UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

BOULDER CANYON PROJECT

FINAL REPORTS

PART VI—HYDRAULIC INVESTIGATIONS

Bulletin 4

MODEL STUDIES OF IMPERIAL DAM, DESILTING WORKS, ALL-AMERICAN CANAL STRUCTURES

UNITED STATES
DEPARTMENT OF THE INTERIOR
J. A. Krug, Secretary

BUREAU OF RECLAMATION
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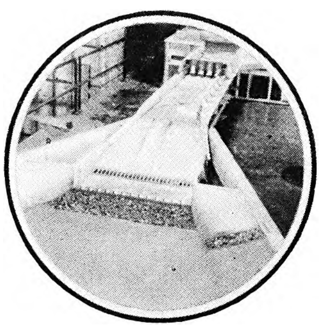
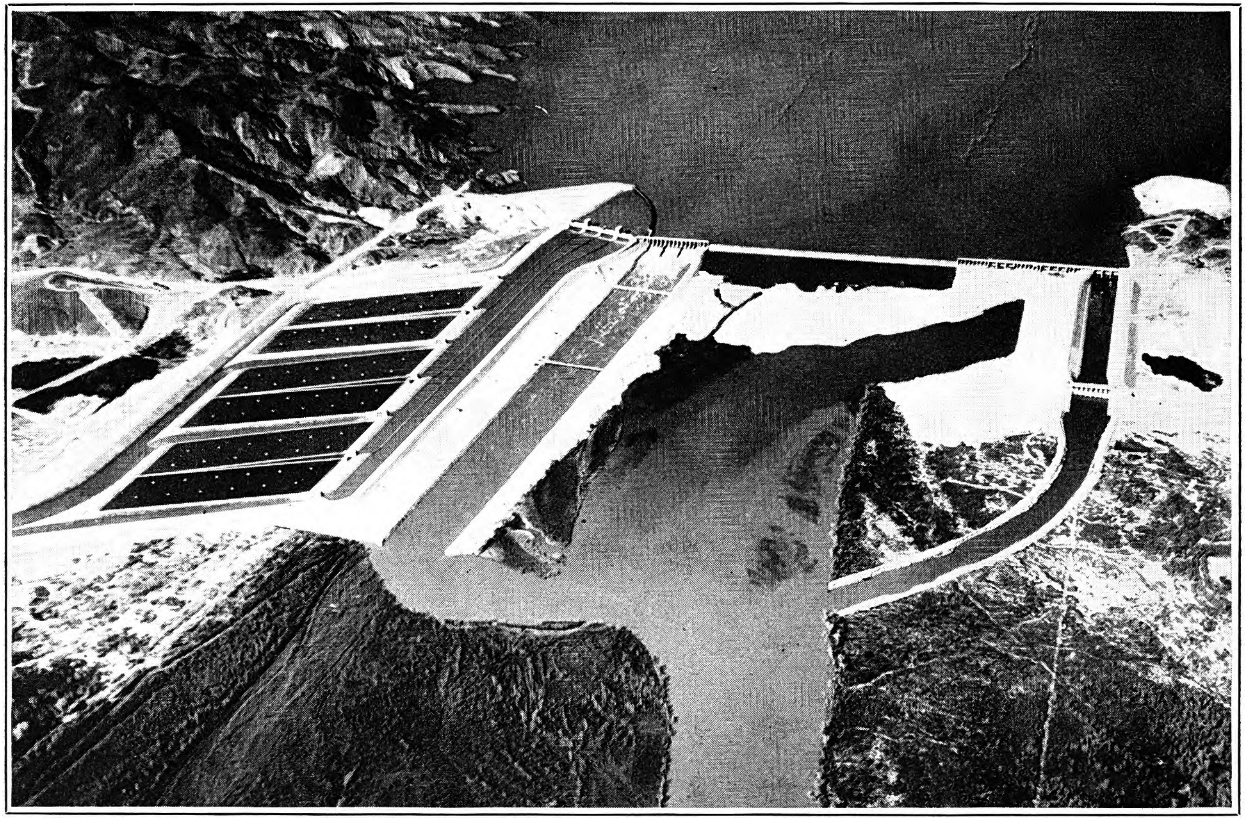


DENVER, COLORADO

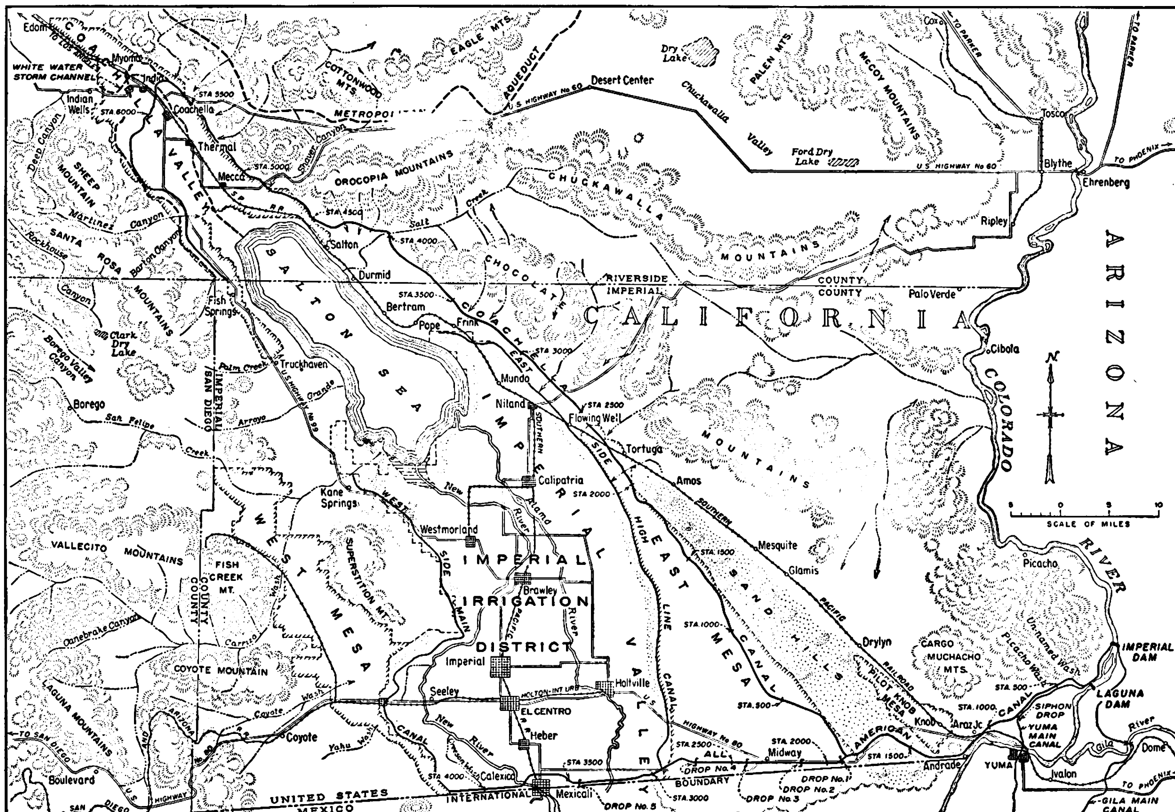
1949

This bulletin is one of a series prepared to record the history of the Boulder Canyon project, the results of technical studies and experimental investigations, and the more unusual features of design and construction. A list of the bulletins available and proposed for publication is given at the back of this report.

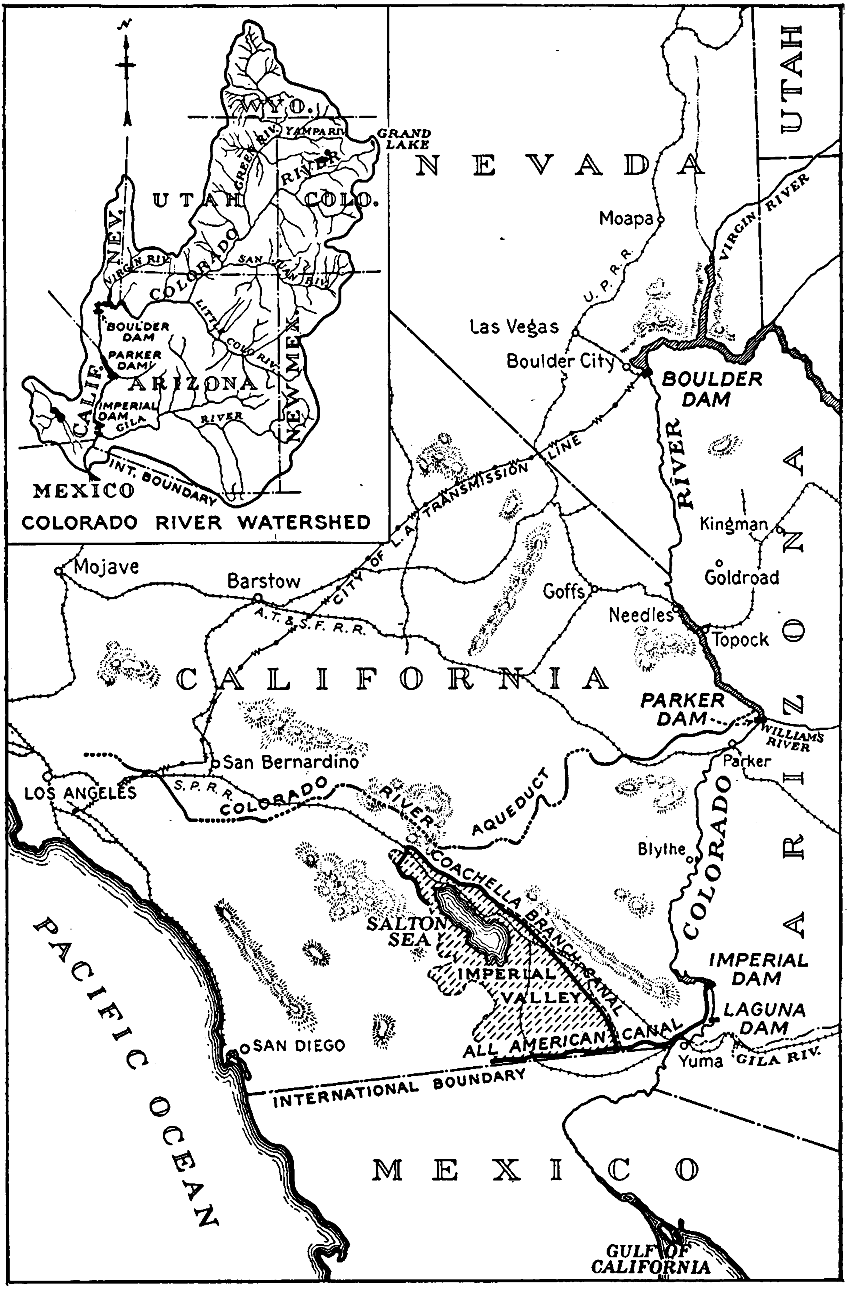
By joint resolution approved by the President April 30, 1947, the United States Congress changed the name of the dam theretofore known as *Boulder* Dam to *Hoover* Dam. This bulletin was too near completion on that date to permit making the appropriate changes in the drawings.



AIR VIEW OF IMPERIAL DAM AND DESILTING WORKS



IMPERIAL DAM AND ALL-AMERICAN CANAL SYSTEM—LOCATION MAP.



BOULDER CANYON PROJECT—LOCATION MAP.

BOULDER CANYON PROJECT

IMPERIAL DAM AND ALL-AMERICAN CANAL STRUCTURES ENGINEERING ORGANIZATION DURING MODEL STUDIES, 1934–39

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FOREWORD

Colorado River, originating in the melting snows of the Wyoming and Colorado Rockies and augmented by rapid run-off from spasmodic rains and cloudbursts over a vast arid region, has menaced life and property in its descent to the Gulf of California since the days of the first covered wagon.

With increased population along the lower reaches of the river the problem of controlling the Colorado became more important. During recent years millions of dollars have been spent in mitigating the evils of silt deposition and in protecting the highly cultivated Imperial Valley lands from annual threats of inundation.

The need for a comprehensive plan of development to check the ravages of Colorado River, to regulate its flow, and to utilize a part of its enormous energy led, first, to investigations by the Reclamation Service of all water storage possibilities; next, to the Colorado River Compact, a mutual agreement for the protection of the seven basin States; and, finally, to the adoption of the Boulder Canyon project, as the initial development.

The Boulder Canyon Project Act, approved December 21, 1928, authorized a total appropriation of $165,000,000 for the various features involved. These include Hoover Dam and appurtenant works, the power plant, the reservoir, and the All-American Canal System. The purposes of the project are: (1) Flood and silt control for protection of land along the lower river; (2) improvement of navigation; (3) river regulation and storage of water for irrigation and municipal use; and (4) development of electric power for domestic and industrial purposes. The project is self-liquidating, largely through contracts for disposal of electrical energy generated by Hoover power plant. The project was constructed and is being operated under the supervision of the Bureau of Reclamation, United States Department of the Interior.

With reference to the All-American Canal system, the Boulder Canyon Project Act provided for the construction of a main canal and appurtenant structures to be located entirely in the United States and to connect Laguna Dam, or some other suitable diversion dam, with the Imperial and Coachella Valleys of California. The canal then serving Imperial Valley had never been satisfactory. A considerable part of it was located in Mexico, and, as stipulated in the right-of-way concession, Mexico was entitled to 50 percent of all water diverted into the canal. The diversion dam, although located in the United States, was a temporary structure that had to be removed during flood periods and replaced after the floods receded. Furthermore, the method of silt disposal was both inadequate and expensive.

It was at first contemplated that Laguna Dam, the diversion structure built for the Yuma project, would be utilized for diversion to a new canal called the All-American. Surveys and investigations, however, showed that a canal 21 feet higher than the Yuma Canal, requiring a new dam about 5 miles farther upstream, would be less expensive, principally for the reason that it involved less excavation through the sand hills. The plan finally adopted required the construction of a dam, upstream from Laguna Dam, to divert water into the All-American Canal. Water for the Yuma project is taken through the All-American Canal, obviating the need for Laguna Dam as a diversion structure. However, it is being maintained to prevent excessive erosion downstream from Imperial Dam.

A great many problems arose in connection with the design of the new dam and canal which could only be solved satisfactorily by studies in the hydraulic laboratory. Some of these problems and their solutions are presented in this bulletin. For a discussion of other features of the Boulder Canyon project the reader is referred to the list of bulletins at the end of this report.

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CHAPTER I—INTRODUCTION

1. General.—As has been stated in the foreword, one of the principal purposes of the Boulder Canyon project is the regulation of the Colorado River and the storage of water for irrigation, power, and municipal use. The flow of the river is extremely variable, but the need for it is much more constant. Therefore, in order to develop the resources of the river to the greatest possible extent, it is necessary to store the water in times of high stream-flow for use when the natural flows are smaller than the quantity needed. This storage of the water has been made possible by the great reservoir named Lake Mead, which was formed by the construction of Hoover Dam. The stored water is released back into the river channel below the dam, flows southward to Needles, thence through the reservoir above Parker Dam, and finally, 300 miles below Hoover Dam, it reaches the head of the delta which was formed by the Colorado River as it entered the Gulf of California. A part of the flow is diverted near Parker Dam for the water supply of Los Angeles and adjacent localities and another small part is diverted to irrigation projects along the river, but the greatest part is used for irrigation near the lower end of the river.

The lands within this region which are supplied with irrigation water from the Colorado River lie partly within the United States and partly within Mexico. The United States lands comprise three general areas: (1) The Imperial Valley area, (2) the Yuma irrigation project, and (3) the Lower Gila River Valley. The first and largest of these areas, known as the Imperial irrigation district, lies in the lower end of California, where 532,000 acres have been developed and were supplied for many years by a canal with headworks a few miles below Yuma, Ariz., as shown in figure 1. Just below the headworks the canal flowed into Mexico, where it supplied water for considerable land before returning to the United States. The second of these areas, the Yuma project, was formerly supplied by a canal heading at the Laguna Dam. This project has an area of 53,000 acres in California and Arizona and can ultimately be extended to include 112,000 acres. The third area is in the lower valley of the Gila River, which enters the Colorado near the town of Yuma.

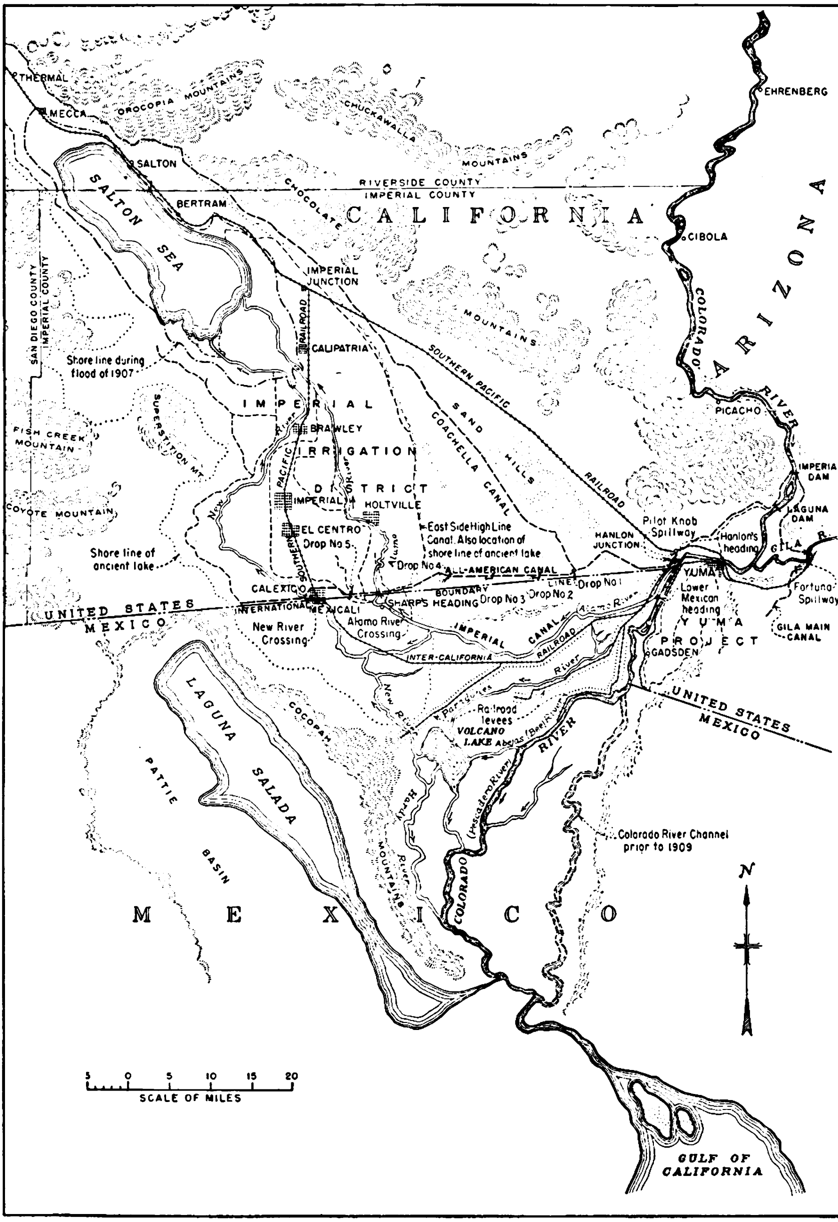


FIGURE 1.—MAP OF IMPERIAL VALLEY AND SURROUNDING TERRITORY.

When the Boulder Canyon project was developed, the Imperial irrigation district and the Yuma project had already been in operation for a considerable period, but the portion in the Gila Valley was largely undeveloped. Great difficulty was experienced in supplying water to the Imperial Valley, due to the large quantities of sediment in the Colorado River water, and expenditures and damages of approximately $1,000,000 per year resulted from sediment deposits. Administrative problems arising from the operation of a canal crossing the international boundary between the United States and Mexico had also proved troublesome. The additional water supply made available by the storage of water in Lake Mead permitted the development of about 80,000 acres of very fertile land, in the vicinity of the Salton Sea, which was too high in elevation to be supplied from the canal leading to the Imperial Valley. It was therefore decided to construct an entirely new and larger canal, called the All-American Canal, as shown in figure 1, which would take water from the Colorado River some distance above the Laguna Dam and carry it, entirely within the United States, to the Imperial Valley; a branch, called the Coachella Canal, was to supply the new areas in the vicinity of the Salton Sea. The All-American Canal would be free from the difficulties of international administration, and was to be supplied with an extensive desilting works which would eliminate the sediment problem. It would incidentally provide for the development of 60,000 kilowatts of electric power. Since 15 miles of this canal paralleled the existing Yuma Canal, the flows were to be combined for this distance. As the plans for the Gila project developed, it became evident that the intake of the canal to supply this region could be close to that of the All-American Canal, and a dam was designed with one of these intakes at either end, to supply desilted water to all of the areas in this vicinity lying within the United States.

The general plan of the Imperial Dam, desilting works, and intakes to the Gila and All-American Canals is shown in figure 2. The dam proper consists of two sections, a sluiceway section about 240 feet long on the right or California side, and an overfall section with uncontrolled crest about 1,200 feet long on the left or Arizona side. Both sections are of hollow concrete construction founded on the sediment of the river bed, with deep cut-offs of sheet piling. Where bearing pressures are high, the sections rest on concrete piles. At the California end of the dam is located the intake for the All-American Canal, which is controlled by four 75-foot roller dam-type gates, each supplying a separate channel. Each of the first three channels leads to a pair of desilting basins, and the fourth leads through a bypass directly to the All-American Canal. The water from each of the first three channels enters a gradually contracting influent channel, dividing each pair of basins, from which it is distributed uniformly to the basins by openings in the side walls. In passing through the basins, the coarser sediment is deposited on the bottom and is scraped toward

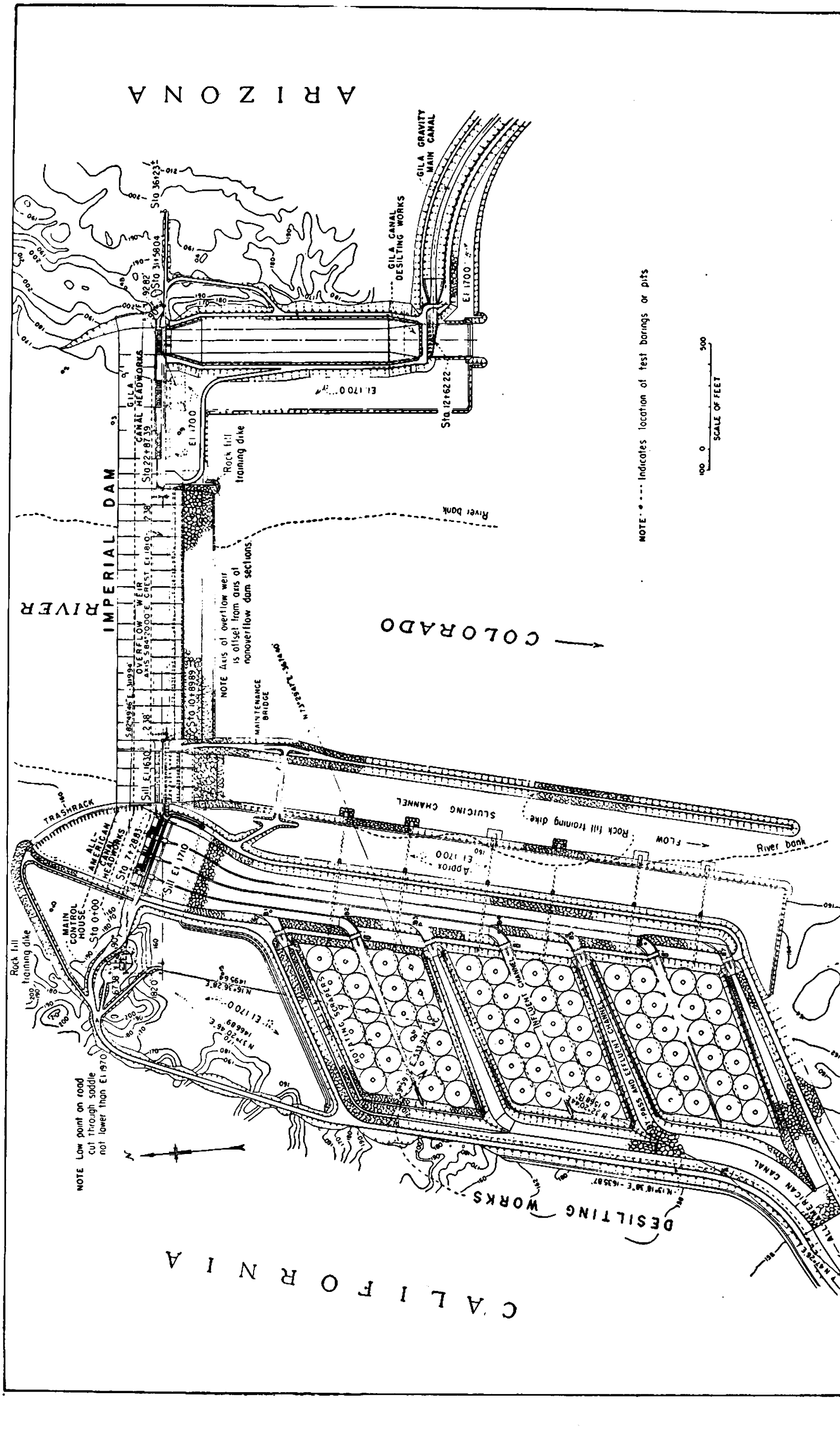


FIGURE 2.—GENERAL PLAN OF IMPERIAL DAM AND DESILTING WORKS.

outlets leading back to the river by a series of rotary scrapers. The clarified water spills over the side walls of the basins into effluent channels which lead to the All-American Canal. The intake for the Gila Canal was placed at the left or Arizona end of the dam. It consists of three gate structures, each of which it is contemplated will lead into a desilting basin; but only one basin has so far been completed. In this basin the sediment settles to the bottom, and at intervals the basin is closed off from the canal and the deposited sediment is washed back to the river through gates leading to culverts passing under the canal.

The All-American Canal, which supplies the Yuma project and the Imperial irrigation district, begins at the desilting works on the California side of the river, as shown in figure 1. At Siphon Drop, 15 miles downstream, the canal supplying the Yuma project branches off. At Pilot Knob, water is supplied to the upper end of the old Imperial Canal, which will supply land in Mexico or be returned to the Colorado River. Just west of the sand hills the Coachella Canal branches off to the northward and irrigates land to the north and northwest of the Salton Sea. The All-American Canal then flows through a series of drops, at several of which power is or will be developed, and then intersects the canals of the Imperial irrigation district system, which were formerly supplied by the Imperial Canal. The Gila Canal flows along the Arizona bank of the Colorado River and crosses the Gila River. Only a small part of the Gila project is at present constructed.

2. Scope of Bulletin.—In connection with the design of the Imperial Dam and desilting works and the canals connecting therewith, a great many problems arose which could only be solved satisfactorily by studies in the hydraulic laboratory. Two of these problems were the determination of the best shape of crest of the overflow dam and of the best design of the stilling pool to dissipate the energy of the overfalling water without undermining or otherwise damaging the structure. A third was the development of the best form of structure for the sluice gates and stilling pool. The development of the roller dam-type headgates for the All-American Canal involved many problems which were attacked through model tests; and other problems requiring model studies arose in connection with the determination of flow conditions through the materials on which the structures were founded, and the developing of measures to insure that this flow would not cause undermining. A large series of questions developed because of the deposit of sediment in the reservoir above the dam, especially in the vicinity of the canal intakes. The design of the desilting basins also involved methods which, to a large extent, were new and had to be developed and perfected. There were a number of problems which involved considerable study in connection with the canals and canal structures, especially the chutes and drops and the structures in which storm drainage water was passed over the canal. The purpose of this report is to describe these problems and the studies which were made on them by the hydraulic laboratory forces of the Bureau of Reclamation, who worked in close coordination with the designing staff. The results of the studies are given in considerable detail to permit more effective use of the underlying principles in the design of other structures of a similar nature which may be built in the future.

3. **Summary of Results.**—The results of the model studies are too voluminous to summarize in a single section of this report; furthermore, the significance of the information would be lost. Accordingly the findings are included in the discussion of each particular feature.

4. **Verification of Results of Investigations.**—Since many of the hydraulic problems encountered in the design of Imperial Dam, desilting works, and pertinent canal structures are without precedent, it is desirable to compare the prototype action with that predicted from the model studies. Although the structures have been completed for several years, no precise studies of the prototype action have been made and are therefore not available for comparison. However, some of the observations made during routine operations are presented in the following paragraphs.

With respect to the desilting basins, it has been noted that a considerable loss of head occurs in the influent slots, whereas the model study predicted a recovery of the velocity head. Measurements and observations should be made to determine the cause of this variation between model and prototype action.

To verify the uplift pressures acting on the dam as determined by hydraulic model and electric analogy studies, pressure cells were installed beneath the spillway. Although some observations utilizing these cells have been made, a complete analysis, reflecting the usefulness of the particular type of cell together with details of installation and location, has not been made. This analysis would be useful in the design of future installations.

Closely related with the amount of percolation is the action of the ejectors on the downstream face of the spillway. These devices are designed to control the elevation of the water inside the dam. Since doubt exists relative to the transference of the action of the ejectors from model to prototype, a field calibration is considered essential. However, to date the spillway has not operated sufficiently to permit such a study.

A determination of the amount of power required to rotate the silt scrapers on the prototype would enable verification of this phase of the hydraulic model design and provide valuable data for the design of future installations.

The ball check valves in the desilting basin of the Gila Gravity Main Canal have not functioned because of lack of water in the earth fill, but a comparatively recent examination revealed that the valves were in proper condition and would operate if necessary.

The stilling pools for the drops and overchutes have performed satisfactorily to date, and no special efforts have been made to correlate their action with that predicted from the model studies. A comparison of water surfaces and velocities in the pools with those predicted from the hydraulic models would be extremely useful in verifying the model studies. Particular attention should be given the condition of the teeth in the dentated sills and steps, as the model revealed a materially different result if the edges became rounded through abrasive action.

5. **Similitude.**—In part VI, bulletin 1, Model Studies of Spillways, Boulder Canyon Project Final Reports, a discussion has been given of the necessary similitude relations between a model and its prototype, together with a bibliography on the subject. Suffice it to say here that all hydraulic model experiments on the Imperial Dam and appurtenant works and the All-American Canal structures were made to conform to the principles of similitude as far as was practicable. Accordingly, model scale ratios, surface roughnesses, velocities, and other factors were considered carefully to assure compliance with the laws applying to the case involved, thereby enabling transference of the model data to the prototype with the least possible error. Where it was impracticable to comply fully with the laws of similitude, analysis of the data took into consideration any digressions therefrom.

6. **The Hydraulic Laboratories.**—Practically all of the studies for the dam and desilting works were conducted at the laboratory in the old customhouse in Denver and in the one at Montrose, Colo., while the tests for many of the canal structures were performed at the laboratory in the new customhouse. The laboratory in the basement of the old customhouse had been built as an emergency to provide for work which could not be handled in the laboratory of the Colorado Agricultural Experiment Station at Fort Collins, Colo. It was constructed in very crowded quarters, where the available floor space was only 1,700 square feet, badly broken by walls and columns, and the head room was only 10 feet. Although the lay-out of the laboratory and models was dictated largely by the space available, and the apparatus was therefore not as convenient to operate as if a specially built laboratory had been available, it is believed that the restricted quarters had no appreciable effect on the quality of the results obtained. The Montrose laboratory was a large outdoor one which received water from the main irrigation canal of the Uncompahgre irrigation project. The customhouse laboratory, built in the basement of the new cusomhouse for the use of the Bureau of Reclamation, is fully equipped with the latest types of equipment. More details of these facilities will be given in connection with the individual tests.

7. **Personnel.**—The hydraulic experiments discussed in this report were begun under the supervision of E. W. Lane and were completed under the supervision of J. E. Warnock. Because of the large number of projects and

individuals involved and the frequent shift in personnel, it is impracticable to outline in detail the part taken by each individual. The following are the names of the engineers who were engaged in the studies, and to whom their success was largely due:

J. W. Ball	J. M. Buswell	R. C. Haven, Jr.	W. M. Borland
D. P. Barnes	F. B. Campbell	D. J. Hebert	D. C. Weed
J. N. Bradley	F. R. Cline	G. J. Hornsby	T. G. Owens
C. W. Thomas	W. E. Corfitzen	L. H. Kristof	F. L. Panuzio
D. M. Lancaster	J. H. Douma	Fred Locher	Samuel Schulist
H. W. Brewer	J. B. Drisko	F. C. Lowe	V. L. Streeter
H. D. Briley	H. G. Dewey, Jr.	C. H. Morris	R. C. Vierck
L. R. Brooks	H. M. Martin	E. J. Nemmers	R. K. Van Horn
R. R. Buirgy	R. A. Goodpasture		

Others, too numerous to mention individually, were associated with these experiments, in the design, construction, and operation of the models and in the analysis of the data. Without their assistance, the successful conduct of the work described herein would not have been possible.

The text was written by E. W. Lane and was reviewed and edited by D. M. Lancaster and E. H. Larson. Drawings were prepared for publication by R. H. Williams and D. F. Wilson.

8. Bibliography.—The following bibliography, relating to the model studies described in this bulletin, consists entirely of unpublished reports originating in the hydraulic laboratory of the Bureau of Reclamation at Denver, Colo. A list of the bulletins being published as Boulder Canyon Project Final Reports, with prices shown for those now available for distribution, is given at the end of this report.

1. Ball, J. W.
   Hydraulic Model Studies for the Design of the Pilot Knob Wasteway, All-American Canal, Boulder Canyon Project, Hyd. Lab. Report No. 184, Oct. 12, 1945.
2. Bradley, J. N., J. B. Drisko, and D. J. Hebert.
   Hydraulic and Electric Analogy Model Studies of the Proposed Imperial Dam and Appurtenant Works, Preliminary Report No. 2, Tech. Memo. No. 471, July 15, 1935.
3. Cline, F. R.
   Preliminary Report on Model Studies of Influent Slots in All-American Canal Desilting Works, Memo. to Chief Designing Engineer, May 7, 1937.
4. ———,———.
   Preliminary Report on Model Studies of New River Crossing and Wasteway, All-American Canal, Hyd. Lab. Report No. 27, Sept. 23, 1937.
5. Douma, J. H.
   Preliminary Report on Model Tests for the Wasteway Chute and Stilling Pool of the Alamo River Crossing, All-American Canal, Boulder Canyon Project, Office Memo., June 9, 1938.
6. ———,———.
   Model Study of Siphon Drop Turnout, All-American Canal System. Memo. to J. E. Warnock, July 20, 1939.
7. ———,———.
   Hydraulic Model Tests of Wash Overchutes and Wasteways, Gila Gravity Main Canal, Gila Valley Project, Hyd. Lab. Report No. 185, Oct. 22, 1945.

8. Locher, Fred.
Model Studies of the Coachella Canal Wash Overchutes, All-American Canal System, California, Hyd. Lab. Report No. 111, Apr. 20, 1942.
9. Pomeroy, R. R.
Hydraulic Model Studies of Coachella Power Drop, Station 290 + 00, All-American Canal Project, Hyd. Lab. Report No. 102, Jan. 31, 1942.
10. Streeter, V. L.
Hydraulic Model Studies of New River Crossing Automatic Gate. Memo. to the Chief Designing Engineer, Apr. 20, 1939.
11. Thomas, C. W., and H. D. Briley.
Results of Model Studies for Groins on Upstream Apron of Imperial Dam, Memo. to Research Engineer Warnock, Sept. 14, 1936.
12. Thomas, C. W., H. D. Briley, and J. H. Douma.
Model Tests Gila Valley Canal Headworks and Ball Check Valves for the Desilting Basin, Imperial Dam, All-American Canal Project, Hyd. Lab. Report No. 183, Oct. 1, 1945.
13. Thomas, C. W., J. E. Warnock, and C. P. Vetter.
Progress Report on Model Studies at Montrose, Colorado, of the Imperial Dam and Desilting Works, All-American Canal Project, Hyd. Lab. Report No. 9, July 25, 1936.

CHAPTER II—SPILLWAY STUDIES

INTRODUCTION

9. **The Prototype.**—The primary purpose of Imperial Dam is to raise the water in the Colorado River so it can be diverted readily into the All-American and Gila Canals, and to provide for the control of the flow. The purpose of the spillway section is to discharge flows which may come down the river in excess of the capacity of the sluiceway gates, which ordinarily control the flow passing the dam. Because of the necessity of founding the dam on the fine sand of the river bed, the hollow concrete type of construction was selected to reduce the bearing pressures to values the river bed material could safely carry. A filling of sand and gravel ballast was added to give weight and stability.

The spillway has a height of 31 feet, and as first designed, had a cross section as shown in figure 3–A. The shape as finally developed is shown in figure 3–B. Briefly, the design considered includes a long upstream apron with three rows of sheet piling to reduce seepage beneath the dam, a hollow concrete Ambursen-type dam, and a heavy downstream slab forming a stilling pool, with a sheet pile cut-off at the downstream edge Downstream from the stilling pool is a wide fill of riprap.

10. **Purpose of the Spillway Model Tests.**—Since the spillway is founded on unconsolidated material, any erosion immediately downstream from the structure due to flow over the spillway would affect seriously its stability. It was urgent, therefore, that an efficient stilling pool be developed to dissipate the energy and reduce the velocity of flow in a hydraulic jump, particularly since it was known that velocities of only 6 feet per second were sufficient to scour the river bed material. Additional complications were foreseen from either (1) a possible lowering of the tailwater, due to the degrading of the river bed downstream as a result of a clarification of the water by settling of the sediment in the pool above the dam, or (2) a possible raising of the tailwater from silt dumped into the river from the desilting basins. Accordingly, the model tests of the

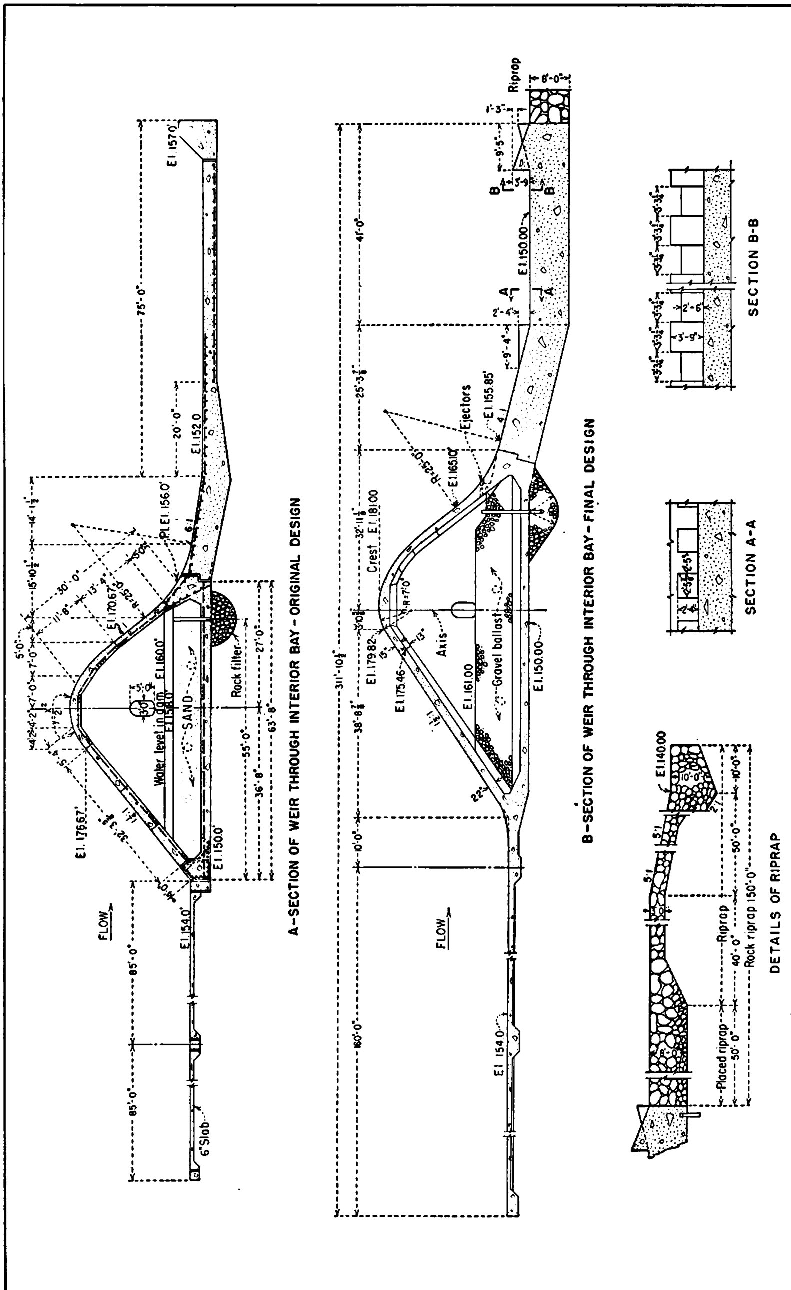


FIGURE 3.—ORIGINAL AND FINAL DESIGN OF IMPERIAL DAM SPILLWAY.

spillway were primarily for the purpose of evolving a hydraulic-jump stilling pool which would minimize scour in the river channel just below the dam, even if the tailwater level for a given discharge varied over a rather wide range.

With a dam of the hollow concrete type it was important that there be no negative pressures beneath the flowing stream which might tend to produce cavitation or to lift the downstream face of the dam. The model tests on the spillway section of the dam therefore covered the two problems of preventing vacuum and scour.

To develop the most desirable form of stilling pool, many tests were performed by changing its elevation and length, and the types and locations of sills. Each arrangement was investigated at various tailwater elevations, at a discharge representing the maximum expected, which was at first estimated at 150,000 second-feet but later changed to 134,500 second-feet. Much has been learned of stilling-pool design since these tests were started in 1935, and it would not now be necessary to conduct as many tests as were then made, but even now a large number would be necessary to insure the development of as satisfactory a structure for the adverse tailwater conditions which may exist at Imperial Dam as was secured by these tests.

INITIAL STILLING-POOL TESTS

11. Model of the Preliminary Design.—The first tests were made on a preliminary shape of spillway section. Because of limited laboratory space and pump capacity, only a 60-foot length of the spillway was constructed in a 1:30 scale model in the old customhouse laboratory, as shown in figure 4. The model crest was made by shaping 20-gage sheet metal over four metal ribs. Fifteen ⅛-inch piezometers were installed in staggered positions in the crest for measuring pressures. The downstream apron was fabricated from heavy sheet metal soldered to angle irons, which were pinned to the dam at the upstream end and bolted to the floor of the flume at the downstream end. Four piezometers were installed along the center line of the apron to measure pressures on the floor, and a wooden sill was fastened to the end of the apron. The paving of the river bed upstream from the dam and downstream from the stilling pool was represented by sand. To observe the performance of the stilling pool, glass windows were inserted in the sides of the steel flume.

The pressures at the piezometers in the model were obtained by connecting each piezometer to a glass manometer, by means of rubber tubing, and obtaining the height of water in the tube, care being taken to exclude all air from the rubber tubing. The head on the crest of the model was measured with a point gage located in the flume upstream from the dam, and the discharge was measured by a 90° V-notch weir. The tailwater elevation was observed at three piezometers located 6, 10, and 14 feet down-

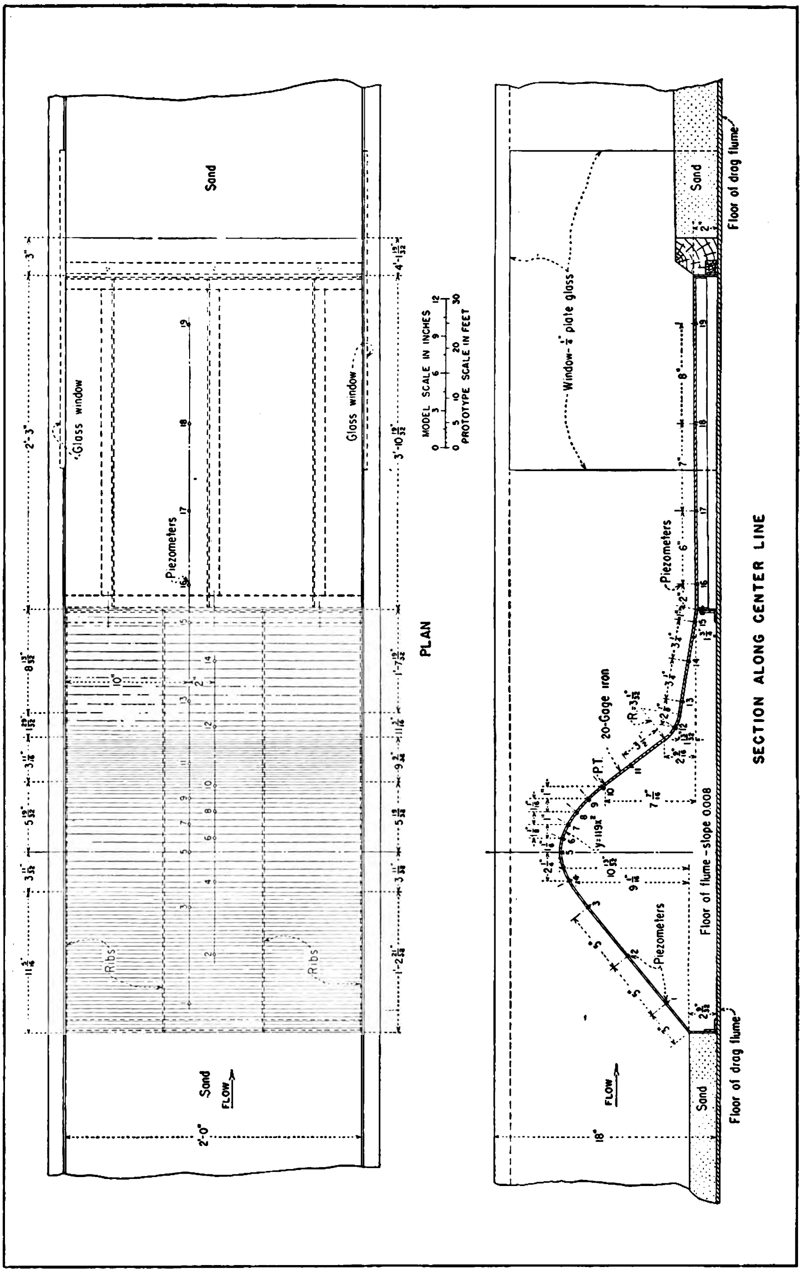


FIGURE 4.—MODEL OF PRELIMINARY DESIGN OF IMPERIAL DAM SPILLWAY.

stream from the axis of the dam. Profiles of the water surfaces and sand surfaces were obtained along the center line of the model with a movable point gage mounted on a carriage which could be moved longitudinally along the flume on two level bars, one mounted on each side of the flume.

12. Spillway 1,072 Feet Long, Pool Floor at Elevation 152.0.—The first design tested was that shown in figure 4. It was tested at a flow representing the maximum unit discharge of 140 second-feet per foot of crest, equivalent to 150,000 second-feet for a length of spillway of 1,072 feet. The tailwater for this discharge was varied from elevation 160 to elevation 168. The latter elevation was established by the requirement that the toe of the hydraulic jump should not rise above elevation 158, at which point it would interfere with the action of the ejectors; the lower elevation was possible because of retrogression which may occur in the river channel downstream. Unless the stilling pool could perform properly at the two extreme elevations, it might be unsatisfactory. In the following paragraphs the term "workable tailwater range" is used to indicate the difference between the maximum and minimum tailwater levels at which a model operated satisfactorily.

Figure 5, test 1–1, shows a profile of the water surface and river bed for the initial test. In this test, the pool action was satisfactory for all discharges when the tailwater was in the region of elevation 168, but, when the tailwater was dropped to elevation 166, the hydraulic jump moved out of the pool at maximum discharge. The profile of the sand surface indicated a tendency for excessive scour. The sizes of the sand grains used in these experiments were not to scale, and the results of scour tests on the spillway and sluice models were therefore only qualitative; nevertheless, by using sand of the same size throughout all tests, comparisons were possible for determining which one of the pool designs was the most satisfactory from the standpoint of scour.

In an attempt to improve the performance of the stilling pool, four additional tests were made at the maximum discharge of 140 second-feet per foot of crest, with tailwater at elevation 168.0. During these tests, a comparison was made of the effectiveness of the hydraulic jump and the degree of scour in the sand bed for various types of sills placed at the end of the horizontal apron. Two of these tests are indicated in figure 5, tests 3–1 and 5–1. It can be seen that, regardless of the size of sill used, the jump generally moved downstream as the tests progressed, although the tailwater was held constant. This downstream movement of the hydraulic jump was at first retarded by a sand bar developed by scour, but as this bar washed away the movement continued. As a result of these tests, it was apparent that the original design would be inadequate, and that the elevation of the river bed downstream from the dam was important.

13. Spillway 1,300 Feet Long, Pool Floor at Elevation 152.0.—Since the model demonstrated that the original design of the stilling pool failed

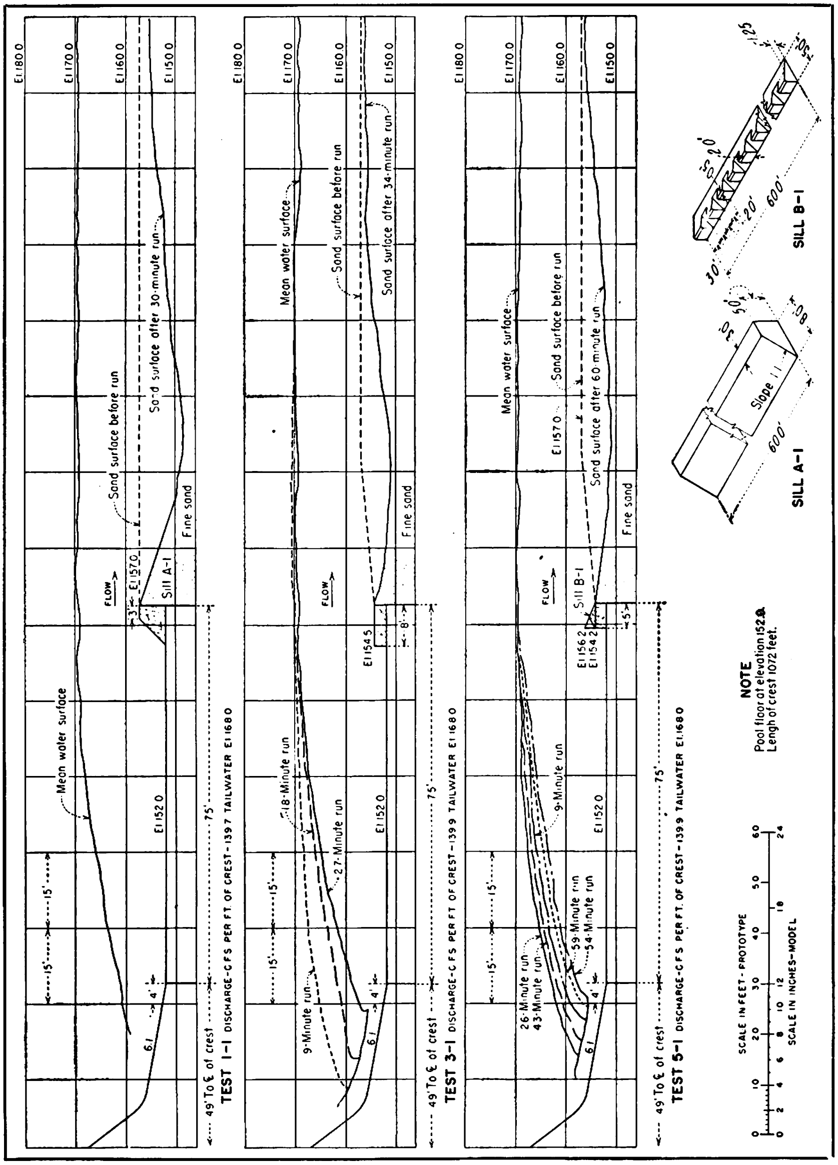


FIGURE 5.—PROFILES OF WATER AND SAND SURFACES FOR PRELIMINARY DESIGN OF IMPERIAL DAM SPILLWAY.

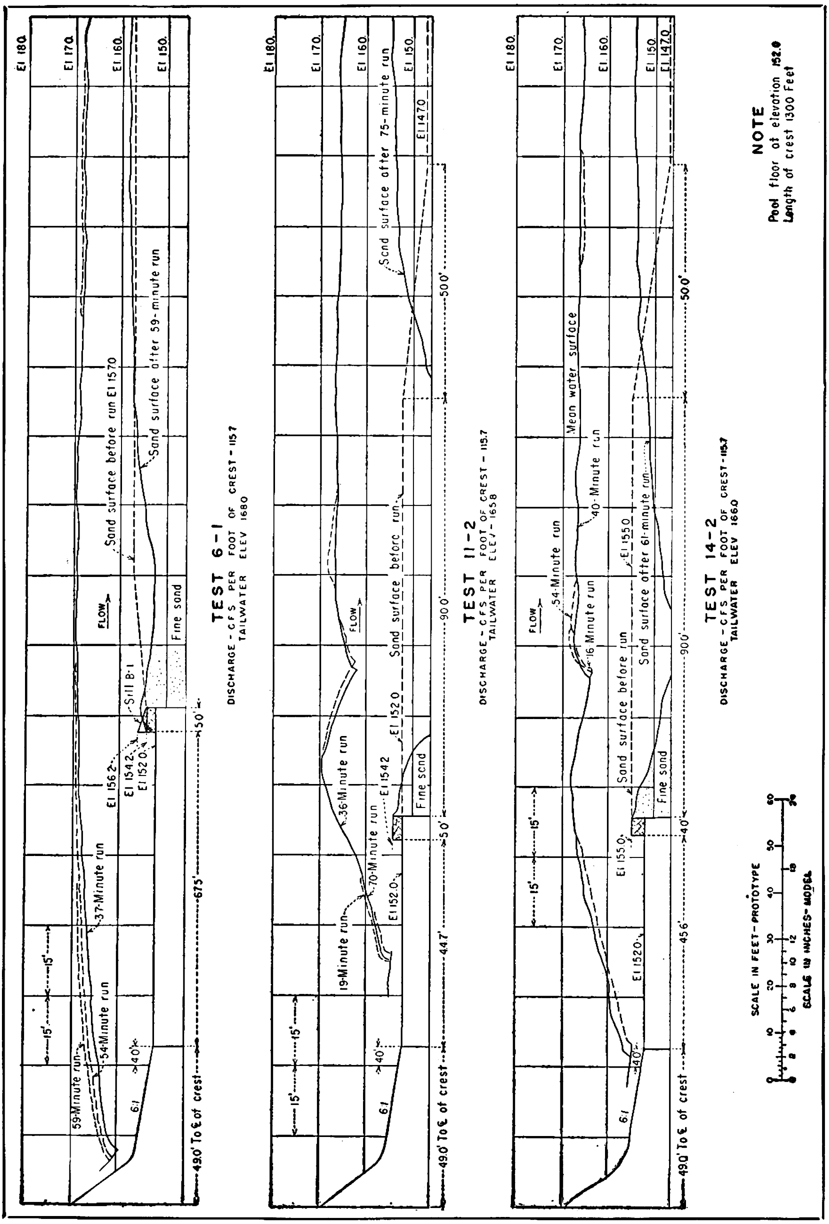


FIGURE 6.—PROFILES OF WATER AND SAND SURFACES FOR IMPERIAL DAM STILLING BASIN.

to satisfy the requirements, the length of the spillway was increased to 1,300 feet. It was reasoned that this would reduce the discharge per foot from 140 to 115 second-feet and correspondingly reduce the energy to be dissipated per foot in the stilling pool, as well as reduce the length of the hydraulic jump. Tests on the longer spillway showed this reasoning to be partly correct, but a satisfactory performance for a wide variation in tailwater was still not realized. In test 6–1, figure 6, the length of pool (which in this report is measured from the foot of the incoming slope to the downstream end of the sill) was reduced from 75 to 72.5 feet, and the height of sill was increased to 4.2 feet. The profiles show that the scour was decreased slightly, but that as the test progressed, the front of the jump advanced up the 6:1 slope of the apron instead of downstream as occurred for tests on the shorter spillway. This change was caused by a sand bar which formed above the original sand bed surface, reducing the effective area of the channel. With a smaller discharge per foot the sand bar remained stationary. The danger of the jump reaching the ejectors, in addition to the unsatisfactory pool operation at lower tailwater elevations, eliminated this design. In test 11–2, figure 6, the pool was shortened to 49.7 feet, the sill was reduced in height to 2.2 feet, and the tailwater was lowered to elevation 165.8 for the maximum discharge. At the beginning of the test the jump swept out of the pool. This produced excessive erosion and formed a large sand bar, which eventually caused the jump to return to the pool; also, an undesirable secondary roller developed just downstream from the apron. Increasing the height of sill to 3.0 feet as shown in test 14–2, figure 6, gave only slight improvement in the scour condition and position of the jump in the pool, even though the sand bed was raised to the top of the sill at the start of the test. Placing the sill farther upstream in the pool, to utilize part of the concrete paving as an apron downstream from the sill, somewhat improved the operation at higher tailwater, but at lower tailwater the performance remained unsatisfactory.

14. Spillway 1,732 Feet Long, Pool Floor at Elevation 152.0.—Since a slight improvement was produced by increasing the crest length from 1,072 feet to 1,300 feet, another increase was made to 1,732 feet. This gave a further reduction in discharge per foot from the previous values of 140 and 115 to 86 second-feet. Several comparative tests were made at this discharge, each test differing with respect to the type of sill, length of pool, and shape of the sand bed. The arrangements for six of the most representative tests and the details of the sills are shown in figure 7. An accurate record of the shapes of the sand beds was not kept for this series of tests; instead, particular attention was paid to the tailwater elevation at which the point of the jump advanced up the face of the dam to elevation 158.0, and the tailwater elevation at which the jump swept out of the pool. The desired stilling pool was the one that required the highest tailwater to force the toe of the jump up the dam to elevation 158.0 and

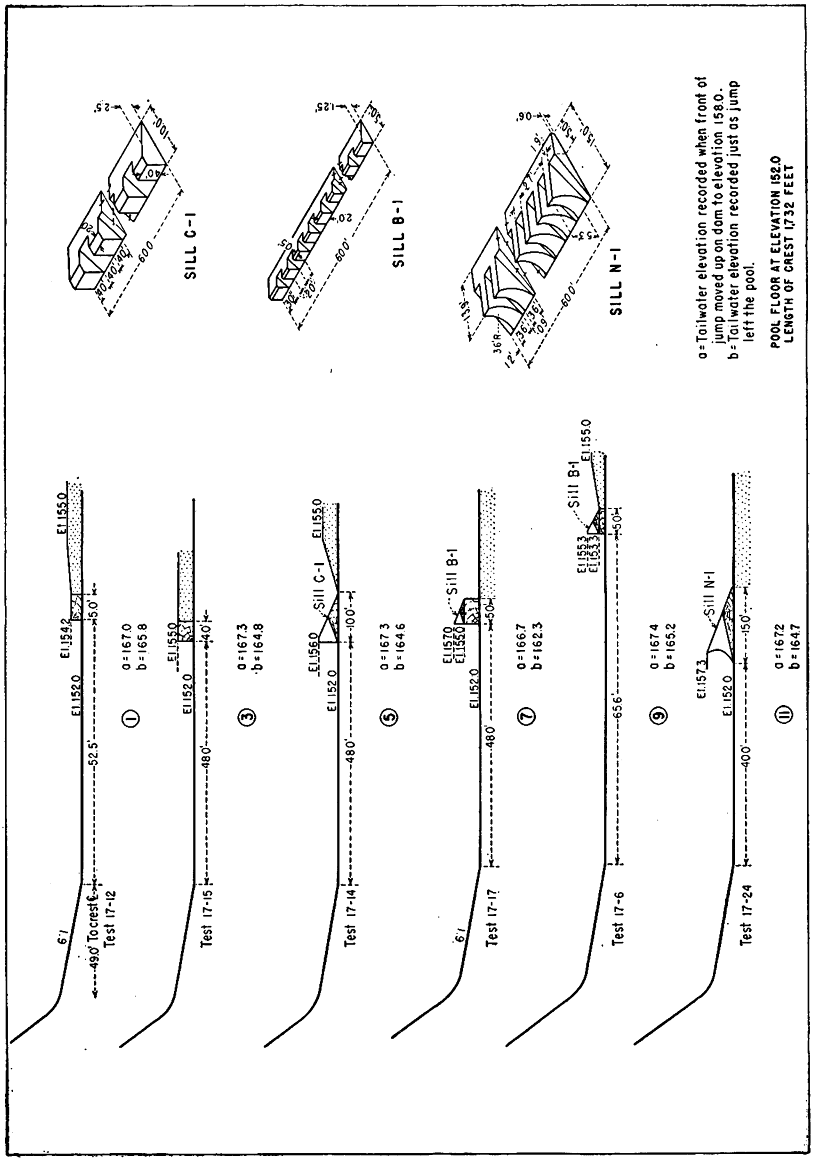


FIGURE 7.—SOME DESIGNS OF STILLING POOL TESTED FOR IMPERIAL DAM.

the lowest tailwater to maintain a jump in the pool. The best arrangement in this series was design 7 in figure 7, which had a workable tailwater range of 4.4 feet.

15. Spillway 1,300 Feet Long, Pool Floor at Elevation 150.0.—All of the tests so far described definitely showed the pool designs would not meet the requirements outlined. Some benefit could be obtained by using a longer spillway, but the variation between the highest and lowest tailwater elevations for the best designs was not sufficient. Accordingly, the 1,300-foot length was retested, but with the depth of the stilling pool increased by lowering the floor from elevation 152.0 to elevation 150.0. A spillway 1,300 feet long was selected because previous tests showed somewhat better results for this length than for a length of 1,072 feet, while the 1,732-foot spillway did not show enough improvement over the 1,300-foot design to warrant the added length. The sloping portion of the apron was changed from 6:1 to 4:1 to confine the jump to a shorter horizontal distance. Both small stones and sand were used to indicate the severity of scour in this series, the former being used when comparisons were made of the tailwater range only.

Comparative tests were continued with this revision, the most representative of the group being shown in figure 8. As before, the arrangements differed in types of sills, length of pool floor, types of stepped aprons, combinations of stepped aprons and sills, and shape of the sand bed. Although there was a wider variation in the tailwater levels, or workable tailwater range, for some of these arrangements, difficulties were encountered from the formation of a secondary roller immediately downstream from the end of the stilling pool. Test 21, figure 9, shows this to occur at the lower tailwater elevations. For test 22–1, however, this secondary jump did not occur even at still lower tailwater, because the elevation of the river bed was raised 3 feet higher; nevertheless, the bottom velocities were excessive, and undesirable surface waves developed farther downstream. Elimination of waves and high bottom velocities was accomplished in succeeding tests by lengthening the pool and using two sills at the end of the apron.

It will be noticed in figures 8 and 9 that a stepped apron was placed at the intersection of the 4:1 apron with the pool floor. This device divides the sheet of water entering the pool into more or less individual jets, making the energy dissipation take place in a shorter length. As a result the hydraulic jump is shortened and more effective energy dissipation is obtained.

An interesting comparison of the effects of placing sills with their vertical faces upstream and downstream is shown in tests 29 and 30, figure 9. The best results are shown clearly to exist when the vertical faces of the sills are upstream.

Designs 42 and 48 in figure 8 were the most promising in this series, especially with respect to the tailwater range. Design 42, which had a

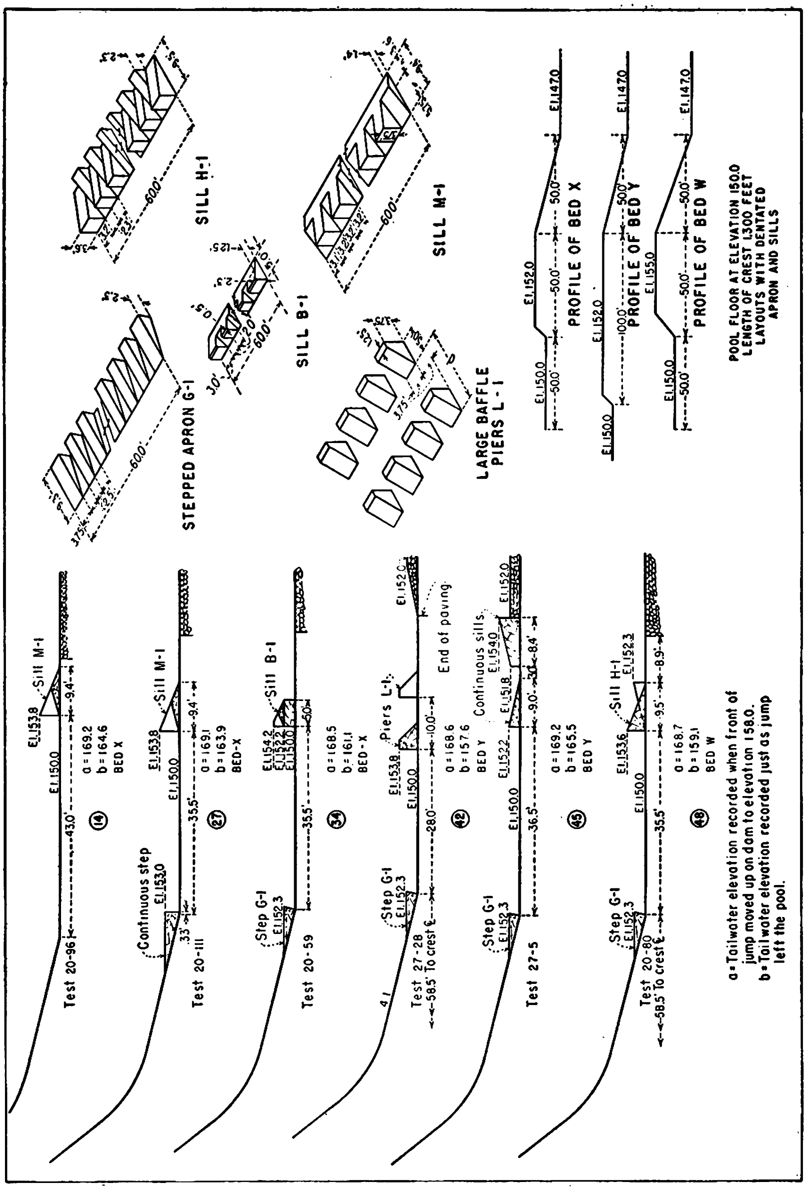


FIGURE 8.—ADDITIONAL DESIGNS OF STILLING POOL TESTED FOR IMPERIAL DAM.

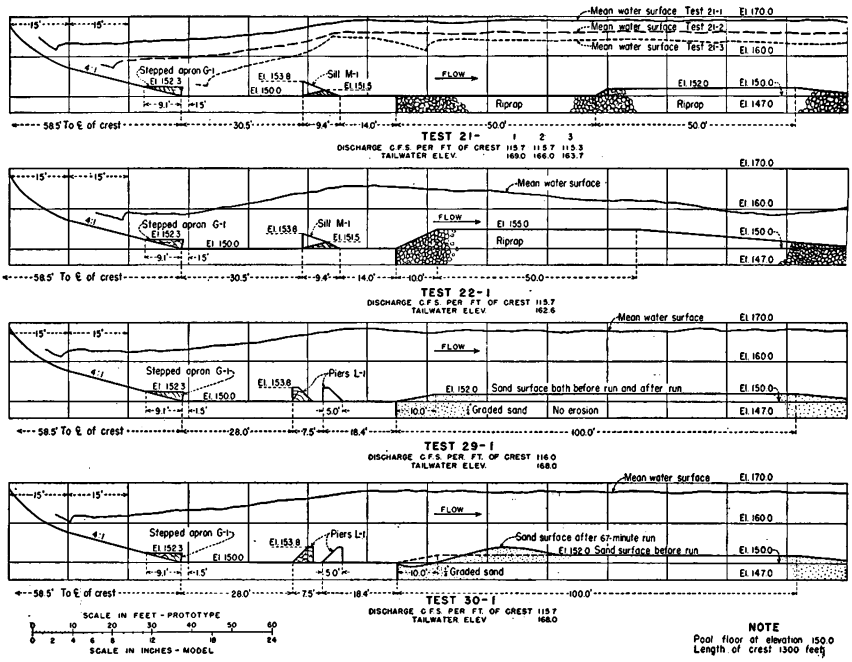


FIGURE 9.—PROFILES OF WATER AND SAND SURFACES FOR IMPERIAL DAM STILLING POOL.

tailwater range of 11 feet with a minimum tailwater at elevation 157.6, is the design used in test 29, figure 9. Design 48 had a tailwater range of 9.5 feet with a minimum tailwater at elevation 159.1, for the maximum discharge.

FINAL STILLING-POOL TESTS

16. Effect of River Bed on Stilling Pool.—It was noticed during the initial stilling-pool tests that the performance of any design was influenced by changes occurring in the river bed as a result of scour, or, when scour was not excessive, by the initial shape of the river bed selected for a test. Therefore, before resuming the detailed tests of the stilling pool with a revised spillway design, a study was made to determine more definitely the effect of the shape of the river bed downstream on the hydraulic performance of the stilling pool.

Using a stilling pool similar to design 48, figure 8, and a river bed constructed of riprap so its shape would not change during a test, an investigation was made at the maximum discharge to determine the tailwater range for several bed shapes. It was found (1) that the tailwater range remained practically constant for beds more than 35 feet long, but decreased for shorter beds; and (2) that, as the elevation of the river bed was increased from 150.0 to 155.0, the workable tailwater range increased, the greatest range occurring with the longer beds for a given bed elevation. Since the bottom velocities increased for the higher beds and nullified any gain in tailwater range, it was concluded that a solution to this phase of the design would be found with a bed at a lower elevation in combination with a sill which would be effective in confining the jump to the stilling pool.

17. Spillway 1,335 Feet Long, Pool Floor at Elevation 150.0.—Detailed tests of the stilling pool were resumed, using a river bed at elevation 150.0 and an arrangement similar to design 48, figure 8. Following the procedure used in the initial tests, the type of stepped apron and sill was varied for numerous lengths of pool, and comparisons were made of each arrangement by determining the range of tailwater and relative scour for the maximum discharge of 150,000 second-feet. It was determined finally that either baffle piers of the L–1 type, figure 8, or a modified Rehbock sill, such as sill H–1 in figure 8, would produce the best hydraulic performance.

Since the effectiveness of any sill depends on the sharp corners of the individual teeth, a study was made to determine the effect on the hydraulic performance of a stilling pool in the prototype, should the sharp corners be removed by abrasion. By chamfering the sharp edges on the model baffle piers to 3 inches (prototype), the workable tailwater range was decreased nearly 30 percent. A reduction of only 16 percent was obtained when the distance between the rows of teeth in the direction of the flow was reduced to zero, the spacing between the teeth in each row remaining the same. Since this approximated a Rehbock sill, it was concluded that the baffle piers would not be as practicable to use as the modified Rehbock sill.

18. Final Design of Stilling Pool.—From the results of the final tests it was decided that the most efficient and economical stilling pool would be as shown in figure 10. Stepped apron G–1, figure 8, was placed at the toe of the sloping apron; and sill S–1, figure 10, was placed on the end of a pool floor 41.5 feet long, at elevation 150.0. The workable tailwater range for this design was 5 feet. Figure 3–B shows the prototype design of this arrangement, with minor revisions as described later. Water surface profiles and piezometer pressures on the pool floor were recorded for eight discharges. The results for the maximum discharge are shown in figure 10.

To complete the studies on this design, a paint test was made by passing the water through the model covered with a thick coat of fresh paint. Figure 11–A shows the stilling-pool arrangement at the conclusion of the test. The pattern formed by the flow as it passes over the stepped apron may be seen in figure 11–B. The function of the stepped apron is to develop many small eddies within the flow to increase the intensity of the turbulence and thereby increase the rate of energy dissipation. It may be seen in

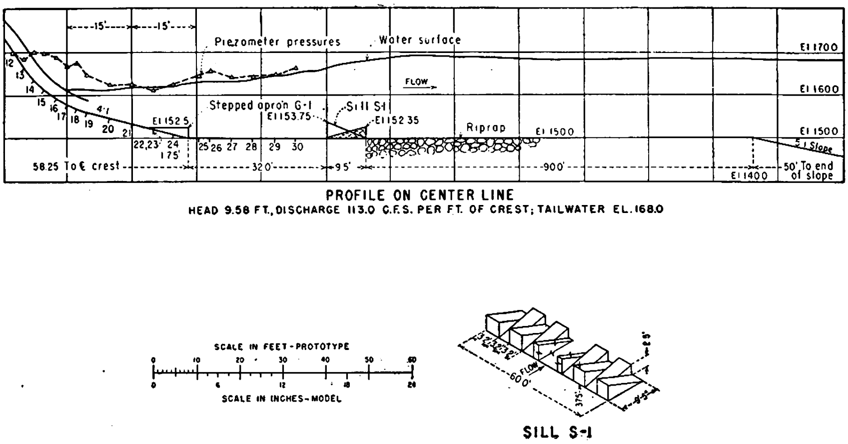
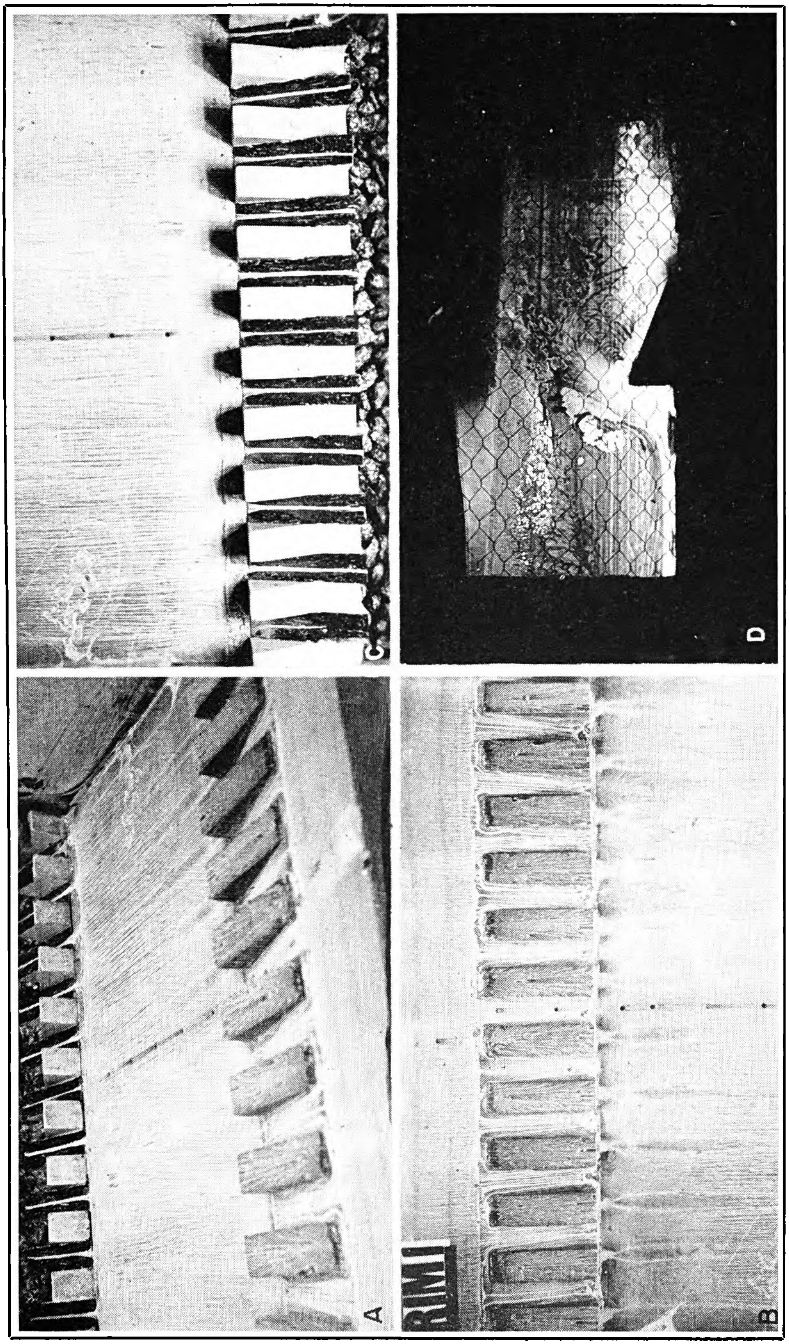


FIGURE 10.—RECOMMENDED DESIGN STILLING BASIN FOR IMPERIAL DAM—PIEZOMETER PRESSURES AND WATER SURFACE PROFILE.

figure 11–B that this is accomplished by the transverse spreading of the flow as it passes over the dentates and as it passes between them. In addition, the teeth cause the jet to expand vertically while at the same time separating it into more or less individual jets, thereby increasing the turbulence throughout the entire depth of flow and thus reducing the velocity. The same action also takes place at the sill at the end of the stilling pool, except that in this case the losses also include those due to impact against the vertical face of the sill. Figure 11–C indicates the flow pattern at the sill; the thin white streaks occur between the teeth with vertical faces and on top of the teeth sloping downstream (see sill S–1, fig. 10). An equally important function of the sill is to deflect the flow vertically away from the river bed immediately downstream; figure 11–D shows how effectively this is accomplished. As a result of using a stepped apron with a sill, it is possible to reduce the length of stilling pool to two-thirds of the length required when no such devices are employed.

MODIFICATIONS IN FINAL SPILLWAY DESIGN AS RESULT OF STILLING-POOL TESTS

19. **Final Modifications.**—Following the completion of the final stilling-pool tests and the dismantling of the model, minor changes were made in the design of the spillway. The over-all length was reduced from 1,335 to 1,197.5 feet, but the discharge per foot of length was not changed. The over-all maximum discharge of 150,000 second-feet was maintained by increasing the capacity of the sluices, as discussed later. The length of the stilling pool was reduced from 41.5 to 41.0 feet. To match the expansion joints in the sill with those in the pool floor, the width of teeth in the sill was increased from 3.20 to 3.27 feet. The spillway as finally constructed is shown in figure 3–B.



A. Looking Downstream. C. Plan View of Modified Rehbock Sill—Flow From Top to Bottom.

B. Plan View of Stepped Apron—Flow From Top to Bottom. D. Side View of Pool Showing Action of Dentates.

FIGURE 11.—PAINT TESTS OF FINAL DESIGN STILLING POOL FOR IMPERIAL DAM.

20. Thoroughness of Stilling-Pool Tests.—To prevent the account of the spillway tests from being too voluminous, only the most important steps have been discussed. The tests were much more complete than this discussion would indicate, and include a total of 414 separate tests. The investigation of roughening of the pool floor was particularly complete, including the study of 32 forms of sills, 7 types of baffle piers, and 4 types of stepped aprons. These forms are shown in figures 12 and 13.

TESTS OF PRESSURES ON FACE OF SPILLWAY

21. Original Design.—Prior to the completion of the initial stilling-pool tests, pressure measurements were made on the spillway face to ascertain the extent of negative pressures, if any, for various discharges. Piezometers were located as shown in figure 4. These measurements covered a range of discharge from 38 to 166 second-feet per foot of crest, with the approach floor at elevation 154.0. Negative pressures of 2 feet of water were developed on the downstream face of the spillway for discharge between 88 and 166 second-feet per foot. Results for the maximum unit discharge of 115 second-feet per foot of crest are given in figure 14, test 16–1.

22. Effect of Depth of Approach.—Since the reservoir above Imperial Dam will gradually fill with silt, the pressure tests were continued to determine the effect of the depth of approach on the spillway pressures. Test 16–1 was made with the reservoir floor at elevation 154.0, and tests 41–1 and 42–1 were made with the reservoir floor at elevations 172.5 and 179.0, respectively, for a maximum discharge of 115 second-feet per foot of crest. The results in figure 14 show that the magnitude of the negative pressures increased as the depth of approach decreased, because the shape of the lower nappe changed with the velocity of approach.

23. Effect of Negative Pressures.—The necessity for consideration of the negative pressures on the spillway of Imperial Dam, to prevent the possibility of a partial vacuum tending to lift the downstream face, has already been mentioned. Also, when these pressure tests were made in 1934 there was less confidence than exists at present regarding the transference of model pressures to the prototype if the pressures were below atmospheric. Furthermore, there was less understanding of the phenomenon of cavitation as related to the separation of nappes from spillway crests.

Since the model tests of the spillway were made prior to general familiarity with the mechanism of cavitation, and because no application of the underlying principles had been made to the flow in open channels, it was believed necessary to eliminate all negative pressures on the spillway crest. Even though present-day practice occasionally permits some negative pressures as a means of increasing the coefficient of discharge, the presence of negative pressures indicates a separation of the nappe from the boundary

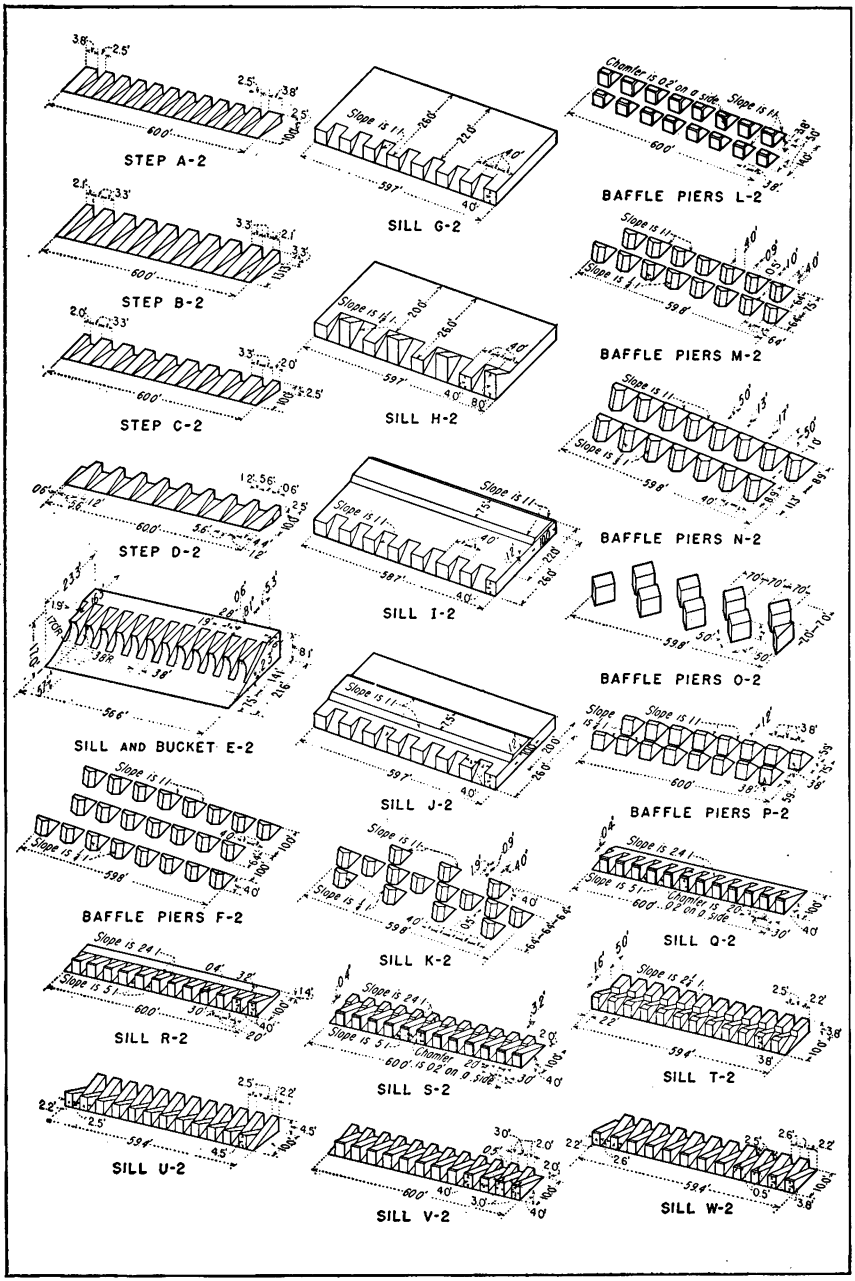


FIGURE 12.—SOME SILLS AND STEPPED APRONS USED IN STUDIES OF STILLING POOL FOR IMPERIAL DAM SPILLWAY.

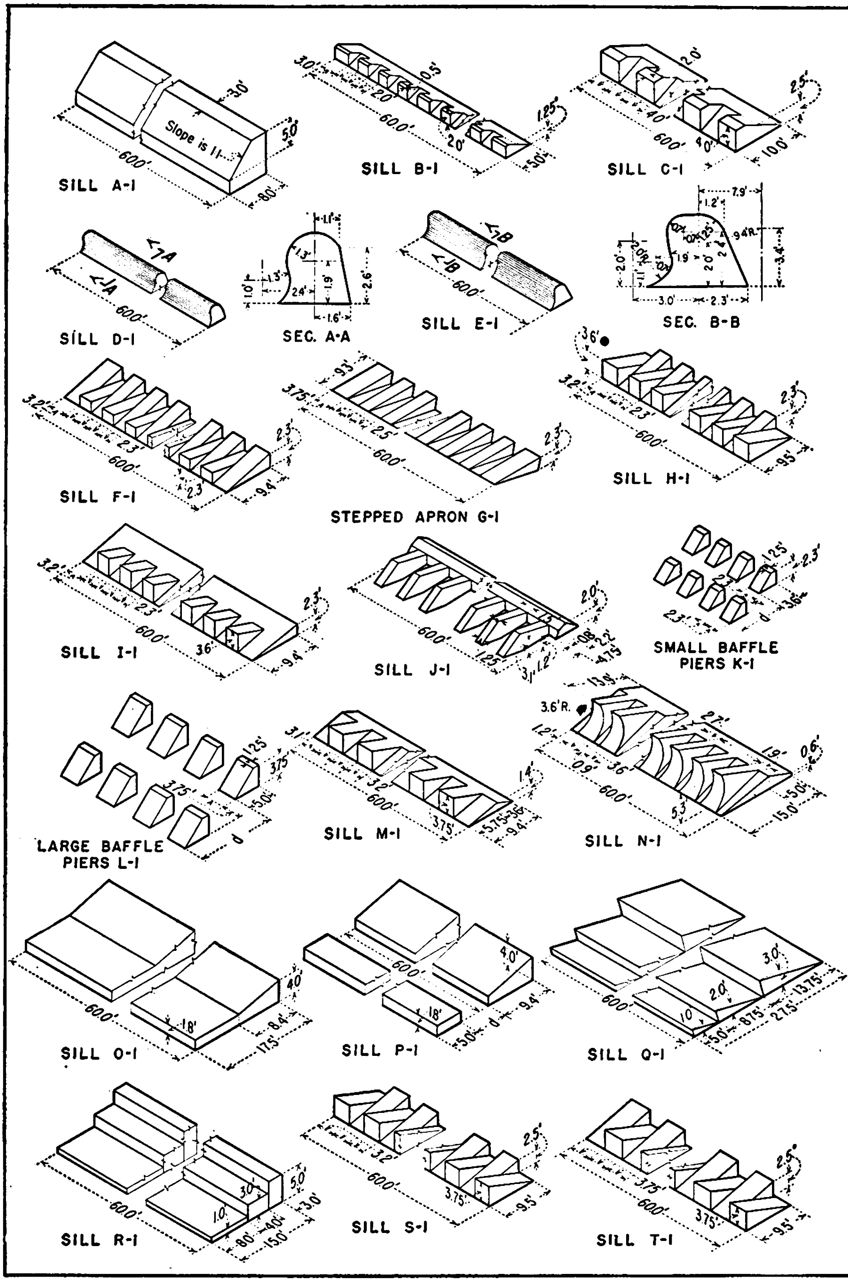
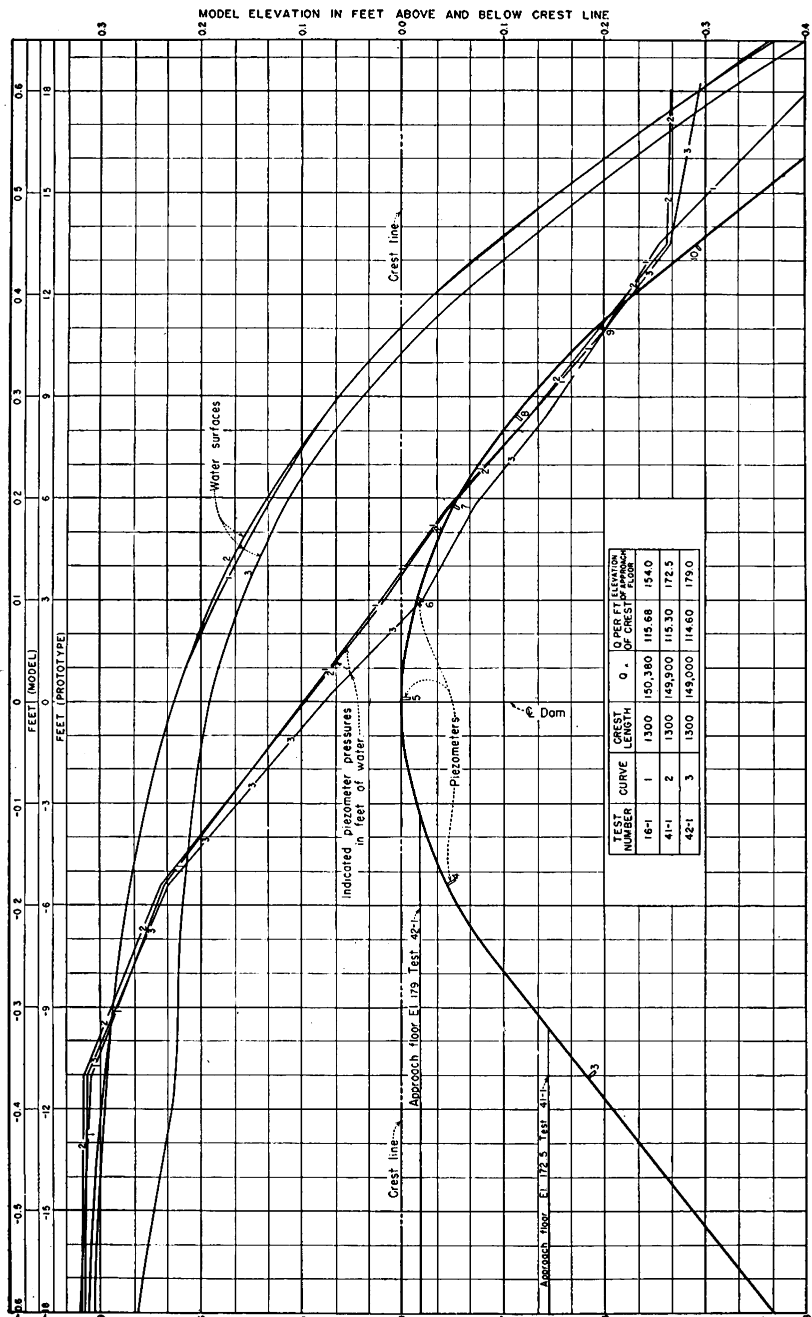


FIGURE 13.—ADDITIONAL SILLS AND STEPPED APRONS USED IN STUDIES OF STILLING POOL FOR IMPERIAL DAM SPILLWAY.



TEST NUMBER	CURVE	CREST LENGTH	Q	Q PER FT OF CREST	ELEVATION OF APPROACH FLOOR
16-1	1	1300	150,380	115.68	154.0
41-1	2	1300	149,900	115.30	172.5
42-1	3	1300	149,000	114.60	179.0

FIGURE 14.—WATER SURFACE AND PRESSURE PROFILES—PRELIMINARY DESIGN IMPERIAL DAM SPILLWAY.

surface. Should this separation be intermittent due to a variable supply of air to the space beneath the nappe, it might affect the stability of the spillway, particularly of a hollow-type section founded on sand. The lack of assurance of the absence of such pulsations was therefore an additional reason for the decision to eliminate all negative pressures.

MODIFICATIONS IN FINAL SPILLWAY DESIGN AS RESULT OF PRESSURE TESTS

24. **Modifications of Preliminary Design.**—To eliminate the negative pressures on the face of the spillway, the original design of figure 4 was changed as shown for the final design in figure 3–B. The crest was raised 1.0 foot to elevation 181.0; the length of the spillway was increased from 1,300 to 1,335 feet; the slope of the upstream face was decreased from 1¼:1 to 1½:1; and the shape of the downstream face was revised to fit the lower nappe of the flow for the maximum discharge. The aprons upstream and downstream remained at their previous elevations of 154.0 and 150.0 feet, respectively.

25. **Pressures on Face of Spillway—Final Design.**—After the model had been revised, pressure measurments were made to determine the results of the modifications. The pressure curves on figure 15 reveal that the revisions were a definite improvement. Piezometer 10 at the maximum discharge indicated a slight negative pressure, but this was of such magnitude as to be negligible. When tests were made to determine the effect of the velocity of approach, it was found that the pressures remained positive, indicating a well-proportioned shape for the downstream face.

26. **Effect of Depth of Approach on Discharge.**—The head-discharge relation for the revised spillway is shown in figure 16. For a crest length of 1,335 feet the model calibration indicated that a head of 9.70 feet would be required on the crest for the maximum discharge of 150,000 second-feet, the approach floor being at elevation 154.0. This was an improvement over the original design, for which a head of 10.0 feet was required to pass the maximum discharge. The coefficient-of-discharge curve, also shown in figure 16, indicates a coefficient for the maximum discharge of 3.73, as compared with the corresponding value for the original design of 3.61.

Since the reservoir will eventually fill with silt, the effect of this on the discharge capacity of the spillway was determined by calibrating the model for various depths of approach. The head required to pass the maximum discharge increased from 9.70 to 9.81 feet as the approach floor was raised from elevation 154.0 to elevation 175.0. Similarly, the discharge coefficient was found to decrease from 3.73 to 3.62.

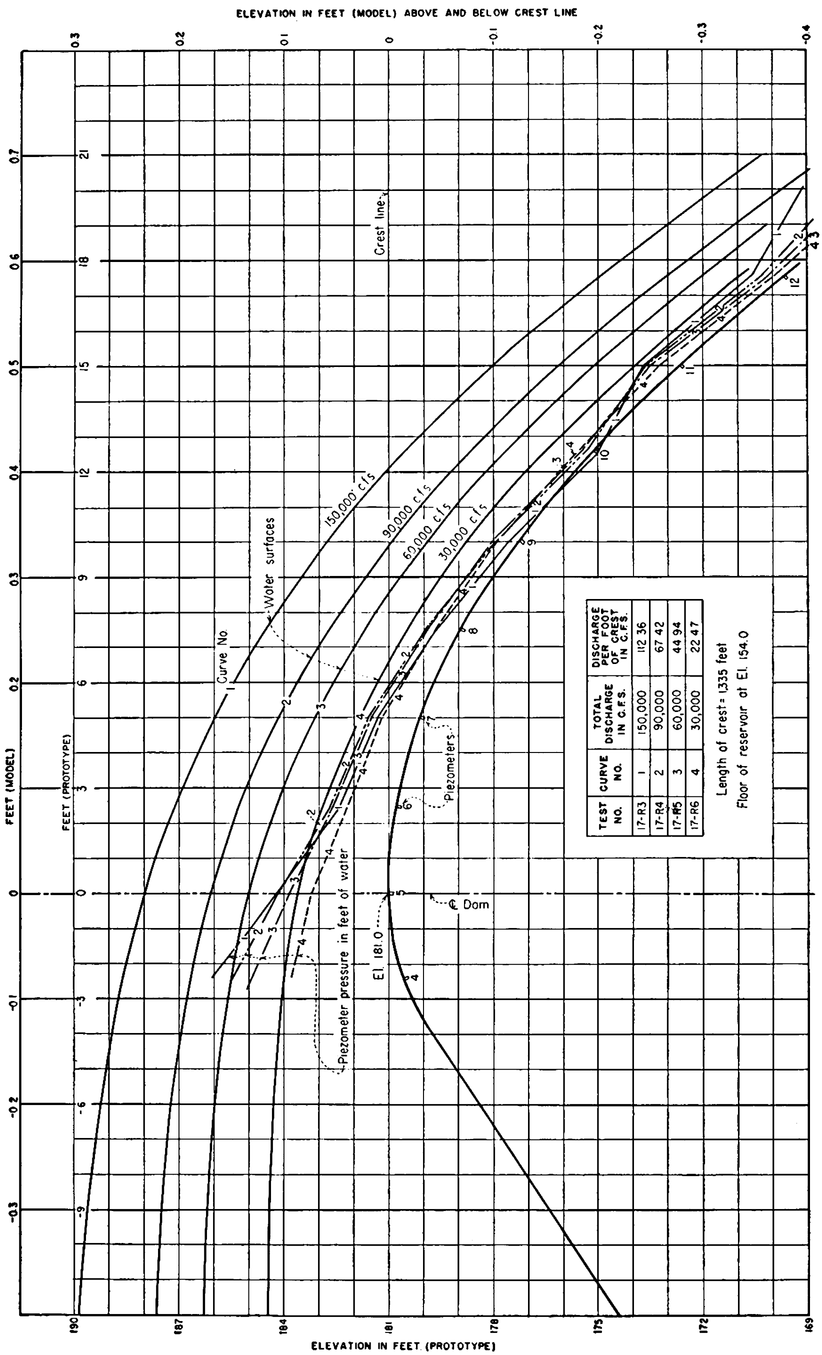


FIGURE 15.—WATER SURFACE AND PRESSURE PROFILES—FINAL DESIGN IMPERIAL DAM SPILLWAY.

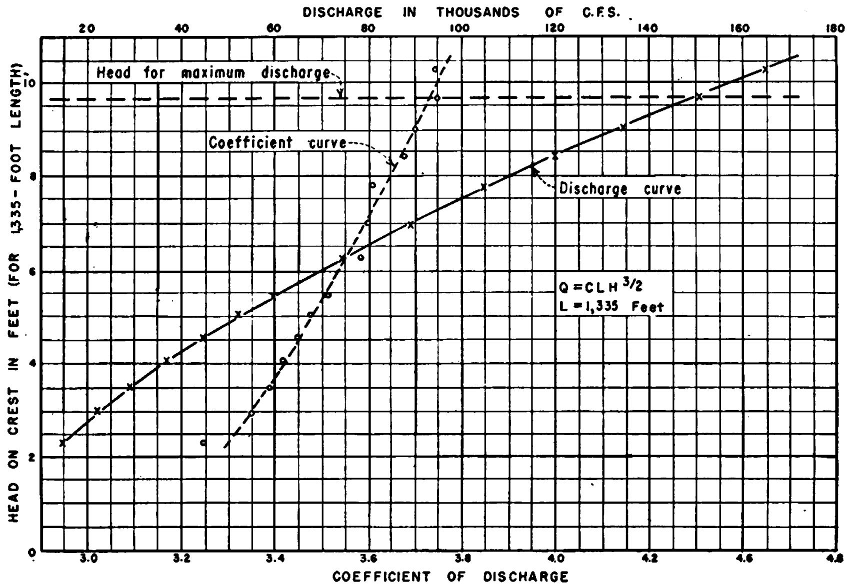


FIGURE 16.—COEFFICIENT AND DISCHARGE CURVES—IMPERIAL DAM SPILLWAY.

CHAPTER III—SLUICEWAY STUDIES

SLUICEWAY TESTS

27. **Model Tests on Imperial Dam Sluiceways.**—As previously mentioned, the sluiceway at Imperial Dam is located between the intake of the All-American Canal and the spillway section, as shown in figure 2. Its principal function is to control the level of the water in the lake above Imperial Dam so as to divert the proper amount down the All-American and Gila Canals. When the flow coming down the Colorado River exceeds the irrigation needs, the excess is discharged past the dam through the sluice gates. In times of large flood, when the excess is greater than the capacity of the sluiceway, the spillway goes into operation. The sluiceway also serves to wash out sediment deposits from the lake above Imperial Dam in the vicinity of the All-American Canal headgates.

To design correctly the sluiceway structure so that the high velocity of the water flowing through it would not erode the stream bed downstream from the control works, extensive model tests were conducted in the hydraulic laboratories at the old customhouse and at Montrose, Colo. The tests were started on a preliminary design of the structure and continued through a large number of forms of sluiceway for a similar discharge. Due to changes in the general plan of the dam, the capacity of the sluiceway was twice increased, and the alterations necessitated by these changes in discharge capacity were also tested.

28. **The Original Design.**—The original design of the sluiceway had a cross section as shown in figure 17. There were six gates 16 feet wide and 7 feet high, separated by 3-foot piers. The maximum discharge was 12,000 second-feet. The bottom of the gates was at elevation 162.0, the upstream apron at 154.0, and the downstream apron at 147.0. An apron on a 6:1 slope was provided between the rounded crest section and the downstream apron to keep the hydraulic jump in the pool for a wide range of tailwater depths. A sill 8 feet high was located at the downstream edge of the apron to aid in the formation of the jump.

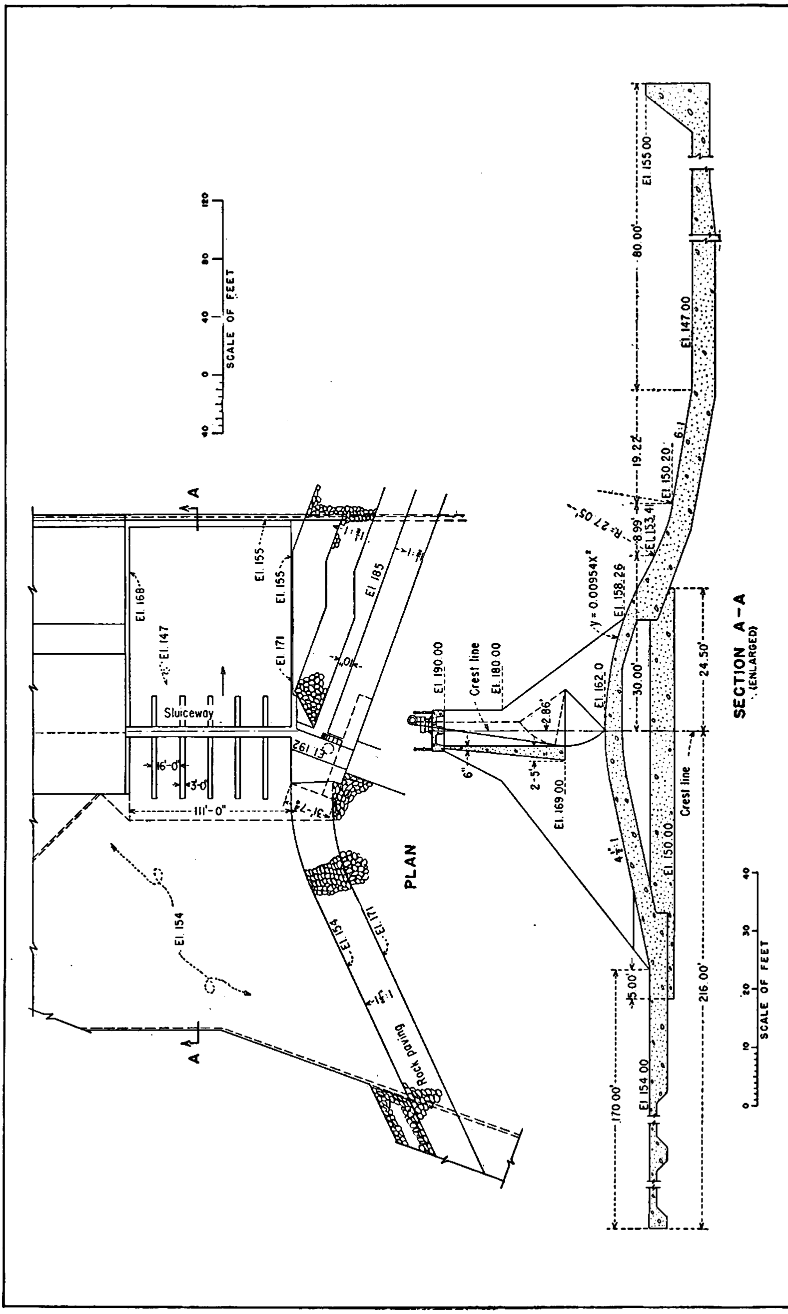


FIGURE 17.—ORIGINAL DESIGN OF IMPERIAL DAM SLUICEWAY.

29. The Original Design Model.—A 1:40 scale model of the original design of the spillway, shown in figure 17, was constructed and tested in the old customhouse laboratory. The model was placed in a wooden flume lined with sheet metal, the upstream apron of the sluiceway being represented by the floor of the flume. The crest or overflow section was made of 20-gage sheet metal soldered over five metal ribs mounted on steel angles fastened to the flume. Redwood was used for the piers, and 18-gage sheet metal for the radial gates. The downstream apron forming the stilling pool was made of wood to facilitate changes as testing progressed. Sand was placed in the flume immediately downstream from the model to permit a study of the effect of scour produced by various pool designs. The tailwater elevation, regulated by a tailgate, was observed by a piezometer connected to the flume; the head upstream from the sluice was observed by a hook gage in the reservoir. Piezometers were installed in the crest section for observing pressures on its face, and water surface profiles through the model were taken by a point gage mounted across the flume.

STILLING-POOL TESTS

30. Tests of the Original Design Pool.—The stilling-pool tests on the model of the original design were not recorded in the form of measurements, since the action of the model was so unsatisfactory that a change was obviously necessary. The flow in the pool was very rough, and a large boil formed over the downstream sill for the maximum discharge and tailwater depth. With the sand representing the stream bed at elevation 155.0, the velocity of the water flowing over it was excessive, and considerable scour resulted.

The tailwater level in this and succeeding tests, unless otherwise noted, was made to conform to a rating curve which was based on the best possible estimate of future conditions. Because of the possibility that the tailwater level might at some time be lowered by the degrading of the channel downstream, or raised by the deposit of sediment from the desilting works, it was important that the sluice operate satisfactorily under a wide range of tailwater conditions. One of the purposes of the model tests was to secure a design which would have this desirable feature.

31. Modifications of the Original Design Pool.—In an effort to improve the action of the stilling pool of the original design, sills of a number of different designs were tried at the end of the downstream apron in place of the continuous sill shown in figure 17. The designs used are shown in figures 18 and 19. It was found that stepped aprons of types A–3 to G–3, figure 18, if installed on the 6:1 sloping apron, generally improved the hydraulic action. In the following tests a wide variety of combinations of these aprons and sills were used. In all, 174 different conditions were tested in the design of these sluices.

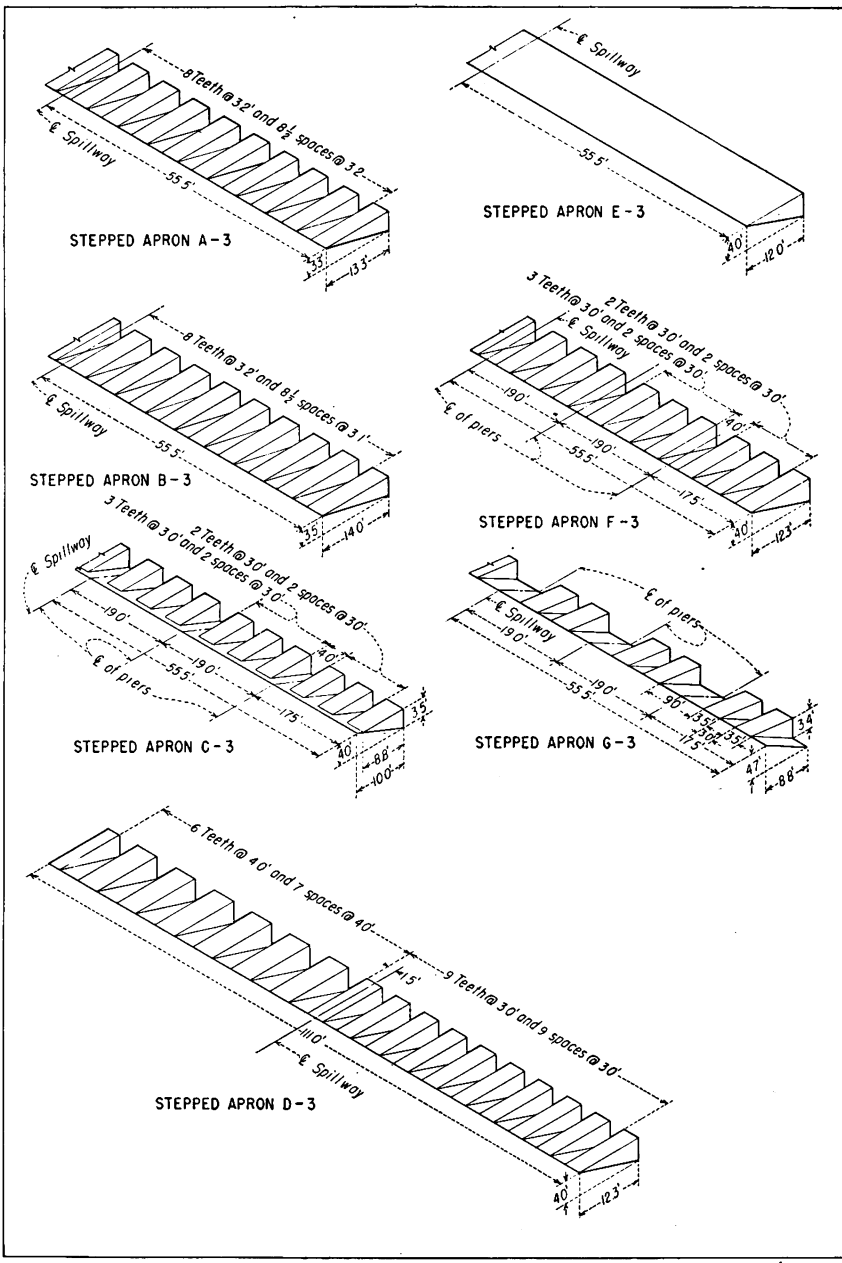


FIGURE 18.—SOME STEPPED APRONS AND SILLS STUDIED FOR THE DESIGN OF A STILLING POOL FOR IMPERIAL DAM SLUICEWAY.

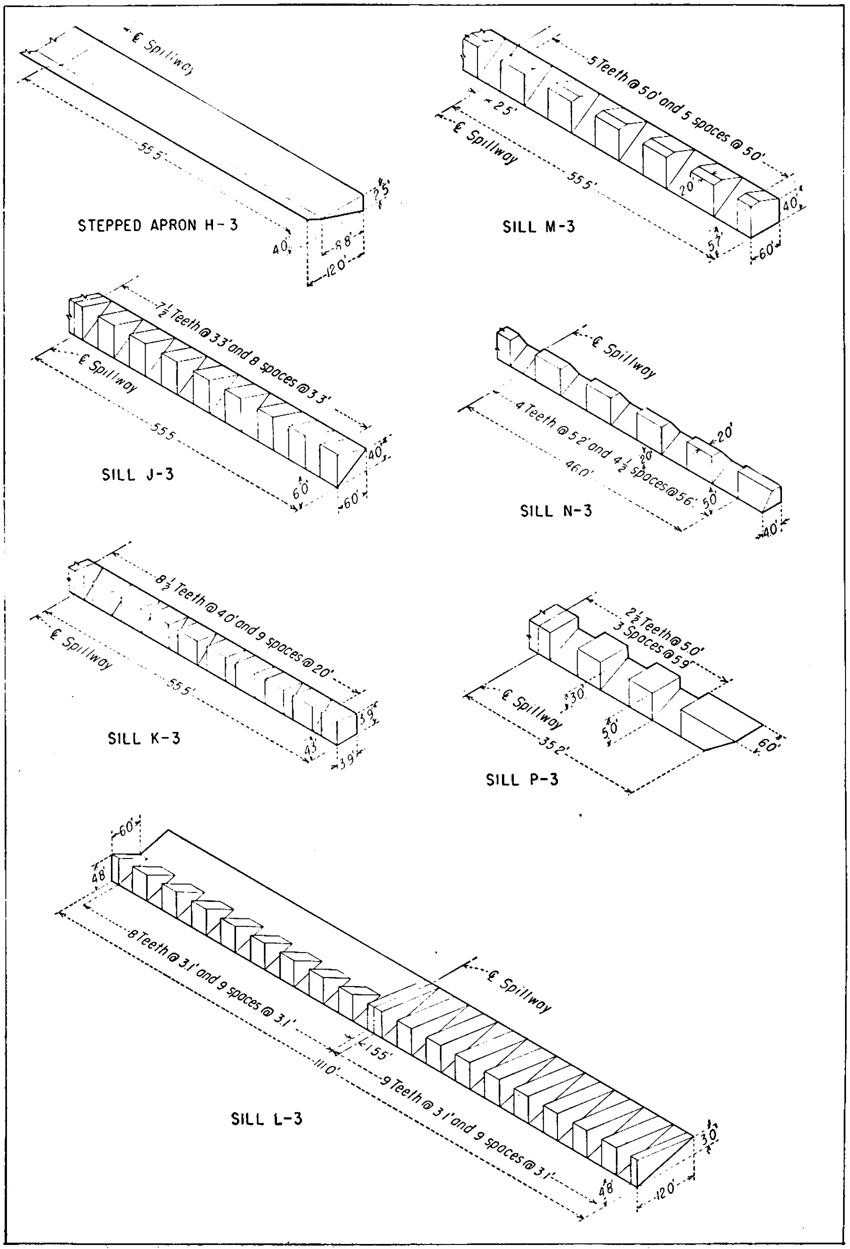


FIGURE 19.—ADDITIONAL STEPPED APRONS AND SILLS STUDIED FOR THE DESIGN OF A STILLING POOL FOR IMPERIAL DAM SLUICEWAY.

Among the first studies were tests 4–1 and 4–2, which were made with stepped apron F–3 placed at the end of the sloping apron, and sill L–3 installed on the end of a pool floor 45.5 feet long, as shown in figure 20. A smooth paving, 58 feet in length, was placed at elevation 150.0 immediately downstream from the sill, and below this the river was paved with riprap for a distance of 70 feet. The riprap started at elevation 150.0 and sloped downward to elevation 140.0. For the purpose of comparison, the teeth were different on the two halves of the sill L–3, as shown in figure 19. The larger teeth gave slightly better results than the smaller ones. The water surface profiles produced in this model are shown in figure 20. Flow conditions were fairly satisfactory for the maximum tailwater elevation of 158.1, as in test 4–1; but when the tailwater was dropped to elevation 155.4, as in test 4–2, the jump threatened to leave the pool. In addition, a large boil formed over the sill, and a series of stationary waves were created by the high-velocity water downstream from the pool. The front of the jump formed near the center of the stilling pool for both tailwater depths. This lay-out was unsatisfactory for the purpose desired, as the workable tailwater range was much too small and the velocities downstream from the pool were excessive.

Attempts were made to improve the action by raising the floor downstream from the sill 1 foot, and by lowering both the pool floor and the floor downstream from the pool 1 foot; but these were unsuccessful.

32. The Double-Jump Pool.—Since the attempts to improve sufficiently the action of the original design model were unsuccessful, a new form called the double-jump pool was devised. In this pool the energy of the high-velocity discharge was dissipated by passing the water through two hydraulic-jump pools in series, as shown in figure 20, test 16. The upstream pool was placed at a higher level than the downstream pool and was provided with a stepped apron and a dentated sill. The downstream pool had a sloping floor section, followed by a level floor with continuous sill at the end. Several forms of this type of pool were tested, the best one being that shown in figure 20, tests 16–1 and 16–2. This form of pool was successful in dissipating the energy of the water with the wide range of tailwater levels which might occur at this location.

33. The Long Sloping-Apron Type.—The type of pool which was incorporated into the final design had a long, sloping apron and used a single jump, as shown in figure 20, test 29–2. In this design there were two rows of dentated steps upstream from the pool floor and a dentated sill on the apron at the end of the pool. The pressures which the water exerted on the floor of the structure were observed and are shown in this figure. The action of the spillway was further perfected by streamlining the downstream ends of the piers. For the maximum discharge of 2,000 second-feet per gate, the jump remained in the stilling pool for all levels down to elevation 150. Limitations of the apparatus prevented tests at lower levels,

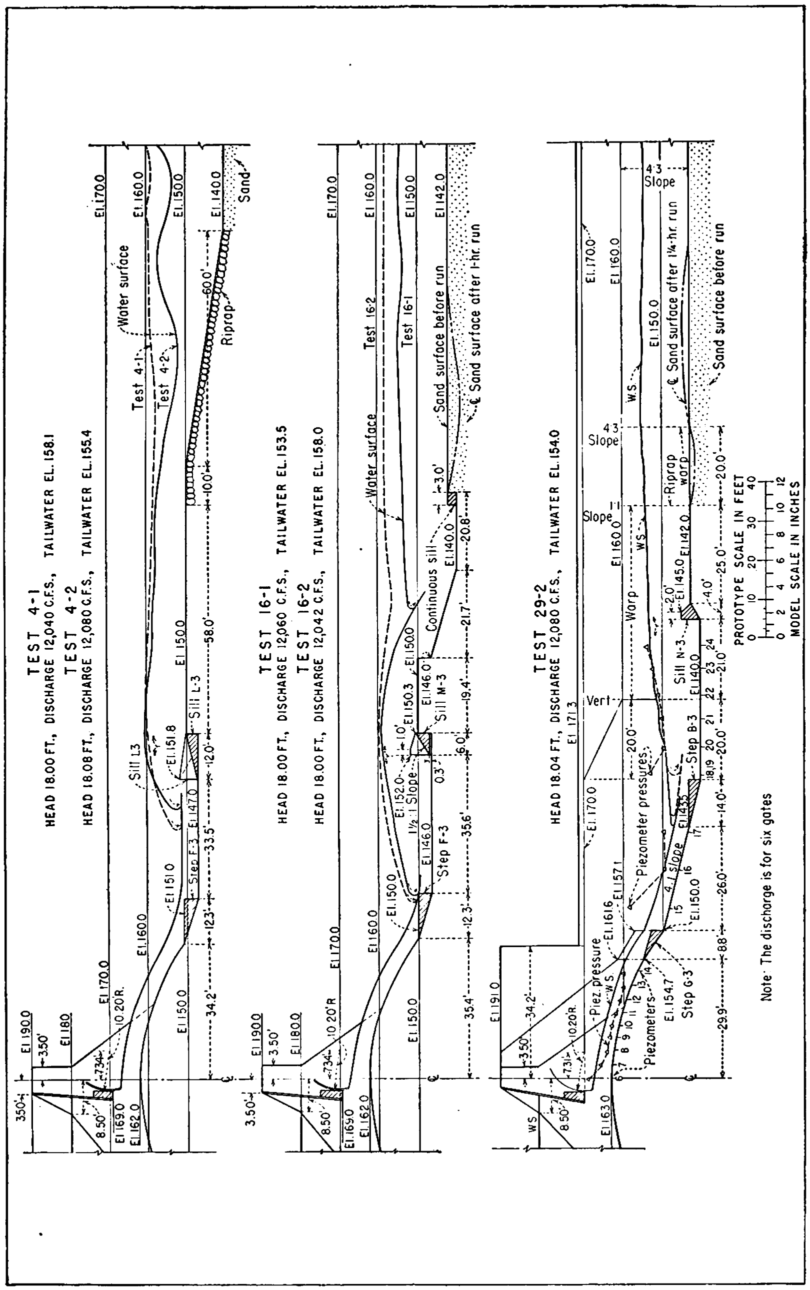


FIGURE 20.—TYPICAL STILLING-POOL TESTS ON SLUICEWAY OF IMPERIAL DAM.

but the indications were that even somewhat lower tailwater levels could have been used.

34. **The Final Design.**—During the course of the studies several changes were made in the design of the dam as a whole which involved changes in the sluiceway design. The principal one was the substitution of 12 gates for the 6 specified in the original design. In the final design the sluiceway gates are arranged in three sets of four each, the sets being separated by division walls. The walls extend through the stilling pool to permit any one set to be operated independently of the others and to facilitate repairs. A cross section of the final design is shown in figure 21, and the plan and upstream elevation are shown in figure 22.

A model of this structure was constructed at the Montrose laboratory to a scale of 1:40 and tested under a wide variety of conditions, some of which are shown in views B, C, and D of figure 23. The action of the structure was very satisfactory over a wide range of tailwater conditons. The pressures on the floor for a wide range of heads and gate openings were observed, and some of the results are shown in figure 24. No negative pressures were observed. The discharges of the gates for various openings were determined and plotted as shown in figure 25.

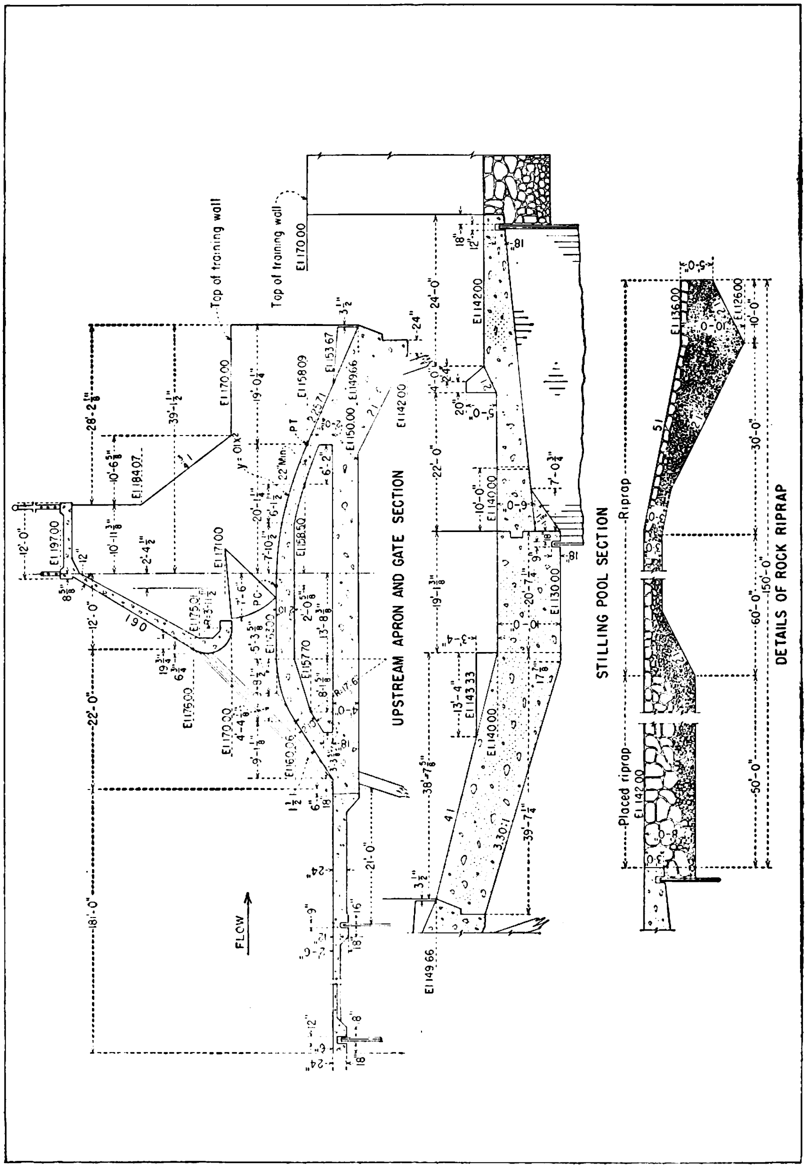


FIGURE 21.—SECTIONS OF IMPERIAL DAM SLUICEWAY—FINAL DESIGN.

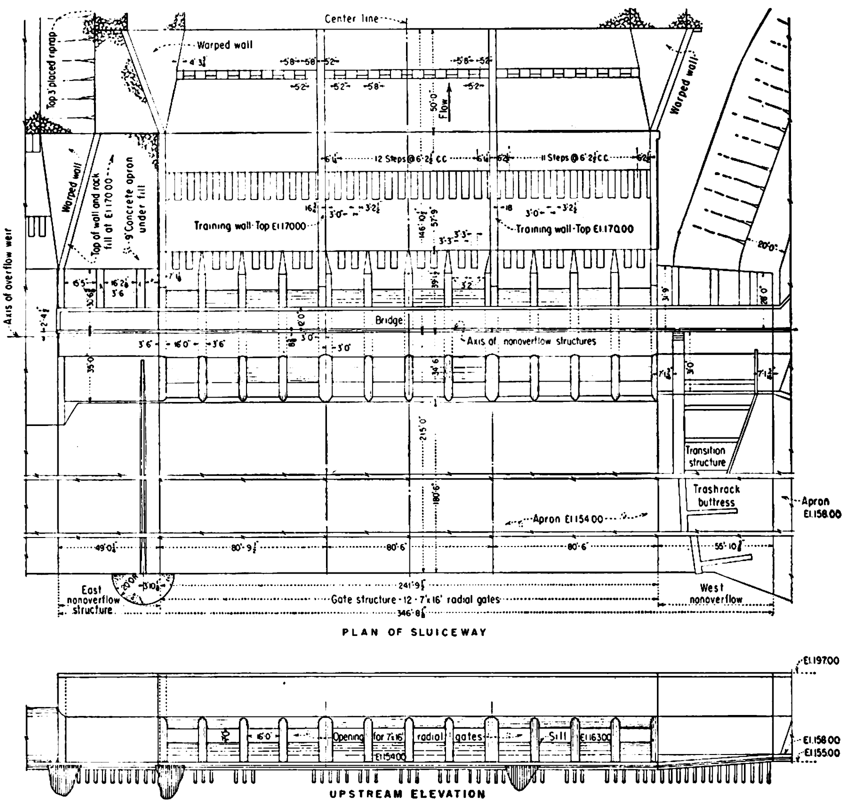
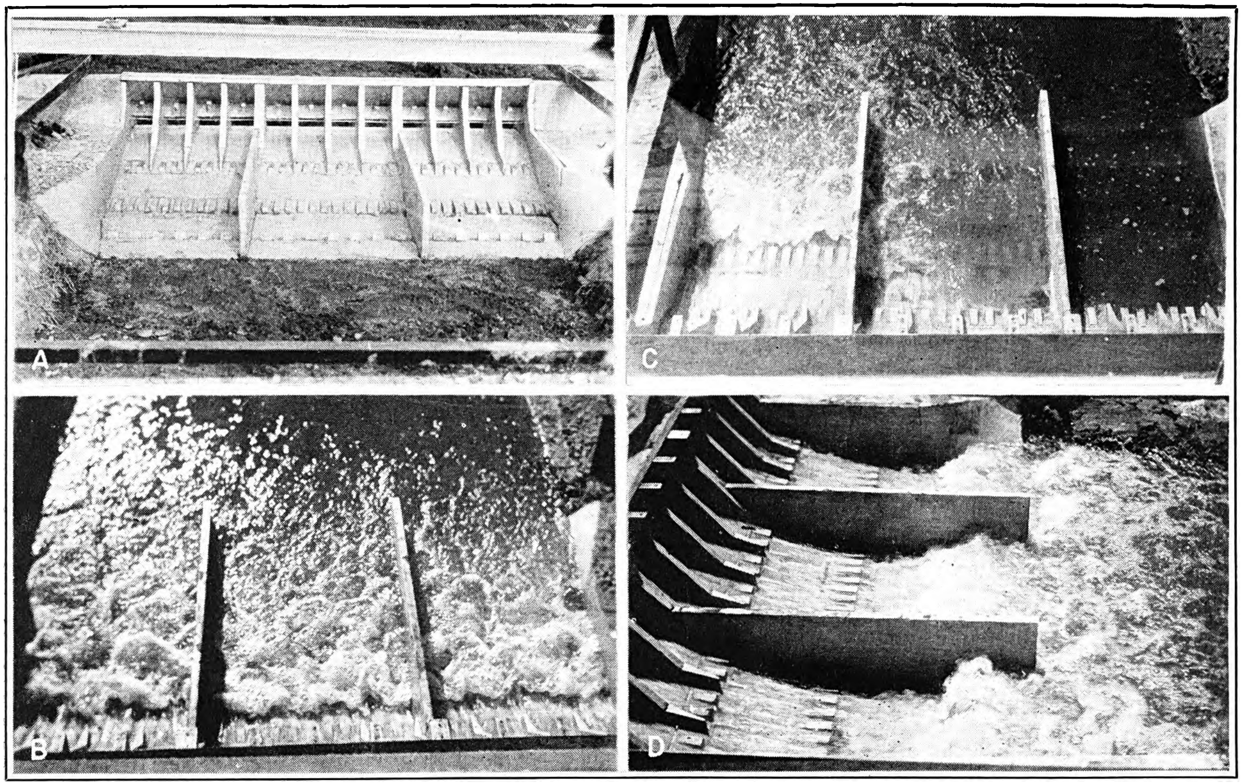


FIGURE 22.—FINAL DESIGN—PLAN AND UPSTREAM ELEVATION OF SLUICEWAY FOR IMPERIAL DAM.



A. Sluiceway Model Looking Upstream.
B. Looking Downstream on Pool—Discharge 24,000 Second-Feet, Tailwater Elevation 158.0.
C. Looking Downstream on Pool—Six Gates on Left Open, Discharge 12,000 Second-Feet, Tailwater Elevation 154.0.
D. Discharge 24,000 Second-Feet, Tailwater Elevation 150, Jump on Verge of Leaving Pool.

FIGURE 23.—MODEL TESTS ON FINAL DESIGN SLUICEWAY FOR IMPERIAL DAM.

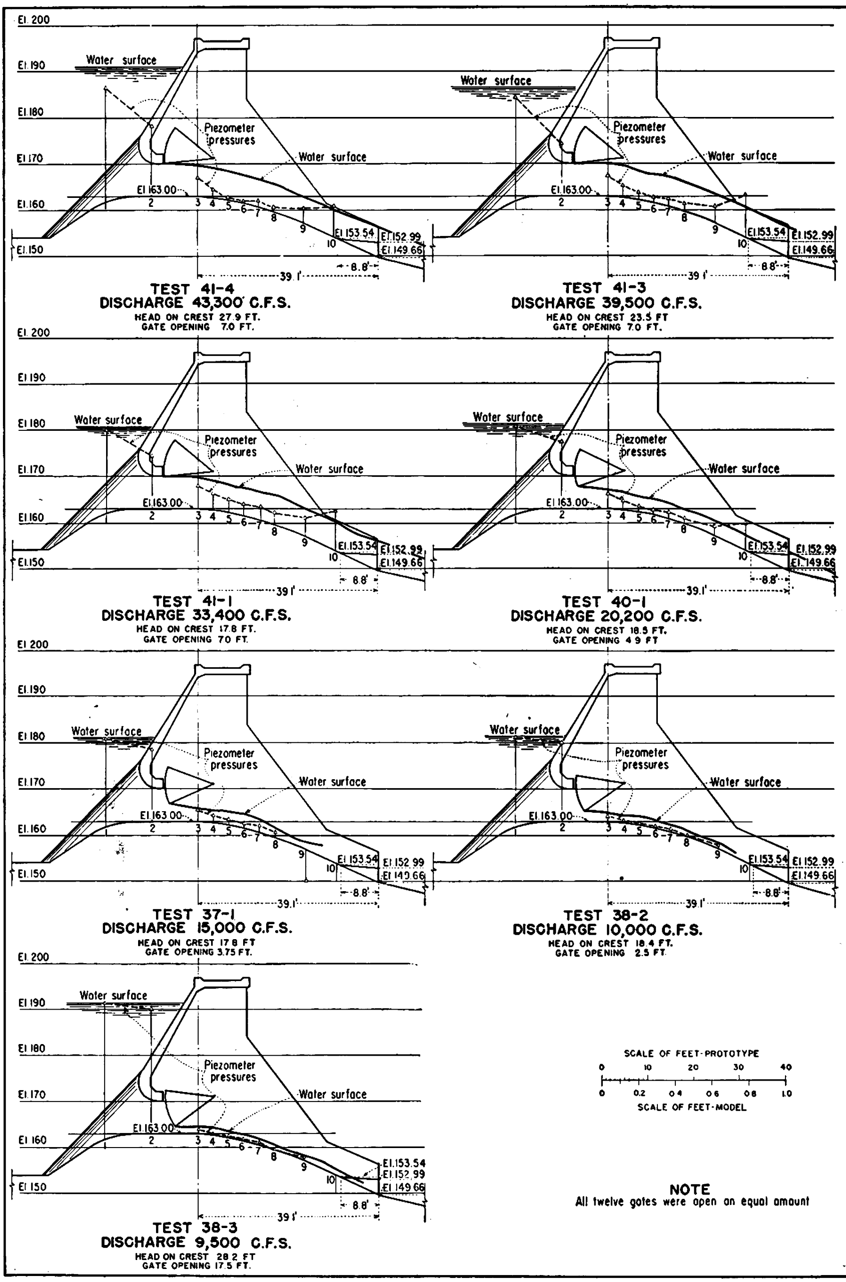


FIGURE 24.—WATER SURFACE AND PRESSURE PROFILES ON FACE OF IMPERIAL DAM SLUICEWAY.

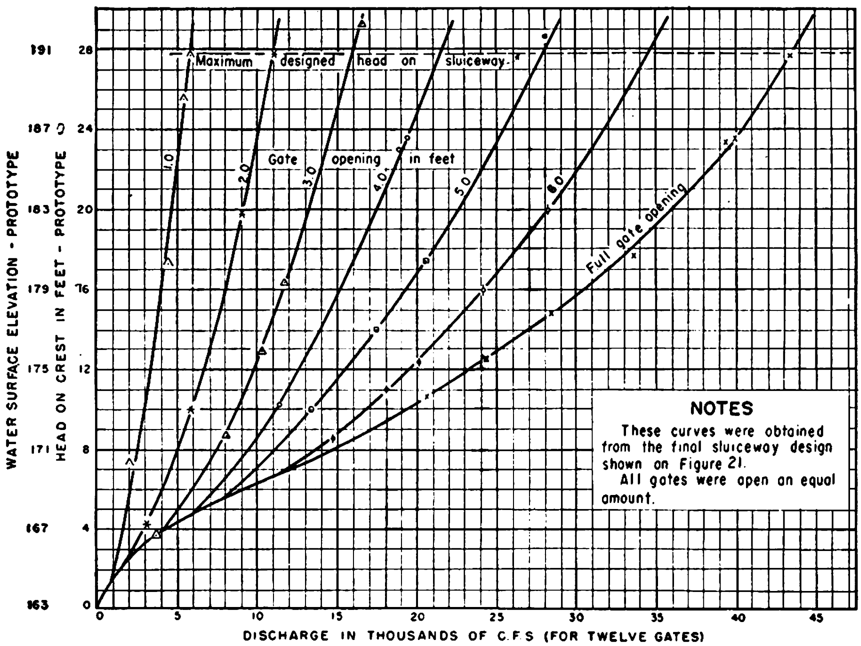


FIGURE 25.—DISCHARGE CURVES FOR FINAL DESIGN OF IMPERIAL DAM SLUICEWAY.

CHAPTER IV—ALL-AMERICAN CANAL HEADGATE STUDIES

35. The Design of the All-American Canal Headworks.—The general design of the headworks for the All-American Canal is shown in figure 2. It consists of a long, curved trash rack, downstream from which are located the gates which control the flow of water into the canal. These gates are of the roller type, frequently used in movable dams. Each of the four gates is 75 feet long and is designed to sustain a hydrostatic head of 20 feet. The channel leading from these control gates to the desilting works is divided into four smaller channels by concrete sheet-piling division walls, each smaller channel being supplied by a single gate. This subdivision was made to provide nonsilting velocities in the channels when the desilting works are being operated at part capacity.

There is a direct connection from each of the four channels to the All-American Canal, bypassing the desilting works. This was provided to enable silt to be passed into the canal during the initial flow to seal any leaks in the porous soil through which much of the canal passes.

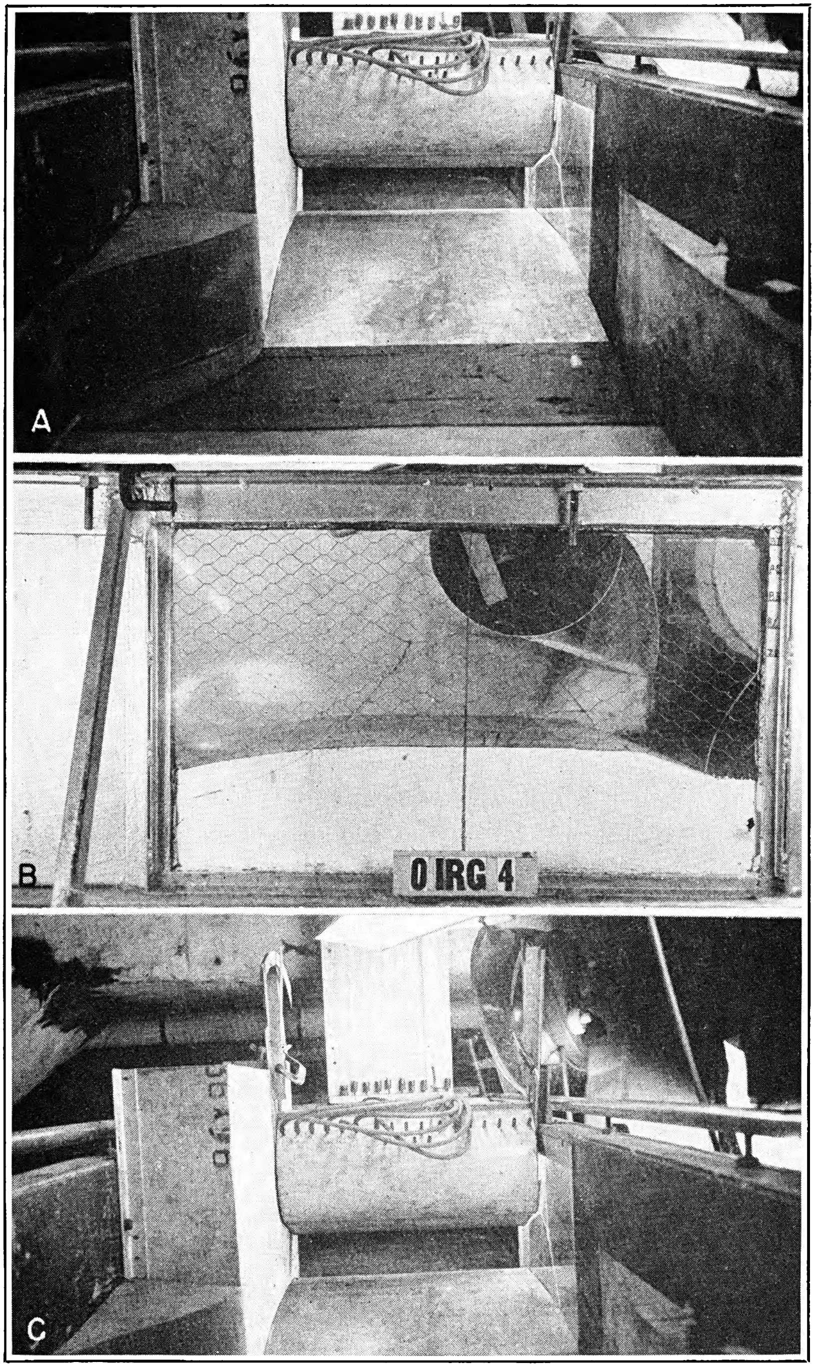
One of the problems in the design of this structure was the dissipation of the energy of the high-velocity water which flowed under the headgates at partial openings, in such a way that no damage would result to the structure or the canal banks. If a hydraulic jump was to be used, it was required that it occur far enough away from the roller gate to prevent the turbulence of the jump from setting up vibrations in the gate. To solve this problem, model tests were conducted at the hydraulic laboratories in the old customhouse and at Montrose, as described in the following paragraphs. Sediment problems encountered in the design of the headworks were investigated at the Montrose laboratory, as described subsequently.

36. The 1:20 Headgate Model.—The first model for the design of the All-American Canal headgate was constructed in the old customhouse laboratory on a scale of 1:20. It was built in a glass-walled flume 2 feet wide, which was too narrow to accommodate a full gate; accordingly, one-half of one pier and 40 percent of the length of the roller were used. The nose and tail of the pier were made of redwood, while the remainder of the model was constructed of sheet metal. In the original design, the floor of the All-American Canal intake was level at elevation 171.0 and continued at this elevation for 100 feet downstream from the gates, then sloped slightly downward. Figure 26 gives several views of a revised design of this model. In view B, the direction of flow is from right to left.

37. Results of Tests on Original Design.—The tests of the original design of the headgate showed several undesirable conditions. At low discharges with the higher heads, the jump swept far downstream and severely eroded the channel bottom. Under other conditions the jump formed immediately downstream from the gate, and at times there was a wave action striking the lower face of the gate. This latter condition was undesirable, as the waves striking the gate tended to set up vibration or chattering, especially under conditions when the gate was partially submerged and the buoyant force reduced its effective weight, thus permitting it to move more easily. Large eddies formed near the downstream end of the pier, and the shape of the upstream end of the pier caused considerable swirl and draw-down at the end of the roller gate.

To eliminate the possibility of vibration or chattering, the following alternative courses were considered: (1) Through control of the boundary layer, to compel the water to cling to the underside of the gate and flow away without pounding, and (2) to design the gate so that the water would break free at the lip of the gate and form a hydraulic jump far enough downstream to prevent any interference with the gate.

38. Experiments With Boundary-Layer Control.—The theory of the boundary-layer control as applied to a contraction of a channel is as follows: When water flows through a contraction, such as a Venturi meter, the water accelerates and the pressure drops until the most contracted section is reached, and at this point the pressure begins to rise and the velocity begins to decrease. Along the surface of the meter, however, is a thin layer called the boundary layer, which moves much slower than the main body of the water. When the main body of the water reaches the most contracted section it begins to flow toward a region of higher pressure. It can do this because its high velocity gives it sufficient momentum. The boundary layer, however, does not have this high velocity and great momentum, and therefore can move only a short distance against the increasing pressure before it comes to rest or stagnates. The point where it stops moving is called the stagnation point. Since at the stagnation point the water in the boundary layer does not move forward, but water is continually brought down to that point from upstream, an accumulation of this practically quiet water tends to form and divert the faster moving water outside the boundary layer away from the side walls. Since the pressure along the walls downstream from the stagnation point decreases in an upstream direction, when the boundary layer diverts the fast-moving water away



A. View Looking Upstream. B. Side View Showing Gate Sill. C. View Showing Manometer Board and Connections.

FIGURE 26.—MODEL OF REVISED DESIGN HEADGATE FOR ALL-AMERICAN CANAL.

from the walls a flow tends to form along the wall downstream from the stagnation point in the direction of this decreasing pressure, that is, in an upstream direction with reference to the main flow. When this occurs the boundary layer breaks free from the surface and an upstream eddy occurs, and the main current moves away from the channel walls. This upstream eddy can be prevented by drawing off through the side walls the boundary layer upstream from the stagnation point, or by increasing the velocity of the boundary layer by mixing it with high-velocity water so it can overcome the adverse pressure gradient.

To cause the boundary layer in the model to move through the gate contraction and hence cause the fast-moving water to cling to the underside of the gate, it was necessary that the shape of the gate be changed from that used in the original design. Consequently, the gate was altered by the addition of a specially formed lip to the apron of the gate, as shown in figure 27. A triangular lip, A, was attached to the apron, to which various curved sections were added to form the other lips shown. Tests with lips B, C, and D proved unsatisfactory since the water would cling to the face only for headwater levels of less than 3 feet (prototype scale) above the tailwater level. On lip E a wire was stretched across the face, as shown in figure 27, to cause the mixing of the high-velocity particles with those of the boundary layer, and thus enable the boundary layer to overcome more easily the adverse pressure gradient. This was an improvement, and caused the water to cling to the face of the lip for a headwater 3 feet higher (prototype scale) than without the wire. Another attempt to cause a mixing of the boundary layer with the high-velocity jet was made by providing an offset in the lip (lip I).

In the next experiments, attempts were made to direct a jet of high-velocity water on the boundary layer near the stagnation point to assist it in carrying through the adverse pressure gradient. On lips of type E a small slot or conduit was formed by a sheet of metal separated from the lip by a wire spacer. The water entered this slot at the upstream end of the lip and was discharged at various distances downstream near where the stagnation point might be expected, see lips F, G, and H, figure 27. These lips with slots were successful in their action, lip G being the most effective; for lip G, separation took place at a head 9 feet greater than for lips without the slot. Although the hydraulic action in this case was satisfactory, anticipated difficulties of construction and operation caused the rejection of the design. The experiments, however, were an interesting demonstration of the possibilities of boundary-layer control, which for some hydraulic structures may lead to very beneficial solutions, as has already been the case in aeronautics.

39. Stilling-Pool Design.—Since the elimination of the possibility of vibration or chattering by the control of the boundary layer did not develop a practicable design, the second method, that of causing the jump to form

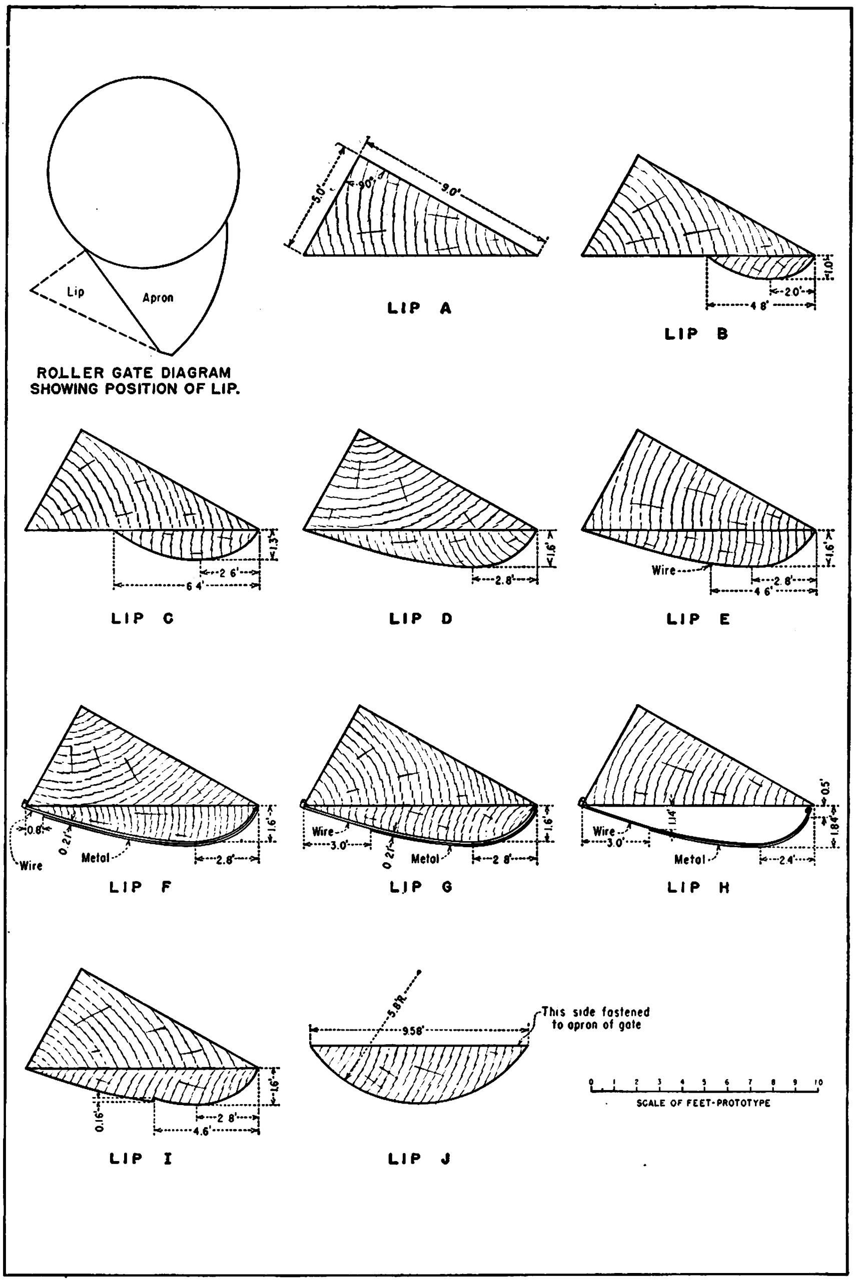


FIGURE 27.—APRON LIP ADDITIONS TO ALL-AMERICAN CANAL HEADGATE.

far enough downstream to be free of the gate, was investigated. The first trial of this type was with the Rehbock dentated sill, K, figure 28, tested at various positions on the level floor; however, no location could be found which would cause the jump to be stable and out of contact with the gate over a range of discharges.

After the experiments on the Rehbock sill, a study was made to determine the effect of shape of stilling basin, in which a large number of shapes were tested. The first shapes were simple, consisting of depressions in the otherwise level channel floor, each depression beginning just downstream from the gate. All pools of this type, see pools L to T, figure 28, proved unsatisfactory as the jump could not be stabilized away from the gate. Each of the next three forms of stilling basin investigated had a raised section of the floor immediately beneath the gate, followed by a sloping section leading into the stilling pool. These gave successively better results, and pool W was adopted as the final design. With this pool operating under the higher heads, the jump formed in the stilling pool without contact with the gate, as shown in figure 29. At the lower heads, the surface of the water passing over the sill was depressed as the velocity decreased; and the water surface rose downstream from this point without forming a hydraulic jump, but remained out of contact with the gate.

40. Revision in Shape of Gate Piers.—The blunt noses on the original piers created undesirable disturbances upstream from the gates. It was desired to streamline these piers, but it was impossible to decrease their width as this was established by the machinery to be contained within them. The alternative was to increase the length of each pier by lengthening its upstream nose. To obtain the most satisfactory flow around the pier it would have been necessary to lengthen the nose considerably. Since for architectural reasons this was not permissible, the resulting pier constituted a compromise.

41. Pressures on Roller Gate.—To obtain a more accurate knowledge of the existing pressures on the face of the roller gate, six piezometers were installed in the upstream face, as shown in figure 26, and one was placed in the floor of the channel 3 feet (model distance) upstream from the gate sill. From the readings, the pressures on the face of the gate were computed. The plotted pressures are shown in figure 30. These represent the measured pressures on the upstream face of the gate for three different pond elevations and a constant discharge of 4,000 second-feet per gate. Attention is called to the manner in which the measured pressure drops as the lip of the gate is approached. The indicated pressure at the tip is zero, as the jet breaks free from the lip at this point. The static pressure given is that due to the elevation of the free water surface above the point considered.

As a check on the hydraulic method of obtaining pressures, an electric analogy model was built on a scale of 1:24, with a salt solution for the conductor. Tests were made using three pond levels with the gate opening

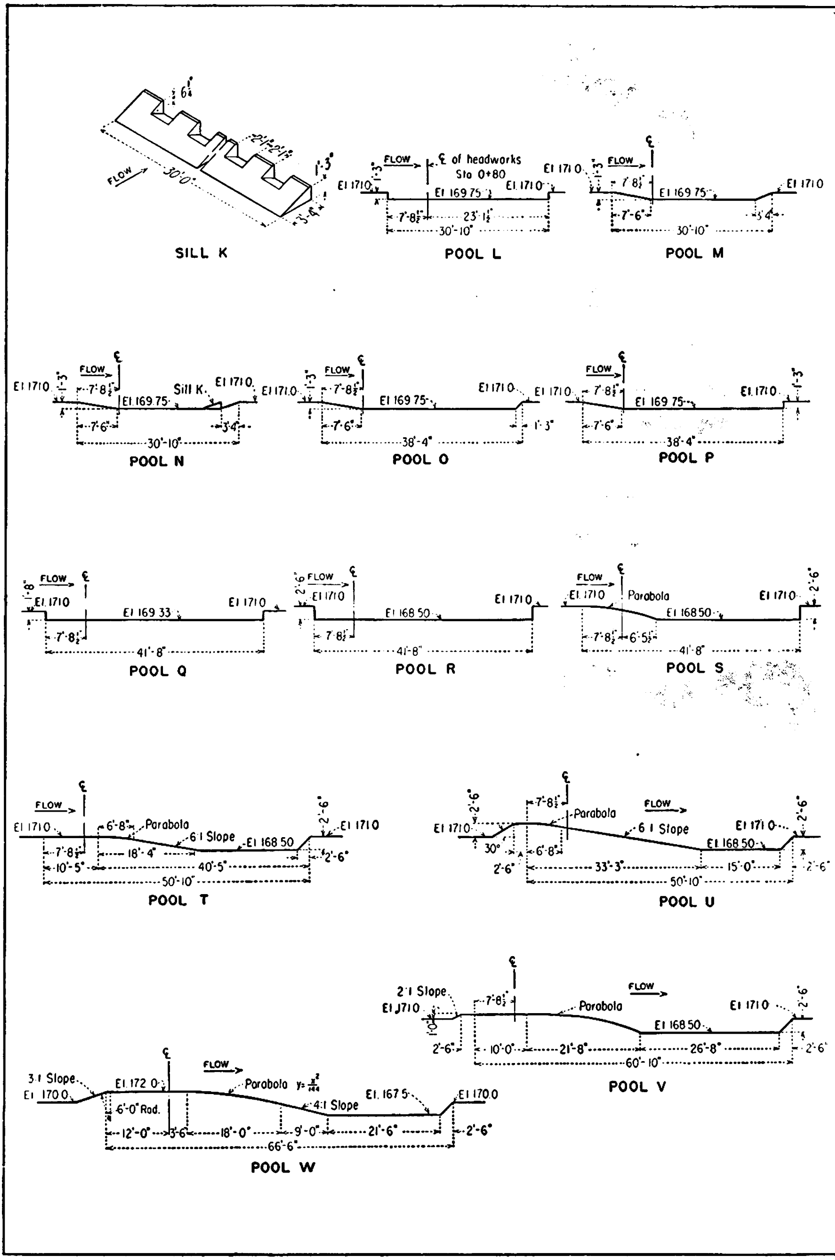
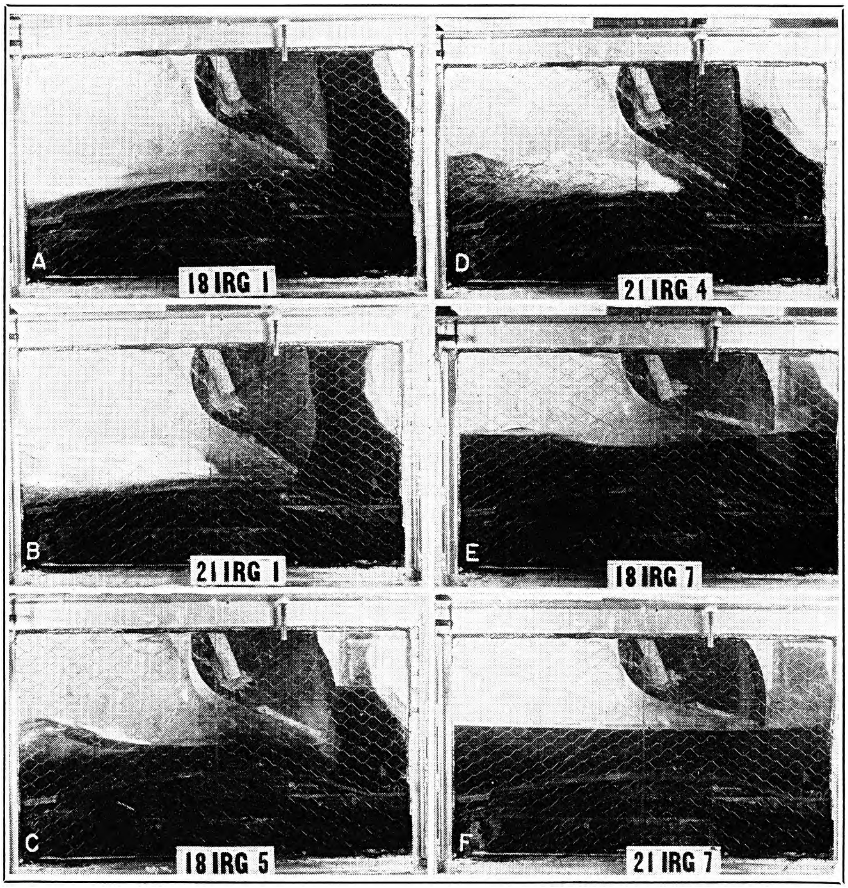


FIGURE 28.—HEADGATE STILLING-POOL SHAPES INVESTIGATED FOR ALL-AMERICAN CANAL.



A. Pond Elevation 191.0—Discharge of 5,000 Second-Feet per Gate.
B. Pond Elevation 191.0—Discharge of 2,500 Second-Feet per Gate.
C. Pond Elevation 181.0—Discharge of 5,000 Second-Feet per Gate.
D. Pond Elevation 183.0—Discharge of 2,500 Second-Feet per Gate.
E. Gate Out of Water—Discharge of 5,000 Second-Feet per Gate.
F. Gate Out of Water—Discharge of 2,500 Second-Feet per Gate.

FIGURE 29.—FINAL DESIGN STILLING POOL FOR ALL-AMERICAN CANAL HEAD-GATE STRUCTURE.

required for a discharge of 4,000 second-feet per gate, duplicating conditions in the hydraulic model. The electric analogy test consists essentially of the location of stream lines, which may be considered as the boundaries of stream tubes each carrying the same percentage of the total flow. Since the flow in these varying tubes is continuous, the velocity is inversely proportional to the cross section of the tube. Thus, when the average velocity is known at some section in a tube, it can be computed for any other section.

On the model, five stream lines were located forming six stream tubes,

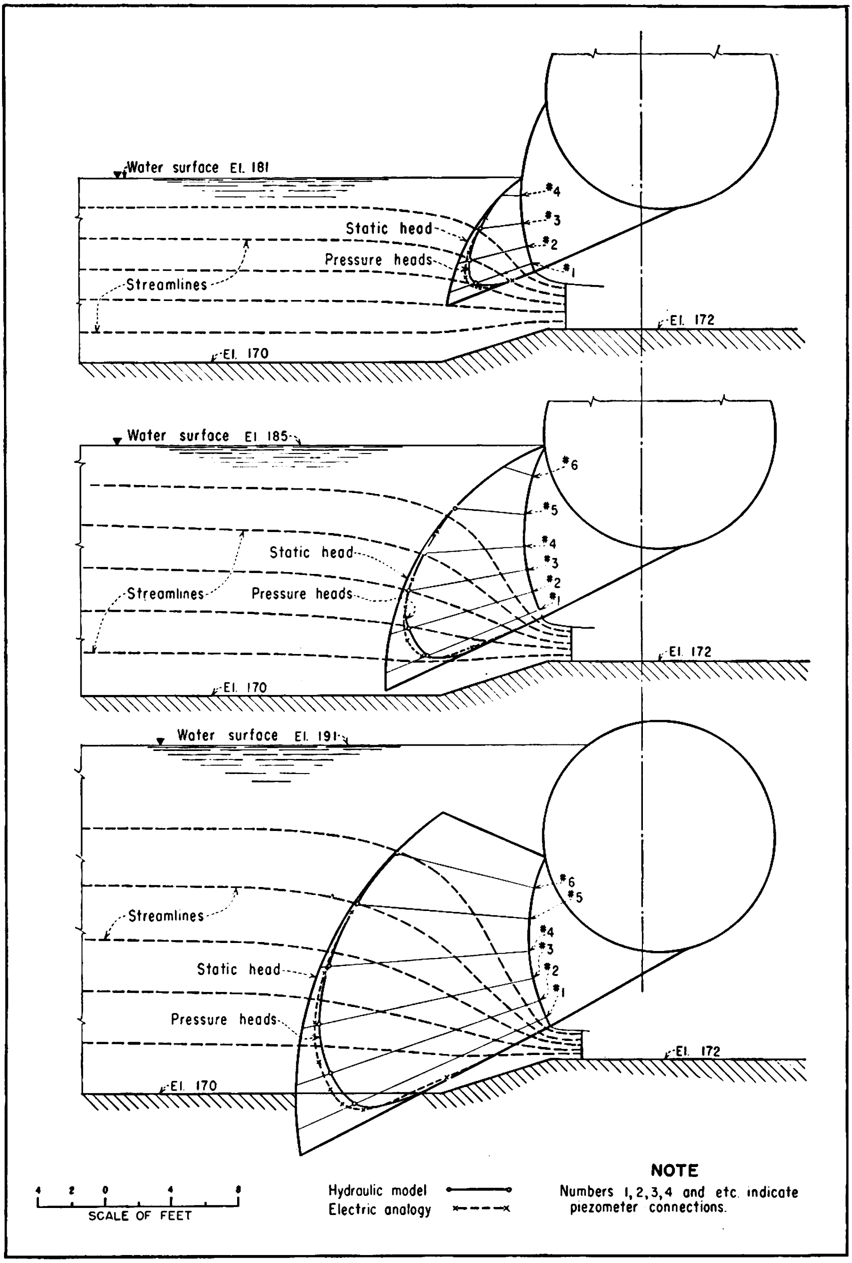


FIGURE 30.—PRESSURES ON ALL-AMERICAN CANAL HEADGATE BY HYDRAULIC AND ELECTRIC-ANALOGY TESTS.

as shown in figure 30. From a sketch of the water surface curve, the depth was determined at a section below the gate and the average velocity was computed from this depth and the discharge. Starting with this velocity at the downstream ends of the stream tubes, the distribution of velocity was determined from the varying widths of the stream tubes. The pressure heads were obtained by subtracting the computed velocity heads from the static head for no flow. These pressures are represented by the broken lines in figure 30. The results agree favorably with those obtained by the hydraulic method.

42. **Calibration of Roller Gate.**—Two calibrations were made on the roller gates; one on the 1:20 scale Denver model, consisting of a partial gate, and the other on a 1:40 scale model of the complete intake structure at the Montrose laboratory.

The calibration of the 1:20 model was made with the gate, the gate sill, and channel floor as in the final design, but with the piers as in the original design. The training dikes and trash rack which exist in the prototype were omitted on the model.

The calibration of the Montrose model was made after the final location of training dikes and trash racks had been developed and incorporated in the model. The gates in this model were calibrated for total discharges of 2,200 to 15,000 second-feet using reservoir elevations ranging from 179.0 to 191.0. To maintain a minimum velocity of 3.0 feet per second in the channels downstream from the gates, an arbitrary distribution of discharge was evolved such that a range of from 2,200 to 6,000 second-feet might be attained through one gate, a range of from 4,400 to 11,000 second-feet through two adjacent gates operating in parallel, and a range of from 6,600 to 15,000 second-feet through three gates operating simultaneously. It was reasoned that, by the time development reaches a stage in which operation of the fourth gate is desired, discharge curves from current-meter measurements, made as part of the routine operation of the structure, should be available; hence, the fourth gate was omitted in the calibration.

The values obtained from the calibration of the Montrose model with one, two, and three gates operating, expressed in terms of the prototype, are plotted in figure 31. Where two or three gates were operated simultaneously, each was opened the same amount. The term "gate opening" signifies the vertical height in feet (prototype) of the lip of the gate above the gate sill, elevation 172.0, in the final design. The values obtained from the Denver model were not used in constructing the discharge diagram since the model differed from the structure as constructed.

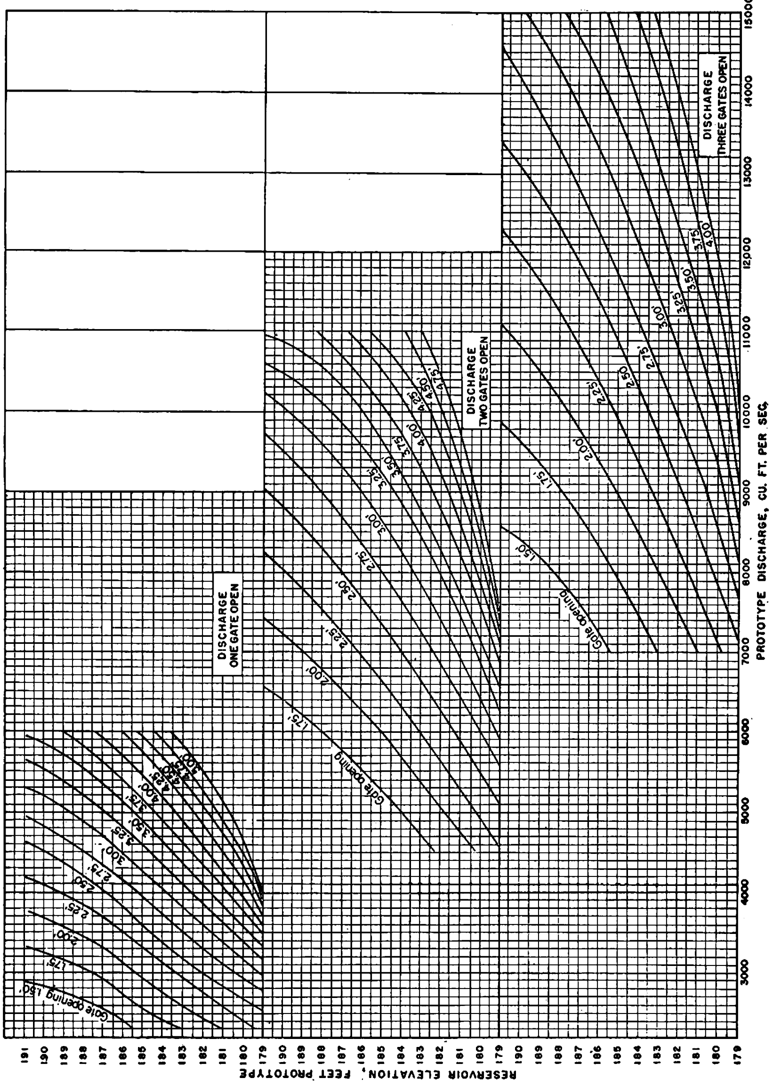


FIGURE 31.—CALIBRATION CURVES FOR ALL-AMERICAN CANAL HEADGATES.

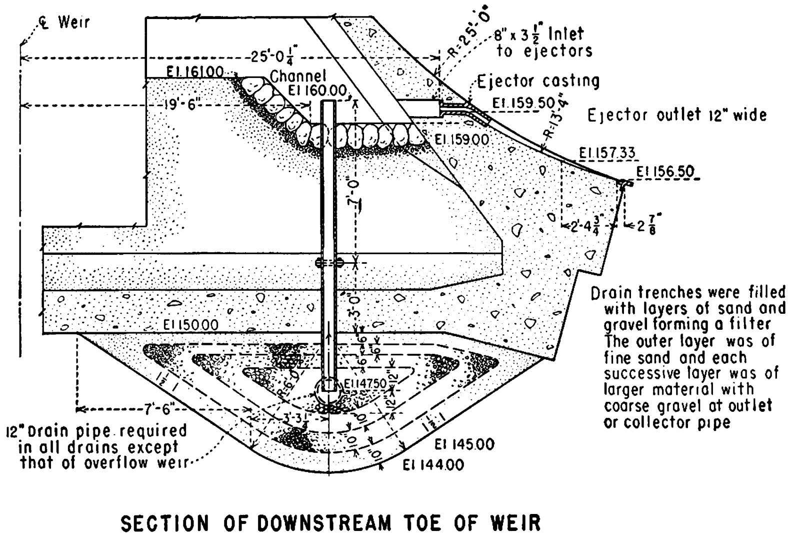
CHAPTER V—PERCOLATION STUDIES

EJECTOR TESTS

43. **History of Ejector Tests.**—The spillway of Imperial Dam, as shown in figure 3, was designed to have a large drain beneath the structure, from which the seepage water would rise through a vertical pipe into a longitudinal channel in the hollow spillway section, and thence escape through the downstream face of the dam. For reasons of stability, it is desirable that the water level inside the spillway be kept at a low elevation. If the water were discharged through the downstream face of the dam in simple drains, at times of high flow the level inside the dam could not be kept down to the desired elevation. It was therefore proposed to place ejectors on the seepage water outlets, through which the energy of the overflowing water would cause a suction to draw the water out rapidly enough to keep the level inside the dam to the desired elevation. Figure 32 shows the lay-out of the drain, riser pipe, channel, and ejector, as finally designed.

Little or no information was available as to the effectiveness and capacity of an installation of this type; therefore, hydraulic model studies were resorted to as a means of determining (1) the form and location of the ejector to obtain the most desirable results, (2) the flow conditions through the ejector in relation to the range of discharge over the spillway, and (3) the rate of flow through the ejector with relation to the discharge over the spillway for the purpose of determining the number of ejectors necessary to insure the relief of seepage waters within the dam during all flow conditions.

The original studies on the ejectors, which were designated as the "F series," were made in the hydraulic laboratory of the Colorado Agricultural Experiment Station, at Fort Collins, using a scale of 1:6. The model represented 12 feet of the spillway. The scale ratio and length of spillway to be represented were determined by the pump capacity available and the width of a previously constructed flume that was readily adapted to the installation of this model. The arrangement of the model is shown in figure 33.



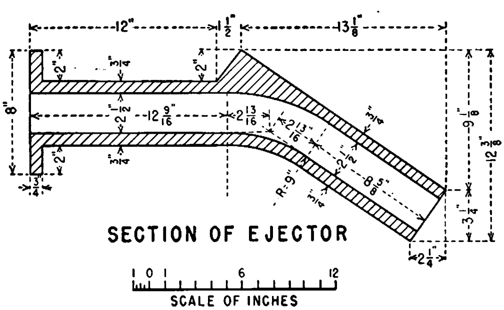


FIGURE 32.—DRAIN, RISER PIPE, CHANNEL, AND EJECTOR IN IMPERIAL DAM.

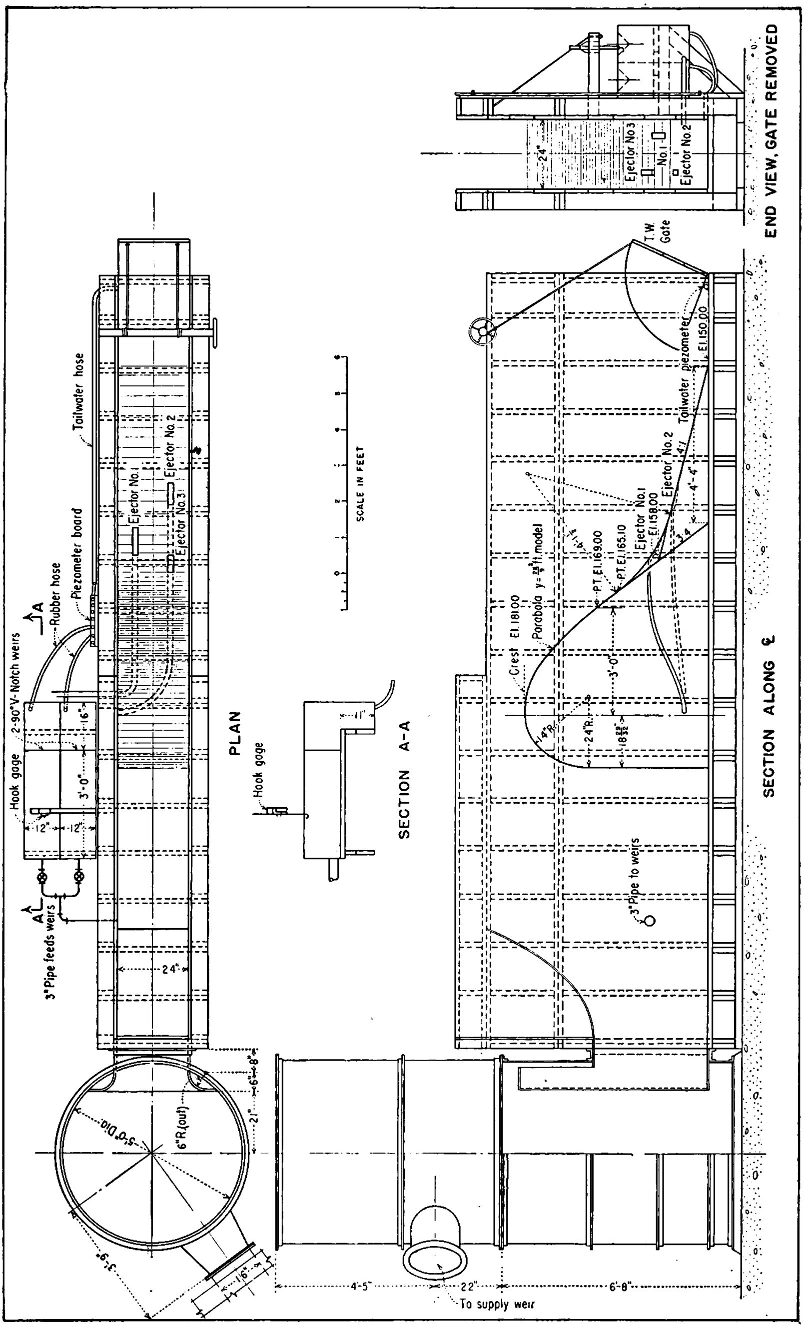


FIGURE 33.—LAY-OUT OF IMPERIAL DAM EJECTOR MODEL AT FORT COLLINS.

Water was supplied to the model by gravity from a storage reservoir and measured by a 2-foot Cipolletti weir. The upstream slope of the dam was omitted to allow better entrance conditions from the surge tank. The level apron downstream from the dam was shortened, due to lack of available space. The spillway section of the model was constructed with an angle-iron framework covered with galvanized iron, and the ejectors were formed of galvanized iron. The ejectors were connected by a pipe to a reservoir outside the flume. The water surface in this reservoir was measured by a piezometer connected to a manometer board and was assumed to be equivalent to the water surface inside the dam on the prototype. The flow through the ejector was measured by two 90° V-notch weirs.

As the studies on the 1:6 model progressed, the limitations and faults in this model became apparent. To eliminate these, a model of the Grand Coulee Dam spillway at the laboratory near Montrose, Colo., was modified to form a model of the Imperial Dam spillway to a scale of 1:1.5. The model represented a 6-foot length of spillway. The discharge over the spillway was measured on a 12-foot Francis weir, and the flow through the ejector on a 90° V-notch weir. The inlet to the ejectors was within the dam, as in the prototype, the side reservoir of the Fort Collins model having been found not truly representative of prototype conditions. One group of tests, designated the "M series," was made, but the studies could not be carried to a satisfactory completion because the oncoming winter necessitated closing of the laboratory. Upon completion of the new hydraulic laboratory in the customhouse in Denver, Colo., a flume was built of sufficient dimensions to remove all the objectionable features previously encountered in both the Fort Collins and Montrose tests. The studies of the D series were then made, which led to the final design.

44. **The Original Model.**—The design of the original ejector was such that the top of the casting forming the opening of the ejector conformed to the outline of the downstream face of the dam. This necessitated a channel set in the surface of the dam, to allow escape of the water flowing from the ejector opening. The channel extended downstream from the mouth of the ejector and varied in depth from the dimension of the ejector opening, to zero at the point of coincidence with the face of the dam, as shown for design F–1, figures 32 and 34.

The spillway section of the dam was designed for a reservoir surface varying from elevation 179.5, the normal operating level, to elevation 191.0, the maximum flood level. To maintain proper equilibrium in the prototype structure, it was desirable to keep the water level inside the dam between elevations 159.9 and 160.0. This made it necessary to place the inlet to the ejector below elevation 160.0, and the invert of the opening between elevations 159.5 and 160.0, so that the level in the dam would be held within this range at times of no flow over the spillway.

45. **Test Procedure.**—During the initial testing, flows over the spillway were calculated to be equivalent to a maximum discharge of 150,000 second-feet over a total crest length of 1,335 feet; later tests were for 120,000 second-feet over a 1,197.5-foot spillway. The tailwater was adjusted to conform to the tailwater curve for the river. Tailwater conditions throughout the range of discharge used on this model were such that the backwater did not cover the ejector located in the curved portion of the apron. However, the behavior of the ejector located in the sloping portion of the apron was influenced slightly by changes in tailwater elevation.

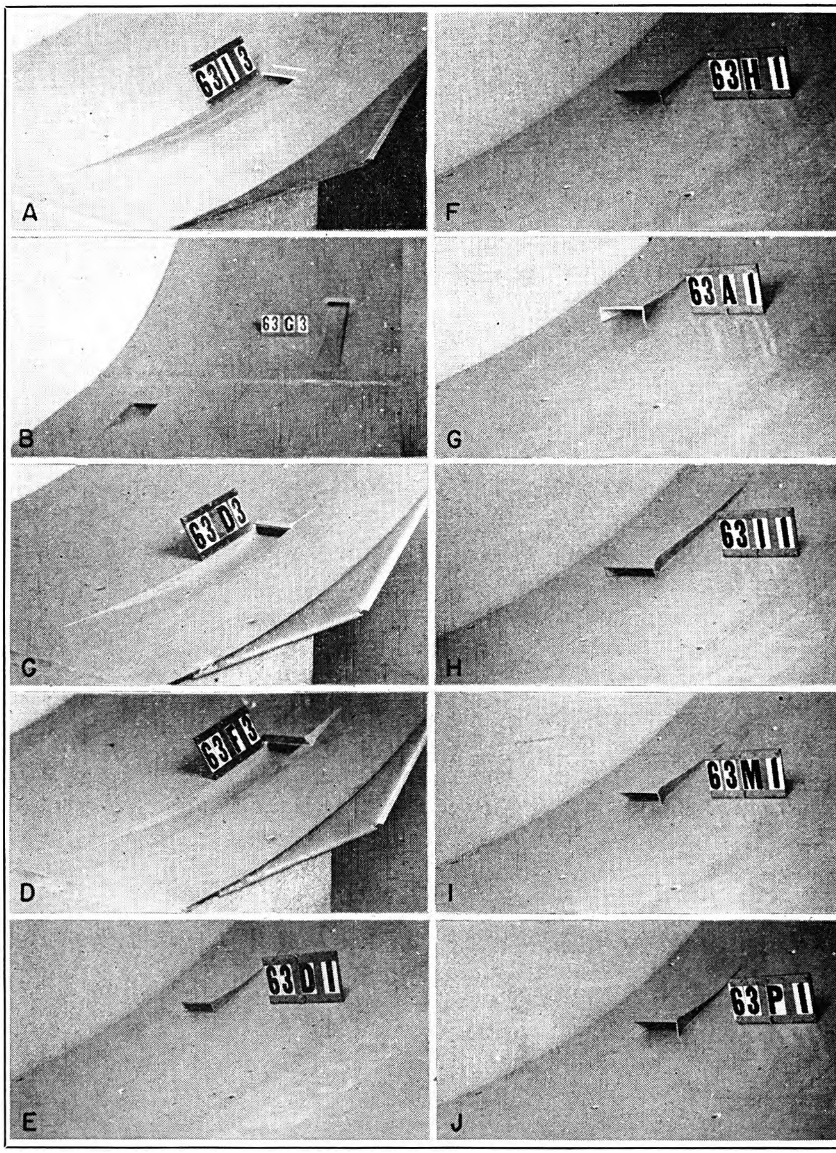
Because of the limitations of the laboratory, the maximum discharge obtainable for the model represented approximately 75 percent of the designed maximum discharge of the spillway. Calibration tests were made of each ejector by determining its capacity for several spillway discharges, distributed over the range permitted by laboratory limitations, and for various elevations of the water surface inside the dam. This was done by setting a constant discharge over the spillway, and then measuring the discharges over the V-notch weirs which would produce various constant water levels between elevation 158.0 and elevation 163.0 in the ejector reservoirs. The elevation in the ejector reservoir was assumed to be the equivalent of that which would have been reached within the prototype dam for the same flow conditions.

46. **Results From the Original Design.**—Calibration curves showing the results from the original design of ejector and a number of later models are shown in figure 35. These curves show the discharge of the ejectors for various flows over the spillway, with the water level inside the dam at elevation 160.0.

The original design, F–1, was unsatisfactory because of the rapidly decreasing discharge through the ejector as the spillway discharge increased, and the failure of the ejector to discharge at a flow over the spillway of less than half its designed capacity. Since the rate of collection of seepage waters inside the dam may be expected to increase as the head on the spillway rises, it is desirable that the discharge of the ejector should also increase as the spillway discharge increases or that the injector at least have a nearly constant discharge rate.

The characteristics of the curve derived from design F–2 were found to be much more desirable than those derived from F–1, which indicated that the location of the ejector opening in the sloping section of the apron was conducive to better efficiency from the standpoint of discharge than a location on the curved face as in design F–1. This might be expected, since the overflowing water exerted a higher pressure on the curved portion of the dam face than on the sloping surface, due to centrifugal forces, and this pressure retarded the outflow from the ejector.

Although hydraulically the ejector on the sloping face was better, this location necessitated crossing a construction joint of the dam with the



A. Design F–1.
B. Design F–2.
C. Design F–3.
D. Design F–4.
E. Design F–6.

F. Design F–7.
G. Design F–8.
H. Design F–9.
I. Design F–10.
J. Design F–11.

FIGURE 34.—IMPERIAL DAM EJECTOR MODELS—SERIES F.

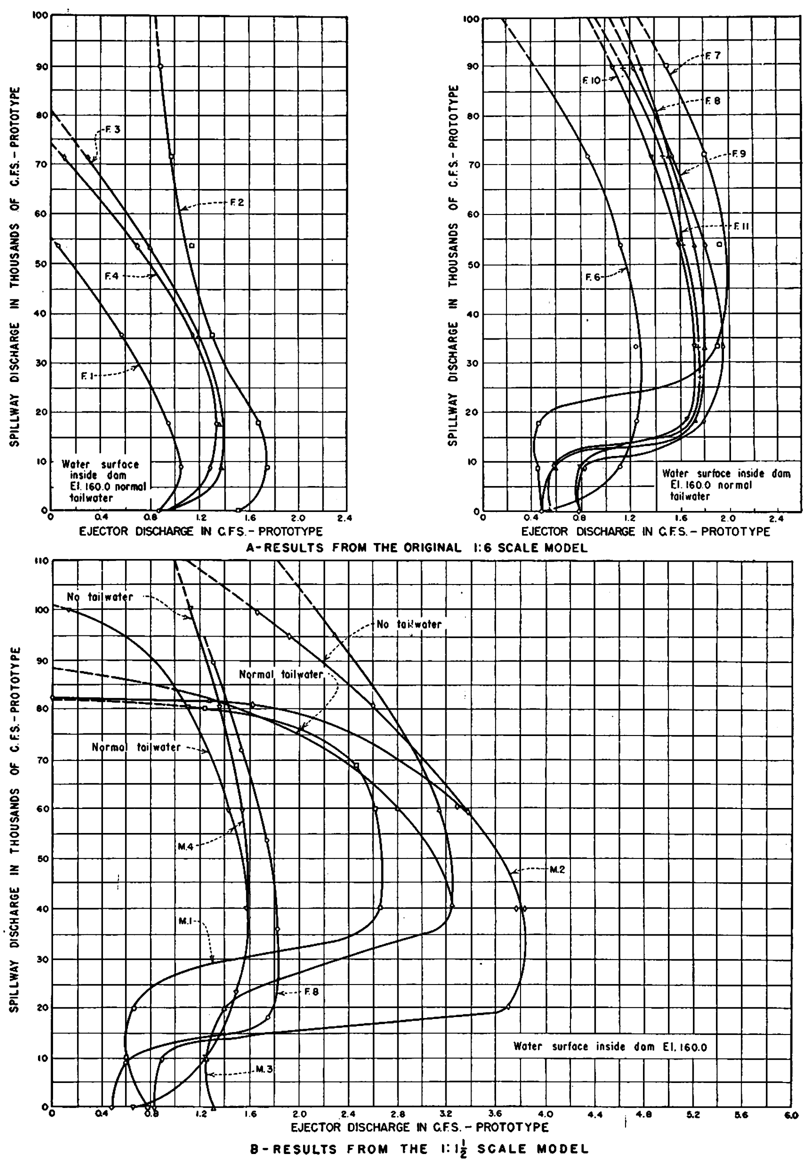


FIGURE 35.—RESULTS FROM SERIES F AND SERIES M IMPERIAL DAM EJECTORS.

ejector, which was very undesirable from a construction standpoint. Accordingly, this location was eliminated from further consideration.

47. Deflectors Over Ejector Openings.—It was believed that eddies and turbulence at the downstream end of the ejectors reduced the flow through them, and that the addition of a deflector immediately above the outlet would increase the outflow by eliminating the turbulence. The effect of the addition of deflector lips to design F–1 was therefore tested. The form of the two shapes tested, designs F–3 and F–4, is shown in figure 34, and the results of the tests are shown in figure 35–A. The addition of the deflector increased the ejector discharge throughout the range of spillway flow. The results, while superior to those of previous designs, were generally unsatisfactory because the flow diminished as the spillway discharge increased, and became zero at approximately 75 percent of the designed spillway capacity.

48. Ejectors Projecting From Face of Dam.—After an analysis of the results of the preceding tests, it appeared that better results would be obtained if the opening of the ejector were placed above the surface of the dam and the projection were suitably streamlined. These changes were incorporated in design F–6, figure 34. As shown in figure 35–A, for spillway discharges between 35,000 and 60,000 second-feet the results were comparable with those from design F–2, but above that range the ejector discharge decreased rapidly, reaching zero with a spillway flow of approximately 100,000 second-feet.

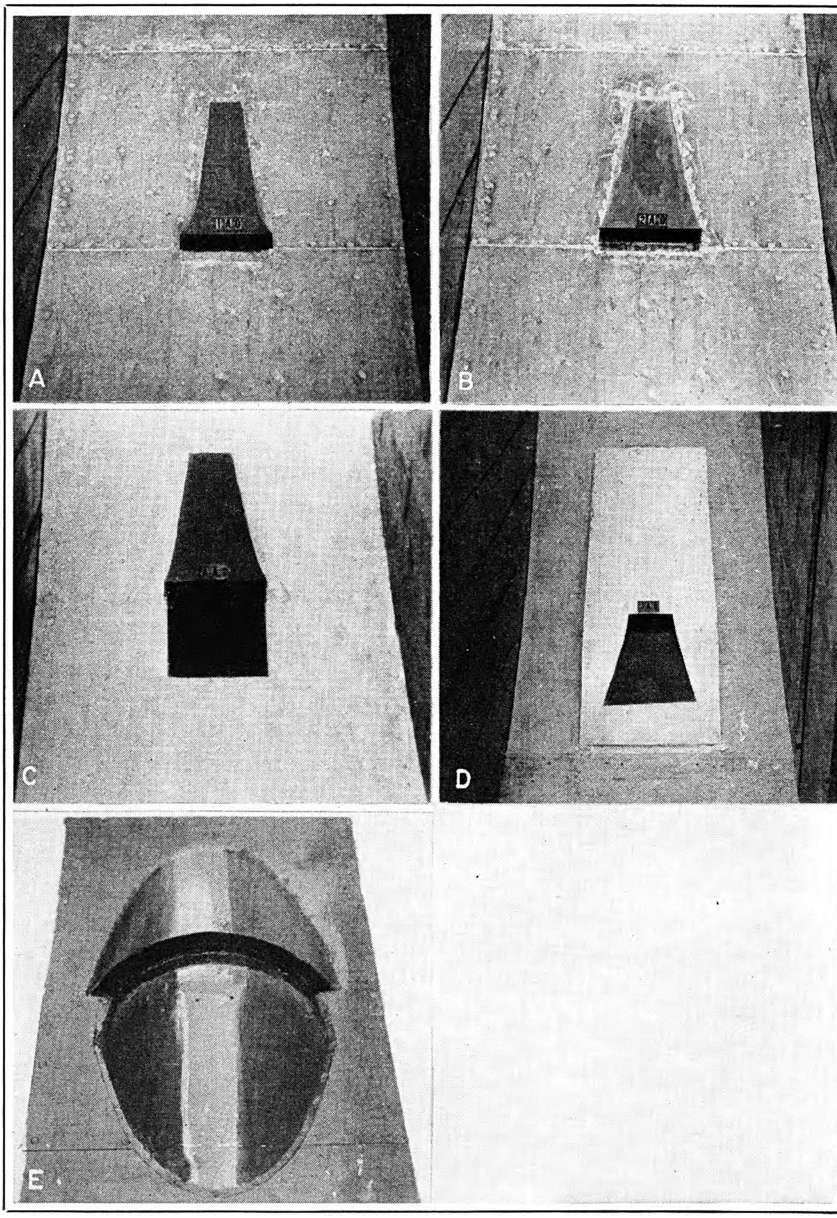
Designs F–7, F–8, F–9, F–10, and F–11 were modifications of design F–6. As shown in figure 35, calibration curves from these designs had the same general characteristic, and the variation in discharge between the designs was small. At low flows over the spillway, the projection of the ejector caused a parting of the overflowing stream, which admitted the atmosphere to the ejector and caused it to have a free discharge. As the overflowing stream became thicker this parting was less pronounced, and a spillway discharge was reached where no opening from the atmosphere to the ejector existed. When this occurred, a partial vacuum formed at the ejector outlet and the discharge from the ejector materially increased, as shown in figure 35. As the spillway discharge increased, however, the pressure of the water at the ejector opening due to centrifugal force increased, which partly offset the partial vacuum produced at the ejector. This resulted in smaller flows from the ejector at the higher spillway discharges. Of the ejectors of the F series, F–8 was believed to be the most satisfactory, as extrapolation indicated a higher discharge at maximum flow than for the others of the series.

49. The Results on the 1:1½ Model.—Because of the uncertainty of the model-prototype relations when applied to ejector action, and the uncertainties growing out of the use of an outside reservoir for the supply of the ejectors in the Fort Collins model, a series of experiments were made on the large-scale model at the Montrose laboratory. Due to the low flow existing in the supply ditch at the time investigations were made, tests were limited to a spillway flow corresponding to 85 percent of the design maximum.

The first ejector tested was a larger model of the F–8 design, and was called the M–1 type, figure 36. The results on this model differed materially from those obtained on the smaller F–8 model, as shown in figure 35–B. This is due to the difference in conditions of construction of the two models and to the greater turbulence existing in the larger model. A revision of design M–1 called the M–2 type, shown in figure 36, produced some improvement. Due to conditions in the model, the ejector was submerged by tailwater levels lower than it was believed would occur in the prototype, and tests were therefore run with the ejector both submerged and unsubmerged. A restriction of the lower half of the M–2 opening produced the M–2A type, which showed some improvement and was incorporated into the M–3 type. A radically different type, M–4, was also tested. Further tests on the 1:1½ model were prevented by the closing of the laboratory for the winter.

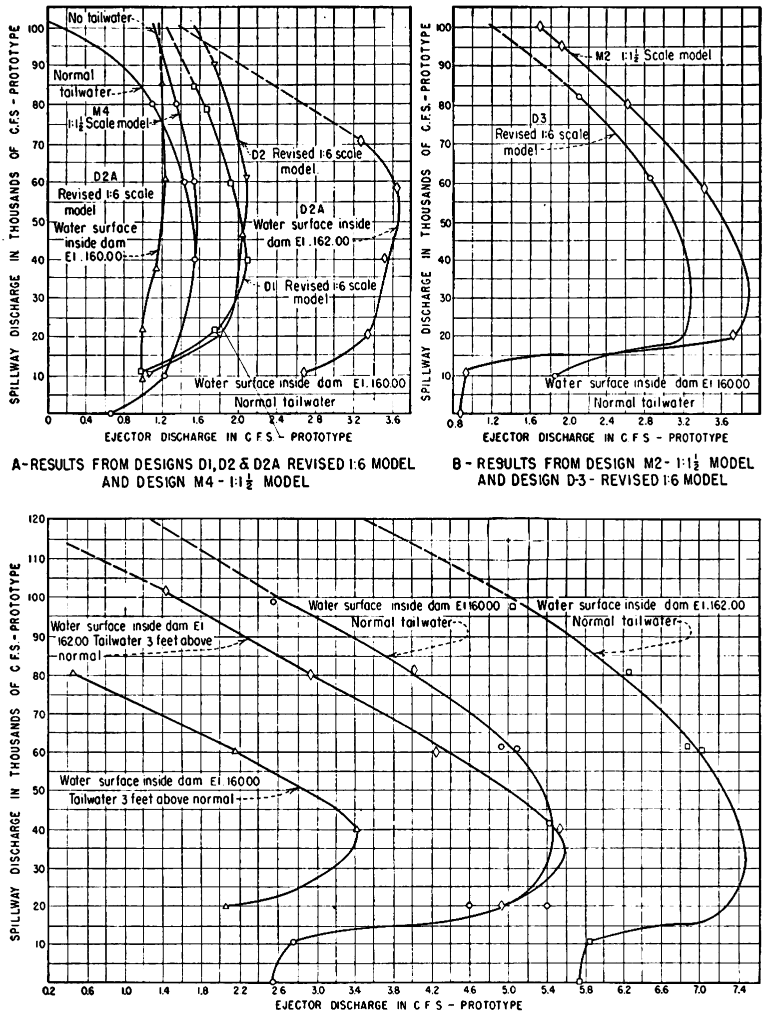
50. Results on the 1:6 Model in the Denver Laboratory.—The testing of ejectors was undertaken as soon as possible in the new laboratory in the customhouse in Denver. A model was constructed on a 1:6 scale, in which approach and tailwater conditions were more nearly similar to those on the prototype than for tests at the other laboratories. The model was similar to the M–4 model and was given the designation D–1. A comparison of the results of the D–1 and M–4 models is shown in figure 37–A. The tailwater in the D–1 model did not submerge the ejector for discharges less than 85,000 second-feet, and the results compare reasonably well with the M–4 model with no tailwater. The extension of the roof of the ejector, creating design D–2, improved the conditions somewhat for high flows.

A revision of the design of the dam changed the upper limit of permissible water level within the structure from elevation 160.0 to 162.0. The addition of a bell-mouth entrance to the ejector created design D–2A, which resulted in an almost constant ejector discharge for all spillway flows when the water inside the dam was held at elevation 160.0. As shown in figure 37–A, tests of the D–2A model, with the higher level inside, showed much greater discharge at medium spillway flows but did not show appreciably greater discharges for high flows. A comparison of the M–2 model on the 1:1½ scale was made with a similar D–3 model on a 1:6 scale. The results were in fair agreement, as shown in figure 37–B.



A. Design F–8 and M–1.
B. Design M–2 and D–3.
C. Design M–2A.
D. Design M–3.
E. Design M–4 and D–1.

FIGURE 36.—IMPERIAL DAM EJECTOR MODELS—SERIES M.



A-RESULTS FROM DESIGNS D1, D2 & D2A REVISED 1:6 MODEL AND DESIGN M4 - 1:1½ MODEL

B-RESULTS FROM DESIGN M2 - 1:1½ MODEL AND DESIGN D-3 - REVISED 1:6 MODEL

C-RESULTS FROM DESIGN D4 (FINAL DESIGN) - REVISED 1:6 MODEL

FIGURE 37.—RESULTS FROM SERIES D IMPERIAL DAM EJECTORS.

51. The Final Ejector Design.—Rapid construction of Imperial Dam made necessary an early decision on the form of ejector to be used, and a design D–4 was prepared, which used standard fittings wherever possible. This final design, shown in figure 38, consists of a specially designed cast-iron pipe ejector section, connected to a 28-inch section of 10-inch flanged cast-iron pipe and a 10-inch flanged ell. The cross section of the ejector decreases in the direction of flow and ends in a nearly rectangular opening with its lower edge flush with the face of the dam. A 1:6 model of this ejector gave materially better results than any other type tested, as shown in figure 37–C. The discharges for low and medium spillway flows were

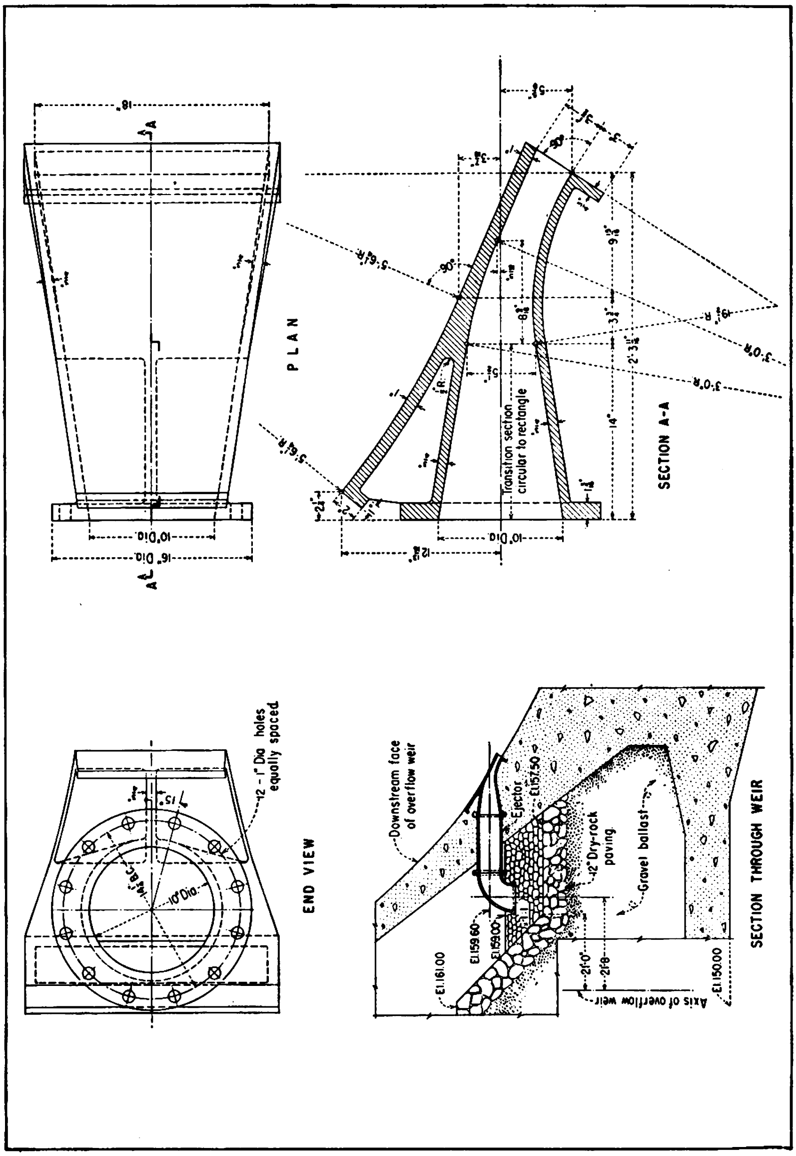


FIGURE 38.—RECOMMENDED DESIGN OF EJECTOR FOR IMPERIAL DAM.

satisfactory with a tailwater at elevation 160.0, but the figure shows that for high flows in the prototype the water level inside the dam might rise to elevation 162.0 before sufficient discharge is obtained. Tests were also made with a tailwater level 3 feet above normal. These showed considerably lower discharges, but since this condition would only occur with a lake upstream largely filled with sediment, and with seepage therefore greatly reduced, the results were believed to be satisfactory. The necessity for immediate construction of the ejectors for incorporation in the dam prevented further study and perfection of these devices.

STUDY OF PERCOLATION BENEATH SPILLWAY

52. Purpose of the Study.—Since the spillway of Imperial Dam is founded on a deep layer of very fine material, a knowledge of the percolation beneath the structure was necessary to secure an economical design. There were three principal problems of design for which solutions were obtained by model studies. One was the determination of the magnitude of the exit gradient of the seepage water as it rises through the foundation material on the downstream side of the dam, in order to insure the safety of the structure from piping. The second problem was the determination of the magnitude of the upward pressure which may exist beneath the dam, to insure that no portion of the structure might be lifted by this pressure. The third principal problem was the determination of the quantity of flow which may be discharged by the drain beneath the dam, to enable provision of a sufficient number of ejectors to take care of this flow.

To obtain the data necessary for the solution of these three problems, observations were made on the percolation of water through models of the dam foundation, and studies were also made of percolation by means of the electric analogy. The tests by these two methods are largely supplementary, since the desired results could be obtained by either method alone. At the time these studies were made, the electric analogy was a comparatively untried device in the hydraulic engineering field and its results were not as widely accepted as at present. It was desirable therefore to make parallel tests by both methods to insure the reliability of the results and also to demonstrate the practicability of the use of the electric analogy. The tests by the two methods were carried on more or less simultaneously, but since those unfamiliar with the electric analogy probably can understand it more readily if the hydraulic method is discussed first, that order of presentation will be used.

53. The Hydraulic Percolation Model Tests.—The hydraulic percolation model consisted of a wooden tank approximately 2 by 2 feet in cross section and 11 feet long, lined with sheet iron, as shown in figure 39. As will be explained later, the size of this box was investigated to be sure that it was large enough to give reliable results. Raised watertight compartments at either end formed headwater and tailwater bays, in which the desired water levels representing various conditions of headwater and tailwater could be obtained. The distance between walls A and D, figure 39, represents the over-all length of the dam foundation in the direction of flow, to a scale of 1 to 50. Walls A and D were extended below the bottom of the dam to form the upstream and downstream cut-off walls. The cut-off walls at B and C were constructed of 16-gage galvanized sheet metal. All four walls extended across the full width of the box and were soldered to the sides of the box with watertight joints.

Nearly a hundred piezometers made of copper tubes were placed in the foundation of the dam and in the sides and bottom of the box. The piezometers through the right side of the model extended 3 inches from the side wall, while the others extended the full width of the model with slotted openings at their centers. All other tubes terminated flush with the inside surface of the metal lining of the box. The apertures of all tubes inside the model were protected with copper screens to prevent silt from entering. Wherever possible the tubes were sloped upward to facilitate the escape of any entrapped air. Piezometers were also placed in the headwater and tailwater bays. All piezometers were connected to a bank of glass manometer tubes.

The inside of the model was coated with asphaltic paint and sprinkled with fine sand to prevent excessive percolation along the walls and bottom. In the first tests Colorado River bed material, shipped from Imperial Dam site, was thoroughly mixed before being placed in the model. This material had a median size of 0.10 millimeter and was largely composed of fine and very fine sand. Difficulty was experienced with this material clogging the piezometer pipes and the drain. In later tests, sand passing the number 28 and retained on the number 48 screen was substituted. To prevent undue segregation, the sand was placed in 3-inch layers while water was entering the model at the proper rate to maintain the sand and water surface at the same level. Each layer was thoroughly rodded. After the box had been completely filled, the sand was allowed to settle for several days while submerged, after which the surface was screened to the approximate outline of the underside of the dam foundation. The outline of the filter trench, see figure 32, was then cut to the proper dimensions by means of a metal template. The various layers forming the filter were then laid with the aid of other templates.

In the first tests the outline of the underside of the foundation was built of redwood blocks resting on the sand, but separation between the blocks and the sand developed. Later tests were therefore made with a topping of cement mortar cast in place.

Constant water level was maintained in the headwater and tailwater bays by Mariotte tubes (the devices frequently used to keep a constant level in drinking pans for chickens), while the water level at the drain

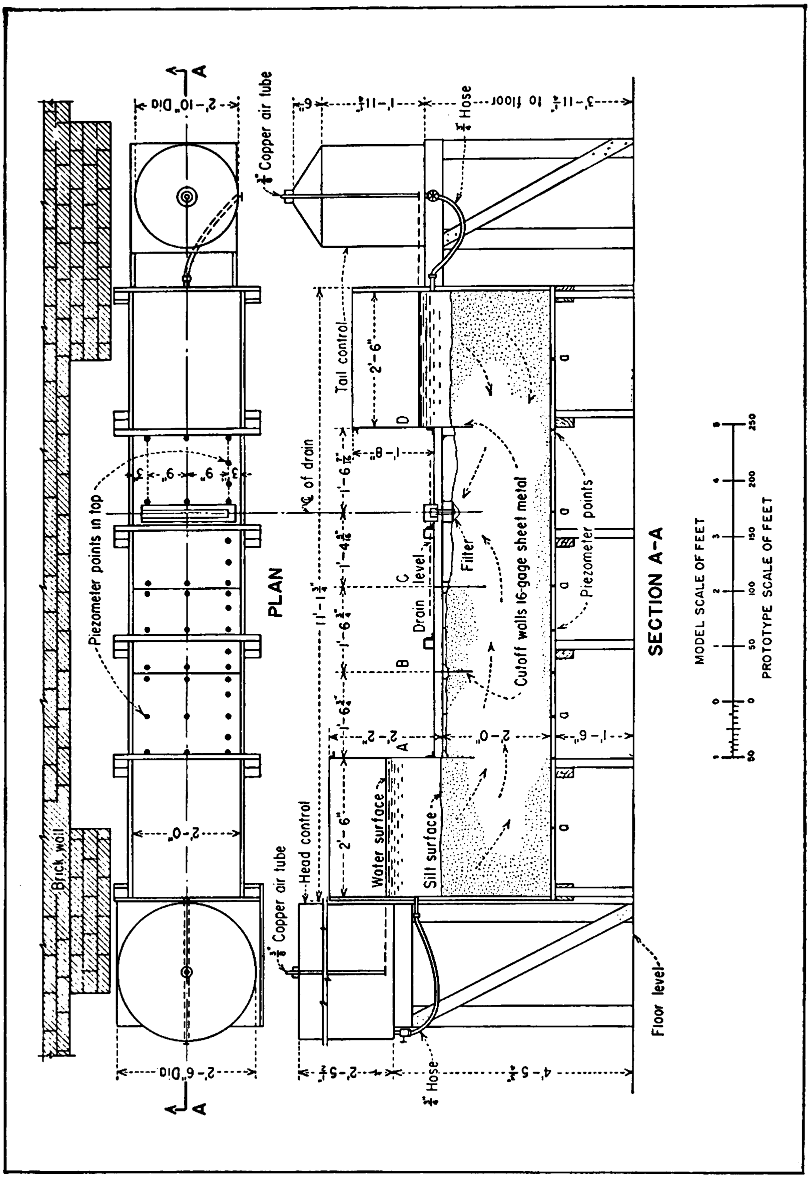


FIGURE 39.—IMPERIAL DAM HYDRAULIC PERCOLATION MODEL.

was maintained by a weir in a narrow box extending across the model and supported on seven ½-inch copper tubes which connected it to the drain filters under the foundation. The crest of the weir represented elevation 160.0 in the prototype. The weir box was connected to a tube on the piezometer board, to indicate the elevation corresponding to the level of the water inside the dam. The water used in the tests was preheated in an open boiler to eliminate the dissolved air, and was stored at room temperature in an 800-gallon elevated steel tank.

The control tanks were filled with water and the water allowed to establish a level. Then the Mariotte tubes were adjusted until the water levels in the headwater and tailwater bays were maintained at the desired elevations.

54. Results of Tests on Hydraulic Percolation Model.—In analyzing the results of the tests on the hydraulic model, the difference between the level in each piezometer and that in the piezometer connected to the headwater bay was computed. These distances were then expressed in percent of the difference in level between the headwater and tailwater bays, or the headwater bay and the drain weir box, whichever of the two was the larger. These ratios were plotted on a cross section of the dam and foundation, at the points representing the places where the measurements were taken, and lines were drawn representing equal values of the ratio, as shown in figure 40. In this figure the solid lines represent the results obtained on the percolation model, and the broken lines represent the results fot the same conditions obtained by the electric analogy, as will be explained later.

The values given represent the head lost in friction from the headwater to the point in question, expressed in percent of the total fall. The lines correspond to the potential lines of a flow net representing the flow of water under the dam. Ten different combinations of headwater, tailwater, and drain level were tested, of which the two shown in figure 40 are typical. Run No. 7 shows the conditions with headwater at elevation 191.0, tailwater at 160.0, and the water level inside the dam also at elevation 160.0. This represents the headwater conditions of the design maximum flood with a very low tailwater equal to the elevation of the level inside the dam. Run No. 10 represents the normal conditions of headwater and tailwater for the design flood, but without flow through the drain.

55. The Electric Analogy Tests.—The electric analogy apparatus used in the percolation tests can be used in solving problems in many other fields as well as hydraulic engineering. In this field it is extensively employed in the construction of flow nets, but the underlying theory is rather involved. In its use in percolation studies,[1] however, the theory can be

[1] Bradley, J. N., J. B. Drisko, and D. J. Hebert, "Hydraulic and Electric Analogy Model Studies of the Proposed Imperial Dam and Appurtenant Works, Preliminary Report No. 2," Tech. Memo. No. 471 (unpublished) Bur. of Recl., July 15, 1935.

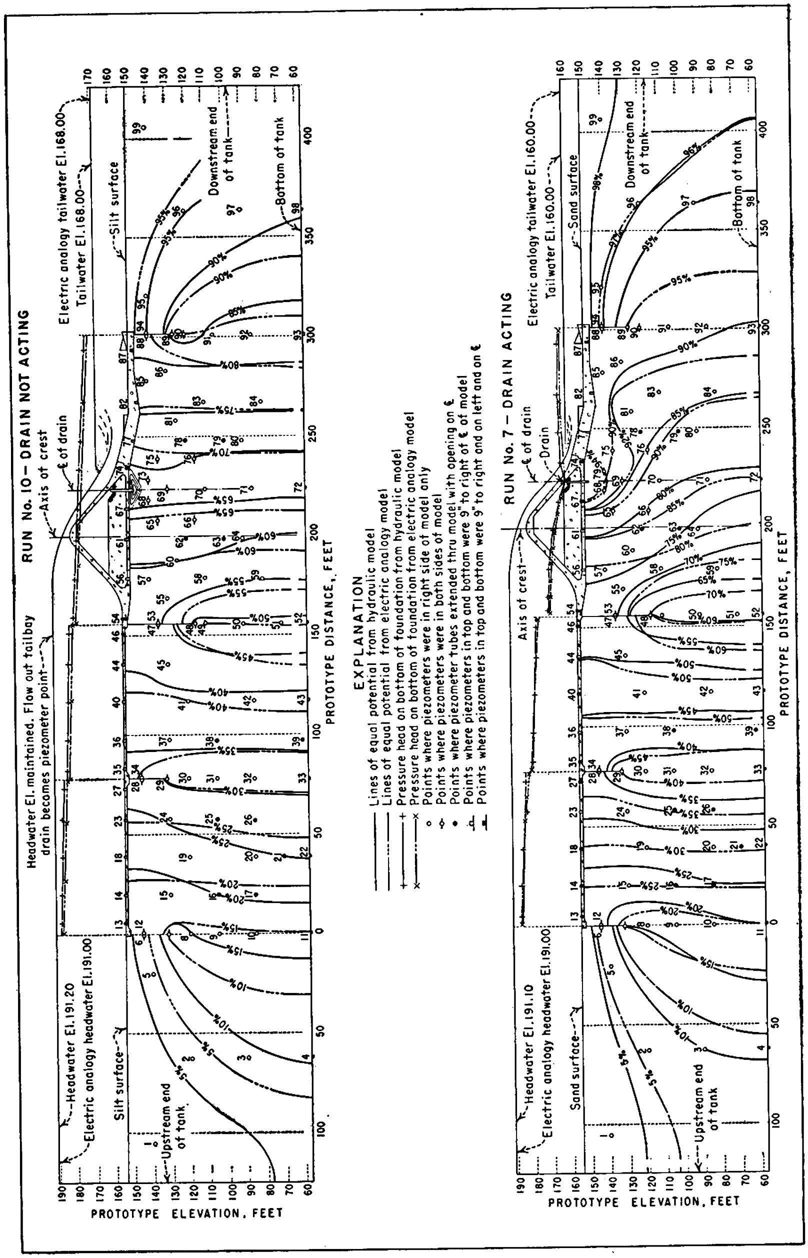


FIGURE 40.—PERCOLATION AND UPLIFT PRESSURES UNDER IMPERIAL DAM.

reduced to very simple terms, because of the similarity of the laws of flow of water through a porous soil and the flow of electricity through a conductor. The flow of water through soil follows what is known as Darcy's law and is expressed by the equation

$$Q=\frac{A}{L}Kh \tag{1}$$

where Q is the rate of flow in units of volume per unit of time, A is the area through which the flow occurs, h is the head applied, L is the length of the path of flow, and K is a constant expressing the permeability of the material, or the resistance of the material to the percolation of water. The equation for flow of electricity through a conductor follows Ohm's law, which may be expressed as

$$I=\frac{A}{L}CE \tag{2}$$

where I is the current in amperes, A is the area of the conductor, E is the electrical potential in volts, L is the length of the conductor, and C is a constant representing the relative resistance of the material of the conductor.

It will be seen that the form of these equations is the same. Therefore, if the shape of a conductor can be made to simulate accurately the shape of the soil mass through which the percolation takes place, the quantity of flow, Q, and the pressure drop, h, can be determined by electrical measurements on the conductor, providing the values of K and C are known. In many studies the variations in head, h, may be studied without a knowledge of K or C.

56. The Salt-Solution Electric Analogy Apparatus.—A very simple form of electric analogy apparatus was used in the early studies of percolation beneath Imperial Dam, as shown in figure 41–A. This apparatus was used to obtain results for comparison with those obtained on the hydraulic percolation model. The model consisted of a tray with a plate-glass bottom, carefully leveled, having a rim on three sides corresponding to the bottom and ends in a cross section of the box used in the hydraulic percolation model. The other side of the rim of the tray was partly formed of insulating material in the shape of the bottom of the Imperial Dam, with its sheet-piling cut-offs. The portions of the percolation model cross section where the headwater and the tailwater came in contact with the sand foundation material were represented by two copper plates. All of these pieces of rim were cemented to the plate glass, producing watertight joints, and the bottom of the tray was covered with a ½-inch layer of a 5-percent salt solution. The two copper plates were connected, through two 100-watt incandescent lamps in parallel, to the 110-volt, 60-cycle lighting circuit, and also by heavy copper wires to the opposite ends of a 100-centimeter length of high-resistance wire resting on a meter-

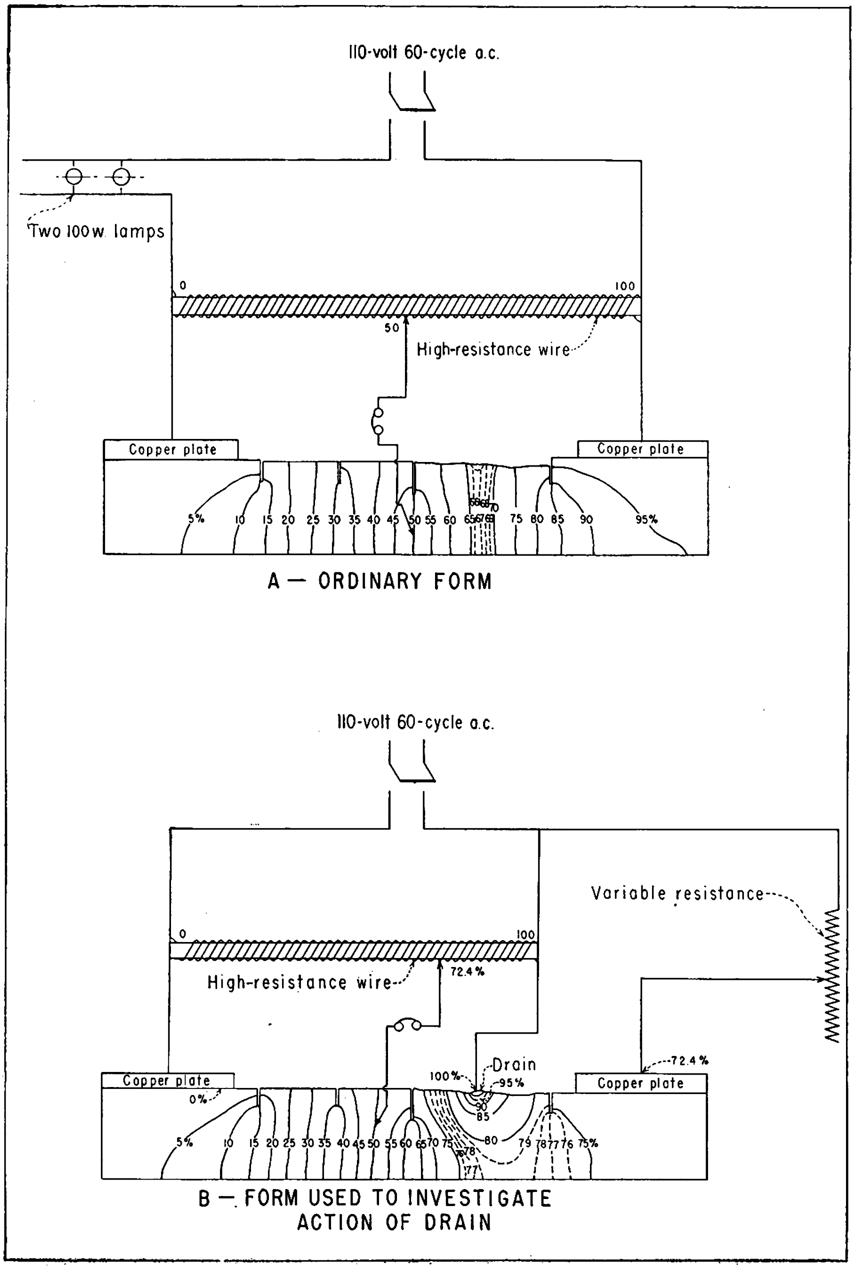


FIGURE 41.—DIAGRAM OF THE SALT-SOLUTION ELECTRIC ANALOGY—IMPERIAL DAM.

stick scale. A sliding contact was placed on the high-resistance wire and connected through a pair of 200-ohm earphones to a metal probe or pencil, the point of which could be placed at any location in the salt solution in the tray.

The operation of the salt-solution electric analogy apparatus is explained as follows: If the sliding contact on the high-resistance wire rests 40/100 of the distance from the left end of the wire, the potential at that point is below the potential at the left end of the wire by an amount equal to 40/100 of the total potential drop through the resistance wire. Since the ends of the resistance wire are connected to the two copper plates by heavy copper wires, the resistance of which is negligible, the potential drop in the high-resistance wire is practically the same as that between the copper plates. If the probe is moved about in the salt solution until it reaches a point with the same potential as the sliding contact (which potential is below the potential of the left plate by an amount equal to 40/100 of the total potential drop between the plates), no current will flow through the wire connecting that point with the resistance wire and no sound will be heard in the earphones. If, however, the probe is placed at a point which is at any other potential, a current will flow through the wire and be heard in the phones. If it is desired to find the line of 40 percent potential drop, a series of points is found where no sound can be heard, through which the 40 percent potential line can be drawn. In this manner a set of lines can be obtained similar to those determined by the hydraulic percolation model, as shown by figure 40. If the sand in the percolation model is of uniform permeability throughout and the liquid in the electric analogy is of uniform resistance throughout, the results obtained with the two methods should be the same, since the lines for the percolation model represent the loss of head in the model and those for the electric analogy represent the loss of electrical potential expressed as a ratio to the total drop of potential.

The lay-out shown in figure 41–A is satisfactory where only headwater and tailwater levels need be considered, but where the drain introduces a third level a modification of this lay-out must be used. If the investigation covers a case where the elevation of the water surface at the drain is below the tailwater level, it is necessary to place a conductor at the location of the drain and connect it by a heavy wire with the tailwater end of the resistance wire as shown in figure 41–B. The copper plate representing the tailwater is also connected with the tailwater end of the resistance wire by a variable resistance, which is varied to give the proper potential to the tailwater plate. For example, suppose the headwater is at elevation 191, the drain at elevation 162, and the tailwater at elevation 170. The drop from headwater to drain is $191-162=29$ feet, and the drop from headwater to tailwater is $191-170=21$ feet, or 72.4 percent of the 29-foot drop to the drain. The potential of the tailwater copper plate should therefore be adjusted by changing the variable resistance, until the plate potential is below the headwater potential by an amount equal to 72.4 percent of the total drop from headwater to drain. This can be done by first placing the sliding contact at 72.4 percent, then placing the probe on the tailwater plate and varying the resistance until the absence of sound in the earphones indicates that the plate is at the desired potential. Equipotential lines can then be plotted as previously described for the case in which headwater and tailwater levels only were considered.

As the work progressed, certain refinements were added to the electric analogy, the principal ones being the substitution of an ordinary resistance box for the high-resistance wire, and the introduction of an amplifier in the headphone circuit to amplify the sound in the earphones.

57. **The Tinfoil Electric Analogy Apparatus.**—Studies were also made with an electric analogy apparatus in which a sheet of tinfoil was used as the irregular-shaped conductor, instead of the water solution. The advantage of this apparatus was the ease with which a model shape could be constructed or altered, simply by cutting out the proper shape on the tinfoil sheet. A commercial tinfoil containing lead was used. It had an average thickness of 0.005 inch with a variation of ± 10 percent, which did not produce any appreciable distortion of the flow pattern. A 6-volt storage battery furnished the current and a microammeter was used in place of the earphones to detect currents. To facilitate handling, the tinfoil sheets were pasted on stiff manila paper, and the surface painted with whitewash to permit plotting of results directly on the foil.

58. **Determination of Size of Percolation Model.**—The Imperial Dam, as previously stated, is founded on a deep deposit of fine, sandy material. The total depth is unknown, but borings have penetrated into it over 60 feet. Since the worst percolation condition would result from the assumption of a very great depth of material, an infinitely deep foundation was assumed. In planning the percolation model, the question arose as to the size of the model required for the results to indicate the assumed conditions on the prototype. To investigate this problem with a percolation model would have been very laborious, but it was comparatively easily solved by the salt-solution electric analogy. The analogy apparatus was set up with a great distance between the parts representing the contact of the dam with the stream bed and the edges of the tray. The effect of reducing the size of the percolation model was studied by progressively cutting off sections of the fluid by insulating walls, and locating a set of potential lines for each step. This process was continued until the changes began to indicate appreciable differences in the location of the potential lines. The tests showed that a model having a distance from the base of the model dam to the bottom of the analogy tray of three times the greatest sheet-pile length, and a distance from the base of the dam to the sides of the tray of four times this length, would be sufficient.

59. **Comparison of Percolation Model and Electric Analogy Results.**—The results obtained by the electric analogy apparatus for two typical

runs are shown as broken lines in figure 40, and can be compared with the solid lines on the same figure representing the results obtained with the percolation model. It will be observed that a fairly close agreement was secured. In both runs the percolation model showed greater loss near the headwater than was indicated by the electric analogy. This was observed in the other cases for which comparisons were made, and probably results from a layer of impervious material which formed on the surface of the percolation model where the water entered. This is a common occurrence under such conditions, and is one of the disadvantages of percolation models.

The results of this comparison showed that the electric analogy could be relied on for percolation studies of this nature, and its greater convenience in both ease of construction and readiness of change has led to its use wherever suitable in preference to the percolation models.

60. Computation of Pressures From Flow Net Results.—Given a set of potential lines, the pressure at any point beneath a dam can be computed as follows: Assume conditions are the same as those in the preceding example, and that the point at which the pressure is desired is at elevation 150 and has a position in the flow net possessing a potential of 25 percent. The distance of the point below the headwater is $191-150=41$ feet. The potential drop from the headwater to the point would be 25 percent of the total drop of 29 feet, or 7.25 feet, and the desired pressure at the point would therefore be $41-7.25=33.75$ feet of water. By similar computations, the upward pressure acting on the bottom of the dam and the pressures acting on the sheet piling can be determined.

In figure 40 is shown the upward pressure on the base of the dam, as computed from both the percolation model and electric analogy results. The close agreement indicates the similarity of the results obtainable by the two methods.

61. Quantitative Relations for Flow Under the Dam.—It is possible to estimate the magnitude of the discharge under the dam from the results obtained on the percolation model, if the percolation capacities of the soils beneath the model and the dam are known. In the first percolation model, the material was the actual Colorado River bed material and may therefore be assumed to have the same permeability as the material under the prototype dam.

Let

h_p=loss of head per unit length in the prototype.

h_m=loss of head per unit length in the model.

L_r=ratio of prototype length to model length$=\frac{L_p}{L_m}$.

Then

$$h_m=\frac{h_p}{L_r}, \text{ and } L_m=\frac{L_p}{L_r}$$

hence

$$\frac{h_m}{L_m}=\frac{h_p/L_r}{L_p/L_r}=\frac{h_p}{L_p} \tag{3}$$

Accordingly, the hydraulic gradient, or loss of head per foot of length, is dimensionless and therefore the same in the model as in the prototype. The velocity of flow at any point under the model would therefore be the same at the corresponding point on the prototype.

Now consider a 1-foot strip in the prototype and a corresponding 1-foot strip in the model.

Let

A_p=the area of the strip in the prototype.

A_m=the corresponding area in the model.

Then

$$A_p=L_p\ (1.0) \text{ and } A_m=L_m\ (1.0)$$

and

$$\frac{A_p}{A_m}=\frac{L_p}{L_m}=L_r \tag{4}$$

Or, the ratio of the area of a strip one foot wide on the prototype to the area of a corresponding strip of equal width on the model would be the same as the linear ratio of the size of the prototype to that of the model.

The flow of water through soil, as previously stated, follows Darcy's law,

$$Q=\frac{A}{L}\ Kh$$

By letting the subscripts p and m designate prototype and model, respectively, this equation may be written as

$$Q_p=\frac{A_p}{L_p}\ K_p h_p$$

and

$$Q_m=\frac{A_m}{L_m}\ K_m h_m$$

Dividing,

$$\frac{Q_p}{Q_m}=\frac{A_p/L_p}{A_m/L_m}\cdot\frac{K_p}{K_m}\frac{h_p}{h_m}$$

or

$$\frac{Q_p}{Q_m}=\frac{A_pL_m}{A_mL_p}\cdot\frac{K_p}{K_m}\frac{h_p}{h_m} \tag{5}$$

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Since Colorado River bed material was used in both the model and prototype, the coefficient of permeability, K, is the same in each case and will drop out of the equation.

Then

$$\frac{Q_p}{Q_m}=\frac{A_p}{A_m}\frac{L_m}{L_p}\frac{h_p}{h_m}$$

From the definition of L_r, the equation becomes

$$\frac{Q_p}{Q_m}=\frac{A_p}{A_m}=\frac{L_p}{L_m} \qquad (6)$$

Or, expressed in words, the ratio of discharge per unit of length under the prototype to that per unit of length under the model would be the same as the ratio of the linear scale of the prototype to that of the model.

It should be observed that if the material used in the percolation model differs from that in the prototype the above relation will not hold, since K will not drop out of the equation. In that case it will be necessary to multiply the model discharge by both the length ratio and the ratio K_p/K_m.

The analysis required in estimating the quantity of water flowing under the dam from the electric analogy results is somewhat more complex. In the following paragraphs will be given an explanation of the method, in simple terms. Consider a small square sheet of a conductive material, such as tinfoil, 1 inch on each side, with heavy copper bars connected along the opposite edges, as in figure 42–A. Suppose the thickness of the material is such that the resistance to flow of an electric current from one bar to the other is exactly 1 ohm. Consider also a sheet 3 inches long in the direction of current flow and 1 inch wide, as in B. Since the path of the current is three times as long in B as in A, the resistance would be three times as great, or 3 ohms. Consider next a square 3 inches on a side, as in C. This may be considered as equivalent to three strips of shape B

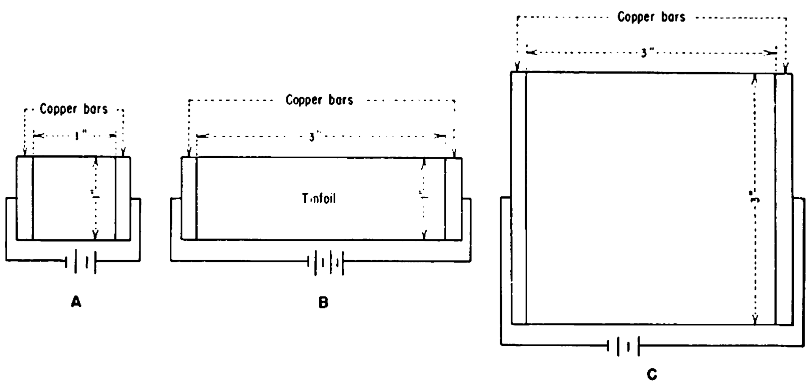


FIGURE 42.—ILLUSTRATION OF METHOD OF DETERMINING FLOW UNDER IMPERIAL DAM.

connected in parallel, and its resistance will therefore be one-third that of B, or 1 ohm. The resistance of the larger square is therefore exactly the same as that of the smaller one. In a similar way it may be shown that any square of this thickness and material will have a resistance of 1 ohm. Similarly, any rectangle with the proportions of shape B will have the same resistance as B if the current passes through in the direction of the larger dimension. It can also be shown that the ratio of the resistance of any rectangle to the resistance of a square of the same thickness and material will be equal to the ratio of the length of the rectangle in the direction of current travel to the length of the other dimension.

In the design of Imperial Dam the quantity of water passing under the dam was the information desired, and this corresponds to the current in the electric analogy. For a constant voltage the current is inversely proportional to the resistance; hence, for a potential of 1 volt, square A, which has a resistance of 1 ohm, would carry a current of 1 ampere, rectangle B would carry a current of one-third ampere, and square C also a current of 1 ampere. It is convenient to compare the currents carried by the various shapes of rectangle by comparing each of them to the current carried by a square as a standard. The rectangle B would carry one-third as much as a square, and its shape factor may therefore be considered to be one-third. Any other rectangle would have a shape factor equal to the ratio of its dimension at right angles with the direction of current to its dimension in the direction of flow. Thus a rectangle 10 inches in the direction of flow and 2 inches in the other direction would have a shape factor of two-tenths, or one-fifth.

It can similarly be shown that the resistance of a plane figure of any other shape bears a definite relation to the resistance of a square, and therefore has a definite shape factor. The tinfoil strip representing the foundation below Imperial Dam, previously described, would therefore have a certain resistance, regardless of the scale to which it was constructed, and for a given potential difference would carry the same current, regardless of its size. It too would have a definite shape factor, which would be equal to the ratio of the current carried by the strip to that carried by a square of the same tinfoil with the same voltage drop. Since the Darcy law controlling the flow of water through a porous material has the same form as Ohm's law controlling the flow of electric current, it follows that for a given head (which corresponds to the electrical potential) the flow through a prism of a certain soil of given thickness would be the same as that through any other prism of the same thickness which had the same height-width ratio and was similarly oriented to the direction of flow. For a given head, therefore, the percolation model using Colorado River sand should carry the same flow as would pass under a length of the Imperial Dam equal to the width of the model used, namely 2 feet. Since the flow of water through a soil varies directly with the head, the discharge per

2-foot length of the Imperial Dam should be equal to the discharge of the model (which had a width of 2 feet) times the ratio of head on the prototype to that on the model, or the model discharge times the ratio of the size of the prototype to that of the model. It will be seen that this is the same conclusion as obtained for the percolation model in a preceding paragraph.

To find the percolation beneath a dam by means of the electric analogy, it is necessary to determine the shape factor of the cross section of the foundation material and combine this with the permeability factor of the foundation material. The shape factor can be found by measuring the current that would flow through a sheet of tinfoil representing the shape of the earth material below the dam for a given potential drop, and measuring also the current which would flow through a square of the same material with the same potential drop. The shape factor is then the ratio of the current for the foundation shape to that for the square. The flow under the dam per foot of length would bear this same ratio to the flow through a volume of soil, one foot thick and square in section, under the same head as the dam. For convenience this volume of soil may be a cube, one foot on a side. The percolation under the dam per foot of length could thus be found by multiplying the percolation through a 1-foot cube of soil, under the head acting on the dam, by the shape factor. Since the percolation through soil varies directly with the head, it may also be found by multiplying the percolation through a 1-foot cube of soil under a 1-foot head by the head acting on the dam and then by the shape factor. The percolation through the 1-foot cube of soil for 1-foot head may be computed from observations on the percolation through permeability cylinders. Summarizing,

$$Q = B \times K \times H \tag{7}$$

where Q = discharge under the dam per lineal foot.
B = shape factor of the foundation.
K = percolation through a 1-foot cube of the foundation material under a 1-foot head.
H = head on the dam in feet.

In determining the shape factor of the foundation by the electric analogy method, the resistance of the tinfoil was too low to be measured with the equipment available, and the salt solution was therefore used for this determination. The resistance of a square of the salt solution of known thickness was measured, and the resistance of a square of the same thickness as used in the analogy tray was computed by multiplying the measured resistance by the ratio of the thickness of the square to that of the solution in the analogy tray.

The percolation coefficient, K, for the Colorado River bed material was determined by using a portion of the hydraulic percolation model in which the potential lines, as shown in figure 40, were nearly parallel entirely across the model normal to the direction of flow. From the distance in the direction of flow between the piezometers in this section, the cross-sectional area at that point, the observed pressure drop between the piezometers, and the rate of flow through the model, the percolation coefficient, K, was computed. For this purpose the portion of the percolation model was considered as though it were a percolation cylinder with that same measured length, cross-sectional area, pressure drop, and discharge. The mean value of the percolation coefficient for Colorado River bed material as thus determined is 0.000101.

By substituting in formula 7 the above value of the percolation coefficient and the values of H and Q obtained by the measurements on the hydraulic percolation model, it was possible to get the shape factor for the hydraulic model also. The following tabulation gives a comparison of the shape factors, for three conditions, as determined by the electric analogy method and the percolation model:

Elevation on prototype			Values of B	
Headwater	Drain	Tailwater	Percolation model	Electric analogy
190	160	168	0.211	0.219
180	160	158	.220	.242
Drain not acting			.122	.167

The agreement is considered very good, in view of the difficulties of getting quantitative results with percolation models. The greatest discharge under the dam will occur for the conditions represented by the first case in the above table, when the drain is below the tailwater level and water flows to the drain from both headwater and tailwater. The discharge for this case was computed from the above equation, where $H=190-160=30$, $B=.219$ (using the electric analogy value), and $K=0.000101$. The computation gave a discharge per lineal foot of dam equal to 0.000664 second-foot, which is the value on which the final drain capacity was based.

CHAPTER VI—SEDIMENTATION STUDIES

SEDIMENT PROBLEMS OF THE RESERVOIR AND ALL-AMERICAN CANAL HEADWORKS

62. **The Montrose Imperial Dam Model.**—In order to study the problems which might arise at the Imperial Dam due to the heavy sediment content of the Colorado River water, an undistorted model of the Imperial Dam and surroundings was constructed at the hydraulic laboratory of the Bureau of Reclamation in Montrose, Colorado,[2] to a scale of 1:40, and extensive experimentation was conducted. Experiments were also performed on this model to determine the action of certain parts of the development under clear-water conditions. Of these experiments the tests on the spillway, sluiceway, and All-American Canal intake gates have previously been described. The plan of the lay-out of this portion of the Montrose laboratory is shown in figure 43 and a view of it in operation is shown in figure 44. At the point of the model intake, the supply channel consisted of a chute in which the water moved at high velocity. The water was taken from the chute by an open-end trough dipping into the high-velocity stream. A 24-inch diameter welded steel pipe carried the water to a weir box, A, from which a long wooden flume, B, conveyed it to the upper end of the Imperial Dam reservoir, E, at D. A sand feed apparatus, C, was located in the flume. The model included (1) the greater part of the lake, E, above Imperial Dam; (2) the complete installation, F, comprising the dam, intake, and control works, and the desilting basins; and (3) the river bed for about 6,500 feet (prototype distance) below the dam. In all, the model covered an area of approximately 60,000 square feet.

63. **Model Tests Qualitative Only.**—It was realized at the time the tests were undertaken that quantitative results could not be obtained, but it was believed the magnitude of the problems involved justified the effort

[2] Thomas, C. W., J. E. Warnock, and C. P. Vetter, "Progress Report on Model Studies at Montrose Colorado, of the Imperial Dam and Desilting Works, All-American Canal Project," Hyd. Lab. Report No. 9 (unpublished) Bur. of Recl., July 25, 1936.

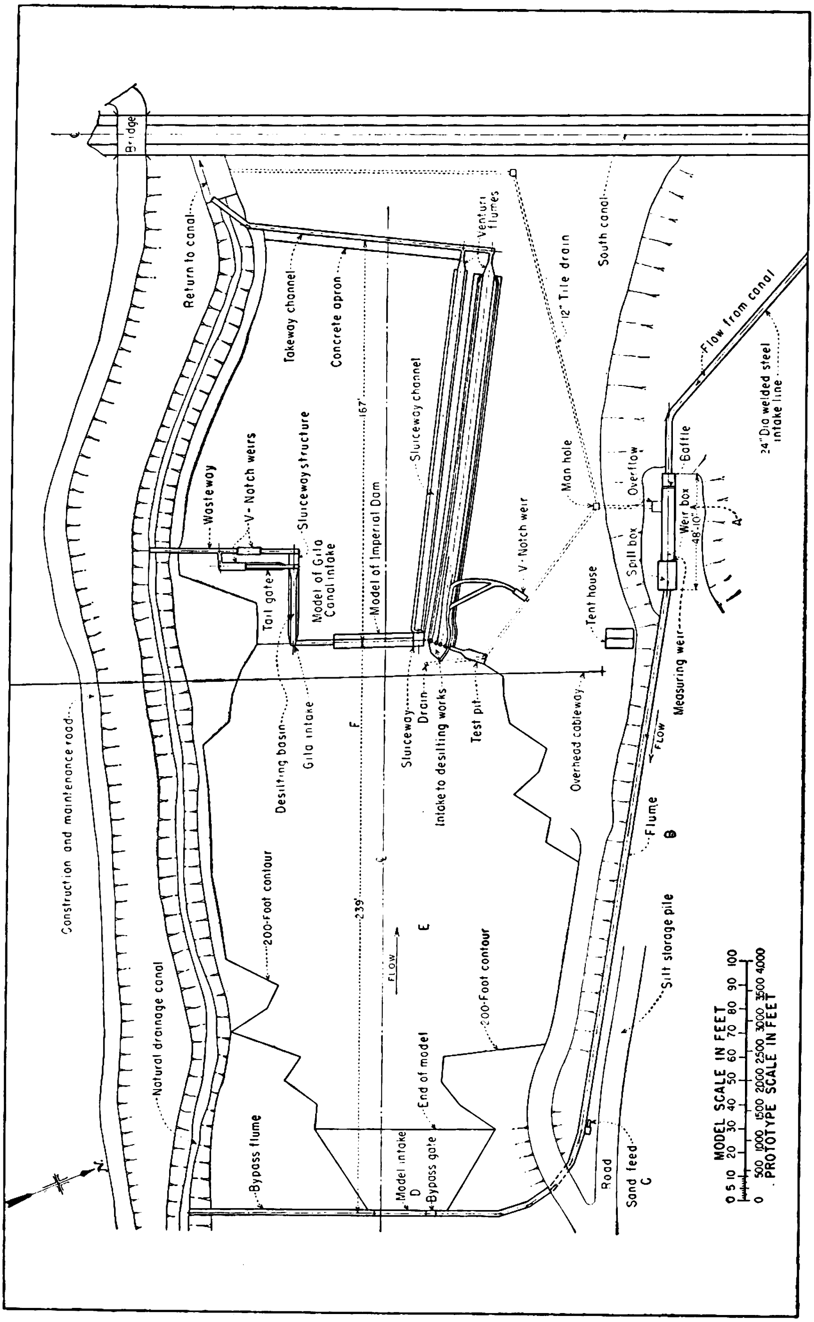


FIGURE 43.—PLAN OF MONTROSE IMPERIAL DAM MODEL.

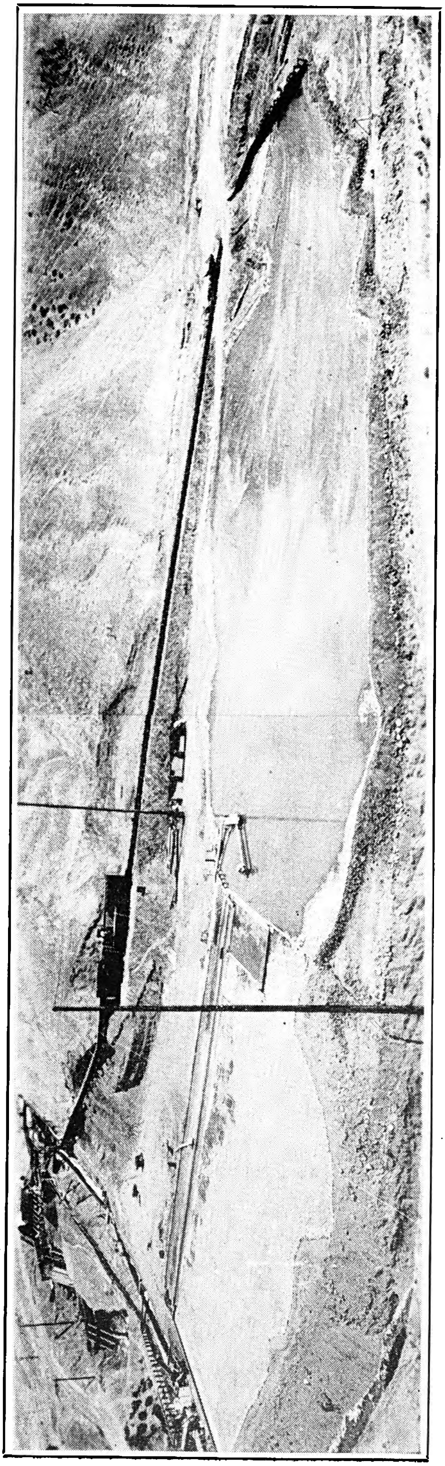


FIGURE 44.—THE IMPERIAL DAM MODEL IN OPERATION AT MONTROSE.

to obtain the best information possible, even though it be only qualitative in nature. As will be shown in the following paragraphs, very valuable information was acquired by means of the model, although exact quantitative results were not obtained. It was impossible to obtain quantitative results because the laws of sediment transportation and deposit were not known, and their relation to the scale of the model could not be ascertained. Figure 45 gives the average composition of the bed material at the Imperial Dam site, and much of the material carried by Colorado River is approximately within the range of these sizes. At times, however, considerable very fine material may come into the reservoir. If one were to make the assumption that the grain size of the bed sediment particles should be reduced to the model-prototype ratio, the material for the model would need to be of the size shown by curve B, but this material would settle under a different law than most of the sediment sizes which occur in the prototype, and would also be subject to coagulation or dispersion. Moreover, sand was available from only one source in adequate quantities and at reasonable cost; hence this was the sand used. This material, the composition of which is shown by curve C, was coarser, instead of finer, than the Colorado River bed material. One advantage of coarse material was that it would settle rapidly and the effect of depositions would be produced more rapidly than if the material were fine. It seems probable, but of course not certain, that somewhat finer material would have reproduced the conditions on the prototype more accurately.

64. **Sediment Deposits in the Reservoir.**—To learn as much as possible about the manner in which the sediment would deposit in the reservoir, a flow of 2.08 second-feet, corresponding to a river flow of 21,000 second-feet, was first discharged through the model. This was later increased to 40,000 second-feet to complete the filling of the reservoir in the time available. The sand was added to the water by a power-driven feeding device which introduced the material at a rate sufficient to produce a concentration of about one percent by weight. At first, practically all the sediment deposited near the upper end of the basin above the dam, and the water discharged over the spillway or through the sluice gates of the model. The deposits formed a delta, however, which continually built out into the lake. The edge of the delta formed roughly at right angles to the direction of flow, and the stream shifted back and forth across the deposits. The large diagram in figure 46 shows the position of the edge of the delta on various dates, and the smaller diagrams show the position of the stream flowing on the delta. The stippled portions of these diagrams represent the dry portions of the delta at the time of observation. At times the water flowed across the delta in a single fairly well-defined stream, and at others it split into a large number of smaller courses as in figure 47–A, which shows the formations of the delta after 333 hours of running. Figure 47–B shows the delta approaching the dam after 522 hours operation.

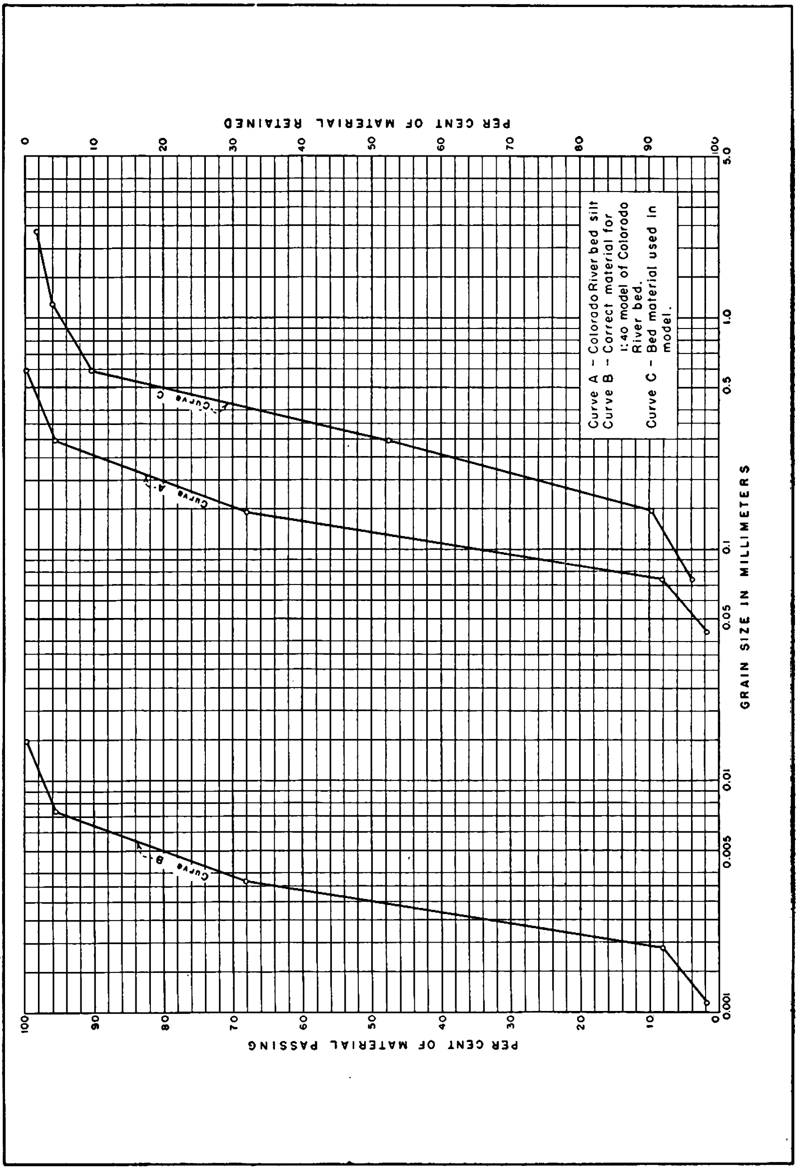
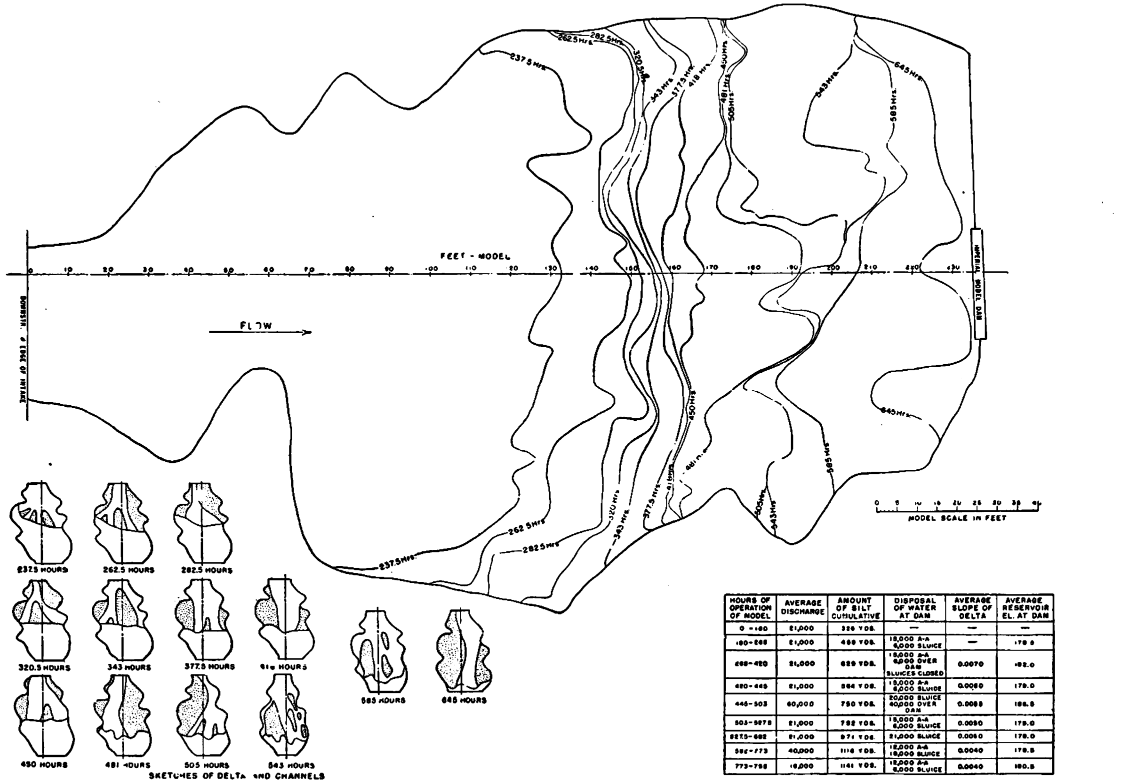


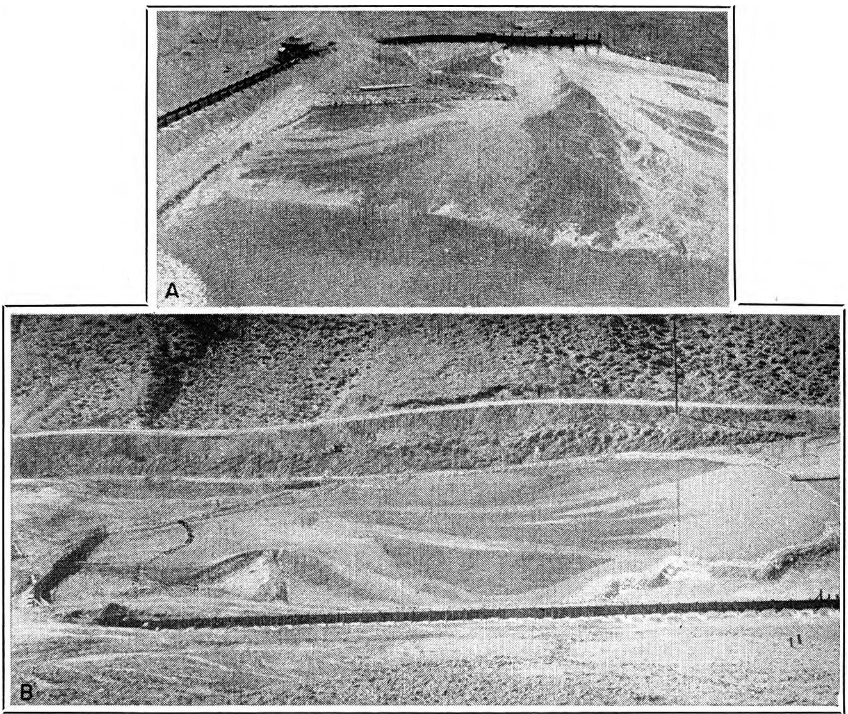
FIGURE 45.—COMPARISON OF BED MATERIALS FOR STUDIES OF IMPERIAL DAM.



HOURS OF OPERATION OF MODEL	AVERAGE DISCHARGE	AMOUNT OF SILT CUMULATIVE	DISPOSAL OF WATER AT DAM	AVERAGE SLOPE OF DELTA	AVERAGE RESERVOIR EL. AT DAM
0 - 160	21,000	326 YDS.	—	—	—
160-268	21,000	468 YDS.	15,000 A-A 6,000 SLUICE	—	178.5
268-420	21,000	629 YDS.	15,000 A-A 6,000 OVER DAM SLUICES CLOSED	0.0070	182.0
420-446	21,000	664 YDS.	15,000 A-A 6,000 SLUICE	0.0060	179.0
446-503	60,000	750 YDS.	20,000 SLUICE 40,000 OVER DAM	0.0088	186.5
503-527.5	21,000	782 YDS.	15,000 A-A 6,000 SLUICE	0.0050	175.0
527.5-582	21,000	971 YDS.	21,000 SLUICE	0.0060	178.0
582-773	40,000	1116 YDS.	12,000 A-A 18,000 SLUICE	0.0040	178.5
773-798	18,000	1141 YDS.	12,000 A-A 6,000 SLUICE	0.0040	180.5

FIGURE 46.—MOVEMENT OF SEDIMENT THROUGH RESERVOIR OF IMPERIAL DAM.

The reservoir was filled after approximately 665 hours. There was a tendency for the coarser particles of sand to deposit near the upper end of the delta and for the finer particles to flow farther down the delta before depositing. For a given discharge and rate of sand feed, the slope of the stream across the delta remained the same; and, as the edge of the delta moved out, layers of equal thickness formed on the delta, raising its level continually higher. If the discharge was increased, the slope of the stream across the delta decreased; if the discharge was restored, the slope was raised to its former value again but at a much slower rate than that of the original decrease.



A. Delta Formation After 333 Hours.
B. Delta Formation After 522 Hours.

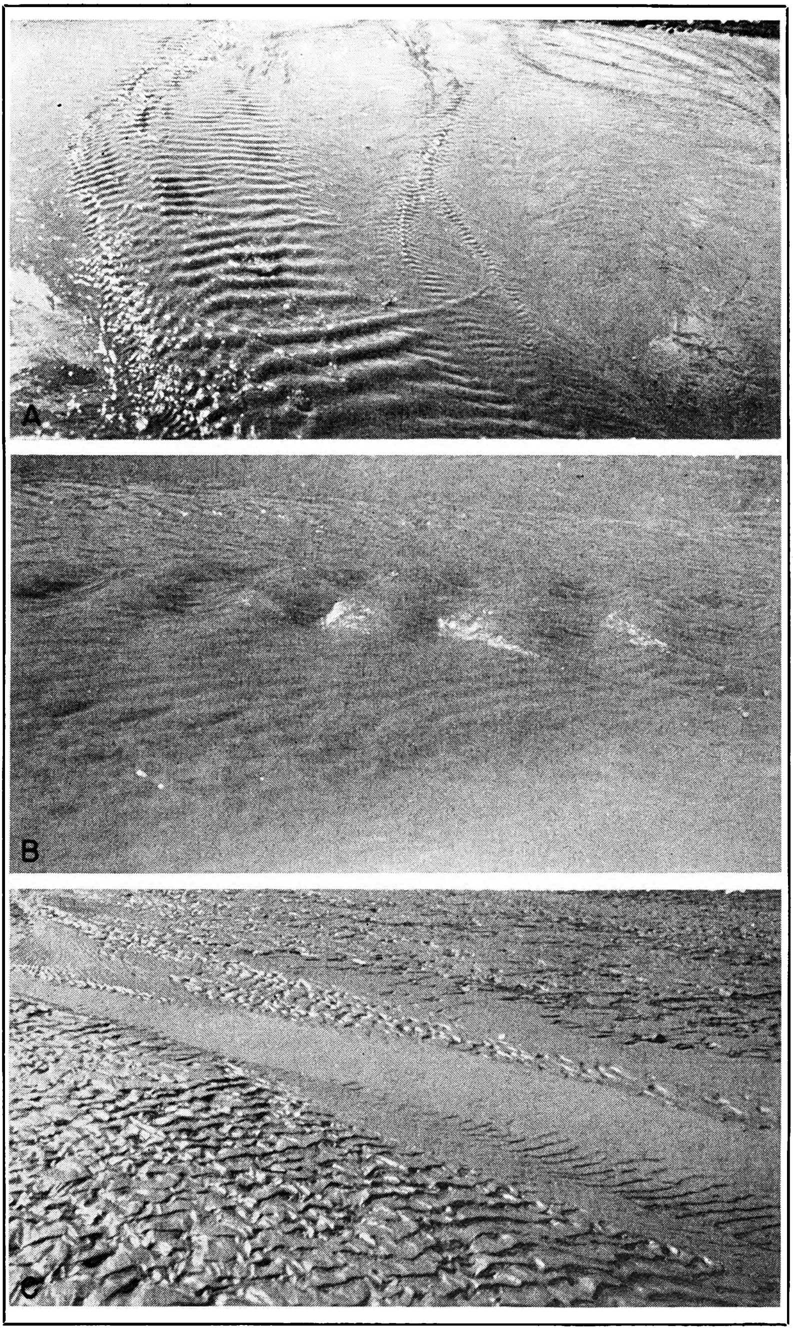
FIGURE 47.—DELTA FORMATION IN RESERVOIR OF THE IMPERIAL DAM.

The action of the model river in the delta section was in some respects very similar to that of the Colorado River itself. One occurrence noted was the formation of sand waves, sometimes called antidunes, in the bed of the channel, which frequently occurs in the Colorado River. The formation in the model of a large number of these antidunes of various sizes produced surface waves as shown in figure 48–A. A closer view of one set of these surface waves is shown in figure 48–B. The antidunes get their name from the fact that they usually move upstream rather than downstream. The bed wave, or antidune, was shaped very much like the surface wave, but with the crest of the bed wave slightly upstream from that of the surface wave. Observations showed that the depth of water on the crest of the bed wave was somewhat less than the depth in the trough, and less on the upstream side than on the downstream side.

The upstream motion of an antidune results because the water in moving along the downstream side of one bed wave removes material and deposits it on the upstream side of the next bed wave. The bed waves therefore tend to move upstream, although the water and sediment both move downstream. These bed waves form, persist for a short time, and then disappear, the whole cycle in the model covering not more than a few minutes. They frequently start where the water is smooth, and gradually grow higher, with the upstream face of each wave becoming steeper until it is nearly vertical. The accompanying surface waves then break like waves in the surf, and the bed waves and surface waves both level off and disappear, the surface becoming smooth again. In figure 48–B, the waves are just beginning to break.

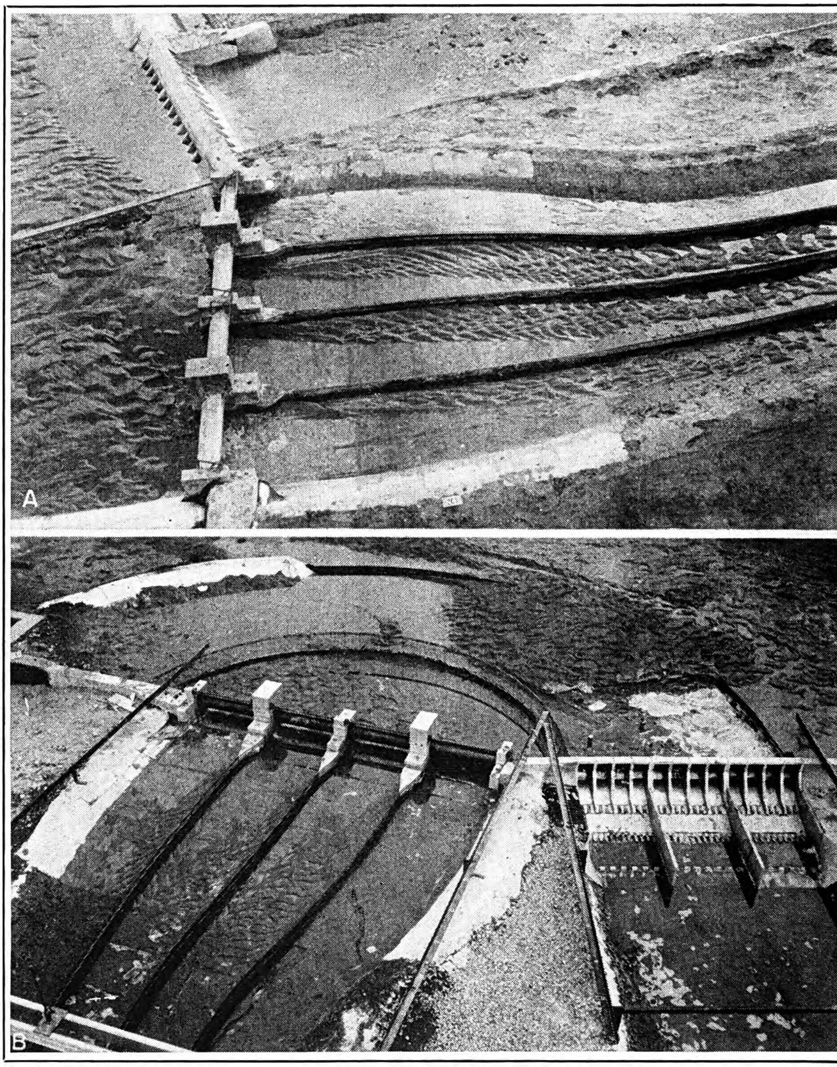
In the model, in addition to the antidunes, which moved upstream, other bed waves, called ripples, were formed which moved downstream by material being carried up the crest of each wave and deposited on the downstream side. In some cases extensive series of ripples were formed, as shown in figure 48–C. Both these ripples and the antidunes were proportionately larger in the model than those occurring in the Colorado River, but the antidunes were nearer to true model size than were the ripples; in fact, the latter were little if any smaller than those which had been observed on the prototype.

65. Effect of River Approach Direction on Intake Conditions.—It was believed that the operation of the All-American Canal headworks would be most effective if they were manipulated to discharge as much of the sediment as possible through the sluice gates, thereby decreasing the load into the desilting works. It appeared probable that more sediment would pass through the sluice gates if the current of the river flowed across in front of the sluice gates before passing through the trash racks and entering the headgates. For the river to have this direction it would necessarily move down the left bank of the reservoir and, near the dam, cross over to the headworks which are on the right bank. To test the soundness of this belief, experiments were made first with the river approaching the headworks along the right bank and then along the left bank. Each case was studied with two model lay-outs: (1) The model with the original design training dikes and trash racks; and (2) the model with no training dikes or trash racks.



A. Various Forms of Antidunes. B. Closer View of Antidunes. C. Ripple Form of Sand Waves in the Model.

FIGURE 48.—FORMATION OF ANTIDUNES IN MODEL OF IMPERIAL DAM.



A. River Approaching Along Left Bank—Original Design Training Dike and Trashrack.
B. River Approaching Along Right Bank—Plan 1 as Shown in Figure 50.

FIGURE 49.—SIX-HOUR DEPOSITION BELOW HEADGATES OF ALL-AMERICAN CANAL.

The results of the tests justified the belief expressed in the preceding paragraph, since much more sediment flowed through the canal headworks when the river approached along the right bank than when it approached along the left bank and crossed the reservoir to the right bank just above the dam. This was shown by the deposits in the channels below the gates. Figure 49–A shows the conditions after a run of 6 hours for a flow representing 30,000 second-feet, with the river approaching along the left bank and with the original design of training dike and trash rack. The effectiveness of the diversion of sediment through the sluice gates when the river passes across in front of them is due to the fact that the greater part of the sediment is carried in the lower layers of the water, which flow more slowly than the water higher in the stream. When the sluice gates are open they deflect a part of the water, and the slower moving portion near the bottom can be more easily deflected into the sluice gate than the faster moving water near the top, as the slower movement allows a greater time for the deflecting forces to act. Hence, much of the heavy-sediment-laden water is deflected into the sluices.

66. Investigation of Trash Rack and Training Dike Lay-Outs.—Although the tests indicated that the best results would be secured when the river flowed across the reservoir from the left to the right bank just above the dam, it might not always be practicable to hold the river to such a course; a study was therefore conducted to develop a form of inlet which would work successfully for either direction of approach. It was realized that if the model river were permitted to meander unrestrictedly from one bank of the stream to the other, a large number of possible directions of approach would result, and the river would not remain constant long enough to make observations practicable. The range of tests was therefore limited to three directions of approach: (1) From the left bank, (2) from the center, and (3) from the right bank.

It was thought that the flow approaching the headgate could be distributed by a submerged weir in such a way that a uniform distribution of velocity through the trash rack and a satisfactory condition of flow in the headworks might be obtained. By correctly placing the weir, a standing wave might be created which would convert part of the bed load into suspended load. The weir might also tend to deflect much of the heavy part of the load to the sluice gates. The preliminary tests of submerged weirs proved to be promising, and efforts were concentrated on determining the position and elevation of the weir, together with the lay-out of the trash racks and training walls, which would form the best entrance conditions for the All-American Canal headgates.

In the preliminary tests three weir lay-outs, each containing several alternative trash rack locations, were tested as shown on plans 1, 2, and 3 of figure 50. The model of plan 1 after test is shown in figure 49–B. It was expected to test each of a number of plans under three discharges with the three directions of approach, but investigation showed that a 6-hour duration of test was necessary to get uniform conditions. This program proved too extensive, and the three discharges were cut to a single discharge of 24,000 second-feet, 10,000 second-feet flowing through the inlet gates and the remainder through the sluices. Since many plans had to be investigated in the time available, the first test of each plan was made with

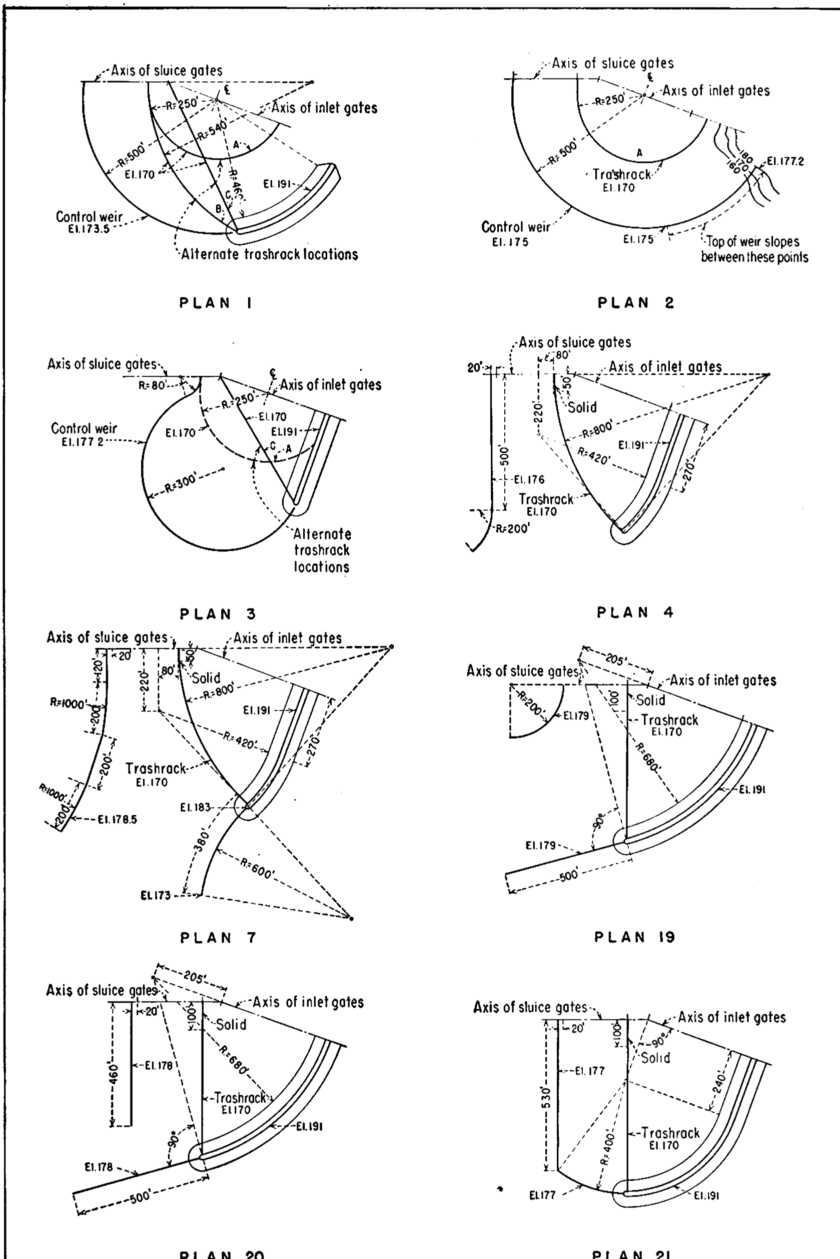


FIGURE 50.—SOME TRASH RACKS AND TRAINING DIKES STUDIED FOR IMPERIAL DAM.

probably the most severe condition, namely, with the stream approaching along the right bank; if this test proved unfavorable, no further studies of the plan were carried out. In all, 43 plans were tested, of which those shown in figures 50 and 51, including plans 1, 2, 3, 4, 7, 19, 20, 21, 22, 25, 30, 34, 36, 37, 39, and 40, are typical. In appraising the merits of these various plans, an attempt was made to select the best combination of the following conditions: (1) A distribution of flow through the trash rack, as nearly uniform as possible along the length of the rack, (2) a minimum of sediment carried through the rack and a minmum deposition within the forebay of the headgates, (3) a maximum of sediment carried through the sluice gates to the river, (4) good hydraulic conditions of discharge through the sluice gates and headgates, and (5) protection of the deposits which occurring on the upstream apron would add to the stability of the dam. Of the plans tested, Nos. 36 and 37 were most satisfactory, and No. 37 was selected as final. A view of the model for plan 37 after test is shown in figure 52–A. It was decided, however, that the construction of the submerged weir part of this plan be deferred until the most desirable characteristics, location, and dimensions could be determined by experience with the river itself.

With the trash racks extending from the bank all the way to a point on the dam between the intake gates and the sluice gates, undesirable conditions of flow occurred in this area. An investigation indicated that conditions would be improved if a solid wall or structure was built to replace about 100 feet of the trash rack adjacent to the dam. The shape and position of this structure was the subject of considerable study. Trials were made with a solid wall replacing 100 feet of trash rack: First, with the wall extending from the east end of the solid dam section between the sluice gates and intake gates; second, with the wall extending from the center of the section; and third, with the wall extending from the west end. A triangular structure about 100 feet long was also tried, as well as a rectangular one. The best results, however, were secured with a section as shown in figure 52–B.

67. Groins on Upstream Spillway Apron.—A deep deposit of sediment on the upstream apron of the spillway is desirable, as it would reduce the percolation beneath the structure and add to its stability. If the river should flow down the left bank of the reservoir and cross to the right bank near the dam (which is desirable in order to direct as much bed load as possible into the sluiceway), it might wash away a large part of the sediment on this apron. In order to find means of preventing it from doing this, experiments were made with various forms of groins [3] extending at

[3] Thomas, C. W., and H. D. Briley, "Results of Model Studies for Groins on Upstream Apron of Imperial Dam," Memo. to Research Engineer Warnock (unpublished), Bur. of Recl., Sept. 14, 1936.

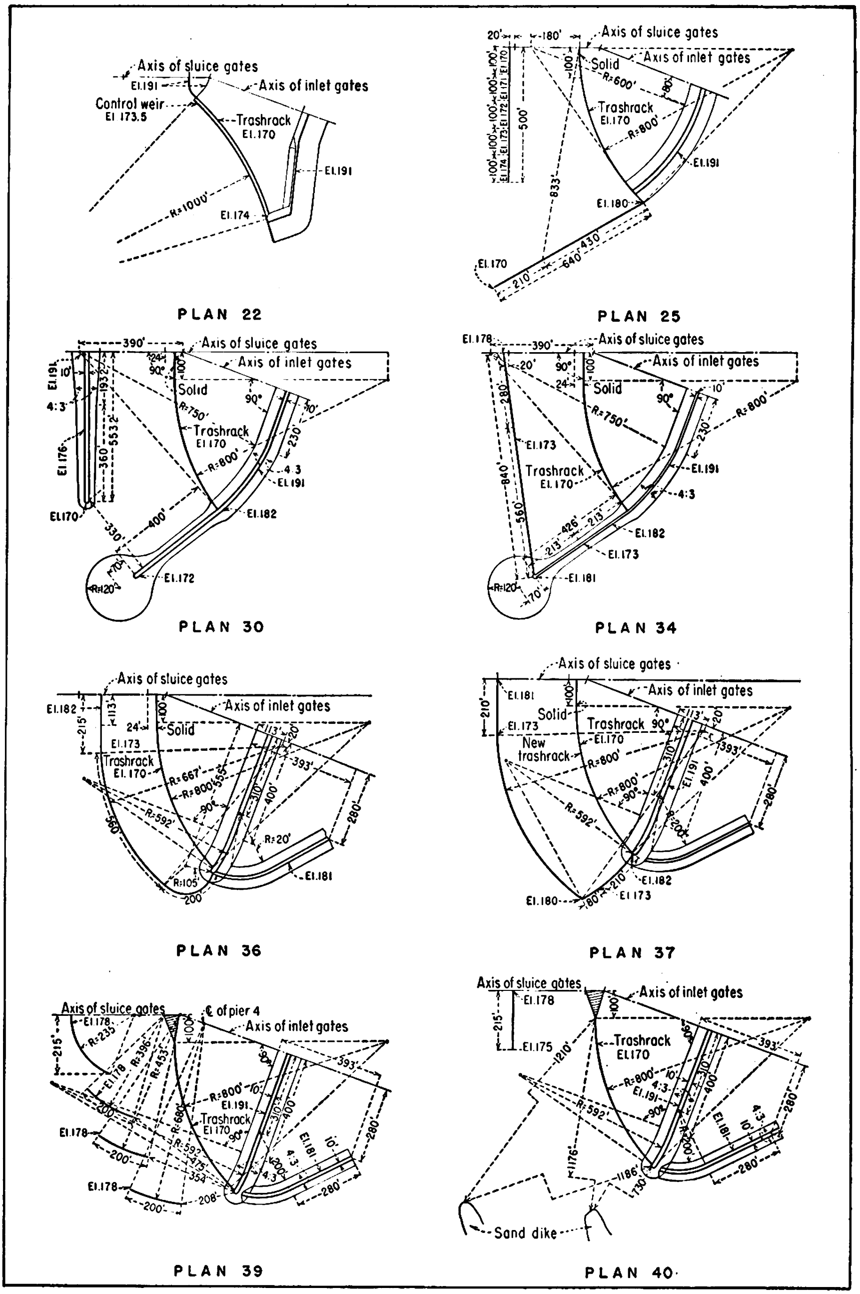
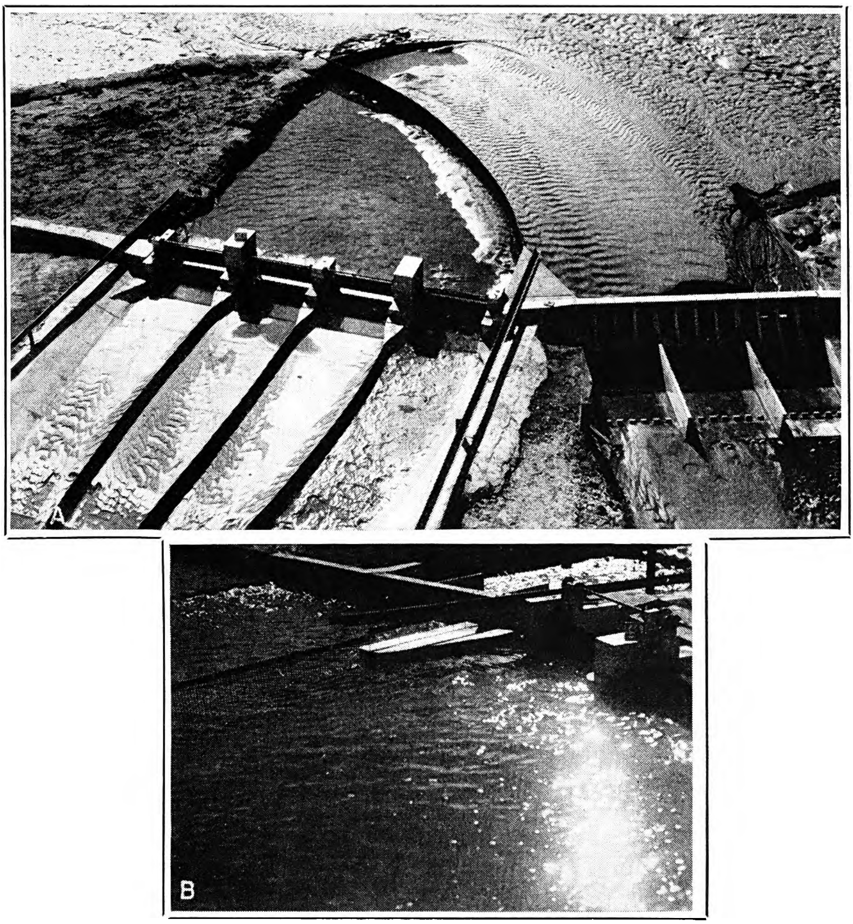


FIGURE 51.—ADDITIONAL TRASH RACKS AND TRAINING DIKES STUDIED FOR IMPERIAL DAM.

right angles to the axis of the dam as shown in figure 53–A. Seven different combinations of length, spacing, and height of groins were tested. The best results were secured with dikes about 200 feet long and 407 feet apart, beginning at the east side of the sluiceway structure. It was decided,



A. Plan 37 After 6-Hour Test With River Flowing Along Right Bank. B. Solid Wall Replacing Portion of Trash Rack.

FIGURE 52.—MODEL OF TRASH RACKS AND TRAINING DIKES FOR ALL-AMERICAN CANAL HEADGATES.

however, that only the dike at the east side of the sluiceway would be built in the initial construction. This dike begins at the upstream edge of the apron, rises to elevation 170.0 in 15 feet, thence to elevation 176.0 in a further distance of 165.5 feet, and thence is level to the face of the dam.

PROBLEMS OF THE ALL-AMERICAN CANAL DESILTING WORKS

68. **Scope of the Investigations.**—In the design of the all-American Canal desilting works several problems were encountered requiring study in the hydraulic laboratory. The lay-out of the desilting works is shown in figure 2. The problems connected with the channels involved the action of the sediment discharged from the desilting basins into the sluiceway below the sluice gates of the Imperial Dam and also of the deposits in the channels leading from the intake gates to the desilting basins. The problems connected with the desilting basin involved the development of the best form of scrapers for removing the sediment from the basin, the power requirement for the scraper system, and the method of distributing uniformly the water flowing into the basins.

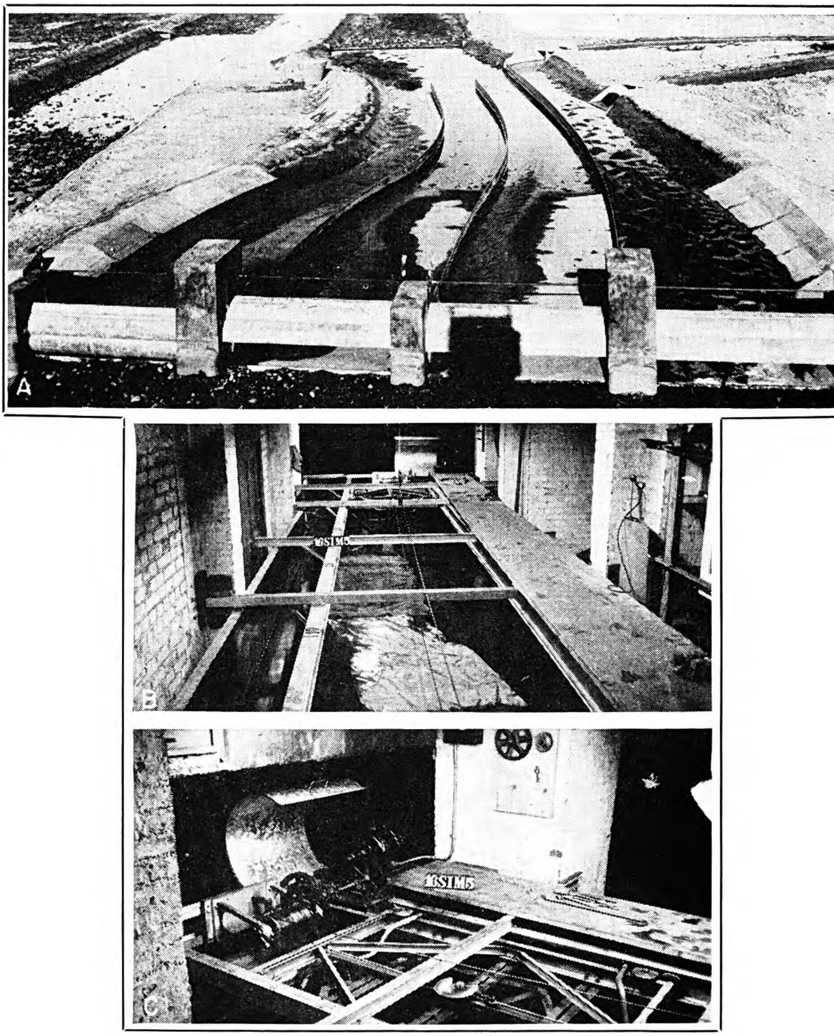
69. **Inlet Channel Alinement.**—The alinement of the inlet channels leading from the All-American Canal headgates to the desilting works was shown by tests to have an effect on the tendency of the sediment passing through them to be deposited before reaching the desilting basins. The original design of these channels had considerable curvature, as shown in figure 53–B. The curvature led to an uneven distribucion of velocity, with low velocities and consequent deposition in certain portions of the channels. This condition was especially pronounced in the outer and more curved channel. To remedy this condition, the alinement of the channels was changed to that shown in figure 54–A, which resulted in materially improved conditions.

70. **Bypass and Influent Gate Structures.**—As shown in figure 2, each of the first three inlet channels below the All-American Canal intake gates leads to an influent channel of the desilting works, through a control gate structure. The fourth channel leads through another control gate and a bypass channel directly to the All-American Canal, but it was planned that this would later lead to another pair of desilting basins to be constructed when required. In the side of each of the first three channels is a sluice gate, connecting to a bypass channel which leads directly to the main canal. Models of the bypass and influent gate structures connecting with the right-hand channel were constructed to determine the flow and sediment conditions under which they would operate. The shape of the transition section in front of the bypass gate structure was conducive to heavy deposits of sediment in front of the gates during periods when they were closed and the water was passing down the channel to the influent gates. A somewhat disturbed condition of flow occurred in the bypass gate structure, but the flow through the influent gate structure was satisfactory. A revised design of the transition was installed in the model and showed satisfactory results, although some sedimentation persisted upstream from the bypass gates.

A. Groins on Upstream Apron of Dam. B. Original Design of Inlet Channels for All-American Canal.

FIGURE 53.—APPROACH CHANNELS TO IMPERIAL DAM AND ALL-AMERICAN CANAL.

71. Sediment Removal in Sluicing Channel.—The sediment removed from the water in the desilting basins is discharged into the sluicing channel below the sluice gates in the dam. The possibility of flushing this material from the sluicing channel was investigated on the model. The model tests demonstrated that sediment could be removed very rapidly by opening the



A. Revised Alinement of Approach Channels to All-American Canal. B. Model for Silt Scraper Studies. C. Carriage to Operate Model Silt Scraper.

FIGURE 54.—MODELS FOR SILT STUDIES AT ALL-AMERICAN CANAL.

gates to full capacity, but this procedure would ordinarily be undesirable in prototype operation. Tests showed that it could also be removed, but less rapidly, by small discharges through the sluice gates. On the model, if water was continuously discharged through the gates while sediment was brought in from the desilting basins, the channel was kept clear of deposits. Material already deposited was removed from the sluicing channel with small flows of water by opening the gates first on one side of the channel and then on the other. Although the sediment used in the model was much coarser than that in the prototype and the exact relation between model and prototype results was uncertain, it was believed the model tests demonstrated that no difficulty would be experienced in keeping the sluicing channel clear.

SEDIMENT SCRAPER INVESTIGATIONS

72. **Purpose of the Investigations.**—To arrive at a decision relative to the form of desilting device to use for the All-American Canal, it was necessary to know the amount of electric power required to drive the machinery used with mechanical methods of sediment collection, since the power cost would be an important factor in comparing the total cost of sediment removal by mechanical means with that of removal by hydraulic or flushing methods, in which no auxiliary power is required. To secure the maximum economy of the mechanical methods, the apparatus should be designed for minimum power requirements. Experiments were therefore undertaken to determine the power required for mechanical methods and the means by which this could be minimized.

The mechanical method investigated involved a settling basin through which the water would pass to deposit the sediment. Some form of mechanically driven scraper would remove the sediment from the basin floor and collect it into channels, through which it could return to the river.

73. **Tests on Normal Scrapers.**—The first device tested consisted of a scraper with a blade normal to the direction of its motion, which would move across the basin and scrape the deposited sediment into trenches in the floor. In the bottom of the trenches were orifices through which the sediment would pass into pipes, where it would mix with water and be returned to the river. A model was constructed on a 1:3 scale for a portion of the basin and scraper with the trenches and pipes. Tests were made to determine the power required by the scraper and the feasibility of the flushing system. The sediment used was Colorado River bed material, the composition of which is shown in figure 45. Various speeds of scraper movement were investigated and various depths of sediment were tried. Three different forms of scraper blade were tested. The tests indicated that this method of sediment removal was feasible, and enabled improvements in the type of orifice through which the sediment would be discharged. They also indicated that a number of factors influenced the power required, one being the way in which the sediment was deposited on the bottom of the basin, and another the time which elapsed between the time of deposit and operation of the scraper. It was therefore concluded that the results obtained, while useful, were only qualitative and that model results could not be transferred to prototype with certainty. Accordingly, a full-scale section of a scraper was constructed and tested under conditions as nearly as possible like those expected in the prototype device.

As shown in B and C of figure 54, the full-scale model consisted of a long tank above which was mounted a set of rails, on which a carriage was pulled by means of a motor-driven reel. This carriage controlled a scraper blade which could be set normal or at an angle to the direction of the movement. The force required to move the carriage forward was measured by a spring dynamometer, and the rate of movement was also measured. The sediment was spread by first drawing the blade through the tank to prepare a smooth bottom. The scraper blade was then raised an amount equal to the thickness of the layer of sediment to be removed, and a layer of material deeper than the layer which it was proposed to excavate was spread over the area to be scraped. The scraper was again run through the tank, leaving a layer of just the desired thickness on top of the original bed. The blade was then set back to its original height and the scraper run, with accompanying observations of the speed and force required. Several types of blade were used, including a vertical-plane blade, three types of plow, and a Fresno-type blade.

The data obtained were erratic and it was not possible to get a quantitative analysis of the results. They indicated, however, that the Fresno-type blade required the least power. The tests revealed that the main force required to move the sediment is a linear function of the sediment depth; also, that for all depths the force increases very rapidly with speed up to 16 feet per minute, and less rapidly above this point. In practically all cases, the force increased if the sediment was allowed to consolidate longer than the usual time.

74. **Tests on Diagonal Scrapers.**—Preliminary tests on vertical-blade scrapers showed that less power was required with the blade held at an angle to the direction of movement, and detailed tests with this blade arrangement were therefore undertaken. The apparatus was that shown in photographs B and C of figure 54, with an 8-foot blade mounted diagonally to the direction of motion.

In the prototype, the diagonal blades are placed along a series of rotating arms, and scrape the deposited material toward the outlet near the axis of rotation. As it passes along, each blade leaves a *windrow* of material which it has moved toward the axis, and which is picked up by the following blade and moved still farther in that direction. In the tests on the model scraper, this condition was simulated by depositing the material in a windrow and moving it by the scraper. The tests covered four speeds of scraper movement with four blade angles and a number of different sizes of windrow.

The results of these tests are summarized in figure 55. These diagrams give the energy required to move 1 pound of sediment 1 foot laterally, for various weights of material per foot of windrow left by the preceding blade. This energy was found also to vary with the speed of motion and angle of the blade.

In figure 55–A is given the total energy required as determined by the experiments, which includes not only the energy required to move the sediment, but also the energy required to overcome the resistance of the water to the motion of the blade and to overcome mechanical friction. Figure 55–B gives the energy required to move the sediment with the mechanical and hydraulic resistances eliminated. There was reason to suppose that the frictional and hydraulic resistances of the revolving scrapers would be less in the model than in the prototype, but it was not possible from the available apparatus to determine the amount of the reduction.

The results show that the energy required to move 1 pound of sediment 1 foot laterally decreases as the weight of material per foot of windrow increases to about 50 pounds. A comparison of the energy required with and without mechanical friction and hydraulic resistance indicates that the higher unit energy requirement at the lower windrow sizes is largely due to the fact that the hydraulic and frictional resistances are independent of the windrow size, and therefore are a greater part of the total resistance when the amount of sediment moved is small. The results also indicate that the total energy required increases with the speed of blade movement, and that it decreases with increasing blade angles up to a certain value, beyond which it increases. The tests were not sufficiently exact to permit an accurate determination of the angle giving minimum energy requirements, but it was believed to be near the upper limit of the angles considered.

STUDY OF INFLUENT SLOTS

75. Purpose of Study.—As described in section 1 and shown in figure 2, water from the All-American Canal headgates flows down the diversion channels to three influent channels,[4] which admit the water into six desilting basins. The basins are arranged in pairs, and each pair is served by one of the influent channels, which extends between the two basins. The width of each influent channel decreases in the direction of flow, and either side of the channel contains 110 vertical slots through which the water is released into the basins. The slots were designed to distribute the water equally over the desilting basins and to reduce the relatively high velocity of flow in the influent channels to a low velocity in the desilting basins with as little turbulence as possible so as to prevent retardation of sedimentation within the basins. The water from the left side of each influent channel flows across one desilting basin in a direction of 60° from that of its flow in the channel, and the water flowing through the right side turns 120° to flow across the other basin. Since, to reduce turbulence, it was desirable that these changes of direction take place within the slots, the slots were designed both to reduce the velocity of the water and

[4] Cline, F. R., "Preliminary Report on Model Studies of Influent Slots in All-American Canal Desilting Works," Memo. to Chief Designing Engineer (unpublished), Bur. of Recl., May 7, 1937.

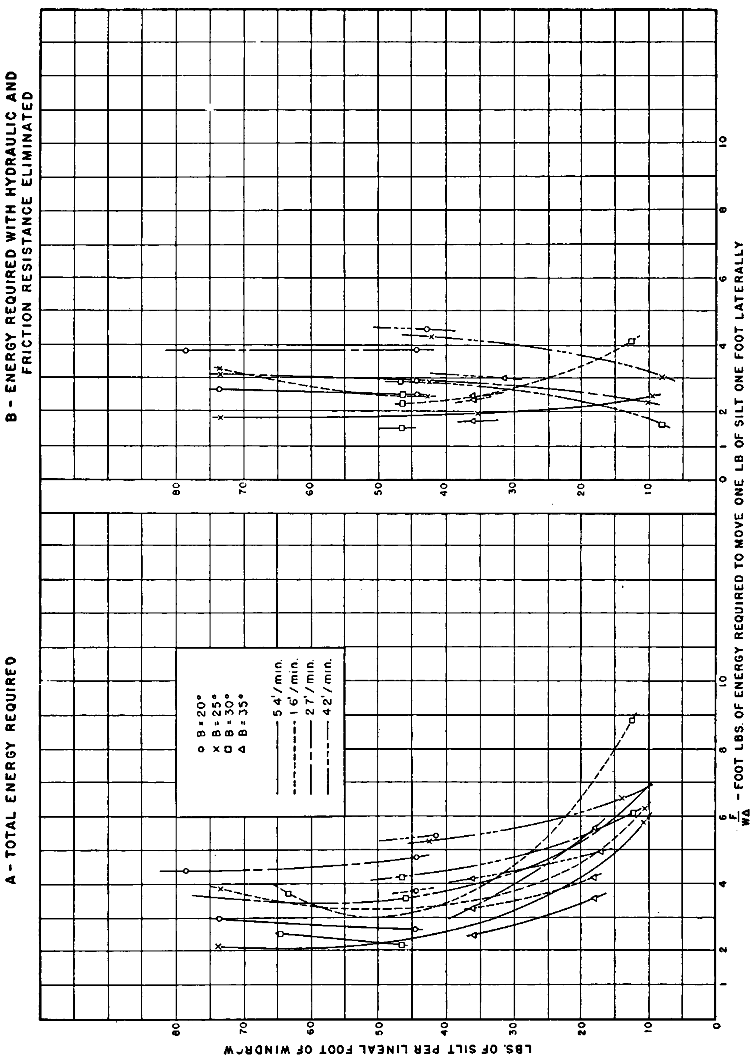


FIGURE 55.—RESULTS OF DIAGONAL SCRAPER TESTS FOR THE ALL-AMERICAN CANAL DESILTING WORKS.

change its direction, one set of slots by 60° and the other by 120°. The sides of the influent channel between adjacent slots are parallel planes, and the reduction in channel width is accomplished by offsetting the walls 2.45 inches at each of the influent slots, which are spaced 7 feet apart in the direction of flow.

The purpose of the model tests of the influent slots was to determine the shape of slot which would change the direction of flow as revised and most efficiently reduce the velocity from the high velocity in the channel to a low velocity within the basin, the direction change and velocity reduction to occur within the slot itself.

76. The Model Tests.—The apparatus used was a full-scale model of an 8-inch length of one gate slot, together with a sufficiently large part of the influent channel and settling basin to determine their action. It was assumed that the action of this depth of water would be the same as that of a layer of corresponding depth at any level in the height of the influent slot. The bottom and top of the portion of the model representing the slot and immediately adjacent areas were formed of sheets of transparent pyralin to enable observation of the motion of the water through the slot. The flow through the slot and the total flow approaching the slot were measured on weirs. Numerous piezometers were placed in both the bottom and top faces of the model in the parts representing the influent channel, the desilting basin, and the center lines of the slot.

The tests were usually made with a flow corresponding to the discharge capacity of the basins, it being assumed that the discharge was distributed equally to all slots. However, some tests were made with somewhat lower and higher flows in order to investigate the effect of an unequal distribution. The conditions of flow through the slot were observed by introducing into the stream a large number of one-quarter-inch black cubes made of live oak wood, which had a specific gravity practically the same as that of water. By placing a bright light below the model, the action of the water containing these cubes could be observed. The conditions in each model were recorded photographically by slow-motion pictures, and notes and sketches were made describing in detail the action observed.

77. Results of Tests.—Tests were conducted on 14 different shapes of 120° influent slots. The results were judged on the basis of two different considerations, which were given equal weight. The first was the excess of the pressure head in the approach channel, upstream from the slot entrance, above that in the basin; or in other words, the magnitude of the transformation of the velocity head in the approach channel to pressure head in passing through the slot. The second criterion was the appearance of the flow, as observed in the model, with regard to its freedom from turbulence and undesirable currents.

The first slot tested for the 120° bend, shape A, proved to be fairly satisfactory. The water passed through on the outside of the bend with little disturbance, but on the inside of the bend there was considerable turbulence. The dimensions of this slot and the details of its action are shown in figure 56. The next tests were made on slots with the change of direction accomplished near the slot entrance. Shape O, as shown in figure 56, proved to be the best of this series. Another type was tried of which shape M in figure 57 is typical, but poor conditions of flow prevailed in most of the expanding section. Of the 14 shapes tested, shape O appeared to be the best and was therefore adopted as the final design.

The problem of the 60° slot was not to secure the most efficient design, but to produce one giving results similar to those obtained on the shape O, selected for the 120° bend. This was necessary in order that symmetry exist in the flow through each pair of desilting basins. Shape S, as shown in figure 57, proved satisfactory and was adopted for the 60° slots.

On the basis of these tests, the following general conclusions were drawn regarding the flow of water in channels of the type used in these slots:

1. To give a uniform velocity distribution at the end of turn, walls should be slightly converging. Otherwise, the water will tend to follow the outside wall.

2. If the channel is narrow and the velocity relatively high at a turn, the flow distribution can be controlled by a small convergence of the walls. In a wider channel, with the radius of the inside wall remaining the same, the convergence will need to be much greater to accomplish an equal result with the same flow.

3. If the widening of a channel is too rapid, the jet will not completely fill the channel and eddies will form on either or both sides. The lower the velocities, the more rapidly the widening can be accomplished.

4. Losses in narrow shapes increase more rapidly with increase in discharge than they do in wider shapes.

5. In the tests, the best streamline appearance usually accompanied narrow shapes and high velocities, while the higher values of velocity-head recovery accompanied the wider slots, in spite of the formation of many small eddies in the latter.

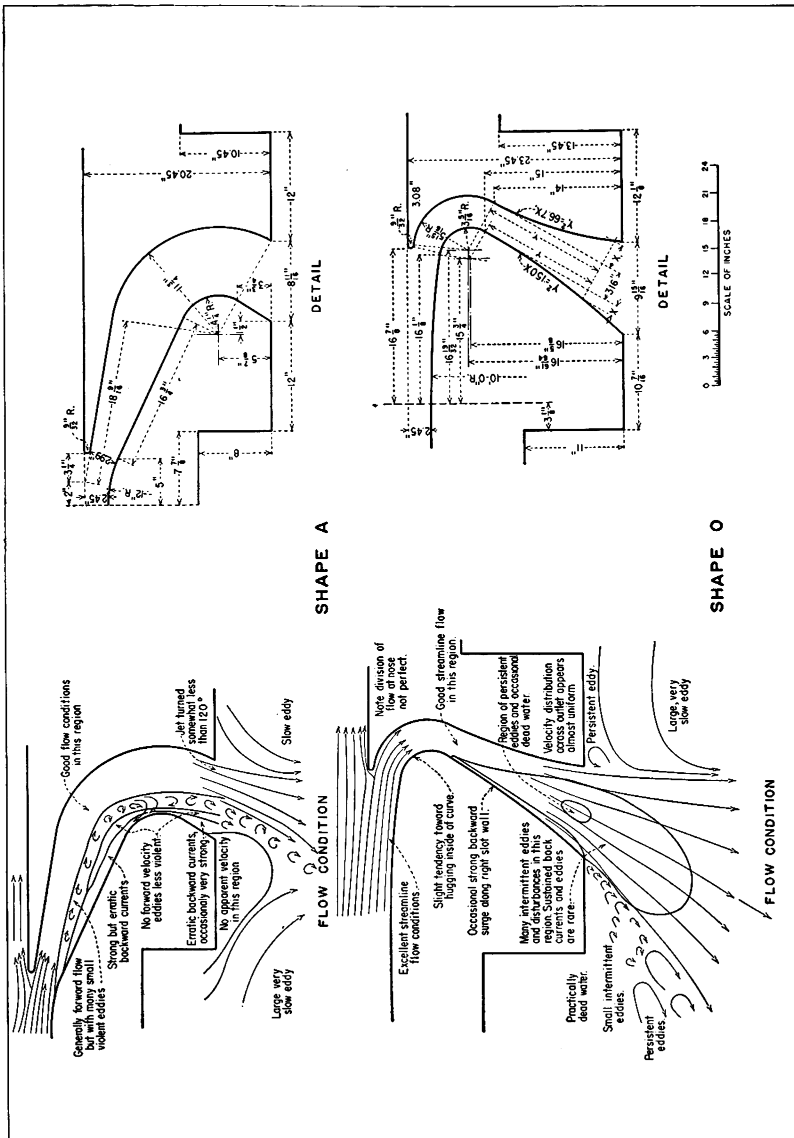


FIGURE 56.—DETAILS AND FLOW CONDITIONS THROUGH INFLUENT SLOTS OF IMPERIAL DAM DESILTING WORKS.

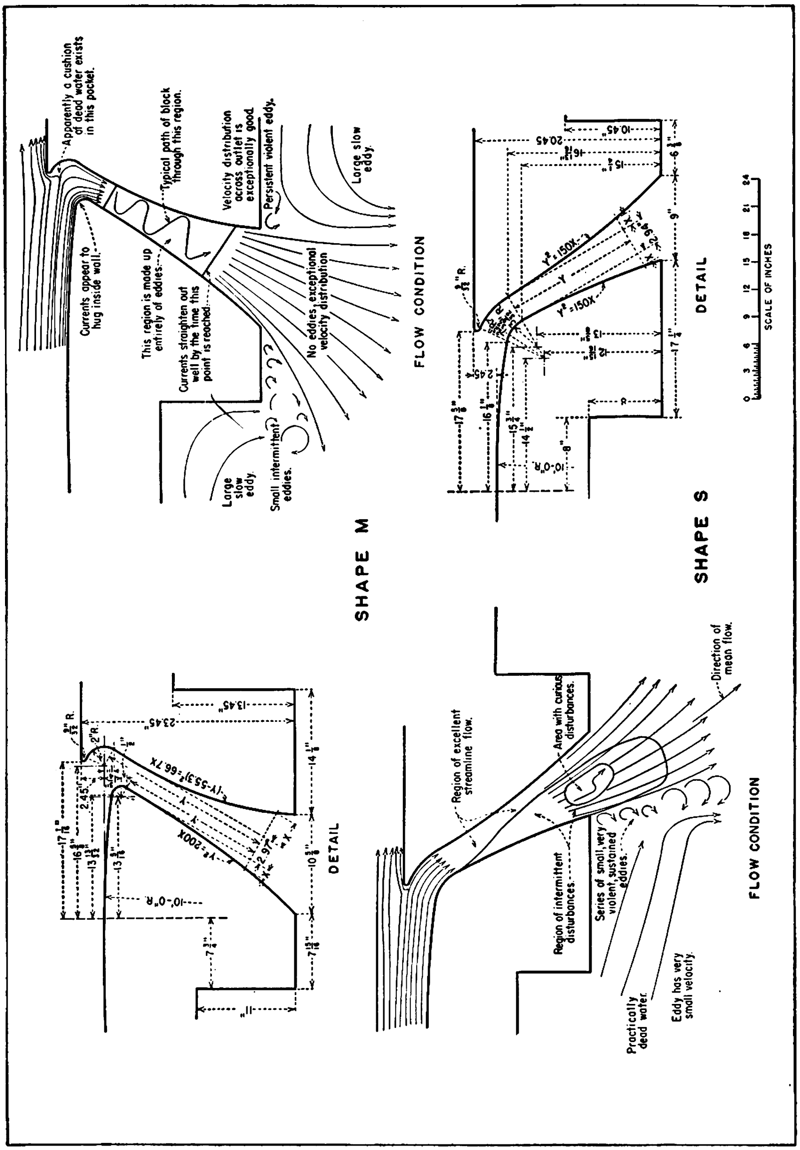


FIGURE 57.—DETAILS AND FLOW CONDITIONS THROUGH INFLUENT SLOTS OF IMPERIAL DAM DESILTING WORKS (Continued).

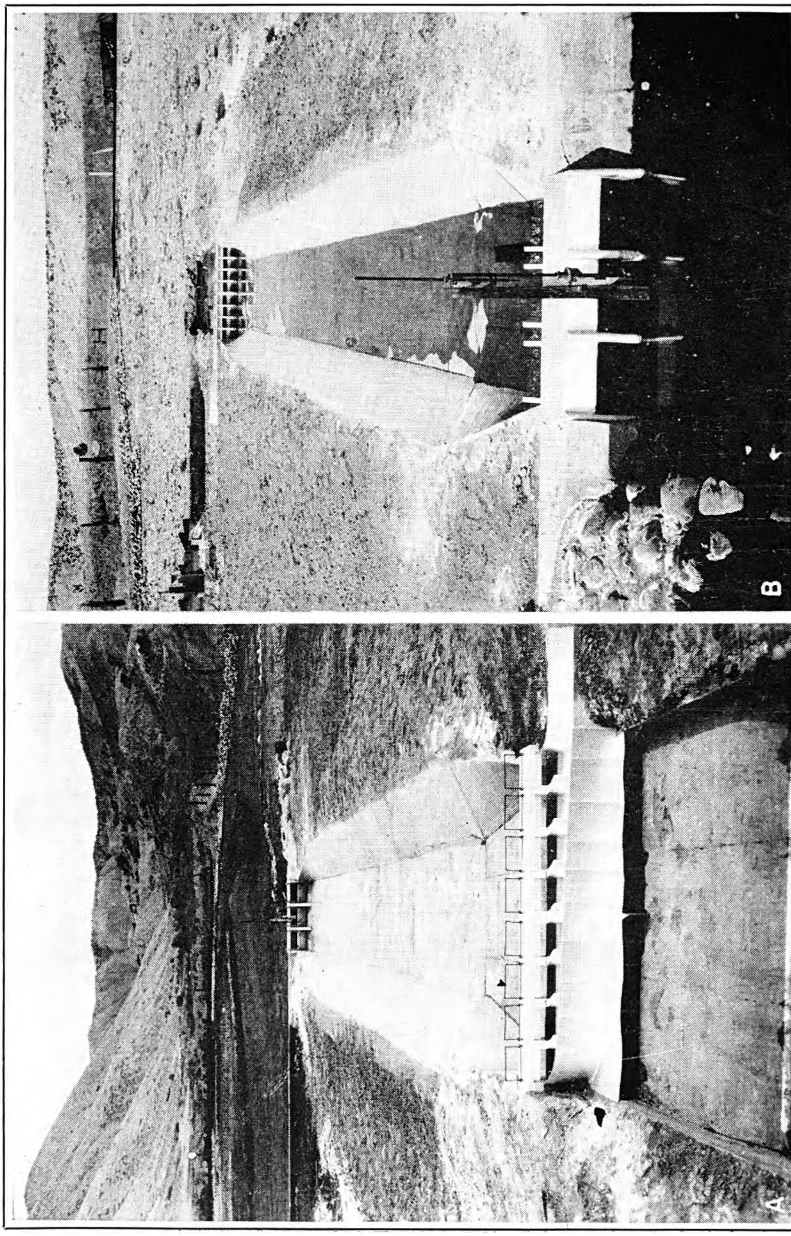
CHAPTER VII—STUDY OF THE GILA CANAL HEADWORKS

78. Problems of the Control Structures.—The headworks of the Gila Gravity Main Canal, at the east end of the Imperial Dam, were also designed with the help of tests on models.[5] The model tests were conducted at the laboratory in Montrose, Colo., on a scale of 1:40, and the model was a part of the large model of the entire dam which was used for the study of sediment problems. In the model tests, only the initial installation, that of one desilting basin, was considered.

It was not economical to provide a desilting works for the Gila Canal similar to that designed for the All-American Canal, since the head loss through the latter structure is relatively high and a high loss in head would be reflected in the cost of pumping the water from the Gila Canal to the point of distribution. Conversely, the desilting basin developed for the Gila Canal, as shown in figure 2, was not adaptable to the All-American Canal because the available crest length was insufficient to discharge the amount of water required.

The original design of the headworks for the Gila Canal consisted of an intake structure in the face of the dam, containing three radial gates through which the water would pass into the desilting basin. This basin was trapezoidal in section and had the following prototype dimensions: Bottom width, 103.5 feet; over-all length, approximately 1,165 feet; and depth, 26.4 feet. In this basin the designed velocity of the water was low and the sediment in the inflow water was expected to deposit. At the lower end of the basin was another structure containing two sets of eight gates each, one set above the other. The lower set connected to a channel returning to the Colorado River below the dam, and the upper set to a channel at right angles to the desilting basin, which led to the Gila Canal. According to the plan, most of the time the water would flow into the

[5] Thomas, C. W., H. D. Briley, and J. H. Douma, "Model Tests Gila Valley Canal Headworks and Ball Chech Valves for the Desilting Basin, Imperial Dam, All-American Canal Project," Hyd. Lab. Report No. 183 (unpublished), Bur. of Recl., Oct. 1, 1945.



A. Looking Upstream Into Desilting Basin B. Looking Downstream Into Desilting Basin.

FIGURE 58.—MODEL OF GILA CANAL, HEADWORKS AND DESILTING BASIN.

desilting basin where the sediment would be deposited, thence to the Gila Canal through the upper set of gates at the downstream end of the desilting basin. Periodically, the deposited sediment would be washed from the desilting basin into the river below the dam by closing the upper set of gates and opening the lower ones, allowing the water to pass through the basin at high velocity. The general appearance of the control works may be seen from the photographs of the model in figure 58.

In connection with these structures, laboratory studies were made of the following: (1) The method of sluicing the sediment from the settling basin, (2) the desirability and manner of streamlining the piers in the downstream control structure, (3) the shape of the channel and guide piers leading to the Gila Canal from the upper set of gates in the downstream control structure, and (4) the relative desirability of directing flow over or under this upper set of gates.

The first of these studies, that of the sluicing methods, was undertaken to determine how to wash the deposited sediment from the basin with the use of as little water as possible. The tests showed that by opening both the upstream and downstream gates the full amount a large flow of water could be produced through the basin, which would rapidly remove the deposited sediment. It was found, however, that the sediment could be washed out with a small flow of water by successively performing the following operations: (1) Closing the upstream gates and the upper set of downstream gates, (2) opening the lower set of downstream gates to drain the basin, and (3) opening the upstream gates slightly, producing a shallow stream of high-velocity water along the floor of the basin, which washed the deposited sediment through the lower downstream gates into the river.

The second study was to determine the advisability and method of streamlining the lower parts of the leading edge of the piers at the downstream control works. In the original design these piers were rounded on their upstream ends, but, under the conditions of flushing with a sheet of high-velocity water flowing along the floor from the upstream gates, the rounded edges were too abrupt, and undesirable waves were formed by impact of the stream on the rounded ends. This condition was materially improved by sharpening the upstream ends of these piers at the lower elevations where the high-velocity flushing water impinged upon them.

In the second study, an investigation was also made of streamlining the downstream edge of the center one of these piers, which was thicker than the others. A wedge-shaped streamlining which extended 20 feet downstream was studied, and also one 10 feet long. The 10-foot length was found to be sufficient and was recommended. However, a decision was later made to build, as part of this initial installation, the foundations and other portions of the walls that will ultimately support the flume carrying the water from the two unconstructed basins across the lower end of the

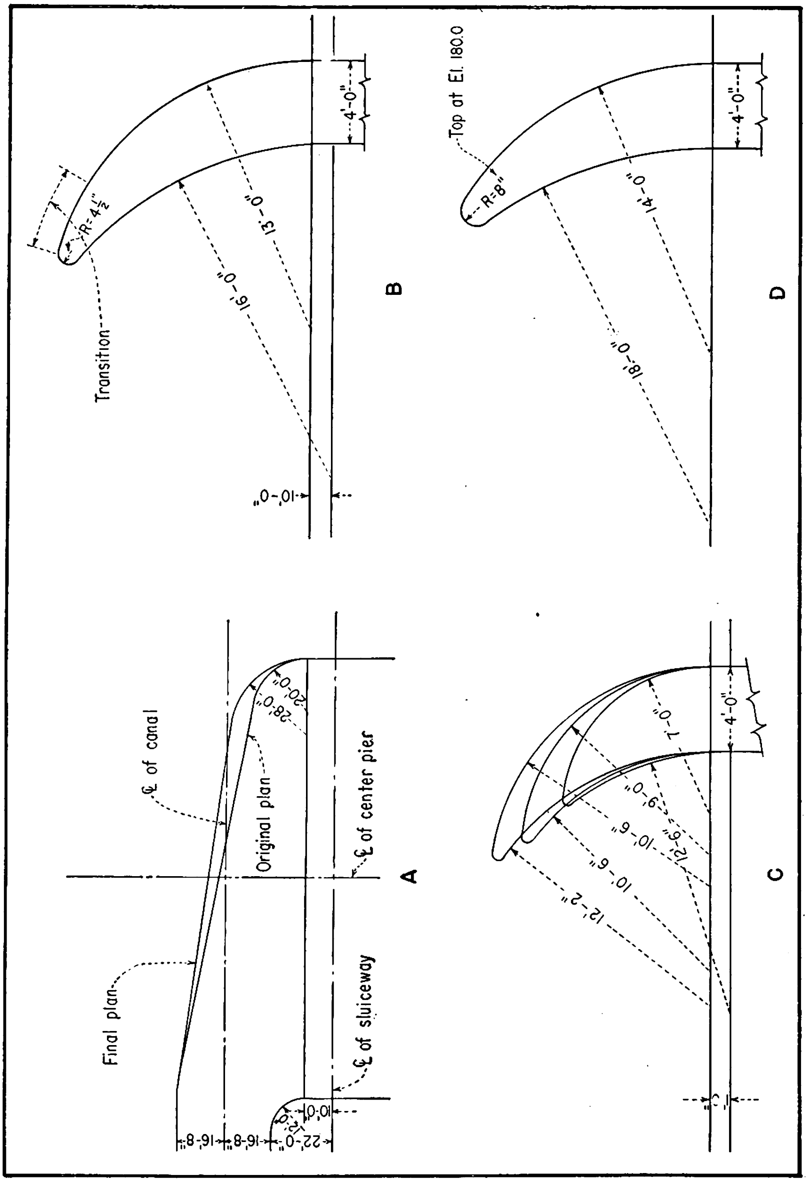


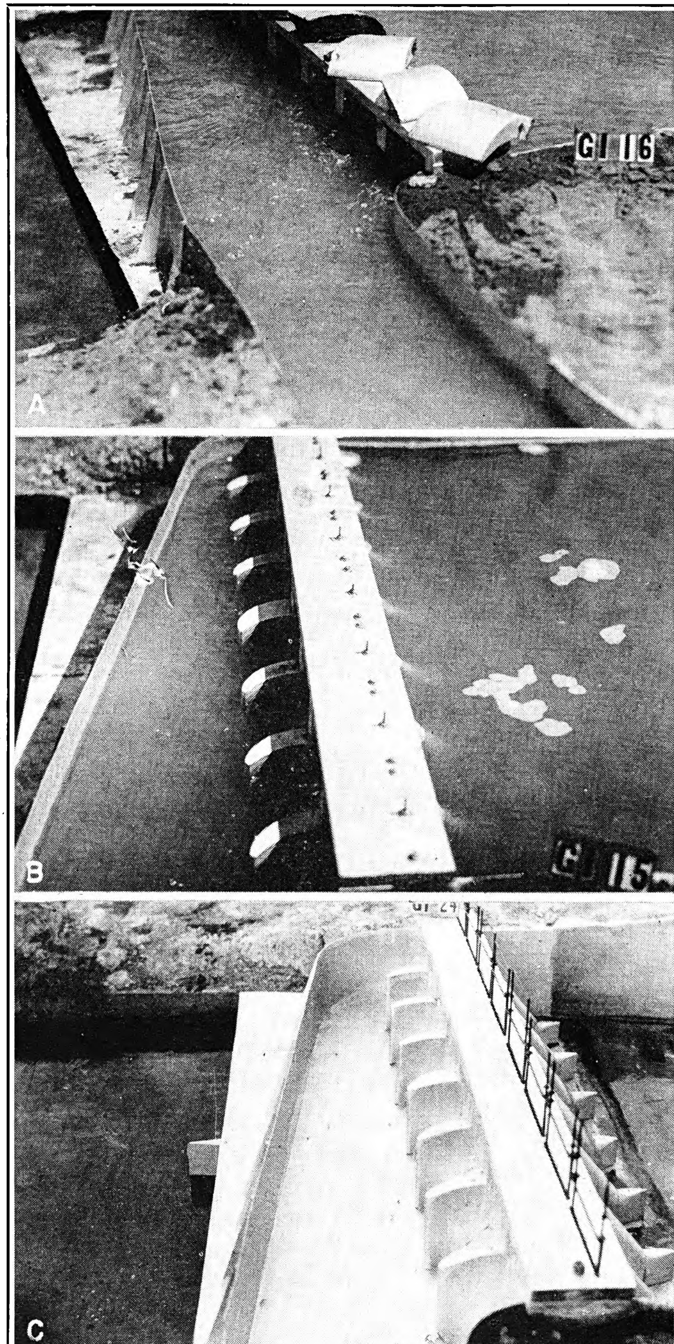
FIGURE 59.—CHANNEL AND GUIDE PIERS FOR ENTRANCE TO GILA GRAVITY MAIN CANAL.

initial one, which made it impractical to install the streamlining on the center pier.

The third model study undertaken was the development of an improved shape for the channel and guide piers leading from the upper gates of the downstream control works into the Gila Canal, see figure 58–A. The original lay-out of channel is shown in figure 59–A and the shape of the original guide piers in figure 59–B. Comparative tests with and without curved piers to help change the direction of the flow of the water showed the need of these devices, see figure 60. The tests with the original lay-out indicated that the distance between the wall and the ends of the pier tails at the upstream end of the channel was insufficient, resulting in turbulence and a high slope in the channel. An improvement of this situation seemed possible either by shortening the curved piers at the upper end of the channel or by widening the upper end of the channel. A trial was therefore made with the upstream three piers shortened various amounts as shown in figure 59–C, the shortening increasing as the channel became narrower. Although some improvement resulted, the flow in the channel was too turbulent, and an entire new set of piers of equal length, each as shown in figure 59–D, was installed. These piers resulted in still further improvement but the upstream portion of the channel remained crowded. The next change was to widen the upstream end of the channel by increasing the radius of the curve from 20 to 28 feet, as shown in figure 59–A, which made the narrowest part of the channel equal to the width of one gate. This change resulted in a considerable improvement in flow conditions and was incorporated in the final design; however, for other than hydraulic reasons, the distance from the center line of the sluiceway to the beginning of the curves at both ends of the sluiceway was increased from 10 feet 0 inch to 13 feet 0 inch. The model of the design as finally developed is shown in figure 60–C.

The conditions at the downstream control structure were such that the gates controlling the flow into the Gila Canal could have been designed to open either by being raised, with the water passing beneath, or lowered, with the water passing over them. In the fourth model study, both methods were tried, and it was found that the conditions of flow in the channel downstream were materially better if the gates were lowered, allowing the water to flow over them. Also, a minor improvement in flow conditions was made by making the height of the gate openings sufficient to allow the water to pass freely through them.

79. The Ball Check Valves.—In operating the Gila Canal desilting basin, great care must be exercised when draining it, to prevent withdrawing the water so fast as to endanger the floor or walls as a result of the pressure behind them. To relieve this pressure the walls and floor of much of the basin are pierced by numerous weep holes through which the water can drain. The downstream control structure, however, acts as a



A. Flow Condition Without Curved Piers. B. Flow Condition With Curved Piers. C. Final Design.

FIGURE 60.—MODEL OF GILA CANAL HEADWORKS.

dam, and weep holes in its upstream portion are therefore undesirable. The pressure beneath the floor and behind the walls in the upstream part of the control structure is therefore relieved through a series of drains that discharge through check valves. These valves relieve the pressure on the walls and floor when the basin is drained, but prevent the flow of water from the basin beneath the floor and behind the walls when the basin is filled. The development of a suitable check valve for this purpose was conducted in the hydraulic laboratory.

The first design of valve tested is shown in figure 61–A. It consisted of a cast-iron casing enclosing a 6½-inch diameter ball which seated on a hard-rubber seat. The ball was prevented from being lifted out of the valve by six cast-iron vanes attached to the casing walls. A hard-rubber ball was first investigated, but was found to be unsatisfactory since it developed a permanent deformation along the line of contact with the seat, which would probably result in leakage since it would not seat in the same place each time. A chromium-plated, hollow, brass ball, with a specific gravity of 1.25, was tested and found satisfactory.

Tests of the original design developed an intensive "hammering" of the ball against the vanes for discharges below 0.75 second-feet; for higher discharges the ball was held in contact with the vanes. Hard-rubber strips or vanes eliminated the noise of the hammering but not the hammering itself. An analysis of the cause of this action revealed that it would occur in any design in which the cross-sectional area downstream from the ball was greater than that upstream, as in the original design. With a constant flow, according to the Bernoulli theorem, the pressure downstream from the ball, where the area is greater, would be more than upstream, where the area is smaller. This difference in pressure on the two sides of the ball would cause it to move upstream. In the tests, with the valve set vertically and the flow in an upward direction, the force of gravity also acted upstream. Thus, both the pressure difference and the force of gravity tended to move the ball toward its seat, tending to cut off the flow of water. As the flow of water decreased, the difference in pressure on the two sides due to the expanding section also decreased, and the pressure beneath the ball tended to raise it upward again. This unbalanced condition caused the ball to move alternately up and down, producing the hammering on the guides as previously mentioned.

To remedy this situation the shape of the valve was changed to that shown in figure 61–B. In this form there is a decrease in cross-sectional area of the housing downstream from the ball, which causes a lowering of pressure and thus a decrease in the force tending to move the ball upstream. This change eliminated the hammering. At low flows the motion occurred in the revised valve with the downward force on the ball supplied by gravity only, but this caused no undesirable effects.

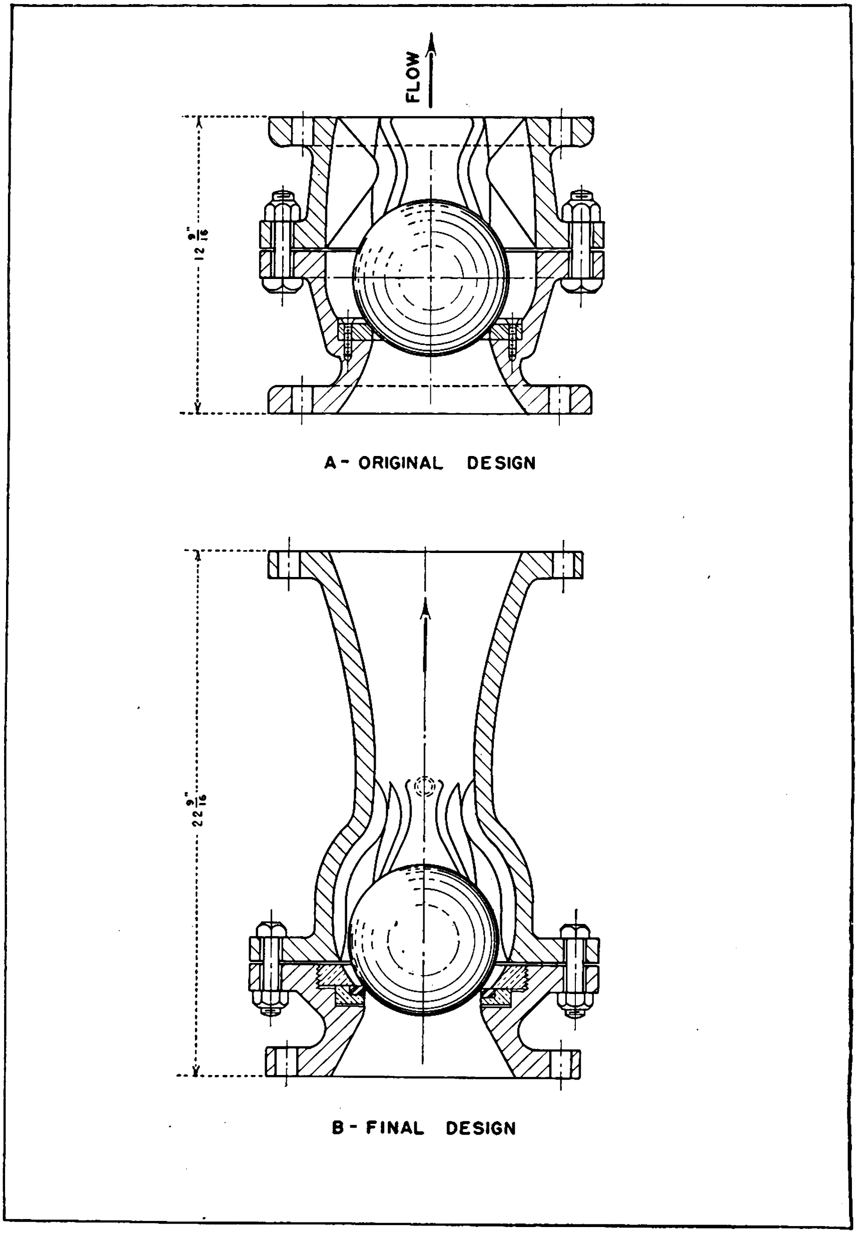


FIGURE 61.—BALL CHECK FOR GILA CANAL DESILTING BASIN.

CHAPTER VIII—CANAL STRUCTURES

INTRODUCTION

80.—Types of Canal Structures Investigated.—Because of the large discharge of the canals taking off from the Imperial Dam, the structures required to control the flow along the canals and to distribute the water were of large size and therefore of considerable cost. The expense which would be incurred in operating the system, should the structure be improperly designed, was believed so great as to justify extensive hydraulic model studies to insure adequate designs. It was necessary that a great variety of structures be tested, as the connection with existing canals and the development of power introduced somewhat unusual conditions. Several chutes or drops were investigated which were of relatively large height, considering the discharges, and which in some cases involved unusual tailwater conditions. Several unusual river crossings were studied; and tests were made on drops bypassing power plants to discharge the flow when the plants are shut down or before they are put into service. Also investigated were several structures designed to carry storm drainage over the canal line. In the following sections the investigations will be described in detail.

Although it has not been found entirely practicable in analyzing these studies to present all the problems of one kind in a single group, this arrangement is followed insofar as feasible. The structures involving chutes are treated first, followed by river crossings, power plant drops, flood overpasses, and automatic gates, in the order named.

CHUTES, WASTEWAYS, AND RIVER CROSSINGS

81. Siphon Drop Turn-Out and Chute.—One of the important canal structures studied by models is the Siphon Drop turn-out and chute.[6] As previously stated, the original Yuma Project Main Canal was replaced in

[6] Douma, J. H., "Model Study of Siphon Drop Turn-Out, All-American Canal System," Memo. to J. E. Warnock (unpublished), Bur. of Recl., July 20, 1939.

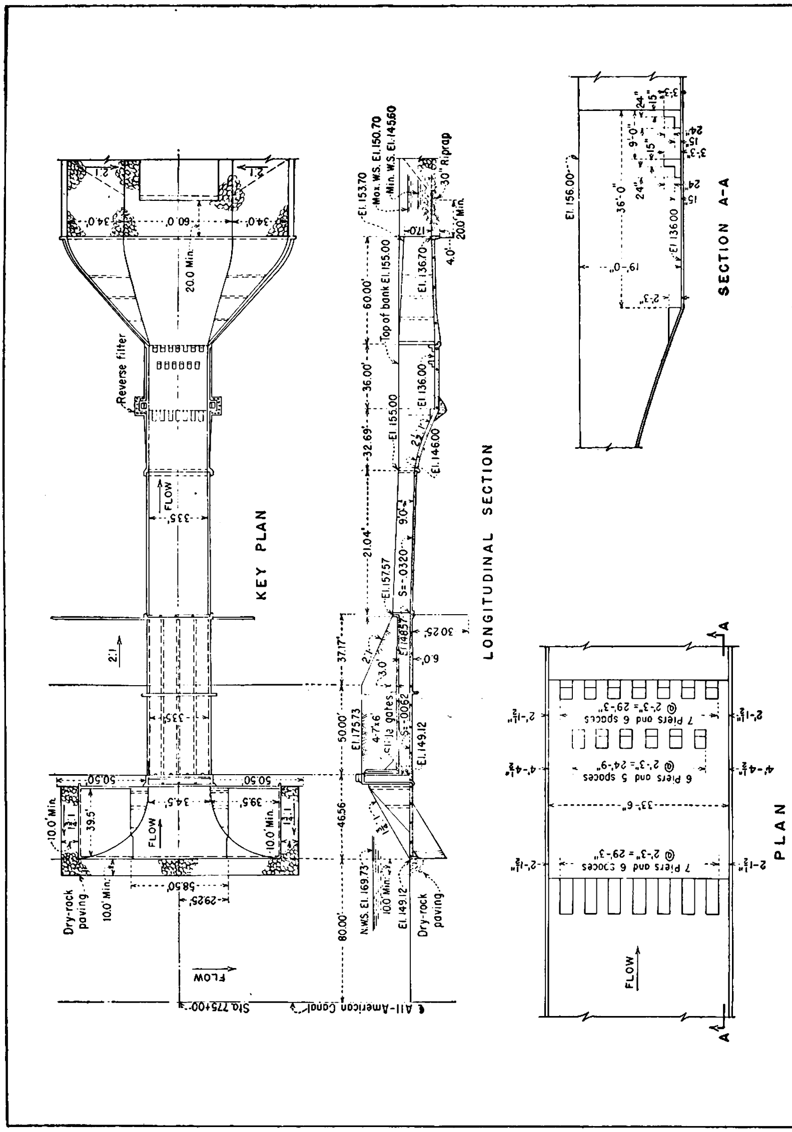


FIGURE 62.—SIPHON DROP TURN-OUT SHOWING FINAL DESIGN OF STILLING BASIN.

its upper 15 miles by the All-American Canal. The Siphon Drop turn-out and chute, located at the end of this 15-mile section, is for the purpose of supplying a discharge of 2,000 second-feet, maximum, to the Yuma Project Main Canal from the All-American Canal. Since the All-American Canal is at an elevation approximately 20 feet higher than the Yuma Canal, a chute and stilling pool are required. The installation, see figure 62, consists of a control structure containing four sluice gates, each discharging into a culvert leading under the canal bank, downstream from which is a single open flume leading to a chute and stilling pool.

The original design provided for an abrupt entrance into the turn-out, but the model tests showed that this would produce undesirable flow conditions in the water entering the structure. Accordingly, the warped surfaces shown in figure 62 were developed, which produced smooth entrance flow conditions under all discharges. The flow conditions in the culvert and chute were found to be satisfactory under all conditions of discharge. The gates when fully open were designed to discharge the maximum flow of 2,000 second-feet with a low water level in the All-American Canal; at all higher canal levels the flow would be controlled by the gates. The total discharge wirh the four gates fully open and the equal gate openings required to discharge 2,000 second-feet are shown in figure 63 for various water levels.

In the original design the floor of the stilling pool was placed at elevation 135.0, which was the elevation required to form the hydraulic jump with the maximum discharge. No baffles were included, but the model tests showed that by the use of the baffles shown in figure 62 the bottom could be raised to elevation 136.0, with a considerable saving. The blocks also reduced the wave action in the channel downstream from the stilling basin. The best conditions of flow were obtained with all four gates opened equally. With unequal gate openings the operation was unsymmetrical, but, because of the cushioning effect of the water in the pool, no damage resulted at ordinary discharges. With the entire maximum discharge of 2,000 second-feet passing through two adjacent gates, however, the stilling-basin action became very unstable, and there was a tendency for the water to sweep out of the basin. This method of operation should therefore never be used on the prototype.

82. The Pilot Knob Wasteway.—At Pilot Knob, 5 miles west of Yuma, Ariz., a check has been placed in the All-American Canal, and a wasteway [7] has been installed to discharge the flow of the canal in an emergency, see figure 64. The structure will also serve to discharge the flow which will later be used by a power development at this point. The waste water empties into the upper end of the Alamo Canal, from which it can be directed either down the canal for irrigation or returned to the Colorado

[7] Ball, J. W., "Hydraulic Model Studies for the Design of the Pilot Knob Wasteway, All-American Canal, Boulder Canyon Project," Hyd. Lab. Report No. 184 (unpublished), Bur. of Recl., Oct. 12, 1945.

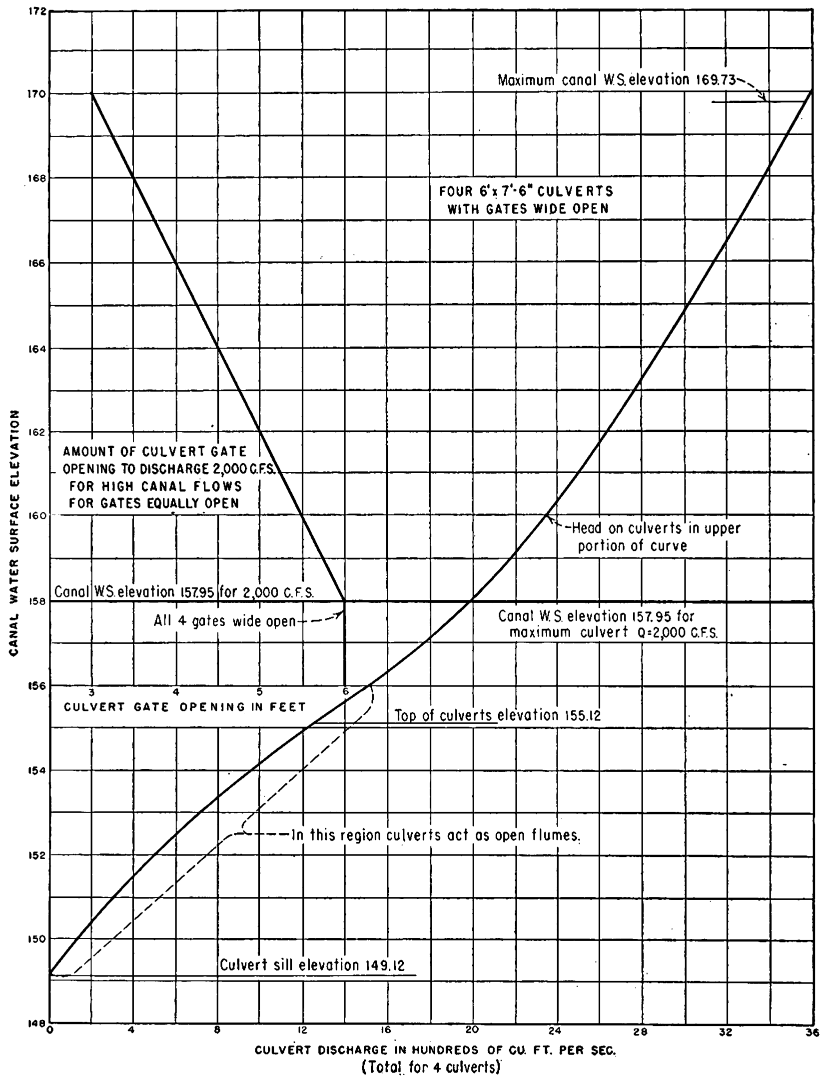


FIGURE 63.—DISCHARGE RATING CURVES FOR SIPHON DROP TURN-OUT—ALL AMERICAN CANAL.

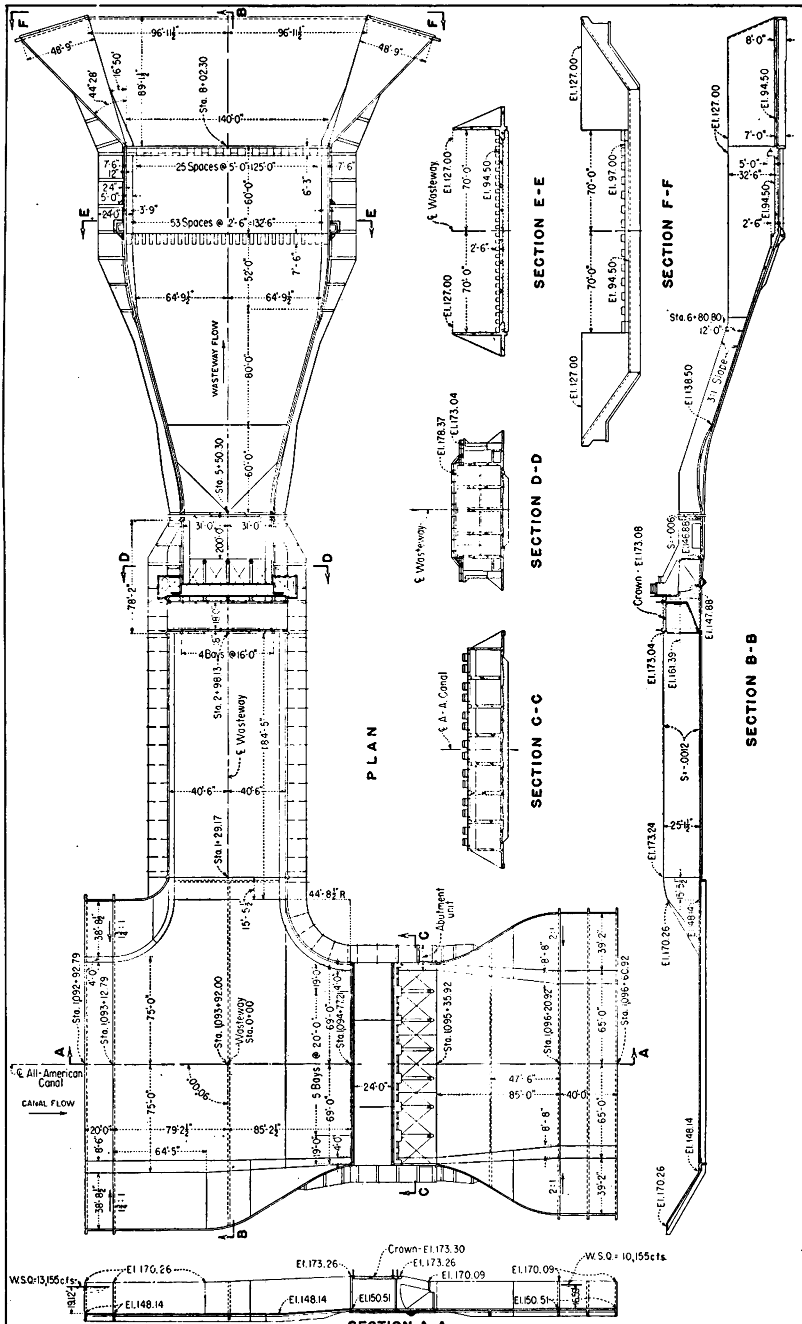


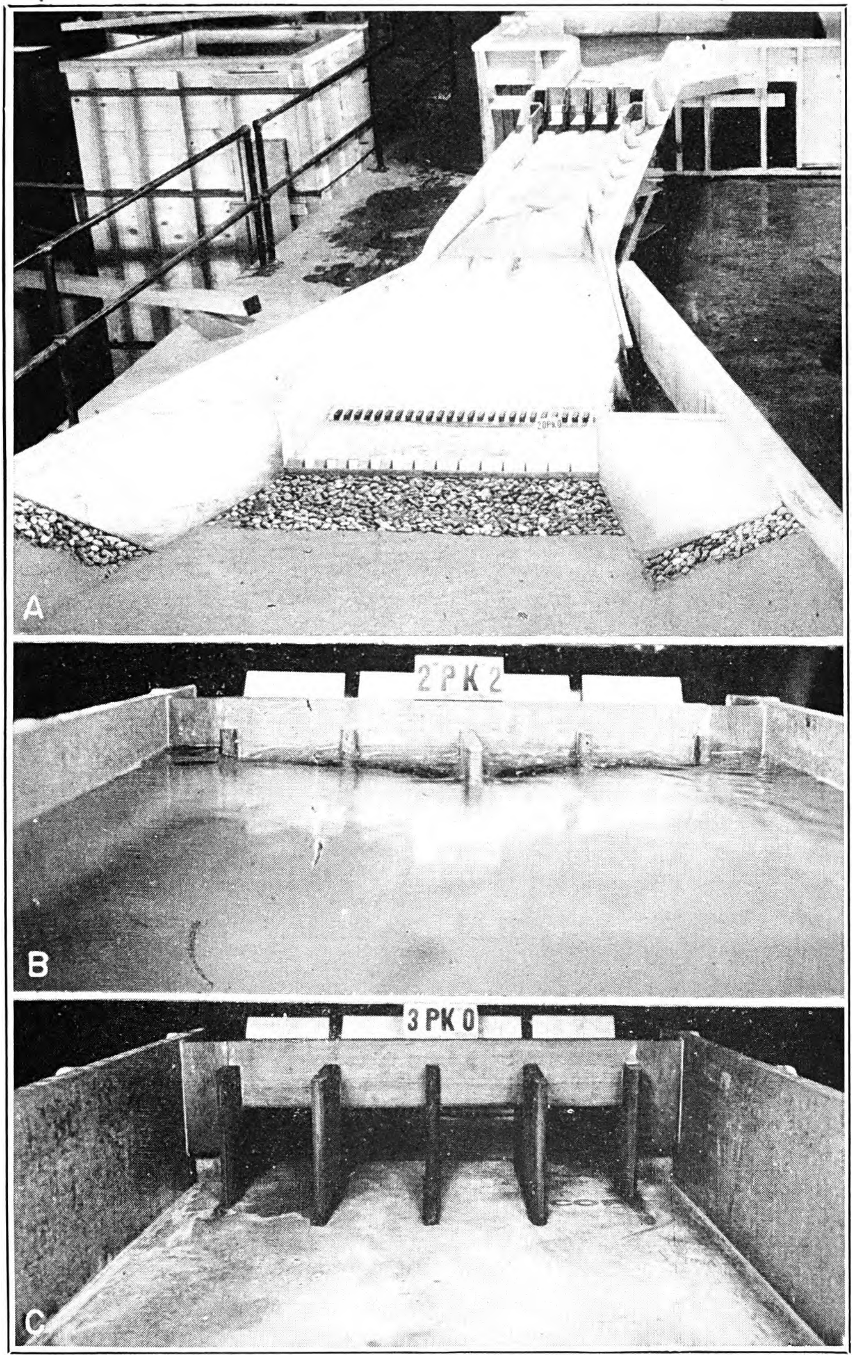
FIGURE 64.—FINAL DESIGN OF PILOT KNOB WASTEWAY.

River. The installation consists of a wasteway channel leading to a control structure with four large gates and two small bypass sluice gates, below which is a concrete flume leading under the Southern Pacific Railway to the head of an expanding chute discharging into a stilling pool. The discharge of this structure may reach 13,155 second-feet, the capacity of the All-American Canal at this point, and the normal drop is 57 feet. The problems of this structure were studied by means of a 1:36 model of the principal installation, see figure 65, and a 1:12 scale model of the bypass sluiceways.

As in the Siphon Drop turn-out, the streamlining of the entrance to the Pilot Knob wasteway was a problem for study, and four designs of transition shapes for the upstream junction of the canal and wasteway bank were investigated. The best design was found to be a segment of an inclined elliptical cylinder, but this was objectionable because of the difficulty of construction and was abandoned in favor of a simpler but less effective design. More turbulence existed at maximum discharge in the simpler design, but since this discharge would occur only at rare intervals it was not believed to be important.

83. Tests on the Control Gate Structure of Pilot Knob Wasteway.—As will be seen in figure 64, the channel approaching the control structure of Pilot Knob wasteway is 81 feet wide and that leading away from the structure is 62 feet wide. As stated above, the upstream channel leads to six gates, the outside two being smaller than the other four and discharging into sluiceways which turn to enter the downstream channel from the sides. In the original design, the dividing walls between the gates extended upstream beyond the face of the control structure and were rounded on the upstream edge. The walls between the smaller and larger gates projected very little, as shown in figure 65–B. The center wall extended far enough upstream to act as a pier for a bridge over the wasteway, and the remaining two walls at the top extended out a distance equal to that of the outer pair, but at the bottom had an extension inclined upward in a downstream direction. The flow conditions for maximum discharge with this design were unsatisfactory, and an attempt was made to improve the design by extending the two outside piers, in addition to the center pier, to support the bridge. The semicircular pier noses were also replaced by sharper ones. This design produced satisfactory flow conditions but was not used, since an equally satisfactory design, made by shortening the center pier and extending the second and fourth piers to support the bridge structure, provided a more economical design of the bridge. In the final design, see figure 65–C, the two outside piers were also somewhat extended.

In the original design, the bypass sluices discharged into the channel downstream from the control structure at an angle of 45°. When these sluices were operated simultaneously, the outflow streams met at an abrupt angle in the center of the channel, with considerable turbulence. This



A. Recommended Design. B. Entrance to Wasteway Showing Short Piers. C. Final Design Piers With Upstream Extended.

FIGURE 65.—MODEL OF PILOT KNOB WASTEWAY.

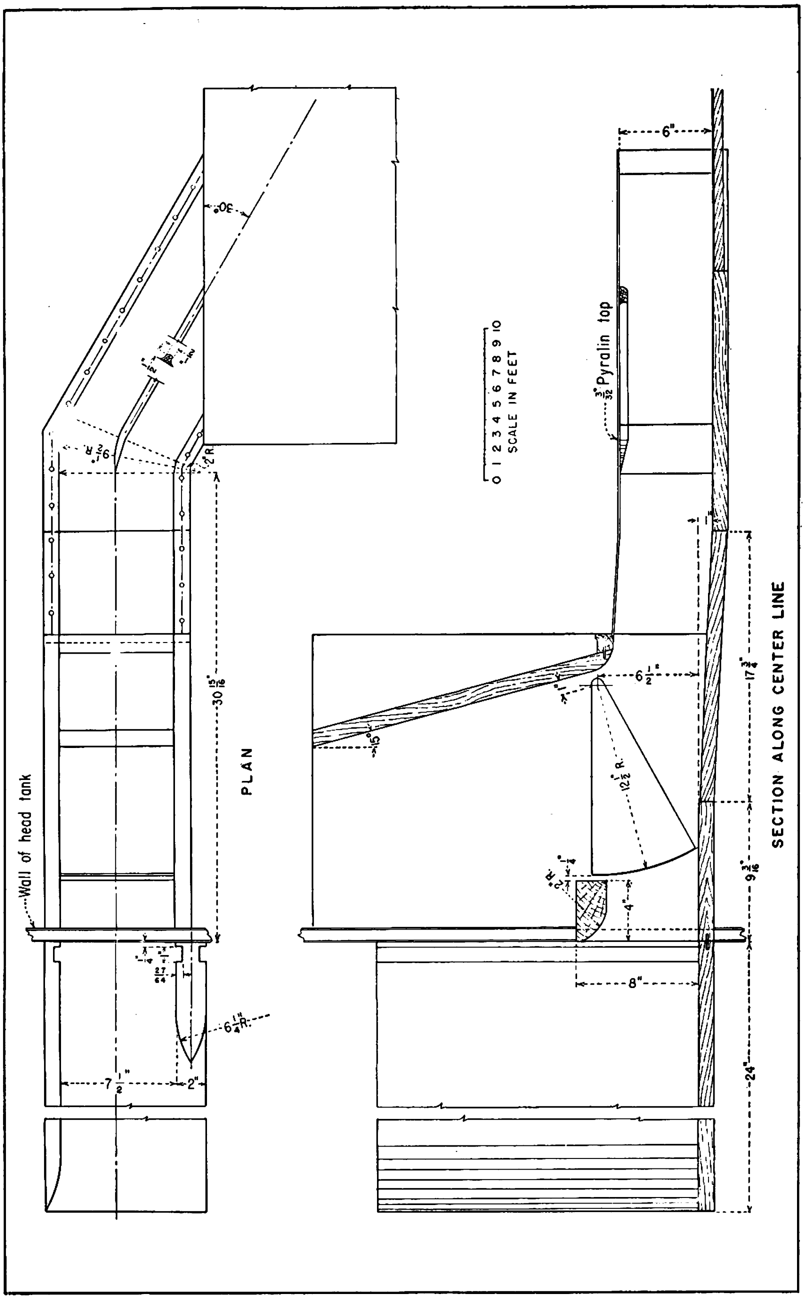


FIGURE 66.—MODEL OF PILOT KNOB BYPASS SLUICEWAY.

condition was considerably improved by changing the angle from 45° to 30°, which was the smallest angle considered practicable. Because of the spray from these outlets at partial gate openings, and since pulsating flow occurred in the sluices immediately downstream from the gates when they were completely raised, a 1:12 model, see figure 66, was constructed to investigate these conditions. In this model the top of the sluice was made of transparent pyralin to permit observation of the flow. The spray was seen to be produced by the spiral motion set up by the 30° angle turn in the direction of flow, which caused part of the water to move upward to the roof of the conduit and flow across the top to the outlet, as shown in figure 67–A. This spray was greatly reduced by placing a protruding beam on the roof of the conduit. The improvement which resulted may be seen by comparing figures 67–A and 67–B.

84. Tests on Chute and Stilling Pool of Pilot Knob Wasteway.—The conditions encountered at the Pilot Knob wasteway site differed from those at Siphon Drop and resulted in a somewhat different type of chute and stilling-basin design. At Siphon Drop the stilling basin was the same width as the channel leading to it. This required a stilling basin of considerable depth, to cause the hydraulic jump to form in the basin at all discharges. At the Pilot Knob site, however, the existing geological conditions made it very undesirable to have such a deep pool, and a design was developed to spread the flowing water coming down the channel into a wider and shallower stilling pool. As shown in figure 64, the chute leading to the stilling pool consists of two parts having different slopes joined by a vertical curve. The upper part has a slope of 0.006 and a constant width, and the lower part has a 3:1 slope and symmetrically flared walls. The flow in the upstream section of the chute was satisfactory at all discharges; but, with a floor which was level at right angles to the center line, the flow in the downstream section would not spread rapidly enough to follow the flared walls, and the water concentrated in the center of the chute as it entered the stilling pool, producing undesirable conditions. An effort was therefore made to cause the flowing stream to spread evenly over the entire width of the pool. This might have been accomplished by starting the flare in the channel just downstream from the gate structure, but that would have added considerably to the cost. Attempts were therefore made to cause the stream to spread by changing the shape of the bottom of the flume, and nine different designs of spreader were investigated, see figure 68. It was found that the spreaders had more effect where the velocity was lower, and they were therefore placed as far upstream as possible without reducing the discharge, preventing the draining of the main canal, or producing undesirable waves at low discharges. Design 7 proved to be the most desirable of all those tested. These spreaders were also effective in distributing the uneven flow present when several combinations of gates were opened.

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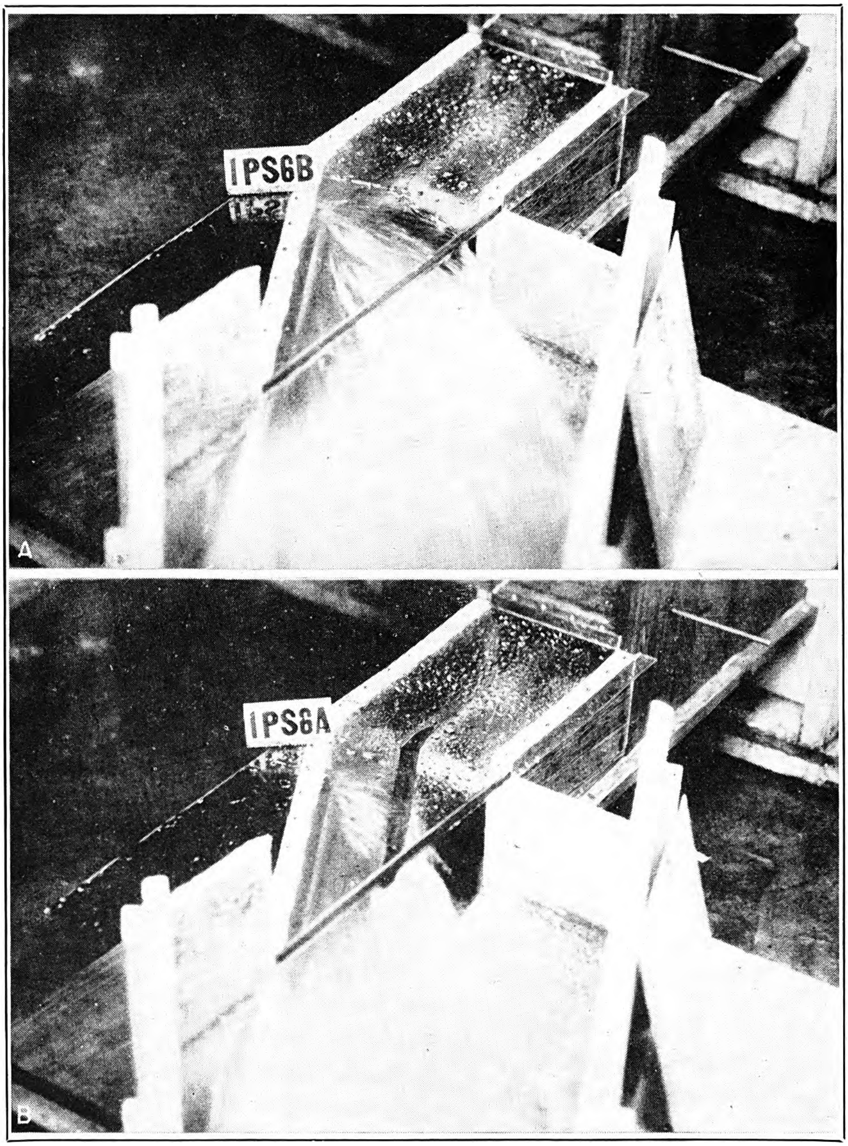

A. Spray in Bypass Sluice. B. Effect of Beam on Roof of Sluice.

FIGURE 67.—FLOW IN PILOT KNOB BYPASS SLUICEWAY.

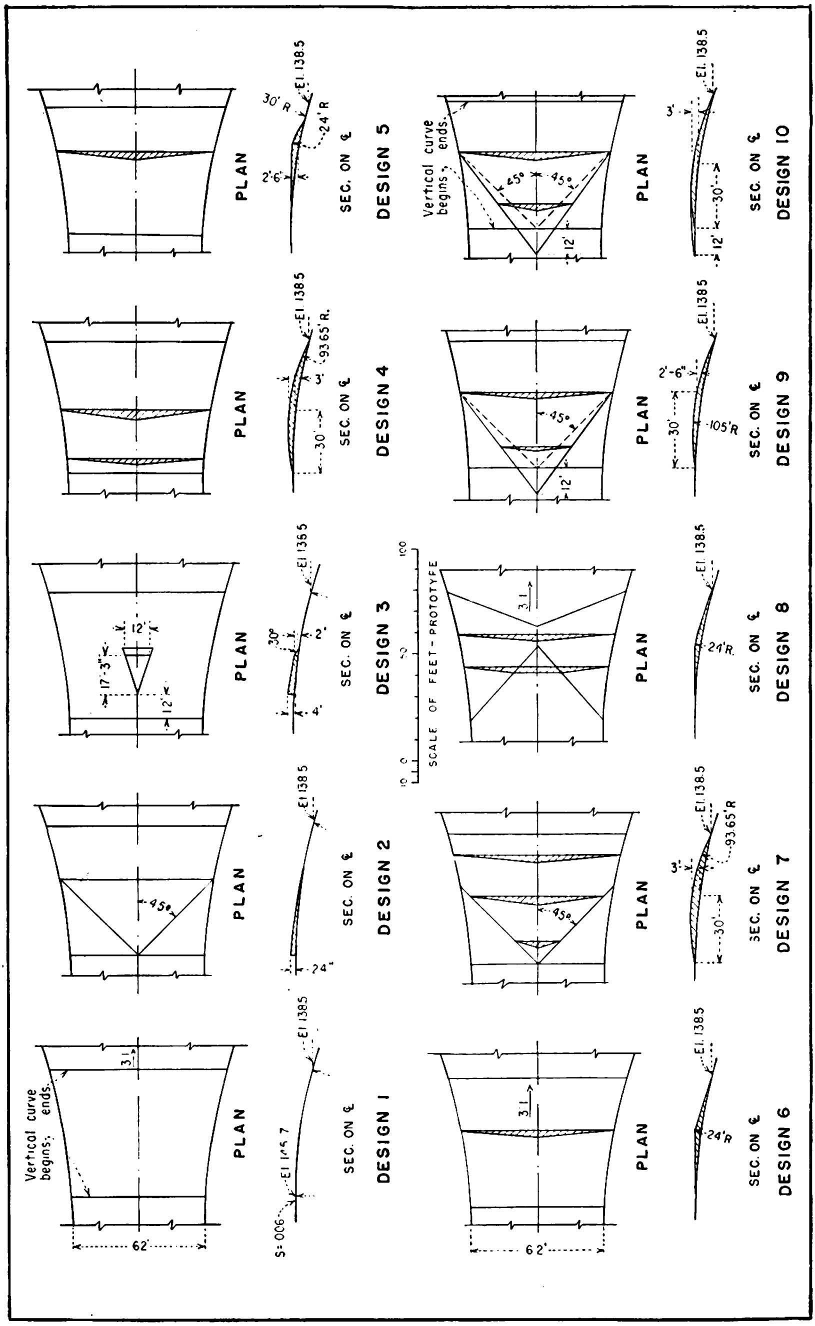

FIGURE 68.—DIFFUSER FOR SPREADING FLOW INTO STILLING POOL OF PILOT KNOB WASTEWAY.

Because of the possibility that the waste gates at Pilot Knob would have to be opened quickly, before the channel downstream from the stilling pool had time to fill, a temporary condition of low tailwater and high discharge might exist. Although it was estimated that the channel would fill to normal height in 15 minutes, the stilling pool would have to be safe from erosion during this interval. With the original design, under the low tailwater condition the high-velocity stream passed entirely through the stilling basin and eroded the channel. It was not considered essential that under this condition the entire jump be in the stilling basin, but it was required that its upstream end be in the pool to permit the jump to move upstream into the basin as the tailwater rises. A number of arrangements of steps, sills, and floor and riprap lengths were tested to arrive at the most economical combination to meet the requirements. It was found that the steps in the sloping apron were very effective in reducing the length of basin required. The most economical combination was the stepped apron and dentated sills shown in figure 64, with riprap on the bottom extending downstream approximately 60 feet from the edge of the concrete floor.

85. Fortuna Wasteway.—The Fortuna wasteway is located on the Gila Gravity Main Canal about 10 miles east of Yuma, Ariz. Its purpose is to provide a method of wasting the water coming down this canal if the pumping plant to which the canal leads should suddenly cease operating. If no outlet were provided, this blocked flow would fill the canal near the pumping plant to the top and overflow the canal banks. The wasteway discharges into a dry channel leading to the Gila River 6,300 feet away, and it is estimated that the discharge from the gates might increase from 0 to 3,000 second-feet in 3 minutes. The wasteway consists of a gate structure containing two automatic gates, which discharge into culverts leading under the canal bank to an expanding section with vertical walls, thence to a warped expanding section, as shown in figure 69. To investigate the action of this structure a model on a 1:30 scale was constructed, at first without baffles or cross walls. When the gate was opened rapidly with the discharge channel empty, simulating the conditions which might occur on the prototype, the high-velocity stream flowed swiftly across the floor and eroded the channel. Several methods of preventing this action were investigated, of which two were successful. One was the placing of two rows of closely spaced, staggered, baffle piers near the end of the expanding section to break up the high-velocity stream and reduce the scour caused by it. The second method, which was more successful, consisted of placing a barrier across the structure, 30 feet from the lower end. This barrier acted somewhat as a dam, and when the gates were opened, caused a pool to form with sufficient depth to develop the hydraulic jump. At the bottom of the barrier were placed four equally spaced holes, 2 feet high and 4 feet wide, through which the water could drain when there was no discharge and which would also reduce the head of water acting on the dam.

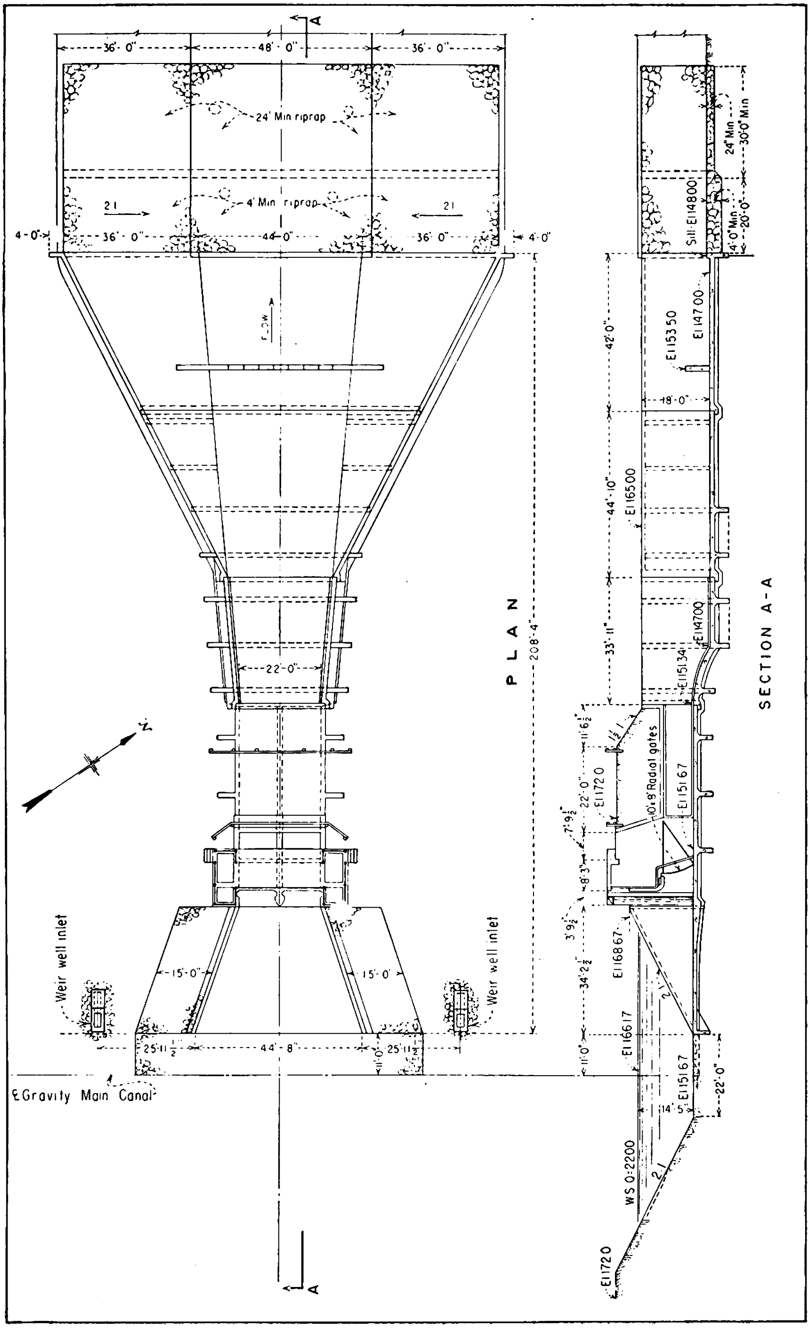

FIGURE 69.—FORTUNA WASTEWAY.

A 1-foot-square sill across the downstream end of the structure formed a secondary stilling pool below the barrier. This design worked very well under all reasonably probable conditions. The barrier not only caused a stilling pool to form but also distributed the flow across the width of the channel. Under the worst possible conditions, which would occur if one gate remained closed and the other open, there would be a heavy concentration of flow over the barrier and down one side of the structure. This might cause some scour of the banks and bed, but it is believed the structure would not be endangered.

86. The New River Crossing.—About 2 miles west of Calexico, Calif., the main line of the All-American Canal, with a capacity of 2,700 second-feet, crosses New River in two 15.5-foot diameter steel pipes, which dip below the grade of the canal in the form of an inverted siphon.[8] The pipes do not pass beneath the river bed, however, since the frequency of earthquakes in this section makes it desirable that they be exposed to permit vibration and facilitate repairs. They are therefore carried across the river on piers, which have sufficient height to provide the waterway area required to carry the maximum expected flow of New River. In the inlet structure at the upstream end of the pipes, two radial gates are installed to control the discharge through the siphon. To provide a means of discharging the water coming down the main canal, should it become necessary to close the inlet gates, a wasteway is constructed, with four gates which have a combined capacity of 3,300 second-feet. These gates, one of which is automatic, prevent the water level at the siphon intake from exceeding any desired elevation, by discharging the surplus into the wasteway. To secure the best possible design of these structures, hydraulic model tests were conducted. It was believed that the outlet end of the siphon did not involve sufficiently difficult problems to justify investigation, and the model therefore included only the upstream end of the siphon and the wasteway, as shown in figure 70.

The hydraulic model was constructed to a scale of 1:18.6, at the laboratory in the old customhouse. Because of the limitations of space and pump capacity, the model was smaller than required to keep the surface roughness in the proper proportion; however, a correction was made by steepening the slope of the chute the required amount. Laboratory conditions necessitated constructing the wasteway on the left side of the siphon instead of the right side as in the prototype, but obviously this would cause no difference in the hydraulic action.

The model studies showed that the original design of the inverted siphon and its control works was satisfactory, but that improvements were needed in the design of the wasteway. The canal upstream from the structure is lined and the velocity of the water approaching the wasteway therefore

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[8] Cline, F. R., "Preliminary Report on Model Studies of New River Crossing and Wasteway, All-American Canal," Hyd. Lab. Report No. 27 (unpublished), Bur. of Recl., Sept. 23, 1937.

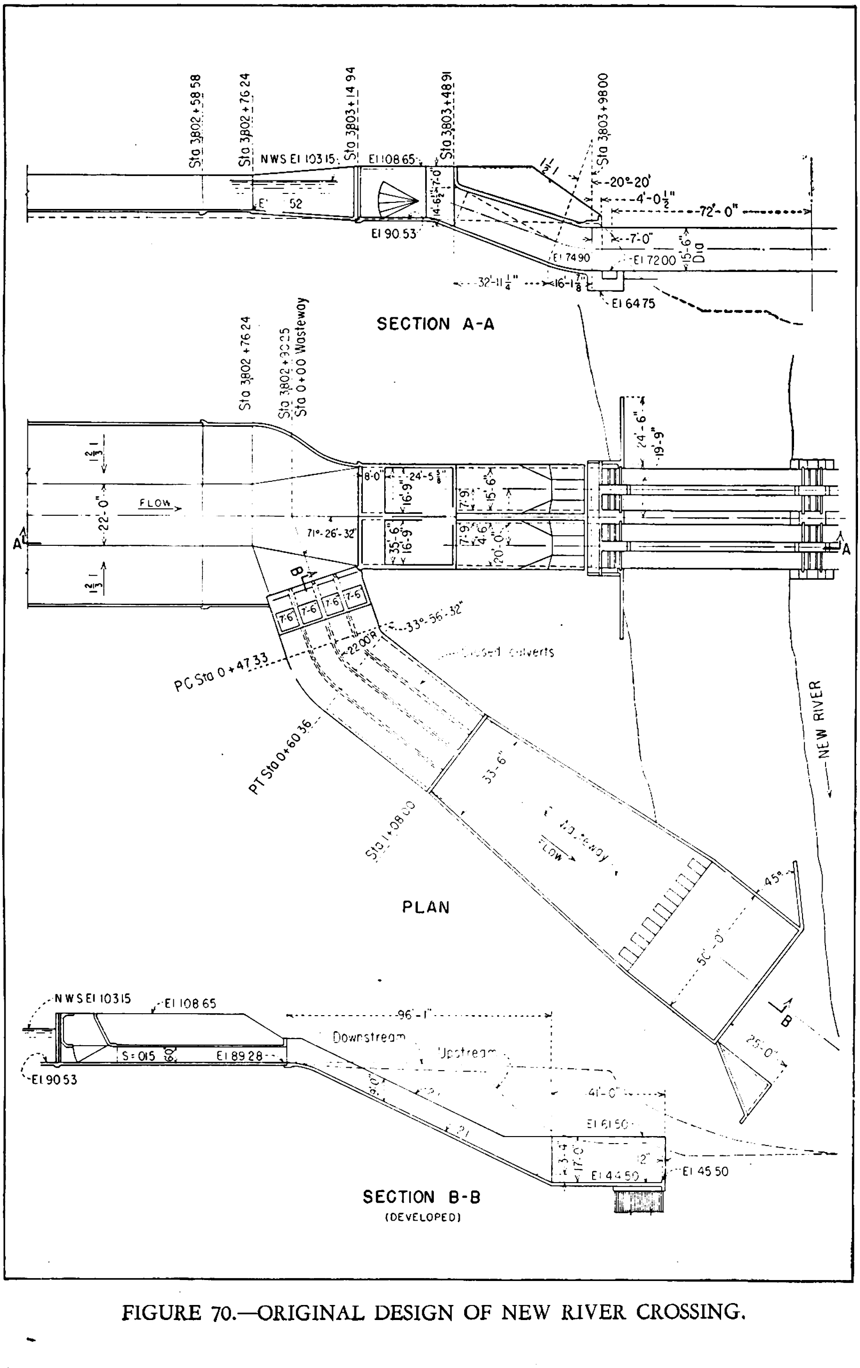

FIGURE 70.—ORIGINAL DESIGN OF NEW RIVER CROSSING.

relatively high, and to enter the closed culverts of the wasteway necessitates an abrupt change in the direction of this rapidly moving stream, see figure 70. The entrance curve of the original design was too abrupt, and the water could not change direction rapidly enough to follow it. A warped surface was substituted for the original entrance curve but resulted in only minor improvement. It was evident that a major improvement could not be obtained by any practicable shape of transition without changing the position of the gate structure. Another difficulty, also caused by abrupt change in flow direction, arose in the curved culverts just downstream from the gates. Under conditions of maximum discharge, the high-velocity streams from beneath the gates rose on the outside of the curve sufficiently to cover about two-thirds of the top of the culvert. The resulting high pressures in the conduit would have necessitated a design adequate to take care of the unusual forces produced. The difficulties due to direction of flow in both the gate approach and the culverts downstream were eliminated by changing the direction of the wasteway control structure to be in line with the chute and thus more nearly in line with the direction of flow in the canal, as shown in figure 71. Several designs of the warped surface connecting the canal to the wasteway gate were investigated, and the one giving the best results, considering the cost involved,. was chosen. With this alinement of gate structure and chute, the flow into all the gates was equal and without undesirable turbulence.

The original design of the chute proved to be satisfactory and was incorporated in the final design of the structure. The vertical curve at the junction with the culvert appeared to fit the trajectory of the stream, and the spread of the walls was such as to cause an equal depth across the flume. The ends of the culvert walls caused only minor disturbances. The elevation of the stilling-pool floor, however, proved to be too high, and it was lowered 2 feet. Additional lowering might have been better hydraulically, but would have been very expensive on account of groundwater conditions existing at the site. With the depth adopted, a 10-percent decrease in the length of the pool over that in the original design was possible.

The design of the stilling pool was complicated by the flow down New River, and transition walls of the normal shape were possible only on the downstream side with respect to the New River flow. Five different designs for the wall on the upstream side of the stilling pool were tested, and the one shown in figure 71 proved to be most satisfactory. The shape of steps in the stepped apron of the stilling pool was also given considerable study, and a size somewhat less than in the original design was chosen. The sill at the downstream end of the pool was investigated in detail. Various sizes of teeth for the Rehbock-type dentated sill were studied, and teeth as shown in figure 71 were adopted. The effect of the height of the sill was observed and found to have greater influence than changes of teeth

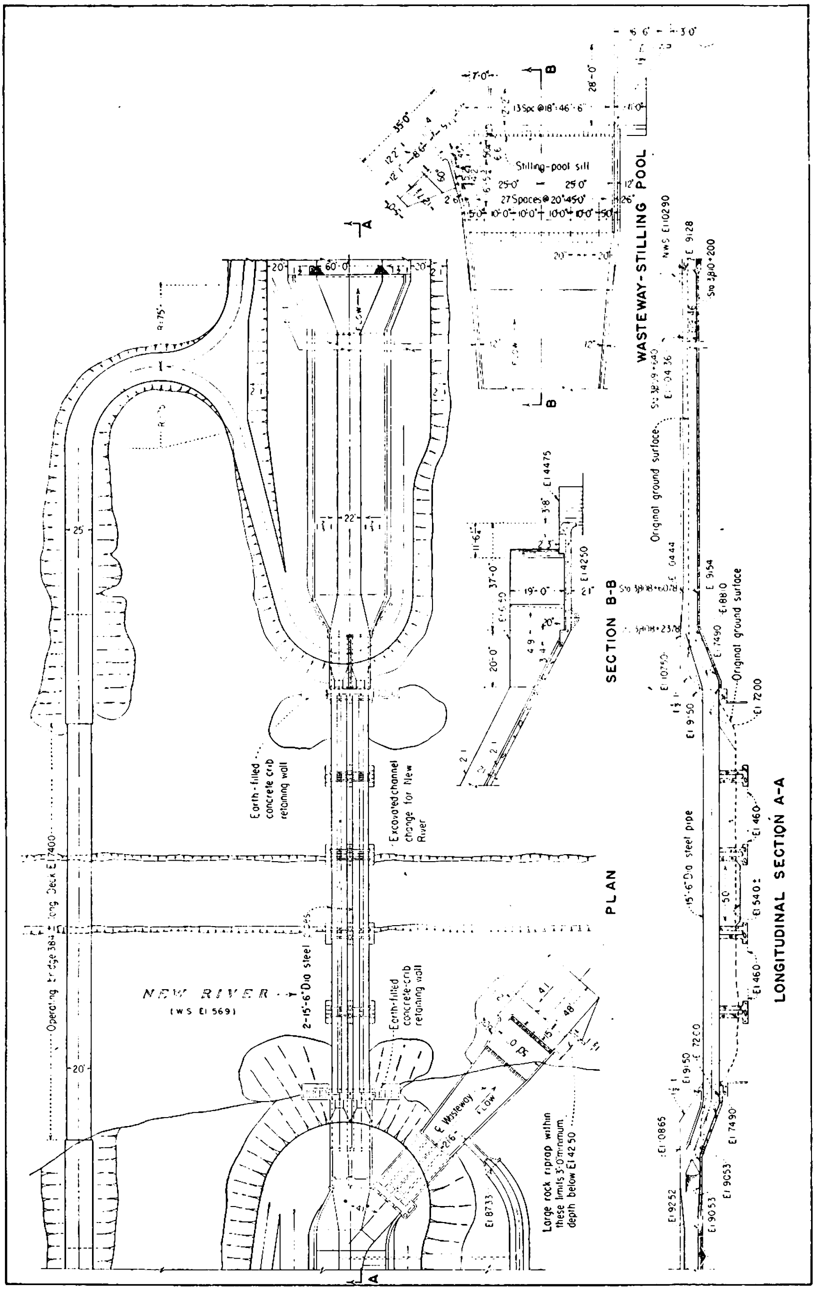

FIGURE 71.—RECOMMENDED DESIGN FOR NEW RIVER CROSSING.

design. Severe scour occurred at the upstream corner of the stilling basin, and various types of sills and groins were studied to prevent it. The use of a solid sill at this upstream corner in place of the dentated type, and two short pile groins, proved to be the best although not a perfect solution, see figure 71.

87. The Alamo River Crossing.—About 7 miles east of Calexico, California, the main line of the All-American Canal crosses the Alamo River in a lined canal constructed on a rolled fill.[9] The canal approaching the crossing is unlined, and at the transition to the lined canal a check and wasteway are installed. The Alamo River is carried in a culvert through the fill supporting the canal. This combination of structures is called the Alamo River crossing, and its general arrangement is shown in figure 72. The discharge capacities of the main canal above and below this structure are 4,700 and 4,300 second-feet, respectively, and the wasteway is designed to discharge 1,500 second-feet. The maximum flow of the Alamo River is estimated at 1,000 second-feet, and the normal flow is 100 to 200 second-feet.

To determine the best design for these structures, a model on a 1:24 scale was constructed of the main canal transition and check, the wasteway, and the culvert. The model did not cover the whole length of the lined canal on the fill, since it was expected that no problems would develop in this section. The conditions of flow in the transitions leading to and away from the check in the main canal were found to be satisfactory in the original design, as were also the flow in the intake and barrel of the culvert. The only problem encountered involved the design of the stilling pools of the wasteway and culvert. Because of the proximity of the two basins, the problem involved not only the functioning of the stilling basins individually but also in combination with each other.

The original design of these stilling basins is shown in figure 73–A, which is a photograph of part of the model. Tests were made on the wasteway stilling basin as originally designed, and with a number of combinations of piers and sills in the basin. With the original design the hydraulic jump did not form in the stilling basin with low tailwaters, but with the piers installed it formed in the basin under all conditions which are expected to be encountered. With the stepped apron, piers, and sills, the scour downstream from the basin was reduced. The best form of baffles tested is shown in figure 72. The performance of the original design of the culvert stilling pool was unsatisfactory, since the flow did not spread evenly over the basin but concentrated on one side, causing high velocities and excessive scour. Several arrangements of dividing walls and piers were investigated, but no satisfactory solution was found until the basin was reduced in size and made symmetrical around the center line

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[9] Douma, J. H., "Preliminary Report on Model Tests for the Wasteway Chute and Stilling Pool of the Alamo River Crossing. All-American Canal, Boulder Canyon Project," Office Memo. (unpublished), Bur. of Recl., June 9, 1938.

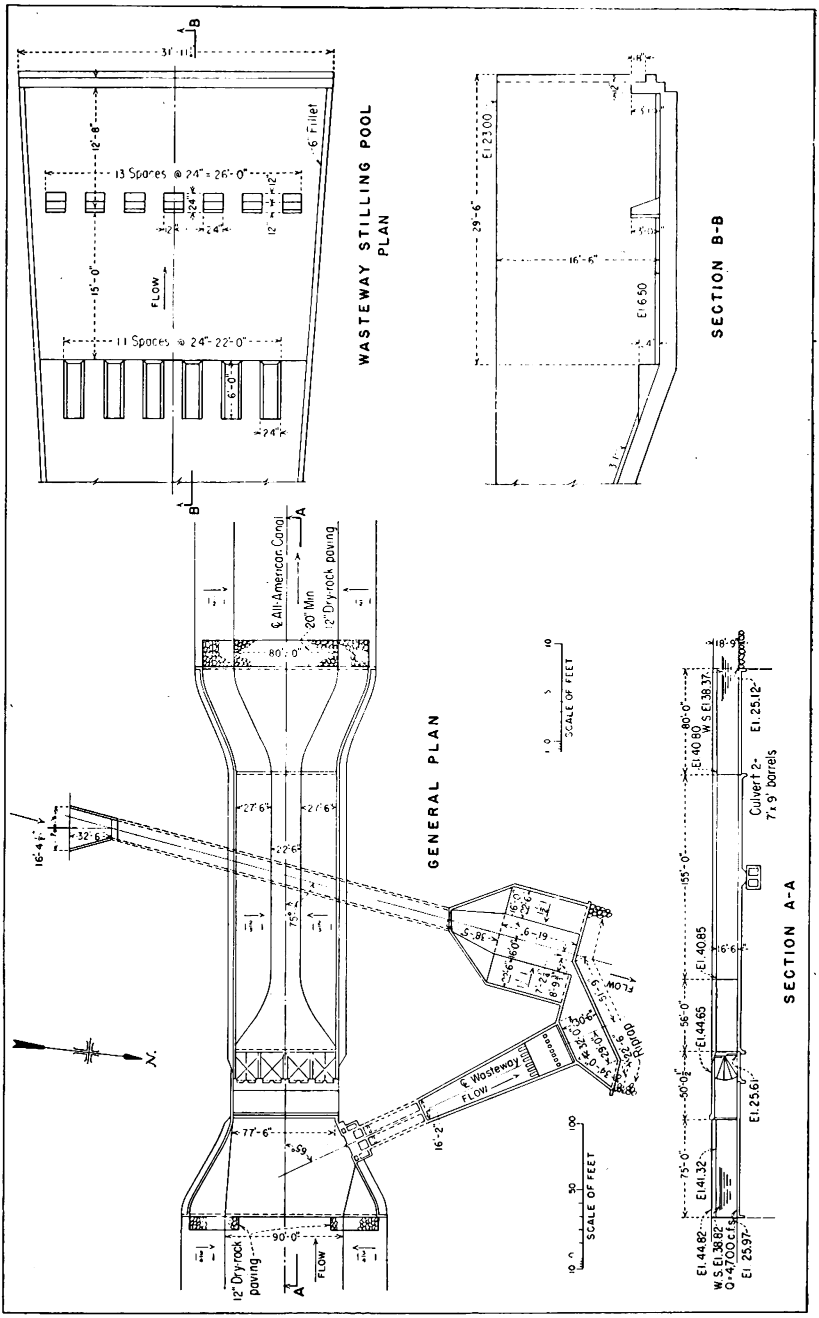
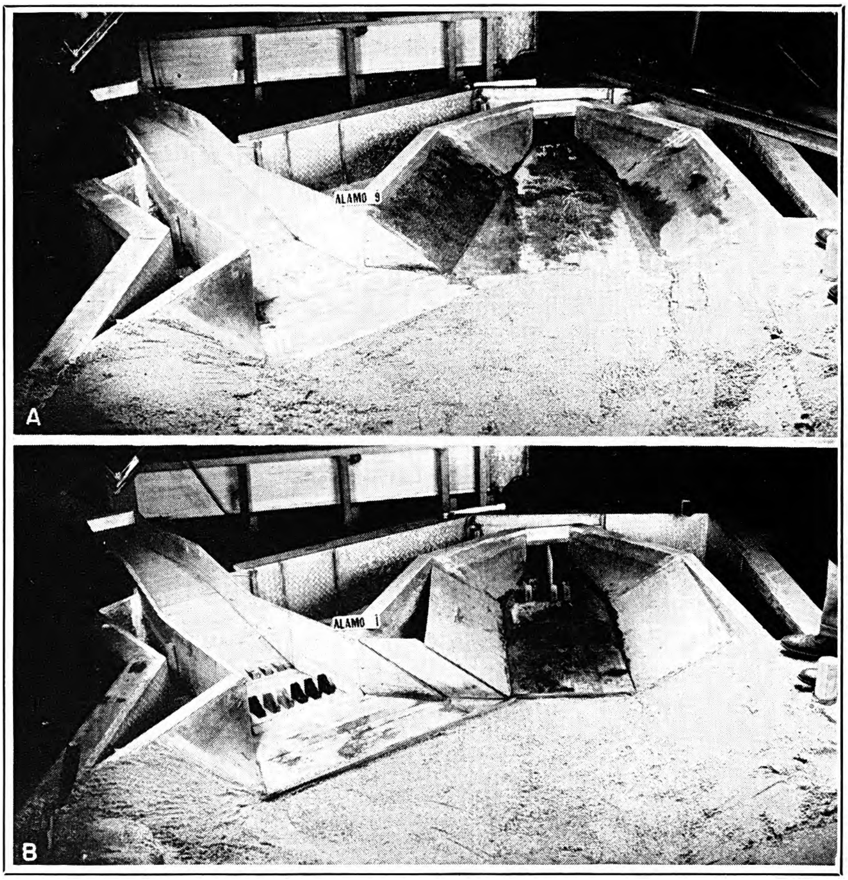


FIGURE 72.—ALAMO RIVER CROSSING.



A. Original Design. B. Recommended Design.

FIGURE 73.—MODEL OF ALAMO RIVER CROSSING.

of the conduit. With this smaller, symmetrical basin, the best solution consisted of a 20-foot dividing wall; five piers, 4 feet high, 28 feet from the culvert exit; and a sill 1 foot high at the downstream edge of the paving. The model of both stilling basins, with the most effective baffle arrangement, is shown in figure 73–B. When both basins were operating at full capacity, the action downstream from the pools was very satisfactory. When only one was operating, an eddy was formed partly in the other basin and partly in the canal, which would probably attack the banks of the canal unless they were protected with riprap. With the baffles shown in figure 73–B, however, this attack on the canal banks would be reduced to a minimum.

In connection with the tests on these structures, a detailed study of the discharge capacity of the sluice gate openings of the wasteway was undertaken. The sides of the opening were rounded, as shown in figure 74.

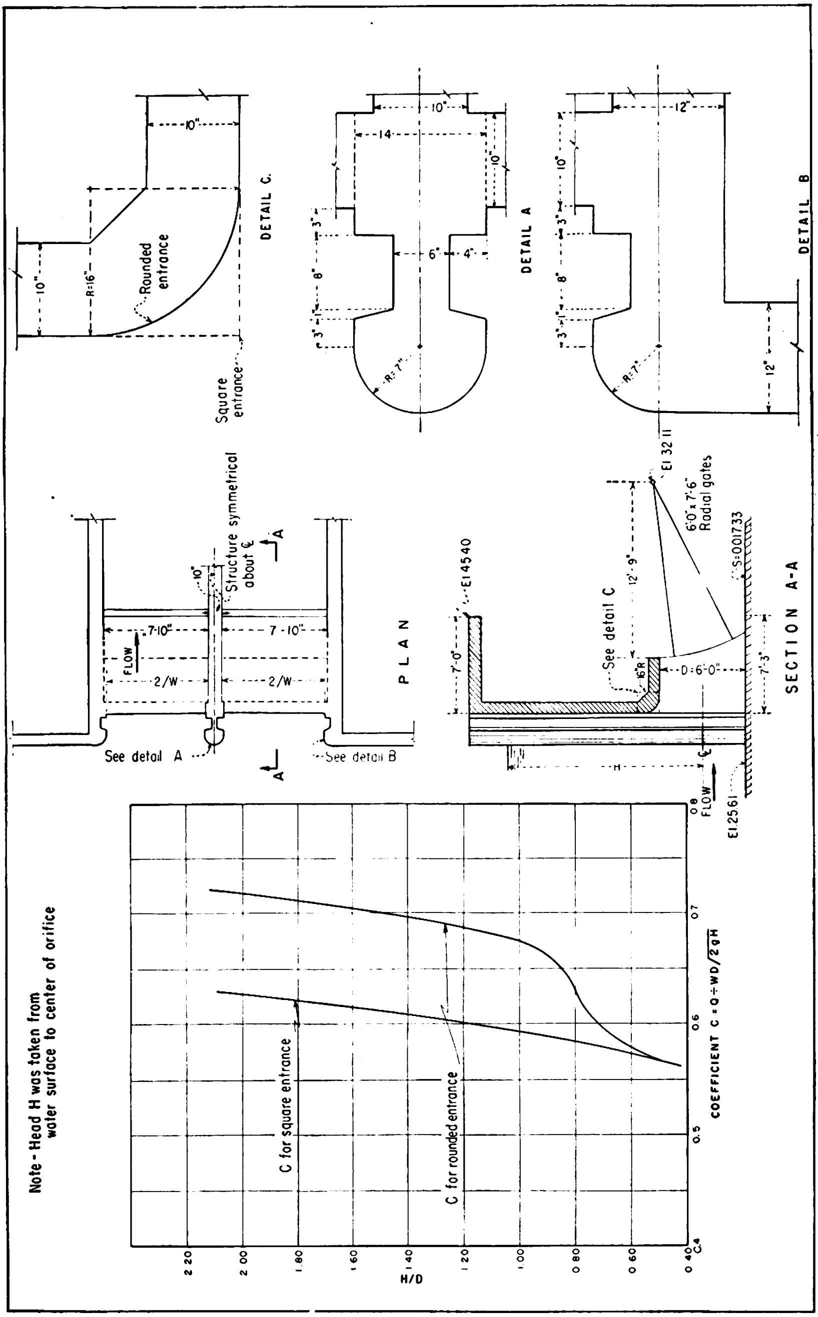


FIGURE 74.—DISCHARGE COEFFICIENTS OF SLUICE GATES—ALAMO RIVER CROSSING.

It was found that the shape of the edge at the top of the opening exerted considerable influence on the discharge capacity; with a sharp 90° corner, under the higher heads the gate had a discharge capacity of about 16 percent less than with the edge rounded to a radius of 16 inches. Since data for estimating the discharge of sluice gates are not extensive, the capacities of this model gate with square and rounded top edge are given in figure 74, in the form of orifice coefficients, the head being measured above the center of the gate opening.

DROPS

88. Drop 1, All-American Canal.—West of Yuma, Ariz., the All-American Canal passes through a ridge, beyond the crest of which the ground surface slopes more steeply than the grade of the canal. After emerging from the deep cut in which it crosses the highest part of the ridge, the ground continues to slope more steeply than the canal, and five drops were designed to keep the canal from rising above the land surface. The first of these is located about 18 miles west of Yuma, just below the turn-out for the Coachella Branch Canal, the structures for the drop and the head-works of the branch canal being contiguous. At the remaining four drops, which are distributed along the next 27 miles of the canal, it is proposed to develop power from the falling water. Power development at a drop in the Coachella Branch was also investigated. The designs of these structures were investigated by model studies in the hydraulic laboratory, as will be discussed in the following paragraphs.

Drop 1 was designed for a difference in water surface of 9.68 feet and a maximum flow of 8,700 second-feet. The original design is shown in figure 75. It consisted of seven radial gates, the center five being 15 feet 10 inches wide and the two end ones 12 feet 11 inches wide. The latter two gates were designed for automatic regulation of the level in the canal above the structure. The gates discharged on a level floor, which extended about 12 feet downstream and then sloped on a curve to another level floor at the elevation of the canal bottom downstream from the structure. A dentated apron was located at the foot of the slope, and a small dentated sill was located at the beginning of the transition to the canal section. A model of this device was constructed on a scale of 1:40. When the flow was equally distributed among all the gates, the results with this design were satisfactory, but with unequal distribution of flow considerable scour occurred. Since two of the seven gates were to be automatically regulated, equal distribution would not be the normal condition. Also, it was found that the number of gates was larger than was necessary for the required discharge.

A number of alterations of the original structure were investigated and a satisfactory form developed, see figure 76–A. The principal change made in the design was in the vertical alinement of the discharge of the radial

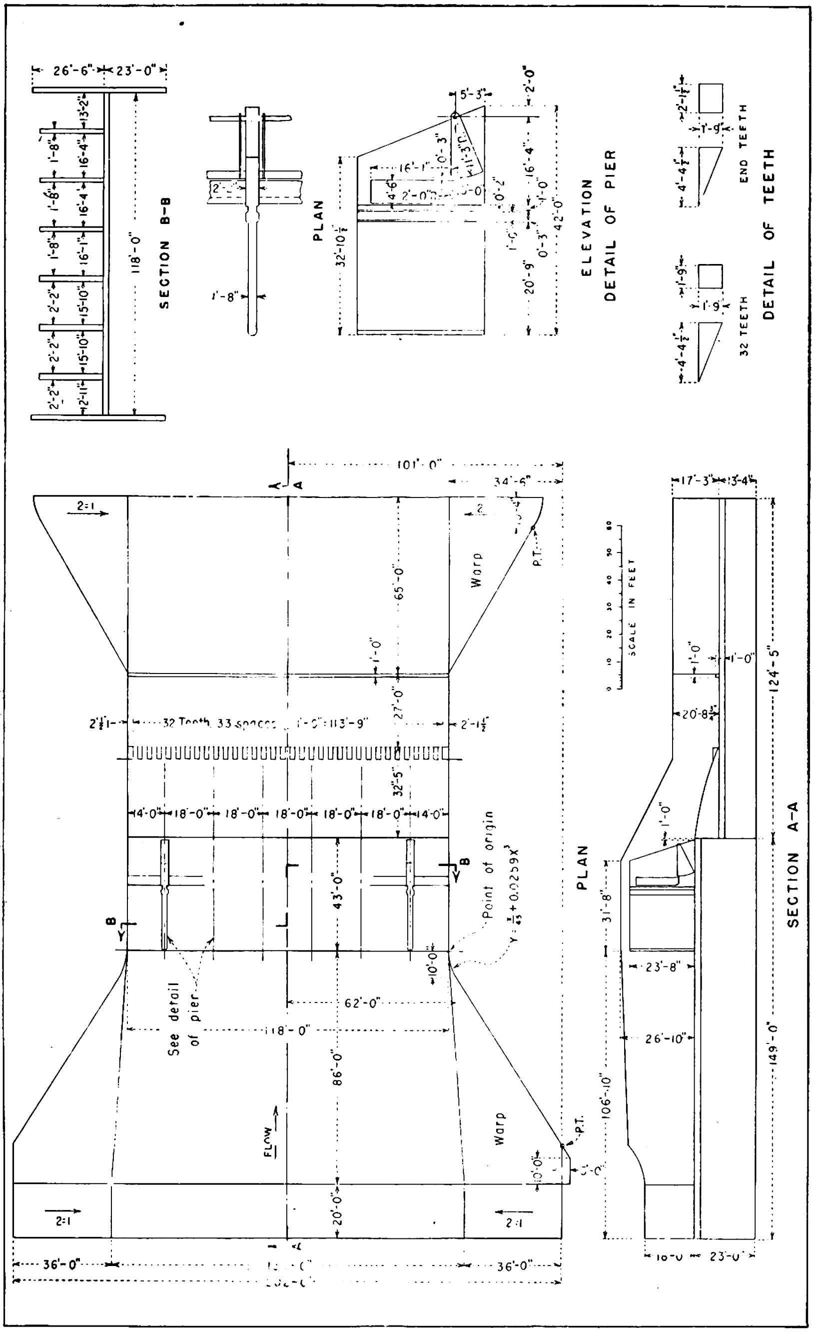
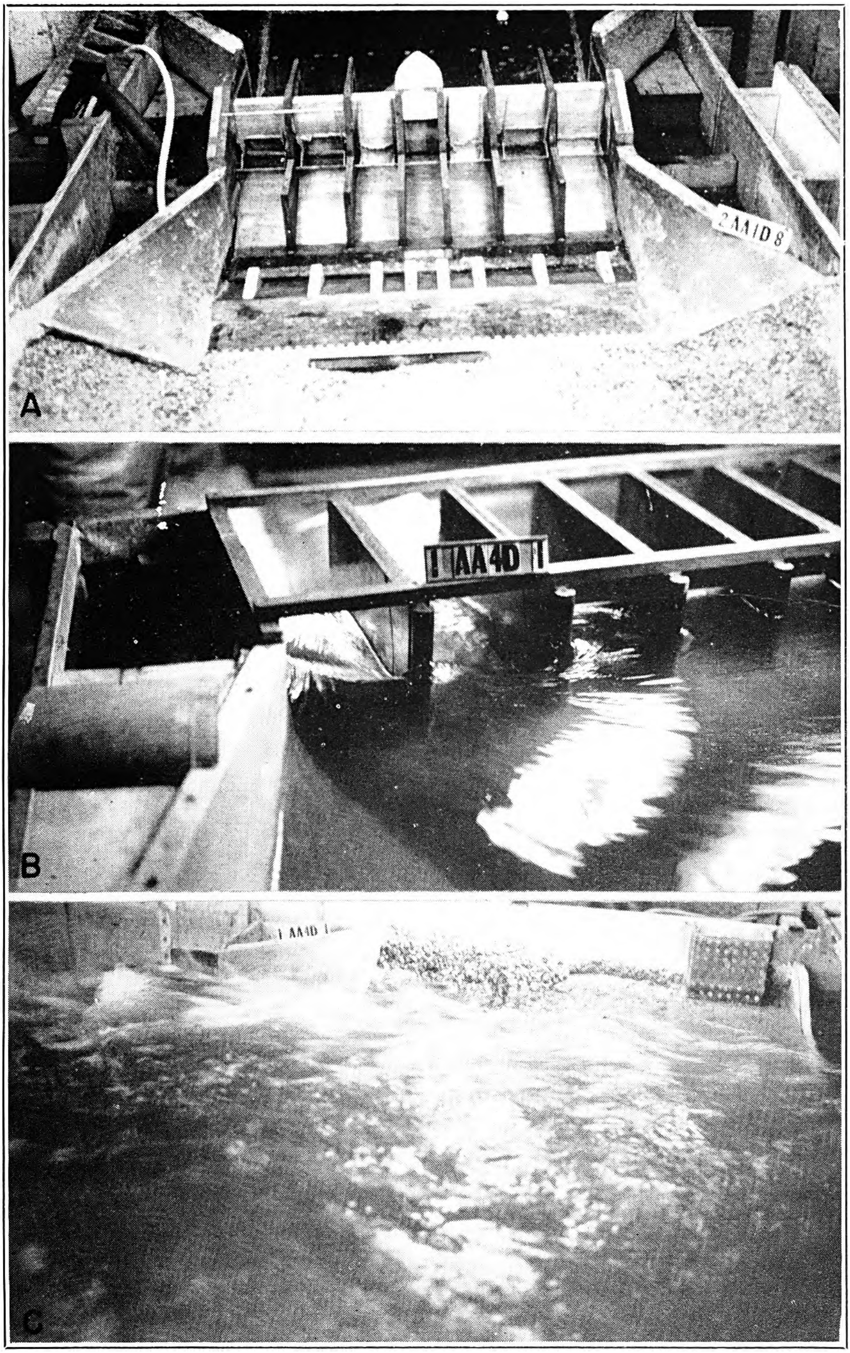


FIGURE 75.—ORIGINAL DESIGN OF DROP 1—ALL-AMERICAN CANAL.

gates. In the original design the flow from the gates was horizontal along a level floor, thence on a curved incline to the bottom of the pool. Due to the drop being small in proportion to the pool depth, the high-velocity water from the gate tended to flow over the top of the water in the pool without thoroughly mixing with it. In the final design, the discharge from the gates is directed downward along a plane sloping floor toward the bottom of the pool, which is placed at a lower elevation than in the first model. A more complete mixing of the water, with consequent dissipation of energy, is thereby accomplished. Other changes are the replacing of the center gate by a pier that houses the mechanism of the two automatic gates, which are placed one at either side of the pier instead of adjacent to the side walls of the structure. Training walls were extended downstream from each gate pier, and two sets of baffles were installed, one consisting of eight piers at the downstream side of the stilling pool and the other a Rehbock sill across the downstream end of the warped section, see figure 76–A.

89. **Drops 2 and 3, All-American Canal.**—Drops 2 and 3, located approximately 5 and 10½ miles, respectively, downstream from drop 1, are identical except that the canal width immediately upstream from drop 3 is 2 feet less, the downstream width 1 foot less, and the total discharge 300 second-feet smaller, than the corresponding values for drop 2. The difference in water surface elevations through the drops is the same. Because of this similarity in design, the following discussion of drop 2 is equally applicable to drop 3.

Since the canal would be operating before the power plants were constructed, and since it would occasionally be necessary to shut down the plants for repair, all drops were designed to carry the maximum discharge of the canal. As the most severe conditions would exist when the entire flow passed through the drop, and since the energy of any water passing through the power plant would be dissipated, the studies were concentrated on the design of the drop without the powerhouse operating. However, in the 1:36 scale model constructed to determine the proper design of the structure, provision was made to operate the powerhouse to study its effect on the operation of the drop. At drop 2 it was necessary to discharge 8,500 second-feet with a difference in water levels of 24 feet 4.4 inches. The original design, see figure 77, consisted of six gates 15 feet 8 inches wide, leading to the powerhouse; and four gates 15 feet wide, two on either side of the structure, which discharged onto a 3:1 sloping apron intersecting a horizontal pool floor. With a normal discharge flowing through the drop, the water entered the unlined portion of the canal at high velocity, causing severe erosion. An oscillation was observed in the approach channel, particularly in that part adjacent to the longer piers. A more detailed description of this action is given in the discussion of drop 4. To remedy the unstable condition, holes were provided through the longer piers to permit



A. Recommended Design of Drop 1. B. Contraction at Pier of Drop 4. C. High-Velocity Water Leaving Left Stilling Pool in Original Design of Drop 4.

FIGURE 76.—HYDRAULIC MODELS OF DROPS 1 AND 4.

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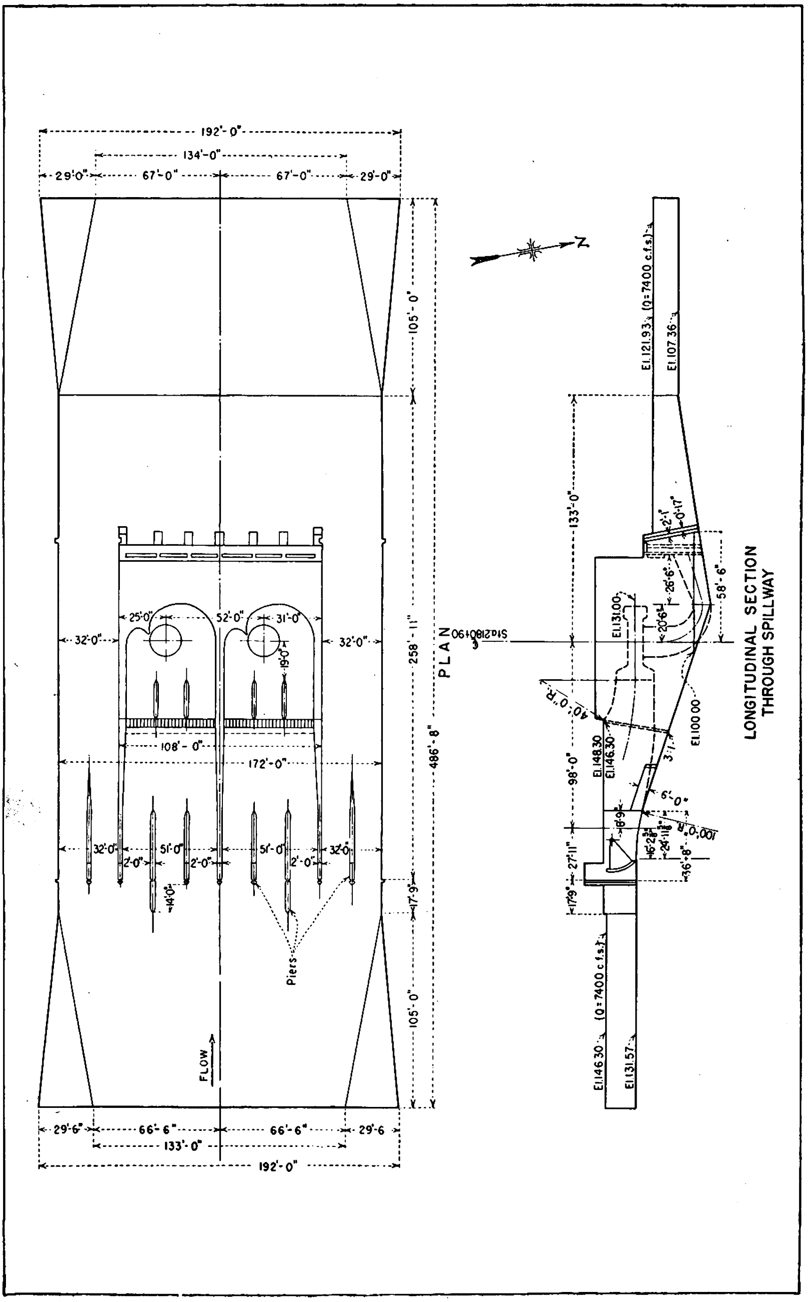


FIGURE 77.—ORIGINAL DESIGN OF DROP 2.

the water pressure to equalize on either side. Details of the final design pier for drops 2, 3, and 4 are shown in the upper part of figure 78.

Because of the unusually large depth of tailwater and thickness of jet entering the pool, it was obvious that visual means could not be relied upon to ascertain the best pool action; hence, in later tests, a pitot tube was used to give an indication of the velocities at the downstream end of the pool and aid in the selection of the proper design. Many combinations of dentated steps and baffle piers were tried in connection with different elevations and slopes of the pool floor. Figure 79 shows the most important of these designs. The dentated step served to break up the jet of water entering the pool, while the baffle piers caused a further disintegration of the jet as well as dissipation of the energy by impact on the upstream face of the baffle piers. Best results were obtained with a space rather than a tooth adjacent to the pool sides. In the recommended design, shown in the lower portion of figure 78, a rectangular sill was placed on the pool floor at the break in slope, which caused the bottom jet to be deflected upward, preventing erosion of the channel bottom. Satisfactory operation was obtained with all flow conditions, including unsymmetrical operation of the drop and flow through the power plant. The velocity distribution at the end of the warp, with symmetrical flow and maximum discharge, is shown in figure 80. For other than hydraulic reasons this design was rejected in favor of a diffusion bucket.

90. Diffusion Bucket for Drop 2.—Three of the diffusion buckets tested for drop 2 are shown in figure 81. Although very little difference existed between the various buckets, the one shown in figure 82 gave slightly lower velocities, hence was chosen as the final design. The stepped apron and rectangular sill were required in connection with the diffusion bucket to minimize the velocities entering the unlined canal. A comparison of the velocity distributions for the final design stilling pool and the recommended design is shown in figure 80 for one condition of flow. The magnitudes of velocity for the two pool designs are very nearly the same.

91. Drop 4, All-American Canal.—Drop 4, located approximately 5½ miles downstream from drop 3, was designed for a maximum discharge of 7,800 second-feet with a differential water surface of 49 feet, 8.3 inches. As in the case of drops 2 and 3, provision was made to construct a power plant in the center of the structure, which would utilize a flow of 5,200 second-feet. In the original design, see figure 83, the discharge passed through the two spillway sections onto a 2.31:1 slope which intersected the horizontal floor of the pool. A dentated step was placed near the upstream end of the pool floor and a Rehbock sill at the downstream end. A 1:28 scale model was constructed in the laboratory of the old customhouse to determine the adequacy of the design.

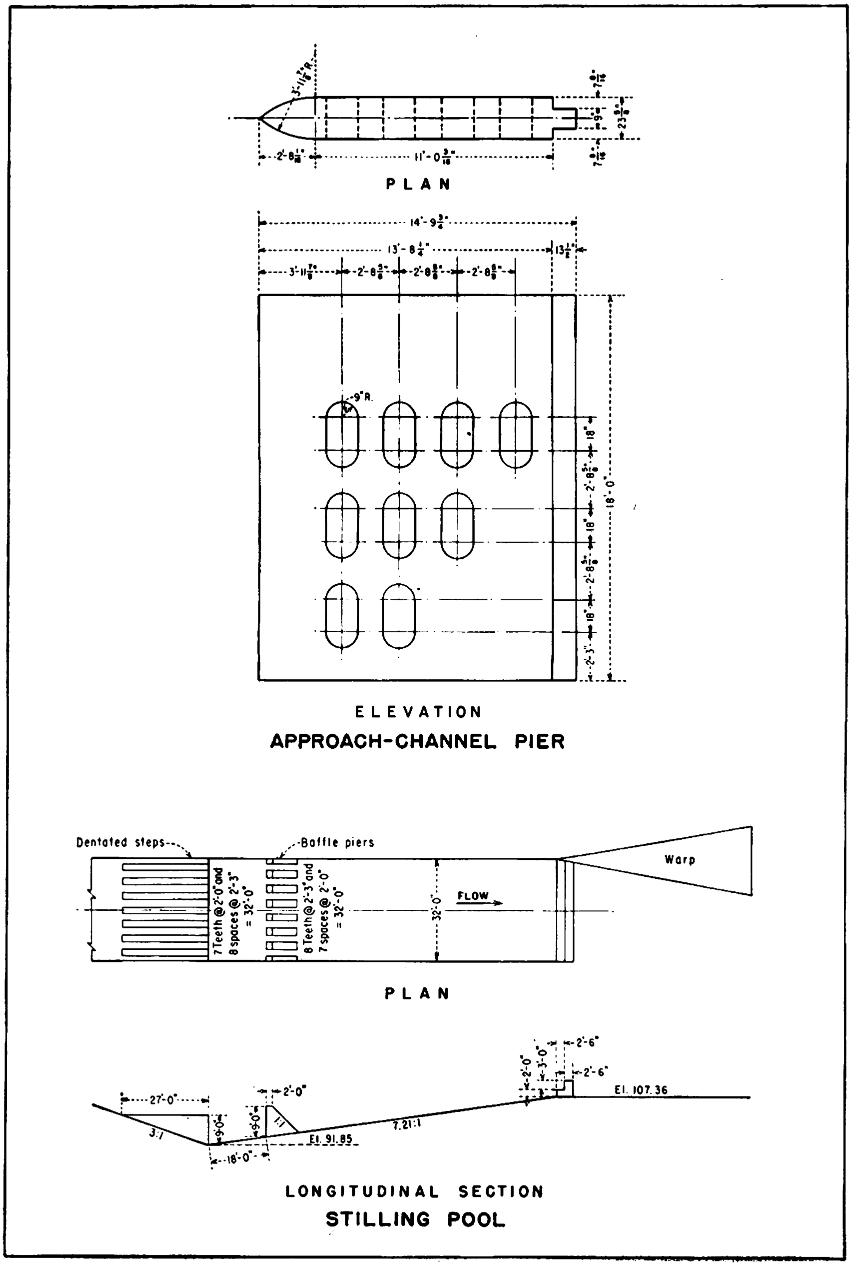


FIGURE 78.—RECOMMENDED DESIGN POOL AND APPROACH-CHANNEL PIER FOR DROP 2.

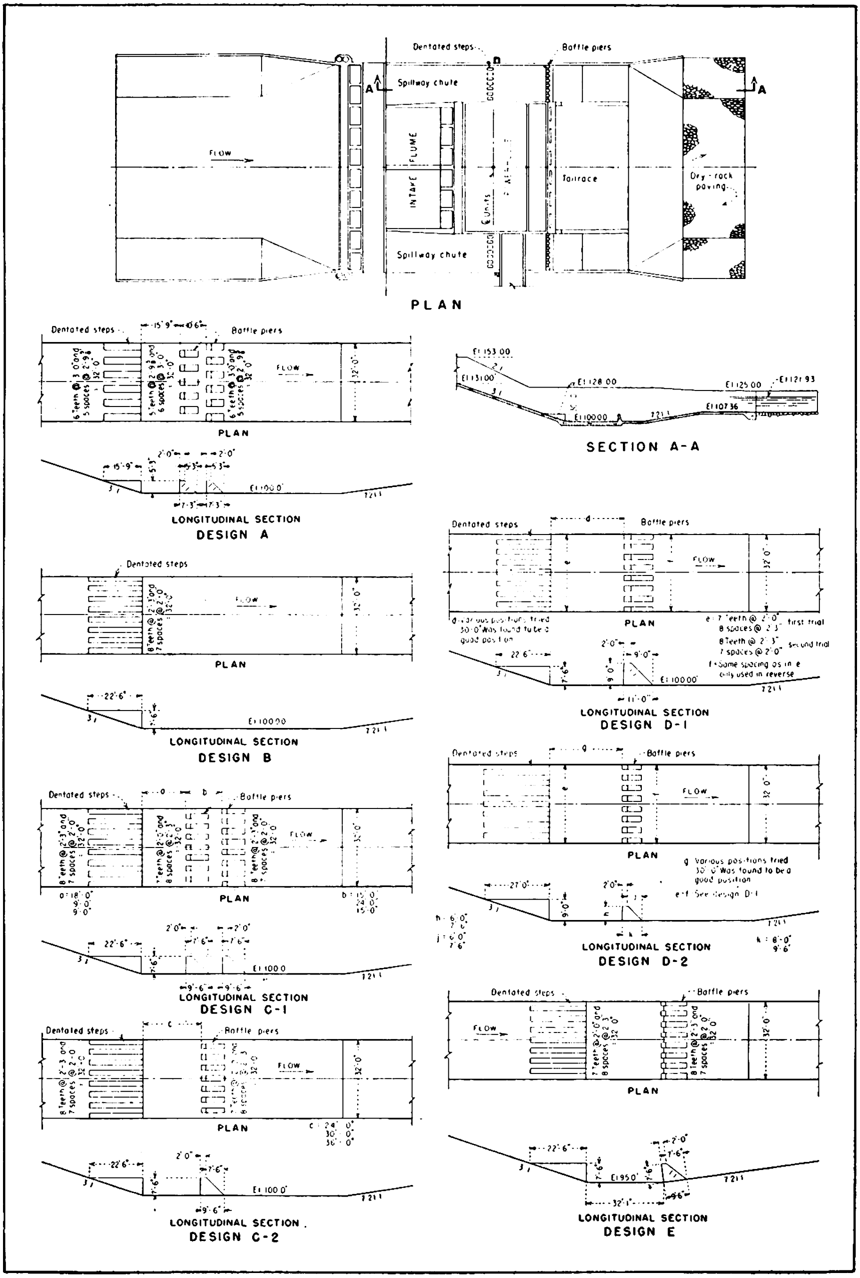


FIGURE 79.—BAFFLE-PIER STILLING POOLS STUDIED FOR DROP 2.

There were two spillway gates at either side of the drop, and the operating schedule required the canal discharge to be controlled through two gates on one side and one on the other. Under this condition of flow, on the side with both gates operating, a very pronounced contraction occurred at the pier nearest the center line of the structure, see figure 76–B; and at the corresponding pier on the opposite side of the drop a similar but less severe contraction occurred, if the outer gate was operating. As a result of these contractions an unequal distribution of flow occurred, which contributed

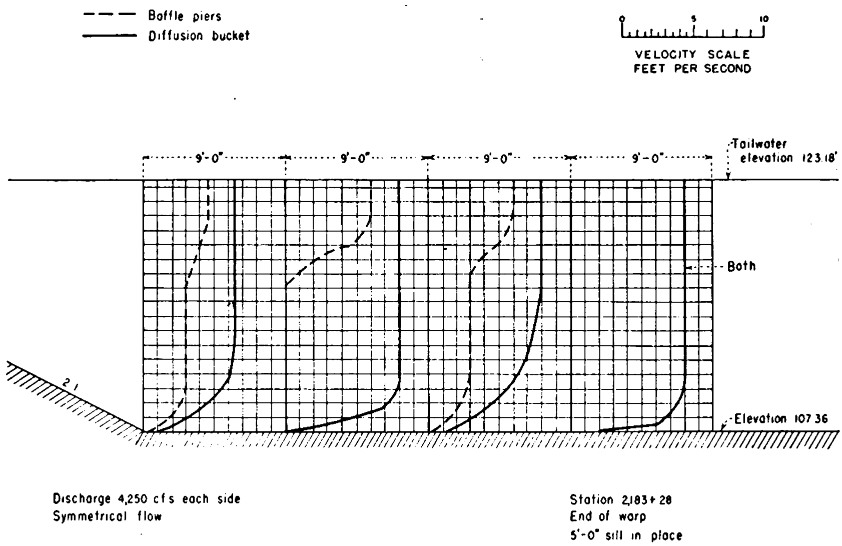


FIGURE 80.—COMPARISON OF VELOCITIES IN DIFFUSION-BUCKET AND BAFFLE-PIER POOLS—DROP 2.

to the unsatisfactory operation of the stilling pool. Since the gate piers were not streamlined, considerable disturbance occurred on the face of the spillway immediately downstream from the pier below which the flows from the two adjacent gates converged. The unsatisfactory operation of the gate section was further aggravated by the sharp break in slope where the approach floor intersected the downstream sloping face of the spillway. The high velocity of the water leaving the stilling pool severely eroded the unlined canal banks, as shown in figure 76–C. Additional erosion resulted from the large eddy currents existing in the tailrace when the power plant was not operating.

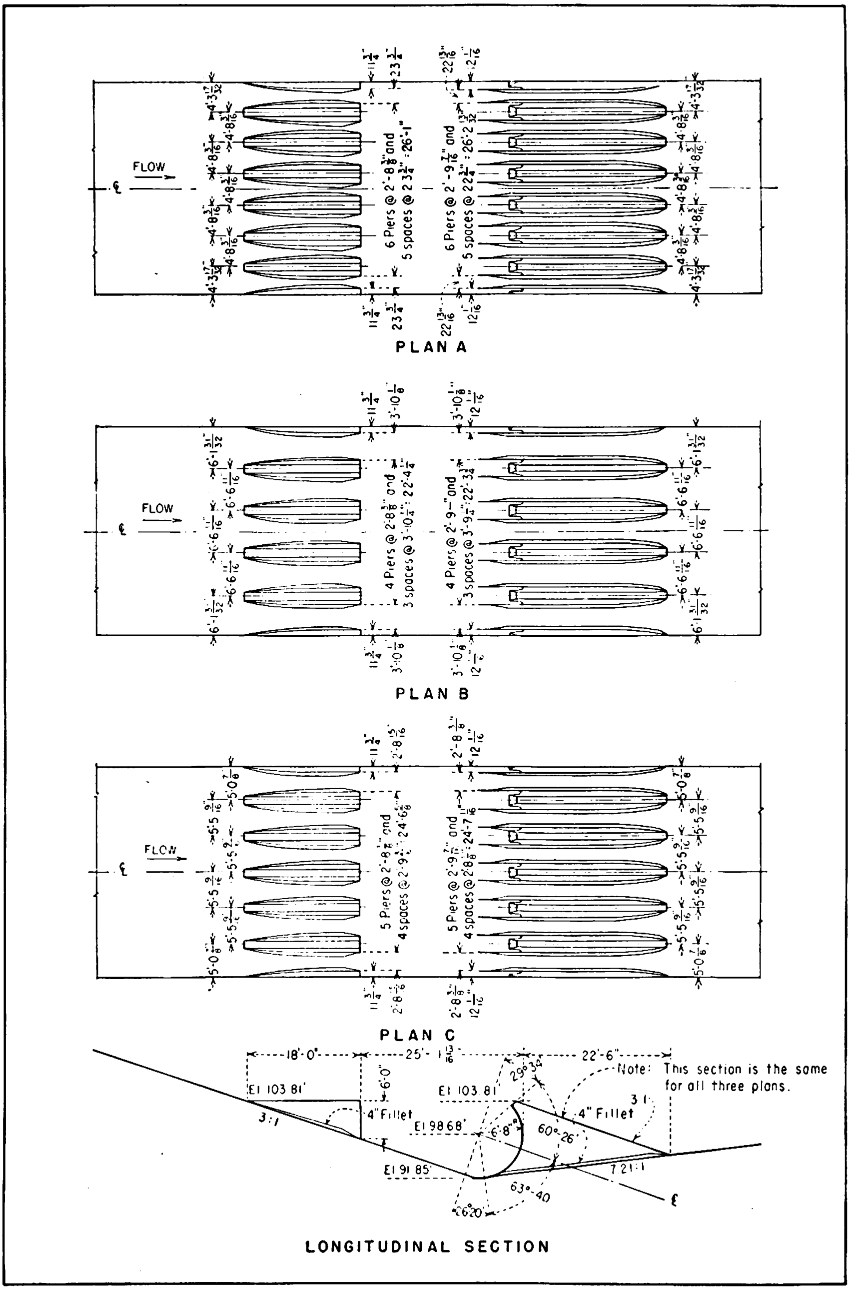


FIGURE 81.—THREE DIFFUSION BUCKETS TESTED FOR DROP 2.

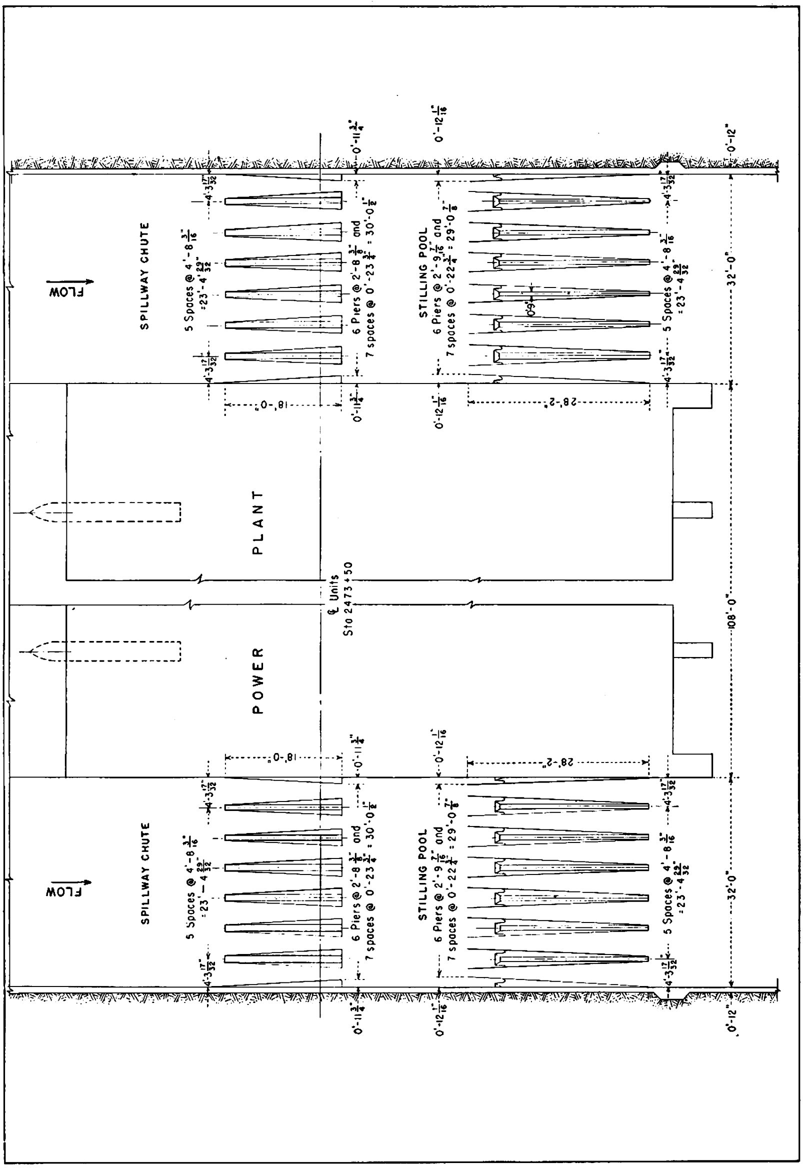


FIGURE 82.—FINAL DESIGN DIFFUSION BUCKET FOR DROP 2.

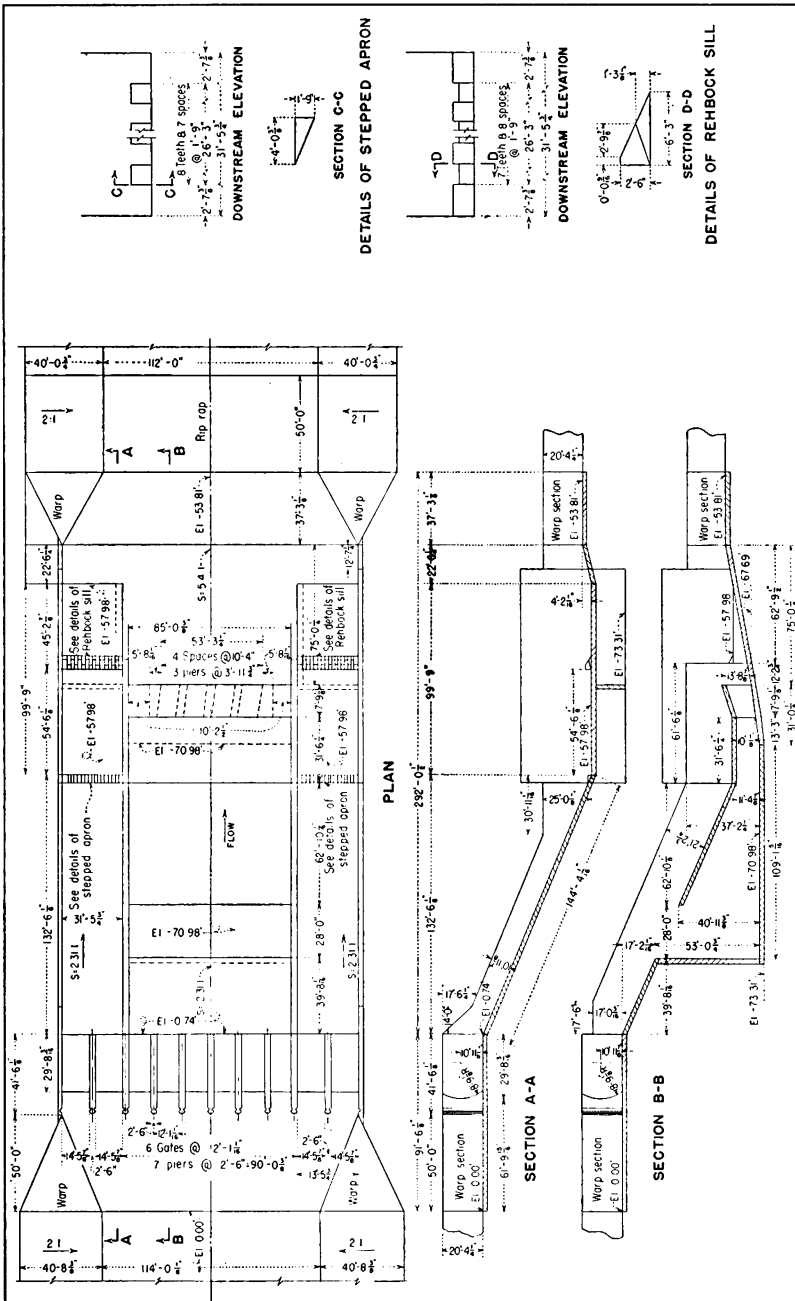


FIGURE 83.—ORIGINAL DESIGN OF DROP 4.

92. **Pier Studies for Drop 4.**—Since a satisfactory pool design could not be obtained without the proper distribution of flow entering it, efforts were first concentrated on the proper pier design. Piers of a number of shapes were investigated, and each of the three piers shown in figure 84 gave satisfactory operation. A further improvement resulted from curving the upstream portion of the central pier of either spillway section into the direction of the current as determined by injecting dye into the approach channel at various elevations; but, because of structural disadvantages and since a simple streamlined pier gave satisfactory results, the curved piers were not incorporated in the final design. Another improvement in the flow through the spillway gates was accomplished by extending in an upstream direction the piers nearest the center line of the structure. However, these longer piers caused an oscillation in the approach channel by the water flowing past the pier and impacting on the adjacent gate. Numerous trials revealed that a 14-foot pier extension produced the least contraction of flow through the gates with a minimum of oscillation. To completely eliminate the oscillation, holes were placed through the longer piers to enable the water to spread uniformly rather than impinge against the gates. As previously stated, tests on the hydraulic model resulted in the development of the perforated pier shown in figure 78.

93. **Revised Design for Drop 4.**—The model of drop 4 was revised to correct the unsatisfactory conditions of the original design. Piers as shown in figure 84–C were installed on the spillway crest, which included a vertical curve connecting the approach floor to the downstream sloping face. In this new design, see figure 85, the slope of the spillway face was changed from 2.31 : 1 to 3 : 1. The gates were necessarily longer because of the decreased thickness of the streamlined piers. Operation of the hydraulic model revealed very satisfactory flow conditions through the gates and down the spillway face; however, the action of the stilling pool was not sufficient to prevent scouring velocities in the unlined canal. The operating schedule was changed at this time to include symmetrical flow through all four gates. Under this condition the stilling pool was more effective but erosive velocities persisted. Various combinations of dentated steps and baffle piers were tested, with varying degrees of success, see figure 86. A satisfactory condition resulted with the set-up in figure 87, and that arrangement was therefore recommended for incorporation in the final structure. The velocities produced by this pool will be discussed later.

94. **Various Types of Stilling Pools Studied, Drop 4.**—For structural reasons the recommended baffle pier was abandoned and studies were immediately undertaken on other methods of dissipating the energy. The first device considered was a submerged weir contemplated to maintain a pool of water in which the energy would be largely dissipated. By making the weir V-shaped in plan, it was expected to cause the high-velocity water leaving the pool to be diverted to the center of the canal, thereby preventing

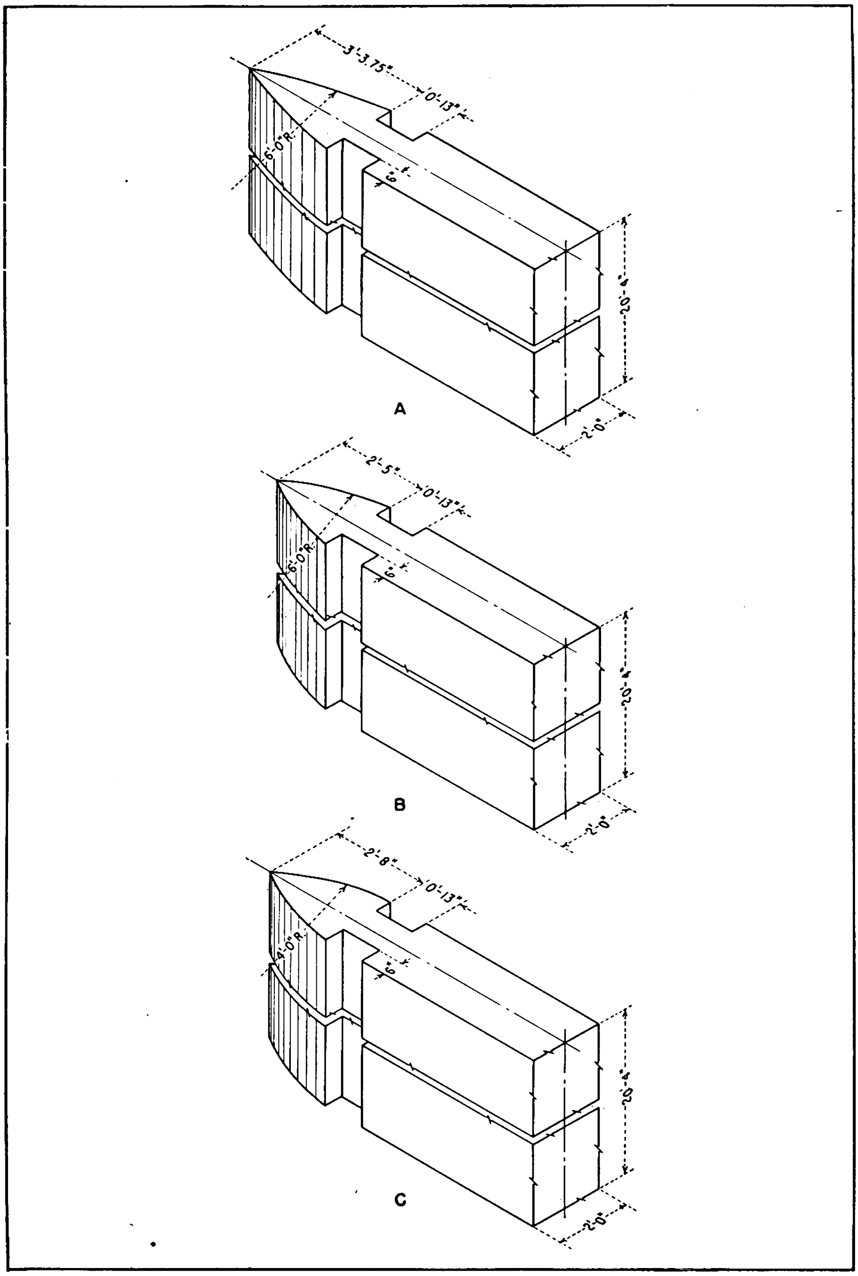


FIGURE 84.—PIERS STUDIED FOR DROP 4.

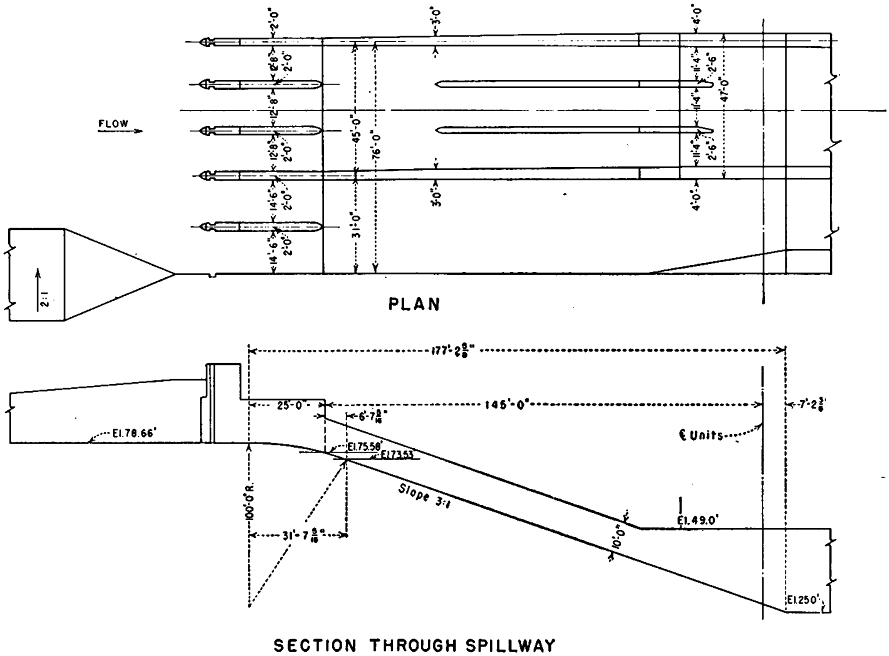


FIGURE 85.—REVISED DESIGN OF DROP 4.

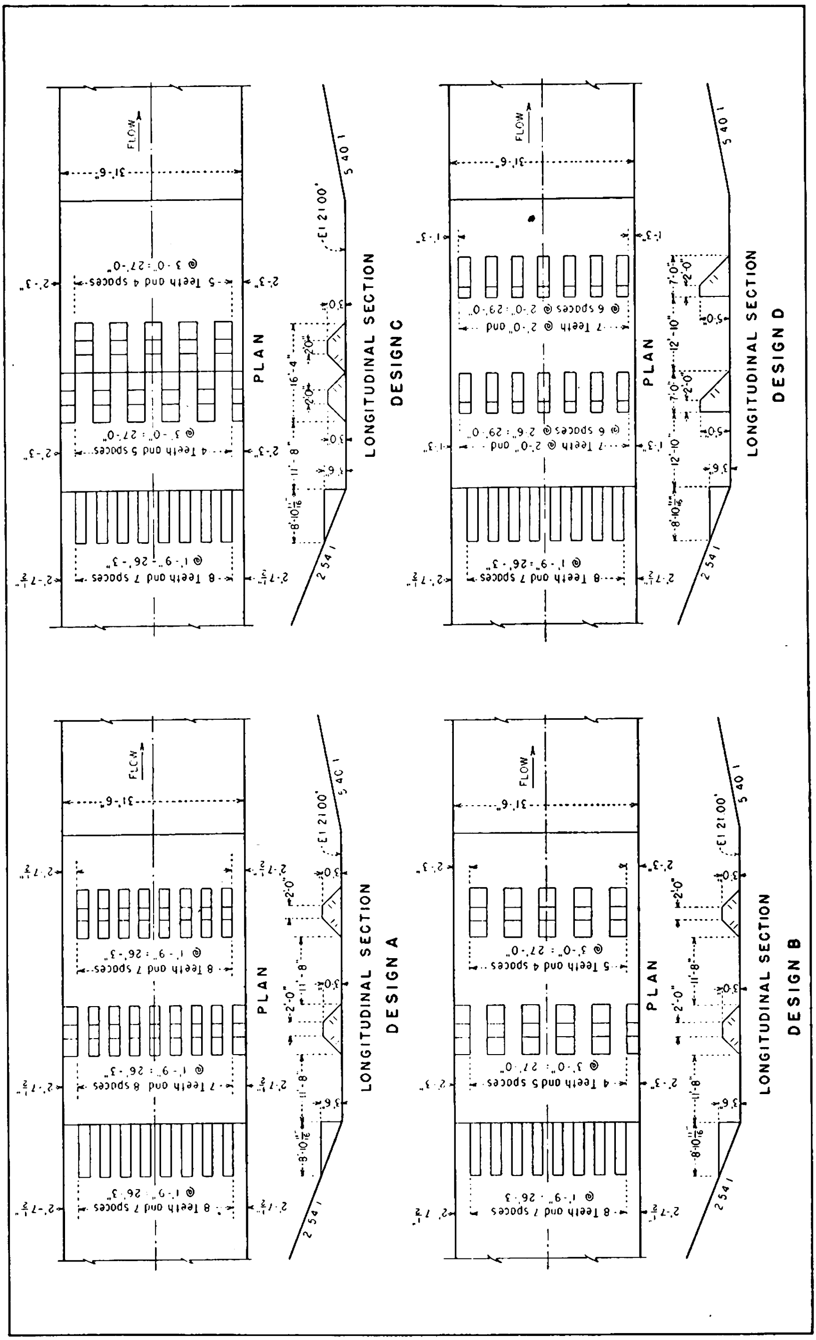


FIGURE 86.—BAFFLE-PIER POOLS FOR DROP 4.

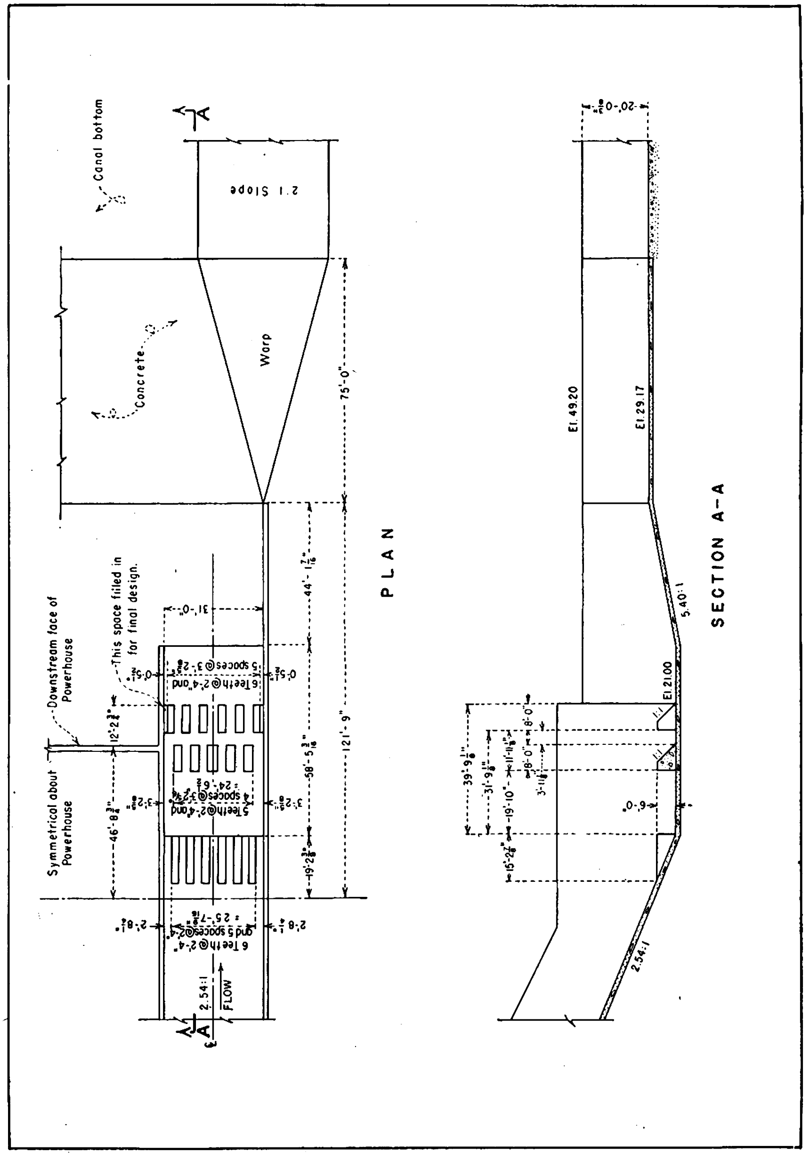


FIGURE 87.—RECOMMENDED DESIGN STILLING POOL FOR DROP 4.

erosion of the canal banks. The two weirs tested, see figure 88, failed to produce acceptable results. The next tests were with a type of diffusion bucket similar to the one used at Grand Coulee Dam. This design, see figure 88, also failed to produce sufficiently low velocities in the canal downstream, chiefly because the tailwater depth was insufficient to prevent a severe boiling action with resultant high velocities. The type of bucket provided for drops 2 and 3 was next studied in the hydraulic model.

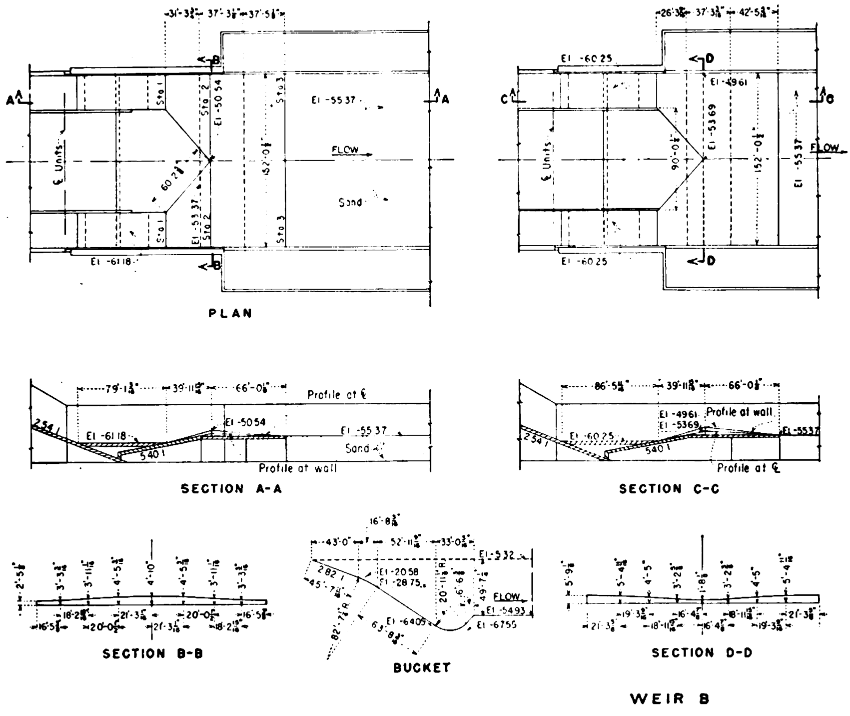


FIGURE 88.—OTHER TYPES OF DISSIPATORS STUDIED FOR DROP 4.

95. Diffusion Bucket for Drop 4.—As in the study of the other drops, various designs of diffusion buckets for drop 4 were tested in conjunction with a dentated sill near the pool entrance. Many of the designs tested are shown in figures 89 and 90. In general, it was planned that each of the several buckets would dissipate a maximum amount of energy, and the water at reduced velocity would be directed towards the center of the canal by a triangular transition connecting the sides of the stilling pool with the banks of the canal, or by the deflector barriers shown in plans C and F. The latter method failed to change the direction of flow, but the triangular transition in conjunction with the diffusion bucket of figure 91 produced acceptable results and was incorporated in the final design. Visual observations with the powerhouse operating in conjunction with

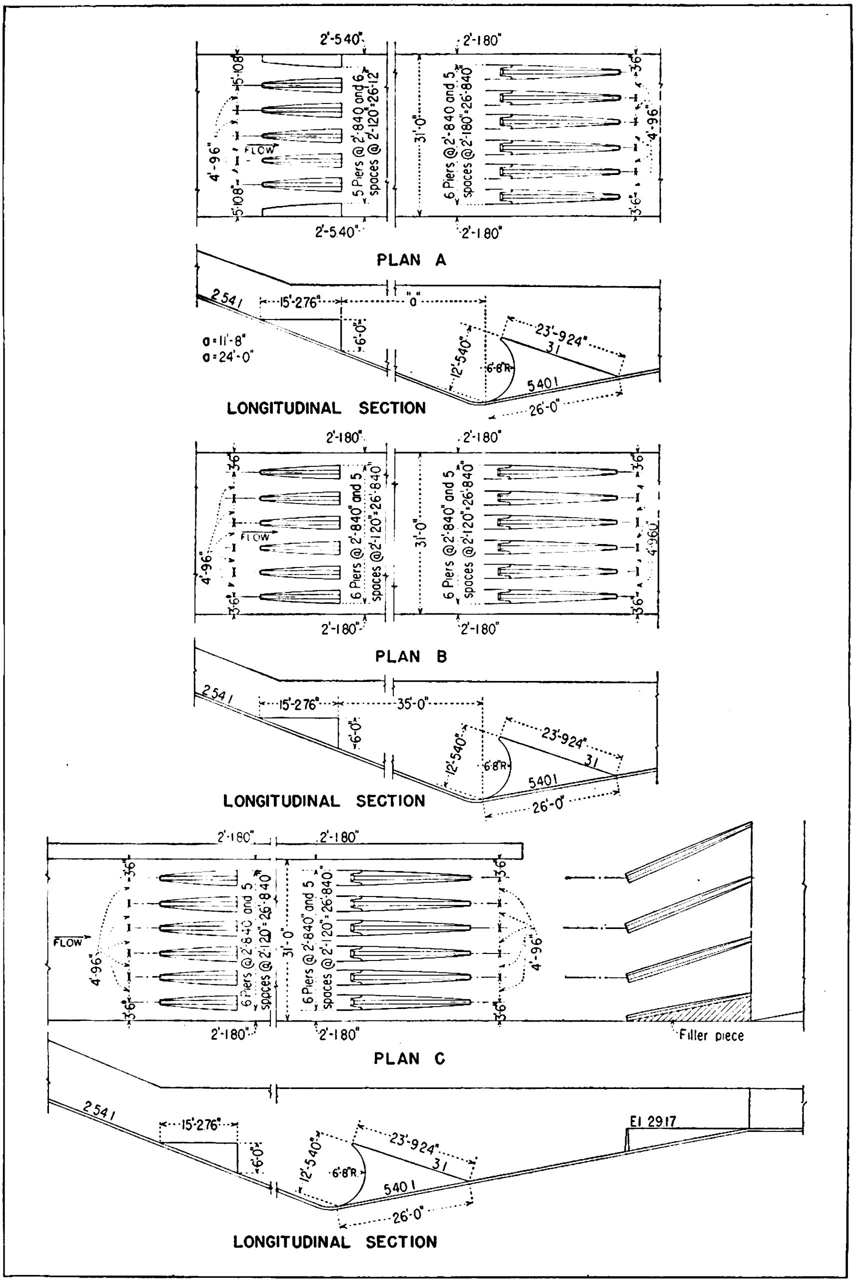


FIGURE 89.—SOME DIFFUSION BUCKETS TESTED FOR DROP 4.

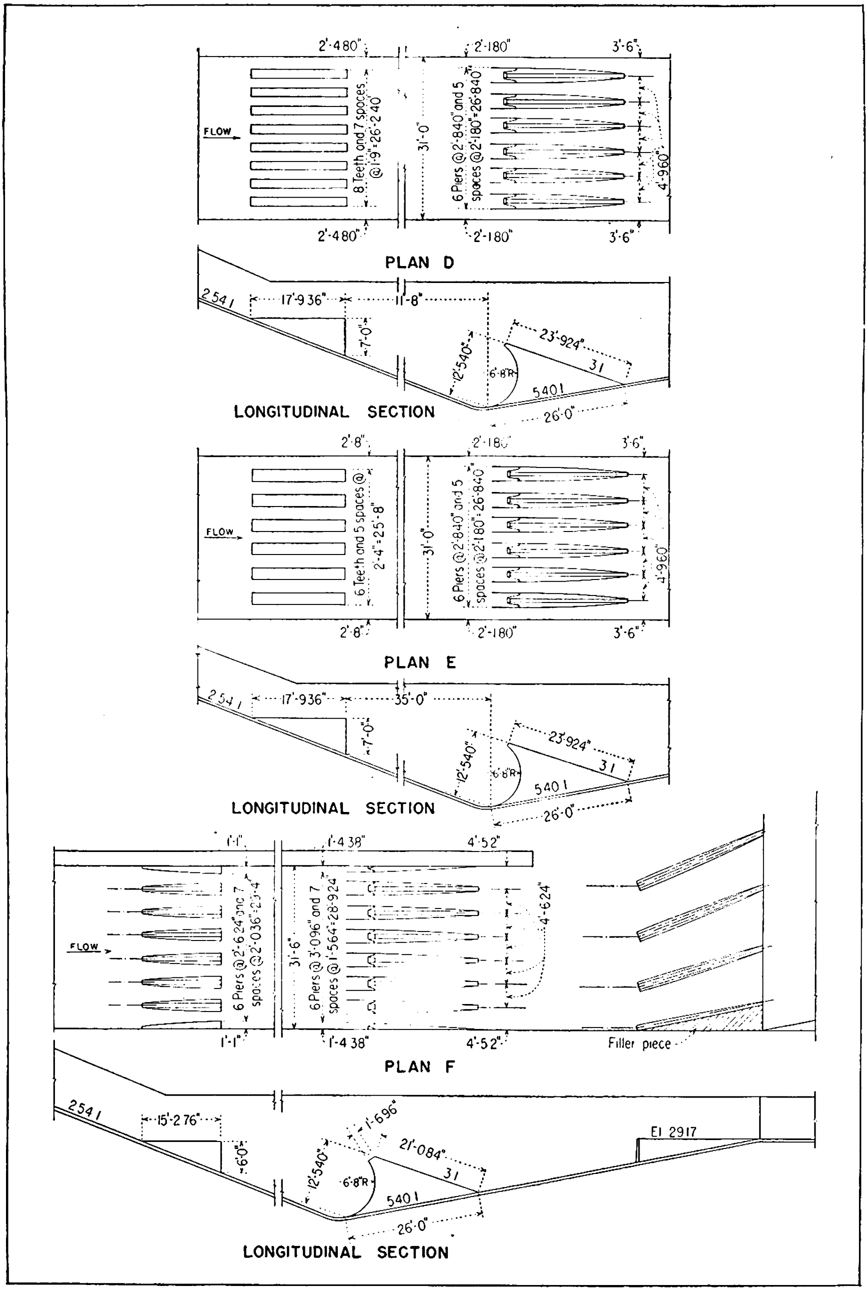


FIGURE 90.—ADDITIONAL DIFFUSION BUCKETS TESTED FOR DROP 4.

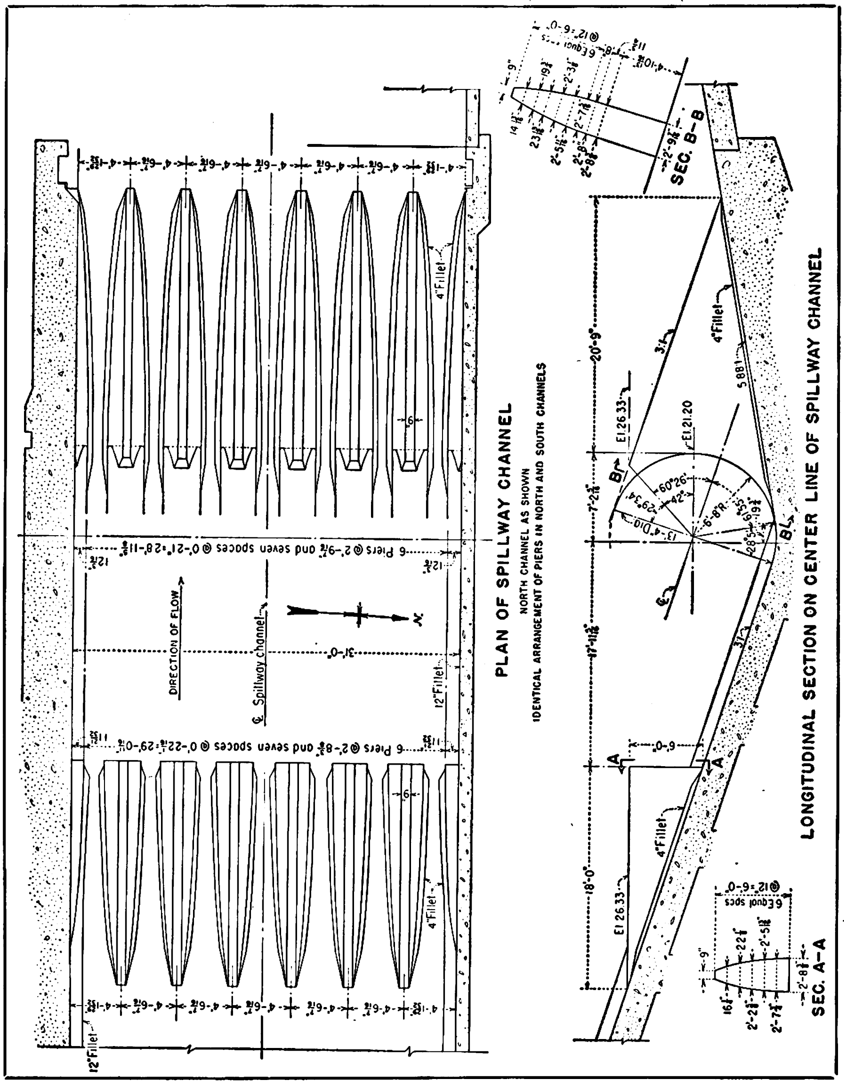
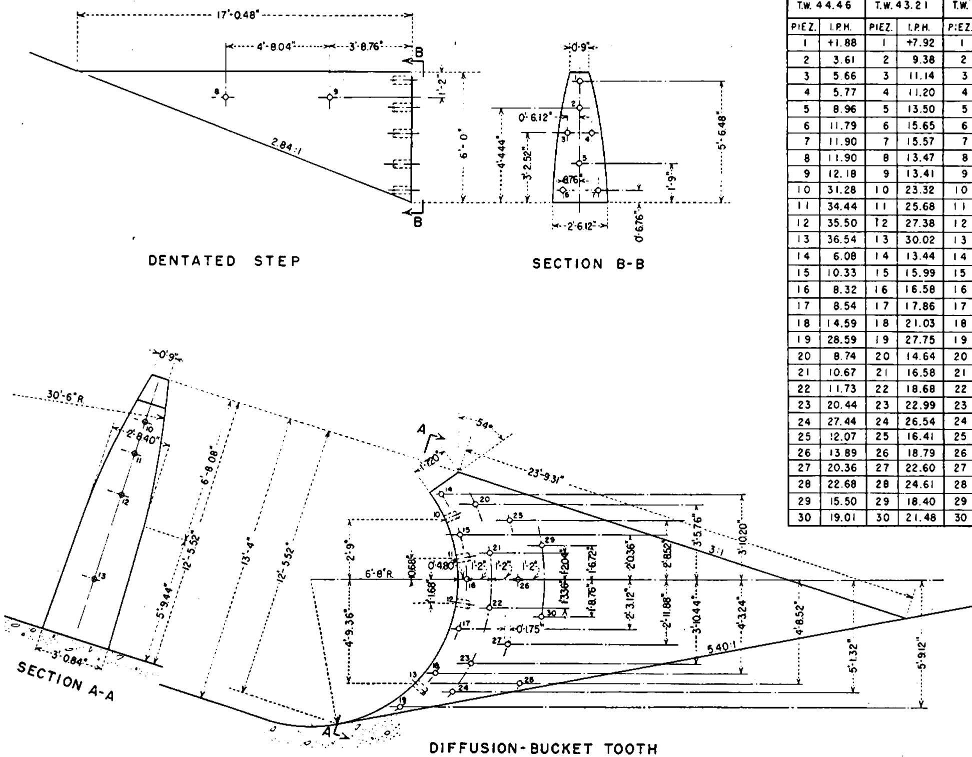


FIGURE 91.—FINAL DESIGN STILLING BASIN FOR DROP 4.

symmetrical and unsymmetrical flow through the gates revealed satisfactory results. Although not tested on the model of drop 4, a rectangular sill at the downstream end of the pool, found necessary in the studies of drops 2 and 3, was provided in the final structure. Pressure measurements taken on one of the teeth of the diffusion bucket are shown in figure 92.

96. **Comparison of Baffle-Pier and Diffusion-Bucket Stilling Pools.**—In comparing the hydraulic action of the two types of pools considered for drop 4, it may be seen that, in general, the final results are the same. The water surface produced by the baffle-pier pool is smoother, as may be seen from figures 93 and 94, which show the profiles for both symmetrical and unsymmetrical flow. It may also be seen that the end of the roller is farther upstream in the case of the baffle-pier pool, while the beginning of the jump is at the same point for either design. In the model tests, velocities were taken at various stations, but those recorded at the end of the concrete pavement are taken as indicative of the pool action. A comparison of these velocities for symmetrical and unsymmetrical flow is shown in figure 95. When the velocities were taken for the baffle-pier pool, a 75-foot warped transition was in place on either side of the canal, while for the measurements in the other pool a 105-foot warped transition was on the left side and a 105-foot three-plane transition on the right side. Because of this difference in transitions, a direct comparison will not indicate the true conditions; the velocities for the two pools are probably of equal magnitude instead of slightly higher for the baffle-pier pool as would be indicated from figure 95 alone. A visual comparison of the flow in the two types of pool is possible in figure 96.

97. **Drop 5, All-American Canal.**—Drop 5 is located approximately 10¾ miles downstream from drop 4. It is designed to operate at a maximum discharge of 4,200 second-feet with a differential water surface of 23 feet 1.3 inches. Because of the similarity of the structures it was feasible to meet the model requirements for drop 5 by a conversion of the 1:36 scale hydraulic model of drop 2. The same scale ratio was maintained on the one side of the model converted. By using the criteria previously established, only a few tests on this structure were necessary. The same piers, including the perforated ones, were used and produced the desired flow conditions. The baffle-pier pool recommended for drop 2 was adjusted for the 10-foot narrower pool width and installed in this model, as shown in figure 97–A. Since satisfactory conditions prevailed during operation, no further tests were conducted with this type of pool.



Q = 5,200 C.F.S. T.W. 44.46		Q = 3,400 C.F.S. T.W. 43.21		Q = 3,900 C.F.S. T.W. 44.46		Q = 5,200 C.F.S. T.W. 42.56	
PIEZ.	I.P.H.	PIEZ.	I.P.H.	PIEZ.	I.P.H.	PIEZ.	I.P.H.
1	+1.88	1	+7.92	1	+7.67	1	-1.23
2	3.61	2	9.38	2	9.63	2	+0.67
3	5.66	3	11.14	3	11.34	3	2.80
4	5.77	4	11.20	4	11.51	4	3.14
5	8.96	5	13.50	5	13.89	5	6.16
6	11.79	6	15.65	6	15.88	6	9.35
7	11.90	7	15.57	7	15.93	7	9.16
8	11.90	8	13.47	8	14.20	8	9.74
9	12.18	9	13.41	9	14.31	9	10.72
10	31.28	10	23.32	10	26.04	10	30.49
11	34.44	11	25.68	11	28.34	11	34.33
12	35.50	12	27.38	12	30.32	12	35.87
13	36.54	13	30.02	13	32.26	13	36.51
14	6.08	14	13.44	14	13.58	14	3.70
15	10.33	15	15.99	15	16.63	15	8.79
16	8.32	16	16.58	16	16.58	16	5.21
17	8.54	17	17.86	17	17.61	17	4.28
18	14.59	18	21.03	18	20.44	18	11.82
19	28.59	19	27.75	19	28.06	19	27.55
20	8.74	20	14.64	20	14.76	20	5.94
21	10.67	21	16.58	21	16.86	21	7.36
22	11.73	22	18.68	22	18.68	22	8.85
23	20.44	23	22.99	23	23.10	23	18.09
24	27.44	24	26.54	24	28.06	24	25.34
25	12.07	25	16.41	25	16.74	25	9.46
26	13.89	26	18.79	26	19.15	26	11.03
27	20.36	27	22.60	27	23.27	27	17.98
28	22.68	28	24.61	28	25.84	28	20.08
29	15.50	29	18.40	29	19.07	29	13.27
30	19.01	30	21.48	30	22.15	30	16.74

Q = 5,200 C.F.S. T.W. 40.73	
PIEZ.	I.P.H.
1	[illegible]
2	[illegible]
3	-0.17
4	-0.03
16	+0.59
17	-0.06

Q = 5,200 C.F.S. T.W. 41.44	
PIEZ.	I.P.H.
1	-4.28
2	-1.82
3	-1.12
4	+2.80
16	+1.62
17	+3.64

FIGURE 92.—PRESSURES ON DIFFUSION BUCKET OF DROP 4.

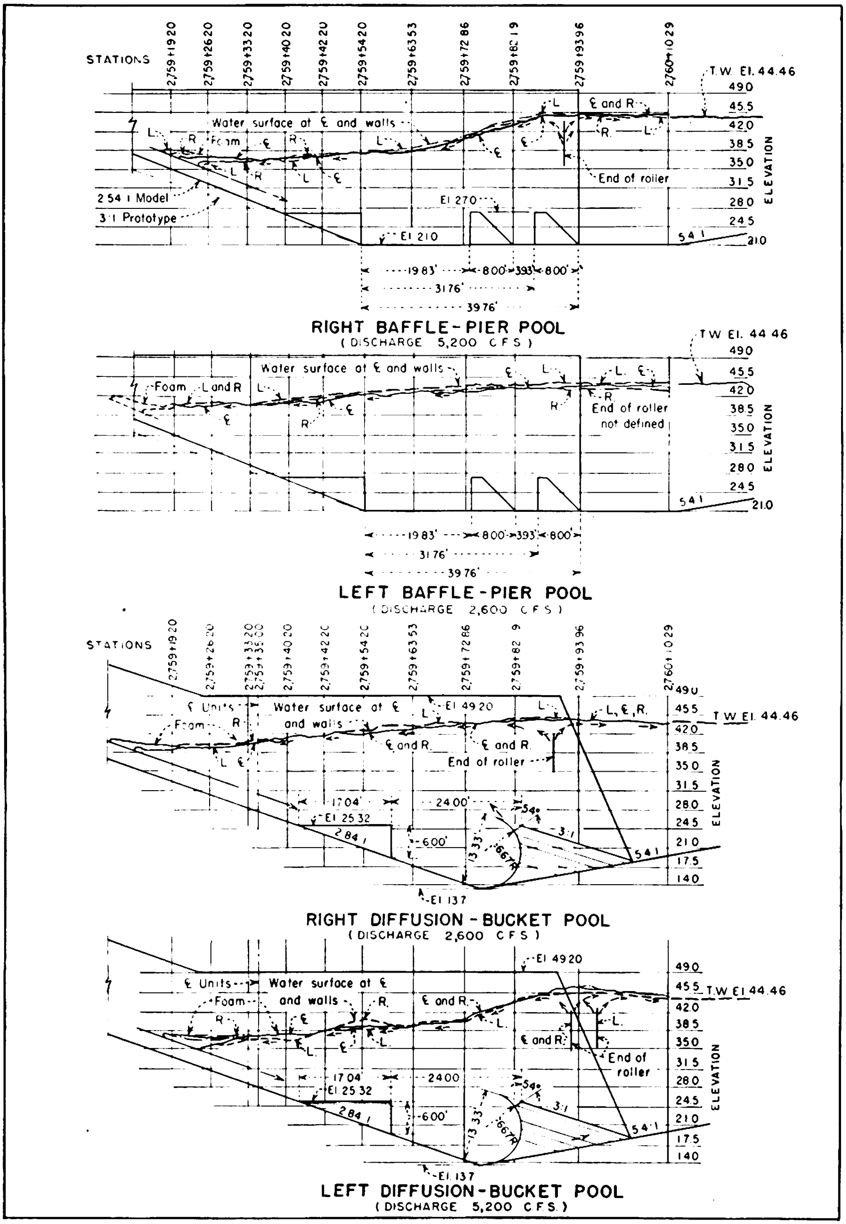


FIGURE 93.—WATER SURFACE PROFILES FOR BAFFLE-PIER AND DIFFUSION-BUCKET STILLING POOLS—DROP 4.

Symmetrical Operation.

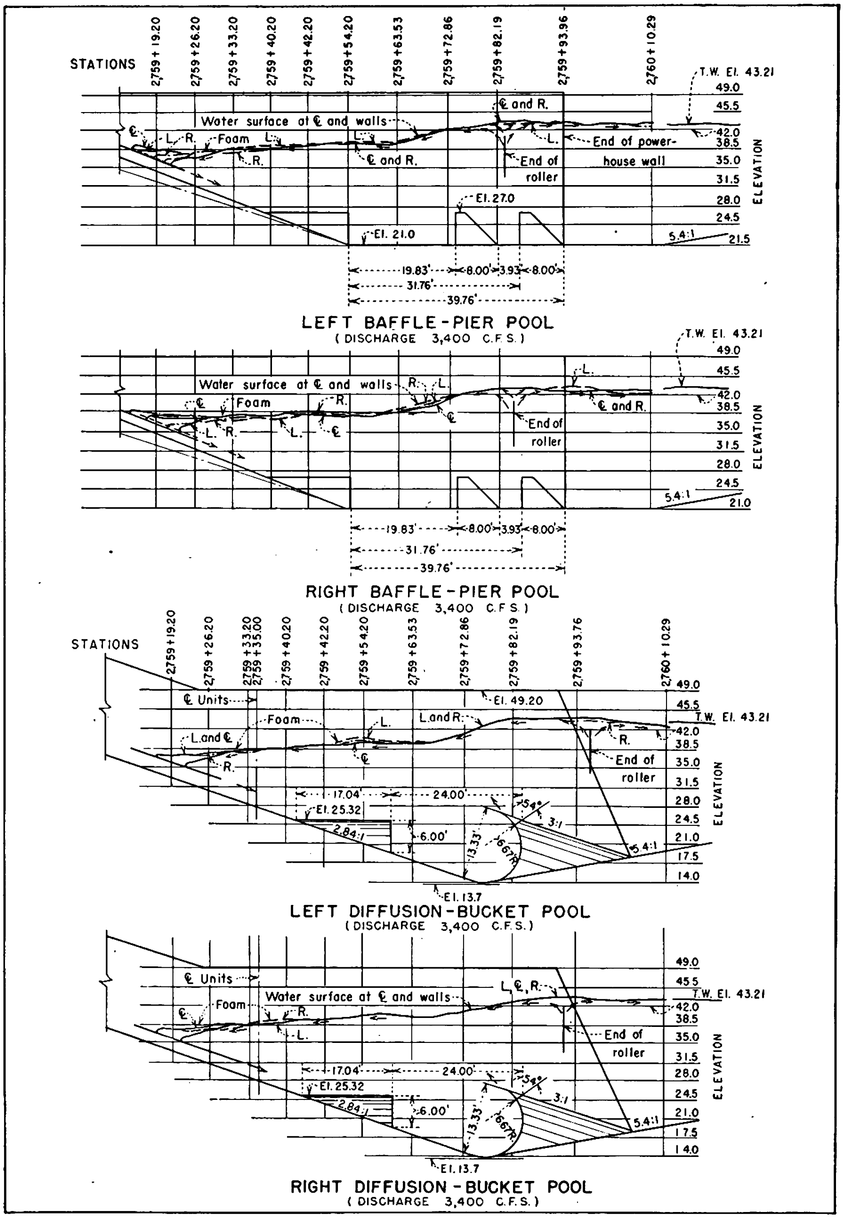


FIGURE 94.—WATER SURFACE PROFILES FOR BAFFLE-PIER AND DIFFUSION-BUCKET STILLING POOLS—DROP 4.

Unsymmetrical Operation.

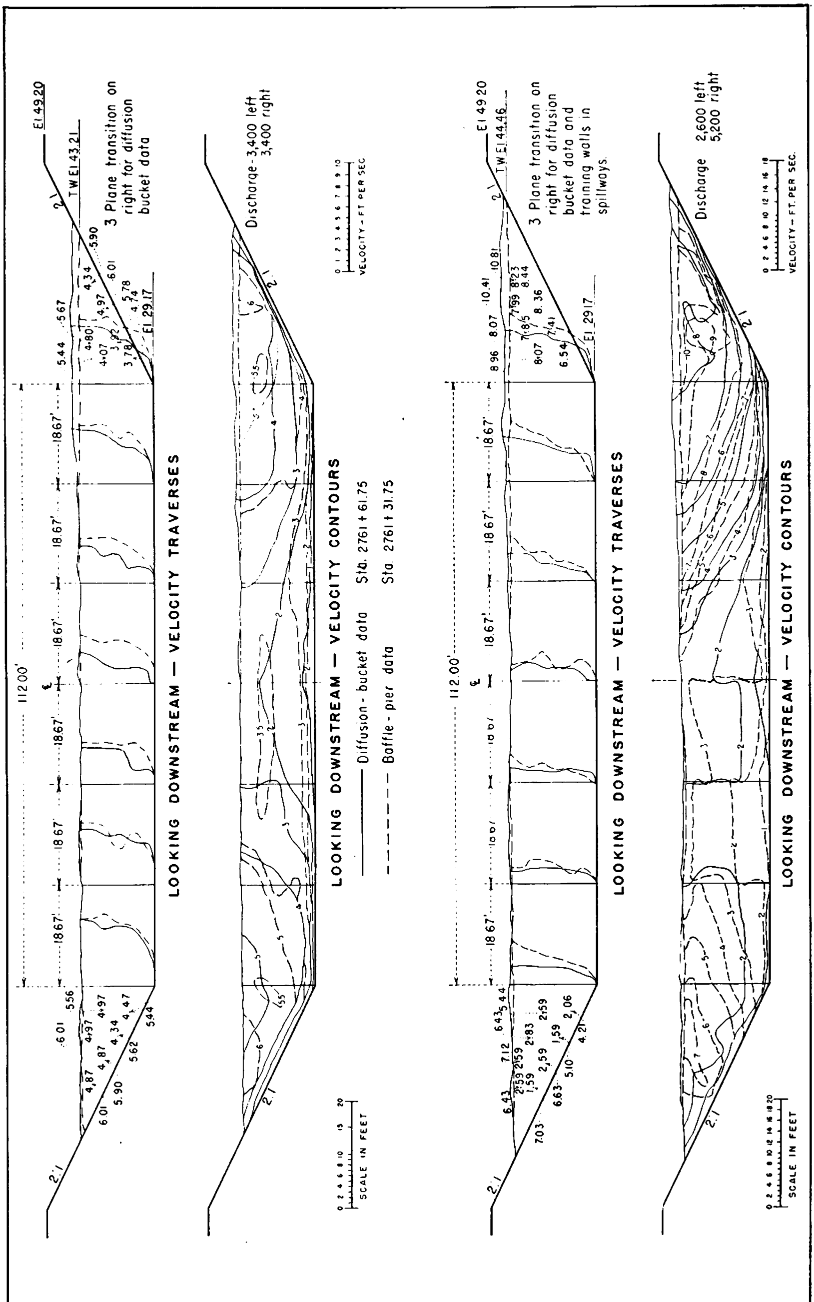
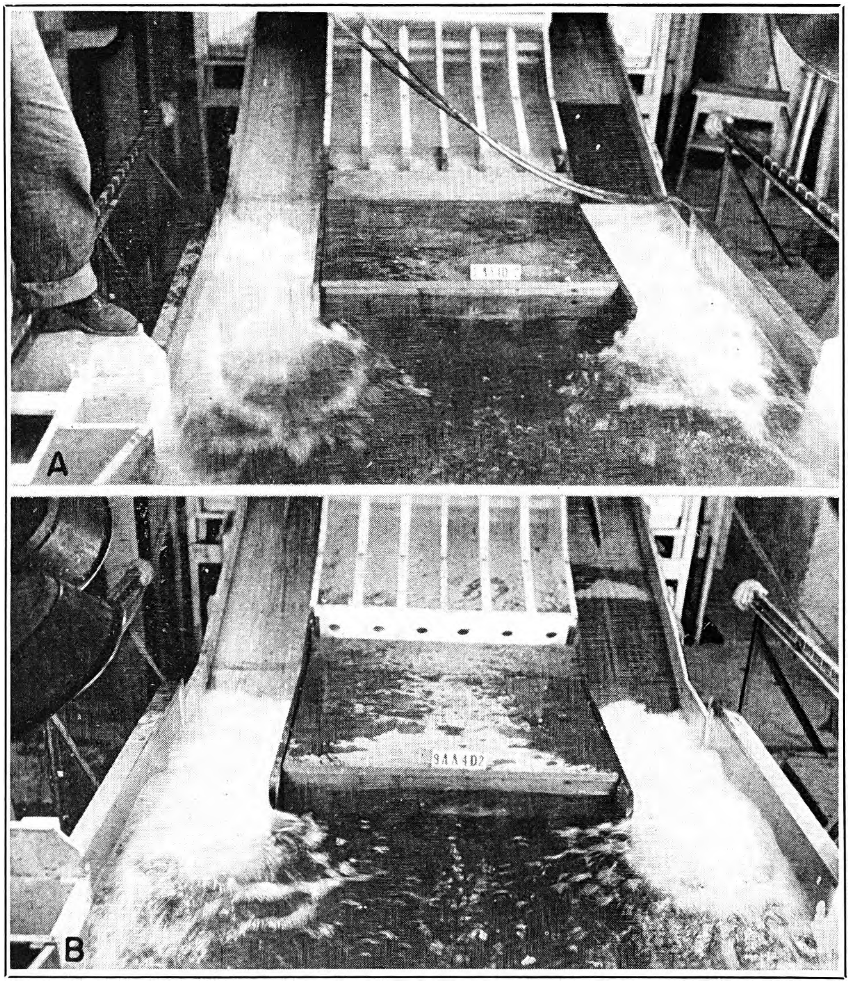


FIGURE 95.—COMPARISON OF VELOCITIES IN TWO TYPES OF STILLING POOLS—DROP 4.



A. Recommended Baffle-Pier Pool—Discharge of 3,400 Second-Feet Through Each Pool, Tailwater at Elevation 43.21. B. Final Design of Diffusion-Bucket Pool—Discharge of 3,400 Second-Feet Through Each Pool, Tailwater at Elevation 43.21.

FIGURE 96.—FLOW THROUGH RECOMMENDED AND FINAL DESIGN STILLING POOLS FOR DROP 4.

In previous tests, the diffusion bucket had been found preferable to other types of energy dissipators, hence tests were made to determine the suitability of a diffusion bucket for drop 5. The final design bucket of drop 2, see figure 97–B, was logically installed for tests in the model of drop 5, but pitot-tube measurements on this installation revealed excessive velocities at the downstream end of the pool. Velocities resulting from use of the bucket shown in figure 98 were satisfactory, hence that design was chosen as final. The velocities produced by the two buckets are shown in figure 99.

Some doubt existed relative to the effectiveness of the diffusion bucket should the sharp corners become eroded by action of sand in the prototype. A test of this foreseeable condition was made by rounding the sharp corners of the first bucket tested for drop 5, on a 4.5-inch radius. The results, see

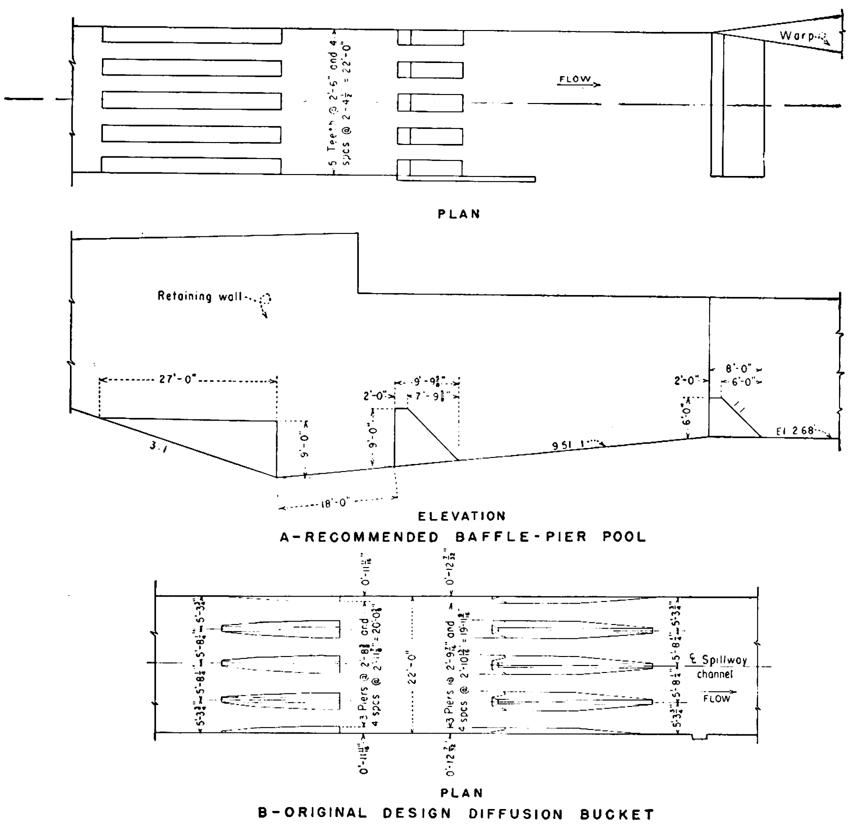


FIGURE 97.—STILLING POOLS FOR DROP 5.

figure 99, clearly illustrate that the rounded corners produced an average velocity increase of approximately 7½ percent.

98. Sill at Downstream End of a Stilling Pool.—Since time was available after completion of the study of the various drops, a comprehensive investigation was made of the best shape and size of sill for use as an energy dissipator at the downstream end of a stilling pool. Figure 100 shows the details of the best sill developed and the effect of the sill on the magnitude of the velocity.

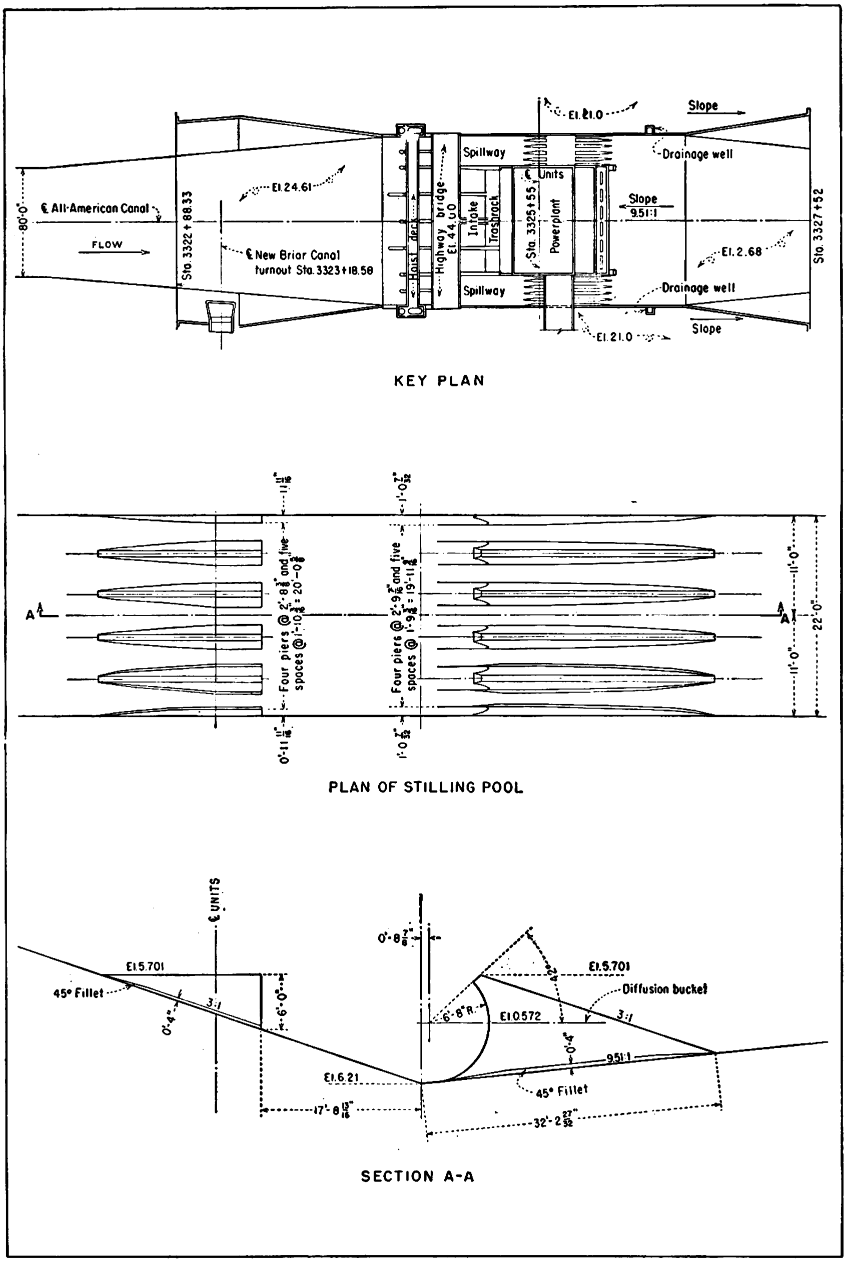


FIGURE 98.—FINAL DESIGN STILLING POOL FOR DROP 5.

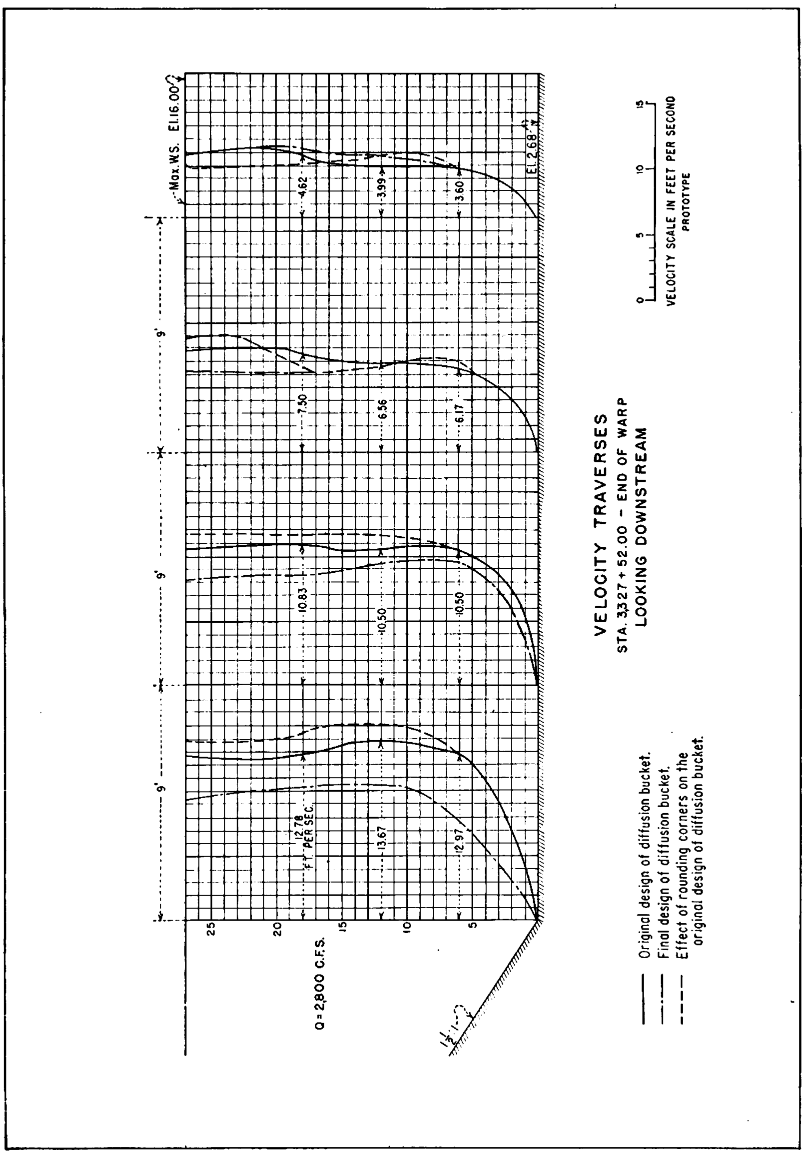


FIGURE 99.—VELOCITY DISTRIBUTION IN FINAL DESIGN POOL OF DROP 5.

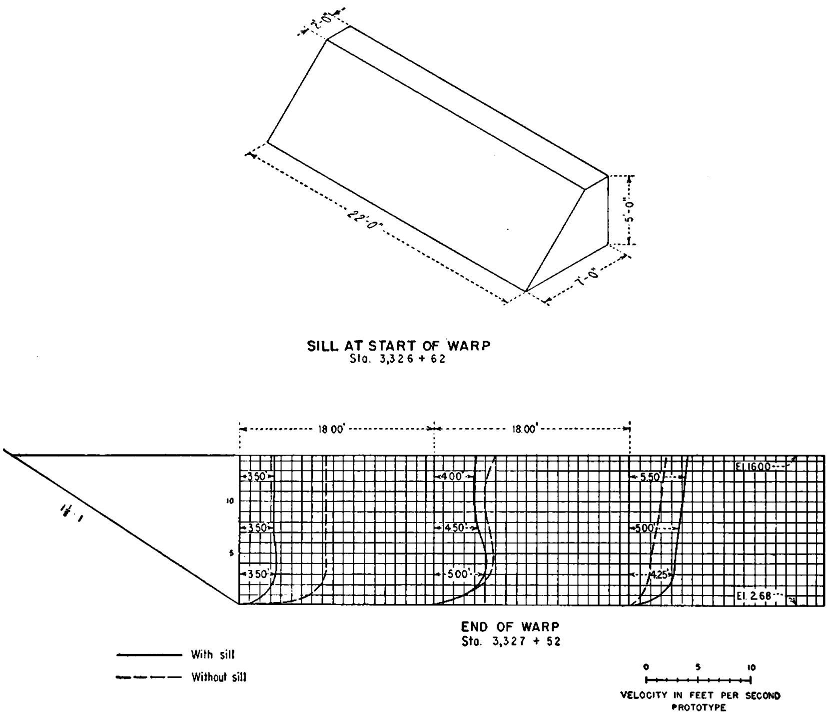


FIGURE 100.—EFFECT OF SILL AT DOWNSTREAM END OF STILLING POOL.

OVERCHUTES

99. Coachella Power Drop.—The original design of the Coachella power drop [10] called for a structure comparable in length to drop 2 of the All-American Canal. Because the discharge was only about one-quarter as large as in drop 2, it was felt that a less elaborate and shorter structure might be developed by model tests; a 1:24 scale model was therefore constructed. Figure 101–A shows the model of the original design of this structure. Tests were made on the model with a flow representing the design discharge of the prototype, 2,200 second-feet, distributed in various proportions between the powerhouse turbines and the spillways. This model performed satisfactorily except for an undesirable contraction in the flow around the piers beneath the bridge upstream from the powerhouse, which was corrected by changing the position of the piers.

It was believed that considerable saving in cost could be obtained by having the spillway all on one side of the powerhouse, since with this design one side of the powerhouse would replace a large section of side wall of the structure. A second model was therefore constructed incorporating

[10] Pomeroy, R. R., "Hydraulic Model Studies of Coachella Power Drop, Station 290+00, All-American Canal Project," Hyd. Lab. Report No. 102 (unpublished), Bur. of Recl., Jan. 31, 1942.

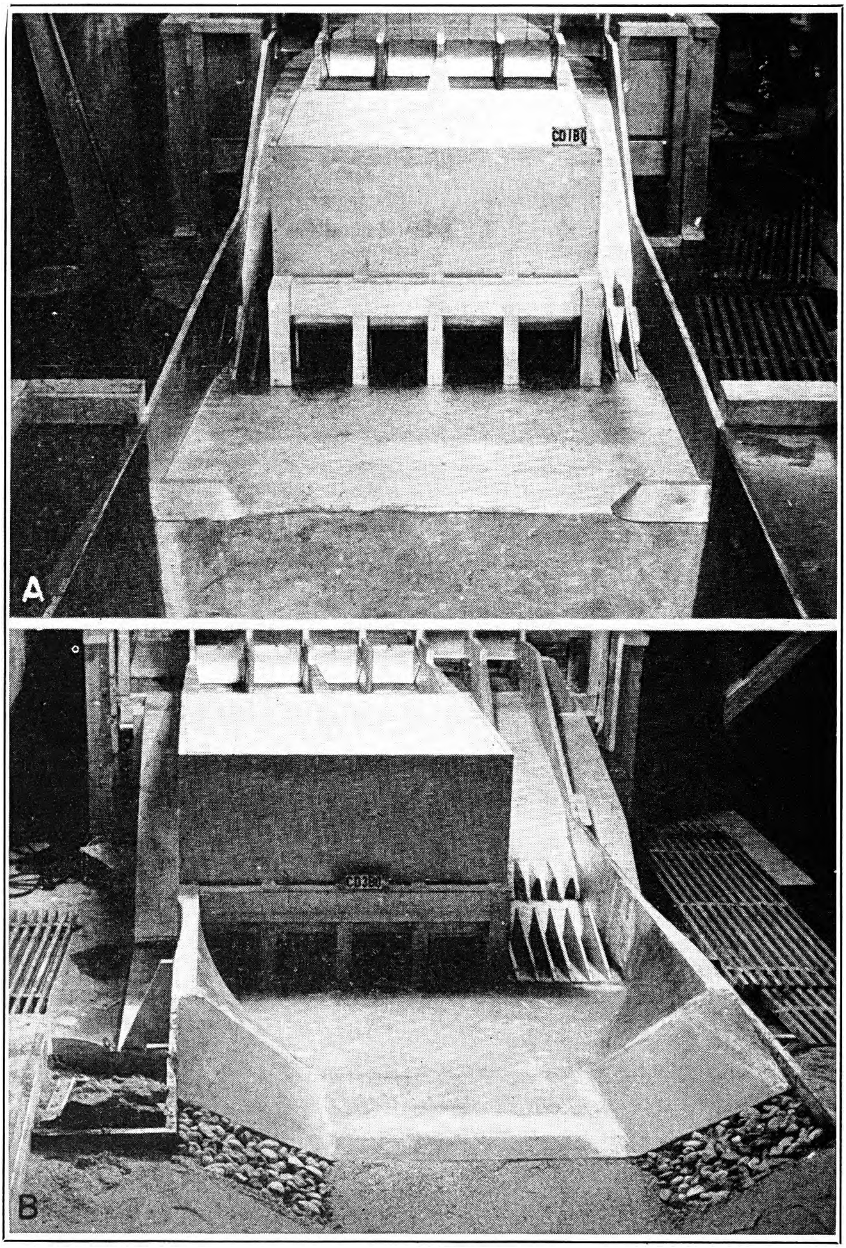
this feature, and was found to operate satisfactorily. A third model was also constructed, which had a total length of 350 feet as compared with 481 feet for the original structure and which included several other minor alterations, see figure 101–B. As the action of this structure was satisfactory, it was recommended, but the plan to develop power at this point was abandoned and the Coachella power drop was not constructed.

100. Cross Drains and Overchutes.—The All-American Canal, the Coachella Branch, and the Gila Canal are all crossed by a number of streams which drain the land on their uphill sides. Because of the very infrequent rainfall in the region these streams are normally dry, but occasionally a cloudburst or general rain occurs over the watersheds causing large discharges in these channels. It was necessary that provision be made for taking care of these discharges without interfering with the operation of the canal. The gradients of the streams are steep and the flows heavily laden with sediment. It was therefore impracticable to discharge them into the canal or to carry them underneath in inverted siphons, because of the objectionable deposits of sediment which would occur. The solution was therefore to carry each stream or wash over the canal, depressing the canal in an inverted siphon if necessary. Since it was expected that the flood discharges might reach as much as 20,500 second-feet, the magnitude and cost of these structures was considerable. Moreover, the problems involved in their design were unusual and difficult of solution. Extensive model tests were therefore conducted to insure satisfactory performance.

The conditions at these washes vary widely and the discharges cover a wide range. Because of the number of structures involved it was impractical to model each one, hence a series of typical cases only were investigated. From the general rules obtained from these tests, diagrams were prepared to cover a wide range of conditions. It was found that the structures could be divided into two important groups or cases: (1) Those in which the construction of the structure would cause degradation in the channel downstream, and (2) those in which no degration would occur. The solutions of the two cases were found to differ considerably.

The first detailed model tested was the overchute for the Gila Gravity Main Canal, located about 3 miles downstream from Imperial Dam, where nondegrading conditions exist. This model test was made applicable to a large variety of conditions by using different model scales and assuming that the action would be the same if the width and the discharge were altered proportionately.

The Gila crossing investigation was followed by tests of three typical crossings for the Coachella Canal. These were first investigated for the condition in which no degradation would occur. The importance and possibility of degradation at such structures were discovered during these tests, and studies were undertaken to perfect designs which would be safe for this condition also.



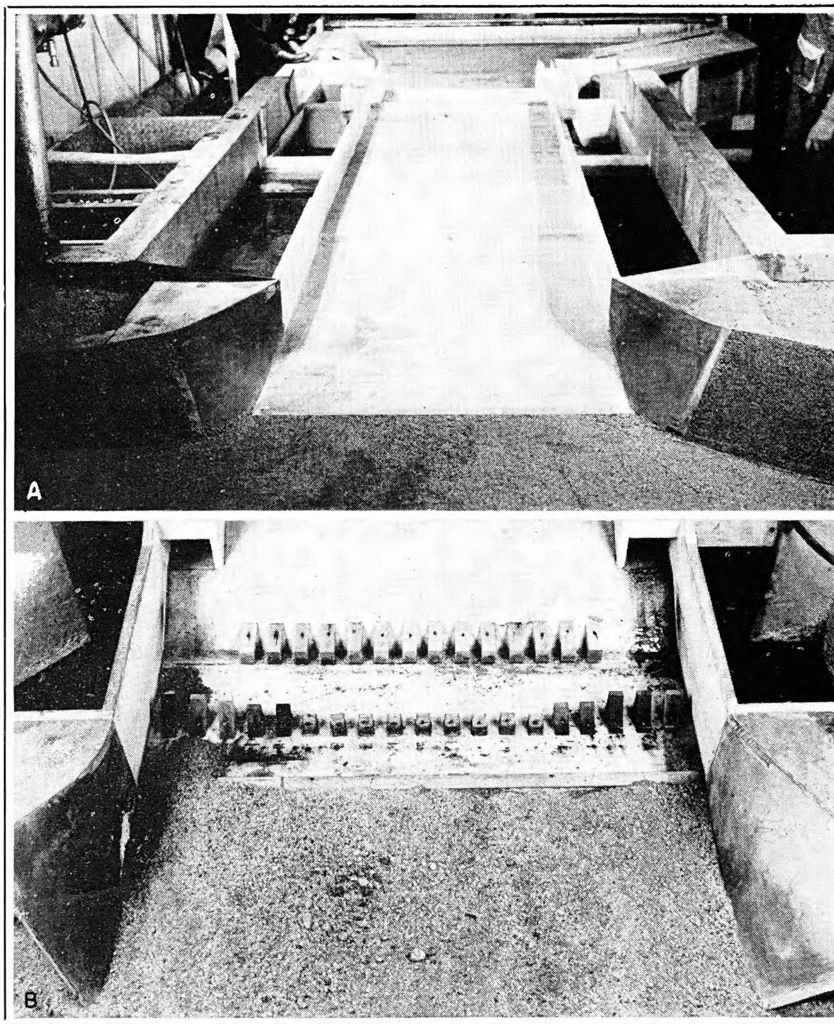
A. Original Design. B. Recommended Design.

FIGURE 101.—HYDRAULIC MODELS OF COACHELLA POWER DROP.

101. The Gila Canal Overchute.—The Gila Canal overchute [11] was designed to carry a flow of 2,500 second-feet. The structure was essentially a rectangular flume with an inlet transition on the upstream side of the canal and a concrete stilling basin on the downstream side. The first model was constructed on a scale of 1:18, but it was later decided to include in the same structure two automatically controlled wasteways, with a combined discharge capacity of 1,500 second-feet. Since the pump capacity available would not provide sufficient supply for this additional flow on the 1:18 model, a second model on a 1:24 scale was constructed. Figure 102–A shows the 1:24 scale model, viewed from upstream. The original design of the inlet transition proved to be satisfactory. The problem of the flow through the flume section was complicated by the use of sills on the floor. The purpose of the sills was to retain on the floor, after the flood, a layer of sediment which would reduce the temperature changes in the thin concrete floor. These temperature changes would otherwise be very severe because of the intense sunlight in that locality. Numerous heights and spacings of sills were tested to find the best design from the standpoint of flow conditions through the flume. Some combinations produced waves of considerable height, which tended to limit the flume capacity. It was found that 8-inch-square sills approximately 10 feet apart were most satisfactory.

The design of the stilling pool for this overchute was the subject of an extensive model study. A hydraulic jump was used, with stepped aprons, baffle piers, and sills, similar to those used in the stilling pools at the Siphon Drop and Alamo River crossing wasteways. The most efficient height of the stepped apron at the foot of the incoming slope, the size and shape of the baffle piers on the floor of the stilling basin, and the height and shape of the sill at the downstream end of the basin were all investigated. It was found that there was an optimum height of stepped apron, as the scour effect increased when the step size was increased above a certain height. Vertical-face baffle piers were found to be more effective than those with sloping faces, but were subject to greater impact from the flowing water. It was observed that these baffles and the sill at the end of the pool floor, if too high, received excessive impact and produced a large roller which caused severe scour on the bottom and sides of the canal. It was also found that the sill must be sufficiently low to prevent a drop in the water surface at any discharge, but that it should be high enough to deflect the flow above the river bed so as to form a ground roller beneath the stream. A ground roller consists of a bottom current in an upstream direction just downstream from the structure, which will move the bed material toward the end of the paving rather than away from it. There appeared to be no difference in effectiveness between the rectangular and trapezoidal sills of the same height, nor did the top width of sill seem to be important. The

[11] Douma, J. H., "Hydraulic Model Tests of Wash Overchutes and Wasteways, Gila Gravity Main Canal, Gila Valley Project," Hyd. Lab. Report No. 185 (unpublished), Bur. of Recl., Oct. 22, 1945.



A. 1:24 Scale Model. B. Wasteways Entering Stilling Pool.

FIGURE 102.—MODEL OF GILA CANAL OVERCHUTE.

height of the baffle piers was found to be of more importance than their location; however, if placed too far upstream they were found to receive excessive impact, and if placed too near the end sill they became less effective. The best location appeared to be about one-third of the floor length from the downstream end of the floor.

If no sills or baffles are employed in a stilling pool, the elevation of the pool floor may be computed by the hydraulic-jump formula, using the maximum discharge. However, tests showed that with use of sills and baffles, the depth of the pool for the Gila Canal overchute could safely be reduced 15 percent under that required to form the jump as computed by the formula. The length of the pool was set at that required to cover the full length of the jump at maximum discharge.

It was reasoned that since the model chute was the same width throughout and the depth of flow was uniform across it, the results could be applied to similar chutes of various widths, by designing on the basis of discharge per foot of width. Accordingly, by applying this principle and numerous model scale ratios, a general diagram was prepared, see figure 103, which is applicable to rectangular stilling pools under a wide range of conditions. However, these relations should be used only for low heads, where cavitation would not be produced.

In using figure 103, the following rules and relationships are applied:

1. The discharge per foot at the pool entrance is equal to the maximum discharge divided by the width at the pool entrance.
2. The theoretical depth, d_1, at the pool entrance is computed by dividing q by v_1.
3. The theoretical velocity, v_1, at the pool entrance is computed by the formula for flow in steep chutes given by King in his "Handbook of Hydraulics."
4. The theoretical jump depth, d_2, is computed from the momentum formula.
5. The experimental jump depth, d'_2, is equal to 85 per cent of d_2.
6. The required stilling-basin floor elevation is equal to the maximum discharge tailwater elevation minus d'_2.
7. The required stilling-basin length, L, is equal to $3d_2$.
8. The height, h_1, of the dentated step is equal to d_1, or $2d_2/9$, whichever is larger.
9. The height, h_2, of baffle piers is equal to $2d_2/9$.
10. The height, h_3, of solid end sill is equal to $d_2/9$.
11. The distance, a, from the end of the stilling basin to the vertical upstream face of the baffle piers is equal to $L/3$.
12. The top dimensions of the baffle piers and end sill parallel to the length of the basin are equal to $h_2/4$ and $h_3/4$ respectively.
13. The width of the teeth of the dentated step and the baffle piers is equal to h_1, and their spacing is equal to $2h_1$.
14. The teeth of the dentated apron and the baffle piers should be staggered, with no blocks or teeth against the side walls, and one more baffle pier than teeth in the dentated apron.
15. The back slopes of the baffle piers and end sill may be such as to be the most economical.
16. The slope of the downstream excavation may vary from horizontal to 1:6.
17. The chute slope entering the pool may vary from horizontal to 1:1.

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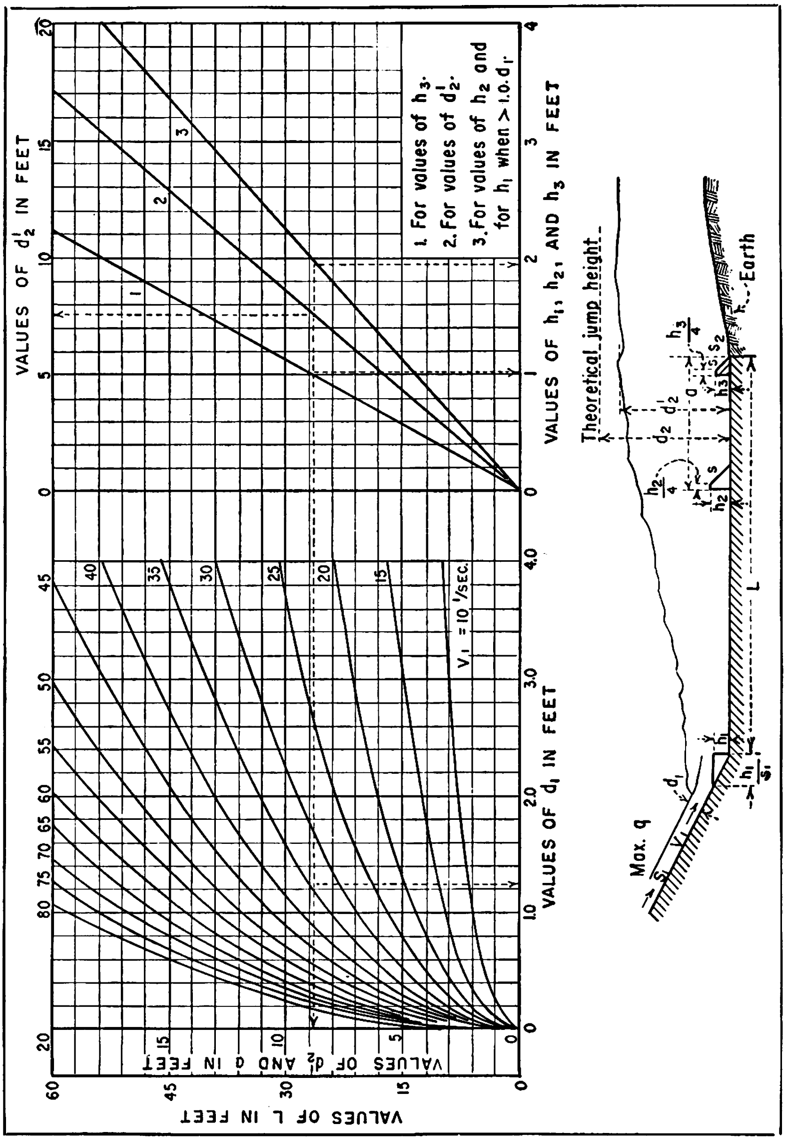

FIGURE 103.—STILLING-BASIN DESIGN CHART.

The foregoing diagram and rules apply only when the inflowing water is uniformly distributed across the width of the pool.

Two wasteways enter at the sides of the Gila Canal overchute stilling pool, see figure 102–B. It was reasoned that since the flow through these structures as well as the overchute would be intermittent and uncontrolled, the flow in the prototype stilling pool would most probably be distributed unequally, and without special provision the resulting hydraulic jump would be unsatisfactory. One solution considered was to have separate stilling pools for the gates and overchute. This would require a lower floor in the wasteway pools to produce the hydraulic jump, and dividing walls between the pools, both of which would be expensive. After extensive investigation, involving about 60 different tests, a solution was developed which did not require a lowered floor for the wasteway sluices. It consisted of higher baffle piers with no stepped apron in front of the sluice outlets, as shown in figure 102–B. The three blocks nearest the sides were increased in height to 4 feet, the next two to 3 feet, and the remainder were unchanged in height at 2 feet with their front faces 12 feet upstream from the end of the concrete floor. The stepped apron in front of the overchute was raised to a height of 3 feet, while the end sill was unchanged. With this solution a satisfactory action of the stilling pool resulted under all combinations of flow.

102. Studies for the Coachella Overchutes.—Since there were about 50 overchute structures along the Coachella Branch Canal, it was impractical to model each structure, but the importance of working out the proper design for each of them was very great.[12] The structures were therefore divided into three general types, and detailed studies were made on one typical structure of each type. The results of each study, with minor changes as required, were then applied to all structures within that classification. The three cases studied were as follows: (1) A comparatively narrow structure for small washes, with concrete sides and bottom and with a contracting entrance section and expanding outlet section formed of rock-fill banks, the canal passing beneath the overchute in a siphon of trapezoidal cross section as shown in figure 104; (2) a wide structure for one or more large washes, consisting of paved side walls, inlet contraction and outlet expansion, but with unpaved bottom, the canal passing beneath in a rectangular, double-barrel siphon as shown in figure 105; (3) a wide structure for large flows, with unpaved bottom and with training walls of earth embankment protected by riprap, crossing over a round, single-barrel siphon deeply buried and protected by riprap and sheet piling. In all cases studied the width of the opening through which the water would flow was considerably less than the width of the wash, because the cost of a siphon wide enough to cover the full wash width would have been prohibitive.

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[12] Locher, Fred, "Model Studies of the Coachella Canal Wash Overchutes, All-American Canal System, California," Hyd. Lab. Report No. 111 (unpublished), Bur. of Recl., April 20, 1942.

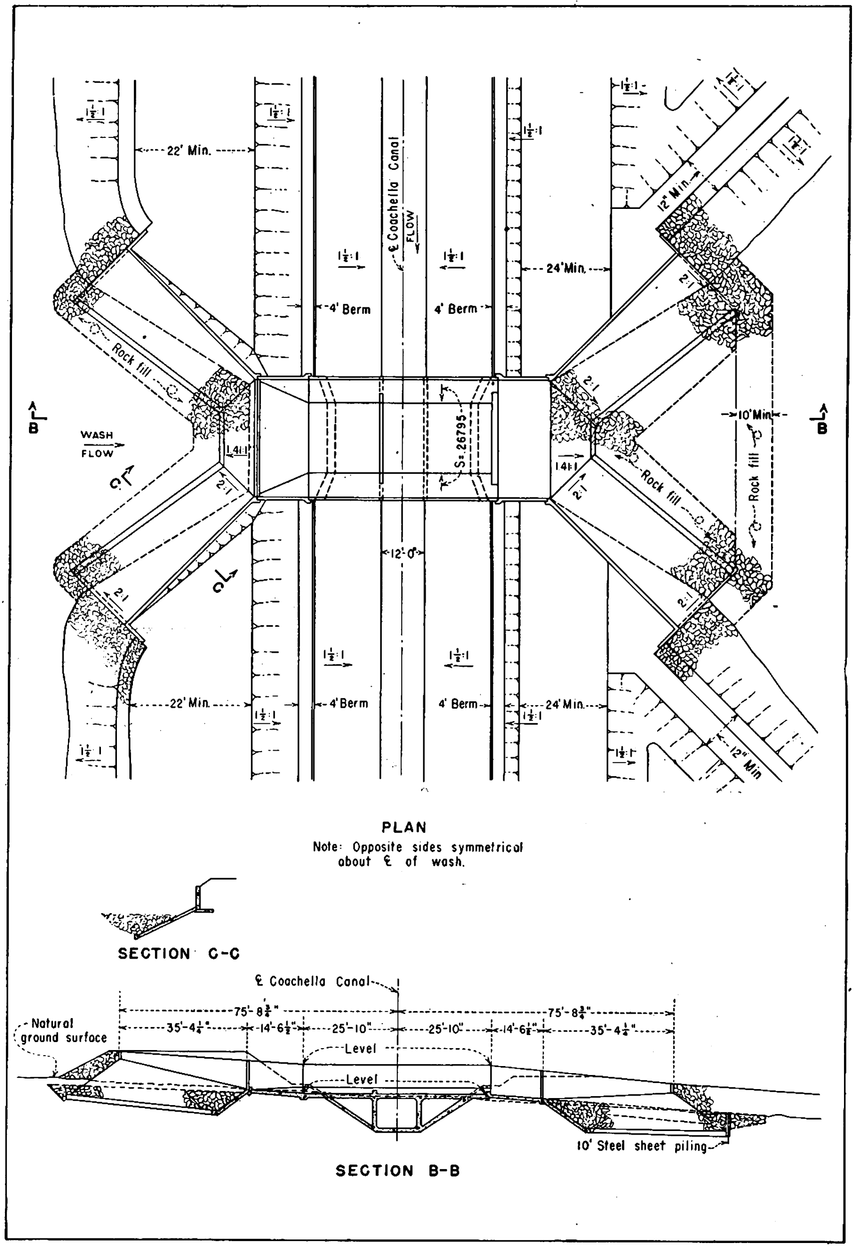


FIGURE 104.—ORIGINAL DESIGN OF NARROW COACHELLA OVERCHUTE.

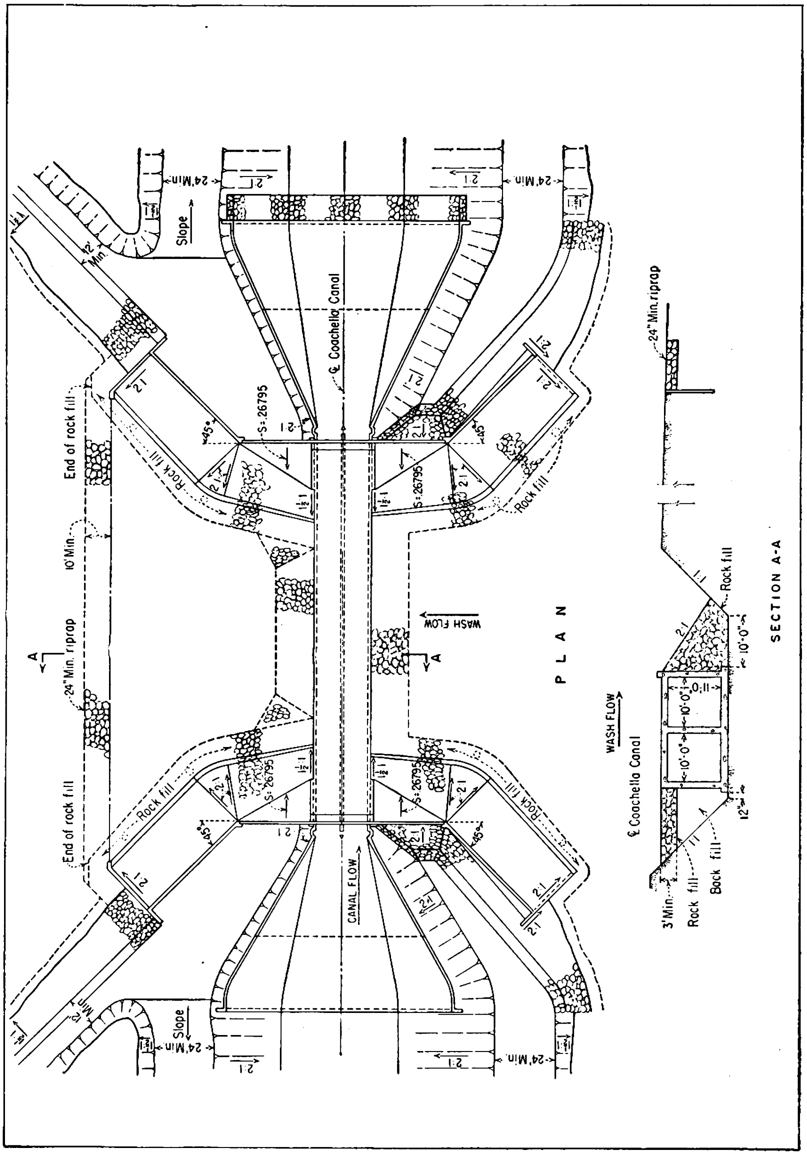


FIGURE 105.—ORIGINAL DESIGN OF A WIDE OVERCHUTE FOR COACHELLA CANAL.

The original design of the typical narrow overchute, case 1, see figure 104, showed excessive scour for all values of wash slope tested, because of the concentration of flow produced. Following the lessons learned in the Fortuna wasteway, an attempt was made to spread the stream after it passed over the canal siphon, by building two cross walls in the expanding section to form a double stilling pool, as shown in figure 106–A. This form proved reasonably satisfactory for cases where the canal bottom downstream from the overchute did not degrade. For the wide overchute with rectangular siphon, case 2, the original design also proved to be inadequate. The double-jump type of pool developed for the narrow overchute was investigated, but the results were unsatisfactory. Continued trial eventually evolved the form shown in figure 106–B, which showed very little scour for nondegrading conditions. A little study was given to case 3, the round-barrel siphon, but no satisfactory solution was reached. This was not important, since the design was an alternative for case 2 and the latter could therefore be substituted.

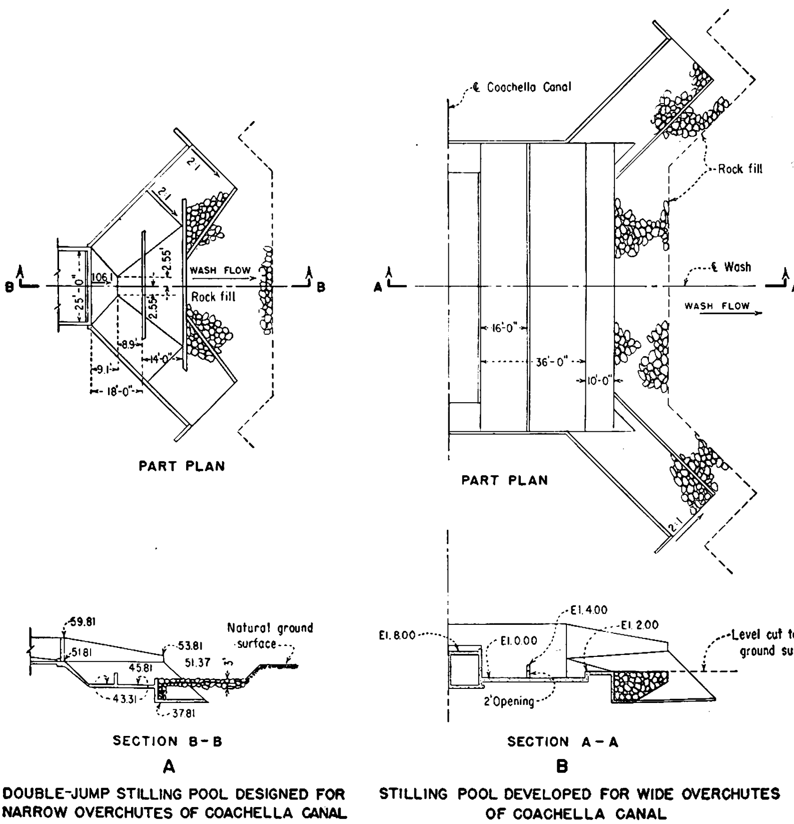


FIGURE 106.—RECOMMENDED DESIGN OF STILLING POOLS FOR OVERCHUTES OF COACHELLA CANAL.

103. Development of Overchutes to Withstand Degradation.—In the experiments leading to the development of the overchutes for the Coachella Canal previously discussed, it became evident that serious effects would result on the stability of the structures if degradation were to take place. Furthermore, a study of conditions along the canal led to the conclusion that degradation would likely occur at many of the structures.

The degrading of the stream bed downstream from an overchute may result from one of two causes, or from a combination of both. The simplest cause is the contraction of the flow into a narrower channel by the overchute. The washes crossing the Coachella Canal are generally very wide and the water flows over them in a shallow sheet. As previously mentioned, for economic reasons the width of each overchute was made less than the width of the wash. During flood flow the discharge per unit width through the overchute will therefore be much greater than it would have been in the wash had no overchute been built. The greater unit discharge through the overchute will have more transporting power than the normal flow in the wash, and will therefore carry away material from the bottom of the channel downstream from the chute, causing the channel to degrade. The depth of degradation is difficult to estimate in advance, but would depend upon the slope of the wash and the distance downstream to the point where the flow would resume its normal width; or to a control such as a rock ledge or lake, which would prevent further degrading at that point. Degrading from this cause may also occur in the narrow section of an overchute unless restrained by some form of bottom protection, and possibly to a limited extent in the upstream entrance. A further increase in the discharge per unit width over that obtained by narrowing the flow from a single wash may occur if the flows from more than one wash are combined. In a number of cases diversion ditches were designed to bring together several washes, in order to reduce the number of structures required. As explained above, this will further constrict the flow, aggravating the tendency to degradation.

Another cause of degrading is the holding back of material carried down by the stream. If the water entering an overchute is retarded, it will deposit part of the material it carries, and, after passing the retarding section, will again pick up a full load of material. If the construction of the entrance to the overchute is such that the water upstream is retarded, part of the material it carries will be deposited; and downstream from the structure a new load will be picked up, equal to that deposited. Thus material will be carried away from the stream bottom, causing it to degrade. Since the washes crossed by the Coachella Canal are steep, during floods they may carry considerable loads of solid material; thus degrading may take place rapidly and large volumes of material may be removed.

Because of the probability that degrading would occur at many of the Coachella Canal overchutes, and since the depth to which the bottom would be lowered was uncertain, it was necessary to design a type of structure which would be safe under such conditions. To function satisfactorily the type of structure which uses a stilling pool requires a tailwater level that is reasonably constant for a given discharge, hence this type of structure could not be used. The device evolved consisted of a sloping chute which could be extended as the stream bottom degraded, with closely spaced baffles on the chute floor to dissipate a large part of the energy of the falling water as it passed down the chute.

The first models of this type consisted of a stepped floor on a 1.5:1 slope, see figure 107–A. With this device the dissipation of energy was not sufficiently complete, and the slope was changed to 3:1, as shown in figure 107–B, with staisfactory results. With so flat a slope, however, a very long structure would be required to extend to the depth to which the stream might be expected to degrade. A 2:1 slope provided with closely spaced baffles, see figure 107–C, was constructed and produced very satisfactory results.

To investigate this device more thoroughly a 1:15 scale model of a section of such a chute was constructed and tested as shown in figure 108–A. For the design discharge of 35 second-feet per foot of width, very little scour resulted at the foot of the slope. The baffles were 3 feet high and 4 feet 6 inches long, and were placed in staggered rows 6 feet apart. The top thickness of the baffles was 1 foot and the bottom thickness 2 feet 6 inches. Piezometric measurements indicated an average pressure of 7.5 feet of water on the baffle face.

This form of energy dissipator was applied both to the narrow overchutes, a model of one of which is shown in figure 109, and the wide overchutes. Experiments were also conducted to investigate its application to several overchutes previously constructed, which were modified as described below. A sloping floor with baffles was placed in the center of the downstream side of each chute, but it was considered impractical to build the baffles on the already completed sloping floor near the sides. An arrangement of baffles, with a low training wall at the end of the first three rows of baffles, was devised. This arrangement may be seen at the right in figure 108–B. The part of the model shown at the left of the photograph was constructed with a vertical wall of the form designed for the new structures for the wide overpasses. A comparison of the flow on the two sides of the structure may be seen in figure 108–C. The resulting scour in figure 108–B is a measure of the effectiveness of the method devised to modify existing structures and of that devised for new structures. It will be seen that the scour at the foot of a modified structure would be more severe than at the foot of a structure based on the new design, but that scour conditions at the modified structure would still be reasonably satisfactory.

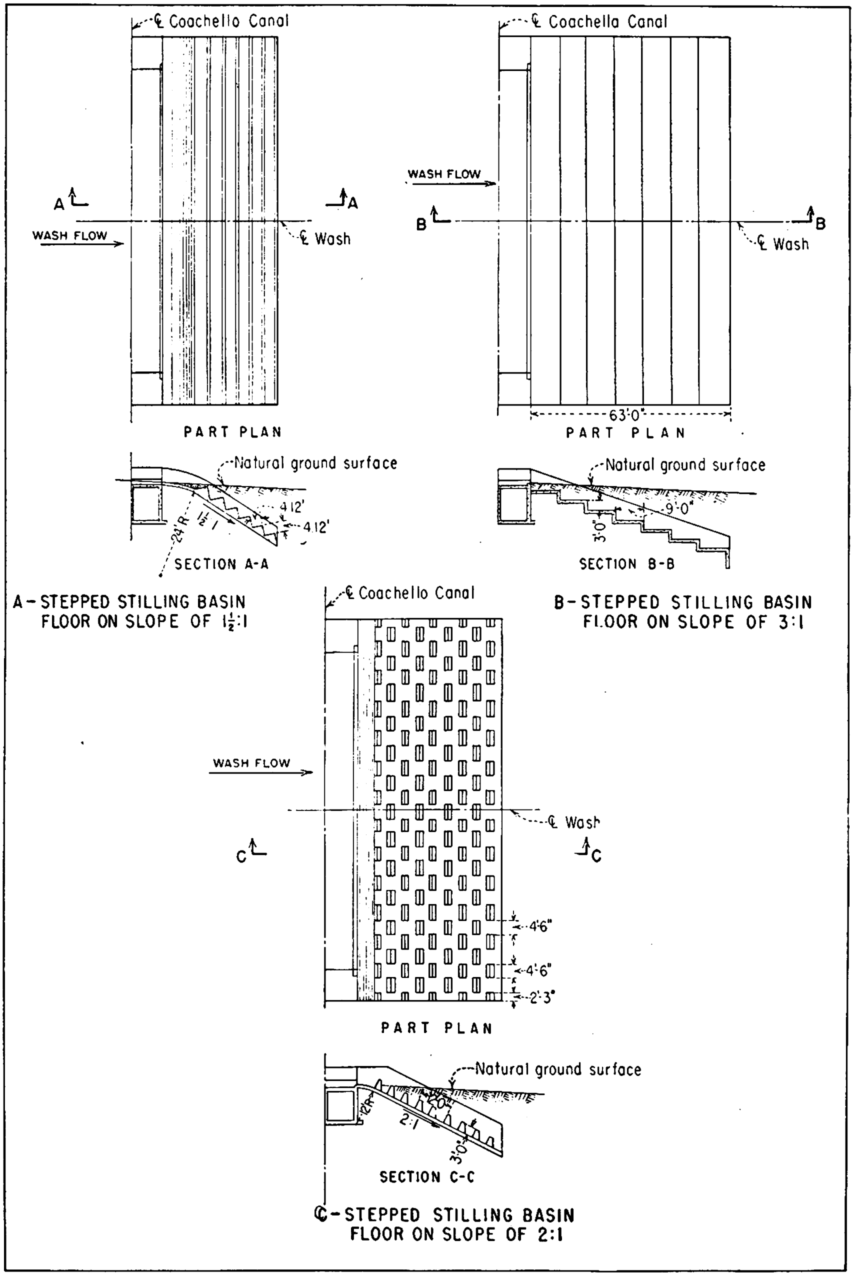
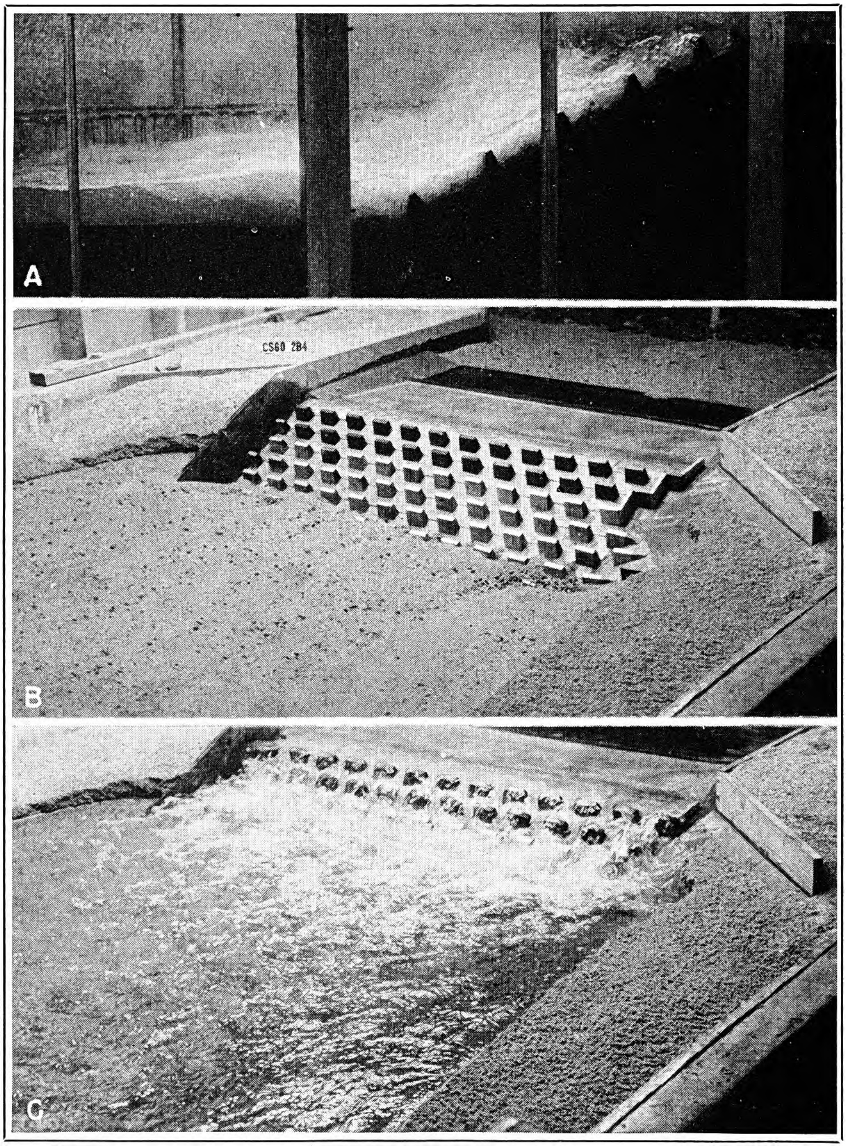


FIGURE 107.—TYPE OF OVERCHUTE INVESTIGATED TO WITHSTAND DEGRADATION—COACHELLA CANAL.



A. Baffles on Sloping Floor. B. Energy Dissipator for Existing Overchutes. C. Flow Conditions at Designed Capacity.

FIGURE 108.—TYPES OF PROTECTION FOR OVERCHUTES TO WITHSTAND DEGRADATION.

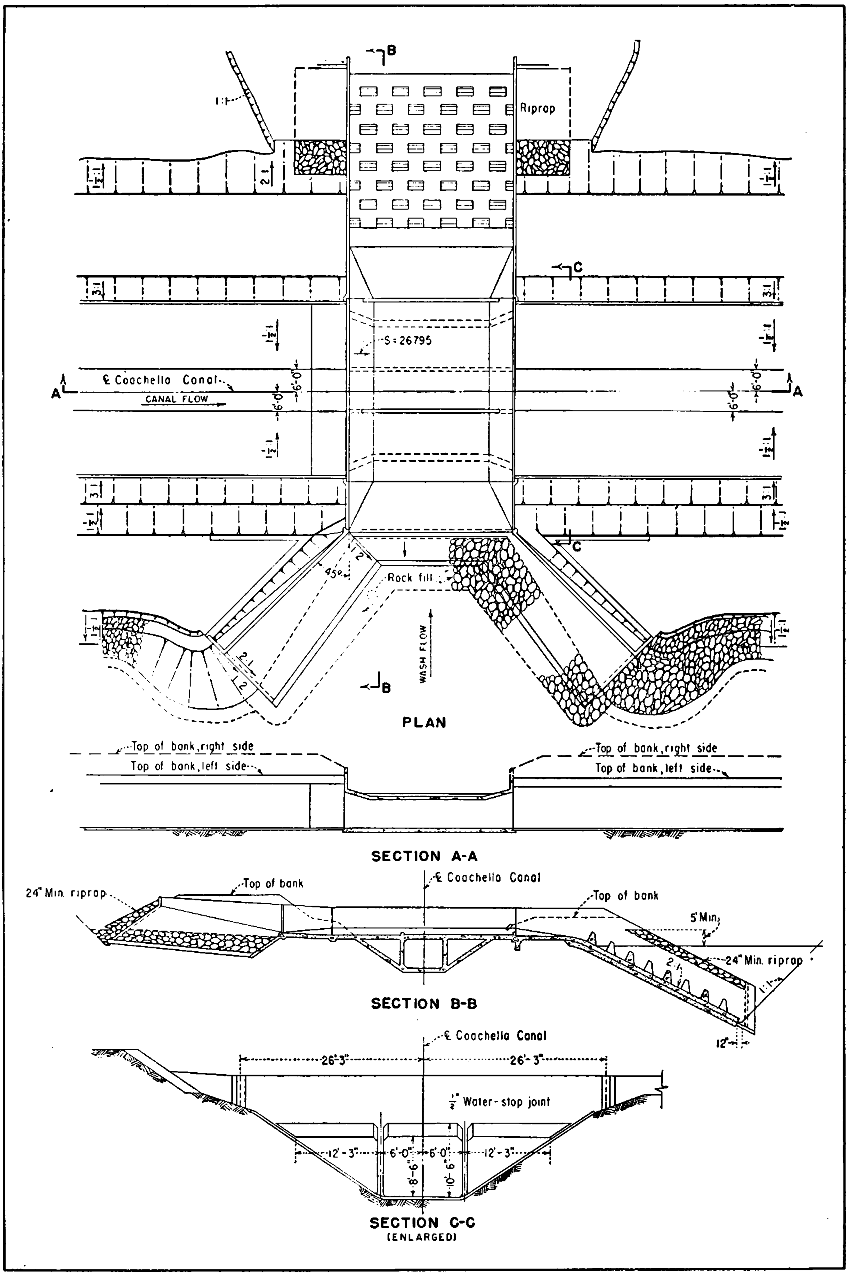


FIGURE 109.—RECOMMENDED STILLING POOL FOR COACHELLA CANAL OVERCHUTES DESIGNED TO WITHSTAND DEGRADATION.

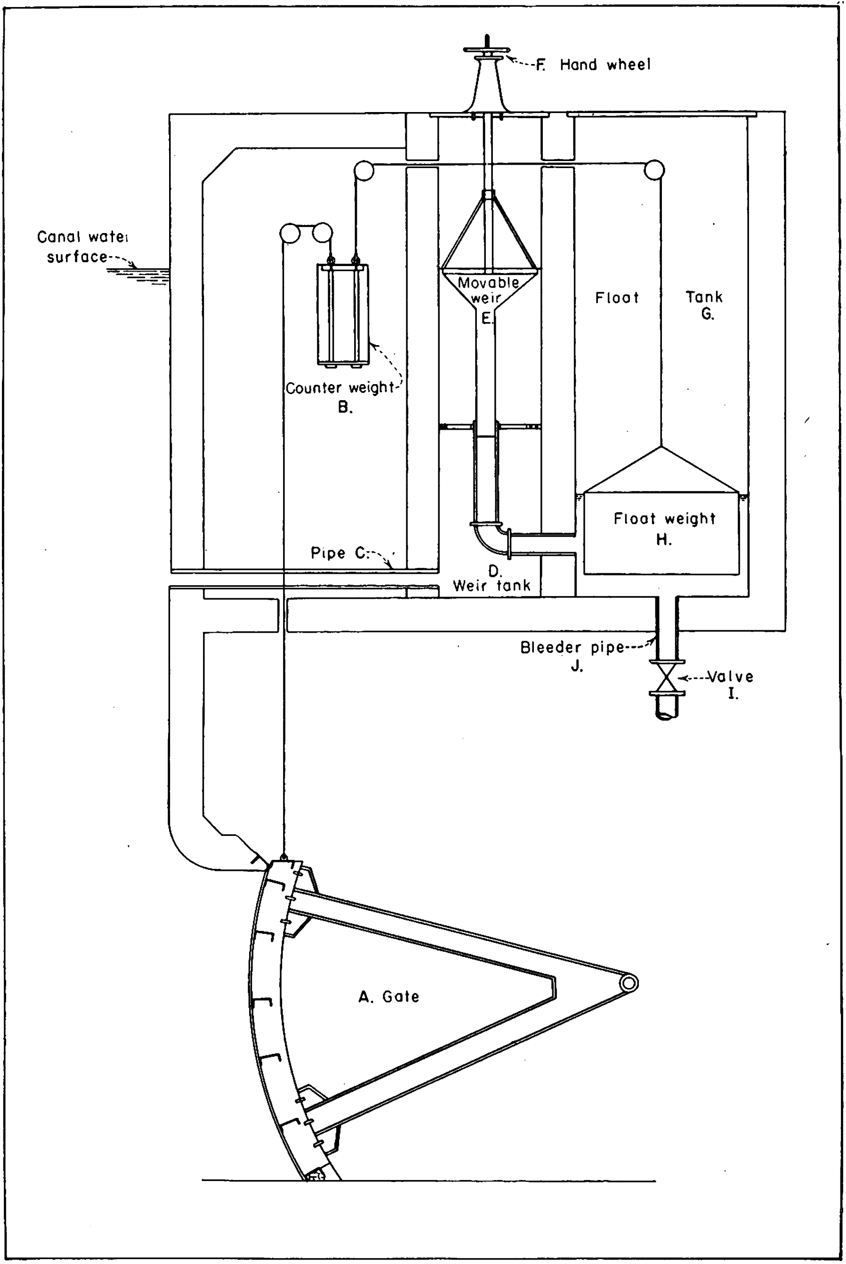


FIGURE 110.—DIAGRAM OF AUTOMATIC REGULATOR GATE FOR ALL-AMERICAN CANAL.

REGULATING GATES

104. Automatic Regulator Gates.—At a number of places along the All-American Canal it was desirable to install automatic devices for regulating the water levels in the canal,[13] to prevent the water from rising due to any cause and overflowing the banks, possibly causing them to break and disrupt the water supply to a large irrigated area. Since it was possible that an outside source of energy would fail, it was desirable to provide the energy from the water in the canal, which would always be present if the canal was in any danger of overflowing.

The device evolved is shown diagrammatically in figure 110. It consists of a sector gate, A, the weight of which is balanced by a counterweight, B, which is just heavy enough to raise the sector gate. From the canal a pipe, C, leads to a weir tank, D, containing a movable weir, E, which can be raised or lowered to a desired elevation by the handwheel, F. The discharge over the weir passes down a pipe leading to a float tank, G, in which is located a heavy float, H, having considerable weight but not as much as the counterweight. This float is connected to the counterweight by a cable so arranged that when the float sinks it pulls upward on the counterweight. From the float tank the water discharges through a bleeder pipe, J, into the canal downstream from the control. The crest of the movable weir is set slightly below the elevation of the water surface which it is desired to hold in the canal. When the water in the canal rises slightly above the weir crest elevation it overflows and discharges into the float tank, which it fills to a level sufficient to cause a discharge from the bleeder pipe equal to the inflow. If the water in the canal continues to rise, however, the float tank fills until it raises the float, allowing the counterweight to fall and open the sector gate, which tends to lower the water level in the canal. As the canal water level lowers, the flow over the weir decreases, the water level in the float tank drops, and the gate lowers. If the water level in the canal changes slowly, the gate will adjust itself to the discharge which will tend to hold the water level in the canal at a constant elevation. By proper adjustment of valve I, in the bleeder pipe, elevations of the movable weir can be found which will correspond to various water levels maintained in the canal. When the weir is set at an elevation corresponding to the canal level desired, the device will hold the water in the canal very close to that desired level.

In order to study the practicability of such a device, and to perfect it, a model of one of the 6-foot 6-inch by 7-foot 6-inch gates of the New River crossing was constructed on a scale of 1:6. The model was made of galvanized iron and steel. An effort was made to reduce friction to a minimum. On account of space limitations the model was first installed in a flume with only 35 square feet of surface area. Under these conditions the water levels

[13] Streeter, V. L., "Hydraulic Model Studies of New River Crossing Automatic Gate," Memo. to the Chief Designing Engineer (unpublished), Bur. of Recl, April 20, 1939.

in the weir and float chambers could not adjust themselves quickly enough, resulting in considerable fluctuation of the water surface before a constant level was established. The model was later connected with a flume of 1,100 square feet area, which corresponded more nearly with conditions in the prototype.

The tests revealed that successful operation of this type of regulator gate depends on observance of the following general conditions:

1. The float should be large enough to respond to small changes in float-chamber water surface elevation. (The size of the float depends upon the weight of the gate and the anticipated friction.)
2. The inlet to the weir, and the passage from the weir to the float chamber, should be designed such that the effect of a change in canal water surface is transmitted to the float chamber as soon as possible.
3. The bleeder outlet should be large and should be provided with a valve so that the opening can be adjusted.
4. The seal friction should be kept at a minimum. The greater the friction the greater the tendency of the gate to "hunt."

LIST OF BULLETINS

The following list shows title and prices of the final reports on the Boulder Canyon project which have been published and are now available for distribution. The list also shows revised titles of those reports which are now planned for publication. Certain of the reports heretofore included in the tentative list of those to be printed have been omitted and the titles of certain others revised.

PART I—INTRODUCTORY

General History and Description of Project
The Financing of the Boulder Canyon Project

PART II—HYDROLOGY

(Not to be published)

PART III—PREPARATORY EXAMINATIONS

Geological Investigations

PART IV—DESIGN AND CONSTRUCTION

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Diversion, Outlet, Spillway, and Structures* (paper, $2.50; cloth, $3.00)
Concrete Manufacture, Handling, and Control* (paper, $2.50; cloth, $3.00)
Penstocks and Outlet Pipes
Imperial Dam and Desilting Works

PART V—TECHNICAL INVESTIGATIONS

Trial Load Method of Analyzing Arch Dams* (paper, $1.50; cloth, $2.00)
Slab Analogy Experiments* (paper, $1.00; cloth, $1.50)
Model Tests of Boulder Dam* (paper, $1.50; cloth, $2.00)
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