UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

MODEL STUDIES OF THE FRIANT-KERN CANAL OUTLETS; FRIANT-MADERA CANAL OUTLETS; AND THE FRIANT SPILLWAY AND RIVER OUTLETS

FRIANT DAM
CENTRAL VALLEY PROJECT – CALIFORNIA

Hydraulic Laboratory Report No. Hyd. 166

ENGINEERING AND GEOLOGICAL
CONTROL AND RESEARCH DIVISION

FEBRUARY 23, 1945

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UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Branch of Design and Construction
Engineering and Geological Control
and Research Division
Denver, Colorado
February 23, 1945

Laboratory Report No. 166
Hydraulic Laboratory

Compiled by: C. V. Adkins
Reviewed by: J. E. Warnock

Subject: Model studies of the Friant-Kern Canal Outlets: Friant-Madera Canal Outlets; and the Friant Spillway and River Outlets--Friant Dam--Central Valley Project, California.

SUMMARY

Development of the Friant-Kern Canal Outlet Stilling Pool and Apron.

The 1:34.29 model. The design as first submitted to the laboratory for testing in 1936, is shown on Figure 3. A 1:34.29 scale ratio was adopted to make use of 2.80-inch needle valves already on hand. The apron design as submitted was never tested. Instead, a trajectory was computed, using a velocity based on the maximum head of 98.0 feet. The model apron was constructed to this trajectory. Due to successive changes from the design department, only observation runs were made on this model before it was abandoned.

The centerline of the valves had been lowered from elevation 466.0 to 464.0 and the pool floor fixed at elevation 450.12. Two test runs were made with one of the 2.80-inch needle valves and trajectory measurements taken. An equation for the apron was developed from these measurements. This apron was installed in the model and initial runs were made. The jump in the pool was unstable, moving from side to side. The jets did not spread soon enough, suggesting that the apron would have to be raised and possibly the pool deepened. Indications were that the action in the pool could be improved materially by changing the alinement of the valves. The apron was gradually raised and lengthened and the valve alinement changed several times before arriving at a combination that produced a satisfactory pool action for all operating conditions. The recommended valve alinement,

the stilling pool design, and the parabolic apron shape are shown on Figure 6.

The 1:34.38 model. Subsequent to the 1936 tests, the two 96-inch needle valves in the center were replaced with two 102-inch tube valves. The maximum reservoir water surface was increased from elevation 564.0 to 578.0 to provide for flood control. The angles of the two center valves were changed from 1 degree 0 minutes to 0 degrees 30 minutes and the apron lengthened from 82.15 to 89.50 feet. The width of the apron at the origin was increased from 74.0 to 78.0 feet. These changes justified additional model tests, so in 1939 a model was constructed to determine the adequacy of the design for the new conditions. Figures 7 and 8 show the design as submitted for model tests.

A vertical sill 3.50 feet high was placed at the end of the jump as a stabilizer and the canal floor at elevation 452.0 extended upstream to the sill. The centerlines of the valves were raised from elevation 464.0 to 464.25 to permit the jets from the tube valves to emerge tangent to the apron. The final arrangement as recommended from this model is shown in Figure 9.

The 1:32 model of 1944. Model tests were requested to determine the action of the pool with four of the newly developed hollow-jet valves. Because of the larger diameter of these valves, the upper part of the apron would have to be removed to allow passage of the jets. The model was built accordingly and the hollow-jet valves installed. The apron was too long, thus protruding up into the jets and spreading them too soon. The result was excessive splashing and turbulence on the apron. The rough condition continued on down into the canal with practically no jump being formed. The entire apron was removed. A trajectory was computed for the hollow-jet valves and a new apron built as shown by the dot-dash lines on Figure 10. The equation of the apron shape was $X^2 = -456.0Y$.

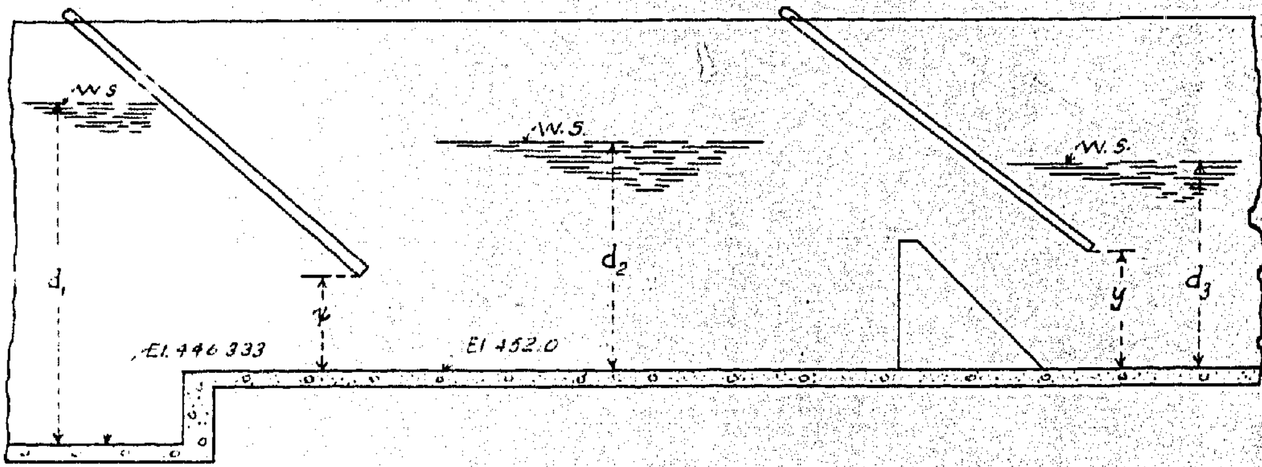
Conditions in the pool were excellent for all flows at all heads for balanced operation of the four valves. For unbalanced operation, the discharges should be decreased, unless the valves are discharging under low heads.

The capacity of the stilling pool and canal as originally designed was 3,500 second-feet, but they functioned satisfactorily with 3,600 second-feet, for balanced operation of the four valves. The canal depth downstream from the sill was 14.65 feet, or the water surface elevation was 466.65.

During the test program in 1944 it was learned that consideration was being given to increasing the capacity of the structure and canal to 4,500 second-feet. Tests showed that the pool would need to be deeper, so the floor was lowered three feet and the apron extended to the new floor level as shown by the dashed lines on Figure 10. The pool action for 4,500 second-feet was comparable to that for 3,500 second-feet, except that a maximum tailwater elevation became an important factor. With the canal water surface above a certain point, the tailwater interferred with the jets, causing considerable splashing and turbulence. The maximum capacity was 4,900 second-feet for balanced operation of the four valves. With the canal water surface at elevation 466.65, a depth of 14.65 feet, for a discharge of 4,500 second-feet under minimum head, the tailwater flooded the valves. The design section had specified a canal depth of 14.65 feet for a discharge of 4,500 second-feet, but it could not be maintained for this operating condition. The dashed lines on Figure 10 indicate the stilling pool design as recommended from this model.

<u>The 1:32 Model of 1945.</u> This model was constructed to investigate the adequacy of the stilling pool and canal for a discharge of 5,000 second-feet, the effect of turning each of the two center valves out one degree farther, and the possibility of damping the waves in the canal. The apron was lengthened and the pool floor lowered to elevation 446.333. The top of the sill at the end of the pool was placed even with the bottom of the canal at elevation 452.0. The side walls were made straight from the downstream ends of the piers to the sill at the end of the pool. They were also raised to elevation 481.67. A training wall 30 inches thick was placed in the center of the pool. The pool worked very well for all flows, when operating symmetrically. For unsymmetrical operation the pool became rough. The change in alinement of the two center valves had no adverse results. There was sufficient clearance between the jets and the center pier.

The wave action in the canal was severe for balanced operation and was worse for unbalanced operation. Waves occasionally slopped over the model walls for a discharge of 5,000 second-feet. Several combinations of stationary baffles and dentates were used in an attempt to quiet the waves in the canal. All wave studies were made with maximum reservoir elevation. Any arrangement giving desirable results for the higher flows had little or no control over the waves at lower flows. If the baffles were set low enough to dampen the waves of the lower flows, the waves of the higher flows were not controlled. When sufficient baffling was installed to obtain a desirable amount of damping of the waves at all flows, the depth of water in the pool became excessive. The result was flooding of the valves, violent turbulence in the pool, and splashing over the sidewalls. The only workable solution found was one set of dentates and two adjustable surface baffles located as shown on Figure 11. These baffles must be rigid with the upstream edges hinged or pinned and the elevation of the downstream edges adjustable. This arrangement did not eliminate the waves completely, but was the minimum amount of baffling required to obtain an appreciable reduction in wave action for three valves discharging under maximum head. A floating baffle was not satisfactory. The impact of the waves caused it to bob up and down, a movement which periodically accentuated the waves downstream. Square-edged baffles were used. The sharp edge biting into the surface velocities was an important damping factor. Rounding the bottom edge would have been of little or no value since the baffle would have to be lowered more to reduce the area of flow under the baffle to obtain the same results as with the sharp edged baffle. Table I gives the distances of the downstream edges from the floor for best results as obtained on the model. It also gives the depths of water upstream and downstream from each baffle. Refer to Sketch I for explanation of table.



SKETCH I

TABLE I

Q Sec.-ft.	Two baffles operating					Upstream baffle operating			Downstream baffle operating		
	d_1	d_2	d_3	x	y	d_1	d_2	x	d_2	d_3	y
5,000	28.00'	19.00'	17.20'	9.16'	8.84'	28.00	17.20	6.97'	18.66'	17.20'	10.66'
4,000	26.68	17.00	15.22	5.50	8.00	25.33	15.22	6.66	17.33	15.22	6.33
3,000	23.33	14.30	12.98	5.34	5.66	22.67	12.98	5.34	14.00	12.98	5.66
2,000	18.00	11.50	10.33	5.00	4.33	16.66	10.33	5.00	11.00	10.33	4.33

Table II shows the maximum splash up the slope at station 6+74.0 for various discharges and combinations of the surface baffles operating. The distances are from crest to trough measured along the slope. The height of splash up the slope from mean tailwater elevation may be obtained by dividing the values in the table by two and adding along the slope above mean tailwater.

TABLE II

Q Second-feet	Number of valves operating	Height of waves on slope from crest to trough: With no baffles operating	Upstream baffle operating	Downstream baffle operating	Both baffles operating
5,000	4	11.16 ft.	3.50 ft.	5.50 ft.	2.17 ft.
5,000	3	12.17	6.50	5.17	2.83
4,000	3	12.17	5.00	5.83	2.83
4,000	4	8.66	3.00	3.33	2.67
3,000	4	8.17	2.67	3.00	1.83
3,000	3	9.83	4.83	4.67	2.33

Pool operation for various discharges with and without the baffles is shown in the photographs of Figures 32, 33, and 34. Several oscillograms were taken of the vertical wave action in the center of the trapezoidal channel. Representative sections of these oscillograms are shown in Figures 35, 36, and 37. They cannot be compared visually but must be compared by the dimensions on the records. The dimensions are for the maximum range from crest to trough only.

There was considerable spray and splashing in the pool, as may be seen from Figures 38 and 39. These photographs were taken after the model had run for a period equivalent to one hour on the prototype.

A subsequent proposal submitted by the design department for damping the waves in the canal is shown and discussed in Appendix B. This proposal was not tested in the model.

The 1:16 sectional model of one outlet. In connection with the above studies on the 1:32 model, it was requested that a 1:16 scale model of one of the center outlets be installed with the six-inch hollow-jet valve to check the clearance between the jet and the center pier. The design department had objected to cutting all of the concrete between the piers down to elevation 457.25, because it would weaken the piers. They proposed to leave footings on each side of the piers at a distance of 5 feet 10½ inches from the centerlines of the valves. The footings were incorporated in this model to see if the jets would clear them.

There were 8 inches clearance, prototype, between the jet and the center pier. The jet impinged on the footings on each side and the stop-log grooves in them. Conditions were worse for lower reservoir elevations since the diameter of the jet dropped down toward the floor. The downstream ends of the footings were cut back still farther. All stop-log grooves were relocated at the downstream ends of the piers, with the intentions of filling the old grooves. These changes are shown on Figure 41.

Conclusions:

a. In every test it was shown that the best pool performance was obtained with four valves operating at equal openings and that conditions were good for any discharge and head for operating the valves in this manner.

b. For any combination of valve operation, the discharges must be so proportioned as to give a uniform spread at the end of the apron to have a good jump. Concentration of flow on either side results in the formation of bad eddies in the pool instead of a jump.

c. Tube valves do not give satisfactory operation for small openings. The jet spreads out into a spray that causes violent agitation in the pool and splashing over the walls.

d. For operation with three hollow-jet valves, the valve in one bay should have twice the opening of the two valves in the other bay for best conditions in the stilling pool.

e. The training wall in the center of the pool, similar to the one in the river outlet pool, improved unbalanced operating conditions considerably and made the pool operation more flexible.

f. The outlet pool as designed for 3,500 second-feet would pass 3,600 second-feet satisfactorily with four hollow-jet valves operating at approximately equal openings.

g. The outlet pool as designed for 4,500 second-feet with the tailwater at elevation 466.65 could not be operated in this manner at low heads. The tailwater came back on the apron and flooded the valves.

h. The present elevation of the walls of the trapezoidal section is 468.87. Unless the waves are dampened they will splash over the walls at discharges of 4,000 second-feet and more. Even with two baffles in operation the waves are approximately one foot over the present elevation of the walls for a discharge of 5,000 second-feet. For no waves at all, there is only 0.30 of a foot freeboard above the design tailwater elevation for 5,000 second-feet. For discharges of 4,000 second-feet and more, the waves should be dampened or else more freeboard added to the canal walls.

i. If the surface baffles are lowered too far, the valves will be flooded and a violent disturbance created in the pool. These conditions were taken into consideration when obtaining the values of x and y in Table II.

j. The baffles must be rigid with the elevation of the downstream edges adjustable to cover the varying depths of tailwater. Stationary baffles cannot be used successfully for all flow conditions.

k. If the surface baffles are not used, there will be no control on the canal for six miles downstream. It is possible that the value of n in the canal downstream will not be as high as 0.014. If the value of n should be too low, the tailwater is likely to sweep out. Such a condition would necessitate installation of some kind of control in the canal.

l. There will be considerable spray rising from the stilling pool at flows of 3,500 second-feet and higher as may be seen from Figures 38 and 39. The sidewalls at elevation 481.67 do not retain this spray. It is recommended that the road and berms on each side of the stilling pool be paved from the valve house to the downstream end of the stilling pool and that drains be provided for carrying away the water resulting from the spray.

<u>Development of the Friant-Madera Canal Outlet Stilling Pool and Apron.</u>

<u>The 1:28.44 model of 1939.</u> The design as submitted for testing is shown on Figure 12. The model indicated the hydraulic features of the structure to be satisfactory except for an unstable jump at some discharges. The pool and apron were shortened and a vertical sill installed at the

lower end of the pool. The canal and pool floors were raised. The pool action was very good. The design as changed was accepted and is shown on Figure 12.

Conclusions.

a. With both valves operating, satisfactory results may be had for any discharge and head.

b. One valve operating under a high head is not acceptable. It is acceptable when operating under a low head since the jet will spread and form a jump in the pool.

Development of the Friant Dam Spillway Crest and Stilling Pool; River Outlets and Stilling Pool; and the Bulkheads in the River Outlet Trashracks.

The 1:25 model of the spillway crest. In 1936 the Bureau of Reclamation Hydraulic Laboratory at Fort Collins was authorized to conduct model studies on the ogee-type overflow spillway crest at Friant Dam. The model as constructed represented a 50-foot section of the spillway crest, or one-half of one of the three 100-by 18-foot drum gates used to control the flow over the crest. These gates, which recede into the crest and take the form of the ogee section when completely lowered, are separated by piers. The pressure-distribution curves obtained for various gate positions and maximum reservoir elevation disclosed positive pressures on the crest for all gate positions, Figure 13. The pressure data was submitted to the design department for making a diagram of the hinge-pin reactions.

The crest was calibrated for various positions of the drum gates and the data plotted into a family of coefficient curves, Figure 14A. These discharge coefficients were for the crest without piers. As the contractions for piers of the type used on Friant Dam decrease the capacity approximately one and one-half percent at maximum discharge, a correction based on the pier tests for Grand Coulee Dam was applied in preparing the set of coefficient curves shown on Figure 14D. Corrections were also applied in determining the discharge diagram of Figure 14B.

Conclusions.

a. The pressure distribution obtained from piezometers installed in the crest and gate should have been of considerable assistance in preparing waterload curves required for the diagrams of the hinge-pin reactions.

b. There will be no negative pressures on the crest profile for discharges up to and including the maximum design discharge of 90,000 second-feet.

c. The spillway crest will have adequate capacity for handling the design discharge without encroachment on the freeboard.

d. The correction used in connection with the pier contractions on the Friant Spillway Crest should give results within the limits of experimental error.

e. The discharge curves should prove helpful in determining the magnitude of floods passing the dam after its completion.

The 1:24 model of the spillway stilling pool. This was a sectional type model built in 1939 to develop a satisfactory stilling pool at the toe of the dam. Since the tailwater drowned the jump at the lower flows the 4:1 sloping apron was replaced by a 7:1 slope. This changed the radius of the curve connecting the 0.7:1 slope of the face of the dam and the apron slope from 60 to 100 feet, prototype. The horizontal section of the pool floor was lowered from elevation 285.5 to 282.5. The resulting pool operation was satisfactory. Several tests were made using different sills at the end of the pool to eliminate scouring of the streambed downstream from the apron. Scour patterns and water surface profiles were obtained for each sill used. A sill having a 3:1 slope on the upstream face was recommended. The proper length of the stilling pool was determined during these tests. Figure 48, Section D-D, and Figure 49A show the spillway stilling pool design as recommended by the laboratory, except for the 2:1 sloping sill which was actually used in the field. This sill will make very little if any difference in the operation of the stilling pool.

The 1:60 model of the spillway crest, river outlets and stilling pool. This model was an assembly of the spillway crest and stilling pool with the river outlets discharging into the spillway stilling pool. It was constructed to study the spacing and the spreading of the four river outlet jets on the spillway apron, and to check the results obtained on the previous 1:24 and 1:25 models.

The stilling pool action was excellent for all flows with only the spillway discharging, Figure 50. Scour patterns and water surface profiles were obtained for operation with both a 2:1 and a 3:1 sill at the downstream end of the pool. These were compared with the results obtained on the 1:24 model and found to be in very close agreement. The water surface profiles are shown on Figure 47. The 3:1 sill was again recommended, although there was very little difference in the effectiveness of the two. The data obtained in the calibration of the crest were in close agreement with those obtained on the 1:25 model.

The model was operated with only the four outlets discharging. Spreading of the jets was not sufficient to produce a uniform jump in the pool. Severe eddies developed in the pool and carried material back on the apron, Figure 51. The jets were isolated by training walls, but the results were unsatisfactory. The outlets were passed straight through the dam at elevation 358.0, but the resulting conditions in the stilling pool were entirely unsatisfactory. They were moved to the left of the spillway where they discharged into a separate stilling pool, Figure 5.

The abandonment of the plans for the outlets discharging into the spillway does not mean that such a plan should be forever condemned. With a stilling pool narrow enough to prevent the jets from creating a whirl at the sides of the pool, and the outlets spaced sufficiently close to prevent whirls forming between the jets, combined with proper tailwater elevations, such a plan could be made to work satisfactorily. This is evident from the fact that satisfactory operation was obtained on the 1:24 model where one outlet was isolated such that whirls could not form at the sides.

The 1:34.38 model of the river outlets and stilling pool. This model was constructed in 1939 to develop a satisfactory stilling pool, using two 105-inch needle and two 102-inch tube valves. The 105-inch needle valves were later replaced by 96-inch needle valves. The design as submitted for testing is shown on Figure 15. The flow conditions in the pool and over the end sill were not satisfactory. The pool length and depth were increased and the valves tipped at an angle of 10° with the horizontal. This arrangement was excellent for discharges of 12,000 second-feet and less. For higher flows, only fair conditions existed in the stilling pool, becoming worse with increase in flow. This design, Figure 16, was accepted as final since the maximum flow of 17,000 second-feet was not expected to occur very often.

The 1:32 model of the river outlets and stilling pool. This study was conducted in 1944 in connection with replacing the tube and needle valves with four 96-inch hollow-jet valves. Transitions were placed between the conduits and the valves to reduce their angle of tip from 10° to 7°. This reduced the amount of concrete excavation required for the installation of the hollow-jet valves. Using the same origin, a new parabolic apron having a flatter slope was extended to an intersection with the original apron at elevation 319.626. The equation of the new apron is $y = -0.0010802X^2 + 0.0466X - 3.82$ and is the lower one shown on Figure 17. The maximum depth of concrete to be removed for the new shape was 15 inches.

As in previous tests, desirable pool conditions ended for a discharge of approximately twelve thousand second-feet. Various flow conditions are shown in Figures 59, 60, and 61. Although the design of Figure 17 was not entirely satisfactory for the higher flows it was accepted as final since those flow conditions were not expected to occur very often.

Following completion of these tests a new transition design, Detail 1, Figure 17, was submitted to the laboratory to be tested for pressures. This design changed the angle of tip of the valves from 7° to 6°. Such a change was not considered to affect the stilling pool enough to warrant further studies thereon. Tests showed that there were no negative pressures in the transition and that the pressures were only slightly reduced on the upper part of the valve body. The coefficient of the valve was not changed. The design of the transition as submitted for testing was recommended by the laboratory.

Conclusions.

a. All tests with any kind of valve have shown the pool to be under-designed for the higher flows. The maximum capacity was approximately sixteen thousand second-feet for operation with the hollow-jet valves. Due to the angle of tip of these valves, the jets were not spread on the apron for operation at high heads and discharges. At higher flows the jets persisted through the pool and struck the vertical end sill. The result was a violent boil at the end of the pool and a sharp increase in velocity as the flow passed over the sill.

b. The training wall down the center of the pool was a definite advantage. It helped stabilize the jump for unbalanced operation of the valves and made possible some increase in flow for unbalanced operating conditions.

c. When operating one or two valves with the corresponding tailwater elevation, the discharge per valve was limited to a value lower than that which could be passed when four valves were operating. When the discharge per valve was high, and only one or two valves were in operation, the energy of the jets was disproportionately high to the corresponding tailwater elevation. Consequently, an insufficient amount of energy was dissipated in the pool to maintain a hydraulic jump. This condition was amplified with two valves discharging into one side of the pool. For operation with two valves, the most satisfactory conditions were obtained with one valve discharging into each side of the pool. The combined discharge should be limited to 5,000 second-feet for any combination of two valves operating, and to 2,000 second-feet with only one valve operating.

further studies thereon. Tests showed that there were no negative pressures in the transition and that the pressures were only slightly reduced on the upper part of the valve body. The coefficient of the valve was not changed. The design of the transition as submitted for testing was recommended by the laboratory.

Conclusions.

a. All tests with any kind of valve have shown the pool to be under-designed for the higher flows. The maximum capacity was approximately sixteen thousand second-feet for operation with the hollow-jet valves. Due to the angle of tip of these valves, the jets were not spread on the apron for operation at high heads and discharges. At higher flows the jets persisted through the pool and struck the vertical end sill. The result was a violent boil at the end of the pool and a sharp increase in velocity as the flow passed over the sill.

b. The training wall down the center of the pool was a definite advantage. It helped stabilize the jump for unbalanced operation of the valves and made possible some increase in flow for unbalanced operating conditions.

c. When operating one or two valves with the corresponding tailwater elevation, the discharge per valve was limited to a value lower than that which could be passed when four valves were operating. When the discharge per valve was high, and only one or two valves were in operation, the energy of the jets was disproportionately high to the corresponding tailwater elevation. Consequently, an insufficient amount of energy was dissipated in the pool to maintain a hydraulic jump. This condition was amplified with two valves discharging into one side of the pool. For operation with two valves, the most satisfactory conditions were obtained with one valve discharging into each side of the pool. The combined discharge should be limited to 5,000 second-feet for any combination of two valves operating, and to 2,000 second-feet with only one valve operating.

The 1:18.33 model of the temporary bulkheads in the river outlet trashracks. A 1:18.33 scale model of one of the Friant River Outlets was constructed with the bulkheads in the trashrack, as they were to be during the construction period. Since each trashrack served two outlets, a wall dividing the trashracks was necessary to account for the effect of an adjacent outlet. Previously, a mathamatical analysis had been made to determine suitable exit diameters for temporary nozzles to be placed on the river outlets. These nozzles were to maintain pressures in the conduits before the control valves were installed. The outlets would be needed during construction of the dam to supplement flow through the diversion tunnels. This analysis appears in Appendix A at the end of this report.

A nozzle having approximately the same exit diameter as developed in the analysis mentioned above was installed on the end of the conduit in this model. The model design is shown on Figure 18.

Several tests were made with the bulkheads in place. Pressures were measured for several water surface elevations by piezometers located on the top and bottom of the bell-mouth entrance. In each test the effect of the air vent upon the pressures was studied. The pressures on the top of the outlet were lower than those on the bottom. The maximum negative pressure of 9 feet prototype occurred with the water surface at elevation 396.0 or 16 feet above the conduit centerline. With the air vent closed, a negative pressure of 16 feet was recorded for a water surface elevation of 389.0.

The removal of the bulkheads decreased the pressures on the bottom of the conduit three to five feet of water but increased the pressures on the top by only one-half to one foot. Conversely, the pressures on the bottom of the conduit were increased with the bulkheads in position, while the pressures on the top were decreased only slightly. Therefore, it can be stated that the bulkheads had no important effect upon the pressures in the conduit. The pressure diagrams are shown on Figure 18.

GENERAL

Description of Project and the Models.

Location. Friant Dam is located on the upper San Joaquin River about twenty miles north of Fresno and 21 miles east of Madera, California, Figure 1. When completed, it will be approximately three hundred feet high and three thousand four hundred and thirty feet long. The reservoir created by this dam will have a gross storage capacity of 520,550 acre-feet which includes a flood-control storage of 70,000 acre-feet, and an active canal storage of 316,500 acre-feet.

Friant Dam will play a large part in the Central Valley Reclamation Project in strengthening flood control and in preventing the loss of thousands of acres of highly productive lands in California because of the invasion of salt water in the Sacramento-San Joaquin Delta and the falling water tables in the San Joaquin Valley. It will be possible to reclaim many acres of highly developed land which have been suffering from drouth.

The San Joaquin River waters impounded in the Millerton Reservoir behind Friant Dam will be distributed for irrigation throughout the San Joaquin Valley chiefly by two large canals. The Friant-Kern Canal will extend 160 miles to the south of Friant Dam to the Kern River west of Bakersfield. The Friant-Madera Canal will extend approximately thirty miles northwest of Friant Dam to the Chowchilla River.

The spillway section, located near the center of the dam, was designed to pass the flood waters coming into Millerton Reservoir. The maximum design capacity of the ogee-type overflow crest is 90,000 second-feet. The flow over the spillway will be regulated by three 100-by 18-foot drum gates which recede into the crest and take the form of the ogee section when completely lowered. The arrangement of all of the outlets and the spillway as of 1936, prior to the beginning of hydraulic model studies, is shown on Figure 2.

The Friant-Kern Canal Outlets. Four model tests have been made on these outlets over a period of several years. The first, in 1936, was a 1:34.29 scale model to determine the best alinement of the needle valves as they entered the stilling pool, and to develop a satisfactory stilling pool. These tests were made on the basis of four 96-inch needle valves with a maximum discharge of 3,500 second-feet and a maximum head of 98 feet.

Later, the maximum reservoir surface was increased from elevation 564.0 to 578.0 and the centerline of the valves lowered from elevation 466.0 to 464.0. This resulted in a maximum head of 114.0 feet. Consideration was given to replacing two of the 96-inch needle valves with two 102-inch tube valves. These changes justified the construction of a 1:34.38 scale model in 1939, using two 96-inch needle valves and two 102-inch tube valves to determine the adequacy of the apron and stilling pool design developed in 1936.

Plans were completed for installation of the tube and needle valves but the war delayed construction. In the meantime, the new hollow-jet valves were developed in the Bureau Design Office. A study was made to determine the relative costs of fabrication and installation of the three types of valves. A saving in initial costs of approximately one hundred seventy-three thousand dollars could be realized by installing four 96-inch hollow-jet valves in place of the tube and needle valves. In addition, the combined weight of the four hollow-jet valves would be approximately seventy-six tons less than that of two each of the tube and needle valves. An itemized account of the study appears in Table III.

The hydraulic characteristics of the hollow-jet valve are described in Hydraulic Laboratory Report No. 148, by Fred Locker. Due to the larger diameter of the hollow-jet valves, part of the apron at the upper end of the stilling pool will have to be removed. In 1944 a 1:32 scale model was constructed to determine the performance of the stilling pool for operation with hollow-jet valves. Tests were also made to determine the feasibility of increasing the maximum discharge to 4,500 second-feet.

The February 15, 1945, monthly report of urgent work and status of active projects contained the following statement: "We have tentatively concluded, based on both the independent and coordinated studies now

TABLE III

Sheet 1

FRIANT DAM
NEEDLE VALVES

12-12-44
B. E. Staats.

ITEM NO.	ITEM	QUANTITY AMOUNT	UNIT	MATERIAL AND LABOR FURNISHED BY THE CONTRACTOR UNIT COST	TOTAL COST	MATERIAL FURNISHED BY THE GOVERNMENT UNIT COST	TOTAL COST	SUMMARY UNIT COST	TOTAL COST
	Jan. 1. 1945 prices.								
	RIVER OUTLETS								
1	4-110" x 105" Needle valves	880,000	lbs	.04	35,200	.415	365,000		400,200
2	Additional needle valve control piping	9,100	lbs	.11	1,000	.175	1,590		2,590
3	Four sets of anchor brackets and bolts	23,000	lbs	.04	920	.20	4,600		5,520
4	Cost of changes in concrete structure and piers.								
	(See Tabor's estimate for item 4)								6,400
									414,710
								Say	415,000
	FRIANT-KERN CANAL								
1	4-110" x 105" Needle valves	844,000	lbs	.04	33,760	.415	350,260		384,020
2	Additional needle valve control piping	7,440	lbs	.11	820	.174	1,290		2,110
3	Four sets of anchor brackets and bolts	23,000	lbs	.04	920	.20	4,600		5,520
4	Cost of changes in concrete structure and piers.								
	(See Tabor's estimate for item 4)								3,700
									395,350
								Say	396,000
							Total		811,000
	RIVER OUTLETS								
1	2-110" x 105" Needle valves	440,000	lbs	.04	17,600	.445	195,800		213,400
2	2-110" x 102" Tube valves	306,000	lbs	.04	12,240	.445	136,170		148,410
3	Four sets of anchor brackets and bolts	23,000	lbs	.04	920	.20	4,600		5,520
4	Cost of concrete changes in piers								
	(See Tabor's estimate for item 4)								6,400
									373,730
								Say	374,000
	FRIANT-KERN CANAL								
1	2-110" x 105" Needle valves	422,000	lbs	.04	16,880	.445	187,790		204,670
2	2-110" x 102" Tube valves	280,000	lbs	.04	11,200	.445	124,600		135,800
3	Four sets of anchor brackets and bolts	23,000	lbs	.04	920	.20	4,600		5,520
									345,990
								Say	346,000
							Total		720,000
	RIVER OUTLETS.								
1	4-96" Hollow jet valves	500,000	lbs	.04	20,000	.42	210,000		230,000
2	4-110" x 96" Adapters (Gray iron castings)	80,000	lbs	.04	3,200	.20	16,000		19,200
3	Pressure, vent and drain piping	10,000	lbs	.11	1,100		Purchased		1,100
4	Four sets of anchor brackets and bolts	25,000	lbs	.04	1,000	.25	6,250		7,250
5	Cost of changes in concrete structure								
	(See Tabor's estimate for item 5)								9,200
									266,750
								Say	267,000
	FRIANT-KERN CANAL								
1	4-96" Hollow jet valves	500,000	lbs	.04	20,000	.42	210,000		230,000
2	4-110" x 96" Adapters (steel plate)	40,000	lbs	.04	1,600	.25	10,000		11,600
3	Pressure vent and drain piping	8,000	lbs	.11	880		Purchased		880
4	Four sets of anchor brackets and bolts	25,000	lbs	.04	1,000	.25	6,250		7,250
5	Cost of changes in concrete structure								
	(See Tabor's estimate for item 5)								30,100
									279,830
								Say	280,000
	NOTE: 4-Hollow jet valves for River outlets exactly the same for Friant-Kern Canal outlets. All 8 valves built from the same patterns and jigs.						Total		547,000

nearing completion, that the canal should be constructed to the Kaweah River with an initial capacity of 4,000 second-feet and with necessary provisions made for its future enlargement to 5,000 second-feet." The tailwater elevations on the curve resulting from the above provisions for corresponding canal flows of 4,000 and 4,500 second-feet were very near the point at which only a fair jump was formed in the design for 4,500 second-feet. These same elevations were very near the "sweepout" point for the design based on a maximum flow of 3,500 second-feet. For a canal flow of 5,000 second-feet, the corresponding tailwater elevation was above the point at which a passable hydraulic jump was formed in the design based on a maximum flow of 4,500 second-feet. The same combination was at the "sweepout" point of the design based on a maximum canal flow of 3,500 second-feet.

Studies on a 24-inch hollow-jet valve under high heads at Boulder Dam showed that the jet expanded slightly immediately after leaving the valve. It appeared that the jets from the two center valves on the Friant-Kern Canal would impinge on the stop-log grooves of the center pier. For this reason the two center valves were each turned out one degree more with respect to the centerline of the outlet structure, to allow the jets to miss the center pier.

Photographs came from Friant Dam showing the severe wave action on the Madera Canal, Figure 4, along with a request that studies be made relative to damping the waves on the Friant-Kern Canal. The 1:32 scale model was reconstructed to study the stilling pool design for 5,000 second-feet, the effect of resetting the two center valves, and the wave conditions in the canal.

In connection with these studies, it was requested that a 1:16 scale model of one of the center outlets be set up with the six-inch hollow-jet valve to check the clearance between the jets and the center pier. The design department had objected to cutting all of the concrete between the piers down to elevation 457.25, because it would weaken the piers. They proposed to leave footings on each side of the piers at a distance of

5 feet $10\frac{1}{2}$ inches from the centerlines of the valves. The footings were incorporated in this model to determine the clearance around the jets.

The Friant-Madera Canal Outlets. In 1939 the hydraulic laboratory was requested to conduct model tests of these outlets to determine the adequacy of the apron and stilling pool design. The pool was designed to pass a maximum flow of 1,500 second-feet discharging from two 78-inch needle valves under a maximum head of 132 feet.

The Friant Dam Spillway and River Outlets. The first model to be tested was a 1:25 scale model constructed in the Bureau of Reclamation Hydraulic Laboratory at Fort Collins in 1936. This model represented a 50-foot section of the crest or one-half of one gate. It was necessary to obtain the water load curves for the gate in various positions. These were needed by the design section to make a diagram of the hinge-pin reactions. The efficiency of the crest was to be determined for various gate positions and used as a measure of the spillway capacity. The presence of negative pressures on the crest and gate was investigated for various gate elevations, including the completely lowered position.

In 1939, a second model was built to a 1:24 scale in the Bureau Laboratory in Denver to design a satisfactory hydraulic jump stilling pool at the toe of the dam. This was a sectional model representing a 90-foot section of crest equally divided on each side of a 12-foot pier. The crest was built to the correct shape, but since the stilling pool was the only concern, the gates were omitted.

Original plans were to have four 102-inch outlets in the spillway of the dam, similar to those in Shasta Dam. These outlets were first designed to be controlled by ring-follower gates near the upstream face of the dam. Tube valves were later installed on the downstream ends of the conduits to replace the ring-follower gates. Following the development of a satisfactory stilling pool design on the 1:24 scale sectional model of the spillway, one of the 102-inch river outlets was installed in the spillway of that model. The development of the transition and "beavertail" on this outlet and the results of the tests thereon may be found in Hydraulic Laboratory Report No. 69, by H. G. Dewey, Jr., December 28, 1939.

A 1:60 scale model was constructed of the entire spillway with the stilling pool and outlets as developed on the 1:24 scale model. The items studied on this model were the performance of the stilling pool with only the outlets operating; the performance of the stilling pool with only the spillway operating; the performance of the pool with both the spillway and outlets operating; and the calibration of the crest as a check on the calibration made on the 1:25 model in Fort Collins.

These tests showed that flow conditions were very unsatisfactory with the river outlets discharging into the spillway. They were moved to the left of the spillway where they discharged into a separate stilling pool, Figure 5. Model tests were then conducted to develop a stilling pool for the new arrangement, which embodied two 102-inch tube and two 105-inch needle valves operating under a maximum head of 258 feet.

Since construction on the river outlets was also delayed, consideration was given to replacing the tube and needle valves with hollow-jet valves as in the case of the Friant-Kern Canal Valves. Table III shows an itemized account of the savings which would be made by making such a replacement. In 1944, model tests were conducted to determine the pool changes that would be necessary for installation of the hollow-jet valves. The upper part of the apron would need to be removed to allow passage of the jets from the larger diameter valves.

Temporary bulkheads in the river outlet trashracks. During construction of the dam, bulkheads were placed in the lower bays of the river outlet trashracks to hold the rising water until the coaster gate tracks and seal seats on the face of the dam could be installed. It was undesirable to remove these bulkheads immediately following the installation of the tracks and seal seats, but preferable to allow the water to rise above them and discharge through the outlets.

The effect of the bulkheads on the pressures in the outlets was uncertain, so hydraulic model studies were utilized in observing the behavior of the outlet and to measure the pressures in the entrance for various water surface elevations.

MODEL TESTS

Tests on the Friant-Kern Canal Outlets.

The 1:34.29 model of 1936. In 1936 the hydraulic laboratory was authorized to make model tests on the outlet design as shown on Figure 3 and to develop a satisfactory apron and stilling pool design. A 1:34.29 scale ratio was adopted so that available 2.80-inch needle valves might be used. The original apron design was never constructed since it was considered to be fundamentally wrong. Instead, a coefficient of velocity of 0.96 was assumed for the vena contracta of the valve and a velocity computed, based upon the maximum head of 98.0 feet. Conduit and trash-rack losses were neglected. A trajectory was computed from the velocity and a parabolic apron developed. Computations were made with a step at the downstream end of the hydraulic jump pool. A minimum difference of 6 inches between the pool and canal floor elevations was recommended for the prototype. Because of successive changes from the design section, only observation runs were made on this model before the design was abandoned. The centerline of the valves had been moved to elevation 464.0, the canal and pool widths were changed, and the pool floor elevation changed to elevation 450.12. The length of the stilling pool was limited to 175.0 feet.

The stilling pool apron was designed to spread the jets sufficiently to form a sheet of water of uniform depth at the toe of the apron. Computing the combined jet area of the four valves and dividing by the pool width of two feet, model, a depth of 1.05 inches was needed at the toe of the apron for the uniformly spread jet. Such a condition was obtained by setting up a definite relation between the jet trajectory and the slope of the apron. A measurement of the trajectory of a model valve was made. From this the parabolic equation of the centerline of the trajectory, $y = -0.00683X^2$, model inches, was computed. The depth of 1.05 inches was measured down from the upper boundary of the jet and the trajectory equation plotted parallel to the upper boundary. By taking the point of intersection of the trajectory equation with elevation 450.12 as the toe of the apron, the depth of 1.05 inches was obtained. The point of intersection of the trajectory equation with elevation 450.12 was taken as the toe of the apron. Since some form of discharge guides were contemplated for the valves, a cone 3.05 inches long and 4.30 inches in exit diameter was sketched for the end of the valve. The lower point of this cone was taken as the origin of the apron. With the origin and the toe of the apron fixed, the apron equation, $y = -0.00451X^2$, model inches, was determined. An apron was constructed from the equation as developed above. A trapezoidal pool section was considered, but was never tested because tests on other installations had yielded adverse results. Instead, a straight rectangular pool 74.70 feet wide was built. The four needle valves were installed, each turned in 3 degrees, 42 minutes, and 40 seconds, toward the centerline of the pool.

A test was made with the two center valves discharging 3,500 second-feet under a 100-foot head and at approximately forty five percent of full-valve opening. The partially formed jump was unstable, moving from side to side with change in tailwater elevation. A bad fin formed on the apron between the jets. The jump could be stabilized by turning the valves out at about two degrees with the centerline of the pool. The jets were not spread soon enough on the apron and the pool appeared to be too shallow.

A second test was made with the same settings as before, except that the two outside valves were used. The jump formed much better in this arrangement, but was accompanied by an intolerable cross surge in the pool. This was practically eliminated by turning the valves toward the centerline about six degrees.

A third test was made with all four valves operating at equal openings of about forty-three percent of full opening. They were discharging 3,500 second-feet under a 25-foot head. The pool action was satisfactory for any valve alinement, indicating that any arrangement which was satisfactory for other operating conditions would suffice here.

The above sequence of runs was repeated several times using modified aprons, various valve alinements, and various spacing of the valves with respect to the centerline of the pool before acceptable results were obtained. The center valves were turned out a maximum of six degrees and the outside valves turned in a maximum of nine degrees during these tests. The most satisfactory conditions were with the inside valves directed out one degree and the outside valves directed in seven degrees. The width of the pool was reduced to 50.0 feet at a point 91.0 feet downstream from the outside edge of the valve chamber, making a rectangular pool and canal 50 feet wide. Figure 6 shows the arrangement as accepted for the final design.

Two outside or two inside valves operating under maximum head and a maximum discharge of 3,335 second-feet gave acceptable pool conditions. Pool action was also acceptable for the same valves operating under minimum head. The maximum discharge for minimum head was 3,300 second-feet. One valve discharging 3,450 second-feet under maximum head was not desirable. Figures 19 and 20 show the model operating for other conditions.

Several types of discharge cones were tried on the final design of the model. In addition, fillets were placed on the apron and up into the cones. It was found that better performance resulted when there were no fillets in the cones or on the apron. Since no definite information could be obtained from the design division regarding the design and use of discharge cones, nothing definite was developed in the laboratory. The ones shown on Figure 6 worked satisfactorily on the model.

<u>The 1:34.38 model of 1939.</u> Following the tests in 1936 the maximum reservoir water surface was increased to elevation 578.0 to provide for flood control. The two center valves were to be replaced by two 102-inch tube valves. In addition, the design section had made a few minor changes in the laboratory design of 1936. The angle of the two center valves was changed from one degree to 0° 30'; the apron length was increased from 82.15 feet to 89.50 feet; the elevation of the pool floor was changed from 450.12 to 450.10 and the width of the apron at the origin increased from 74.0 to 78.0 feet. These changes justified model tests, so in 1939

the hydraulic laboratory constructed a 1:34.38 scale model from the details of Figures 7 and 8 to determine the adequacy of the pool for the new conditions.

The design was satisfactory in every way except that the jump was unstable at higher discharges. A vertical sill 3.50 feet high was placed at the end of the jump as a stabilizer, and the canal floor at elevation 452.00 extended upstream to the sill. The centerline of the valves was raised to elevation 464.25 to allow the invert of the tube valves to become tangent to the apron at elevation 460.0. The canal water surface was set at elevation 466.65 by the design division for a discharge of 3,500 second-feet. Figures 9 and 21A show the design as recommended by the laboratory.

The valves were numbered from left to right, looking downstream. Unless otherwise stated, the valves were operated at equal discharges and maximum head for all runs. With four valves operating, pool conditions were good for discharges from 3,500 down to 1,600 second-feet. At lower flows the discharge characteristics of the tube valves became intolerable. Pool conditions were good for valves one and four discharging 3,500 down to about two thousand second-feet. Below these discharges the jets did not spread sufficiently to form a good jump nor to prevent water from coming back up the center of the apron. Pool conditions were good for valves two and three for flows down to about two thousand second-feet and could be accepted down to 1,500 second-feet. One center and two outside valves gave passable pool operation for flows down to 1,500 second-feet. At lower discharges the tube valve flow became bad and caused a whirl in the pool. Pool operation was improved at higher flows by allowing the center valve to pass more than one-third of the total discharge.

One center and one outside valve did not give satisfactory pool operation at any discharge. Valves one, two and three gave good results for all flows down to 1,200 second-feet if valve two passed more than one-third of the total flow. Valves one and three were acceptable down to 800 second-feet. Below this discharge the tube valve flow became unsatisfactory.

For any combination of valve operation, the discharges must be so proportioned as to give a uniform spread at the end of the apron to have a good jump. Concentration of flow on either side results in bad eddies in the pool instead of a jump. Figures 21B and C and 22 show the model operating for several valve combinations. Figure 23 is the discharge curve used in setting the tailwater on the model.

The 1:32 model of 1944. In January 1944, the hydraulic laboratory constructed a 1:32 scale model of the Kern Canal Outlets with four 96-inch hollow-jet valves. Since the diameter of these valves was greater than that of either the tube or needle valves, troughs would have to be chipped out of the upper part of the apron in the prototype to allow passage of the jets. Instead of building the troughs in the model, the entire parabolic apron was cut off level at elevation 457.0, as shown by the solid lines on Figure 10.

Only observation runs were made on this arrangement since the apron was too long and extended too far up into the jets. The result was excessive splash and turbulence that extended from the apron down into the canal. A jump did not form in the pool. A trajectory equation was computed, based upon the maximum head of 114.0 feet and a coefficient of velocity of 0.98 for the valve. Taking the origin at the invert of the valves, an apron was built from this equation. The upper portion of the apron was then cut off horizontally at elevation 457.0, as shown by the dot-dash line on Figure 10 and pictorially on Figure 24A.

The valves were operated at equal openings at all times. Several runs were made with four valves operating at various discharges under maximum head. For each run the tailwater sweep-out elevation and the tailwater elevation giving the best jump in the pool were determined. These are shown on Figure 25 along with the design tailwater elevations.

Flow conditions were very good with four valves operating at any discharge under any head, Figure 24B and C. The flow in the pool was very rough with three valves discharging 3,500 second-feet and became worse at higher heads. Conditions were better for two outside valves and one inside valve operating than for two inside valves and one outside valve operating, Figure 26A. These conditions might be tolerated for short periods of time. Either combination of three valves operating at low heads was satisfactory.

Two center valves discharging 3,500 second-feet under maximum head resulted in a rough and unstable pool accompanied with considerable splashing over the walls, Figure 26B. Pool operation was satisfactory for this discharge under low heads. For two outside valves discharging 3,500 second-feet the pool was rough, accompanied by splashing and a cross surge. Water ran back on the apron between the jets, and at low heads water splashed over the walls before the jets left the apron. Two valves operating on one side caused a whirl in the pool instead of a jump. Water ran back on the apron adjacent to the discharging valves, thereby interfering with the jet of the center valve.

One valve operating wide open under maximum head was definitely unsatisfactory, since no jump formed in the pool. Low discharges at high heads might be tolerated for a short time. Low discharges at low heads would be acceptable for any one valve operating.

The jets issuing from the two center valves spread enough to catch on the ends of the piers. To prevent this, the construction of the valves was altered slightly. The tangent running back from the valve face on the inside of the valve body was replaced by continuing the curve out to the valve face. This reduced the areas of the jets sufficiently to permit them to pass the piers undisturbed.

Before the test program was completed, it was learned that the capacity of the structure might be increased to 4,500 second-feet. Previous tests indicated that the pool would have to be deeper. The floor was lowered three feet and the old apron extended to the new floor level as shown by the dashed lines on Figure 10. Specifications were to hold the tailwater elevation at 466.65 for a discharge of 4,500 second-feet.

Four valves discharging 4,500 second-feet under maximum head developed a very good jump in the pool, Figure 27A. When operating down in the low head range the pool action was also good, but the tailwater flooded the valves, Figure 27B. Operation in this manner would be undesirable.

Two outside valves and one center valve discharging 4,500 second-feet under maximum head resulted in a rough pool with some splashing, but it was not serious, Figure 28A. Two center valves and one outside valve operating under the same conditions produced a rougher pool, Figure 28B. Operation in this manner might be permitted for short periods.

The two center valves discharging 4,500 second-feet under maximum head produced a rough pool, but it was not considered to be serious enough to be objectionable. The pool conditions were worse for the two outside valves operating under the same conditions. As the head was decreased a cross surge developed in the pool and became serious at low heads. Two valves on one side discharging 4,500 second-feet under maximum head created a whirl in the pool instead of a jump. The pool was very rough and was accompanied by considerable splashing. Figure 29 shows the three methods of operation as just described.

On Figure 30 may be found the tailwater sweep-out elevations, the tailwater elevations giving the best jump in the pool, and the maximum allowable tailwater elevations. Above given elevations the tailwater would interfere with the jets and cause considerable turbulence and splashing on the apron. Figure 31 shows the minimum reservoir elevations for various discharges with four valves operating wide open.

<u>The 1:32 model of 1945.</u> From the February 15, 1945, monthly report of urgent work, it was learned that further changes in design were being made on the Friant-Kern Canal. The maximum flow was to be 5,000 second-feet and the bottom width of the canal was increased from 30 to 36 feet. The tailwater elevations from the new curve for corresponding canal flows of 4,000 and 4,500 second-feet were very near the point at which only a fair jump was formed in the design for 4,500 second-feet. These same elevations were very near the "sweepout" point for the design based on a maximum flow of 3,500 second-feet. For a canal flow of 5,000 second-feet, the corresponding tailwater elevation was above the point at which

a passable hydraulic jump was formed in the design based on a maximum flow of 4,500 second-feet. The same combination is at the "sweepout" point of the design based on a maximum canal flow of 3,500 second-feet.

Studies made on a 24-inch hollow-jet valve under high heads at Boulder Dam showed that the jet expanded slightly immediately after leaving the valve. It appeared that the jets from the two center valves on the Friant-Kern Canal would impinge on the stop-log grooves of the center pier. For this reason the two center valves were each turned out one degree more with the centerline of the outlet structure, to allow the jets to miss the center pier.

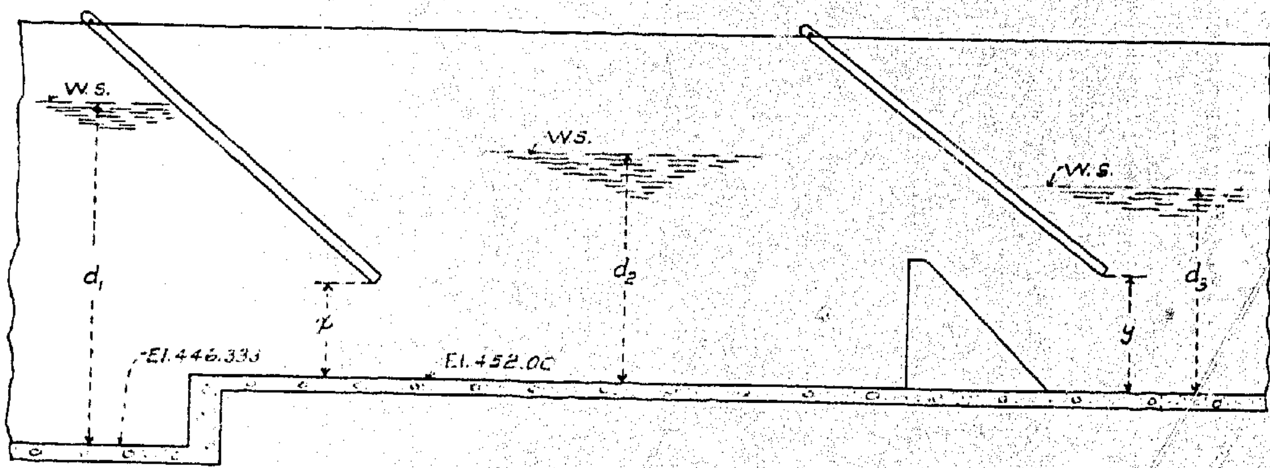
Photographs were received from Friant Dam showing the severe wave action on the Madera Canal, Figure 4, and also a request that some study be made relative to damping the waves on the Friant-Kern Canal. The above-mentioned problems resulted in the reconstruction of the 1:32 scale model.

Previous model tests indicated that the pool would work better for a discharge of 5,000 second-feet if the elevation of the pool floor was lowered and the apron lengthened. The side walls were straightened between stations 1+70.17 and 3+12.17 so that they come into the rectangular section at station 3+12.17 rather than at station 2+59.42 as before. They were also raised to elevation 481.667 to curtail some of the splash accompanying unsymmetrical operation at the higher discharges. Computations were made to determine the depth d_2 required in the pool for the maximum possible discharge of 2,885 second-feet through one valve. The length of the pool was taken as $5d_2$. The pool floor was placed at elevation 437.0 and the downstream end at station 3+88.97, where there was a 15-foot vertical sill extending to the canal elevation of 452.0. A trajectory equation was computed from the horizontal centerline of the valve face at elevation 464.0 to the pool floor at elevation 437.0. The trajectory was then shifted so as to place the origin at elevation 457.25 and station 1+11.416, which is the upstream face of the piers. The model was built and operated accordingly. The pool was too deep and too long. These results upset all the theory and computations of the hydraulic jump. The hydraulic jump equation is based upon uniform distribution of depth and energy in a uniform channel. This was a converging channel which constantly decreased in area in the lateral direction. The energy was concentrated in a smaller area than it should have been. The result was an increased depth over what would have formed normally, thereby making the pool deeper than necessary.

The apron did not extend into the jets far enough to spread them properly. It was lengthened until satisfactory spreading was obtained. The length and depth of the pool were gradually reduced until satisfactory operation was obtained with four valves discharging 5,000 second-feet at maximum reservoir elevation. The increased space between the two center valves resulting from turning each one out one degree more, allowed water to run back on the apron and cause a whirl between the two center valves when four valves were operating. A 30-inch training wall was placed in the center of the pool to eliminate the whirl and to improve conditions in the pool when operating unsymmetrically. There was plenty of clearance between the center pier and the jets from the two center valves. This was checked later on a 1:16 scale sectional model. The recommended apron and stilling pool design are shown on Figure 11.

For a discharge of 5,000 second-feet through three valves, the pool works better if the two valves in one bay have equal openings and the one valve in the other bay has twice their opening. The energy distribution is more even. This combination works best for reservoir elevations below 540.0. With three valves discharging 5,000 second-feet at equal openings, the tailwater in the bay with the two valves is swept out. The same combination of valve openings applies to a discharge of 4,000 second-feet through three valves for reservoir elevations above 535.0. Below a reservoir elevation of 535.0, 4,000 second-feet may be passed through three valves at equal openings, but the two-to-one ratio of valve openings is recommended. Three valves will discharge 3,000 second-feet satisfactorily at equal openings for reservoir elevations below 570.0. The above conditions were observed without the surface baffles which were later recommended for damping the waves in the canal. With these baffles in place, the possibilities of the tailwater sweeping out during unbalanced operation will be eliminated.

The wave action in the canal was severe for balanced operation and was worse for unbalanced operation. The top of the walls of the trapezoidal section on the model were at elevation 472.767 which was an increase of 4 feet 8.0 inches, prototype, above the elevation of the walls on the prototype design. Waves occasionally slopped over the model walls for a discharge of 5,000 second-feet. Several combinations of stationary baffles and dentates were used in an attempt to quiet the waves in the canal. All wave studies were made with maximum reservoir elevation. Any arrangement giving desirable results for the higher flows had little or no control over the waves at lower flows. If the baffles were set low enough to dampen the waves of the lower flows, the waves of the higher flows were not controlled. When sufficient baffling was installed to obtain a desirable amount of damping of the waves at all flows, the depth of water in the pool became excessive. The result was flooding of the valves, violent turbulence in the pool, and splashing over the sidewalls. The only workable solution found was one set of dentates and two adjustable surface baffles located as shown on Figure 11. These baffles must be rigid with the upstream edges hinged or pinned and the elevation of the downstream edges adjustable. This arrangement did not eliminate the waves completely. It was the minimum amount of baffling required to obtain an appreciable reduction in wave action for three valves discharging under maximum head. A floating baffle was not satisfactory. The impact of the waves caused it to bob up and down, a movement which periodically accentuated the waves downstream. Square-edged baffles were used. The sharp edge biting into the surface velocities was an important damping factor. Rounding the bottom edge would have been of little or no value since the baffle would have to be lowered more to reduce the area of flow under the baffle to obtain the same results as with the sharp-edged baffle. Table I gives the distances of the downstream edges from the floor for best results as obtained on the model. It also gives the depths of water upstream and downstream from each baffle. Refer to Sketch I for explanation of table.



SKETCH I

TABLE I

Q second-feet	Two baffles operating					Upstream baffle operating			Downstream baffle operating		
	d_1	d_2	d_3	x	y	d_1	d_2	x	d_2	d_3	y
5,000	28.0'	19.0'	17.2'	9.16'	8.64'	28.0'	17.2'	6.97'	18.66'	17.2'	10.66'
4,000	26.68	17.0	15.22	5.5	8.0	25.33	15.22	6.66	17.33	15.22	6.33
3,000	23.33	14.3	12.98	5.34	5.66	22.67	12.98	5.34	14.0	12.98	5.66
2,000	18.0	11.5	10.33	5.0	4.33	16.66	10.33	5.0	11.0	10.33	4.33

Table II shows the maximum splash up the slope at station 6+74.0 for various discharges and combinations of the surface baffles operating. The distances are from crest to trough measured along the slope. The height of splash up the slope from mean tailwater elevation may be obtained by dividing the values in the table by two and adding along the slope above mean tailwater.

TABLE II

Q Second-feet	Number of valves operating	Height of waves on slope from crest to trough: With no baffles operating	Upstream baffle operating	Downstream baffle operating	Both baffles operating
5,000	4	11.16 ft.	3.50 ft.	5.50 ft.	2.17 ft.
5,000	3	12.17	6.50	5.17	2.83
4,000	3	12.17	5.00	5.83	2.67
4,000	4	8.66	3.00	3.33	1.83
3,000	4	8.17	2.67	3.00	2.17
3,000	3	9.83	4.83	4.67	2.33

Pool operation with the baffles is shown in the photographs of Figures 32 and 33. A comparison of the wave action in the canal for discharges of 4,000 and 5,000 second-feet with and without baffles is shown in the photographs of Figure 34.

An oscillograph was installed to record the wave heights for various conditions of flow. Part of the equipment is shown in Figure 34B at the end of the pencil. A detailed description of the equipment and method used is in Hydraulic Laboratory Report No. 205, by C. R. Daum. Several oscillograms were taken of the vertical wave action in the center of the trapezoidal channel. Each record was run for one minute, which was equivalent to 5.6 minutes on the prototype. A representative section containing a maximum wave height was taken from each record for reproduction. These records are shown in Figures 35, 36 and 37, indicating conditions for discharges of 3,000, 4,000 and 5,000 second-feet, respectively. A linear electrode could not be developed in the available time, so the records are not linear. They cannot be used for visual comparison, but must be compared by the dimensions shown on the records. These dimensions are for the maximum range from crest to trough only. Apparently, the height of the waves was magnified on the slope. For example, the elevation of the floor in the trapezoidal section is 451.37 and the elevation of the canal lining is 468.87. Figure 36A is an oscillogram for a discharge of 4,000 second-feet without baffles. The maximum range of wave action was 1.44 feet vertically. Taking one-half of the value, or 0.72 foot, and adding to the mean tailwater depth of 15.22 feet gives an elevation of 467.31 which leaves 1.56 feet freeboard. Referring to line three in Table II the maximum range of wave action up the slope is 12.17 feet. Half of this equals 6.08 feet up the slope above mean tailwater. Converting 6.08 feet to the vertical by the function of the $1\frac{1}{4}$:1 slope, gives 3.8 feet. Adding this to the mean tailwater elevation gives elevation 470.39 which is 1.52 feet above the canal lining at elevation 468.87. This shows that the wave heights are magnified on the slope and that there is not enough freeboard on the canal for a discharge of 4,000 second-feet with no baffles to dampen the waves. The tailwater on all tests was regulated according to the curve furnished by the design department and is the one shown on Figure 30 for an n of 0.014.

There was considerable spray and splashing in the pool. The additional height on the sidewalls did not retain it all, as may be seen from Figures 38 and 39. These photographs were taken after the model had run for a period equivalent to one hour on the prototype. The surface baffles were in place during this time. The general arrangement of the entire model, with and without the surface baffles is shown in Figure 40.

<u>The 1:16 sectional model of one center outlet.</u> Requests were made for a 1:16 scale sectional model of one of the center outlets using the six-inch hollow-jet valve to determine if the jets from the center valves would clear the center pier. The design department had objected to

cutting all of the concrete between the piers down to elevation 457.25, because it would weaken the piers. They proposed to leave footings on each side of the piers at a distance of 5 feet $10\frac{1}{2}$ inches from and parallel to the extended centerlines of the valves. These footings were incorporated in the model. The six-inch valve did not have the two degree convergence on the inside as the prototype valves will have.

The model was operated on the basis of 5,000 second-feet discharging through three valves, or 1,667 second-feet through one valve, for both maximum and minimum reservoir elevations. There were eight inches clearance, prototype, between the jet and the center pier. The clearance on the prototype should be more since those values will have the two degree convergence. The jet impinged on the footings on each side and in the stop-log grooves in them. Conditions were worse for lower reservoir elevations since the diameter of the jet dropped down toward the floor.

The design department agreed to cut the downstream ends of the footings back still farther. The thickness of the training wall in the pool was increased from 30 inches to 48 inches for a distance of 5 feet downstream from the center pier. This was to allow stop-log grooves to be placed in the wall just below the pier. All of the old stop-log grooves are to be filled. New grooves will be cut in the outside walls in line with grooves in the center training wall. The stop-log grooves in the two outside piers will be replaced by H-beams at the downstream ends of these piers. The jets will not clear the footings completely for all conditions of flow, but with the stop-log grooves relocated no damage can result. These changes are shown on Figure 41. The outside outlet was not built in the model, but the arrangement shown in the figure is the result of the studies made on the center outlet.

Tests on the Friant-Madera Canal Outlets.

The 1:28.44 model of 1944. In 1939 the hydraulic laboratory constructed a 1:28.44 scale model of the Madera Canal Outlets as shown on Figure 12 to determine the adequacy of the pool and apron design. The maximum discharge of the two 78-inch needle valves was 1,500 second-feet and the maximum head was 132 feet.

Initial runs indicated the design to be satisfactory except for a few minor items. The jump was unstable at some discharges and the pool appeared to be longer than necessary. The pool was shortened by 50 feet, but the jump was still unstable. The pool floor was raised 1.25 feet and a vertical sill placed at the downstream end of the pool. The raising of the floor shortened the parabolic apron approximately four and five-tenths feet. The top of the sill was fixed at the same elevation as that of the canal bottom downstream from the pool.

The performance of the pool was excellent for the new arrangement. In addition to stabilizing the jump, considerable excavation was saved. The final design is shown on Figures 12 and 42.

Pool conditions were satisfactory when two valves were discharging 1,500 second-feet at maximum head, or when discharging 1,500 second-feet at low heads, Figures 43A and B. One valve operating partially open under high heads was not acceptable. An eddy formed in the pool and eventually swept the tailwater out of the pool. One valve operating wide open under low heads was satisfactory since the jet spread on the apron and formed a jump in the pool. Operation under these conditions is shown in Figures 43C and D. The tailwater rating curve used for model operation is shown on Figure 44.

Tests on the Friant Dam Spillway Crest and Stilling Pool; River Outlets and Stilling Pool; and the Bulkheads in the River Outlet Trashracks.

The 1:25 model of the spillway crest. In 1936, the Bureau of Reclamation Hydraulic Laboratory at Fort Collins was authorized to conduct model studies on the ogee-type overflow spillway crest at Friant Dam. The design of this crest is very similar to that of the Grand Coulee Crest. As a result of this similarity, the model of the Grand Coulee Crest was used. Very few changes were required in adapting the new model to this crest. This procedure resulted in the 1:25 scale ratio. The model represented a 50-foot section of the spillway crest or one-half of one of the three 100-by 18-foot drum gates used to control the flow over the crest. These gates, which recede into the crest and take the form of the ogee section when completely lowered, are separated by piers.

At the time these tests were conducted, it was planned to place the spillway crest at elevation 545.0 and provide for a maximum head of 19 feet. Subsequent plans raised the crest to elevation 560.0, but kept the maximum head at 19 feet. Because of this fact, only the approach conditions were changed. Since the coefficients were corrected for velocity of approach, and since a noticeable change in the pressures could not result from such a small change on the model, these data are still applicable to the Friant Dam. Positive pressures were found to exist for the maximum discharge with the gate completely lowered, Figure 13. Because the pressures on a crest of this type increase as the discharge decreases, positive pressures will exist for all flows below the maximum. The pressure distribution on the upstream portion of the crest and on the top of the drum gate was obtained for various gate positions at maximum reservoir elevation. The piezometer data were plotted showing the pressure distribution as determined by the model tests, Figure 13. These data were submitted to the design section for making a diagram of the hinge-pin reactions.

Thorough calibration of a model dam is often important because it affords a reliable source for determining the magnitude of floods in the field. It was for this reason and for checking the adequacy of the spillway that the model of the Friant Spillway Crest was calibrated with the drum gates at various positions. The discharge coefficients were obtained for several reservoir elevations at each gate position. The plotted data formed a family of coefficient curves, Figure 14A.

Corrections for pier contractions were determined from results obtained in connection with the Grand Coulee pier tests, in which the shapes of the crest and piers were similar. This procedure was justified since the correction at maximum discharge was only 1.5 percent for the Grand Coulee piers. The magnitude of the adjustments was determined by obtaining the difference in coefficients for the crest with and without piers and dividing this difference by the value of the coefficient without piers. This procedure was followed for various reservoir elevations and the results plotted against the head expressed in percent of the maximum head, Figure 14C. The values of the coefficients thus obtained were approximate, but are within the limits of experimental error. In applying the corrections to the coefficient curves for each gate elevation, the head on the high point of the gate was expressed in terms of percent of the maximum head, in this case 19 feet, and the corresponding values taken from the curve. These were then subtracted from the coefficients, for the proper reservoir elevation, resulting in a new set of coefficient curves, Figure 14D. Values were taken from this new set of curves and the discharge computed from the formula $Q = CL\,(H)^{3/2}$ where

Q = discharge in second-feet

C = coefficient of discharge taken from the new set of curves.

L = net length of spillway

H = total head on crest = $(H_s + h_v)$

H_s = static head on crest

h_v = velocity head of approach.

The values of the discharge thus obtained were then plotted against the drum gate elevations for constant reservoir elevations to form the discharge diagram of Figure 14B.

The 1:24 model of the spillway stilling pool. This model, a sectional type, was built in 1939 for developing a satisfactory hydraulic jump stilling pool at the toe of the dam. Frequently, it is necessary to use large sectional models instead of complete models because of the size of the prototype, and because more accurate results are possible on the larger models. Moreover, flow conditions are more readily observed as to the effect of changes in design. This model represented a prototype width of 90 feet equally divided on each side of one 12-foot pier. Since the stilling pool was the only concern, the crest was built to correct shape and the gates omitted.

An agreement between the tailwater and the height of water necessary to form an efficient hydraulic jump was lacking. As may be seen on Figure 45, the tailwater was too deep for flows up to approximately seventy thousand second-feet. This indicated a drowned jump at the lower flows. For such conditions, the jet passing down the face of the dam would dive under the tailwater and flow along the bottom of the pool. There would, therefore, be insufficient energy dissipation in the pool to prevent scouring of the riverbed downstream from the apron. For flows above approximately seventy thousand second-feet, the tailwater is not deep enough, in which case the jump may be swept out of the pool. The result would be severe scouring of the riverbed downstream from the apron. From these observations, it was concluded that a sloping apron of some type should be used to obtain a satisfactory jump in the pool.

To obtain the correct slope of the apron, values of D_2, or height required for a jump, were computed for several discharges, including the maximum, from the momentum formula of the hydraulic jump:

$$D_2 = \frac{-D_1}{2} + \sqrt{\frac{D_1^2}{4} + \frac{2D_1v_1^2}{g}}$$

The length of the jump was assumed to be four times the depth D_2 downstream from the origin of the jump. With the pool entrance as origin, length of pool as abscissa, and elevation as ordinate, the tailwater elevation for each discharge was plotted a distance of 4 D_2 from the origin. Using these points as centers and radii equal to the respective D_2, arcs were drawn below. A line tangent (approximately) to these arcs was the slope of the apron to be used, Figure 46. The apron slope thus obtained was approximately 7:1. In the process of changing the apron slope from 4:1 to 7:1, the radius of the curve connecting the slope of the apron with the 0.7:1 slope of the face of the dam was increased from 60 to 100 feet, prototype. The horizontal section of the pool floor was lowered from elevation 285.5 to 282.5.

Several tests were made at various discharges to observe the hydraulic jump performance without any sill at the end of the apron. The computed apron slope of 7:1 was satisfactory for all discharges. Further tests were made to determine the proper length of horizontal apron required below the sloping apron. Several types and sizes of sills were used at the end of the apron to eliminate scouring of the streambed. Progressive scour tests were made for every sill used. A profile was established in the sandbed downstream from the apron, representing the prototype excavation. The model was then operated over a range of several discharges. Photographs were also taken of the hydraulic jump during each operating period. Each discharge was run until the streambed had become stabilized, or approximately forty-five minutes for these conditions. Following each run, photographs were taken and a profile of the sand was taken with a point gage. A comparison of results revealed very little difference in the scour patterns resulting from runs made with sills having a 2:1 or a 3:1 slope on the upstream face. Figure 47 is a comparison of the water surface elevations in the pool when operating with the two different sills. These were obtained on the 1:60 model and were nearly identical with those obtained on the 1:24 model. The hydraulic jump performance led to the recommendation of a solid sill 3.25 feet high and having a 3:1 slope on the upstream face. Figure 48, Section D-D, and Figure 49A show the spillway stilling pool design as recommended by the laboratory, except for the 2:1 sloping sill having been used instead of the 3:1. Flows of 70,000 and 90,000 second-feet being discharged into this pool are shown in photographs B and C of Figure 49.

The 1:60 model of the spillway crest, river outlets, and stilling pool. This model was a complete arrangement of the spillway crest and stilling pool with the river outlets discharging into the spillway stilling pool. It was constructed to observe the spillway pool action with only the crest discharging; to check the scour patterns obtained on the 1:24 model; to check the crest calibration made on the 1:25 model; and to study the spacing and the spreading of the four river outlet jets on the spillway apron.

The first observation was made with only the spillway discharging. The pool performance was excellent for all flows, indicating that the design as developed on the 1:24 model was satisfactory, Figure 50. Water surface and sand profiles were taken for operation with both a 2:1 and a 3:1 sill. The results were compared with data taken on the 1:24 model and found to be practically the same. The 3:1 sill was again recommended, although there was very little difference in the effectiveness of the two.

The design department had proceeded on the basis of a 2:1 slope before the recommendation of a 3:1 slope was received from the laboratory. As a result the design having the 2:1 slope was sent to the field and was constructed on the prototype. As previously indicated, no serious results are expected from such procedure. Section D-D, Figure 48, shows the design as recommended by the laboratory except for the 2:1 sloping sill having been used instead of the 3:1. The data obtained in the calibration of the crest on this model were in close agreement with those obtained on the 1:25 model.

The model was operated with only the four outlets discharging into the spillway stilling pool. Spreading of the jets was somewhat as had been expected, but was not sufficient to produce a uniform jump in the pool. The lack of spreading was due in part to incorrect spacing of the outlets. The jets, after they had plunged into the tailwater, were diffused by the tailwater and prevented from spreading by the hydrostatic pressure in the pool. Because of this insufficient spreading, severe eddies developed in the pool and downstream in the river channel. These eddies carried sand onto the apron of the model, as shown by Figure 51. Such condition in the prototype would cause erosion to both the apron and the riverbed. Each jet was isolated by training walls placed in the pool, but the results were not satisfactory. Moreover, the cost was objectionable, as was also the impact effect of the spillway flow upon the training walls.

A telegram from the project requested results of a test with the outlets passing straight through the spillway section at elevation 358.0. The 1:60 model was changed accordingly with the outlets spaced 17.50 feet apart in pairs. Various combinations of the outlets were run with a full reservoir. Serious flow conditions in the pool removed all of the loose material for a distance of 175 feet, prototype, downstream from the apron. The large whirl which formed in the stilling pool became worse with one outlet on one side operating. Flow conditions in the pool are shown in Figures 52 and 53 with the tops of the pegs in Figure 53C representing the original ground surface. This is equivalent to the removal of 10 feet of erodible material on the prototype. The plan for the river outlets in the spillway section was abandoned, partly for the above reasons and, in part, because of the inability to regulate the outlet flow with the tube valves on the upstream ends of the outlets. The difficulty of operation of the tube valves was the excessive negative pressures in the valves and in the outlets below the valves in a similar design being studied for Shasta Dam. The outlets were moved to the left of the spillway, where they discharged into a separate stilling pool, Figure 5.

The 1:34.38 model of the river outlets. This model of the river outlets and stilling pool was constructed as located at the side of the spillway. Figure 15 shows the design as submitted for laboratory tests. During a preliminary run it was observed that at the maximum discharge of 17,000 second-feet the jets impinged on the sill at the end of the pool, causing excessive turbulence and preventing the hydraulic jump from forming. The jump formed at lower discharges, but the tailwater worked upstream into the jets and became violently agitated. At all discharges there was a sharp increase in velocity as the water plunged over the end sill and into the river below.

The stilling pool was lengthened in an attempt to have the hydraulic jump form at the higher discharges. This was accomplished, but the problem of the flow plunging over the sill into the river was still prevalent. The pool floor was then lowered from elevation 305.0 to 289.0 and a parabolic apron was installed according to the jet trajectory of a 96-inch needle valve. The pool was shortened to accompany the increase in its depth. A satisfactory jump was formed at all discharges except at those approaching the maximum of 17,000 second-feet.

The valves were tilted to 17 degrees to eliminate the bend in the conduit immediately upstream from the valves. This change shortened the pool and the apron which was designed according to the trajectory of a 102-inch tube valve. This apron was too short to allow complete spreading of the jets. It was difficult to determine the correct tilt for the valves. A slight tilt made a longer pool, but eliminated excavation at the apron. A greater tilt shortened the pool, but increased excavation at the apron. This test indicated that a tilt greater than 10 degrees would not provide an apron long enough to obtain proper spreading of the jets. Therefore, a longer apron and deeper pool must be provided. A 10 degree tilt was taken to be the most economical solution, balancing excavation, length of structure, and hydraulic performance.

A compromise between the advocates of the two types of valves resulted in the placing of two 102-inch tube valves in the right pool and two 96-inch needle valves in the left pool. The horizontal centerlines of the valves were raised to elevation 330.0 to clear a gallery in the dam. The design section fixed the pool floor at elevation 289.0, thereby lengthening the parabolic apron considerably. A 16-foot vertical sill was placed at the downstream end of the pool. With this arrangement the jump formed too far down in the pool at the higher discharges, so the pool was gradually shortened until the most satisfactory operation was obtained. Even then the jump on the tube-valve side was a little downstream for a discharge of 17,000 second-feet. These jets were not spread as much as had been expected. This arrangement, Figures 16 and 54, was excellent for discharges of 12,000 second-feet and less. For higher flows, only fair conditions existed in the stilling pool, becoming worse with increase in flow. It was accepted as final since maximum discharge was not expected to occur very often, and inasmuch as a drawing of this arrangement has been sent to the field previous to the model tests. Since the channel was to be in rock cut, the water plunging over the end sill into the river was not considered to be extremely serious. Various stages of operation of this design are shown in Figures 55, 56, and 57.

<u>The 1:32 model of the river outlets.</u> In 1944 the hydraulic laboratory was asked to conduct model studies in connection with replacing the tube and needle valves with four of the newly-developed hollow-jet valves having a 96-inch diameter. Due to the larger diameter of these valves part of the concrete apron would have to be removed to allow passage of the jets. It was suggested that transitions be placed between the conduits and the valves such as to raise the valves and decrease their angle of tip from 10 degrees to 7 degrees. Preliminary computations indicated that clearance could be provided by removing about 12.5 inches of concrete at the upper end of the apron and letting it run out to zero at some point down on the apron. Using the same origin a new parabolic apron having a flatter slope was extended to an intersection with the original apron at elevation 319.626. The equation of the new apron is $y = -0.0010802X^2 + 0.0466X - 3.82$ and is the lower one shown on Figure 17. The maximum

depth of concrete to be removed for the new shape was 15 inches. The model as constructed in the laboratory is shown on Figures 17 and 58A.

The model operating with four valves at various discharges is shown in the photographs of Figures 59B, C, D and 59A. From these figures it is evident that 16,000 second-feet is very near the maximum capacity of the structure.

Various combinations of two valves in operation are shown in Figures 59B, C, D, and 60. The maximum desirable discharge for two valves operating was approximately five thousand second-feet. The one valve operating as shown in Figure 61 clearly indicates 3,000 second-feet to be the maximum allowable discharge for this operating condition, and that this would not be desirable for any length of time. The maximum desirable flow for one valve operating was approximately two thousand second-feet.

As in previous model tests, there was a drop in water surface as the water passed over the sill at the end of the pool. This was true for all discharges and became more pronounced with increase in discharge. The design was considered satisfactory for operation with the hollow-jet valves for all discharges up to 16,000 second-feet. Symmetrical operation is not compulsory but would be desirable, especially for higher flows being discharged under high heads.

The streamlined design of the transition between the conduit and valves as submitted for testing fixed the angle of tip at 6 degrees instead of 7 degrees, Detail 1, Figure 17. The difference in pool operation was not considered to be enough to justify another test on the pool. However, a 6-inch model of the transition was built and tested for pressures within the valve and transition. There were no negative pressures in the transition, and the pressures were only slightly reduced on the upper part of the valve body. The coefficient of the valve was the same as that previously determined in the laboratory with the valve attached to a straight approach pipe. The design was considered satisfactory and its use was recommended by the laboratory.

The 1:18.33 model of the temporary bulkheads in the river outlet trashracks. During the construction of Friant Dam it was necessary to place bulkheads in the lower bays of the outlet trashracks to impound the rising water until the coaster-gate tracks and seal seats on the face of the dam were installed. It was undesirable to remove these bulkheads immediately after the tracks and seal seats were installed, but preferable to let the water rise above them and discharge through the outlets. The effect the bulkheads would have upon the pressures in the outlets was uncertain, so hydraulic model studies were utilized to observe their behavior and to measure the pressures in the entrance for various water surface elevations.

A 1:18.33 scale model of one of the outlets was constructed with the bulkheads in the trashrack as they would be during the construction period, Figure 18. Since each trashrack served two outlets, a wall dividing the trashrack was necessary to account for the effect of an adjacent outlet. The length of the model conduit was made less than that of the prototype to compensate for the greater friction factor in the model.

Previously, a mathematical analysis had been made to determine the proper exit diameter of temporary nozzles to be placed on the river outlets. The analysis appears in Appendix A. During construction of the dam the outlets were needed to supplement the flow through the diversion tunnels. These nozzles were to maintain positive pressures in the conduits, thereby avoiding cavitation and pitting. The recommended exit area of the nozzles was 65 percent of the area of the conduit. A nozzle having an exit area of 64 percent of the conduit area was installed on the outlet of this model, the 64 percent resulting from the use of an even diameter.

A group of tests was run with the bulkheads in position and the pressures measured for various water surface elevations by piezometers located on the top and bottom of the bellmouth entrance. In each test the effect of the air vent upon the pressures was studied.

The pressures on the top of the outlet were lower than those on the bottom. The maximum negative pressure of 9 feet, prototype, occurred with the water surface at elevation 396.0 or 16 feet above the centerline of the conduit. With the air vent closed a negative pressure of 15 feet was recorded for a water surface elevation of 389.0 feet. The pressures on the top and bottom of the outlet for given water surface elevations with the bulkheads in place are shown on Figure 18.

Removing the bulkheads decreased the pressures on the bottom of the conduit three to five feet of water, prototype, but increased the pressures on the top by only one-half to one foot, prototype. Conversely, the pressures on the bottom of the conduit were increased with the bulkheads in position and the pressures on the top decreased only slightly. Therefore, the bulkheads have no important effect upon the pressures in the conduit.

The types of flow through the outlets appeared to control the pressures. For reservoir water surfaces less than elevation 390, typical open channel flow existed with the conduit flowing only partially full. Since large quantities of air could pass through the outlet, atmospheric pressures existed along the top of the conduit. The jet issuing from the exit was smooth except for fins which tended to develop along the upper edges.

At water surfaces between elevations 390 and 410, the conduit filled but negative pressures developed. The air vent relieved these negative pressures considerably. Nevertheless, if the outlets were discharged at these elevations for a long period of time, it might be necessary to close the air vents. In the model the air would not mix with the water but appeared to gather in the conduit in slugs which, when ejected, caused spray. Pressure changes were evidenced by a pounding in the model in a manner similar to a repeated water hammer and by considerable fluctuations in piezometer readings. Such conditions might cause serious vibration in the prototype. Closing the air vents produced a smooth flow with no spray or fluctuations of pressure even though the pressures were lowered.

With water surfaces above elevation 410, positive pressures existed in all portions of the outlet and the air vent filled with water. The tests thus indicated that the critical conditions would occur at water surfaces between elevation 390 and 410.

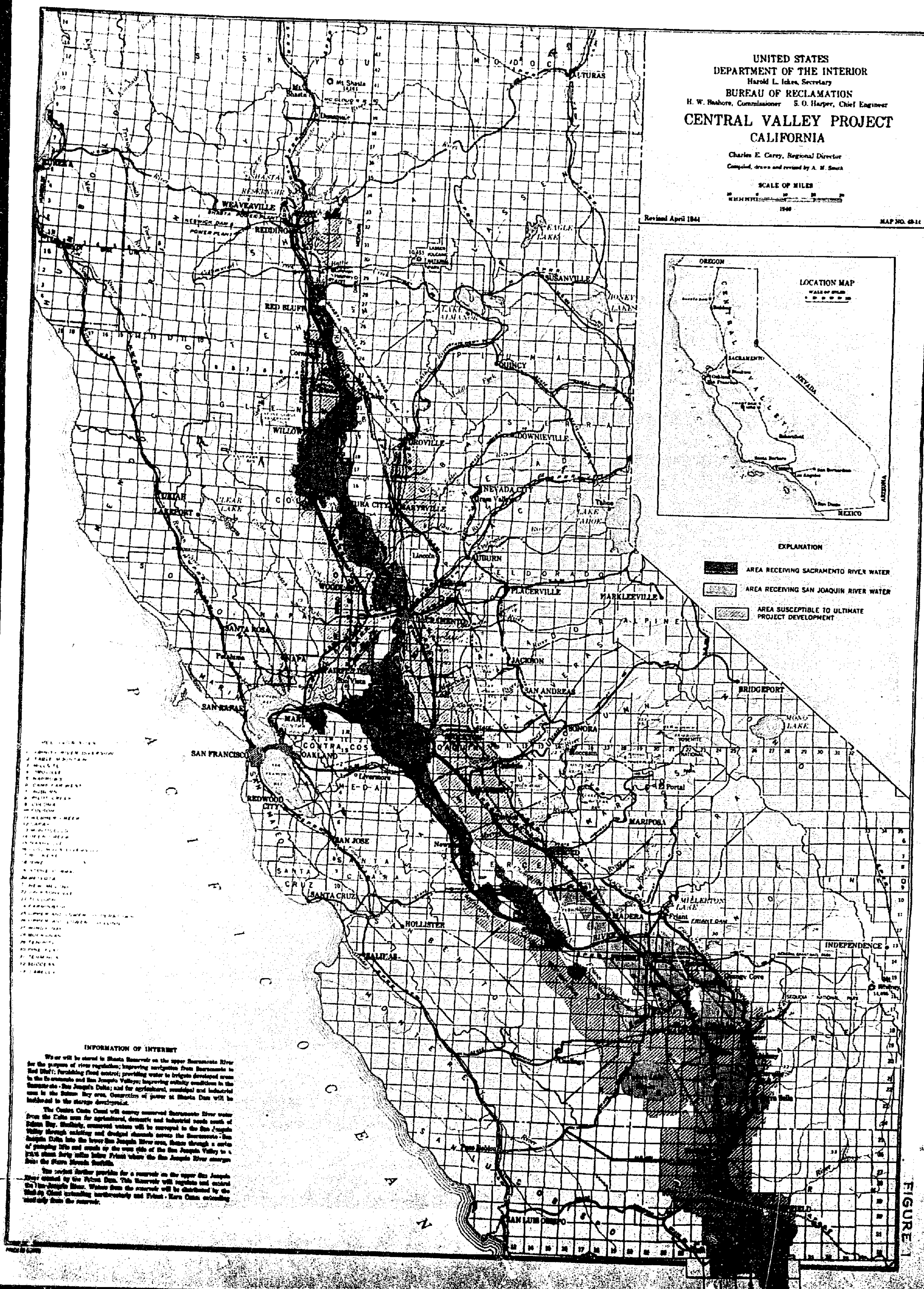
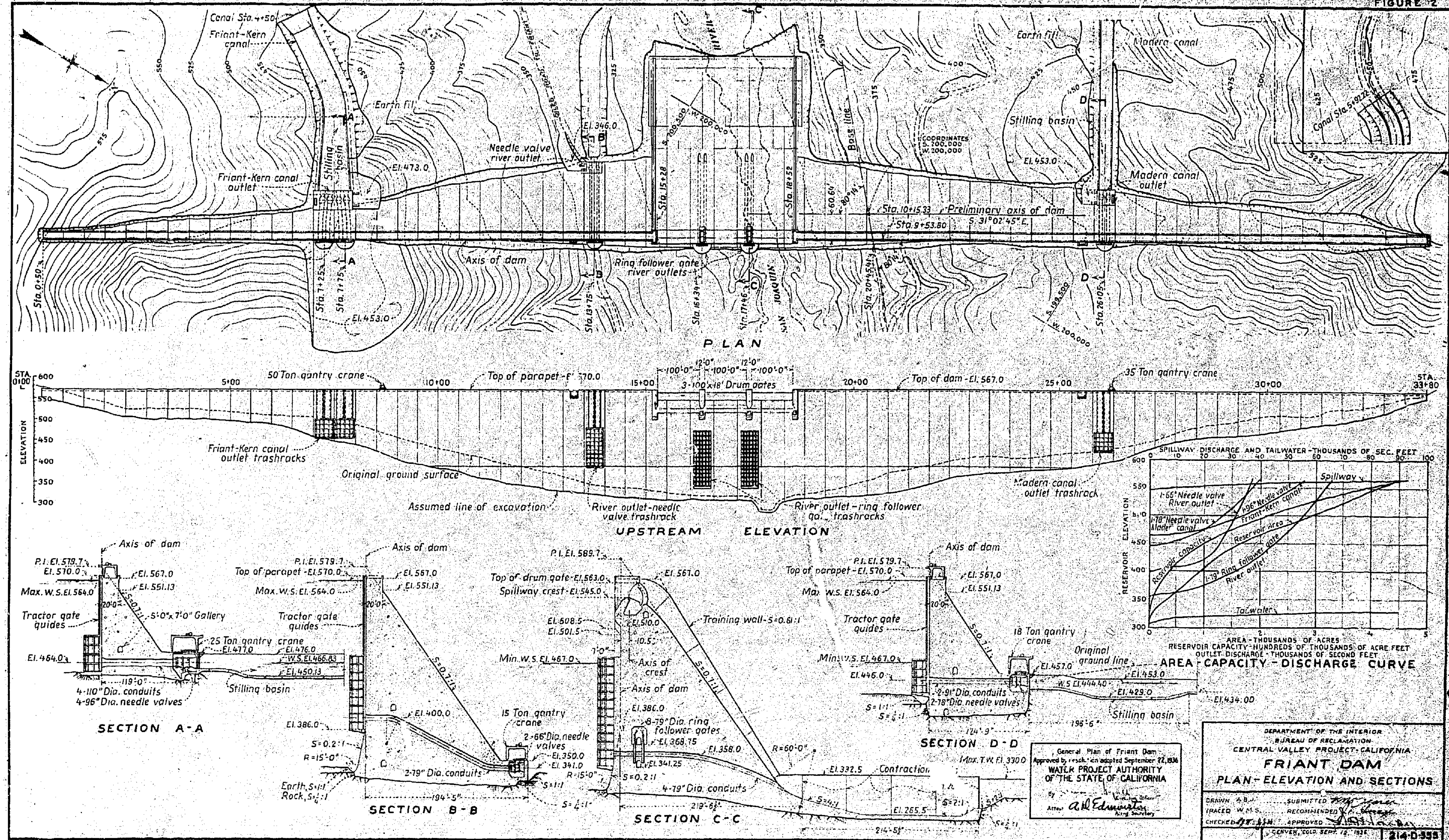
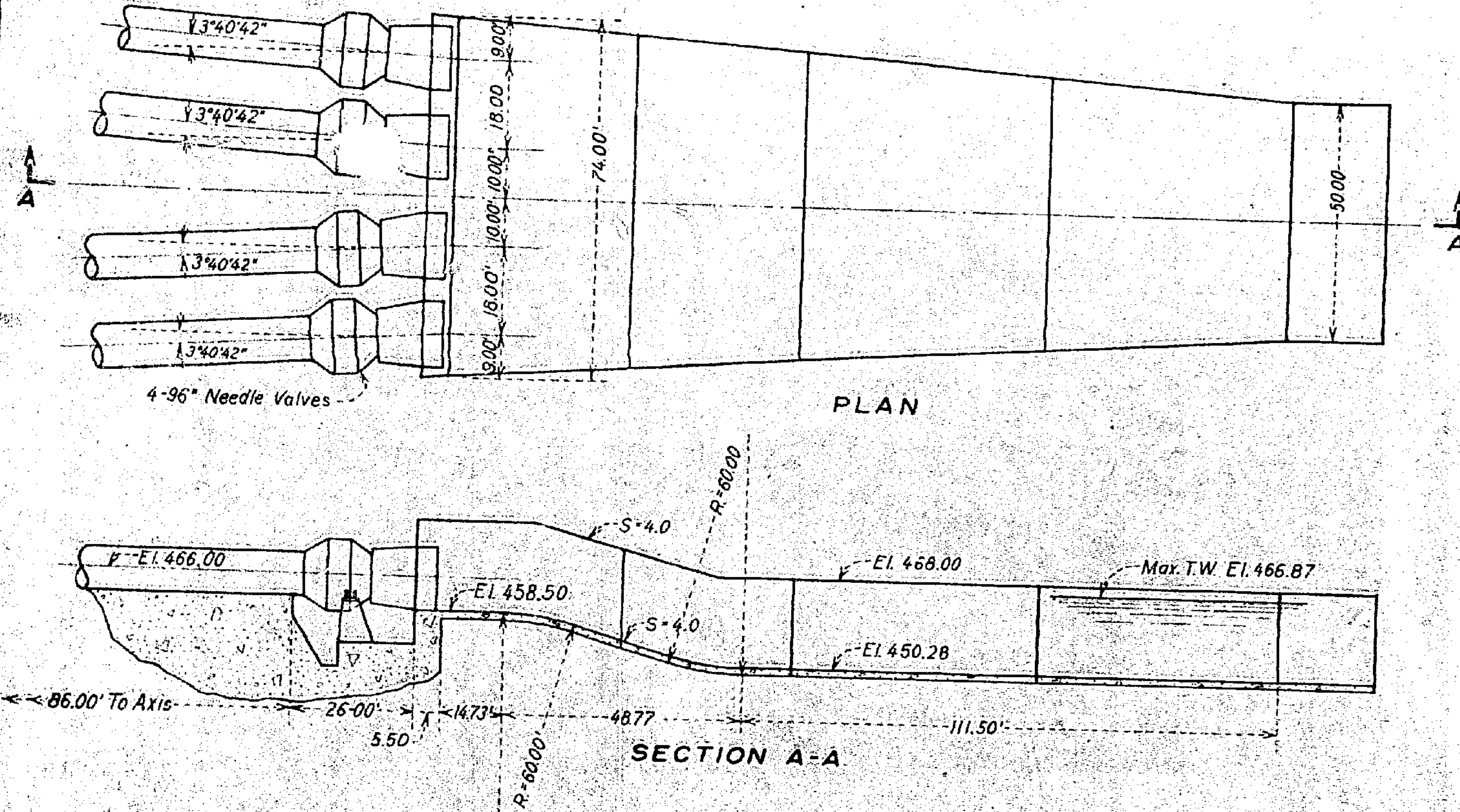

UNITED STATES
DEPARTMENT OF THE INTERIOR
Harold L. Ickes, Secretary
BUREAU OF RECLAMATION
H. W. Bashore, Commissioner S. O. Harper, Chief Engineer
CENTRAL VALLEY PROJECT
CALIFORNIA
Charles E. Carey, Regional Director
SCALE OF MILES
Revised April 1944
LOCATION MAP
EXPLANATION
AREA RECEIVING SACRAMENTO RIVER WATER
AREA RECEIVING SAN JOAQUIN RIVER WATER
AREA SUSCEPTIBLE TO ULTIMATE PROJECT DEVELOPMENT
INFORMATION OF INTEREST
PACIFIC OCEAN
SAN FRANCISCO
SACRAMENTO
REDDING
RED BLUFF
OROVILLE
MADERA
MILLERTON LAKE
FRIANT DAM
SAN LUIS OBISPO
INDEPENDENCE
BRIDGEPORT
MONO LAKE
SANTA CRUZ
SAN JOSE
OAKLAND
HOLLISTER
WEAVERVILLE
SUSANVILLE
ALTURAS
QUINCY
DOWNIEVILLE
NEVADA CITY
AUBURN
PLACERVILLE
MARKLEEVILLE
JACKSON
SAN ANDREAS
SONORA
MARIPOSA
SANTA ROSA
NAPA
SAN RAFAEL
REDWOOD CITY
LAKEPORT
UKIAH
EUREKA

FIGURE 1

SPECIFICATIONS No.

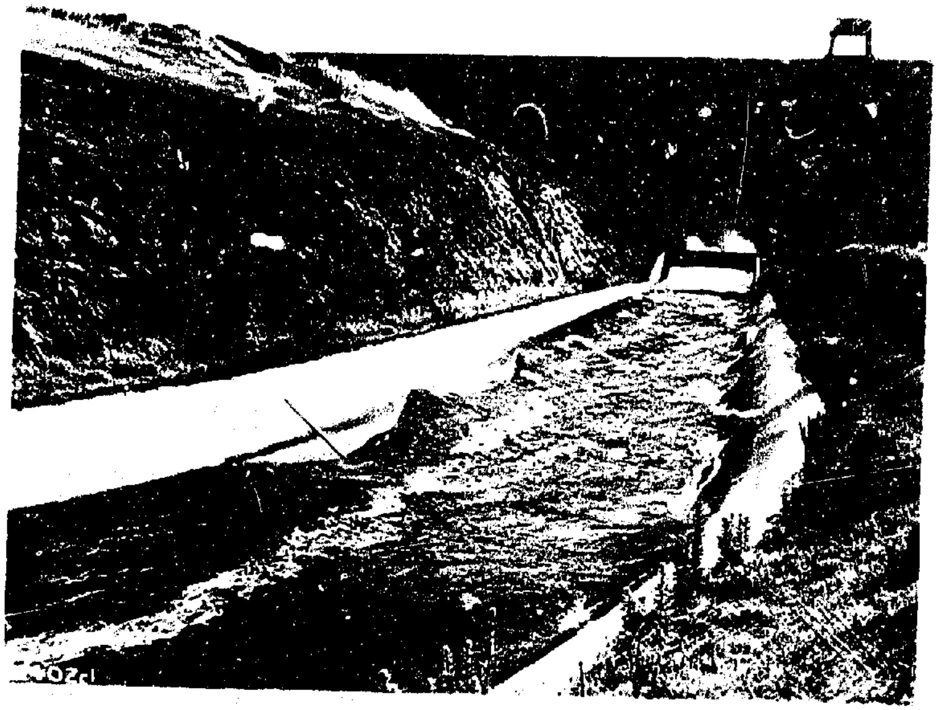
FIGURE 2





CENTRAL VALLEY PROJECT – CALIFORNIA
FRIANT-KERN CANAL OUTLETS
ORIGINAL APRON AND STILLING POOL
DESIGN SUBMITTED FOR MODEL STUDIES – 1936

A - Discharge = 472 s.f. through one valve 13% open. Res. elev. 559.7. Looking downstream from valve house.



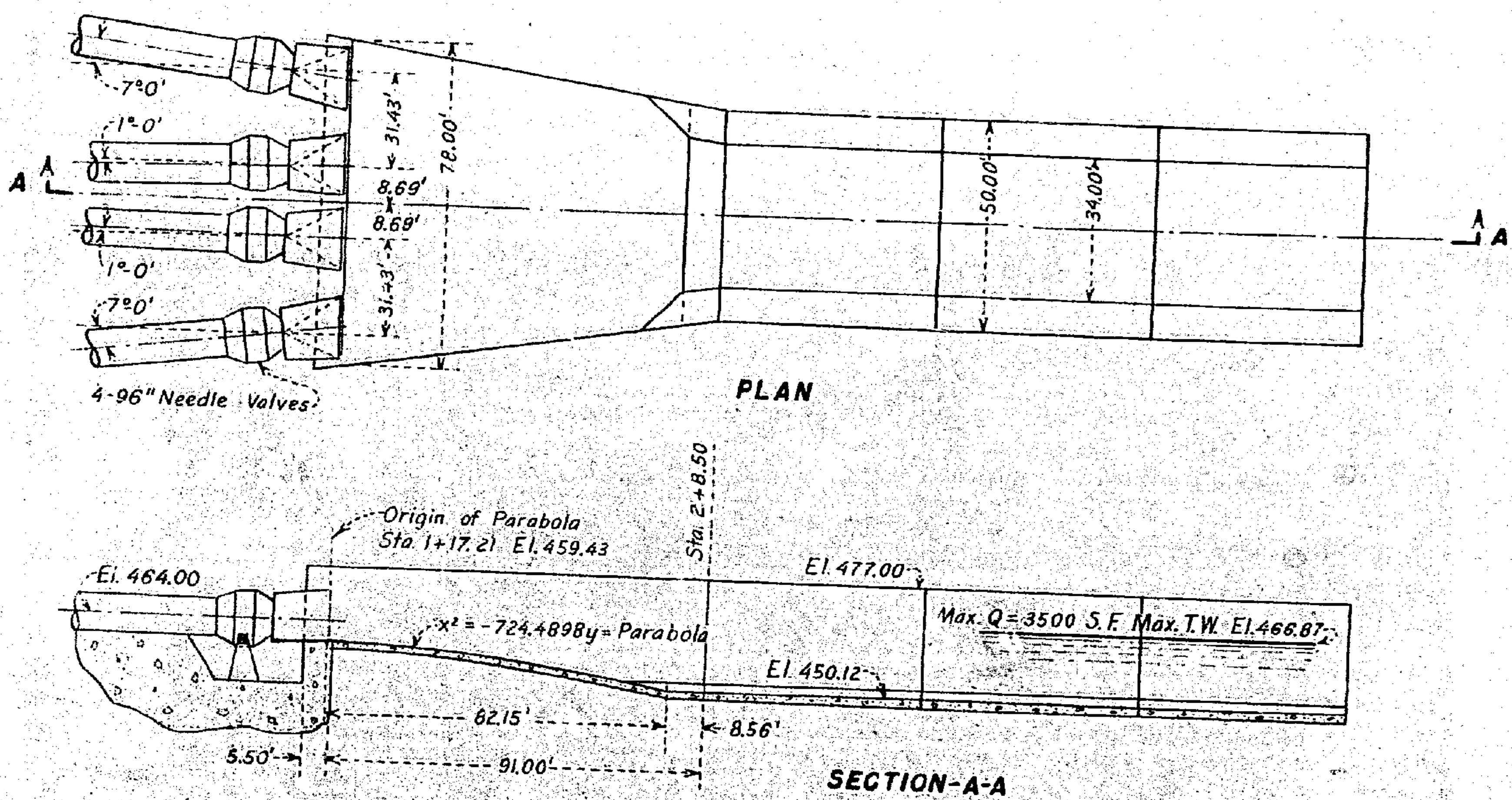
B - Discharge = 472 s.f. through one valve 13% open. Waves have overtopped 9 foot gage in left foreground.

FRIANT-MADERA CANAL OUTLETS
WAVE ACTION IN PROTOTYPE FOR ONE VALVE OPERATING

A. River outlets relocated at side of spillway.

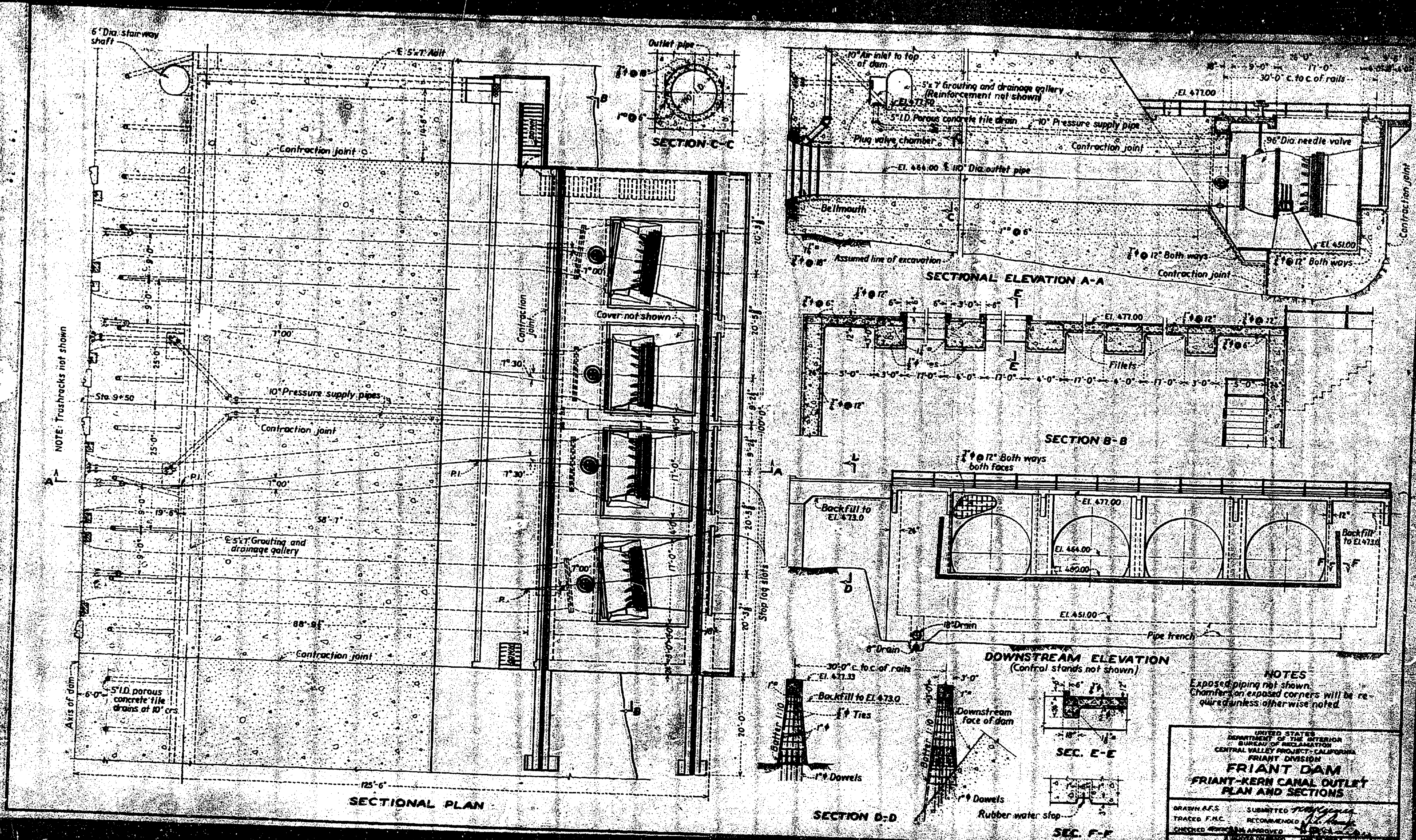
B. River outlets operating at side of spillway.

FRIANT DAM SPILLWAY AND RIVER OUTLETS
1:60 Model



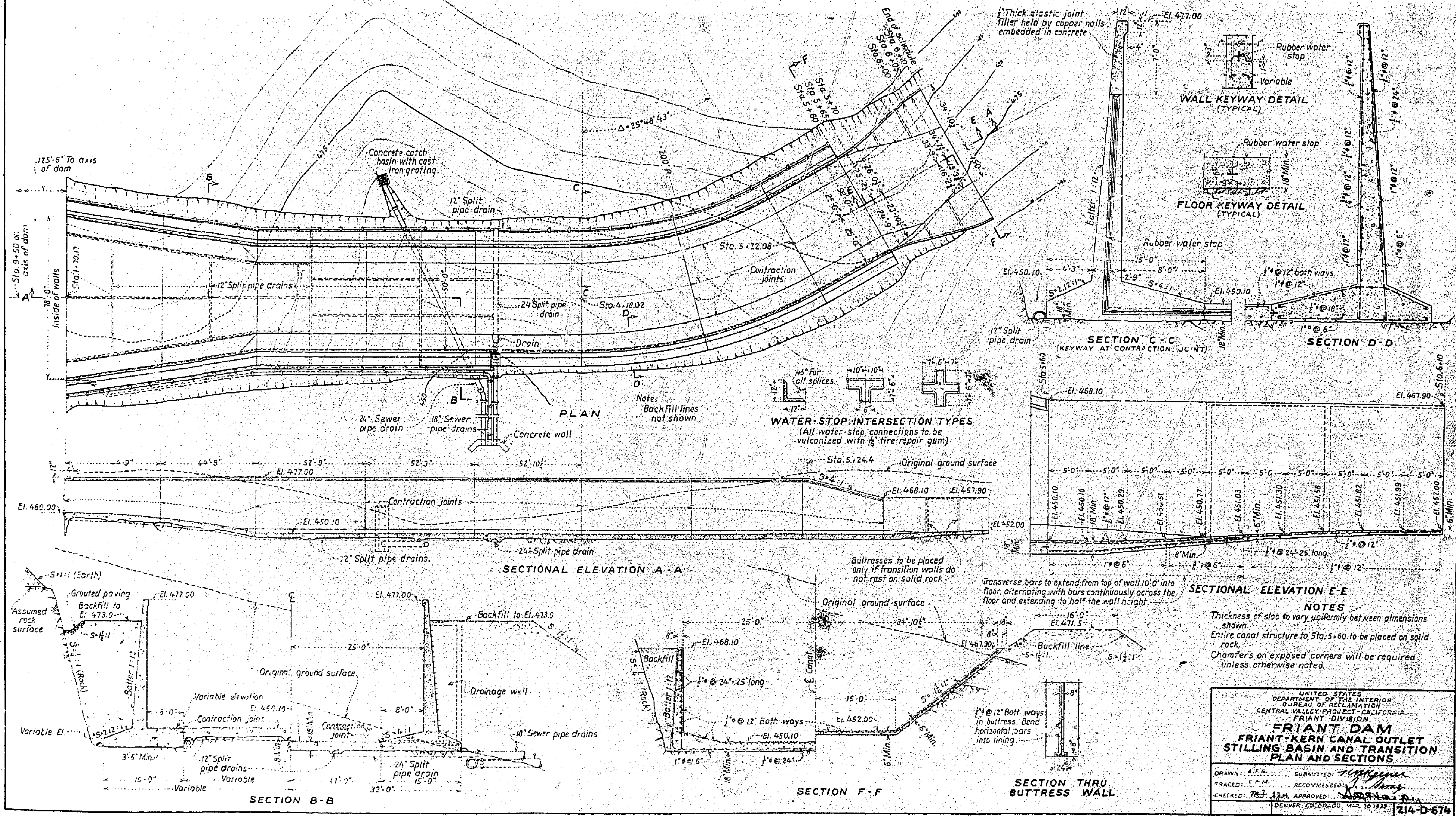
CENTRAL VALLEY PROJECT-CALIFORNIA
FRIANT KERN CANAL OUTLETS
APRON AND STILLING POOL DESIGN
1:34.29 SCALE MODEL

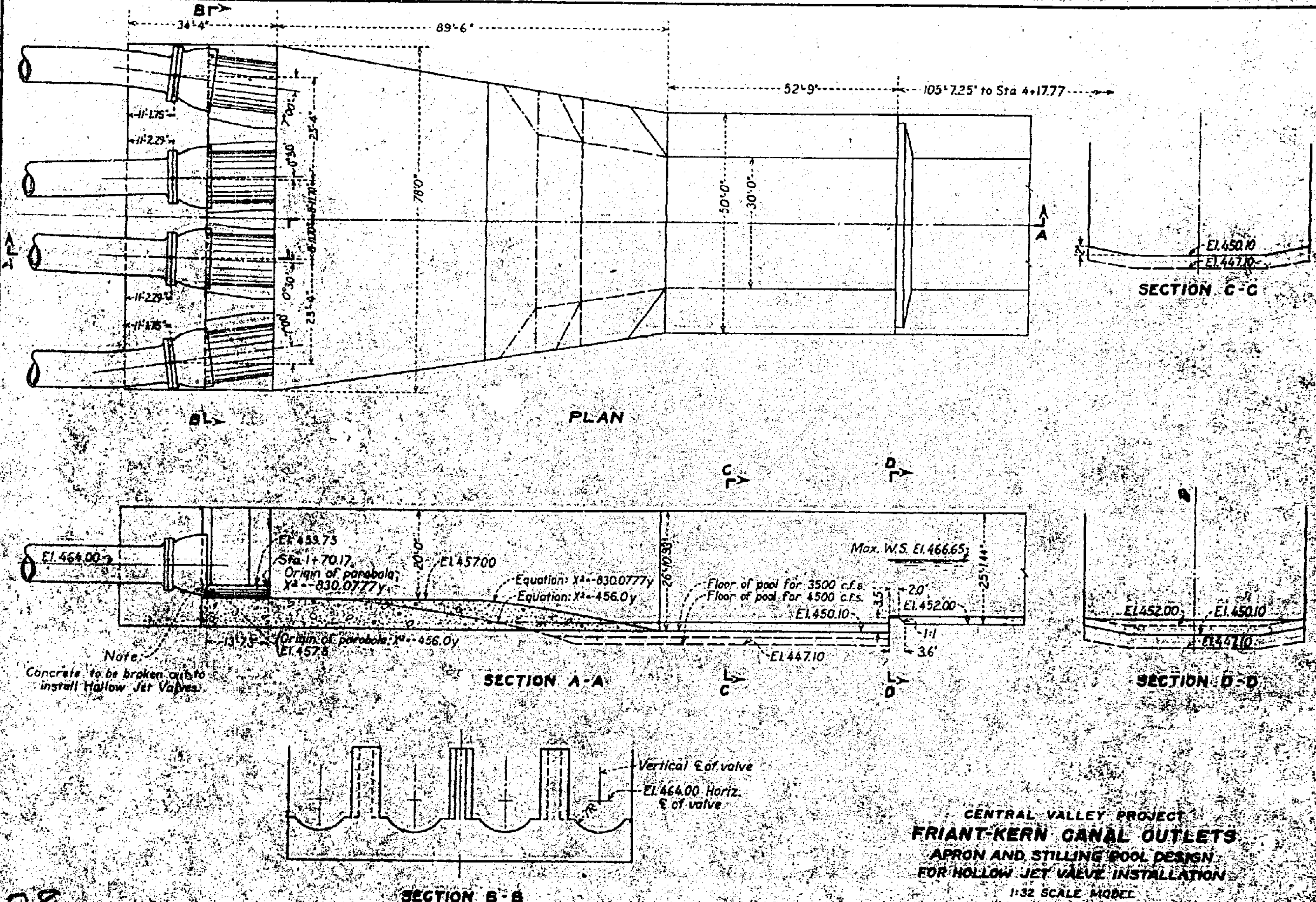
FIGURE 6



SECTIONAL PLAN
SECTION C-C
SECTIONAL ELEVATION A-A
SECTION B-B
DOWNSTREAM ELEVATION
(Control stands not shown)
SECTION D-D
SEC. E-E
SEC. F-F
NOTE: Trashracks not shown
6' Dia. stairway shaft
5'x7' Adit
Contraction joint
10" Pressure supply pipes
5'x7' Grouting and drainage gallery
5" I.D. porous concrete tile drains at 10' crs.
Axis of dam
Sta. 9+50
Cover not shown
Stop log slots
125'-6"
Outlet pipe
10" Air inlet to top of dam
5'x7' Grouting and drainage gallery (Reinforcement not shown)
5" I.D. Porous concrete tile drain
10" Pressure supply pipe
Plug valve chamber
96" Dia. needle valve
El. 464.00
110" Dia. outlet pipe
Bellmouth
Assumed line of excavation
El. 477.00
El. 451.00
30'-0" c. to c. of rails
Fillets
Backfill to El. 473.0
18" Drain
8" Drain
Pipe trench
El. 460.00
Ties
1" Dowels
Downstream face of dam
Rubber water stop
NOTES
Exposed piping not shown.
Chamfers on exposed corners will be required unless otherwise noted.
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT - CALIFORNIA
FRIANT DIVISION
FRIANT DAM
FRIANT-KERN CANAL OUTLET
PLAN AND SECTIONS
DRAWN R.F.S.
TRACED F.M.C.
CHECKED
SUBMITTED
RECOMMENDED
APPROVED

FIGURE 8


PLAN
Note: Backfill lines not shown.
Concrete catch basin with cast iron grating.
12" Split pipe drain
12" Split pipe drains
24" Split pipe drain
Drain
24" Sewer pipe drain
18" Sewer pipe drains
Concrete wall
Contraction joints
Sta. 5+22.08
Sta. 4+18.02
Sta. 9+50 on axis of dam
Inside of walls
125'-6" To axis of dam
200' R.
Δ=29°48'43"
End of schedule
WATER-STOP INTERSECTION TYPES
(All water-stop connections to be vulcanized with 1/8" tire repair gum)
45° for all splices
SECTIONAL ELEVATION A-A
Original ground surface
Contraction joints
El. 477.00
El. 460.00
El. 450.10
El. 468.10
El. 467.90
El. 452.00
12" Split pipe drains.
24" Split pipe drain
WALL KEYWAY DETAIL
(TYPICAL)
Rubber water stop
Variable
FLOOR KEYWAY DETAIL
(TYPICAL)
1" Thick elastic joint filler held by copper nails embedded in concrete
El. 477.00
El. 450.10
Batter 1:12
SECTION C-C
(KEYWAY AT CONTRACTION JOINT)
SECTION D-D
SECTIONAL ELEVATION E-E
Transverse bars to extend from top of wall 10'-0" into floor, alternating with bars continuously across the floor and extending to half the wall height.
Buttresses to be placed only if transition walls do not rest on solid rock.
SECTION B-B
Grouted paving
Backfill to El. 473.0
Assumed rock surface
Backfill to El. 473.0
Original ground surface
Drainage well
Variable elevation
Contraction joint
Variable El.
18" Sewer pipe drains
SECTION F-F
Original ground surface
Backfill line
SECTION THRU BUTTRESS WALL
NOTES
Thickness of slab to vary uniformly between dimensions shown.
Entire canal structure to Sta. 5+60 to be placed on solid rock.
Chamfers on exposed corners will be required unless otherwise noted.
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT-CALIFORNIA
FRIANT DIVISION
FRIANT DAM
FRIANT-KERN CANAL OUTLET
STILLING BASIN AND TRANSITION
PLAN AND SECTIONS
DENVER, COLORADO
214-D-674



CENTRAL VALLEY PROJECT

FRIANT-KERN CANAL OUTLETS

APRON AND STILLING POOL DESIGN FOR HOLLOW JET VALVE INSTALLATION

1:32 SCALE MODEL

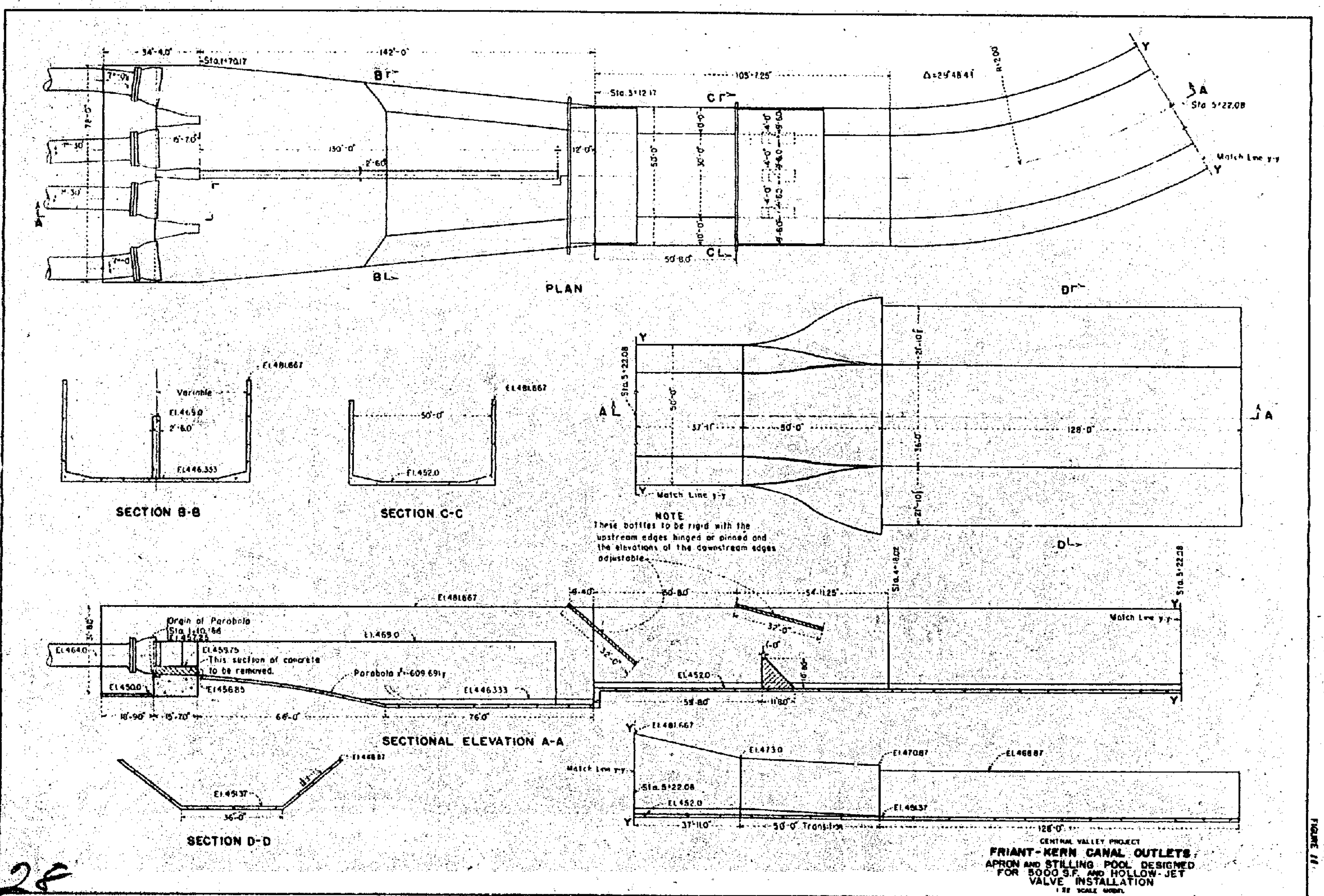

PLAN
SECTION B-B
SECTION C-C
SECTION D-D
SECTIONAL ELEVATION A-A
NOTE
These bottles to be rigid with the upstream edges hinged or pinned and the elevations of the downstream edges adjustable.
Origin of Parabola
This section of concrete to be removed.
Match Line y-y
CENTRAL VALLEY PROJECT
FRIANT-KERN CANAL OUTLETS
APRON AND STILLING POOL DESIGNED FOR 5000 S.F. AND HOLLOW-JET VALVE INSTALLATION
1:32 SCALE MODEL

FIGURE 11

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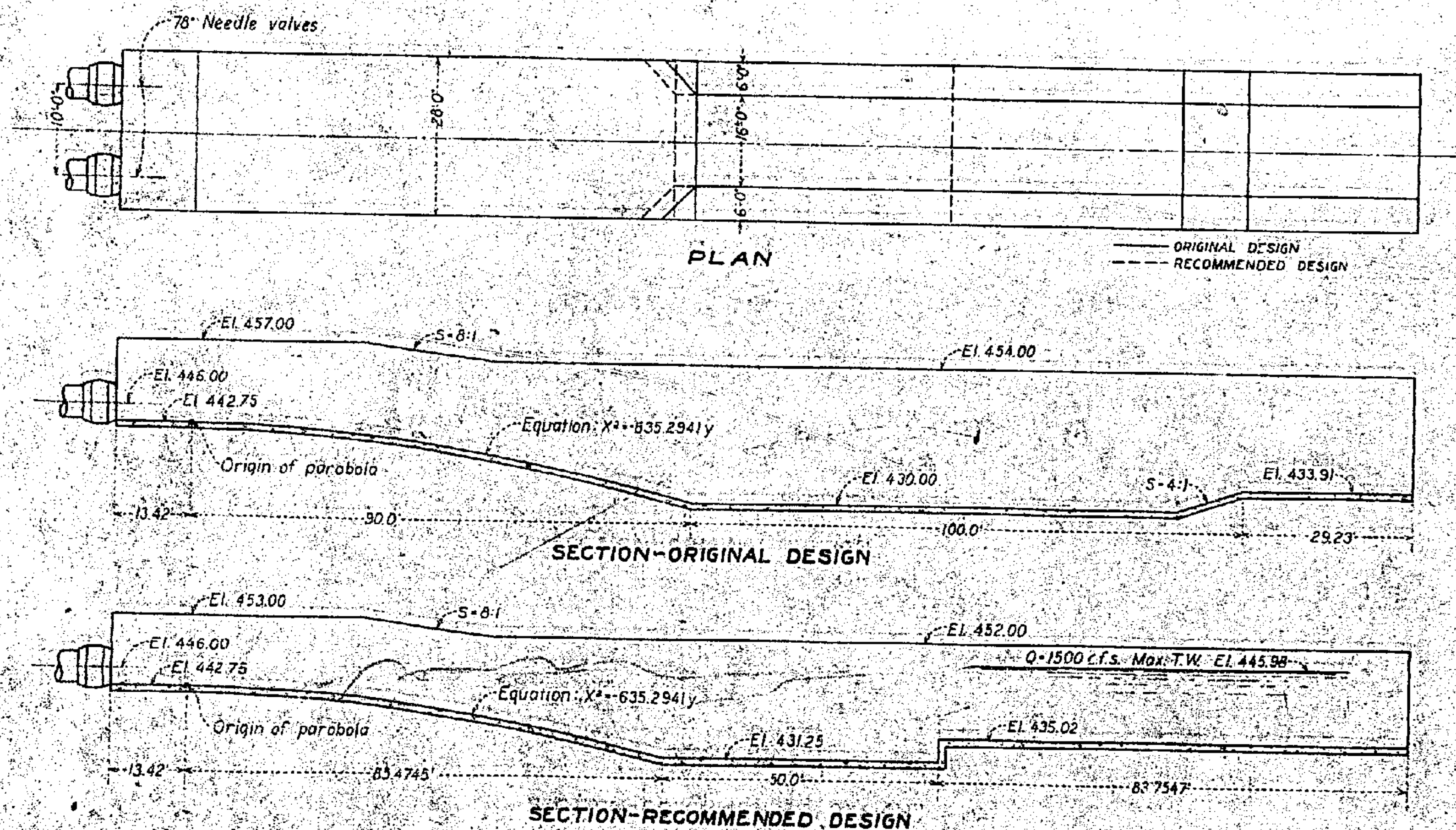
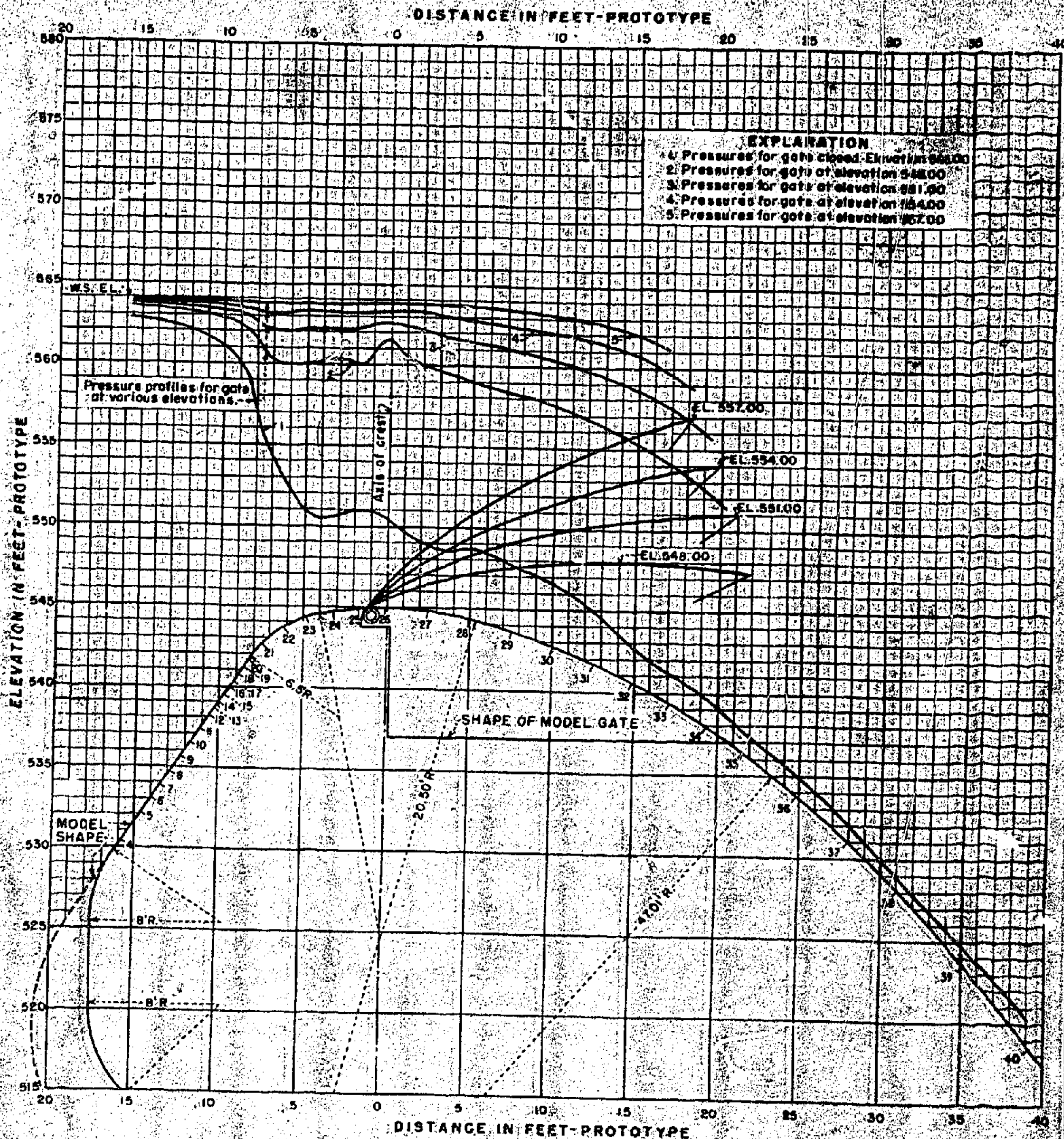
FIGURE 12



CENTRAL VALLEY PROJECT
FRIANT DAM
MADERA CANAL OUTLET
NEAT LINES OF WATER PASSAGES



NOTE

The upstream portion of the model crest was not made to conform to the shape of the prototype because it was believed that this portion would not effect the pressures on the gate. Pressures were taken for various gate elevations with maximum reservoir elevation 564.00 only.

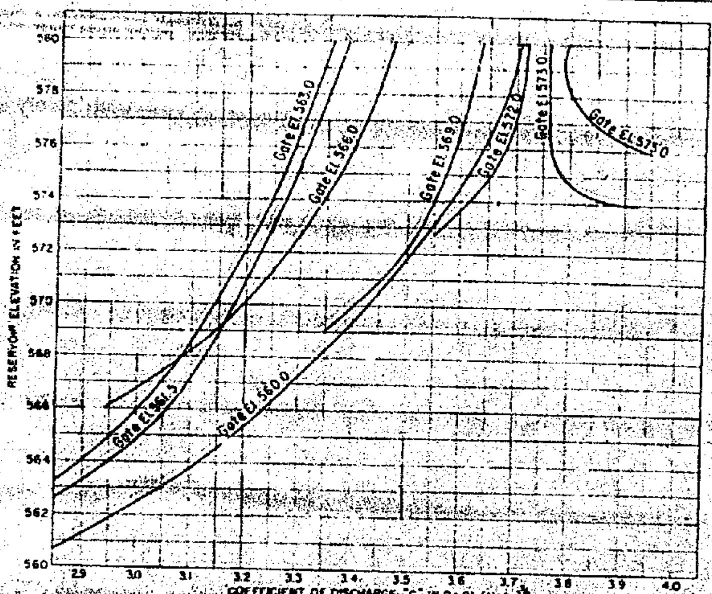
CENTRAL VALLEY PROJECT-CALIFORNIA

FRIANT DAM

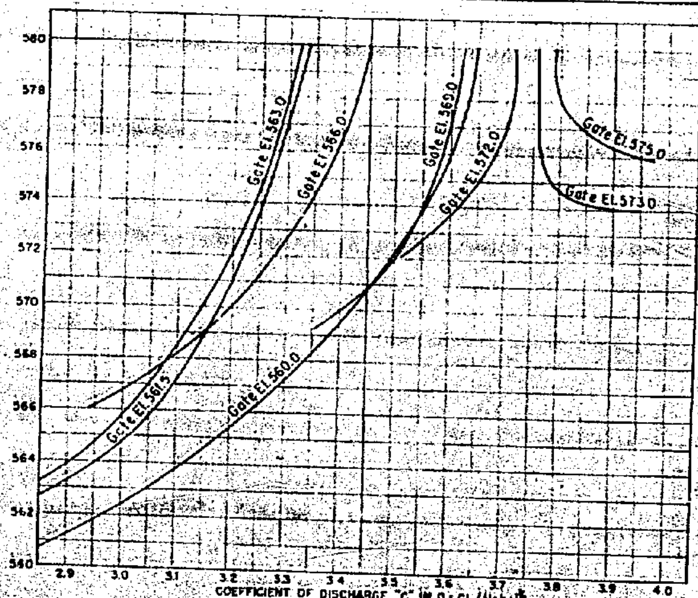
PRESSURES ON CREST AND GATE

HYDRAULIC STUDIES ON A 1:25 SCALE MODEL

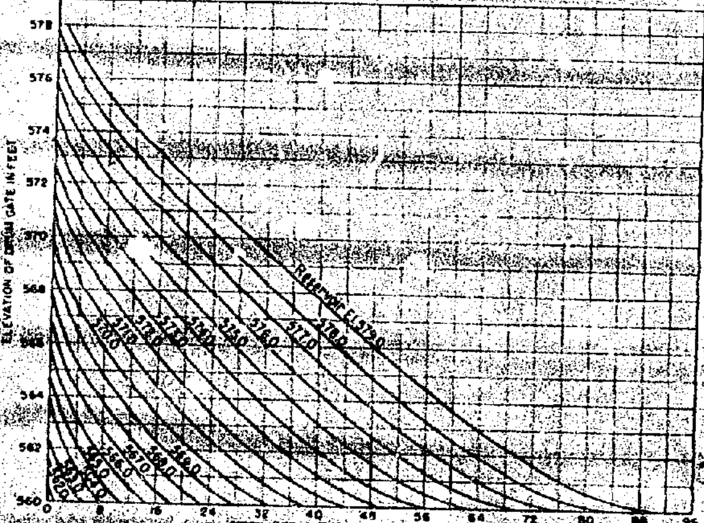
36



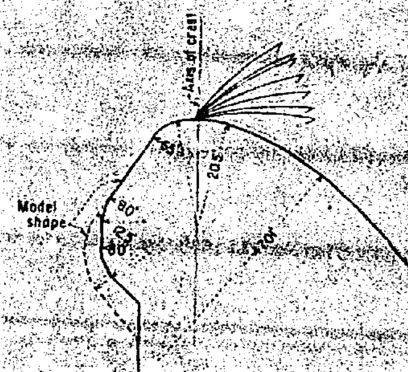
A. COEFFICIENT CURVES FOR CREST WITHOUT PIERS



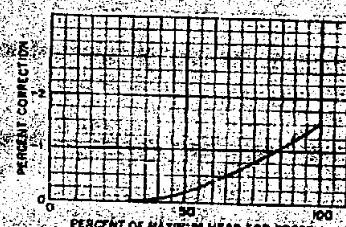
D. COEFFICIENT CURVES FOR CREST WITH PIERS



B. DISCHARGE DIAGRAM FOR THREE 100'x18' DRUM GATES



E. CREST SHAPE



C. PERCENT CORRECTION FOR COEFFICIENT CURVES FOR CREST WITH PIERS

NOTE

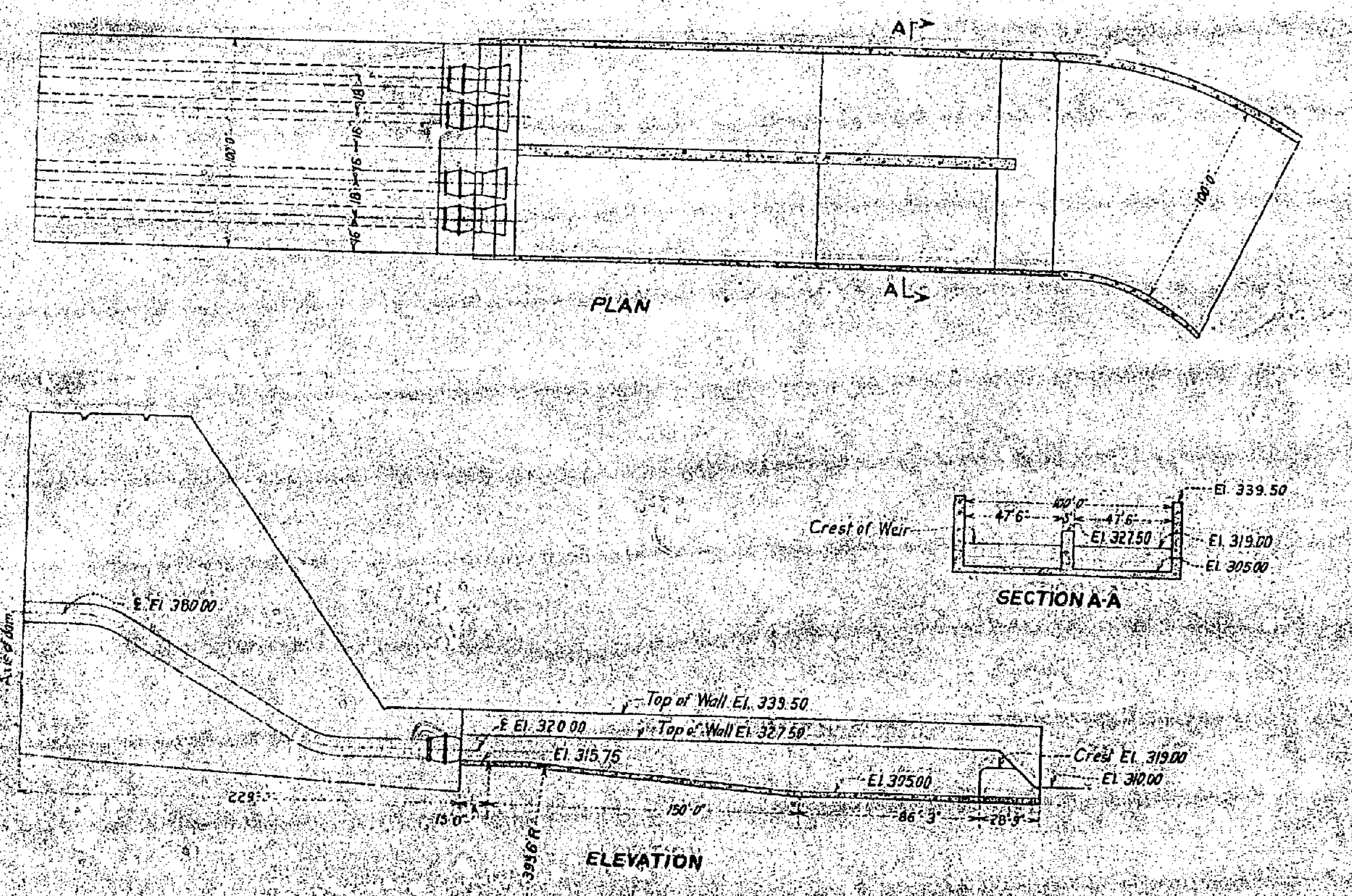
Head was measured from reservoir surface to high point of crest.

CENTRAL VALLEY PROJECT—CALIFORNIA

FRIANT DAM

SPILLWAY DISCHARGE DIAGRAM AND COEFFICIENTS

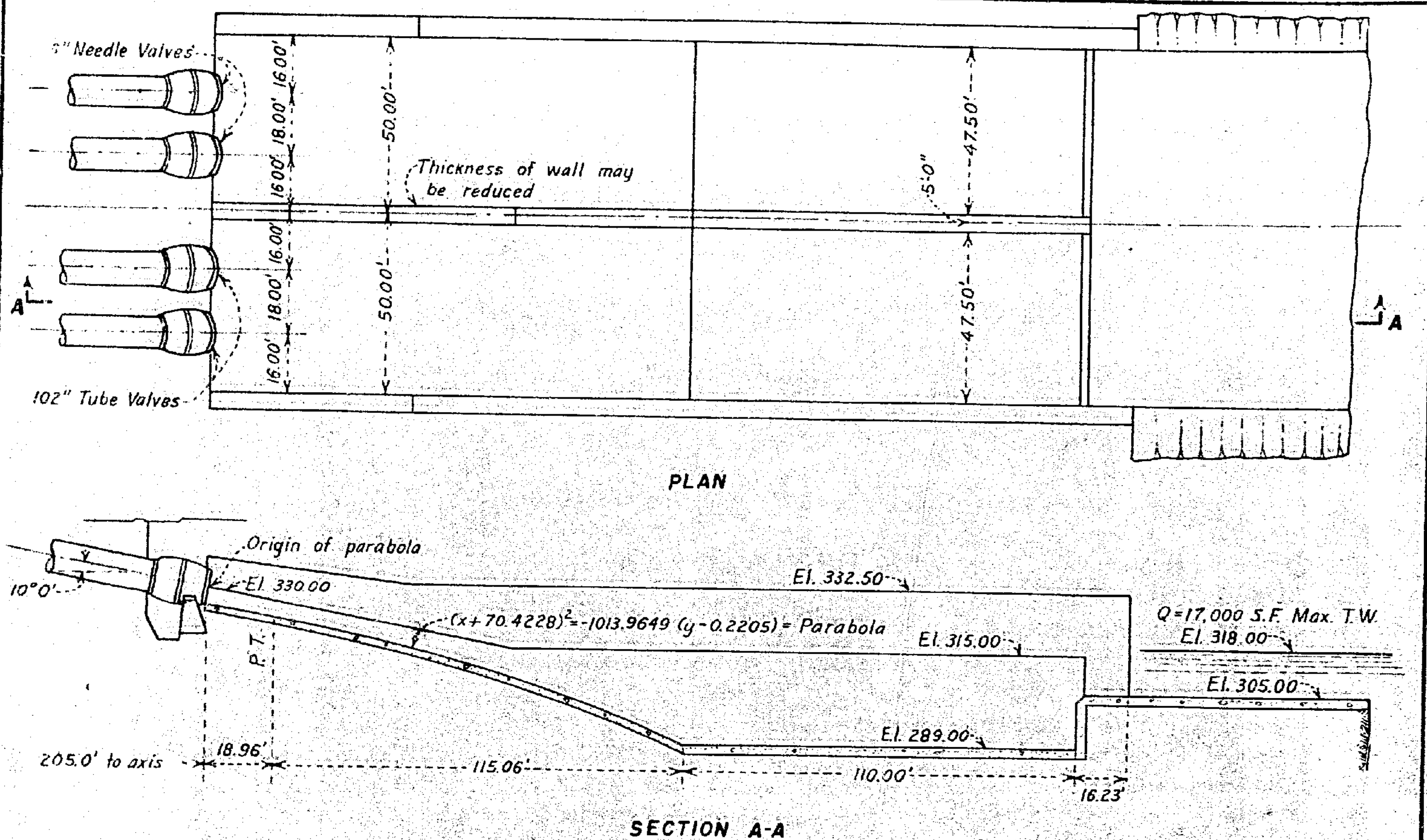
HYDRAULIC STUDIES ON A SCALE MODEL



CENTRAL VALLEY PROJECT
FRIANT RIVER OUTLETS
ORIGINAL DESIGN

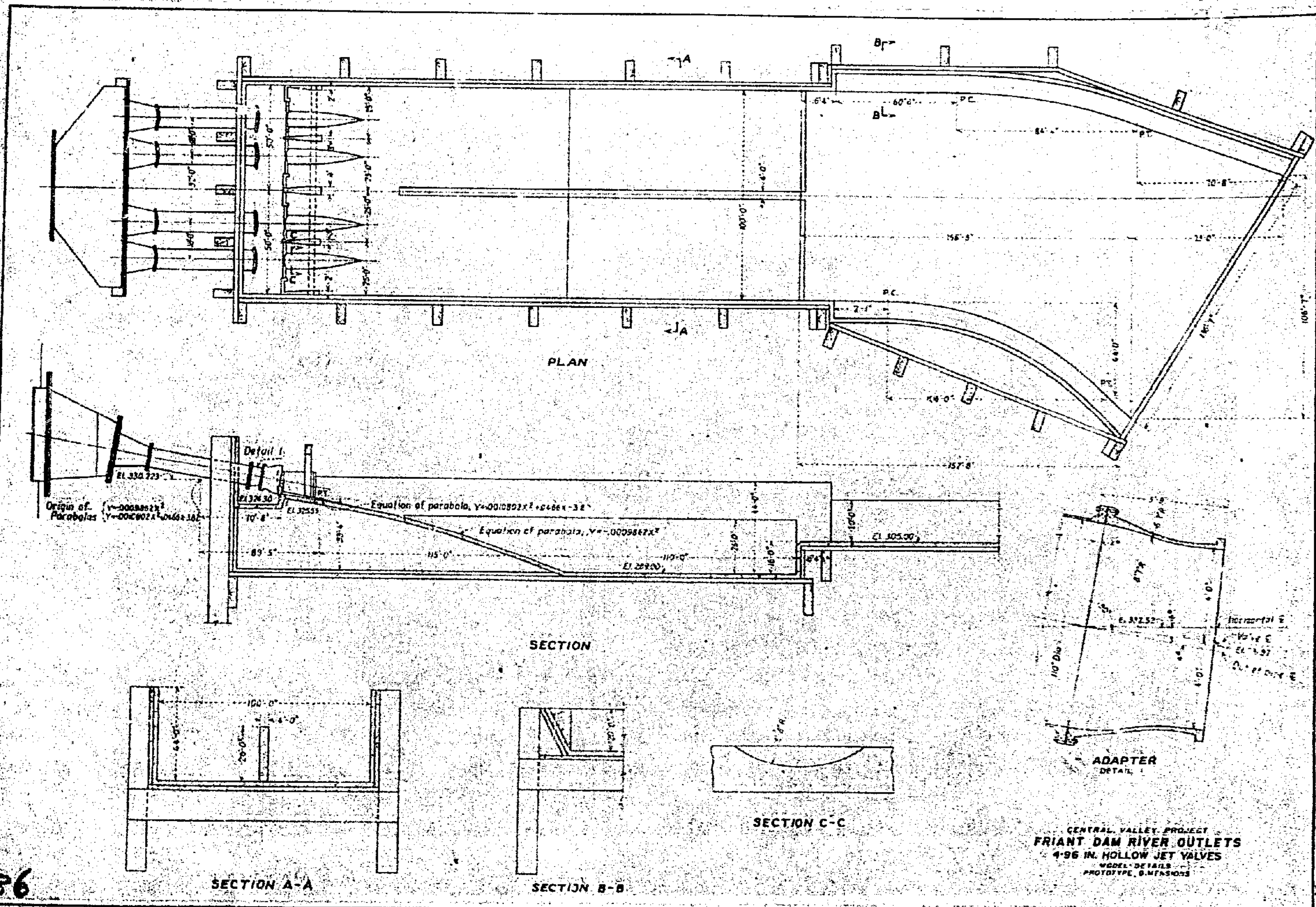
FIGURE 15

36



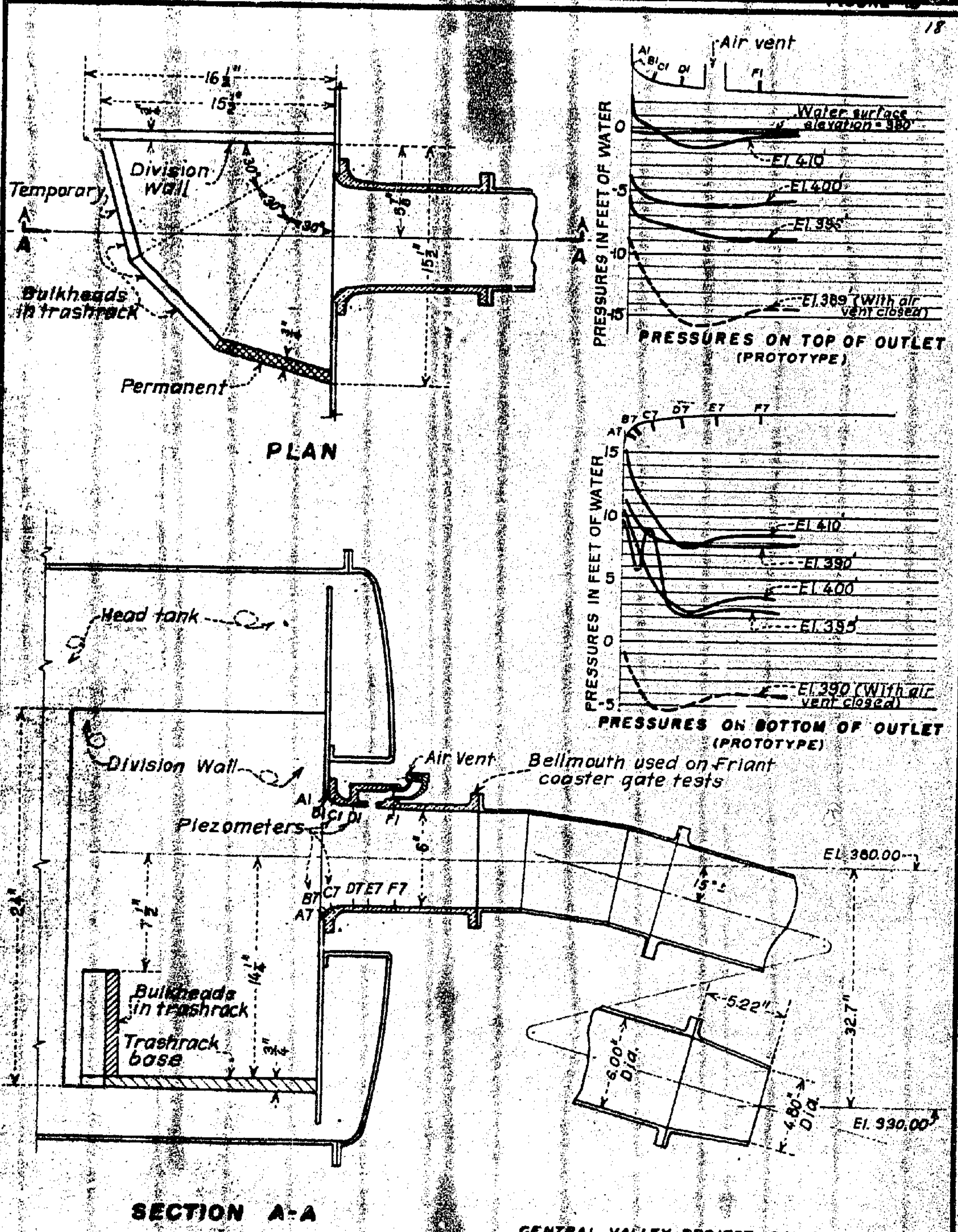
CENTRAL VALLEY PROJECT - CALIFORNIA
FRIANT DAM RIVER OUTLETS
APRON AND STILLING POOL DESIGN
1:34.38 SCALE MODEL

FIGURE 16

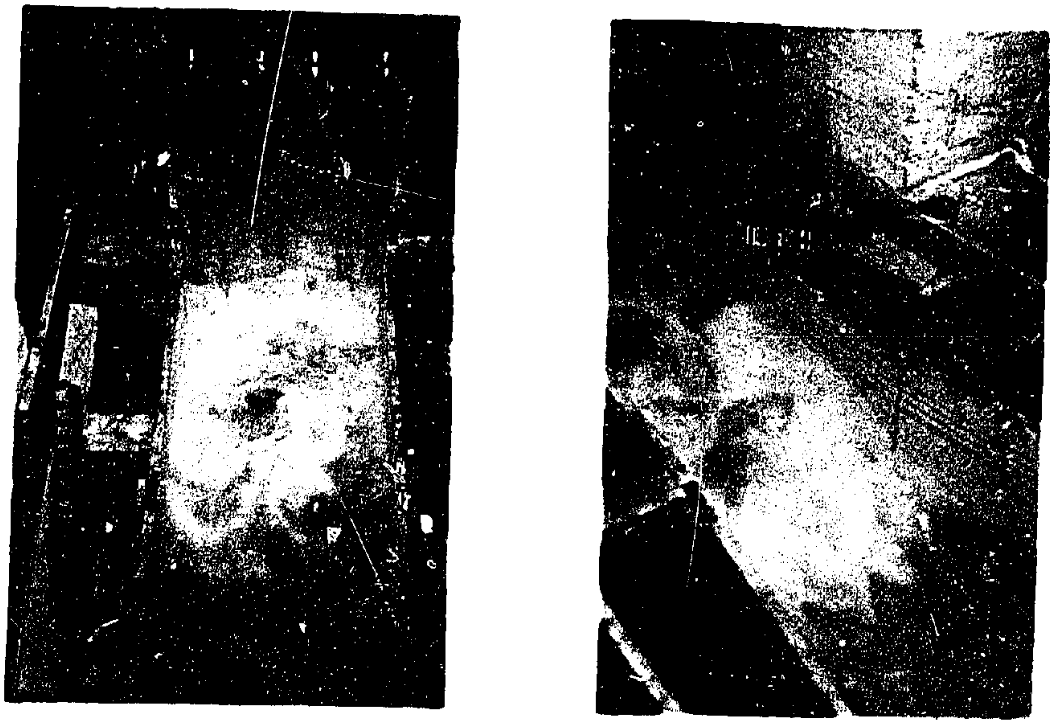


PLAN
SECTION
SECTION A-A
SECTION B-B
SECTION C-C
ADAPTER
DETAIL 1
Detail 1
Origin of Parabolas
Equation of parabola
CENTRAL VALLEY PROJECT
FRIANT DAM RIVER OUTLETS
4-96 IN. HOLLOW JET VALVES
MODEL DETAILS
PROTOTYPE DIMENSIONS
FIGURE 17

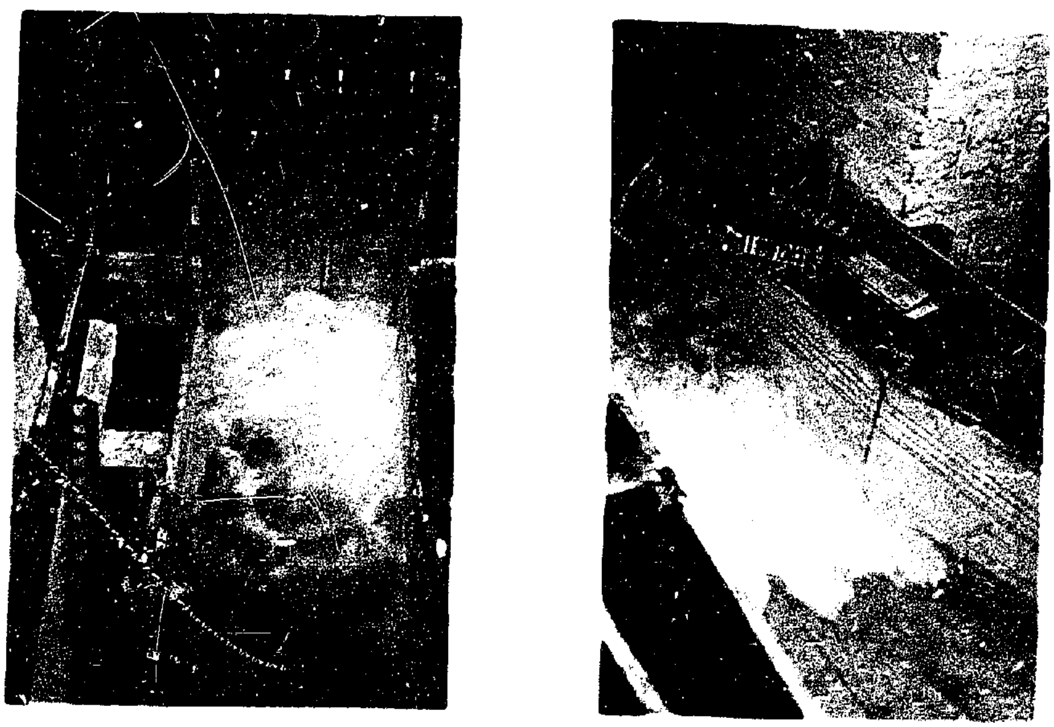
36



CENTRAL VALLEY PROJECT—CALIFORNIA
FRIANT DAM RIVER OUTLETS
EFFECT OF BULKHEADS IN TRASHRACKS
ON PRESSURES IN OUTLETS
1:8.33 SCALE MODEL

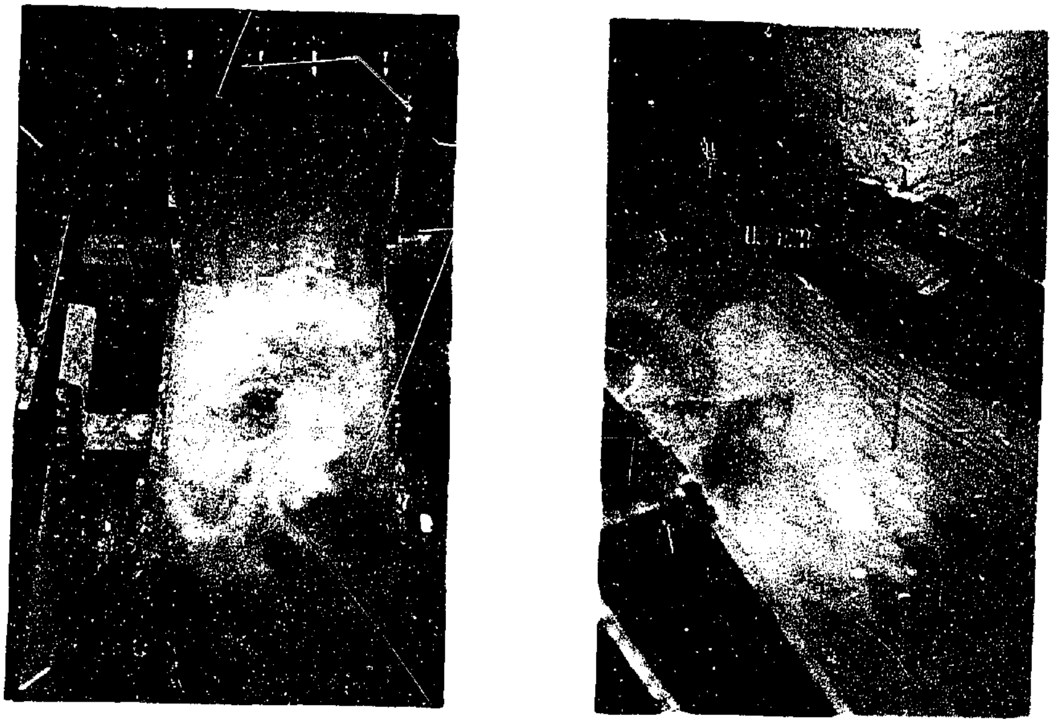


A. Discharge 3,440 second-feet. Reservoir elevation 563.25
Four valves 23 percent open.

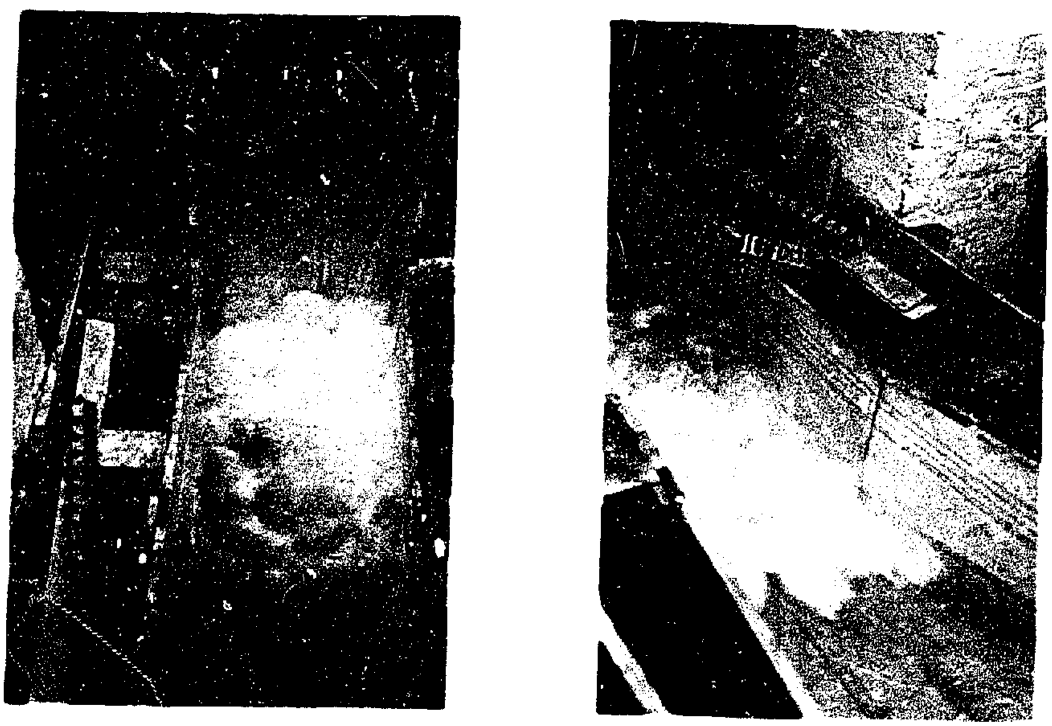


B. Discharge 3,300 second-feet. Reservoir elevation 563.00
Two valves 44 percent open.

FRIANT-KERN CANAL OUTLETS. NEEDLE VALVE INSTALLATION
Two valves and four valves operating.
1:34.29 Model



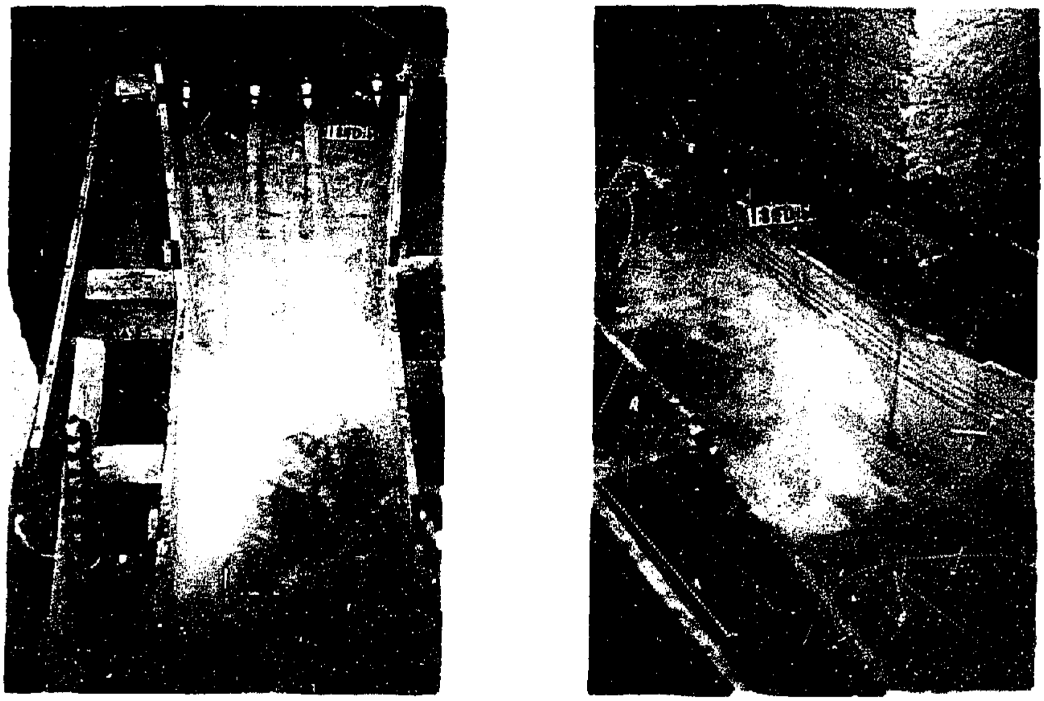
A. Discharge 3,440 second-feet. Reservoir elevation 563.25
Four valves 23 percent open.



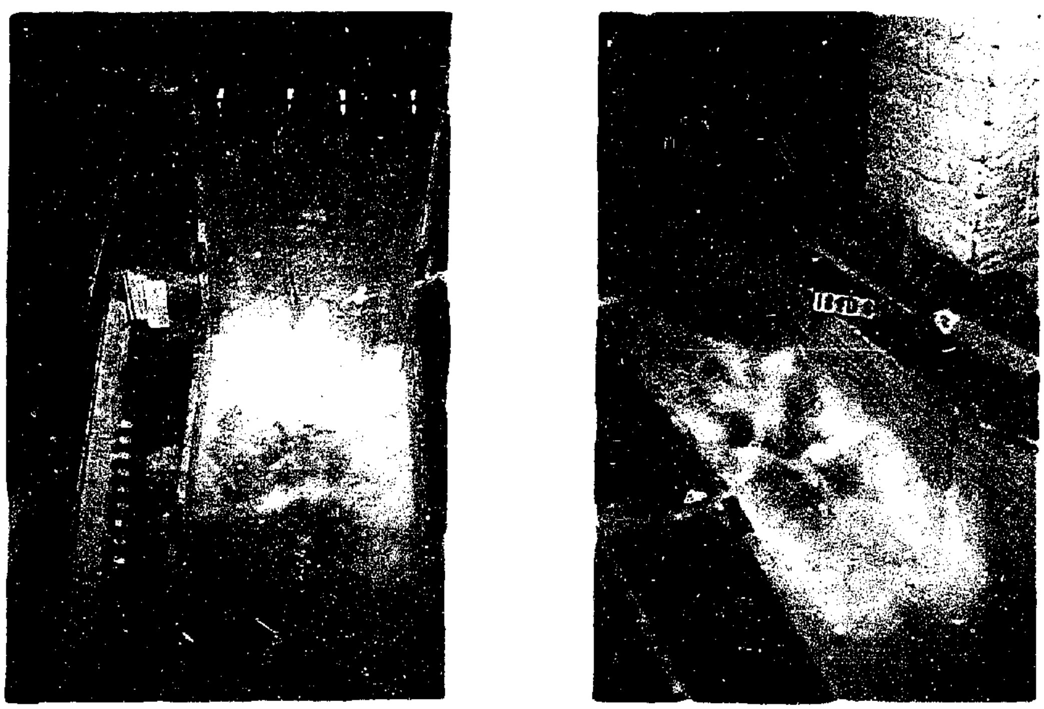
B. Discharge 3,300 second-feet. Reservoir elevation 563.00
Two valves 44 percent open.

FRIANT-KERN CANAL OUTLETS. NEEDLE VALVE INSTALLATION
Two valves and four valves operating.
1:34.29 Model

FIGURE 20

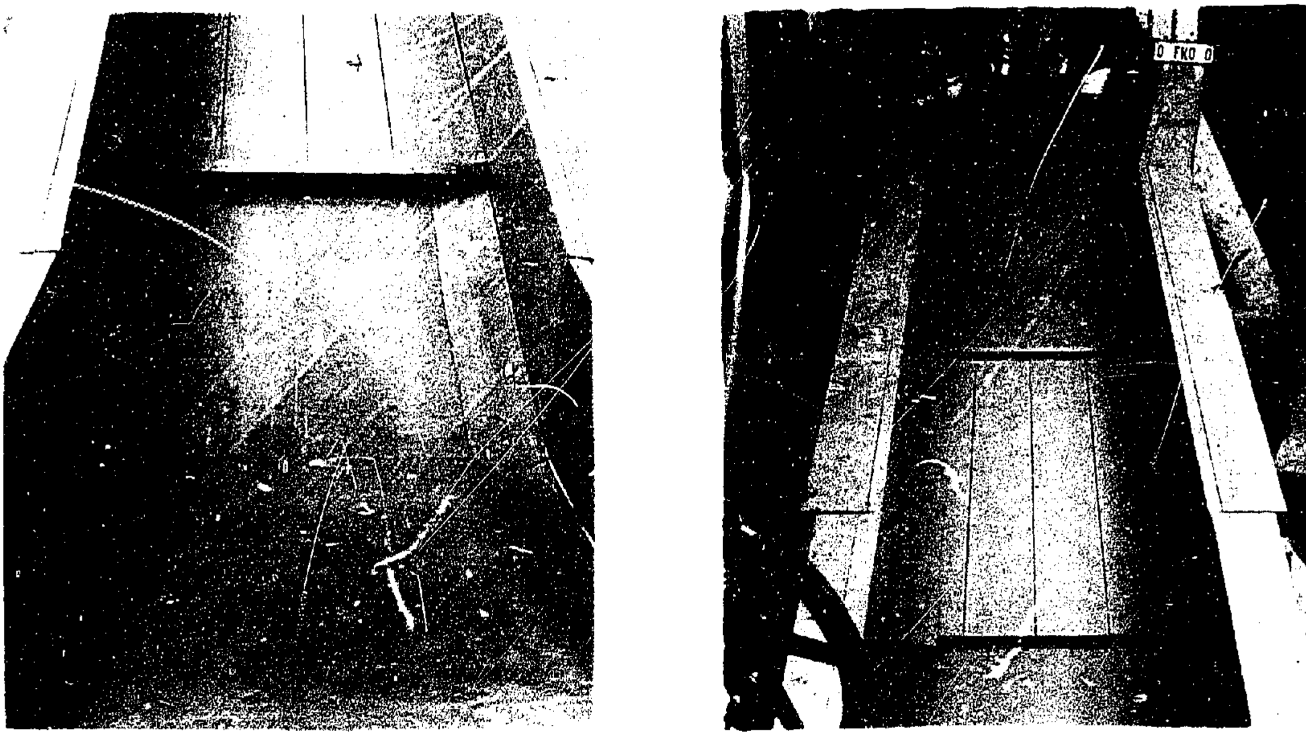


A. Discharge 3,595 second-feet. Reservoir elevation 563.25
Three valves 32 percent open.



B. Discharge 3,600 second-feet. Reservoir elevation 563.50
Three valves 32 percent open.

FRIANT-KERN CANAL OUTLETS. NEEDLE VALVE INSTALLATION
Three valves operating
1:34.29 Model



A. Model arrangement.

B. Discharge 3,500 s.f. Maximum head.
Tailwater elevation 466.65.

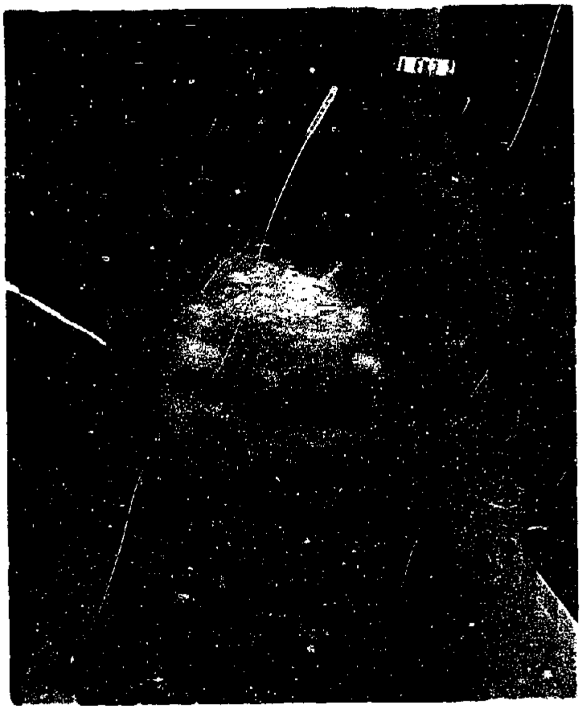
C. Discharge 3,500 s.f. Minimum head.
Tailwater elevation 466.65.

FRIANT-KERN CANAL OUTLETS WITH TWO NEEDLE AND TWO TUBE VALVES
1:34.38 Model

FIGURE 22

A. Discharge 3,500 s.f. Maximum head.
Tailwater elevation 466.65
Two valves operating

B. Discharge 3,500 s.f. Maximum head.
Tailwater elevation 466.65
Tube and needle valve operating

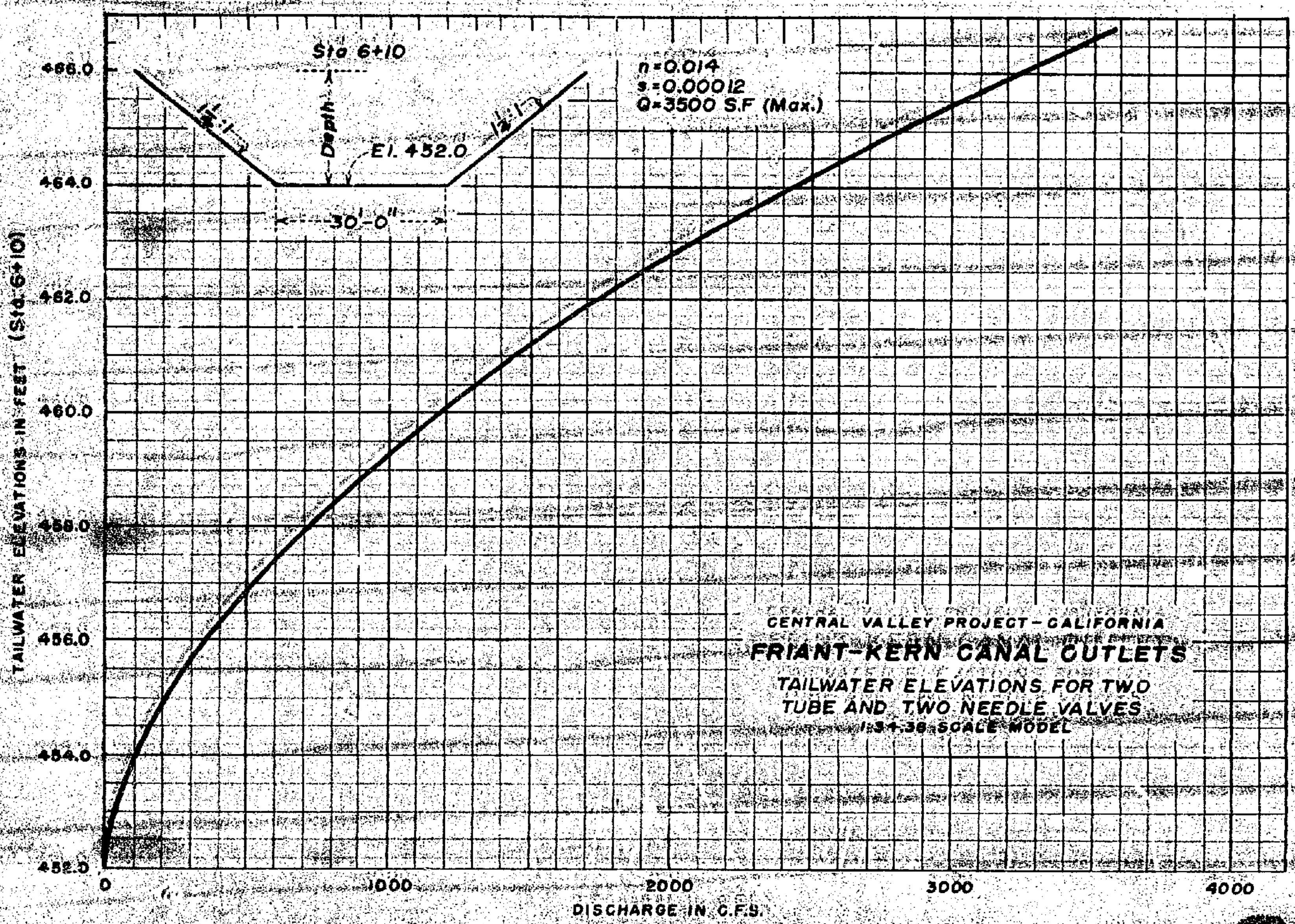


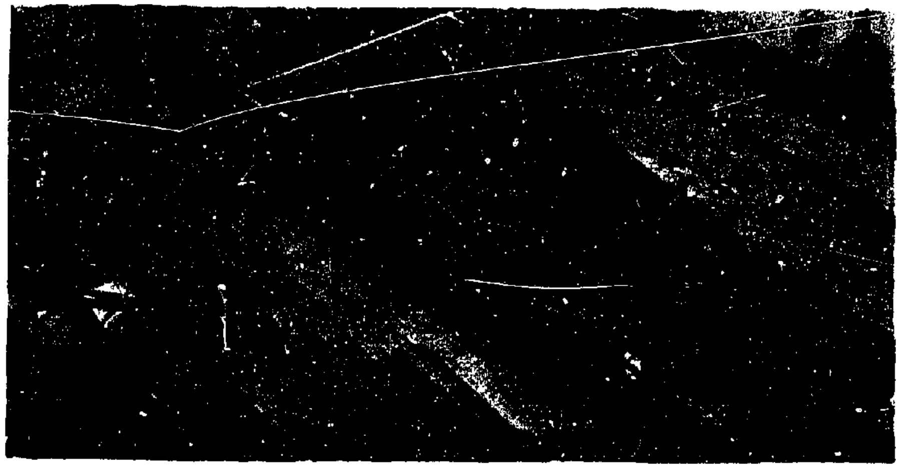
C. Discharge 3,500 s.f. Maximum head.
Tailwater elevation 466.65
Needle valves operating

D. Discharge 1,750 s.f. Maximum head.
Tailwater elevation 462.00
Tube and needle valve operating

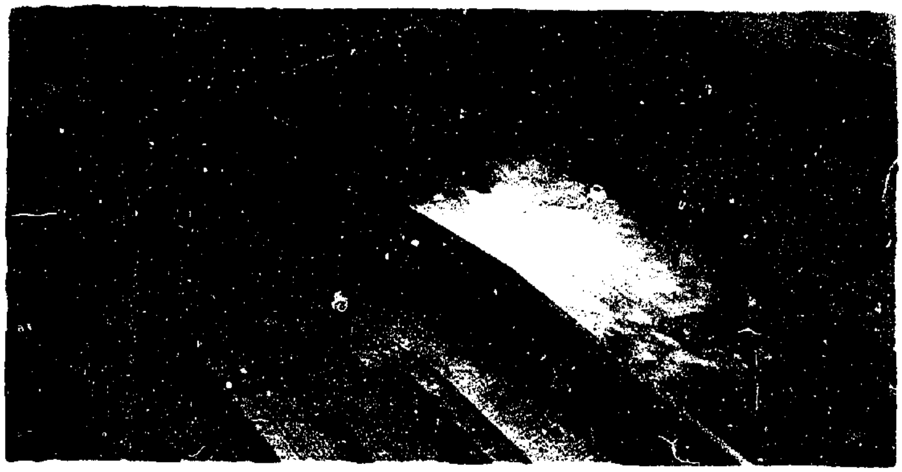
FRIANT-KERN CANAL OUTLETS WITH TWO TUBE AND TWO NEEDLE VALVES
1:34.38 Model

FIGURE 2

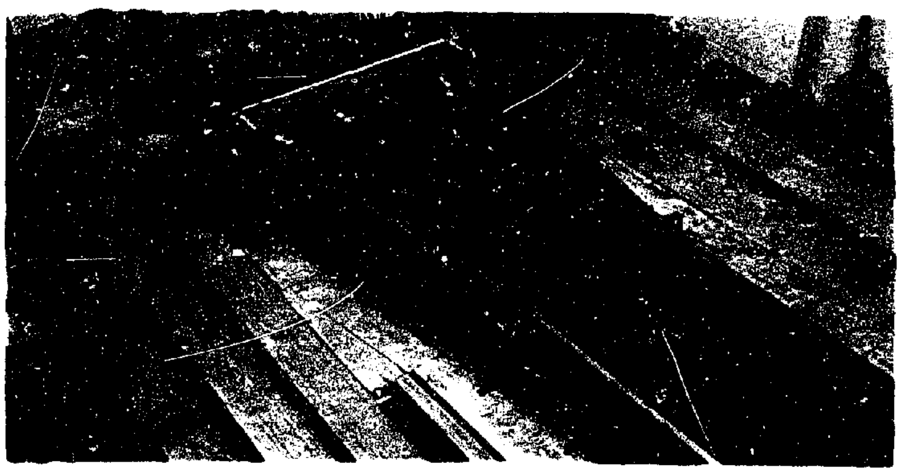

Sta 6+10
Depth
El. 452.0
1½:1
1½:1
30'-0"
n=0.014
s=0.00012
Q=3500 S.F (Max.)
466.0
464.0
462.0
460.0
458.0
456.0
454.0
452.0
TAILWATER ELEVATIONS IN FEET (Sta 6+10)
0
1000
2000
3000
4000
DISCHARGE IN C.F.S.
CENTRAL VALLEY PROJECT - CALIFORNIA
FRIANT-KERN CANAL OUTLETS
TAILWATER ELEVATIONS FOR TWO
TUBE AND TWO NEEDLE VALVES
1:34.38 SCALE MODEL



A. Model arrangement. Pool designed for 3,500 second-feet.



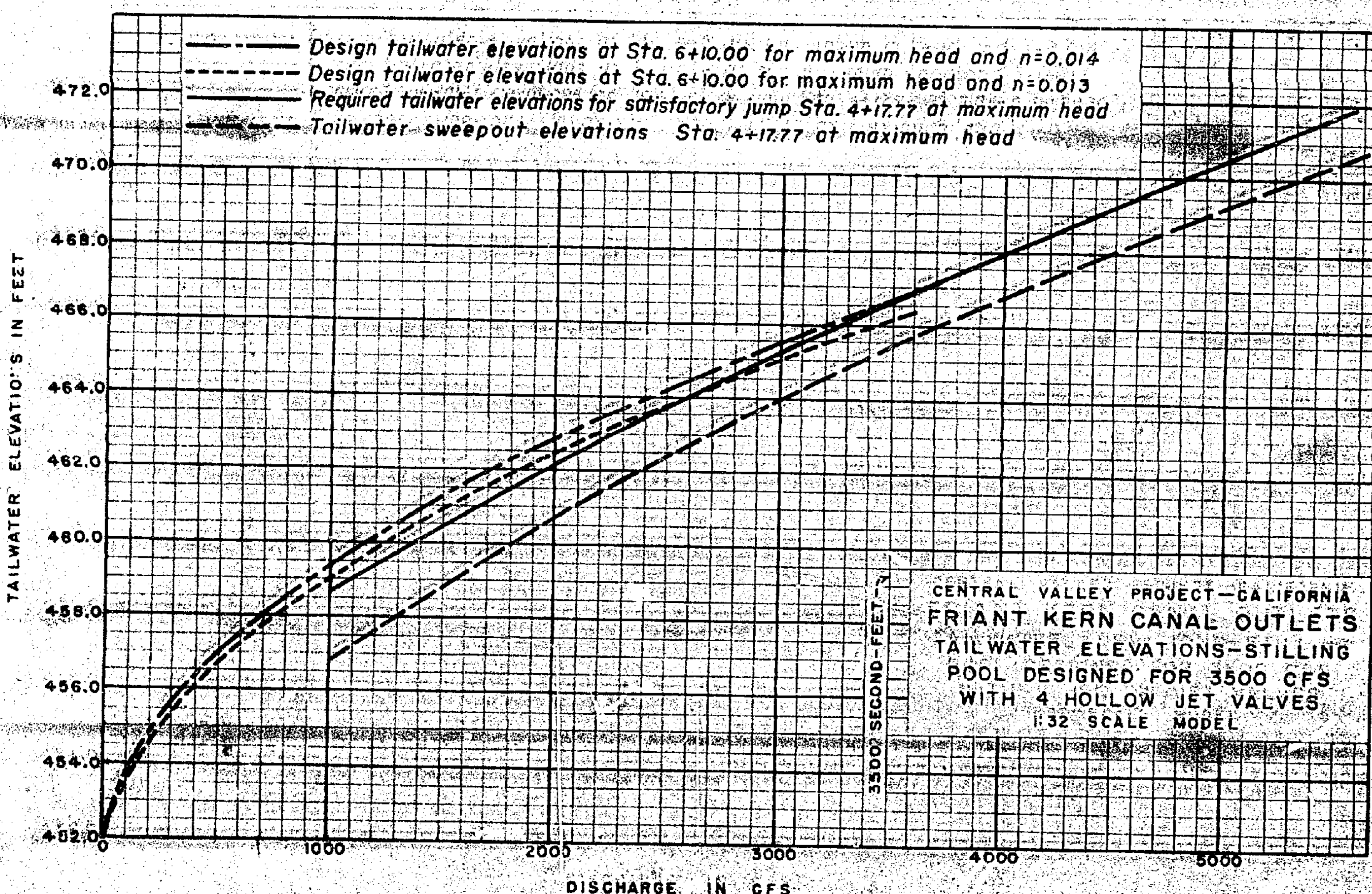
B. Discharge 3,500 second-feet. Reservoir elev. 578.0.
Tailwater elev. 466.65. Four valves 26 percent open.

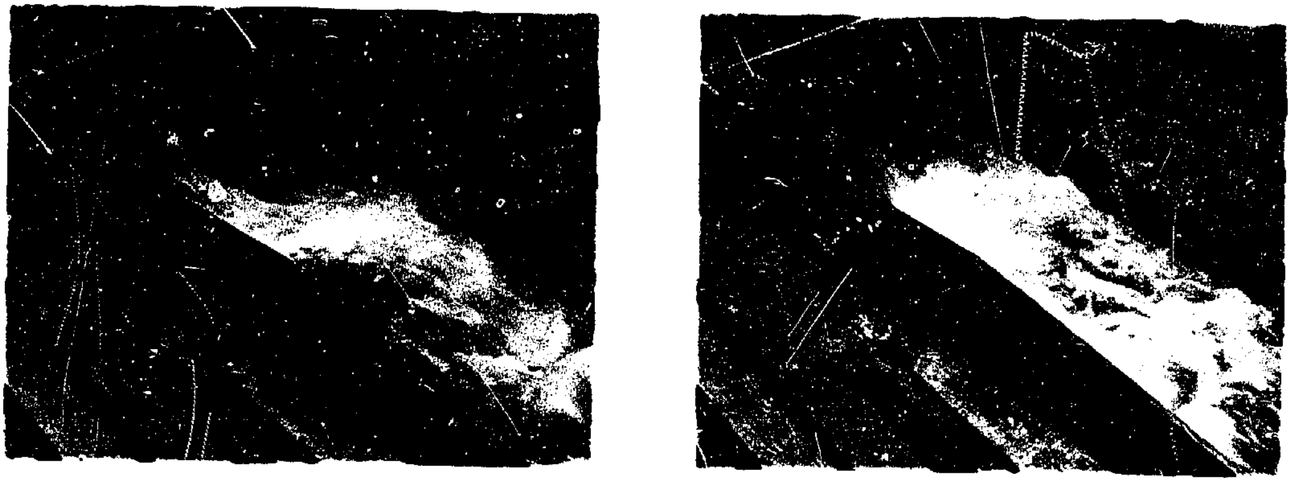


C. Discharge 3,500 second-feet. Reservoir elev. 479.0.
Tailwater elev. 466.65. Four valves 100 percent open.

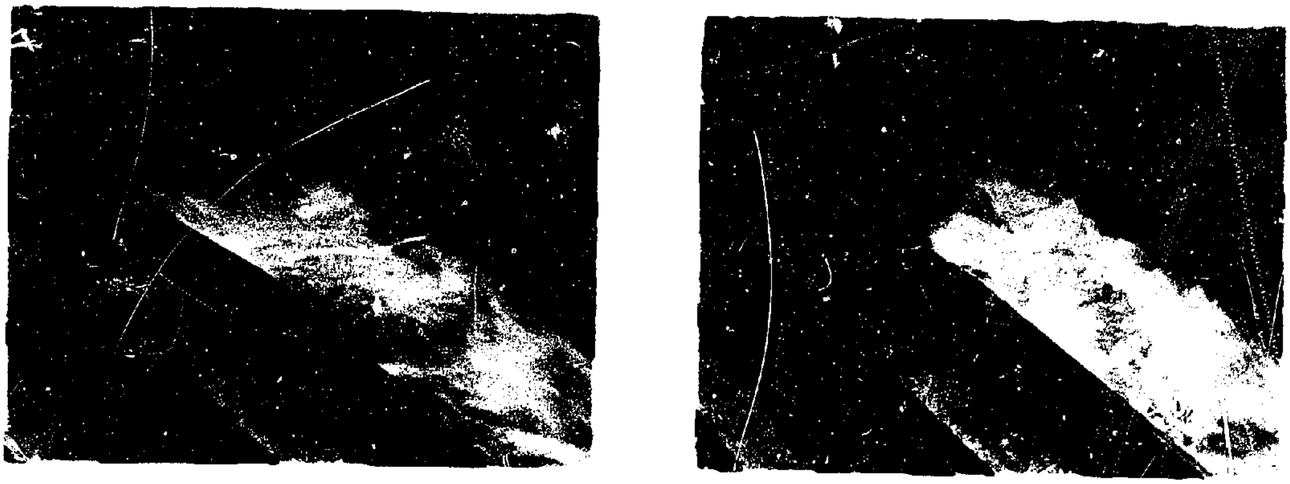
FRIANT-KERN CANAL OUTLETS WITH FOUR HOLLOW-JET VALVES

1:32 Model

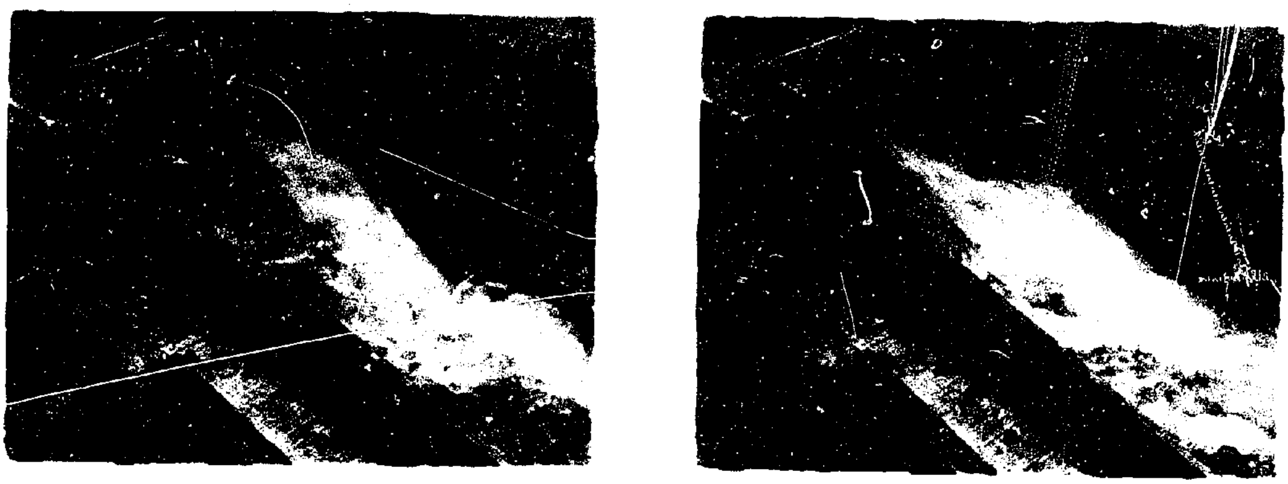

Design tailwater elevations at Sta. 6+10.00 for maximum head and n=0.014
Design tailwater elevations at Sta. 6+10.00 for maximum head and n=0.013
Required tailwater elevations for satisfactory jump Sta. 4+17.77 at maximum head
Tailwater sweepout elevations Sta. 4+17.77 at maximum head
TAILWATER ELEVATIONS IN FEET
472.0
470.0
468.0
466.0
464.0
462.0
460.0
458.0
456.0
454.0
452.0
0
1000
2000
3000
4000
5000
DISCHARGE IN CFS
3500 SECOND-FEET
CENTRAL VALLEY PROJECT—CALIFORNIA
FRIANT KERN CANAL OUTLETS
TAILWATER ELEVATIONS—STILLING POOL DESIGNED FOR 3500 CFS WITH 4 HOLLOW JET VALVES
1:32 SCALE MODEL



A. Discharge 3,500 second-feet. Reservoir elev. 578.0. Tailwater elev. 466.65.
Three valves 100 percent open.

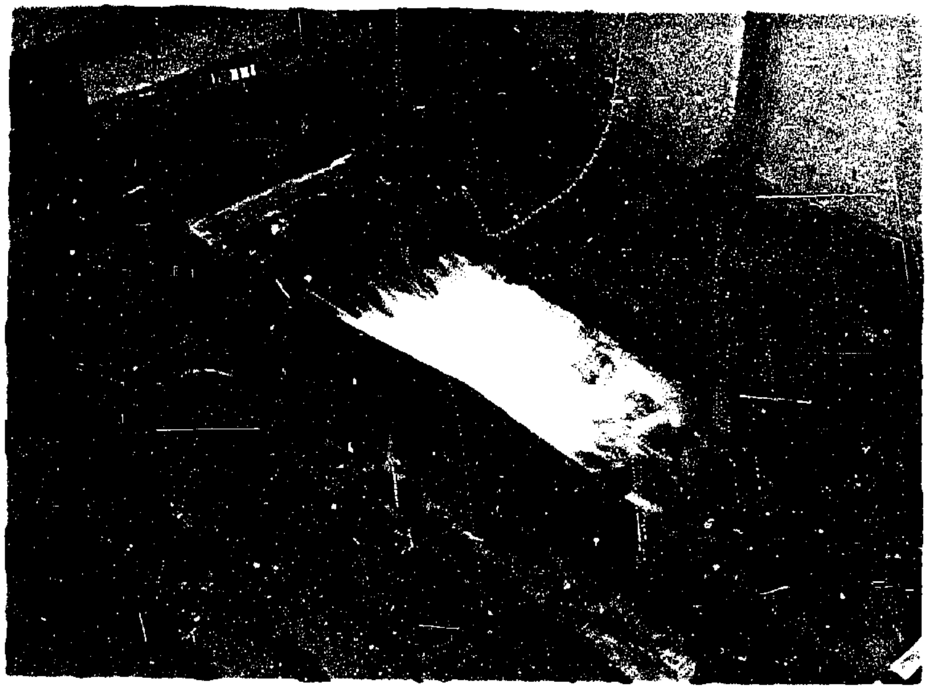


B. Discharge 3,500 second-feet. Reservoir elev. 578.0. Tailwater elev. 466.65.
Two valves 100 percent open.



C. Discharge 2,900 second-feet. Reservoir elev. 578.0. Tailwater elev. 466.65.
One valve 100 percent open.

FRIANT-KERN CANAL OUTLETS WITH FOUR HOLLOW-JET VALVES
Pool designed for 3,500 second-feet.
1:32 Model



A. Discharge 4,500 second-feet. Reservoir elev. 578.0. Tailwater elev. 466.65. Four valves 27 percent open.

B. Discharge 4,500 second-feet. Reservoir elev. 479.0. Tailwater elev. 466.65. Four valves 100 percent open.

FRIANT-KERN CANAL OUTLETS WITH FOUR HOLLOW-JET VALVES
Pool designed for 4,500 second-feet.
1:32 Model

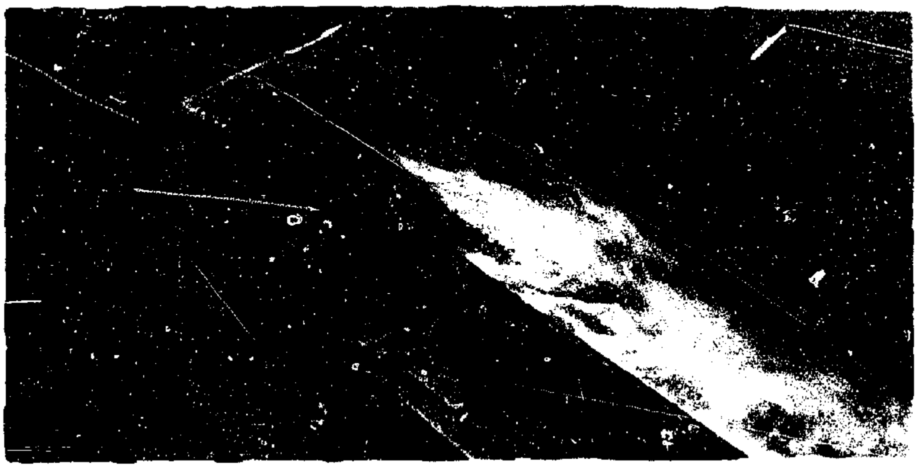
A. Discharge 4,500 second-feet. Reservoir elev. 578.0. Tailwater elev. 466.65. Three valves 33 percent open.

B. Discharge 4,500 second-feet. Reservoir elev. 578.0. Tailwater elev. 466.65. Three valves 33 percent open.

FRIANT-KERN CANAL OUTLETS WITH FOUR HOLLOW-JET VALVES
Pool designed for 4,500 second-feet.
1:32 Model

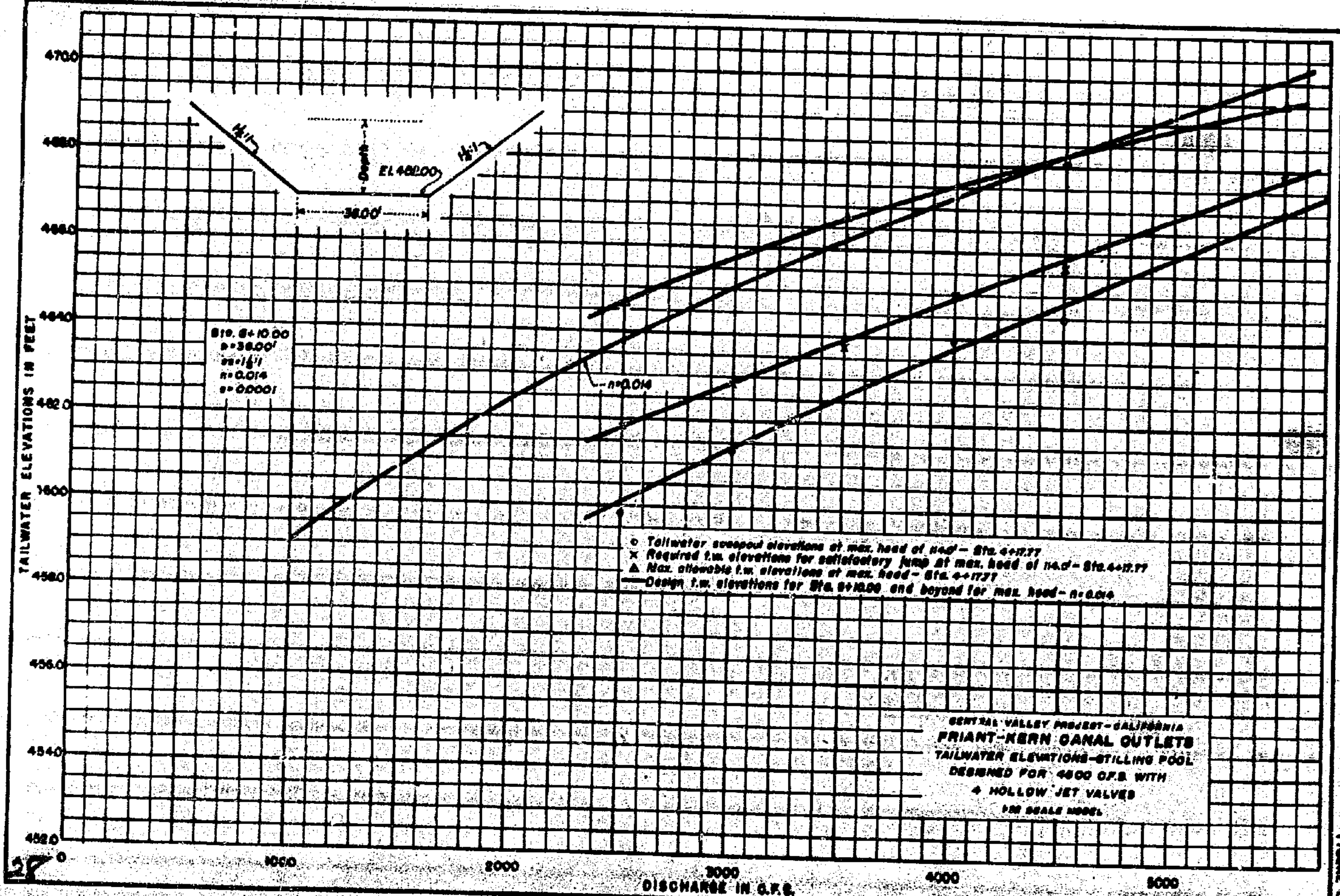
A. Discharge 4,500 second-feet. Reservoir elev. 578.0. Tailwater elev. 466.65. Two valves 51 percent open.

B. Discharge 4,500 second-feet. Reservoir elev. 578.0. Tailwater elev. 466.65. Two valves 51 percent open.



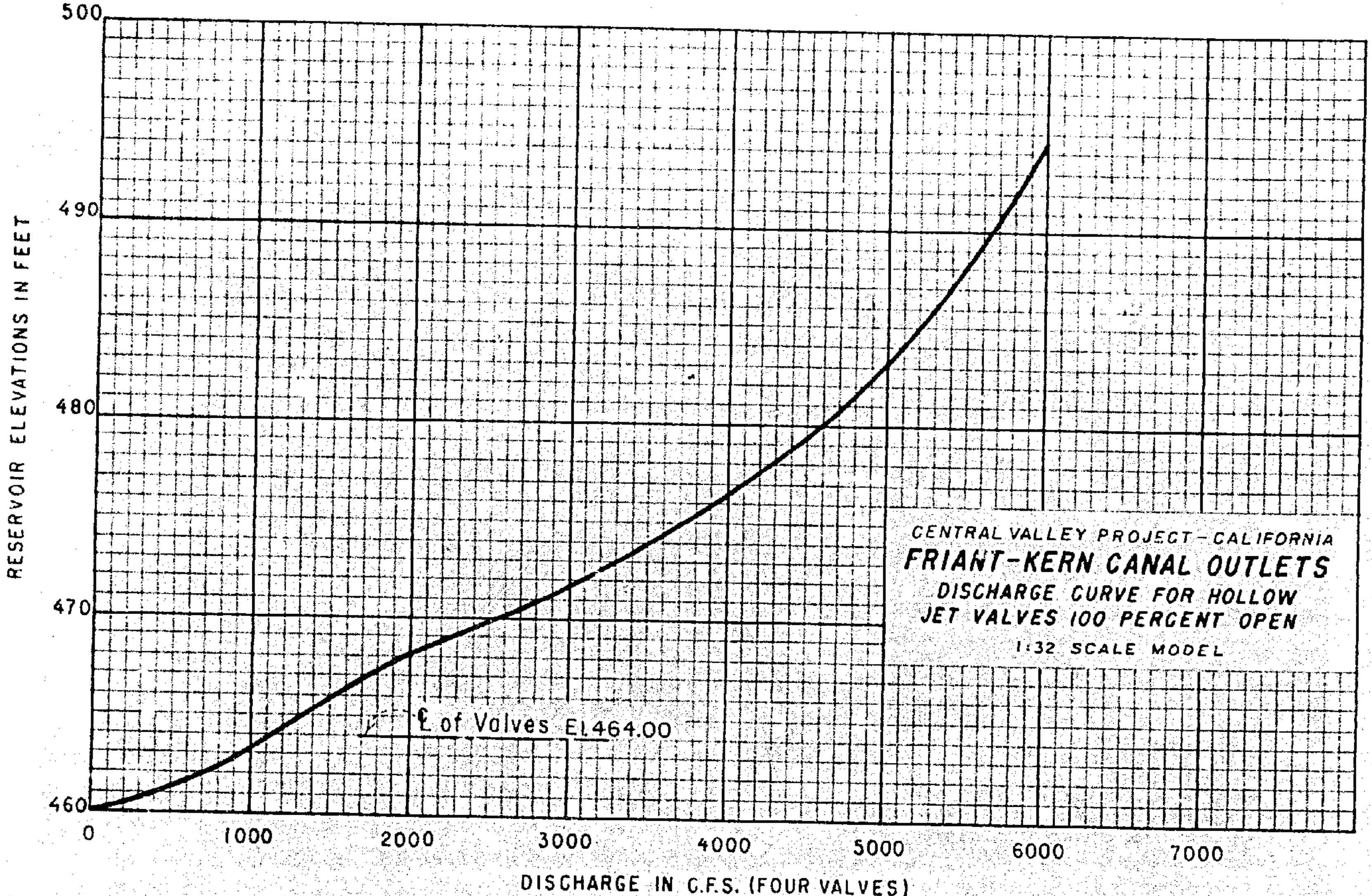
C. Discharge 4,500 second-feet. Reservoir elev. 578.0. Tailwater elev. 466.65. Two valves 48 percent open.

FRIANT-KERN CANAL OUTLETS WITH FOUR HOLLOW-JET VALVES
Pool designed for 4,500 second-feet.
1:32 Model

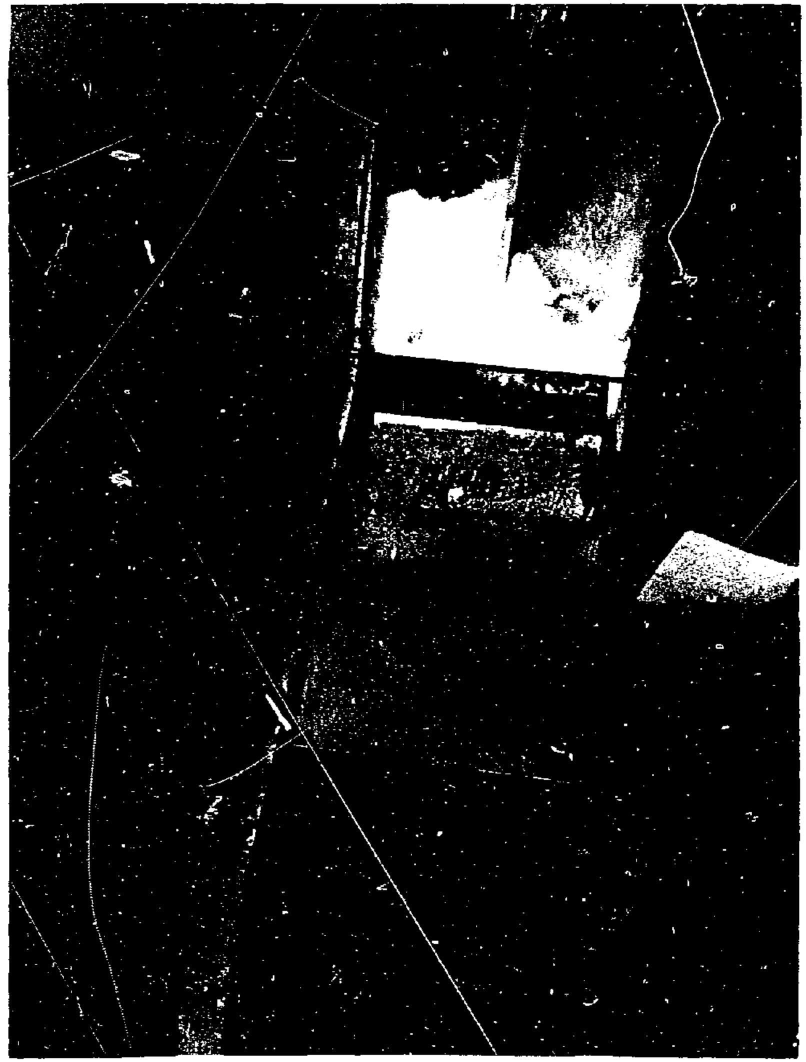


TAILWATER ELEVATIONS IN FEET
DISCHARGE IN C.F.S.
0
1000
2000
3000
4000
5000
El. 4620.0
Depth
38.00'
1½:1
1½:1
Sta. 6+10.00
b=38.00'
ss=1½:1
n=0.014
s=0.0001
n=0.014
o Tailwater scourpool elevations at max. head of 114.0' - Sta. 4+17.77
x Required t.w. elevations for satisfactory jump at max. head of 114.0' - Sta. 4+17.77
▲ Max. allowable t.w. elevations at max. head - Sta. 4+17.77
—— Design t.w. elevations for Sta. 6+10.00 and beyond for max. head - n=0.014
CENTRAL VALLEY PROJECT - CALIFORNIA
FRIANT-KERN CANAL OUTLETS
TAILWATER ELEVATIONS - STILLING POOL
DESIGNED FOR 4600 C.F.S. WITH
4 HOLLOW JET VALVES
1:20 SCALE MODEL

FIGURE 32


CENTRAL VALLEY PROJECT - CALIFORNIA
FRIANT-KERN CANAL OUTLETS
DISCHARGE CURVE FOR HOLLOW
JET VALVES 100 PERCENT OPEN
1:32 SCALE MODEL
RESERVOIR ELEVATIONS IN FEET
DISCHARGE IN C.F.S. (FOUR VALVES)
℄ of Valves El.464.00
500
490
480
470
460
0
1000
2000
3000
4000
5000
6000
7000

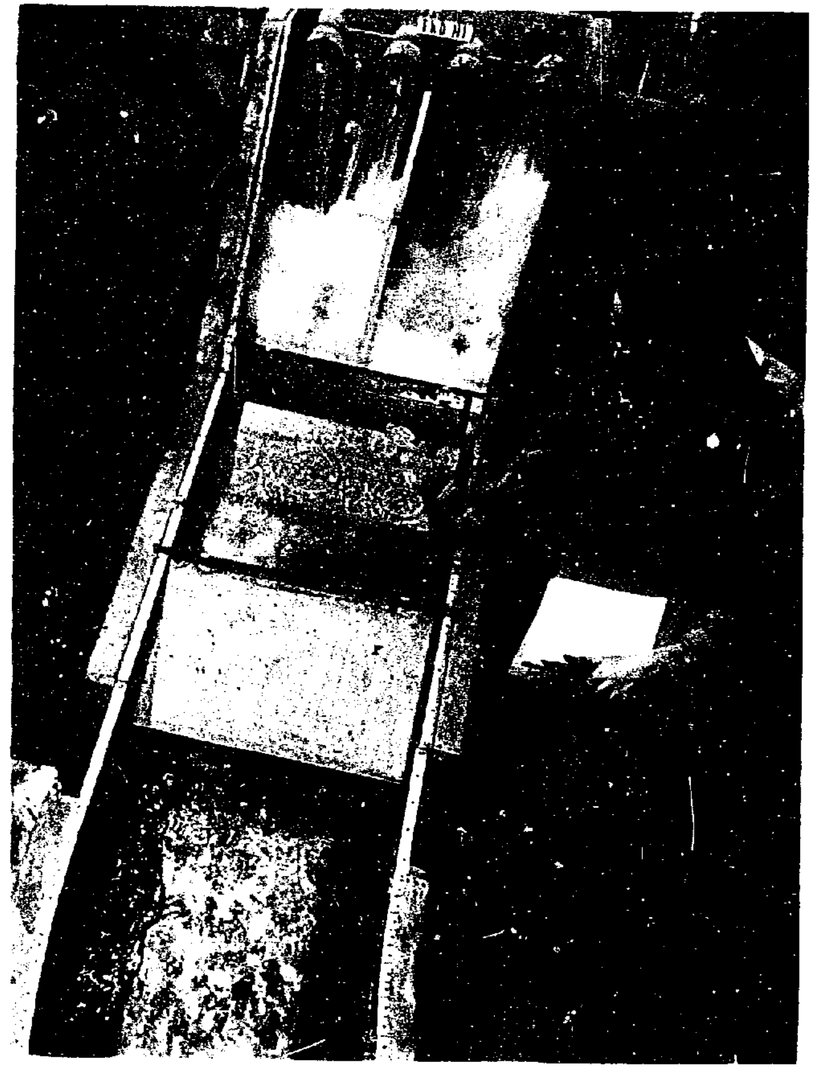
B - Discharge 5,000 s.f. through three valves. Res. elev. 578.0 baffles in place.



A - Discharge 5,000 s.f. through four valves. Res. elev. 578.0 baffles in place.

FRIANT-KERN CANAL OUTLETS WITH FOUR HOLLOW-JET VALVES.
POOL DESIGNED FOR 5,000 S.F., SHOWING BAFFLE ARRANGEMENT.
1:32 MODEL

A - Discharge 4,000 s.f. through four valves. Res. elev. 578.0 baffles



B - Discharge 4,000 s.f. through three valves. Res. elev. 578.0 baffles in place.

FRIANT-KERN CANAL OUTLETS WITH FOUR HOLLOW-JET VALVES.
POOL DESIGNED FOR 5,000 S.F., SHOWING BAFFLE ARRANGEMENT.
1:32 MODEL

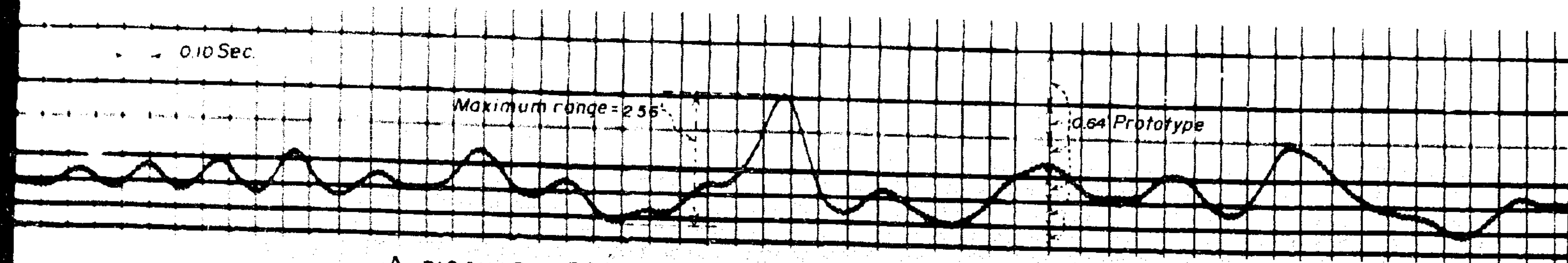
A - Discharge 5,000 s.f. through four valves. Res. elev. 578.0. No baffles in place.

B - Discharge 5,000 s.f. through four valves. Res. elev. 578.0. Two baffles in place.

C - Discharge 4,000 s.f. through three valves. Res. elev. 578.0. No baffles in place.

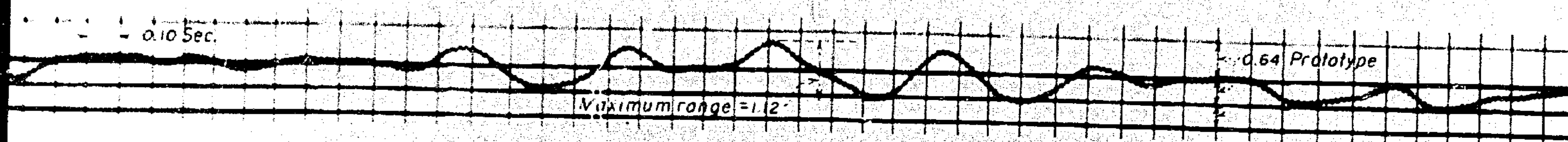
D - Discharge 4,000 s.f. through three valves. Res. elev. 578.0. Two baffles in place.

FRIANT-KERN CANAL OUTLETS WITH FOUR HOLLOW-JET VALVES.
POOL DESIGNED FOR 5,000 S.F., WAVE ACTION DOWNSTREAM FROM BAFFLES
1:32 MODEL

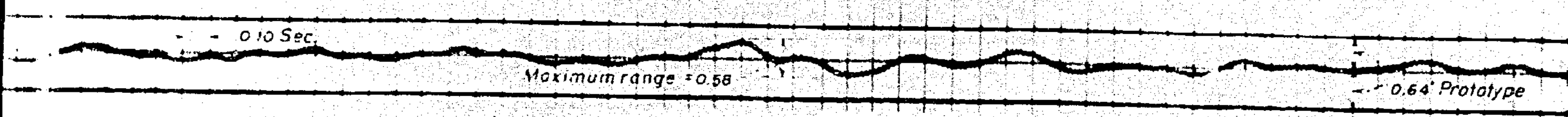


A. DISCHARGE 3000 S.F. THROUGH THREE VALVES-NO BAFFLING

B. DISCHARGE 3000 S.F. THROUGH THREE VALVES-TWO BAFFLES



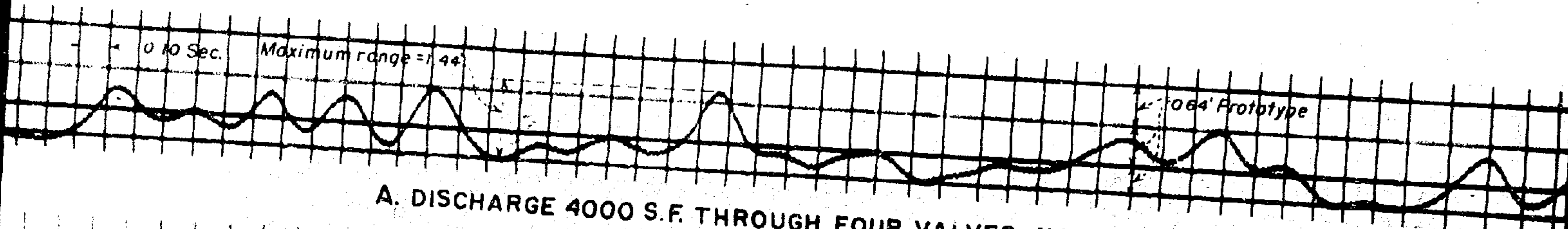
C. DISCHARGE 3000 S.F. THROUGH FOUR VALVES-NO BAFFLING



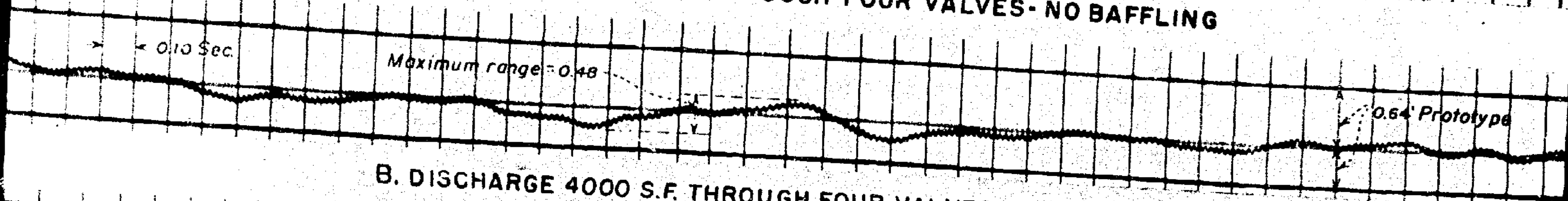
D. DISCHARGE 3000 S.F. THROUGH FOUR VALVES-TWO BAFFLES

RELATIVE ROUGHNESS OF WATER SURFACE BELOW STA. 6+10.0-DISCHARGE 3000 S.F.

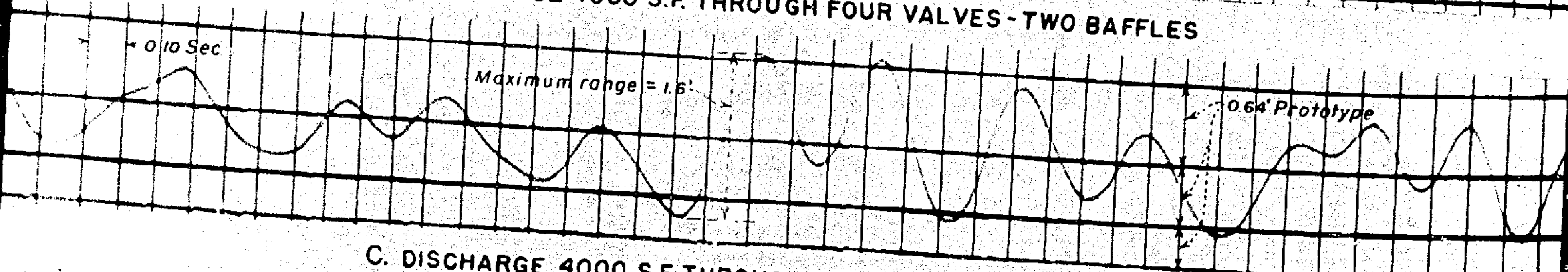
FRIANT-KERN CANAL OUTLETS



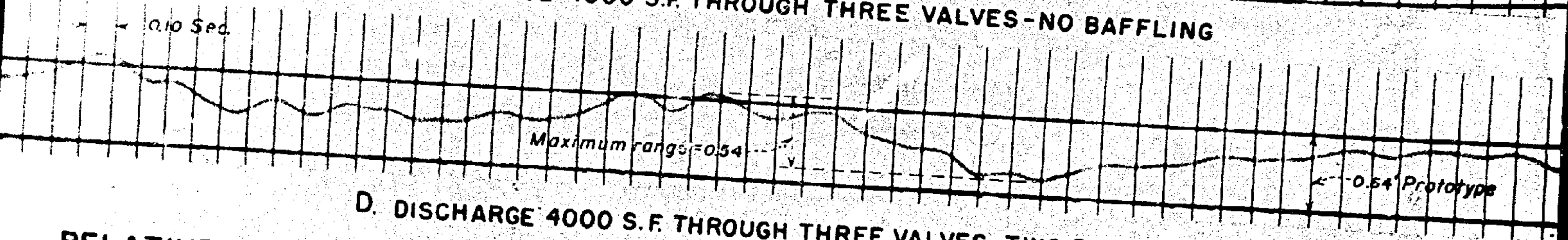
A. DISCHARGE 4000 S.F. THROUGH FOUR VALVES- NO BAFFLING



B. DISCHARGE 4000 S.F. THROUGH FOUR VALVES - TWO BAFFLES



C. DISCHARGE 4000 S.F. THROUGH THREE VALVES - NO BAFFLING

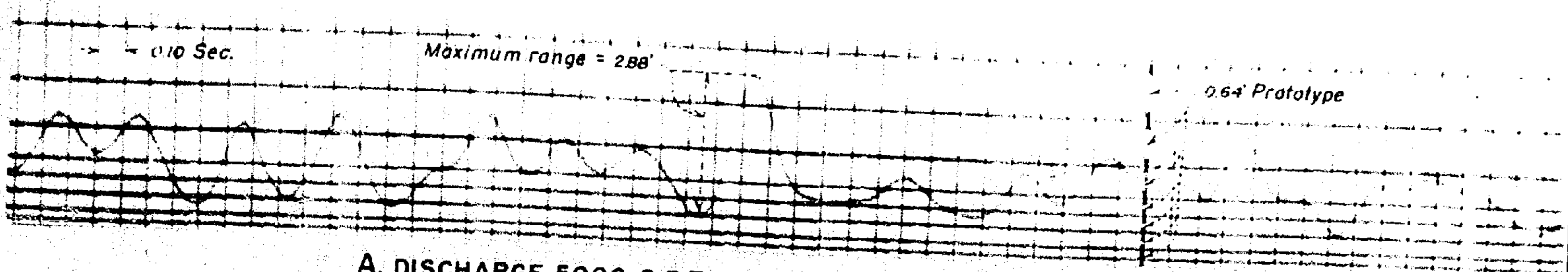


D. DISCHARGE 4000 S.F. THROUGH THREE VALVES - TWO BAFFLES

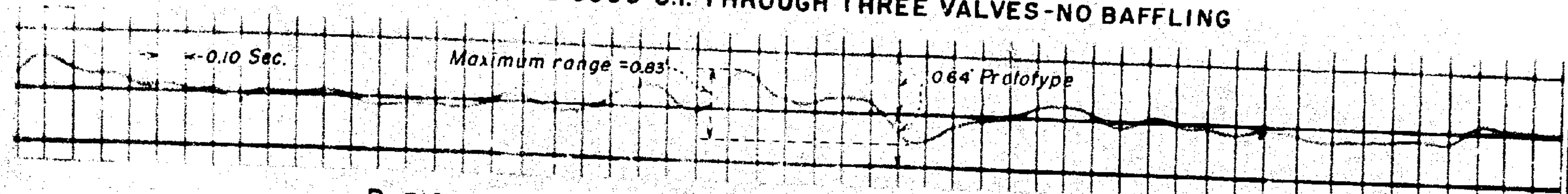
RELATIVE ROUGHNESS OF WATER SURFACE BELOW STA. 6+10.0 - DISCHARGE 4000 S.F.

FRIANT-KERN CANAL OUTLETS

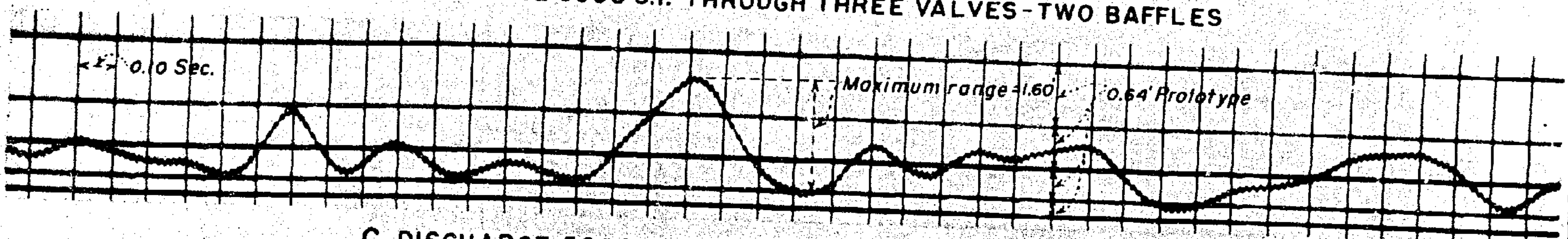
FIGURE



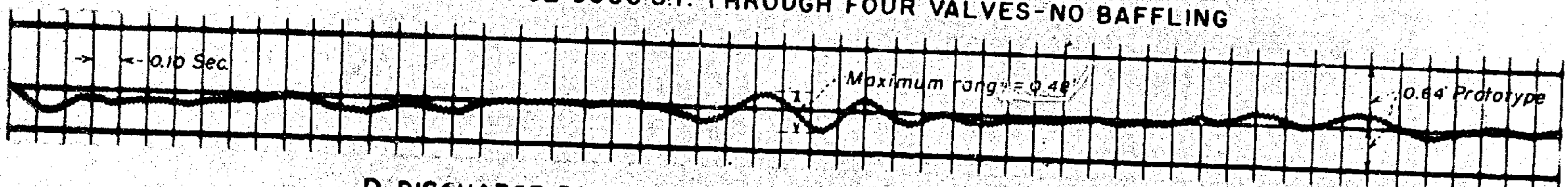
A. DISCHARGE 5000 S.F. THROUGH THREE VALVES-NO BAFFLING



B. DISCHARGE 5000 S.F. THROUGH THREE VALVES-TWO BAFFLES



C. DISCHARGE 5000 S.F. THROUGH FOUR VALVES-NO BAFFLING

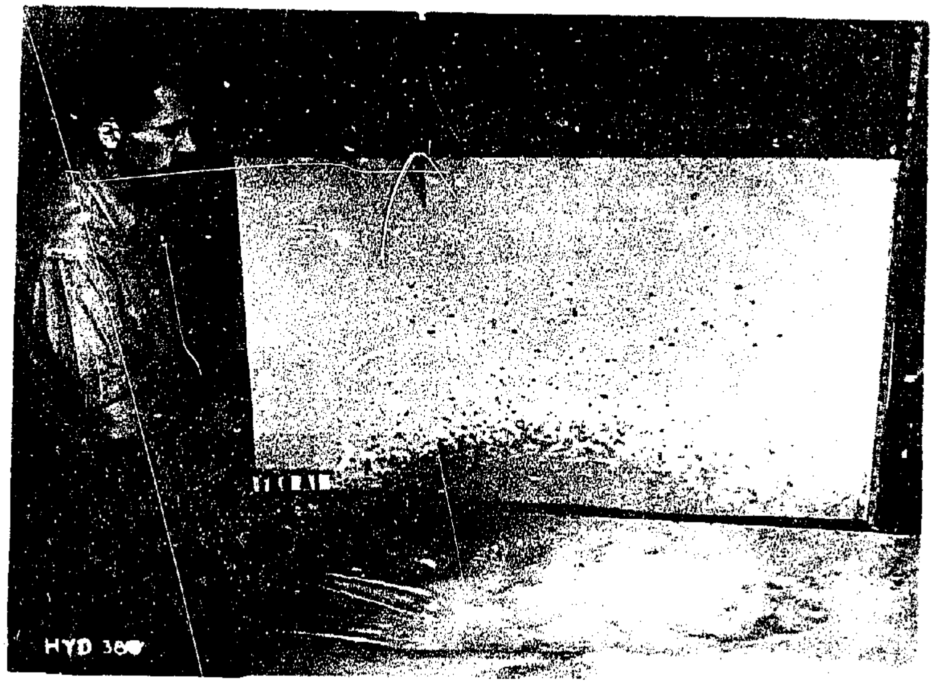


D. DISCHARGE 5000 S.F. THROUGH FOUR VALVES-TWO BAFFLES

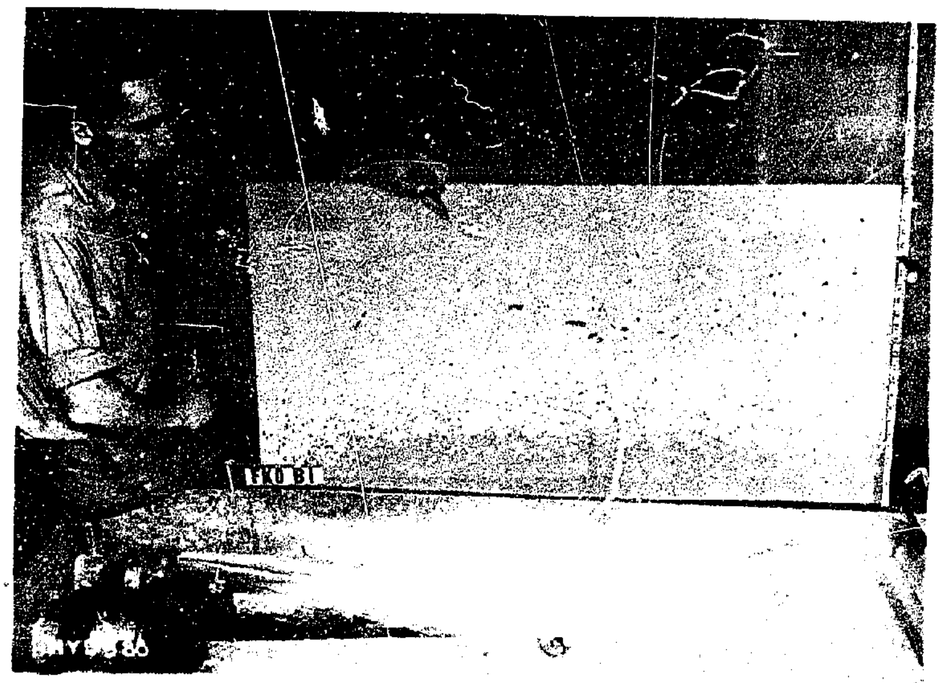
RELATIVE ROUGHNESS OF WATER SURFACE BELOW STA. 6+10.0-DISCHARGE 5000 S.F.

FRIANT-KERN CANAL OUTLETS

FIGURE 37



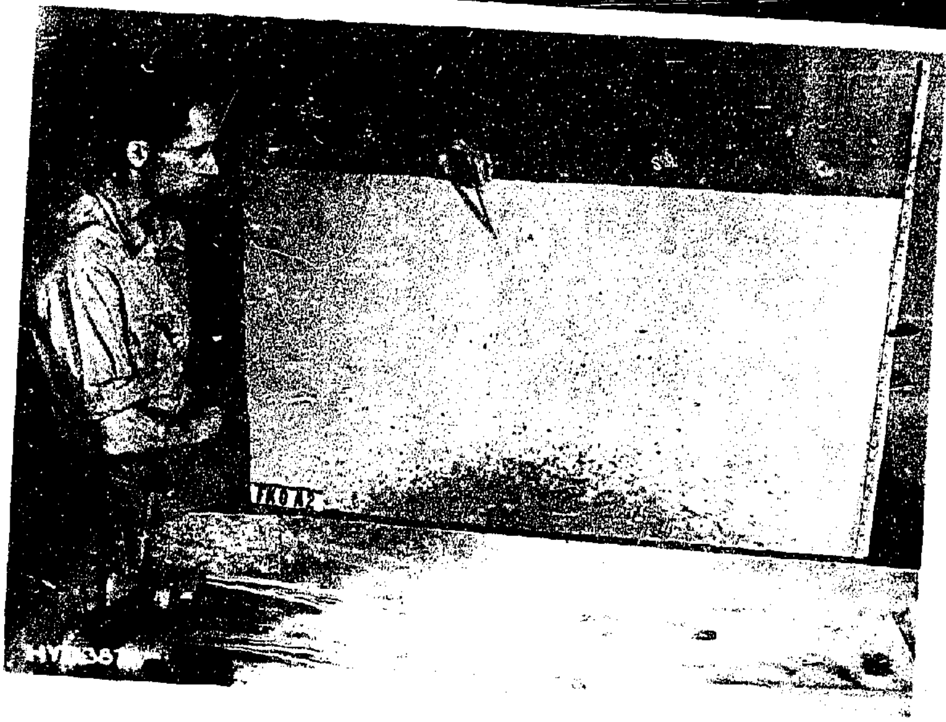
A - Discharge 5,000 s.f. through four valves.
Res. elev. 578.0.



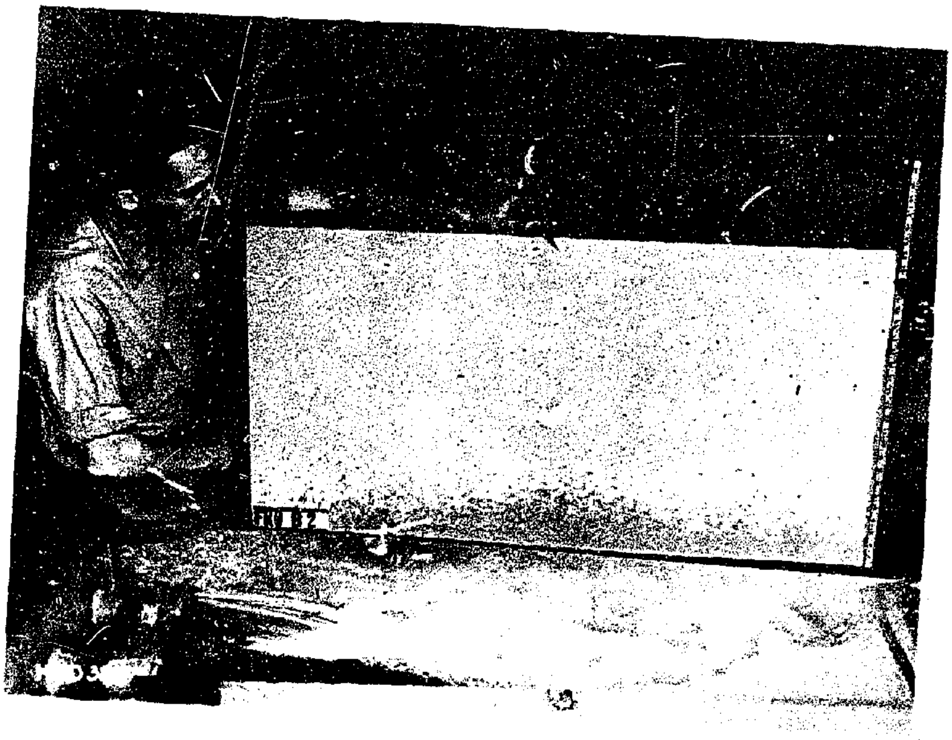
B - Discharge 5,000 s.f. through three valves.
Res. elev. 578.0

FRIANT-KERN CANAL OUTLETS WITH FOUR HOLLOW-JET VALVES
POOL DESIGNED FOR 5,000 S.F. SPRAY OVER SIDEWALLS
1:32 MODEL

FIGURE 39

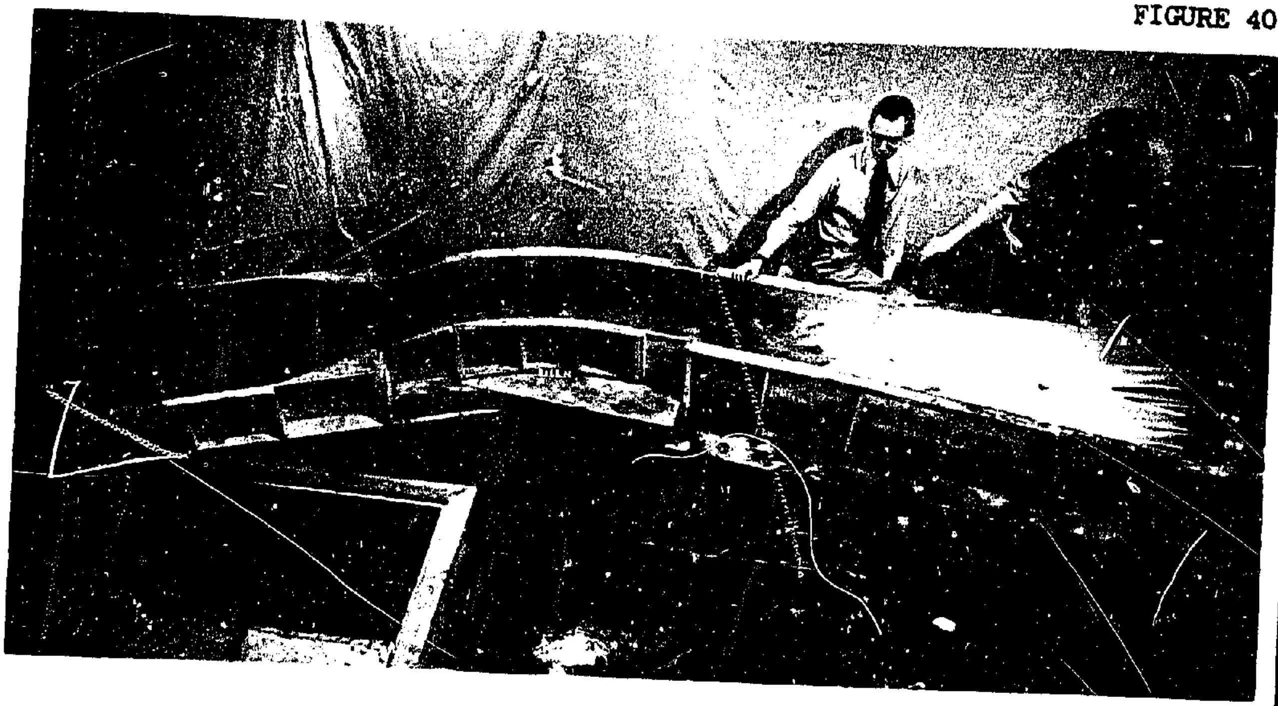


A - Discharge 4,000 s.f. through three valves.
Res. elev. 578.0.

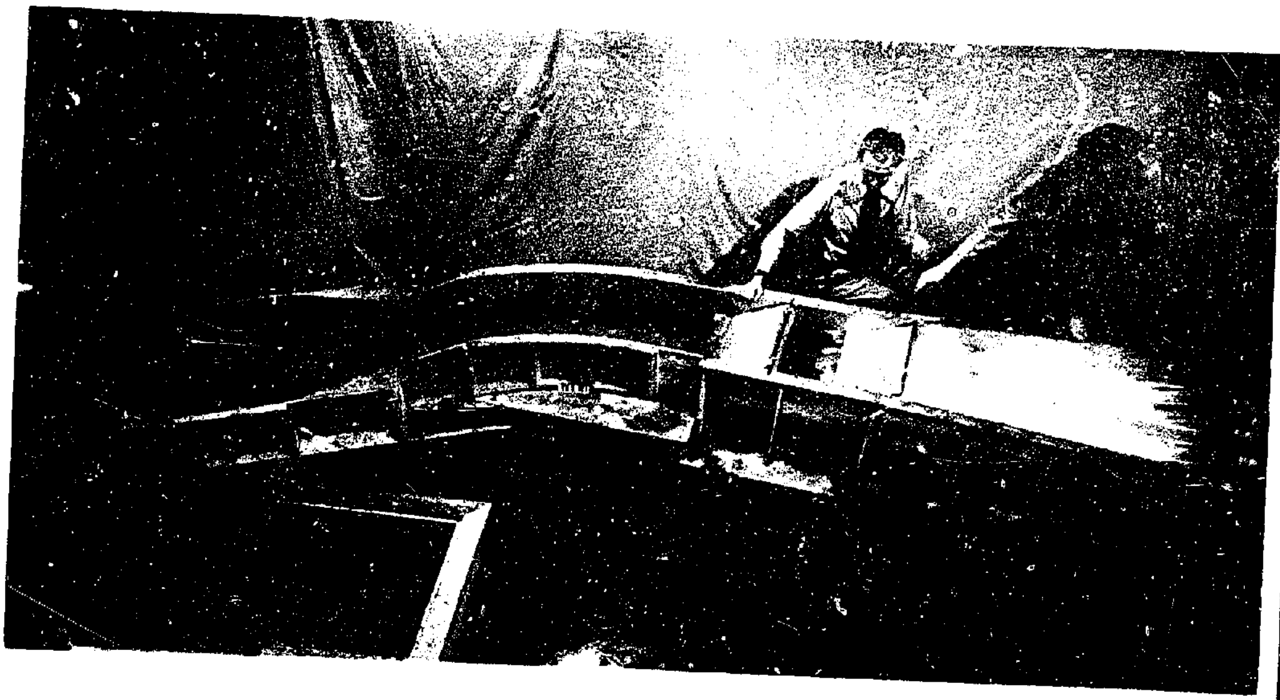


B - Discharge 4,000 s.f. through three valves.
Res. elev. 578.0.

FRIANT-KERN CANAL OUTLETS WITH FOUR HOLLOW-JET VALVES.
POOL DESIGNED FOR 5,000 S.F. SPRAY OVER SIDEWALLS
1:32 MODEL

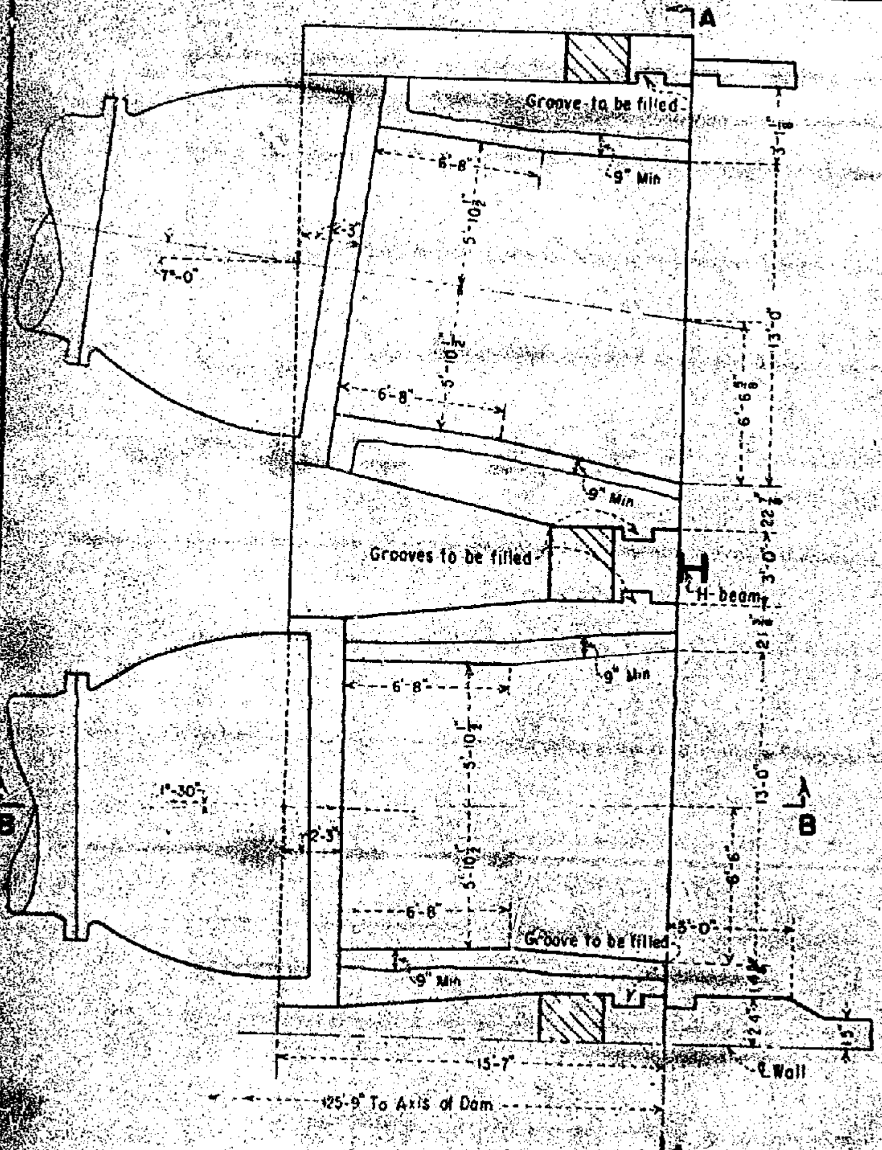


A - Discharge 5,000 s.f. through four valves with no baffles in channel. Res. elev. 578.0.

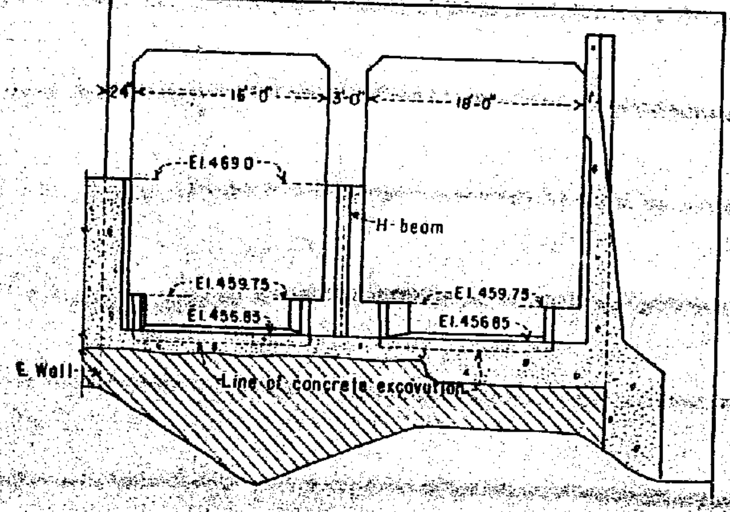


B - Discharge 5,000 s.f. through four valves with two baffles in channel. Res. elev. 578.0.

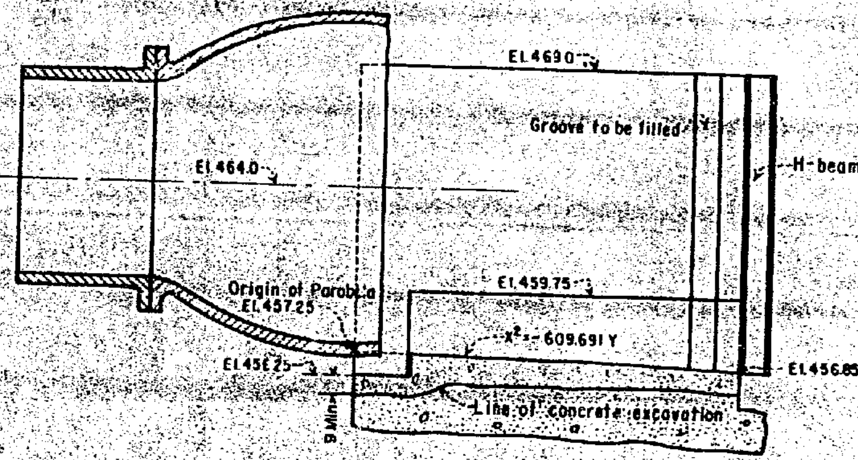
FRIANT-KERN CANAL OUTLETS WITH FOUR HOLLOW-JET VALVES.
POOL DESIGNED FOR 5,000 S.F. GENERAL ARRANGEMENT
1:32 MODEL



PLAN

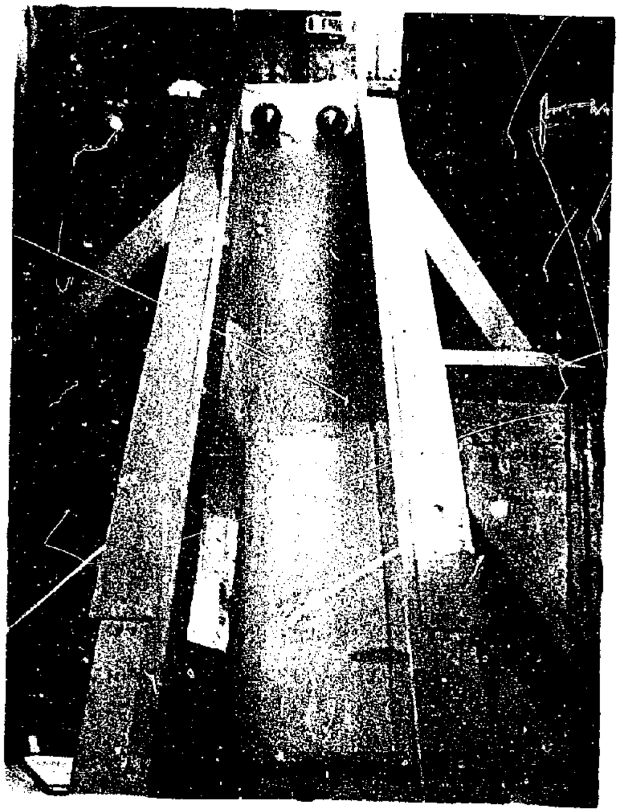


SECTIONAL ELEVATION A-A



SECTION B-B

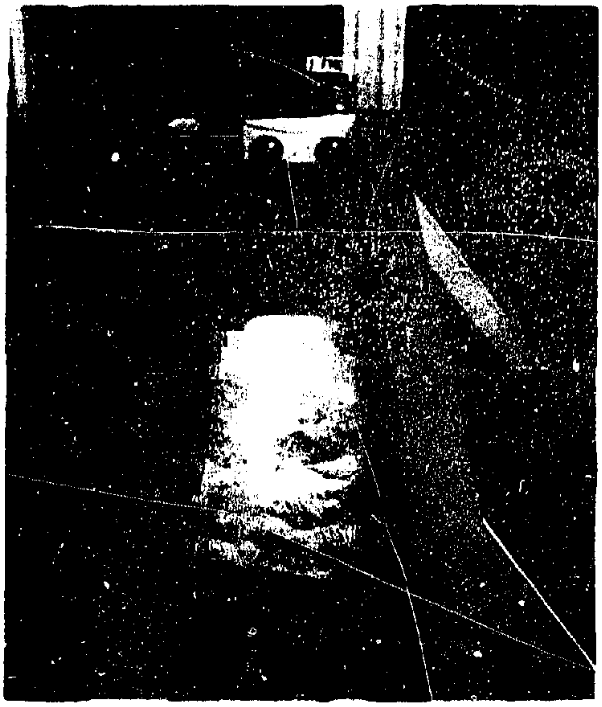
CENTRAL VALLEY PROJECT - CALIFORNIA
FRIANT-KERN CANAL OUTLETS
DETERMINATION OF CONCRETE EXCAVATION FOR
HOLLOW-JET VALVE INSTALLATION
1:16 SCALE SECTIONAL MODEL



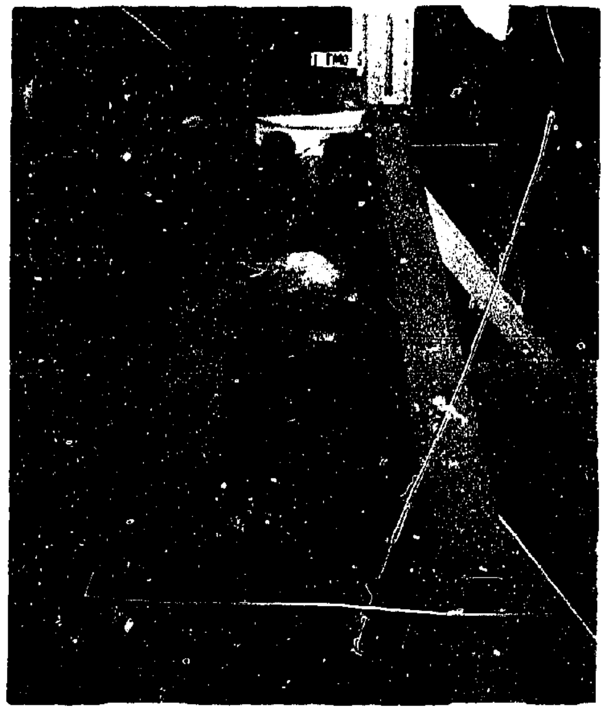
A. Looking upstream into pool.

B. Looking downstream into pool.

MADERA CANAL OUTLETS WITH TWO 78-INCH NEEDLE VALVES
1:28.44 Model
FINAL DESIGN

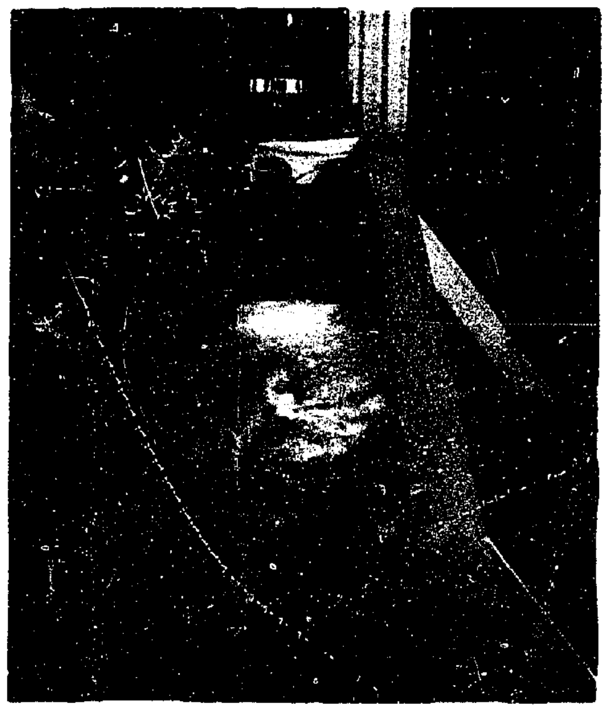


A. Discharge 1,500 s.f. Res. elev. 578.0.
Tailwater elev. 445.98.
Two valves partially open.



B. Discharge 1,500 s.f. Minimum head.
Tailwater elev. 445.98.
Two valves 100 percent open.

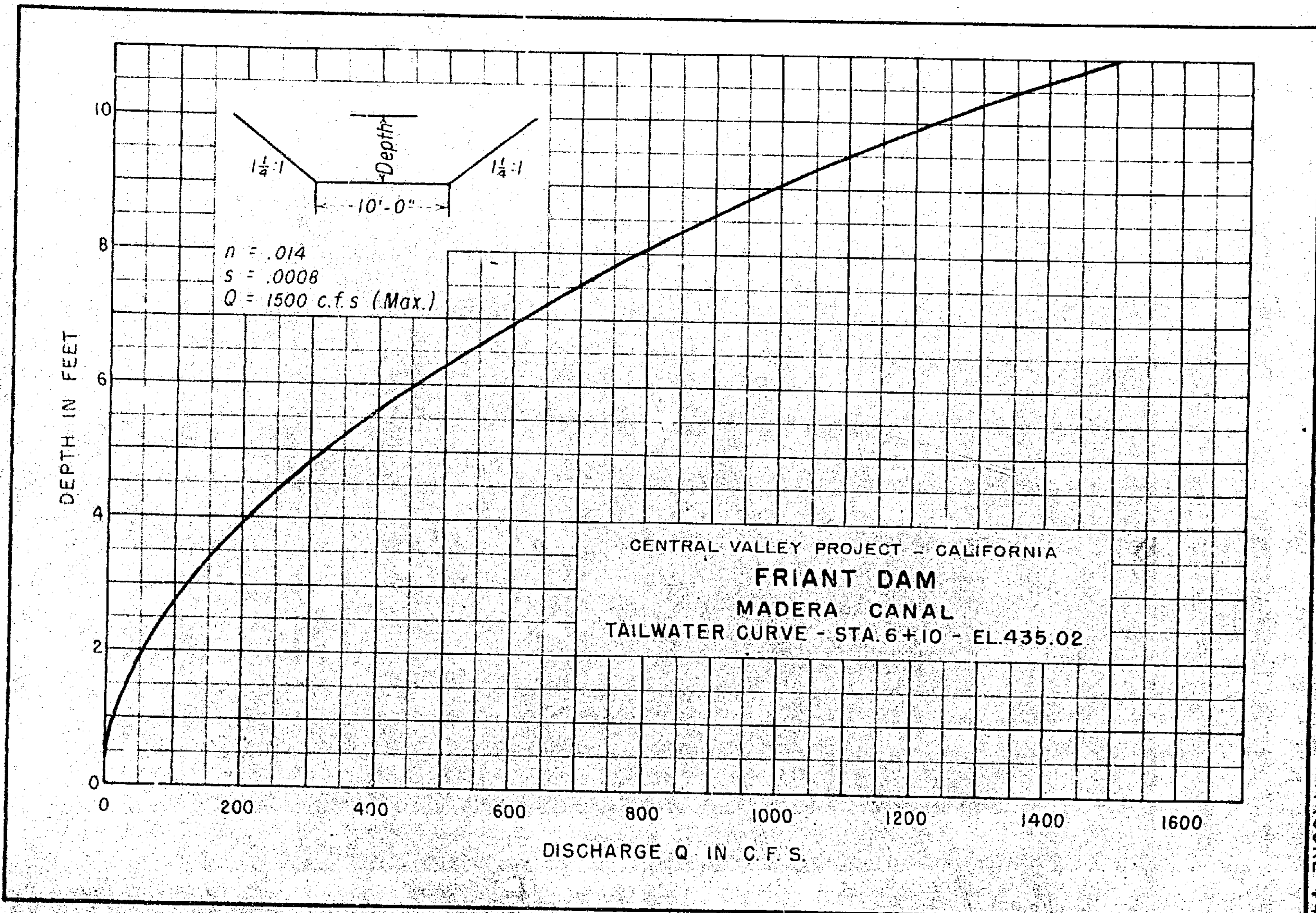
C. Discharge 750 s.f. Res. elev. 578.0
Tailwater elev. 442.82.
One valve partially open.

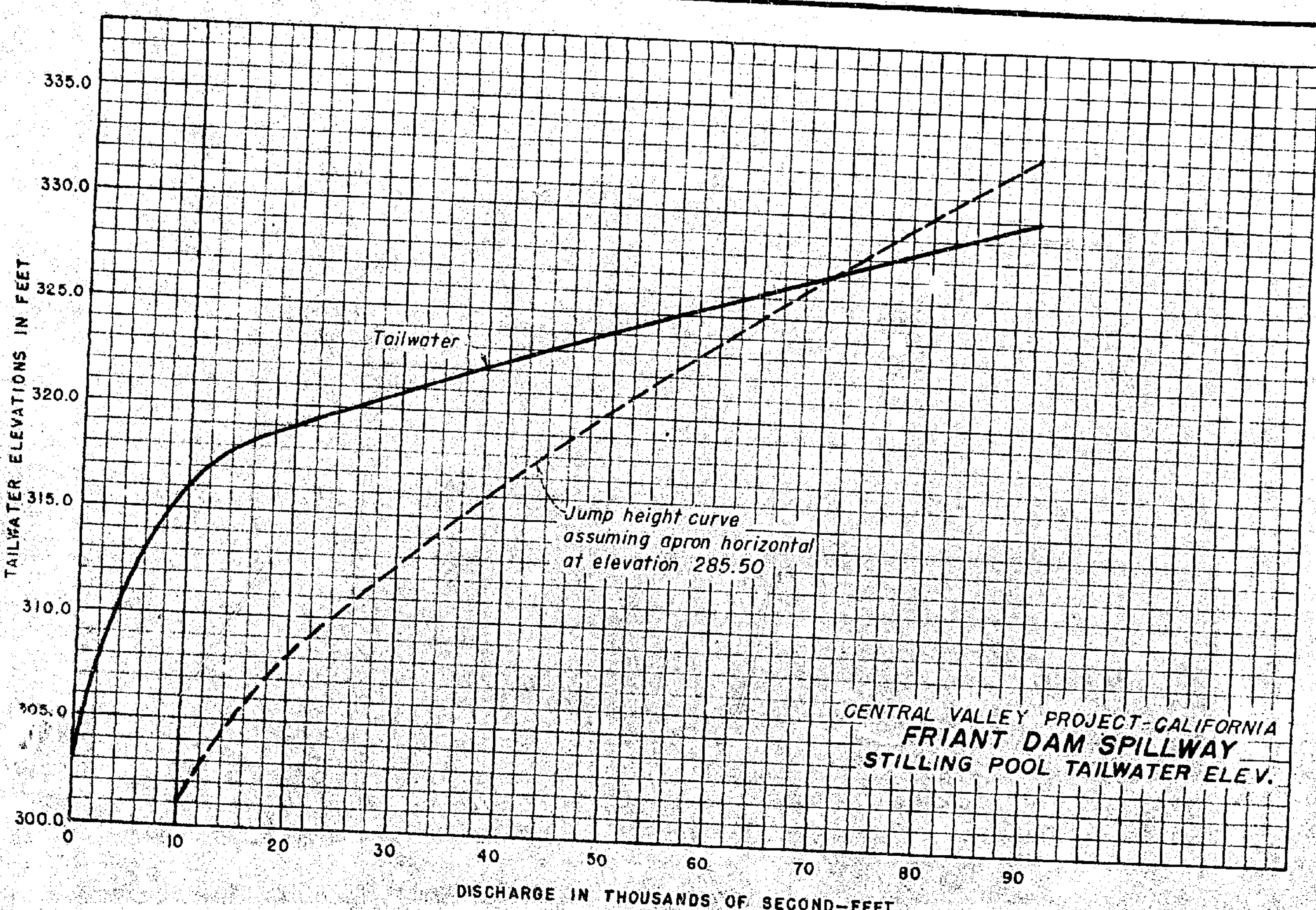


D. Discharge 1,500 s.f. Minimum head.
Tailwater elev. 445.98.
One valve 100 percent open.

MADERA CANAL OUTLETS WITH TWO 78-INCH NEEDLE VALVES
1:28.44 Model
FINAL DESIGN

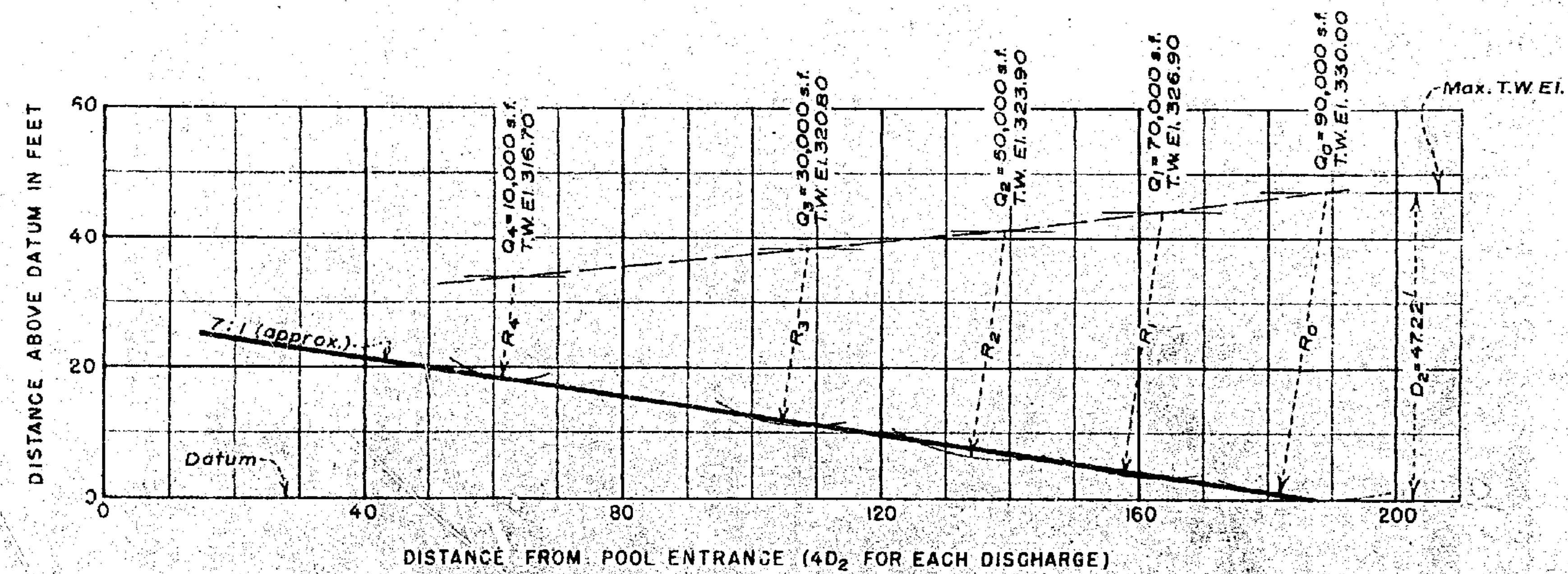
FIGURE 44


CENTRAL VALLEY PROJECT - CALIFORNIA
FRIANT DAM
MADERA CANAL
TAILWATER CURVE - STA. 6+10 - EL.435.02
DISCHARGE Q IN C.F.S.
DEPTH IN FEET
0
2
4
6
8
10
0
200
400
600
800
1000
1200
1400
1600
n = .014
s = .0008
Q = 1500 c.f.s (Max.)
Depth
10'-0"
1 1/4:1
1 1/4:1



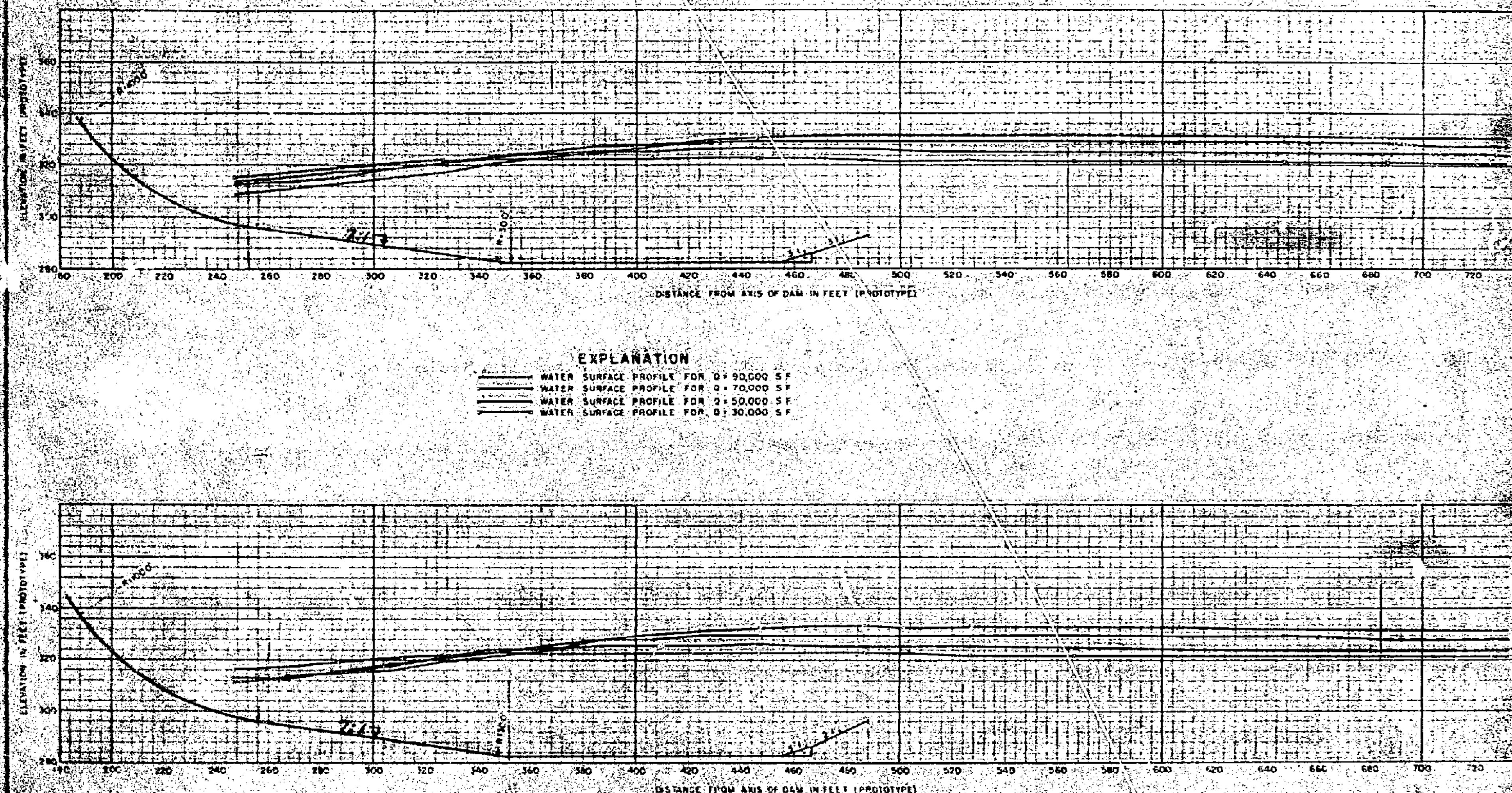
CENTRAL VALLEY PROJECT-CALIFORNIA
FRIANT DAM SPILLWAY
STILLING POOL TAILWATER ELEV.
Tailwater
Jump height curve assuming apron horizontal at elevation 285.50
TAILWATER ELEVATIONS IN FEET
335.0
330.0
325.0
320.0
315.0
310.0
305.0
300.0
0
10
20
30
40
50
60
70
80
90
DISCHARGE IN THOUSANDS OF SECOND-FEET

FIGURE 45



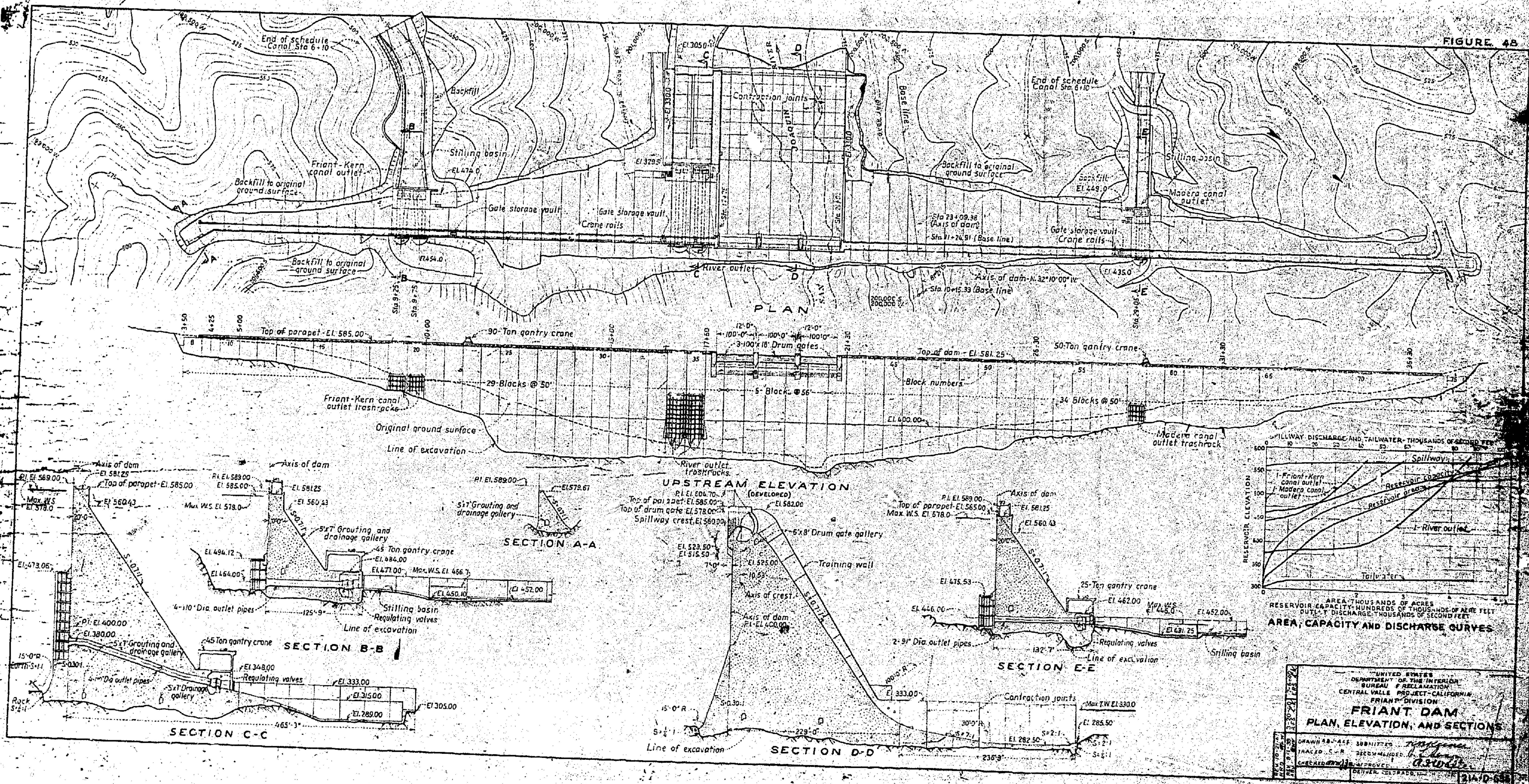
NOTE:
R_0, R_1, R_2, R_3, and R_4, are equal to the values of D_2 for the respective discharges, Q_0, Q_1, Q_2, Q_3, and Q_4.

CENTRAL VALLEY PROJECT—CALIFORNIA
FRIANT DAM SPILLWAY
DESIGN OF SLOPING APRON
1:24 SCALE MODEL



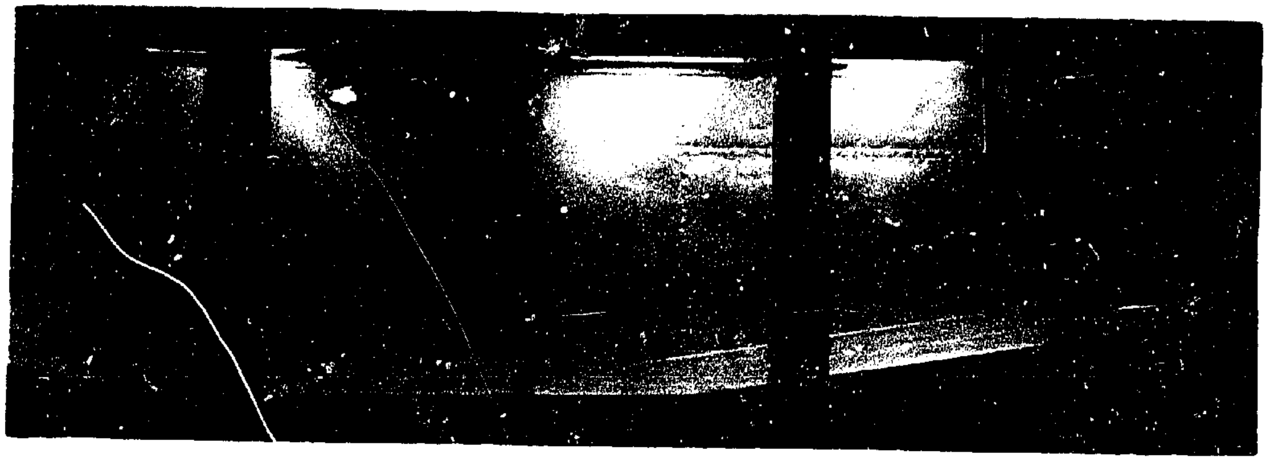
FRIANT DAM
COMPARISON OF EXCAVATION SLOPES
HYDRAULIC STUDIES ON A 1:60 MODEL

36

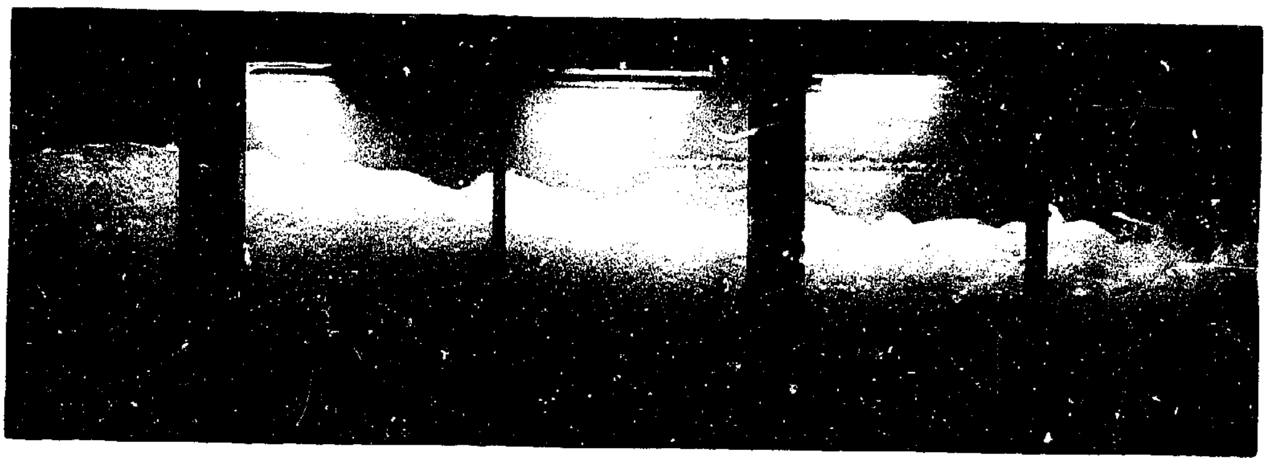


PLAN
UPSTREAM ELEVATION (DEVELOPED)
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E
AREA, CAPACITY AND DISCHARGE CURVES
Friant-Kern canal outlet
Madera canal outlet
Stilling basin
Gate storage vault
Crane rails
River outlet
Axis of dam
Contraction joints
Top of parapet - El. 585.00
90-Ton gantry crane
50-Ton gantry crane
3-100'x18' Drum gates
Top of dam - El. 581.25
29-Blocks @ 50'
5-Block @ 56'
34 Blocks @ 50'
Block numbers
Friant-Kern canal outlet trashracks
Madera canal outlet trashrack
River outlet trashracks
Original ground surface
Line of excavation
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT-CALIFORNIA
FRIANT DIVISION
FRIANT DAM
PLAN, ELEVATION, AND SECTIONS
DENVER, COLORADO
214-D-636

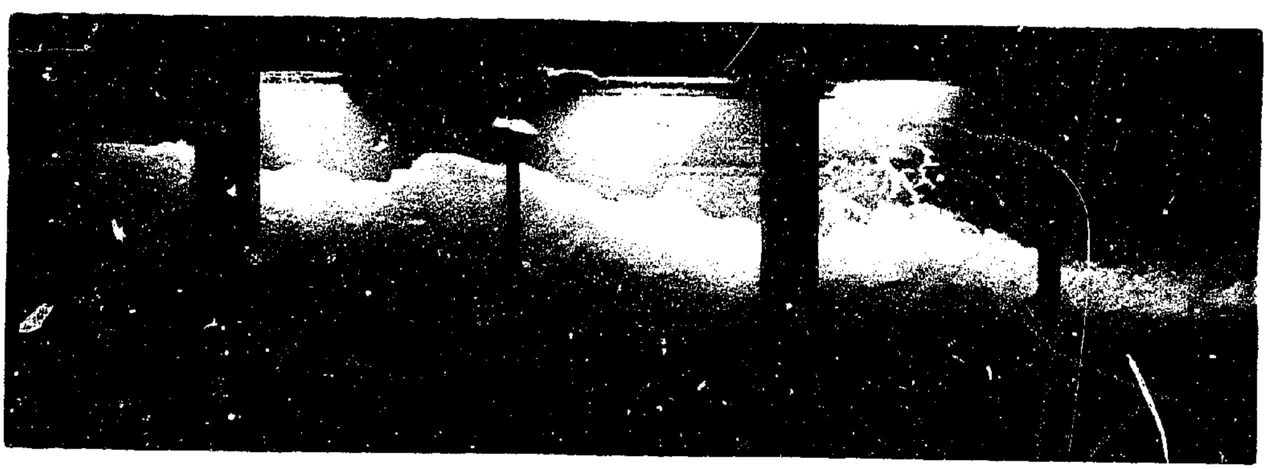
FIGURE 48



A. Recommended stilling pool design. 3:1 sloping sill in place.



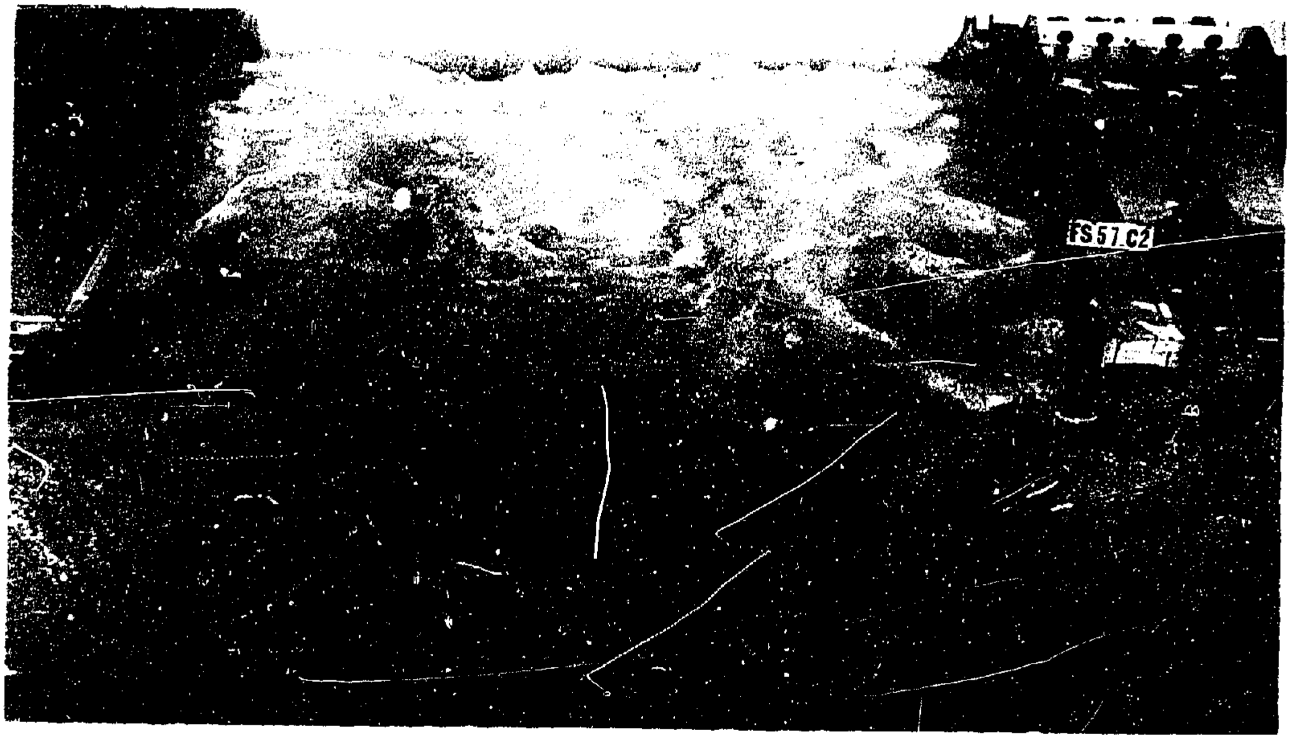
B. Discharge 70,000 second-feet. Normal tailwater.



C. Discharge 90,000 second-feet. Normal tailwater.

FRIANT DAM SPILLWAY
1:24 Model

FIGURE 50

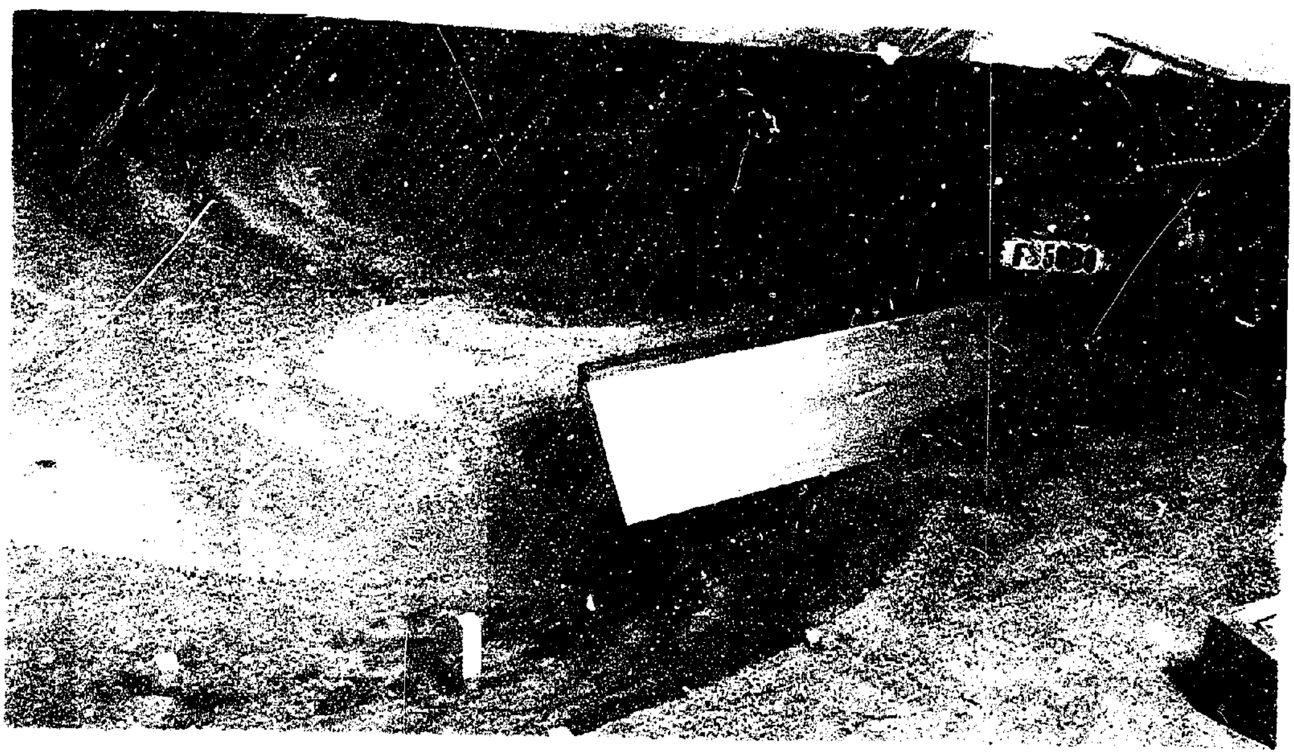


A. Discharge 50,000 second-feet.

B. Discharge 90,000 second-feet.

FRIANT DAM SPILLWAY AND STILLING POOL
1:60 Model

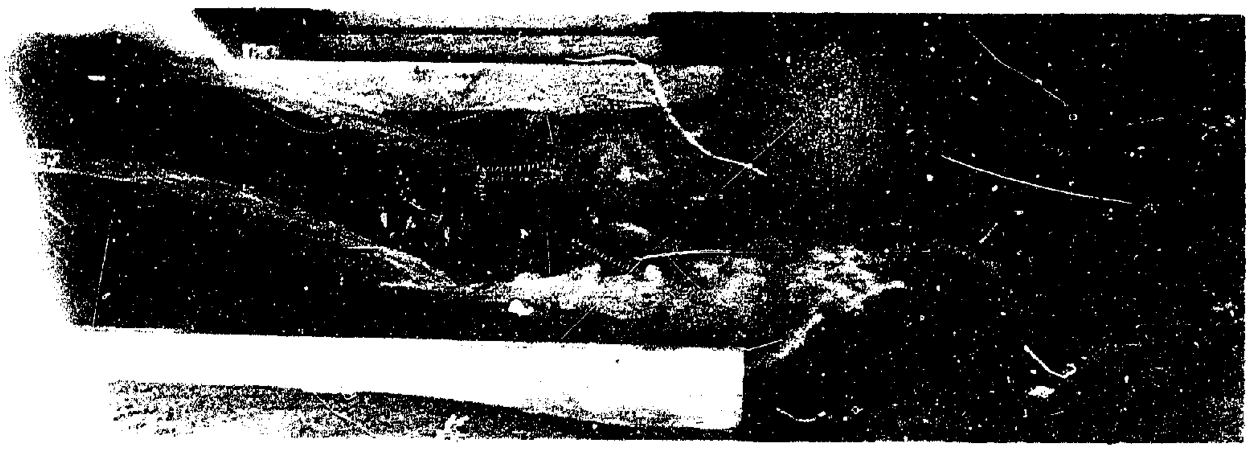
A. Plan view of erosion caused by passing 20,000 s.f. through the outlets.



B. Side view of erosion caused by passing 20,000 s.f. through the outlets.

FRIANT DAM SPILLWAY AND RIVER OUTLETS
1:60 Model

A. Discharge 4,000 second-feet. Reservoir elevation 568.0.
Tailwater elevation 309.50.

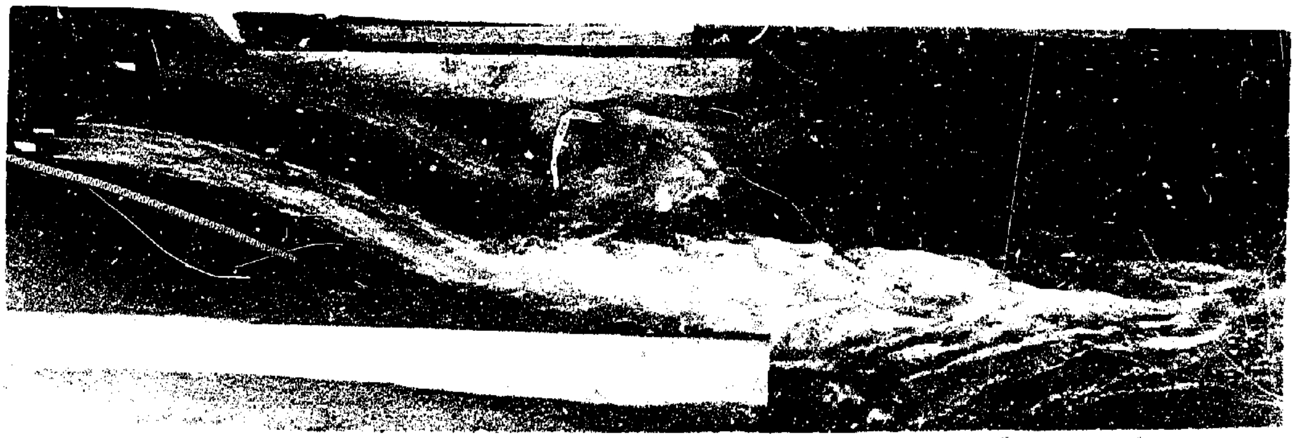


B. Discharge 8,000 second-feet. Reservoir elevation 568.0.
Tailwater elevation 315.0.

C. Discharge 8,000 second-feet. Reservoir elevation 568.0.
Tailwater elevation 315.0.

FRIANT DAM SPILLWAY WITH RIVER OUTLETS AT ELEVATION 358.0
1:60 Model

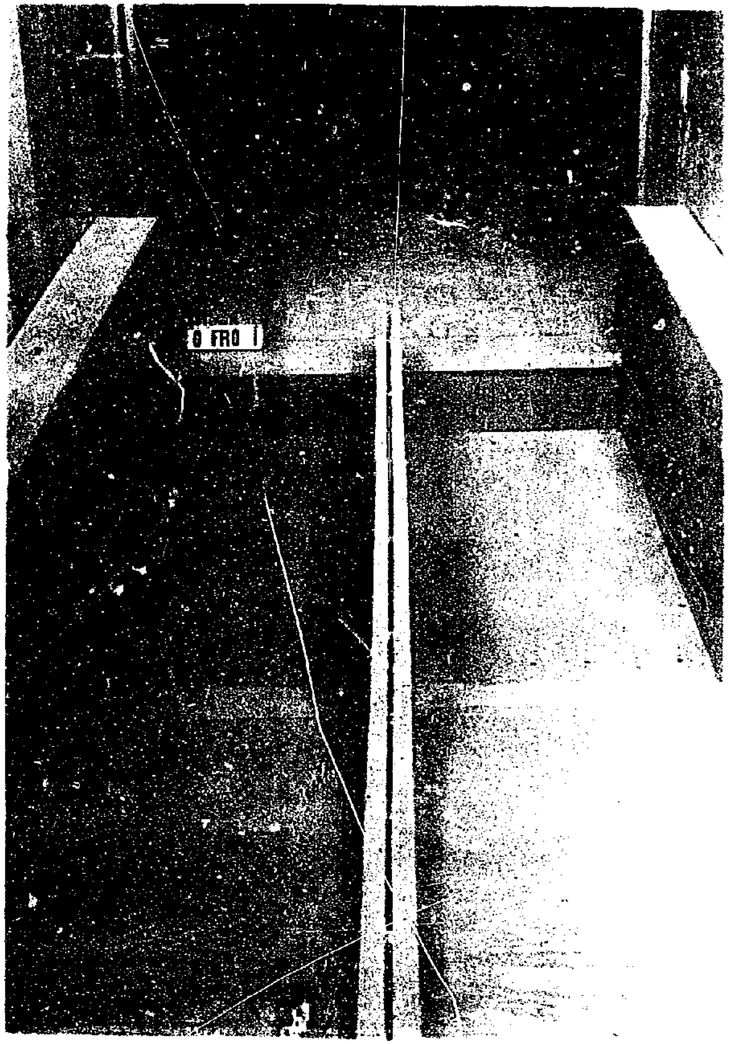
A. Discharge 12,000 second-feet. Reservoir elevation 568.0. Tailwater elevation 318.0.



B. Discharge 16,000 second-feet. Reservoir elevation 568.0. Tailwater elevation 318.0.

C. Erosion produced by operation of the outlets. Tops of pegs represent original ground surface. This is equivalent to removal of 10 feet of erodible material.

FRIANT DAM SPILLWAY WITH RIVER OUTLETS AT ELEVATION 358.0
1:60 Model

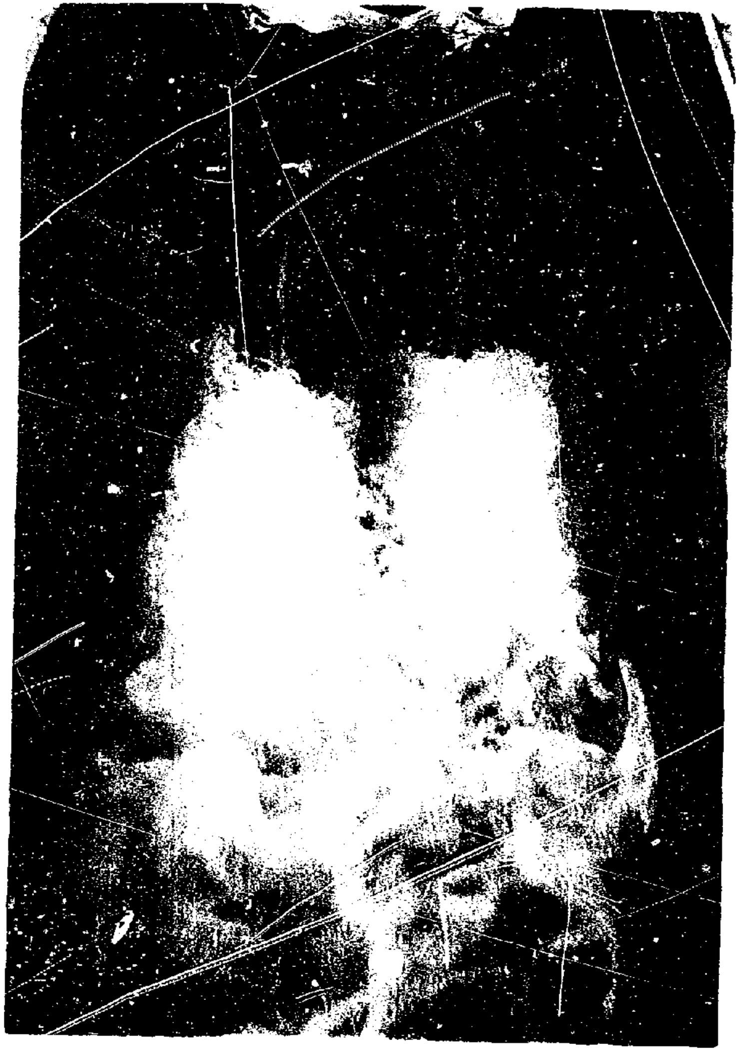


B. Model arrangement. Looking downstream.

A. Model arrangement. Looking upstream.

FRIANT RIVER OUTLETS WITH TWO NEEDLE AND TWO TUBE VALVES
1:34.38 Model

FIGURE 55



A. Discharge 17,000 second-feet.
Tailwater elevation 318.0.
Four valves 100 percent open.

B. Discharge 16,000 second-feet.
Tailwater elevation 318.0.
Four valves 100 percent open.

FRIANT RIVER OUTLETS WITH TWO NEEDLE AND TWO TUBE VALVES
1:34.38 Model

FIGURE 56

A. Discharge 12,000 second-feet. Minimum head.
Tailwater elevation 317.0.
Four valves 100 percent open.

B. Discharge 12,000 second-feet. Maximum head.
Tailwater elevation 317.0.
Four valves partially open.

FRIANT RIVER OUTLETS WITH TWO NEEDLE AND TWO TUBE VALVES
1:34.38 Model

FIGURE 57

A. Discharge 8,500 second-feet.
Tailwater elevation 315.2.
Two needle valves 100 percent open.

B. Discharge 8,500 second-feet.
Tailwater elevation 315.2.
Two tube valves 100 percent open.

FRIANT RIVER OUTLETS WITH TWO NEEDLE AND TWO TUBE VALVES
1:34.38 Model

A. Model arrangement.
Looking upstream into pool

B. Discharge 2,000 second-feet.
Four valves approximately 12 percent open.

C. Discharge 6,000 second-feet.
Four valves approximately 30 percent open.

D. Discharge 16,000 second-feet.
Four valves approximately 91 percent open.

FRIANT RIVER OUTLETS WITH FOUR HOLLOW-JET VALVES
1:32 Model
FINAL DESIGN

A. Discharge 20,000 second-feet. Four valves 100 percent open.

B. Discharge 5,000 second-feet. Two valves approximately 50 percent open.

C. Discharge 5,000 second-feet. Two valves approximately 50 percent open.

D. Discharge 5,000 second-feet. Two valves approximately 50 percent open.

FRIANT RIVER OUTLETS WITH FOUR HOLLOW-JET VALVES
1:32 Model
FINAL DESIGN

A. Discharge 2,000 second-feet. Two valves approximately 30 percent open.

B. Discharge 10,000 second-feet. Two valves 100 percent open.

C. Discharge 8,000 second-feet. Two valves approximately 81 percent open.

D. Discharge 8,000 second-feet. Two valves approximately 81 percent open.

FRIANT RIVER OUTLETS WITH FOUR HOLLOW-JET VALVES
1:32 Model
FINAL DESIGN

A. Discharge 3,000 second-feet. One valve approximately 71 percent open.

B. Discharge 4,000 second-feet. One valve approximately 83 percent open.

C. Discharge 3,000 second-feet. One valve approximately 76 percent open.

D. Discharge 4,000 second-feet. One valve approximately 81 percent open.

FRIANT RIVER OUTLETS WITH FOUR HOLLOW-JET VALVES
1:32 Model
FINAL DESIGN

APPENDICES

APPENDIX A

Mathematical Analysis for Selection of Temporary Nozzles for the Friant Dam River Outlets.

During construction of the dam it was necessary to divert river flow through the 110-inch river-outlet conduits to supplement the flow through the regular diversion tunnels. The river-outlet stilling pool was completed at the time of the river diversion, but the valves were not in place. Therefore, it was necessary to provide nozzles at the ends of the conduits to maintain sufficient pressure in the conduits to prevent cavitation and pitting. The nozzles were to be located so that the flow would enter tangent to the parabolic apron of the outlet pool, thus duplicating, as far as possible, the discharge of the valves. The nozzle is shown on Figure 1.

Conduit losses.

From axis of dam to P. C. of upper bend	12.78 feet
Length of upper bend	18.30 feet
Length of lower bend	7.81 feet
From P. T. of upper bend to P. C. of lower bend	121.40 feet
From P. T. of lower bend to flange	37.10 feet
Total length along centerline of 110-inch pipe =	197.39 feet

A length of 200 feet was used for computing friction losses.

Length of tangent at upper bend = 9.22 feet.

Length of tangent at lower bend = 3.91 feet.

Bernoulli's equation written between the reservoir water surface and the flange, assuming no nozzle at the flange:

$$H = h_t + h_e + h_{B_1} + h_{B_2} + \frac{V^2}{2g} + h_f$$

where

V = velocity in the 110-inch pipe

$\frac{V^2}{2g}$ = velocity head at exit = $1.00\frac{V^2}{2g}$

h_t trashrack losses = $0.01\frac{V^2}{2g}$

h_e = entrance losses = $0.10\frac{V^2}{2g}$

h_{B_1} = loss in upper bend = $0.11\frac{V^2}{2g}$

h_{B_2} = loss in lower bend = $0.07\frac{V^2}{2g}$

h_f = friction loss in any length of pipe
equal $0.011\frac{V^2L}{2gd} = 0.24\frac{V^2}{2g}$ for 200 feet.

Therefore, $H = 1.53\frac{V^2}{2g}$

Assume a maximum total discharge of 17,000 second-feet or 4,250 second-feet for each outlet. The area of the 110-inch pipe = 66.00 square feet.

$$V = \frac{4250}{66.00} = 64.40 \text{ fps}$$

$$\frac{V^2}{2g} = 64.40 \text{ feet}$$

Therefore, the head required to pass 4,250 second-feet with no nozzle at the flange is:

$$H = 1.53\frac{V^2}{2g} = 1.53 \times 64.4 = 98.53 \text{ feet.}$$

Elevation of the P. I. of the upper bend = 380.00

Elevation of the P. I. of the lower bend = 339.64

Elevation of the centerline of the flange = 332.52

Then the reservoir elevation required to pass 4,250 second-feet = elevation 332.52 + 98.53 = elevation 431.05.

Theoretical pressure-heads along the centerline of the conduit passing 4,250 second-feet with no nozzle. The pressure-heads were computed at piezometers 1, 2, 3, and 4 where Piezometer No. 2 is on the center of the tangent between the P. I.'s of the bends and Piezometer No. 4 is on the center between the flange and the P. I. of the lower bend.

Pressure-head at No. 1.

$$431.05-380.00 = 0.01\frac{V^2}{2g} + 0.10\frac{V^2}{2g} + 1/2\left(0.11\frac{V^2}{2g}\right) + 0.011\frac{L}{d}\frac{V^2}{2g} + \frac{V^2}{2g} + \frac{P_1}{w}$$

$$L = 220 \text{ feet; } h_f = 0.011 \times \frac{22}{9.17}\frac{V^2}{2g} = 0.026\frac{V^2}{2g}$$

$$\frac{P_1}{w} = 51.05-(1.191 \times 64.40) = -25.65 \text{ feet}$$

All pressure-heads were computed in a similar manner. Table 1 gives the pressure-heads for no nozzle at the flange.

TABLE 1

Q in cfs	Area of pipe in sq. ft.	Velocity in pipe	$V^2/2g$	H in feet on conduit outlet	Reservoir elevation	Pressure-heads in feet of H_2O Piez#1	Piez#2	Piez#3	Piez#4
4,250	66.00	64.40	64.40	98.53	431.05	-25.65	-14.42	-1.46	-1.95
3,250	66.00	49.24	37.65	57.60	390.12	-34.72	-19.77	-3.85	-2.62

Assume that the minimum reservoir elevation which will cause the pipe to flow full is such that the water surface will be approximately five feet above the top of the bellmouth at the face of the dam. This corresponds approximately to elevation 391.0. From Table 1 it is seen that, for no nozzle, the value c^ Q = 3,250 second-feet is the approximate minimum discharge at which the conduit will flow full. It is obvious from the above figures that a nozzle should be placed at the flange to reduce the negative pressures and the flow in the conduit.

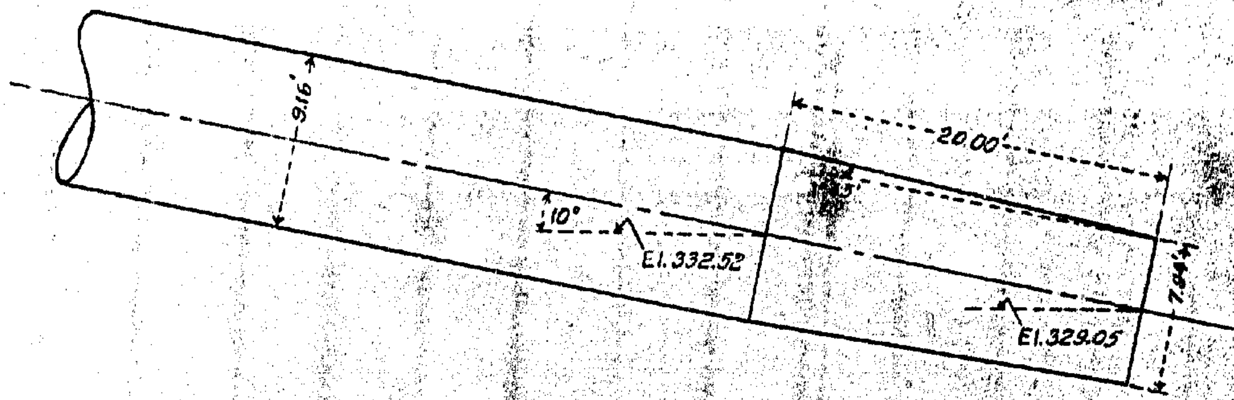
<u>Selection of a nozzle.</u> Assume a nozzle 20 feet in length with an exit area equal to 75 percent of the pipe area.

Area of 110-inch pipe = 66.00 sq. ft.

Area of nozzle exit = 49.50 sq. ft.

Diameter of nozzle = 7.94 ft.

V_n = velocity at nozzle exit.



$$\theta = \text{arc tan} = \frac{4.58-3.97}{20} = 0.0305$$

$$\theta = 1^\circ\ 45' \qquad 2\theta = 3^\circ\ 30'$$

$$\frac{d_1}{d_2} = \frac{9.17}{7.94} = 1.155$$ (see Table 74, Page 231--King's Handbook)

$K_2 = 0.015$ and the nozzle loss $= 0.015 \frac{V_n^2}{2g}$

Then

$$H = 0.01\frac{V^2}{2g} + 0.10\frac{V^2}{2g} + 0.11\frac{V^2}{2g} + 0.07\frac{V^2}{2g} + \frac{V_n^2}{2g} + 0.011 \times \frac{200}{9.17}\frac{V^2}{2g} + 0.015\frac{V_n^2}{2g}$$

$$H = 0.53\frac{V^2}{2g} + 1.015\frac{V_n^2}{2g}$$

to pass a given discharge with this nozzle in place.

The theoretical pressure-heads along the centerline of the conduit passing various discharges with a nozzle having an exit area equal to 75 percent of the conduit area were computed. These values are found in the first three lines of Table 2. They show that a smaller nozzle was required to further reduce the negative pressures at low discharges.

Assume a nozzle having an exit area equal to 65 percent of the area of the conduit.

Then $H = 0.53\frac{V^2}{2g} + 1.020\frac{V_n^2}{2g}$

to pass a given discharge with this nozzle in place. The theoretical pressure-heads along the centerline of the conduit are found in the lower part of Table 2. The pressure gradients for the two nozzles appear on Figure 1. A nozzle having an exit area equal to 65 percent of the conduit area would work satisfactorily for the river outlets.

A nozzle was actually built, installed and used on the outlets with satisfactory results. An even exit diameter of 88 inches was used, giving an area ratio of 64 percent instead of the 65 percent recommended. It is shown as Detail 1, Figure 1.

TABLE 2

Q in cfs	Area of pipe in sq. ft.	Velocity in pipe V_p	Area of nozzle in sq. ft.	Velocity in nozzle Vn	$\frac{V_p^2}{2g}$	$\frac{V_n^2}{2g}$	Reservoir elevation	H in feet on conduit outlet	Pressure-head in ft. of H_2O Piez. #1	Piez. #2	Piez. #3	Piez. #4
4,250	66.00	64.40	49.50	85.86	64.40	114.47	479.37	150.32	+ 22.67	+ 33.90	+ 46.80	+ 46.37
3,250	66.00	49.24	49.50	65.66	37.65	66.94	416.94	87.89	- 7.90	+ 7.05	+ 22.97	+ 24.20
2,750	66.00	41.67	49.50	55.55	26.96	47.92	391.98	62.93	- 20.13	- 3.70	+ 13.44	+ 15.33
4,250	66.00	64.40	42.90	99.07	64.40	152.40	518.63	189.58	+ 61.93	+ 73.16	+ 86.06	+ 85.63
3,250	66.00	49.24	42.90	75.76	37.65	89.12	439.90	110.85	+ 15.06	+ 30.01	+ 45.93	+ 47.16
2,500	66.00	37.88	42.90	58.28	22.28	52.74	394.65	65.60	- 11.89	+ 5.20	+ 22.85	+ 25.04

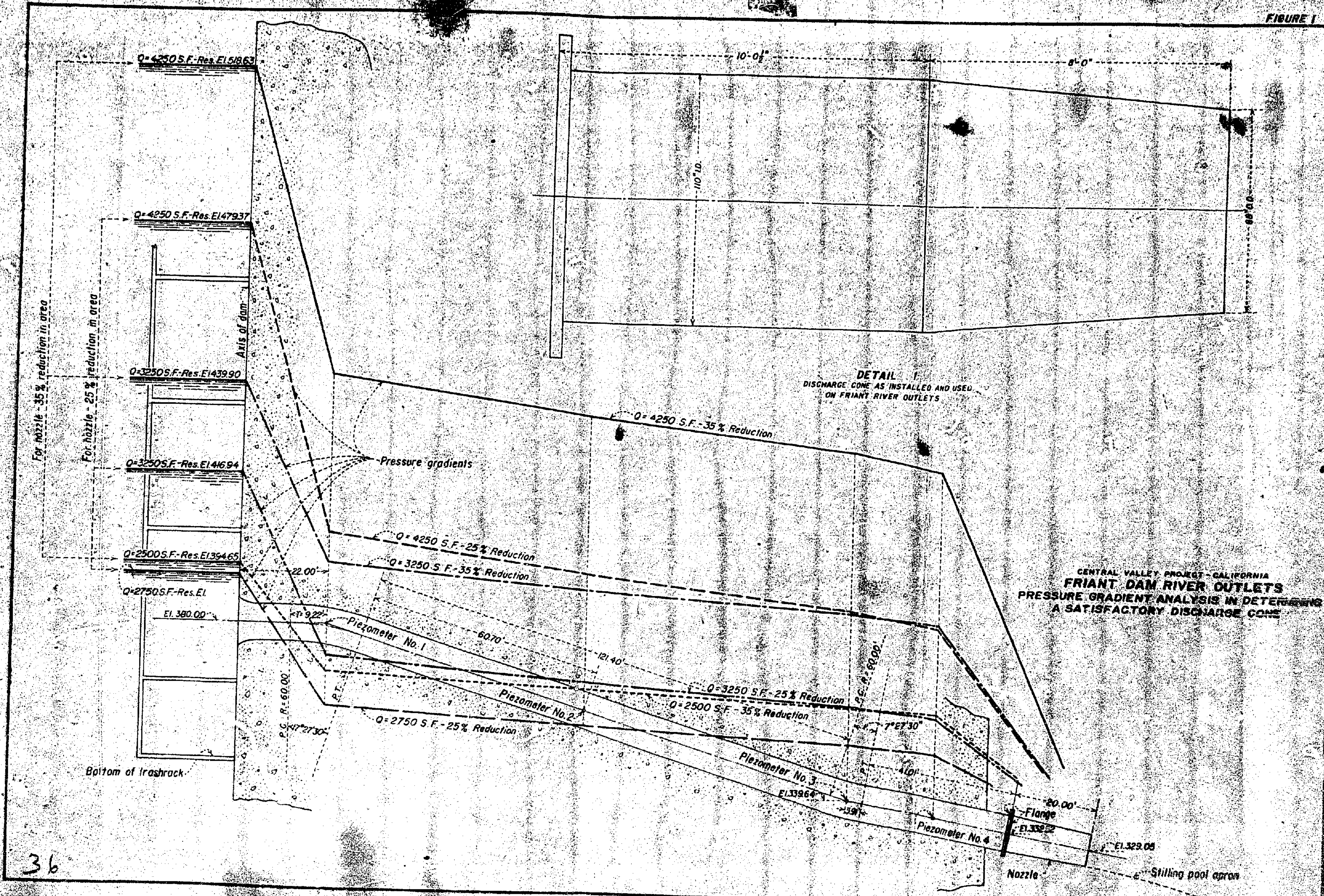


FIGURE I
10'-0½"
8'-0"
110" I.D.
88" O.D.
DETAIL I
DISCHARGE CONE AS INSTALLED AND USED ON FRIANT RIVER OUTLETS
Q=4250 S.F.-Res. El.518.63
Q=4250 S.F.-Res. El.479.37
Q=3250 S.F.-Res. El.439.90
Q=3250 S.F.-Res. El.416.94
Q=2500 S.F.-Res. El.394.65
Q=2750 S.F.-Res. El.
For nozzle - 35% reduction in area
For nozzle - 25% reduction in area
Axis of dam
Pressure gradients
Q=4250 S.F.-35% Reduction
Q=4250 S.F.-25% Reduction
Q=3250 S.F.-35% Reduction
Q=3250 S.F.-25% Reduction
Q=2500 S.F.-35% Reduction
Q=2750 S.F.-25% Reduction
22.00'
El. 380.00
Piezometer No. 1
Piezometer No. 2
Piezometer No. 3
Piezometer No. 4
60.70'
121.40'
P.C. R=60.00'
17°27'30"
7°27'30"
El.339.64
Flange
Nozzle
20.00'
El.329.05
Stilling pool apron
Bottom of trashrack
CENTRAL VALLEY PROJECT - CALIFORNIA
FRIANT DAM RIVER OUTLETS
PRESSURE GRADIENT ANALYSIS IN DETERMINING A SATISFACTORY DISCHARGE CONE
36

Mr. Warnock

Friant-Kern. Canal Stilling Basin

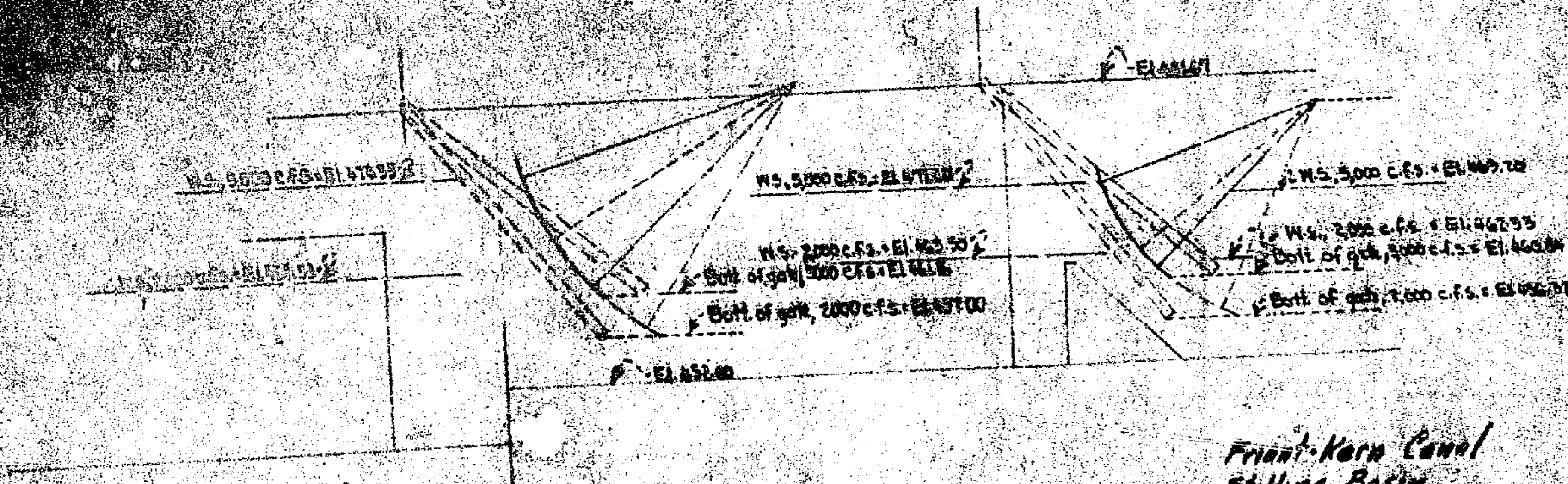
The attached sketch shows the proposed radial gates for dampening the waves in the Friant-Kern Canal-

It appears the radial gates would cost less and give equal results as compared with the flat leaf gates-

Comments are requested-

L. G. Puls-

OK, +7H.



Friant-Kern Canal
Stilling Basin
Proposed Radial Gates for dampening Waves

Return to Puls.

FIGURE V

Denver 2, Colorado, May 9, 1946.

MEMORANDUM TO L. G. PULS:
(C. V. ADKINS THROUGH J. E. WARNOCK)

Subject: Wave damping devices for the Friant-Kern Canal Stilling Basin.

1. In reply to the radial gate design proposed by Mr. Puls for wave damping, the coefficients of the radial gates would be higher than those of the flat leaf gates. This would require the radial gates to be lowered more than the flat leaf gates to obtain the same results. For high flows the angles at which the faces of the radial gates set with the direction of flow are steeper than with the flat leaf gates. The downstream radial gate is closer to the dentates at higher flows than recommended. This may require a larger head differential across the gate to pass the flow. These items will tend to change the nature of the flow from that obtained with the flat leaf gates, and it is not possible to state the magnitude of this difference. Neither is it possible to state the ideal relative location of the gates and dentates, without additional model studies.

2. The flat leaf gates did not eliminate the waves completely. They would have to be regulated by some mechanism which would hold the gates rigid at any and all positions. No practical means of fulfilling this requirement are available. The radial gates will lower of their own weights and will remain in whatever position they may be placed. A comparatively simple method of raising and lowering these gates is available. It is estimated that the complete installation of the radial gates would cost approximately seventy-five thousand dollars and that the installation of the flat leaf gates and regulating mechanism would be at least twice that amount. If the radial gates are operated to maintain the same differences in water surfaces as was recommended for the flat leaf gates, there should not be too much difference in the damping effect of the two installations. With the above points in mind, it is believed that installation of the radial gates is justified. An

operating schedule for these gates should be determined in the field, using the values obtained for the flat leaf gates as a guide. The operating schedule for the flat leaf gates as determined by model studies does not necessarily apply to the radial gates.