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ROCK-LINED TRANSITIONS: HYDRAULIC MODEL INVESTIGATION.(U)
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ROCK-LINED TRANSITIONS

Hydraulic Model Investigation

Report No. 1-110

January 1978

U. S. Army Engineer District, Los Angeles
CORPS OF ENGINEERS
Los Angeles, California

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

Model investigations to determine the adequacy of the theoretical design of four rock-lined transitions in providing the desired boundary roughness for reducing velocities from supercritical (rapid) to subcritical (tranquil) are reported herein. Satisfactory design was developed for each project. Dumped-stone requirements in the transition were determined. The investigation of each rock-lined transition was concerned with its flow conditions in the approach channel, the performance of the energy dissipator, the adequacy of the boundary roughness, and the magnitude of scour in the earth channel downstream from the transition structure.

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FOREWORD

The hydraulic studies reported herein were conducted in the Hydraulic Laboratory of the U. S. Army Engineer District, Los Angeles, during the period 1956 to 1962. Preparation and publication of the report were authorized by the Office, Chief of Engineers, in a letter dated 21 August 1969 to the Director, U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Miss.

This report was prepared by Mr. D. A. Barela, Hydraulics Section, Los Angeles District, under the supervision of Mr. A. Robles, Jr., Chief of the Hydrology and Hydraulics Branch. Certain portions of two earlier reports entitled "Channel Transition for East Twin and Warm Creeks" and "Transition for Chino Creek Channel," by the Los Angeles District Office, dated August 1961 and October 1962, respectively, are inclosed as Appendices I and II to this report. The material presented in Appendices I and II is supplementary to the material presented in the main report. COL John V. Foley, CE, was District Engineer during publication of the report.

The report was reviewed and published by WES.

CONTENTS

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	25.4	millimetres
feet	0.3048	metres
miles (U. S. statute)	1.609344	kilometres
feet per second	0.3048	metres per second
cubic feet per second	0.02831685	cubic metres per second
pounds (mass)	0.4535924	kilograms

SUMMARY

Model investigations to determine the adequacy of the theoretical design of four rock-lined transitions in providing the desired boundary roughness for reducing velocities from supercritical (rapid) to subcritical (tranquil) are reported herein. Satisfactory design was developed for each project. Dumped-stone requirements in the transition were determined.

The investigation of each rock-lined transition was concerned with its flow conditions in the approach channel, the performance of the energy dissipator, the adequacy of the boundary roughness, and the magnitude of scour in the earth channel downstream from the transition structure.

ROCK-LINED TRANSITIONS

Hydraulic Model Investigation

INTRODUCTION

1. The problem involved in the design of rock-lined transitions is to effect low velocities and prevent scouring by developing a design that provides reasonably uniform and nonscour flow velocities along the bottom and side slopes of the downstream channel.

2. This report presents the results of four laboratory tests conducted with hydraulic models of rock-lined energy-dissipating transitions. The model studies were conducted to verify the effectiveness of each proposed design and to develop any necessary modifications to effect acceptable flow conditions. Each model reproduced a portion of the approach channel, the rock-lined transition, and a significant reach of natural wash downstream from the transition structure.

3. A discussion of the reach of the prototype structure as constructed in the model, followed by a description of the model, the sequence of tests, and the test results of the four hydraulic models are presented in this report.

PURPOSE OF STUDY

4. The purpose of the model studies was to observe and analyze the performance of four rock-lined transitions in order to determine their proper design. The main area and elements of concern which had to be investigated by the models were the flow in the simulated concrete reach, the flow pattern in the transition, the effectiveness of the boundary roughness of the transition lining in reducing velocities from supercritical to subcritical, the outflow condition in the earth channel, and the scour that would result to the dumped stone and earth channel downstream from the transition structure.

MODEL APPURTENANCES

5. Water used in the operation of the models was pumped from a sump through a supply line equipped with a venturi meter to measure flow rate. The flow from the supply line discharged into a forebay where it was stilled by baffles prior to entering the model. After passing through the model, the water returned by gravity flow to the sump for recirculating. A tailgate installed at the downstream end of each model was used to produce the required tailwater elevation. Wooden rails set to grade alongside the model provided a datum plane for measuring devices. Water-surface elevations were measured with a point gage and velocities were measured with a Prandtl pitot tube. Selected flow conditions were recorded photographically.

6. Before each run, water was slowly pumped into the area below the end sill to a level equal to the natural tailwater elevation for that run. This water cushion prevented disproportionate scour at the beginning of each run.

SCALE RELATIONS

7. Hydraulic similitude based upon the Froudian relations were used to express mathematical relations between model and prototype. Model data have been transferred to prototype equivalences or vice versa by the scale relations shown in Table 1.

8. Discharge, depth of flow, and velocity in the models can be transferred quantitatively to prototype equivalents with the scale relations. Scour in the models was analyzed qualitatively since it has not yet been found practicable to reproduce quantitatively in the model the resistance to erosion of a prototype bed material. The observed scour data served as a guide for determining the reach of earth channel that would require riprap protection.

Table 1

Scale Relations

		Rock-Lined Transitions			
Dimension	Ratio	Bautista Creek Outlet	Devil Creek Diversion	Tucson Diversion Channel	Trilby Wash Basin and Outlet Channel
Length	L_r	1:48	1:25	1:40	1:18
Area	$A_r = L_r^2$	1:2,304	1:625	1:1,600	1:324
Velocity	$V_r = L_r^{1/2}$	1:6.93	1:5	1:6.324	1:4.243
Discharge	$Q_r = L_r^{5/2}$	1:15,963	1:3,125	1:10,119	1:1,375
Time	$T_r = L_r^{1/2}$	1:6.93	1:5	1:6.324	1:4.243
Roughness	$N_r = L_r^{1/6}$	1:1.91	1:1.71	1:1.85	1:1.62
Volume	$V_r = L_r^3$	1:110,592	1:15,625	1:64,000	1:5,832

BAUTISTA CREEK OUTLET CHANNEL

The Prototype

9. The Bautista Creek levees are part of a unit, comprising the San Jacinto River and Bautista Creek levee project, under the general comprehensive plan for flood control in the Santa Ana River Basin, Calif.

10. The Bautista Creek Channel outlet, reproduced in the model, is located in Riverside County near the community of Valle Vista in an unincorporated area 5.3 miles* east of Hemet and 85 miles southwest of the city of Los Angeles (Plate 1). The outlet channel, part of the overall flood-control system for the area in Riverside County, will dissipate some of the energy before it discharges into the existing natural channel. The reach of channel upstream from the transition is an open trapezoidal reinforced concrete channel that extends 103 ft downstream from the Florida Avenue Bridge (Hwy 74), sta 99+40. The channel has a base width of 25 ft, side slopes of 1V on 2.25H, and an invert slope of 0.012195.

11. The trapezoidal grouted-stone channel outlet extends from sta 99+40, the downstream end of the concrete-lined channel, to sta 96+90. The channel diverges from a base width of 25 ft at sta 99+40 to a base width of 87.5 ft at sta 96+90. The invert and 1V-on-2.25H side slopes are revetted with 1000-lb grouted derrick stone. The trapezoidal ungrouted stone channel extends from sta 96+90 to sta 94+40, where the invert width is 187.5 ft. The invert and 1V-on-2.25H side slopes are revetted with 3000-lb ungrouted derrick stone. The invert slope of the overall transition is 0.000500.

The Model

12. The model, constructed to an undistorted scale ratio of 1:48,

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page vii.

reproduced about 660 ft of approach channel and the 600-ft-long transition. A sand bed downstream of the transition simulated 800 ft of the prototype area of the San Jacinto River.

13. The material used in constructing the model consisted of wood, sand, and stone. The approach channel upstream from the transition was constructed of timber and plywood. The joists that supported the deck were attached to stringers by means of adjustable bolts. This facility enabled the slope to be varied readily. The remainder of the model was sand and stone construction.

14. To simulate the roughness coefficients of the prototype, certain adjustments were made in the model. Upstream from the transition, supplementary slope was added. In the grouted stone portion of the transition, the boundary roughness was obtained by cementing crushed stone passing and retained on 1/4-in. and No. 4 sieves, respectively, to the plywood decks and sidewalls. The ungrouted dumped-stone portion of the transition was simulated by use of 3/4-in. crushed stone having the same specific weight as the stone to be used in the prototype. Downstream from the transition, sand having a mean diameter of 0.20 mm was used to simulate the riverbed.

Original Design

15. In the original design, the flare of the base width was 1 ft in 20 ft for each side of the grouted reach of the transition and 1 ft in 10 ft for each side of the ungrouted reach. The ungrouted stone in this design had a maximum size of 3000 lb. The base width at the downstream end of the grouted reach was 50 ft, and the base width at the downstream end of the transition or ungrouted reach was 120 ft. The overall length of transition was 600 ft. The layout of the original design is shown in Photo 1. The design proved to be unsatisfactory for this transition. Erosion in the ungrouted section began to occur immediately for the lower discharges. For the design discharge of 16,500 cfs, the hydraulic jump began to form near the downstream end of the transition, sta 93+50. Flow observations indicated that the energy from the

flow was not diminished sufficiently while passing through the transition. The test indicated that the base width in the transition did not flare out sufficiently to dissipate the energy before it reached the natural wash. The water profile for the original design is shown in Plate 2. Photographs of the flow conditions for this design were not available. Excessive scour occurred adjacent to and downstream of the transition (Photo 2). A hydrograph run produced the scour pattern shown in Plate 3. The hydrograph had a peak discharge of 16,500 cfs as shown in Plate 4.

Alternative Design

16. The results of the original design indicated that greater energy dissipation through the transition must be obtained. Tests were conducted using different flares for the base width of the transition. In this design, the length of the transition was the same as that of the original design. The base width at the downstream end of the transition was 128.33 ft. Photo 3 shows the model before the hydrograph run. With this design, a discharge of 8000 cfs was reached before erosion of the ungrouted stone invert near the end of the transition began to occur. Photo 4 shows the flow conditions for various discharges of the hydrograph. The flow at the downstream end of the transition was more evenly distributed. The velocities were not reduced sufficiently to eliminate erosion at the downstream end of the transition. Photo 5 shows the scour due to the hydrograph run. The water-surface profile in Plate 5 indicates that there is sufficient wall height throughout the entire reach of the channel.

Final Design

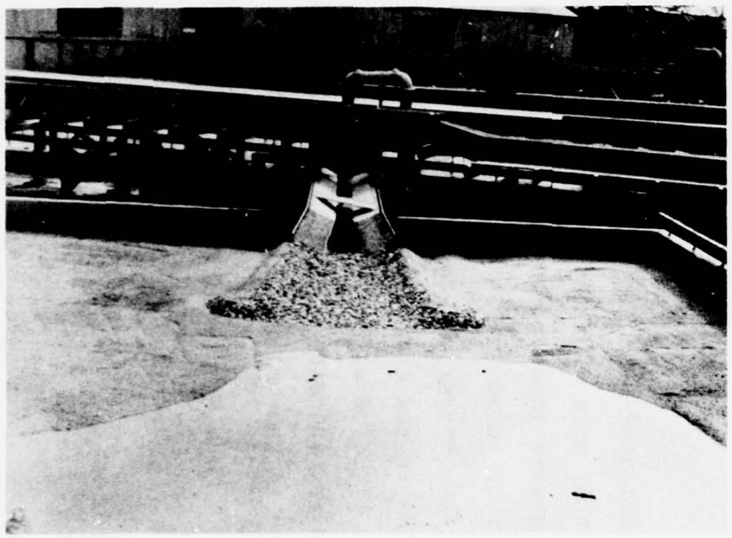
17. On the basis of the previous test, the flare was again increased. This design, together with the 500-lb (maximum) stone used in the transition, provided a maximum reduction in flow velocities for a minimum length of transition. The transition was shortened 100 ft. The

ungrouted stone portion of the transition would extend from sta 96+90 where the invert width is 87.5 ft, to sta 94+90, where the invert width is 187.5 ft (Photo 6).

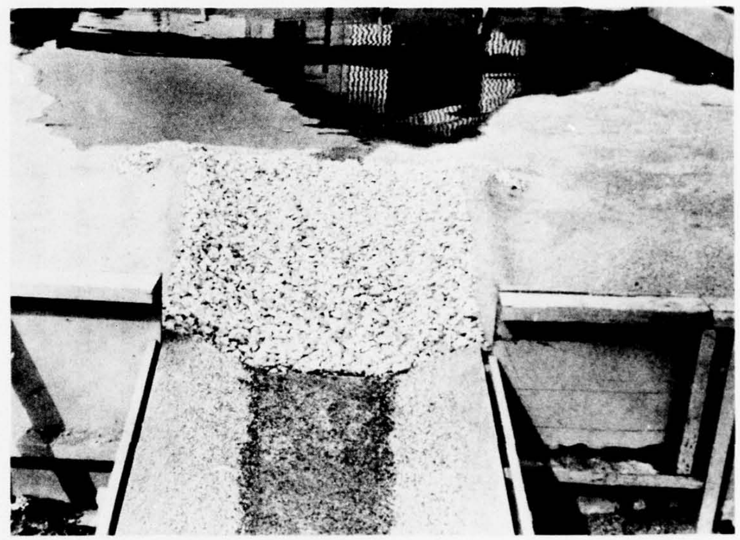
18. The channel outlet incorporating the most desirable features investigated during the course of the model tests together with the average water surface is shown in Plate 6. This design was recommended for the prototype. Flood conditions were excellent for all discharges. Surface flow patterns for some of the discharges of the hydrograph are shown in Photos 7 and 8. The maximum discharge condition is shown in each photograph. Results of scour after the hydrograph run are shown in Photo 9. Comparison of this scour with that obtained downstream from the transition of the original design indicates considerable improvement. The effectiveness of this design can be evaluated from the scour pattern shown in Plate 3 which indicates that the scour is considerably less than the scour which occurred in the original design. The symmetry of the scour pattern would indicate a more uniform flow distribution.

Discussion

19. The tests indicated that flow conditions with the final design were better than they were with the original and alternative designs. Flow conditions were improved by shortening the transition and widening the base width as shown in Plate 6. The final design dissipated most of the excessive flow energy before it discharged into the natural wash. The velocity flow in the channel outlet was reduced from 39.0 fps at the upstream end to 17.0 fps at the downstream end. The use of larger stone in the invert and side slopes in the final design reduced the scour that occurred in the original design.

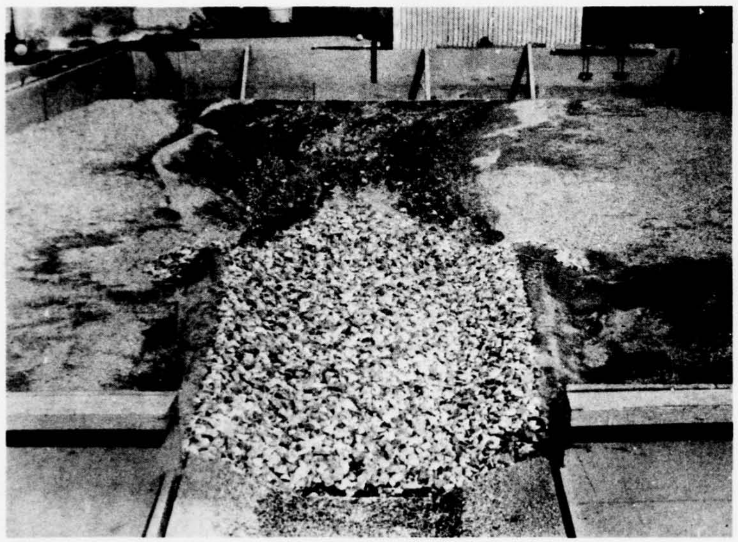


Looking upstream



Looking from right side

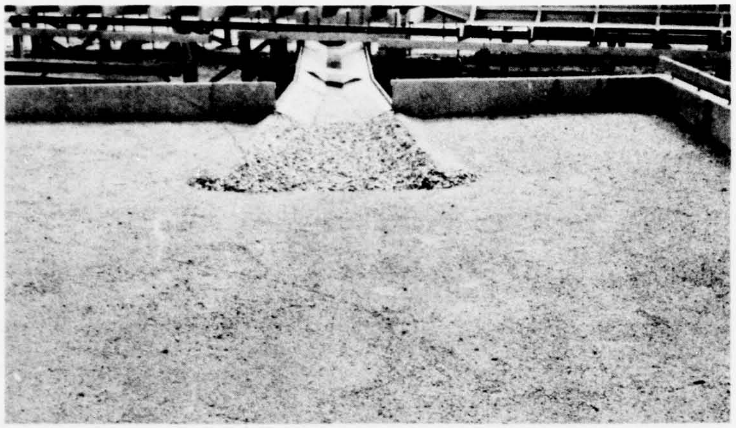
Photo 6. Final design, Bautista Creek Outlet



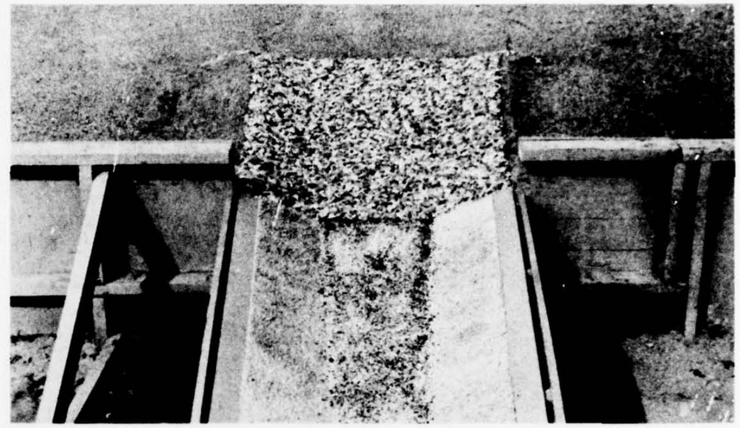
Looking downstream

Looking upstream

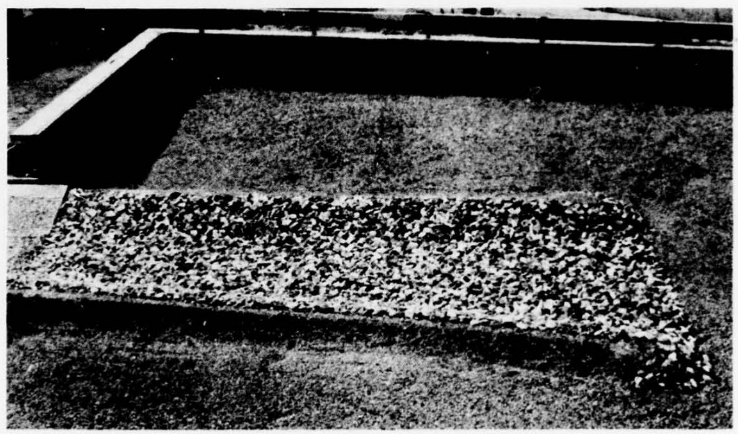
Photo 2. Original design, scour due to hydrograph run



Looking upstream

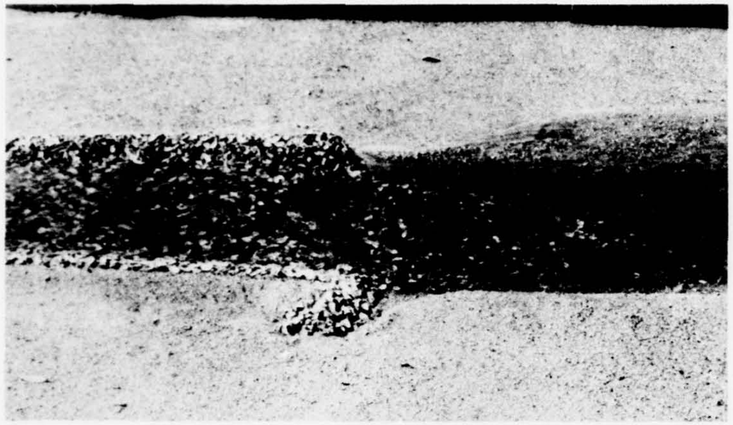


Looking downstream

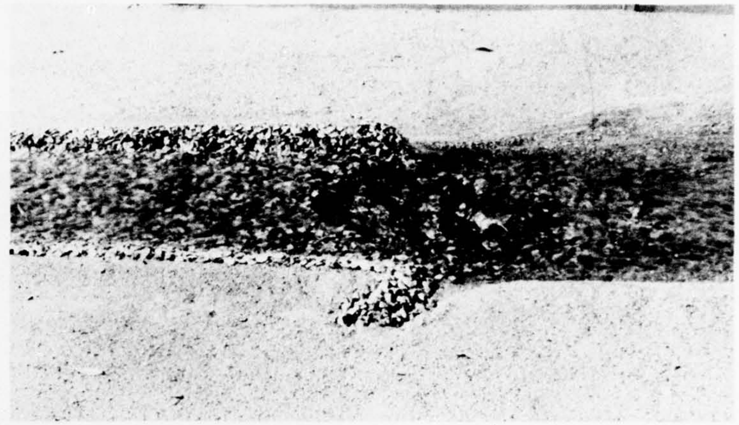


Looking from right side

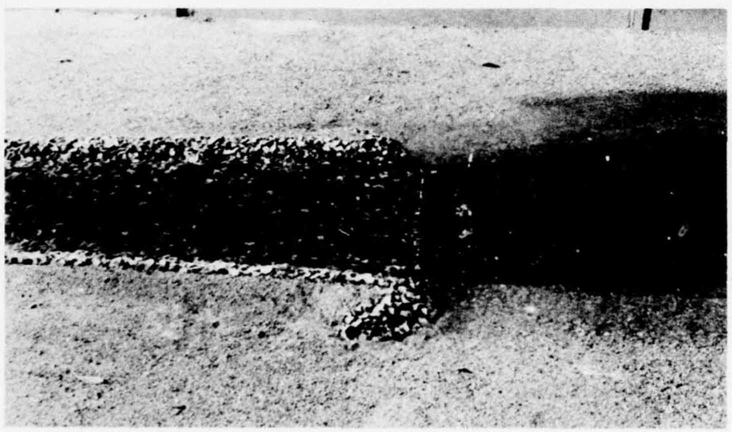
Photo 3. Alternative design, Bautista Creek Outlet



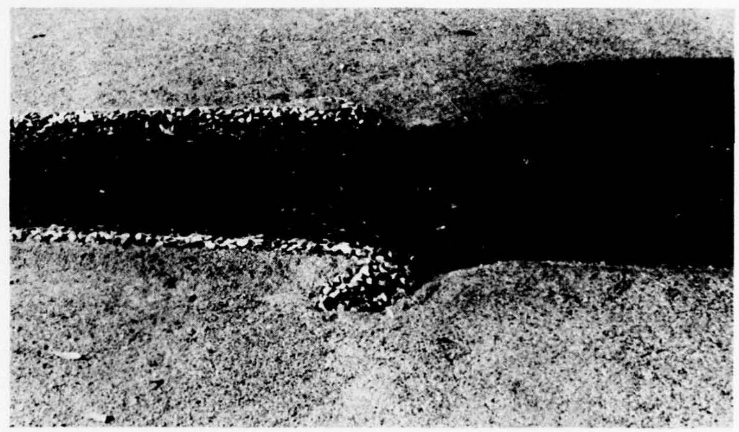
Discharge 4848 cfs



Discharge 6960 cfs

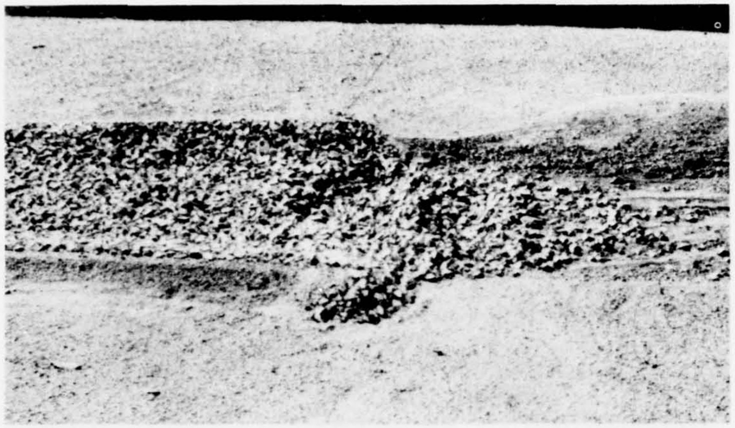


Discharge 9235 cfs



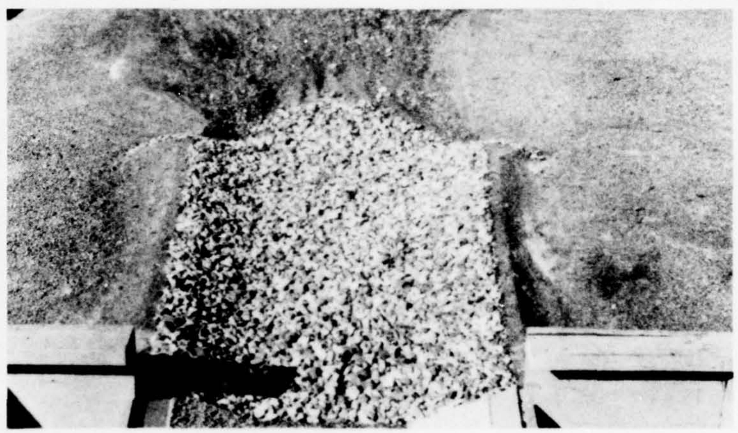
Discharge 16,500 cfs

Photo 4. Alternative design flow conditions, looking from right side



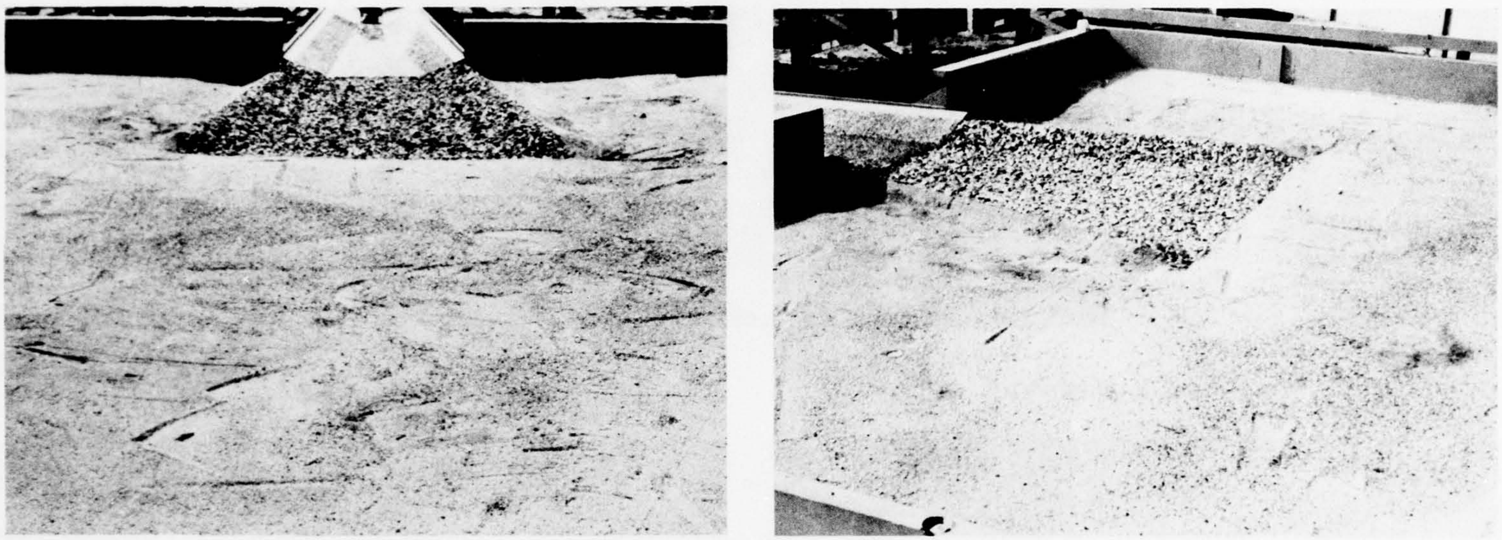
Looking from right side

Looking upstream



Looking downstream

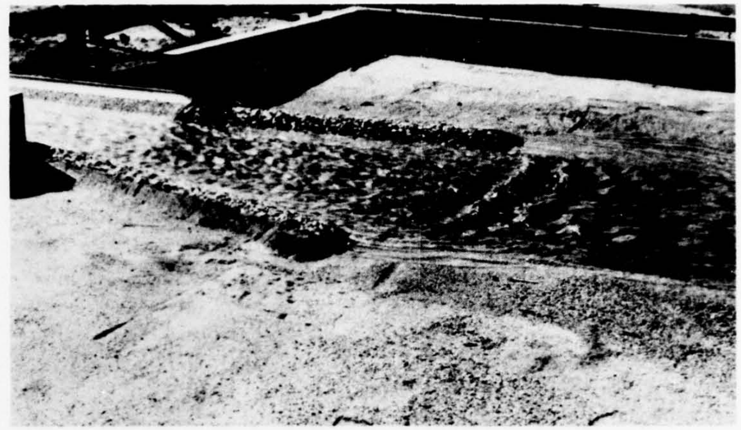
Photo 5. Alternative design, scour due to hydrograph run



Looking upstream   Looking downstream

Photo 1. Original design, Bautista Creek Outlet

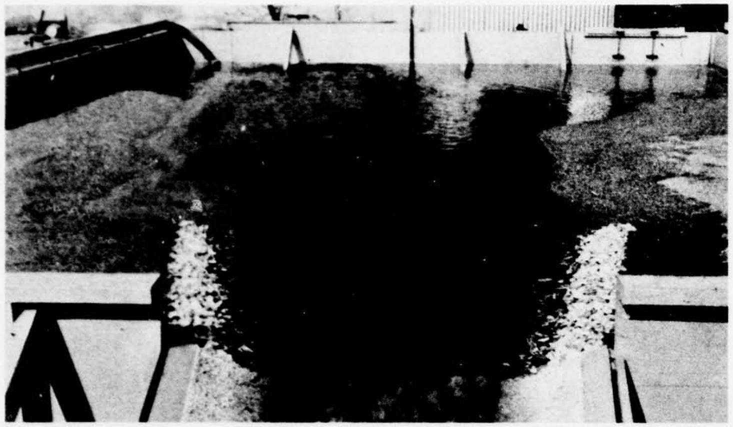
Discharge 9235 cfs



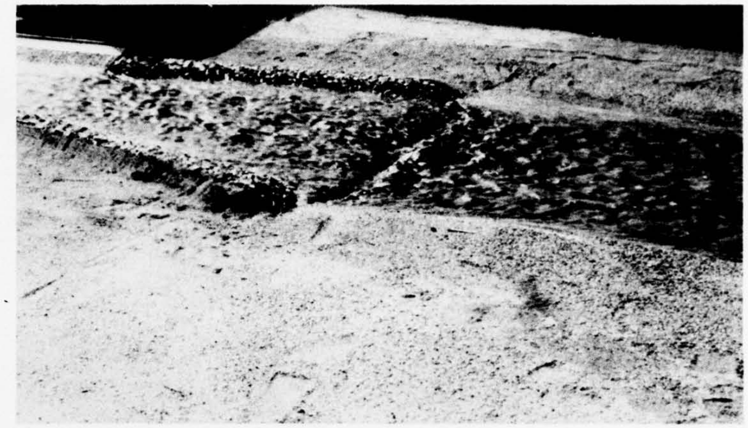
Discharge 13,070 cfs

Discharge 16,500 cfs

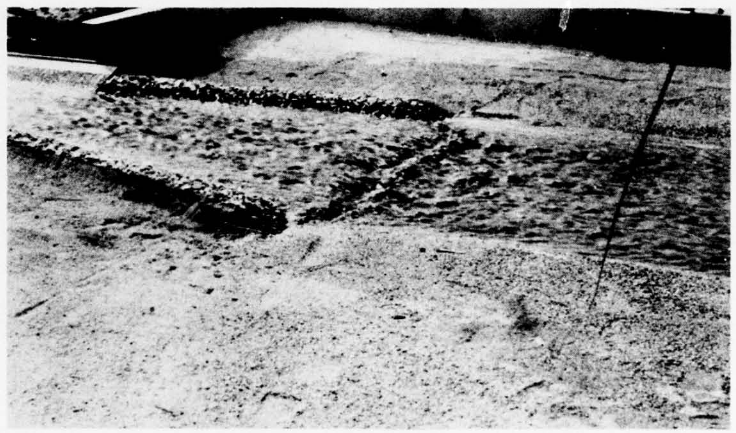
Photo 7. Final design flow conditions, looking from right side



Looking downstream, discharge 16,500 cfs



Looking from right side, discharge 12,440 cfs



Looking from right side, discharge 9190 cfs

Photo 8. Final design flow conditions, Bautista Creek Outlet

Looking from right side

Looking upstream

Photo 9. Final design, scour due to hydrograph run

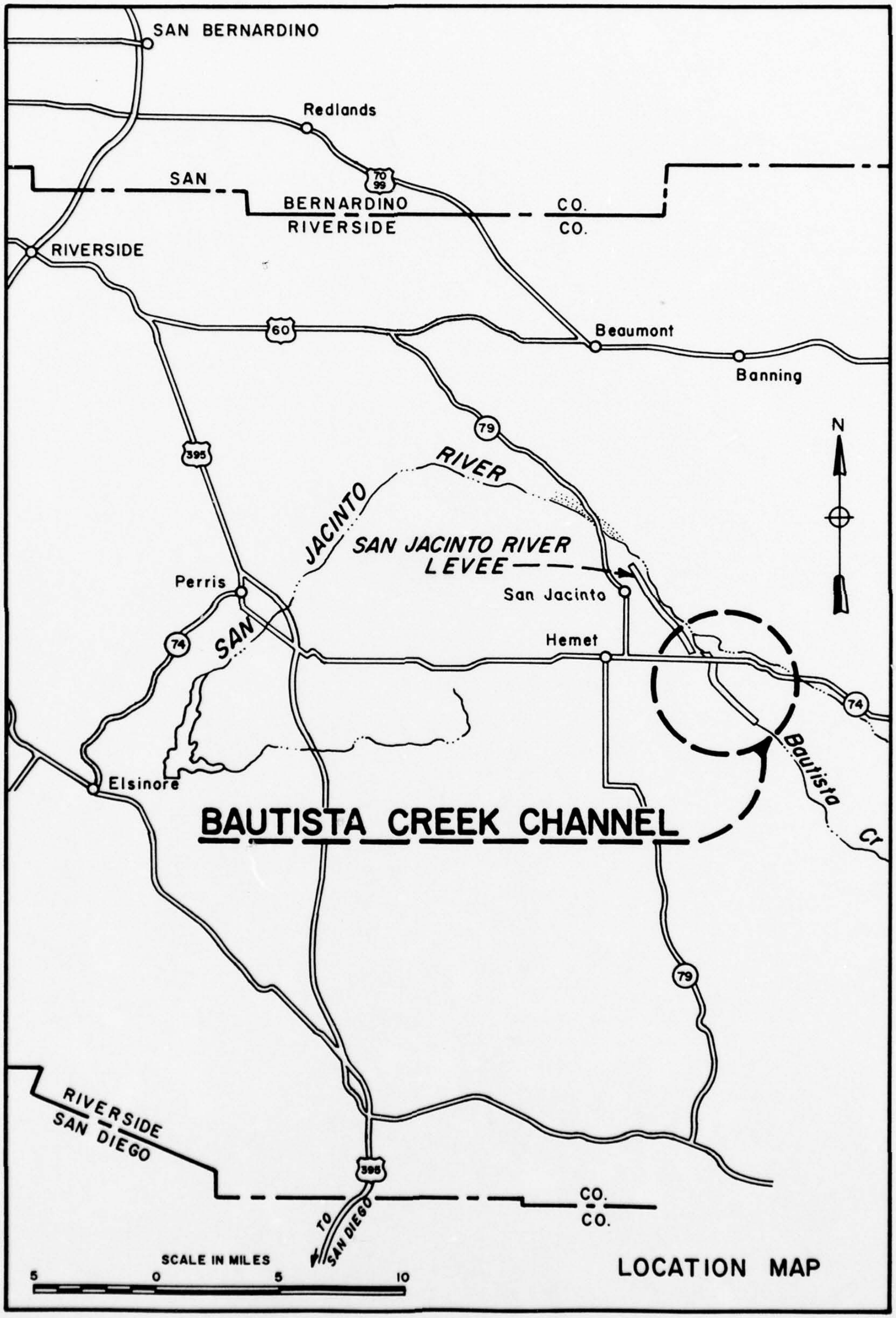
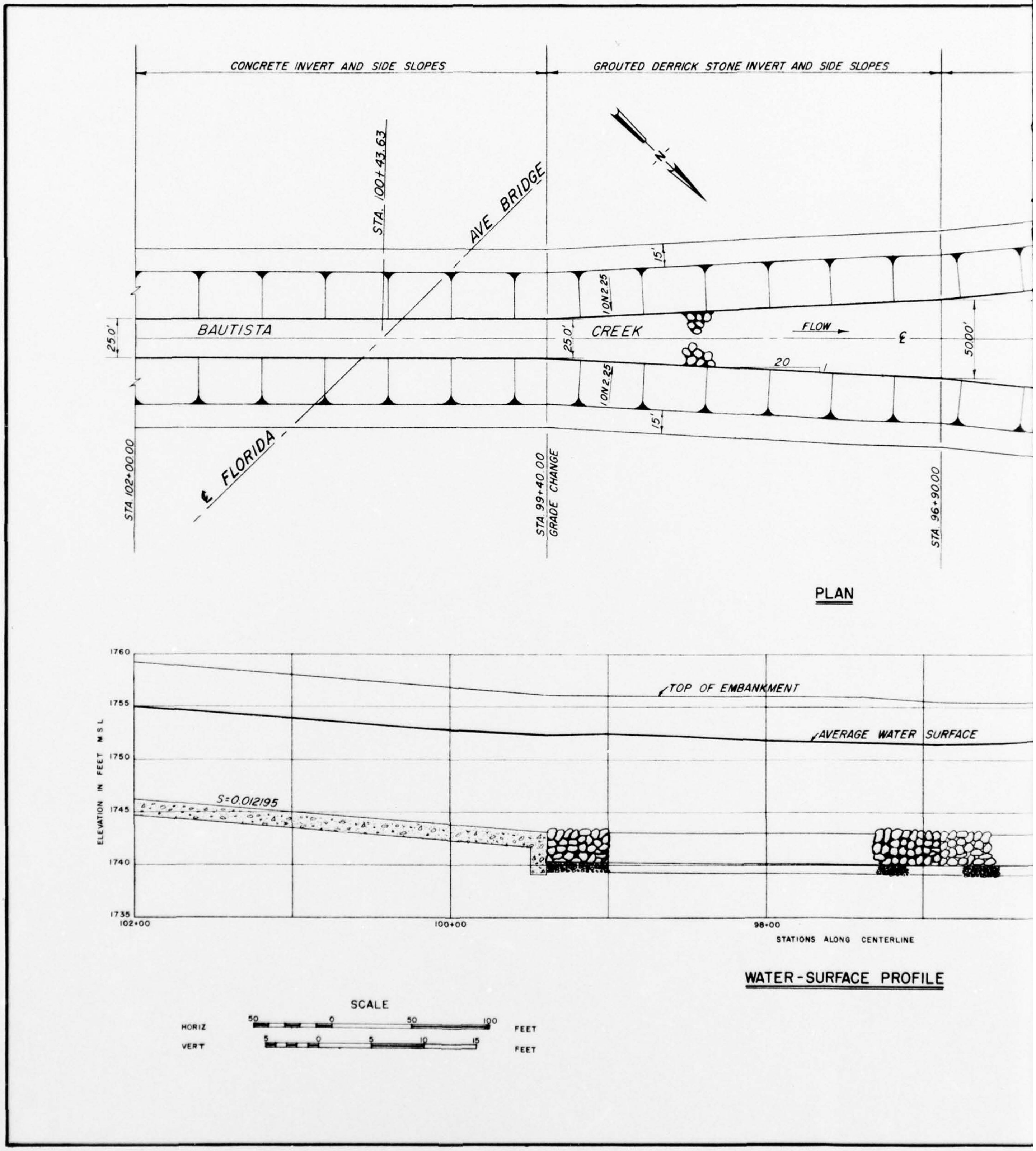
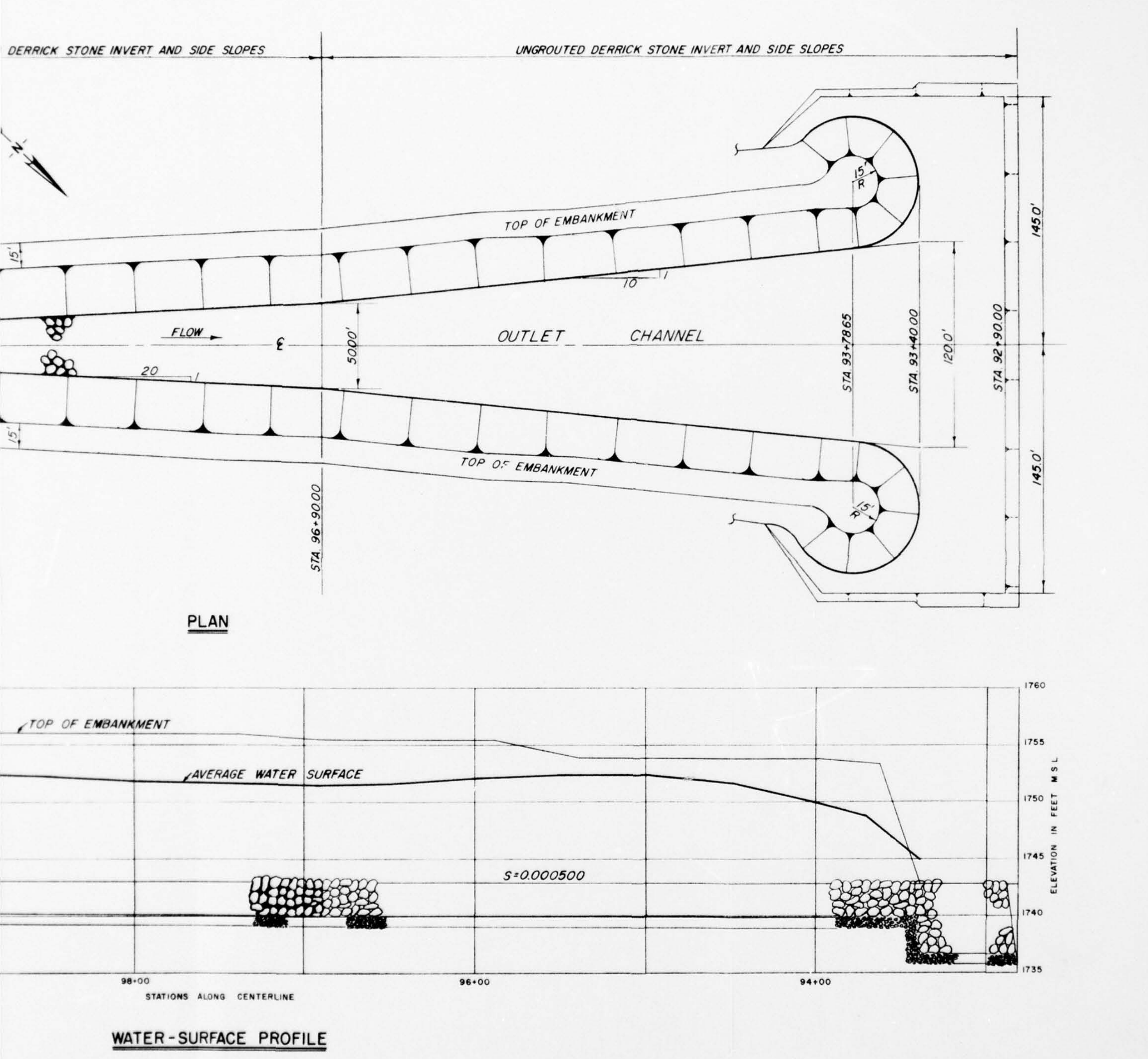

SAN BERNARDINO
Redlands
70
99
SAN
BERNARDINO
RIVERSIDE
CO.
CO.
RIVERSIDE
60
Beaumont
Banning
79
395
N
RIVER
JACINTO
SAN JACINTO RIVER
LEVEE
SAN
Perris
San Jacinto
Hemet
74
74
Bautista
Cr
Elsinore
BAUTISTA CREEK CHANNEL
79
RIVERSIDE
SAN DIEGO
395
CO.
CO.
TO SAN DIEGO
SCALE IN MILES
5
0
5
10
LOCATION MAP

PLATE 1



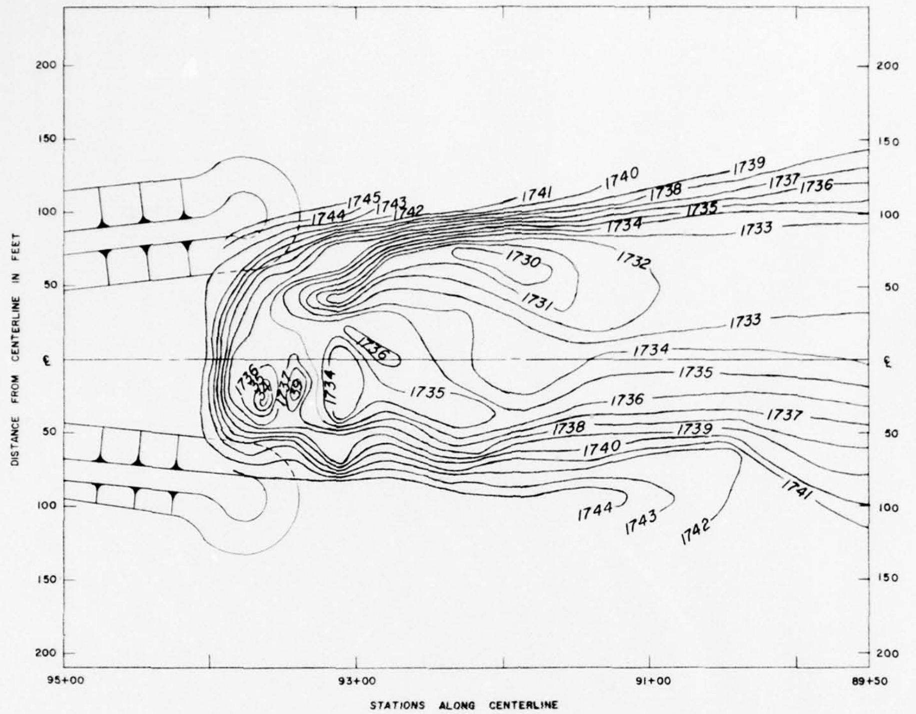
CONCRETE INVERT AND SIDE SLOPES
GROUTED DERRICK STONE INVERT AND SIDE SLOPES
STA. 100+43.63
AVE BRIDGE
℄ FLORIDA
N
15'
1 ON 2.25
BAUTISTA
CREEK
FLOW
℄
25.0'
50.00'
20
1
STA 102+00.00
STA 99+40.00
GRADE CHANGE
STA 96+90.00
PLAN
TOP OF EMBANKMENT
AVERAGE WATER SURFACE
S=0.012195
ELEVATION IN FEET MSL
1760
1755
1750
1745
1740
1735
102+00
100+00
98+00
STATIONS ALONG CENTERLINE
WATER-SURFACE PROFILE
SCALE
HORIZ
VERT
FEET

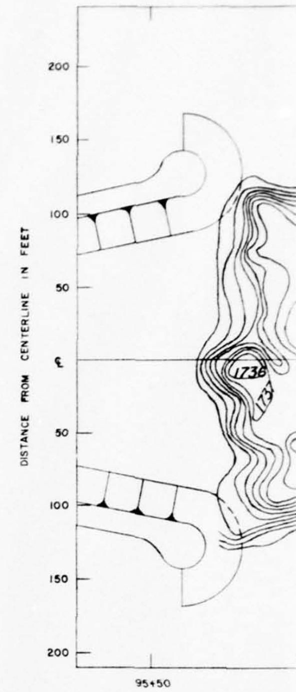


PLAN AND PROFILE
ORIGINAL DESIGN
DISCHARGE 16,500 CFS

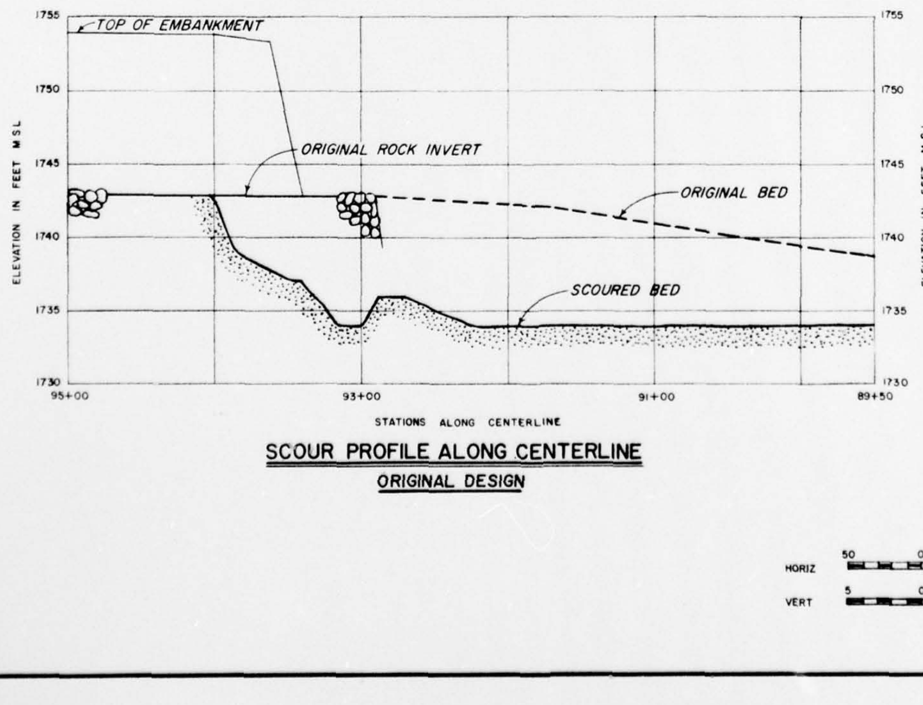
PLATE 2

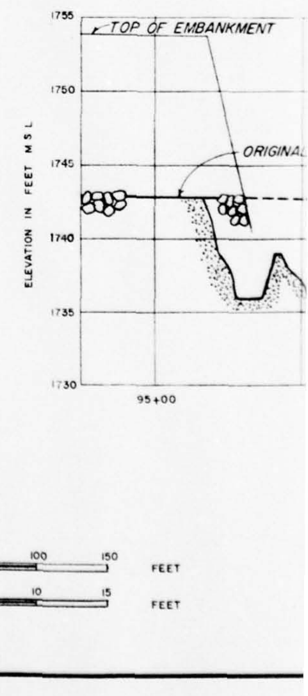
2





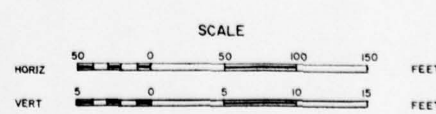
SCOUR PATTERN

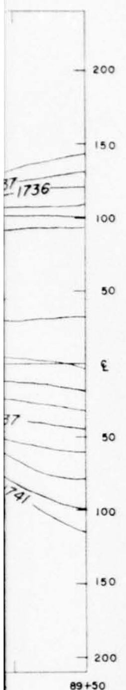


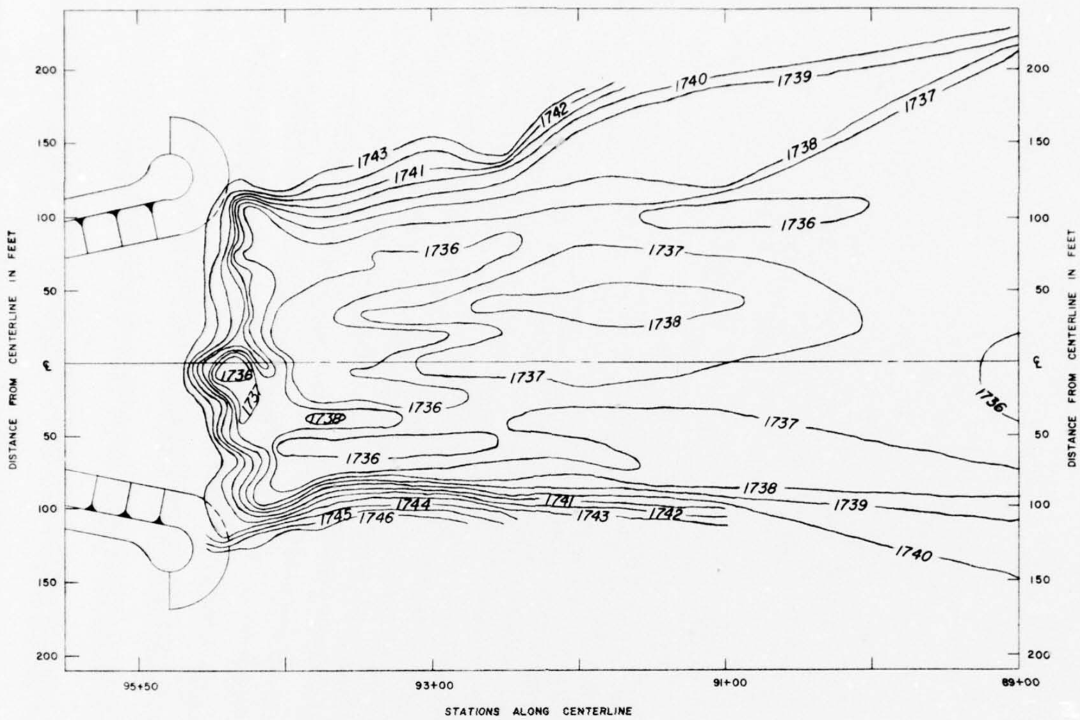


SCOUR PROFILE ALONG CENTERLINE

ORIGINAL DESIGN







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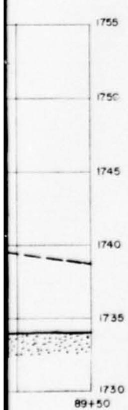
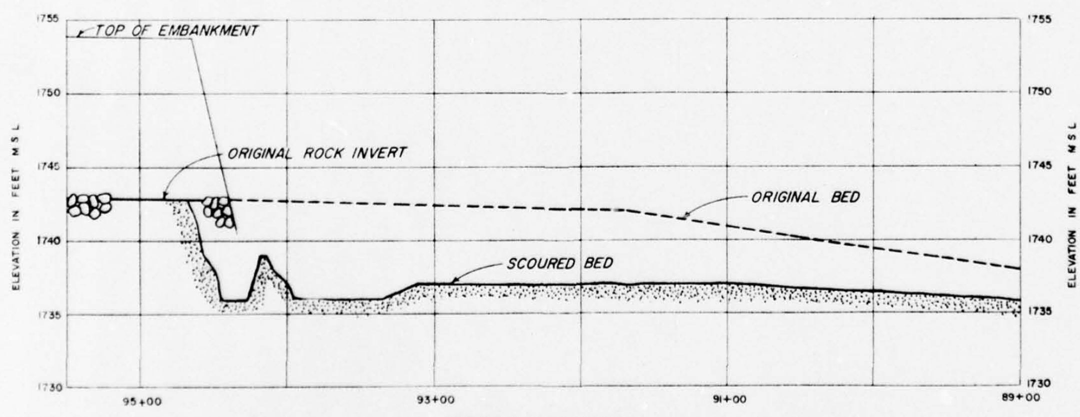




SCOUR PROFILE ALONG CENTERLINE

FINAL DESIGN

SCALE

ORIZ 50 0 50 100 150 FEET

ERT 5 0 5 10 15 FEET

BAUTISTA CREEK

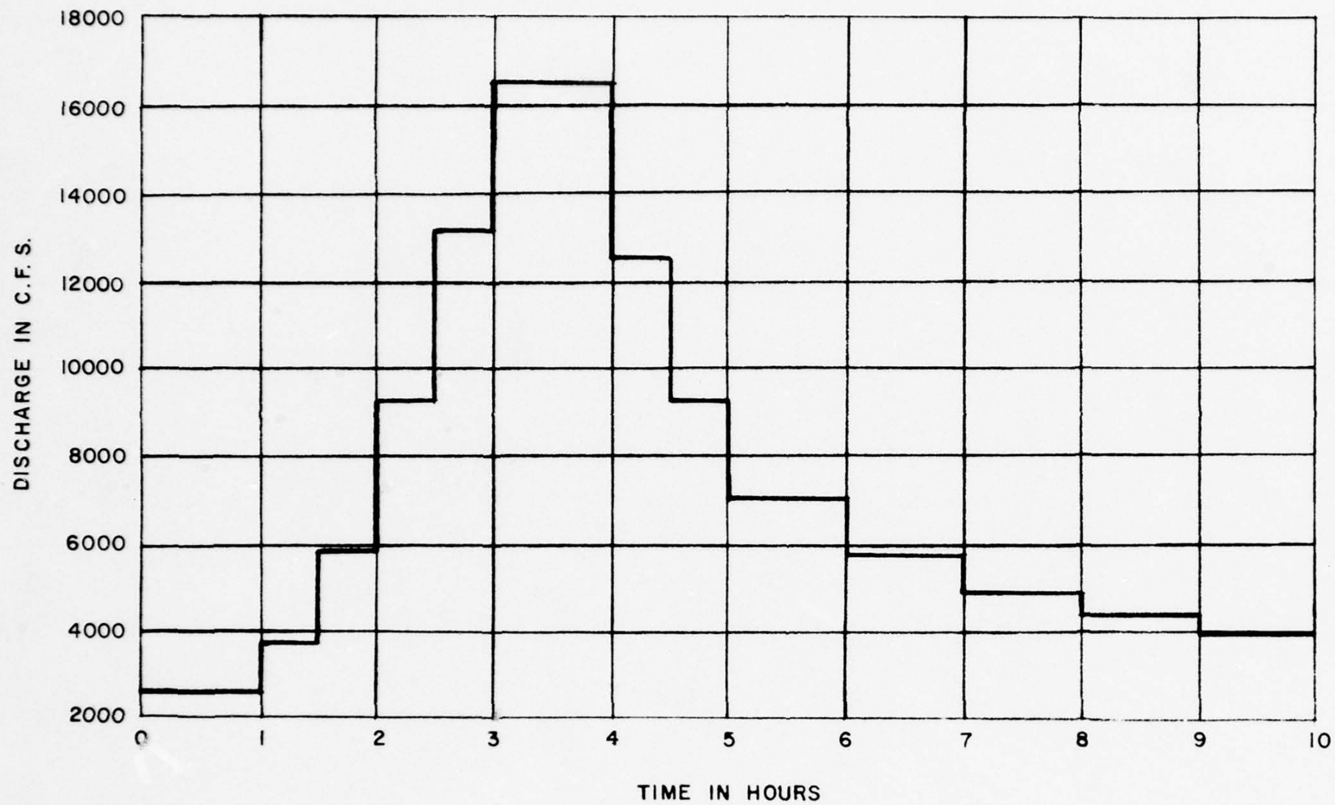
SCOUR PATTERNS

ORIGINAL AND FINAL DESIGNS

AFTER HYDROGRAPH

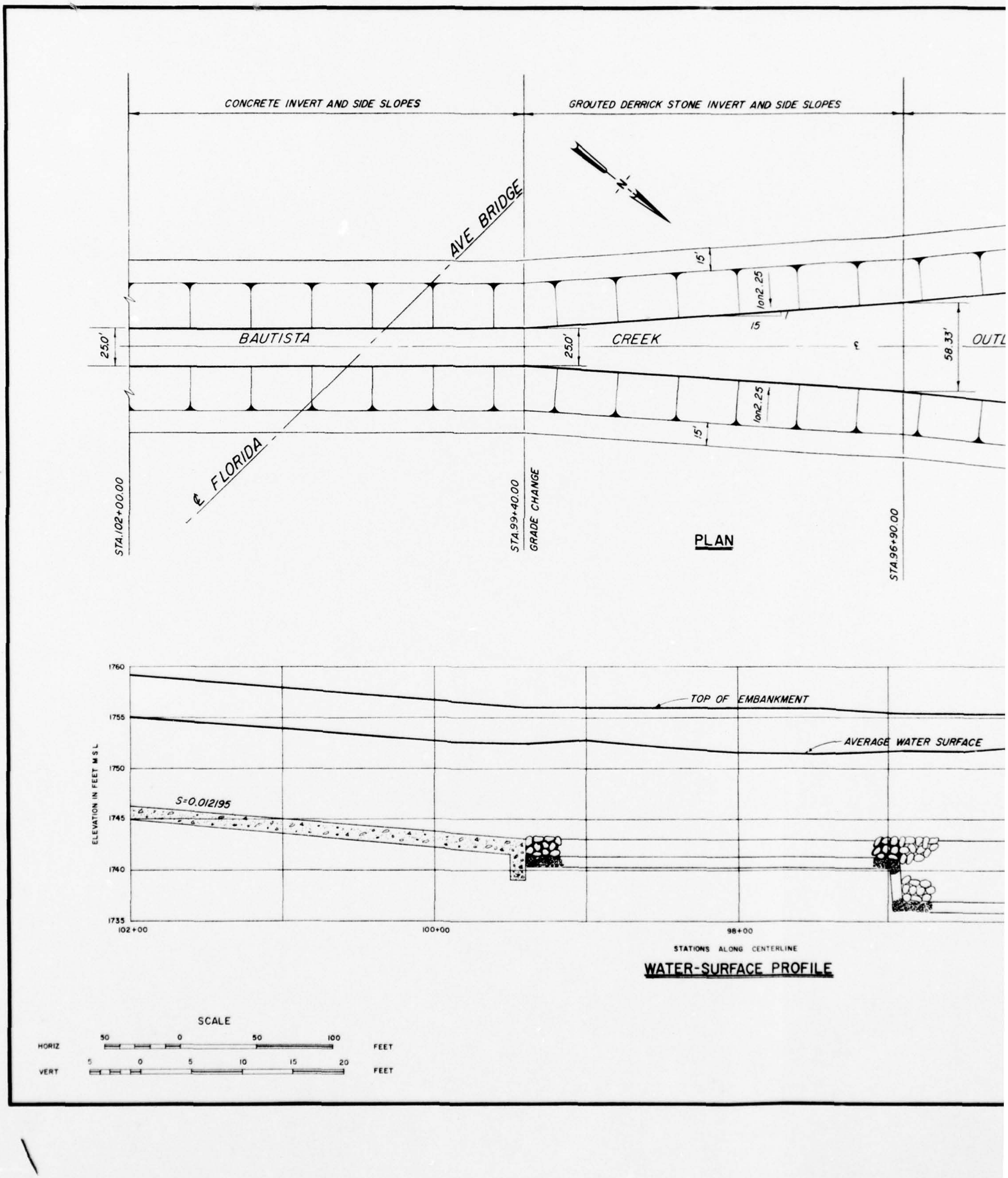
PLATE 3

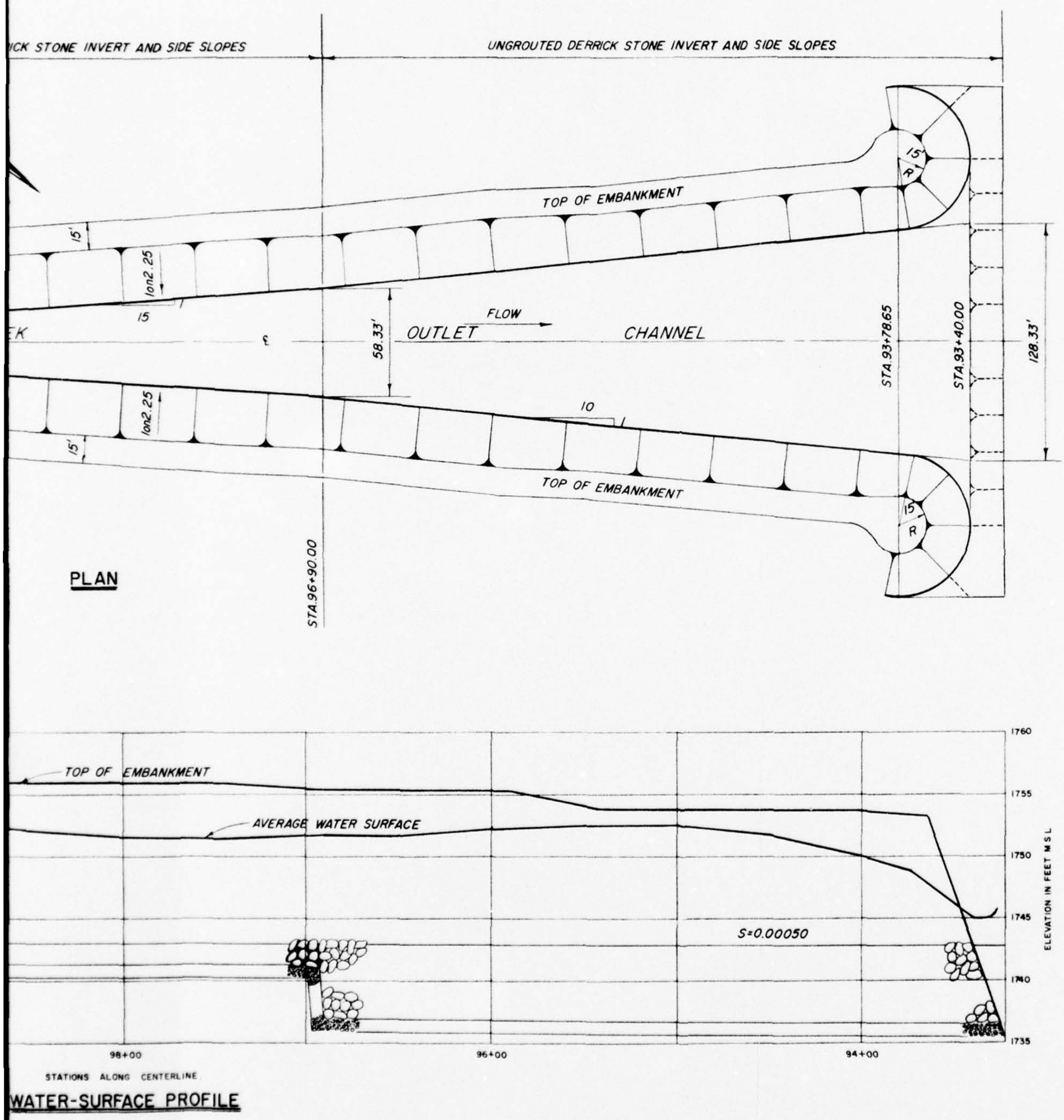
2



HYDROGRAPH

BAUTISTA CREEK OUTLET CHANNEL


CONCRETE INVERT AND SIDE SLOPES
GROUTED DERRICK STONE INVERT AND SIDE SLOPES
N
AVE BRIDGE
℄ FLORIDA
15'
1on2.25
15
25.0'
BAUTISTA
CREEK
℄
58.33'
OUTL
STA.102+00.00
STA.99+40.00
GRADE CHANGE
STA.96+90.00
PLAN
TOP OF EMBANKMENT
AVERAGE WATER SURFACE
S=0.012195
ELEVATION IN FEET M S L
1760
1755
1750
1745
1740
1735
102+00
100+00
98+00
STATIONS ALONG CENTERLINE
WATER-SURFACE PROFILE
SCALE
HORIZ
50 0 50 100
FEET
VERT
5 0 5 10 15 20
FEET



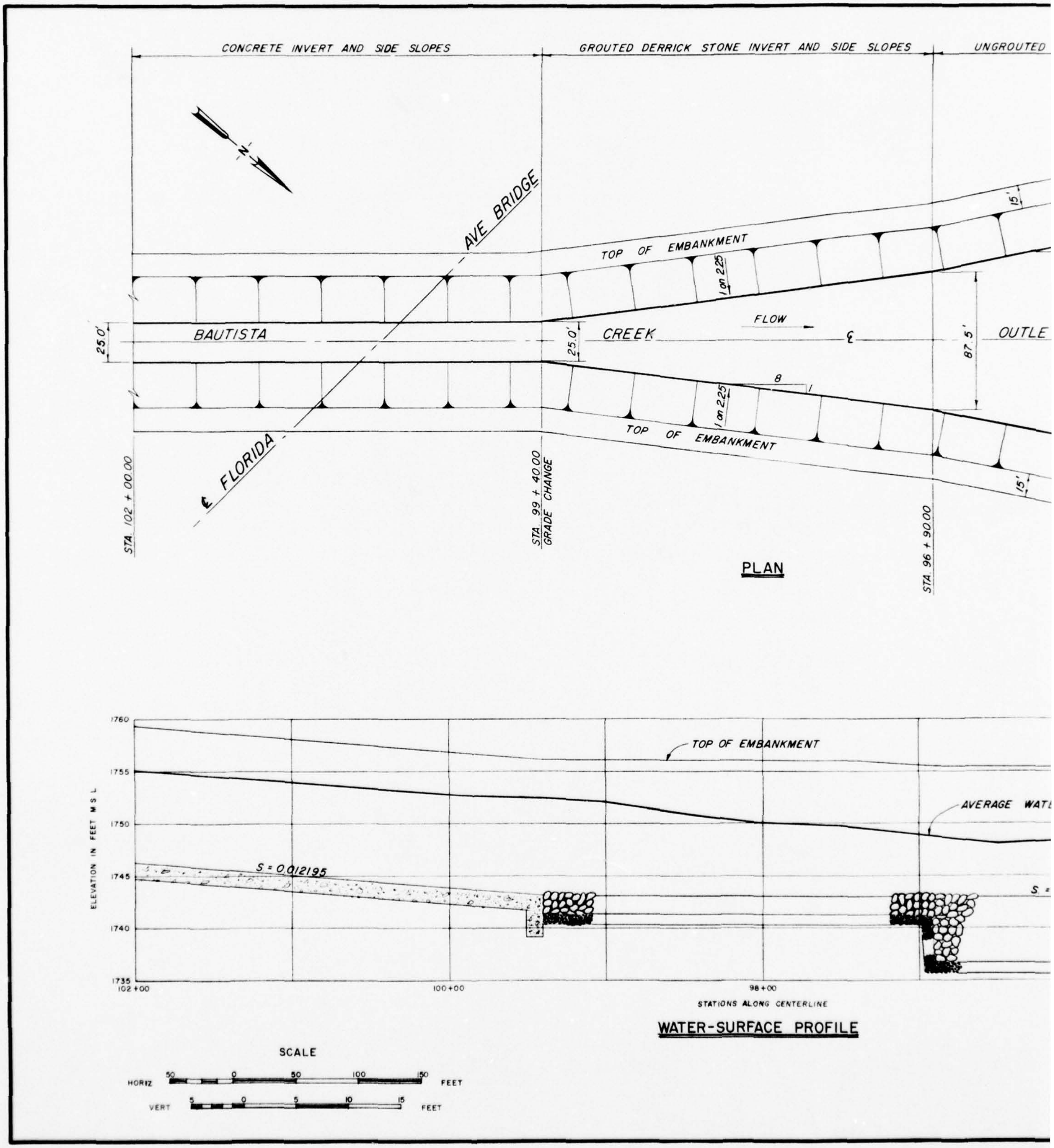
PLAN AND PROFILE

ALTERNATIVE DESIGN

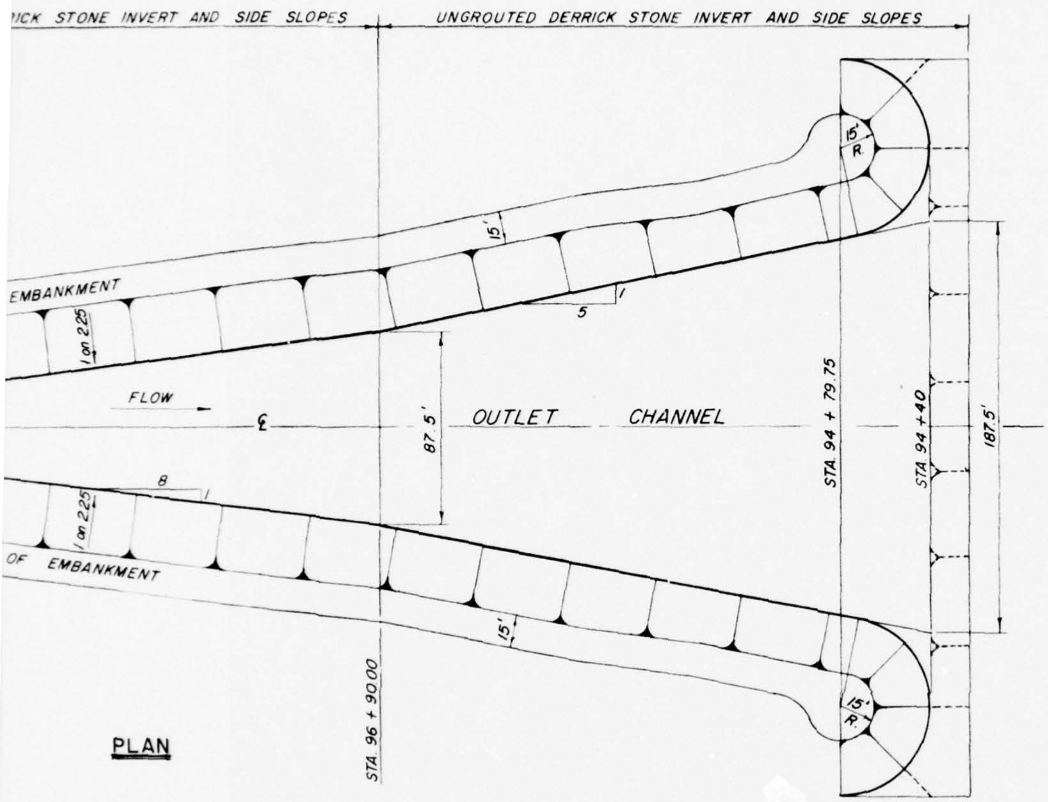
DISCHARGE 16,500 cfs

PLATE 5

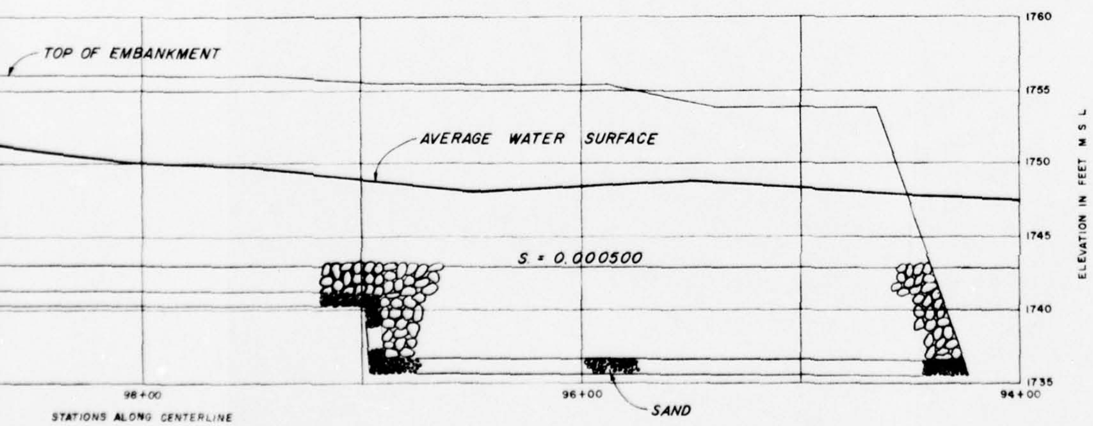
2



CONCRETE INVERT AND SIDE SLOPES
GROUTED DERRICK STONE INVERT AND SIDE SLOPES
UNGROUTED
N
AVE BRIDGE
℄ FLORIDA
TOP OF EMBANKMENT
1 on 2.25
25.0'
BAUTISTA
CREEK
FLOW
℄
87.5'
OUTLE
8
1
15'
STA 102 + 00.00
STA 99 + 40.00
GRADE CHANGE
STA 96 + 90.00
PLAN
ELEVATION IN FEET M S L
1760
1755
1750
1745
1740
1735
TOP OF EMBANKMENT
AVERAGE WATE
S = 0.012195
S =
102+00
100+00
98+00
STATIONS ALONG CENTERLINE
WATER-SURFACE PROFILE
SCALE
HORIZ
50
0
50
100
150
FEET
VERT
5
0
5
10
15
FEET



PLAN



WATER-SURFACE PROFILE

PLAN AND PROFILE

FINAL DESIGN

DISCHARGE 16,500 CFS

PLATE 6

2

DEVIL CREEK DIVERSION CHANNEL

The Prototype

20. The Devil Creek Diversion is about 5 miles northwest of the central part of the city of San Bernardino, Calif. The project consists of a Devil Creek Diversion Levee and a Devil Creek Diversion Channel. The diversion levee extends from a point near the base of the San Bernardino mountains at Badger Canyon to a low range of hills along Kendall Drive. The Devil Creek Diversion Channel extends from a point 900 ft above Kendall Drive to the channel outlet at Cajon Creek, a length of about 10,500 ft. The diversion channel consists of a rectangular section and a trapezoidal section. The channel is connected to the inlet structure at the upstream end and terminates at the downstream end in a flared transition. The general location of the project is shown in Plate 7.

The Model

21. The model, constructed to a scale ratio of 1:25, reproduced a short reach of the Devil Creek Diversion Channel and the outlet channel into Cajon Creek. The model channel, constructed with plywood surfaces that were fitted to shaped wooden ribs mounted on a plywood base, was covered with three coats of high-gloss enamel. This reach of model was steepened to reproduce the prototype "n" value of 0.014.

22. Cajon Creek, an earth-bottom channel, was reproduced in a 20- by 22-ft box filled with concrete sand (mean diameter 0.20 mm) to a depth of 2 ft. A section of an existing groin, with its toe stone protection and levee stone facing, was also reproduced to make certain that it would not be destroyed or in any way be damaged by the outlet flow from Devil Creek Diversion Channel. The model stone passing and retained on the 1-1/4-in. and 1-in. sieves, respectively, was used to simulate 1000- to 2000-lb toe stone. The size of the stone used in the model to simulate the facing stone was 3/4 in.

Original Design

23. The original design, trapezoidal in cross section, had a base width of 20 ft to sta 8+59.31. A terminal bucket at the downstream end between sta 8+59.32 and 8+20.72 flared out to 60 ft at the lip of the bucket. The terminal bucket was protected from undermining, as a result of scouring, by a cutoff wall and by 3000-lb maximum rock protection. The rock protection varied in thickness from 10 ft at the cutoff wall to 20 ft at a distance 50 ft downstream. The model detail and rock placement are shown in Plate 8.

24. In this design, the bulk of the flow was concentrated at the center of the terminal bucket. Eddies that formed around both walls caused considerable damage to the structure.

25. Since this design was found to be inadequate, a series of modifications were made to the original design in an attempt to improve the hydraulic efficiency of the structure but to no avail. Data or photographs of the original design and intermediate designs were not recorded.

Final Design

26. In the final design, the 20-ft-wide trapezoidal section with 1V-on-2.25H side slopes terminated at sta 12+50. The bottom width of the trapezoidal channel increased from 20 ft at sta 12+50 to 60 ft at sta 9+50. The structure terminated in a dentated sill at sta 9+25.25. The dentated sill had 15 piers, 5 ft high, 3 ft wide, and spaced 6 ft on centers. The piers rose in the direction of flow on a 1V-on-2H slope (Plate 9). The 3000-lb rock protection at the end of the structure was 10 ft thick and was placed on a 1V-on-2H slope for a distance of 50 ft downstream. The model detail and rock placement are shown in Plate 10. Photo 10 shows the excavation for the rock fill and the rock protection in place.

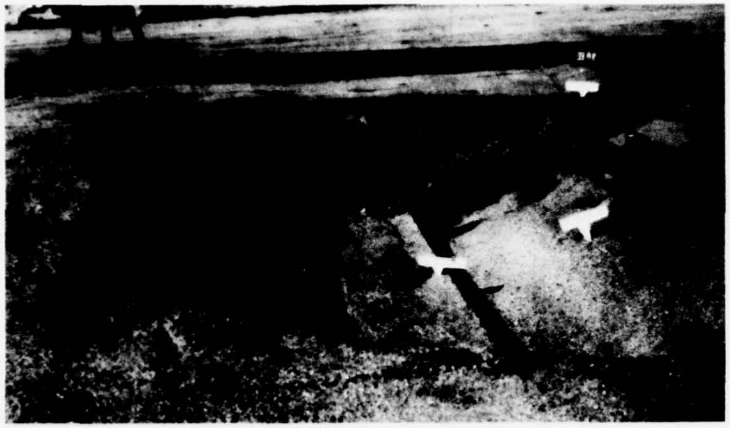
27. The test involved two different numbers of dentates, 10 and 15. The same type of dentates was used and the model was the same

throughout (Photo 11). The purpose of the test runs, made in accordance with the hydrograph in Plate 11, was to compare the hydraulic action and resulting scour of the 10-dentate design with that of the 15-dentate design. The test of the 15-dentate sill indicated better energy dissipation characteristics. A slight improvement over the 10-dentate sill results was noted in the flow condition over the terminal end sill and in the scour patterns, with good protection to the cutoff wall. Since the flow was much more evenly distributed across the end sill and the flow was spread over a larger area of Cajon Creek, it was considered best to go with the 15-dentate sill. Photo 12 shows the flow conditions for the 10- and 15-dentate sills.

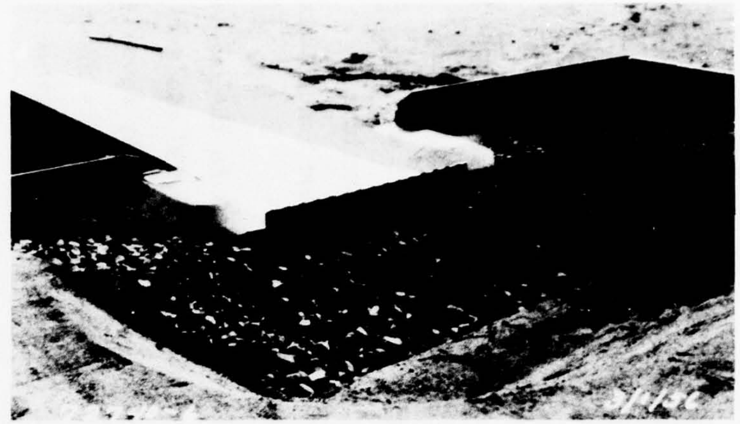
28. Water-surface profiles and velocity measurements in the approach channel and transition at the design discharge of 16,030 cfs are shown in Plates 12, 13, and 14. The scour occurring immediately downstream from the terminal sill as a result of the hydrograph run is shown in Photos 13 and 14. Comparison of the scour patterns with and without dentates is shown in Plates 15 and 16.

Discussion

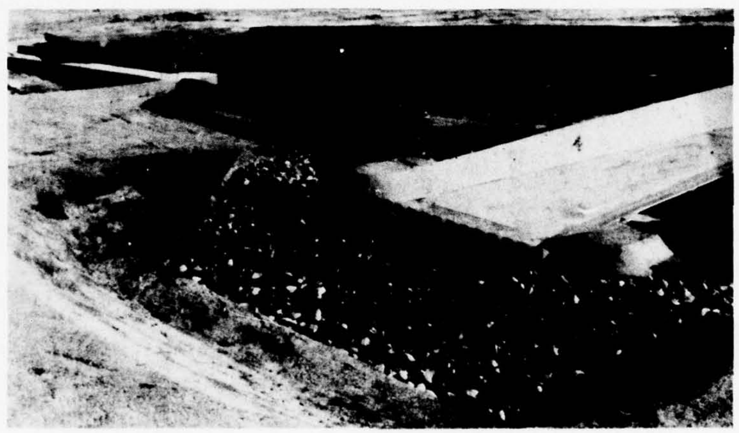
29. The original design afforded little protection to the cutoff wall. The original and intermediate designs proved inadequate for the peak discharge. The 300-ft transition, with a 60-ft base width at the downstream end and dentates at the end sill, provided adequate dissipation of the energy of the high-velocity flow from the channel upstream; however, the most desirable hydraulic action for all flows up to and including 16,030 cfs was obtained with the 15-dentate sill. The favorable results prompt its adoption for the prototype design.



Excavation for rock fill



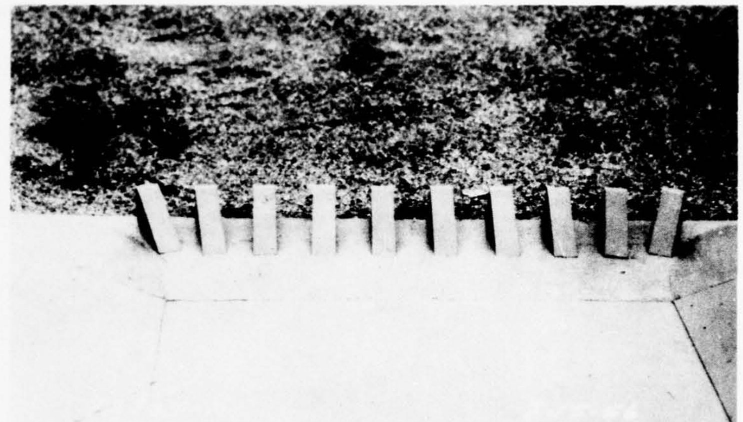
Rock in place



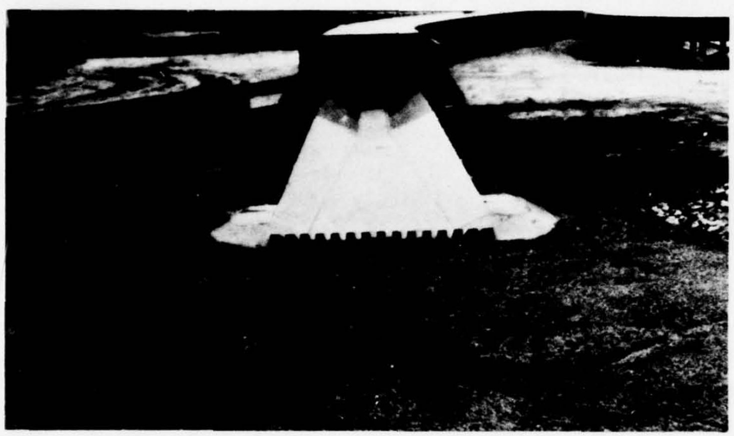
Rock in place

Photo 10. Final design, Devil Creek Diversion

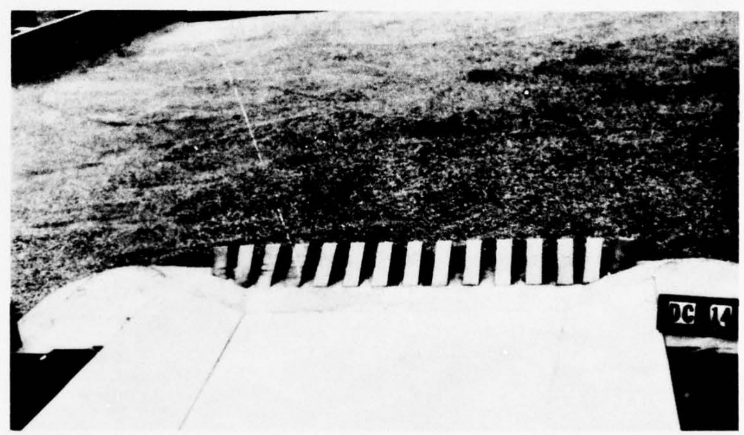
Looking upstream, 10 dentates



Looking downstream, 10 dentates



Looking upstream, 15 dentates

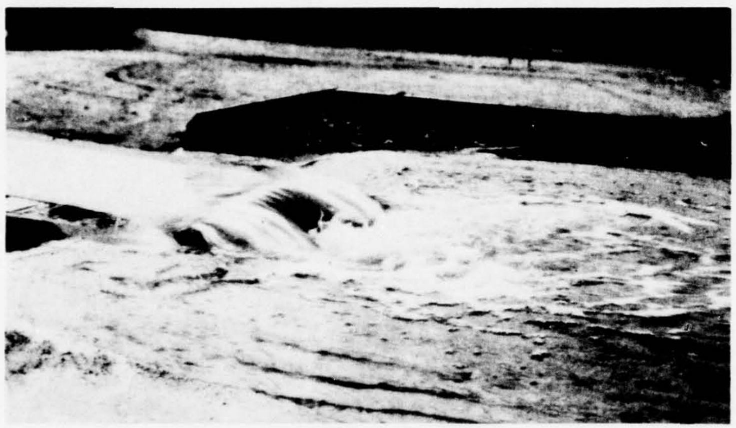


Looking downstream, 15 dentates

Photo 11. Final design, Devil Creek Diversion, with 10 and 15 dentates

10 dentates, discharge 5600 cfs

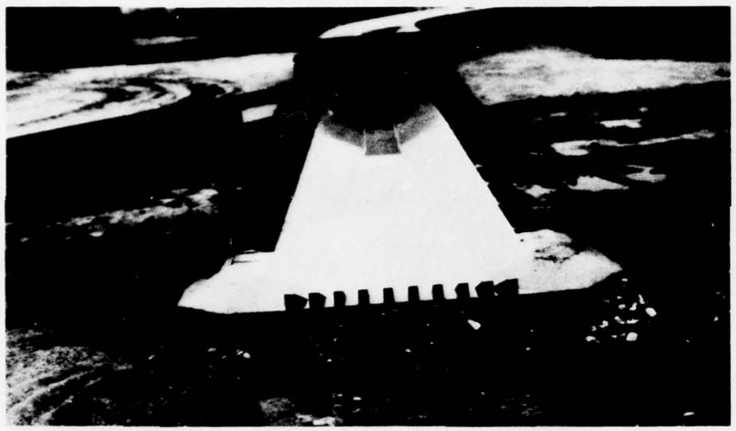
10 dentates, discharge 13,600 cfs



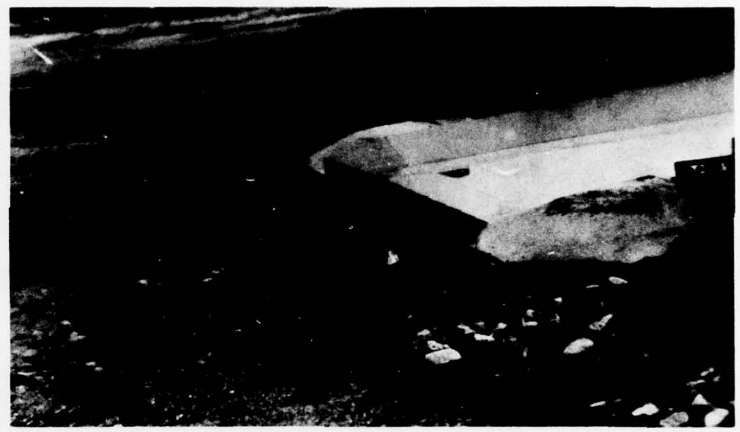
15 dentates, discharge 13,600 cfs

15 dentates, discharge 16,030 cfs

Photo 12. Final design flow conditions, Devil Creek Diversion

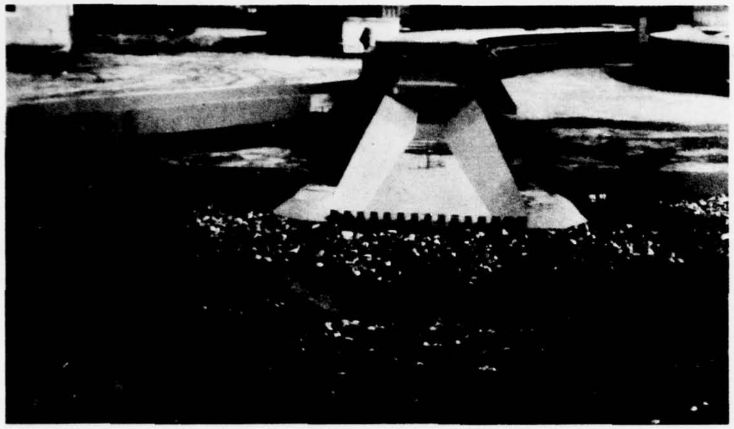


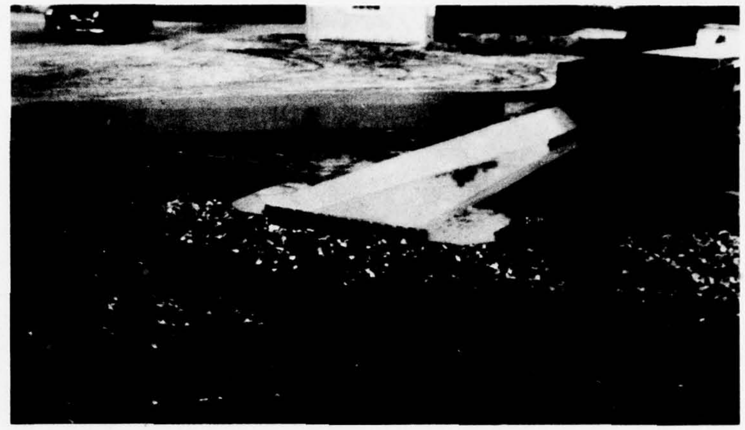
Looking upstream



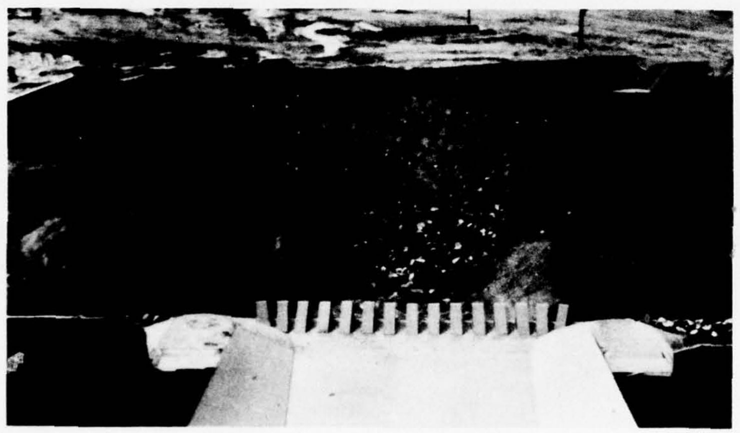
Looking from left side

Photo 13. Final design, Devil Creek Diversion, scour after hydrograph run, 10 dentates



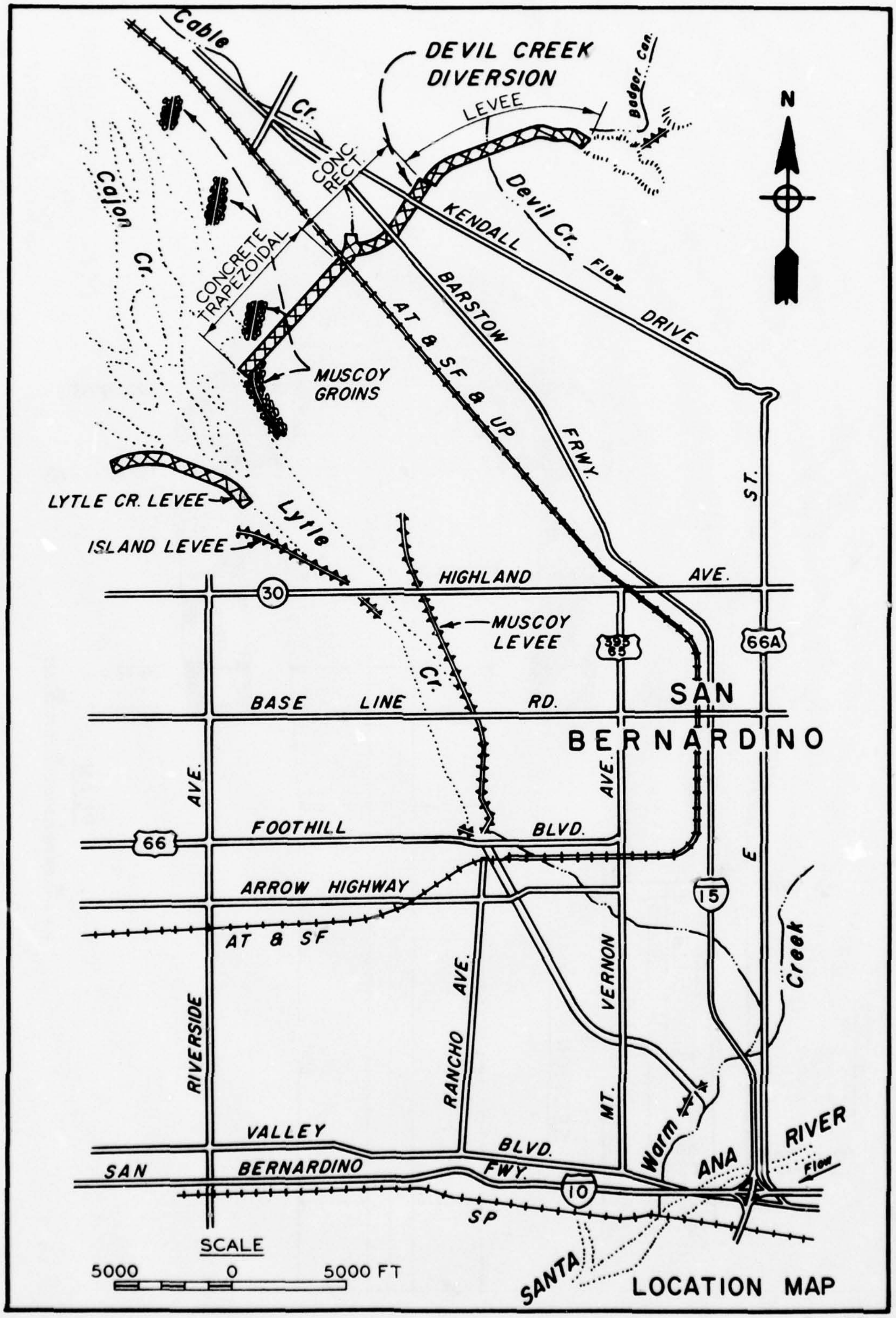


Looking upstream



Looking downstream

Photo 14. Final design, Devil Creek Diversion, scour after hydrograph run, 15 dentates



DEVIL CREEK DIVERSION
LEVEE
Badger Can.
Cable
Cr.
CONC RECT.
CONCRETE TRAPEZOIDAL
Cajon
Cr.
Devil Cr.
Flow
KENDALL
DRIVE
BARSTOW
FRWY.
AT & SF & UP
MUSCOY GROINS
LYTLE CR. LEVEE
Lytle
ISLAND LEVEE
HIGHLAND
AVE.
30
MUSCOY LEVEE
395 65
66A
Cr.
ST.
BASE
LINE
RD.
SAN BERNARDINO
AVE.
AVE.
FOOTHILL
BLVD.
66
15
E
ARROW HIGHWAY
AT & SF
Creek
RIVERSIDE
RANCHO AVE.
MT. VERNON
VALLEY
BLVD.
Warm
SAN
BERNARDINO
FWY.
10
ANA
RIVER
Flow
SP
SANTA
SCALE
5000
0
5000 FT
LOCATION MAP

PLATE 7

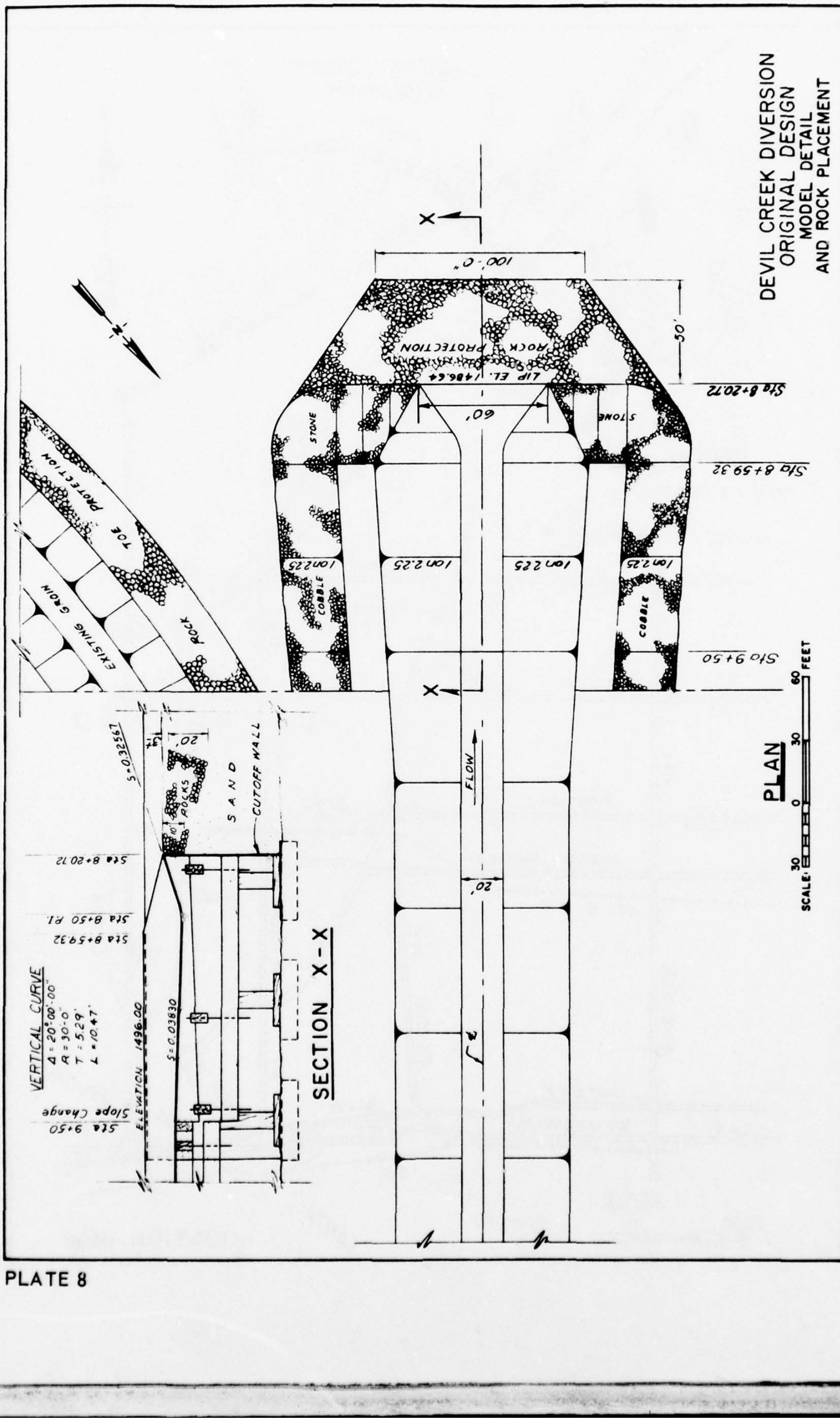
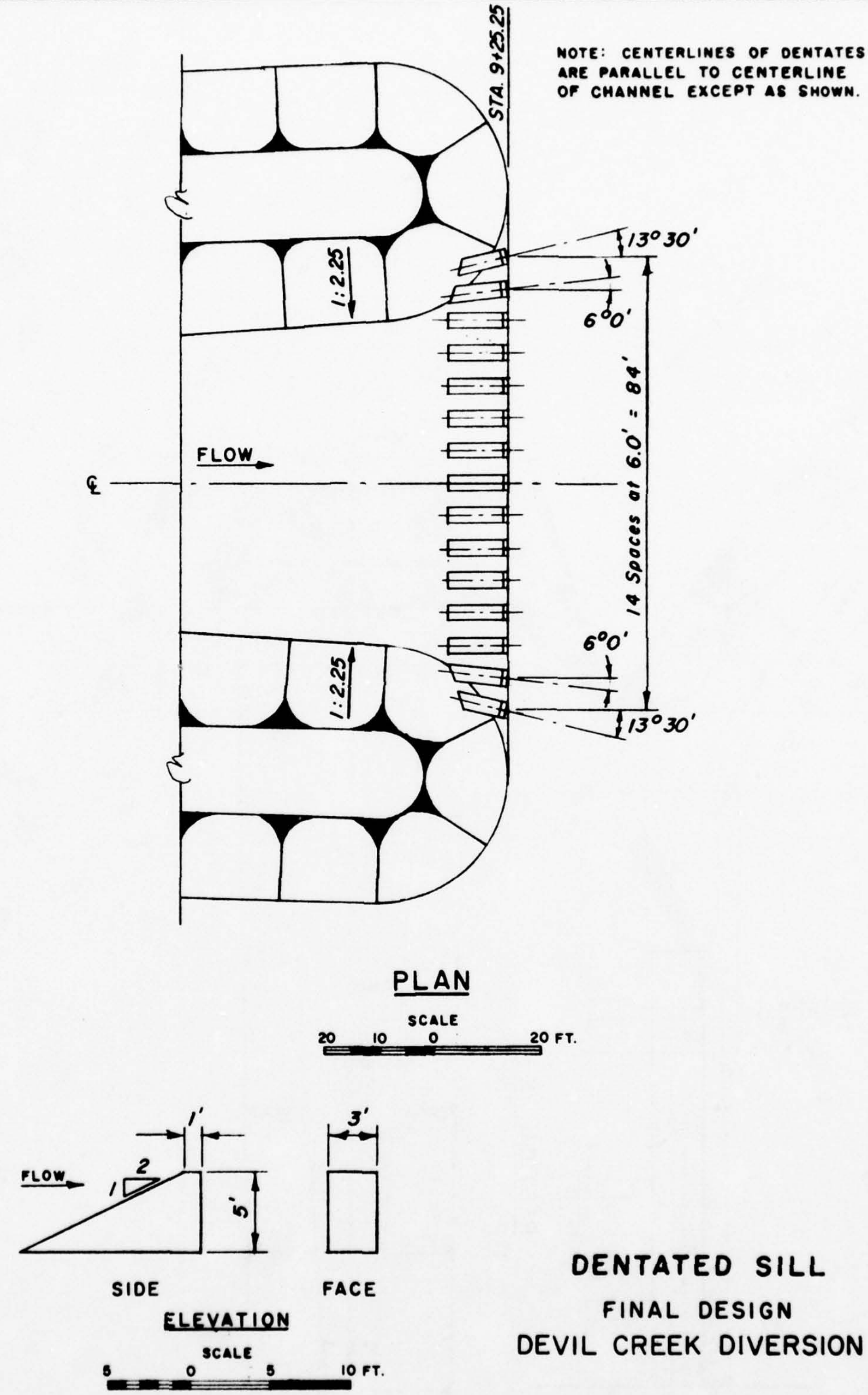

DEVIL CREEK DIVERSION
ORIGINAL DESIGN
MODEL DETAIL
AND ROCK PLACEMENT
PLAN
SCALE: 30 0 30 60 FEET
100'-0"
50'
60'
20'
LIP EL. 1486.64
ROCK PROTECTION
STONE
COBBLE
1 on 2.25
Sta 8+20.72
Sta 8+59.32
Sta 9+50
FLOW
X
EXISTING GROIN
ROCK TOE PROTECTION
SECTION X-X
VERTICAL CURVE
Δ = 20°-00'-00"
R = 30'-0"
T = 5.29'
L = 10.47'
Sta 9+50
Slope Change
Sta 8+59.32
Sta 8+50 P.I.
Sta 8+20.72
ELEVATION 1496.00
S = 0.03830
S = 0.32567
ROCKS
SAND
CUTOFF WALL
20'

PLATE 8



DENTATED SILL

FINAL DESIGN

DEVIL CREEK DIVERSION

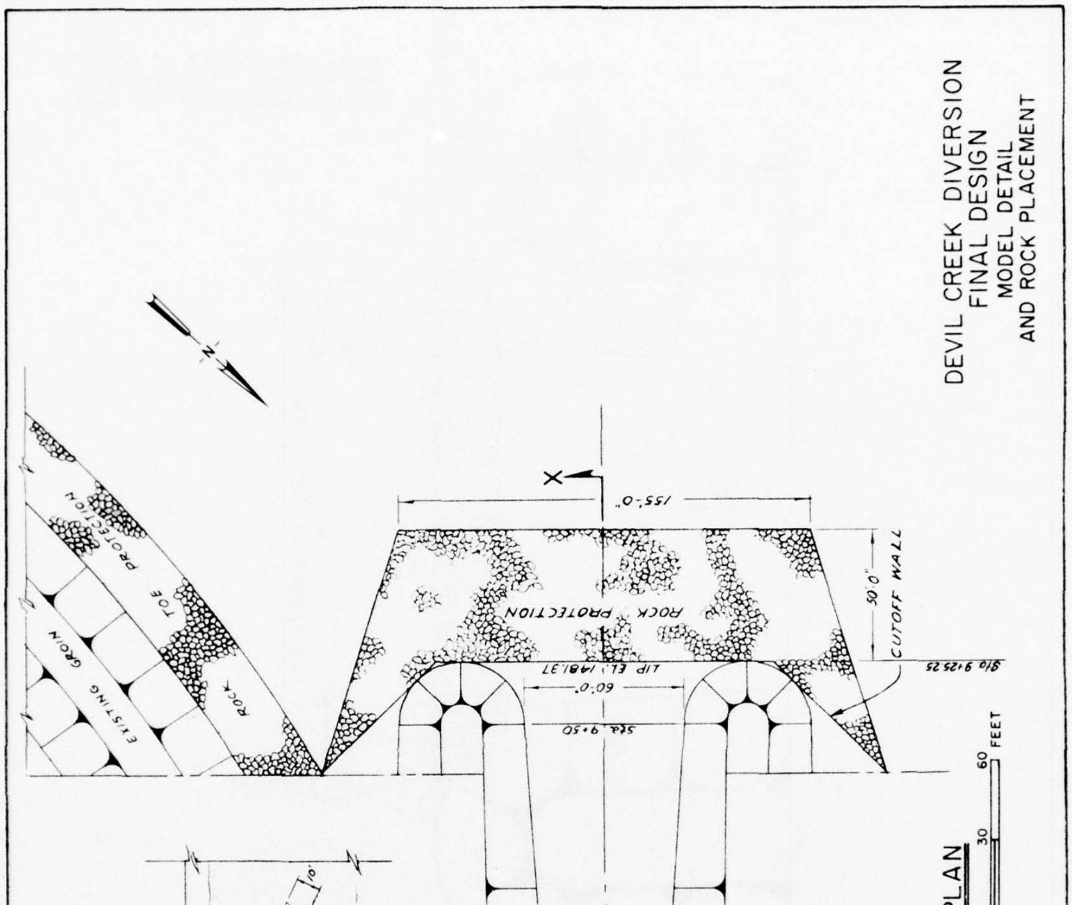
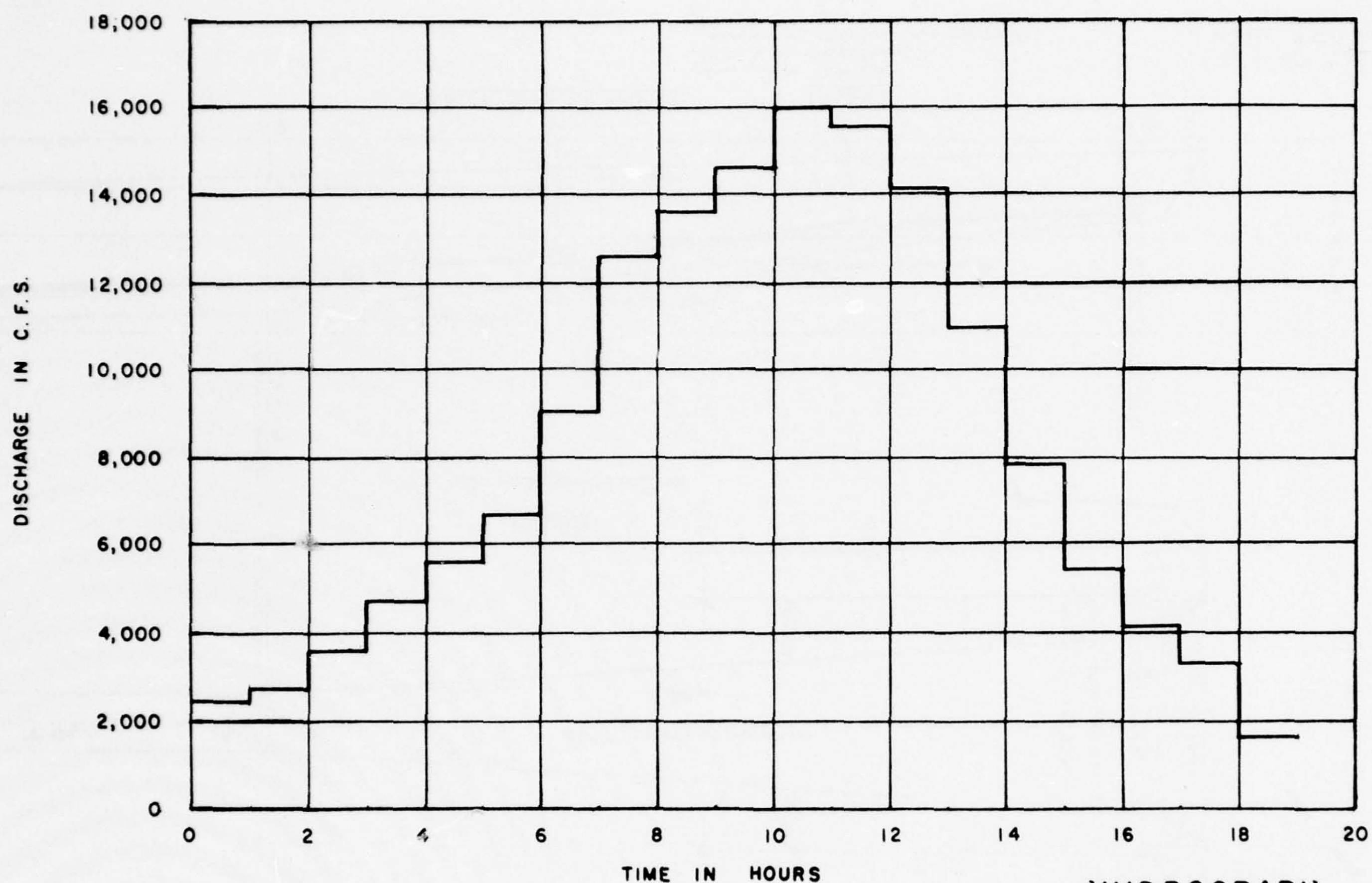
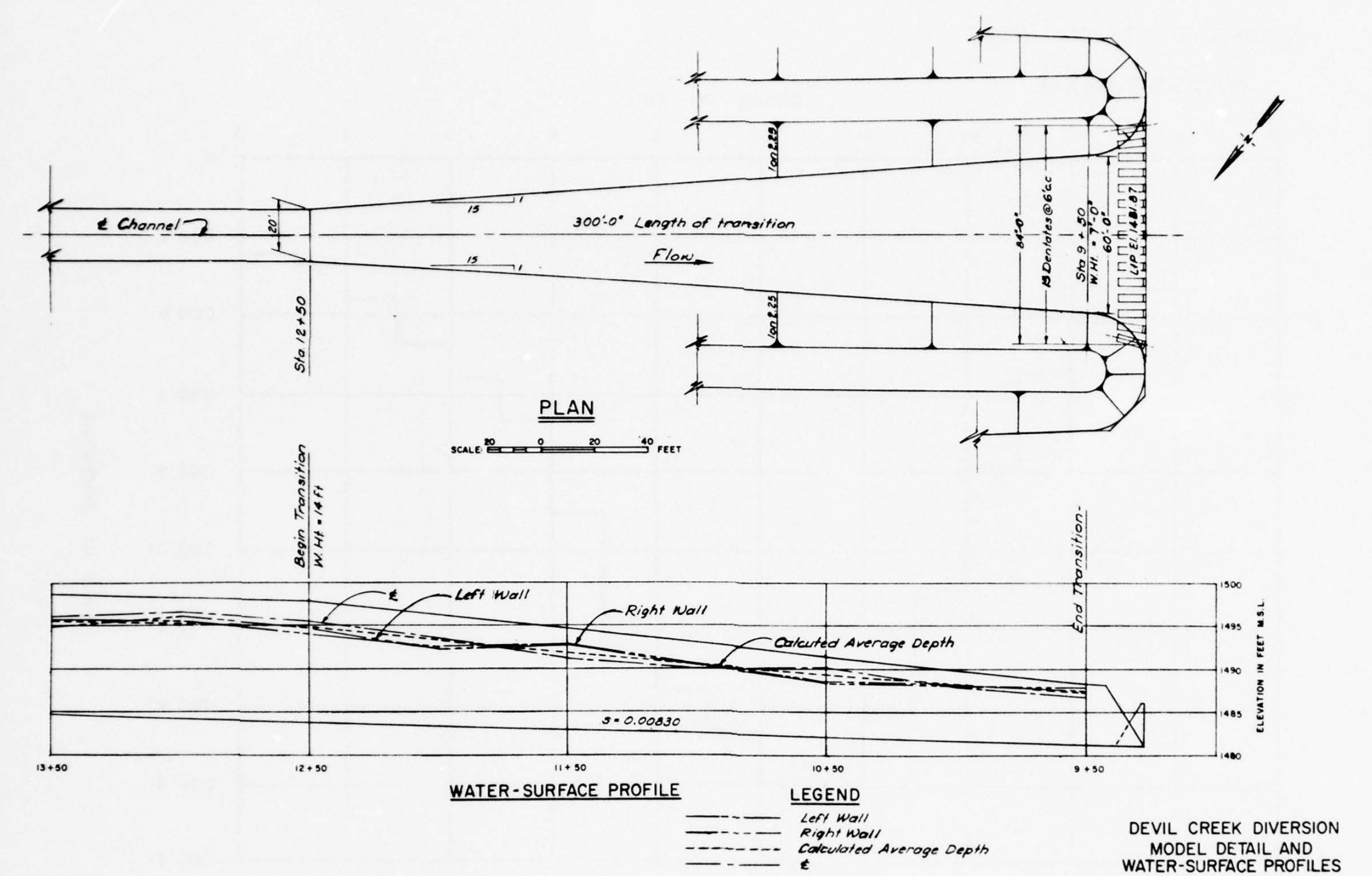

DEVIL CREEK DIVERSION
FINAL DESIGN
MODEL DETAIL
AND ROCK PLACEMENT
PLAN
SECTION X-X
SCALE 30 0 30 60 FEET
ROCK PROTECTION
CUTOFF WALL
EXISTING GROIN
ROCK TOE PROTECTION
155'-0"
50'0"
60'0"
LIP EL. 1481.37
STA. 9+25.25
STA. 9+50
STA. 12+50
FLOW
1 on 2.25
20'0"
SAND
S=0.00830
N

PLATE 10



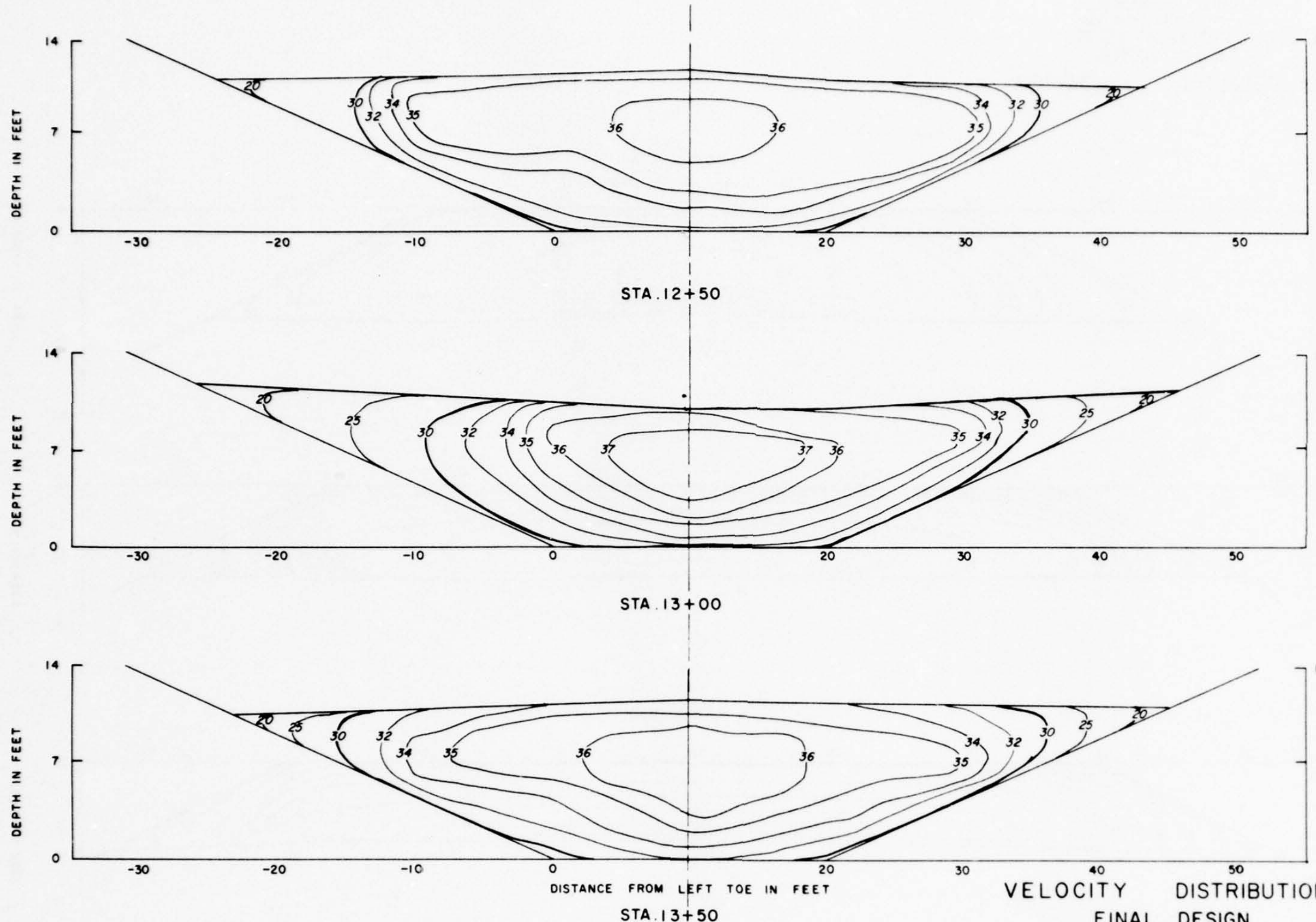
HYDROGRAPH

DEVIL CREEK DIVERSION



DEVIL CREEK DIVERSION
MODEL DETAIL AND
WATER-SURFACE PROFILES
FINAL DESIGN
DISCHARGE: 16,030 cfs

PLATE 12



VELOCITY DISTRIBUTION

FINAL DESIGN

DEVIL CREEK DIVERSION

STA 12+50, 13+00, AND 13+50

DISCHARGE 16,030 CFS

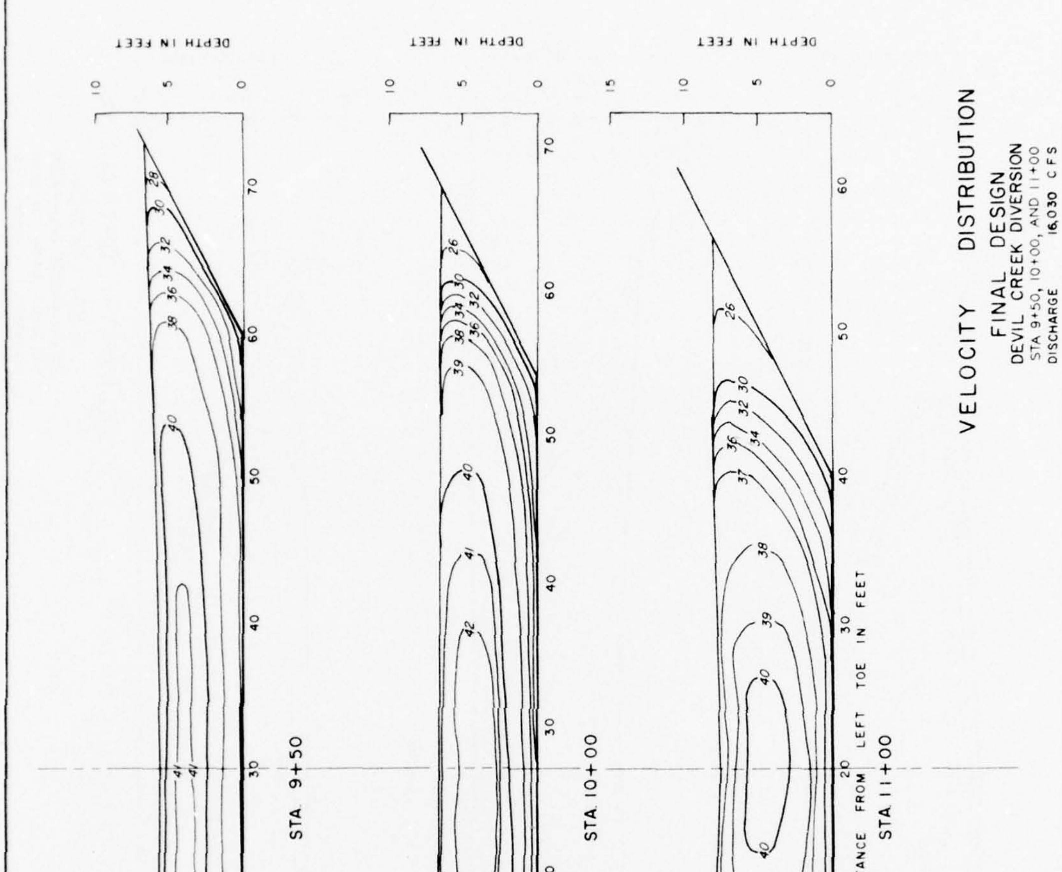

VELOCITY DISTRIBUTION
FINAL DESIGN
DEVIL CREEK DIVERSION
STA 9+50, 10+00, AND 11+00
DISCHARGE 16,030 CFS
STA 9+50
STA 10+00
STA 11+00
DISTANCE FROM LEFT TOE IN FEET
DEPTH IN FEET

PLATE 14

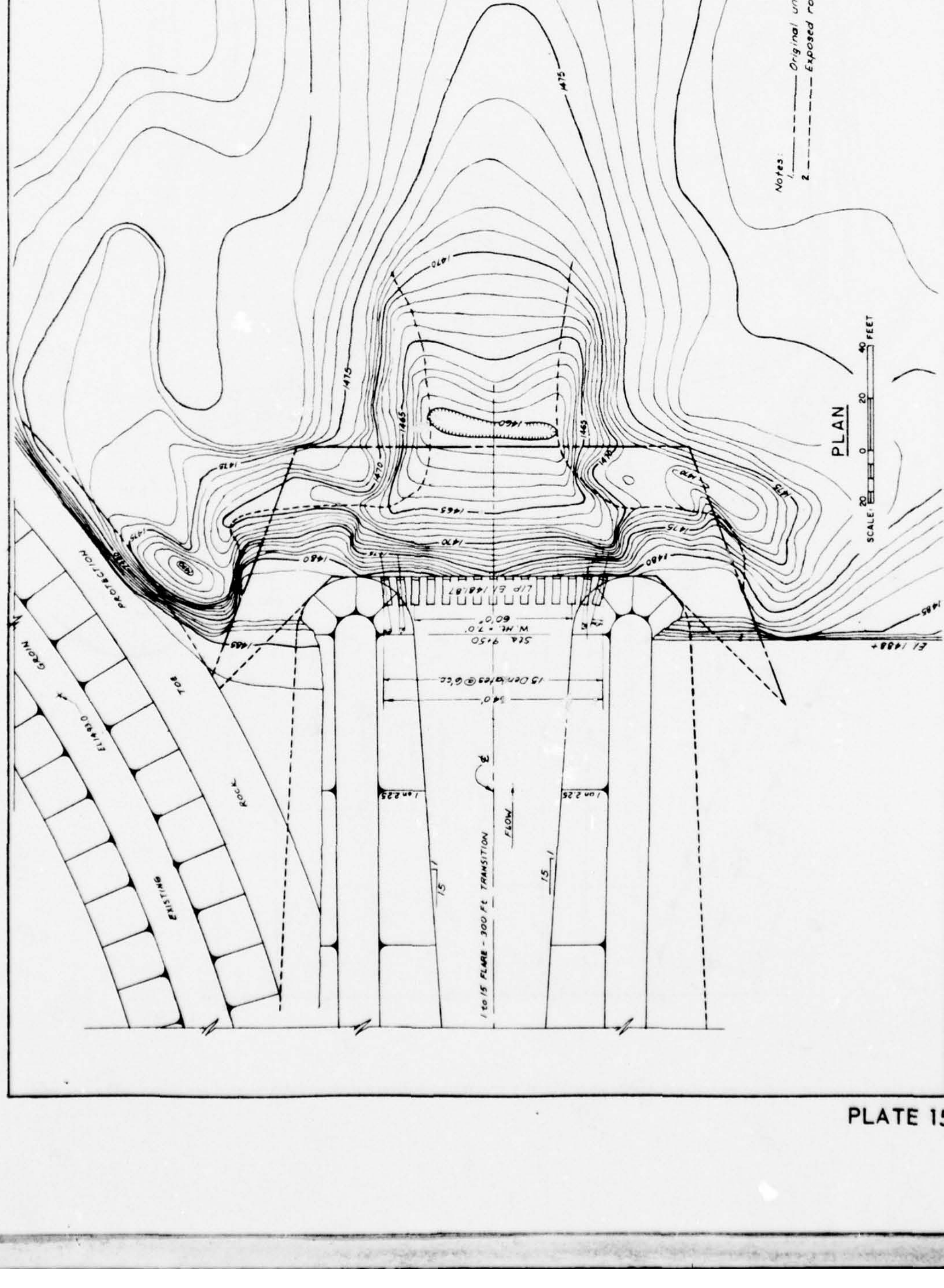

DEVIL CREEK DIVERSION
SCOUR PATTERN
FINAL DESIGN
WITH DENTATES
(AFTER HYDROGRAPH RUN)
ORIGINAL ROCK REVETMENT
ORIGINAL BED
SCOURED BED
ELEVATION IN FEET MSL
STATIONS ALONG CENTERLINE
Notes:
1. Original unexposed rock outline
2. Exposed rock outline after hydrograph run.
PLAN
SCALE
FEET
EXISTING ROCK TOE PROTECTION
GROIN
FLOW
1 to 15 FLARE - 300 Ft TRANSITION
LIP El 1481.87
Sta 9+50
60.0'
54.0'
El 1488+

PLATE 15

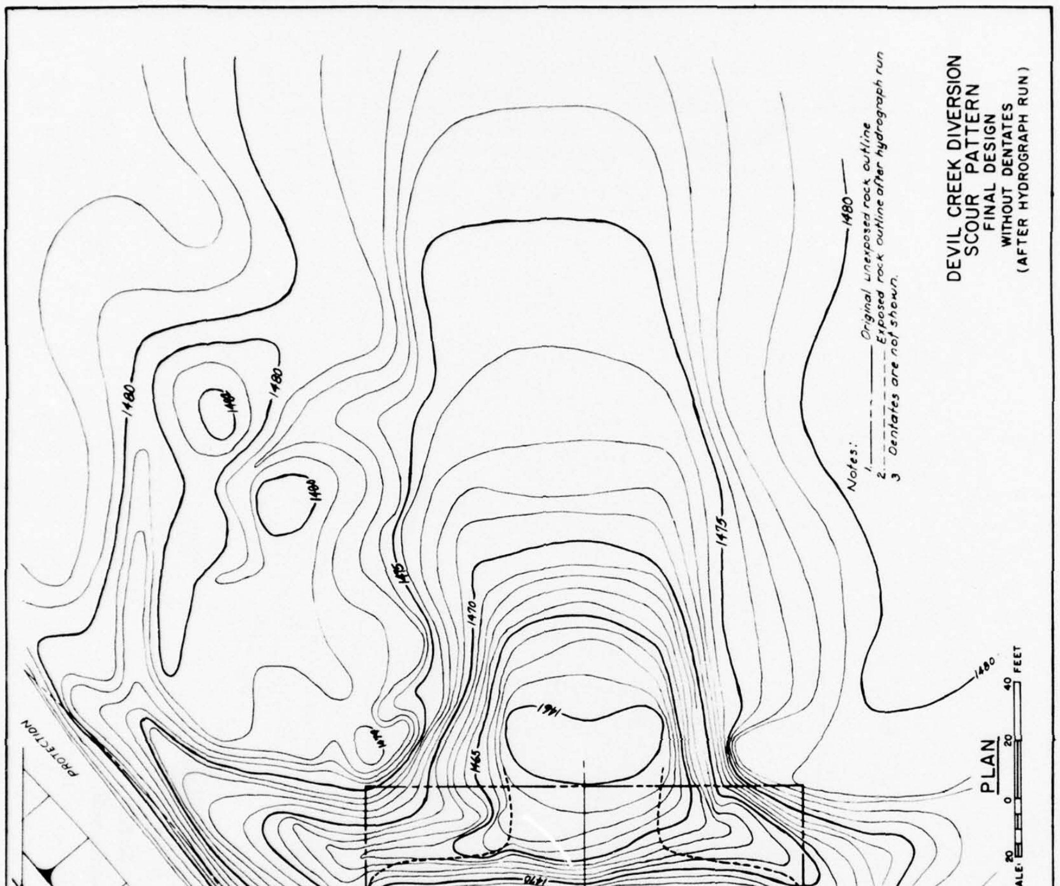

DEVIL CREEK DIVERSION
SCOUR PATTERN
FINAL DESIGN
WITHOUT DENTATES
(AFTER HYDROGRAPH RUN)
Notes:
1. Original unexposed rock outline
2. Exposed rock outline after hydrograph run
3. Dentates are not shown.
PLAN
SCALE: 20 0 20 40 FEET
1480
1475
1470
1465
EXISTING ROCK TOE PROTECTION
GROIN
300-FT TRANSITION
FLOW
1 on 2.25
60.0'

PLATE 16

TUCSON DIVERSION CHANNEL

The Prototype

30. The Tucson Diversion Channel discharges into the Santa Cruz River approximately 1 mile west of the intersection of U. S. Highways 80 and 89 in South Tucson, Ariz. (Plate 17). The reach of the prototype channel reproduced in the model consists of four types of channel. The first or the upstream reach of channel between sta 30+00 and 27+00 is a reinforced-concrete rectangular channel, with a width of 40 ft. The second reach, which is a trapezoidal channel with side slopes of 1V on 2.25H and a base width of 40 ft, extends downstream from sta 27+00 to 23+00. The third reach is a transition, trapezoidal in cross section with base widths varying from 40 ft at sta 23+00 to 160 ft at sta 16+46. This reach contains a 500-ft radius curve with the B.C. at sta 19+19.19 and the E.C. at sta 22+82.81. The fourth reach is trapezoidal in cross section with a 160 ft base width, 1V-on-2.25H side slope, and extends from sta 16+46 to 11+70. The diversion channel from sta 27+00 to 18+20 is lined with grouted stone and from sta 18+20 to 16+10 with dumped stone. The channel from sta 16+10 to 11+70 is an excavated earth channel in the Santa Cruz River bed which consists of sandy silt. The grouted stone was specified as 3000-lb maximum grouted with 1 ft of stone projection and the dumped stone specified was 5000-lb maximum.

31. The Tucson Diversion Channel was designed to convey a peak discharge of 17,800 cfs when the flow in the Santa Cruz River might vary between 0 and 24,600 cfs. At peak discharge in the concrete reach, the flow has a supercritical velocity of 41 fps. At the downstream end of the transition, the velocity varies from 9 to 15 fps, depending upon the discharge in the Santa Cruz River. The "n" values, used in the Manning formula, were assumed to be 0.014 for the reach of rectangular concrete channel, 0.040 for the grouted and dumped quarrystone reaches, and 0.025 for the excavated reach.

The Model

32. The available space and facilities in the model laboratory

were such that it was necessary for the model to be reversed from the prototype. The model was constructed to an undistorted scale ratio of 1:40. In addition to the reach of the diversion channel as described in a preceding paragraph, a short reach of the Santa Cruz River at the confluence was simulated in the model to provide the backwater effect in the diversion channel and flow characteristics at the junction.

33. The materials used to construct the model were wood, sand, gravel, and cement. The reach of reinforced-concrete channel was simulated by wood in the form of timber and plywood finished with high-gloss enamel. The grouted-stone reach was simulated by cementing pea gravel to the plywood facing. The reach of dumped stone was simulated by placing crushed rock which varied from 1-1/4 in. to 3/8 in. in molded sand. The channel downstream of the dumped stone and the portion of the Santa Cruz River were molded in a bed of sand (Photo 15).

34. The "n" value in the reach of reinforced-concrete channel was simulated to the design value of 0.014 by the addition of a supplementary invert slope. The roughness coefficient ("n") of the painted-plywood surface had previously been determined to be 0.0088. Based on similitude, this value was equivalent to a prototype value of 0.0016. The "n" value in the grouted reach was simulated to the design value of 0.040 by cementing gravel particles to the plywood surface. The size and amount of gravel required to simulate the proper "n" value was determined by trial runs.

35. The reach of the model that simulated the dumped stone and natural-earth channel was molded in sand having a mean diameter of 0.20 mm. The crushed rock placed in the sand to simulate dumped stone had a specific gravity of 2.64 which was assumed equal to the prototype. This was necessary to maintain similitude of geometry and density. Crushed rock was used to represent the shape of the quarrystone used in the prototype and the stone was graded to ensure proper particle size distribution.

36. The average size of grouted stone to be used on the prototype invert and side slopes would be 1000 lb. The dumped stone used in the prototype would be 4 ft thick on the invert and side slopes,

conforming to the following gradations:

Quarrystone

Weight of Stone, lb	Percent Smaller by Weight
5000	100
3000	75
2000	50
1000	25
250	0

Final Design

37. Two flow conditions were considered pertinent to the design of the outlet channel for the Tucson Diversion Channel: (a) 17,800 cfs in Tucson Diversion Channel and 24,600 cfs in Santa Cruz River, and (b) 17,800 cfs in Tucson Diversion Channel and zero flow in Santa Cruz River. Data obtained during the tests consisted of water-surface profiles, velocity distribution, and photographs of flow conditions.

Test 1

38. Test 1 was based on the assumption that the Santa Cruz River would be passing 24,600 cfs at the beginning of the Tucson Diversion Channel design flood. Therefore, the design was tested by running flows representing the project-design-flood hydrograph of a 9-hr period with a peak discharge of 17,800 cfs in the Tucson Diversion Channel and a constant discharge of 24,600 cfs in the Santa Cruz River. The magnitude and duration of the design flood of the Tucson Diversion Channel are shown by the hydrograph in Plate 18. With the peak flows, the discharge from Santa Cruz River produced a backwater effect in the outlet channel. Quantitative data obtained for the above-mentioned flow combination included the water-surface profiles and velocity distribution within the diversion channel outlet. The water surface in the channel was not exceptionally rough; however, the surface currents indicated the concentration of flow along the right bank. Water-surface profiles in the

diversion channel outlet are shown in Plate 19. Photo 16 shows flow conditions through the curved transition. The combined flow conditions at the outlet channel and Santa Cruz River are also shown in Photo 16. The velocity distribution cross sections shown in Plate 20 indicate an uneven distribution of velocities through the transition. The supercritical velocity was greatly reduced as the design discharge passed through the grouted and dumped stone reaches. The average bottom velocity was about 10 fps at the downstream end of the outlet channel. The scour, which occurred immediately downstream from the dumped stone as a result of this test, is shown in Photo 17. The amount of scour was moderated and the damage to the dumped stone invert and side slopes was negligible.

Test 2

39. Test 2 was conducted with zero flow in Santa Cruz River and the range of discharges shown by the hydrograph in Plate 18 for the Tucson Diversion Channel. At the beginning of the test, water was admitted into the area below the ungrouted stone until natural tailwater level was reached. This water cushion prevented disproportionate scour at the beginning of the hydrograph run. Sufficient tailwater within the Santa Cruz River aided in producing an effective jump (Photo 18) at the downstream end of the ungrouted stone reach, resulting in a shallow depth of scour. The extent of the scour is shown in Photo 19. Since the scour was not excessive, no measurements were taken of the scour patterns for Test 1 or Test 2. Water-surface profiles and velocity cross sections for a discharge of 17,800 cfs are shown in Plates 21 and 22, respectively. The velocity cross sections indicate that the flow distribution through the lower reach of the transition was more uniform than in Test 1. Although the velocities were higher, there was little damage to the ungrouted stone lining.

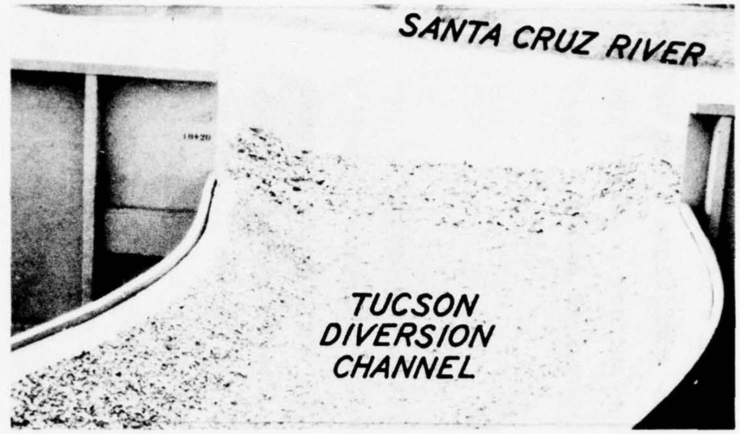
40. When all the desired data with the model simulating an "n" value of 0.014 were obtained, the model was adjusted to simulate an "n" value of 0.011 by adding more supplementary slope to the invert grade. Tests were then made to show the operation of the transition with the lower roughness coefficient. Comparing the results of these tests with

those of Test 1 and Test 2, it was found that with a discharge of 17,800 cfs, no apparent change occurred in flow conditions or in the resulting scour patterns. Measurements of water-surface profiles and velocity distribution were made but are not illustrated in this report.

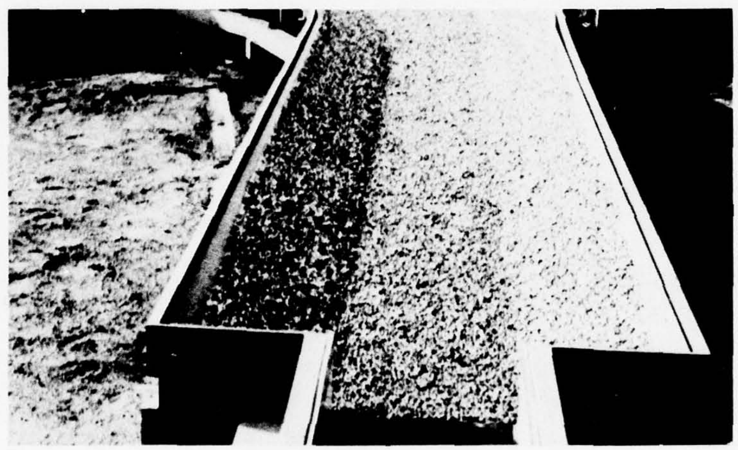
Discussion

41. The results of the tests indicate that the design of the outlet channel for the Tucson Diversion Channel was generally satisfactory. The high-velocity flow emerging from the rectangular channel upstream was sufficiently reduced by the boundary roughness of the grouted and ungrouted stone transition. When the Tucson Diversion Channel and Santa Cruz River were both discharging their design quantities, the flow from Santa Cruz River did not totally submerge the flow from the Tucson Diversion Channel. Flow throughout the transition was quite stable. When only the Tucson Diversion Channel was discharging its design quantity, a jump was formed at the end of the ungrouted stone reach but did not appreciably affect flow conditions in the transition. No unsatisfactory waves or flow conditions developed in the excavated channel downstream from the jump.

Looking upstream, Tucson Diversion Channel and Santa Cruz River

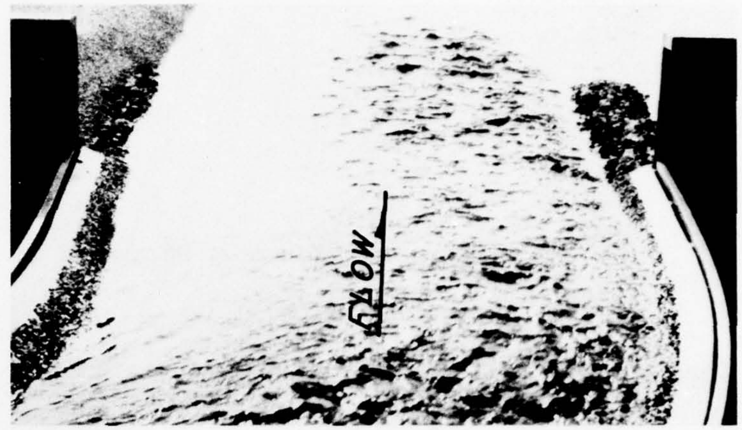


Looking downstream, Tucson Diversion Channel and Santa Cruz River

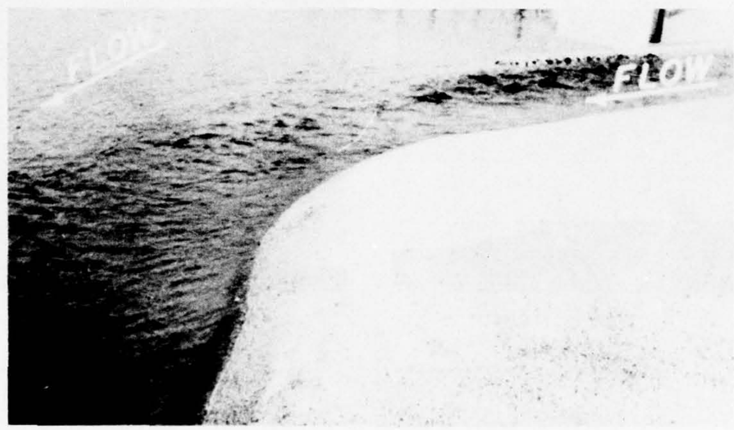


Looking downstream, Tucson Diversion Channel

Photo 15. Final design, Tucson Diversion Channel and Santa Cruz River



Looking downstream



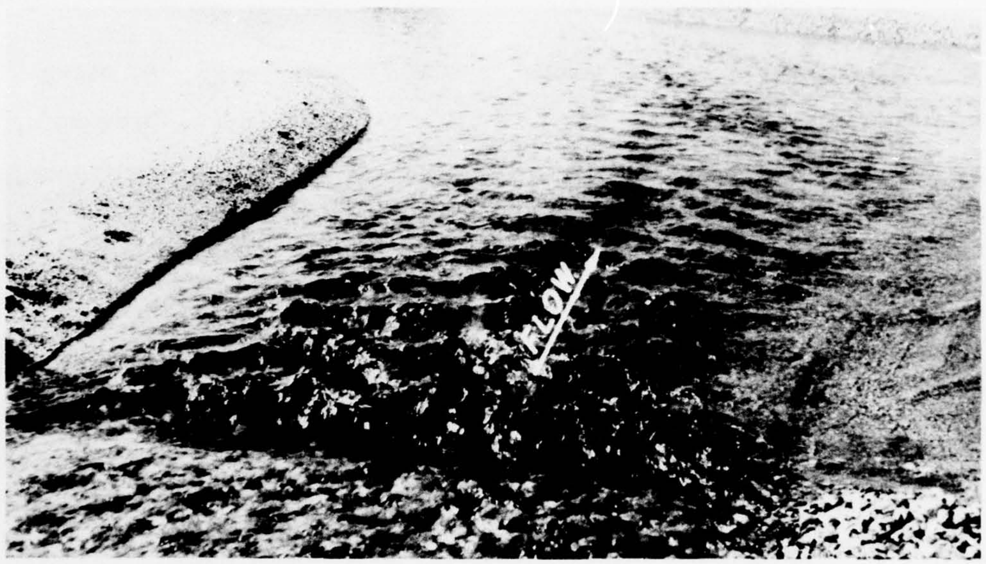
Looking upstream

Photo 16. Final design flow conditions, Test 1; discharge: Tucson Diversion Channel 17,800 cfs, Santa Cruz River 24,600 cfs

Looking downstream

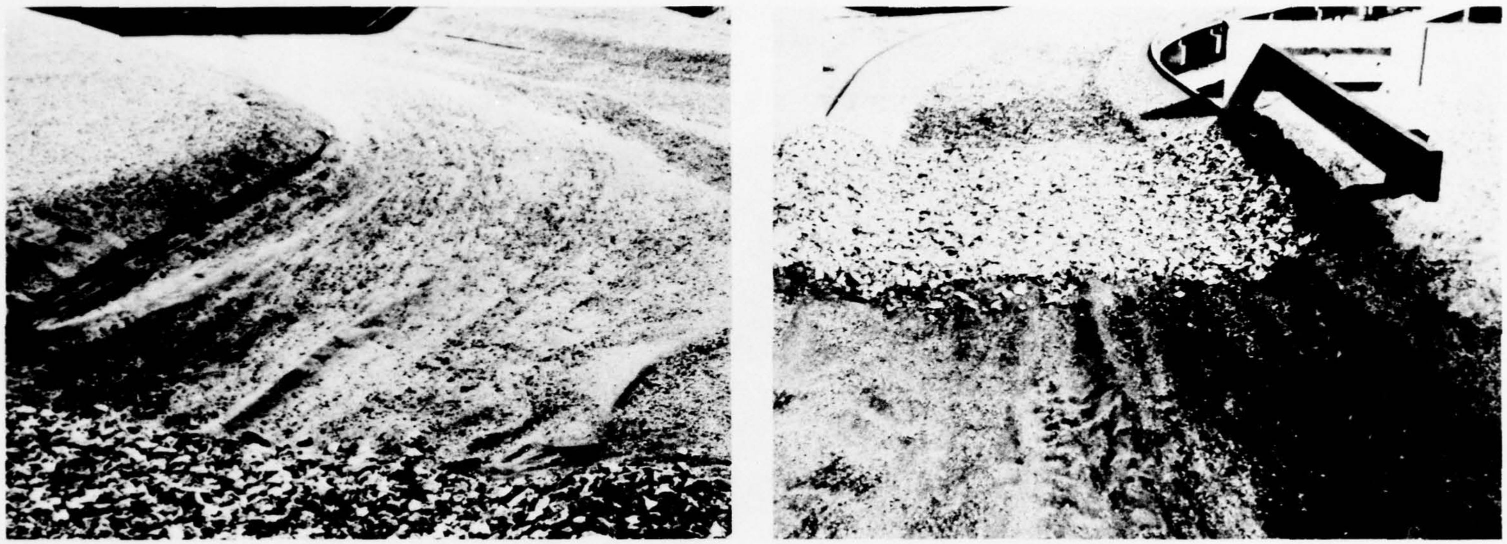
Looking upstream

Photo 17. Final design, Test 1, scour resulting from hydrograph run



Looking at downstream end of transition

Photo 18. Final design, Test 2; discharge:
Tucson Diversion Channel 17,800 cfs,
Santa Cruz River 0 cfs



Looking downstream — Looking upstream

Photo 19. Final design, Test 2, scour resulting from hydrograph run

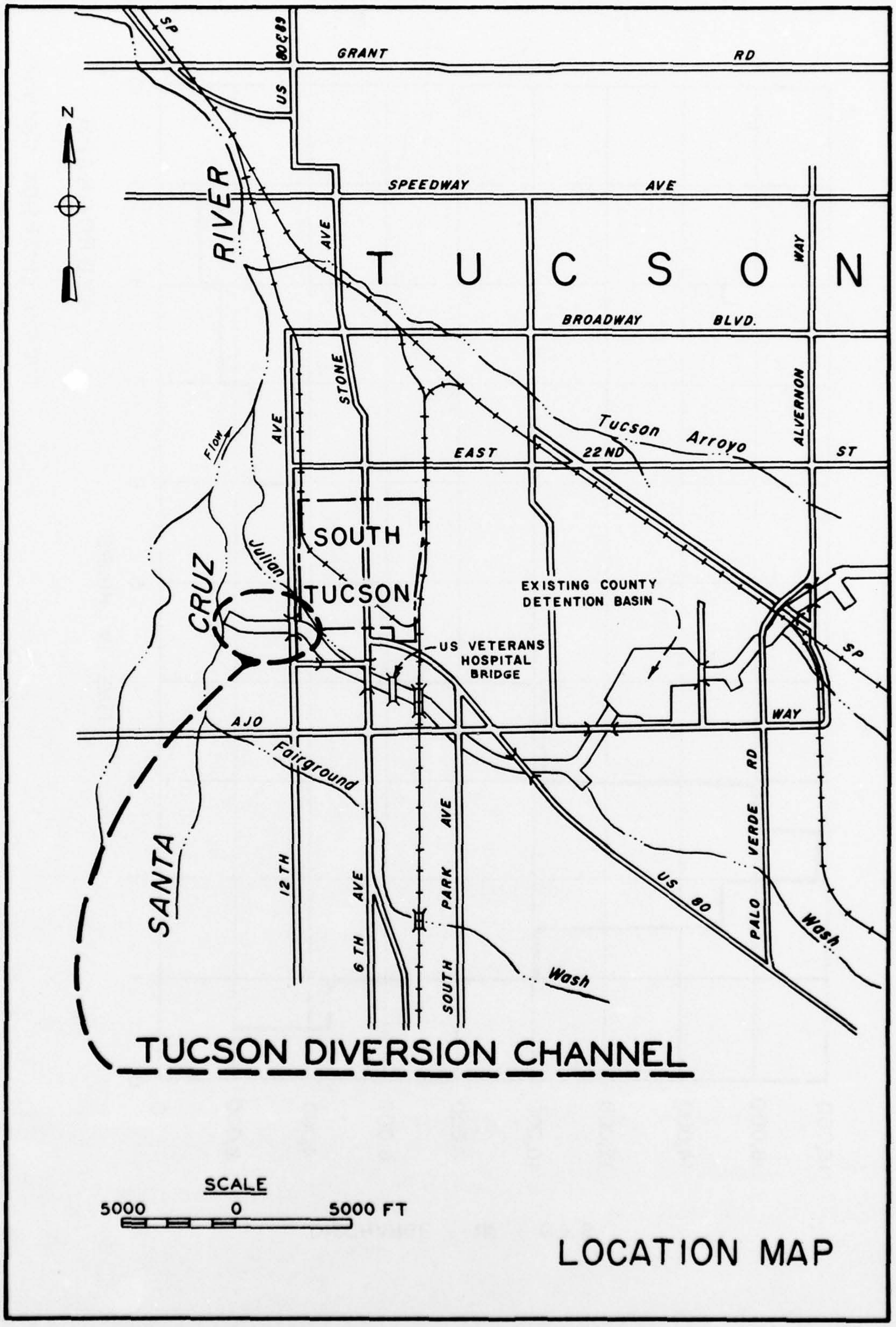
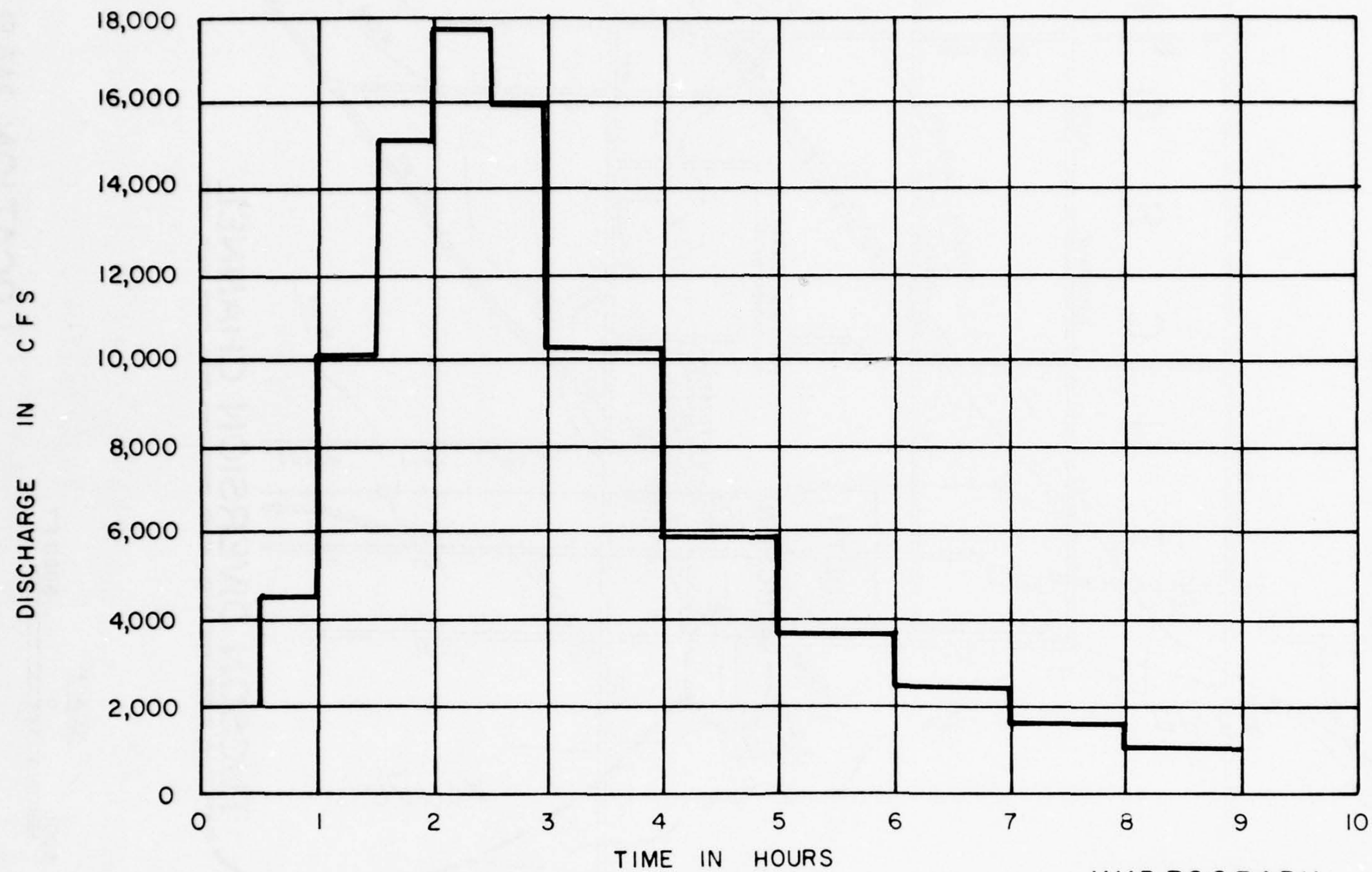

SP
US 80 & 89
GRANT
RD
N
SPEEDWAY
AVE
RIVER
AVE
TUCSON
WAY
BROADWAY
BLVD.
STONE
AVE
Flow
Tucson Arroyo
ALVERNON
EAST
22 ND
ST
SOUTH
TUCSON
CRUZ
Julian
EXISTING COUNTY
DETENTION BASIN
US VETERANS
HOSPITAL
BRIDGE
SP
AJO
WAY
Fairground
RD
SANTA
12 TH
6 TH
AVE
PARK
AVE
SOUTH
US 80
VERDE
PALO
Wash
Wash
TUCSON DIVERSION CHANNEL
SCALE
5000
0
5000 FT
LOCATION MAP

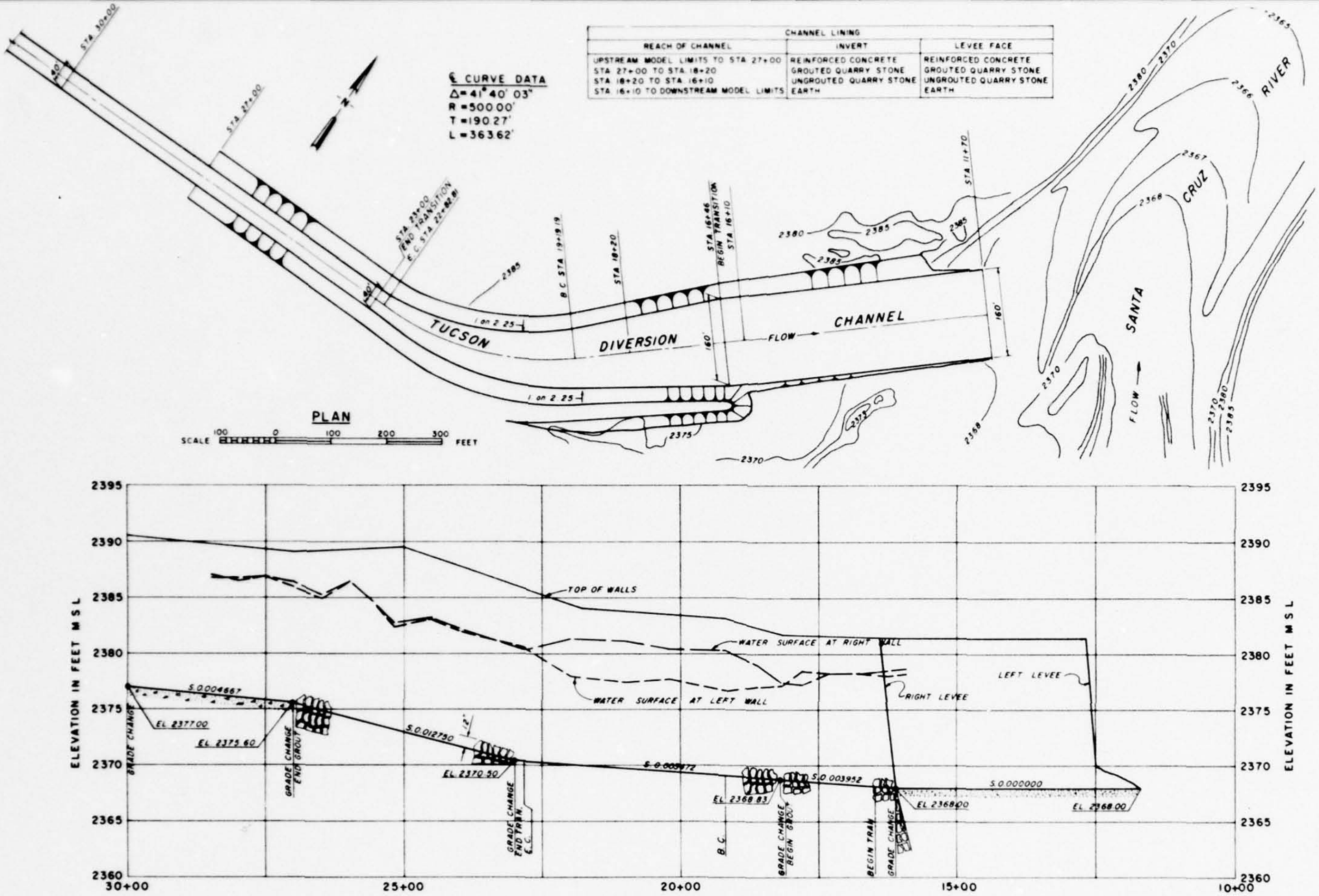
PLATE 17



HYDROGRAPH

TUCSON DIVERSION CHANNEL

PLATE 18



REACH OF CHANNEL	CHANNEL LINING: INVERT	CHANNEL LINING: LEVEE FACE
UPSTREAM MODEL LIMITS TO STA 27+00	REINFORCED CONCRETE	REINFORCED CONCRETE
STA 27+00 TO STA 18+20	GROUTED QUARRY STONE	GROUTED QUARRY STONE
STA 18+20 TO STA 16+10	UNGROUTED QUARRY STONE	UNGROUTED QUARRY STONE
STA 16+10 TO DOWNSTREAM MODEL LIMITS	EARTH	EARTH

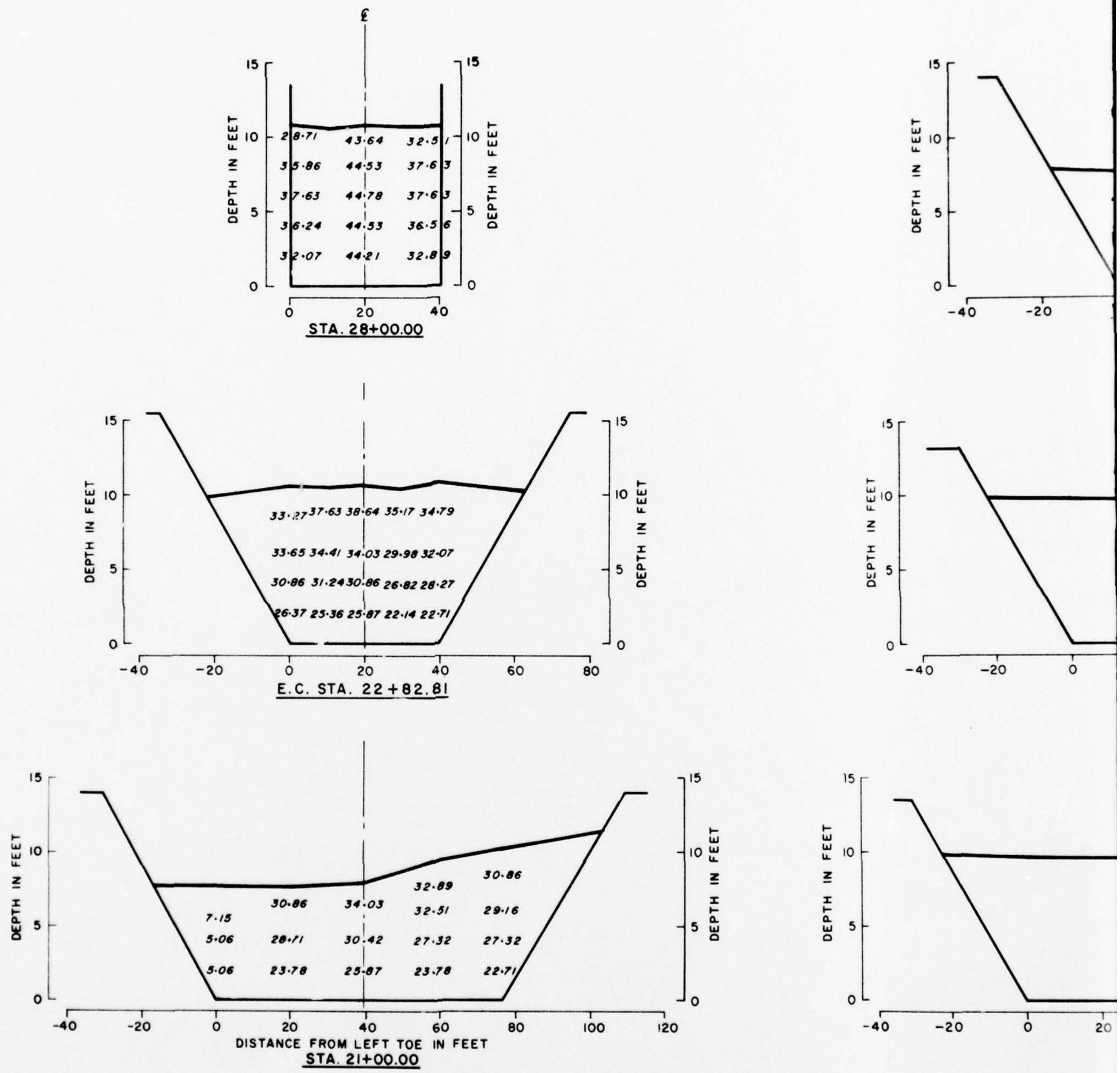
WATER-SURFACE PROFILES

DISCHARGES

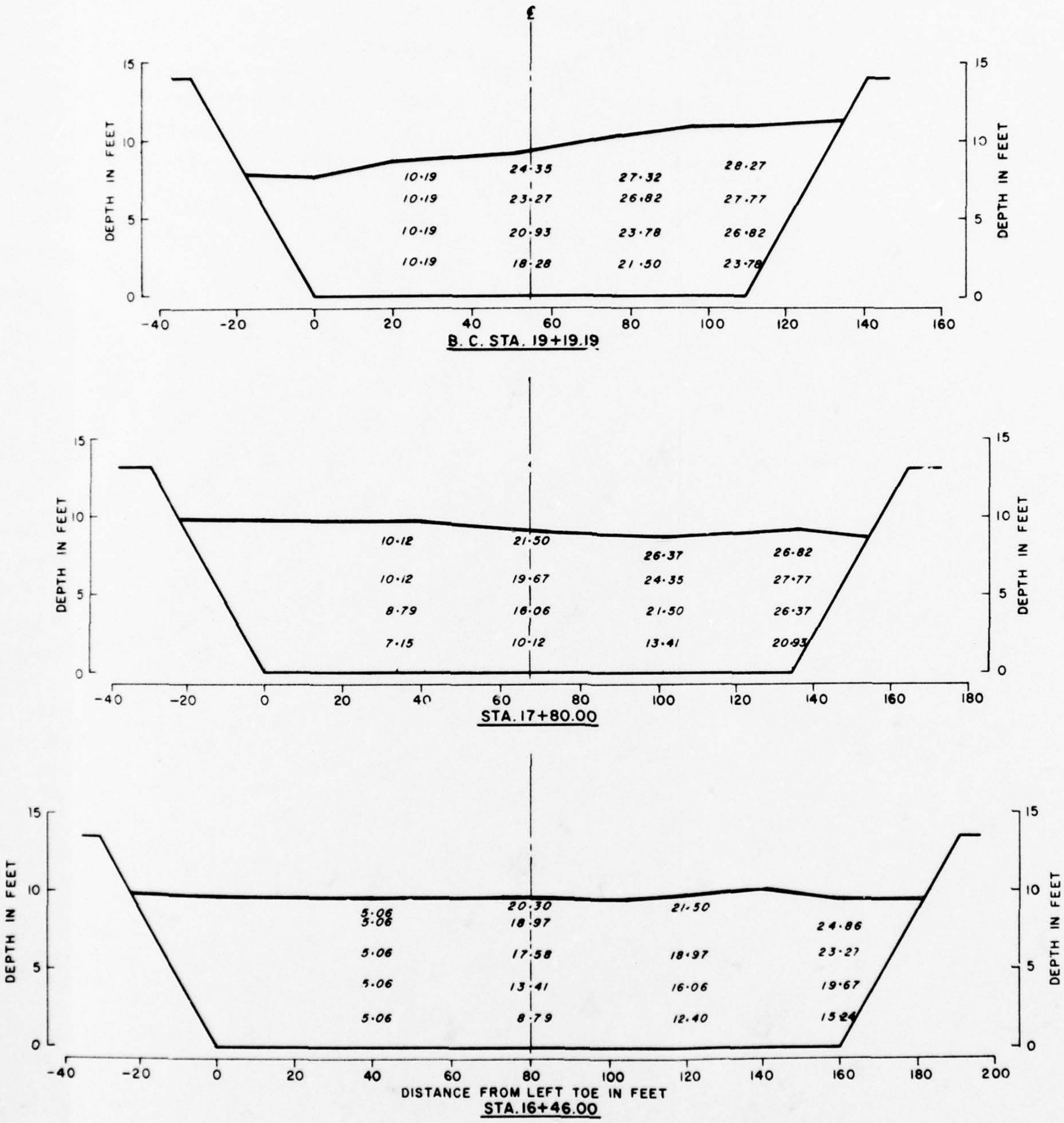
TUCSON DIVERSION CHANNEL 17,800 CFS
SANTA CRUZ RIVER 24,600 CFS

MODEL DETAIL AND WATER-SURFACE PROFILES

FINAL DESIGN TEST NO. 1

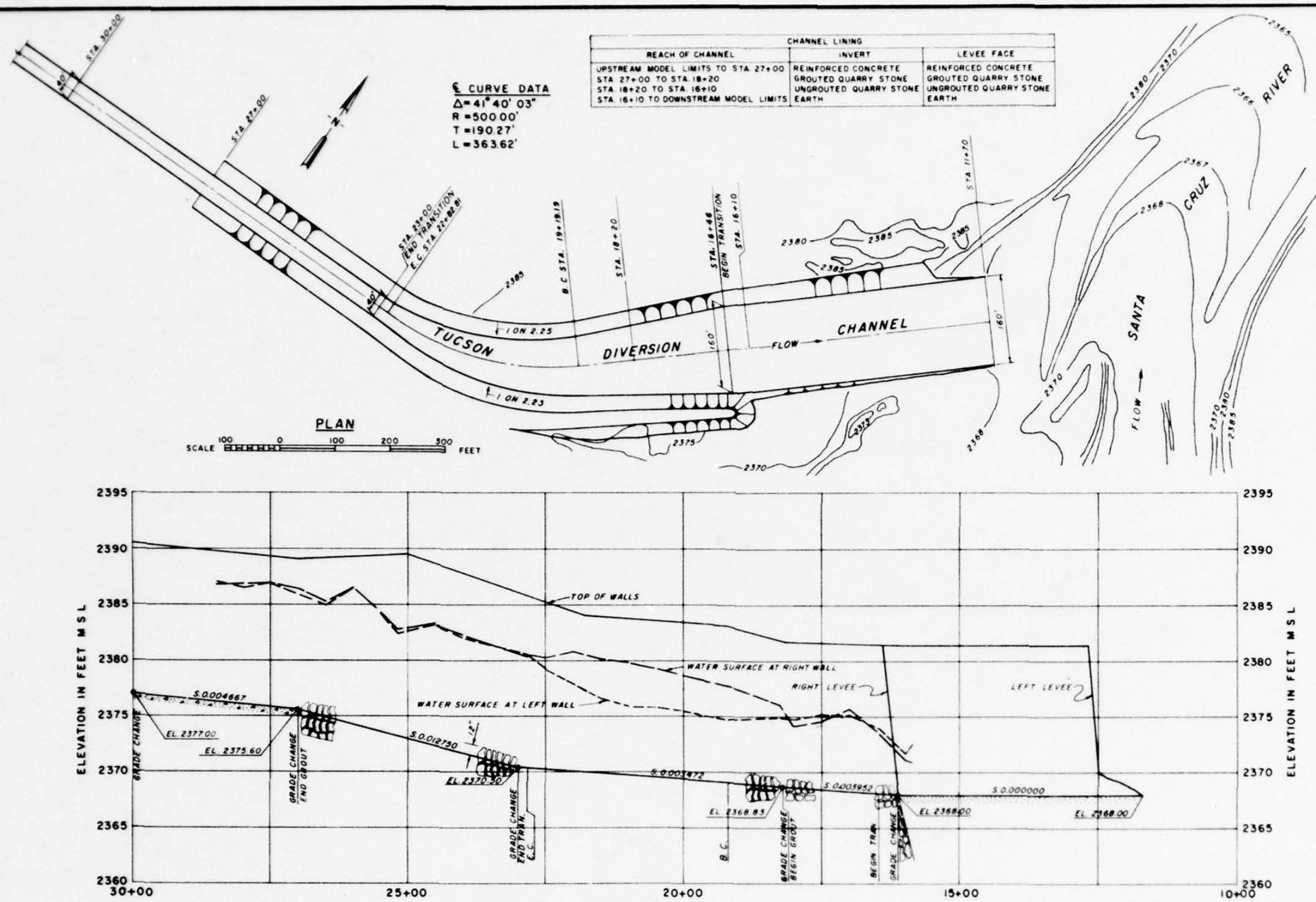


NOTE:
ALL SECTIONS SHOWN LOOKING DOWNSTREAM.
VELOCITIES ARE IN FEET PER SECOND.
DISCHARGE IN SANTA CRUZ RIVER = 24,600 CFS.



VELOCITY DISTRIBUTION
TUCSON DIVERSION CHANNEL
FINAL DESIGN TEST NO. 1
DISCHARGE 17,800 CFS

2



CHANNEL LINING		
REACH OF CHANNEL	INVERT	LEVEE FACE
UPSTREAM MODEL LIMITS TO STA. 27+00	REINFORCED CONCRETE	REINFORCED CONCRETE
STA. 27+00 TO STA. 18+20	GROUTED QUARRY STONE	GROUTED QUARRY STONE
STA. 18+20 TO STA. 16+10	UNGROUTED QUARRY STONE	UNGROUTED QUARRY STONE
STA. 16+10 TO DOWNSTREAM MODEL LIMITS	EARTH	EARTH

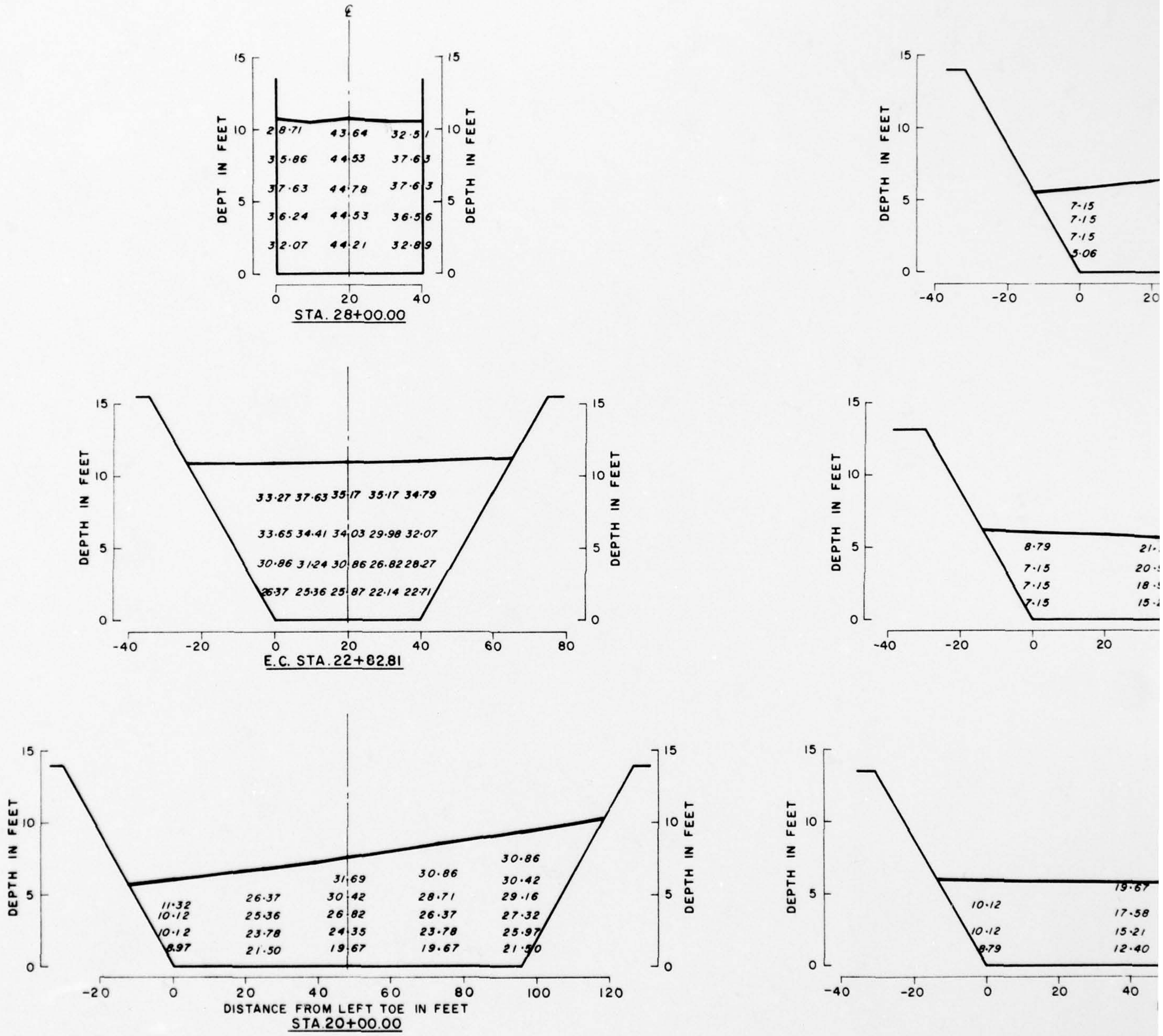
WATER-SURFACE PROFILES

DISCHARGES

TUCSON DIVERSION CHANNEL 17,800 CFS
SANTA CRUZ RIVER NONE

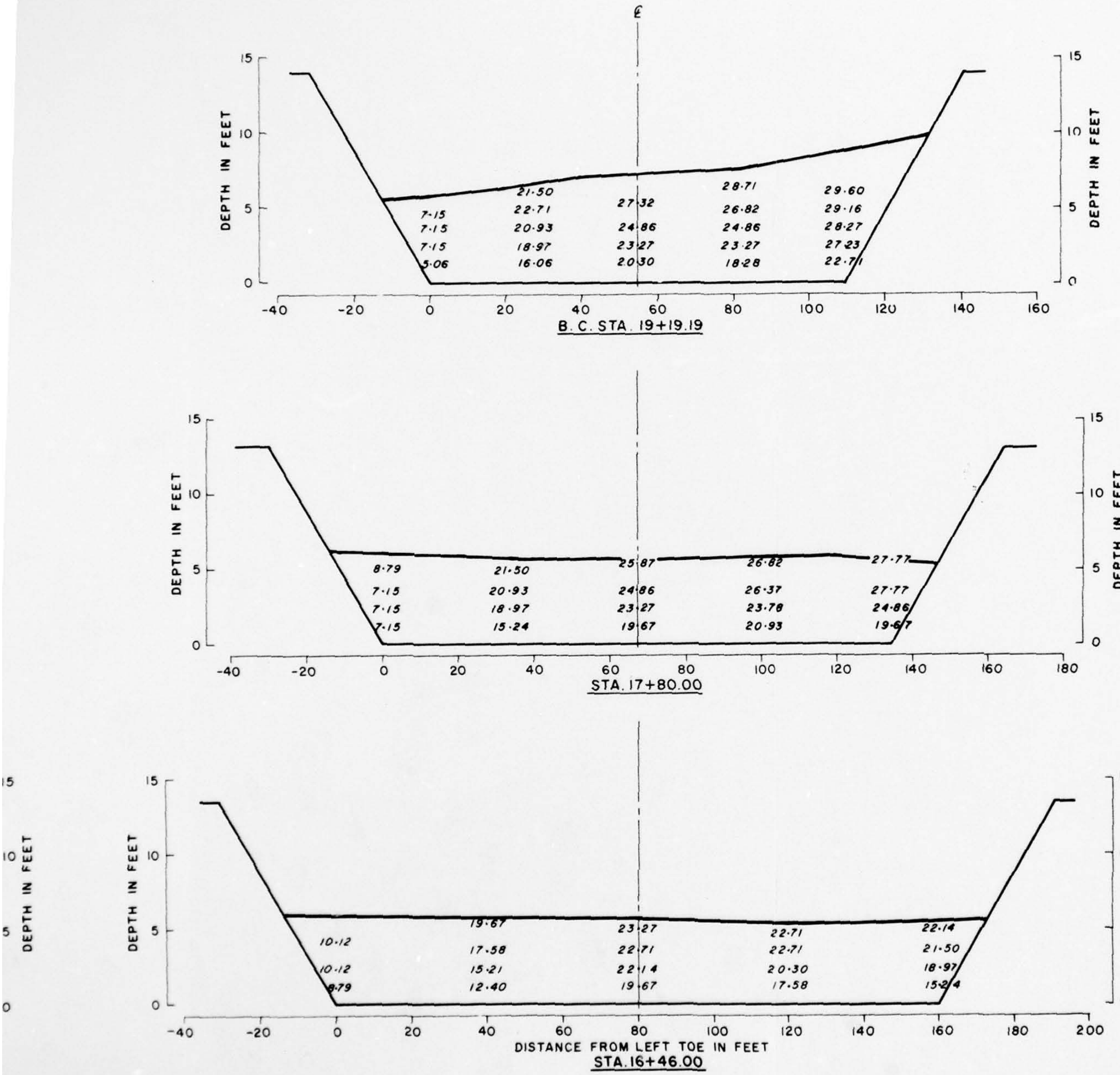
MODEL DETAIL AND WATER-SURFACE PROFILES

FINAL DESIGN TEST NO. 2



NOTE:

ALL SECTIONS SHOWN LOOKING DOWNSTREAM.
VELOCITIES ARE IN FEET PER SECOND.
NO FLOW IN SANTA CRUZ RIVER.



VELOCITY DISTRIBUTION
TUCSON DIVERSION CHANNEL
FINAL DESIGN TEST NO. 2

DISCHARGE 17,800 CFS

TRILBY WASH DETENTION BASIN AND OUTLET CHANNEL

The Prototype

42. Trilby Wash Detention Basin is an earth-fill structure located northwest of Luke Air Force Base near Highways 60, 70, and 89 (Plate 23). The crest of the dam, el 1361.0, is about 50,100 ft long. The outlet works is an uncontrolled reinforced-concrete structure located near the north end of the dam about 5000 ft from the left abutment. Plan and profile of the outlet structure are shown in Plate 24. The overall length of the outlet structure is 188.33 ft which includes a 20-ft-wide entrance channel, a transition section, and a 30-ft-wide stilling basin. The invert elevation at the entrance of the outlet structure is 1335.0 and slopes to 1333.80 at the entrance of the transition section. A reinforced-concrete breastwall is located 65.5 ft from the upstream end of the structure and limits the channel to an 11-ft-high by 20-ft-wide rectangular orifice. The size of orifice was necessary to obtain the desired discharge of 4450 cfs at a pool elevation of 1354.0. The transition extends downstream from sta 10+16 for a distance of 35 ft. At the downstream end of the transition the base width becomes 30 ft. The invert slopes to el 1320.0 through the transition. The stilling basin is 30 ft wide by 65 ft long with an invert elevation of 1320.0 for the entire length of the stilling basin. Two rows of 3-ft-wide by 3-ft-high baffle blocks, spaced on 6-ft centers, are provided 32 ft and 43 ft, respectively, from the upstream end of basin. A 1.5-ft-high end sill is constructed at the downstream end of the stilling basin. A trapezoidal transition channel, flaring from a bottom width of 30 ft at sta 11+16 to 60 ft at sta 11+91, with 1V-on-2.50H side slopes is provided at the downstream end of the basin. The transition side slopes and invert are riprap-lined with maximum size stone weighing 2000 lb. The trapezoidal channel downstream from the transition has a natural earth invert and levees.

The Model

43. The model was constructed to an undistorted, model-to-prototype scale ratio of 1:18. The prototype structures reproduced in the model were the outlet works, stilling basin, and approximately 300 ft of outlet channel downstream of the stilling basin. Part of the detention basin area was simulated by a wooded forebay equipped with baffles to ensure tranquil flow conditions.

44. The entire model, with the exception of the outlet channel, was constructed of marine plywood. The portion of the model representing the outlet channel downstream of the stilling basin was molded of sand and 1-1/2-in. maximum size rock. The baffle blocks were installed with screws to facilitate expedient alterations.

Test and Results

45. The original design called for an ungated outlet to pass a discharge of 4450 cfs at a pool elevation of 1354.0. The hydraulic computations indicated that this requirement could be met by a 10-ft-high by 20-ft-wide opening between the invert of the flume and the bottom of the breastwall. The hydraulic computations were based on standard methods of computing flow in open channels and orifices. Manning's formula was used in the computation of open-channel flow and the general formula, $Q = CA\sqrt{2gh}$, was used for orifice flow. A roughness coefficient of $n = 0.012$ was used for the concrete flume and a discharge coefficient of $C = 0.878$ was used for orifice flow computations for the discharge rating shown in Tables 2 and 3 and the discharge rating curve is shown in Plate 25.

46. The original design as shown in Plate 24 and Photo 20 called for a 10-ft-high by 20-ft-wide rectangular opening. Tests of the outlet works and stilling basin were concerned primarily with assuring (a) satisfactory flow condition, and (b) passage of 4450 cfs at a pool elevation of 1354.0. Initial test results indicated that for open channel conditions the model discharge was about 10 percent less than the

computed discharge due to the fact that entrance contraction was greater than contemplated. For controlled orifice flow, the entrance contraction combined with considerable vortical action at the breastwall made it necessary to increase the size of the orifice to obtain the desired discharge of 4450 cfs at a pool elevation of 1354.0. This was accomplished by raising the bottom of the breastwall to elevation from 1344.1 to 1345.1, giving an 11-ft by 20-ft opening. Table 4 gives the model discharges for the various pool elevations; results are plotted in Plate 25 in comparison with computed discharges.

47. Tests were made to determine the adequacy of the structure under two tailwater conditions. The tests were conducted with a design discharge of 4450 cfs and tailwater elevations of 1334.5 and 1335.5 For a tailwater elevation of 1334.5, the flow produced a very unstable hydraulic jump. This tailwater produced an unsteady flow in which the jump fluctuated in position and height above the end sill. This is objectionable since the disturbances caused by the pulsating jump do not have a chance to dissipate before they reach the erodible section. Photo 21 shows the jump in its highest extremity and a view of the 4450-cfs flow as it passed through the rectangular orifice down the curved chute and stilling basin, through the rock-lined transition, and thence down the outlet channel. Scour occurred immediately downstream from the end sill and progressed to the downstream limit of the rock-lined transition. The scour hole was quite deep as shown in Photo 22.

48. Tests with the tailwater at el 1335.5 indicated a favorable hydraulic jump. The flow conditions are shown in Photo 23. The jump was stable and little wave action was propagated downstream. Test results (Photo 24) show that this tailwater produced a lesser scour hole than the previous test. The scouring action was rather weak and the end sill was well protected.

Table 2

Computed Discharge Rating for Orifice and Critical Depth Flow

Orifice Flow				Critical Depth Flow*				
Pool Elevation	h Control el 1344.1	$\sqrt{h}$	$Q = 1410 \sqrt{h}$	d_c	d_c^3	$d_c^3 g$	$q = \sqrt{d_c^3 g}$	$Q = 20\ q$
1345.0	0.9	0.949	1338	0.5	0.125	4.0	2.0	40
1346.0	1.9	1.378	1943	1.0	1.000	32.2	5.67	113
1347.0	2.9	1.703	2401	2.0	8.000	257.6	16.05	321
1348.0	3.9	1.975	2785	3.0	27.00	869.0	29.48	590
1349.0	4.9	2.214	3122	4.0	64.00	2061.0	45.40	908
1350.0	5.9	2.429	3425	5.0	125.0	4025.0	63.44	1269
1351.0	6.9	2.627	3704	6.0	216.0	6955.0	83.40	1668
1352.0	7.9	2.811	3964	7.0	343.0	11045.0	105.10	2102
1353.0	8.9	2.983	4206	9.0	729.0	23474.0	153.21	3064
1354.0	9.9	3.146	4436	11.0	1331.0	42858.0	207.02	4140
1355.0	10.9	3.302	4656					
1356.0	11.9	3.450	4865					
1357.0	12.9	3.592	5065					
1358.0	13.9	3.73	5260					
1361.0	16.9	4.11	5800					

Note: Assume $K_e = 0.30$

$$C = \frac{1}{\sqrt{K_e + 1}} = 0.878$$

$$Q = CA\sqrt{2gh} \qquad \text{(10-ft × 20-ft opening)}$$

$$Q = 0.878 \times A \times 8.02 \sqrt{135.0 - 13441.1} \qquad Q = 1410 \sqrt{h}$$

$$A = \frac{4450}{22.15} = 200.9 \text{ sq ft}$$

* Control section is at sta 9+27.67 for Q = 0 cfs and moves downstream as Q increases.

Table 3

Computed Discharge Rating for Open-Channel Flow

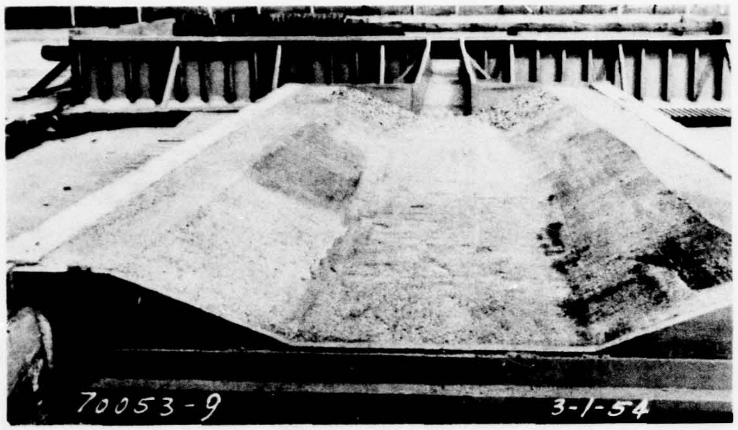
Q_c	d_c	A_c	P_c	R_c	$\frac{1}{2.21 R_c^{4/5}}$	V_c	h_{v_c}	f_c	sta for d_c	Pool Elevation
40	0.50	10.00	21.00	0.48	1.200	4.00	0.25	0.00278	9+30	1335.78
113	1.00	20.00	22.00	0.91	0.514	5.65	0.50	0.00236	9+31	1336.55
321	2.00	40.00	24.00	1.67	0.229	8.02	1.00	0.00212	9+34	1338.08
590	3.00	60.00	26.00	2.31	0.148	9.83	1.50	0.00206	9+36	1339.63
908	4.00	80.00	28.00	2.86	0.112	11.35	2.00	0.00208	9+39	1341.16
1269	5.00	100.00	30.00	3.33	0.0911	12.69	2.50	0.00211	9+41	1342.71
1668	6.00	120.00	32.00	3.75	0.0777	13.90	3.00	0.00216	9+44	1344.24
2102	7.00	140.00	34.00	4.12	0.0686	15.00	3.50	0.00222	9+46	1345.80
3064	9.00	180.00	38.00	4.74	0.0569	17.00	4.50	0.00237	9+51	1348.88
4140	11.00	220.00	42.00	5.24	0.0498	18.80	5.50	0.00254	9+56	1351.96

Note: Assume loss from critical depth section to pool = $0.15\ h_{v_c}$; pool elevation = invert el (at section of critical depth) + $d_c + h_{v_c} + 0.15\ h_{v_c}$; for all above discharges invert slope $= 0.0136 > f_c$, $n = 0.012$.

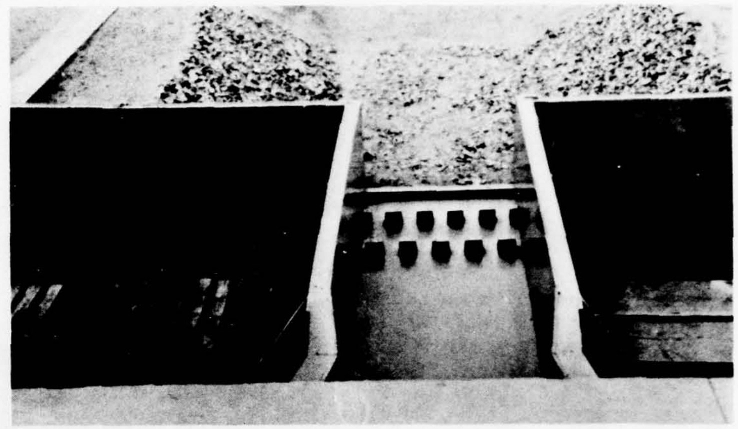
Table 4

Discharge Rating Observed in Model

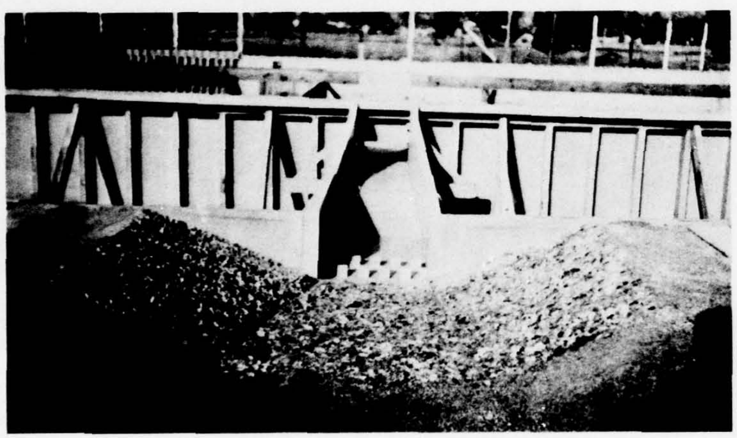
Pool Elevation ft	Discharge Actual cfs	Discharge Adjusted cfs
1335.0	0	0
1337.0	0	156
1338.3	0	340
1338.7	399	398
1339.7	591	591
1340.8	825	822
1341.8	1045	1047
1342.8	1265	1275
1343.8	1512	1509
1344.8	1788	1765
1346.0	2076	2090
1346.9	2337	2337
1347.9	2612	2612
1348.8	2887	2870
1350.1	3300	3290
1351.0	3547	3575
1352.2	3987	3990
1353.0	4262	4280
1354.2	4675	4655
1357.1	4812	4830
1359.2	5087	5065
1359.9	5155	5155
1360.8	5225	5245



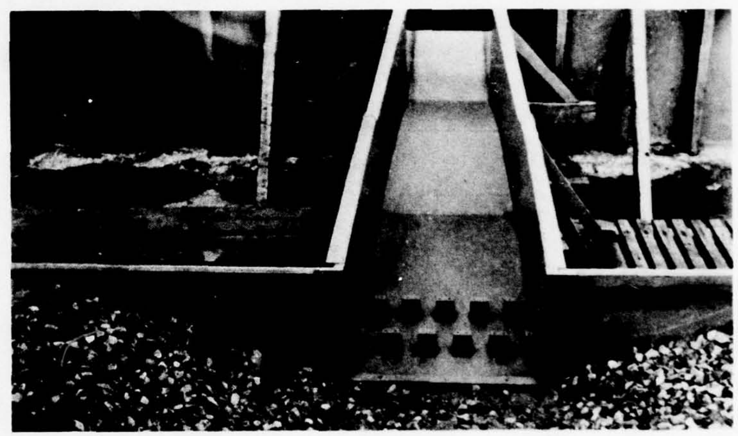
Looking downstream at baffle blocks and rock-lined transition



General view, looking upstream

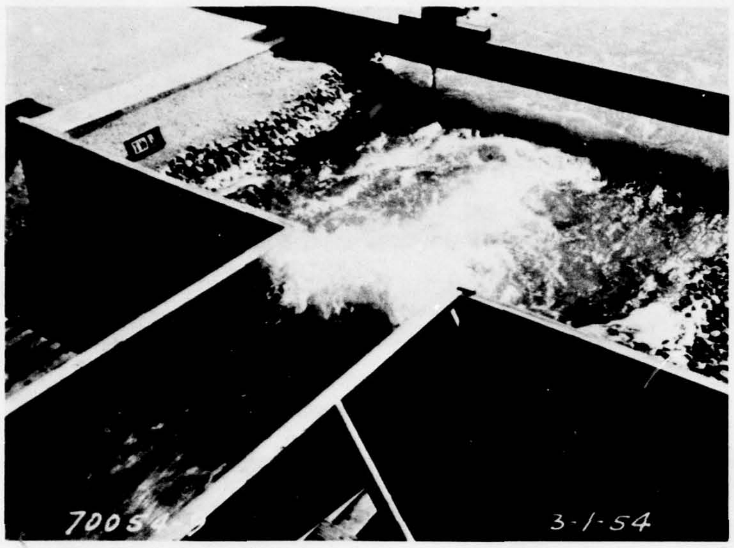


Looking upstream at rock-lined transition and baffle blocks

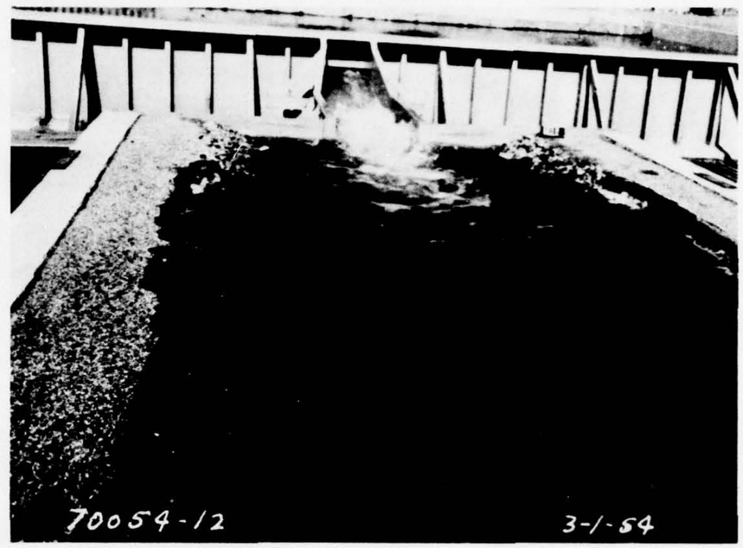


Looking upstream at baffle blocks

Photo 20. Original design stilling basin and outlet channel, Trilby Wash Detention Basin

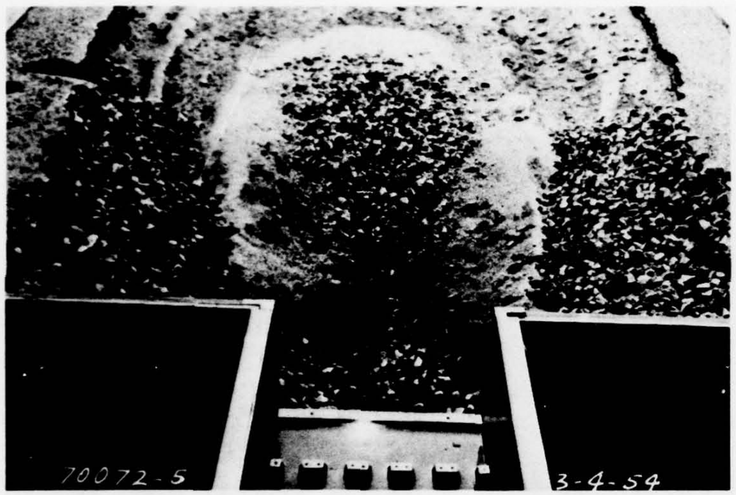


Looking downstream

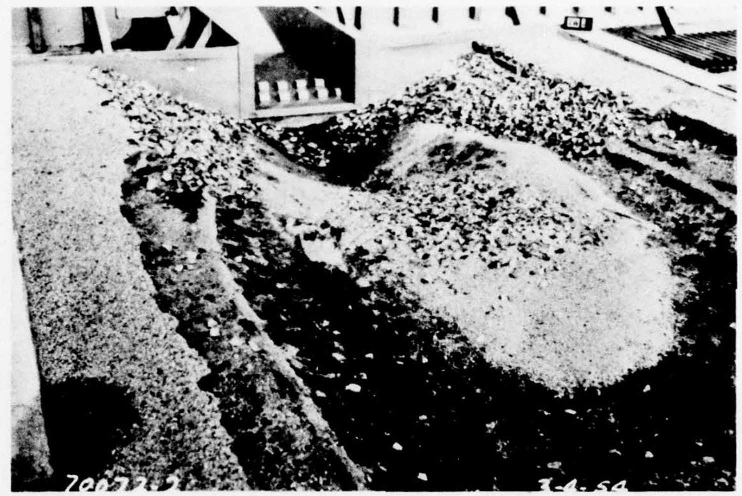


Looking upstream

Photo 21. Stilling basin and outlet channel flow conditions, Trilby Wash Detention Basin; discharge 4450 cfs, tailwater elevation 1334.5 msl



Looking downstream



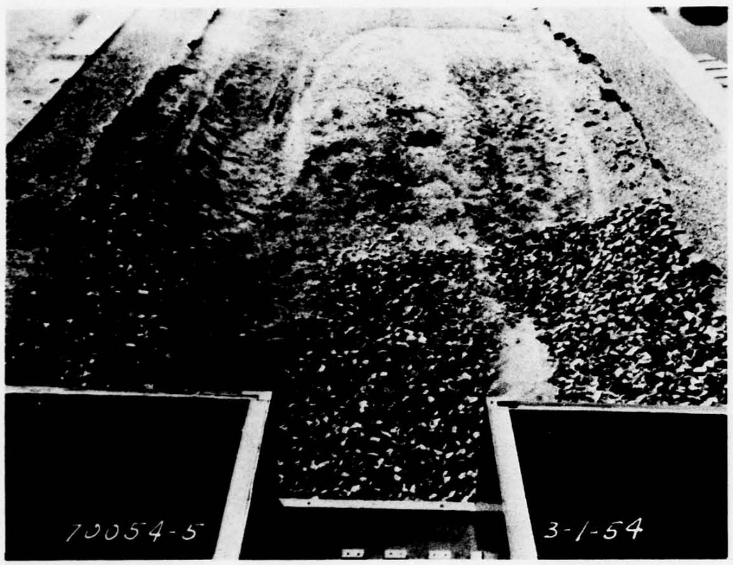
Looking upstream

Photo 22. Scour after 1-hr test, Trilby Wash Detention Basin; discharge 4450 cfs, tailwater elevation 1334.5 msl

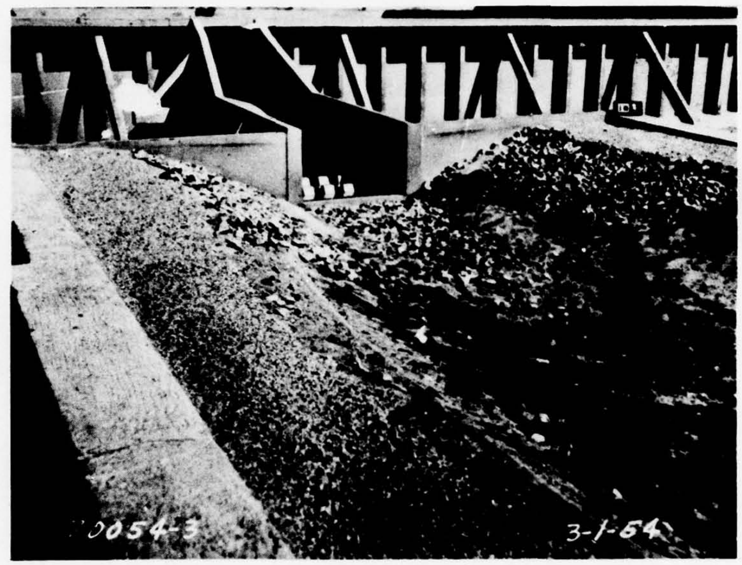
Looking upstream. Stable flow condition over end sill

General view looking upstream

Photo 23. Stilling basin and outlet channel flow conditions, Trilby Wash Detention Basin; discharge 4450 cfs, tailwater elevation 1335.5 msl



Looking downstream



Looking upstream

Photo 24. Scour after 1-hr test, Trilby Wash Detention Basin; discharge 4450 cfs, tailwater elevation 1335.5 msl

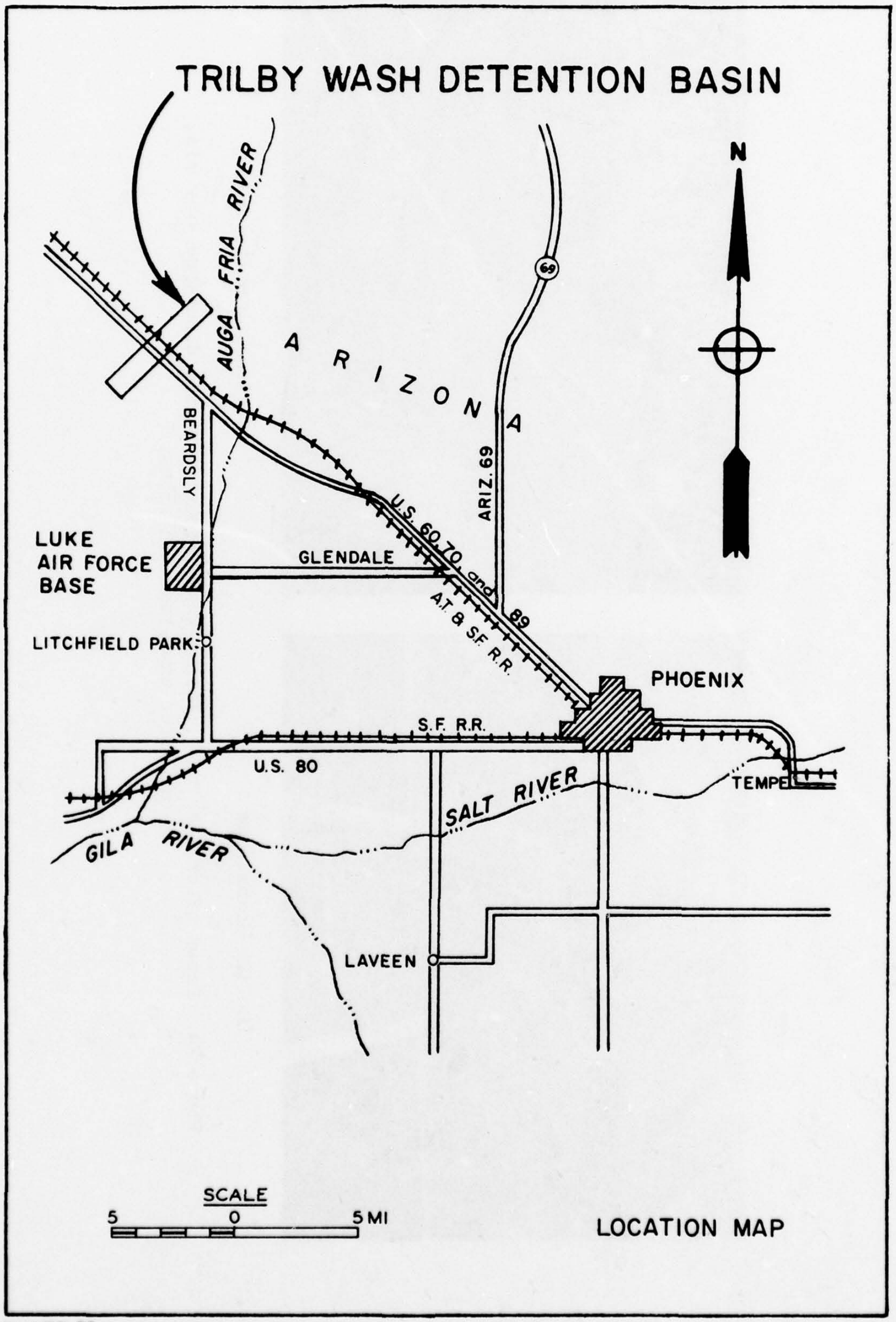
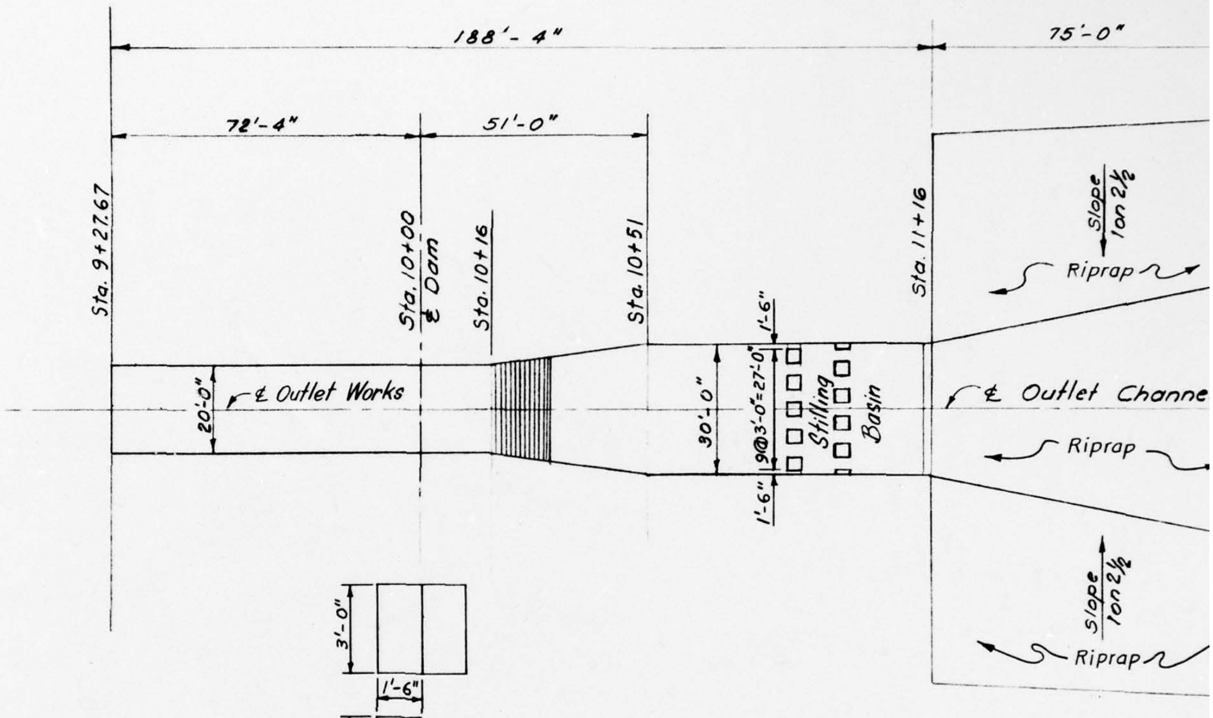
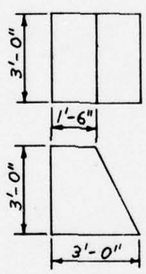

TRILBY WASH DETENTION BASIN
N
AUGA FRIA RIVER
ARIZONA
BEARDSLY
ARIZ. 69
69
U.S. 60,70 and 89
A.T. & S.F. R.R.
LUKE
AIR FORCE
BASE
GLENDALE
LITCHFIELD PARK
PHOENIX
S.F. R.R.
U.S. 80
TEMPE
SALT RIVER
GILA RIVER
LAVEEN
SCALE
5
0
5 MI
LOCATION MAP

PLATE 23





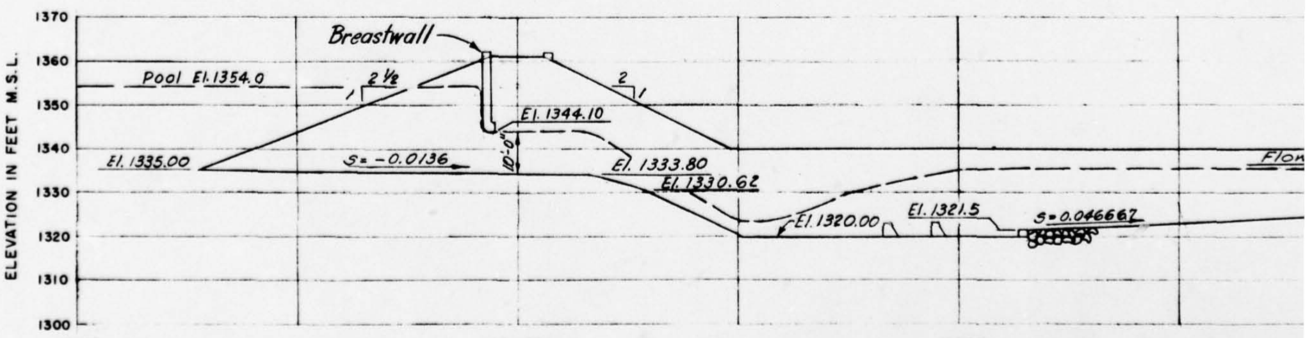
PLAN

SCALE: 1 IN = 20 FT.

SCALE: 10 0 10 20 30 40 50 FEET

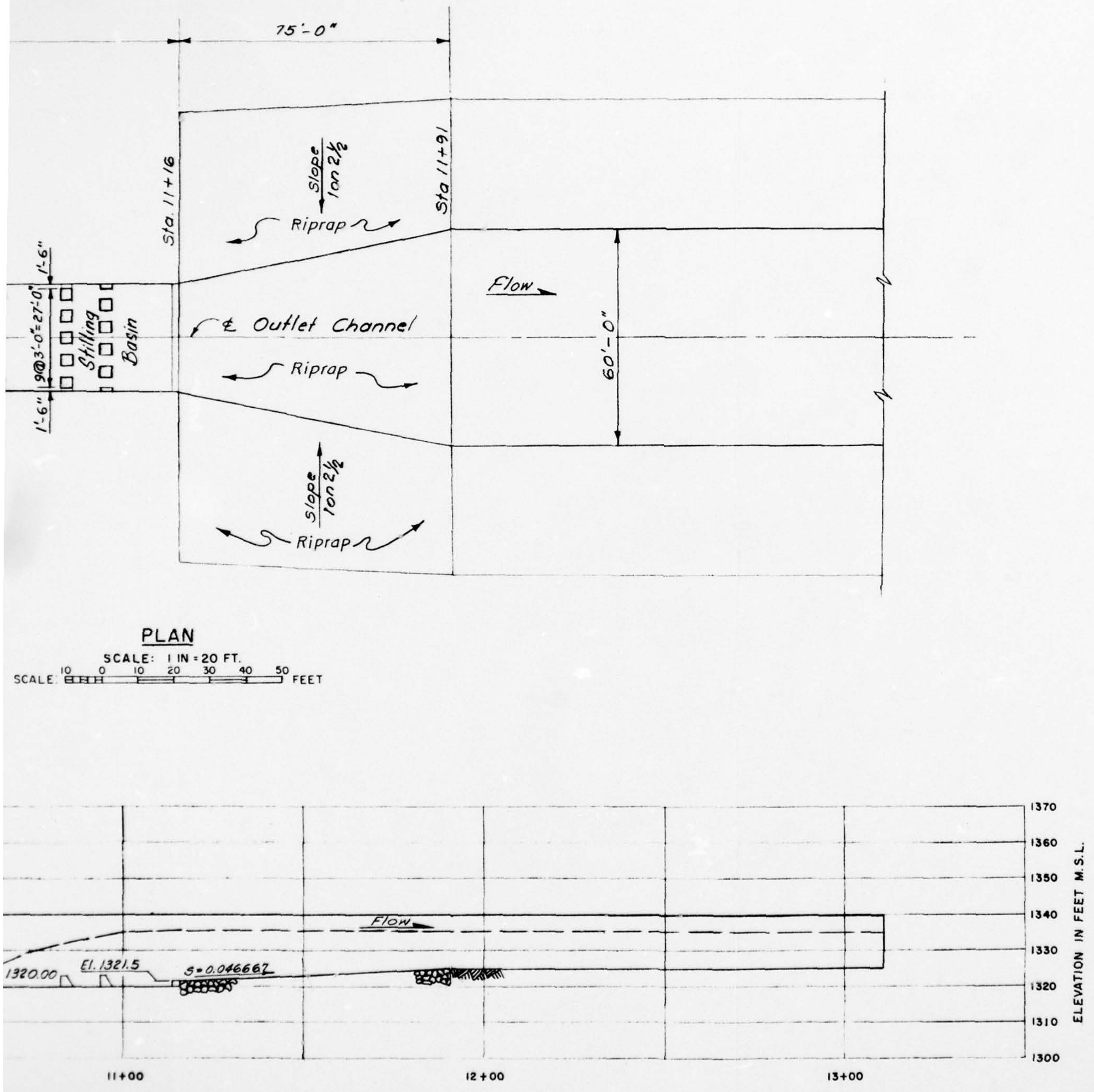
BAFFLE BLOCK DETAIL

SCALE: 30 0 30 60 90 FEET



WATER-SURFACE PROFILE

SCALE: 1 IN = 20 FT.

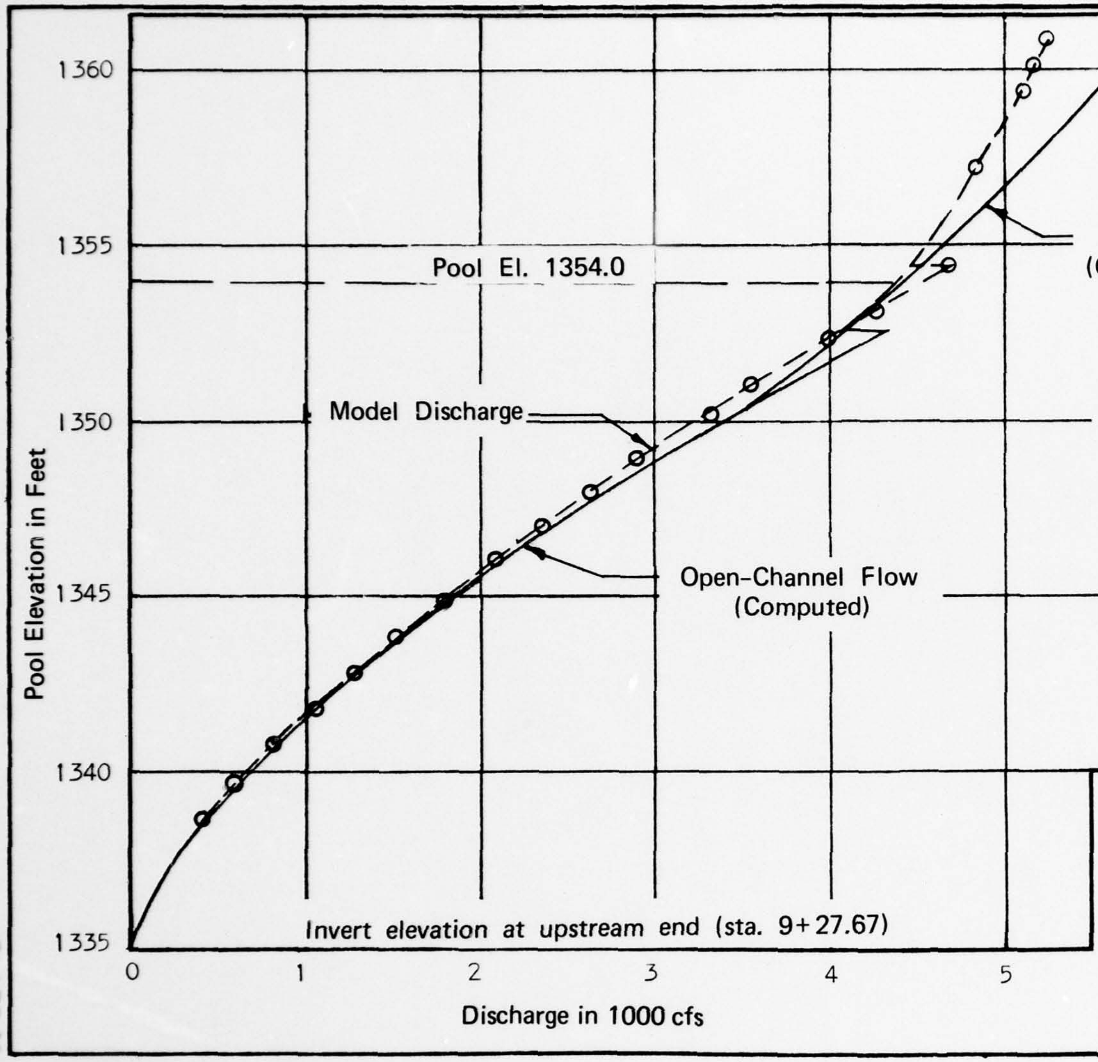


ATER-SURFACE PROFILE

SCALE: 1 IN = 20 FT.

TRILBY WASH DETENTION BASIN
OUTLET WORKS AND STILLING BASIN
DISCHARGE 4450 C.F.S.

PLATE 24



TRILBY WASH DETENTION BASIN

Outlet Works Discharge Rating Curves

APPENDIX I

EXTRACTS FROM REPORT

ENTITLED

CHANNEL TRANSITION

FOR

EAST TWIN AND WARM CREEKS

REPORT NO. 1-105

AUGUST 1961

U. S. ARMY ENGINEER DISTRICT, LOS ANGELES

CORPS OF ENGINEERS

LOS ANGELES, CA.

INDEX TO APPENDIX I

The prototype.

CHANNEL TRANSITION FOR EAST TWIN AND WARM CREEKS

The Prototype

1. The channel transition for East Twin and Warm Creeks is located near the southern boundary of the city of San Bernardino, San Bernardino County, California (plate 1). The improved reach of channel upstream from the transition is rectangular in cross section and has a width of 60 ft. The transition, which is trapezoidal in cross section and having 1 on 2.25 levee slopes, extends downstream from station 169+63 for a distance of 1,050 ft. At the downstream end of the transition the base width of the channel becomes 200 ft. The 200-ft base width and trapezoidal cross section are typical for the channel downstream from the end of the transition.

2. Upstream and downstream from the transition the channel will carry a design discharge of 22,000 cfs at depths and velocities ranging from 10.4 ft to 8 ft and 36 fps to 11.1 fps, respectively. The cross sections, material, and coefficient of roughness used in various channel reaches were as follows: (a) the rectangular channel was of reinforced concrete having an "n" value of 0.014, (b) the trapezoidal channel for the transition was paved with a reach of grouted stone and a reach of dumped stone which had an average "n" value 0.040, and (c) the trapezoidal channel downstream from the transition had a natural earth invert and revetted levees and was considered to have an average "n" value of 0.025.

The Model

3. The model was constructed to an undistorted, model-to-prototype scale ratio of 1:48. The prototype reach simulated in the model was as follows: (a) 537 ft of rectangular channel upstream from the transition, (b) the 1,050-ft transition, and (c) 1,213 ft of trapezoidal channel downstream from the transition.

4. The materials used in the construction of the model were wood,

sand, cement, and gravel. The wood, in the form of timbers and plywood, was used to construct that part of the model upstream from the transition. The grouted stone invert of the transition was simulated by cementing gravel particles to the plywood surface, and the part of the invert lined with dumped stone was simulated by using various sizes of crushed stone. The channel downstream from the transition was molded in a bed of sand. In this reach of channel, the dumped stone levees were simulated with roughened concrete.

5. Two methods were applied in the model to simulate the slope of energy gradient of the prototype. One method was to increase the model slope, and the other was to artificially roughen the surface of the model. The first of the above two methods was applied to that part of the model upstream from the transition. The coefficient of roughness of the painted surface used in this part of the model had previously been determined to be 0.0088. Based upon similitude relationship, the above "n" value would be equivalent to a prototype "n" value of 0.017. The "n" value used in design for prototype concrete linings was 0.014. To compensate for the difference in the two prototype coefficients of roughness, the slope of the invert in the model was increased about 10 ft in 1,000 ft. The second method was applied in reproducing the boundary roughness in the grouted stone part of the transition. This was done by cementing particles of gravel to the plywood invert of the model.

6. The part of the model that simulated the natural earth invert of the prototype was molded in sand having a mean diameter of 0.20 mm. To maintain similitude of mass and not destroy the geometric similarity, it was necessary for the stone in the model to have the same specific gravity as the stone in the prototype. The stone used in the model had a specific gravity of 2.64 (assumed equal to prototype) and was graded to provide a uniform distribution of particle size. Crushed stone was used because it more nearly represented the shape of the quarrystone to be used in the prototype.

Description of Tests

7. Comparison of the qualitative results of the tests for the

different designs was made possible by simulating the flow that would occur during the design flood. The magnitude and duration of the design flood are shown by the hydrograph on plate 2. The above-mentioned results were of scour that would occur downstream from the end of the transition. The quantitative data were obtained for a design discharge of 22,000 cfs. These data included the water surface and distribution of velocities within the improved channel.

8. The same length of prototype channel was simulated in the model for each of the designs that was tested. The changes to the models were limited to the length of the dumped stone used for the invert of the transition. The channel cross sections remainded the same for all of the designs tested.

Original Design

9. The layout of the original design is shown in photographs 1 and 2 and on plate 3. For the design discharge of 22,000 cfs flowing in the structure, it can be seen in photographs 3 and 4 that the water surface was not very rough. The velocity of the flow decreased as the design discharge passed through the transition (plate 6). This showed that the boundary roughness, produced by the grouted and dumped stone linings, was sufficient to overcome acceleration of the flow which resulted from the expanding channel cross section. At the downstream end of the transition the flow reached a subcritical velocity of about 11 fps. Photographs 5 and 6 show what is considered to be a minor amount of scour movement of stone at the downstream end of the transition.

10. Hydraulically, the structure was satisfactory. It was therefore feasible to pursue the design from an economic standpoint. Saving could be realized by reducing the length of the invert lining located at the downstream end of the transition. This served as the basis for tests conducted upon the alternative design.

Alternative Design

11. In this design the dumped stone downstream from station

162+88 was reduced in length to 88 ft (photograph 7). The grouted stone lining remained the same length as in the original design.

12. The tests were conducted with a design discharge of 22,000 cfs. Photograph 8 shows that the water surface in the transition was not exceptionally rough. The scour that resulted from the flows which were simulated in accordance with the hydrograph is shown in photographs 9 and 10, on plate 5. Evaluation of the scour indicated that the roughness and length of the transition lining were sufficient to negate the supercritical velocities occurring at the upstream end of the transition.

13. In the above tests the results were satisfactory, therefore, it was decided that future reduction to the length of the invert lining for the transition would be feasible. It was upon this basis that the final design was determined.

Final Design

14. The dumped stone reach was reduced to a length of 375 ft, as shown in photographs 11 and 12, and plate 4. The 375-ft length was made up of a 275-ft length of derrick stone followed by a 100-ft length of toe stone.

15. For the design discharge of 22,000 cfs, the flow within the transition was satisfactory, as shown in photographs 13 and 14. The velocity distribution cross sections shown on plate 6 indicated a fairly even distribution of velocities through the transition. The scour occurring immediately downstream from the stone protection, as a result of the hydrograph run, is shown in photographs 15 and 16, and on plate 5.

Discussion of Results

16. The tests conducted on the models of the original, alternative, and final designs show that the hydraulic characteristics of each structure were satisfactory. Therefore, the acceptance of the best design would be based on economics rather than hydraulics. The choice of

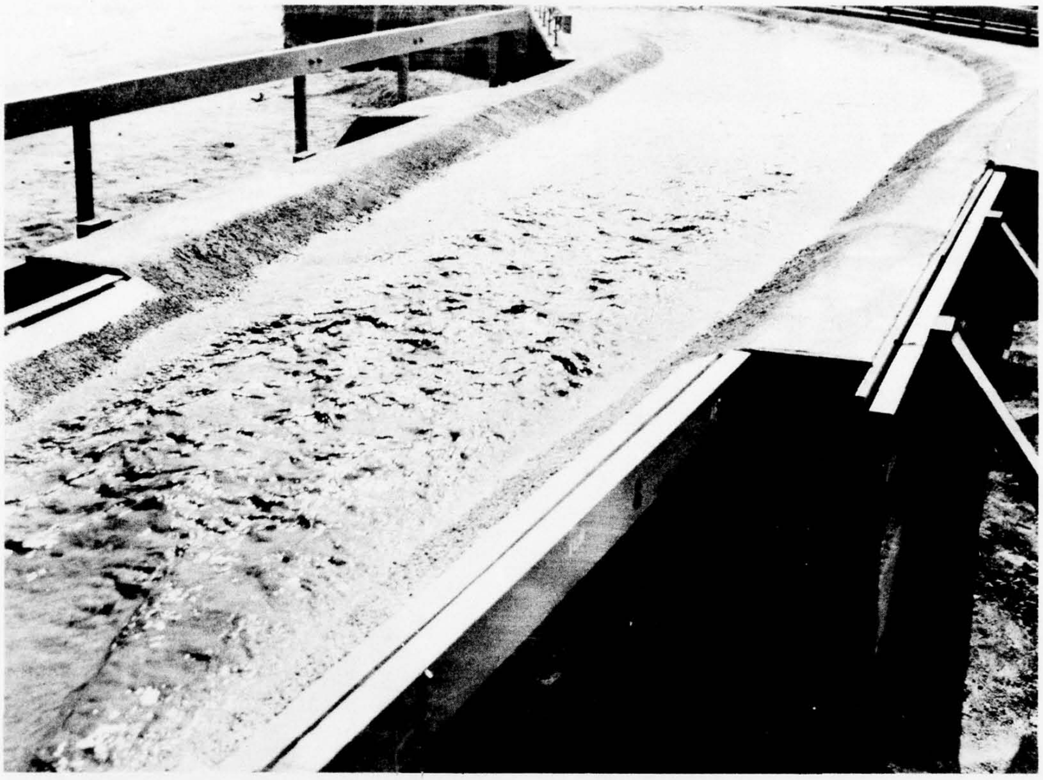
design would be determined by the length of stone lining in the transition. The most noticeable difference between the original and final design is shown by the velocity cross sections. It can be seen that a slightly higher velocity occurs at the downstream end of the stone lining for the final design. The velocity did not increase to the extent that it would cause noticeable change in scour at the downstream end of the transition.

17. A problem common to this type of transition is the lack of uniform distribution of flow in the channel cross section. This occurs as the flow passes through the transition and is identified by the formation of an eddy which extends over the length and greater part of the width of the transition. This eddy forces the flow to pass along side of the transition and causes a greater depth of scour in a localized area at the downstream end of the stone lining. Proper tailwater and rate of flare of the levees in the transition eliminate the abovementioned condition. In general, the best results with this type of transition have been obtained when the rate of flare is not greater than 1 in 15 and there is no excess tailwater.

(1)

(2)

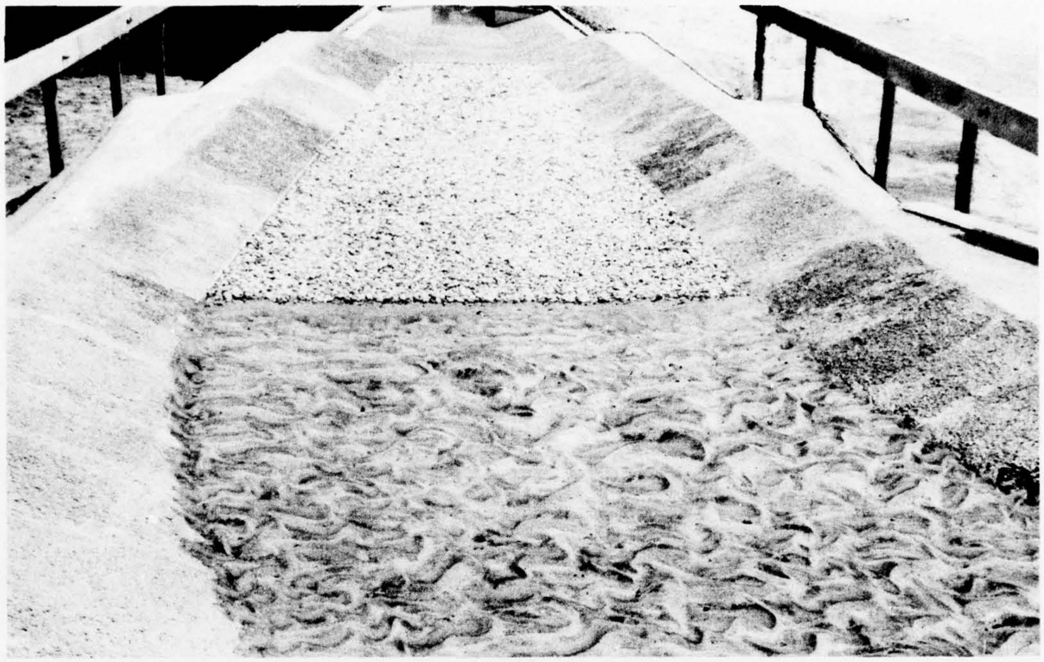
The original design. (1) Downstream view. (2) Upstream view.



(3)

(4)

Flow conditions, original design. Discharge, 20,950 cfs.
(3) Downstream view of flow through transition.
(4) Downstream view of flow approaching bridge.



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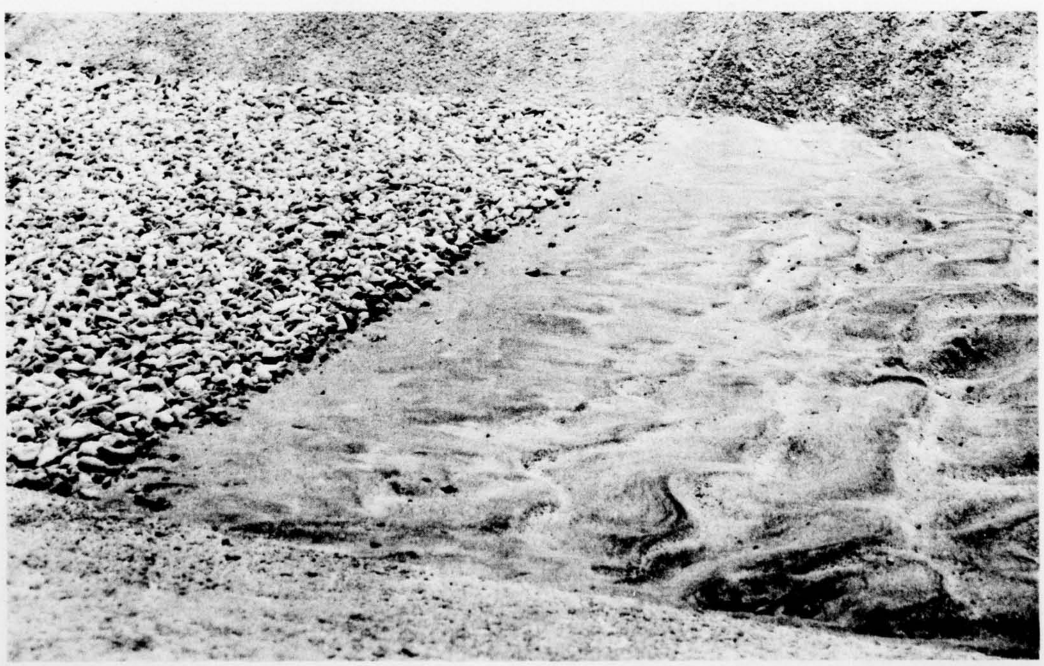

(6)

Scour resulting from hydrograph run, original design.
(5) Upstream view. (6) View at downstream end of transition.

(7)

Downstream view of alternative design.

(8)

Flow conditions, alternative design, looking downstream.
Discharge, 20,950 cfs.

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ROCK-LINED TRANSITIONS: HYDRAULIC MODEL INVESTIGATION.(U)
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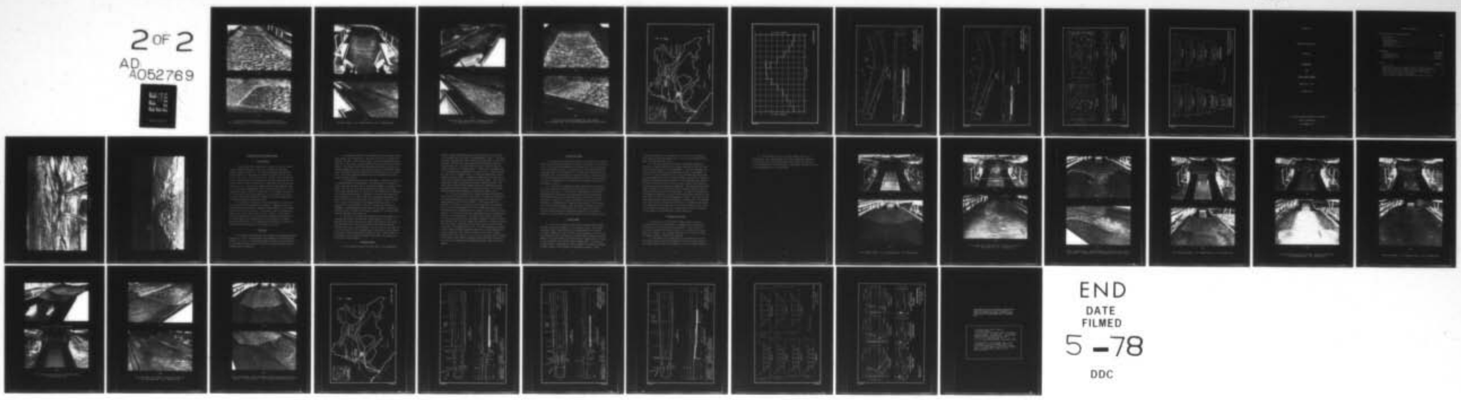

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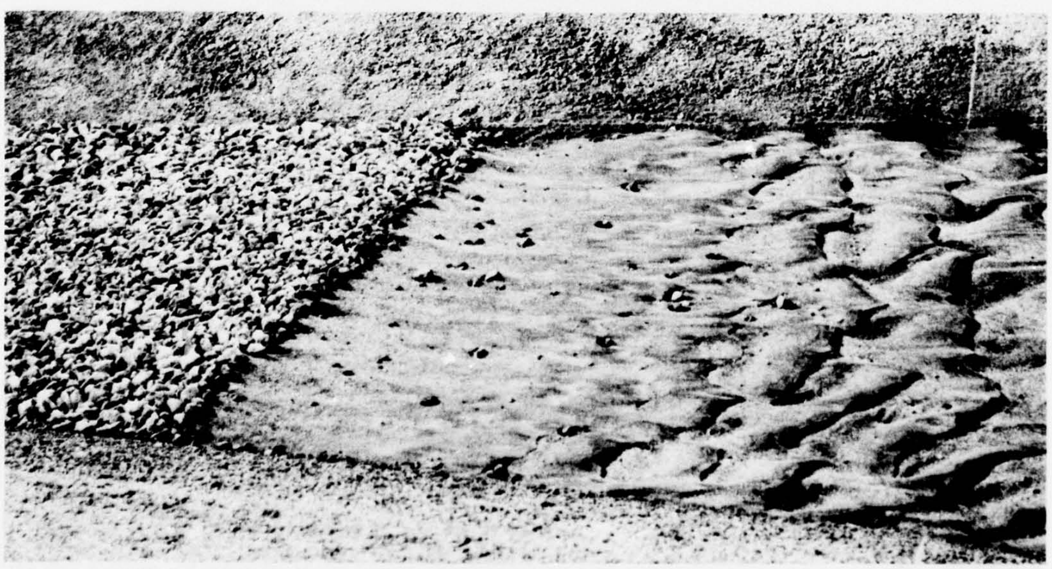
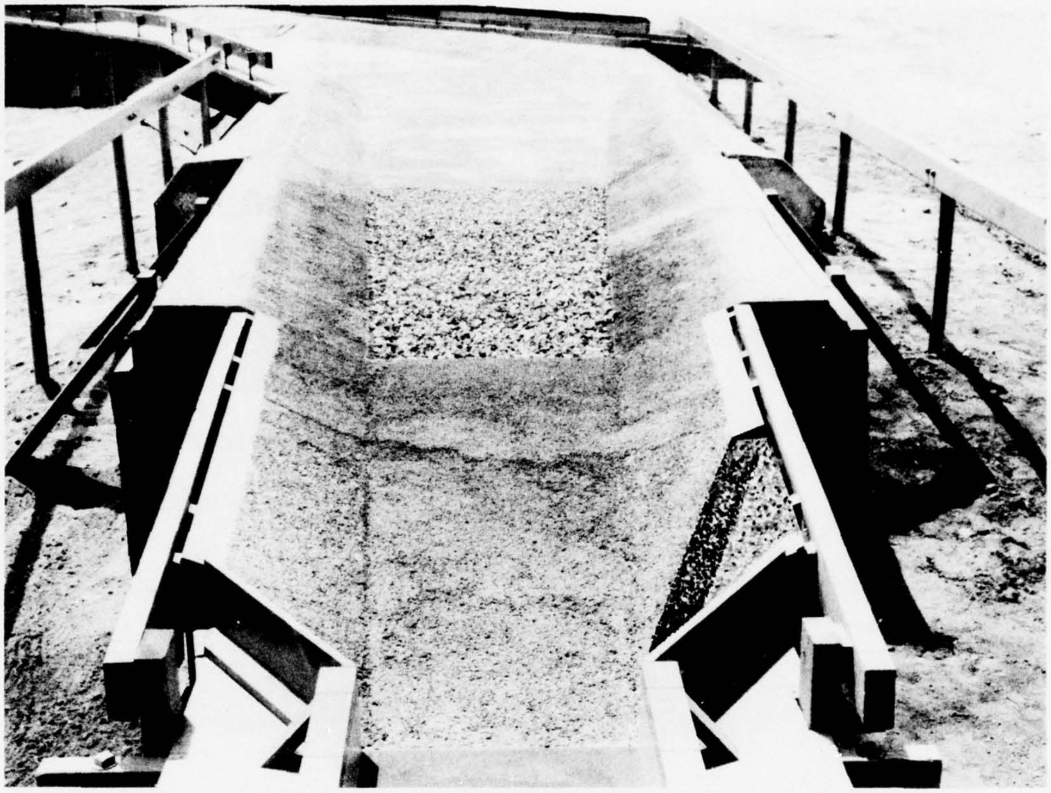
(9)



(10)

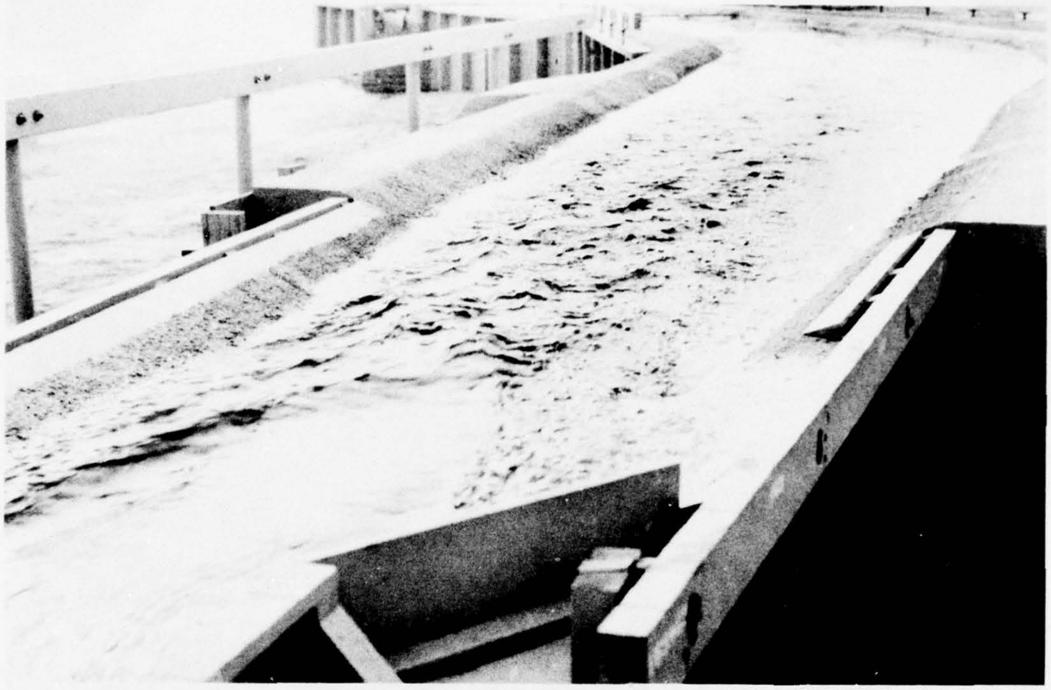
Scour resulting from hydrograph run, alternative design.
(9) Upstream view. (10) View at downstream end of stone invert.



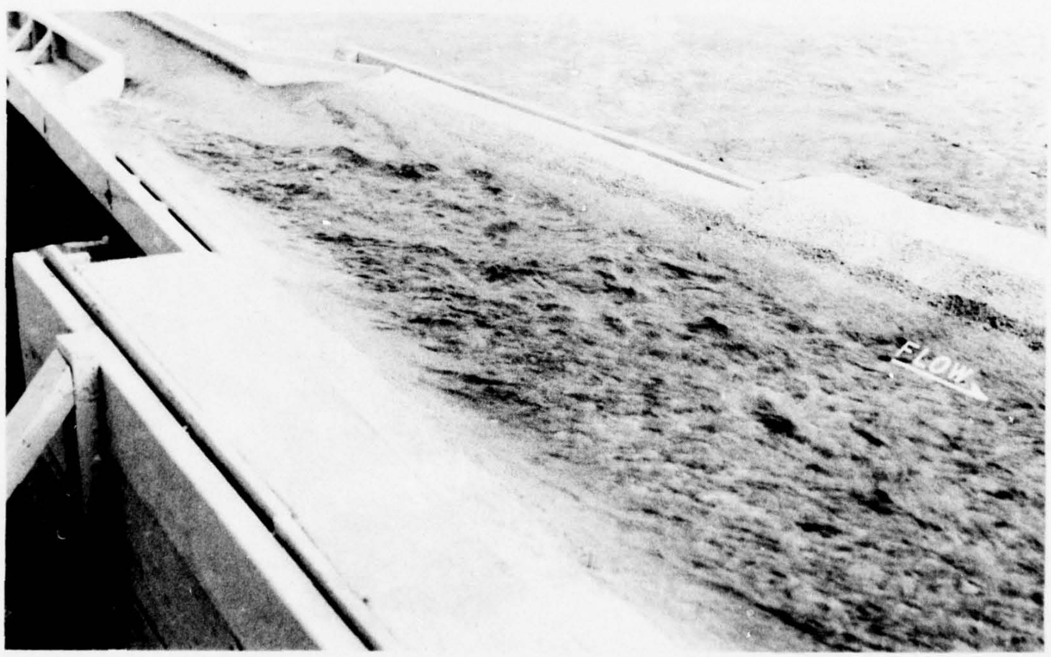
(11)

(12)

The final design. (11) Downstream view. (12) Upstream view.

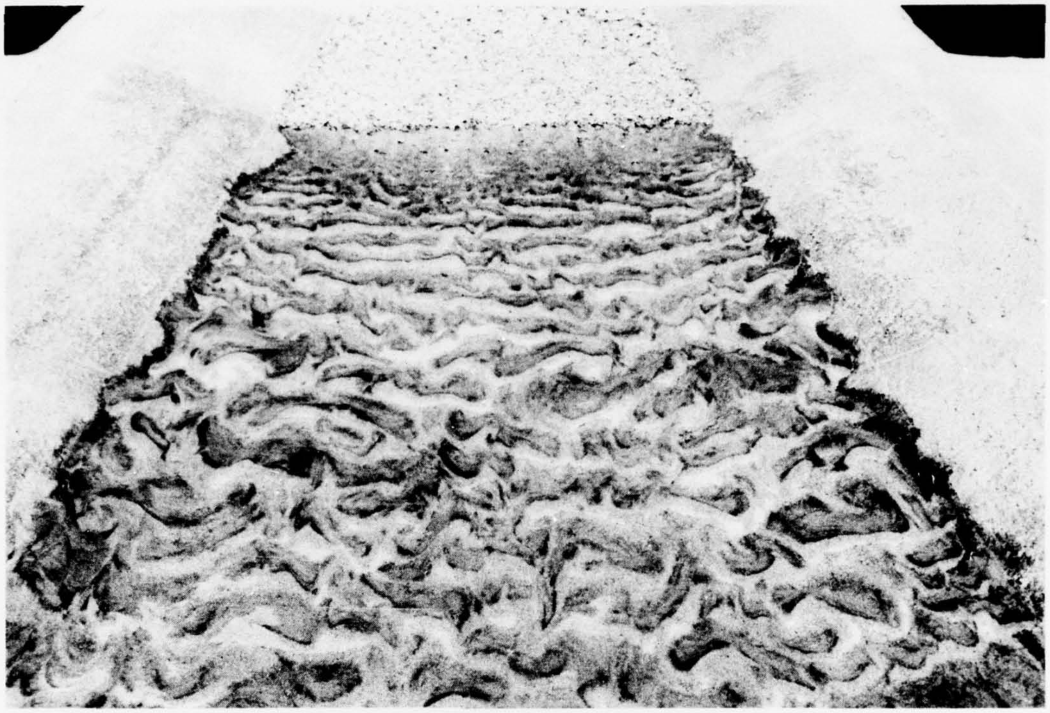


(13)



(14)

Flow conditions, final design. Discharge, 20,950 cfs.
(13) Downstream view. (14) Upstream view.



(15)

(16)

Scour resulting from hydrograph run, final design.
(15) Upstream view. (16) View at downstream end of stone invert.

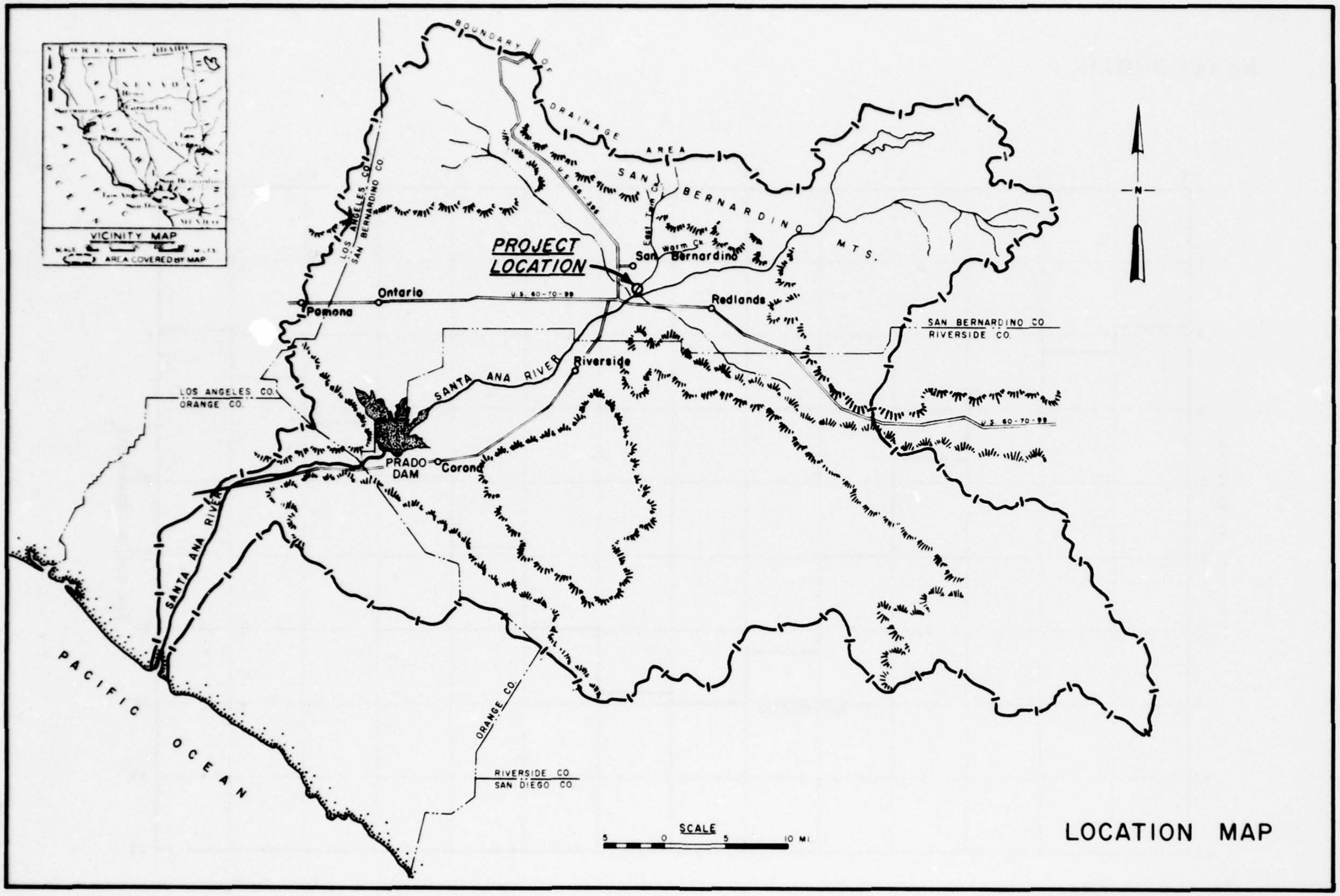

VICINITY MAP
AREA COVERED BY MAP
BOUNDARY OF DRAINAGE AREA
SAN BERNARDINO MTS.
PROJECT LOCATION
San Bernardino
East Twin Ck.
Warm Ck.
Redlands
Ontario
Pomona
U.S. 60-70-99
U.S. 66-395
LOS ANGELES CO.
SAN BERNARDINO CO.
SAN BERNARDINO CO.
RIVERSIDE CO.
Riverside
SANTA ANA RIVER
LOS ANGELES CO.
ORANGE CO.
PRADO DAM
Corona
SANTA ANA RIVER
ORANGE CO.
RIVERSIDE CO.
SAN DIEGO CO.
PACIFIC OCEAN
N
SCALE
5 0 5 10 MI.
LOCATION MAP

PLATE 1

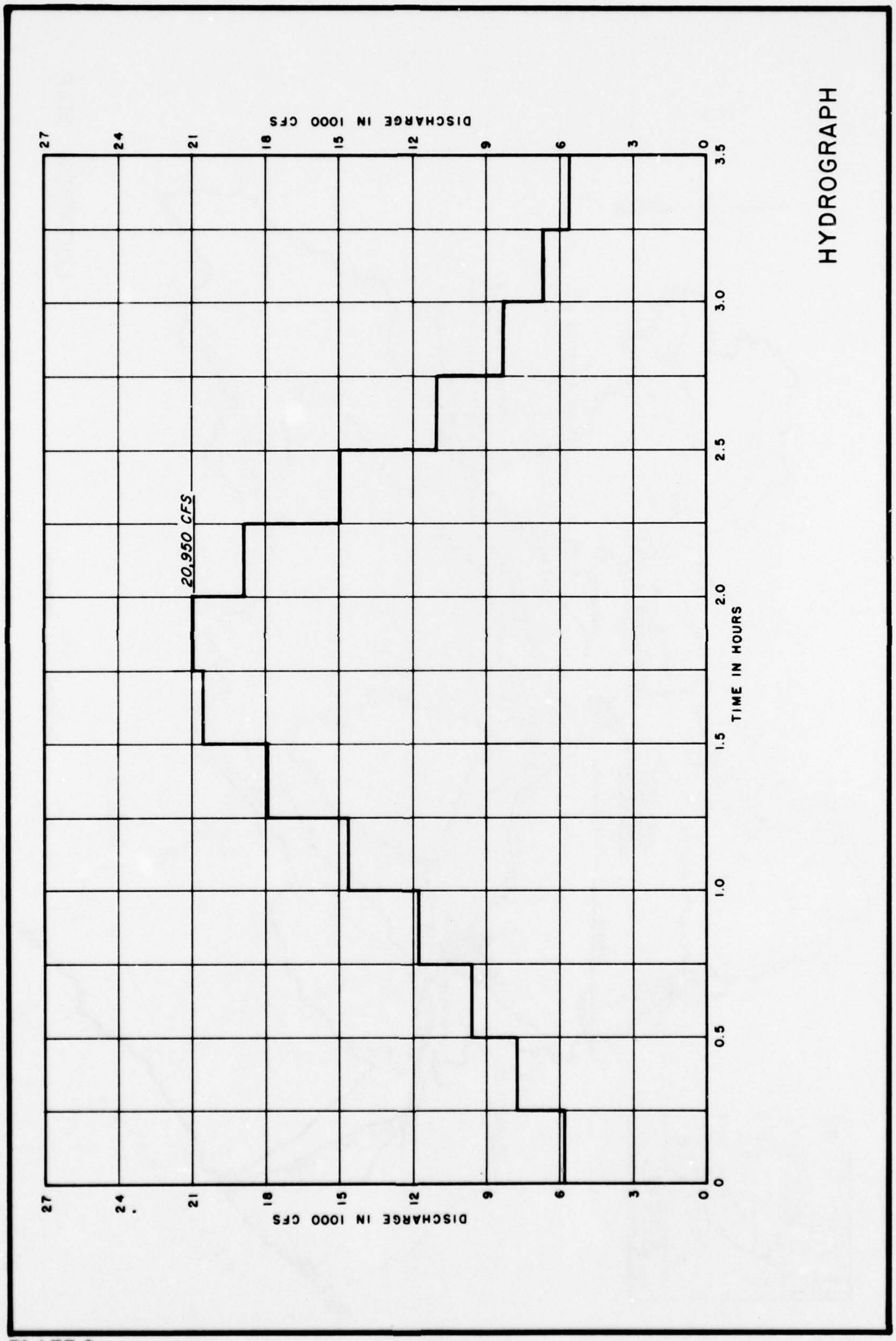

HYDROGRAPH
DISCHARGE IN 1000 CFS
0
3
6
9
12
15
18
21
24
27
TIME IN HOURS
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
20,950 CFS

PLATE 2

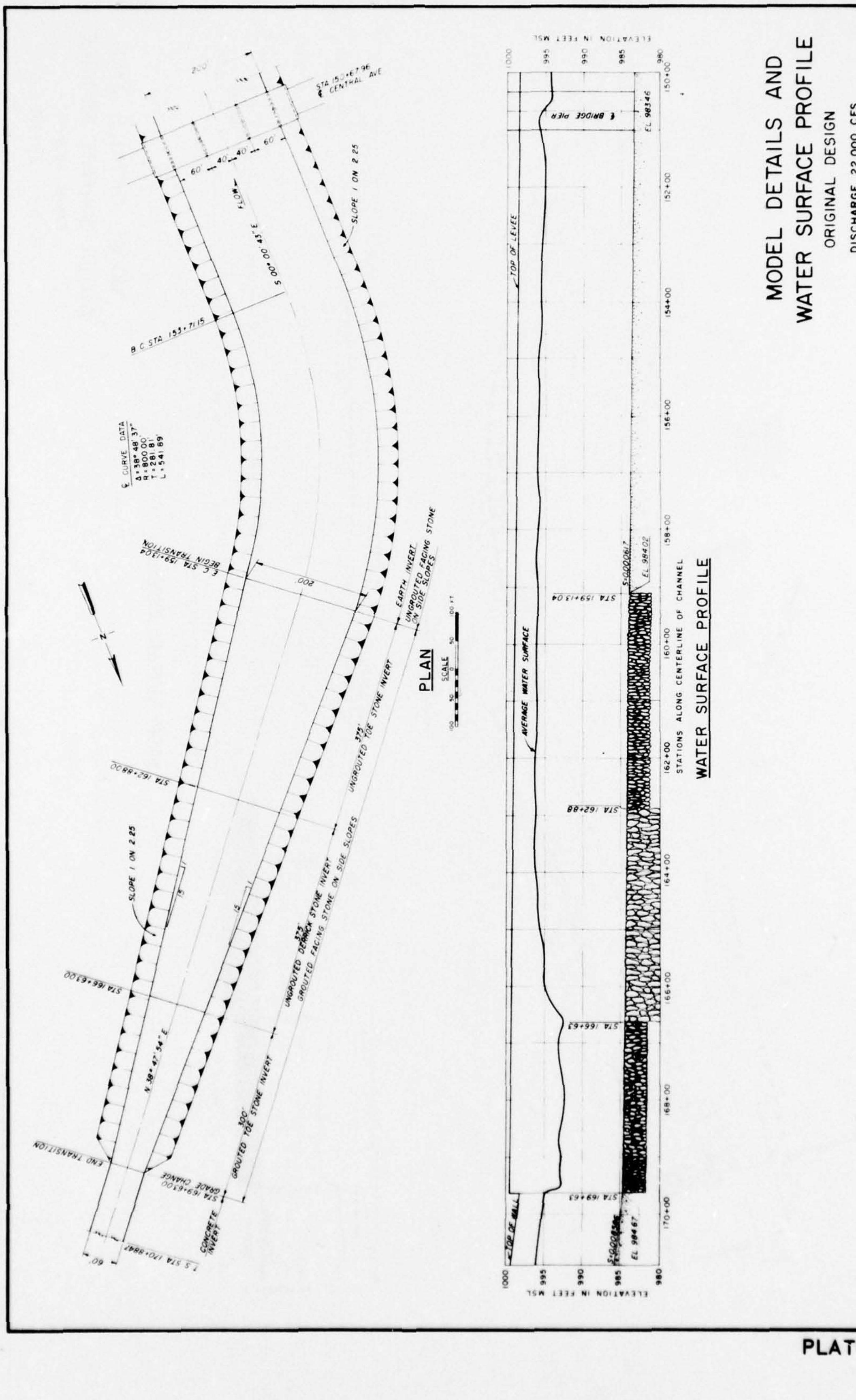
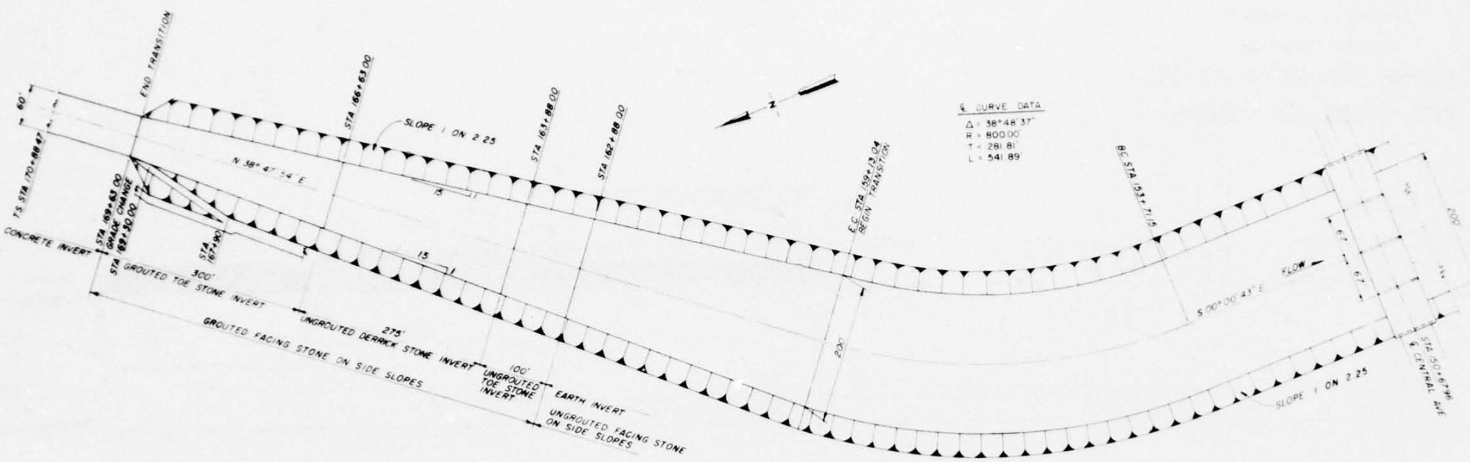
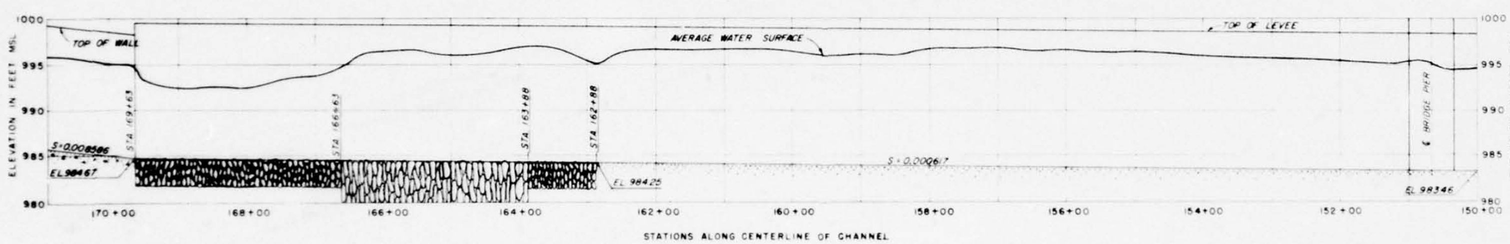

MODEL DETAILS AND
WATER SURFACE PROFILE
ORIGINAL DESIGN
DISCHARGE 22,000 CFS
PLAN
SCALE
WATER SURFACE PROFILE
STATIONS ALONG CENTERLINE OF CHANNEL
ELEVATION IN FEET MSL
TOP OF LEVEE
AVERAGE WATER SURFACE
TOP OF WALL
℄ BRIDGE PIER
FLOW
SLOPE 1 ON 2.25
B C STA 153+71.15
E C STA 159+13.04
BEGIN TRANSITION
END TRANSITION
℄ CURVE DATA
EARTH INVERT
UNGROUTED FACING STONE ON SIDE SLOPES
UNGROUTED TOE STONE INVERT
GROUTED FACING STONE ON SIDE SLOPES
GROUTED TOE STONE INVERT
CONCRETE INVERT
GRADE CHANGE
℄ CENTRAL AVE

PLATE 3



PLAN

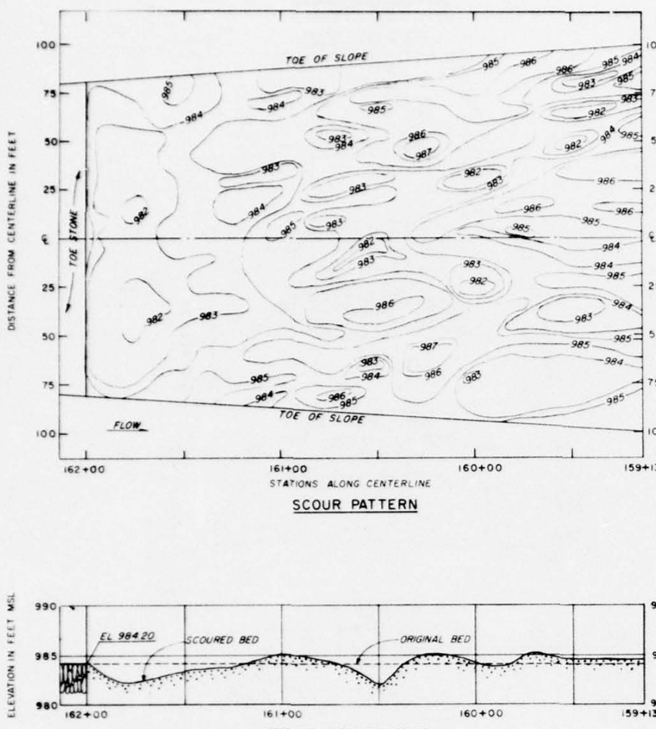
SCALE



WATER SURFACE PROFILE

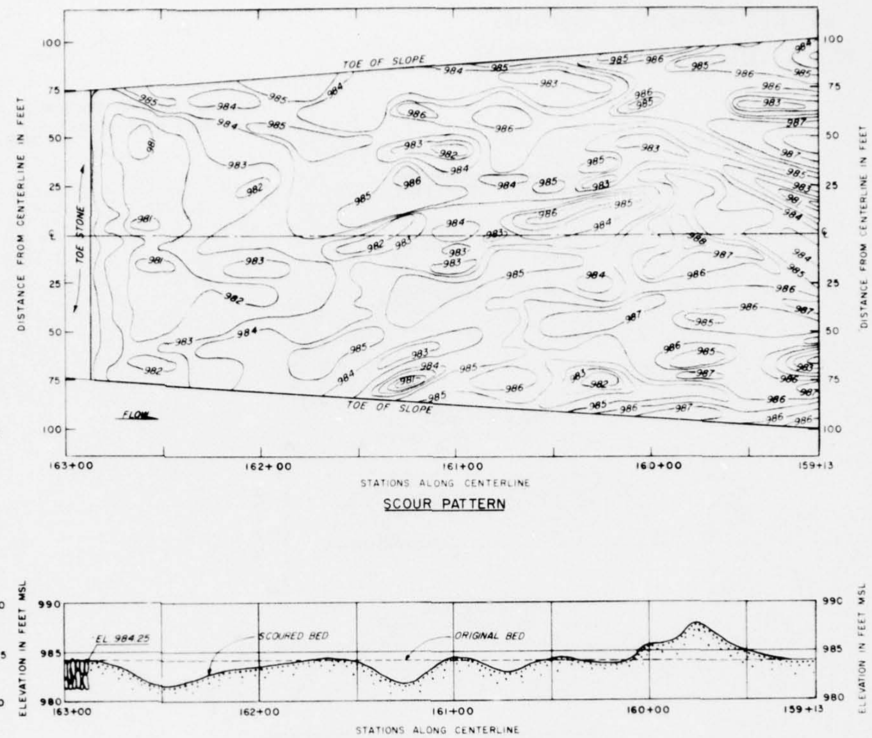
MODEL DETAILS AND
WATER SURFACE PROFILE
FINAL DESIGN
DISCHARGE 22,000 CFS

PLATE 4



SCOUR PROFILE ALONG CENTERLINE

ALTERNATIVE DESIGN

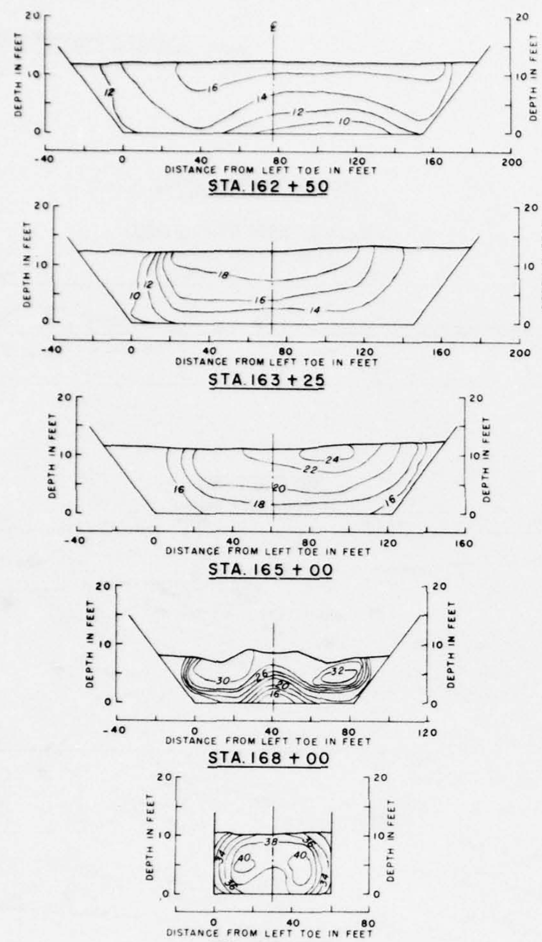


SCOUR PROFILE ALONG CENTERLINE

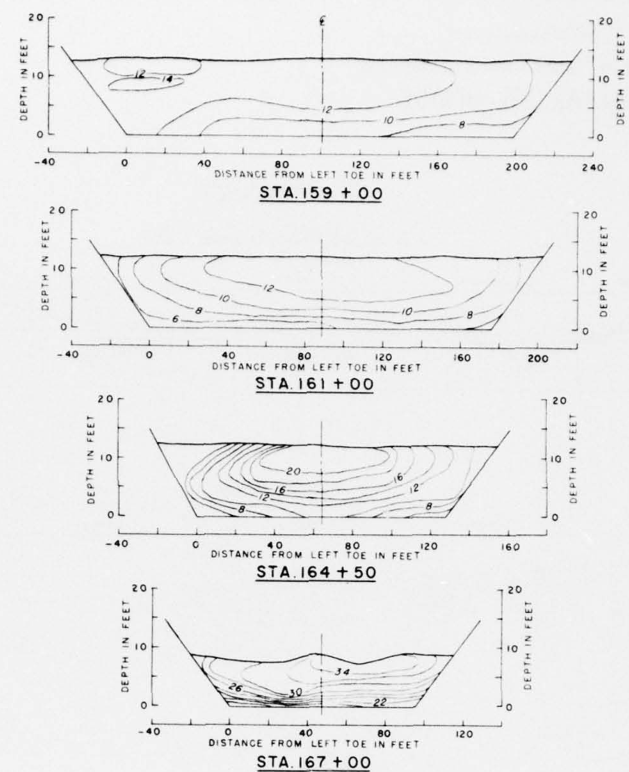
FINAL DESIGN

NOTE:
SCOUR DATA OBTAINED FROM A 3.5-HOUR RUN USING THE TIME-DISCHARGE RELATIONSHIP SHOWN BY THE HYDROGRAPH ON PLATE 2.

SCOUR PATTERNS
ALTERNATIVE AND FINAL DESIGNS
AFTER HYDROGRAPH



FINAL DESIGN



ORIGINAL DESIGN

NOTE
ALL SECTIONS SHOWN LOOKING DOWNSTREAM
VELOCITIES ARE IN FEET PER SECOND

VELOCITY DISTRIBUTION
ORIGINAL AND FINAL DESIGNS
DISCHARGE 22,000 CFS

APPENDIX II

EXTRACTS FROM REPORT

ENTITLED

TRANSITION

FOR

CHINO CREEK CHANNEL

REPORT NO. 1-106

OCTOBER 1962

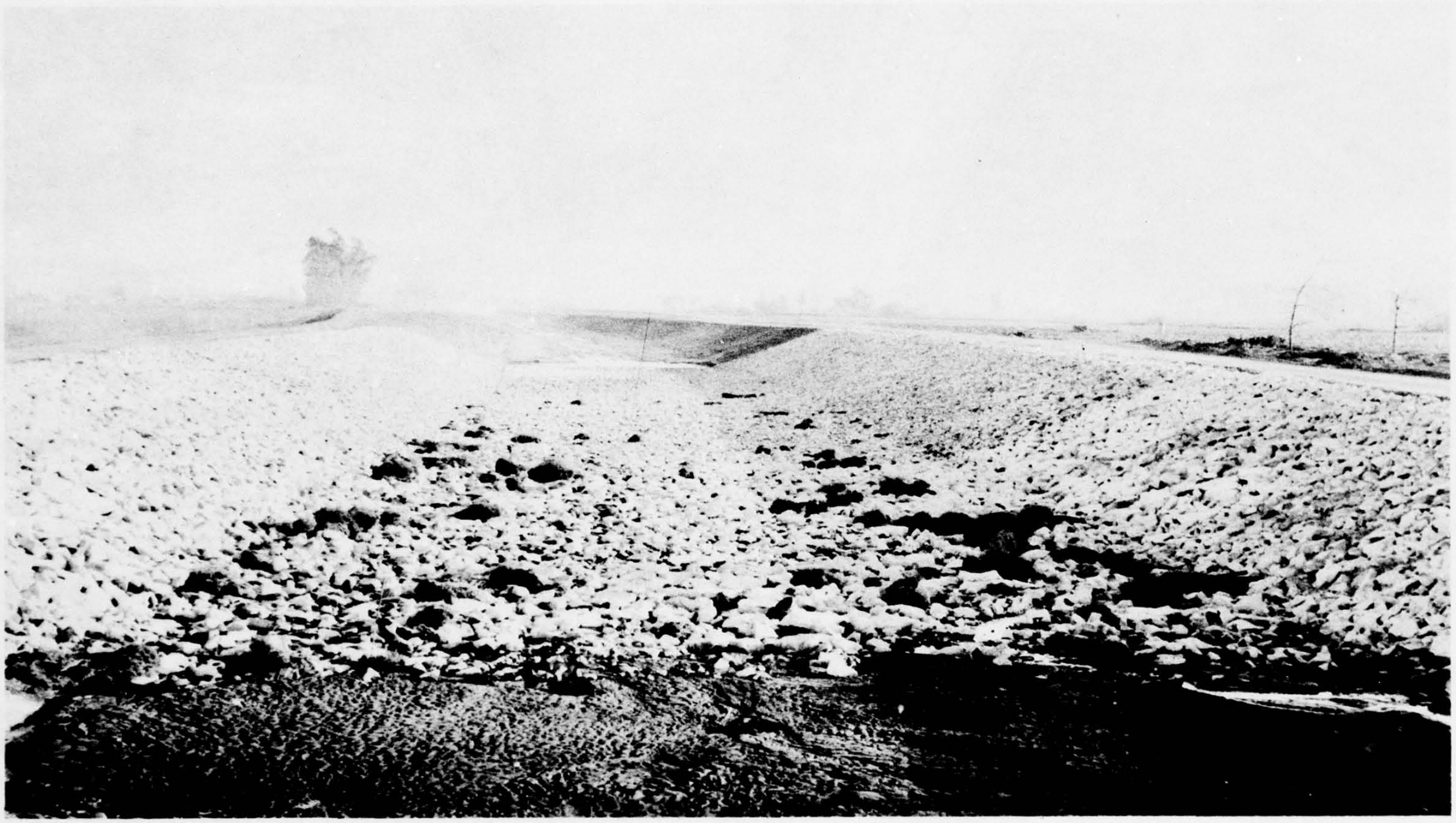
U. S. ARMY ENGINEER DISTRICT, LOS ANGELES

CORPS OF ENGINEERS

LOS ANGELES, CA.

INDEX TO APPENDIX II

The prototype.



The prototype, looking downstream from concrete section.

TRANSITION FOR CHINO CREEK CHANNEL

The Prototype

1. The channel reach, trapezoidal in cross section, is located near the southern boundary of the city of Pomona, San Bernardino County, California (plate 1). The channel, extending upstream from station 365+00, is constructed of reinforced concrete and has a base width of 90 ft and 1 on 2.25 side slopes. Between station 365+00 and station 359+00 the levee toes diverge and the side slopes are varied so that at station 359+00 the channel has a base width of 100 ft and side slopes of 1 on 3. This reach of channel was lined with grouted stone for the first 200 ft and dumped stone for the remaining 400 ft. Downstream from station 359+00, the base width of the channel continues to be 100 ft and the side slopes remain at 1 on 3. However, only 200 ft of channel downstream from station 359+00 is lined with dumped stone, the remainder being constructed of earth.

2. The channel is designed to carry a discharge of 26,000 cfs at both supercritical and subcritical velocities. Computations for the hydraulic design were based on Manning's formula wherein the resistance coefficient "n" was as follows: (1) concrete, 0.014, (2) grouted and dumped stone, 0.040, and (3) earth, 0.025. The part of the channel lined with concrete will carry the flow at supercritical velocities of 29.5 fps; however, the classification of flow changes as it passes through the transition. Upon reaching the earth channel the flow will have subcritical velocities in the vicinity of 11 fps.

The Model

3. The model was constructed to an undistorted, model-to-prototype scale ratio of 1:48. The prototype reach simulated in the model was as follows: (a) 550 ft of trapezoidal channel upstream from the transition, (b) the 600-ft transition, and (c) 500 ft of trapezoidal channel downstream from the transition.

4. The materials used in the construction of the model were wood, sand, gravel, and crushed stone. The wood, in the form of timbers and plywood, was used to construct that part of the model upstream from the transition. This reach of model simulated the reinforced concrete-lined channel. The channel reach lined with grouted stone was simulated by cementing gravel particles to the plywood surface, and the channel reach lined with dumped stone was simulated by using various sizes of crushed stone. The channel downstream from the end of the dumped stone was molded in a bed of sand.

5. Two methods were applied in the model to simulate the slope of the energy gradient of the prototype. One method was to increase the model slope, and the other was to artificially roughen the surface of the model. The first of the above two methods was applied to that part of the model upstream from the transition. The coefficient of roughness of the painted surface used in this part of the model had previously been determined to be 0.0088. Based upon similitude relationship, the above "n" value would be equivalent to a prototype "n" value of 0.017. The "n" value used in design for prototype linings was 0.014. To compensate for the difference in the two prototype coefficients of roughness, a supplementary slope was added to the model. The second method was used to simulate the boundary roughness for the grouted and ungrouted linings of the transition.

6. The part of the model that simulated the earth channel of the prototype was molded in sand having a mean diameter of 0.20 mm. To maintain similitude of mass and not destroy the geometric similarity, it was necessary for the stone in the model to have the same specific gravity as the stone in the prototype. The stone used in the model had a specific gravity of 2.64 (assumed equal to prototype) and was graded to provide a uniform distribution of particle size. Crushed stone was used because it more nearly represented the shape of the quarrystone to be used in the prototype.

Original Design

7. In the original design, the base widths of the trapezoidal

channel upstream and downstream from station 363+00 were 90 ft and 100 ft respectively (see plate 2 and photographs 1 and 2). The reach of channel upstream from station 363+00 was constructed of reinforced concrete. Two piers for Los Serranos Road bridge were located at station 366+42.78. The channel between station 365+00 and station 363+00 consisted of a stilling basin. A 100-ft reach of dumped stone, 5 ft deep, was placed downstream from the end sill to station 362+00. The next 400 ft of dumped stone, 3 ft deep, was placed from station 362+00 to station 358+00. In addition, dumped stone was used on the levee slopes between station 363+00 and station 358+00. The reach of trapezoidal channel downstream from station 358+00 was earth. The levee slopes between station 365+00 and station 363+80 were warped. Upstream from station 365+00, the levees were sloped 1 on 2.25, and downstream from station 363+80, the levees were sloped 1 on 3.

8. Tests were conducted for a design discharge of 26,000 cfs. Supercritical velocities were to be reduced to subcritical velocities after passing through the stilling basin, which was located at the downstream end of the concrete channel. The stilling basin was designed to dissipate the energy in the flow by means of a hydraulic jump. As shown in photograph 3 and 4, the jump that formed in the stilling basin was unsymmetrical with respect to the centerline of the channel. It can also be noted that downstream from the stilling basin the bulk of the flow passed to the right of the channel centerline. The water surface profile for the design discharge is shown on plate 2, and indicated that the levee heights are sufficient to contain the flow. Tests were made to determine the scour that would occur at the downstream end of the dumped stone invert. This was accomplished by passing the design discharge through the model for a period equivalent to 20 prototype hours. The results of this test are shown in photographs 5 and 6 and on plate 6. It can be seen that more scour occurred to the right of the channel centerline, which indicates that most of the flow in the channel was concentrated along the right side of the channel.

Alternative Design

9. The alternative design was based on the premise that it would provide more uniform distribution of flow at the entrance to the stilling basin. This uniform distribution of flow would result in the formation of a better hydraulic jump. To accomplish this the stilling basin was moved upstream to station 366+16.65. Training walls were added to the downstream ends of the bridge piers and were aligned so that an angle of 10° existed between the projected centerline of the pier and the centerline of the training wall. The extensions are shown on plate 3 and in photographs 7 and 8.

10. Tests with the design discharge indicated that the training walls were effective in distributing the flow across the entire width of the stilling basin. It can be seen in photographs 9 and 10 that undulating flow occurred in, and downstream from the stilling basin. This indicated that the stilling basin was not an effective energy dissipator. However, the overall distribution of flow within the channel was considerably better than the distribution of flow experienced in the previous design. Tests were made to determine the scour that would occur at the downstream end of the dumped stone invert. This was accomplished by the same method employed in the previous design. The scour pattern resulting from this test is shown on plate 6.

Final Design

11. The reach of channel upstream from station 365+00 remained the same in the final design as in the original design. This design contained a 600-ft long transition, located between station 365+00 and station 359+00, as shown on plate 4 and photographs 11, 12, and 13. The channel was lined with grouted stone from station 365+00 to station 363+00. This was followed by a 200-ft reach of dumped stone, 5 ft deep, to station 361+00, and a 400-ft reach of dumped stone, 3 ft deep, to station 357+00. Downstream from station 357+00, the channel was molded in sand, similar to the original and alternative designs. Levee

slopes were warped in the transition from 1 on 2.25 at the upstream end, where the base width was 90 ft, to 1 on 3 at the downstream end, where the base width was 100 ft.

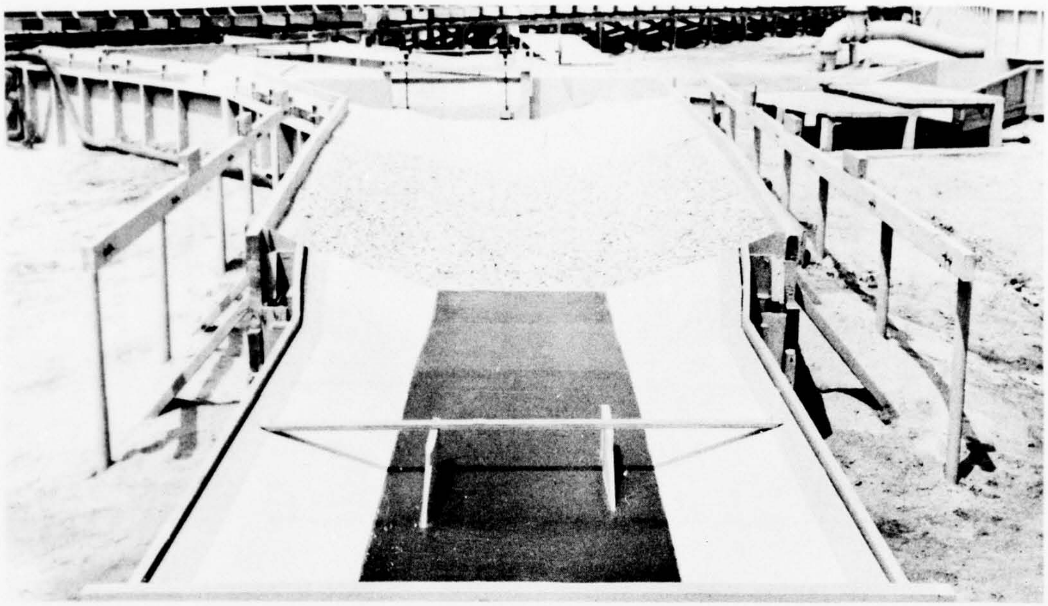
12. The final design was tested for the design discharge of 26,000 cfs. Data obtained for this design consisted of depth of flow, velocity distribution and scour measurements. The flow throughout the entire reach of channel was satisfactory, as shown in photographs 14, 15 and 16. Visual observation in the model indicated that the grouted stone and dumped stone reach of channel formed a satisfactory transition between the upstream concrete channel and the downstream earth channel. The supercritical velocities from the concrete channel were reduced to subcritical velocities within the transition. The velocity distribution shown on plate 5 for this reach of channel indicated that the flow was evenly distributed across the channel base. The model was subjected to the design discharge for a period of 20 prototype hours resulting in the scour shown on plate 6 and in photographs 17 and 18. The scour pictures show that there was very little movement of surface stone. The symmetry of the scour pattern verifies the existence of a uniform flow distribution downstream of the dumped stone. The water surface profile for the final design is shown on plate 4. Measured water surface, in general, was the same as the computed water surface elevations.

Discussion of Results

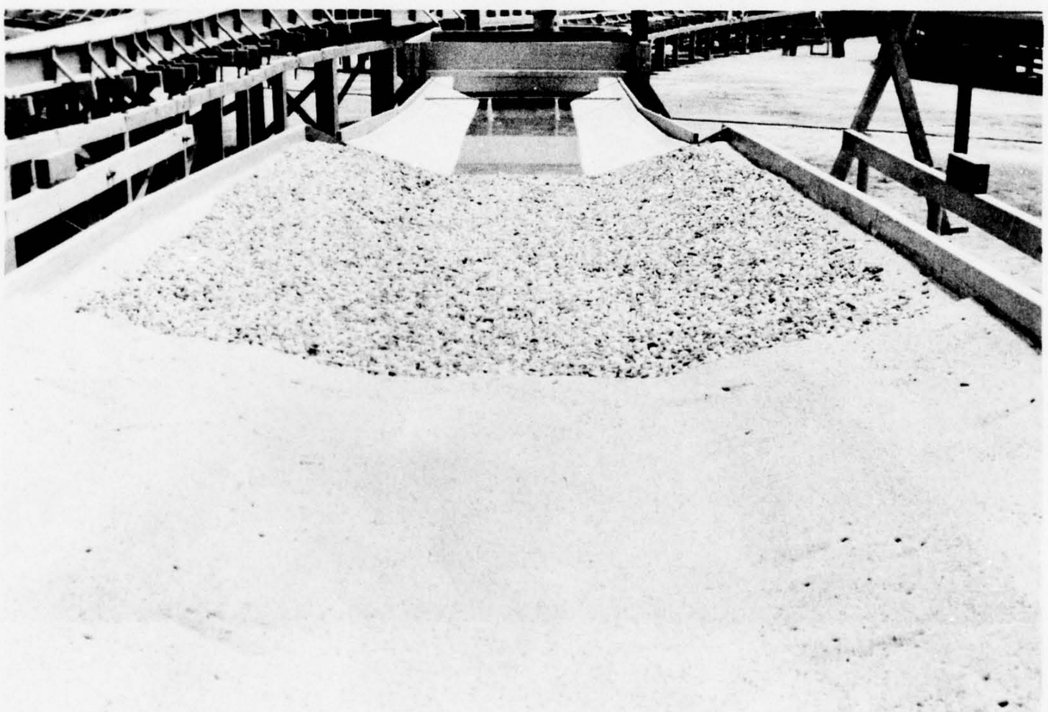
13. Model tests of the original design indicated that the stilling basin was not satisfactory. Unsymmetrical flow conditions existed as the flow entered the stilling basin. These conditions continued downstream and caused the flow to be confined to the right side of the channel. The stilling basin was not an effective energy dissipator because of insufficient tailwater.

14. The addition of training walls on the bridge piers in the alternative design improved the symmetry of flow but did not improve the effectiveness of the stilling basin to act as an energy dissipator.

15. The transition used in the final design proved to be satisfactory. The flow throughout the entire channel was improved. It was effective in reducing the supercritical velocities (30 fps) at the upstream end of the transition to subcritical velocities (11 fps) at the downstream end of the transition.

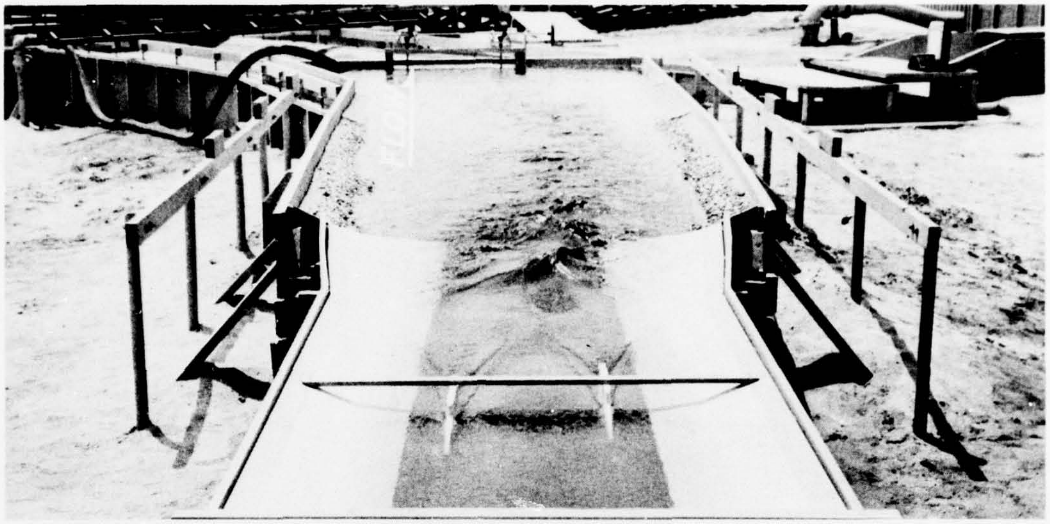


(1)

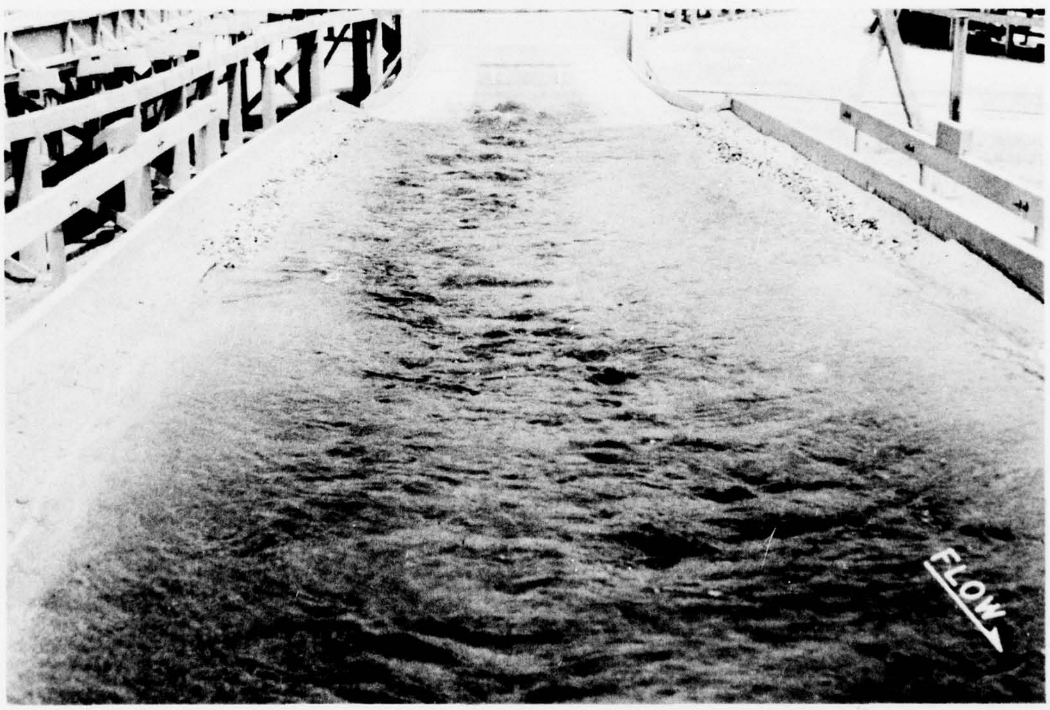


(2)

The original design. (1) Downstream view. (2) Upstream view.



(3)



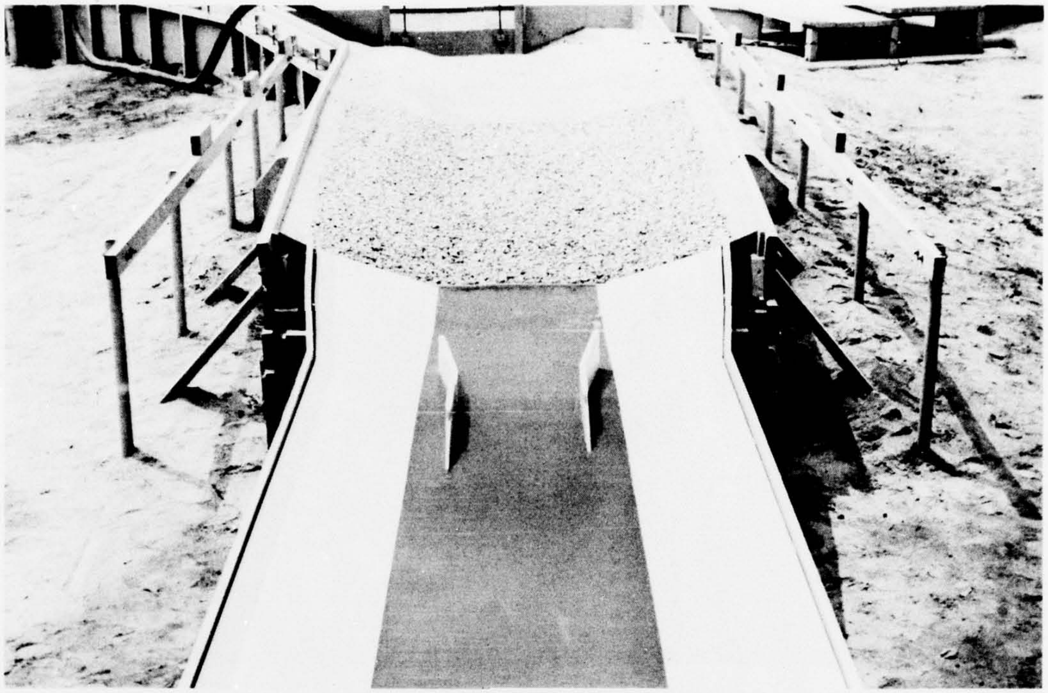
(4)

Flow conditions, original design. Discharge, 26,000 cfs.
(3) Downstream view. (4) Upstream view.

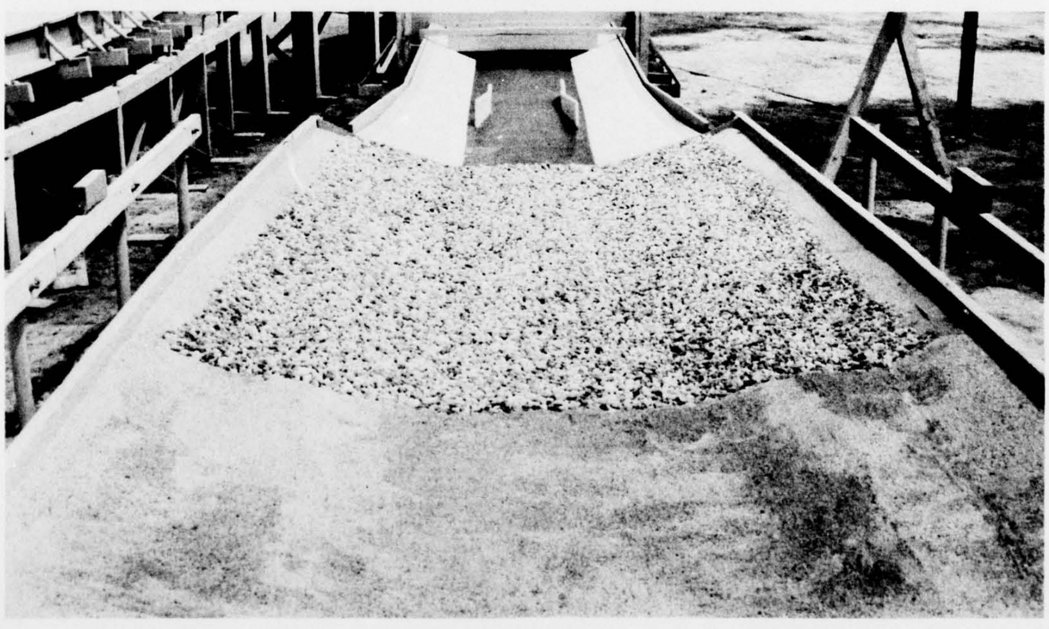
(5)

(6)

Scour, original design. Design discharge of 26,000 cfs was run for a 20-hr period. (5) Upstream view. (6) Closeup at end of stone lining.

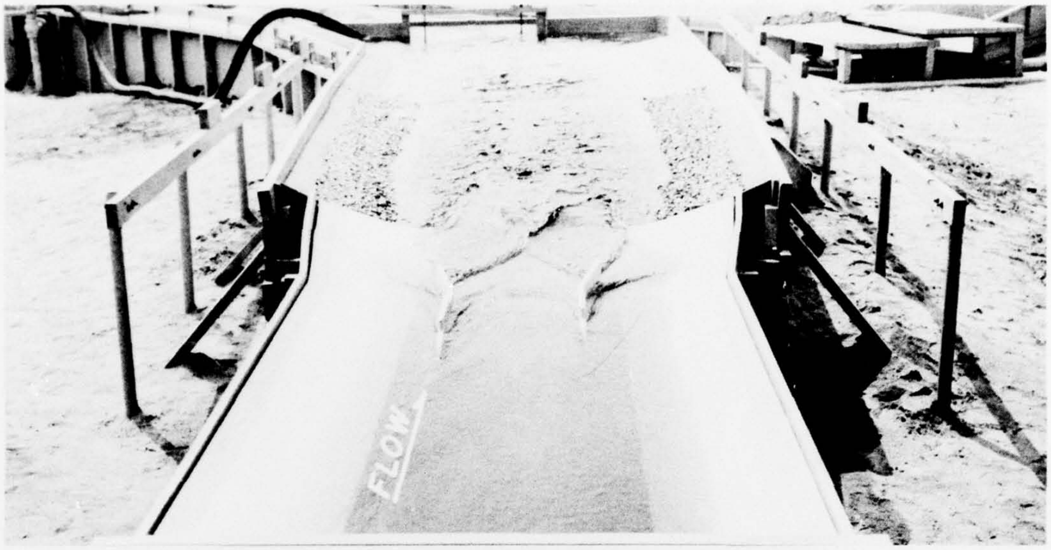


(7)



(8)

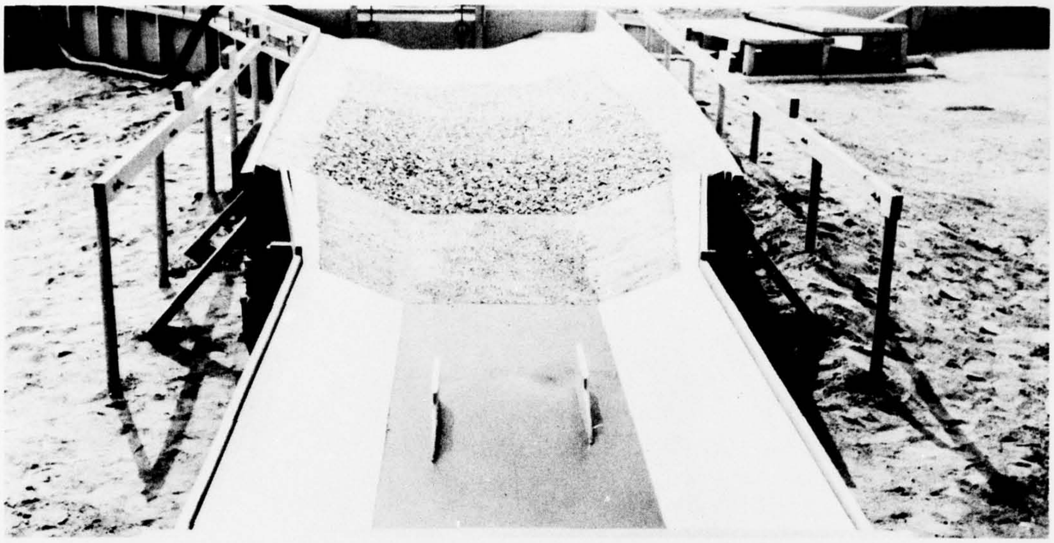
The alternative design. (7) Downstream view. (8) Upstream view.



(9)

(10)

Flow conditions, alternative design. Discharge, 26,000 cfs.
(9) Downstream view. (10) Upstream view.



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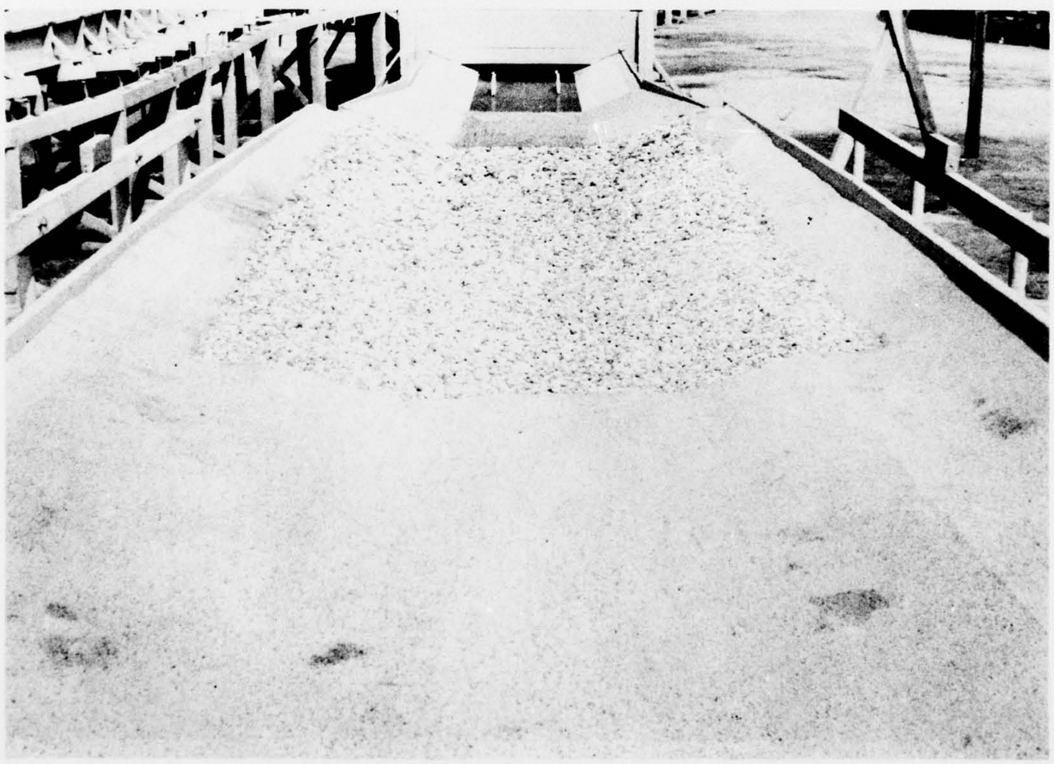
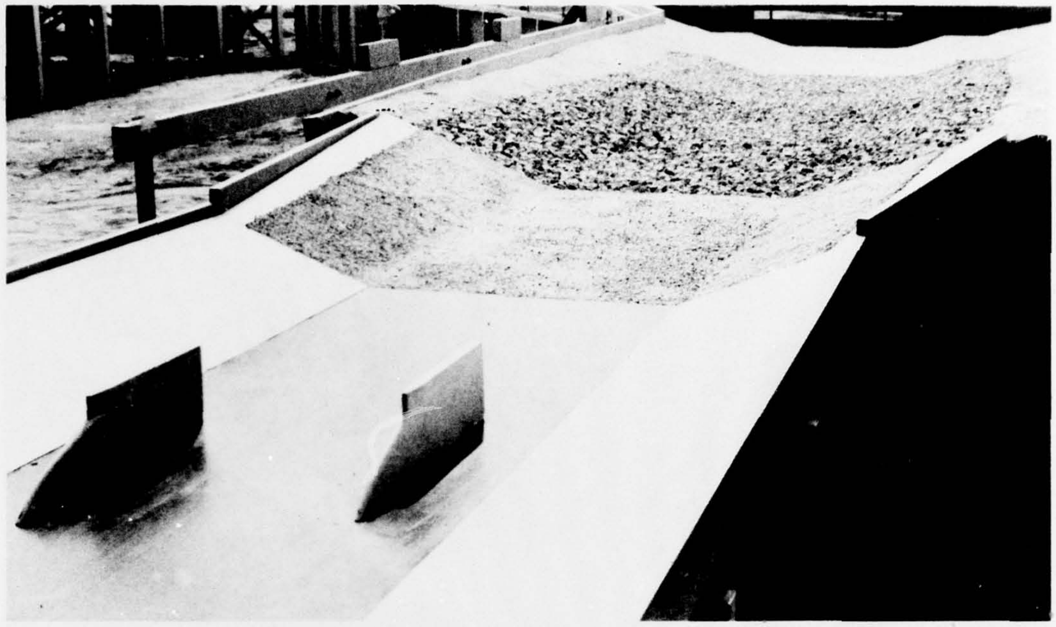


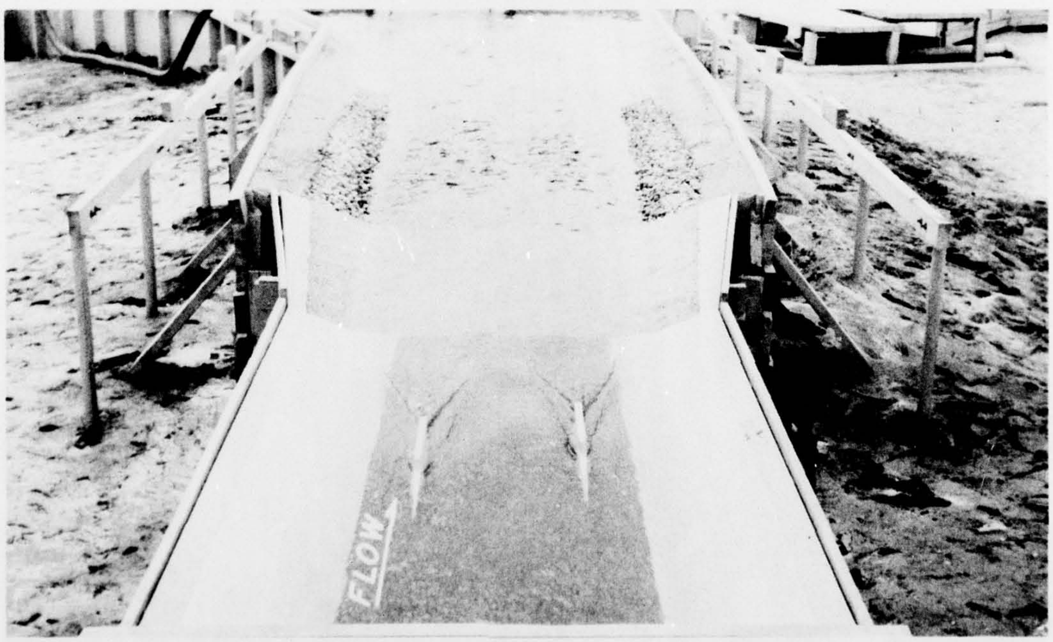
(12)

The final design. (11) Downstream view. (12) Upstream view.



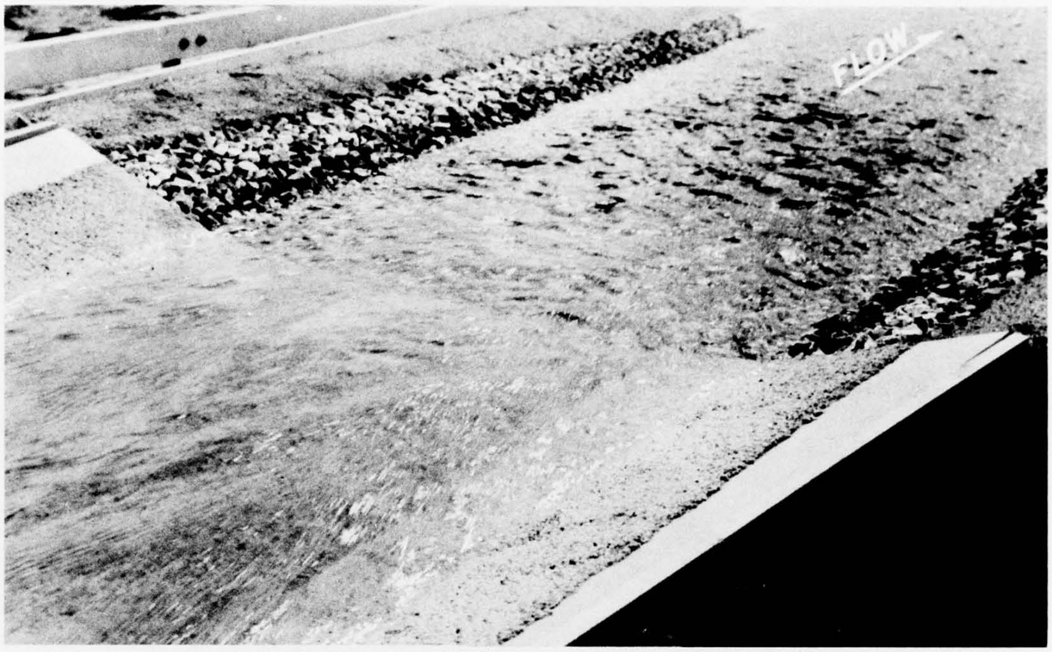
(13)

Downstream view of the final design.



(14)

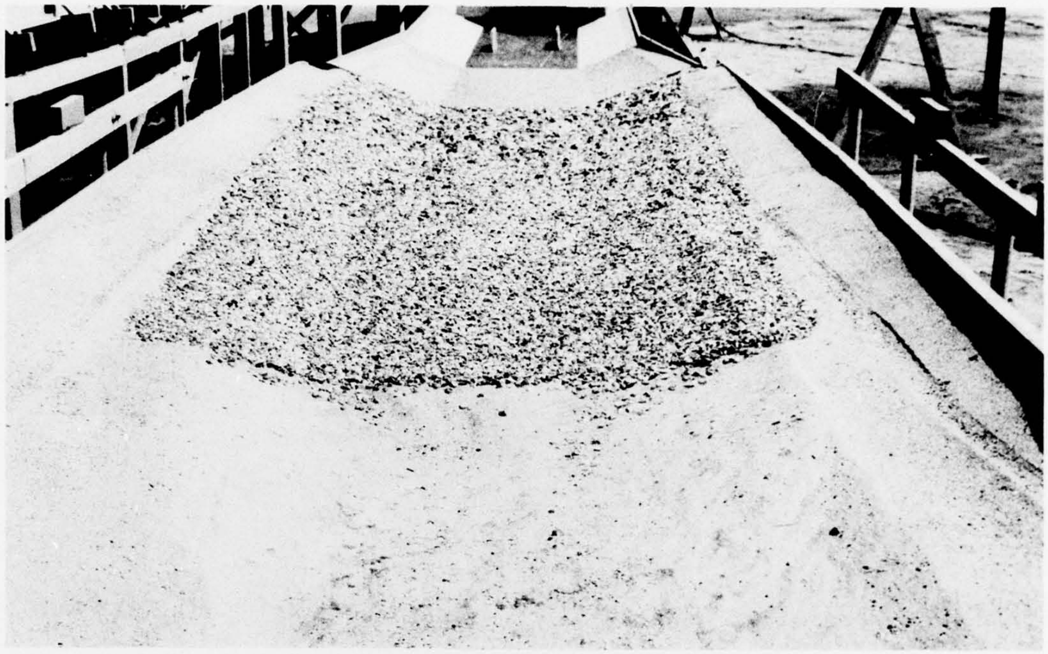
Flow conditions, final design, looking downstream.
Discharge, 26,000 cfs.



(15)

(16)

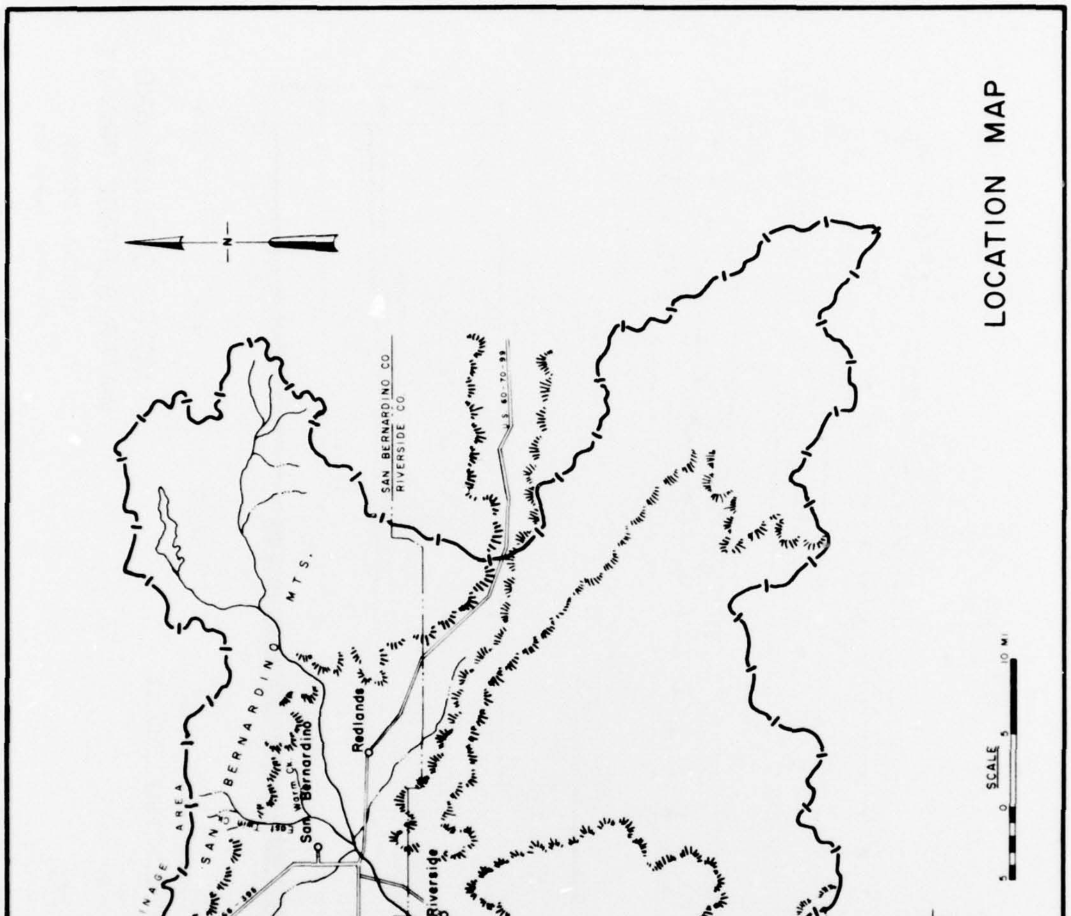
Flow conditions, final design. Discharge, 26,000 cfs.
(15) Downstream view. (16) Upstream view.



(17)

(18)

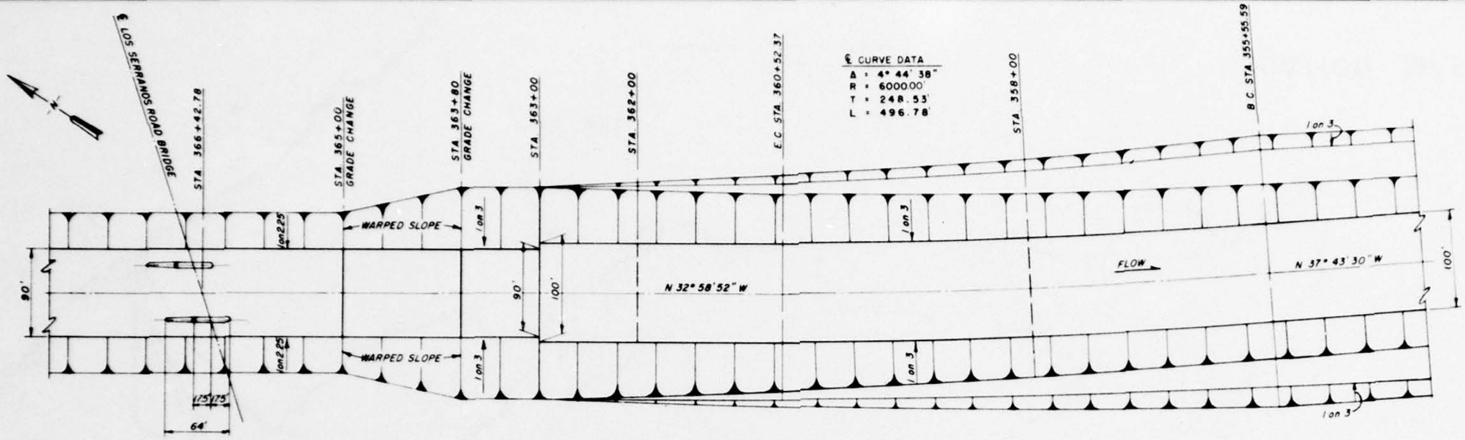
Scour, final design. Design discharge of 26,000 cfs was run for a 20-hr period. (17) Upstream view. (18) Closeup at end of stone lining.



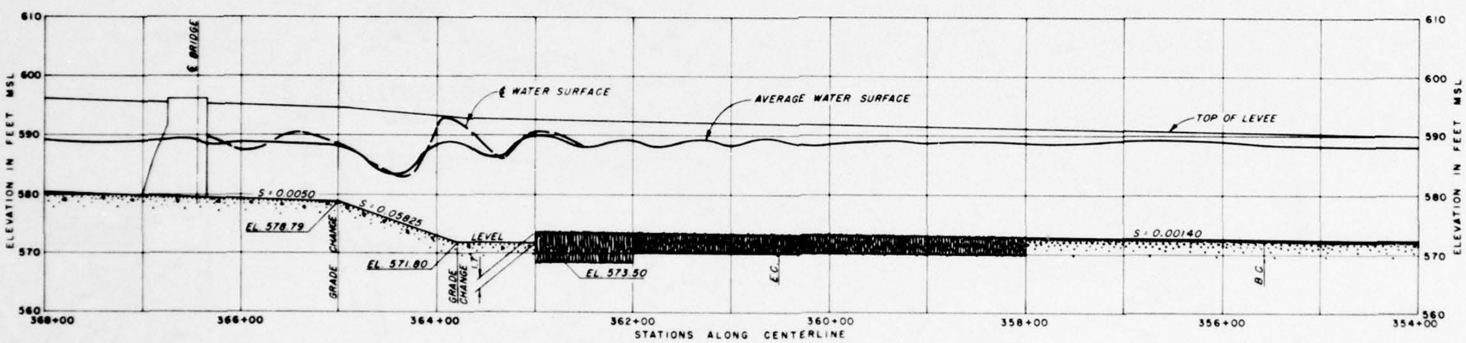
LOCATION MAP
SCALE
N
PROJECT LOCATION
SAN ANTONIO DAM
PRADO DAM
Ontario
Pomona
Redlands
San Bernardino
Riverside
Corona
Chino Creek
San Antonio
SANTA ANA RIVER
SAN BERNARDINO MTS.
BOUNDARY OF DRAINAGE AREA
SAN BERNARDINO CO.
RIVERSIDE CO.
LOS ANGELES CO.
ORANGE CO.
RIVERSIDE CO.
SAN DIEGO CO.
PACIFIC OCEAN
VICINITY MAP
AREA COVERED BY MAP

PLATE 1

PLATE 2



PLAN



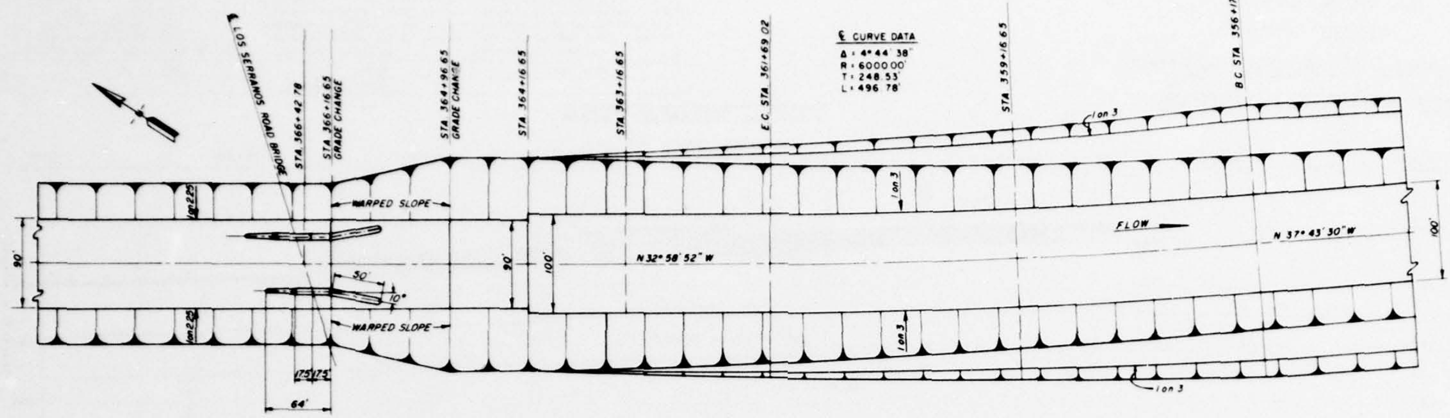
WATER SURFACE PROFILE

REACH OF CHANNEL	CHANNEL LINING: INVERT	CHANNEL LINING: LEVEE FACE
UPSTREAM MODEL LIMITS TO STA 363+00	REINFORCED CONCRETE	REINFORCED CONCRETE
STA 363+00 TO STA 362+00	UNGROUTED QUARRY STONE, TYPE A	UNGROUTED QUARRY STONE, TYPE B
STA 362+00 TO STA 358+00	UNGROUTED QUARRY STONE, TYPE B	UNGROUTED QUARRY STONE, TYPE B
STA 358+00 TO DOWNSTREAM MODEL LIMITS	EARTH	EARTH

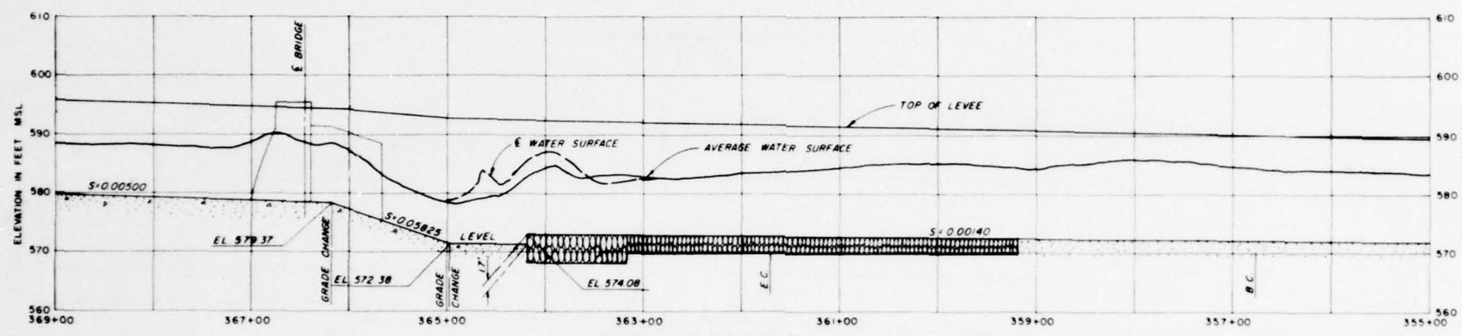
MODEL DETAILS AND WATER SURFACE PROFILE

ORIGINAL DESIGN

DISCHARGE 26,000 CFS



PLAN



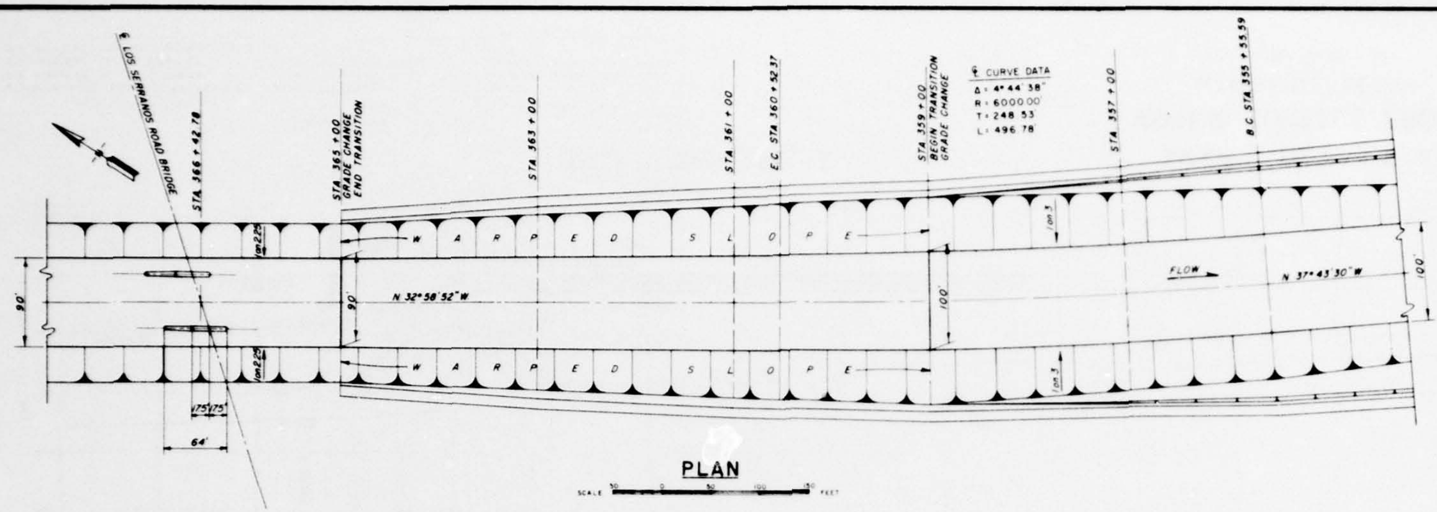
WATER SURFACE PROFILE

REACH OF CHANNEL	CHANNEL LINING: INVERT	CHANNEL LINING: LEVEE FACE
UPSTREAM MODEL LIMITS TO STA 364+16.65	REINFORCED CONCRETE	REINFORCED CONCRETE
STA 364+16.65 TO STA 363+16.65	UNGROUTED QUARRY STONE, TYPE A	UNGROUTED QUARRY STONE, TYPE B
STA 363+16.65 TO STA 359+16.65	UNGROUTED QUARRY STONE, TYPE B	UNGROUTED QUARRY STONE, TYPE B
STA 359+16.65 TO DOWNSTREAM MODEL LIMITS	EARTH	EARTH

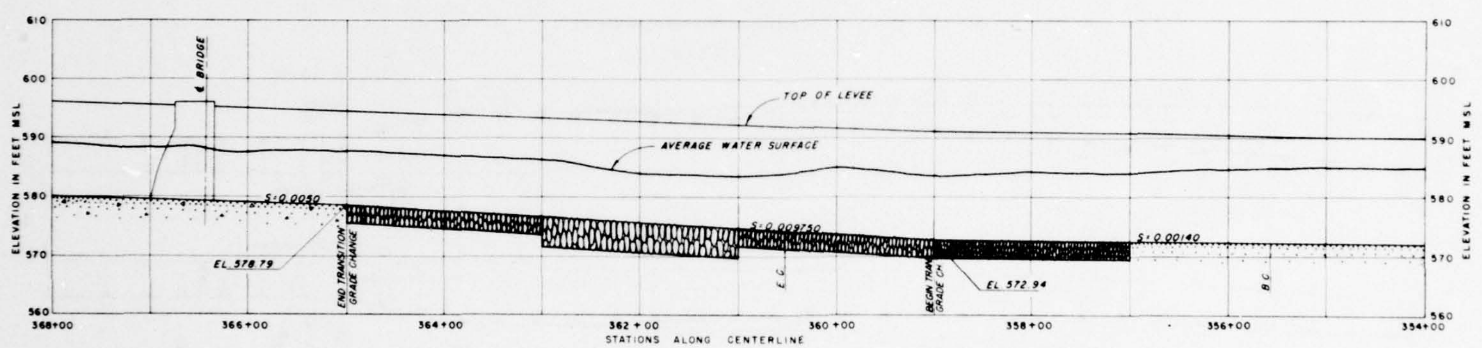
MODEL DETAILS AND WATER SURFACE PROFILE

ALTERNATIVE DESIGN

DISCHARGE 26,000 CFS



PLAN



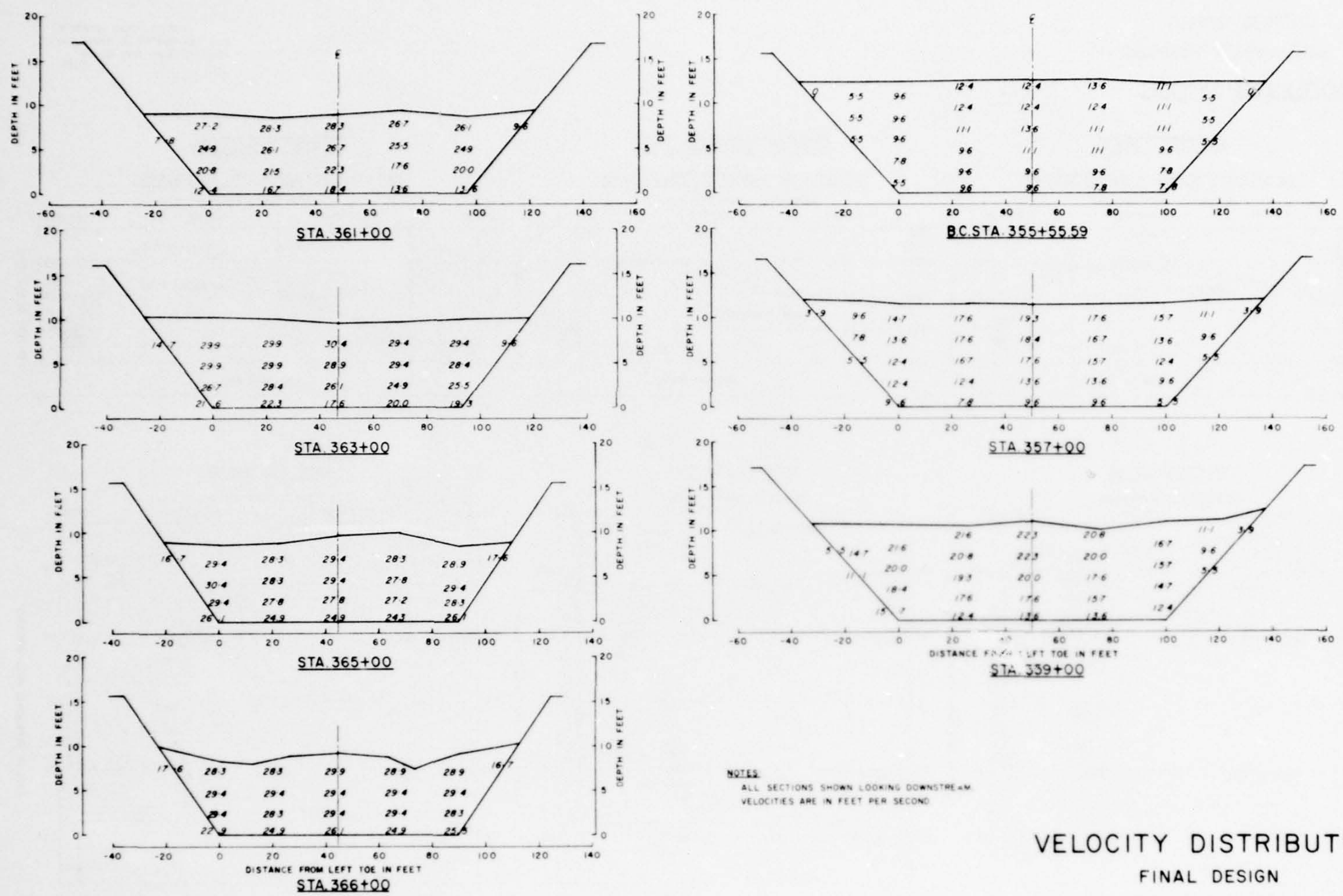
WATER SURFACE PROFILE

REACH OF CHANNEL	CHANNEL LINING: INVERT	CHANNEL LINING: LEVEE FACE
UPSTREAM MODEL LIMITS TO STA. 365+00	REINFORCED CONCRETE	REINFORCED CONCRETE
STA. 365+00 TO STA. 363+00	GROUTED QUARRY STONE, TYPE B	GROUTED QUARRY STONE, TYPE B
STA. 363+00 TO STA. 361+00	UNGROUTED QUARRY STONE, TYPE A	UNGROUTED QUARRY STONE, TYPE A
STA. 361+00 TO STA. 359+00	UNGROUTED QUARRY STONE, TYPE B	UNGROUTED QUARRY STONE, TYPE B
STA. 359+00 TO STA. 357+00	UNGROUTED QUARRY STONE, TYPE C	UNGROUTED QUARRY STONE, TYPE C
STA. 357+00 TO DOWNSTREAM MODEL LIMITS	EARTH	EARTH

MODEL DETAILS AND WATER SURFACE PROFILE

FINAL DESIGN

DISCHARGE 26,000 CFS

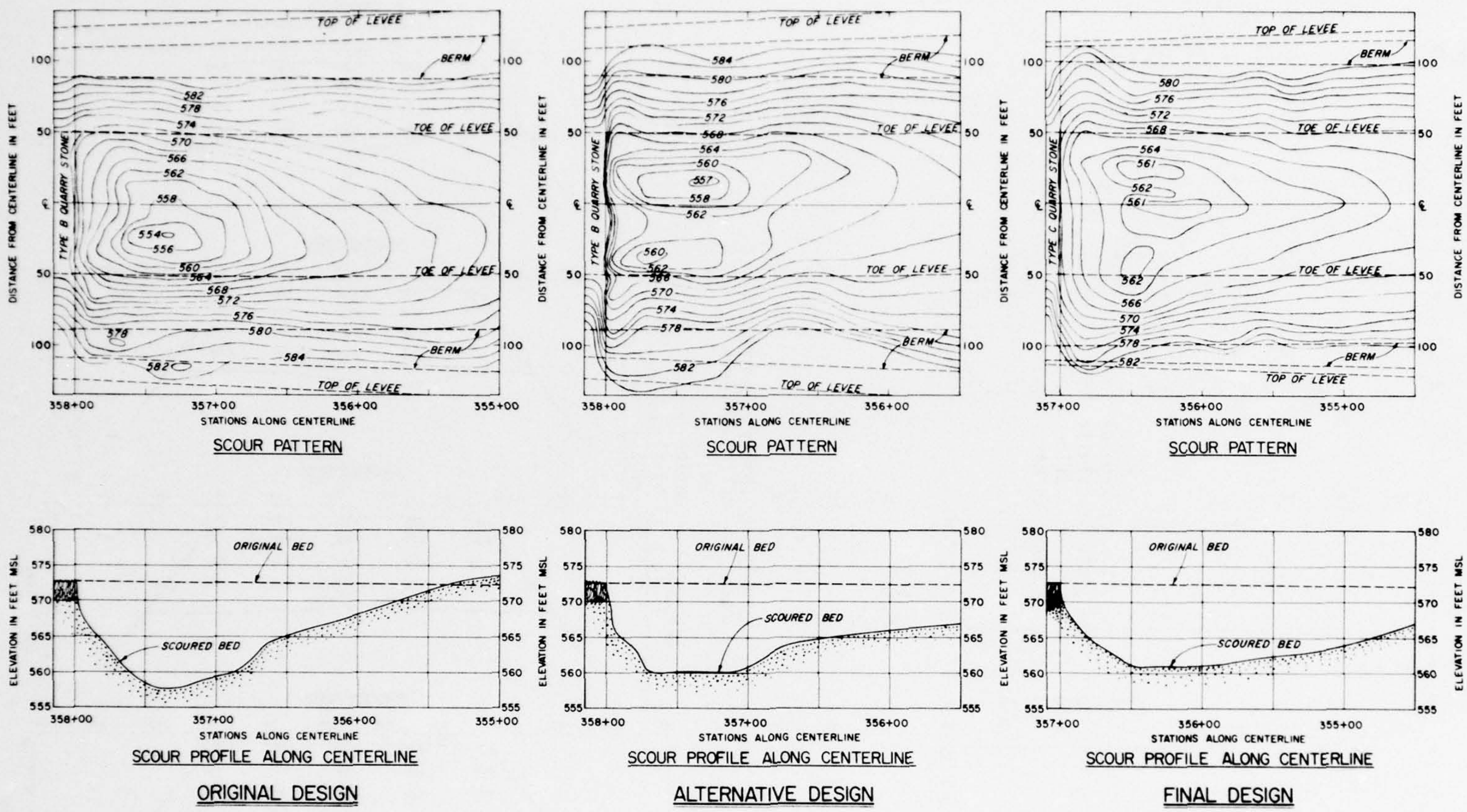


NOTES:
ALL SECTIONS SHOWN LOOKING DOWNSTREAM.
VELOCITIES ARE IN FEET PER SECOND.

VELOCITY DISTRIBUTION

FINAL DESIGN

DISCHARGE 26,000 CFS



NOTE
SCOUR DATA OBTAINED FROM A DISCHARGE OF 26,000 CFS FOR A PERIOD OF 20 HOURS.

SCOUR PATTERNS

ORIGINAL, ALTERNATIVE AND FINAL DESIGNS

PLATE 6

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

United States. Army. Corps of Engineers.
  Los Angeles District.
  Rock-lined transitions; hydraulic model investigation / [by Dave A. Barela]. [Vicksburg, Miss. : U. S. Waterways Experiment Station] ; Springfield, Va. : available from National Technical Information Service, 1978.
  77, [50] p., 37 leaves of plates : ill. ; 27 cm. (Report - U. S. Army Engineer District, Los Angeles ; 1-110)

1. Hydraulic models. 2. Open-channel flow. 5. Rock-lined channels. 4. Surface roughness (Hydraulics). 5. Transitions (Hydraulics). 6. Water flow. I. Barela, Dave A. II. Series: United States. Army. Corps of Engineers. Los Angeles District. Report ; 1-110.
TC159.L6 no.1-110