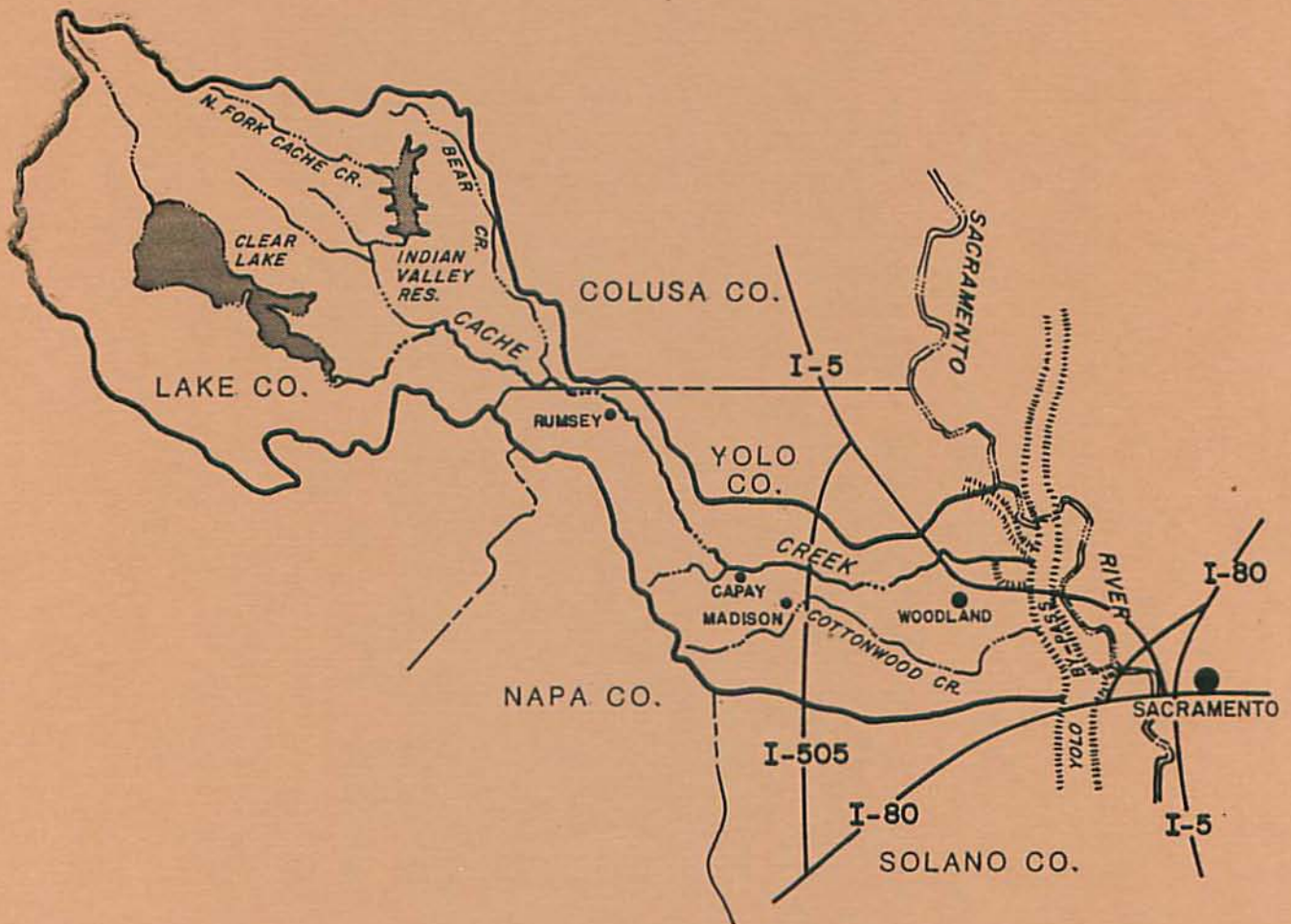


Foley

GEOMORPHIC ANALYSIS OF CACHE CREEK BETWEEN RUMSEY AND MADISON YOLO COUNTY, CALIFORNIA



GEOMORPHIC ANALYSIS
OF
CACHE CREEK
BETWEEN RUMSEY AND MADISON
YULO COUNTY, CALIFORNIA

PHASE I FINAL REPORT

Submitted to

Department of the Army
Los Angeles District
Corps of Engineers
Los Angeles, California 90053

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I. INTRODUCTION

1.1 Study Purpose

This report documents the first phase of a two-phase geomorphic analysis being conducted as part of a detailed investigation by the U.S. Army Corps of Engineers (COE) to assess potential impacts of the authorized Cache Creek Basin, California (Clear Lake Outlet Channel) project on sediment-transport characteristics of Cache Creek between Rumsey, approximately 20 miles downstream of Clear Lake and Madison (Interstate Highway 505). Figure 1.1 shows a general location map of the study reach.

The purpose of this report is to identify and qualitatively evaluate the geomorphic characteristics of Cache Creek and its watershed. The results of this report will be utilized in the Phase II quantitative geomorphic analysis and in a quantitative sediment-routing analysis being conducted concurrently by the COE Hydrologic Engineering Center.

The proposed flood-control project consists of modifications to the Clear Lake outlet channel. The 5-mile long outlet channel between Clear Lake and the Clear Lake Dam presently limits the rate at which water can be released from the Lake. This limitation, combined with large inflows, has resulted in flooding around the lake rim. To control such flooding the discharge capacity of the outlet channel is proposed to be increased. The increased discharge capacity of the outlet channel will alter the flow frequency characteristics of Cache Creek. Concern exists that such alteration may induce channel instability problems along downstream reaches of the stream. Of particular concern is the effect of the alteration of flow frequency on bank erosion and other sediment-transport-related problems.

1.2 Authorization

Simons, Li & Associates, Inc. (SLA) was contracted to conduct this study by the U.S. Army Corps of Engineers (COE), Los Angeles District, as Delivery Order Number 4 under contract number DACW09-86-D-0016. Mr. Rene Vermereen is the project manager, Los Angeles District. Mr. Edward F. Sing is the project coordinator, Sacramento District.

1.3 Principal SLA Investigators

A list of principal investigators for SLA is shown in Table 1.1.

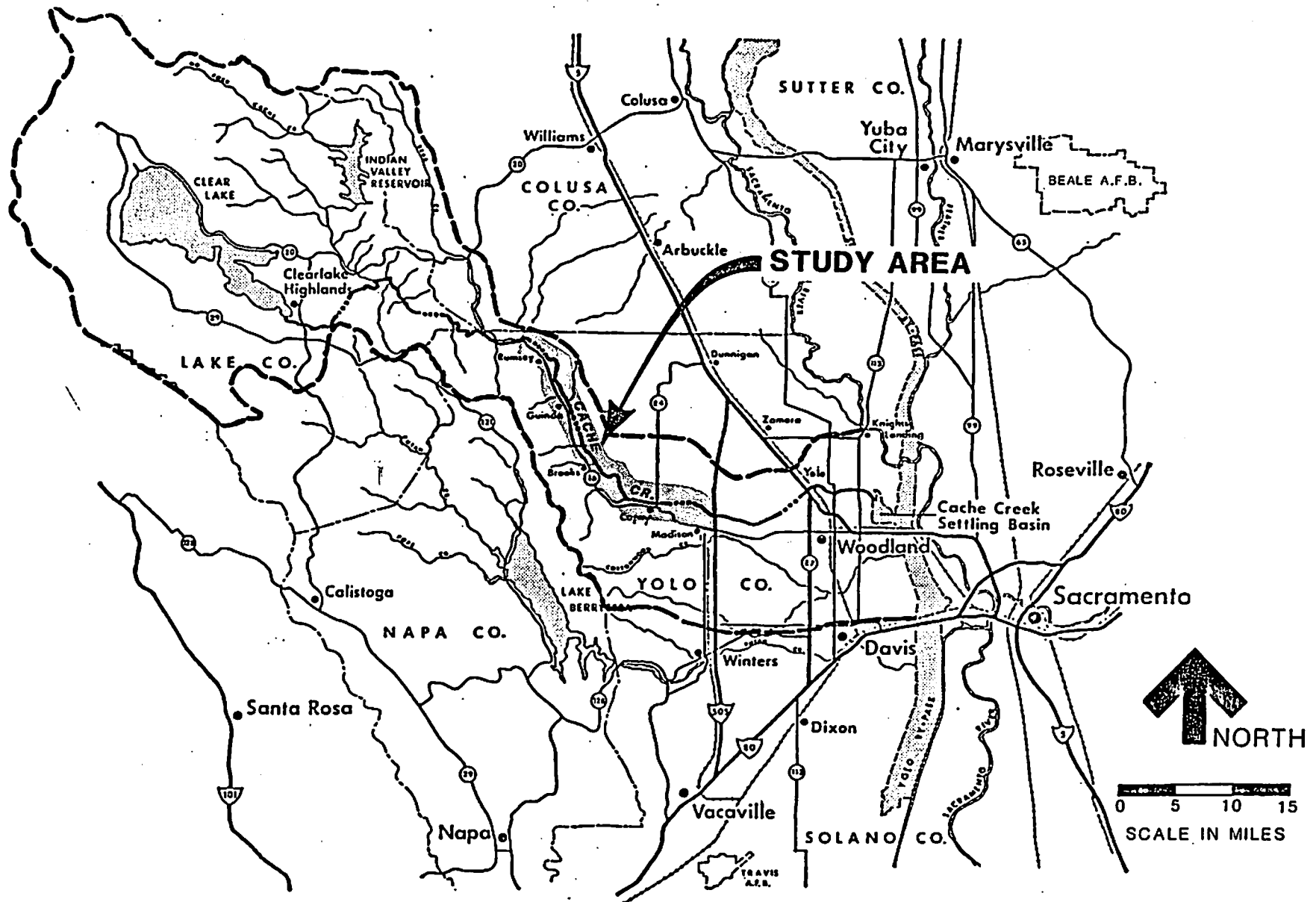


Figure 1.1. General location map of the study reach.

Table 1.1. Principal Project Investigators of
Simons, Li & Associates, Inc.

| Investigator | Function |
|----------------------------------|-----------------------|
| Ruh-Ming Li, Ph.D., P.E. | Principal In Charge |
| Robert C. MacArthur, Ph.D., P.E. | Technical Review |
| Robert A. Mussetter, P.E. | Project Manager |
| Thomas R. Grindeland, P.E. | Lead Project Engineer |
| C. Gary Wolff, E.I.T. | Hydraulic Engineer |
| Ann K. Woods | Project Engineer |

1.4 Specific Tasks Conducted for Phase I

The specific tasks conducted for Phase I of this study were as follows:

1. Conduct a site visit to project area.
2. Collection of all pertinent data for conduct of the geomorphic analyses.
3. Evaluation of channel morphology.
4. Evaluation of basin and channel geology.
5. Integration of basin and river hydrology with channel morphology and behavior.
6. Evaluation of sedimentology of basin and channel.
7. Evaluation of effects of vegetation on channel behavior.
8. Evaluation of combined effects of geology, hydrology, vegetation, and hydraulics on sediment transport and potential impacts of project.
9. Development of data and recommendations for use in developing a sediment-transport model for the study reach.
10. Preparation of a brief summary report.

1.5 Data Sources

Data used in the conduct of this analysis were collected from a variety of sources. Sources including the COE, Sacramento District, the U.S. Geological Survey, the Soil Conservation Service, and Yolo County. A complete tabulation of specific references used is provided at the end of this report.

II. FIELD RECONNAISSANCE

2.1 General

As a first step in evaluating the geomorphic characteristics of the Cache Creek study reach, a detailed field reconnaissance was conducted. The field reconnaissance included both aerial and ground surveys of channels, banks, and watershed areas. Observations were made of channel bed and bank characteristics, land use, channel stability, structures along the channel and water discharge, sediment transport, and active erosion monitoring locations. The purpose of the reconnaissance was to familiarize the study team with the watershed, observe the general tendencies of the watershed system, identify influencing factors, and document the existing condition of the study reach.

On December 2, 1986, an aerial reconnaissance of the Cache Creek watershed was conducted. Participants in the aerial survey included Dr. Robert MacArthur (SLA), Mr. Thomas Grindeland (SLA), Dr. Michael Gee [U.S. Army Corps of Engineers Hydrologic Engineering Center (HEC)], and Mr. Edward Sing [Sacramento District, Corps of Engineers (SPK)]. The survey covered Cache Creek from its confluence with the Yolo Bypass of the Sacramento River to its outlet from Clear Lake. The survey also covered the two major tributaries to Cache Creek downstream of Clear Lake: the North Fork of Cache Creek and Bear Creek. The aerial survey presented the opportunity to observe the overall Cache Creek watershed and inspection of locations with difficult ground access.

On December 3, 1986, a ground reconnaissance of the Cache Creek study reach was conducted. Participants in the ground survey included Dr. Robert MacArthur (SLA), Mr. Thomas Grindeland (SLA), Mr. Jim Blodgett (USGS), Mr. Jerry Harman [Sacramento Office, United States Geological Survey (USGS)], Mr. Michael Gee (HEC), and Mr. Edward Sing (SPK). The site visit covered Cache Creek from the upstream limit of the study reach to the Interstate Highway 505 bridge. The purpose of the site visit was to allow personnel from the USGS and SPK to familiarize HEC and SLA personnel with the general condition of the stream, point out locations of known erosion problems, and to observe water and sediment measurement stations. A second ground reconnaissance was conducted by SLA personnel on December 4 through 7, 1986. This reconnaissance effort involved walking the entire 30.5-mile study reach and making additional observations of the watershed and stream channel.

2.2 General Characteristics

Several important observations were made during the course of the aerial and ground reconnaissance of the Cache Creek study reach. These observations are grouped into two categories: watershed characteristics and study reach characteristics. The presentation of these observations are intended to give a general overview of the conditions in the watershed and document the existing conditions found within the study reach.

2.2.1 Watershed Characteristics

Cache Creek is located in Lake, Yolo, and Colusa Counties in the Coast Ranges of Northern California. Cache Creek drains approximately 1,300 square miles and trends in a northwest direction. The major tributaries to Cache Creek are Clear Lake, North Fork Cache Creek, and Bear Creek. Table 2.1 lists the drainage areas of the major tributaries to Cache Creek. Numerous minor tributaries join Cache Creek as its flows towards its confluence with the Yolo Bypass of the Sacramento River. Figure 2.1 is a map of the Cache Creek watershed.

The characteristics of the Cache Creek watershed change dramatically with location. In the upper portion of the watershed, elevations reach as much as 4,000 feet msl. The orographic effect of the Coast Range causes heavy precipitation. Consequently, vegetation in the upper watershed is quite dense. Stream channels in the upper watershed contain much coarse material and channel gradients are quite steep. Valleys are generally narrow and deep.

Cache Creek flows from the outlet of Clear Lake (Figure 2.2). The lake traps virtually all sediment from tributary watershed areas. The Cache Creek channel, downstream to its confluence with the North Fork, has large amounts of coarse bed material. The general topography surrounding Cache Creek in this reach changes with elevation to a "badlands" type setting of steep dissected gullies (Figure 2.3). This is due to a change in the general geology and a decrease in precipitation. This badland topography is thought to supply substantial amounts of sediment to the stream.

The North Fork of Cache Creek begins in an alluvial valley surrounded by hills much lower in elevation than those surrounding Clear Lake. Sediment produced by this area, however, is trapped in the Indian Valley Reservoir (Figure 2.4). Below the reservoir, the topography changes to a badlands-type condition similar to mainstem Cache Creek below Clear Lake (Figure 2.5). There is a large availability of sediment for transport from this lower reach.

Table 2.1. Drainage Areas of Major Tributaries to Cache Creek.

| Tributary Name | Drainage Area* (square miles) | Percent of Total Watershed Area |
|------------------------|----------------------------------|---------------------------------------|
| Clear Lake Basin | 520 | 40 |
| North Fork Cache Creek | 225 | 17 |
| Bear Creek | 102 | 8 |

Total Cache Creek drainage area is approximately 1,300 square miles.**

* Drainage area values of tributaries taken from communication from the Assistance Secretary of the Army Transmitting A Report from the Chief of Engineers, Department of the Army, on Cache Creek Basin, California, 1983, Appendix 1, Section B, Table B-1, Basin Physical Data.

** Total basin drainage area taken from Geological Survey Professional Paper 562-A, 1967 Sediment Transport in Cache Creek Drainage Basin in the Coast Ranges West of Sacramento, California, by L. K. Lustig and R. D. Bush.

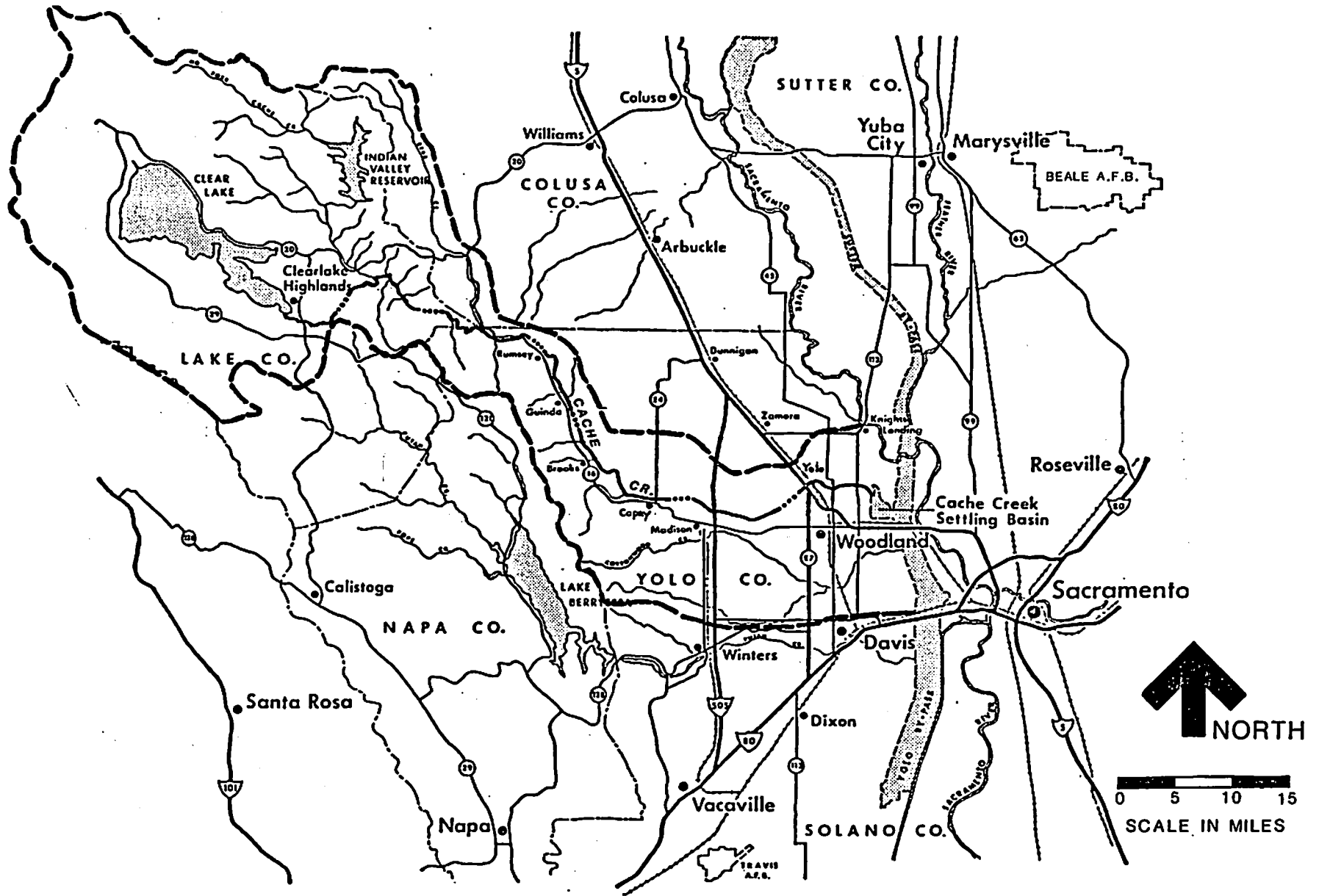


Figure 2.1. Cache Creek watershed vicinity map.



Figure 2.2. Outlet channel from Clear Lake (looking upstream).



Figure 2.3. Badland topography in upper Cache Creek watershed.



Figure 2.4. Indian Valley Reservoir on North Fork Cache Creek.



Figure 2.5. Badland topography along North Fork Cache Creek (looking upstream). Indian Valley Reservoir is located in upper right of picture.

Bear Creek is similar to the North Fork in that it also flows through a long alluvial valley in its upper reach. The gradient of this upper valley is quite flat (Figure 2.6). The supply of sediment from this area to Cache Creek is probably limited by the gradient of the valley. Below the valley, however, Bear Creek flows through steep canyons for approximately 12 miles (Figure 2.7). The watershed surrounding the channel changes, like Cache Creek below Clear Lake and the lower North Fork, to a badlands-type setting. The easily erodible badlands represent a potentially large sediment supply to Cache Creek.

Below the confluence of Bear Creek, the Cache Creek channel intersects outcrops of moderate- to deep-dipping sedimentary rock. In several locations, the stream flows across ledges of these units. The channel appears to rest virtually on bedrock. This condition prevails to the approximate upstream study limit at the head of the Capay Valley near Rumsey.

As mentioned previously, the amount of precipitation lessens with elevation. At the upstream study limit, the amount of annual precipitation is approximately half that experienced in the upper watershed. The density of vegetation, therefore, decreases with elevation. The amount of sediment supplied to the channel from watershed areas between Rumsey and Capay is thought to be much greater than predicted based on drainage area compared to the upstream watershed. This is probably due to conditions related to increased badland topography in surrounding hill areas, land use, and effects of decreased precipitation (Figure 2.8).

Below Capay, the Cache Creek channel flows into a wide alluvial fan area in which deposition of sediment is the dominant process (Figure 2.9). The supply of sediment to the channel from this area is primarily from overland flow and local gullying along the stream. The Cache Creek channel in this area has been extensively influenced by sand and gravel extraction operations.

2.2.2 Study Reach Characteristics

2.2.2.1 General

Field reconnaissance of the study reach was made in order to familiarize the project team with the study area, identify the general processes affecting the stream, and to document the existing condition of the study reach. As part of the field reconnaissance, the entire study reach was walked, obser-



Figure 2.6. Upper valley along Bear Creek (looking east).

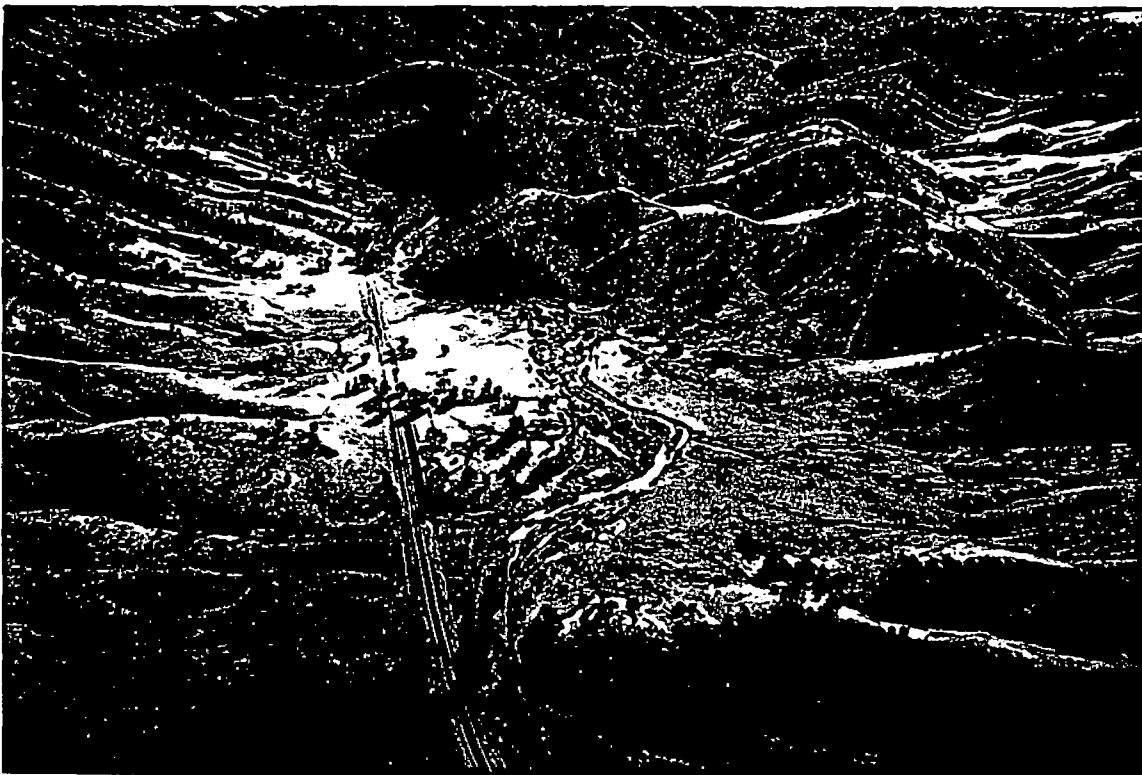


Figure 2.7. Upstream entrance to Bear Creek Canyon (looking downstream).



Figure 2.8. Cache Creek at approximate location of upstream study limit (looking downstream).



Figure 2.9. Cache Creek at Esparto Bridge (looking downstream).

vations were made of channel characteristics, bank instability, tributaries, gullying, bed and bank materials, hillslope instability, and structures which might influence the stream. Also observed were locations of bank erosion that are being monitored by the USGS as part of an ongoing sediment investigation. The purpose of the USGS study is to determine the amount of bank movement with time and discharge.

In the following sections, descriptions of the various items observed along the stream are presented. To aid in referencing locations referred to in the discussion, Plates 1 through 6 were prepared. A stationing system along the study reach was established on the plates which delineates river miles upstream of the Interstate 505 crossing. The upstream study limit is located at River Mile (RM) 30.5 (California Highway 16 bridge near Rumsey). Figure 2.10 is a map showing the study reach.

To aid in the description, the study reach was also divided into smaller subreaches of similar characteristics. Notable features were tabulated for easy reference. Table 2.2 lists all man-made structures, along with their location and description. Table 2.3 lists the description and location of geologic features. Table 2.4 lists tributaries and significant gullies. A table of bank erosion sites, including bank height and description, is given in Table 2.5.

2.2.2.2 Reach 1

A narrow canyon with steep hillslopes and dense vegetation characterize Reach 1. This reach begins at RM 29.2 and ends where California State Highway 16 crosses Cache Creek at RM 30.5. A natural outcrop of bedrock runs perpendicular to the creek at the downstream limit. The streambed is trapezoidal in shape and well armored with large boulders and some bedrock outcrops. A USGS gaging station has been established at RM 30.1. Bank slopes are steep and heavily vegetated. The stability of the banks appear to be good as only a short area 300 feet downstream of the California State Highway 16 bridge was eroding. Typical conditions are shown in Figures 2.11 and 2.12.

2.2.2.3 Reach 2

Reach 2 extends along the Cache Creek channel from the rock outcrop in the narrow canyon at RM 29.2 into the Capay Valley where the channel is more active. The reach starts at Rumsey Bridge, RM 27.0 and ends at RM 29.2. The

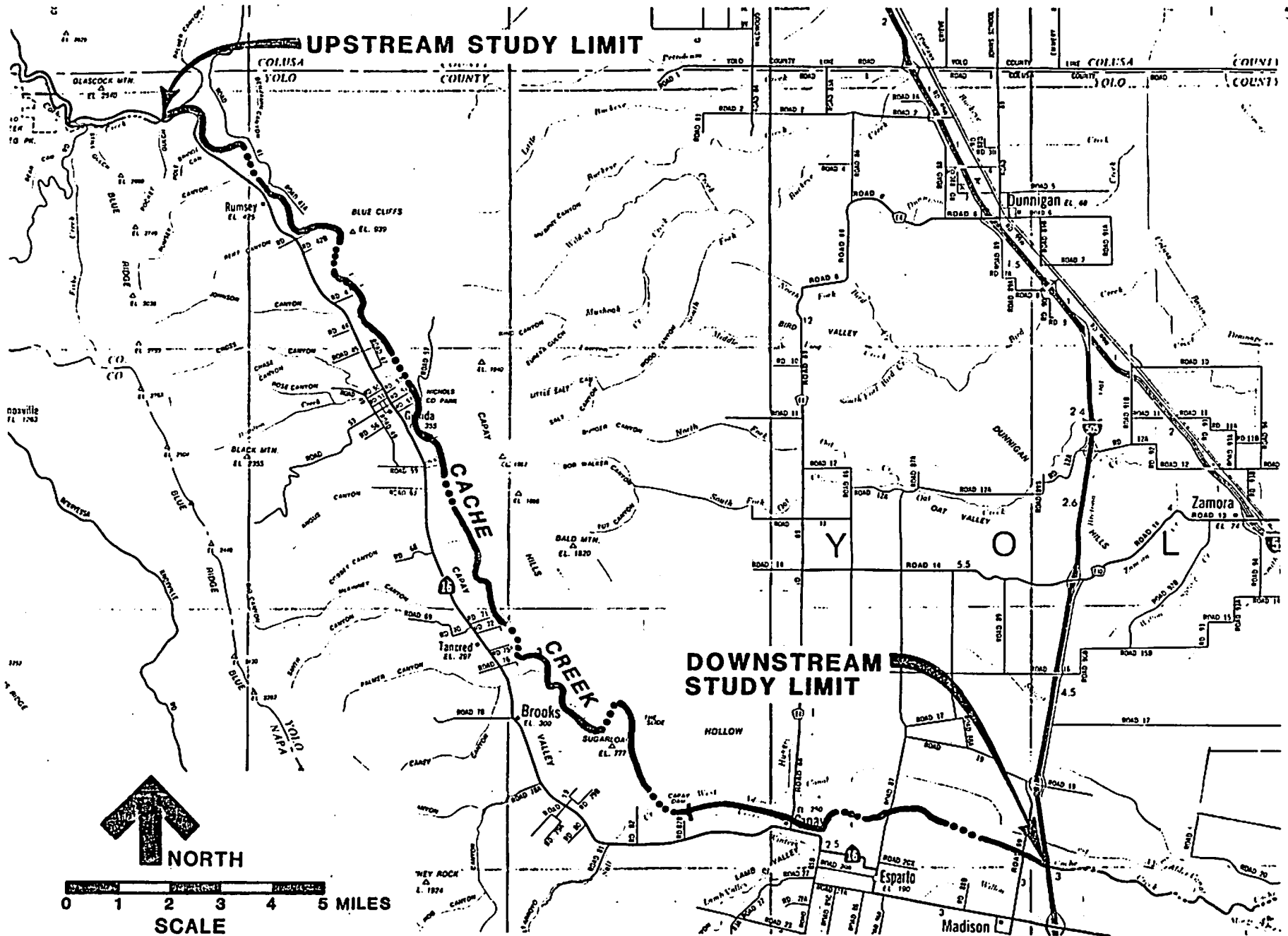


Figure 2.10. Cache Creek study reach.

Table 2.2. Man-Made Structures Along Cache Creek.

| Reach | Type | Location (River Mile) | Comments |
|-------|-------------------------|--------------------------|---|
| 10 | I-505 Bridge | 0.00 | Concrete structure at downstream study limit. |
| 9 | Abandoned bridge | 0.56 | Structure removed. |
| 9 | Gravel mining operation | 0.58 - 1.20 | Current mining operation in channel. |
| 9 | Dikes | 2.59 | Transverse earth dikes along right bank. |
| 8 | Esparto Bridge | 3.52 | Concrete structure approximately 640 feet wide; bridge constricts flow. |
| 8 | Dikes | 4.03 - 4.18 | Transerve earth dikes along right bank. |
| 8 | Graded slope | 5.20 - 5.40 | Left bank has been graded to a 3:1 slope. |
| 8 | Rebuilt bank | 5.00 | Right bank is eroded along the outside of bend. Soil has been dumped to replace it. |
| 8 | Rebuilt bank | 5.45 | Right bank is eroded along outside of bend. Soil has been dumped to replace it. |
| 7 | Capay Bridge | 5.62 | Concrete structure, 430 feet wide. |
| 7 | Bank protection | 7.62 | Concrete bank protection along left bank immediately downstream of Capay Dam. |
| 6 | Capay Dam | 7.68 | Concrete weir used to supply water to West Adams Canal and Winters Canal. |
| 6 | Bank protection | 7.79 - 7.80 | Car bodies piled along right bank upstream of Capay Dam. |

Table 2.2. Man-Made Structures Along Cache Creek (continued).

| Reach | Type | Location (River Mile) | Comments |
|-------|--------------------------|--------------------------|---|
| 6 | Bank protection | 8.00 - 8.11 | Car bodies piled along right bank to protect outside of bend. |
| 6 | Bank protection | 8.52 | Riprap along right bank. |
| 6 | Pump | 8.58 | Pump for irrigation supply on right bank. |
| 6 | Wood platform | 8.78 | Wood platform on right bank. |
| 6 | Abandoned wood structure | 9.15 | Old wood bridge; no longer used; abutments remain in low-flow channel. |
| 6 | Brooks gaging station | 11.64 | Gage set by USGS on right bank. |
| 5 | Bank protection | 12.24 | Car bodies piled along right bank on outside of bend; USGS active site 10. |
| 5 | Channel excavation | 13.62 - 13.75 | Channel excavated. |
| 5 | Channel excavation | 14.15 - 14.32 | Channel excavated. |
| 5 | Bank protection | 14.15 - 14.20 | Riprap (18-inch diameter) bank protection along outside of bend on right bank. |
| 5 | Footbridge | 14.75 | Small suspended footbridge spanning full width of channel. |
| 5 | Channel excavation | 16.98 - 17.02 | Channel excavated. |
| 5 | Bank protection | 17.70 | Concrete slabs and rubble placed on outside of bend along right bank. |
| 4 | Rebuilt bank | 20.00 | Right bank eroded along outside curve. Soil has been dumped to replace eroded material. |
| 4 | Guinda Bridge | 20.02 | Steel structure, 270 feet wide. |

Table 2.2. Man-Made Structures Along Cache Creek (continued).

| Reach | Type | Location (River Mile) | Comments |
|-------|----------------------------------|--------------------------|---|
| 4 | Bank protection | 21.00 | Bank protection along left bank; constructed of tires wired together. |
| 4 | Bank protection | 21.60 - 21.70 | Riprap along right bank. |
| 3 | Bank protection | 24.11 - 24.24 | Riprap along right bank. |
| 2 | Rumsey Bridge | 27.03 | Concrete structure, 200 feet wide. DWR gaging station located on downstream face of bridge. |
| 1 | Gaging station | 30.01 | USGS gaging station (above Rumsey) on right bank. |
| 1 | California State Route 16 Bridge | 30.46 | Concrete structure, 300 feet wide; upstream study limit. |

Table 2.3. Geologic Features Along Cache Creek.

| Reach | Type | Location (River Mile) | Comments |
|-------|-------------------|--------------------------|--|
| 6 | Grade control | 10.50 | Landslide has deposited large boulders into the stream. Finer material has been carried away. Boulders act as grade control. |
| 6 | Groundwater | 11.49 | Groundwater is seeping through right bank. |
| 4 | Landslide | 18.50 | Hillslide on left bank has slumped and pushed into the stream. The toe of the slide has been eroded away by the stream. Bank height along the toe of the slide is approximately 30 feet. |
| 4 | Clay outcrop | 19.39 - 19.7 | A clay seam runs along the toe of both banks. |
| 4 | Clay outcrop | 20.02 - 20.22 | A clay seam runs along the toe of the left bank. |
| 4 | Bedrock | 21.00 | Isolated bedrock outcrop in middle of channel. Outcrop is not exposed along banks. |
| 4 | Landslide | 21.34 | A small portion of the landslide has pushed into the stream. |
| 4 | Clay outcrop | 22.22 - 22.54 | A clay seam runs along the toe of the left bank. |
| 4 | Sandstone outcrop | 23.46 | A sandstone seam runs along the toe of the right bank. |
| 3 | Sandstone outcrop | 26.37 | A sandstone seam is exposed at the toe of the right bank. |
| 3 | Sandstone outcrop | 26.56 - 26.89 | A sandstone seam is exposed at the toe of the left bank. |
| 2 | Clay outcrop | 27.65 - 28.11 | A clay seam is exposed at the toe of both banks. |
| 2 | Clay outcrop | 28.25 - 28.73 | A clay seam is exposed at the toe of the both banks. |

Table 2.3. Geologic Features Along Cache Creek (continued).

| Reach | Type | Location (River Mile) | Comments |
|-------|--------------|--------------------------|--|
| 1 | Bedrock | 29.24 | Bedrock outcrop crosses the channel. Outcrop is acting as a grade control. |
| 1 | Clay outcrop | 29.24 - 29.50 | A clay seam is exposed at the toe of the right bank. |

Table 2.4. Tributaries and Significant Gullies Along Cache Creek.

| Reach | Type | Location | | Tributary Name or Source* | Comments |
|-------|-------|------------|-------|---|--|
| | | River Mile | Bank | | |
| 6 | Trib. | 8.26 | Right | Pipeline Canyon Unnamed canyon | Contribution of sediment from tributary is very small. |
| 6 | Trib. | 8.41 | Right | Salt Creek | Contribution of sediment from tributary is very small. |
| 5 | Trib. | 12.98 | Left | Unnamed | Contribution of sediment from tributary is very small. |
| 5 | Trib. | 13.28 | Right | Palmer Unnamed canyon Carey Canyon Unnamed canyon | Contribution of sediment from tributary is very small. |
| 5 | Trib. | 13.77 | Left | Unnamed canyon | Contribution of sediment from tributary is very small. |
| 5 | Trib. | 15.49 | Right | McKinney Canyon Smith Canyon Unnamed canyon (Patterson Spring) | Contribution of sediment from tributary is very small. |
| 5 | Trib. | 15.50 | Left | Unnamed canyon | Contribution of sediment from tributary is very small. |
| 5 | Gully | 16.13 | Left | Unnamed gully | Contribution of sediment from tributary is very small. |
| 5 | Trib. | 16.64 | Left | Unnamed canyon | Contribution of sediment from tributary is very small. |
| 5 | Trib. | 16.90 | Right | Cobby Canyon | Contribution of sediment from tributary is very small. |

* Tributaries with two or more names have several sources that come together to form one tributary flowing into Cache Creek.

Table 2.4. Tributaries and Significant Gullies Along Cache Creek (continued).

| Reach | Type | Location | | Tributary Name or Source* | Comments |
|-------|-------|------------|-------|--|--|
| | | River Mile | Bank | | |
| 5 | Trib. | 17.17 | Left | Unnamed canyon | Contribution of sediment from tributary is very small. |
| 5 | Gully | 17.35 | Left | Unnamed canyon | Contribution of sediment from tributary is very small. |
| 5 | Trib. | 17.47 | Left | Unnamed canyon | Contribution of sediment from tributary is very small. |
| 5 | Gully | 17.89 | Left | Unnamed canyon | Contribution of sediment from tributary is very small. |
| 4 | Trib. | 18.25 | Right | Pierce Canyon Angus Canyon | Contribution of sediment from tributary is very small. |
| 4 | Trib. | 18.85 | Left | Unnamed | Contribution of sediment from tributary is very small. |
| 4 | Trib. | 19.07 | Left | Unnamed canyon | Contribution of sediment from tributary is very small. |
| 4 | Trib. | 19.70 | Left | Unnamed canyon | Contribution of sediment from tributary is very small. |
| 4 | Trib. | 20.89 | Right | Cross Canyon Chase Canyon Rose Canyon Hamilton Canyon | Contribution of sediment from tributary is very small. |
| 4 | Trib. | 22.17 | Left | Unnamed canyon | Contribution of sediment from tributary is very small. |
| 3 | Trib. | 22.55 | Right | Johnson Canyon | Contribution of sediment from tributary is very small. |

* Tributaries with two or more names have several sources that come together to form one tributary flowing into Cache Creek.

Table 2.4. Tributaries and Significant Gullies Along Cache Creek (continued).

| Reach | Type | Location | | Tributary Name or Source* | Comments |
|-------|-------|------------|-------|---------------------------|--|
| | | River Mile | Bank | | |
| 3 | Trib. | 23.03 | Left | Unnamed canyon | Contribution of sediment from tributary is very small. |
| 3 | Trib. | 23.58 | Right | Unnamed canyon | Contribution of sediment from tributary is very small. |
| 3 | Gully | 24.77 | Left | Blue Cliffs | Gully outlet is higher in elevation than channel bed. Sediment in gully is similar to sediment in channel. |
| 3 | Trib. | 25.43 | Left | Unnamed canyon | Contribution of sediment from tributary is very small. |
| 3 | Trib. | 25.85 | Right | Reiff Canyon | Contribution of sediment from tributary is very small. |
| 3 | Trib. | 26.37 | Right | Unnamed canyon | Contribution of sediment from tributary is very small. |
| 3 | Trib. | 26.79 | Left | Unnamed canyon | Tributary outlet is higher in elevation than channel bed. Sediment in tributary is similar to sediment in channel. |
| 2 | Trib. | 27.49 | Right | Rumsey Canyon | Contribution of sediment from tributary is very small. |
| 2 | Trib. | 27.71 | Left | Benjamin Canyon | Tributary outlet is at same elevation as channel; sediment in tributary is similar to sediment in channel. |

* Tributaries with two or more names have several sources that come together to form one tributary flowing into Cache Creek.

Table 2.4. Tributaries and Significant Gullies Along Cache Creek (continued).

| Reach | Type | Location | | Tributary Name or Source* | Comments |
|-------|-------|------------|-------|---------------------------|--|
| | | River Mile | Bank | | |
| 2 | Trib. | 28.17 | Left | Unnamed canyon | Tributary outlet is higher in elevation than channel bed. Sediment in tributary is smaller than sediment in channel. |
| 2 | Trib. | 28.53 | Left | Unnamed canyon | Contribution of sediment from tributary is very small. |
| 2 | Trib. | 29.22 | Right | Pole Bridge Canyon | Contribution of sediment from tributary is very small. |

* Tributaries with two or more names have several sources that come together to form one tributary flowing into Cache Creek.

Table 2.5. Bank Erosion Along Cache Creek.

| Reach | Location (River Mile) | Height (ft) | Bank | Comments |
|-------|--------------------------|----------------|-------|--|
| 10 | 0.10 - 0.50 | 10-15 | Right | Nearly vertical bank with toe composed of large cobbles, located on outside of bend. |
| 9 | 3.00 - 3.18 | 10-15 | Right | Nearly vertical bank with toe composed of large cobbles, located on outside of bend. |
| 9 | 3.35 | 15 | Right | Nearly vertical bank with toe composed of large cobbles, located on outside of bend. |
| 8 | 4.05-4.89 | 20-25 | Right | Nearly vertical bank with toe composed of large cobbles, located on outside of bend. |
| 8 | 4.41-4.50 | 10 | Left | Nearly vertical bank with toe composed of large cobbles, located on outside of bend. |
| 8 | 4.83-4.97 | 8 | Left | Nearly vertical bank with toe composed of large cobbles, located on outside of bend. |
| 8 | 5.18-5.62 | 15 | Right | Nearly vertical bank with toe composed of large cobbles, located on outside of bend. |
| 7 | 5.62-5.89 | 8 | Left | Nearly vertical bank with toe composed of large cobbles, located on outside of bend. |
| 7 | 7.20-7.68 | 2-5 | Right | Nearly vertical bank downstream of Capay Dam on inside of bend. |
| 6 | 7.68-7.96 | 10 | Right | Nearly vertical bank upstream of Capay Dam on outside of bend. |

Table 2.5. Bank Erosion Along Cache Creek (continued).

| Reach | Location (River Mile) | Height (ft) | Bank | Comments |
|-------|--------------------------|----------------|-------|---|
| 6 | 8.06-8.30 | 8 | Left | Nearly vertical bank with toe composed of large cobbles. |
| 6 | 8.79-8.85 | 15 | Right | Steep bank on outside of bend. |
| 6 | 8.95-9.48 | 5-10 | Left | Nearly vertical bank with toe composed of large cobbles, located on outside of bend. |
| 6 | 9.17-9.20 | 8 | Right | Steep bank with no vegetation, located on a straight reach of river. |
| 6 | 9.46 | 8 | Right | Steep bank with no vegetation, located on a straight reach of river. |
| 6 | 9.48-9.85 | 5 | Left | Undercut terraced bank erosion, 2-3 foot terraces with long grass protecting top layer of soil and erosion undercutting grass protection. |
| 6 | 10.11-10.77 | 10-70 | Left | Nearly vertical bank cut into steep valley. Toe of bank composed of large boulders. A landslide has occurred in this area and hillsides are unstable. |
| 6 | 10.11-10.56 | 5-10 | Right | Nearly vertical bank with toe composed of large cobbles, located on outside of bend. |
| 6 | 10.85 | 8 | Left | Nearly vertical bank with no vegetation for protection, located on a straight reach of river. |
| 6 | 11.15-11.41 | 20 | Right | Vertical bank along outside of bend. |

Table 2.5. Bank Erosion Along Cache Creek (continued).

| Reach | Location (River Mile) | Height (ft) | Bank | Comments |
|-------|--------------------------|----------------|--------|--|
| 6 | 11.52-11.62 | 15 | Right | Nearly vertical bank along outside of bend. |
| 6 | 11.80-11.92 | 2-5 | Left | Terraced erosion, 2-3 foot terraces with grass protecting top layer of soil along outside of bend. |
| 6 | 11.92-12.01 | 10 | Left | Nearly vertical bank with toe composed of large cobbles, located on outside of bend. |
| 6 | 12.0-12.24 | 5-10 | Right | Nearly vertical bank with toe composed of large cobbles, located on outside of bend. (USGS sediment engineering investigation; "active" bank erosion site 10). |
| 5 | 12.56-12.72 | 5 | Left | Nearly vertical bank along outside of bend. |
| 5 | 12.73-13.45 | 8-15 | Left | Nearly vertical slopes with toe composed of large cobbles. This is a divided flow area in high flows with erosion running along both banks of high-flow channel. |
| 5 | 12.80-12.89 | 10 | Right | Nearly vertical bank along outside of bend. |
| 5 | 12.73-13.45 | 8 | Island | Nearly vertical bank running along both the heel and the toe of the island. |
| 5 | 13.45-13.68 | 5 | Left | Nearly vertical bank with toe composed of large cobbles, located on outside of bend. |

Table 2.5. Bank Erosion Along Cache Creek (continued).

| Reach | Location (River Mile) | Height (ft) | Bank | Comments |
|-------|--------------------------|----------------|-------|--|
| 5 | 13.68-13.70 | 5 | Right | Nearly vertical bank with toe composed of large cobbles, located on outside of bend. |
| 5 | 13.88-13.96 | 5 | Left | Nearly vertical bank with toe composed of large cobbles on outside of bend. |
| 5 | 14.00-14.32 | 8 | Right | Nearly vertical bank with about 600 feet of bank protection (riprap) starting at RM 14.15 and ending at 14.2. Bank erosion located on outside of bend. |
| 5 | 14.63-14.75 | 5-20 | Left | Nearly vertical bank along outside of bend. |
| 5 | 15.72-16.40 | 5-20 | Left | Nearly vertical bank along outside curve of large meander loop. |
| 5 | 16.51-16.62 | 10 | Right | Nearly vertical bank with toe composed of large cobbles, located on outside of bend. |
| 5 | 16.62-16.75 | 10 | Left | Nearly vertical bank with toe composed of large cobbles, located on outside of bend. |
| 5 | 16.85-16.94 | 10 | Left | Nearly vertical bank with toe composed of large cobbles, located on outside of bend. |
| 5 | 16.85-17.0 | 10 | Right | Nearly vertical bank with toe composed of large cobbles located on outside of bend. |
| 5 | 17.26-17.72 | 5-15 | Left | Nearly vertical bank with toe composed of large cobbles located on outside curve of large meander loop. |

Table 2.5. Bank Erosion Along Cache Creek (continued).

| Reach | Location (River Mile) | Height (ft) | Bank | Comments |
|-------|--------------------------|----------------|-------|---|
| 5 | 17.66-17.77 | 10 | Right | Nearly vertical bank with some bank protection. Erosion located on outside of bend. |
| 5 | 17.98-18.02 | 5 | Left | Steep bank with toe composed of large cobbles. |
| 4 | 18.17-18.34 | 8 | Right | Nearly vertical bank with toe comprised of large cobbles located on outside of bend. |
| 4 | 18.43-18.66 | 30-50 | Left | Landslide pushing into stream. Toe of landslide being eroded. |
| 4 | 18.49-18.60 | 1 | Right | Terraced erosion on sandbar (see Section 2.2.2.5 for explanation). |
| 4 | 18.75-19.22 | 5-20 | Left | Nearly vertical bank with toe composed of large cobbles. Located on outside of bend and downstream along unstable hillside. |
| 4 | 18.85-19.70 | 5-30 | Right | Nearly vertical bank with toe composed of large cobbles around outside of bend and downstream along straight reach. |
| 4 | 19.60-19.89 | 10 | Left | Nearly vertical bank with toe composed of large cobbles. Located at upstream end of island. |
| 4 | 19.83-20.03 | 10 | Right | Nearly vertical bank. Portions of bank rebuilt. Located on outside of bend downstream of Guinda Bridge. |

Table 2.5. Bank Erosion Along Cache Creek (continued).

| Reach | Location (River Mile) | Height (ft) | Bank | Comments |
|-------|--------------------------|----------------|-------|--|
| 4 | 19.98-20.32 | 10-20 | Left | Steep slope located on outside of bend just upstream of Guinda Bridge. |
| 4 | 20.24-20.73 | 5-20 | Right | Nearly vertical bank eroded into orchard. Toe of bank is comprised of large cobbles. Located on outside of bend. (USGS "active" bank erosion site 24). |
| 4 | 20.56-20.78 | 15 | Left | Nearly vertical bank with toe composed of large cobbles located on outside of bend. |
| 4 | 20.79-21.11 | 10 | Right | Nearly vertical bank with toe composed of large cobbles located on outside of bend. A tributary enters channel at same location. |
| 4 | 20.98-21.76 | 5-25 | Left | Nearly vertical bank along toe of landslide which is pushing into the channel. |
| 4 | 21.49-21.74 | 20 | Right | Nearly vertical bank with toe composed of large cobbles and a portion has riprap protection. Erosion is located along outside of bend. |
| 4 | 21.81-21.89 | 10 | Right | Nearly vertical bank located along outside of bend. |
| 4 | 21.98-22.18 | 10 | Left | Nearly vertical bank located along unstable hillside. |

Table 2.5. Bank Erosion Along Cache Creek (continued).

| Reach | Location (River Mile) | Height (ft) | Bank | Comments |
|-------|--------------------------|----------------|-------|---|
| 4 | 22.22-22.62 | 15-28 | Left | Nearly vertical bank located along outside of bend. Bank has a clay seam running along toe. |
| 3 | 22.49-22.87 | 10 | Right | Nearly vertical bank located along outside of bend. |
| 3 | 22.97-23.07 | 20 | Left | Steep bank located along outside of bend. |
| 3 | 22.98-23.56 | 5-30 | Right | Steep bank beginning on outside of bend and continuing down through sandbar. A sandstone layer runs along the toe for a short distance. (USGS "active" bank erosion site 30). |
| 3 | 23.60-23.73 | 10 | Right | Steep bank located along outside of bend. |
| 3 | 23.79-24.05 | 5 | Right | Steep bank located along outside of bend along historic flow channel. |
| 3 | 24.26-24.64 | 5 | Right | Steep bank located along straight constricted channel. |
| 3 | 24.39-24.83 | 25-100 | Left | Unstable hillside along outside of bend located at Blue Cliffs area. |
| 3 | 25.00-25.15 | 8 | Right | Steep banks along constricted channel with sparse vegetation. |
| 3 | 25.18-25.26 | 8 | Left | Steep banks along constricted channel with sparse vegetation. |
| 3 | 25.30-25.37 | 8 | Right | Steep banks along constricted channel with sparse vegetation. |

Table 2.5. Bank Erosion Along Cache Creek (continued).

| Reach | Location (River Mile) | Height (ft) | Bank | Comments |
|-------|--------------------------|----------------|-------|---|
| 3 | 25.41-25.58 | 10 | Left | Steep bank along outside of bend. |
| 3 | 25.68-25.75 | 8 | Left | Steep banks along constricted channel with sparse vegetation. |
| 3 | 25.73-26.35 | 5-10 | Right | Steep banks with toe composed of large cobbles located on outside of bend. |
| 3 | 26.43-26.90 | 5-40 | Left | Nearly vertical bank located on outside of bend. (USGS "active" erosion site 35). A sandstone seam runs along the toe. |
| 2 | 27.03-27.45 | 10 | Right | Steep banks located on outside of bend. (USGS "active" bank erosion site 36). |
| 2 | 27.45-27.55 | 30 | Left | Nearly vertical bank located on outside of bend. (USGS "active" bank erosion site 37). |
| 2 | 27-75-28.11 | 10 | Right | Steep bank along constricted channel with sparse vegetation. Bank has clay seam at toe. |
| 2 | 28.15-28.39 | 15 | Left | Nearly vertical bank located on outside of bend. Toe composed of large cobbles and a clay seam follows the toe. (USGS "active" bank erosion site 38). |

Table 2.5. Bank Erosion Along Cache Creek (continued).

| Reach | Location (River Mile) | Height (ft) | Bank | Comments |
|-------|--------------------------|----------------|-------|--|
| 2 | 28.64-28.81 | 15 | Left | Nearly vertical bank with toe composed of large cobbles located on outside of bend. |
| 2 | 28.80-28.85 | 8 | Right | Nearly vertical bank with toe composed of large cobbles located on outside of bend. |
| 2 | 29.02-29.16 | 15 | Left | Steep bank with toe comprised of large cobbles located on outside of bend. |
| 1 | 29.16-29.62 | 10 | Right | Steep bank located with toe composed of large cobbles located on outside of bend and a clay seam runs along toe. |
| 1 | 30.40 | 20 | Left | Steep bank with toe comprised of large cobbles. |



Figure 2.11. Bedrock control along Cache Creek, River Mile 29.2 (looking downstream).



Figure 2.12. Cache Creek near USGS gaging station above Rumsey, River Mile 30.3 (looking upstream at California Highway 16 Bridge).

channel within the Capay Valley is wide and meandering with large bars of cobbles and eroded banks. The bed material of the channel consists of cobbles and small boulders between 6 to 18 inches in diameter with significant quantities of sand and gravel. Several small tributaries carry sediment into the creek from the Capay Hills along the reach.

The banks of the channel are eroding in many areas, especially on the outside of bends. Along the toe of several eroding bank locations, a clay outcrop was observed. The banks are stratified with layers of sand, gravel, and clay in some locations. The slopes of the eroding banks are steep, almost vertical. The toe of the bank are covered with large cobbles. Erosive action causes the banks to slump into the river. Vegetation along the banks is sparse, providing little protection from erosion. The valley floor surrounding the creek is mainly irrigated agricultural land. Photographs showing typical conditions within the reach are shown in Figures 2.13 through 2.21.

2.2.2.4 Reach 3

Reach 3 extends from RM 22.5 upstream to Rumsey Bridge at RM 27.0. The channel meanders along the toe of the Capay Hills. Meanders are large and the sand-and-gravel bars show signs of many historic flow paths. Vegetation on the bars is well established, consisting of large oak trees and thick brush. Bed material consists of sand, gravel, and cobbles up to 12 inches in diameter. Several tributaries and gullies load the system with sediment from the Capay Hills, particularly in the Blue Cliff area. Layers of different materials can be seen along the banks. Bank erosion has occurred on the outside of the meander bends. Eroded banks have a layer of cobbles along the toe and some banks have outcrops of sandstone or clay. Vegetation along the banks consists of dense brush and trees. The vegetation appears to provide some protection against erosion. Photographs showing typical conditions within the reach are presented in Figures 2.22 and 2.29.

2.2.2.5 Reach 4

This reach extends from RM 18 to 22.5. It is a fairly straight channel with alternating sand-and-gravel bars of a much smaller size than those in Reach 3. The width of the active channel is smaller than the previous reach. There are several areas where the hillsides along the channel are unstable.

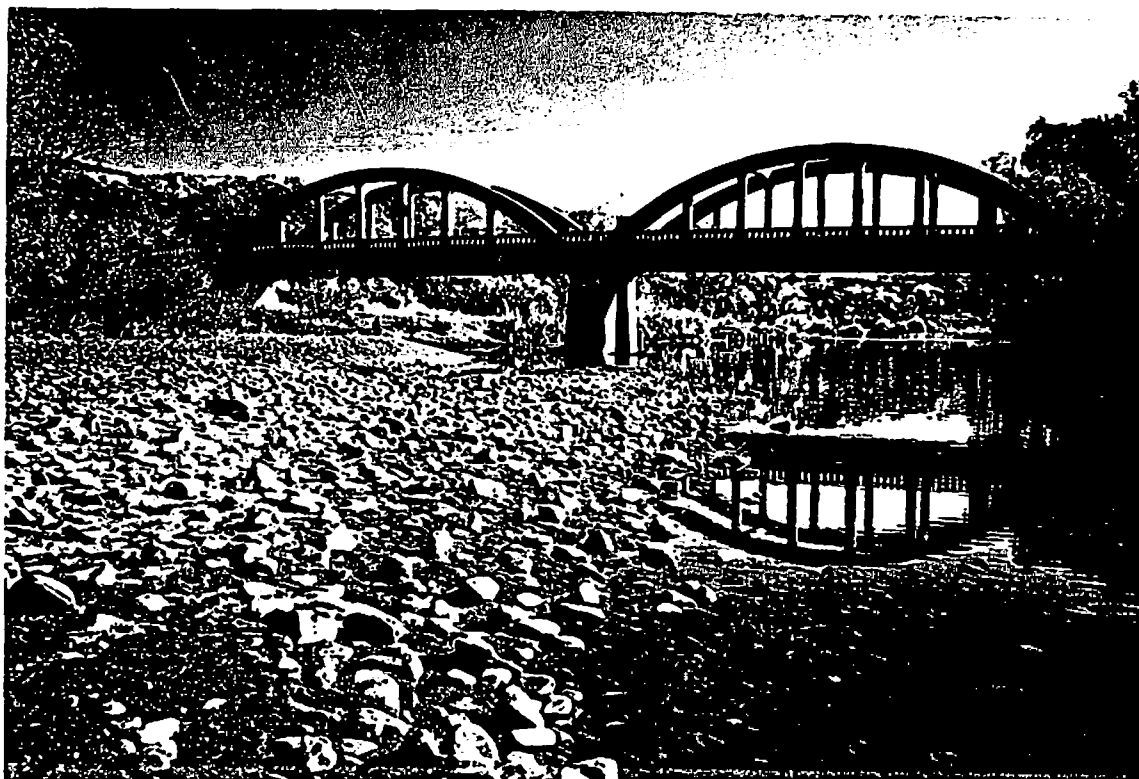


Figure 2.13. Cache Creek at Rumsey Bridge, River Mile 27.03.
(looking downstream). Location of California Department
of Water Resources gaging station.



Figure 2.14. Left Bank of Cache Creek below confluence with
Benjamin Canyon, River Mile 27.5. Approximate
location, USGS "Active Site 37."



Figure 2.15. Bed material in Benjamin Canyon, River Mile 27.5.



Figure 2.16. Clay layer at base of left bank Cache Creek, River Mile 28.3 (looking downstream). Approximate location, USGS "Active Site 38."



Figure 2.17. Sand-and-gravel bar along Cache Creek, River Mile 27.6 (looking downstream).



Figure 2.18. Left bank Cache Creek, River Mile 28.2 (looking downstream).



Figure 2.19. Left bank Cache Creek, River Mile 29.2.



Figure 2.20. Upper watershed and bank erosion along Cache Creek, River Mile 27.0 (looking upstream from Rumsey Bridge).



Figure 2.21. Cache Creek approximately one mile above Rumsey Bridge (looking downstream).



Figure 2.22. Clay outcrop along left bank Cache Creek, River Mile 22.5 (looking downstream).



Figure 2.23. Left bank Cache Creek, River Mile 22.8
(looking upstream).



Figure 2.24. Sand-and-gravel bar along Cache Creek, River Mile 23.4
(looking upstream).



Figure 2.25. Bank erosion along Cache Creek, River Mile 23.4 (looking downstream). Approximate location, USGS "Active Site 30."



Figure 2.26. "Blue Cliffs" along left bank Cache Creek, River Mile 24.8 (looking downstream).



Figure 2.27. Cache Creek, River Mile 25.1 (looking downstream).



Figure 2.28. Sandstone outcrop along left bank Cache Creek, River Mile 26.3. Approximate location, USGS "Active Site 35A."



Figure 2.29. Tributary along left bank Cache Creek, River Mile 26.7 (note mouth of tributary higher than channel bed).



Figure 2.30. Landslide along left bank Cache Creek, River Mile 18.5.

Landslides and hillside slumping were observed at RM 18.5 and 21.3. The bed material is similar to Reach 3, composed of sand, gravel, and cobbles up to 12 inches in diameter. The banks are active along the outside of bends with the toe lined with cobbles. Again, the banks are layered and some locations have clay outcrops running along the toe. At one location, broken concrete slabs were dumped along the bank for protection. Vegetation is scattered and a small amount has established itself on sandbars. Erosion noted on one sandbar has created a stairstep effect on its banks. This "terraced erosion" appears to be due to fluctuations in water level and the effect of vegetation, mainly grasses, growing on the bar. Photographs showing typical conditions within the reach are presented in Figures 2.30 through 2.36.

2.2.2.6 Reach 5

Reach 5 starts at RM 12.3 and ends at RM 18. The channel has large meanders similar to that of Reach 3. The active channel is wide, and in parts, braided. There is evidence of old meanders and historical flow paths. Surrounding hillslopes are similar in character to the previous reach, but no landslides were seen. Along the reach, portions of the channel had been recently excavated. Bed material consists of sand, gravel, and cobbles up to 6 inches in diameter. Bank erosion occurs along the outside of the bends. The toe of banks are covered with large cobbles. Banks are not as layered as in previous reaches.

Vegetation consists of scattered brush and trees, providing little bank protection. Photographs showing typical features along the reach are presented in Figures 2.37 and 2.38.

2.2.2.7 Reach 6

Reach 6 contains the Cache Creek channel from the Capay Hills near Sugarloaf Hill (RM 12.3) to the Capay Dam (RM 7.7). This reach is similar to Reach 1. The narrow valley and steep hillslopes along the reach constrict the flow. Upstream of the dam the river is wide and trapezoidal in shape with one old meander loop. Through the hills the channel is narrow. Hillside slopes are sloughing and sliding. Bed material consists of sand, gravel, and cobbles up to 4 inches in diameter. At RM 10.5 a slide has deposited large boulders, 5 to 10 feet in diameter, into the river. Upstream of the dam, the bed material in the channel changes to fine sand, apparently due to the effect of



Figure 2.31. Landslide along left bank Cache Creek, River Mile 18.7 (looking downstream).



Figure 2.32. Erosion along right bank Cache Creek, River Mile 21.9 (looking downstream).



Figure 2.33. Guinda Bridge, River Mile 20.0 (looking downstream).



Figure 2.34. Right bank Cache Creek, River Mile 20.4 (looking downstream). Approximate location, USGS "Active Site 24."



Figure 2.35. Landslide along left bank Cache Creek, River Mile 21.34.



Figure 2.36. Right bank Cache Creek, River Mile 21.7.



Figure 2.37. Excavation along left bank Cache Creek, River Mile 17.7.



Figure 2.38. Excavated sand-and-gravel bar along Cache Creek, River Mile 17.2 (looking towards left bank).

the dam on the hydraulics of the creek. The banks just upstream of the dam are similar to banks in the upper reaches with erosion occurring on the outside of bends. Through the hills the banks are more stable, except along the area where the slide occurred. Vegetation along banks consists of grass, brush, and trees, providing some protection of the banks. In some locations, erosion has terraced the banks in a stairstep manner. Photographs depicting typical conditions along the reach are shown in Figures 2.39 through 2.43.

2.2.2.8 Reach 7

Reach 7 starts at Capay Bridge (RM 5.6) and ends at the Capay Dam (RM 7.7). This reach is straight with alternating sand-and-gravel bars. The channel is approximately trapezoidal in shape and is fairly stable. Bed material consists of sands and gravels up to 2 inches in diameter. Near the Capay Dam, bedrock is exposed along its downstream toe. Banks are well vegetated with grass, thick brush, and trees. Small amounts of erosion were observed at locations downstream of the dam and upstream of the Capay Bridge. Figure 2.44 is an aerial view of the channel at the Capay Bridge.

2.2.2.9 Reach 8

Reach 8 extends from Esparto Bridge (RM 3.5) to Capay Bridge (RM 5.6). The channel within this reach is very wide due to the effects of gravel extraction operations. This reach was extensively excavated during the summer of 1986. Approximately 300 feet below Capay Bridge, the bed of the channel has been lowered considerably. It was estimated that approximately 5 to 10 feet of material was removed from the bed of the channel. A large stockpile of the excavated material has been placed along the left overbank. As part of the excavation, numerous bank protection measures were constructed. Protection measures observed included levees, transverse dikes, and concrete slab bank protection. Bed material consists of sand and gravel up to 2 inches in diameter. Banks unaffected by the recent excavation show signs of erosion similar to upstream reaches. Vegetation along banks is generally sparse consisting of mainly low brush and grass. Banks affected by the excavation were bare of vegetation. Typical photographs of the channel are shown in Figures 2.45 through 2.47.



Figure 2.39. Capay Dam, River Mile 7.7 (looking upstream).



Figure 2.40. Bed material along Cache Creek upstream of Capay Dam (looking downstream).

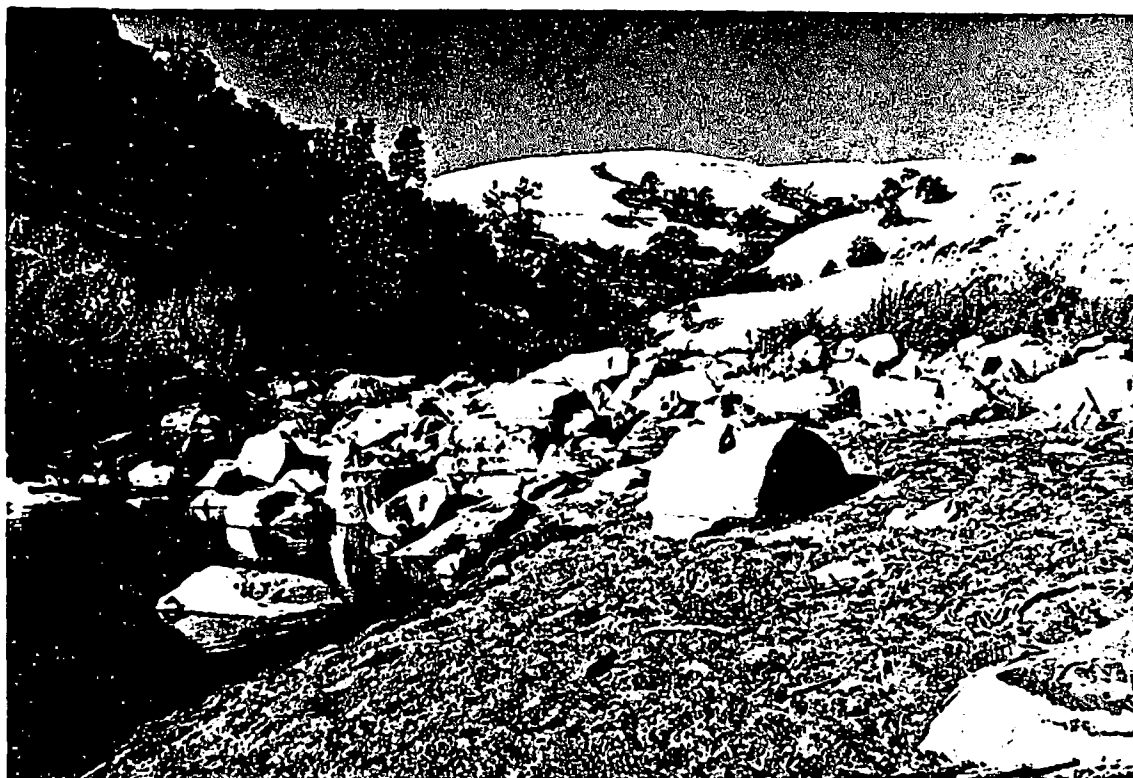


Figure 2.41. Landslide along Cache Creek, River Mile 10.6 (looking upstream).



Figure 2.42. Landslide and hillside slumping along Cache Creek, River Mile 10.6 (looking downstream).



Figure 2.43. USGS gaging station at Brooks along right bank Cache Creek, River Mile 11.6.



Figure 2.44. Capay Bridge, River Mile 5.62 (looking upstream).



Figure 2.45. Cache Creek at Capay Bridge (looking upstream).



Figure 2.46. Cache Creek downstream of Capay Bridge (looking downstream).



Figure 2.47. Bank protection along Cache Creek between Esparto Bridge and Capay Bridge (looking south).



Figure 2.48. Cache Creek downstream of Esparto Bridge (Esparto Bridge in foreground).

2.2.2.10 Reach 9

This reach extends from RM 0.6 to Esparto Bridge (RM 3.5). Channel characteristics are similar to Reach 8. The channel is very wide. Significant gravel mining activity is occurring in this reach. The low-flow channel in this reach is very straight. Low levees and transverse dikes have been constructed to contain the low-flow channel. The high-flow banks of the reach show the meander pattern of the historic form of the creek. Material in the channel consists of sand and gravel up to 2 inches in diameter. The banks are eroding similar to upstream reaches. Vegetation is absent within the channel due to mining activity. Vegetation along high-flow banks consists of grass and brush. Typical photographs of the channel in this area are shown in Figures 2.48 and 2.49.

2.2.2.11 Reach 10

Reach 10 extends from I-505 upstream to RM 0.6. This reach is presently undisturbed by man's activities. The effects of past gravel mining activities are, however, apparent. Vegetation is seen to be reestablishing itself on bars within the channel. It is narrower than the two previous reaches, but wide compared to reaches further upstream. The bed material is sand and gravel up to 2 inches in diameter. Vegetation along banks consists of grasses and provides little bank protection. A typical view of the channel in this reach is shown in Figure 2.50.

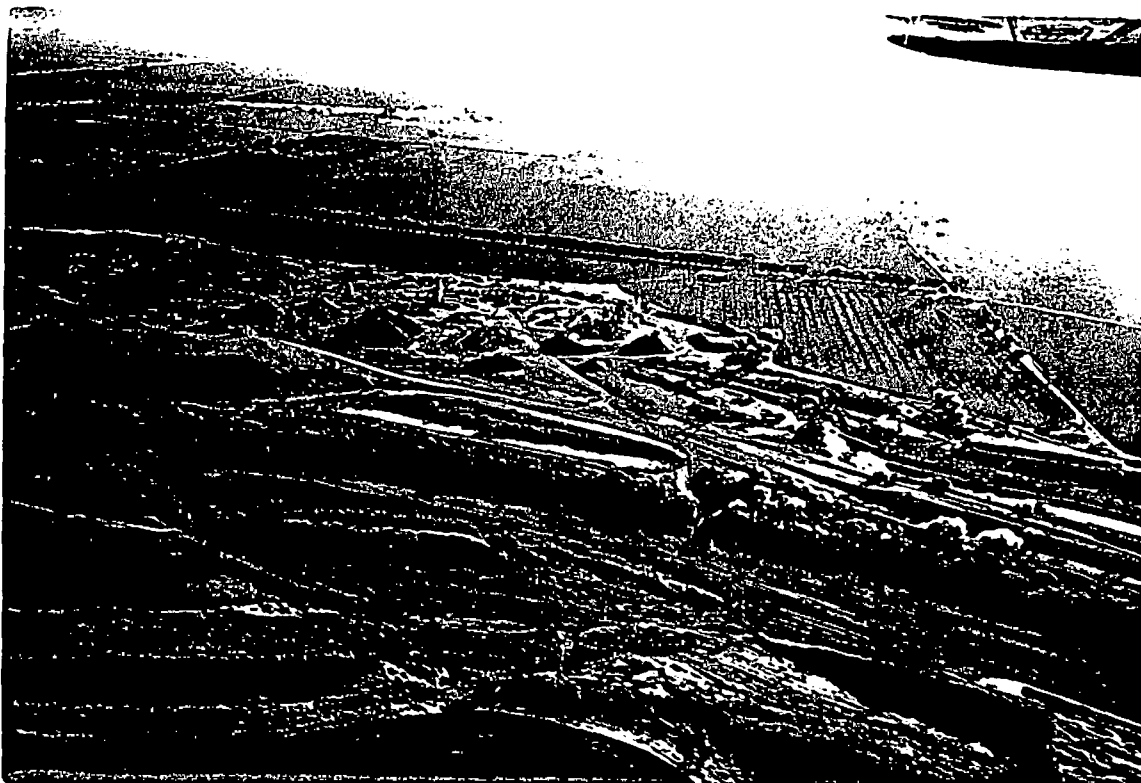


Figure 2.49. Gravel mining operation along right bank Cache Creek upstream of I-505.



Figure 2.50. I-505 Bridge, River Mile 0.0 (looking downstream).

III. HYDROLOGY

Cache Creek, a tributary to the Sacramento River, is a seasonal stream which runs dry much of the summer and fall and experiences heavy flows in the winter and spring. Flows range from 0 to over 40,000 cubic feet per second (cfs) at the Yolo gage. The drainage area above Yolo is 1,139 square miles. According to the COE (1983a), the drainage area above Capay is 1,044 square miles, of which 55 percent empties into Clear Lake and 13 percent drains into the Indian Valley Reservoir. Peak flows at Capay and Yolo are influenced by these two reservoirs.

About 85 percent of the precipitation occurs from November to March. Normal annual precipitation varies from a maximum of more than 60 inches in the upper portion of the watershed to a minimum of about 17 inches near Yolo and averages about 32 inches over the entire watershed. A high percentage of the rainfall that occurs in the mountainous areas of the watershed results in direct runoff due to the nature of the soils and the steepness of the topography.

The variation in precipitation throughout the drainage basin is an important factor in the production of sediment and sediment yield from the basin. As precipitation decreases with a decrease in elevation, it is accompanied by a general decrease in the abundance of vegetation. This results in an increase in sediment yield.

Annual runoff in the drainage basin fluctuates over a wide range. Streamflow data are available from the U.S. Geological Survey (USGS) and the California Department of Water Resources (DWR) for streams in the drainage basin. Average annual runoff measured at some of the major gaging stations for the period of record is tabulated in Table 3.1. According to Woodward-Clyde Consultants (1976), Cache Creek has an average discharge of 516 cfs (379,600 acre-feet per year) based on 70 years of record at the Yolo gaging station.

Throughout the recorded hydrologic history of Cache Creek, many floods have occurred. The most notable floods of recent years are listed in Table 3.2. The 1958 and 1983 floods are the two largest floods on record according to the Corps of Engineers.

The COE (1985b) prepared rain-flood frequency curves from historical data for Cache Creek above Rumsey and for Cache Creek near Capay. The curves are presented in Figures 3.1 and 3.2. They do not reflect the effects of Indian

Table 3.1. Stream Gaging Stations.

| Location | Drainage Area (sq. mi.) | Period of Record Used | Length of Record (years) | Average Annual Runoff (acre-ft) | Station Operator |
|---|-------------------------|-----------------------|--------------------------|---------------------------------|------------------|
| Clear Lake at Lakeport | 528 | 1913-1984 | 72 | 5.00 ¹ | USGS |
| Cache Creek near Lower Lake | 528 | 1944-1984 | 41 | 235,600 | USGS |
| North Fork Cache Creek at Hough Springs Near Loner Lake | 60.2 | 1972-1984 | 13 | 170,000 | USGS |
| North Fork Cache Creek near Lower Lake | 197.0 | 1930-1981 | 52 | 136,500 | USGS |
| Bear Creek Near Rumsey | 100 | 1958-1980 | 23 | 35,760 | DWR, Calif. |
| Cache Creek Above Rumsey | 955.0 | 1965-1984 | 20 | 549,400 | DWR, Calif. |
| Cache Creek Near Capay | 1,044.0 | 1942-1976 | 35 | 556,900 | USGS |
| Cache Creek At Yolo | 1,139.0 | 1903-1981 | 69 | 379,600 | USGS |

¹ Average annual lake stage in feet above datum of gage, 1,318.65 ft.

Table 3.2. Notable Floods of Recent Years.¹

| Gaging Station | Flood Date | Peak Flow (cfs) | Three-Day Volume (acre-ft) |
|--|------------|---------------------|----------------------------|
| Cache Creek Near Lower Lake | 02/24/58 | 8,000 | 30,550 |
| | 12/22/64 | -- | -- |
| | 01/05/65 | 5,320 | 23,270 |
| | 01/23/70 | 6,320 | 26,620 |
| | 02/14/80 | 5,290 | -- |
| | 03/01/83 | 6,070 | -- |
| North Fork Cache Creek Near Lower Lake | 02/24/58 | 13,500 | 31,860 |
| | 12/22/64 | 19,700 | 61,800 |
| | 01/05/65 | 15,700 | 40,060 |
| | 01/23/70 | 16,000 | 37,410 |
| Bear Creek Near Rumsey | 12/22/64 | 6,820 | 10,680 |
| | 01/05/65 | 9,720 | 12,710 |
| | 01/23/70 | 5,900 | 10,400 |
| | 02/19/80 | 6,820 | -- |
| Cache Creek Above Rumsey | 01/05/65 | 59,000 ² | -- |
| | 01/24/70 | 43,400 ² | 99,970 |
| Cache Creek At Rumsey | 01/26/83 | 53,500 | 84,500 |
| Cache Creek Near Capay | 02/24/58 | 51,600 | 98,980 |
| | 12/23/64 | 32,400 | 84,350 |
| | 01/05/65 | 44,500 | 96,620 |
| | 01/24/70 | 36,200 | 92,230 |
| Cache Creek At Yolo | 02/25/58 | 41,400 | 102,230 |
| | 12/23/64 | 26,200 | 79,360 |
| | 01/06/65 | 37,800 | 97,420 |
| | 01/24/70 | 34,600 | 97,730 |
| | 01/27/83 | 44,560 | 125,170 |

¹ Data from U.S. Army Corps of Engineers (1979) and Corps of Engineers (1985 personal communication).

² Value appears unreasonably high, possibly due to the extension of the low-flow rating curve and slope-area measurements.

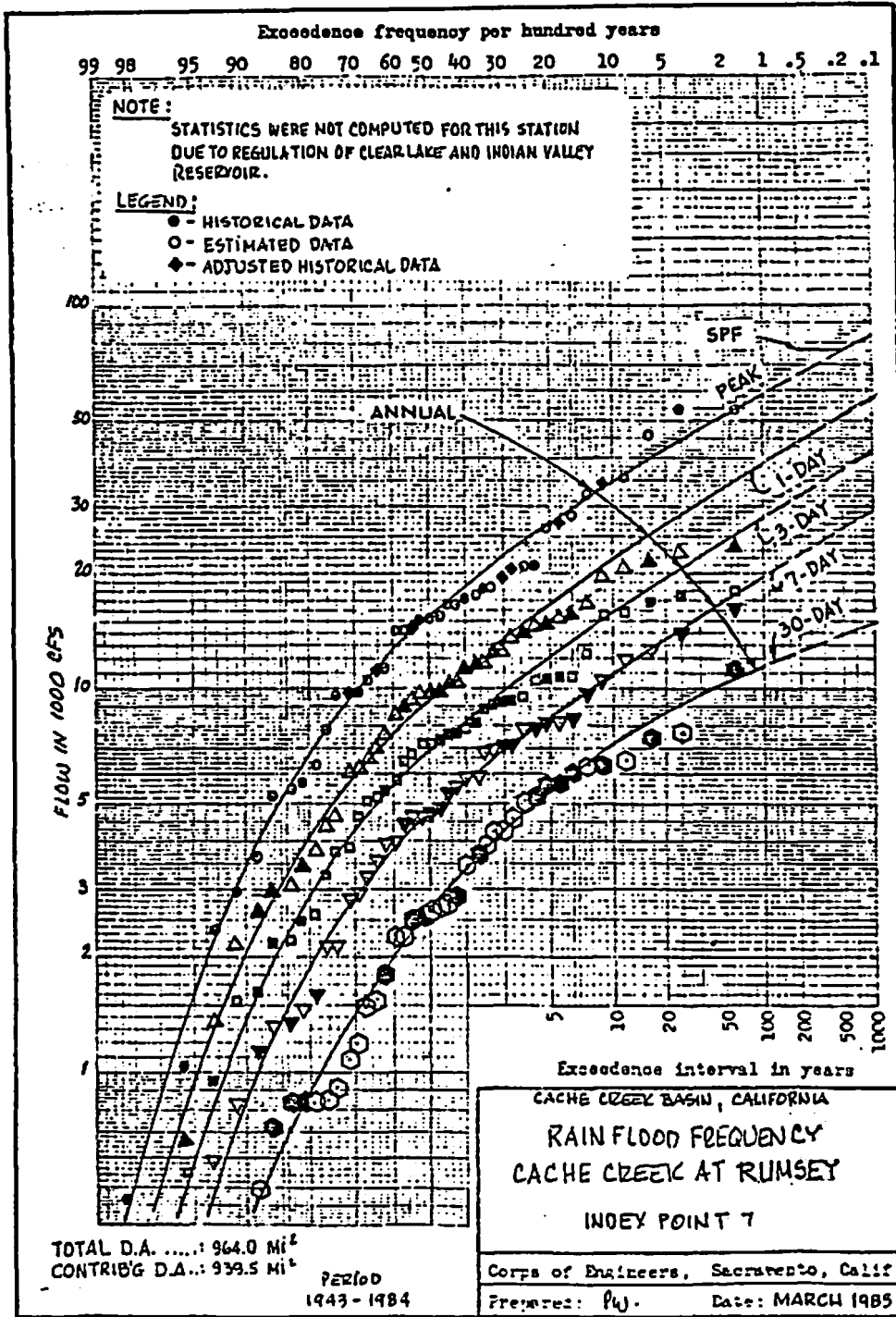


Figure 3.1. Flood frequency curves for Cache Creek at Rumsey (USCOE, 1985b).

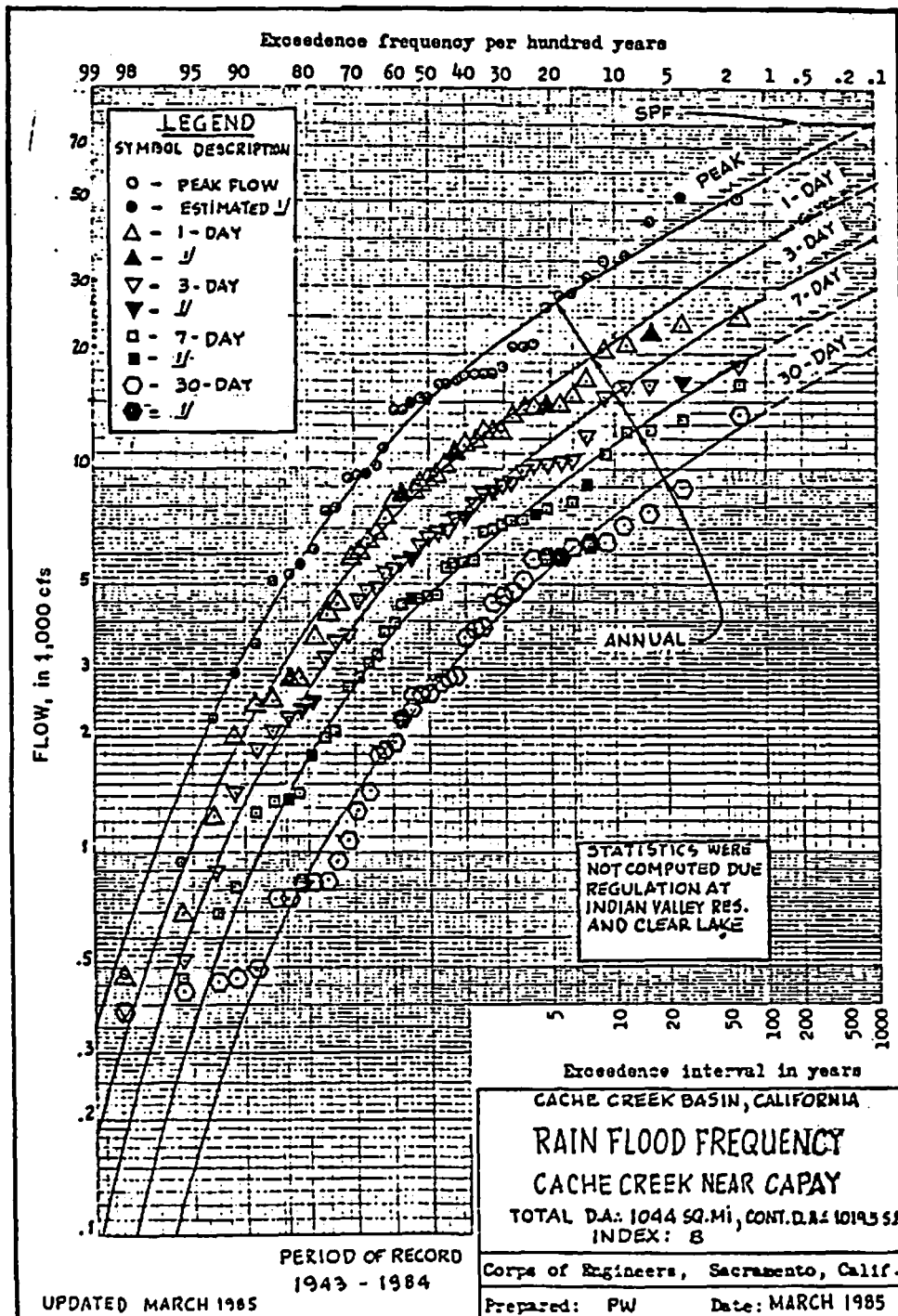


Figure 3.2. Flood frequency curves for Cache Creek near Capay (USCOE, 1985b).

Valley Reservoir. Peak flow frequency curves were also prepared by the COE, which do reflect the influence of Indian Valley Reservoir operation, for existing (preproject) operating conditions of Clear Lake Dam and for the approved operation alternative for the dam (Case A) that is being considered as part of the Clear Lake flood-control project (USCOE, 1985c).

The Case A alternative involves operating Clear Lake Dam by the present outlet channel capacity (Figure 3.3) as long as the Clear Lake stage remains below 7.56 feet. Once Clear Lake stage rises above 7.56 feet, releases are based on the proposed 8,000 cfs outlet channel capacity curve (Figure 3.3), as long as flows at Rumsey do not exceed 20,000 cfs. If flows at Rumsey exceed 20,000 cfs, release from Clear Lake Dam are based on the present channel capacity curve. When Clear Lake stage is reduced back to 7.56 feet, or below, the release rate reverts back to the present channel capacity curve.

Preproject (existing) and postproject (Case A) peak flow frequency curves for Cache Creek above Rumsey, which do reflect the influence of Indian Valley Reservoir operation are presented on Figure 3.4. These curves indicate an increase in the peak discharge for flow frequencies between approximately 1.5 and 4 years. Peak flow data for various locations and return periods are presented in Table 3.3.

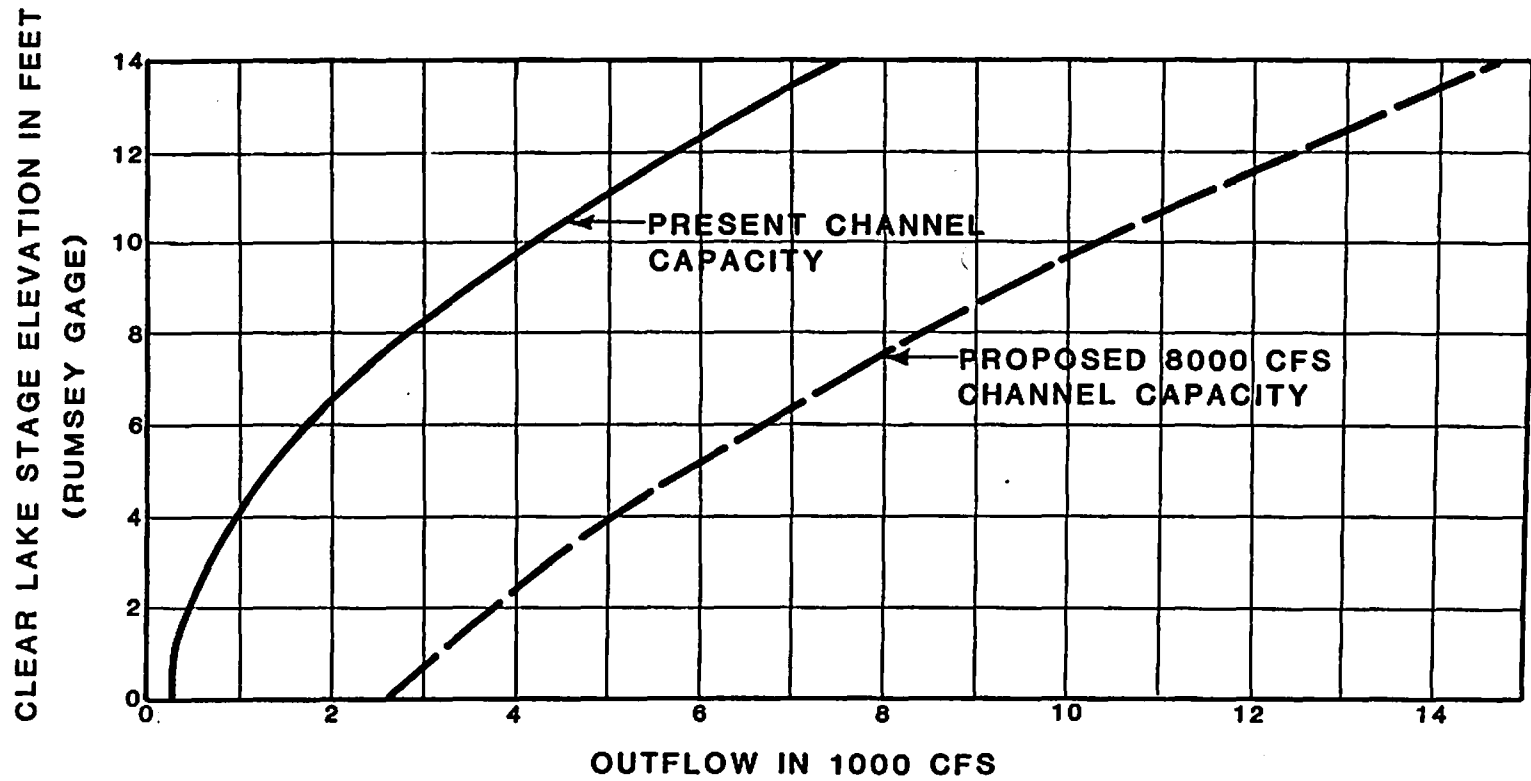


Figure 3.3. Clear Lake outlet channel rating curves.

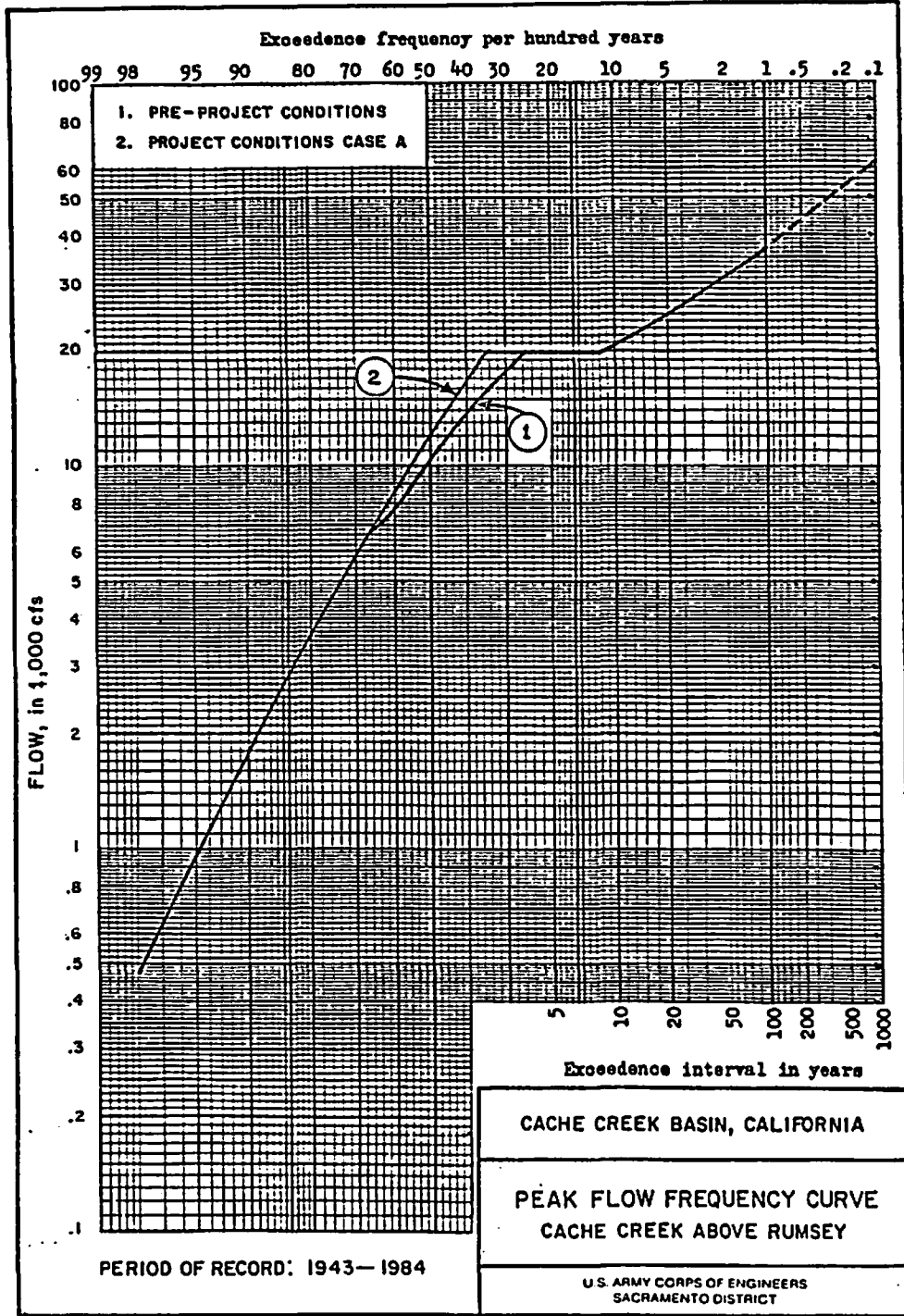


Figure 3.4. Preproject (existing) and postproject (Case A) frequency curves for Cache Creek above Rumsey (USCOE, 1985b).

Table 3.3. Peak Discharge of Various Return Period Along Cache Creek
(USCOE, 1985b).

| Location | Case | Return Period | | | | | |
|---------------------------|--|-----------------|-----------------|------------------|------------------|------------------|-------------------|
| | | 2-year (cfs) | 5-year (cfs) | 10-year (cfs) | 25-year (cfs) | 50-year (cfs) | 100-year (cfs) |
| Cache Creek at Rumsey | Historic Without Influence of Indian Valley Reservoir | 15,500 | 27,000 | 35,000 | 44,500 | 52,000 | 58,500 |
| | Historic With Influence of Indian Valley Reservoir | 10,500 | 20,000 | 20,500 | 25,500 | 31,500 | 37,000 |
| | Project Case A | 12,000 | 20,000 | 20,500 | 25,500 | 31,500 | 37,000 |
| Cache Creek Near Capay | Historic Without Influence of Indian Valley Reservoir | 15,000 | 26,500 | 34,000 | 43,500 | 51,000 | 58,000 |

IV. GEOMORPHIC ANALYSIS

4.1 General

In Phase I of this study, a qualitative geomorphic analysis of Cache Creek in the study area was conducted to identify the general sediment-transport characteristics of the channel and define problems related to erosion and sedimentation. The importance of a geomorphic approach is its ability to identify the general tendencies of the stream system and assess the response of the system to change. This will allow effective planning of improvements to the channel to alleviate problems of flooding, bed and bank erosion, and sedimentation. The geomorphic approach also provides a check of the reasonableness of results from quantitative analyses of the system.

A geomorphic analysis utilizes several sources of information. These sources include site visits, aerial photographs, geological and soils information, land-use information, bed- and bank-size distributions, and hydraulic properties. This chapter describes how these sources of information are utilized in defining the geomorphic characteristics of Cache Creek in the study area.

4.2 Geology

The Cache Creek drainage basin lies within the Coast Ranges of northern California, northwest of Sacramento. The ranges are structurally controlled, with major topographic and structural features trending approximately N30W. The structural controls are reflected in the shape and orientation of the drainage basin. Figure 4.1 shows the major geologic features of the drainage basin.

The Franciscan Assemblage of Mesozoic age forms the core of much of the Coast Ranges and outcrops in the upper part of the Cache Creek basin. The formation contains a heterogeneous assemblage of clastic marine sedimentary rocks, mafic volcanic rocks, and mafic and ultramafic intrusive rocks that show varying degrees of metamorphism. Also included in lesser abundance are chert and limestone. Other Mesozoic rocks that form much of the upper and middle parts of the basin are serpentized ultramafic rocks, shales, sandstones, and conglomerates.

The study reach of Cache Creek begins in a hilly region just upstream of Rumsey in the middle part of the basin. This area contains a sequence of Upper Cretaceous marine formations, consisting primarily of sandstone units.

From Rumsey, the stream flows through the Capay Valley consisting of Quaternary alluvium approximately 1 to 2 miles wide. Areas of Pliocene and Pleistocene continental deposits outcrop along the valley walls. These deposits consist of silt-clay, sand, and gravel and occur as discrete units and heterogeneous mixtures. The deposits are dissected by gulleys and rills to form a badlands-type topography which contribute large amounts of sediment. Several tributaries drain Upper Cretaceous and Tertiary sedimentary rocks on the west side of the valley.

Below the Capay Valley, Cache Creek flows through an area of Upper Cretaceous marine sedimentary rocks. Surficial slumps and associated gulleys are a significant source of sediment in this area. From here the creek flows out onto a broad alluvial plain with scattered low gravel hills. The plain contains a large alluvial fan with a high gravel content, over which Cache Creek meandered before the settling basin and other controls were established.

4.3 Soils, Land Use and Watershed Characteristics

Watershed and climatic characteristics throughout the Cache Creek basin have a significant influence on the sediment yield to the study reach. According to the Cache Creek Basin Report by the SPK, 1983, the sediment yield in the Capay Valley between Rumsey and Capay is approximately 3.5 times higher than upstream portions of the basin. Average annual suspended load in this area is approximately 1,600 tons per square mile. A variety of factors contribute to this condition.

Precipitation in the upper part of the basin is significantly higher than the lower basin due to the orographic-type storms which occur in this area. Because of the higher precipitation, vegetation on the hillslopes and valley bottoms is relatively dense, limiting the tendency for erosion of the overland areas. In addition, roughly 68 percent of the watershed area upstream of Capay drains into either Clear Lake or Indian Valley Reservoir. These lakes effectively trap all of the sediment delivered from upstream areas.

The drainage area of the North Fork Cache Creek below Indian Valley Reservoir and the main fork for a short distance upstream of the confluence are characterized by relatively heavy vegetation, narrow, deep valleys, and steep channel gradients. Soils in the watershed are primarily silt-clay layers, sands, and gravels. Steep bluffs in the area contribute to a badland-type topography which is subject to intense erosion and resulting high sedi-

ment yield. Drainage areas of Bear Creek above the confluence with the mainstem of Cache Creek are composed of steep sedimentary outcrops which also produce a relatively high sediment yield.

The middle portions of the watershed (between Rumsey and Capay) exhibit topography and soils conditions similar to the upper watershed, but are subject to significantly less precipitation. The resulting lack of vegetation coupled with steep topography and high silt-clay content of the soils result in an increased sediment yield in this area. Additionally, cultivation of the valley floor for agricultural purposes is more intense in this area which also contributes to the increased erosion potential. Soils in the valley bottoms are generally silt loams to silty clay loams on alluvial fans explaining the tendency of the channel to contain coarser alluvial material in the bed with finer, erodible banks.

In general, the decrease in precipitation in the downstream direction results in a decrease in vegetation in both the upland areas and valley bottoms. Coupled with the relatively erodible nature of the watershed soils, this results in a tendency for significant increase in sediment yield in the lower portions of the study area. Lack of vegetation along the valley bottoms and channel banks also contribute to a greater tendency for bank erosion in this area.

An additional factor which may contribute to the relatively high sediment yield from the basin is the impact of forest fires. According to U.S. Forest Service data, forested areas in this general location are on an approximately 25-year burn cycle. As shown by Tatum (COE, 1963) sediment and debris production from watershed areas can be as much as 25 times higher immediately after a burn than for the normal vegetated conditions. High sediment yield from burned areas as well as a general tendency for landslides and soil creep in steep areas may introduce large quantities of sediment to the system which may temporarily overload the stream channel resulting in unusual deposition patterns, bank erosion, and convulsions of the channel.

4.4 Bed- and Bank-Size Distributions

Bed and bank sediment samples were collected along Cache Creek by the USGS in 1984 and 1985. Bed samples were taken at intervals approximately one-half to 1 mile apart with bank samples taken less frequently. At locations where the gradation of material varied across the channel, several samples

were taken. Gradation curves were developed for these samples to characterize the material forming the bed and banks along the study reach. Gradations were determined by sieve analysis for the majority of samples. In some locations, however, the coarseness of the material required gradations to be determined by hand count (Wolman method).

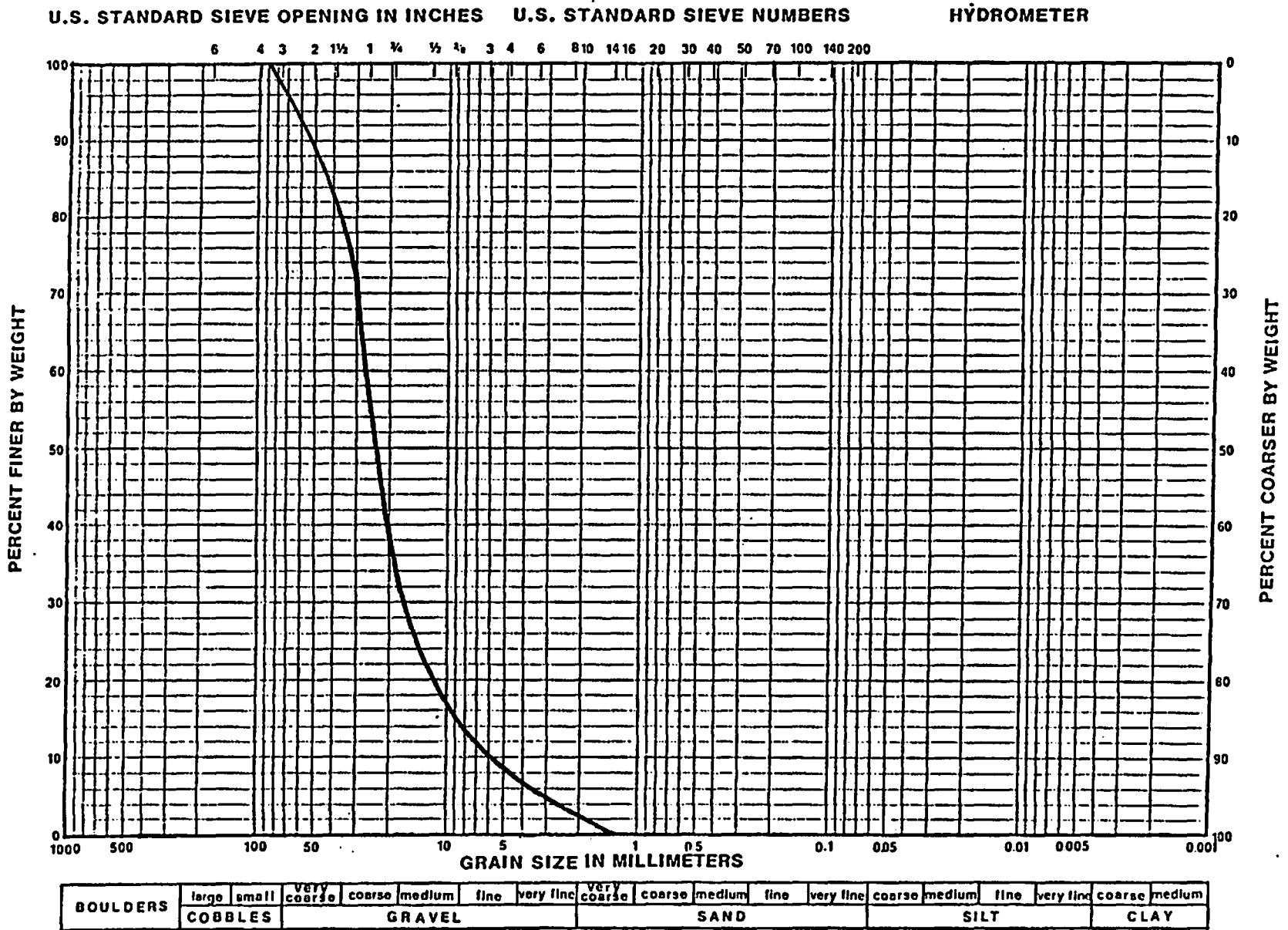
Figure 4.2 illustrates the variation in size of the bed material collected along the study reach. The values plotted in Figure 4.2 are from composite size distributions developed for each sampling location. Composite distributions were developed by averaging the gradations of all samples collected. Both sieve gradations and hand count gradations were included. The figure shows for each sample the sizes at which 10, 50, and 90 percent of the material is finer (D_{10} , D_{50} , and D_{90}). Considerable scatter exist in the data with a detectable increase in sizes in the upstream direction. Figure 4.3 gives an average size distribution for the material downstream of Capay Dam (RM 0.0 to 7.7). The material consists mostly of gravels. Figure 4.4 gives the size distribution of material collected in the pool above Capay Dam (RM 7.7). This material is much finer and reflects the potential for finer material to settle out behind the dam. Figure 4.5 gives an average size distribution for material upstream of the pool of Capay Dam (RM 7.7 to 27.0). The material contains more cobble-size material than that below the dam. Figure 4.6 shows an average size distribution for Cache Creek above Rumsey Bridge, Reach 2 (RM 27.0 to 29.2). This area contains considerably larger material. Figure 4.7 shows an average size distribution near the upstream study limit (RM 29.2 to 30.5). This area has a higher percent of coarse material than Reach 2. The plotted bed-material size distributions agree with field observations (See Chapter II).

Bank samples collected along the study reach showed no detectable trends in size distributions. The material generally contained fine to medium sand with 10 to 20 percent silts and clays. Figure 4.8 gives a representative average size distribution for the collected bank samples. Some samples contained layers considerably larger material but were not found to be representative of any trends in the data.

4.5 Aerial Photograph Analysis

4.5.1 General

Comparison of historical aerial photographs is useful in identifying changes in the topwidths of channels and flood plains, locations of bank ero-



4.7

Figure 4.3. Representative bed-size distribution, Cache Creek, downstream of Capay Dam (RM 0.0 to 7.7).

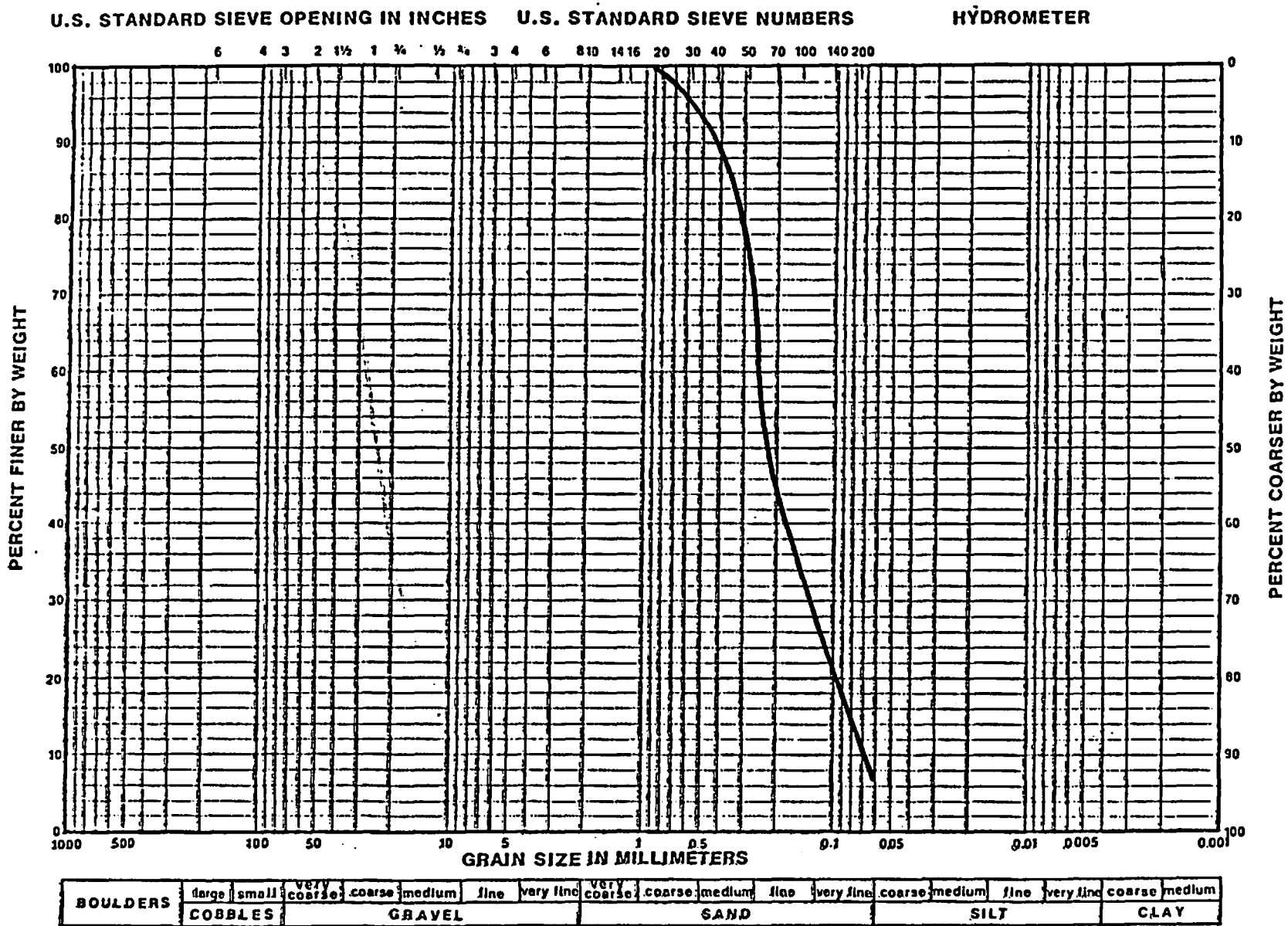


Figure 4.4. Composite bed-size distribution for cross section just upstream of Capay Dam (RM 7.7).

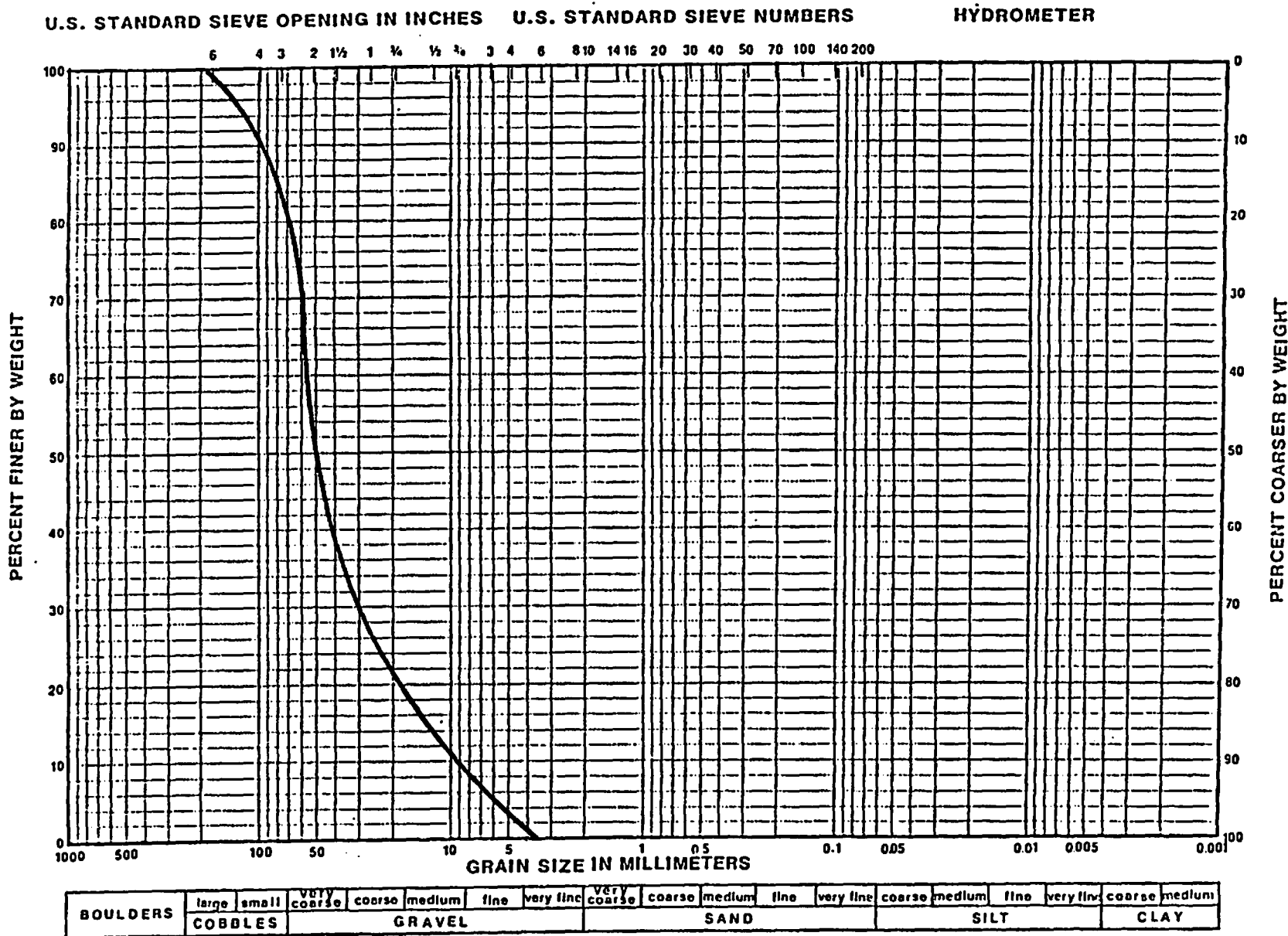


Figure 4.5. Representative bed-size distribution, Cache Creek, upstream of Capay Dam (RM 7.7 to 27.0).

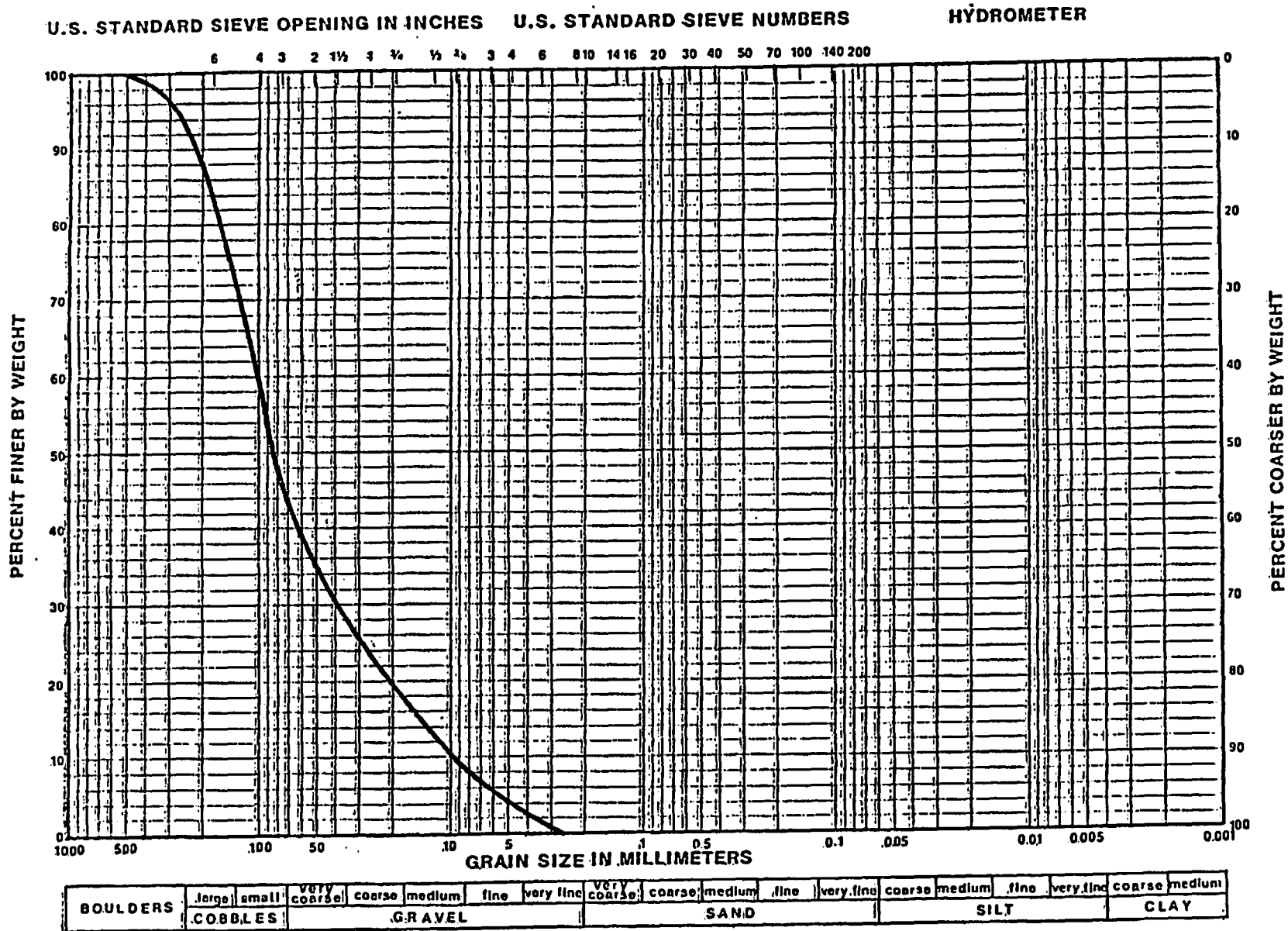


Figure 4.6. Representative bed-size distribution for Reach 2, Cache Creek (RM 27.0 to 29.2).

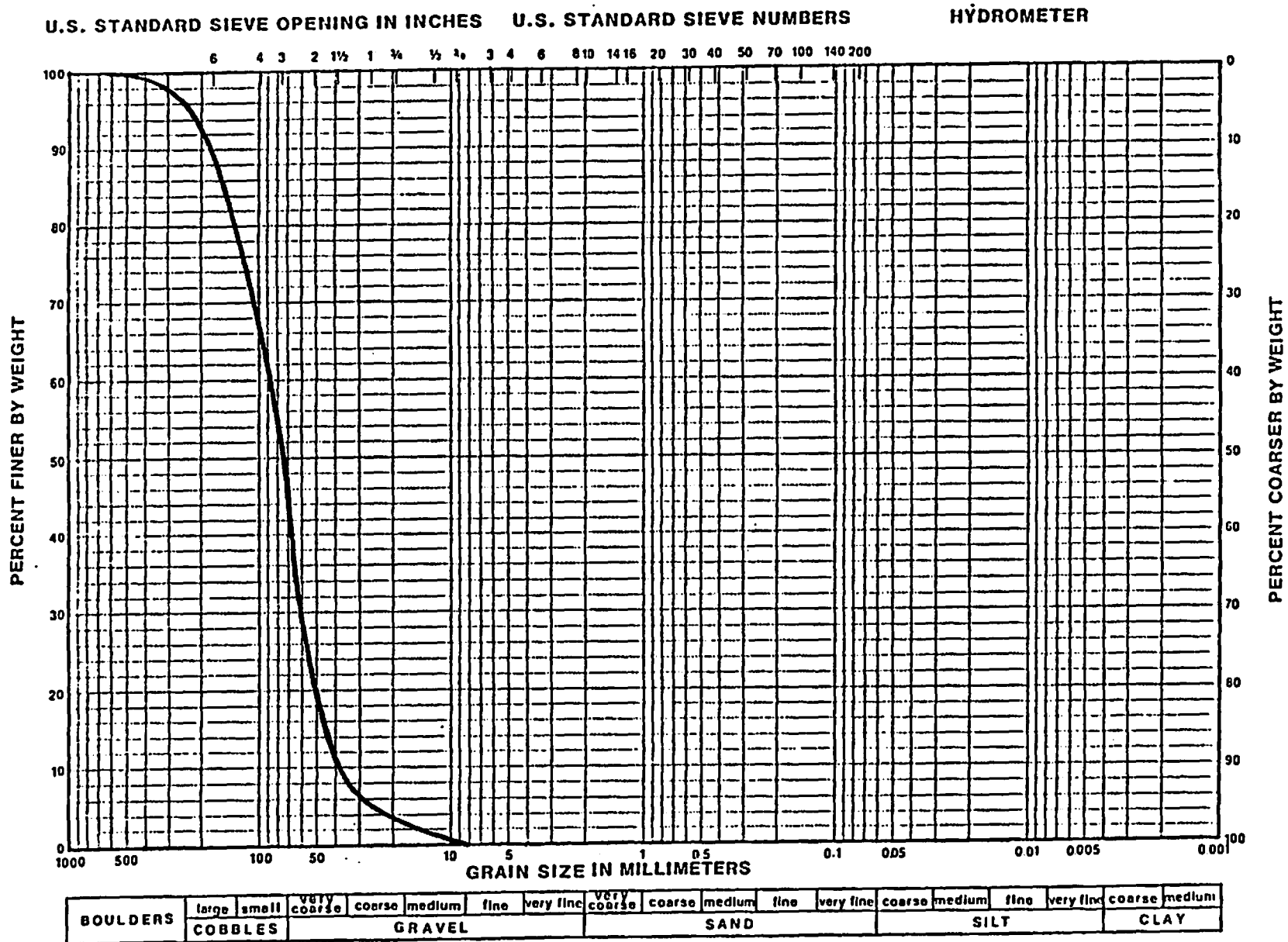


Figure 4.7. Representative bed-size distribution, Cache Creek at upstream end of study reach (RM 29.2 to 30.5).

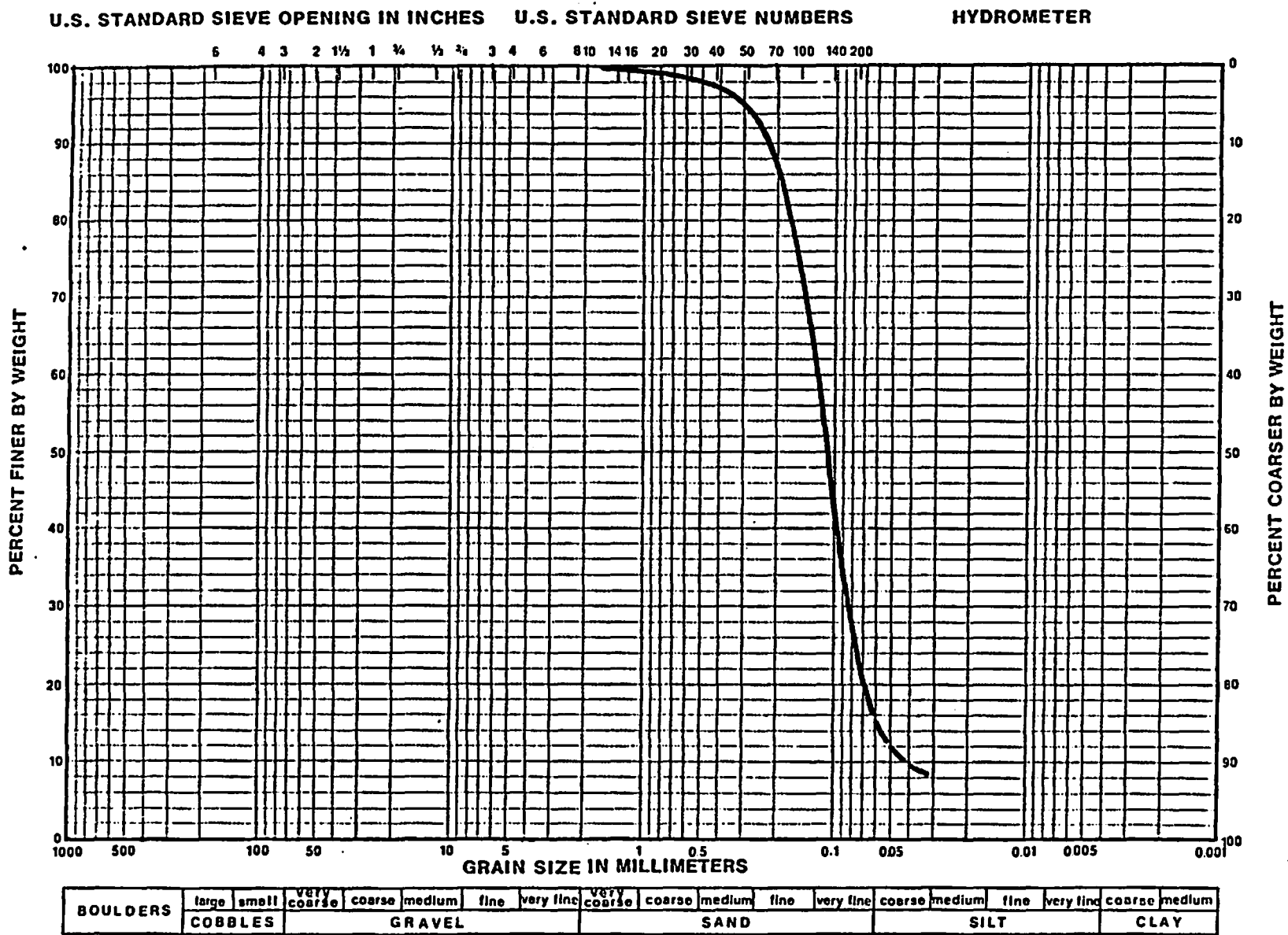


Figure 4.8. Representative bank-size distribution.

sion, man-made influences, and channel form. Identification of such changes assist in understanding the system's behavior. A large number of aerial photographs of Cache Creek within the study area are available. Years for which photos were found included 1939, 1969, 1972, 1974, 1979, 1983, 1984, 1985, and 1986.

Differences in scale, extent of coverage, seasonal change and variations in discharge, made comparison difficult. Only large changes in the stream were readily observable. In the following sections, the various observations made along each reach through the study area are summarized. Additionally, quantification of the areas of land affected by bank erosion was also attempted.

4.5.2 Aerial Photograph Comparison

Aerial photographs of the study area were available for a period covering 47 years (1939 to 1986). Significant changes to the channel have occurred during this period. Changes noted included an overall narrowing of the active channel downstream of the Capay Bridge, shifts in bank alignments throughout the study reach, and gravel mining. In the following sections, descriptions of observations made from the aerial photographs for each reach in the study area are presented.

4.5.2.1 Reach 1 (River Miles 29.2 to 30.5)

The channel in this reach is confined to a narrow valley between steep hillslopes. The channel appears to be controlled by bedrock and has a riffle-pool-type morphology. Measured 1986 active channel topwidths are approximately 100 to 150 feet and appear relatively uniform. The channel appears very stable with no significant changes detectable in the photographs.

4.5.2.2 Reach 2 (River Miles 27.0 to 29.2)

The channel in this reach flows through the upper part of the alluvial Capay Valley. The channel is meandering with alternate bars. Measured active channel topwidths vary between approximately 150 to 200 feet. There is some evidence of bank erosion, but no significant changes were observed over most of the reach. Measurable bank erosion (50 feet in 3 years) was observed at the COE's active site at RM 27.5. Some evidence of old channel shifting could be observed in flood plain features.

4.5.2.3 Reach 3 (River Miles 22.5 to 27.0)

The channel in this reach flows along the toe of the Capay Hills. The Blue Cliffs are located in this reach. The channel is sinuous with numerous alternating bars and islands. Measured topwidths of the active channel vary between 150 and 1,400 feet. The channel appears more active than Reach 2.

4.5.2.4 Reach 4 (River Miles 18.0 to 22.5)

This reach is fairly straight and narrow. The active flood channel is sinuous with relatively long meanders of small amplitude. One large meander exists downstream of the Guinda bridge. Agricultural fields border the right bank of the stream throughout the reach. Measured topwidths of the active channel vary between 150 and 200 feet. Widths of the apparent flood plain vary from 300 feet over most of the reach to 1,250 feet in the vicinity of the large meander downstream of Guinda bridge.

4.5.2.5 Reach 5 (River Miles 12.3 to 18.0)

Reach 5 is also relatively straight and narrow. The active channel is quite sinuous. Active channel topwidths vary between 200 and 750 feet. Numerous minor shifts in the location of the low-flow channel of the creek were also observed. Active bank erosion and shifting of bars is evident throughout the reach. In the vicinity of RM 16, a meander has cutoff between the 1972 and 1974 aerial photographs. This is the only meander cutoff observed in the study reach.

4.5.2.6 Reach 6 (River Miles 7.7 to 12.3)

The channel in this reach is confined by steep hillslopes as the creek flows around Sugarloaf Hill. Channel widths were measured to be approximately 100 feet. Downstream of Sugarloaf Hill the channel widens to an approximately 500-foot width. The channel is relatively stable, no significant changes were noted throughout the series of available photographs.

4.5.2.7 Reach 7 (River Miles 5.6 to 7.7)

Reach 7 is located downstream of Capay dam. The channel in this area is relatively straight and lined with trees and low vegetation. The apparent flood plain in this area varies from 800 to 1,600 feet. The active channel width is approximately 750 feet. The channel in this reach appears relatively stable; no significant changes were detectable between 1939 and 1986.

4.5.2.8 Reach 8 (River Miles 3.5 to 5.6)

The channel downstream of the Capay bridge is significantly different than any of the reaches observed upstream. The channel is considerably wider and shows significant deposition patterns. In 1939, the channel had very little vegetation and widths of up to 2,500 feet. The channel had a distinct braided pattern. The apparent flood plain in the area is up to 4,000 feet in width. No significant change was noted in this pattern between 1939 and 1986 aerial photographs.

Field observations of the reach (December 1986) showed significant alteration of the channel caused by gravel extraction operations. The channel bed in this area had been lowered by approximately 10 feet since the time of the June 1986 aerial photographs; a defined straight low-flow channel had been excavated and numerous types of bank protection measures had been constructed.

4.5.2.9 Reach 9 (River Miles 0.6 to 3.5)

In 1939, Reach 9 was seen as a wide, braided channel with a sinuous low-flow path. At its widest, the apparent flood plain along the reach was measured to be 2,500 feet. Towards its downstream limit, the reach narrows and vegetation increases. At the downstream limit of the reach the flood plain was measured to be approximately 1,500 feet in width.

The characteristics of the channel first observed in 1939 did not significantly change through the 1969 aerial photographs. In 1972, however, a gravel mining operation was established along the reach. The effects of gravel mining observed included removal of numerous braids, the construction of haul roads, and the establishment of an aggregate processing facility along the right bank near the downstream reach limit. Gravel mining effects were limited to the downstream one-half of the reach. Successive sets of photographs (1972 to 1986) showed these effects to be continuing.

The 1986 aerial photographs showed gravel mining effects to still be limited to the lower one-half of Reach 9. Field observations (December 1986), however, revealed that the gravel mining activities had been extended upstream throughout Reaches 8 and 9. A large trapezoidal low-flow channel had been excavated throughout these reaches to approximately 300 feet downstream of the

Capay bridge. The channel bed appeared to have been lowered by as much as 10 feet.

4.5.2.10 Reach 10 (River Miles 0.0 to 0.6)

Reach 10 is a short reach at the downstream limit of the study. In 1939, the flood plain in the area had a width of approximately 1,500 feet, vegetation was quite dense along both banks. The active channel had a width of approximately 300 feet.

In 1969, the reach was seen to be much straighter. The alignment of I-505 was visible at the downstream study limit. Vegetation was quite dense along banks. Between 1969 and 1972, the characteristics of the reach changed significantly. The channel appeared much narrower and uniform. The width of the channel was measured to be approximately 500 feet. Vegetation along the reach was almost completely absent. An aggregate processing facility had been established along the right bank immediately downstream of the I-505 crossing.

Between 1972 to 1986 no significant changes were noted in the overall alignment of the channel. Vegetation increased in density along both banks.

4.5.3 Areas of Land Lost to Bank Erosion

In an effort to quantify the areas of land which have been lost to bank erosion along the study reach, a comparison was made between successive sets of aerial photographs. A comparison was made by plotting the planform of the stream from 1939, 1972, and 1986 aeriels, and measuring the areas of land located between the active bank lines identified from each set of photographs. The 1939, 1972, and 1986 aerial photographs were chosen for the comparison as they were the best photographs which spanned the length of available information. This task was difficult due to differences in scale, distortion resulting from enlargement and alignment of photographs, problems in identifying active bank lines, and differences in the extent of coverage of aerial photographs with significant time differences between them. Because of these difficulties, the accuracy of the area changes may be somewhat questionable. It is believed, however, that the results do give a qualitatively correct description of the overall changes which have occurred in planform along the study reach.

Figure 4.9 details the variation in active channel topwidth which has occurred between 1939, 1972, and 1986. As can be seen from the figure, the

width of the active channel has stayed fairly consistent through time along most reaches. Reaches 8, 9, and 10, however, show an overall narrowing of the channel. This is likely due to the effects of gravel mining. Upstream reaches show only slight overall changes. Differences along upstream reaches are believed to be mainly due to translation of meanders along the creek.

Table 4.1 details the change in land areas which have occurred along the study reach due to shifts in the active bank line between 1939, 1972, and 1986. The active bank line is the boundary of main Cache Creek channel. As can be seen from the table, there has been substantial shifts in the location of the active channel. Between 1939 and 1972, there appears to have been an overall narrowing of the active channel in Reaches 7, 8, 9, and 10. This can likely be attributed to the effects of gravel mining. Between 1972 and 1986 bank erosion appeared to be the overall trend along most of the study reach. Due to the sinuous nature of the creek, the amount of bank erosion on one bank is compensated to some degree by a corresponding gain in land area along the opposite bank. The change from narrowing to bank erosion in Reaches 7, 8, 9, and 10 can likely be related to the several significant flood flows experienced along Cache Creek between 1972 and 1986.

4.5.4 Sinuosity

Sinuosity is a measure of a channel's meandering. It is defined as the length of the channel divided by the valley length. The sinuosity of the various subreaches within the study area was calculated for the years 1939, 1969, 1972, and 1986. The results of the calculations are shown on Table 4.2. Only minor changes in the sinuosity of the various reaches has occurred. No significant trend in the form of the river within the study area is evident.

4.6. Stream Profile

Historic stream profile data are available for Cache Creek from cross section surveyed in 1977 and 1983. The 1977 data cover only the area between RM 7.7 and 25.1, while the 1983 data cover the entire study reach. A 3.0-foot datum difference in the surveyed profiles are taken into consideration in comparison between the profiles. The profiles were compared where possible to detect areas of aggradation or degradation over the 6-year time span between the surveys. Conclusions regarding erosion or sedimentation at specific loca-

Table 4.1. Land Area Changes Along Study Reach.*

| Reach | 1939 to 1972 | | | 1972 to 1986 | | | 1939 to 1986 | | |
|-------|------------------|-------------------|-------------|------------------|-------------------|-------------|------------------|-------------------|-------------|
| | Rt. Bank (acres) | Left Bank (acres) | Total Acres | Rt. Bank (acres) | Left Bank (acres) | Total Acres | Rt. Bank (acres) | Left Bank (acres) | Total Acres |
| 10 | + 24.33 | + 8.78 | + 33.11 | - 15.78 | - 10.22 | - 26.00 | + 9.85 | 0 | + 9.85 |
| 9 | -147.30 | +222.68 | + 75.38 | - 68.81 | + 57.34 | - 11.47 | -159.79 | +292.99 | +133.20 |
| 8 | +130.85 | +102.31 | +233.16 | - 8.48 | - 32.60 | - 41.08 | +134.35 | + 72.89 | +207.24 |
| 7 | + 7.33 | + 34.95 | + 42.28 | - 12.06 | - 9.92 | - 21.98 | - 5.17 | + 30.99 | + 25.82 |
| 6 | ** | | | + 36.33 | - 68.15 | - 31.82 | ** | | |
| 5 | ** | | | -157.70 | + 81.06 | - 76.64 | ** | | |
| 4 | ** | | | + 26.31 | - 48.78 | - 22.47 | ** | | |
| 3 | ** | | | + 30.85 | - 35.91 | - 5.06 | ** | | |
| 2 | ** | | | | ** | | ** | | |
| 1 | ** | | | ** | | | ** | | |
| TOTAL | + 15.21 | +368.72 | +383.93 | -169.34 | - 67.18 | -236.52 | - 20.76 | +396.87 | +376.11 |

4.19

* - Indicates land lost to bank erosion, + indicates land gained.

** Reaches not included in aerial photography.

Table 4.2. Sinuosity of the Cache Creek Study Reach.

| Reach | 1939 Aerial Photos | 1969 Aerial Photos | 1972 Aerial Photos | 1986 Aerial Photos |
|-------|--------------------------|--------------------------|--------------------------|--------------------------|
| 10 | 1.05 | 1.06 | 1.10 | 1.09 |
| 9 | 1.05 | 1.06 | 1.07 | 1.12 |
| 8 | 1.12 | 1.21 | 1.22 | 1.12 |
| 7 | * | 1.00 | 1.00 | 1.04 |
| 6 | * | 1.04 | 1.04 | 1.09 |
| 5 | * | 1.19 | 1.18 | 1.15 |
| 4 | * | 1.14 | 1.14 | 1.25 |
| 3 | * | 1.20 | 1.29 | 1.33 |
| 2 | * | * | * | 1.45 |
| 1 | * | * | * | 1.00 |

*Not included in aerial photography.

tions are subject to some question as the cross sections were not surveyed at the same locations.

The following paragraphs discuss the stream profile on a reach-by-reach basis. The profile is shown in Figure 4.10.

4.6.1 Reach 1 (River Miles 29.2 to 30.5)

Reach 1 is the steepest reach in the study with an average gradient of 50 feet per mile. No data from the 1977 survey are available for comparison.

4.6.2 Reach 2 (River Miles 27.0 to 29.2)

The average gradient of Reach 2 is 13.9 feet per mile with on 1977 data available for comparison.

4.6.3 Reach 3 (River Miles 22.5 to 27.0)

The average gradient of Reach 3 is 10.0 feet per mile. Data from 1977 area available for the lower half of the reach. Comparison of profiles in this area shows slight aggradation (approximately 2 feet) in the middle of the reach between RM 24 and 25. This aggradation could be from sediment inflow from the Blue Cliffs area. There is little change elsewhere.

4.6.4 Reach 4 (River Miles 18.0 to 22.5)

Comparison of profiles in this reach show both areas of aggradation and degradation. As much as 3 to 4 feet of aggradation occur near RM 20. Areas of significant degradation occur near the Guinda Bridge (3 to 4 feet) and near RM 19 (2 to 2.5 feet). The average gradient of the reach from the 1983 profile is 9.8 feet per mile.

4.6.5 Reach 5 (River Miles 12.3 to 18.0)

Areas of both aggradation and degradation occur in this reach. As much as 3 to 6 feet of degradation occur at several locations. Dredging in the reach may have contributed to the degradation. Areas of aggradation are generally minor (1 to 2 feet) except near RM 15 where almost 5 feet of aggradation has occurred. The average gradient of the reach (1983 profile) is 9.8 feet per mile.

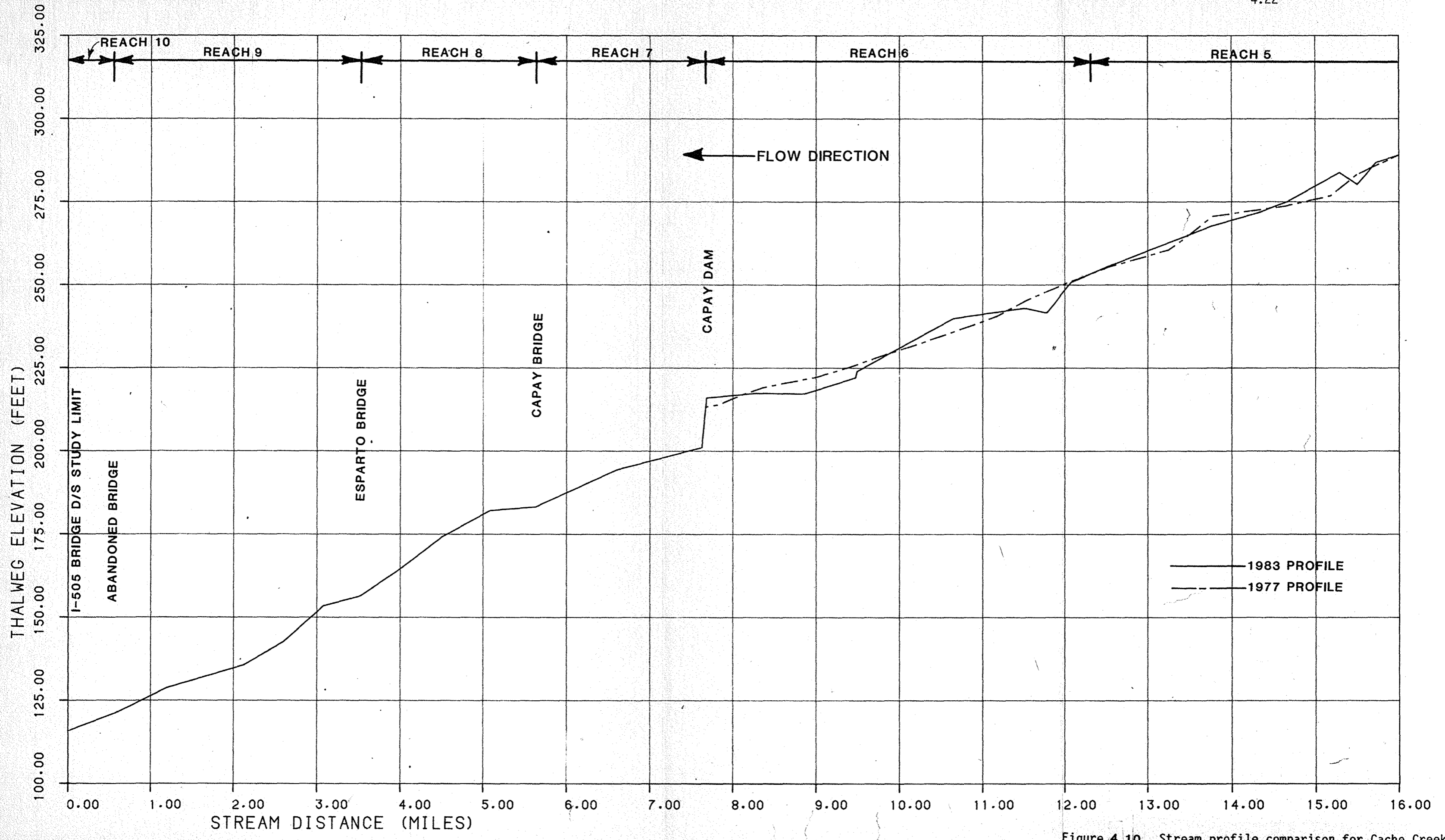


Figure 4.10. Stream profile comparison for Cache Creek

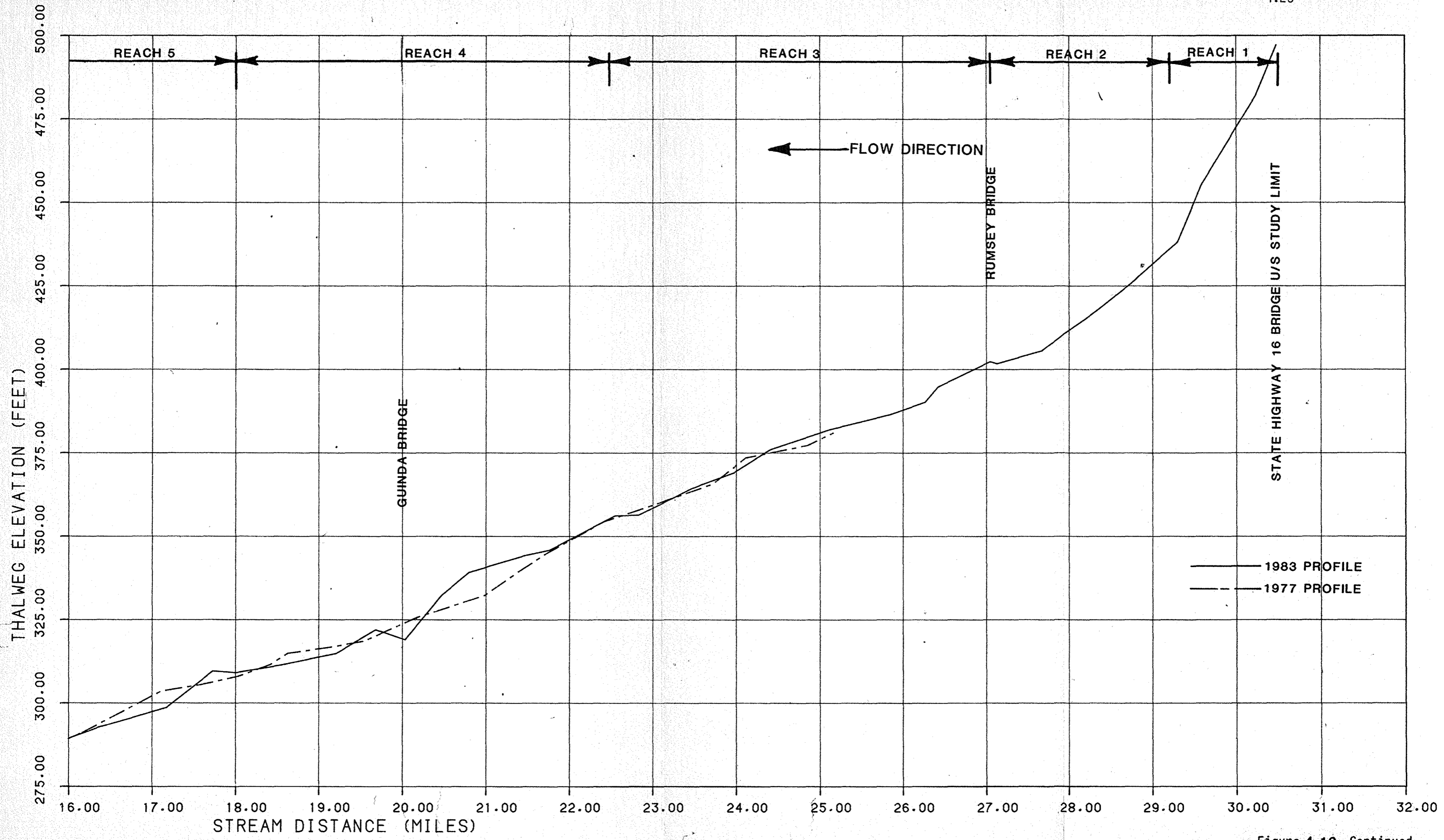


Figure 4.10. Continued

4.6.6 Reach 6 (River Miles 7.7 to 12.3)

This reach shows nearly 3 to 4 feet of aggradation in the upstream half. As much as 3 to 4 feet of degradation occurs in the downstream half. Approximately 2 to 3 feet of aggradation occurs in the pool area just above the dam. The average gradient of the reach (1983 profile) is 7.9 feet per mile.

4.6.7 Reach 7 (River Miles 5.6 to 7.7)

No profile data from 1977 is available for comparison in this reach. A large drop at the Capay Dam site indicates possible long-term aggradation above the dam and degradation downstream of the dam. The reach below the dam appears flatter, indicative of the channel adjusting to reduced sediment loads below the dam. The bed below the dam currently is controlled by a rock outcrop (see Chapter II) which will limit any further degradation. The average gradient of the reach is 8.9 feet per mile.

4.6.8 Reach 8 (River Miles 3.5 to 7.7)

No profile data from 1977 are available for comparison in this reach. The reach is slightly steeper (16.5 feet per mile) than those just upstream or downstream. This may be related to previous mining in the reach and downstream.

4.6.9 Reach 9 (River Miles 0.6 to 3.5)

No profile information is available from 1977 for comparison in this mined reach. The 1983 profile shows an average slope of 10.9 feet per mile.

4.6.10 Reach 10 (River Miles 0.0 to 0.6)

No profile information is available from 1977 for this reach. The 1983 profile shows an average gradient of 9.6 feet per mile.

4.7 Channel Morphology

Channel morphology of Cache Creek varies through the study area in response to geologic and man-made controls. Previous discussion (see Section 4.5 for example) has described the morphology on a reach by reach basis. This section summarizes the observations and relates the changes to the geologic and man-made controls.

The channel at the upstream limit of the study is confined to a narrow valley. The channel here has a riffle-pool-type morphology typical of mountain streams. The channel is steep (50 feet per mile), has a relatively uniform width, and is stable due to the presence of large boulders and geologic controls.

From the hills, the channel flows through the alluvial Capay Valley. Here the channel has movable beds and banks and exhibits a meandering pattern with a moderate channel slope (approximately 10 feet per mile). The degree of meandering varies with the more sinuous reaches showing more bank instability and variation in topwidth.

Below the Capay Valley, the channel flows through the Sugarloaf area of the Capay Hills. Here the channel is confined by a narrow valley and exhibits a relatively straight pattern with a relatively uniform width. The channel appears stable in this area.

Below the Sugarloaf area, the channel enters a wide alluvial valley and returns to a more meandering pattern. Extensive gravel mining in this area has caused the channel to become very wide and unstable. The channel exhibits a braided pattern in this area at higher flows.

Schumm (1977) has classified alluvial channels according to the type of sediment load being transported. Bed-load channels transport sediment predominantly as bed load and typically exhibit a braided pattern. Mixed load channels transport both bed and suspended load and typically exhibit a moderately meandering pattern. Suspended-load channels transport a significant portion as suspended load and typically have a high sinuosity. Morphologic evidence for the alluvial reaches of Cache Creek suggests the channel to be of the mixed-load type. Coarse material in the channel bed suggests significant bed-load transport in the stream. Fine material in the high sediment production areas of the watershed (see Section 4.3) suggests that suspended load transport in the stream is also significant. The channel pattern of the alluvial reaches of Cache Creek appears to be near a threshold where a change in hydrologic variables or sediment load can produce drastic changes in channel pattern. This is exhibited in the reaches influenced by gravel mining.

The mining operations have disturbed the channel banks and overbanks to a degree where a large supply of coarse sediment is available to the stream during flood flows. The availability of coarse sediment in these reaches appears to have caused the channel to shift to a more braided-type pattern

typical of a bed-load channel. Changes of the stream from a moderately meandering channel to a braided channel follows the following relationship given by Schumm (1977):

$$Q_s^+ \sim b^+, d^-, \lambda^+, S^+, p^-$$

where Q_s^+ = increase in sediment load

b^+ = increase in channel width

d^- = decrease in channel depth

λ^+ = increase in meander wavelength

S^+ = increase in channel slope

p^- = decrease in channel sinuosity.

The relationship defines the indicated changes in the given morphologic variables to an increase in sediment load. For the mined reaches of Cache Creek, the previously meandering channel (as presumed from undisturbed reaches in the area) became wider, shallower, steeper, and less sinuous as a result of the mining. At the present time, the channel in this area is oversized, resulting in deposition of sediment carried to the area from upstream reaches. Other reaches upstream that show increased sediment loading also appear to be closer to a braided pattern with the formation of islands and a wider active channel width.

4.8 Critical Erosive Flows

As an initial step in identifying the flows, flow ranges and durations at which sediment-related problems are occurring, some simple qualitative calculations were performed to evaluate the armoring potential in degradational reaches, height of inundation of banks and relative shear stresses acting on the banks. These calculations were performed only at selected cross sections to indicate the general characteristics of the study reach. The Phase II work will provide more detailed quantitative analysis of the above factors.

In general, the site reconnaissance and subsequent geomorphic analysis of the system indicated that the channel bed is composed of sand, gravel, and cobble material with a general increase in the sizes (particularly at the large end of the size distribution) in the upstream direction. From the sizes of bed material observed in the channel, there may be significant armoring potential in areas that exhibit a degradational tendency. The channel banks throughout the study reach, on the other hand, are composed primarily of sand, silt, and clay material with some lenses of gravel. Although the bank

material appeared to be somewhat cemented in most areas, it did not exhibit a significantly cohesive nature. This type of material is typically quite stable when dry, as evidenced by the vertical banks in many areas. It may, however, become highly erosive when saturated and subjected to channel flows.

A total of six cross sections were selected to represent typical areas within the study reach. Hydraulic conditions were estimated for a range of flows from approximately the 2-year flood peak to the 100-year flood peak using the HEC-2 model. Rough estimates of shear stress acting on the channel bed were made and used to evaluate the incipient particle size and armoring potential. At locations where a significant bend in the channel occurred, the shear stress acting on the channel banks was then estimated. The selected cross sections were located at RM 9.5, 19.5, 20.5, 22.5, 23.5, and 24.4. (Cross sections at RM 20.5 and 23.5 are located at USGS "active erosion" sites 27 and 30, respectively.) Of these areas, only RM 9.5 and 20.5 showed evidence of degradation from the profile comparison. The calculations indicated little potential for armoring at RM 9.5. At RM 20.5, however, significant armoring potential exists. The bridge located downstream of this area may cause a backwater effect which may tend to reduce the scour potential for some distance upstream at higher flows. Although the other areas do not appear to be degrading, the calculations indicated that the channel would armor rather quickly if a degradational tendency did exist.

As previously discussed, comparison of the peak flow frequency curves for the pre and postproject conditions indicates an increase in peak discharge for floods between approximately 1.5- and 4.0-year return period. Evaluation of the change in armoring potential related to the proposed modifications to the Clear Lake outlet channel indicated insignificant differences.

The cross sections at RM 19.7, 20.5, 23.5, and 24.4 occur at channel bends. In all cases, erosion of the bank on the outside of the curve is occurring. Estimates of the shear stress acting on the outside of the bends indicates that sufficient tractive force is available to move significant quantities of material for the range of flows being considered. It should be emphasized that the moisture conditions within the banks at the time they are subjected to flow as well as the duration of the flows are critical factors affecting the amount of erosion that will occur. Comparison of the changes to the postproject frequency curve indicates that the shear stress at the peak flow may increase by as much as 30 percent. As discussed by others (USCOE,

1978), in previous work regarding the study reach, however, the decrease in duration of the flows may be sufficient to compensate for the increased erosive tendency at the peak flow.

In evaluating the range of flows where bank erosion problems may occur, the depth of inundation of the banks may be a significant factor. In many areas, the toe of the banks appeared to be protected to some degree by rubble consisting of cobbles which have either been washed into the channel from upstream or were left behind as the bank eroded. A significant mechanism of bank failure in this system appears to be derived from undercutting of the toe of the slope. The rubble material acts to limit this tendency. In these areas, erosion of the banks will not occur until the flows are sufficiently deep to attack the unprotected portions. From the rough hydraulic calculations performed for this analysis, it appeared that the flow depths were generally sufficient to attack the unprotected areas for even the 2-year flood peak. Further analysis will be conducted in the Phase II study to more definitively determine at what flows this does occur.

V. CONCLUSIONS

This report presents the background data, analysis, and results of a general (Phase I) qualitative geomorphic analysis of the Cache Creek channel between Clear Lake and Madison, and the contributing watershed area. More specific analysis and observations are delineated for the approximately 30-mile detailed study reach between Rumsey, approximately 20 miles downstream of Clear Lake to the Interstate 505 bridge near Madison. The general purpose of the study was to identify the geomorphic characteristics of the study area which may affect the form and overall stability of the existing channel, document areas of active channel change, and identify significant factors which should be considered in the concurrent sediment study being conducted by the Hydrologic Engineering Center. The ultimate goal of these studies is to evaluate the potential impact of the authorized modifications to the Clear Lake outlet channel on the stability of the downstream channel. Of particular concern is the potential for increased bank erosion. The following specific observations and conclusions were derived from this preliminary study:

1. The approximately 1,300-square-mile watershed area of Cache Creek is characterized by highly variable precipitation patterns which appears to have a significant influence on the morphology of the channel. Upstream portions of the watershed are subjected to an average of approximately 60 inches of rainfall. The average rainfall amount reduces significantly in the downstream direction with the lower portions of the watershed receiving only approximately 12 inches of rain per year. The variability of rainfall has a significant impact on the vegetation characteristics on the hillslopes and channel banks. In areas of high precipitation, vegetation tends to be more dense limiting the amount of overland erosion and providing erosion protection for the channel banks. In the drier downstream areas, the vegetation is relatively sparse resulting in increased overland erosion and sediment yield to the channel and increasing the tendency for bank erosion.
2. The general geology and topography of the watershed also significantly influences the character of the channel. Steep hillsides in the watershed, particularly in the relatively dry areas adjacent to the detailed study reach, contribute to a relatively high sediment yield. Areas of badland-type topography on the North Fork of Cache Creek below Indian Valley Reservoir, and on the mainstem above the confluence with the North Fork indicate significant gullying and resulting high sediment load. Potential for mass wasting of the hillslopes in the form of landslides and slumping is also high in many areas in the lower watershed which may contribute larger slugs of sediment to the system. Several active areas were noted near the Guinda Bridge during the site reconnaissance, for example.
3. Soils in the watershed are generally silt-loam material which tends to be very erosive if not protected by vegetation.

4. These soil types also appear to make up the majority of the channel banks. Some lenses of coarse gravel and clay were also noted in the banks. Due to the lenticular nature of these deposits, however, they are not felt to contribute significantly to the stability of the banks. The stability of banks is thought to be highly influenced by the moisture content of the bank material. Typically, the banks along the creek were seen to be nearly vertical and quite stable when dry. When saturated, however, the banks become unstable. Saturation and drawdown of banks caused by flood flows most likely has a tremendous impact on the stability of banks. In some locations, clay outcrops were observed along the toe of banks. Erosion at these locations was seen to be occurring primarily to the overlying silty material. This may be attributed to the saturation of the less cohesive silty material by flood flows or groundwater which may be traveling along the interface between the two types of material.

In many areas, however, the toe of actively eroding banks is armored with large cobbles that appear to have been deposited through the bank erosion process. This armoring may provide limited protection for the toe, limiting the tendency for undercutting of the banks.

5. Numerous areas of active bank erosion were observed. These locations are listed in Table 2.4 and shown on Plates 1 through 5. Limited areas have been protected from erosion in localized areas. In most cases, the protection measures are not engineered structures and have not been constructed as part of an integrated bank protection plan. Their impact on the overall bank stability in the system is probably very limited.
6. Movable bed material in the study reach consists of particle sizes ranging from sand to cobbles. As described in the previous chapter, there is a general coarsening of material in the upstream direction. The median bed-material size in the Capay area is approximately 25 mm, increasing to 50 mm near Rumsey. The amount of large cobbles also increase. The average D_{90} size near Capay is roughly 75 mm, increasing to 125 mm near Rumsey.
7. Comparison of channel profiles surveyed in 1977 and 1983 between Capay Dam and approximately 3 miles downstream of the Rumsey Bridge indicate that there is no general aggradation or degradation trend. As discussed in Section 4.6, however, limited areas of both aggradation and degradation do exist. A landslide near RM 21.3, for example, appears to have raised the channel bed by 4 to 5 feet. Several areas of degradation were also observed, including the reach near the Guinda Bridge and in the downstream reaches where active gravel mining is occurring. The uncharacteristically flat gradient in the downstream channel and drop in elevation across the Capay Dam, shown in the 1983 profile, also indicate probable degradation in that area. However, historical profiles were not available to verify this observation.
8. Based on preliminary hydraulic calculations and the documented bed-material sizes, significant potential for armoring of the channel during degradation exists in the majority of the study reach. The exception to this general observation occurs in the reach near RM 10 where all of the material is sufficiently fine to be moved by flood flows. In addition to

the armoring potential, several natural and man-made controls exist throughout the study reach. Significant natural controls occur near RM 29.2 where a bedrock outcrop is exposed in the channel bed and near RM 10.5 where a landslide has deposited large boulders 5 to 10 feet in diameter into the channel. Man-made controls include the Capay Dam at RM 7.7 and the various bridge structures which cross the channel within the study reach. Complete documentation of other less significant controls is presented in Table 2.1.

9. Active channel widths throughout the study reach vary widely from 100 to 150 feet in the narrow canyon area between RM 29.5 and 30.5 and RM 9.4 and 11.7 to over 1,000 feet in the active mining areas downstream of the Capay Bridge. Factors contributing to the variation in channel width include encroachment from geologic formations in the canyon areas to a braiding tendency in mining areas where the valley floor is wider. Sand and gravel mining in the downstream reaches coupled with active bank erosion also contributes to an uncharacteristically wide channel in some areas.
10. Considering the implications of the various observation discussed above, it appears that the study reach, in general, may be somewhat overloaded with sediment due to high watershed and tributary sediment yield and contributions from mass wasting and active bank erosion. This is evidenced by the wide variation in channel width and presence of numerous alternate and middle bars in unconfined areas of the study reach. Field observations have shown removal of sand and gravel material from the channel bed has had a profound impact on the channel characteristics in the study reach downstream of the Capay Bridge.
11. Preliminary assessment of the proposed modifications to the Clear Lake outlet channel indicate that the impacts may be limited. The magnitude of the peak flows between the 1.5- and 4.0-year return period floods will increase after the modifications are implemented which may tend to slightly increase the bank erosion tendencies. Since the volume of water that passes through the system is nearly the same for those floods, however, the duration of erosive flows may decrease, which might compensate for the higher flows. More detailed quantitative analysis of this phenomenon will be conducted as part of the Phase II study.

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GEOMORPHIC ANALYSIS
OF
CACHE CREEK
BETWEEN RUMSEY AND MADISON
YOLO COUNTY, CALIFORNIA

PHASE II FINAL REPORT

Submitted to

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Los Angeles District
Corps of Engineers
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I. INTRODUCTION

1.1 Study Purpose

This report documents the second phase of a two-phase geomorphic analysis being conducted as part of a detailed investigation by the U.S. Army Corps of Engineers (COE) to assess potential impacts of the authorized Cache Creek Basin, California (Clear Lake Outlet Channel) project on sediment-transport characteristics of Cache Creek between Rumsey, approximately 20 miles downstream of Clear Lake and Madison (Interstate Highway 505). Figure 1.1 shows a general location map of the study reach.

The purpose of this report is to provide a quantitative geomorphic analysis of Cache Creek along the approximately 30-mile study reach. The results of the study will be used in conjunction with the quantitative sediment-routing analysis being conducted concurrently by the COE Hydrologic Engineering Center (HEC) to predict the impact of the Cache Creek Basin project on the stability of the channel along the study reach.

The authorized flood-control project consists of modifications to the Clear Lake Outlet Channel. The 5-mile long outlet channel between Clear Lake and the Clear Lake Dam presently limits the rate at which water can be released from the Lake. This limitation, combined with large inflows, has resulted in flooding around the lake rim. To control such flooding the discharge capacity of the outlet channel is proposed to be increased. The increased discharge capacity of the outlet channel will alter the flow frequency characteristics of Cache Creek. Concern exists that such alteration may induce channel instability problems along downstream reaches of the stream. Of particular concern is the effect of the alteration of flow frequency on bank erosion and other sediment-transport-related problems.

1.2 Authorization

Simons, Li & Associates, Inc. (SLA) was contracted to conduct this study by the U.S. Army Corps of Engineers (COE), Los Angeles District, as Delivery Order Number 4 under Contract Number DACW09-86-D-0016. Mr. Rene Vermereen is the project manager, Los Angeles District. Mr. Edward F. Sing is the project coordinator, Sacramento District.

1.3 Principal SLA Investigators

A list of principal investigators for SLA is shown in Table 1.1.

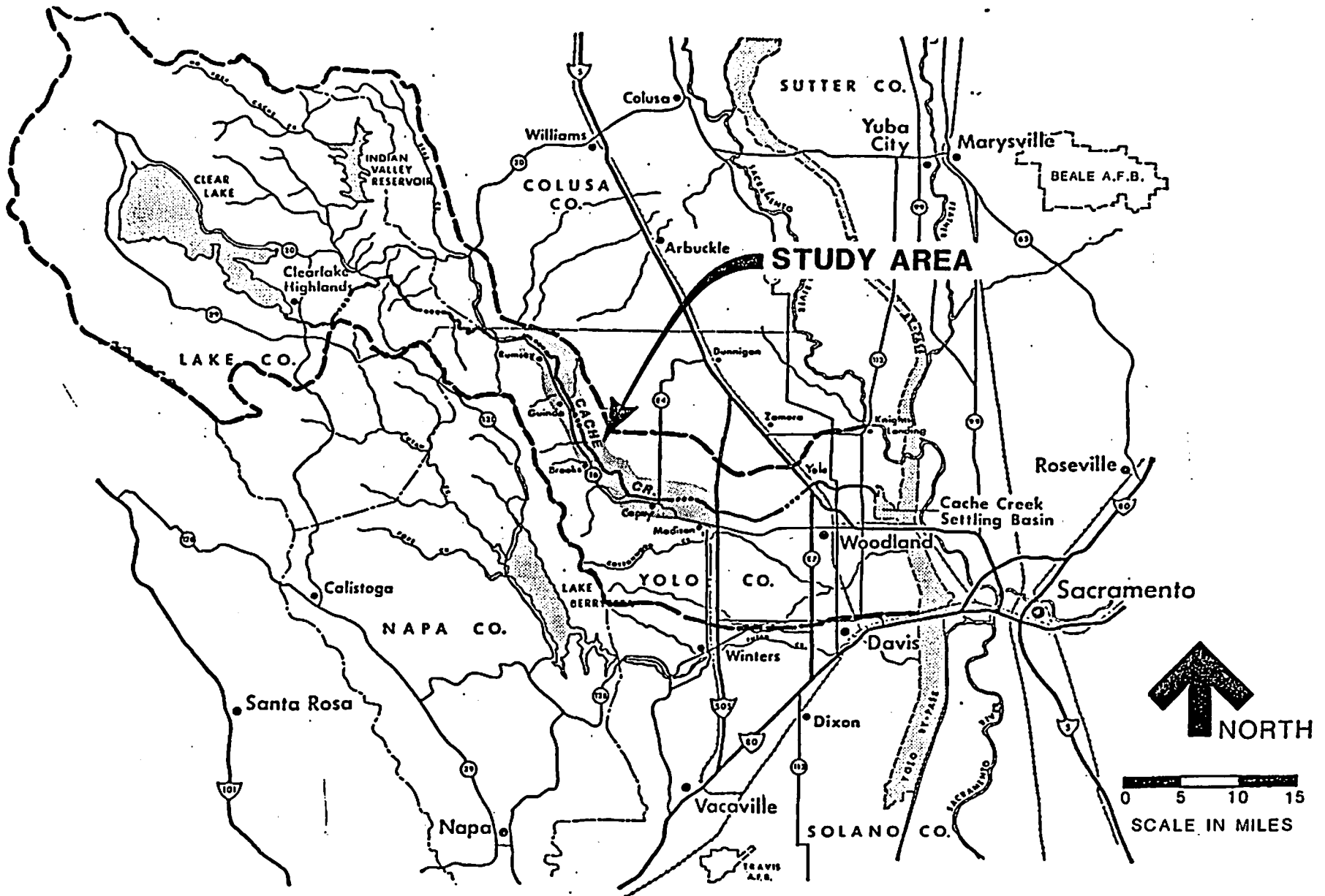


Figure 1.1. General location map of the study reach.

Table 1.1. Principal Project Investigators of
Simons, Li & Associates, Inc.

| Investigator | Function |
|----------------------------------|-----------------------|
| Ruh-Ming Li, Ph.D., P.E. | Principal In Charge |
| Robert C. MacArthur, Ph.D., P.E. | Technical Review |
| Robert A. Mussetter, P.E. | Project Manager |
| Thomas R. Grindeland, P.E. | Lead Project Engineer |
| C. Gary Wolff, E.I.T. | Hydraulic Engineer |
| Ann K. Woods | Project Engineer |

1.4 Specific Tasks Conducted for Phase II

The specific tasks conducted for Phase II of this study included:

1. Incorporation of data from Phase I study into analysis for Phase II.
2. Analysis of COE historic flood routings for pre- and postproject conditions. This included the development of flow-duration curves for the available data.
3. Calibration and implementation of HEC-2 water-surface profile program for analyzing hydraulic conditions along the study reach.
4. Development of sediment-transport relationships applicable to different sections of the study reach.
5. Sediment-continuity analysis using hydrology developed from COE historic flood routings to determine erosion/sedimentation trends along the study reach for pre- and postproject conditions.
6. Determination of the frequency of flows causing bank erosion problems along the study reach for pre- and postproject conditions.
7. Determination of the relative magnitude of bank erosion potential along the study reach for pre- and postproject conditions.
8. Determination of the impact of the project on channel morphology along the study reach.
9. Description of channel stabilization techniques which would be effective in controlling erosion problems.
10. Preparation of a brief summary report.

1.5 Data Sources

Data used in the conduct of this analysis were collected from a variety of sources. Sources including the COE, Sacramento District, the U.S. Geological Survey, the Soil Conservation Service, and Yolo County. A complete tabulation of specific references used is provided at the end of this report.

II. HYDROLOGY

2.1 Operation Changes for Clear Lake

Cache Creek, historically a tributary to the Sacramento River is a seasonal stream which runs dry much of the summer and fall and experiences heavy flows in the winter and spring. Flows along the study reach are influenced by Clear Lake and Indian Valley Reservoir located in the upstream watershed (Figure 1.1). The Cache Creek Basin project involves changing the operation of flow releases from Clear Lake which will alter downstream flow frequency characteristics. The operation was designed to minimize flooding around the lake and at the same time minimize impacts on the downstream channel.

The new Clear Lake operation will be based on two rating curves shown in Figure 2.1. As long as the Clear Lake stage remains below 7.56 feet, the operation will be based on the present channel-capacity curve. Once the Clear Lake stage rises above 7.56 feet, the releases will be based on the new 8,000 cfs channel-capacity curve as long as flows at the Rumsey gage do not exceed 20,000 cfs. If flows at Rumsey exceed 20,000 cfs, releases from Clear Lake will be based on the present channel-capacity curve. When Clear Lake stage is reduced back to 7.56 feet or below, releases will revert back to the present channel-capacity curve.

2.2 Corps of Engineers Historic Flood Routings

To determine changes in the downstream flow characteristics due to the authorized project, the COE developed a flood-routing model for Cache Creek to simulate pre- and postproject conditions. Historic flow data from 8 runoff seasons were used in the simulation. The 8 water years selected were 1952, 1956, 1958, 1965, 1970, 1974, 1978, and 1983. Table 2.1 summarizes the period of runoff covered by data. All years had flow peaks above 10,000 cfs at Rumsey. The flow data covers only the significant runoff portion of each water year. The simulation included both mean daily and bihourly flow data.

Figure 2.2 shows a schematic of the COE routing model. The flows computed at Rumsey, Capay, and Yolo were used in this study to encompass the study reach. The historic data were used in the model to calibrate the routing coefficients and determine reservoir inflows. Indian Valley Dam started operation in June 1974 which is in the middle of the historic data set. All historic flows before this time were modified to include operation

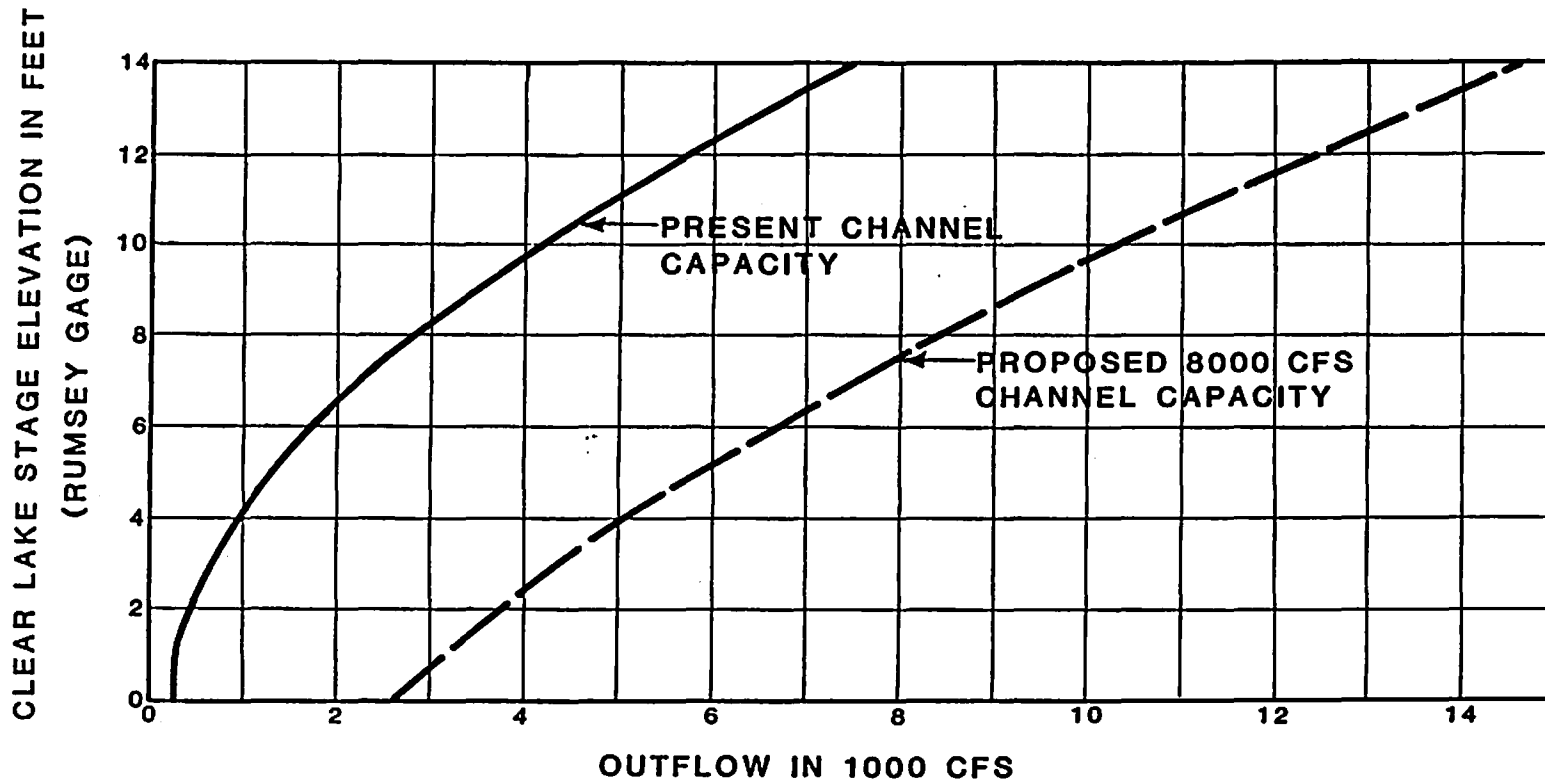
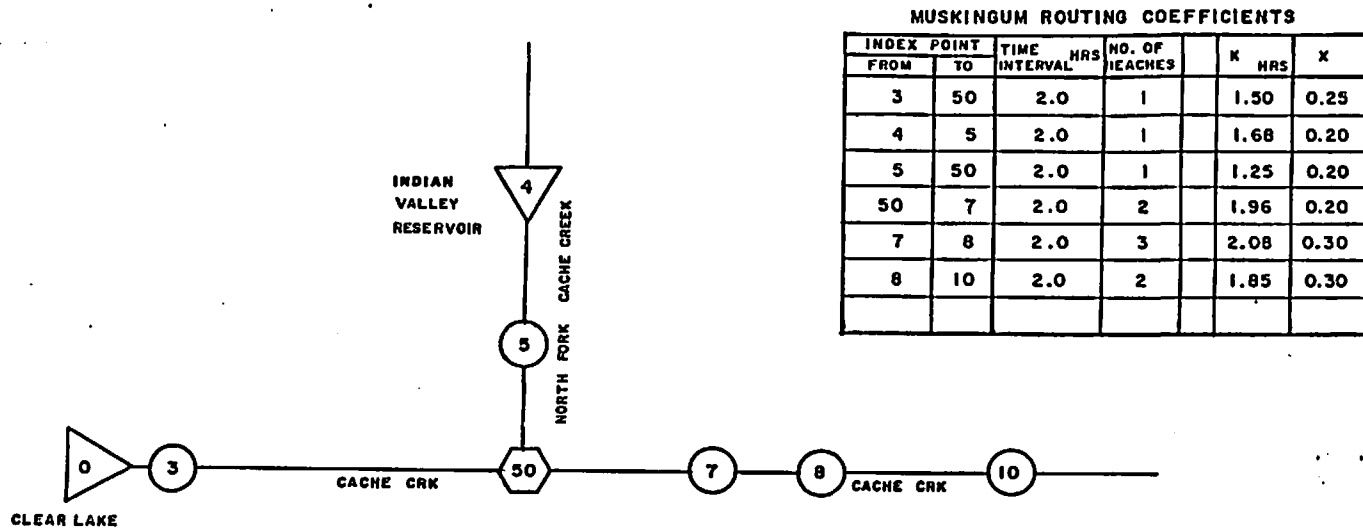


Figure 2.1. Clear Lake outlet channel rating curves proposed for the Clear Lake flood-control project.

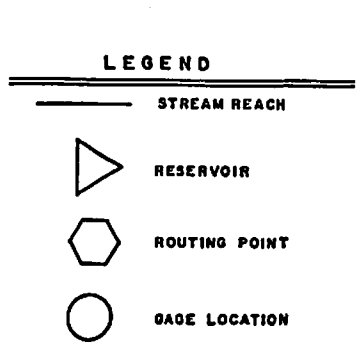
Table 2.1. Periods of Historic Flow Data Used in Corps of Engineers Flood-Routing Simulations.

| Water Year | Period of Historic Flow Data |
|------------|---|
| 1952 | January 1, 1952, to February 21, 1952 |
| 1956 | December 19, 1955, to March 14, 1956 |
| 1958 | January 24, 1958, to April 27, 1958 |
| 1965 | December 22, 1964, to February 16, 1965 |
| 1970 | January 14, 1970, to March 5, 1970 |
| 1974 | December 10, 1973, to April 16, 1974 |
| 1978 | February 6, 1978, to April 15, 1978 |
| 1983 | January 6, 1983, to May 9, 1983 |



MUSKINGUM ROUTING COEFFICIENTS

| INDEX POINT FROM | INDEX POINT TO | TIME INTERVAL HRS | NO. OF REACHES | K HRS | X |
|------------------|----------------|-------------------|----------------|-------|------|
| 3 | 50 | 2.0 | 1 | 1.50 | 0.25 |
| 4 | 5 | 2.0 | 1 | 1.68 | 0.20 |
| 5 | 50 | 2.0 | 1 | 1.25 | 0.20 |
| 50 | 7 | 2.0 | 2 | 1.96 | 0.20 |
| 7 | 8 | 2.0 | 3 | 2.08 | 0.30 |
| 8 | 10 | 2.0 | 2 | 1.85 | 0.30 |
| | | | | | |



INDEX POINT DESCRIPTION

| | |
|----|---|
| 0 | CLEAR LAKE (LAKEPORT STAGE) |
| 3 | CACHE CREEK NR. LOWER LAKE |
| 4 | INDIAN VALLEY RESERVOIR |
| 5 | NORTH FORK CACHE CREEK NR. LOWER LAKE |
| 50 | CACHE CRK. JUNCTION WITH N. F. CACHE CRK. |
| 7 | CACHE CREEK AT (ABOVE) RUMSEY |
| 8 | CACHE CREEK AT CAPAY |
| 10 | CACHE CREEK AT YOLO |

CACHE CREEK BASIN, CA.

HISTORICAL FLOOD ROUTINGS

ROUTING DIAGRAM

CORPS OF ENGINEERS, SACRAMENTO, CALIFORNIA

Prepared: E.W.P. Date: 2 APRIL 1985

Drawn:

Figure 2.2. Schematic diagram of the COE routing model for Cache Creek.

of Indian Valley Dam. This allowed the development of a uniform data set representative of present conditions. The proposed operation of Clear Lake described above was then implemented in the model to develop flows representative of conditions with the Clear Lake flood-control project in place.

2.3 Flow-Duration Analysis

Using results from the COE historic flood routings, SLA developed flow-duration curves at Rumsey, Capay, and Yolo for pre- and postproject conditions. Curves were developed for each runoff season and for the entire period of simulation (i.e., the 8 high-flow periods described above). The resulting curves for the entire period of simulation are given in Figures 2.3 through 2.5. Pre- and postproject results are plotted on the same graph to illustrate differences. Tabulation of results at Rumsey, Capay, and Yolo for the individual runoff seasons are given in Appendix A. The plotted curves for Rumsey are also included.

The flow-duration curves at each of the gage sites illustrate the effects of the project on the frequency of flows. The duration of lower flows (less than 5,000 to 6,000 cfs) is greater under preproject than project conditions. The duration of higher flows (between approximately 5,000 to 6,000 cfs to 20,000 cfs) is greater under project conditions. These results agree with the operation of the dam. Under project conditions, more frequent releases under flooding conditions will result in less frequent lower flows. For flows above 20,000 cfs, the flow-duration curves at Rumsey are nearly identical, with preproject flows being slightly more frequent. The curves are nearly identical in this region as the reservoir will be operated the same under both conditions for flows above 20,000 cfs. The slightly more frequent preproject floods result from a higher reservoir level when flows at Rumsey reach 20,000 cfs (due to a difference in operation up to 20,000 cfs). At Capay and Yolo, the preproject flood levels above 20,000 cfs become even more frequent which is likely due to routing effects (the preproject hydrographs at Rumsey would be less spiked).

It should be noted that the flow-duration curves cover only the runoff season and are biased toward seasons with peak flows exceeding 10,000 cfs. This bias should be considered in interpreting the results of the sediment transport and bank erosion analyses given later in this report. The results will still be qualitatively correct since sediment transport and bank erosion is most significant during high-flow periods.

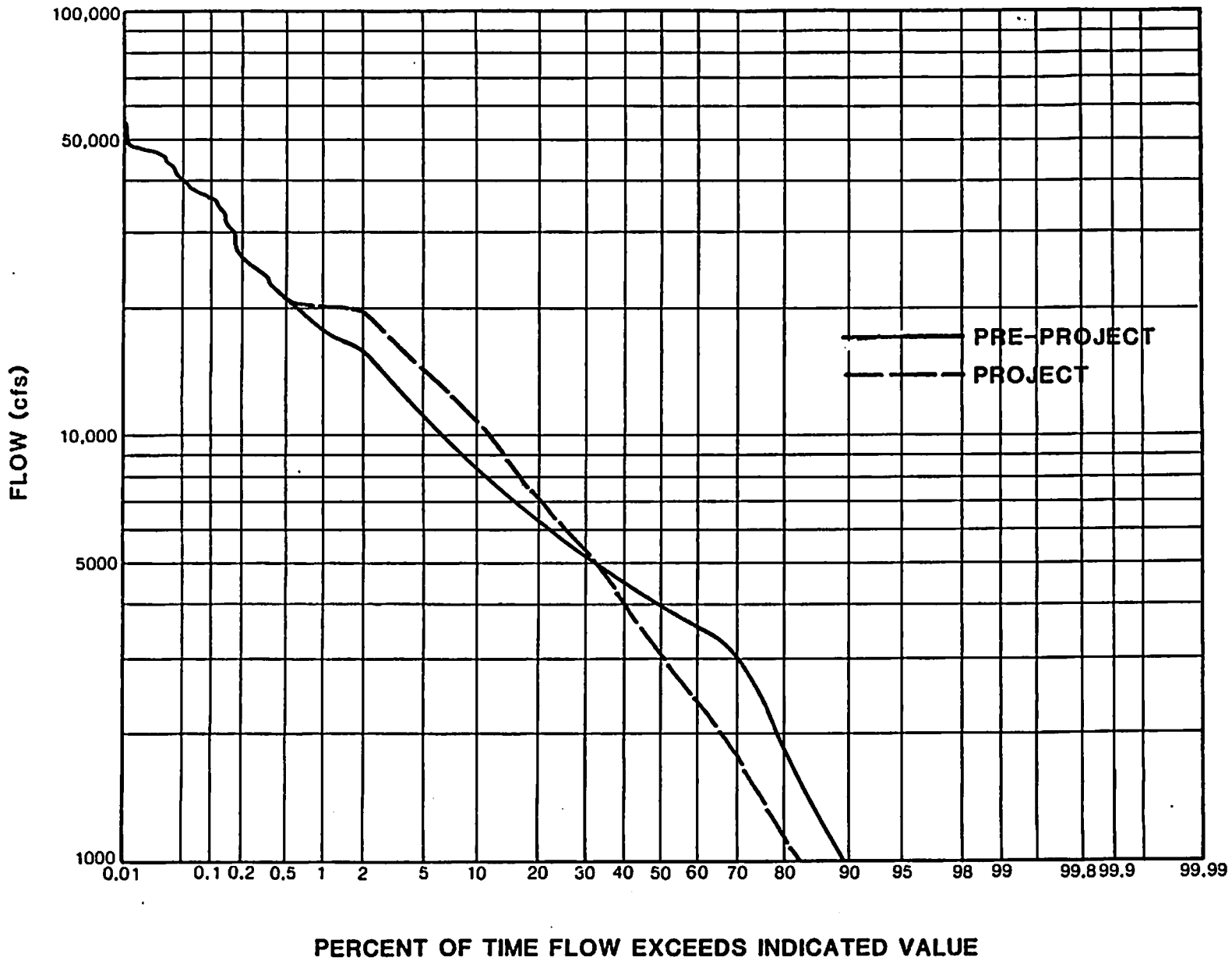


Figure 2.3. Pre- and postproject flow-duration curves computed at Rumsey for the entire period of simulated flows.

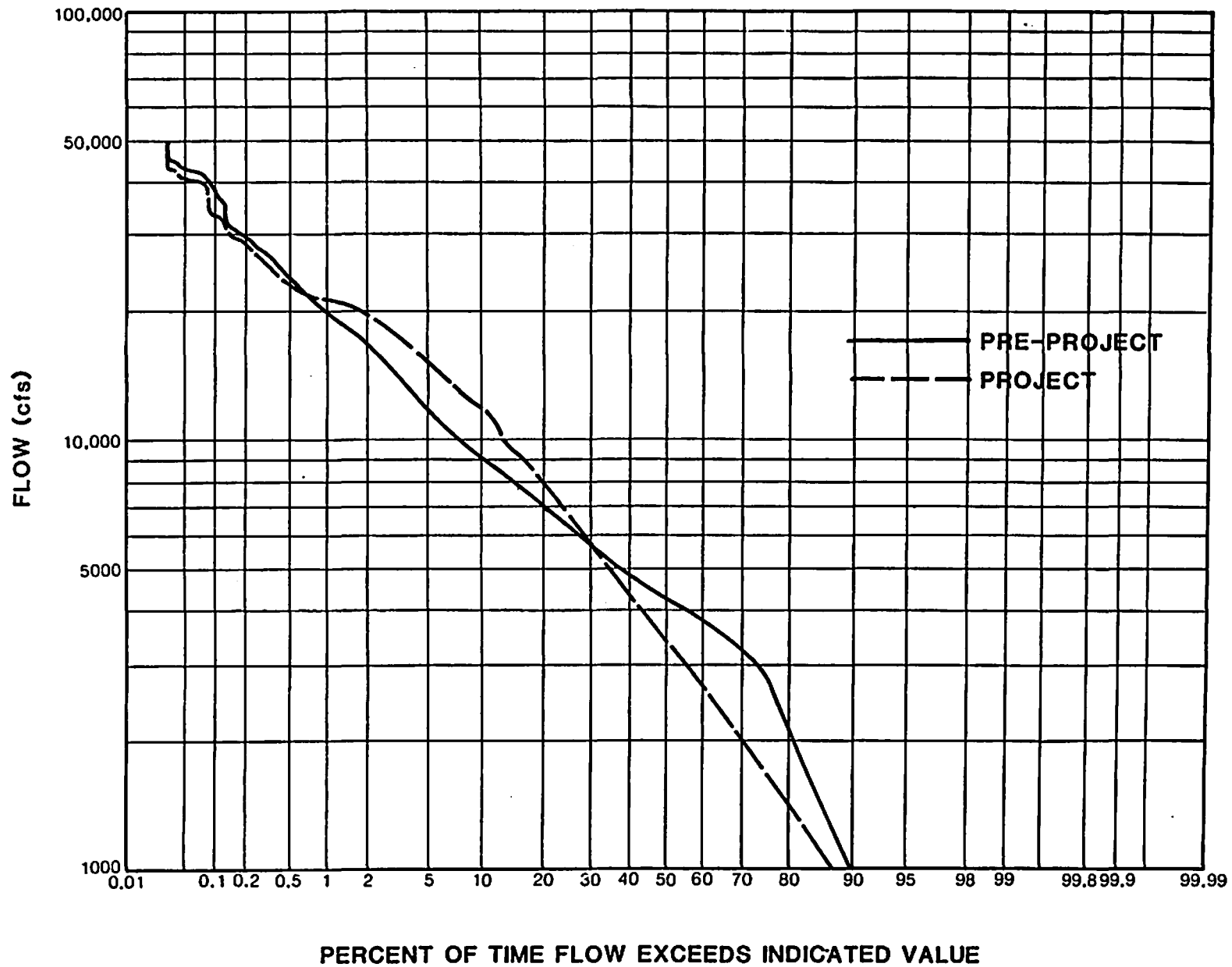


Figure 2.4. Pre- and postproject flow-duration curves computed for Capay for the entire period of simulated flows.

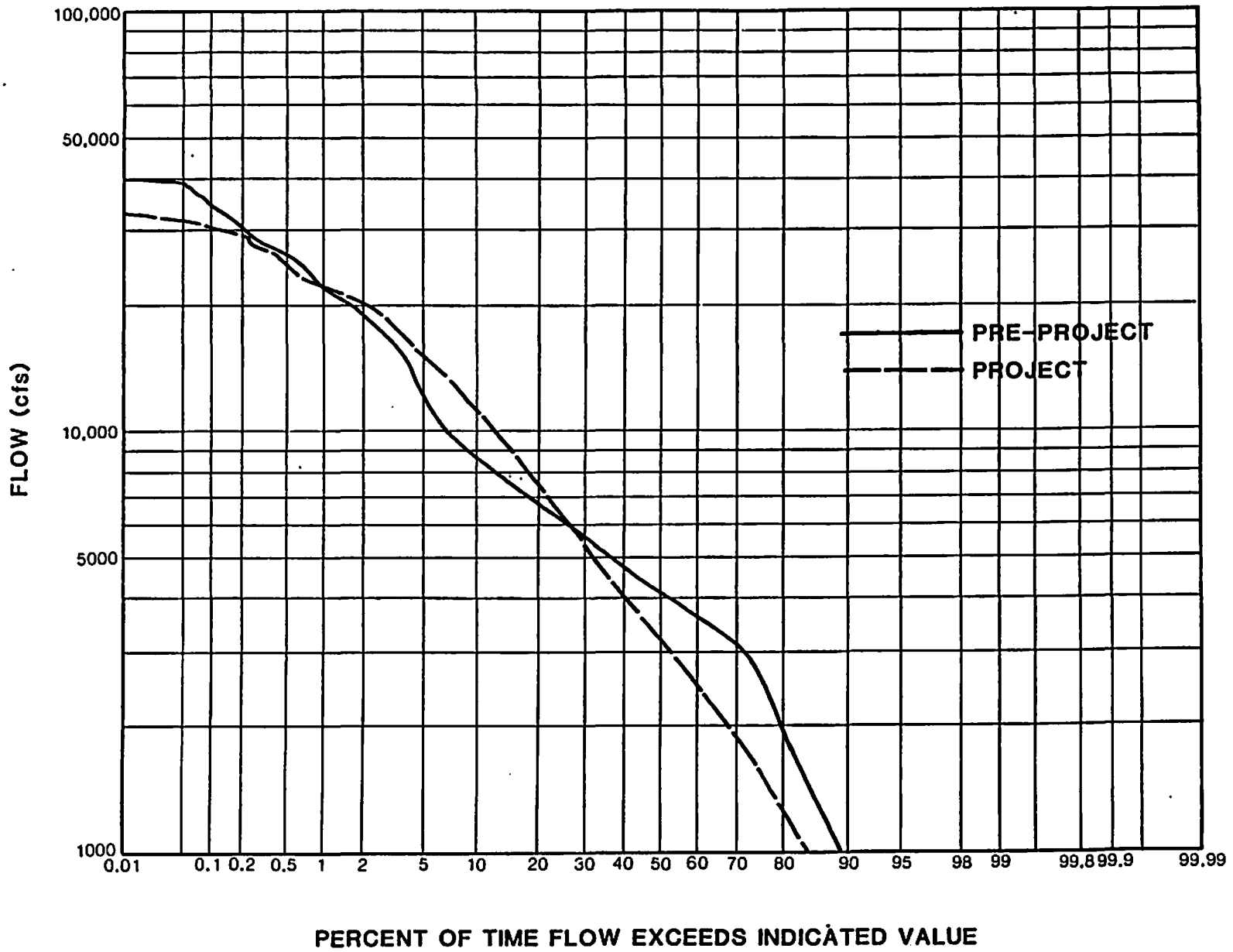


Figure 2.5. Pre- and postproject flow-duration curves computed at Yolo for the entire period of simulated flows.

III. HYDRAULICS

Hydraulic data used to perform the quantitative geomorphic analysis of the Cache Creek study reach were obtained using the HEC-2 water-surface profile program (COE, 1982). Basic data input for the model were obtained from cross sections surveyed by the U.S. Geological Survey (USGS) in 1983 through 1985. The data were modified to include geometric information on bridge crossings obtained from plans provided by the Yolo County Department of Public Works and the California Department of Highways. Starting water-surface elevations were determined using the slope-area method.

The hydraulic model was calibrated by adjusting main channel and overbank roughness coefficients until a reasonable match could be made between computed water-surface elevations and high-water marks at known discharges. The high-water marks and corresponding discharges were provided by the USGS. Table 3.1 gives the results of the calibration at selected discharges and cross sections. Table 3.2 gives the calibrated roughness coefficients used in the model. The roughness values increased in the upstream direction due to a general increase in the size of bed material.

Discharge values covering a wide range were used in the model to develop the necessary hydraulic data for subsequent analysis. Using these data and aerial photographs to identify changes in channel geometry and hydraulic characteristics, the creek was divided into 10 reaches to be used in the sediment-transport analysis. The reach delineation is shown in Figure 3.1 with a tabular listing of river miles and cross sections given in Table 3.2.

Hydraulic variables of primary interest in sediment-transport calculations include main channel velocity, hydraulic depth (area divided by topwidth), and topwidth. Average values for these variables for each reach are given in Tables 3.4 through 3.7 for a range of discharges from 3,000 to 40,000 cfs. These values were used to develop sediment-transport relationships and sediment-rating curves described in the following chapter.

Table 3.1. Calculated Versus Observed Water-Surface Elevations.

| Cross Section | Discharge near Brooks (cfs) | Observed Elevation (ft) | Computed Elevation (ft) | Difference (ft) |
|---------------|-----------------------------|-------------------------|-------------------------|-----------------|
| 3.0 | 6,960 | 209.9 | 209.8 | -0.1 |
| | 11,000 | 212.2 | 211.7 | -0.5 |
| | 26,000 | 216.2 | 217.3 | 1.1 |
| 9.0 | 6,960 | 255.5 | 255.5 | 0.0 |
| | 11,000 | 258.1 | 257.7 | -0.4 |
| | 26,000 | 264.1 | 263.7 | -0.4 |
| 14.0 | 6,960 | 283.2 | 284.3 | 1.1 |
| | 11,000 | 285.0 | 286.0 | 1.0 |
| | 26,000 | 287.6 | 289.3 | 1.7 |
| 24.0 | 6,960 | 339.4 | 338.6 | -0.8 |
| | 11,000 | 341.7 | 339.9 | -1.8 |
| | 26,000 | 344.0 | 343.7 | -0.3 |
| 30.0 | 6,960 | 371.7 | 371.7 | 0.0 |
| | 11,000 | 373.6 | 373.5 | -0.1 |
| | 26,000 | 376.6 | 376.8 | 0.2 |
| 34.0 | 6,960 | 396.2 | 397.0 | 0.8 |
| | 11,000 | 400.4 | 398.9 | -1.5 |
| | 26,000 | 402.2 | 403.4 | 1.2 |
| 43.0 | 6,960 | 491.3 | 491.0 | -0.3 |
| | 11,000 | 493.6 | 493.1 | -0.5 |
| | 26,000 | 497.6 | 497.4 | -0.2 |

Table 3.2. Roughness Coefficients Used to Determine Hydraulic Conditions Along Cache Creek.

| River Miles | Main Channel Roughness (Manning's n) | Overbank Roughness (Manning's n) |
|---------------|--------------------------------------|----------------------------------|
| 0.00 - 11.92 | 0.030 | 0.050 |
| 11.92 - 26.84 | 0.040 | 0.050 |
| 26.84 - 28.80 | 0.045 | 0.080 |
| 28.80 - 30.65 | 0.058 | 0.080 |

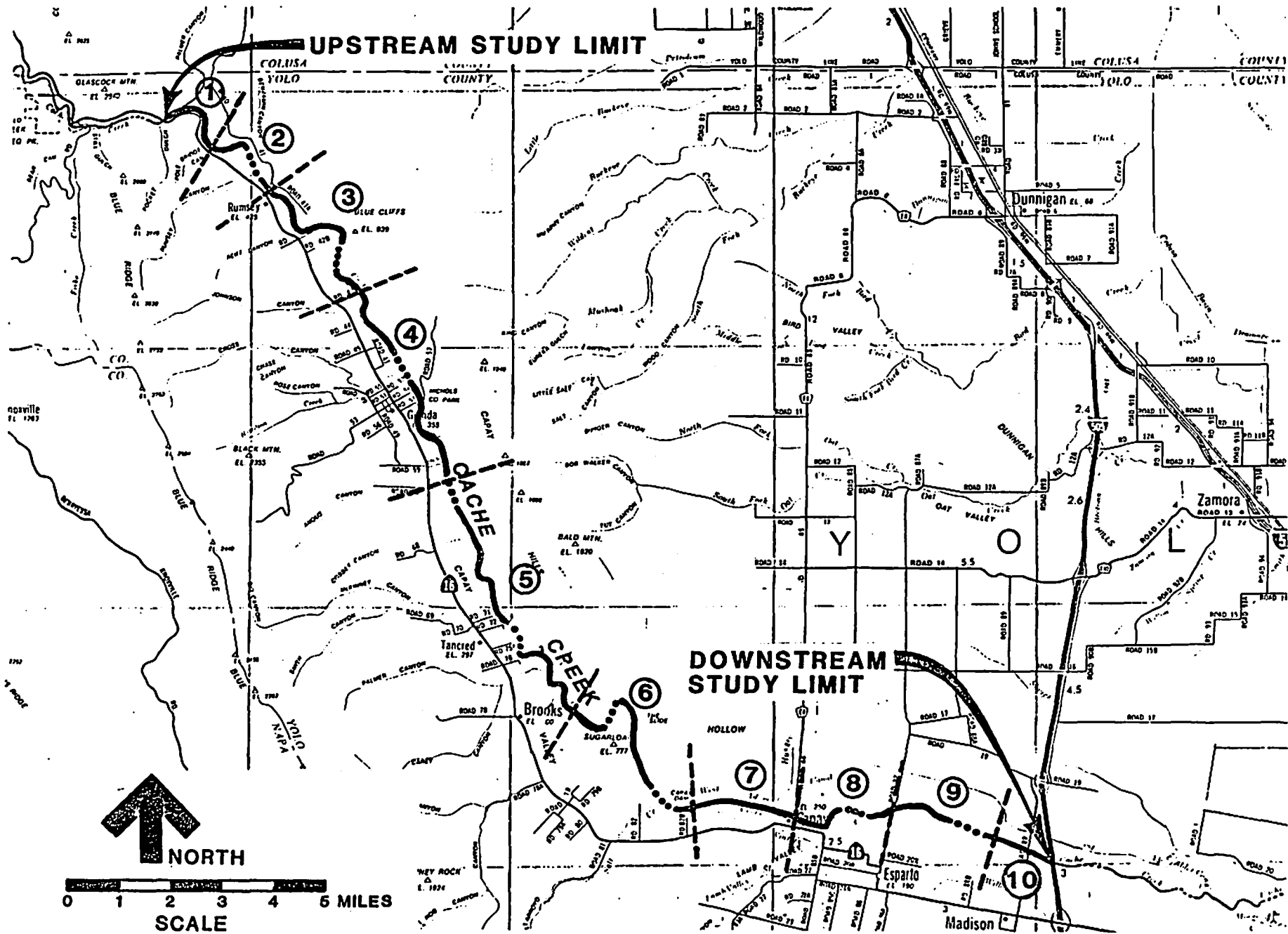


Figure 3.1. Reaches used in the Cache Creek Quantitative Geomorphic Analysis.

Table 3.3. River Miles and Cross Sections Defining Reaches Used in Cache Creek Quantitative Geomorphic Analysis.

| Reach | Location (River Miles) | Length (mi) | Cross Sections |
|-------|---------------------------|----------------|-------------------|
| 1 | 28.80 - 30.65 | 1.85 | 40.0 - 44.0 |
| 2 | 27.03 - 28.80 | 1.77 | 36.0 - 38.0 |
| 3 | 23.14 - 27.03 | 3.89 | 30.0 - 35.99 |
| 4 | 18.13 - 23.14 | 5.01 | 20.0 - 29.0 |
| 5 | 11.92 - 18.13 | 6.21 | 10.0 - 19.0 |
| 6 | 7.97 - 11.92 | 3.95 | 4.0 - 9.0 |
| 7 | 5.62 - 7.97 | 2.35 | 1.0 - 3.1 |
| 8 | 3.52 - 5.62 | 2.10 | 0.6 - 0.99 |
| 9 | 1.66 - 3.52 | 1.86 | 0.3 - 0.59 |
| 10 | 0.0 - 1.66 | 1.66 | 0.0 - 0.2 |

Table 3.4. Average Hydraulic Variables for Reaches
Along Cache Creek, Q = 3,000 cfs.

| Reach | Main Channel Velocity (ft/s) | Hydraulic Depth (ft) | Main Channel Topwidth (ft) |
|-------|------------------------------------|----------------------------|----------------------------------|
| 1 | 6.7 | 3.2 | 141 |
| 2 | 4.2 | 4.2 | 169 |
| 3 | 4.6 | 3.1 | 212 |
| 4 | 4.1 | 3.0 | 243 |
| 5 | 3.7 | 2.8 | 292 |
| 6 | 5.1 | 3.4 | 175 |
| 7 | 4.5 | 3.5 | 192 |
| 8 | 3.5 | 1.4 | 605 |
| 9 | 3.3 | 1.1 | 866 |
| 10 | 4.5 | 2.0 | 334 |

Table 3.5. Average Hydraulic Variables for Reaches
Along Cache Creek, Q = 10,000 cfs.

| Reach | Main Channel Velocity (ft/s) | Hydraulic Depth (ft) | Main Channel Topwidth (ft) |
|-------|------------------------------------|----------------------------|----------------------------------|
| 1 | 9.2 | 6.2 | 173 |
| 2 | 5.4 | 5.1 | 364 |
| 3 | 5.8 | 4.6 | 360 |
| 4 | 4.8 | 5.2 | 398 |
| 5 | 4.7 | 4.4 | 487 |
| 6 | 7.2 | 7.2 | 191 |
| 7 | 6.7 | 6.9 | 217 |
| 8 | 4.3 | 2.2 | 1,082 |
| 9 | 4.2 | 1.8 | 1,333 |
| 10 | 6.7 | 4.0 | 375 |

Table 3.6. Average Hydraulic Variables for Reaches
Along Cache Creek, Q = 25,000 cfs.

| Reach | Main Channel Velocity (ft/s) | Hydraulic Depth (ft) | Main Channel Topwidth (ft) |
|-------|------------------------------------|----------------------------|----------------------------------|
| 1 | 11.4 | 10.6 | 202 |
| 2 | 6.5 | 8.6 | 436 |
| 3 | 8.0 | 7.5 | 372 |
| 4 | 6.5 | 9.0 | 423 |
| 5 | 6.1 | 7.0 | 580 |
| 6 | 10.3 | 11.7 | 206 |
| 7 | 8.6 | 9.6 | 302 |
| 8 | 5.7 | 3.1 | 1,435 |
| 9 | 5.2 | 2.9 | 1,677 |
| 10 | 8.7 | 6.9 | 420 |

Table 3.7. Average Hydraulic Variables for Reaches
Along Cache Creek, Q = 40,000 cfs.

| Reach | Main Channel Velocity (ft/s) | Hydraulic Depth (ft) | Main Channel Topwidth (ft) |
|-------|------------------------------------|----------------------------|----------------------------------|
| 1 | 13.3 | 14.1 | 202 |
| 2 | 7.1 | 12.0 | 436 |
| 3 | 8.6 | 10.3 | 375 |
| 4 | 7.7 | 11.7 | 426 |
| 5 | 6.9 | 9.3 | 580 |
| 6 | 12.3 | 14.9 | 212 |
| 7 | 11.1 | 11.8 | 302 |
| 8 | 6.4 | 4.0 | 1,577 |
| 9 | 5.9 | 3.9 | 1,753 |
| 10 | 10.0 | 9.1 | 439 |

IV. CHANNEL STABILITY ANALYSIS

4.1 Sediment-Continuity Analysis

To evaluate project impacts on the erosion/sedimentation potential of the study reach, a relatively simple model based on sediment continuity was used. The model is based on the concept that for a given storm, the volume of the sediment eroded or deposited in a given stream reach is the difference between the volume of sediment supplied and the volume of sediment transported. If supply is greater than transport potential, the reach is aggradational. If transport potential is greater than supply, the reach is degradational. Sediment supplied to a reach is equal to the sediment transported by the adjoining upstream reach plus any sediment inflow from tributaries. The steps used to perform the sediment-continuity analysis are described in the following sections.

4.1.1 Transport Relationship Development

The first step in the sediment-continuity analysis for Cache Creek was the development of sediment-transport relationships applicable over different segments of the study reach. The relationships are based on sediment-transport theory and give the unit width bed-material sediment discharge as a function of velocity, depth, and channel width. Different relationships were developed for different segments of the study reach to account for varying bed-material and hydraulic characteristics. Four river segments (containing one or more reaches) were defined where bed-material characteristics were assumed constant (based on analysis of bed-material data) for the development of the transport relationships. These segments cover the following river miles: (1) RM 0.0 to 7.97, (2) RM 7.97 to 27.03, (3) RM 27.03 to 28.80, and (4) RM 28.80 to 30.65. Figure 4.1 shows the average bed-material size distributions developed for each segment. The size distributions were developed based on data collected by the USGS in 1983 through 1985. The procedures used to develop the relations are described in the Phase I report.

Unit width sediment discharges were computed using the Meyer-Peter, Muller bed-load equation and Einstein's method for computing suspended bed-material load for the anticipated range of hydraulic conditions. Multiple linear regressions were then performed to develop sediment-transport relationships of the form:

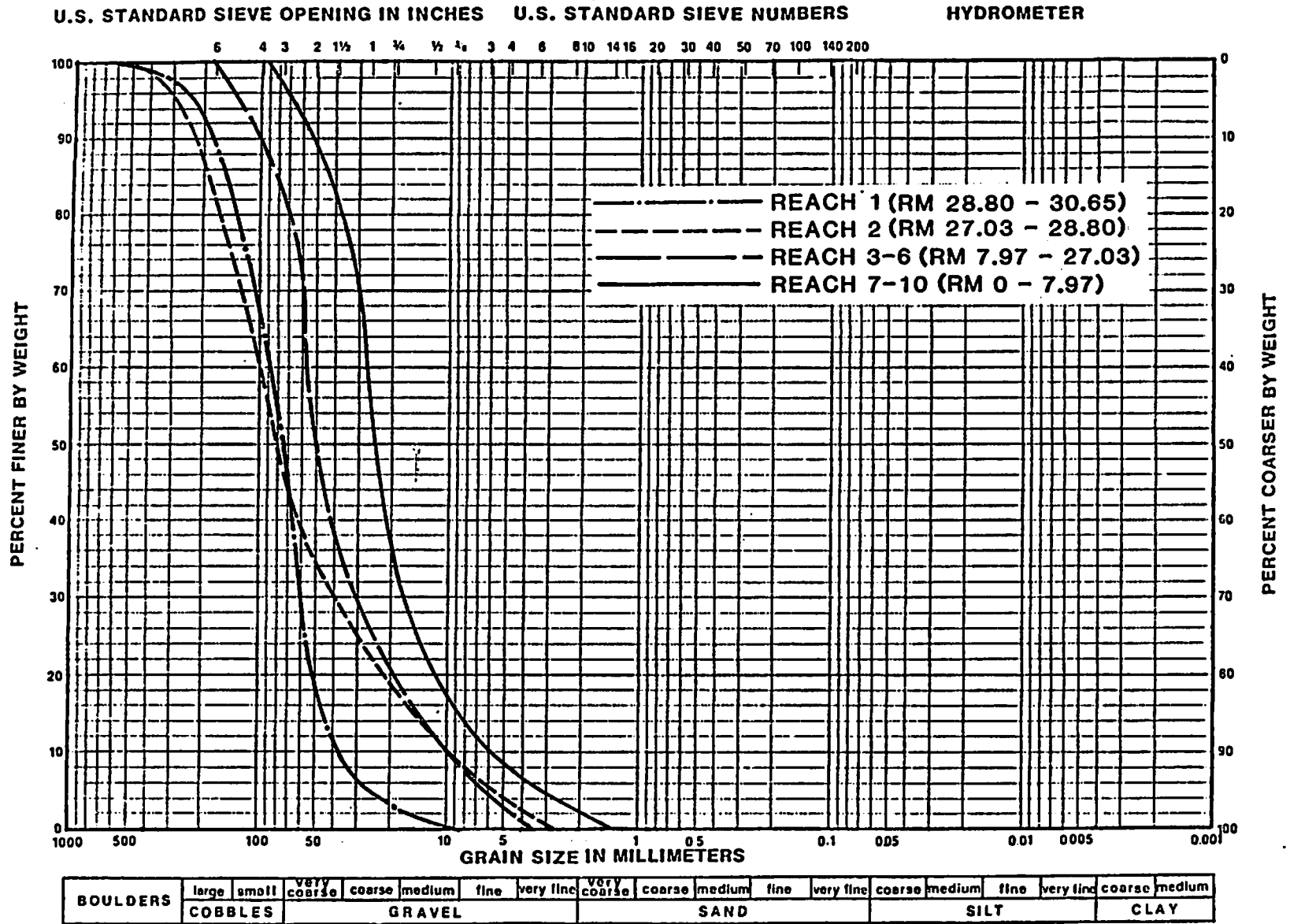
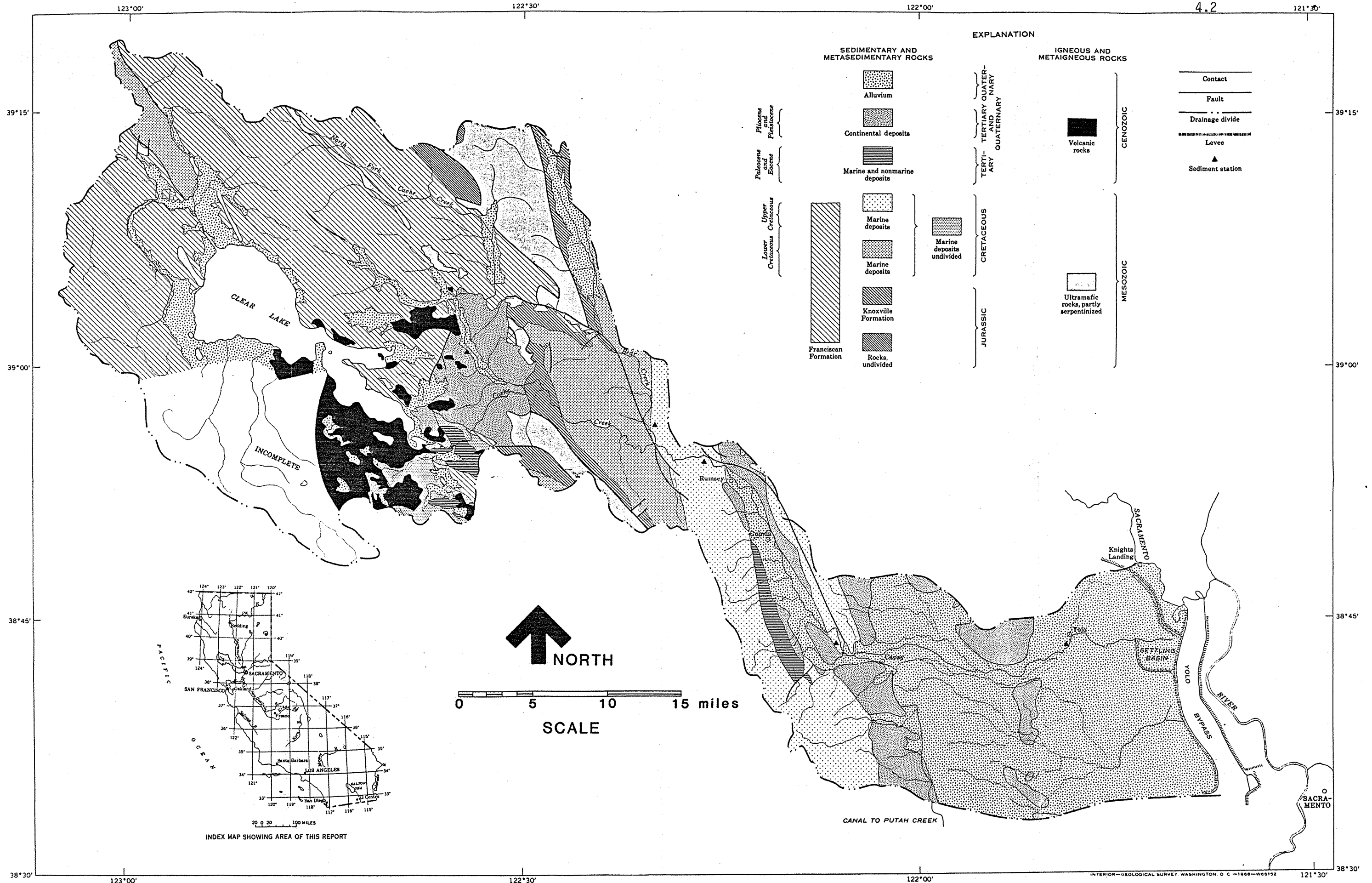


Figure 4.1. Average bed-material size distributions used to develop sediment-transport relationships along Cache Creek.



Base from Army Map Service 1:250 000 series: Ukiah, 1960; Sacramento, 1961; and Santa Rosa, 1964

Figure 4.1. Geologic map of Cache Creek basin (from Lustig and Busch, 1967).

$$q_s = aV^b d_H^c \quad (4.1)$$

where q_s = unit width bed-material sediment discharge (cfs/ft),
 V = average main channel velocity (ft/sec),
 d_H = hydraulic depth (ft), and
 a , b , and c are constants.

A description of the procedure is included in Appendix B. Table 4.1 presents the resulting equations. Two equations applicable over different velocity ranges were developed for Reach 1 to adequately define the relationship.

4.1.2 Reach Rating Curves

Using the sediment-transport relationships derived for Cache Creek in conjunction with average reach hydraulics computed in the hydraulic analysis, sediment-rating curves were developed for each reach. The sediment-rating curves were developed by first using average reach main-channel velocity and hydraulic depth as input to Equation 4.1 to compute unit bed-material sediment discharge for a range of discharges. Total bed-material sediment discharge for each discharge was computed by multiplying the unit width bed-material discharge by the channel topwidth. Power curves of the following form were then fit to the data for each reach:

$$Q_s = aQ^b \quad (4.2)$$

where Q_s equals total bed-material sediment discharge (cfs), Q equals water discharge (cfs), and a and b are constants. The constants for the sediment-rating curves developed for this study are summarized in Table 4.2. Two power equations applicable over different discharge ranges were developed for most reaches to adequately define the rating curve.

4.1.3 Calibration

To assess the accuracy of the computed sediment-transport relationships and sediment-rating curves, a comparison was made to actual measured sediment-transport data for the stream. Sediment-transport data have been collected by the USGS at two locations: above Rumsey and near Brooks (see Figure 1.1). The collected data are for total suspended load which includes both suspended bed-material load and wash load. The USGS estimated the total load by estimating the unmeasured load and applying an adjustment to the measured data. Since

Table 4.1. Derived Constants for the Sediment-Transport Relationships for Cache Creek, $q_s = aV_d^b c^c$.

| Reach | a | b | c | Velocity Range (ft/s) |
|-------|-----------------------|------|--------|-----------------------|
| 1 | 1.02×10^{-9} | 8.08 | -1.25 | 0 - 9 |
| | 9.58×10^{-8} | 5.87 | -0.985 | 9 - 20 |
| 2 | 9.02×10^{-7} | 4.96 | -0.714 | 0 - 20 |
| 3-6 | 4.78×10^{-7} | 5.26 | -0.861 | 0 - 20 |
| 7-10 | 8.38×10^{-7} | 5.02 | -0.797 | 0 - 20 |

Table 4.2. Derived Constants for the Sediment-Rating Curves for Cache Creek, $Q_s = aQ^b$.

| Reach | a | b | Discharge Range (cfs) |
|-------|------------------------|-------|-----------------------|
| 1 | 2.77×10^{-8} | 1.91 | 0 - 9,000 |
| | 1.11×10^{-4} | 1.00 | 9,000 - 55,000 |
| 2 | 1.12×10^{-6} | 1.39 | 0 - 12,000 |
| | 9.48×10^{-5} | 0.912 | 12,000 - 55,000 |
| 3 | 6.62×10^{-8} | 1.86 | 0 - 1,500 |
| | 1.03×10^{-5} | 1.17 | 1,500 - 55,000 |
| 4 | 1.49×10^{-6} | 1.35 | 0 - 3,200 |
| | 8.98×10^{-4} | 0.559 | 3,200 - 6,800 |
| | 1.65×10^{-6} | 1.27 | 6,800 - 55,000 |
| 5 | 6.49×10^{-6} | 1.14 | 0 - 55,000 |
| 6 | 3.46×10^{-6} | 1.24 | 0 - 2,500 |
| | 1.13×10^{-17} | 4.62 | 2,500 - 3,000 |
| | 1.42×10^{-6} | 1.43 | 3,000 - 55,000 |
| 7 | 4.26×10^{-7} | 1.57 | 0 - 2,000 |
| | 3.43×10^{-6} | 1.30 | 2,000 - 55,000 |
| 8 | 2.17×10^{-5} | 1.14 | 0 - 10,000 |
| | 2.11×10^{-6} | 1.39 | 10,000 - 55,000 |
| 9 | 1.36×10^{-4} | 0.955 | 0 - 55,000 |
| 10 | 4.03×10^{-7} | 1.67 | 0 - 5,000 |
| | 4.41×10^{-5} | 1.12 | 5,000 - 55,000 |

the sediment-transport relationships and sediment-rating curves developed for this study apply only for bed-material load, it is necessary to estimate the percentages of bed-material load and wash load in the measured data to allow direct comparison.

Wash load is typically assumed to be composed of material in the silt and clay size ranges (finer than 0.062 mm). However, by its strict definition, wash load is that portion of the transported material not found in significant quantities in the bed of the channel (Vanoni, 1977). Review of the basic bed-material data (collected by the USGS) used to develop the representative bed-material gradations for this study indicates that virtually no material finer than 0.125 mm is found in the bed. It was, therefore, assumed that wash load includes very fine sand and smaller material ($D < 0.125$ mm). Based on data presented in a 1977 office report prepared by SPK and USGS total suspended load data collected in 1983 through 1985, approximately 12 percent of the total suspended load is coarser than 0.125 mm (fine sand). It was therefore assumed that approximately 12 percent of the total load from the USGS data is bed-material load. For purposes of comparison, the total load curve at Rumsey based on the SPK 1977 office report and 1983 through 1985 USGS data, the bed-material load curve estimated as 12 percent of the total load and the bed-material rating curve developed for this study by SLA for the upstream supply reach are shown in Figure 4.2. Review of the figure indicates that, within the accuracy of the sediment measurements and calculation procedures, the developed equations reflect the relationship between bed-material transport capacity and discharge with sufficient accuracy to accomplish the goals of this study.

4.1.4 Sediment-Continuity Calculations

To perform the sediment-continuity analysis, flow-duration curves for the pre- and postproject conditions were developed for each reach. This was accomplished by averaging the flow-duration curves at Rumsey, Capay, and Yolo (see Section 2.3) based on the distance of the stations from the center of each reach. The sediment-rating curves and time period of simulation used to develop the flow-duration curves were then used to integrate the flow-duration curves to determine a volume of sediment transport for each reach. The difference between the volume of sediment supplied to each reach and its

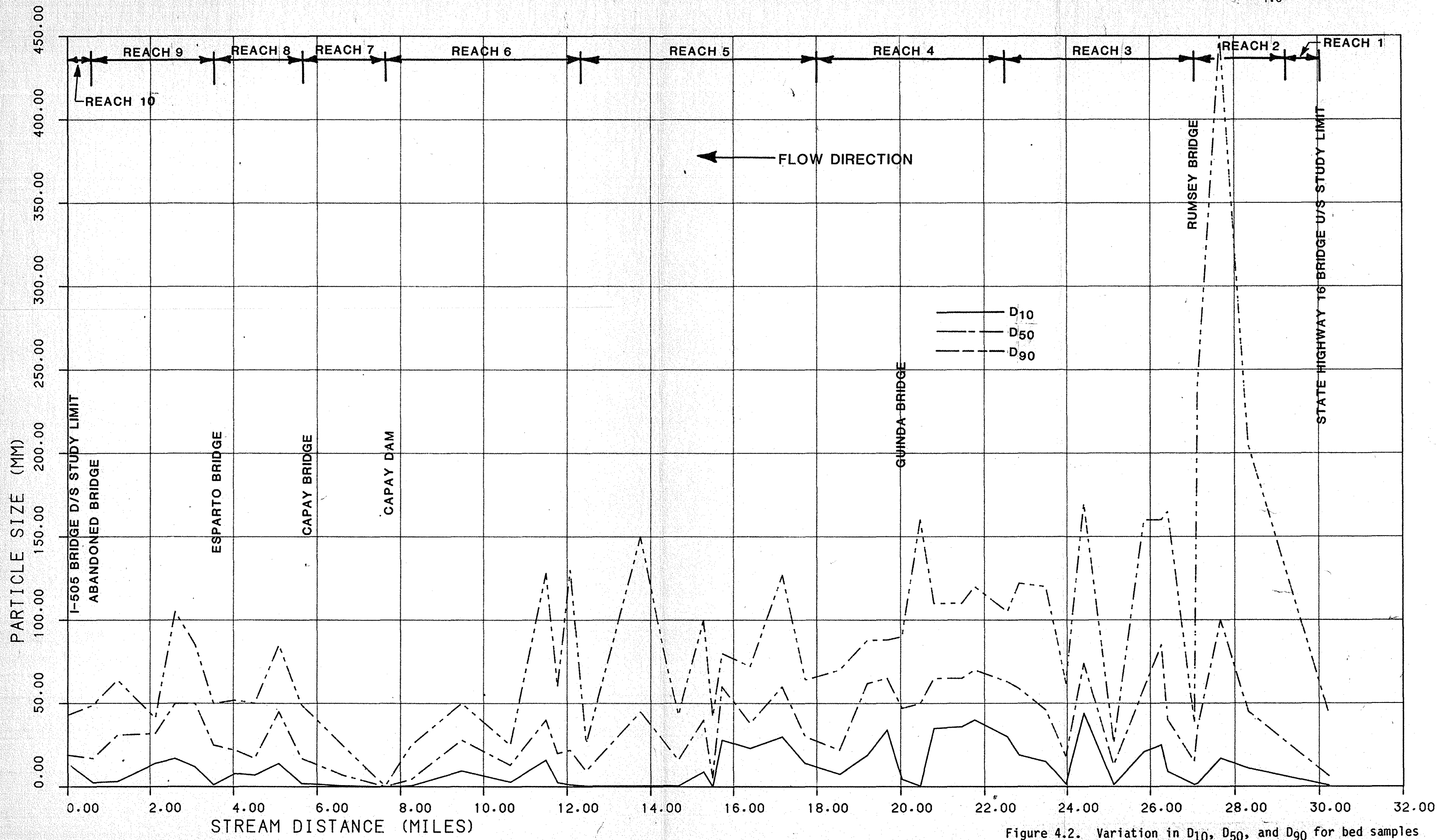


Figure 4.2. Variation in D₁₀, D₅₀, and D₉₀ for bed samples collected through the study reach

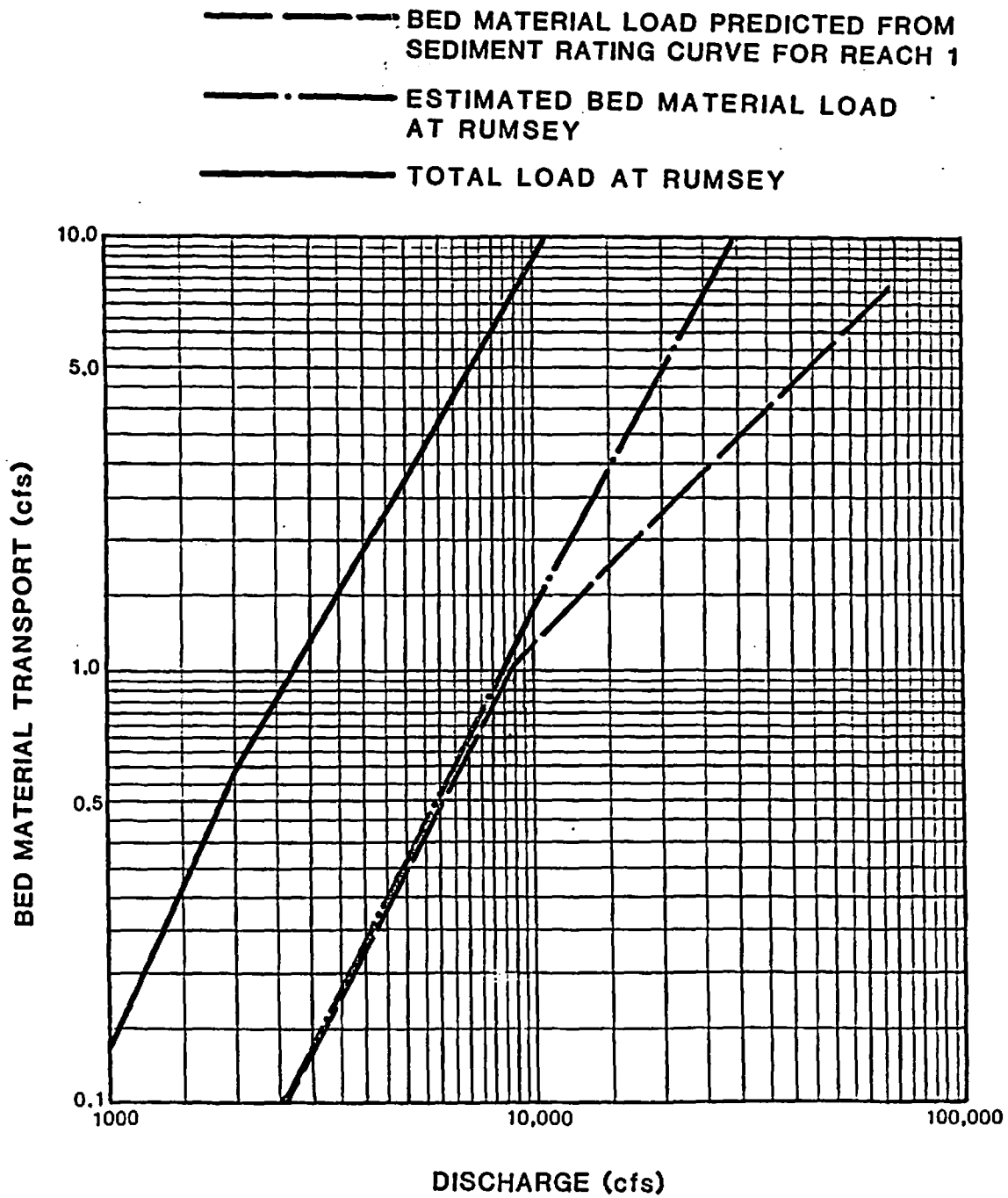


Figure 4.2. Comparison of bed-material load, derived from measured total load, and predicted bed-material load at the Rumsey gage.

transport capacity is the volume of erosion or sedimentation. Sediment supply from tributaries was neglected in the calculations due to a lack of adequate data to accurately estimate the inflow rate. Tributary inflows will be unaffected by the project and hence will not affect changes on the mainstem caused by the project. Upstream supply for the continuity calculations was determined using the transport from Reach 1 where no erosion/sedimentation volume was computed. The sediment-continuity analysis does not reflect changes in transport capacity due to changes in bed elevations, channel geometry, or bed-material gradation and hence should be used for qualitative analysis only.

Table 4.3 compares the computed pre- and postproject sediment-transport capacities for each reach for the entire period of simulated flows. The table shows that, in general, the project increases the sediment-transport potential of the stream with the largest increase in Reach 1 (8.8 percent) decreasing in the downstream direction. The transport capacity for the project conditions actually decreases in Reaches 9 and 10 which is likely due to less frequent very high flows in the downstream direction (see Section 2.3 and Figures 2.3 through 2.5).

Sediment-continuity results for pre- and postproject conditions for the entire period of simulated flows are given in Tables 4.4 and 4.5, respectively. Results for individual years are provided in Appendix C. Results given in Table 4.4 for the preproject conditions show aggradation in Reaches 2, 4, and 7 with degradation in the remaining reaches. Similar trends exist under project conditions with the aggradation/degradation volumes generally being slightly greater.

Table 4.6 provides a comparison of approximate erosion/sedimentation depths computed using the entire period of simulated flows. The depths given are average depths computed by dividing the bulked erosion/sedimentation volume by the reach length and average topwidth. Actual erosion/sedimentation depths may vary significantly within each reach. This was shown in the historic profile comparison given in the Phase I report where areas of both erosion and sedimentation occurred in each reach (a comparison to verify the model was thus not possible). The depths would also be limited by channel armoring which is not considered in the sediment-continuity analysis (see Section 4.1.5). Table 4.6 shows the project to generally increase erosion/sedimentation depths, but the increases are, in general, small. Reaches 3 and 9 actually show a slight decrease. The project will thus have

Table 4.3. Comparison of Pre- and Postproject Sediment-Transport Capacities Computed for the Entire Period of Simulated Flows.

| Reach | Preproject Transport Capacity (1,000 ft ³) | Postproject Transport Capacity (1,000 ft ³) | Percent Difference |
|-------|---|--|-----------------------|
| 1 | 19,730 | 21,468 | 8.8 |
| 2 | 8,325 | 8,733 | 4.9 |
| 3 | 11,646 | 11,956 | 2.7 |
| 4 | 5,793 | 5,857 | 1.1 |
| 5 | 5,802 | 5,916 | 2.0 |
| 6 | 17,031 | 18,044 | 5.9 |
| 7 | 13,583 | 14,194 | 4.5 |
| 8 | 20,706 | 21,447 | 3.6 |
| 9 | 25,796 | 25,433 | -1.4 |
| 10 | 34,146 | 33,984 | -0.5 |

Table 4.4. Sediment-Continuity Results for the Entire
Period of Simulated Flows, Preproject Conditions.

| Reach | Upstream Supply (1,000 ft ³) | Transport Capacity (1,000 ft ³) | Erosion/Sedimentation* Volume (1,000 ft ³) |
|-------|--|---|--|
| 1 | -----Supply Reach----- | | |
| 2 | 19,730 | 8,325 | 11,405 |
| 3 | 8,325 | 11,646 | -3,321 |
| 4 | 11,646 | 5,793 | 5,583 |
| 5 | 5,793 | 5,802 | -9 |
| 6 | 5,802 | 17,031 | -11,229 |
| 7 | 17,031 | 13,583 | 3,448 |
| 8 | 13,583 | 20,706 | -7,123 |
| 9 | 20,706 | 25,796 | -5,090 |
| 10 | 25,796 | 34,146 | -8,350 |

* Negative numbers denote erosion.

Table 4.5. Sediment-Continuity Results for the Entire
Period of Simulated Flows, Project Conditions.

| Reach | Upstream Supply (1,000 ft ³) | Transport Capacity (1,000 ft ³) | Erosion/Sedimentation* Volume (1,000 ft ³) |
|-------|--|---|--|
| 1 | -----Supply Reach----- | | |
| 2 | 21,468 | 8,743 | 12,734 |
| 3 | 8,734 | 11,956 | -3,222 |
| 4 | 11,956 | 5,857 | 6,099 |
| 5 | 5,857 | 5,916 | -59 |
| 6 | 5,916 | 18,044 | -12,128 |
| 7 | 18,044 | 14,194 | 3,850 |
| 8 | 14,194 | 21,447 | -7,253 |
| 9 | 21,447 | 25,433 | -3,986 |
| 10 | 25,433 | 33,984 | -8,551 |

* Negative numbers denote erosion.

Table 4.6. Comparison of Approximate Pre- and Postproject Erosion/Sedimentation Depths Computed for the Entire Period of Simulated Flows.

| Reach | Preproject* Depth (ft) | Postproject* Depth (ft) | Difference** (ft) |
|-------|------------------------------|-------------------------------|----------------------|
| 1 | -----Supply Reach----- | | |
| 2 | 7.66 | 8.55 | 1.12 |
| 3 | -0.94 | -0.91 | -0.03 |
| 4 | 1.08 | 1.12 | 0.04 |
| 5 | -0.001 | -0.008 | 0.007 |
| 6 | -4.90 | -5.30 | 0.4 |
| 7 | 2.26 | 2.52 | 0.26 |
| 8 | -1.27 | -1.29 | 0.02 |
| 9 | -0.79 | -0.61 | -0.18 |
| 10 | -4.47 | -4.58 | 0.11 |

* Negative numbers denote erosion.

** Difference in absolute magnitude of computed erosion/sedimentation depths. A negative number denotes a decrease in magnitude for project conditions.

some impact on erosion/sedimentation trends along the study reach, but this impact appears small.

4.1.5 Armoring Calculations

The sediment-continuity analysis given above is based solely on differences in computed sediment transport and does not take into account armoring which may limit the amount of degradation at a particular location. Armoring is the process in an alluvial channel where the bed material is sorted with finer material removed by transport leaving behind the coarser material. Eventually, enough coarse particles accumulate on the surface to shield, or "armor" the bed, preventing further degradation. An armor layer sufficient to protect the bed against moderate discharges can be disrupted during high flow, but may be restored as flows diminish.

The armoring potential for reaches determined to be degradational in the sediment-continuity analysis was computed using Shields criteria for incipient motion and a representative bed-material composition. Shields criteria for determining incipient motion is given by:

$$D_c = \frac{\tau}{0.047 (\gamma_s - \gamma)} \quad (4.3)$$

where D_c is the diameter of the sediment particle at incipient motion, τ is the boundary shear stress, γ_s and γ are the specific weights of sediment and water, respectively, and 0.047 is a dimensionless coefficient referred to as the Shields parameter. Equation 4.3 was used to establish the incipient particle size for each reach for a given discharge. The depth of scour necessary to establish an armor layer (ΔZ_a) was then calculated from (USBR, 1984):

$$\Delta Z_a = y_a \left(\frac{1}{p_c} - 1 \right) \quad (4.4)$$

where y_a is the thickness of the armor layer and p_c is the fraction of material coarser than the armor size (determined from the representative bed-material gradation). The thickness of the armor layer was taken to equal twice the diameter of the incipient particle size (SLA, 1982).

The armoring potential of the degradational reaches was computed for discharge levels which are exceeded 50 and 10 percent of the time (determined from flow-duration analysis, Section 2.3). The 50 percent exceedence discharge level was chosen to compute the armor depths that would develop under near-average flow conditions and the 10 percent exceedence discharge level was chosen to compute armor depths that would develop under high-flow conditions. The results are given in Tables 4.7 and 4.8, which show the necessary amount of degradation to establish an armor layer at the given discharge levels for both pre- and postproject conditions. The results show that significant armoring potential exists at these discharge levels for all reaches since the armor depths are relatively small compared to degradation depths computed in the sediment-continuity analysis. Degradation may not be limited to the depths given in Tables 4.7 and 4.8, however. These depths were computed using average reach velocities which may be significantly exceeded in local areas resulting in somewhat greater degradation potential. The armor layer that does form under the given discharges may also be broken up during times of very high flows.

4.1.6 Discussion

The sediment-continuity analysis given in this section shows that the proposed Clear Lake flood-control project will have some impact on sediment-transport characteristics of the study reach. The impact is insignificant, showing, in general, slightly increased sediment-transport capacity and slightly increased erosion/sedimentation trends. The analysis has also shown that the computed degradation potential will be limited by armoring under both pre- and postproject conditions.

4.2 Bank Erosion

4.2.1 General

Bank erosion along the study reach has been and is occurring. It is a significant and continuing problem. The large flood flows which occurred in 1983 dramatically illustrated the problem, causing extensive bank loss at numerous locations. The quantification of potential impacts of the proposed project on this situation is therefore of prime importance.

Table 4.7. Armoring Potential for Degradational Reaches at a Discharge Level Equaled or Exceeded 50 Percent of the Time, Pre- and Postproject Conditions.

| Reach | Preproject Conditions | | | | Postproject Conditions | | | |
|-------|-----------------------|-----------------|------------|--------------------------------|------------------------|-----------------|-------|------------------------------|
| | Discharge (cfs) | D_c^* (mm) | p_c^{**} | Degradational Depth (ft) | Discharge (cfs) | D_c^* (mm) | p_c | Depth at Armoring (ft) |
| 3 | 4,000 | 22.9 | 0.76 | 0.05 | 3,200 | 20.2 | 0.78 | 0.04 |
| 5 | 4,100 | 13.2 | 0.86 | 0.01 | 3,300 | 12.5 | 0.87 | 0.01 |
| 6 | 4,200 | 29.8 | 0.71 | 0.08 | 3,400 | 26.7 | 0.74 | 0.06 |
| 8 | 4,300 | 13.2 | 0.80 | 0.02 | 3,500 | 12.5 | 0.80 | 0.02 |
| 9 | 4,300 | 11.2 | 0.82 | 0.02 | 3,500 | 10.6 | 0.82 | 0.02 |
| 10 | 4,300 | 23.8 | 0.54 | 0.14 | 3,500 | 21.1 | 0.58 | 0.10 |

* Diameter of partical at incipient motion (armor size).

** Fraction of material coarser than armor size.

Table 4.8. Armoring Potential for Degradational Reaches at a Discharge Level Equaled or Exceeded 10 Percent of the Time, Pre- and Postproject Conditions.

| Reach | Preproject Conditions | | | | Postproject Conditions | | | |
|-------|-----------------------|--------------|------------|--------------------------|------------------------|--------------|-------|------------------------|
| | Discharge (cfs) | D_c^* (mm) | p_c^{**} | Degradational Depth (ft) | Discharge (cfs) | D_c^* (mm) | p_c | Depth at Armoring (ft) |
| 3 | 8,200 | 28.7 | 0.71 | 0.08 | 11,500 | 33.0 | 0.68 | 0.10 |
| 5 | 8,800 | 19.4 | 0.79 | 0.03 | 11,800 | 22.0 | 0.78 | 0.04 |
| 6 | 9,800 | 47.5 | 0.52 | 0.29 | 11,500 | 51.5 | 0.48 | 0.37 |
| 8 | 8,800 | 16.2 | 0.75 | 0.04 | 11,500 | 18.6 | 0.66 | 0.06 |
| 9 | 9,000 | 14.7 | 0.76 | 0.03 | 11,800 | 17.7 | 0.70 | 0.05 |
| 10 | 9,000 | 38.7 | 0.18 | 1.14 | 11,800 | 46.2 | 0.14 | 1.84 |

* Diameter of partical at incipient motion (armor size).

** Fraction of material coarser than armor size.

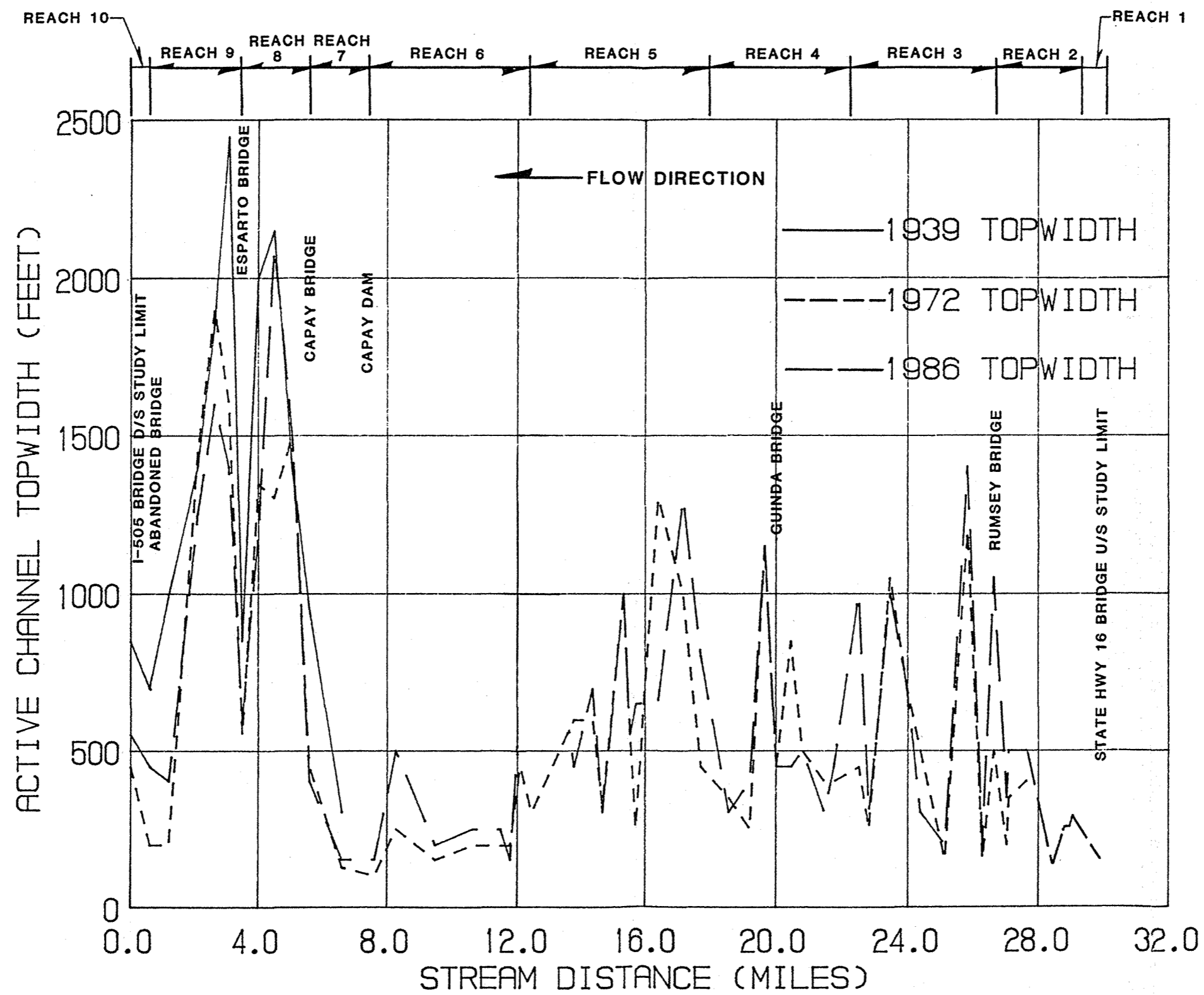


Figure 4.9. Variation in active channel topwidth along the study reach.

Materials found in banks along the study reach consist of medium- to fine-sized sand with 10 to 20 percent silts and clays (determined from the site visit and bank-material gradations provided by the USGS). Lenses of coarse gravel and clay are located along banks throughout the study reach. The height of banks vary with location. Generally, bank heights are in the range of 5 to 15 feet. In some locations, the river flows along the base of a hillslope; in these areas bank heights were seen to vary from 20 to 100 feet.

Typically, the sideslopes of banks in the study area are composed of a lower 2 horizontal to 1 vertical (2H:1V) section capped by a nearly vertical layer. Material exposed in the upper layer consists of the fine sediments previously described. Material in the lower section is composed of eroded upper bank material armored by and interspersed with cobbles and gravels. A definition sketch of this typical bank sideslope is shown on Figure 4.3.

In view of the general configuration and composition of banks along the study reach, the predominant cause of bank erosion is believed to be due to flows acting on the fine-grained material located in the vertical bank face. In low-flow situations, the armored debris piles located along the toe of most banks are believed to buttress the vertical slope and prevent flows from acting directly on the bank. In high-flow situations, the debris piles can be overtopped. This may allow high velocity flows to act on the fine-grained upper bank, saturation of upper bank materials, disruption of armor on debris piles, and erosion of material buttressing the toe of the vertical slope.

Other localized causes of bank failure which have been noted along the stream include gulying from concentrated overland runoff, flow acceleration caused by vegetative debris blockages, and slumps caused by groundwater at interfaces between materials with different permeabilities. Gulying was noted at numerous locations along the study reach during field reconnaissance for the Phase I report. Gulying was seen to occur wherever concentrated overland runoff was discharging over banks. Flow accelerations due to debris blockages have been noted in locations where trees or brush have lodged to accelerate and impinge on the adjacent bank. In certain locations, the fine-grained bank material was seen to overlie clay outcrops. In many of these areas, erosion of the bank is believed to be due to piping of the fine-grained material by groundwater flowing at the interface between the materials of significantly different permeability. The project is not expected to impact any of these localized causes of bank erosion.

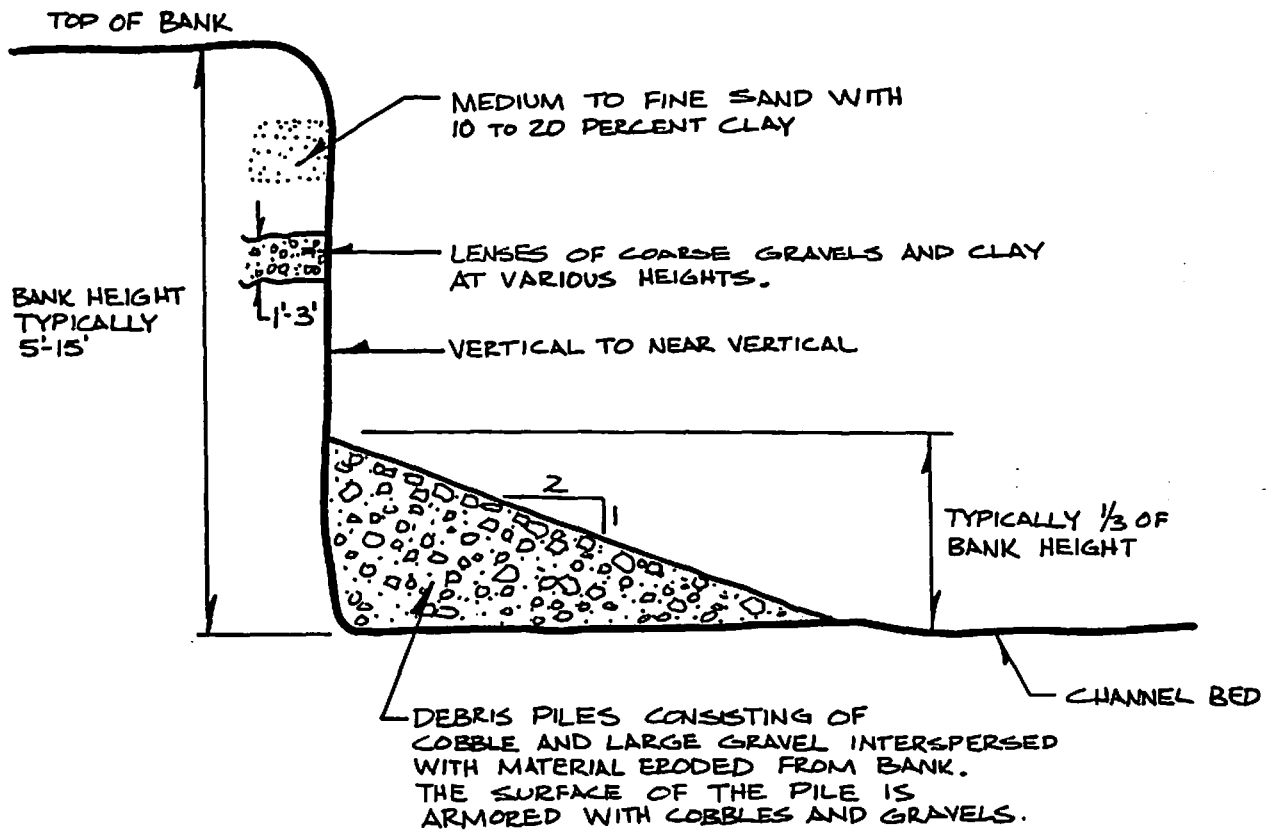


Figure 4.3. Typical bank configuration.

4.2.2 Bank Erosion Analysis

A quantitative analysis of the effects of the project on bank erosion is presented in this section. The analysis examines the change in frequency of flows between preproject and project conditions causing bank erosion and bank overtopping. A quantitative measure of the change in bank erosion potential is also presented.

To perform the bank erosion analysis representative sites throughout the study reach were chosen. The sites were chosen at representative bends in the channel throughout the study reach where bank erosion is known to be occurring. Table 4.9 lists the locations for which the analysis was performed. The locations are shown in Figure 4.4.

4.2.3 Frequency of Overtopping Armor and Banks

The first step in the analysis of bank erosion was to determine the frequency of flows causing bank erosion. This was determined by first noting that large cobbles exist at the toe of most banks, providing an "armor" against erosion (see Section 4.2.1, Figure 4.3). This armor must be overtopped before bank erosion can occur. Cross section plots, field notes, and photographs were used to estimate the elevation of the top of the armor layer at each cross section. The discharge level at which this occurs was then determined from the hydraulic analysis.

Table 4.10 gives the computed discharge level at which overtopping of bank armor occurs at each cross section used in the bank erosion analysis. The percent of time these discharges are exceeded under pre- and postproject conditions for the entire period of simulated flows is also provided (determined from flow-duration analysis, see Section 2.3). This indicates the percentage of time bank erosion is possible at each location. Table 4.10 shows that at most sections, bank erosion will occur less frequently under project conditions. This occurs as the discharge levels just overtopping the bank armor are generally less than 5,000 to 6,000 cfs, below which preproject flows are more frequent (see Figures 2.3 through 2.5). Banks that require flows above this range to overtop the bank armor generally show an increased frequency of overtopping under project conditions. The duration of flows overtopping the bank armor does not address the magnitude of bank erosion, which also depends on the magnitude of flows that overtop the armor. This is addressed in Section 4.2.4.

Table 4.9. Cross Sections Used in the Bank Erosion Analysis.

| Cross Section | River Mile | Reach*** | Bank* |
|---------------|------------|----------|-------|
| 0.9 | 5.1 | 8 | L |
| 2.0 | 6.6 | 7 | R |
| 4.0 | 8.2 | 6 | L |
| 7.0 | 10.6 | 6 | L |
| 10.0** | 12.1 | 5 | R |
| 13.0 | 14.3 | 5 | R |
| 17.0 | 16.4 | 5 | L |
| 20.0 | 18.5 | 4 | L |
| 22.0 | 19.7 | 4 | R |
| 24.0** | 20.5 | 4 | R |
| 26.0 | 21.4 | 4 | L |
| 28.0 | 22.5 | 4 | R |
| 30.0** | 23.5 | 3 | R |
| 35A** | 26.6 | 3 | L |
| 36A** | 27.1 | 2 | R |
| 37.0** | 27.6 | 2 | L |
| 38.0** | 28.3 | 2 | L |
| 40.0 | 29.3 | 1 | L |

* L = Left bank, looking downstream
R = Right bank, looking downstream

** USGS active erosion sites

*** See Figure 3.1 and Table 3.3.

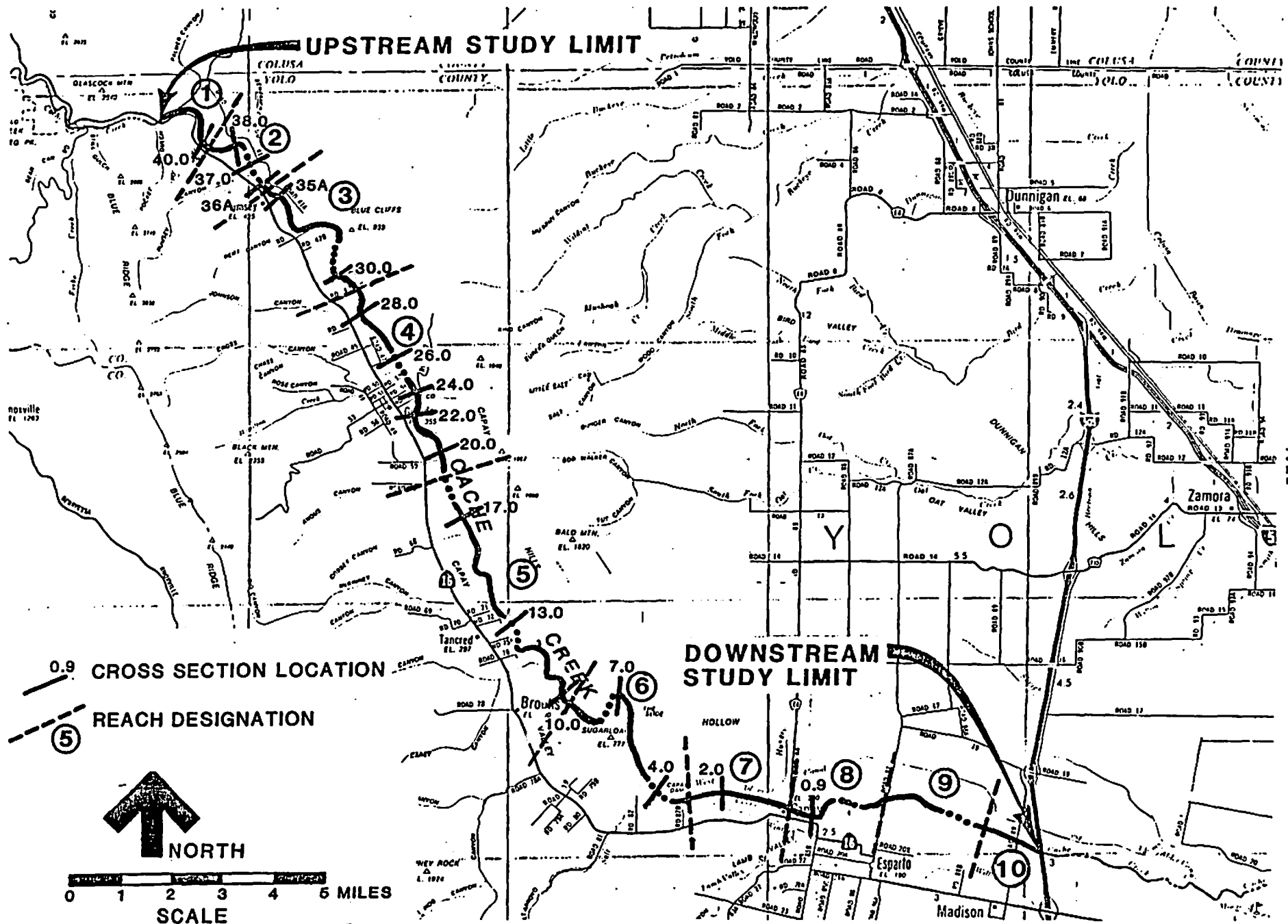


Figure 4.4. Cross section locations used in the bank erosion analysis.

Table 4.10. Frequency of Overtopping Bank Toe Armor Considering Entire Period of Simulated Flows, Pre- and Postproject Conditions.

| Cross Section | Bank Toe Armor Height (ft) | Discharge** (cfs) | Percent Time Exceeded | |
|---------------|----------------------------|-------------------|-----------------------|-------------|
| | | | Preproject | Postproject |
| 0.9 | 8.9 | 35,500 | 0.10 | 0.07 |
| 2.0 | 3.1 | 2,500 | 77.2 | 62.1 |
| 4.0 | 1.6 | 3,500 | 64.4 | 48.3 |
| 7.0 | 3.1 | 750 | 92.4 | 89.2 |
| 10.0* | 4.1 | 3,000 | 73.6 | 54.5 |
| 13.0 | 9.2 | 6,250 | 23.6 | 25.2 |
| 17.0 | 7.0 | 6,500 | 21.8 | 24.1 |
| 20.0 | 3.1 | 2,500 | 76.4 | 60.4 |
| 22.0 | 8.5 | 6,000 | 24.3 | 25.5 |
| 24.0* | 3.2 | 1,000 | 89.6 | 84.4 |
| 26.0 | 7.8 | 7,000 | 17.5 | 21.3 |
| 28.0 | 5.3 | 4,000 | 50.8 | 40.0 |
| 30.0* | 8.9 | 8,000 | 12.0 | 17.4 |
| 35A* | 6.1 | 2,000 | 79.8 | 67.3 |
| 36A* | 6.4 | 5,000 | 32.6 | 31.7 |
| 37.0* | 4.0 | 3,000 | 72.4 | 51.8 |
| 38.0* | 3.8 | 1,500 | 84.5 | 75.2 |
| 40.0 | 2.8 | 2,000 | 79.5 | 66.8 |

* USGS active erosion sites

** Discharge at which bank toe armor is overtopped.

Although not directly related to bank erosion, overtopping the top of the bank is also of interest in analyzing the impacts of the project. Table 4.11 gives the computed discharge levels and frequencies at which bank overtopping occurs for the entire period of simulated flows. The table shows sections where banks are both more and less frequently overtopped under project conditions. Sections where bank overtopping is less frequent under project conditions require very high discharges to be overtopped. These high discharges fall on the flow-duration curves in the area where preproject flows are slightly more frequent (see Section 2.3 and Figures 2.3 through 2.5).

4.2.4 Shear Stress Duration Analysis

To assess potential change in the rate of bank erosion resulting from the project an approach suggested by Professor John F. Kennedy of the University of Iowa was utilized. The approach relates the erosion potential of the flow to the product of the shear stress acting on the bank and duration of the flow producing it. The flow-duration curves were utilized to determine this quantity at each bank erosion site for pre- and postproject conditions. The condition that produces the highest erosion potential calculated using this approach is expected to produce the largest amount of bank erosion.

The bank erosion potential is computed as follows:

$$I = \sum_{i=1}^N \tau t \quad (4.5)$$

where I is defined as the bank erosion potential index, τ is the total shear stress acting on the bank, t is the duration of flow, and N is the number of flows. The total shear stress acting on the banks is given by

$$\tau = \sqrt{\tau_0^2 + \tau_r^2} \quad (4.6)$$

where τ_0 is the bed shear stress and τ_r is the transverse shear stress produced by a bend in the channel (see Figure 4.5). Bed shear stress is given by

$$\tau_0 = \frac{1}{8} f \rho V^2 \quad (4.7)$$

where f is the Darcy-Weisbach friction factor, ρ is the density of water and V is the mean velocity in the main channel. The transverse bed shear stress is given by Zimmerman and Kennedy (1978):

Table 4.11. Frequency of Overtopping Bank Considering Entire Period of Simulated Flows, Pre- and Postproject Conditions.

| Cross Section | Bank Height (ft) | Discharge** (cfs) | Percent Time Exceeded | |
|---------------|------------------|-------------------|-----------------------|-------------|
| | | | Preproject | Postproject |
| 0.9 | 11.7 | 55,000 | 0.0*** | 0.0*** |
| 2.0 | 15.8 | 25,500 | 0.35 | 0.25 |
| 4.0 | 8.8 | 12,500 | 4.4 | 8.7 |
| 7.0 | 13.8 | 26,000 | 0.32 | 0.27 |
| 10.0* | 14.6 | 23,500 | 0.45 | 0.42 |
| 13.0 | 14.0 | 23,750 | 0.42 | 0.38 |
| 17.0 | 11.0 | 20,000 | 0.85 | 1.9 |
| 20.0 | 11.5 | 8,500 | 10.6 | 16.5 |
| 22.0 | 21.2 | 47,500 | 0.015 | 0.015 |
| 24.0* | 11.6 | 16,500 | 1.2 | 3.7 |
| 26.0 | 11.5 | 17,500 | 1.4 | 3.2 |
| 28.0 | 10.1 | 21,500 | 0.50 | 0.60 |
| 30.0* | 12.9 | 27,000 | 0.20 | 0.18 |
| 35A* | 17.4 | 27,000 | 0.20 | 0.18 |
| 36A* | 14.9 | 11,000 | 5.1 | 11.0 |
| 37.0* | 19.4 | 24,500 | 0.28 | 0.28 |
| 38.0* | 17.4 | 40,500 | 0.035 | 0.035 |
| 40.0 | 14.8 | 17,500 | 1.2 | 3.1 |

* USGS active erosion sites

** Discharge at which channel bank is overtopped.

*** No flows in simulation used to develop flow-duration curves exceeded this discharge value.

$$\tau_r = \frac{1}{3} \frac{(n+1)}{n(n+2)} \frac{d}{r_c} \rho V^2 \quad (4.8)$$

where n is the exponent on the power law velocity profile formula, d is the depth of flow, and r_c is the radius of curvature of the channel bend measured at the channel centerline. The exponent n is given by

$$n = K \sqrt{\frac{8}{f}} \quad (4.9)$$

where K is the von Karman constant equal to 0.4 and f is the Darcy-Weisbach friction factor.

The shear stress duration analysis was performed by integrating the flow-duration curves using Equation 4.5 for flows that exceed the bank armor height as given in Table 4.10. The results of the analysis considering the entire period of simulated flows for pre- and postproject conditions is given in Table 4.12. Results for individual years are given in Appendix D. The results show that at most sections, bank erosion potential will be increased by the project. The increases occur because the erosion potential of the more frequent higher flows under project conditions more than offsets the shorter overall duration of the higher flows under these conditions. Bank locations at Cross Sections 0.9, 10.0, and 37.0, however, show a decrease in erosion potential under project conditions. At these sections, the combined effects of shorter durations of erosion (Table 4.9) and the less duration of the highest flows (Section 2.3) dominate.

The results of the shear stress duration analysis given in Table 4.12 indicate that the forces causing bank erosion will in general increase under project conditions. The actual increase in bank erosion will not necessarily be as great as the computed increased in erosion potential, however. The analysis does not consider the resistance of the bank material to erosion which may limit the amount of erosion that will actually occur. This is illustrated by examining the USGS active erosion sites. The site at Section 35A has experienced relatively little erosion while a relatively large bank erosion potential index was computed. The site at Section 10 has experienced relatively high erosion although a relatively low bank erosion potential index was computed.

Table 4.12. Potential Bank Erosion Computed Using Entire Period of Simulated Flows, Pre- and Postproject Conditions.

| Cross Section | Bank Erosion Potential Index* | | Percent Difference |
|---------------|-------------------------------|-------------|--------------------|
| | Preproject | Postproject | |
| 0.9 | 29.0 | 20.3 | -30.0 |
| 2.0 | 5,028 | 5,662 | 12.6 |
| 4.0 | 3,555 | 3,928 | 10.5 |
| 7.0 | 33,999 | 36,846 | 8.4 |
| 10.0** | 3,726 | 2,602 | -30.2 |
| 13.0 | 2,491 | 3,136 | 25.9 |
| 17.0 | 2,702 | 3,551 | 31.4 |
| 20.0 | 2,908 | 3,089 | 6.3 |
| 22.0 | 2,587 | 3,209 | 24.0 |
| 24.0** | 16,941 | 17,040 | 0.6 |
| 26.0 | 1,311 | 1,879 | 43.3 |
| 28.0 | 2,667 | 2,815 | 5.6 |
| 30.0** | 1,101 | 1,719 | 56.1 |
| 35A** | 26,027 | 29,002 | 11.4 |
| 36A** | 1,421 | 1,696 | 19.3 |
| 37.0** | 3,363 | 2,950 | -12.3 |
| 38.0** | 15,769 | 16,455 | 4.4 |
| 40.0 | 5,344 | 5,529 | 3.5 |

* Larger number indicates higher erosion potential.

** USGS active erosion sites

4.3 Channel Form

Cache Creek along the study reach is a dynamic stream with significant bank erosion. To determine qualitatively the potential impact of the project on channel form, relationships developed by Schumm (1977) were utilized. These relationships relate changes in dominant discharge and bed load to changes in morphologic variables. Assuming that the mean annual flood represents the dominant discharge, this variable will increase under project conditions (see Phase I report, Figure 3.4). With a general increase in sediment-transport capacity and bank erosion, bed-material load can be expected to increase. Schumm's relationship for an increase in both dominant discharge and bed-material load is given as follows (Schumm, 1977):

$$Q^+ Q_s^+ \sim b^+ d^+ \lambda^+ S^- P^- F^+ \quad (4.10)$$

where Q^+ = increase in dominant discharge

Q_s^+ = increase in bed-material load

b^+ = increase in channel width

d^+ = unknown change in channel depth

λ^+ = increase in meander wavelength

S^- = unknown change in channel slope

P^- = decrease in channel sinuosity

F^+ = increase in width/depth ratio

These changes indicate a potential for the channel to become wider and less sinuous as a result of the project. Similar changes have presently occurred on the lower reaches of the study area where gravel mining activities have artificially widened and to some degree straightened the stream. These areas of the channel appear closer to a braided pattern with the formation of islands and a wider active channel topwidth. At the present time, the channel in this area is oversized, resulting in deposition of sediment carried to the area from upstream reaches. The changes brought on by gravel mining activities are probably much greater than those that will result from the proposed project.

4.4 Discussion

The sediment-continuity analysis given in Section 4.1 shows that the proposed Clear Lake flood-control project will have some impact on the sediment-transport characteristics of the study reach. Sediment-transport capacity will, in general, increase with decreasing impact in the downstream direction. Erosion/sedimentation potential will, in general, increase slightly with two reaches (Reaches 3 and 9) showing a slight decrease. The analysis also shows that significant armoring potential of the channel bed exists under both pre- and postproject conditions limiting the amount of bed degradation that will occur. It can thus be concluded that the project will have little impact on the channel profile.

The bank erosion analysis given in Section 4.2 shows that although the duration of potentially erosive flows will be reduced by the project, the actual potential for bank erosion is, in general, increased. This is due to the more frequent higher flows that will exist under project conditions.

As stated in Section 2.3, the flow-duration curves used in the sediment-continuity and bank erosion analysis cover only the high runoff season and are biased towards seasons with peak flows exceeding 10,000 cfs. Analysis of gaging records along the study reach shows that the average annual runoff for the 8 years used to develop the flow-duration curves is approximately twice the average annual runoff for the period of record. The bias included in the analysis by only considering these wet years results in a conservative estimate of project impacts. The bias is of little importance in the sediment-continuity analysis since insignificant project impacts were shown. The bias is of importance in the bank erosion analysis, however, since this analysis did show more significant impacts. No quantitative method of adjusting for this bias is available without a simulation of project changes on flows for the lower flow years.

The computed bank erosion index values represent cumulative values for the 8 high-flow years and do not represent values for average annual conditions. Average annual values of the index would be considerably less considering the entire period of simulation than the average annual values computed for the 8 high-flow years. The cumulative values of the index considering the entire period of record would likely be nearly the same, however, since the lower flow periods not included in the 8 years of simulation would add little to the cumulative values. This can be illustrated by comparing the

flow-duration curves developed for the period of record with those developed for the 8 high-flow years. Such a comparison is given in Figure 4.6 for the Rumsey gage (period of record 1961-1983). It should be noted that the curves cannot be directly compared since the period of record curve was developed for mean daily flows while the 8-year simulation includes bihourly data. Also, the high-flow curves for years prior to 1974 have been adjusted to simulate the operation of Indian Valley Reservoir while the period of record curves were developed from actual flow data. Qualitative comparison, however, shows that the percent of time a given discharge is exceeded is considerably less for the period of record compared to the 8-year flow period. Examination of the flow-duration curve for the period of record indicates that the percent of time that flows actually exceed the critical flow to overtop the armor and cause bank erosion is very small. Thus, it is likely that little additional cumulative bank erosion potential would be computed considering the entire simulation period. The analysis of impacts using only the 8 high-flow years is, therefore, believed to be at least qualitatively correct.

Both the sediment-continuity and bank erosion analyses are based on the historical flood routings that cover the runoff seasons with peak flows greater than 10,000 cfs. The analysis thus reflects long-term changes over several runoff seasons. The shorter term impacts depend on the occurrence of large flood events. If no erosion-producing flood event occurs, then bank erosion impacts due to the project are negligible. If a season with large flooding occurs, the potential for short-term impacts is increased.

The sediment-continuity analysis showed insignificant project impacts on all reaches. The bank erosion analysis showed increased bank erosion potential at representative cross sections throughout the study areas. The calculations were very site-specific reflecting local variations in cross-sectional shape and bend geometry. No generalizations could be made concerning increased bank erosion potential on a reach basis.

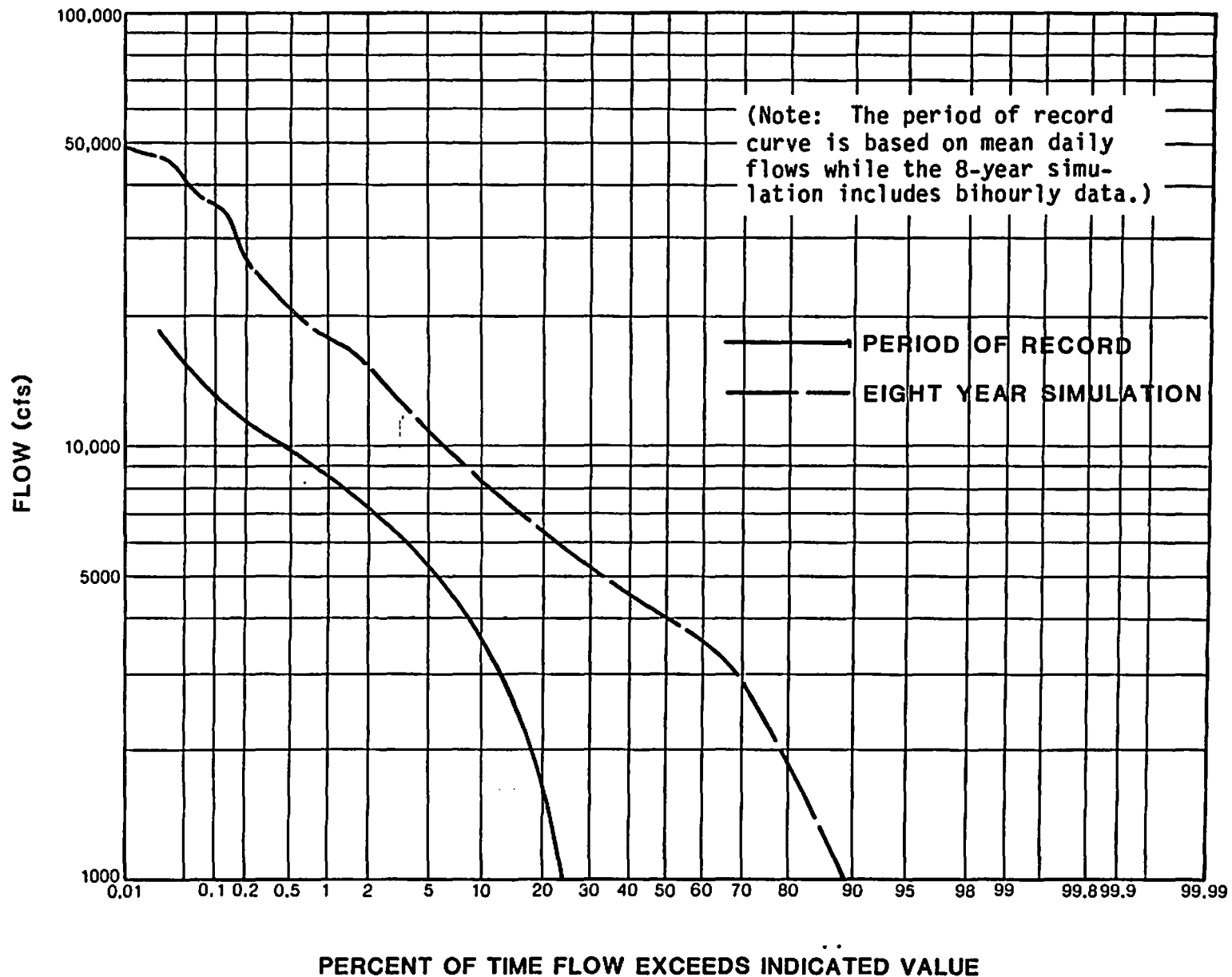


Figure 4.6. Comparison of flow-duration curves developed at the Rumsey gage, period of record versus 8-year simulation.

V. CHANNEL STABILIZATION TECHNIQUES

Bank erosion has been and will continue to be a significant problem along Cache Creek. This chapter reviews general types of protection measures which might be utilized to control bank erosion in the study reach. The suggested measures include various levels of sophistication. The applicability and design of bank protection measures for a particular location should be determined by a qualified engineer. Construction of bank protection measures may require permitting by local, state, and federal regulatory agencies.

5.1 Causes of Bank Erosion

The primary cause of bank erosion along the Cache Creek study reach has been described in previous chapters to be related to the influence of flood flows which overtop debris piles located at the toe of most banks. The mechanisms of bank erosion which occur during these events are complex and often-times interrelated. They include direct erosion of sediments by high-velocity flows, geotechnical instabilities caused by saturation and draining of bank materials, and erosion of material from the toe of banks allowing direct impingement of flow causing undercutting and increased geotechnical instabilities.

Less common causes of erosion noted in localized areas of the study reach include water entering the channel as concentrated overland runoff, vegetative debris in the channel which direct flows against banks, and piping of bank materials due to groundwater flow at interfaces between materials of different permeabilities.

Aggradation/degradation trends of the channel are not thought to significantly impact the stability of banks in the study area at the present time.

5.2 Bank Protection Techniques

Because of the variety of causes of bank erosion along Cache Creek, protection techniques were identified to accomplish the following objectives:

1. Direct protection of the fine-grained bank materials,
2. Reduction of flow velocities along banks,
3. Alignment of flows away from banks or reduction of sharp curvature along the channel,

4. Protection of the toe of the bank to buttress the slope and prevent overtopping,
5. Improvement of the structural stability of streambanks, and
6. Control of concentrated overland runoff over banks.

There are many techniques for accomplishing the above-stated objectives. The choice for a particular location depends on the level of protection desired, the geotechnical and hydraulic conditions that exist, and the resources available for implementation and maintenance. In the following sections, several erosion protection methods are identified. Various advantages and disadvantages associated with each method are also described.

5.2.1 Direct Protection

Direct protection techniques of streambanks are those that protect the bank material without reducing the erosive force of the flow. Examples of direct protection believed to be suitable along the study reach include riprap, concrete rubble, soil cement, and vegetative covers.

Riprap is probably the most commonly used direct protection method. If properly sized and emplaced, it can provide long-term erosion protection with little or no maintenance. Stability of riprap bank protection is highly dependent on the installation of an adequate gravel or geotextile filter between the native bank material and the riprap and upstream and downstream end treatments. Advantages of riprap over other types of direct bank protection include its relatively "natural" appearance, ease of installation with common earthmoving equipment, and ease of repair, if necessary. Disadvantages are generally related to its cost. Figure 5.1 shows a typical riprap bank protection installation. It should be noted that the design presented on Figure 5.1 is not for areas where flows will overtop the bank. In areas of overtopping, additional "rollback" rock must be provided along the top of the bank.

Soil cement is a relatively new direct protection technique. It has been used extensively as protection in Arizona and is formed by mixing a relatively low percentage (8 to 12 percent by volume) of portland cement with the native bank material. Typically, the native soil should be between 5 to 20 percent by weight of silt- and clay-sized material (passing No. 200 sieve) and no more than 10 percent by weight of material coarser than gravel (No. 4 sieve). The bank material found along many areas of the study reach appears to be suitable for preparing soil cement.

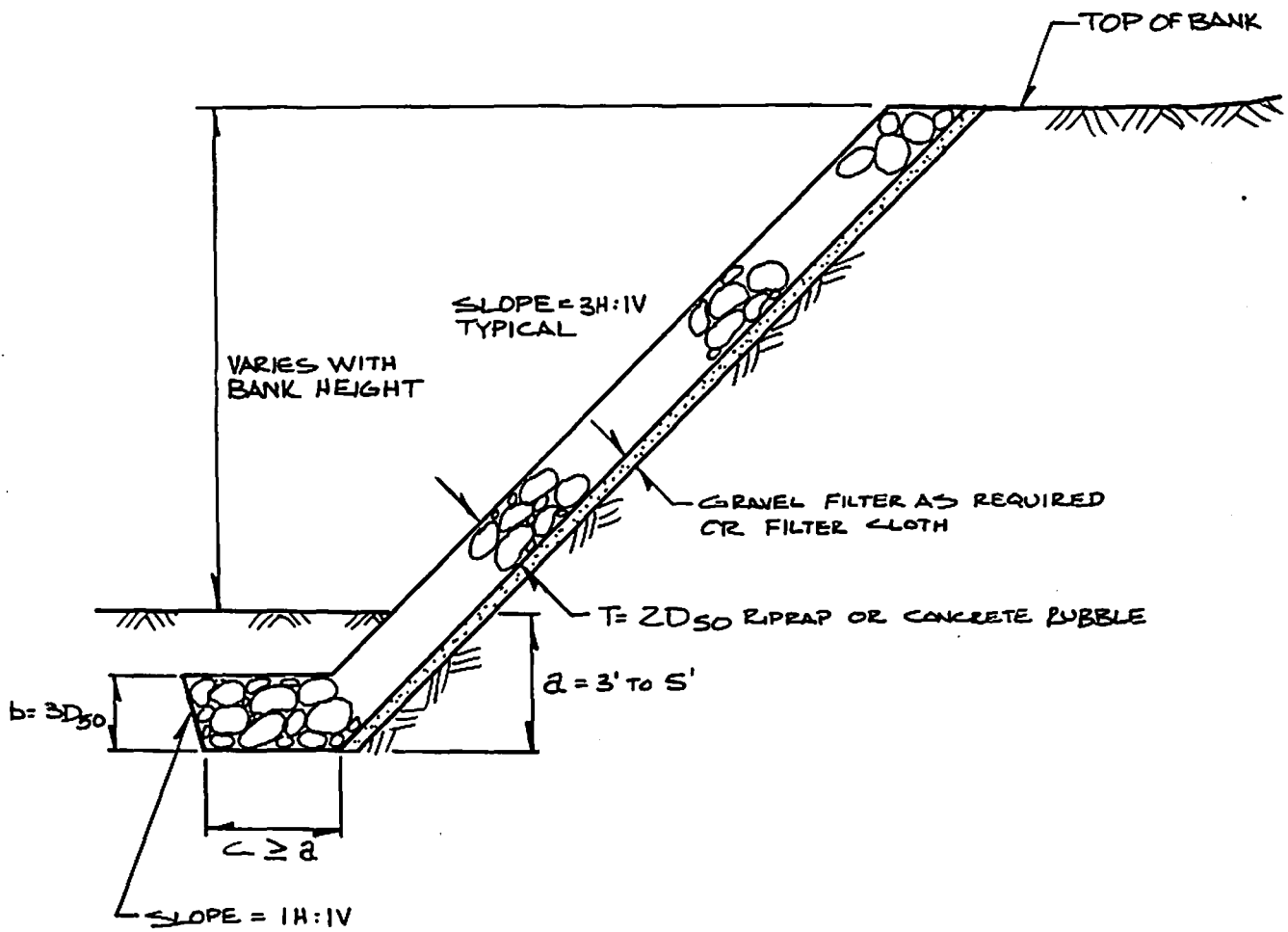


Figure 5.1. Typical riprap or concrete rubble bank protection.

Preparation and placement of soil cement can be done with a common batch plant and ordinary earthmoving equipment. Advantages of soil cement include utilization of locally available materials and economy of installation (in large quantities). Disadvantages include the requirement for more sophisticated engineering and construction support. A typical soil cement bank protection installation is shown on Figure 5.2.

Concrete rubble is another commonly used direct protection method. Numerous installations of concrete rubble protection already exist along the study reach. Installation of concrete rubble has the same general requirements as riprap. Advantages of concrete rubble include availability and ease of installation. Disadvantages include its "unnatural" appearance and difficulties in achieving an adequate gradation to prevent piping of bank material. Also, individual pieces of concrete rubble tend to be somewhat lighter than riprap, requiring larger particle sizes to insure stability.

Vegetative covers for direct protection from erosion have a more natural appearance than other erosion methods. Stability of vegetative covers require selection of plant species which are compatible with the environmental conditions of the study area and can resist erosive forces of flow. The advantage relates mainly to their "natural" appearance. Disadvantages of vegetative covers include problems with establishing and maintaining adequate cover, environmental uncertainties, such as drought, and the relatively low velocities (5 fps \pm) for which they can be expected to remain stable. For most locations in the study reach, particularly along the outside of bends, normal flow velocities are thought to be excessive for satisfactory performance of this method.

5.2.2 Longitudinal Controls

Longitudinal controls are protection techniques that run parallel to the eroding bank. They reduce the erosive forces on the bank by keeping flows from impinging directly on the bank. Longitudinal controls which would be useful at certain locations along the study reach include longitudinal stone dikes, board fences, and windrow revetment.

Figure 5.3 shows typical longitudinal stone dikes. They are constructed by placing riprap parallel to the toe of the streambank. The dike deflects flows and protects the toe of the slope. In many respects, the existing debris piles found at the toe of most banks act in much the same way. In many

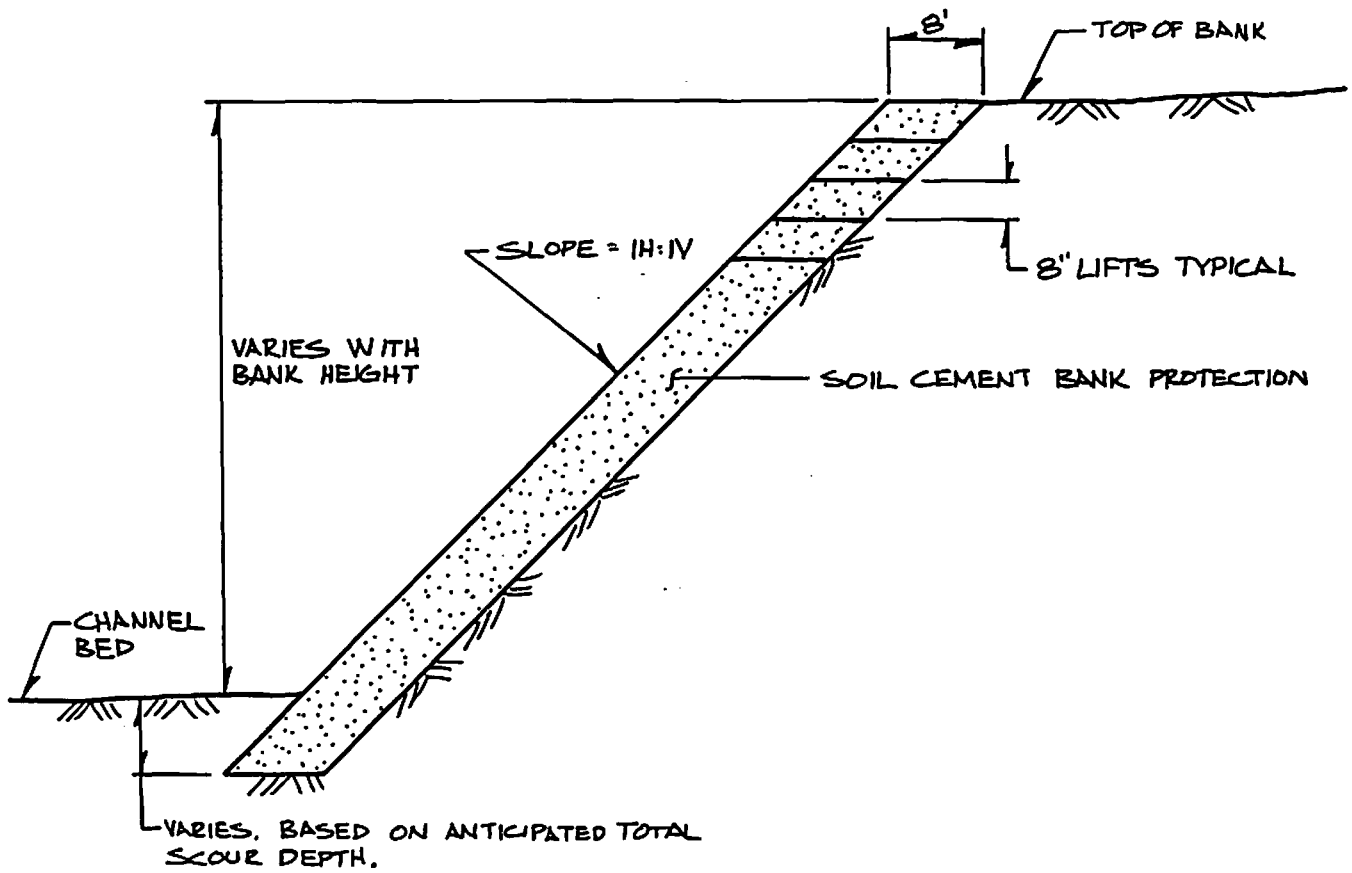
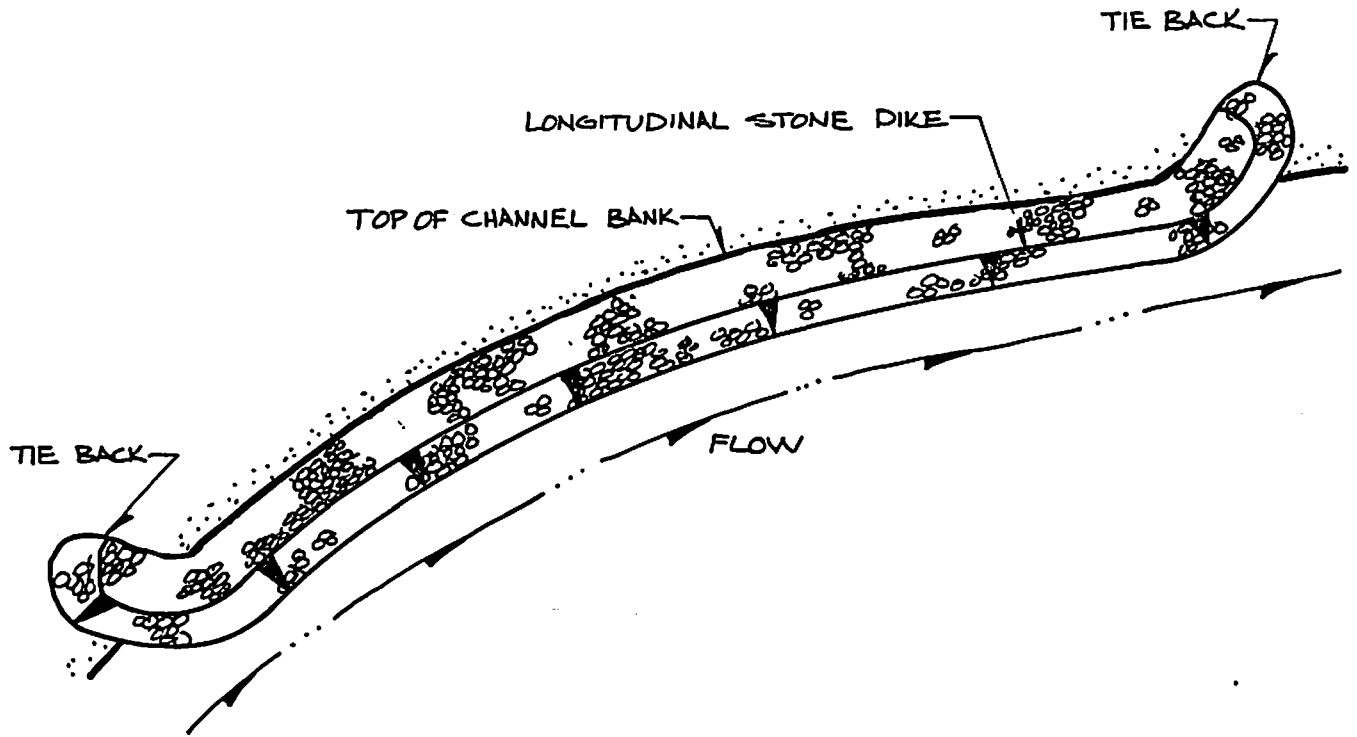
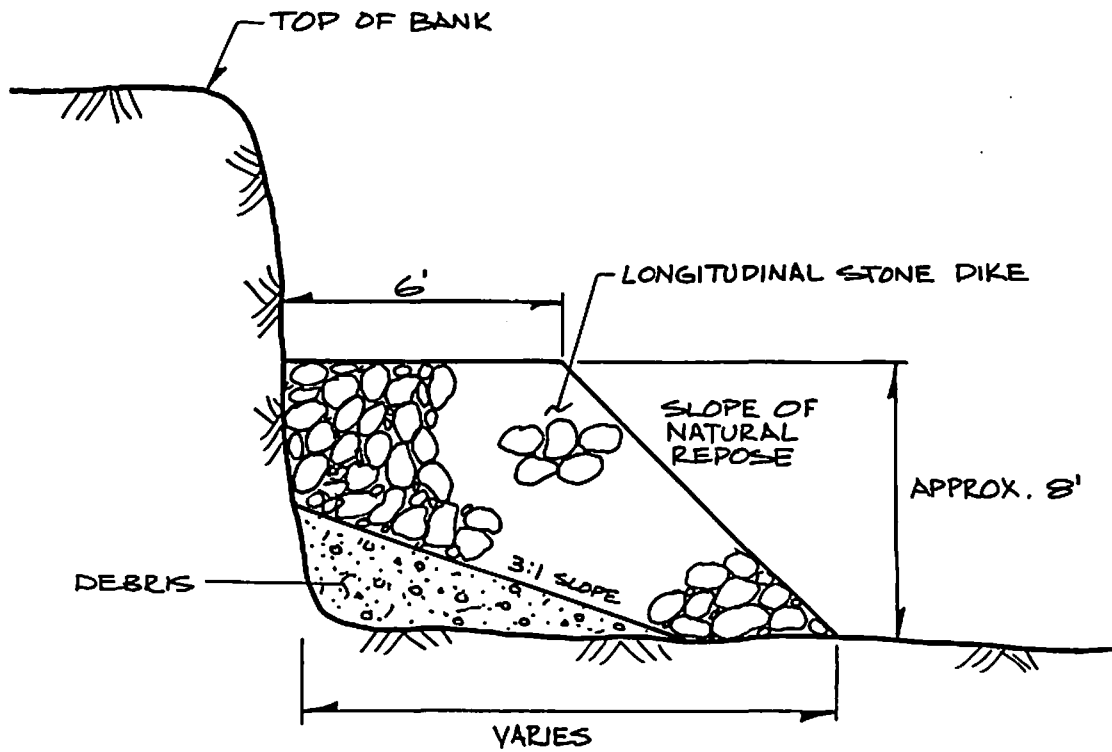


Figure 5.2. Typical soil cement bank protection.



PLAN VIEW



SECTIONAL VIEW

Figure 5.3. Typical longitudinal stone dike.

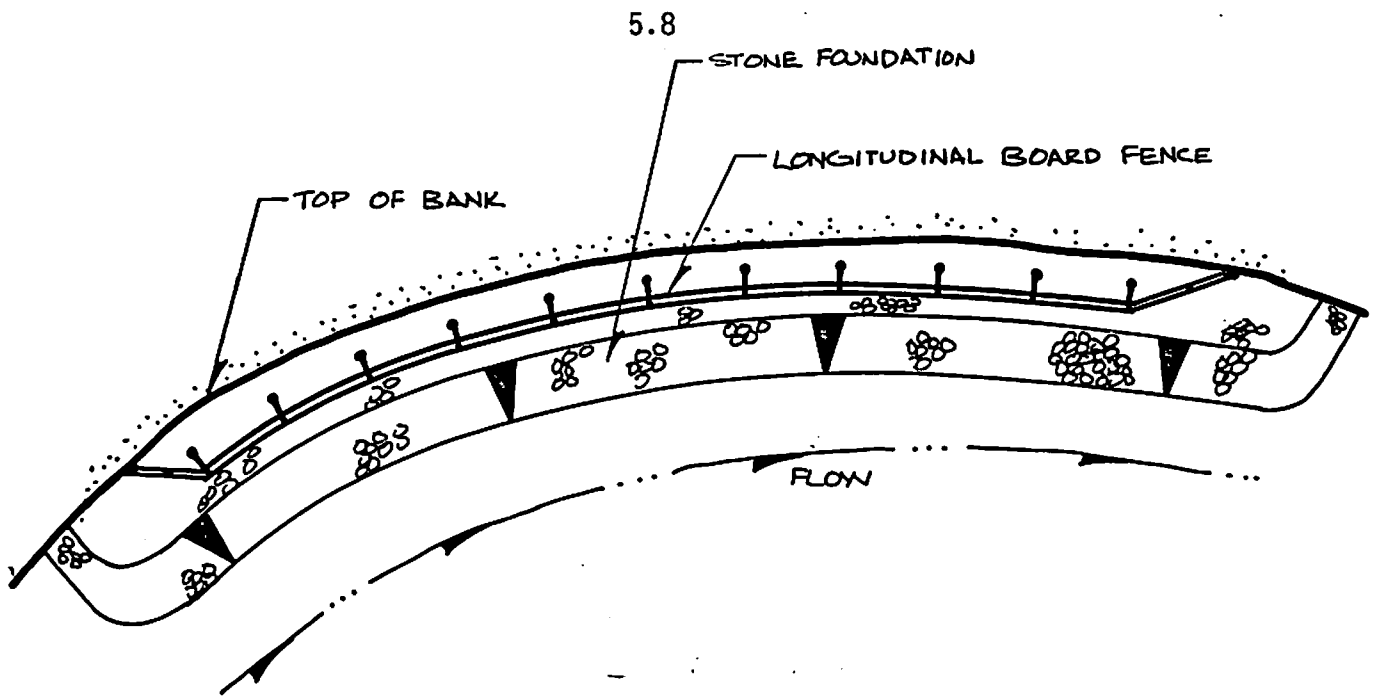
areas, the stone dikes could be built over the debris piles to provide protection to a greater height on the bank and increase the stability of both the debris pile and the bank under high-flow conditions. The longitudinal stone dike could be used in combination with a vegetative cover along the upper bank to provide complete protection of the bank.

The advantages of a longitudinal stone dike include a relatively natural appearance, ease of construction, and relatively low cost. Disadvantages may be related to its incomplete protection of the upper bank and the availability of suitable material.

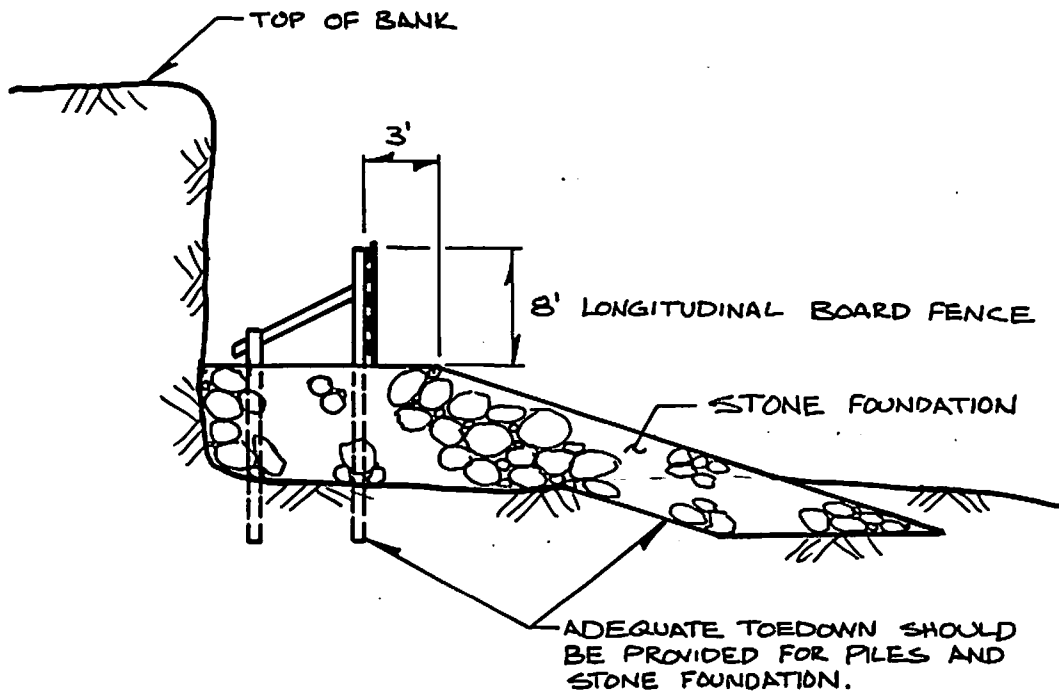
Figure 5.4 details a typical installation for a longitudinal board fence. The longitudinal board fence is intended to deflect flows away from the bank and reduce flow velocities along the bank. The fences are constructed of treated wood piles and boards. Riprap is placed along the foundation of the fence to prevent local scour. The major advantages of the longitudinal board fence are its simple construction, ease of repair, and relative low cost. Disadvantages include its need for maintenance, incomplete protection provided for either the toe or upper slope of the bank, and potential for being flanked by the stream.

Windrow revetment consists of a mound of stone placed on the ground, or partially or totally buried, immediately adjacent to the top of the eroding bank. As bank-line erosion reaches the windrow, the stone is undercut, causing it to "self-launch" down the bank. Figure 5.5 details typical windrow revetment installations.

Advantages of windrow revetment include relatively simple construction procedures, the avoidance of construction problems and hazards within the channel, minimal impact on the hydraulic characteristics of the channel, and the ability to use less expensive quarry run material rather than a specified gradation. Disadvantages of this method of bank protection include the additional land area necessary for placement of the revetment, the bank loss which must occur for function of the revetment, and the periodic monitoring needed to insure adequacy of design quantities. Additionally, the "maximum" bank height for which windrow revetment should be utilized is thought to be approximately 15 feet (SLA, 1985).



PLAN VIEW



SECTIONAL VIEW

Figure 5.4. Typical longitudinal board fence.

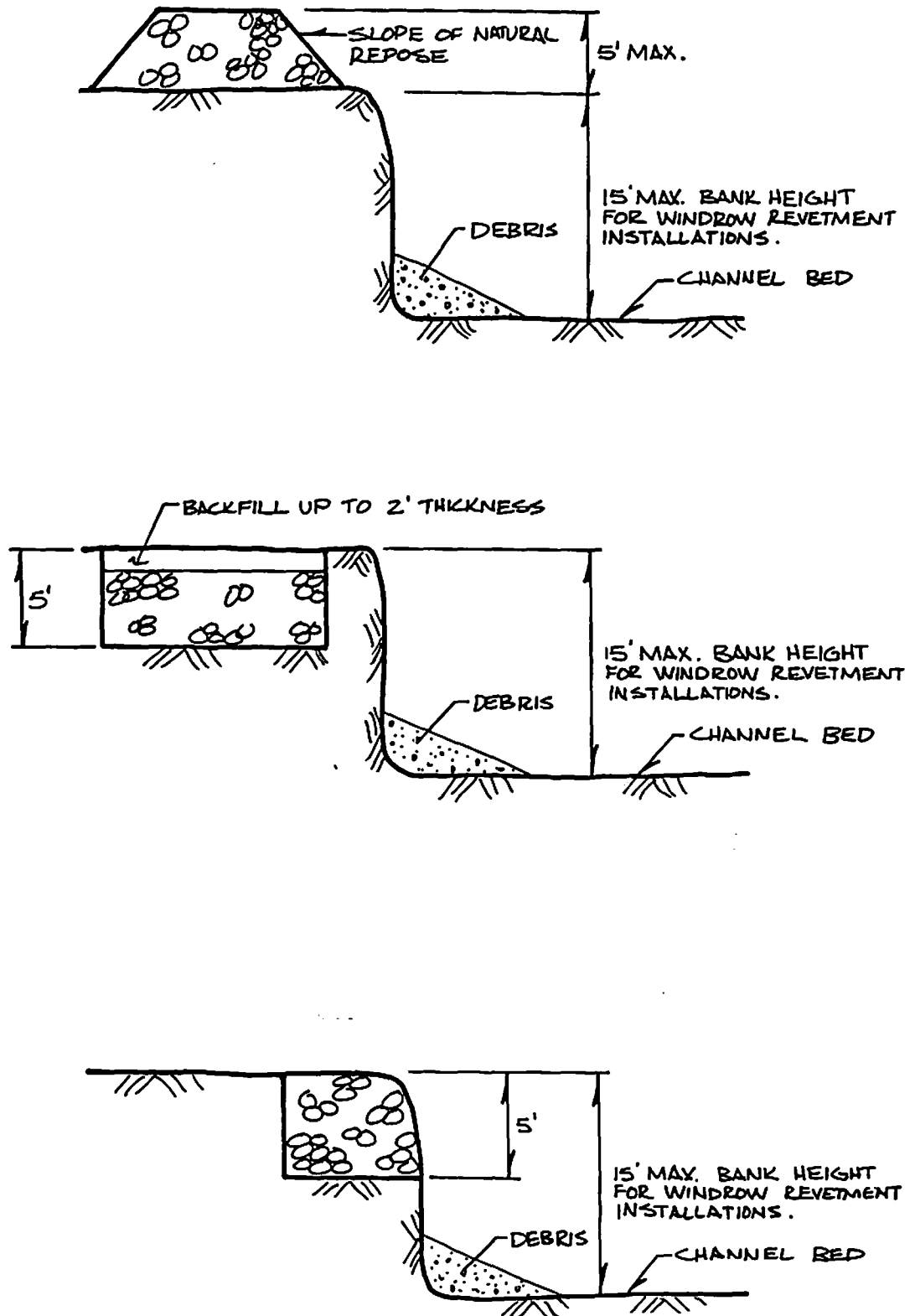


Figure 5.5. Typical windrow revetment installations.

5.2.3 Transverse Controls

Transverse controls are bank protection structures which protrude directly from the bank. They are intended to direct flows away from the bank and reduce velocities along the bank by creating flow-separation zones. Types of transverse controls include stone dikes and board fences.

Transverse stone dikes are riprap structures that protrude from the bank as a "hardpoint." A typical transverse stone dike installation is shown in Figure 5.6. Generally, a series of transverse dikes are used, such as at a bend in a river. They can also be used as tiebacks to prevent flanking of structures such as longitudinal stone dikes. Advantages of transverse stone dikes include their ease of construction and maintenance. Disadvantages include their incomplete protection of upper bank, difficulty in assessing effect on the hydraulics of the river, and the availability of suitable materials.

Transverse board fences are constructed of treated wood piles and boards. They also are intended to deflect flows and slow flow velocities along the bank. A typical installation is shown on Figure 5.7. Advantages of transverse board fences include their ease of construction and maintenance, and relative low cost. Disadvantages include the requirement for maintenance and the potential for vegetative debris hanging up on the fence. Additionally, transverse fences do not provide protection for the upper bank.

5.2.4 Control of Overland Runoff

The flow of concentrated overland runoff over banks along the study reach was a cause of bank erosion at several locations. Prevention of such erosion can be accomplished by providing controlled transitions for such flows. Transitions can vary from simple riprap-lined channels to collection ponds with corrugated metal drop pipes. The type of structure to be used is dependent on the area of contributing land and degree of protection required. Typical transitions are shown on Figures 5.8 and 5.9.

5.3 Discussion

In the previous sections, various methods for providing protection against bank erosion along the Cache Creek study area were presented. The methods presented represent a range of protection levels and therefore a range of resource requirements. The type of protection to be used at any particular

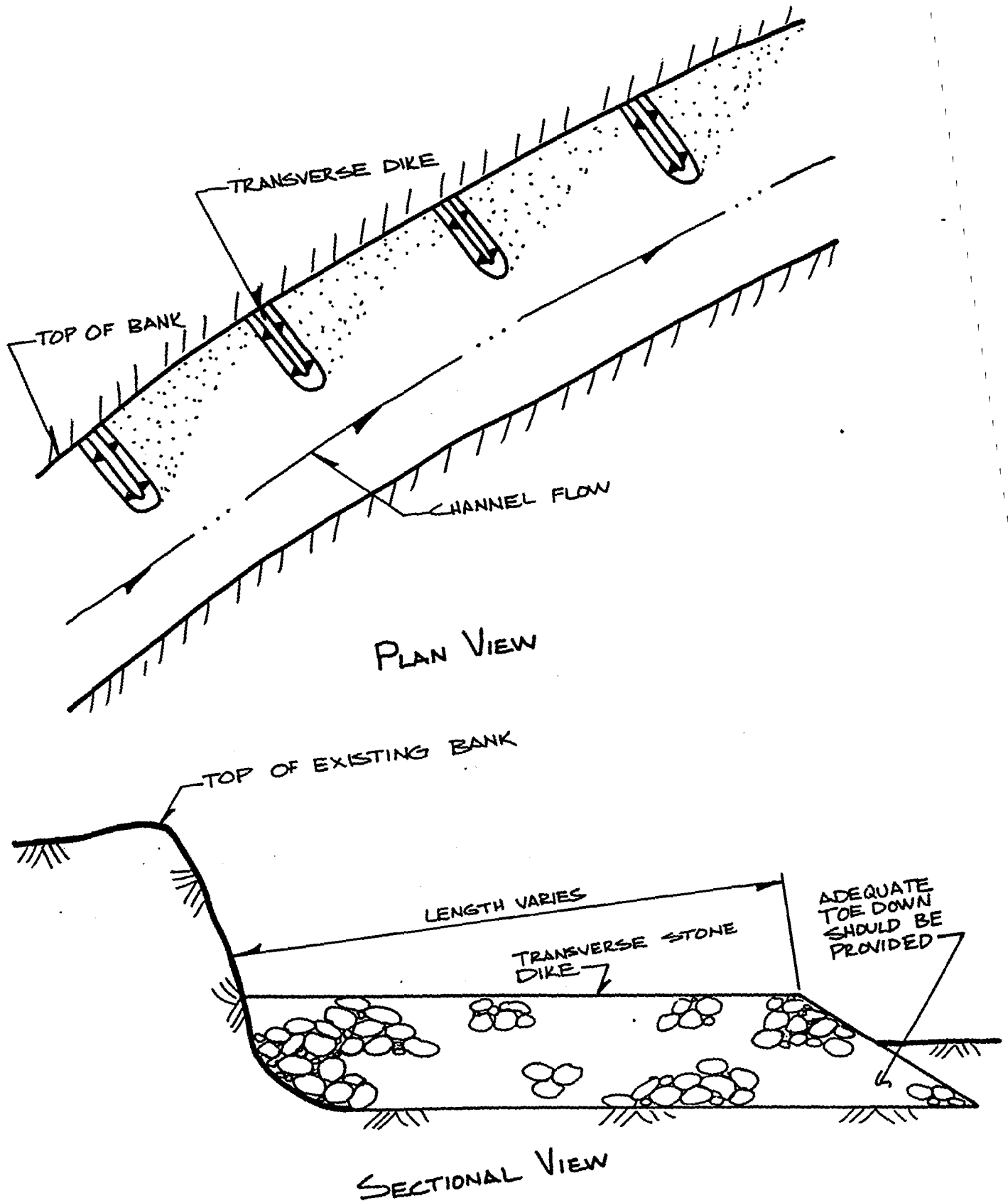


Figure 5.6. Typical transverse stone dike.

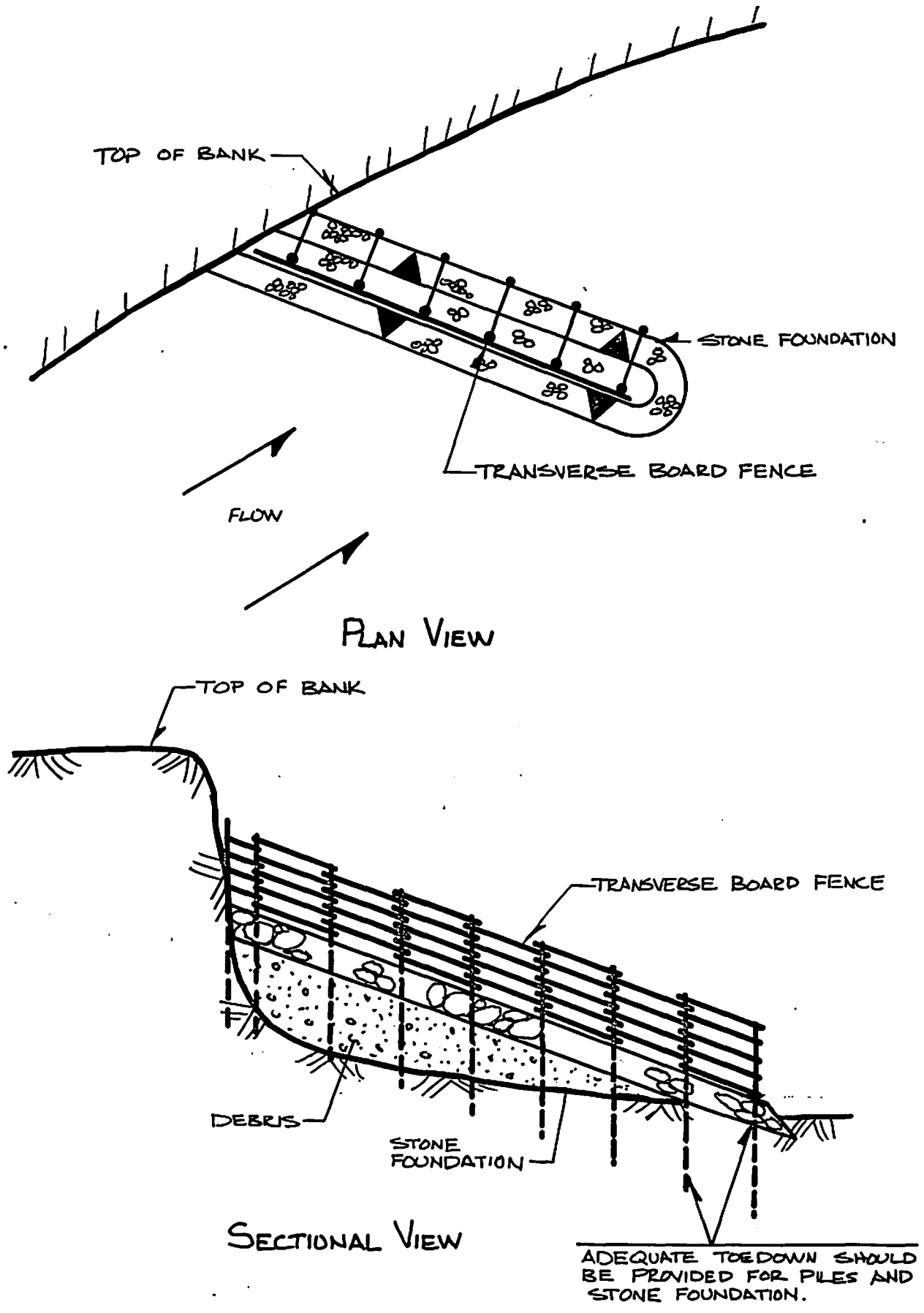


Figure 5.7. Typical transverse board fence.

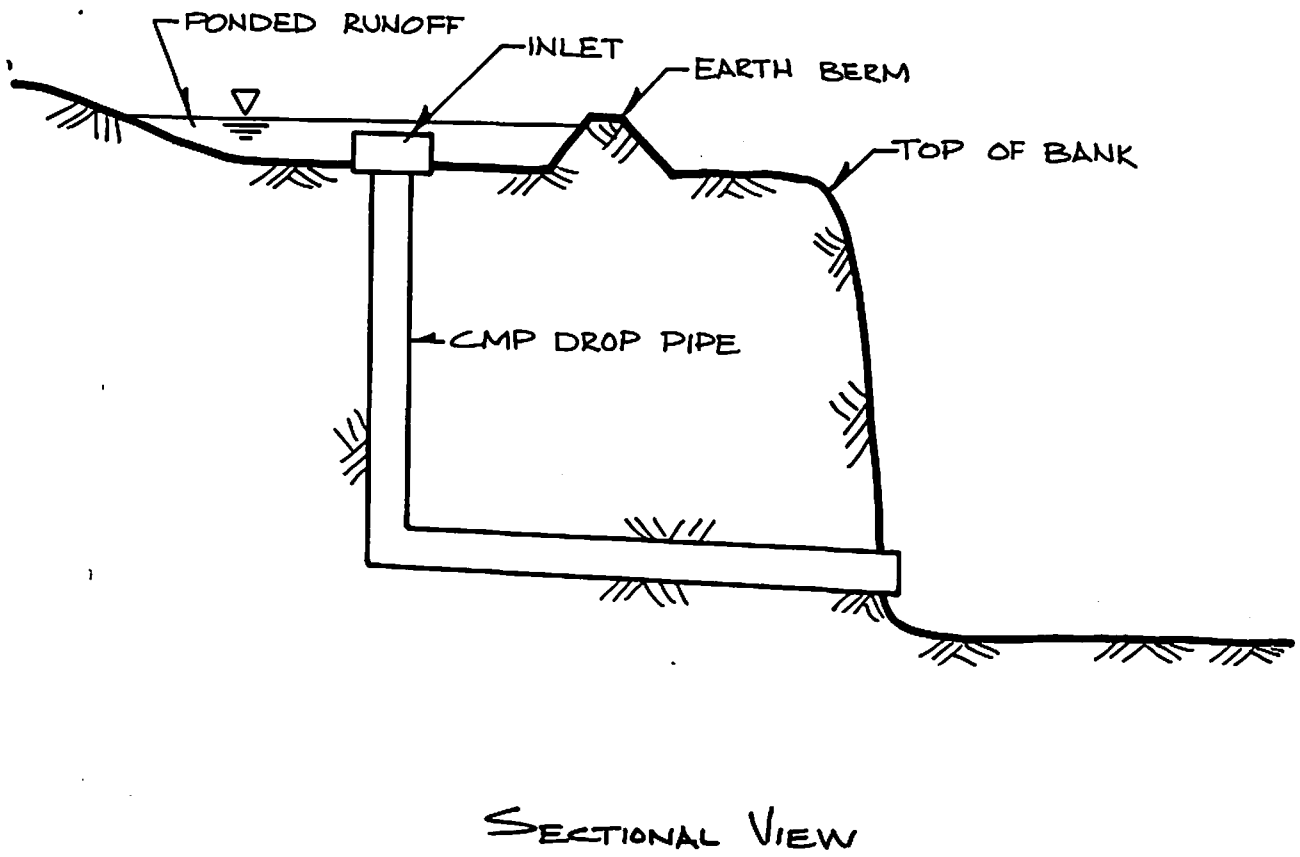


Figure 5.8. Typical drop pipe transition.

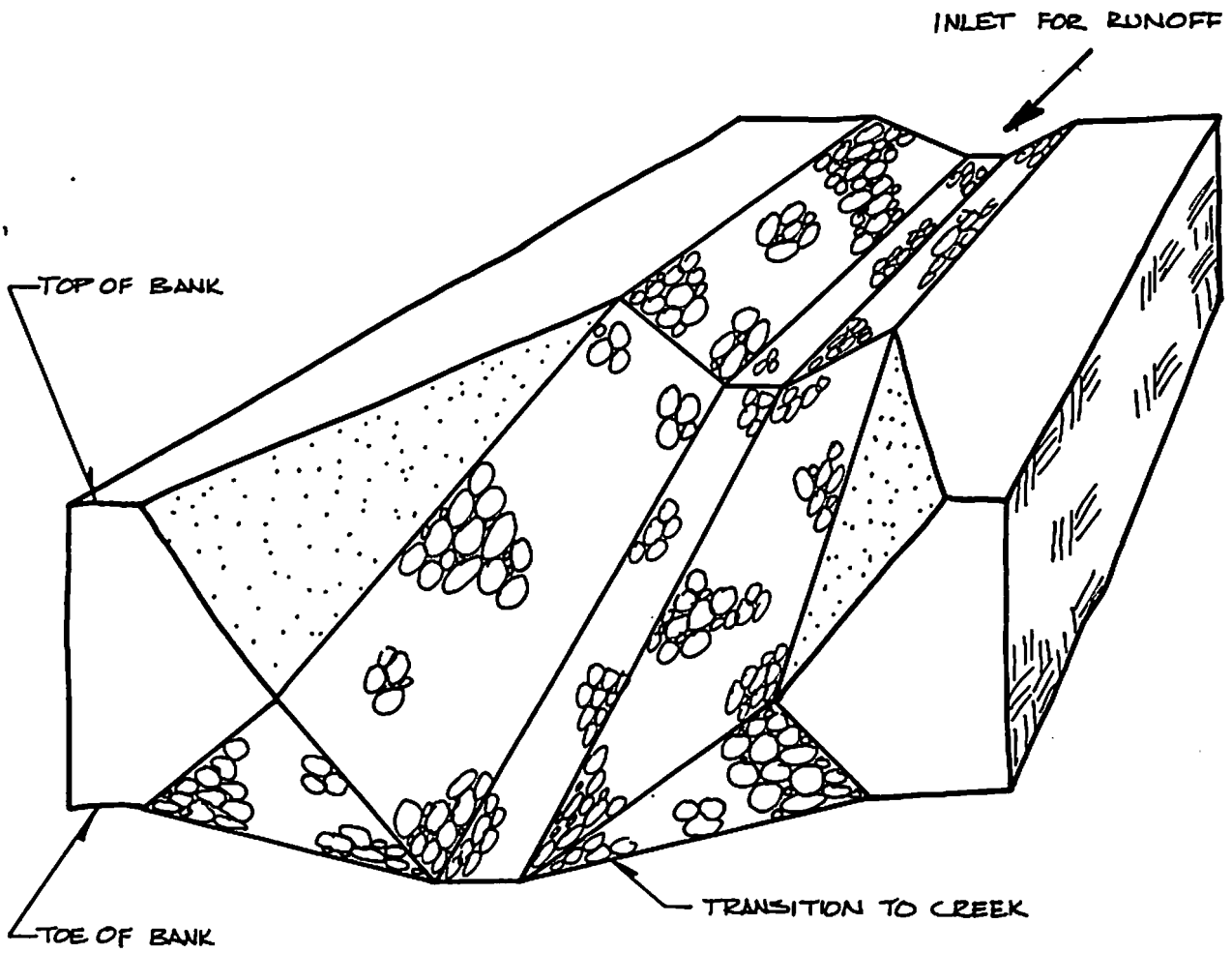


Figure 5.9. Typical riprap-lined channel transition.

location will depend on the level of protection necessary and resources available. Each erosion site should be evaluated separately to determine the best erosion protection measure and the potential impacts on adjacent areas. Regulatory or environmental constraints may also influence the decision.

VI. CONCLUSIONS

This report presents the analysis and results of a quantitative (Phase II) geomorphic analysis of the Cache Creek channel between Rumsey and Madison. The purpose of this phase of the study is to provide quantitative estimates of the impact of the proposed modification to the Clear Lake outlet channel on the stability of the channel along the study reach. Of particular concern is the potential for increased bank erosion. The following observations and conclusions were derived from this phase of the study:

1. The authorized changes to the Clear Lake outlet channel will change the frequency and duration of flood flows along the downstream channel. The flow-duration analysis of COE historic flood routings for eight runoff seasons indicates the proposed project will increase the duration of flood flows between 5,000 to 6,000 cfs and 20,000 cfs. The duration of lower flows will be reduced under the project condition. The duration of flows above 20,000 cfs will be approximately the same with a slight reduction for the highest discharges under project conditions. The duration of the highest flows decreases under project conditions in the downstream direction due to routing effects.
2. The sediment-continuity analysis for both pre- and postproject conditions indicates significant variability in aggradation/degradation potential throughout the study reach. For example, the reach upstream of Rumsey (Reach 2) shows relatively large aggradation potential while the reaches near Sugarloof Hill (Reach 6) and immediately upstream of I505 (Reach 10) show significant degradation potential. Due to the relatively coarse bed material throughout the study reach, the average degradation depth indicated by the continuity analysis will probably not be achieved due to channel armoring. Dampening of the indicated aggradation during the actual flow record is also likely since the continuity analysis does not reflect the dynamic effects of channel adjustment during the aggradation process. Nevertheless, it is felt that the continuity results are representative of the actual trends. Comparison of the results with the historical thalweg profile changes from 1977 to 1983 indicates a similar trend with wide variability in aggradation/degradation throughout the study reach.
3. Based on data from the COE historic flood routings, the sediment-transport analysis shows that the authorized Clear Lake flood-control project will, in general, increase the sediment-transport capacity along the study reach with a decreasing impact in the downstream direction. The increase is due to an increase in the duration of flows between 5,000 to 6,000 cfs and 20,000 cfs. The sediment-continuity analysis shows that the proposed project will, in general, increase the erosion/sedimentation trends slightly along the study reach. Two reaches show a slight decrease in erosion/sedimentation trends. The analysis also shows that significant armoring potential of the channel bed exists under both pre- and postproject conditions limiting the amount of bed degradation that will occur. It can be concluded that the authorized project will have no significant impact on the channel profile.

4. Analysis of the flow-duration data developed from the COE historic flood routings show that the duration of flows potentially causing bank erosion will be reduced by the proposed project. Bank erosion potential will be increased, however, due to the longer duration of flows between 5,000 to 6,000 cfs and 20,000 cfs. The relative increase in bank erosion potential may be less than predicted due to a bias included in the analysis by using the COE historic flood routings which only include runoff seasons with peak flows greater than 10,000 cfs. This bias is probably small, however, since the the low-flow periods which make up most of the time not simulated produce little or no bank erosion. Although the project is shown to increase bank erosion potential, the actual increase to bank erosion is not predicted since local variations in resistance of the material to erosion has not been considered.
5. Bank erosion has been and will continue to be a significant problem along the study reach. The primary cause of bank erosion along the study reach is related to the influence of flood flows which overtop protective debris piles located at the toe of most banks. The major mechanisms of bank erosion include direct erosion of sediments by high-velocity flow, geotechnical instabilities caused by saturation and draining of bank materials, and erosion of materials from the toe of banks allowing direct impingement of flow causing undercutting and increased geotechnical instabilities. Less common causes of bank erosion noted in localized areas include water entering the channel as concentrated overland runoff, vegetative debris in the channel which direct flows against banks, and piping of bank material due to groundwater flow at interfaces between materials of different permeabilities. Bank protection measures which may be appropriate in controlling these problems have been presented.

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APPENDIX A

Flow-duration data
for individual runoff seasons

Table A.1. Pre- and Postproject Flow-Duration Data for Rumsey, 1952 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 92.08 | 100.00 |
| 2000. | 86.14 | 80.20 |
| 3000. | 80.20 | 66.34 |
| 4000. | 44.55 | 46.20 |
| 5000. | 22.28 | 24.26 |
| 6000. | 10.23 | 12.21 |
| 7000. | 6.27 | 8.25 |
| 8000. | 4.13 | 6.11 |
| 9000. | 3.96 | 5.94 |
| 10000. | 3.14 | 3.30 |
| 11000. | 2.15 | 2.81 |
| 12000. | 1.32 | 2.48 |
| 13000. | 0.83 | 2.15 |
| 14000. | 0.00 | 1.82 |
| 15000. | 0.00 | 1.49 |
| 16000. | 0.00 | 0.99 |
| 17000. | 0.00 | 0.83 |
| 18000. | 0.00 | 0.50 |
| 19000. | 0.00 | 0.17 |
| 20000. | 0.00 | 0.00 |

Table A.2. Pre- and Postproject Flow-Duration Data for Rumsey, 1956 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 96.28 | 80.14 |
| 2000. | 91.21 | 76.42 |
| 3000. | 78.80 | 65.25 |
| 4000. | 51.40 | 46.54 |
| 5000. | 30.30 | 36.61 |
| 6000. | 17.48 | 23.78 |
| 7000. | 13.65 | 21.20 |
| 8000. | 10.96 | 18.51 |
| 9000. | 10.03 | 18.10 |
| 10000. | 8.89 | 16.44 |
| 11000. | 8.17 | 14.68 |
| 12000. | 7.86 | 11.48 |
| 13000. | 6.83 | 8.27 |
| 14000. | 6.00 | 7.55 |
| 15000. | 5.17 | 7.03 |
| 16000. | 3.83 | 6.72 |
| 17000. | 3.10 | 6.31 |
| 18000. | 2.79 | 5.79 |
| 19000. | 1.96 | 4.76 |
| 20000. | 1.45 | 3.93 |
| 21000. | 0.93 | 0.93 |
| 22000. | 0.72 | 0.72 |
| 23000. | 0.52 | 0.52 |
| 24000. | 0.52 | 0.52 |
| 25000. | 0.21 | 0.21 |
| 26000. | 0.10 | 0.10 |
| 27000. | 0.00 | 0.00 |

Table A.3. Pre- and Postproject Flow-Duration Data for Rumsey, 1958 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 98.88 | 78.75 |
| 2000. | 97.76 | 67.57 |
| 3000. | 96.64 | 57.50 |
| 4000. | 74.28 | 50.70 |
| 5000. | 55.27 | 46.04 |
| 6000. | 37.09 | 40.17 |
| 7000. | 23.30 | 37.47 |
| 8000. | 15.10 | 30.66 |
| 9000. | 8.20 | 26.00 |
| 10000. | 6.24 | 19.85 |
| 11000. | 5.31 | 14.82 |
| 12000. | 4.66 | 10.90 |
| 13000. | 3.73 | 7.55 |
| 14000. | 3.17 | 6.15 |
| 15000. | 2.42 | 4.75 |
| 16000. | 1.96 | 3.82 |
| 17000. | 1.40 | 3.45 |
| 18000. | 0.93 | 2.80 |
| 19000. | 0.84 | 2.61 |
| 20000. | 0.75 | 1.68 |
| 21000. | 0.47 | 0.47 |
| 22000. | 0.47 | 0.47 |
| 23000. | 0.37 | 0.37 |
| 24000. | 0.37 | 0.37 |
| 25000. | 0.37 | 0.37 |
| 26000. | 0.37 | 0.37 |
| 27000. | 0.28 | 0.28 |
| 28000. | 0.19 | 0.19 |
| 29000. | 0.19 | 0.19 |
| 30000. | 0.19 | 0.19 |
| 31000. | 0.19 | 0.19 |
| 32000. | 0.19 | 0.19 |
| 33000. | 0.19 | 0.19 |
| 34000. | 0.19 | 0.19 |
| 35000. | 0.19 | 0.19 |
| 36000. | 0.19 | 0.19 |
| 37000. | 0.09 | 0.09 |
| 38000. | 0.09 | 0.09 |
| 39000. | 0.09 | 0.09 |
| 40000. | 0.09 | 0.09 |
| 41000. | 0.00 | 0.00 |

Table A.4. Pre- and Postproject Flow-Duration Data for Rumsey, 1965 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 87.50 | 73.21 |
| 2000. | 69.64 | 69.64 |
| 3000. | 64.29 | 53.57 |
| 4000. | 41.07 | 32.14 |
| 5000. | 25.00 | 21.43 |
| 6000. | 14.29 | 17.86 |
| 7000. | 12.35 | 14.14 |
| 8000. | 8.04 | 11.61 |
| 9000. | 6.85 | 11.16 |
| 10000. | 6.55 | 11.01 |
| 11000. | 5.95 | 9.08 |
| 12000. | 5.51 | 8.78 |
| 13000. | 5.06 | 6.85 |
| 14000. | 4.61 | 6.55 |
| 15000. | 4.02 | 5.65 |
| 16000. | 3.27 | 5.21 |
| 17000. | 2.38 | 4.91 |
| 18000. | 1.49 | 4.32 |
| 19000. | 1.49 | 4.02 |
| 20000. | 1.19 | 3.57 |
| 21000. | 1.19 | 1.19 |
| 22000. | 1.04 | 1.04 |
| 23000. | 0.89 | 0.89 |
| 24000. | 0.74 | 0.74 |
| 25000. | 0.74 | 0.74 |
| 26000. | 0.45 | 0.45 |
| 27000. | 0.45 | 0.45 |
| 28000. | 0.45 | 0.45 |
| 29000. | 0.45 | 0.45 |
| 30000. | 0.45 | 0.45 |
| 31000. | 0.30 | 0.30 |
| 32000. | 0.15 | 0.15 |
| 33000. | 0.15 | 0.15 |
| 34000. | 0.15 | 0.15 |
| 35000. | 0.15 | 0.15 |
| 36000. | 0.15 | 0.15 |
| 37000. | 0.15 | 0.15 |
| 38000. | 0.00 | 0.00 |

Table A.5. Pre- and Postproject Flow-Duration Data for Rumsey, 1970 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 100.00 | 82.69 |
| 2000. | 98.08 | 69.23 |
| 3000. | 88.46 | 59.62 |
| 4000. | 65.22 | 42.31 |
| 5000. | 45.99 | 36.54 |
| 6000. | 30.77 | 31.73 |
| 7000. | 25.00 | 31.09 |
| 8000. | 19.23 | 25.80 |
| 9000. | 14.58 | 24.68 |
| 10000. | 10.74 | 22.12 |
| 11000. | 9.13 | 21.31 |
| 12000. | 7.53 | 18.75 |
| 13000. | 6.41 | 15.71 |
| 14000. | 5.61 | 11.38 |
| 15000. | 4.81 | 9.13 |
| 16000. | 4.01 | 7.53 |
| 17000. | 2.88 | 5.93 |
| 18000. | 2.24 | 4.97 |
| 19000. | 1.44 | 4.17 |
| 20000. | 1.12 | 3.85 |
| 21000. | 0.96 | 1.12 |
| 22000. | 0.96 | 1.12 |
| 23000. | 0.96 | 0.96 |
| 24000. | 0.96 | 0.96 |
| 25000. | 0.80 | 0.80 |
| 26000. | 0.64 | 0.64 |
| 27000. | 0.64 | 0.64 |
| 28000. | 0.64 | 0.64 |
| 29000. | 0.64 | 0.64 |
| 30000. | 0.64 | 0.64 |
| 31000. | 0.64 | 0.64 |
| 32000. | 0.64 | 0.64 |
| 33000. | 0.64 | 0.64 |
| 34000. | 0.64 | 0.64 |
| 35000. | 0.32 | 0.32 |
| 36000. | 0.32 | 0.32 |
| 37000. | 0.32 | 0.32 |
| 38000. | 0.16 | 0.16 |
| 39000. | 0.16 | 0.16 |
| 40000. | 0.00 | 0.00 |

Table A.6. Pre- and Postproject Flow-Duration Data for Rumsey, 1974 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 88.33 | 81.32 |
| 2000. | 62.65 | 54.86 |
| 3000. | 46.82 | 26.72 |
| 4000. | 26.39 | 18.03 |
| 5000. | 10.83 | 13.36 |
| 6000. | 7.98 | 11.28 |
| 7000. | 5.77 | 10.44 |
| 8000. | 3.37 | 7.26 |
| 9000. | 2.14 | 6.16 |
| 10000. | 1.56 | 6.03 |
| 11000. | 1.23 | 4.99 |
| 12000. | 0.91 | 3.44 |
| 13000. | 0.58 | 2.98 |
| 14000. | 0.39 | 1.43 |
| 15000. | 0.39 | 1.04 |
| 16000. | 0.32 | 0.91 |
| 17000. | 0.26 | 0.65 |
| 18000. | 0.26 | 0.52 |
| 19000. | 0.19 | 0.26 |
| 20000. | 0.19 | 0.19 |
| 21000. | 0.19 | 0.19 |
| 22000. | 0.19 | 0.13 |
| 23000. | 0.06 | 0.06 |
| 24000. | 0.06 | 0.06 |
| 25000. | 0.00 | 0.00 |

Table A.7. Pre- and Postproject Flow-Duration Data for Rumsey, 1978 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 63.77 | 81.16 |
| 2000. | 52.17 | 56.52 |
| 3000. | 44.93 | 33.33 |
| 4000. | 21.74 | 18.84 |
| 5000. | 2.90 | 11.59 |
| 6000. | 0.00 | 5.80 |
| 7000. | 0.00 | 1.45 |
| 8000. | 0.00 | 0.00 |

Table A.8. Pre- and Postproject Flow-Duration Data for Rumsey, 1983 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 88.69 | 89.50 |
| 2000. | 85.46 | 70.12 |
| 3000. | 85.46 | 63.66 |
| 4000. | 67.70 | 58.82 |
| 5000. | 57.00 | 52.89 |
| 6000. | 46.43 | 40.65 |
| 7000. | 36.41 | 31.56 |
| 8000. | 24.16 | 27.99 |
| 9000. | 16.76 | 21.20 |
| 10000. | 12.58 | 19.31 |
| 11000. | 8.95 | 17.50 |
| 12000. | 7.20 | 14.67 |
| 13000. | 5.18 | 11.37 |
| 14000. | 4.58 | 9.42 |
| 15000. | 4.17 | 7.67 |
| 16000. | 3.77 | 5.92 |
| 17000. | 1.48 | 4.91 |
| 18000. | 1.14 | 4.44 |
| 19000. | 0.87 | 3.90 |
| 20000. | 0.67 | 3.70 |
| 21000. | 0.47 | 0.47 |
| 22000. | 0.34 | 0.34 |
| 23000. | 0.34 | 0.34 |
| 24000. | 0.34 | 0.27 |
| 25000. | 0.27 | 0.27 |
| 26000. | 0.27 | 0.27 |
| 27000. | 0.27 | 0.27 |
| 28000. | 0.27 | 0.27 |
| 29000. | 0.27 | 0.27 |
| 30000. | 0.27 | 0.27 |
| 31000. | 0.27 | 0.27 |
| 32000. | 0.27 | 0.27 |
| 33000. | 0.27 | 0.27 |
| 34000. | 0.27 | 0.27 |
| 35000. | 0.27 | 0.27 |
| 36000. | 0.27 | 0.27 |
| 37000. | 0.20 | 0.20 |
| 38000. | 0.20 | 0.20 |
| 39000. | 0.20 | 0.20 |
| 40000. | 0.20 | 0.20 |
| 41000. | 0.20 | 0.20 |
| 42000. | 0.20 | 0.20 |
| 43000. | 0.20 | 0.13 |
| 44000. | 0.13 | 0.13 |

Table A.8. Pre- and Postproject Flow-Duration Data for Rumsey, 1983 Runoff Season, Continued.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 45000. | 0.13 | 0.13 |
| 46000. | 0.13 | 0.13 |
| 47000. | 0.13 | 0.13 |
| 48000. | 0.07 | 0.07 |
| 49000. | 0.07 | 0.07 |
| 50000. | 0.07 | 0.07 |
| 51000. | 0.07 | 0.07 |
| 52000. | 0.07 | 0.07 |
| 53000. | 0.07 | 0.07 |
| 54000. | 0.07 | 0.07 |
| 55000. | 0.00 | 0.00 |

Table A.9. Pre- and Postproject Flow-Duration Data for Capay, 1952 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 92.08 | 100.00 |
| 2000. | 86.14 | 88.12 |
| 3000. | 80.20 | 68.32 |
| 4000. | 58.42 | 52.48 |
| 5000. | 24.26 | 26.24 |
| 6000. | 16.17 | 20.13 |
| 7000. | 8.25 | 10.23 |
| 8000. | 6.27 | 8.25 |
| 9000. | 4.13 | 6.11 |
| 10000. | 3.80 | 5.78 |
| 11000. | 3.14 | 3.14 |
| 12000. | 2.15 | 2.64 |
| 13000. | 1.32 | 2.31 |
| 14000. | 0.99 | 1.98 |
| 15000. | 0.17 | 1.49 |
| 16000. | 0.00 | 1.32 |
| 17000. | 0.00 | 0.99 |
| 18000. | 0.00 | 0.66 |
| 19000. | 0.00 | 0.33 |
| 20000. | 0.00 | 0.00 |

Table A.10. Pre- and Postproject Flow-Duration Data for Capay, 1956 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 96.28 | 86.35 |
| 2000. | 92.55 | 77.66 |
| 3000. | 83.87 | 70.22 |
| 4000. | 60.19 | 50.36 |
| 5000. | 36.61 | 40.33 |
| 6000. | 21.72 | 27.61 |
| 7000. | 14.99 | 22.44 |
| 8000. | 12.51 | 19.96 |
| 9000. | 10.75 | 18.61 |
| 10000. | 9.51 | 18.10 |
| 11000. | 8.38 | 16.34 |
| 12000. | 7.86 | 13.34 |
| 13000. | 7.34 | 9.72 |
| 14000. | 6.52 | 8.07 |
| 15000. | 5.79 | 7.55 |
| 16000. | 4.45 | 7.14 |
| 17000. | 3.83 | 6.72 |
| 18000. | 3.41 | 6.20 |
| 19000. | 3.00 | 5.38 |
| 20000. | 3.00 | 3.41 |
| 21000. | 2.28 | 2.07 |
| 22000. | 1.76 | 1.55 |
| 23000. | 1.14 | 1.03 |
| 24000. | 0.93 | 0.72 |
| 25000. | 0.72 | 0.62 |
| 26000. | 0.62 | 0.31 |
| 27000. | 0.41 | 0.21 |
| 28000. | 0.31 | 0.21 |
| 29000. | 0.21 | 0.21 |
| 30000. | 0.00 | 0.00 |

Table A.11. Pre- and Postproject Flow-Duration Data for Capay, 1958 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 98.88 | 87.70 |
| 2000. | 97.76 | 69.80 |
| 3000. | 97.76 | 60.86 |
| 4000. | 83.22 | 54.15 |
| 5000. | 63.09 | 50.51 |
| 6000. | 49.58 | 44.73 |
| 7000. | 30.20 | 39.98 |
| 8000. | 21.44 | 35.97 |
| 9000. | 10.16 | 30.01 |
| 10000. | 8.20 | 24.23 |
| 11000. | 6.34 | 19.38 |
| 12000. | 5.31 | 12.30 |
| 13000. | 4.29 | 8.67 |
| 14000. | 3.73 | 7.27 |
| 15000. | 2.98 | 6.15 |
| 16000. | 2.33 | 4.75 |
| 17000. | 1.86 | 3.91 |
| 18000. | 1.68 | 3.17 |
| 19000. | 1.03 | 1.68 |
| 20000. | 0.93 | 1.21 |
| 21000. | 0.84 | 1.03 |
| 22000. | 0.84 | 0.93 |
| 23000. | 0.56 | 0.75 |
| 24000. | 0.56 | 0.75 |
| 25000. | 0.47 | 0.37 |
| 26000. | 0.37 | 0.28 |
| 27000. | 0.37 | 0.28 |
| 28000. | 0.37 | 0.19 |
| 29000. | 0.37 | 0.19 |
| 30000. | 0.37 | 0.19 |
| 31000. | 0.28 | 0.19 |
| 32000. | 0.28 | 0.19 |
| 33000. | 0.19 | 0.19 |
| 34000. | 0.19 | 0.09 |
| 35000. | 0.19 | 0.09 |
| 36000. | 0.19 | 0.09 |
| 37000. | 0.19 | 0.09 |
| 38000. | 0.19 | 0.09 |
| 39000. | 0.19 | 0.09 |
| 40000. | 0.19 | 0.00 |
| 41000. | 0.19 | 0.00 |
| 42000. | 0.19 | 0.00 |
| 43000. | 0.09 | 0.00 |
| 44000. | 0.09 | 0.00 |
| 45000. | 0.09 | 0.00 |
| 46000. | 0.00 | 0.00 |

Table A.12. Pre- and Postproject Flow-Duration Data for Capay, 1965 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 87.50 | 73.21 |
| 2000. | 71.43 | 69.64 |
| 3000. | 66.07 | 60.71 |
| 4000. | 48.21 | 39.29 |
| 5000. | 28.57 | 25.00 |
| 6000. | 19.64 | 21.43 |
| 7000. | 12.50 | 16.07 |
| 8000. | 8.63 | 12.20 |
| 9000. | 7.44 | 11.01 |
| 10000. | 6.40 | 10.57 |
| 11000. | 5.80 | 8.63 |
| 12000. | 5.65 | 8.48 |
| 13000. | 5.21 | 6.70 |
| 14000. | 4.61 | 6.55 |
| 15000. | 4.02 | 6.25 |
| 16000. | 3.27 | 5.65 |
| 17000. | 2.53 | 5.21 |
| 18000. | 1.93 | 4.32 |
| 19000. | 1.93 | 3.87 |
| 20000. | 1.93 | 2.98 |
| 21000. | 1.64 | 2.08 |
| 22000. | 1.49 | 1.49 |
| 23000. | 1.19 | 1.34 |
| 24000. | 1.19 | 1.04 |
| 25000. | 1.19 | 1.04 |
| 26000. | 0.89 | 0.89 |
| 27000. | 0.89 | 0.74 |
| 28000. | 0.89 | 0.74 |
| 29000. | 0.74 | 0.60 |
| 30000. | 0.60 | 0.45 |
| 31000. | 0.60 | 0.45 |
| 32000. | 0.45 | 0.45 |
| 33000. | 0.45 | 0.45 |
| 34000. | 0.45 | 0.30 |
| 35000. | 0.45 | 0.15 |
| 36000. | 0.45 | 0.15 |
| 37000. | 0.30 | 0.15 |
| 38000. | 0.30 | 0.15 |
| 39000. | 0.15 | 0.15 |
| 40000. | 0.15 | 0.00 |
| 41000. | 0.15 | 0.00 |
| 42000. | 0.15 | 0.00 |
| 43000. | 0.15 | 0.00 |
| 44000. | 0.00 | 0.00 |

Table A.13. Pre- and Postproject Flow-Duration Data for Capay, 1970 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 100.00 | 90.38 |
| 2000. | 100.00 | 73.08 |
| 3000. | 94.23 | 63.46 |
| 4000. | 73.08 | 50.00 |
| 5000. | 51.60 | 40.38 |
| 6000. | 39.90 | 34.46 |
| 7000. | 24.68 | 31.57 |
| 8000. | 20.83 | 30.45 |
| 9000. | 17.31 | 25.16 |
| 10000. | 11.86 | 22.12 |
| 11000. | 9.78 | 21.47 |
| 12000. | 8.01 | 18.75 |
| 13000. | 6.09 | 17.95 |
| 14000. | 4.81 | 12.82 |
| 15000. | 4.01 | 8.65 |
| 16000. | 3.21 | 7.05 |
| 17000. | 1.60 | 5.61 |
| 18000. | 1.28 | 4.01 |
| 19000. | 0.96 | 2.72 |
| 20000. | 0.80 | 1.12 |
| 21000. | 0.80 | 1.12 |
| 22000. | 0.80 | 0.96 |
| 23000. | 0.80 | 0.80 |
| 24000. | 0.64 | 0.80 |
| 25000. | 0.64 | 0.80 |
| 26000. | 0.64 | 0.80 |
| 27000. | 0.64 | 0.48 |
| 28000. | 0.48 | 0.48 |
| 29000. | 0.32 | 0.48 |
| 30000. | 0.32 | 0.32 |
| 31000. | 0.16 | 0.16 |
| 32000. | 0.00 | 0.16 |
| 33000. | 0.00 | 0.00 |

Table A.14. Pre- and Postproject Flow-Duration Data for Capay, 1974 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 89.11 | 80.54 |
| 2000. | 64.98 | 56.42 |
| 3000. | 46.89 | 32.17 |
| 4000. | 31.19 | 18.74 |
| 5000. | 14.01 | 14.07 |
| 6000. | 9.99 | 12.39 |
| 7000. | 7.00 | 11.93 |
| 8000. | 3.05 | 9.47 |
| 9000. | 2.33 | 7.85 |
| 10000. | 1.88 | 6.10 |
| 11000. | 1.62 | 5.19 |
| 12000. | 1.36 | 4.60 |
| 13000. | 0.84 | 2.20 |
| 14000. | 0.78 | 1.62 |
| 15000. | 0.52 | 1.36 |
| 16000. | 0.39 | 1.10 |
| 17000. | 0.39 | 0.91 |
| 18000. | 0.32 | 0.71 |
| 19000. | 0.26 | 0.52 |
| 20000. | 0.26 | 0.52 |
| 21000. | 0.19 | 0.32 |
| 22000. | 0.19 | 0.06 |
| 23000. | 0.19 | 0.00 |
| 24000. | 0.19 | 0.00 |
| 25000. | 0.13 | 0.00 |
| 26000. | 0.06 | 0.00 |
| 27000. | 0.06 | 0.00 |
| 28000. | 0.00 | 0.00 |

Table A.15. Pre- and Postproject Flow-Duration Data for Capay, 1978 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 68.12 | 81.16 |
| 2000. | 53.62 | 57.97 |
| 3000. | 47.83 | 34.78 |
| 4000. | 28.99 | 23.19 |
| 5000. | 10.14 | 13.04 |
| 6000. | 2.90 | 7.25 |
| 7000. | 0.00 | 2.90 |
| 8000. | 0.00 | 0.00 |

Table A.16. Pre- and Postproject Flow-Duration Data for Capay, 1983 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 88.69 | 91.86 |
| 2000. | 85.46 | 74.09 |
| 3000. | 85.40 | 65.21 |
| 4000. | 72.48 | 60.30 |
| 5000. | 61.10 | 56.93 |
| 6000. | 50.81 | 45.83 |
| 7000. | 42.13 | 38.09 |
| 8000. | 33.78 | 30.62 |
| 9000. | 22.21 | 25.30 |
| 10000. | 16.22 | 19.92 |
| 11000. | 10.90 | 18.91 |
| 12000. | 8.61 | 17.23 |
| 13000. | 7.54 | 14.47 |
| 14000. | 5.59 | 10.50 |
| 15000. | 4.78 | 9.22 |
| 16000. | 4.31 | 7.54 |
| 17000. | 3.84 | 6.12 |
| 18000. | 2.49 | 5.25 |
| 19000. | 1.35 | 4.51 |
| 20000. | 1.01 | 4.10 |
| 21000. | 0.81 | 3.57 |
| 22000. | 0.47 | 0.61 |
| 23000. | 0.40 | 0.40 |
| 24000. | 0.40 | 0.40 |
| 25000. | 0.40 | 0.40 |
| 26000. | 0.40 | 0.40 |
| 27000. | 0.40 | 0.40 |
| 28000. | 0.27 | 0.27 |
| 29000. | 0.27 | 0.27 |
| 30000. | 0.27 | 0.27 |
| 31000. | 0.27 | 0.27 |
| 32000. | 0.27 | 0.27 |
| 33000. | 0.27 | 0.27 |
| 34000. | 0.27 | 0.27 |
| 35000. | 0.27 | 0.27 |
| 36000. | 0.27 | 0.27 |
| 37000. | 0.27 | 0.27 |
| 38000. | 0.27 | 0.27 |
| 39000. | 0.27 | 0.27 |
| 40000. | 0.27 | 0.27 |
| 41000. | 0.20 | 0.20 |
| 42000. | 0.20 | 0.20 |
| 43000. | 0.13 | 0.13 |
| 44000. | 0.13 | 0.13 |

Table A.16. Pre- and Postproject Flow-Duration Data for Capay, 1983 Runoff Season, Continued.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 45000. | 0.13 | 0.13 |
| 46000. | 0.13 | 0.13 |
| 47000. | 0.13 | 0.13 |
| 48000. | 0.13 | 0.13 |
| 49000. | 0.13 | 0.13 |
| 50000. | 0.13 | 0.13 |
| 51000. | 0.07 | 0.07 |
| 52000. | 0.07 | 0.00 |
| 53000. | 0.00 | 0.00 |

Table A.17. Pre- and Postproject Flow-Duration Data for Yolo, 1952 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 90.10 | 100.00 |
| 2000. | 86.14 | 86.14 |
| 3000. | 80.20 | 68.32 |
| 4000. | 56.44 | 52.48 |
| 5000. | 27.89 | 29.70 |
| 6000. | 17.49 | 21.29 |
| 7000. | 11.39 | 9.41 |
| 8000. | 5.12 | 9.08 |
| 9000. | 3.14 | 5.12 |
| 10000. | 2.81 | 4.79 |
| 11000. | 2.81 | 2.64 |
| 12000. | 2.48 | 2.31 |
| 13000. | 2.15 | 1.98 |
| 14000. | 1.82 | 1.49 |
| 15000. | 1.32 | 1.16 |
| 16000. | 0.99 | 0.99 |
| 17000. | 0.66 | 0.99 |
| 18000. | 0.50 | 0.83 |
| 19000. | 0.00 | 0.50 |
| 20000. | 0.00 | 0.33 |
| 21000. | 0.00 | 0.00 |

Table A.18. Pre- and Postproject Flow-Duration Data for Yolo, 1956 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 97.31 | 87.49 |
| 2000. | 92.24 | 78.59 |
| 3000. | 87.07 | 71.04 |
| 4000. | 61.01 | 50.98 |
| 5000. | 39.71 | 42.19 |
| 6000. | 23.37 | 29.58 |
| 7000. | 14.48 | 22.85 |
| 8000. | 11.89 | 20.06 |
| 9000. | 9.20 | 18.51 |
| 10000. | 8.58 | 16.65 |
| 11000. | 8.27 | 14.06 |
| 12000. | 7.96 | 12.00 |
| 13000. | 7.76 | 10.44 |
| 14000. | 7.14 | 8.69 |
| 15000. | 6.51 | 6.72 |
| 16000. | 6.20 | 6.10 |
| 17000. | 5.48 | 5.69 |
| 18000. | 4.86 | 5.27 |
| 19000. | 4.14 | 3.93 |
| 20000. | 2.90 | 3.31 |
| 21000. | 2.38 | 2.38 |
| 22000. | 1.76 | 1.96 |
| 23000. | 1.55 | 0.52 |
| 24000. | 1.03 | 0.21 |
| 25000. | 0.93 | 0.10 |
| 26000. | 0.72 | 0.00 |
| 27000. | 0.31 | 0.00 |
| 28000. | 0.10 | 0.00 |
| 29000. | 0.10 | 0.00 |
| 30000. | 0.10 | 0.00 |
| 31000. | 0.10 | 0.00 |
| 32000. | 0.10 | 0.00 |
| 33000. | 0.10 | 0.00 |
| 34000. | 0.00 | 0.00 |

Table A.19. Pre- and Postproject Flow-Duration Data for Yolo, 1958 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 98.88 | 86.58 |
| 2000. | 97.76 | 70.92 |
| 3000. | 97.76 | 63.09 |
| 4000. | 83.22 | 53.87 |
| 5000. | 62.72 | 48.84 |
| 6000. | 51.16 | 46.32 |
| 7000. | 30.29 | 39.42 |
| 8000. | 24.32 | 36.72 |
| 9000. | 18.55 | 31.87 |
| 10000. | 9.51 | 27.87 |
| 11000. | 7.83 | 19.66 |
| 12000. | 7.08 | 16.03 |
| 13000. | 6.34 | 11.18 |
| 14000. | 5.59 | 8.29 |
| 15000. | 5.22 | 6.24 |
| 16000. | 4.57 | 5.78 |
| 17000. | 4.29 | 5.13 |
| 18000. | 3.54 | 4.38 |
| 19000. | 3.17 | 3.91 |
| 20000. | 2.24 | 3.45 |
| 21000. | 1.86 | 3.17 |
| 22000. | 1.58 | 2.33 |
| 23000. | 1.30 | 1.68 |
| 24000. | 1.21 | 1.21 |
| 25000. | 0.84 | 1.12 |
| 26000. | 0.65 | 0.93 |
| 27000. | 0.65 | 0.84 |
| 28000. | 0.56 | 0.56 |
| 29000. | 0.56 | 0.47 |
| 30000. | 0.56 | 0.28 |
| 31000. | 0.47 | 0.09 |
| 32000. | 0.37 | 0.09 |
| 33000. | 0.37 | 0.00 |
| 34000. | 0.37 | 0.00 |
| 35000. | 0.37 | 0.00 |
| 36000. | 0.28 | 0.00 |
| 37000. | 0.28 | 0.00 |
| 38000. | 0.28 | 0.00 |
| 39000. | 0.19 | 0.00 |
| 40000. | 0.09 | 0.00 |
| 41000. | 0.09 | 0.00 |
| 42000. | 0.00 | 0.00 |

Table A.20. Pre- and Postproject Flow-Duration Data for Yolo, 1965 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 89.14 | 71.28 |
| 2000. | 69.49 | 69.49 |
| 3000. | 65.77 | 53.27 |
| 4000. | 44.35 | 33.63 |
| 5000. | 24.70 | 24.55 |
| 6000. | 18.90 | 20.68 |
| 7000. | 11.61 | 15.18 |
| 8000. | 9.52 | 13.10 |
| 9000. | 7.14 | 10.86 |
| 10000. | 6.85 | 10.12 |
| 11000. | 6.10 | 8.04 |
| 12000. | 6.10 | 8.04 |
| 13000. | 5.80 | 6.25 |
| 14000. | 5.80 | 5.95 |
| 15000. | 5.65 | 5.95 |
| 16000. | 5.51 | 5.51 |
| 17000. | 4.61 | 5.21 |
| 18000. | 4.17 | 4.46 |
| 19000. | 3.72 | 3.87 |
| 20000. | 3.42 | 2.83 |
| 21000. | 2.83 | 2.08 |
| 22000. | 2.23 | 1.79 |
| 23000. | 1.93 | 1.04 |
| 24000. | 1.79 | 1.04 |
| 25000. | 1.49 | 0.89 |
| 26000. | 1.34 | 0.74 |
| 27000. | 1.04 | 0.74 |
| 28000. | 0.74 | 0.60 |
| 29000. | 0.74 | 0.45 |
| 30000. | 0.74 | 0.30 |
| 31000. | 0.60 | 0.30 |
| 32000. | 0.60 | 0.15 |
| 33000. | 0.60 | 0.00 |
| 34000. | 0.60 | 0.00 |
| 35000. | 0.45 | 0.00 |
| 36000. | 0.45 | 0.00 |
| 37000. | 0.30 | 0.00 |
| 38000. | 0.30 | 0.00 |
| 39000. | 0.30 | 0.00 |
| 40000. | 0.00 | 0.00 |

Table A.21. Pre- and Postproject Flow-Duration Data for Yolo, 1970 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 100.00 | 88.46 |
| 2000. | 100.00 | 75.00 |
| 3000. | 90.06 | 63.46 |
| 4000. | 72.44 | 46.15 |
| 5000. | 43.27 | 36.22 |
| 6000. | 31.09 | 33.65 |
| 7000. | 23.72 | 30.29 |
| 8000. | 19.71 | 25.16 |
| 9000. | 15.87 | 24.68 |
| 10000. | 13.46 | 23.72 |
| 11000. | 11.70 | 21.31 |
| 12000. | 10.42 | 18.75 |
| 13000. | 9.13 | 16.67 |
| 14000. | 8.33 | 11.06 |
| 15000. | 7.53 | 9.13 |
| 16000. | 6.89 | 7.85 |
| 17000. | 4.97 | 6.57 |
| 18000. | 4.33 | 5.45 |
| 19000. | 3.69 | 4.49 |
| 20000. | 3.04 | 2.40 |
| 21000. | 2.40 | 1.60 |
| 22000. | 1.28 | 1.12 |
| 23000. | 1.28 | 1.12 |
| 24000. | 1.12 | 0.80 |
| 25000. | 0.96 | 0.80 |
| 26000. | 0.80 | 0.80 |
| 27000. | 0.80 | 0.64 |
| 28000. | 0.80 | 0.48 |
| 29000. | 0.48 | 0.48 |
| 30000. | 0.48 | 0.48 |
| 31000. | 0.48 | 0.16 |
| 32000. | 0.48 | 0.00 |
| 33000. | 0.16 | 0.00 |
| 34000. | 0.00 | 0.00 |

Table A.22. Pre- and Postproject Flow-Duration Data for Yolo, 1974 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 87.29 | 78.92 |
| 2000. | 57.59 | 50.45 |
| 3000. | 45.78 | 27.11 |
| 4000. | 28.53 | 15.37 |
| 5000. | 12.13 | 13.75 |
| 6000. | 9.79 | 12.13 |
| 7000. | 7.78 | 11.74 |
| 8000. | 4.47 | 9.99 |
| 9000. | 3.37 | 8.37 |
| 10000. | 3.11 | 5.97 |
| 11000. | 2.59 | 5.12 |
| 12000. | 2.14 | 4.67 |
| 13000. | 1.82 | 2.14 |
| 14000. | 1.43 | 1.82 |
| 15000. | 1.23 | 1.56 |
| 16000. | 0.97 | 1.49 |
| 17000. | 0.84 | 1.23 |
| 18000. | 0.65 | 0.97 |
| 19000. | 0.45 | 0.52 |
| 20000. | 0.45 | 0.45 |
| 21000. | 0.39 | 0.26 |
| 22000. | 0.39 | 0.00 |
| 23000. | 0.32 | 0.00 |
| 24000. | 0.26 | 0.00 |
| 25000. | 0.26 | 0.00 |
| 26000. | 0.26 | 0.00 |
| 27000. | 0.13 | 0.00 |
| 28000. | 0.06 | 0.00 |
| 29000. | 0.00 | 0.00 |

Table A.23. Pre- and Postproject Flow-Duration Data for Yolo, 1978 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 59.42 | 72.46 |
| 2000. | 52.17 | 49.28 |
| 3000. | 39.13 | 27.54 |
| 4000. | 11.59 | 14.49 |
| 5000. | 5.80 | 10.14 |
| 6000. | 1.45 | 5.80 |
| 7000. | 0.00 | 1.45 |
| 8000. | 0.00 | 0.00 |

Table A.24. Pre- and Postproject Flow-Duration Data for Yolo, 1983 Runoff Season.

| Discharge (cfs) | Percent Time Exceeded | |
|--------------------|-----------------------|-------------|
| | Preproject | Postproject |
| 1000. | 91.12 | 91.92 |
| 2000. | 86.27 | 71.74 |
| 3000. | 85.46 | 64.40 |
| 4000. | 73.35 | 61.98 |
| 5000. | 61.64 | 53.30 |
| 6000. | 51.14 | 43.61 |
| 7000. | 37.95 | 32.77 |
| 8000. | 21.53 | 26.04 |
| 9000. | 17.97 | 22.61 |
| 10000. | 13.46 | 18.98 |
| 11000. | 12.05 | 18.03 |
| 12000. | 7.54 | 15.34 |
| 13000. | 6.66 | 13.26 |
| 14000. | 6.06 | 10.50 |
| 15000. | 5.85 | 8.48 |
| 16000. | 5.18 | 7.60 |
| 17000. | 4.31 | 5.92 |
| 18000. | 2.76 | 5.38 |
| 19000. | 2.02 | 5.18 |
| 20000. | 1.62 | 4.85 |
| 21000. | 1.35 | 3.50 |
| 22000. | 0.94 | 2.56 |
| 23000. | 0.94 | 1.28 |
| 24000. | 0.81 | 1.01 |
| 25000. | 0.74 | 0.94 |
| 26000. | 0.67 | 0.81 |
| 27000. | 0.54 | 0.67 |
| 28000. | 0.40 | 0.47 |
| 29000. | 0.34 | 0.40 |
| 30000. | 0.20 | 0.34 |
| 31000. | 0.13 | 0.13 |
| 32000. | 0.07 | 0.07 |
| 33000. | 0.00 | 0.00 |

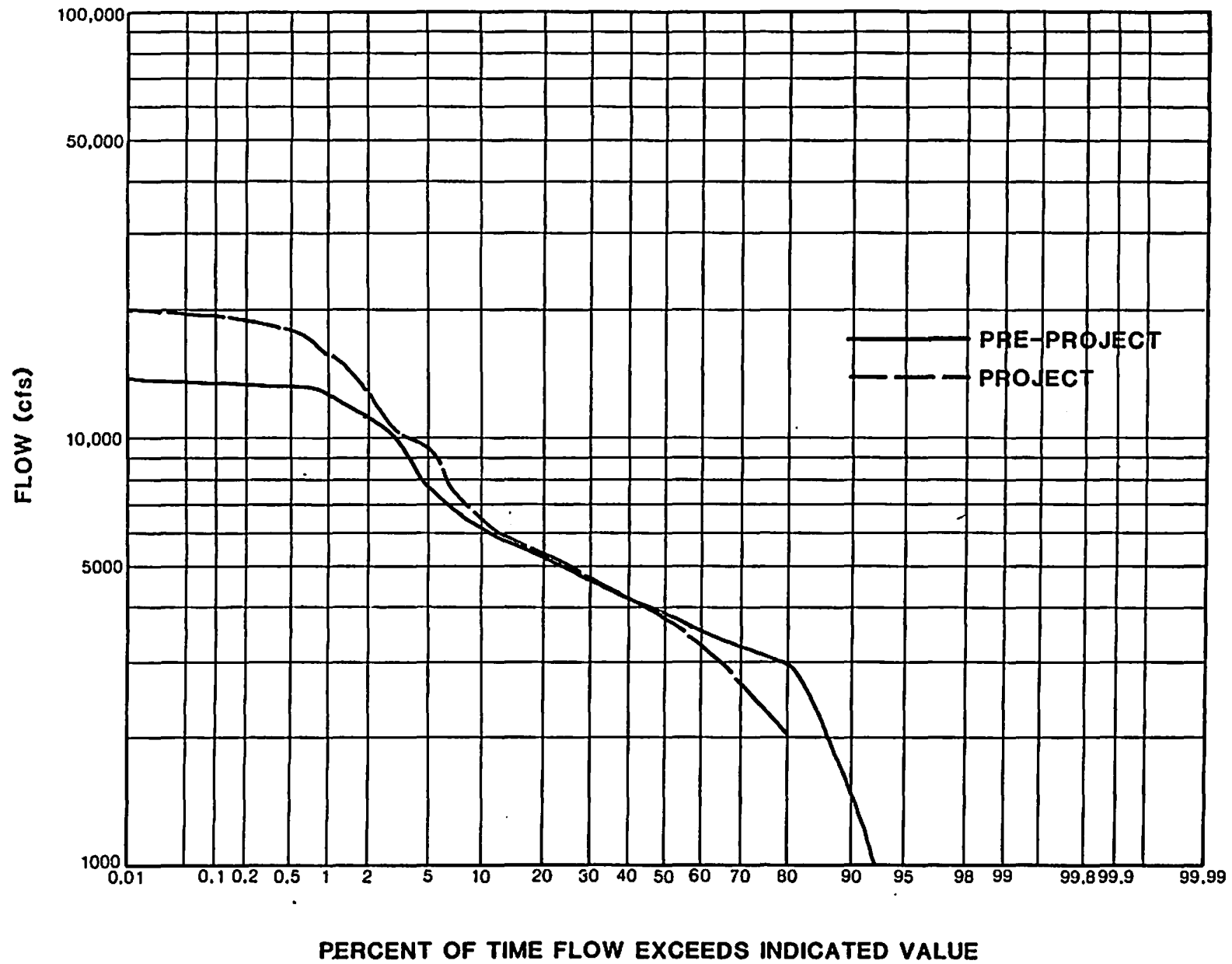


Figure A.1. Pre- and postproject flow-duration curves computed at Rumsey for 1952 flows.

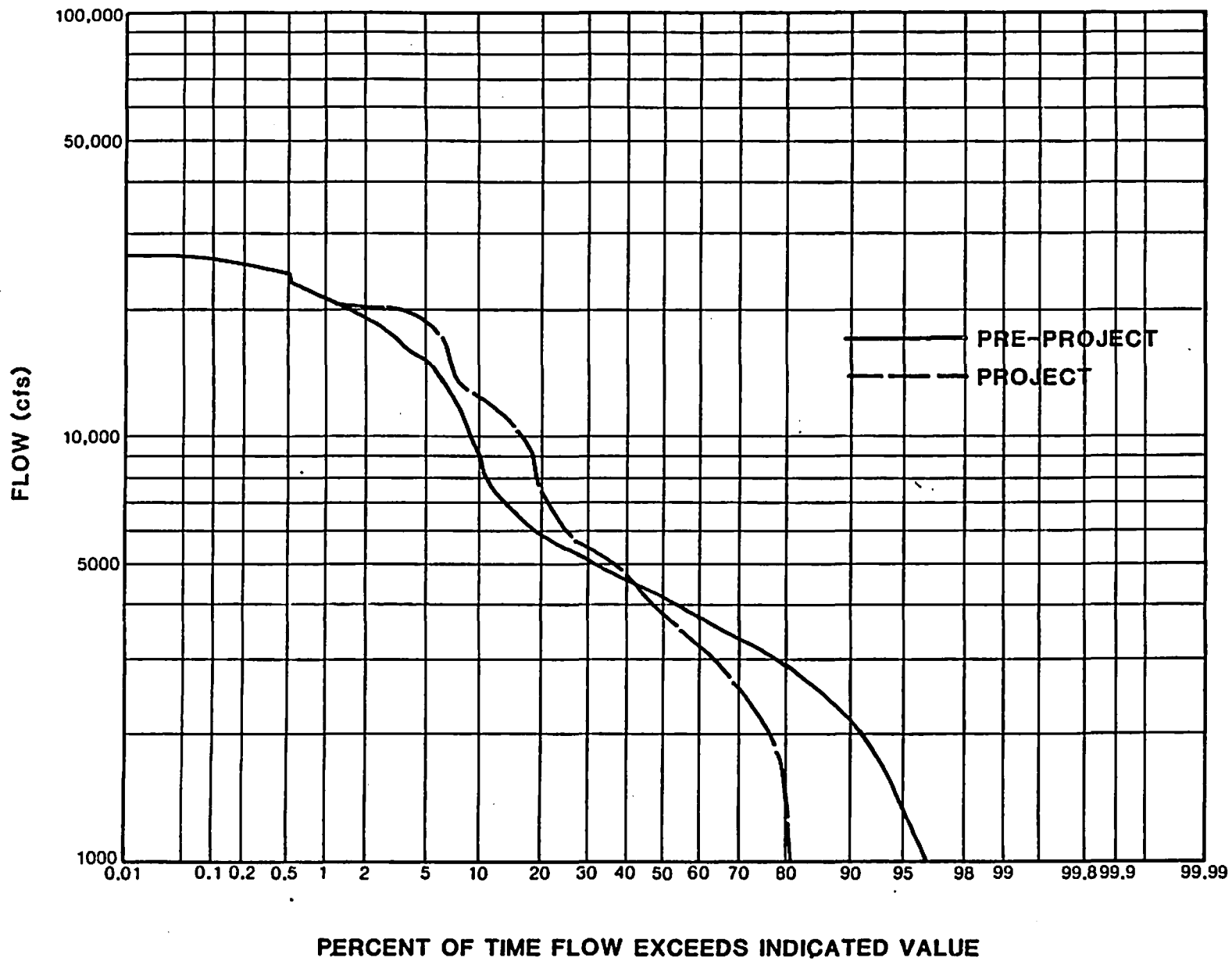


Figure A.2. Pre- and postproject flow-duration curves computed at Rumsey for 1956 flows.

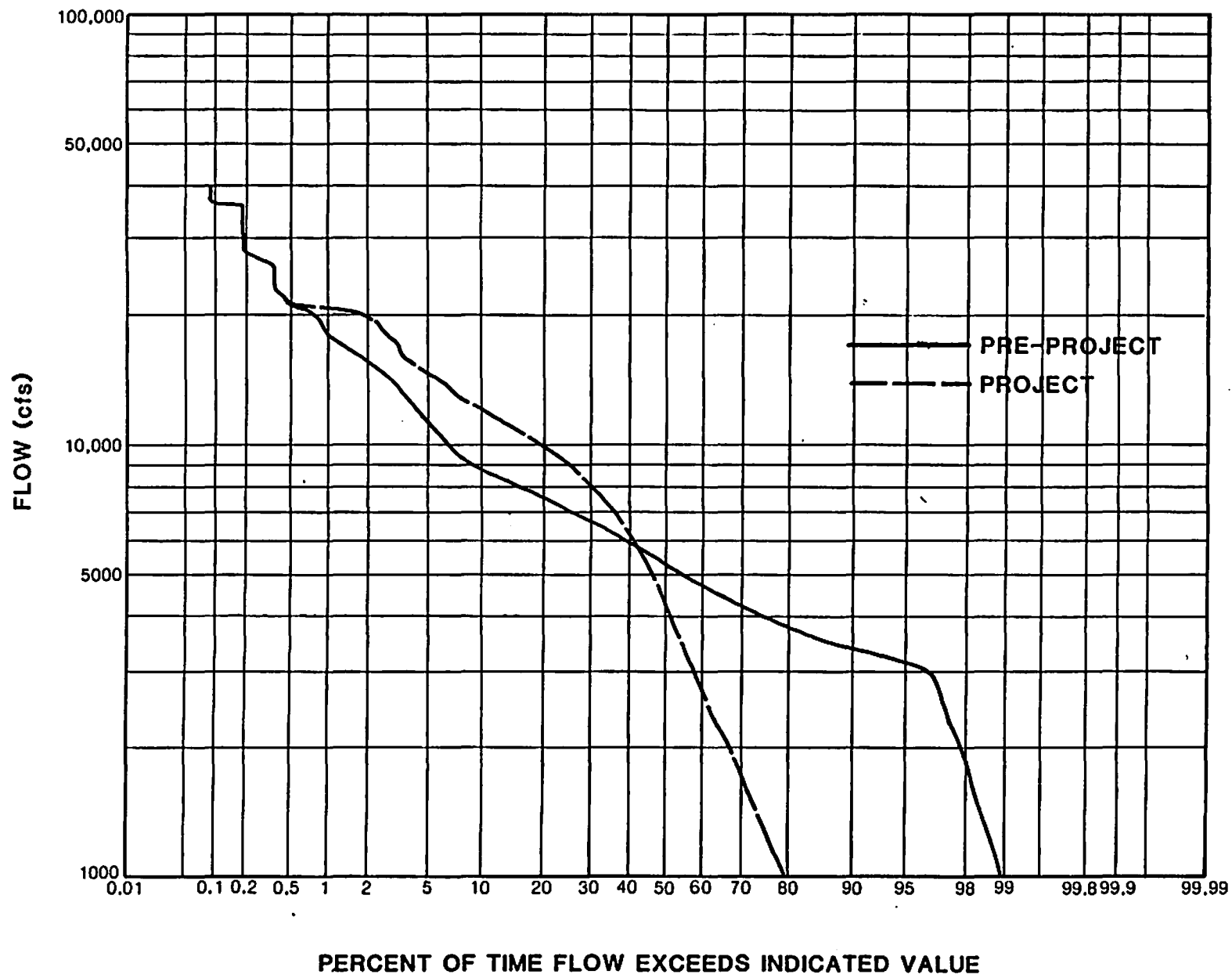


Figure A.3. Pre- and postproject flow-duration curves computed at Rumsey for 1958 flows.

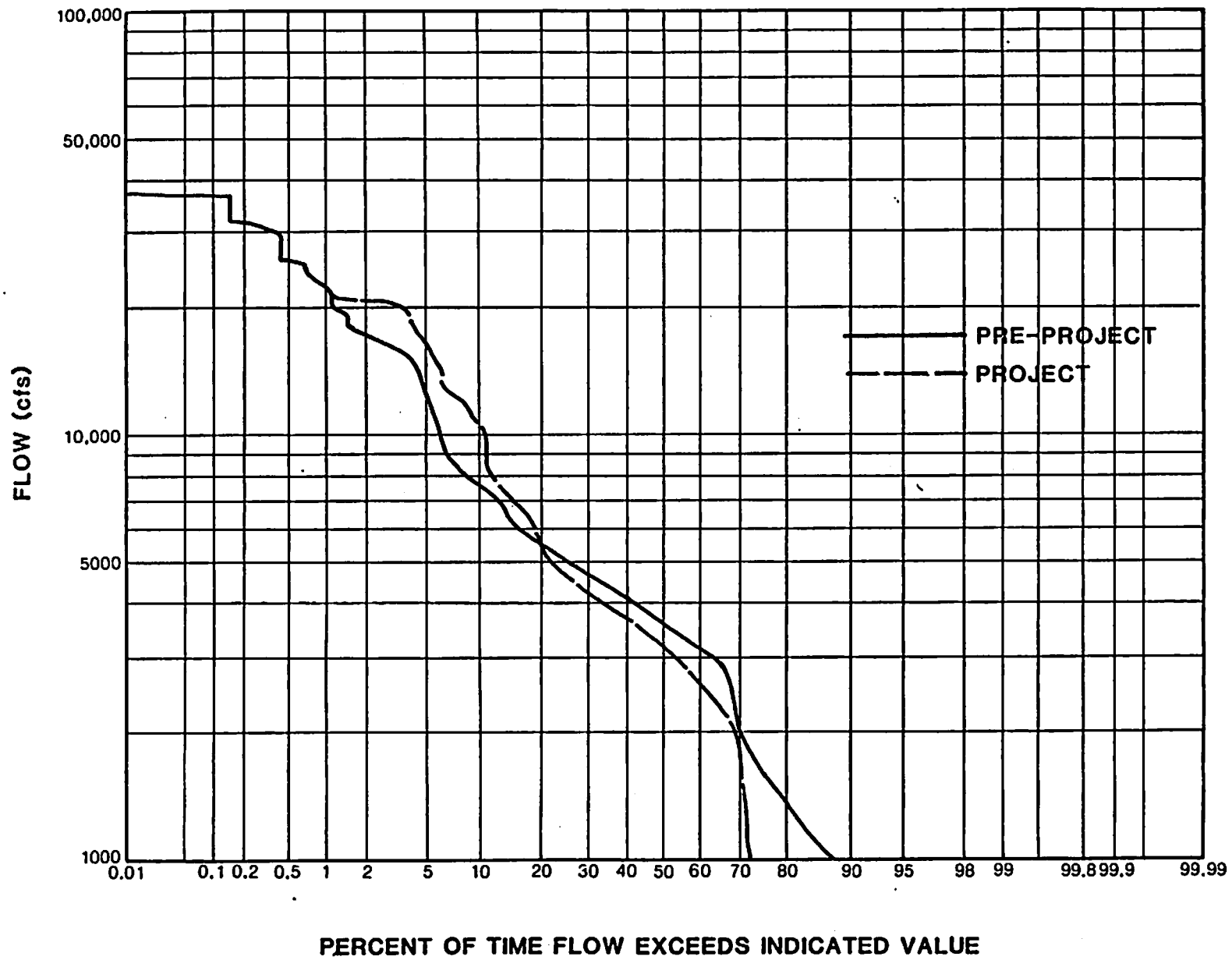


Figure A.4. Pre- and postproject flow-duration curves computed at Rumsey for 1965 flows.

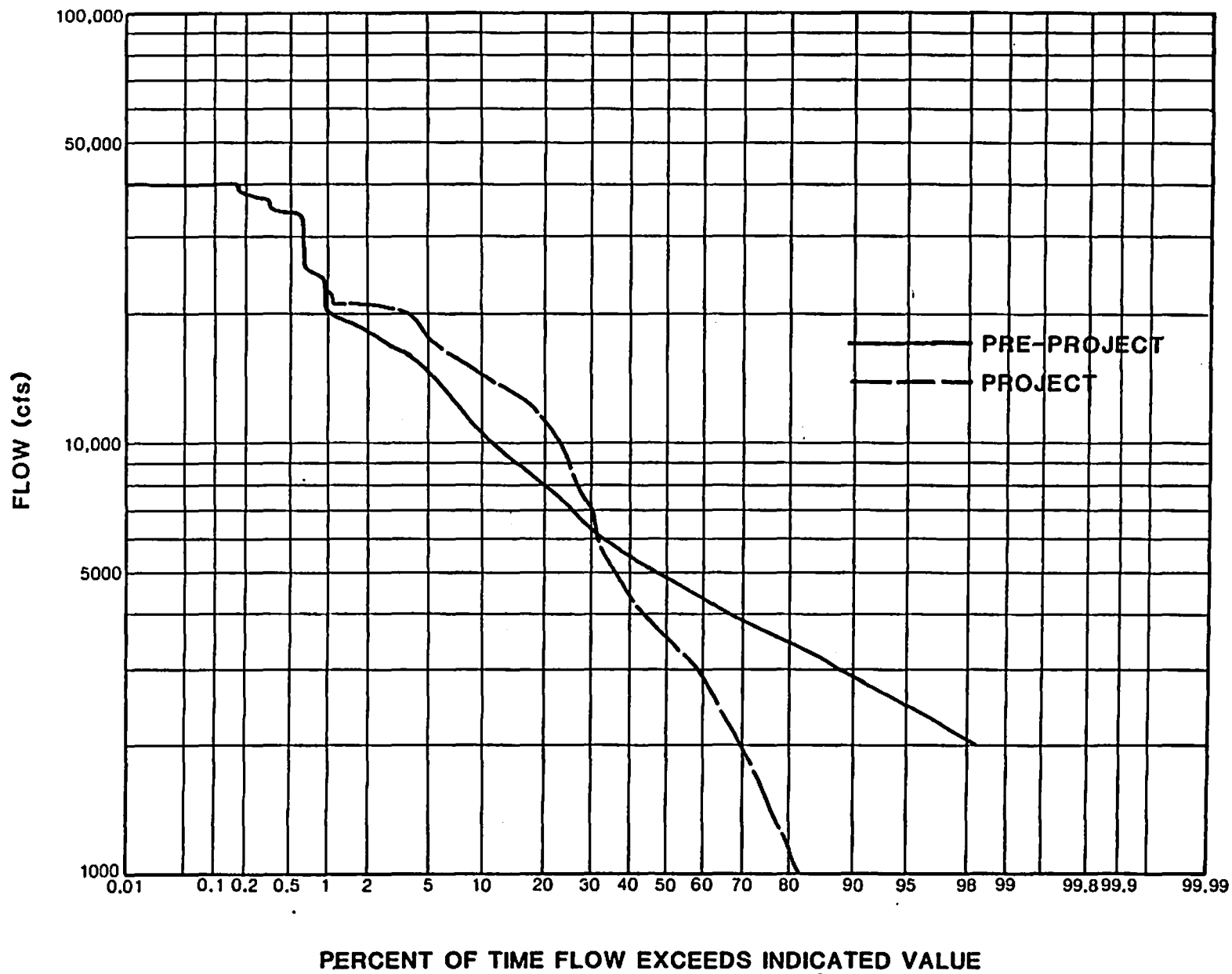


Figure A.5. Pre- and postproject flow-duration curves computed at Rumsey for 1970 flows.

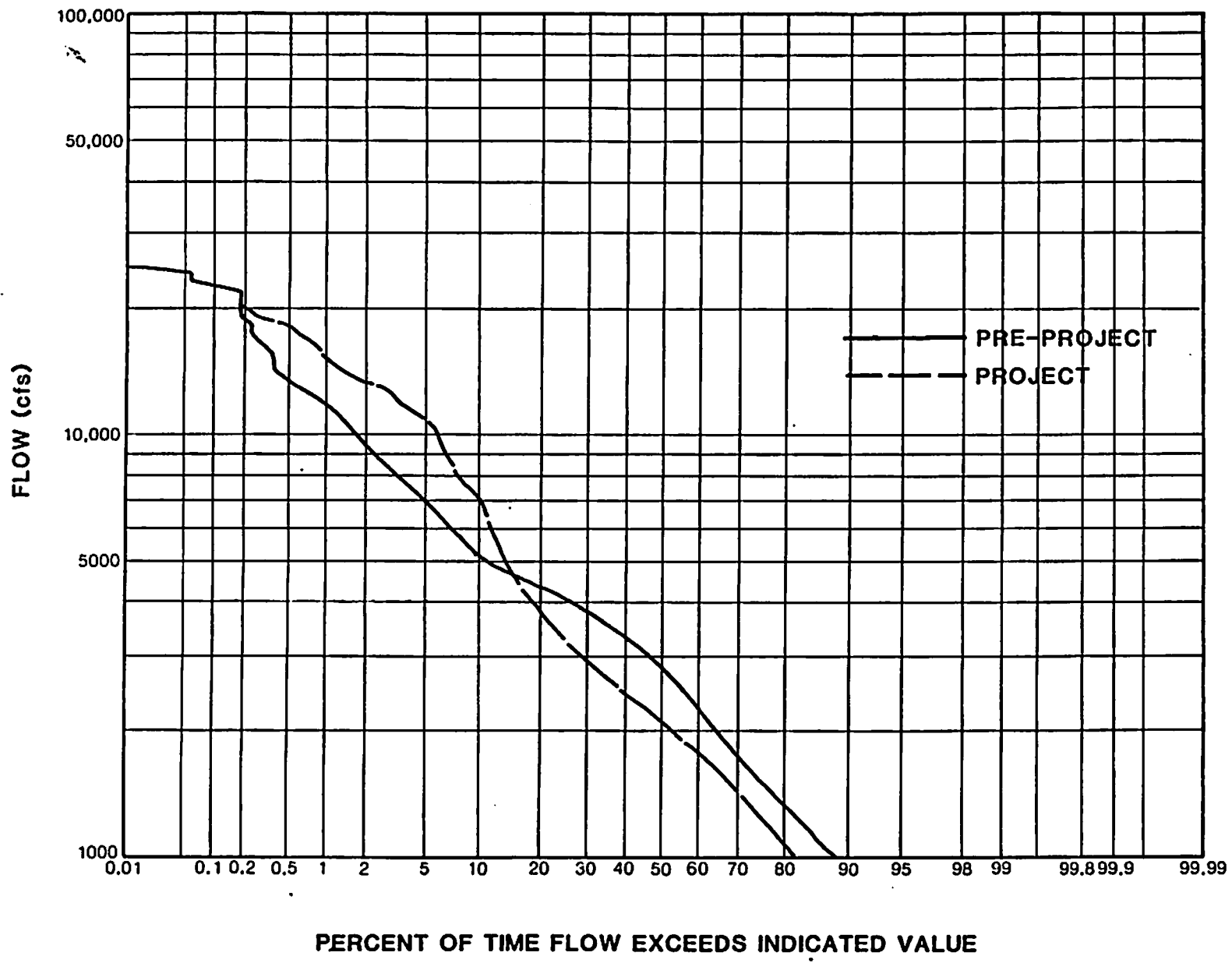


Figure A.6. Pre- and postproject flow-duration curves computed at Rumsey for 1974 flows.

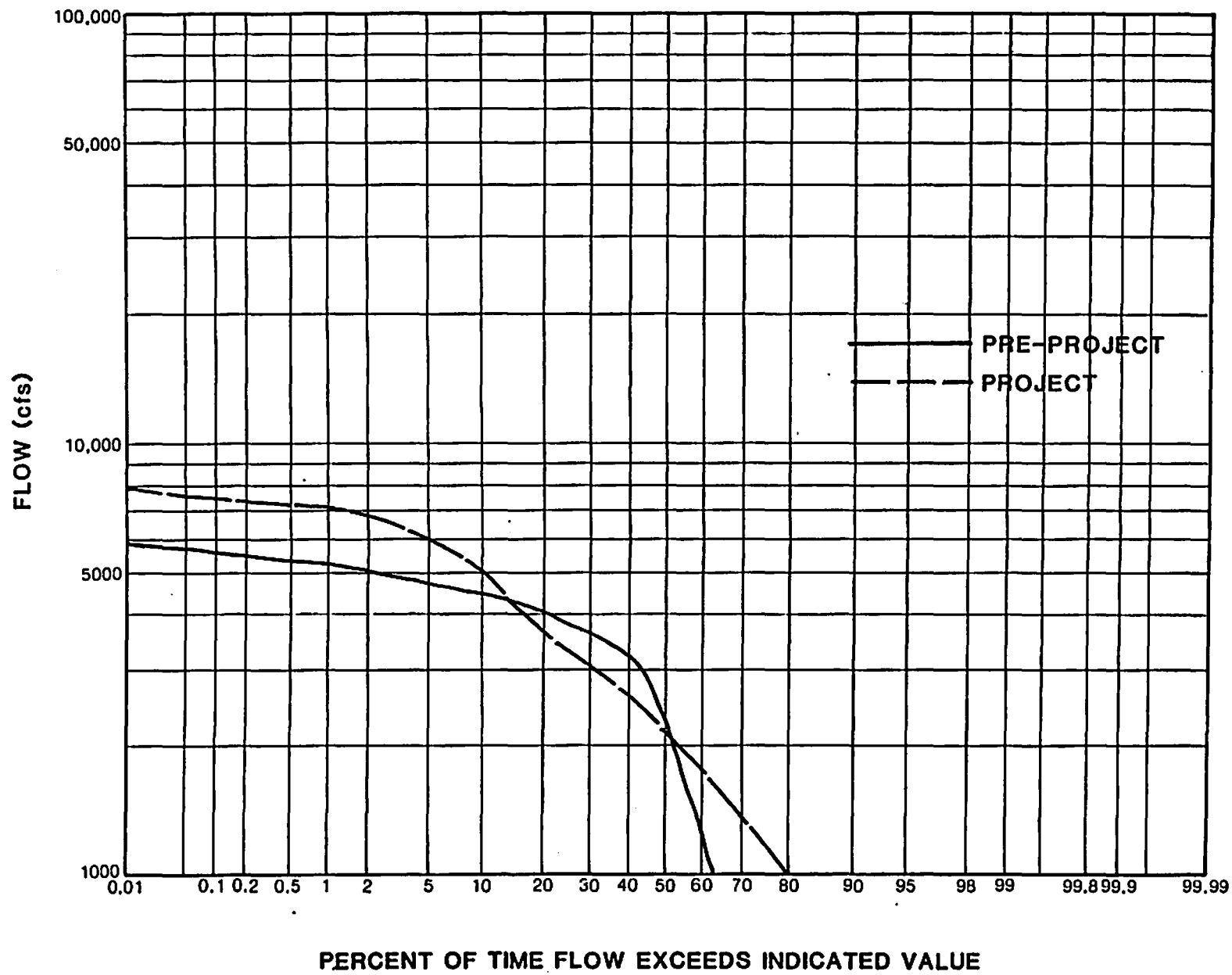


Figure A.7. Pre- and postproject flow-duration curves computed at Rumsey for 1978 flows.

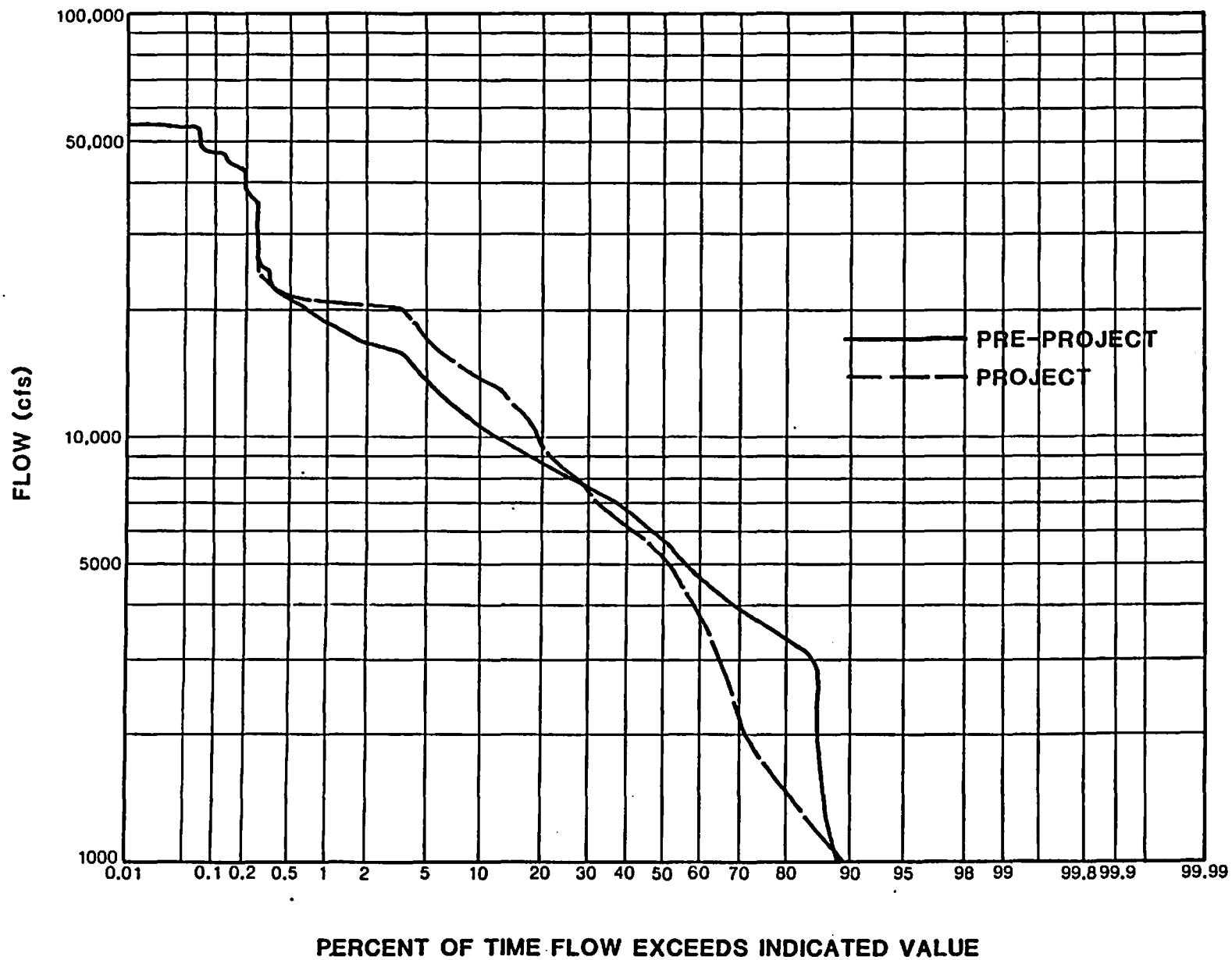


Figure A.8. Pre- and postproject flow-duration curves computed at Rumsey for 1983 flows.

APPENDIX B

Procedure for developing sediment-
transport relationships

APPENDIX B

Procedure for Developing Sediment-
Transport Relationships

B.1 GENERAL

Transport of the bed-material load in a river channel is divided into two zones. The sediment moving in a layer close to the bed is referred to as the bed load. The sediment carried in the remaining upper region of the flow is referred to as the suspended bed load. The total bed-material load is the sum of the bed load and suspended bed load. The turbulent mixing process and the action of gravity on the sediment particles cause continual transfer between the two zones. Although there is no a distinct line between the zones, the definitions are made in order to aid in the mathematical description of the process. The wash load occupies the entire depth of flow but consists of fine particles that are not present in the bed in appreciable quantities, and will not easily settle out. Wash load is primarily controlled by the supply of fine silts and clays from the watershed, while bed load and suspended bed load are primarily controlled by the transport capacity of the river and availability of material in the channel bed and banks.

Sediments of different sizes will experience different rates of transport. Therefore, the transport capacities for a range of sediment sizes are determined and totaled to produce an acceptable determination of total transport capacity. The total bed-material transport capacity for a channel section is

$$Q_s = T \sum P_i (q_{bi} + q_{si}) \quad (1)$$

In Equation 1, T is the top width of the channel, P_i is the fraction of one sediment size, q_{bi} is the bed-load transport rate per unit width for the i th size, and q_{si} is the suspended load transport rate for the i th size.

B.2 BED-LOAD TRANSPORT CAPACITY

The Meyer-Peter, Muller formula gives good results for bed-load transport over a wide range of sediment sizes. The Meyer-Peter, Muller formula is well suited to model the dynamics of channel armoring processes as well as transport of sand sizes with little armoring potential. The formula is

$$q_{bi} = \frac{12.85}{\sqrt{\rho} \gamma_s} (\tau_o - \tau_c)^{1.5} \quad (2)$$

in which

$$\tau_c = F_* (\gamma_s - \gamma) d_{si} \quad (3)$$

In Equations 2 and 3, q_{bi} is the bed-load transport rate in volume per unit width for a specific size of sediment, τ_c is the critical tractive force necessary to initiate particle motion, ρ is the density of water, γ_s is the specific weight of sediment, γ is the specific weight of water, d_s is the sediment size, and F_* is the Shield's parameter which ranges between 0.030 and 0.060 inclusive.

The boundary shear stress acting on the grain is

$$\tau_o = \frac{f_o}{8} \rho V^2 \quad (4)$$

where ρ is the density of flowing water and f_o is the Darcy-Weisbach friction factor, and V is the mean velocity of the flow.

The Darcy-Weisbach friction factor is related to Manning's n by:

$$f_o = n^2 \frac{8g}{1.49^2 R^{1/3}} \quad (5)$$

where g is the acceleration due to gravity and R is hydraulic radius. Assuming the wide channel approximation is valid, then the hydraulic radius R is equal to depth.

For sediment-transport calculations, Manning's n in Equation 5 represents the skin resistance only, which is typically 0.6 to 0.9 times the total Manning's n for the channel depending upon the preponderance and type of bed forms.

B.3 SUSPENDED BED-MATERIAL TRANSPORT CAPACITY

The suspended sediment-transport capacity is determined by using a solution developed by Einstein. This method relies upon integration of the sediment concentration profile as a function of depth. The nature of the profile is determined using turbulent transport theory. The sediment profile is assumed to be in equilibrium, and therefore the rate at which sediment is transported upward due to turbulence and the concentration gradient is exactly equal to the rate at which gravity is transporting sediment downward. The resulting concentration profile is given by:

$$\frac{C}{C_a} = \left[\frac{Y-y}{y} \frac{a}{Y-a} \right]^z \quad (6)$$

where C is the concentration at a point y , C_a is the concentration at point a generally considered to be the bed layer thickness, Y is the depth and z is given by:

$$z = \frac{w}{\kappa U_*} \quad (7)$$

where w is the fall velocity for a given size sediment, U_* is the shear velocity, and κ is the von Karman constant (0.4). U_* is defined as $\sqrt{\tau/\rho}$ where τ is the shear stress on the bed and ρ is the density of the fluid. (Note: τ should be the total shear stress on the bed and not the grain-associated shear stress. The shear velocity U_* characterizes both the turbulence and grain resistance, and the fall velocity, w , characterizes the sediment properties.)

A logarithmic velocity profile is used to describe the velocity distribution in turbulent flows. The equation utilized is

$$\frac{U_\xi}{U_*} = B + 2.5 \ln \left(\frac{\xi}{\eta_s} \right) \quad (8)$$

in which u_ξ is the point mean velocity at the distance ξ from the bed, B is a constant dependent on roughness, and η_s is the roughness height.

The integral of suspended load above the bed layer is obtained by combining Equations 6 and 7 or

$$q_{si} = \int_a^Y u_\xi C_\xi d\xi = C_a U_* \int_a^Y \left[B + 2.5 \ln \left(\frac{\xi}{\eta_s} \right) \right] \left(\frac{Y-\xi}{\xi} \frac{a}{Y-a} \right)^z d\xi \quad (9)$$

where q_{si} is the suspended bed-material transport for a specific size sediment above the bed layer. In this equation

$$\sigma = \frac{\xi}{Y} \quad (10)$$

where σ is the dimensionless relative depth and

$$G = \frac{a}{Y}$$

in which G is the relative depth of the bed layer. Integration of Equation 9 yields

$$q_{si} = C_a U_* a \frac{G^{z-1}}{(1-G)^z} \left\{ [B + 2.5 \ln\left(\frac{Y}{\eta_s}\right)] \int_G^1 \left(\frac{1-\sigma}{\sigma}\right)^z d\sigma + 2.5 \int_G^1 \left(\frac{1-s}{\sigma}\right)^z \ln\sigma d\sigma \right\} \quad (11)$$

The average flow velocity V is defined by the equation

$$V = \frac{\int_0^Y u_\xi d\xi}{\int_0^Y d\xi} \quad (12)$$

Substituting Equation 8 into Equation 12 and integrating yields

$$\frac{V}{U_*} = B + 2.5 \ln\left(\frac{Y}{\eta_s}\right) - 2.5 \quad (13)$$

The two integrals in Equation 11 can be defined as

$$J_1 = \int_G^1 \left(\frac{1-\sigma}{\sigma}\right)^z d\sigma \quad (14)$$

and

$$J_2 = \int_G^1 \left(\frac{1-\sigma}{\sigma}\right)^z \ln\sigma d\sigma \quad (15)$$

Substituting Equations 13, 14, and 15 reduces Equation 11 to

$$q_{si} = C_a U_* a \frac{G^{z-1}}{1-G^z} \left[\left(\frac{V}{U_*} + 2.5\right) J_1 + 2.5 J_2 \right] \quad (16)$$

According to Einstein, the concentration at the upper level of the bed layer is related to the total transport in the bed layer by

$$q_{bi} = 11.6 C_a U_* a \quad (17)$$

Substituting into Equation 16 produces

$$q_{si} = \frac{q_{bi}}{11.6} \frac{G^{z-1}}{1-G^z} \left[\left(\frac{V}{U_*} + 2.5\right) J_1 + 2.5 J_2 \right] \quad (18)$$

Generally sediment measurements are taken in a zone approximately 0.3 feet above the bed and higher since commonly used measuring devices are incapable of getting closer to the bed. Hence, commonly reported sediment

measurements consist of wash load and suspended bed load. This zone of measured sediment transport will be denoted the "measured zone" in the following discussion.

For practical purposes wash load consists of no particles greater than 0.062 mm. If size analyses of suspended sediment and corresponding concentration measurements have been made, then the concentration of suspended bed material in the measured zone is given by

$$C_{bm} = P * C_m \quad (19)$$

where C_{bm} is the concentration of suspended bed material in the measured zone, P is percent of suspended bed-material load and C_m is total measured concentration.

Now C_{bm} may be calculated theoretically using the following procedure. First the suspended bed-material concentration at 0.3 feet above the bed is calculated from:

$$C_{0.3} = C_a \left(\frac{Y - 0.3}{0.3} \frac{a}{Y - a} \right)^z \quad (20)$$

Substituting this concentration into Equation 16 and changing the limits of integration appropriately yields the transport rate of suspended bed material for a specific size in the zone extending from 0.3 feet above the bed to the water surface. The resulting relation is

$$q_{0.3i} = C_{0.3} U_* 0.3 \frac{G'^z - 1}{(1 - G')^z} \left[\left(\frac{V}{U_*} + 2.5 \right) J_1' + 2.5 J_2' \right] \quad (21)$$

where

$$G' = \frac{0.3}{Y} \quad (22)$$

$$J_1' = \int_{G'}^1 \left(\frac{1 - \sigma}{\sigma} \right)^z d\sigma \quad (23)$$

and

$$J_2' = \int_{G'}^1 \left(\frac{1 - \sigma}{\sigma} \right)^z \ln \sigma d\sigma \quad (24)$$

Rewriting Equation 21 and using Equation 20 produces the relation

$$q_{0.3i} = C_a U_*^{0.3} \left(\frac{Y-0.3}{0.3} \frac{a}{y-a} \right)^z \frac{G'^{z-1}}{(1-G')^z} \left[\left(\frac{V}{U_*} + 2.5 \right) J_1' + 2.5 J_2' \right] \quad (25)$$

Introducing Equation 17 modifies the foregoing relation to read

$$q_{0.3i} = \frac{q_{bi}}{11.6} \frac{0.3}{a} \left(\frac{Y-0.3}{0.3} \right)^z \frac{G'^{z-1}}{(1-G')^z} \left[\left(\frac{V}{U_*} + 2.5 \right) J_1' + 2.5 J_2' \right] \quad (26)$$

The total concentration of suspended bed material in the measured zone is given by

$$C'_{bm} = \sum_{i=1}^n (P_i * q_{0.3i}) / q * S.G. \quad (27)$$

where C'_{bm} is the theoretically computed bed-material concentration, q is water discharge, and $S.G.$ is the specific gravity of the sediment, n is the total number of sediment sizes.

Comparing C_{bm} and C'_{bm} for a wide range of discharges yields a measure of the accuracy of the theoretical computation procedure.

This procedure can be "calibrated" so that the C'_{bm} values agree closely with the C_{bm} values by adjusting one or more parameters involved in the procedure. These parameters are:

1. Shield's parameter F_* . This is generally assumed equal to 0.047 but may range in value from 0.030 to approximately 0.060.
2. Manning's n for skin resistance only. Typical values are 0.01 to 0.03. This parameter may be estimated from Stickler's relation:

$$n = \frac{D_{90}^{1/6}}{26}$$

where D_{90} is in meters.

3. Bed layer thickness a . This is an extremely important parameter since it determines the concentration C_a which defines the suspended bed-material concentration profile. The value of a is typically taken as two times the D_{65} or D_{90} of the bed material.
4. As a last resort, the exponent 1.5 and the coefficient 12.85 in Equation 2 may be adjusted.

The next step is to use to "calibrated" transport procedure to generate values for total bed-material transport for a wide range of river discharges. HEC-2 was used to determine the hydraulics for these discharges. A regression analysis was performed on the resulting data to obtain a relation of the form:

$$q_s = a v^b Y^c \quad (28)$$

where q_s is unit bed-material transport, v is velocity, Y is depth, and a , b , and c are coefficients.

The calibrated transport relations are then applied using the average hydraulic and sediment conditions in each of the computational reaches to determine the bed-material rating curves.

$$Q_s = a Q^b \quad (29)$$

for each reach, where Q_s is total bed-material transport within the reach and Q is river discharge. These relationships form the basis of the continuity calculations.

APPENDIX C

Sediment-continuity results for
individual runoff seasons

Table C.1. Sediment-Continuity Results for the 1952
Runoff Season, Preproject Conditions.

| Reach | Upstream Supply (1,000 ft ³) | Transport Capacity (1,000 ft ³) | Erosion/Sedimentation* Volume (1,000 ft ³) |
|-------|--|---|--|
| 1 | -----Supply Reach----- | | |
| 2 | 1,136 | 527 | 609 |
| 3 | 527 | 759 | -232 |
| 4 | 759 | 399 | 360 |
| 5 | 399 | 381 | 18 |
| 6 | 381 | 1,022 | -641 |
| 7 | 1,022 | 851 | 171 |
| 8 | 851 | 1,326 | -475 |
| 9 | 1,326 | 1,774 | -448 |
| 10 | 1,774 | 2,219 | -444 |

* Negative numbers denote erosion.

Table C.2. Sediment-Continuity Results for the 1952
Runoff Season, Project Conditions.

| Reach | Upstream Supply (1,000 ft ³) | Transport Capacity (1,000 ft ³) | Erosion/Sedimentation* Volume (1,000 ft ³) |
|-------|--|---|--|
| 1 | -----Supply Reach----- | | |
| 2 | 1,205 | 545 | 659 |
| 3 | 545 | 788 | -242 |
| 4 | 788 | 407 | 381 |
| 5 | 407 | 394 | 13 |
| 6 | 394 | 1,051 | -657 |
| 7 | 1,051 | 890 | 161 |
| 8 | 890 | 1,382 | -492 |
| 9 | 1,382 | 1,819 | -437 |
| 10 | 1,819 | 2,253 | -434 |

* Negative numbers denote erosion.

Table C.3. Sediment-Continuity Results for the 1956
Runoff Season, Preproject Conditions.

| Reach | Upstream Supply (1,000 ft ³) | Transport Capacity (1,000 ft ³) | Erosion/Sedimentation* Volume (1,000 ft ³) |
|-------|--|---|--|
| 1 | -----Supply Reach----- | | |
| 2 | 2,662 | 1,140 | 1,522 |
| 3 | 1,140 | 1,607 | -467 |
| 4 | 1,607 | 814 | 793 |
| 5 | 814 | 798 | 16 |
| 6 | 798 | 2,404 | -1,606 |
| 7 | 2,404 | 1,902 | 502 |
| 8 | 1,902 | 2,916 | -1,015 |
| 9 | 2,916 | 3,522 | -606 |
| 10 | 3,522 | 4,749 | -1,226 |

* Negative numbers denote erosion.

Table C.4. Sediment-Continuity Results for the 1956
Runoff Season, Project Conditions.

| Reach | Upstream Supply (1,000 ft ³) | Transport Capacity (1,000 ft ³) | Erosion/Sedimentation* Volume (1,000 ft ³) |
|-------|--|---|--|
| 1 | -----Supply Reach----- | | |
| 2 | 3,151 | 1,270 | 1,881 |
| 3 | 1,270 | 1,731 | -461 |
| 4 | 1,731 | 849 | 882 |
| 5 | 849 | 854 | -5 |
| 6 | 854 | 2,722 | -1,869 |
| 7 | 2,722 | 2,088 | 634 |
| 8 | 2,088 | 3,146 | -1,058 |
| 9 | 3,146 | 3,612 | -465 |
| 10 | 3,612 | 5,000 | -1,389 |

* Negative numbers denote erosion.

Table C.5. Sediment-Continuity Results for the 1958
Runoff Season, Preproject Conditions.

| Reach | Upstream Supply (1,000 ft ³) | Transport Capacity (1,000 ft ³) | Erosion/Sedimentation* Volume (1,000 ft ³) |
|-------|--|---|--|
| 1 | -----Supply Reach----- | | |
| 2 | 3,705 | 1,514 | 2,190 |
| 3 | 1,514 | 2,088 | -573 |
| 4 | 2,088 | 997 | 1,091 |
| 5 | 997 | 1,040 | -43 |
| 6 | 1,040 | 3,174 | -2,134 |
| 7 | 3,174 | 2,494 | 680 |
| 8 | 2,494 | 3,751 | -1,257 |
| 9 | 3,751 | 4,616 | -865 |
| 10 | 4,616 | 6,496 | -1,880 |

* Negative numbers denote erosion.

Table C.6. Sediment-Continuity Results for the 1958
Runoff Season, Project Conditions.

| Reach | Upstream Supply (1,000 ft ³) | Transport Capacity (1,000 ft ³) | Erosion/Sedimentation* Volume (1,000 ft ³) |
|-------|--|---|--|
| 1 | -----Supply Reach----- | | |
| 2 | 4,093 | 1,573 | 2,520 |
| 3 | 1,573 | 2,075 | -502 |
| 4 | 2,075 | 973 | 1,101 |
| 5 | 973 | 1,029 | -56 |
| 6 | 1,029 | 3,314 | -2,284 |
| 7 | 3,314 | 2,551 | 763 |
| 8 | 2,551 | 3,779 | -1,229 |
| 9 | 3,779 | 4,387 | -607 |
| 10 | 4,387 | 6,258 | -1,871 |

* Negative numbers denote erosion.

Table C.7. Sediment-Continuity Results for the 1965
Runoff Season, Preproject Conditions.

| Reach | Upstream Supply (1,000 ft ³) | Transport Capacity (1,000 ft ³) | Erosion/Sedimentation* Volume (1,000 ft ³) |
|-------|--|---|--|
| 1 | -----Supply Reach----- | | |
| 2 | 1,513 | 651 | 862 |
| 3 | 651 | 926 | -276 |
| 4 | 926 | 471 | 455 |
| 5 | 471 | 460 | 11 |
| 6 | 460 | 1,374 | -913 |
| 7 | 1,374 | 1,085 | 289 |
| 8 | 1,085 | 1,691 | -606 |
| 9 | 1,691 | 2,042 | -351 |
| 10 | 2,042 | 2,690 | -648 |

* Negative numbers denote erosion.

Table C.8. Sediment-Continuity Results for the 1965
Runoff Season, Project Conditions.

| Reach | Upstream Supply (1,000 ft ³) | Transport Capacity (1,000 ft ³) | Erosion/Sedimentation* Volume (1,000 ft ³) |
|-------|--|---|--|
| 1 | -----Supply Reach----- | | |
| 2 | 1,645 | 684 | 962 |
| 3 | 684 | 955 | -272 |
| 4 | 955 | 485 | 470 |
| 5 | 485 | 473 | 12 |
| 6 | 473 | 1,476 | -1,003 |
| 7 | 1,476 | 1,138 | 338 |
| 8 | 1,138 | 1,744 | -606 |
| 9 | 1,744 | 2,015 | -270 |
| 10 | 2,015 | 2,683 | -668 |

* Negative numbers denote erosion.

Table C.9. Sediment-Continuity Results for the 1970
Runoff Season, Preproject Conditions.

| Reach | Upstream Supply (1,000 ft ³) | Transport Capacity (1,000 ft ³) | Erosion/Sedimentation* Volume (1,000 ft ³) |
|-------|--|---|--|
| 1 | -----Supply Reach----- | | |
| 2 | 2,244 | 907 | 1,337 |
| 3 | 907 | 1,243 | -336 |
| 4 | 1,243 | 600 | 643 |
| 5 | 600 | 602 | -2 |
| 6 | 602 | 1,821 | -1,219 |
| 7 | 1,821 | 1,416 | 405 |
| 8 | 1,416 | 2,124 | -708 |
| 9 | 2,124 | 2,587 | -463 |
| 10 | 2,587 | 3,579 | -992 |

* Negative numbers denote erosion.

Table C.10. Sediment-Continuity Results for the 1970
Runoff Season, Project Conditions.

| Reach | Upstream Supply (1,000 ft ³) | Transport Capacity (1,000 ft ³) | Erosion/Sedimentation* Volume (1,000 ft ³) |
|-------|--|---|--|
| 1 | -----Supply Reach----- | | |
| 2 | 2,419 | 943 | 1,476 |
| 3 | 943 | 1,261 | -319 |
| 4 | 1,261 | 605 | 657 |
| 5 | 605 | 609 | -4 |
| 6 | 609 | 1,976 | -1,366 |
| 7 | 1,976 | 1,493 | 483 |
| 8 | 1,493 | 2,224 | -731 |
| 9 | 2,224 | 2,502 | -278 |
| 10 | 2,502 | 3,513 | -1,011 |

* Negative numbers denote erosion.

Table C.11. Sediment-Continuity Results for the 1974
Runoff Season, Preproject Conditions.

| Reach | Upstream Supply (1,000 ft ³) | Transport Capacity (1,000 ft ³) | Erosion/Sedimentation* Volume (1,000 ft ³) |
|-------|--|---|--|
| 1 | -----Supply Reach----- | | |
| 2 | 1,972 | 956 | 1,016 |
| 3 | 956 | 1,419 | -463 |
| 4 | 1,419 | 766 | 653 |
| 5 | 766 | 713 | 53 |
| 6 | 713 | 1,718 | -1,005 |
| 7 | 1,718 | 1,534 | 184 |
| 8 | 1,534 | 2,470 | -936 |
| 9 | 2,470 | 3,395 | -924 |
| 10 | 3,395 | 3,801 | -407 |

* Negative numbers denote erosion.

Table C.12. Sediment-Continuity Results for the 1974
Runoff Season, Project Conditions.

| Reach | Upstream Supply (1,000 ft ³) | Transport Capacity (1,000 ft ³) | Erosion/Sedimentation* Volume (1,000 ft ³) |
|-------|--|---|--|
| 1 | -----Supply Reach----- | | |
| 2 | 2,170 | 988 | 1,182 |
| 3 | 988 | 1,411 | -423 |
| 4 | 1,411 | 744 | 667 |
| 5 | 744 | 708 | 36 |
| 6 | 708 | 1,790 | -1,083 |
| 7 | 1,790 | 1,568 | 223 |
| 8 | 1,568 | 2,482 | -914 |
| 9 | 2,482 | 3,255 | -773 |
| 10 | 3,255 | 3,662 | -408 |

* Negative numbers denote erosion.

Table C.13. Sediment-Continuity Results for the 1978
Runoff Season, Preproject Conditions.

| Reach | Upstream Supply (1,000 ft ³) | Transport Capacity (1,000 ft ³) | Erosion/Sedimentation* Volume (1,000 ft ³) |
|-------|--|---|--|
| 1 | -----Supply Reach----- | | |
| 2 | 662 | 361 | 301 |
| 3 | 361 | 552 | -191 |
| 4 | 552 | 320 | 232 |
| 5 | 320 | 292 | 28 |
| 6 | 292 | 685 | -393 |
| 7 | 685 | 613 | 72 |
| 8 | 613 | 1,009 | -396 |
| 9 | 1,009 | 1,425 | -416 |
| 10 | 1,425 | 1,466 | -41 |

* Negative numbers denote erosion.

Table C.14. Sediment-Continuity Results for the 1978
Runoff Season, Project Conditions.

| Reach | Upstream Supply (1,000 ft ³) | Transport Capacity (1,000 ft ³) | Erosion/Sedimentation* Volume (1,000 ft ³) |
|-------|--|---|--|
| 1 | -----'-----Supply Reach----- | | |
| 2 | 763 | 400 | 363 |
| 3 | 400 | 609 | -209 |
| 4 | 609 | 336 | 273 |
| 5 | 336 | 308 | 28 |
| 6 | 308 | 662 | -354 |
| 7 | 662 | 634 | 28 |
| 8 | 634 | 1,042 | -408 |
| 9 | 1,042 | 1,478 | -436 |
| 10 | 1,478 | 1,481 | -3 |

* Negative numbers denote erosion.

Table C.15. Sediment-Continuity Results for the 1983
Runoff Season, Preproject Conditions.

| Reach | Upstream Supply (1,000 ft ³) | Transport Capacity (1,000 ft ³) | Erosion/Sedimentation* Volume (1,000 ft ³) |
|-------|--|---|--|
| 1 | -----Supply Reach----- | | |
| 2 | 5,866 | 2,289 | 3,576 |
| 3 | 2,289 | 3,072 | -783 |
| 4 | 3,072 | 1,437 | 1,636 |
| 5 | 1,437 | 1,522 | -85 |
| 6 | 1,522 | 4,853 | -3,331 |
| 7 | 4,853 | 3,721 | 1,132 |
| 8 | 3,721 | 5,490 | -1,768 |
| 9 | 5,490 | 6,444 | -954 |
| 10 | 6,444 | 9,143 | -2,699 |

* Negative numbers denote erosion.

Table C.16. Sediment-Continuity Results for the 1983
Runoff Season, Project Conditions.

| Reach | Upstream Supply (1,000 ft ³) | Transport Capacity (1,000 ft ³) | Erosion/Sedimentation* Volume (1,000 ft ³) |
|-------|--|---|--|
| 1 | -----Supply Reach----- | | |
| 2 | 6,040 | 2,344 | 3,696 |
| 3 | 2,344 | 3,144 | -800 |
| 4 | 3,144 | 1,473 | 1,671 |
| 5 | 1,473 | 1,547 | -74 |
| 6 | 1,547 | 5,084 | -3,536 |
| 7 | 5,084 | 3,862 | 1,221 |
| 8 | 3,862 | 5,742 | -1,879 |
| 9 | 5,742 | 6,397 | -656 |
| 10 | 6,397 | 9,136 | -2,739 |

* Negative numbers denote erosion.

APPENDIX D

Potential bank erosion computed
for individual runoff seasons

Table D.1. Potential Bank Erosion Computed Using 1952
Flows, Pre- and Postproject Conditions.

| Cross Section | Shear Stress Duration Index* | | Percent Difference |
|------------------|------------------------------|-------------|-----------------------|
| | Preproject | Postproject | |
| 0.9 | 0 | 0 | 0.0 |
| 2.0 | 274 | 295 | 7.7 |
| 4.0 | 195 | 201 | 3.1 |
| 7.0 | 2,032 | 2,183 | 7.4 |
| 10.0** | 331 | 274 | -17.2 |
| 13.0 | 88 | 117 | 33.0 |
| 17.0 | 90 | 123 | 36.7 |
| 20.0 | 178 | 185 | 3.9 |
| 22.0 | 90 | 119 | 32.2 |
| 24.0** | 1,186 | 1,270 | 7.1 |
| 26.0 | 36 | 52 | 44.4 |
| 28.0 | 149 | 163 | 9.4 |
| 30.0** | 30 | 46 | 53.3 |
| 35A** | 1,497 | 1,580 | 5.5 |
| 36A** | 61 | 73 | 19.7 |
| 37.0** | 247 | 221 | -10.5 |
| 38.0** | 1,037 | 1,107 | 6.8 |
| 40.0 | 346 | 348 | 0.6 |

* Larger number indicates higher erosion potential.

** USGS active erosion sites

Table D.2. Potential Bank Erosion Computed Using 1956
Flows, Pre- and Postproject Conditions.

| Cross Section | Shear Stress Duration Index* | | Percent Difference |
|------------------|------------------------------|-------------|-----------------------|
| | Preproject | Postproject | |
| 0.9 | 0 | 0 | 0.0 |
| 2.0 | 736 | 870 | 18.2 |
| 4.0 | 515 | 612 | 18.8 |
| 7.0 | 4,825 | 5,437 | 12.7 |
| 10.0** | 527 | 418 | -20.7 |
| 13.0 | 311 | 447 | 43.7 |
| 17.0 | 357 | 520 | 45.7 |
| 20.0 | 416 | 466 | 12.0 |
| 22.0 | 311 | 448 | 44.1 |
| 24.0** | 2,321 | 2,324 | 0.1 |
| 26.0 | 176 | 276 | 56.8 |
| 28.0 | 367 | 427 | 16.3 |
| 30.0** | 153 | 252 | 64.7 |
| 35A** | 3,762 | 4,439 | 18.0 |
| 36A** | 186 | 255 | 37.1 |
| 37.0** | 463 | 459 | -0.9 |
| 38.0** | 2,190 | 2,321 | 6.0 |
| 40.0 | 759 | 830 | 9.4 |

* Larger number indicates higher erosion potential.

** USGS active erosion sites

Table D.3. Potential Bank Erosion Computed Using 1958
Flows, Pre- and Postproject Conditions.

| Cross Section | Shear Stress Duration Index* | | Percent Difference |
|------------------|------------------------------|-------------|-----------------------|
| | Preproject | Postproject | |
| 0.9 | 8 | 3 | -62.5 |
| 2.0 | 946 | 1,059 | 11.9 |
| 4.0 | 675 | 742 | 9.9 |
| 7.0 | 6,137 | 6,559 | 6.9 |
| 10.0** | 663 | 366 | -44.8 |
| 13.0 | 530 | 667 | 25.8 |
| 17.0 | 555 | 744 | 34.1 |
| 20.0 | 531 | 547 | 3.0 |
| 22.0 | 556 | 692 | 24.5 |
| 24.0** | 2,847 | 2,711 | -4.8 |
| 26.0 | 243 | 414 | 70.4 |
| 28.0 | 530 | 538 | 1.5 |
| 30.0** | 196 | 395 | 101.5 |
| 35A** | 4,761 | 5,257 | 10.4 |
| 36A** | 304 | 342 | 12.5 |
| 37.0** | 626 | 507 | -19.0 |
| 38.0** | 2,720 | 2,722 | 0.1 |
| 40.0 | 963 | 956 | -0.7 |

* Larger number indicates higher erosion potential.

** USGS active erosion sites

Table D.4. Potential Bank Erosion Computed Using 1965
Flows, Pre- and Postproject Conditions.

| Cross Section | Shear Stress Duration Index* | | Percent Difference |
|------------------|------------------------------|-------------|-----------------------|
| | Preproject | Postproject | |
| 0.9 | 10 | 3 | -70.0 |
| 2.0 | 420 | 472 | 12.4 |
| 4.0 | 296 | 330 | 11.5 |
| 7.0 | 2,833 | 3,000 | 5.9 |
| 10.0** | 290 | 258 | -11.0 |
| 13.0 | 187 | 238 | 27.3 |
| 17.0 | 215 | 279 | 29.8 |
| 20.0 | 237 | 262 | 10.5 |
| 22.0 | 185 | 239 | 29.2 |
| 24.0** | 1,398 | 1,337 | -4.4 |
| 26.0 | 106 | 143 | 34.9 |
| 28.0 | 209 | 218 | 4.3 |
| 30.0** | 82 | 121 | 47.6 |
| 35A** | 2,136 | 2,444 | 14.4 |
| 36A** | 108 | 121 | 12.0 |
| 37.0** | 265 | 254 | -4.2 |
| 38.0** | 1,303 | 1,323 | 1.5 |
| 40.0 | 425 | 468 | 10.1 |

* Larger number indicates higher erosion potential.

** USGS active erosion sites

Table D.5. Potential Bank Erosion Computed Using 1970
Flows, Pre- and Postproject Conditions.

| Cross Section | Shear Stress Duration Index* | | Percent Difference |
|------------------|------------------------------|-------------|-----------------------|
| | Preproject | Postproject | |
| 0.9 | 0 | 0 | 0.0 |
| 2.0 | 542 | 636 | 17.3 |
| 4.0 | 390 | 452 | 15.9 |
| 7.0 | 3,567 | 3,970 | 11.3 |
| 10.0** | 372 | 231 | -37.9 |
| 13.0 | 294 | 374 | 27.2 |
| 17.0 | 321 | 436 | 35.8 |
| 20.0 | 323 | 345 | 6.8 |
| 22.0 | 309 | 383 | 23.9 |
| 24.0** | 1,690 | 1,660 | -1.8 |
| 26.0 | 161 | 247 | 53.4 |
| 28.0 | 308 | 320 | 3.9 |
| 30.0** | 149 | 229 | 53.7 |
| 35A** | 2,997 | 3,397 | 13.3 |
| 36A** | 175 | 199 | 13.7 |
| 37.0** | 358 | 317 | -11.5 |
| 38.0** | 1,677 | 1,732 | 3.3 |
| 40.0 | 600 | 612 | 2.0 |

* Larger number indicates higher erosion potential.

** USGS active erosion sites

Table D.6. Potential Bank Erosion Computed Using 1974
Flows, Pre- and Postproject Conditions.

| Cross Section | Shear Stress Duration Index* | | Percent Difference |
|------------------|------------------------------|-------------|-----------------------|
| | Preproject | Postproject | |
| 0.9 | 0 | 0 | 0.0 |
| 2.0 | 458 | 524 | 14.4 |
| 4.0 | 302 | 335 | 10.9 |
| 7.0 | 3,807 | 4,009 | 5.3 |
| 10.0** | 493 | 310 | -37.1 |
| 13.0 | 155 | 258 | 66.5 |
| 17.0 | 162 | 285 | 75.9 |
| 20.0 | 308 | 322 | 4.5 |
| 22.0 | 164 | 270 | 64.6 |
| 24.0** | 2,550 | 2,452 | -3.8 |
| 26.0 | 77 | 162 | 110.4 |
| 28.0 | 228 | 232 | 1.8 |
| 30.0** | 55 | 139 | 152.7 |
| 35A** | 2,559 | 2,823 | 10.3 |
| 36A** | 87 | 135 | 55.2 |
| 37.0** | 372 | 276 | -25.8 |
| 38.0** | 2,114 | 2,110 | -0.2 |
| 40.0 | 603 | 609 | 1.0 |

* Larger number indicates higher erosion potential.

** USGS active erosion sites

Table D.7. Potential Bank Erosion Computed Using 1978
Flows, Pre- and Postproject Conditions.

| Cross Section | Shear Stress Duration Index* | | Percent Difference |
|------------------|------------------------------|-------------|-----------------------|
| | Preproject | Postproject | |
| 0.9 | 0 | 0 | 0.0 |
| 2.0 | 164 | 166 | 1.2 |
| 4.0 | 114 | 101 | -11.4 |
| 7.0 | 1,437 | 1,560 | 8.6 |
| 10.0** | 276 | 196 | -29.0 |
| 13.0 | 11 | 45 | 309.1 |
| 17.0 | 10 | 42 | 320.0 |
| 20.0 | 118 | 125 | 5.9 |
| 22.0 | 9 | 51 | 466.7 |
| 24.0** | 991 | 1,172 | 18.3 |
| 26.0 | 0 | 10 | -- |
| 28.0 | 80 | 80 | 0.0 |
| 30.0** | 0 | 0 | 0.0 |
| 35A** | 924 | 995 | 7.7 |
| 36A** | 10 | 38 | 280.0 |
| 37.0** | 173 | 138 | -20.2 |
| 38.0** | 789 | 937 | 18.8 |
| 40.0 | 235 | 254 | 8.1 |

* Larger number indicates higher erosion potential.

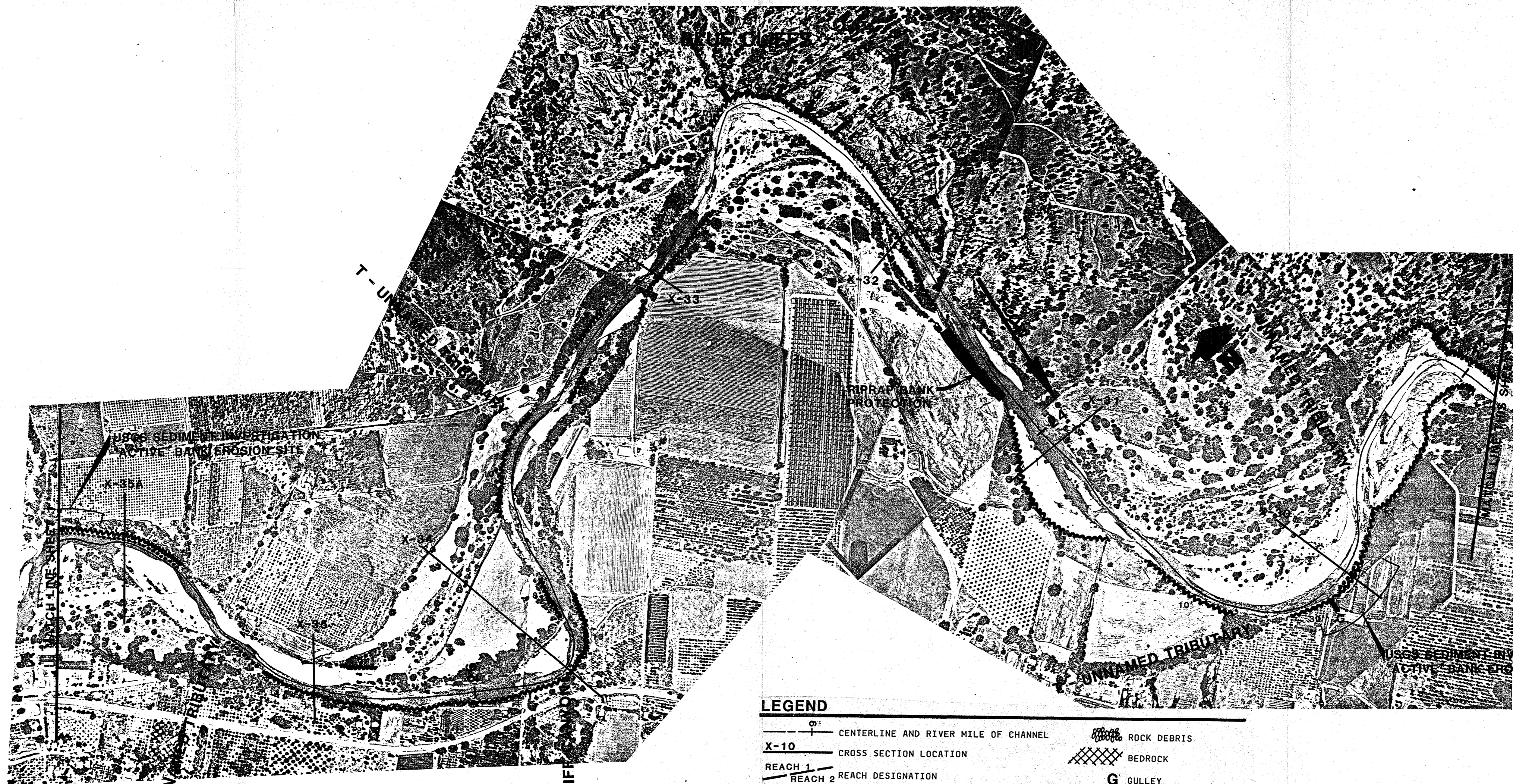
** USGS active erosion sites

Table D.8. Potential Bank Erosion Computed Using 1983
Flows, Pre- and Postproject Conditions.

| Cross Section | Shear Stress Duration Index* | | Percent Difference |
|------------------|------------------------------|-------------|-----------------------|
| | Preproject | Postproject | |
| 0.9 | 17 | 17 | 0.0 |
| 2.0 | 1,509 | 1,660 | 10.0 |
| 4.0 | 1,075 | 1,165 | 8.4 |
| 7.0 | 9,424 | 10,211 | 8.4 |
| 10.0** | 771 | 549 | -28.8 |
| 13.0 | 923 | 999 | 8.2 |
| 17.0 | 1,002 | 1,134 | 13.2 |
| 20.0 | 809 | 852 | 5.3 |
| 22.0 | 973 | 1,021 | 4.9 |
| 24.0** | 3,986 | 4,154 | 4.2 |
| 26.0 | 521 | 586 | 12.5 |
| 28.0 | 803 | 845 | 5.2 |
| 30.0** | 440 | 540 | 22.7 |
| 35A** | 7,500 | 8,169 | 8.9 |
| 36A** | 496 | 538 | 8.5 |
| 37.0** | 867 | 785 | -9.5 |
| 38.0** | 3,996 | 4,242 | 6.2 |
| 40.0 | 1,427 | 1,461 | 2.4 |

* Larger number indicates higher erosion potential.

** USGS active erosion sites

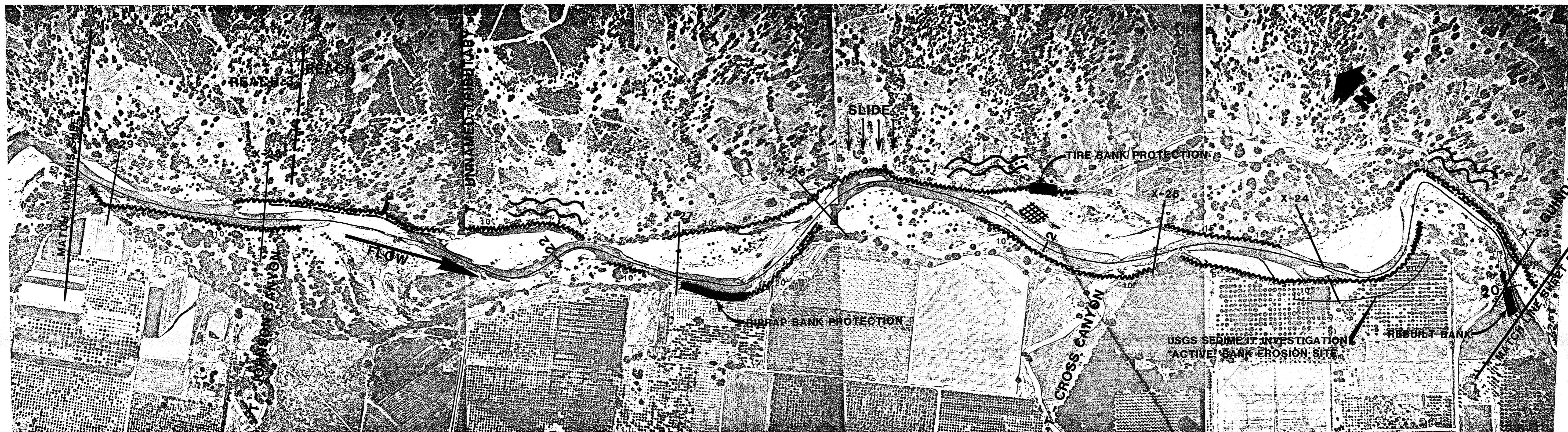


LEGEND

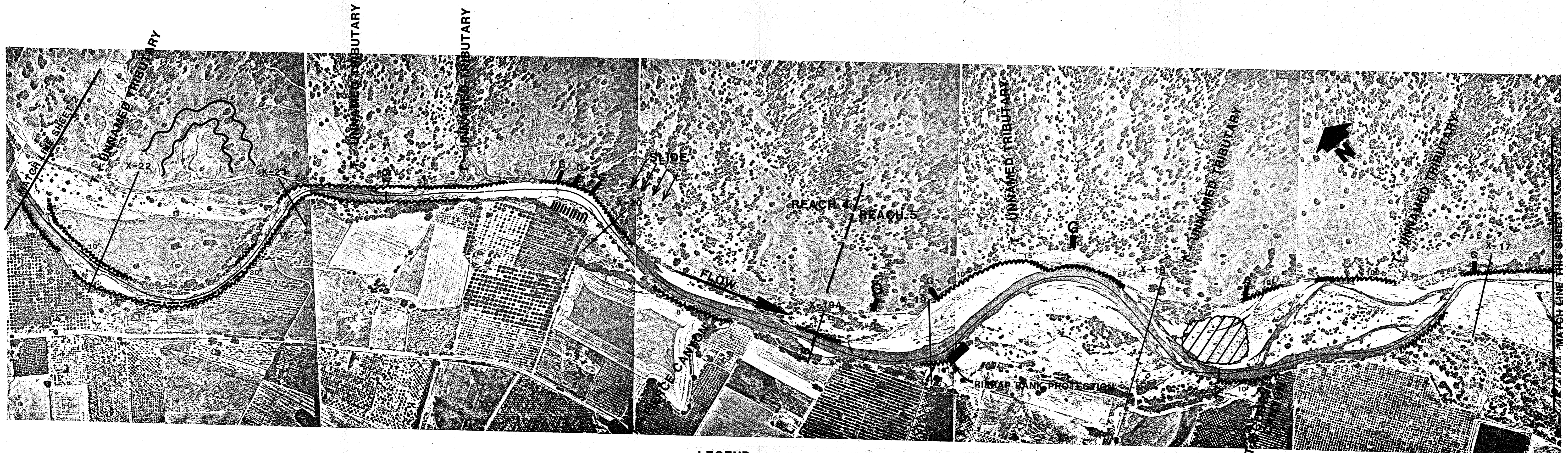
- CENTERLINE AND RIVER MILE OF CHANNEL
- X-10 CROSS SECTION LOCATION
- REACH 1 REACH DESIGNATION
- REACH 2 REACH DESIGNATION
- EROSION AND HEIGHT OF BANK
- MAN-MADE BANK
- CLAY OUTCROP
- SANDSTONE OUTCROP
- TERRACED EROSION
- ROCK DEBRIS
- BEDROCK
- G GULLEY
- T TRIBUTARY
- SLIDE
- UNSTABLE HILLSIDE
- CHANNEL EXCAVATION

AERIAL PHOTOGRAPHY TAKEN 6/24/86

SCALE: 1" = 500'



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| GEOMORPHIC ANALYSIS OF CACHE CREEK | | Proj. No.: Date: Design: Drawn: Checked: Revisions: |
| SIMONS, LI & ASSOCIATES, INC. Colorado Springs, CO • DENVER, CO • FORT COLLINS, CO NEWPORT BEACH, CA • PHOENIX, AZ • TUCSON, AZ | | 2 |



LEGEND

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|---------|--------------------------------------|---|--------------------|
| | CENTERLINE AND RIVER MILE OF CHANNEL | | ROCK DEBRIS |
| X-10 | CROSS SECTION LOCATION | | BEDROCK |
| REACH 1 | REACH DESIGNATION | G | GULLY |
| REACH 2 | REACH DESIGNATION | T | TRIBUTARY |
| | EROSION AND HEIGHT OF BANK | | SLIDE |
| 10' | EROSION AND HEIGHT OF BANK | | UNSTABLE HILLSIDE |
| | MAN-MADE BANK | | CHANNEL EXCAVATION |
| | CLAY OUTCROP | | |
| | SANDSTONE OUTCROP | | |
| | TERRACED EROSION | | |

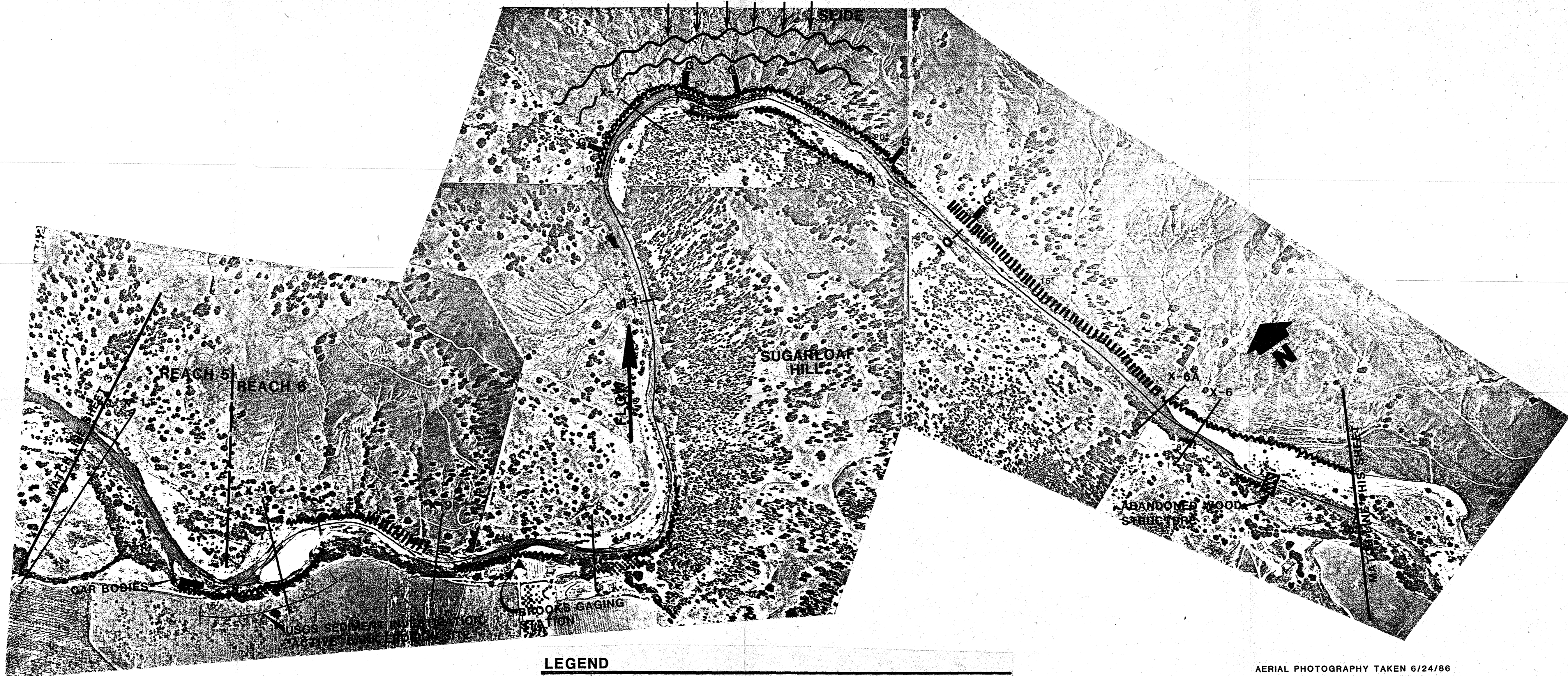
AERIAL PHOTOGRAPHY TAKEN 6/24/86
SCALE: 1" = 500'



**GEOMORPHIC ANALYSIS
OF
CACHE CREEK**

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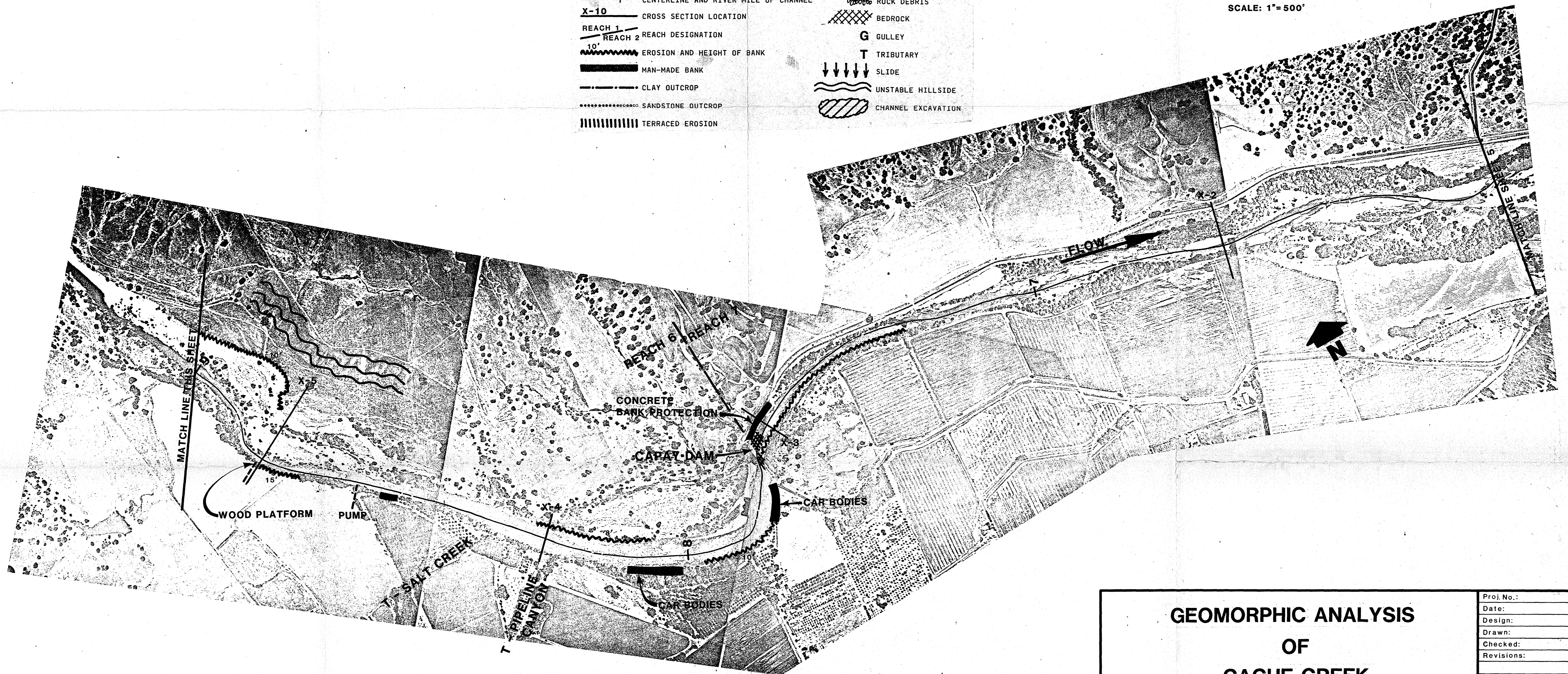


AERIAL PHOTOGRAPHY TAKEN 6/24/86

SCALE: 1" = 500'

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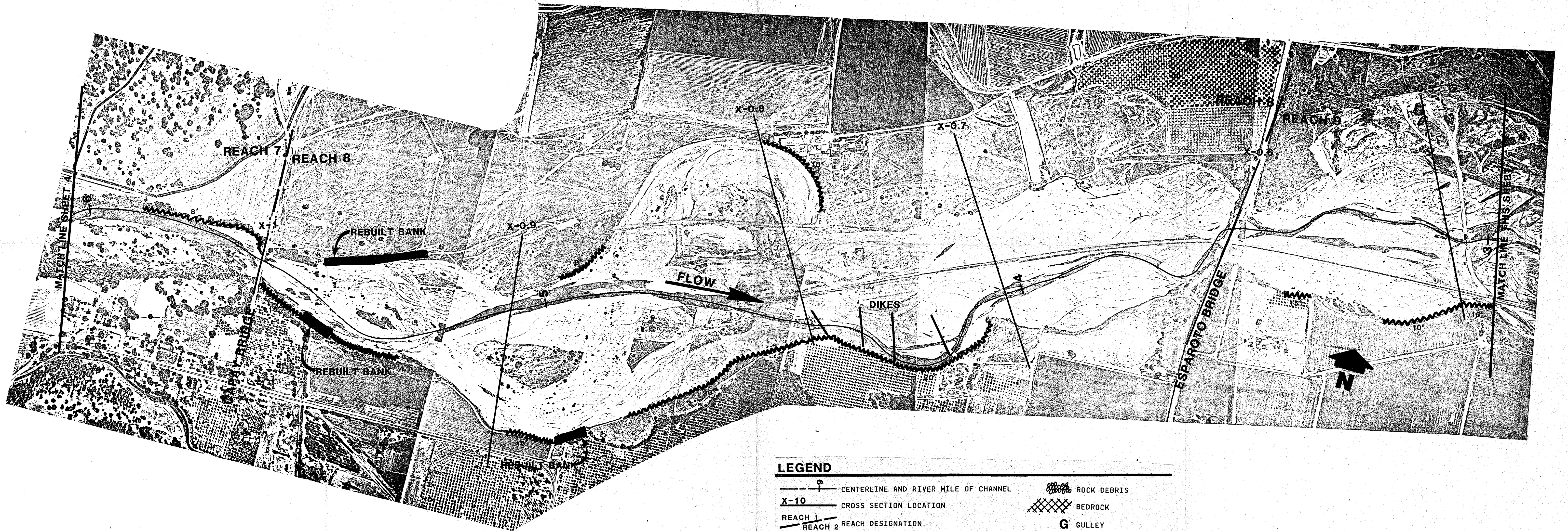
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|--------------------------------------|--------------------|
| CENTERLINE AND RIVER MILE OF CHANNEL | ROCK DEBRIS |
| X-10 CROSS SECTION LOCATION | BEDROCK |
| REACH 1 REACH 2 REACH DESIGNATION | G GULLEY |
| EROSION AND HEIGHT OF BANK | T TRIBUTARY |
| MAN-MADE BANK | SLIDE |
| CLAY OUTCROP | UNSTABLE HILLSIDE |
| SANDSTONE OUTCROP | CHANNEL EXCAVATION |
| TERRACED EROSION | |



**GEOMORPHIC ANALYSIS
OF
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| Revisions: |

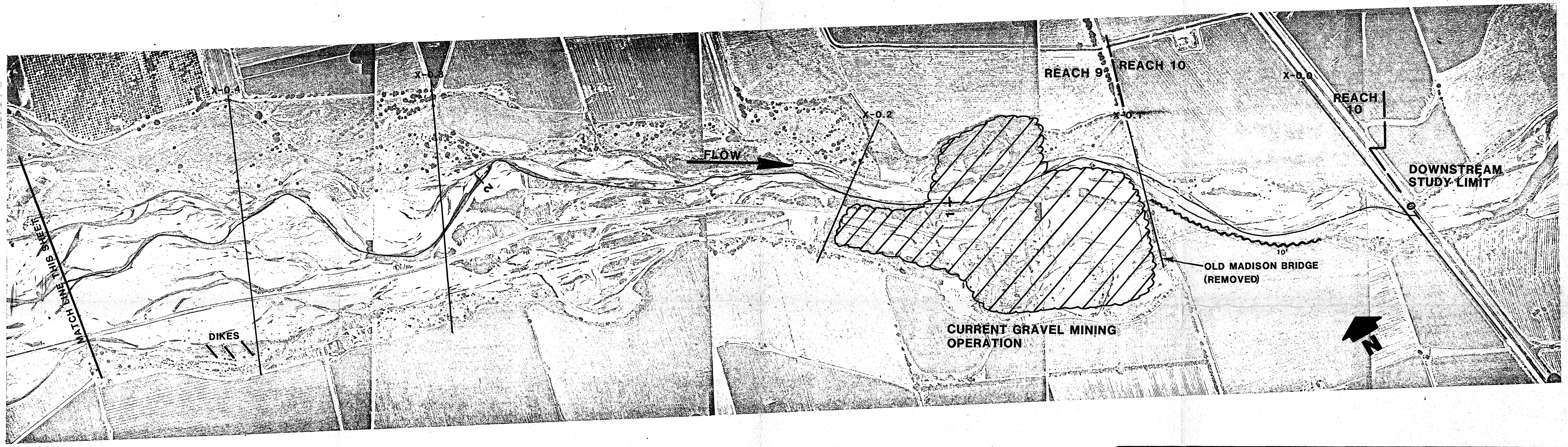
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 Newport Beach, CA • Phoenix, AZ • Tucson, AZ



LEGEND

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|---------|--------------------------------------|--|--------------------|
| —○— | CENTERLINE AND RIVER MILE OF CHANNEL | | ROCK DEBRIS |
| X-10 | CROSS SECTION LOCATION | | BEDROCK |
| REACH 1 | REACH DESIGNATION | | GULLY |
| REACH 2 | REACH DESIGNATION | | TRIBUTARY |
| | EROSION AND HEIGHT OF BANK | | SLIDE |
| | MAN-MADE BANK | | UNSTABLE HILLSIDE |
| | CLAY OUTCROP | | CHANNEL EXCAVATION |
| | SANDSTONE OUTCROP | | |
| | TERRACED EROSION | | |

AERIAL PHOTOGRAPHY TAKEN 6/24/88
SCALE: 1"=500'



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| | | Drawn: |
| 5 | | Checked: |
| | | Revisions: |