Division of Engineering Laboratories
Hydraulic Laboratory Branch

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES OF PALO VERDE DIVERSION DAM

Hydraulic Laboratory Report No. Hyd-408

DIVISION OF ENGINEERING LABORATORIES

COMMISSIONER'S OFFICE
DENVER, COLORADO

June 4, 1956

FOREWORD

Hydraulic model studies of Palo Verde Diversion Dam, a part of the Palo Verde Diversion Project, were conducted in the laboratory of the Bureau of Reclamation at Denver, Colorado, during the period from October 1954 to December 1955.

The final plans evolved from this study were developed through the cooperation of the staffs of the Dams Branch, the Canals Branch, the Mechanical Branch, the Hydrology Branch, and the Hydraulic Laboratory Branch.

During the course of the model studies, Messrs. L. G. Puls, F. A. Houck, and E. L. Watson of the Dams Branch, and A. W. Kidder and J. A. Hufferd of the Canals Branch frequently visited the laboratory to observe the model tests and to discuss results with A. J. Peterka, E. J. Carlson, and G. L. Beichley of the Hydraulic Laboratory Branch.

These studies were conducted by G. L. Beichley and supervised by A. J. Peterka under the direction of H. M. Martin.

CONTENTS

CONTENTS--Continued

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Laboratory Report No. Hyd-408
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Subject: Hydraulic model studies of Palo Verde Diversion Dam

SUMMARY

Hydraulic model studies of Palo Verde Diversion Dam, including the spillway, the canal headworks, the existing temporary rock weir, and the adjacent reaches of the river channel, Figures 1 through 14, were made on two models including a 1:28.3 scale sectional model of one spillway bay, Figure 15, and a 1:50 scale overall model, Figures 16 and 17. Model data showed that the general concept of the preliminary design was satisfactory, however, several modifications were necessary to develop the final plans.

The shape of the spillway section, including the stilling basin, was developed from comprehensive tests on the 1:28.3 scale sectional model, Figures 18 through 28. A stilling basin with a horizontal apron and a solid-type end sill, Figure 3, proved to be satisfactory in preventing excessive erosion and water surface roughness. This relatively simple basin which contained no baffle piers or dentates on the end sill will be economical to construct and should require the minimum in maintenance.

Studies were made on the riprap to be used to protect the riverbed downstream from the stilling basin, Figures 41 and 42. Both models were used in estimating the probable necessary stone size. Stone sizes of 24- to 36-inch diameter were recommended.

The recommended location of the canal headworks structure in relation to spillway was determined by trial in the 1:50 scale overall model after studying the flow characteristics in the approach area for other arrangements, Figures 29 through 39. It was found necessary to move the headworks inward toward the spillway and to reshape the right bank to eliminate a large eddy at the headworks entrance and numerous small eddies along the right bank which would tend to deposit sediment at the headworks entrance.

Air pockets that collected along the roof of the conduits in the canal headworks structure were eliminated by sloping the roofs upward.

The left approach training wall to the spillway was reduced in length and the curvature revised to eliminate an excessive contraction. The recommended wall provided smooth flow along its face, Figures 4 and 32, and was more economical to construct.

Flow through the gate section was satisfactory; however, for the design flow the gate trunnions were submerged, Figure 39. It was noted, too, that any floating debris will flow along the right side of each bay when the flow is uncontrolled.

The water surface elevation in the discharge channel was higher than expected, Figure 40B. Therefore, it was recommended that the discharge channel be widened and that its banks be elevated.

Surface waves were found to be highest for a controlled discharge of approximately 15,000 second-feet; those for an uncontrolled flow of 75,000 second-feet were nearly as high, Figure 43. Throughout the discharge range the waves along the channel banks were not objectionable.

The rock weir which is presently used as a diversion dam was studied to determine its backwater effect after completion of the diversion dam. The elevation to which the temporary rock weir should be removed, Figures 7 through 13, and the method of removing it were determined with the aid of the overall model, Figures 44 and 45.

The spillway was calibrated for both gate controlled and uncontrolled flow, Figures 46 through 49, both for present tail water conditions and for lower tail water elevations to be expected as a result of channel degradation. Over most of the discharge range the spillway discharge is of the submerged type. Preliminary calibration was done on the sectional model for preliminary use by the designers, Figures 46 and 47, and to aid in obtaining final calibration data on the overall model for prototype use, Figure 48. Water surface elevations at various places throughout the reservoir, in the discharge channel, and in the river channel were also recorded during the final calibration tests for possible prototype use, Figure 49.

Flow currents in the river channel downstream from the discharge channel were investigated in the overall model and it was determined that lands of the Colorado Indian Reservation along the left bank would not be endangered by excessive erosion. Velocity measurements and photographs of surface currents, Figure 50, showed that no erosion of any consequence would occur along the left bank.

INTRODUCTION

Palo Verde Diversion Dam is part of the Palo Verde Diversion Project located on the Colorado River about 9 miles northeast of Blythe, California, Figure 1. The dam, Figures 2, 3, 4, 5, and 6, will divert up to 1,800 second-feet of water to the Palo Verde Irrigation District and will replace the existing rock weir diversion dam shown in Figures 7, 8, 9, 10, 11, 12, and 13. The rock weir, at present, requires frequent repair and maintenance because floodwaters remove portions of the weir as shown in Figures 9, 10, 11, and 12.

The dam, Figure 2, an earth and rockfill structure approximately 1,850 feet long, including spillway, is approximately 40 feet high above the riverbed. On the right the earth dam butts against the rock knoll near the old intake structure for the Palo Verde Irrigation District. On the left the dam joins a levee designed to protect the lands of the Colorado River Indian Reservation. The crest of the dam and levee is 20 feet wide at elevation 296.

The spillway, Figures 2, 3, and 4, designed to pass 75,000 second-feet at reservoir elevation 290 will not reach this capacity until retrogression of the river channel and the discharge channel has lowered the tail water surface in the discharge channel a sufficient amount at some future date. Retrogression is expected to occur downstream from the dam as the clear reservoir water picks up a sediment load in the downstream channel. As a result, the tail water elevation is expected to become lower at the rate shown in Figure 14. Therefore, the capacity of the submerged spillway is expected to increase in future years.

The spillway approach channel is excavated to slope downward toward the crest at 10:1 from elevation 273 in the river channel to a level area at elevation 256 which is 3 feet below the crest. The level area extends upstream from the crest as shown in Figure 2. A curved training wall on the left, Figure 4, guides the water into the spillway. On the right the canal headworks structure, Figures 2 and 6, adjoining the spillway structure is designed to pass 1,800 second-feet at reservoir elevation 283.5 with the water surface elevation at the head of the canal at elevation 282.9.

The spillway crest is at elevation 259.0 and is equipped with three 50- by 24.91-foot-high radial gates, Figure 5. The spillway piers are 6 feet thick and extend to the downstream end of the spillway apron. Stoplog slots are provided at the upstream and downstream ends of the piers for unwatering the gate section and stilling basin area. The stilling basin, founded on rock, has a horizontal apron at elevation 245 with a solid-type end sill.

The headworks structure, Figure 6, consists of four conduits each 8 feet high by 12 feet wide and controlled by a top seal radial gate. The sill or bottom of the openings is at elevation 273.5. A concrete transition section is provided between the conduits and the canal. The canal joins the existing Palo Verde Irrigation District desilting basin near the point where the old intake canal emptied into it.

The excavated spillway discharge channel, Figure 2, slopes upward 6:1 from the stilling basin end sill at elevation 251.5 to elevation 260. After 100 feet of level bed the channel slopes upward again at the rate of 10:1 to elevation 270, the elevation of the river channel. As the channel bottom approaches elevation 270 its breadth is gradually increased to about 500 feet.

THE MODELS

Two models were used in the investigation. Each was constructed and tested in the Bureau of Reclamation Hydraulic Laboratory at the Denver Federal Center. The first was a 1:28.3 scale sectional model of one spillway bay, Figure 15. The other was an overall 1:50 scale model of the earth dam, spillway, canal headworks, and surrounding topography upstream and downstream of the site, Figures 7, 16, and 17.

The 1:28.3 Scale Sectional Model

The 1:28.3 scale sectional model, Figure 15, was installed in one of the laboratory's test flumes. The flume is approximately 24 inches wide by 43 feet 6 inches long, and is provided with a 4- by 13-foot 8-inch glass window on one side for observing and photographing flow characteristics.

The spillway crest, the radial gate, and the bucket energy dissipator of the preliminary spillway were constructed of sheet metal. The spillway piers and energy dissipating baffle piers were made of wood and soldered in place by means of clip angles. The spillway approach channel was formed with 1-1/2-inch rock which did not move during the tests, while the river channel bed downstream from the spillway was formed in 1/4-inch pea-gravel to provide a movable bed for erosion tests. A sample of the gravel used had the following analysis:

Mechanical Sieve Analysis of
1/4-inch Gravel Sample

Sieve size	Percent retained on sieve by weight
3/8-inch	2.0
No. 4 sieve	89.5
No. 8 sieve	8.2
Less than No. 8 sieve	0.3

Water was supplied to the model from the laboratory's underfloor reservoir through the permanently installed 12-inch supply line that encircles the interior of the laboratory. The quantity of flow was measured by use of the permanently installed venturi meters. The elevation of the reservoir water surface was measured using a hook gage in a transparent plastic well attached to the flume. The head was measured at a point approximately 4.6 feet upstream from the spillway crest line. A hinged tailgate controlled the elevation of the water surface downstream from the structure. Tail water elevations were measured at a point approximately 13 feet downstream from the crest line using an arrangement similar to the headwater gage.

The 1:50 Scale Overall Model

The 1:50 scale model, Figures 7, 16, and 17, of the earth dam, spillway, canal headworks, temporary rock weir, and surrounding topography upstream and downstream of the site was constructed in an existing sheet-metal-lined box approximately 26 feet wide by 60 feet long and 24 inches deep.

The dam was constructed across the width of the box at about the midpoint. The interior of the model dam was provided with a sheet-metal bulkhead to prevent water passing through the sand and concrete. The top and downstream face of the dam were molded in concrete on metal lath to provide a walkway across the model. The upstream face of the dam was molded in sand except for a short piece adjacent to the spillway where flow velocities along the face of the dam might cause sloughing. The excavated spillway approach area was also formed in concrete. All other topography upstream of the dam was molded in sand to simplify model construction. For some of the tests a 1/4-inch layer of fine, white sand was used to cover the concrete approach area to indicate bottom flow currents. The white sand has a mean diameter of approximately 0.2 mm with 90 percent passing the 40-mesh and 10 percent being retained on the 100-mesh U. S. Standard screens (0.43 to 0.15 mm).

The model topography upstream from the dam included the river channel and adjacent area for an overall width approximately 1,300 feet, and extended upstream approximately 1,200 feet from the dam to include the presently used rock weir diversion dam and a short reach of river upstream from it. The rock weir in the model was constructed of 1-1/2-inch rock placed on top of the sand topography to weir crest elevation.

The downstream topography was of approximately the same area as the upstream topography and included the spillway discharge channel and a reach of river downstream from the discharge channel. The knoll to the left of the spillway was molded in concrete up to elevation 300. Contours were not reproduced above elevation 300, to provide flat working areas on the model. The riverbanks and the banks of the discharge channel were molded in concrete to provide bank stability but were covered with sand about 3/4 inch thick to provide a surface that would show bank erosion tendencies. The river and discharge channels, between banks, were molded entirely in sand to provide a movable bed for erosion tests. Bed rock in the discharge channel adjacent to the stilling basin was molded in concrete, since the rock quality was sufficiently high to resist erosive forces to be expected in the area.

Riprap in the discharge channel was reproduced in the model in various ways and subjected to a series of tests. Sizes varied from pea-gravel, the same as used in the 1:28.3 model, to 3/4-inch gravel which represented prototype stones 36 inches in diameter.

Samples of the movable bed sand and the 3/4-inch riprap gravel had the following size analysis:

Mechanical Sieve Analysis of the Sand Sample

Sieve size	Percent retained on sieve by weight
No. 4	--
No. 8	3
No. 16	27
No. 30	36
No. 50	27
No. 100	7
Pan	0

Mechanical Sieve Analysis of the 3/4-inch Gravel Sample

Sieve size	Percent retained on sieve by weight
3/4-inch	4.6
3/8-inch	75.3
No. 4	20.1

The spillway crest and stilling basin were constructed of concrete. Five sheet-metal templates were cut to the exact shape of the crest and basin. The templates were equally spaced and soldered to a supporting framework which in turn was soldered to the floor of the model box. Gravel was placed between templates to within 3/4 inch of the final profile. Concrete was placed over the gravel and screeded smooth to the sheet-metal templates. The end sill and piers were made of wood, soaked in linseed oil, and painted to prevent warping. The radial gates were constructed of sheet metal and pivoted on a 1/8-inch rod that extended across the full width of the spillway at the gate pin location. The vertical face of the left approach training wall was also constructed of sheet metal and was fitted into the adjacent concrete parts of the structure.

The canal headworks structure, including the radial control gates, was constructed mostly of sheet metal and soldered to the floor of the model box. The roof of the conduits was of transparent plastic so that flow through the conduits could be observed. The walls between conduits were made of treated and painted wood.

Water was supplied to the model from a portable vertical pump through an 8-inch pipe. An orifice venturi meter in the pipe was used to measure discharges. Flow entered the upstream end of the head box and was quieted by an 8-inch rock baffle before it entered the model reservoir upstream of the dam. Flow entering the model reservoir had the same relative direction as in the prototype.

The water surface elevation in the river channel downstream from the dam was controlled by a hinged tailgate across the downstream end of the model. A sand trap was constructed immediately upstream of the gate to prevent loss of sand from the model.

Flow through the canal headworks emptied into a separate wasteway channel along the inside of the model box. A 90° V-notch weir, preceded by a rock baffle, was installed in this wasteway to measure the canal headworks discharge.

Point gages were used to measure water surface elevations at many places in the model. The location of these gages is shown in Figure 17.

THE INVESTIGATION

In the sectional model the investigation was concerned with developing the spillway and stilling basin to provide optimum performance with a minimum of structure. In the overall model the investigation was concerned with checking the spillway and stilling basin performance particularly for side effects, waves, and riprap sizes required, and, in addition, developing the design of the entire arrangement of structures from the hydraulic viewpoint.

To complete the entire study in the overall model extensive tests were made to (1) determine the most effective location of the canal headworks structure in relation to the spillway to reduce the amount of silt entering the canal intakes, (2) determine the effect of the rock weir for full and partial heights on water surface elevations both upstream and downstream of the weir so that weir removal recommendations could be made, (3) study flow currents in the spillway approach to determine their effect on spillway and canal headworks performance, (4) determine the capacity of the spillway for the changing tail water levels expected over a period of years as a result of river channel degradation, (5) calibrate the spillway for the range of submergence to be expected, including the effect of the riprap in the discharge channel in preventing a lowering of the tail water in the discharge channel, and (6) study the downstream flow conditions for a range of flood flows to determine the possibility of riverbank erosion.

The spillway was designed to pass 75,000 second-feet or 25,000 second-feet per bay at reservoir elevation 290. Normally the reservoir would be controlled at elevation 283.5 by means of a radial gate in each bay.

Tail water elevations for the design flow and the lower discharges used in the model tests were determined from the tail water curves in Figure 14, which show the expected rate of riverbed degradation in terms of tail water elevations. Tail water references in this report, therefore, contain the year in which the elevation is first expected to occur.

Development of Spillway in the Sectional Model

Tests to develop the shape of the spillway and the stilling basin were conducted on the 1:28.3 scale sectional model of one spillway bay, Figure 15. The section developed from these studies was then installed in the 1:50 scale overall model and tested to verify the capacity and performance of the entire spillway.

The Preliminary Spillway Section

The preliminary spillway consisted of an ogee section, shown in Figure 18, and a 30-foot-radius Angostura-type slotted bucket energy dissipator. The spillway crest was at elevation 259 and the invert of the bucket at elevation 240. The radial gates, 50 feet wide by 25.5 feet high, were placed between spillway piers 6 feet thick that extended downstream to the bucket invert. The gravel in the movable bed was molded on a 6:1 slope upward from the bucket lip.

The model was operated for a range of uncontrolled flows up to 25,000 second-feet, first, with the present expected tail water elevation and then with the lower tail water expected in year 1972. When present expected tail water elevation was used with each of these flow conditions, the water surface throughout the model was smooth and no erosion of the riverbed occurred for any discharge, probably because of the relatively deep tail water. Flow did not follow the spillway profile into the bucket but rather passed over the crest and expanded gradually to fill the flow area at the end of the stilling basin. The fall from reservoir to tail water under these conditions was hardly enough to require an energy dissipator.

The model discharging the design flow uncontrolled with present tail water is shown in Figure 19. The water surface downstream shows only small standing waves which are less harmful to the channel banks than traveling waves.

The model showed the need for an energy dissipator for gate controlled flows, particularly when the tail water elevation was at the expected year 1972 elevation. For this operation the flow currents followed the crest shape into the bucket. However, water surface roughness and erosion of the river channel were still not considered excessive. The most severe operating condition occurred for a gate controlled flow of 8,000 second-feet at normal reservoir elevation 283.5 and low tail water, Figure 20A. Retrogression is expected to lower the tail water 8 feet to elevation 272 by the year 1972. Elevation 271, however, was used in the model test to represent the most severe operating condition.

The water surface was rough in the bucket but smooth in the channel. Bed erosion stabilized at elevation 245 after a 30-minute model scour test, as shown in Figure 20A. The elevation of the eroded hole is the same as the bucket invert elevation. The material adjacent to the bucket lip was eroded only slightly.

A series of 30-minute model erosion tests was then made to determine whether the baffles in the Angostura-type bucket energy dissipator could be eliminated or modified to provide a more economical design without sacrificing good hydraulic performance. The tests were all conducted using the most severe operating condition described above.

In the first modification the baffles, Figure 20B, were removed. As the flow currents passed over and left the apron they scoured the channel bed for quite some distance downstream. The currents then turned upward producing a rise in the water surface. Since the main current followed along the channel bed, the water surface was smooth except for the slight rise in the water surface but erosion of the channel bed was excessive.

It was concluded that the baffles were valuable in that they directed part of the flow currents in the basin upward without producing excessive disturbances in the water surface. Only the currents that passed through the slots reached the end of the apron to erode the channel bed. Therefore, erosion of the channel bed was reduced by the use of baffles.

In the second modification a solid sill replaced the baffles, Figure 20C. With this arrangement none of the currents reached the end of the apron; instead, all currents were directed upward. As a result, the water surface was very rough and an upstream ground roller developed downstream from the sill which deposited bed material on the apron. Performance was considered to be very poor. It was concluded from these two modifications that slots were beneficial in minimizing water surface roughness and that baffles were helpful in minimizing the erosion of the channel bed.

A simpler and perhaps more economical version of the Angostura-type baffles, shown in Figure 21, was tested. It was believed that the curved faces or rounded edges of the baffle piers were not necessary since they would not be subjected to high velocity flows. Also, it was believed that fewer baffles might be used if the same ratio of solid to open spaces could be maintained. Therefore, modified baffles nearly twice the width of the Angostura baffle and spaced twice as far apart as in the Angostura design were tested. Operation for a range of discharges showed that the flow currents through the slots caused deeper and more extensive erosion than that in the preliminary design. Water surface roughness was about the same. No further consideration was given to this design.

Modified baffles four times the width of the Angostura baffles and spaced about 1-1/2 times farther apart, shown in Figure 22C, did not perform satisfactorily either for two reasons. First, because of the fewer number of slots the water surface was rougher than in the preliminary design. Second, the slots were so far apart that side eddies carried bed material back onto the apron and swirled the material around behind the baffles. In the prototype structure, material moving back and forth on the apron could cause severe damage to the concrete.

The baffles 7 feet wide, shown in Figure 21, were again tested. This time, however, 18-inch spacing, as shown in Figure 18, was used. The erosion and water surface roughness were about the same as for the preliminary design without modification. Since this 7-foot-wide modified baffle, Figure 21, was more economical to construct than the preliminary Angostura-type baffle, this design was considered to be the best tested thus far.

To improve on this arrangement, the baffles were tapered, Figure 22C. In tapering the 7-foot-wide baffles the spacing was increased to about 1-1/2 times the Angostura spacing to provide lower velocities at the downstream end and to allow trapped material to become free if it once started through the slot. The baffles were tapered from 7 feet wide upstream to 6 feet 9 inches downstream, making the upstream slot width 24 inches and the downstream width 27 inches. Erosion tests showed that the tapered slots were almost as good as the narrower slots (compare Figure 22B with 22C). Since the water surface was smooth this arrangement was indicated as suitable for use in the prototype. However, at this same time, the designers decided that the spillway piers should be extended to the end of the bucket for structural reasons. This decision, combined with the opinion in the laboratory that the entire stilling basin could be made smaller, led to tests on another design.

Spillway Section with 25-foot-radius Bucket

Before discussing the next tests a brief discussion of the findings of the earlier tests may be of interest. The tests showed that the baffles aid in breaking up the flow currents in the bucket. Some of the currents pass through the slots unobstructed, while others are turned upward at a steeper angle by the baffles. The currents that pass through the slots are spread out on the apron and are the cause of the erosion of the channel bed. The currents turned upward by the baffles cause water surface roughness. If the slots are too wide excessive erosion occurs while if the baffles are too wide the flow through the slots will not spread to the full width of the baffles at the end of the apron, producing eddies which move bed material on and off the apron. Wider baffles direct more water upward and the water surface becomes rougher. If the slots are eliminated completely, bed material is deposited on the apron by the induced ground roller and the water surface is still rougher. It is therefore necessary to determine the best balance between baffle and slot width.

In testing the 25-foot-radius Angostura-type bucket, Figure 23, the modified baffles, Figure 21, were used. Since the spillway piers in these next designs extended to the end of the apron, the spacing of the baffles was adjusted to place five baffles in each bay. The baffles tapered from 7 feet to 6 feet 9 inches and the slots from 32 to 35 inches.

Scour occurred along the apron lip and was deeper than for the preliminary design, which indicated that the slots were too wide. This was also evident from the fact that the water surface was quite smooth.

Tests using six baffles per bay were then made. The slots tapered from 18 to 21 inches wide. With this spacing no scour occurred along the apron lip and very little in the bed downstream but the water surface was a little rougher and a few particles of bed material swirled onto the apron.

For the next test, the slot width was increased to 24 inches at the upstream end and 27 inches at the downstream end, Figure 24A, which is the same as recommended with the preliminary

30-foot-radius bucket. To accomplish this it was necessary to use six baffles, tapering from 5 feet 9 inches upstream to approximately 5 feet 6 inches downstream, in each bay. Erosion was negligible along the apron lip and in the river channel. The water surface was fairly smooth. Performance in both respects was considered to be better than for the preliminary design.

A solid end sill, shown in Figure 25, was placed in the 25-foot-radius bucket for the next tests. With the horizontal top, Figure 25, very little erosion occurred, Figure 24B, and there was no general movement of material along the apron. However, a considerable amount of bed material did swirl on and off the apron which was objectionable. Also, the water surface was rougher than when baffles were used.

The horizontal top was changed to a sloping top as shown in Figure 25. For this arrangement very little erosion occurred, as shown in Figure 24C; practically no movement of material occurred along the lip, and no material swirled onto the apron. The water surface was rougher than when baffles were used, but the designers believed that the waves in evidence would be tolerable in the prototype.

This design was recommended for prototype use, however, data received from the field by the designers indicated that the elevation of good foundation rock was sufficiently high to allow placing the invert of the bucket as high as elevation 245. This information, combined with the laboratory's suggestion that a horizontal apron and end sill could be substituted for the bucket and sill, led to the final tests.

Recommended Spillway Section with Horizontal Apron

A horizontal apron at elevation 245, shown in Figure 3, with spillway piers extending to the end of the apron was tested to determine the shape of the end sill and whether the horizontal apron could be substituted for the bucket. The most severe operating condition was used in the tests--a gate controlled flow of 8,000 second-feet with reservoir elevation 283.5 and tail water elevation 271.

The first sill tested had a 2:1 upstream slope, as shown in Figure 26A. Erosion of the bed material was very minor, Figure 27A, and very little movement of material occurred along the lip of the apron. The water surface was a little rough, with wave heights from crest to trough reaching 3 to 4 feet. Although the design appeared adequate, it was felt that improvements could be made.

A sill with a double slope, shown in Figure 26B, similar to the one tested with the 25-foot-radius bucket was investigated. Results were practically identical to those for the 2:1 sloping sill, Figure 27B.

A 4:1 sloping sill, Figure 26C, was tested. Again, practically the same results were obtained, Figure 27C.

Six baffles 5 feet 6 inches wide and spaced 27 inches apart, previously used in the 25-foot-radius bucket, were placed on the 4:1 sloping sill as shown in Figure 26D. Erosion was reduced and water surface roughness was improved as shown in Figure 28A. When the baffles were moved upstream onto the apron, as shown in Figure 26E, some of the water surface roughness was moved upstream into the basin between the spillway piers. Erosion was the same as with the piers on the slope, Figure 28B.

Since in the previous arrangement the 4:1 slope appeared to be of little benefit, it was replaced by a 2:1 sloping sill as shown in Figure 26F. Bed erosion and water surface roughness results were better than for any arrangement tested, Figure 28C. Erosion was negligible and wave heights were reduced to about 1-1/2 to 2 feet. Tests for other operating conditions also showed good results. For maximum discharge, 25,000 second-feet per bay, the water surface was smooth and appeared similar to that for the preliminary bucket. Standing waves occurred in both cases but were not considered objectionable. A horizontal apron at elevation 245, with a 2:1 end sill and intermediate baffles, as shown in Figure 26F, was recommended for prototype use on the basis of the sectional model studies.

Spillway in Overall Model

The designers found it necessary to alter the end sill of the recommended basin, as shown in Figure 3, to provide a horizontal bearing surface for the stoplogs. It was also their opinion that the baffles in the recommended design did not add sufficient improvement to warrant their extra cost. Consequently, it was decided to test the spillway section without baffles in the overall model, Figures 16 and 17. The spillway crest section was also modified at this time, as shown in Figure 3, to provide a better seal between stoplogs and crest upstream of the crest.

Flow Through the Approach Channel

The preliminary arrangement of the spillway, canal headworks, and approach channel is shown in Figures 29 and 30. The center line of the headworks is at an angle of 60° to the center line of the spillway. Between the two structures is a 30-foot-radius training wall tangent to the right wall of the spillway and joining the left wall of the headworks. The sill of the headworks, Figure 31, is at elevation 275.5, 16.5 feet above the spillway crest and the spillway approach channel bed.

During the tests on the approach area it was necessary to modify the left approach wall, the right bank, and the location of the canal headworks structure. These modifications are discussed as they occurred during the testing program in the following sections.

A discharge of 15,000 second-feet was expected to be a common occurrence in the prototype with the reservoir controlled to elevation 283.5, therefore, the model study was concerned with the spillway discharging 15,000 second-feet as well as with discharges up to and including the design flow of 75,000 second-feet. The design flow approaching the spillway is shown in Figure 30B. A flow disturbance occurred along the left spillway approach wall and mild eddies occurred in front of the headworks structure and along the right bank upstream from the headworks.

Flow at left wall. The flow disturbance along the left approach wall occurred for uncontrolled discharges of 34,000 second-feet or greater and increased with the higher flows. The drawdown water surface caused by an excessive flow contraction is shown in Figures 30A and 32A as it occurred for 75,000 second-feet. This action caused a visible as well as measurable reduction in discharge. To remedy the flow conditions and reduce the cost of this long wall, two revised walls were tested. The first had a 24-foot-radius section with one end tangent to the left wall of the spillway and the other extended parallel to the dam axis as shown in Figure 32B. The 24-foot-radius turn proved to be too sharp and the drawdown in the water surface was even more pronounced, as shown in Figure 32B. The 120°, 51-foot-radius curve, shown in Figure 4, was found to be very satisfactory, as shown in Figure 32C. At maximum discharge the drawdown was barely noticeable. The latter wall was therefore recommended for prototype use.

Flow at right bank. The eddies near the canal intake structure occurred for all discharges tested including 15,000, 34,000, and 75,000 second-feet, as shown in Figures 33 and 34A. Although the eddies were mild they indicated inefficient use of the approach channel. To determine whether other objectionable flow patterns occurred below the water surface, a layer of the fine, white sand 1/4 inch thick was placed on the concrete surface of the approach channel before each test for 15,000, 34,000, and 75,000 second-feet. For 15,000 second-feet the white sand did not move sufficiently to clearly define a flow pattern, but for 34,000 and 75,000 second-feet sufficient sand was moved to establish the direction of the bottom currents, as shown in Figure 34B and C. The riffles in the sand in front of the headworks structure and along the warped transition showed the bottom currents to be in an upstream direction. Therefore, the eddy action was concluded to be general throughout the depth of the flow.

To reduce the eddies in the spillway and canal intake approach, the right approach bank line was made straight, as shown in Figure 35. To do this the projecting point upstream of the headworks, shown in Figure

30, that extended out into the approach as far as the right spillway pier was excavated about 50 feet, while the bank just upstream of the headworks was filled in the model (not excavated in the prototype) to form a straight line. Operation with this revision showed that a large eddy persisted for all discharges both on the water surface and on the approach bed, as shown in Figure 35, for 75,000 second-feet.

To improve the approach conditions, the headworks structure was moved out into the spillway approach channel so that the upstream face of the intake was at an angle of 10° with the right training wall of the spillway, eliminating the curved wall between the structures. The entire right bank was also moved inward to form a smooth spiral curve, as shown in Figure 36A. To accomplish this in the prototype, part of the bank would have to be constructed with fill material. A warped transition was used between the vertical intake face and the sloping bank.

The purpose in curving the right bank was to provide a flow with a "roping action." This action may be described as follows. Surface water should roll off the main flow approaching the spillway and enter the headworks. Thus, water with the least amount of suspended material would enter the canal system while the heavier silt laden currents would pass through the spillway.

The flow currents observed for a range of discharges occurred very much as anticipated. Flow was smooth along the right bank and no eddies occurred, as shown in Figure 36B. However, the narrower approach channel created by realigning the right bank produced higher velocities; as a result, the flow around the left training wall was again disturbed, as shown in Figure 36A and C.

To compensate for the narrower approach channel, the right bank was realigned as shown in the recommended layout in Figures 16B, 37, and 38. The position of the headworks was not changed, but the right bank was excavated in a straight line to the right of the preliminary bank line, similar to the second trial layout shown in Figure 35.

While these modifications were being made, other changes were made in the headworks structure itself mainly for structural reasons. The principal change was to move the conduit entrances and control gates downstream along the headworks center line so that the entrance to the conduits was recessed 13 feet from the face of the structure, as shown in Figures 6 and 16B. Also, the sill crest and floor of the structure were lowered from elevation 275.5 to 273.5.

In the recommended arrangement, flow currents, shown in Figure 38, approached the spillway and headworks in a very satisfactory manner. No large eddies occurred near the headworks. Surface currents flowed directly to the spillway and headworks, as shown by the confetti streaks in Figure 38, for uncontrolled spillway flow, while dye injected along the approach bottom showed that bottom currents flowed directly to the spillway. Similar flow conditions also occurred when the spillway flow was gate controlled. This was a very desirable flow pattern since the currents along the bottom that either carried or might move sediment passed through the spillway, while the surface currents passed through the headworks.

General flow conditions. For one operating condition, 50,000 second-feet with tail water expected in 1967, a very rough water surface in the form of standing waves occurred in the spillway approach area, as shown in Figure 39A. This condition might occur in future years when the tail water elevation is low and the spillway discharge is, therefore, greater at normal reservoir elevation 283.5 than at present. The rough water surface evident in the approach area was caused by the high areas located just upstream from the excavated slope in the approach channel. In this reach, almost the entire width of the approach channel is above elevation 270 with a high point at elevation 273. Waves formed just downstream from the high areas and were estimated to be 3 to 4 feet high. Since the waves were standing waves, however, and did not extend into the downstream channel, they were not particularly harmful. It is also possible that waves of this type will never occur in the prototype because the prototype bed is erodible and probably will degrade sufficiently to provide a deeper channel. No recommendations for relieving this condition were therefore made.

Flow Through the Spillway

Flow currents through the spillway structure were photographed for the design flow of 75,000 second-feet and are shown in Figure 39B. The gate trunnions will be submerged; floating debris will flow along the right side of each bay. If the debris is large enough, the right side of the radial gates or trunnions might be damaged. However, this is not considered to be very likely.

Flow Through the Discharge Channel

The model is shown in Figure 40A discharging 15,000 second-feet equally distributed between the three bays, and reservoir elevation 283.5 with present expected tail water elevation: The water surface in the discharge channel was not rough and very little erosion occurred. The channel bed of the model had been molded in sand for these early tests, and the foundation rock that extended along the toe of the basin and along the left training wall had been molded in concrete. Riprap was not used in these first tests except along the left bank of the discharge channel to prevent sloughing of the sand in the model.

The model discharging 75,000 second-feet with the expected present tail water elevation is shown in Figure 40B. The water surface in the discharge channel was not very rough; however, the flow washed over the discharge channel banks as shown and the flow eroded the channel bed from elevation 260 to 227 in a very short while, as shown in Figure 40C.

Channel erosion. Erosion of the magnitude shown in Figure 40C did not occur in the sectional model nor did the sectional model indicate excessive turbulence along the level bed for the design discharge of 75,000 second-feet. Therefore, it was apparent that the relatively fine sand was too small for the average velocity of flow in the overall model. The sand represents, on a linear scale basis, loose gravel in the prototype ranging in size from 3/4 inch to 2-1/4 inches in diameter. In the sectional model the velocity of flow for 75,000 second-feet discharging under present tail water conditions was not sufficient to move the 1/4-inch pea-gravel which represents, on a linear scale basis, approximately 7-inch-diameter stones in the prototype.

To determine the size of riprap necessary in the discharge channel for the most severe operating condition, the maximum anticipated velocity was computed and the riprap size determined by use of Figure 41. It was anticipated that a discharge of 75,000 second-feet with tail water conditions for the year 1972 would produce the greatest velocity of flow. The average velocity of flow for this condition was computed to be about 18 feet per second, with an estimated bottom velocity of 3/4 of the average, or about 13.5 feet per second. Figure 41 shows that individual rock pieces should be about 27 inches in diameter. However, if it is assumed that the velocity may not be uniform across the channel width or that velocities might become even greater after year 1972, the maximum bottom velocity might become sufficient to move riprap greater than 3 feet in diameter.

As a result of the above analysis, gravel from 3/8 to 3/4 inch (analysis shown on page 6) was used in the model. This material represented prototype riprap ranging in size from 18 to 36 inches in diameter. It was placed as shown in Figures 2 and 42A about 0.1 of a foot deep to represent a 5-foot depth in the prototype.

Tests were begun with 15,000 second-feet and tail water at the elevation expected in year 1962. Discharges were then increased in increments of about 15,000 second-feet up to 75,000 second-feet. Each discharge was tested for about 30 minutes. No erosion of the riprap occurred, even for the design discharge of 75,000 second-feet. A discharge of 75,000 second-feet with tail water elevation for year 1972 could not be set in the model without partial removal of the riverbed, but it appeared from the other tests that 36-inch rock pieces and probably even 24-inch pieces would be sufficient.

The tests also showed that for 45,000 second-feet with the 1962 expected tail water, the channel bed downstream from the riprap began to erode. In 30 minutes of model operation the unprotected part of the bed eroded to the pattern shown in Figure 42B. It was noted that heavier scour occurred along the right bank and that sand was deposited along the left bank and along the left training wall of the stilling basin. Bottom currents along the left wall of the stilling basin were apparently in an upstream direction, but no serious consequences appeared. When the discharge was increased to 75,000 second-feet for 30 more minutes of operation (73,000 second-feet through spillway, 2,000 through canal headworks), erosion of the sand continued, producing the pattern shown in Figure 42C. This scour pattern, as well as the one in Figure 42B for 45,000 second-feet, shows the velocity of flow to be greater near the right bank of the channel, probably caused by more water being discharged by the right bay of the spillway. However, riprap along the right channel bank or on the channel bed did not move.

Tests were then made on the 1:50 scale model with the sand bed of the discharge channel, downstream from the riprap area, covered with a 1/2-inch layer of 1/4-inch pea-gravel to elevation 270. The gravel was the same as used in the 1:28.3 scale model. In the 1:50 model the pea-gravel represented prototype stones approximately 12 inches in diameter. Using present expected tail water elevations with each discharge, a stone here and there began to move only when 50,000 second-feet was reached. The stones that did move were smaller than the average and probably represented stones about 6 inches in diameter. The water surface directly above the gravel for 50,000 second-feet was at elevation 284, and the channel width was approximately 250 feet so that the average velocity that moved these stones was approximately 14 feet per second. This test shows again that 36-inch riprap is probably more than ample to protect the channel.

Channel water surface. The model discharging 75,000 second-feet with the expected present tail water elevation 287.5 feet in the river channel is shown in Figure 40A. The water surface in the discharge channel was not very rough; however, the flow washed over the discharge channel banks.

The top of the training wall along the left side of the discharge channel and extending downstream from the stilling basin was at elevation 280 and was submerged by the design flow. The riprapped banks of the discharge channel at elevation 290 were also submerged and it was decided to raise the banks to elevation 296 and widen the preliminary discharge channel. Increasing the channel width also improved the flow conditions for smaller discharges since they showed a tendency to cut across the downstream end of the right bank of the discharge channel. As a result of these studies the preliminary channel, beginning at the downstream end of the basin, was widened to the dimensions shown in Figure 2.

Widening the channel reduced the water surface elevation in the discharge channel below elevation 290; however, the designers wished to maintain the banks of the discharge channel at elevation 296.

Water surface roughness in the discharge channel was investigated in the 1:50 model to determine the maximum wave heights to be expected in the prototype. Tests showed that the highest waves occurred just downstream of the center bay for 15,000 second-feet, Figure 43A, with reservoir elevation 283.5 and tail water at the 1962 expected elevation. The wave heights measured along the left and right training walls were 1-1/2 and occasionally 2 feet high. It was estimated that the waves in the middle of the channel downstream from the center bay were 3 to 4 feet high, which is in agreement with the waves measured in the 1:28.3 scale model.

As the spillway gates were opened and the discharge increased, maintaining the reservoir at elevation 283.5, the wave heights became less and less until the water surface became very smooth for uncontrolled flow at reservoir elevation 283.5, measured near the headworks. Even with the tail water lowered to the expected 1962 elevation, the water surface in the discharge channel was smooth.

As the discharge was increased still further to 75,000 second-feet, the waves again became higher. For 75,000 second-feet with tail water expected in year 1962, the waves from crest to trough measured 2 to 3 feet high downstream from the right bay, as shown in Figure 43B. Exact wave measurements were not made since no severe waves appeared to exist along the channel banks.

Canal Headworks Structure

No extensive tests were made on the hydraulic characteristics of the 1:50 scale model of the headworks structure itself, Figure 30; but while operating the headworks structure, it was noted that air pockets collected along the roof of the conduits. Since these might affect the accuracy of any type of metering device, the roof of the conduits was sloped upward to provide a 3-inch rise, as shown in Figure 6. Thus, air entering from the upstream end will flow along the roof and exit at the downstream end.

A vortex was noted above the entrance to the right conduit when the gate was open. No serious consequences from the vortex could be contemplated however.

Rock Weir

General Description

The rock weir, Figures 7 through 13, is located in the reservoir area of the new dam approximately 800 feet upstream from the spillway and is approximately 550 feet long. It was modeled as shown in Figures 16A and 44.

The elevation of the rock weir crest could not be determined precisely from available prototype data; however, it was known that the reservoir surface upstream from the weir is at elevation 288 when the river is discharging 15,000 second-feet. The approximate elevation of the rock weir in the model was, therefore, determined by building up the weir crest until the water surface upstream reached elevation 288 for a discharge of 15,000 second-feet. The prototype weir crest elevation was thus determined to be at elevation 286. The prototype weir might, however, be more or less porous than the model which would affect the crest elevation determined above. Since the determined elevation agreed with the best estimates of the prototype elevation, crest elevation 286 was used in all tests.

It is interesting to note in Figure 44A the two rather large slow moving eddies that occur downstream from the weir near the right and left banks. These two eddies are probably responsible for the two major depressions in the prototype riverbed topography shown in Figure 13. The depressions are scoured holes with bottom elevation 240 and are located directly beneath the eddies. Note, too, that the large eddy on the right creates another small eddy rotating in reverse direction. These facts are pointed out here to indicate that the model is capable of representing flow characteristics which may or may not have been observed with even careful study of the prototype. For example, the scour holes which were molded in sand during the model construction could not be explained until the model was operated.

Removal of the Weir

In the planning of the permanent dam, it was proposed that the rock weir be removed sufficiently, after completion of the dam and spillway, to prevent backwater effects. Backwater effects in the form of increased reservoir elevations were to be avoided since the top of the Indian levees along the left bank upstream are at elevation 296 and it is important that they not be overtopped.

Breaching the weir. The designers proposed to remove the weir by breaching it near the right bank for a length of 50 to 100 feet which would lower the water surface behind the weir sufficiently to drive equipment out on the weir to remove the remainder. To follow this plan it would be necessary to have assurance that the initial breach could be made wide enough to lower the water surface upstream of the weir below weir crest elevation so that a temporary road could be built on top of the weir out from the left bank.

It was apparent that if the spillway gates could be fully opened during the weir removal operation the elevation of the water surface both upstream and downstream of the breach would be lower than if normal reservoir elevation 283.5 was maintained at the headworks. However, with a lower reservoir elevation the full quota of 1,800 second-feet of irrigation water could not be delivered to the Palo Verde Irrigation District. Model studies were therefore made to obtain data for this and other weir removal plans.

Three arrangements of the rock weir were tested. The first was with the entire weir in place with weir crest elevation 286, the second was with a 50-foot-wide breach excavated to elevation 270, and the third was with a 100-foot-wide breach excavated to elevation 270.

The tests were run with the spillway gates fully open so that the water surface elevation at the canal intake structure was for uncontrolled spillway flow. Curves plotted from this data for the three different weir conditions are shown in Figure 45A.

For 15,000 second-feet the 50-foot breach reduced the water surface elevation upstream of the rock weir from elevation 288 to 285.3 feet which is slightly below the estimated rock weir crest elevation. The water surface near the canal intake was found to be at elevation 278.5, 3 feet above the preliminary sill elevation of the headworks which was the established floor elevation at the time of the weir removal tests. The irrigation district, therefore, would receive about one-third of the full water quota, or 600 second-feet, during the removal operation. The 100-foot breach reduced the water surface upstream of the weir further to elevation 283.3, but the water surface near the canal intake remained at elevation 278.5.

When the spillway gates were closed to control the flow and maintain reservoir elevation 283.5 at the headworks during the weir removal, the 50-foot breach lowered the water surface upstream of the weir from elevation 288.4 to 286.3. This may or may not be sufficiently low for the proposed method of construction of a temporary road on the weir.

These tests indicated that a 50-foot breach would lower the water surface sufficiently to remove the weir as planned if the 15,000 second-feet was passed through the spillway with the gates fully open. During weir removal, however, the available irrigation water would be reduced. On the other hand, increasing the breach width would not increase the available irrigation water. The next tests were therefore made to determine the elevation to which the remainder of the weir should be removed.

Lowering the weir. Assuming the 50-foot breach to be sufficient, the water surface elevations upstream and downstream of the weir for a discharge of 75,000 second-feet were determined with the remainder of the weir lowered to elevation 282, elevation 278, and elevation 272. The water surface elevations upstream of the weir, on the upstream face of the dam and near the canal intake, for the three weir elevations are plotted in Figure 45B. The curves in Figure 45B show that as the weir is lowered, the water surface elevations both upstream and downstream of the weir are reduced, but little is gained by lowering the weir below elevation 278. With the weir at elevation 278 the water surface upstream of the weir was reduced to approximately elevation 293.2 and to about 291.3 on the dam, which is higher than desirable. However, since little additional reduction in water surface elevation can be obtained, removing the weir below elevation 278 becomes an uneconomical operation.

As a check on the data above, further tests were conducted with most of the staff gages replaced with water surface point gages and additional point gages installed at new locations as shown in Figure 17. The river channel upstream of the weir was filled to elevation 278, since it is believed that sediment has probably deposited behind the rock weir to make the channel at or near the elevation of the excavated weir crest. For previous test runs the channel bed was at about elevation 272. The check tests showed the water surface at gage "G," upstream of the weir, at elevation 293 and at gage "F," on the dam, at 290.9. Both elevations were still higher than the proposed maximum water surface elevation 290.

Lowering the tail water. To determine how much of the backwater effect was caused by tail water, the tail water downstream from the diversion dam at gage "A" was lowered to the predicted elevation for the year 1962, shown in Figure 14. The water surface at gage "G" upstream from weir was then at elevation 292.0 and at gage "F" 289.7. The latter figure is satisfactory, but at gage "G" the surface was still 2 feet higher than desirable. For 1972, tail water tests showed that gage "G" would be at elevation 291.4 and gage "F" at 288.8.

Further tests showed that if the weir crest and channel bed upstream from weir were lowered to elevation 272, the water surface at gage "G" would be at elevation 292.5 for 75,000 second-feet with present tail water. For 1962 tail water, gage "G" would be at elevation 291 and for 1972 at 290.

An appraisal of the data and conclusions, up to this time, indicated a general belief among all persons concerned in the tests that the year 1972 was too far in the future to depend on retrogression to produce full design capacity at the design head. It was felt that a severe flood might develop prior to 1972 which might endanger the dikes. On the other hand, it was felt that to lower the weir to elevation 272 would be a very costly operation. General feeling, however, was that 1962 would not be too long to wait for a satisfactory condition to develop and that elevation 278 was an economical elevation to which to lower the crest of the rock weir. Other methods of reducing the upstream water surface elevations for the design capacity were then investigated.

Suggested modifications. It was suggested that the spillway be widened, but model tests showed that it would be necessary to increase the spillway width by about 33 percent in addition to lowering the weir to elevation 278 and the downstream tail water to the 1962 anticipated elevation. The model spillway discharged only about 67 percent of 75,000 second-feet when the reservoir water surface upstream of the weir was at elevation 290 and the tail water was set for the 1962 anticipated elevation. To widen the spillway 33 percent was not economically possible because foundation rock was lacking, therefore, this idea was abandoned.

Further modifications to the rock weir were suggested and tested using the 1962 anticipated water surface elevation in the channel downstream from the dam at gage "A". With 450 feet of the 550-foot-long weir removed to elevation 278 and 100 feet of it removed to elevation 272, the water surface upstream of the dam was at elevation 291.5 for 75,000 second-feet and at elevation 289.4 on the upstream face of the dam. By widening the channel to 800 feet and excavating to elevation 278 except for 100 feet at elevation 272, the water surface upstream of the weir was reduced to elevation 291.0. At the same time, the water surface increased at the upstream face of the dam to elevation 289.9. Further experiments showed, however, that if the channel widening is accomplished entirely on the right-hand side, the water surface elevation at the dam is affected very little, if any.

Recommendations. It was concluded from these tests that it was impossible to lower the reservoir surface to elevation 290 for a discharge of 75,000 second-feet by any practical and economical means. The designers and field representatives of the project therefore decided to lower the weir a reasonable and economical amount. It was decided that the river channel cross section (or an equivalent area) that existed in 1954, prior to the construction of rock weir, should be re-established at the present weir site.

An equivalent cross section, shown in Figure 13, was agreed upon. About 250 feet of weir and part of the left bank were to be removed to elevation 278, about 250 feet in the center of the channel and 50 feet near the right bank to elevation 275, and 100 feet near the right bank to elevation 272. The deepest excavations, at elevation 272, near the right bank provided a direct flow line to the spillway which probably offers some advantages. This arrangement, according to tests previously described, will maintain water surface elevations along the upstream face of dam at a slightly lower level than if the deep excavation was on the left. Another advantage is that the remaining material near the left bank may wash out of its own accord during flood times since the model indicated that the left end of the rock weir washed out more easily than the right end.

Spillway Capacity

Both models were used to determine the ultimate capacity of the spillway and calibrate the spillway for the full range of reservoir elevations, gate openings, and tail water elevations. The 1:50 scale overall model was particularly useful in determining suitable locations for measuring the reservoir and tail water elevations in the prototype.

Since the sectional model was constructed and tested first, discharge calibration data obtained from it were used to give approximate results and curves that could be refined using the more reliable data taken on the overall model. It was known that the sectional model could not give reservoir and tail water elevations which would occur in areas where they could be measured in the prototype. The sectional model gives correct values, however, only if it can be assumed that all water approaching and leaving the spillway is moving at the same velocity as the water approaching and leaving the center bay of the spillway.

Sectional Model

Uncontrolled flow. Data from the sectional model were used to obtain the preliminary uncontrolled discharge curve, Curve (A), in Figure 46 since the overall model was not yet constructed. Present expected tail water elevations, shown in Figure 14, were used with the discharge and approach channel beds at elevation 259. For the design flow of 75,000 second-feet (25,000 per bay), the reservoir water surface was at elevation 289.96. However, the velocity head at the reservoir gage was computed to be 3.0 feet which, when added to 289.96, gave the static reservoir elevation as 292.96 feet. This preliminary data on the sectional model indicated that the capacity of the spillway might not be sufficient to discharge the design flow at the design reservoir elevation 290 if the reservoir elevation was to be measured outside the limits of the approach channel.

The preliminary approach bed at elevation 259 was lowered to elevation 256 to increase the efficiency of the spillway and for other design reasons. The lower approach bed was installed in the sectional model and the spillway recalibrated for uncontrolled flow. The reservoir water surface was found to be at elevation 290.37 for the design flow of 75,000 second-feet. The velocity head was 2.51 feet which gave the static reservoir as elevation 292.88 feet. Therefore, the efficiency of the submerged crest was increased by very little by lowering the elevation of the approach bed; the headwater elevation necessary to pass 75,000 second-feet was reduced only 0.08 foot.

Gate controlled flow. In the prototype, the reservoir elevation at the canal headworks is to be maintained at elevation 283.5 whenever possible; therefore, all gate controlled flow will occur with headwater elevation 283.5. The gate controlled discharge curves shown in Figure 47 were obtained from the sectional model. The anticipated tail water curves of Figure 14 for the present, year 1962, and year 1972 are also shown in Figure 47 so that discharges for future years as well as the present may be estimated. With fixed reservoir elevation 283.5, the discharge should depend on the gate opening and the tail water elevation. However, the elevation of the discharge channel in the sectional model was also found to affect the quantity of flow for any given tail water elevation. With the discharge channel bed at sill elevation 251, the spillway discharged more water for the same reservoir and tail water elevations than when the bed sloped up to elevation 259. The higher bed submerged the flow over the spillway to a greater degree in the stilling basin, and, therefore, reduced the discharge. The elevation of water surface at the tail water gage was not affected by the higher bed, however, because the water surface sloped downstream from the basin to the tail water gage located approximately 300 feet downstream from the end of the basin. When the channel bed was molded level at sill elevation 251, the water surface from the stilling basin to the gage was more nearly level.

The curves in Figure 47 were also used to determine the safe gate opening increment for operating the prototype gates. If the prototype gates are opened rapidly or if the increment is too large, the increased discharge may be too great for the existing tail water elevation, resulting in sweep out and the possibility of serious erosion downstream from the apron. In future years when the tail water is expected to be lower than at present this type of operation is more likely to occur.

The curves of Figure 47 show discharges for 5-foot increments of gate opening. After the three gates have been opened 5 feet, the tail water should be allowed to stabilize before increasing the gate opening. This might take a matter of minutes or hours, depending on downstream conditions. With 1972 tail water, a 5-foot increment from a 5-foot opening to a 10-foot opening produces a discharge of about 18,000 second-feet with tail water elevation 270.2. Opening another 5-foot increment before the tail water had risen would result in near sweep out conditions. The tail water should be allowed to rise to about elevation 274.6 before a second 5-foot increment is attempted. As the tail water rises, the discharge will be reduced according to the slope of the gate opening curves which also adds to the margin of safety in opening the next increment. It is therefore recommended that gate opening increments do not exceed 5 feet and that successive 5-foot increments be made only after the tail water elevation has reached a safe elevation. The dotted horizontal lines on Figure 47 show the maximum discharge and lowest tail water that will result from this procedure for the 1972 tail water conditions. For present and 1962 tail water the increment would not be critical.

The Overall Model

Gate controlled and uncontrolled flow. Before discharge calibrations were started, the locations for measuring headwater and tail water elevation in the model were selected, keeping in mind that these locations had to be duplicated in the prototype. The water surface along the right bank upstream from the headworks was fairly quiet for all discharges, therefore, location "E" in Figure 17 was selected for the headwater or reservoir gage. In the downstream area, the water surface to the right of the downstream right-hand corner of the stilling basin was stable for all discharges, therefore, a gage was installed at location "C" in the model to measure tail water elevations. Both of these gages could be constructed into the walls of the prototype structure and could easily be piped to a location convenient for the gate operator.

Using an erodible sand bed downstream from the riprap in the discharge channel that eroded as shown in Figure 42A, the spillway was calibrated for gate controlled flow with reservoir at elevation 283.5 and for uncontrolled flow with higher reservoir elevations. These discharge curves are shown in Figure 48 and may be used for prototype operation of the spillway. All three gates are to be opened the same amount.

To determine the reliability of the discharge curves in Figure 48 for varying conditions, the erodible sand bed in the discharge channel was replaced to about elevation 270, as shown in Figure 2, and covered with a layer of 1/4-inch pea-gravel to prevent erosion of the discharge channel bed. Discharge calibration tests were then made to determine whether the curves in Figure 48, which were obtained with an erodible sand bed downstream from the riprap that did erode as shown in Figure 42, were still reliable. No measurable differences in the discharge curves could be detected. It was therefore concluded that the curves of Figure 48 were reliable for any anticipated condition of the discharge channel bed.

The 20-foot gate opening curve, in Figure 48, had a shape somewhat different from the other gate opening curves. This appeared to be due to a pileup in the water surface at the headwater gage that occurred only for this gate opening. As the tail water elevation was lowered, the pileup became more pronounced. Since the same condition is expected to occur in the prototype, the 20-foot gate opening curve is reliable for determining the discharge in the prototype; but the true reservoir water surface near the intake will not be quite as high as elevation 283.5.

The gate controlled discharge curves from the overall model, Figure 48, agreed quite well with those from the sectional model, Figure 47. The 5-foot gate opening curves from the two models are nearly identical while the larger gate openings show less discharge in the overall model. The reason for this is that the tail water gages were not in the same location in both models and this becomes an important factor as the discharge is increased. The tail water gage at point "C," in Figure 17, measured a lower water surface than the tail water gage in the sectional model because in the overall model the water surface in the center of the channel was slightly higher than at the banks.

The uncontrolled discharge data from the 1:50 scale overall model also shows close agreement with the sectional model data. The uncontrolled discharge Curve (A) in Figure 46 was obtained from the sectional model. The uncontrolled discharge Curve (C) in Figure 46 was obtained from the overall model, using tail water and bed elevations in the overall model similar to those used in the sectional model. The two curves check very closely. In the overall model the reservoir elevation for 75,000 second-feet is about 1 foot higher than in the sectional model; however, the velocity of approach at reservoir gage "E" located near the headworks in the overall model is not as great as at the reservoir gage in the 2-foot flume of the sectional model. Making allowances for the different velocity heads, the static reservoir elevations are about the same.

<u>Discharge channel bed elevation affects spillway capacity.</u> Although the gages at "C" and "E" are used to determine the discharge through the spillway, gage "C" does not reflect the tail water elevation as it occurs in the river channel since the discharge channel acts as a

constriction to make gage "C" read higher than gage "A." Figure 49 includes five pairs of curves showing elevations for gages "A" and "C" with the gage at "E" held at reservoir elevations 283.5, 285, 287.5, 290, and 292.5 (left-hand ordinate). Intermediate headwater elevation may be interpolated. In all cases the curves show the elevation at gage "C" to be higher than at gage "A" when the discharge channel bed is at elevation 270. When the discharge channel bed erodes, however, gages "A" and "C" read practically the same and the spillway will pass more discharge at the same headwater. Thus, Figure 49 may be used to estimate the probable capacity of the spillway for a range of tail water elevations and discharge channel bed elevations. With a given reservoir elevation at gage "E" (left-hand ordinate in Figure 49) intersect the gage "A" curve with the tail water curve for any particular year. This gives the spillway capacity with discharge channel bed at elevation 270. When the discharge channel bed has eroded to elevation 260, gage "A" will read approximately the same as gage "C" because of the increased discharge; therefore, intersect curve "C" with tail water curve for the particular year to obtain increased spillway capacity. For example, for reservoir elevation 283.5 at gage "E" near the canal headworks, with the discharge channel bed at elevation 270, the maximum uncontrolled discharge through the spillway is 29,000 second-feet for present tail water elevation in the river channel as shown at Intersection (1) in Figure 49. However, if sufficient erosion in the discharge channel occurs, the water surface elevation at gage "C" in the discharge channel will become practically the same as in the river channel at gage "A," and Intersection (2) in Figure 49 shows that the uncontrolled discharge for present tail water elevation will be increased to 36,000 second-feet. For tail water elevation in year 1962 the difference would be between 34,000 and 45,000 second-feet as shown at Intersections (3) and (4) in Figure 49.

The same comparison for reservoir elevation 290 at gage "E" is shown at Intersections (5), (6), (7), and (8) in Figure 49. For present tail water conditions the spillway discharges 59,000 second-feet, Intersection (5), and in 1962 it will discharge 64,000 second-feet, Intersection (6); if the tail water elevation in the discharge channel is reduced by erosion to elevation 260, the spillway will discharge about 70,000 second-feet, Intersection (7), with present tail water and 78,000 second-feet, Intersection (8), in 1962. Curves (B), (C), (D), and (E) in Figure 46 can be used to verify the comparison made above in Figure 49.

It may therefore be concluded that the maximum capacity of the spillway with present expected tail water elevation will be 59,000 second-feet. An additional 1,800 second-feet may be discharged through the canal headworks, making the total capacity of the structures about 61,000 second-feet. Additional capacity will be available only after erosion of the discharge channel and degradation of the river channel have reduced the submergence effect on the spillway.

Reservoir elevations upstream from the headworks. With the maximum discharge conditions described above, the reservoir water surface will be at elevation 290 at the headworks, gage "E," but will be higher in the reservoir upstream, as shown in Figure 49. For example, for 59,000 second-feet, Intersection (5) in Figure 49, the elevation at gage "F" on the upstream face of the dam, see Figure 17, is at elevation 290.6 and upstream of the rock weir at gage "G" the elevation is 291.3. For 70,000 second-feet, Intersection (7), the water surface on the face of the dam is at elevation 290.8 and upstream of the weir at elevation 291.6 feet.

By interpolation between curves, Figure 49 also shows that if the design spillway flow of 75,000 second-feet occurs with present tail water elevation and before erosion of the discharge channel bed occurs, the elevation at the headworks, gage "E," will be about 292.4; at the dam face, gage "F," about 293.6; and upstream of the weir at gage "G," about 294.6. Extensive excavation of the discharge channel bed to below elevation 270 when constructing the prototype would be required to lower these elevations. The designers believed, however, that natural erosion of the bed would accomplish this within a few years.

River Currents

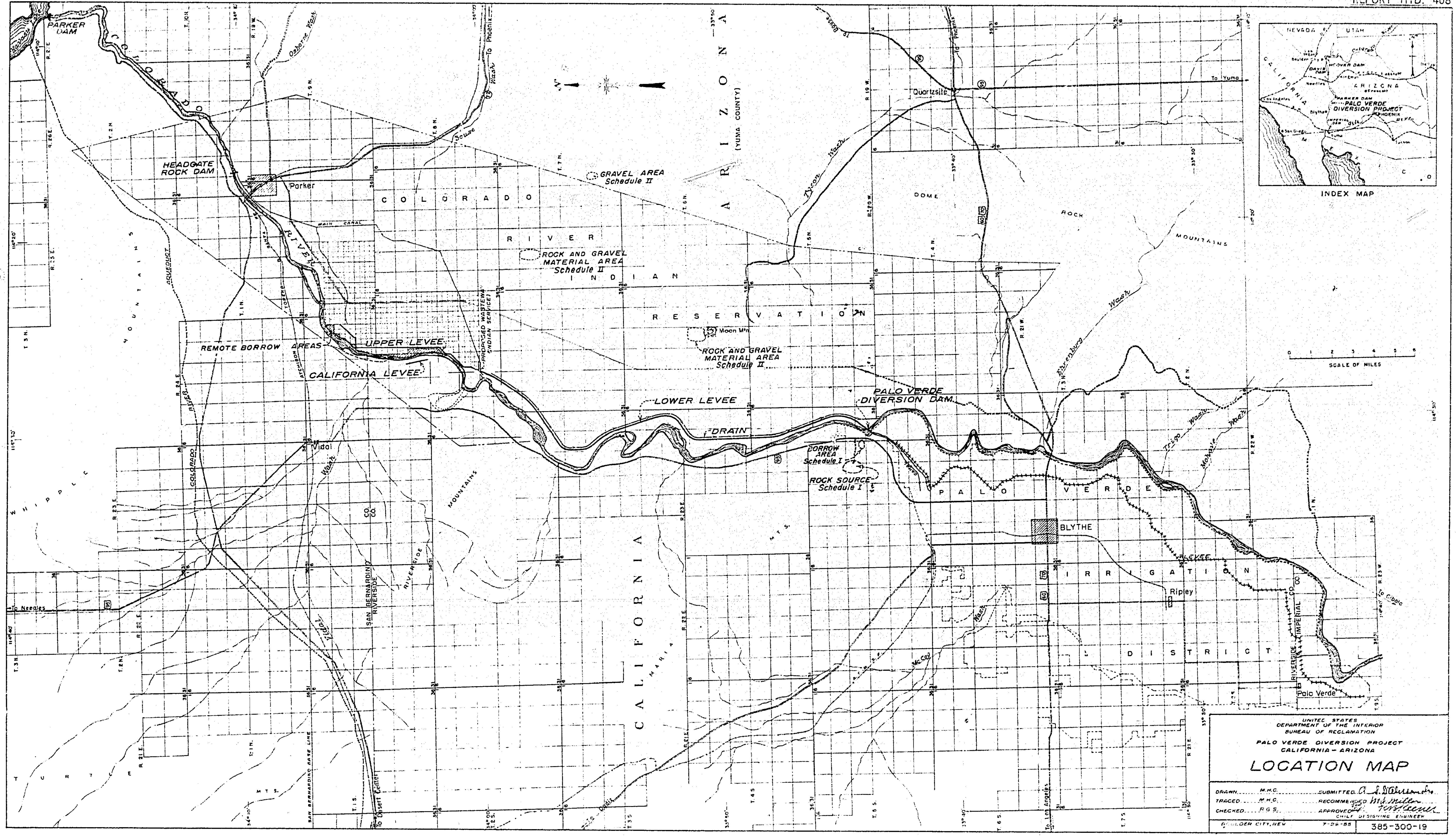
The intensity and direction of flow currents in the downstream river channel were investigated to be sure that Indian reservation lands along the left bank were not endangered by flow currents crossing the river near the discharge channel exit. The direction and relative intensity of flow currents for discharges of 15,000, 34,000, and 75,000 second-feet are shown in Figure 50. Paper confetti and a 1/5-second exposure were used to show the currents. Therefore, the longer streaks indicate faster velocities.

For 15,000 second-feet through the spillway with tail water at the present expected elevation, flow currents from the discharge channel cross the river channel from right to left in a diagonal direction, as shown in Figure 50A. The main current is near the left bank. However, the currents leaving the discharge channel are spread to the full width of the river, and cutting velocities do not occur adjacent to the left bank. The velocity of flow was measured near the left bank, right bank, and center of the channel by use of a Bentzel tube and are shown in Figure 50. Along the left bank the velocity did not exceed 3 prototype feet per second.

For 34,000 second-feet, Figure 50B, with the river tail water at about elevation 282, the flow was not directed into the left bank. Velocities measured near the left bank did not exceed 4 feet per second.

For 75,000 second-feet, Figure 50C, with the river tail water at elevation 285.5, which is the expected elevation in 1962, the main current was near the center of the channel and measured about 7 feet per second. Velocities near the left bank did not exceed 4 feet per second; these occurred 50 to 100 feet from the shoreline. Water along the shoreline was dead water, but when the tail water elevation was increased to 287.5 feet the velocity had an upstream direction.

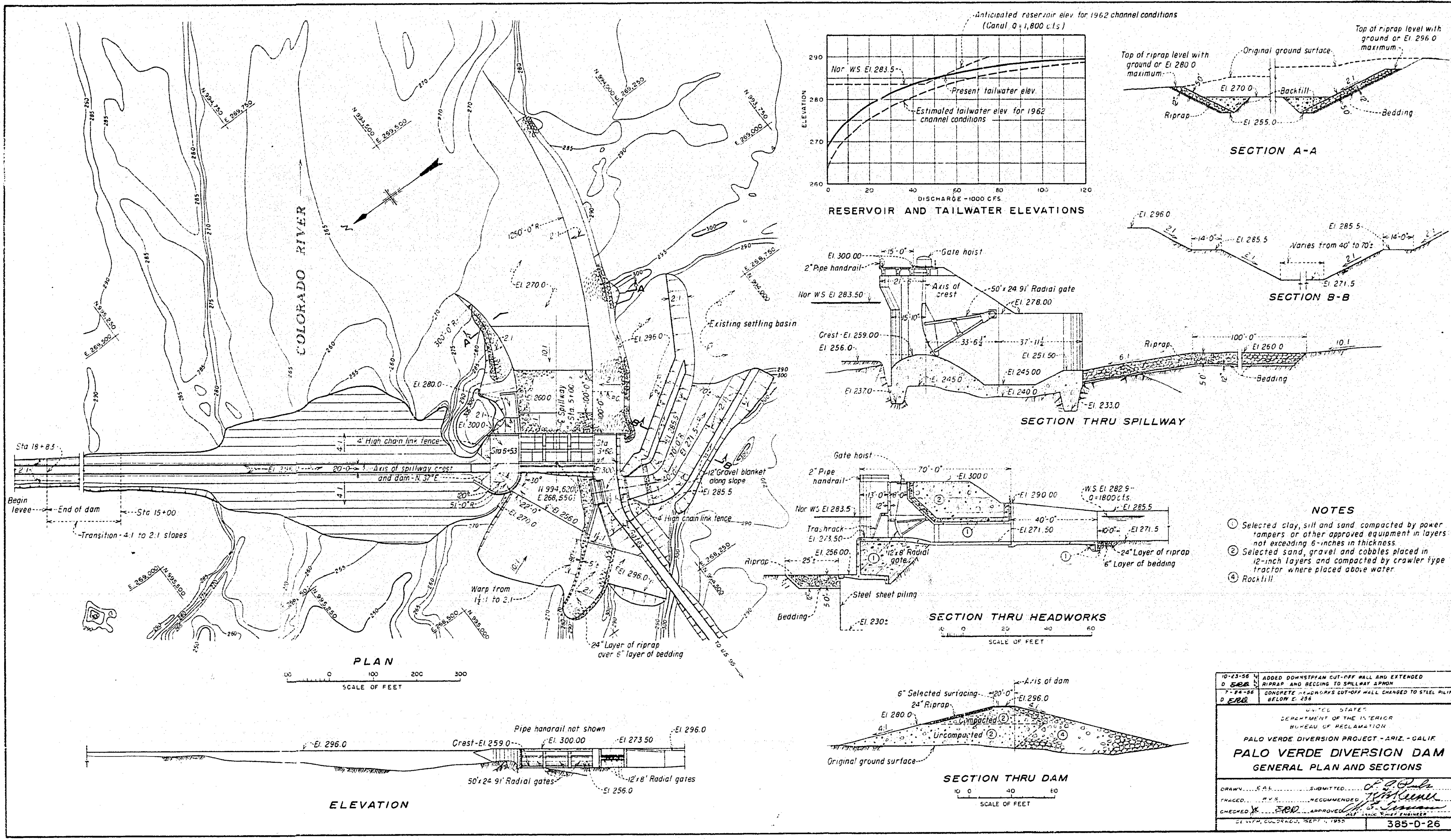
For 34,000 and 75,000 second-feet the ends of the discharge channel banks are submerged and therefore do not turn the flow toward the left bank. Instead, the flow cuts across the right bank as it leaves the discharge channel. The main flow, therefore, is nearer to the right bank than the left as indicated in Figure 50C and by the scour pattern in Figure 50D. In fact, the scour pattern shows a tendency for material to deposit along the left bank rather than to scour material from it. Therefore, it is believed that the left bank is not in danger of extensive erosion.

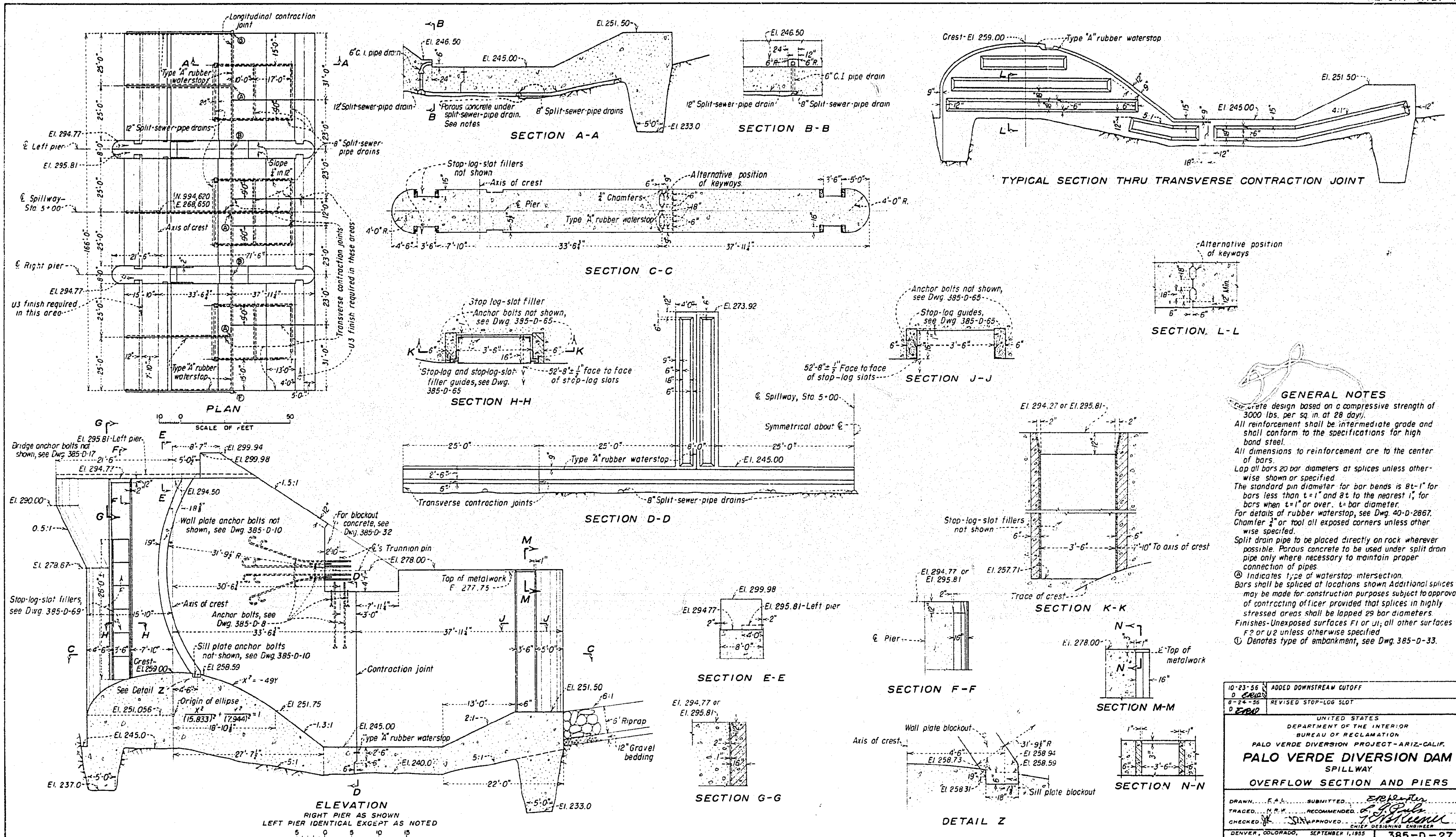


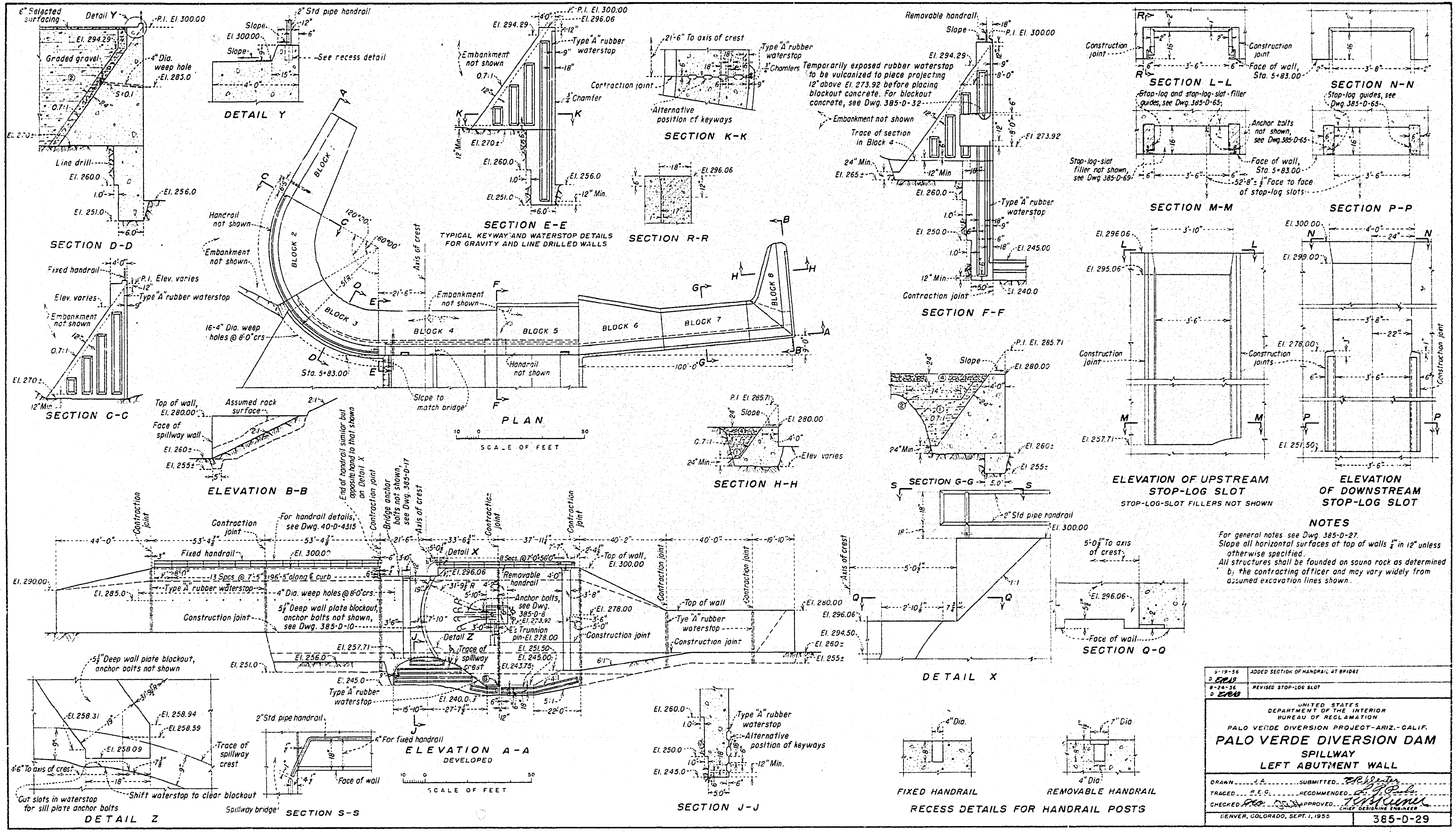
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
PALO VERDE DIVERSION PROJECT
CALIFORNIA - ARIZONA

LOCATION MAP

DRAWN M.H.C. SUBMITTED
TRACED M.H.C. RECOMMENDED
CHECKED R.G.S. APPROVED
CHIEF DESIGNING ENGINEER
BOULDER CITY, NEV. 7-25-55 385-300-19


COLORADO RIVER
PLAN
SCALE OF FEET
ELEVATION
Pipe handrail not shown
50'x24.91' Radial gates
12'x8' Radial gates
RESERVOIR AND TAILWATER ELEVATIONS
Anticipated reservoir elev. for 1962 channel conditions (Canal Q = 1,800 c.f.s.)
Nor. WS. El. 283.5
Present tailwater elev.
Estimated tailwater elev. for 1962 channel conditions
ELEVATION
DISCHARGE - 1000 CFS
SECTION A-A
Top of riprap level with ground or El. 280.0 maximum
Original ground surface
Top of riprap level with ground or El. 296.0 maximum
Backfill
Riprap
Bedding
SECTION B-B
Varies from 40' to 70'±
SECTION THRU SPILLWAY
Gate hoist
2" Pipe handrail
Axis of crest
50'x24.91' Radial gate
Existing settling basin
SECTION THRU HEADWORKS
Trashrack
Steel sheet piling
24" Layer of riprap
6" Layer of bedding
SECTION THRU DAM
6" Selected surfacing
24" Riprap
Axis of dam
Compacted
Uncompacted
Original ground surface
NOTES
(1) Selected clay, silt and sand compacted by power tampers or other approved equipment in layers not exceeding 6-inches in thickness.
(2) Selected sand, gravel and cobbles placed in 12-inch layers and compacted by crawler type tractor where placed above water.
(4) Rockfill
10-23-56 ADDED DOWNSTREAM CUT-OFF WALL AND EXTENDED RIPRAP AND BEDDING TO SPILLWAY APRON
7-24-56 CONCRETE HEADWORKS CUT-OFF WALL CHANGED TO STEEL PILING BELOW El. 256
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
PALO VERDE DIVERSION PROJECT - ARIZ. - CALIF.
PALO VERDE DIVERSION DAM
GENERAL PLAN AND SECTIONS
DENVER, COLORADO, SEPT. 1955
385-D-26


PLAN
SECTION A-A
SECTION B-B
TYPICAL SECTION THRU TRANSVERSE CONTRACTION JOINT
SECTION C-C
SECTION L-L
SECTION H-H
SECTION J-J
SECTION D-D
SECTION K-K
ELEVATION
RIGHT PIER AS SHOWN
LEFT PIER IDENTICAL EXCEPT AS NOTED
SECTION E-E
SECTION F-F
SECTION M-M
SECTION G-G
DETAIL Z
SECTION N-N
GENERAL NOTES
Concrete design based on a compressive strength of 3000 lbs. per sq. in. at 28 days.
All reinforcement shall be intermediate grade and shall conform to the specifications for high bond steel.
All dimensions to reinforcement are to the center of bars.
Lap all bars 20 bar diameters at splices unless otherwise shown or specified.
The standard pin diameter for bar bends is 8t-1" for bars less than t=1" and 8t to the nearest 1" for bars when t=1" or over. t= bar diameter.
For details of rubber waterstop, see Dwg. 40-D-2867.
Chamfer 3/4" or tool all exposed corners unless otherwise specified.
Split drain pipe to be placed directly on rock wherever possible. Porous concrete to be used under split drain pipe only where necessary to maintain proper connection of pipes.
Ⓐ Indicates type of waterstop intersection.
Bars shall be spliced at locations shown. Additional splices may be made for construction purposes subject to approval of contracting officer provided that splices in highly stressed areas shall be lapped 29 bar diameters.
Finishes-Unexposed surfaces F1 or U1; all other surfaces F2 or U2 unless otherwise specified.
① Denotes type of embankment, see Dwg. 385-D-33.
10-23-56 ADDED DOWNSTREAM CUTOFF
8-24-56 REVISED STOP-LOG SLOT
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
PALO VERDE DIVERSION PROJECT-ARIZ.-CALIF.
PALO VERDE DIVERSION DAM
SPILLWAY
OVERFLOW SECTION AND PIERS
DRAWN E.A.L. SUBMITTED
TRACED N.R.P. RECOMMENDED
CHECKED APPROVED
CHIEF DESIGNING ENGINEER
DENVER, COLORADO, SEPTEMBER 1, 1955
385-D-27


SECTION D-D
DETAIL Y
SECTION E-E
TYPICAL KEYWAY AND WATERSTOP DETAILS FOR GRAVITY AND LINE DRILLED WALLS
SECTION K-K
SECTION R-R
SECTION F-F
SECTION L-L
SECTION N-N
SECTION M-M
SECTION P-P
SECTION C-C
ELEVATION B-B
PLAN
SCALE OF FEET
BLOCK 1
BLOCK 2
BLOCK 3
BLOCK 4
BLOCK 5
BLOCK 6
BLOCK 7
BLOCK 8
SECTION H-H
SECTION G-G
ELEVATION OF UPSTREAM STOP-LOG SLOT
STOP-LOG-SLOT FILLERS NOT SHOWN
ELEVATION OF DOWNSTREAM STOP-LOG SLOT
NOTES
For general notes see Dwg. 385-D-27.
Slope all horizontal surfaces at top of walls 1/4" in 12" unless otherwise specified.
All structures shall be founded on sound rock as determined by the contracting officer and may vary widely from assumed excavation lines shown.
ELEVATION A-A
DEVELOPED
DETAIL X
SECTION Q-Q
DETAIL Z
SECTION S-S
SECTION J-J
FIXED HANDRAIL
REMOVABLE HANDRAIL
RECESS DETAILS FOR HANDRAIL POSTS
9-19-56 ADDED SECTION OF HANDRAIL AT BRIDGE
8-24-56 REVISED STOP-LOG SLOT
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
PALO VERDE DIVERSION PROJECT-ARIZ.-CALIF.
PALO VERDE DIVERSION DAM
SPILLWAY
LEFT ABUTMENT WALL
DRAWN
SUBMITTED
TRACED
RECOMMENDED
CHECKED
APPROVED
CHIEF DESIGNING ENGINEER
DENVER, COLORADO, SEPT. 1, 1955
385-D-29

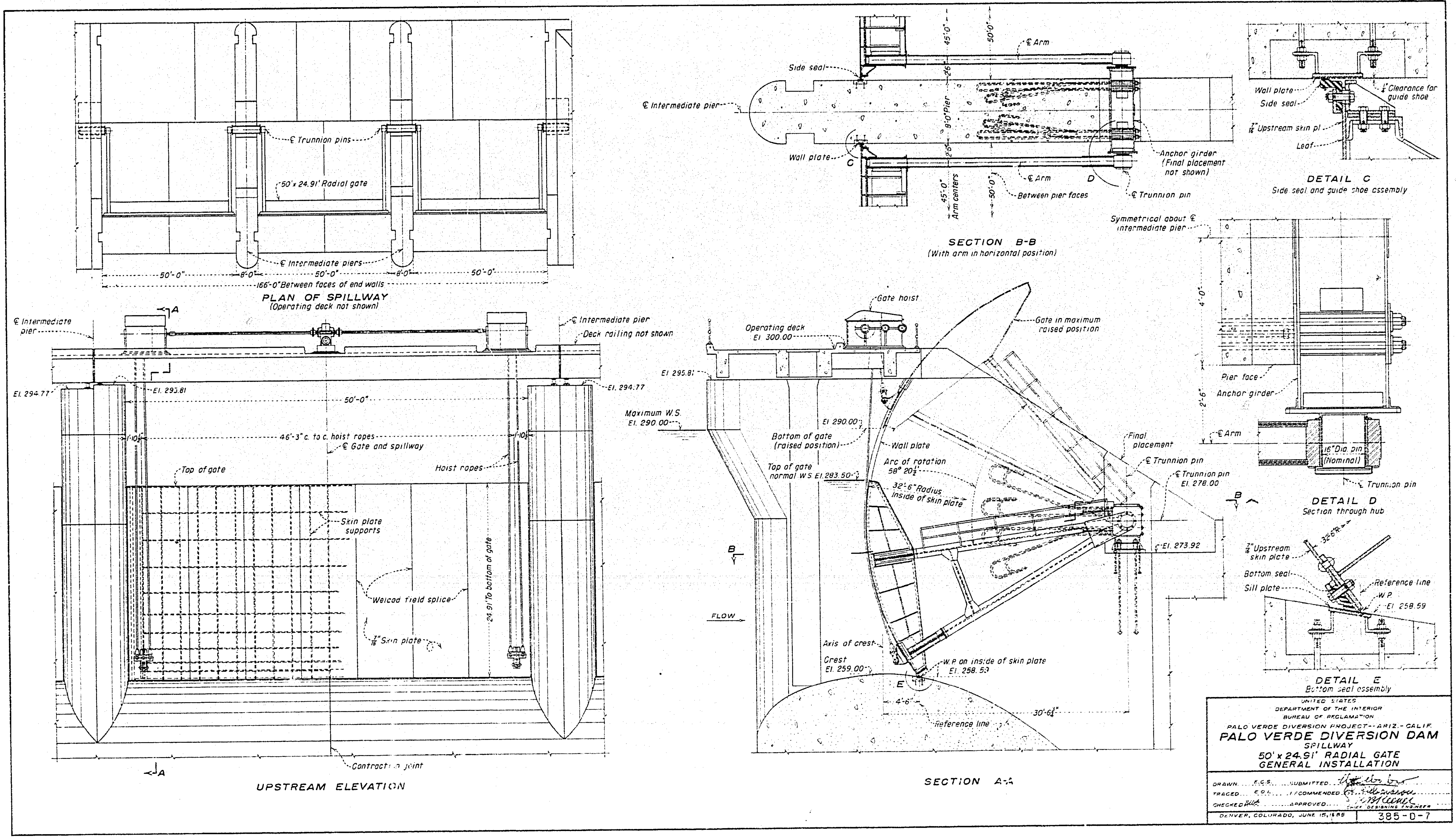
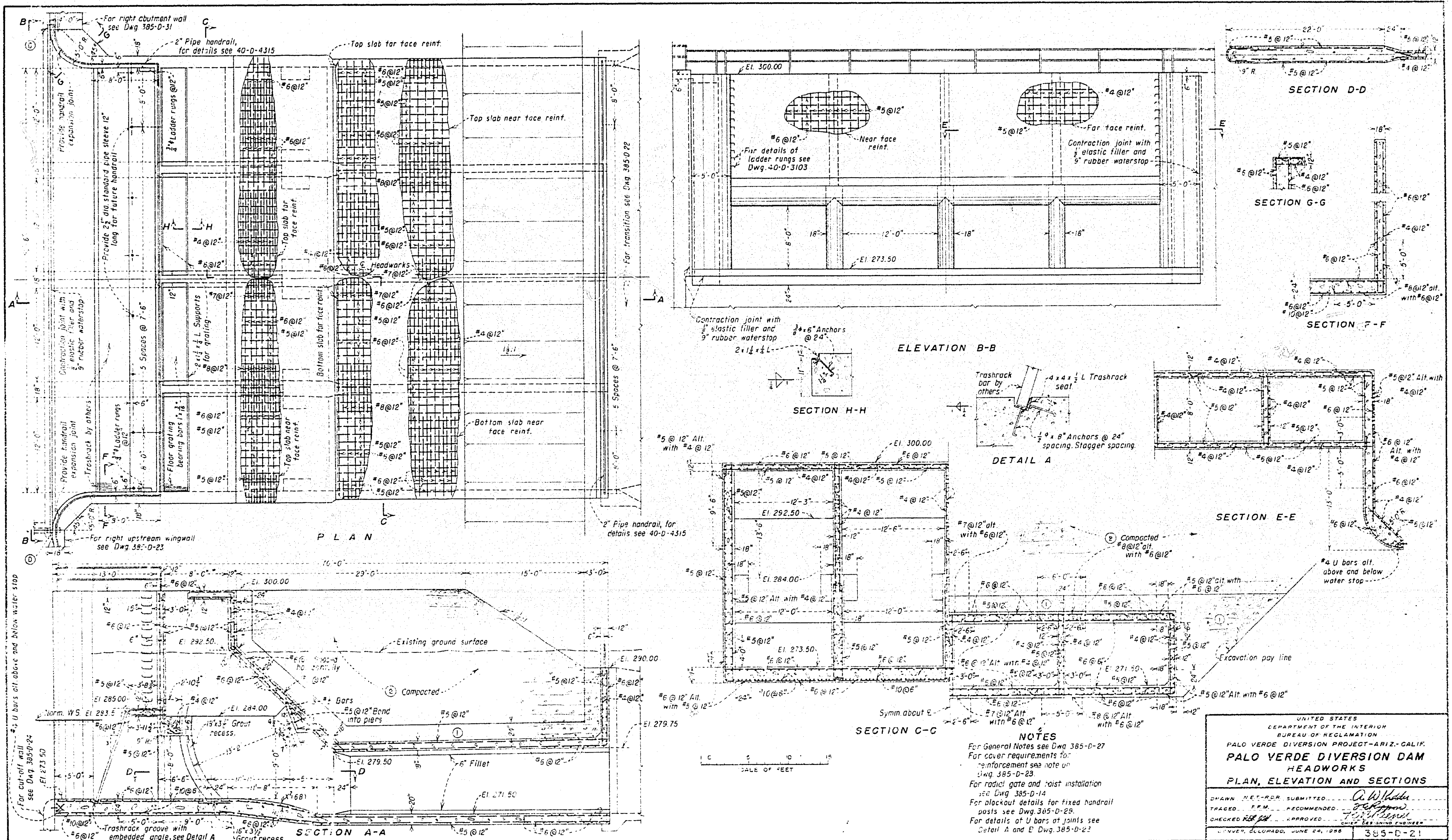
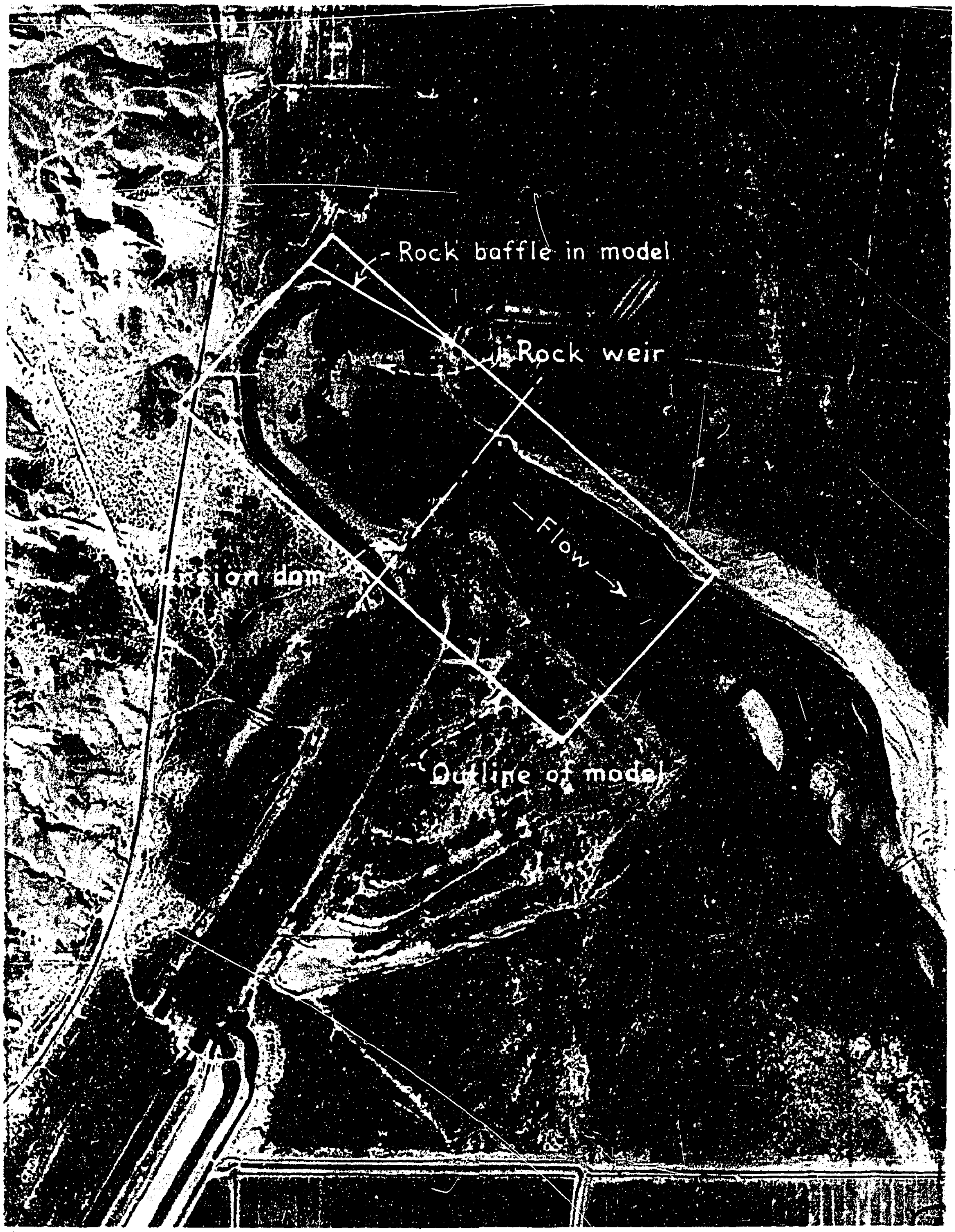

PLAN OF SPILLWAY
(Operating deck not shown)
℄ Trunnion pins
50'x 24.91' Radial gate
℄ Intermediate piers
50'-0"
8'-0"
50'-0"
8'-0"
50'-0"
166'-0" Between faces of end walls
SECTION B-B
(With arm in horizontal position)
Side seal
℄ Intermediate pier
Wall plate
℄ Arm
Anchor girder (Final placement not shown)
℄ Trunnion pin
Between pier faces
45'-0" Arm centers
50'-0"
8'-0" Pier
DETAIL C
Side seal and guide shoe assembly
Wall plate
Side seal
7/16" Upstream skin pl
Leaf
1/2" Clearance for guide shoe
DETAIL D
Section through hub
Symmetrical about ℄ intermediate pier
Pier face
Anchor girder
℄ Arm
16" Dia. pin (Nominal)
℄ Trunnion pin
UPSTREAM ELEVATION
℄ Intermediate pier
℄ Intermediate pier
Deck railing not shown
El. 294.77
El. 293.81
El. 294.77
50'-0"
46'-3" c. to c. hoist ropes
℄ Gate and spillway
Top of gate
Hoist ropes
Skin plate supports
Welded field splice
7/16" Skin plate
24.91' To bottom of gate
Contraction joint
SECTION A-A
Gate hoist
Operating deck El. 300.00
El. 295.81
Maximum W.S. El. 290.00
Gate in maximum raised position
El. 290.00
Bottom of gate (raised position)
Wall plate
Top of gate normal W.S. El. 283.50
Arc of rotation 58° 20 1/2'
32'-6" Radius inside of skin plate
Final placement
℄ Trunnion pin
℄ Trunnion pin El. 278.00
El. 273.92
FLOW
Axis of crest
Crest El. 259.00
W.P. on inside of skin plate El. 258.59
4'-6"
30'-6 1/4"
Reference line
DETAIL E
Bottom seal assembly
7/16" Upstream skin plate
Bottom seal
Sill plate
Reference line
W.P.
El. 258.59
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
PALO VERDE DIVERSION PROJECT--ARIZ.-CALIF.
PALO VERDE DIVERSION DAM
SPILLWAY
50' x 24.91' RADIAL GATE
GENERAL INSTALLATION
DRAWN
TRACED
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
CHIEF DESIGNING ENGINEER
DENVER, COLORADO, JUNE 15, 1955
385-D-7

FIGURE 6
REPORT HYD.





PALO VERDE DIVERSION DAM
OUTLINE OF PROTOTYPE AREA MODELED

PALO VERDE DIVERSION DAM
AERIAL VIEW OF TEMPORARY ROCK WEIR

FIGURE 8
REPORT HYD 408

PALO VERDE DIVERSION DAM
TEMPORARY ROCK WEIR DISCHARGING 16,000 SECOND FEET

FIGURE 9
REPORT HYD 408

PALO VERDE DIVERSION DAM
FLOOD DAMAGE TO TEMPORARY ROCK WEIR LOOKING UPSTREAM

FIGURE 10
REPORT HYD 408

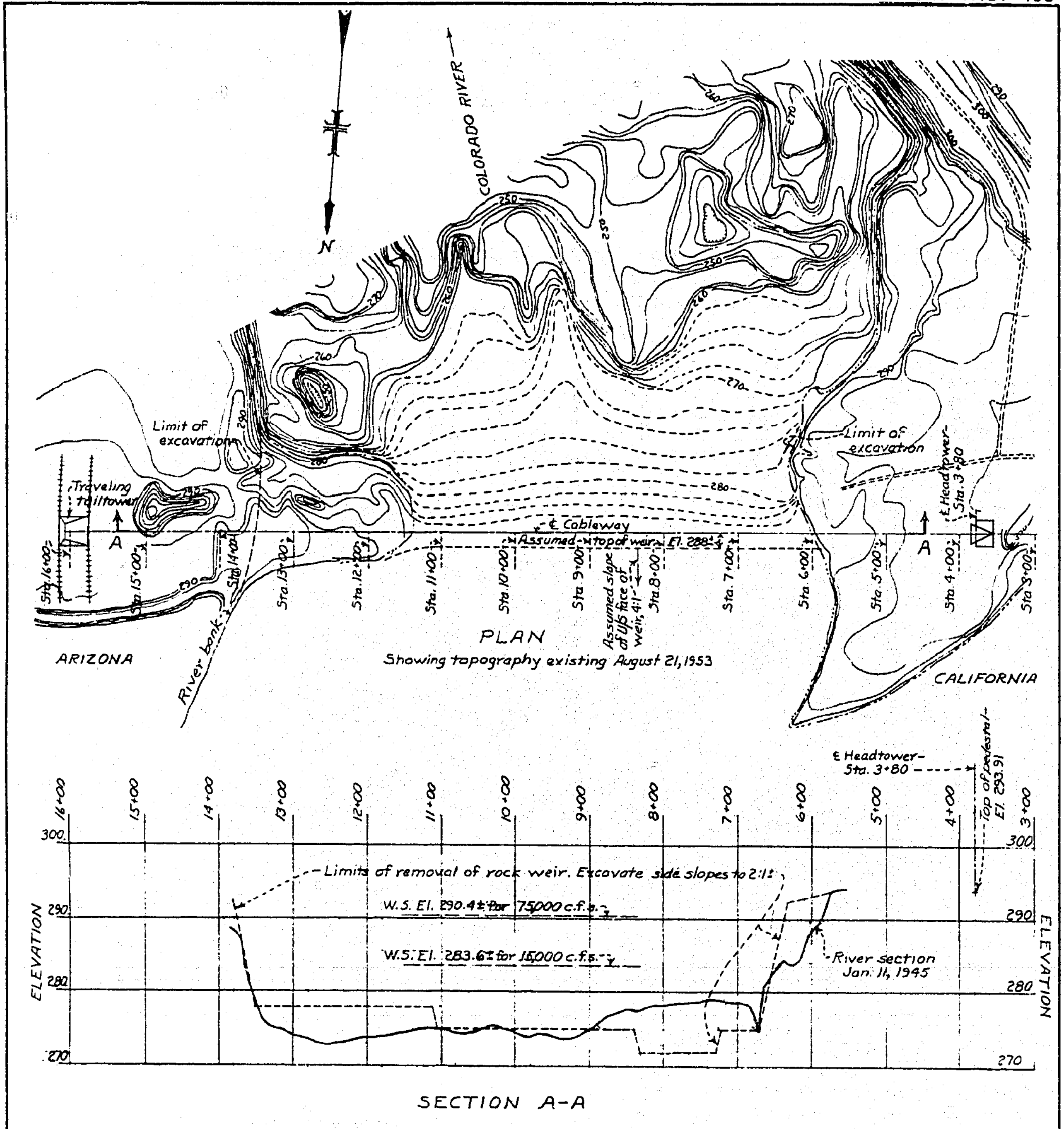
PALO VERDE DIVERSION DAM
FLOOD DAMAGE TO TEMPORARY ROCK WEIR - LOOKING TOWARD LEFT BANK

FIGURE 11
REPORT HYD 408

PALO VERDE DIVERSION DAM
FLOOD DAMAGE TO TEMPORARY ROCK WEIR - LOOKING TOWARD LEFT BANK

FIGURE 12
REPORT HYD 408

FIGURE 13
REPORT HYD. 408



NOTES

Topography taken from Dwg. 423-306-727, dated Aug. 21, 1953.
Dotted contours on downstream side of weir are estimated.
No data available on upstream face of weir, however, slope of this face assumed to be 4:1.
River section on ℄ of cableway plotted from survey of Jan. 11, 1945, prior to construction of weir.

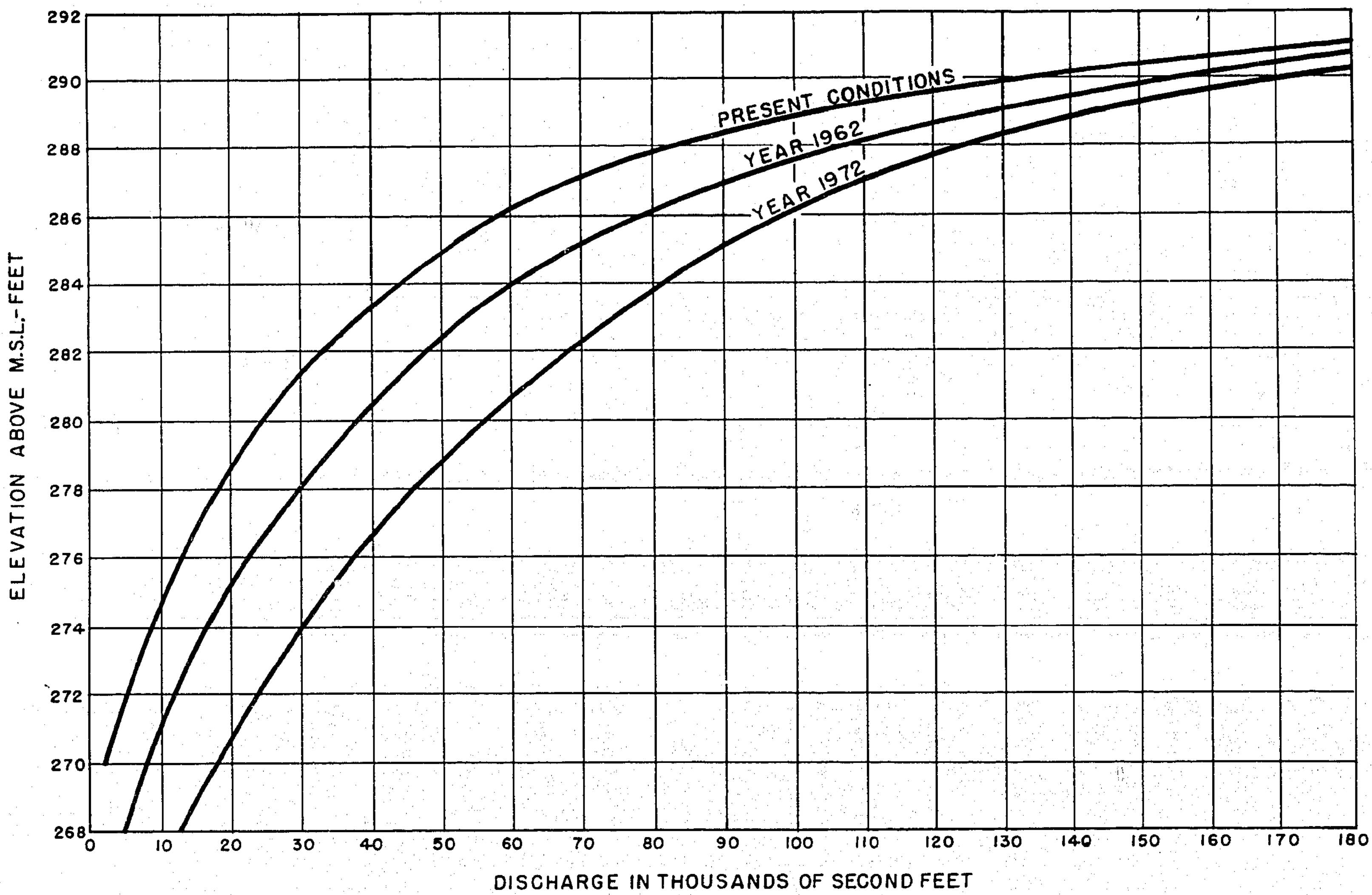
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
PALO VERDE DIVERSION PROJECT — ARIZ. - CALIF.

PALO VERDE DIVERSION DAM

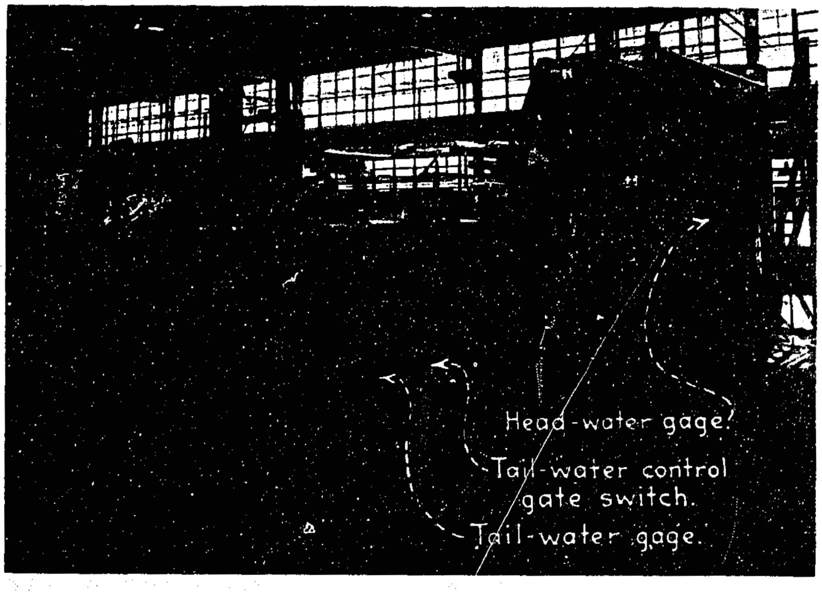
REMOVAL OF TEMPORARY ROCK WEIR

DRAWN LB. SUBMITTED L. G. Puls
TRACED RECOMMENDED A. J. Rice
CHECKED RECA APPROVED ~~CHIEF DESIGNING ENGINEER~~
DENVER, COLORADO, SEPT. 1, 1955 | 385-D-56

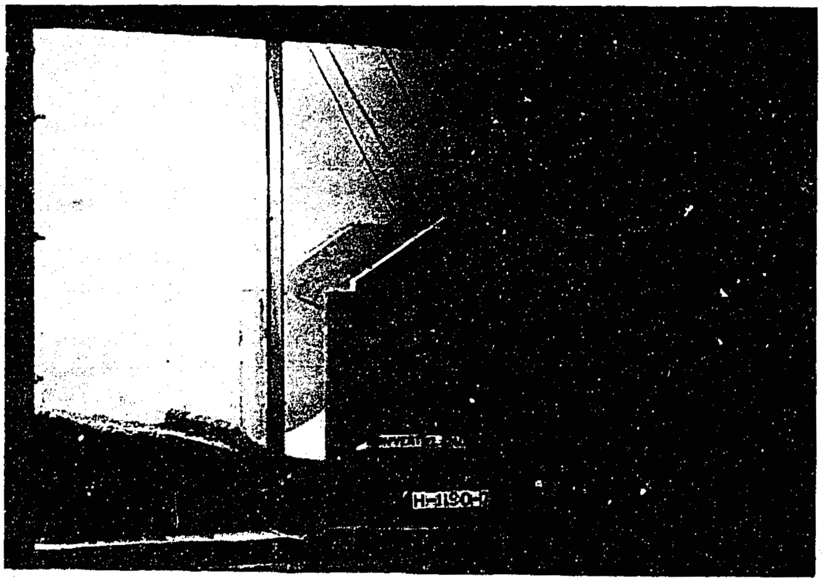
FIGURE 14
REPORT HYD. 408



PALO VERDE DIVERSION DAM
TAILWATER CURVES

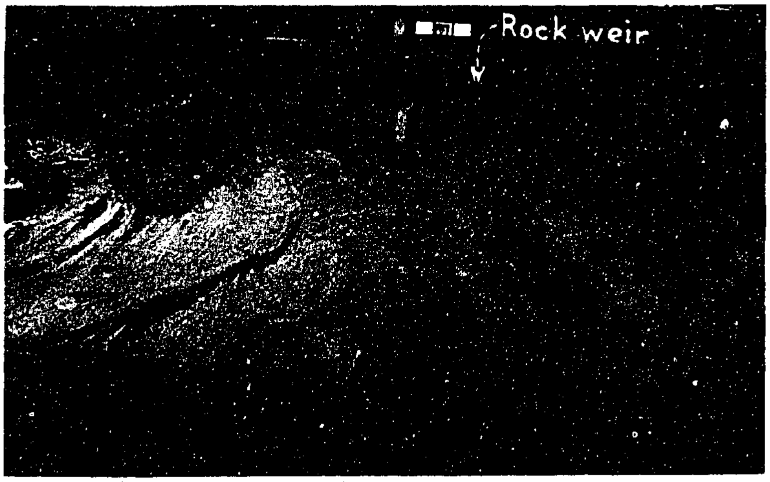


Test flume and equipment used to
test sectional model



Single bay sectional model with
preliminary bucket type energy
dissipator in place

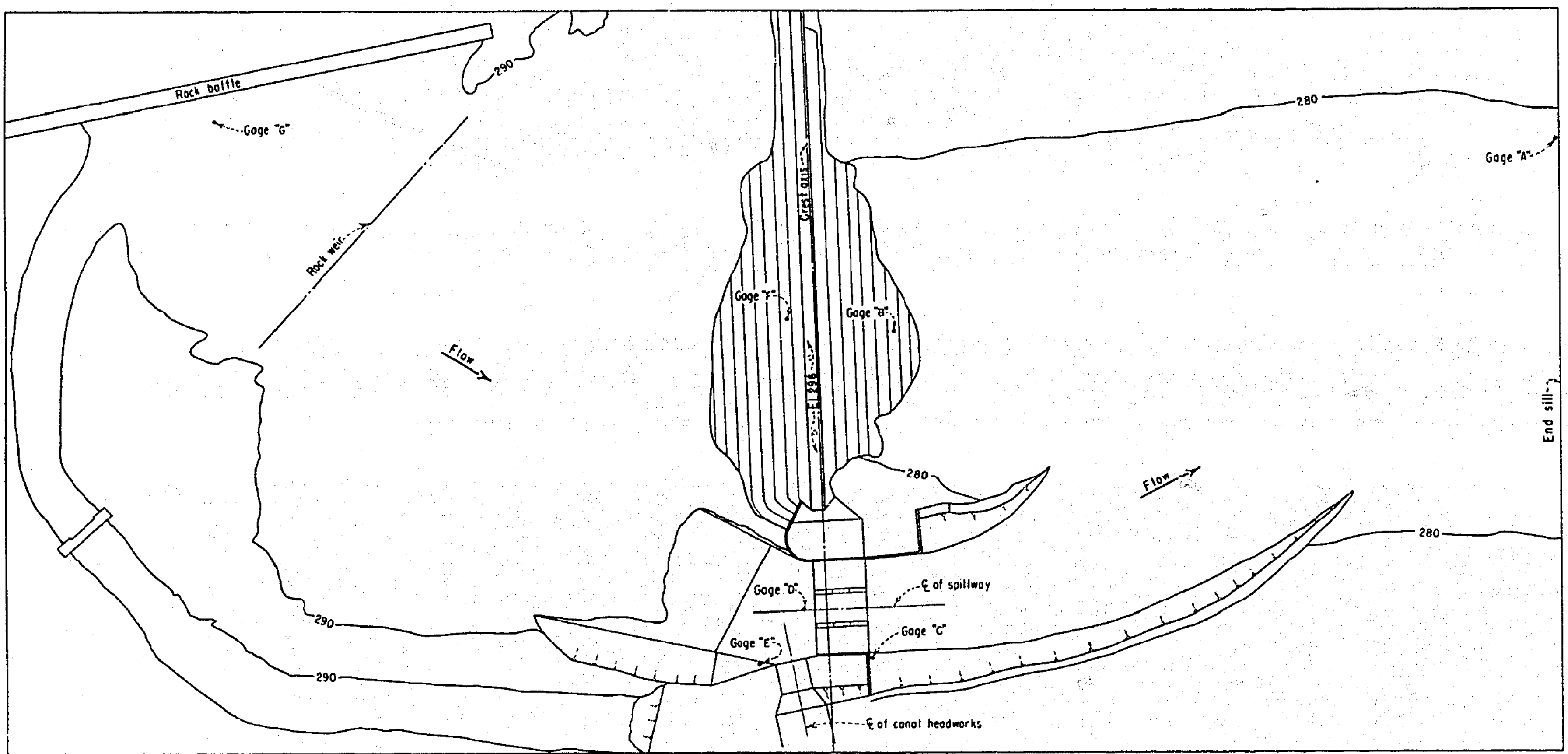
PALO VERDE DIVERSION DAM
SECTIONAL SPILLWAY MODEL
1:28.3 SCALE MODEL



A. Looking upstream into model. Spillway discharge channel is at left.

B. Recommended arrangement of Spillway and Canal Headworks structures.

PALO VERDE DIVERSION DAM
OVERALL MODEL
1:50 SCALE MODEL



PLAN

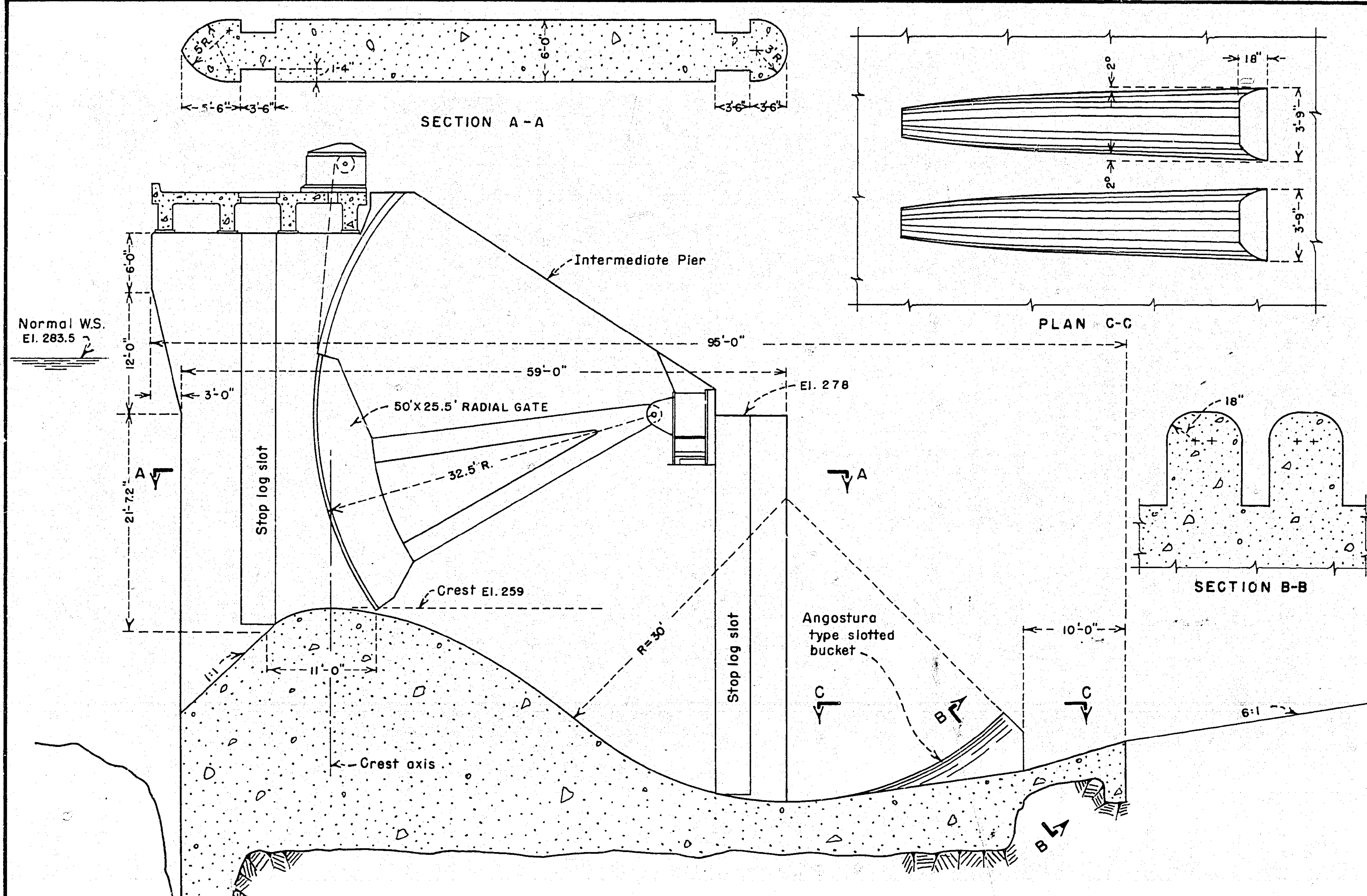
NOTE: Gages "A" through "G" are water surface point gages.

PALO VERDE DIVERSION DAM

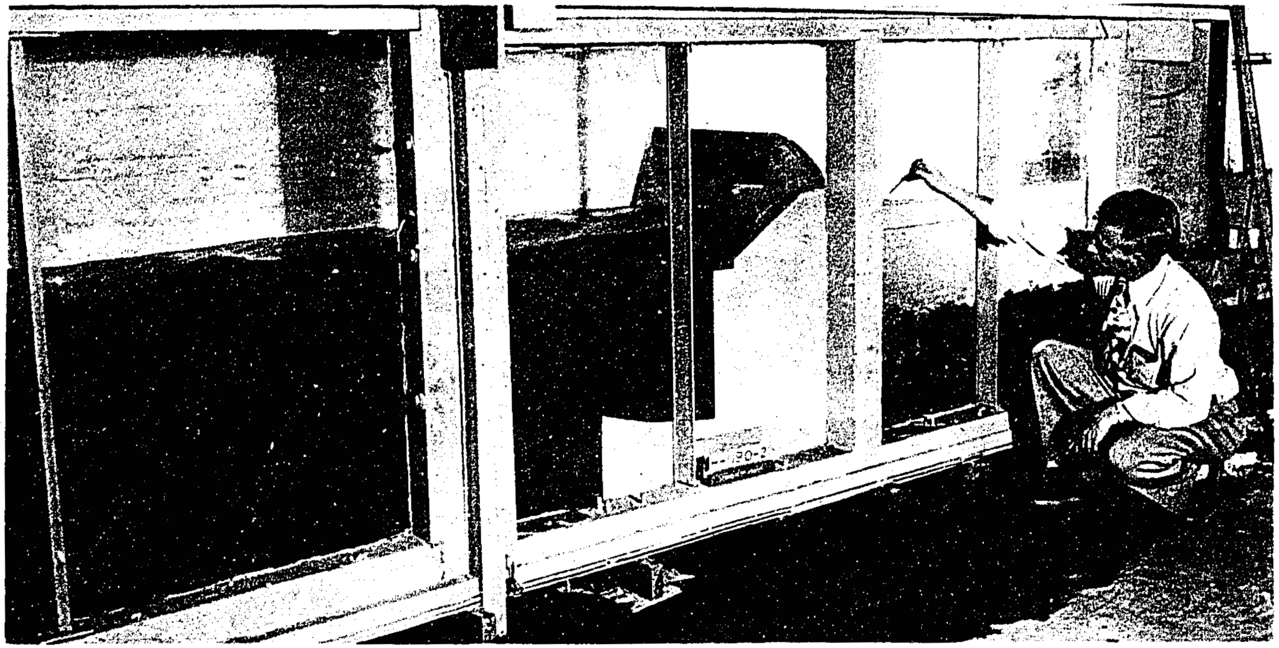
OVERALL MODEL LAYOUT

1:50 SCALE MODEL

FIGURE 17
REPORT HYD. 408



PALO VERDE DIVERSION DAM
PRELIMINARY 30 FOOT RADIUS SPILLWAY BUCKET

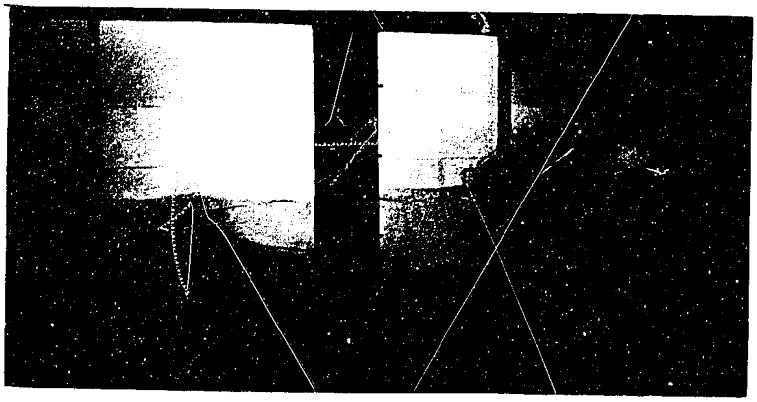


One bay discharging 25, 000 second feet with
reservoir at elevation 290

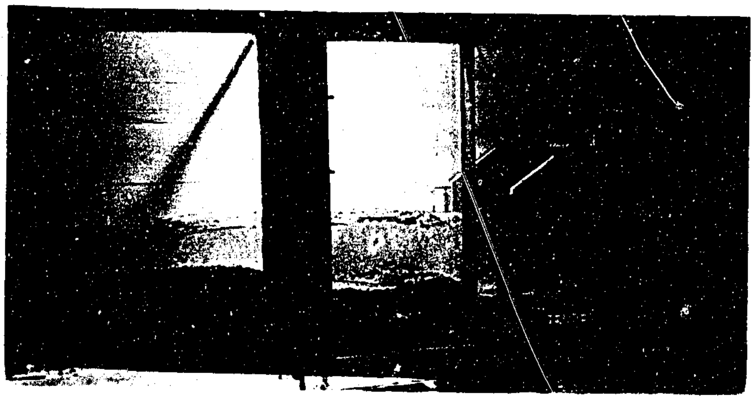
PALO VERDE DIVERSION DAM
PRELIMINARY SPILLWAY BUCKET DISCHARGING
1:28.3 SCALE MODEL

FIGURE 19
REPORT HYD 408

A. Preliminary Bucket - (slotted-baffle type - see Figure 18).



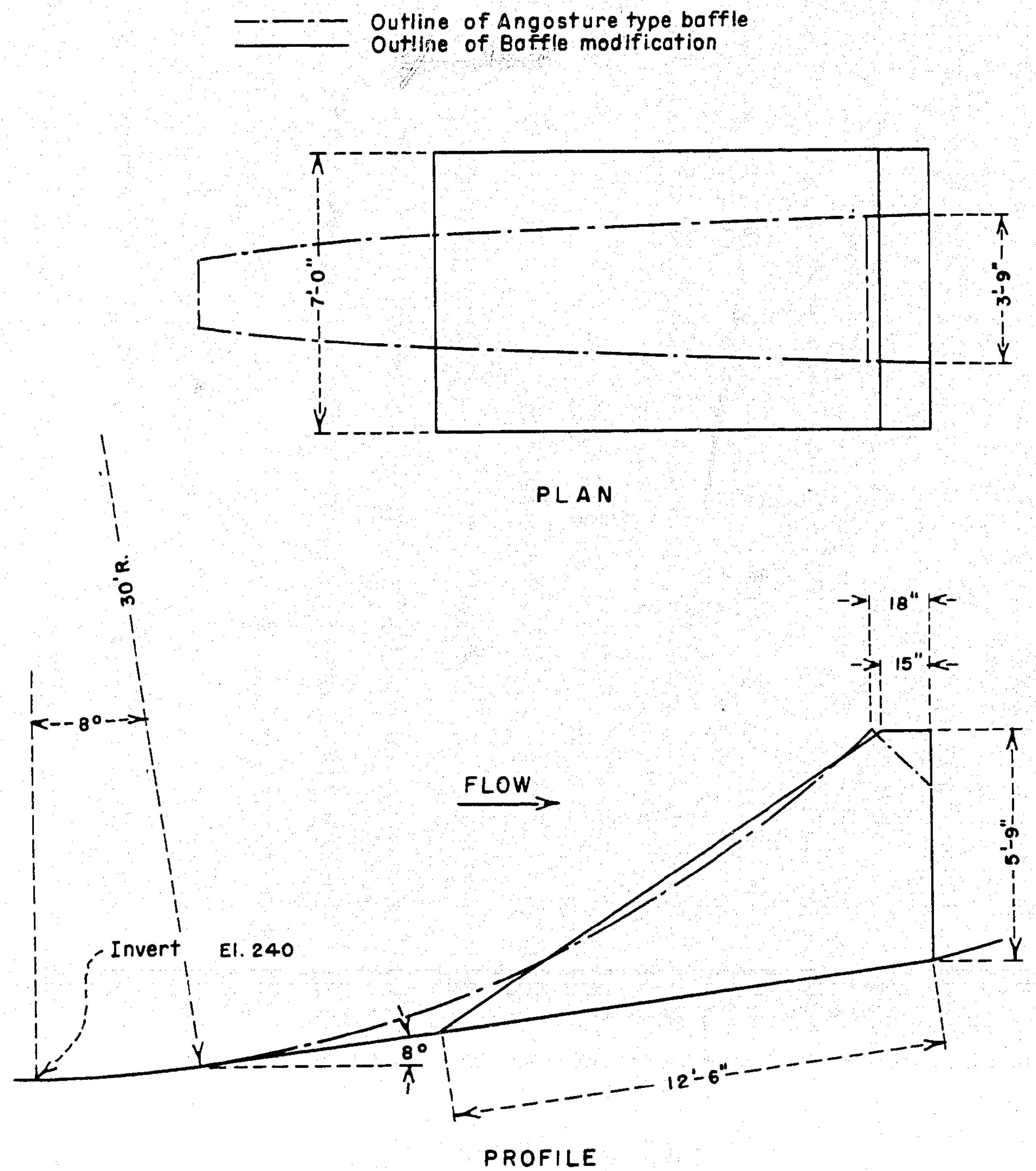
B. Baffles removed



C. Solid sill in place of baffles.

Note: The conditions shown are at the conclusion of a 30-minute model erosion test in which the discharge is 8,000 second feet, the reservoir at elevation 283.5, and the tailwater at elevation 271.

PALO VERDE DIVERSION DAM
PRELIMINARY SPILLWAY BUCKET DISCHARGING
WITH & WITHOUT MODIFICATIONS
1:28.3 SCALE MODEL

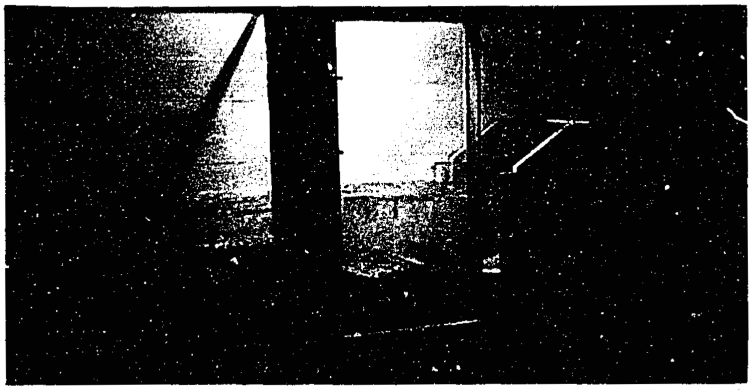


PALO VERDE DIVERSION DAM

MODIFICATION OF THE PRELIMINARY BAFFLE IN THE SPILLWAY BUCKET

A. Baffles 14-feet wide, slots 25-inches

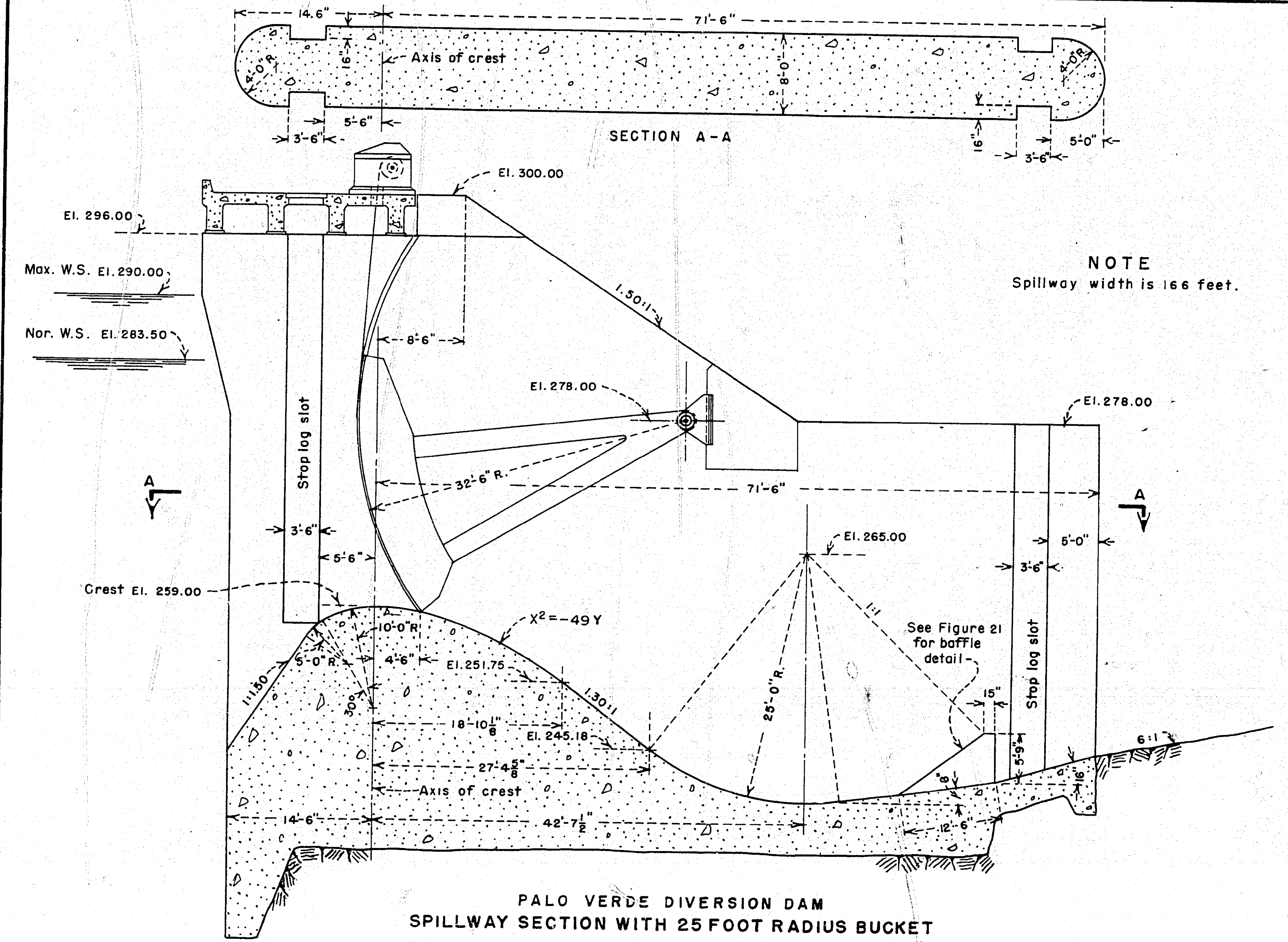
B. Baffles 7-feet wide, slots 18-inches.



C. Baffles taper from 7-feet upstream to 6-feet 9 inches downstream, slots from 24 inches to 27 inches.

Note: The conditions shown are at the conclusion of a 30 minute model erosion test in which the discharge is 8,000 second feet, the reservoir at elevation 283.5, and the tailwater at elevation 271.

PALO VERDE DIVERSION DAM
PRELIMINARY SPILLWAY BUCKET WITH BAFFLE MODIFICATIONS
1:28.3 SCALE MODEL

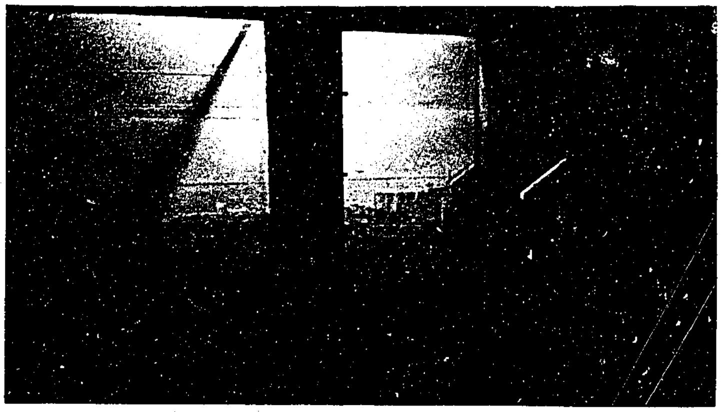


PALO VERDE DIVERSION DAM
SPILLWAY SECTION WITH 25 FOOT RADIUS BUCKET

543

A. Baffles taper from 5 feet 9 inches upstream to 5 feet 6 inches down stream, slots from 24 inches to 27 inches. See Figure 21 for baffles profile dimensions.

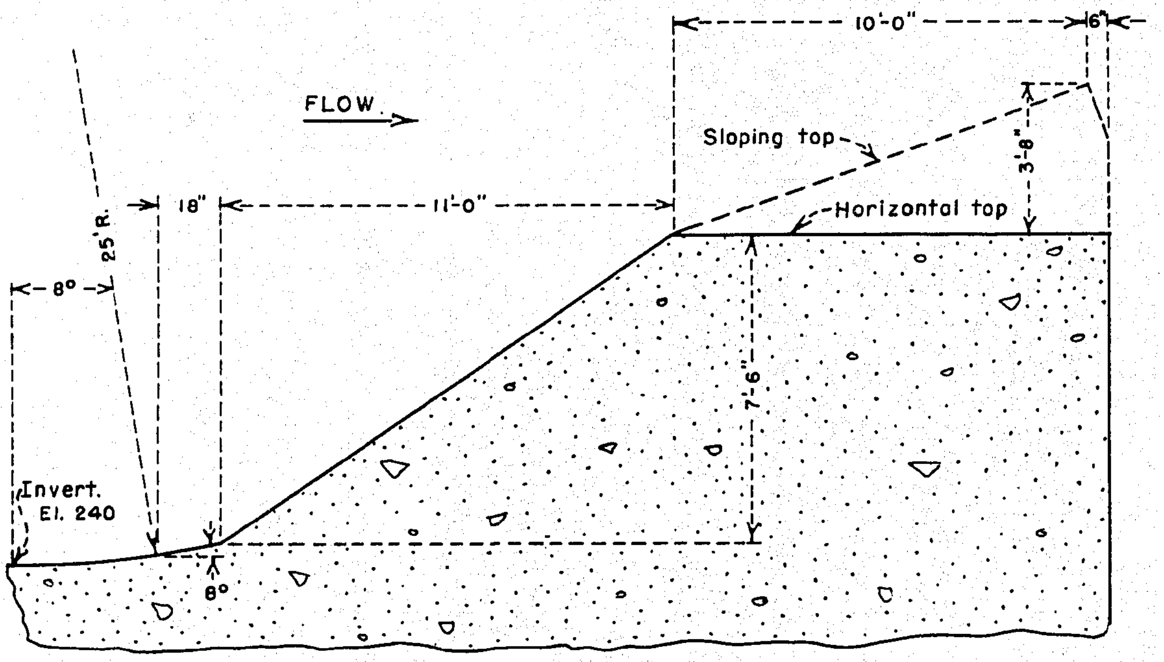
B. Solid bucket with horizontal top on end sill. See Figure 25.



C. Solid bucket with sloping top on end sill. See Figure 25.

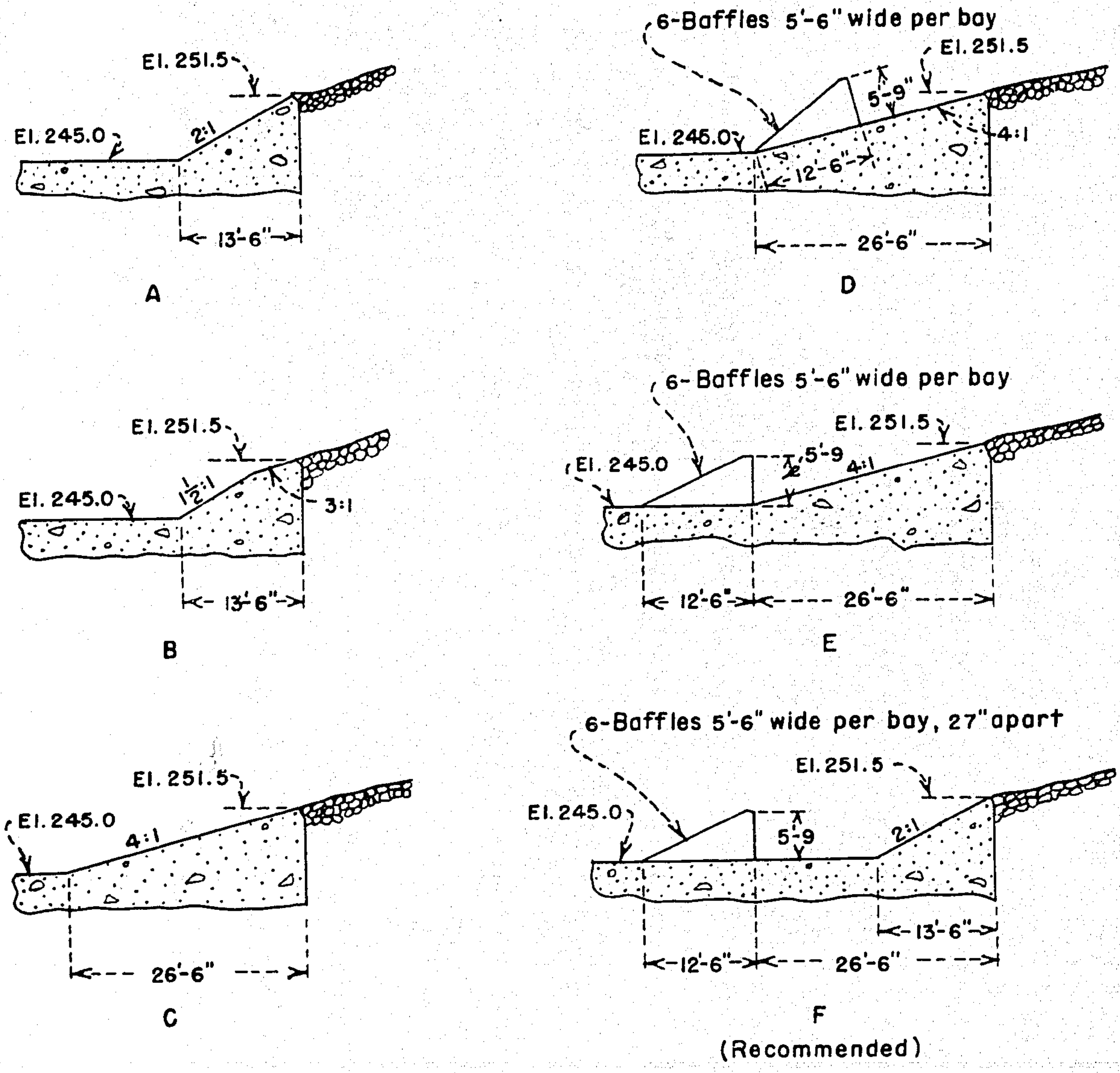
Note: The conditions shown are at the conclusion of a 30-minute model erosion test in which the discharge is 8,000 second feet, the reservoir at elevation 283.5, and the tailwater at elevation 271.

PALO VERDE DIVERSION DAM
25-FOOT RADIUS SPILLWAY BUCKET WITH MODIFICATIONS
1:28.3 SCALE MODEL



SECTION

PALO VERDE DIVERSION DAM
25 FOOT RADIUS SOLID SPILLWAY BUCKET

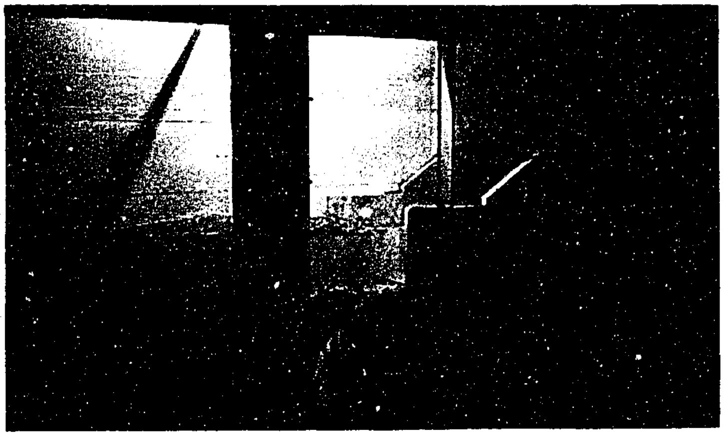


NOTE: End of basin is 71'-6" downstream from crest axis. See Figure 3 for crest shape and elevation.

PALO VERDE DIVERSION DAM
SILL AND BAFFLE ARRANGEMENTS
ON HORIZONTAL SPILLWAY APRON

A. 2:1 sloping end sill.
See Figure 26A

B. Double slope end sill.
See Figure 26B.



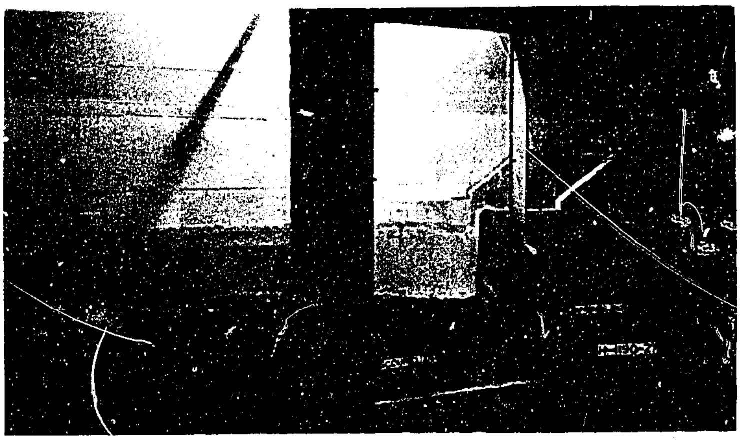
C. 4:1 sloping end sill.
See Figure 26C.

Note: The conditions shown are at the conclusion of a 30-minute model erosion test in which the discharge is 8,000 second feet, the reservoir at elevation 283.5, and the tailwater at elevation 271.

PALO VERDE DIVERSION DAM
HORIZONTAL APRON AT ELEVATION 245
WITH VARIOUS END SILL ARRANGEMENTS
1:28.3 SCALE MODEL

A. 6 baffles on 4:1 sloping end sill. See Figure 26D.

B. 6 baffles on apron upstream from 4:1 sill. See Figure 26E.

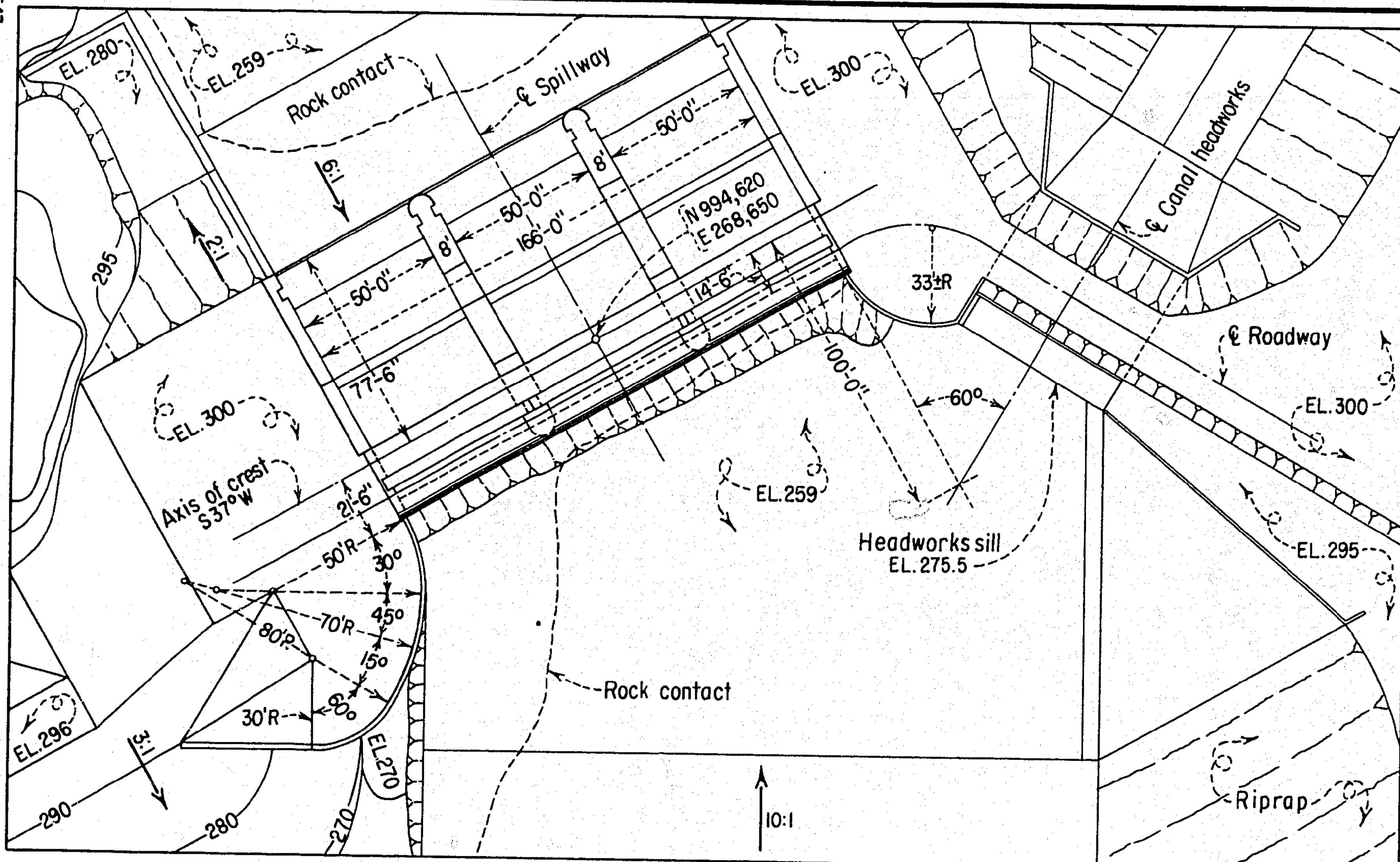


C. 6 baffles on apron upstream from 2:1 end sill. See Figure 26F. Hydraulic performance best of any design tested.

Note: The conditions shown are at the conclusion of a 30 minute model erosion test in which the discharge is 8,000 second feet, the reservoir at elevation 283.5, and the tailwater at elevation 271.

PALO VERDE DIVERSION DAM
HORIZONTAL APRON AT ELEVATION 245
WITH VARIOUS END SILL AND BAFFLE ARRANGEMENTS
1:28.3 SCALE MODEL

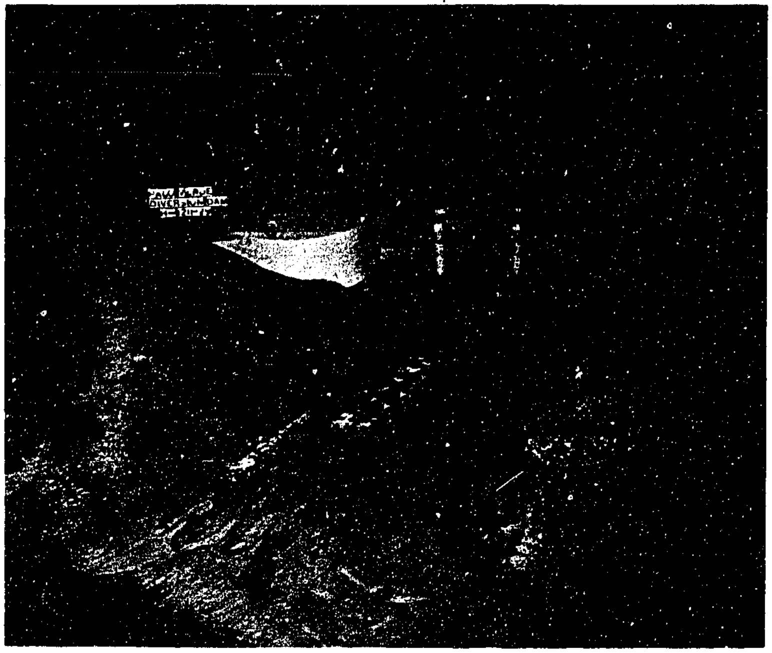
543



PALO VERDE DIVERSION DAM
PRELIMINARY SPILLWAY APPROACH AREA

FIGURE 29
REPORT HYD. 408

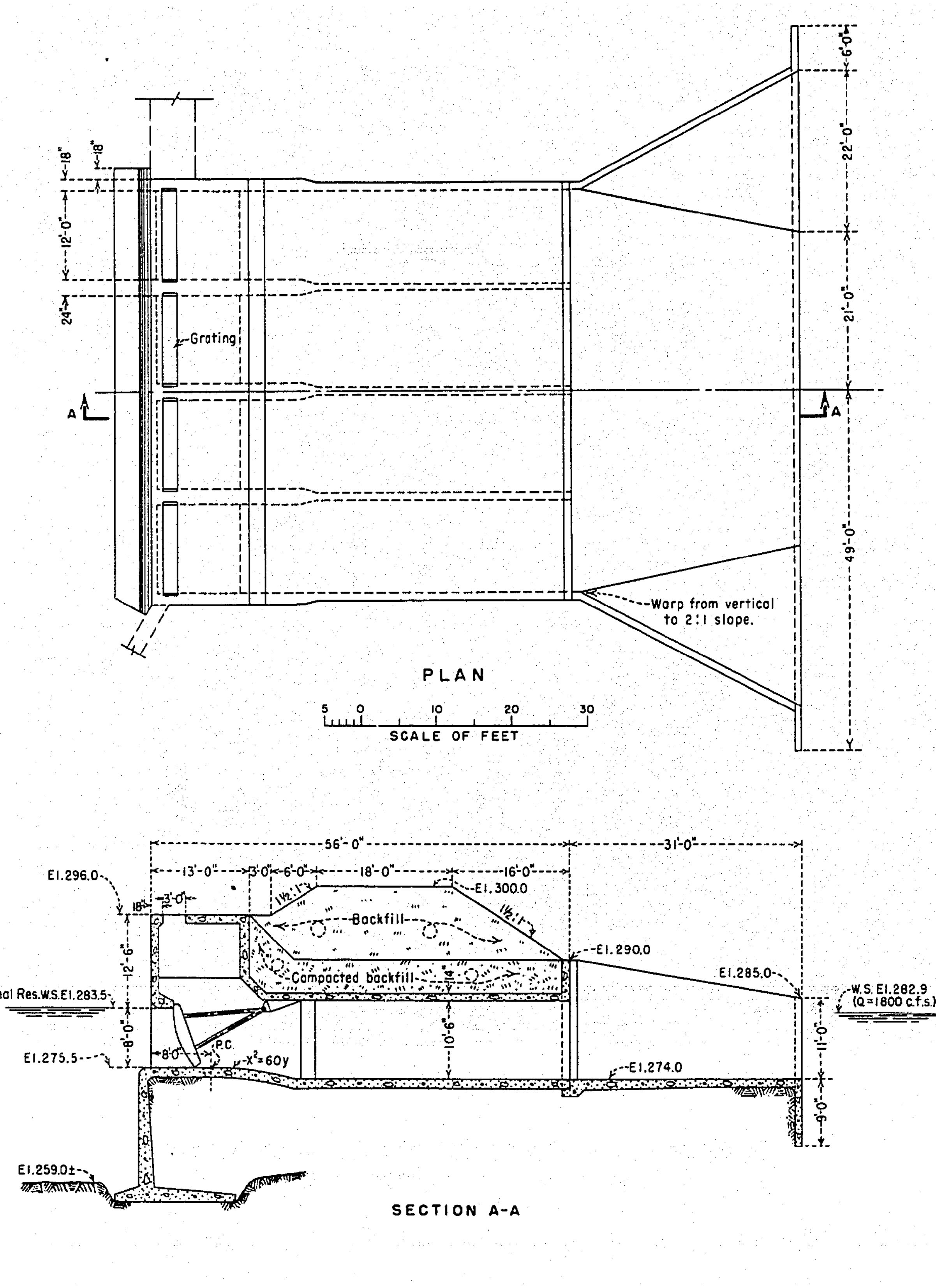
FIGURE 30
REPORT HYD 408



A. Spillway approach channel. Note the sand deposit near the canal headworks.

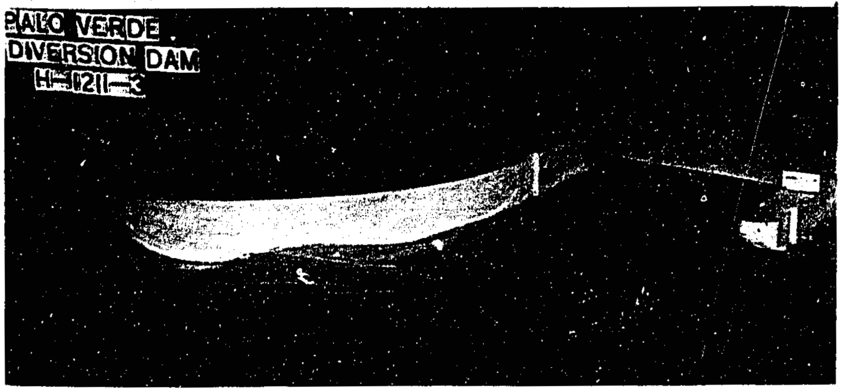
B. 75,000 second feet.

PALO VERDE DIVERSION DAM
PRELIMINARY SPILLWAY APPROACH AREA
MODEL VIEWS
1:50 SCALE MODEL

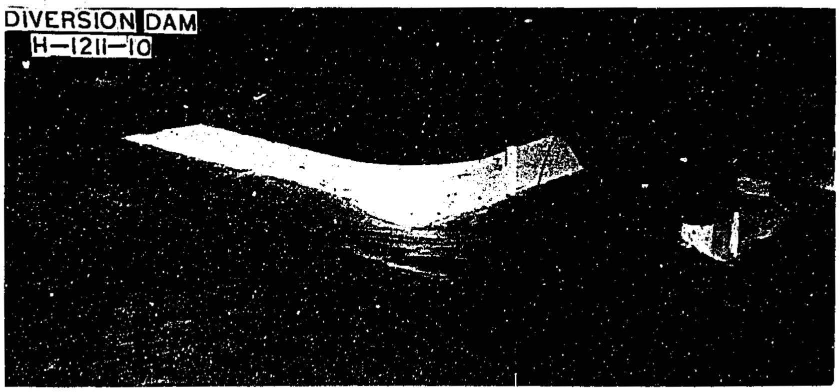


PALO VERDE DIVERSION DAM
PRELIMINARY CANAL HEADWORKS STRUCTURE

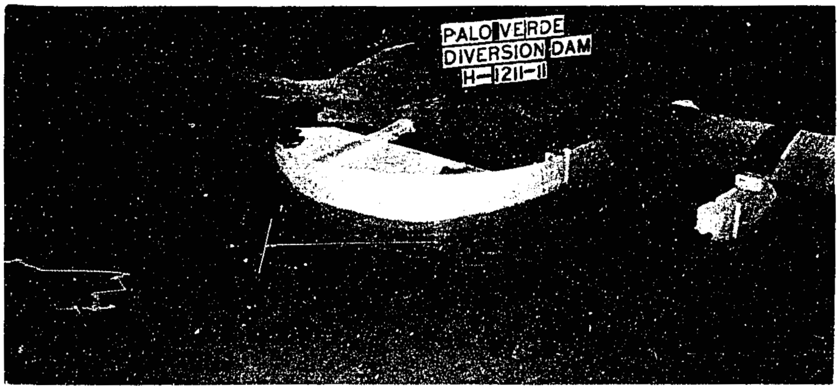
543



A. Preliminary left wall.

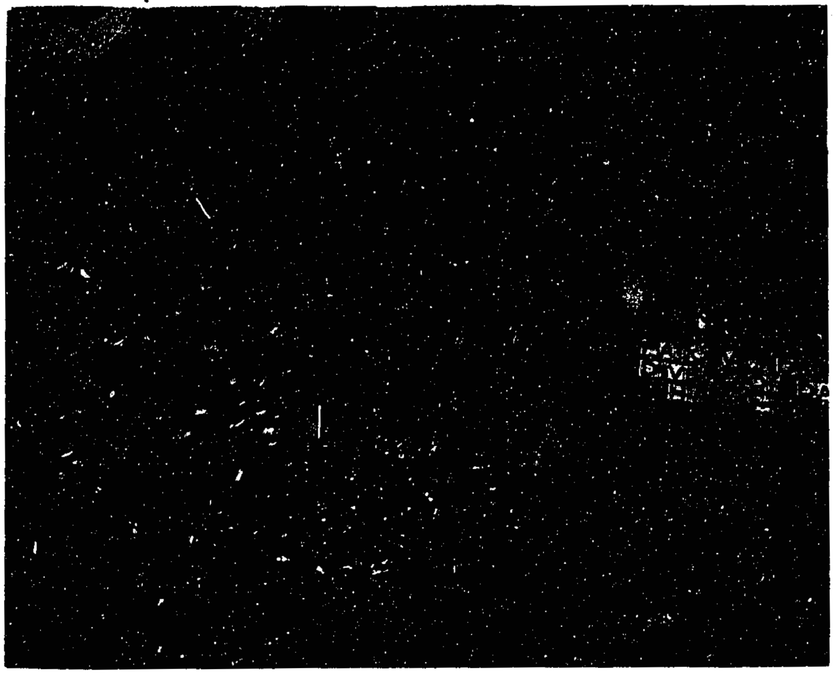


B. 24 foot radius wall

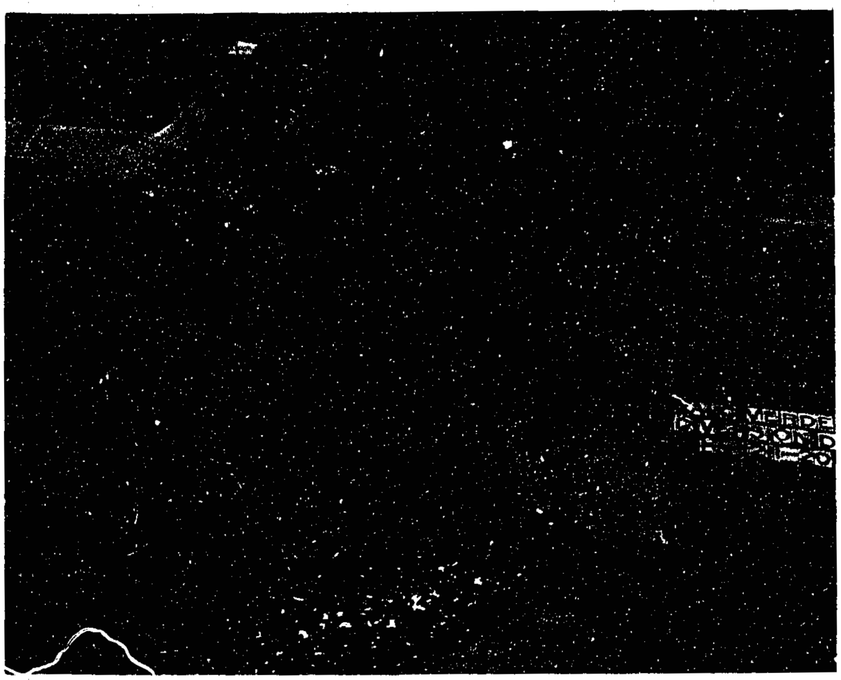


C. 51 foot radius wall - Recommended

PALO VERDE DIVERSION DAM
LEFT SPILLWAY APPROACH TRAINING
WALL 75,000 SECOND FEET
1:50 SCALE MODEL

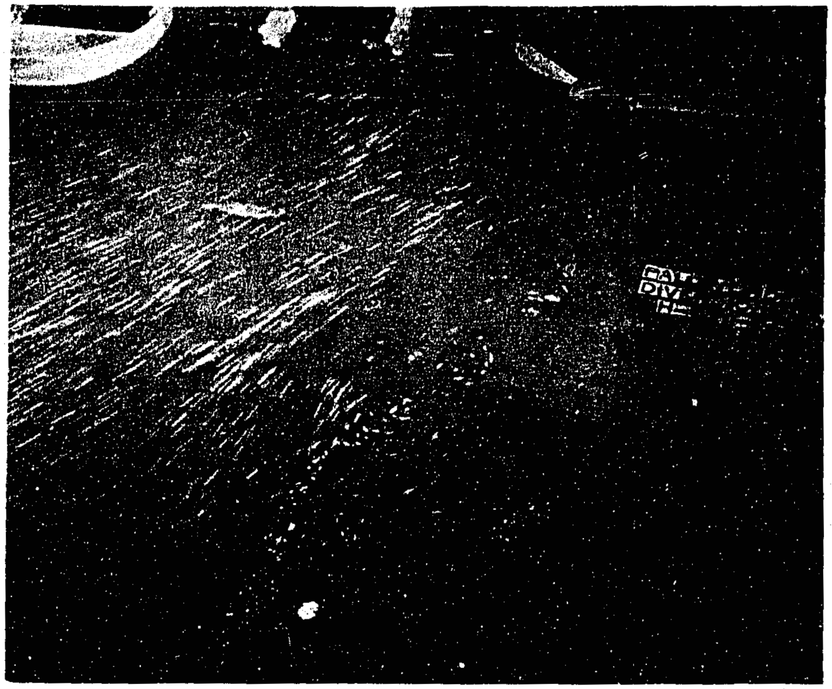


A. 15,000 second feet
(1/5 second exposure).

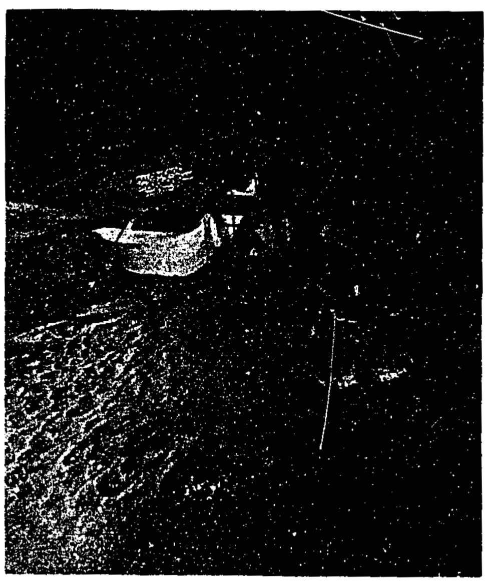


B. 34,000 second feet
(1/5 second exposure).

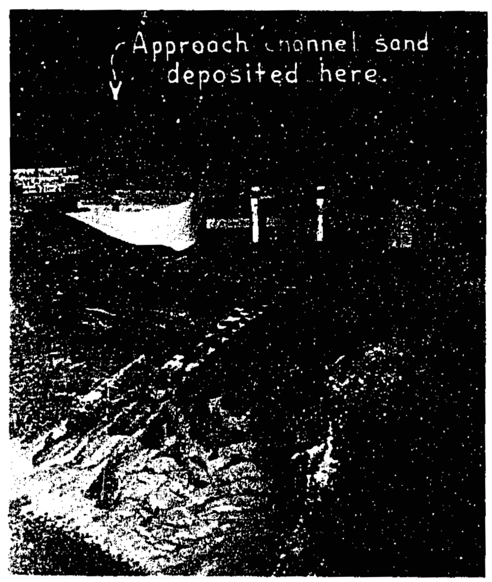
PALO VERDE DIVERSION DAM
FLOW PATTERNS IN THE PRELIMINARY SPILLWAY APPROACH
1:50 SCALE MODEL



A. Flow pattern on water surface
75,000 second feet discharging
(1/10 second exposure).

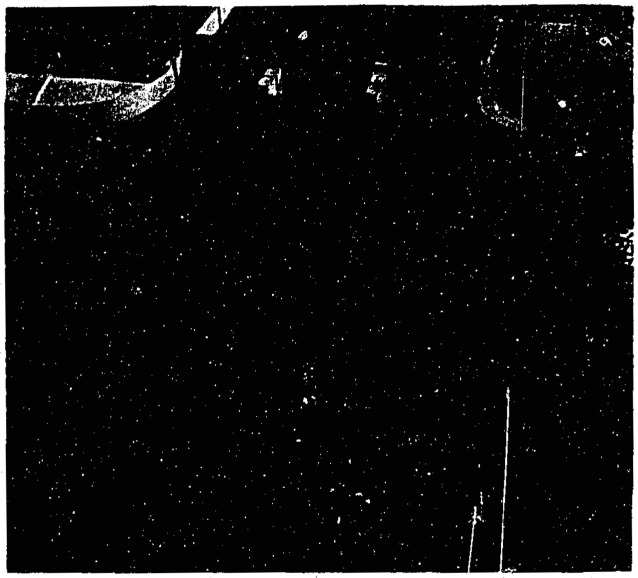


B. Flow pattern on approach
channel bed for 34,000
second feet.

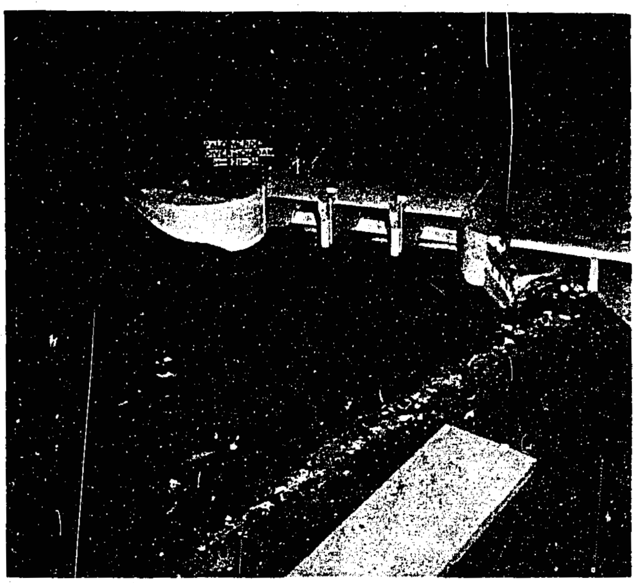


C. Flow pattern on approach
channel bed for 75,000
second feet.

PALO VERDE DIVERSION DAM
FLOW PATTERNS IN THE PRELIMINARY SPILLWAY APPROACH
1:50 SCALE MODEL

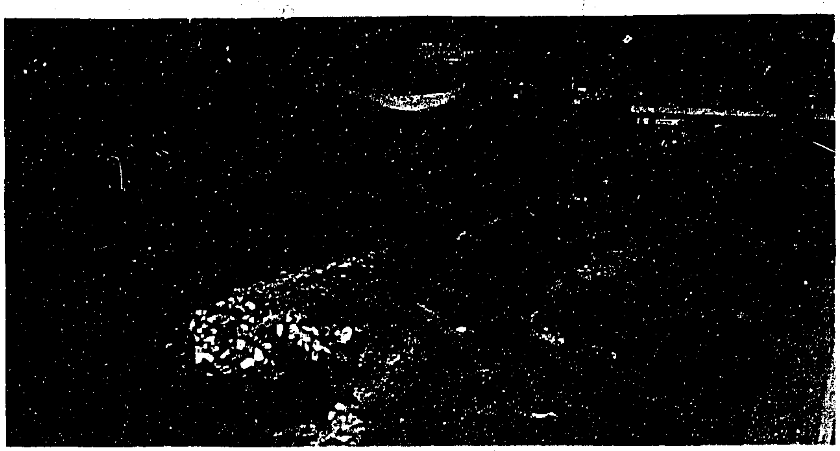


A. Flow pattern on water surface (1/10 second exposure)

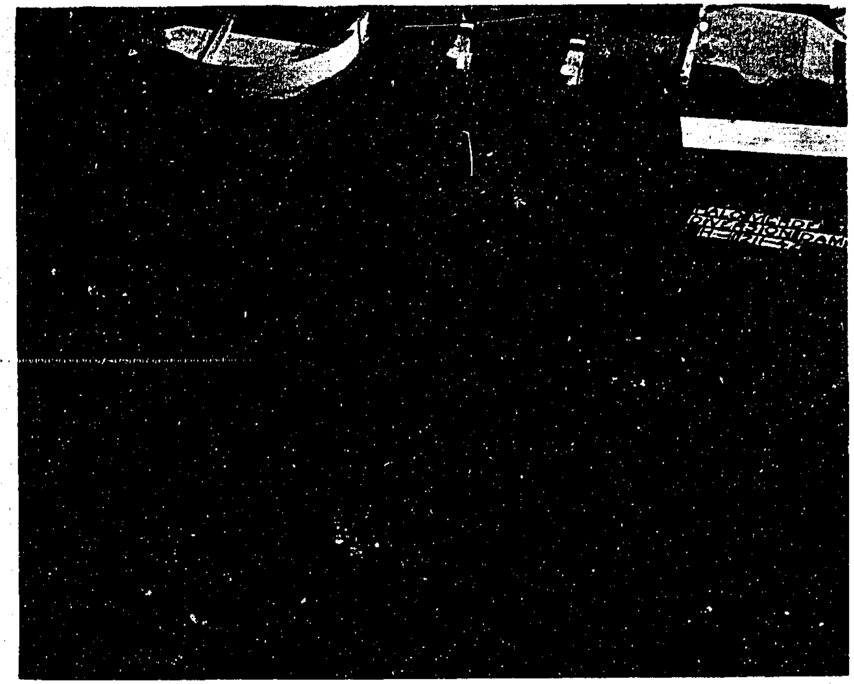


B. Flow pattern on approach channel bed.

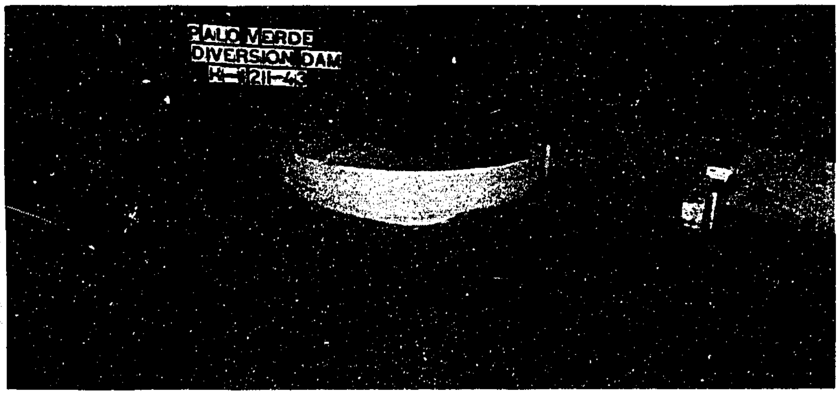
PALO VERDE DIVERSION DAM
FLOW PATTERNS IN THE SECOND SPILLWAY APPROACH
75,000 SECOND FEET
1:50 SCALE MODEL



A. The approach area.

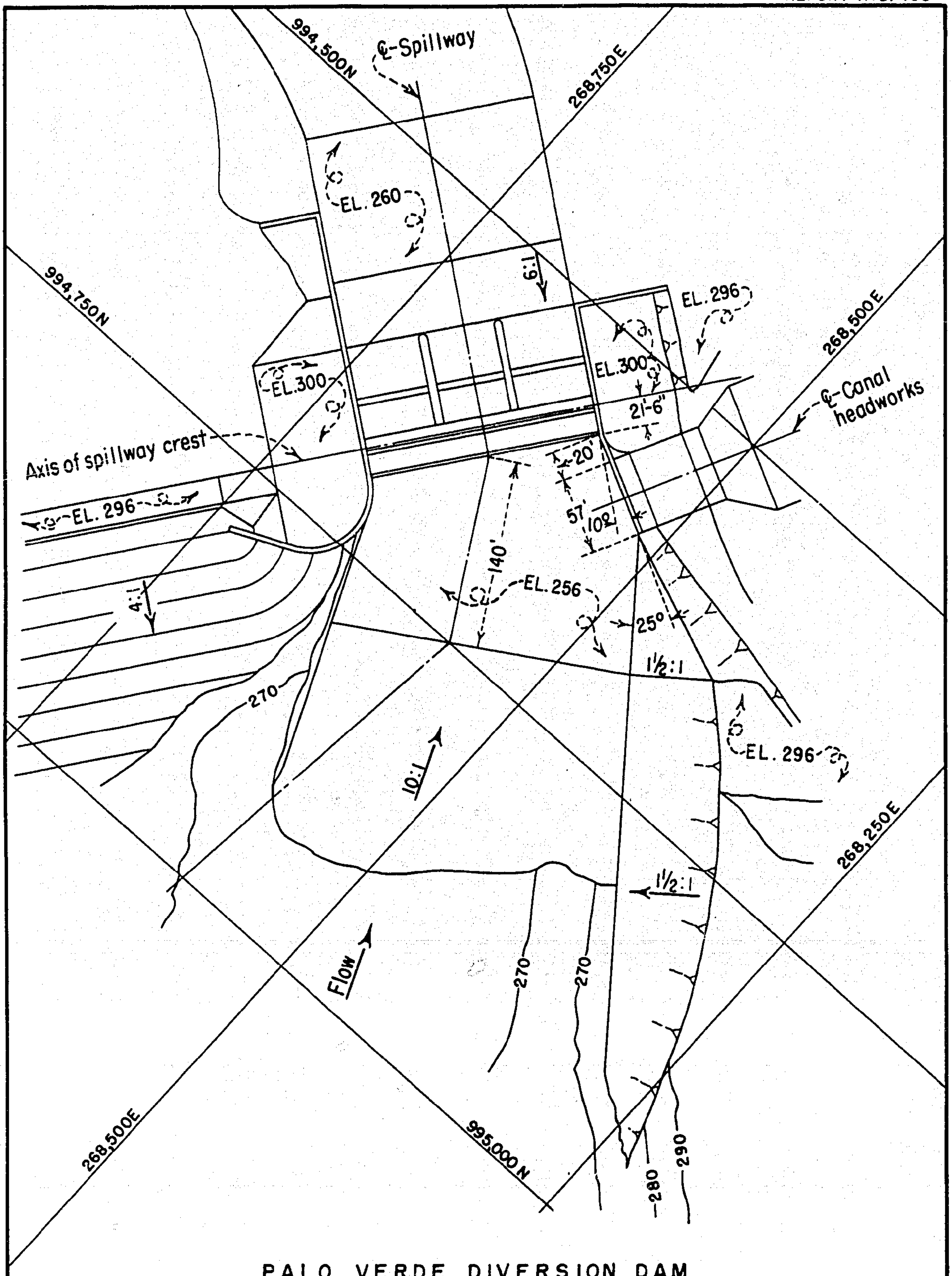


B. Flow pattern on water surface (1/10 second exposure).



C. Flow around left training wall.

PALO VERDE DIVERSION DAM
FLOW PATTERNS IN THE THIRD SPILLWAY APPROACH
75,000 SECOND FEET
1:50 SCALE MODEL



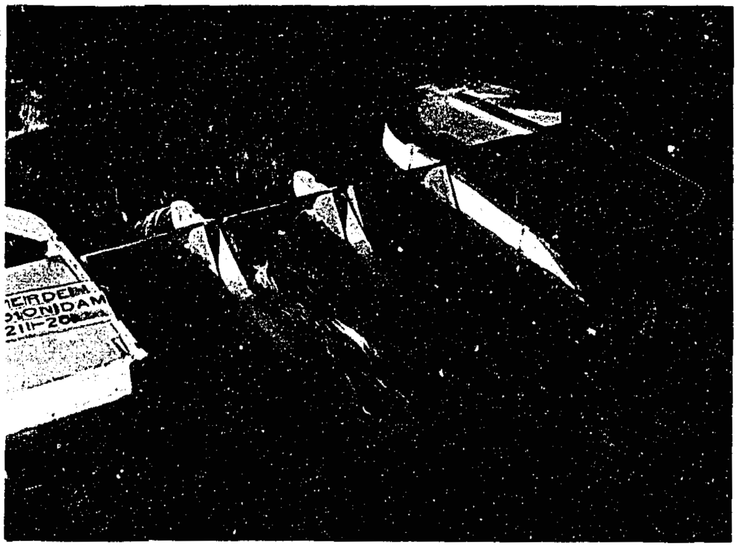
PALO VERDE DIVERSION DAM
RECOMMENDED SPILLWAY APPROACH AREA

A. Water surface flow patterns
34, 000 second feet discharging
(1/5 second exposure).

B. Water surface flow patterns
75, 000 second feet discharging
(1/10 second exposure).

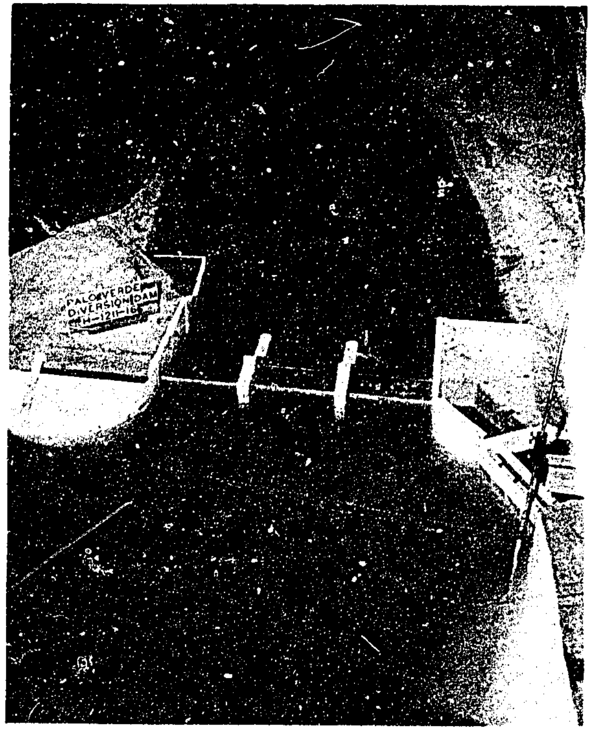
PALO VERDE DIVERSION DAM
FLOW PATTERNS IN THE RECOMMENDED SPILLWAY APPROACH
1:50 SCALE MODEL

A. Flow through the spillway approach 50,000 second feet with reservoir elevation 283.5 and tailwater elevation 281, expected in 1967.

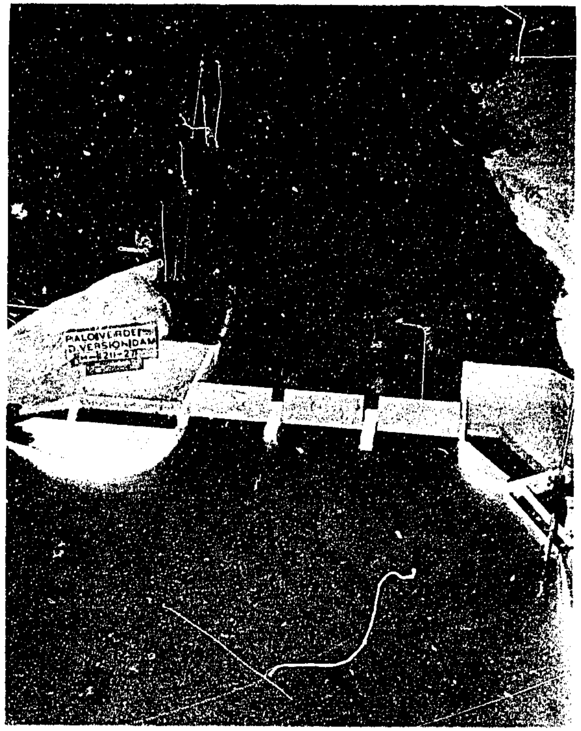


B. Flow through the spillway gate section 75,000 second feet. Present tailwater elevation 287.5.

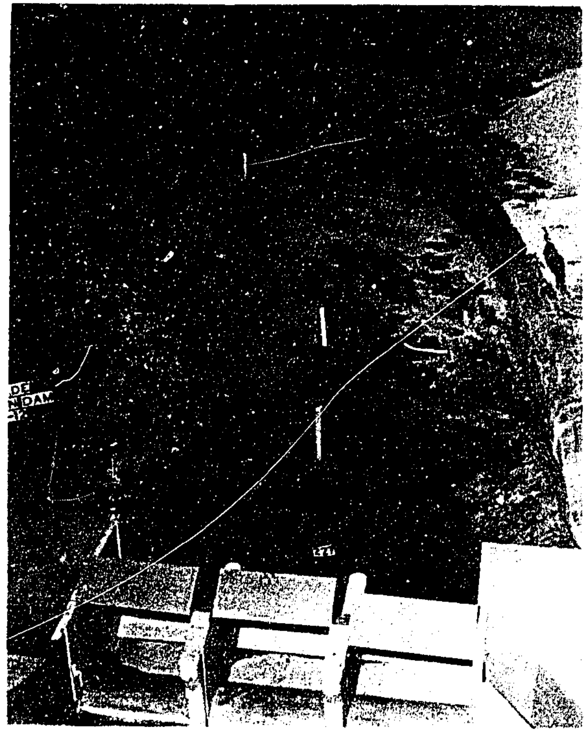
PALO VERDE DIVERSION DAM
FLOW CURRENTS THROUGH THE RECOMMENDED SPILLWAY APPROACH AND GATE SECTION
1:50 SCALE MODEL



A. 15,000 second feet with present expected tailwater elevation 277 in river channel. Reservoir elevation 283.5 near canal intake.

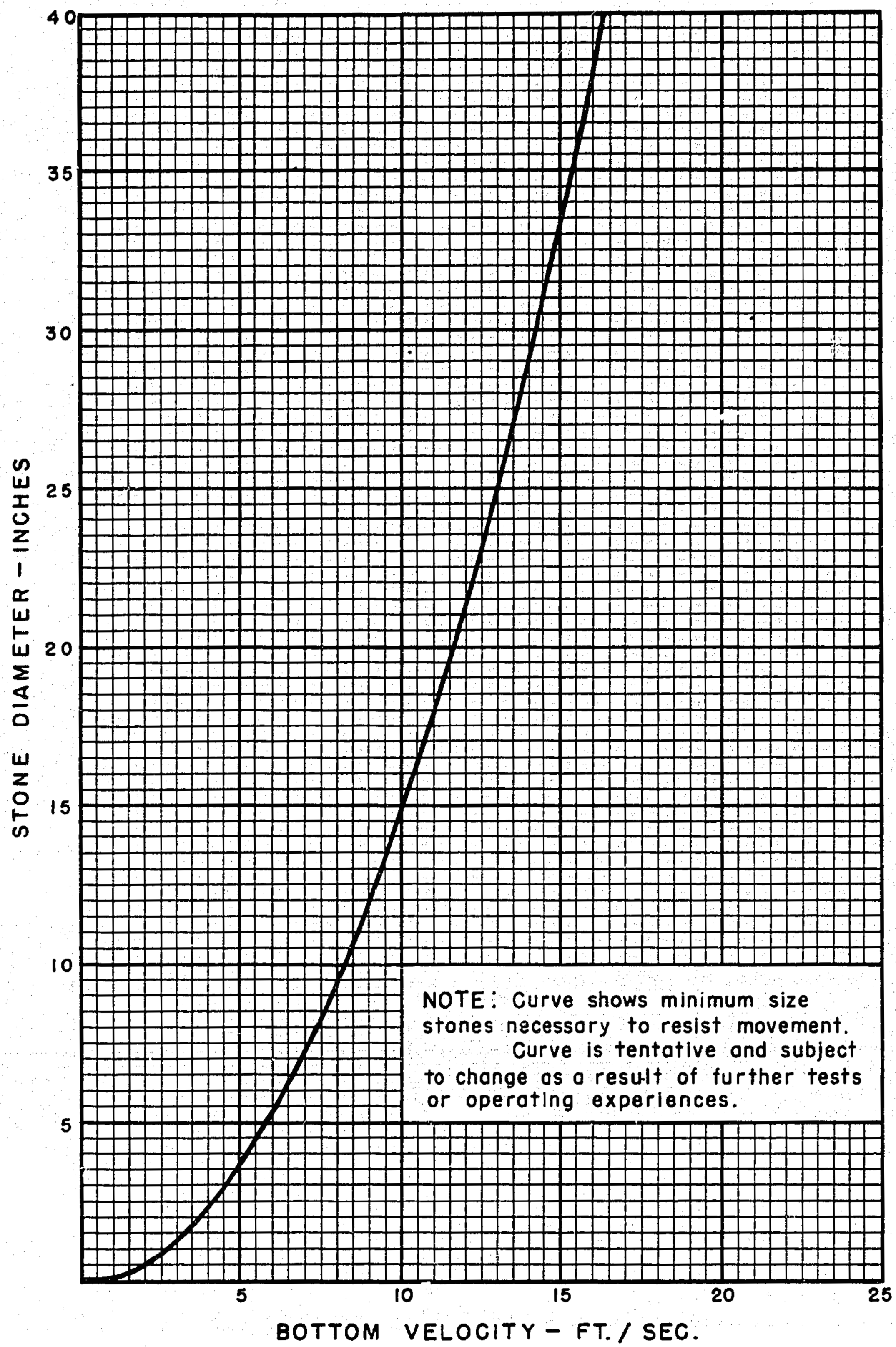


B. 75,000 second feet with present expected tailwater elevation 287.5 in river channel. Reservoir elevation 291.5 near headworks.



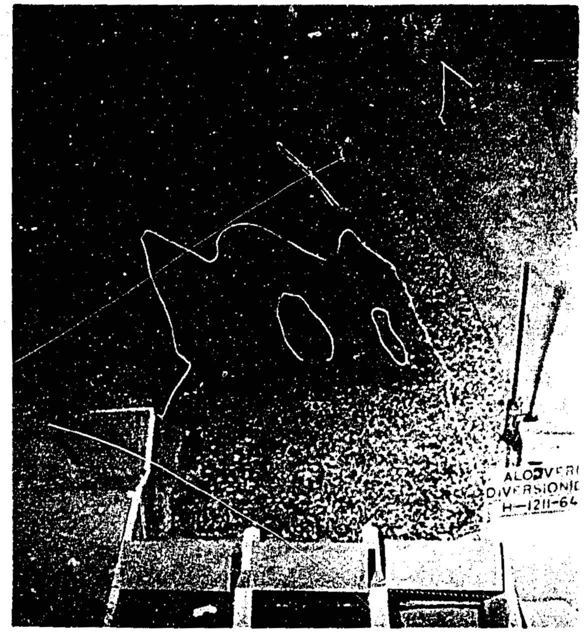
C. Erosion after 30 min. model test at 75,000 second feet with present expected tailwater elevation.

PALO VERDE DIVERSION DAM
RECOMMEND SPILLWAY DISCHARGING
1:50 SCALE MODEL



TENTATIVE CURVE TO AID IN DETERMING RIPRAP SIZES

A. 18 to 36 inch riprap in discharge channel prior to erosion test.

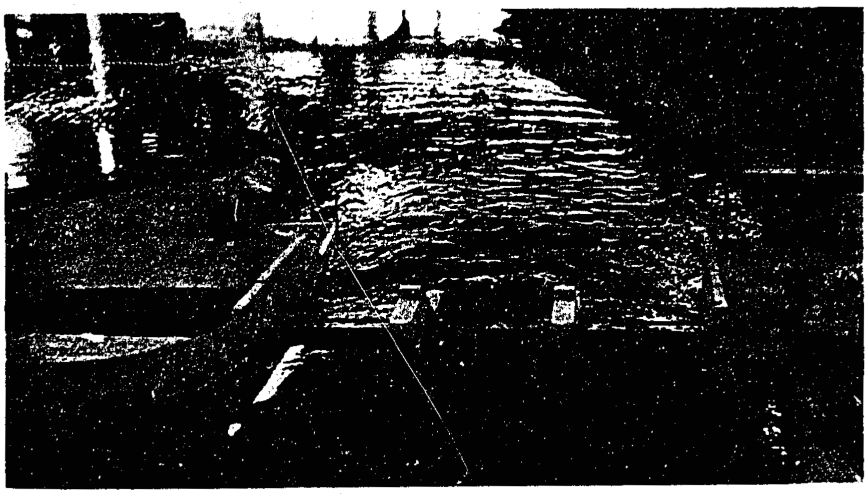


B. Erosion after 45,000 second feet for 30 minutes with tailwater at elevation 281 expected in year 1967. Reservoir at elevation 283.5 near headworks.

C. Erosion after 75,000 second feet for 30 minutes with tailwater at elevation 283 expected in year 1970. Reservoir at elevation 288 near headworks.

PALO VERDE DIVERSION DAM
DISCHARGE CHANNEL EROSION TRENDS
RECOMMENDED SPILLWAY
1:50 SCALE MODEL

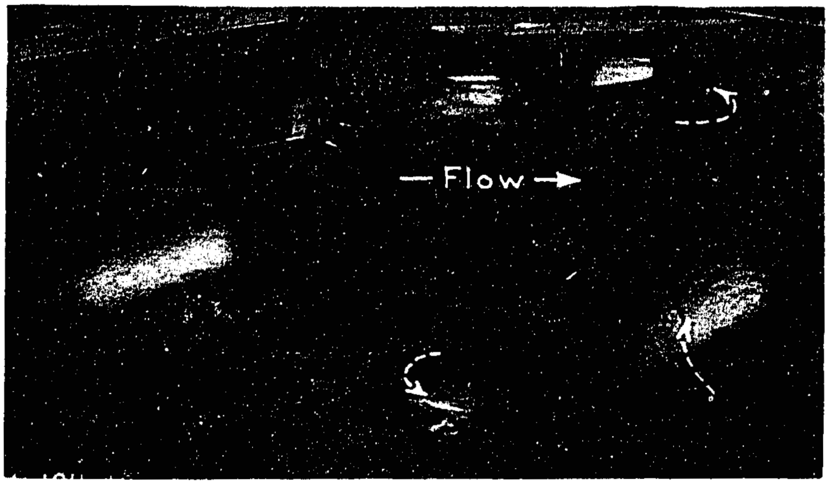
FIGURE 42
REPORT HYD 408



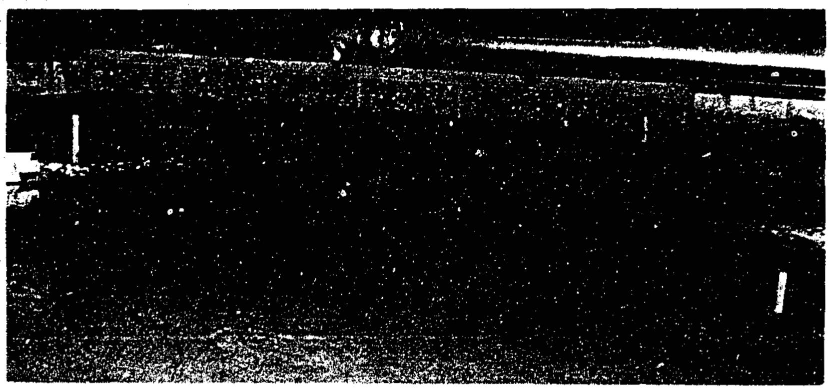
A. 15,000 second feet. Tailwater elevation 274 in river channel for year 1962. Reservoir elevation 283.5 near canal headworks.

B. 75,000 second feet. Tailwater elevation 286 in river channel for year 1962. Reservoir elevation 290 near canal headworks.

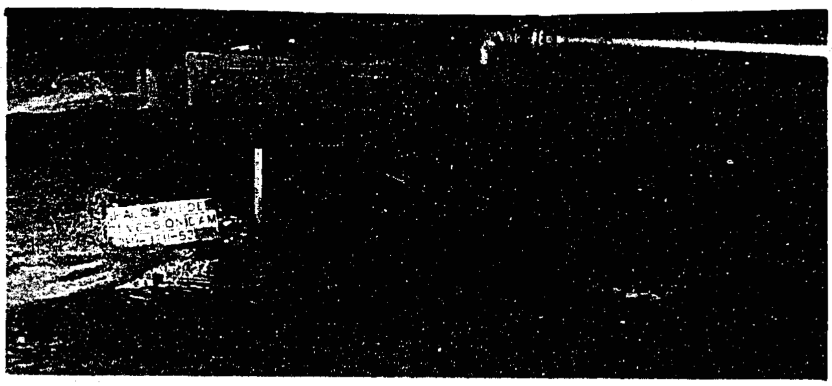
PALO VERDE DIVERSION DAM
DISCHARGE CHANNEL WAVES
RECOMMENDED SPILLWAY
1:50 SCALE MODEL



A. Water surface upstream of weir approximately elevation 288, downstream approximately 278. Discharge approximately 15,000 second feet. (See Figure 9 for prototype comparison).

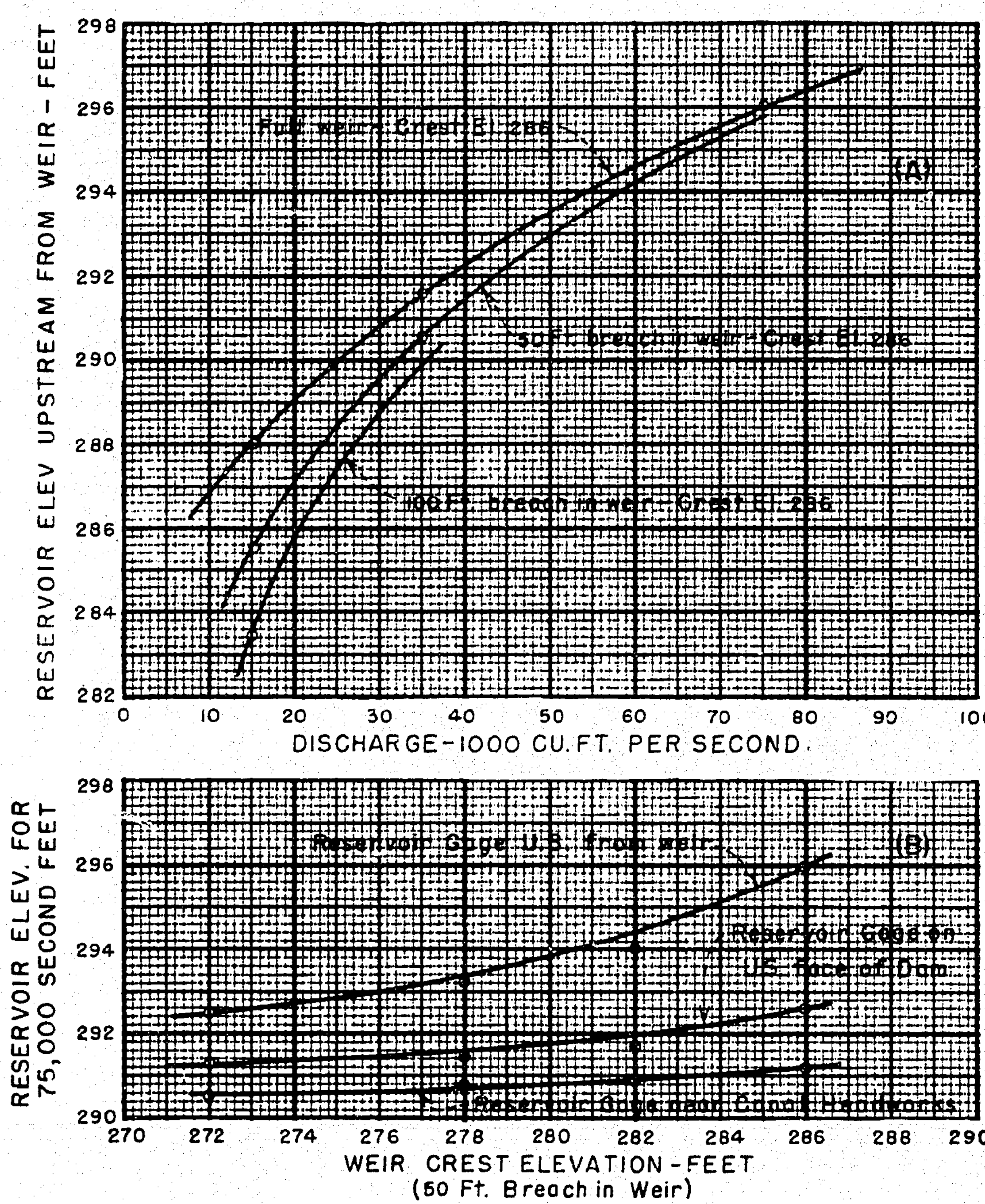


B. Discharge 15,000 second feet, water surface upstream of weir at elevation 288.4 and water surface downstream of weir at elevation 283.5.



C. 50' Breach--Discharge 15,000 second feet, water surface upstream of weir at elevation 286.8, and downstream of weir at elevation 283.5.

PALO VERDE DIVERSION DAM
TEMPORARY ROCK WEIR AT ELEVATION 286
1:50 SCALE MODEL



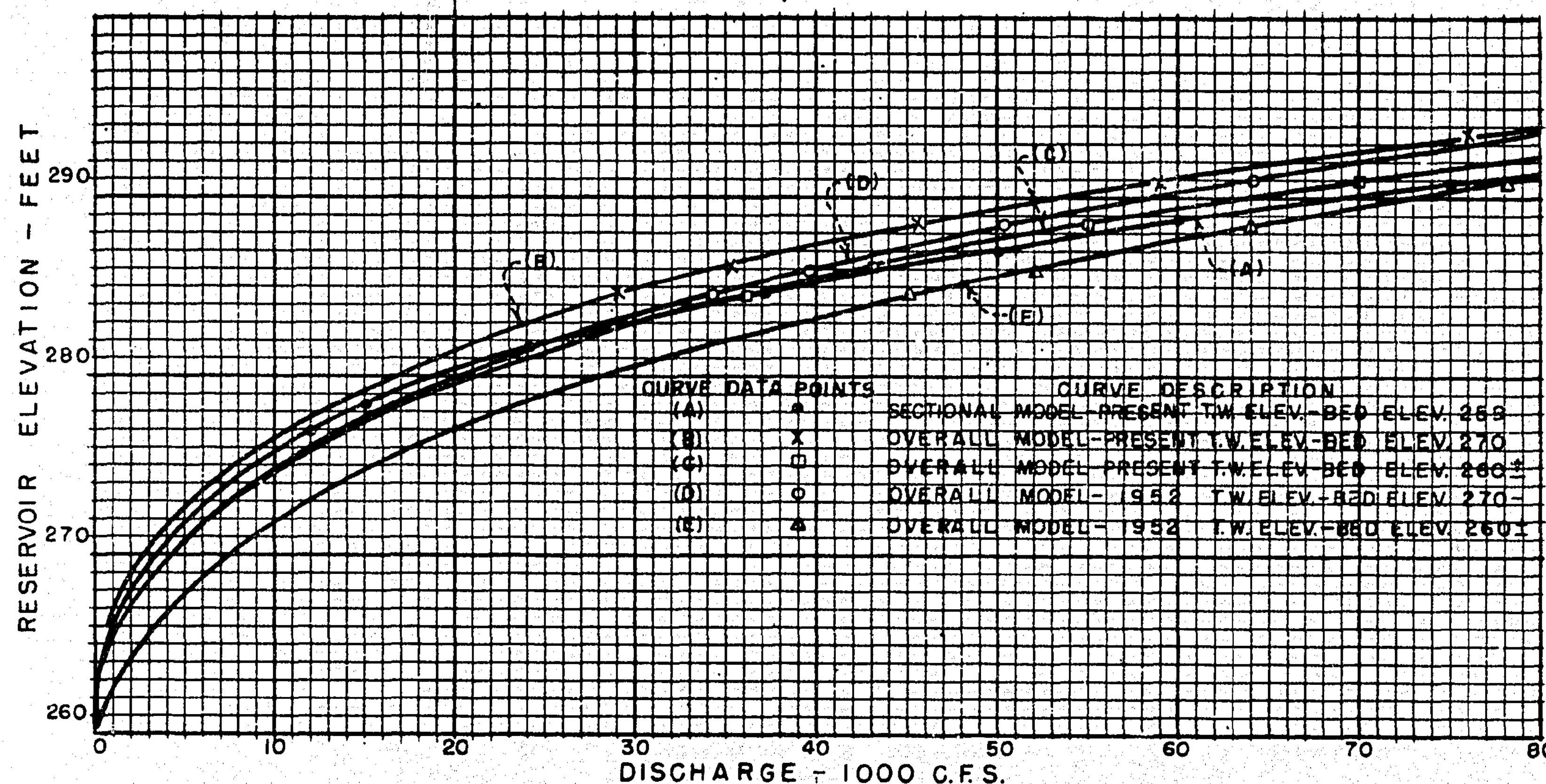
NOTES

The reservoir elevation was not controlled by the spillway gates. The tailwater elevation in river channel downstream from spillway was controlled to the expected present tailwater elevation shown in Figure 14.

PALO VERDE DIVERSION DAM

ROCK WEIR DISCHARGE AND RESERVOIR ELEVATION CURVES

1:50 SCALE MODEL

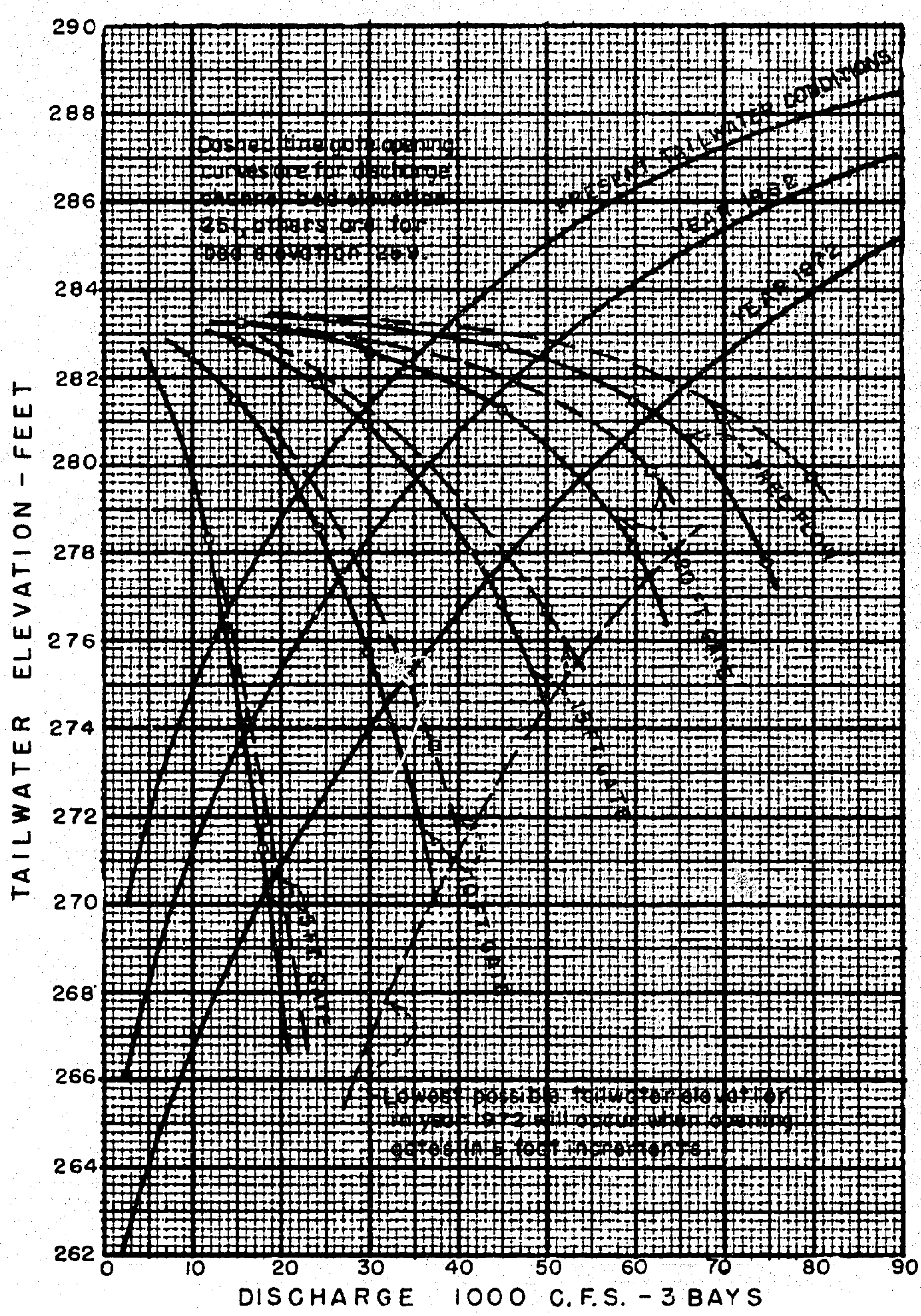


Tailwater (T.W.) elevation in the overall model was measured in the river channel at Gage A in Figure 17. Bed elevation shown above is the highest elevation of the discharge channel bed. Reservoir elevation in the overall model was measured at gage E in Figure 17.

PALO VERDE DIVERSION DAM

UNCONTROLLED SPILLWAY DISCHARGE CURVES

1:28.3 AND 1:50 SCALE MODELS



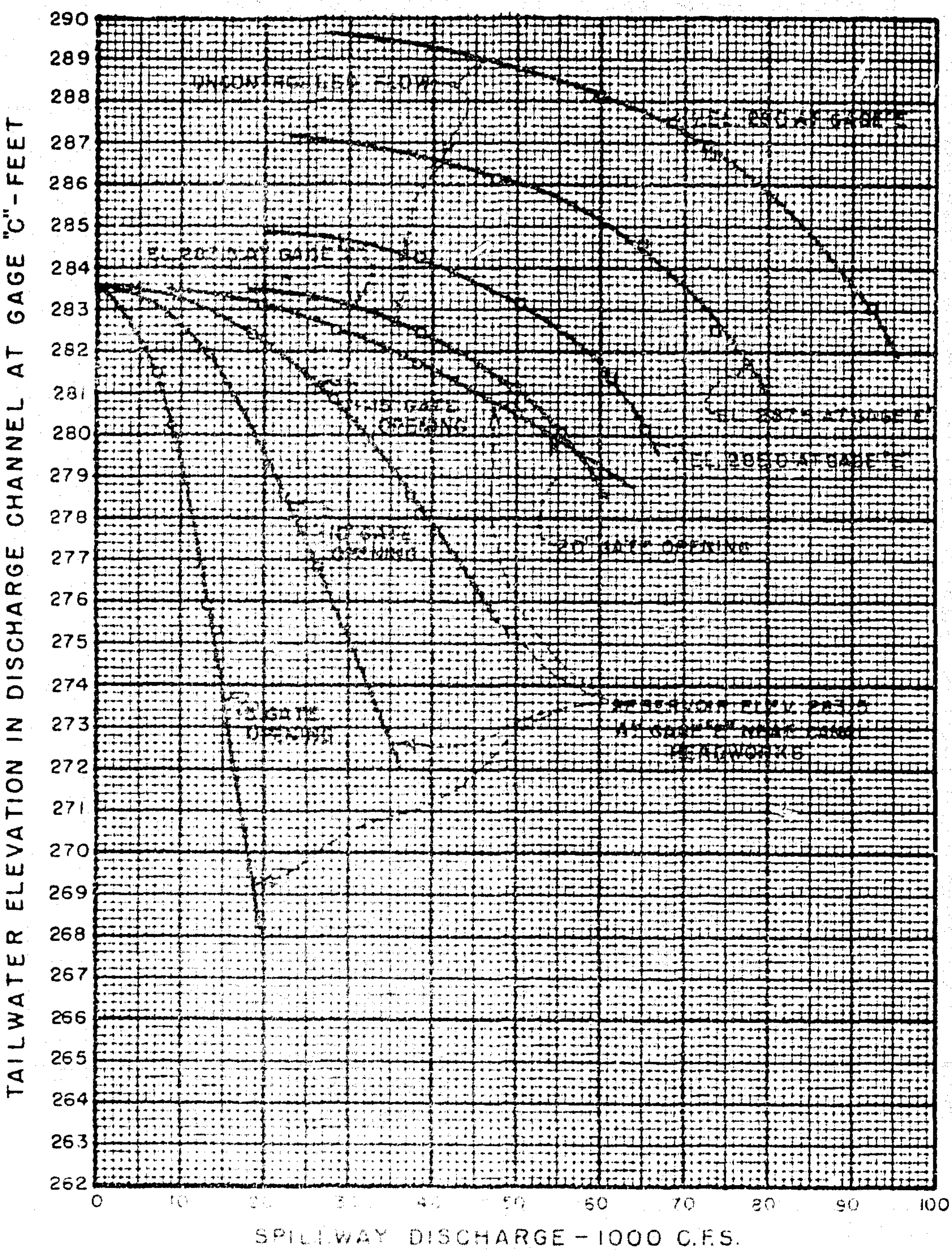
NOTE

Reservoir elevation 283.5 was measured at approximately 130 feet upstream from crest. Tailwater elevation was measured at approximately 370 feet downstream from crest.

PALO VERDE DIVERSION DAM

PRELIMINARY SPILLWAY CALIBRATION FOR RESERVOIR ELEVATION 283.5 FEET

1:28.3 SCALE MODEL

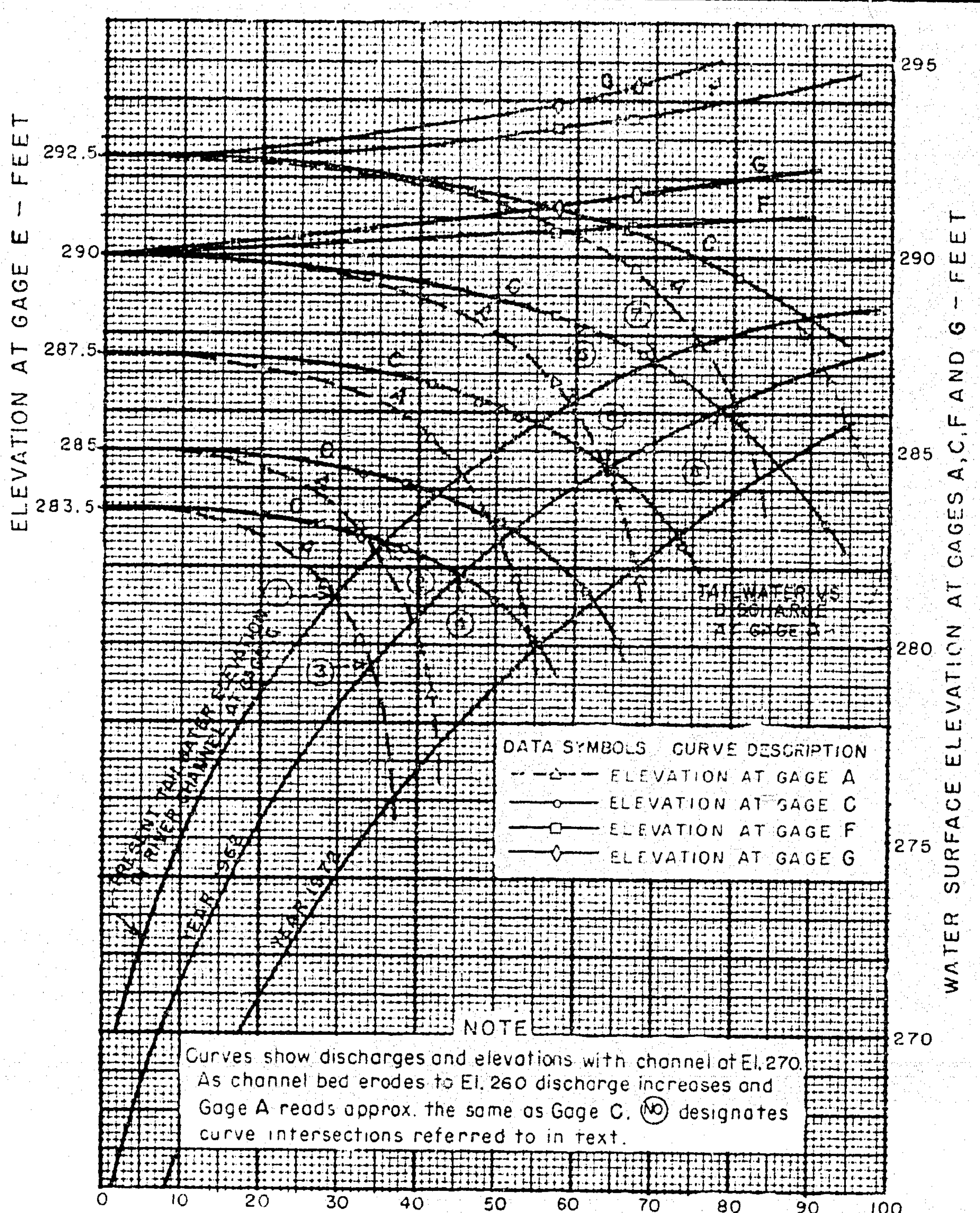


NOTE
See Figure 17 for Gage locations

PALO VERDE DIVERSION DAM

FINAL SPILLWAY CALIBRATION CURVES FOR CONTROLLED AND UNCONTROLLED FLOW

1:60 SCALE MODEL

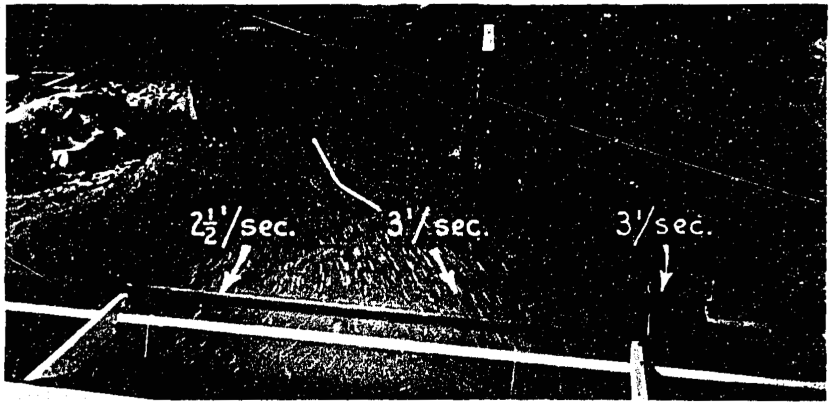


SPILLWAY DISCHARGE - 1000 SECOND FEET
(The canal headworks discharged an additional 2000 second feet approx.)

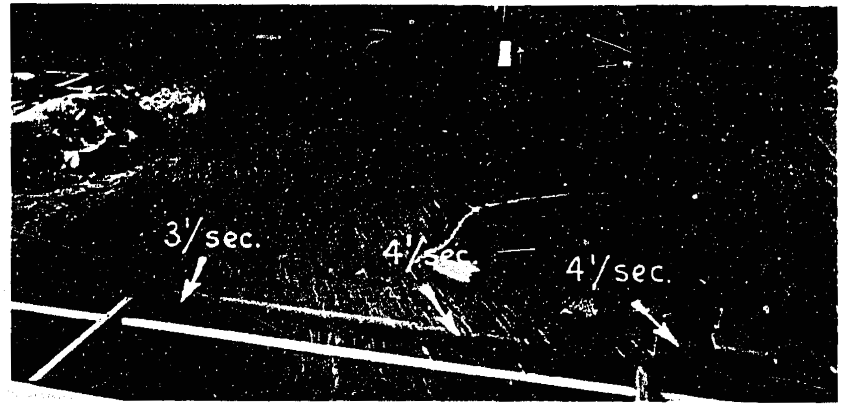
GAGE LOCATION (See Figure 17 for exact location)

E	Upstream of spillway near canal headworks
A	River channel
C	Spillway discharge channel
F	Upstream face of dam
G	Upstream from rock weir

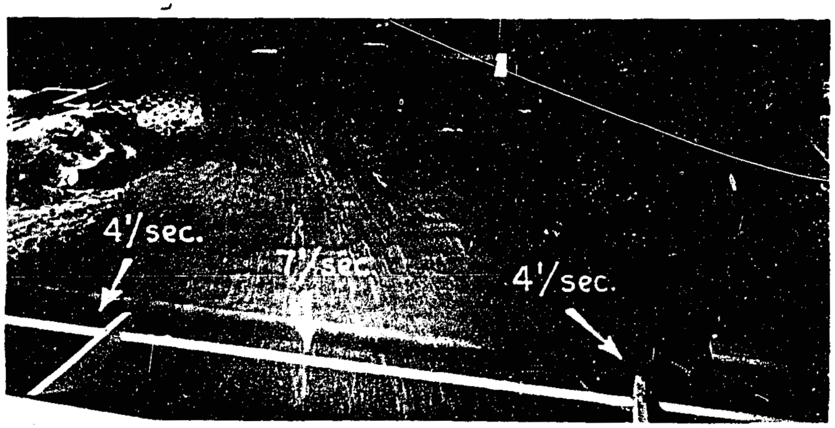
PALO VERDE DIVERSION DAM
SPILLWAY DISCHARGE VERSUS WATER SURFACE ELEVATIONS
FOR UNCONTROLLED FLOW
1 : 50 SCALE MODEL



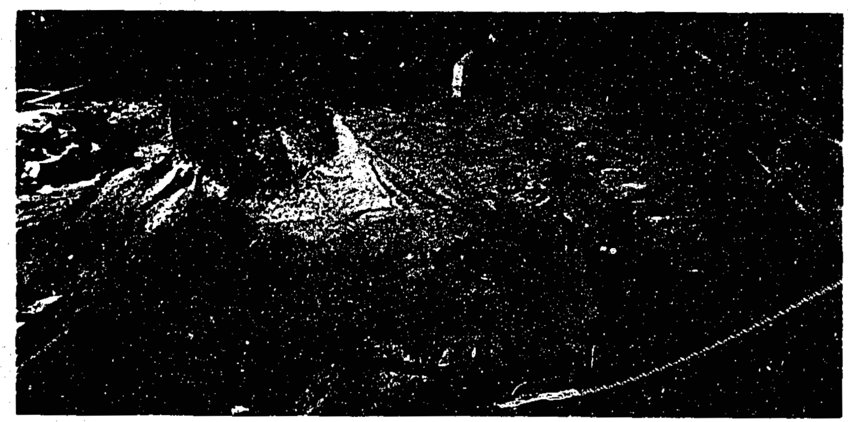
A. 15,000 second feet



B. 34,000 second feet.



C. 75,000 second feet



D. Erosion following 75,000 second feet

PALO VERDE DIVERSION DAM
RIVER FLOW CURRENTS
1:50 SCALE MODEL

FIGURE 50
REPORT HYD 408