US Army Corps of Engineers

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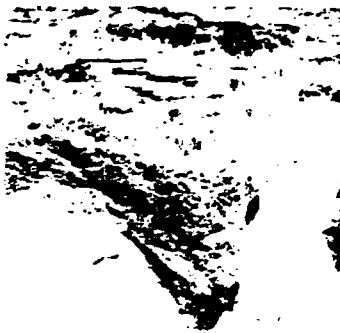# EVALUATION OF BANK EROSION POTENTIAL CLIFTON, ARIZONA

## Numerical Model Investigation

by

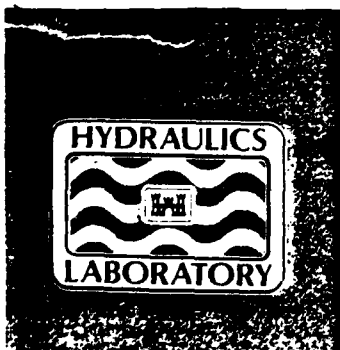Ronald R. Copeland, William A. Thomas

Hydraulics Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39181-0631

July 1989
Final Report

Approved For Public Release; Distribution Unlimited

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| REPORT DOCUMENTATION PAGE | | | | *Form Approved* *OMB No. 0704-0188* |
|---|---|---|---|---|

| 1a. REPORT SECURITY CLASSIFICATION Unclassified | | | 1b. RESTRICTIVE MARKINGS | |
|---|---|---|---|---|
| 2a. SECURITY CLASSIFICATION AUTHORITY | | | 3. DISTRIBUTION/AVAILABILITY OF REPORT Approved for public release; distribution unlimited. | |
| 2b. DECLASSIFICATION/DOWNGRADING SCHEDULE | | | | |

| 4. PERFORMING ORGANIZATION REPORT NUMBER(S) Miscellaneous Paper HL-89-3 | | 5. MONITORING ORGANIZATION REPORT NUMBER(S) | |
|---|---|---|---|

| 6a. NAME OF PERFORMING ORGANIZATION USAEWES Hydraulics Laboratory | 6b. OFFICE SYMBOL *(If applicable)* CEWES-HR-N | 7a. NAME OF MONITORING ORGANIZATION |
|---|---|---|

| 6c. ADDRESS *(City, State, and ZIP Code)* PO Box 631 Vicksburg, MS  39181-0631 | 7b. ADDRESS *(City, State, and ZIP Code)* |
|---|---|

| 8a. NAME OF FUNDING/SPONSORING ORGANIZATION USAED, Los Angeles | 8b. OFFICE SYMBOL *(If applicable)* | 9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER | | |
|---|---|---|---|---|

| 8c. ADDRESS *(City, State, and ZIP Code)* PO Box 2711 Los Angeles, CA  90053-2325 | 10. SOURCE OF FUNDING NUMBERS | | | |
|---|---|---|---|---|
| | PROGRAM ELEMENT NO. | PROJECT NO. | TASK NO. | WORK UNIT ACCESSION NO. |

**11. TITLE** *(Include Security Classification)*

Evaluation of Bank Erosion Potential, Clifton, Arizona; Numerical Model Investigation

**12. PERSONAL AUTHOR(S)**
Copeland, Ronald R., and Thomas, William A.

| 13a. TYPE OF REPORT Final report | 13b. TIME COVERED FROM _____ TO _____ | 14. DATE OF REPORT *(Year, Month, Day)* July 1989 | 15. PAGE COUNT 54 |
|---|---|---|---|

**16. SUPPLEMENTARY NOTATION**
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA  22161.

| 17. | COSATI CODES | | 18. SUBJECT TERMS *(Continue on reverse if necessary and identify by block number)* | |
|---|---|---|---|---|
| FIELD | GROUP | SUB-GROUP | Bank erosion | Numerical model |
| | | | Gravel-bed rivers | San Francisco River |
| | | | HEC-6 computer program | Sedimentation |

**19. ABSTRACT** *(Continue on reverse if necessary and identify by block number)*

A sediment impact assessment study was conducted to evaluate the potential for bank erosion adjacent to a proposed levee along the San Francisco River in Clifton, AZ. Degradation potential was evaluated using a one-dimensional numerical sediment transport model. Historical data and a field reconnaissance were used to evaluate historical stability of the river. Generalized empirical regression equations were employed to evaluate potential for channel widening. The study identified zones along the levee where erosion potential was high.

| 20. DISTRIBUTION/AVAILABILITY OF ABSTRACT ☒ UNCLASSIFIED/UNLIMITED  ☐ SAME AS RPT.  ☐ DTIC USERS | 21. ABSTRACT SECURITY CLASSIFICATION Unclassified | |
|---|---|---|
| 22a. NAME OF RESPONSIBLE INDIVIDUAL | 22b. TELEPHONE *(Include Area Code)* | 22c. OFFICE SYMBOL |

**DD Form 1473, JUN 86**          *Previous editions are obsolete.*          SECURITY CLASSIFICATION OF THIS PAGE

# PREFACE

The numerical model investigation of the San Francisco River through Clifton, AZ, reported herein, was conducted at the US Army Engineer Waterways Experiment Station (WES), at the request of the US Army Engineer District, Los Angeles (SPL).

This investigation was conducted during the period May to December 1988 in the Hydraulics Laboratory of WES under the direction of Mr. Frank A. Herrmann, Jr., Chief of the Hydraulics Laboratory, Mr. Marden B. Boyd, Chief of the Waterways Division (WD), and Mr. Michael J. Trawle, Leader of the Math Modeling Group (MMG). Mr. William A. Thomas, WD, provided general guidance and review. The project engineer was Mr. Ronald R. Copeland, MMG. Authors of this report were Mr. Copeland and Mr. Thomas. Technical assistance was provided by Ms. Brenda L. Martin, MMG.

During the course of this study, close working contact was maintained with Mr. Rene Vermeeren and Ms. Rachel Korkos of the Hydraulics Section, SPL, who served as coordinating engineers, providing required data and technical assistance.

Acting Commander and Director of WES during preparation of this report was LTC Jack R. Stephens, EN. Technical Director was Dr. Robert W. Whalin.

1

## CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI
(metric) units as follows:

| Multiply | By | To Obtain |
| --- | --- | --- |
| cubic feet | 0.02831685 | cubic metres |
| cubic yards | 0.7645549 | cubic metres |
| feet | 0.3048 | metres |
| inches | 2.54 | centimetres |
| miles (US statute) | 1.609347 | kilometres |
| square miles (US statute) | 2.589998 | square kilometres |
| tons (2,000 pounds, mass) | 907.1847 | kilograms |

# EVALUATION OF BANK EROSION POTENTIAL, CLIFTON, ARIZONA

## Numerical Model Investigation

## PART I: INTRODUCTION

### Description of Study Area

1. The town of Clifton is located in mountainous terrain in southeastern Arizona (Figure 1). The San Francisco River flows through the town toward its confluence with the Gila River. The river's drainage area above Clifton is approximately 2,800 square miles.* Elevations in the basin range from 9,200 ft** near the headwaters to 3,500 ft near Clifton. A single levee is proposed to protect south Clifton for a one-half-mile reach adjacent to the San Francisco River beginning immediately below the Southern Pacific Railroad and Highway 666 Bridges (sta 568+00) and extending downstream to Ward Canyon at sta 534+00 (Figure 2). The project reach extends upstream through north Clifton for a total project length of about 2.3 miles. The project upstream of the railroad bridge would consist of housing relocation and floodproofing but no improvements to the existing channel. The project is designed for the 125-year-frequency flood of 145,000 cfs. The Standard Project Flood (SPF) is 167,000 cfs.

2. Existing channel improvements through Clifton include masonry floodwalls and dumped molten slag deposits from local copper smelters (Figure 2). These bank protection measures were constructed between 1906 and 1920. The right descending bank downstream from the railroad and highway bridges has slag deposit bank protection for 1,200 ft and then a nearly vertical bluff wall composed of rock or slag through the rest of the study reach. Downstream from the bridges, on the left descending bank, a masonry wall extends for about 1,100 ft until it intersects a slag deposit of uncertain continuity. An investigative study (US Army Engineer District (USAED), Los Angeles, 1988) concluded that this slag deposit was probably continuous between river

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\* A table of factors for converting non-SI units of measurement to SI (metric) units is found on page 3.
\*\* All elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).
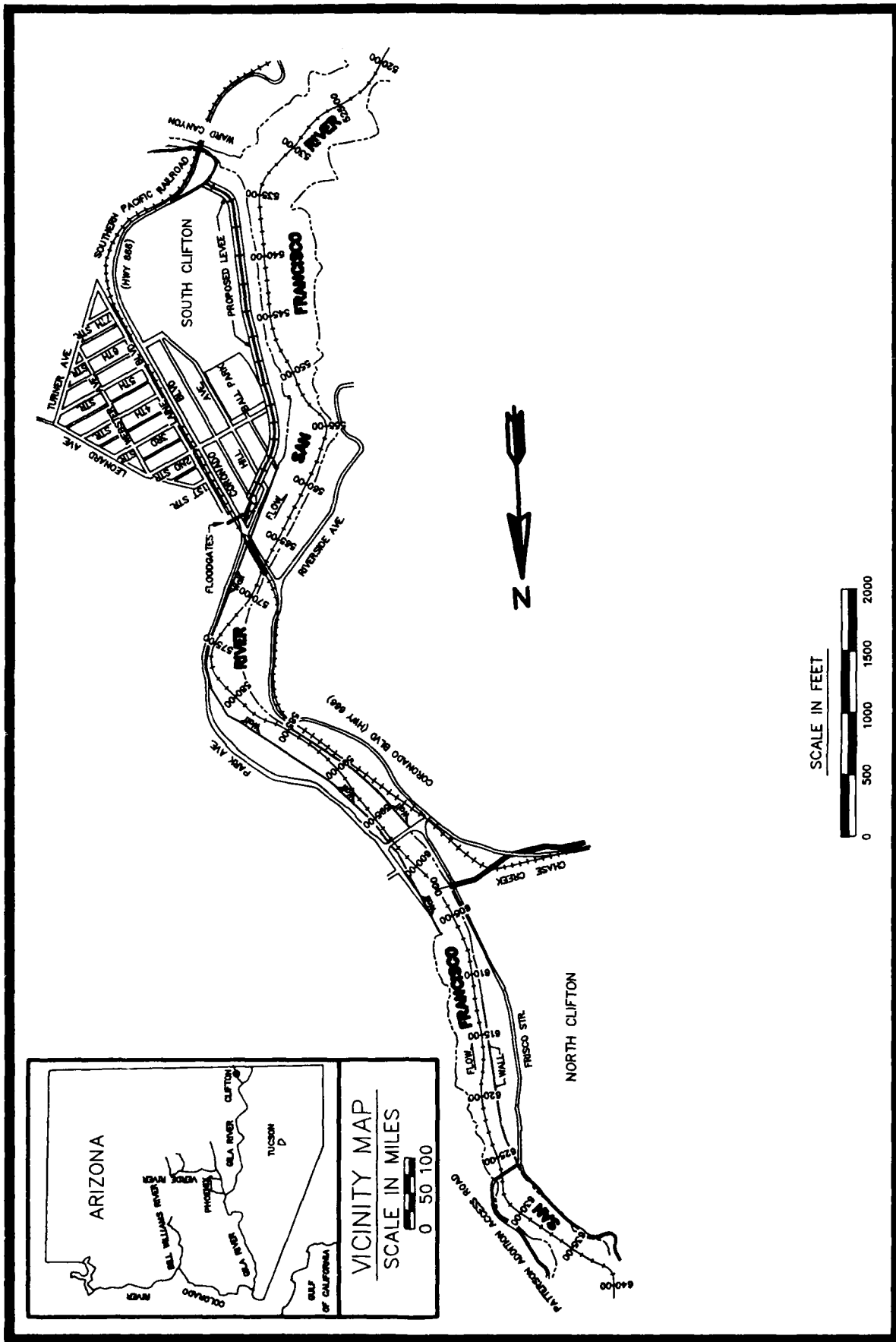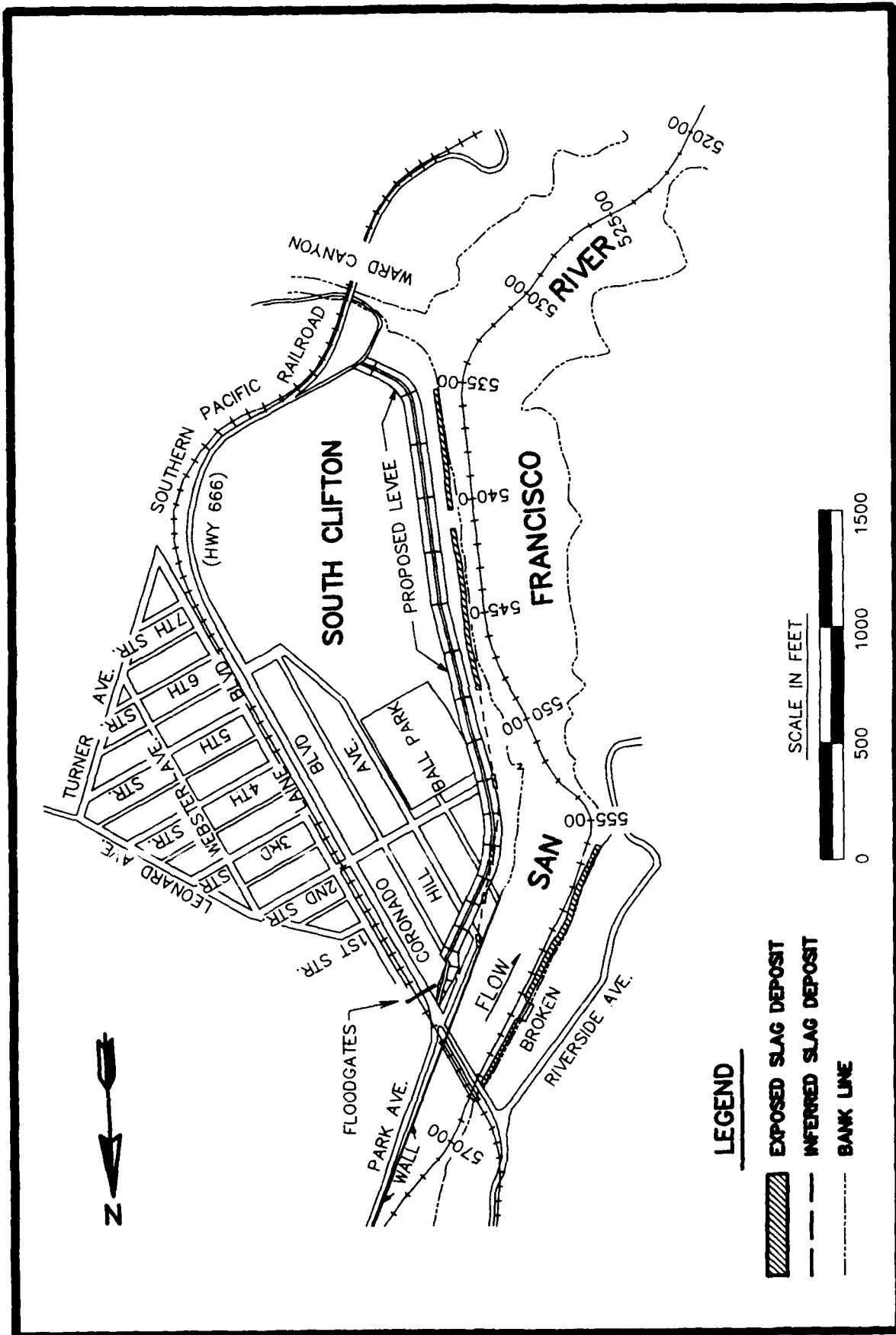
Figure 1. Clifton, AZ, location map

5

Figure 2. Clifton, AZ, project features

sta 535+00 and sta 562+00 except for a 150-ft-long section near sta 541+00. However, the existence of the slag deposit between sta 548+00 and sta 562+00 is uncertain because the area is covered by fill material. The existence of the slag deposit was inferred based on conversations with local residents and dark lineaments on aerial photos. The deposit was reportedly dumped from narrow-gaged tracks laid along the riverside edge of mine tailing piles. The condition and configuration of the deposit varies. Thickness varies between 3 and 25 ft; height between 5 and 18 ft; toe elevations from below the waterline to 5 ft above the waterline; slopes from near-vertical to 1V on 4H; and foundation material from loose silt to hard conglomerate. The slag was found to be generally competent, with zones of deterioration and slumping.

3. The alignment of the proposed levee is set back about 100 ft from the existing main river channel. The 100-ft setback was established based on hydraulic calculations that predicted low velocities adjacent to the levee toe, on historical flood indications that the riverbank is stable, and on the geotechnical analysis of the bank soils. The levee alignment was also coordinated with local interests. The proposed earth levee includes grouted stone side-slope protection extending 5 ft below existing ground elevation. The levee is 2,500 ft long with 1V on 2.5H side slopes. The levee ranges in height from 7 to 17 ft, with a minimum 3 ft of freeboard. Two manually operated floodgates are proposed at the intersection of the levee alignment and the railroad and highway. These structures include swing and roller steel floodgates 15 ft high, and concrete retaining walls having a maximum height of 18 ft.

## Purpose and Approach

4. The purpose of this study was to evaluate the potential for bank erosion adjacent to the proposed levee. This was accomplished indirectly by identifying potential for degradation and meandering in the main river channel. Degradation potential was evaluated using the TABS-1 one-dimensional numerical model. Meandering potential was evaluated by reviewing historical river data and by application of generalized empirical regression equations for channel width.

5. The numerical model study evaluated the potential magnitude of scour and deposition in the study reach for the SPF, and for a flood with the same

7

peak discharge as the flood of record. Model adjustment was based on the assumption that the existing channel is essentially stable. The effect of backwater upstream from the railroad and highway bridges on sediment transport and bed change was determined.

6. Historical river data, including river profiles, aerial photos, and topographic maps, were reviewed. The historical data were supplemented by a field reconnaissance to observe existing channel conditions. These data were used to evaluate the historical stability of the river.

7. Generalized empirical regression equations that predict stable channel width were used to evaluate the potential for bank erosion. Due to the generalized nature of these equations, their regional applicability, and the uncertainty related to the influence of bank stability, results from this type of analysis are valuable only in the sense that they provide knowledge of probable trends in channel geometry change.

PART II: INPUT DATA DESCRIPTION

## Description of Numerical Model

8. The TABS-1 one-dimensional sedimentation program was used to develop the numerical model for this study. Development of this computer program was initiated by Mr. William Thomas at the US Army Engineer District, Little Rock, in 1967. Further development at the Hydrologic Engineering Center (HEC) by Mr. Thomas produced the widely used HEC-6 generalized computer program for calculating scour and deposition in rivers and reservoirs (HEC 1977). Additional modification and enhancement to the basic program by Mr. Thomas at the US Army Engineer Waterways Experiment Station (WES) led to the TABS-1 program currently in use. TABS-1 is considered experimental in that it is not documented to the point that it can be made available for general use, but can be made available by special request. The program produces a one-dimensional model that simulates a series of steady-state discharge events and their effect on the sediment transport capacity at cross sections and the resulting degradation or aggradation.

## Channel Geometry

9. Channel geometry for the numerical model was developed from a HEC-2 backwater model prepared by the Los Angeles District in December 1985, and adjusted in February 1986. This model included the proposed levee and other design features and extended from sta 514+42 upstream to sta 653+20. Cross-section elevations in the channel were taken from a 1977 survey of the channel, and overbank elevations from a 1939 topographic map. The TABS-1 sediment model does not have a bridge analysis capability. Therefore, backwater effects from the highway and railroad bridges were accounted for by setting the water-surface elevation upstream from the bridges based on a rating curve developed from the HEC-2 backwater calculations. The upstream boundary of the numerical model at sta 653+20 was artificially extended 5,000 ft to remove the boundary from backwater effects caused by the constricted channel sections in the vicinity of the Patterson Addition Access Bridge near sta 626+50 (Figure 1). This was accomplished in the model by repeating the upstream cross

section five times at intervals of 1,000 ft and increasing bed elevations based on a slope of 0.0033.

## Hydrographs

10. The discharge hydrographs were simulated in the numerical model by a series of steady-state events. The duration of each event was chosen such that changes in bed elevation, due to deposition or scour, did not significantly change the hydraulic parameters during that event. At relatively high discharges, durations need to be short. At low discharges, the time interval may be extended. Time intervals between 15 minutes and 2 hours were used in this study.

11. A hydrograph simulated by a series of steady-state events of varying durations is called a histograph. The SPF histograph used in the numerical model investigation was based on a hydrograph developed by the Los Angeles District (USAED, Los Angeles, 1986). This flood had a peak discharge of 167,000 cfs. The histograph is shown in Plate 1.

12. A flood with a peak discharge equal to that of the flood of record (90,900 cfs in 1983) was also tested. This histograph was assumed to have the same duration as the SPF, and each discharge event was reduced by the ratio of the flood peaks (0.544). This histograph is shown in Plate 2.

## Downstream Water-surface Elevation

13. The water-surface elevation rating curve at the downstream boundary (sta 514+42) was based on the HEC-2 calculations by the Los Angeles District (Plate 3). The rating curve at the downstream boundary was based on the slope-area option in the HEC-2 code. A rating curve upstream from the railroad and highway bridges (sta 566+80) was based on District desk-top calculations using orifice and weir equations at high flow and the HEC-2 special bridge routine at low flows (Plate 4).

## Bed Material and Overbank Gradations

14. Size-class characteristics of the bed material and the overbank were developed from 1985 test trench data (USAED, Los Angeles, 1986). These

10

data were collected as part of a geologic and soils investigation to determine suitable borrow sites, foundation conditions, and soil design values for the levee project. Locations of the test trenches are shown in Plate 5.

15. Ten test trenches were dug in the riverbed to depths of 12 ft. Gradations of samples from these test trenches were determined by sieve analyses. Although boulders with diameters as large as 2 ft were observed in the trenches, all material larger than 3 in. was excluded from the samples. No field estimate of the percentage of this coarse material in the bed was made. In the numerical model, 8 percent of the bed material was arbitrarily assumed to be greater than 3 in. An analysis of the test trench data revealed that all but two of the 17 sample gradations could be identified as either sandy silt or gravelly sand. Interestingly, samples from a given site, but at different depths, could fall into both groups. This vertical variation in bed material indicates active bed movement either due to bedforms or shifting gravel bars and sinuosity patterns. Average gradation curves for the two types of samples and their ranges are shown in Plate 6. Also shown are the two intermediate samples that did not fall into either category. The average gradation for the gravelly sand material is considered most representative of the active layer of bed material during major flow events and was used in the numerical model. A sensitivity study was conducted to determine the effect of using the coarser and finer limits of the gravelly-sand gradations.

16. Lateral and longitudinal variations in the bed material gradation can significantly affect scour and deposition in gravel-bed rivers. These variations can be especially important at channel constrictions, expansions, and bends. However, the San Francisco River data are insufficient to define either the lateral or longitudinal variation in bed material gradation.

17. On the overbank, five trenches were dug along the proposed levee alignment to depths of 12 ft. These data indicate that the overbank is composed primarily of mine tailings consisting of stratified layers of clay, clayey silt, silty clay, silt, silty sand, sandy gravel, and gravelly sand. Detailed size-class analysis was not conducted, but general size descriptions show that the silt-clay percentage in the overbank varies between 18 and 100 percent. The sand and gravel layers give the overbank a high potential for undermining and bank failure. The upstream 500 ft of the proposed levee would rest on a fairly erosion-resistant conglomerate formation.

11

## Roughness

18. Channel roughness in the TABS-1 numerical model was based on results from the HEC-2 backwater study. Manning's roughness coefficients in the backwater model had been adjusted to reproduce high-water stages from historical flood events. Channel roughness coefficients varied between 0.025 and 0.030. The channel roughness coefficient at the upstream cross-section (653+20) in the HEC-2 model was changed from 0.050 to 0.025 in the TABS-1 model to be consistent with downstream values. Overbank roughness coefficients varied between 0.045 and 0.060. Major obstructions in the overbank were accounted for by inclusion in the cross-section geometry.

## Sediment Inflow

19. Between 1982 and 1984, suspended sediment samples were collected at the Clifton gage, which is located about one mile downstream from the town at the sewage treatment plant (sta 515+00). The maximum discharge at which samples were obtained was 2,000 cfs. Particle size analyses were not performed on these samples so silt and clay percentages are unknown. The data are useful only for general comparisons with calculated bed-material loads.

20. Sediment inflow for the TABS-1 numerical model was estimated by assuming equilibrium transport potential at the upstream boundary. It was necessary to extend the model's upstream boundary 5,000 ft due to backwater created by the constriction near the Patterson Addition Access Road Bridge (sta 626+50). Due to the uncertainty related to the sediment inflow, sensitivity calculations were made with sediment inflows greater and less than estimated average values.

## Transport Function

21. Four transport functions were evaluated for use in the numerical model. These were: Meyer-Peter and Muller (1948) bedload formula, Laursen-Madden (HEC 1977) bed material load formula, a combination of the Schoklitsch (Schulits 1935) bedload formula and the Toffaleti (1966) bed-material load formula, and a combination of the Meyer-Peter and Muller and Toffaleti formulas. Hydraulic parameters for the evaluation were taken from fixed-bed

backwater runs at the upstream cross section. Results of calculations using these functions are compared to measured suspended data in Plate 7. These data are comparable only in a general sense because the measured data include silts and clays and excludes bedload, while the calculated data include total sand and gravel transport but exclude silts and clays. The combination of Toffaleti and Meyer-Peter and Muller formulas was selected because it appeared to give reasonable results for sand transport and was capable of transporting the larger gravel sizes. Sediment inflow rating curves for each size class used in the numerical study are shown in Plates 8 and 9.

PART III:  STUDY RESULTS

## Average-Annual Flood

22.  To get a general idea of aggradation and degradation potential in
the study reach, the numerical model was used to simulate a steady-state dis-
charge of 7,500 cfs (2.33-year frequency) for a duration of five days.  This
simulation indicated a degradation trend downstream from the railroad and
highway bridges.  The model also indicated that the riverbed through Clifton
is highly dynamic, with sediment transport potential varying significantly
with distance along the channel.  Due to the dynamic characteristics of the
sediment movement, it was decided that the SPF hydrograph should be simulated
to determine the effect of duration on scour and deposition in the study
reach.

## Standard Project Flood

23.  The SPF histograph was run with 1977 channel geometry and equilib-
rium sediment inflow.  Insufficient data were available to adjust or circum-
stantiate the numerical model, therefore results can be used only in a quali-
tative sense.  This qualitative evaluation is valuable in determining the
potential for bank erosion adjacent to the proposed levee alignment.  Sections
that show significant scour or aggradation are indications of a section with
high erosion potential.

24.  The model showed that the riverbed through Clifton is highly dy-
namic during the passage of the SPF.  Thalweg profiles at the peak and at the
end of the simulation are compared to the initial thalweg in Figure 3.  This
figure shows scour on the order of 5 ft downstream from the railroad and high-
way bridges at the peak of the flood.  This scour trend does not continue
throughout the leveed reach because of the constriction at sta 521+80.  This
constriction creates a backwater that prevents degradation at discharges
greater than 40,000 cfs.  The backwater effect disappears at lower discharges
which helps explain the incised channel observed in the field.  The constric-
tion itself is subject to significant degradation which results in deposition
downstream.  This condition does not make for a favorable downstream boundary
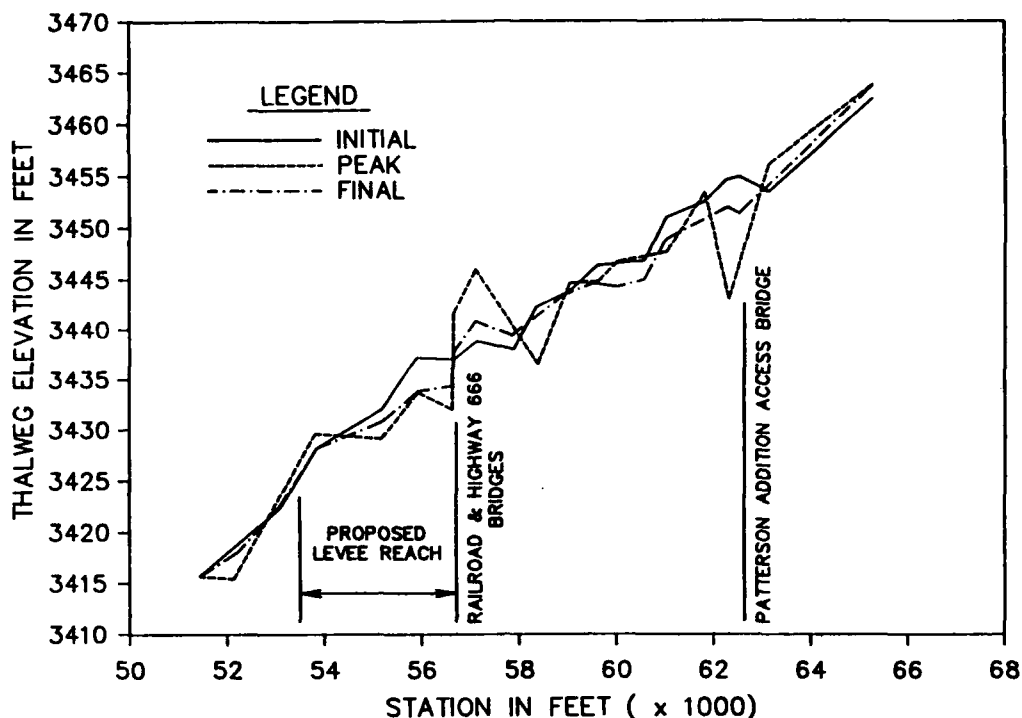in the model.  Downstream control should be investigated in detail to

14

Figure 3.   Standard project flood with average sediment discharge

determine if there is invert control at the constriction.  Bed changes during
the course of the SPF simulation at five cross sections in the leveed reach
are shown in Plates 10-14.

25.   The model results upstream from the leveed reach are also interest-
ing.  The San Francisco River is constricted through the town of Clifton, and
as a result, significant scour occurs at the first sections inside the im-
proved reach of river.  The model showed up to 13 ft of scour near the SPF
peak.  Unless the Patterson Addition Access road bridge is built on bedrock or
has extensive pile footings, its integrity is questionable during a major
event.  In general, the entire reach of river through Clifton is subject to
degradation during a major flood.  This is due to the increased velocities
caused by the constriction through town and the backwater created by the
constriction upstream from town that reduces sediment transport potential to
the improved reaches.  Backwater from bridges and local constrictions serve to
create local areas of deposition and scour.

## Sensitivity Analysis

26.   Sensitivity of the model to sediment inflow was tested by

15

increasing and decreasing the sediment inflow for each size class by a constant ratio of 2.0 and 0.5, respectively. Comparisons of the total sand and gravel inflow curves to the measured data are shown in Figure 4. Calculated thalweg elevations at the peak of the SPF with the different sediment inflow assumptions are compared in Figure 5. Calculated thalweg elevations at the end of the SPF are compared in Figure 6. These comparisons show that bed elevations in the leveed reach are not significantly affected by this range of sediment inflow assumptions.

27. Additional sensitivity studies were conducted to determine the effect of the bed material gradation on degradation and aggradation in the study reach. The average bed material gradation from the test trench data had a $D_{50}$ = 3.5mm. A coarser gradation, $D_{50}$ = 9mm, and a finer gradation, $D_{50}$ = 1.5mm, were obtained from the limits of the gradation data for gravelly-sands from the test trenches (Plate 6). Results of the sensitivity test are shown in Figure 7. The bed material gradation did not significantly affect scour or deposition in the study reach. The sensitivity test does not address the possible effects of longitudinal variation in the bed material gradation.

28. A sensitivity test was conducted to determine the calculated scour and deposition for a flood event similar to the 1983 flood of record. Results of the simulation are shown in Figure 8, and compared to SPF results in Plates 15 and 16. The magnitude of scour for the 1983 simulated flood was similar to the SPF scour in the leveed reach.

## Historical Analysis

29. Historical accounts of numerous floods show that the alignment of the main river channel has remained relatively straight, and the banks essentially stable, through the proposed levee reach. Bank erosion along the left descending bank adjacent to the proposed levee was reported during interviews with local residents after the 1983 flood (USAED, Los Angeles, 1986). Less than 20 ft of bank erosion was reported at the upstream end of the reach near the bridges and less than 50 ft at the downstream end near Ward Canyon. The existing masonry floodwall at the bridges sustained some damage during the 1983 floods. Repair work observed during a May 1988 field reconnaissance indicated that damage was caused by floodflows returning to the channel rather than by undercutting of the toe.
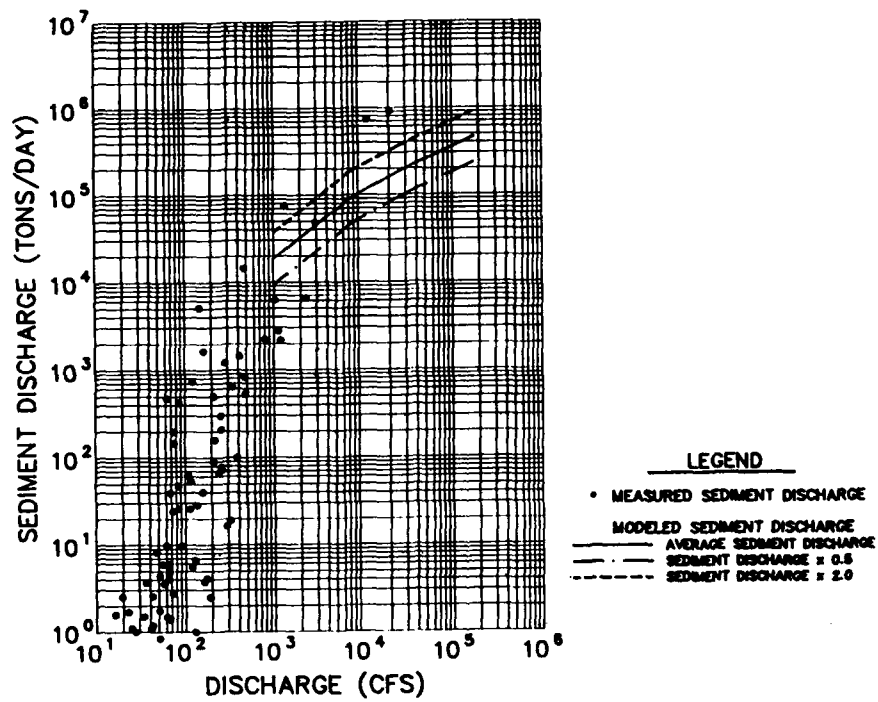
16

Figure 4. Measured and modeled suspended sediment, San Francisco River
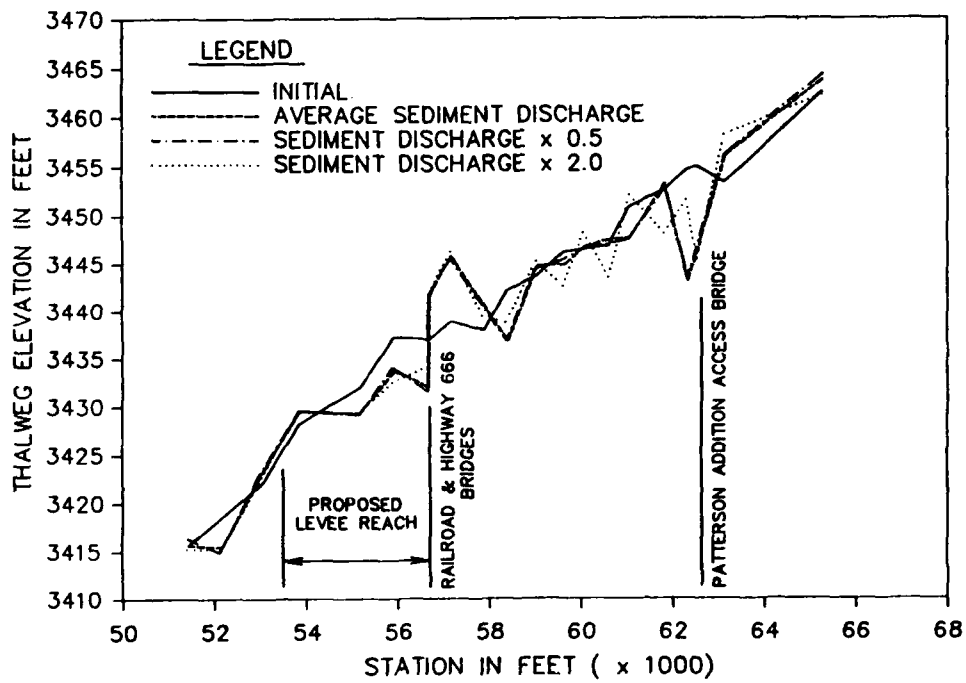


Figure 5. Sensitivity of calculated thalweg to sediment inflow at peak standard project flood
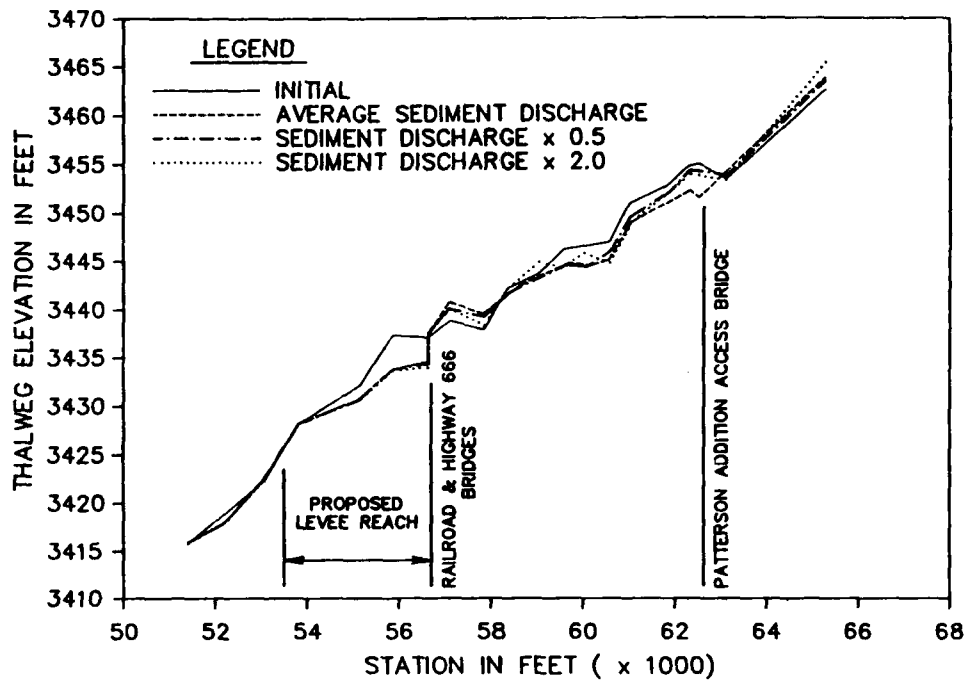
Figure 6. Sensitivity of calculated thalweg to sediment
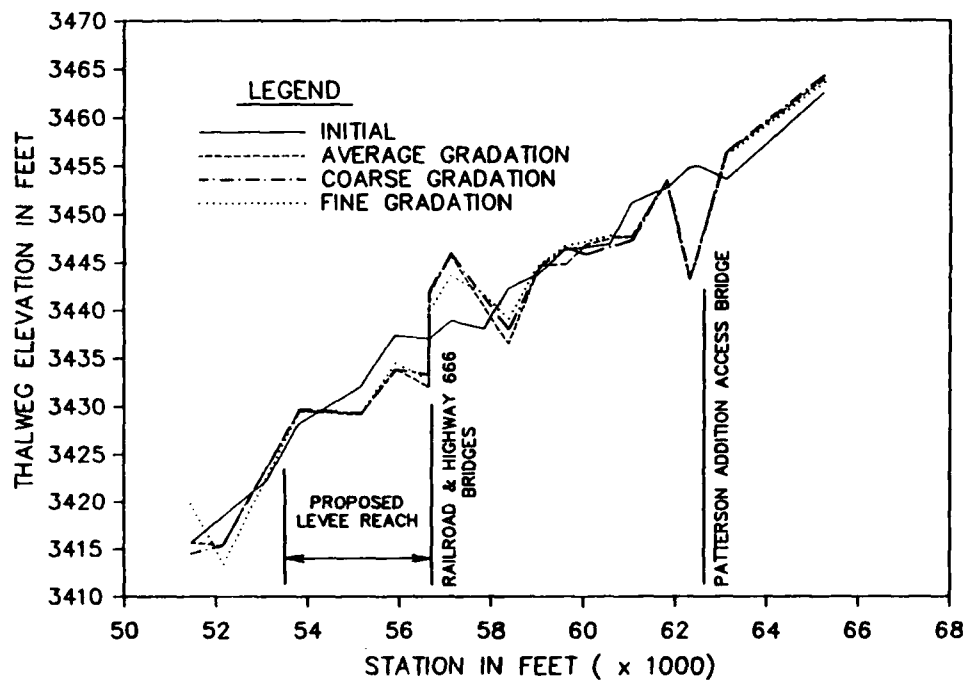inflow at end of standard project flood



Figure 7. Sensitivity of calculated thalweg to bed material
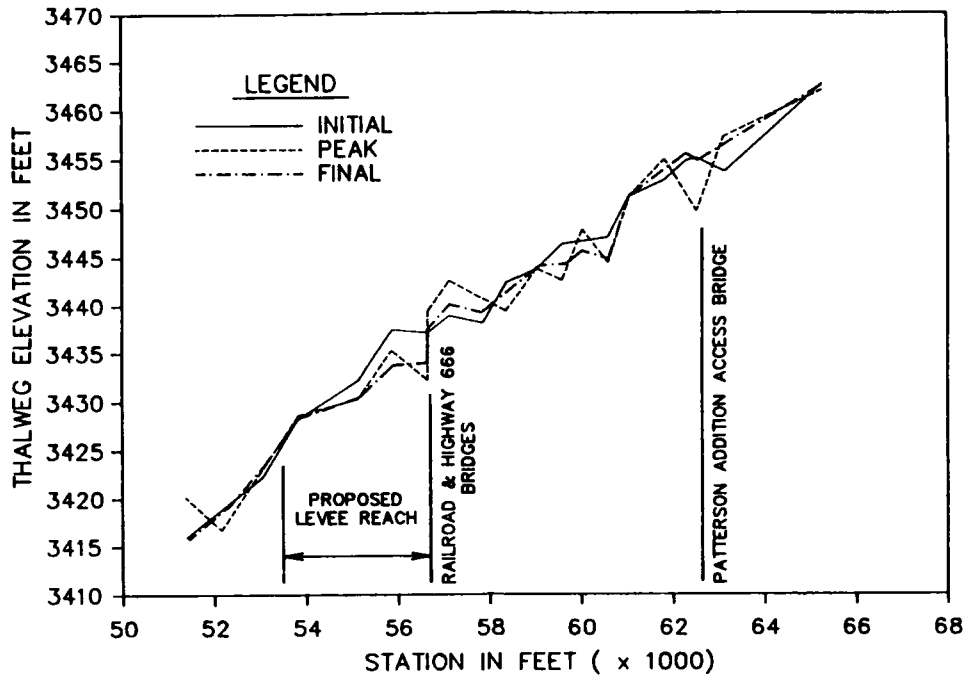gradation at peak standard project flood

18

Figure 8. 1983 hydrograph

30. Riffle and pool sequences, characteristic of gravel-bed rivers, typically have regular spacings with impinging flows at pools on alternating sides of the channel. The San Francisco River, through Clifton, even with its width constraints, has developed a typical riffle-pool sequence (Plate 17). Aerial photos taken after the 1983 flood reveal a consistent pattern upstream of the railroad bridge with pools alternating from one side of the channel to the other, with spacings of 5 to 8 times the channel width or about 1,000 ft. Downstream from Clifton, where the channel is no longer encroached on by development, the natural channel also has a consistent sequence of alternating pools and riffles, with spacings of about five channel widths or 1,800 ft. The consistent alternating sequence upstream and downstream of the proposed leveed reach is discontinuous in the leveed reach itself. If the upstream sequence were to extend downstream past the highway bridge, the flow would impinge against the existing left-descending bank near sta 552+00. This is the area where the continuity of the slag deposit is uncertain, and the proposed levee alignment sets on top of the inferred slag deposit alignment. However, the existing thread of the channel does not follow the upstream pattern and does not impinge on the left-descending bank until further downstream where the slag deposit is exposed, and the levee alignment is set back

19

100 ft. Comparison of the 1939 topographic map and the 1983 aerial photos indicates that the low-flow channel has moved away from the left descending bank adjacent to the proposed levee between river stations 544+00 and 552+00. This favorable condition is attributed to longitudinal progression of the gravel bar along the left descending side of the channel. Current plans to excavate this gravel bar could result in unfavorable realignment of the low-flow channel toward the left bank.

31. Historical survey data were reviewed to evaluate riverbed stability over time. Historical profile and cross-section data were published in the Planning Assistance Study (USAED, Los Angeles, 1979) and are shown in Plates 18-22. About 4 ft of degradation occurred in the low-flow channel at the downstream end of the project reach between 1939 and 1977. This condition was confirmed during site reconnaissance in May 1988. The degradation has undercut the adjacent slag deposit and exposed a utility pipe crossing the river. Upstream migration of the headcut has apparently been arrested by a collection of large rocks near sta 539+00. These boulders, some reaching 20 ft in diameter, extend from the left-descending bank across the low-flow channel, and about 100 ft onto the adjacent gravel bar. There is no observable bedrock at this location; the boulders are reported to be remnants of a rock promontory from the right bank that was blasted away in 1979 (USAED, Los Angeles, 1988). Structural measures such as this may be required to prevent future degradation of the low-flow channel and ensure a thalweg elevation above the slag deposit toe.

## Regression Analysis

32. A river that erodes its boundary will form a channel that effectively carries its water and sediment load. Theories of equilibrium channel formation are based on the premise that hydraulic parameters can be determined, on the average, as a function of an independent variable, usually discharge. Regression equations have been developed by a number of investigators for width, depth, velocity, resistance, and slope. Of these, predictors for width are the most reliable.

33. Hydraulic geometry regression equations were determined by Leopold and Maddock (1953) for selected rivers in the midwestern United States. The generalized equation for width is expressed:

$$w = aQ^b$$

where:

  w = width

  Q = characteristic discharge

  a = coefficient characteristic of the particular river

  b = exponent determined to generally be a constant

Potential for bank erosion adjacent to the proposed levee at Clifton was evaluated by calculating predicted widths using the regression equations of Northwest Hydraulic Consultants (NHC) and Williams (1978).

34. NHC (1984) proposed that hydraulic geometry could be used for preliminary design of stable channels. They came up with an exponent b of 0.50 for gravel bed streams. NHC also determined that the coefficient a could be determined depending on the nature of the bank material. Banks with high resistance (riprap, cobbles and boulders, or stiff clay) are assigned a coefficient of 1.6, moderate resistance (coarse gravel, medium clay) are assigned a coefficient of 2.1, and low resistance (sand and silt, soft clay) are assigned a coefficient of 2.7. In the NHC procedure, it is recommended that either the 10-yr-frequency peak discharge or 50 percent of the design discharge, whichever is larger, be used for Q in the regression equation. Using the NHC procedure with a coefficient of 2.1, widths of 565 and 606 ft were calculated for discharges of 72,500 and 83,500 cfs, respectively (50% of design and SPF discharges).

35. At-a-station hydraulic geometry regression equations consider temporal variations in channel geometry as discharge fluctuates at a cross section. The exponent b in equations for width are significantly affected by bank cohesion. A river with near-vertical, cohesive banks would have an exponent near zero because the width would essentially be constant with changes in discharge. Based on empirical data from various types of channels, Williams (1978) came up with an exponent of 0.40 for sandy-gravelly rivers with one cohesive bank and one noncohesive bank. According to Williams, this result compares to a theoretical value of 0.45 using Langbein's (1965) minimum variance theory. Coefficients for Williams' equation were determined for the San Francisco River using an aerial photo dated 7 October 1983. An average channel width of 110 ft in the study reach was measured from the photos. Using the mean daily flow (1110 cfs) for 7 October 1983, a coefficient of 6.7

21

was calculated. Using the channel trace cleared of vegetation as indicative of channel width at the flood peak, an average channel width of 340 ft was determined from the aerial photos for the flood that occurred on 2 October 1983. Using the mean daily flow (52,200 cfs) for the discharge in the hydraulic geometry equation, a value of 4.4 was calculated for coefficient $a$. Using the peak flow of 90,900 cfs, a value of 3.5 was calculated for $a$. The hydraulic geometry equation using an exponent of 0.40 and a coefficient of 3.5 yielded a predicted width of 406 ft and 430 ft for peak discharges of 145,000 cfs and 167,000 cfs, respectively. With a coefficient of 6.7, predicted widths of 777 ft and 873 ft were calculated for peak discharges of 145,000 cfs and 167,000 cfs, respectively.

36. Proposed widths between the existing rock right bank and the proposed levee varies between 300 and 500 ft. This analysis indicates that, unrestrained, there would be a tendency for the river to widen past the proposed levee alignment. This regression analysis does not account for the erosion-resistant slag deposit adjacent to the left descending bank. In addition, the application of these generalized equations to flashy streams like the San Francisco River is questionable. It can be concluded however that the integrity of the slag deposit is important because it arrests the river's tendency to widen at high flows.

PART IV: CONCLUSIONS AND RECOMMENDATIONS

37. Results of the numerical model study indicated that the channel bed elevation would vary significantly during the passage of a major flood. Significant scour would occur downstream from the Highway 666 bridge where existing bank protection is provided by a masonry wall with unknown toe protection, and the proposed levee toe would have less than the 100-ft setback. Degradation in the lower portion of the project reach, during the passage of a major flood, is prevented by a backwater condition created by a downstream constriction. However, degradation in this lower reach occurs during low flows when the backwater condition does not exist.

38. The review of historical data, including aerial photos, surveyed profiles, and eye witness accounts, indicates that there is some potential for bank erosion in the reach downstream from the bridge at sta 566+80 during high flows and in the lower reaches during lower flows.

39. Hydraulic geometry regression analysis indicates there would be a tendency for channel widening during flood flows.

40. The existing masonry wall, downstream from the bridges, is in a zone of high bank erosion and degradation potential. It is recommended that the existing toe elevation be identified, and if necessary, additional toe protection provided. Numerical model study results indicate scour depths on the order of 5 ft with existing channel geometry. However, these calculations are qualitative. Engineering judgment must be relied on heavily to determine an adequate depth for toe protection. Downstream from the masonry wall, the levee alignment lies on top of the inferred slag deposit. In this reach, the slag must be considered an integral part of the levee design and toe protection is recommended to insure the integrity of the slag deposit. Historical data indicate that the gravel bar in this location has existed for a long time, and provides some toe protection for bank improvements. However, if removed, the flow could redirect itself and move toward the left-descending bank. It is recommended that the gravel bar be allowed to remain in place.

41. Downstream from sta 550+00, the existing slag deposit along the left descending bank is critical to the integrity of the proposed levee. In areas where the slag deposit is being undercut, remedial efforts are recommended to halt undermining of the existing toe. This could be accomplished by dumped stone along the toe or by grade control structures. The longitudinal

23

continuity of the slag deposit is essential to the integrity of the levee. Its existence should be confirmed.

42. Downstream from sta 550+00, setback distance for the levee is not especially critical with respect to bank erosion. As long as the slag deposit remains in place, the levee toe will be protected. Some slumping or breaking of the slag deposit during a single flood event would not affect the proposed levee toe. The mass of the deposit would be sufficient to continue to provide toe protection for the duration of the flood. However, it is recommended that the integrity of the slag deposit be included as a maintenance requirement for the project. The levee should be set back far enough so that the levee toe is not structurally effected by the presence of the slag deposit. Setback distance can be determined based on real estate, structural, geotechnical, and construction considerations.

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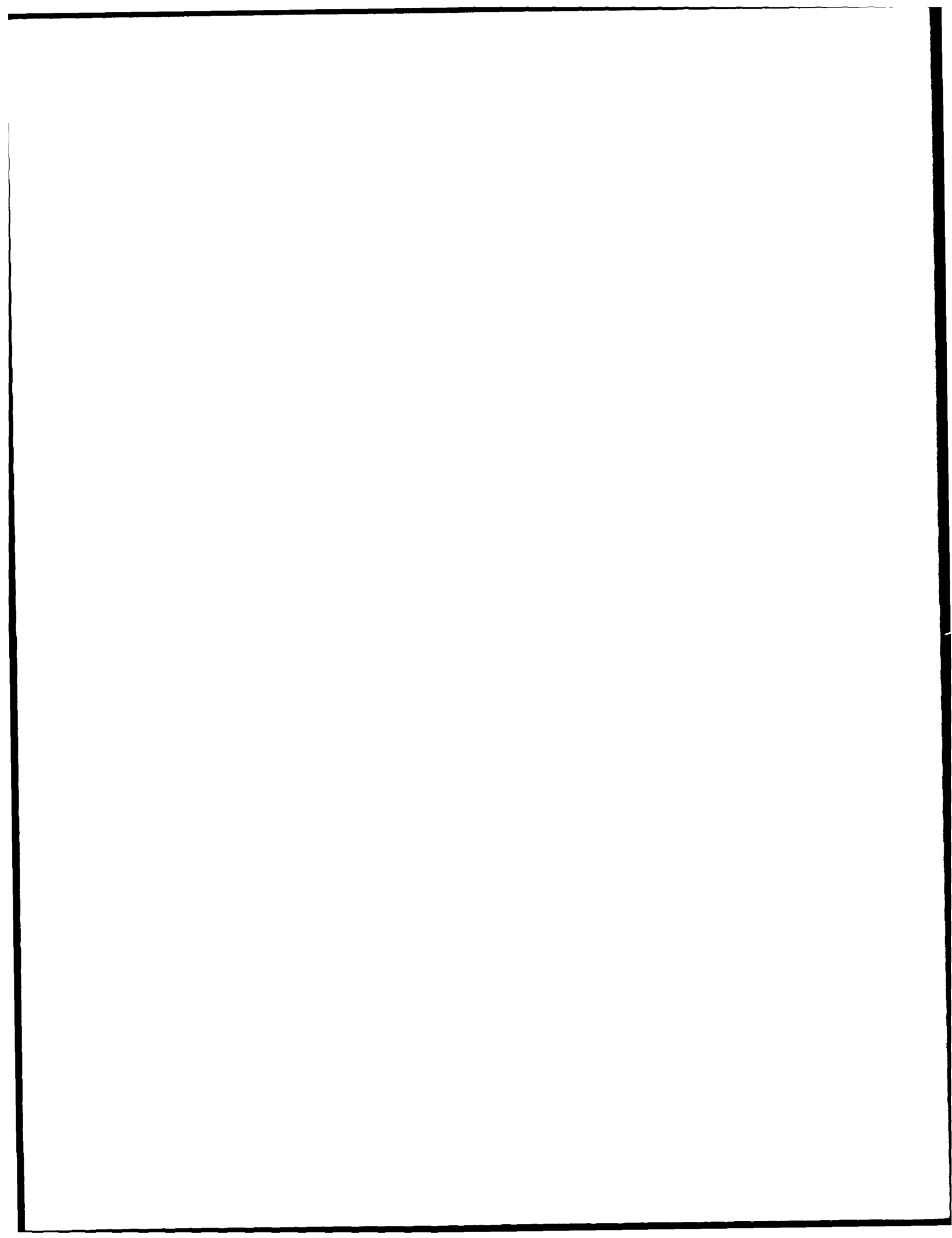
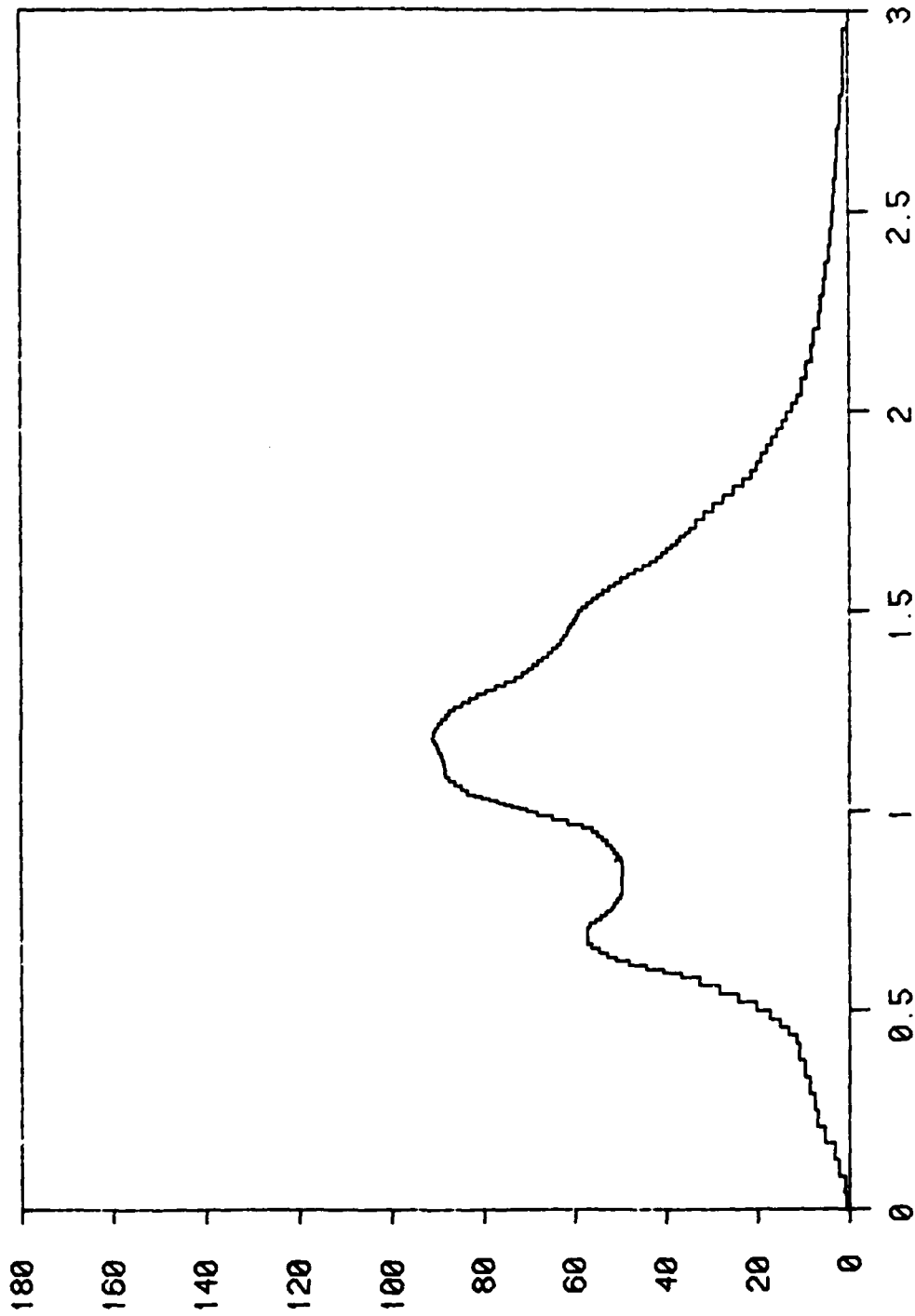STANDARD PROJECT HISTOGRAPH
SAN FRANCISCO RIVER



THOUSANDS

DISCHARGE IN CFS

180
160
140
120
100
80
60
40
20
0

0    0.5    1    1.5    2    2.5    3

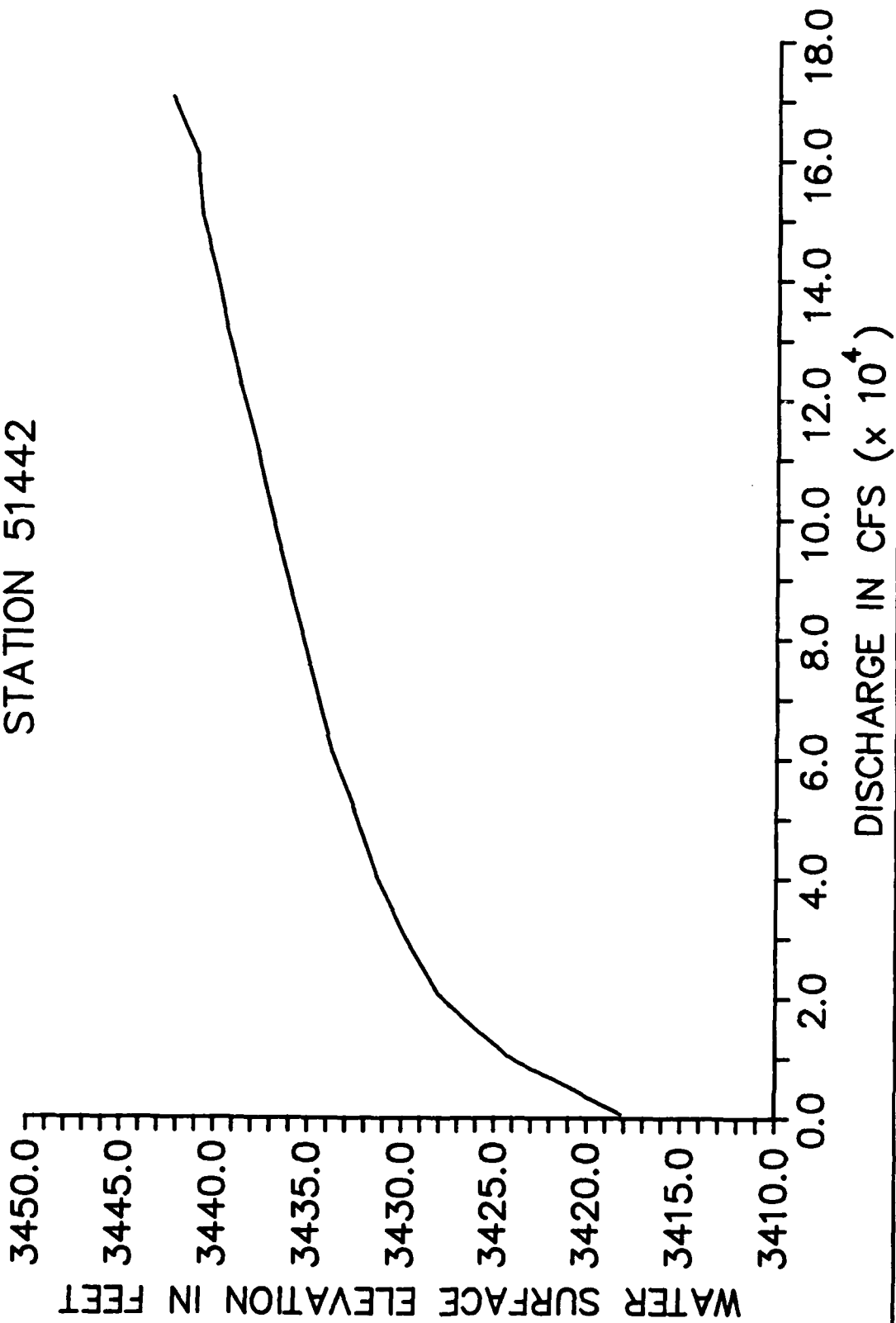TIME IN DAYS

PLATE 1
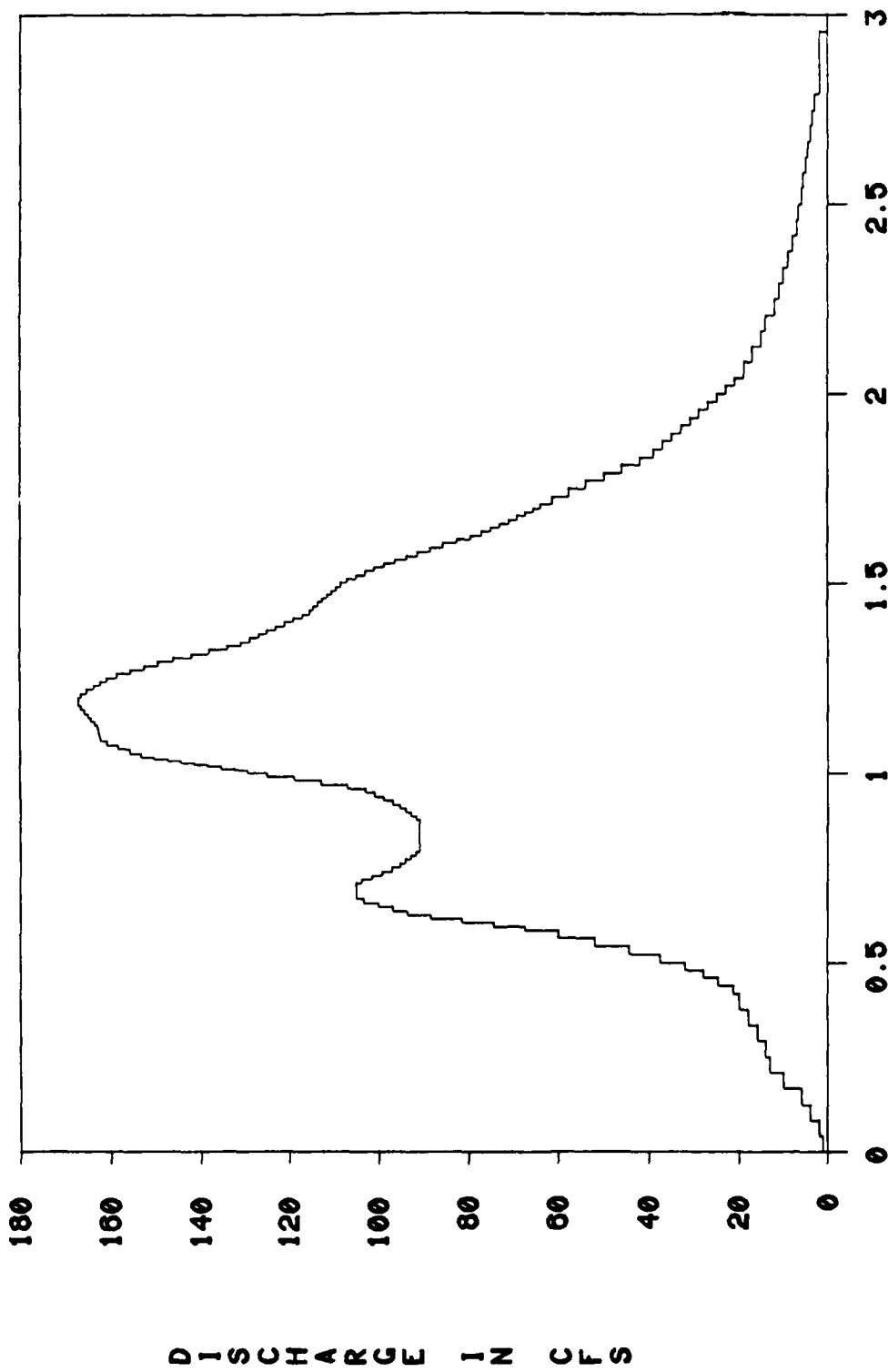
1983 HISTOGRAPH
SAN FRANCISCO RIVER

THOUSANDS

DISCHARGE IN CFS

TIME IN DAYS

PLATE 2

DOWNSTREAM RATING CURVE
STATION 51442

WATER SURFACE ELEVATION IN FEET

DISCHARGE IN CFS ($\times 10^4$)

PLATE 3

# RATING CURVE UPSTREAM FROM RAILROAD BRIDGE STATION 56680

WATER SURFACE ELEVATION IN FEET

3480.0 3475.0 3470.0 3465.0 3460.0 3455.0 3450.0 3445.0 3440.0

DISCHARGE IN CFS ($\times 10^4$)

0.0   2.0   4.0   6.0   8.0   10.0   12.0   14.0   16.0   18.0
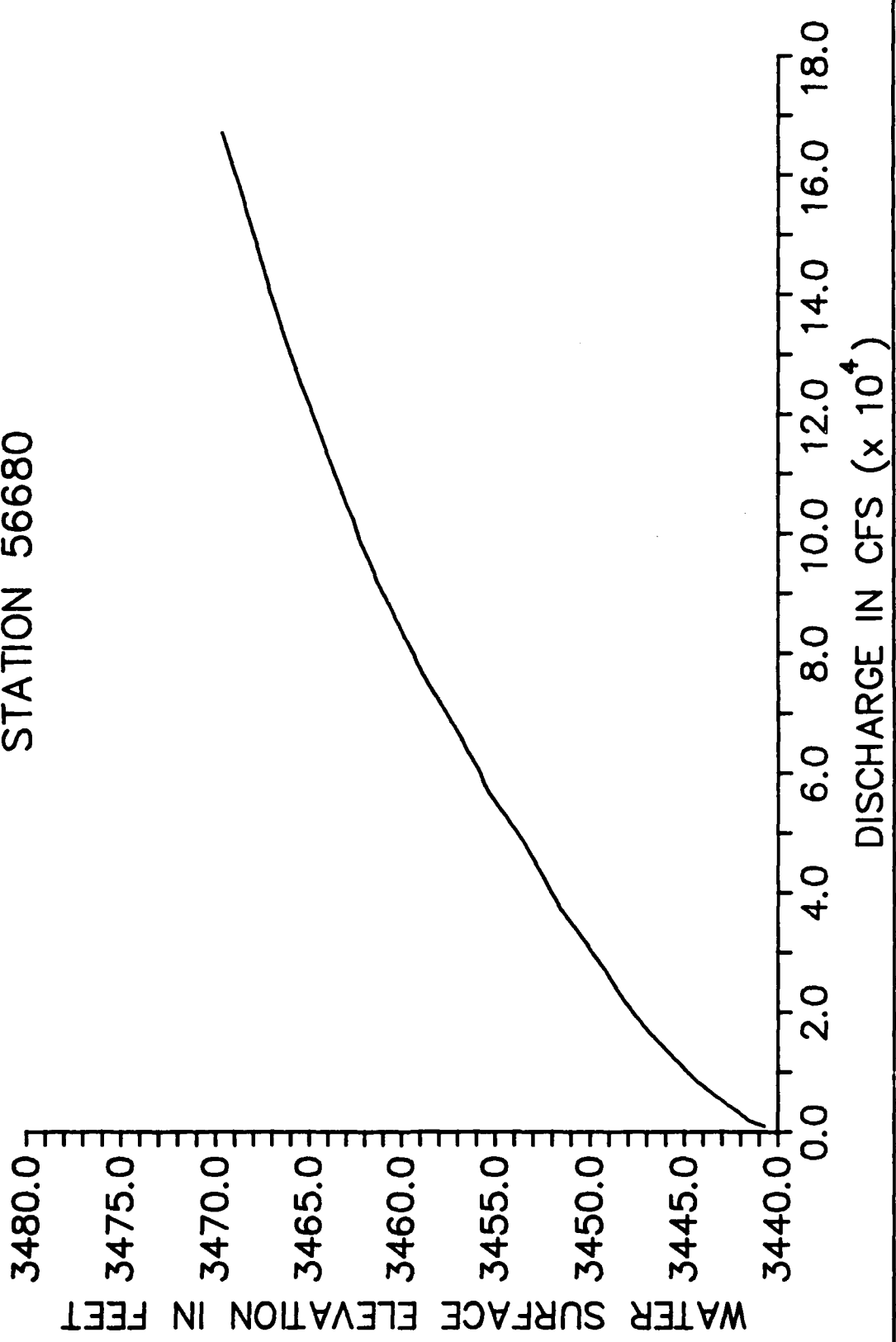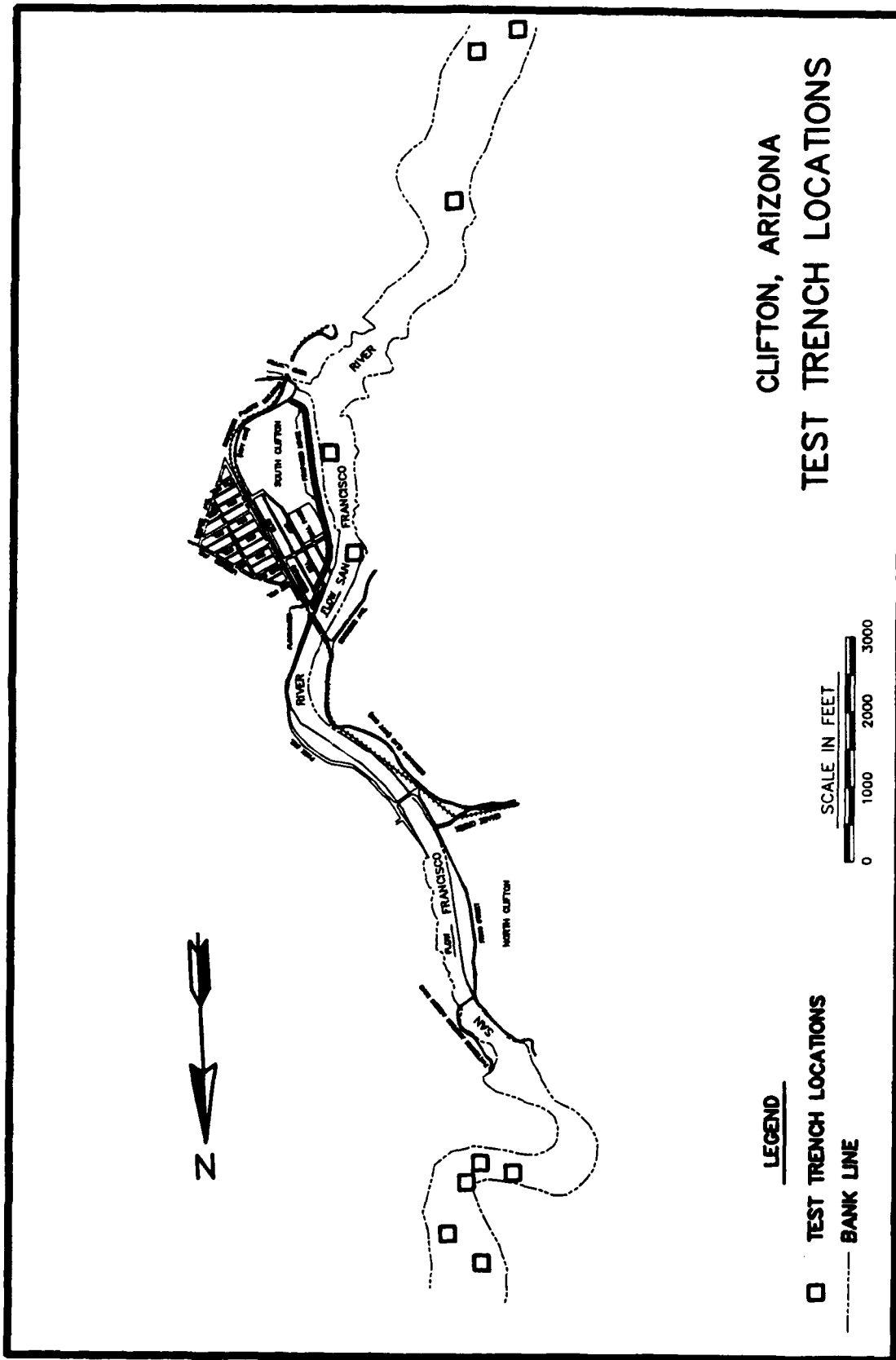
PLATE 4

CLIFTON, ARIZONA
TEST TRENCH LOCATIONS

N

SCALE IN FEET

0    1000    2000    3000

LEGEND

□  TEST TRENCH LOCATIONS

─────  BANK LINE
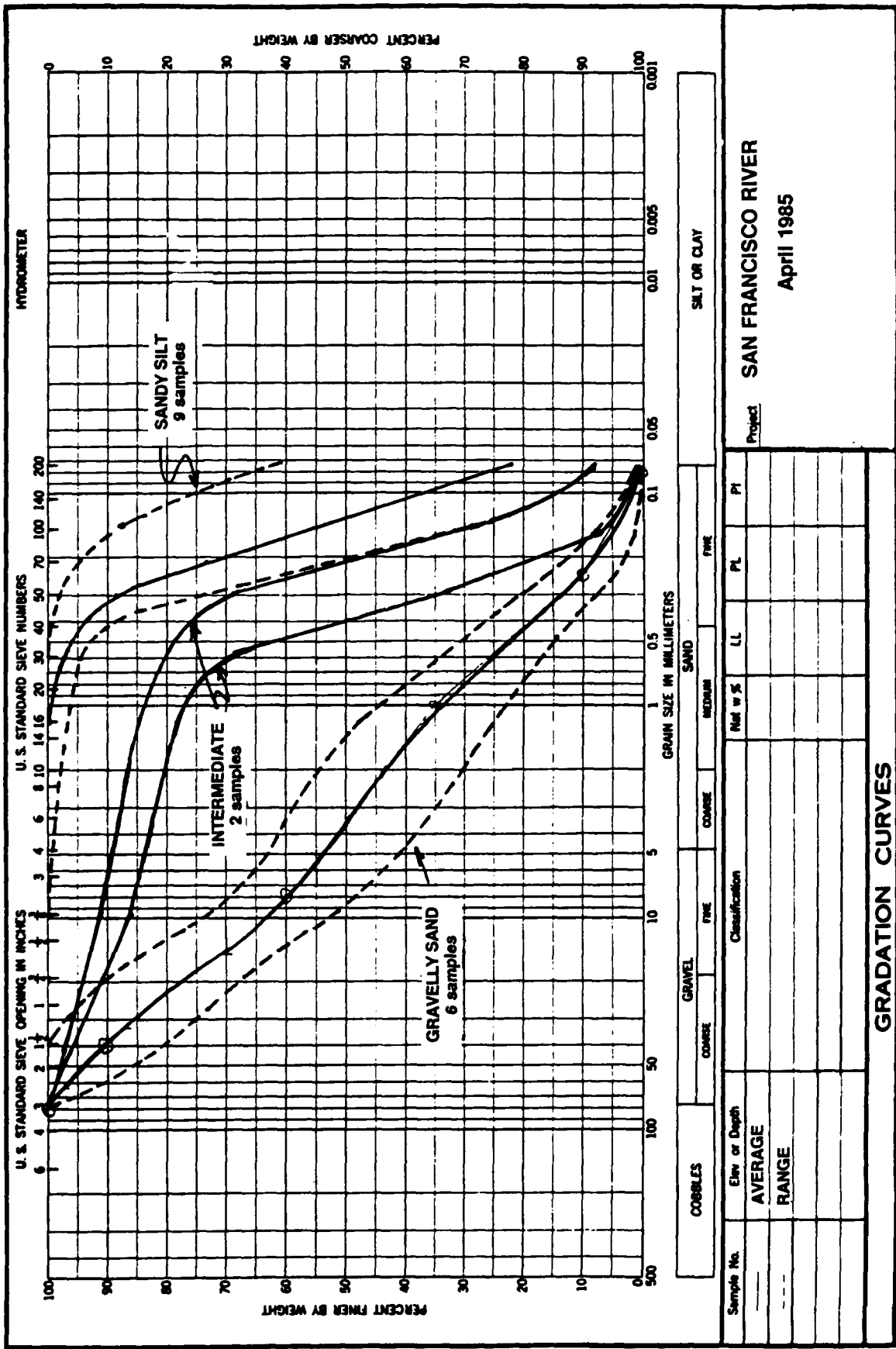
PLATE 5

GRADATION CURVES

SAN FRANCISCO RIVER

April 1985

PLATE 6

# MEASURED AND MODELED SUSPENDED SEDIMENT
## SAN FRANCISCO RIVER
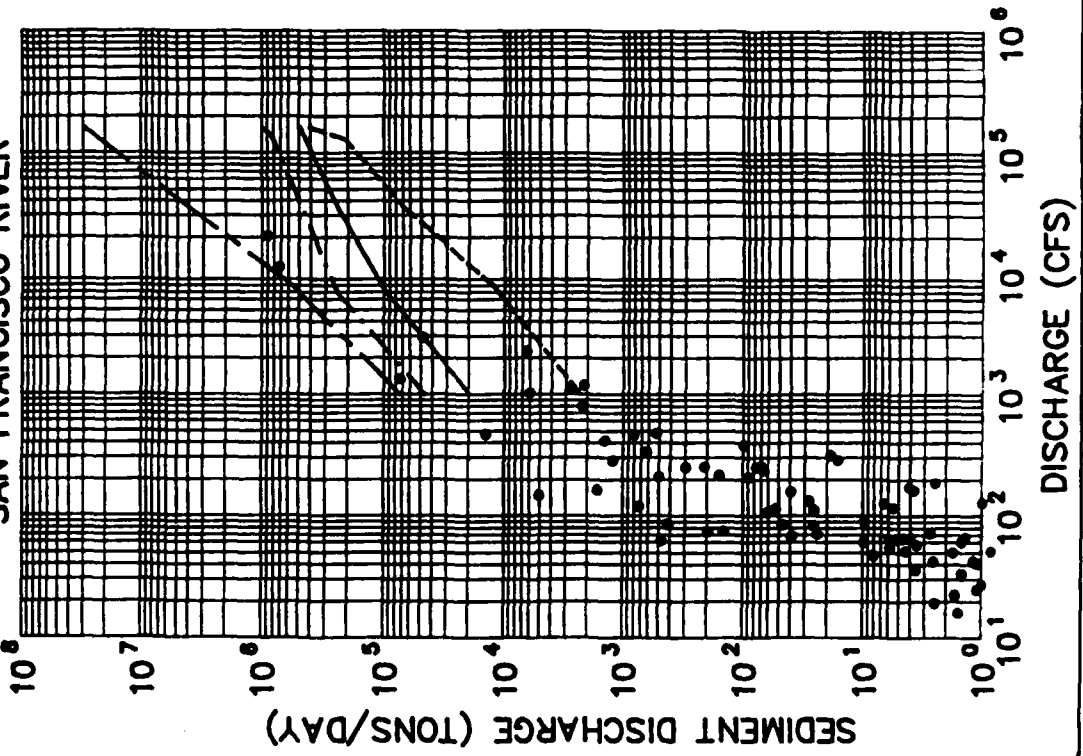


PLATE 7

AVERAGE SEDIMENT DISCHARGE OF SAND
SAN FRANCISCO RIVER AT CLIFTON, ARIZONA

DISCHARGE (CFS)

SEDIMENT DISCHARGE (TONS/DAY)

LEGEND

VERY FINE
FINE
MEDIUM
COARSE
VERY COARSE

PLATE 8
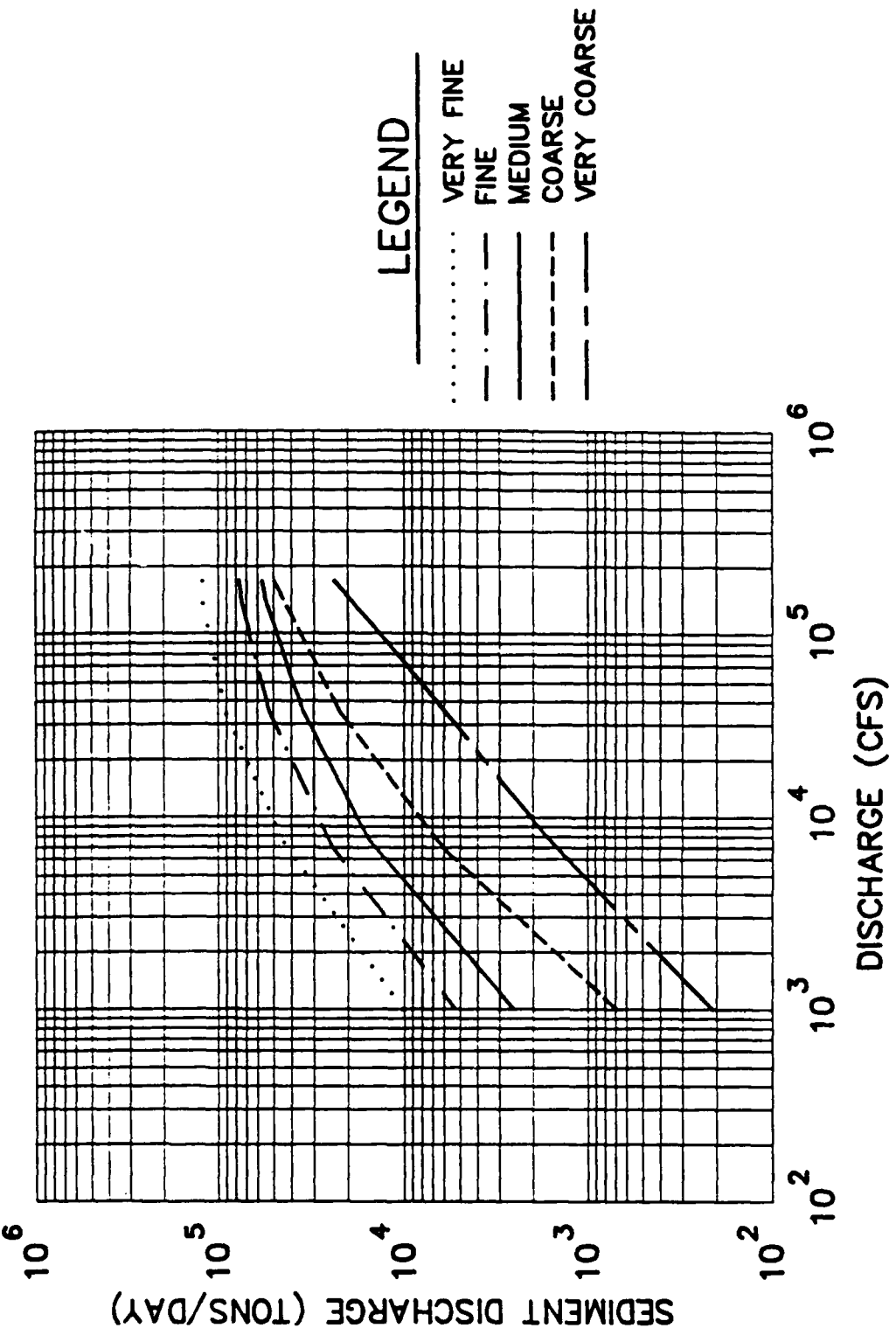
AVERAGE SEDIMENT DISCHARGE OF GRAVEL
SAN FRANCISCO RIVER AT CLIFTON, ARIZONA

LEGEND

VERY FINE
FINE
MEDIUM
COARSE
VERY COARSE

SEDIMENT DISCHARGE (TONS/DAY)

DISCHARGE (CFS)

PLATE 9
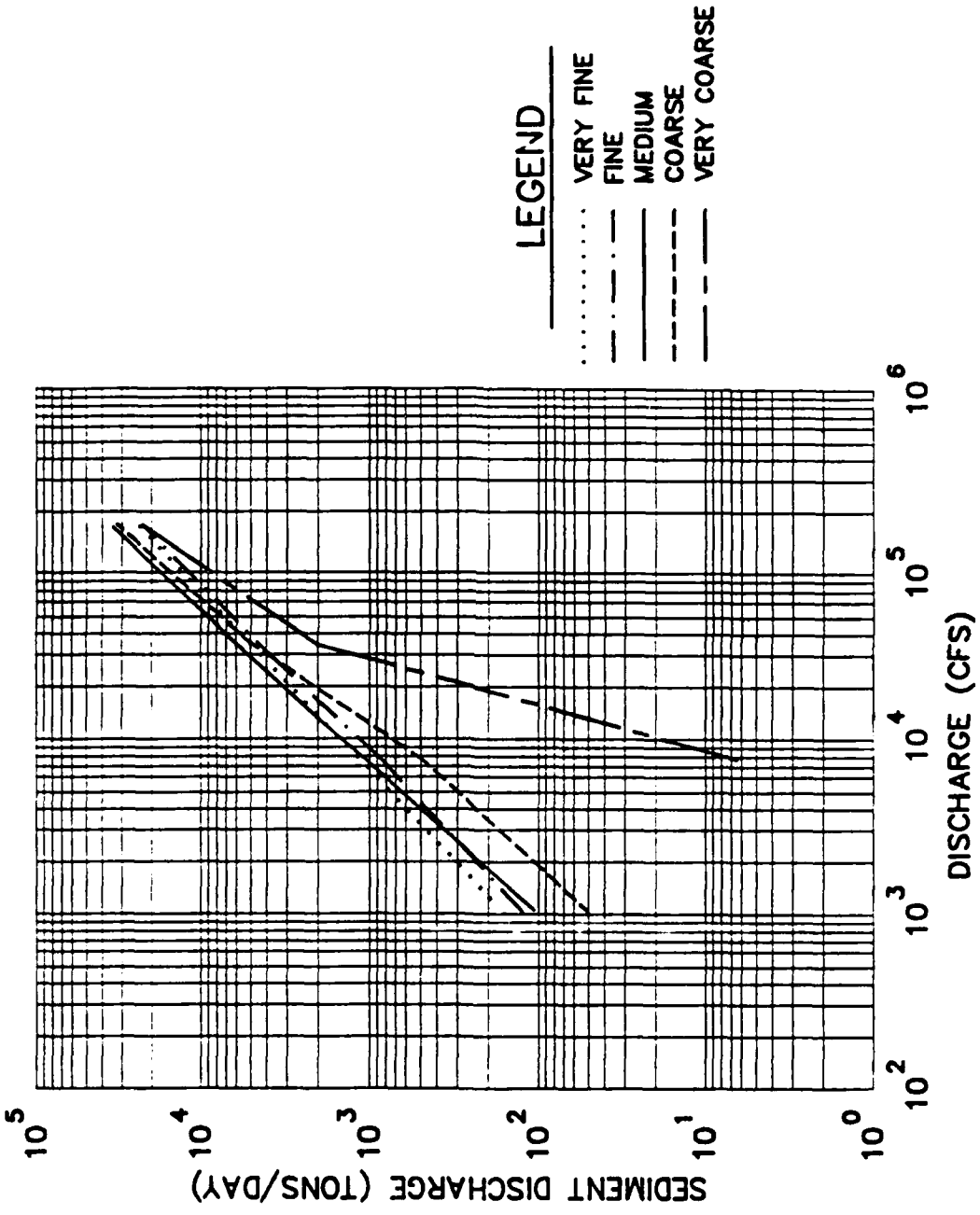
BED CHANGE DURING STANDARD PROJECT FLOOD
STATION 56660

PLATE 10

BED CHANGE DURING STANDARD PROJECT FLOOD
STATION 55930
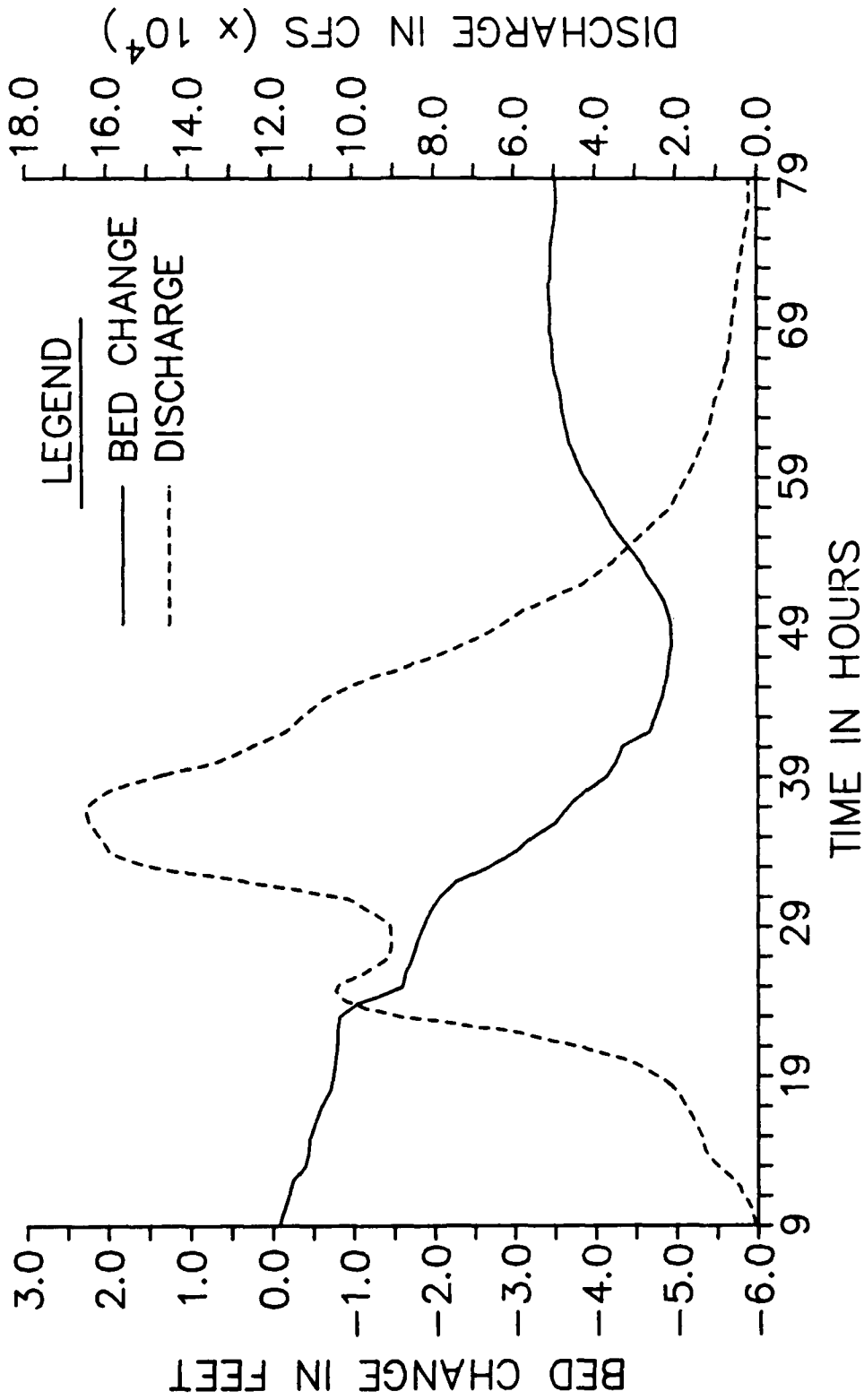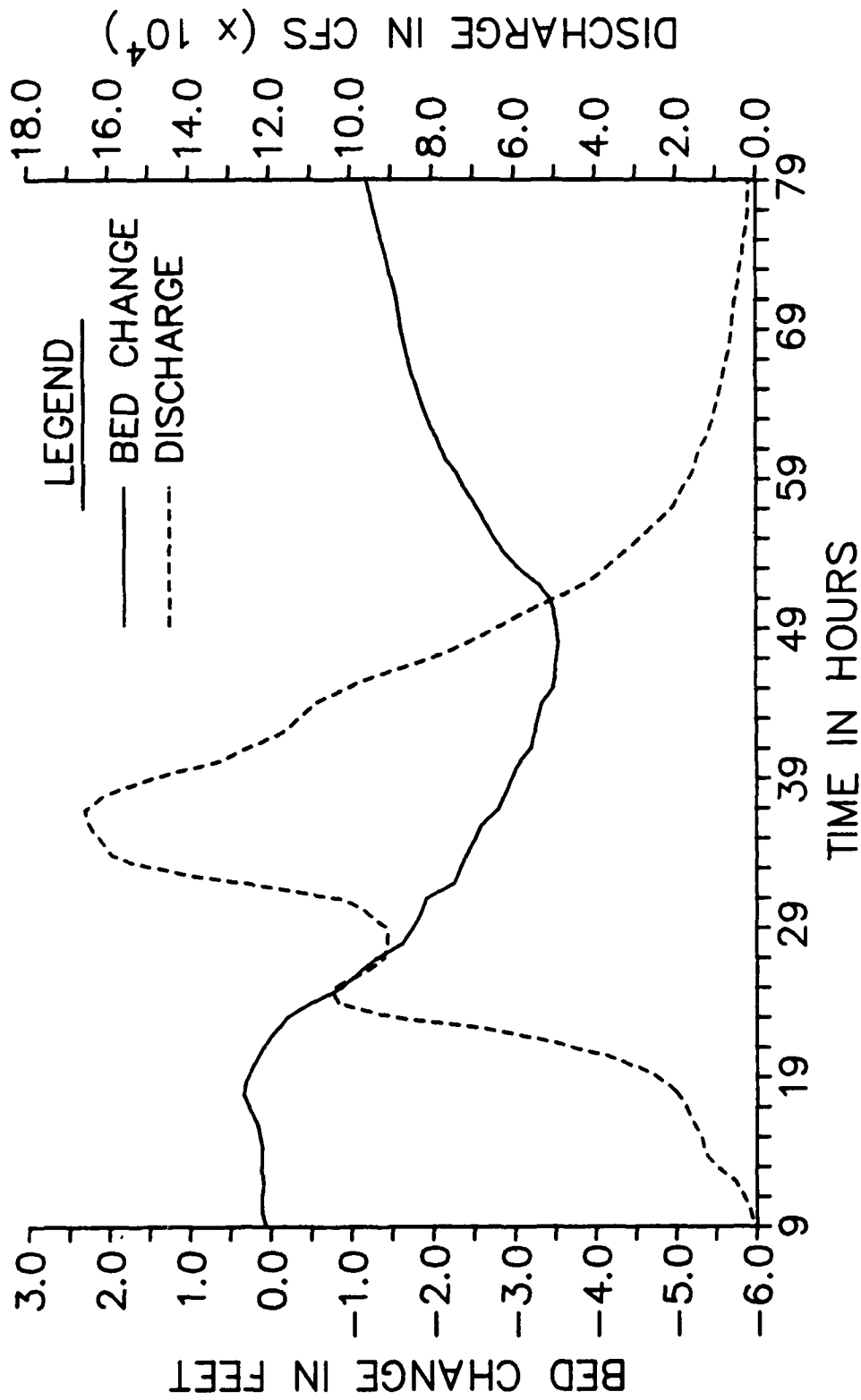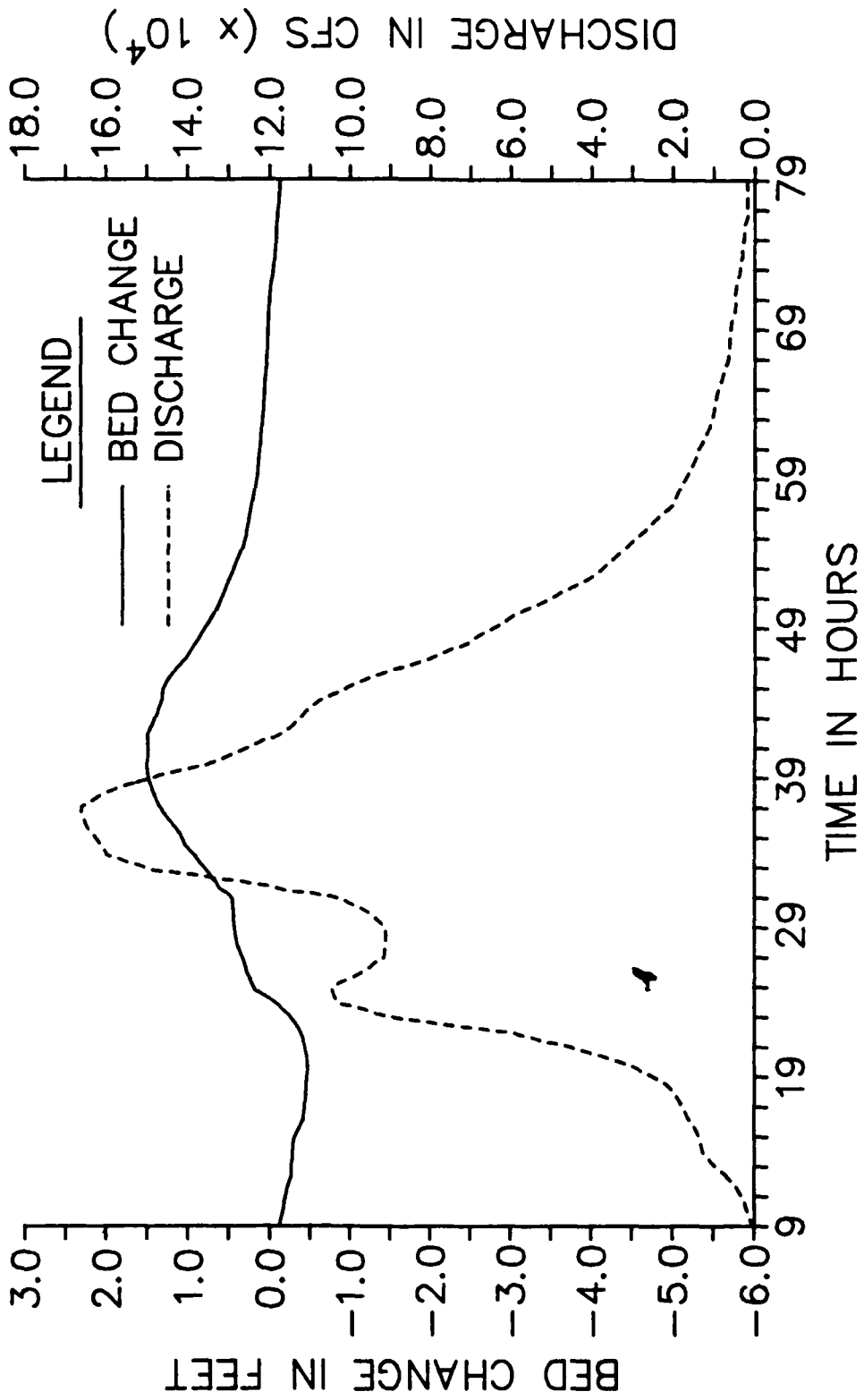
LEGEND
BED CHANGE
DISCHARGE

DISCHARGE IN CFS (x 10⁴)

BED CHANGE IN FEET

TIME IN HOURS

PLATE 11

BED CHANGE DURING STANDARD PROJECT FLOOD
STATION 55180

LEGEND
—— BED CHANGE
---- DISCHARGE

DISCHARGE IN CFS (x 10⁴)

BED CHANGE IN FEET

TIME IN HOURS

PLATE 12

BED CHANGE DURING STANDARD PROJECT FLOOD
STATION 53840

LEGEND
——— BED CHANGE
------ DISCHARGE

DISCHARGE IN CFS (x 10⁴)

BED CHANGE IN FEET

TIME IN HOURS

PLATE 13

BED CHANGE DURING STANDARD PROJECT FLOOD
STATION 53100

DISCHARGE IN CFS (x 10⁴)

LEGEND

BED CHANGE
DISCHARGE

TIME IN HOURS

BED CHANGE IN FEET
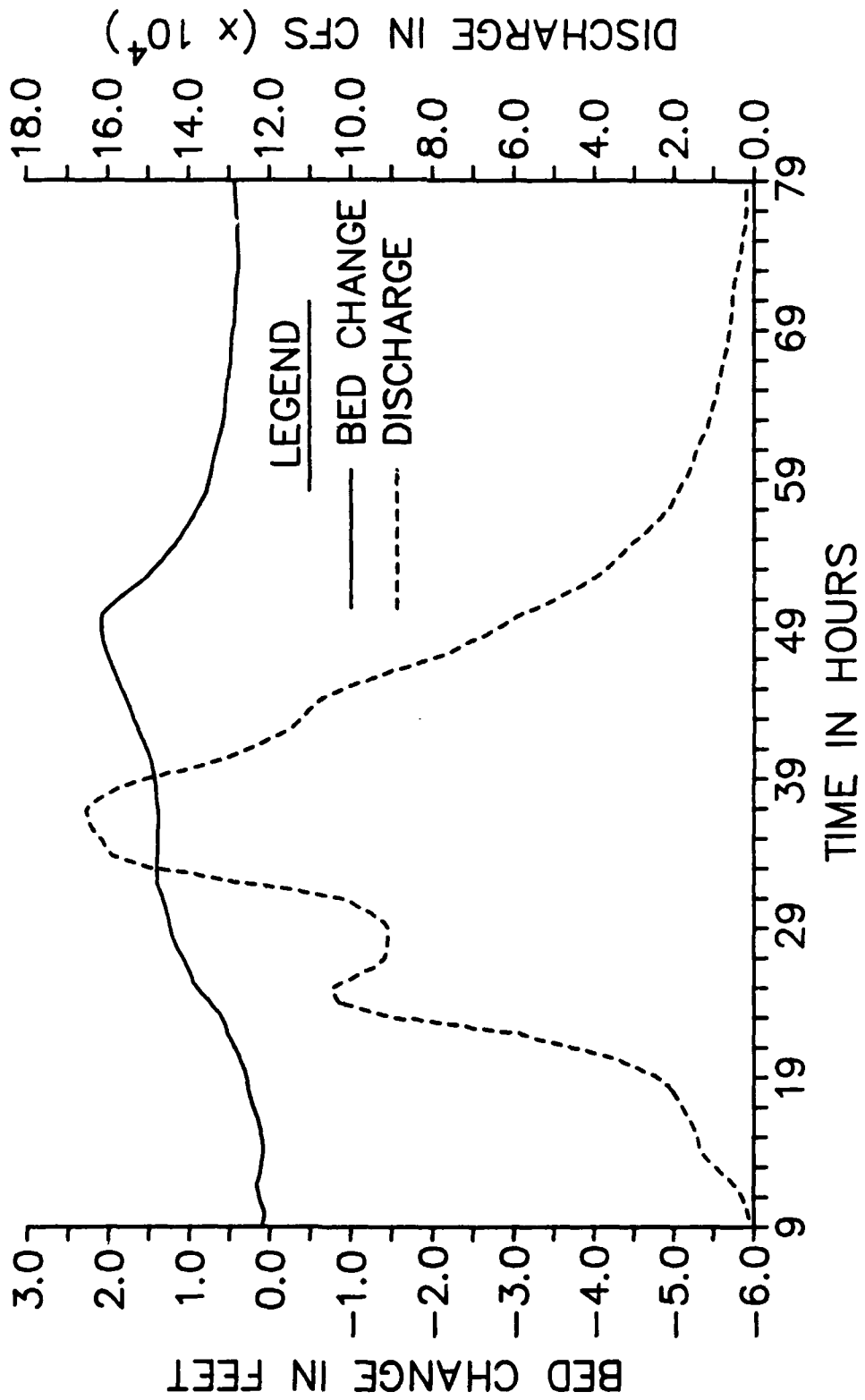
PLATE 14

SAN FRANCISCO RIVER AT CLIFTON, ARIZONA
PEAK OF FLOOD

LEGEND

INITIAL
PEAK (1983)
PEAK (STANDARD PROJECT FLOOD)

PATTERSON ADDITION ACCESS BRIDGE

RAILROAD & HIGHWAY 666 BRIDGES

PROPOSED LEVEE REACH

STATION IN FEET ( x 1000)

THALWEG ELEVATION IN FEET
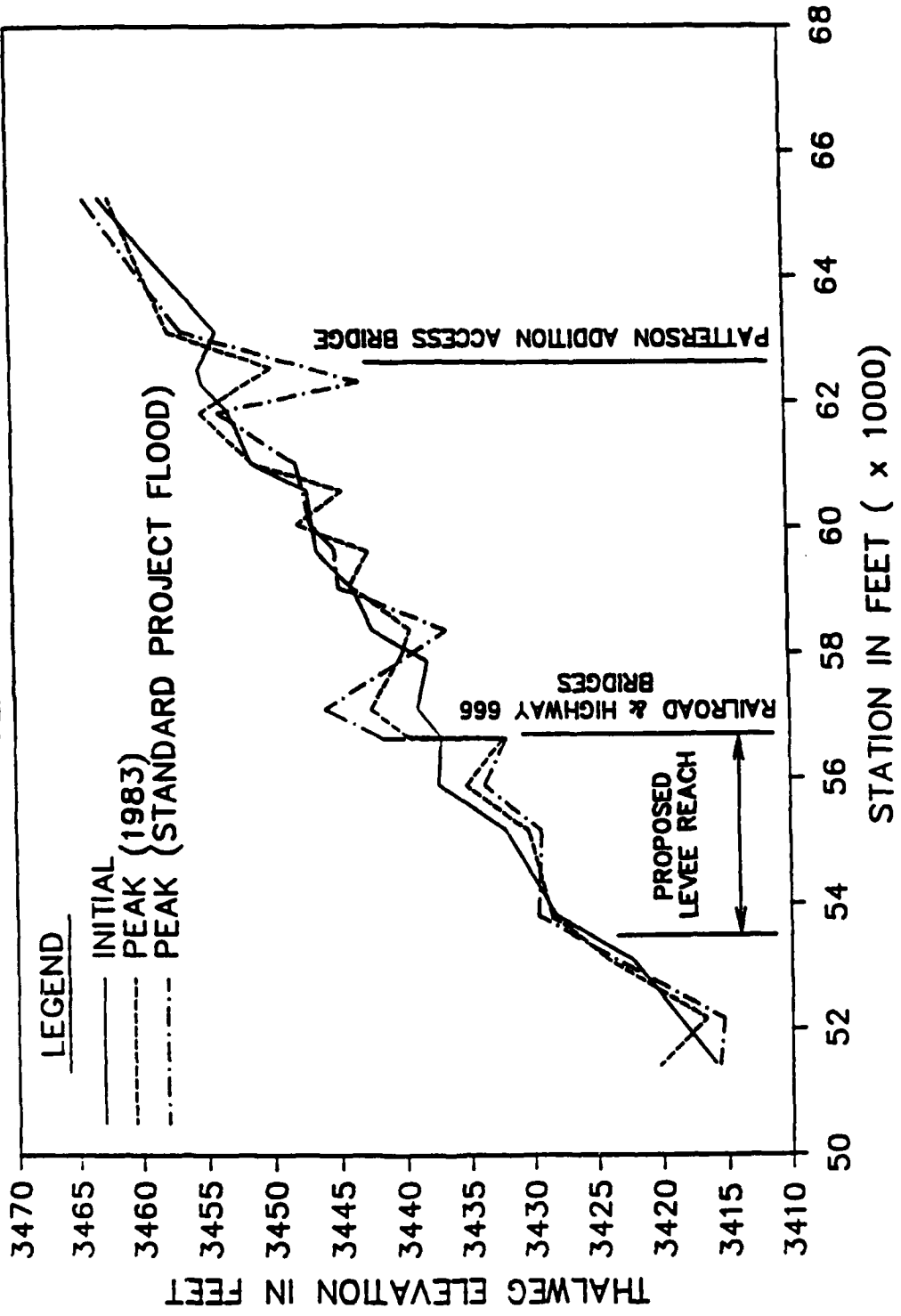
PLATE 15

SAN FRANCISCO RIVER AT CLIFTON, ARIZONA
END OF FLOOD

LEGEND
INITIAL
FINAL (1983)
FINAL (STANDARD PROJECT FLOOD)

PATTERSON ADDITION ACCESS BRIDGE

RAILROAD & HIGHWAY 666
BRIDGES

PROPOSED
LEVEE REACH

STATION IN FEET ( x 1000)

THALWEG ELEVATION IN FEET
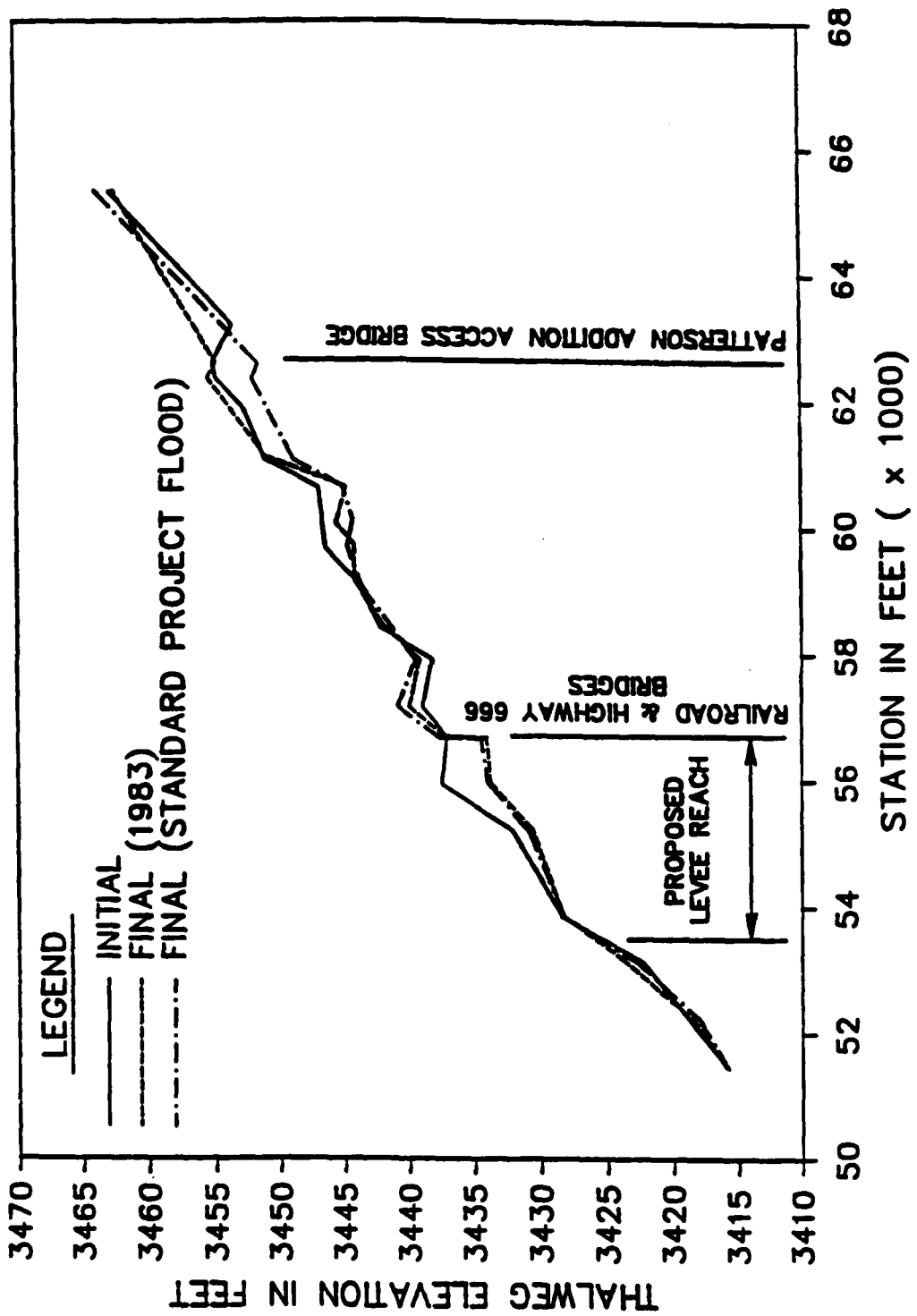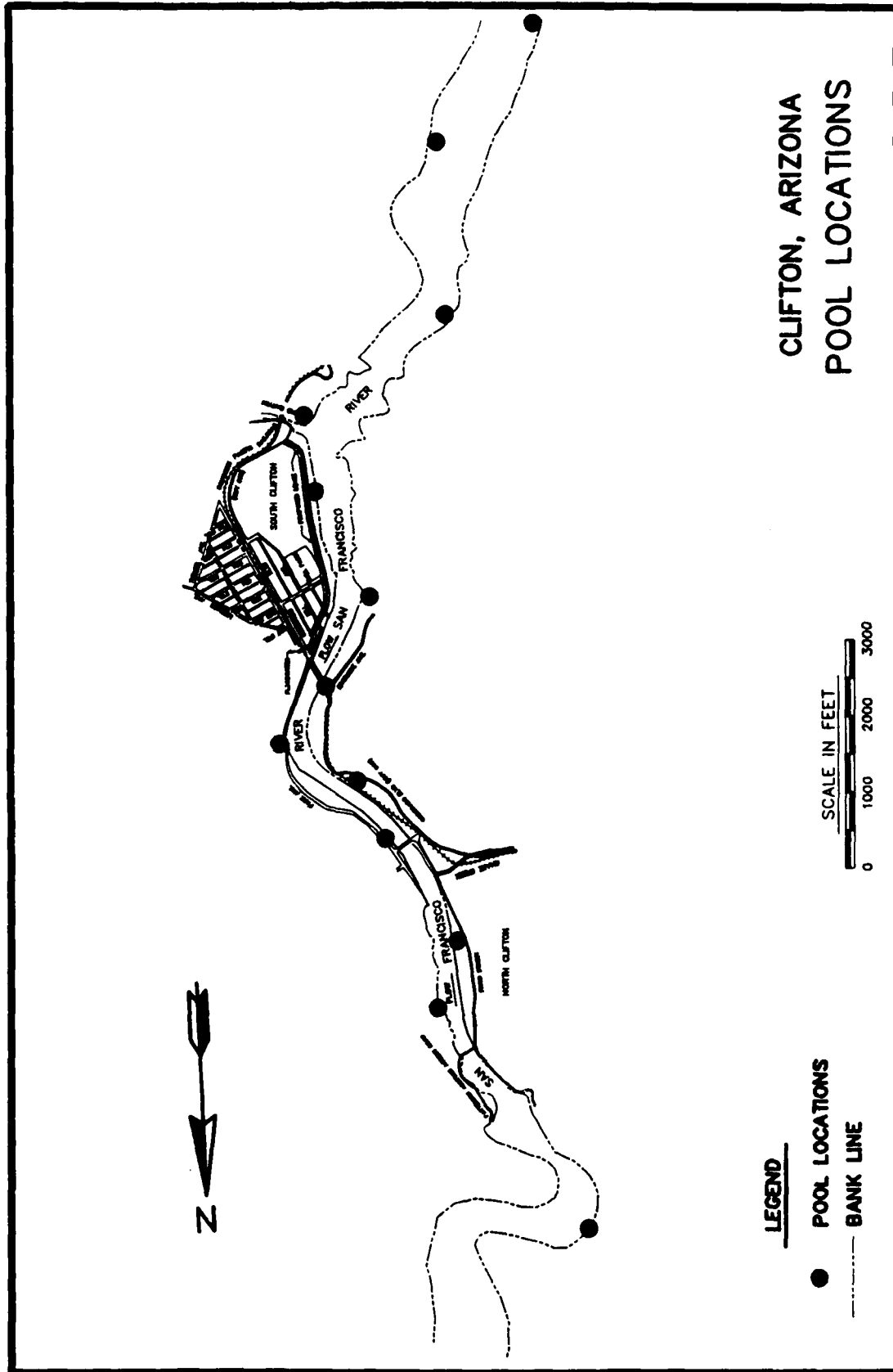
PLATE 16

CLIFTON, ARIZONA
POOL LOCATIONS

SCALE IN FEET

0    1000    2000    3000

LEGEND

●    POOL LOCATIONS
------    BANK LINE

N

PLATE 17

SAN FRANSISCO RIVER
CLIFTON
GREENLEE COUNTY, ARIZONA
HISTORIC STREAMBED PROFILES

STA 522+00 TO 590+00

LEGEND

△ - - △  STREAMBED - 1977 SURVEY (USCE)
◯—◯  STREAMBED - 1939 TOPO (USCE)
✛  SPOT ELEVATIONS

STATIONING IN FEET ABOVE MOUTH

ELEVATION IN FT, NGVD

PLATE 18

SAN FRANSISCO RIVER
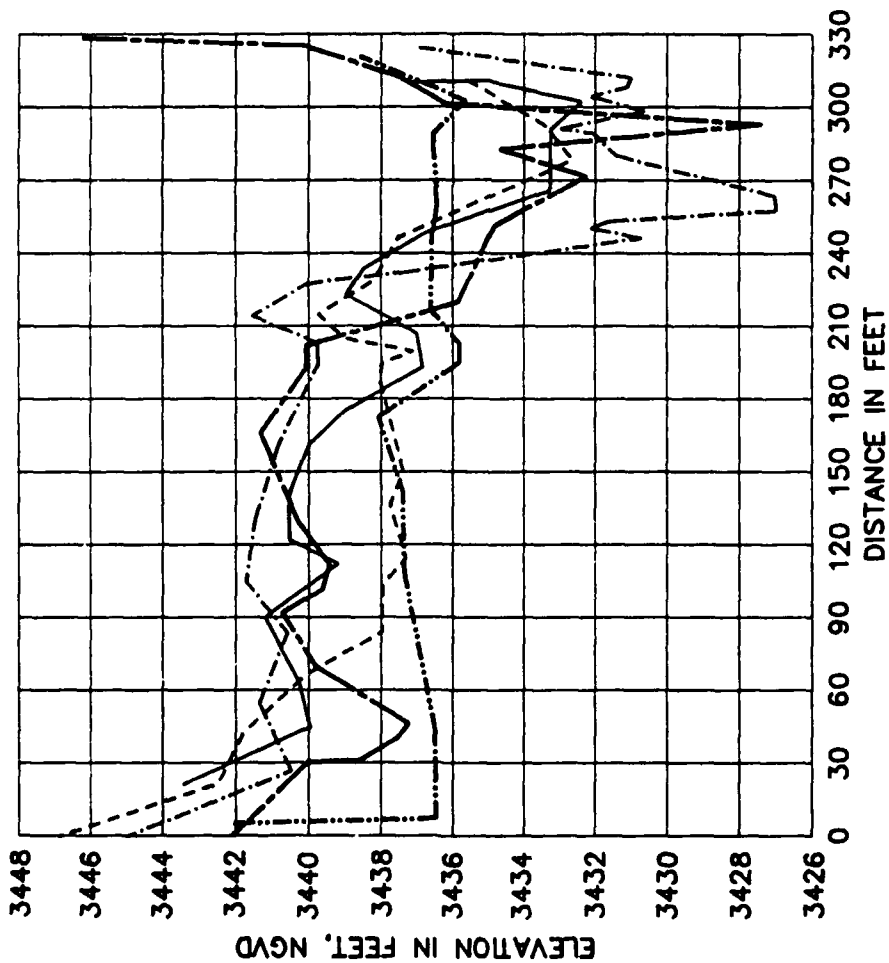CLIFTON
GREENLEE COUNTY, ARIZONA
HISTORIC STREAMBED PROFILES
STA 586+00 TO 654+00
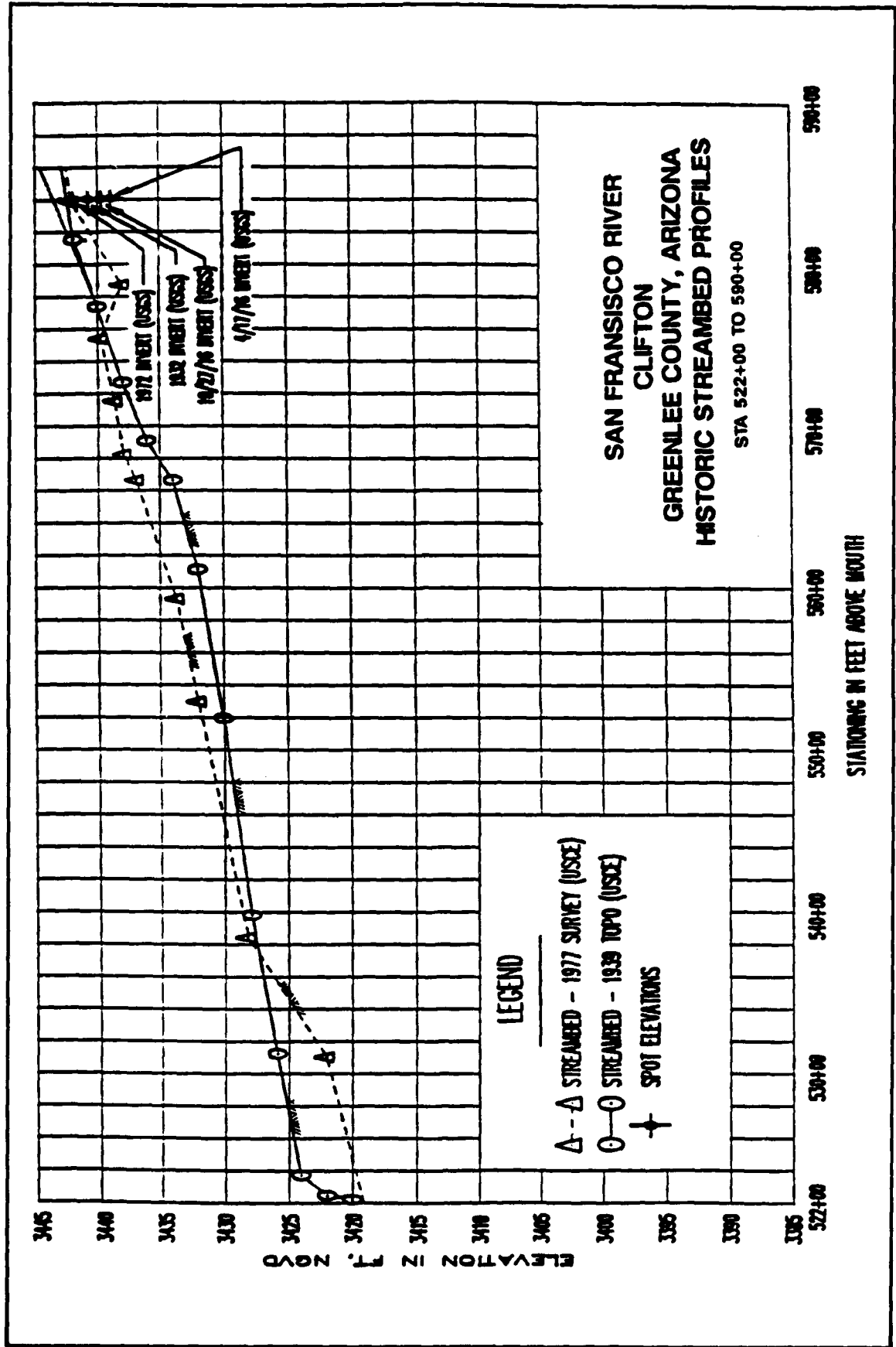
PLATE 19
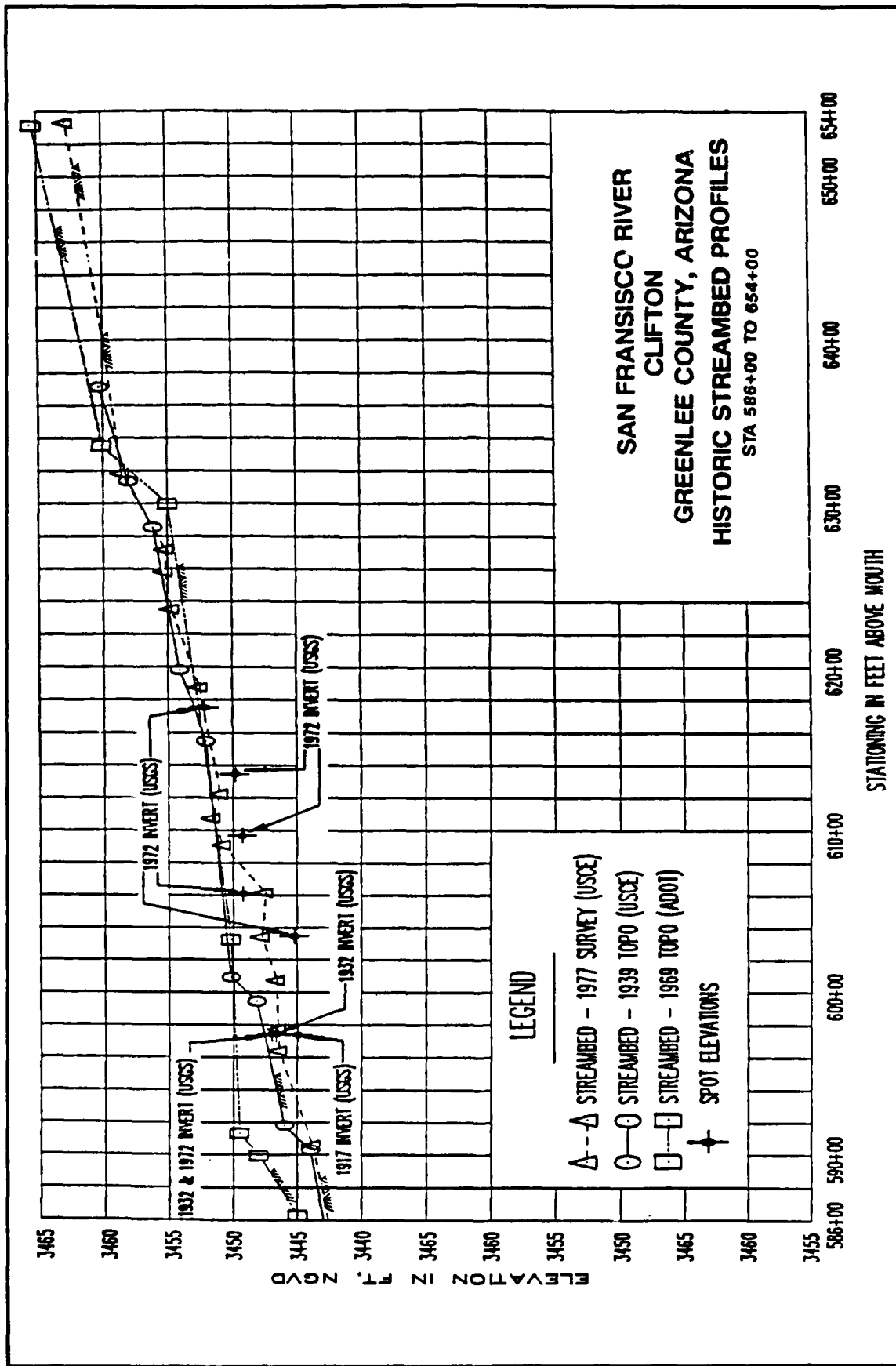
LEGEND

SURVEY OF 8-12-1967
SURVEY OF 12-30-1965
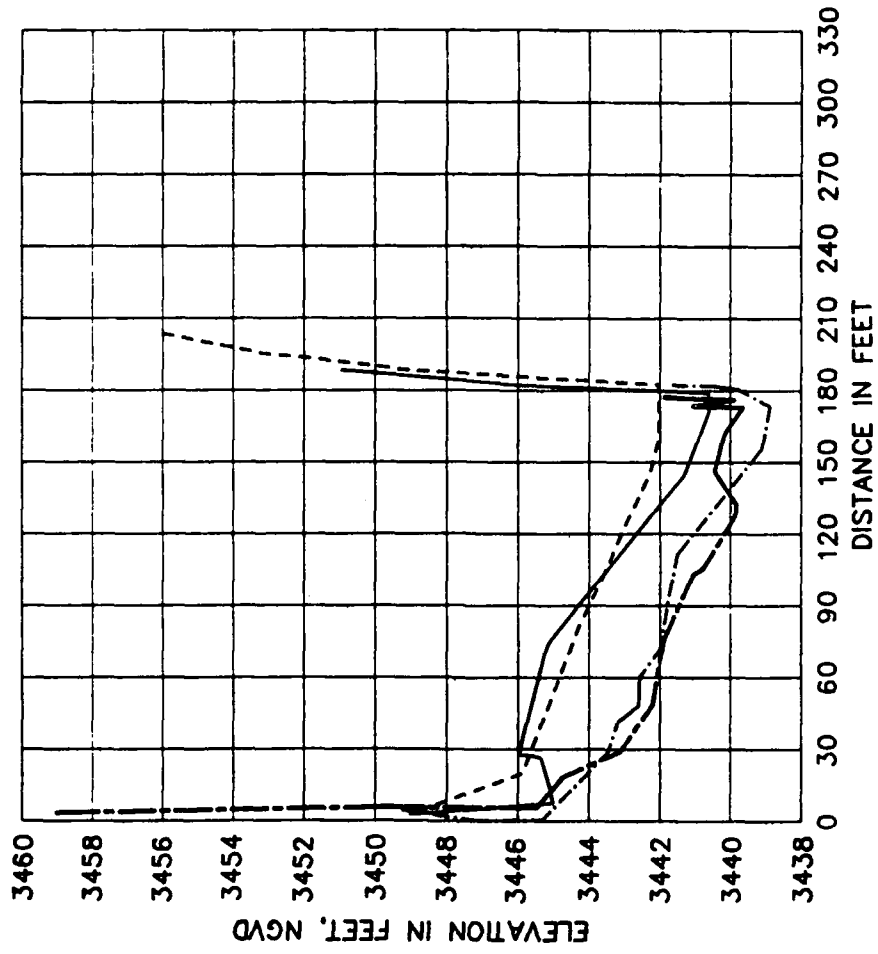SURVEY OF 12-22-1965
SURVEY OF 8-24-1931
SURVEY OF 10-14-1914

SAN FRANCISCO RIVER
CLIFTON
GREENLEE COUNTY, ARIZONA

HISTORICAL CROSS SECTION
STATION 567+70
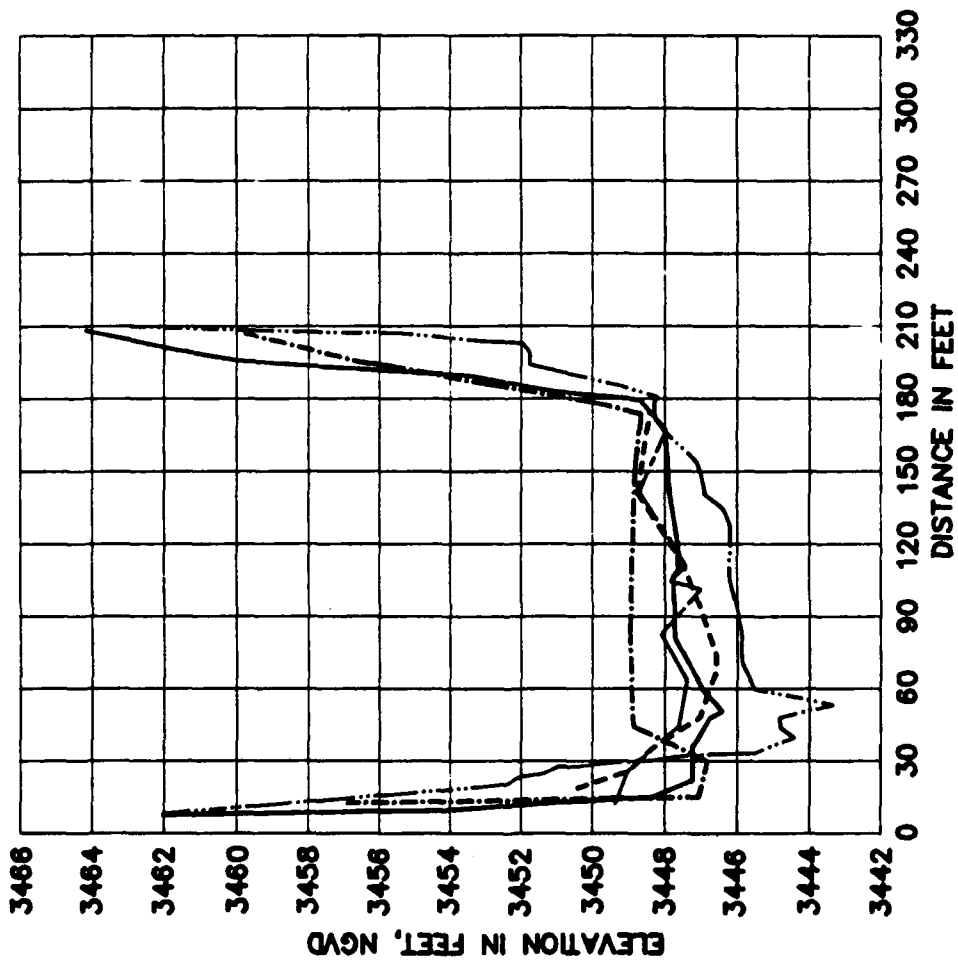
PLATE 20

PLATE 21

LEGEND

SURVEY OF 11-21-1972
SURVEY OF 3-17-1932
SURVEY OF 4-7-1916
SURVEY OF 10-27-1916

SAN FRANCISCO RIVER
CLIFTON
GREENLEE COUNTY, ARIZONA

HISTORICAL CROSS SECTION
STATION 584+10

DISTANCE IN FEET

ELEVATION IN FEET, NGVD

LEGEND

SURVEY OF 11-20-1972
SURVEY OF 10-21-1972
SURVEY OF 3-17-1932
SURVEY OF      1927
SURVEY OF 1-5-1917

SAN FRANCISCO RIVER
CLIFTON
GREENLEE COUNTY, ARIZONA

HISTORICAL CROSS SECTION
STATION 597+40

DISTANCE IN FEET

ELEVATION IN FEET, NGVD

PLATE 22