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UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

BOULDER CANYON PROJECT
FINAL REPORTS

PART VI—HYDRAULIC INVESTIGATIONS

Bulletin 4

MODEL STUDIES OF SPILLWAYS

UNITED STATES
DEPARTMENT OF THE INTERIOR
HAROLD L. ICKES, Secretary

BUREAU OF RECLAMATION
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BOULDER CANYON PROJECT
FINAL REPORTS

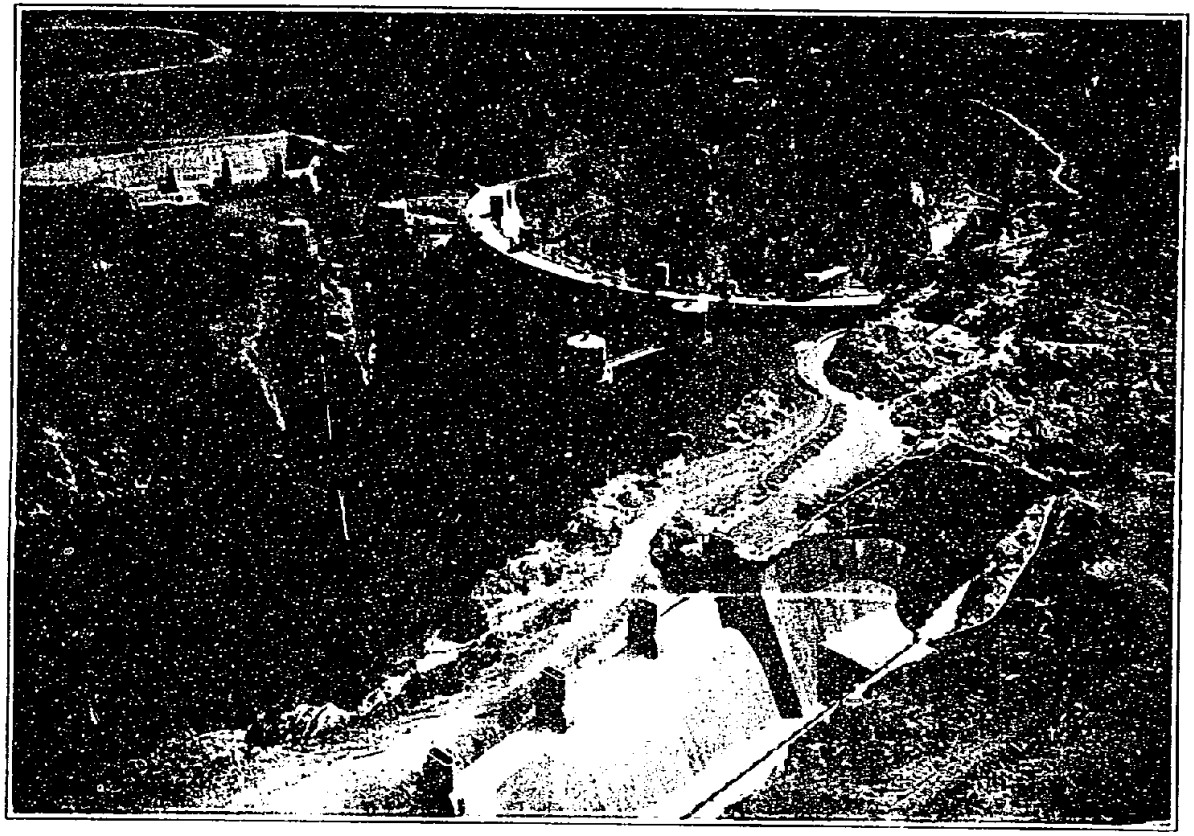
PART VI—HYDRAULIC INVESTIGATIONS

Bulletin 1

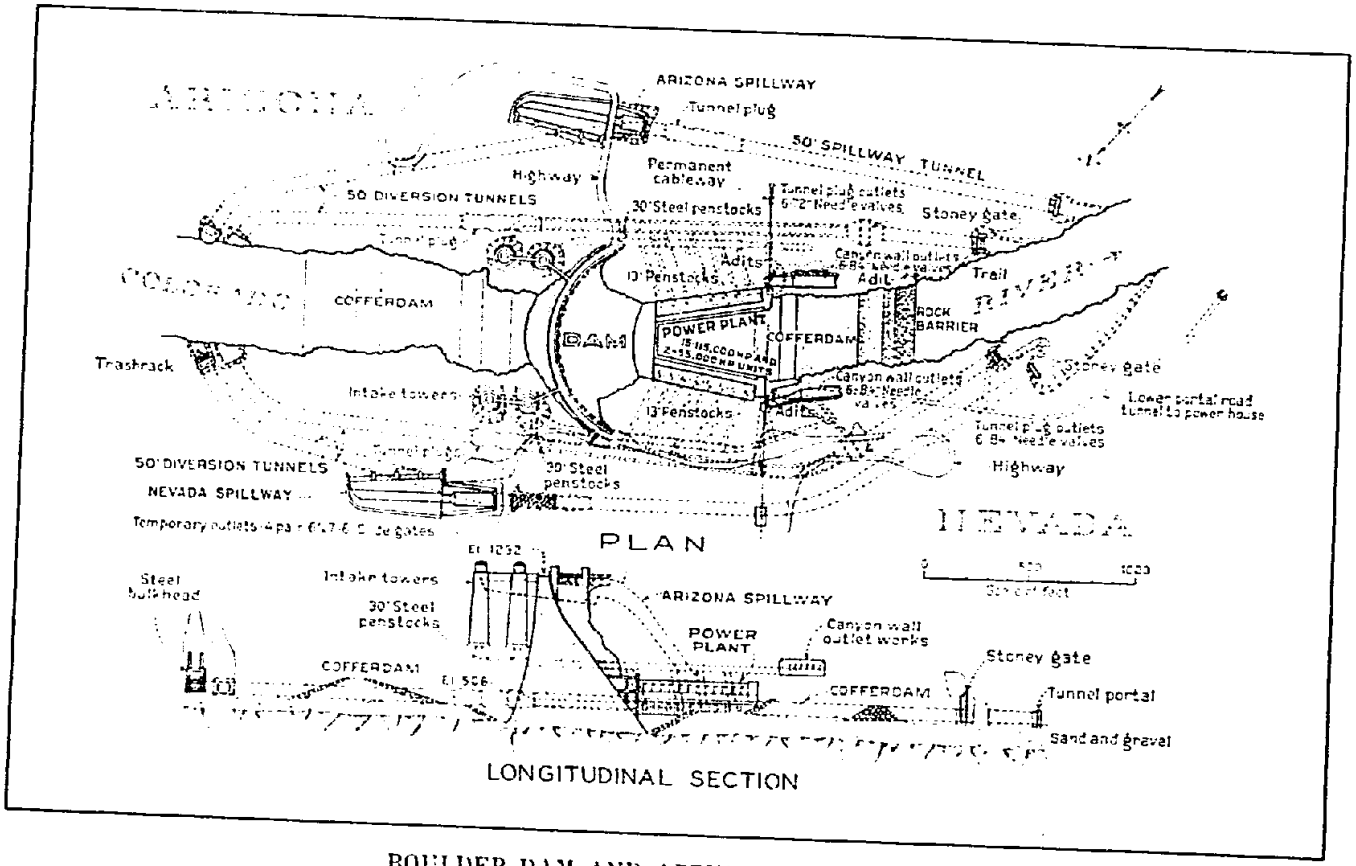
MODEL STUDIES OF SPILLWAYS

DENVER, COLORADO
1938

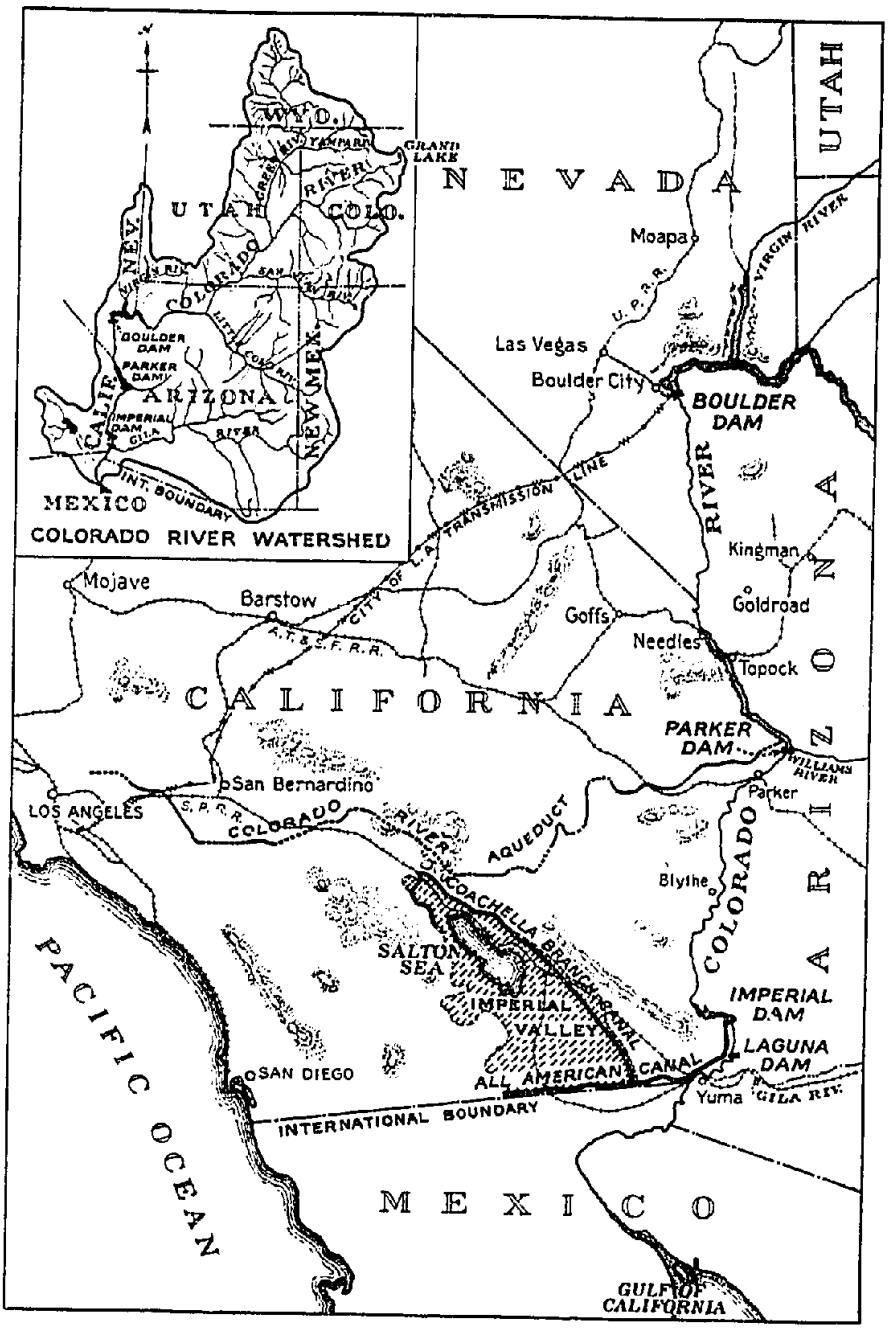
This bulletin is one of a series prepared to record the history of the Boulder Canyon Project, the results of technical studies and experimental investigations, and the more unusual features of design and construction. A list of the bulletins available and tentatively proposed for publication is given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



FRONTISPIECE—BOULDER DAM FROM NEVADA SIDE SHOWING INTAKE TOWERS AND SPILLWAYS



BOULDER DAM AND APPURTENANT WORKS



BOULDER CANYON PROJECT—LOCATION MAP

BOULDER CANYON PROJECT

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John L. Savage ..... Chief Designing Engineer
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William H. Nalder ..... Assistant Chief Designing Engineer
Byram W. Steele ..... Civil Engineer-Dams
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Charles M. Day* ..... Mechanical Engineer
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Gordon B. Kaufmann ..... Architectural Features

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* Deceased.

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 Boulder 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 lands 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. It was constructed and is being operated under the supervision of the Bureau of Reclamation, United States Department of the Interior.

Boulder Dam is located on the Nevada-Arizona boundary near Las Vegas, Nevada, at a place where Colorado River has carved a deep gorge between towering rock cliffs, known as Black Canyon. The dam is a concrete arch gravity structure with a maximum height of 726 feet above foundation rock, a maximum base thickness of 660 feet, and a crest length of 1,244 feet. The dam and appurtenant works contain 4,400,000 cubic yards of concrete, of which 3,250,000 cubic yards were required in the dam.

During construction the river was diverted through four 50-

foot diameter, concrete-lined tunnels, two on each side of the river. These tunnels were subsequently plugged near the upstream ends. The spillways, each of 200,000 second-feet capacity, are connected through inclined shafts to the two outer tunnels. A 30-foot diameter steel power penstock is installed in each of the inner tunnels. Discharge from the reservoir is controlled by cylinder gates in four intake towers, founded on the canyon walls near the upstream face of the dam. Four 30-foot steel penstocks, connected to the bases of the intake towers, conduct water to the power plant and to the outlet valves for release of flood, irrigation, and domestic water supply when the power plant discharge is insufficient for such purposes. The reservoir above the dam is 115 miles long and has a capacity of 30,500,000 acre-feet, the equivalent of two years' normal river flow.

The power plant is in a U-shaped, reinforced concrete structure, over 200 feet high and 1,500 feet long, located immediately downstream from the dam. The plant is designed for an ultimate installation of fifteen 115,000 and two 55,000 horsepower units, making a total installed capacity of 1,835,000 horsepower.

The All-American Canal, located near the Mexican border, will carry water to irrigate lands in the Imperial and Coachella Valleys. The canal proper, with a diversion capacity of 15,000 second-feet, is the largest ever constructed for irrigation purposes in America.

The entire Boulder Canyon Project is characterized by the extraordinary. The height and base thickness of the dam, the size of the power units, the dimensions of the fusion-welded, plate-steel pipes, the novel system of artificially cooling the concrete, the speed and coordination of construction, and other major features of the project are without precedent. The magnitude of the undertaking introduced many new problems and intensified many usual ones, requiring investigations of an extensive and diversified character to insure structures representing the utmost in efficiency, safety, and economy of construction and operation.

The major credit for the conception of the project and the initiation of investigations leading to its adoption must be given to the late Arthur P. Davis, former Director of the Reclamation Service. Dr. Elwood Mead, Commissioner of Reclamation during the greater part of the construction period, passed away January 26, 1936, four months after the dedication of Boulder Dam. In commemoration of his untiring services on the Boulder Canyon Project, the reservoir created by the construction of the dam has been officially named "Lake Mead".

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CHAPTER I—INTRODUCTION AND DESCRIPTION OF HYDRAULIC LABORATORIES

INTRODUCTION

1. **History of Spillway Tests.**—The unprecedented height of Boulder Dam required a thorough study of all phases of design to insure the safety of the structure. The spillways were one feature which demanded careful attention. It was found necessary to provide for a flow of 400,000 second-feet, which, with the fall of 500 feet, represents an energy of 22,700,000 horsepower, about seven times the energy of the water flowing over Niagara Falls. Moreover, the theoretical velocity of the water is 175 feet per second,[1] or two miles per minute, which considerably exceeds the velocity in any similar structure so far constructed. It is obvious, therefore, that the greatest possible care was necessary to be sure that the spillways will act as expected, so that no damage will result.

To discharge so great a quantity of water over the top of the dam, permitting it to flow down the face to the river below, would involve great danger to the security of the dam foundations. The large tunnels which were necessary to divert the river during construction provided a possible outlet for the water, since they discharge into the river several hundred feet downstream from the dam. Spillway plans were developed utilizing these tunnels. The first designs included one spillway of the shaft or glory-hole type on each side of the river, but this was recognized as only a preliminary layout. When funds for extensive study of the project became available, all practical forms of spillways were studied and cost estimates compared. These included glory holes, side channels with and without gates, and various combinations of such forms. Spillways using an entirely separate tunnel system were also planned and estimated. It was evident that extensive model tests would be necessary to secure an economical and safe design. The problem of providing a spillway form which would discharge through the

[1]Recent investigations have shown that actual velocities in open channels may be materially less due to entrainment of air. A. Schoklitsch: Kolkabwehr und Stauraumverlandung (Prevention of Scour and Silting of Reservoirs). Julius Springer, Berlin, 1935.

long tunnels without undesirable effects proved to be unsually exacting. The spillways as finally evolved represent the result of an intensive process of design, estimation, experimentation, and elimination, covering a period of more than two years.

When the design of side-channel spillways for Boulder Dam was undertaken, many points which were necessary to consider in preparing plans were uncertain. As a result of the model tests most of the uncertainties were removed. Consequently future designs can be made with much greater confidence.

Some of the questions which arose in preparing the spillway designs were:

1. Can reliance be placed on the theory developed for flow in side-channel spillways?

2. How far can tests on small models be depended upon to show quantitatively how large spillways will perform?

3. What air content must be assumed for the water in the channel?

4. What shape is necessary in the transition floor in order that a vacuum may not form under the flowing water?

5. Will water pass around vertical and horizontal bends without undesirable effects?

6. Will concrete withstand the erosive action of water flowing at velocities of 175 feet per second?

The hydraulic model tests were begun at the laboratory of the Colorado Agricultural Experiment Station at Fort Collins, Colorado. The first model was of the glory-hole type, on a scale of 1 to 60, designed for a capacity corresponding to 100,000 second-feet in the prototype. Expediting Boulder Dam project as an unemployment relief measure made necessary a decision on the general type of spillway without extensive tests on this model.

The second model was of the side-channel type with a free crest, also on a 1:60 scale, designed for a prototype capacity of 100,000 second-feet with the reservoir water surface at elevation 1234.5, and 140,000 second-feet with the water surface at the top of the dam. Before the experiments on the model were completed, it was decided to increase each spillway capacity to 200,000 second-feet. The next experiments were on a 1:60 model of a side-channel spillway with a Stoney gate at one end. While experiments on this model were being conducted, a large 1:20 scale model of the side-

channel type, with drum gates on the crest, was built at a specially constructed laboratory at Montrose, Colorado.

The Stoney gate spillway did not prove satisfactory but tests were conducted on it as a matter of record. From the results of the tests on the 1:20 drum-gate model at Montrose, a new 1:60 drum-gate design was prepared and the model tested at Colorado State College. After many changes a satisfactory form of spillway was developed and another 1:20 model of this form was built at the Montrose laboratory.

Experiments in the Colorado State College laboratory were then undertaken to develop the form of spillway crest giving the greatest discharge capacity. A more efficient form of crest was developed and useful data were obtained for the design of the crest gates. As a result of these studies, the overflow crest of the side-channel model previously developed was altered to increase its capacity. Models of the final design were constructed on scales of 1:100, 1:60, and 1:20. Comparisons of results obtained on the three models indicated that substantially similar results would be obtained on the prototype.

Throughout the entire series of tests, close contact was maintained with the design office in Denver; so that the plans developed would be sound, not only from the hydraulic, but also from the structural and construction standpoint.

Experiments were made to determine the resistance of concrete to erosive action of clear water flowing at extremely high velocities. Although the experiments were not a part of the laboratory tests on hydraulic properties of spillways, they were definitely related to the practical performance of the spillway structures. Since no previously existent structures had been known to carry water at velocities of nearly 180 feet per second, the investigations were of considerable importance.

Studies made in 1931, on existing structures exposed to clear water flowing at velocities up to 130 feet per second, indicated that in most instances there had been no erosion. However, lack of data regarding properties of concrete, details of construction, and conditions of flow made the data unreliable and indicated the necessity for controlled tests. Consequently, test specimens were fabricated in the Denver laboratories of the Bureau of Reclamation and the Bureau of Standards, and tested in a specially designed apparatus at the Guernsey power plant near Guernsey, Wyoming. The results of the tests are described in chapter VI.

2. Results of Model Tests.—The results of the model tests were highly satisfactory. The final designs of the spillways as constructed on the Arizona and Nevada sides of the canyon are shown in figures 1 and 2. The locations of the spillways with respect to the dam and other features of the development are shown in the frontispiece. The spillways amply satisfy the exacting requirements imposed by the enormous but erratic flood flows of Colorado River, at a moderate cost and with a reasonable assurance of safety. Without the model experiments, the same degree of security could only have been obtained by the use of designs costing several million dollars more than those adopted. The results of the hydraulic laboratory investigations herein presented should furnish valuable assistance in designing spillways at other dams.

3. Model and Prototype Relations.—The soundness of conclusions reached from model tests depends on the accuracy with which the action of the prototype can be predicted from observations on models. This question has been the subject of considerable controversy. The problem can be investigated from two viewpoints: first, by theoretical reasoning based on the application of known laws of hydraulics and mechanics to the action of both model and prototype; and second, by experimentation and observation. Because of the complexity of the problem, it was not considered safe to depend entirely on either method in developing designs for the Boulder Dam spillways.

The theory of model testing has been developed in considerable detail. Excellent articles on model testing have appeared in many publications. These have conclusively demonstrated the reliability of model tests, when certain practical limitations are kept in mind. In the Boulder Dam experiments, considerable attention was given to theoretical phases of the subject; but within the field covered by the tests, theoretical considerations were already so well developed that little was added to existing knowledge. The principal emphasis was placed on the other line of attack; that is, on the experimental confirmation of theoretical deductions. It was, of course, impractical to make comparisons of tests on the model and prototype. However, if tests on models extending over a considerable range of sizes show virtually identical results, a further extension of size to that of the protoype could be expected to produce similar results. Any uncertainty in the comparative action of model and prototype in the Boulder Dam tests was reduced to a

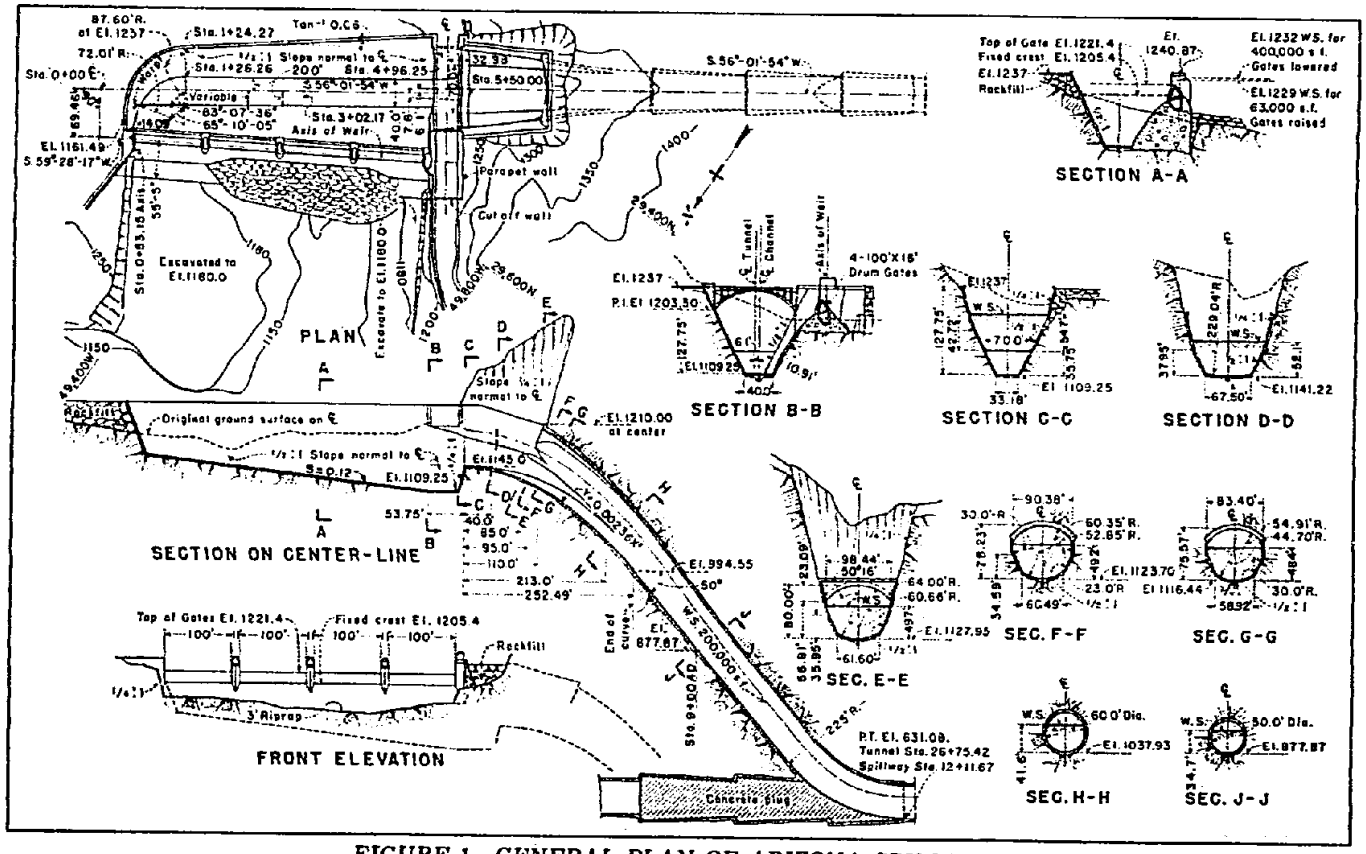

FIGURE 1—GENERAL PLAN OF ARIZONA SPILLWAY

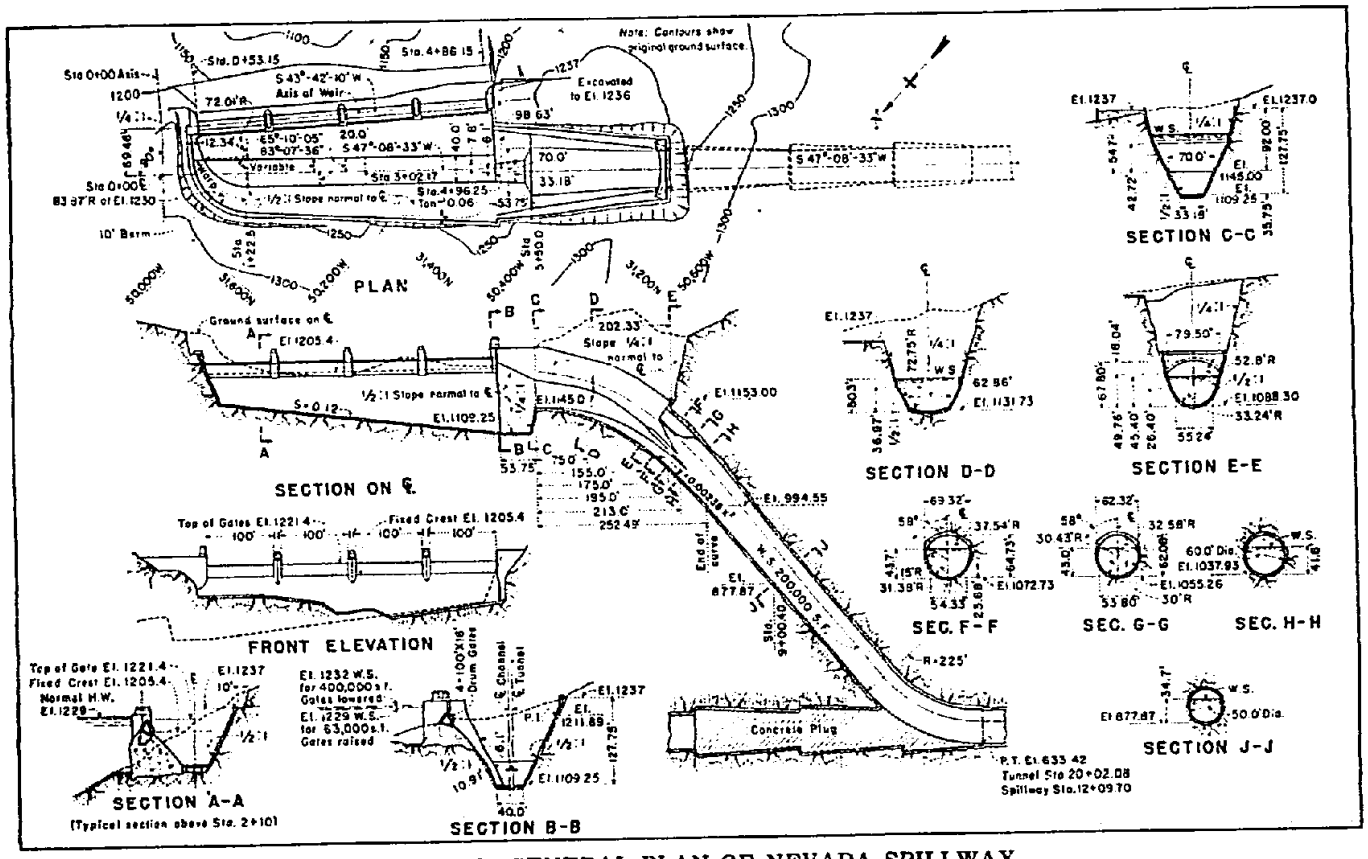

FIGURE 2—GENERAL PLAN OF NEVADA SPILLWAY

minimum by making the largest models as near as practical to the prototype size.

Due to the fact that the results of the Boulder Dam experiments are largely a confirmation of conclusions reached by model theory, rather than additions to the theory itself, the results are here presented without extensive theoretical discussions. In this form they are believed to be more useful to engineers engaged in designing similar structures, and more convincing to those who may not already be satisfied as to the reliability of model results. It is believed, however, that those interested in the more theoretical phases will find the results of value, especially the quantitative comparisons of the results of the three sizes of final design models. The relations of the quantities in the models to those in the prototype are given in the following table:

RATIO OF PROTOTYPE TO MODEL

Quantities	In Terms of N*	N=20	N=60
Length	N	20	60
Area	N^2	400	3,600
Volume	N^3	8,000	216,000
Velocity	$N^{0.5}$	4.472	7.746
Discharge	$N^{2.5}$	1,788.8	27,885
Time	$N^{0.5}$	4.472	7.746
Energy	N^4	160,000	12,960,000
Impulse	$N^{3.5}$	35,776	1,673,136

N*=Ratio of linear dimensions of the prototype to those of the model.

The model ratio is usually expressed in terms of 1:N.

As a means of avoiding confusion and for the sake of brevity, each model was designated by a letter denoting its chronological order. Thus, the model of the glory-hole spillway, the first model tested at Fort Collins, is shown as the C-1 model.

As the tests were conducted with models of different scales, each of which had different relations to the prototype, confusion is avoided by referring to the corresponding dimensions and discharges of the prototype rather than the actual quantities observed on the model. This enables comparisons to be made between the performance of models of different scales without the necessity of reducing the values on one to corresponding values on the other.

Moreover, it enables one to visualize and appreciate the true significance of the effects as they will appear in the actual spillway in a way that expressions in terms of actual model size could not do without considerable computation. Throughout this report, therefore, all quantities such as lengths, elevations, and discharges, unless otherwise stated, are given in terms of the corresponding quantities on the prototype rather than the actual quantities observed on the model.

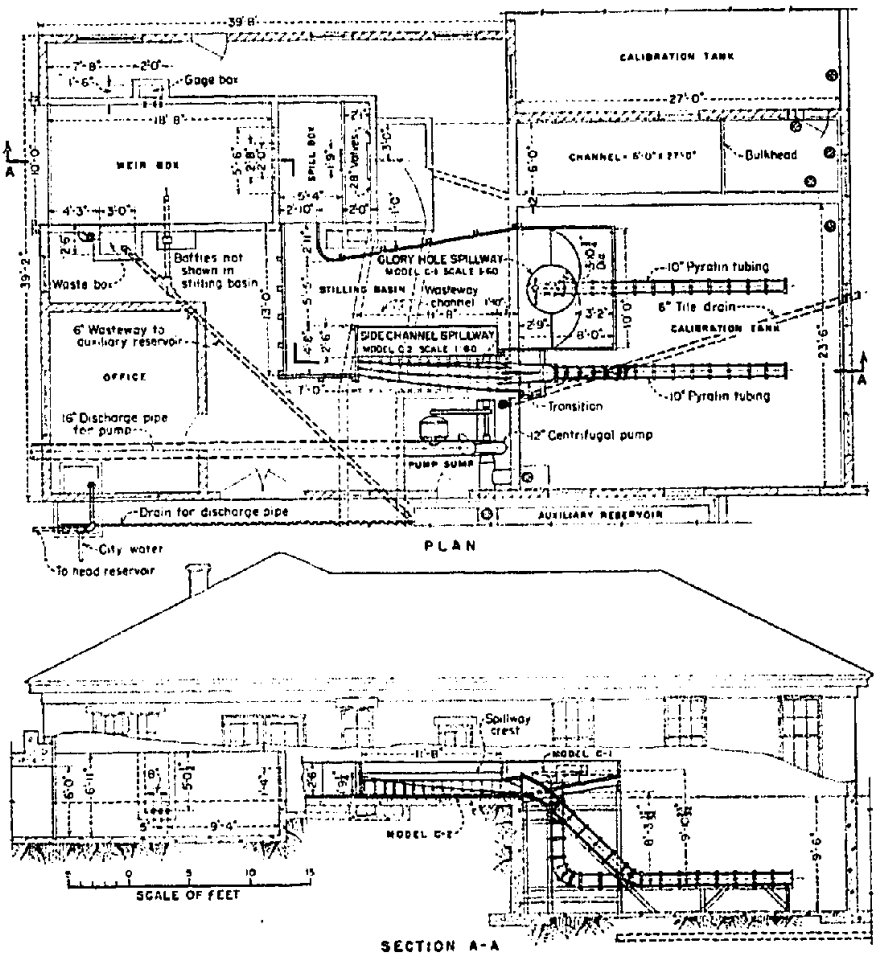

FIGURE 3—HYDRAULIC LABORATORY AT COLORADO STATE COLLEGE

A bibliography is included at the back of this bulletin for the benefit of those who may wish to study theoretical phases of model testing. General references on model testing and the theory of similitude are listed first. These are followed by references on side-channel spillways.

HYDRAULIC LABORATORIES

4. **Colorado State College Laboratory.**—The small scale experiments for the design of the Boulder Dam spillways were conducted in the hydraulic laboratory of the Colorado Agricultural Experiment Station at Fort Collins, Colorado. The laboratory was described by V. M. Cone in Engineering News, October 2, 1913, pages 662 to 665. The facilities of the laboratory were adequate for tests of Boulder spillway models as large as 1:60 scale. The generous permission to use the laboratory and the hearty cooperation of the college staff and the staff of the United States Bureau of Agricultural Engineering, who ordinarily use the laboratory, are greatly appreciated.

The general plan of the laboratory and the locations of the first two models are shown in figure 3. Water was obtained from a reservoir of 30,000 cubic feet capacity, located on a hill adjacent to the laboratory. The flow from the reservoir was controlled by hand-operated gates at the upstream end of a weir box, 19.5 feet long, 10 feet wide, and 7 feet deep. Close adjustment of the quantity discharged through the model was made by varying the flow through two bypasses in the side of the box, 13 feet upstream from the weir, one controlled by a hinged weir, and the other by a valve. The head on the weir was observed by a float gage similar to that developed at Cornell University.[7] The gage was located in a stilling well connected with the main channel by a pipe.

Discharges for all tests, except the series to determine the best overflow crest shape, were measured by either a 90-degree V-notch weir or a two-foot Cipolletti weir, both of which had been volumetrically calibrated. During the experiments on the drum-gate model, the crest of the Cipolletti weir was partially submerged. An extensive set of volumetric calibration tests was made to determine weir discharges under submerged conditions.

[7]Trans. Am. Soc. C. E., vol. 83, 1920, p. 1154.

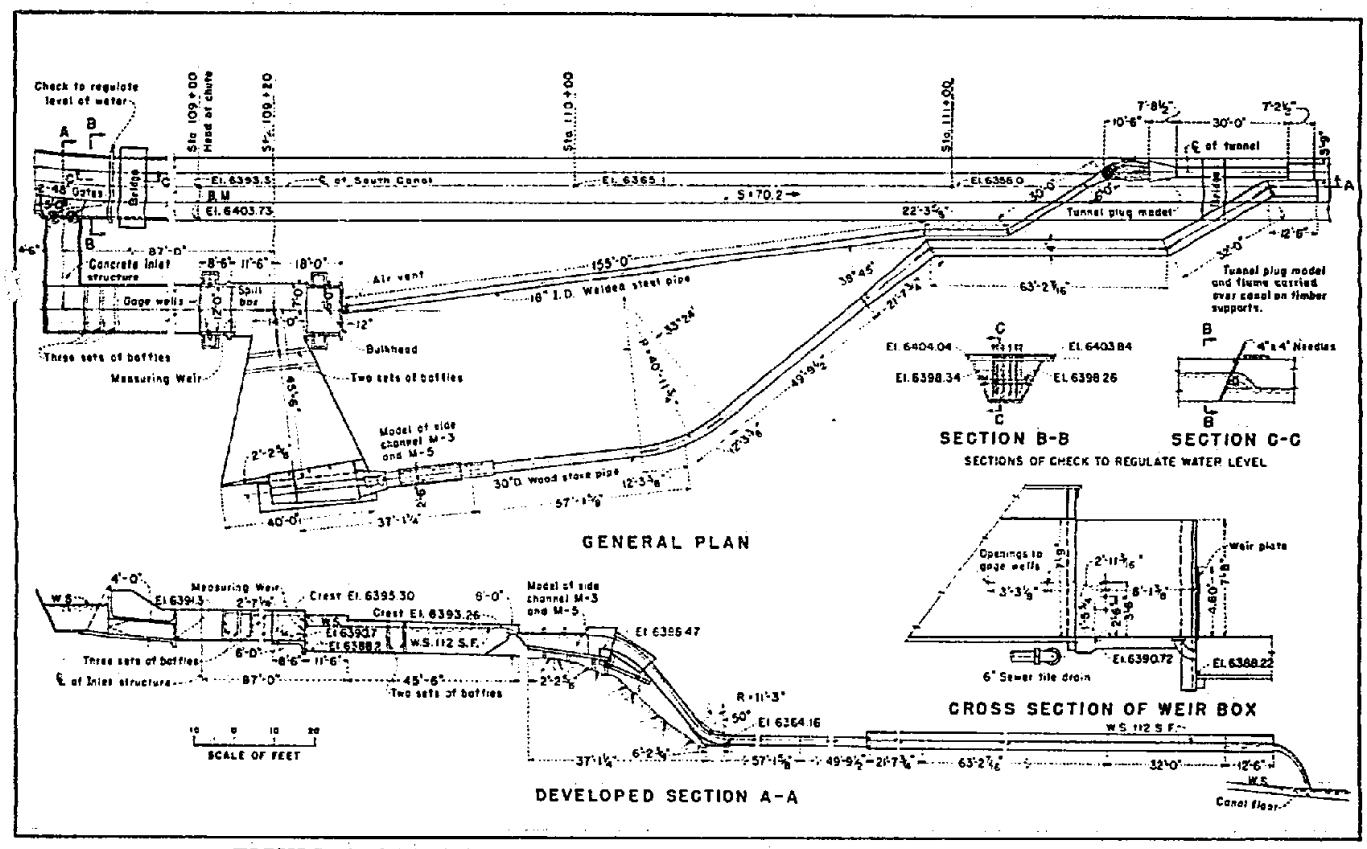

FIGURE 4—DIAGRAMMATIC LAYOUT, MONTROSE, COLO., LABORATORY

5. Bureau of Reclamation Laboratory.—Because of the tremendous power of the water to be handled by the spillways, it was necessary to take every reasonable precaution to insure that the finished structure will function as intended. It was therefore decided to build a model on a scale as large as practicable, in order to indicate any discrepancies between the action of the model and prototype which might arise from using too small a model. A model scale of 1:20 was decided upon for the tests. This required a discharge of 112 second-feet and a fall within the model of approximately 30 feet. As no laboratory was available which provided satisfactory facilities, one was built on the Uncompahgre Irrigation project at Montrose, Colorado, as shown in figure 4. At the location selected a drop in the South Canal provides a fall of approximately 50 feet and a flow of 200 second-feet or more throughout the irrigation season.

Water was diverted from the canal above the chute through two 48-inch circular gates. A check, controlled by 4- by 4-inch needles, was installed below the gates to raise the water in the ditch sufficiently at times of low flow to permit diversion of the desired quantity. From the intake gates the water flowed into the upper end of the weir channel, which was of wood frame construction, 12 feet wide with walls 8 feet high. The total length of the channel upstream from the weir was 80 feet. Three sets of baffles, consisting of panels of cross-slats, were located at the upper end to quiet the water approaching the weir. The spacing of the slats was varied as a result of experiment until a uniform distribution of velocity was obtained.

The weir box was constructed of concrete on a foundation of shale so that the elevation of the weir crest would not be subject to change. The weir was a duplicate of the Francis weir with suppressed contractions, except that a length of 12 feet was used instead of 10 feet. The crest was 4.6 feet above the approach floor and the walls were offset, downstream from the crest, to permit aeration of the nappe. The weir was cut in a steel plate, braced with structural steel shapes; and the weir crest was formed of a steel angle with a lip one-quarter inch wide. The upstream face of the weir was smooth in the vicinity of the crest and care was taken to keep the upstream corner true and sharp. The head on the weir was observed from float gages of the Cornell type, set in concrete pits on either side of the weir box. The gage scales were

supported by metal frames attached to the concrete pits for permanency. The float wells were connected through a short section of garden hose to piezometers in the weir-box walls, located 6 feet upstream and 1.0 foot below the weir crest, as on the Francis weir.

After flowing over the weir, the water turned at right angles and passed through another set of baffles into an expanding forebay at the end of which was located the spillway model. This particular shape of the laboratory was dictated by the necessity of having the water approach the model at right angles to the overflow section. After passing through the model spillway and tunnel, the water was discharged through a rectangular wooden flume into the irrigation canal.

6. **Personnel.**—The hydraulic laboratory tests described in this bulletin were begun under the supervision of E. W. Lane, former research engineer, and completed under the supervision of Jacob E. Warnock, present research engineer. The erosion tests on concrete were made under the supervision of J. F. Gleason, superintendent of power at Guernsey, and Arthur Ruettgers, engineer in charge of concrete research. Chapter VI was prepared by G. A. Marston and C. C. McNamara, junior engineers.

R. R. Randolph, asistant engineer, aided in supervising the work at the Fort Collins laboratory; and Whitney M. Borland, assistant engineer, aided in supervising the program at the Montrose laboratory.

During the course of the investigations, material assistance in design, construction, and operation of the models and in analysis of technical data was given by J. N. Bradley and W. H. Price, assistant engineers, and C. W. Thomas, J. W. Ball, R. A. Goodpasture, A. N. Smith, and R. R. Buirgy, junior engineers.

CHAPTER II—TESTS ON PRELIMINARY DESIGNS OF SPILLWAYS

GLORY-HOLE SPILLWAY MODEL (C-1)

7. **Construction of Model.**—Preliminary designs for the Boulder Dam spillways were of the glory-hole or shaft type. The design of this type of spillway contemplated a discharge of 100,000 second-feet for each shaft, with the reservoir water surface at elevation 1234.5. One of the conditions specified for the project was that the discharge from a flood as large as that of 1884 should be carried safely in the river channel below the dam. To meet this specification, the combined discharge of the two spillways should not exceed 62,500 second-feet with the reservoir water surface at elevation 1229.0.

The spillway designed to meet these requirements is shown

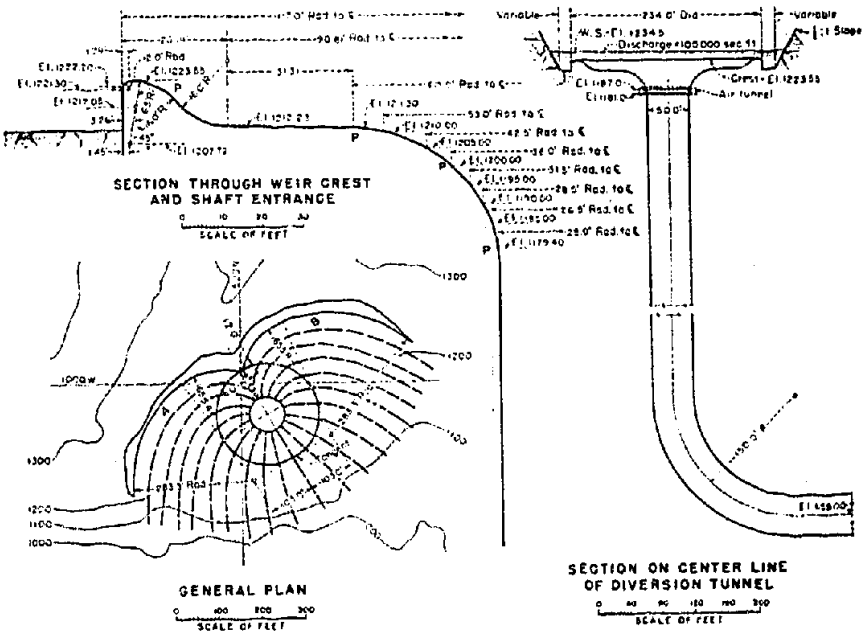

FIGURE 5—GLORY HOLE OR SHAFT SPILLWAY PRELIMINARY DESIGN

in figure 5. It consisted of a circular ogee crest, 234 feet in diameter, below which was a morning-glory-shaped funnel leading to a 50-foot diameter shaft. The purpose of the ogee crest was to produce a high discharge coefficient, thus providing a larger discharge capacity per foot of crest and a reduction in cost of construction.

A model of the glory-hole spillway, built on a scale of 1:60, was tested in the hydraulic laboratory of Colorado State College at Fort Collins, Colorado. Discharges through the model were measured by a two-foot Cipolletti weir. Water flowed to the spillway through a series of baffles, thus producing quiet water upstream from the structure.

The crest section of the spillway was built of paraffin. The paraffin block was first cast approximately to the shape desired, then cut accurately to dimensions by a template in the form of a knife or screed revolved about an axis in the center of the spillway shaft as shown in figure 6. Although paraffin is a rather soft material to use in building a model, it is easily patched when it becomes scarred or nicked. For this type of model, it is believed to be as satisfactory a material as can be selected. Some checking occurred in the model but this was easily remedied; the only permanent change being in the appearance.

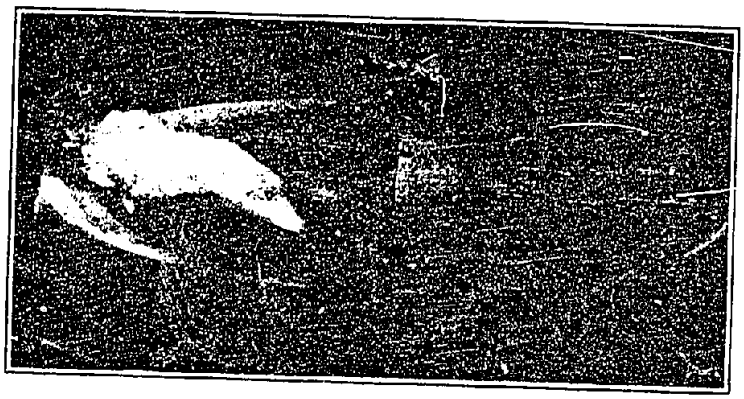

FIGURE 6—CONSTRUCTION OF PARAFFIN CREST OF GLORY-HOLE MODEL

The vertical tunnel, vertical bend, and horizontal tunnel were formed of transparent pyralin, a material similar to celluloid, so that the action of the water could be observed. The various sections

of the bend were made of pyralin sheets, one-eighth inch thick, formed in a hydraulic press. The straight sections were about 18 inches long, flanged at the ends, and were connected by longitudinal butt joints with outside straps. The 90-degree vertical bend was made up of four 22½-degree flanged bends, each bend being divided into two similar flanged sections on the plane passing through the center line of the bend. The 90-degree bend was made in sections to facilitate construction and to enable experiments to be made with 22½-degree, 45-degree, and 67½-degree bends.

8. **Results of Tests.**—Due to expediting Boulder Dam project as an unemployment relief measure, it was necessary to make a decision on the general type of spillway without conducting the extensive tests first contemplated. To avoid delaying work on the side-channel models, experiments on the glory-hole type were few, consisting only of visual observations.

In the design of the glory-hole spillway, it was assumed that the water would flow over the crest and toward the spillway shaft in a radial direction, with equal depths at all points equally distant from the axis of the shaft. This would produce a smooth condition of flow in the crest section and down the vertical shaft. The model showed, however, that this condition would not be obtained in the spillway. A distinct concentration of flow occurred along the radial line from the center to the point at the back side of the spillway formed by the ends of the two approach channels. This concentration took the form of a ridge across the intake section. As a result, water did not flow down the shaft equally distributed around the walls, but in a concentrated stream that jumped across the top of the shaft from the canyon wall to the lake side, finally dropping down the shaft with an irregular distribution. For large flows the ridge or jet tended to seal the top of the shaft, causing an undesirable suction and pulsation. This action passed through a cycle of two stages. In the first stage, the stream flowing across the top of the shaft sealed the top and resulted in a suction which drew water down the shaft. When the vacuum in the shaft was broken by the entrance of air from above, a distinct piling up of water occurred over the shaft. This was followed by a return to the first condition and a repetition of the cycle.

Studies were made to obtain a more equal distribution of flow in the inlet section of the spillway. The concentration of flow was found to be caused by water not entering the intake section in a

radial direction. To bring the cost of the spillway to a reasonable amount, it was necessary to set the structure partly in excavation in the side of the canyon. About one-half the water passing over the crest approached it through the excavated channels, A and B, in which the direction of flow was not at right angles to the crest but as indicated by the lines in figure 5. As the water approached the center of the morning-glory-shaped section, more of it tended to concentrate at C and enter the shaft on that side, causing a disturbed condition of flow. The directions of the currents were determined by inserting coloring matter in the water. The direction of the stream along the ridge of concentration, leading from the ends of the two approach channels, was radial, since the flows from the two approach channels were inclined in opposite directions and balanced each other. At 90 degrees to this line, the flow was not radial, but was nearly tangent to the edge of the shaft. It was the concentration of the non-radial flows that formed the ridge.

It is believed that a more uniform distribution of the flow entering the spillway shaft could have been secured with radial piers on the crest and on the morning-glory-shaped section, similar to those used on the Davis Bridge spillway.[3] A similar set of piers in the approach channel was tested with fairly favorable results. The effect of the impact of the water at the bottom of the vertical shaft would obviously be much more severe, even with a perfect distribution of flow, than after flowing down the inclined shaft from the side channel. With the unprecedented velocities resulting from the 600-foot drop at Boulder Dam, it was believed to be safer to use the side-channel type of spillway. Consequently, work on the glory-hole type was discontinued and the side-channel type adopted.

SIDE-CHANNEL SPILLWAY WITH FREE CREST (C-2)

9. Design and Construction of Model.—The first side-channel design for the Boulder Dam spillways proposed to use a plain overflow crest without gates. The design of the spillway, shown in figure 7, provided for a discharge capacity of 31,250 second-feet with the reservoir water surface at elevation 1229.0 and 100,000 second-feet with the water surface at elevation 1234.5, the same as

[3]Kurtz, Ford, "The Hydraulic Design of the Shaft-Type Spillway for the Davis Bridge Dam and Hydraulic Tests on Working Models", Trans. Am. Soc. C. E., vol. 88, 1925, p 1.

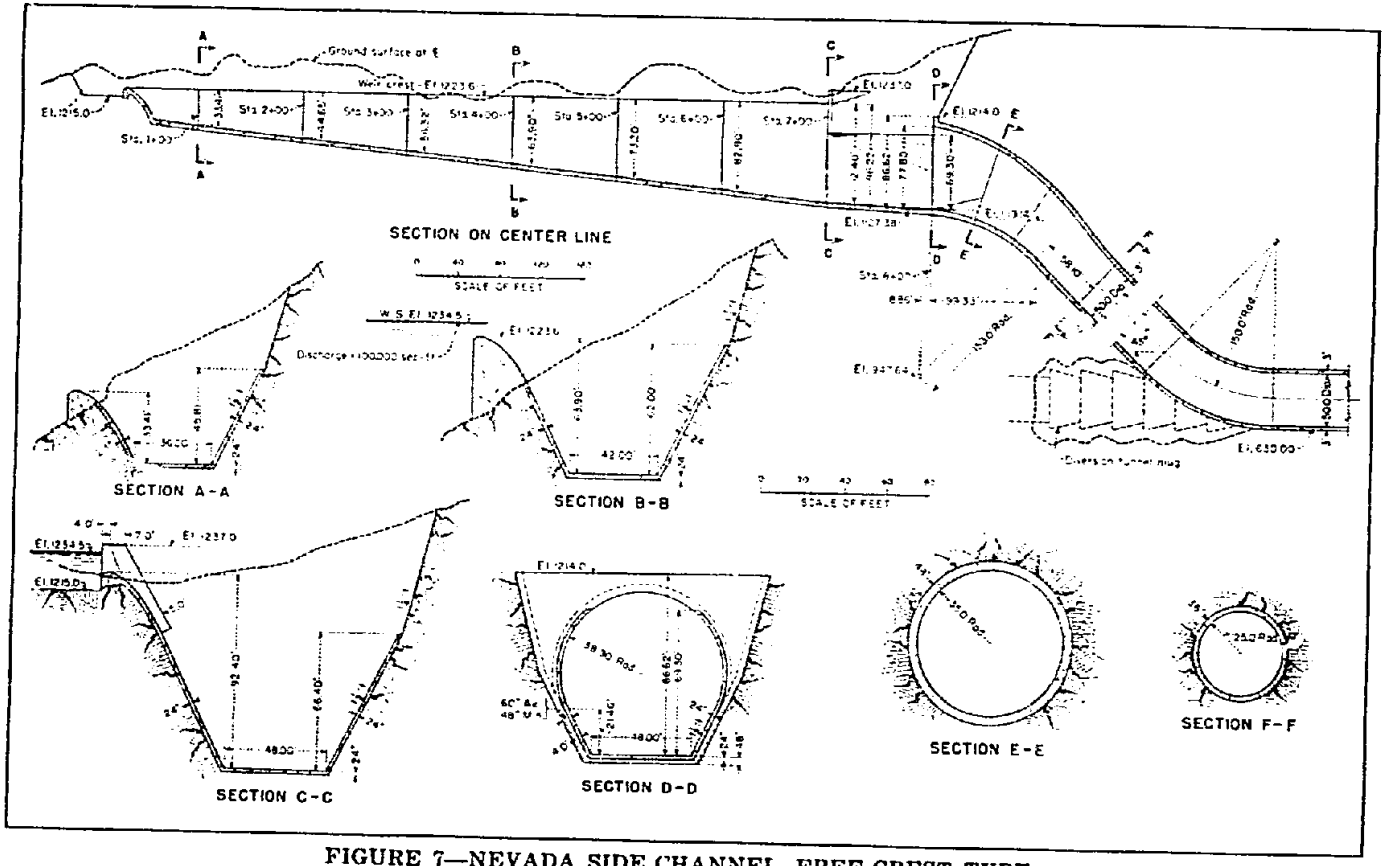

FIGURE 7—NEVADA SIDE CHANNEL, FREE-CREST TYPE

the glory-hole type. However, the spillway side channel and tunnel had sufficient capacity to handle the flow over the crest with the water surface at the top of the dam, approximately 140,000 second-feet. A model of the Nevada spillway was built in the Colorado State College laboratory on a 1:60 scale. The position of the model is shown in figure 3. The water measuring apparatus was the same as previously described.

The spillway channel was constructed of galvanized iron, supported on sheet-metal buttresses. It was expected that several sizes of channel would be tested and the buttresses were designed so that various cross sections could be obtained by slight changes of the positions of the buttresses. The overflow crest of the structure was made of paraffin. A sloping floor, simulating the mountain side, intersecting the vertical face of the spillway crest 35 feet below the top, was placed in the approach channel. The transition from the trapezoidal section of the side channel to the circular 50-foot diameter tunnel was constructed of reinforced concrete. It was built in halves, divided by a vertical plane along the center line. Plaster of Paris cores for the opening were cast around accurately spaced guides, held in place by reinforcing rods. The transition was cast around the core using quick-setting cement. When the cement had set, the core was removed and the two halves bolted together. Three windows were cast into the top, so the flow in the transition could be observed. These could be closed with concrete plugs accurately cast to fit the windows. The pyralin tube which formed the model of the vertical shaft and tunnel for the glory-hole model was used for the inclined and horizontal tunnels. The slope of the inclined tunnel was 45 degrees and the bend between the two sections of tunnel was formed by two 22½-degree bends.

10. Results of Visual Observations.—For flows of 50,000, 100,000, and 150,000 second-feet, the sheet of water flowing over the spillway crest plunged under the water in the channel, producing a spiral motion. The motion at the bottom of the channel was from the crest side diagonally across to the opposite side. Under favorable conditions, the spiral motion was so pronounced that the centrifugal force tended to move the denser water to the outside and the lighter air bubbles toward the center, forming a core of air in the center for the full length of the channel. When the discharge was increased above that for which the spillway was de-

signed, the depth of flow at the upper end of the channel was raised until the nappe no longer plunged under the water in the trough, but passed over the top. This action tended to cause a spiral at the upper end of the channel in the opposite direction to that formed at the lower end. At the junction of the two spirals there was an unstable condition. The point where the direction of rotation changed moved up and down the channel over a considerable range, causing a pulsating condition of flow.

11. Measurements of Flow Conditions.—Soon after the model was completed, it was decided to increase the capacity of each spillway from 100,000 to 200,000 second-feet. This required a redesign of the spillway, and while the office studies to develop the most economic design were under way, an extensive series of experiments was conducted to check the theory of side-channel spillways and to investigate methods of reducing the cost.

The basic theory of side-channel spillways was developed by Julian Hinds.[1] It is based on the assumption that all the energy of the water flowing over the spillway crest is dissipated as heat and the flow down the spillway trough is caused entirely by the water surface slope in the channel.

Four series of tests were conducted under the following conditions:

1. The side-channel spillway, with trapezoidal channel cross section as designed, carrying various discharges.
2. Various discharges with submerged weirs across the downstream end of the spillway channel.
3. A channel with the same sides as the original design, but with a flat floor at a higher elevation formed by a false bottom.
4. A channel with practically the same sides as the original design but with the bottom formed by a circular arc tangent to the sides as shown in figure 8. The bottom of the channel of this model was slightly higher than the original design.

Hydraulically, the side-channel spillway is an inefficient device, because much of the energy of the water which falls over the crest is dissipated in heat and does not cause flow down the side channel. If part of the energy could be used in producing flow along the

[1]Hinds, Julian, "Side-Channel Spillways", Trans. Am. Soc. C. E., vol. 89, 1926, p. 881.

FIGURE 8—MODEL OF SIDE-CHANNEL SPILLWAY WITH ROUNDED BOTTOM

channel, a greater velocity would result and a smaller and cheaper channel could be used. To a certain extent this can be accomplished by vanes which guide the water from the crest into a direction more or less parallel to the center line of the channel. Different types and combinations of vanes were tried to determine if the spillway cost could be reduced by this means.

One of the uncertainties of side-channel spillway design is the allowance which must be made for air entrained due to turbulent conditions of flow. The theory as developed considered only air-free water. If a quantity of air is entrained the volume of the water is correspondingly increased and the size of the channel is also increased. Measurements were therefore made of the air entrained in the flow through the model. The results of the tests were largely negative from the viewpoint of producing smoother conditions of flow. However, they are of interest from the hydraulic viewpoint.

Observations to test the side-channel theory consisted in measurements of the cross section of flow in the side channel for different discharges through the model. For this purpose two parallel bars were placed along the sides of the channel, about one and one-half feet above the crest. The bars were carefully leveled with their upper surfaces in the same horizontal plane. A point gage extending down from a movable beam, spanning the distance from

one bar to the other, was used to determine the elevation of the water surface at any point in the channel. Ordinarily, cross sections were taken at 20-inch intervals, which, for the 1:60 scale model, correspond to 100-foot intervals in the prototype.

Sufficient points were determined to permit plotting of each cross section to scale. The area of each was determined with a planimeter. A typical set of cross sections for a discharge of

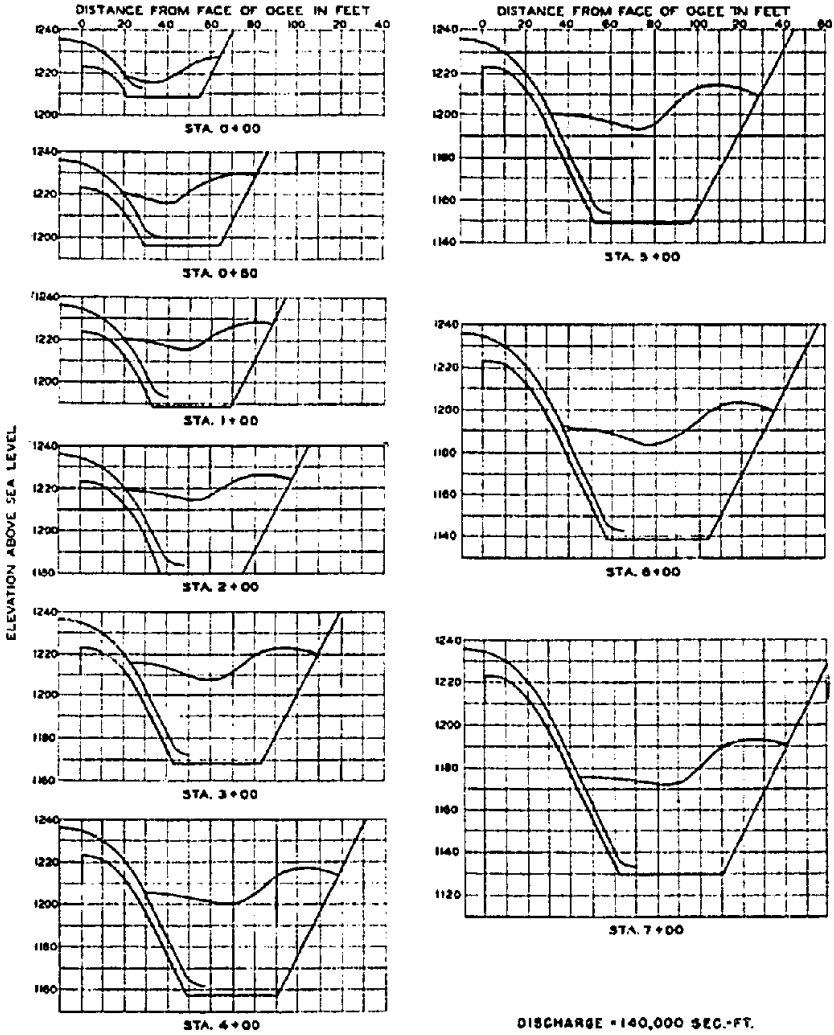

FIGURE 9—TYPICAL CROSS SECTIONS OF FLOW IN TRAPEZOIDAL CHANNEL OF MODEL C-2, DISCHARGE 140,000 SECOND-FEET

140,000 second-feet is shown in figure 9. From the cross sections the mean depth of flow was obtained for use in the computations. The water surface on the crest side was lower, and on the opposite side higher, than the mean water surface. By means of piezometer openings in the floor of the channel, connected to glass tube manometers, observations were made of the downward pressure on the bottom of the channel at four points along the center line.

12. Actual and Theoretical Conditions.—Quantitative results of observations on the model of the original design are shown on figure 10. The first diagram shows mean water surfaces for discharges between 50,000 and 200,000 second-feet. The remaining diagrams show analyses of individual runs. The average water surface, determined as described above; the theoretical water surface, where it could be computed; the depth of water equivalent to the downward pressure on the channel bottom; and the critical depth of flow for the given channel section are shown for each discharge. Elevations of reservoir water surface, spillway crest, and channel floor are also indicated. The theoretical water surface was computed on the assumption that all the energy of the water flowing over the crest is lost.[a] The computations were usually started with the observed water surface at station 7+00, and theoretical water surfaces computed upstream by a trial method, using the laws of inelastic impact instead of the Bernoulli Theorem. The computed curves are similar to ordinary backwater curves, since they cannot extend across the critical depth. The logical place to start the computations is at a known water surface elevation at the downstream end of the channel, since backwater curves for supercritical depths can be worked upstream more effectively than downstream.

It was not always possible to commence the computations at station 7+00 because, for some conditions of discharge, the critical depth was upstream from this point. In this case, the computations could be carried upstream only to the critical depth. It was then necessary to make an observation on the water surface, somewhere between the critical depth and the upper end of the channel; and commence a new set of computations, working upstream and downstream from the new point. Where the computations were commenced upstream from the critical depth, the starting point in each case is indicated by the letter S on the diagrams.

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[a]Trans. Am. Soc. C. E., vol. 89, 1926, pp. 898-899.

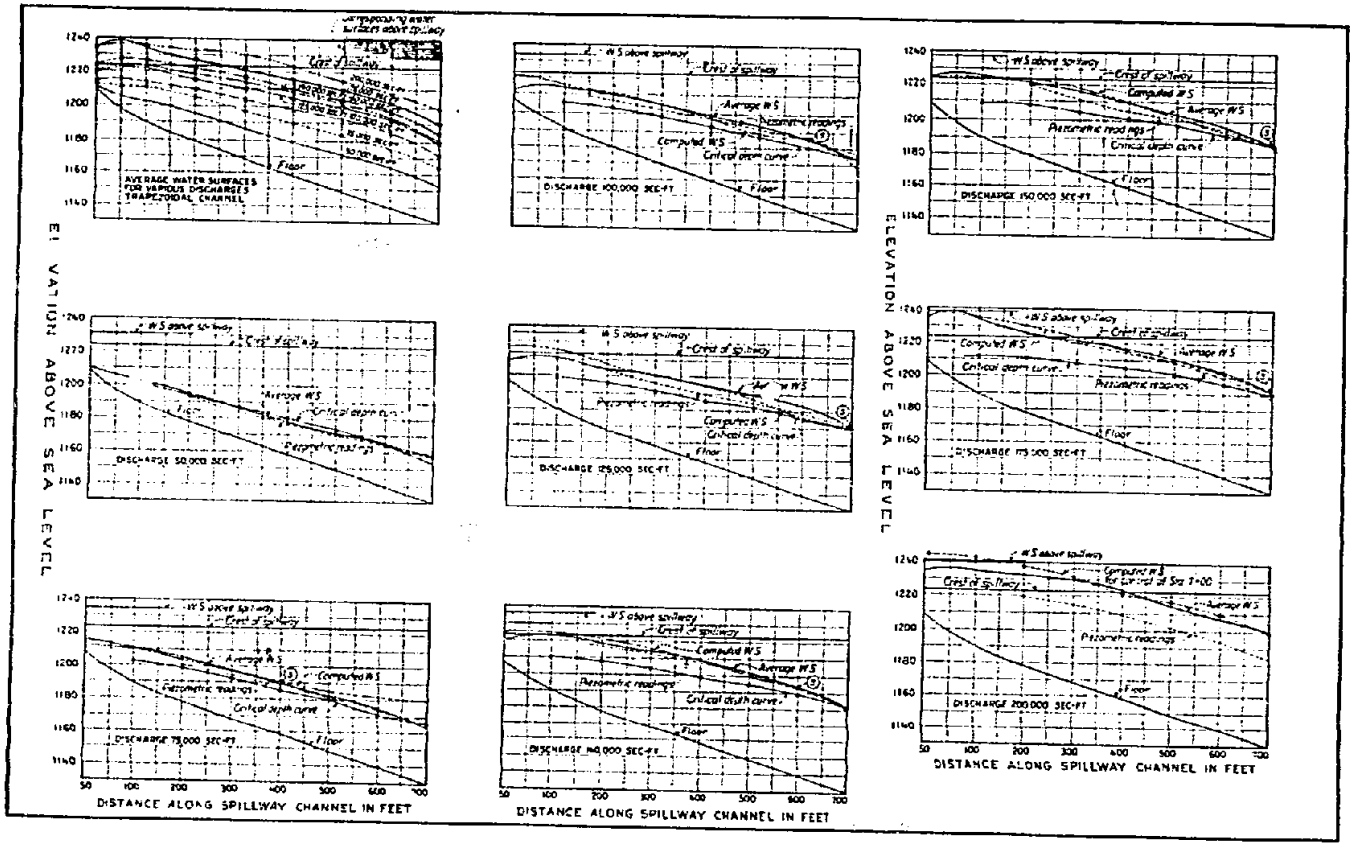


FIGURE 10—WATER SURFACE PROFILES IN TRAPEZOIDAL CHANNEL OF MODEL C-2

The flow line for 50,000 second-feet was so near critical depth that the backwater curve could not be computed. For 75,000 second-feet, the computations were started at station 4+00 and carried in both directions. The results departed considerably from the observed water surface below station 4+00, indicating that where the computations are started near the critical depth, the water surface cannot always be computed accurately.

There is a satisfactory agreement between observed and computed water surfaces for discharges above 75,000 second-feet, which demonstrates the substantial soundness of the basic theory. The computed flow line for 200,000 second-feet is considerably higher than the observed surface at the upper end. This may be explained by the fact that a uniform discharge per foot of crest was assumed. Actually the crest was submerged to sufficient depth to reduce the flow at the upstream end.

The line representing the piezometric pressure on the bottom of the channel falls uniformly below the observed mean water surface. This has been assumed by some experimenters to be due

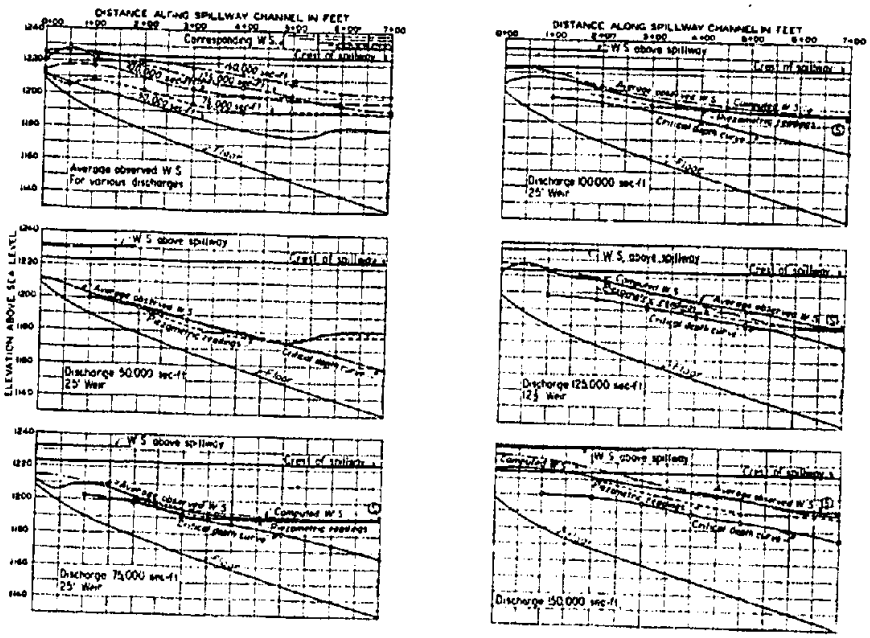


FIGURE 11—WATER SURFACE PROFILES IN TRAPEZOIDAL CHANNEL, WEIR ACROSS LOWER END, MODEL C-2

to the air content of the water; but measurement of air has shown that this is not always true. The difference may be due to the fact that the actual water depth over the piezometer openings, which were on the center line of the channel, was less than the mean depth of flow.

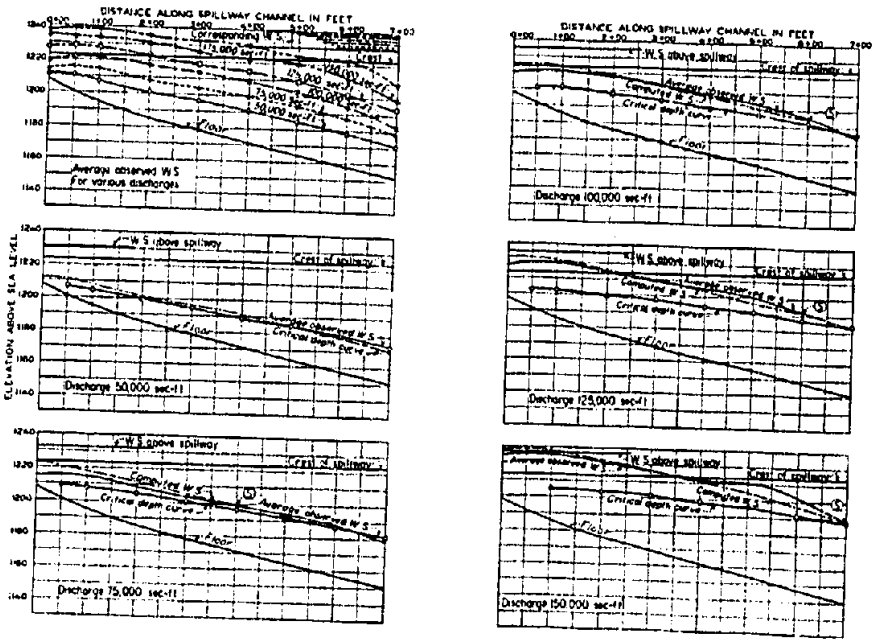
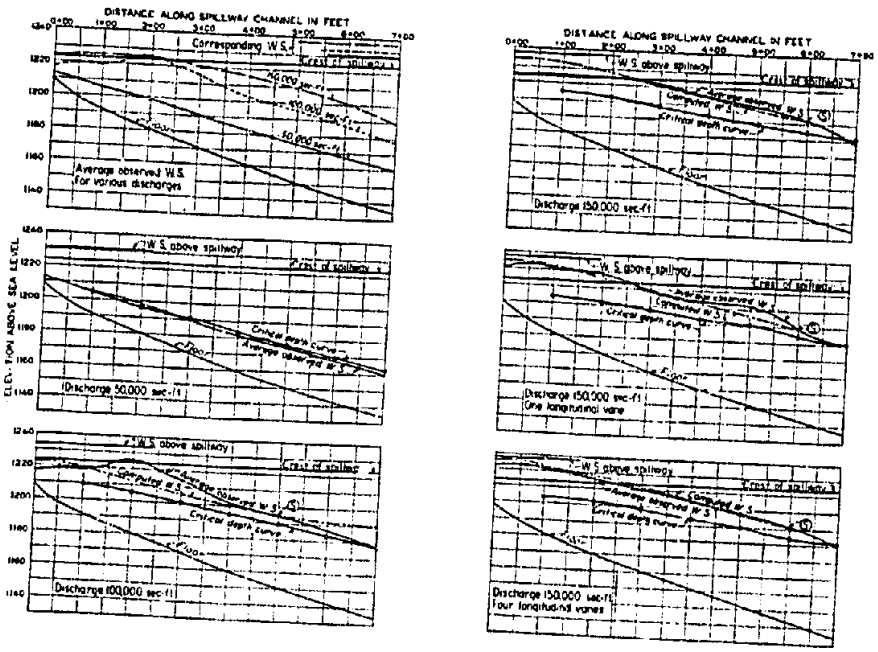


FIGURE 12—WATER SURFACE PROFILES IN TRAPEZOIDAL CHANNEL, FALSE BOTTOM IN MODEL C-2

The results of experiments with a cross weir at the lower end of the channel, shown in figure 11, also indicate the substantial accuracy of the theory. With this revision in design, the flow for 50,000 second-feet was again too near critical to enable a water surface to be computed. For the higher discharges there is satisfactory agreement between the computed and observed water surfaces, except at the lower end of the channel, where a persistent wave made measurement difficult. The cross weir caused submergence of the upstream end of the crest for a discharge of 150,000 second-feet.

The results of experiments using a false bottom in the spillway channel, shown in figure 12, also confirm the basic theory. The



Ⓢ Starting point for computed Water Surface

FIGURE 13—WATER SURFACE PROFILES IN CHANNEL WITH ROUNDED BOTTOM, MODEL C-2

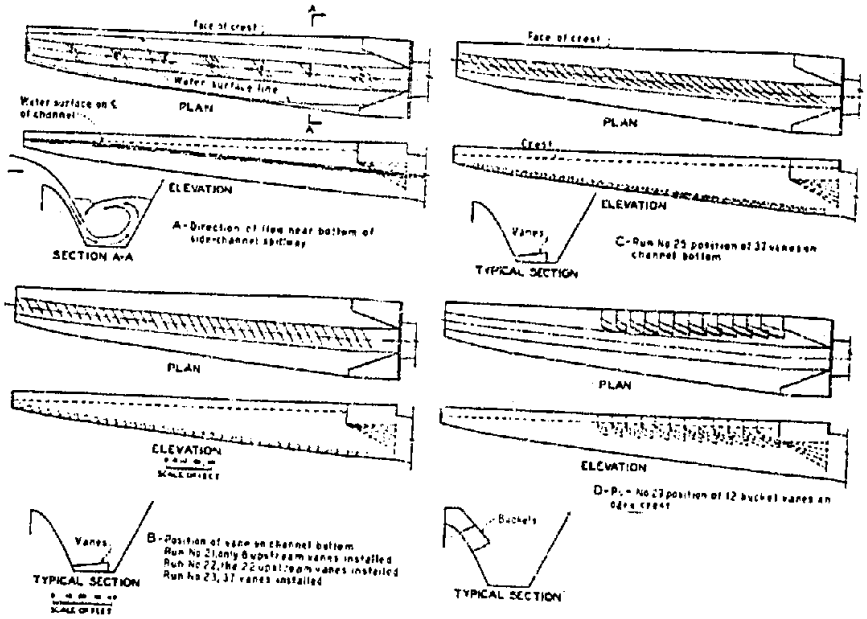


FIGURE 14—POSITION OF VANES AND DIRECTION OF FLOW, MODEL C-2

computed water surface, where it could be obtained, conforms with the observed water surface. The false floor also caused submergence of the upstream end of the crest for 150,000 second-feet.

Experiments made with a rounded bottom in the original channel, in general, substantiate the fundamental theory. The revised model is shown on figure 9 and the results of the tests are plotted on figure 13. An unexplainable wave formation occurred during a discharge of 100,000 second-feet.

13. Effect of Guide Vanes.—A number of attempts to increase the efficiency and decrease the cost of the spillway were made by

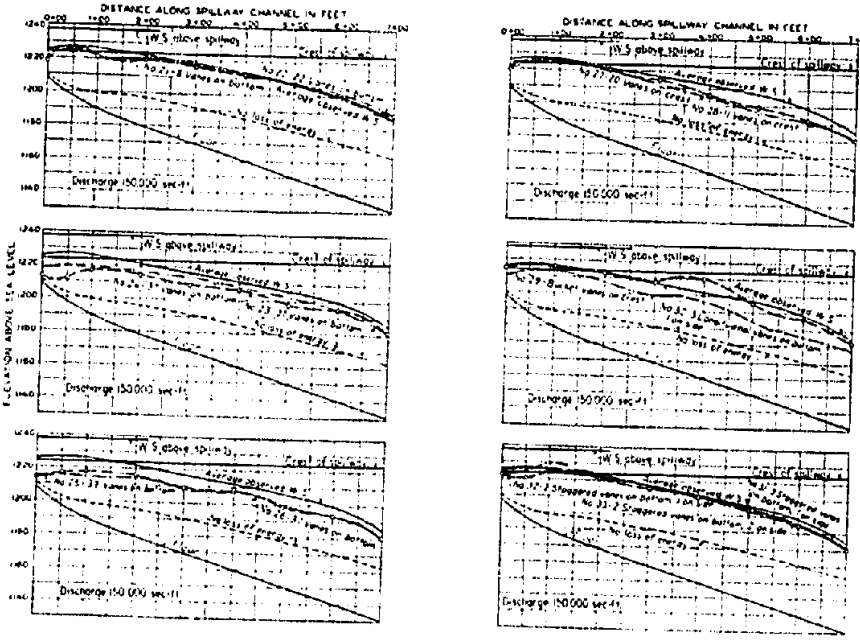


FIGURE 15—WATER SURFACE PROFILES IN CHANNEL WITH GUIDE VANES

introducing curved guide vanes in the channel or on the overflow section. Without vanes, nearly all the energy of the water flowing over the spillway crest was dissipated in the channel. With vanes, it was possible to deflect some of the water in a downstream direction, thus increasing the channel velocities and permitting the use of a smaller trough.

The first guide vanes were placed on the bottom of the channel. Observations with color injected into the water showed that the sheet flowing over the spillway crest plunged to the bottom of the channel, then moved across the bottom and flowed up the opposite side. The direction of the current is shown on figure 14-A. In the first test, eight vanes were placed on the bottom and their effect noted, see run 21, figure 14-B. Additional vanes, making totals of 22 and 37, were then added, see runs 22 and 23. The results of the tests are shown on figure 15. The average water surface was practically the same as in the original design, which shows that the velocity of flow in the channel was not materially increased. It was concluded that the vanes were not sufficiently inclined downstream. Runs 24, 25, and 26 were then made with 37 vanes at greater inclinations. The position of the vanes for run 25 is shown on figure 14-C. These runs showed lower water surfaces than those obtained with the vanes omitted, indicating that the vanes increased the velocity of the flow in the channel. Assuming no loss of energy, the water surface was computed to determine the limit to which the surface would fall if all the energy of the water flowing over the crest was utilized in producing velocity along the channel. The position of this surface is indicated by the line "No Loss of Energy" on figure 15. The results of experiments with vanes showed that it was possible to recover only a small part of the energy of the falling water. The recovery was not sufficient to greatly reduce the necessary size of channel, and it was evident that the cost of installing guide vanes of sufficient strength to resist the forces involved would be greater than the saving produced.

A series of vanes was also placed on the overfall section. The first vanes had surfaces curved in the downstream direction, see runs 27 and 28. Since the water passed off the side of the vanes, a set with edges curved upward was tested, see run 29, figure 14-D. These gave a better recovery, but in neither case was the saving sufficient to justify the cost of constructing the vanes.

Experiments were also performed with longitudinal vanes along the bottom and rear wall of the spillway, with both the trapezoidal and rounded bottom. The impingement on the vanes decreased the height reached by the water on the rear wall of the channel. They were effective in smoothing the water surface across the channel, but effected no recovery of energy. The mean water surface for these conditions was practically the same as without the vanes.

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computed water surface, where it could be obtained, conforms with the observed water surface. The false floor also caused submergence of the upstream end of the crest for 150,000 second-feet.

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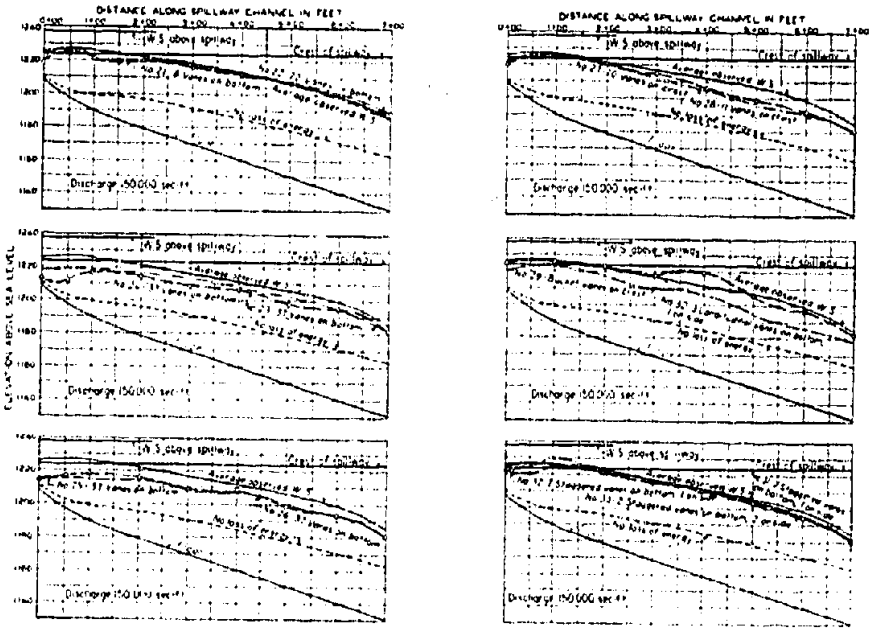

FIGURE 15—WATER SURFACE PROFILES IN CHANNEL WITH GUIDE VANES

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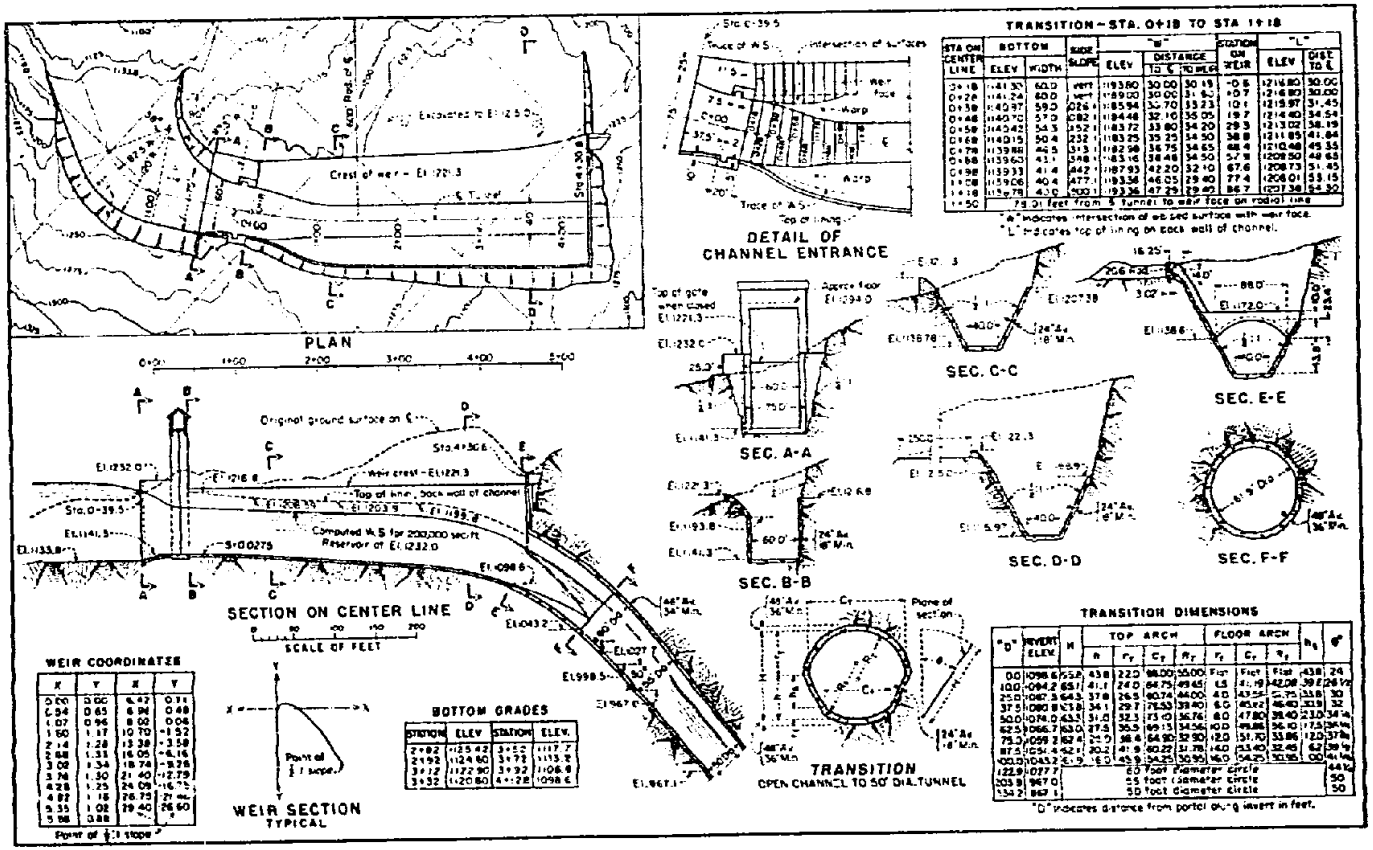


FIGURE 16—NEVADA SIDE-CHANNEL SPILLWAY WITH 400-FOOT FREE CREST AND 60-FOOT BY 80-FOOT STONEY GATE

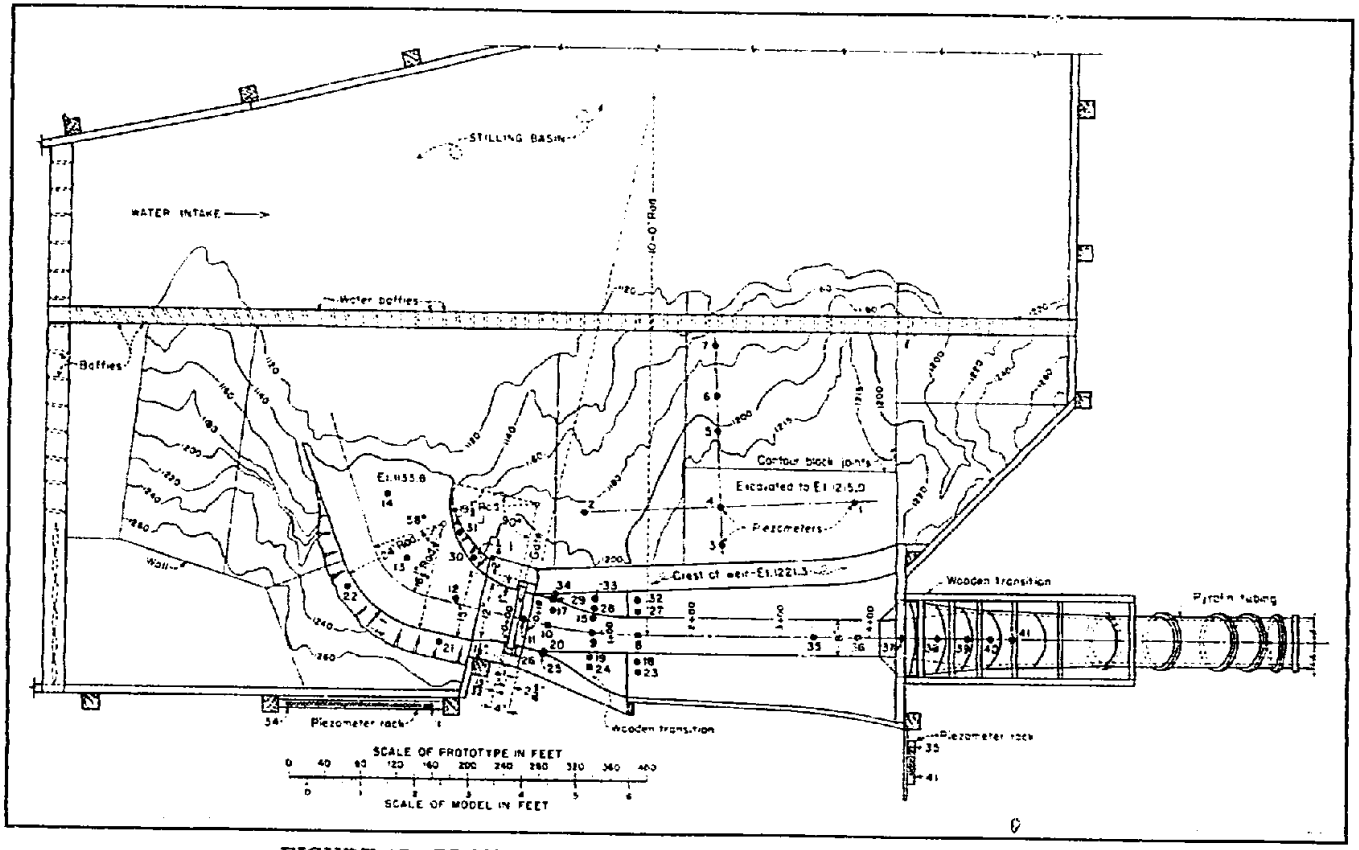
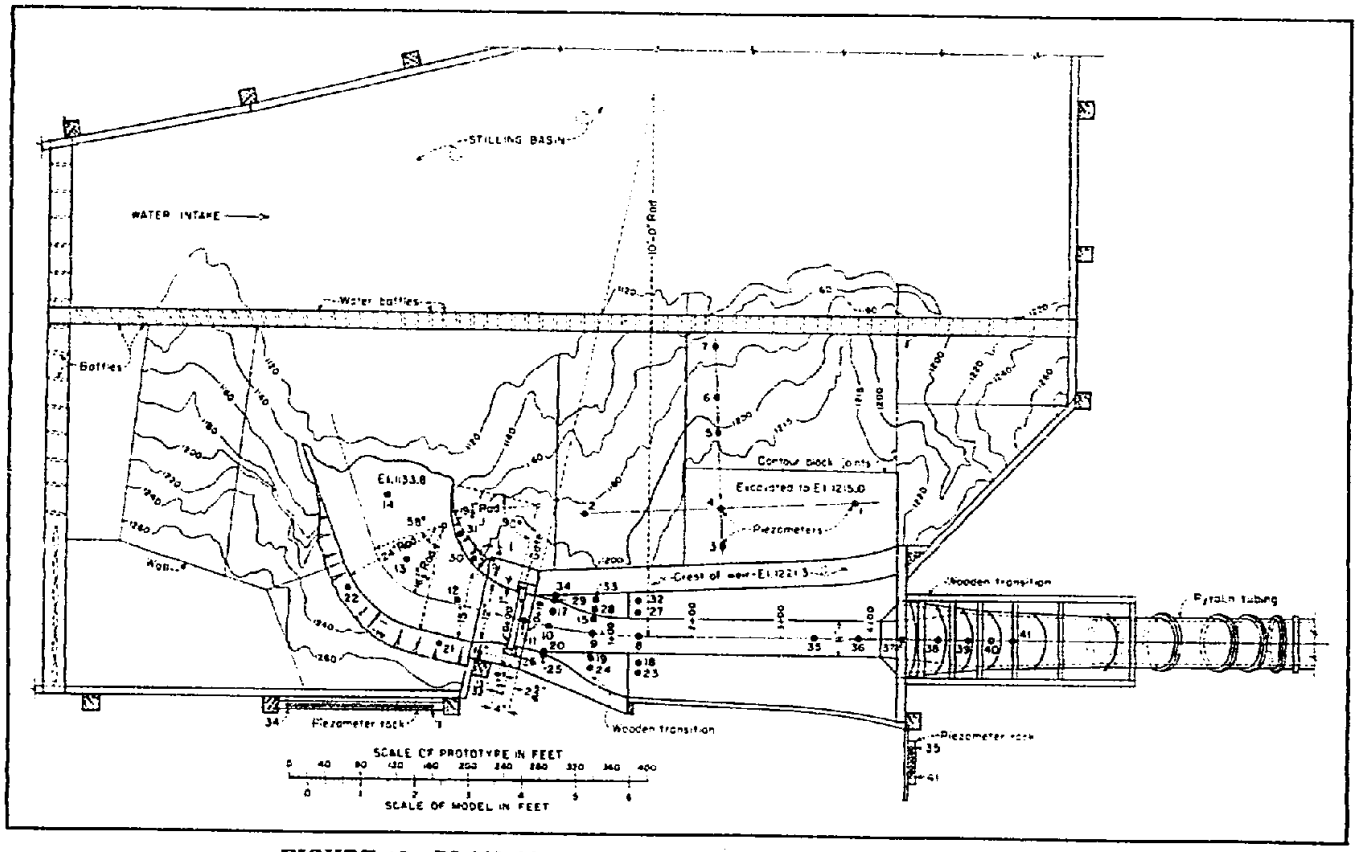


FIGURE 17—PLAN OF 1:60 MODEL OF STONEY GATE SPILLWAY



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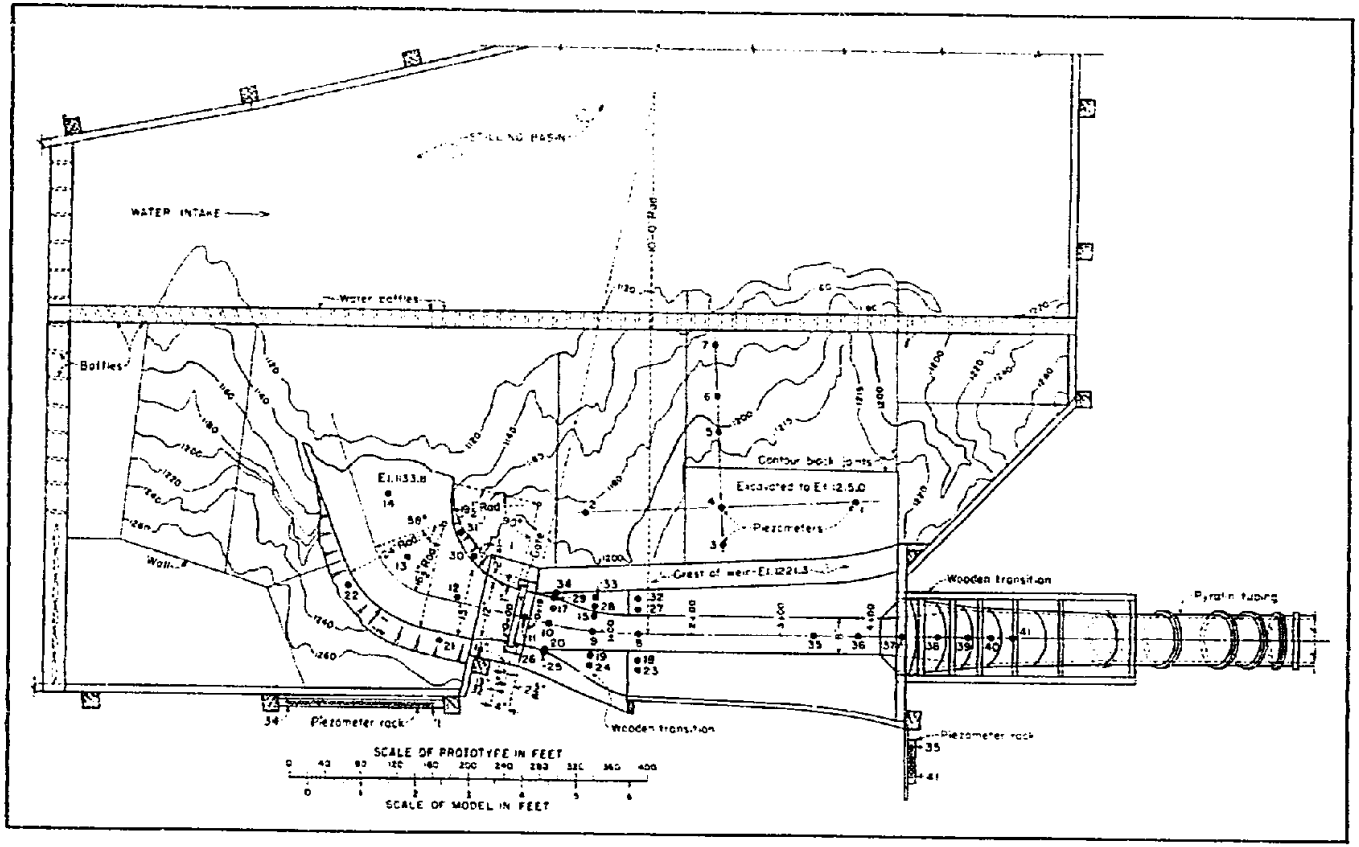

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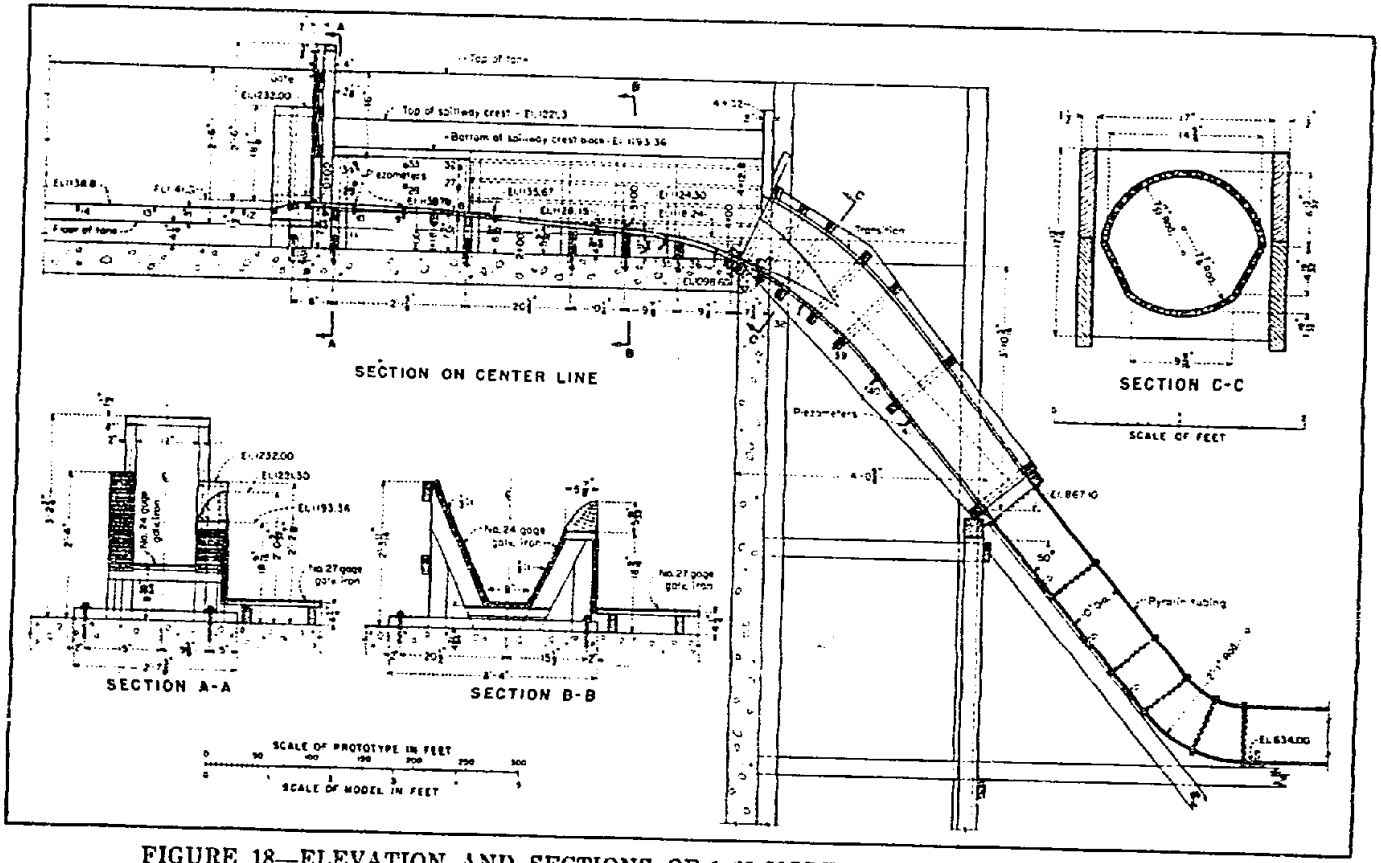


FIGURE 18—ELEVATION AND SECTIONS OF 1:60 MODEL OF STONEY GATE SPILLWAY

SIDE-CHANNEL SPILLWAY WITH STONEY GATE AND FREE CREST (C-3)

14. Design and Construction of Model.—When the decision was made to provide for a total spillway capacity of 400,000 instead of 200,000 second-feet, the first design developed consisted of a side-channel spillway with a crest length of 400 feet discharging a maximum of 125,000 second-feet, in combination with a 50- by 50-foot Stoney gate at the end having a capacity of 75,000 second-feet. Further study showed that a reduction in cost could be effected by increasing the size of the Stoney gate. This design, shown in figure 16, had a free crest length of 400 feet, capable of discharging 50,000 second-feet at a 10.7-foot head, and a Stoney gate, 60 feet wide by 80 feet high, with a capacity of 150,000 second-feet. The top of the Stoney gate, when closed, was at the same elevation as the spillway crest. Under this condition water could flow over the top of the gate as well as over the rounded crest.

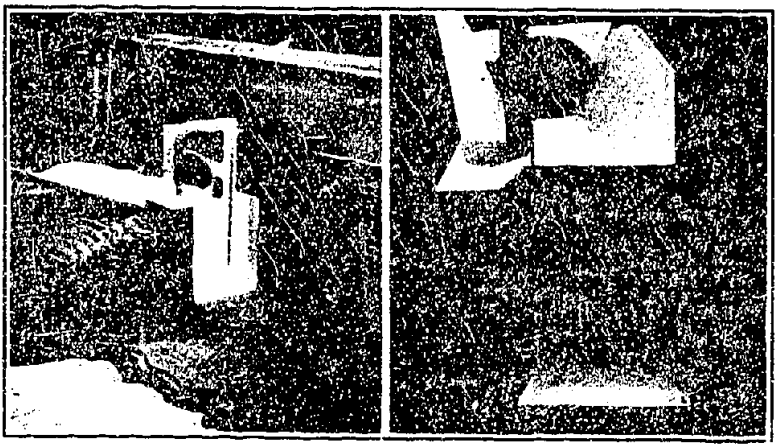


FIGURE 19—STONEY GATE SPILLWAY MODEL

A model of the Stoney gate spillway, shown in figures 17 and 18, was constructed on a scale of 1:60, at the Colorado State College laboratory. The model, not including the tunnel, was about 7 feet long, 1.5 feet high, and had a capacity of 7.2 second-feet.

The trough of the model, shown in figure 19, was constructed of a wooden frame covered with galvanized iron. The crest portion and the warped section, immediately downstream from the gate,

were of wood. In the assembled model, surfaces representing concrete were painted gray and the remainder painted with red enamel to furnish contrast in photographs. The topography of the canyon side in the vicinity of the spillway was of laminar wood construction. The transition from the trapezoidal side channel to the circular inclined tunnel was constructed of wood staves, held in place by a number of collars. The staves were painted on the inside and outside and coated with a thick layer of paraffin to prevent swelling.

Flow through the model was measured by the two-foot Cipolletti weir. The water surface in the channel was observed by a point gage, mounted on a carriage that operated along level parallel bars above the channel. Piezometers were installed in the channel transition, immediately downstream from the Stoney gate, to observe the pressures in that region. The locations of the piezometer openings are shown in figures 17 and 18.

15. Outline of Tests.—A large number of tests was made on the Stoney gate model to observe flow conditions and determine discharge capacities. A number of changes were made in the model in attempts to improve flow conditions. The following is an outline of the tests:

1. Original Design.
   - (a) Flow for probable conditions of operation.
   - (b) Flow over spillway crest only.
   - (c) Flow through gate only.
   - (d) Determination of discharge coefficient for Stoney gate.
   - (e) Investigation of effect of draw-down at Stoney gate on the discharge over the overflow crest.
2. Alterations of the Original Design.
   - (a) Effect of dentated baffles on channel floor.
   - (b) Flow conditions with various types of piers supporting the Stoney gate.
   - (c) Effect of alterations in approach channel to gate.
   - (d) Result of lengthening the transition between the rectangular gate section and the trapezoidal channel section.
   - (e) Effect of guide walls at ends of spillway crest.
   - (f) Flow conditions with channel floor raised; tests made to obtain data for drum-gate design.

3. Determination of Discharge Coefficients.
   (a) Determination of discharge coefficient for overflow section.

These tests are discussed in "Studies of Crests for Overfall Dams" Part IV, Bulletin 3, Final Reports, Boulder Canyon Project.

16. Normal Operating Conditions.—In the spillway shown on figure 16, the top of the Stoney gate, when closed, was at the same elevation as the top of the ogee crest. Under normal operating conditions the Stoney gate would be closed until the reservoir reached elevation 1232.0, at which time the combined flow over the gate and spillway crest would be approximately 60,000 second-feet. For greater discharges, the Stoney gate would be partially raised to prevent the reservoir from rising above elevation 1232.0. For flows between 60,000 and slightly over 80,000 second-feet, the gate would not be opened sufficiently to bring its top above the reservoir water level and there would be flow over, as well as under, the gate. For discharges above 80,000 second-feet, flow over the top of the gate would cease.

Flow conditions in the model were observed for each multiple of 20,000 second-feet. With the lower discharges, most of the water entered the spillway over the overflow crest and the conditions in the channel were similar to those in the free crest side-channel type. The water moved up the side of the channel opposite the crest to a higher elevation than on the side adjacent to the crest. For a discharge of 60,000 second-feet, there was a distinct standing wave at the back side of the channel, downstream from the gate. As the flow was increased, this wave became higher and moved downstream. The flow in the channel was very turbulent with a higher water surface on the back side of the channel. As the discharge approached design capacity, flow conditions became smoother, due to the greater discharge from beneath the gate and the relatively smaller flow over the crest. The most undesirable condition was for discharges between 120,000 and 130,000 second-feet. For all discharges, the water had a spiral motion in a counter-clockwise direction, looking downstream in the Nevada spillway, and the greater depth on the back side of the channel continued through the transition and into the inclined tunnel.

When water flowed over both the gate and crest, with the reservoir surface at elevation 1232.0, the flow at the downstream end of the transition was only slightly spiral in direction. With a

200,000 second-foot discharge, the water had some spiral motion, but the velocity of flow from the gate gave it a greater longitudinal component. With all the flow passing under the gate there was no spiral motion, but for 200,000 second-feet over the crest, with the gate closed, the spiral was very pronounced.

The design of the Stoney gate was based on the assumption that flow would drop to a depth of two-thirds the height of the reservoir level above the gate sill in accordance with the theory of critical flow. To this was applied a slight friction loss correction. Downstream from the gate, the design was based on the theory for side-channel spillways. A comparison of a computed and observed mean water surface profile, made to check these assumptions, is shown in figure 20-A. The agreement is remarkably close, except in the vicinity of the Stoney gate, where the observed water surface was substantially higher than the computed height. This discrepancy did not affect the flow over the overfall portion of the spillway, since the observed water surface was still below the crest. It does indicate, however, that there is a difference between the action of the water flowing through the gate and the theory of critical flow. That this disagreement with theory is not a special case is shown by similar results with flow through the gate only, plotted in figure 21, and by additional experiments conducted elsewhere.

Although the mean water surface in the channel for most runs agreed with the computed mean water surface, there were large differences in surface elevations at the two sides of the channel. The water surface levels at the two sides and the center line, for different flows over the entire range of capacity, are shown in figure 20.

A comparison of the depth of flow in the spillway and the pressure on the channel floor for no flow over the crest is shown on figure 21. The solid lines show the water surface along the center line, which in this case is very close to the mean depth. The dotted lines indicate the piezometric pressures on the channel floor, directly beneath the lines along which the water surfaces were observed.

The pressure at some points on the bottom was greater than indicated by the depth of flow, and at other points less. In general, the pressure was greater than the depth where the water surface was concave, viewed from above, and less than the depth where the water surface was convex. This difference was a result of non-parallel flow of the water particles. Where the surface is

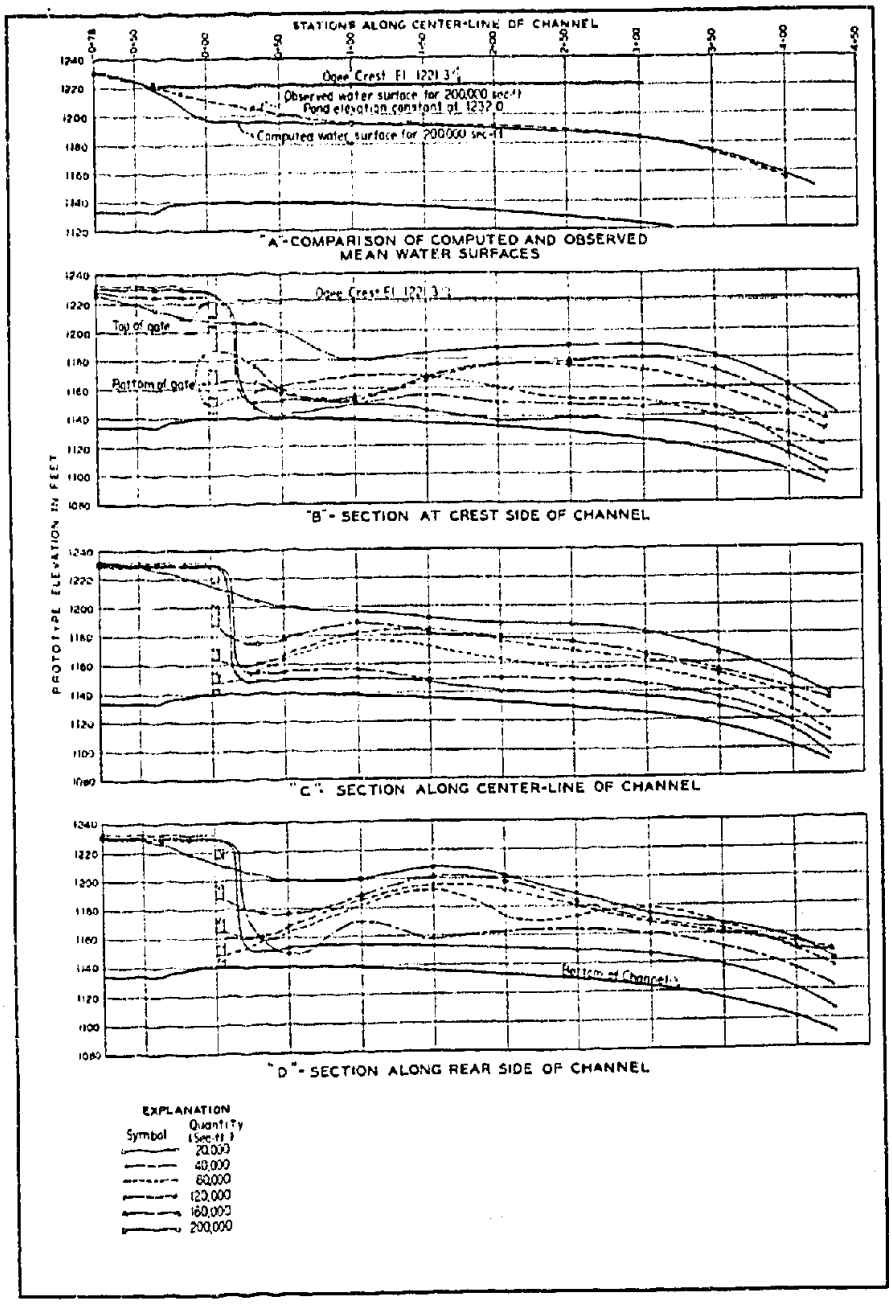


FIGURE 20—WATER SURFACE PROFILES OF STONEY GATE SPILLWAY WITH COMBINED FLOW OF GATE AND CREST, MODEL C-3

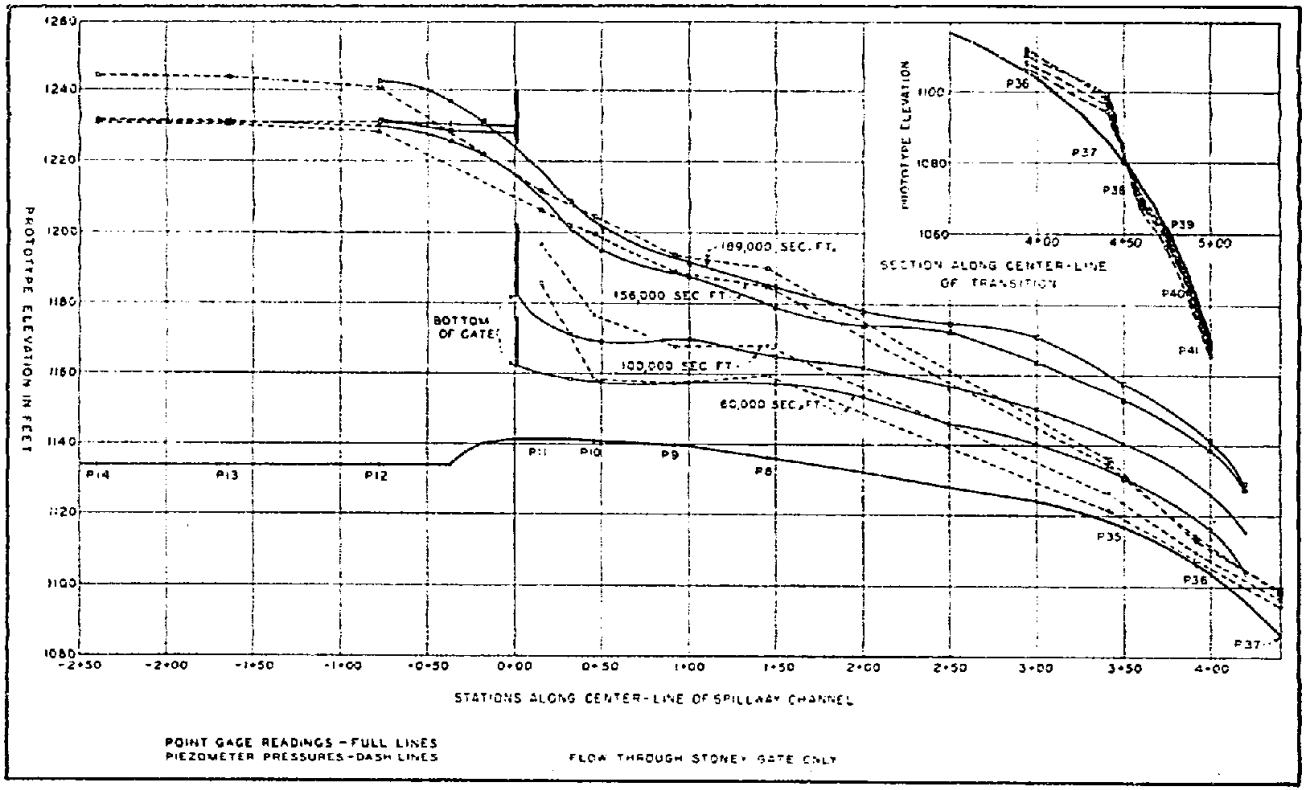


FIGURE 21—WATER SURFACE PROFILES AND PRESSURES ON CHANNEL BOTTOM, STONEY GATE SPILLWAY MODEL C-3

concave, particles near the surface move in curved paths, with an initial motion directed toward the channel bottom; but are deflected from this direction as they move along. According to Newton's laws of motion, they can only be diverted from a straight path by the application of a force. The force is applied by the pressure of the particles moving beneath. If these particles exert a force, there must be a corresponding reaction. The reaction is exerted downward through successive layers of particles until the bottom is reached. The excess pressure at the bottom is indicated by the difference in the height of the water surface in piezometers connected to the channel bottom, and the height of the surface of the flowing water in the channel. Similarly, when the surface is convex, pressure on the bottom is less than that indicated by the depth, since the force of gravity is influential in diverting the particles from their straight-line directions.

17. **Flow Over Spillway Crest.**—Observations were made on the flow in the channel with all the water flowing over the spillway crest. The discharges were increased to 200,000 second-feet, which was greater than the discharge for which the crest was designed, and necessitated a higher level in the reservoir than would be possible at Boulder Dam. Conditions of flow were similar to those found in the free-crest side channel, model C-2.

18. **Flow Through Stoney Gate.**—Observations were made on flow through the Stoney gate with the spillway crest blocked. A distinct wave occurred at all flows, commencing at the curve in the channel, immediately downstream from the gate, and taking a diagonal course to the opposite or crest side of the channel near the tunnel entrance. The disturbance was especially undesirable at the higher discharges. Water flowed down the channel with little disturbance during smaller discharges and the wave was not of sufficient size to cause spiral flow in the tunnel.

Since a large part of the flow was expected to pass through the gate, it was necessary to know the gate capacity with considerable accuracy. General theories of discharge for such conditions were known, but little experimental data existed from which accurate estimates could be made. A series of experiments on the flow through the Stoney gate was performed to furnish the necessary data.

During the passage of a flood, the gate could operate under four conditions:

1. Gate closed and water flowing over the top.
2. Gate partly open and water flowing both over and under it.
3. Gate partly open and water flowing under it.
4. Gate entirely open and not in contact with the water.

The flow for the first condition would be similar to the flow over a flat-crested weir. It was not believed that results obtained from the model would be any more reliable than those obtained by using coefficients derived from experiments on flat-crested weirs. Since the discharge over the gate would always be small, accuracy in estimating it was not important. Detailed designs for the gate had already been prepared and consequently no measurements were made on the model for this condition.

The second condition was virtually a combination of the first and third. Since flows under and over the gate would be small and would not appreciably interfere with each other, the discharges could be estimated with sufficient accuracy. In addition, it would have been difficult to measure the two discharges when both were flowing at the same time. Therefore quantitative results for this condition were not secured.

As the greater discharge would occur for the third and fourth conditions, extensive experiments were made to determine discharge coefficients for these two stages of operation. Since different discharge formulas apply to the two cases, they will be discussed separately.

When the Stoney gate was partly open with no water passing over the top, the condition was analogous to flow through an orifice with suppressed contractions on the bottom and the two sides upstream from the orifice, and confined on the sides and bottom on the downstream side. The downstream depth was not sufficient to submerge the orifice and under these conditions the general formula for discharge is:

$$Q = CWD\sqrt{2gH'}$$

where

C = Coefficient representing the ratio of the measured discharge to the theoretical discharge, see figure 22.
W = Width of gate opening, feet.

H' = Head measured from the water surface in the reservoir, where the velocity of approach was negligible, to the bottom of the gate, feet.

D = Opening of the gate or the vertical height of the orifice opening, feet.

Since the outflowing stream was supported by the floor and confined by the side walls of the channel, it did not fall freely as in the usual case of an orifice. Therefore, a back pressure was exerted as shown by the arrows in the triangle D-J-E of figure 22. The pressure acting downstream, tending to cause flow, is shown by the arrows of the trapezoid D-F-B-J. The net effective pressure is shown by the heavy arrows of the rectangle F-D-J-G, which is that due to a head H' measured to the bottom of the gate.

A better theoretical formula might be developed by measuring D and H' from the most contracted section of the issuing stream, but the elevation of this point was not observed. Moreover, such a form would have a limited practical application.

The results of the coefficient determinations are shown on figure 22. The curve shows the value of the coefficient C for various values of D/H'. This curve has been prepared so as to make it readily applicable to other cases.

It is believed, because of the gradually converging section of the approach channel, that the effect of the side contractions on the orifice is negligible. If this is true, then by the principle of similitude, the results on the model should be applicable to other cases in which the stream is supported on the bottom and confined by suppressed side contraction, providing the values of D/H' are the same.

The curve on figure 22 is well established up to values of D/H' slightly above 0.8. No observations were made for higher values except for free flow conditions where the bottom of the gate merely touched the surface of the water. This occurred with $D/H' = 3.42$. The curve was extended as a dotted line above the D/H' value of 0.8, which indicates that the values in this range are approximate.

With the probable plan of operation of the outlets during flood, the Stoney gates would be completely open only when maximum discharges of the spillway were required. For this condition the reservoir would be at elevation 1232.0. To make the tests as complete as possible, discharges through the gate opening were made for reservoir water levels above, below, and at elevation 1232.0.

Since the channel bottom, downstream from the Stoney gate opening, sloped steeply and expanded slightly, it maintained the stream at subcritical depths. The flow passed through the critical

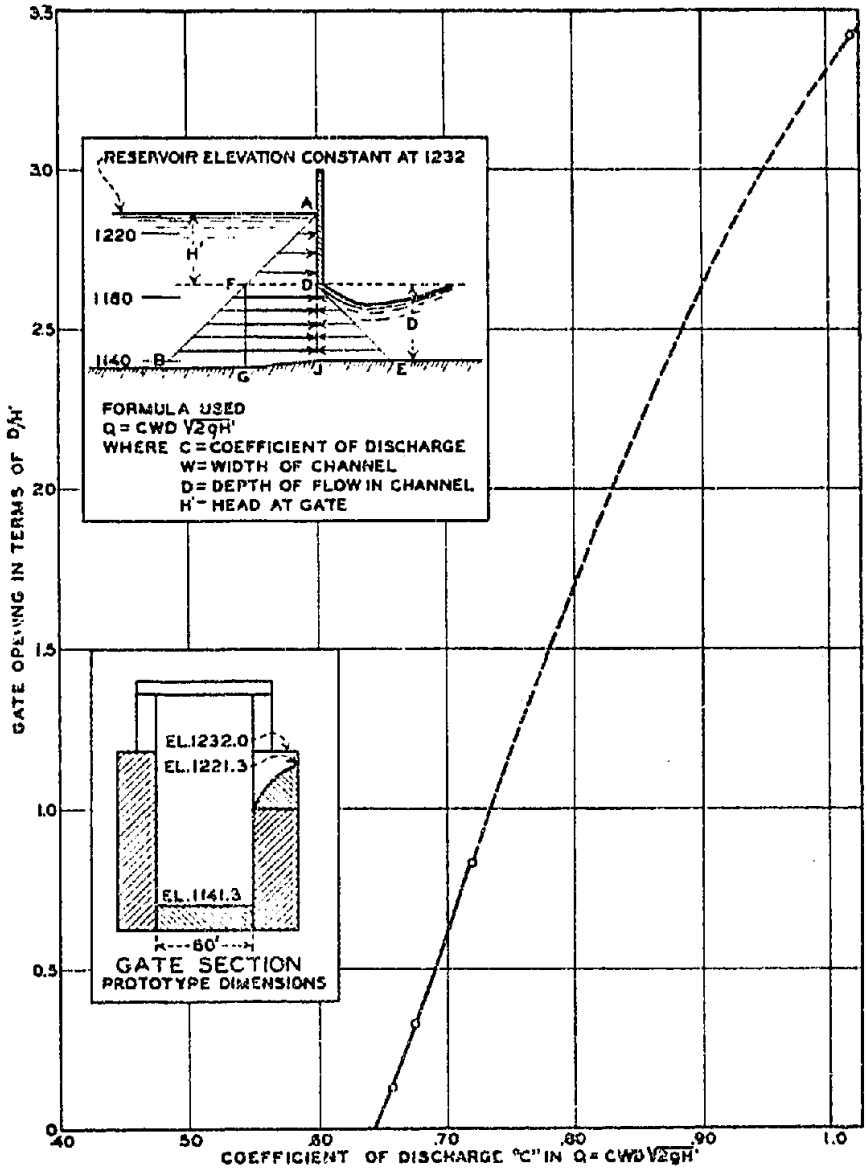


FIGURE 22—DISCHARGE COEFFICIENTS FOR FLOW UNDER STONEY GATE

depth near the Stoney gate structure which acted as a control. According to the theory of critical flow, the water surface, as it passed through the gate, should theoretically have dropped to a depth of two-thirds the height of the reservoir level above the gate sill, and the velocity head should have been one-third of this height, or one-half the depth of the water passing over the sill. The theoretical velocity head would be $\sqrt{2gH/3}$ and the area of flow $\frac{2}{3}WH$, giving a discharge of $\frac{2}{3}WH\sqrt{2gH/3}$, in which W is the width of the gate opening and H is the height of the reservoir above the gate sill. The velocity of approach in the reservoir is considered negligible. Substituting the numerical value of g in the above equation, the formula becomes:

$$Q = 3.089\ WH^{3/2}$$

The results of the experiments were substituted in the formula

$$Q = 3.089\ KWH^{3/2}$$

in which K represents the ratio of the observed to the theoretical discharge. They were also substituted in the more useful formula:

$$Q = CWH^{3/2}$$

Forms of the approach channel, leading to the gate, were tested and the observed values of the coefficients, C and K, plotted in figure 23. The coefficients for all types of gate entrance with the topography in place, show little variation; values of C ranged from 2.93 to 3.075, and values of K, from 0.950 to 0.996. This indicates that the theoretical value can be approached with almost any similar converging channel and that small differences in shape are relatively unimportant. The best results at the higher flows were obtained with the type IV entrance, test 18, figure 23. This was to be expected, as a more gradually contracted entrance was obtained; and the false wall offered less friction to the flow than the model topography. The results, without the model topography, test 23, show considerably lower coefficients due to the sharp corners around which it was necessary for the water to flow in entering the gate.

Although the discharge seems to agree closely with the theoretical formula, based on ordinary assumptions of flow through a contraction in a channel, the water surface, as previously mentioned, departs considerably from the position which this theory would indicate. The theory indicated that the drop in water surface level should be one-third the difference in elevation between the reservoir surface and the gate sill; but the observed surface on figure 20-A shows a drop of about one-sixth the difference, or only

one-half that indicated by the theory. Much the same conditions are shown for the flow through the Stoney gate on figure 21. Experiments performed elsewhere show similar results[6]. The cause of this divergence of fact and theory is not definitely known, but is probably caused, as previously explained, by the difference in the observed pressure in the channel and the depth of flow. The theory which indicates a one-third drop assumes flow in parallel stream lines which would produce pressures corresponding to the depth. The conditions of flow with a one-third drop are such that a slight difference of pressure can cause a material effect on the discharge. Time was not available for a more thorough study of this phenomena, but it offers an interesting field for further experimentation.

19. Interference of Flow.—One of the uncertainties in the design of the Stoney gate spillway was the extent to which interference would occur between the discharge from the Stoney gate and the spillway crest. Since the capacity of the gate was large, it was feared that the draw-down produced in the reservoir near the gate would greatly reduce the effective head on the overflow crest, and thus materially reduce the crest discharge. In turn the flow over the overflow section would produce a draw-down of its own which would reduce the flow through the Stoney gate. A study was made to determine the magnitude of the combined draw-down effect. The results of the study, given on figure 24, show the magnitude of the interference when the reservoir is held at elevation 1232.0 and different flows passed through the Stoney gate. The flow over the crest, without interference, was computed using crest coefficients determined from experiments with the Stoney gate closed. The flow through the Stoney gate, without interference, was computed using coefficients shown on figure 21, determined for no flow over the crest. A comparison of the sum of the computed flows with the observed flow, with both crest and gate acting together, is also shown on figure 24. With a discharge of 200,000 second-feet, the interference reduced the capacity 5,000 second-feet, or 2.5 percent. This represents 9.2 percent of the discharge over the spillway crest. Since the observed profile of flow through the Stoney gate shows only a slight drop upstream from the gate, near the adjacent end of the crest, a reduction of 9.2 percent is large for the overflow crest. Since time was not available to run

[6]Trans. Am. Soc. C. E., vol. 83, 1920, pp. 1183-1185.

additional tests to determine this effect, and data from other tests were meager, the interference effect may be somewhat in error.

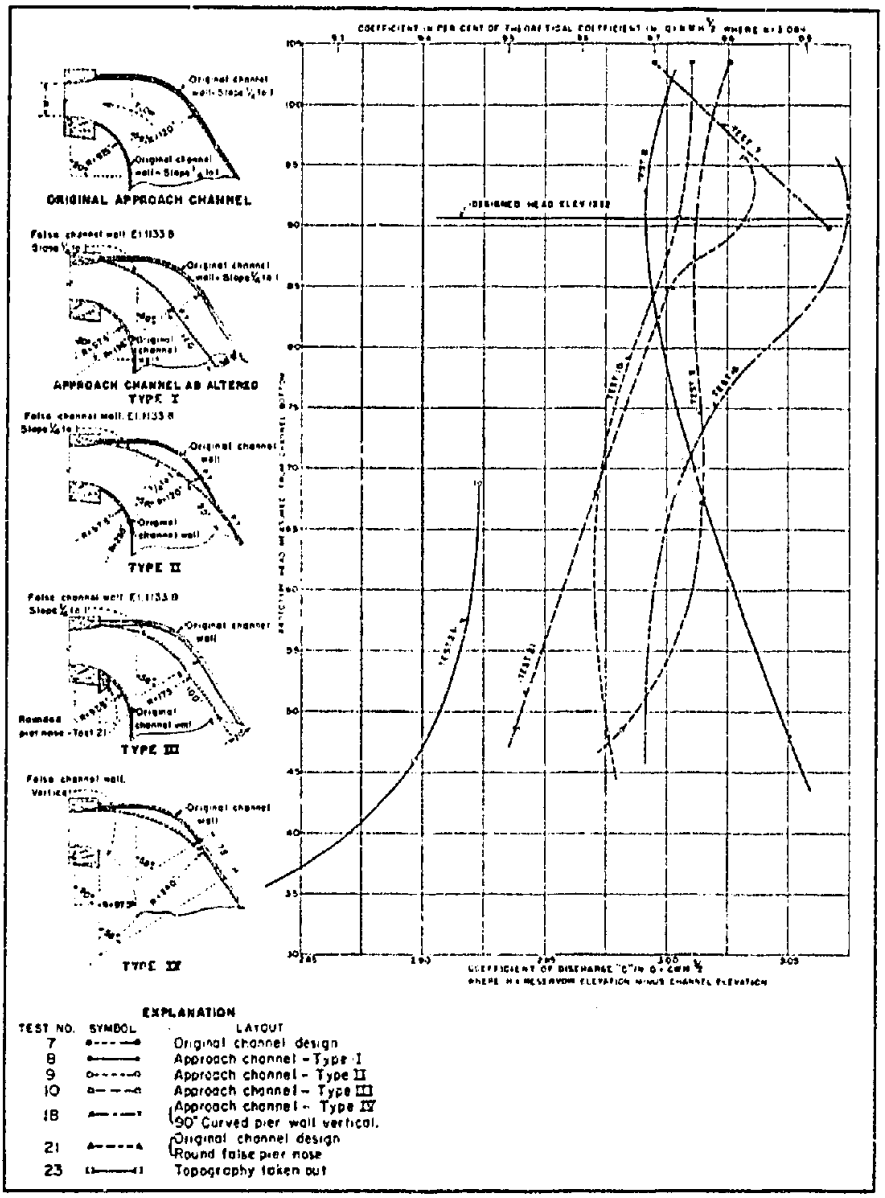


FIGURE 23—DISCHARGE COEFFICIENTS, STONEY GATE ENTIRELY OPEN, MODEL C-3

Visual observations of the model in action indicate that the interference was small and probably did not exceed 2.5 percent.

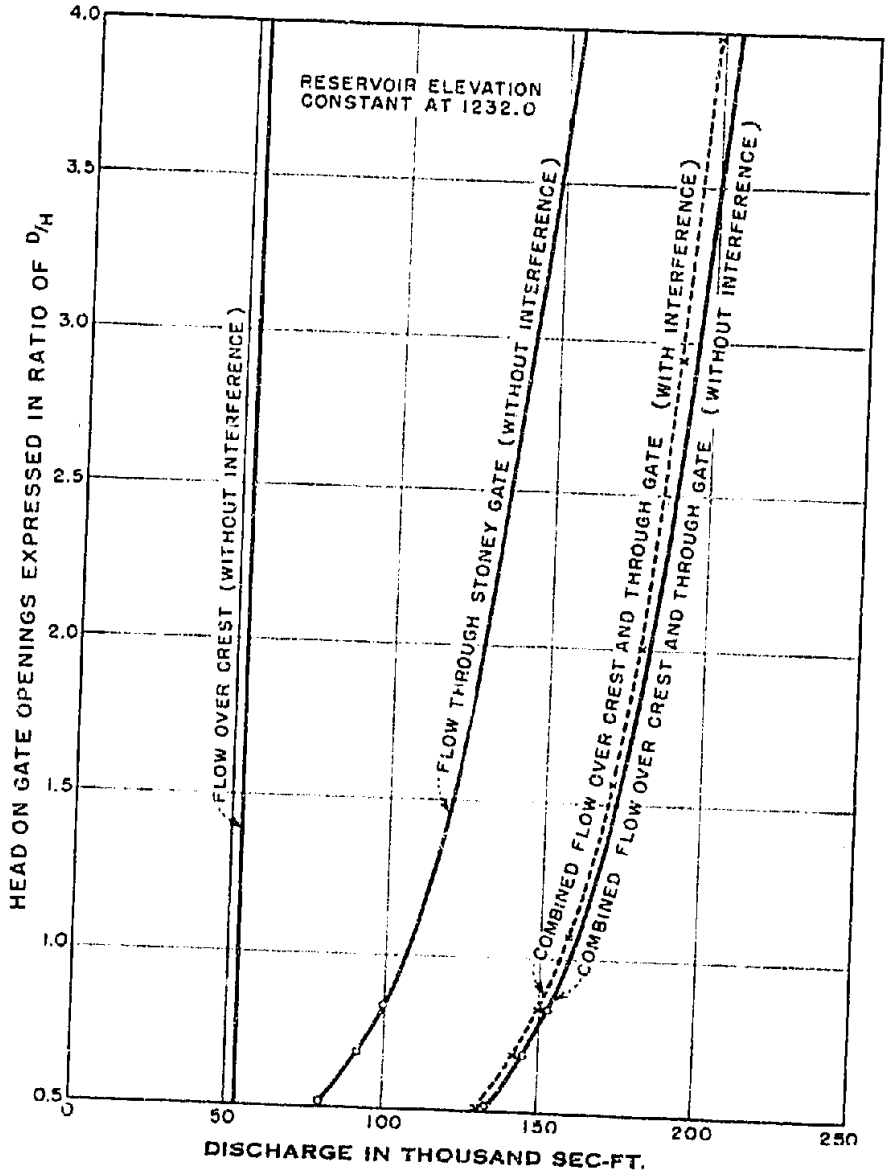


FIGURE 24—INTERFERENCE OF FLOWS THROUGH STONEY GATE AND OVER SPILLWAY CREST

20. Dentated Baffles on Channel Floor.—A row of dentated baffles was placed on the channel floor to dissipate the energy of the water falling over the spillway crest. The baffles, shown in figure 25, had curved faces which tended to deflect the water back toward the crest side. For lower discharges, the action of the baffles was satisfactory because they reduced the height to which the water would rise on the rear wall of the spillway. For higher discharges, the effect of the baffles was less, since the water passing over the crest represented a smaller proportion of the total discharge and the deeper water in the channel afforded a more efficient dissipation of energy. As the baffles obstructed the flow from the gate and were not a practical installation, no further study was made of this phase of the problem.

FIGURE 25—STONEY GATE MODEL WITH 15-FOOT DENTATED BAFFLE ON CHANNEL FLOOR

21. Conditions Above and Below Gate.—As originally designed, both sides of the gate structure presented sharp corners to the flow entering the gate. The corner on the hill side caused little disturbance; but a pronounced swirl occurred on the opposite side, due, largely, to the water approaching the gate at a sharp angle. A rounded nose was placed on the outer pier to eliminate the swirl;

but this did not appreciably improve the flow around the end of the gate pier. A curved approach wall, actually an extension of the curved face of the outer pier, was then installed as shown in figure 26. This eliminated the flow around the end of the pier and produced smooth entrance conditions.

FIGURE 26—APPROACH CHANNEL TO STONEY GATE

Experiments were made to determine the possibility of reducing the cost of the spillway by reducing the excavation in the channel approaching the Stoney gate. Four shapes of approach channel, formed by temporary sheet-iron walls, were tested, as shown in figure 23. In the original design, a zone of eddying water existed along the hill-side wall of the approach channel. With the channel narrowed, this condition was removed and the flow approaching the gate was not adversely affected. It was found that the size of the approach channel could be reduced nearly one-third without appreciable detriment to the action of the entering water. Especially good conditions were obtained by narrowing the approach channel on the hill-side wall and extending the outer curved wall as shown for type IV, figure 23. This produced very smooth, well balanced entrance conditions.

The shape of the channel approaching the gate did not greatly influence the flow in the side-channel section, as the water upstream from the gate was flowing above the critical depth. It passed through the critical depth near the gate section and flowed at less than critical depth downstream from that point. When water is

flowing at a supercritical velocity, it is very difficult to change its direction without producing a disturbance. The transition from the gate section to the trapezoidal channel section was quite short, causing a rapid enlargement of the channel on the right side, see figure 17. This condition, together with the fact that the center line of the channel was on a curve, produced a sizable wave which rose partly up the right wall, immediately downstream from the transition. By making the transition longer, and the change of direction less abrupt, the height of the wave was considerably reduced, although it was still pronounced.

It was desired to make a few major changes in the model, one of which was to straighten the spillway channel below the Stoney gate; but time was not available for further testing of this type of spillway. It is believed that by straightening the spillway channel and placing all the curvature in the approach to the gate, much better flow conditions in the spillway and tunnel could have been obtained. This, however, would have greatly increased the cost of the structure. Estimates showed that a side-channel spillway with drum gates could be adapted to the conditions at a reasonable cost. Attempts to improve the Stoney gate model were discontinued and further tests on it were made only as an aid in the design of the drum-gate type.

22. Effect of Channel Slope.—The latter part of the testing on the Stoney gate model was carried on simultaneously with tests on the first drum-gate model at the Montrose laboratory. Experiments were made on the Stoney gate model, with the gate blocked, to determine the maximum slope of floor satisfactory for the drum-gate spillway. False floors, having uniform slopes of 0.037, 0.075, and 0.112, were tested in the channel.

At the lower discharges, the flow over the crest swept entirely across the channel floor, forming a high wave on the side opposite the spillway crest. At maximum discharge, rough conditions were produced with all slopes. The flatter slopes produced less longitudinal velocity and greater depth of water in the channel, and were therefore more desirable.

SIDE-CHANNEL SPILLWAY WITH DRUM GATES (M-1 AND M-2)

23. Design and Construction of 1:20 Model.—Estimates

showed that a spillway with drum gates could be built at a lower cost than the Stoney gate spillway structure. As the model experiments indicated that the Stoney gate design would not be entirely satisfactory, a model of a drum-gate spillway was constructed on a 1:20 scale at the Montrose laboratory. The model, shown in figure 27, was supported on piles, driven to firm shale to prevent possible settlement. A sloping floor was first constructed. This served as the side-channel bottom, carried the buttresses supporting the rear wall of the spillway channel, and supported one end of the ribs for the downstream face of the overfall section. The other end of the ribs on the overfall side was supported by a framing which rested directly on the piling. The piers were accurately shaped from solid blocks of wood. The gates were built to represent only the upper faces and were raised and lowered by adjustable screws located below the crest, as shown on figure 27. This permitted a much simpler construction than would be required by an exact duplication of the drum gate, and from the standpoint of hydraulic action produced the same result. The completed spillway model is shown in figure 28.

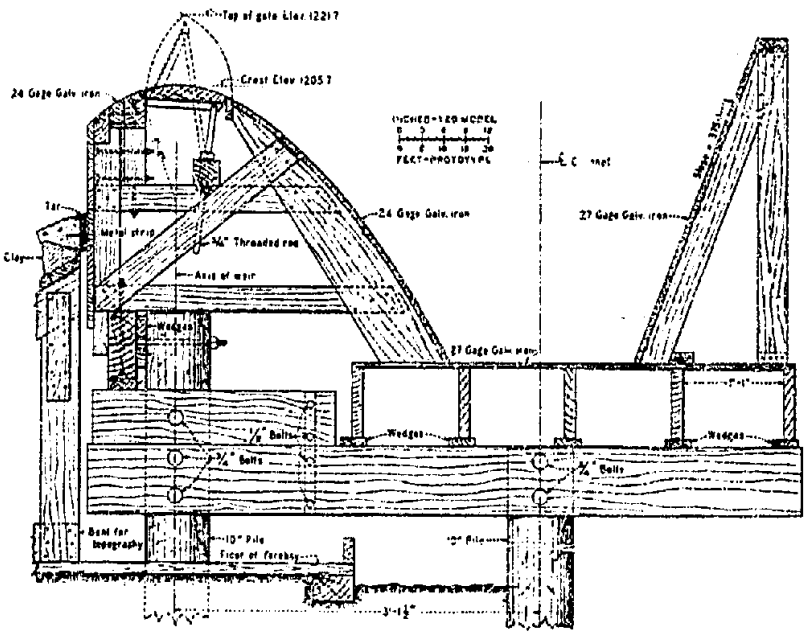
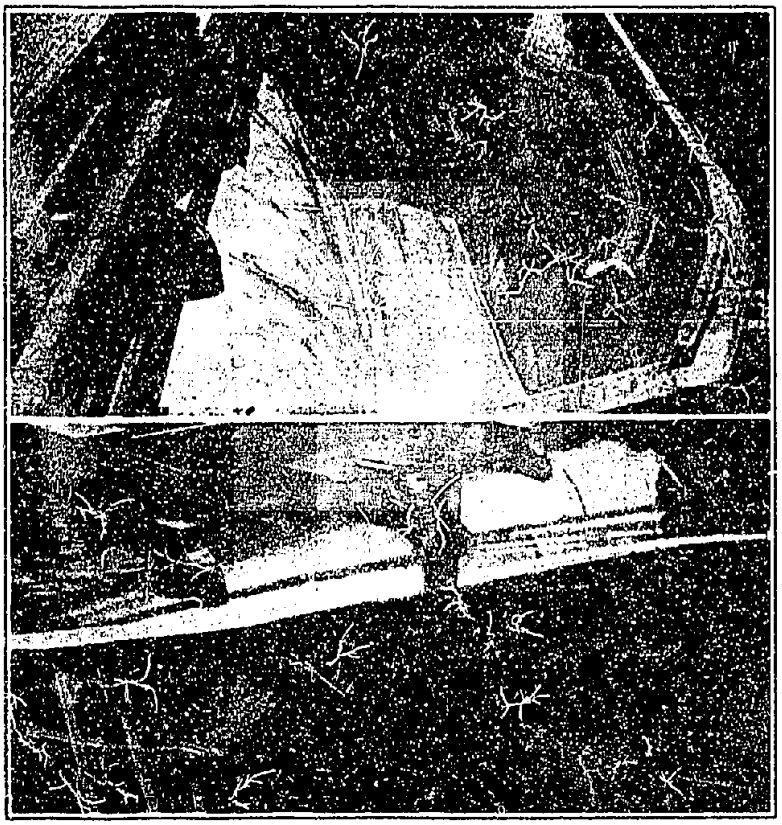


FIGURE 27—TYPICAL CROSS SECTION OF FIRST MODEL OF DRUM-GATE SPILLWAY



A. (Upper) Looking Downstream
B. (Lower) Side View Showing Overflow Face

FIGURE 28—MODEL OF ORIGINAL DESIGN OF DRUM-GATE SPILLWAY

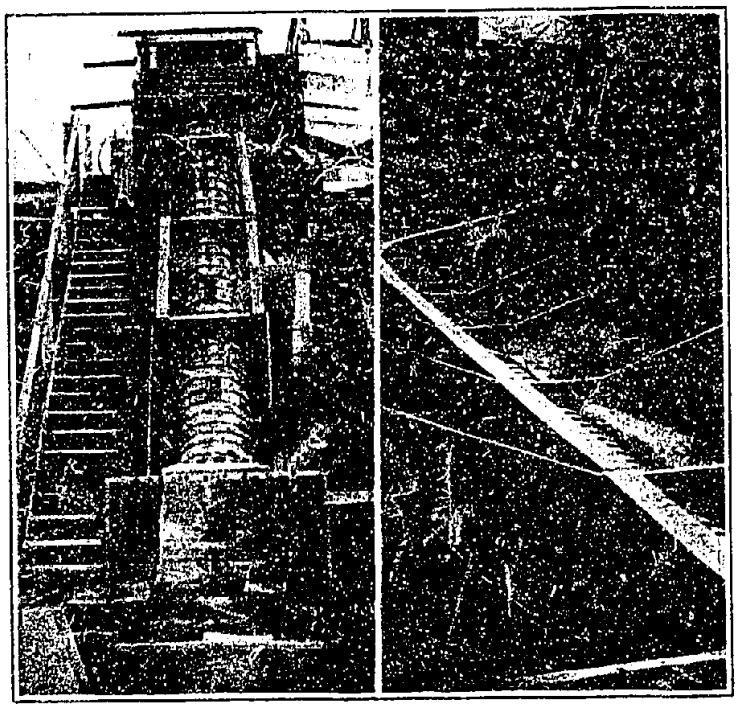
The model of the transition from the trapezoidal channel section to the circular tunnel section, see figure 29-A, was formed by fitting splined redwood staves to the inside of accurately cut wooden collars, similar to those used in the transition of the Stoney gate model. These collars were held in position by iron rods, to prevent distortion from the forces arising from the bent staves. The transition was supported on piles driven to shale.

The converging circular section of the inclined tunnel was formed by wedge-shaped staves. The pipe was carried in a cradle, supported at its upper end by piles, and at the lower end by a

concrete block forming the downstream vertical bend of the inclined tunnel. The concrete block was connected to the wood-stave pipes by steel sleeves. Two windows, fitted with removable concrete plugs, were cast in the top of the bend to permit observation of the action of the water.

The horizontal tunnel and horizontal bend, figure 29-B, were formed of 30-inch wood-stave pipe. The pipe was carried in collars and braced on the sides of the trench to prevent displacement in case of an accident which might fill the trench with water and tend to float the pipe. The horizontal bend had a radius of 41 feet, model dimension, which was much shorter than ordinarily considered feasible for a wood-stave pipe. Windows were placed in the top of the pipe to permit observation of flow conditions.

Except for a shifting of the inclined tunnel and vertical bend a short distance downstream, the design of the entire tunnel re-



A. Inclined Tunnel and Vertical Bend B. Horizontal Bend

FIGURE 29—MODEL OF TUNNEL DOWSTREAM FROM SPILLWAY, SCALE 1:20

molded the same for Models M-1, M-3, and M-5. Retaining walls were placed in the trench at the vertical bend and a gravel-covered tile drain was laid along the bottom of the trench to intercept any leakage or seepage and carry it back to the irrigation canal as a safeguard against sliding banks.

24. Results from Original Design.—Extensive experiments were conducted on the first model, M-1, of the drum-gate side-channel spillway. A large variety of devices was tried in an attempt to improve flow conditions in the spillway channel and inclined tunnel. Quantitative experiments were made on the depth of flow in the spillway channel and on the discharge coefficients of the spillway crest.

The prototype design for model M-1 is shown on figure 30. It had four drum gates, each 100 feet long, separated by piers 10 feet in width. The channel bottom had a constant slope of 0.0232 and a width varying from 26.5 to 52.4 feet. The rear wall had a slope of 0.375:1. The dimensions were computed according to the side-channel spillway theory previously discussed.

Flow conditions in the model were not favorable. The velocity of flow along the channel was high because of the steep bottom slope, and the cross section of the flowing water was relatively small. The depth of flow was therefore less than in the models previously investigated, and the average fall between the reservoir and the water surface in the channel was greater.

The combination of greater fall and smaller depth of flow resulted in a high wave on the back wall of the channel, which caused considerable impact on the end wall at the portal of the tunnel during discharges of 100,000 second-feet and over. The action of the model for a discharge of 100,000 second-feet is shown on figure 31. The water surface in the channel was not as rough during higher discharges. The flow for a discharge of 200,000 second-feet is shown on figure 32. For discharges less than 150,000 second-feet, the sheet of water flowing over the spillway crest swept the floor clear at the upper end of the channel. Figure 33 shows this action during a discharge of 100,000 second-feet.

Cross sections of the water surface were taken at a number of points along the side channel. Typical cross sections for three conditions of flow are shown on figure 34 and longitudinal profiles of the mean water surface for two conditions are shown on figure 35. The upper profile shows that the mean water surface for the

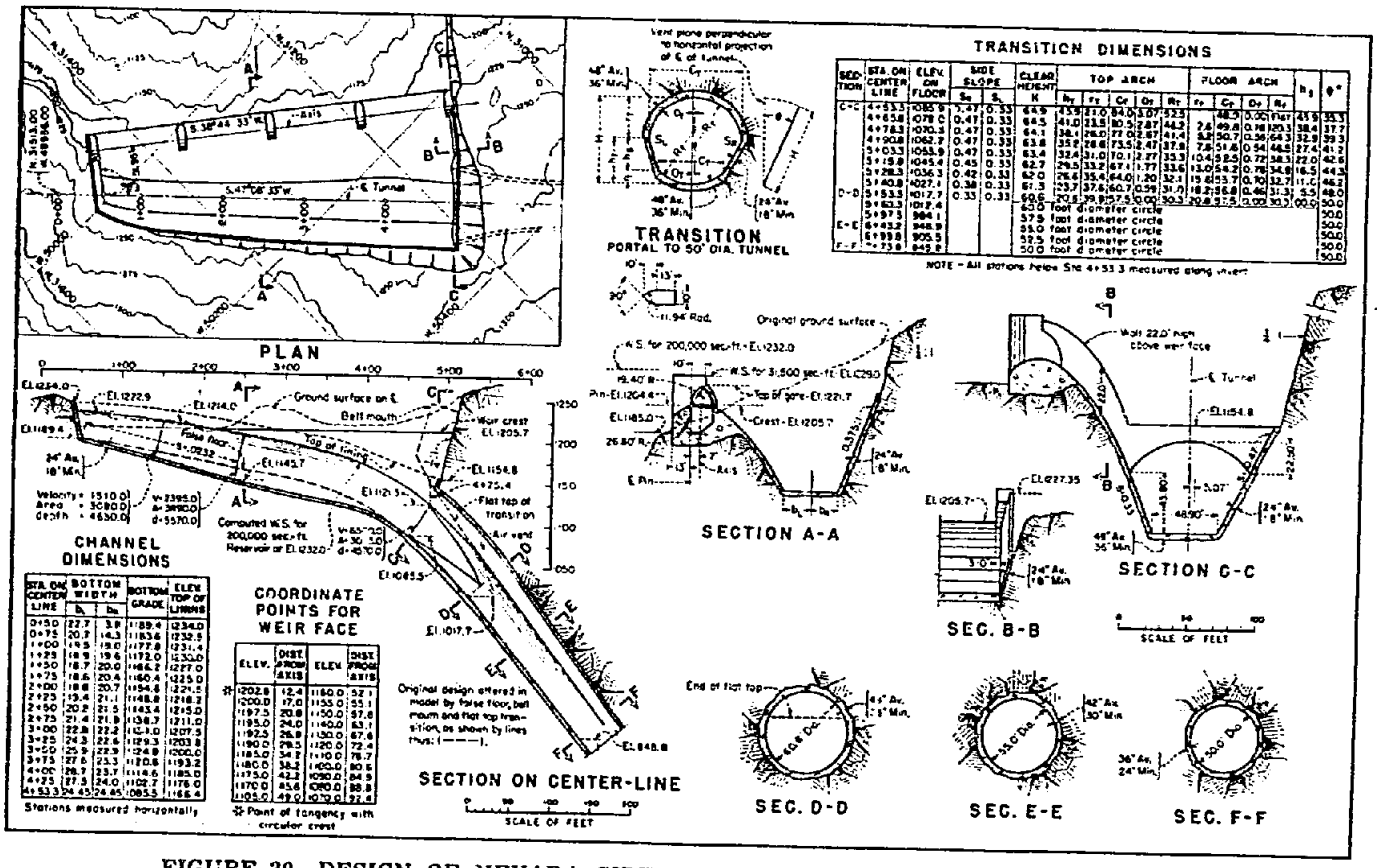


FIGURE 30—DESIGN OF NEVADA SIDE-CHANNEL SPILLWAY WITH FOUR 16-FOOT BY 100-FOOT DRUM GATES

FIGURE 31—FLOW CONDITIONS WITH 100,000 SECOND-FEET DISCHARGE

designed discharge was above the spillway crest at the upstream end of the channel, and also submerged the top of the transition portal. The depth of water at the lower end of the channel indicated that the area of the portal section of the transition was insufficient. In spite of the fact that the portal of the transition was submerged, a decided spiral motion occurred in the greater portion of the channel. This created considerable disturbance in the inclined transition section. Severe splashing also occurred in

FIGURE 32—CONDITIONS WITH DESIGNED FLOW OF 200,000 SECOND-FEET

FIGURE 33—WAVE AT UPPER END OF CHANNEL WITH FLOW OF 100,000 SECOND-FEET

the vertical bend at the bottom of the inclined tunnel. Water began to splash out of the lower window in the bend at a discharge of 110,000 second-feet, and severe splashing occurred at 120,000 second-feet. With a discharge of 200,000 second-feet, there was a very disturbed condition of flow at this point; and the splashing, together with the air carried down by the water, exerted considerable pressure on the windows in the top of the bend. The flow through the horizontal section, as observed through the windows, was very rough and severe vibration occurred at the end of the tunnel.

The cause of the vibration or thumping is not definitely known. It seems to be related to the fact that resistance to flow of water in pipes decreases as the depth of flow increases until a depth of about 0.95 the diameter is reached, beyond which the resistance rapidly increases. There is a generally accepted fact that when the depth of flow in a pipe reaches 0.95 of the diameter, the pipe is carrying its maximum discharge. If water is flowing in an inclined pipe at a depth of 0.87 the diameter and a disturbance occurs which produces a splash against the top, the pipe may suddenly flow full, with the result that the discharge is instantaneously reduced.

As the water flowed down the tunnel, its velocity was reduced by friction and the depth of flow increased. Near the downstream end, for discharges of about 200,000 second-feet, the depth approached 0.87 the diameter of the tunnel. When a surge or splash

occurred the flow suddenly jumped to the full condition, instantly increasing the resistance to flow and causing a hammer or thump in the tunnel section. The flow did not continue at full depth, how-

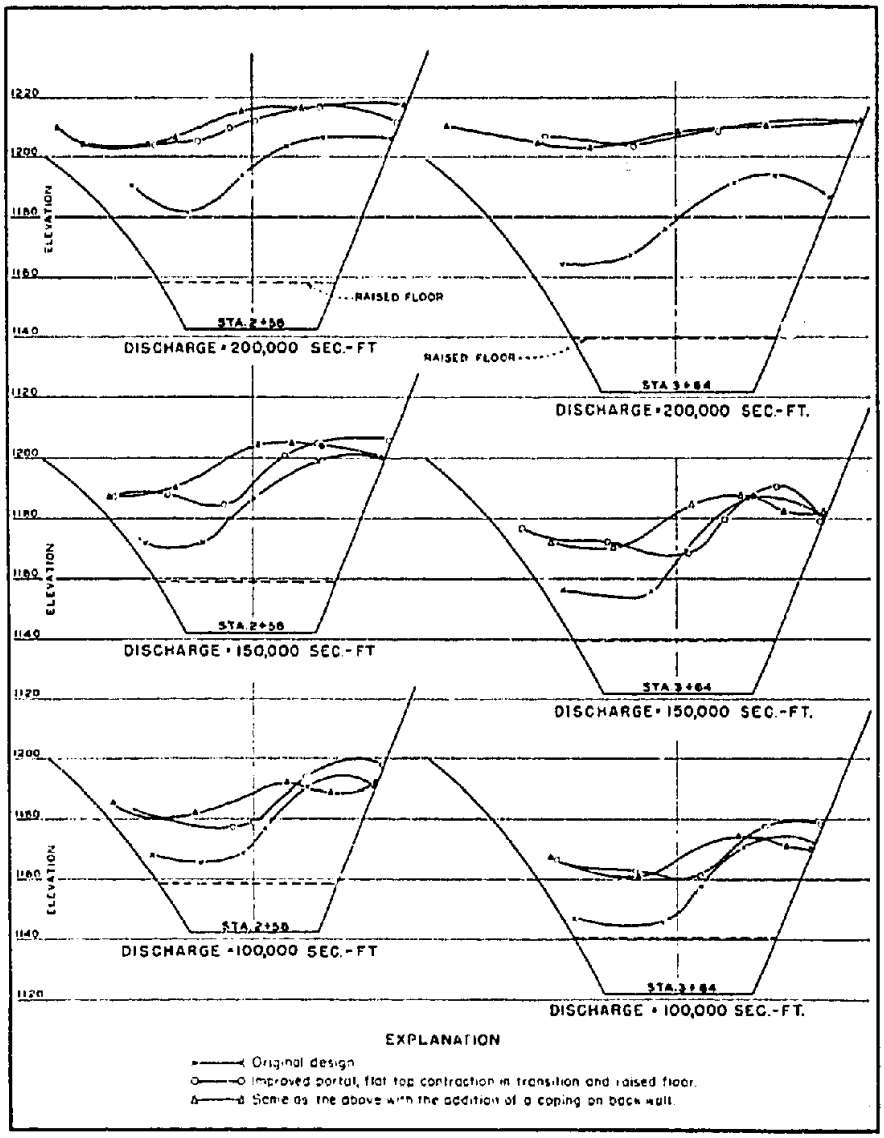


FIGURE 34—COMPARISON OF EFFECT OF RAISED FLOOR, FLAT-TOP TRANSITION AND COPING BAFFLE ON REAR WALL, MODEL M-1

ever, but quickly dropped to partial-depth flow. The disturbance causing this condition may have been due to air entering the upper end of the tunnel and forcing its way out at the lower end. Another possible explanation is that the sudden increase in resistance tended to form, temporarily, a hydraulic jump, which could not maintain itself because the friction loss to the end of the tunnel

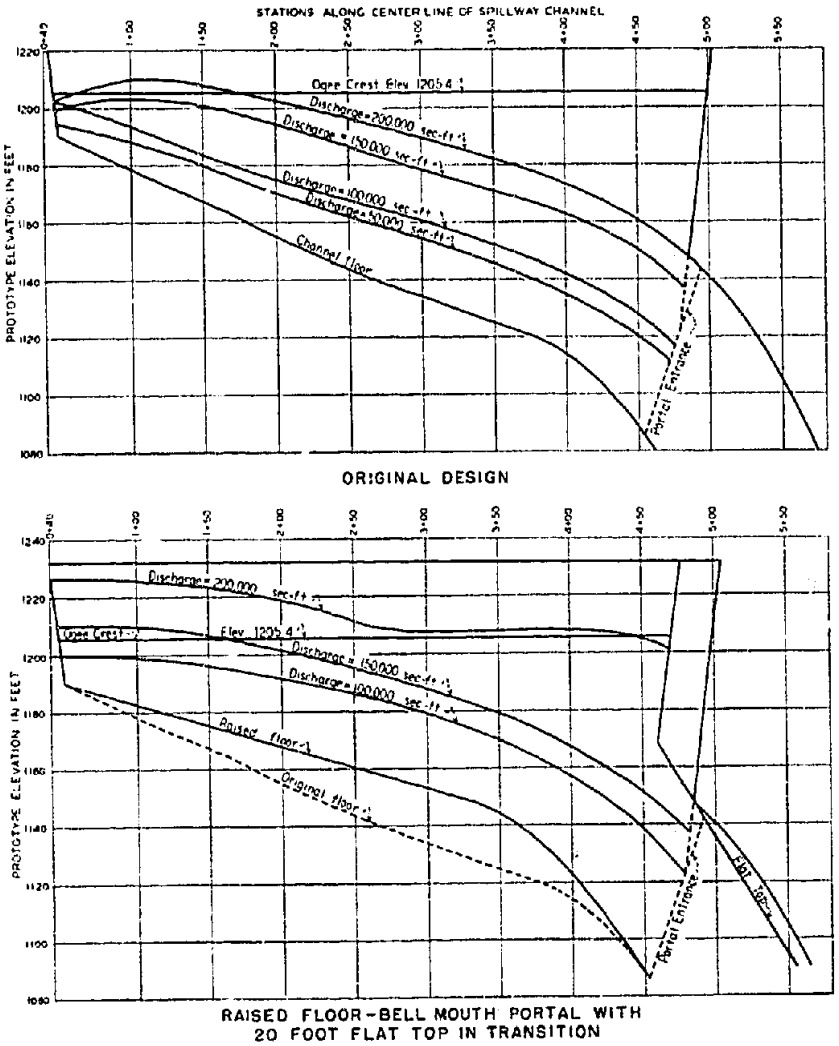


FIGURE 35—MEAN WATER SURFACE PROFILES IN SIDE CHANNEL, MODEL M-1

was not sufficient to create the required back pressure. These changes between full and partly full flow gave rise to a series of thumps or blows of varying intensity at irregular intervals, which were felt only near the downstream end of the tunnel. For the original design, a severe blow came about once every ten seconds. These disturbances would no doubt be undesirable in the prototype, as they would probably set up a perceptible vibration in the rock of the canyon wall.

25. Increased Slope in Channel Floor.—The floor of the channel was raised a maximum of 19 feet at station 3+00 and sloped from this point upward to the original level at the upper end of the channel and downward to the beginning of the transition. The grade of the channel bottom is shown by the dotted line on figure 30. This decreased the slope of the upper end of the channel, and decreased the fall over the crest. Conditions of flow in the channel were improved; but there was still considerable splashing at the portal of the tunnel. The effect of the raised floor for a discharge of 100,000 second-feet may be seen by comparing figure 31 with figure 36. In the latter, the spillway was provided with a different shaped tunnel portal and transition. The revisions did not affect the flow for discharges as low as 100,000 second-feet.

A smoother condition of flow is evident near the downstream end of the channel, with a minor improvement of the wave formation at the upper end. The water, however, still rose to an un-

FIGURE 36—DISCHARGE OF 100,000 SECOND-FEET WITH 20-FOOT FLAT TOP IN TRANSITION AND IMPROVED PORTAL SHAPE

desirable height on the rear wall. Cross sections are shown on figure 34, and longitudinal profiles on figure 35, for the flow in the model with the raised floor and flat-top transition. The contraction in the transition did not affect the flow for discharges below 175,000 second-feet, since the water did not come in contact with the top of the portal. For the design discharge, however, it restricted the flow and caused the water surface at the lower end of the channel to rise considerably above the top of the portal.

The original design was computed to have a cross-sectional area for 200,000 second-feet without causing submergence of the spillway crest, at the upstream end of the channel, sufficient to reduce the crest discharge. It was found that the floor could be raised and the flow over the crest maintained. This was probably due to the fact that computed water surfaces in the channel were mean values, while the submergence was controlled by the height of the water surface on the crest side of the channel. The false floor remained in the channel during succeeding tests.

26. Coping on Rear Wall of Channel.—A curved-faced baffle in the form of a coping was placed along the back wall of the channel to reduce the height to which the water rose on the rear wall. The position of the baffle is shown on figure 37. The baffle tended to deflect the water toward the center of the channel, reducing the height of the wave against the wall. It considerably improved conditions at the tunnel entrance and reduced disturbances through-

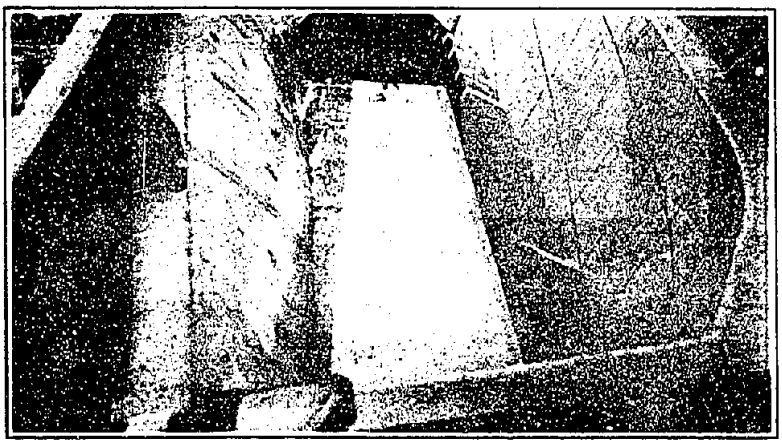


FIGURE 37—SPILLWAY CHANNEL WITH COPING BAFFLE

out the tunnel. The effect produced by the baffle on a flow of 100,000 second-feet can be seen by comparing figures 38 and 39. The height of the wave on the rear wall was materially less near the lower end of the channel when the coping baffle was used.

27. Flat-Top Contraction in Transition.—The original design

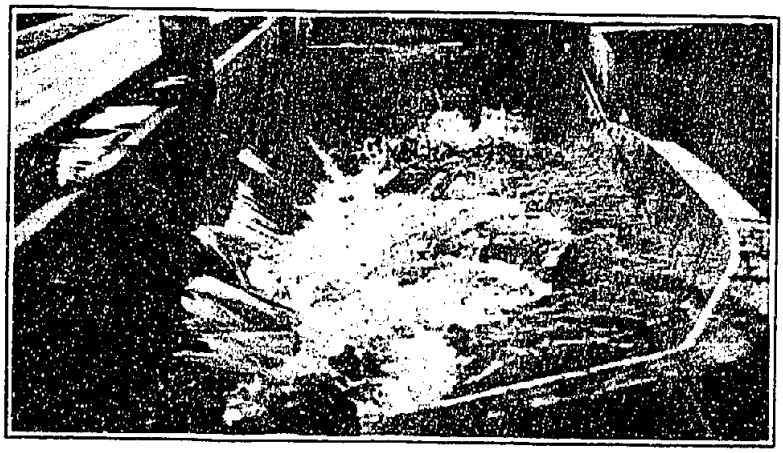


FIGURE 38—FLOW OF 200,000 SECOND-FEET WITH COPING BAFFLE, 15-FOOT FLAT TOP IN TRANSITION AND IMPROVED PORTAL

FIGURE 39—FLOW OF 200,000 SECOND-FEET WITHOUT COPING BAFFLE BUT WITH 20-FOOT FLAT TOP IN TRANSITION AND IMPROVED PORTAL

contemplated a free surface for the water flowing in the spillway and tunnel. The surface was very rough, however, and produced considerable splashing and impact. In an attempt to smooth the water surface, a contraction was placed in the transition section as shown in figures 30 and 40. This consisted of substituting a flat top, extending 20 feet downstream, for the curved arch. A vent was installed at the lower end, downstream from the contraction, to admit air above the water flowing in the inclined tunnel. For discharges over 175,000 second-feet the portal formed an orifice, with a horizontal top edge, which caused the water to start down the tunnel with a flat upper surface, thus reducing the former splashing. When this contraction acted as an orifice it improved flow conditions in the side channel by increasing the depth of flow and decreasing the height of fall over the crest.

Although the contraction considerably raised the water surface at the lower end of the channel, it did not materially reduce the discharge over the upper end of the crest. The conditions of flow in the tunnel were considerably improved, both in regard to splashing at the vertical bend and thumping at the end.

A contraction in the transition, which diminished the restriction, was also tested. The flat top in this case extended 15 feet downstream from the top of the transition section as compared with 20 feet in the previous case. Conditions of flow in the channel, in the vertical bend, and at the end of the tunnel were less

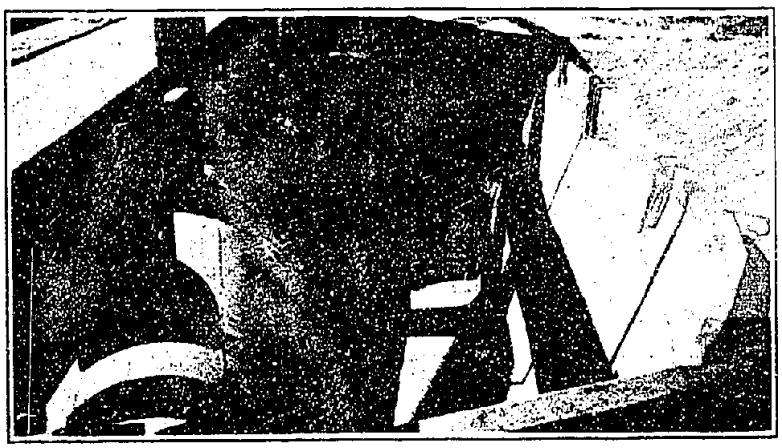


FIGURE 40—CHANNEL WITH 20-FOOT FLAT TOP IN TRANSITION AND IMPROVED PORTAL

desirable than with the greater contraction. The greater contraction was used in all subsequent experiments.

28. Improved Portal Shape.—The face of the portal in the original design was subject to considerable impact. To improve entrance conditions, the portal was altered to the form of a bell mouth, as shown in figure 41. Since the tunnel portal, as originally built, was practically opposite the end of the overflow crest, the bell mouth extended beyond the end of the crest. It was therefore necessary to omit a portion of the bell mouth on the crest side to prevent it from interfering with flow over the spillway crest. This would not be a practical form of construction for the prototype, but the design served to indicate the results that can be secured with a bell-mouth entrance. The bell mouth improved entrance conditions considerably and it was used throughout the remainder of the tests.

The design was also tested with the coping baffle removed, as shown on figure 40. However, flow conditions in the channel, in the vertical bend, and at the end of the tunnel were all less satisfactory, so the coping baffle was reinstalled.

29. Vanes in Inclined Tunnel.—The spiral motion of the water in the side channel, produced by the water flowing over the over-

FIGURE 41—CHANNEL WITH COPING BAFFLE, 20-FOOT FLAT TOP AND IMPROVED PORTAL SHAPE

fall crest, persisted for a long distance down the tunnel and caused at least a portion of the disturbance in flow. Vanes were tried in various positions in the transition and inclined tunnel to reduce the spiral motion. The first vane was about six feet high and extended down the center of the flat-top contraction in the transition. No beneficial effect was observed. A vane six feet high was then placed along the bottom of the inclined tunnel, from the end of the transition to the beginning of the vertical bend. This caused a marked improvement in splashing conditions at the vertical bend. With the flat-top contraction and vane in the transition and the coping on the rear wall, the discharge at which splashing began at the lower window was increased from 130,000 to 162,000 second-feet. Severe splashing did not occur until the discharge reached 190,000 second-feet.

As splashing at the window always occurred on the left side, looking downstream, it was thought that by inclining the vane toward the right side of the tunnel, more water could be forced to that side and the splashing relieved. The direction of the vane was changed so that it extended from the center line of the bottom of the tunnel at the upper end to five feet to the right of the center line at the lower end. This change reduced the discharge at which splashing occurred from 162,000 to 155,000 second-feet, and the discharge for severe splashing from 190,000 to 170,000 second-feet. The vane was then moved so that the lower end was five feet to the left of the center of the bottom. In this position, splashing from the lower window occurred at about the same discharge as with the vane on the center line; but no severe splashing occurred for discharges up to 200,000 second-feet.

The water approached the vertical bend with a greater depth on the right side, which is the probable explanation of this unexpected action. In passing around the vertical bend it was acted upon by a centrifugal force. Since the mass of water on the right side was greater than that on the left, the force was greater on the right side, and likewise greater on the bottom. This differential in pressure caused the water to move from the right to the left side. The greater mass of water at the beginning of the bend was on the right side. In passing around the bend it moved in a spiral clockwise direction across the bottom, up the left side, and across the top of the bend toward the right. The vane, with its end toward the left, forced more water toward the left side of the bend, producing more nearly equal centrifugal forces on the two

sides of the tunnel. Moving the vane toward the right side at the lower end increased the mass of water on the right side, making the two sides more unequal, which increased the cross motion causing the splash.

30. Cylindrically-Faced Baffles.—Combinations of cylindrically-faced baffles were investigated as a means of diminishing the height of wave which formed on the back wall of the spillway and to more equally distribute the flowing water in the channel. The first set of cylindrical baffles is shown in figure 42. It consisted of baffles, 10 feet long with 10-foot spaces between, set on the center line of the channel. The cylindrical face had a radius of six feet, and the baffle had an over-all height of 14 feet. These baffles improved the flow in the channel and tunnel; but were less effective than the coping baffle on the rear wall. Splashing occurred at the lower window in the vertical bend during a discharge of 130,000 second-feet. The baffles considerably obstructed the flow in the channel and submerged the weir at the upper end sufficiently to decrease the flow over the crest during maximum reservoir level.

The faces of the baffles, located on the center line of the channel, were not exactly at right angles to the direction of flow. Consequently, portions of the water were deflected downstream, impinging on the adjacent downstream baffles. For the next tests the baffles were set with their faces inclined upstream, 15

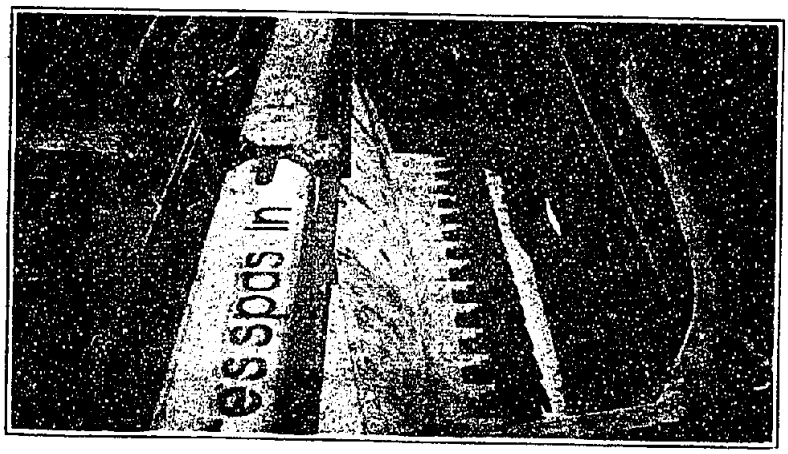


FIGURE 42—CYLINDRICALLY-FACED DENTATED BAFFLES ON CENTER LINE OF CHANNEL

degrees with the center line of the channel. Flow conditions were slightly improved, both in the channel and in the tunnel; but the baffles still obstructed the flow in the channel and submerged the crest sufficiently to reduce the discharge during maximum reservoir stage. The baffles were then placed midway between the center line and the left side of the channel and again inclined upstream, 15 degrees with the center line. This arrangement was slightly better than the preceding one, but the obstruction caused by the baffles was still sufficient to submerge the crest. As the two upstream baffles appeared to obstruct the flow without bettering conditions, they were removed. The conditions were not materially improved.

Cylindrically-faced baffles were next placed in a continuous line down the channel, beginning 6.5 feet to the right of the center line at station 1+00 and reaching 15 feet to the left of the center line at station 3+50. The submergence effect was not as great as for the preceding tests, but was still sufficient to submerge the crest. Severe thumping also occurred at the downstream end of the tunnel.

Smaller baffles were next used in an attempt to reduce the submergence. These had a radius of 4.5 feet and an over-all height of 11 feet. They extended in a line from 8.5 feet to the right of the center line at station 0+65 to 20 feet to the left of the center line at station 3+75. Submergence of the crest did not occur with these baffles in place and flow conditions were slightly improved. The baffles were extended downstream to station 4+50, but the extension resulted in bad splashing at the vertical bend. The splashing began at 132,000 second-feet, and was severe at 150,000 second-feet. Shocks occurred at the lower end of the tunnel at average intervals of five seconds, but were not severe. This continuous set of baffles was built using blocks 10 feet long. A test was made with alternate blocks removed and the results procured were similar to the first run with the cylindrically-faced baffles. Conditions of flow in the channel were fairly satisfactory and excessive submergence did not occur at the upper end. Splashing from the lower window began at 155,000 second-feet and became severe at 178,000 second-feet. Flow conditions at the end of the tunnel were more desirable, with shocks occurring at average intervals of 40 seconds.

Cylindrically-faced baffles, inclined downstream, were placed as shown in figure 43. Upon striking the baffles, the water flow-

FIGURE 43—CYLINDRICALLY-FACED BAFFLES INCLINED DOWNSTREAM

ing from the crest was deflected downstream, increasing the velocity in the channel and reducing the necessary cross section. The action of the baffles in deflecting the flow downstream could be observed at small discharges. The same action probably continued during higher flows. The obstruction of the channel caused by the baffles offset the advantage gained, so that conditions of flow for maximum discharge were not appreciably improved over those obtained without baffles.

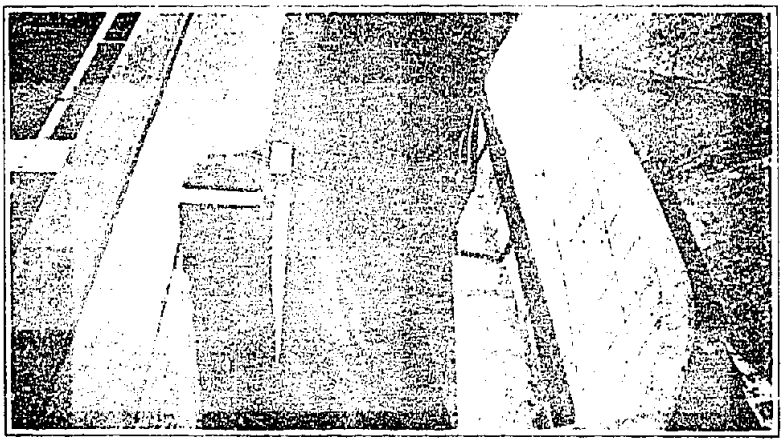


FIGURE 44—PLAIN SILL ON CHANNEL FLOOR

31. Sills on Channel Floor.—Experiments were made with a plain sill on the channel floor, having a vertical face six feet high, as shown in figure 44. At high flows this produced very satisfactory conditions in the channel and at the end of the tunnel, with small shocks occurring about once in 15 seconds. At low flows, however, conditions in the channel were undesirable. For a flow of 15,000 second-feet, the stream was thrown vertically into the air in a fountain formation. At higher flows, a pressure area formed between the baffle and overflow face, forcing the stream upward and over the baffle without impingement.

The plain sill was replaced by a dentated sill, six feet high, extending from a point 23 feet to the left of the center line at station 3+00 to a point 14 feet to the left at station 0+50. This improved the flow in the channel at low discharges; but severe shocks occurred at the lower end of the tunnel at intervals averaging 1.3 seconds.

32. Deflector Vanes on Spillway.—Deflector vanes were tried on the spillway face to direct the falling water downstream and permit the use of a smaller channel. The arrangement of the vanes for the first trial is shown in figure 45. The water flowing over the crest was not deflected directly downstream, but had a component across the channel. At the larger discharges, the vanes obstructed the flow and caused excessive submergence at the

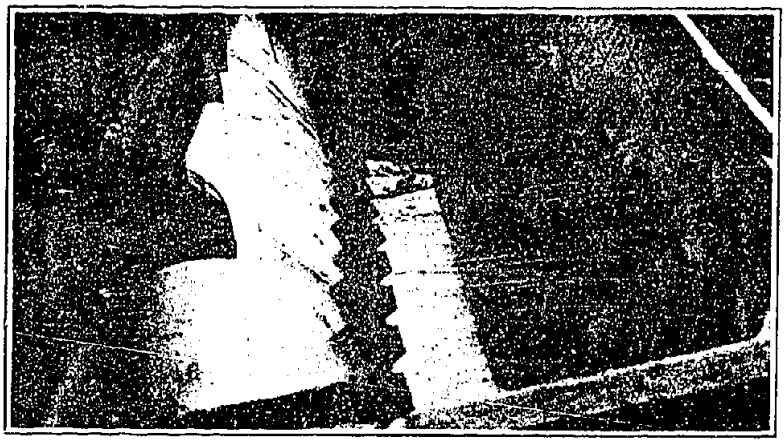


FIGURE 45—FULL SET OF VANES ON SPILLWAY CREST
MODEL SCALE 1:20

upper end of the crest. Considerable vibration also resulted at the downstream end of the tunnel, the average period for severe shocks being four seconds. The four upstream vanes were removed, as they appeared to obstruct the flow. Slightly improved conditions resulted in the channel but excessive submergence was still present and the frequency of severe shocks at the tunnel exit was increased to a three-second average. The four downstream vanes were then raised so they would be less effective in obstructing the flow. This procedure eliminated the vibration at the end of the tunnel, but produced an unstable condition in the channel which caused the water surface at the lower end to fluctuate as much as 20 feet.

33. Troughs in Floor of Channel.—This revision, shown in figure 46, was made by removing the false floor and building a raised step on the wall side. For flows up to 100,000 second-feet, the raised section caused disturbed conditions in the channel, with a stream of water rising in a fountain effect along the center line. Disturbed conditions still existed at 150,000 second-feet; but at 200,000 second-feet, the conditions in the channel were satisfactory. The severe shocks at the lower end of the tunnel occurred at intervals of about 15 seconds.

A dentated sill, six feet high, was placed on the left edge of the raised portion of the channel floor. This tended to hold the water at the right side of the channel. During low flows the trough at the upper end was swept out, producing turbulent con-

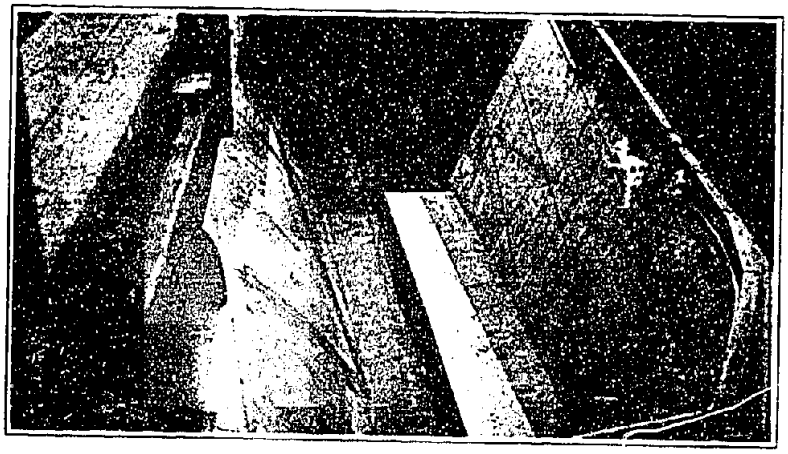


FIGURE 46—TROUGH ON CREST SIDE OF CHANNEL FLOOR
MODEL SCALE 1:20

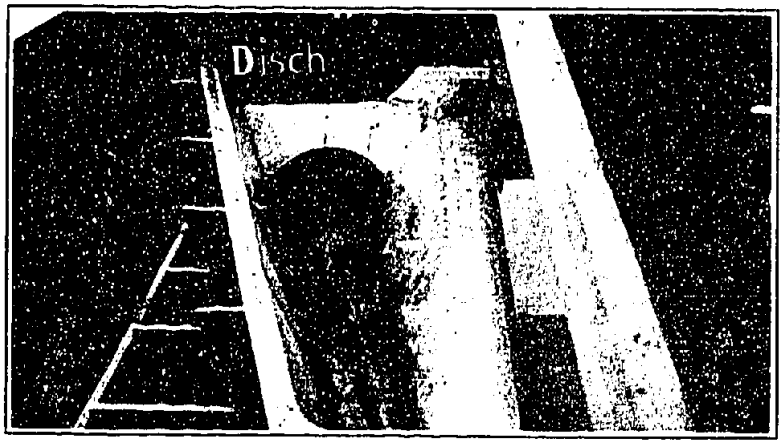


FIGURE 47—CHANNEL WITH TROUGH IN FLOOR ALONG REAR WALL MODEL SCALE 1:100

ditions. A cross weir, placed across the channel at the downstream end, did not produce a noticeable improvement.

A model, M-2, of the drum-gate type of spillway was constructed on a 1:100 scale. A test was made to determine the possibility of using a trough on the floor of the channel along the bottom, opposite the crest side, to reduce the height of the wave along the rear wall. The channel of the model was approximately the same design as in the 1:20 model. To simplify construction, it was built without gates or intermediate piers. Figure 47 shows the spillway with no flow. The trough on the wall side proved ineffective in reducing the height of the wave. The sheet of water flowed over the edge of the trough in an inclined direction, moved downward to the bottom of the trough, then across the bottom, and up the rear wall, with practically no dissipation of energy. The trough was then moved to the crest side with better results. The action in this case was very similar to that for the 1:20 model.

CHAPTER III—MODELS OF SEMI-FINAL DESIGN OF SPILLWAYS

MODEL WITH DRUM GATES, 1:60 SCALE, (C-4)

34. **Design and Construction of Model.**—The experiments on the first model types, particularly those on the first model at the Montrose laboratory, showed that unusual care would be necessary in developing a spillway which would provide satisfactory flow conditions in the spillway channel and through the tunnels leading to the river.

Since a satisfactory type was not evolved by the tests on the first 1:20 and 1:100 models at the Montrose laboratory, a new model was installed in the Fort Collins laboratory which embodied the pertinent results of the observations on the previous models. The principal difference between this model as originally constructed and the first 1:20 Montrose model was that the slope of the channel bottom in the new model was reduced to obtain a greater depth of water and reduce turbulence.

The prototype of the C-4 model is shown on figure 48. The model was built on a scale of 1:60, or one inch on the model equal to five feet on the prototype. It had four drum gates, each 100 feet long, and a side-channel trough approximately 45 feet deep at the upper end and 95 feet deep at the lower end. The crest and side channel were constructed of wooden frames covered with galvanized iron. In the first tests, the model did not have movable gates, the solid crest being shaped to represent the condition with the drum gates in the lowered position. Gates which could be regulated were added later. The transition from the trapezoidal channel to the circular 50-foot diameter tunnel was made by fitting laths between accurately shaped and spaced collars and plastering them with a mixture of quick-setting cement, lime, and sand. The top of the upstream portion of the transition was omitted for some distance, so that flow conditions might be observed. Unfortunately, no satisfactory pictures of the flow could be obtained, due to interference of the framing and lack of light.

The model was located in the same position in the laboratory

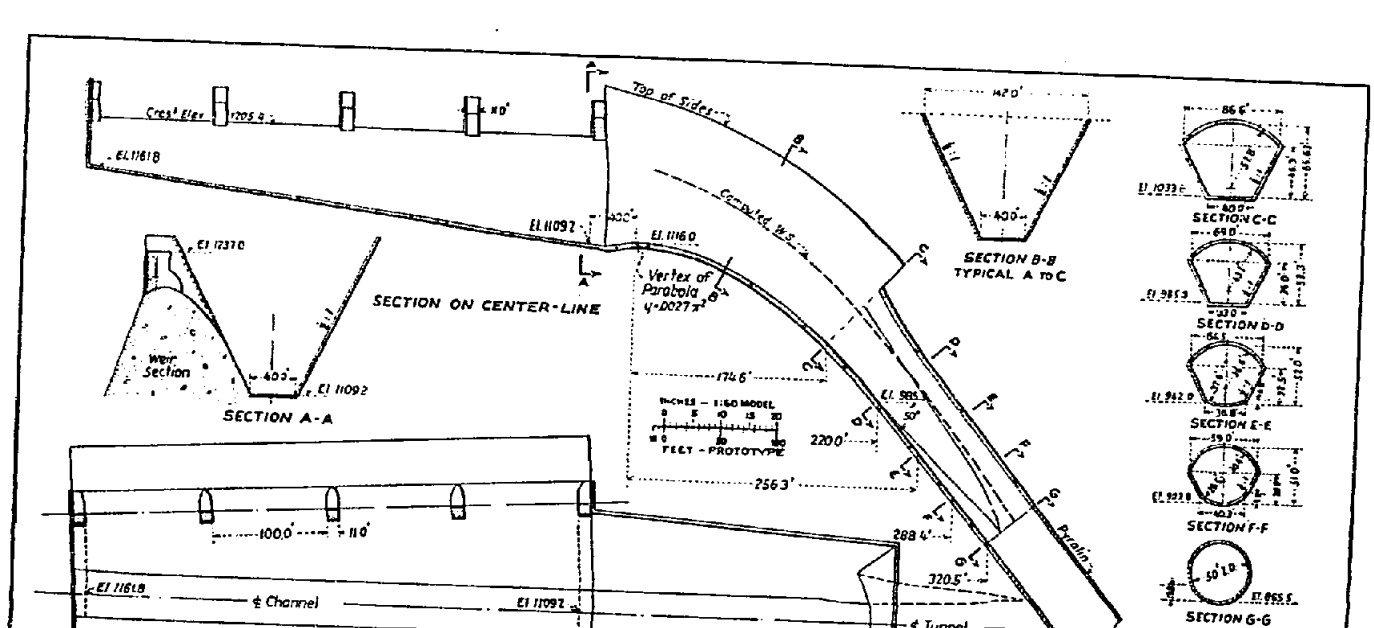


FIGURE 48—DRUM-GATE SPILLWAY, MODEL C-4 WITH TRANSITION 1

as the C-2 model shown on figure 3. Flow was measured by the laboratory two-foot Cipolletti weir. The height of the model crest was sufficient to submerge the measuring weir at high discharges, and an extensive series of volumetric measurements was made to calibrate the measuring weir under submerged conditions. The maximum discharge over the model was about 7.2 second-feet and the corresponding depth of flow on the model crest approximated 5 inches.

Except in a few instances no major changes were made in the shape of either side of the channel. The principal changes were in the transition from the trapezoidal channel to the tunnel section, in the addition of baffles in the channel, and in the presence or absence of model topography representing the hillside.

The main object of the experiments was to develop a design which would allow water to flow down the tunnel as smoothly as possible without completely filling the section. A large number of tests was also performed to investigate the effect of various devices on the flow.

Considerable quantitative data were observed, consisting of cross sections of channel flow, depths of flow in transitions, discharge coefficients for ogee crest forms, and flow over crest gates in various positions.

35. Results from Original Design.—In the original design of the C-4 spillway, water entering the tunnel was much higher against the rear wall than against the wall on the crest side, causing a transverse flow in the upper end of the tunnel and undesirably rough conditions down the inclined shaft. A space of 60 feet at the downstream end of the spillway was blocked in an effort to improve this condition. The reduction in flow over the crest was compensated by a gate at the upper end of the channel which permitted water to flow directly into the channel along its axis. These modifications indicated that a space of 60 feet between the end of the spillway section and the entrance to the steep slope of the transition would allow the difference in water surface elevation on the two sides of the channel to equalize upstream from the transition, thus eliminating cross flow at the tunnel entrance and improving hydraulic conditions down the inclined tunnel.

36. Channel Extension of 60 Feet.—The spillway model was modified by moving the transition 60 feet downstream, with a

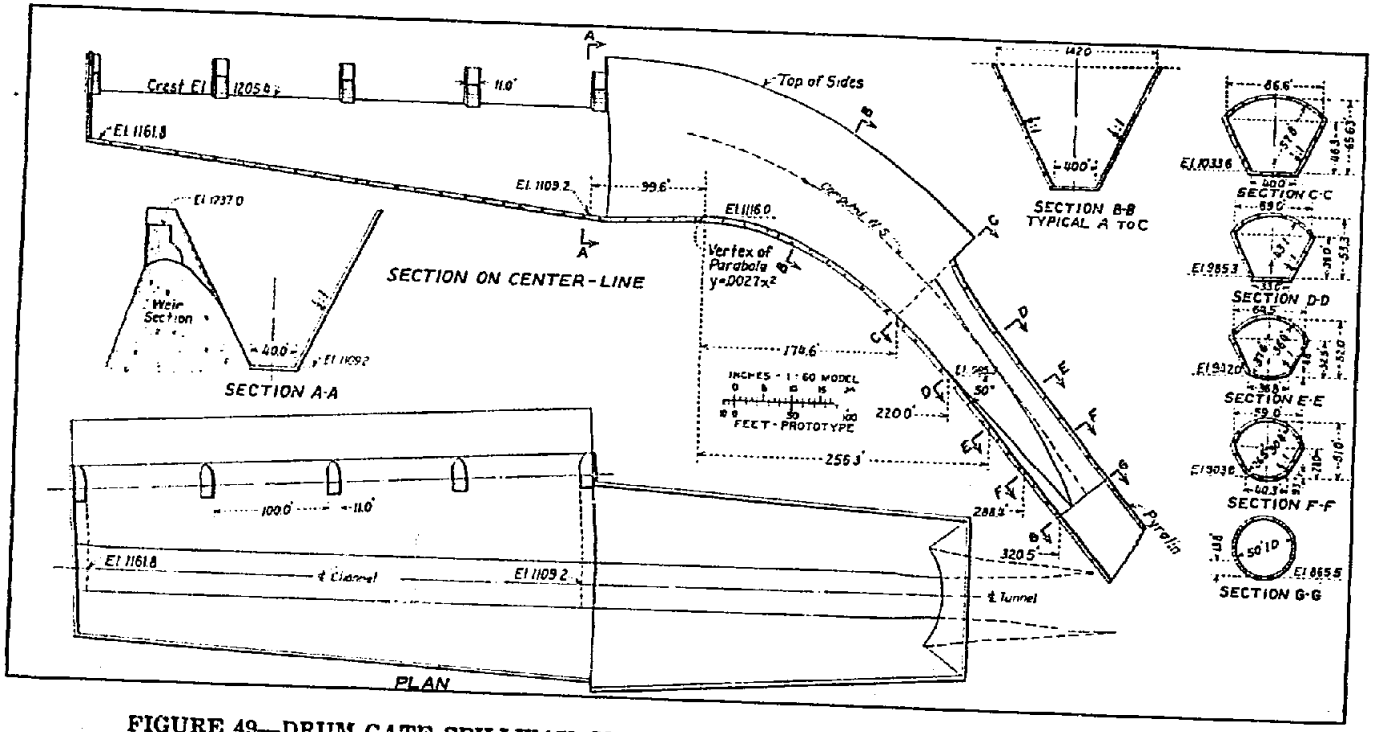


FIGURE 49—DRUM-GATE SPILLWAY, MODEL C-4 WITH 60-FOOT CHANNEL EXTENSION

straight section having a relatively level bottom inserted between the end of the spillway section and the transition, as shown on figure 49. The elongated transition caused a greater depth of water, resulting in smoother flow along the channel and in the region where it entered the sloping section. This greater depth in the channel was not sufficient to reduce the coefficient of crest discharge since it did not produce excessive submergence at the upper end.

37. Cross-Weir at End of Channel.—The extension of the channel section below the spillway crest was considered beneficial and was incorporated in subsequent steps leading to the final design. Since the increased depth in the channel seemed advantageous, an investigation was made of the possibility of improving hydraulic conditions by constructing a weir across the lower end of the channel to provide a still greater depth of flow. A set of experiments was made with weir heights of 15, 20, and 30 feet at the end of the 60-foot extension. These tests indicated that flow conditions improved as weir heights increased.

38. Offset in Channel Wall.—For all conditions of operation, part of the channel adjacent to the overflow section was occupied by that portion of the crest flow which had a small component in the direction of the channel. The space occupied by water flowing along the channel was, therefore, practically that between the rear wall and the outer face of the nappe. At the downstream end of the crest, the effective channel area suddenly increased to the full width, since there was no nappe to reduce the section. This abrupt enlargement caused a whirl on the crest side and a cross movement of the water on the opposite side as the water adjusted itself to the new channel shape. Both of these actions resulted in undesirable conditions of flow down the tunnel.

To remedy this condition, an offset was constructed on the overfall side of the transition, beginning just downstream from the end of the crest, as shown in figure 50. This tended to make the side of the downstream portion of the channel more nearly a continuation of the effective side of the upstream portion. The offset also had the effect of reducing the channel section in the transition, thus producing a greater depth of flow. Both effects acted together to improve the nature of the flow in the channel, transition, and tunnel.

39. Improvement of Transition Shape.—Hydraulic conditions were further tested by the construction of transition 2, see figure 51, which incorporated a 15.3-foot offset on the crest side and a cross-weir with crest at elevation 1125.0 at the entrance to the

FIGURE 50—MODEL WITH 21-FOOT OFFSET

transition. This caused an improvement in the flow entering the tunnel, but was less desirable than that secured in the final design, as it filled the shaft to a greater extent.

By raising or lowering the entire transition section, keeping the rear wall of the transition in line with the rear wall of the channel, the height of the cross-weir could be changed; and at the same time, the offset on the crest side could be varied, the offset increasing as the cross-weir height increased. An offset of 10 feet was found to give the best flow conditions and a greater width was found to be detrimental. When the transition was raised sufficiently to produce a cross-weir 15.8 feet in height, the resulting offset was too large.

Transition 2 did not include the 60-foot length between the end of the crest and the beginning of the steep section, the lack of which was evident in the results obtained. A third transition, number 3, was built, including the 60-foot extension, the 10-foot offset at the end of the crest, and a cross-weir 43 feet high with crest at elevation 1145.0, the intention being to incorporate all points tending toward good results which had been evolved in the previous tests.

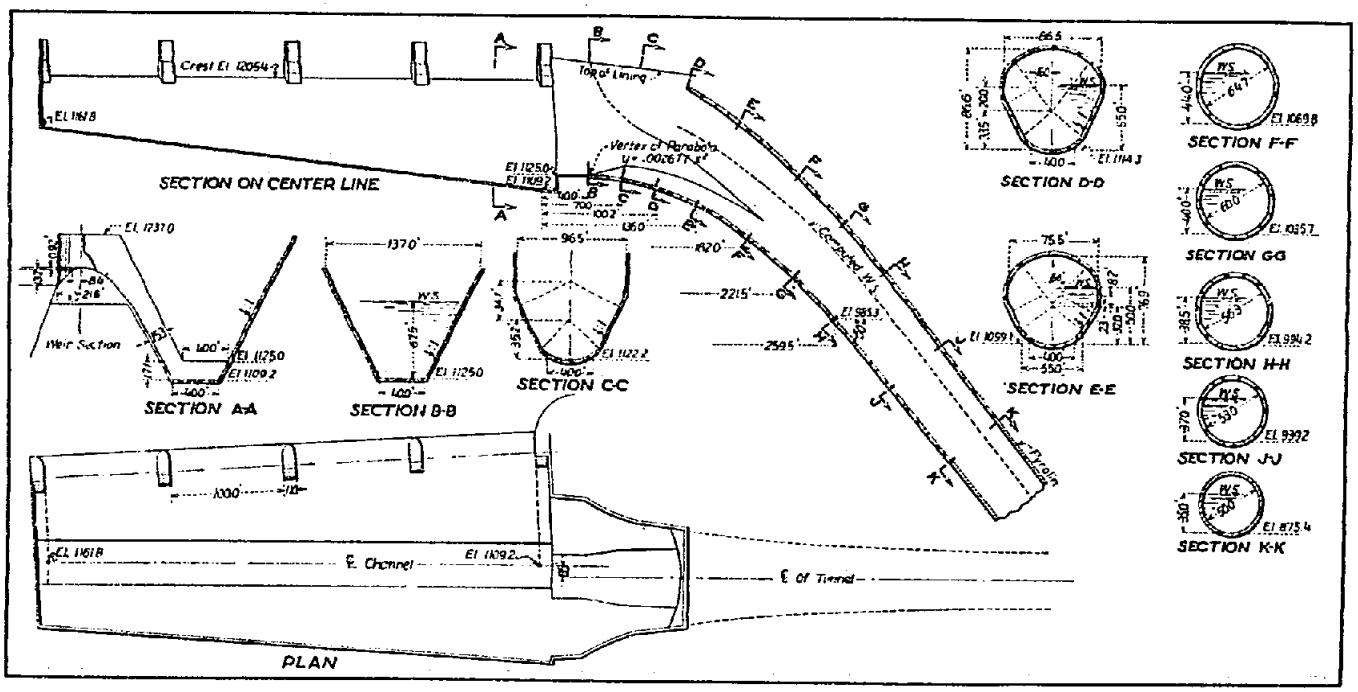


FIGURE 51—DRUM-GATE SPILLWAY, MODEL C-4 WITH 15.3-FOOT OFFSET ON CREST SIDE OF CHANNEL

The dimensions of the prototype of transition 3 are given on figure 52. This form of transition gave exceptionally smooth flow into the tunnel during maximum discharge. However, at this discharge, it backed the water in the channel, causing sufficient submergence of the crest to reduce the spillway capacity. This transition, with minor changes, was incorporated in the final design.

In transition 3-A, shown on figure 53, the number 3 transition was lowered five feet, making the cross-weir at the entrance at elevation 1140.0. This decreased the submergence of the crest and increased the discharge capacity. The action in the spillway channel and transition, however, was not as favorable as with transition 3.

Transition 3-C, shown by figure 54, was made from transition 3 by adding a level section of channel bottom at elevation 1110.0 from the end of the spillway section 107 feet downstream to the cross-weir, the latter being at elevation 1140.0. This was done to test the advantage of a longer section between the end of the spillway crest and the inclined portion of the transition. The increased length was of no advantage, and no further tests were made with this construction.

Two other forms of transition were tested. One, number 4, was very similar to number 3-A, except the parabola of the transition invert was much steeper, as shown on figure 55. The results of the steeper parabola were not satisfactory. Transition 5, shown on figure 56, had a low cross-weir at elevation 1125.0 and an offset on the crest side, the same as transition 2, but a flatter parabolic slope for the transition invert. This transition did not improve the flow and seemed to indicate that the high cross-weir was necessary with a flat parabolic curve. A curved guide wall was placed upstream from the crest at each end, as shown on figure 57, to improve the approach to the spillway during tests of transition 4. This wall improved flow conditions and was retained in the final design.

40. Curved Wall at Upstream End.—Since transitions 4 and 5 proved less satisfactory than 3, the latter was replaced in position and tests continued. Curved walls, shown in figure 58, were introduced from the upstream end of the spillway face to the back of the channel to smooth the flow at the upper end. At low flows an undesirable wave formed at the upper end of the channel, while at high flows the curved wall had little effect. The general appearance of the spillway was much improved with the curved upstream end,

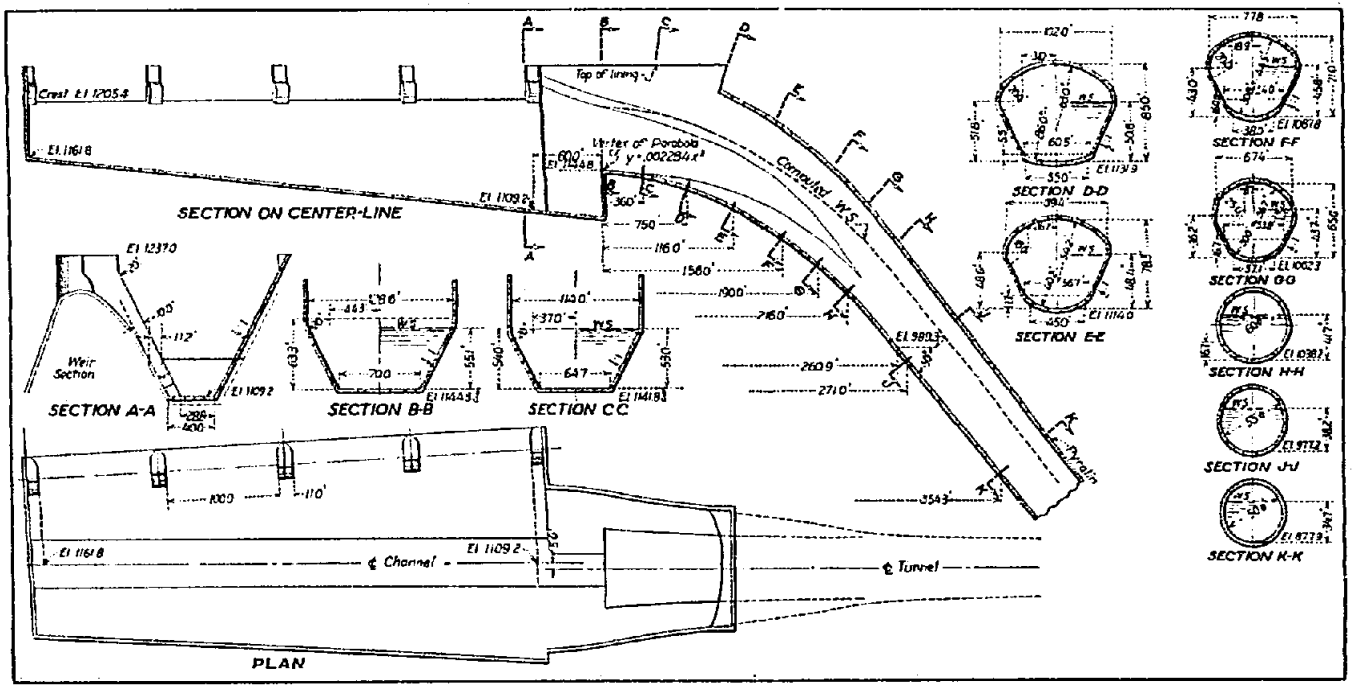


FIGURE 52—DRUM-GATE SPILLWAY, MODEL C-4 WITH TRANSITION 3

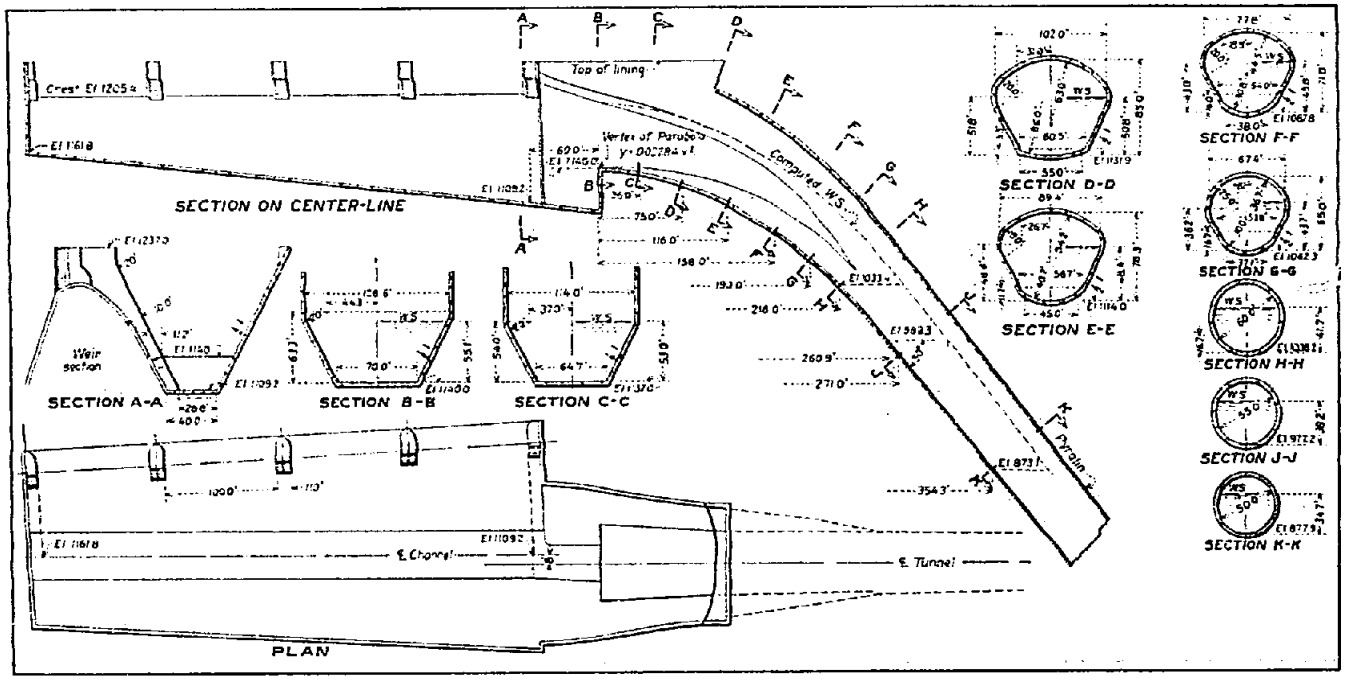


FIGURE 53—DRUM-GATE SPILLWAY, MODEL C-4 WITH TRANSITION 3-A

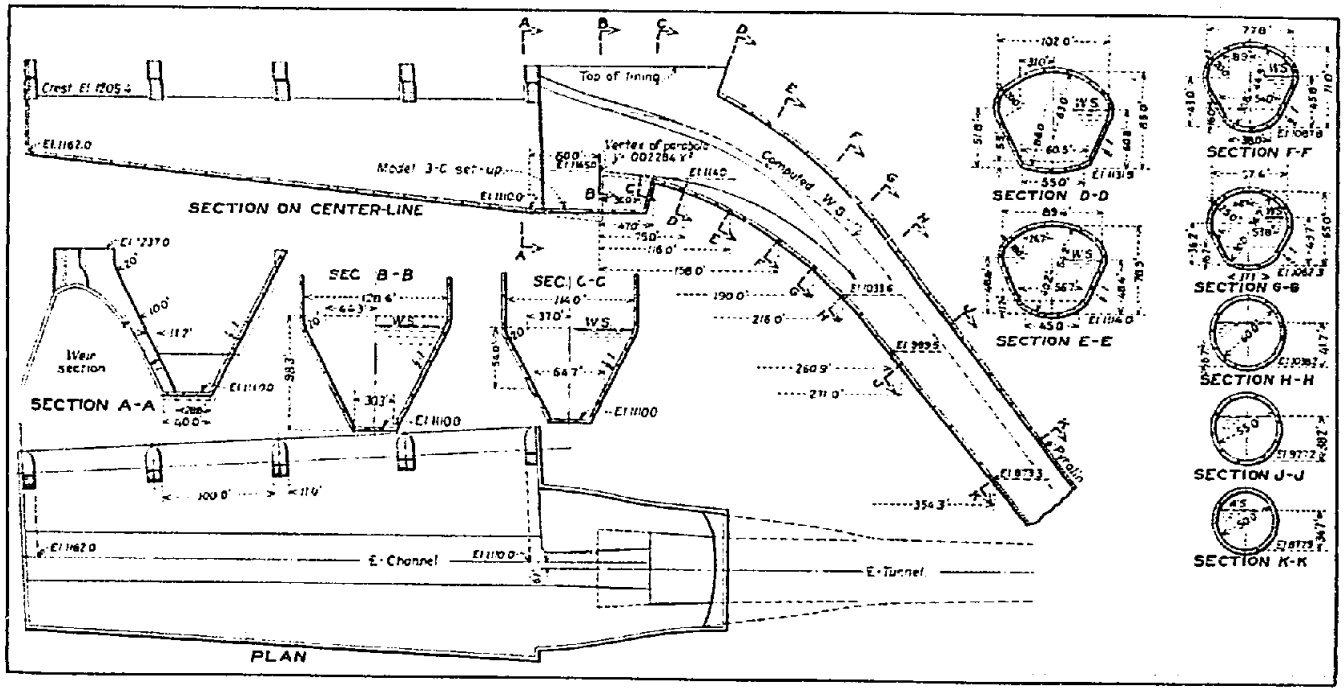


FIGURE 54—DRUM-GATE SPILLWAY, MODEL C-4 WITH TRANSITION 3-C

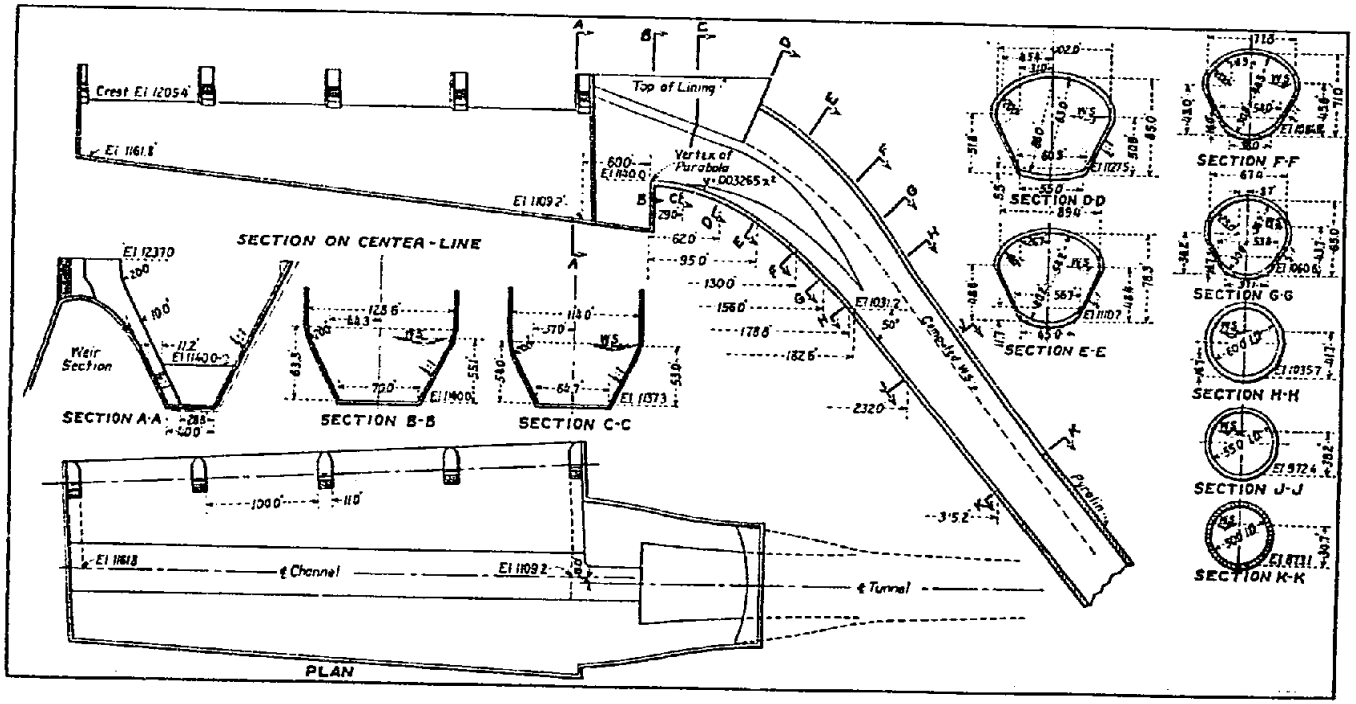


FIGURE 55—DRUM-GATE SPILLWAY, MODEL C-4 WITH TRANSITION 4

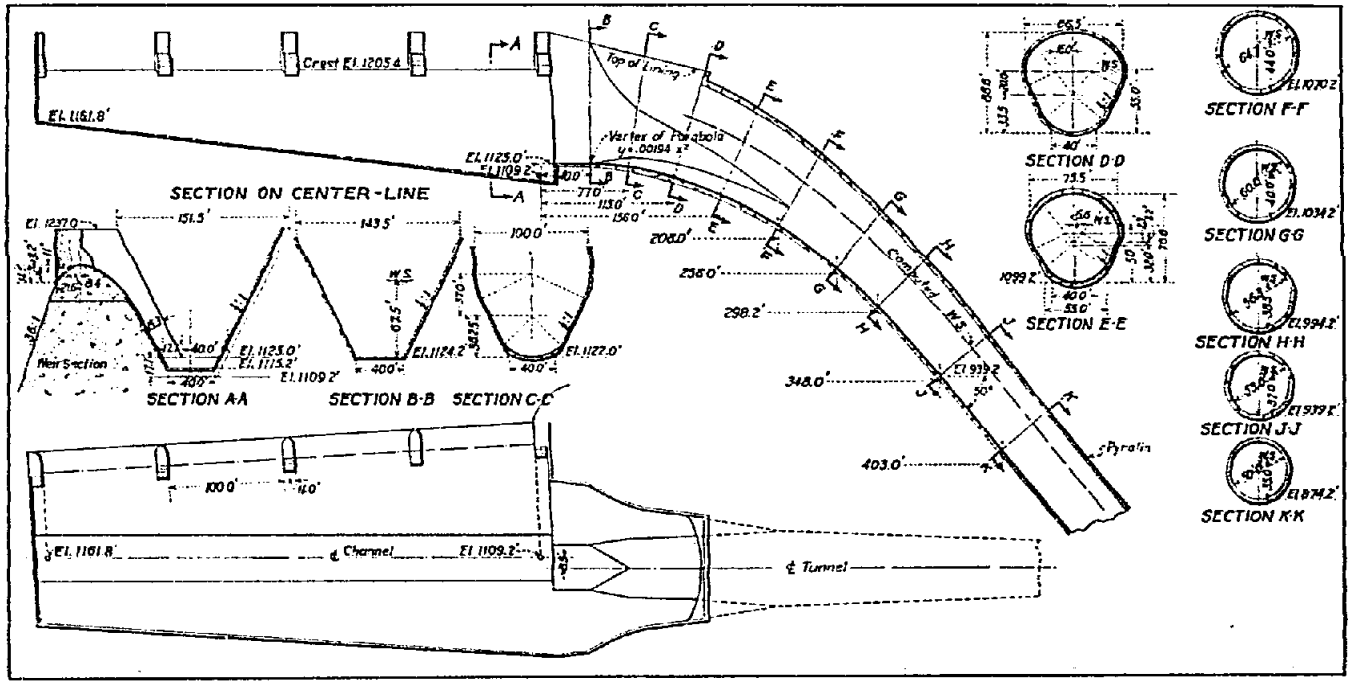


FIGURE 56—DRUM-GATE SPILLWAY, MODEL C-4 WITH TRANSITION 5

FIGURE 57—CURVED GUIDE WALL AT END OF SPILLWAY CREST

and by proper gate manipulation it would never be necessary to form the wave. The curved wall was incorporated in the final design.

In testing model C-4, three different forms of spillway crest were used in an attempt to improve discharge coefficients. The use of the three types had no apparent effect on the action of the spillway. Their quantitative effect on the discharge is discussed in Bulletin 3 of this series: "Studies of Crests for Overall Dams."

41. Installation of Baffles. — The form of spillway finally adopted contains no baffles or other devices to break up currents and smooth the flow. However, the tests leading to this selection included extensive experiments on such devices. Most of the tests were performed on the C-4 model. Although considerable progress was made in improving flow conditions, satisfactory results were secured without the baffles and the additional improvement obtained by their use was not believed to warrant the extra expense involved in their construction. It is believed, however, that a record of the baffle experiments may be of value to other designers of spillways.

The tests were comprehensive, including a variety of basic forms with variations and combinations. In all, about one hundred different arrangements were tried.

The tests were conducted simultaneously with the developments leading to the final design, and therefore include runs with baffles using all the various transition forms previously discussed.

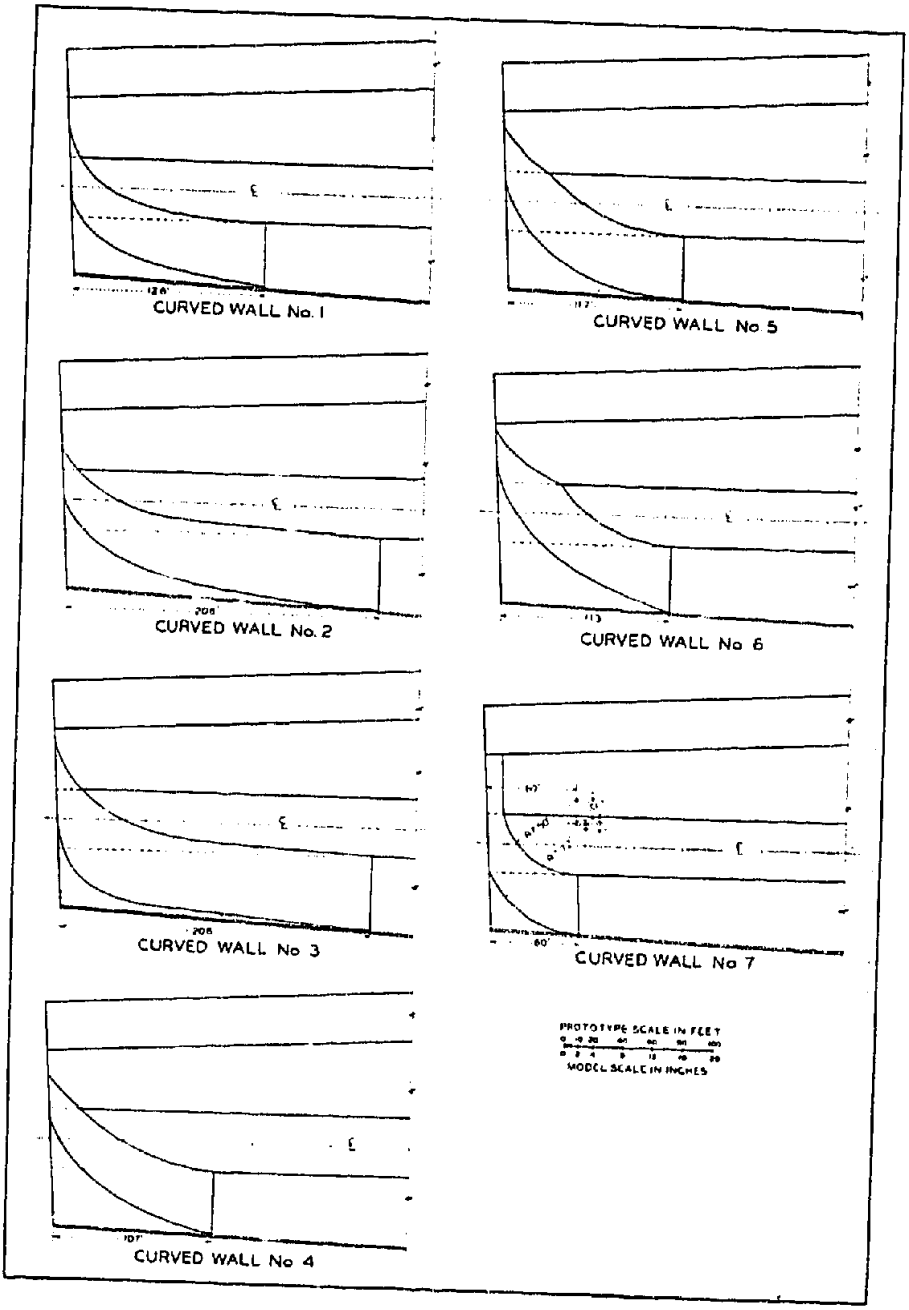


FIGURE 58—CURVED WALL AT UPPER END OF SPILLWAY CHANNEL

In order that the developments leading to the final design may not be obscured, it is believed better to discuss the main steps separately from the tests with baffles, rather than to cover both in a chronological order. In discussing the baffles, only the steps in the development of the principal forms are described.

The purpose of the various baffle devices was to check the velocity produced by the fall over the spillway crest, lessen the turbulence, and reduce the height of wave on the rear side of the channel. The accomplishment of these results improved flow conditions down the tunnel.

42. False Floor Across Channel.—One of the first layouts tested was the installation of a false floor sloping across the channel, intersecting the rear wall at an angle of 96 degrees at the elevation of the channel bottom, as shown on figure 59. This alteration was expected to raise the water surface on the crest side and direct the jet against a normal surface, on which the tendency to rise would be less than in the case of the original form where the angle was obtuse. The results indicated but little improvement over the original form. The effect of adding curved-face baffles in three positions on the cross sloping floor was also tried; one row near the rear wall, one on the center line, and the third near the crest side. The last position was most beneficial as the baffles tended to smooth the water surface across the channel during all conditions except very low flows. Some other minor changes of general form are shown on figure 59.

43. Trough in Front Half of Channel.—Another form of baffle tested was a ledge on the rear half of the channel, leaving a trough on the crest side as shown on figure 60. This is approximately the same form that was tested on the M-1 model, shown on figure 45. At high flows, the effect of the trough was beneficial; but at low flows it was probably less desirable than the original design. Details of some of the variations of the trough form are shown on figure 60.

Dentated baffles on the front edge of the ledge resulted in improved conditions. This form led to cutting teeth in the ledge itself, and then to the addition of projecting blocks, which made up the best form of baffle devised, see figure 61.

In its best form, the baffle construction was 20 feet high at the upper end, 30 feet high at the lower end, and was placed diago-

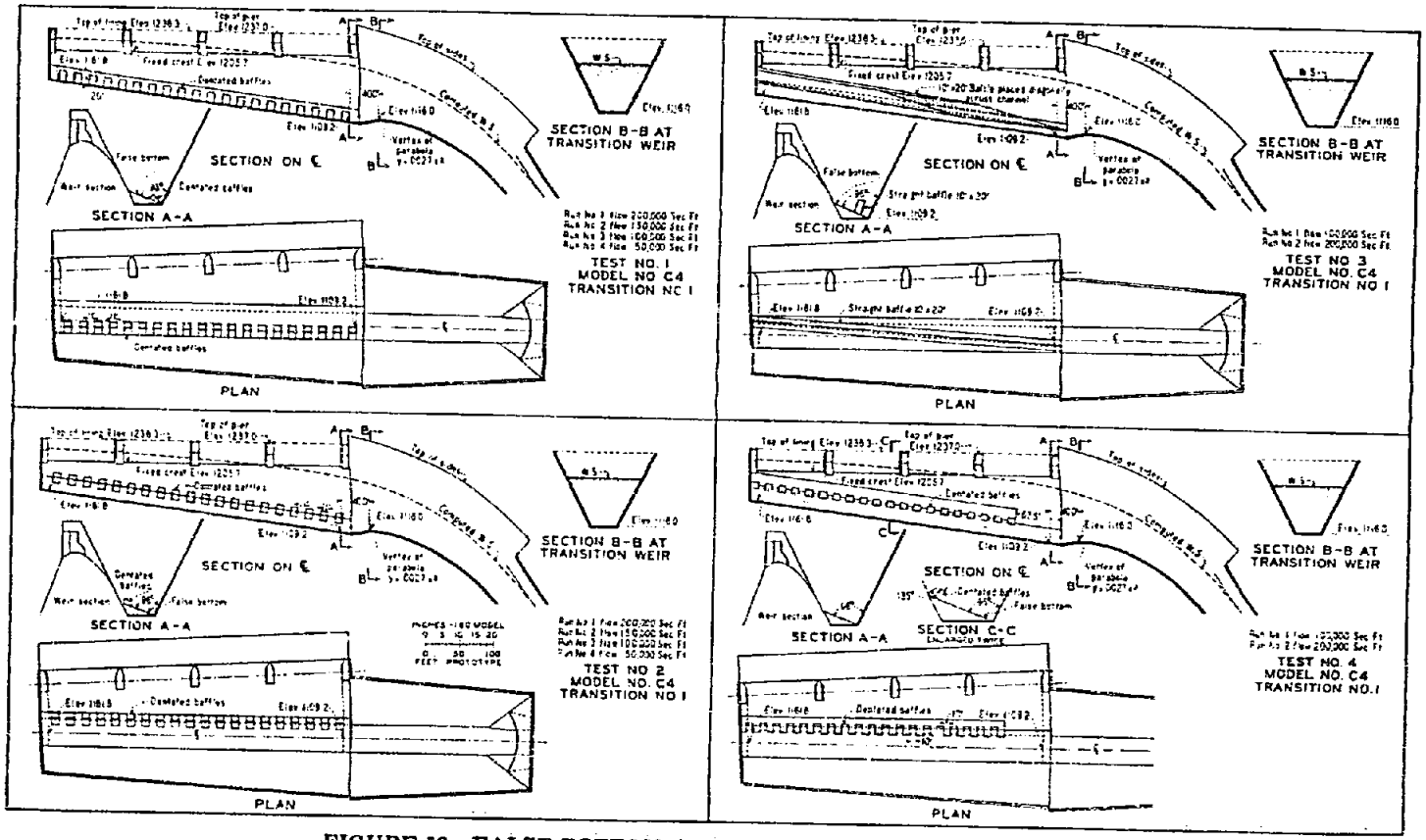


FIGURE 59—FALSE BOTTOM AT 96 DEGREES WITH REAR WALL

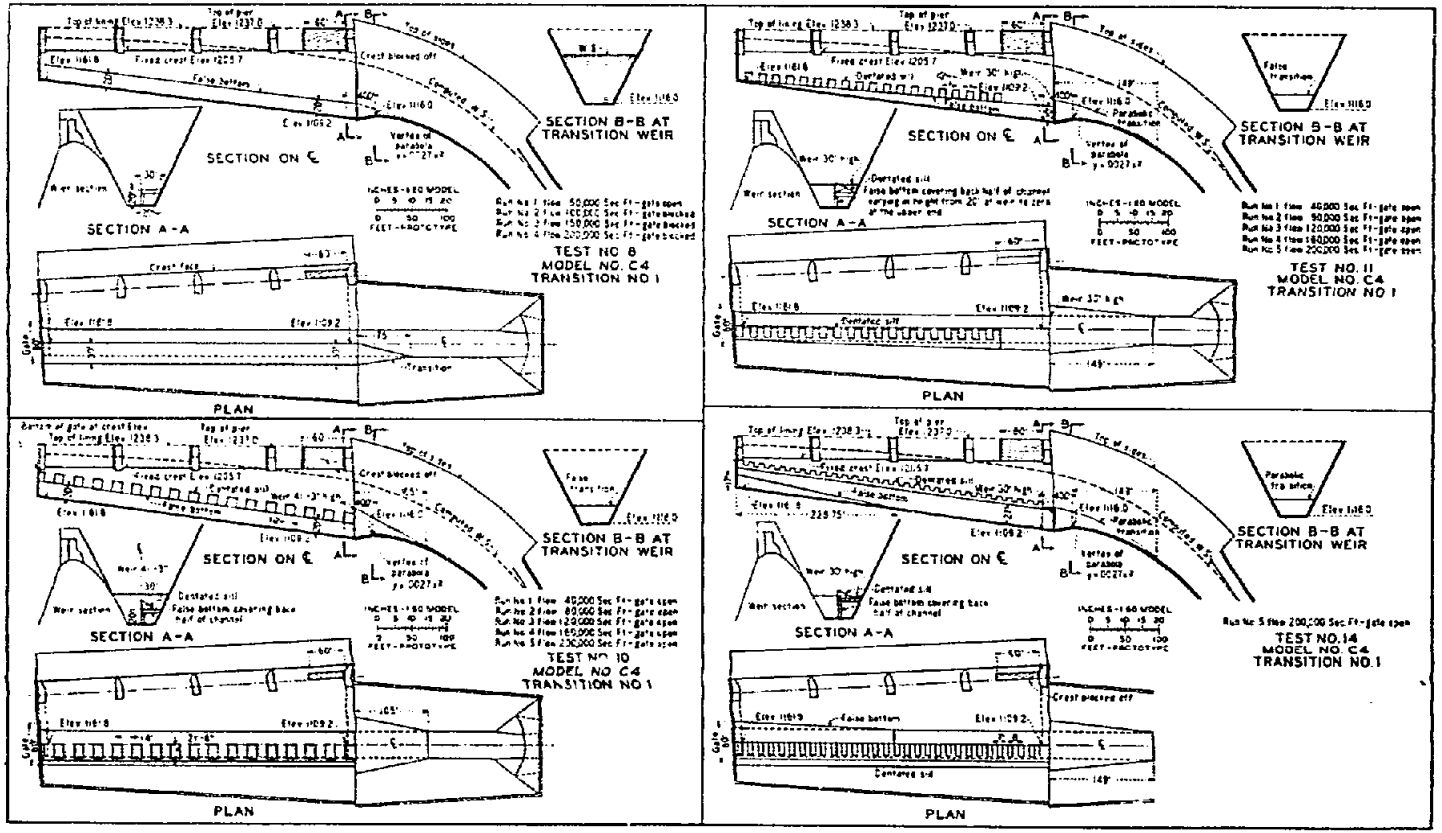


FIGURE 60—RIGHT ANGLE BAFFLE IN BACK HALF OF CHANNEL

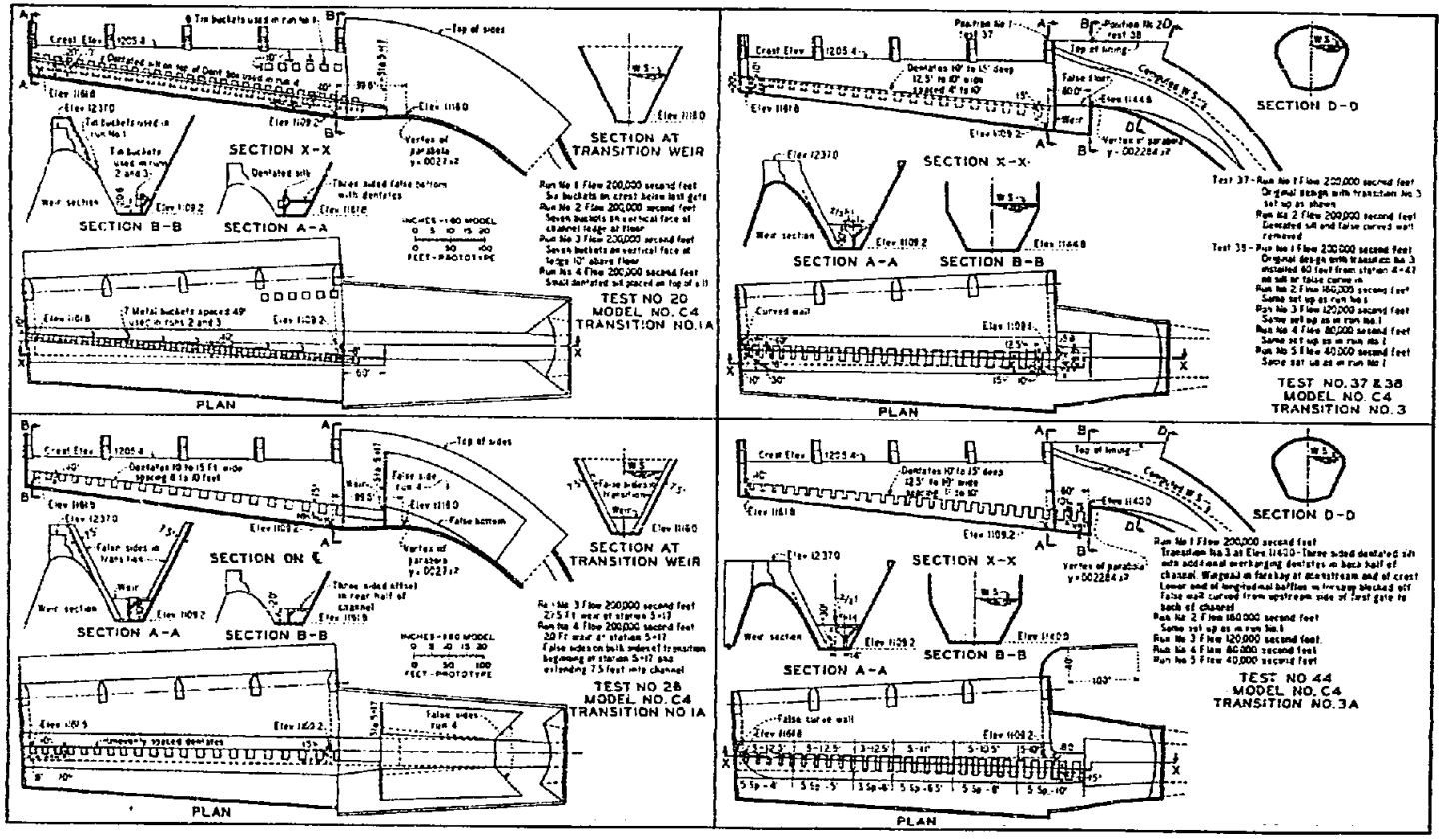


FIGURE 61—DENTATED BAFFLE IN BACK HALF OF CHANNEL

nally across the channel bottom from a position 10 feet in front of the channel center line at the upper end to 10 feet in the rear of the center line at the beginning of the transition. Teeth were made by cutting notches in the leading edge of the baffle, from 8 to 10 feet apart, varying from 10 to 15 feet deep along an angle of 45 degrees. Overhanging blocks were about two-thirds the width of the teeth. This type of baffle was tested with all the different transitions and cross-weirs. Combined with transition 3-A, figure 62, it gave the best results of any combination using baffles.

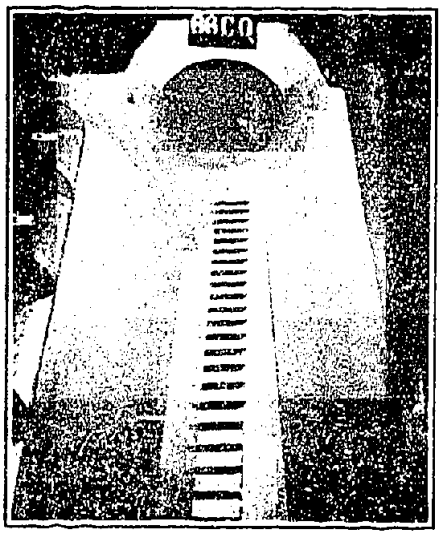


FIGURE 62—BEST FORM OF BAFFLE DEVELOPED

By the action of this baffle, part of the water falling over the crest was deflected back toward the crest side, thus tending to make the water surface more nearly equal on the two sides of the channel. The baffle face was set at a slight angle with the direction of the center line, since with the face set on the center line, there was a tendency for the water to pulsate back and forth across the channel in a form of tidal motion. At a discharge of 200,000 second-feet, the upper end of the crest was submerged and the overflowing sheet of water did not dive to the bottom of the channel and strike the baffles but passed over the top. The effect of the baffles at the upper end was, therefore, not as great at maximum discharge as at somewhat lower flows.

44. Deflectors on Crest Side.—Several other forms of baffles and deflectors were tested, the details of which are shown on figure 63. The first was a straight deflector with curved upper surface attached to the overflow face 20 feet below the crest elevation.

The form shown on figure 63-C gave better results. This form caused two opposing currents. The portion of the overfalling nappe which passed between the teeth followed around the channel perimeter and impinged on that portion which was deflected in a nearly horizontal direction from the vertical faces of the teeth, resulting in an efficient means of dissipating energy. The blocks had to be set high enough to allow the under-roll to form beneath. This was objectionable at low flows, particularly at the upper end of the channel where no water cushion was available to receive the impact of the jet.

A combination of the curved deflectors with a coping baffle, similar to that tested on the M-1 model, is shown on figure 63-C. This combination gave a satisfactory dissipation of energy but introduced so much obstruction to longitudinal flow that the discharge over the upstream end of the crest was materially decreased. The form shown on figure 63-D seemed fairly satisfactory for high flows but gave undesirable results for low flows.

A trough on the rear side of the channel, similar to one tested on the M-2 model, did not give satisfactory results.

45. False Wall in Channel.—In addition to the baffle studies, a few tests were made to investigate the possibilities of changing the channel cross section. This was done by inserting a false rear wall in the channel, making an angle of 60 degrees with the horizontal. This contraction improved flow conditions for a given height of cross-weir. As the height of the cross-weir was increased, the flow was improved. The result was not so satisfactory as with the larger channel and cross-weirs producing the same degree of submergence.

MODEL WITH DRUM GATES, 1:20 SCALE (M-3)

46. Design and Construction of Model.—In order to secure larger scale tests of the results obtained on the 1:60 drum-gate model at the Colorado Agricultural College laboratory, a similar model was constructed on a 1:20 scale at the Montrose laboratory.

This model was of the same design as that tested at Fort Collins, using the 3-A transition shown in figure 53. The construction was similar to that of the M-1 model. Three windows were placed in

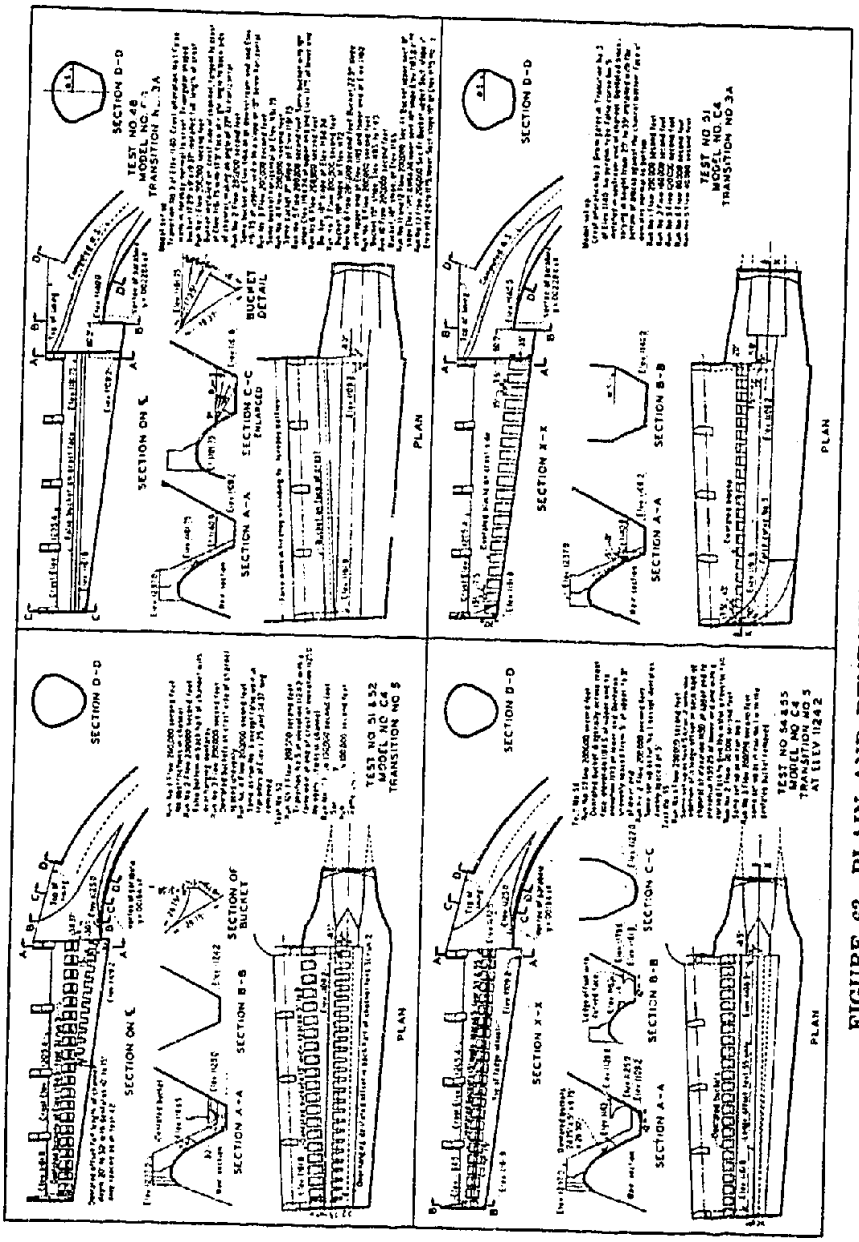
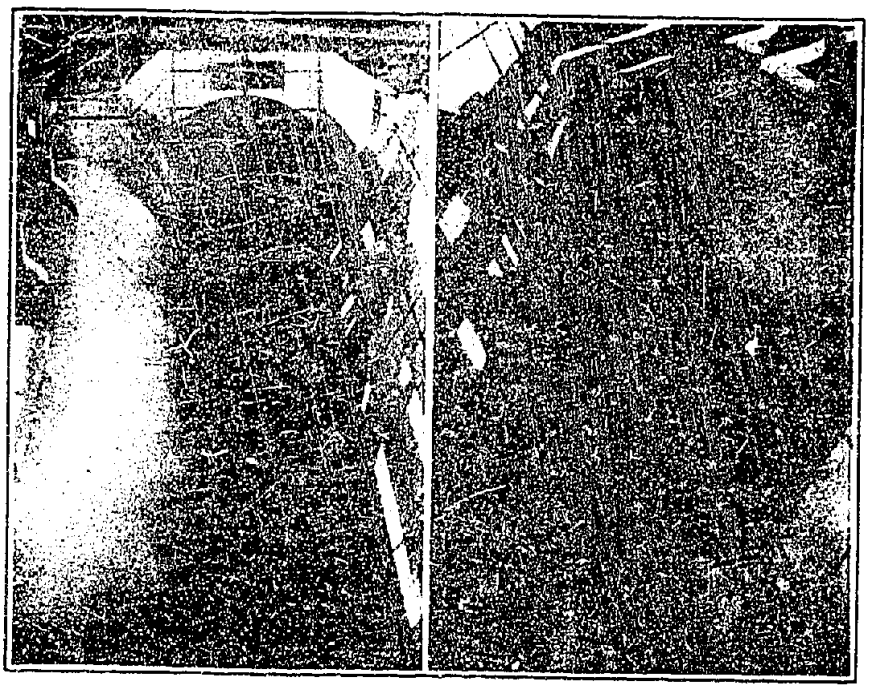


FIGURE 63—PLAIN AND DENTATED BUCKET ON CREST SIDE

the rear wall so that the action of the water within the spillway channel could be observed.

47. Results from Original Design.—A number of experiments were conducted on the M-3 model. Extensive quantitative observations were made, including cross sections of channel flow, pressures on channel bottom and sidewalls, pressures on the transition bottom, coefficients of discharge for various forms of crest, coefficients of discharge for drum gates in different positions, measurements of air content in the spillway channel, and measurements of velocities in the spillway tunnel. Quantitative results are discussed in detail in a later section of this bulletin.

Photographs of the model are shown on figure 64 and the action in the model for a discharge of 200,000 second-feet is shown on figure 65. At the lower discharges, the actions of the 1:20 and 1:60 models were very similar. The photographs for the 200,000 second-feet discharge did not show exact similitude as they were taken



A. Looking Downstream B. Looking Upstream

FIGURE 64—DRUM-GATE SIDE-CHANNEL SPILLWAY, MODEL M-3

FIGURE 65—200,000 SECOND-FEET DISCHARGE IN ORIGINAL DESIGN OF MODEL M-3

at different time intervals in a surge which existed at this discharge.

48. **Curved Wall at Upstream End.**—Experiments were made with a curved wall at the upper end of the channel, similar to those investigated in the 1:60 model. Figure 66 shows one of the walls in place and the action of the spillway with a 200,000 second-foot discharge.

49. **Installation of Dentated Baffles.**—Two forms of dentated baffles were studied, similar to the overhanging dentated form in the 1:60 model. The first form was an exact duplicate of that shown in figure 62. In the second form alternate teeth of the first were removed.

The action in the spillway channel with the first form of dentated baffles and the curved wall at the upstream end is shown on figure 67. The action with the second form was quite similar to that produced by closer spaced teeth. Both baffles resulted in the smoothing of the water surface in the channel, but offered some obstruction to the flow. The improvement was not believed to be

A. View of Wall B. Discharge 200,000 Second-Feet

FIGURE 66—CURVED WALL AT UPPER END OF CHANNEL

FIGURE 67—ACTION WITH DENTATED BAFFLE AND CURVED WALL, DISCHARGE 200,000 SECOND-FEET

sufficient to justify the expensive construction necessitated by baffles, and therefore they were omitted in the final design.

50. Flow Conditions, Drum Gates Raised.—A series of experiments was made with the drum gates raised, to determine the action of the spillway and the possibility of obtaining better flow conditions at low discharges by adjusting the gates. Figure 68 shows the conditions in the spillway when discharging 31,500 second-feet per spillway with all gates at their maximum elevation, 1221.7. This is the condition estimated for a discharge equal to the 1884 flood if the flow is distributed equally between the two spillways. The 1884 flood was the largest for which reliable records are available.

Experiments were made to determine the effect of different gate elevations on the channel flow. Two combinations of gates were tried. In the first, the upstream gate, number 1, was entirely down, gate 2 was raised to the quarter position, gate 3 to the half position, and gate 4 to the three-quarter position. In the second combination, the positions of the gates were reversed. The discharge in each case was 144,000 second-feet, which was the largest flow the gates would handle with the maximum reservoir elevation of 1232.0.

These experiments showed that at low discharges, improvements in flow conditions could be secured by having the greatest flow over the downstream end of the spillway; while at larger flows

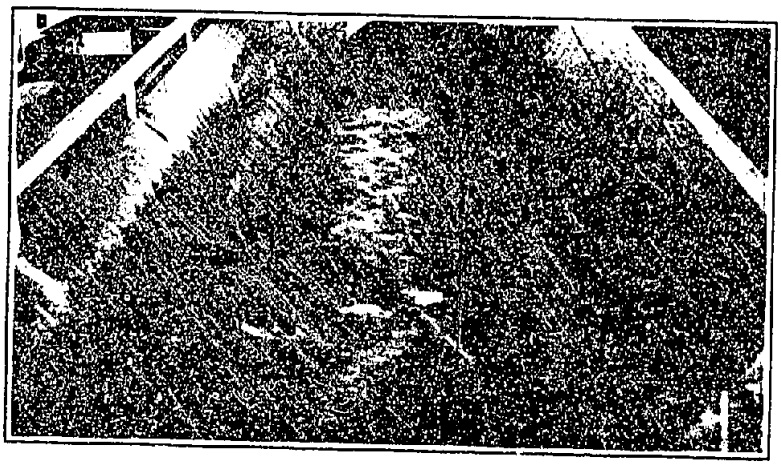
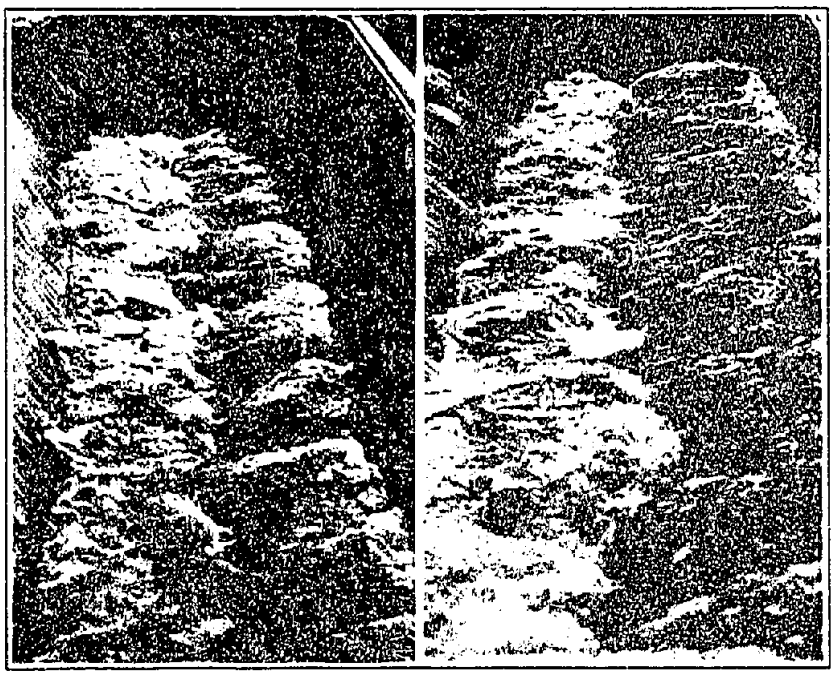


FIGURE 68—31,500 SECOND-FEET DISCHARGE WITH ALL GATES RAISED TO ELEVATION 1221.7

the best results were obtained with the greater flows over the upstream end of the crest. Due to the formation of a pool immediately upstream from the cross-weir the depth of water at the lower end, for small discharges, was relatively greater with respect to that at the upper end, than for large flows. This greater depth made possible the dissipation of energy at small flows with relatively little disturbance, while water passing over the upper end of the crest did not fall into the pool. At larger discharges the depths at the two ends of the channel were more nearly equal; but the drop from reservoir level to channel was less at the upper end and the disturbance there was not so pronounced as at the lower end.

Flow conditions for two discharges with gates uniformly raised one-fourth are shown on figure 69. In this case, the turbulence in the channel was slightly greater than for the gates down, since the water level in the reservoir was higher for the same discharge. The drop, therefore, between the reservoir surface and the water level



A. Discharge 50,000 Second-Feet B. Discharge 150,000 Second-Feet

FIGURE 69—ACTION IN CHANNEL WITH GATES RAISED ONE-FOURTH

in the channel was greater, causing more energy to be dissipated in the side channel with more turbulence.

51. Direction of Currents.—Studies of directions of currents in the side channel were made by observing paths of colored water introduced into the flow immediately in front of the windows in the rear side of the channel. In all cases the color indicated that the flow was diagonally upward in a downstream direction. This observation confirms the results obtained on the C-2 model. The color showed an upward direction of flow at the downstream window, due to the water rising to pass over the weir at the entrance to the transition. Rising flow was also evident at the windows upstream, due to a combination of the rising effect on the back side of the channel and the longitudinal flow in the channel.

52. Air Content of Water.—Extensive experiments were made to determine the air content in the water passing down the side channel and to determine the pressures on the sidewalls and in the transition section. Proper allowance for air content has been one of the uncertainties in the design of side-channel spillways. The flow over the spillway crest falls into the water in the channel with considerable turbulence and carries with it air, which occupies space and must be provided for in the channel design. This subject had been previously investigated, both by model experiments, and by observations on the side-channel spillway at Arrowrock Dam[1]. Those experiments indicated that under certain conditions the air content might be large. As the cost of providing the additional channel capacity in the Boulder Dam spillways would be great, it was desirable to determine as accurately as possible the percentage of air that would be mixed with the water. Consequently, observations were made on the model spillways to obtain data which would be of value on this subject.

53. Measurement of Air Content.—Two methods of measuring air content were tried. One had been used in the previously cited experiments. It consisted of observing pressures on the bottom of the spillway channel with piezometers and computing the air content from the differences between the pressures indicated and the pressures which would be caused by the observed depths of water if no air were present.

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[1]Trans. Am. Soc. C. E., vol. 89, pp. 912 and 919.

The second method was based on sampling. A sample of the flow was withdrawn from the side channel through a metal tube at the point where it was desired to determine the air content. The tube was connected by a rubber hose to a closed air tank in which the air and water separated. The air rose to the top of the tank and the water was withdrawn from the bottom. The water was measured volumetrically on the small model and by a water meter on the larger model. The quantity of air was determined from the increase in the volume of the air in the tank, as observed from a gage glass on the side, care being taken to have the air in the tank at atmospheric pressure and at the same temperature as the water when the volume was observed.

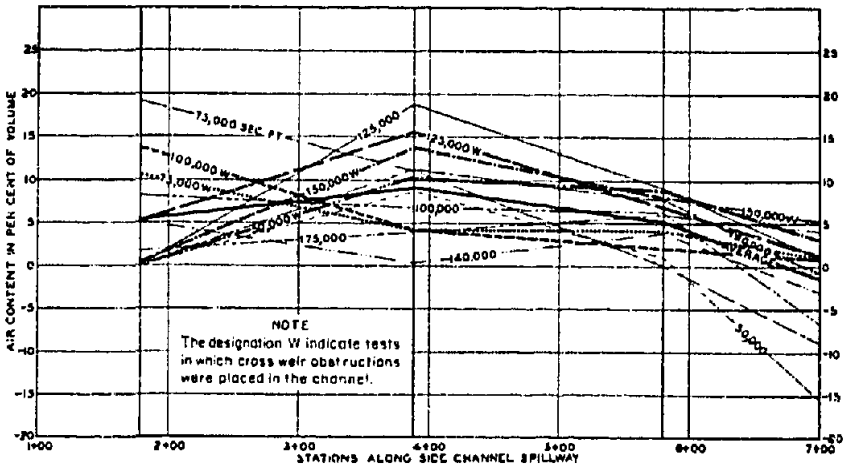


FIGURE 70—AIR CONTENT IN SPILLWAY CHANNEL OF MODEL C-2

54. **Results of Air Measurements.**—Observations in the first side-channel spillway, model C-2, were made by the first method. Piezometers were connected on the center line of the channel bottom at four points along its length. Observations were made in the trapezoidal channel with and without the cross-weir. The results of the estimates obtained by this method are shown on figure 70. The results are quite variable, extending from +19 percent to −15 percent, with averages considerably higher than shown by the second method. It is possible that closer agreement would have been secured if the elevation of the water surface had been taken

immediately above the piezometer opening instead of using the average water surface at the cross section. A distinct reduction of air content is indicated at the lower end of the channel, due no doubt to the reduction of pressure of the water on the bottom as it accelerated toward the inclined shaft.

Measurements of air content were made by the second method at two cross sections in the free-crest side channel, model C-2. The first section was at the middle of the channel length. The results are shown on figure 71. The maximum air content was 1.41 percent and occurred in the region of the core of air which ran through the center of the channel. The average content was 0.5 percent. The second cross section was at station 7+00, located near the lower end of the channel. The water passing this end of the spillway crest fell a greater distance to the water surface in the channel, causing

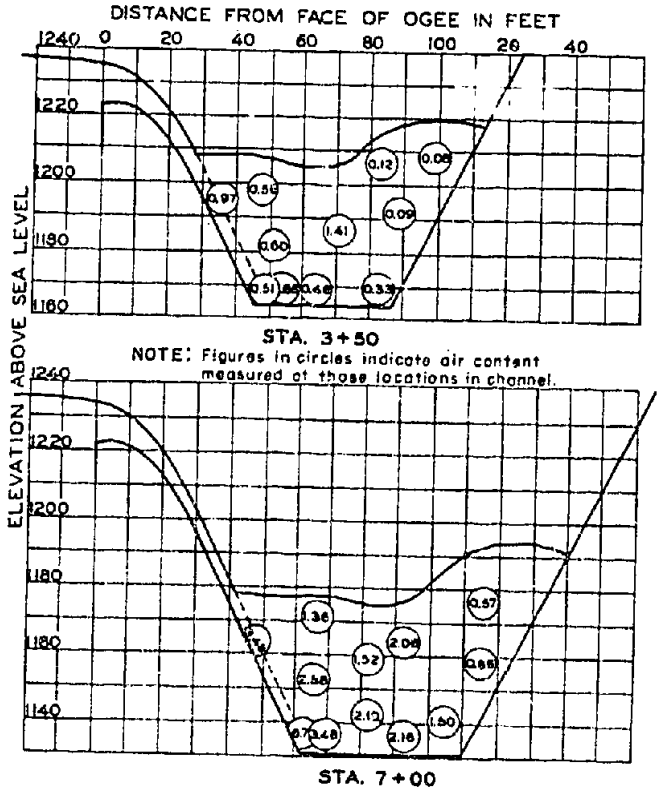


FIGURE 71—PERCENT OF ENTRAINED AIR IN SIDE-CHANNEL SPILLWAY MODEL C-2

more turbulence and a greater air content. The maximum content in this case was 6.7 percent and the average 2.2 percent. These contents are based on air at atmospheric pressure and are slightly larger than for the pressures which existed in the model where the air was under a slight pressure.

Observations were made by the second method on the M-3 model, at the three stations along the spillway shown on figure 72. The results were not definite, but tended to indicate more air toward the downstream end. The average air content at the downstream section was the largest of the three and reached only 1.08 percent.

Observations were made by the second method at the downstream end of the channel of the M-5 model, for discharges of 200,000, 160,000, and 120,000 second-feet. Here again the results were not definite, but the largest average content was only 1.09 percent.

The fact that the air content was small was further demonstrated by the comparative transparency of the water. If the water had contained a large proportion of air it would have been opaque or translucent, rather than nearly transparent.

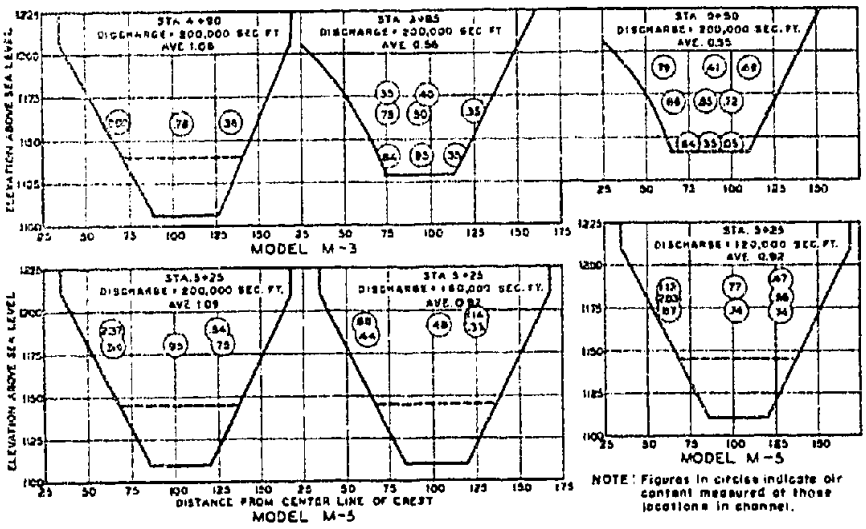


FIGURE 72—PERCENT OF ENTRAINED AIR IN CHANNEL, MODELS M-3 AND M-5

55. Accuracy of Methods.—The first method of measuring air content assumes that the flow in the channel in the vicinity of the piezometer is normal to the piezometer opening and is not accelerating. This is only approximately true in the side-channel spillway. Observations on piezometers located at various points in the channel indicated that high local pressures against the floor and walls existed at certain points. Whether or not such pressures would act at the location of the piezometers for measuring the air content is uncertain. Near the lower end of the channel the water was accelerating and therefore did not exert as much pressure on the bottom as its depth and normal weight would indicate. This effect appeared in the difference between the two heights from which the air content was determined, producing serious errors, as in some cases negative air contents were indicated. In general, it may be said that while this method might give good results under certain conditions, it was not reliable for the conditions existing in the model spillways discussed in this report.

The second method should give reliable results if a representative sample is secured. Two of the possible sources of error were leaks in the air tank and the possibility that air escaped with the water discharged from the tank. The tank was tested by allowing it to remain long periods under pressure.

The most representative sample would be secured if the orifice through which the sample was taken was so placed that it caused as little disturbance in the flow lines as possible. In these experiments the samples were drawn through a 7/8-inch brass tube with its open end pointed upstream. If the cylinder of water flowing directly toward the end of the pipe was intercepted it would cause no appreciable disturbance. If more or less water than that in the cylinder approaching the orifice was intercepted, some deflection of the stream lines would result. If the sampling tube intercepted more of the mixture than was flowing directly towards it, adjacent currents would be drawn into the tube which would probably contain more air than water, since the mass per unit volume of air is less than that of water. If less water flowed through the orifice than was directly approaching it, relatively more air than water would be deflected and the sample would contain less air than it should. In the tests, no attempt was made to have the sampler intercept just the quantity of water which was directly approaching it, but it is believed that the results obtained are reasonably near the true conditions. It is believed that this method of measuring

air content in the water was reasonably reliable and sufficiently accurate for practical purposes.

56. Model and Prototype Results.—The relation between the air content in the model and that which will occur on the prototype is not well established. Where opportunity has existed to observe both prototype and model actions, the flow in the prototype has appeared to be relatively rougher with more foam and mist.

Air in the model spillway channels, when carried down into the channel, was subjected to a pressure of only a few feet at the most. In the prototype, however, it would be subjected to many times this pressure. Consequently, the same quantity of air would occupy a relatively smaller space in the prototype. These two effects would tend to offset each other.

To obtain comparable results of spray and foam in the model and prototype, the viscosity of the liquid must be to scale along with other necessary factors. Air resistance is practically negligible in models of moderate size in which water is used for the fluid as air resistance does not appreciably influence the flow until the velocity reaches about 15 feet per second. In neither of the larger spillway models did the velocity reach this value. This undoubtedly explains why the air content appeared practically the same in both the 1:60 and 1:20 models. From what information is available,[1] the air content increases appreciably with the velocity of flow for velocities greater than about 15 feet per second. The prototype, accordingly, will contain a larger percentage of air than did the 1:60 and 1:20 models. It appears impossible, therefore, to determine with any degree of accuracy the air content in the prototype from a model using water as a test fluid.

57. Conclusions on Air Content.—Although the quantitative results do not apply, the general observations indicated that for the Boulder Dam spillways, as finally designed, the air content at maximum discharge will be negligible. It was therefore neglected in the design of the side channel. An allowance of about 35 percent air content was made in the tunnel transition section to insure a free air space above the water. At low discharges, the air content may be appreciable; but such discharges do not control the size

[1]Schoklitsch, A., Kolkabwehr und Stauraumverlandung, Julius Springer, Berlin, 1935. Translation, "Prevention of Scour and Energy Dissipation" by E. F. Wilsey (Unpublished article in U. S. Bureau of Reclamation technical library).

of the transition and channel. Conditions are favorable for low air content at maximum flows because the spillway channel has a relatively large cross section and the drop of the water over the crest is small.

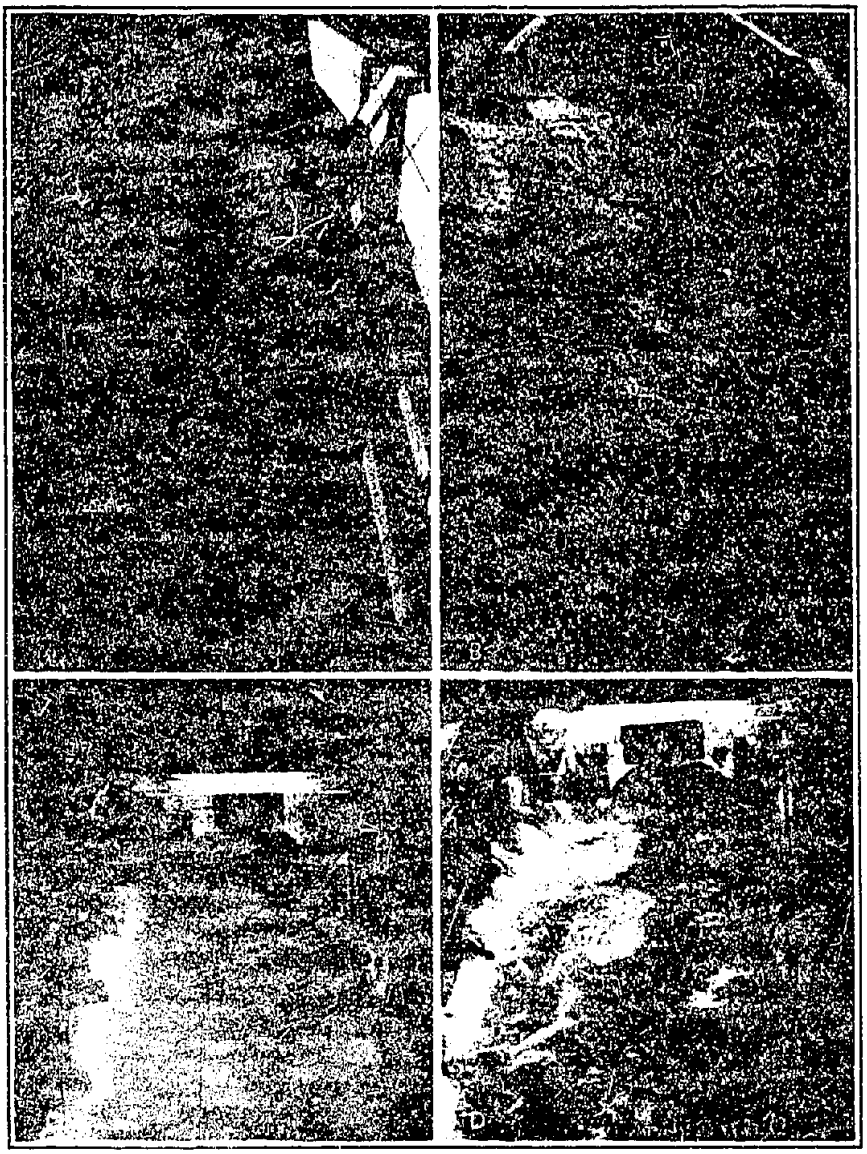
For designs of other dams it will usually be economical to have a low drop; but ordinarily a smaller channel cross section will be used, because the desirability of smooth flow down the chute will usually not be so important as in the inclined tunnels at Boulder Dam. With a smaller channel section or greater drop, the air content will probably be higher. In designing spillways where the air content will be great, it will be expedient to assume a varying air content along the channel, with a small air content at the upper end where the drop is small, and an increasing content down the spillway channel.

DRUM GATE MODEL, 1:100 SCALE (M-4)

58. Description and Results.—A model on a 1:100 scale was constructed of the same shape as that of the 1:20 model at Montrose and the 1:60 model at the Fort Collins laboratory with the 3-A transition. Prototype dimensions are given on figure 53. The transition was constructed with an open top so that flow could be observed and the water surface measured. The results were very similar to those obtained on the 1:20 and 1:60 scale models. Visual and quantitative comparisons of the results obtained on the three models are included in the following pages.

SIMILARITY RESULTS, PRELIMINARY DRUM GATE MODELS

59. Visual Results, C-4 and M-3 Models. — Results obtained from models C-4, M-3, and M-4 showed such close agreement that confidence was established in the principles of model testing, and it was felt that results obtained on models could be extended to the prototypes. The appearance of comparable tests are shown on figures 73 and 74. Figure 73-A and C show the M-3 and C-4 models and B and D show the action in the two channels with flows corresponding to 200,000 second-feet. Figure 74 shows the action in the two models for discharges of 160,000 and 120,000 second-feet. The appearances of the two were very similar, for all corresponding discharges, there being a strong resemblance even in



A. Model M-3, 1:20

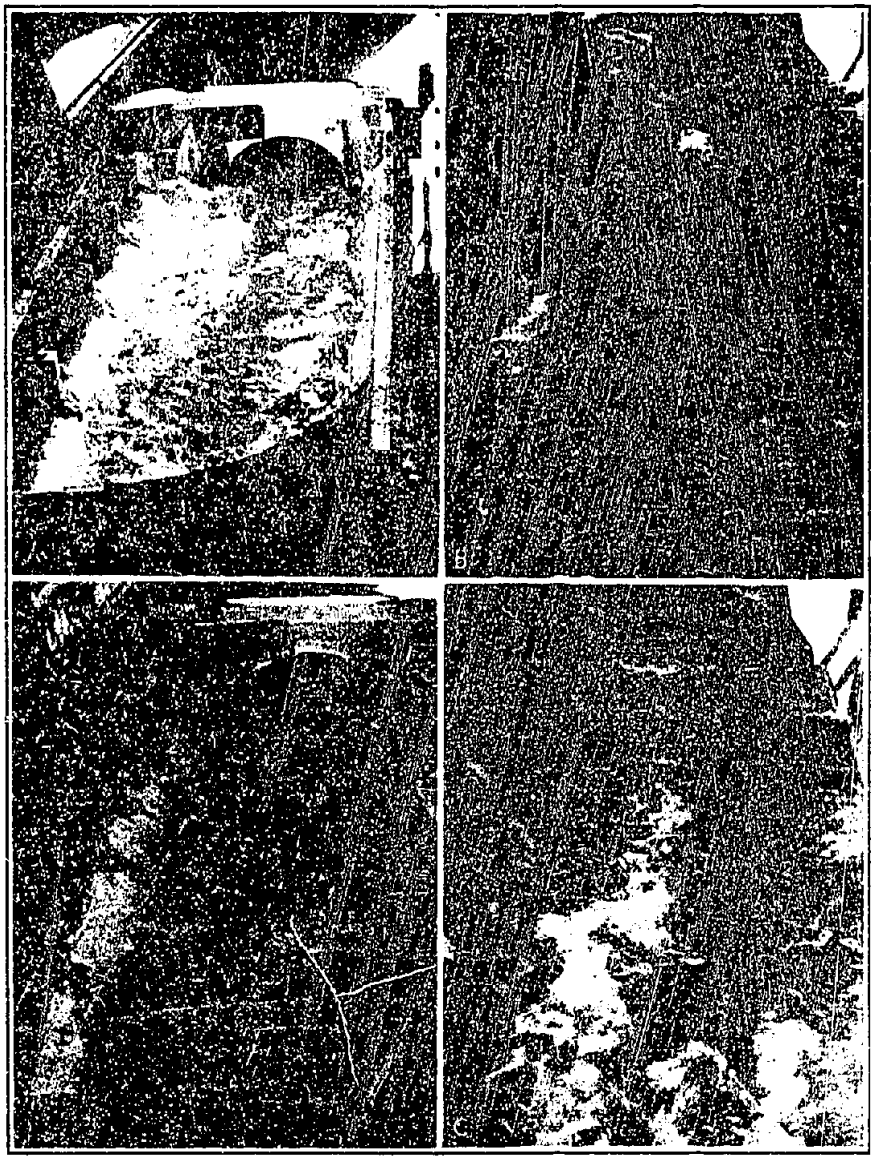
B. Model M-3

C. Model C-4, 1:60

D. Model C-4

Discharge 200,000 Second-Feet

FIGURE 73—COMPARISON OF FLOW CONDITIONS IN MODELS C-4 AND M-3 WITHOUT BAFFLE IN CHANNEL



A. Model C-4 B. Model M-3

Discharge 160,000 Second-Feet

D. Model C-4 C. Model M-3

Discharge 120,000 Second-Feet

FIGURE 74—COMPARISON OF FLOW CONDITIONS IN MODELS C-4 AND M-3 WITH DENTATED BAFFLE IN CHANNEL

the case of minor wave formations. In the 1:20 model, the water surface was more frothy than in the 1:60 model, due to the fact that the surface tension was relatively smaller in the case of the larger model and air resistance greater. Comparison of model and prototype results on other structures where available also show a similar relation, there being more spray and foam in the prototype. Very close similarity was also obtained with the quantitative comparisons of the 1:100, 1:60, and 1:20 scale models, as will be discussed in the following section. Visual comparison is not possible in the case of the 1:100 model because of the lack of satisfactory photographs.

60. Quantitative Results, Models C-4, M-3 and M-4.—The similarity of action of the M-3, M-4 and C-4 models is shown by the longitudinal profiles of mean water surfaces on figure 75 and by the typical cross sections on figure 76. A comparison of the water surfaces on the three models at station 4+00 is shown on figure 77.

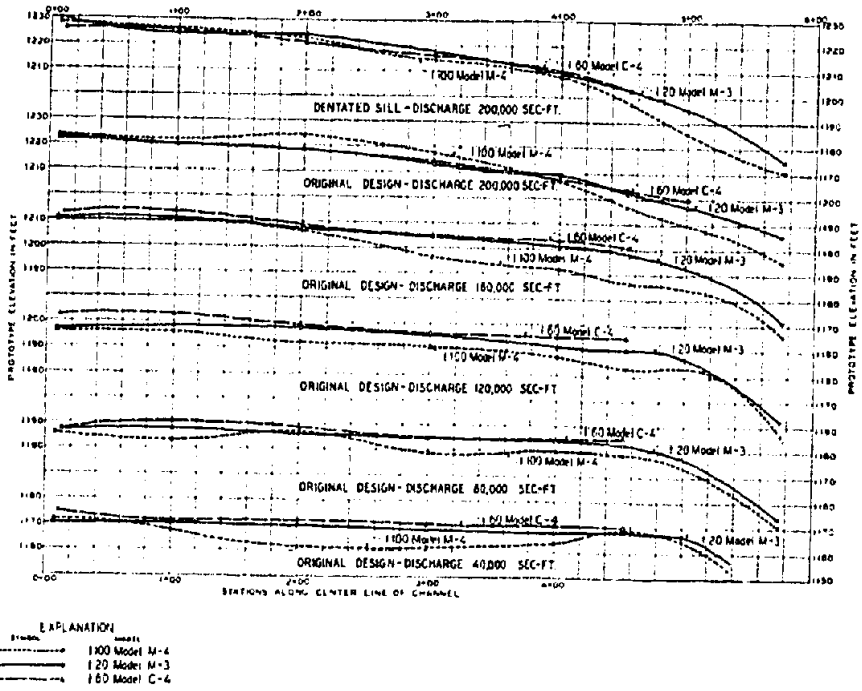


FIGURE 75—MEAN WATER SURFACE PROFILES
MODELS M-3, M-4, AND C-4

Very close agreement is shown from the profiles and sections. Water surface elevations on the 1:100, M-4, model were generally lower than on the other two models. Discharges were not measured on the 1:100 model, but were estimated from the head on the crest, using an assumed coefficient. For the lower discharges, the value of the coefficient is now known to have been too high. Consequently, the discharges used were relatively less than those on the other models. This probably accounts for the lower water surface elevations obtained on the 1:100 model.

Figure 78 shows the longitudinal profiles of the mean water surface for the semi-final design of the channel of the 1:20 model, as compared with those obtained with the dentated baffle on the back side of the channel. This comparison shows the effect of the baffle in obstructing flow down the channel, as indicated by the fact that the water surfaces for the same flows were considerably higher with the baffle in place. As the crest of the spillway in this model was at elevation 1205.4, the water surface in the channel at the upstream end was far above the crest and materially

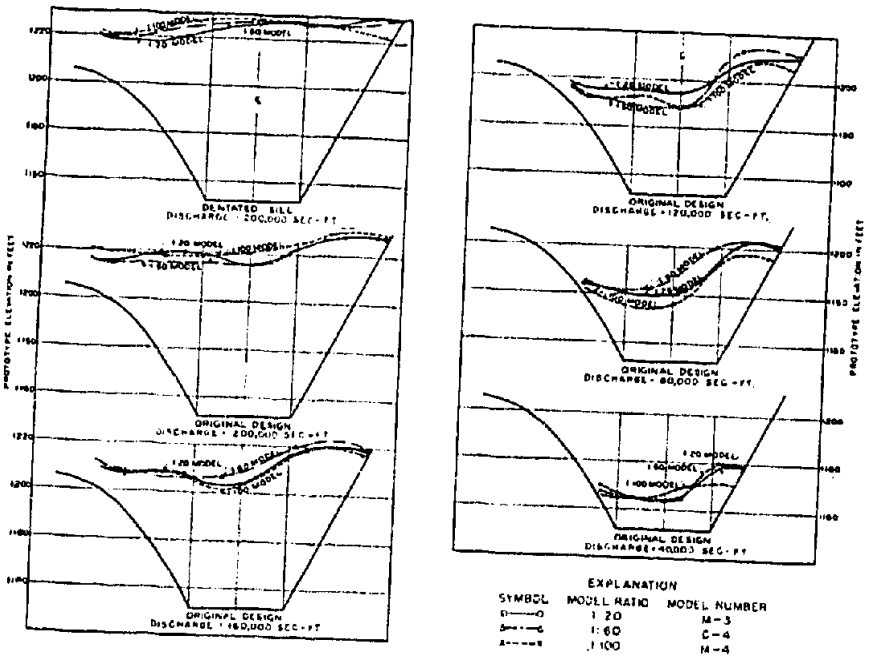


FIGURE 76—WATER SURFACE PROFILES AT STATION 1+00 FOR MODELS C-4, M-3, AND M-4

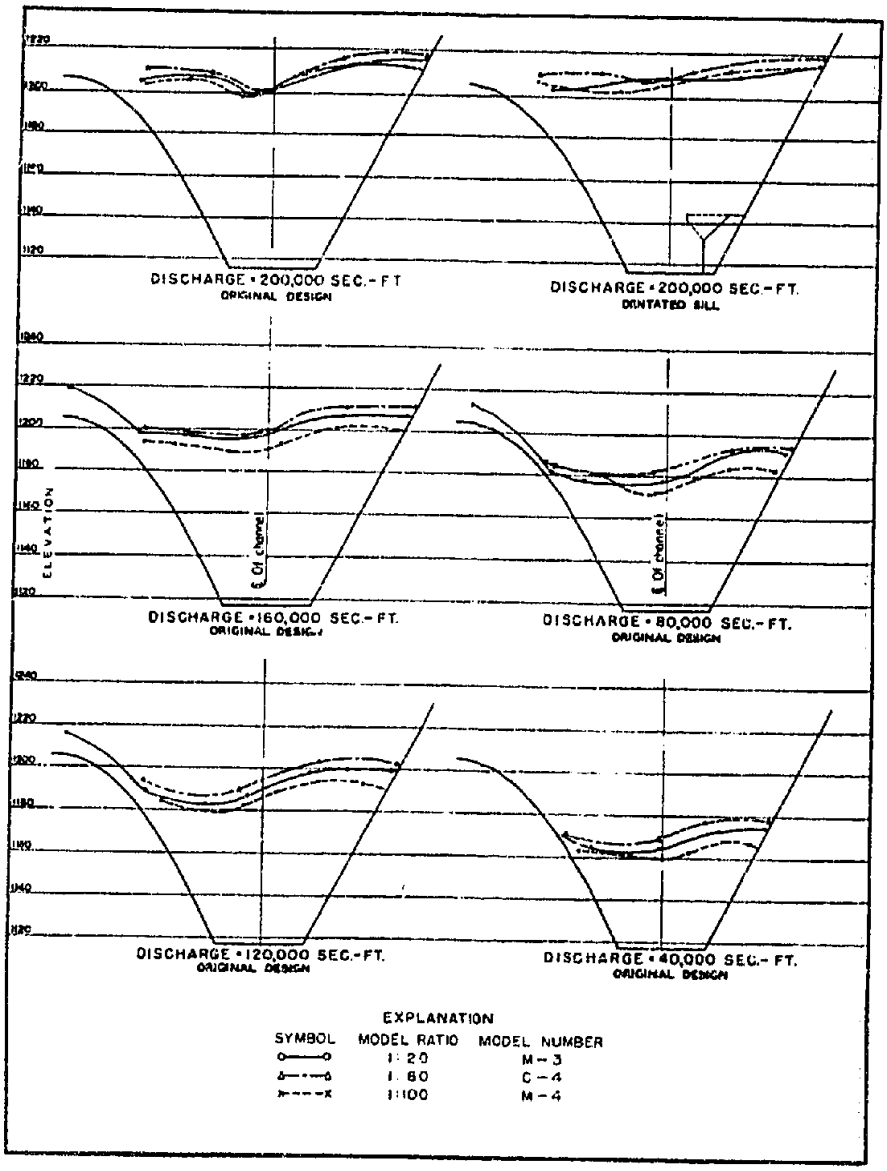
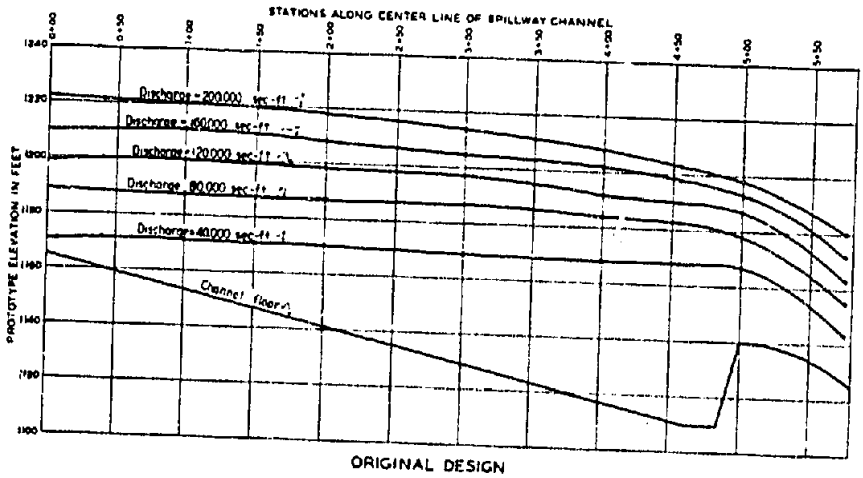


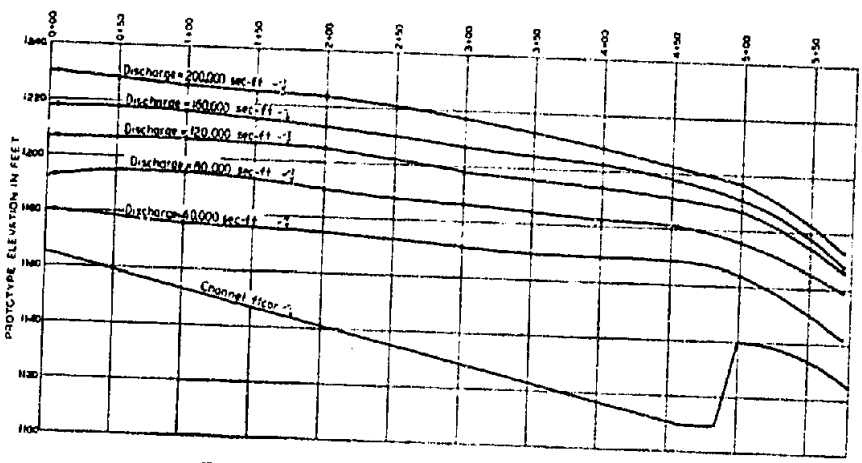
FIGURE 77—WATER SURFACE PROFILES AT STATION 4+00 FOR MODELS C-4, M-3, AND M-4

reduced the flow over the the overflow section, even when the baffle was omitted. With the baffle installed, more reduction resulted.

Piezometers were installed in the sides of the channel of the 1:20 model, M-3, and observations of water pressures made. For these runs no baffles were in the channel. The results of most of the observations are shown on figure 79. The solid line represents



ORIGINAL DESIGN



DENTATED BAFFLE IN BACK SIDE OF CHANNEL

FIGURE 78—MEAN WATER SURFACE PROFILES, SIDE-CHANNEL SPILLWAY MODEL M-3

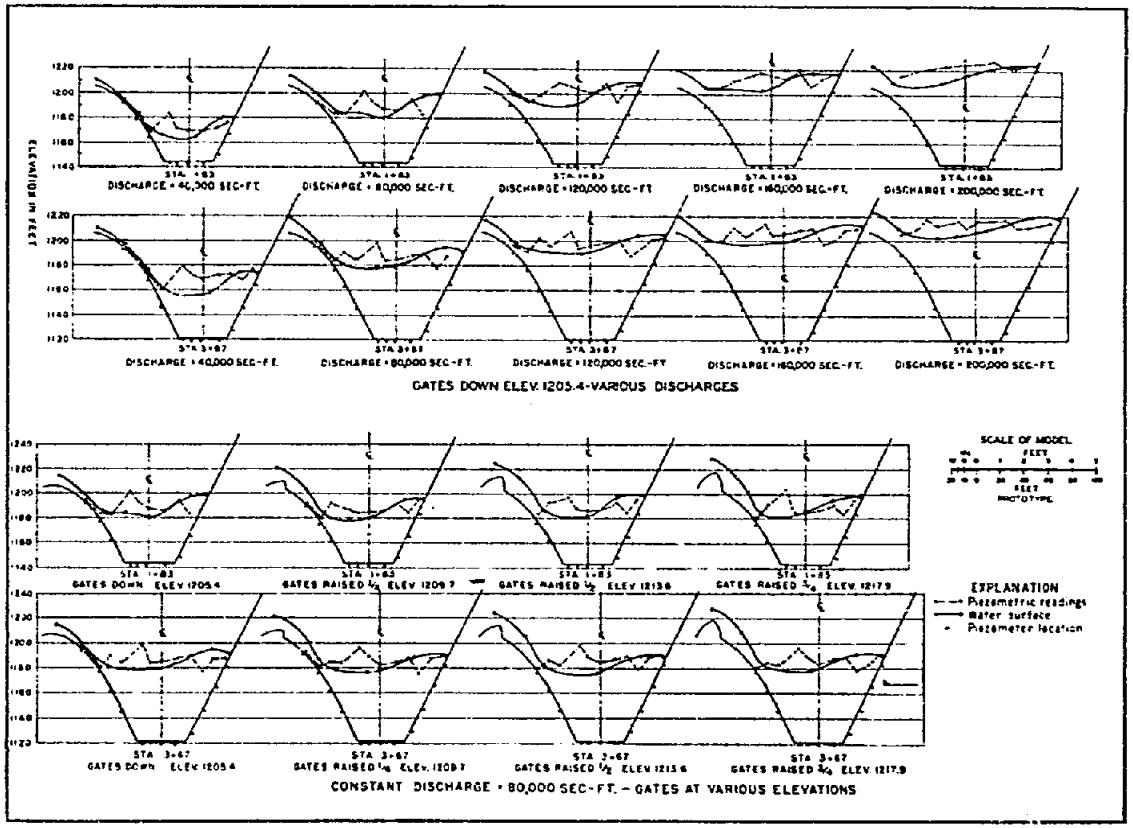


FIGURE 79—PIEZOMETRIC PRESSURES FOR VARIOUS GATE POSITIONS AND DISCHARGES, MODEL M-3

the observed water surface in the channel and the broken line connects the readings of the piezometer heights, the height of each piezometer being recorded vertically above the piezometer opening. The difference between the height of the piezometer surface and the height of the water surface represents mainly the excess or deficiency of pressure due to the impact or high velocity of the flowing water. In this difference is also included the effect of air in the water, as previously explained. Locations of excess pressure, in general, occurred on the bottom near the foot of the spill face and on the rear side, just above the bottom. These are the points at which the stream of water, first falling over the crest and then flowing across the channel bottom, was forced to abruptly change its direction. The excess pressure was the reaction of the force exerted on the moving water to cause the change in direction.

In general, the intensity of excess pressure was larger in the channel with low flows, since the drop of the water passing over the overflow crest was greater. The depth of water in the channel was less for the low discharges and exerted less cushioning effect. The greater the depth of water, the more the stream tended to spread, causing a decrease in the pressure intensity. This is no doubt one reason why the maximum intensity of excess pressure was greater at the foot of the spill face than on the rear wall. Additional factors contributing to this effect were impact and friction losses of the water on the bottom of the channel, reducing the velocity of flow and therefore reducing the pressure required to deflect the water.

One of the surprising results was the existence of a reduced pressure on the rear wall of the channel some distance above the floor. This was not an observational error, since it appeared on all three models. Its cause is not apparent.

According to the theory of the side-channel spillway, none of the energy of the water falling over the spillway crest is available for inducing flow down the channel. If this is the case, the mean water surface in the channel for a given discharge should be independent of the magnitude of the fall of the water flowing over the overfall section. That this is, in general, correct is indicated by the results from the 1:20 model, shown on figure 80, where the mean surface profiles in the channel are compared for various discharges with the drum gates in different positions. With the gates raised, the reservoir water levels were higher and therefore the fall to the channel greater. The profiles on figure 80 show no con-

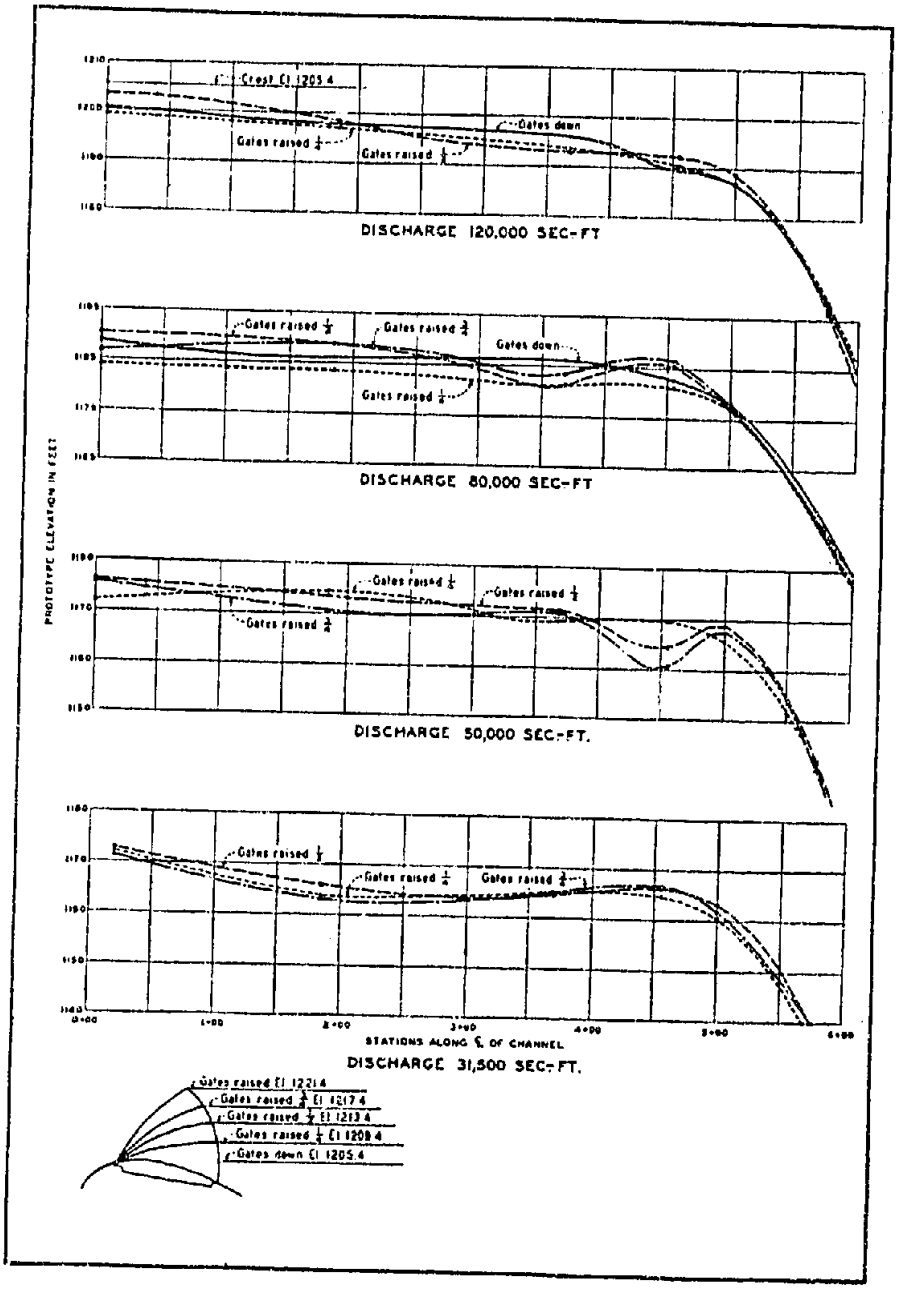


FIGURE 80—COMPARISON OF WATER SURFACE PROFILES, MODEL M-3

sistent variation of level with various positions of the gate, which tends to substantiate the above theory.

For hydraulic model tests to give the same results as the prototype, not only must the models be geometrically similar but there must also be similarity of the forces involved. In general, there are three types of forces,* (1) inertia forces, (2) viscosity forces, and (3) gravity forces. It is not always feasible to obtain exact similarity of model and prototype, as it is often impractical or impossible to obtain an exact similarity of all forces. Usually, however, certain of the forces so predominate that the lack of exact duplication of the others is negligible from a practical standpoint. In the spillway models the viscosity forces were relatively small. For such conditions geometrical similarity of the model and prototype will insure comparable results, except for the effects of air resistance. The only point in the spillway tests at which the viscosity forces were significant was in the model of the spillway tunnel at the Montrose laboratory. In this case the model tunnel was constructed on a grade which compensated for the relatively greater viscosity effects in the model.

*Tietjens, O. G., The Use of Models in Aerodynamics and Hydrodynamics, Trans. Am. Soc. M. E., Sept. 30, 1932, vol. 54, no. 18, p. 225.

CHAPTER IV—MODELS OF FINAL SPILLWAY DESIGN

FINAL DESIGN TESTS

61. General.—Studies were made of shapes of weir nappes and overflow crests to determine the form providing the greatest capacity of the drum-gate crest. A spillway crest was developed which gave a greater capacity at maximum reservoir elevation than the forms used on the semifinal models.

Observations on the C-4 model with transition 3 indicated that the pulsating conditions in the channel would be materially reduced, in fact, practically eliminated, by raising the cross-weir at the downstream end from elevation 1140.0 to 1145.0. With the crest shape used on this model, however, the spillway would not handle the maximum capacity of 200,000 second-feet, due to backwater effects present with a reservoir elevation of 1232.0.

By combining the more efficient spillway crest shape with the cross-weir at elevation 1145.0, conditions in the spillway channel and tunnel were materially improved and the spillway capacity was not reduced.

62. Program of Tests.—Tests on models of the final design fall into two classes, visual and quantitative. The visual tests consisted of:

1. Observation of flow conditions with all gates down and all gates partly and completely raised.
2. Improvement at the downstream end of the piers on the overflow crest.
3. Improvement of conditions at the upstream end of the spillway channel.
4. Development of a program for manipulating spillway gates during flood conditions.

The quantitative tests consisted of:

1. Determination of crest discharge coefficients for gates in different operating positions and a comparison of results obtained on the three models. These results are in-

corporated in Bulletin 3, Part VI, "Studies of Crests for Overfall Dams."

2. Observations of depths of flow in the spillway channel and a comparison of results obtained on the different models.

3. Determination of pressures on the spillway crest and in the channel, and a comparison of results.

4. Development of a program of gate operation during flood conditions.

Visual studies of flow conditions were made on all three models and a pictorial comparison prepared to determine the reliability of the principles of similitude.

The operating program was developed principally on the 1:60 model, where the gates were easier to manipulate, and the results checked by similar studies on the 1:20 model. Tests on the gate piers were made on both the 1:60 and 1:20 models. Studies for the improvement of flow conditions at the upper end of the channel were made only on the 1:20 model.

In the quantitative tests, crest coefficient determinations were made on all three models. Observations of pressures on the crest, channel, and transition were made on the 1:20 and 1:60 models. Observations on depths of flow in the channel and transition were made on all models. Air content measurements were made only on the 1:20 model.

63. Models M-5, C-5, and C-6.—The final design of the 1:20 scale model at the Montrose laboratory, M-5, was constructed of galvanized sheet iron with a timber frame similar to the M-1 and M-3 models. A plan and vertical section of the model is shown on figure 81. A cross section with the adjacent topography is shown on figure 82. The gates of the model could be raised to any desired height by screws operated from the inside. Windows were installed in the side channel to permit observation of flow conditions. The model was painted with a light gray paint, with black lines on the channel walls to indicate stations and elevations. The transition was a slightly modified form of the M-3 transition, and the tunnel was the same as for the M-1 and M-3 models.

The 1:60 model of the final design of the spillway, C-5, was constructed similar to the C-4 model. The design is shown on figure 83. It differs from the other models in having gates which could be quickly raised or lowered by push rods actuated through

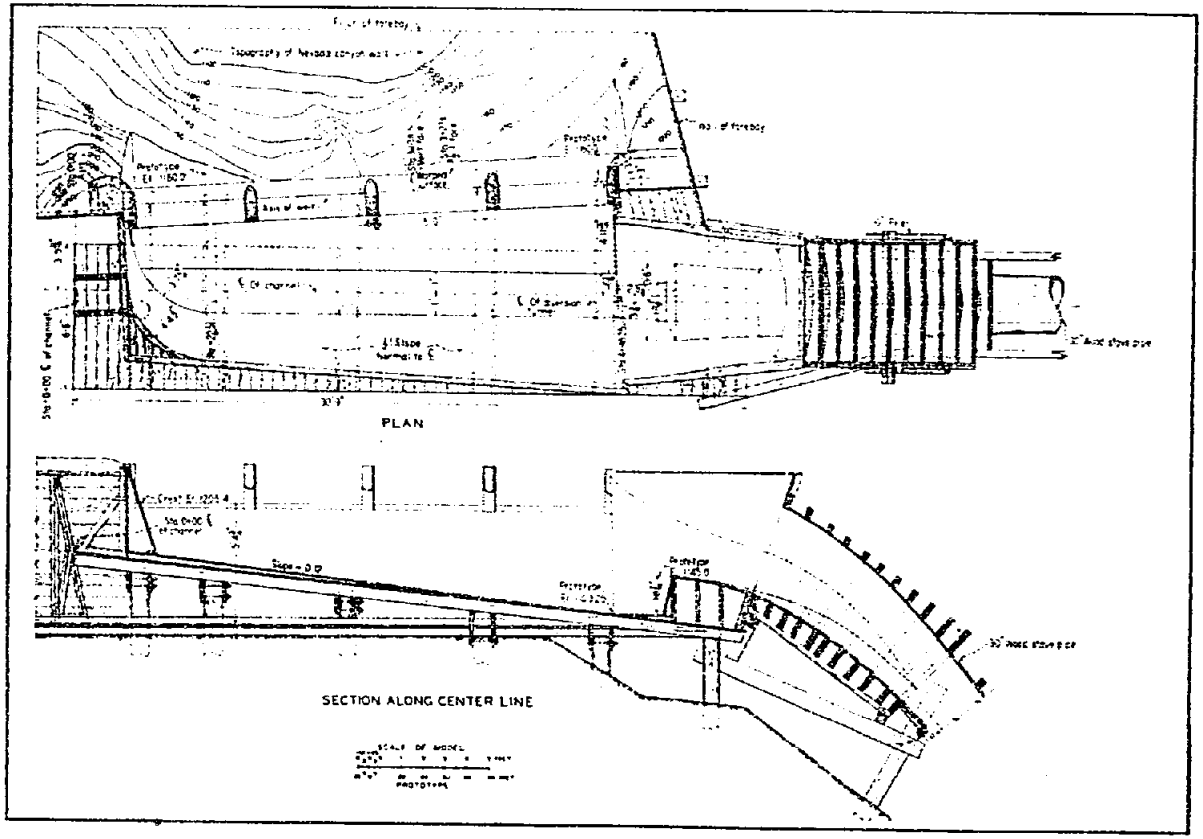


FIGURE 81—FINAL DESIGN OF SIDE-CHANNEL SPILLWAY, MODEL M-5, SCALE RATIO 1:20

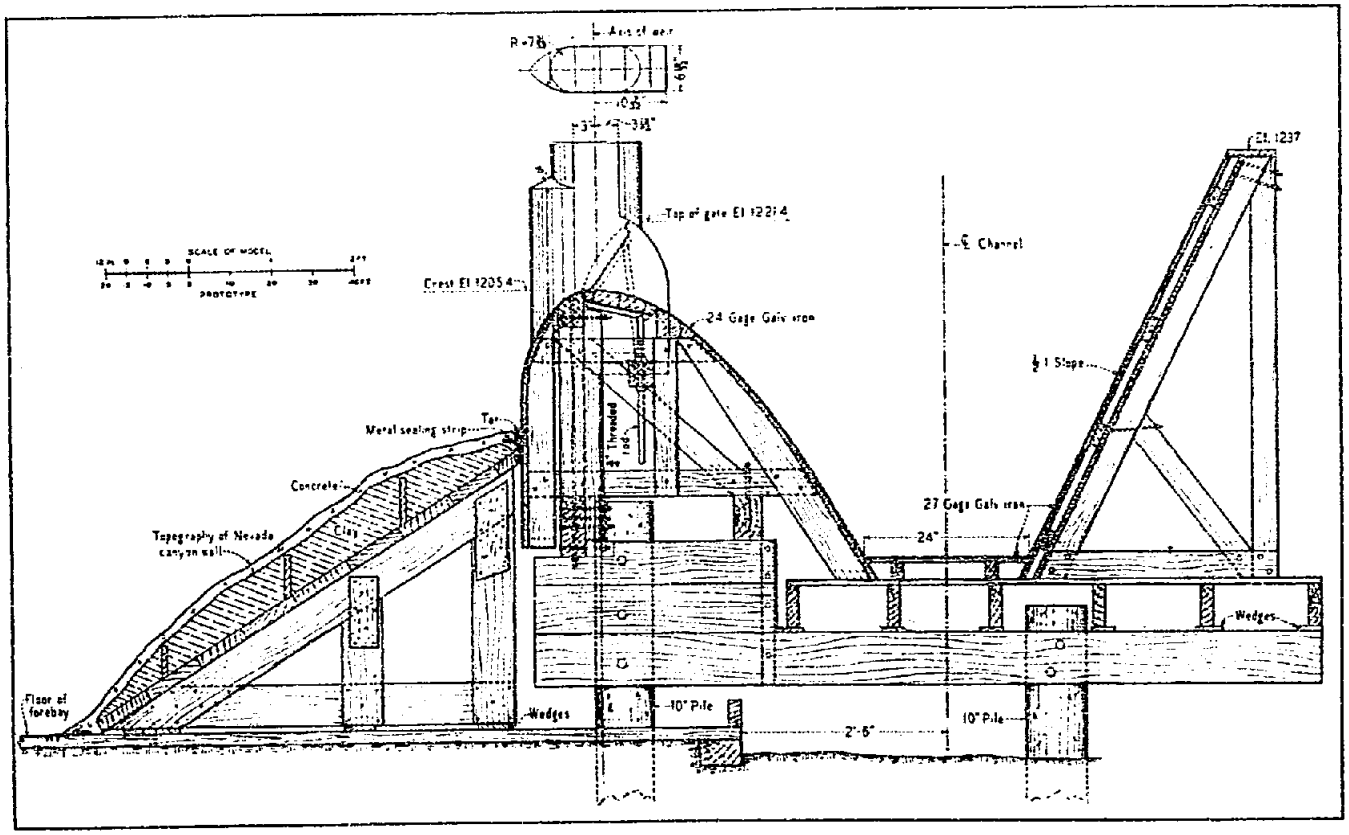


FIGURE 82—TYPICAL CROSS SECTION OF MODEL M-5

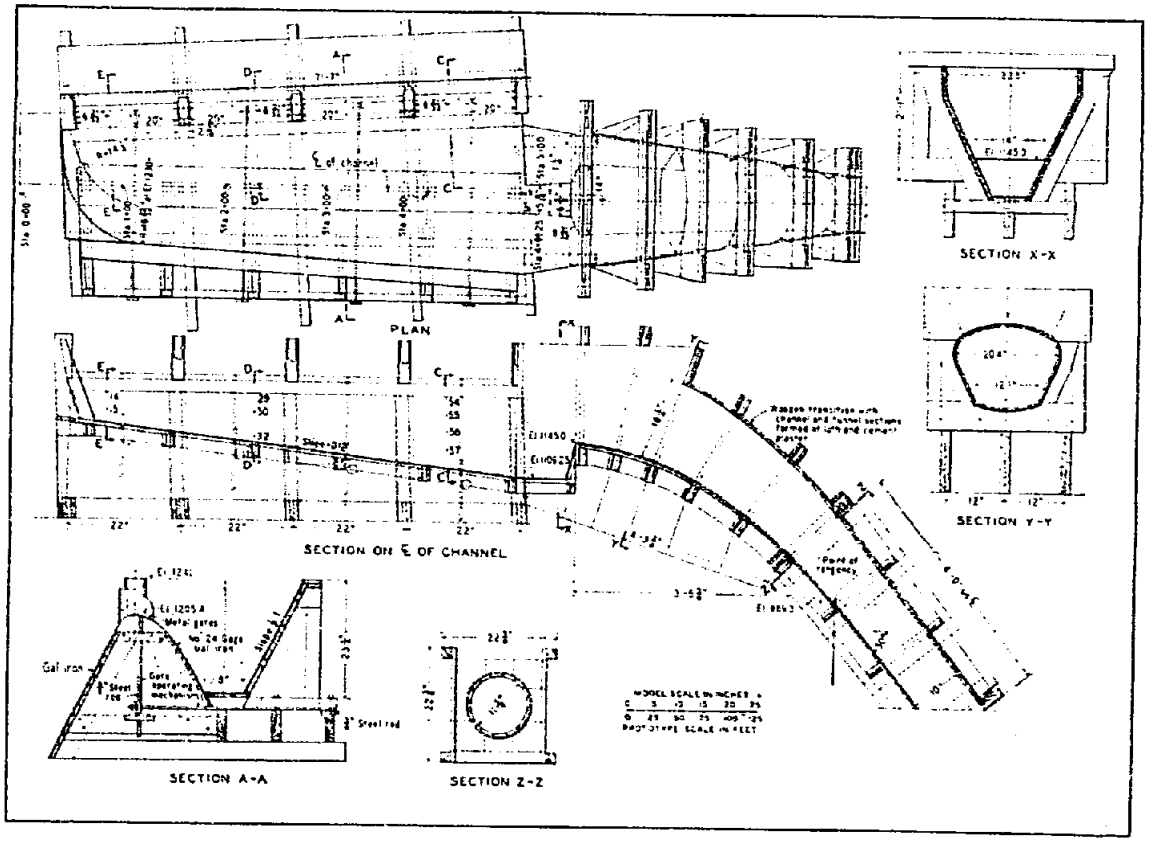
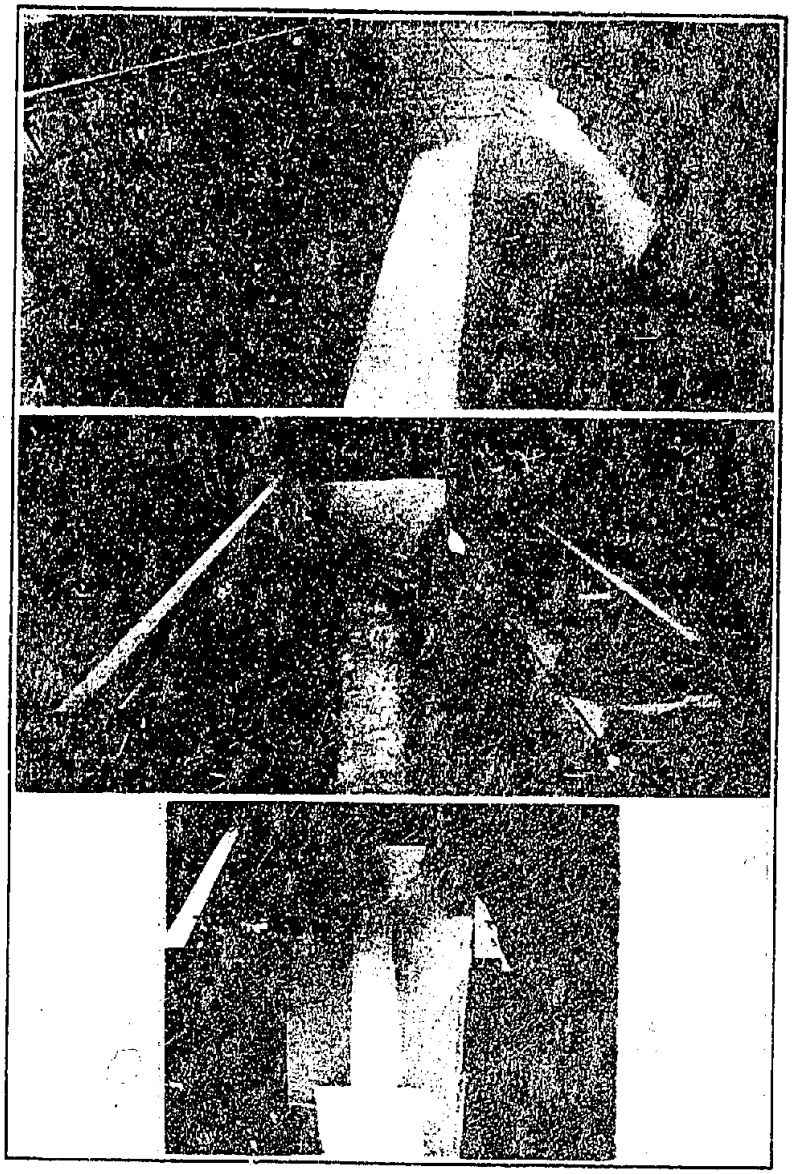


FIGURE 83—SIDE-CHANNEL SPILLWAY, MODEL C-5, SCALE RATIO 1:60



A. Scale Ratio 1:20
B. Scale Ratio 1:60
C. Scale Ratio 1:100

FIGURE 84—COMPARISON OF SPILLWAY MODELS, FINAL DESIGN

rack and pinions from the outside of the model. This arrangement greatly facilitated the experiments for determining flow effects caused by gates in various positions. The topography in front of the model was constructed of lean cinder concrete.

The 1:100 model was also constructed of galvanized iron with a wooden frame. There were no gates on this model, but the crest was built to represent the shape with the drum gates down. Views of the three models are shown on figure 84.

SIMILARITY CONDITIONS

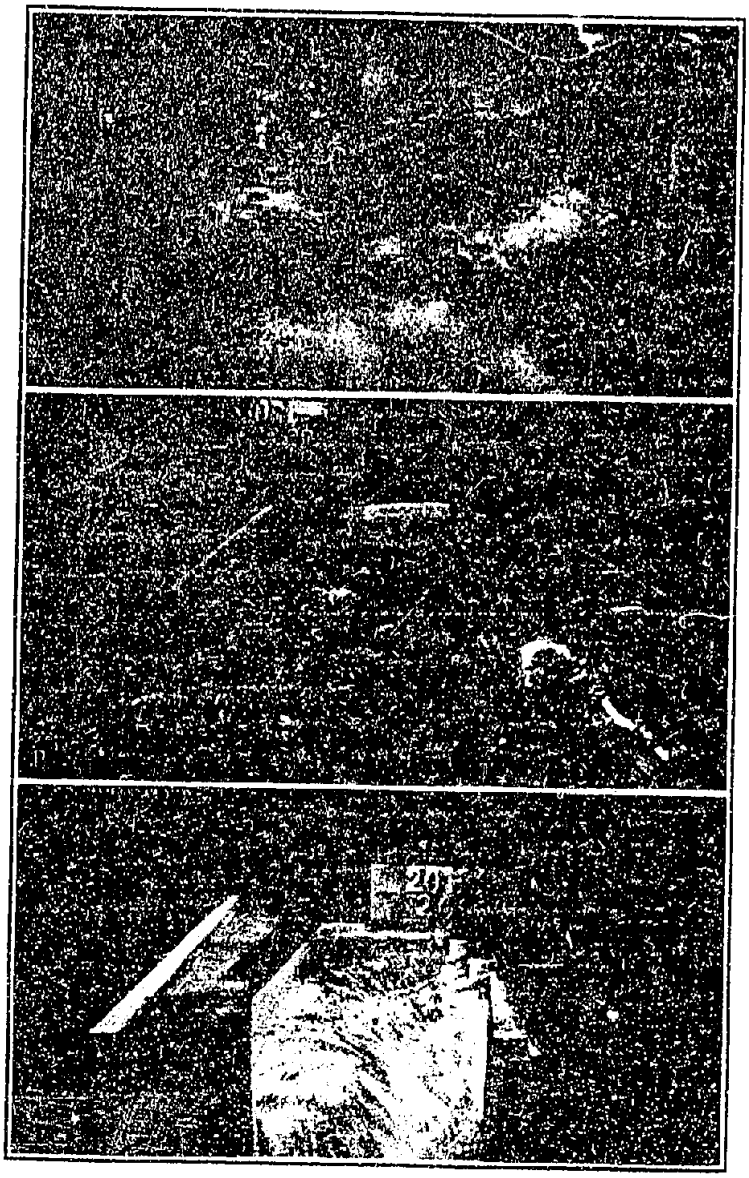
64. Visual Results, M-5, C-5, and C-6 Models.—Actions of the three models for all conditions were found to be surprisingly similar. This is shown by the photographs in figures 85 to 89, inclusive. Figure 84 shows the three models with no flow. Figure 85 shows the three models with discharges corresponding to 200,000 second-feet. A close correspondence in general features is noted, and in most cases extending even to details of wave action.

Flow conditions in the channel for discharges of 160,000 second-feet and over followed a cycle in which certain forms of flow appeared at intervals. Unless photographs were taken at the same state in the cycle, they would appear to indicate different flow conditions, even though the entire cycle was exactly the same. For this reason the similarity of flow may have been even better than indicated by the photographs.

In all three models, the surface roughness increased with the size of the model, due to the relative reduction in surface tension and the increase in air friction. Figure 86 shows conditions in the three models for a flow of 120,000 second-feet with all gates down. Figure 87 shows the action for a discharge of 40,000 second-feet. Here again the close similarity, even in wave forms, is noticeable.

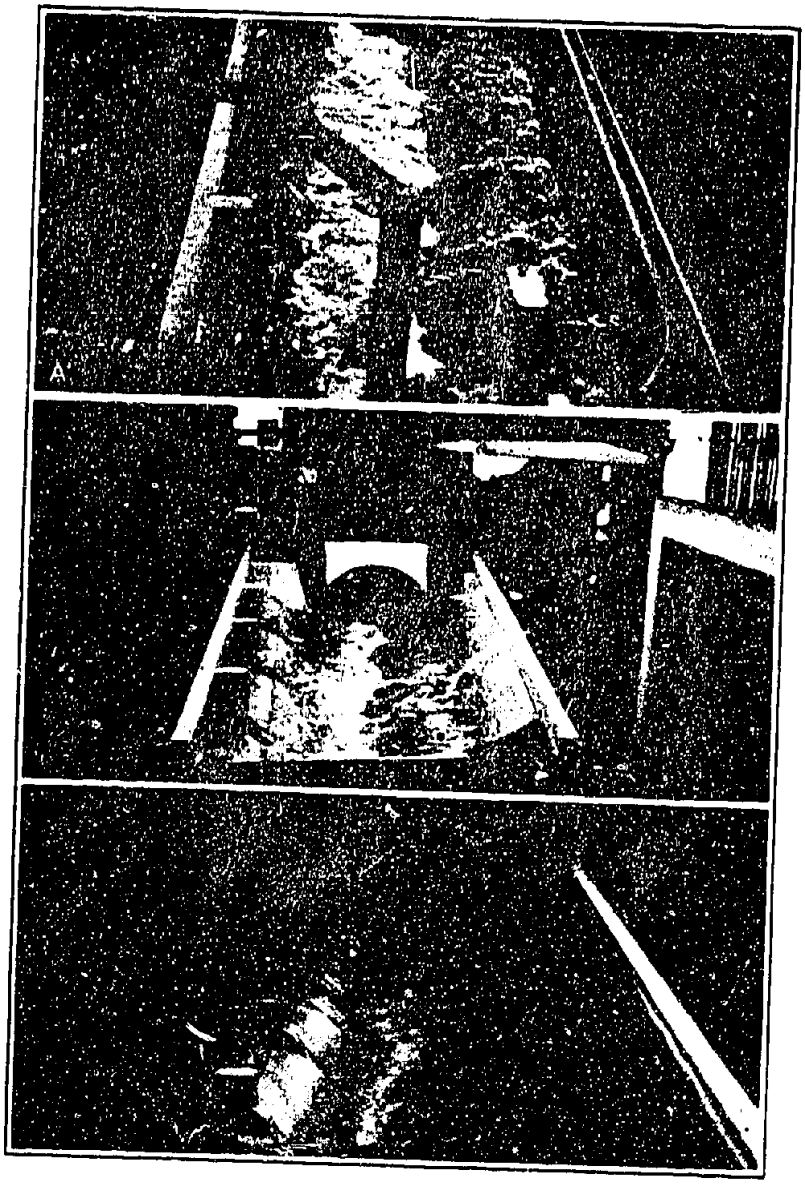
Flow conditions with the drum gates in raised positions for the M-5 and C-5 models show the same close correspondence. Figure 88 shows a comparison for flows of 40,000 and 80,000 second-feet with the gates raised one-half. A similar comparison is shown on figure 89 for the same discharges with the gates completely raised.

A detailed study of the results of comparable runs on the three models, as shown by the photographs, demonstrates the close similarity of the action of the models, the smallest of which was one-fifth the size of the largest. In addition, whenever it has been possi-



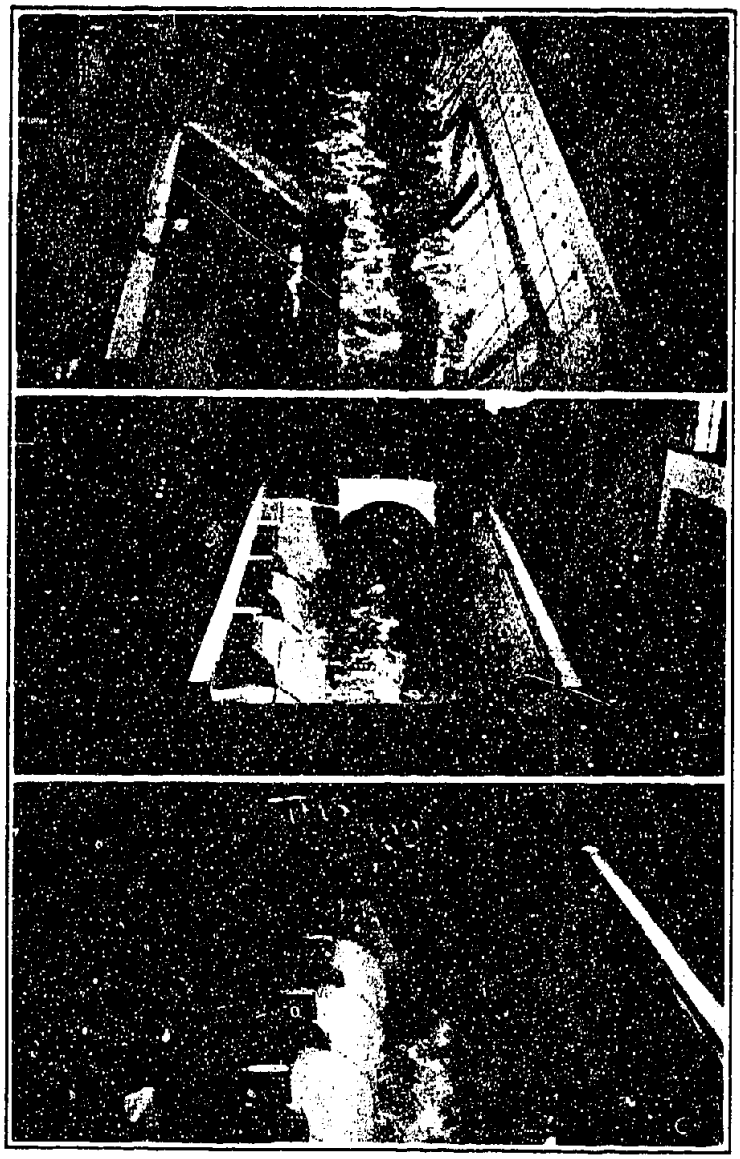
A. Scale Ratio 1:20
B. Scale Ratio 1:60
C. Scale Ratio 1:100

FIGURE 85—FINAL DESIGN WITH DISCHARGE OF 200,000 SECOND-FEET



A. Scale Ratio 1:20
B. Scale Ratio 1:60
C. Scale Ratio 1:100

FIGURE 86—FINAL DESIGN WITH DISCHARGE OF 120,000 SECOND-FEET



A. Scale Ratio 1:20
B. Scale Ratio 1:60
C. Scale Ratio 1:100

FIGURE 87—FINAL DESIGN WITH DISCHARGE OF 40,000 SECOND-FEET

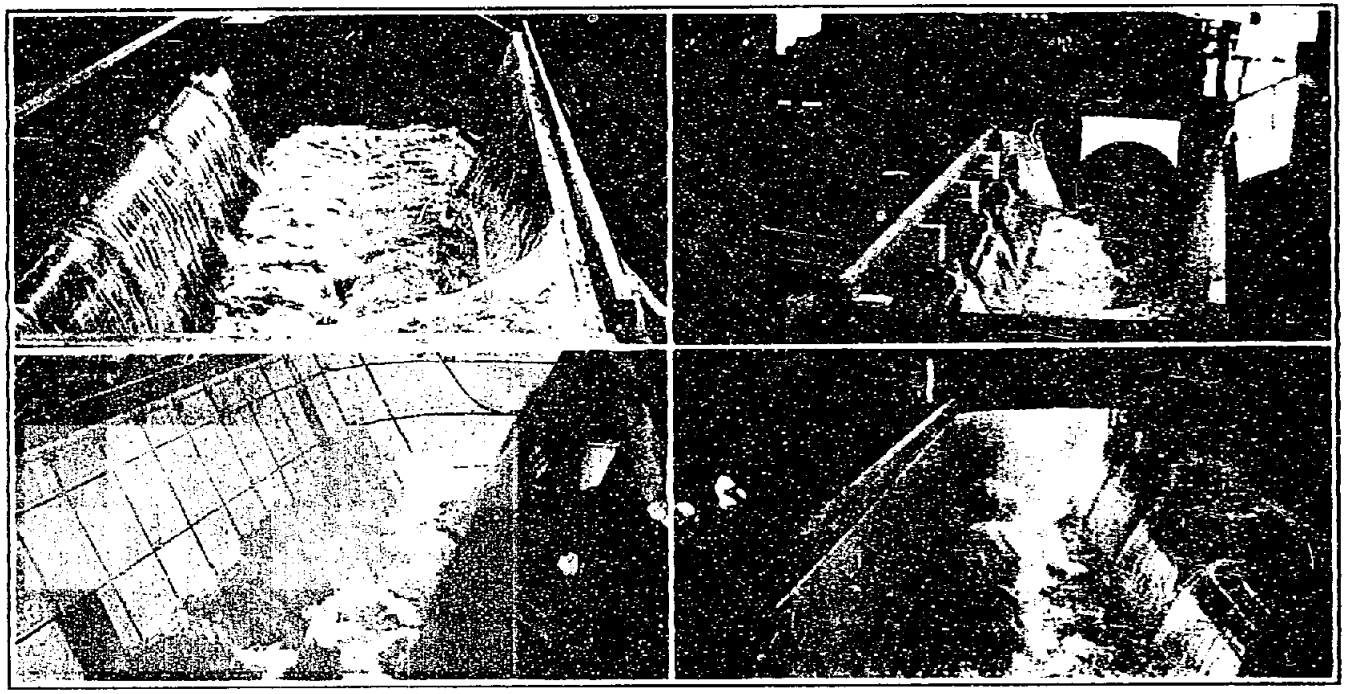
A. Scale Ratio 1:20 B. Scale Ratio 1:60

Discharge 80,000 Second-feet

C. Scale Ratio 1:20 D. Scale Ratio 1:60

Discharge 40,000 Second-feet

FIGURE 88—FINAL DESIGN WITH GATES RAISED TO HALF-POSITION



A. Scale Ratio 1:20 B. Scale Ratio 1:60

Gates Raised Three-fourths

C. Scale Ratio 1:20 D. Scale Ratio 1:60

Gates Completely Raised

FIGURE 89—FINAL DESIGN WITH DISCHARGE OF 40,000 SECOND-FEET

ble to obtain comparisons between models and their prototypes, a close similarity has been found. This is believed to warrant the conclusion that the spillways of Boulder Dam will perform in a manner very similar to that indicated by the model tests; also that the volumes of water involved will be safely handled. A further confirmation is obtained from quantitative comparisons of the results obtained on the spillway models.

65. Quantitative Results, M-5, C-5, and C-6 Models.—To further establish the reliability of predicting the action in prototype structures from the results of model tests, extensive comparisons were made of the quantitative results obtained on the 1:20, 1:60, and 1:100 scale models of the final spillway design. In general, the results were very satisfactory and they materially confirm the reliability of model tests. The results are shown in detail on figures 90 to 101, inclusive.

Comparisons of water surfaces on the three models for flows with the crest gates down are shown on figures 90 and 91. Longitudinal profiles of comparable conditions on the 1:20 and 1:60 models with crest gates raised to various heights are shown on figures 92 and 94. Cross sections of the water surfaces in the channel for the same conditions are shown on figures 93, 95, and 96. The 1:100 model was without movable gates, and therefore was not included in this comparison.

Comparisons of the water surfaces for the same discharge with gates at various elevations are shown for each model, on figure 97. As previously explained, the agreement of the water surfaces indicates the reliability of the assumption usually made in side-channel spillway design: namely, that the entire energy of the overfalling water is dissipated without causing longitudinal flow in the spillway channel.

Pressures acting on the overflow face, gates, and in the side channel, obtained from piezometric observations on the 1:20 and 1:60 models, are shown on figures 98 to 101, inclusive. Considering the difficulties of obtaining consistent piezometric readings where high velocities and impacts are involved, the results are very consistent. In general, they show the same pressure concentrations and reductions as the observations previously described on the M-3 model.

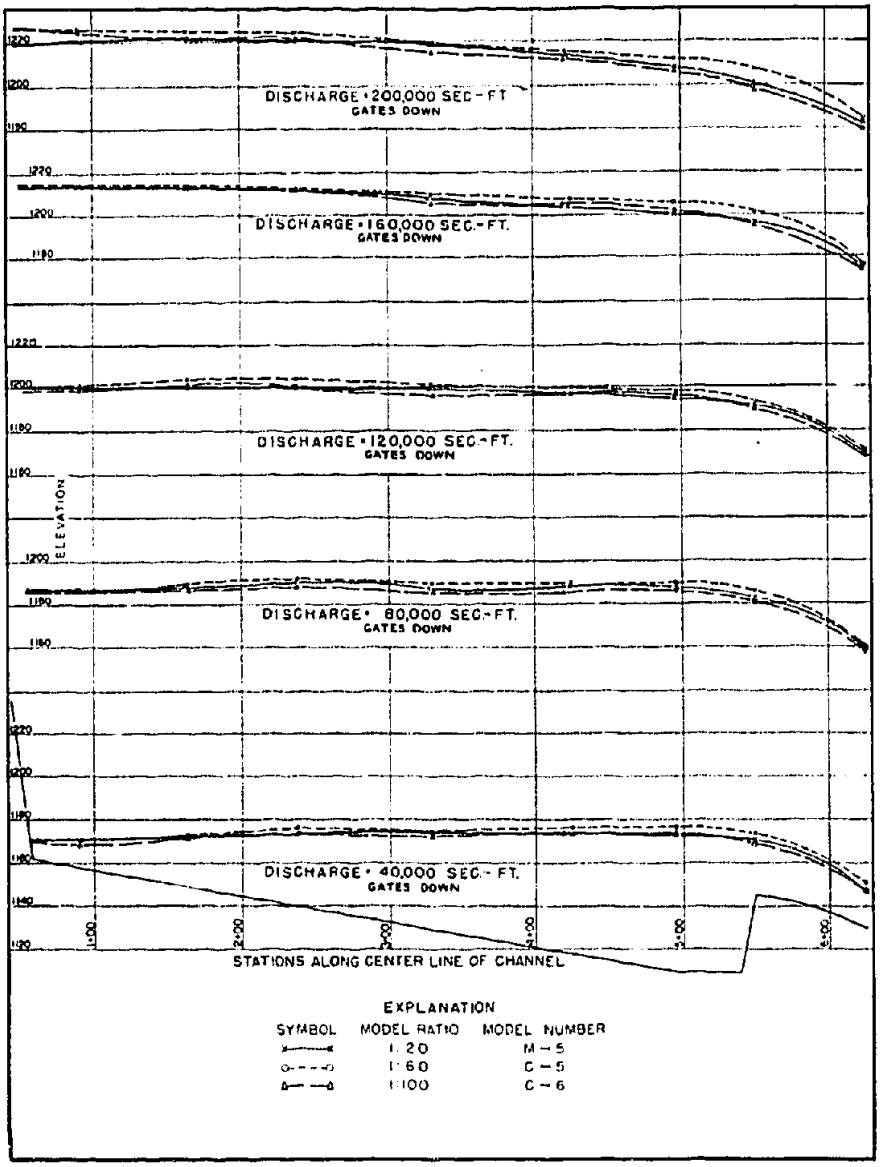


FIGURE 90—WATER SURFACE WITH DRUM GATES DOWN

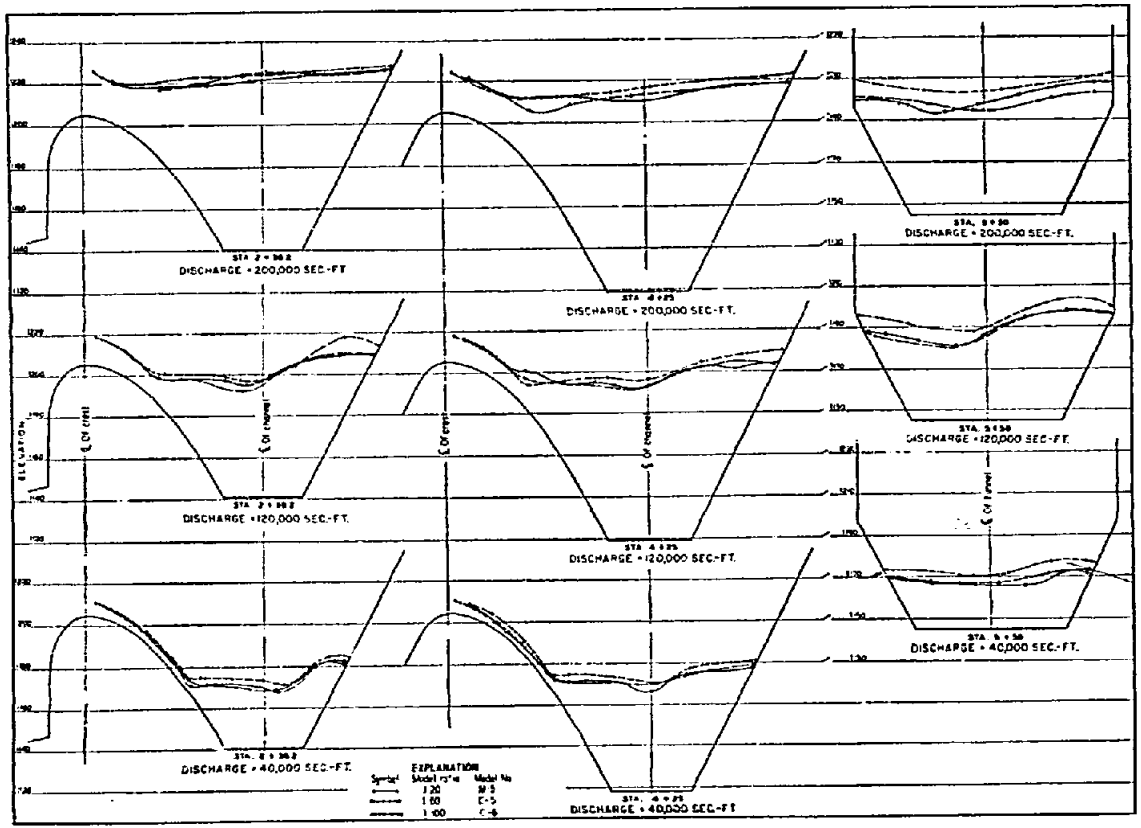


FIGURE 91—WATER SURFACE WITH DRUM GATES DOWN

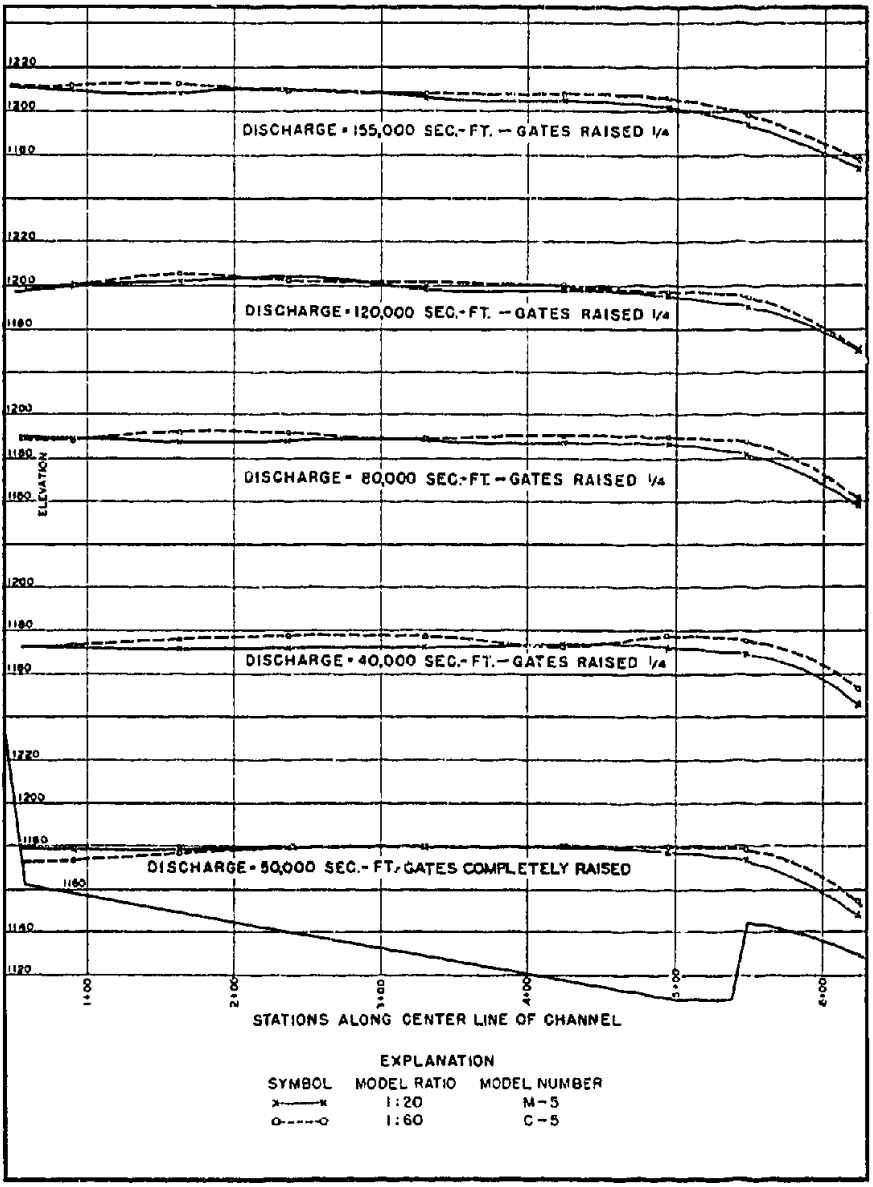


FIGURE 92—WATER SURFACES WITH DRUM GATES RAISED ONE-FOURTH AND FULL HEIGHT

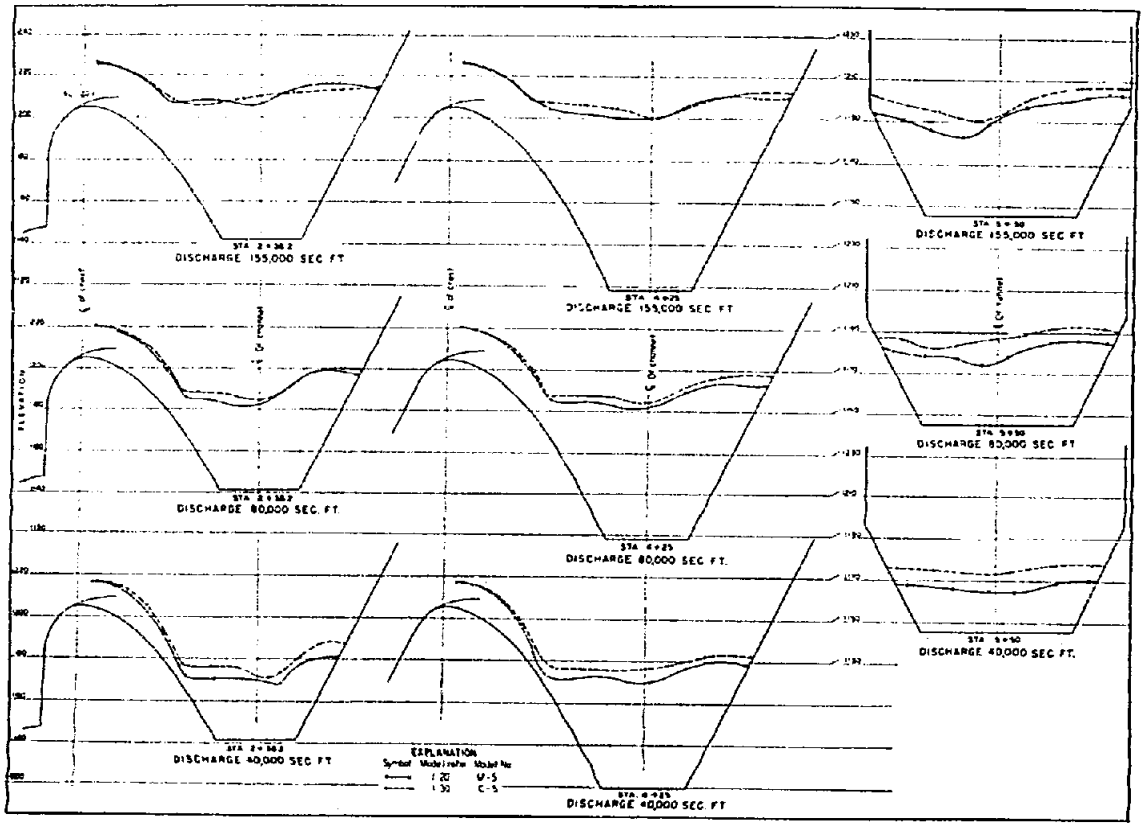


FIGURE 93—WATER SURFACES WITH DRUM GATES RAISED ONE-FOURTH

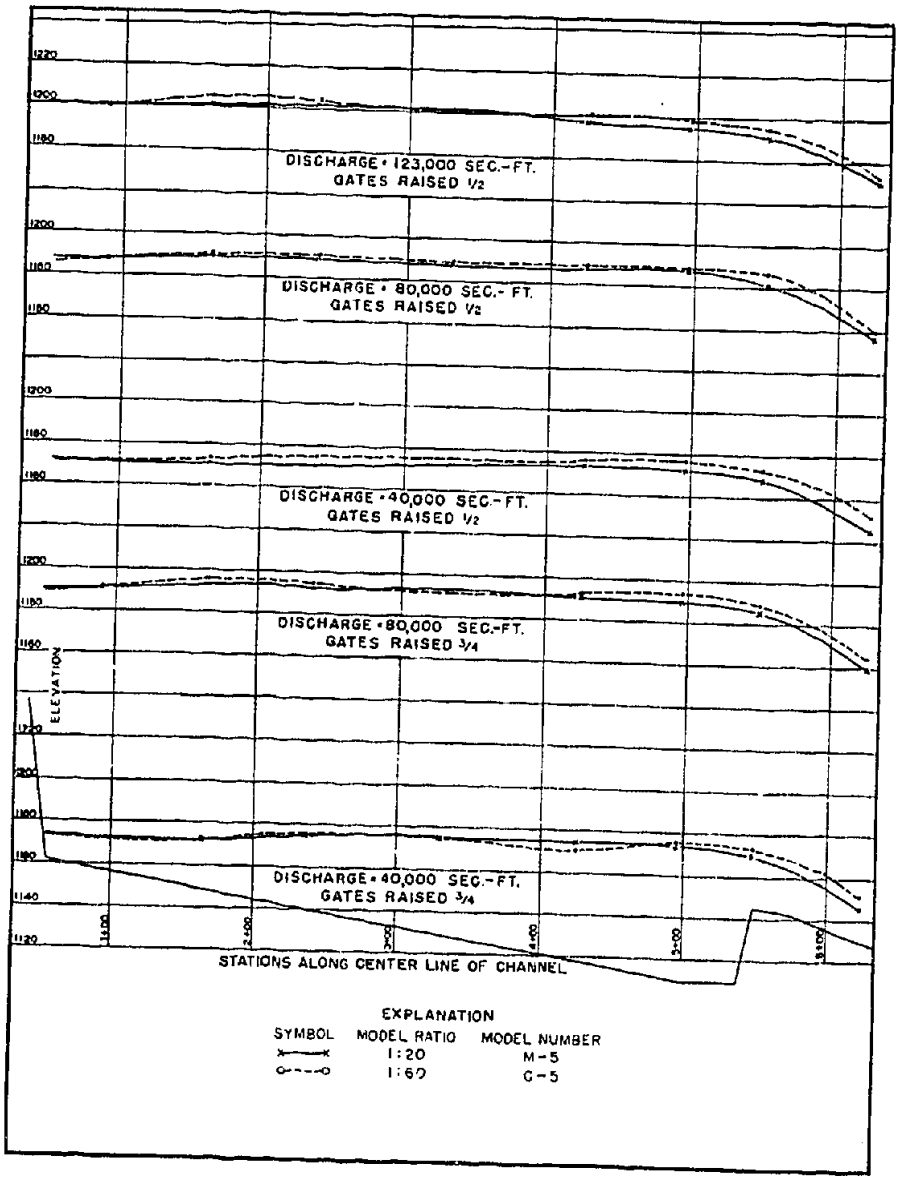


FIGURE 94—WATER SURFACES WITH DRUM GATES RAISED ONE-HALF AND THREE-FOURTHS

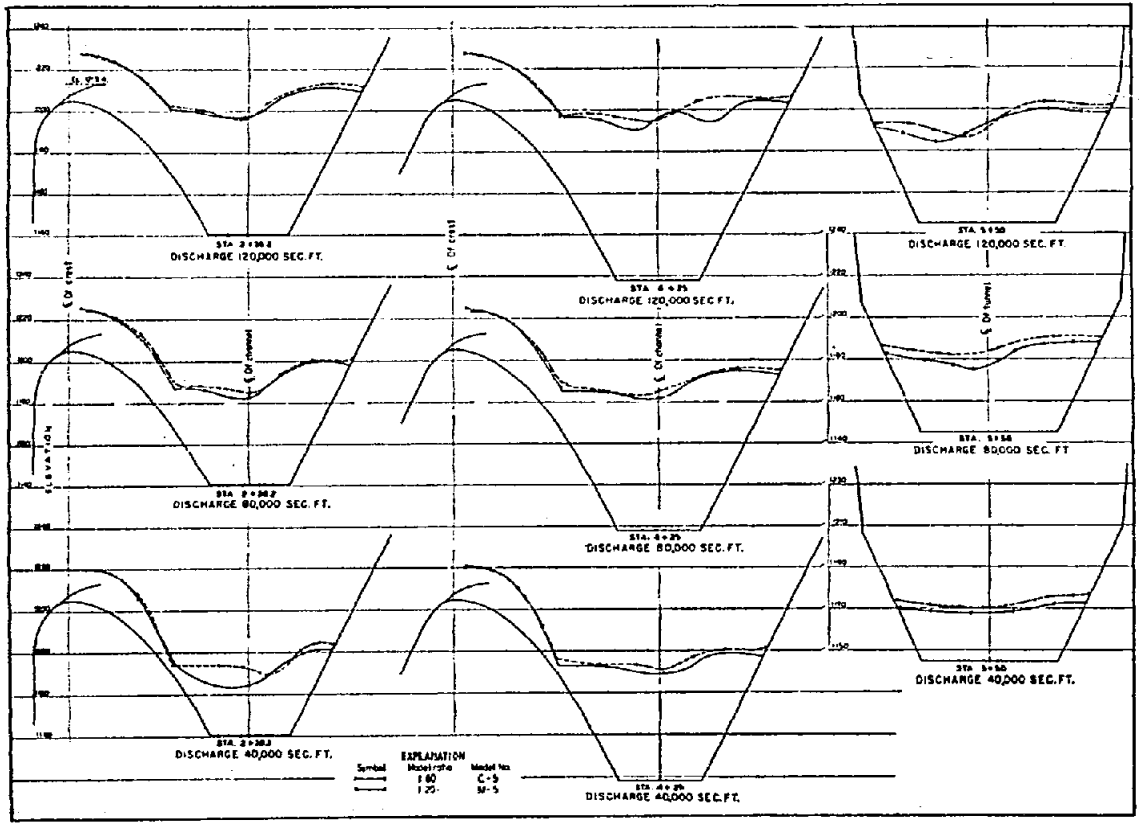


FIGURE 95—WATER SURFACES WITH DRUM GATES RAISED ONE-HALF

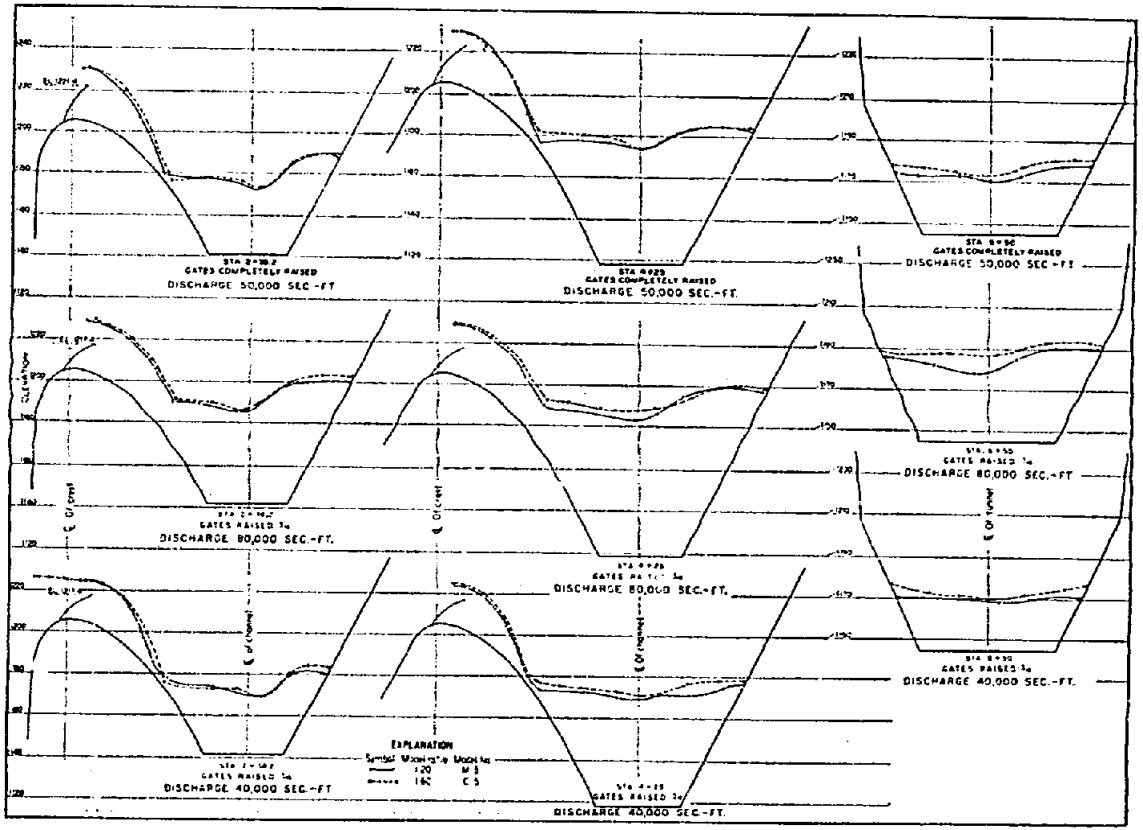


FIGURE 96—WATER SURFACES WITH DRUM GATES RAISED THREE-FOURTHS AND FULL HEIGHT

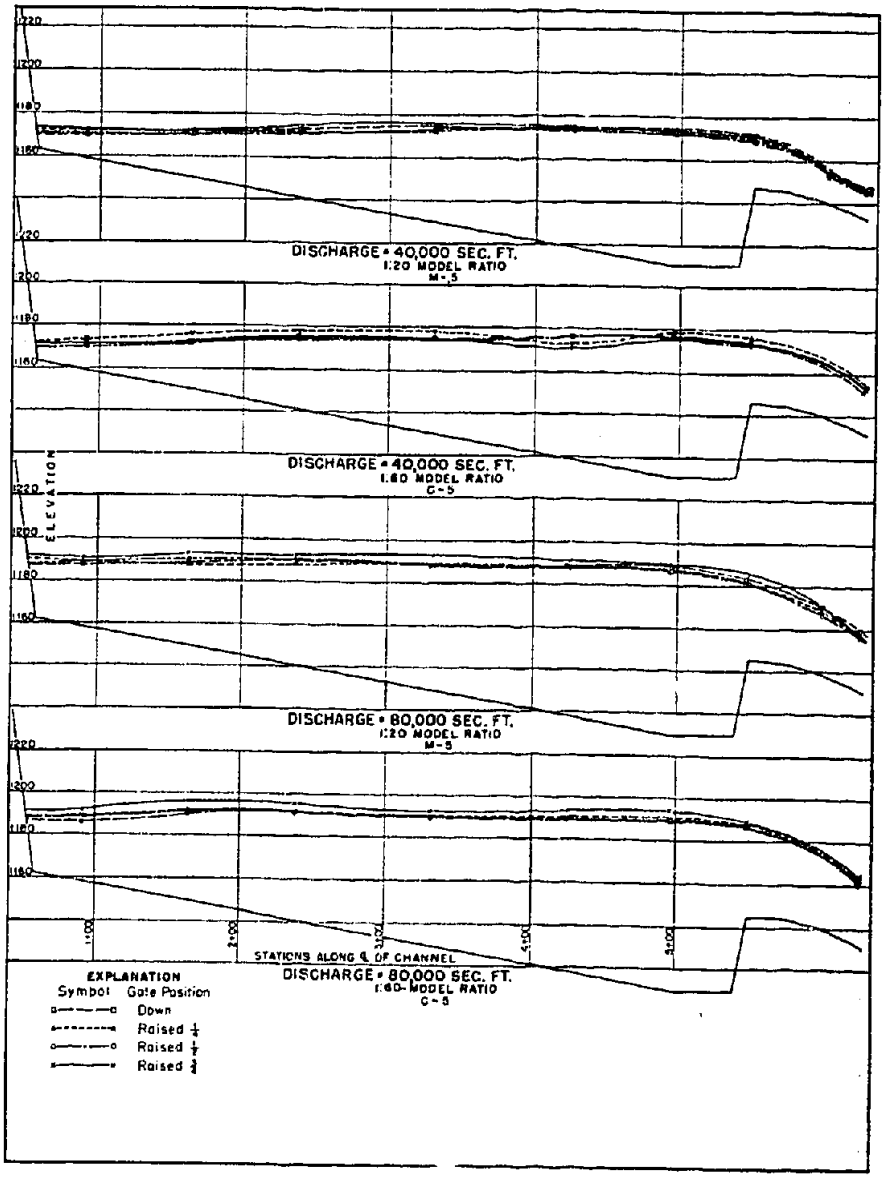


FIGURE 97—WATER SURFACE PROFILES WITH DRUM GATES AT DIFFERENT ELEVATIONS

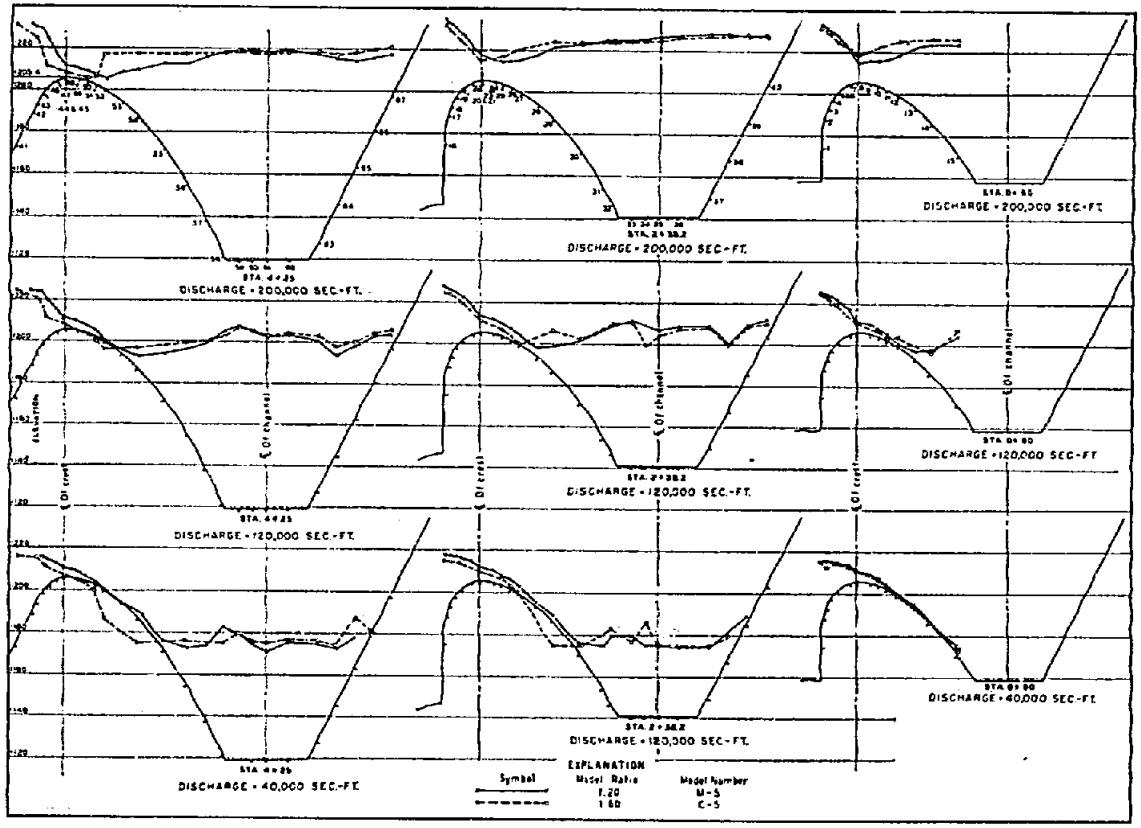


FIGURE 98—PIEZOMETRIC PRESSURES WITH DRUM GATES DOWN

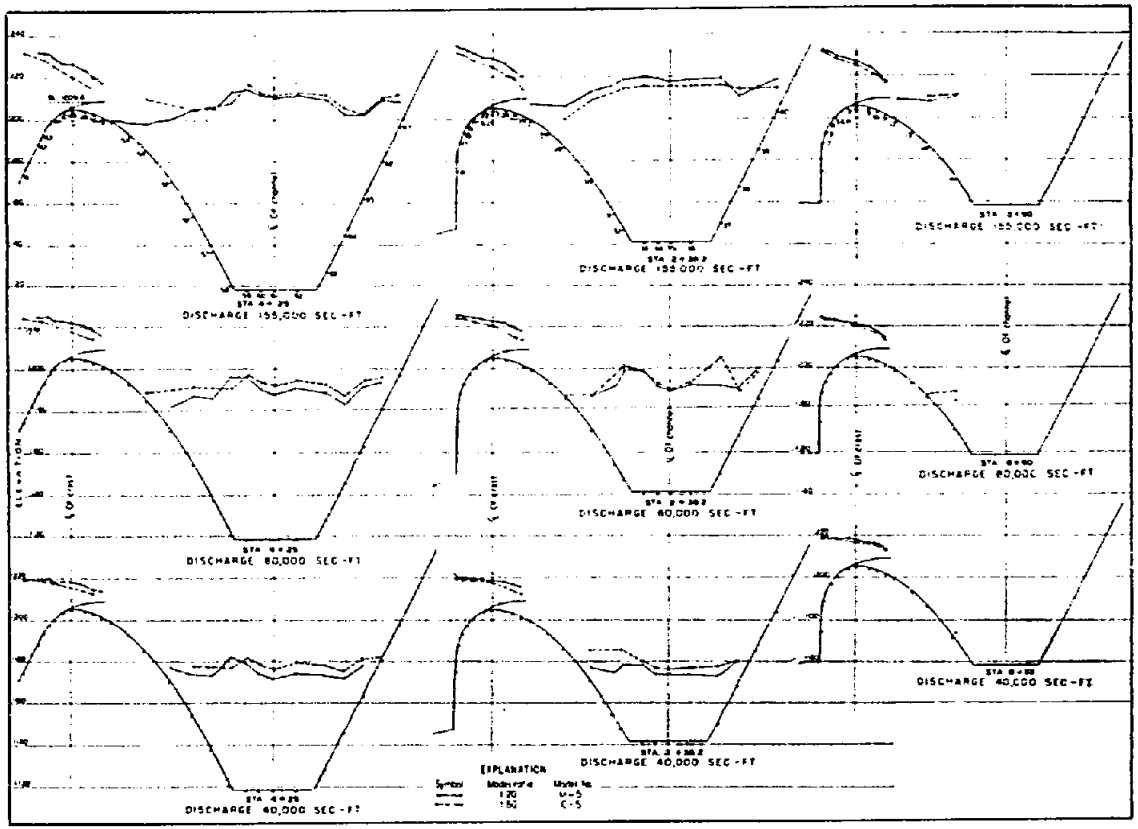


FIGURE 99—PIEZOMETRIC PRESSURES WITH DRUM GATES RAISED ONE-FOURTH

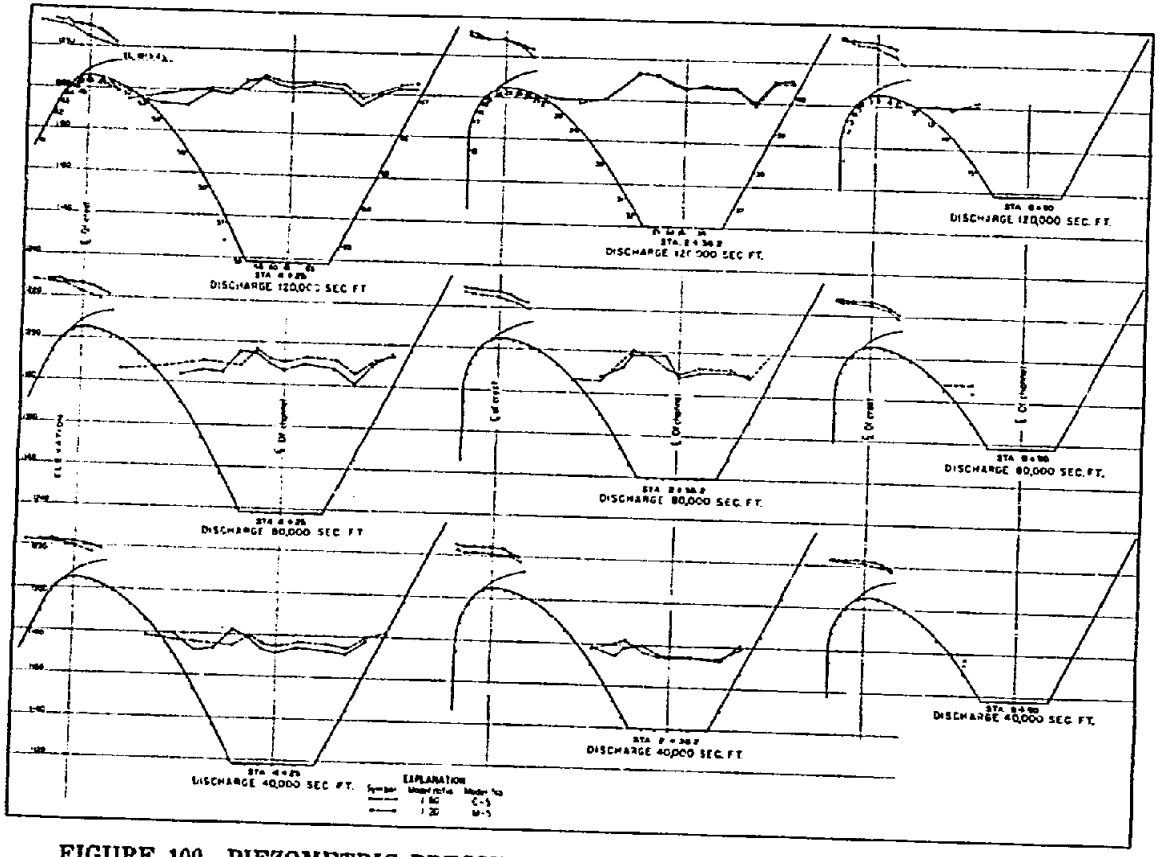


FIGURE 100—PIEZOMETRIC PRESSURES WITH DRUM GATES RAISED ONE-HALF

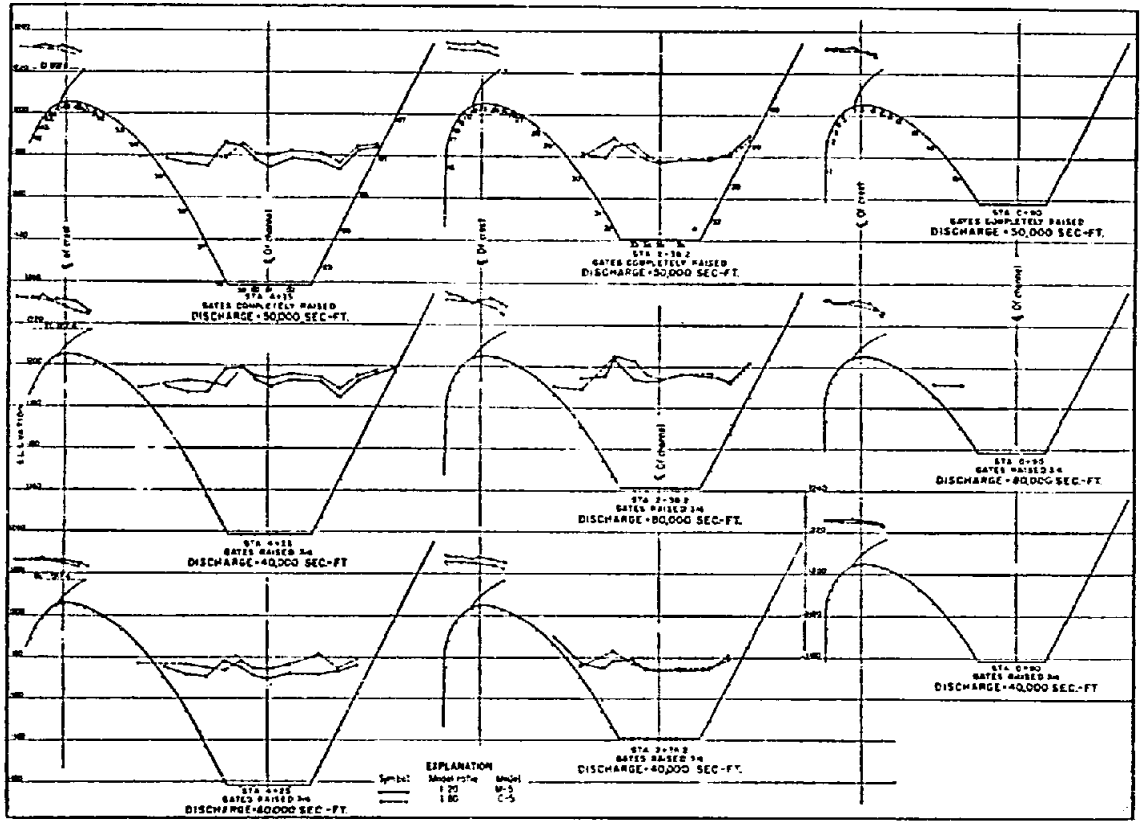


FIGURE 101—PIEZOMETRIC PRESSURES WITH DRUM GATES RAISED THREE-FOURTHS AND FULL HEIGHT

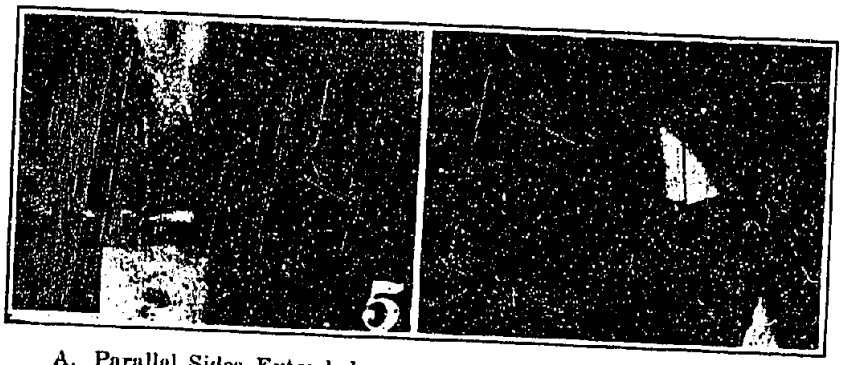
A. Gates Down, Discharge 200,000 Second-Feet

B. Gates Raised Three-Fourths, Discharge 90,000 Second-Feet

FIGURE 102—CONDITION AT DOWNSTREAM END OF GATE PIERS, ORIGINAL DESIGN OF C-5 MODEL

MISCELLANEOUS TESTS

66. Shape of Downstream End of Crest Piers.—One of the valuable results obtained from the model tests was the check on the performance of structural details. When the M-5 and C-5 models of the spillway were first tested, it was discovered that a partial vacuum formed under the nappe of water flowing over the gates when the gates were partially raised. The water flowed around the piers in such a manner that no openings existed at the downstream end through which air could flow to the space



A. Parallel Sides Extended 1.5 Feet Downstream

B. Parallel Sides Extended 2.0 Feet Downstream

FIGURE 103—CONDITION AT DOWNSTREAM ENDS OF PIERS WITH PARALLEL SIDES EXTENDED, CYLINDRICAL RADIUS DECREASED

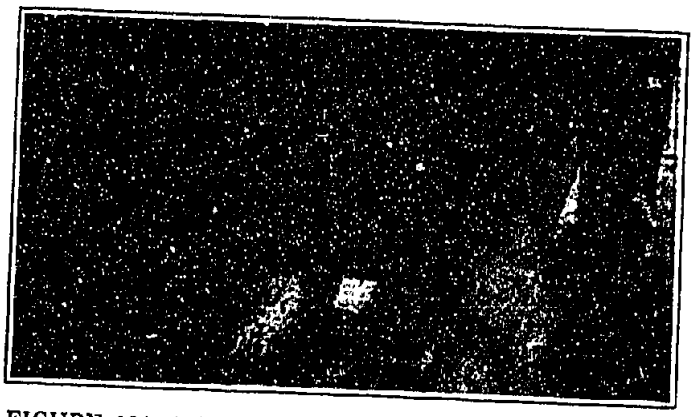


FIGURE 104—BAFFLE AT UPPER END OF CHANNEL

beneath the nappe. Instead, air was continually being exhausted from beneath the nappe. Figure 102-A shows the condition with the gates completely down; and figure 102-B, the flow at the downstream end of a pier with the gates in a three-quarter raised position. The water at the ends of the gate spread rapidly in a lateral direction and maintained constant contact with the pier faces. To break this contact at the tail end of the piers so as to form open spaces through which aeration of the nappe could continue, the parallel sides of the piers were extended downstream and the radius of the cylindrical portion at the downstream end was reduced. This resulted in an offset at the downstream edge of the parallel sides of the pier. Several combinations of radii and extension of parallel sides were tested.

Figure 103-A shows an extension of the parallel sides of one pier 1.5 feet farther downstream than in the original design, with 1.0-foot decrease in the radius, thus producing a 1.0-foot offset on each side of the pier. A space is visible between the edges of the nappes from the adjacent two gates and the faces of the pier through which continuous aeration was possible. Figure 103-B shows the same pier with a 2.0-foot extension downstream. It was desirable to reduce the extension to a minimum in order to economize on the volume of concrete in the piers. An extension of 1.33 feet and a decrease of pier radius of 1.0 foot were finally adopted.

67. Baffle at Upper End of Channel.—With a low discharge and the gates at the same elevation, an undesirable wave formed at the upper end of the channel, due to the curved wall. As a remedy

for this condition, a form of baffle was tested in the 1:20 scale model. It consisted of two steps, as shown in figure 104. The baffle, however, did not remove the wave. As the discharge increased, the baffle became more effective; but even without the baffle the wave action tended to decrease as the discharge increased. Hence little improvement was effected by the baffle at any discharge. Slightly better results were obtained by removing the lower of the two steps and placing an overhanging coping on the higher step. The wave can be largely eliminated at all except low discharges by the proper manipulation of the gates as described in section 68.

68. **Gate Operating Program.**—An extensive study was made on the C-5 model to determine a schedule for operating the gates during flood periods. Most of the previous experiments had been carried on with the gates at the same elevation, although on the M-3 model some studies were made with the gates at various elevations. In all, 75 different operating combinations were tried. These combinations were divided into five groups, according to the operations of the gates as the flow increased, which were as follows: (1) all gates moving at the same rate and elevation; (2) each gate lowered entirely before the next gate started; (3) each gate lowered five feet before the next successive gate started, after which they all move down at the same rate; (4) each gate lowered 10 feet before the next successive gate started, after which they all move down at the same rate; and (5) miscellaneous orders.

Of the seventy-five combinations, three were nearly equally desirable and were studied in considerable detail. In program A, gates 1 and 4, numbering from the upstream end of the channel, were lowered equally a distance of 5 feet, at which time the lowering of the two middle gates was started, and all four gates were lowered at the same rate until they reached the fully lowered position. In program B, the upstream gate was lowered 5 feet, at which point the lowering of the remaining three gates was started and all four gates were lowered at the same rate. In program C, all gates were lowered at the same time and rate. Water surfaces in the channel are shown quantitatively on figures 105 and 106 for the three operating programs. The action of the spillway for the three programs was equally desirable and it was difficult to decide which was best. Program C, in which all gates move to-

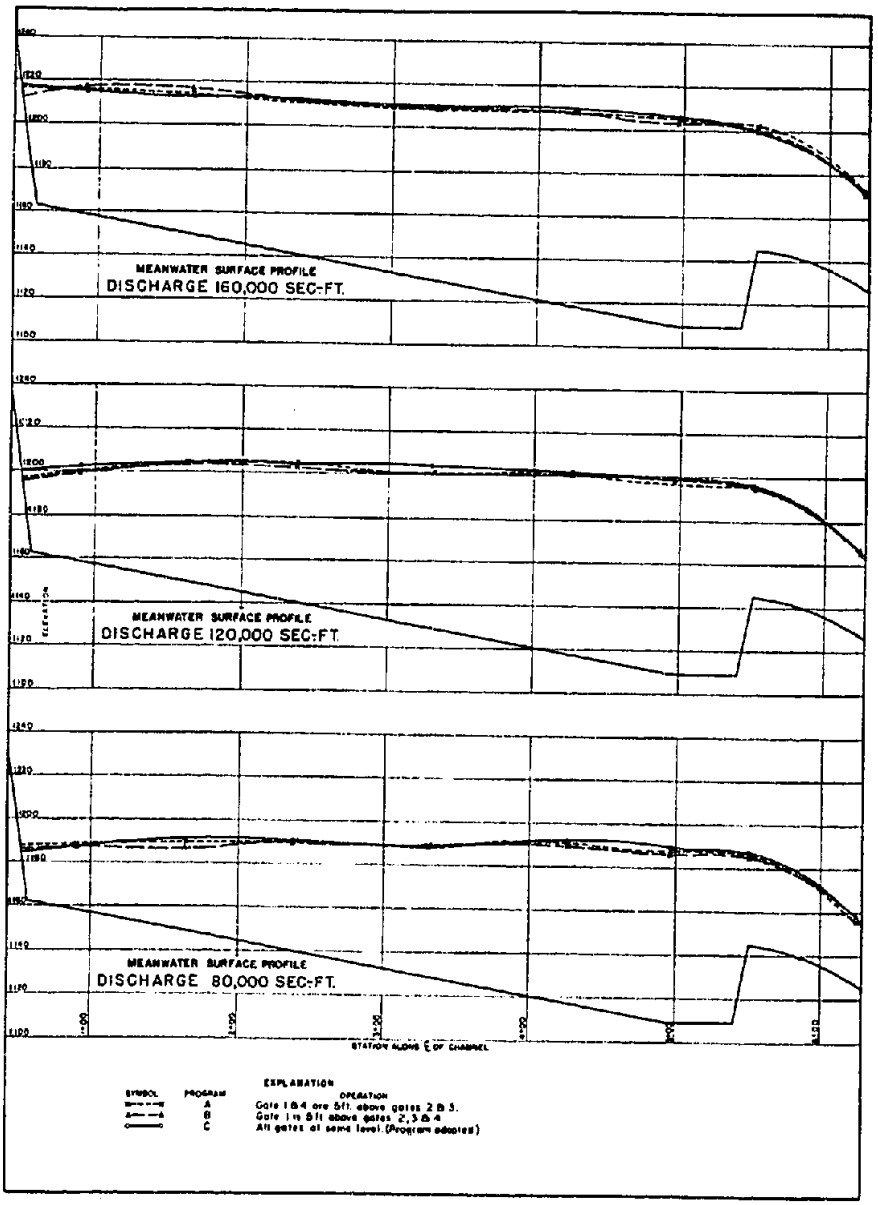


FIGURE 105—COMPARISON OF GATE OPERATING PROGRAMS, MODEL C-5

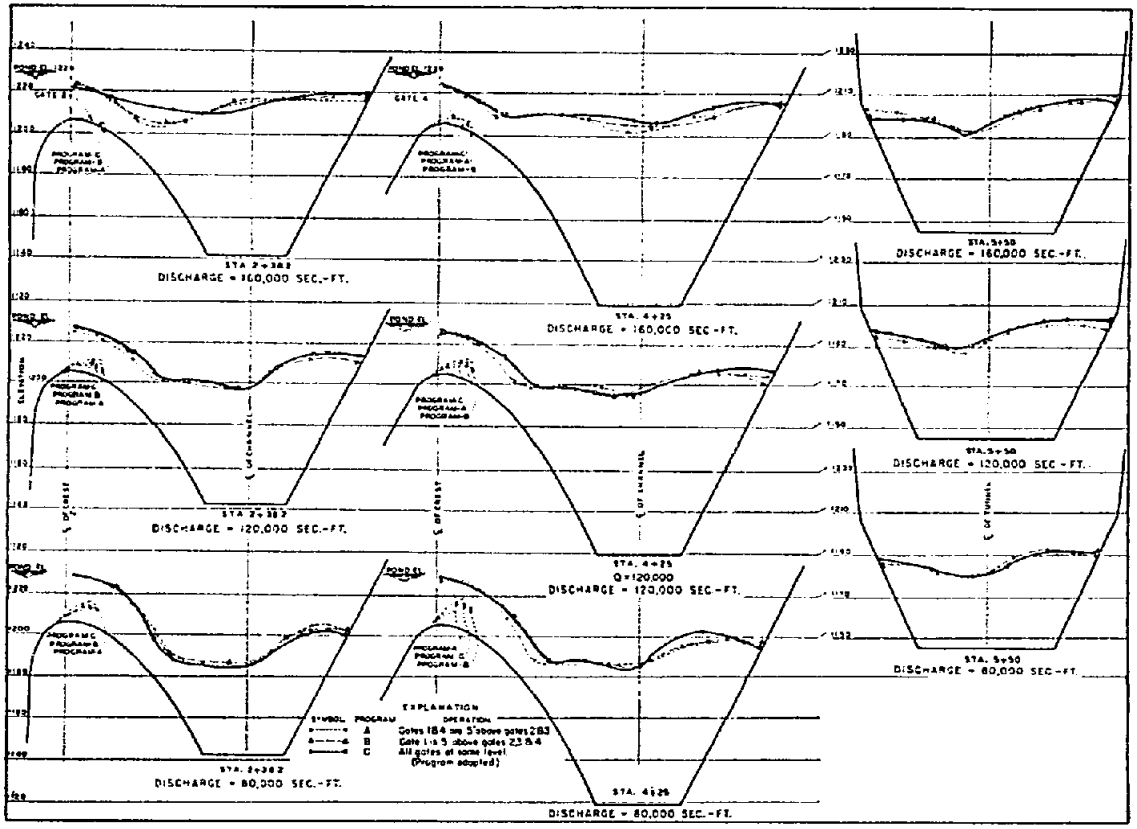
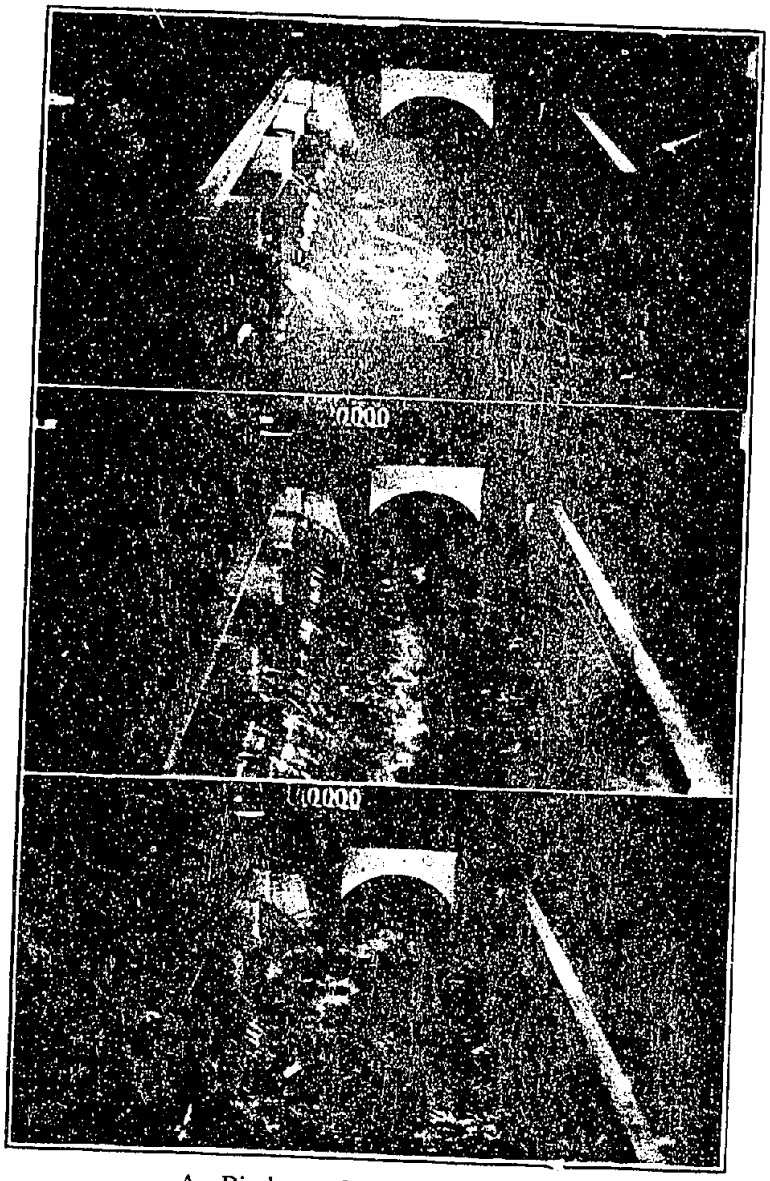


FIGURE 106—COMPARISON OF GATE OPERATING PROGRAMS, MODEL C-5



A. Discharge 80,000 Second-Feet
B. Discharge 120,000 Second-Feet
C. Discharge 160,000 Second-Feet

FIGURE 107—ADOPTED GATE OPERATING PROGRAM, ALL GATES AT SAME ELEVATION

gether, was finally selected, largely because of simplicity. Flow conditions in the channel for three discharges, with three positions of the gates, operating according to program C, are shown on figure 107. This is the arrangement recommended for use when it is necessary to discharge water over the spillways of the Boulder Dam.

CHAPTER V—FLOW IN SPILLWAY TUNNELS

TRANSITION SECTION

69. General.—The portion of the side-channel spillways in which the change was made from the trapezoidal section of the side channel to the circular section of the tunnel was a transition in which the flow changed from slightly inclined to steeply inclined. Since water entered the transition section with an initial velocity in a slightly inclined direction, the change to a steeply inclined direction could not be made suddenly without danger of the water springing free from the invert. Intermittent partial vacuums between the water stream and the invert might cause hammering or pulsations. This situation resembles flow over an ogee dam. Cavitation effects of such action in a tunnel might be more severe than on a dam, however, as it occurs in a partially enclosed space.

The slopes of the transition bottoms were designed somewhat flatter than the trajectory of a freely falling stream having the initial velocity and direction of the water entering the section. The action of the water in this case was complicated by the contraction of the stream in a horizontal direction. Piezometers were installed along the bottoms of the transitions to study vacuum effects.

70. Piezometer Measurements.—Figure 21 shows elevations of water surfaces in open tubes connected to piezometers in the transition of the Stoney gate side-channel model. The heavy line shows the elevation of the transition invert grade. Heights of the piezometric levels above the invert grade indicate pressures of water on the transition bottom. Piezometric heights below the invert grade indicate partial vacuums. Figure 21 shows a slight vacuum at the lower end of the transition. The results at piezometer 37 are probably high, due to a defect in the piezometer opening. These observations were taken with the entire discharge passing through the Stoney gate. With a portion of the flow passing over the spillway crest, which is the normal operating condition, the initial velocity entering the transition would be less. This would result in a steeper trajectory, and a reduction in negative pressures.

Pressures on the transition bottom of the semifinal 1:20 model, M-3, as indicated by piezometers along the center line, are shown in figure 108. The upper set shows results for various discharges in the original design; the middle set shows results for various discharges with a dentated baffle installed in the spillway channel; and the lower set shows effects produced on water surfaces in the original design by setting the gates at different elevations during a constant discharge of 80,000 second-feet. As the results show that under all conditions positive pressures exist on the transition bottom, vacuum effects need not be expected in the prototype. The high pressures on the sloping face of the cross weir were caused by the velocity of the water approaching the transition. When dentated baffles were used they interfered with the piezometers at this point. These pressures are therefore not shown for the middle set of results.

The results in figure 108 show that the pressures depended upon the flow and were practically independent of the channel or weir-crest conditions, as indicated by the similarity of results obtained with and without the dentated baffle, and the close agreement of the results for an 80,000 second-foot discharge with the drum gates at various elevations.

Figure 109 shows water surfaces and piezometric pressures observed in the transition of the 1:20 final design model, M-5, with the gates down. Piezometric lines are considerably below their corresponding mean water surfaces since the water was accelerating under the action of gravity and did not exert so much pressure as it would under static conditions. In all cases positive pressures existed on the transition bottom. Cross sections of the water surface normal to the flow are also shown in figure 109. Observations were made on water surfaces and piezometric pressures for a complete range of flows with the drum gates in different positions. The results agreed so closely with those obtained for the same discharge with the gates down that they are not included.

FLOW AT BENDS

71. Vertical Bend—1:60 Scale.—Conditions of flow in the 1:60 model of the vertical bend were satisfactory for all radii tested. There was, however, a tendency toward splashing near the center of the 150-foot radius bend which formed a ridge of detached par-

ticles that travelled a short distance down the horizontal pipe. A bend with a radius of 200 feet was then tested and was found to result in less pronounced splashing. Radii of 300, 468, and 582 feet, obtained by using a flexible rubber pipe, were then tried; but showed little improvement over the 200-foot radius.

Flow in the 1:20 model of the vertical bend having a radius of 150 feet was similar to that for the same radius on the 1:60

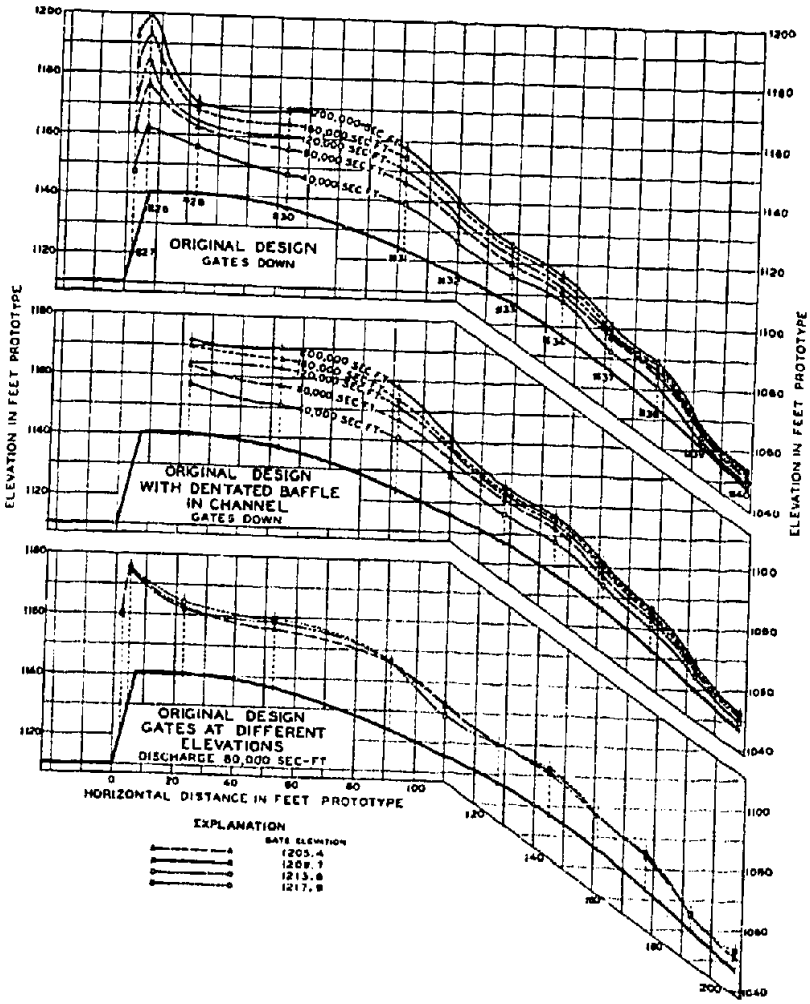


FIGURE 108—PRESSURES ON BOTTOM OF TRANSITION DRUM GATE MODEL M-3

model. The same tendency was present for splashing near the center of the bend. The installation of a 200-foot radius bend on this model reduced the splashing. As a result of the experiments on these two models, a 200-foot radius vertical bend was adopted in the final design. Unfortunately, it was not possible to obtain satisfactory photographs of flow in the vertical bends.

72. **Horizontal Bend.**—For the designed discharge of 200,000 second-feet, the theoretical velocity of flow in the horizontal tunnels is approximately 173 feet per second, representing an energy content of 10,560,000 horsepower in each tunnel. The most advantageous alinement for the spillway tunnel on the Nevada side required a horizontal bend with a central angle of 32 degrees 15 minutes. It would have been difficult to predict the action of the water in in these bends without constructing models. Laboratory space at the Colorado Agricultural College did not permit an exact duplication of the entire length of horizontal tunnel; therefore, the distance between the vertical and the horizontal bend was shortened in the 1:60 model.

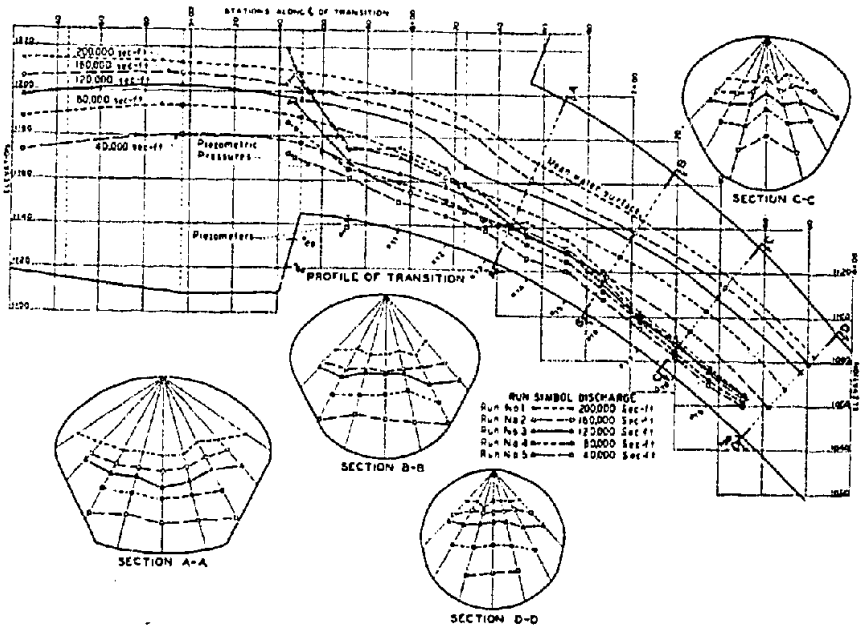


FIGURE 109—WATER SURFACE AND PIEZOMETRIC PRESSURE IN TRANSITION MODEL M-5

To determine the action of the water in the horizontal bend on the Nevada side, several models on a 1:60 scale were tested using different degrees of curvature. The first model represented the bend shown on the contract drawings. It had a central angle of 32 degrees 15 minutes and a radius of 819 feet, measured from the center line of the pipe. The top portion of the pipe was cut away so that the action of the water could be observed. The water climbed the outside wall at the bend, then fell toward the center of the pipe at the same time continuing its motion down the tunnel. The water falling toward the center, however, had a component of flow across the pipe which caused the water to also climb to a lesser extent on the inside wall of the bend. This second rise occurred in the straight section of pipe downstream from the bend. The action was much the same for all discharges.

In order to determine the direction of the currents in the bend, the inside of the pipe was coated with a layer of thick white lead

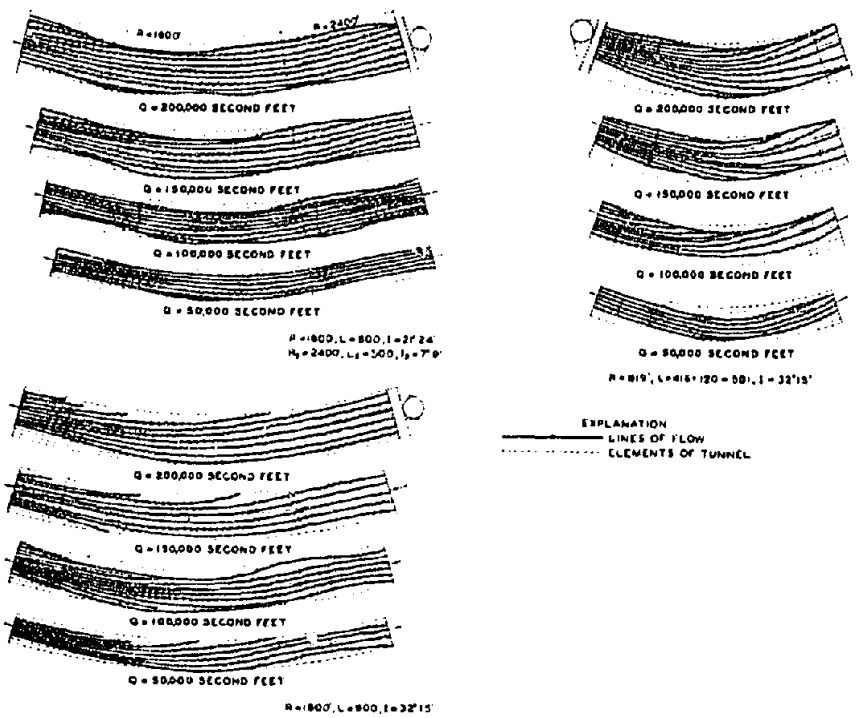


FIGURE 110—DIRECTION OF FLOW IN HORIZONTAL BEND OF SPILLWAY TUNNEL, MODEL SCALE RATIO 1:20

and water permitted to flow for a short time while the paint was wet. This produced streaks or flow lines on the inside surface as shown on figure 110. It was not possible to obtain satisfactory photographs of flow lines.

In an attempt to improve flow conditions in the bend, a model was constructed of wood, plastered on the inside with cement mortar, having a radius corresponding to 1,600 feet in the prototype. Flow lines for this bend are also recorded on figure 110. The action of the flow was not improved by the longer radius.

A model was then constructed with a long radius spiral downstream from the 1,600-foot radius curve; but no noticeable improvement was indicated by this layout, see figure 110. It was therefore decided to return to the 819-foot radius, and this was incorporated in the Nevada spillway tunnel model under construction on a 1:20 scale at the Montrose laboratory, see figure 29.

The nature of the flow in the 1:20 scale horizontal tunnel for the M-1, M-3, and M-5 models is shown on figure 111. Although the spillways were revised, the tunnel remained the same for all models. Figure 111 shows positions of the water surface at various points along the horizontal tunnel as observed through windows in the top of the pipe. The water surface was comparatively rough in all cases. Conditions of flow in the tunnel for all three models were practically the same. The water surface in the horizontal section upstream from the bend was nearly level for all flows. At the beginning of the horizontal bend, the water surface rose at the outside and receded at the inside, reaching a nearly vertical position at the midpoint of the bend. After passing the midpoint, the water surface dropped back, assuming a horizontal position, and did not swing on over in a spiral. In all cases on the final and semifinal models, M-3 and M-5, ample air space existed at the top of the tunnel. A distinct pulsation occurred in the M-3 model for certain conditions, during which the pipe was completely filled at the lower end.

Velocities in the tunnel for the M-3 model, measured during a wide range of flows, are recorded in terms of prototype values on figure 112. Observations were made with a pitot tube and pressures were measured with a mercury manometer. Velocities were observed at two points; 135 feet downstream from the vertical bend and at the end of the pipe. Velocities were generally higher during the larger discharges, as friction effects were relatively less for the larger flows. Since the frictional resistances in model and

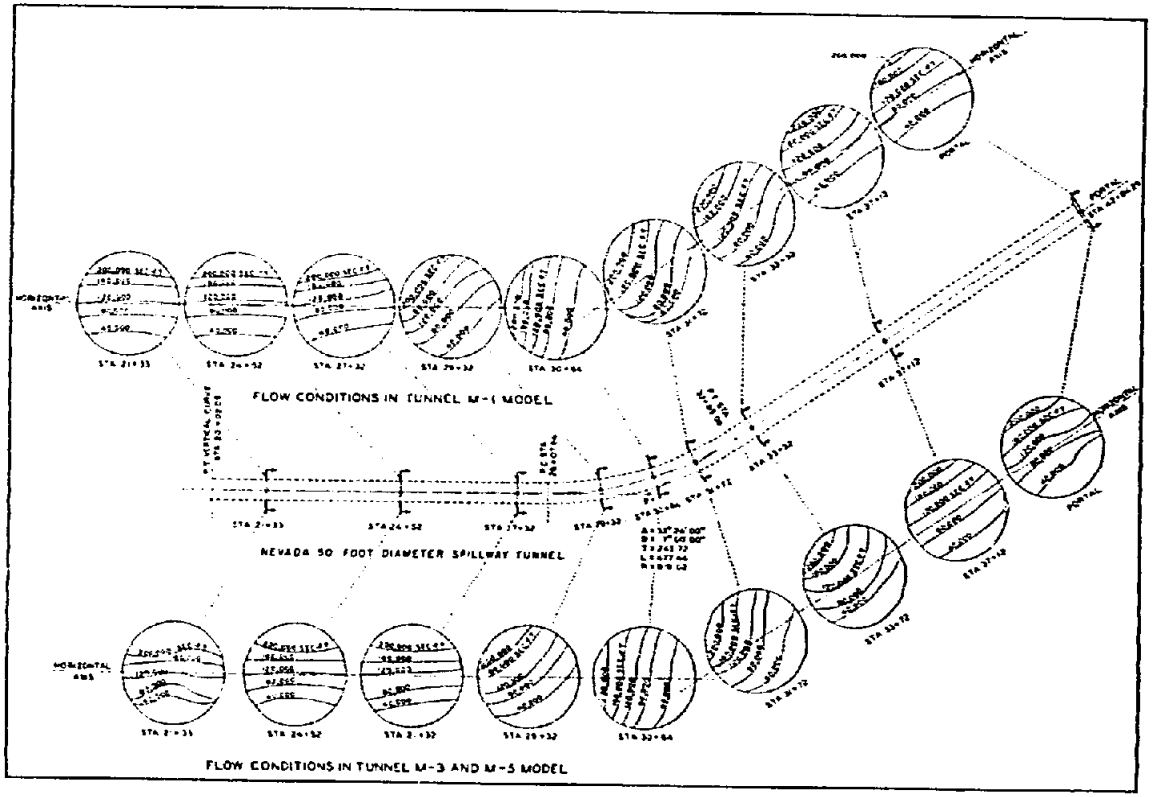


FIGURE 111—WATER SURFACES IN NEVADA SPILLWAY TUNNEL FOR VARIOUS DISCHARGES, MODELS M-1, M-3, AND M-5

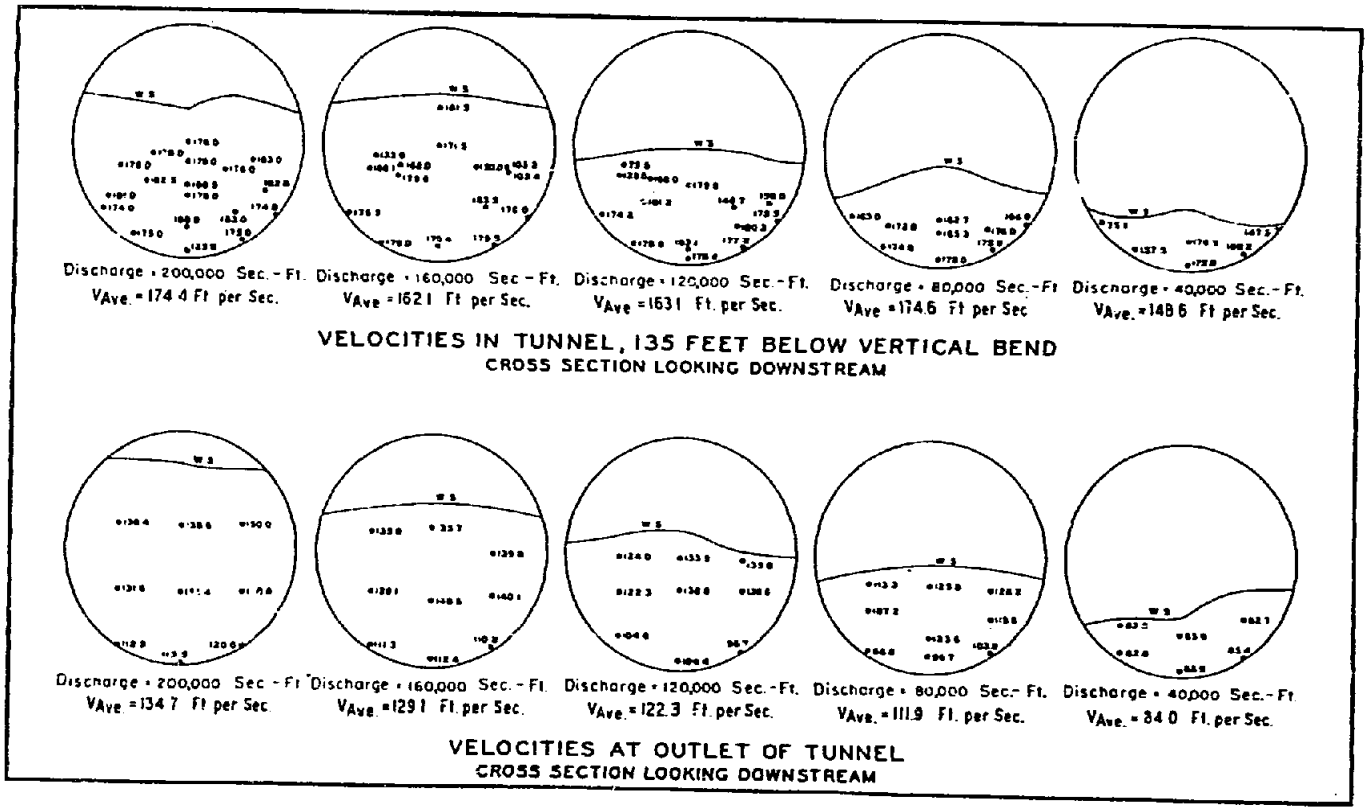


FIGURE 112—VELOCITY DISTRIBUTION IN NEVADA SPILLWAY TUNNEL, MODEL SCALE 1:20

prototype were not to scale, the slope of the model tunnel was increased to partially compensate for this discrepancy. Results obtained for maximum discharge were very similar to those computed in the spillway design. The approximate average velocity observed for 200,000 second-feet was 174 feet per second immediately downstream from the vertical bend, and 135 feet per second at the end of the pipe. The corresponding computed velocities were 173 and 124 feet per second, respectively. Due to air resistance and other factors which enter the problem when extremely high velocities are involved, it is not expected that velocities in the prototype will reach these figures.

CHAPTER VI—EROSION OF CONCRETE BLOCKS UNDER HIGH VELOCITIES

73. Nature of Tests.—Concrete blocks were subjected to the action of a continuous stream of water issuing from a nozzle at velocities between 100 and 175 feet per second. Surfaces of the blocks were either untreated or coated with tar and asphalt preparations. The water jet was applied by one of the following methods:

1. Passed through a straight circular hole.
2. Followed a semicircular groove.
3. Passed over or entered an open joint.
4. Impinged against or over a surface offset.
5. Impinged against a smooth surface at angles from zero to 90 degrees.

Visual inspection and photographic records were made of the concrete surfaces after various periods of testing.

MATERIALS, TEST BLOCKS, EQUIPMENT, AND TESTS

74. Materials.—Arizona aggregate and cement D_4 44, as described in the bulletin on "Mass Concrete Investigations", were used in all blocks. The aggregate was graded as shown in the following tabulation:

	Cumulative Percents Retained									
Sieve	1½ in.	¾ in.	⅜ in.	No. 4	No. 8	No. 14	No. 28	No. 48	No. 100	F.M.
Sand[a]	0	0	0	0	9.4	29.5	41.0	69.2	98.7	2.48
Sand[b]	0	0	0	0	16.3	27.5	42.3	88.5	100.0	2.75
Gravel	0	42.0	77.0	100.0	100.0	100.0	100.0	100.0	100.0	7.19

[a]The fineness modulus of the total aggregate with this sand was 5.66.

[b]The fineness modulus of the total aggregate with this sand was 5.74, used in specimens A-13, A-14, C-1, G-1, and G-2 only.

Denver city water was used for mixing and curing.

Water used for testing was drawn from a penstock supplying

the Guernsey power plant. Although no analysis was made, the testing water was believed to have been essentially free from silt or sand.

Tar and asphaltic coating materials were not analyzed.

75. Test Blocks.—Cement, sand, and gravel ingredients were mixed in the ratio of 1:2:4, by dry-rodded volumes, quantities for each batch being measured by weight. Proportions by weight were 1:2.31:4.76. Sufficient water was added to give a slump of 3 to 5 inches. A summary of test blocks, composition, and tests is given in the tabulations at the end of the chapter.

The unit weight of the fresh concrete in all blocks except G-1 and G-2 was between 149.9 and 150.1 pounds per cubic foot. The cement in each block was equivalent to 1.24 barrels per cubic yard.

For most blocks, the materials were mixed by hand according to the American Society for Testing Materials, standard method, Designation: C 39-27, and were compacted into forms by hand rodding. Concrete for blocks G-1 and G-2 was mixed in the laboratory mixer[10] for 3½ minutes, then compacted in two layers by vibration for approximately one minute each, on a suspended vibrated platform.

Blocks were cured in water or moist air until tested, or until test surfaces were coated. After surface coatings were applied, blocks were stored in the laboratory until tested.

Test blocks were shaped as shown in figure 113. In each corner of the block, two inches from each edge, a ¾-inch diameter pipe with a lock nut on one end was cast to provide a means of anchoring the block to the support bars in the testing apparatus. The hole in the D and E blocks was formed by turned, wooden rods well oiled and wrapped with oiled paper. The G blocks were cast on end with the test surfaces against sheet-metal-lined forms. Each of the A blocks was cast in a single six-inch layer, the B and C blocks in two sections, the D and E blocks in two three-inch layers, the F blocks in two 5-inch layers and each of the G blocks in two equal layers.

Test surfaces were, in most cases, untreated; but some of them were coated with water-gas tar, coal-gas tar, or a commercial paint made of pure asphalt. All coatings of water-gas tar and asphaltic

[10]Described in the bulletin on "Mass Concrete Investigations" Part VII, Bulletin 4, Final Reports, Boulder Canyon Project.

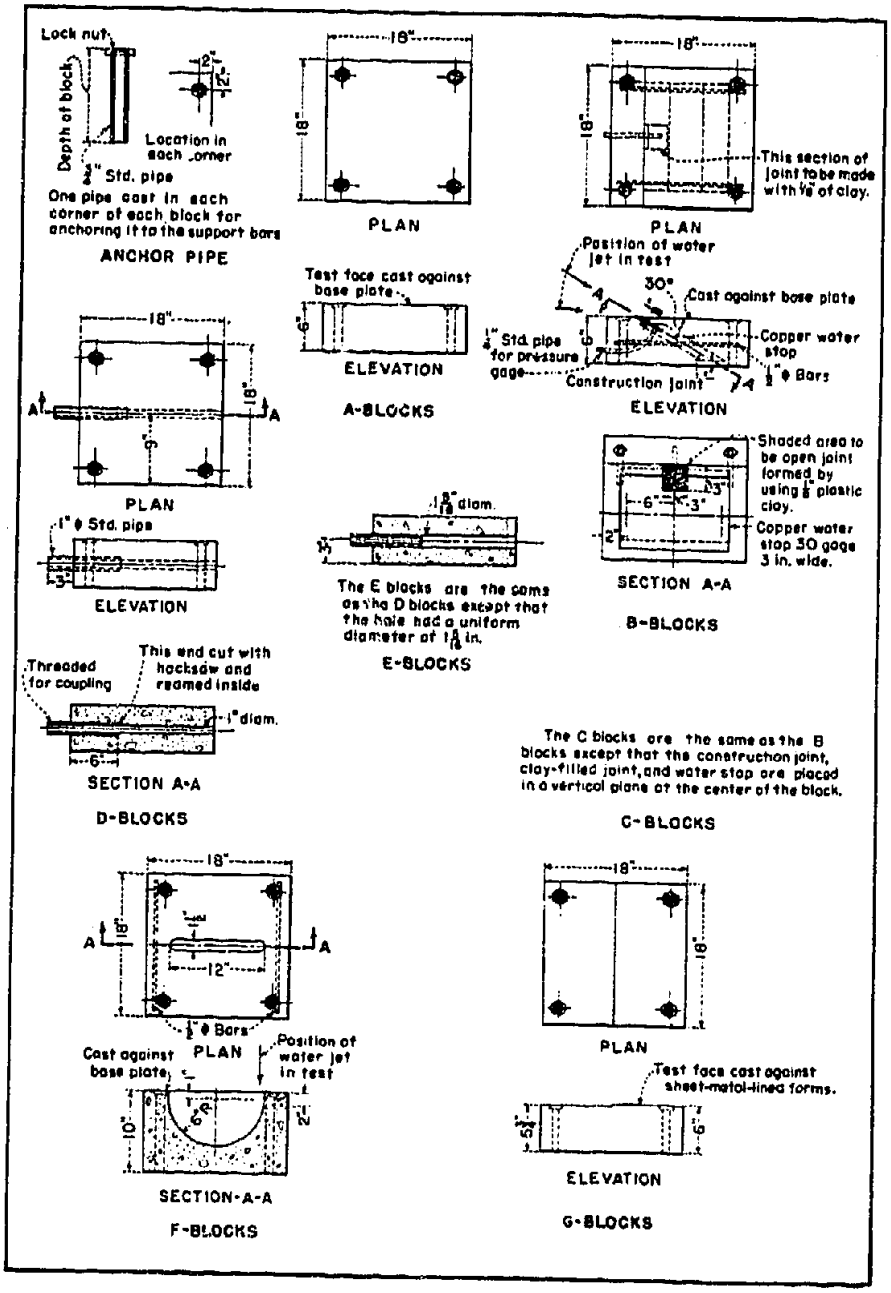


FIGURE 113—TEST BLOCKS

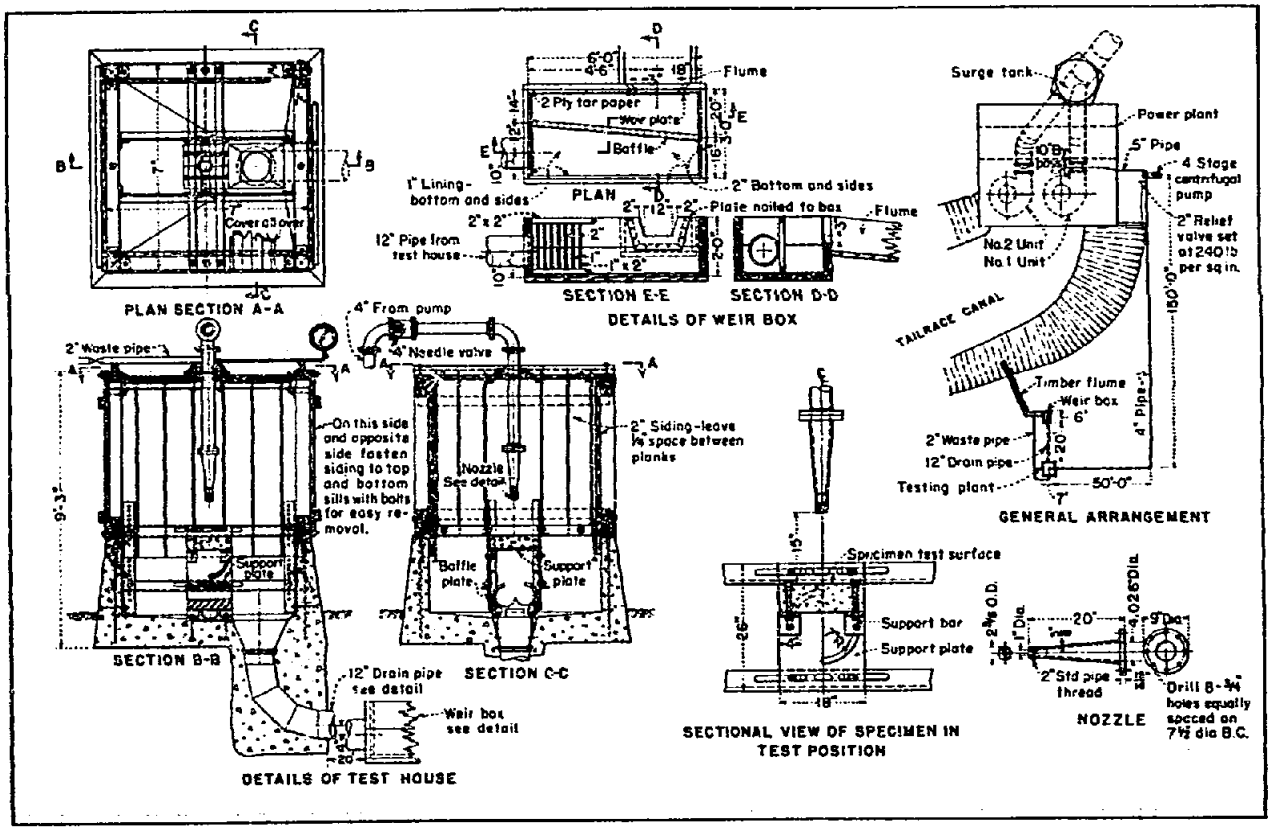


FIGURE 114—JET TEST APPARATUS

paint were applied cold, and all coatings of coal-gas tar were applied hot. Each coating was allowed to set before application of a succeeding coating.

76. Test Apparatus.—The testing apparatus is shown in figures 114 and 115. It consisted essentially of a four-stage centrifugal pump that supplied water to a one-inch diameter nozzle at velocities up to 175 feet per second. A relief valve limited the discharge pressure to 240 pounds per square inch. The nozzle was located in a test house, and was mounted so that the jet could be impinged on the test blocks at any angle between 0 and 90 degrees. The test block was bolted through the four embedded pipes to the support bars which were in turn bolted to the support plates. The water passing over the block was carried away in a 12-inch drain pipe to a weir box from which it escaped over a Cippoletti weir, see figure 116. Depths over the weir were read on a gage and jet velocities were computed by the aid of a specially prepared curve. A pressure gage, mounted on the supply pipe above the nozzle, aided in the control of jet velocities.

77. Tests.—The test surface of a block, cured or coated and ready for test, was photographed just before being mounted on the test frame. All except the D, E, and F blocks were mounted so that the center of the test face was about 15 inches from the nozzle. The embedded pipes of D and E blocks were directly connected to the nozzle and block F-1 was mounted so that the nozzle was about

FIGURE 115—TEST HOUSE

1½ inches from one end of the semicircular groove. Blocks were set at angles with the jet, as indicated in the tabulations. The velocity of the jet was controlled by regulation of the water discharged from the nozzle-feeder line through the two-inch waste pipe. Readings of the pressure gage were helpful in regulating the velocity; but velocities, computed from rates of outflow over the

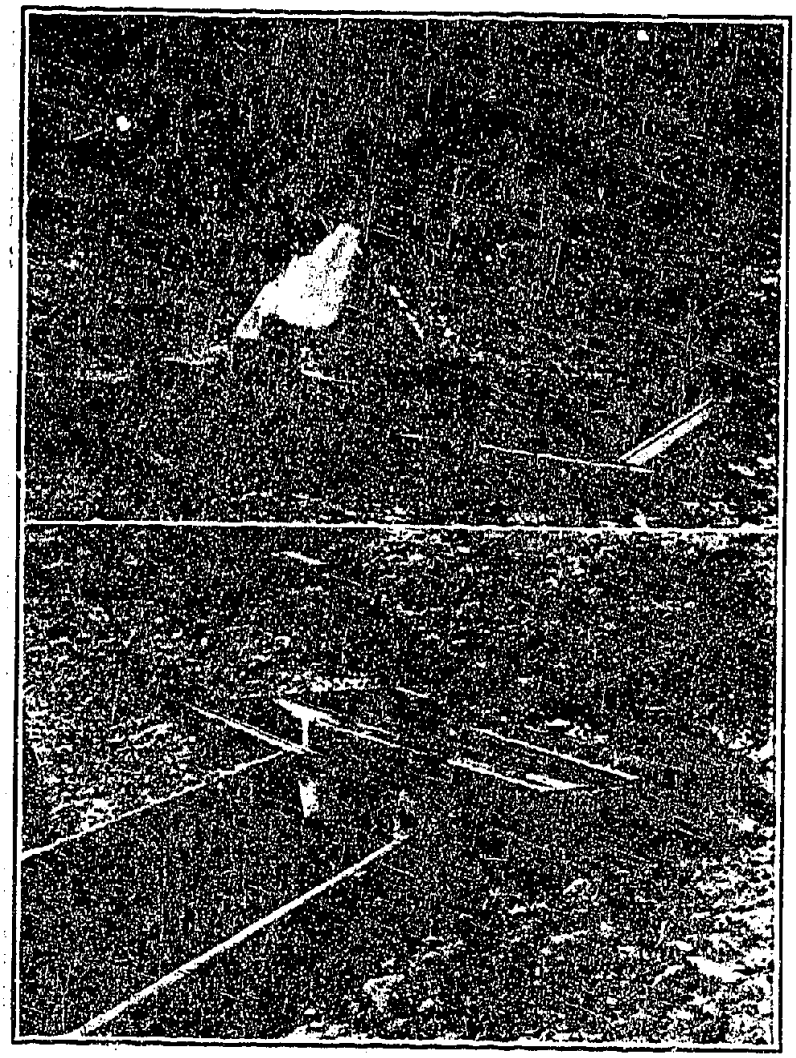


FIGURE 116—WEIR BOX
Upper View Shows Weir Box and Waste Pipe in Operation

weir, were recorded in the tabulations. Most blocks were removed from the frame at about two-day intervals to permit photographing test surfaces. Upon continuation of the test, many blocks were reset at angles differing from those previously used. After completion of testing, test surfaces were again photographed.

TEST RESULTS AND THEIR APPLICATION

78. **Test Results.**—All test results are shown in the tabulations and on figures 117 to 132. Examination of test blocks proved to be more satisfactory than photographs for judging the character and extent of erosion, since the photographs tended, in general, to exaggerate actual conditions.

There was practically no evidence of wear or erosion of the concrete on any plane or jointed surface subjected to the water jet at any velocity when the angle between the jet and the block was small, see records for blocks A-1, B-1, C-1, G-1, and G-2. For this reason most of the plane-surface blocks were tested under jet

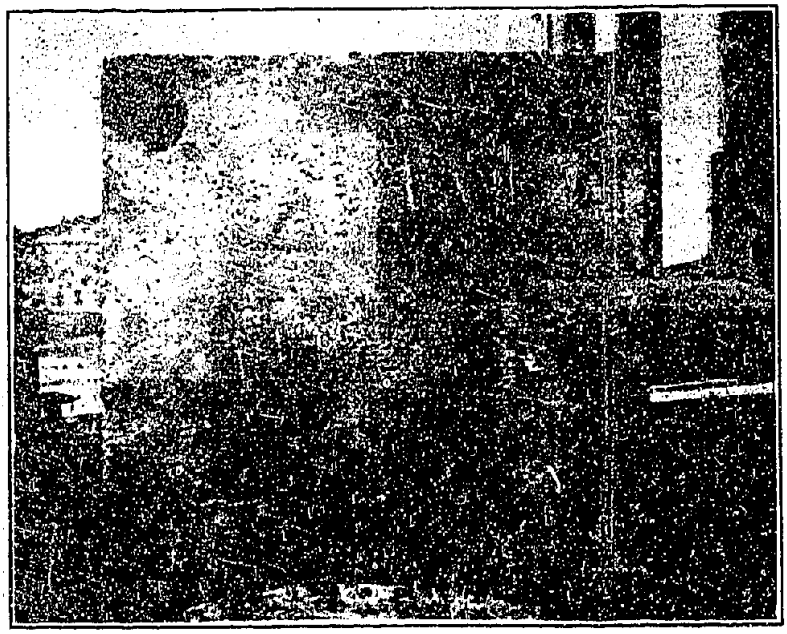


FIGURE 117—BLOCK A-1 AFTER 18.5 DAYS ON TEST
Jet angle, 5°: Velocity, 125 fps, 1 day; 150 fps, 1 day; 175 fps, 1 day. Jet angle, 45°: Velocity, 175 fps, 6.9 days

velocities of 175 feet per second and jet angles of 45° and 90° only, in order to determine the erosion produced by such extremely severe conditions. A roughening or slight pitting of the mortar surface generally resulted from two days' exposure to the jet at the 45° angle, see records for blocks A-1, A-6, and A-7. At the 90° jet angle effects were similar but more pronounced. Areas affected had the shape of an annular ring approximately one inch wide around a two-inch inside diameter circle, see records for blocks A-2, A-6, and A-7.

Even long-time tests failed to show appreciable damage to plane surfaces. Block A-1, tested for 18½ days with small jet angles, had only a roughened surface at conclusion of the test, see figure 117. Block A-2, tested under a jet velocity of 175 feet per second and a jet angle of 90°, was pitted to a depth of one-quarter inch at the end of two weeks, see figure 119; and at the end of five weeks it was pitted to an average depth of about one-half inch with one hole about an inch deep, see figure 120. This is a remarkably small amount of erosion for such severe exposure.

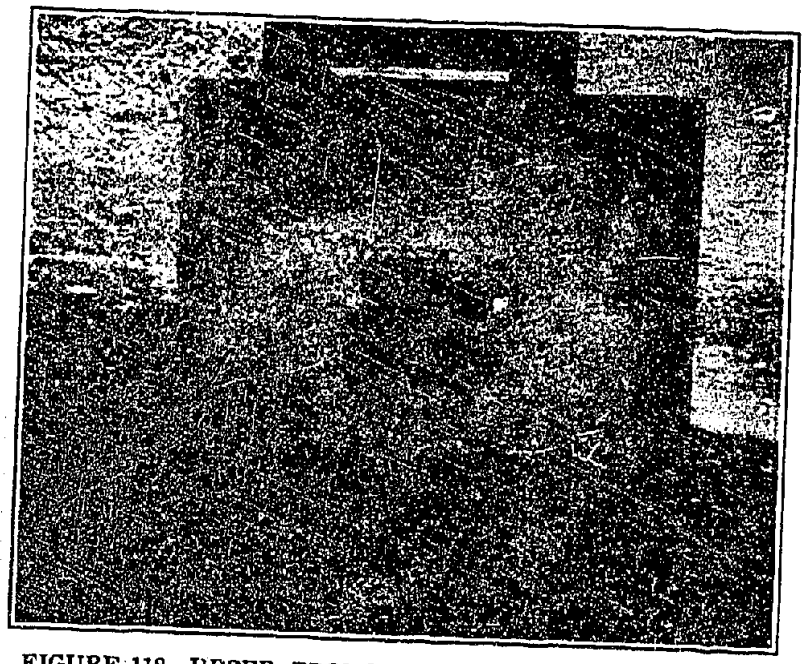


FIGURE 118—UPPER, TROWELED SURFACE OF BLOCK A-2 AFTER 5 DAYS ON TEST
Jet angle, 90°: Velocity, 175 fps, 5 days

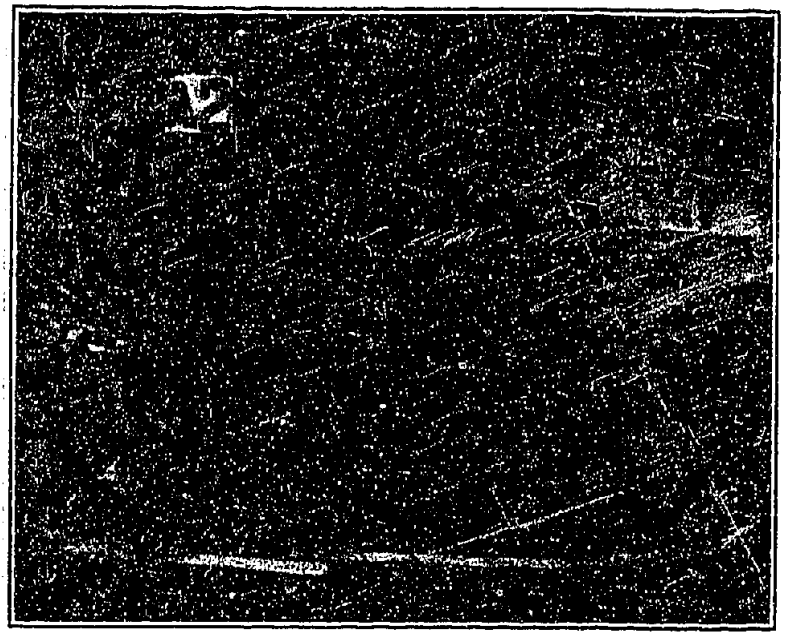


FIGURE 119—BLOCK A-2 AFTER 13.3 DAYS ON TEST
Jet angle, 90°: Velocity, 175 fps, 13.3 days

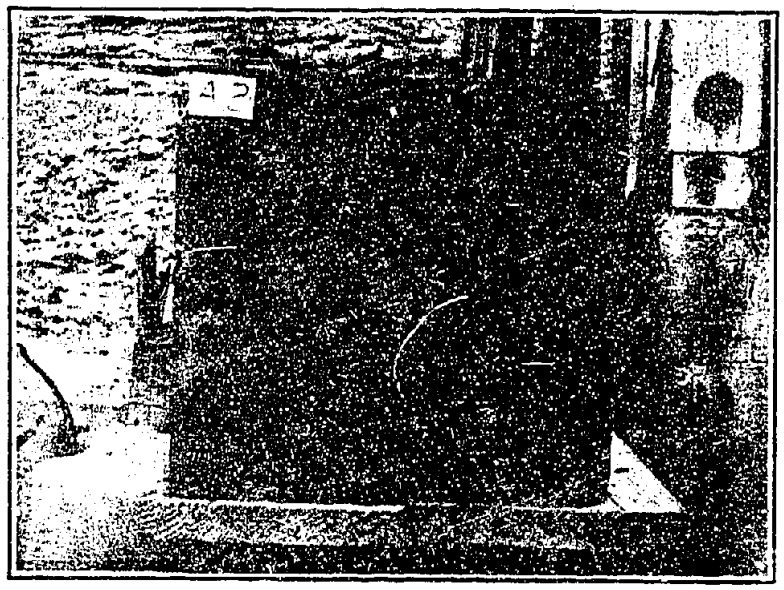


FIGURE 120—BLOCK A-2 AFTER 34.3 DAYS ON TEST
Jet angle, 90°: Velocity, 175 fps, 34.3 days

The surface texture of the concrete had some influence on the resistance to erosion. Most of the blocks were exposed to the jet on the bottom side, which had been cast against the base plate. Surfaces on blocks G-1 and G-2, which were tested on a face cast vertically against sheet-metal-lined forms, resisted erosion as well as test faces on the bottom of the blocks, see figures 131 and 132. The upper, troweled, laitance-bearing surface of block A-2 was unintentionally subjected to the water jet for five days, and was found to be far less resistant to erosion than the opposite, bottom face of the same block, see figure 118.

Paint coatings appeared to furnish little if any protection to the concrete surfaces under the conditions of the tests, see records for blocks A-3, A-4, A-5, A-13, and A-14.

After four days' testing at a jet velocity of 175 feet per second and a jet angle of 5° and 30°, a block with a one-quarter-inch offset showed only slight wear from the impingement of water against the offset, see figure 131. A companion block similarly tested four days with the water jet impinging over, not against, the one-quarter-inch offset showed no appreciable erosion, see figure 132.

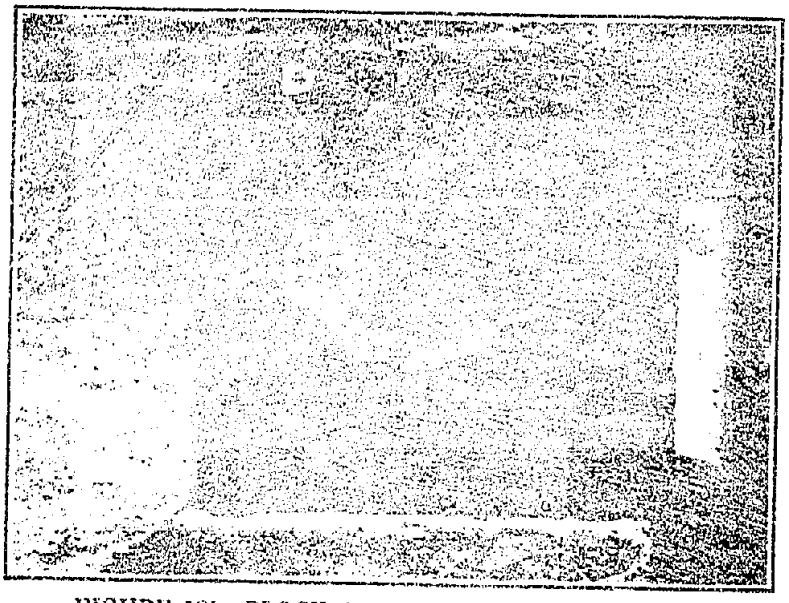


FIGURE 121—BLOCK A-4 AFTER 4 DAYS ON TEST
Jet angle, 45°: Velocity, 175 fps, 2 days
Jet angle, 90°: Velocity, 175 fps, 2 days

In blocks having a cylindrical hole or semicircular groove the apparent wear was inappreciable, except near the point of water exit from the block, see figures 129 and 130. At those points, substantial cavitation occurred under jet velocities of 105 to 175 feet per second. The cavitation is attributed principally to outlet disturbances accompanying the sudden release of the stream from its constricting channel.

Blocks with vertical or sloping joints were badly damaged under jet velocities of 175 feet per second and jet angles of 30°, see figures 126 and 128. The damage was evidently due to the disrupting effect of water pressure in the joint.

79. **Application to Boulder Dam Spillways.**—The Boulder Dam spillway system is designed with the expectation of obtaining practically streamline flow in the lower tunnel sections where maximum velocities occur. Comprehensive specification provisions for accuracy of alinement and rigid control of concrete manufacture and placement help to insure this.

The only tests really applicable to this condition of smooth unobstructed flow are those in which the water was applied at

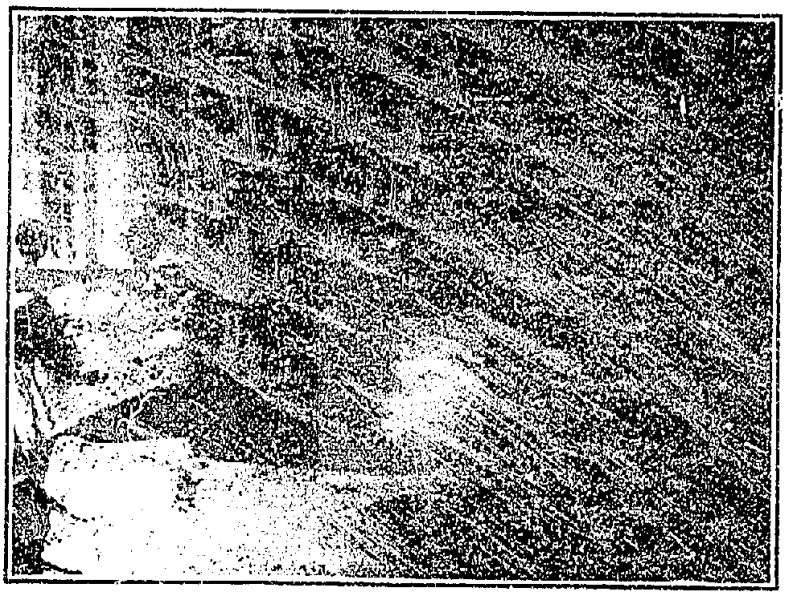


FIGURE 122—BLOCK A-5 AFTER 4 DAYS ON TEST
Jet angle, 45°: Velocity, 175 fps, 2 days
Jet angle, 90°: Velocity, 175 fps, 2 days

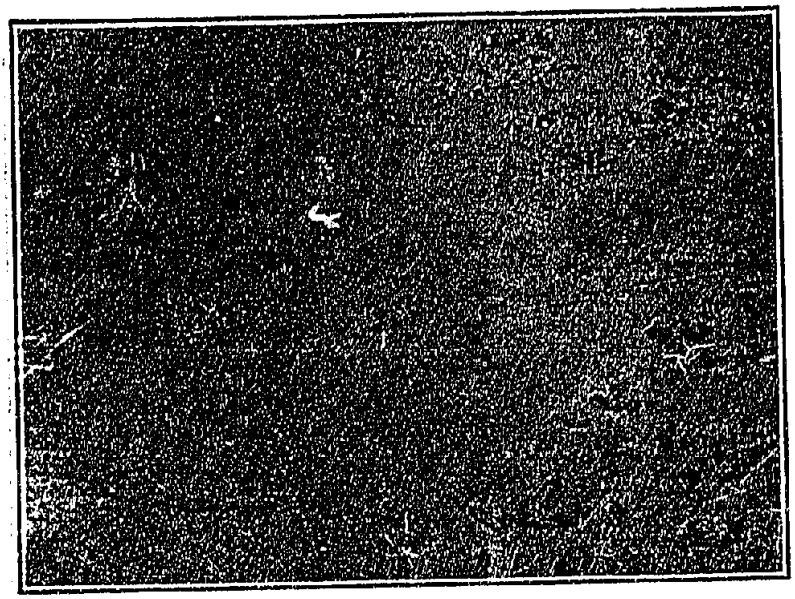


FIGURE 123—BLOCK A-7 AFTER 4 DAYS ON TEST
Jet angle, 45°: Velocity, 175 fps, 2 days
Jet angle, 90°: Velocity, 175 fps, 2 days

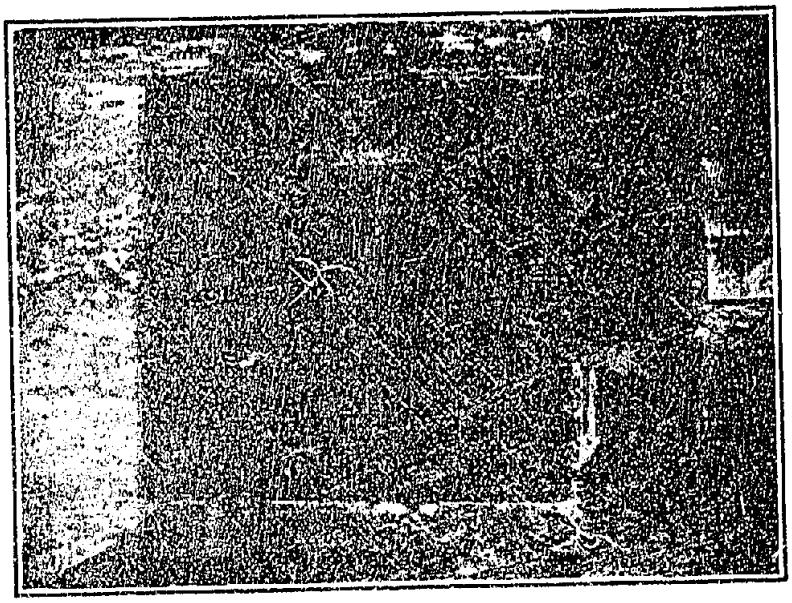


FIGURE 124—BLOCK A-13 AFTER 4 DAYS ON TEST
Jet angle, 45°: Velocity, 175 fps, 2 days
Jet angle, 90°: Velocity, 175 fps, 2 days

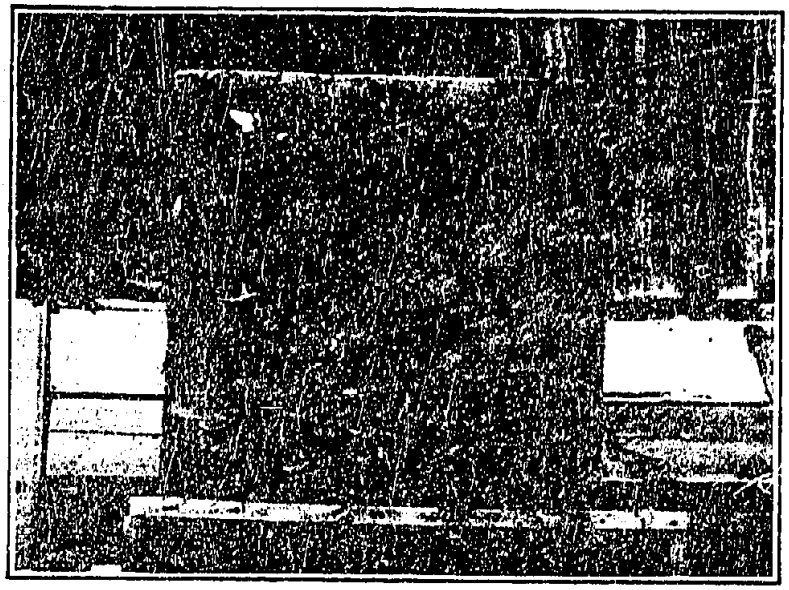


FIGURE 125—BLOCK B-1 BEFORE TEST

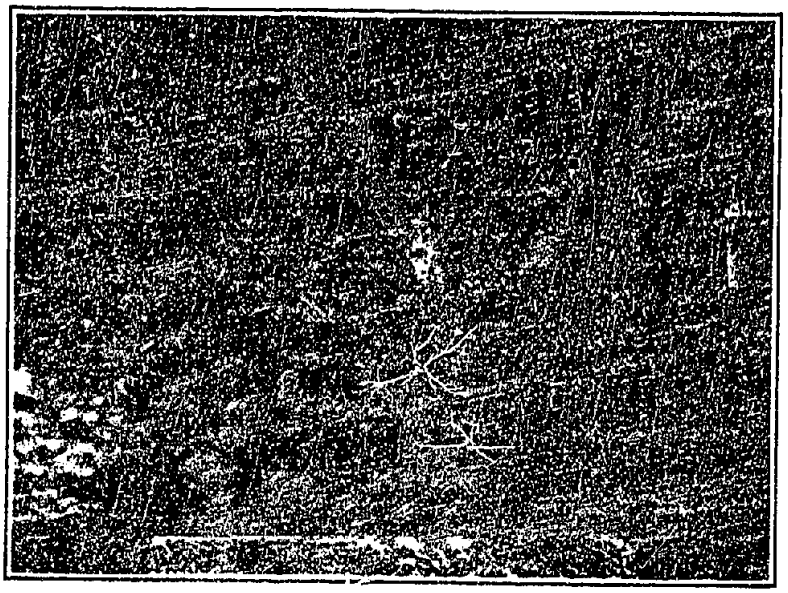


FIGURE 126—BLOCK B-1 AFTER 2.1 DAYS ON TEST
Jet angle, 5°: Velocity, 175 fps, 2 days. Jet angle, 30°: Velocity, 175 fps, 0.1 day; pressure dropped from 75 to 15 lbs. per sq. in. after 2½ hours on test

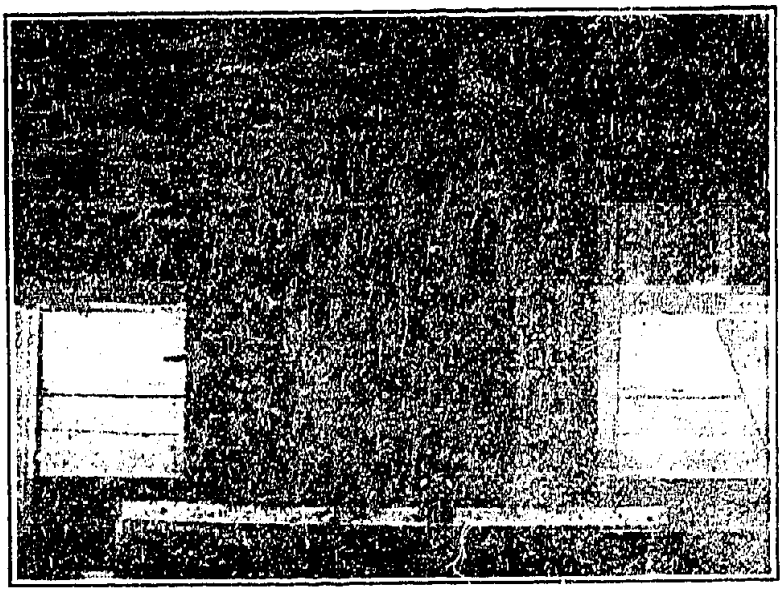


FIGURE 127—BLOCK C-1 BEFORE TEST

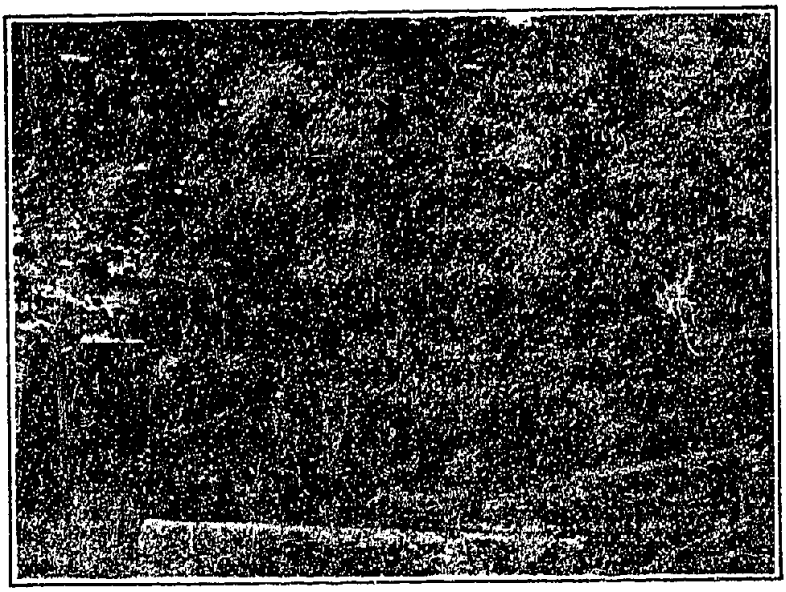


FIGURE 128—BLOCK C-1 AFTER 4 DAYS ON TEST
Jet angle, 5°: Velocity, 175 fps, 2 days. Jet angle, 30°: Velocity, 175 fps, 2 days; pressure in slot, 28 lbs. per sq. in.

small jet angles. It is significant, therefore, that in no case did the gliding velocity tests develop more than a slight roughening of the surface, even after ten days of continuous application at maximum velocity.

Other tests at a jet angle of 45° and 90° were intentionally extreme or accelerative and have no direct application to the spillways. These tests are enlightening, however, in revealing the resistance of the concrete to severe or abusive treatment. The fact that block A-2 developed pitting to an average depth of only one-half inch, which is a relatively minor surface impairment, after five weeks of continuous application of water directed normal to the surface at a velocity of 175 feet per second, seems a remarkable testimonial of wear resistance.

It should also be noted that the concrete actually placed in the spillway tunnels at Boulder Dam is of a quality superior to that of which the test blocks were made. The average water-cement ratio used in the tunnel concrete was 0.48, by weight, as compared to values of 0.53 to 0.63 used in the test blocks.

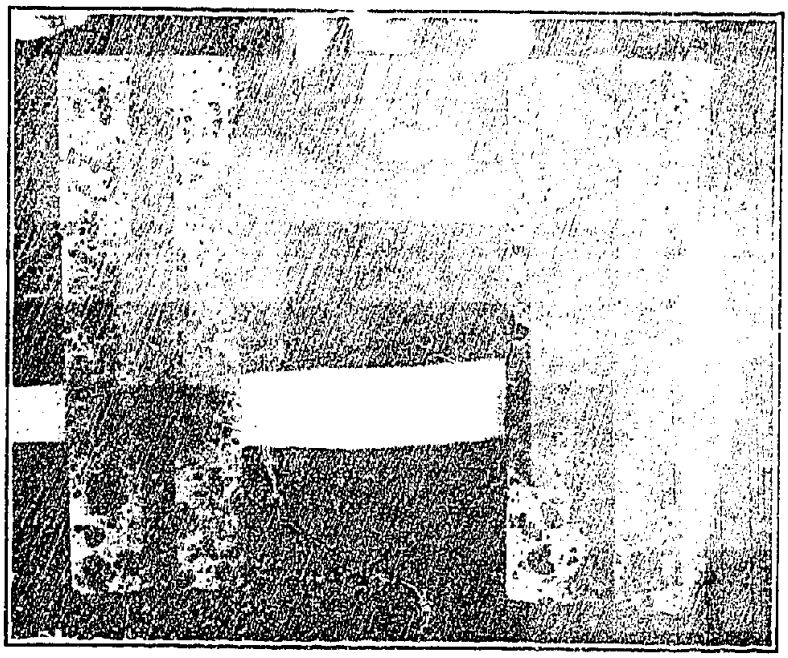


FIGURE 129—CROSS SECTIONS OF BLOCKS D-1 AND E-1 AFTER TESTS
Block D-1: Velocity, 105 fps, 1 day; 140 fps, 2 days
Block E-1: Velocity, 125 fps, 1 day; 150 fps, 1.5 days; 160 fps, 1.3 days

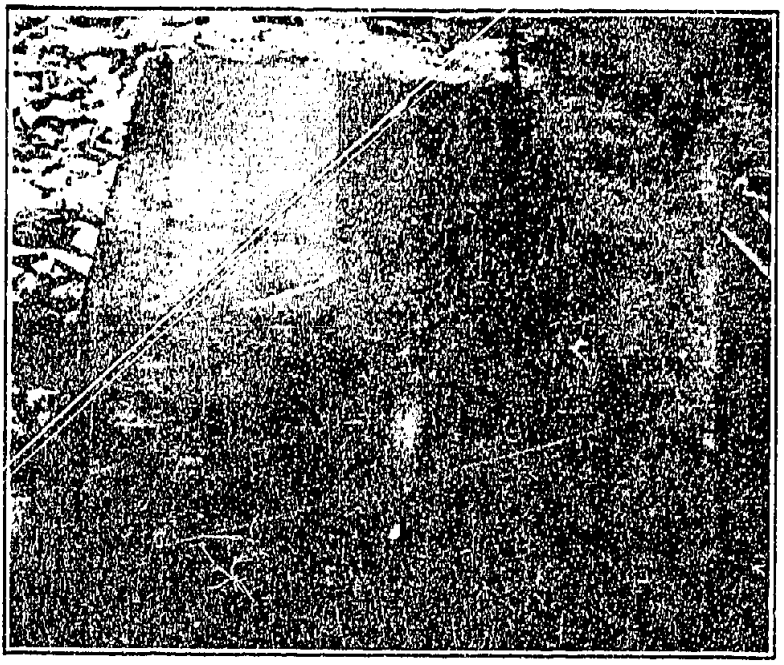


FIGURE 130—BLOCK F-1 AFTER 3 DAYS ON TEST
Jet angle, 90°: Velocity, 125 fps, 1 day; 150 fps, 1 day; 175 fps, 1 day

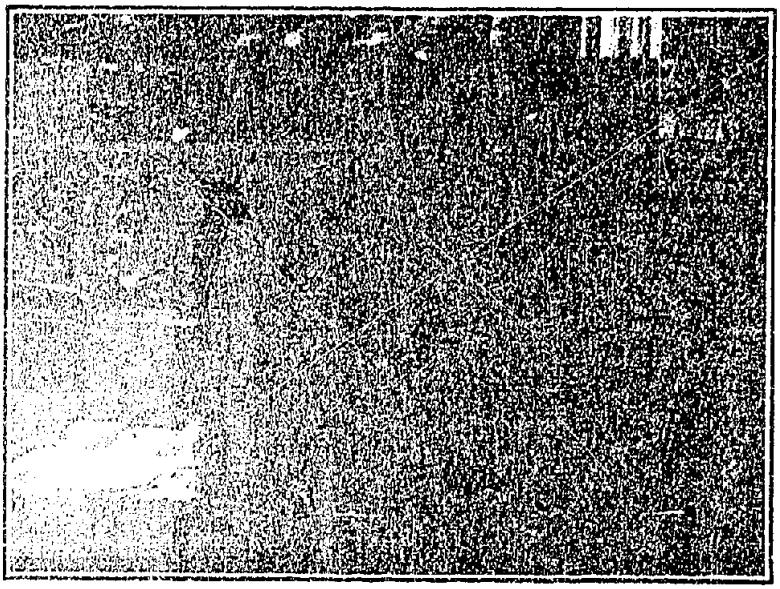


FIGURE 131—BLOCK G-1 AFTER 4 DAYS ON TEST
Jet angle, 5°: Velocity, 175 fps, 2 days
Jet angle, 30°: Velocity, 175 fps, 2 days

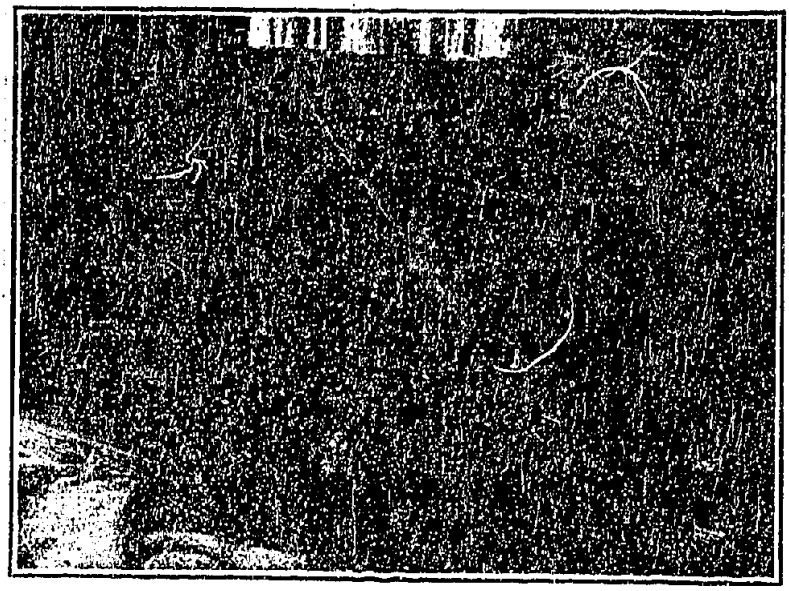


FIGURE 132—BLOCK G-2 AFTER 4 DAYS ON TEST
Jet angle, 5°: Velocity, 175 fps, 2 days
Jet angle, 30°: Velocity, 175 fps, 2 days

SUMMARY—TEST BLOCKS AND RESULTS

Block No.	Type of Block	W/C by Wt.	Slump, In.	Surface Treatment	Moist Curing, Days	Age at Test, Days	Approx. Jet Velocity Ft. per sec.	Angle of Jet with Test Face	Duration of Test, Days	Observed Data	Block No.
A-1	Plain face	0.62	3 to 4	None	91	91	.....	.....	0	Photo taken before test.	A-1
							125	5°	1	No appreciable erosion. Keel marks unaffected.	
							150	5°	1	No appreciable erosion. Keel marks unaffected.	
							175	5°	9.6	Small rough spot, some enlargement of surface pits.	
							175	5°	6.9	Surface rough. Greater part of roughening occurred during last three days	
A-2	Plain face	0.62	3 to 4	None	112	112	.....	.....	0	Photo taken before test.	A-2
							175	90°	5	Unintentionally tested on upper, trowelled surface (as cast). The mortar surface was worn off in the shape of an annular ring. Other tests were made on the bottom of specimens, as cast.	
							175	90°	6.3	Mortar surface worn off in spots. Wear was materially less than on trowelled surface.	
							175	90°	7	Gradual erosion in the form of a circle. Pits approximately ¼-inch deep.	
							175	90°	7	Erosion was confined to strip 1-inch wide on outside of a circle of approximately 4-inch diameter. Pits less than ⅛-inch deep.	
							175	90°	7	Erosion around a circular ring a little deeper than after 14-day test.	
							175	90°	7	Largest hole in block 1-inch deep and 12-inches square at the bottom. Rest of erosion less than ⅛-inch deep.	
A-3	Plain face	0.63	4½ to 5	One coat of water-gas tar and 1 coat of coal-gas tar	28	73	.....	.....	0	Photo taken before test.	A-3
							175	45°	2	Paint removed during first day. Hole approximately ¼-inch deep over 5-sq. inch area.	
							175	90°	2	Little additional erosion.	
A-4	Plain face	0.63	4½ to 5	One coat of water-gas tar 2 coats coal-gas tar	28	66	.....	.....	0	Photo taken before test.	A-4
							175	45°	2	Paint worn off during first day.	
							175	90°	2	Very little erosion of concrete.	
A-5	Plain face	0.63	4½ to 5	One coat water-gas tar	28	76	.....	.....	0	Photo taken before test.	A-5
							175	45°	2	Spots of paint still adhering to block.	
							175	90°	2	Very little change, slight roughening of surface.	
A-6	Plain face	0.63	4½ to 5	None	40	40	.....	.....	0	Photo taken before test. Poor specimen. Concrete honey-combed.	A-6
							175	45°	2	Wear imperceptible. No picture taken.	
							175	90°	2	Mortar coating removed. Test duplicated using block A-7.	

SUMMARY—TEST BLOCKS AND RESULTS (Continued)

Block No.	Type of Block	W/C by Wt.	Slump, In.	Surface Treatment	Moist Curing, Days	Age at Test, Days	Approx. Jet Velocity ft. per sec.	Angle of Jet with Test Face	Duration of Test, Days	Observed Data	Block No.
A-7	Plain face	0.63	4½ to 5	None	60	60	175	45°	2	Small pits formed.	A-7
							175	90°	2	Pits more pronounced.	
A-13	Plain face	0.60	5	Two coats asphaltic paint	36	55	.....	.....	0	Photo taken before test.	A-13
							175	45°	2	Paint partially removed.	
							175	90°	2	Paint removed. But little wear of concrete noted.	
A-14	Plain face	0.60	5	Three coats asphaltic paint	36	57	175	45°	2	Paint partially removed. No appreciable erosion of concrete.	A-14
							175	90°	2	Paint removed. Hole approximately ½-inch deep cut in concrete.	
B-1	Noping joint. 30° angle.	0.63	4½ to 5	None	125	125	.....	.....	0	Photo taken before test. This specimen was tested with jet directed down the slot.	B-1
							175	5°	2	*Very little erosion. No water pressure in slot.*	
							175	30°	0.1	Hole having approximately 3 sq. in. surface area, adjacent to slot developed during first 2½ hr. coincident with pressure drop in slot from 75 to 15 lb. per sq. in.	
							175	30°	1.9	Coincident with pressure drop to zero, large piece of concrete having 1 in. thickness and approximately 12 sq. in. average surface area broken off, due apparently to water under pressure severing bond between concrete and embedded copper strip.	
C-1	Vertical joint	0.60	5	None	72	72	.....	.....	0	Photo taken before test. [illegible] specimen was tested with the jet directed t[illegible] one side of the slot.	C-1
							175	5°	2	Very little erosion. No water pressure in slot.	
							175	30°	2	Hole 1 in. square and 1 in. deep worn in block. Water pressure of 28 lb. per sq. in. in slot.	
D-1	Hole through block flush with inside of embedded pipe	0.63	4½ to 5	None	127	127				The 1-inch diameter cylindrical test hole was imperfectly cast.	D-1
							105		1		
							140	.....	2	Some indication of erosion of thin surface film in half adjacent to pipe. Principal erosion confined to 5-inch length at outlet end of hole, the cavitation starting at exit and gradually progressing backwardly. Maximum depth of cavitation approximately ⅛ inch. Maximum velocity through hole limited by maximum allowable pressure (220 lb. per sq. in.) in supply pipe.	

SUMMARY—TEST BLOCKS AND RESULTS (Continued)

Block No.	Type of Block	W/C by Wt.	Slump, In.	Surface Treatment	Moist Curing, Days	Age at Test, Days	Approx. Jet Velocity ft. per sec.	Angle of Jet with Test Face	Duration of Test, Days	Observed Data	Block No.
E-1	Hole through block flush with outside of embedded pipe	0.63	4½ to 5	None	130	130				The cylindrical test hole (1¾ inch diameter) was imperfectly cast.	E-1
							125	.....	1		
							150	.....	1.5		
							160	.....	1.3	Some indication of erosion of thin surface film in half adjacent to pipe. Principal erosion confined to 5-inch length of outlet end of hole, the cavitation starting at exit and gradually progressing backwardly. Pitting confined to 1-inch length of hole during first 24 hours. Maximum depth of cavitation during entire test approximately ¼ inch. Maximum velocity through hole limited by maximum allowable pressure (220 lb. per sq. in.) in supply pipe.	
F-1	12-inch dia. semicircular groove	0.63	4½ to 5	None	138	138				This specimen was tested with the jet directed down one end of the slot.	F-1
							125	90°	1		
							150	90°	1		
							175	90°	1	Erosion of concrete practically confined to approximately 6-inch length of arc at outlet end of semicircular groove, the pitting in general diminishing with distance from exit. Maximum depth of cavitation approximately ¼ inch.	
G-1	¼-inch offset	0.53	4	None	60	60	.....	.....	0	This specimen tested with jet directed toward ¼-inch offset face.	G-1
							175	5°	2	No evidence of erosion	
							175	30°	2	Erosion at two points approximately 1 inch and ¼ inch long about 4 inches apart along edge of offset.	
G-2	¼-inch offset	0.53	4	None	65	65	.....	.....	0	This specimen tested with jet directed to pass over (not against) the offset.	G-2
							175	5°	2	No evidence of erosion.	
							175	30°	2	No appreciable erosion.	

CHAPTER VII—CONCLUSIONS

SUMMARY OF RESULTS

80. Spillway Channel.—Although the experiments described in this report were made primarily for the purpose of developing spillways for the unusual conditions existing at Boulder Dam, many of the results will be of value to designers of other spillways.

The experiments on the C-2 model demonstrated the reliability of the method of analyzing side-channel spillways by the theory of conservation of linear momentum.[10] It was found, however, that difficulty was encountered in computing results by this method when the flow was near the critical depth. Therefore results of computations made under such conditions are not always reliable.

Guide vanes on the spill face or on the channel bottom in many cases improve the flow and reduce the size of channel, but the expense of constructing them would probably be greater than the saving effected.

If it is necessary or desirable to secure a relatively smooth flow in the spillway channel, it may be done by increasing the cross section of flow, or by the introduction of baffles. For conditions existing at Boulder Dam, the former method proved most desirable. Of the many forms tested, the overhanging dentated baffle and the coping baffle on the rear wall produced the best results, although other forms were satisfactory. As baffles are expensive to construct, better results might possibly be obtained by an equal expenditure in channel enlargement. This can be determined by model tests.

A channel bottom with a steep slope will cause a shallow depth of flow and a high water surface in the channel opposite the spillway crest. This surface irregularity is likely to produce undesirable roughness in the flow down the inclined tunnel. A greater depth of flow may be obtained by placing a control section across the channel below the downstream end of the spillway crest, similar to the cross-weir used in the Boulder Dam spillways.

It was found that considerable care was necessary in designing

[10]Trans. Am. Soc. C. E., vol. 89, 1926, page 881.

the piers between the drum gates, to prevent the formation of a partial vacuum which would interfere with the operation of the gates.

The air content of the turbulent water in the model spillway channels was determined to be negligible, particularly at the maximum designed discharge. With that flow the drop in water surface between the reservoir and the channel was slight enough to prevent the entrainment of excessive quantities of air. At lesser discharges, where the drop was greater, the air content was greater; but with those conditions there was excess channel capacity available so that air content was not a critical factor to be considered. The air content in the prototype will undoubtedly be greater than indicated by the models.

In spillways similar to the Stoney gate model, it is undesirable to have changes in the direction of flow downstream from the gate where the water is moving at a velocity near or greater than critical velocity, since it is difficult to change the direction of flow under these circumstances. The change should be made upstream from the gate, where the velocity is below the critical value.

81. **Transition.**—It was found in all side-channel designs tested that it was difficult to obtain symmetrical flow in the transition when the center line of the transition was coincident with the center line of the spillway. In all tests there was a concentration of flow on the side opposite the overflow crest which induced a spiral flow throughout the transition and also in the tunnel downstream. The most effective means of correcting this condition was to raise the entire transition 40 feet and at the same time shift it so that the wall of the channel and the wall of the transition were in line on the side opposite the overflow crest. By keeping the same size transition section, this produced an offset at the entrance on the side of the overflow crest. Due to the nature of the flow down side-channel spillways it may be found necessary to use similar designs to obtain satisfactory flow conditions in other installations.

82. **Tunnels.**—Flow in the inclined and horizontal tunnels was satisfactory for all discharges after the later alterations were made in the spillway channel and inclined transition. Experiments on flow in bends indicated that the radius of the vertical bend at the bottom of the inclined tunnel section did not have an important effect on the character of the flow, although increasing the radius

of the bend by 50 feet gave sufficient improvement to justify the change. Flow through the horizontal bend was satisfactory and this remained as in the original design.

The close agreement of results obtained in the similar models creates confidence that the results on the models will be duplicated in the prototype and that the Boulder Dam spillways will perform in a satisfactory manner. The results also furnish considerable evidence of the reliability of hydraulic models for solving complicated hydraulic problems.

The agreement of results from models of different sizes tends to emphasize and encourage the use of comparatively small models. It was found that the smaller models were less expensive to construct and much easier to alter. It is concluded, therefore, that the most satisfactory procedure in model testing can be obtained by making the first tests on small scale models. After the designs are perfected, the tests can be checked on larger models. The unprecedented magnitude of the work at Boulder Dam will rarely be encountered, however, and large models such as these will not often be required.

83. **Resistance to Erosion.**—Since considerable care was taken in designing the shapes of the spillway transitions to eliminate negative pressures or vacuums, it is not expected that erosion will occur due to cavitation. In the regions of the tunnels where the maximum velocities occur, virtually streamline flow is expected. Under this condition, it is concluded from the results of the jet tests that concrete of ordinary proportions and controlled quality will safely withstand any erosive action produced under conditions of maximum discharge in the spillways.

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PART I—INTRODUCTORY

General Description of Project
History of Project
Legal and Financial Problems

PART II—HYDROLOGY

Stream Flow and Reservoir Operation
Lower Basin Utilization
Upper Basin Utilization

PART III—PREPARATORY EXAMINATIONS AND CONSTRUCTION

Geologic Investigations
Surveys and Preliminary Construction
(Topographic surveys, highways, railways, power lines, substations, and Boulder City).

PART IV—DESIGN AND CONSTRUCTION

General Features
Concrete Manufacture, Handling and Control
Boulder Dam
Diversion Structures and Spillways
Intake Towers and Outlet Works
Penstocks and Outlet Pipes
Hydraulic Valves and Gates
Handling Facilities
(Cableway, cranes, and other permanent handling facilities).
Power Plant Building
Generating, Transforming, and Switching Equipment
Turbines, Governors, and Mechanical Auxiliaries
Control, Communication, and Electrical Auxiliaries
Imperial Dam and Desilting Works
All-American Canal and Canal Structures

LIST OF BULLETINS—(Continued)

PART V—TECHNICAL INVESTIGATIONS

Trial Load Method of Analysing Concrete Dams
Model Tests of Boulder Dam
Slab Analogy Experiments
Stress Studies for Boulder Dam
Penstock Analysis and Stiffener Design
Model Tests of Arch and Cantilever Elements
Research Measurements at Dam

PART VI—HYDRAULIC INVESTIGATIONS

Model Studies of Spillways* (paper cover, $1.00) (cloth cover, $1.50)
Model Studies of Penstock and Outlet Works* (paper cover, $1.00) (cloth cover, $1.50)
Studies of Crests for Overfall Dams
Model Studies of Imperial Dam and Desilting Works
Model Studies of All-American Canal Structures
Silt Movement in Colorado River

PART VII—CEMENT AND CONCRETE INVESTIGATIONS

Thermal Properties of Concrete
Investigation of Portland Cements
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Mass Concrete Investigations
Contraction Joint Grouting
Volume Changes in Mass Concrete
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