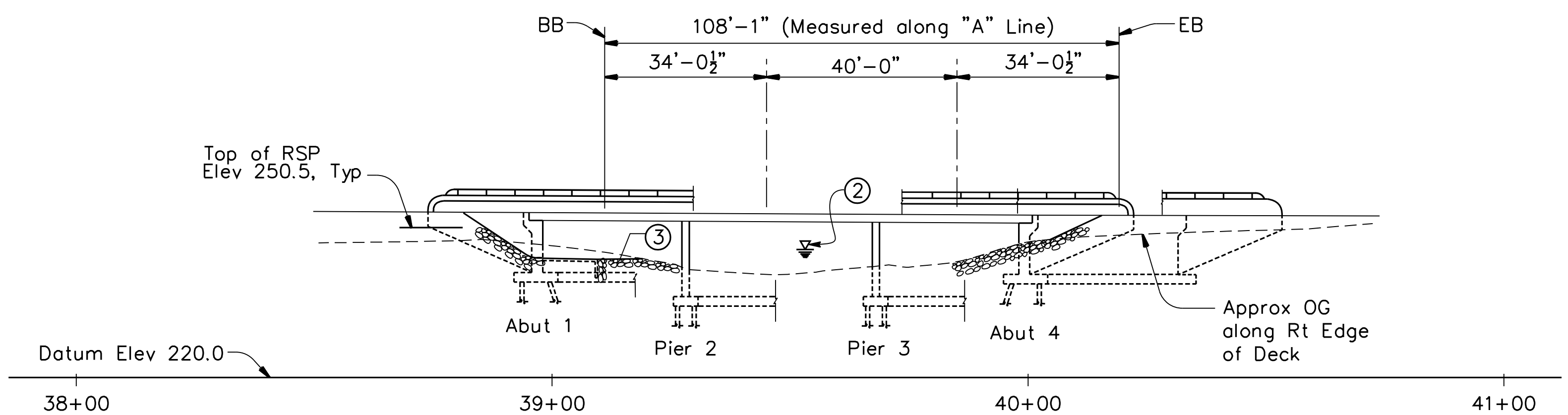
Project Number:
Drawing Number
Scale: As Shown
Sheet 2 of _

Plan Status: ☐ Conceptual, ☐ Approved Final, ☐ Record

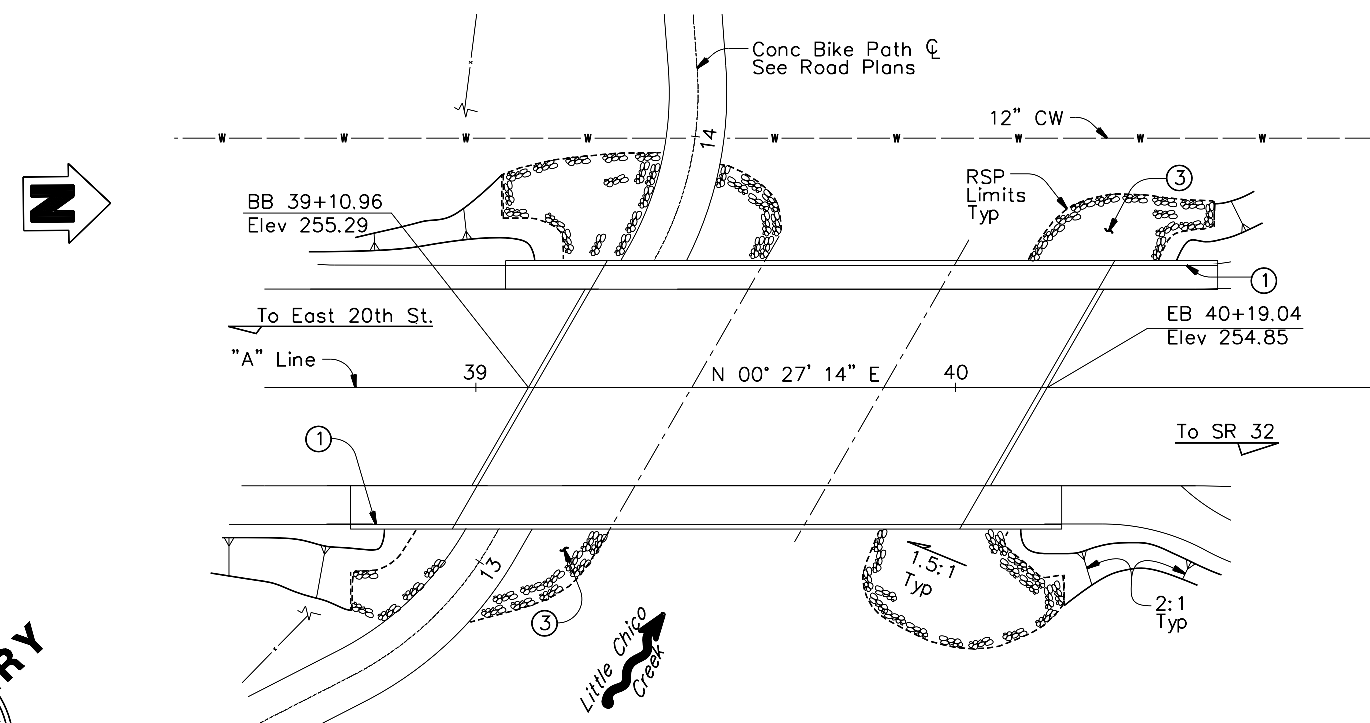
GEOTECHNICAL PROFESSIONAL
APPROVAL DATE
Drawing Number
Sheet _ of _

DATE PLOTTED => $DATE
TIME PLOTTED => $TIME

PROFILE GRADE
No Scale



ELEVATION
1' = 20'-0"

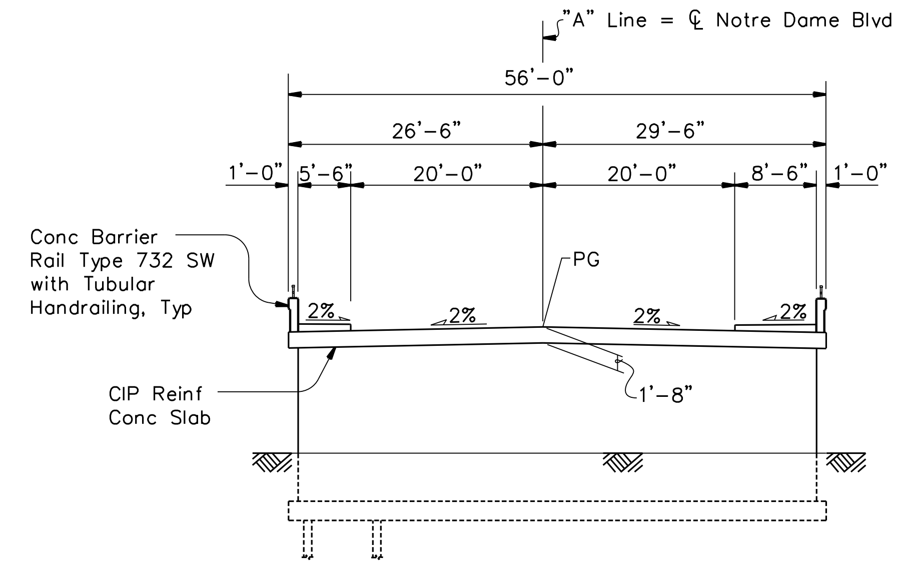


PLAN
1' = 20'-0"

Notes:
1. Paint "LITTLE CHICO CREEK BRIDGE" and "BR NO XXX-XXXX"
2. See Hydrologic Summary on "XXX" Sheet.
3. RSP Method B placement. for details see Rock Slope Protection Detail on "XXX" Sheet.

Standard Plan Sheet No.
Detail No.

For General Notes, see "XXX" sheet.



TYPICAL SECTION
1' = 10'-0"

Index To Standard Plans

A3A-C	Abbreviations (3 Sheets)
A10A-H	Lines and Symbols (8 Sheets)
A62C	Limits of Payment for Excavation and Backfill - Bridge
B0-1	Bridge Details
B0-3	Bridge Details
B6-21	Joint Seals (Maximum Movement Rating = 2")
B11-51	Tubular Handrailing
B11-58	Concrete Barrier Type 732SW No. 1
B11-59	Concrete Barrier Type 732SW No. 2

REGISTERED PROFESSIONAL ENGINEER
Robert Morrison Jr.
No. ...7
Exp. ...31-23
STRUCTURAL
STATE OF CALIFORNIA
PRELIMINARY

Plan Status: ☐ Conceptual, ☐ Approved Final, ☐ Record

Designed: R.Morrison, Jr	Approved:	Revision	Date	By
Drawn By: J.Gallino				
Checked:	Date: 4/25/21			

CITY OF CHICO
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CITY of CHICO INC 1872

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LITTLE CHICO CREEK BRIDGE ON NOTRE DAME BLVD.
GENERAL PLAN

Project Number:
Drawing Number
Scale: As Shown
Sheet 1 of XX

Sheet __ of __
Title:
Drawing Number

APPENDIX C

SITE PHOTOGRAPHS

Site photographs were taken November 2020 and February 2021

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Photos generally progress from the upstream end of the study reach and progress downstream (down-station). Refer to Plan View Exhibit for Channel Stationing. “Right” and “Left” bank orientation based on viewing downstream. “u/s” = upstream; “d/s” = downstream

Sta 6+93, view d/s
Note: When the site photographs were collected there was a recent vegetation fire in the area, this portion of the creek typically is moderately dense with vegetation.

Stat 6+11; view d/s

Sta 4+11; view d/s

Stat 4+11; view southerly

Sta 4+11; view northerly

Sta 2+29; view d/s

Sta 0+41; right bank, view d/s

APPENDIX D

CVHS AND FEMA DATA

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Central Valley Hydrology Study:

Butte Creek watershed hydrologic analysis

January 10, 2014

U.S. Army Corps of Engineers
Sacramento District

CENTRAL VALLEY HYDROLOGY STUDY (CVHS)
COMPLETION OF AGENCY TECHNICAL REVIEW (ATR)

BUTTE CREEK WATERSHED

HYDROLOGIC ANALYSIS

ATR has been completed for the above-specified rainfall-runoff analysis. The ATR was conducted as defined in the project's Review Plan to comply with the requirements of EC 1165-2-209. During the ATR, compliance with established policy principles and procedures, utilizing justified and valid assumptions, was verified. This included review of: assumptions, methods, procedures, and material used in analyses, alternatives evaluated, the appropriateness of data used and level obtained, and reasonableness of the results. All comments resulting from the ATR have been resolved and the comments have been closed in DrChecks[sm].

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Jay Pak, Ph.D, P.E.
ATR Reviewer
CEIWR-HEC-HH

Date

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Judy Soutiere, CFM
Project Manager
CESPK-PD-WF

Date

Document revisions

Date (1)	Summary of action or revision (2)
December 30, 2011	Draft report submitted to Corps.
July 24, 2012	Draft report revised according to district quality control (DQC) comments provided by the Corps.
January 25, 2013	Draft report revised after completion of modified rainfall-runoff analysis. This modified analysis, completed based on technical direction provided by the Corps, included: • Use of alternative depth-area reduction factors. • Use of 4-day, center peaked design storms. • Use of 1-hr design storm rainfall increments. • Calibration of constant loss rates to achieve design storm flows consistent with at-site flow-frequency curves provided by the Corps.
March 12, 2013	Draft report modified in response to agency technical review (ATR) comments, which are on file with the Corps.
January 10, 2014	Updated the Little Chico Creek inflow-discharge diversion relationship (Table 4, Figure 3). Calibrated Little Chico Creek subbasin parameters based on observed diversion flow. Also added a Soil Map (Figure 2).

Contents

Tables

Figures

Executive summary

Situation

The California Department of Water Resources (DWR) and the US Army Corps of Engineers, Sacramento District (Corps) are involved in a collaborative effort, the Central Valley Hydrology Study (CVHS), to develop flood flow-frequency relationships at various analysis points in California's Central Valley. In most cases, these flow-frequency relationships can be defined through analyses that use historical streamflow data. However, for some streams, particularly when historical streamflow data are poor or unavailable, rainfall-runoff modeling can be used to characterize flood flow-frequency. These locations and their respective watersheds are listed in Table 1 of *Central Valley Hydrology Study: Ungaged watershed analysis procedures*, dated November 14, 2011. The *Ungaged watershed analysis procedures* document also outlines the analysis approach used to develop frequency curves at these locations.

This report, which is 1 of 6 similar reports, describes our analysis of the Butte Creek watershed (ungaged watershed 2 of Table 1 from the *Ungaged watershed analysis procedures*).

Tasks

We developed flow-frequency curves for the Butte Creek watershed at the designated CVHS analysis points. (For CVHS, an analysis point is a selected, agreed upon location where flow-frequency curves and flood volumes are required.)

Actions

To develop the required flow-frequency curves, we followed the steps described in the *Ungaged watershed analysis procedures*. Specifically, we:

1. Gathered information about the watershed, including past studies.
2. Delineated watershed subbasins using US Geological Survey (USGS) 10-m digital elevation model (DEM) topographic data and HEC-GeoHMS tools.
3. Developed an HEC-HMS basin model using the delineated watershed from step 2.
4. Selected rainfall-runoff modeling methods, parameter values, and an appropriate model time step.
5. Collected available historical precipitation and flow data for model calibration.
6. Corrected or removed precipitation data where errors were identified.
7. Calibrated model parameters to observed flows at gages within the watershed.
8. Identified 1 storm centering for calculating the appropriate depth-area reduction factors for each analysis point. Despite the presence of 3 analysis points in the Butte Creek watershed, we used only 1 storm centering because all 3 analysis points have similar contributing areas.

9. Extracted precipitation-frequency estimates for each subbasin from *NOAA Atlas 14 Volume 6, Version 2: Precipitation-frequency atlas of the United States, California* (USDC-NOAA 2011).
10. Lumped subbasins with similar precipitation-frequency estimates into 7 zones and computed area-weighted average precipitation for each zone.
11. Applied depth-area reduction factors from Hydrometeorological Report No. 59 (HMR 59 [NWS 1999]) to the zonal precipitation-frequency estimates computed in step 10.
12. Used the HEC-HMS frequency storm meteorologic model as a preprocessor to create all required design storm hyetographs. Specifications and inputs to the design storm preprocessing model included the reduced zonal precipitation-frequency estimates from step 11, a maximum rainfall increment centering of 50 percent, a 4-day total storm duration, and a center-peaked design storm hyetograph.
13. Configured the calibrated HEC-HMS basin model of the Butte Creek watershed to use the design storm hyetographs from step 12 as input.
14. Simulated all required design storms and assigned the annual exceedence probabilities (AEPs) of the design precipitation to the resulting peak flows.
15. Compared the computed flow-frequency estimates for the Butte Creek near Chico gage location to an at-site flow-frequency curve provided by the Corps for the same gage location. We compared flow-frequency estimates for peak, 1-day, and 3-day flows.
16. Scaled calibrated constant loss rates to achieve an acceptable match in peak, 1-day, and 3-day flow-frequency estimates between the current study and the Corps' at-site curve.
17. Evaluated the reasonableness of the flow-frequency curves developed by comparing to previous studies.
18. Adopted the flow-frequency curves.

Results

The peak flow-frequency curves developed in this study for each analysis point are presented in Table 1. Although the complete hydrographs are available for computing volume-frequency curves, we only present the peak flow-frequency curves in Table 1 for brevity.

The results of this analysis are intended to be used in conjunction with hydraulic models of the Butte Creek watershed's channels. As such, the results presented here do not reflect backwater conditions resulting from high flows occurring outside of the study area. Furthermore, the results do not reflect any cross-basin or watershed transfers resulting from computed flows exceeding channel capacities.

Table 1. Analysis point descriptions and design storm runoff peaks at each analysis point (flow, in cfs)

Stream (1)	Analysis point (2)	Location description (3)	Contributing area (mi^2) (4)	Annual exceedence probability				
				0.1 (5)	0.02 (6)	0.01 (7)	0.005 (8)	0.002 (9)
Butte Creek	BUT-43	Butte Creek at the confluence of Butte Creek and Little Chico Creek diversion channel	183.11	16,684	27,472	32,398	37,534	44,572
	BUT-37	Butte Creek d/s[1] of the Durham-Dayton Highway	183.80	16,657	27,421	32,341	37,472	44,548
	BUT-27	Butte Creek at the Butte Sink d/s of Durham Slough	185.66	16,475	27,250	32,159	37,309	44,390

1. d/s = upstream.
2. u/s = downstream.

Study purpose

The document *Central Valley hydrology study: Ungaged watershed analysis procedures* dated November 14, 2011, describes the procedures to be used for locations in which rainfall-runoff modeling must be used to characterize flood flow-frequency. The watersheds that contain analysis points that fall into this analysis category are listed in Table 1 of the *Ungaged watershed analysis procedures* document.

The Butte Creek watershed is one of the identified ungaged watersheds. Thus, the purpose of this study is to compute flood flow-frequency relationships for the Butte Creek watershed at 3 analysis points for floods of various exceedence probabilities and durations. These analysis points are listed in Table 1.

Watershed description

Watershed overview

The Butte Creek and Little Chico Creek watershed is located in the eastern portion of the Sacramento River watershed. It is located northwest of the Feather River and southeast of Big Chico Creek. The watershed is located in Butte County, with small portions of the watershed located in Tehama and Plumas counties.

The watershed drains approximately 185.65 mi^2 and contains 2 main streams: Butte Creek and Little Chico Creek. Butte Creek originates in Jonesville Basin (Lassen National Forest) and flows to the confluence with the Sacramento River. Butte Creek enters the Sacramento River at 2 locations: through Butte Slough and through the Sutter Bypass. Little Chico Creek originates north of Butte Creek, and enters Butte Creek through the Little Chico Creek diversion near the city of Chico. The Little Chico Creek diversion provides flood protection to the City of Chico by diverting flood flows to Butte Creek by way of the diversion weir and diversion channel. For this study, we set an inflow-diversion function obtained from the Corps' *Design memorandum no.2, Sacramento River and major and minor tributaries, CA: Butte Creek, Little Chico, Butte Creek levees general design* (1957) to allow flow down Little Chico Creek downstream of the diversion.

The Butte Creek watershed is shown in Figure 1.

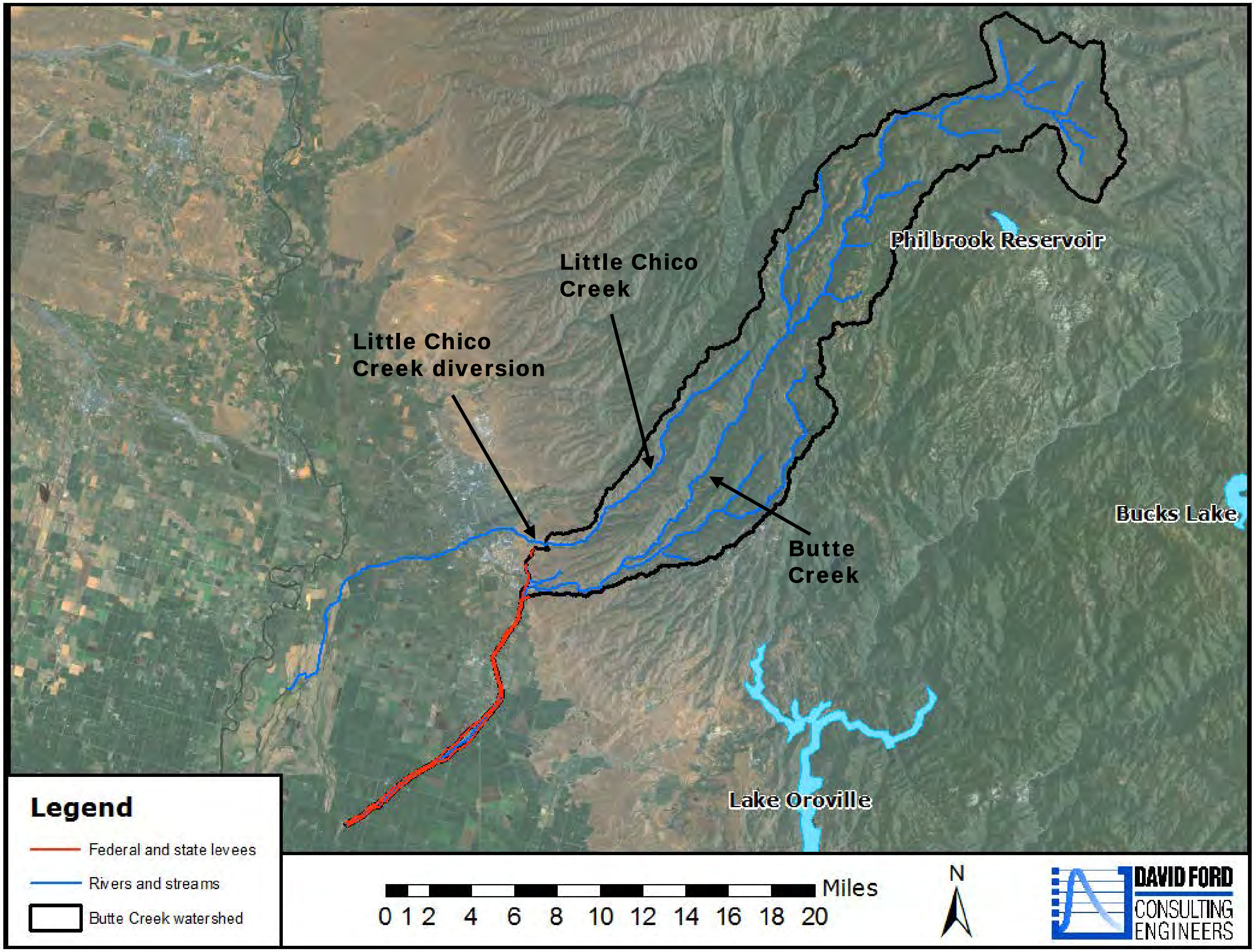

Figure 1. Butte Creek watershed

Watershed properties

The general properties of the Butte Creek watershed are described in Table 2.

Table 2. Butte Creek watershed characteristics

Watershed characteristics (1)	**Description (2)**
Climate	Hot, dry summers and cool, wet winters. Most precipitation falls as rain during the months of November through March across the majority of the watershed. Snow is more common at higher elevations in the upstream (eastern) portion of the watershed.
Elevation range[1]	The elevation ranges from approximately 50 ft to 7,190 ft, with an average elevation of approximately 3,240 ft.
Average slope[1]	25.7%
Predominant soil type	Hydrologic soil group B (Figure 2: Butte Creek Hydrologic Soil Map)
Predominant land use	Evergreen forests are predominant in the upper portion of the watershed, with herbaceous vegetation and cultivated crops predominant in the downstream portion.
Urbanized areas	Chico and Durham
Agricultural use	The watershed has areas of agriculture, which include rice fields in the downstream portion of the watershed. Rice fields present unique challenges to hydrologic modeling due to limited outlets for water to drain during the winter months. Fields are drained in late February and early March to allow the soil to dry so that farmers can access the fields in April.
Little Chico Creek Diversion	The project design flow for Little Chico Creek upstream of the diversion is estimated to be 6,700 cfs. Downstream of the diversion structure the capacity is 2,200 cfs (the non-damaging capacity of the channel through the City of Chico). The remaining flow is diverted over the concrete weir and through the diversion channel to Butte Creek. Table 4 describes the inflow-diversion flow to Butte Creek, which was based on the discharge rating curve provided in Corps' *Design memorandum no.2, Sacramento River and major and minor tributaries, CA: Butte Creek, Little Chico, Butte Creek levees general design* (1957).
Levees	Levees are located along the Little Chico Creek diversion channel and Butte Creek.

1. Elevations and average slopes are based on the 10-m USGS DEM used to delineate the watershed.

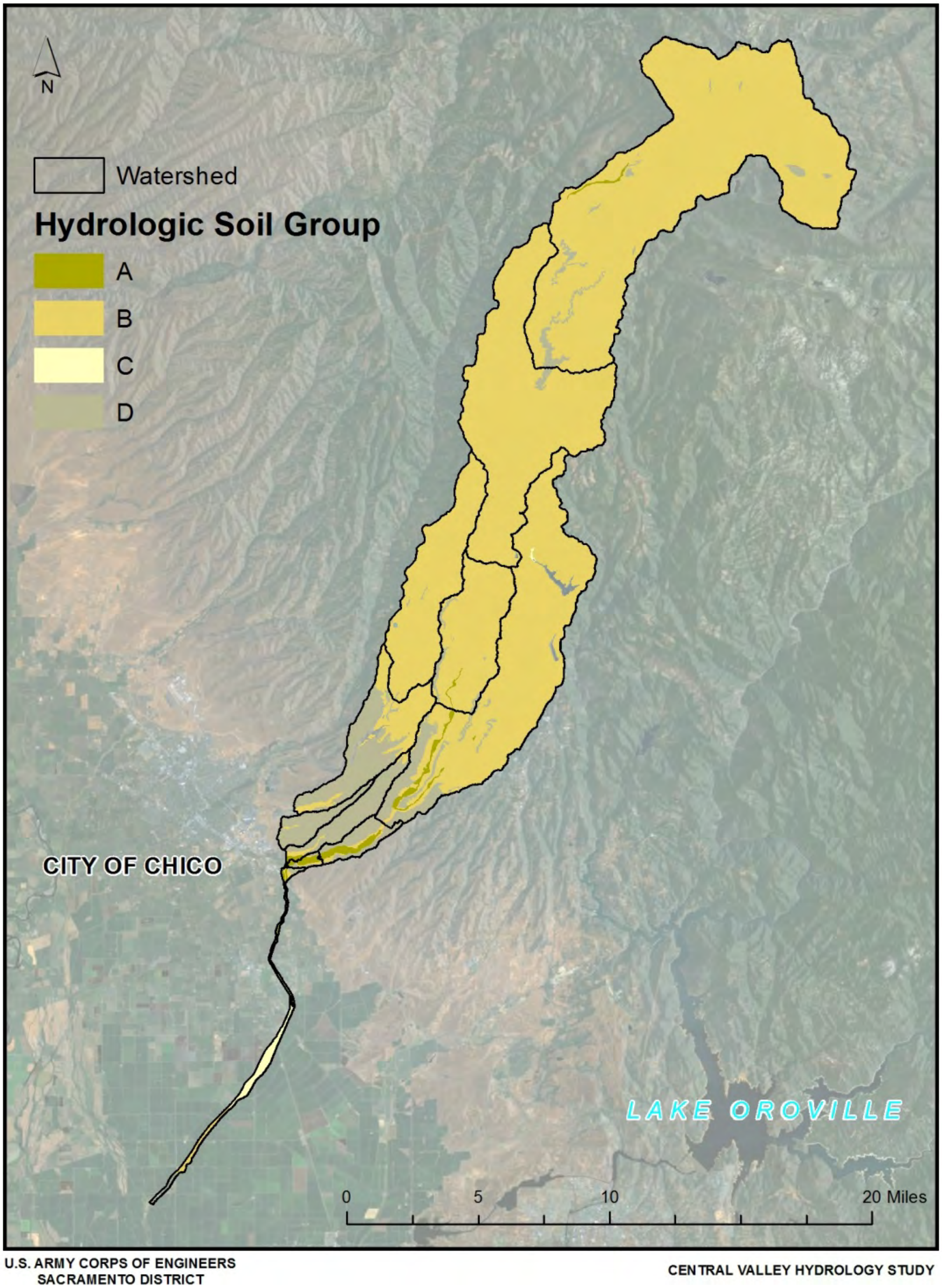

Figure 2: Butte Creek Hydrologic Soil Map

Modeling complexities

In Table 3, we describe modeling complexities of the Butte Creek watershed and how we accounted for them in the watershed model and simulations.

Table 3. Butte Creek watershed modeling complexities

Watershed complexity (1)	Modeling approach (2)
Little Chico Creek diversion	The Little Chico Creek diversion provides flood protection to the City of Chico by diverting flood flows to Butte Creek through the Little Chico Creek concrete weir and diversion channel. We developed an inflow-diversion function to model the Little Chico Creek diversion using the Little Chico Creek channel and diversion capacities described in the Corps' *Design memorandum no.2, Sacramento River and major and minor tributaries, CA: Butte Creek, Little Chico, Butte Creek levees general design* (1957). We display the inflow-diversion function in Table 4 and Figure 3.
Parrot Diversion from Butte Creek (Parrot-Phelan Diversion Dam)	This structure diverts water year round from Butte Creek into Comanche Creek. Based on a time series of flows (1996 to 2006) developed by DWR, we determined that the magnitude of diverted flows ranged between 0 and 120 cfs with an average flow of approximately 40 cfs. We will not model this development because the amount of water diverted is negligible compared to the flood flows in this area.
Upper Centerville Canal[1]	These conveyance and control structures are part of the PG&E power generation infrastructure in the Butte Creek watershed. We will not model these structures because the amount of water diverted and conveyed is negligible compared to the flood flows in this area.
Lower Centerville Dam and Canal[1]	
Centerville Powerhouse[1]	
Centerville Development[1]	
Upper Centerville Canal[1]	
Lower Centerville Diversion Dam and Canal[1]	
Centerville Powerhouse[1]	
DeSabla Development[1]	
Butte Division Dam[1]	
Toadtown Canal[1]	
DeSabla Forebay and Dam[1]	
DeSabla Powerhouse[1]	
Leveed reaches	We accounted for levees along Butte Creek during watershed delineation.

Watershed complexity (1)	Modeling approach (2)
Urbanization	The town of Durham is located in the low elevation and low slope downstream end of the watershed. We modeled these subbasins with smaller loss rates and different land use characteristics than other areas of the watershed.
Snowmelt	Snow is common in the upper portions of the watershed. A melting snowpack can increase the runoff volume in the watershed. The *Ungaged watershed analysis procedures* document states that the impacts of snow on runoff will be considered for watersheds having more than 1/3 of the basin area above 5,000 ft. Although there are portions of the Butte Creek watershed above 5,000 ft, that area is small and does not meet the 1/3 criteria. Therefore, the impacts of snow on runoff are not considered in the frequency-based design storm analysis.

1. Pacific Gas and Electric Company, *DeSabla-Centerville Hydroelectric Project FERC Project No.803*, 2008

Table 4. Little Chico Creek diversion inflow-diversion function

Inflow (cfs) (1)	Diversion to Butte Creek (cfs) (2)
0	0
888	0
1,167	107
1,568	327
2,468	1,018
3,046	1,472
3,445	1,780
3,714	1,990
4,424	2,562
5,005	3,040
5,158	3,168
5,900	3,790
6,585	4,360
6,712	4,465
6,750	4,497

1. Inflow diversion table based on Little Chico Creek Diversion Structure discharge rating curve

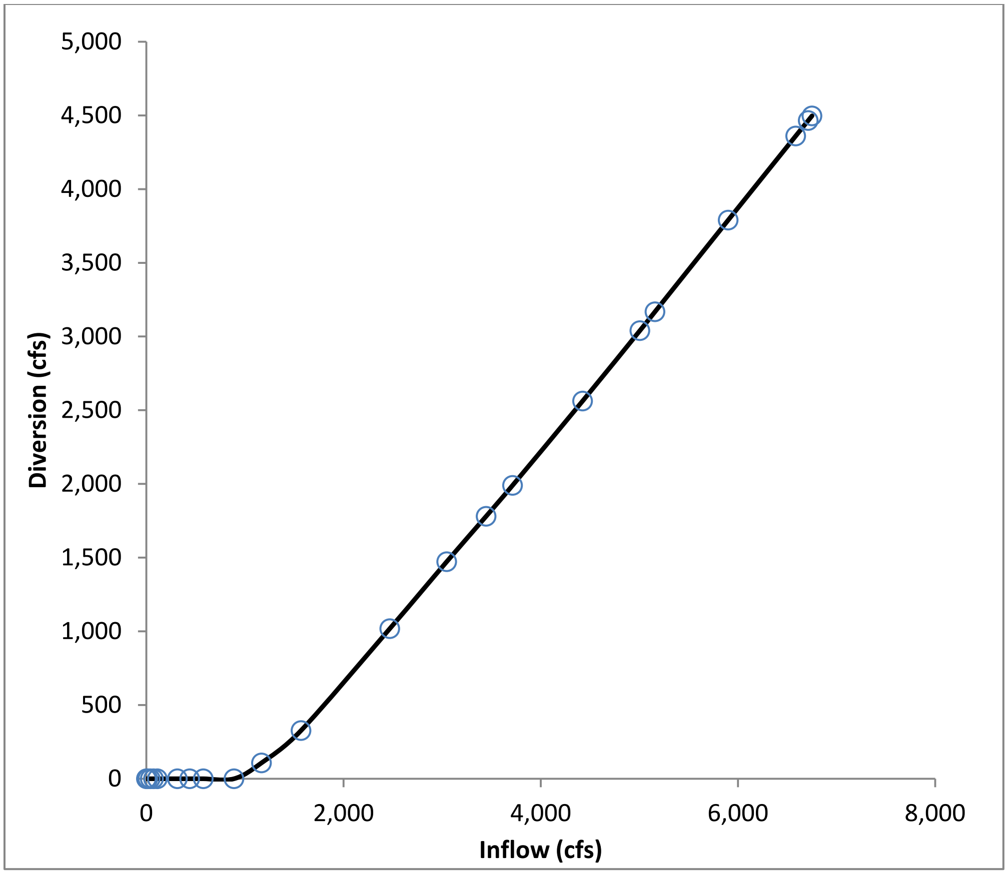

Figure 3: Little Chico Creek inflow-diversion flow curve to Butte Creek

Previous studies of the Butte Creek watershed

At the onset of the analysis, we reviewed several previous flood studies of the Butte Creek watershed. The initial intent of the review was to identify previous watershed models developed, methods used in those analyses, and existing flow-frequency curves. None of the studies we reviewed included previous watershed models. Therefore, the focus of the review was to identify existing flow-frequency curves and to identify the Little Chico Creek diversion structure's operating capacity. The previous studies are summarized in Table 5. Column 2 of Table 5 summarizes our key findings relevant to the current effort. The CVHS project team decided that a new watershed model consistent with the guidelines in the *Ungaged watershed analysis procedures* was needed because none had been developed for previous studies. The remainder of this document describes the development and application of that model. However, specific modeling details from the previous studies were retained and carried forward as noted in this document.

Table 5. Summary of previous studies

Previous study (1)	**Summary (2)**
Corps, *Design memorandum no. 2, Sacramento River and major and minor tributaries, CA: Butte Creek, Little Chico, Butte Creek levees general design*, 1957	This memorandum covers the general design for levee construction and channel improvements on Butte Creek upsteam of the Highway 99 bridge including the Little Chico Creek diversion structure and diversion channel. The memorandum describes preproject conditions and establishes design criteria. The channel capacity of Little Chico Creek upstream of the diversion structure is estimated to be 6,700 cfs. Downstream of the diversion structure the flow of Little Chico Creek is limited to 2,200 cfs. Therefore, 4,500 cfs will be diverted over the diversion structure's concrete weir and through the Little Chico Creek diversion channel to Butte Creek.
Corps, *Office Study: Little Chico Creek*, 1963	This study summarizes a preliminary appraisal by the USACE of the flooding problem along Little Chico Creek through the town of Chico. In addition to the preliminary appraisal, the study describes alternative solutions investigated by the USACE. The study determined that sedimentation, vegetative growth, and other obstructions had reduced the controlling capacity of Little Chico Creek below the diversion through Chico. Little Chico Creek could no longer pass the design flow rate of 2,200 cfs. The study concluded that a "clear and snag" operation would be economically feasible and would be implemented to improve channel capacity of Little Chico Creek below the diversion structure.
Sacramento and San Joaquin river basins comprehensive study (Comp Study (USACE 2002))	The Comp Study was a system-wide analysis of the Central Valley. The study included hydrologic, reservoir, hydraulic, and economic analyses. Flow-frequency curves were developed at numerous locations throughout the Sacramento and San Joaquin river basins. A flow-frequency curve was developed for the Butte Creek near Chico streamgage.

Strategy

Our overarching goal was to calculate flow-frequency curves for the Butte Creek watershed's 3 analysis points. To meet this goal, the project team devised a strategy for using all available data and information to formulate best estimates of flow-frequency within the watershed. To develop the flow-frequency curves according to this agreed-upon strategy, we:

1. Developed an HEC-HMS watershed model of the Butte Creek watershed.
2. Calibrated the HEC-HMS watershed model to selected historical high water events.
3. Developed design storm hyetographs for the Butte Creek watershed based on NOAA Atlas 14 and depth-area reduction factors from HMR 59.
4. Applied the design storm hyetographs to the calibrated HEC-HMS basin model.
5. Compared resulting flow-frequency curves for the Butte Creek near Chico gage location to at-site peak, 1-day, and 3-day flow-frequency curves, for the same location, provided by the Corps.
6. Adjusted subbasin constant loss rates in the calibrated HEC-HMS model to achieve an acceptable match between the rainfall-runoff-generated and at-site flow-frequency curves.
7. Compared the resulting flow-frequency curves to those computed using other methods or published in prior studies to determine whether the flow-frequency curves fell within an expected range.
8. Adopted the flow-frequency curves for all 3 Butte Creek analysis points.

Delineation of the watershed for modeling

The watershed was delineated to determine the extents of the watershed and the location and size of subbains to be configured in the HEC-HMS model. Prior to delineating the watershed, we used the GIS tools included in HEC-GeoHMS to develop the drainage network. We did this by following the terrain preprocessing steps outlined in Chapter 6 of the *HEC-GeoHMS user's manual* (HEC 2009). During terrain preprocessing, we filled sinks in the USGS DEM to ensure that all runoff drains from the watershed. We also built walls onto the USGS DEM to impede the flow of water where levees are present in the watershed.

Following terrain preprocessing, we delineated the watershed into subbasins. We adjusted the default watershed delineations to fit the study needs. The watershed was delineated with the intent of simulating large storm events covering the entire watershed area. In general, we adjusted delineations using the following criteria:

- Watersheds should be broken into as few subbasins as possible.
- Subbasin outlets should be specified at all locations where flow information is needed, such as at a CVHS analysis point or a streamgage.
- Subbasin characteristics should be approximately homogenous over a given subbasin area.

In all, we delineated 13 subbasins for the Butte Creek watershed area. The subbasins range in size from 0.31 mi^2 to 66.14 mi^2, with an average subbasin area of 14.28 mi^2. A figure of the delineated subbasins and figures showing screenshots of the HEC-HMS basin model are located in Appendix I.

After delineating the watershed, we extracted the physical features from each subbasin that are necessary to develop the rainfall-runoff transform, routing reach models, and loss characteristics. Data and information sources used for watershed delineation and estimating model parameters are listed in Table 6.

Table 6. Data and information sources used for watershed delineation and estimation of model parameters

Watershed data or information (1)	**Source (2)**	**Remarks (3)**
Topographical data	10-m USGS DEM	Topographical data are required for watershed delineation and determination of basin and stream slopes.
Watershed boundaries	USGS 8-digit hydrologic unit code (HUC-8) boundaries	We used published watershed boundaries from HUC-8 to check the reasonableness of delineations. We did not force watershed delineations to follow HUC-8 boundaries, as watershed boundaries delineated from the USGS DEM matched closely with those from HUC-8.
Stream alignments	National Hydrography Dataset (NHD) flowlines	We "burned" known stream alignments into an USGS DEM to force the flow of water to follow known flow paths.
Levee locations	Federal levee segment shapefile	We used a shapefile of existing levees to impose walls onto the USGS DEM, essentially impeding the flow of water where levees are present.
Soils	Soil Survey Geographic (SSURGO) database (USDA-NRCS 2010)	We used the SSURGO database to determine appropriate locations for subdividing the watersheds and to estimate the most predominant soil type/classification in each subbasin. We then used the *HEC-HMS technical reference manual* (HEC 2000; Chapter 5) to estimate the constant loss rates for each subbasin.
Land use/cover	USGS National Land Cover Dataset of 2001 (NLCD 2001) (http://landcover.usgs.gov; Homer et al. 2007)	We used NLCD 2001 to determine appropriate locations for subdividing the watersheds and to estimate the most predominant land cover types and average percent impervious values for all subbasins.
Analysis point locations	Analysis point shapefile	We added delineation points to correspond with analysis point locations. Delineations were not made at analysis points where the incremental area between analysis points was small.
Streamgage locations	Streamgage shapefile	In addition to analysis points, we also delineated subbasins to have outlets corresponding with the Butte Creek near Chico, Butte Creek near Durham, and Little Chico Creek diversion gages to facilitate model calibration.

Required model parameters, transforms, and routings

The modeling methods we intended to use for the Butte Creek watershed before model development began are outlined in the *Ungaged watershed analysis procedures* document. The reviewed reports did not include any previous model; therefore, we found no reason to change our original analysis plan. Below, we describe the selected models to represent runoff volume, baseflow, and rainfall-runoff transforms.

Runoff volume

We used the initial and constant loss method for runoff volume modeling. Initial losses were estimated from Table 5-1 of the *Sacramento City/County drainage manual volume 2: hydrology standards* (Sacramento County 2006). We optimized initial losses during model calibration to represent accurately the time runoff production should begin for each event.

We used the SSURGO database (USDA-NRCS 2010) to determine the predominant hydrologic soil group for each subbasin. We then used Table 11 in the *HEC-HMS technical reference manual* (HEC 2000) to estimate the constant loss rates that correspond with these soil groups. These constant loss rates are presented in a table in Appendix I. We adjusted constant loss rates during model calibration, as necessary, to improve the relationship between simulated and observed flows.

Baseflow

We began model development assuming no baseflow. However, during calibration, we determined that baseflow was necessary to simulate accurately historical events in the watershed. We used the recession baseflow model in HEC-HMS and optimized the initial baseflow, ratio-to-peak threshold, and baseflow recession constants during model calibration. Baseflow represented a small component of the total flow for all calibration events.

Rainfall-runoff transform

We used the user-specified S-graph rainfall-runoff transform for the watershed. To model the watershed we used 3 S-graphs: mountain, foothills, and valley. These S-graphs are provided in *LAPRE-1: Los Angeles District preprocessor to HEC-1* (USACE 1989). The valley S-graph is recommended for areas with a basin slope less than 200 ft/mi, the foothill S-graph for areas with a basin slope between 200 and 400 ft/mi, and the mountain S-graph for areas with a basin slope greater than 400 ft/mi.

The lag time for each subbasin was computed as described in *Improved procedures for determining drainage area lag values* (USACE 1962) using the equation:

$$T_{lag} = 24n\left(\frac{LL_{ca}}{S^{0.5}}\right)^{0.38} \quad (1)$$

where T_{lag} = the unit hydrograph lag, in min; S = watershed slope, in ft/mi; L = length of longest watercourse, in mi; L_{ca} = length along the longest watercourse to the centroid, in mi; and n = basin roughness coefficient. These parameters are shown for each subbasin in Appendix I. We adjusted basin lag times during model calibration on an event-by-event basis to improve the relationship between simulated and observed flows in the watershed. Equation 1 and the recommended S-graphs are consistent with the *Ungaged watershed analysis procedures* document.

Flow routing

Based on guidance from the *HEC-HMS technical reference manual* (HEC 2000), we used Muskingum-Cunge routing to represent channel flow in the watershed. We used the USGS DEM and HEC-GeoRAS version 4.2.93 to cut several cross sections for each routing reach in the watershed model. We then selected a representative cross section for each reach to estimate the geometry, which was either triangular or trapezoidal in shape.

Manning's *n*-values were assigned for each reach based on channel characteristics observed in aerial photos and Manning's *n* tables in the *HEC-RAS hydraulic reference manual* (HEC 2010). The routing geometries and Manning's *n*-values for each reach are presented in Appendix I. We confirmed final Manning's *n*-values and channel geometries as reasonable for selected reaches during a field visit to the watershed.

Computation time step

The computation time step for the HEC-HMS model is 15 minutes. This time step was chosen so that it adequately captures the runoff peak. The computation time step does not exceed 1/5 the final basin lag time for the smallest subbasin, a general rule for hydrologic modeling (USACE 1993). We also ensured that the travel time through each routing reach was greater than the model time step.

Calibration of model to historical events

Historical calibration strategy

The Butte Creek near Chico, Butte Creek near Durham, and Little Chico Creek diversion near Chico streamgages are the only locations in the Butte Creek watershed with sufficient data suitable for watershed model calibration. We calibrated the watershed model using available precipitation data in and near the watershed available through the California Data Exchange Center (CDEC). Historical flow data used for calibration were collected by the Corps during the early stages of CVHS.

We were not able to calibrate the watershed downstream of the Butte Creek near Durham gage. This ungaged area is approximately 2.2 mi^2, a small portion of the 185.7 mi^2 Butte Creek watershed's total area. We adjusted initial conditions and model parameters in the ungaged portion of the Butte Creek watershed according to adjustments we made during calibration of the gaged portion of the watershed.

Historical hydrometeorological data collection

We collected hourly precipitation data for calibration from rain gages in and near the Butte Creek watershed. The 3 selected events and the gages used for calibration for each event are presented in Table 7. The gage names in columns 2 through 5 of Table 7 correspond with the 3-letter CDEC identifier for each gage. A map of the gages used for calibration is shown in Figure 4.

Table 7. Precipitation gages in and near the Butte Creek watershed used for calibration

Event	Precipitation gages used for calibration[1]			
	CAR (2)	CST (3)	CHI (4)	BMT (5)
(1)				
December 24, 1996 – January 08, 1997		●	●	
January 25, 1998 – March 01, 1998		●	●	●
December 18, 2005 – January 10, 2006	●	●	●	

1. CDEC identifier.

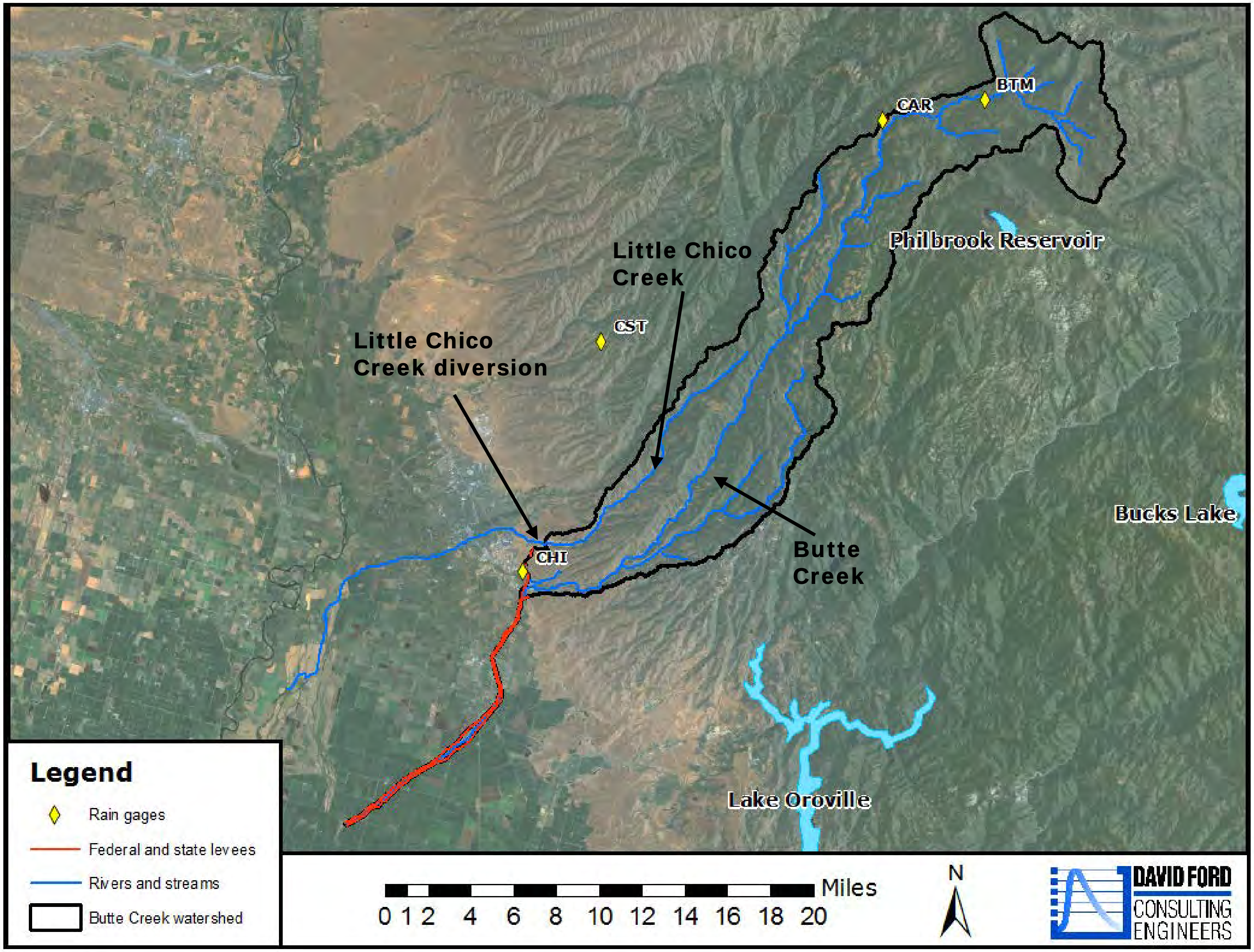

Figure 4. Precipitation gages (labeled with CDEC IDs) used for calibration of the Butte Creek watershed model

Historical precipitation input development

The Butte Creek watershed has an elevation range of approximately 50 ft to 7,190 ft. There is also a wide range in the elevation of the precipitation gages used to calibrate the model. Due to the orographic enhancement of precipitation along the Sierra Nevada, the mean annual precipitation (MAP) varies greatly throughout the watershed. To account for this, we created an indexing system for each gage and each subbasin centroid based on the MAP at those locations, where the index number is the MAP. The indexing system allowed for elevation differences throughout the watershed to be accounted for when spatially interpolating precipitation across the watershed.

To determine the MAP for each subbasin centroid and precipitation gage, we overlaid shapefiles of each with gridded MAP generated using the Parameter-elevation Regressions on Independent Slopes Model (PRISM). PRISM is an analytical tool that uses a DEM, point data, and other spatial datasets to generate gridded estimates of climate parameters. For this study, we chose the 30-year climatological average from 1971-2000 to represent the MAP, which we downloaded directly from the PRISM website (http://www.prism.oregonstate.edu). Gridded PRISM precipitation data account for spatial variations in precipitation due to:

- Ground elevation.
- Terrain orientation and steepness.
- Moisture regime.
- Coastal proximity.
- Temperature inversions.

To distribute point rainfall throughout the watershed, we chose the inverse distance squared weighting method. In this method, precipitation gage weights are computed and assigned based on the distances from the gages to the centroid of each subbasin. More information regarding this method can be found in the *HEC-HMS technical reference manual* (HEC 2000).

In HEC-HMS, the user can control how far the program will search from the subbasin node for a precipitation gage. If the precipitation gage is farther from the node than the specified distance, the program will not use that precipitation gage when calculating the rainfall for that node. For all gages, a search distance of 20 mi was specified.

Historical calibration simulations

We calibrated the Butte Creek watershed model to the 3 historical events listed in Table 7. Our goal during calibration was to simulate accurately historical flow time series at the Butte Creek near Chico, Butte Creek near Durham, and Little Chico Creek diversion near Chico streamgage locations. To do this, we adjusted:

- Basin lag times.
- Initial losses.
- Constant loss rates.
- Baseflow characteristics.

Historical calibration results

Optimized basin lag times for each historical event and the final basin lag times used for simulating all design storms are presented in Appendix II. Final basin lag times, which are weighted averages of the 3 event-specific values, are approximately 70 percent greater than the original basin lag time estimates computed as a function of subbasin physical properties. We did not adjust final basin lag times during the calibration of the model to the at-site flow-frequency curves for the Butte Creek near Chico streamgage.

Although we optimized subbasin initial losses for each historical simulation, we used the initial loss values from the *Sacramento City/County drainage manual* (Sacramento County 2006) for simulating all frequency-based design storms. Adjusting the initial losses to match better the beginning of the events did not influence our calibration of the event peaks, and thus did not affect our calibrated constant loss rates.

Optimized constant loss rates for each historical event and the provisional constant loss rates used for simulating preliminary flow-frequency estimates are also presented in Appendix II. The provisional constant loss rates are weighted averages of the 3 sets of event-specific constant loss rates. These loss rates are provisional because they were subject to further refinements during calibration of the basin model to the Corps' at-site flow-frequency curves for the Butte Creek near Chico streamgage. The provisional constant loss rates are approximately 10 percent to 60 percent smaller than the original constant loss rate estimates, which we derived from hydrologic soil group classifications.

Optimized initial baseflows, baseflow recession constants, and ratio-to-peak thresholds for each historical event and the final baseflow characteristics used for simulating all design storms are presented in Appendix II. We did not adjust the final baseflow characteristics during the calibration of the model to the at-site flow-frequency curves for the Butte Creek near Chico streamgage.

We did not calibrate the basin models for subbasins downstream of the Butte Creek near Durham gage because these subbasins are ungaged. Instead, we adjusted the initial conditions and calibration parameters for this ungaged area using the same relative adjustments we made upstream in the gaged portion of the watershed.

Since we made no changes to the hydrologic routing reach parameters during calibration, the physical properties and parameter values presented in

Appendix I are the final values used to simulate the design storms. Furthermore, we made no changes to these final values during calibration of the model to the at-site flow-frequency curves for the Butte Creek near Chico streamgage.

After developing final parameter sets and provisional constant loss rates, we re-ran the model for each historical event. Results for these verification simulations can be found in Appendix II. The verification simulation results match well the timing of the observed hydrograph peaks. Since the final parameters are weighted averages of the optimized parameters for each event, the magnitudes of the verification simulation peaks and volumes overestimate or underestimate observed peaks and volumes depending on the event. The HEC-HMS model that we used for model verification served as a starting point during the next step of model calibration where we scaled the provisional constant loss rates to match suitably the peak, 1-day, and 3-day flows associated with the at-site flow-frequency curves for the Butte Creek near Chico streamgage.

Development of design precipitation input

Overview

Using the rainfall-runoff model we developed, we used frequency-based design storms to compute flow-frequency curves at 3 analysis points in the Butte Creek watershed. To do this, we applied design storm hyetographs to the watershed model, computed the runoff hydrographs, and assigned the AEPs of the design storm hyetographs to the computed runoff hydrograph peaks.

Design storm development

Spatially interpolated, high-resolution precipitation-frequency estimates from NOAA Atlas 14 were used to simulate the design storms. Depth-duration-frequency curves are published in NOAA Atlas 14 for a wide range of durations and frequencies. To develop the precipitation input for the HEC-HMS basin model, we:

1. Downloaded gridded precipitation-frequency estimates from the Hydrometeorological Design Studies Center (HDSC) website (http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_gis.html). Precipitation estimates were downloaded for 5 AEPs (p=0.1, p=0.02, p=0.01, p=0.005, and p=0.002) and 15 durations ranging from 5 minutes to 10 days. This resulted in 5 x 15 = 75 precipitation-frequency grids for the Butte Creek watershed. We extracted precipitation-frequency estimates from the annual maximum time series analysis only.
2. Converted each ASCII grid to a raster grid and reprojected each to an Albers equal-area projection.
3. Computed the average cumulative precipitation in each subbasin for the 5 frequencies and 15 durations.
4. Created 7 precipitation zones across the watershed by grouping together subbasins with similar precipitation-frequency estimates. The zones were created based on the average 100-year precipitation estimates in each subbasin. A map of the zones is shown in Figure 5.
5. Calculated an area-weighted precipitation depth for each zone for all frequencies and durations. Hereafter, we will refer to these depths as *zonal precipitation-frequency estimates*.
6. Determined 1 storm centering for calculating the depth-area reduction factors appropriate for all 3 analysis points. We used 1 storm centering to represent all 3 analysis points because the contributing areas for those analysis points were similar in magnitude, ranging from 183.11 to 185.66 mi^2. Table 8 shows the grouping of the analysis points, the contributing areas of each, and the representative area.
7. Determined and applied depth-area reduction factors to the zonal precipitation-frequency estimates from step 5. We applied a unique depth-area reduction factor for each durational precipitation depth. For this, we used guidance and depth-area reduction factor tables from HMR 59.
8. Developed temporally-balanced hyetographs based on the reduced zonal precipitation-frequency estimates from step 7. We developed a design storm hyetograph for each of the 7 zones, 5 frequencies, and 1 storm

centering. We specified the maximum precipitation intensity of each design storm hyetograph to occur at 50 percent of the total 4-day storm duration. We developed the balanced design storm hyetographs using the HEC-HMS frequency storm meteorologic model as a precipitation preprocessor.

In all, we developed 35 design storm hyetographs (7 zones × 5 frequencies × 1 centering). We developed these hyetographs using a 1-hr time step.

Because HEC-HMS does not allow the user to input precipitation depths for the 10-minute, 30-minute, and 3-day durations, which are part of the NOAA Atlas 14 dataset, we did not use these precipitation estimates when developing the balanced hyetographs. Furthermore, we did not develop balanced hyetographs using depth durations greater than 4 days.

A balanced hyetograph, with depth-area reduction factors applied, is shown in Figure 6. All balanced hyetographs developed have the same temporal pattern shown in Figure 6.

9. Applied each of the balanced hyetographs developed in step 8 to the HEC-HMS basin model calibrated to historical events. We configured the balanced hyetographs in the HEC-HMS model as precipitation gages and linked those gages to the appropriate subbasins based on the zones in Figure 5.
10. Simulated 5 design storms using the HEC-HMS basin model calibrated to historical events (5 frequencies × 1 storm centering). The flows resulting from these simulations served as preliminary flow-frequency estimates subject to further refinement by calibrating the HEC-HMS basin model to at-site flow-frequency curves for the Butte Creek near Chico streamgage.

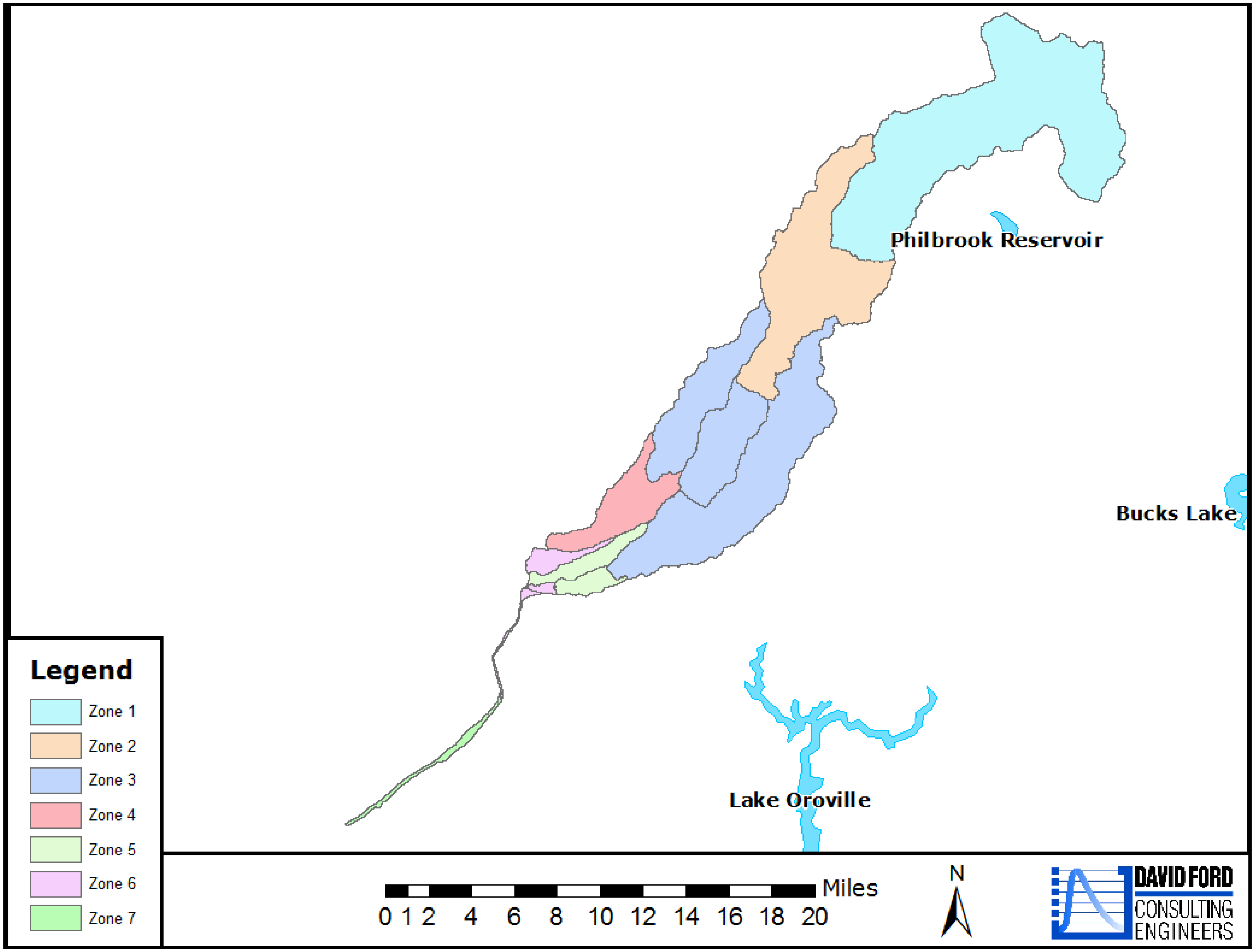

Figure 5. Zones created in the Butte Creek watershed based on subbasins with similar precipitation-frequency estimates

Table 8. Butte Creek watershed storm centering: The representative areas for each centering are used to determine depth-area reduction factors for that centering.

Storm centering (1)	Analysis point (2)	Contributing area (mi^2) (3)	Representative area (mi^2)[1] (4)
1	BUT-43	183.11	183.11
	BUT-37	183.80	
	BUT-27	185.66	

1. We chose the smallest area in each grouping as the "representative area" for computing depth-area reductions.

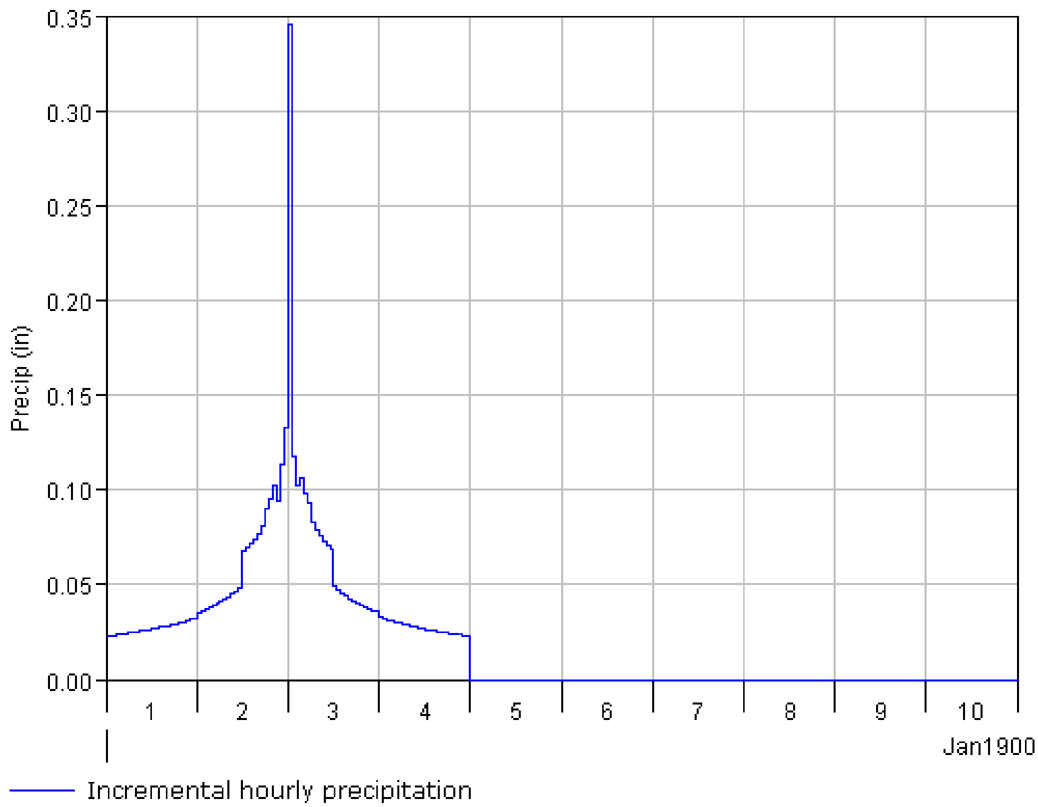

Figure 6. Balanced hyetograph for subbasin Sub_BUT-43_10 for the p=0.01 event with a storm centering of 183.11 mi^2

Computation of flow-frequency curves

Overview

Development of the design storms and the configuration of those design storms in the HEC-HMS basin models were described in the previous chapter. In this chapter, we describe the computation of flow-frequency curves at the Butte Creek near Chico streamgage and the 3 Butte Creek analysis points.

To develop frequency curves at all locations of interest in the Butte Creek watershed, we:

1. Configured and simulated 5 design storms (5 frequencies × 1 storm centering) with the HEC-HMS basin model calibrated to historical events. The results of these simulations were preliminary design storm hydrographs at each of the 3 analysis points in the watershed and the Butte Creek near Chico streamgage. We assigned the frequency of the applied design storm hyetographs to the corresponding computed hydrograph peaks, maximum 1-day flows, and maximum 3-day flows at all locations of interest.
2. Compared the computed peak, 1-day, and 3-day flow-frequency curves for the Butte Creek near Chico streamgage location to the at-site peak, 1-day, and 3-day flow-frequency curves for that same location. The Corps developed these at-site curves using flow-frequency analysis applied to historical streamgage data.
3. Adjusted the HEC-HMS basin models' provisional constant loss rates in a uniform fashion to achieve acceptable comparisons between the resulting peak, 1-day, and 3-day flow-frequency curves for the Butte Creek near Chico streamgage location and the Corps' at-site curves.
4. Extracted the computed design storm hydrographs for the 3 Butte Creek analysis points for all 5 frequencies. We extracted the hydrographs corresponding with the optimized constant loss rates from step 3.
5. Used the extracted hydrographs to determine the peak flow-frequency curves for the 3 Butte Creek analysis points.

Results

We compare preliminary and at-site peak, 1-day, and 3-day flow-frequency curves for the Butte Creek near Chico streamgage location in Table 9. Columns 2-4 compare the peak flow-frequency curves, columns 5-7 compare the 1-day flow-frequency curves, and columns 8-10 compare the 3-day flow-frequency curves. As can be seen, the preliminary curves are significantly greater in magnitude than the at-site curves for the Butte Creek near Chico streamgage. This suggests that the provisional constant loss rates should be increased, which will, in turn, decrease the resulting flood peaks and volumes.

In Table 10, we show the final and at-site peak, 1-day, and 3-day flow-frequency curves for the Butte Creek near Chico streamgage location. We developed the final flow-frequency curves by scaling the provisional constant loss rates by a factor of 2 for all subbasins. As can be seen, the peak and 1-day flow-frequency curves match much more closely after we applied this scale factor. The final 3-day flow-frequency curves, however, are approximately 20 to 30 percent lower than the at-site curves. After coordinating with the project team, we determined that this discrepancy was acceptable because, for the largely unregulated Butte Creek watershed, peak and 1-day flows generally define floodplain mapping.

In Table 11, we compare the provisional and final constant loss rates against the constant loss rates indicated by the predominant soil type for each subbasin. HEC (2000) includes a table that relates hydrologic soil group classifications to expected constant loss rates. We show these expected constant loss rates, for each subbasin, in column 2. We show the provisional constant loss rates, which we determined by calibrating the HEC-HMS basin model to historical events, in column 3. As can be seen, the provisional constant loss rates often fall below the expected range, which may explain why preliminary flow-frequency curves significantly exceeded the at-site curves. In column 4, we show the final constant loss rates, which are 2 times the provisional rates. The final constant loss rates generally fall within the expected ranges.

Figure 7 shows example computed design storm hydrographs for the Butte Creek near Chico streamgage location. We show these example hydrographs, which represent the p=0.01 event, computed using both the provisional and final constant loss rates. As can be seen, application of the final constant loss rates result in a significantly smaller hydrograph peak and volume.

The Butte Creek near Chico streamgage is located upstream of all 3 analysis points. The streamgage has a contributing area of 147 mi^2, which is smaller than the 183.11 mi^2 centering we used for final calibration and design storm simulations. Nonetheless, for a duration of 1 hr, the HMR 59 depth-area reduction factors compare closely for areas of 147 mi^2 and 183.11 mi^2—78.7 percent versus 76.6 percent, respectively. Because of this, we used the final constant loss rates to compute design storm flows at all 3 Butte Creek analysis points.

Table 9. Preliminary flow-frequency curve comparisons for Butte Creek near Chico: Results from using the provisional constant loss rates

AEP (1)	At-site peak[1] (cfs) (2)	Preliminary peak[2] (cfs) (3)	Difference (%) (4)	At-site 1-day[1] (cfs) (5)	Preliminary 1-day[2] (cfs) (6)	Difference (%) (7)	At-site 3-day[1] (cfs) (8)	Preliminary 3-day[2] (cfs) (9)	Difference (%) (10)
p=0.1	13,944	21,310	52.8	10,219	15,347	50.2	7,315	9,177	25.5
p=0.02	23,275	31,137	33.8	17,045	23,327	36.9	12,443	14,634	17.6
p=0.01	27,741	35,529	28.1	20,201	26,825	32.8	14,877	17,053	14.6
p=0.005	32,497	40,077	23.3	23,486	30,388	29.4	17,450	19,516	11.8
p=0.002	39,245	46,347	18.1	28,021	35,205	25.6	21,065	22,830	8.4

1. At-site flows were determined using statistical flood-frequency analysis on historical streamgage data.
2. Preliminary flows were determined using rainfall-runoff modeling with the provisional constant loss rates.

Table 10. Final flow-frequency curve comparisons for Butte Creek near Chico: Results from using the final (provisional scaled by 2) constant loss rates

AEP (1)	At-site peak[1] (cfs) (2)	Final peak[2] (cfs) (3)	Difference (%) (4)	At-site 1-day[1] (cfs) (5)	Final 1-day[2] (cfs) (6)	Difference (%) (7)	At-site 3-day[1] (cfs) (8)	Final 3-day[2] (cfs) (9)	Difference (%) (10)
p=0.1	13,944	15,060	8.0	10,219	9,281	-9.2	7,315	5,267	-28.0
p=0.02	23,275	24,359	4.7	17,045	16,547	-2.9	12,443	9,454	-24.0
p=0.01	27,741	28,633	3.2	20,201	19,905	-1.5	14,877	11,507	-22.7
p=0.005	32,497	33,114	1.9	23,486	23,393	-0.4	17,450	13,698	-21.5
p=0.002	39,245	39,319	0.2	28,021	28,151	0.5	21,065	16,749	-20.5

1. At-site flows were determined using statistical flood-frequency analysis on historical streamgage data.
2. Final flows were determined using rainfall-runoff modeling with the final constant loss rates.

Table 11. Provisional and final constant loss rates for Butte Creek watershed

Subbasin (1)	**Expected loss rates (in/hr)[1] (2)**	**Provisional constant loss rates (in/hr)[2] (3)**	**Final constant loss rates (in/hr)[3] (4)**
Sub_BUT-27_1	0.00-0.05	0.021	0.042
Sub_BUT-37_1	0.00-0.05	0.021	0.042
Sub_BUT-37_2	0.15-0.30	0.061	0.122
Sub_BUT-43_1	0.00-0.05	0.021	0.042
Sub_BUT-43_10[4]	0.15-0.30	0.192	0.192
Sub_BUT-43_2	0.00-0.05	0.023	0.046
Sub_BUT-43_3	0.00-0.05	0.023	0.046
Sub_BUT-43_4	0.15-0.30	0.08	0.16
Sub_BUT-43_5	0.15-0.30	0.08	0.16
Sub_BUT-43_6	0.15-0.30	0.08	0.16
Sub_BUT-43_7	0.15-0.30	0.08	0.16
Sub_BUT-43_8	0.00-0.05	0.023	0.046
Sub_BUT-43_9[5]	0.05-0.015	0.086	0.086

1. Expected constant loss rates are indicated by the predominant hydrologic soil group classification for each subbasin.
2. Provisional constant loss rates were determined by calibrating the HEC-HMS basin model to 3 historical events.
3. Final constant loss rates were optimized by uniformly scaling the provisional constant loss rates to match the at-site Butte Creek near Chico flow-frequency curves.
4. Loss rate value was not scaled by 2
5. Loss rate value was not scaled by 2

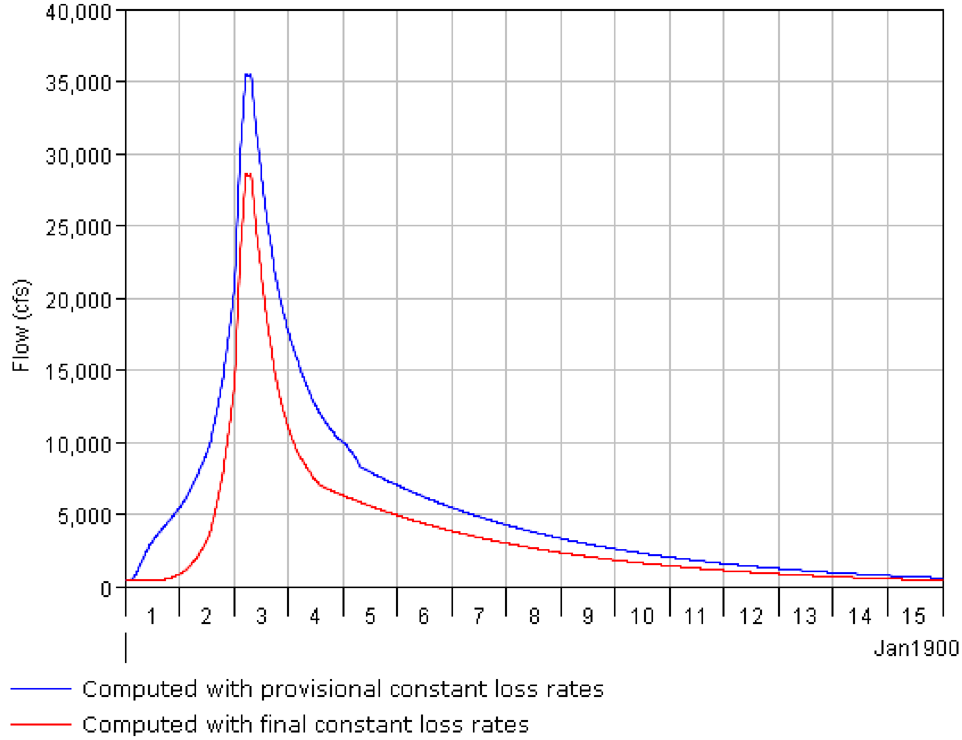

Figure 7. Design storm hydrographs for Butte Creek near Chico computed using the provisional and final constant loss rates (p=0.01, 183.11 mi^2 centering)

Evaluation of results

Overview

Before adopting the flow-frequency curves developed in this study, we evaluated the reasonableness of our model results. To do this, we compared the results of this study to peak flows calculated using USGS regression equations and results from the *Sacramento and San Joaquin river basins comprehensive study* (Comp Study).

Comparison to USGS regression equations

We compared the flow-frequency curves developed in this study to peak flow estimates obtained from USGS regional regression equations. The regression equations are used for estimating peak flows in a watershed for recurrence intervals ranging from p=0.5 to p=0.01.

Regression equations for California are published in *Magnitude and frequency of floods in California* (Waananen and Crippen 1977). These equations can also be found on the USGS website: http://water.usgs.gov/software/NFF/manual/ca.

6 sets of regression equations are available for California. Each set of regression equations is applicable to a specific hydrologic region of the state. The Butte Creek watershed lies within the Sierra hydrologic region.

The equation for the p=0.01 event for the Sierra region is:

$$Q_{1\%} = 15.7A^{0.77}\, P^{1.02}\ \ H^{-0.43} \qquad (2)$$

where $Q_{1\%}$ = the peak flow for the p=0.01 event, in cfs; A = the drainage area, in mi^2; P = the mean annual precipitation, in inches; and H = the average main channel elevation at 10 percent and 85 percent points along the main channel length, in 1,000 ft.

We used an interactive mapping application called StreamStats to estimate the peak flows at 1 location in the watershed. StreamStats delineates the watershed and invokes the USGS regression equations to calculate the peak flows for the watershed area. The StreamStats interactive map can be found at: http://water.usgs.gov/osw/streamstats/california.html.

A comparison of the results from this study and those using StreamStats to apply the USGS regional regression equations is presented in Table 12. In Table 12, we compare the peak flows at analysis point BUT-27. Since the regression equations do not estimate peak flows beyond the p=0.01 event, we only compare results for the p=0.1, p=0.2, and p=0.01 events. StreamStats will estimate the p=0.002 event by extrapolating the flow-frequency curve. However, we do not show or compare these results.

Table 12 shows that the peak flows developed in this study are lower than those calculated using the USGS regression equations for all 3 AEPs. While the peak flows associated with the p=0.1 and p=0.02 events compare well, the current study's p=0.01 peak flow is nearly 50 percent less than the USGS regression equation peak flow.

Table 12. Comparison of peak flows (in cfs) derived in this study to those derived from USGS regression equations

Analysis point (1)	Watershed area (mi^2) (2)	AEP (3)	Current study (4)	USGS regression equations (5)	Difference (%)[1] (6)
BUT-27	185.66	0.1	16,475	17,600	-6.4
		0.02	27,250	31,400	-13.2
		0.01	32,158	61,800	-48.0

1. Difference = [(current study – USGS) / USGS] × 100.

Comparison to Comp Study

The Comp Study developed flow-frequency curves at the Butte Creek near Chico gage by completing a flow-frequency analysis of historical streamflow data. We compared the Comp Study 1-day flow-frequency curve to the results from this study. (It is important to note that the Comp Study extracted annual maximum 1-day flows using a midnight-to-midnight window; for the current study and CVHS in general, we extract annual maximum 1-day flows using a moving 24-hr window.)

Table 13 shows that the 1-day flows computed for this study are consistently smaller than those from the Comp Study. Furthermore, the discrepancies between the 2 studies increase as the AEP decreases. Nonetheless, even for the p=0.002 event, the current study's 1-day flow remains within 20 percent of the Comp Study's 1-day flow.

Table 13. Comparison of maximum average 1-day flows (in cfs) between results derived in this study and the Comp Study

Location (1)	AEP (2)	Current study (3)	Comp Study[1] (4)	Difference (%)[2] (5)
Butte Creek near Chico	0.1	9,281	10,000	-7.2
	0.02	16,547	19,000	-12.9
	0.01	19,905	23,000	-13.5
	0.002	28,151	35,000	-19.6

1. Flows were estimated from the 1-day frequency curve.
2. Difference = [(current study – Comp Study) / Comp Study] × 100.

Conclusions

The flow-frequency curves that we developed for this study are based on calibration to an updated at-site flow-frequency curve for the Butte Creek near Chico streamgage. Here, we compared this study's results to those computed using USGS regression equations and those developed as part of the Comp Study. While results varied, the discrepancies are within ranges that we consider acceptable given the different analysis methodologies employed for each case.

Adoption of flow-frequency curves

We have adopted the flow-frequency curves and associated design storm hydrographs computed for this study. The adopted peak flow-frequency curves for the 3 analysis points are presented in Table 1.

In Figure 8, we show a peak flow rate per square mile (CSM) plot for CVHS ungaged watershed analysis points. Each point on the plot represents the computed peak p=0.01 flow rate resulting from our analyses. As can be seen, the 3 points associated with the Butte Creek watershed, which are essentially overlapping in the figure, fall within a general range indicated by the results from the other ungaged watersheds.

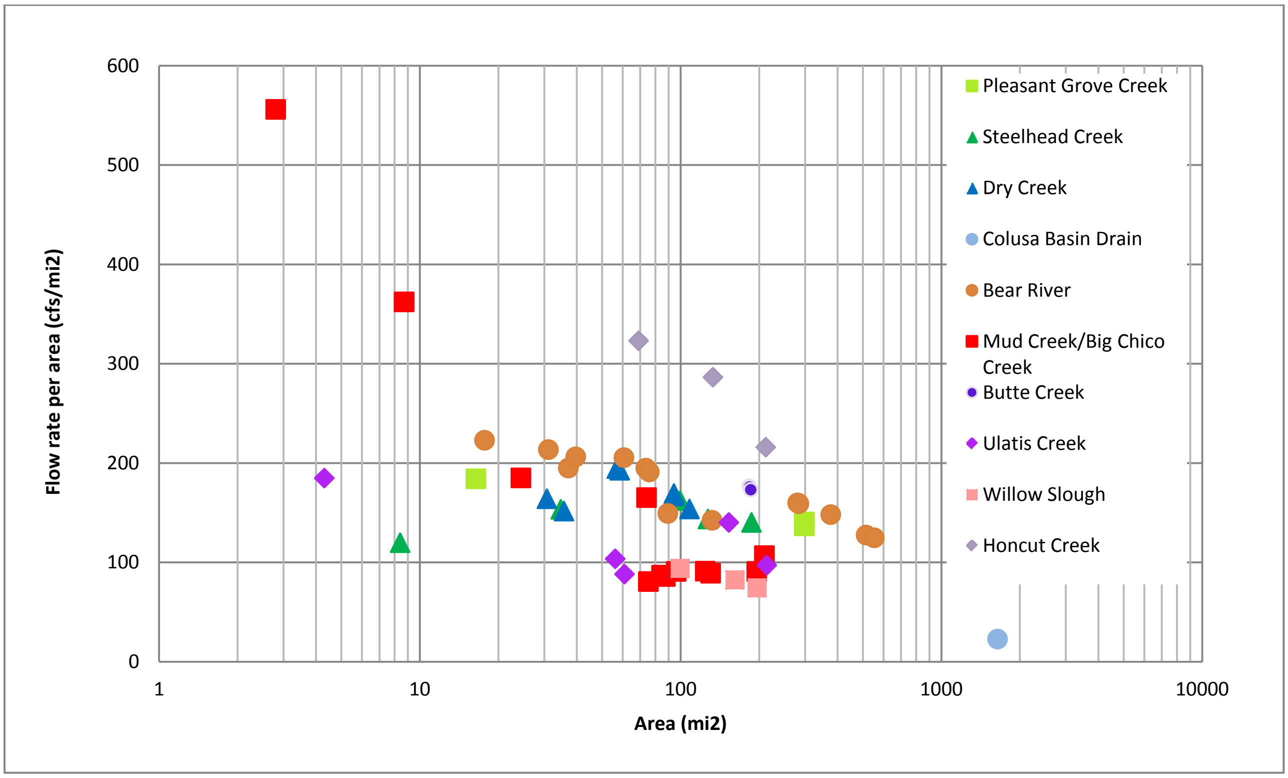

Figure 8. CSM plot for CVHS ungaged watersheds: Each point on the curve represents the peak p=0.01 flow at an analysis point in a CVHS ungaged watershed.

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Appendix I. Watershed delineation and properties

Watershed delineation

Details regarding delineation of the Butte Creek watershed were provided in the main body of this report. Figure 9 and Figure 10 show the delineation of the Butte Creek watershed model. Figure 12 through Figure 14 show screenshots of the HEC-HMS basin model.

Model parameters, transforms, and routings

The subbasin loss parameters, transform-related characteristics, and reach routing parameters are presented in Table 14, Table 15, and Table 16, respectively. The values listed in these tables are the initial values derived from the data sources and references described in the main body of this report. We adjusted some of these values during model calibration, if necessary, to improve the relationship between simulated and observed flows.

Since calibration of the routing reach parameters was not required, the parameter values in Table 15 and Table 16 are the final values used for simulating the design storms.

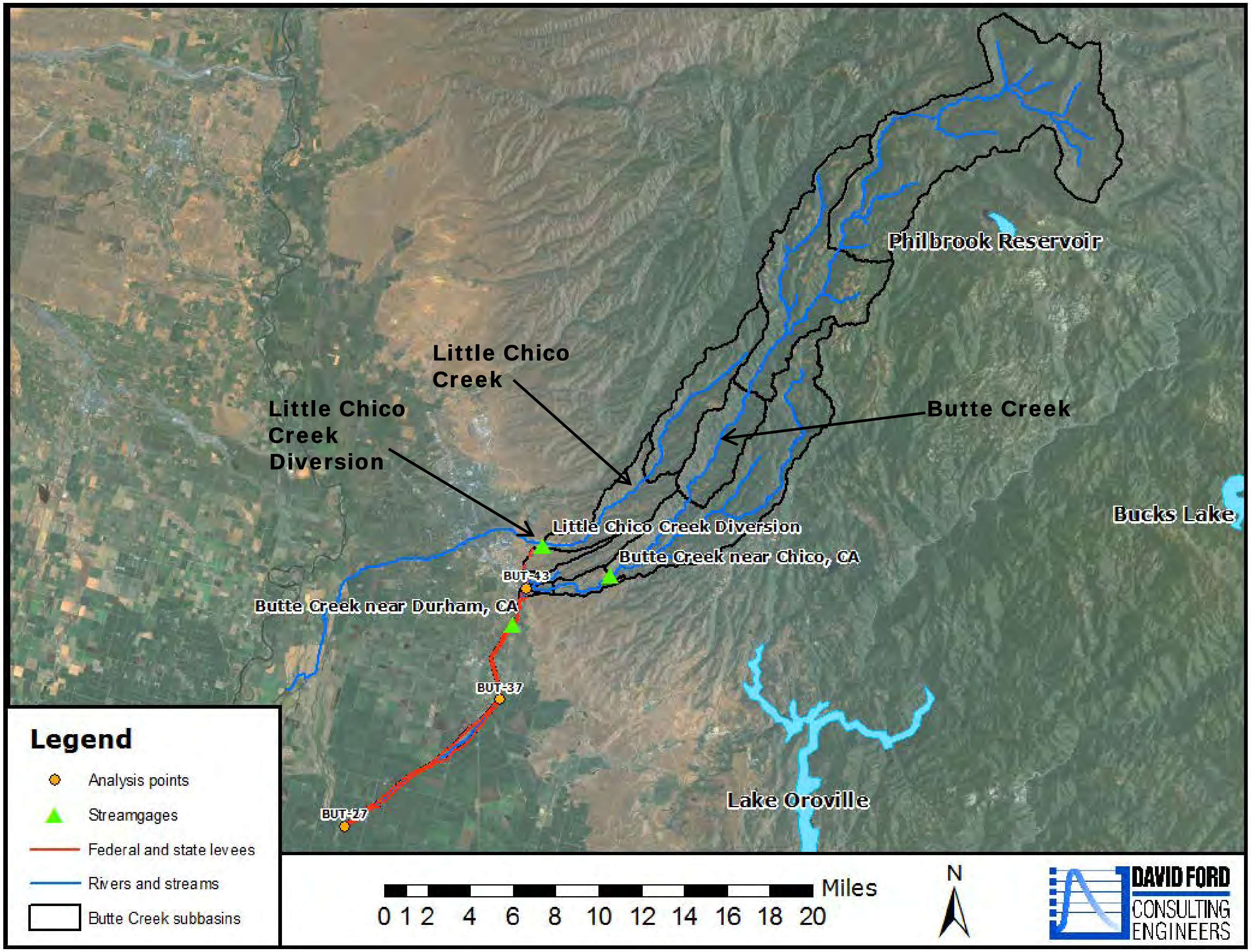

Figure 9. Butte Creek watershed delineation

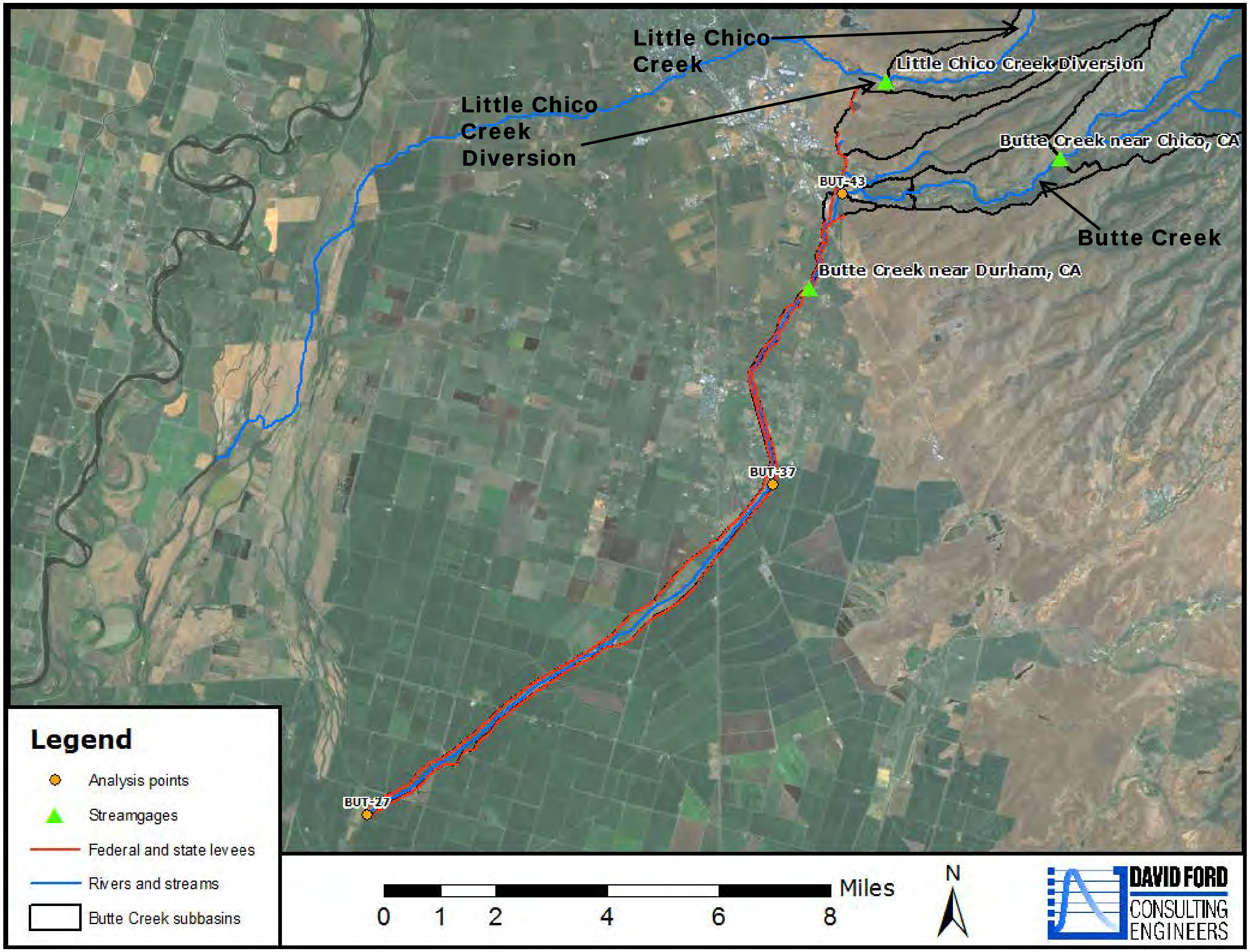

Figure 10. Butte Creek watershed delineation (zoomed to show subbasin delineations at the analysis points and streamgage)

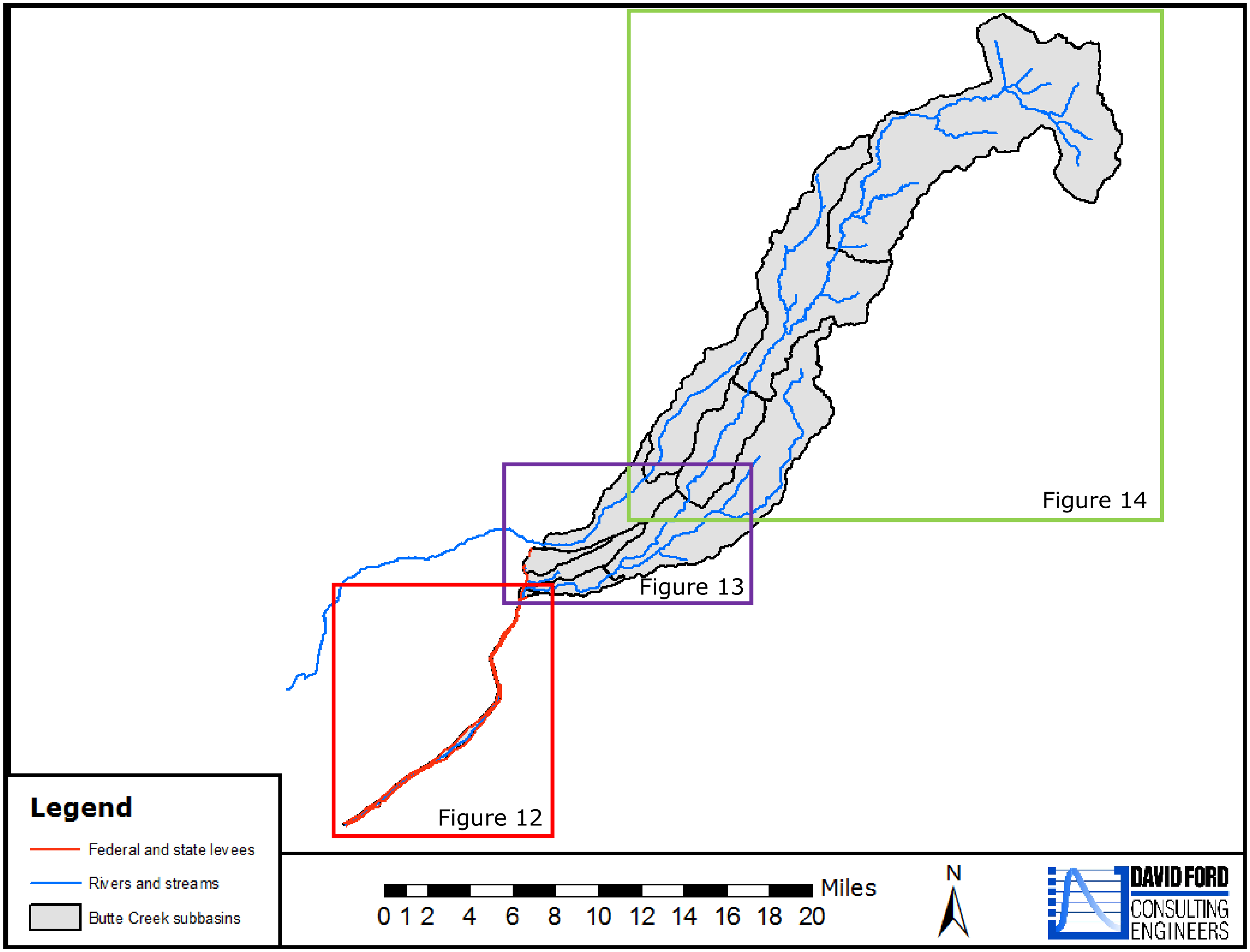

Figure 11. Model schematic for the Butte Creek watershed

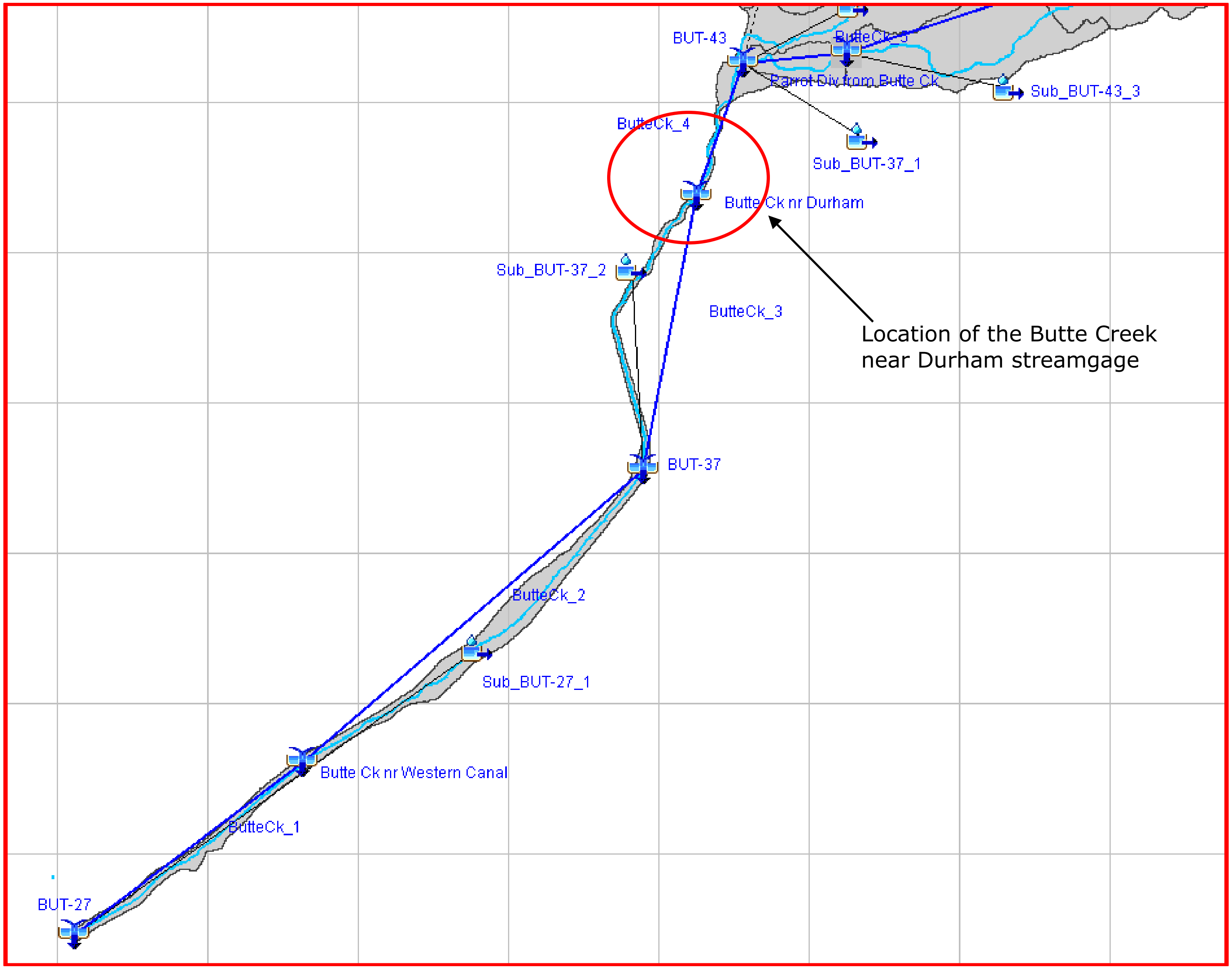

Figure 12. HEC-HMS model schematic (zoomed to the downstream portion of the watershed)

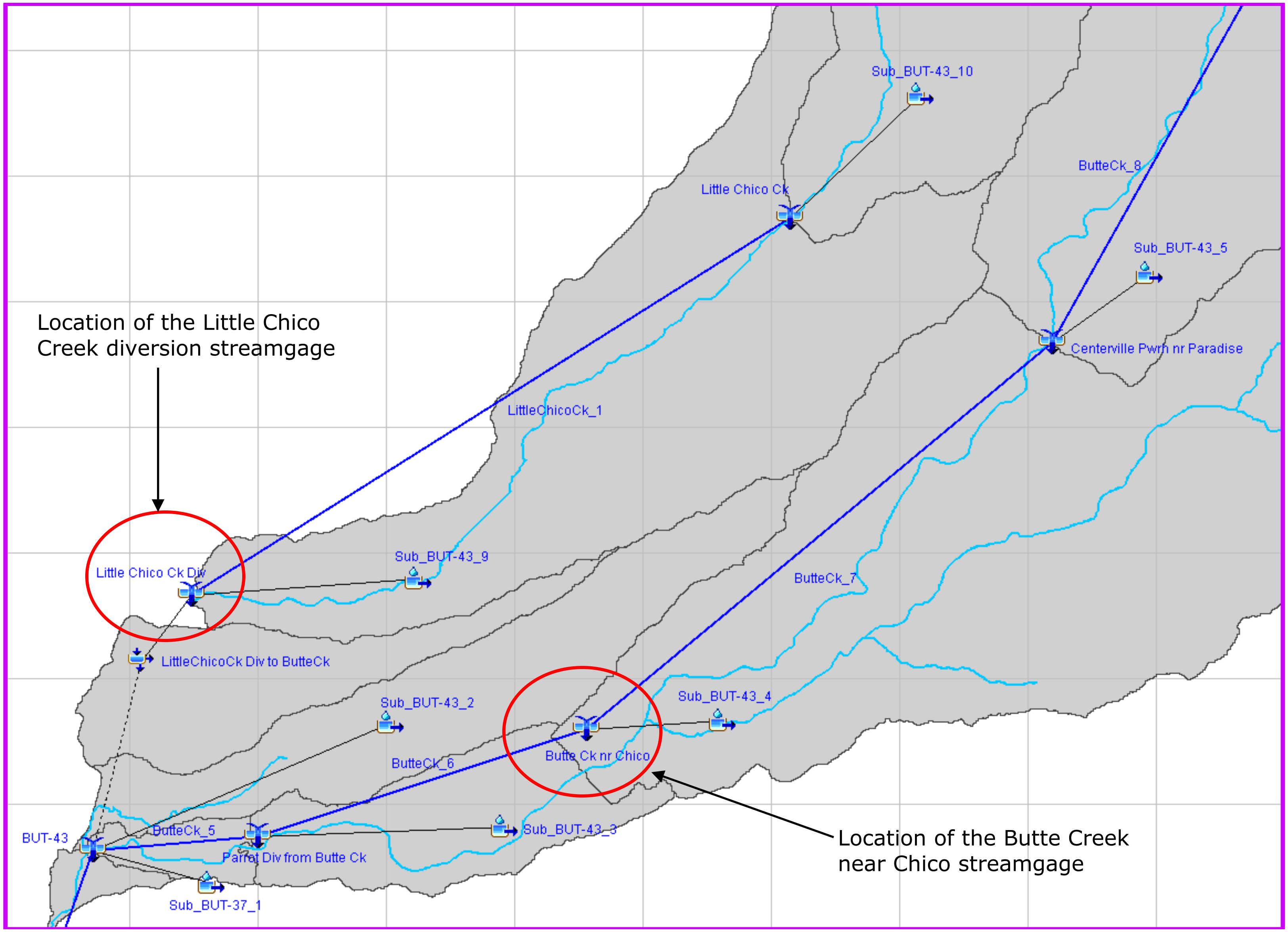

Figure 13. HEC-HMS model schematic (zoomed to the central portion of the watershed)

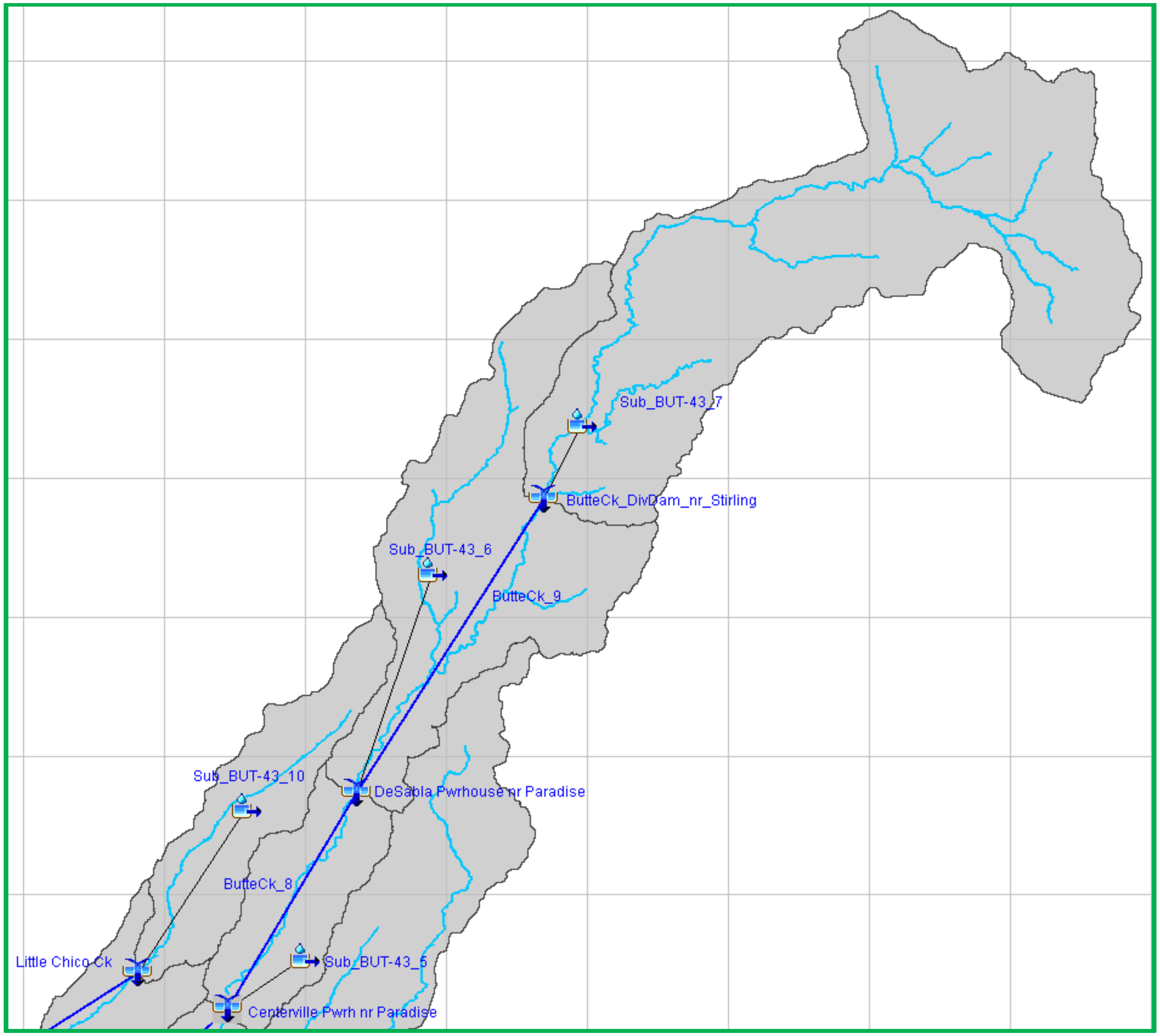

Figure 14. HEC-HMS model schematic (zoomed to the upstream portion of the watershed)

Table 14. Subbasin loss parameters: Expected constant loss rates were adjusted during model calibration. Those shown here are initial estimates only.

Subbasin (1)	**Subbasin area (mi^2) (2)**	**Impervious area (%) (3)**	**Soil type classification (4)**	**Expected constant loss rate[1] (in/hr) (5)**
Sub_BUT-27_1	1.86	0.44	D	0.025
Sub_BUT-37_1	0.37	13.85	D	0.025
Sub_BUT-37_2	0.31	0.66	B	0.225
Sub_BUT-43_1	0.50	6.10	D	0.025
Sub_BUT-43_10	14.79	0.45	B	0.225
Sub_BUT-43_2	4.00	0.48	D	0.025
Sub_BUT-43_3	2.47	0.89	D	0.025
Sub_BUT-43_4	35.74	1.58	B	0.225
Sub_BUT-43_5	12.09	0.17	B	0.225
Sub_BUT-43_6	34.22	0.20	B	0.225
Sub_BUT-43_7	66.14	0.17	B	0.225
Sub_BUT-43_8	2.46	0.14	D	0.025
Sub_BUT-43_9	10.71	0.47	D	0.025

1. Constant loss rates estimated using the *HEC-HMS technical reference manual* (HEC 2000).

Table 15. Subbasin transform-related characteristics: Expected basin lag times were adjusted during model calibration. Those shown here are initial estimates only.

Subbasin (1)	Predominant landcover / use (2)	Basin roughness coefficient[1] (3)	Longest flow path (mi) (4)	Centroidal flow path (mi) (5)	Overall basin slope (%) (6)	S-graph[2] (7)	Expected basin lag time (hr) (8)
Sub_BUT-27_1	Cultivated crops	0.115	10.31	5.45	1.05	valley	5.95
Sub_BUT-37_1	Herbaceous	0.115	2.49	1.51	2.27	valley	1.84
Sub_BUT-37_2	Cultivated crops	0.115	4.25	1.93	1.42	valley	2.70
Sub_BUT-43_1	Woody wetlands	0.12	1.74	0.88	6.24	foothill	1.12
Sub_BUT-43_10	Evergreen forest	0.12	11.77	5.54	25.77	mountain	3.58
Sub_BUT-43_2	Herbaceous	0.115	7.89	4.15	15.4	mountain	2.91
Sub_BUT-43_3	Herbaceous	0.115	4.42	2.22	19.44	mountain	1.76
Sub_BUT-43_4	Evergreen forest	0.12	22.02	10.04	23.96	mountain	5.77
Sub_BUT-43_5	Evergreen forest	0.12	7.90	3.64	42.04	mountain	2.39
Sub_BUT-43_6	Evergreen forest	0.12	17.19	7.81	30.96	mountain	4.54
Sub_BUT-43_7	Evergreen forest	0.12	24.76	14.44	23.96	mountain	6.92
Sub_BUT-43_8	Herbaceous	0.115	5.42	2.07	7.46	foothill	2.22
Sub_BUT-43_9	Herbaceous	0.115	9.29	5.06	23.57	mountain	3.08

1. Coefficients were estimated using the *Sacramento City/County drainage manual* (Sacramento County 2006).
2. Valley S-graphs were assigned when the basin slope was less than 200 ft/mi, foothill when the basin slope was between 200 and 400 ft/mi, and mountain when the basin slope was greater than 400 ft/mi.

Table 16. Hydrologic routing reach characteristics and parameters: We did not adjust hydrologic routing reach models during calibration.

Routing reach name (1)	Length (mi) (2)	Slope (ft/ft) (3)	Shape (4)	Bottom width (5)	Side slope (H:V) (6)	Manning's *n*-value (7)
ButteCk_1	3.81	0.0006	trapezoid	66	54	0.035
ButteCk_2	6.05	0.0012	trapezoid	98	38	0.035
ButteCk_3	4.22	0.0018	trapezoid	220	35	0.035
ButteCk_4	2.02	0.0041	trapezoid	187	30	0.04
ButteCk_5	1.42	0.0057	trapezoid	1641	4	0.04
ButteCk_6	3.55	0.0034	trapezoid	1641	3	0.04
ButteCk_7	6.24	0.0055	triangle	-	3	0.04
ButteCk_8	7.02	0.0195	triangle	-	2	0.04
ButteCk_9	10.15	0.0282	triangle	-	3	0.05
LittleChicoCk_1	6.81	0.015	trapezoid	-	4	0.07

Appendix II. HEC-HMS model calibration to historical events

Historical calibration simulations

We calibrated the Butte Creek watershed model to the 3 events listed in Table 7. Figure 4 shows the locations of the precipitation gages used to calibrate the watershed model, in relation to the delineated watershed. We calibrated the model to observed streamflow data from the Butte Creek near Chico, Butte Creek near Durham, and the Little Chico Creek diversion at Chico gages. Figure 15 through Figure 22 show the calibration results for each event. In each figure, the blue line represents the simulation results at the model time step (15 min) and the red line represents the appropriate observed flow time series. For several events, high flows were observed over a 2-day period. For these events, the model was calibrated to match the 2-day volume instead of the peak. Summary tables of the calibration results are presented in Table 17, Table 18, and Table 19.

For each of the calibration events, we changed the basin lag times and constant loss rates and applied a recession baseflow model to reconstitute the observed streamgage hydrographs. For the 3 calibration events, we:

- Increased the initial estimates of basin lag times.
- Decreased the initial estimates of constant loss rates.
- Applied baseflow using the recession baseflow model in HEC-HMS. We calculated initial baseflow by distributing the initial event flow observed at the streamgages to the upstream contributing subbasins. We specified a baseflow recession constant and baseflow ratio-to-peak threshold.

Table 17. Calibration summary at the Butte Creek near Chico gage

Event (1)	Property[1] (2)	Simulated (3)	Observed (4)	Difference (%)[3] (5)
1997	Peak flow (cfs)	35,678	35,600[2]	0.2
	Volume (in)[1]	19.99	22.36	-10.6
1998	Peak flow (cfs)	12,372	10,400[2]	19.0
	Volume (in)[1]	16.20	18.05	-10.2
2006	Peak flow (cfs)	12,719	14,800[2]	-14.1
	Volume (in)[1]	16.67	16.24	2.6

1. The volume is the average runoff depth above the Butte Creek near Chico gage.
2. USGS reported peak flow rate.
3. Difference = (simulated − observed) / observed × 100.

Table 18. Calibration summary at the Butte Creek near Durham gage

Event (1)	Property[1] (2)	Simulated (3)	Observed (4)	Difference (%)[2] (5)
1997	Peak flow (cfs)	39,042	37,021	5.5
	Volume (in)[1]	19.59	18.93	3.5
1998	Peak flow (cfs)	13,619	9,788	39.1
	Volume (in)[1]	17.31	17.03	1.6
2006	Peak flow (cfs)	14,311	18,358	-22.0
	Volume (in)[1]	17.02	17.77	-4.2

1. The volume is the average runoff depth above the Butte Creek near Durham gage.
2. Difference = (simulated − observed) / observed × 100.

Table 19. Calibration summary at the Little Chico Creek diversion at Chico gage

Event (1)	Property[1] (2)	Simulated (3)	Observed (4)	Difference (%)[2] (5)
1997	Peak flow (cfs)	1,419	1,410	0.61
	Volume (in)[1]	1.09	1.99	-45.0
1998	Peak flow (cfs)	1,510	751	101.0
	Volume (in)[1]	1.2	1.79	-33.0
2006	Peak flow (cfs)	993[3]	993	0.0
	Volume (in)[1]	2.61	2.16	21.0

1. The volume is the average runoff depth above the Little Chico Creek Diversion at Chico gage.
2. Difference = (simulated − observed) / observed × 100.
3. Value is the peak of the second wave

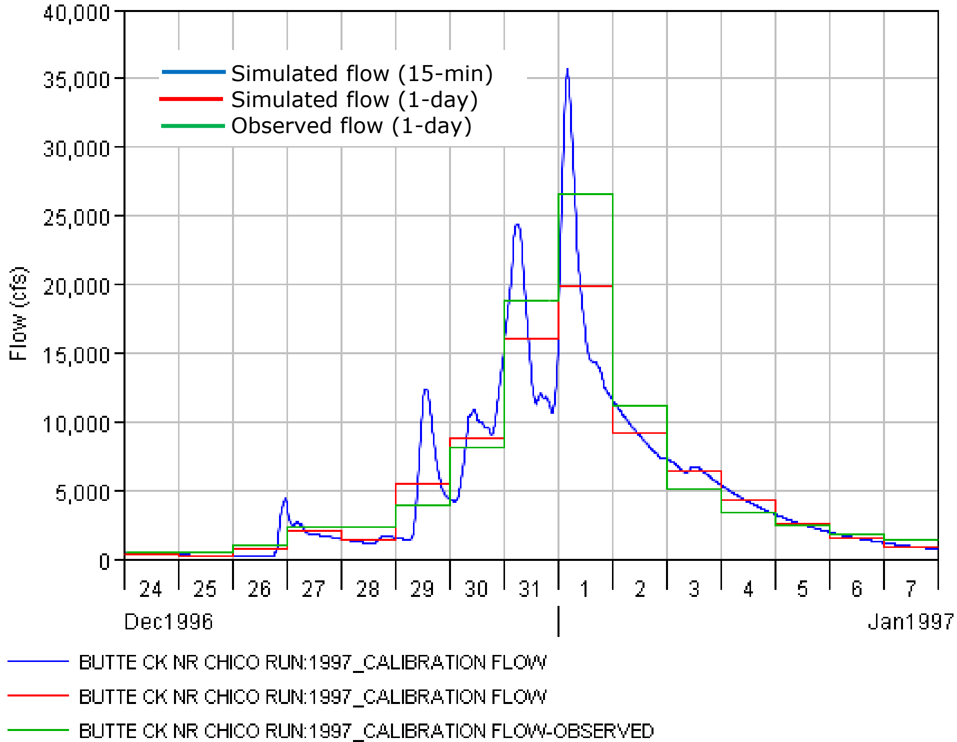

Figure 15. 1997 calibration results for the Butte Creek near Chico gage

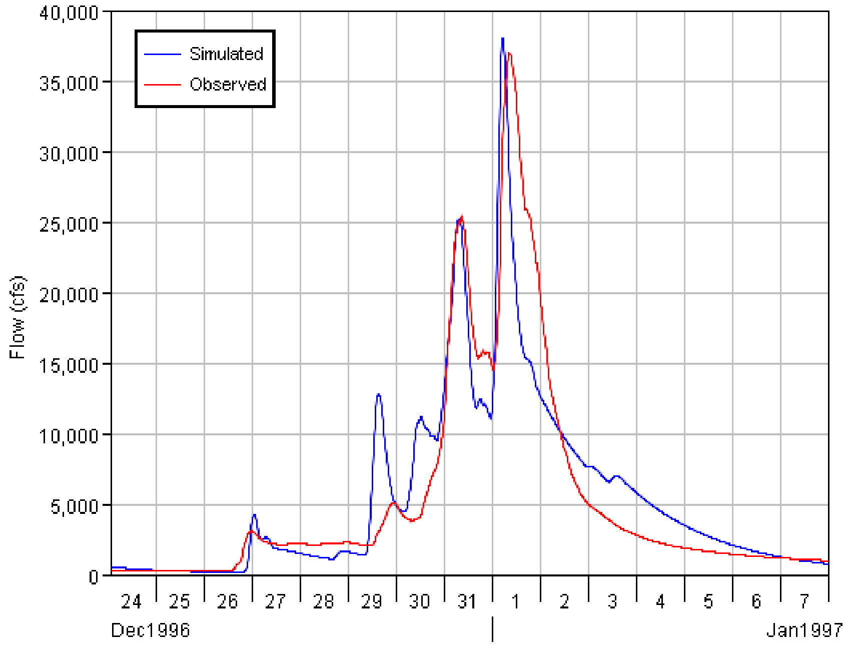

Figure 16. 1997 calibration results for the Butte Creek near Durham gage

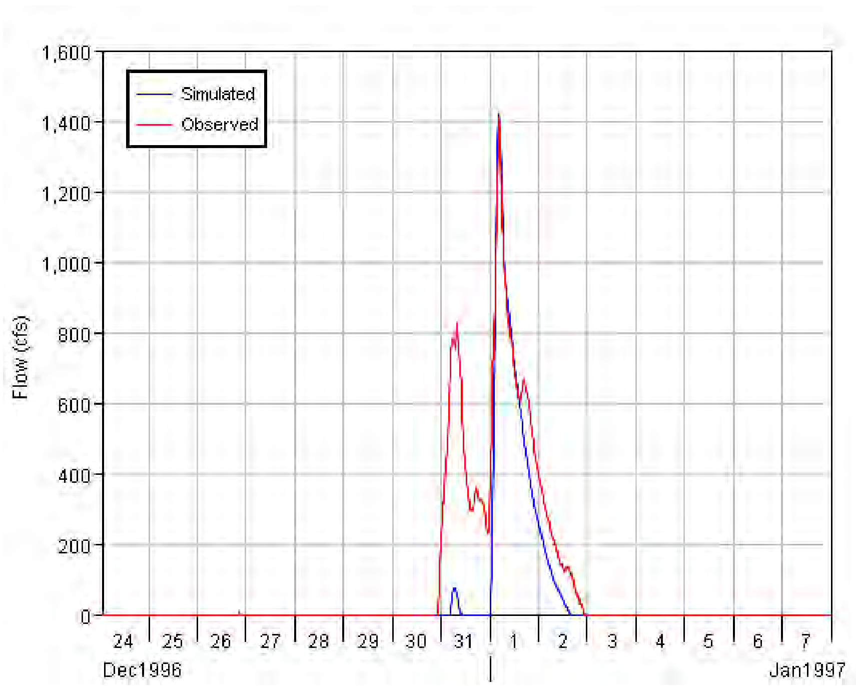

Figure 17. 1997 calibration results for the Little Chico Creek diversion gage

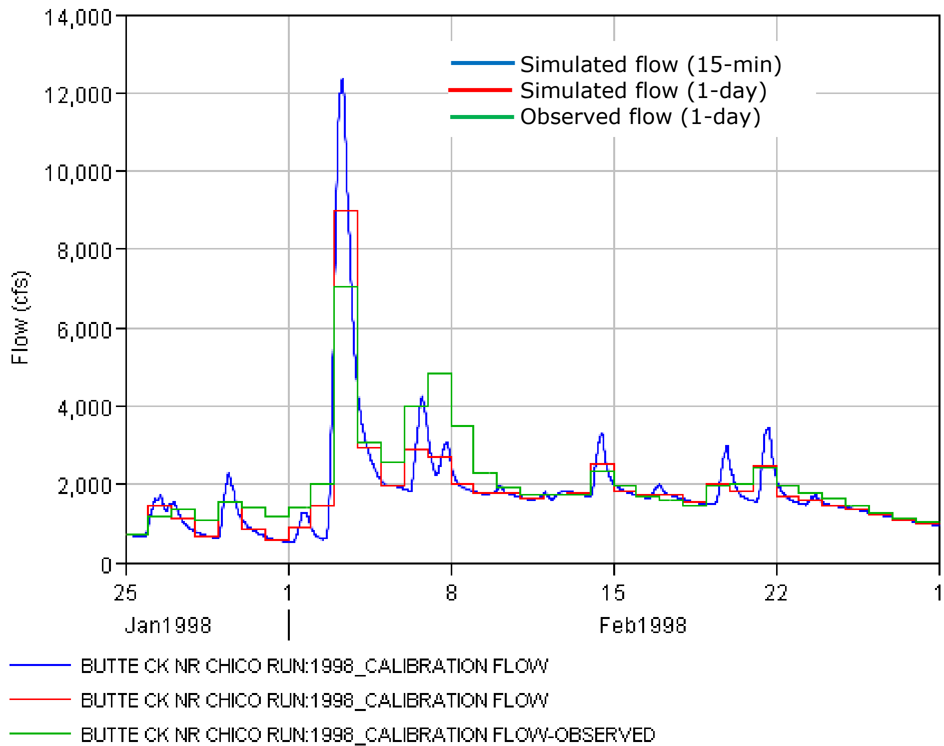

Figure 18. 1998 calibration results for the Butte Creek near Chico gage

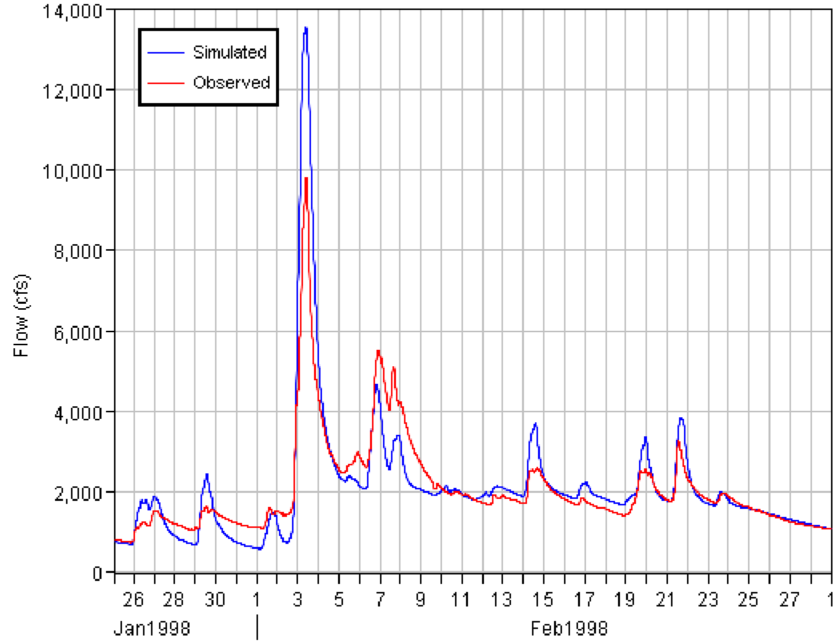

Figure 19. 1998 calibration results for the Butte Creek near Durham gage

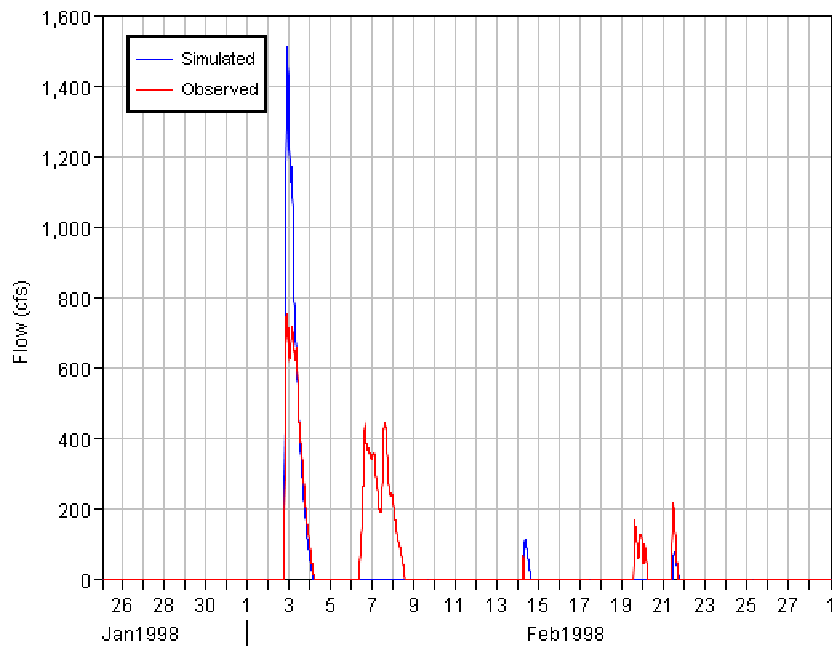

Figure 20. 1998 calibration results for the Little Chico Creek diversion gage

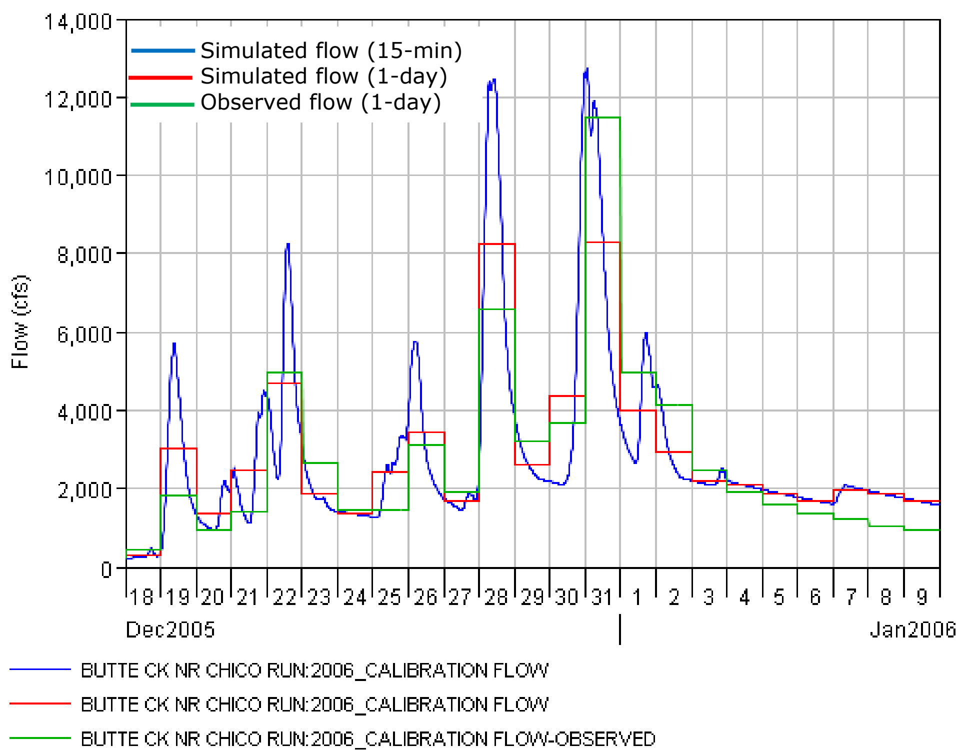

Figure 21. 2006 calibration results for the Butte Creek near Chico gage

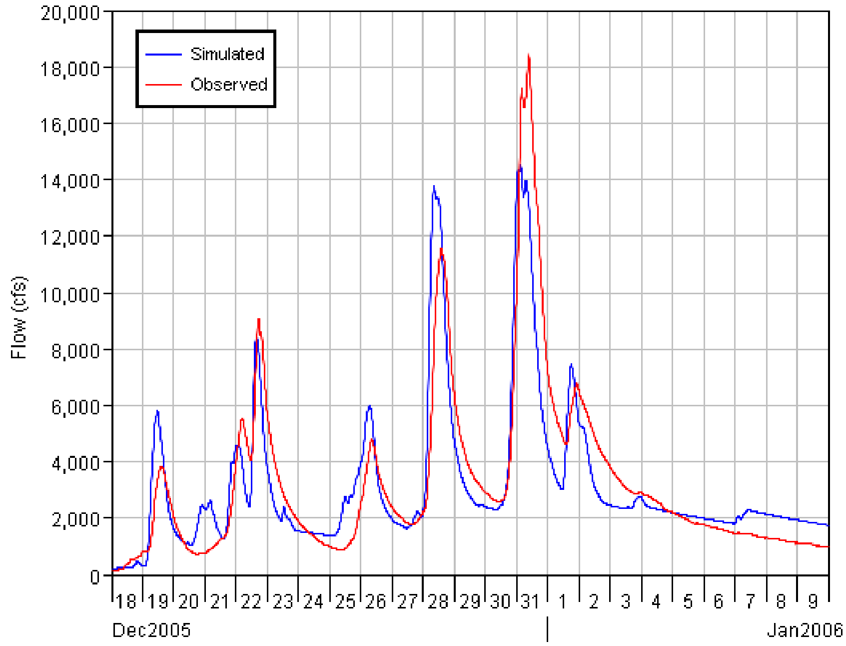

Figure 22. 2006 calibration results for the Butte Creek near Durham gage

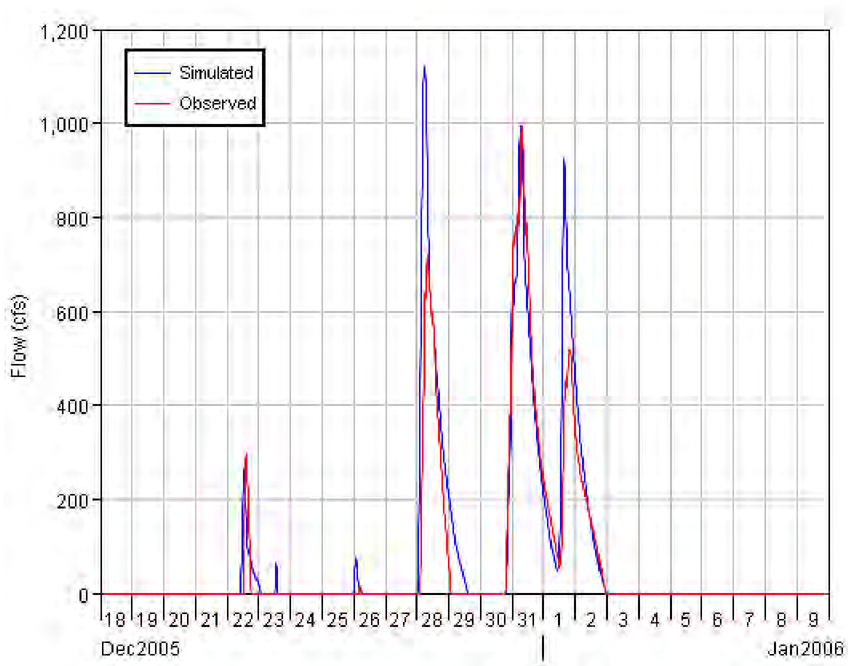

Figure 23. 2006 calibration results for the Little Chico Creek diversion gage

Historical calibration result summary

The calibration results show that our model is able to reconstitute the observed hydrographs at the Butte Creek near Chico and Butte Creek near Durham streamgages. Calibrating to the Little Chico Creek diversion gage proved challenging. For the 1997 and 1998 calibrations, not all the observed waves could be simulated; however, parameter adjustments were done to match the peak, volume, and timing of the largest wave. The following describes the calibrating process for Little Chico Creek and Butte Creek.

The final basin lag times, provisional constant loss rates, and final baseflow characteristics are presented in Table 20 through Table 24. (The constant loss rates are provisional because we later adjusted them to simulate flows consistent with the at-site flow-frequency curves for the Butte Creek near Chico streamgage.) To develop the final basin lag times, provisional constant loss rates, and final baseflow characteristics, we calculated a weighted average of the event-specific basin lag times, constant loss rates, and baseflow characteristics for the 1997, 1998, and 2006 events. We placed 40 percent weight on each of the 1997 and 2006 events and 20 percent weight on the 1998 event. We weighted them this way because we were able to reconstitute the observed hydrographs for the 1997 and 2006 events better than the 1998 event. Little Chico Creek was weighted separately from Butte Creek since Little Chico Creek was calibrated to the Little Chico Creek diversion gage. Since the 1998 event did not calibrate well with observed flow at the diversion gage, the final basin lag times along Chico Creek was weighted equally between the 1997 and 2006 events. Although the 1998 simulated flow captures the rising and falling limb of the observed hydrograph, the simulation significantly overestimates the hydrographs' observed peak and volume. It's likely the rainfall gage may not have adequately captured the precipitation pattern and/or intensity. Therefore, 1998 calibration did not provide useful information and calibrated values were not incorporated into final results for all other calibrated parameters (loss rates, baseflow, etc.).

The final basin lag times presented in Table 20 are approximately 70 percent greater than the original basin lag time estimates. Increasing the basin lag times was necessary to match the hydrograph attenuation observed in the streamgage data. Following calibration, we back-calculated basin roughness coefficients from the increased basin lag times. The back-calculated average basin roughness coefficient was 0.19, which is within the range of acceptable roughness coefficients for overland flow of natural surfaces according to *Hydrology and floodplain analysis* (Bedient et al. 2008). In general, all 6 CVHS ungaged watersheds required increases to basin lag times during HEC-HMS model calibration.

The provisional constant loss rates presented in Table 21 are approximately 35 percent to 95 percent of the original loss rates derived from subbasin hydrologic soil group classifications. Later, to match the at-site flow-frequency curves for the Butte Creek near Chico streamgage, we scaled the provisional constant loss rates by a factor of 2. The final constant loss rates resulting from this scaling fall within the expected ranges indicated by hydrologic soil group classifications. Constant loss rates for Little Chico Creek were not scaled by 2. The upper half of Little Chico Creek contains mainly B soils, while the lower half (Sub_BUT-43_9) contains B and D type. Since both B and D are dominate in the lower half of Little Chico, soil loss rates averaged to a C type soil.

We calculated initial baseflow by distributing the initial event flow observed at the streamgages to the upstream contributing subbasins. The final initial baseflow averaged 39.4 cfs, the final ratio-to-peak threshold value averaged 0.23, and the final recession constant averaged 0.79. Little Chico Creek final peak-to-ratio threshold value averaged 0.775, and the final baseflow recession constant averaged 0.50. The initial baseflow is very small compared to peak flows for each calibration event, and we selected acceptable values of recession constants for surface runoff or interflow according to the *HEC-HMS technical reference manual* (HEC 2000). Baseflow is a small component of the total runoff hydrographs produced by the Butte Creek watershed.

Using the final basin lag times, the provisional constant loss rates, the final ratio-to-peak thresholds, and the final baseflow recession constants, we reran the simulations for each event. Results for these verification simulations are shown in Figure 24 through Figure 32. These figures suggest that, in general, the HEC-HMS basin model calibrated to historical events can reliably simulate the approximate shapes of the historical event hydrographs in the Butte Creek watershed. Nonetheless, significant discrepancies between the magnitudes of simulated and observed hydrographs provided support for our strategy to adjust further the provisional constant loss rates in an attempt to match better the at-site flow-frequency curves for the Butte Creek near Chico streamgage location.

Table 20. Event-specific and final basin lag times: The final basin lag times were determined by taking a weighted average of the event-specific basin lag times. We used the final basin lag times for all design storm simulations.

Subbasin (1)	1997 Basin lag time (hr) (2)	1998 Basin lag time (hr) (3)	2006 Basin lag time (hr) (4)	Final basin lag time (hr) (5)
Sub_BUT-27_1	8.92	17.84	7.73	10.23
Sub_BUT-37_1	2.76	5.52	2.39	3.17
Sub_BUT-37_2	4.06	8.11	3.52	4.65
Sub_BUT-43_1	1.68	3.37	1.46	1.93
Sub_BUT-43_10	6.09	5.37	5.73	5.91
Sub_BUT-43_2	4.36	8.7	3.78	5.00
Sub_BUT-43_3	2.64	5.3	2.29	3.03
Sub_BUT-43_4	5.19	17.3	7.50	8.54
Sub_BUT-43_5	2.15	7.2	3.10	3.53
Sub_BUT-43_6	4.09	13.6	5.91	6.72
Sub_BUT-43_7	6.23	20.8	9.00	10.25
Sub_BUT-43_8	3.33	6.7	2.89	3.82
Sub_BUT-43_9	5.24	4.62	4.93	5.08

Table 21. Event-specific and provisional constant loss rates: The provisional constant loss rates were determined by taking a weighted average of the event-specific constant loss rates. We scaled the provisional constant loss rates by 2 to develop the final constant loss rates used for all design storm simulations.

Subbasin (1)	**1997 constant loss rates (in/hr) (2)**	**1998 constant loss rates (in/hr) (3)**	**2006 constant loss rates (in/hr) (4)**	**Provisional constant loss rates (in/hr) (5)**
Sub_BUT-27_1	0.02	0.025	0.02	0.021
Sub_BUT-37_1	0.02	0.025	0.02	0.021
Sub_BUT-37_2	0.02	0.225	0.02	0.061
Sub_BUT-43_1	0.02	0.025	0.02	0.021
Sub_BUT-43_10	0.225	0.225	0.16	0.192
Sub_BUT-43_2	0.02	0.025	0.025	0.023
Sub_BUT-43_3	0.02	0.025	0.025	0.023
Sub_BUT-43_4	0.022	0.12	0.12	0.08
Sub_BUT-43_5	0.022	0.12	0.12	0.08
Sub_BUT-43_6	0.022	0.12	0.12	0.08
Sub_BUT-43_7	0.022	0.12	0.12	0.08
Sub_BUT-43_8	0.02	0.025	0.025	0.023
Sub_BUT-43_9	0.10	0.10	0.071	0.086

*Little Chico Creek constant loss rates were not scaled by 2.

Table 22. Event-specific and final initial baseflow values: The final initial baseflow values were determined by taking a weighted average of the event-specific initial baseflow values. We used the final initial baseflow values for all design storm simulations, but used event-specific initial baseflows for the verification simulations.

Subbasin (1)	**1997 initial baseflow (cfs) (2)**	**1998 initial baseflow (cfs) (3)**	**2006 initial baseflow (cfs) (4)**	**Final initial baseflow (cfs) (5)**
Sub_BUT-27_1	5.86	8.91	2.51	5.13
Sub_BUT-37_1	1.17	1.77	0.50	1.02
Sub_BUT-37_2	0.98	1.49	0.42	0.86
Sub_BUT-43_1	1.58	2.40	0.67	1.38
Sub_BUT-43_10	46.61	70.86	19.96	40.80
Sub_BUT-43_2	12.61	19.16	5.40	11.03
Sub_BUT-43_3	7.78	11.83	3.33	6.81
Sub_BUT-43_4	112.63	171.24	48.24	98.59
Sub_BUT-43_5	38.10	57.92	16.32	33.35
Sub_BUT-43_6	107.84	163.95	46.18	94.40
Sub_BUT-43_7	208.43	316.89	89.26	182.46
Sub_BUT-43_8	7.75	11.79	3.32	6.79
Sub_BUT-43_9	33.75	51.31	14.45	29.54

Table 23. Event-specific and final baseflow recession constants: The final baseflow recession constants were determined by taking a weighted average of the event-specific baseflow recession constants. We used the final baseflow recession constants for all design storm simulations.

Subbasin (1)	1997 recession constants (2)	1998 recession constants (3)	2006 recession constants (4)	Final recession constants (5)
Sub_BUT-27_1	0.60	0.90	0.90	0.78
Sub_BUT-37_1	0.60	0.90	0.90	0.78
Sub_BUT-37_2	0.60	0.90	0.90	0.78
Sub_BUT-43_1	0.60	0.90	0.90	0.78
Sub_BUT-43_10	0.50	0.40	0.50	0.50
Sub_BUT-43_2	0.60	0.95	0.95	0.81
Sub_BUT-43_3	0.60	0.90	0.95	0.80
Sub_BUT-43_4	0.60	0.90	0.90	0.78
Sub_BUT-43_5	0.60	0.90	0.90	0.78
Sub_BUT-43_6	0.60	0.90	0.90	0.78
Sub_BUT-43_7	0.60	0.90	0.90	0.78
Sub_BUT-43_8	0.60	0.95	0.95	0.81
Sub_BUT-43_9	0.45	0.35	0.50	0.475

Table 24. Event-specific and final ratio-to-peak thresholds: The final ratio-to-peak thresholds were determined by taking a weighted average of the event-specific ratio-to-peak thresholds. We used the final ratio-to-peak thresholds for all design storm simulations.

Subbasin (1)	**1997 ratio-to-peak thresholds (2)**	**1998 ratio-to-peak threshold (3)**	**2006 ratio-to-peak thresholds (4)**	**Final ratio-to-peak thresholds (5)**
Sub_BUT-27_1	0.35	0.15	0.15	0.23
Sub_BUT-37_1	0.35	0.15	0.15	0.23
Sub_BUT-37_2	0.35	0.15	0.15	0.23
Sub_BUT-43_1	0.35	0.15	0.15	0.23
Sub_BUT-43_10	0.80	0.70	0.75	0.775
Sub_BUT-43_2	0.35	0.15	0.15	0.23
Sub_BUT-43_3	0.35	0.15	0.15	0.23
Sub_BUT-43_4	0.35	0.15	0.15	0.23
Sub_BUT-43_5	0.35	0.15	0.15	0.23
Sub_BUT-43_6	0.35	0.15	0.15	0.23
Sub_BUT-43_7	0.35	0.15	0.15	0.23
Sub_BUT-43_8	0.35	0.15	0.15	0.23
Sub_BUT-43_9	0.80	0.70	0.75	0.775

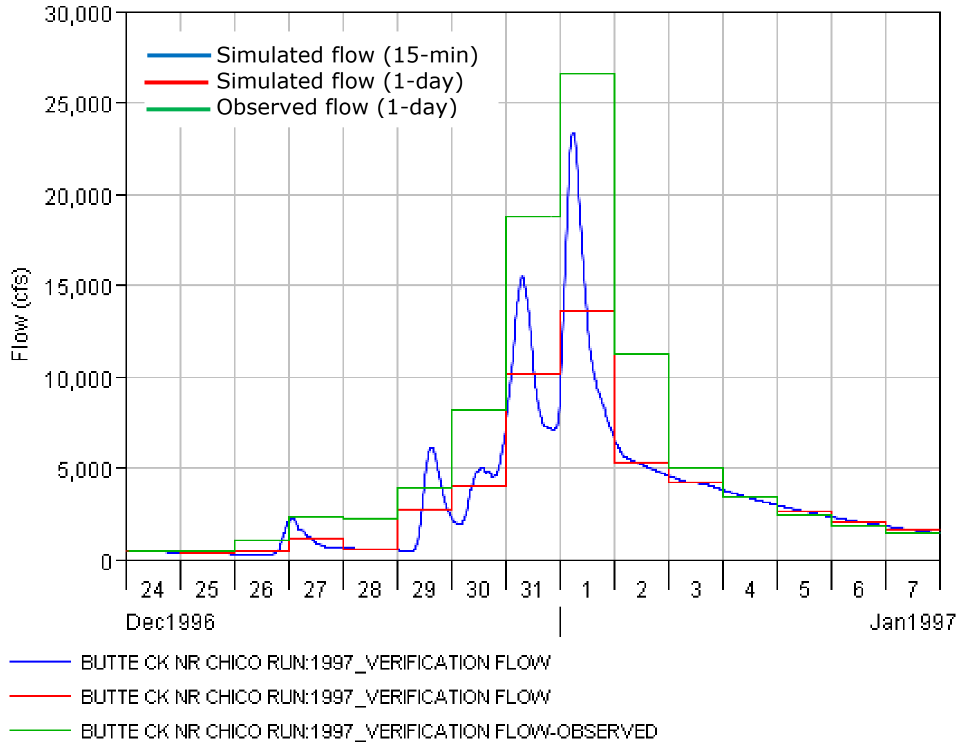

Figure 24. 1997 verification simulation results for the Butte Creek near Chico gage

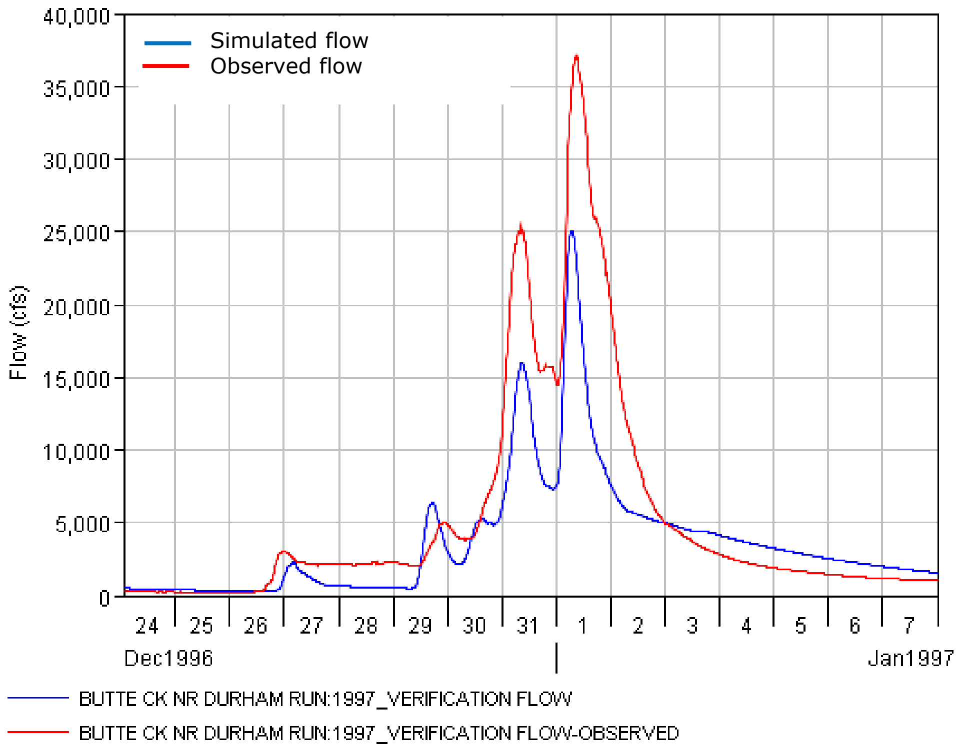

Figure 25. 1997 verification simulation results for the Butte Creek near Durham gage

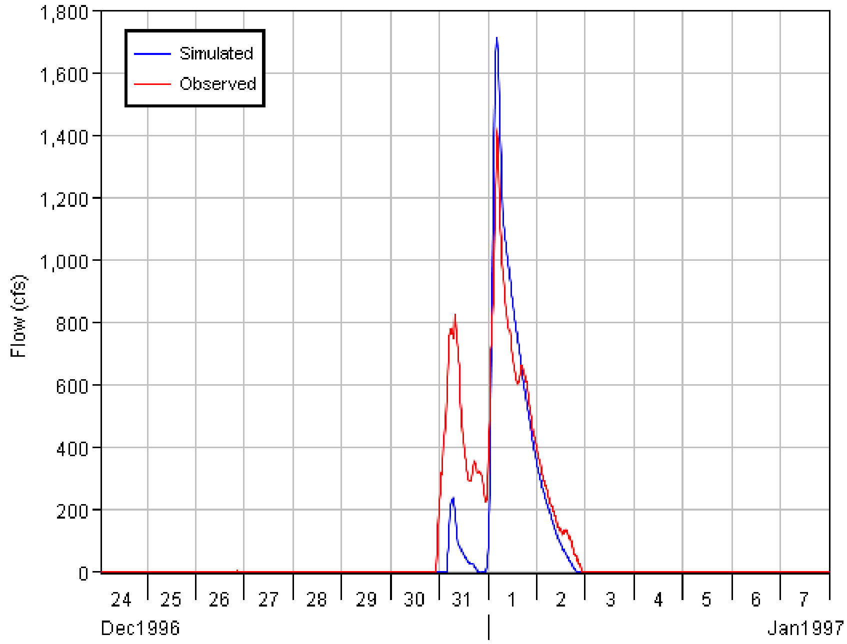

Figure 26. 1997 verification simulation results for the Little Chico Creek diversion gage

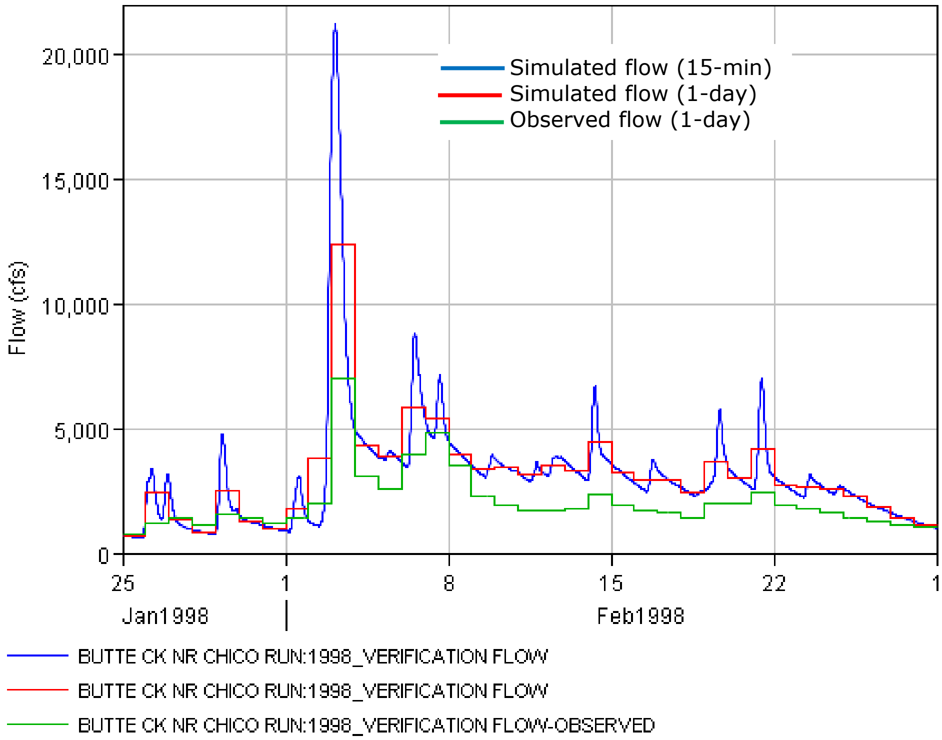

Figure 27. 1998 verification simulation results for the Butte Creek near Chico gage

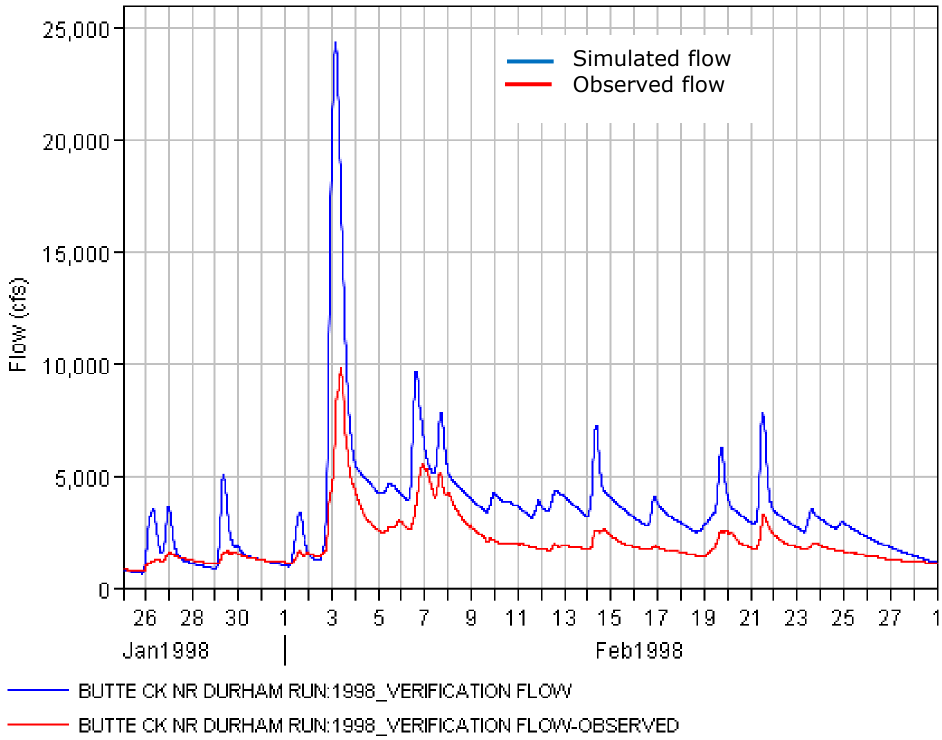

Figure 28. 1998 verification simulation results for the Butte Creek near Durham gage

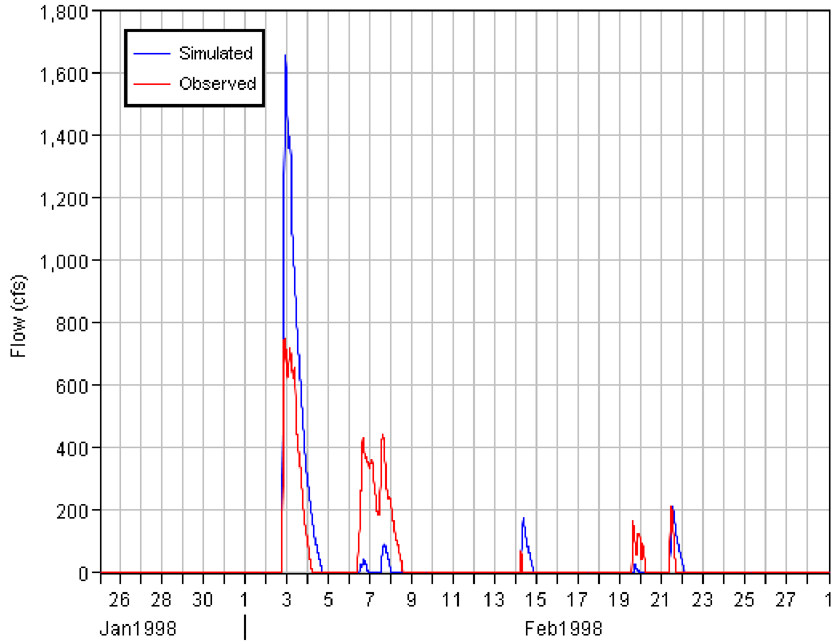

Figure 29. 1998 verification simulation results for the Little Chico Creek diversion gage

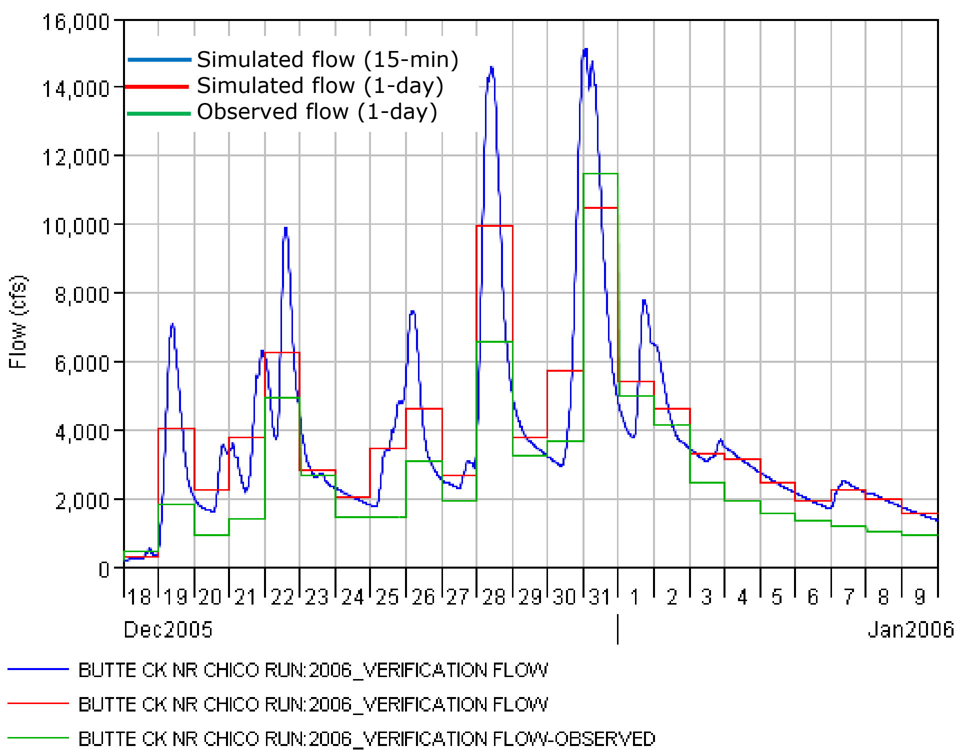

Figure 30. 2006 verification simulation results for the Butte Creek near Chico gage

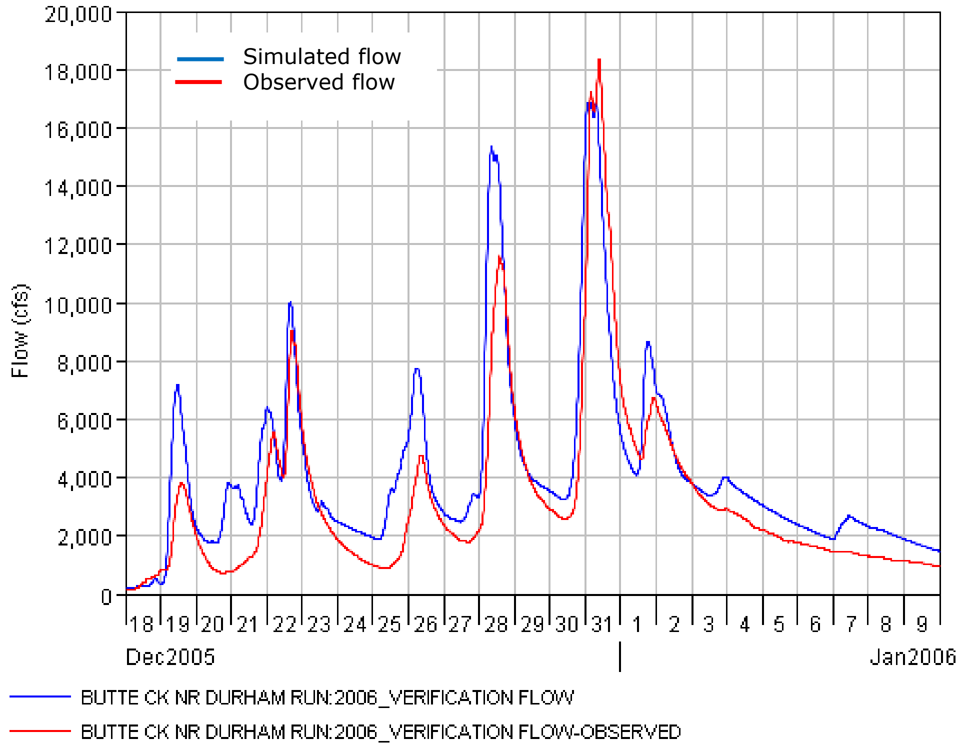

Figure 31. 2006 verification simulation results for the Butte Creek near Durham gage

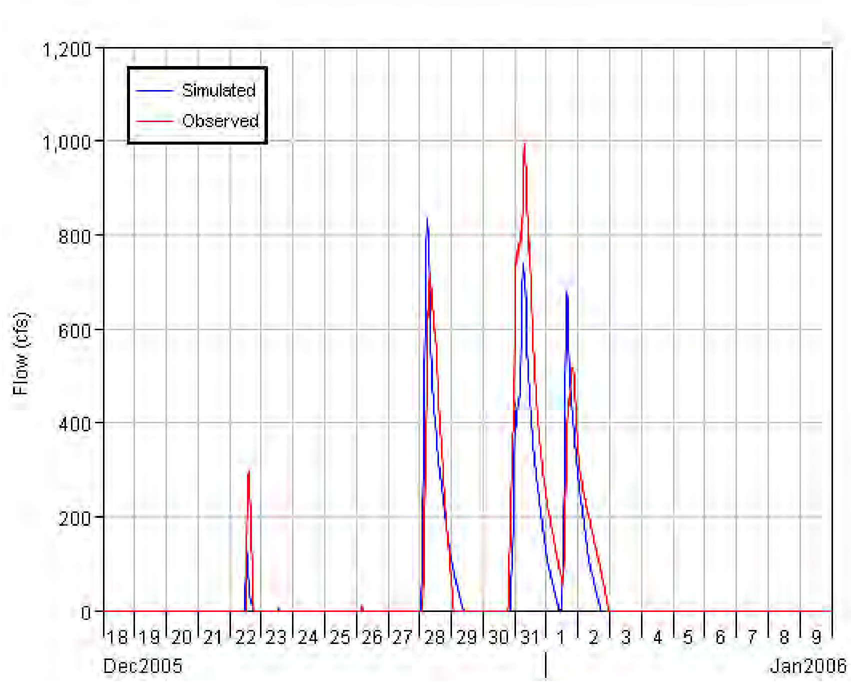

Figure 32. 2006 verification simulation results for the Little Chico Creek diversion gage

Appendix III. Guide to accompanying CD

The accompanying CD contains 3 HEC-HMS models. The folder named "Butte_Creek" contains the HEC-HMS model used for model calibration and verification. The contents and naming conventions for this model are described in Table 25.

The folder named "Butte_Creek_Design_Storms_Preprocessing" contains the HEC-HMS model used to develop the balanced hyetographs. In all, 35 balanced hyetographs were created for the watershed. The contents and naming conventions for this model are described in Table 26.

The folder named "ButteCreek_DesignStorms" contains the HEC-HMS model used for simulating the design storms. The contents and naming conventions for this model are described in Table 27.

All model output from the design storm simulations is stored in the HEC-DSS file Butte_Watershed.dss. The flow hydrographs at the analysis points have been copied to the HEC-DSS file Butte_Ck_Flows_at_Analysis_Points_Final. In Table 28, we list the HEC-DSS pathname for obtaining the flow at each analysis point. In column 2 of Table 28, we specify the appropriate storm centering for each analysis point. Results are only valid at each analysis point for the storm centering listed in column 2. The storm centering used for each simulation is found in the F-part, along with the frequency.

Table 25. Contents and naming conventions for the Butte_Creek HEC-HMS model used for calibration and verification

Property (1)	Naming convention (2)	Information (3)
Basin models	ButteCr_LittleChicoCk_[year]_[description]	The HEC-HMS model contains 6 basin models: 3 basin models each with calibrated parameters for each event and 3 basin models each with the weighted average model parameters for verification of each event. The 3 basin models for verification have event-specific initial baseflows and initial losses.
Meteorological models	ID2W_[year]	The HEC-HMS model contains 4 meteorological models: 1 model for each of the 3 calibration events and 1 base model that contains the index value for every gage.
Precipitation gages	[CDEC_ID]	4 precipitation gages are configured in the model. The precipitation gages are named according to their 3-letter CDEC identifier.
GIS data	---	Shapefiles of analysis points, subbasins, and streams are in the "maps" folder located within the HEC-HMS project folder.

Table 26. Contents and naming conventions for the Butte_Creek_Design_Storms_Preprocessing HEC-HMS model used for developing the balanced hyetographs

Property (1)	Naming convention (2)	Information (3)
HEC-HMS models	DesignStorm.hms	There are 7 HEC-HMS models: 1 storm centering and 7 zones.
Basin models	Basin 1	1 basin model is configured in each of the 7 HEC-HMS models. Each basin model has 1 subbasin named after the zone the HEC-HMS model represents.
Meteorological models	Met [frequency]	5 meteorological models are configured in each HEC-HMS model: 1 meteorological model for each of the 5 frequencies.
Simulation runs[1]	[frequency]_C=[storm centering]	5 simulation runs are configured in each of the 7 HEC-HMS models: 1 simulation run for each frequency. In all, we configured 35 simulation runs to compute the 35 balanced hyetographs: 1 centering × 7 zones × 5 frequencies.

Table 27. Contents and naming conventions for the ButteCreek_DesignStorms HEC-HMS model used for simulating the design storms

Property (1)	Naming convention (2)	Information (3)
Basin models	ButteCr_LittleChicoCk _[frequency]	The HEC-HMS model contains 5 basin models: 1 model for each frequency. The only difference between the basin models is the initial loss.
Meteorological models	[frequency]_C=[storm centering]	The HEC-HMS model contains 5 meteorological models: 1 meteorological model for each of the 5 frequencies and 1 storm centering.
Time series data	[zone]_[frequency]_C=[storm centering]	The HEC-HMS model contains 35 precipitation gage records. These records are balanced hyetographs, with areal reduction factors applied. Each of the 7 zones has 5 balanced hyetographs (5 frequencies and 1 storm centering) for a total of 35.
Simulation runs	[frequency]_C=[centering]	5 simulation runs are configured in the HEC-HMS model: 1 run for each of the 5 frequencies and 1 storm centering.
GIS data	---	There are shapefiles of analysis points, subbasins, and streams in the “maps” folder located within the HEC-HMS project folder.

Table 28. Storm centering and HEC-DSS pathnames for each analysis point

Analysis point (1)	Storm centering (mi^2)[1] (2)	HEC-DSS pathname[2] (3)
BUT-43	183.11	//BUT-43/FLOW/31DEC1899 – 29JAN1900/15MIN/ RUN:[frequency]_C=183.11
BUT-37		//BUT-37/FLOW/31DEC1899 – 29JAN1900/15MIN/ RUN:[frequency]_C=183.11
BUT-27		//BUT-27/FLOW/31DEC1899 – 29JAN1900/15MIN/ RUN:[frequency]_C=183.11

1. The storm centering for which results are valid for each analysis point.
2. The F-part for each DSS record has the naming convention RUN:[frequency]_C=[storm centering].

Appendix IV. Contributors to report

This analysis was completed, documented, and prepared by, and under the direction of, the following:

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FLOOD INSURANCE STUDY

BUTTE COUNTY, CALIFORNIA AND INCORPORATED AREAS

COMMUNITY NAME	COMMUNITY NUMBER
BIGGS, CITY OF	060437
BUTTE COUNTY (UNINCORPORATED AREAS)	060017
CHICO, CITY OF	060746
GRIDLEY, CITY OF	060019
OROVILLE, CITY OF	060020
PARADISE, TOWN OF[1]	060748

[1]No Special Flood Hazard Areas

Revised:
January 6, 2011

Federal Emergency Management Agency

FLOOD INSURANCE STUDY NUMBER
06007CV000A

NOTICE TO
FLOOD INSURANCE STUDY USERS

Communities participating in the National Flood Insurance Program have established repositories of flood hazard data for floodplain management and flood insurance purposes. This Flood Insurance Study (FIS) may not contain all data available within the repository. It is advisable to contact the community repository for any additional data.

Part or all of this FIS may be revised and republished at any time. In addition, part of this FIS may be revised by the Letter of Map Revision process, which does not involve republication or redistribution of the FIS. It is, therefore, the responsibility of the user to consult with community officials and to check the community repository to obtain the most current FIS components.

Initial Countywide FIS Effective Date: June 8, 1998

Revised Countywide FIS Effective Date(s): April 20, 2000
January 6, 2011

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FLOOD INSURANCE STUDY
BUTTE COUNTY, CALIFORNIA AND INCORPORATED AREAS

1.0 INTRODUCTION

1.1 Purpose of Study

This countywide Flood Insurance Study (FIS) investigates the existence and severity of flood hazards in, or revises and updates previous FIS/Flood Insurance Rate Maps (FIRMs) for the geographic area of Butte County, California, including: the Cities of Biggs, Chico, Gridley, Oroville, the Town of Paradise, and the unincorporated areas of Butte County (hereinafter referred to collectively as Butte County). The Town of Paradise is a non-floodprone community.

This FIS aids in the administration of the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973. This FIS has developed flood risk data for various areas of the county that will be used to establish actuarial flood insurance rates. This information will also be used by Butte County to update existing floodplain regulations as part of the Regular Phase of the NFIP, and will also be used by local and regional planners to further promote sound land use and floodplain development. Minimum floodplain management requirements for participation in the NFIP are set forth in the Code of Federal Regulations at 44 CFR, 60.3.

In some States or communities, floodplain management criteria or regulations may exist that are more restrictive or comprehensive than the minimum Federal requirements. In such cases, the more restrictive criteria take precedence and the State (or other jurisdictional agency) will be able to explain them.

1.2 Authority and Acknowledgments

The sources of authority for this FIS are the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973.

The hydrologic and hydraulic analyses for the original June 8, 1998 study were performed for the Federal Emergency Management Agency (FEMA) by Gill & Pulver Engineers, Inc., under Contract No. EMW-85-C-1891, and was completed in February 1987; the study was also performed by Borcalli & Associates, Inc., under Contract No. EMW-91-C-3375, and was completed in April 1993. The hydrologic and hydraulic analyses were also performed by Schaaf & Wheeler, under Contract No. EMW-92-C-4071, and was completed in April 1993.

This study was revised on April 20, 2000, to incorporate approximate flood-hazard information along Dead Horse and Keefer Sloughs and Wyman Ravine in the vicinity of Butte County. This restudy incorporates the results of a study performed by Borcalli & Associates, Inc., for FEMA, under Contract No. EMF-96-CO-0097. This work was completed on November 13, 1997.

For this countywide study, MAP IX-Mainland was contracted by FEMA, under contract number EMF-2003-CO-0047, to revise the 1998 Butte Countywide FIS and DFIRM. This work was completed in May 2009.

Behind levee analyses was completed for Biggs Extension, Cherokee Canal, Comanche Creek, Dead Horse Slough, Feather River, Little Chico-Butte Creek Diversion Channel, Little Dry Creek, Main Drainage Canal, Mud Creek, Sycamore Creek, and Western Canal; the studies were performed by Nolte Associates, Inc. for FEMA, and was completed in May 2007.

Additional behind levee analyses was completed for Butte Creek, Comanche Creek, Dry Creek, Feather River, Lindo Channel, Little Chico-Butte Creek Diversion Channel, Big Chico Creek Diversion Channel, and Sycamore Creek; these studies were performed by URS Corporation for FEMA, and were completed in May 2009.

Reaches of the Upper Feather River from the mouth of Yuba River to Oroville Dam were restudied in May 2008. This countywide revision incorporates the results of the study performed by the U.S. Army Corps of Engineers (USACE) Sacramento District for FEMA (Reference 26).

Base map information shown on this FIRM was derived from multiple sources. Street centerlines and political boundaries were provided by Butte County Development Services – GIS Division. This information was derived at a scale of 1:24,000 and was adjusted to fit digital orthophotos created by Butte County Association of Governments in 2002 and 2004 respectively. Additional information was derived from FEMA FIRM maps dated 1998 or later.

The projection used in the preparation of this map was California State Plane II FIPS 402. The horizontal datum was NAD 83, GRS80 spheroid. Differences in datum, spheroid, projection or State Plane zones used in the production of FIRMs for adjacent jurisdictions may result in slight positional differences in map features across jurisdiction boundaries. These differences do not affect the accuracy of this FIRM.

1.3 Coordination

Consultation Coordination Officer's (CCO) meetings may be held for each jurisdiction in this countywide FIS. An initial CCO meeting is held typically with representatives of FEMA, the community, and the study contractor to explain the nature and purpose of a FIS, and to identify the streams to be studied by detailed methods. A final CCO meeting is held typically with representatives of FEMA, the community, and the study contractor to review the results of the study.

The dates of the initial and final CCO meetings held for the original June 8, 1998 countywide FIS and the April 20, 2000 countywide revision for Butte County and the incorporated communities within its boundaries are shown in Table 1, "Initial and Final CCO Meetings."

Table 1 – Initial and Final CCO Meetings

Community	Initial CCO Date	Final CCO Date
Butte County	December 1984	August 24, 1988[1]
and Incorporated Areas	July 1990	September 21, 1995[1]
	*	April 8, 1997[1]
	*	July 21, 1998[2]

**Data not available*

[1] June 8, 1998 initial countywide

[2] April 20, 2000 countywide revision

For this countywide revision, an initial CCO meeting was held on June 30, 2005, and was attended by representatives of FEMA, the communities, and the study contractor. The final CCO meeting was held on July 9, 2009, and was attended by representatives of FEMA, the communities, and the study contractor.

2.0 AREA STUDIED

2.1 Scope of Study

This FIS covers the geographic area of Butte County, California.

All or portions of the flooding sources listed in Table 2, "Flooding Sources Studied by Detailed Methods," were studied by detailed methods. Limits of detailed study area indicated on the Flood Profiles (Exhibit 1) and on the FIRM (Exhibit 2).

Table 2 – Flooding Sources Studied by Detailed Methods

Butte Creek	Little Chico-Butte Creek Diversion Channel
Big Chico Creek	Little Dry Creek
Big Chico Creek Diversion Channel	Mud Creek
Big Chico Creek Split Flow	Palermo Tributary
Comanche Creek	Ruddy Creek
Dead Horse Slough	Ruddy Creek Tributary
Durham Slough	Sycamore Creek
Hamlin Slough	Wyman Ravine
Lindo Channel	Wyman Ravine Tributary 1
Little Chico Creek	

Numerous flooding sources in the county were studied by approximate methods. Approximate analyses were used to study those areas having a low development potential or minimal flood hazards. The scope and methods of study were proposed to, and agreed upon by, FEMA and the communities.

This countywide FIS also incorporates the determination of Letter of Map Revision (LOMR) case number 04-09-0415P, dated March 31, 2005, for the City of Chico and the Unincorporated Areas of Butte County, California.

2.2 Community Description

Butte County, founded in 1850, was one of the original 27 counties in California. Gold was discovered approximately 12 miles downstream from Oroville, the county seat, in 1848.

Butte County is bounded to the west by Glenn and Colusa Counties, with the Sacramento River forming half of the western boundary; to the north and northwest by Tehama County; to the east by Plumas County; to the south by Sutter County; and to the southeast by Yuba County, with Honcut Creek forming the southeastern boundary (Reference 1).

Butte County, with an area of 1,054,320 acres or 1,680 square miles, contains a wide range of climatic and topographic conditions. The county is geographically divided into a portion that lies in the northeastern part of the Sacramento Valley and the mountainous area surrounding the valley (Reference 1). The topography of the county varies, from the relatively flat Sacramento Valley floor, with an elevation ranging from 60 to 200 feet, and associated alluvial fans; to extensive rolling foothills, with elevations ranging from 200 to 2,100 feet; and to the Cascade and Sierra Nevada Mountain Ranges, with elevations ranging from 2,100 to greater than 6,000 feet above sea level. The valley comprises 45 percent of the county, the foothills comprise 23 percent, and the mountains comprise 31 percent (Reference 2). The valley floor and foothill country encompass approximately 1,100 square miles. Much of the valley floor is alluvial deposit accumulated through time by materials washed down from the face of the Sierras (Reference 1). Soil types in the county include the deep, nearly level, very fertile valley basin and alluvial soils of the Sacramento Valley and associated alluvial fans, which support extensive agriculture; the shallow, gentle to steep sloping, less fertile residual soils of the foothill areas; and the shallow to deep, moderate to steep sloping residual soils of the mountain areas, which are suitable for rangeland, forestry, and wildlife habitat uses. High clay-content expansive soil conditions (creating shrink-swell soil characteristics) predominate the southwestern portion of the county and some of the western portion (Reference 2).

Butte County has a typical Mediterranean climate with hot, dry summers and cool, wet winters. Cooler summers and cold winters are common in the areas of higher elevation. Annual precipitation, generally in the form of rain, ranges from 18 inches along the Sacramento River to 80 inches in high elevation areas, where snow falls regularly. Easterly winds are common above 3,500 feet in elevation. Average wind speeds are less than 8 miles per hour, and prolonged calm periods are common.

Prevailing winds are generally from the southwest during half of the year and from the northwest for the remainder. Southerly winds are normally associated with approaching winter storms and are usually moisture-bearing because of their origin over the Pacific Ocean. Northerly winds are usually associated with winter and spring high pressure ridging (fair weather) and occasional summer daytime breezes. Northerly winds tend to be dry.

Butte County contains abundant and diversified vegetations types, including the non-native agricultural crops and pastures in the valley, native foothill and mountain oak and conifer forests, dryland chaparral areas and riparian and marshland areas of restricted and diminishing distribution, which have high value as wildlife habitats (Reference 2).

No large, natural lakes exist within the county's boundaries. Several artificial lakes serve as domestic water, irrigation, and power dam reservoirs and are located in the mountain and foothill areas. Some examples of these are the Oroville, Philbrook, and Madrone reservoirs (Reference 1).

State Highway 99 and the main line of the Union Pacific Railroad cross the western lowland portion of Butte County. State Highway 70 runs northeasterly from Oroville into the scenic Feather River Canyon. The Western Pacific Railroad follows a similar route. The eastern part of Butte County is very mountainous, but most parts can be reached by car. There are airports at Chico and Oroville (Reference 3).

Butte County's agricultural products include rice, almonds, seed crops, vegetables, peaches, prunes, olives, and walnuts. Livestock and livestock products are also produced. Lumber, minerals, and food processing make up a large portion of the county's economy (Reference 3).

2.3 Principal Flood Problems

A variety of conditions cause flooding in Butte County.

<u>Butte Creek</u>

Floods of record in Butte Creek occurred in December 1937, December 1955, December 1964, and February 1986 (Reference 4). The recurrence intervals for these flows are approximately 20 years, 30 years, 50 years, and 50 years, respectively.

Keefer Slough

Flooding along Keefer Slough is primarily due to water being diverted into Keefer Slough from Rock Creek. The frequency of flooding has historically been dependent on the debris and vegetation in Rock Creek between State Highway 99 and its confluence with Keefer Slough. Farmers in the vicinity have periodically cleared Rock Creek to reduce spills into Keefer Slough. During periods when Rock Creek has not been cleared, Keefer Slough has spilled its banks. The most notable recent flood occurred in March 1983 when Keefer Slough flooded homes in the vicinity of Keefer Road and the area southwest of State Highway 99. State Highway 99 was overtopped for 11.5 hours. These floodflows continued southwest, affecting much of the area between State Highway 99 and the Union Pacific Railroad, including the community of Nord and its vicinity (References 5 and 6).

Little Chico Creek

Flows of record measured in Little Chico Creek occurred in December 1964, March 1978, and March 1974 (Reference 7). The recurrence intervals for these three storms are approximately 10 years, 15 years, and 30 years, respectively.

Ruddy Creek and Ruddy Creek Tributary

Areas of flooding along Ruddy Creek have been at the crossings of Nelson, Tehama, and Biggs Avenues. Minor flood damage was reported after the February 1986 storm. The March 1983 storm caused the most recent widespread flooding (Reference 5).

Wyman Ravine and Tributaries

As Wyman Ravine flows out of the steep foothills, its bed slope flattens, downstream of Lincoln Boulevard. Sheetflow and shallow flooding occur every few years in the orchards west of the Western Pacific Railroad. Floodflows over Palermo Road have extended east of Wyman Ravine almost as far as Occidental Avenue. With few exceptions, the reach of Wyman Ravine between Stimpson Lane and Lone Tree Road experiences annual flooding. The storm of February 1986 produced flow over Lone Tree Road, extending 500 feet north and 1,000 feet south of the creek (Reference 5).

The area to the south of Wyman Ravine Tributary 1, between the Western Pacific Railway embankment and Melvina Avenue, experiences chronic flooding, flow historically crosses over Melina Avenue south of Wyman Ravine Tributary 1 and continues west and southwest across the farm fields. Additional flow spills to the south between the Western Pacific Railway embankment and Railroad Avenue (References 5 and 8).

Palermo Tributary floods during the 10-percent-annual-chance flood and greater discharges. Sheetflow across roads and between homes occurs between approximately once in five years (Reference 5).

2.4 Flood Protection Measures

Several small lakes or ponds are located within the watersheds contributing to the studied reaches, but none have effects on the peak discharges. The largest of these are two water supply reservoirs located at Little Butte Creek, a tributary to Butte Creek. Historically, these reservoirs have been full and spilling during the occurrence of large floods and have not had an appreciable effect on floodflows (Reference 9).

Levees have been erected along the banks of a large portion of Wyman Ravine. The levees range in height from approximately 1 foot to 4 feet. The levees extending from the lower study limits to a point approximately 45,510 feet upstream do not continuously contain the 10-percent-annual-chance flood discharges. Their effectiveness in containing the 1-percent-annual-chance flood discharges is negligible, according to the analysis done in this study. The levee extending from a point approximately 3,500 feet north of Palermo Road to approximately 2,000 feet upstream of Lincoln Boulevard is more significant.

Several levee systems have been constructed along Butte Creek, Cherokee Canal, Big Chico Creek, Hamlin Slough, the Little Chico Butte Creek Diversion Channel, Comanche Creek, and Little Chico Creek. Through hydraulic investigations, these levees were determined to provide protection from less than the 1-percent-annual-chance flood, and/or certification of the levees for 1-percent-annual-chance flood protection could not be obtained from the responsible agency. Therefore, they have been shown on the FIRM as not containing the 1-percent-annual-chance flood.

3.0 ENGINEERING METHODS

For the flooding sources studied in detail in the community, standard hydrologic and hydraulic study methods were used to determine the flood hazard data required for this FIS. Flood events of a magnitude, which are expected to be equaled or exceeded once on the average during any 10-, 50-, 100-, or 500-year period (recurrence interval) have been selected as having special significance for floodplain management and for flood insurance rates. These events, commonly termed the 10-, 50-, 100-, and 500-year floods, have a 10-, 2-, 1-, and 0.2-percent chance, respectively, of being equaled or exceeded during any year. Although the recurrence interval represents the long-term average period between floods of a specific magnitude, rare floods could occur at short intervals or even within the same year. The risk of experiencing a rare flood increases when periods greater than 1 year are considered. For example, the risk of having a flood, which equals or exceeds the 1-percent-annual-chance flood in any 50-year period is approximately 40 percent (4 in 10), and, for any 90-year period, the risk increases to approximately 60 percent (6 in 10). The analyses reported herein reflect flooding potentials based on conditions existing in the community at the time of completion of this FIS. Maps and flood elevations will be amended periodically to reflect future changes.

3.1 Hydrologic Analyses

Hydrologic analyses were carried out to establish the peak discharge-frequency relationships for each flooding source studied in detail affecting the community.

April 20, 2000 Countywide Analyses

Twenty years of peak flow data from the period 1959 to 1984 were available from the California Department of Water Resources (CADWR) for Little Chico Creek (Reference 7). Fifty-two years of peak flow data were available for Butte Creek at U.S. Geological Survey (USGS) gage 1139000 from the period 1931 to 1982 (Reference 4). The location of the flow measurements coincided approximately with the downstream limit of study for both creeks.

A log-Pearson Type III analysis was conducted using the computer program HECWRC (Reference 10), in accordance with the guidelines of the Water Resources Council Bulletin 17B (Reference 11). The resulting peak discharges for the 1-percent-annual-chance recurrence interval flood for Little Chico Creek and Butte Creek were 5,000 and 25,000 cubic feet per second (cfs), respectively. The 1-percent-annual-chance flood discharges presented in an unpublished USACE Office Report in 1976 (Reference 9) were 6,700 and 30,000 cfs for these respective locations. The discrepancy in discharges is because of the inclusion of additional years of record, and the application of the Water Resources Council Guidelines regarding the exclusion of extreme data points and the incorporation of a non-zero skew.

For Ruddy Creek, Wyman Ravine, and their tributaries, runoff was developed using the HEC-l computer program (Reference 12). Six-hour storms were constructed using precipitation statistics for 29 years of record from the rainfall gage at the Oroville Ranger Station. Unit hydrographs were developed using the USACE procedures, as discussed earlier in this section, and the S-curve adopted by the Natural Resources Conservation Service (NRCS) (formerly the Soil Conservation Service). Loss rates were adjusted to produce a discharge for the 1-percent-annual-chance storm that agrees closely with the discharge published in the study by Cook Associates (Reference 8). The point at which the discharges were compared was the point of concentration for approximately 50 percent of the drainage of the Wyman Ravine watershed upstream of the lower study limit.

Wyman Ravine, Wyman Ravine Tributary 1, and Palermo Tributary all have reaches where some flow spills out of the channel and does not return for several thousand feet, if at all. The HEC-2 computer program has the capability of determining where water leaves the channel, but does not adequately account for the downstream effects of the flow transfers. To more accurately model the flow transfers, the hydrologic and hydraulic models were developed simultaneously. A discussion of the development of the discharges presented in Table 3, "Summary of Discharges", is presented in Section 3.2. The 1-percent-annual-chance flood peak discharge at the Stimpson Road crossing is 2,390 cfs. The only other reported discharge at this location was by Cook Associates (Reference 8), which assigns a discharge of 3,300 cfs to the same stream location. The difference is due primarily to the more detailed analysis of this study and the consideration of flow leaving the watershed before it reaches Stimpson Road.

The primary source of the peak discharge in Keefer Slough is the overflow from Rock Creek at their upstream confluence. Rock Creek is an integral part of the hydrology of Keefer Slough.

Rainfall runoff was modeled using the HEC-l computer model. Storms each having a duration of 6 hours for different return periods were developed by obtaining 6-hour rainfall depths from the National Oceanic and Atmospheric Administration (NOAA) precipitation maps (Reference 13) and distributing the storm totals according to the statistics of 30 years of recorded precipitation in the nearby City of Chico. Unit hydrographs for the Rock Creek and Keefer Slough subbasins were developed using a method developed by the USACE. This method utilizes a dimensionless S-curve unit hydrograph in combination with a relationship that relates lag time to various physical parameters of the watershed. The USACE work for the 1975 Office File Report on Pine and Rock Creeks (Reference 14) was used as a basis for the selection of the Valley and Cottonwood S-curves and some of the parameters related to the lag time.

The hydrologic model was calibrated using the adopted peak discharges for Little Chico Creek. The drainage basin of Rock Creek upstream of Keefer Slough and that of Little Chico Creek upstream of the study limits are very similar with respect to size, orientation, topography, and ground cover. For this reason the peak discharges in Rock Creek upstream of its confluence with Keefer Slough were assumed to be the same as the discharges determined for Little Chico Creek.

Loss rates were adjusted to produce peak discharges in Rock Creek equal to the discharges of Little Chico Creek at the point of comparison. A rating curve was developed to represent the division of the Rock Creek total discharge between that portion of the discharge that is diverted into Keefer Slough and the balance of the discharge, which continues down the Rock Creek main channel. This rating was based on the normal depth computations in each channel by modeling a representative channel cross section near their confluence in a hydraulic computer program (Reference 15). The result of this rating is that approximately 44 percent of the 1-percent-annual-chance total Rock Creek discharge is diverted into Keefer Slough. This analysis increases the discharge in Keefer Slough by approximately 1,800 cfs from the original study. Due to the increase in discharge, the detailed study area between State Highway 99 and Keefer Lane was redelineated using an approximate method.

The adopted peak discharges in Keefer Slough are presented in Table 3, "Summary of Discharges." In the cases of the 1-percent and 0.2-percent-annual-chance flood events, the discharges decrease downstream between Garner Lane and State Highway 99. The channel capacity in this reach is 525 cfs. Any additional discharge spills over the left bank and flows away from Keefer Slough. The total 2-percent-, 1-percent, and 0.2-percent-annual-chance flood discharges at State Highway 99 are 760, 840, and 1,200 cfs, respectively. The difference between these discharges and those listed in Table 3 constitutes sheetflow across State Highway 99. The discharges at State Highway 99 exceed the flows presented in previous studies. The USACE (Reference 14) computed a peak discharge of 470 cfs for the 1-percent-annual-chance flood event and McCain Associates (Reference 16) published a flow of 566 cfs for the same return period. This study considered the contribution from Rock Creek and has resulted in a higher total discharge.

Rainfall-runoff modeling was performed for Butte Creek, Hamlin Slough, Comanche Creek, Little Chico-Butte Creek Diversion Channel, and Little Chico Creek, using the HEC-1 computer model. The purpose of the modeling was to estimate peak discharges for performing the floodplain analysis.

Storms having a duration of 24 hours were developed by obtaining rainfall depths from precipitation maps contained in the NOAA precipitation maps and distributing the storms in accordance with the Type lA distribution contained in the NRCS Technical Release 55 (Reference 17).

Precipitation losses were calculated based upon developed NRCS curve numbers (CN). Soil parameters were obtained from NRCS soil surveys and U.S. Forest Service soil vegetation maps. Land use characteristics are based on field investigation, aerial photos, quadrangle maps and Forest Service timber stand and vegetation maps. CN are selected according to soil type and land use, and are based on a set of CN developed by the NRCS for a watershed in Contra Costa County, California. The synthetic unit hydrographs were developed using the NRCS dimensionless unit hydrograph and channel routing was accomplished using the Muskingum and Muskingum-Cunge Methods.

A log-Pearson Type III analysis was performed for Little Chico Creek near Chico, reflecting the period of record from 1931-1988 and for Butte Creek at Durham reflecting the period of record from 1959-1981 and 1983-1990. The results of the analyses at the gages were used as the targets for adjusting the interception/infiltration losses.

The adopted peak discharges in Butte Creek, Hamlin Slough, Little Chico-Butte Creek Diversion Channel, Comanche Creek, and Little Chico Creek are shown in Table 3, "Summary of Discharges."

The USGS and CADWR streamflow gages are located on several streams in the study area; however, only the discharge determined by frequency analysis of data from USGS gage 1138400 on Big Chico Creek may be used in the FIS. The required assumption of annual peak streamflows as independent, random events is invalidated by upstream diversions for all other gage data within the study limits. Additionally, the gage on Lindo Channel was moved about 3 miles upstream in 1974, so any analysis that combines data from the two gage stations would not be valid, since heterogeneity has been introduced. Statistical analysis follows the guidelines set forth in Bulletin 17B of the Interagency Advisory Committee on Water Data (Reference 18).

A summary of the drainage area-peak discharge relationships for the streams studied by detailed methods is shown in Table 3, "Summary of Discharges."

Table 3 – Summary of Discharges

		Peak Discharges (cfs)			
Flooding Source and Location	**Drainage Area (sq mi)**	**10-Percent-Annual-Chance**	**2-Percent-Annual-Chance**	**1-Percent-Annual-Chance**	**0.2-Percent-Annual-Chance**
BIG CHICO CREEK					
Upstream of Big Chico Creek Diversion Structure	73.65	*	*	11,000	*
Downstream of Diversion Structure (Upstream of Manzanita)	73.65	*	*	1,400	*
Road Bend At Bidwell Avenue (2.4 miles Downstream of Rose Avenue)	75.56	*	*	1,730	*
BIG CHICO CREEK DIVERSION CHANNEL[1]					
Downstream of Lindo Channel Diversion Structure	*	*	*	5,600	*
Upstream of Confluence with Sycamore Creek	4.69	*	*	6,070	*
BUTTE CREEK					
At Hamlin Slough	*	13,200	24,400	30,300	44,800
At Aquas Frias Road	*	13,600	28,000	34,900	51,100
Approximately 930 feet upstream of confluence with Little Butte Creek	117.6	10,560	17,040	20,000	27,200
At Skyway	151.4	13,200	21,300	25,000	34,000
COMANCHE CREEK					
Approximately one mile above Midway	*	300	550	6,300	16,800
Approximately 1,500 feet above Midway	*	300	550	3,000	3,000
At Midway	*	300	550	2,300	2,300

[1]*ExcessBig Chico Creek flows are diverted northerly to Lindo Channel and Sycamore Creek. Sycamore Creek merges with Mud Creek upstream of Highway 99.*

**Data not available*

Table 3 – Summary of Discharges, continued

Flooding Source and Location	Drainage Area (sq mi)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
COMANCHE CREEK, continued					
At Union Pacific Railroad	*	400	800	2,100	2,100
Approximately 1,300 feet below Union Pacific Railroad	*	500	900	2,300	2,300
Approximately 1,500 feet above Dayton Road	*	500	900	1,600	1,600
At Lone Pine Road	*	500	900	900	900
Sacramento River Floodplain	*	500	900	1,200	1,200
DEAD HORSE SLOUGH					
At confluence with Little Chico Creek	5.36	750	1,500	1,900	*
HAMLIN SLOUGH					
North Branch at confluence	9.3	523	1,380	1,820	2,640
South Branch at confluence	10.16	741	1,710	2,300	3,290
Hamlin Canyon	33.85	2,300	4,700	6,200	8,650
Hayes Canyon	37.75	2,570	5,210	6,720	9,330
At confluence with Butte Creek	40.12	2,670	5,330	6,830	9,430
KEEFER SLOUGH[1]					
Approximately 1,125 feet downstream of Hicks Lane	0.3	130	400	560	750
Approximately 500 feet upstream of Garner Lane	2.9	275	500	680	850
At State Highway 99[2]	4.4	415	525	525	525
LINDO CHANNEL					
Upstream of confluence with Channel Slough/Sandy Gulch (0.6 miles Downstream of Highway 32)	5.25	*	*	4,600	*
Downstream of Big Chico Creek Diversion Structure	*	*	*	4,000	*
LITTLE CHICO-BUTTE CREEK DIVERSION CHANNEL					
At Diversion Structure	*	700	2,200	3,100	4,900
Approximately 1,500 feet below Warfield	*	800	2,400	3,300	5,200
Approximately 2,000 feet below Skyway	*	1,100	3,000	3,900	6,000

[1]*Drainage area only refers to Keefer Slough local drainage; diversions from Rock Creek are a major source of the listed discharges.*

[2]*See Section 3.1 for an explanation of the reduction in flow.*

**Data not available*

Table 3 – Summary of Discharges, continued

This is "above" diversion structure?

Flooding Source and Location	Drainage Area (sq mi)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
LITTLE CHICO CREEK					
Below Diversion Structure	*	2,300	4,400	5,600	7,800
At Forest Avenue	*	1,500	2,000	2,200	2,500
At State Highway 99	*	2,100	3,400	3,700	*
Approximately 100 feet above Bruce Street	*	2,100	3,400	3,500	3,700
At Bruce Street	*	2,200	3,100	3,100	3,100
At Mills Street	*	2,200	2,800	2,800	2,800
At Crouch Road	*	2,200	2,500	2,500	2,500
Approximately 3,000 feet below Alberton	*	2,300	2,600	2,600	2,600
Sacramento River Floodplain	*	2,300	2,700	2,700	2,700
MUD CREEK					
Downstream of Confluence with Sycamore Circle	44.89[2]	*	*	10,410	*
At Nord Highway	45.44[2]	*	*	10,700	*
PALERMO TRIBUTARY					
At Baldwin Avenue	1.0	255	355	390	470
Approximately 100 feet downstream of Palermo Road	1.7	500	690	760	920
Approximately 550 feet downstream of South Villa Avenue[1]	1.7	126	126	126	126
At confluence with Wyman Ravine Tributary 1	2.1	500	690	760	920
RUDDY CREEK					
Just upstream of confluence with Ruddy Creek Tributary	0.7	255	350	380	460
Approximately 350 feet upstream of Feather River	1.9	580	790	870	1,050
Entire Reach	0.5	165	220	250	300

[1]*See Section 3.2 for an explanation of the reduction in flow.*

[2]*Includes Big Chico Creek Diversion Channel and Sycamore Creek drainage area.*

**Data not available*

Table 3 – Summary of Discharges, continued

Flooding Source and Location	Drainage Area (sq mi)	Peak Discharges (cfs) 10-Percent-Annual-Chance	2-Percent-Annual-Chance	1-Percent-Annual-Chance	0.2-Percent-Annual-Chance
SYCAMORE CREEK					
Upstream of confluence with Big Chico Creek Diversion Channel	8.60	*	*	2,170	*
Downstream of confluence with Diversion	13.29[2]	*	*	7,080	*
Upstream of confluence with Mud Creek	24.99[2]	*	*	8,100	*
WYMAN RAVINE					
Approximately 220 feet downstream of Lincoln Boulevard	12.6	1,670	2,390	2,625	2,970
Approximately 90 feet downstream of Western Pacific Railroad[1]	12.6	1,660	2,200	2,310	2,465
Approximately 2,470 feet downstream of Western Pacific Railroad[1]	14.3	340	385	400	425
Approximately 690 feet downstream of Palermo Road	16.0	1,950	2,620	2,770	3,020
Approximately 200 feet upstream of confluence with Wyman Ravine Tributary 1	16.4	1,950	2,710	2,930	3,390
Approximately3,580 feet downstream of confluence with Wyman Ravine Tributary 1[1]	21.6	2,145	3,010	3,290	3,840
Approximately 6,800 feet downstream of Lone Tree Road	26.2	1,570	1,845	1,920	2,060
At Stimpson Lane	28.4	1,775	2,230	2,390	2,700
WYMAN RAVINE TRIBUTARY 1					
Approximately 60 feet upstream of Melvina Avenue	2.8	560	790	870	1,070
Approximately 950 feet downstream of Melvina Avenue[1]	2.8	80	100	100	110
At confluence with Palermo Tributary[1]	4.9	490	610	660	740
At Western Pacific Railway culvert[1]	4.9	370	430	450	480
At confluence with Wyman Ravine	5.2	440	530	550	600

[1]*See Section 3.2 for an explanation of the reduction in flow.*

[2]*Includes Big Chico Creek Diversion Channel drainage area.*

**Data not available*

2010 Countywide Revision

The drainage area for the Feather River extends from the confluence of the Feather River at the Yuba River down to the confluence of the Feather River and the Sacramento River encompassing over 26,000 square miles.

Historically, large events occurring at the Shanghai Bend have resulted from rare events occurring on the Upper Feather River (above Oroville) and also on the Yuba River, with one of these rivers having a slightly rarer event then the other. Because of the possibility that either scenario could happen, two different hypothetical storm patterns were produced. The differences in the storm patterns lies within the index locations on the Feather and Yuba Rivers.

For the seven hypothetical storms (10-, 2-, 1-, and 0.2- percent chance exceedences) no other location in the Sacramento River Basin experiences a larger flood than at Shanghai Bend and the Latitude of Verona. The distribution of storm intensity for the Upper Feather and Yuba river basins were developed. Initial exceedence frequency values were assigned to the Yuba River and Feather River index locations. Hydrographs were then constructed at these locations and routed through the system to Shanghai Bend. Duration maxima (peak 1-, 3-, 7-, 15- and 30-day) were computed for the hydrographs at Shanghai Bend and compared with the average flows from the frequency curves. The initial pattern was then increased or decreased and the comparison process was repeated until results agreed reasonably with the unregulated rain flood frequency curves.

Once this portion of the pattern was set, the same process was followed for the Latitude of Verona index location. The storm pattern for the rest of the tributary index locations were based upon the average of the Feather and Yuba River storm centerings generated for the Comprehensive Study (Reference 23). This pattern was iteratively adjusted by a fixed percentage until the duration maxima (1-, 3-, 7-, 15-, and 30-day) computed at the Latitude of Verona agreed reasonably with the unregulated rain flood frequency curve at the index locations.

Hypothetical hourly hydrographs consisting of six 5-day waves were generated based on the unregulated frequency curves obtained from the Comprehensive Study (Reference 23). No adjustments were made to any of the frequency curves except for the peak curve for Shanqhai Bend. The 1997 flood was chosen as the pattern for the five – day wave patterns. These wave patterns were constructed by adjusting regulated gage records for the 1997 flood event in accordance with changes in upstream storage.

Reservoir routing for the Feather River system was accomplished using both the HEC-5 and the ResSim modeling package produced by the Hydrologic Engineering Center (HEC). A ResSim model was used to model the Feather – Yuba system and the HEC-5 model completed as part of the Comprehensive Study (Reference 23) was used to model the Sacramento River system down to the confluence with the Feather River (Verona). Output hydrographs from both of these models were used as input into the hydraulic models, which cover the majority of the main river system.

A summary of the regulated peak discharges along the Feather River is shown on Table 4, "Regulated Peak Flows."

Table 4 – Regulated Peak Flows

% Chance Exceedence	Feather River at Oroville	North Yuba River at new Bullards Bar Dam	Yuba River at Marysville	Feather River at Shanghai Bend	Feather River at Nicolaus
10	100,000	44,400	92,400	200,000	219,000
2	150,000	50,000	150,000	293,000	323,000
1	150,000	66,100	155,000	296,000	323,000
0.2	327,000	150,000	313,000	607,000	668,000

3.2 Hydraulic Analyses

Analyses of the hydraulic characteristics of flooding from the sources studied were carried out to provide estimates of the elevations of floods of the selected recurrence intervals. Users should be aware that flood elevations shown on the FIRM represent rounded whole-foot elevations and may not exactly reflect the elevations shown on the Flood Profiles or in the Floodway Data tables in the FIS report. For construction and/or floodplain management purposes, users are encouraged to use the flood elevation data presented in this FIS in conjunction with the data shown on the FIRM.

The hydraulic analysis for this revision was based on unobstructed flow. The flood elevations shown on the flood profiles (Exhibit 1) are thus considered valid only if hydraulic structures remain unobstructed, operate properly, and do not fail.

Locations of selected cross sections used in the hydraulic analyses are shown on the Flood Profiles (Exhibit 1). For stream segments for which a floodway is computed (Section 4.2), selected cross section locations are also shown on the FIRM (Exhibit 2).

April 20, 2000 Countywide Analyses

Cross sections for the detailed analyses of Keefer Slough, Butte Creek, and Wyman Ravine between the lower study limits and a point 35,480 feet upstream were obtained by field survey and extended where necessary from USGS 7.5-minute series topographic maps (Reference 19). Cross sections for the detailed analysis of Wyman Ravine Tributary 1, Palermo Tributary, and Wyman Ravine, between a point 35,480 feet upstream of Stimpson Road and Lincoln Boulevard, were obtained from topographic mapping (Reference 20). For the backwater analysis of Ruddy Creek and Ruddy Creek Tributary, cross sections were obtained from aerial photographs (Reference 21). Cross sections for all approximate method study reaches were obtained from USGS topographic maps (Reference 19). All bridges and culverts were field measured to obtain dimensions, geometry, and elevations.

Water-surface elevations of floods of the selected recurrence intervals were computed through the use the USACE HEC-2 step-backwater computer program (Reference 22).

Starting water-surface elevations for the backwater analyses of all the streams studied by detailed methods were determined by normal depth analysis. In the cases of Wyman Ravine and Keefer Slough, the detailed study started at the upstream face of constricting road crossings. In these cases the models were extended several hundred feet downstream of the structure to a location where normal depth approximations were appropriate.

The hydraulic characteristics of Wyman Ravine and its tributaries require special attention because of the existence of levees and the occurrences of low channel capacity, resulting in sheetflow breaking out of the channel and not returning for several thousand feet, if at all.

The next several paragraphs describe the major occurrences of water spilling out of the channel and the transfer of flow between channels of the Wyman Ravine system. These spills are the reason for the downstream reductions in peak discharge as presented in Table 3, "Summary of Discharges." The hydrology and hydraulic models were developed simultaneously in order to reflect all of these spills and flow transfers.

Levees have been erected along much of Wyman Ravine ranging in height from approximately 1 foot to 4 feet. The levees extending from the downstream study limit to a point approximately 45,510 feet upstream do not contain the 10-percent-annual-chance flood discharges and hence their existence does not affect the flood limits presented in this report. However, the levee that extends from a point approximately 3,500 feet north of Palermo Road to approximately 2,000 feet beyond Lincoln Boulevard restricts some of the flow from leaving the channel and affects the downstream flooding. In the analysis of Wyman Ravine, two cases of channel performance were considered. Case 1 considered the possibility of the latter levee remaining intact, and Case 2 considered the possibility of the same levee failing under flood conditions. The discharges listed in Table 3, "Summary of Discharges," and the profiles in Exhibit 1 represent Case 1, which considers the greater discharge in the channel. The associated flood boundary maps (Exhibit 2) reflect a combination of both cases. The right overbank flood limits result from the larger channel flows and the sheetflow and ponding zones indicated to the south of the ravine were determined assuming that the levee failed entirely.

It should be noted that, even in the case of the levee remaining intact, the large majority of the 1-percent-annual-chance flood streamflow spills out of the ravine before the channel bends sharply south at a point approximately 3,500 feet upstream of Palermo Road.

Some of the flow that spills out of Wyman Ravine between the Western Pacific Railroad and Lincoln Boulevard returns to Wyman Ravine after passing through a railway culvert 300 feet north of North Villa Avenue.

The reach of Wyman Ravine extending from Lone Tree Road to a point approximately 6,060 feet upstream is inadequate to contain the 10-percent-annual-chance flood discharge. Some of the flood discharge flows south and does not re-enter Wyman Ravine within the limits of the study.

The reach of Wyman Ravine Tributary 1 between the Western Pacific Railway embankment and Melvina Avenue is inadequate to contain the 10-percent-annual-chance flood discharge. The majority of the flow upstream of Melvina Avenue spills over the road south of the bridge crossing and continues westerly and southwesterly across the farm fields. Additional flow spills to the south between the Western Pacific Railway embankment and Railroad Avenue.

Palermo Tributary is inadequate to contain the 10-percent-annual-chance flood discharge. Upstream of Palermo Road the flow is confined between the high ground on the east and Lincoln Boulevard on the west. Between Palermo Road and South Villa Road the channel will not contain the 10-percent-annual-chance flood discharge. Any spill over the right bank (east bank) continues southwesterly away from the channel as sheetflow. The Western Pacific Railway embankment stops the westerly movement of the floodflow and directs the sheetflow south across South Villa and into Wyman Ravine Tributary 1. Some water that spills from Wyman Ravine upstream of the Western Pacific Railway embankment enters the Palermo drainage area but the timing of the peak discharge is such that it does not increase the peak discharge in Palermo Tributary or Wyman Ravine Tributary 1.

The approximate study portion of Wyman Ravine and Wyman Ravine Tributary 2 were analyzed using HEC-2. Little Chico Creek and the approximate study portion of Butte Creek were analyzed assuming that the flow traveled at normal depth.

The approximate study portion of Keefer Slough was modeled using HEC-2. The shallow flooding southwest of the channel was computed as normal depth flow. However, based on conversations with the County Department of Public Works, sheetflow southwest of State Highway 99 has occurred more extensively than can be simulated with normal depth approximations (Reference 5). The area is very flat with a mild slope to the southwest. Small farm levees can significantly alter the course of the overland flow. To account for this uncertainty in the path of sheetflow, and to include areas of observed flooding, the flood limits shown on the FIRM (Exhibit 2) are shown wide enough to encompass all possible paths of sheetflow.

Cross sections for detailed analysis of Butte Creek, Hamlin Slough, Little Chico-Butte Creek Diversion Channel, Comanche Creek, and Little Chico Creek were obtained by aerial and field surveys. On Butte Creek and Hamlin Slough, cross sections were extended where necessary using the topographic mapping prepared for this FIS and the USGS 7.5-minute series topographic mapping. All bridges and culverts were field measured to obtain dimensions, geometry, and elevations.

Starting water-surface elevations for the backwater analysis of the streams were determined by normal depth analysis, with the exception of Hamlin Slough and the Little Chico-Butte Creek Diversion Channel. For these streams, the starting water-surface was based upon the estimated water-surface elevation on Butte Creek that would be present at the time of the peak in the respective tributary.

The Butte Creek levee system located downstream of the Skyway could not be reflected as providing 1-percent-annual-chance flood protection in this FIS. Therefore, according to FEMA criteria, the system was evaluated for the three conditions reflecting both levees intact, the left levee failed, and the right levee failed.

The Hamlin Slough levee system located downstream of the Chico-Oroville Highway could not be reflected as providing 1-percent-annual-chance flood protection in this FIS. Therefore, according to FEMA criteria, the system was evaluated for the three conditions reflecting both levees intact, the left levee failed, and the right levee failed.

The Little Chico-Butte Creek Diversion Channel has reaches that consist of a levee along its right bank. The levee could not be reflected as providing 1-percent-annual-chance flood protection in this FIS. Therefore, according to FEMA criteria, the system was evaluated reflecting the levee intact and the levee failed.

The Little Chico-Butte Creek Diversion Channel crosses Comanche Creek. Therefore, under the failed levee scenario, the discharge in the diversion channel would flow down Comanche Creek instead of being delivered to Butte Creek. The hydraulic analysis of Comanche Creek for the 1-percent and 0.2-percent-annual-chance flood events reflects failed levee conditions on the diversion channel.

The levees located along the lower reaches of Comanche Creek could not be reflected as providing 1-percent-annual-chance flood protection in the FIS. Therefore, according to FEMA criteria, the system was evaluated for the three conditions reflecting both levees intact, the left levee failed, and the right levee failed.

The hydraulic analysis of Little Chico Creek reflects the diversion of flow into the Little Chico-Butte Creek Diversion Channel. The levees located in the lower reaches could not be reflected as providing 1-percent-annual-chance flood protection in this FIS. Therefore, according to FEMA criteria, the system was evaluated for the three conditions reflecting both levees intact, the left levee failed, and the right levee failed.

Reaches of Butte Creek downstream of the Skyway, Hamlin Slough, the Little Chico-Butte Creek Diversion Channel, Comanche Creek, and Little Chico Creek all have the occurrences of inadequate levees and/or channel capacities, resulting in flow breaking out of the channel and not returning for several thousand feet, if at all.

A detailed hydraulic analysis was prepared for Lindo Channel beginning approximately 2,000 feet downstream of the Nord Highway Bridge, upstream to its confluence with the Big Chico Creek Diversion Channel.

HEC-2 backwater analyses were run for Lindo Channel so that water surface elevations balance at bifurcations and diversions. The diversion structure is modeled using HEC-2 special culvert routines. Backwater computations were started by assuming normal depth downstream of the Nord Highway bridge. At each bridge or culvert, a 1:1 flow contraction into the opening and a 4:1 flow expansion out of the opening was modeled using encroachments.

Analysis indicates that the estimated 1-percent-annual-chance flood discharge is contained within the creek channel for the entire study reach. Downstream of Esplande, however, Lindo Channel is near bank capacity for the 1-percent-annual-chance flood discharge. Within this reach the channel is perched, so flows that overtopped the banks would tend to run away from the channel as shallow overland flooding. It should be noted that, while the estimated 1-percent-annual-chance flood discharge is significantly less than the channel's design capacity, that capacity was based on a clean channel. Vegetation growth has since reduced that capacity.

Since the estimated 1-percent-annual-chance flood discharge is contained within the channel for Lindo Channel, a floodway was not computed.

Diversion structures on Big Chico Creek and Lindo Channel affect discharges for every stream reach within the study limits, except Sycamore Creek upstream from its confluence with the Big Chico Creek Diversion Channel.

A recreational swimming pool was formed in the past at the diversion structure using temporary flashboards on the upstream faces of the culvert structures on Big Chico Creek and Lindo Channel. For the purposes of hydraulic analyses for this FIS, these flashboards are assumed to be removed prior to the flood season. While this is part of the City of Chico's operational procedure, it is not clear whether or not the flashboards have actually been removed prior to every flood season.

A detailed hydraulic analysis has been prepared for the Big Chico Creek diversion system, beginning at the Nord Highway bridge on Mud Creek. The studied river system includes Mud Creek from Nord Highway to the confluence with Sycamore Creek; Sycamore Creek from the confluence with Mud Creek to a point 1 mile above the tributary diversion canal; and the diversion canal from its outfall into Sycamore Creek to the diversion point at Big Chico Creek.

The USACE Sacramento District surveyed project levee crown elevations and found that the levees are currently at or near design grade. The USACE certifies that the levees are well maintained, do not have any known stability or foundation problems, and, with the exception of Sycamore Creek upstream from Sheep Hollow Creek, the project will pass design flows within the design water surface profile provided that adequate maintenance continues.

HEC-2 backwater analyses were run for each of the study reaches so that water-surface elevations balance at bifurcations and diversions. The ogee spillway on the Big Chico Creek Diversion Channel is modeled using a rating curve based on data found in the U.S. Bureau of Reclamation's Design of Small Dams.

For a balanced water surface with Lindo Creek, the estimated discharge over the spillway is 5,600 cfs.

Backwater computations were started by assuming normal depth downstream of the Nord Highway bridge, and normal depth in North Sycamore Creek. For freeboard determination, encroachments were placed at levee crests. At each bridge or culvert, a 1:1 flow contraction into the opening and a 4:1 flow expansion out of the opening were modeled using encroachments.

The USACE certified their project levees for grade and structural integrity. Adequate freeboard exists for all study reaches with the exception of 100 feet downstream of the Cohasset Road Bridge to just upstream of the bridge.

Following FEMA guidelines, levees without adequate freeboard are assumed not to exist when mapping flood elevations on the protected side of the levee. For this study reach, only about 100 lineal feet of right bank levee on each side of the Cohasset Road Bridge does not meet freeboard criteria. The configuration of the bridge is such that levee failure immediately upstream of the bridge merely causes water to back up into the right overbank without spilling over the road, which is on fill. Effective flow is not changed and the mapped water surface is contiguous with the main channel water surface.

Since the estimated 1-percent-annual-chance flood discharge is contained within the leveed channel for Mud Creek and the Big Chico Creek Diversion Channel, a floodway was not computed.

A detailed hydraulic analysis was prepared for North Sycamore Creek beginning at its confluence with the Big Chico Creek Diversion Channel (Sycamore Creek). North Sycamore Creek is studied for approximately 1 mile upstream of its confluence with the Big Chico Creek Diversion Channel.

North Sycamore Creek was not improved as part of the Sacramento River and Major and Minor Tributaries Project. There are no levees along the creek bank for this study reach.

A HEC-2 backwater analysis was run for North Sycamore Creek. Backwater computations were started by assuming normal depth within the reach of North Sycamore Creek just upstream of the confluence with the Big Chico Creek Diversion Channel. There are no bridges or culverts, nor channel expansions or contractions.

A floodway was established by encroaching to the natural channel banks, and then slightly relaxing the encroachments in order to provide a smooth floodway with a fairly constant width. The floodway results in a maximum rise over the base flood elevation of 0.5 foot.

In Keefer Slough, a rating curve was developed to represent the division of the Rock Creek total discharge between that portion of the discharge that is diverted into Keefer Slough and the balance of the discharge, which continues down the Rock Creek main channel. This rating was based on the normal-depth computations in each channel by modeling a representative channel cross section near their confluence using the USACE HEC-2 computer program. The result of this rating is that approximately 44 percent of the 1-percent-annual-chance total Rock Creek discharge is diverted. This analysis increases the discharge in Keefer Slough by approximately 1,800 cfs from the original study. Due to the increase in discharge, the detailed study area between Highway 99 and Keefer Lane was redelineated using an approximate method. The approximate studies for Dead Horse and Keefer Sloughs and Wyman Ravine were based on a HEC-2 analysis. Cross sections for the studied streams were compiled using available topographic mapping, USGS quadrangle maps, and as-built information. Hydraulic structure dimensions were determined using as-built construction plans and existing HEC-2 models.

Roughness factors (Manning's "n") used in the hydraulic computations were chosen based on engineering judgment and field observations of the streams and floodplain areas. The roughness values used for the channels and overbank floodplains are shown in Table 5, "Manning's "n" Values."

Table 5 – Manning's "n" Values

Community Name	Roughness Values Channel	Overbank
Big Chico Creek	0.045 – 0.1	0.045 – 0.1
Butte Creek	0.040 – 0.054	0.036 – 0.077
Comanche Creek	0.035 – 0.058	0.040 – 0.077
Dead Horse Slough	0.040	0.060
Hamlin Slough	0.035 – 0.050	0.036 – 0.048
Keefer Slough	0.040	0.060
Lindo Channel	0.040 – 0.070	0.045
Little Chico Creek	0.035 – 0.060	0.048 – 0.080
Mud Creek	0.035	0.045
North Sycamore Creek	0.045	0.045
Palermo Tributary	0.050 – 0.060	0.060 – 0.080
Rock Creek	0.060	0.060
Ruddy Creek	0.015 – 0.060	0.050 – 0.100
Ruddy Creek Tributary	0.015 – 0.040	0.040
Wyman Ravine	0.050	0.070
Wyman Ravine Tributary 1	0.080	0.080

2010 Countywide Revision

The study for the Upper Feather model extends from the mouth of the Yuba River upstream to Oroville Dam, approximately 44 miles in length. The basis of the model is the HEC-RAS hydraulic model generated for the Yuba River Basin, California, General Re-evaluation (Yuba GRR) Study (Reference 24).

Cross sections were taken from the hydraulic model for the Yuba GRR study (Reference 24) Refinements to the existing cross sections were performed at the locations where the extents of the floodplain boundaries were uncertain and questionable, and the cross sections of the existing geometry were too far apart. For these areas, more cross sections were generated utilizing DTM surfaces of the Feather River from the Comprehensive Study topographic data (Reference 25). The developed cross sections were imported in the geometry of HEC-RAS model for a more concise definition of floodplain boundaries. Furthermore, some of the already existing cross sections, whose lengths were not sufficient enough to capture the entire floodplain extents, were further extended into the left and right overbank.

Upstream and Downstream conditions for the HEC-RAS model were taken from the Lower Feather model/Yuba GRR models. Upstream boundary conditions consist of inflow hydrographs. Downstream boundary conditions consist of rating curves.

The channel model was calibrated to the 1997 storm event. The model was calibrated by adjusting the Manning's n values to provide a reasonable fit to the observed peak stages from 1997. Extensive effort was undertaken to model the area within HEC-RAS to match the gage data, without using unrealistic Manning's n values.

The limitations associated with the HEC-RAS modeling being one dimensional necessitated the selection of the FLO-2D hydraulic program for delineating flooding in the overbank area resulting from levee failure scenarios. FLO-2D model development constituted generation of a separate left overbank model and a right overbank model.

The FLO-2D grid model for the left bank extends from Oroville wildlife area on the left bank at river mile 58.6 to RM 27 downstream of city of Marysville. The horizontal extents are from the outskirts of Brown Valley Ridge. The levees that have been modeled extend from RM 56 near Oak Grove to Honcut Creek. The other levee encompasses the Honcut area on the four sides. The other two levees consist of the levee on Highway 20 and Marysville Ring Levee. The study limits cover approximately 400 square miles of Yuba County, Sutter County, and Butte County.

The FLO-2D model on the right bank extends approximately from the downstream edge of the Thermalito After Bay at River mile 55.6 of Feather River which is the upstream limit of the grid model, while the downstream limit of the grids is approximately at the confluence of Sutter Bypass and Feather River at RM 7.775 of Feather River. The horizontal extent of the model encompasses the area around Cherokee Canal, Butte Sink, Sutter Buttes, and Sutter Bypass. The levee reaches that have been incorporated into the model are the Feather River right bank levee extending from RM 59.6 to Feather River 7.7 and the Sutter Bypass left bank levee.

Levee breach locations were determined from the levee breach analysis performed in HEC-RAS and based on the recommendations provided by the geotechnical report. Also, the FEMA based levee failure standards have been incorporated into the modeling efforts. The outflow hydrographs resulting from the channel model simulations with the breaches of the levees were utilized as flow input to the FLO-2D models.

The results from the geotechnical levee failure and FEMA based failures were merged to delineate the extent of flooding on the left and right overbanks.

Qualifying bench marks within a given jurisdiction that are cataloged by the National Geodetic Survey (NGS) and entered into the National Spatial Reference System (NSRS) as First or Second Order Vertical and have a vertical stability classification of A, B, or C are shown and labeled on the FIRM with their 6-character NSRS Permanent Identifier.

Benchmarks cataloged by the NGS and entered into the NSRS vary widely in vertical stability classification. NSRS vertical stability classifications are as follows:

- Stability A: Monuments of the most reliable nature, expected to hold position/elevation well (e.g., mounted in bedrock)
- Stability B: Monuments which generally hold their position/elevation well (e.g., concrete bridge abutment)
- Stability C: Monuments which may be affected by surface ground movements (e.g., concrete monument below frost line)
- Stability D: Mark of questionable or unknown vertical stability (e.g., concrete monument above frost line, or steel witness post)

In addition to NSRS benchmarks, the FIRM may also show vertical control monuments established by a local jurisdiction; these monuments will be shown on the FIRM with the appropriate designations. Local monuments will only be placed on the FIRM if the community has requested that they be included, and if the monuments meet the aforementioned NSRS inclusion criteria.

To obtain current elevation, description, and/or location information for benchmarks shown on the FIRM for this jurisdiction, please contact the Information Services Branch of the NGS at (301) 713-3242, or visit their Web site at www.ngs.noaa.gov.

It is important to note that temporary vertical monuments are often established during the preparation of a flood hazard analysis for the purpose of establishing local vertical control. Although these monuments are not shown on the FIRM, they may be found in the Technical Support Data Notebook associated with this FIS and FIRM. Interested individuals may contact FEMA to access this data.

Levee Hazard Analysis

Some flood hazard information presented in prior FIRMs and in prior FIS reports for Butte County and its incorporated communities was based on flood protection provided by levees. Based on the information available and the mapping standards of the NFIP at the time that the prior FISs and FIRMs were prepared, FEMA accredited the levees as providing protection from the flood that has a 1-percent-chance of being equaled or exceeded in any given year. For FEMA to continue to accredit the identified levees with providing protection from the base flood, the levees must meet the criteria of the Code of Federal Regulations, Title 44, Section 65.10 (44 CFR 65.10), titled "Mapping of Areas Protected by Levee Systems."

On August 22, 2005, FEMA issued Procedure Memorandum No. 34 - Interim Guidance for Studies Including Levees. The purpose of the memorandum was to help clarify the responsibility of community officials or other parties seeking recognition of a levee by providing information identified during a study/mapping project. Often, documentation regarding levee design, accreditation, and the impacts on flood hazard mapping is outdated or missing altogether. To remedy this, Procedure Memorandum No. 34 provides interim guidance on procedures to minimize delays in near-term studies/mapping projects, to help our mapping partners properly assess how to handle levee mapping issues.

While 44 CFR Section 65.10 documentation is being compiled, the release of more up-to-date FIRM panels for other parts of a community or county may be delayed. To minimize the impact of the levee recognition and certification process, FEMA issued Procedure Memorandum No. 43 - Guidelines for Identifying Provisionally Accredited Levees on March 16, 2007. These guidelines will allow issuance of preliminary and effective versions of FIRMs while the levee owners or communities are compiling the full documentation required to show compliance with 44 CFR Section 65.10. The guidelines also explain that preliminary FIRMs can be issued while providing the communities and levee owners with a specified timeframe to correct any maintenance deficiencies associated with a levee and to show compliance with 44 CFR Section 65.10.

FEMA contacted the communities within Butte County to obtain data required under 44 CFR 65.10 to continue to show the levees as providing protection from the flood that has a 1-percent-chance of being equaled or exceeded in any given year.

FEMA understood that it might take time to acquire and/or assemble the documentation necessary to fully comply with 44 CFR 65.10. Therefore, FEMA put forth a process to provide the communities with additional time to submit all the necessary documentation. For a community to avail itself of the additional time, it had to sign an agreement with FEMA. Levees for which such agreements were signed are shown on the final effective FIRM as providing protection from the flood that has a 1-percent-chance of being equaled or exceeded in any given year and labeled as a Provisionally Accredited Levee (PAL). Communities have two years from the date of FEMA's initial coordination to submit to FEMA final accreditation data for all PALs. Following receipt of final accreditation data, FEMA will revise the FIS and FIRM as warranted.

FEMA coordinated with the USACE, the local communities, and other organizations to compile a list of levees that exist within Butte County. Table 6, "List of Structures Requiring Flood Hazard Revisions," lists all levees shown on the FIRM, to include PALs, for which corresponding flood hazard revisions were made.

Approximate analyses of "behind levee" flooding were conducted for all the levees in Table 6 to indicate the extent of the "behind levee" floodplains. The methodology used in these analyses is discussed below.

The approximate levee analysis was conducted using information from existing hydraulic models (where applicable) and USGS topographic maps.

The extent of the 1-percent-annual-chance flood in the event of levee failure was determined. Normal-depth calculations were used to estimate the base flood elevation (BFE) if detailed topographic or representative cross section information was available. The remaining BFEs were estimated from effective FIRM maps. The 1-percent-annual-chance floodplain boundary was traced along the contour line representing the estimated BFE. Topographic features such as highways, railroads, and high ground were used to refine approximate floodplain boundary limits. The 1-pecent annual chance peak flow and floodplain widths and depth (assumed at 1 foot) were used to ensure the floodplain boundary was not overly conservative

Table 6 – List of Structures Requiring Flood Hazard Revisions

Community	Flood Source	Levee Inventory ID (Lat. /Long. Coordinates. ; FIRM panel)	USACE Levee
City of Chico	Butte Creek Diversion Channel	1113 (-121.78, 39.73; -121.774, 39.732 06007C0510E)	No
City of Chico	Butte Creek Diversion Channel	1131 (-121.78, 39.722; -121.78, 39.73 06007C0506E/06007C0510E)	Yes
City of Chico	Butte Creek Diversion Channel	1305 (-121.783, 39.718; -121.78, 39.722 06007C0506E/06007C0510E)	No; not a levee
City of Chico	Dead Horse Slough	1269 (-121.794, 39.744; -121.793, 39.744 06007C0506E)	No
City of Chico	Unknown	1317 (-121.849, 39.784; -121.849, 39.787 06007C0340D)	No
City of Oroville	Lake Oroville	1291 (-121.595, 39.526; -121.579, 39.531 06007C0788D/ 06007C0790D)	Dam; not a levee

Table 6 – List of Structures Requiring Flood Hazard Revisions, continued

Community	Flood Source	Levee Inventory ID (Lat. /Long. Coordinates. ; FIRM panel)	USACE Levee
Butte County (Unincorporated Areas)	Butte Creek	1301 (-121.777, 39.694; -121.774, 39.697 06007C0510E)	Yes
Butte County (Unincorporated Areas)	Butte Creek Diversion Channel	1114 (-121.779, 39.695; -121.779, 39.698 06007C0510E)	Yes
Butte County (Unincorporated Areas)	Butte Creek Diversion Channel	1281 (-121.779, 39.698; -121.779, 39.703 06007C0510E)	Yes
Butte County (Unincorporated Areas)	Cherokee Canal	1284 (-121.882, 39.355; -121.867, 39.363 06007C1075D/06007C1100D)	Yes
Butte County (Unincorporated Areas)	Comanche Creek	1081 (-121.864, 39.701; -121.844, 39.702 06007C0505D)	No
Butte County (Unincorporated Areas)	Comanche Creek	1258 (-121.921, 39.667; -121.887, 39.681 06007C0495D)	No
Butte County (Unincorporated Areas)	Drainage Canal	1012 (-121.855, 39.32; -121.855, 39.33 06007C1100D)	No
Butte County (Unincorporated Areas)	Drainage Canal	1190 (-121.85, 39.315; -121.836, 39.315 06007C1100D)	No
Butte County (Unincorporated Areas)	Drainage Canal	1226 (-121.882, 39.328; -121.846, 39.347 06007C1075D /06007C1100D)	No
Butte County (Unincorporated Areas)	Drainage Canal	1287 (-121.854, 39.315; -121.85, 39.315 06007C1100D)	No
Butte County (Unincorporated Areas)	Drainage Canal	1288 (-121.854, 39.332; -121.845, 39.336 06007C1100D)	No

Table 6 – List of Structures Requiring Flood Hazard Revisions, continued

Community	Flood Source	Levee Inventory ID (Lat. /Long. Coordinates. ; FIRM panel)	USACE Levee
Butte County (Unincorporated Areas)	Drainage Canal	1289 (-121.845, 39.336; -121.838, 39.34 06007C1100D)	No
Butte County (Unincorporated Areas)	Drainage Canal	1290 (-121.855, 39.33; -121.854, 39.332 06007C1100D)	No
Butte County (Unincorporated Areas)	Dry Creek	1314 (-121.702, 39.572; -121.701, 39.574 06007C0755D)	Yes
Butte County (Unincorporated Areas)	Feather River	1026 (-121.621, 39.423; -121.605, 39.451 06007C0960D/06007C0975D/06007C0980 D/06007C0990D)	No
Butte County (Unincorporated Areas)	Feather River	1050 (-121.627, 39.419; -121.641, 39.44 06007C0960D/06007C0975D)	No
Butte County (Unincorporated Areas)	Feather River	1053 (-121.631, 39.46; -121.609, 39.47 06007C0960D/06007C0980D)	Yes
Butte County (Unincorporated Areas)	Feather River	1055 (-121.625, 39.396; -121.641, 39.44 06007C0960D/06007C0975D)	Yes
Butte County (Unincorporated Areas)	Feather River	1060 (-121.63, 39.457; -121.595, 39.471 06007C0960D/06007C0980D)	No
Butte County (Unincorporated Areas)	Feather River	1062 (-121.593, 39.472; -121.581, 39.494 06007C0980D)	No
Butte County (Unincorporated Areas)	Feather River	1078 (-121.625, 39.396; -121.632, 39.413 06007C0975D/06007C0990D)	No
Butte County (Unincorporated Areas)	Feather River	1092 (-121.641, 39.44; -121.64, 39.458 06007C0960D)	Yes

Table 6 – List of Structures Requiring Flood Hazard Revisions, continued

Community	Flood Source	Levee Inventory ID (Lat. /Long. Coordinates. ; FIRM panel)	USACE Levee
Butte County (Unincorporated Areas)	Feather River	1184 (-121.638, 39.306; -121.637, 39.313 06007C1125D)	Yes
Butte County (Unincorporated Areas)	Feather River	1229 (-121.637, 39.313; -121.625, 39.396 06007C0975D/06007C1125D)	Yes
Butte County (Unincorporated Areas)	Feather River	1265 (-121.623, 39.422; -121.621, 39.423 06007C0975D/06007C0990D)	No
Butte County (Unincorporated Areas)	Feather River	1266 (-121.621, 39.423; -121.605, 39.425 06007C0990D)	No
Butte County (Unincorporated Areas)	Mud Creek	1241 (-121.883, 39.786; -121.876, 39.802 06007C0320E)	Yes
Butte County (Unincorporated Areas)	Sacramento River-Eddy Lake	1141 (-121.973, 39.529; -121.97, 39.534 06007C0725D)	Yes
Butte County (Unincorporated Areas)	Thermalito Afterbay	1119 (-121.686, 39.505; -121.64, 39.458 06007C0770D/06007C0960D/ 06007C0975D)	Dam; not a levee
Butte County (Unincorporated Areas)	Thermalito Afterbay	1120 (-121.639, 39.458; -121.629, 39.464 06007C0960D)	Dam; not a levee
Butte County (Unincorporated Areas)	Thermalito Afterbay	1238 (-121.686, 39.505; -121.684, 39.509 06007C0770D)	Dam; not a levee
Butte County (Unincorporated Areas)	Thermalito Forebay	1221 (-121.626, 39.514; -121.595, 39.526 06007C0770D/06007C0788D)	Dam; not a levee
Butte County (Unincorporated Areas)	Thermalito Forebay	1263 (-121.63, 39.515; -121.626, 39.514 06007C0770D)	Dam; not a levee

Table 6 – List of Structures Requiring Flood Hazard Revisions, continued

Community	Flood Source	Levee Inventory ID (Lat. /Long. Coordinates. ; FIRM panel)	USACE Levee
Butte County (Unincorporated Areas)	Unknown	1018 (-121.712, 39.523; -121.712, 39.538 06007C0765D)	No
Butte County (Unincorporated Areas)	Unknown	1037 (-121.754, 39.583; -121.748, 39.587 06007C0735D/06007C0755D)	No
Butte County (Unincorporated Areas)	Western Canal	1014 (-121.882, 39.329; -121.882, 39.355 06007C1075D)	No
Butte County (Unincorporated Areas)	Western Canal	1059 (-121.605, 39.47; -121.595, 39.471 06007C0980D)	No
Butte County (Unincorporated Areas)	Western Canal	1061 (-121.605, 39.471; -121.593, 39.472 06007C0980D)	No

Several levees within Butte County and its incorporated communities meet the criteria of the Code of Federal Regulations, Title 44, Section 65.10 (44 CFR 65.10), titled "Mapping of Areas Protected by Levee Systems." Table 7, "List of Certified and Accredited Levees," lists all levees shown on the FIRM that meet the requirements of 44 CFR 65.10 and have been determined to provide protection from the flood that has a 1-percent-chance of being equaled or exceeded in any given year.

Table 7 – List of Certified and Accredited Levees

Community	Flood Source	Levee Inventory ID (Lat. /Long. Coordinates. ; FIRM panel)	USACE Levee
City of Chico	Big Chico Diversion Channel	1306 (-121.81, 39.775; -121.793, 39.762 06007C0339D/ 06007C0343D)	Yes
City of Chico	Mud Creek Diversion Channel	1308 (-121.797, 39.761; -121.793, 39.762 06007C0343D)	Yes

Table 7 – List of Certified and Accredited Levees, continued

City of Chico	Sycamore Creek	1161 (-121.852, 39.78; -121.849, 39.784 06007C0340D)	Yes
City of Chico	Sycamore Creek	1277 (-121.855, 39.779; -121.852, 39.78 06007C0340D)	Yes
City of Chico	Sycamore Creek	1300 (-121.843, 39.778; -121.841, 39.78 06007C0339D)	Yes
City of Chico	Sycamore Creek	1304 (-121.851, 39.776; -121.848, 39.775 06007C0340D)	Yes
City of Chico Butte County (Unincorporated Areas)	Mud Creek	1243 (-121.913, 39.757; -121.883, 39.785 06007C0320E)	Yes
City of Chico Butte County (Unincorporated Areas)	Sycamore Creek	1160 (-121.883, 39.786; -121.855, 39.779 06007C0320E/06007C0340D)	Yes
City of Chico Butte County (Unincorporated Areas)	Sycamore Creek	1164 (-121.883, 39.785; -121.851, 39.776 06007C0320E/06007C0340D)	Yes
City of Chico Butte County (Unincorporated Areas)	Sycamore Creek	1173 (-121.851, 39.776; -121.849, 39.774 06007C0340D)	Yes
City of Chico Butte County (Unincorporated Areas)	Sycamore Creek	1244 (-121.85, 39.777; -121.846, 39.776 06007C0340D)	Yes
City of Chico Butte County (Unincorporated Areas)	Sycamore Creek	1278 (-121.851, 39.776; -121.85, 39.777 06007C0340D)	Yes
City of Oroville	Feather River	1233 (-121.573, 39.511; -121.551, 39.516 06007C0790D/ 06007C0795D)	No
Butte County (Unincorporated Areas)	Mud Creek	1034 (-121.927, 39.741; -121.886, 39.784 06007C0320E/06007C0485D)	Yes

Table 7 – List of Certified and Accredited Levees, continued

Community	Flood Source	Levee Inventory ID (Lat. /Long. Coordinates. ; FIRM panel)	USACE Levee
Butte County (Unincorporated Areas)	Mud Creek	1256 (-121.885, 39.785; -121.876, 39.802 06007C0320E)	Yes
Butte County (Unincorporated Areas)	Mud Creek	1297 (-121.927, 39.741; -121.913, 39.757 06007C0320E/06007C0485D)	Yes
Butte County (Unincorporated Areas)	Western Canal	1090 (-121.706, 39.522; -121.686, 39.505 06007C0765D/06007C0770D)	No
Butte County (Unincorporated Areas)	Western Canal	1218 (-121.703, 39.523; -121.686, 39.505 06007C0765D/06007C0770D)	Yes

3.3 Vertical Datum

All FISs and FIRMs are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum in use for newly created or revised FISs and FIRMs was the National Geodetic Vertical Datum of 1929 (NGVD29). With the finalization of the North American Vertical Datum of 1988 (NAVD88), many FIS reports and FIRMs are being prepared using NAVD88 as the referenced vertical datum.

All flood elevations shown in this FIS report and on the FIRM are referenced to NAVD88. Structure and ground elevations in the community must, therefore, be referenced to NAVD88. It is important to note that adjacent communities may be referenced to NGVD29. This may result in differences in BFEs across the corporate limits between the communities.

The conversion factor from NGVD29 to NAVD88 was 2.35 for all streams in Butte County.

As noted above, the elevations shown in the FIS report and on the FIRM for Butte County are referenced to NAVD88. Ground, structure, and flood elevations may be compared and/or referenced to NGVD29 by applying a standard conversion factor.

The BFEs shown on the FIRM represent whole-foot rounded values. For example, a BFE of 102.4 will appear as 102 on the FIRM and 102.6 will appear as 103. Therefore, users that wish to convert the elevations in this FIS to NGVD29 should apply the stated conversion factors to elevations shown on the Flood Profiles and supporting data tables in the FIS report.

For more information on NAVD88, see Converting the National Flood Insurance Program to the North American Vertical Datum of 1988, FEMA Publication FIA-20/June 1992, or contact the Spatial Reference System Division, National Geodetic Survey, NOAA, Silver Spring Metro Center, 1315 East-West Highway, Silver Spring, Maryland 20910 (Internet address http://www.ngs.noaa.gov).

4.0 FLOODPLAIN MANAGEMENT APPLICATIONS

The NFIP encourages State and local governments to adopt sound floodplain management programs. To assist in this endeavor, each FIS provides 1-percent-annual-chance floodplain data, which may include a combination of the following: 10-percent, 2-percent, 1-percent, and 0.2-percent-annual-chance flood elevations; delineations of the 1-percent and 0.2-percent-annual-chance floodplains; and 1-percent-annual-chance floodway. This information is presented on the FIRM and in components of the FIS, including Flood Profiles. Users should reference the data presented in the FIS as well as additional information that may be available at the local community map repository before making flood elevation and/or floodplain boundary determinations.

4.1 Floodplain Boundaries

To provide a national standard without regional discrimination, the 1-percent-annual-chance flood has been adopted by FEMA as the base flood for floodplain management purposes. The 0.2-percent-annual-chance flood is employed to indicate additional areas of flood risk in the community. For the stream studied in detail, the 1-percent and 0.2-percent-annual-chance floodplains have been delineated using the flood elevations determined at each cross section. Between cross sections, the boundaries were interpolated using topographic maps at a scale and a contour interval as shown on Table 8, "Topographic Map Information."

The 1-percent and 0.2-percent-annual-chance floodplain boundaries are shown on the FIRM (Exhibit 2). On this map, the 1-percent-annual-chance floodplain boundary corresponds to the boundary of the areas of special flood hazards (Zones A, AE, AH, and AO), and the 0.2-percent-annual-chance floodplain boundary corresponds to the boundary of areas of moderate flood hazards. In cases where the 1-percent and 0.2-percent-annual-chance floodplain boundaries are close together, only the 1-percent-annual-chance floodplain boundary has been shown. Small areas within the floodplain boundaries may lie above the flood elevations but cannot be shown due to limitations of the map scale and/or lack of detailed topographic data.

For the streams studied by approximate methods, only the 1-percent-annual-chance floodplain boundary is shown on the FIRM (Exhibit 2).

Table 8 – Topographic Map Information

Flooding Source	Scale	Contour Interval	Reference
Big Chico Creek	1:400	4 foot	[1]
Butte Creek	1:24,000	5 & 40 foot	19
Keefer Slough	1:24,000	5 foot	19
Little Chico Creek	1:24,000	5 & 40 foot	19
Palermo Tributary	1:2,400	2 foot	20
Ruddy Creek	1:4,800	4 foot	21
Ruddy Creek Tributary	1:4,800	4 foot	21
Wyman Ravine	1:24,000	5 foot	19
	1:2,400	2 foot	20
Wyman Ravine Tributary 1	1:2,400	2 foot	20

[1] *Data not available*

There are several locations along Wyman Ravine and its tributaries, as well as Butte Creek downstream of the Skyway, Hamlin Slough, Comanche Creek, and Little Chico Creek, where flow spills from the channel as sheetflow. The limits of this shallow flooding were determined by normal depth analysis. Only the 1-percent-annual-chance floodplain boundaries are indicated for the shallow flooding reaches. Shallow flooding occurs on Wyman Ravine between Lone Tree Road and a point approximately 8,750 feet upstream of Lone Tree Road and again between a point 1,330 feet downstream of Palermo Road and Lincoln Boulevard. Shallow flooding occurs on Wyman Ravine Tributary 1 between the Western Pacific Railroad embankment and Melvina Avenue and on Palermo Tributary between South Villa Avenue and Palermo Road.

4.2 Floodways

Encroachment on floodplains, such as structures and fill, reduces flood-carrying capacity, increases flood heights and velocities, and increases flood hazards in areas beyond the encroachment itself. One aspect of floodplain management involves balancing the economic gain from floodplain development against the resulting increase in flood hazard. For purposes of the NFIP, a floodway is used as a tool to assist local communities in this aspect of floodplain management. Under this concept, the area of the 1-percent-annual-chance floodplain is divided into a floodway and a floodway fringe. The floodway is the channel of a stream, plus any adjacent floodplain areas, that must be kept free of encroachment so that the 1-percent-annual-chance flood can be carried without substantial increases in flood heights. Minimum Federal standards limit such increases to 1.0 foot, provided that hazardous velocities are not produced. The floodways in this study are presented to local agencies as a minimum standard that can be adopted directly or that can be used as a basis for additional floodway studies.

The floodways presented in this study were computed for certain stream segments on the basis of equal-conveyance reduction from each side of the floodplain. Floodway widths were computed at cross sections. Between cross sections, the floodway boundaries were interpolated. The results of the floodway computations are tabulated for selected cross sections (Table 9). The computed floodways are shown on the revised FIRM (Exhibit 2). In cases where the floodway and 1-percent-annual-chance floodplain boundaries are either close together or collinear, only the floodway boundary is shown.

As discussed in Sections 3.2 and 4.1 of this report, there are several reaches of Wyman Ravine and its tributaries, as well as Butte Creek downstream of the Skyway, Hamlin Slough, Comanche Creek, and Little Chico Creek, where the overbank does not confine the flow. In these reaches some of the flow leaves the channel and becomes shallow flooding. Consequently, floodways have not been determined in these reaches.

The area between the floodway and 1-percent-annual-chance floodplain boundaries is termed the floodway fringe. The floodway fringe encompasses the portion of the floodplain that could be completely obstructed without increasing the water-surface elevation of the 1-percent-annual-chance flood by more than 1.0 foot at any point. Typical relationships between the floodway and the floodway fringe and their significance to floodplain development are shown in Figure 1, "Floodway Schematic."

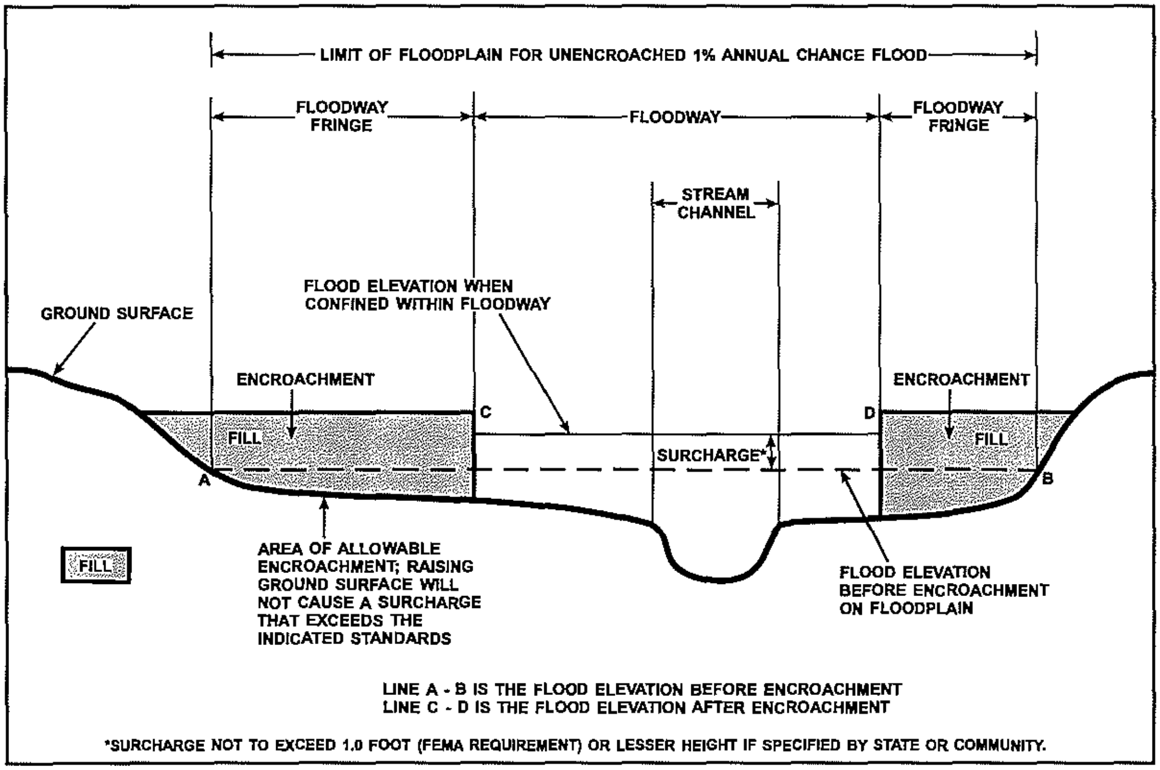

Figure 1 – Floodway Schematic

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Big Chico Creek								
A	0	40	294	5.4	160.9	160.9	161.9	1.0
B	630	57	446	3.6	163.4	163.4	163.8	0.4
C	1,170	48	334	4.8	164.7	164.7	165.0	0.3
D	1,640	38	303	5.3	166.5	166.5	166.6	0.1
E	2,260	35	311	5.1	168.8	168.8	168.8	0.0
F	2,890	44	392	4.1	170.6	170.6	170.6	0.0
G	3,445	41	330	4.8	171.9	171.9	171.9	0.0
H	4,390	47	397	4.0	174.5	174.5	174.5	0.0
I	5,610	53	428	3.7	177.0	177.0	177.0	0.0
J	6,410	57	380	4.2	178.6	178.6	178.6	0.0
K	7,060	48	357	4.5	180.4	180.4	180.4	0.0
L	8,065	61	468	3.4	183.4	183.4	183.4	0.0
M	9,335	79	602	2.7	187.0	187.0	187.0	0.0
N	10,340	66	474	3.2	190.3	190.3	190.4	0.1
O	11,367	64	486	3.1	192.3	192.3	192.8	0.5
P	11,954	130	953	1.6	193.6	193.6	194.0	0.4
Q	12,711	164	1,308	1.1	196.7	196.7	196.8	0.1
R	13,072	178	909	1.6	196.8	196.8	196.9	0.1
S	13,409	64	411	3.6	197.0	197.0	197.1	0.1
T	14,124	95	555	2.7	198.3	198.3	198.3	0.0
U	14,314	80	469	3.2	198.7	198.7	199.7	1.0
V	14,829	120	709	2.1	199.9	199.9	200.5	0.6
W	15,349	45	279	5.4	201.1	201.1	201.4	0.3
X	15,854	315	1,172	1.3	203.6	203.6	203.6	0.0
Y	16,189	110	578	2.6	203.8	203.8	203.8	0.0
Z	16,340	140	738	2.0	204.6	204.6	204.6	0.0

[1]Feet above road bend at Bidwell Avenue

TABLE 9

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA AND INCORPORATED AREAS

FLOODWAY DATA

BIG CHICO CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Big Chico Creek								
AA	17,550	92	367	4.1	206.8	206.8	206.8	0.0
AB	18,248	120	750	2.0	210.1	210.1	210.2	0.1
AC	18,638	205	1,062	1.4	210.3	210.3	210.4	0.1
AD	19,568	93	265	5.7	211.9	211.9	211.9	0.0
AE	20,358	176	850	1.8	215.1	215.1	215.2	0.1
AF	20,949	160	786	1.9	218.6	218.6	219.2	0.6
AG	21,209	171	779	1.8	219.2	219.2	219.8	0.6
AH	22,209	124	656	2.1	220.7	220.7	221.4	0.7
AI	22,879	164	578	2.4	223.0	223.0	223.3	0.3
AJ	23,709	111	591	2.4	225.7	225.7	225.7	0.0
AK	24,719	112	398	3.5	229.2	229.2	229.2	0.0
AL	25,658	120	776	1.8	231.9	231.9	232.2	0.3
AM	26,598	74	218	4.2	233.2	233.2	234.0	0.8
AN	27,448	66	290	3.1	241.7	241.7	241.7	0.0
AO	27,558	124	448	2.0	242.3	242.3	242.3	0.0
AP	28,303	216	642	2.2	246.3	246.3	246.3	0.0
AQ	28,853	139	557	2.5	248.4	248.4	248.4	0.0
AR	29,963	98	498	2.8	252.3	252.3	252.4	0.1
AS	30,993	92	457	3.1	256.0	256.0	256.0	0.0
AT	32,013	109	681	2.1	258.6	258.6	258.6	0.0
AU	33,143	72	161	8.7	263.4	263.4	263.4	0.0
AV	33,778	98	582	2.4	267.0	267.0	267.1	0.1
AW	34,268	247	1,123	1.2	267.5	267.5	267.6	0.1
AX	34,883	30	161	8.7	267.7	267.7	267.8	0.1

[1]Feet above road bend at Bidwell Avenue

TABLE 9	FEDERAL EMERGENCY MANAGEMENT AGENCY BUTTE COUNTY, CA AND INCORPORATED AREAS	FLOODWAY DATA BIG CHICO CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Big Chico Creek Split Flow								
AL	25,658[1]	120	775	1.8	231.9	231.9	232.2	0.3
AM	26,598[1]	144	102	4.8	237.7	237.7	237.7	0.0
AN	27,448[1]	187	424	1.2	241.2	241.2	241.8	0.6
AO	27,558[1]	244	689	0.7	241.3	241.3	241.9	0.6
Butte Creek								
P	2,050[2]	732	5,316	4.7	249.4	249.4	250.2	0.8
Q	8,575[2]	869	5,336	4.7	277.9	277.9	278.8	0.9
R	10,850[2]	640	4,120	6.1	287.4	287.4	288.0	0.6
S	13,750[2]	900	3,775	6.6	300.8	300.8	301.2	0.4
T	17,000[2]	752	4,831	5.2	317.5	317.5	317.7	0.2
U	21,200[2]	300	1,909	13.1	333.5	333.5	333.5	0.0
V	23,850[2]	346	3,039	8.2	344.2	344.2	344.2	0.0
W	25,500[2]	430	3,411	7.3	352.5	352.5	352.5	0.0
X	27,250[2]	232	1,853	10.8	358.7	358.7	358.7	0.0

[1]Feet above road bend at Bidwell Avenue

[2]Feet above Skyway Street

TABLE 9

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

BIG CHICO CREEK SPLIT FLOW - BUTTE CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Palermo Tributary								
A	600 [1]	208	379	2.0	150.8	150.8	151.8	1.0
B	1,100 [1]	150	329	2.3	152.1	152.1	152.9	0.8
C	3,795 [1]	300	432	1.8	161.0	161.0	161.0	0.0
D	4,835 [1]	140	166	2.3	163.8	163.8	164.3	0.5
E	5,595 [1]	120	175	2.2	166.7	166.7	167.6	0.9
F	6,415 [1]	100	232	1.7	170.2	170.2	171.1	0.9
Ruddy Creek								
A	700 [2]	119	287	3.5	155.7	155.7	156.7	1.0
B	2,100 [2]	150	339	3.0	164.1	164.1	165.1	1.0
C	3,600 [2]	130	368	2.7	168.6	168.6	169.2	0.6
D	4,570 [2]	90	339	2.6	173.0	173.0	173.9	0.9
E	5,100 [2]	111	406	2.1	174.2	174.2	175.2	1.0
F	6,700 [2]	64	245	3.6	182.0	182.0	182.6	0.6
G	8,600 [2]	60	225	3.9	186.8	186.8	187.5	0.7
H	10,250 [2]	50	166	2.3	191.7	191.7	192.2	0.5

[1]Feet above confluence with Wyman Ravine Tributary 1
[2]Feet above mouth

TABLE 9

FEDERAL EMERGENCY MANAGEMENT AGENCY
BUTTE COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

PALERMO TRIBUTARY - RUDDY CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Ruddy Creek Tributary								
A	1,100[1]	50	125	2.0	193.6	193.6	194.0	0.4
B	1,800[1]	50	304	0.8	198.0	198.0	198.7	0.7
C	3,250[1]	50	225	1.1	198.0	198.0	198.9	0.9
D	4,350[1]	90	165	1.5	198.1	198.1	199.1	1.0
Sycamore Creek								
A - J[3]								
K	14,760[2]	114	383	5.7	195.3	195.3	195.3	0.0
L	15,720[2]	78	325	6.7	201.1	201.1	201.1	0.0
M	16,870[2]	158	351	6.2	208.9	208.9	208.9	0.0
N	17,925[2]	163	397	5.5	217.1	217.1	217.3	0.2
O	19,047[2]	125	360	6.0	226.1	226.1	226.6	0.5
P	20,285[2]	144	385	5.6	236.6	236.6	236.8	0.2

[1]Feet above confluence with Ruddy Creek

[2]Feet above State Highway 99

[3] No Floodway determined

TABLE 9

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

RUDDY CREEK TRIBUTARY - SYCAMORE CREEK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Wyman Ravine								
A	150	1,000	3,776	0.9	97.6	97.6	98.6	1.0
B	2,200	1,300	3,439	1.0	98.0	98.0	99.0	1.0
C	4,500	1,300	3,913	0.9	98.7	98.7	99.7	1.0
D	6,400	700	1,219	2.8	100.0	100.0	100.8	0.8
E	8,270	700	2,248	1.5	102.9	102.9	103.6	0.7
F	10,580	800	3,086	1.1	104.0	104.0	104.9	0.9
G	11,910	700	2,422	1.4	104.4	104.4	105.4	1.0
H	13,430	512	1,990	1.7	105.5	105.5	106.4	0.9
I	16,170	559	1,921	1.8	107.9	107.9	108.8	0.9
J	17,570	495	1,151	2.9	109.5	109.5	110.3	0.8
K	19,810	650	2,031	1.7	111.6	111.6	112.6	1.0
L	21,640	600	1,493	2.2	114.8	114.8	115.7	0.9
M	24,030	600	1,641	2.0	117.4	117.4	118.4	1.0
N	25,880	550	1,501	2.2	119.2	119.2	120.2	1.0
O	28,000	605	1,330	2.5	122.4	122.4	123.3	0.9
P	30,570	660	1,848	1.8	126.2	126.2	126.7	0.5
Q	33,470	301	869	3.8	132.6	132.6	133.2	0.6
R	34,830	180	640	5.1	137.2	137.2	137.3	0.1
S	36,180	245	970	3.4	140.2	140.2	141.1	0.9
T	37,140	660	1,901	1.7	143.1	143.1	143.9	0.8
U	37,740	220	853	3.4	145.1	145.1	145.9	0.8
V	38,540	179	575	5.1	146.7	146.7	147.6	0.9
W	39,700	269	762	3.8	151.3	151.3	152.2	0.9
X	40,680	166	738	4.0	154.2	154.2	155.0	0.8

[1]Feet above Stimpson Road

TABLE 9

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA
AND INCORPORATED AREAS

FLOODWAY DATA

WYMAN RAVINE

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Wyman Ravine Tributary 1								
A	830	110	187	2.9	143.6	143.6	144.6	1.0
B	1,320	55	134	4.1	147.1	147.1	147.3	0.2
C	3,830	190	368	2.4	158.5	158.5	159.5	1.0
D	5,150	170	394	2.2	165.5	165.5	166.1	0.6
E	6,370	253	540	1.6	173.4	173.4	174.4	1.0

[1]Feet above confluence with Wyman Ravine

TABLE 9

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA AND INCORPORATED AREAS

FLOODWAY DATA

WYMAN RAVINE TRIBUTARY 1

5.0 INSURANCE APPLICATIONS

For flood insurance rating purposes, flood insurance zone designations are assigned to a community based on the results of the engineering analyses. The zones are as follows:

Zone A

Zone A is the flood insurance rate zone that corresponds to the 1-percent-annual-chance floodplains that are determined in the FIS by approximate methods. Because detailed hydraulic analyses are not performed for such areas, no BFEs or depths are shown within this zone.

Zone AE

Zone AE is the flood insurance rate zone that corresponds to the 1-percent-annual-chance floodplains that are determined in the FIS by detailed methods. In most instances, whole-foot BFEs derived from the detailed hydraulic analyses are shown at selected intervals within this zone.

Zone AH

Zone AH is the flood insurance rate zone that corresponds to the areas of 1-percent-annual-chance shallow flooding (usually areas of ponding) where average depths are between 1 and 3 feet. Whole-foot BFEs derived from the detailed hydraulic analyses are shown at selected intervals within this zone.

Zone AO

Zone AO is the flood insurance rate zone that corresponds to the areas of 1-percent-annual-chance shallow flooding (usually sheet flow on sloping terrain) where average depths are between 1 and 3 feet. Average whole-foot depths derived from the detailed hydraulic analyses are shown within this zone.

Zone D

Zone D is the flood insurance rate zone that corresponds to unstudied areas where flood hazards are undetermined, but possible.

Zone X

Zone X is the flood insurance rate zone that corresponds to areas outside the 0.2-percent-annual-chance floodplain, areas within the 0.2-percent-annual-chance floodplain, and areas of 1-percent-annual-chance flooding where average depths are less than 1 foot, areas of 1-percent-annual-chance flooding where the contributing drainage area is less than 1 square mile, and areas protected from the 1-percent-annual-chance flood by levees. No BFEs or depths are shown within this zone.

COMMUNITY NAME	INITIAL IDENTIFICATION	FLOOD HAZARD BOUNDARY MAP REVISION DATE	FIRM EFFECTIVE DATE	FIRM REVISIONS DATE
Biggs City of	June 8, 1998	NONE	June 8, 1998	NONE
Butte, County of	September 6, 1974	December 27, 1977	September 29, 1989	NONE
Chico, City of	June 8, 1998	NONE	June 8, 1998	NONE
Gridley City of	June 8, 1998	NONE	June 8, 1998	NONE
Oroville, City of	June 7, 1974	September 19, 1975	September 24, 1984	NONE
Paradise, Town of [1]	N/A	NONE	N/A	NONE

[1] No Special Flood Hazards

TABLE 10

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA
AND INCORPORATED AREAS

COMMUNITY MAP HISTORY

6.0 FLOOD INSURANCE RATE MAP

The FIRM is designed for flood insurance and floodplain management applications.

For flood insurance applications, the map designates flood insurance rate zones as described in Section 5.0 and, in the 1-percent-annual-chance floodplains that were studied by detailed methods, shows selected whole-foot BFEs or average depths. Insurance agents use the zones and BFEs in conjunction with information on structures and their contents to assign premium rates for flood insurance policies.

For floodplain management applications, the map shows by tints, screens, and symbols, the 1- and 0.2-annual chance floodplains. Floodways and the locations of selected cross sections used in the hydraulic analyses and floodway computations are shown where applicable.

This FIRM includes some flood hazard information that was presented separately on the Flood Boundary and Floodway Maps, where applicable. Historical data relating to the maps prepared for each community up to and including this countywide FIS are presented in Table 10, "Community Map History."

7.0 OTHER STUDIES

Information pertaining to revised and unrevised flood hazards for each jurisdiction within Butte County has been compiled into this FIS. Therefore, this FIS supersedes all previously printed FIS Reports, FHBMs, FBFMs, and FIRMs for all of the incorporated and unincorporated jurisdictions within Butte County

8.0 LOCATION OF DATA

Information concerning the pertinent data used in the preparation of this FIS can be obtained by contacting FEMA, Federal Insurance and Mitigation Division, 1111 Broadway, Suite 1200, Oakland, California 94607-4052.

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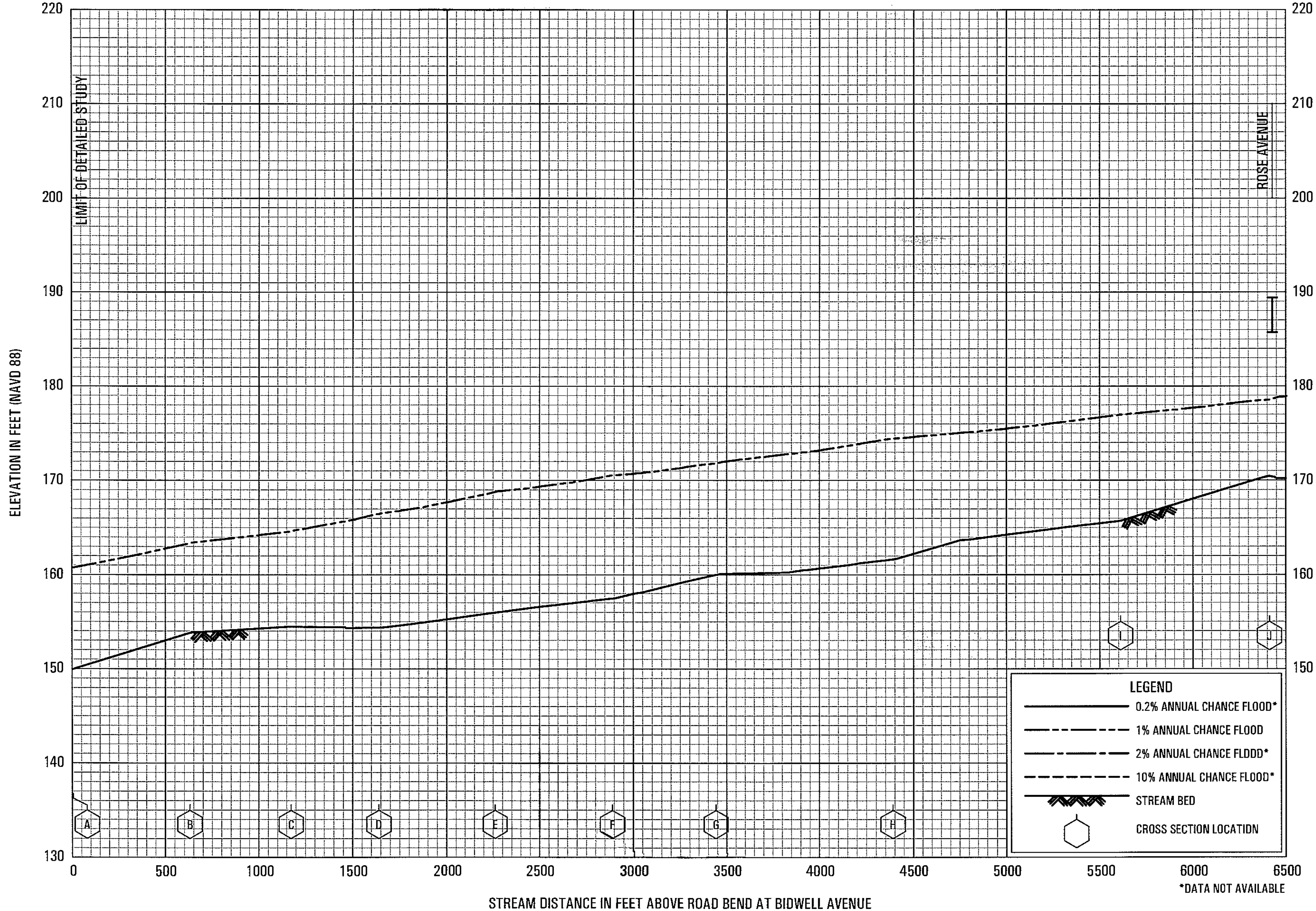
25. U.S. Army Corps of Engineers, Sacramento District, "Topographic and Hydrographic Surveys of the Feather River System for the Sacramento and San Joaquin River Basins Comprehensive Study, California, Report," May 2003.

26. U.S. Army Corps of Engineers, Sacramento District, "Upper Feather River, Floodplain Mapping Study," May 2008.



FLOOD PROFILES

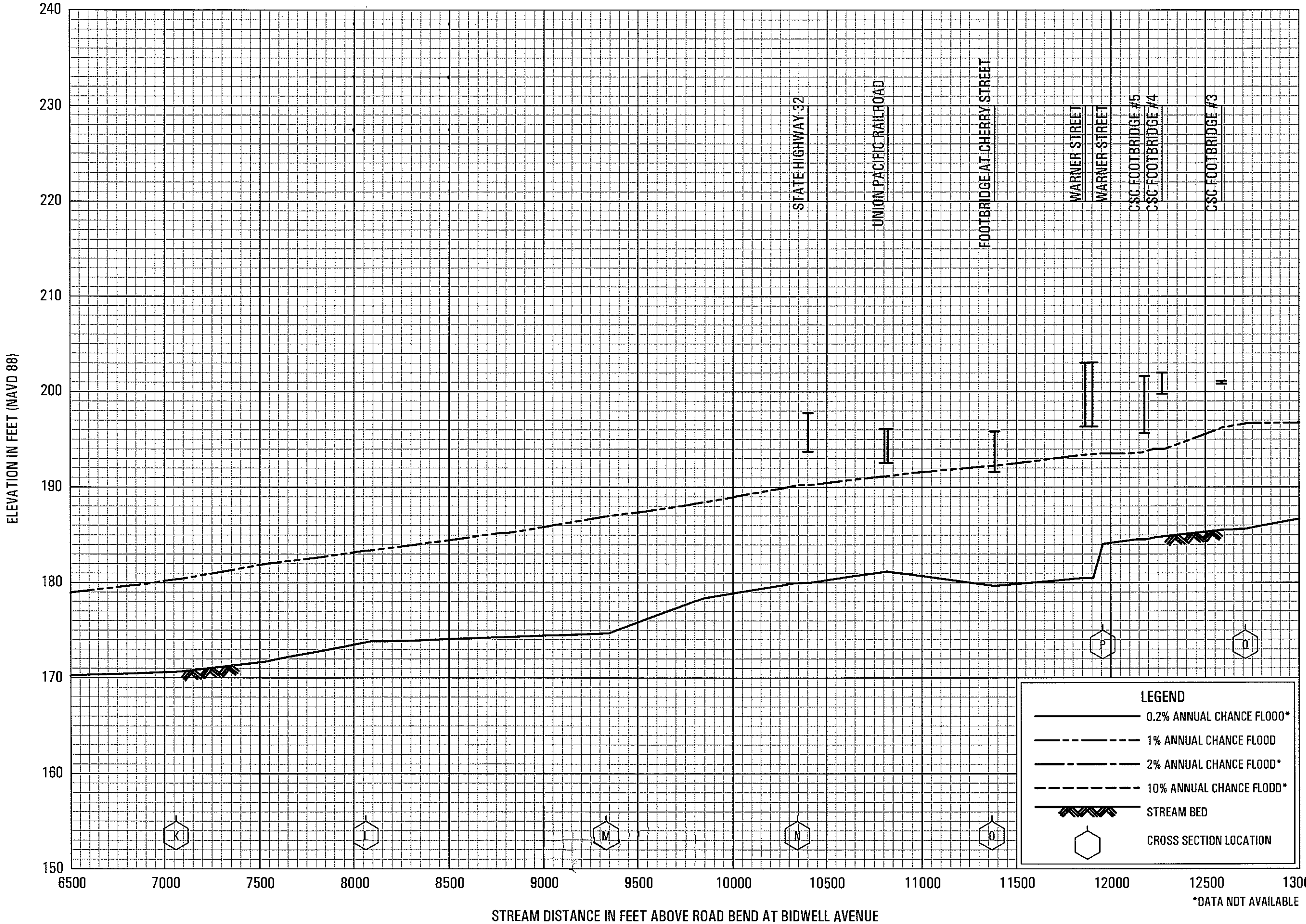
BIG CHICO CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

01P



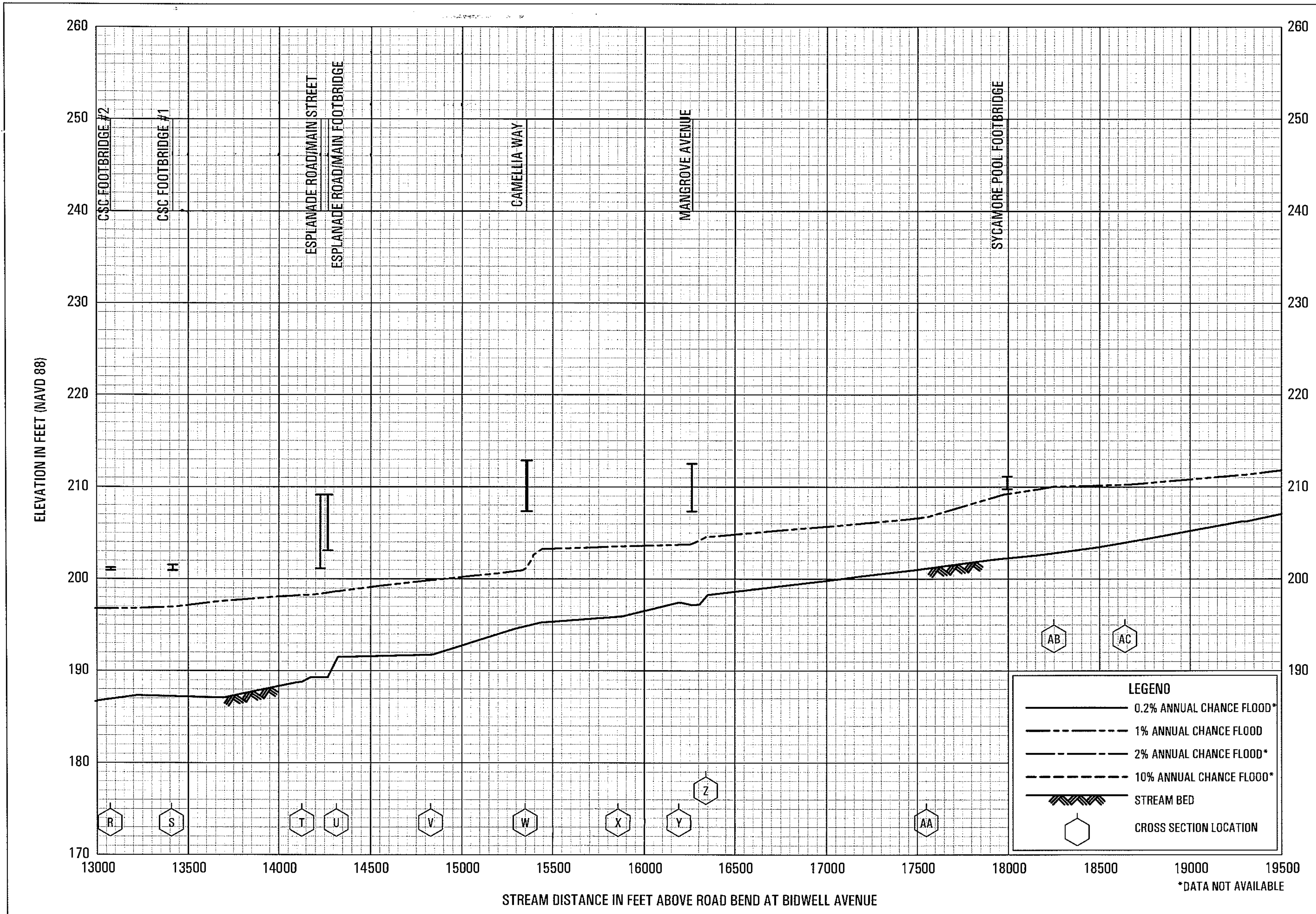
FLOOD PROFILES

BIG CHICO CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA
AND INCORPORATED AREAS

02P



FLOOD PROFILES

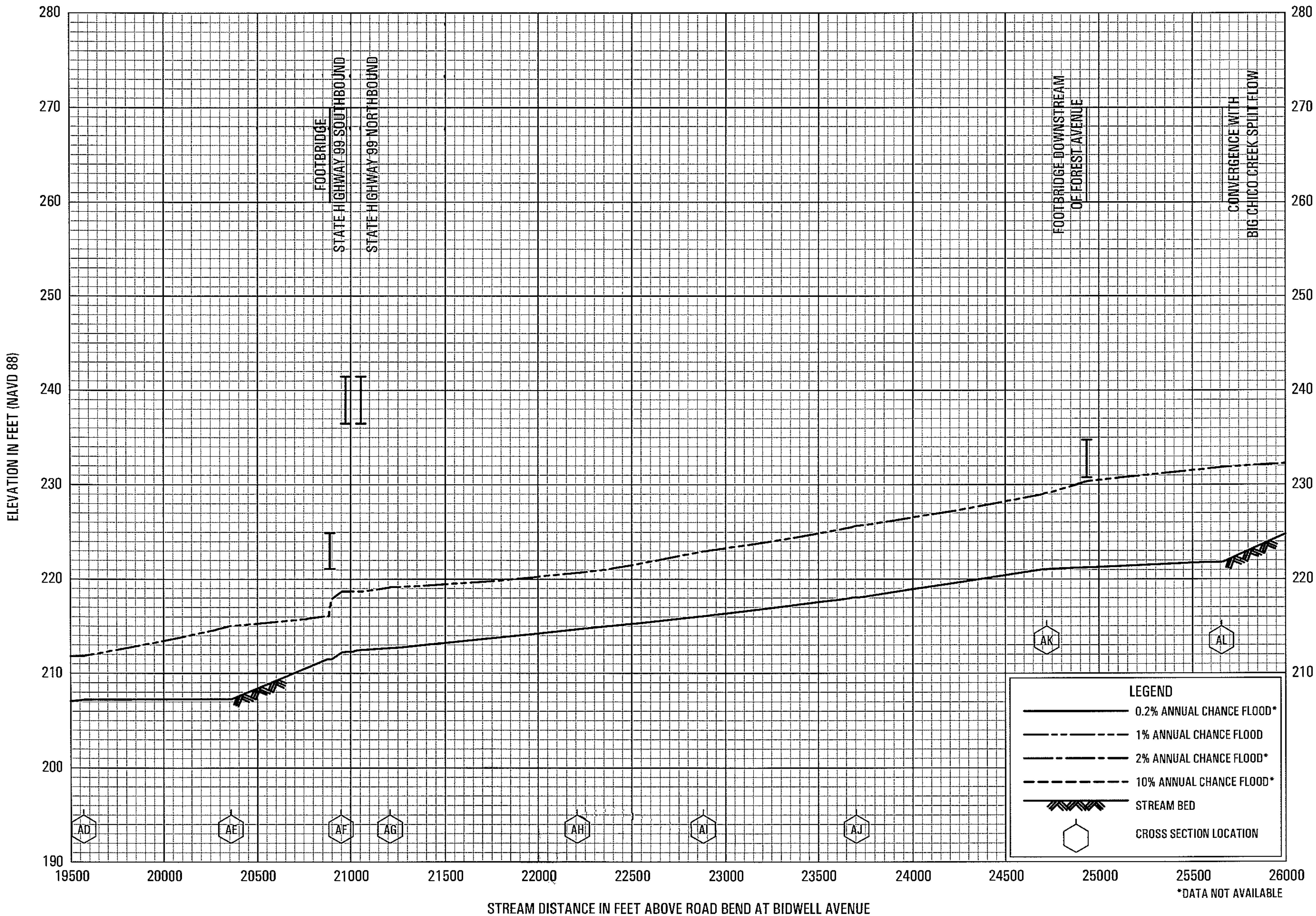
BIG CHICO CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

03P



FLOOD PROFILES

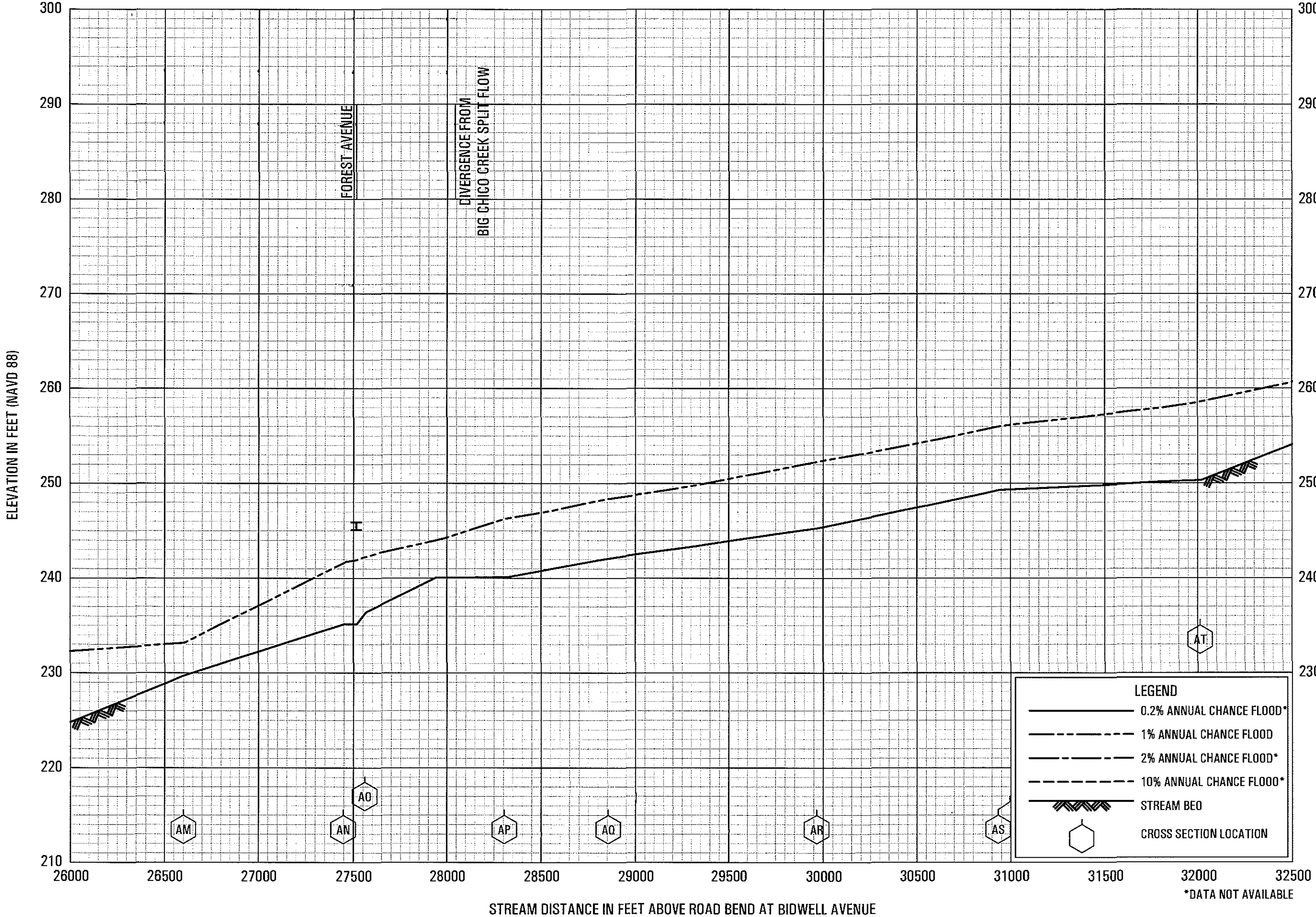
BIG CHICO CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

04P



FLOOD PROFILES

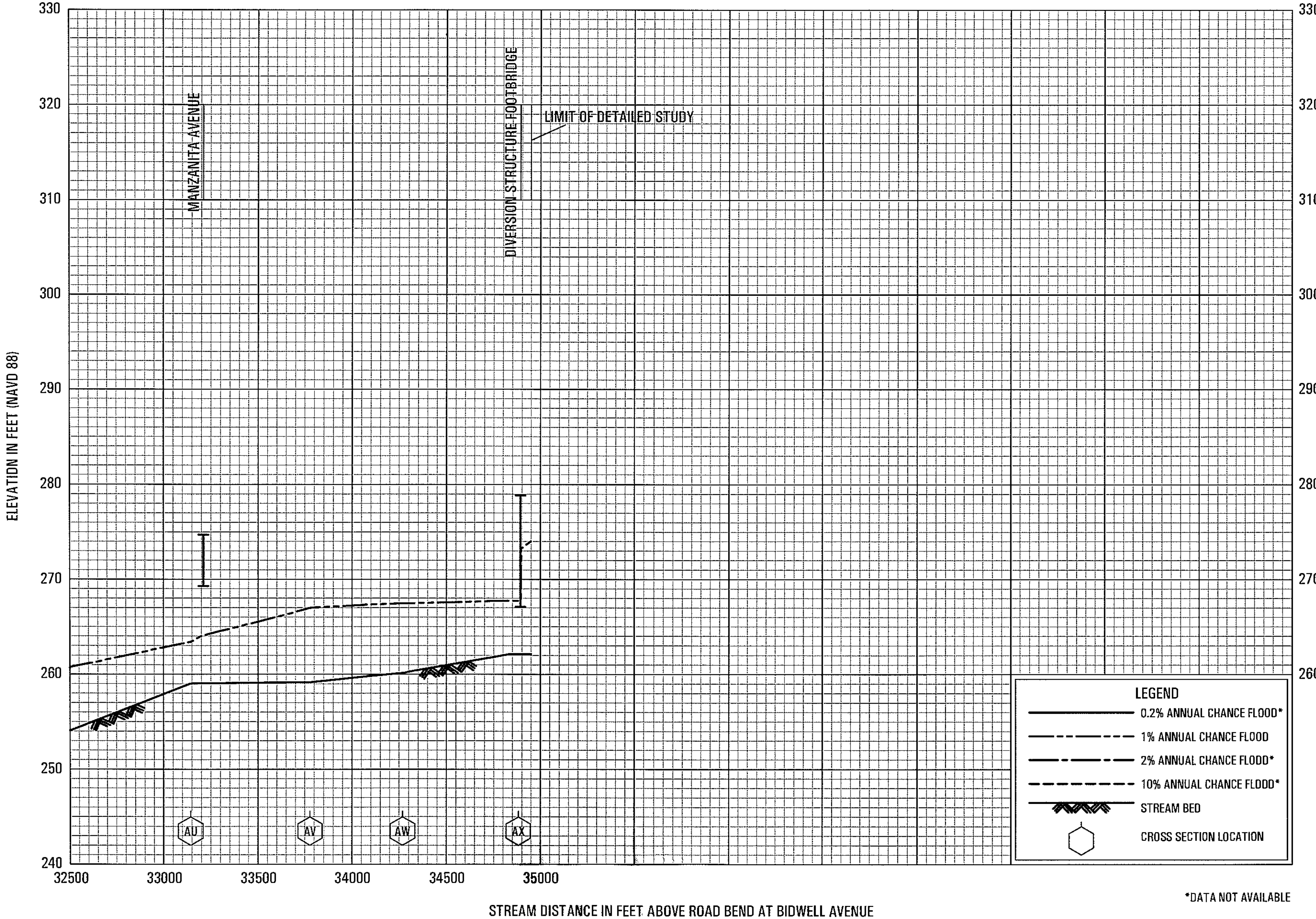
BIG CHICO CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

05P



FLOOD PROFILES

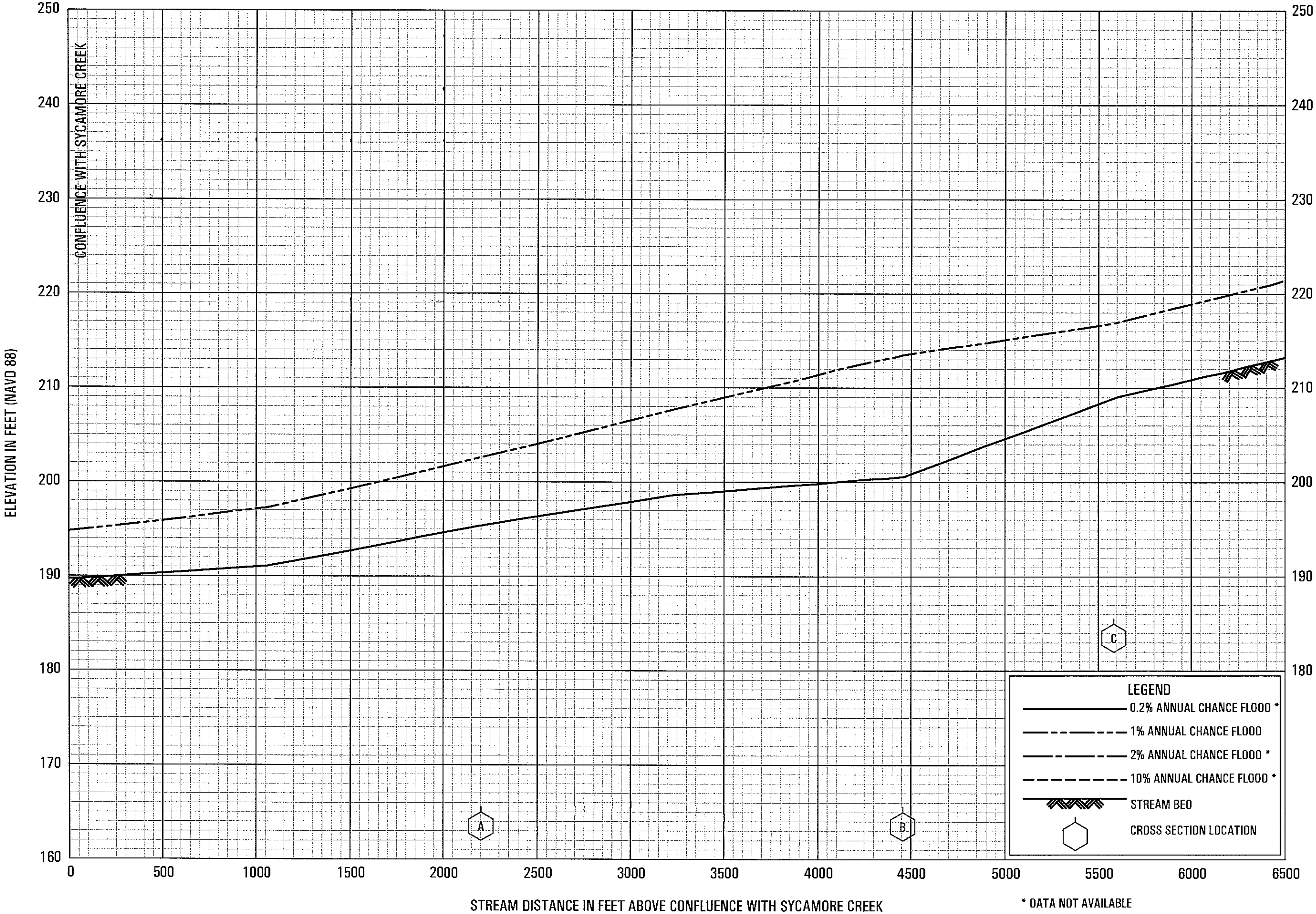
BIG CHICO CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

06P



FLOOD PROFILES

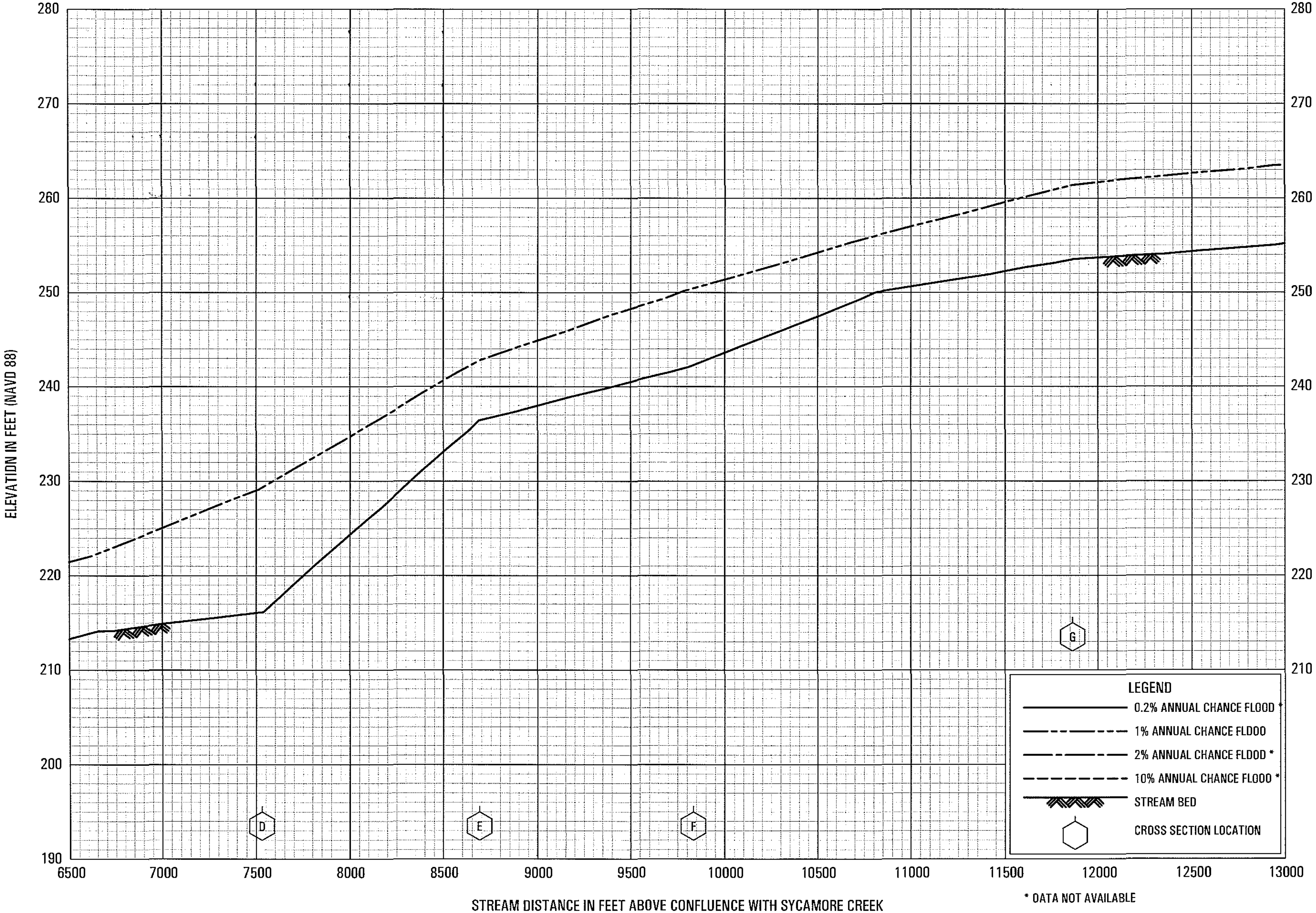
BIG CHICO CREEK DIVERSION CHANNEL

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

07P



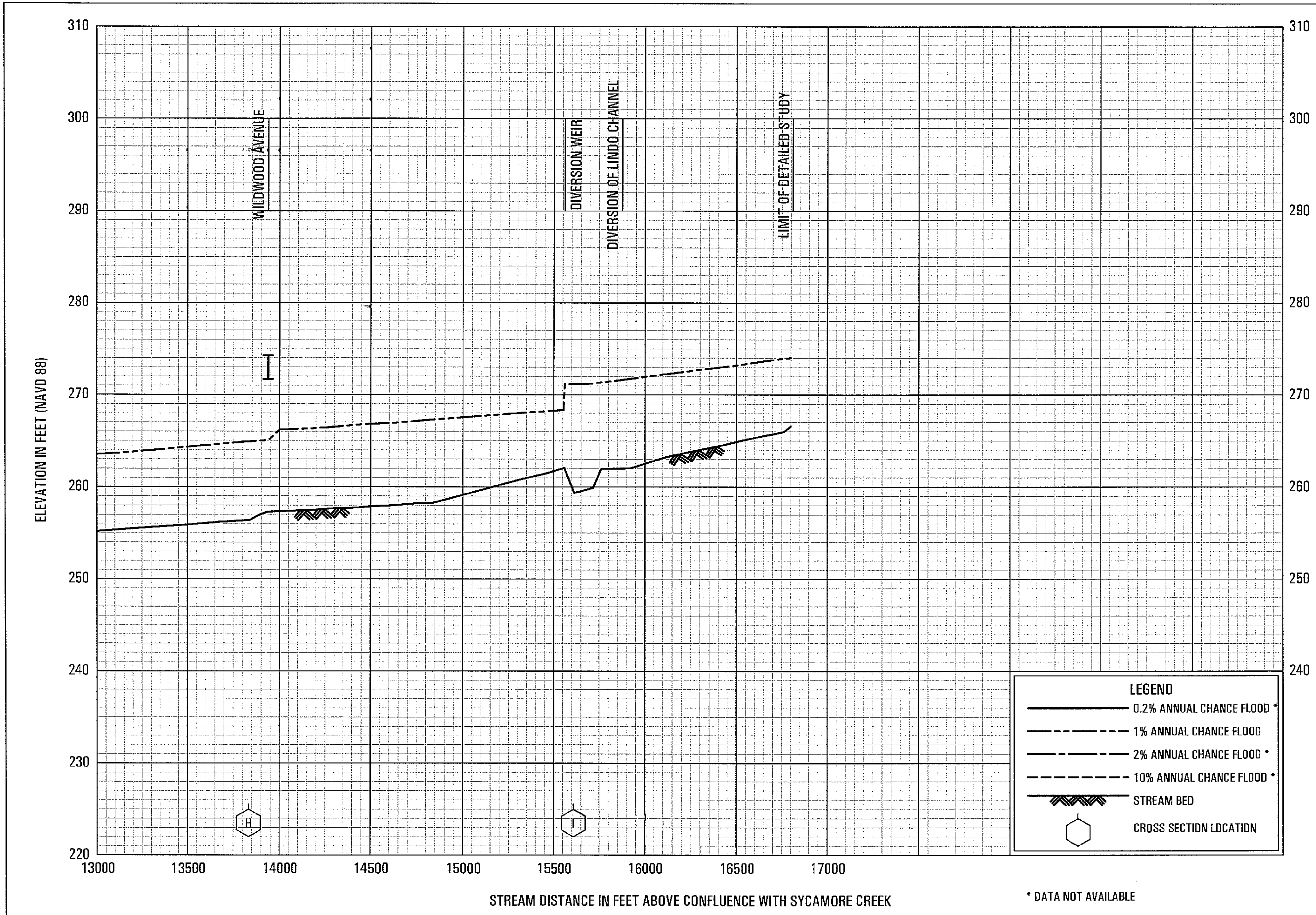
FLOOD PROFILES

BIG CHICO CREEK DIVERSION CHANNEL

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA
AND INCORPORATED AREAS

08P



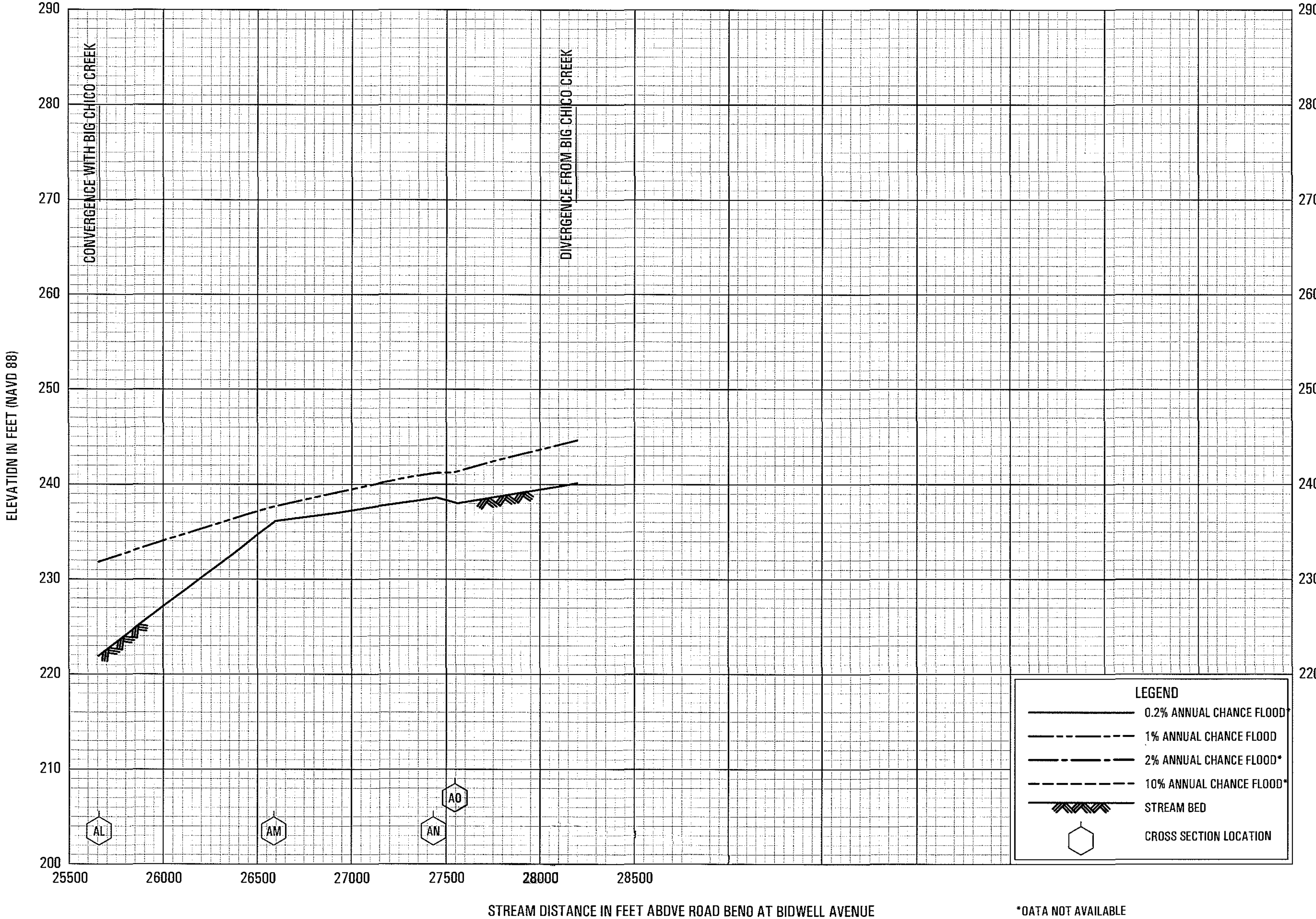
FLOOD PROFILES

BIG CHICO CREEK DIVERSION CHANNEL

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA
AND INCORPORATED AREAS

09P



FLOOD PROFILES

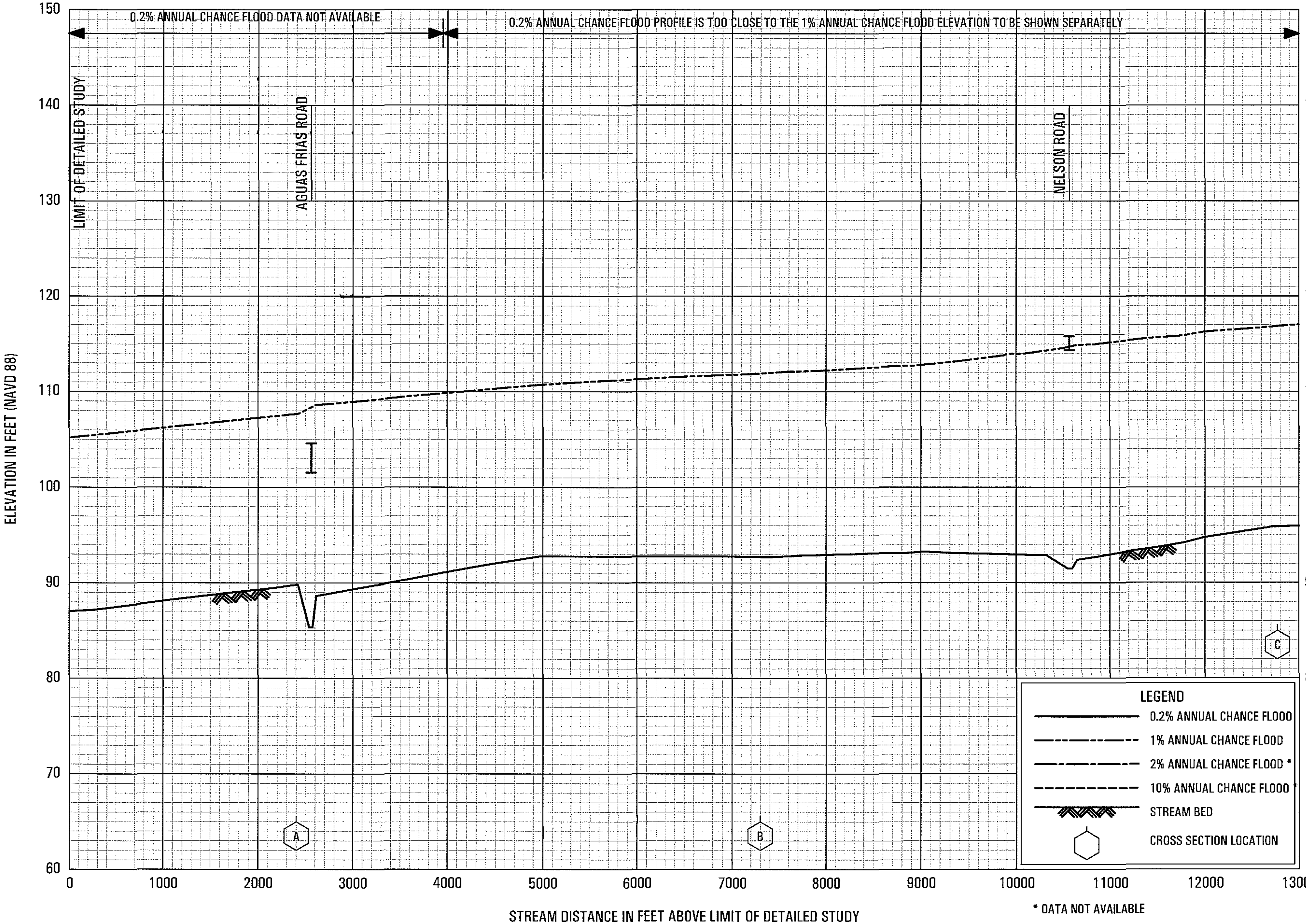
BIG CHICO CREEK SPLIT FLOW

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

10P

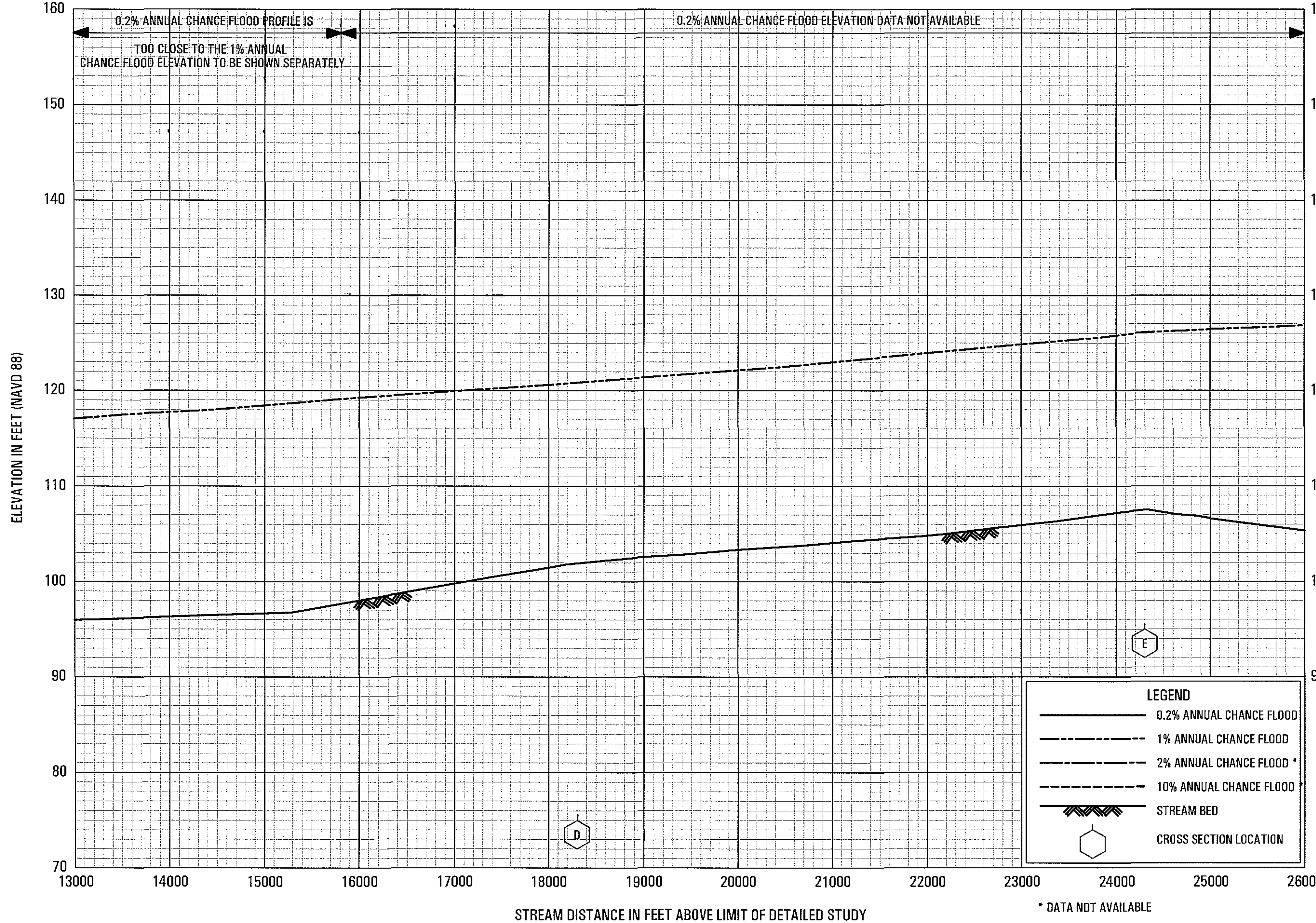


FLOOD PROFILES

BUTTE CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA
AND INCORPORATED AREAS



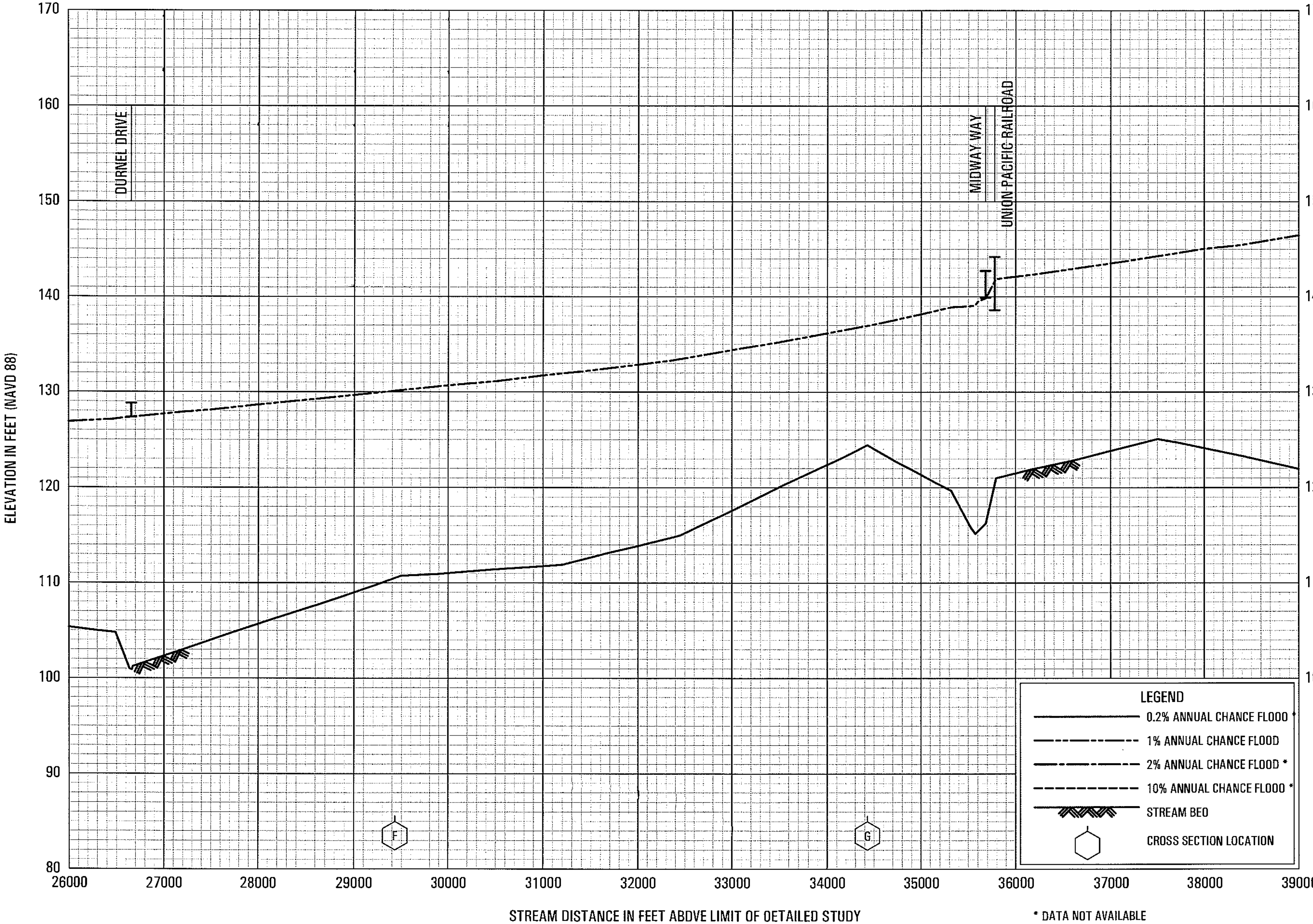
FLOOD PROFILES

BUTTE CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA
AND INCORPORATED AREAS

12P



FLOOD PROFILES

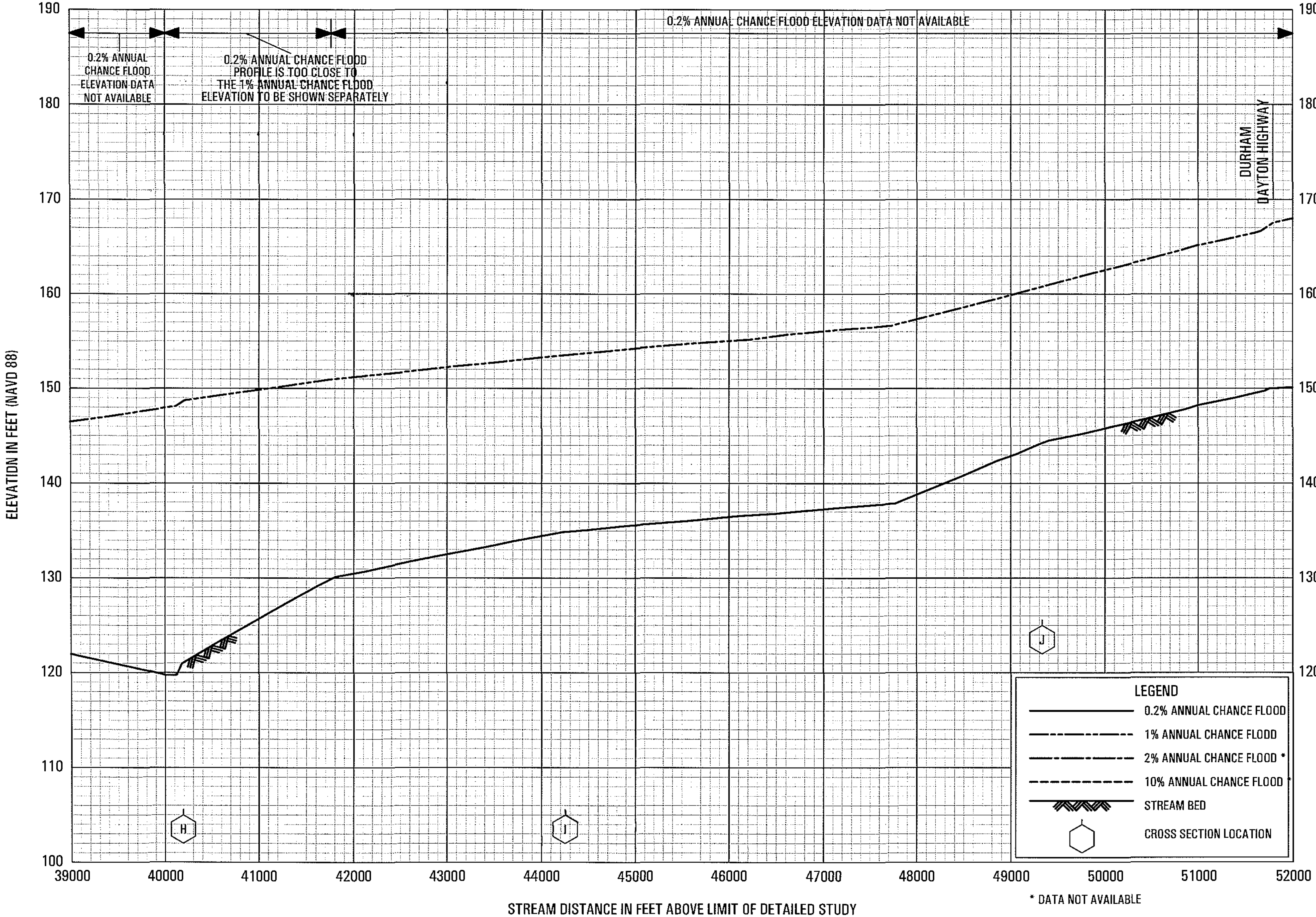
BUTTE CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

13P



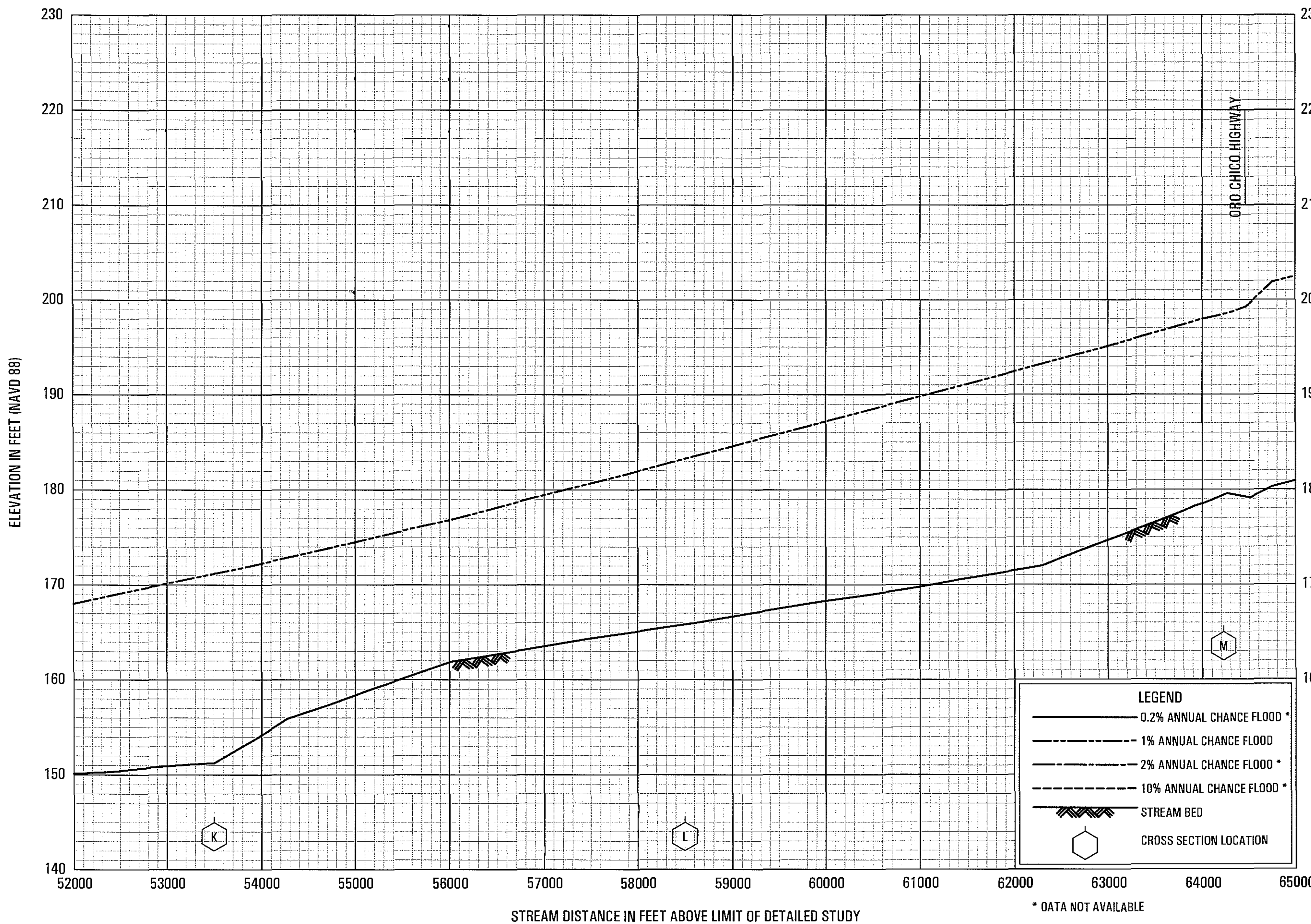
FLOOD PROFILES

BUTTE CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



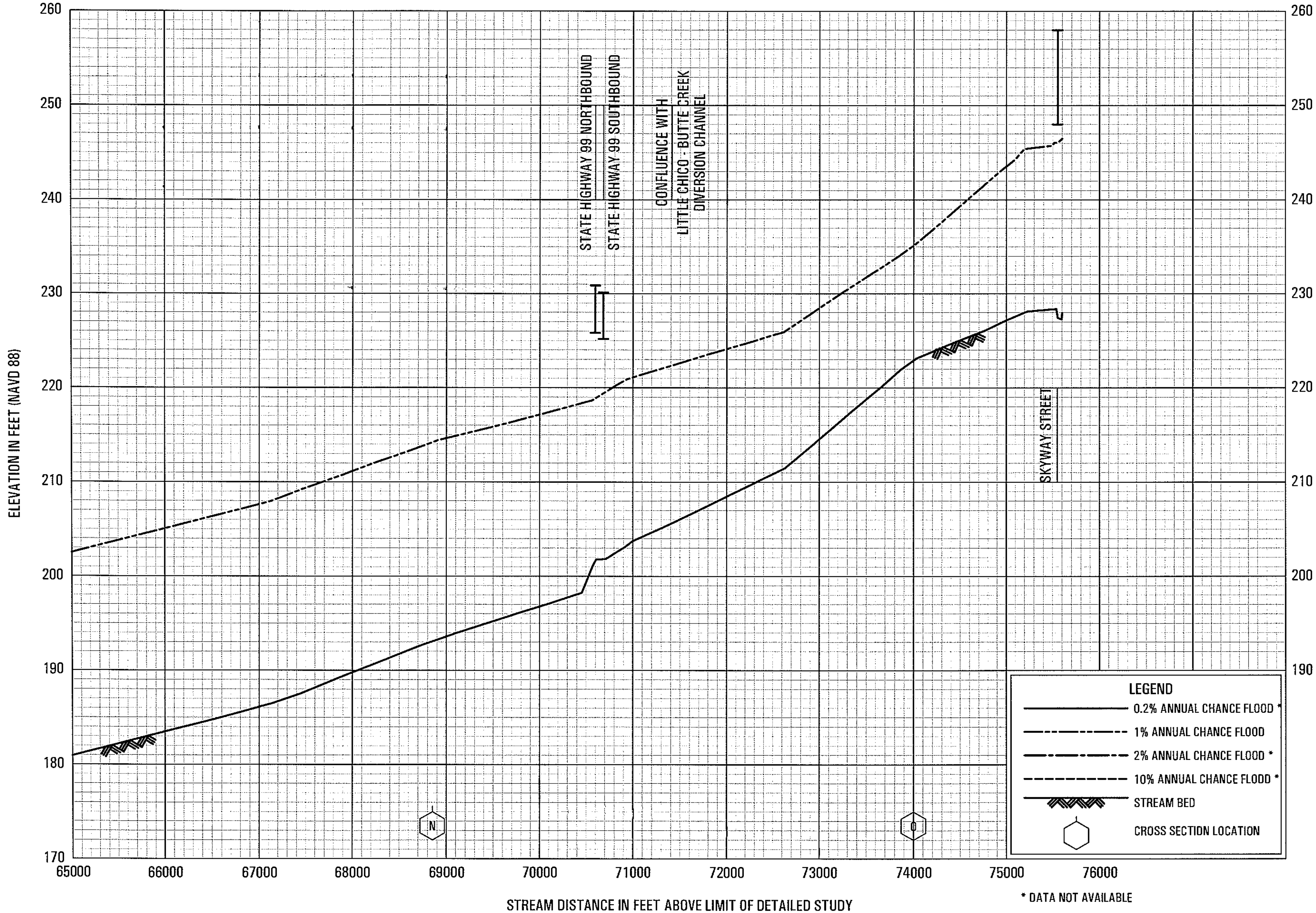
FLOOD PROFILES

BUTTE CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



FLOOD PROFILES

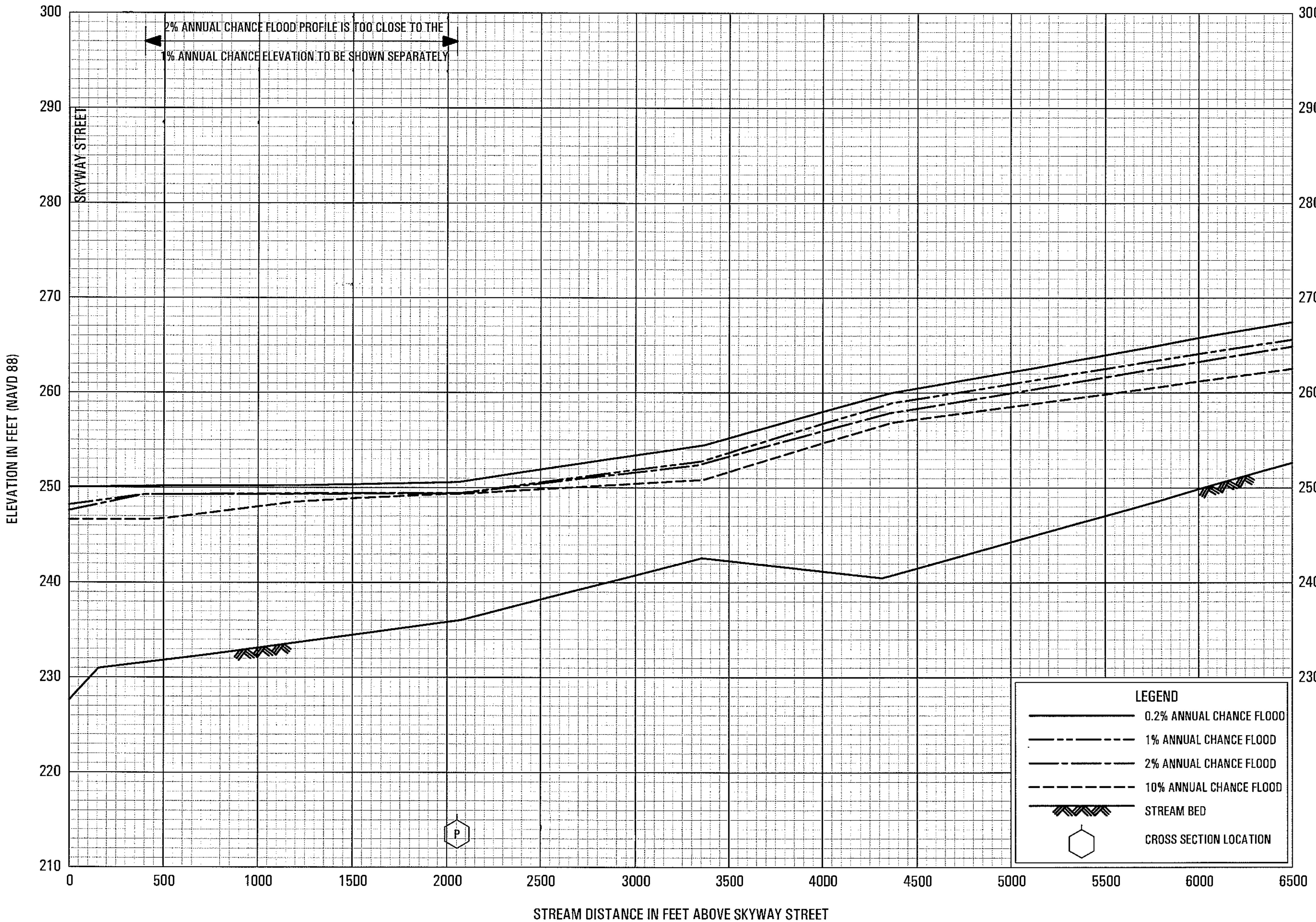
BUTTE CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

16P



FLOOD PROFILES

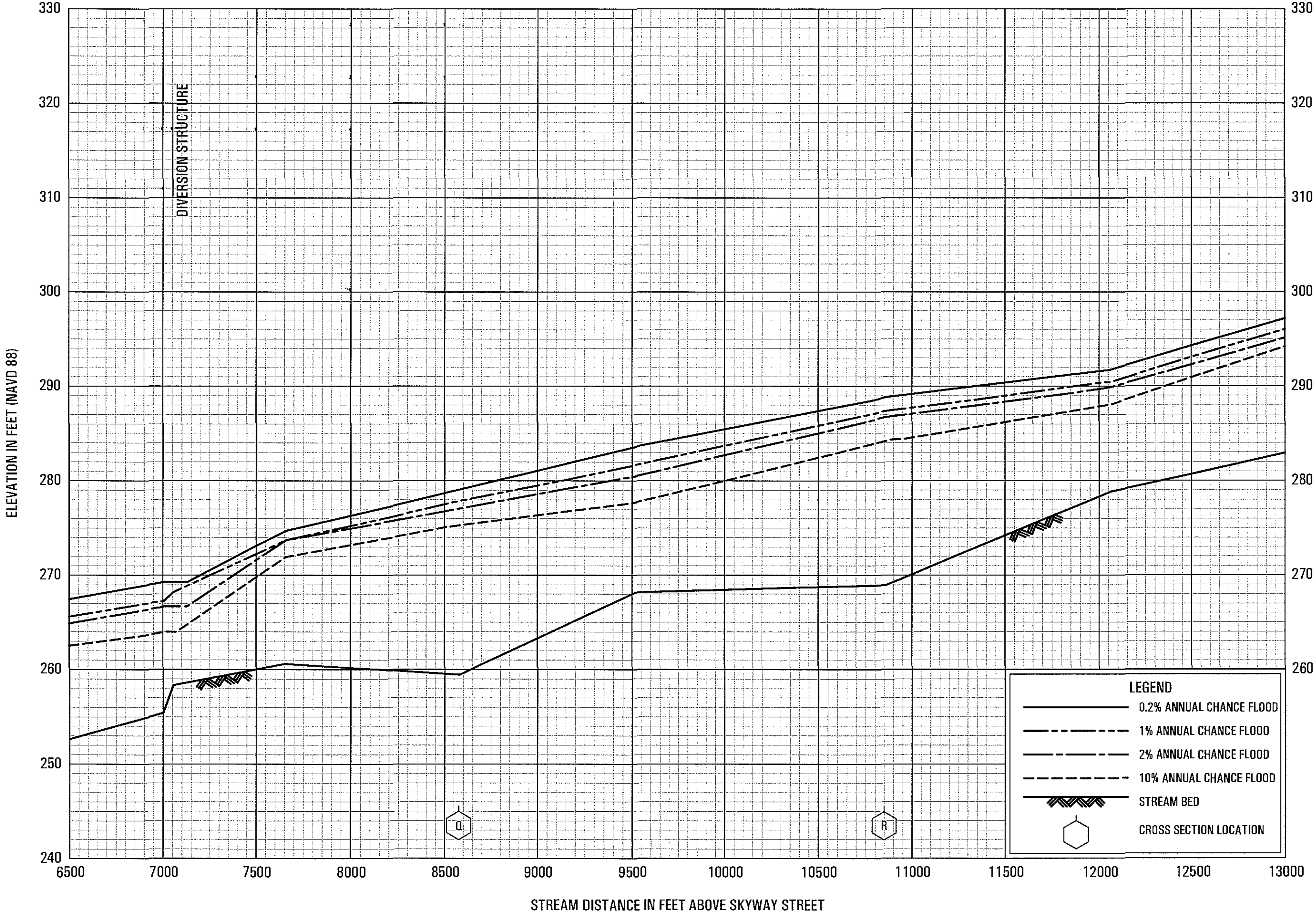
BUTTE CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

17P



FLOOD PROFILES

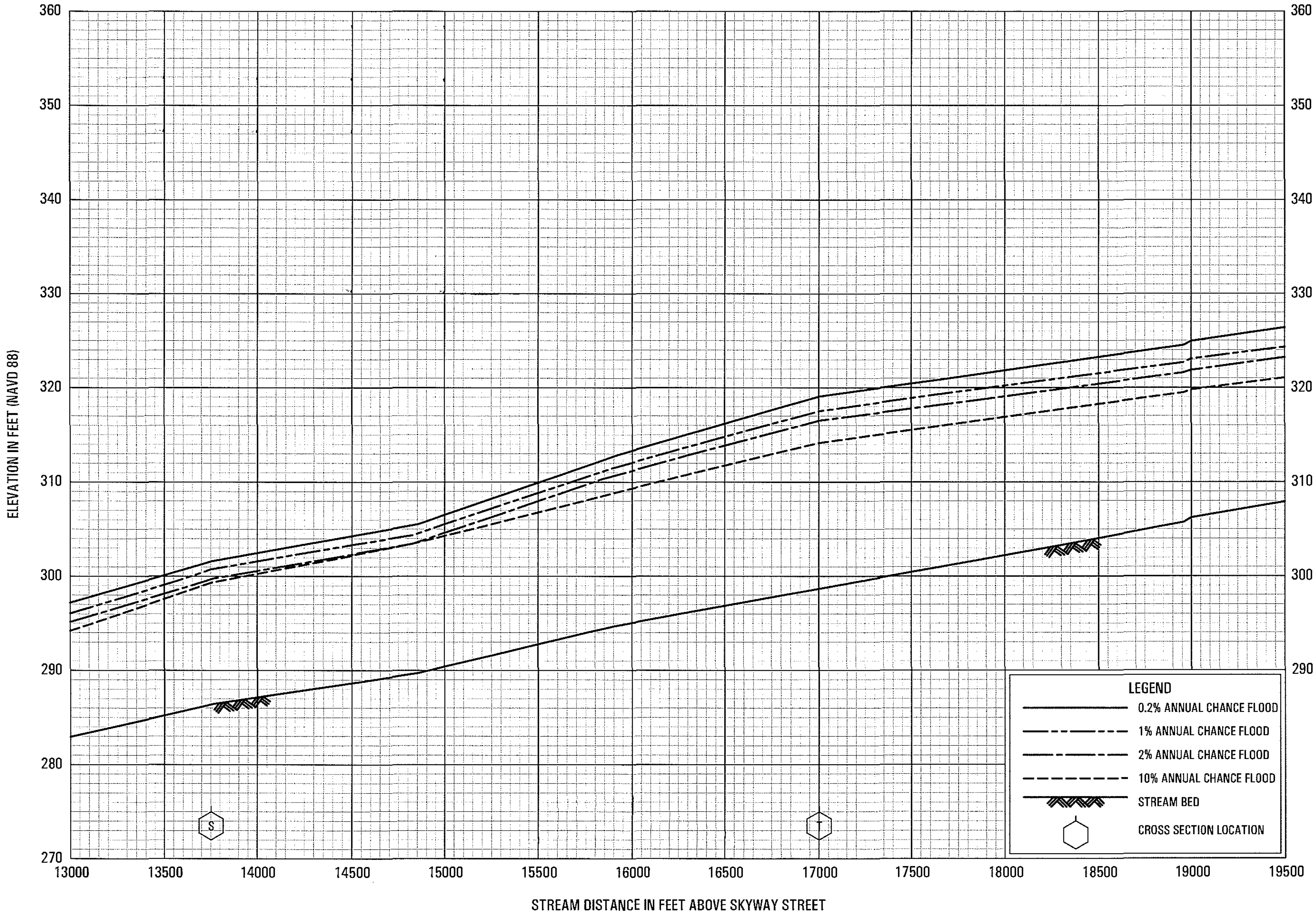
BUTTE CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

18P



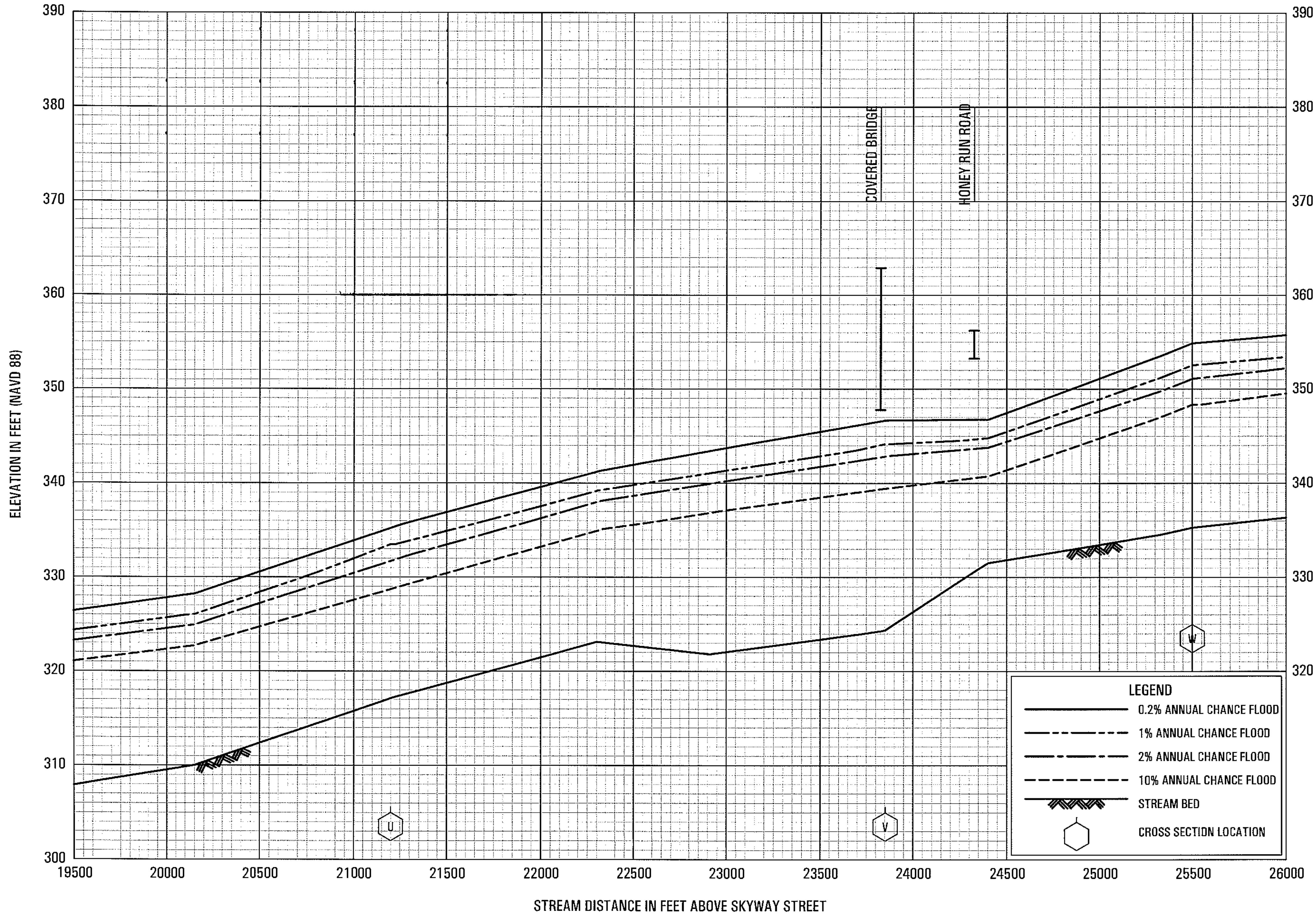
FLOOD PROFILES

BUTTE CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



FLOOD PROFILES

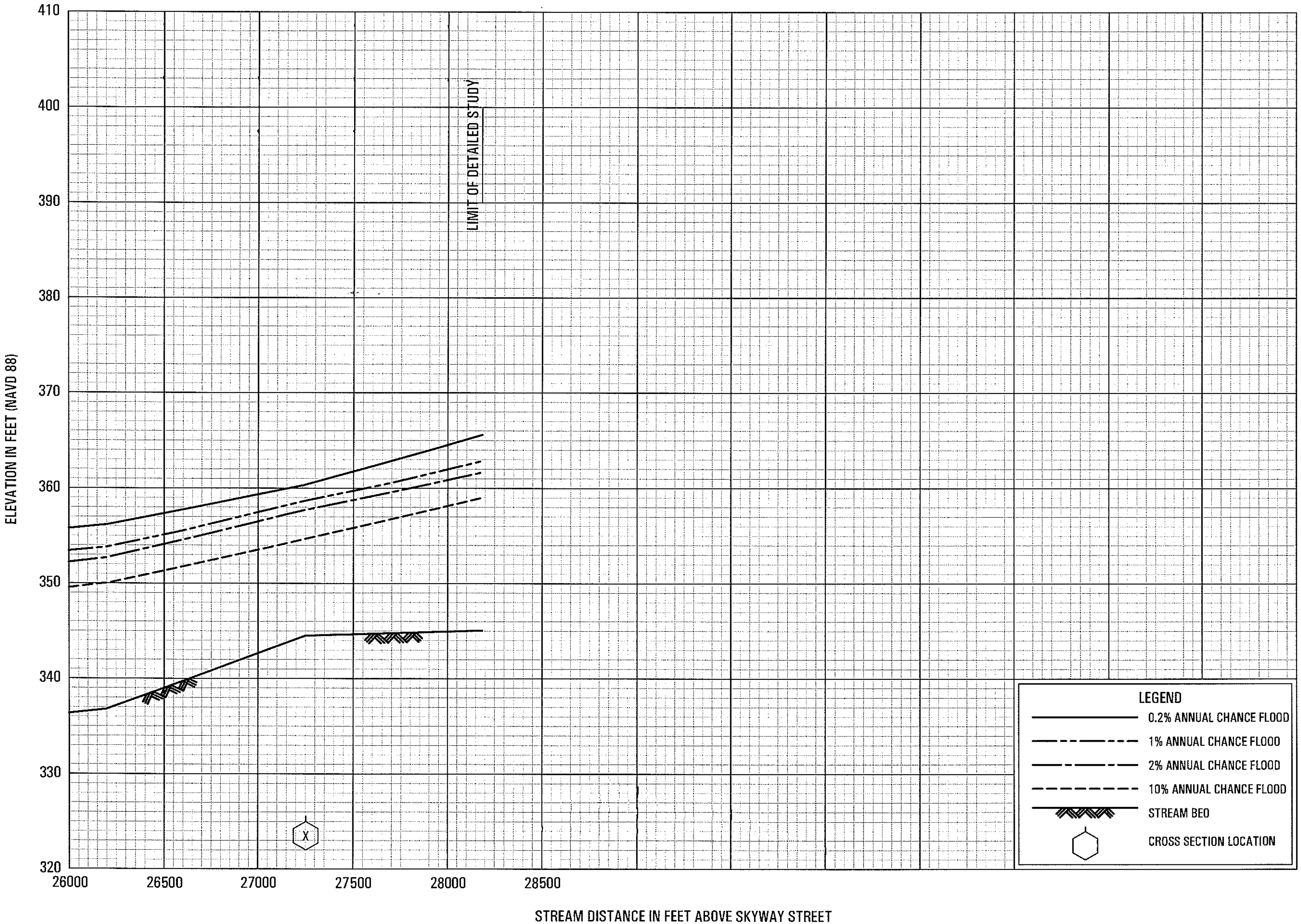
BUTTE CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

20P



FLOOD PROFILES

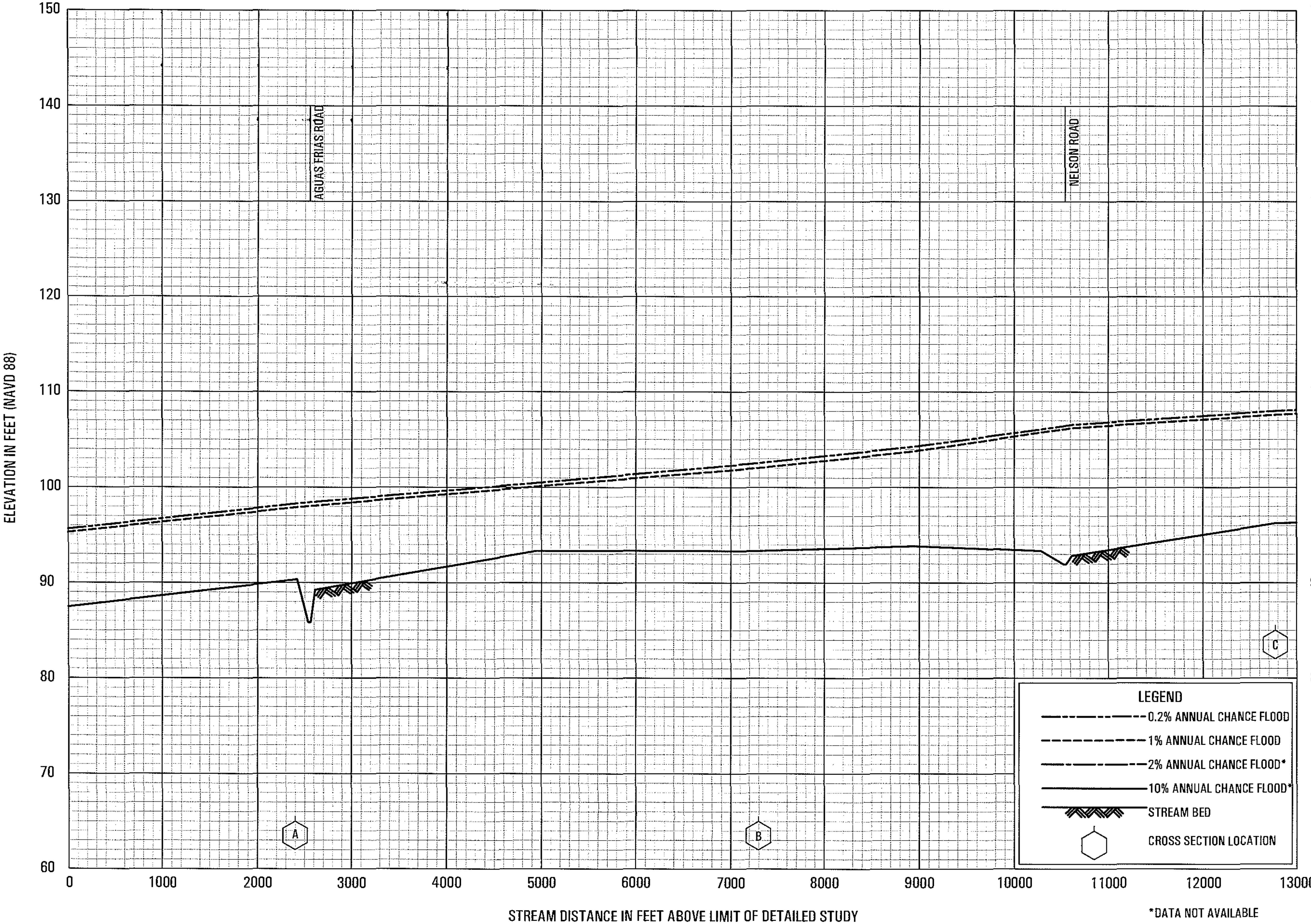
BUTTE CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

21P



FLOOD PROFILES

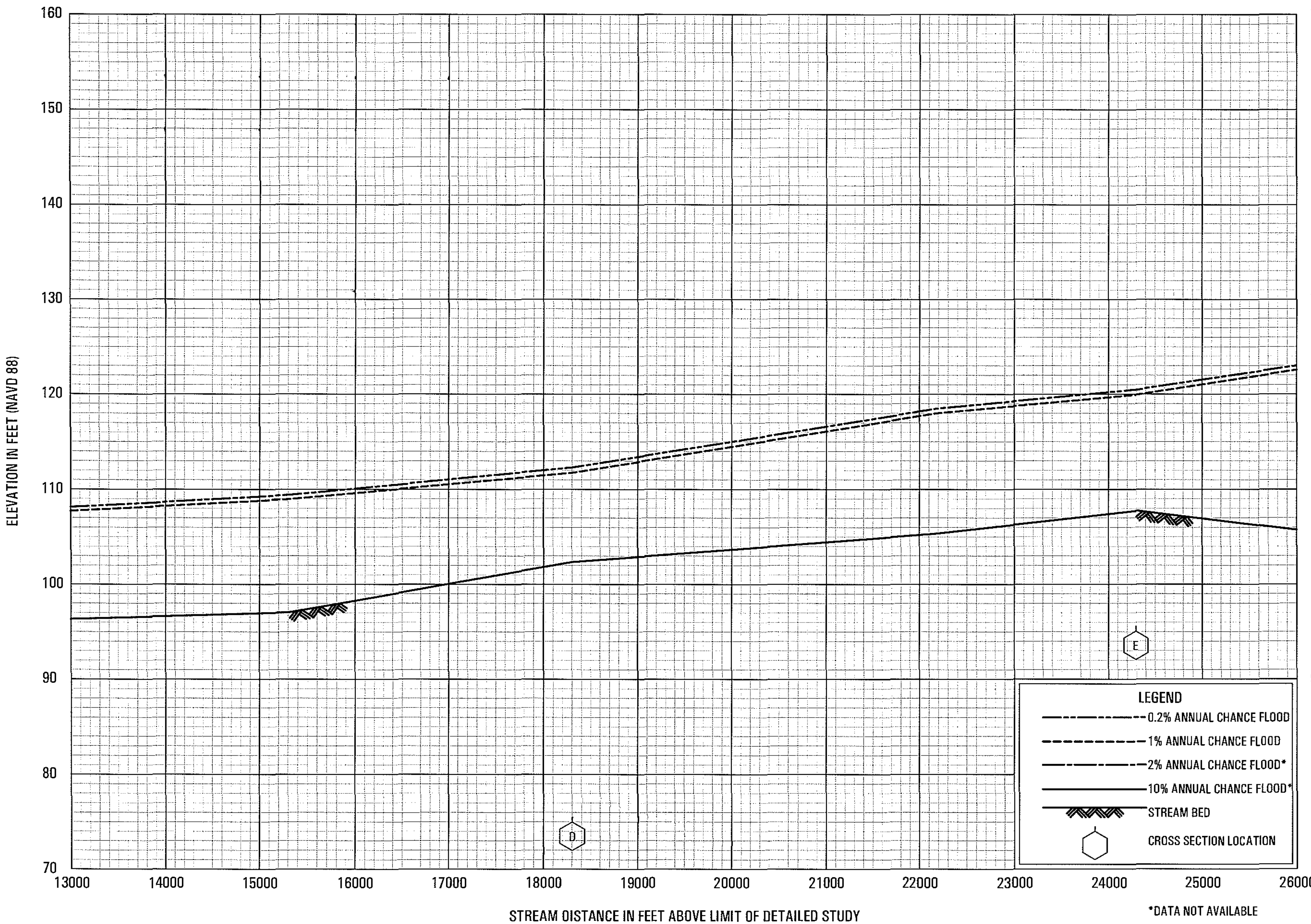
BUTTE CREEK LEFT LEVEE FAILED

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

21aP



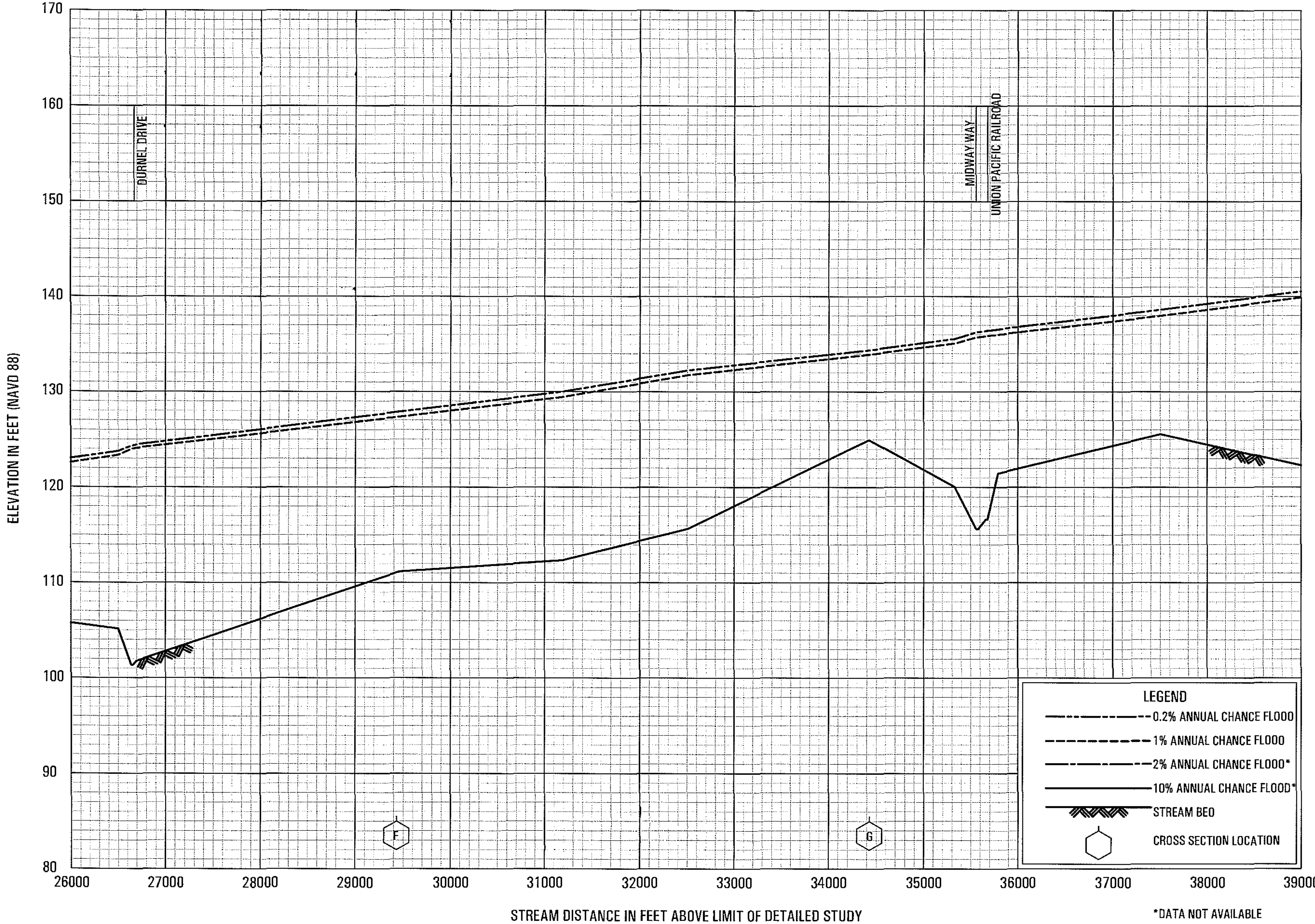
FLOOD PROFILES

BUTTE CREEK LEFT LEVEE FAILED

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



FLOOD PROFILES

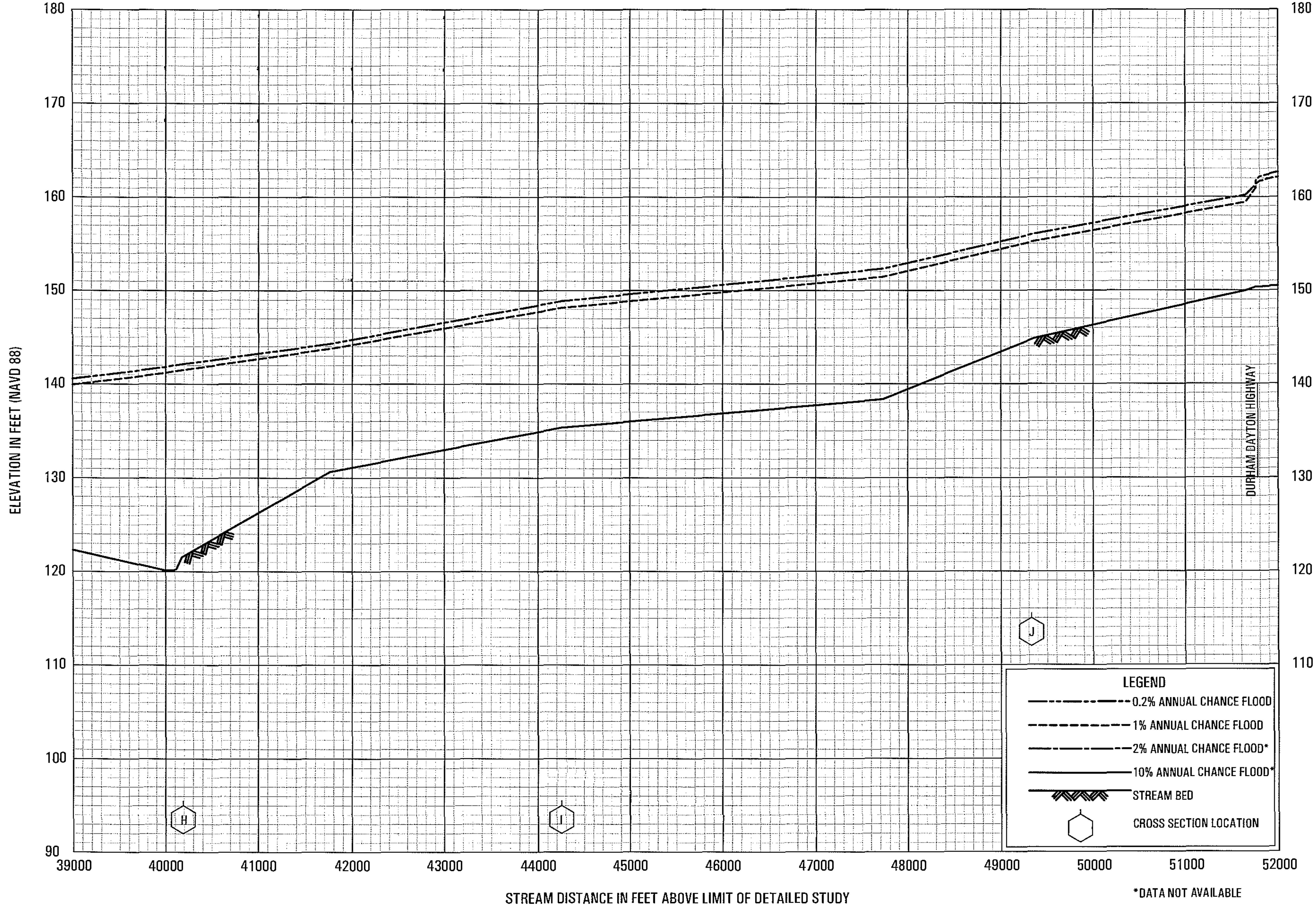
BUTTE CREEK LEFT LEVEE FAILED

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

21cP



FLOOD PROFILES

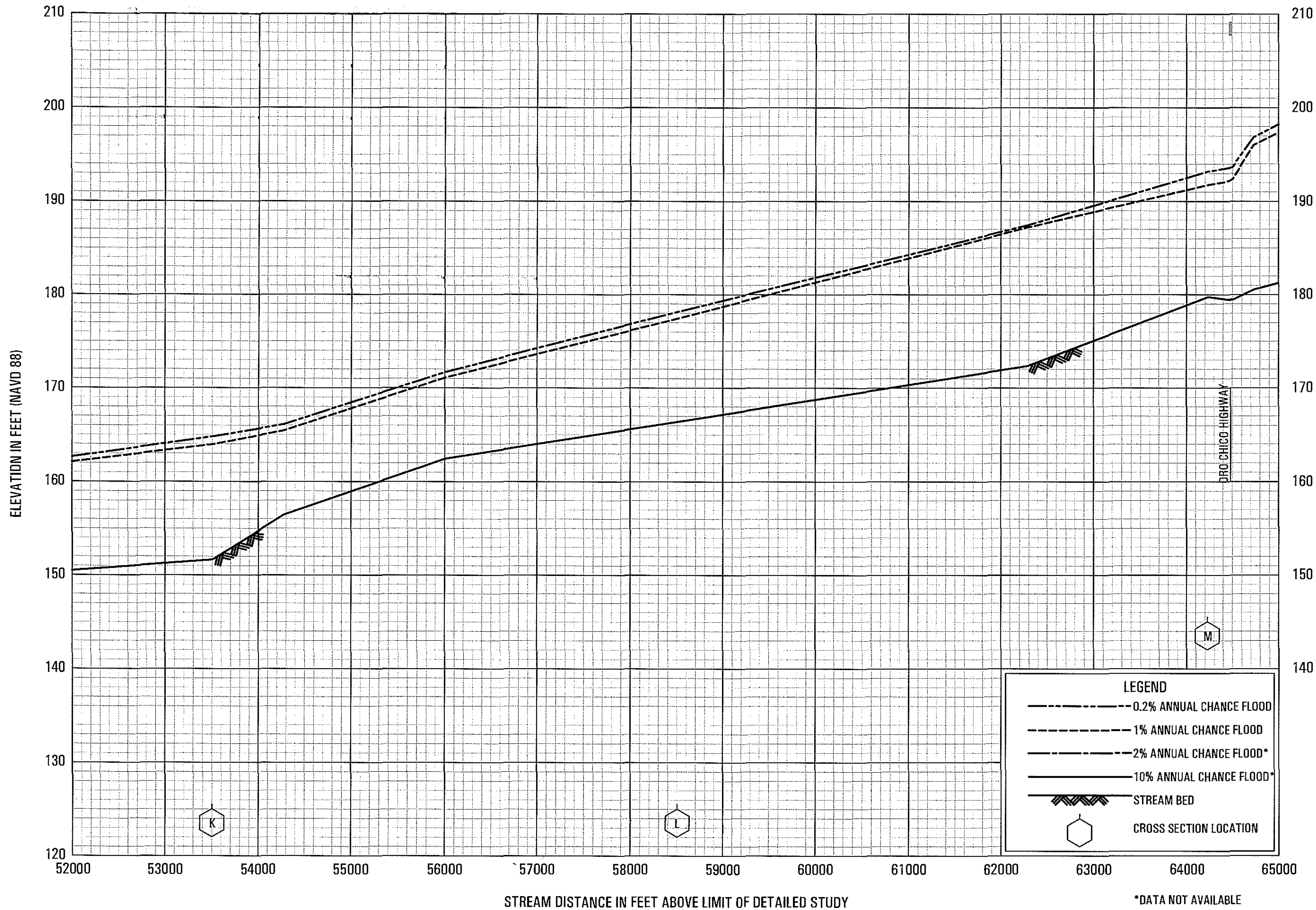
BUTTE CREEK LEFT LEVEE FAILED

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

21dP



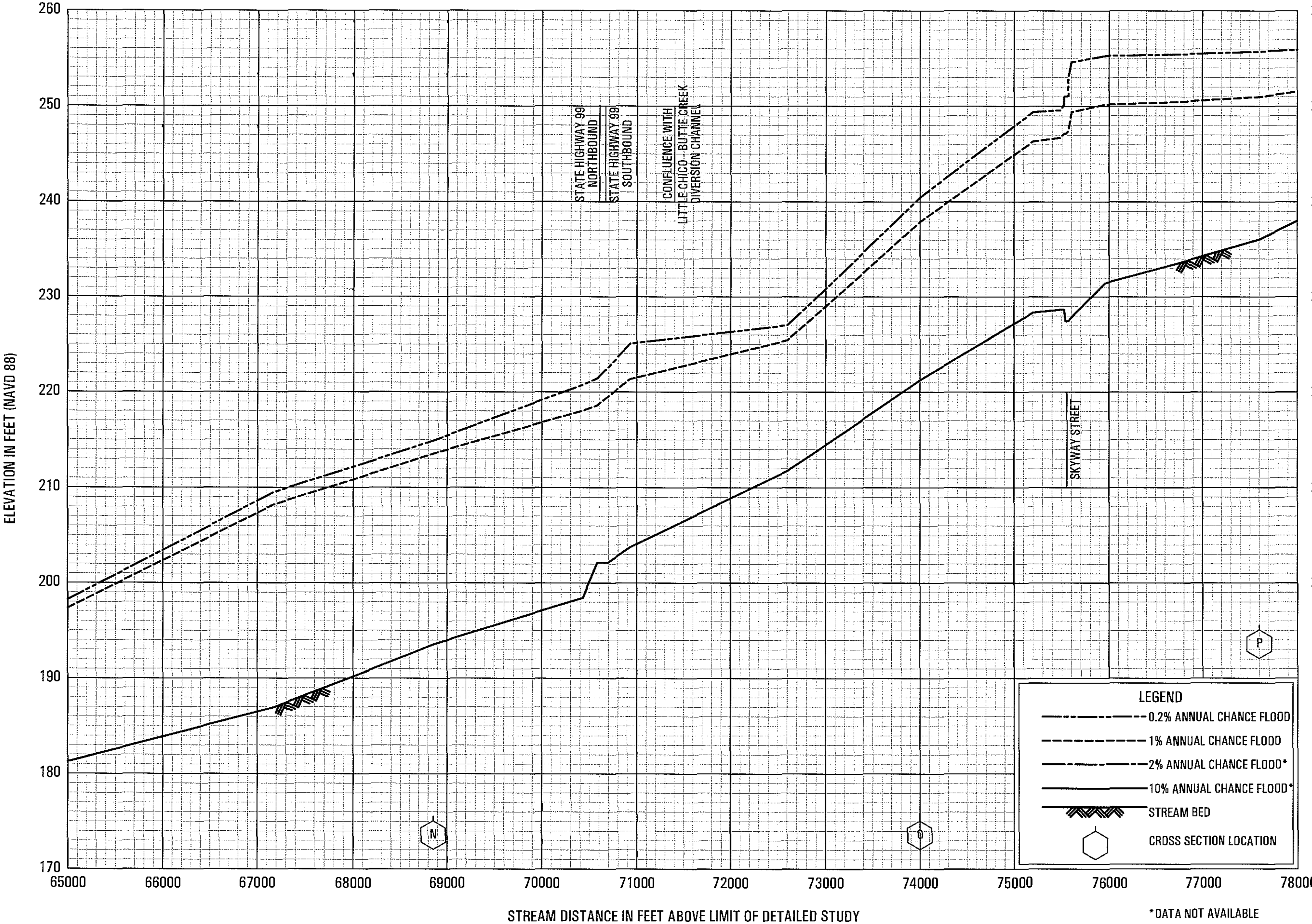
FLOOD PROFILES

BUTTE CREEK LEFT LEVEE FAILED

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE CREEK, CA

AND INCORPORATED AREAS



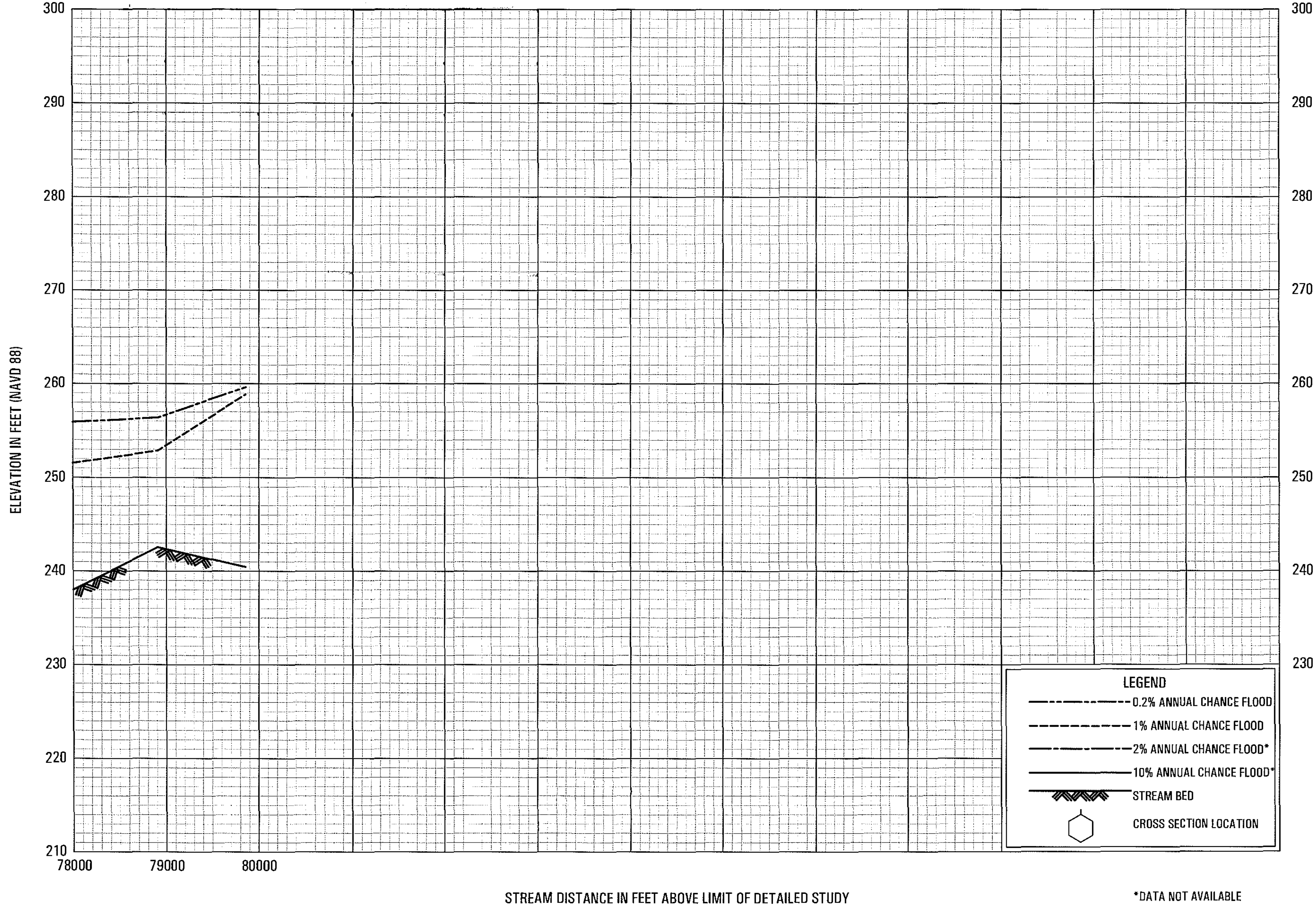
FLOOD PROFILES

BUTTE CREEK LEFT LEVEE FAILED

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



FLOOD PROFILES

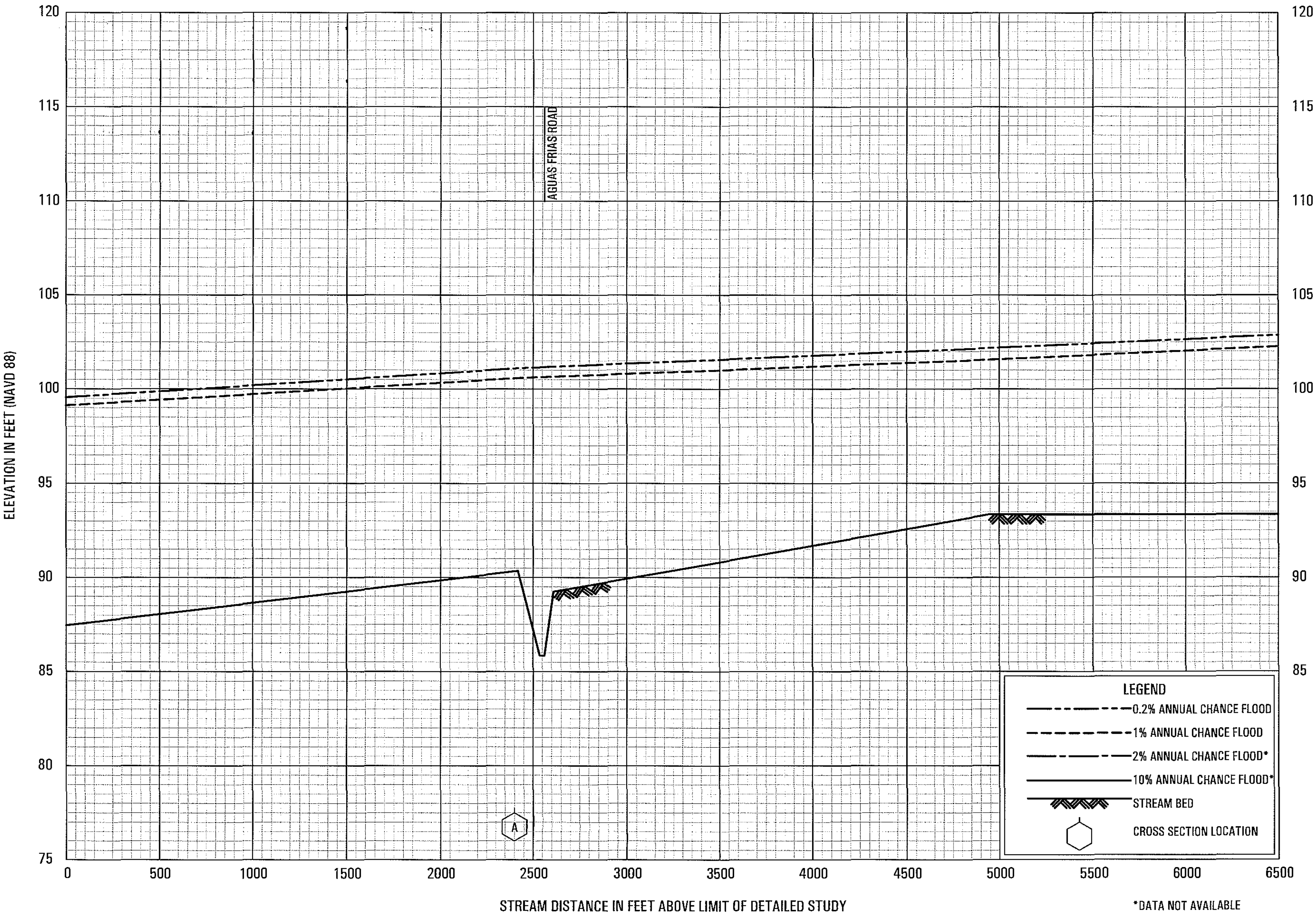
BUTTE CREEK LEFT LEVEE FAILED

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE CREEK, CA

AND INCORPORATED AREAS

21gP



FLOOD PROFILES

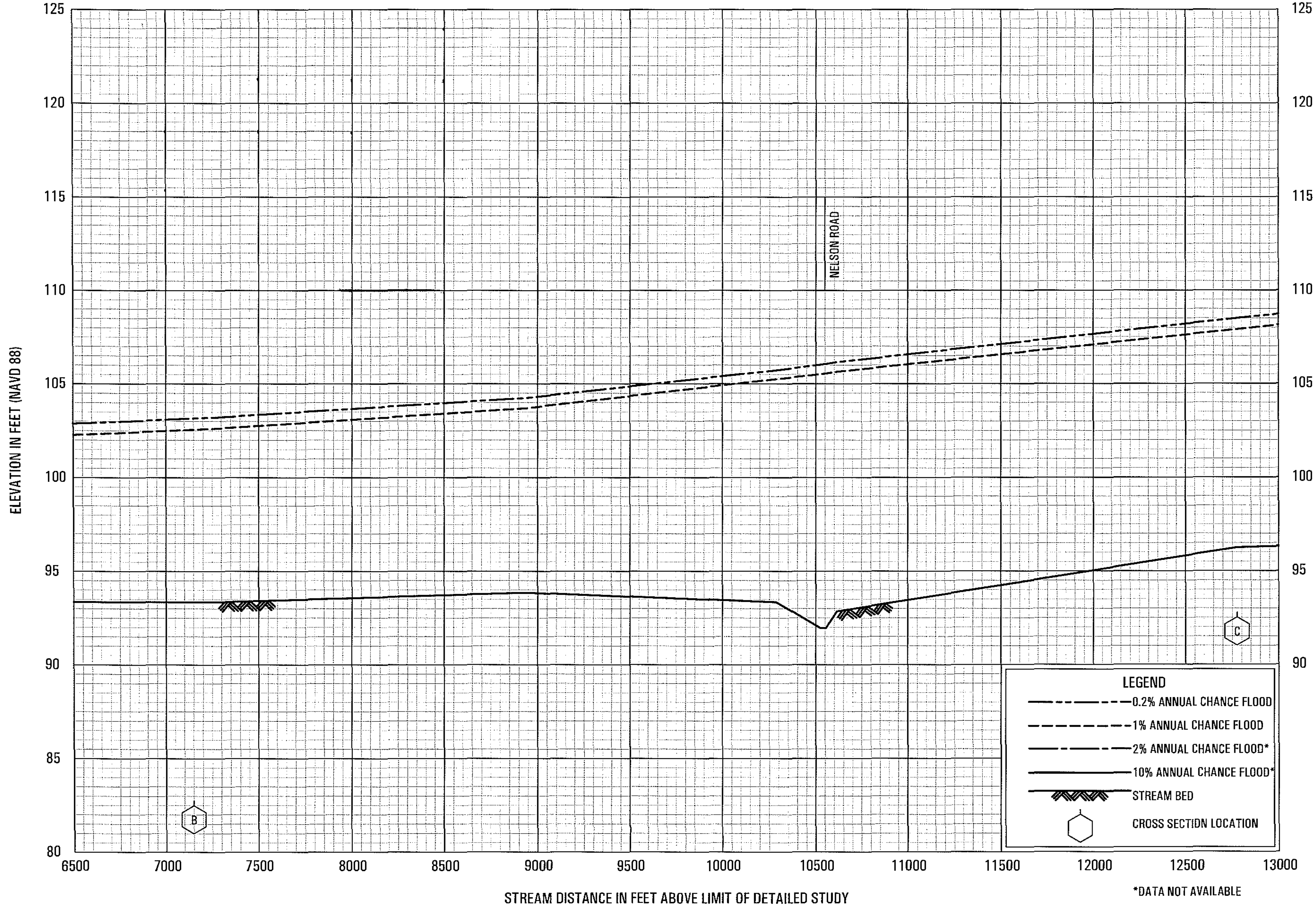
BUTTE CREEK RIGHT LEVEE FAILED - BELOW MIDWAY

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

21hP



FLOOD PROFILES

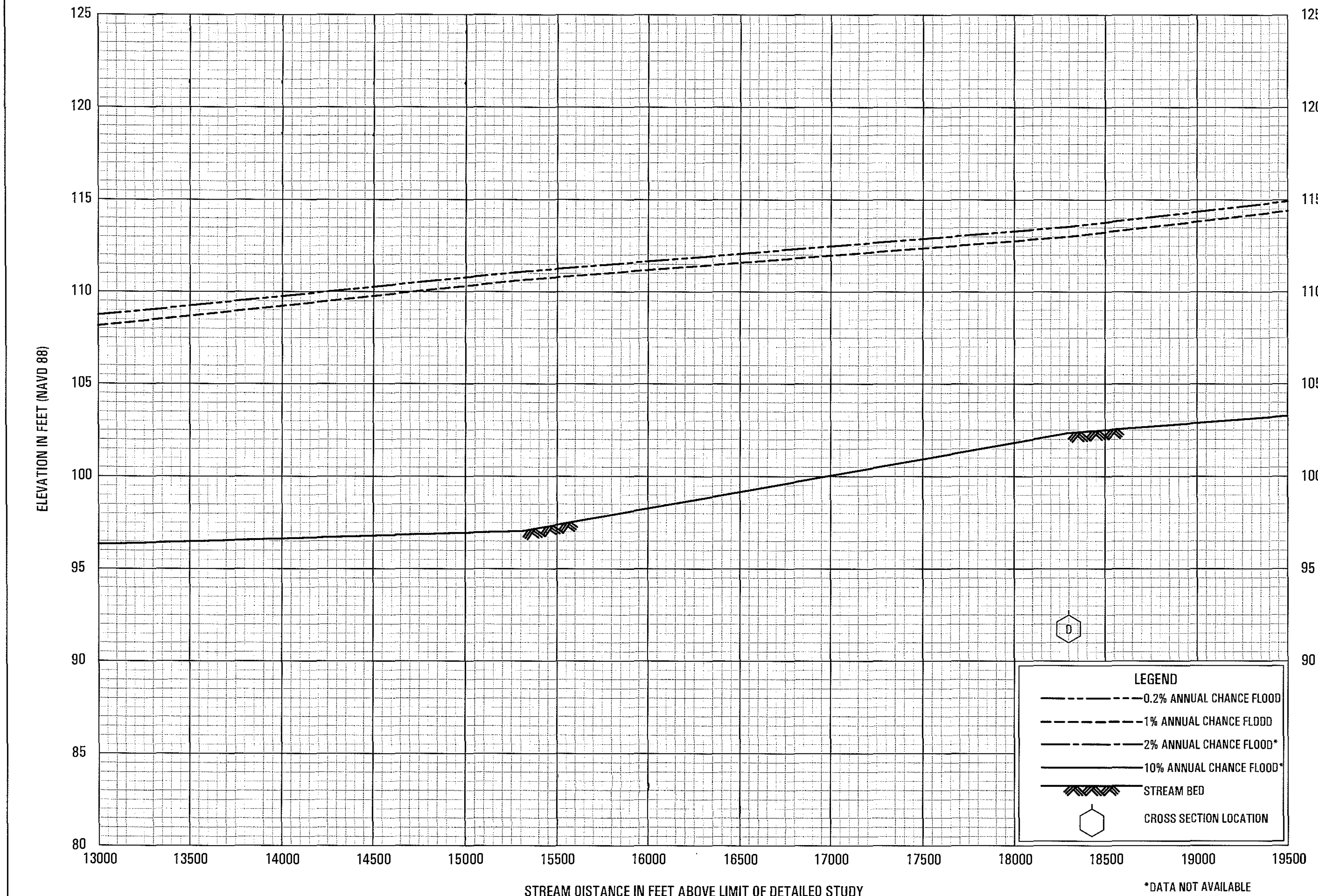
BUTTE CREEK RIGHT LEVEE FAILED - BELOW MIDWAY

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

21iP



FLOOD PROFILES

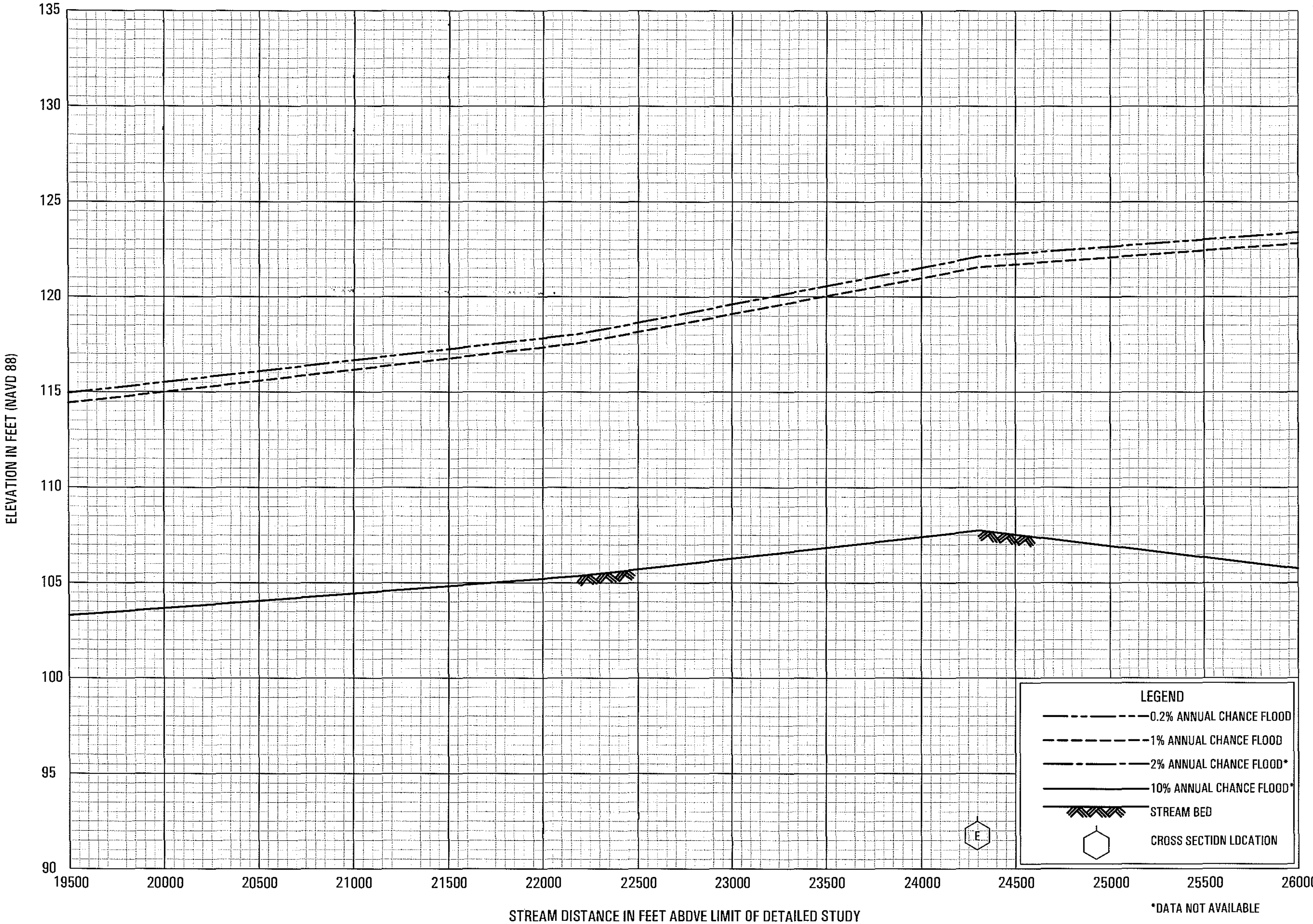
BUTTE CREEK RIGHT LEVEE FAILED - BELOW MIDWAY

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

21jP



*DATA NOT AVAILABLE

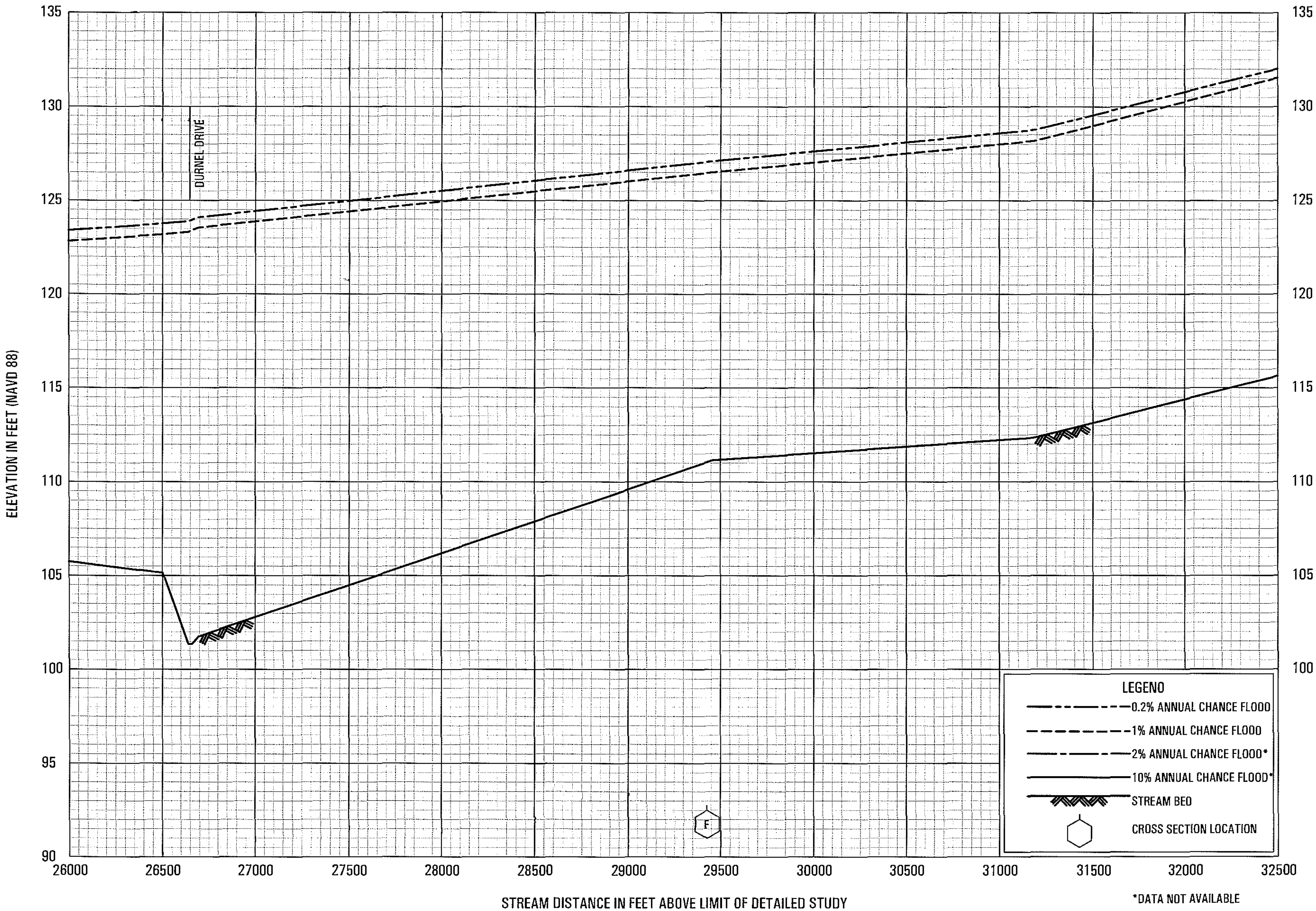
FLOOD PROFILES

BUTTE CREEK RIGHT LEVEE FAILED - BELOW MIDWAY

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



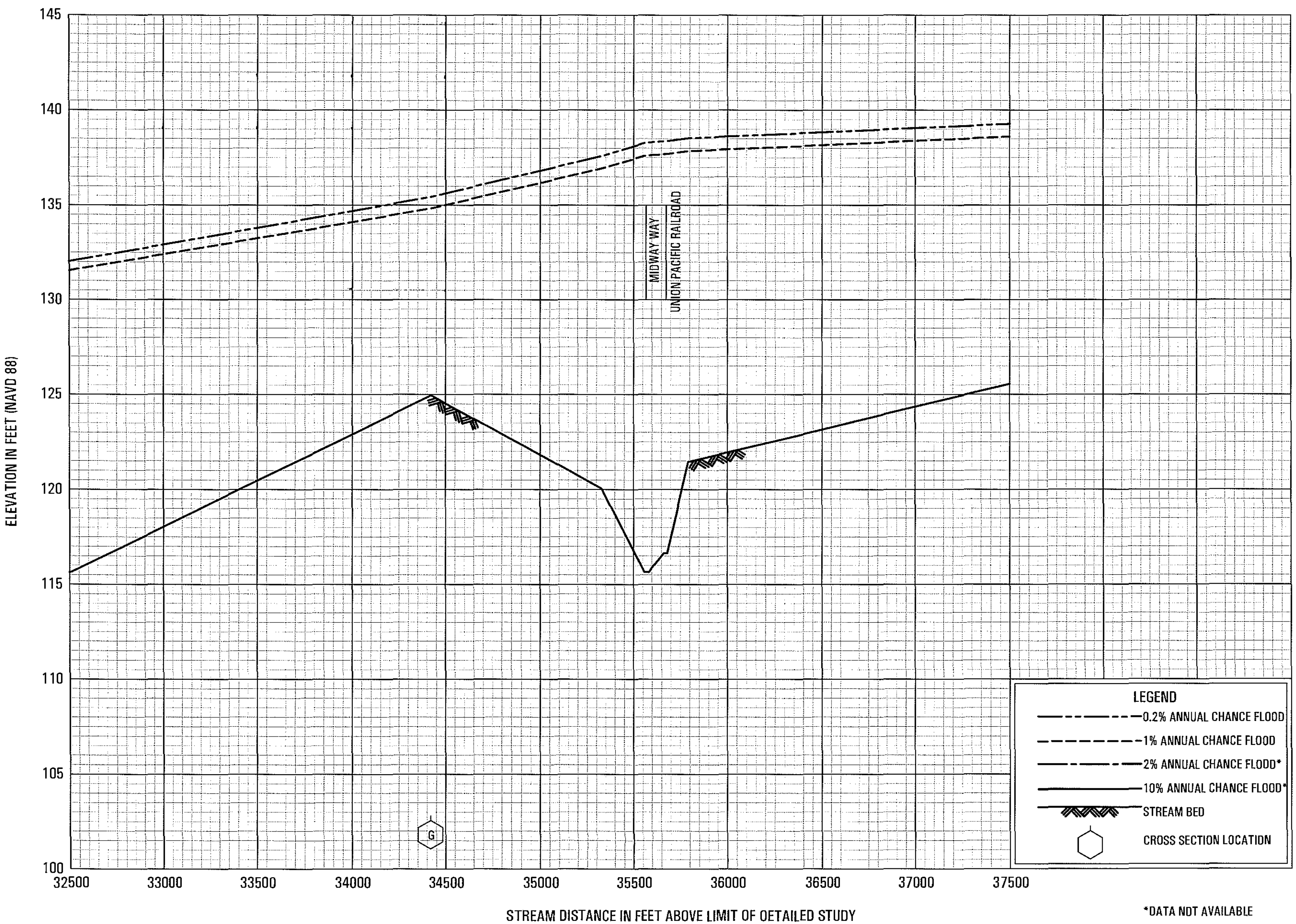
FLOOD PROFILES

BUTTE CREEK RIGHT LEVEE FAILED - BELOW MIDWAY

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA
AND INCORPORATED AREAS

21IP



FLOOD PROFILES

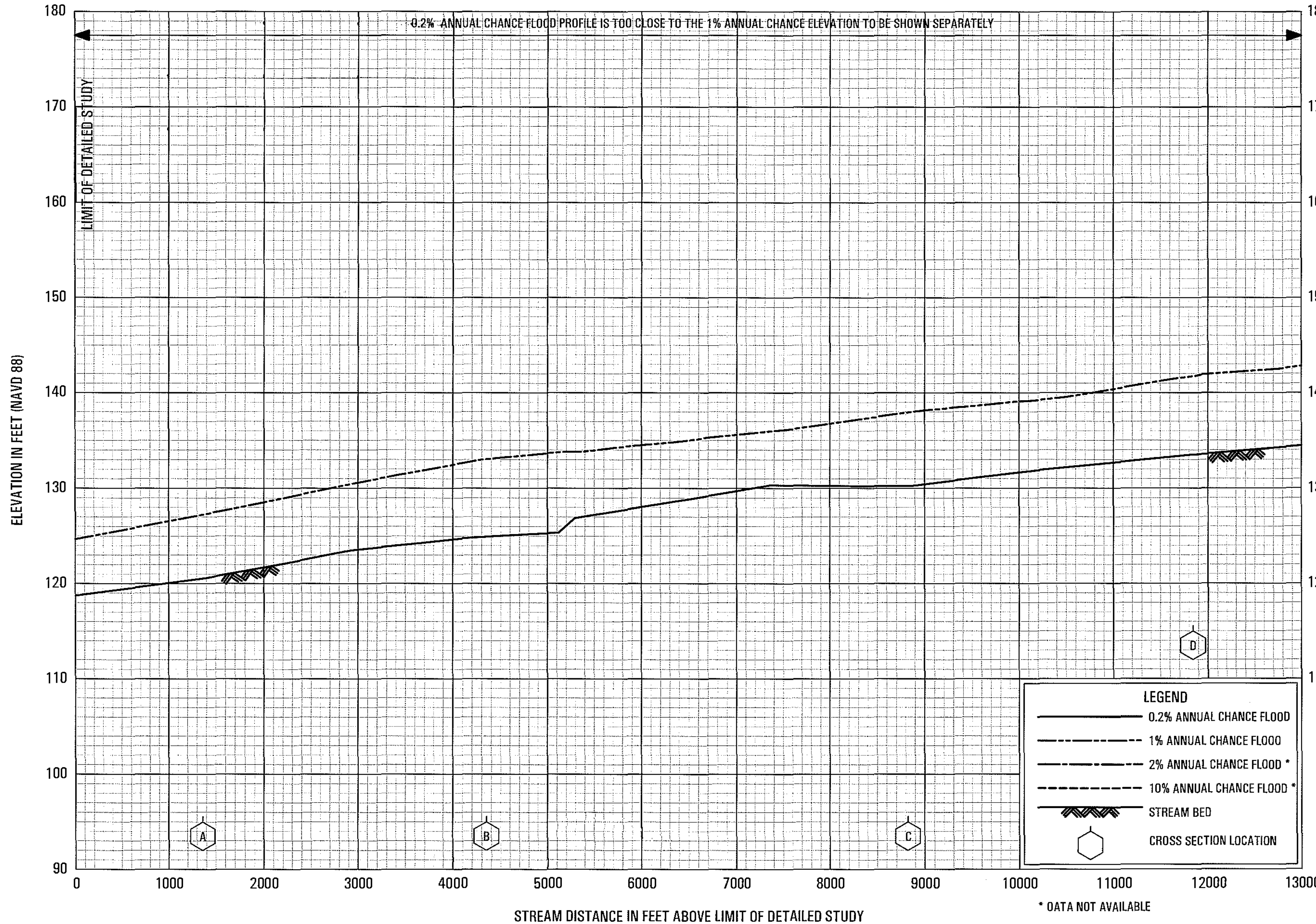
BUTTE CREEK RIGHT LEVEE FAILED - BELOW MIDWAY

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

21mP



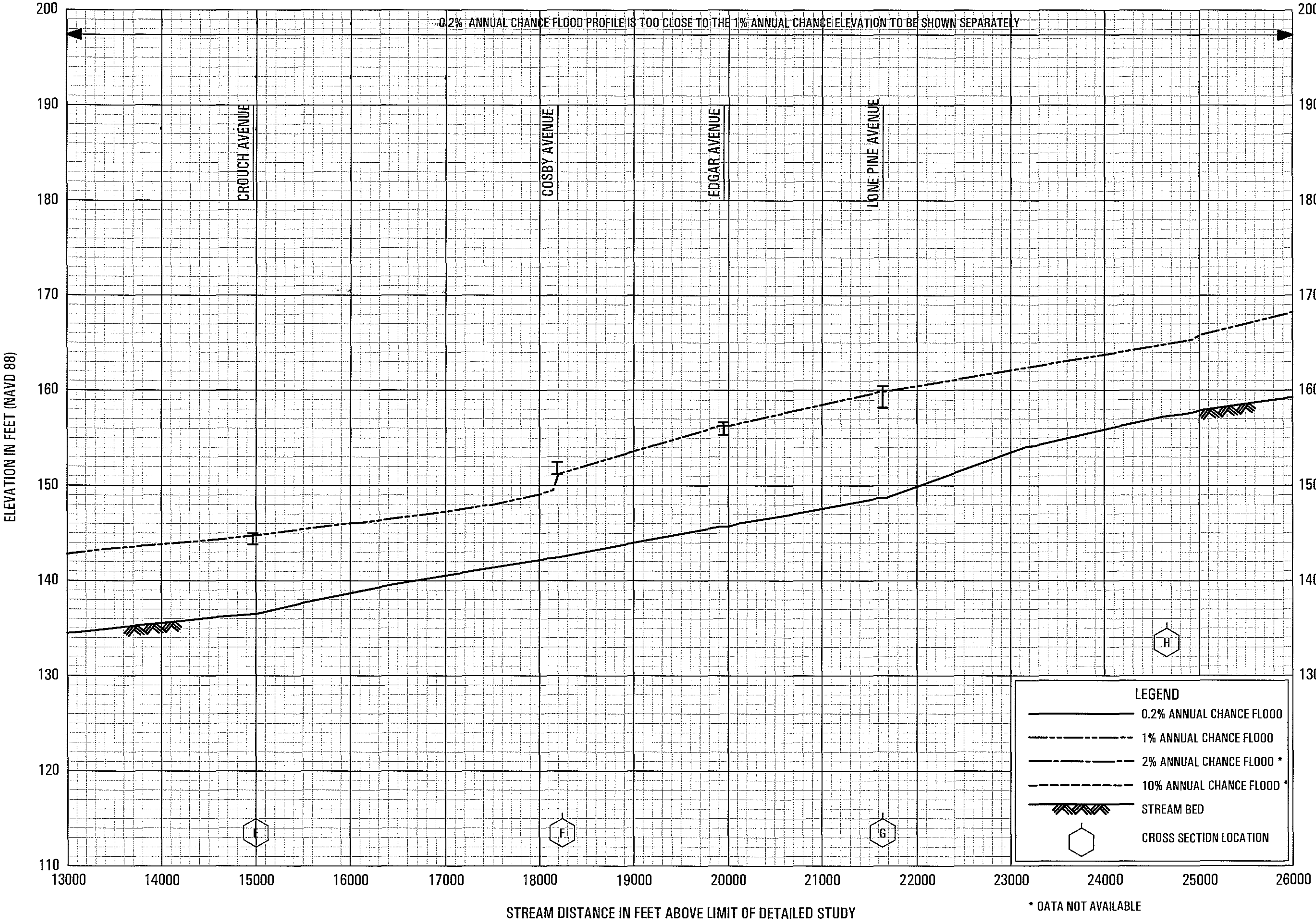
FLOOD PROFILES

COMANCHE CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORORATED AREAS



FLOOD PROFILES

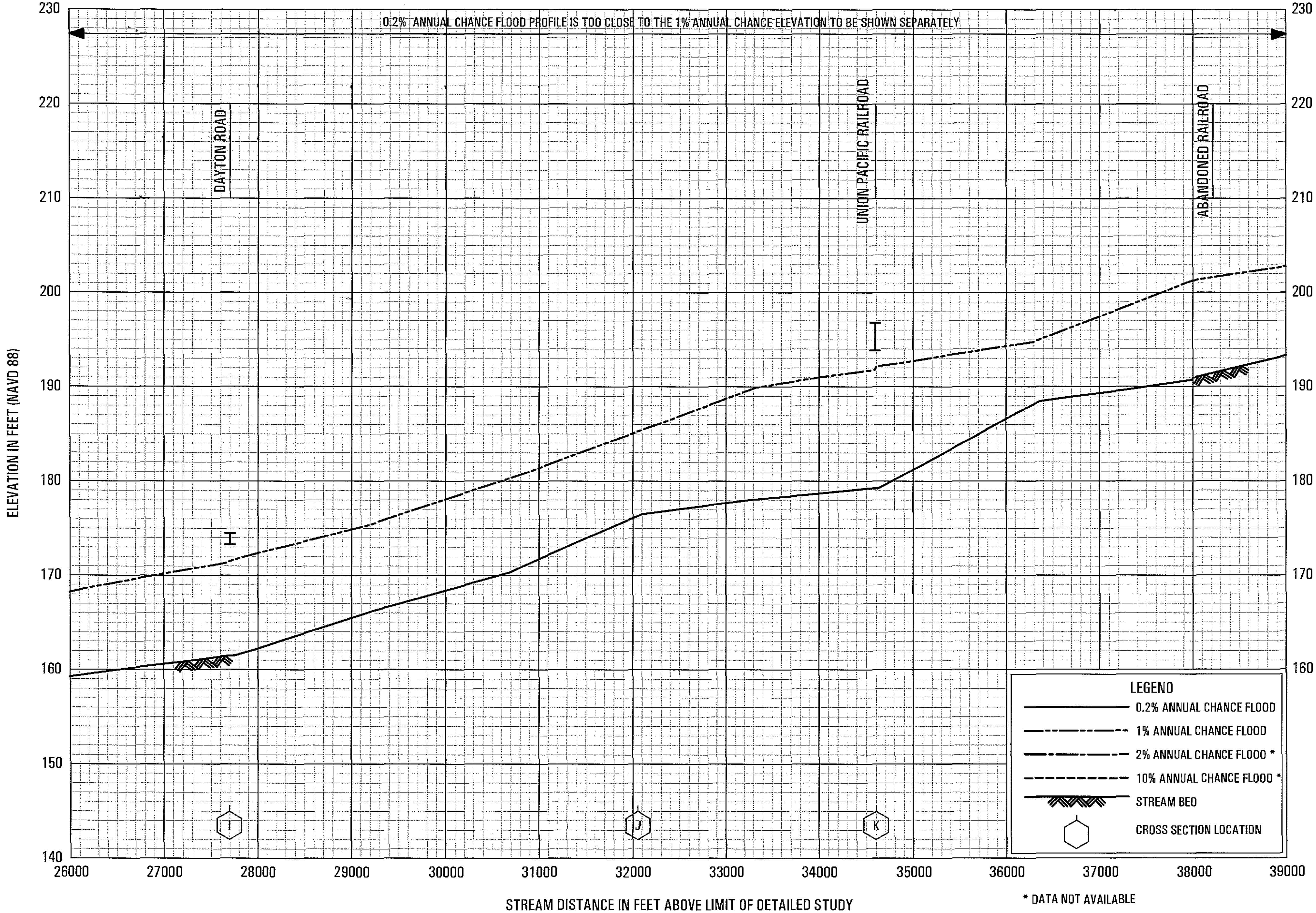
COMANCHE CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORORATED AREAS

23P



FLOOD PROFILES

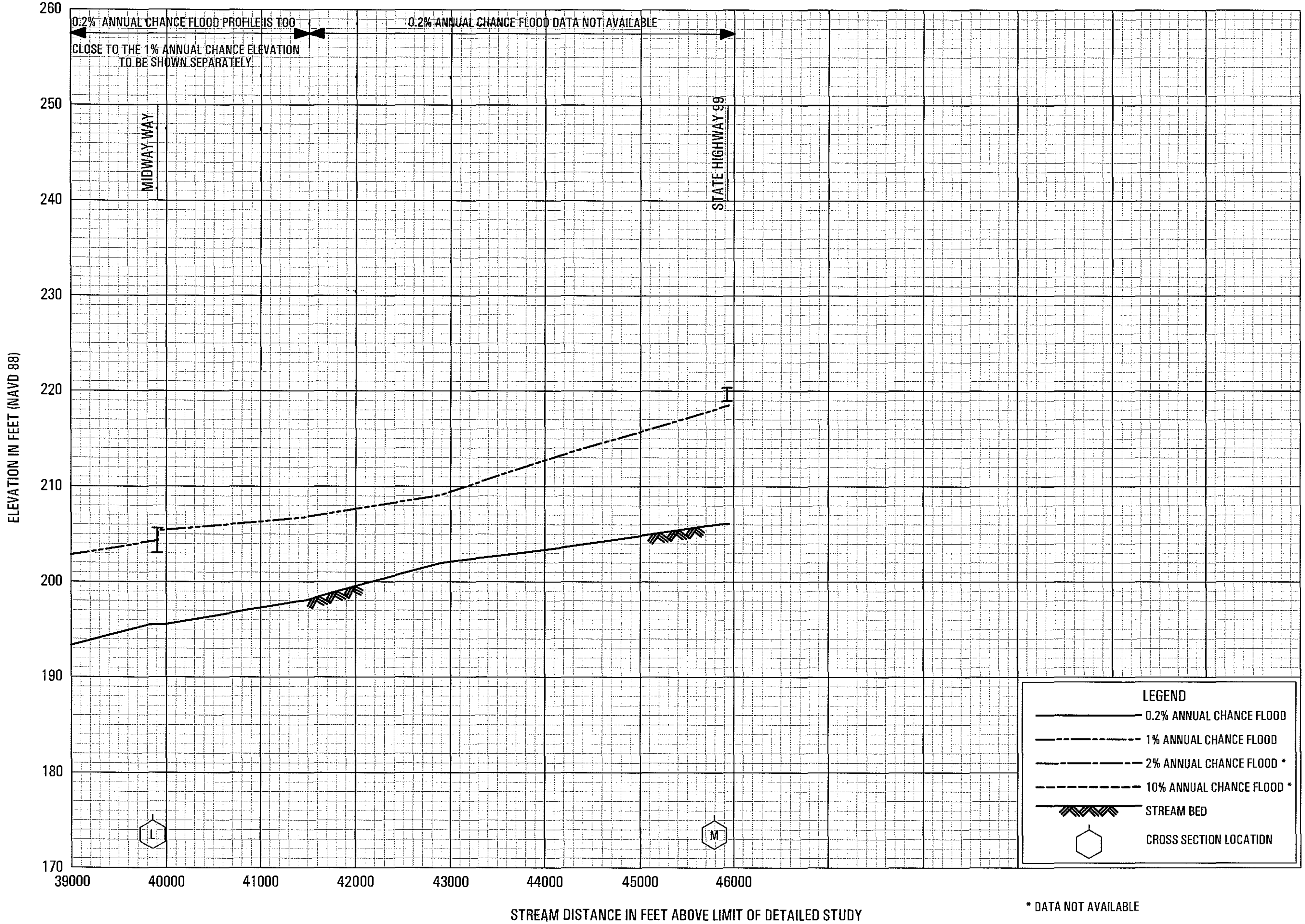
COMANCHE CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

24P



FLOOD PROFILES

COMANCHE CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

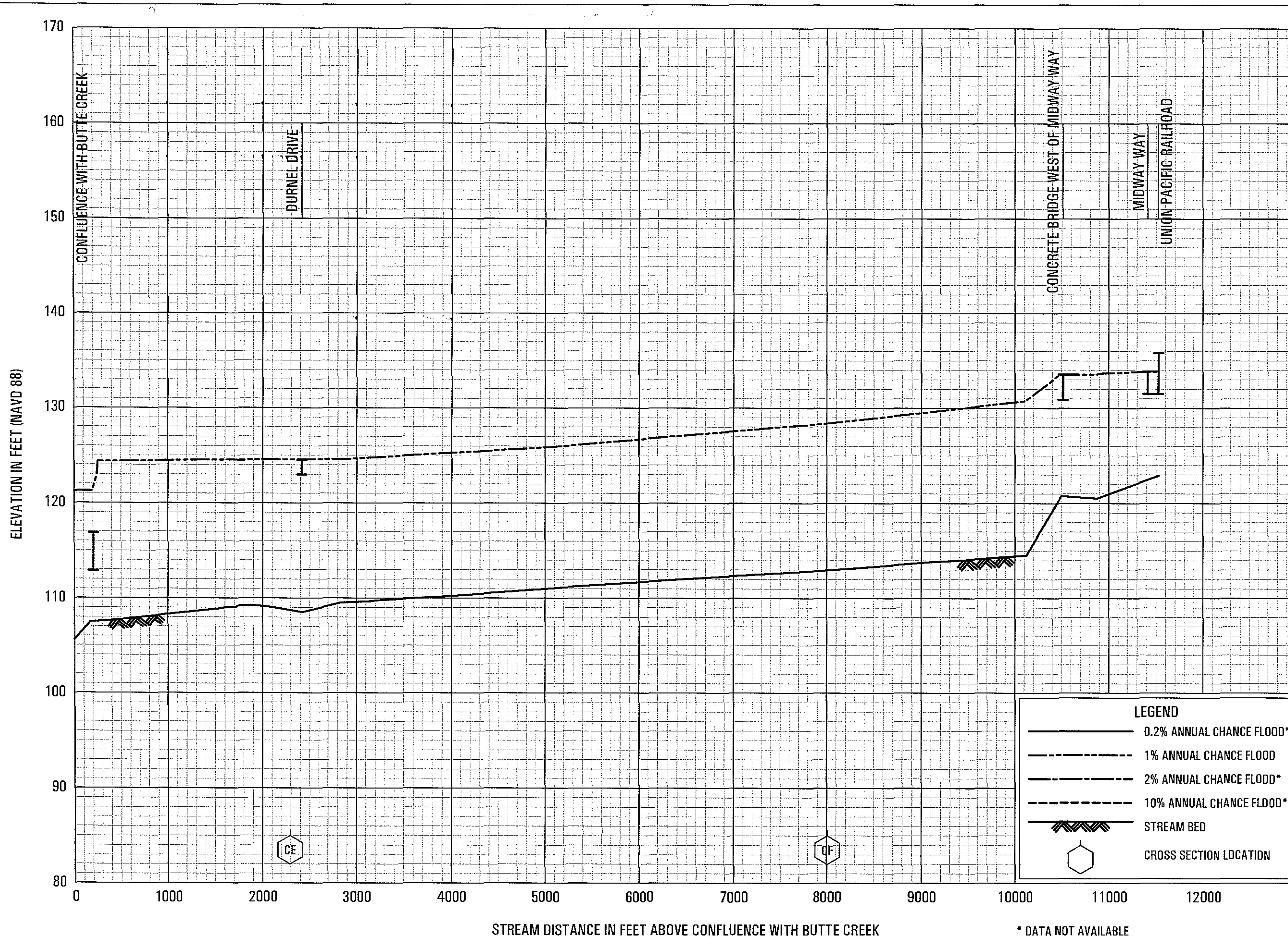
AND INCORORATED AREAS

FLOOD PROFILES

DEAD HORSE SLOUGH

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA
AND INCORPORATED AREAS



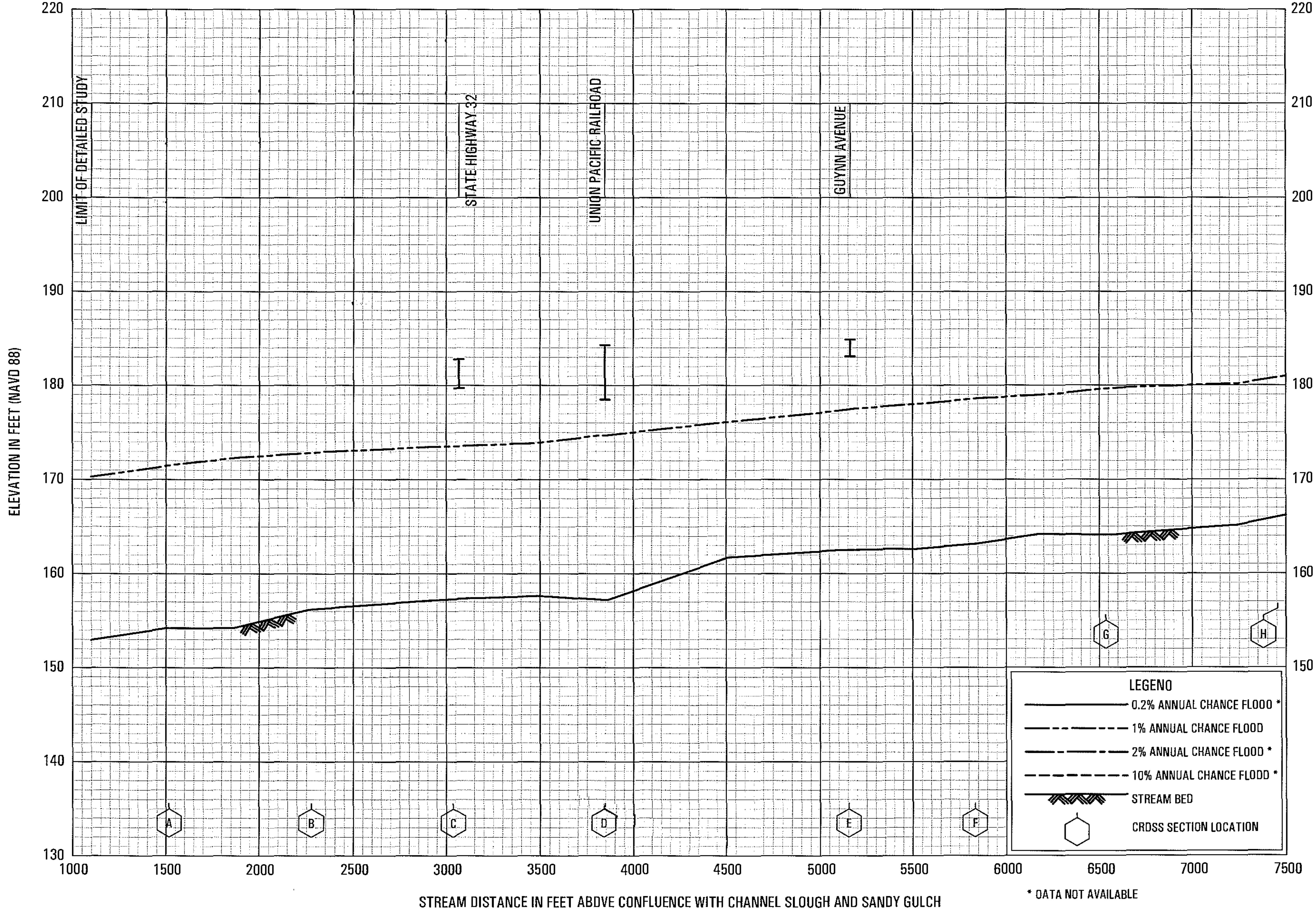
FLOOD PROFILES

HAMLIN SLOUGH

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



FLOOD PROFILES

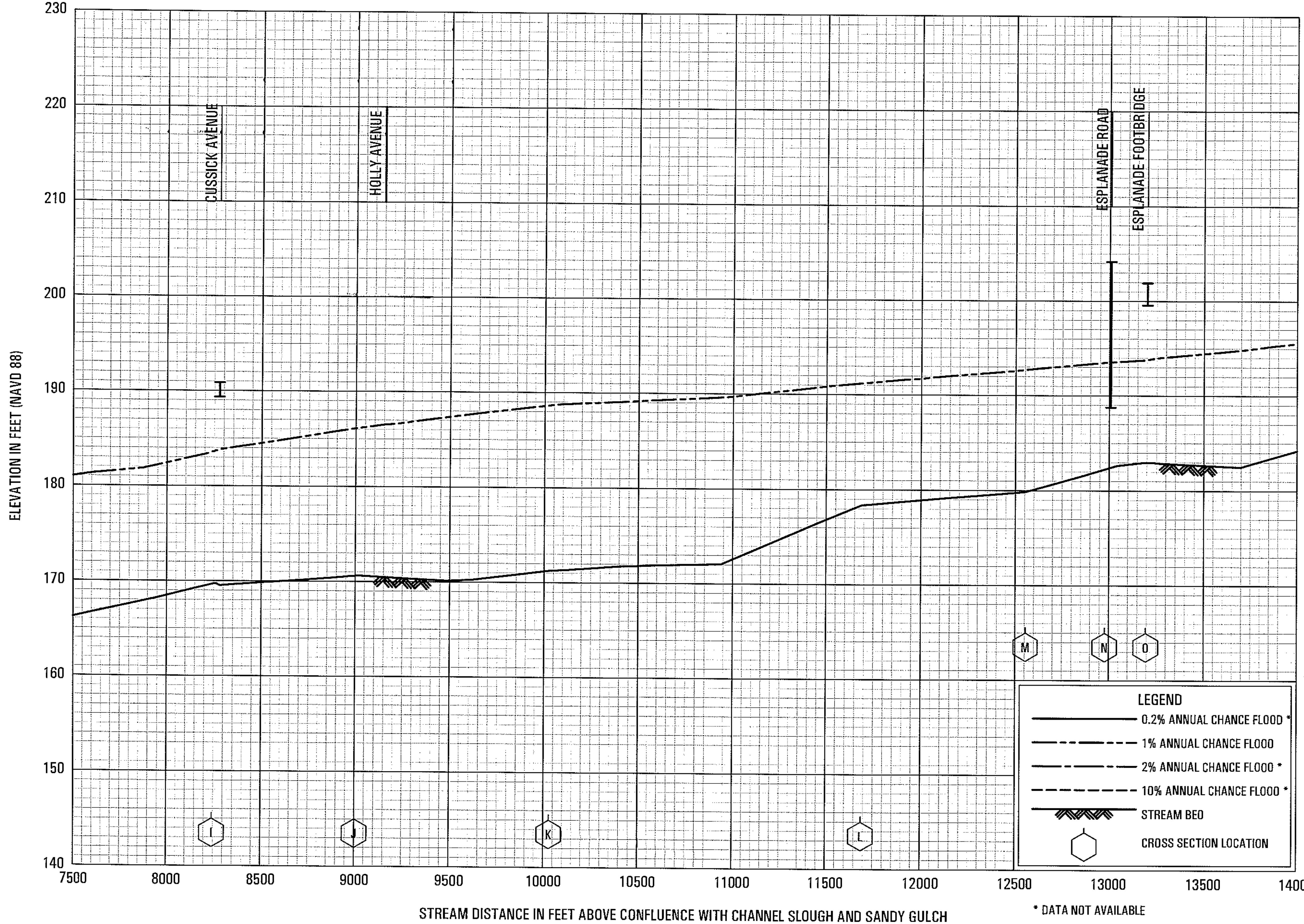
LINDO CHANNEL

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

28P



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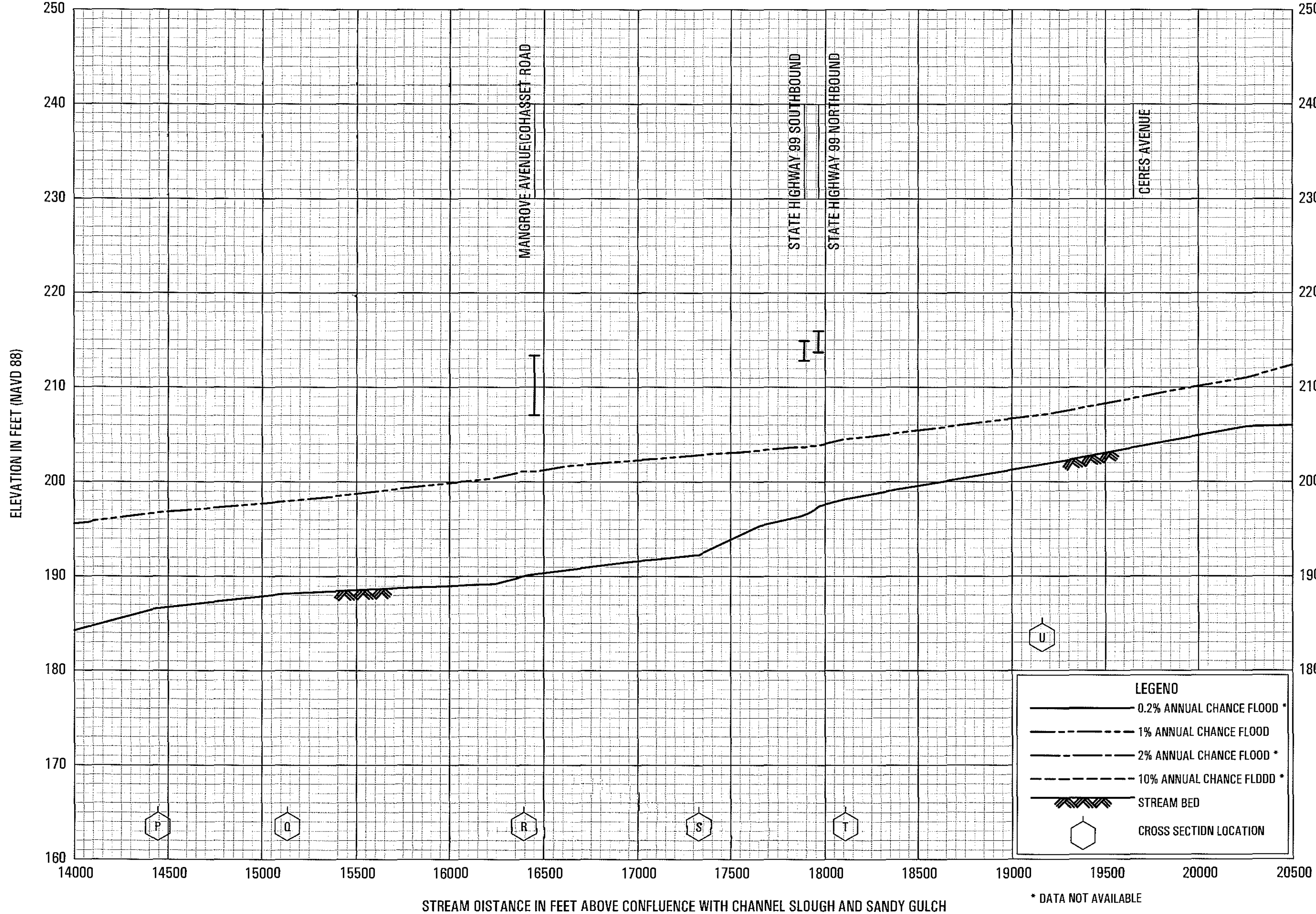
FLOOD PROFILES

LINDO CHANNEL

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA
AND INCORPORATED AREAS



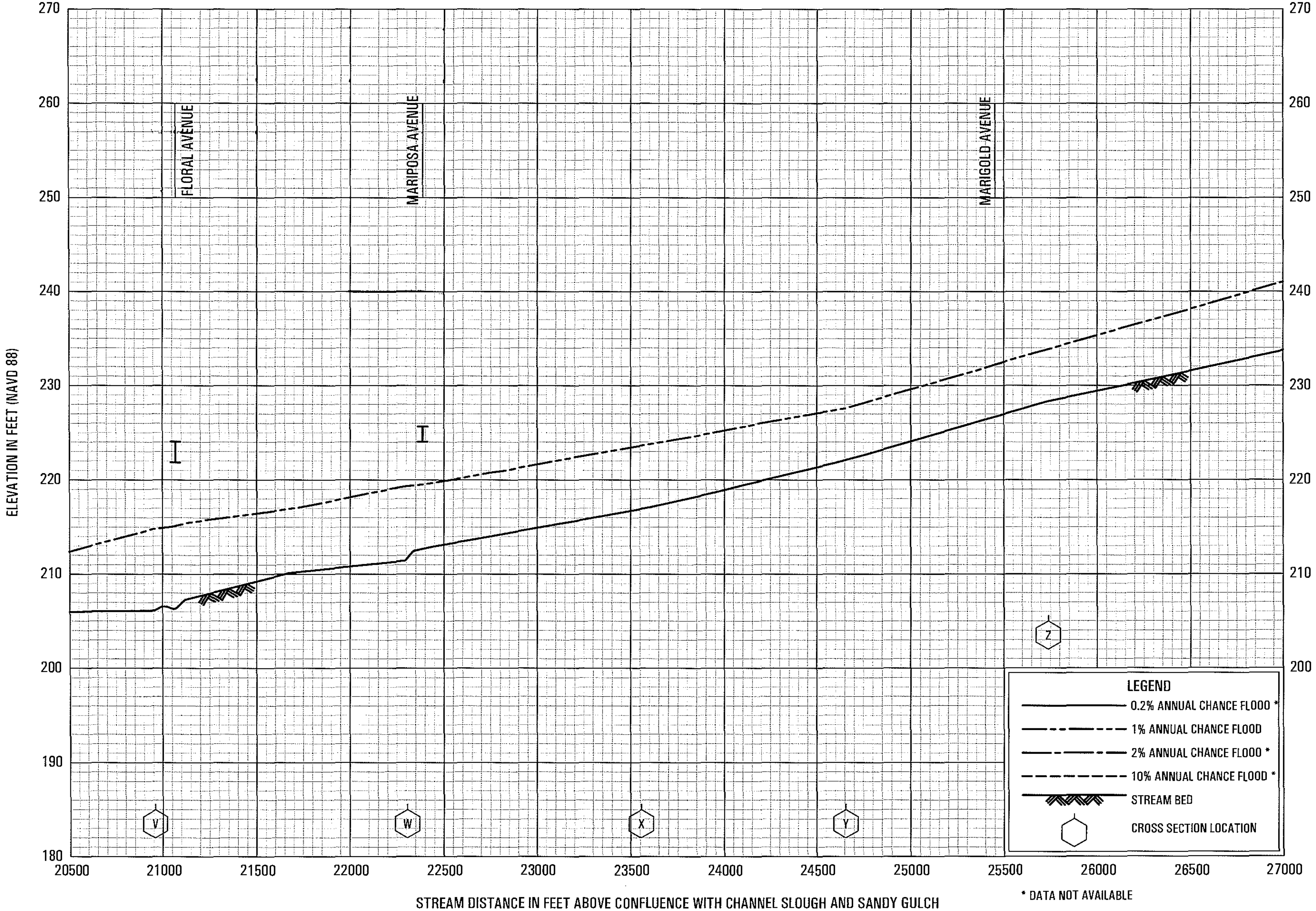
FLOOD PROFILES

LINDO CHANNEL

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



FLOOD PROFILES

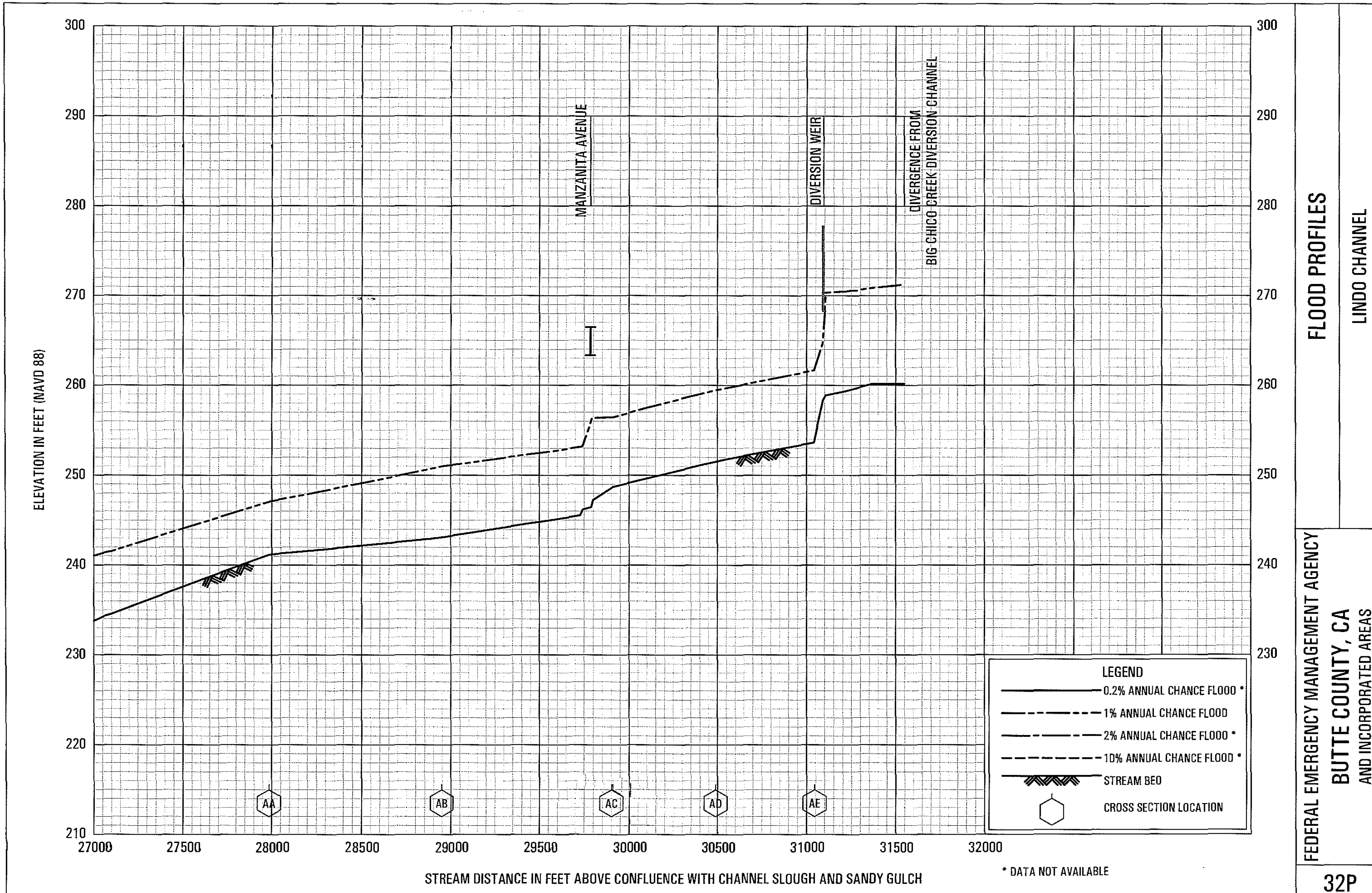
LINDO CHANNEL

FEDERAL EMERGENCY MANAGEMENT AGENCY

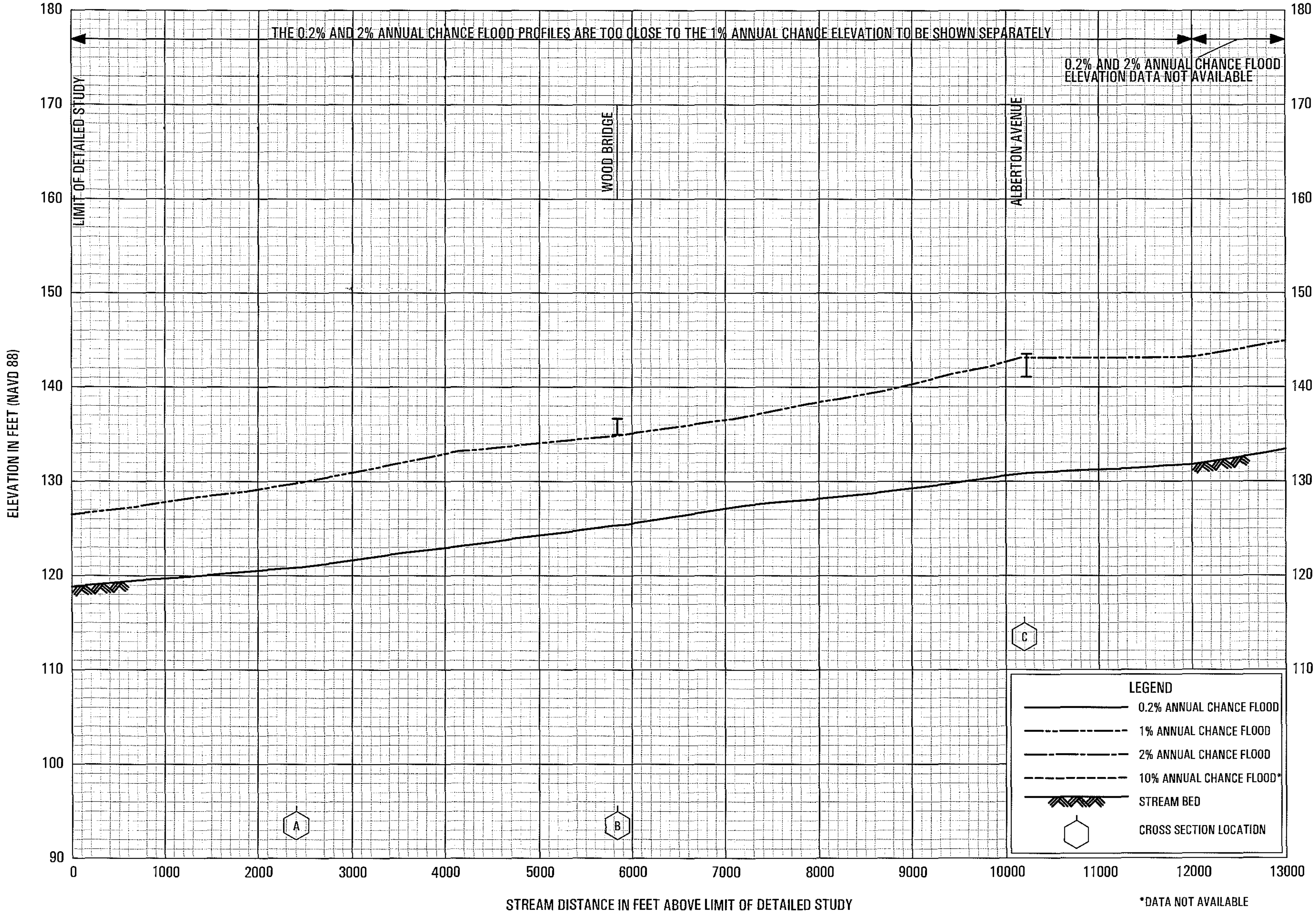
BUTTE COUNTY, CA

AND INCORPORATED AREAS

31P



ELEVATION IN FEET (NAVD 88)
300
290
280
270
260
250
240
230
220
210
27000
27500
28000
28500
29000
29500
30000
30500
31000
31500
32000
STREAM DISTANCE IN FEET ABOVE CONFLUENCE WITH CHANNEL SLOUGH AND SANDY GULCH
MANZANITA AVENUE
DIVERSION WEIR
DIVERGENCE FROM BIG CHICO CREEK DIVERSION CHANNEL
AA
AB
AC
AD
AE
LEGEND
0.2% ANNUAL CHANCE FLOOD *
1% ANNUAL CHANCE FLOOD
2% ANNUAL CHANCE FLOOD *
10% ANNUAL CHANCE FLOOD *
STREAM BED
CROSS SECTION LOCATION
* DATA NOT AVAILABLE
FLOOD PROFILES
LINDO CHANNEL
FEDERAL EMERGENCY MANAGEMENT AGENCY
BUTTE COUNTY, CA
AND INCORPORATED AREAS
32P

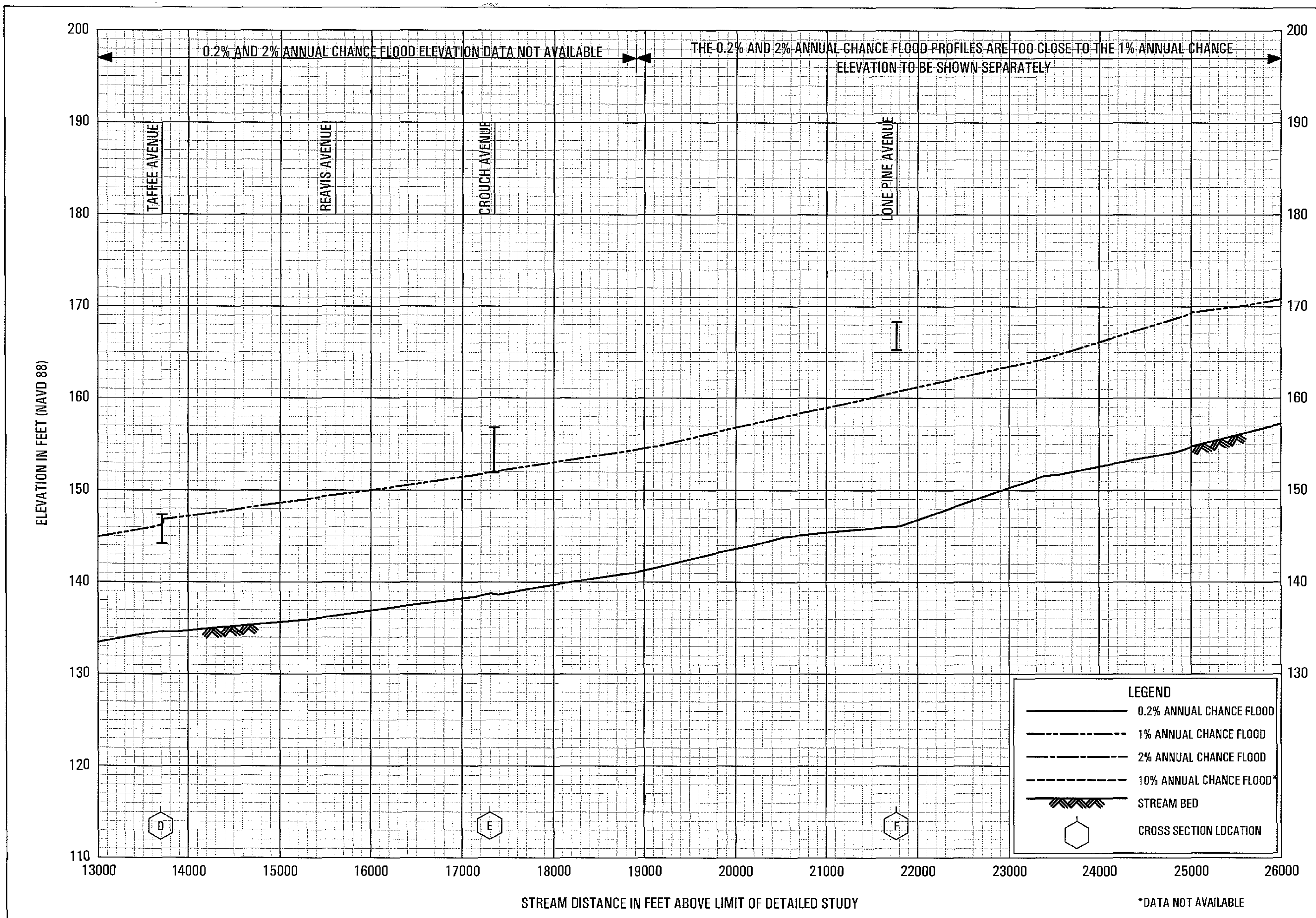


FLOOD PROFILES

LITTLE CHICO CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA
AND INCORPORATED AREAS



FLOOD PROFILES

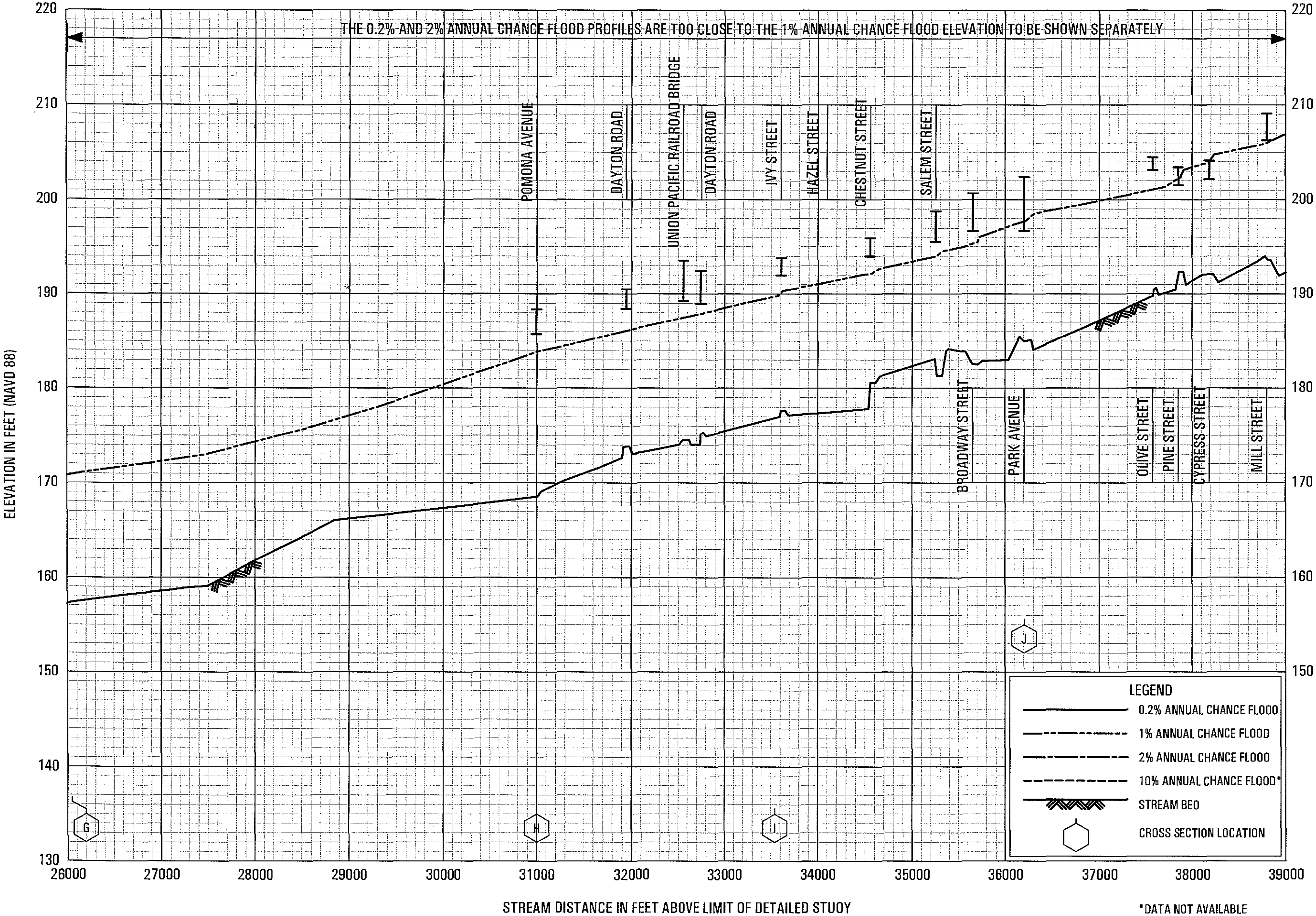
LITTLE CHICO CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

34P



FLOOD PROFILES

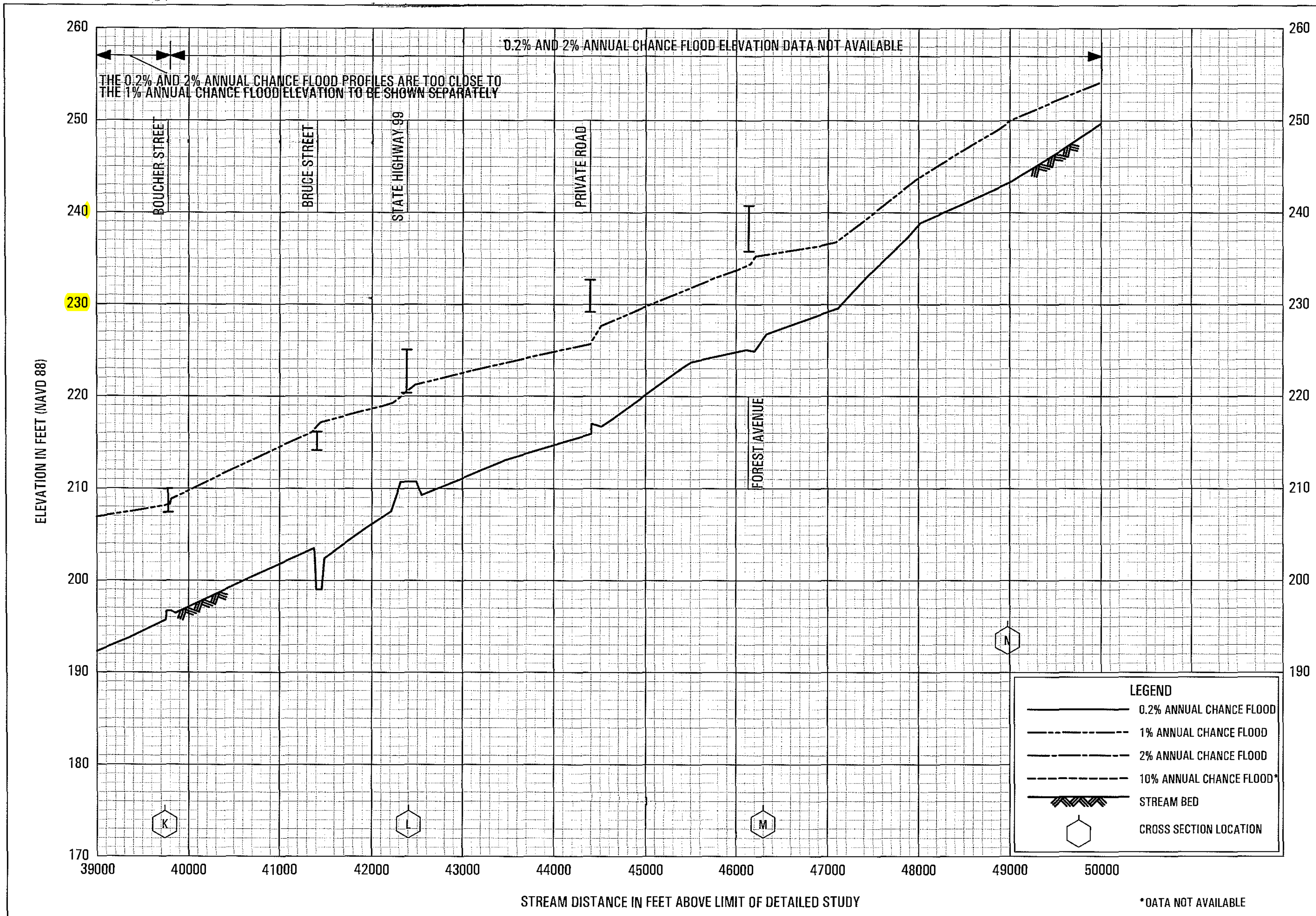
LITTLE CHICO CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

35P

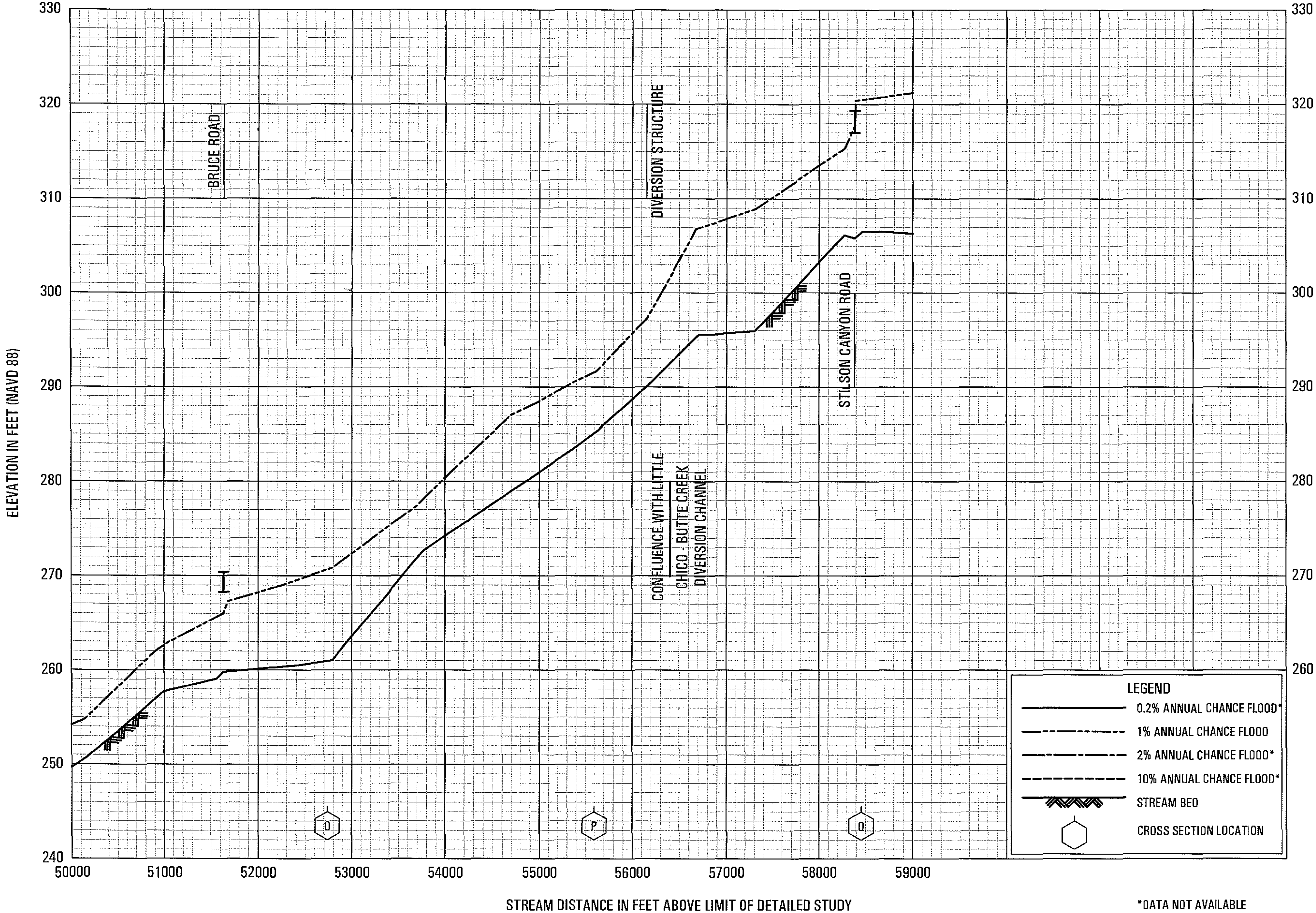


FLOOD PROFILES

LITTLE CHICO CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA AND INCORPORATED AREAS

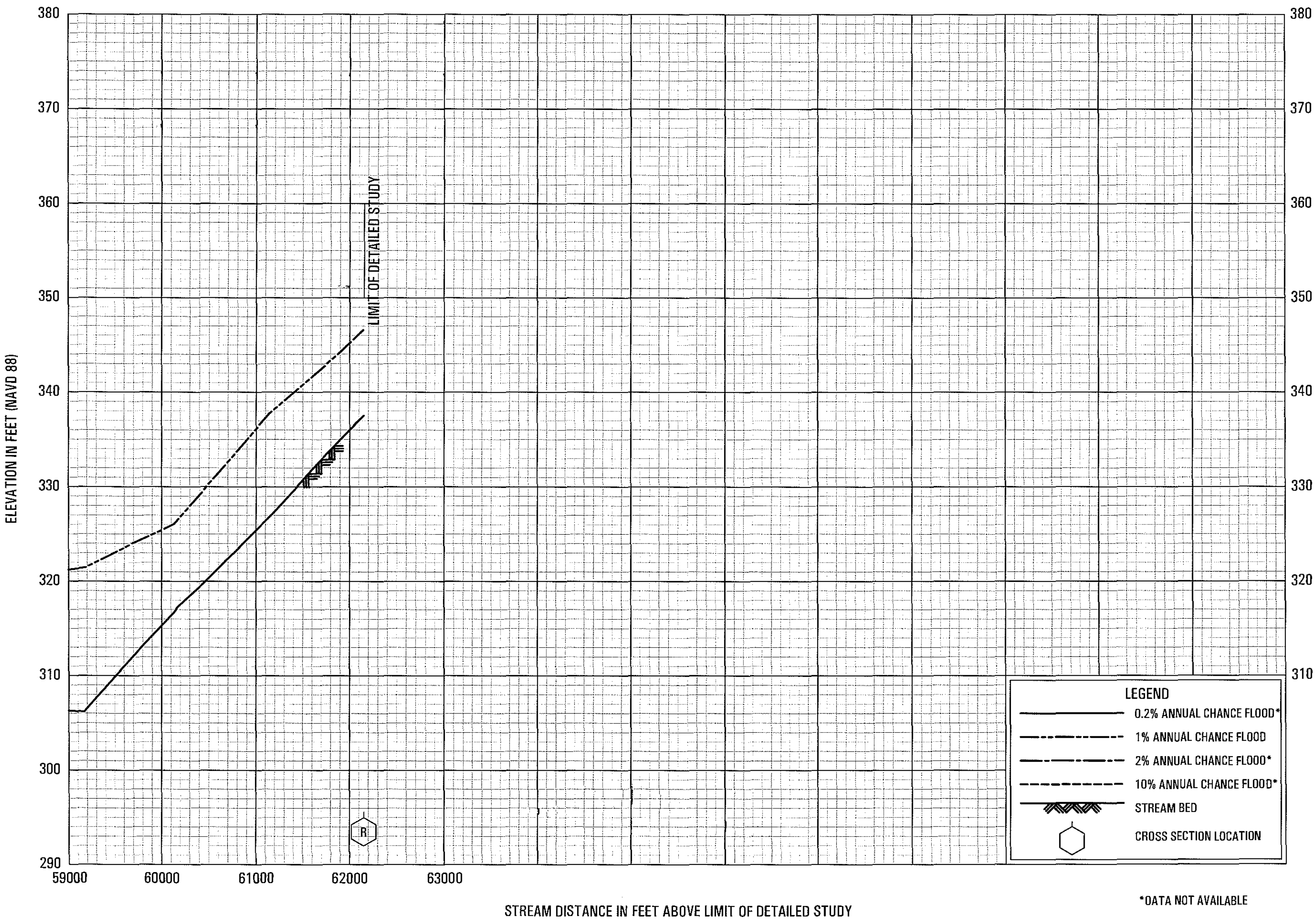


FLOOD PROFILES

LITTLE CHICO CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA AND INCORPORATED AREAS



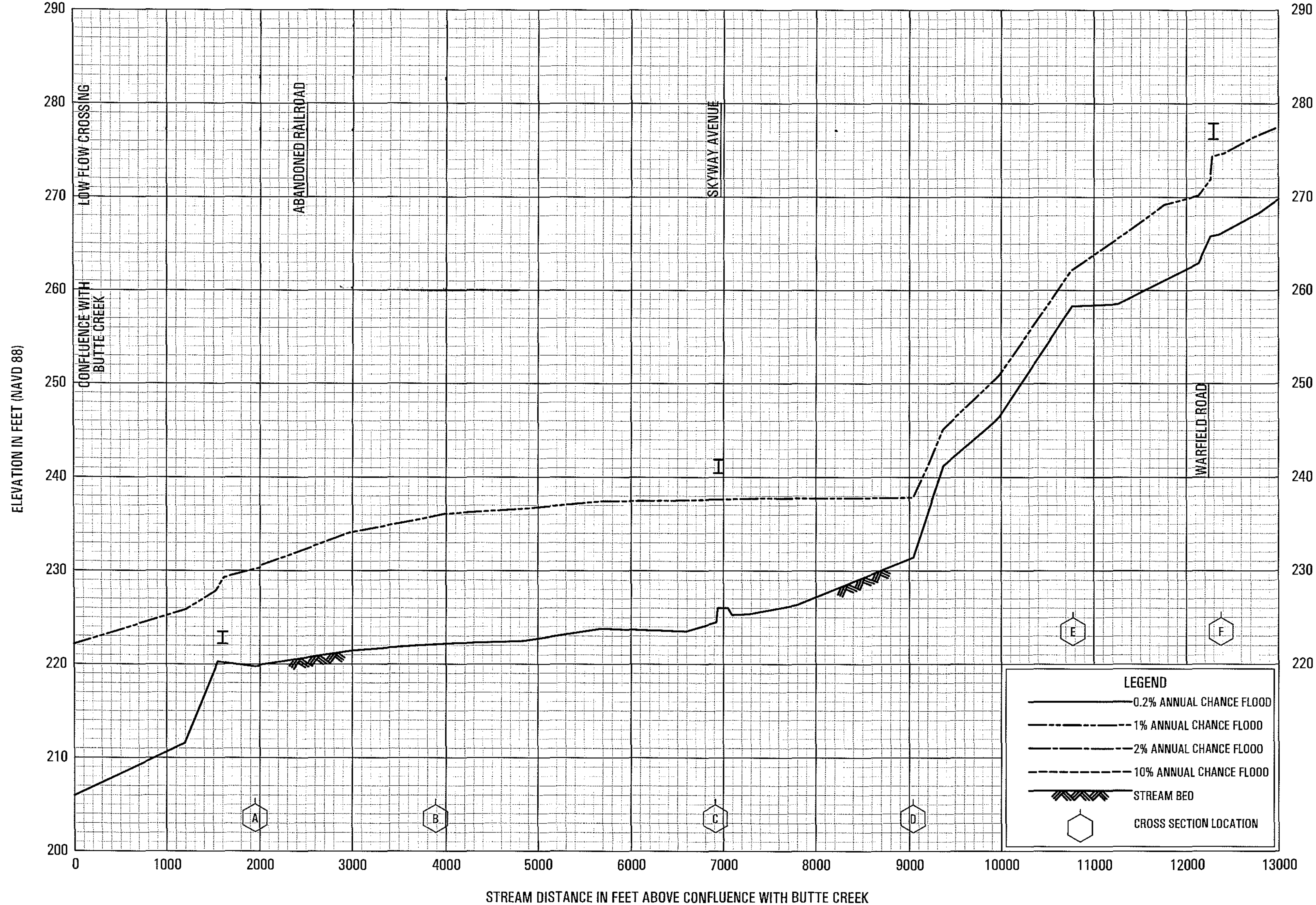
FLOOD PROFILES

LITTLE CHICO CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



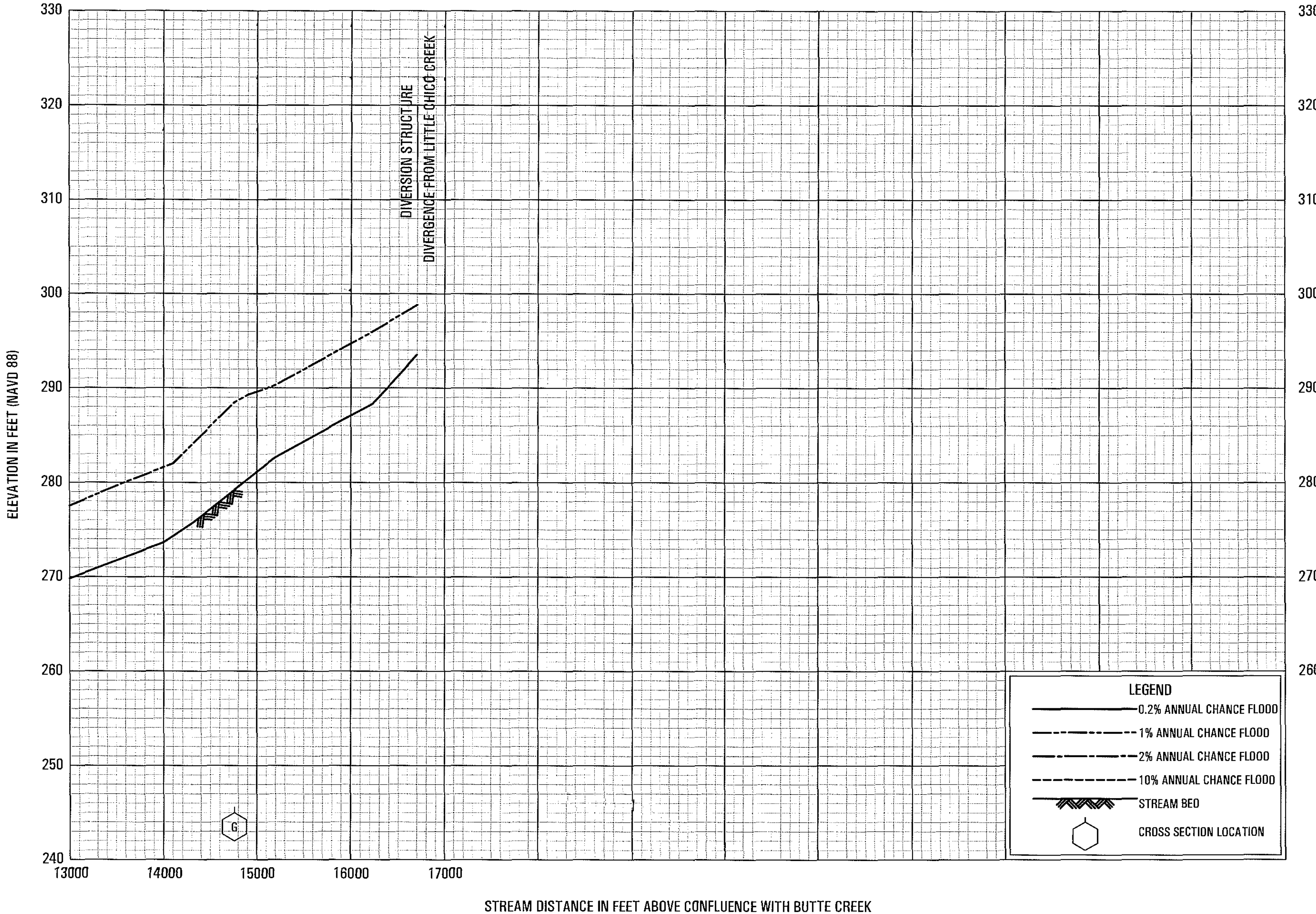
FLOOD PROFILES

LITTLE CHICO-BUTTE CREEK DIVERSION CHANNEL

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



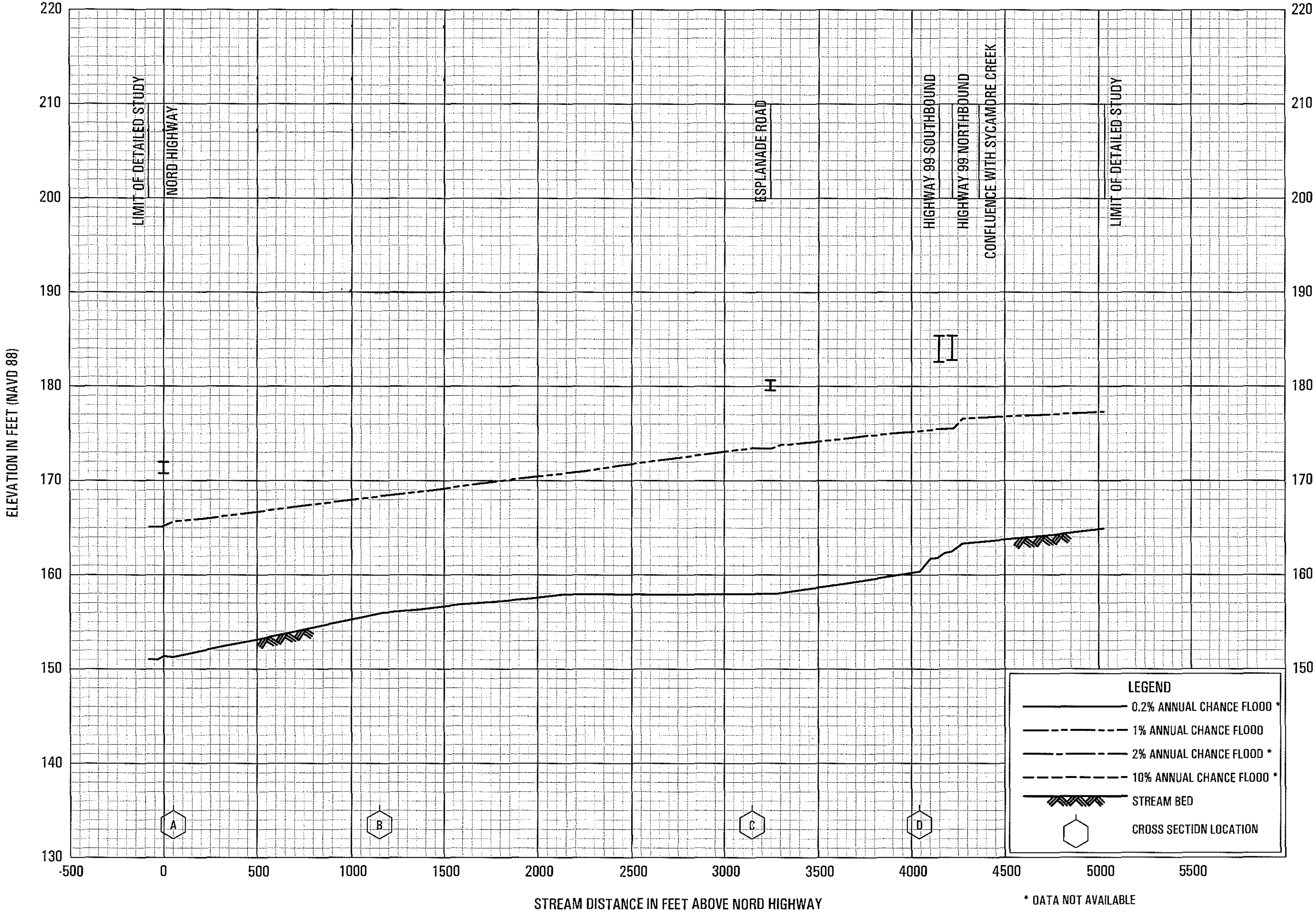
FLOOD PROFILES

LITTLE CHICO-BUTTE CREEK DIVERSION CHANNEL

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



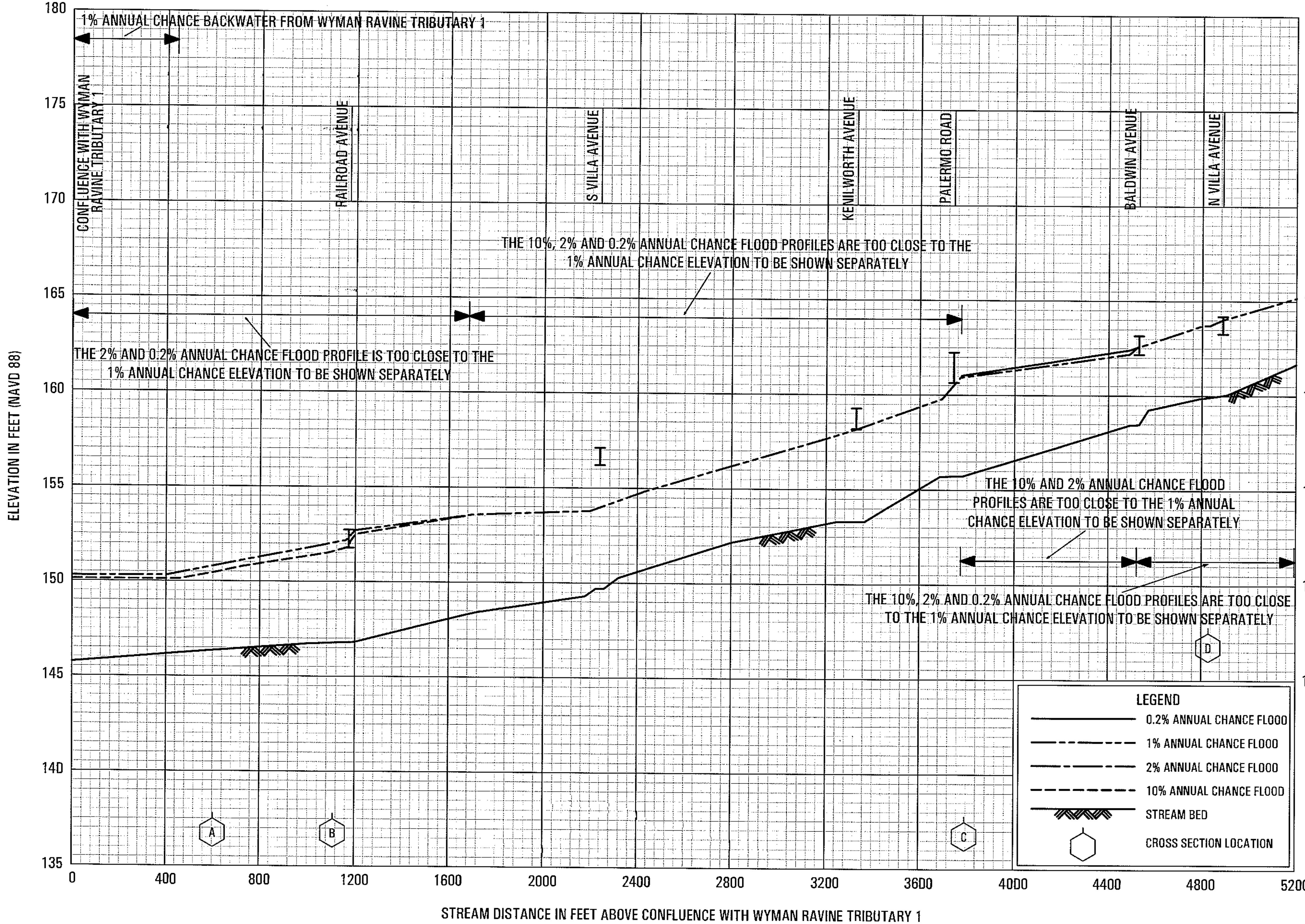
FLOOD PROFILES

MUD CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



FLOOD PROFILES

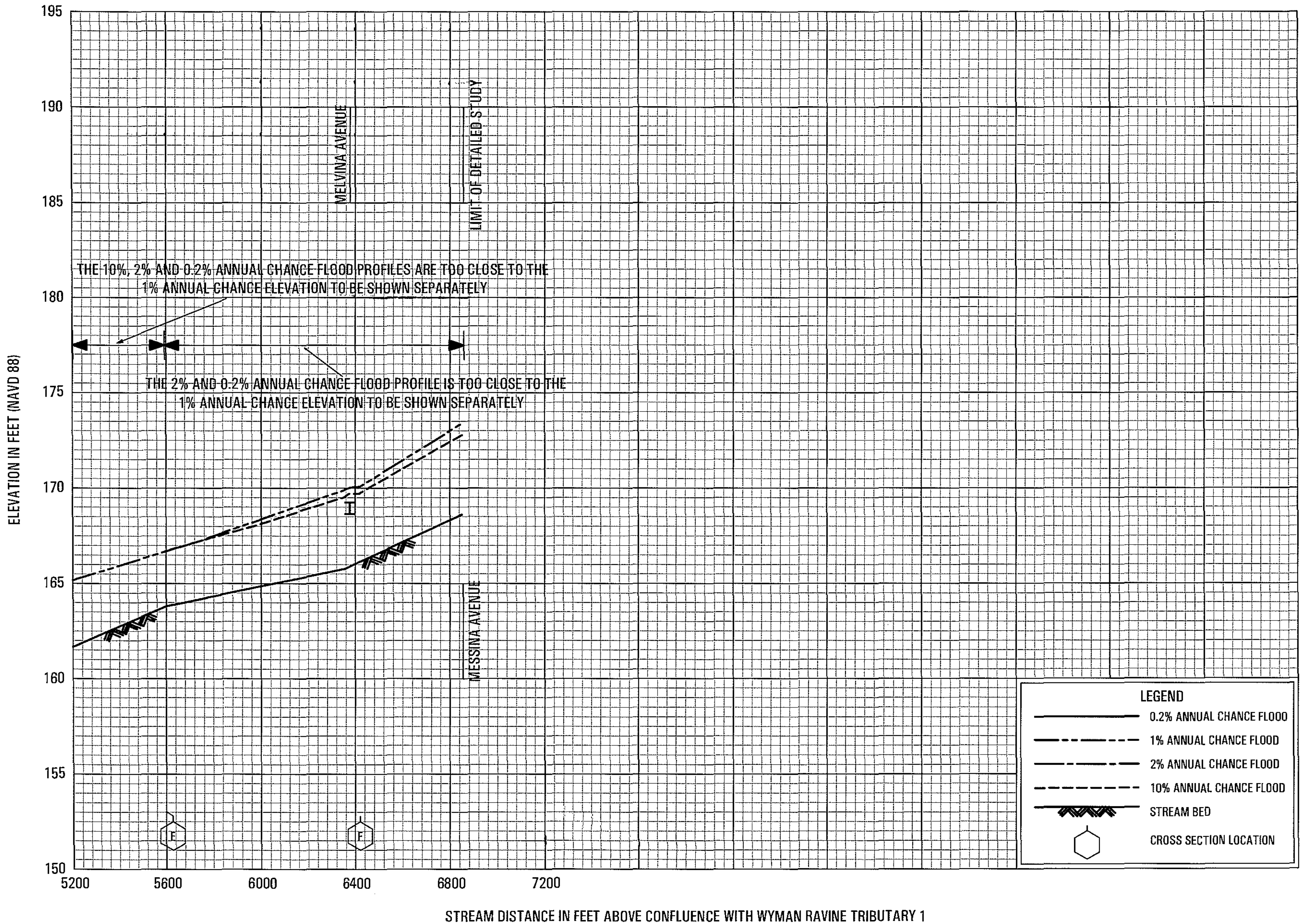
PALERMO TRIBUTARY

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

42P



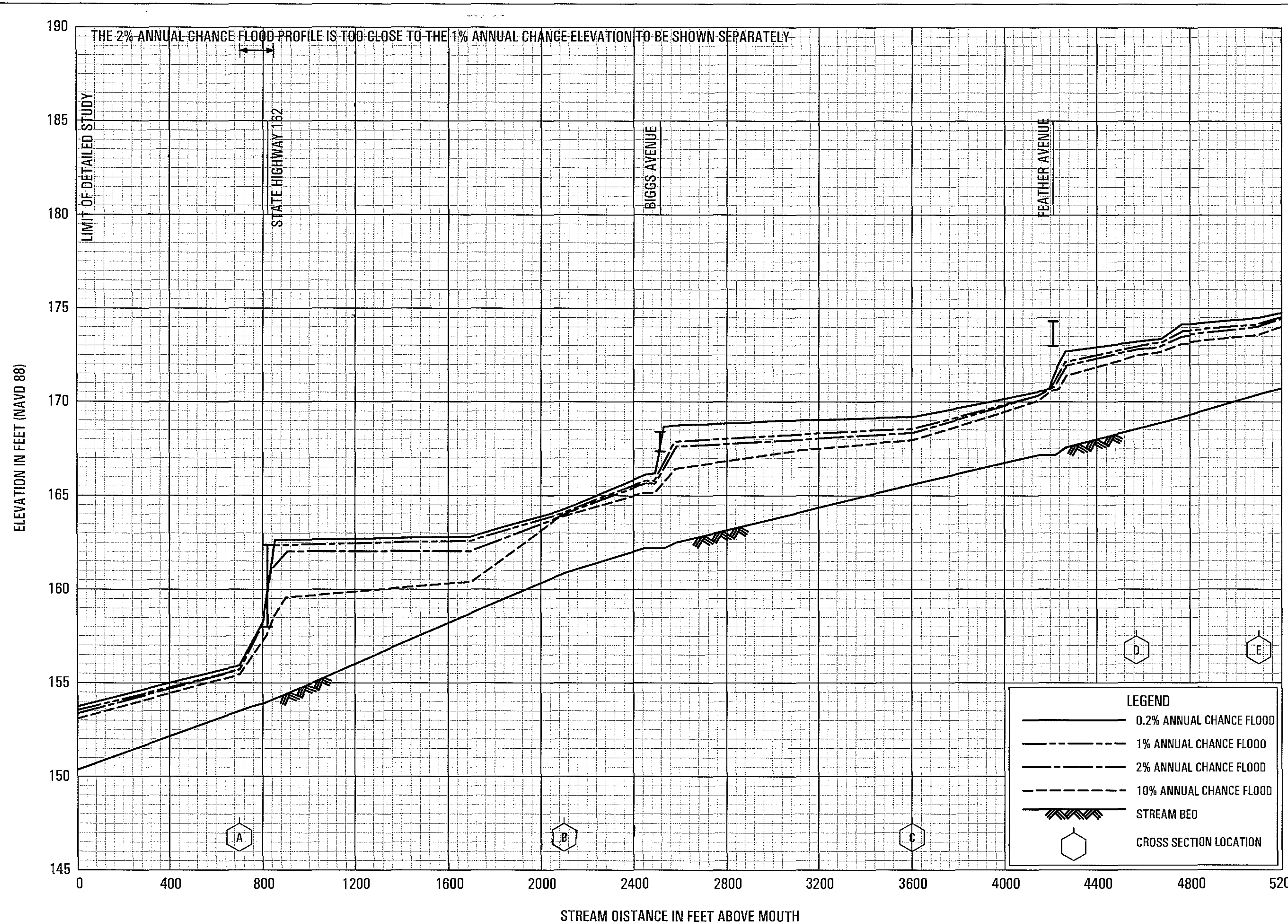
FLOOD PROFILES

PALERMO TRIBUTARY

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



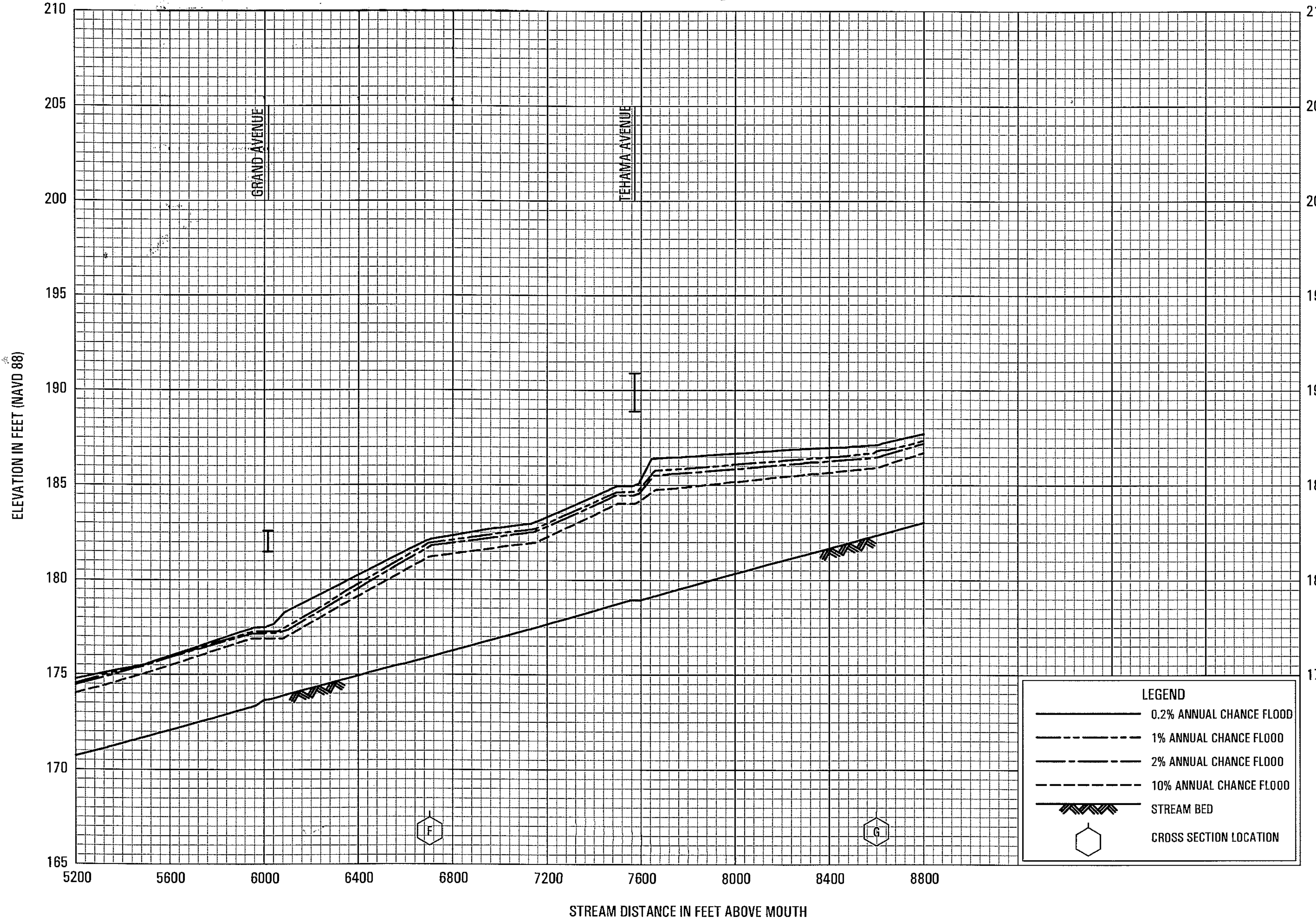
FLOOD PROFILES

RUDDY CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



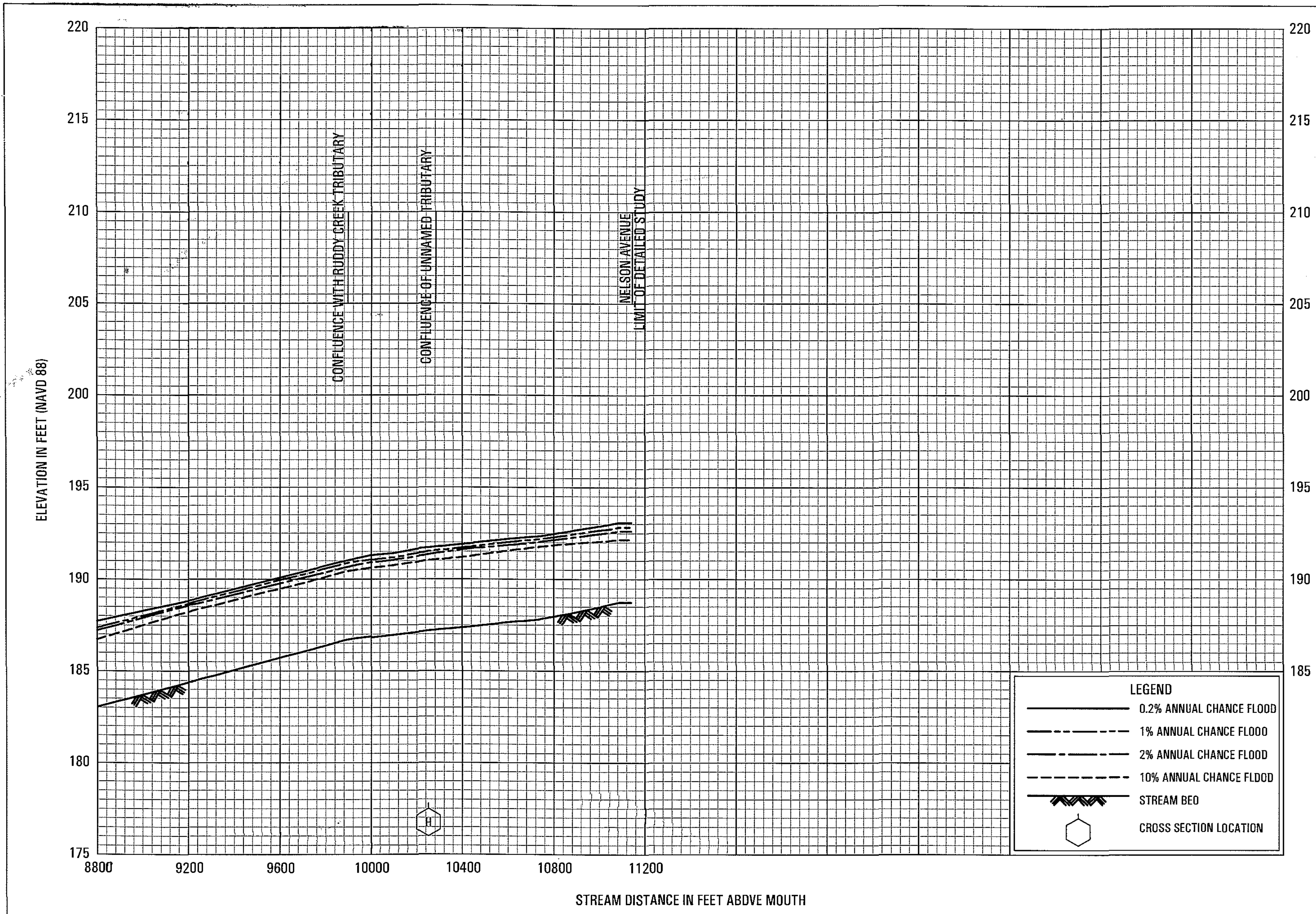
FLOOD PROFILES

RUDDY CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



FLOOD PROFILES

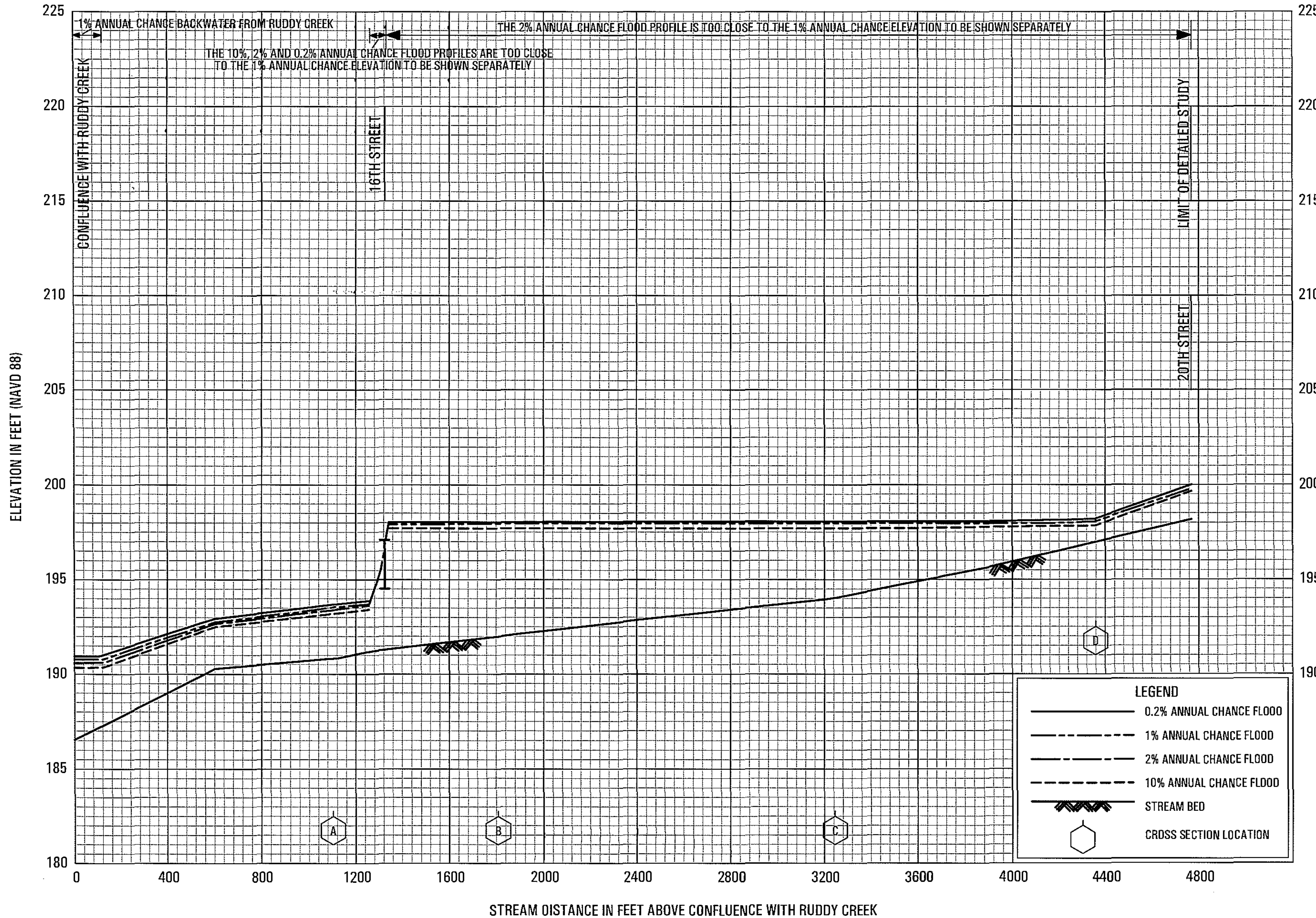
RUDDY CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

46P



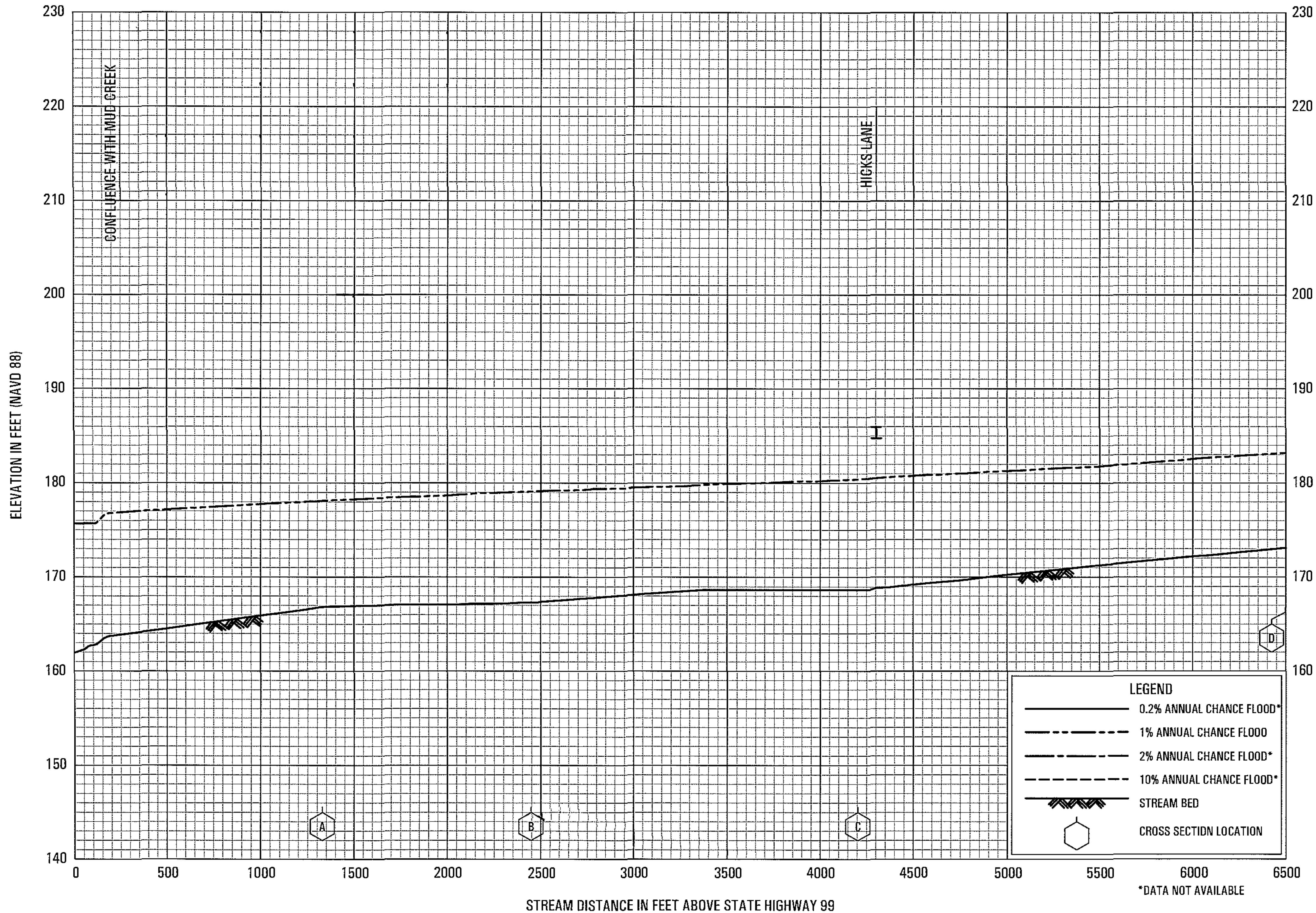
FLOOD PROFILES

RUDDY CREEK TRIBUTARY

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



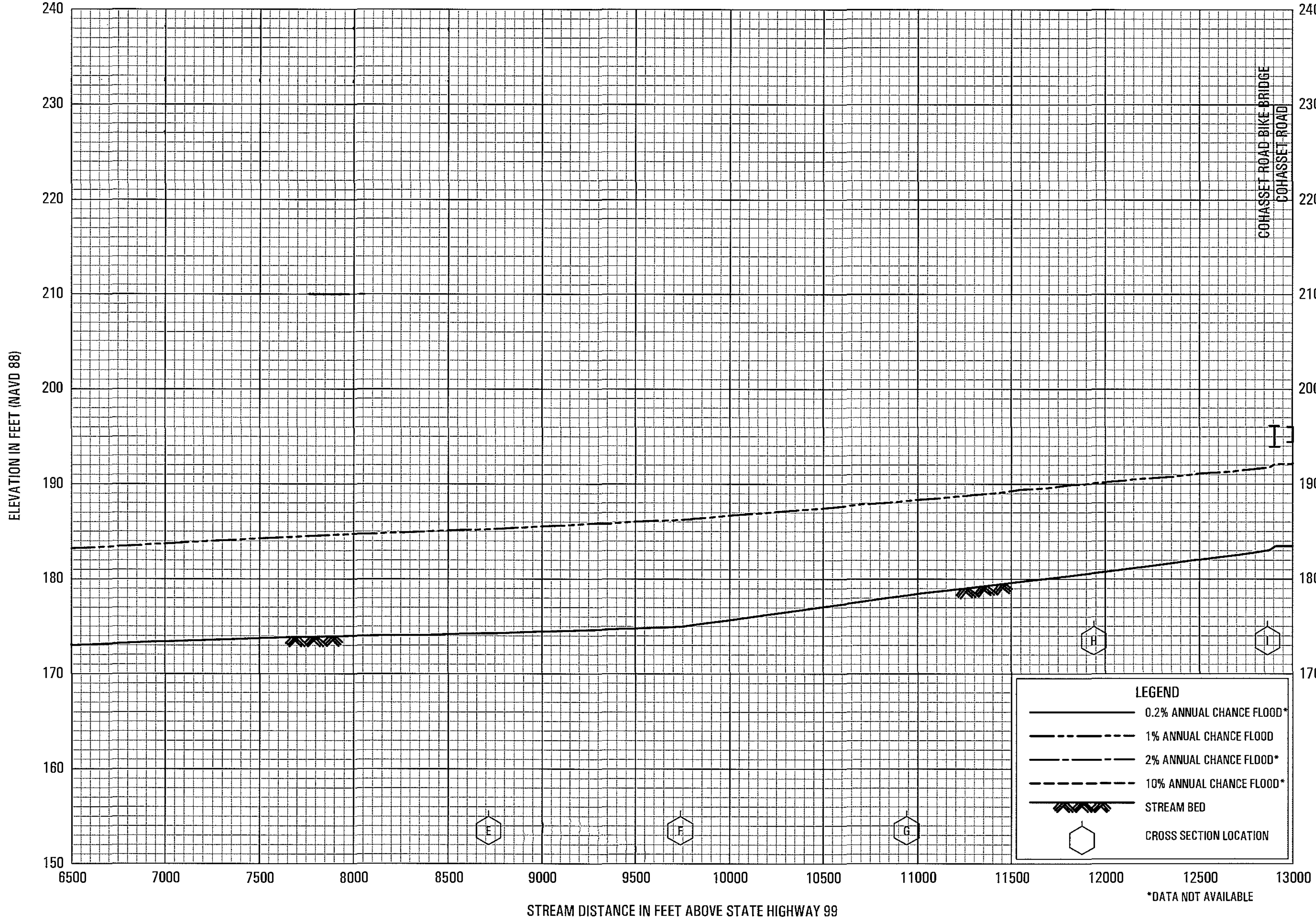
FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

FLOOD PROFILES

SYCAMORE CREEK



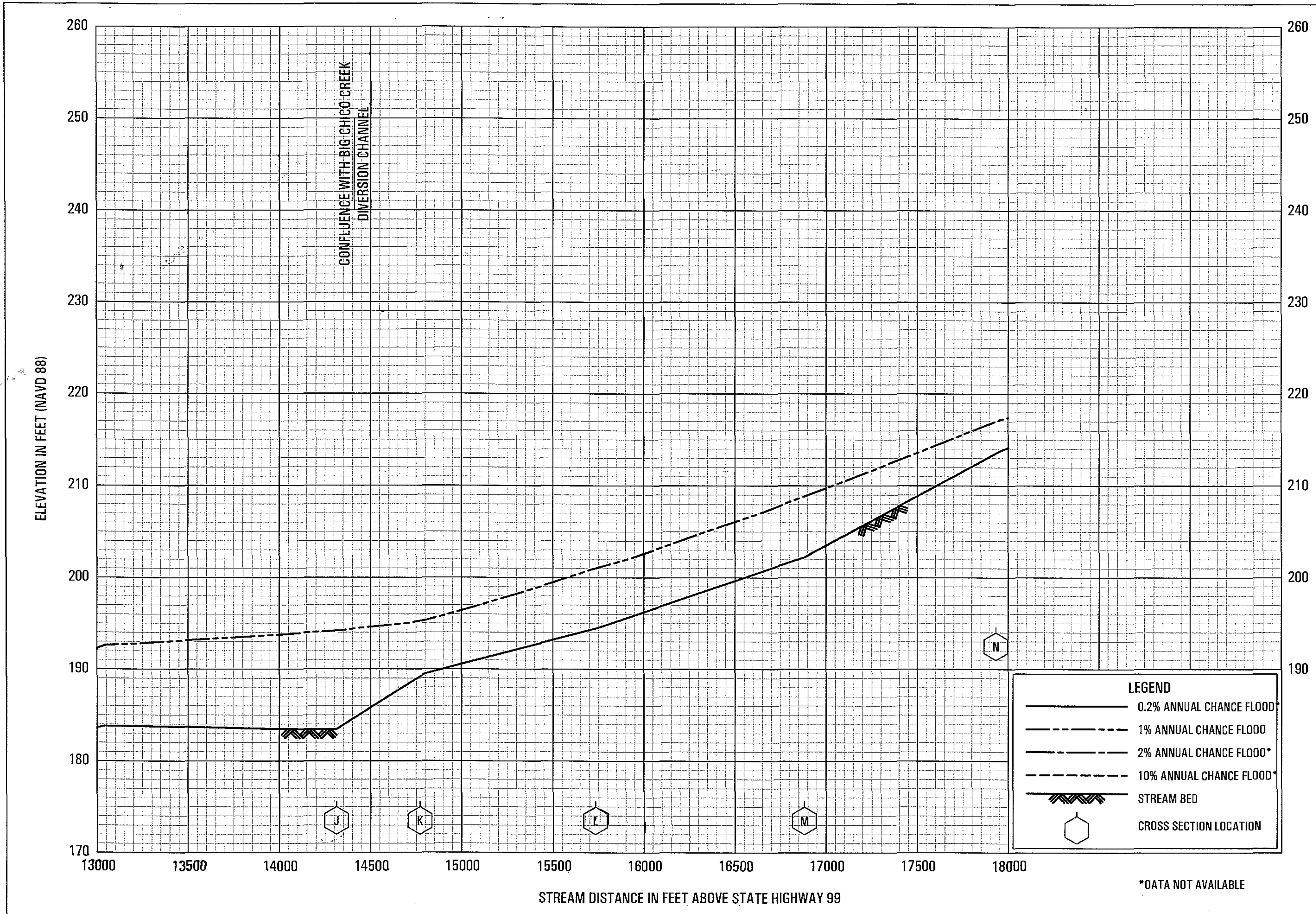
FLOOD PROFILES

SYCAMORE CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



FLOOD PROFILES

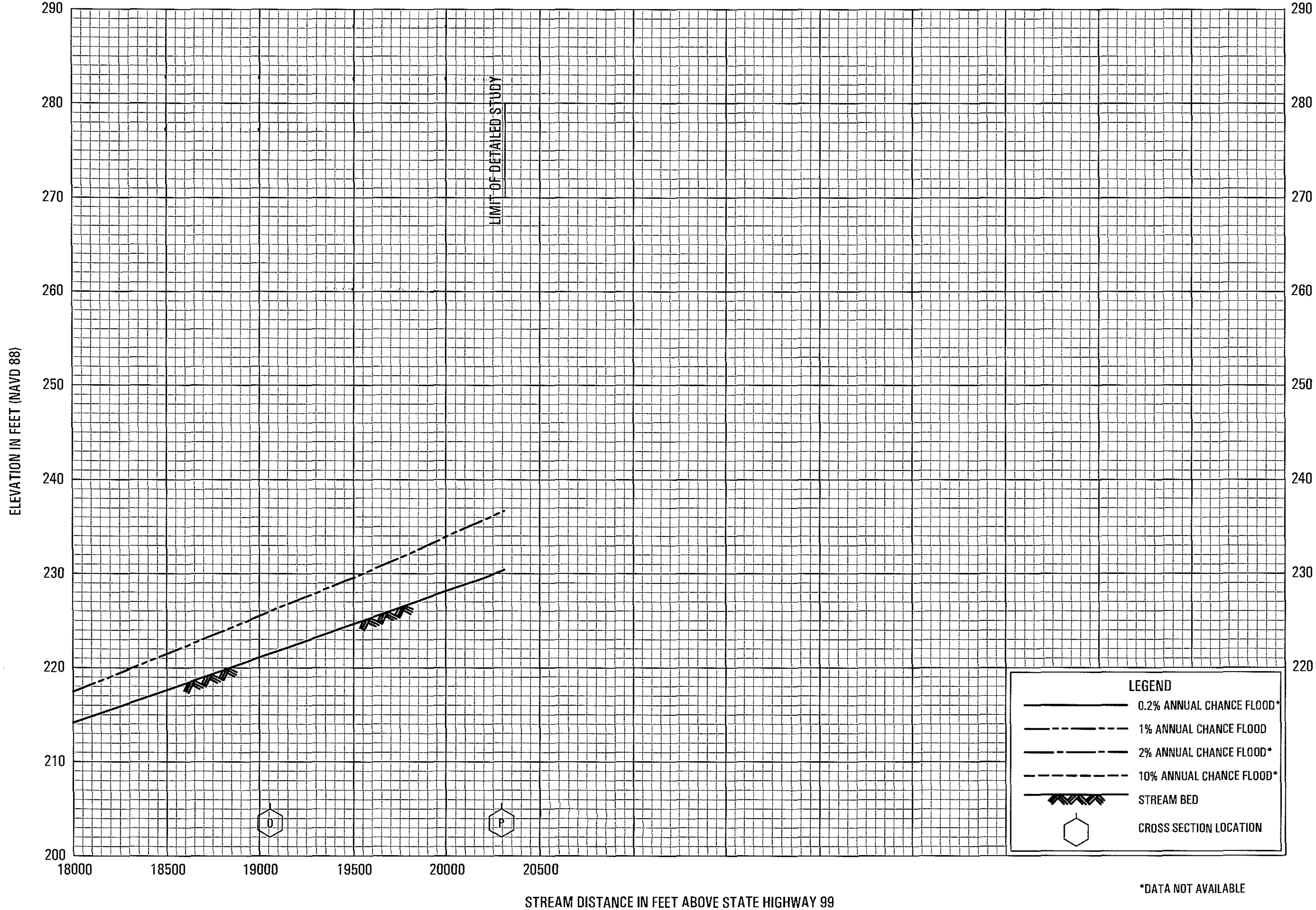
SYCAMORE CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

50P



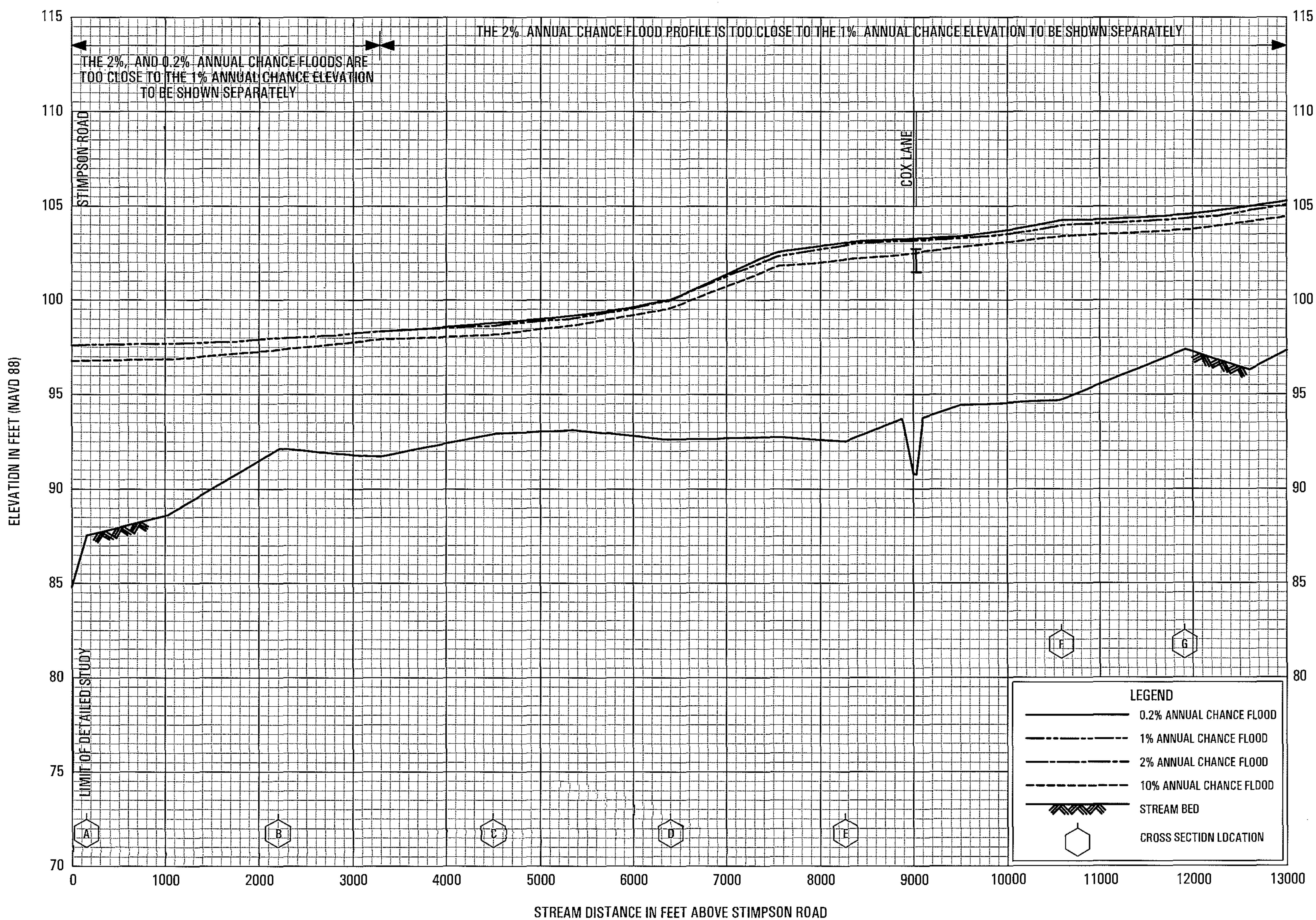
FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

FLOOD PROFILES

SYCAMORE CREEK



FLOOD PROFILES

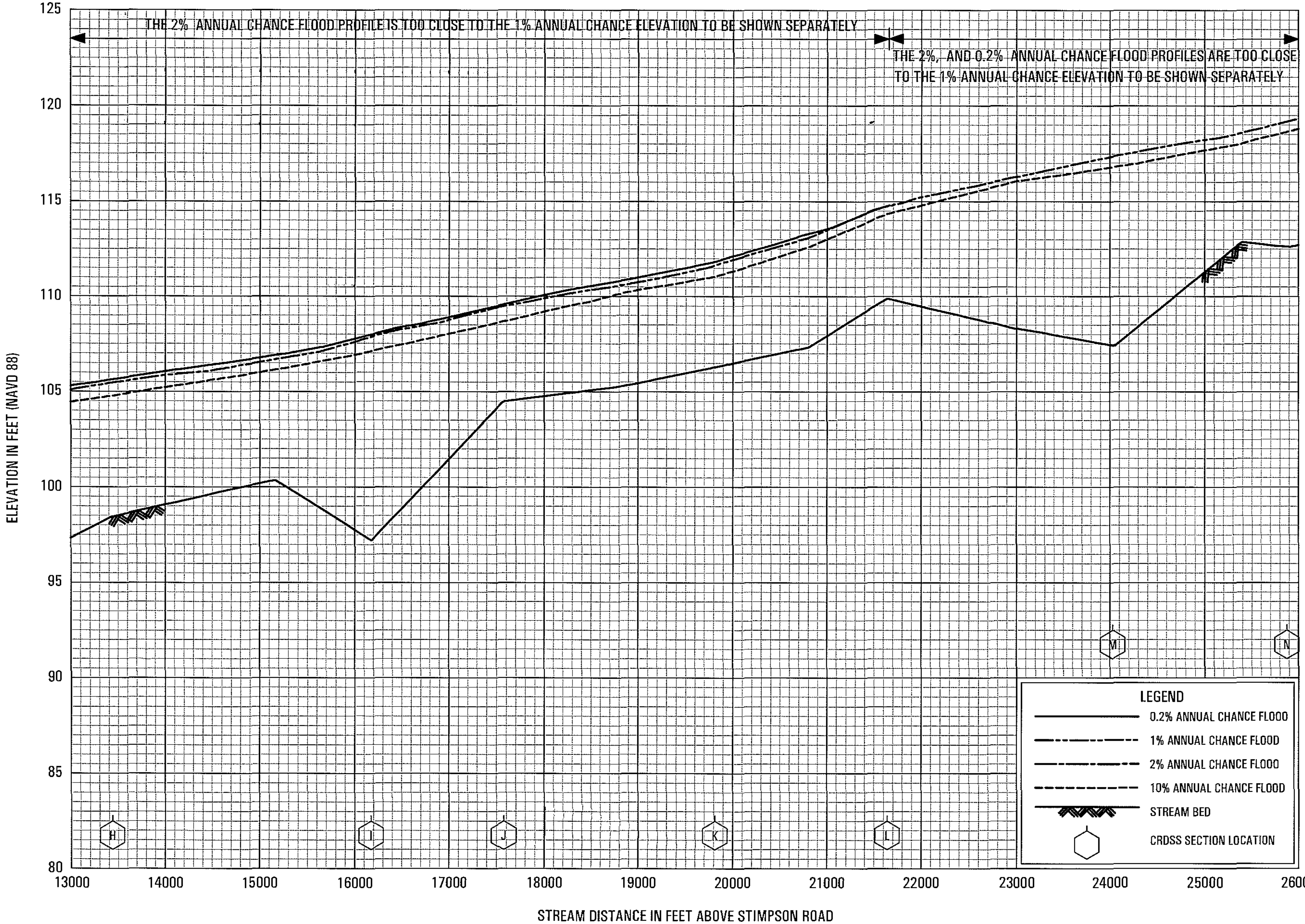
WYMAN RAVINE

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

52P



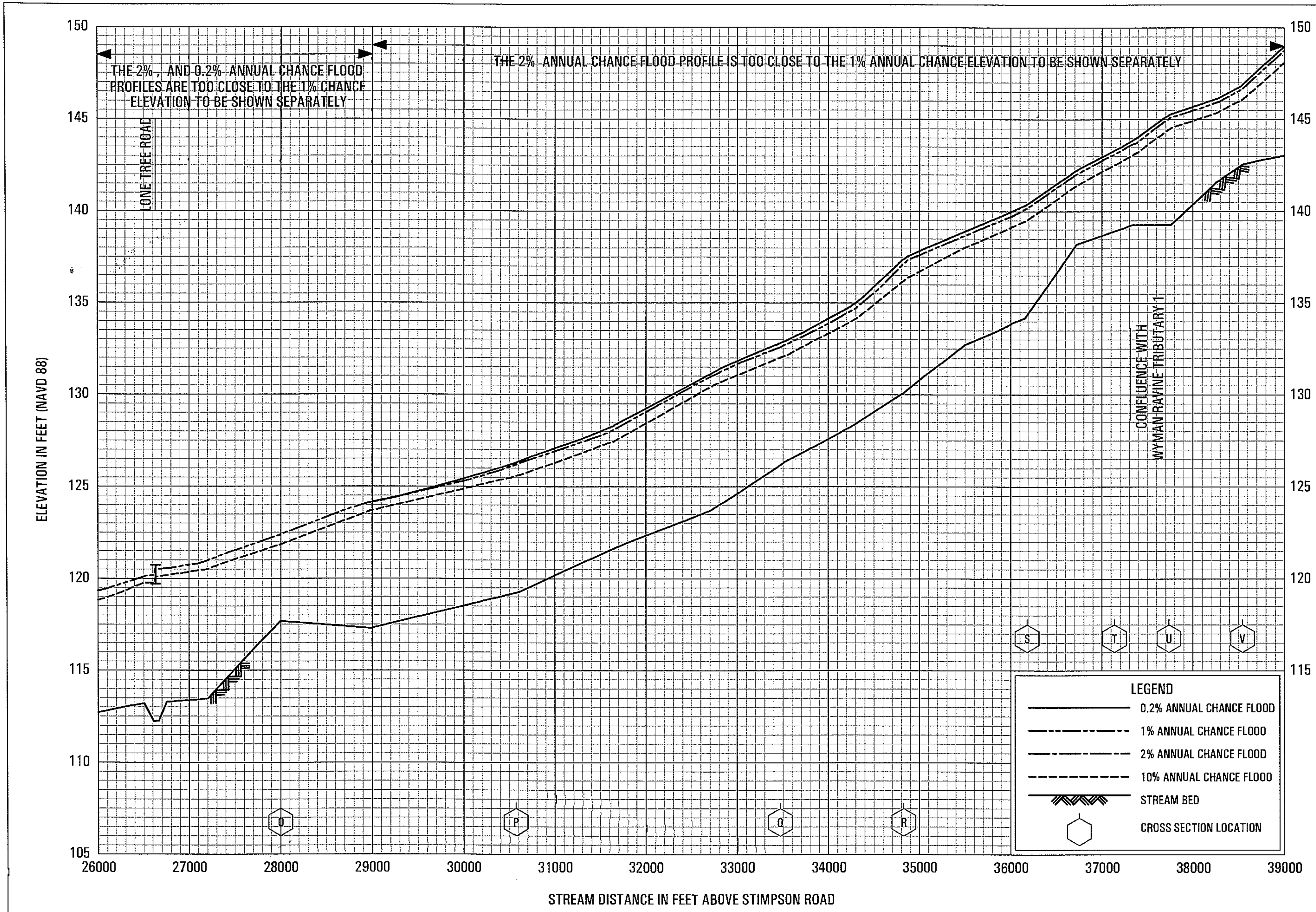
FLOOD PROFILES

WYMAN RAVINE

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



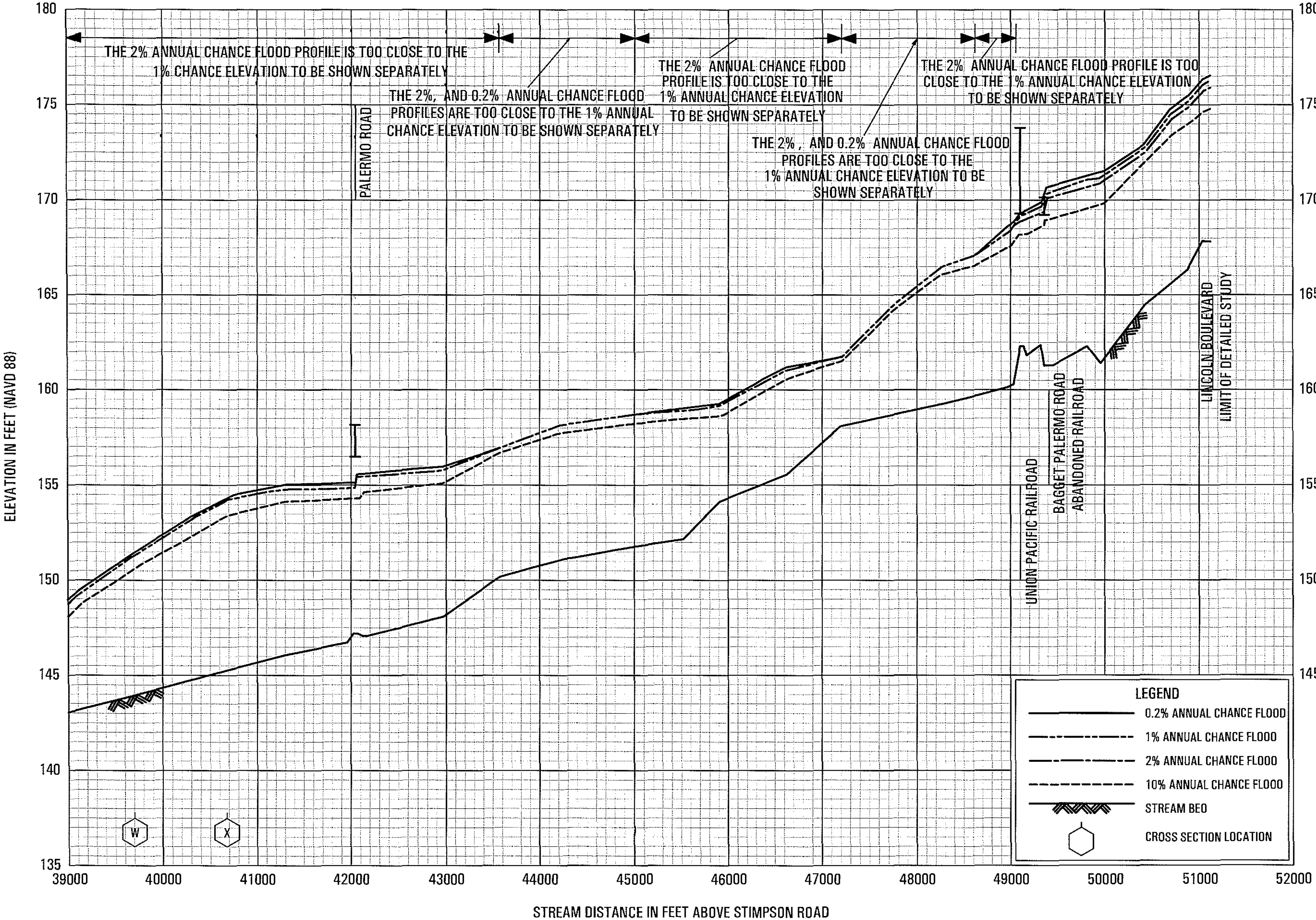
FLOOD PROFILES

WYMAN RAVINE

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



FLOOD PROFILES

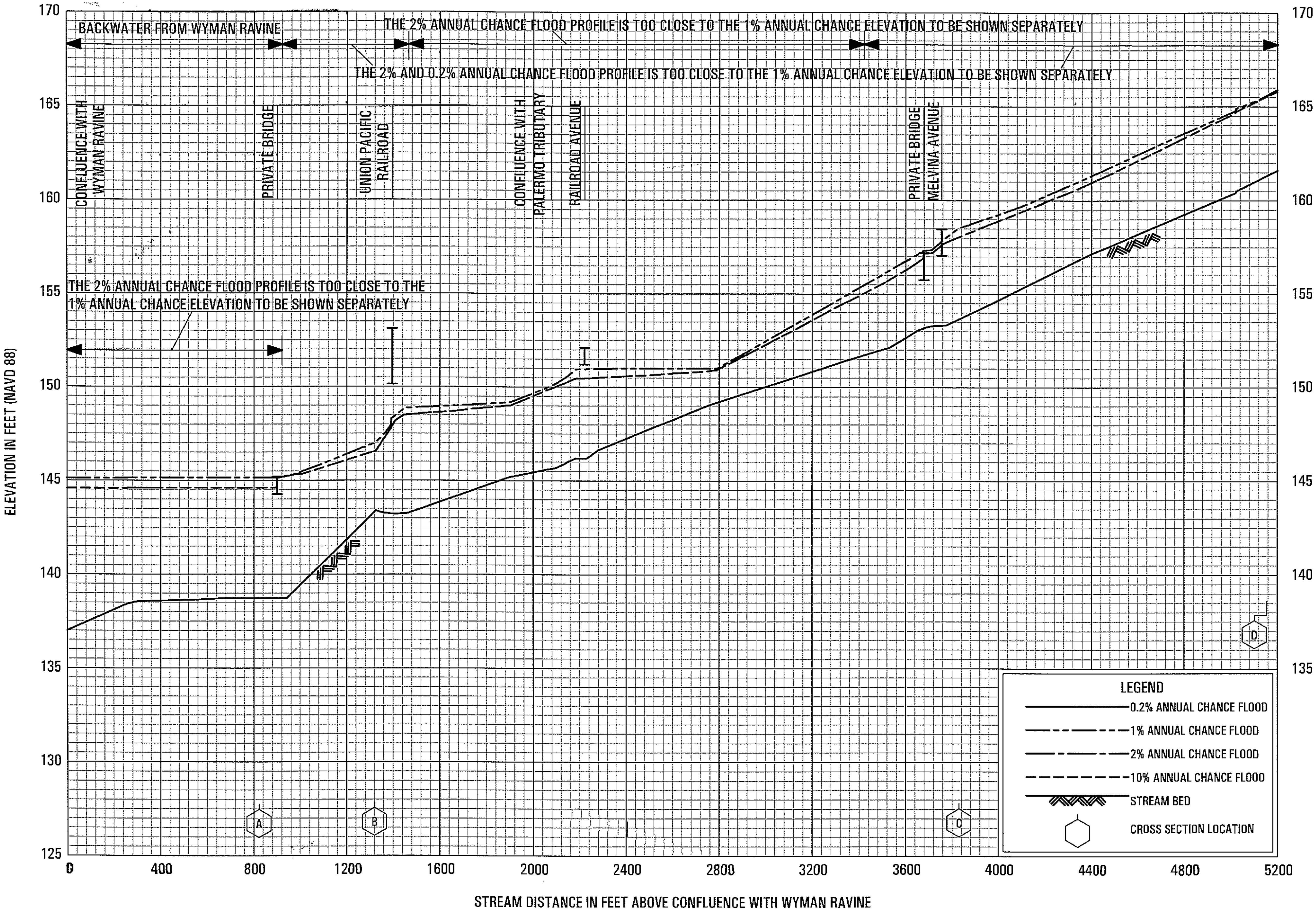
WYMAN RAVINE

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

55P



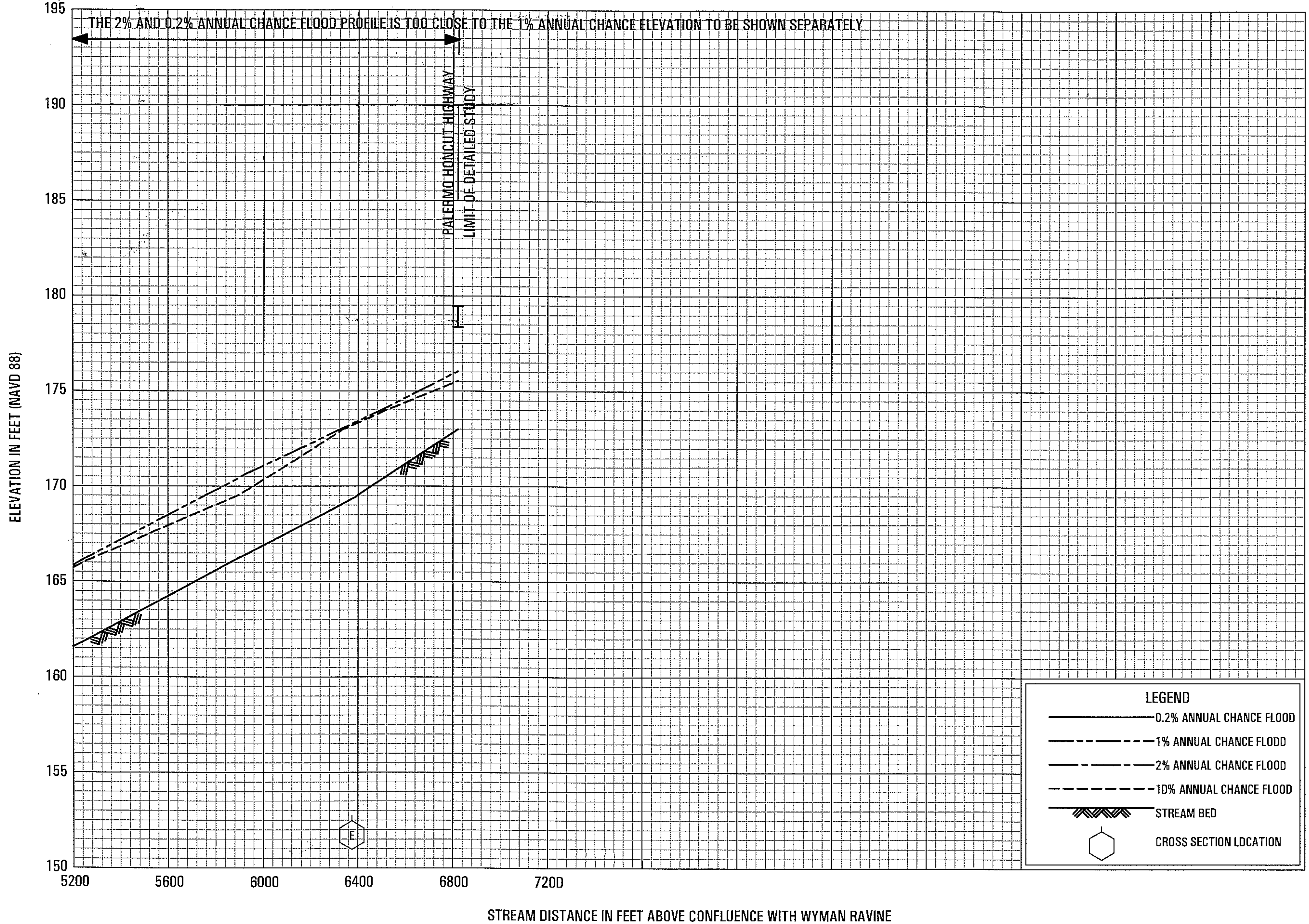
FLOOD PROFILES

WYMAN RAVINE TRIBUTARY 1

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS



FLOOD PROFILES

WYMAN RAVINE TRIBUTARY 1

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUTTE COUNTY, CA

AND INCORPORATED AREAS

APPENDIX E

HEC-RAS HYDRAULIC MODEL OUTPUT- EXISTING CONDITIONS

- Existing Conditions Model
    - HEC-RAS Channel Profile View
    - HEC-RAS Results Tables
    - HEC-RAS Bridge Results Table
    - HEC-RAS Cross-Sections

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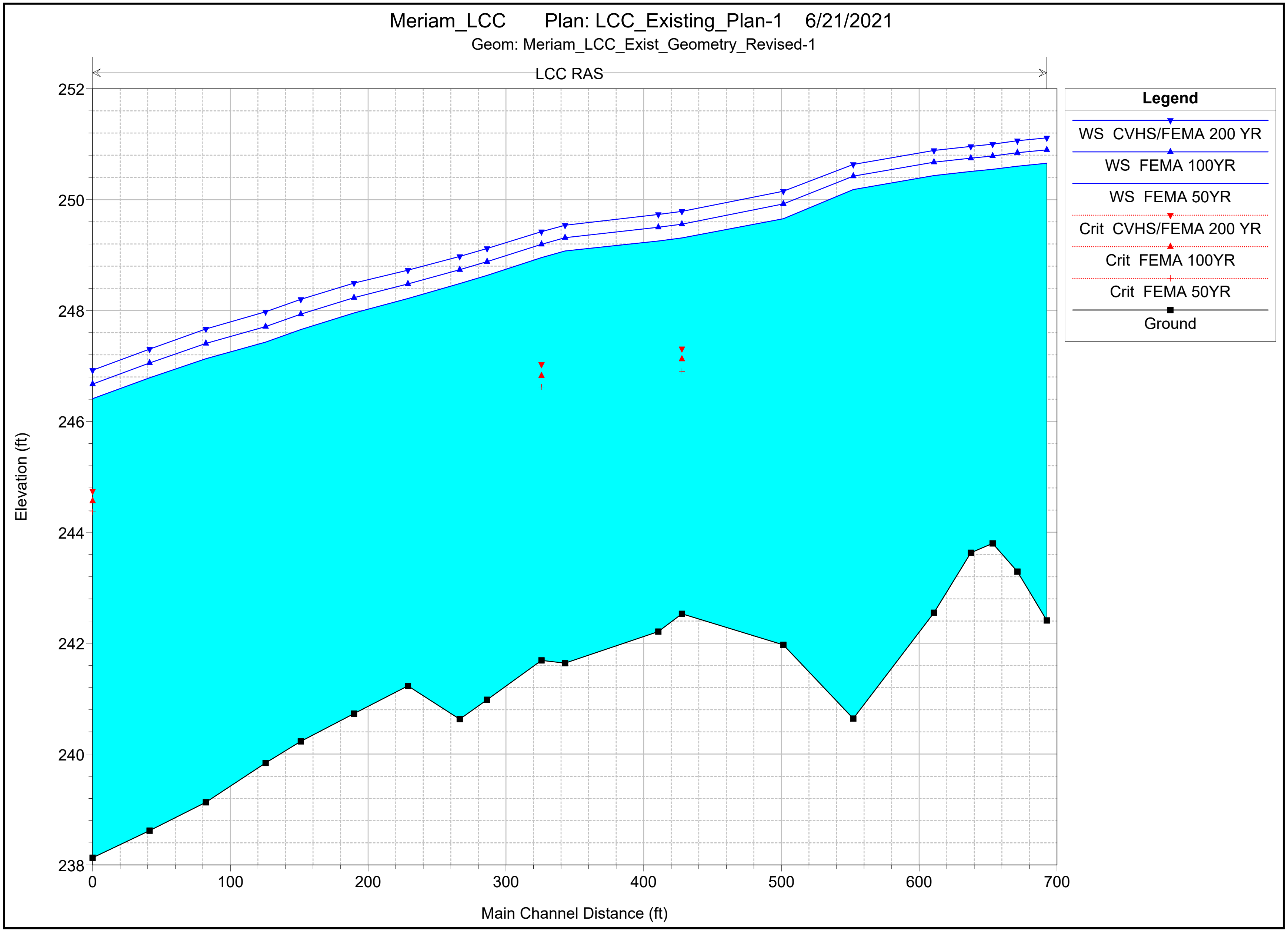
HEC-RAS Plan: Meriam_Exist_LCC-1 River: LCC Reach: RAS

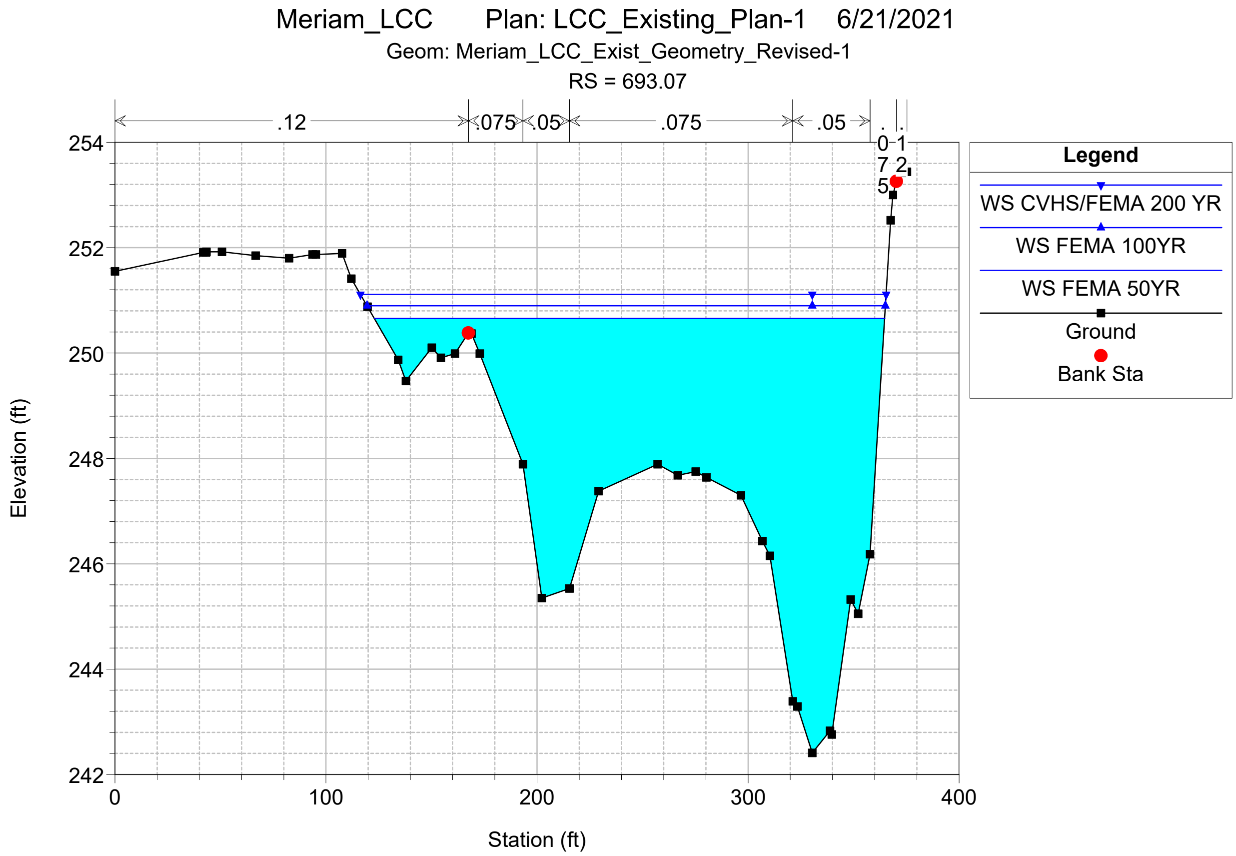
Reach	River Sta	Profile	Q Total	Q Left	Q Right	Q Channel	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Vel Left	Vel Right	Flow Area	Hydr Depth	Froude # Chl
			(cfs)	(cfs)	(cfs)	(cfs)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(ft/s)	(ft/s)	(sq ft)	(ft)	
RAS	693.07	FEMA 50YR	2000.00	12.62		1987.38	250.66		250.75	0.002132	2.52	0.43		818.73	3.39	0.22
RAS	693.07	FEMA 100YR	2200.00	20.53		2179.47	250.90		251.00	0.002116	2.60	0.51		877.69	3.57	0.22
RAS	693.07	CVHS/FEMA 200 YR	2400.00	29.17		2370.83	251.11		251.22	0.002129	2.69	0.57		930.91	3.74	0.23
RAS	671.79	FEMA 50YR	2000.00			2000.00	250.60		250.70	0.002953	2.51			797.11	3.35	0.24
RAS	671.79	FEMA 100YR	2200.00			2200.00	250.85		250.95	0.002905	2.57			855.45	3.55	0.24
RAS	671.79	CVHS/FEMA 200 YR	2400.00			2400.00	251.06		251.17	0.002909	2.64			907.64	3.74	0.24
RAS	653.79	FEMA 50YR	2000.00	35.83		1964.17	250.55		250.66	0.002070	2.68	0.55		797.73	3.42	0.23
RAS	653.79	FEMA 100YR	2200.00	51.41		2148.59	250.79		250.90	0.002082	2.78	0.63		854.16	3.63	0.23
RAS	653.79	CVHS/FEMA 200 YR	2400.00	66.57		2333.43	251.00		251.13	0.002123	2.89	0.69		904.52	3.78	0.23
RAS	637.96	FEMA 50YR	2000.00	29.53		1970.47	250.51		250.62	0.001873	2.75	0.49		776.20	3.37	0.23
RAS	637.96	FEMA 100YR	2200.00	42.36		2157.64	250.75		250.87	0.001892	2.86	0.55		831.84	3.50	0.23
RAS	637.96	CVHS/FEMA 200 YR	2400.00	54.12		2345.88	250.96		251.09	0.001937	2.97	0.58		883.26	3.50	0.24
RAS	611.2	FEMA 50YR	2000.00	91.71		1908.29	250.43		250.56	0.002791	2.94	0.74		773.55	2.99	0.25
RAS	611.2	FEMA 100YR	2200.00	121.55		2078.45	250.68		250.81	0.002757	3.03	0.81		836.56	3.19	0.25
RAS	611.2	CVHS/FEMA 200 YR	2400.00	152.32		2247.68	250.89		251.03	0.002768	3.12	0.88		892.25	3.38	0.26
RAS	552.6	FEMA 50YR	2000.00	138.34		1861.66	250.18		250.37	0.003816	3.58	0.88		677.96	2.73	0.30
RAS	552.6	FEMA 100YR	2200.00	179.75		2020.25	250.42		250.62	0.003759	3.67	0.95		739.84	2.88	0.30
RAS	552.6	CVHS/FEMA 200 YR	2400.00	225.05		2174.95	250.63		250.84	0.003762	3.78	1.03		794.05	3.06	0.31
RAS	501.91	FEMA 50YR	2000.00	124.44		1875.57	249.65		250.05	0.010170	5.19	1.13		472.14	2.06	0.48
RAS	501.91	FEMA 100YR	2200.00	186.85		2013.15	249.92		250.30	0.010170	5.16	1.27		537.51	2.13	0.48
RAS	501.91	CVHS/FEMA 200 YR	2400.00	245.91		2154.09	250.15		250.53	0.009542	5.19	1.36		596.25	2.28	0.47
RAS	428.29	FEMA 50YR	2000.00	58.94		1941.06	249.31	246.90	249.60	0.003659	4.36	0.65		535.16	2.37	0.39
RAS	428.29	FEMA 100YR	2200.00	86.53		2113.47	249.56	247.12	249.85	0.003705	4.45	0.72		594.77	2.34	0.39
RAS	428.29	CVHS/FEMA 200 YR	2400.00	117.92		2282.08	249.79	247.31	250.09	0.003723	4.54	0.77		655.71	2.43	0.39
RAS	411.11	FEMA 50YR	2000.00	61.84		1938.16	249.25		249.53	0.003347	4.30	0.63		548.94	2.39	0.38
RAS	411.11	FEMA 100YR	2200.00	90.04		2109.96	249.50		249.79	0.003426	4.41	0.69		608.98	2.39	0.38
RAS	411.11	CVHS/FEMA 200 YR	2400.00	127.78	0.02	2272.20	249.73		250.03	0.003444	4.49	0.78	0.14	669.01	2.52	0.38
RAS	343.42	FEMA 50YR	2000.00	54.90	0.00	1945.09	249.07		249.32	0.002684	4.05	0.62	0.09	569.04	2.73	0.35
RAS	343.42	FEMA 100YR	2200.00	77.15	0.06	2122.79	249.31		249.58	0.002760	4.17	0.69	0.20	621.07	2.74	0.35
RAS	343.42	CVHS/FEMA 200 YR	2400.00	101.05	0.24	2298.72	249.54		249.81	0.002832	4.30	0.73	0.28	672.97	2.85	0.35
RAS	326.31	FEMA 50YR	2000.00	138.47		1861.53	248.95	246.62	249.25	0.006197	4.52	1.05		543.61	2.56	0.38
RAS	326.31	FEMA 100YR	2200.00	181.78		2018.22	249.19	246.82	249.50	0.006294	4.64	1.13		596.80	2.61	0.39
RAS	326.31	CVHS/FEMA 200 YR	2400.00	235.42	0.00	2164.58	249.42	247.03	249.74	0.006198	4.73	1.22	0.09	650.29	2.77	0.39
RAS	286.88	FEMA 50YR	2000.00	127.94	0.48	1871.58	248.63		248.96	0.008723	4.72	1.27	3.01	496.94	2.57	0.43
RAS	286.88	FEMA 100YR	2200.00	166.11	2.63	2031.26	248.88		249.22	0.008292	4.81	1.35	4.52	546.18	2.72	0.42
RAS	286.88	CVHS/FEMA 200 YR	2400.00	207.64	6.98	2185.38	249.12		249.46	0.007922	4.88	1.43	5.67	594.62	2.87	0.42
RAS	266.95	FEMA 50YR	2000.00	137.03		1862.97	248.49		248.80	0.006797	4.66	1.20		513.34	2.81	0.40
RAS	266.95	FEMA 100YR	2200.00	175.93		2024.07	248.74		249.07	0.006727	4.78	1.29		560.21	2.93	0.40
RAS	266.95	CVHS/FEMA 200 YR	2400.00	217.75	0.01	2182.24	248.98		249.32	0.006612	4.89	1.36	0.17	606.73	3.06	0.40

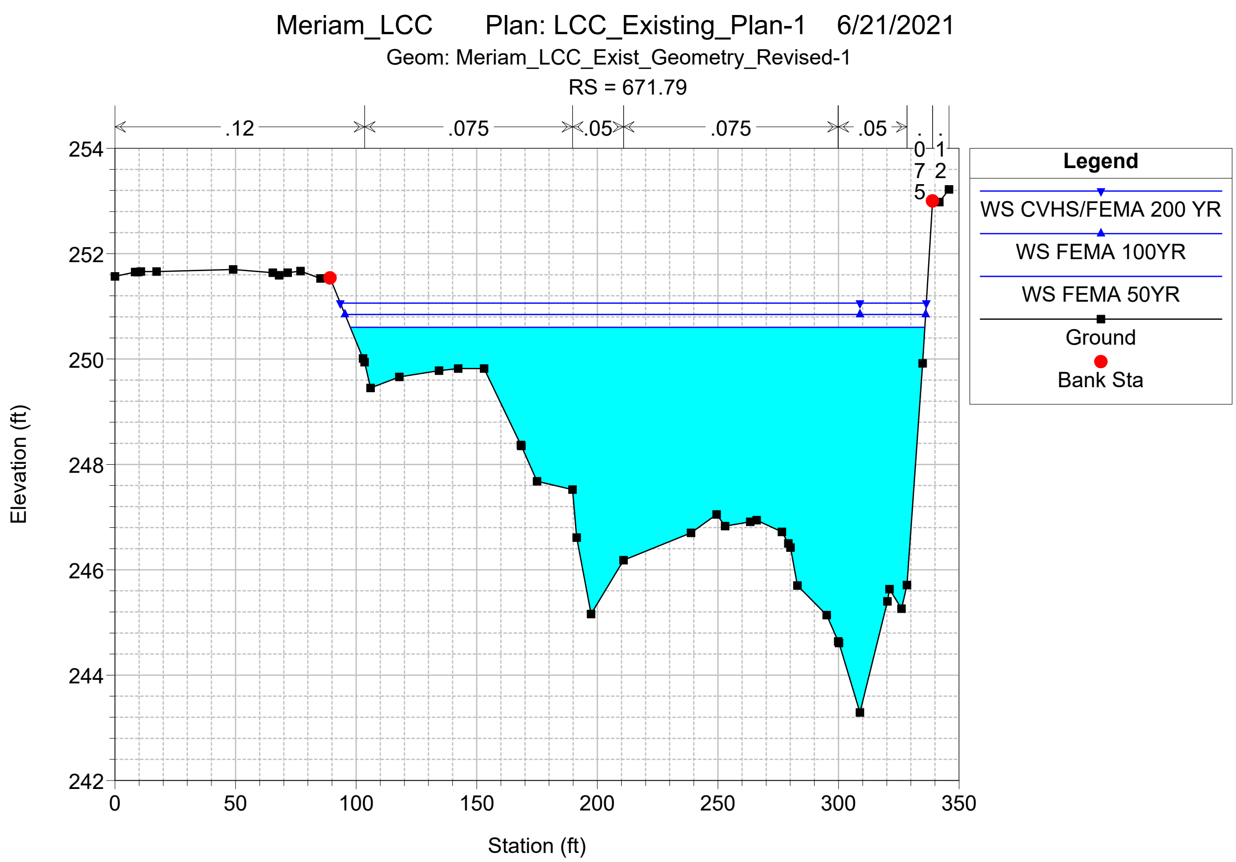
HEC-RAS Plan: Meriam_Exist_LCC-1 River: LCC Reach: RAS (Continued)

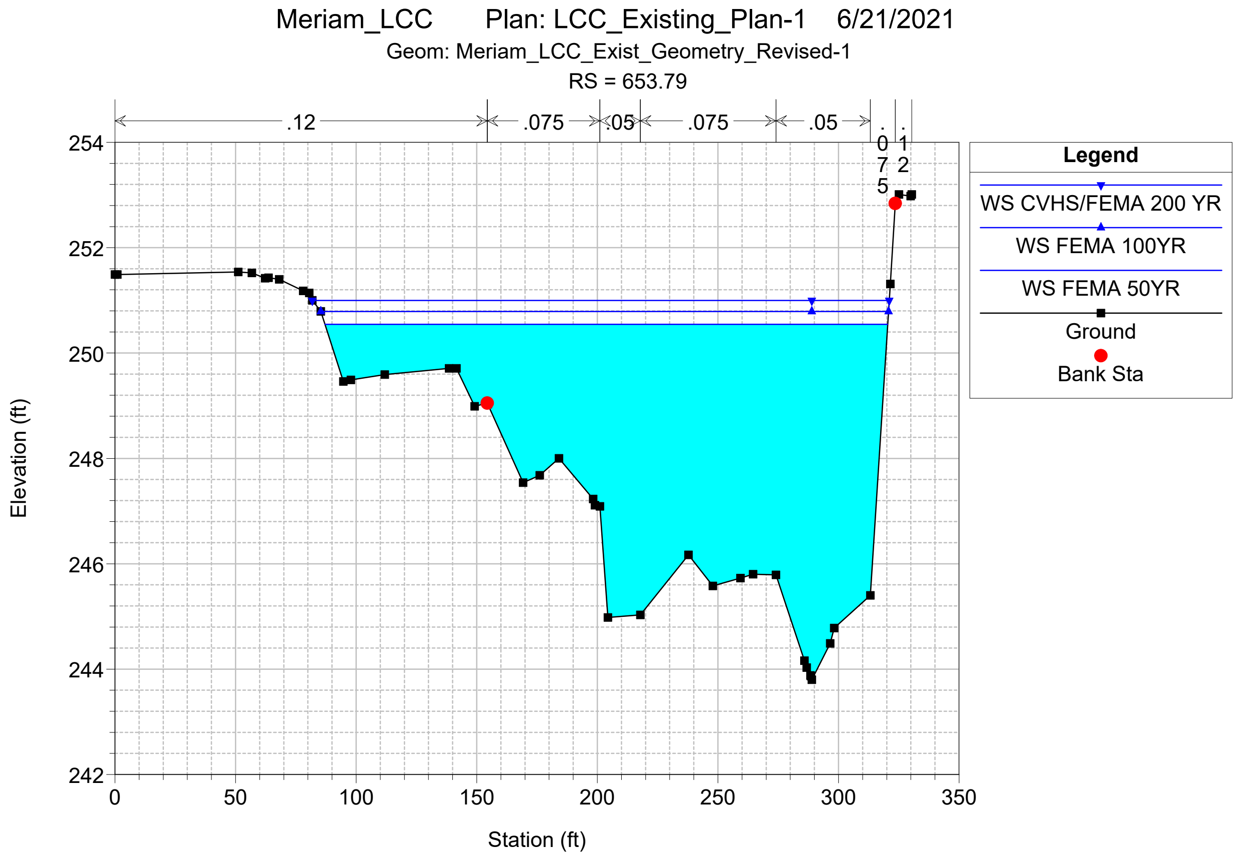
Reach	River Sta	Profile	Q Total	Q Left	Q Right	Q Channel	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Vel Left	Vel Right	Flow Area	Hydr Depth	Froude # Chl
			(cfs)	(cfs)	(cfs)	(cfs)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(ft/s)	(ft/s)	(sq ft)	(ft)	
RAS	229.39	FEMA 50YR	2000.00	88.76		1911.25	248.22		248.54	0.007122	4.66	1.09		491.66	2.70	0.42
RAS	229.39	FEMA 100YR	2200.00	123.32		2076.68	248.48		248.81	0.006837	4.74	1.21		539.78	2.91	0.41
RAS	229.39	CVHS/FEMA 200 YR	2400.00	160.05		2239.95	248.73		249.07	0.006611	4.83	1.31		586.22	3.11	0.41
RAS	190.29	FEMA 50YR	2000.00	19.17		1980.83	247.96		248.26	0.007109	4.42	0.60		479.80	2.50	0.40
RAS	190.29	FEMA 100YR	2200.00	41.56		2158.44	248.23		248.54	0.006710	4.48	0.80		533.70	2.72	0.40
RAS	190.29	CVHS/FEMA 200 YR	2400.00	68.20	0.00	2331.81	248.49		248.81	0.006385	4.54	0.95	0.05	585.22	2.95	0.39
RAS	151.66	FEMA 50YR	2000.00	24.78		1975.22	247.66		247.98	0.007143	4.58	0.83		461.41	2.88	0.40
RAS	151.66	FEMA 100YR	2200.00	37.46		2162.54	247.93		248.27	0.006946	4.70	0.75		510.33	2.72	0.39
RAS	151.66	CVHS/FEMA 200 YR	2400.00	62.11	0.00	2337.89	248.20		248.55	0.006709	4.79	0.82	0.11	563.42	2.72	0.39
RAS	126.11	FEMA 50YR	2000.00	2.76		1997.24	247.43		247.79	0.007757	4.78	0.49		423.47	3.19	0.41
RAS	126.11	FEMA 100YR	2200.00	9.77		2190.23	247.71		248.08	0.007549	4.91	0.54		464.07	2.94	0.41
RAS	126.11	CVHS/FEMA 200 YR	2400.00	23.76	0.00	2376.24	247.98		248.37	0.007337	5.02	0.67	0.09	508.53	2.92	0.41
RAS	82.75	FEMA 50YR	2000.00	0.42		1999.58	247.13		247.49	0.006201	4.77	0.35		420.03	4.27	0.39
RAS	82.75	FEMA 100YR	2200.00	2.18		2197.82	247.41		247.79	0.006214	4.95	0.40		449.62	3.84	0.40
RAS	82.75	CVHS/FEMA 200 YR	2400.00	6.85		2393.15	247.67		248.07	0.006226	5.11	0.48		482.55	3.54	0.40
RAS	41.88	FEMA 50YR	2000.00	0.35		1999.65	246.79		247.20	0.007728	5.15	0.37		389.60	4.34	0.42
RAS	41.88	FEMA 100YR	2200.00	1.38	0.00	2198.62	247.05		247.50	0.007796	5.35	0.52	0.04	413.99	4.43	0.43
RAS	41.88	CVHS/FEMA 200 YR	2400.00	3.27	0.13	2396.60	247.30		247.78	0.007816	5.54	0.65	0.28	438.35	4.39	0.43
RAS	0.5	FEMA 50YR	2000.00		0.00	2000.00	246.41	244.37	246.85	0.009004	5.32		0.13	375.84	4.39	0.45
RAS	0.5	FEMA 100YR	2200.00	0.07	0.26	2199.68	246.67	244.56	247.15	0.009014	5.53	0.24	0.35	399.07	4.33	0.45
RAS	0.5	CVHS/FEMA 200 YR	2400.00	0.65	1.20	2398.15	246.92	244.75	247.43	0.009012	5.72	0.42	0.61	422.77	4.36	0.45



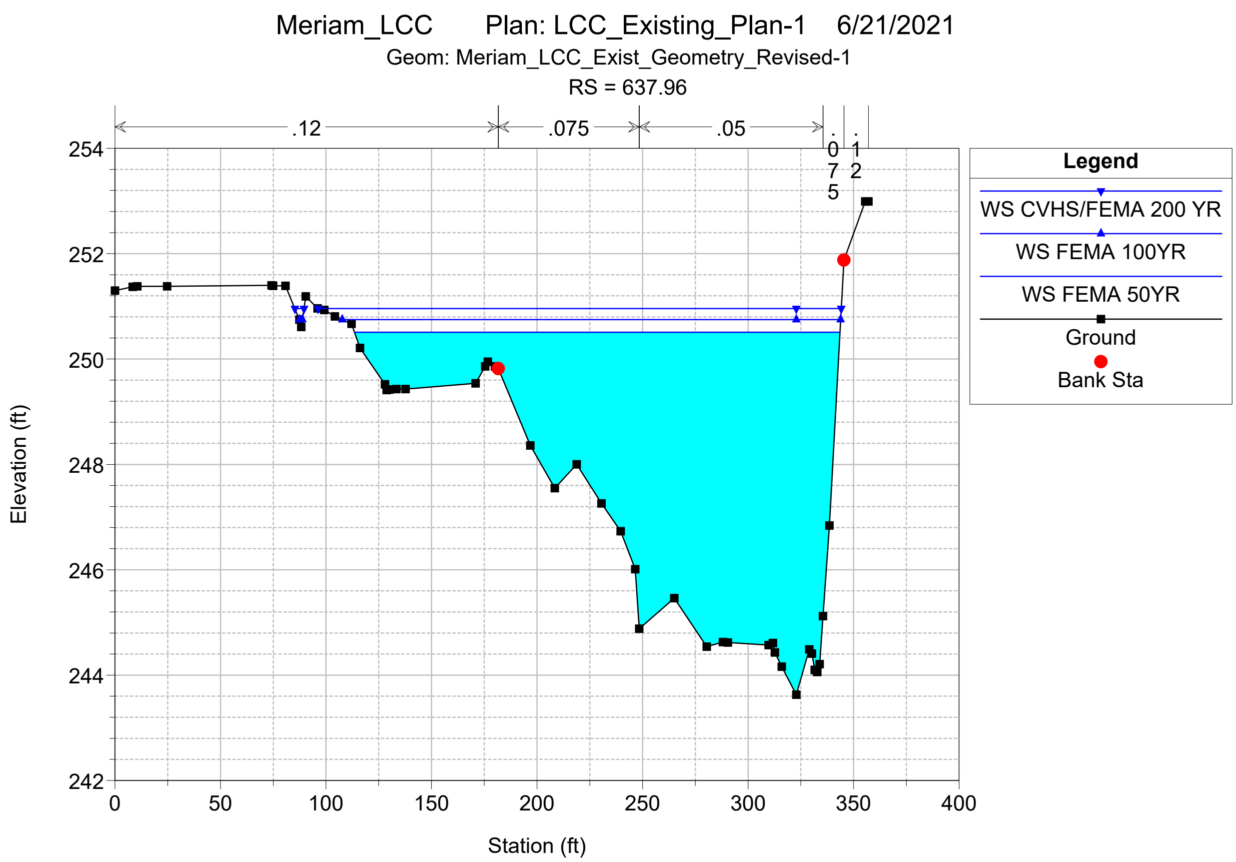
Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
LCC RAS
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Crit CVHS/FEMA 200 YR
Crit FEMA 100YR
Crit FEMA 50YR
Ground
Elevation (ft)
252
250
248
246
244
242
240
238
0
100
200
300
400
500
600
700
Main Channel Distance (ft)


Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 693.07
.12
.075
.05
.075
.05
.075
.12
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)

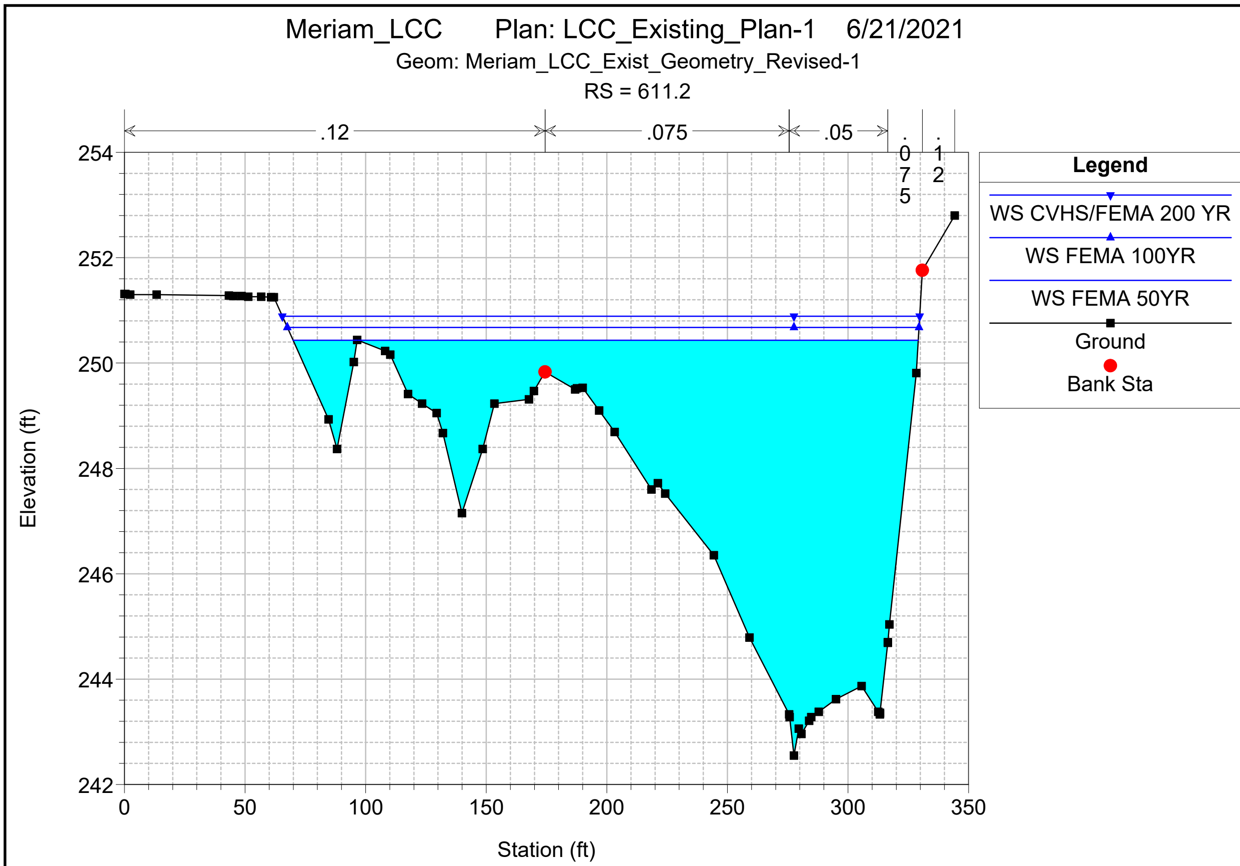

Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 671.79
.12
.075
.05
.075
.05
.075
.12
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)

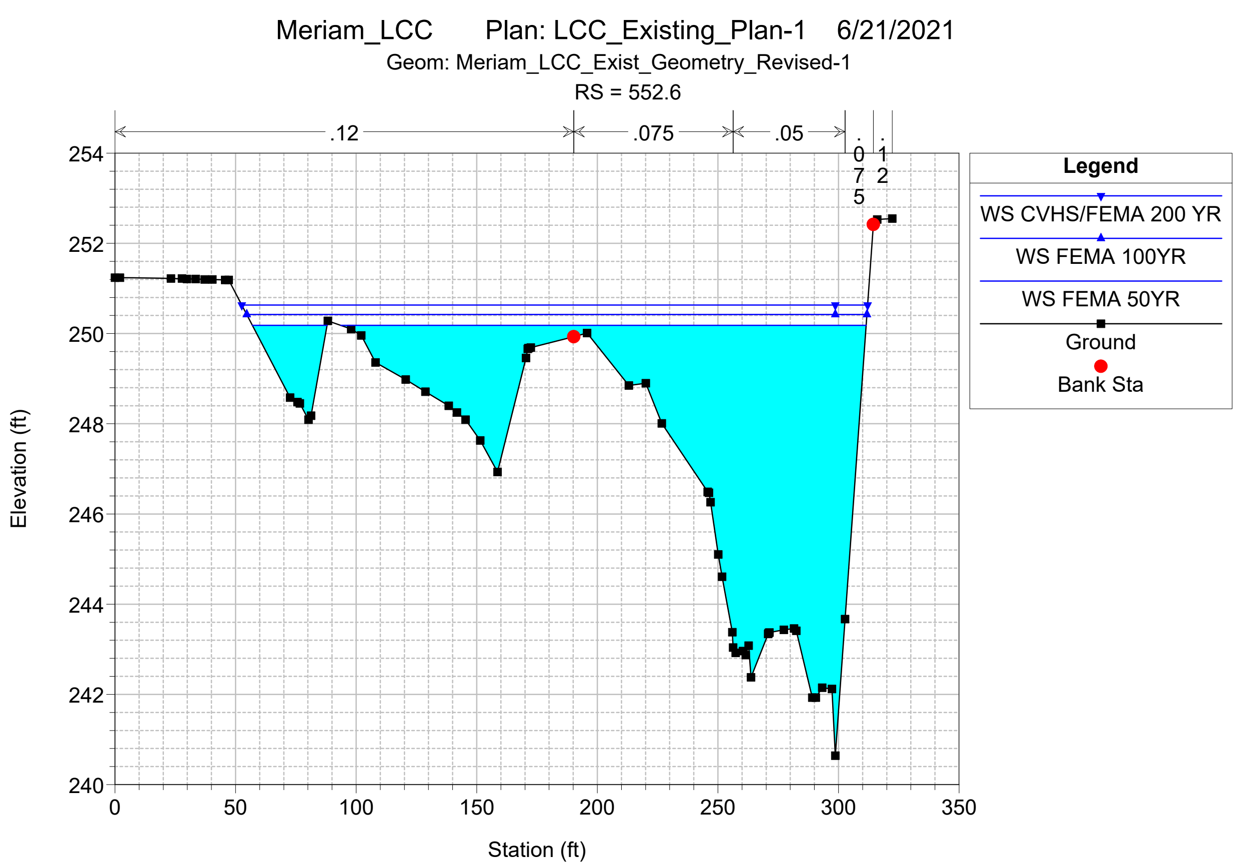


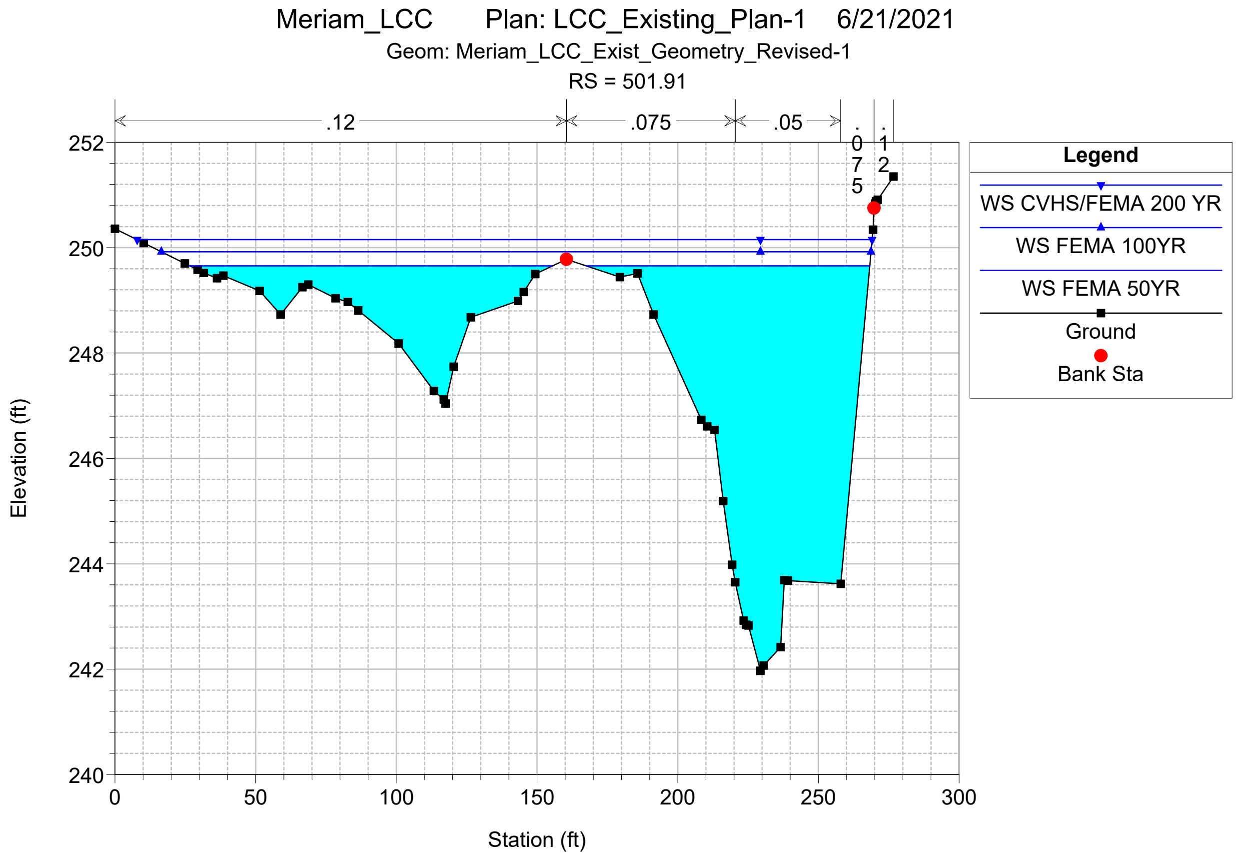
Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 653.79
.12
.075
.05
.075
.05
.075
.12
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)

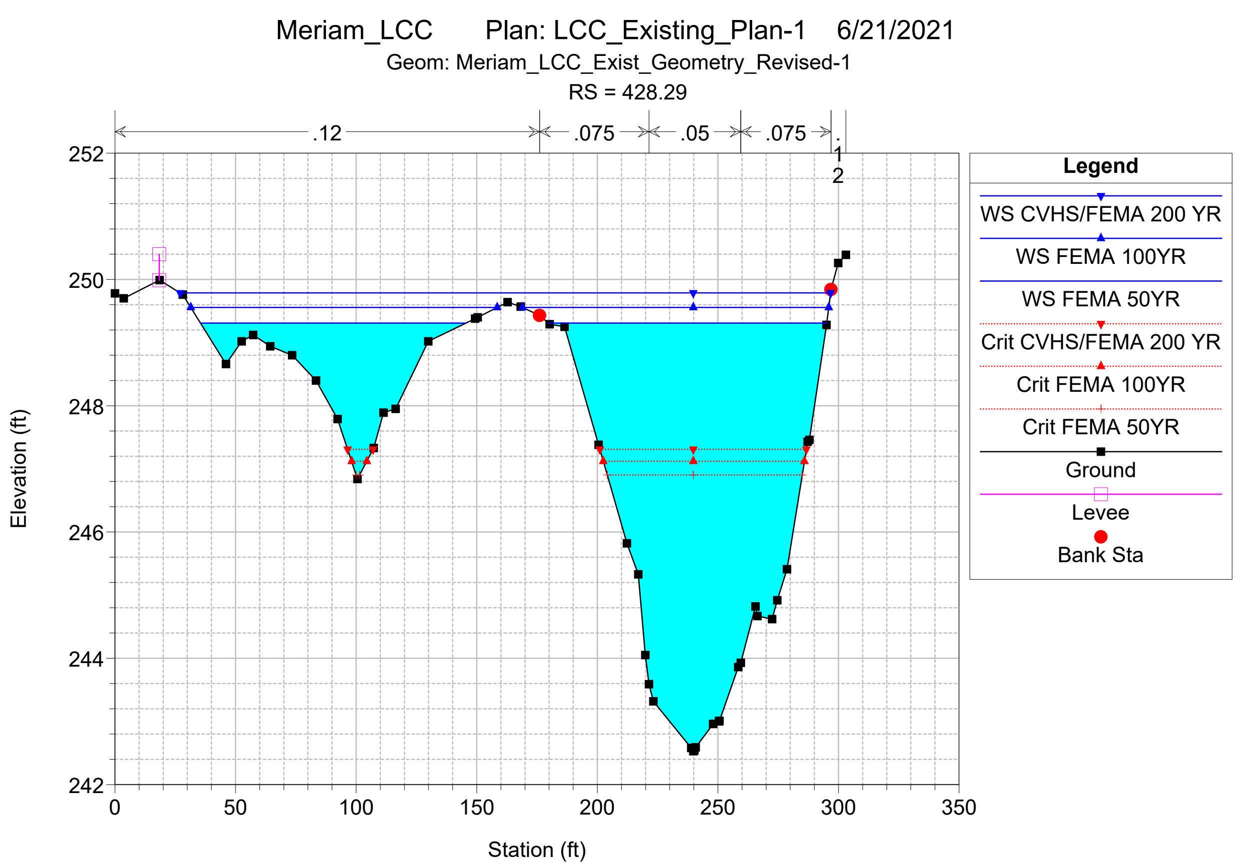


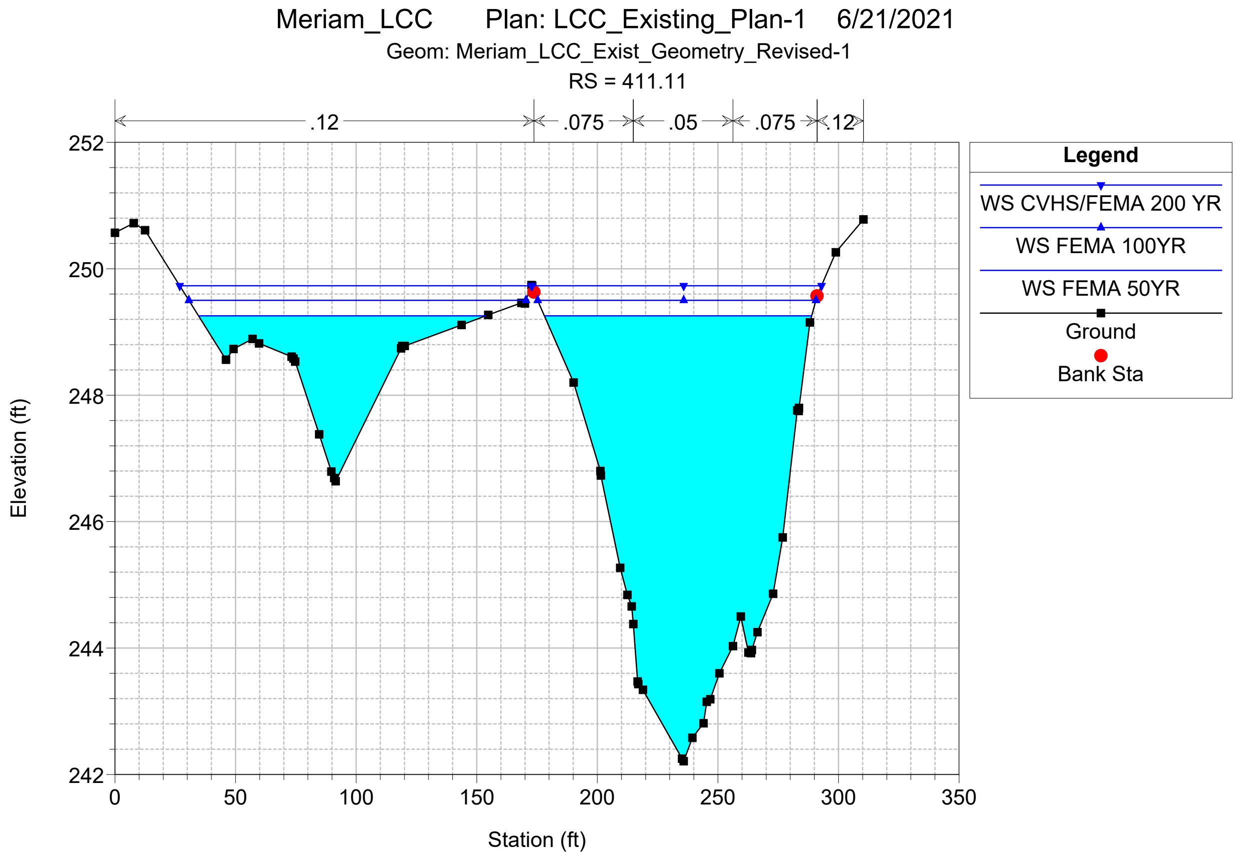
Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 637.96
.12
.075
.05
.075
.12
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)

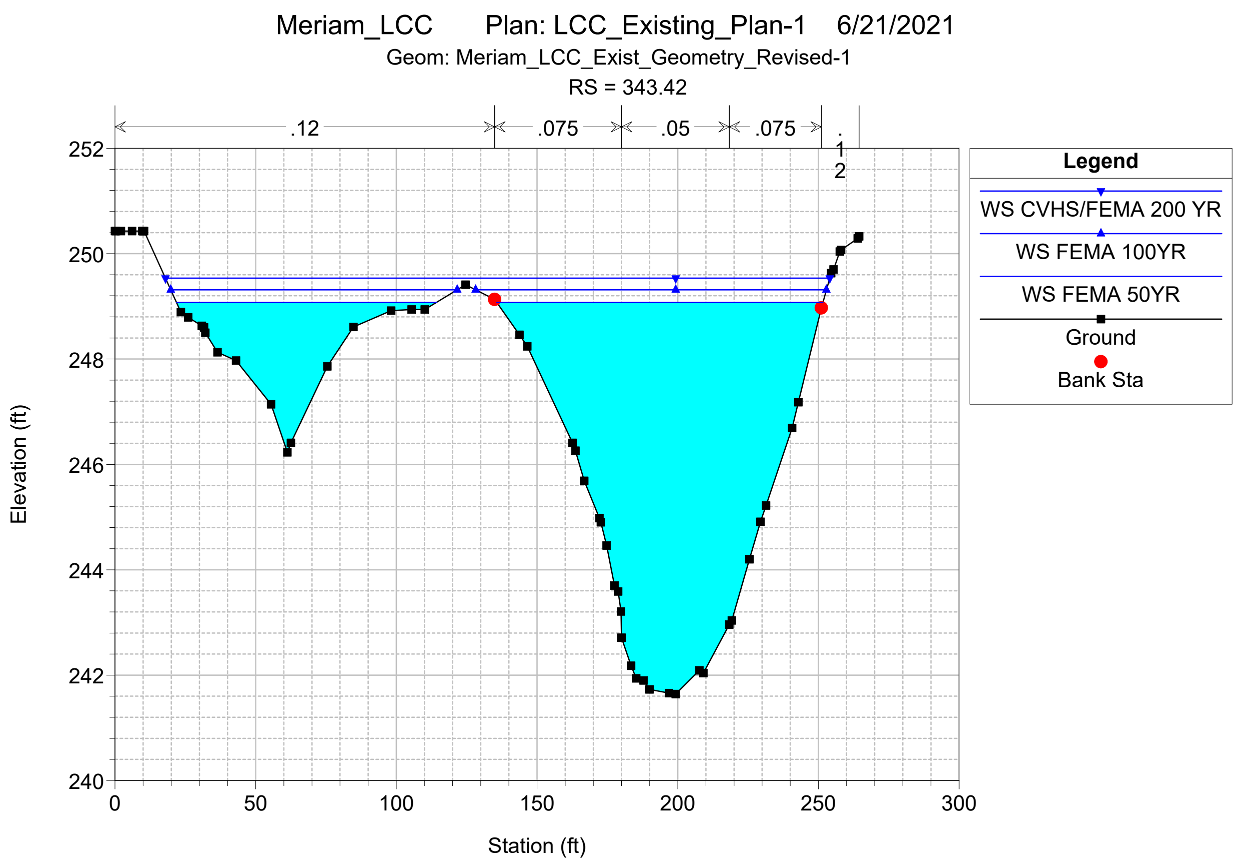

Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 611.2
.12
.075
.05
.075
.12
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)
254
252
250
248
246
244
242
0
50
100
150
200
250
300
350

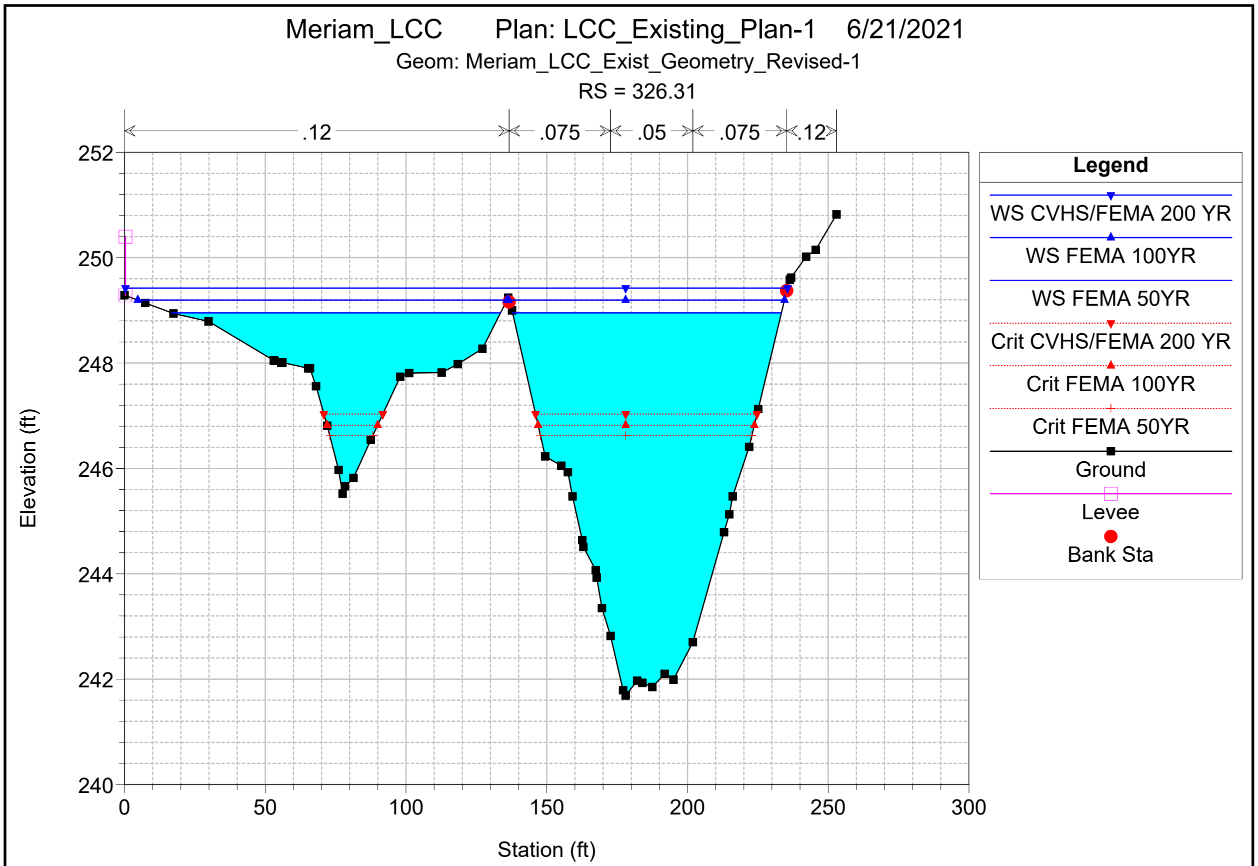

Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 552.6
.12
.075
.05
.075
.12
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)
254
252
250
248
246
244
242
240
0
50
100
150
200
250
300
350

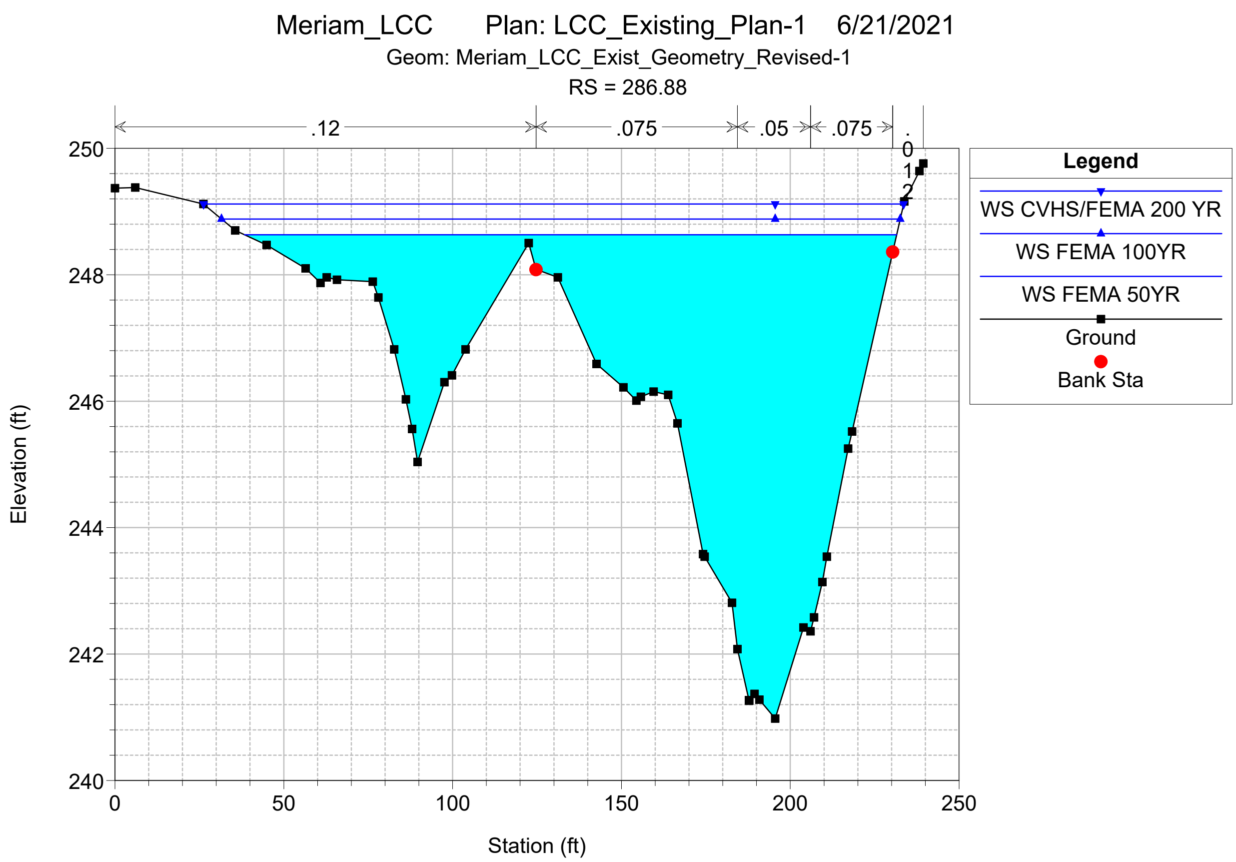

Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 501.91
.12
.075
.05
.075
.12
Elevation (ft)
Station (ft)
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta

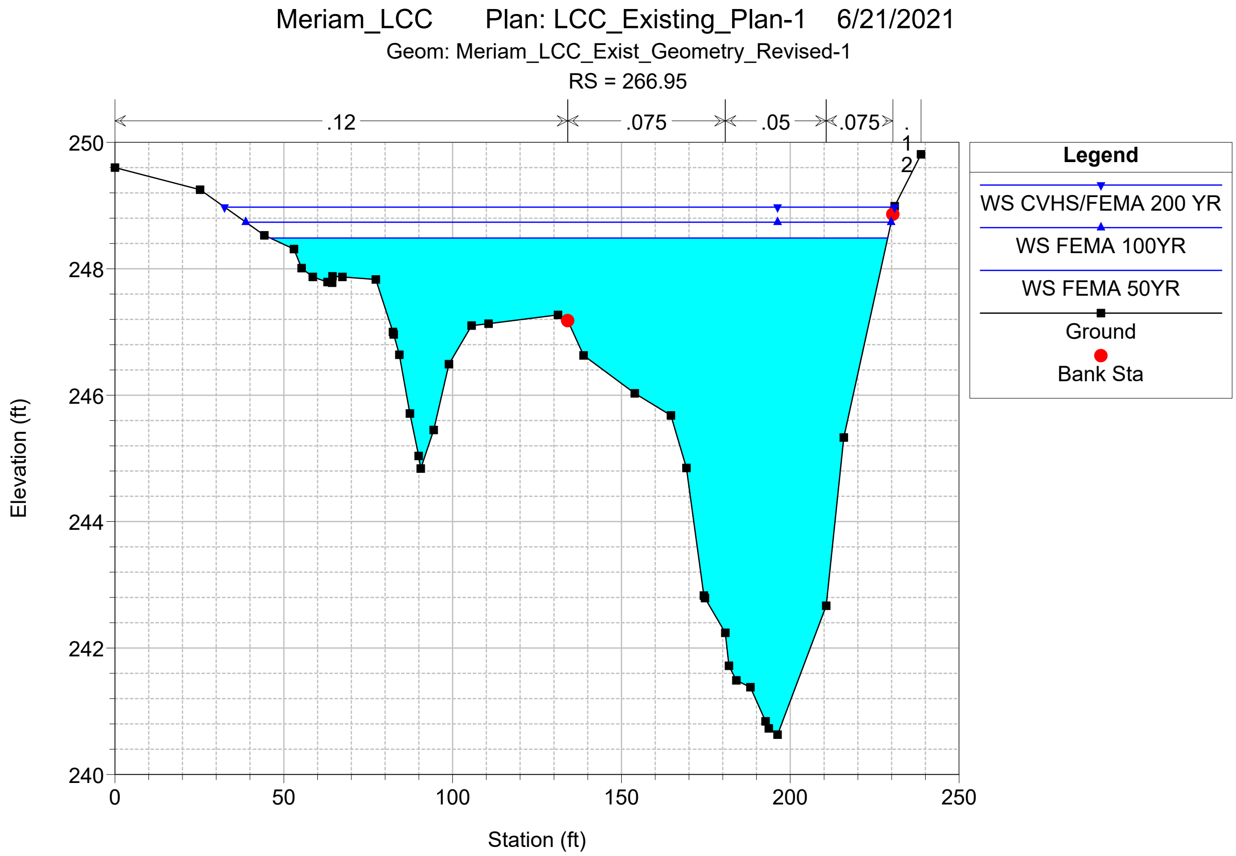

Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 428.29
.12
.075
.05
.075
.12
Elevation (ft)
Station (ft)
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Crit CVHS/FEMA 200 YR
Crit FEMA 100YR
Crit FEMA 50YR
Ground
Levee
Bank Sta

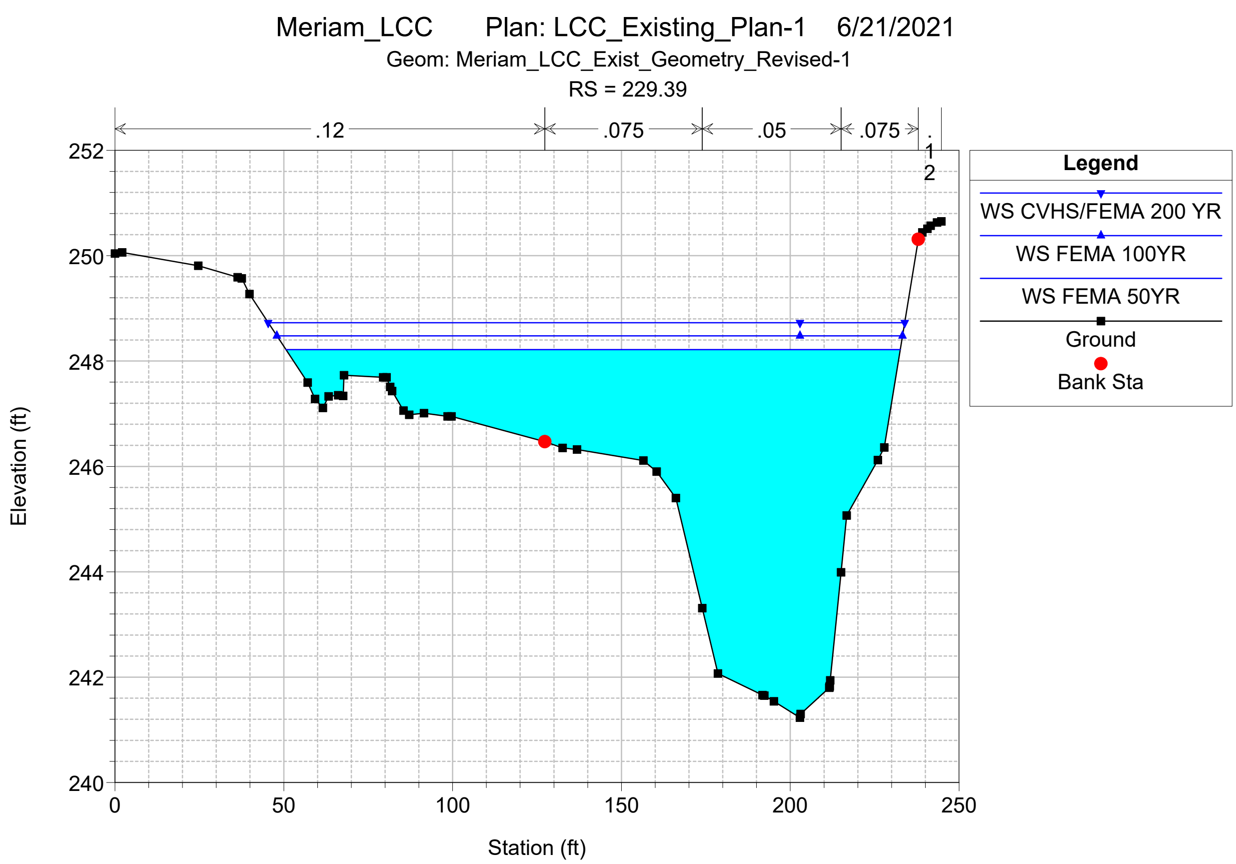

Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 411.11
.12
.075
.05
.075
.12
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)

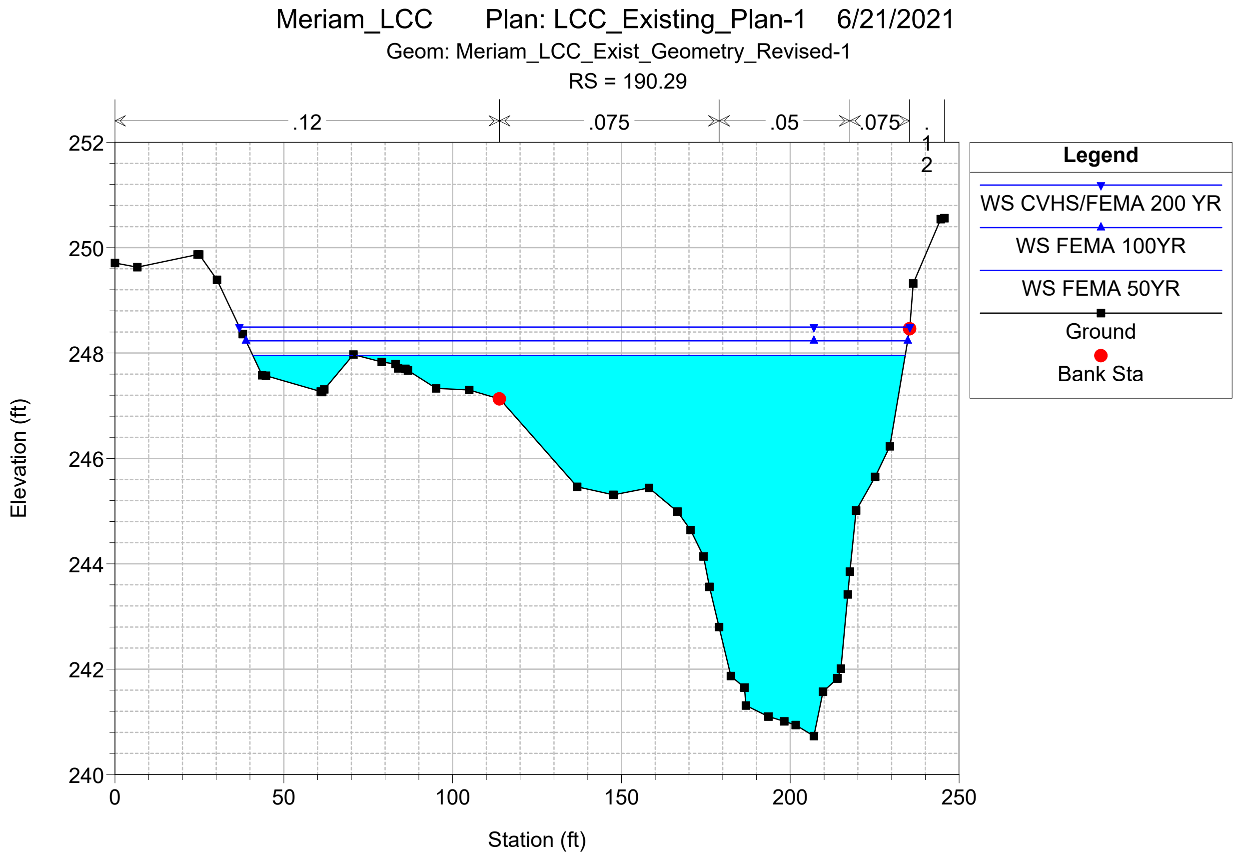

Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 343.42
.12
.075
.05
.075
.12
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)

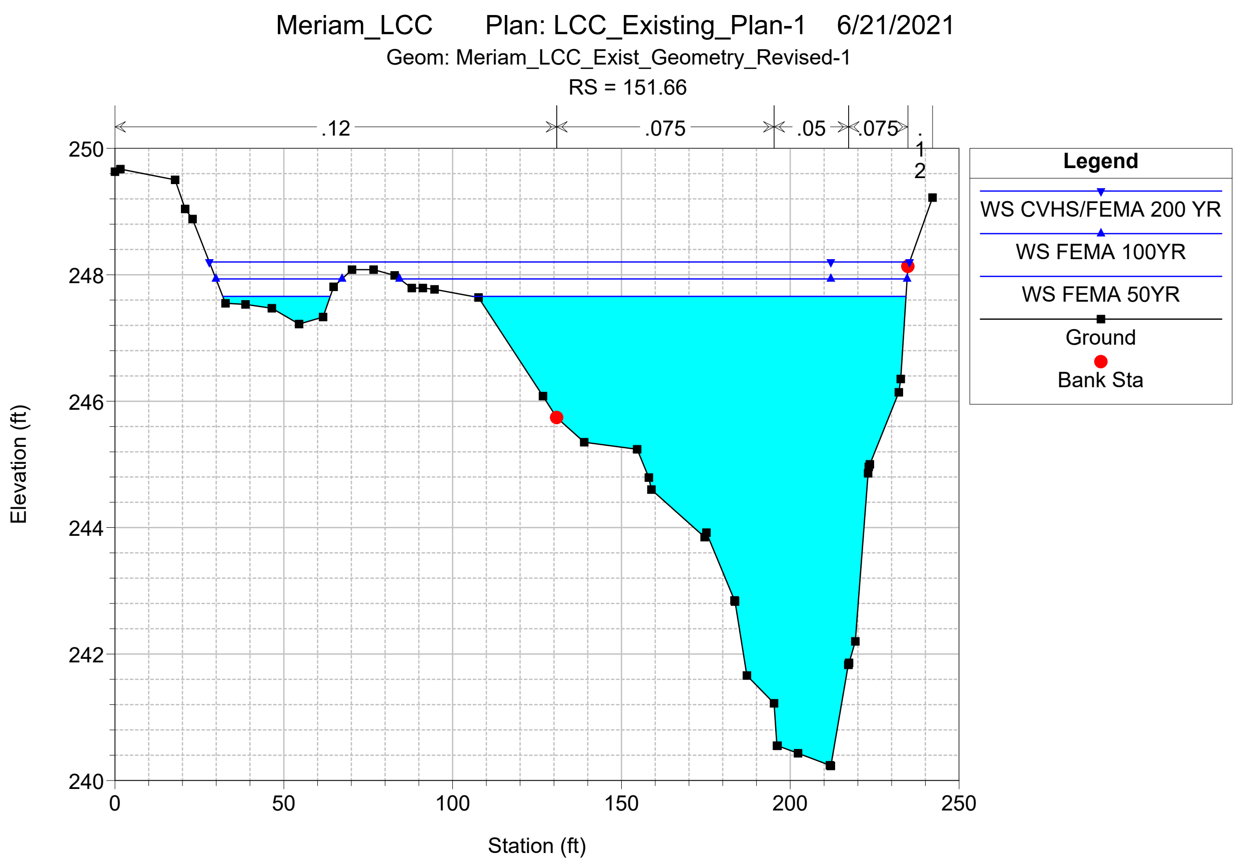

Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 326.31
.12
.075
.05
.075
.12
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Crit CVHS/FEMA 200 YR
Crit FEMA 100YR
Crit FEMA 50YR
Ground
Levee
Bank Sta
Elevation (ft)
240
242
244
246
248
250
252
0
50
100
150
200
250
300
Station (ft)

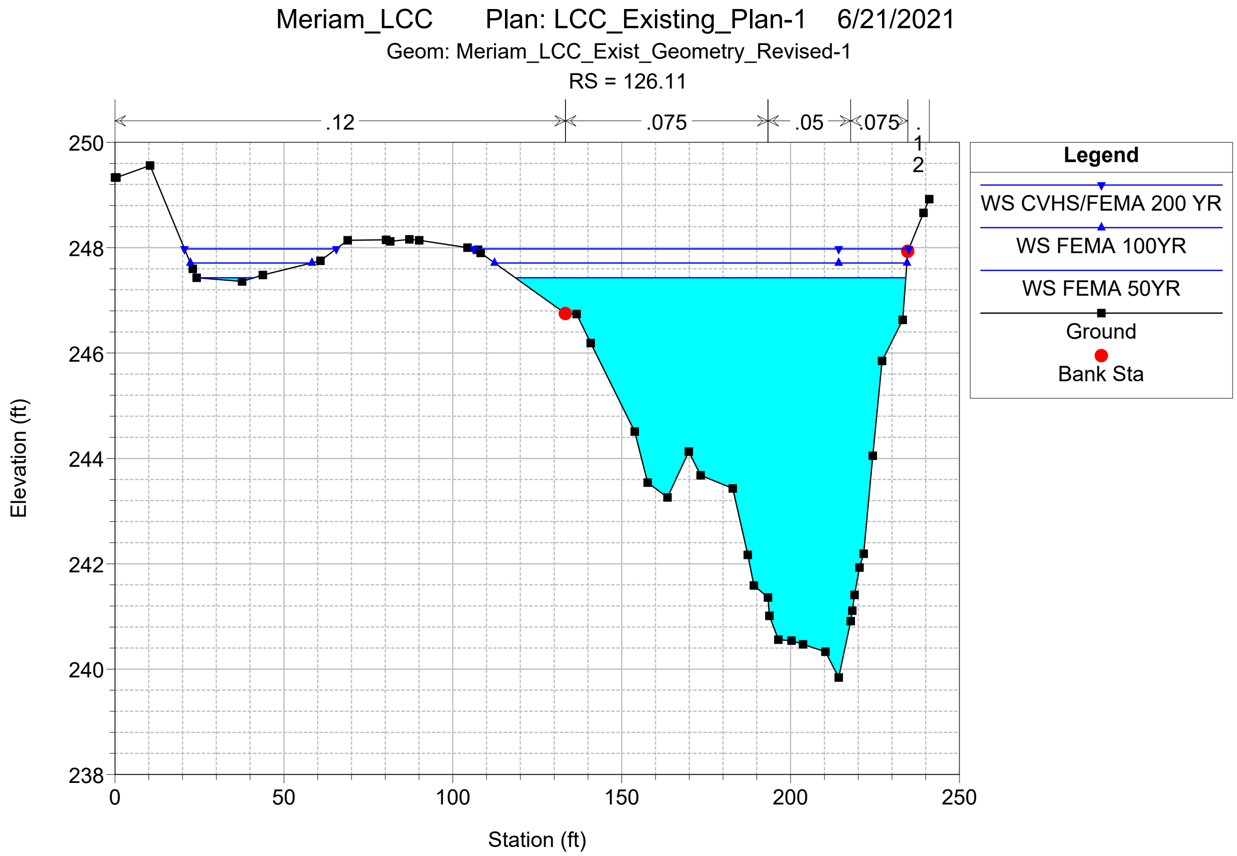

Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 286.88
.12
.075
.05
.075
.
0
1
2
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
240
242
244
246
248
250
0
50
100
150
200
250
Station (ft)


Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 266.95
.12
.075
.05
.075
.1
2
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)
240
242
244
246
248
250
0
50
100
150
200
250

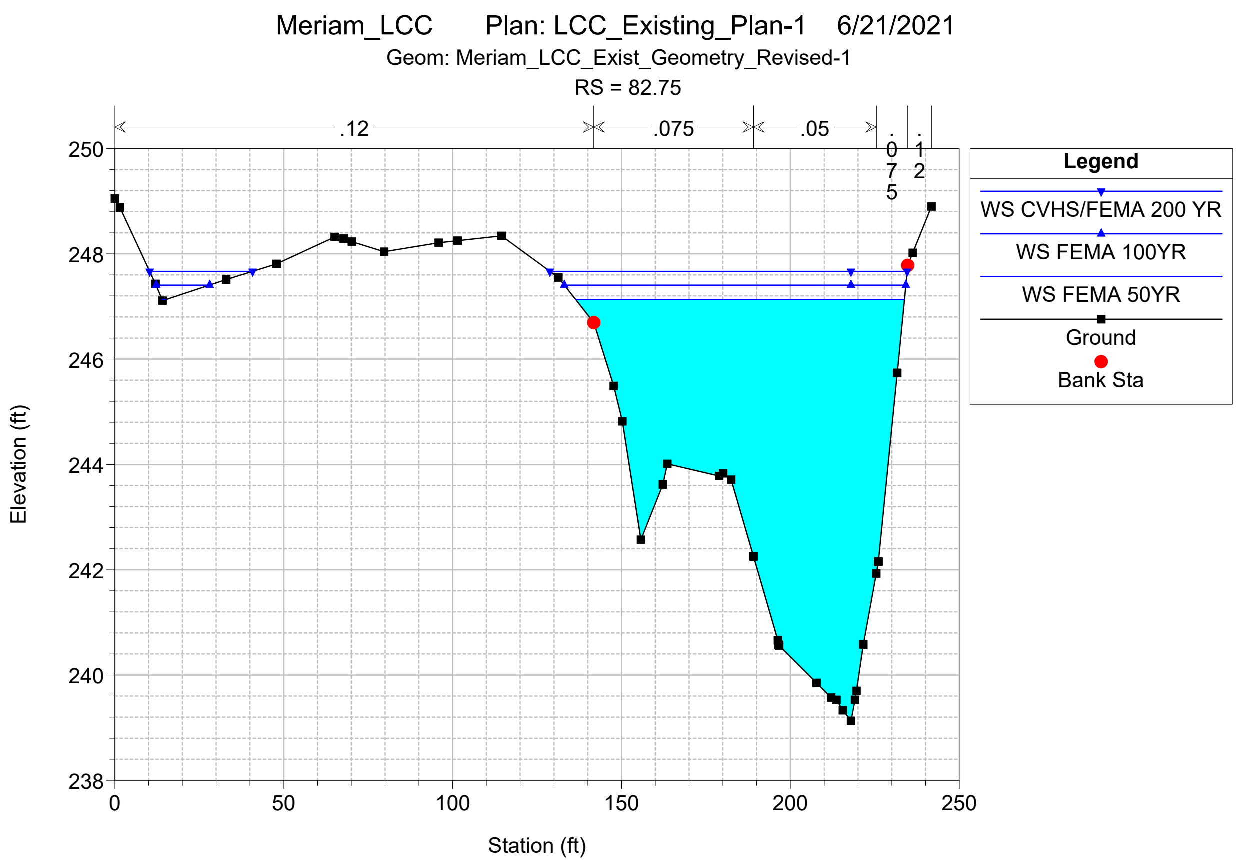

Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 229.39
.12
.075
.05
.075
.1
2
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)
240
242
244
246
248
250
252
0
50
100
150
200
250


Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 190.29
.12
.075
.05
.075
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)

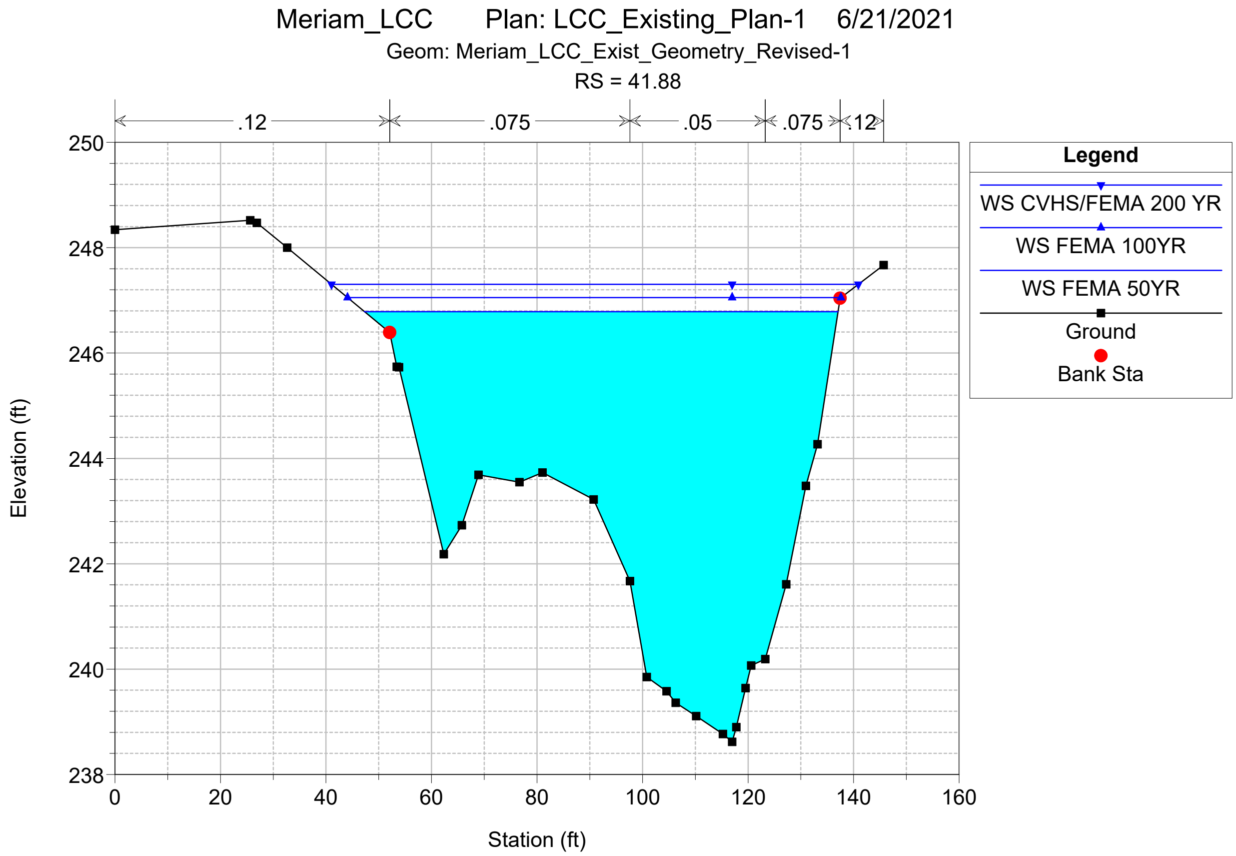

Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 151.66
.12
.075
.05
.075
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)

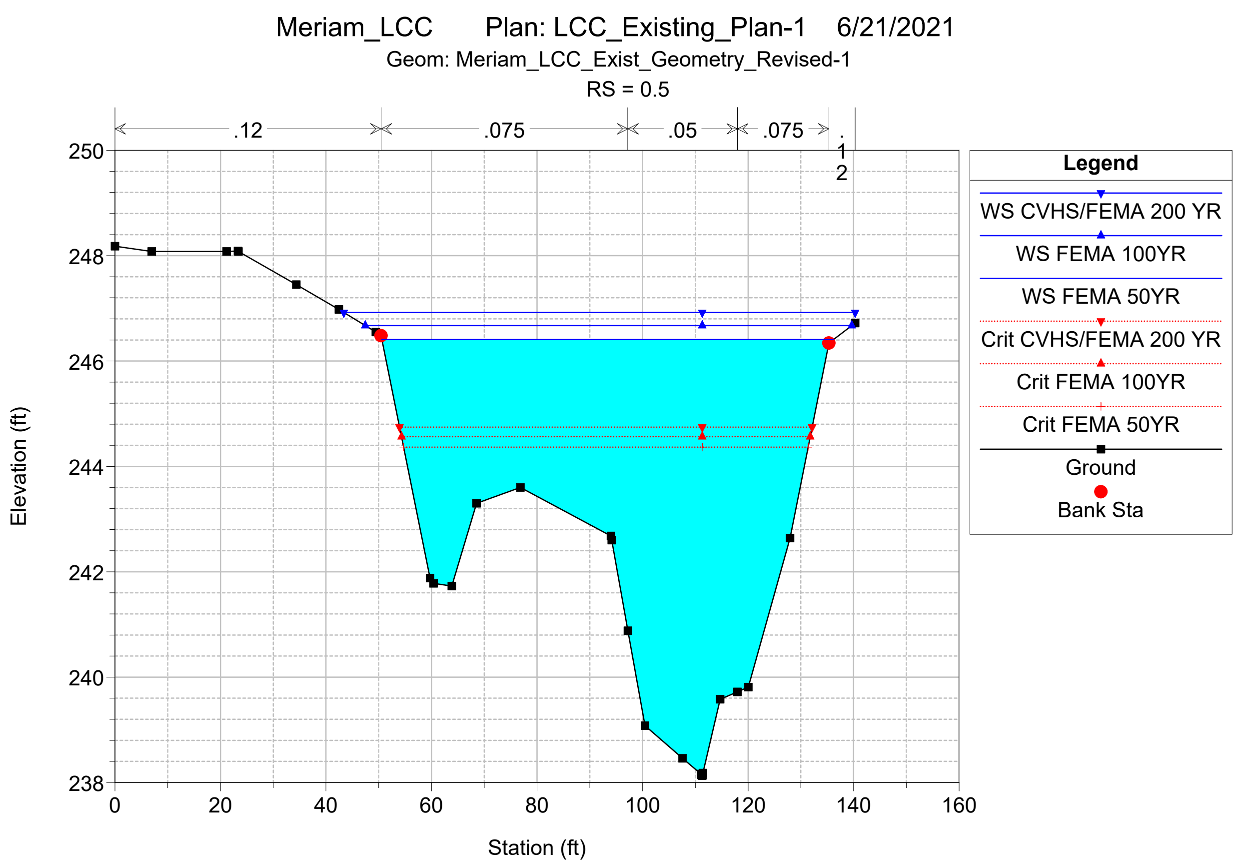


Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 126.11
.12
.075
.05
.075
.12
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)



Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 82.75
.12
.075
.05
.075
.12
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)


Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 41.88
.12
.075
.05
.075
.12
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)


Meriam_LCC Plan: LCC_Existing_Plan-1 6/21/2021
Geom: Meriam_LCC_Exist_Geometry_Revised-1
RS = 0.5
.12
.075
.05
.075
.12
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Crit CVHS/FEMA 200 YR
Crit FEMA 100YR
Crit FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)

APPENDIX F

HEC-RAS HYDRAULIC MODEL OUTPUT- PROPOSED CONDITIONS

- Design Conditions Model
  - HEC-RAS Channel Profile View
  - HEC-RAS Bridge Results Tables
  - HEC-RAS Results Table
  - HEC-RAS Cross-Sections

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HEC-RAS Plan: Meriam_LCC-Prop-1 River: LCC Reach: RAS

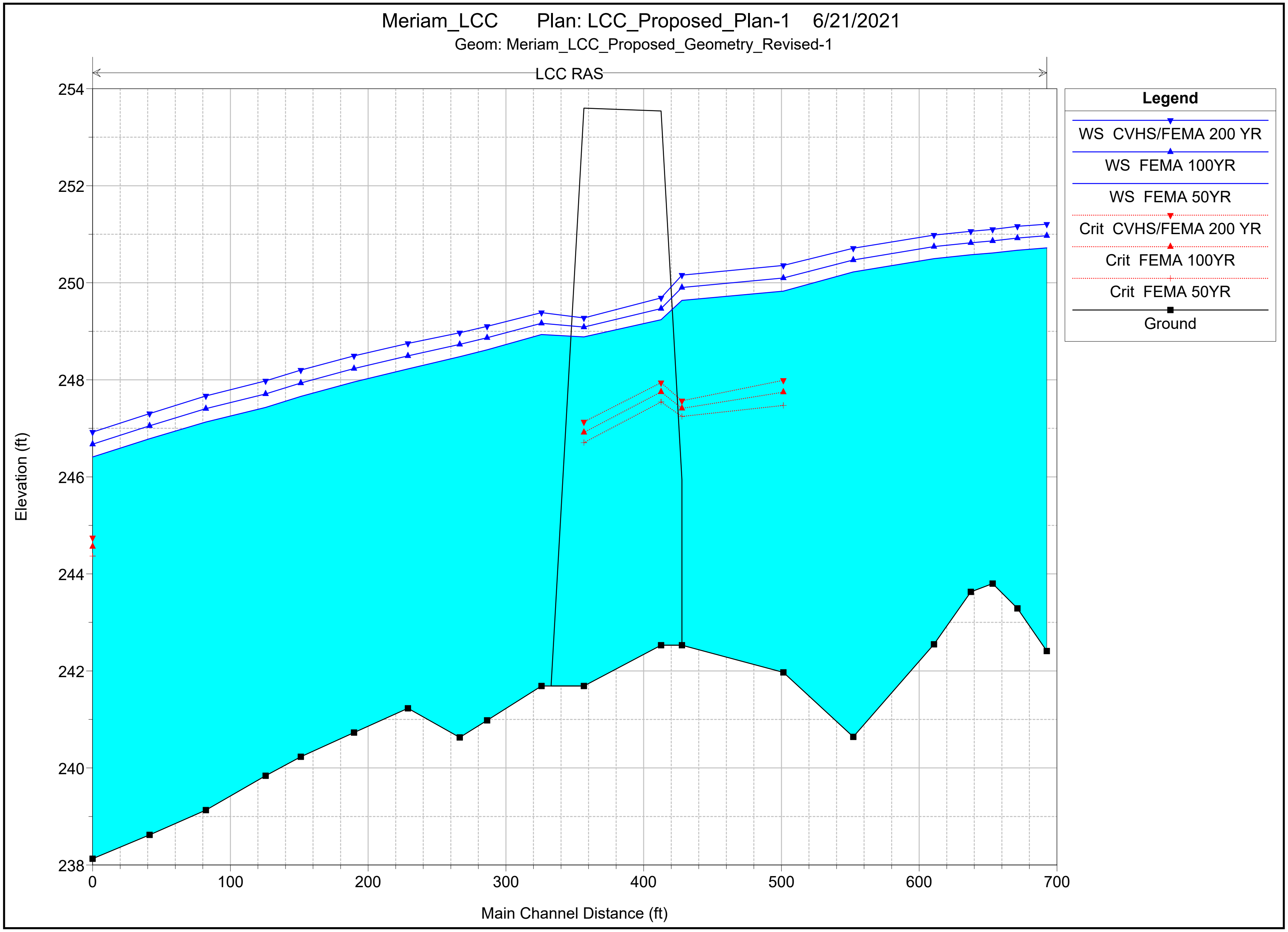
Reach	River Sta	Profile	Q Total	Q Left	Q Right	Q Channel	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Vel Left	Vel Right	Flow Area	Hydr Depth	Froude # Chl
			(cfs)	(cfs)	(cfs)	(cfs)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(ft/s)	(ft/s)	(sq ft)	(ft)	
RAS	693.07	FEMA 50YR	2000.00	14.18		1985.82	250.72		250.81	0.002021	2.48	0.44		834.20	3.44	0.22
RAS	693.07	FEMA 100YR	2200.00	22.57		2177.43	250.97		251.07	0.001999	2.56	0.51		895.35	3.63	0.22
RAS	693.07	CVHS/FEMA 200 YR	2400.00	32.23		2367.77	251.21		251.31	0.001981	2.63	0.57		954.67	3.81	0.22
RAS	671.79	FEMA 50YR	2000.00			2000.00	250.67		250.77	0.002659	2.46			811.90	3.46	0.23
RAS	671.79	FEMA 100YR	2200.00			2200.00	250.92		251.02	0.002572	2.53			871.05	3.70	0.23
RAS	671.79	CVHS/FEMA 200 YR	2400.00			2400.00	251.16		251.27	0.002506	2.59			927.69	3.92	0.23
RAS	653.79	FEMA 50YR	2000.00	25.13		1974.87	250.61		250.72	0.001989	2.65	0.55		789.68	3.73	0.22
RAS	653.79	FEMA 100YR	2200.00	35.81		2164.20	250.86		250.98	0.002003	2.76	0.63		842.63	3.94	0.22
RAS	653.79	CVHS/FEMA 200 YR	2400.00	47.55		2352.45	251.10		251.22	0.002017	2.85	0.69		893.64	4.13	0.23
RAS	637.96	FEMA 50YR	2000.00	13.47		1986.53	250.58		250.69	0.001810	2.73	0.43		758.64	3.66	0.23
RAS	637.96	FEMA 100YR	2200.00	21.03		2178.98	250.82		250.95	0.001829	2.84	0.48		811.56	3.63	0.23
RAS	637.96	CVHS/FEMA 200 YR	2400.00	31.56		2368.44	251.06		251.19	0.001845	2.94	0.52		866.62	3.54	0.23
RAS	611.2	FEMA 50YR	2000.00	42.97		1957.03	250.50		250.63	0.002796	2.97	0.58		732.86	3.02	0.25
RAS	611.2	FEMA 100YR	2200.00	63.95		2136.05	250.75		250.89	0.002766	3.06	0.66		793.93	3.20	0.25
RAS	611.2	CVHS/FEMA 200 YR	2400.00	90.45		2309.56	250.98		251.13	0.002732	3.14	0.75		855.04	3.23	0.26
RAS	552.6	FEMA 50YR	2000.00	59.74		1940.27	250.22		250.43	0.004013	3.69	0.72		608.79	2.70	0.31
RAS	552.6	FEMA 100YR	2200.00	86.67		2113.34	250.47		250.69	0.003977	3.80	0.78		666.36	2.77	0.31
RAS	552.6	CVHS/FEMA 200 YR	2400.00	127.15		2272.85	250.71		250.93	0.003894	3.88	0.90		726.77	2.80	0.31
RAS	501.91	FEMA 50YR	2000.00	136.09		1863.91	249.83	247.47	250.14	0.007652	4.66	1.28		505.92	2.51	0.45
RAS	501.91	FEMA 100YR	2200.00	176.15		2023.85	250.10	247.75	250.41	0.007014	4.68	1.37		561.00	2.74	0.43
RAS	501.91	CVHS/FEMA 200 YR	2400.00	217.37		2182.63	250.36	247.99	250.67	0.006522	4.71	1.44		614.36	2.96	0.42
RAS	428.29	FEMA 50YR	2000.00	147.50		1852.50	249.64	247.24	249.90	0.001667	3.88	6.28		500.92	4.09	0.33
RAS	428.29	FEMA 100YR	2200.00	180.71	0.00	2019.29	249.90	247.41	250.19	0.001634	3.99	6.56	0.05	533.80	4.27	0.33
RAS	428.29	CVHS/FEMA 200 YR	2400.00	215.97	0.05	2183.98	250.16	247.57	250.46	0.001601	4.09	6.81	0.14	565.85	4.43	0.33
RAS	377.85		Bridge													
RAS	326.31	FEMA 50YR	2000.00	490.72		1509.28	248.93		249.13	0.003079	3.82	2.57		586.02	2.72	0.31
RAS	326.31	FEMA 100YR	2200.00	582.19		1617.81	249.16		249.37	0.003083	3.91	2.61		637.23	2.79	0.31
RAS	326.31	CVHS/FEMA 200 YR	2400.00	679.39	0.00	1720.61	249.39		249.59	0.003069	3.97	2.66	0.03	688.56	2.93	0.31
RAS	286.88	FEMA 50YR	2000.00	169.04	0.41	1830.55	248.62		248.93	0.008453	4.64	2.23	2.85	470.62	2.44	0.42
RAS	286.88	FEMA 100YR	2200.00	245.74	2.35	1951.92	248.87		249.17	0.007757	4.64	2.51	4.29	519.55	2.59	0.41
RAS	286.88	CVHS/FEMA 200 YR	2400.00	327.35	6.24	2066.41	249.10		249.41	0.007182	4.63	2.72	5.32	567.43	2.74	0.40
RAS	266.95	FEMA 50YR	2000.00	168.41		1831.59	248.48		248.78	0.006625	4.60	1.65		500.68	2.75	0.40
RAS	266.95	FEMA 100YR	2200.00	240.39		1959.61	248.73		249.03	0.006338	4.64	1.92		548.21	2.87	0.39
RAS	266.95	CVHS/FEMA 200 YR	2400.00	318.64	0.01	2081.36	248.97		249.27	0.006035	4.67	2.14	0.16	595.22	3.00	0.38
RAS	229.39	FEMA 50YR	2000.00	132.55		1867.45	248.23		248.53	0.006755	4.55	1.43		503.31	2.76	0.41
RAS	229.39	FEMA 100YR	2200.00	194.32		2005.68	248.49		248.79	0.006304	4.57	1.71		552.79	2.98	0.40
RAS	229.39	CVHS/FEMA 200 YR	2400.00	262.73		2137.27	248.75		249.05	0.005921	4.58	1.96		600.69	3.18	0.39
RAS	190.29	FEMA 50YR	2000.00	19.17		1980.83	247.96		248.26	0.007109	4.42	0.60		479.80	2.50	0.40
RAS	190.29	FEMA 100YR	2200.00	41.56		2158.44	248.23		248.54	0.006710	4.48	0.80		533.70	2.72	0.40

HEC-RAS Plan: Meriam_LCC-Prop-1   River: LCC   Reach: RAS (Continued)

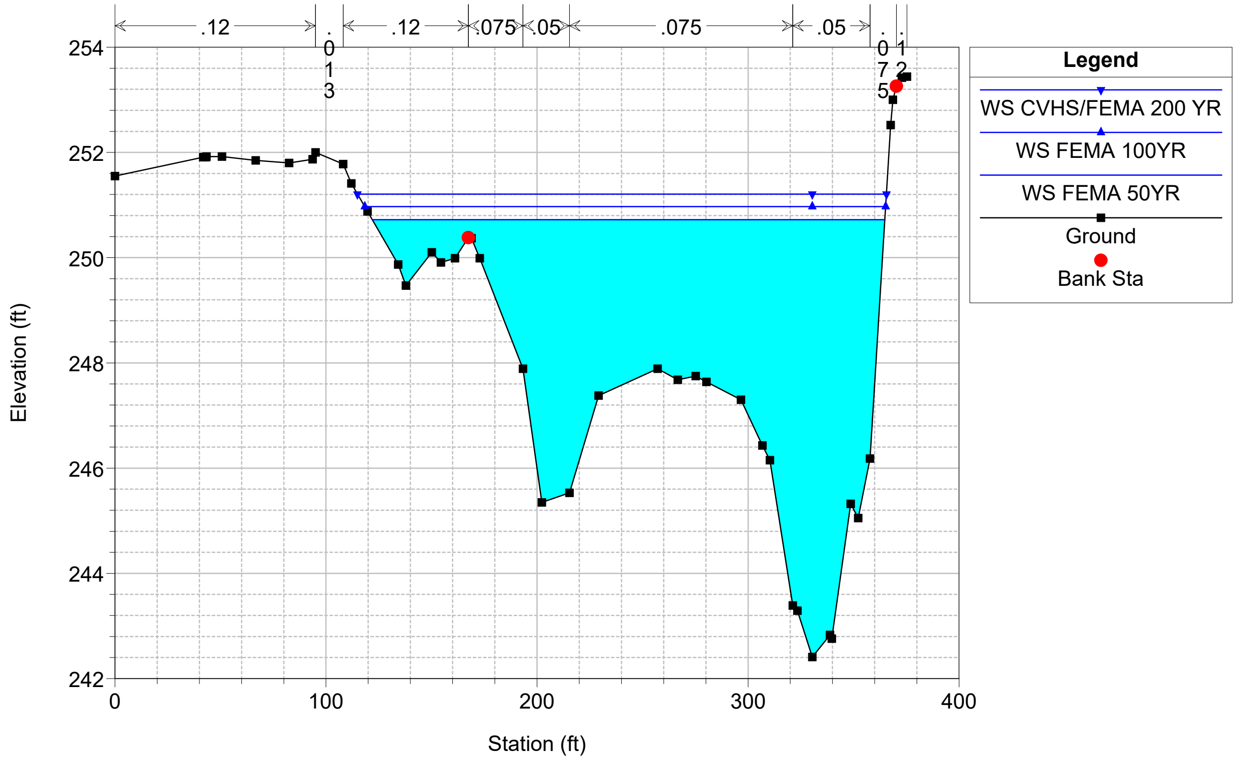
Reach	River Sta	Profile	Q Total	Q Left	Q Right	Q Channel	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Vel Left	Vel Right	Flow Area	Hydr Depth	Froude # Chl
			(cfs)	(cfs)	(cfs)	(cfs)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(ft/s)	(ft/s)	(sq ft)	(ft)	
RAS	190.29	CVHS/FEMA 200 YR	2400.00	68.20	0.00	2331.81	248.49		248.81	0.006385	4.54	0.95	0.05	585.22	2.95	0.39
RAS	151.66	FEMA 50YR	2000.00	24.78		1975.22	247.66		247.98	0.007143	4.58	0.83		461.41	2.88	0.40
RAS	151.66	FEMA 100YR	2200.00	37.46		2162.54	247.93		248.27	0.006946	4.70	0.75		510.33	2.72	0.39
RAS	151.66	CVHS/FEMA 200 YR	2400.00	62.11	0.00	2337.89	248.20		248.55	0.006709	4.79	0.82	0.11	563.42	2.72	0.39
RAS	126.11	FEMA 50YR	2000.00	2.76		1997.24	247.43		247.79	0.007757	4.78	0.49		423.47	3.19	0.41
RAS	126.11	FEMA 100YR	2200.00	9.77		2190.23	247.71		248.08	0.007549	4.91	0.54		464.07	2.94	0.41
RAS	126.11	CVHS/FEMA 200 YR	2400.00	23.76	0.00	2376.24	247.98		248.37	0.007337	5.02	0.67	0.09	508.53	2.92	0.41
RAS	82.75	FEMA 50YR	2000.00	0.42		1999.58	247.13		247.49	0.006201	4.77	0.35		420.03	4.27	0.39
RAS	82.75	FEMA 100YR	2200.00	2.18		2197.82	247.41		247.79	0.006214	4.95	0.40		449.62	3.84	0.40
RAS	82.75	CVHS/FEMA 200 YR	2400.00	6.85		2393.15	247.67		248.07	0.006226	5.11	0.48		482.55	3.54	0.40
RAS	41.88	FEMA 50YR	2000.00	0.35		1999.65	246.79		247.20	0.007728	5.15	0.37		389.60	4.34	0.42
RAS	41.88	FEMA 100YR	2200.00	1.38	0.00	2198.62	247.05		247.50	0.007796	5.35	0.52	0.04	413.99	4.43	0.43
RAS	41.88	CVHS/FEMA 200 YR	2400.00	3.27	0.13	2396.60	247.30		247.78	0.007816	5.54	0.65	0.28	438.35	4.39	0.43
RAS	0.5	FEMA 50YR	2000.00		0.00	2000.00	246.41	244.37	246.85	0.009004	5.32		0.13	375.84	4.39	0.45
RAS	0.5	FEMA 100YR	2200.00	0.07	0.26	2199.68	246.67	244.56	247.15	0.009014	5.53	0.24	0.35	399.07	4.33	0.45
RAS	0.5	CVHS/FEMA 200 YR	2400.00	0.65	1.20	2398.15	246.92	244.75	247.43	0.009012	5.72	0.42	0.61	422.77	4.36	0.45

HEC-RAS Plan: Meriam_LCC-Prop-1 River: LCC Reach: RAS

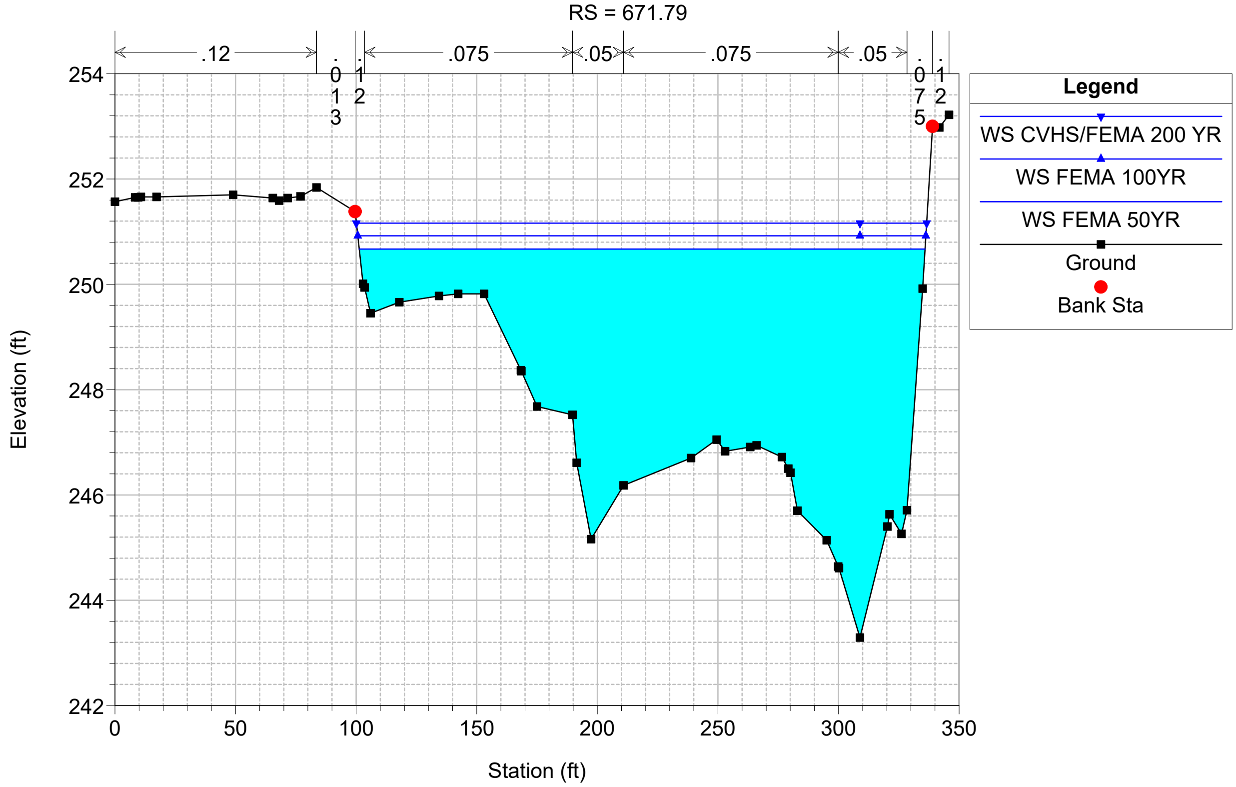
Reach	River Sta	Profile	E.G. Elev	W.S. Elev	Crit W.S.	Frctn Loss	C & E Loss	Top Width	Q Left	Q Channel	Q Right	Vel Chnl	Vel Total	Vel Left	Vel Right	Hydr Depth	Froude # Chl
			(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(cfs)	(cfs)	(cfs)	(ft/s)	(ft/s)	(ft/s)	(ft/s)	(ft)	
RAS	501.91	FEMA 50YR	250.14	249.83	247.47	0.23	0.02	201.59	136.09	1863.91		4.66	3.95	1.28		2.51	0.45
RAS	501.91	FEMA 100YR	250.41	250.10	247.75	0.22	0.01	204.72	176.15	2023.85		4.68	3.92	1.37		2.74	0.43
RAS	501.91	CVHS/FEMA 200 YR	250.67	250.36	247.99	0.21	0.00	207.70	217.37	2182.63		4.71	3.91	1.44		2.96	0.42
RAS	428.29	FEMA 50YR	249.90	249.64	247.24	0.05	0.03	122.52	147.50	1852.50		3.88	3.99	6.28		4.09	0.33
RAS	428.29	FEMA 100YR	250.19	249.90	247.41	0.05	0.03	124.87	180.71	2019.29	0.00	3.99	4.12	6.56	0.05	4.27	0.33
RAS	428.29	CVHS/FEMA 200 YR	250.46	250.16	247.57	0.05	0.04	127.84	215.97	2183.98	0.05	4.09	4.24	6.81	0.14	4.43	0.33
RAS	377.85 BR U	FEMA 50YR	249.82	249.24	247.54	0.44	0.04	75.85		2000.00		6.10	6.10			4.33	0.52
RAS	377.85 BR U	FEMA 100YR	250.10	249.47	247.75	0.47	0.04	76.65		2200.00		6.36	6.36			4.51	0.53
RAS	377.85 BR U	CVHS/FEMA 200 YR	250.37	249.69	247.94	0.50	0.04	77.38		2400.00		6.62	6.62			4.69	0.54
RAS	377.85 BR D	FEMA 50YR	249.33	248.88	246.71	0.13	0.08	79.11	22.04	1977.97		5.42	5.32	2.01		4.75	0.35
RAS	377.85 BR D	FEMA 100YR	249.59	249.09	246.92	0.13	0.09	79.11	26.04	2173.96		5.72	5.61	2.17		4.96	0.37
RAS	377.85 BR D	CVHS/FEMA 200 YR	249.83	249.28	247.13	0.13	0.10	79.11	30.12	2369.88		6.01	5.89	2.32		5.15	0.38
RAS	326.31	FEMA 50YR	249.13	248.93		0.19	0.01	215.38	490.72	1509.28		3.82	3.41	2.57		2.72	0.31
RAS	326.31	FEMA 100YR	249.37	249.16		0.18	0.01	228.17	582.19	1617.81		3.91	3.45	2.61		2.79	0.31
RAS	326.31	CVHS/FEMA 200 YR	249.59	249.39		0.18	0.01	235.33	679.39	1720.61	0.00	3.97	3.49	2.66	0.03	2.93	0.31
RAS	286.88	FEMA 50YR	248.93	248.62		0.15	0.00	192.49	169.04	1830.55	0.41	4.64	4.25	2.23	2.85	2.44	0.42
RAS	286.88	FEMA 100YR	249.17	248.87		0.14	0.00	200.60	245.74	1951.92	2.35	4.64	4.23	2.51	4.29	2.59	0.41
RAS	286.88	CVHS/FEMA 200 YR	249.41	249.10		0.13	0.00	206.87	327.35	2066.41	6.24	4.63	4.23	2.72	5.32	2.74	0.40

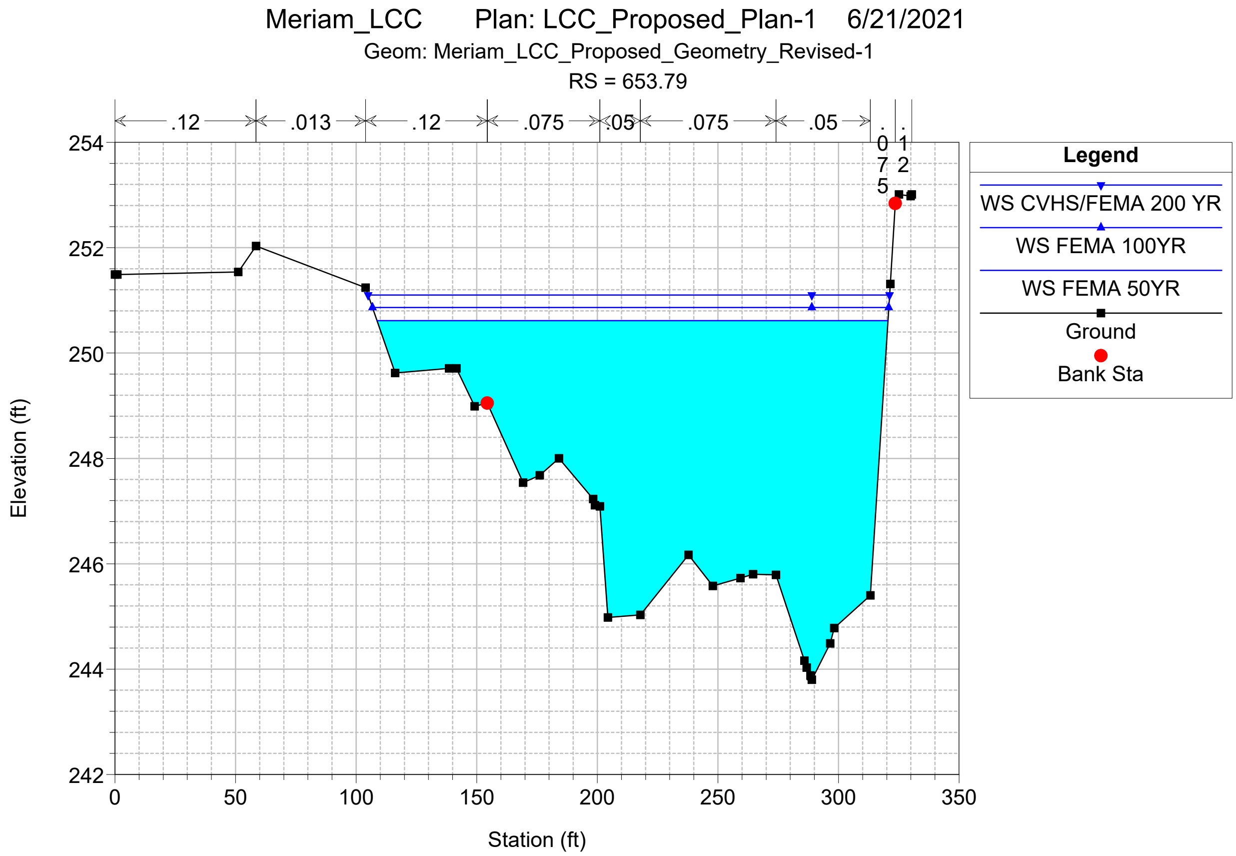

Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
LCC RAS
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Crit CVHS/FEMA 200 YR
Crit FEMA 100YR
Crit FEMA 50YR
Ground
Elevation (ft)
Main Channel Distance (ft)
254
252
250
248
246
244
242
240
238
0
100
200
300
400
500
600
700

Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 693.07

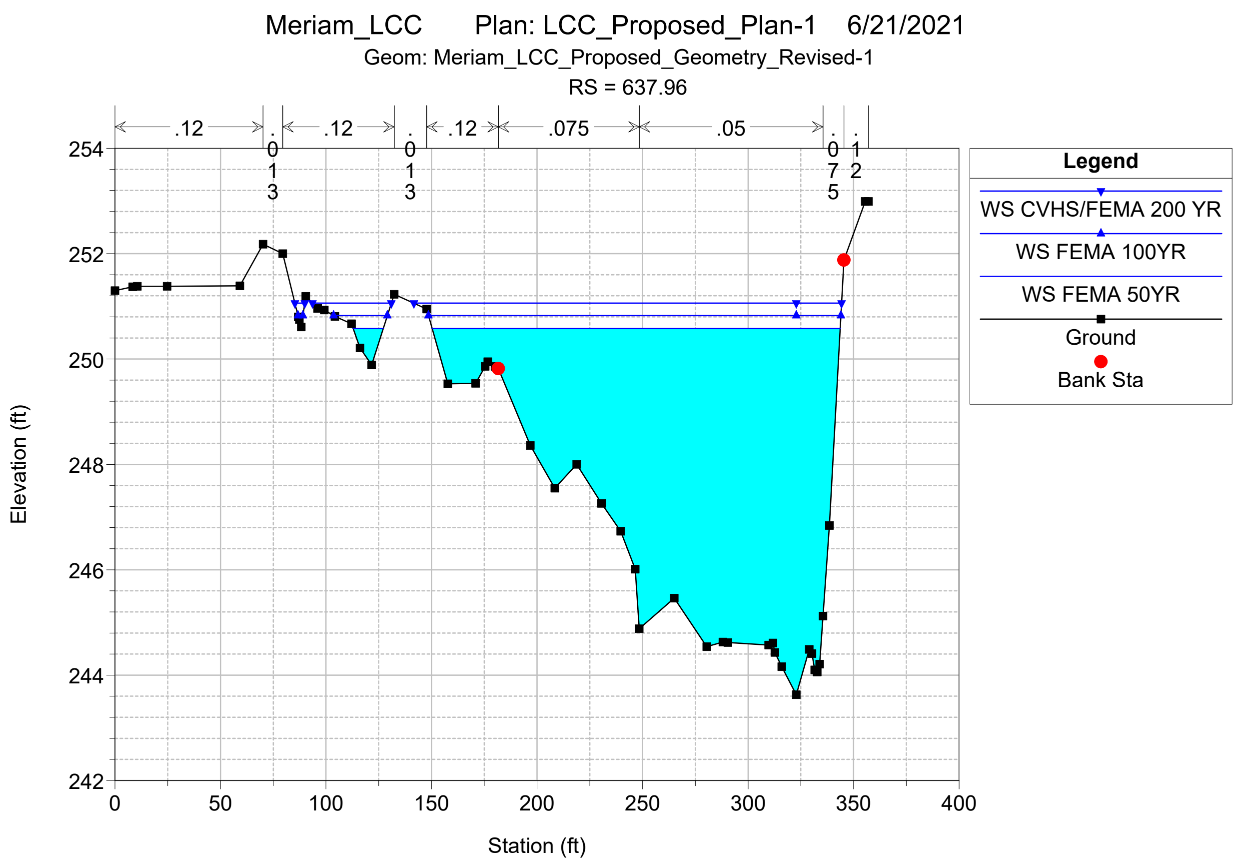


Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 671.79

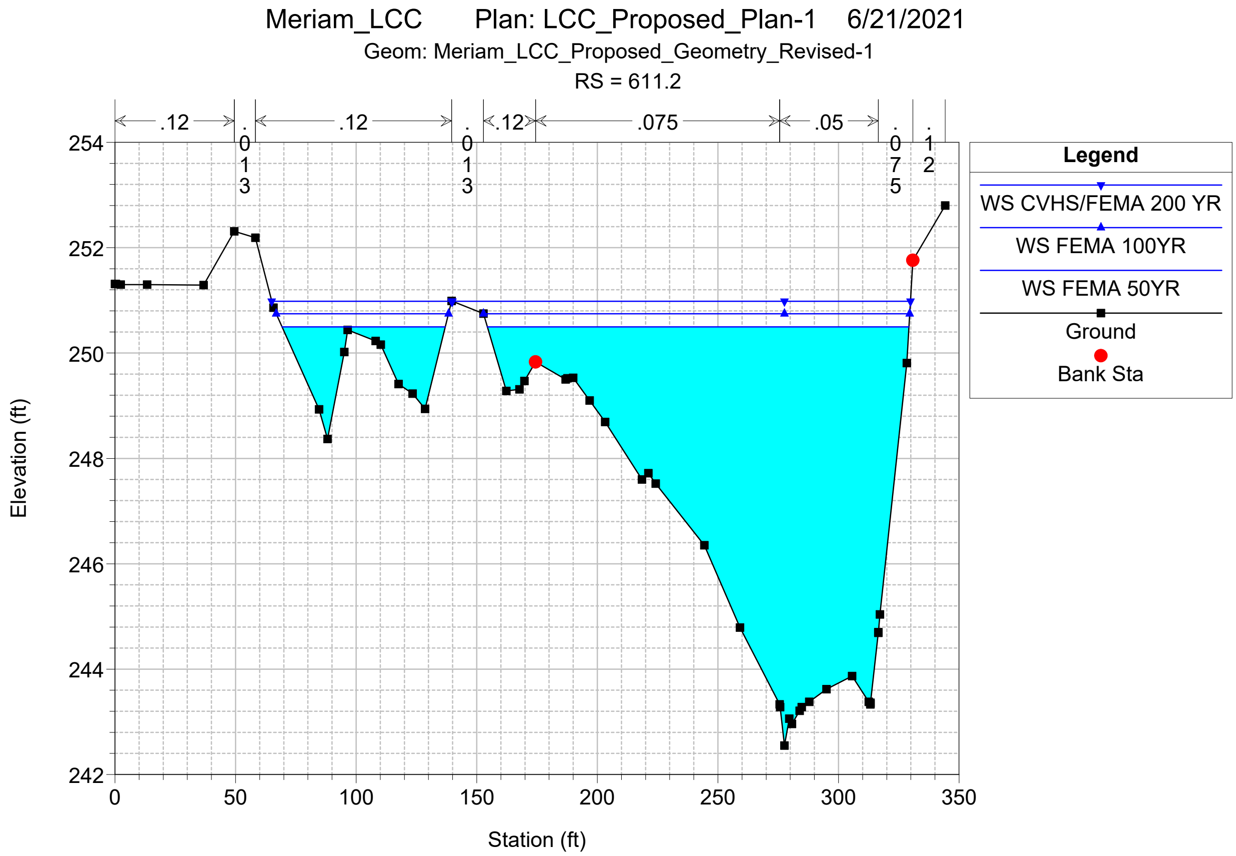


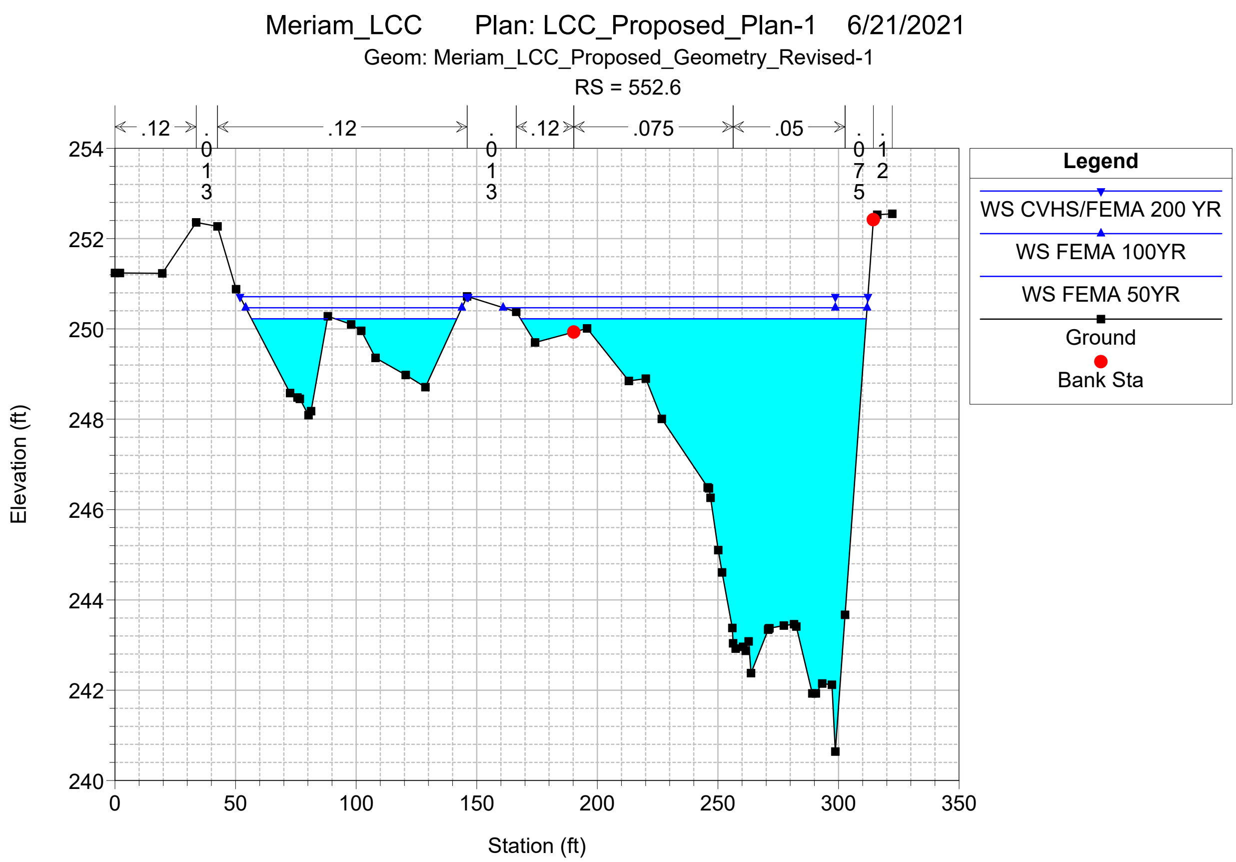


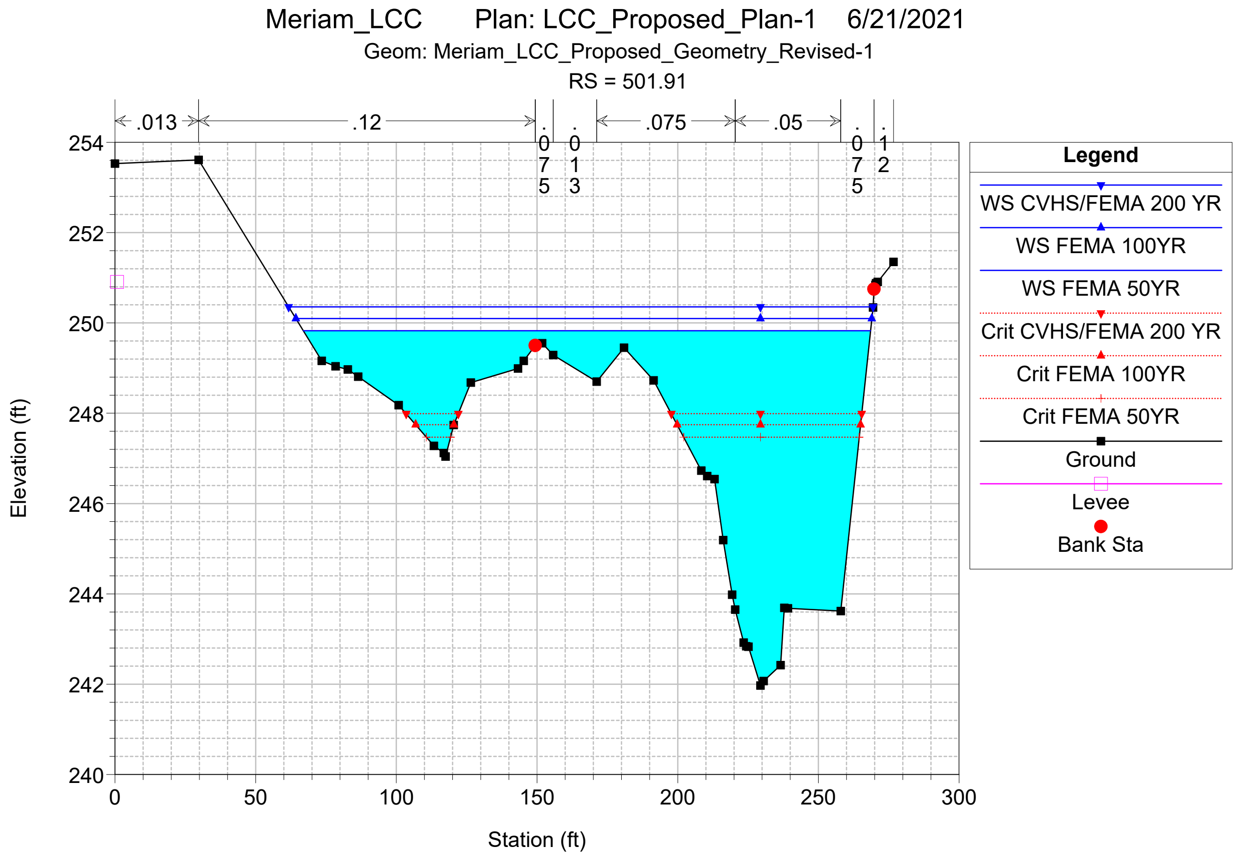
Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 653.79
.12
.013
.12
.075
.05
.075
.05
.075
.12
Elevation (ft)
Station (ft)
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta

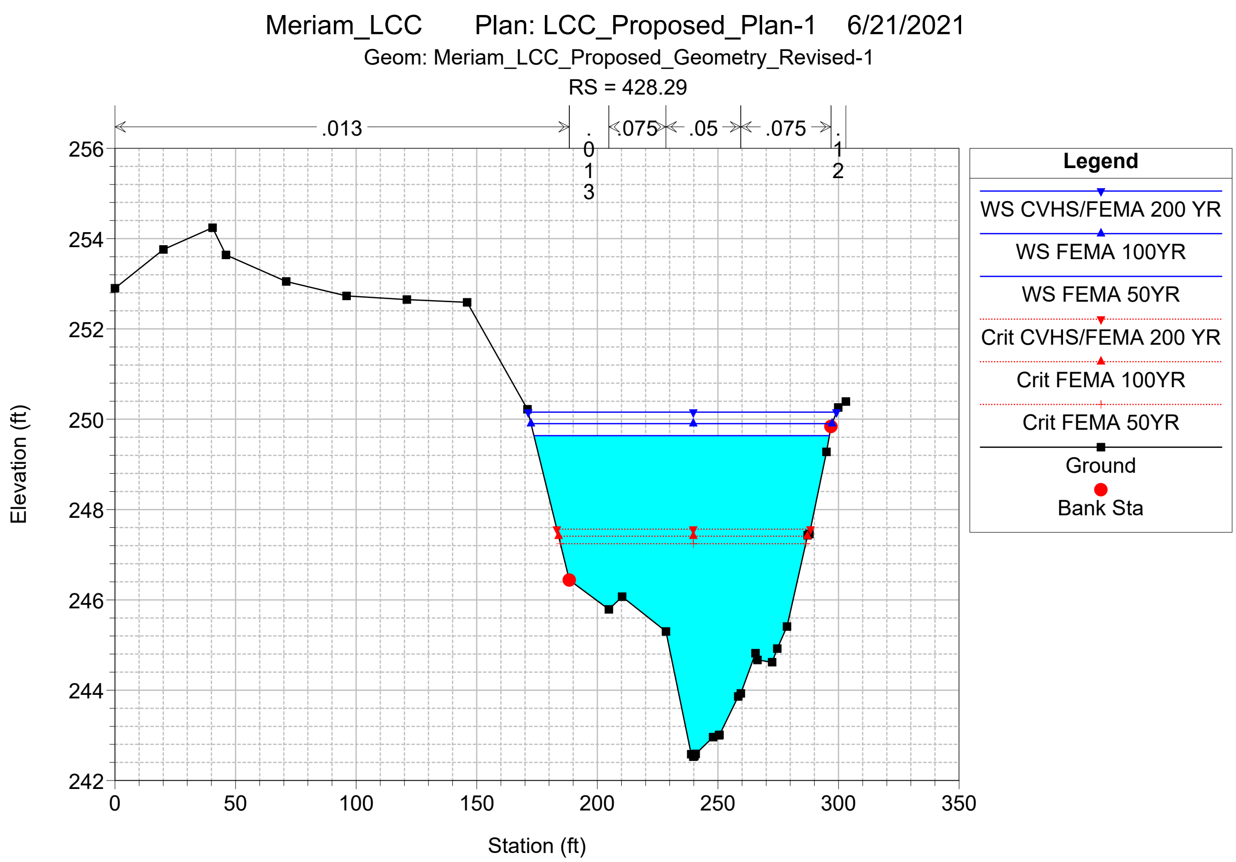


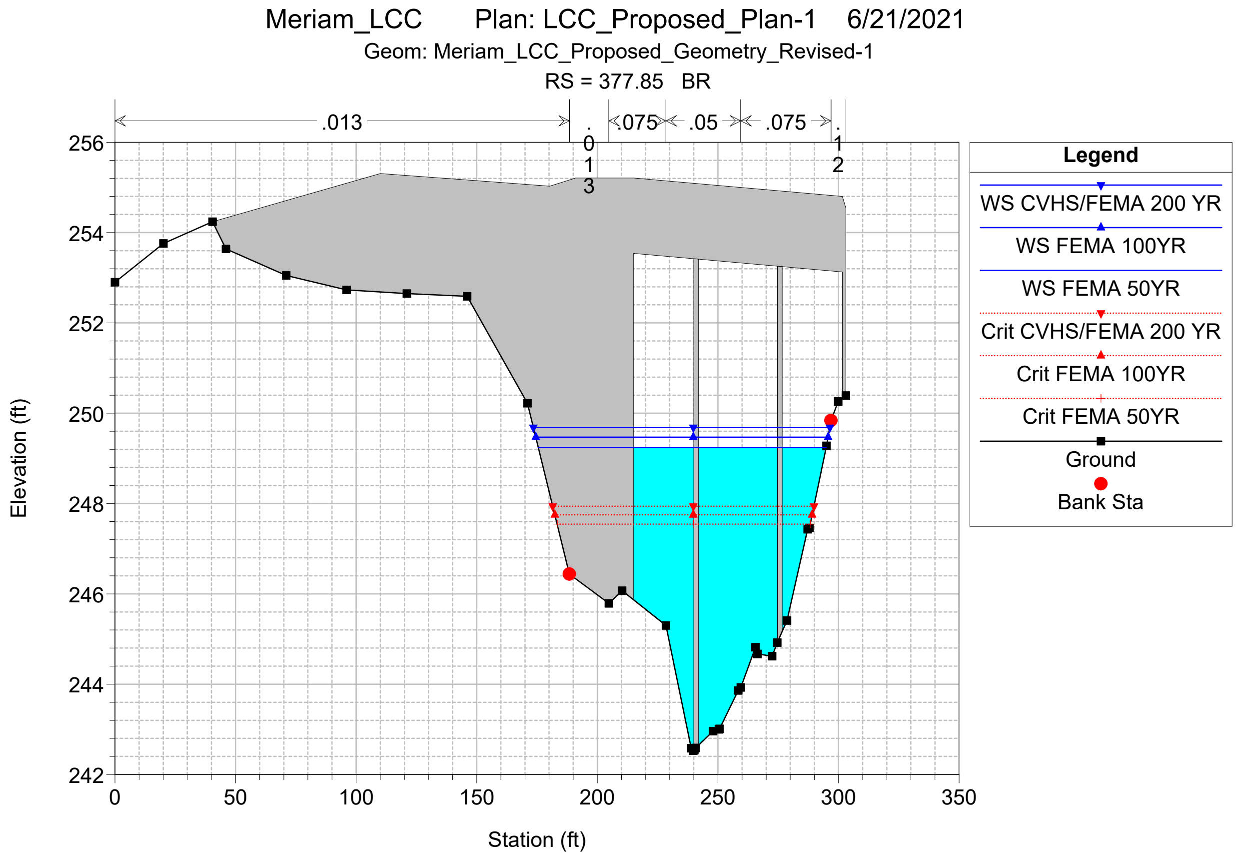
Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 637.96
.12
.013
.12
.013
.12
.075
.05
.075
.12
Elevation (ft)
Station (ft)
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta


Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 611.2
.12
.013
.12
.013
.12
.075
.05
.075
.12
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)
254
252
250
248
246
244
242
0
50
100
150
200
250
300
350

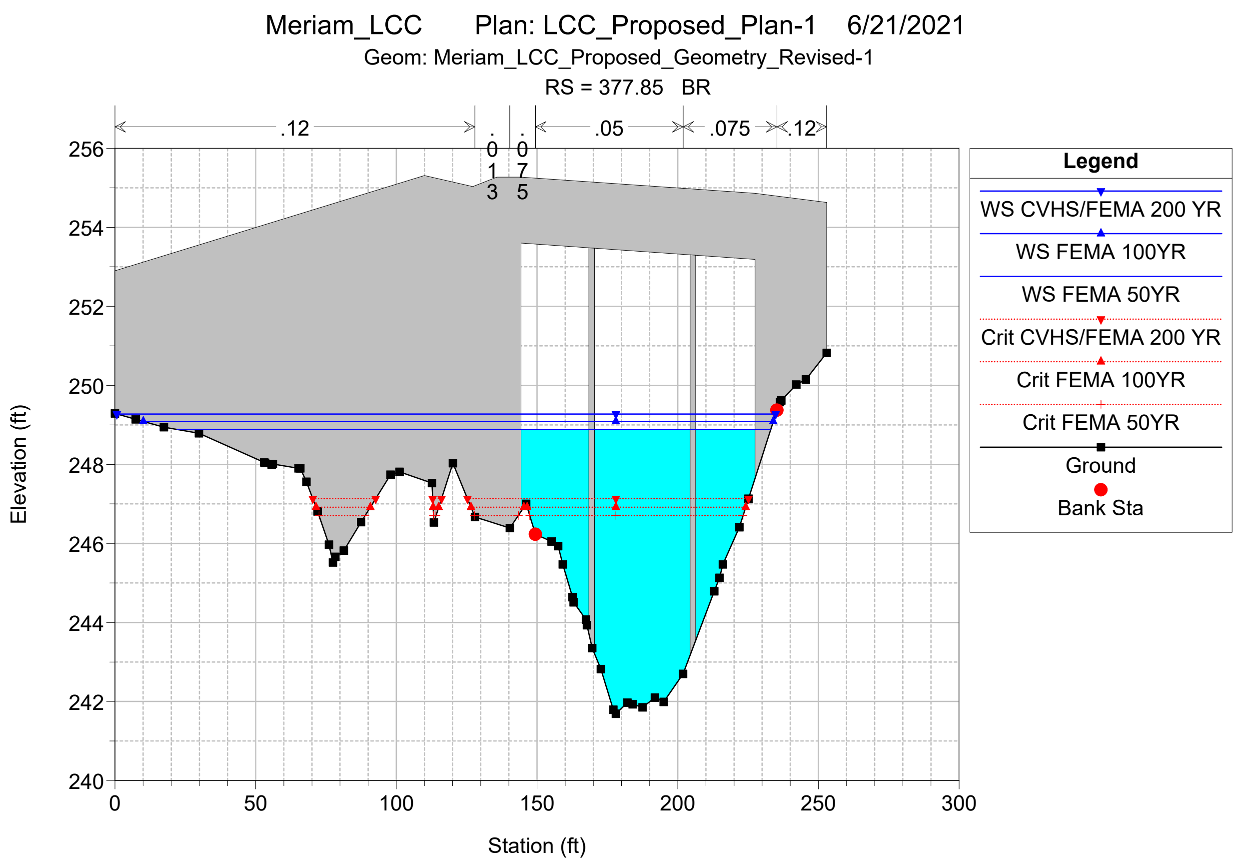

Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 552.6
.12
.013
.12
.013
.12
.075
.05
.075
.12
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)
254
252
250
248
246
244
242
240
0
50
100
150
200
250
300
350


Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 501.91
.013
.12
.075
.013
.075
.05
.075
.12
Elevation (ft)
Station (ft)
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Crit CVHS/FEMA 200 YR
Crit FEMA 100YR
Crit FEMA 50YR
Ground
Levee
Bank Sta

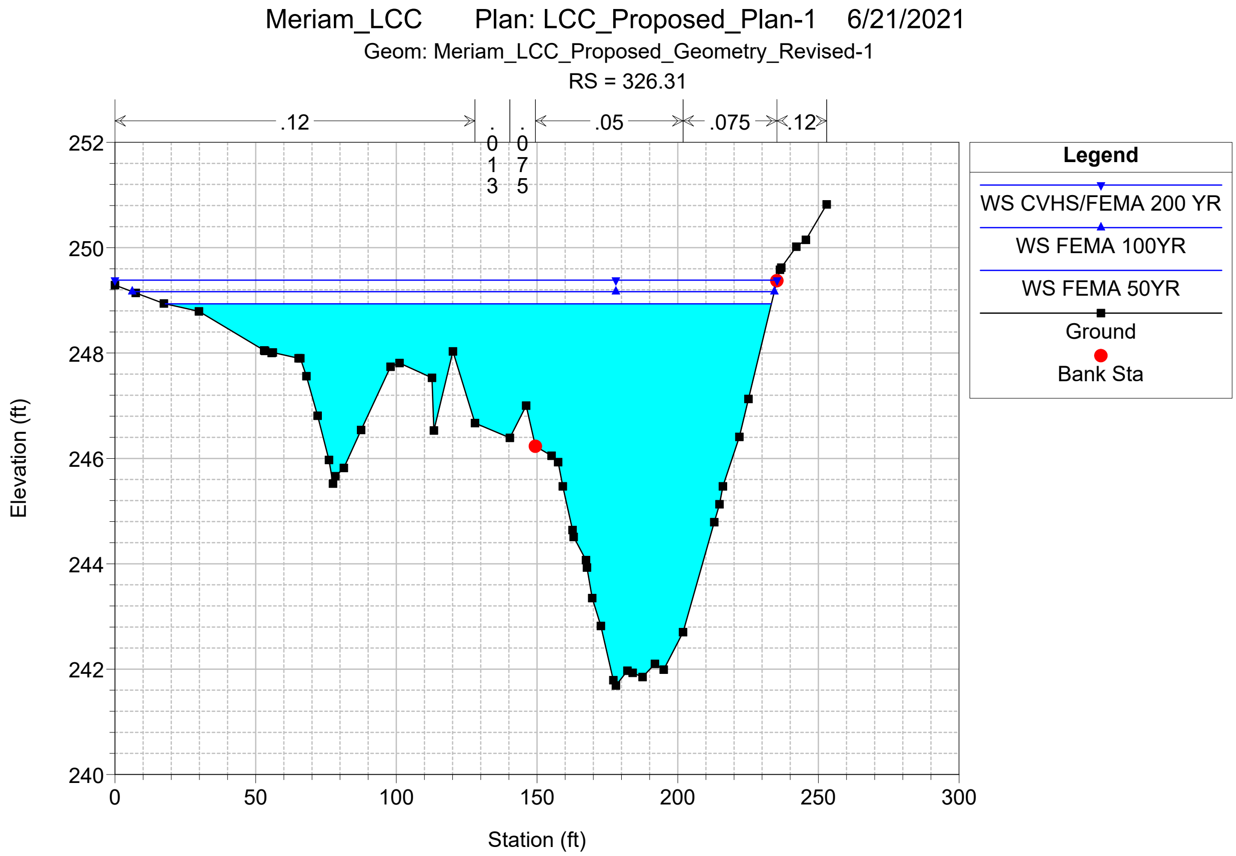

Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 428.29
.013
.013
.075
.05
.075
.12
Elevation (ft)
Station (ft)
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Crit CVHS/FEMA 200 YR
Crit FEMA 100YR
Crit FEMA 50YR
Ground
Bank Sta

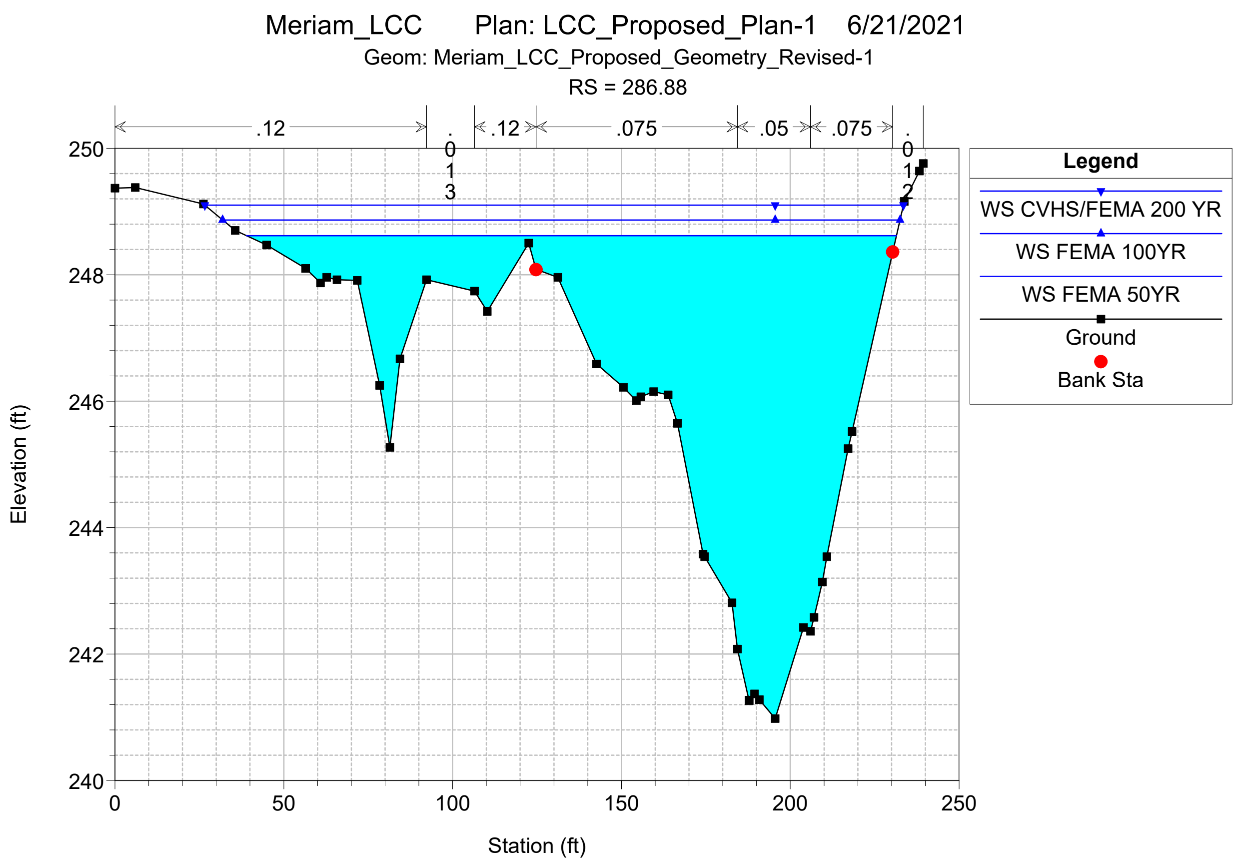


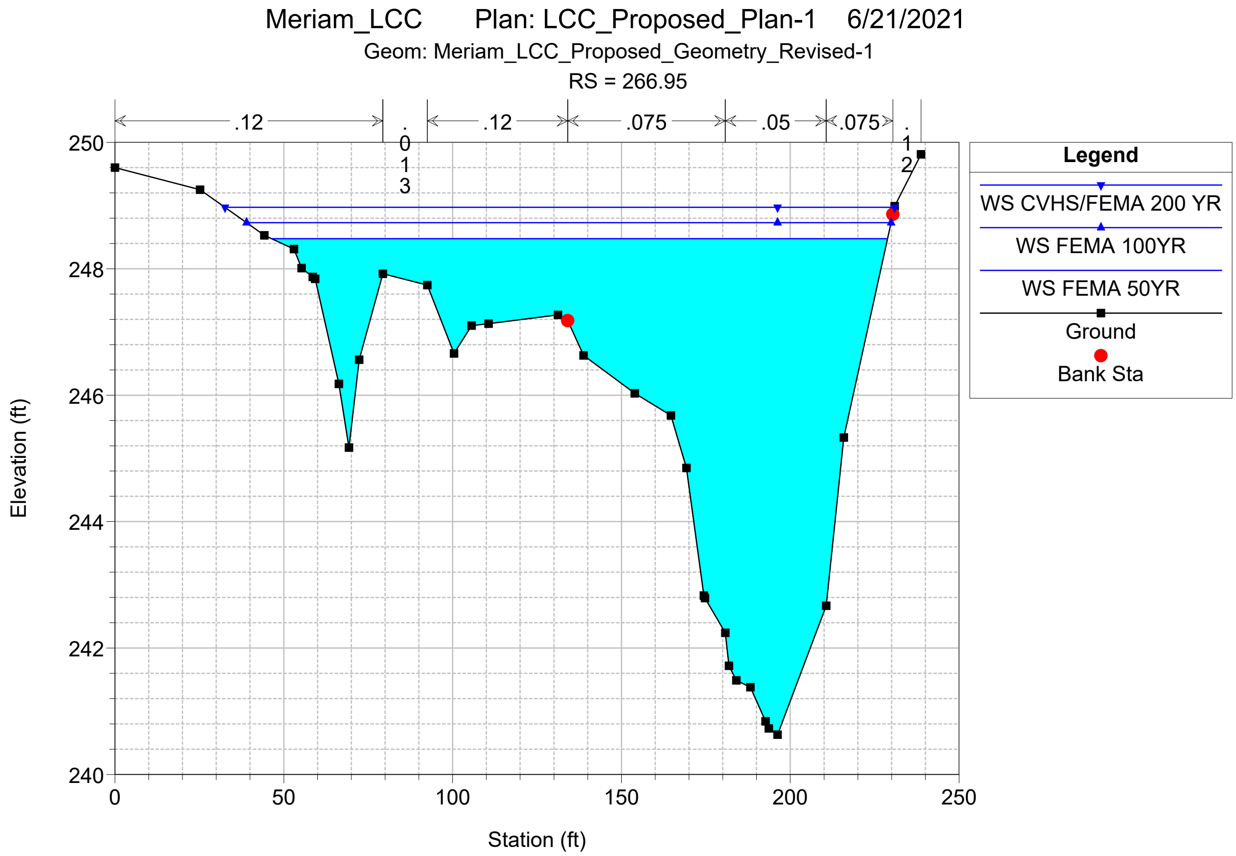
Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 377.85 BR
.013
.013
.075
.05
.075
.12
Elevation (ft)
Station (ft)
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Crit CVHS/FEMA 200 YR
Crit FEMA 100YR
Crit FEMA 50YR
Ground
Bank Sta

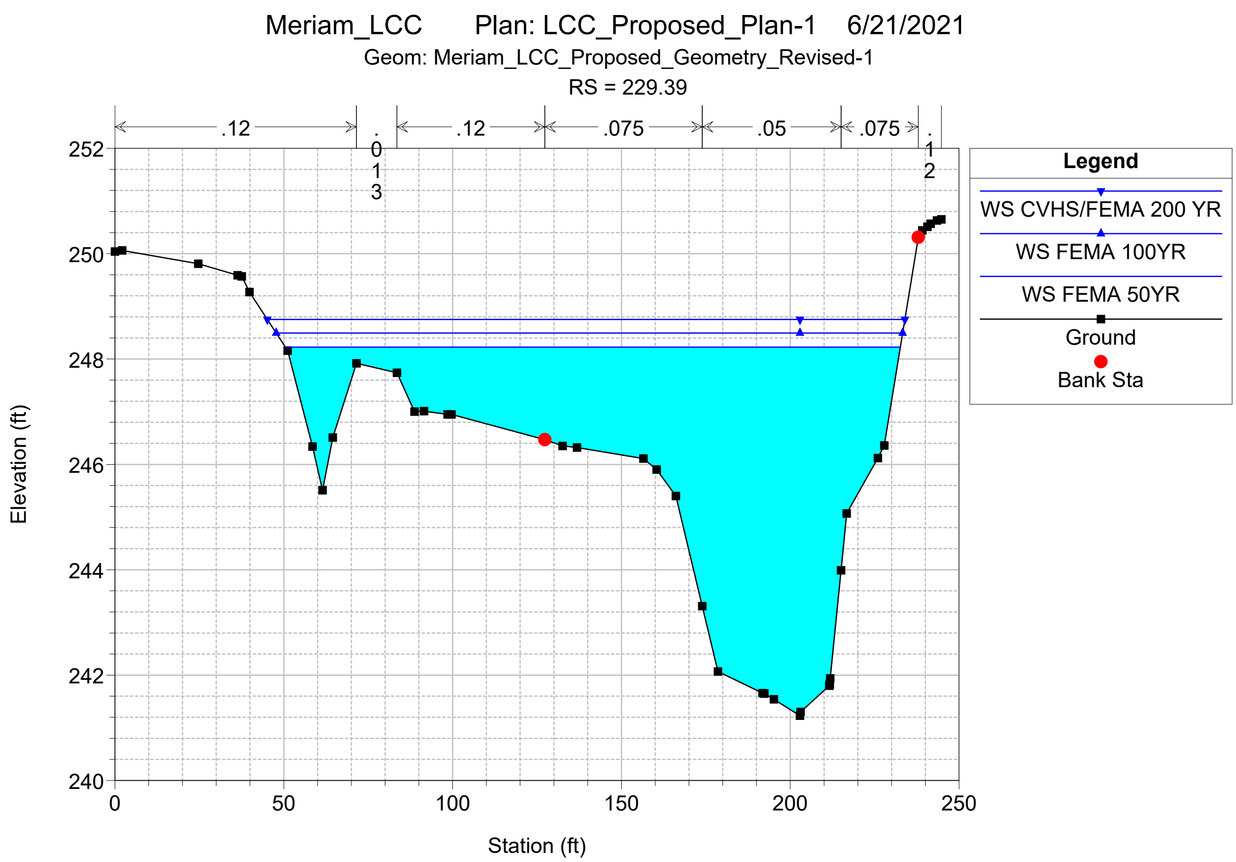


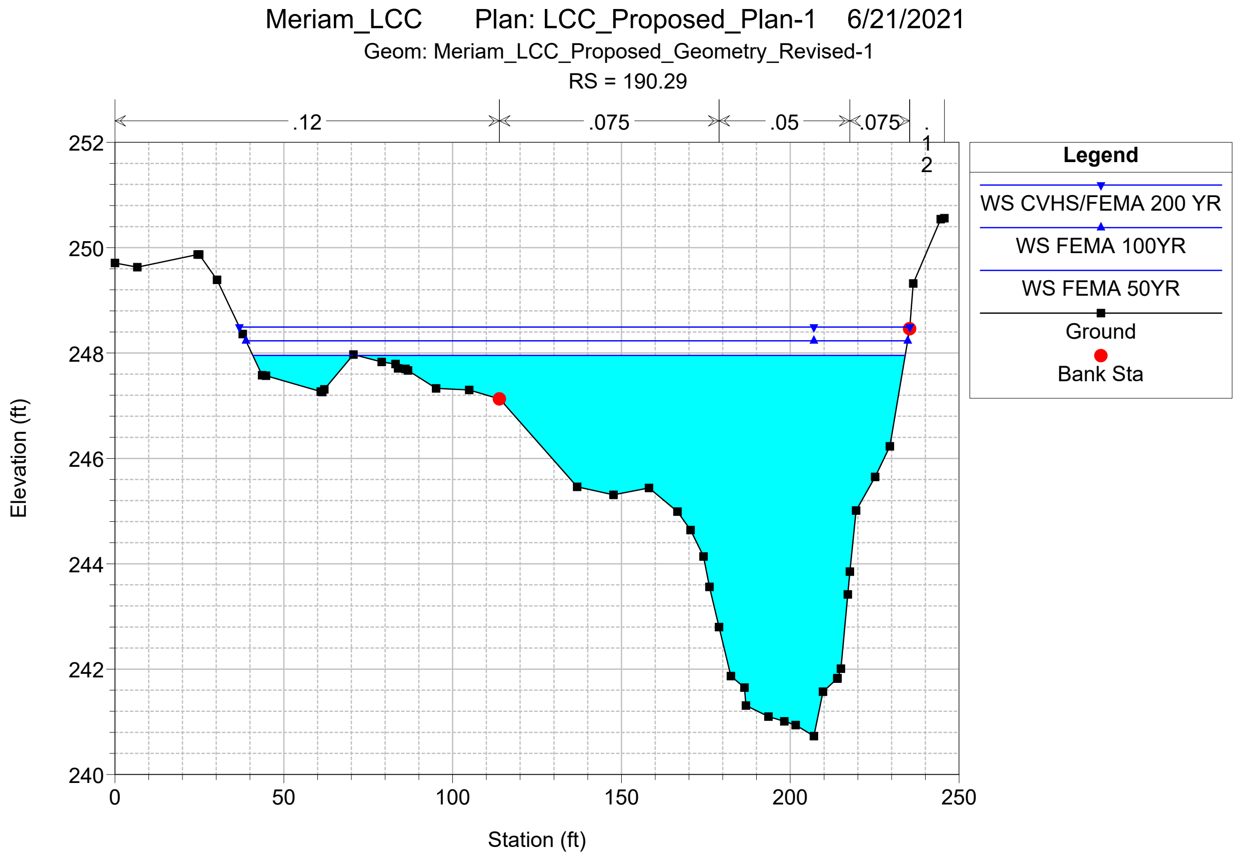
Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 377.85 BR
.12
.013
.075
.05
.075
.12
Elevation (ft)
Station (ft)
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Crit CVHS/FEMA 200 YR
Crit FEMA 100YR
Crit FEMA 50YR
Ground
Bank Sta


Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 326.31
.12
.013
.075
.05
.075
.12
Elevation (ft)
Station (ft)
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta

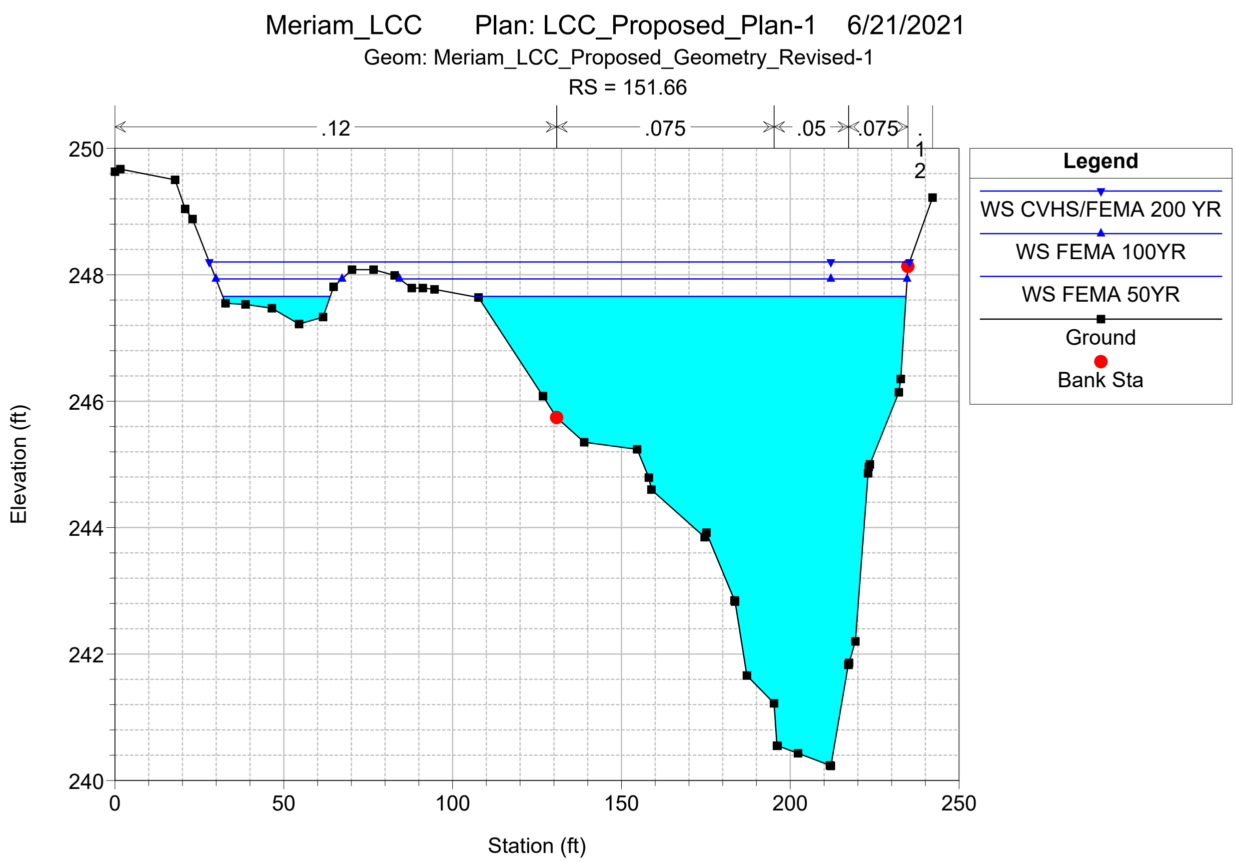

Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 286.88
.12
.013
.12
.075
.05
.075
.012
Elevation (ft)
Station (ft)
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta


Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 266.95
.12
.013
.12
.075
.05
.075
.12
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)
250
248
246
244
242
240
0
50
100
150
200
250


Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 229.39
.12
.013
.12
.075
.05
.075
.12
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)
252
250
248
246
244
242
240
0
50
100
150
200
250

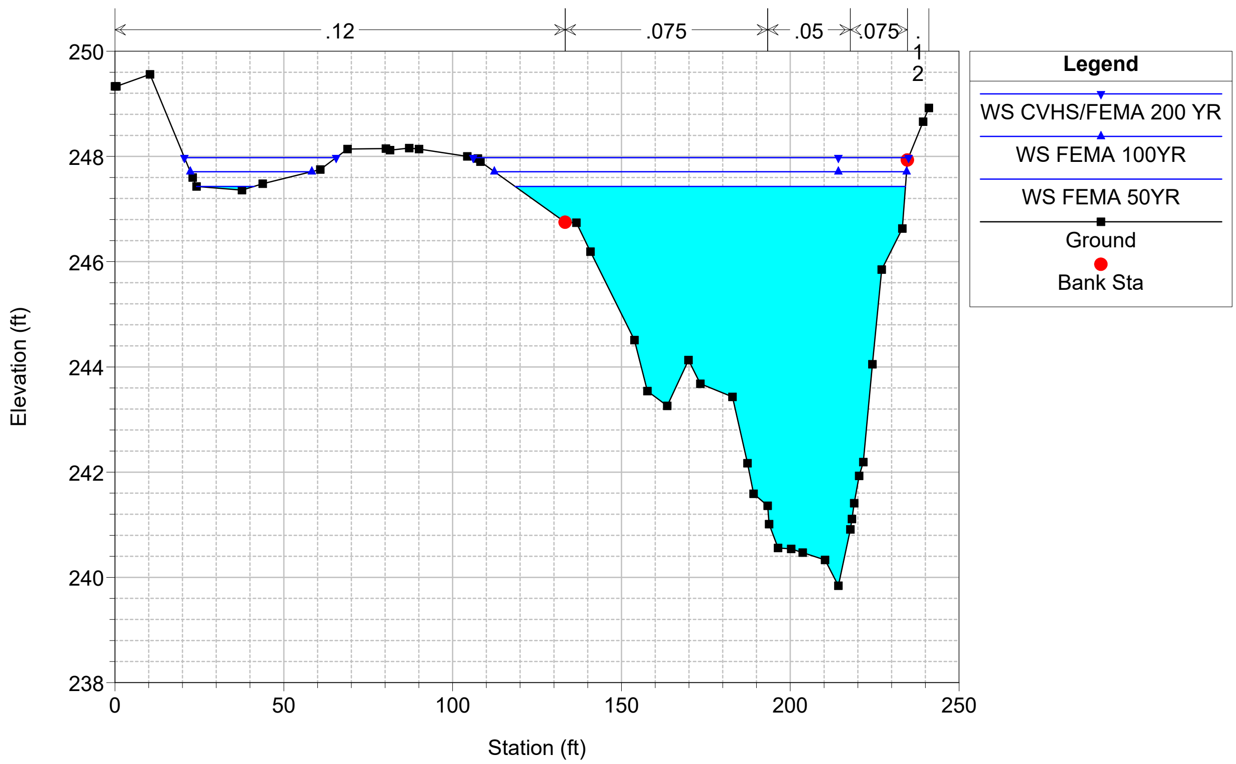


Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 190.29
.12
.075
.05
.075
1
2
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)

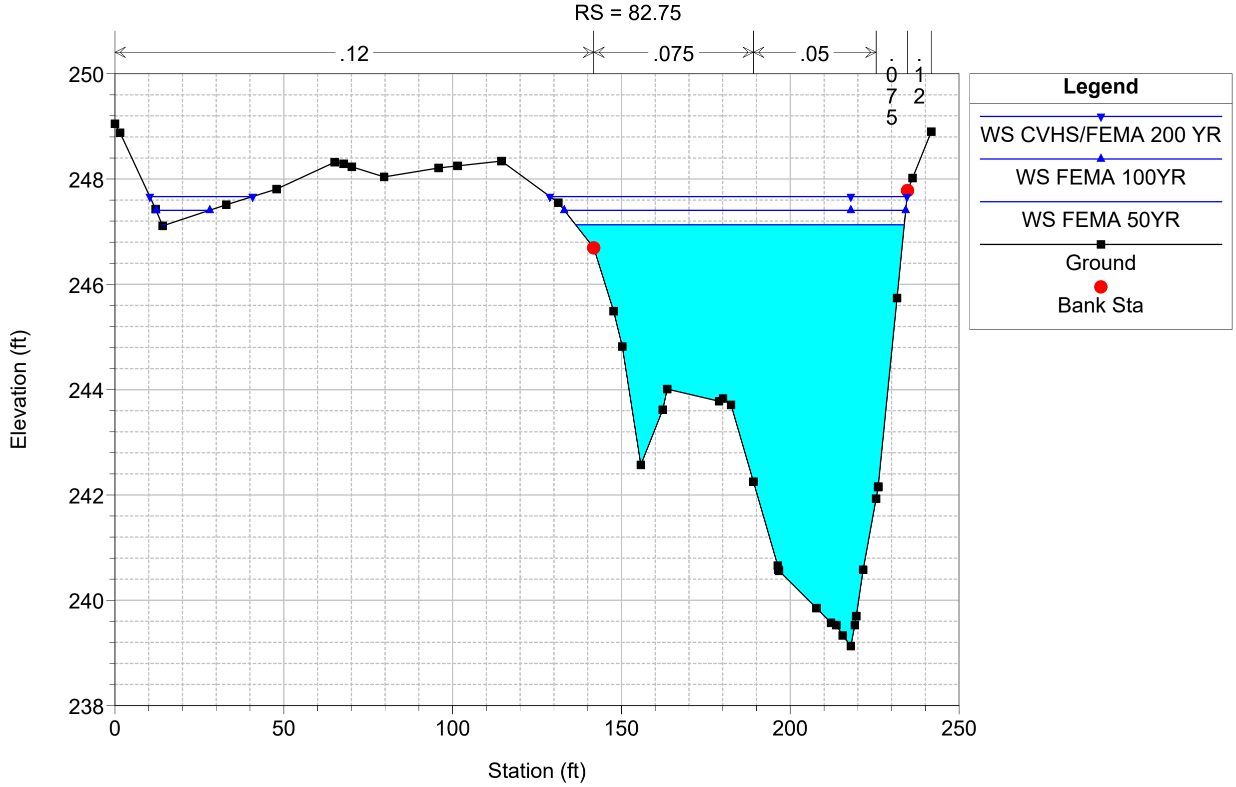


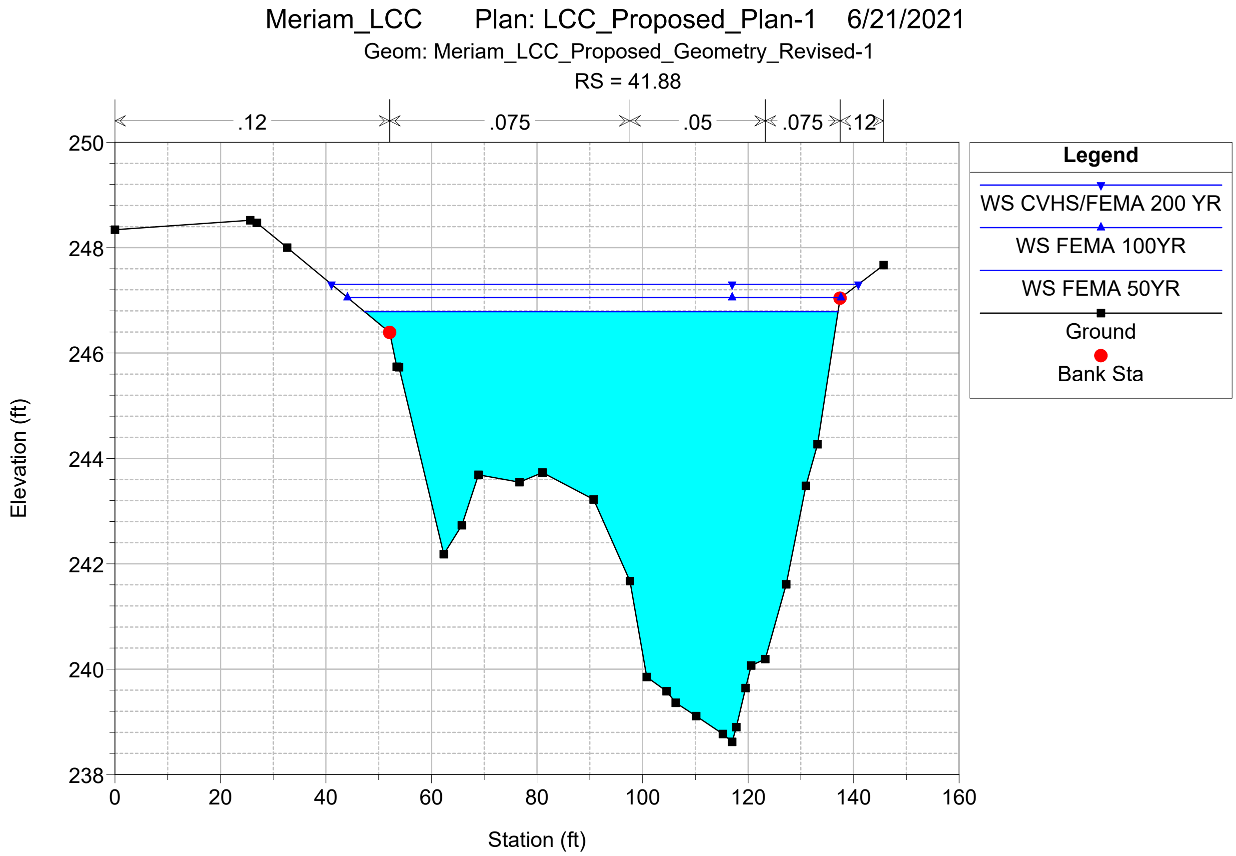
Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 151.66
.12
.075
.05
.075
1
2
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)

Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 126.11

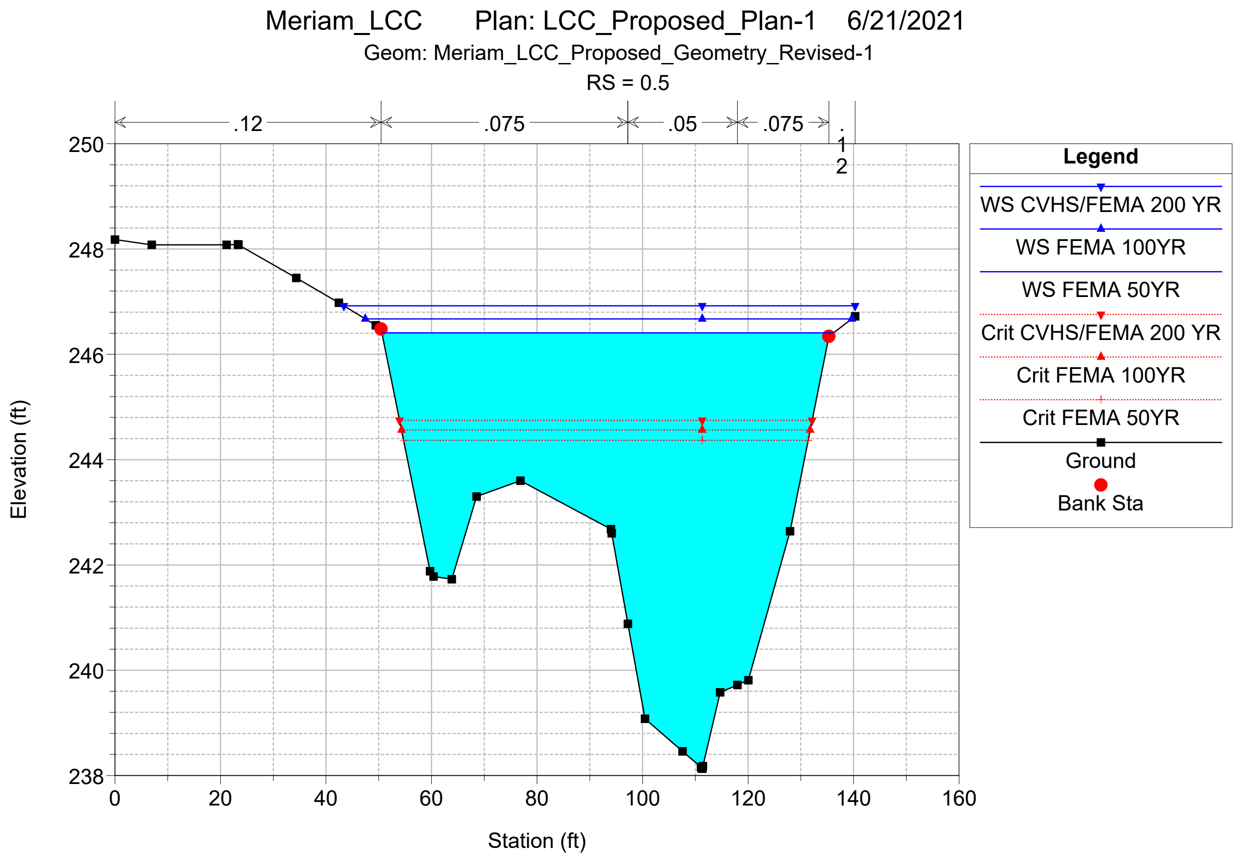


Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 82.75





Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 41.88
.12
.075
.05
.075
.12
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)



Meriam_LCC Plan: LCC_Proposed_Plan-1 6/21/2021
Geom: Meriam_LCC_Proposed_Geometry_Revised-1
RS = 0.5
.12
.075
.05
.075
Legend
WS CVHS/FEMA 200 YR
WS FEMA 100YR
WS FEMA 50YR
Crit CVHS/FEMA 200 YR
Crit FEMA 100YR
Crit FEMA 50YR
Ground
Bank Sta
Elevation (ft)
Station (ft)

APPENDIX G

HEC-RAS SCOUR CALCULATIONS – PROPOSED CONDITIONS

- Design Conditions Model
    - Scour Results

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HEC-RAS Scour Calculations
Proposed Conditions
200-Year Storm Event

Contraction Scour

		Left	Channel	Right
Input Data				
	Average Depth (ft):	1.73	3.86	
	Approach Velocity (ft/s):	1.44	4.71	
	Br Average Depth (ft):		4.69	
	BR Opening Flow (cfs):		2400.00	
	BR Top WD (ft):		77.38	
	Grain Size D50 (mm):	450.00	225.00	450.00
	Approach Flow (cfs):	217.37	2182.63	
	Approach Top WD (ft):	87.66	120.04	
	K1 Coefficient:	0.590	0.590	
Results				
	Scour Depth Ys (ft):		0.00	
	Critical Velocity (ft/s):		12.65	
	Equation:		Clear	

Pier Scour

All piers have the same scour depth

Input Data		
	Pier Shape:	Round nose
	Pier Width (ft):	2.00
	Grain Size D50 (mm):	225.00000
	Depth Upstream (ft):	1.86
	Velocity Upstream (ft/s):	6.81
	K1 Nose Shape:	1.00
	Pier Angle:	0.00
	Pier Length (ft):	56.00
	K2 Angle Coef:	1.00
	K3 Bed Cond Coef:	1.10
	Grain Size D90 (mm):	400.00000
	K4 Armouring Coef:	0.40
Results		
	Scour Depth Ys (ft):	1.62
	Froude #:	0.88
	Equation:	CSU equation

Abutment Scour

		Left	Right
Input Data			
	Station at Toe (ft):	215.02	301.62
	Toe Sta at appr (ft):	176.00	274.48
	Abutment Length (ft):	114.21	0.00
	Depth at Toe (ft):	4.29	-0.17
	K1 Shape Coef:	1.00 - Vertical abutment	
	Degree of Skew (degrees):	60	60
	K2 Skew Coef:	0.95	0.95
	Projected Length L' (ft):	98.91	0.00
	Avg Depth Obstructed Ya (ft):	2.22	
	Flow Obstructed Qe (cfs):	700.00	
	Area Obstructed Ae (sq ft):	253.75	
Results			
	Scour Depth Ys (ft):	14.59	
	Qe/Ae = Ve:	2.76	

Froude #:	0.33	
Equation:	Froehlich	Default

HEC-RAS Scour Calculations
Proposed Conditions
500-Year Storm Event

Contraction Scour

		Left	Channel	Right
Input Data				
	Average Depth (ft):	1.83	3.98	
	Approach Velocity (ft/s):	1.47	4.73	
	Br Average Depth (ft):		4.77	
	BR Opening Flow (cfs):		2500.00	
	BR Top WD (ft):		77.73	
	Grain Size D50 (mm):	450.00	225.00	450.00
	Approach Flow (cfs):	238.31	2261.69	
	Approach Top WD (ft):	88.90	120.15	
	K1 Coefficient:	0.590	0.590	
Results				
	Scour Depth Ys (ft):		0.00	
	Critical Velocity (ft/s):		12.71	
	Equation:		Clear	

Pier Scour

All piers have the same scour depth

Input Data		
	Pier Shape:	Round nose
	Pier Width (ft):	2.00
	Grain Size D50 (mm):	225.00000
	Depth Upstream (ft):	1.88
	Velocity Upstream (ft/s):	6.84
	K1 Nose Shape:	1.00
	Pier Angle:	0.00
	Pier Length (ft):	56.00
	K2 Angle Coef:	1.00
	K3 Bed Cond Coef:	1.10
	Grain Size D90 (mm):	400.00000
	K4 Armouring Coef:	0.40
Results		
	Scour Depth Ys (ft):	1.63
	Froude #:	0.88
	Equation:	CSU equation

Abutment Scour

		Left	Right
Input Data			
	Station at Toe (ft):	215.02	301.62
	Toe Sta at appr (ft):	176.00	274.48
	Abutment Length (ft):	115.47	0.00
	Depth at Toe (ft):	4.41	-0.05
	K1 Shape Coef:	1.00 - Vertical abutment	
	Degree of Skew (degrees):	60	60
	K2 Skew Coef:	0.95	0.95
	Projected Length L' (ft):	100.00	0.00
	Avg Depth Obstructed Ya (ft):	2.32	
	Flow Obstructed Qe (cfs):	738.42	
	Area Obstructed Ae (sq ft):	268.20	
Results			
	Scour Depth Ys (ft):	14.88	
	Qe/Ae = Ve:	2.75	

Froude #:	0.32	
Equation:	Froehlich	Default