Duplicating line plates on text and figures

HYD-247

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES FOR SPILLWAY CHANNEL AT STEWART MOUNTAIN DAM-- SALT RIVER PROJECT, ARIZONA

Hydraulic Laboratory Report No. Hyd.-247

RESEARCH AND GEOLOGY DIVISION

BRANCH OF DESIGN AND CONSTRUCTION
DENVER, COLORADO

SEPTEMBER 13, 1948

FOREWORD

The hydraulic model studies of the spillway channel at Stewart Mountain Dam were conducted by the personnel of the Bureau of Reclamation, in the hydraulic laboratory on the campus of the Colorado State College of Agriculture and Mechanic Arts, at Fort Collins, Colorado. These studies were begun in May 1935 and completed in March 1936, and were initiated under the direction of E. W. Lane, but finished under J. E. Warnock.

These studies were made in the hydraulic laboratory, in conjunction with computations by Mr. Raymond A. Hill of Leeds, Hill, and Jewett, Los Angeles, California, and Mr. D. C. McConaughy of the Dams Division; however, such computations are not included in this report.

A report of these studies was begun in 1937 under the direction of J. W. Ball, and a preliminary draft was written by R. R. Buirgy. This report could not be completed at that time because of more urgent work in the laboratory.

CONTENTS

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Branch of Design and Construction
Reasearch and Geology Division
Denver, Colorado
September 13, 1948

Laboratory Report No. 247
Hydraulic Laboratory
Compiled by: F. C. Lowe
Reviewed by J. E. Warnock

Subject: Hydraulic Model Studies for Spillway Channel at Stewart Mountain Dam--Salt River Project, Arizona.

SUMMARY

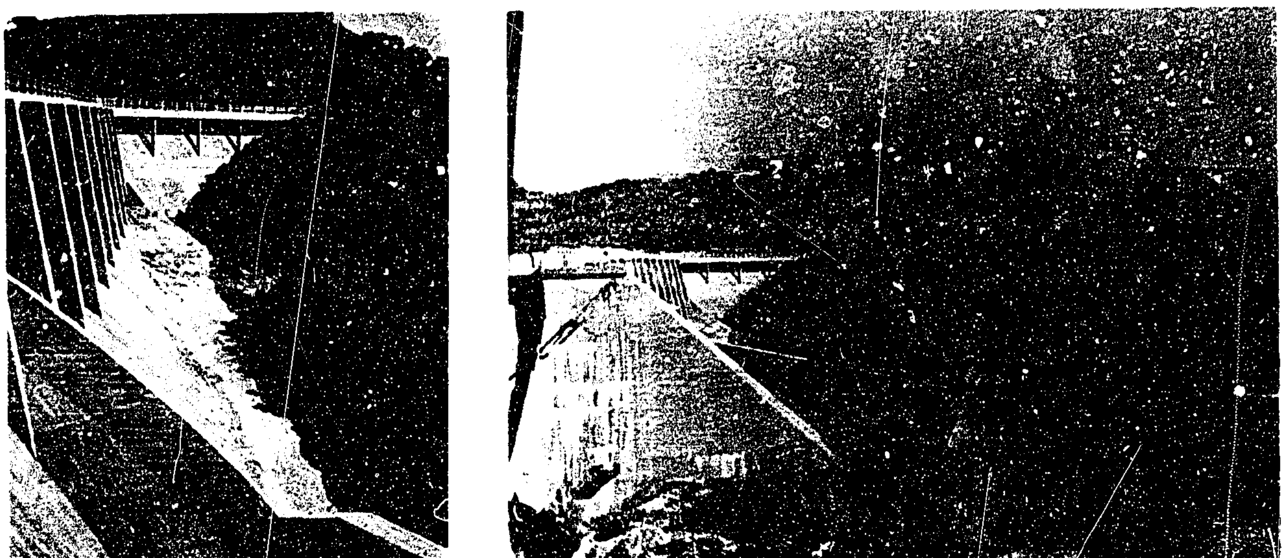
1. Extension of the Stewart Mountain Dam Spillway

In 1936 a concrete discharge channel, 265 feet wide and 450 feet long, was built at Stewart Mountain Dam to replace the natural sidechannel below the ogee crest of the spillway. (Figure 1.) This new channel was curved to conform to the terrain and superelevated to assure satisfactory operation at any flow condition. The design was tested and improved by hydraulic model studies as narrated in this report.

The Stewart Mountain Dam (Figure 2) is located about 30 miles east of Phoenix, Arizona. Completed in 1930 by the Salt River Valley Water Users Association, it was the third dam built in the Salt River Canyon below the Roosevelt Dam, and is downstream from Horse Mesa and Mormon Flat Dams. The Stewart Mountain Dam consists of a central arch between massive concrete abutments with gravity sections on each side to close the gap between the abutments and the sides of the canyon. The height from the stream bed to maximum water level is 119 feet, and a power head of 116 feet is available for operating a 17,500-horsepower hydroelectric plant. In addition to power production, the dam is used for irrigation storage, flood control, and to re-regulate releases from the dams above to fit irrigation requirements downstream.

Figure 1

A. The spillway channel built in 1936.



B. The original side-channel spillway.

SPILLWAY ALTERATIONS AT STEWART MOUNTAIN DAM

Figure 2

THE STEWART MOUNTAIN DAM

The spillway, located on the left gravity section of the dam, is a 267-foot ogee crest, at elevation 1506, placed between two training walls, which extend upwards to the walkway on top of the dam. Eight piers 3 feet wide are placed on this crest to form nine openings to accommodate nine radial gates 27 feet wide by 23 feet high. The water surface in the reservoir may thus be regulated between elevations 1506 and 1529, although during floods the water could rise to elevation 1535 before overtopping the dam. The topography downstream from the crest forms a natural sidechannel (Figure 1B), and when the dam was built, further provision for returning the water to the riverbed was considered unnecessary. A small wing wall was built to protect the east abutment of the main arch from scour by the spillway discharge.

At small discharges, there was a concentration of flow from this natural sidechannel around the east abutment of the arch indicating that large discharges would endanger the structure regardless of the protective wing wall. Any undercutting at the abutment would be especially serious, for the main arch of the dam was designed with high unit stresses and the safety of the dam depends upon the stability of the abutment. Moreover, a large discharge over the spillway would wash soil and rocks from the banks of the sidechannel into the riverbed, increasing the tailwater elevation at the powerhouse and reducing the available power head. It would even be possible to wash a bar across the river and flood the powerhouse.

2. The Proposed Spillway Channel

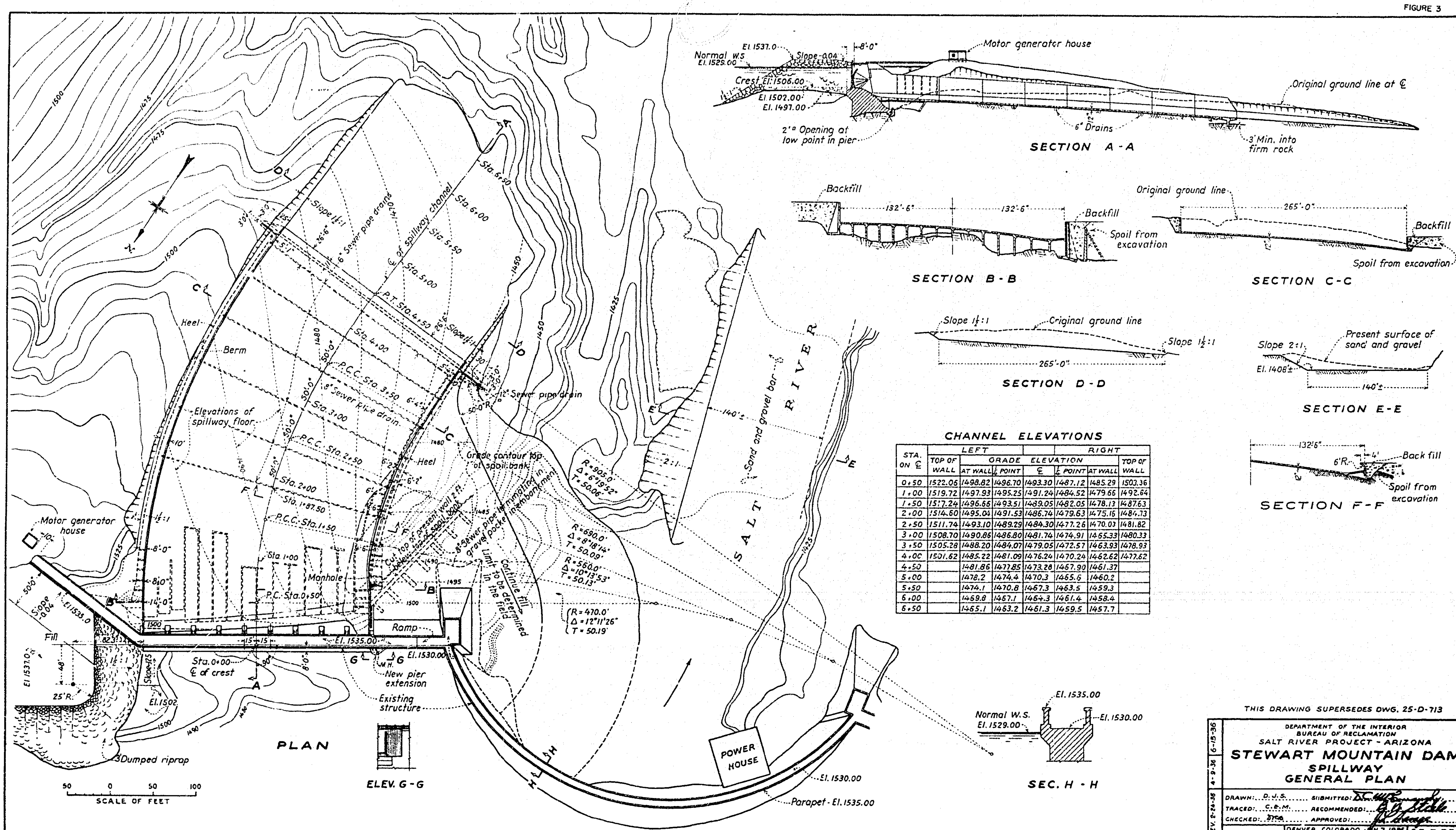
An improvement of the spillway channel was necessary to carry the water along the hillside and into the river at a point further downstream.

In the design of the new channel, the original sidechannel was filled, and a new channel, 265 feet wide, was extended 450 feet to a knoll over which the water could flow (Figure 3). It was contemplated that erosion of this knoll would be negligible once the overburden was removed, for the main rock was of granite in fairly good condition. To reach this knoll, the channel had to be turned through an arc of about 32°. At the same time, the drop was from elevation 1506 at the crest to approximately elevation 1475 at the end of the channel.

The channel was designed for a maximum flood of 140,000 second-feet, and for smaller discharges when the flow was unbalanced, such as operation with one gate only. The design was further complicated by the fact that the channel turned through an arc, thus the floor had to be superelevated to maintain a uniform water depth to avoid the construction of unreasonably high sidewalls. A channel, superelevated to handle a discharge of 140,000 second-feet, may be unsatisfactory if a single gate is opened, for the water may flow to the lower side of the superelevated curve, strike and overtop the lower sidewall. While it was possible to prepare an analytical design of a spillway channel with the degree of superelevation necessary for a maximum flood, the actual success of the spillway operation was uncertain because of the above conditions. A hydraulic model study was necessary to check and improve the proposed spillway channel.

The Stewart Mountain Spillway was completed in 1936, shortly after the tests were completed. Since then, to the knowledge of the laboratory personnel, no large discharges have passed through the spillway, the

FIGURE 3



CHANNEL ELEVATIONS

STA. ON ℄	LEFT				RIGHT		
	TOP OF WALL	GRADE ELEVATION AT WALL	GRADE ELEVATION ¼ POINT	GRADE ELEVATION ℄	GRADE ELEVATION ¼ POINT	GRADE ELEVATION AT WALL	TOP OF WALL
0+50	1522.06	1498.82	1496.70	1493.30	1487.12	1485.29	1503.36
1+00	1519.72	1497.93	1495.25	1491.24	1484.52	1479.66	1492.64
1+50	1517.24	1496.66	1493.51	1489.05	1482.05	1478.13	1487.63
2+00	1514.60	1495.04	1491.53	1486.74	1479.63	1475.16	1484.73
2+50	1511.74	1493.10	1489.29	1484.30	1477.26	1470.03	1481.82
3+00	1508.70	1490.86	1486.80	1481.74	1474.91	1465.33	1480.33
3+50	1505.28	1488.20	1484.07	1479.05	1472.57	1463.93	1478.93
4+00	1501.62	1485.22	1481.09	1476.24	1470.24	1462.62	1477.62
4+50		1481.86	1477.85	1473.28	1467.90	1461.37	
5+00		1478.2	1474.4	1470.3	1465.6	1460.2	
5+50		1474.1	1470.8	1467.3	1463.5	1459.3	
6+00		1469.8	1467.1	1464.3	1461.4	1458.4	
6+50		1465.1	1463.2	1461.3	1459.5	1457.7	

THIS DRAWING SUPERSEDES DWG. 25-D-713

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
SALT RIVER PROJECT - ARIZONA

STEWART MOUNTAIN DAM
SPILLWAY
GENERAL PLAN

DRAWN: D.J.S. SUBMITTED:
TRACED: C.E.M. RECOMMENDED:
CHECKED: APPROVED:
DENVER, COLORADO, JAN. 7, 1936 25-D-752

REV. 2-24-36 4-9-36 6-15-36

normal release of water being through the powerhouse. It was reported that a small discharge or leakage through the gates flowing down the right side of the channel has undercut the foundation to some extent at the downstream end, and that the granite knoll downstream from the channel has decomposed at the surface.

3. Summary of the Tests

The tests were made with two models, one built on a scale of 1:100 and the other a scale of 1:50, and the testing program may be considered as divided into two parts, the preliminary studies and the development of the final design.

The preliminary studies included four tests. The first test was on a 1:100 model of the original design with a curved superelevated channel in which general flow characteristics were observed. A gate operation procedure was studied, discharge capacity, water surface and velocities in the channel measured, and the erosion on the knoll downstream observed. The second test was on a 1:50 model of the crest without the channel downstream. In that test the crest itself was studied by measurements of pressure and discharge. The third test was on the 1:100 model with channel downstream alined on a compound curve in which the superelevation was too steep. The fourth test was with a similar channel, alined on a compound curve. In that test, the flow in the channel was studied, the hydraulic losses measured, and a false floor and fillet placed alongside the right, or inside, wall to reduce the water depth and a fin which formed at the wall.

The final design was developed on the 1:50 model. The channel was revised and alined on a spiral curve which became the final design.

Flow conditions were observed and the discharge capacity measured. The flow over the crest and into the channel was improved, the tests being concerned with the effects of wing walls at the upstream end of the piers, fillets in the pier offsets, a false floor in the channel alongside the right wall, and modification of the topography upstream from the crest and determination of the height of the channel walls. The development of the final design included (1) modification of the sloping bank on the left side of the approach channel to the spillway and to a streamlined-shaped wing wall at the head of the right end pier to decrease contraction of flow at the end piers, thereby increasing the discharge capacity, (2) fillets in pier offsets to smooth the flow, (3) a fillet on the channel floor alongside the right wall to decrease the depth of flow at that point, and (4) an overhanging lip, or sea wall at the top of this wall to divert splash at that wall back into the channel. The flow through the various gates was studied and a gate-operating schedule established. When the final design was developed, discharge measurements were made to establish rating curves for various conditions of operation. The final tests consisted in observation of splash over the channel walls, and discharge measurements to note the effect of the channel upon the capacity and measurements of the velocity distribution in the channel.

4. Results and Conclusions

While the original tests indicated the flow conditions which would occur in the prototype, it became apparent that the 1:100 model was too small for the development of the final design. This was especially noticeable in the velocity measurements in the channel which were inconsistent and the losses larger than anticipated. From this 1:100 model it was concluded that erosion of the overburden on the knoll downstream from the channel would have no serious consequences.

Subatmospheric pressure conditions would occur on the crest; however, no changes were made since this structure was already constructed.

When it was found that the superelevation of the channel was too great, it was realized the basic problem was to design a curved channel superelevated in such a manner as to keep the flow at a constant depth across the section and that it was necessary to account for frictional losses. Because of the friction factor, the model studies had to be interpreted with caution. If any large difference in relative friction were to exist between the model and prototype, the superelevation as indicated by the model would be inadequate. No slope or friction corrections were made in the design of the model, for the very uncertainties involved in friction corrections render them as unlikely to represent the true prototype conditions as does the undisturbed model. Moreover, the channel was comparatively short and the losses were not large.

The spiral channel of the 1:50 model was used for the final design. A number of minor alterations, to improve the flow appearance and the discharge capacity was necessary. The principal trouble was the flow was too deep at the right channel wall requiring an undesirably high wall. A high fin formed along this wall immediately downstream from the crest. These conditions were alleviated largely by placing a fillet on the channel floor along the wall, and constructing an overhanging lip or sea wall at the top. Splash still occurred in the rough flow downstream from crest to an estimated height 40 feet above the right wall. It is uncertain as to what the actual splash condition will be in the prototype structure.

Improvements to increase the discharge capacity were based upon reducing the contraction of flow at the piers to increase the effective crest length. A streamlined wing wall or pier nose was placed at the front of the right pier and tests on various types indicated this was probably an optimum design of wing wall considering effectiveness and construction costs. From the improvements made, it appeared that the maximum capacity was increased about two percent.

THE PRELIMINARY STUDIES

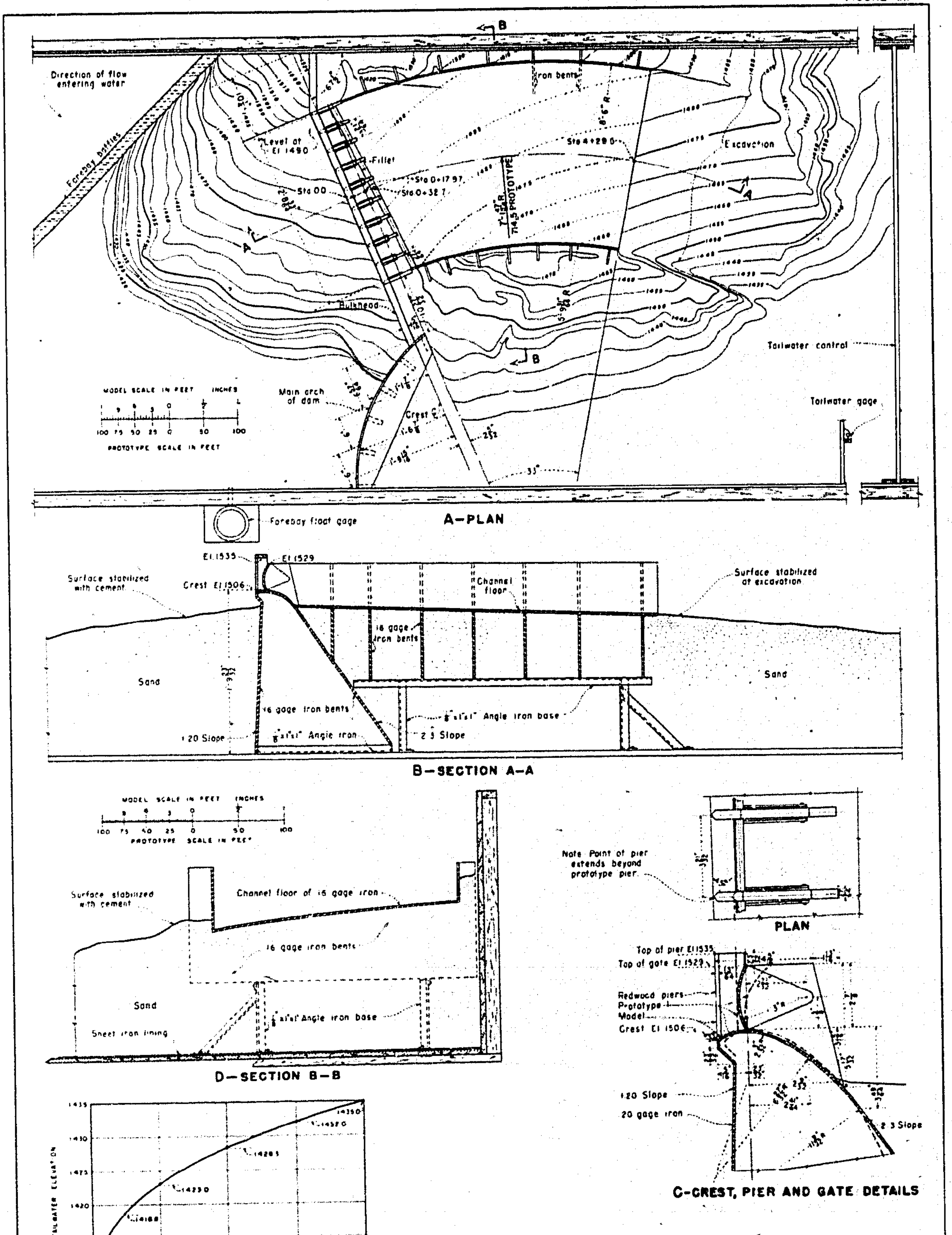
5. The 1:100 Model of the Original Design of Spillway Apron

A channel 271 feet wide and 429.5 feet long, beginning at the spillway crest, was contemplated in the original design of the spillway apron for Stewart Mountain Dam. The centerline of this channel was normal to the crest at Station 0+00 and extended in a straight line to Station 0+17.97. It then curved on a radius of 714.5 feet through an arc of 33° to Station 4+29.50. This channel was superelevated to give uniform depth of flow across the entire section with all gates opened an equal amount and with the water surface in the reservoir at elevation 1529. This superelevated section was faired into the ogee crest at approximately elevation 1492 at the top or outer edge of the curved channel and at elevation 1475 at the lower edge. As the channel dropped in elevation, the angle of superelevation increased to compensate for increased velocities.

A model of this original design, on a scale of 1:100, was built in the Colorado Agricultural College Experiment Station Hydraulic Laboratory, Fort Collins, Colorado. This first model included a portion of the main arch, the spillway crest, and the adjacent topography extending 325 feet upstream and 1,050 feet downstream from the crest (Figure 3.1). The crest, gates, channel, and main arch of the dam were built of sheet iron, the piers of redwood, and the topography shaped with sand. Aluminum paint on the structural parts protected them and improved the photographic qualities. Portions of the sand topography which represented solid rock was stabilized with cement, while that portion representing overburden and soil was of loose sand to show the effects of erosion. The forebay, representing a portion of the reservoir upstream from the crest, was sufficiently large to permit water to approach the crest in a uniform manner. To further assure this, a baffle was placed between the forebay and the source of supply in the flume upstream. The downstream section of the model was extended to include sufficient length of channel to determine whether material eroding from the knoll below the spillway would form a bar across the riverbed. The model terminated with an adjustable tailwater board.

Several features of this model were not similar to the prototype due to the lack of detailed information in the early stages of model studies. The spillway crest section upstream from the crest centerline was 3 feet too wide, the piers extended 7-1/2 feet too far upstream and the radius of the radial gates was 25 feet (prototype) instead of 20 feet. These differences, as shown by dotted line on Figure 3.1C were

FIGURE 3.1



SALT RIVER PROJECT ARIZONA
STEWART MOUNTAIN DAM
HYDRAULIC MODEL STUDIES OF SPILLWAY CHANNEL
1:100 MODEL OF ORIGINAL DESIGN

not significant in the preliminary studies because the flow in the channel downstream would not be materially affected. The height of the channel walls was not to scale, but was extended to elevation 1529, as the proper wall height was to be determined in the model studies.

Instrumentation of this model included a float gage in the forebay to measure the reservoir water surface, a point gage to measure tailwater, a movable point gage to observe the water surface on the crest and in the superelevated channel, and a single leg pitot tube to measure velocities. Discharge was measured by a weir in the flume upstream from the model. The movable point gage and pitot tube were mounted on a channel bar, which in turn was set on two horizontal, parallel bars fastened to the sides of the flume in which the model was placed.

6. Initial Studies on the 1:100 Model

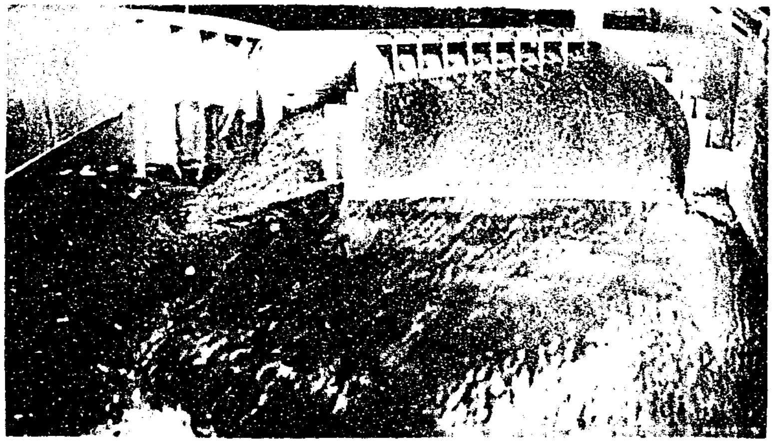
The tests on the 1:100 model included (1) a study of gate operation procedures with respect to flow conditions in the channel downstream, (2) determination of spillway capacity, (3) measurement of water surface and velocity distribution in the channel, (4) general observations of flow over the spillway crest, and (5) an estimate of the probable extent of erosion in the river channel downstream. In these tests, the water surface of the reservoir was held at or above elevation 1529, the top of the radial gates, for it was assumed that water would rise to this elevation before the gates would be opened. The degree of superelevation in the channel and this anticipated operating procedure was established for flow at velocities acquired with the water surface at elevation 1529.

Two methods of spillway gate operation were studied: (1) open all gates the same amount, and (2) open individual gates completely, the number depending upon the discharge. With all gates open an equal amount, the flow was comparatively uniform in the channel, regardless of discharge (Figure 4a). The only undesirable condition observed was the tendency to wash away the bank downstream from the right training wall. By operating individual gates, the flow through the channel appeared satisfactory with combinations of three, five, and seven gates open, although it was preferable to use the gates on the right end of the crest (Figure 4C). When gates on the left end were opened, some flow crossed the channel and piled against the right wall (Figure 4D).

With all gates wide open, the spillway discharged 100,000 second-feet when the water surface in the reservoir was at elevation 1529.7. As the discharge increased, the water surface continued to rise. At a discharge of 139,200 second-feet, the water reached the top of the parapet on the reservoir, elevation 1535. This was some 10,000 second-feet short of the design capacity of 150,000 second-feet. By operating individual gates with the water surface at elevation 1529, three gates completely opened would discharge approximately 25,000 second-feet, five gates 50,000, and seven gates 75,000 second-feet. All these measurements were qualitative and no discharge coefficients were determined in this test for the shape of the crest of the model was not similar to that of the prototype as previously mentioned; moreover, it was found that the 1:100 model was too small to obtain precise discharge measurements.

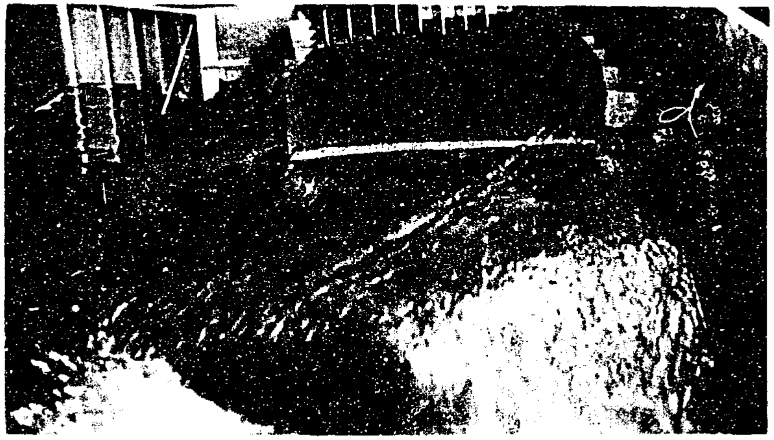
The tests included measurements of the water surface and velocity to ascertain the applicability of the theoretical design of the

Figure 4

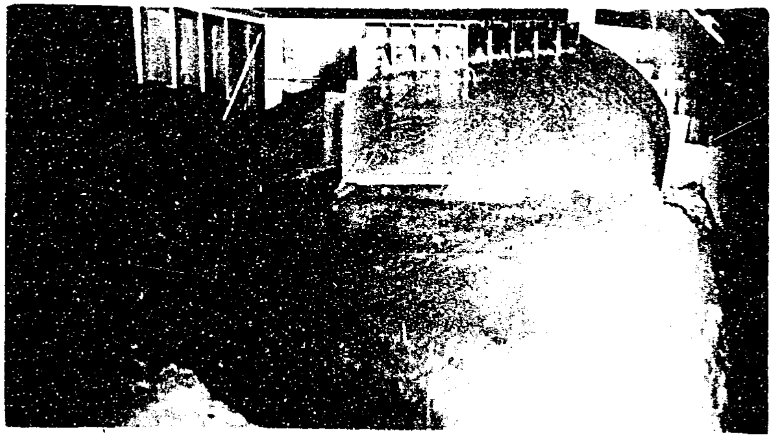


A. The model, all gates partially open. Discharge 75,000 c.f.s. Reservoir at elevation 1529.0.

B. Erosion of river bed after discharge at 139,200 c.f.s.



C. Five gates at right side open Discharge 50,000 c.f.s., Reservoir at elevation 1529.0.



D. Five gates at left side open. Discharge 50,000 c.f.s. Reservoir at elevation 1529.0. (Note water piling against right wall.)

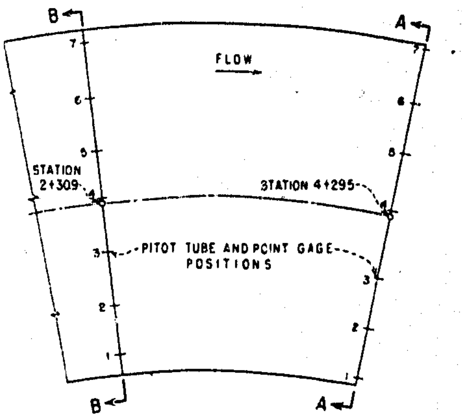
HYDRAULIC MODEL STUDIES OF STEWART MOUNTAIN DAM SPILLWAY
1:100 MODEL OF ORIGINAL DESIGN

superelevated channel.[1/] These measurements were made at two radial sections of the channel (Figure 5A), by placing the pitot tube and point gage at seven positions across each section (Figure 5B). Water surface profiles for discharges of 50,000, 100,000, and 139,200 second-feet are shown on Figure 5B. The depth of water is the minimum at the inner edge of the curve, the right side, increasing gradually to a maximum at the outer edge. This condition was theoretically correct because the velocities, a function of the energy head above the channel floor were less at the higher elevations at the outer edge of the curve and it follows that a greater depth is required at the outer edge to maintain uniform discharge across the section. The velocity distribution at the 14 positions is shown in Figure 5C for discharges of 25,000, 50,000, 75,000, 100,000, and 139,200 second-feet. While some variation exists, the velocities are, roughly, independent of discharge. Theoretically, this is correct except for a discharge of 139,200 second-feet when the water surface is at elevation 1535 instead of 1529. With an ideal, nonviscous fluid, and with the water surface in the reservoir at elevation 1529, the velocity at any point may be expressed as $\sqrt{2gh}$, where h is the drop from elevation 1529. Actually, frictional losses over the crest and down the channel reduced the effective head and velocity. By estimating such losses, the design section obtained curves for discharge

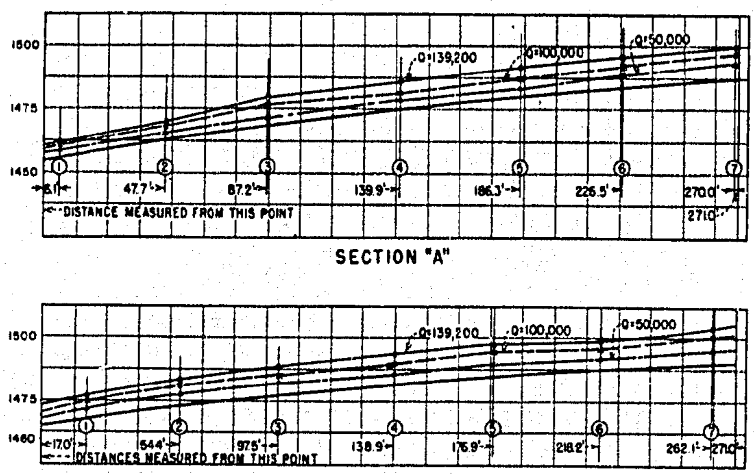
---

1/ For a review of the design of a superelevated spillway channel, see Laboratory Report Hyd. 74 "Flow of Water in Superelevated Channels at Velocities both Above and Below Critical," by T. G. Owen. See also "Design of Spiral Spillway Chutes," by Raymond A. Hill and D. C. McConaughy in Civil Engineering, November 1945, Volume 15, No. 11, Page 499.

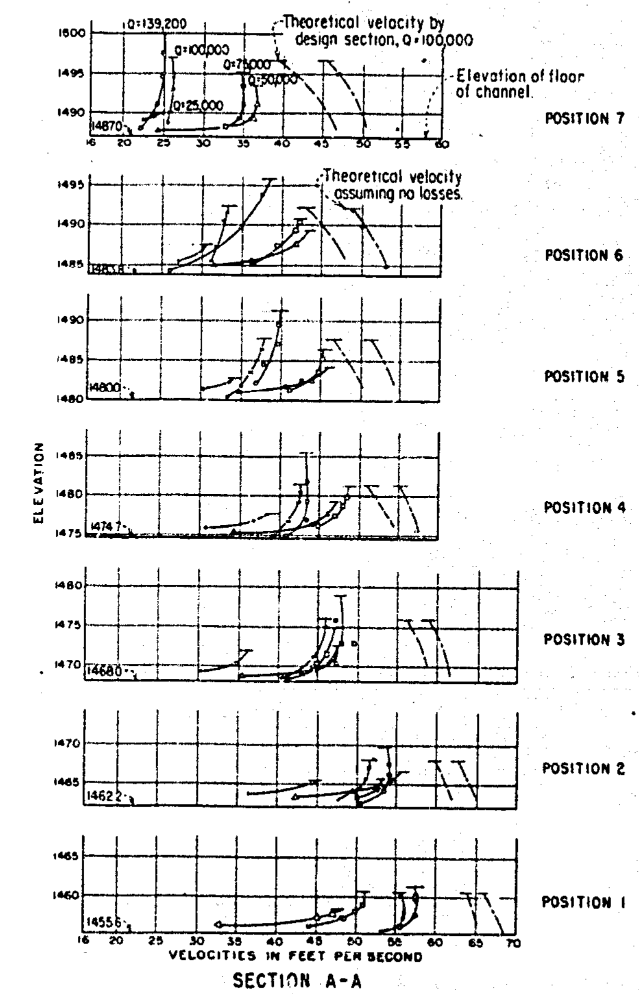
FIGURE 5



A. LOCATION OF SECTIONS A-A AND B-B IN CHANNEL



B. WATER SURFACES AT SECTIONS A-A AND B-B



C. RELATION OF VELOCITY TO DEPTH

NOTES

1. For discharges 25,000, 50,000, 75,000 and 100,000 C.F.S. water surface in reservoir at elevation 1529.00. For 139,200 C.F.S water surface at elevation 1535.00.
2. Water surface elevations obtained by point gage. Velocities by single leg pitot tube

SALT RIVER PROJECT - ARIZONA

STEWART MOUNTAIN DAM

HYDRAULIC MODEL STUDIES OF SPILLWAY CHANNEL

MEASUREMENTS OF WATER SURFACE AND VELOCITIES IN ORIGINAL 1:100 MODEL of 100,000 second-feet as shown in Section A-A, Figure 5C. The measured velocities follow the same general pattern as the computed velocities, but are less indicating greater losses in the model. An investigation suggested that the losses in the model were so large that they were questionable. In fact, it was believed that the model data should lie close to the computed values. Although the values obtained from model data for different discharges follow a logical pattern, they appear widely spread. The only logical explanation was that the model was so small that the frictional losses in it were comparatively large.

The flow over the crest contracted at the ends toward the center, rendering a portion of the spillway crest useless. If these entrance conditions could be improved, the discharge might be increased from a maximum of 139,200 second-feet to the desired 150,000 second-feet. However, any studies of this nature would have to be done on another model with the crest and piers properly built.

One important factor to be shown by this model was the probable erosion from the hillside to the riverbed downstream. In the prototype, the granite rock of the knoll, where the spillway discharges, was covered with an overburden of soil and loose rocks. A conflict of thought arose as to the manner of disposing of this overburden. If it were permitted to remain, it would eventually be washed into the riverbed. The cost of removal would not be increased, and this method would be desirable if a bar were not formed in the river downstream which would cause water to back into the powerhouse. The model was run at various discharges to ascertain the effect of erosion of the knoll upon the riverbed. The erosion patterns were similar, becoming more pronounced as the discharge increased. Figure 4B shows the erosion downstream after a discharge of 139,200 second-feet. From this test, it was concluded that a bar would not be formed which would seriously retard the flow of the river below the powerhouse.

7. Studies of Crest by 1:50 Model

A spillway capacity of 150,000 second-feet was desired, but the 1:100 model of the original design indicated that 139,200 second-feet was the maximum obtainable. Since this model was too small to permit precise measurements and the crest was not built correctly, the original 1:100 model was removed, and a 1:50 model of the spillway crest and approach channel only was installed (Figure 6A). The piers and topography upstream were included in this model, but the regulating gates, the spillway channel, and the topography downstream were not included. Such features were unnecessary, for the purpose of this model at this stage of investigations was to study the properties of the crest itself, the pressure distribution, the discharge coefficient, and the extent of contraction of flow at the end walls and around the piers. The omission of the nine radial gates was the equivalent of assuming that they would always be operated in a wide-open position.

Piezometers were installed at the center of the crest to measure pressures (Figure 6B). The piers were removed to eliminate any effect of contraction of flow. Pressure curves for discharges of 10,200, 26,100, 49,900, 73,600, 100,500, 125,500, 155,000, and 178,700 second-feet are shown on Figures 6C and 6D. These curves represent the elevation of the pressure on the crest compared with the elevation of the crest itself, and indicate subatmospheric pressure at discharges greater than

100,000 second-feet. Subatmospheric pressures on a crest of too great magnitude are not desirable because air might be indrawn periodically between the nappe of water and crest in such a manner as to cause vibration, or cavitation might occur with its attendant vibration and pitting. The necessity for alteration of the crest to eliminate negative pressure was not indicated as they were not in excess of 15 feet of water. Further investigation of pressures were not deemed necessary although unfavorable negative pressures were anticipated in the prototype not only with the gates fully open, but also with the gates partially open.

From the discharge coefficients obtained in this test, it was indicated that the capacity of the spillway would be 150,000 second-feet with the reservoir at elevation 1535 with the piers removed to eliminate the effect of contraction of flow. To study the crest further, calibrations were made over a range of heads and discharges, for the following: (1) without piers, (2) with piers, and (3) with piers and with upstream topography removed to change the approach conditions. Without the piers, the capacity of the spillway was 155,000 second-feet with the water surface at elevation 1533.6.

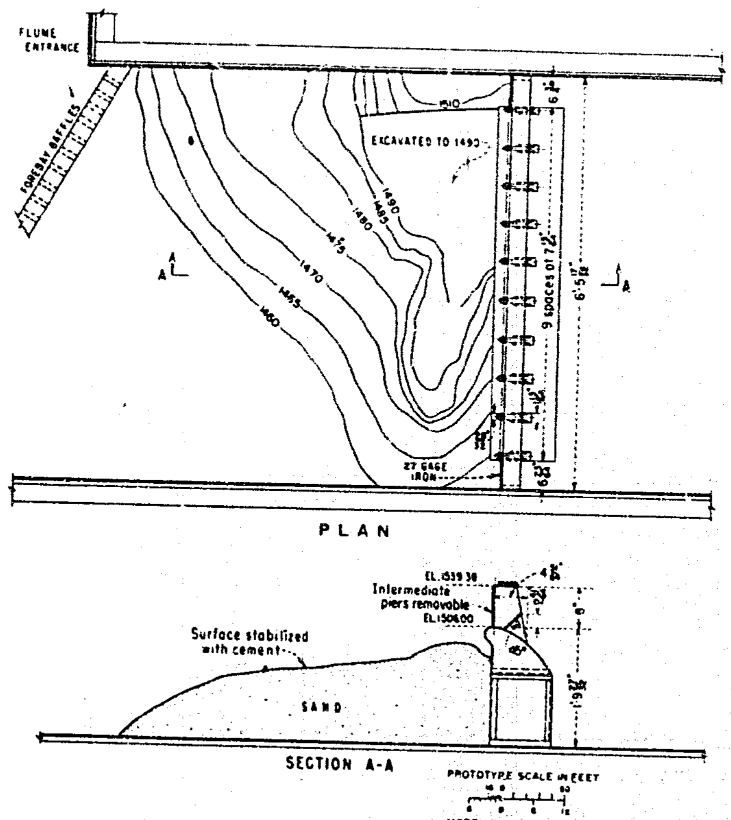
These figures have little significance unless the capacity is expressed as a coefficient independent of the crest length which may be compared with a similar coefficient with the piers in place.

For flow over a rectangular weir, similar to the Stewart Mountain Dam Spillway, the coefficient of discharge, C, may be expressed as

$C = \dfrac{Q}{LH^{3/2}}$ where Q = discharge, L = length of the crest, and H = head over the crest. In an ideal situation, this coefficient would be a constant, but in this case it increases with head (Figure 6E). Several factors cause this variation: The crest shape, the contraction of the water toward the center of the nappe away from the end walls or piers, the contraction of water away from the center piers, and the topography upstream from the crest. At large discharges, the water was observed to spring free of the end walls, both with and without the piers in place. This naturally reduced the effective crest length. It follows that a larger coefficient might be possible if this contraction were eliminated. The effect of the piers was to further reduce the effective crest length, and small coefficients were obtained with the piers in place, as shown in Figure 6E. The removal of topography upstream from the crest did not materially affect the discharge capacity. This suggested that a change of the upstream topography was not necessary.

8. Studies of the Revised 1:100 Model

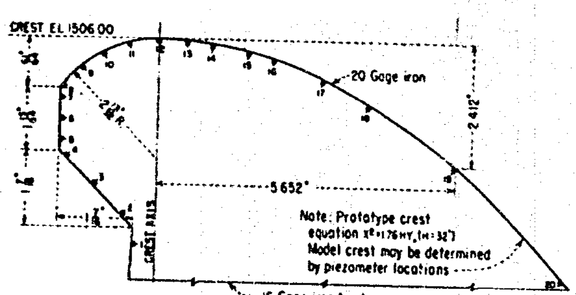
Upon completion of the crest studies, the 1:50 model was removed and replaced by a second 1:100 model of the entire spillway and adjacent topography. The crest was the same as used in the original 1:100 model, but the channel was revised. It was alined on a compound curve extending to Station 4 + 50, and set approximately five feet higher than the channel in the original design (Figures 7A, B, and C). Instead of extending the channel walls to the top of the gate, elevation 1529, as was done before, the right wall was only 14 feet above the channel floor and the left or outside wall extended to elevation 1522.28. The spillway crest, which was incorrectly built in the original 1:100 model, was not changed, but new piers, properly proportioned, were used (Figure 7D).



PLAN

SECTION A-A

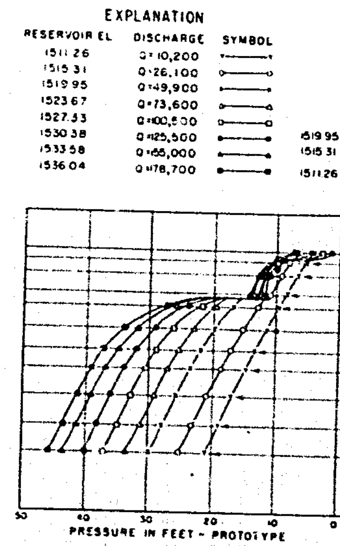
A-DETAILS OF MODEL



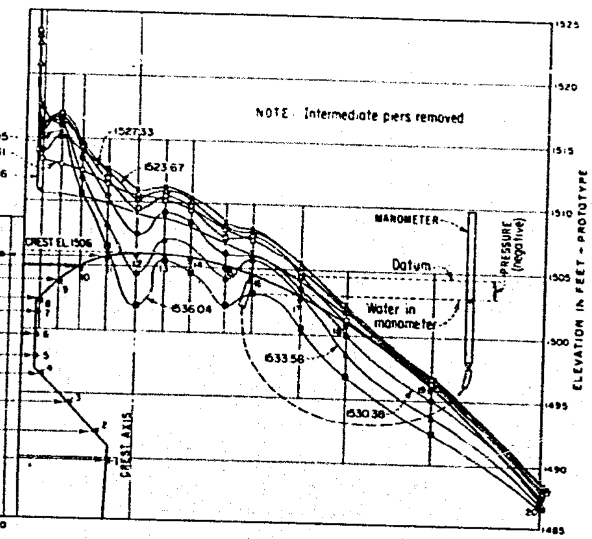
B-CREST DETAILS

PIEZOMETER LOCATIONS

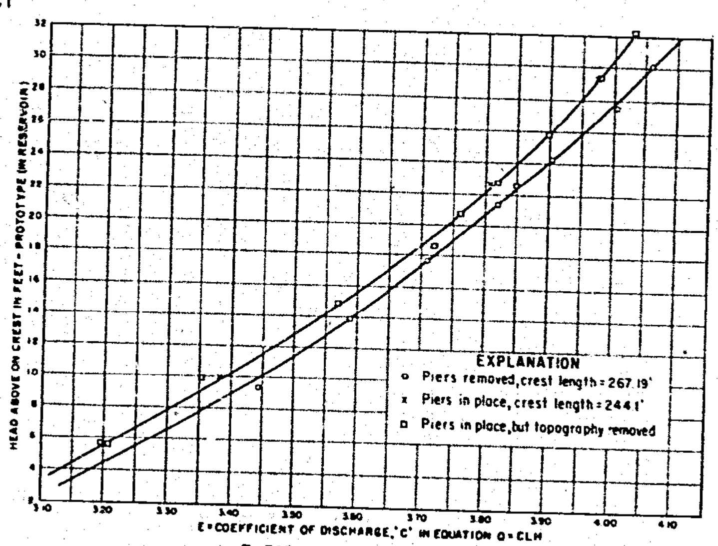
PIEZ. No.	HORIZ. DIST. FROM AXIS PROTO-FT.	HORIZ. DIST. FROM AXIS MODEL-IN.	ELEVATION PROTO FT.	DIST. BELOW CREST MODEL-INCHES
1	-2.05'	-0.492"	1489.96	3.850"
2	-3.03'	-0.727"	1492.03	3.353"
3	-5.24'	-1.257"	1494.24	2.822"
4	-7.45'	-1.788"	1496.45	2.292"
5	-8.00'	-1.920"	1497.84	1.958"
6	-8.00'	-1.920"	1499.66	1.522"
7	-8.00'	-1.920"	1501.48	1.085"
8	-7.62'	-1.828"	1502.42	0.859"
9	-6.19'	-1.485"	1503.82	0.523"
10	-4.37'	-1.049"	1504.93	0.257"
11	-2.41'	-0.578"	1505.71	0.069"
12	0.00'	0.000"	1506.00	0.000"
13	+2.15'	+0.516"	1505.92	0.019"
14	+4.23'	+1.015"	1505.68	0.077"
15	+7.09'	+1.702"	1505.06	0.226"
16	+9.12'	+2.189"	1504.48	0.364"
17	+13.05'	+3.132"	1502.91	0.741
18	+16.75'	+4.020"	1500.89	1.226
19	+23.55'	+5.652"	1495.95	2.412
20	+32.45'	+7.788"	1487.16	4.522



C-PRESSURE ON UPSTREAM FACE OF CREST



D-PRESSURE ON TOP OF CREST



E-DISCHARGE COEFFICIENTS

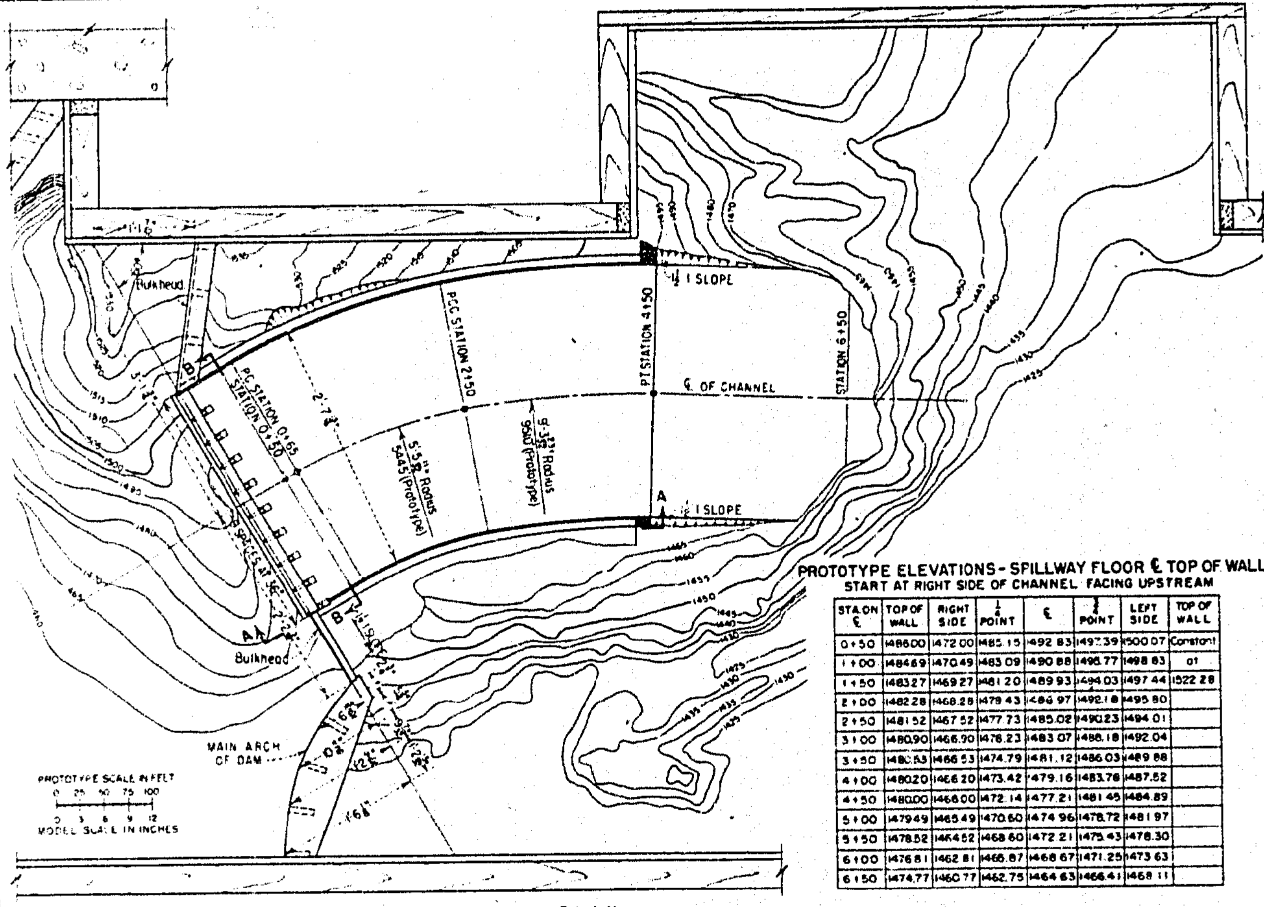
SALT RIVER PROJECT-ARIZONA

STEWART MOUNTAIN DAM

HYDRAULIC MODEL STUDIES OF SPILLWAY

TEST 2-CREST STUDIES ON 1:50 MODEL

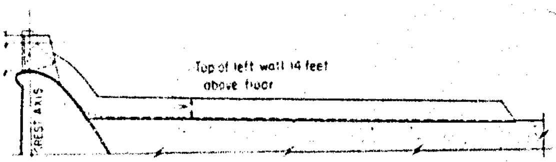
FIGURE 6



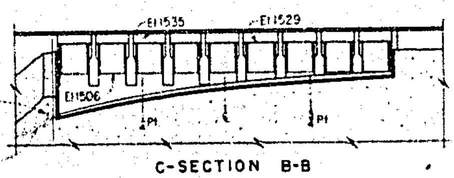
PROTOTYPE ELEVATIONS - SPILLWAY FLOOR ℄ TOP OF WALL
START AT RIGHT SIDE OF CHANNEL FACING UPSTREAM

STATION ℄	TOP OF WALL	RIGHT SIDE	¼ POINT	℄	¾ POINT	LEFT SIDE	TOP OF WALL
0+50	1486.00	1472.00	1485.15	1492.83	1497.39	1500.07	Constant
1+00	1484.69	1470.49	1483.09	1490.88	1496.77	1498.83	at
1+50	1483.27	1469.27	1481.20	1489.93	1494.03	1497.44	1522.28
2+00	1482.28	1468.28	1479.43	1486.97	1492.18	1495.80	
2+50	1481.52	1467.52	1477.73	1485.02	1490.23	1494.01	
3+00	1480.90	1466.90	1476.23	1483.07	1488.18	1492.04	
3+50	1480.53	1466.53	1474.79	1481.12	1486.03	1489.88	
4+00	1480.20	1466.20	1473.42	1479.16	1483.76	1487.52	
4+50	1480.00	1466.00	1472.14	1477.21	1481.45	1484.89	
5+00	1479.49	1465.49	1470.60	1474.96	1478.72	1481.97	
5+50	1478.52	1464.52	1468.60	1472.21	1475.43	1478.30	
6+00	1476.81	1462.81	1466.87	1468.67	1471.25	1473.63	
6+50	1474.77	1460.77	1462.75	1464.63	1466.41	1468.11	

A-PLAN

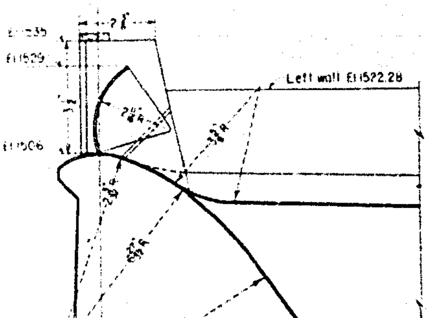


B-SECTION A-A

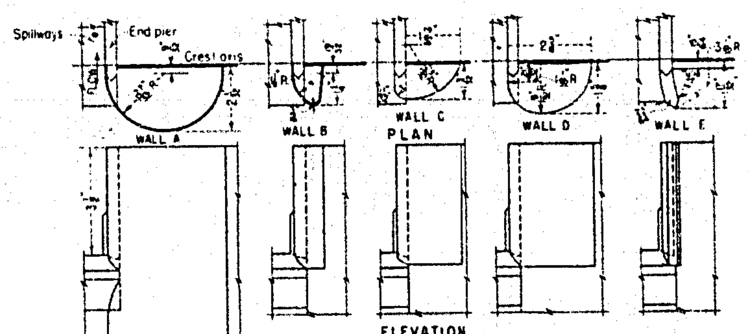


C-SECTION B-B

PROTOTYPE SCALE IN FEET
0 20 40 60 80 100
0 1 2 3 4 5 6 7 8 9 10 11 12
MODEL SCALE IN INCHES



D-CREST, PIER AND GATE DETAILS AT ℄



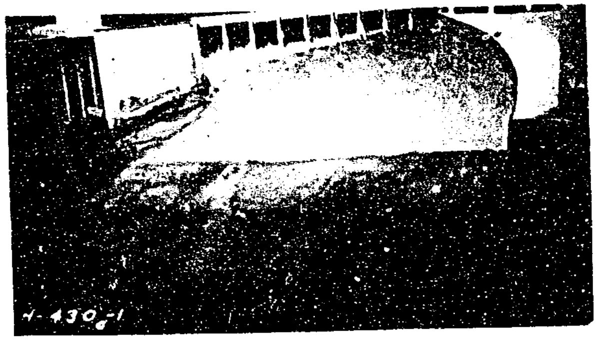
E-EXTENSIONS OR WING WALLS TO STREAMLINE PIERS

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
SALT RIVER PROJECT ARIZONA

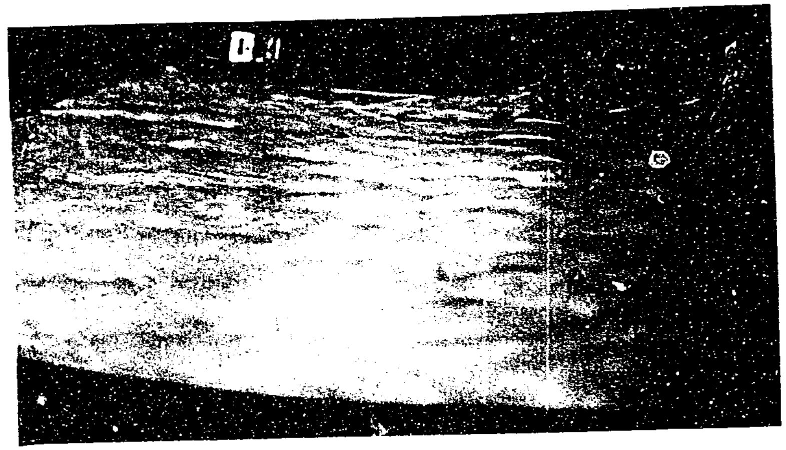
STEWART MOUNTAIN DAM
HYDRAULIC MODEL STUDIES OF SPILLWAY
TEST 3 - REVISED 1:100 MODEL

This was necessary to further study the undesirable contraction of flow at the end piers.

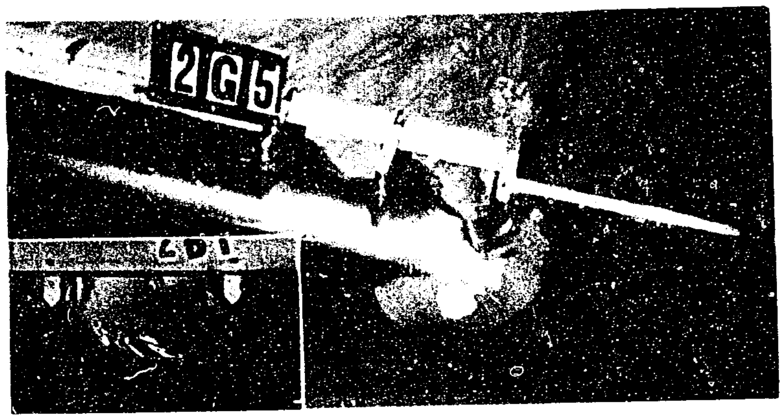
Tests on this model consisted of: (1) observations of flow in the channel at several discharges and various gate opening combinations, (2) observation of the erosion of the knoll upon which the channel discharged, (3) a comparison of the spillway capacity with that of the original 1:100 model to show the effect of raising the channel floor, and (4) installation of the several types of pier extensions to improve the flow passing over the crest. The model was operated at discharges of 25,000, 50,000,75,000,100,000,and 131,800 second-feet. The water surface was held at elevation 1529 for the smaller discharges and at 1535 for the maximum discharge. The tests were made with all gates equally open and with individual gates opened wide, the number depending upon the discharge. With all gates open an equal amount, there was a tendency for the water to shift to the right side of the channel overtopping the 14-foot wall at discharges greater than 75,000 second-feet (Figure 8A). This indicated that the degree of superelevation was too great for the curvature, and that the channel would not be satisfactory. Another undesirable condition was a high fin alongside the right wall immediately downstream from the crest (Figure 8B). Operation with individual gates wide open in groups of three, five, and seven gave flow patterns in the channel similar to those observed in the original design (Figures 4C and D) except for the tendency of the water to flow against the right wall. The flow was undesirable when the gates on the left end of the crest were opened. Water crossing the channel piled against and overtopped the right wall.



A. The model, all gates open. Discharge 100,000 c.f.s. (Note water overtopping right channel wall.)



B. Flow in channel below crest. Discharge 100,000 c.f.s. (Note fin at right channel wall.)



C. Contraction of flow at right wing wall. Discharge 140,000 second-feet.

D. Condition of flow with curved Wing Wall "C". Discharge 140,000 second-feet.

HYDRAULIC MODEL STUDIES OF STEWART MOUNTAIN DAM SPILLWAY

TEST 3 - REVISED 1:100 MODEL

This overtopping of the channel walls could not be permitted in the prototype because the earth fill alongside the wall would wash away and the channel foundations undermined. In the model, loose sand, representing overburden, was washed into the riverbed, not only from the knoll at the channel exit, but also alongside the channel wall almost back to the crest.

The discharge capacity was 131,800 second-feet with the water surface in the reservoir at elevation 1535, while the desired maximum of 150,000 second-feet was with the reservoir at elevation 1537.4. This was less than that of the original model, whose capacity was 139,200 second-feet.

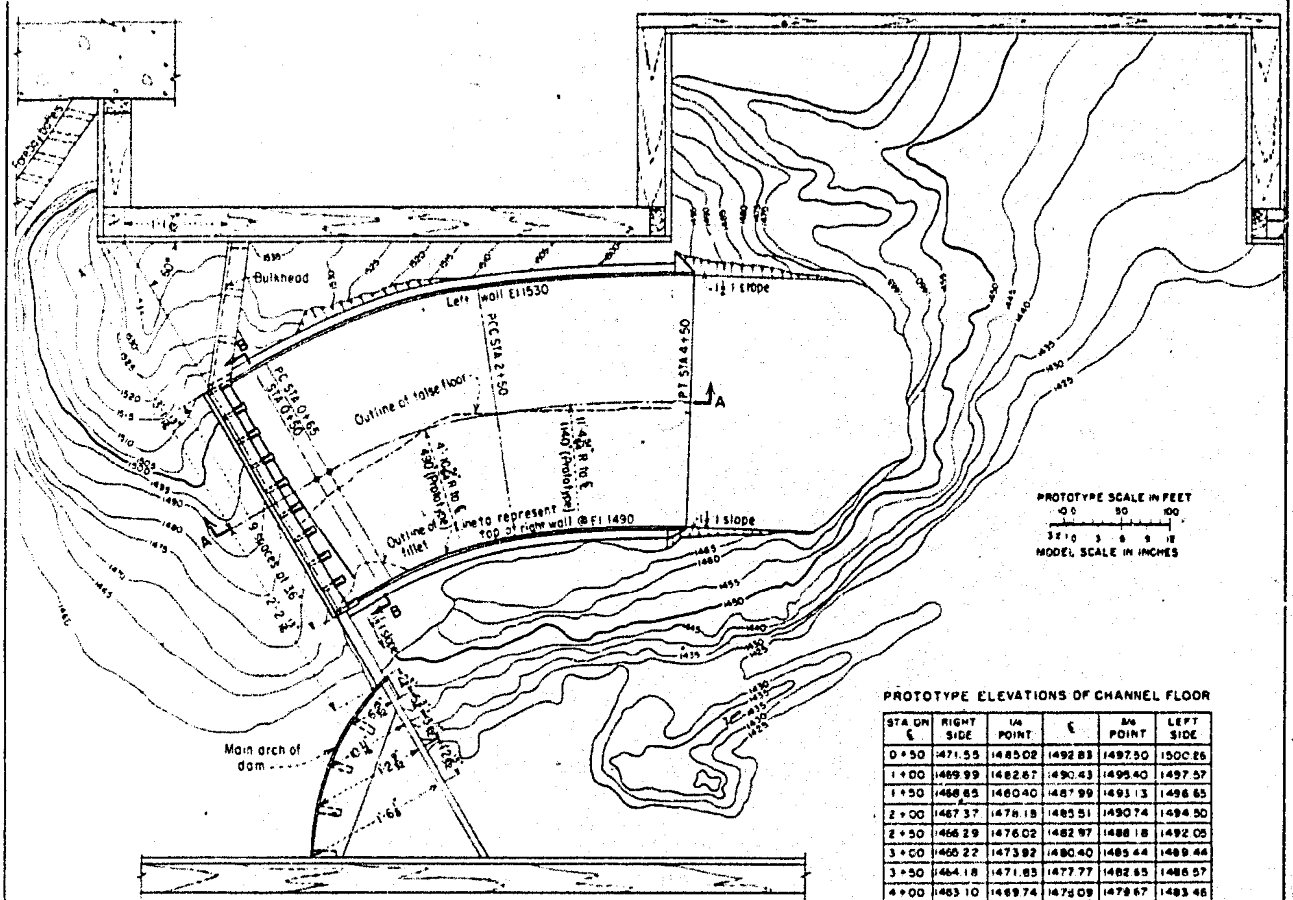
Tests to improve flow over the crest were necessary because water tended to spring away from the end piers, shortening the effective crest length (Figure 8C). This condition was especially bad at the right end pier, in the deeper water near the main arch of the dam. The solution appeared to be the use of some type of wing wall upstream from the end pier to suppress this contraction. To study the problem, five curved wing walls or pier extensions were proposed (Figure 7E). All of these designs appeared successful in that the contraction was eliminated to a large degree. Of the five tested, wall "C" was considered most practical (Figure 8D). However, the improvement obtained was not as much as was desired, and a further test was made by extending a wall upstream from the right end pier about 50 feet, set at an angle of about 15° with the pier centerline. With this arrangement, the water surface was almost level in passing through the gates.

9. Redesign of Channel of 1:100 Model

The channel used in the revised 1:100 model was unsatisfactory mainly because the degree of superelevation was so great that water piled against and overtopped the 14-foot right wall. The channel appeared to have been designed for higher velocities than those actually occurring. A new channel was built, on a compound curve as before, but using radii of 490 and 1,140 feet to the channel centerline (Figure 9A). The upstream end of the channel floor was approximately at the same elevation as that of the previous model, but the downstream end was approximately 4.5 feet lower. In this model, the top of the channel walls was at elevation 1530, but a line was drawn on the right wall at elevation 1490 to represent the proposed top of the wall. The short cutoff wall between the main arch and the crest was extended upstream until it was flush with the leading edge of the piers.

The model was operated by opening all gates an equal amount, and by opening individual gates with water surface in the reservoir held at elevation 1529 for all discharges less than 100,000 second-feet. With all gates opened an equal amount, the flow was spread uniformly across the channel, being somewhat deeper at the right side. Using individual gates, better flow conditions appeared when those on the right side were opened. When gates on the left side were opened the flow crossed the channel and piled against the right wall. An undesirable fin appeared alongside the right wall (Figure 8B).

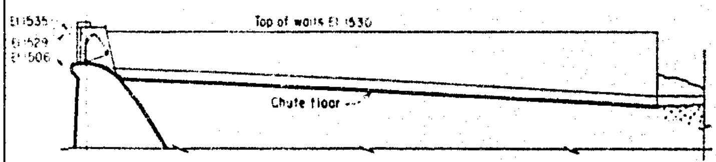
A study was made to estimate the losses in the channel by plotting a hydraulic gradient comparing the results with losses found in the original design. In the original design, the losses were calculated



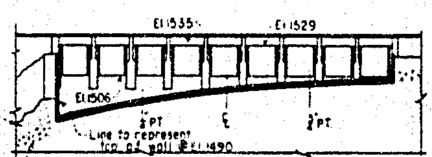
A-PLAN

PROTOTYPE ELEVATIONS OF CHANNEL FLOOR

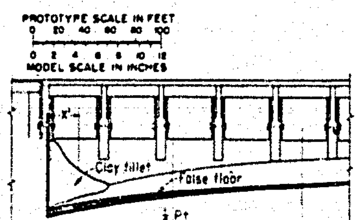
STA ON ℄	RIGHT SIDE	1/4 POINT	℄	3/4 POINT	LEFT SIDE
0+50	1471.55	1485.02	1492.83	1497.50	1500.26
1+00	1469.99	1482.67	1490.43	1495.40	1497.57
1+50	1468.65	1480.40	1487.99	1493.13	1496.65
2+00	1467.37	1478.18	1485.51	1490.74	1494.50
2+50	1466.29	1476.02	1482.97	1488.18	1492.05
3+00	1465.22	1473.92	1480.40	1485.44	1489.44
3+50	1464.18	1471.83	1477.77	1482.65	1486.57
4+00	1463.10	1469.74	1475.09	1479.67	1483.46
4+50	1462.04	1467.69	1472.37	1476.56	1480.14



B-SECTION A-A

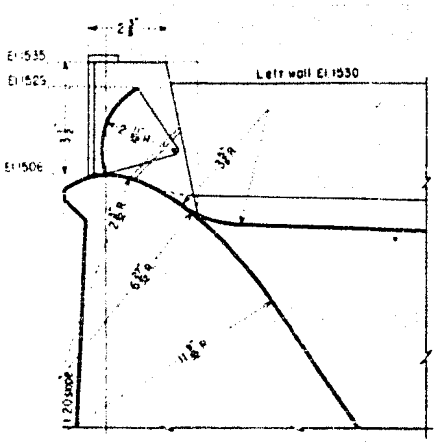


C-SECTION B-B



SECTION - STA. 2+50

E-FALSE FLOOR AND FILLET IN PLACE IN RIGHT HALF OF CHANNEL



D-CREST, PIER AND GATE DETAILS AT ℄

COORDINATES OF CLAY FILLET

STA	DIST. X'	ELEV	STA	DIST X'	ELEV
0+10	1.0	1504.7	0+50	0.0	1467.8
	3.0	1503.8		5.0	1483.0
	6.0	1503.0		10.0	1480.8
0+20	1.0	1501.8		20.0	1478.8
	5.0	1498.5		27.0	1478.9
	10.0	1496.9	0+65	0.0	1481.+
0+30	0.0	1498.1		5.0	1477.5
	6.0	1493.2		10.0	1476.2
	9.0	1490.8		20.0	1477.0
	15.0	1488.4	0+72	0.0	1480.0
	20.0	1487.8		5.0	1477.0
0+40	0.0	1492.6		10.0	1475.6
	4.0	1490.1		15.0	1476.7
	9.0	1485.0	0+86	0.0	1476.8
	15.0	1482.8		6.0	1476.3
	21.0	1481.3		10.0	[illegible]
	27.0	1480.6		20.0	1476.4
	30.0	1480.9		30.0	1478.3

ELEVATIONS OF CHANNEL FLOOR MODIFIED BY CLAY FILLETS

STA ON ℄	RIGHT SIDE	1/4 POINT	℄	3/4 POINT	LEFT SIDE
0+50	1474.20	1485.85	1492.75	1497.50	1500.26
1+00	1472.65	1484.25	1490.43	1495.40	1498.57
1+50	1471.31	1482.04	1487.99	1493.13	1496.65
2+00	1470.05	1479.45	1485.51	1490.74	1494.50
2+50	1468.95	1478.25	1482.97	1498.18	1492.05
3+00	1467.88	1474.55	1480.40	1485.44	1489.44
3+50	1466.84	1472.95	1477.77	1482.65	1486.57
4+00	1465.76	1471.15	1475.65	1479.67	1483.46
4+50	1464.70	1469.05	1472.37	1476.56	1480.14

SALT RIVER PROJECT - ARIZONA

STEWART MOUNTAIN DAM

HYDRAULIC MODEL STUDIES OF SPILLWAY CHANNEL
TEST 4: REDESIGN OF CHANNEL - 1:100 MODEL

from velocities determined by pitot-tube measurements. The losses to a station in the channel in this study were expressed as the difference between the water surface elevation in the reservoir and the elevation of the energy head above the station considered. Since the energy head is the depth of water plus the velocity head, $\frac{V^2}{2g}$, the problem resolves itself into a measurement of the depth of water in the channel, and determination of velocity.

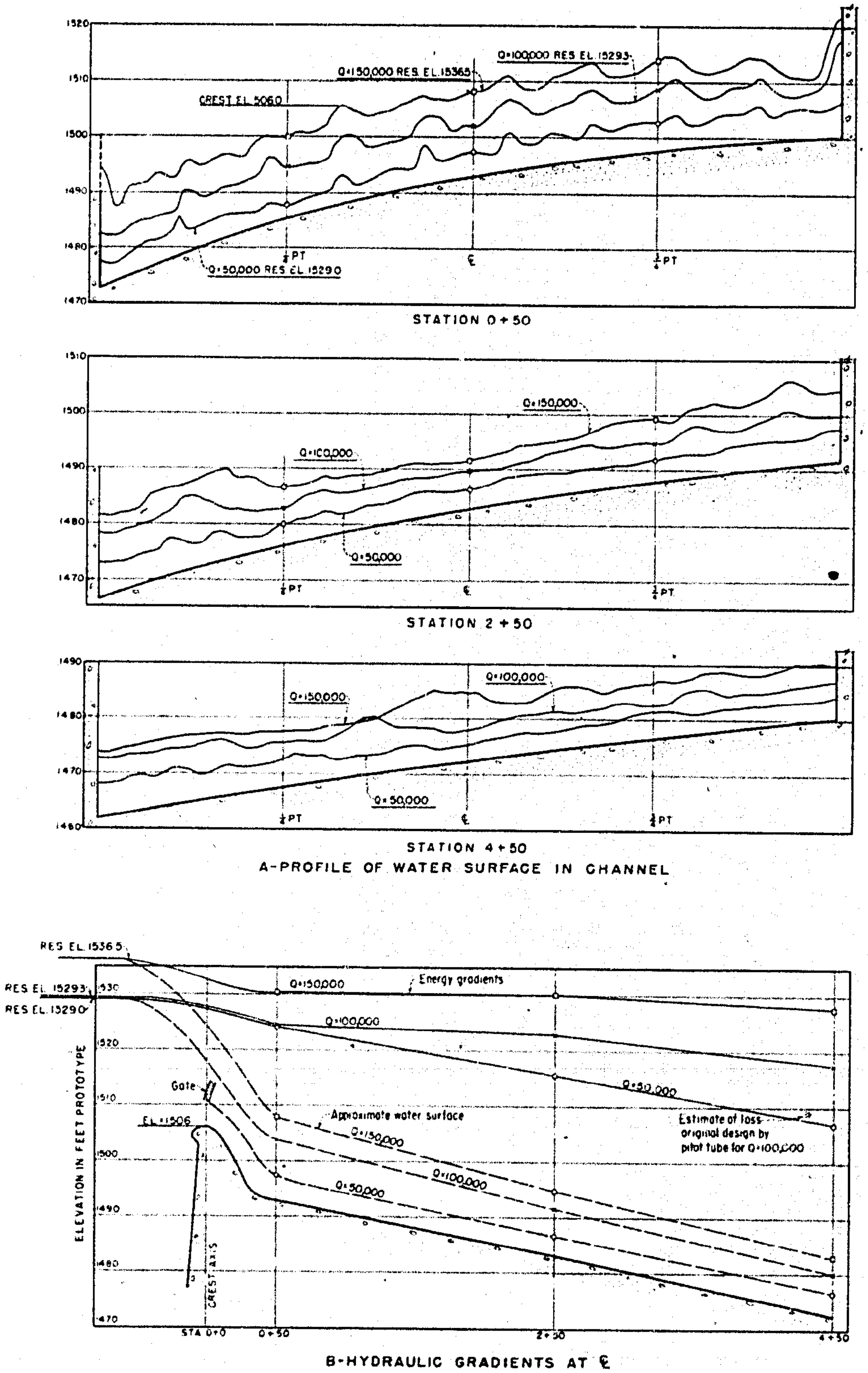
Stations 0+50, 2+50, and 4+50 were selected and the gradient assumed to lie along the centerline of the channel. The water surface was profiled across the channel with a point gage for discharges of 50,000, 100,000, and 150,000 second-feet (Figure 10A). The area was calculated by integration of the depths across the section, and the average velocity obtained by dividing the known discharge by the area. The gradients of Figure 10B were based upon the velocity head of these average velocities. While the results are approximate, the losses show a reasonable relation with discharge, in that greater loss occurred in the channel when there was less discharge. In contrast, the test on the original design by pitot-tube measurements was quite inconsistent in this respect (Figure 5).

The reservoir elevation was determined with the gates raised, at discharges of 25,000, 50,000,75,000, 100,000, 125,000 second-feet. respectively, and a capacity of 139,000 second-feet at elevation 1535 was found by interpolation. This was essentially the same discharge as was found on the original design.

10. The Use of False Floor and Fillet to Improve Flow

In designing a superelevated spillway channel, the depth of flow at the lower side or inside of the curve should be less than that at the

FIGURE 10



NOTE

1-Hydraulic gradients = depth + $\frac{V^2}{2g}$

Where V = average velocity as determined by $\frac{\text{discharge}}{\text{water area under profile}}$

2-For plan of channel see Figure 9.

SALT RIVER PROJECT-ARIZONA

STEWART MOUNTAIN DAM

HYDRAULIC MODEL STUDIES OF SPILLWAY

TEST 4-HYDRAULIC GRADIENTS BASED ON AVERAGE VELOCITIES outer side. Since the depth of flow was greater in the lower or right side of the channel of the model (Figure 10A), it indicated that the degree of superelevation was too great. The channel was modified by constructing a false floor on the right side (Figure 9E). At the same time, a clay fillet was built against the right wall to eliminate the undesirable fin previously described (Figures 8B, 9A, and 9E). The improvements with these modifications were substantial because the depth of flow was uniform across the channel, and the fillet materially reduced the size of the fin.

This test concluded the series of studies with the 1:100 model. The development of the final design was made with a model to a 1:50 scale.

DEVELOPMENT OF THE FINAL DESIGN

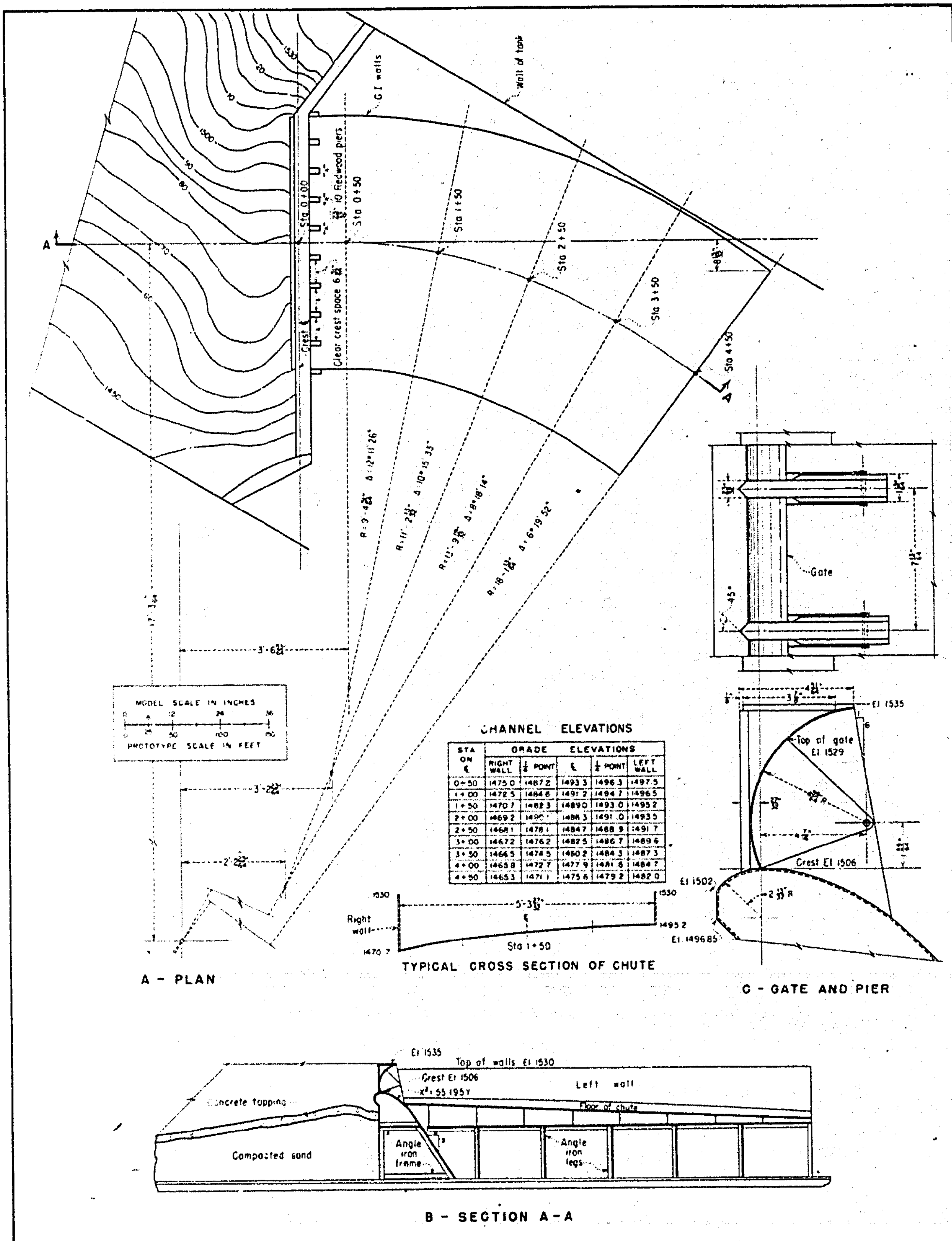
11. Design and Construction of the 1:50 Model

The final design studies of the Stewart Mountain Spillway were made on a 1:50 model. This larger scale was necessary because the 1:100 model was too small to study detailed improvements and to obtain sufficiently accurate discharge measurements required for the final design. This 1:50 model was built similar to the former 1:100, and included a portion of topography upstream, the crest, piers, gates, and channel (Figure 11). The knoll and riverbed downstream from the chute were omitted. This omission was not significant in that flow over this knoll was not to be studied, and its presence or absence would not affect the flow in the channel which was at supercritical velocities. Topography upstream from the crest was made of sand stabilized with cement. The crest was the same as used in Test 2 of the preliminary design, but with piers and gates added.

Two channels were contemplated, first, the same design as the 1:100 model of Test 4 alined on a compound curve having radii of 490 and 1,140 feet, and second, a channel alined on a spiral curve with radii of 470, 560, 690, and 905 feet (Figure 11A). The channel similar to the 1:100 model, was not built because of the necessity of proceeding with studies of the final design. In view of the fact that the final design was to be a channel alined on a spiral, the only value of tests on this first channel would be to compare the flow with that of the smaller 1:100 model. Even then, such a comparison would be of little value since the roughness factor would not change as the same materials were used in both models. In all, 36 individual tests were made on this model of the spiraled channel.

12. General Performance of Model and Discharge Capacity

The model was studied in Tests 1, 2, and 3 without any refinements, with the crest section as it existed in the field, and with the spiral channel attached (Figure 11). Test 1 consisted of three runs at discharges 50,000, 100,000, and approximately 139,000 second-feet with the gates opened partially at the lower discharge to hold the water surface at elevation 1529, and opened wide at the larger discharges. The flow appeared to spread uniformly across the channel, indicating that it was correctly designed. There was some splash against the right wall downstream from the crest in a manner similar to that of the 1:100 model (Figure 8B). Flow over the crest was similar to that of the 1:100 model, Figure 8C, contracting at the ends and reducing the effective length of the crest.



CHANNEL ELEVATIONS

STA ON ℄	GRADE ELEVATIONS				
	RIGHT WALL	¼ POINT	℄	¼ POINT	LEFT WALL
0+50	1475.0	1487.2	1493.3	1496.3	1497.5
1+00	1472.5	1484.6	1491.2	1494.7	1496.5
1+50	1470.7	1482.3	1489.0	1493.0	1495.2
2+00	1469.2	[illegible]	1486.3	1491.0	1493.5
2+50	1468.1	1478.1	1484.7	1488.9	1491.7
3+00	1467.2	1476.2	1482.5	1486.7	1489.6
3+50	1466.5	1474.5	1480.2	1484.3	1487.3
4+00	1465.8	1472.7	1477.9	1481.6	1484.7
4+50	1465.3	1471.1	1475.6	1479.2	1482.0

SALT RIVER PROJECT - ARIZONA

STEWART MOUNTAIN DAM

HYDRAULIC MODEL STUDIES OF SPILLWAY CHANNEL
ORIGINAL DESIGN OF 1:50 MODEL

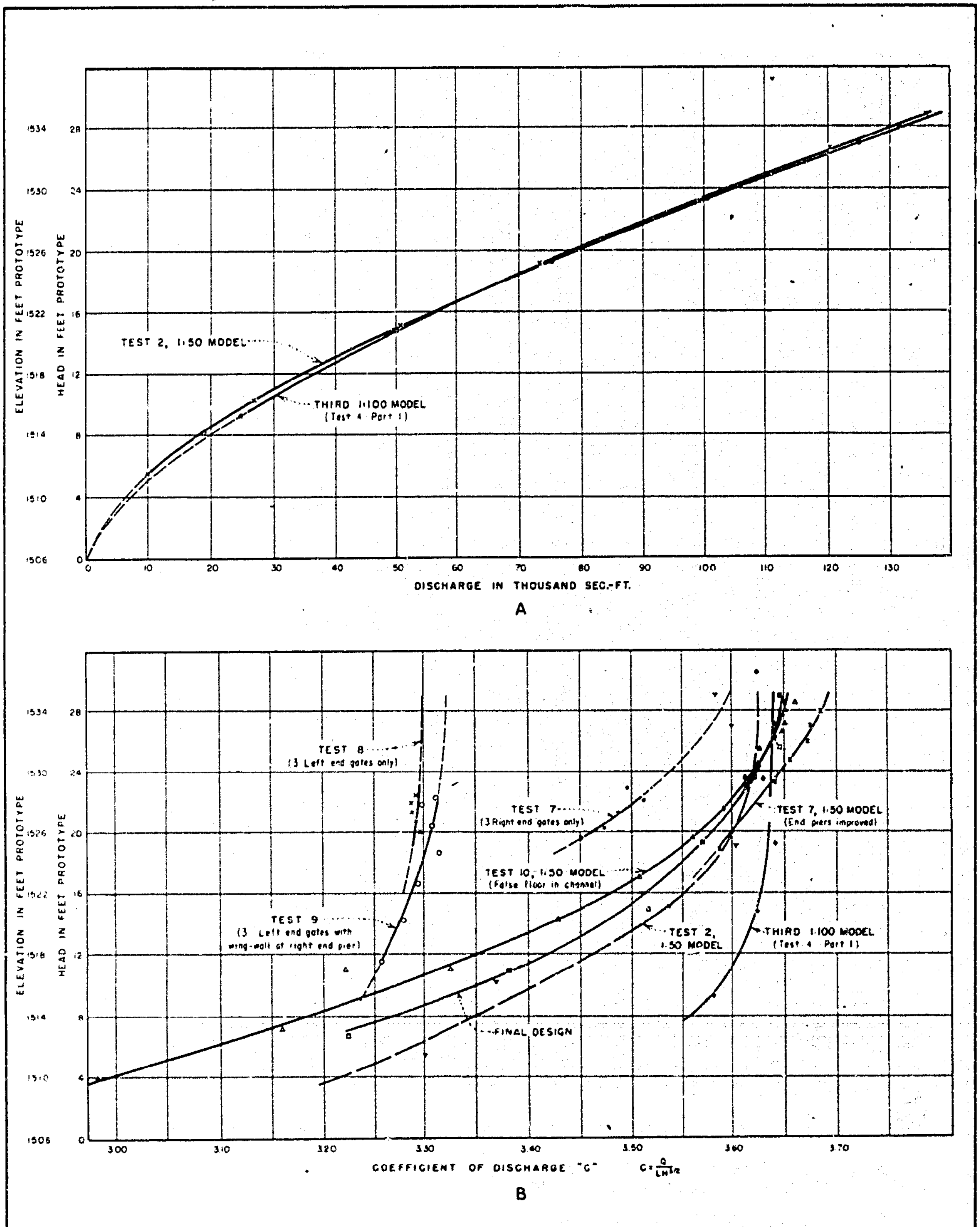
In Test 2, discharge measurements were made to compare the spillway capacity with that of the Test 4 on the 1:100 model, and to provide a basis of comparison for later tests after the flow over the crest was improved. These measurements were made with all gates wide open, and the discharges ranged from 12,500 second-feet to 136,000 second-feet, the maximum capacity with the water surface at elevation 1535. The most significant aspect of this test was that the capacity of the 1:50 model was less than that of Test 4 on the 1:100 model, but the comparison was so close when presented as head-discharge curves that the differences were not significant (Figure 12A). A more desirable comparison was to express the capacities as a coefficient of discharge, as shown by curves of Figure 12B.

Test 3 was made with the gates removed to eliminate the effect of the sheet metal gate arms. This resulted in an increase in the spillway capacity to approximately 138,000 second-feet, but this test was an impossible condition for even with the gates completely raised and out of the water, the radial arms were still partially submerged in the stream. The fact that the resistance of these thin arms changed the capacity a measurable amount indicated that minor changes could be made to improve the flow over the crest.

13. Improvement of Flow Over Crest and Into Channel

The undesirable aspects of flow over the crest observed in the 1:100 models were present in the 1:50 model. There was a definite contraction at the end piers, similar to Figure 8C, and a high fin at the right wall downstream from the crest (Figure 8B). Elimination of this

FIGURE 12



SALT RIVER PROJECT - ARIZONA

STEWART MOUNTAIN DAM

HYDRAULIC MODEL STUDIES OF SPILLWAY CHANNEL

DISCHARGE AND COEFFICIENT CURVES

contraction would increase the discharge capacity. If the high fin at the right wall were eliminated, the height of the channel wall in that region could be materially reduced. Tests 4, 5, 6, and 7 considered means of improving the flow over the crest and into the channel.

In Test 4, 1-inch radius wing walls (50 inches, prototype) were placed at each end pier tangent to the 45° nose of the piers, as shown in Figure 13A. Although satisfactory at small discharges, the wing wall on the right end pier was completely inadequate at larger discharges. In Test 5, 3/4-inch radius wing walls (37.5 inches, prototype) were placed at each end pier tangent to the inside face of the end piers. These wing walls were too small to improve the flow especially at the right end pier. It was concluded that a larger wing wall was needed at the right end pier, but not at the left end pier.

In Test 6, a more general study was made by including four alterations, or improvements to the model: (1) a 2-1/4-inch radius wing wall (9.39 feet, prototype) at the right end pier (Figure 13A); (2) a clay fillet on the right end pier between the nose of the pier and the offset downstream (Figures 13A and B); (3) a false floor about 50 feet wide at the right wall extending from Station 0+10 to Station 2+50, (Figure 13C); and (4) a piece of sheet metal placed upstream from the left end pier to represent a modification of the natural topography upstream from the crest to a slope of 1-1/2:1 (Figure 13D). These alterations considerably improved the flow over the crest and into the channel. The large wing wall on the right end pier, and modified topography at the left end pier held the water surface at both ends of the crest nearly level with contraction of flow practically eliminated. The clay fillet on the right end pier streamlined the pier wall, tending to hold the water against the wall and submerge the objectionable fin downstream from the crest.

To ascertain the probable height of the right wall, point gage readings were taken to establish the water surface on this wall, using a discharge of 132,000 second-feet (Figure 13E).

The principal objection to this arrangement was the large circular wing wall at the right end pier, which was extended below the crest approximately 40 feet (prototype) to the ground surface. Therefore, Test 7 considered a more economical type of wing wall (Figures 13A and B), which extended only a short distance below the crest. With this wing wall, the flow was nearly the same as before, and the design was considered tentatively satisfactory.

14. Spillway Capacity

In Test 7, six runs were made with the gates wide open with the discharge varied from 71,700 second-feet to 131,900 second-feet to obtain a coefficient curve. The spillway capacity was materially increased such that a flow of 140,100 second-feet could be obtained with the water surface in the reservoir at elevation 1535 (Figure 12B). It was, therefore, concluded that the proposed spillway alterations were worthwhile.

15. Comparison of Flow Through End Gates

A wing wall was placed at the right end pier, while the only alteration at the left end pier was to grade the ground surface upstream on a 1-1/2:1 slope. Since the approach conditions to the spillway were not symmetrical, it was desirable to know the comparative discharge next to the right and left end piers. To do this, Test 7 included five runs to measure discharge with the three right gates open, and Test 8 included five similar runs to measure discharge with the three left gates open. In Test 8, the flow spread across the channel to such an extent that it was believed that a fair test could be obtained only if it were confined to the left third of the channel. This was accomplished by placing a row of bricks in the channel (Figure 13C).

The procedure of the tests was to hold the gates open and measure discharge at various water surface elevations in the reservoir. Discharge coefficients were then computed in the same manner as with all gates open, and are comparable because the coefficient of discharge is theoretically independent of crest length. However, as shown by curves on Figure 12B, discharge coefficients for three gates only are less than coefficients with all gates open. This was anticipated because with three gates open there exists a contraction not only at the end pier, but also at the fourth pier and this contraction at the fourth pier reduces the effective crest length.

More significant, however, was the fact that the coefficients for the three left gates were materially less than those for the three right gates. This difference was caused by three factors: (1) the lack of a wing wall at the left end pier; (2) the ground surface upstream was close to the crest near the left end pier, but approximately 40 feet below the crest near the right end pier; and (3) in the superelevated channel downstream, the drop from the crest to the channel was less at the left side, and any tendency for water in the channel to submerge the crest would be prevalent at that side. While all three factors contributed to the unsymmetrical flow over the spillway, it was not certain which was more important, therefore, tests were made to establish this point.

16. Use of Wing Wall on Left End Pier

Without any other changes on the model, a wing wall having a radius of 5.13 inches (21.35 feet, prototype) was placed on the left end pier to establish the effect of this factor on discharge (Figure 13D). The runs were made with three left gates open in the same manner as in Test 8. The coefficients indicated that the wing wall increased the discharge capacity a small amount (Figure 12B), but not sufficient to warrant the construction in the prototype.

17. Effect of Training Wall in Channel

In tests to measure flow through the three left end gates, a row of bricks were used to represent a training wall to keep the flow in the left third of the channel, since water passing over the crest tended to spread across the channel. To show the effect of this training wall, a run was made in Test 8 in which the water surface below the centerline of the first gate was point gaged, with and without the training wall. No significant differences could be observed at least 100 feet downstream. The flow appeared similar in Test 9, where it was further observed that when the training wall was taken from the channel, there was no change in the reservoir water surface. This would indicate that the training wall had no effect upon the discharge.

18. Effect of Wing Walls on Water Surface at Right Channel Wall

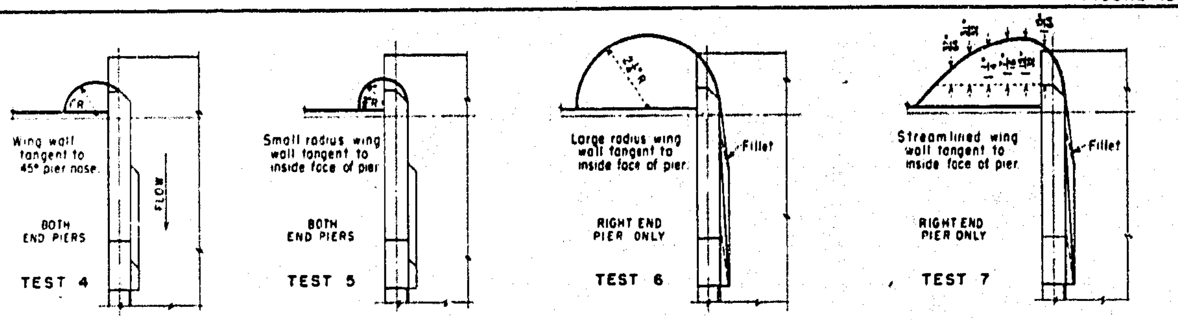
The water surface along the right channel wall was point gaged in Test 6 (Figure 13E). In that test, there was a circular wing wall at the right end pier, and no wing wall on the left end pier. Later, the wing wall at the right end pier was changed to a streamline shape, Test 7, and a temporary wing wall was placed at the left end pier, Test 9. In Test 9, the water surface along the right channel wall was measured to observe any effects caused by changing the wing walls. All gates were open and the reservoir was held at elevation 1533.88, virtually the same as in Test 6. The water surface was nearly identical to that in Test 6, demonstrating that the different wing wall arrangement had no appreciable effect upon the flow in the channel.

19. Revision to False Floor

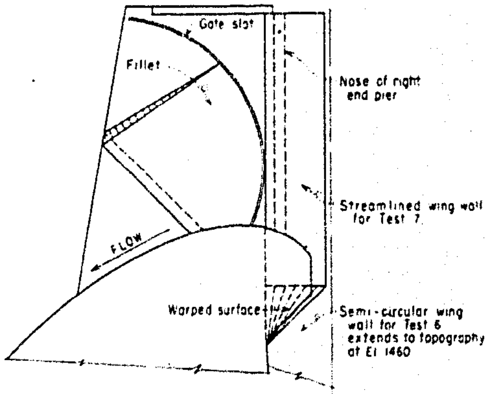
The false floor used in Test 6 was removed and replaced by a new false floor extending from Station 0+50 to 4+50, as shown by Figure 14A. At the same time, the 1-1/2:1 sloping bank on the left side of the spillway, upstream from the crest, represented by a piece of sheet metal in Test 6 through 9, was replaced by sand as permanent topography, for this modification of the natural bank was desirable.

Test 10 included only discharge measurements with all gates wide open to ascertain the capacity of this design. A comparison of the coefficient curves, Figure 12B, shows that the capacity of this design was less at low heads, but compared favorably with other designs at the maximum head. In this test, the head was measured in a more refined manner than formerly in that three gages at various points in the forebay

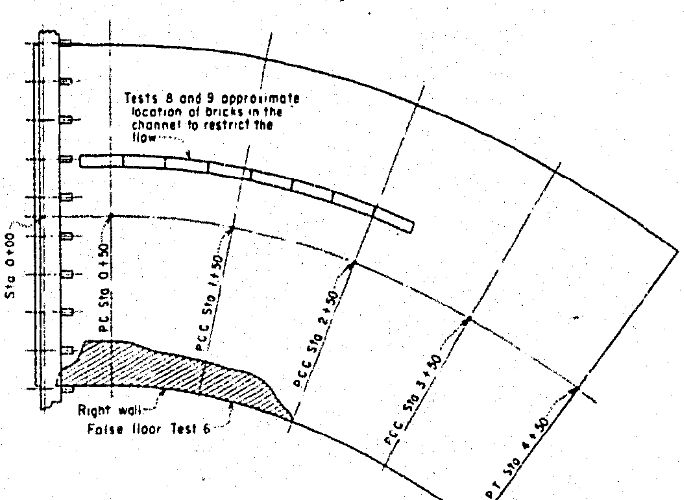
FIGURE 13



A - PROPOSED WING WALLS



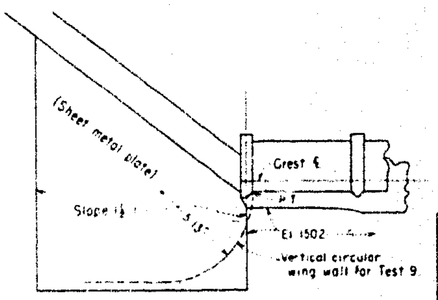
B - SIDE VIEW OF WING WALLS AND PIER FILLET



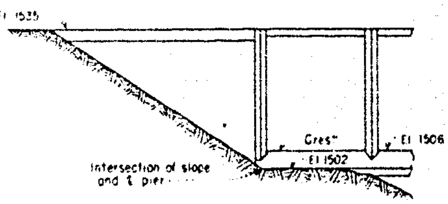
C - PLAN SHOWING LOCATION OF FALSE FLOOR-TEST 6 AND BRICK WALL TESTS 8 AND 9

ELEVATIONS OF FALSE FLOOR

STA ON ℄	DISTANCE FROM WALL PROTO	DISTANCE FROM WALL MODEL	ELEV.	STA ON ℄	DISTANCE FROM WALL PROTO	DISTANCE FROM WALL MODEL	ELEV.	STA ON ℄	DISTANCE FROM WALL PROTO	DISTANCE FROM WALL MODEL	ELEV.	STA ON ℄	DISTANCE FROM WALL PROTO	DISTANCE FROM WALL MODEL	ELEV.
0+10	1.00'	0.020'	1504.7	0+37.08	4.00'	0.080'	1486.65	1+00	3.25'	0.065'	1479.05	1+75	3.25'	0.065'	1474.80
	3.00'	0.060'	1503.8		12.50'	0.250'	1486.55		12.50'	0.250'	1479.35		12.50'	0.250'	1475.85
	5.00'	0.100'	1503.4		22.50'	0.450'	1486.10		22.50'	0.450'	1479.75		22.50'	0.450'	1476.85
Edge of floor	6.00'	0.120'	1503.0		32.50'	0.650'	1486.00		32.50'	0.650'	1480.65		32.50'	0.650'	1477.85
0+15	1.00'	0.020'	1503.1		42.50'	0.850'	1486.55		42.50'	0.850'	1481.70		42.50'	0.850'	1478.70
	3.00'	0.060'	1502.5		52.50'	1.050'	1487.30		52.50'	1.050'	1482.65		52.50'	1.050'	1479.40
	5.00'	0.100'	1502.5	Edge of floor	60.00'	1.200'	1487.75	Edge of floor	54.50'	1.090'	1482.95	Edge of floor	59.50'	1.190'	1479.90
	7.00'	0.140'	1500.9	0+50	4.00'	0.080'	1482.40	1+25	3.25'	0.065'	1477.70	2+00	3.25'	0.065'	1472.75
Edge of floor	8.50'	0.170'	1500.5		12.50'	0.250'	1482.40		12.50'	0.250'	1477.85		12.50'	0.250'	1473.85
0+20	1.00'	0.020'	1501.8		22.50'	0.450'	1483.05		22.50'	0.450'	1478.95		22.50'	0.450'	1475.10
	3.00'	0.060'	1500.5		32.50'	0.650'	1483.75		32.50'	0.650'	1477.45		32.50'	0.650'	1475.90
	5.00'	0.100'	1498.9		42.50'	0.850'	1481.80		42.50'	0.850'	1480.90		42.50'	0.850'	1476.80
	8.50'	0.170'	1497.9		52.50'	1.050'	1482.75		52.50'	1.050'	1481.75		52.50'	1.050'	1478.50
Edge of floor	12.00'	0.200'	[illegible]	Edge of floor	61.75'	1.235'	1483.95	Edge of floor	54.50'	1.090'	1481.75	Edge of floor	55.00'	1.100'	1478.15
0+25	0.00'	0.000'	1500.3	0+75	3.75'	0.075'	1479.70	1+50	3.25'	0.065'	1475.90	2+25	3.25'	0.065'	1469.25
	3.00'	0.060'	1498.5		12.50'	0.250'	1480.60		12.50'	0.250'	1476.90		7.50'	0.150'	1469.50
	6.00'	0.120'	1495.9		22.50'	0.450'	1481.50		22.50'	0.450'	1477.95		12.50'	0.250'	1470.55
	9.00'	0.180'	1494.6		32.50'	0.650'	1482.05		32.50'	0.650'	1478.95				
	12.00'	0.240'	1493.5		42.50'	0.850'	1483.05		42.50'	0.850'	1479.75				
Edge of floor	15.00'	0.300'	1493.2		52.50'	1.050'	1484.15		52.50'	1.050'	1480.45				
				Edge of floor	61.00'	1.222'	1483.35	Edge of floor	56.50'	1.130'	1480.70				

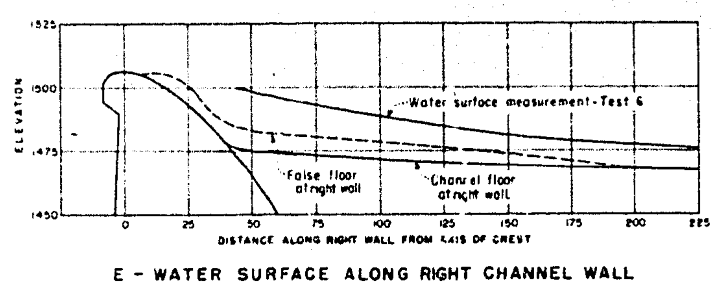


PLAN



UPSTREAM ELEVATION

D - TOPOGRAPHY AT LEFT END OF SPILLWAY - TEST 6



E - WATER SURFACE ALONG RIGHT CHANNEL WALL

SALT RIVER PROJECT - ARIZONA

STEWART MOUNTAIN DAM

HYDRAULIC MODEL STUDIES OF SPILLWAY

TESTS 4-9 IMPROVEMENTS TO SPILLWAY 1:50 MODEL were used to compensate for any uncertainties of velocity effects. Differences in the three gages were less than 0.10 foot, prototype, and this small difference was not significant being within the accuracy of the gage itself. The location of the forebay gage was not critical as long as it was placed at a reasonable distance upstream from the crest.

20. Effect on Discharge of Excavating Bank Upstream From the Crest

It was suggested that the spillway capacity might be increased by excavating the left bank upstream from the crest down to elevation 1490 along the 1-1/2:1 slope, as shown in Figure 14A. Several discharge runs were made and coefficients compared with Test 10. The results indicated an increase in discharge capacity of approximately 0.7 of 1 percent, but such a small gain was not sufficient to justify the cost of the proposed excavation. The idea was abandoned and the embankment replaced as in Test 10.

21. Measurements of Water Surface in Channel

With the water surface in the reservoir at elevation 1535 and with the spillway discharging 137,200 second-feet, the water surface in the channel was measured along the channel walls and across the channel at Stations 0+50, 1+00, 1+50, 2+00, 2+50, 3+00, 3+50, and 4+00 (Test 12). The water surface was considered satisfactory along the left wall, but too high along the right wall immediately downstream from the crest (Figure 14B). Further changes were indicated to reduce the height of the water along the right wall.

The measurements of the water surface across the channel are not included as the model was materially altered in later tests by removing the false floor and installing in its place a fillet. It is sufficient to note that the water surface was reasonably smooth, and that the depth was greater at the outside of the curved channel (that is the left side) as in the case of a theoretically correct design.

22. Use of Circular Wing Wall at End Pier

In Test 6, a circular wing wall on the right end pier was used, which was replaced by the smaller streamlined wing wall in Test 7 (Figures 13A and B). In Test 9, it was demonstrated that the smaller streamlined wing wall would be as efficient as the circular wing wall, but to verify this further, the circular wall was rebuilt and several runs were made to measure discharge. The discharge capacity was increased less than 1 percent over that measured in Test 10, and as far as increasing the discharge is concerned, it was apparent that the circular wall had little advantage over the streamlined type.

To compare this wing wall further, the water surface along the right channel wall was measured, but the results were not significantly different from that in Test 12 (Figure 14B). It was conclusively shown that the performance of the smaller streamlined wing wall, introduced in Test 7, was as efficient as the circular wall used in Test 6.

23. Use of Fillet in Channel Downstream From the Crest

The primary objection to the design of Test 10 was the high water surface along the right wall. In former tests on the 1:100 model, the flow was improved by placing a fillet on the channel floor alongside the wall, immediately downstream from the crest (Figure 9E). To demonstrate again the advantages of such a fillet, one was placed in the 1:50 model (Test 13A), extending from the crest to Station 0+80, patterned after the fillet used in the 1:100 model. In demonstration runs the height of splash along the right wall was reduced. Measurements of the water surface or coordinates of the fillet were not made, since it was planned to improve the shape of this fillet in later tests.

24. Use of Long Fillet Alongside Left Channel Wall

After preliminary runs with the arrangement of Test 13A, the fillet and false floor of Test 10 were replaced (Test 14) with a long fillet alongside the right channel wall (Figure 14A). This new fillet was formed by shaping and reshaping until satisfactory flow conditions were obtained. Coordinates of the fillet were then determined by point gaging (see table for Test 14 in Figure 14).

With the gates completely raised, and with the reservoir level at elevation 1535.0, the water surface along the right channel wall was point gaged. Comparison with the data of Test 12 (Figure 14B) shows that the height of the water surface along this wall was appreciably reduced indicating a definite advantage in the use of the fillet.

25. Modification of Sloping Bank Upstream From Left Pier

The sloping bank upstream from the left end pier was revised (Test 14) as shown in Figure 14A to improve the approach conditions and increase the discharge capacity as it was possible that the embankment developed in Test 10 was not large enough. Preliminary runs indicated that the flow with this new embankment would be satisfactory, but no discharge measurements were made until other tests were completed.

26. Gate Operating Schedule

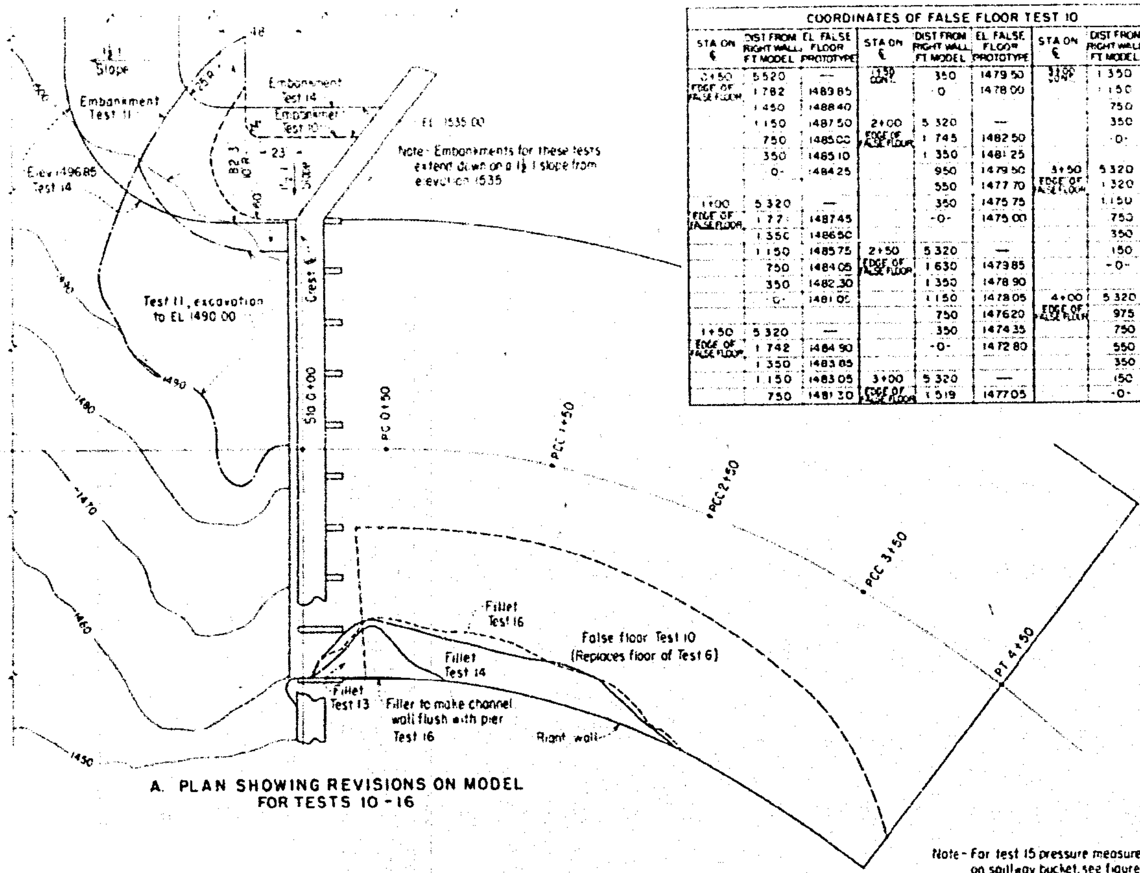
The need for a gate operating schedule was indicated. Prior tests on the 1:50 model had been made with all gates opened equally, resulting in a flow down the spillway of uniform depth across the entire section, the design condition for this channel. In actual operation of the prototype structure, it will seldon be necessary to require all gates to be opened, and for most releases it may be more convenient to use individual gates fully opened. A study (Test 14) was made by operating the individual gates, then various combinations of two, three, and four gates. Throughout this study, the water surface was held at the normal reservoir elevation of 1529.0. It was found that when certain gates and certain groups of gates were opened the flow conditions in the channel were not satisfactory. To facilitate the following discussion, the gates will be numbered consecutively, 1 to 9, from right to left, as means of their identification.

When Gate 1 or any consecutive group of gates, beginning with Gate 1, were opened the flow conditions were satisfactory.

When Gate 1 was closed and Gate 2 or any consecutive group of gates, beginning with Gate 2, were opened the conditions were not satisfactory because the flow piled against the right wall to form a fin as shown in Figure 15C.

When Gates 1 and 2 were closed and Gate 3 or any consecutive groups of gates, beginning with Gate 3, were opened the conditions were not satisfactory because the flow piled against the right wall in the same manner as with Gate 2 open.

With Gate 4 open the water piled against the right wall, but not sufficiently high to overtop it.



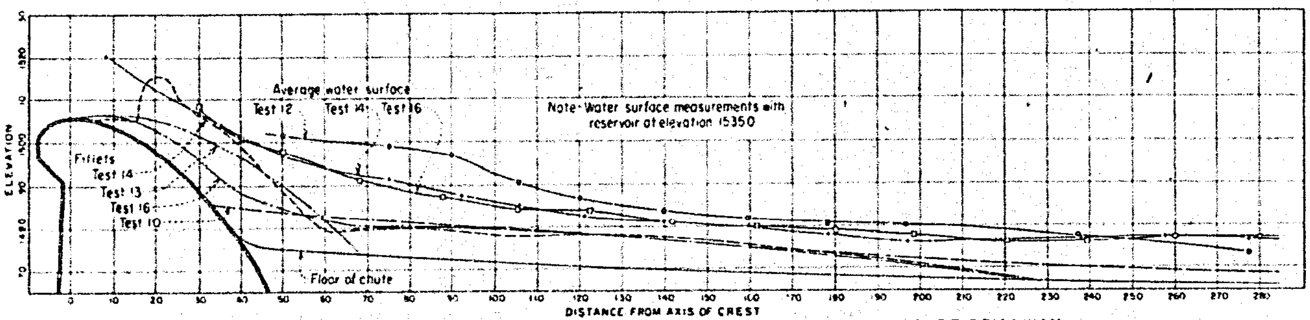
A. PLAN SHOWING REVISIONS ON MODEL FOR TESTS 10 - 16

Note - For test 15 pressure measurements on spillway bucket, see figure 16

COORDINATES OF FALSE FLOOR TEST 10								
STA ON ℄	DIST FROM RIGHT WALL FT MODEL	EL FALSE FLOOR PROTOTYPE	STA ON ℄	DIST FROM RIGHT WALL FT MODEL	EL FALSE FLOOR PROTOTYPE	STA ON ℄	DIST FROM RIGHT WALL FT MODEL	EL FALSE FLOOR PROTOTYPE
0+50	5 520	—	1+50 CONT.	350	1479 50	3+00 CONT.	1 350	1476 50
EDGE OF FALSE FLOOR	1 782	1483 85		-0-	1478 00		1 150	1475 90
	1 450	1488 40					750	1474 30
	1 150	1487 50	2+00	5 320	—		350	1472 65
	750	1485 00	EDGE OF FALSE FLOOR	1 745	1482 50		-0-	1470 90
	350	1485 10		1 350	1481 25			
	-0-	1484 25		950	1479 50	3+50	5 320	—
				550	1477 70	EDGE OF FALSE FLOOR	1 320	1474 40
1+00	5 320	—		350	1475 75		1 150	1472 95
EDGE OF FALSE FLOOR	1 771	1487 45		-0-	1475 00		750	1472 40
	1 350	1486 50					350	1470 50
	1 150	1485 75	2+50	5 320	—		150	1469 75
	750	1484 05	EDGE OF FALSE FLOOR	1 630	1479 85		-0-	1469 05
	350	1482 30		1 350	1478 90			
	-0-	1481 05		1 150	1478 05	4+00	5 320	—
				750	1476 20	EDGE OF FALSE FLOOR	975	1470 75
1+50	5 320	—		350	1474 35		750	1470 20
EDGE OF FALSE FLOOR	1 742	1484 90		-0-	1472 80		550	1469 50
	1 350	1483 85					350	1468 80
	1 150	1483 05	3+00	5 320	—		150	1468 45
	750	1481 30	EDGE OF FALSE FLOOR	1 519	1477 05		-0-	1467 65

COORDINATES OF FILLET TEST 14								
STA ON ℄	DIST FROM RIGHT WALL	ELEV OF FILLET	STA ON ℄	DIST FROM RIGHT WALL	ELEV OF FILLET	STA ON ℄	DIST FROM RIGHT WALL	ELEV OF FILLET
0+05	-0-	1505 23		-0-	1498 25		2 50	1478 45
0+10	9 90	1503 95	0+45	32 50	1482 00		-0-	1479 15
	3 35	1504 50		5 00	1486 30	1+20	23 00	1476 45
0+12½	1 50	1503 35		35	1488 50		20 00	1476 25
	5 00	1503 85		20	1492 30		5 00	1478 60
	-0-	1505 65		-0-	1492 75		-0-	1478 85
0+15	12 55	1501 95	0+50	32 00	1481 50	1+30	22 50	1476 05
	5 00	1502 90		25 00	1481 25		20 00	1475 85
	-0-	1500 00		5 00	1483 75		10 00	1478 00
0+20	14 05	1498 85		50	1485 85		-0-	1478 60
	7 50	1500 05		-0-	1488 10	1+40	22 00	1475 60
	1 20	1513 10	0+60	32 50	1481 00		0 00	1477 50
	-0-	1515 40		25 00	1479 85		-0-	1478 45
0+25	27 05	1494 85		10 00	1481 35	1+60	21 45	1474 75
	20 00	1495 10		-0-	1482 15		0 00	1475 55
	2 50	1499 60	0+70	30 00	1480 00		-0-	1477 85
	90	1509 25		22 50	1479 35	1+80	23 45	1474 30
	-0-	1510 65		10 00	1480 50		15 00	1474 45
0+30	31 45	1489 95		-0-	1480 20		7 50	1475 90
	10 00	1493 55	0+80	27 50	1478 90		-0-	1476 75
	70	1497 50		25 00	1478 60	2+00	23 75	1473 50
	65	1505 85		7 50	1480 15		17 50	1473 30
	-0-	1506 35		2 50	1480 05		10 00	1474 25
0+35	31 75	1486 20		-0-	1480 10		-0-	1476 10
	10 00	1490 60	0+90	25 50	1477 95	2+20	19 05	1472 00
	45	1494 35		12 50	1479 40		7 50	1472 75
	.35	1501 75		-0-	1480 15		-0-	1474 85
	-0-	1502 85	1+00	24 50	1477 40	2+50	9 70	1469 60
0+40	31 75	1483 80		7 50	1479 45		-0-	1471 45
	10 00	1487 75		2 50	1479 35	2+70	4 20	1468 45
	45	1491 60		-0-	1479 75		-0-	1469 15
	25	1407 50	1+10	23 65	1476 85	2+85	-0-	1467 45

COORDINATES OF FILLET TEST 16								
STA ON ℄	DIST FROM RIGHT WALL	ELEV OF FILLET	STA ON ℄	DIST FROM RIGHT WALL	ELEV OF FILLET	STA ON ℄	DIST FROM RIGHT WALL	ELEV OF FILLET
0+05	0	1505 87	0+35	32 70	1486 30	2+00	22 00	1473 40
	4 00	1505 52	0+50	0	1485 24	2+25	0	1473 85
0+10	0	1505 50		16 00	1483 27		10 00	1471 18
	9 50	1504 22		34 00	1482 30		20 00	1472 30
0+15	0	1504 72	0+75	0	1480 65	2+37.5	0	1472 54
	6 00	1502 86		16 00	1479 74		8 00	1470 73
	12 90	1502 00		31 20	1480 17		16 00	1471 28
0+20	0	1503 40	1+00	0	1479 65	2+50	0	1471 30
	8 00	1500 22		16 00	1478 30		8 00	1469 86
	15 60	1498 90		30 00	1478 83		14 00	1470 58
0+25	0	1501 00	1+50	0	1479 50	2+62.5	0	1469 77
	10 00	1496 92		16 00	1475 80		8 00	1469 28
	18 90	1494 90		28 00	1476 34	2+75	0	1468 55
0+30	0	1498 00	1+75	0	1477 46		6 00	1468 60
	12 00	1492 96		12 00	1475 20	2+87.5	0	1467 72
	24 20	1490 02		23 00	1474 50		3 00	1467 87
0+35	0	1494 65	2+00	0	1475 84			
	16 00	1488 38		12 00	1473 70			



B. ELEVATIONS OF FILLETS AND AVERAGE WATER SURFACE ALONG RIGHT WALL OF SPILLWAY

SALT RIVER PROJECT — ARIZONA

STEWART MOUNTAIN DAM

HYDRAULIC MODEL STUDIES OF SPILLWAY TESTS 10-16 IMPROVEMENTS TO THE CHANNEL 1:50 MODEL

With Gate 5 open the water spread and piled against both walls, but not sufficient to overtop them.

With Gates 6, 7, 8, or 9 open the water piled against the left wall, but not sufficient to overtop it.

It follows that consecutive combinations beginning with Gate 1 or combinations of Gates 4, 5, 6, 7, 8, and 9 may be used satisfactorily.

When alternate gates were discharging, high fins formed between them directly downstream from the closed gate (Figure 15D). If alternate combinations of gates were desired, it appeared that combinations such as 1 and 4, 1 and 5, 1 and 9 would be more suitable.

From the observations made in this test, it was recommended that the gates be opened consecutively beginning with Gate 1, and under no circumstances should Gates 2 or 3 be opened unless Gate 1 was already open.

27. Pressures in Bucket Downstream From Crest

The proposed spillway channel will be joined to the existing crest section by a 30-foot radius bucket faired into the crest to complete the ogee from the crest to the spillway channel. It was desirable to ascertain the pressures in this bucket (Test 15). Piezometers were installed along the centerlines of the second and the fourth gates from the right end of the spillway (Figure 16A). Pressures were first measured with all gates open wide and the water surface in the reservoir at elevation 1535.0, and then measured for various gate openings with the water surface at elevation 1529.0. In all cases pressures were positive and apparently greater than the depth of water over the piezometer. As indicated on Figure 16A, the pressures were similar at both positions.

28. Revision of Fillet in Channel and Modification of Right Wall

The fillet along the right wall required simplification for construction in the prototype since the fillet developed in Test 14 was too high near the crest. The fillet was altered as given by the table for Test 16, in Figure 14. It also appeared desirable to change the alinement of the right wall immediately downstream from the crest by shifting it inward to be flush with the end pier and eliminate the step or offset between the wall and pier. This alteration was accomplished in the model by a filler as shown in Figure 14A.

To ascertain if these changes made any improvement, point gage readings of the water surface alongside the left wall were taken in the same manner as those of Test 12 and 14. As shown by Figure 14C, some improvement was obtained in that the height of the water surface was decreased at several points.

29. Streamlined Wing Wall and Pressures in Water Below Crest

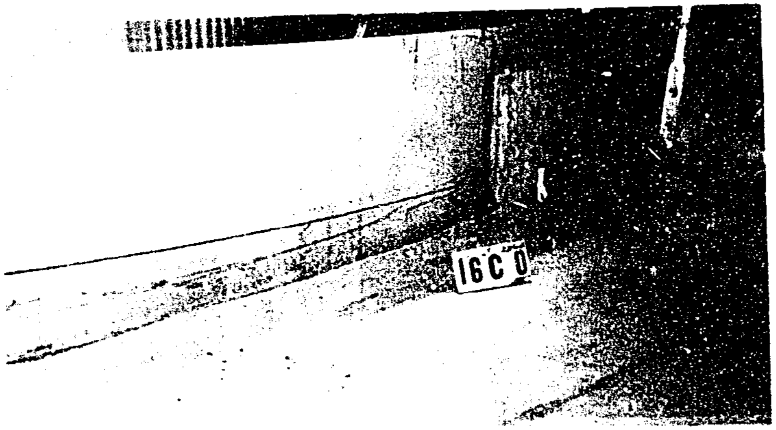
In Test 13 the streamlined wing wall on the right end pier, introduced in Test 7, was replaced by a circular wing wall previously used in Test 6 (Figure 11). At that time it was concluded that the different wing walls had no appreciable effect upon flow in the channel. As the streamlined wing wall was smaller and more economical to build, it was selected for the final design and was again placed on the model. Test 17 was made to measure the water surface alongside the right channel wall to compare with measurements obtained in Test 16, when the circular wing wall was used. The results were nearly identical.

Water coming down the spillway was guided into the channel by the bucket where pressures were increased by centrifugal force. In the channel itself, there was parallel flow and a normal hydrostatic variation of pressure with depth. It was uncertain as to how far downstream the effect of the curvilinear flow extended. To determine this, the variation of pressure with depth was measured by static tubes at several sections downstream from the bucket, and along the centerline of the second gate from the right end pier. Two types of static tubes were used to check the results of one against the other. As shown in Figure 16B, pressures at the end of the bucket at Station 0+50 were nearly twice the normal hydrostatic variation indicating curvilinear flow in this section. The measurements of pressure with depth extending downstream 60 feet (prototype) indicate that the variation of pressure with depth becomes nearly hydrostatic approximately 30 feet downstream from the bucket.

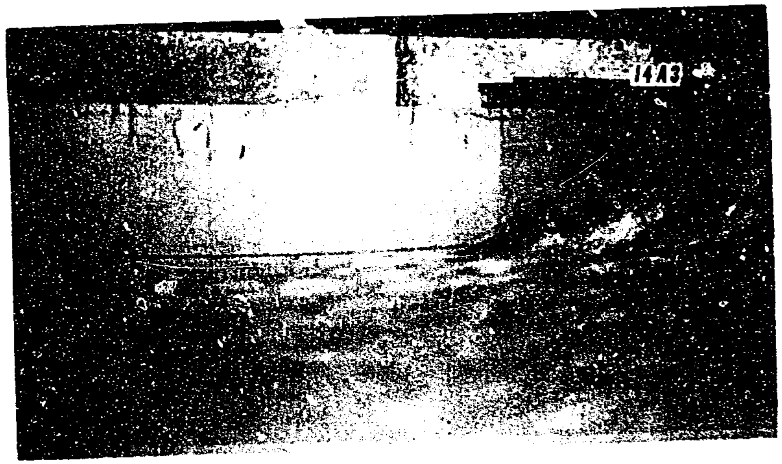
30. Determination of Height of Channel Walls and Use of Overhanging Sea Wall on Right Channel Wall

When improvements to the spillway and channel floor were completed, steps were taken to determine the final height of the channel walls. The proposed walls in the prototype were to extend 5 feet above the maximum water surface computed by the Design Section. The actual top of the walls in the model was at elevation 1529.0 (Figure 15A). As revisions were made to the crest and channel floor, the height of the water surface was measured along the walls. The top of the proposed and revised walls were sketched on this test wall as shown by the light lines of Figure 15A.

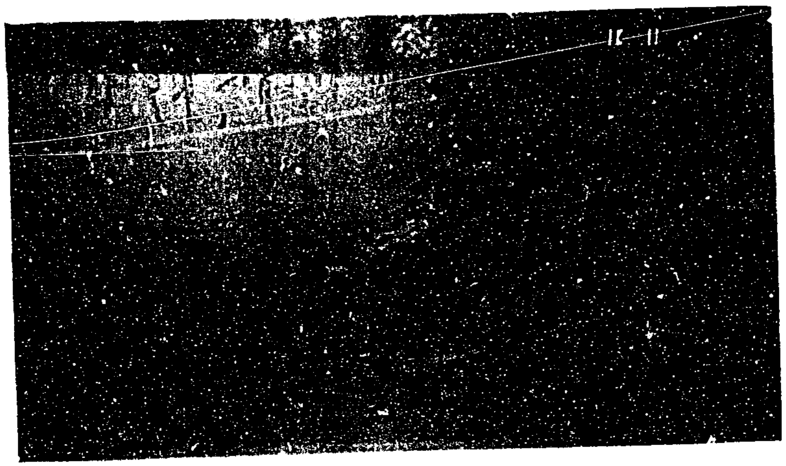
Figure 15



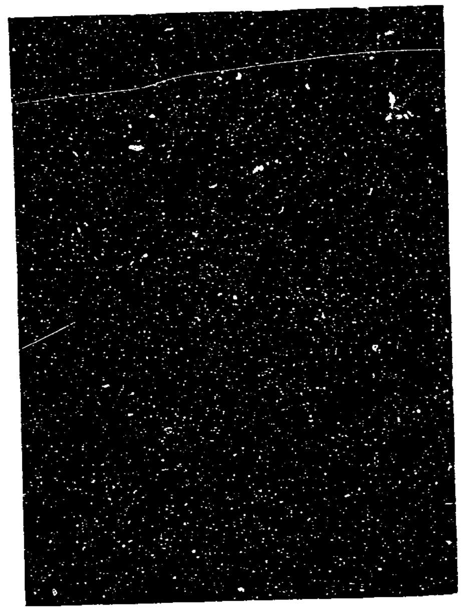
A. Fillet on channel floor.



B. Discharge with all gates open. Reservoir at elevation 1535.0.

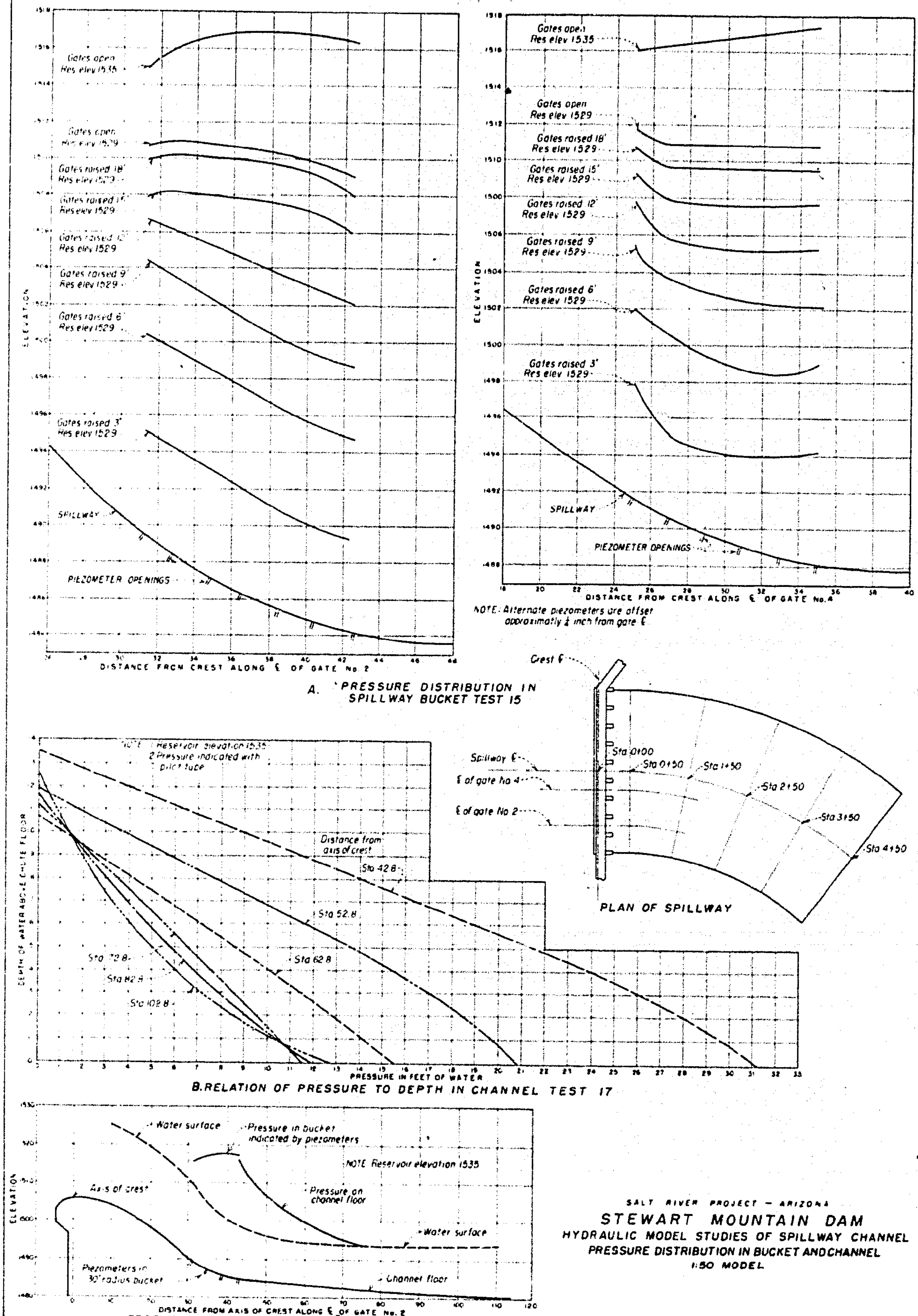


C. Discharge with gate No. 3 open. Reservoir at elevation 1529.0.



D. Discharge with gates No. 1 and 3 open. Reservoir at elevation 1529.0.

HYDRAULIC MODEL STUDIES OF STEWART MOUNTAIN DAM SPILLWAY
FLOW IN 1:50 MODEL WITH FILLET ON CHANNEL FLOOR



SALT RIVER PROJECT — ARIZONA

STEWART MOUNTAIN DAM

HYDRAULIC MODEL STUDIES OF SPILLWAY CHANNEL

PRESSURE DISTRIBUTION IN BUCKET AND CHANNEL

1:50 MODEL

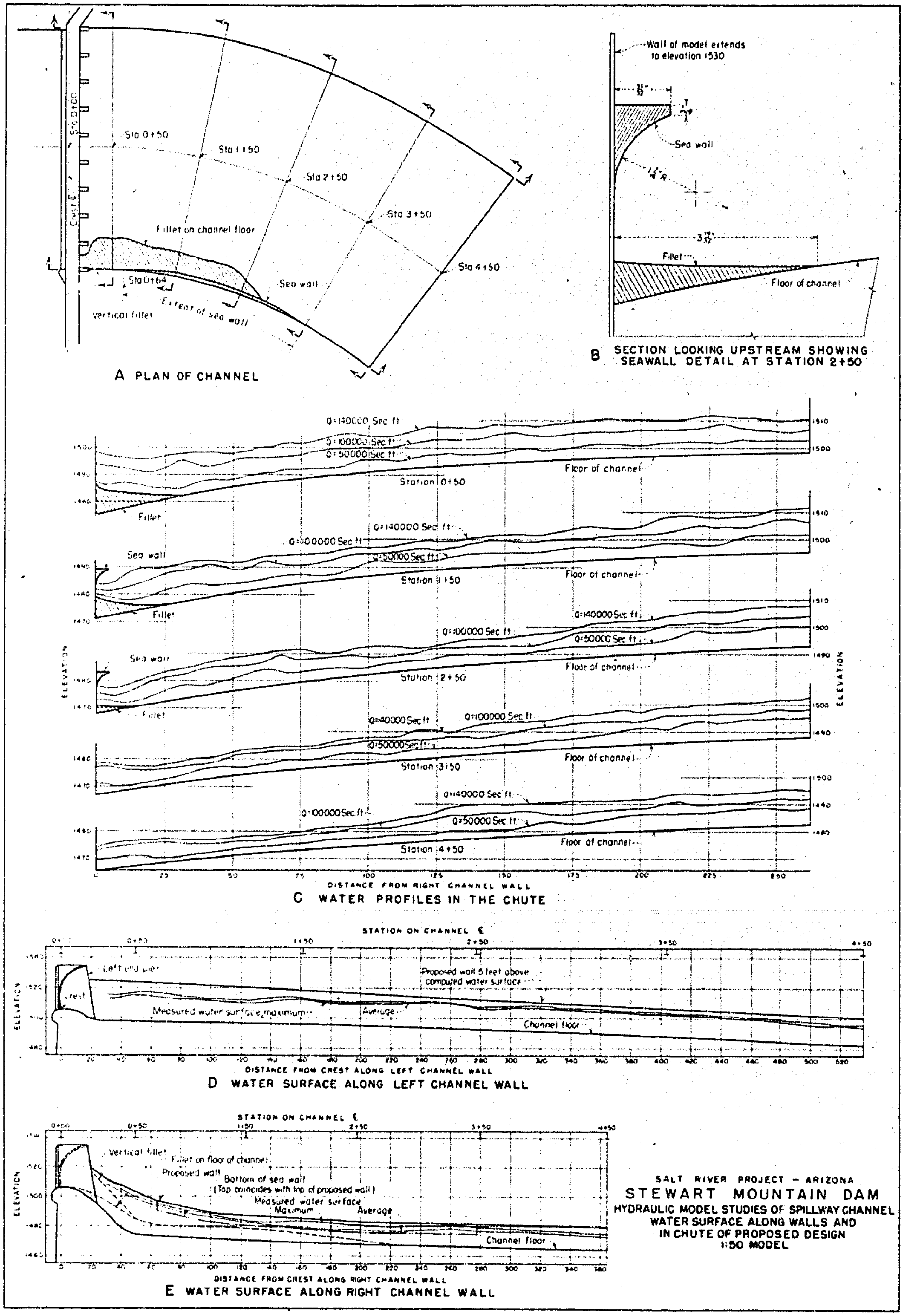
The first measurements were concerned with the left wall (Test 18). The average and maximum water surface, for maximum discharge, was measured by a point gage and plotted as shown in Figure 17D. This water surface was quite uniform and close to the theoretical water surface computed by the Design Section. The proposed wall, 5 feet above the theoretical water surface, was satisfactory and could be used in the final design.

The height of the right wall could not be established in this manner (Test 19). The proposed wall, 5 feet above the theoretical water surface, was below the actual water surface between Stations 0+30 and 1+25, and between Stations 2+00 and 2+25 (Figure 17E). In previous tests, the height of the water surface was reduced as much as possible by using a fillet on the channel floor alongside the right wall, but it was necessary to increase the height of the wall over the designed height. This was done between Stations 0+37.5 and 2+50 as shown in Figure 17E. Even with this higher wall, the water surface came nearly to the top, and the desirable freeboard of 5 feet did not exist. Since it would not be economical to increase the height of the wall, an overhanging lip, or sea wall, was placed along the top between Stations 0+70 and 3+00 as shown in Figures 17B, 17C, 17E, and 18D. Flow conditions along this wall were observed for various combinations of gates discharging to observe the effectiveness of the sea wall (Figure 18D). The study indicated that the sea wall should be extended upstream to Station 0+64 and downstream to Station 3+50 to control all high points of the water surface along the wall.

31. Water Surface Profiles Across Channel

With the sea wall installed, the final design was obtained, the water surface across the channel was measured (Test 19) at various stations and

FIGURE 17



A PLAN OF CHANNEL

B SECTION LOOKING UPSTREAM SHOWING SEAWALL DETAIL AT STATION 2+50

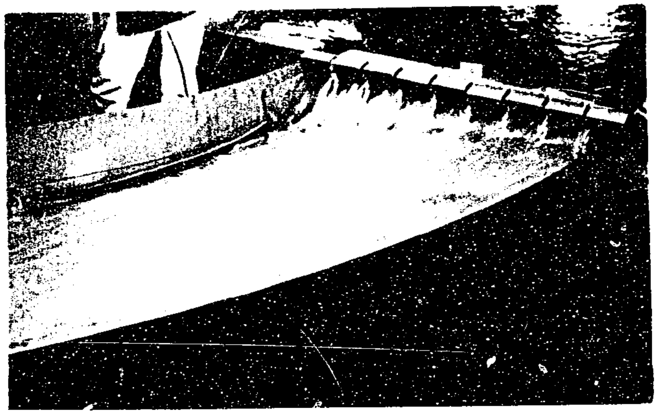
C WATER PROFILES IN THE CHUTE

D WATER SURFACE ALONG LEFT CHANNEL WALL

E WATER SURFACE ALONG RIGHT CHANNEL WALL

SALT RIVER PROJECT – ARIZONA
STEWART MOUNTAIN DAM
HYDRAULIC MODEL STUDIES OF SPILLWAY CHANNEL
WATER SURFACE ALONG WALLS AND
IN CHUTE OF PROPOSED DESIGN
1:50 MODEL

Figure 18



A. View of model showing position of overhanging sea wall.

B. Looking downstream, showing sea wall and fillets on piers and channel floor.

C. All gates partially open to discharge 85,000 c.f.s, to show action of sea wall.

D. Second gate from end open showing water being turned into channel by sea wall.

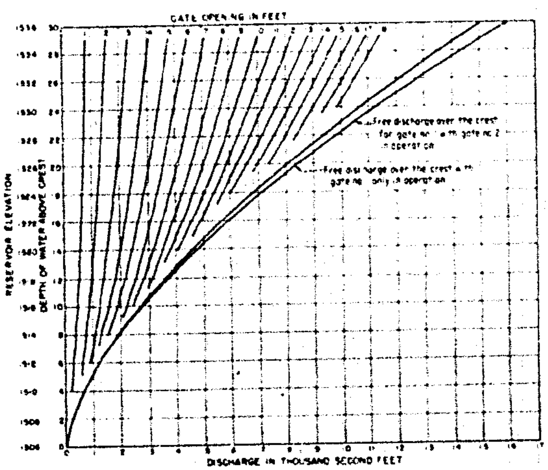
HYDRAULIC MODEL STUDIES OF STEWART MOUNTAIN DAM SPILLWAY
USE OF SEA WALL ON LEFT CHANNEL WALL, 1:50 MODEL

for several discharges. In the first run, the radial gates were raised sufficiently to maintain the reservoir at elevation 1529.0 with a discharge of 50,000 second-feet. Point gage readings were taken across the channel at Stations 0+50, 1+50, 2+50, 3+50, and 4+50, to measure the depth of water in the channel. The gates were completely raised and the water surface held at elevation 1529.2 to give a discharge of 100,000 second-feet, and point gage readings were again taken across the sections. The third run was similar, with the gates raised, the water surface at elevation 1535.0 and the discharge 140,000 second-feet. The results are shown on Figure 17C.

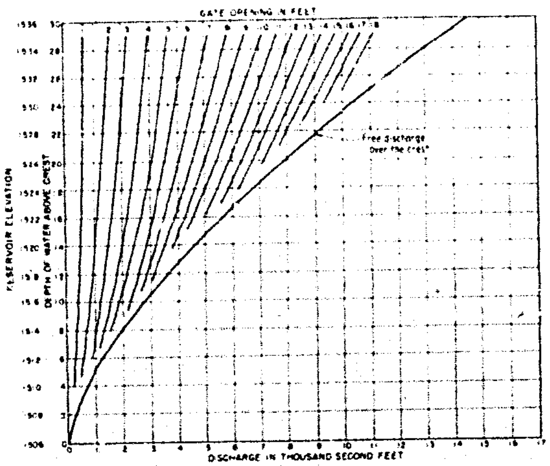
32. Calibration of Model for Free Discharge Over Crest, and With All Gates at Same Openings

When an acceptable design of the spillway was obtained, the model was calibrated (Test 20) to provide rating curves of discharge in the prototype (Figure 19) showing the relation of discharge to water surface elevation. In previous tests, coefficient curves were used to compare the capacity of the different spillway designs.

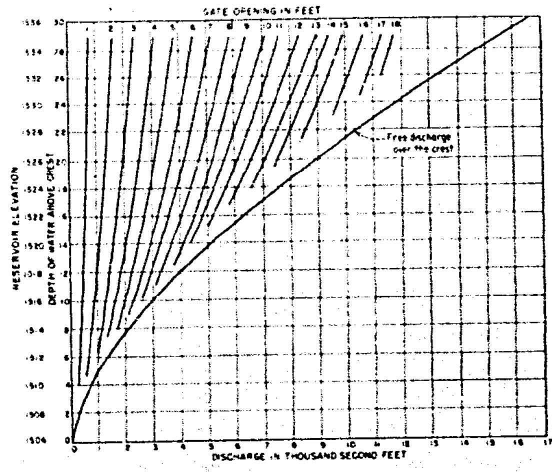
A prototype rating curve for free discharge over the crest, showing the relation between reservoir elevation and discharge, was computed from a model coefficient curve (Figure 19E). Seventeen runs were made on the model with the gates fully raised to measure discharge at various water surfaces between elevation 1506 and 1535. The coefficients were computing using the relation: $Q = CLH^{3/2}$, where Q = discharge, C = the coefficient, L = the crest length, and H = head in reservoir above the crest.



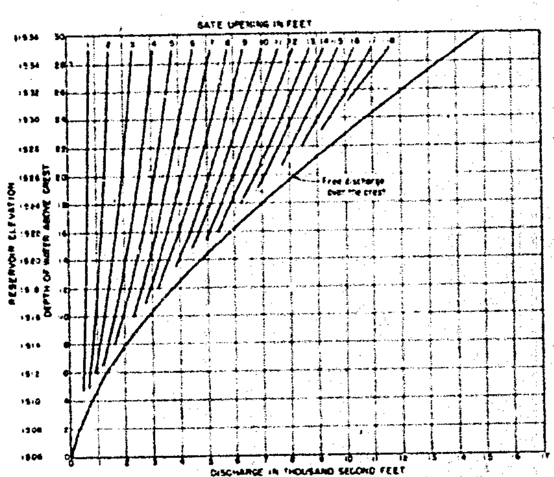
A DISCHARGE THROUGH GATE NO. 1.



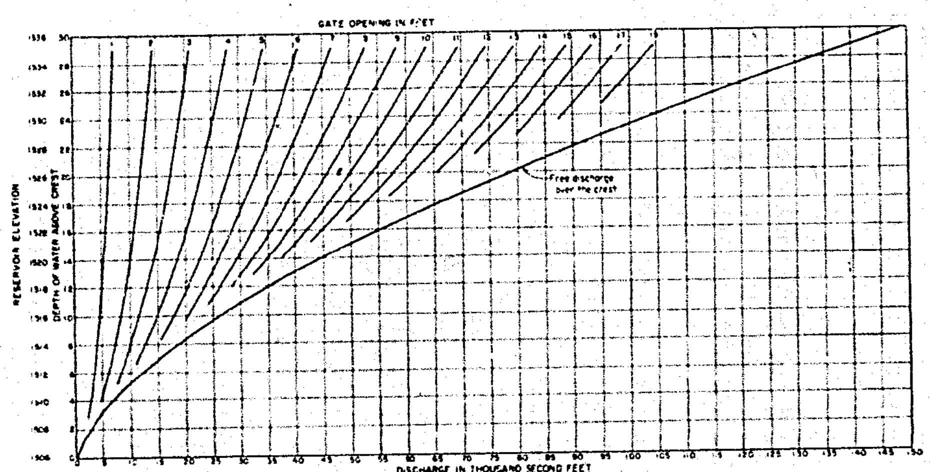
B DISCHARGE THROUGH GATES 2 TO 8 INCLUSIVE, ONE ADJACENT GATE DISCHARGING



C DISCHARGE THROUGH GATES 2 TO 8 INCLUSIVE, BOTH ADJACENT GATES DISCHARGING



D DISCHARGE THROUGH GATE NO. 9 WITH ADJACENT GATE DISCHARGING



E DISCHARGE THROUGH ALL GATES

SALT RIVER PROJECT — ARIZONA
STEWART MOUNTAIN DAM
HYDRAULIC MODEL STUDIES OF SPILLWAY CHANNEL
HEAD-DISCHARGE CURVES
1:50 MODEL

FIGURE 19

Calibration measurements were made for gate openings as follows:

Gate opening feet	Number runs made	Range of water surface elevation in test
18	7	1530.6 to 1535
15	9	1527.0 to 1535
12	10	1523.9 to 1535
9	22	1518.5 to 1535
6	10	1515.9 to 1535
3	13	1511.4 to 1535
1-1/2	11	1509.8 to 1535

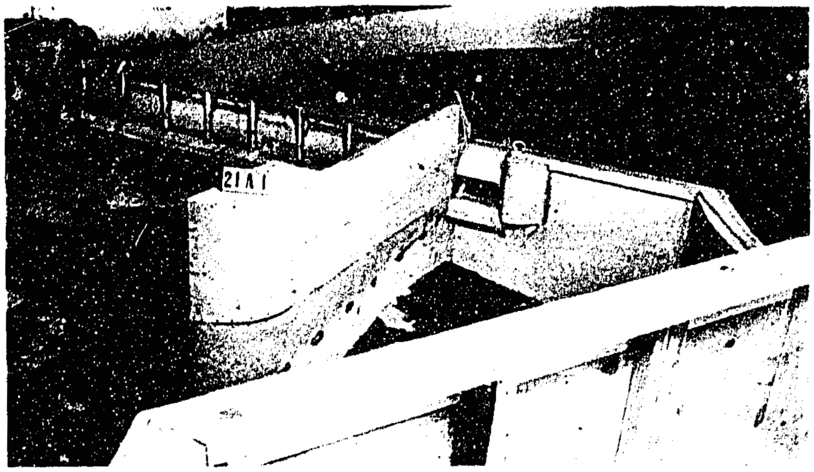
In applying this data to prototype, a method of analysis was used which subordinated the influence of individual readings to the general trend in a manner similar to the analysis with the gates completely raised. The empirical expression $Q = K\ (H-b)^n$ was used where Q = discharge, H = the head, and b, K, and n = constants to be determined. The relations of Q and H were plotted on log-log coordinates. By a proper selection of the value of the constant b, the relations of Q and H-b plotted as a straight line on log-log coordinates. Then from the expressions of data as log Q and log H-b the values of K and n may be determined least squares, by connecting to prototype terms, and by interpolation the curves of Figure 19A, for various gate openings were obtained.

The curve for free flow over the crest was not joined with the curves for discharge with the gate partially open since there is a discontinuity or unbalanced condition between the two types of flow. If the water surface in the reservoir is held constant, and the gates are closed from the wide open position, free discharge over the crest will occur until the gates touch the flowing nappe. By impact, a head of water is created against the gate of sufficient force to cause the nappe downstream from the gate to contract reducing the effective area of the jet and reducing the discharge. A similar situation occurs when the gates are being raised. At the opening where the nappe breaks from the gate to flow free, the discharge increases. No tests were made to study this condition beyond general observation because of the difficulty of holding the water surface in the forebay of the model constant while a sudden change of discharge occurred.

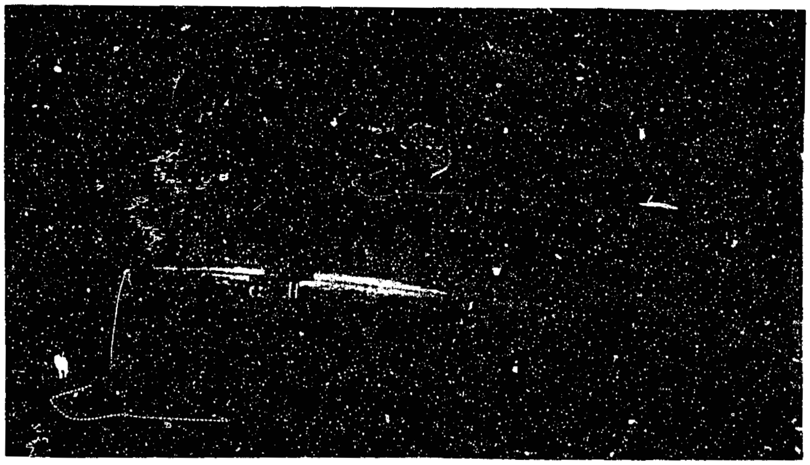
33. Simplification of Right Wing Wall of Spillway

As shown in Figure 13B, the bottom of the wing wall at the right side of the spillway was formed by a sloping fillet with a warped surface below elevation 1502.0. As such a fillet would be difficult to form in concrete, it was proposed that a flat bottom be used if similar flow conditions could be obtained by extending the wing wall downward several feet below the original position. A long wall was placed in the forebay at the left to isolate this gate (Figure 20A).

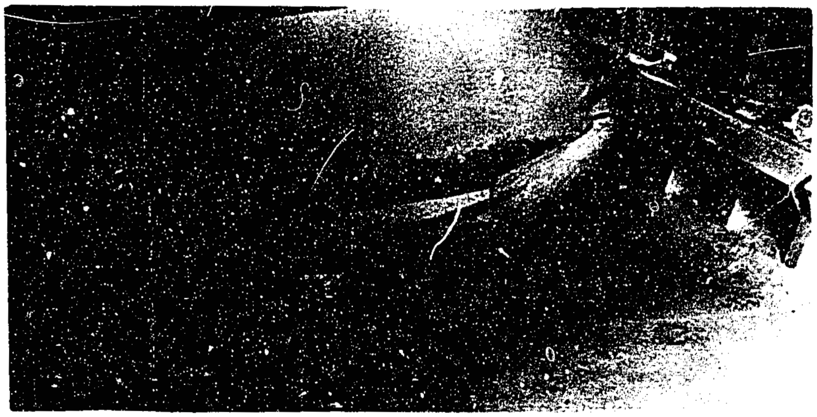
The discharge and head were measured with free discharge first, with the fillet on the bottom, Figure 13B, (Test 21); second, with a flat bottom and the wing wall at elevation 1497.0 (Test 22); and third, with a flat bottom at elevation 1502 (Test 23). There was no appreciable effect on the discharge capacity. Therefore, the prototype design included a wing wall at the right end with a flat bottom at elevation 1502.00 (Figure 20A).



A. Left end gate open with wall at right pier to obtain same approach conditions as if adjacent gate were opened.



B. Building paper along right wall to show extent of splash at maximum discharge.



C. Building paper along left wall to show extent of splash.

HYDRAULIC MODEL STUDIES OF STEWART MOUNTAIN DAM SPILLWAY
WING WALL ON PIER IN CALIBRATION TESTS
AND EXTENT OF SPLASH OVER CHANNEL WALLS.

34. Calibration of Flow in Right End Gate, Gate 1

In addition to operating the spillway with all gates open the same amount, it may also be operated by opening individual gates. As suggested in Test 14, a satisfactory schedule would be to open the right gate, Gate 1, first, then the adjacent gates consecutively. The right gate was calibrated with the adjacent gate closed, and then calibrated with it open, because the contraction of the nappe at the left side will be changed, affecting the discharge capacity. In Test 23, the gate was held open and the free flow over the crest measured. The long wing wall shown in Figure 20A was placed at the left pier to suppress the flow in a manner similar to the adjacent gate being open. Test 28 was similar, but with the long wing wall removed to represent free flow through the right end gate with the adjacent gates closed. The procedure of the tests and method for obtaining these curves was similar to that described in Test 20.

In Test 29, the right end gate was calibrated for partial openings with the water surface in the reservoir at various elevations. The curves for partial openings and free discharge are shown in Figure 19A. Since the long wing wall at the left pier used in Test 23 has been removed, Test 29 represented the flow through the right end gate with the adjacent gate closed. During this test, it was observed that vortices would form at either, or both, sides of the gate, usually when the gate was at a wide opening and when the head of water on it was not large. Some tests were made by placing the long wall on the left pier to suppress these vortices. No material change could be observed and it was concluded that the model was not sensitive enough to warrant a study of their effect.

Moreover, the gate was not calibrated for partial openings with the long wall at the left pier in place which would represent the condition with the adjacent gates also partially open. It was believed that such a calibration would not differ greatly from the curves shown in Figure 19A.

35. Calibration of Flow in Center Gates

If the recommended operating schedule is followed, after the right end gate, Gate 1, is open, the adjacent gates, Gates 2 to 8, will be opened consecutively. Thus the flow through each of these gates must be calibrated for two conditions: (1) with the adjacent gate to the right open, (2) with the gates on each side opened. If each gate were thus calibrated, there would be a duplication of work, for it may be reasonably assumed that the calibration of any one of the seven center gates will be representative of the others. Gate 5 in the center of the crest was selected for the tests. In Tests 30 and 31, a single long wing wall similar to that shown in Figure 20A, was placed at the right pier of Gate 5 to give the same approach conditions that would occur if the adjacent gates to the right were opened. In Tests 24 and 25, two wing walls were used, one at each side of the gate, to give the same approach conditions that would occur if adjacent gates on both sides were opened. These tests were conducted in the same manner as the calibration of the right end gate. The results of Tests 30 and 31 are shown in Figure 19B and the results of Tests 24 and 25 in Figure 19C. The curves for a gate partially open are not valid if the adjacent gates are not at the same opening as the one under consideration.

36. Calibration of Flow in Left End Gate

In the suggested procedure for opening the gates indivudually, Gate 9, at the left end, will be opened last. Since the embankment at the side would cause an approach condition peculiar to this gate, it was calibrated separately. A long wing wall was placed at the right pier of this gate to represent the approach condition that would occur if an adjacent gate to the right were opened. The procedure for the calibration was similar to that for the previous tests, and the results are presented as shown in Figure 19D. The curves for specific gate openings are valid only if the adjacent gates are opened the same amount as Gate 9.

37. Velocity Distribution in the Channel

It was desirable to measure the velocity distribution in this final design for a comparison with theoretical velocities upon which the design was based. Velocities were measured on the original design of the 1:100 model, but the results were unsatisfactory because that model was too small. In this Test 32, on the final design, the velocities were measured at 15 positions indicated in Figure 21B. The discharges were 50,000, 100,000, and 139,000 second-feet, with the water surface at elevation 1529.0 for the smaller discharges and elevation 1536.0 for the maximum discharge. The pitot tube used was a special static leg type being small in diameter to give a minimum disturbance to the flow. Since the dynamic pressure only was recorded, the static pressure had to be determined by the depth of the water. The results of these measurements are shown on Figure 21A, and are compared with the theoretical velocities. The theoretical velocities were based upon a drop from elevation 1529.0

to the point in question assuming no loss. Therefore, the theoretical velocities are shown as increasing with depth although it might be argued that they should be constant with respect to depth to account for the static head. The velocities were higher (Figure 21A) at the right side of the channel as anticipated, and somewhat less than the theoretical velocities. It appears from these curves that the losses increase down the channel for the difference between the measured and theoretical velocities at Station 4+50 is greater than upstream, but this data is too irregular to draw any definite conclusions.

38. Splash Over Spillway Walls

During demonstration runs on the final design, it was observed that particles of water splashed over the walls, especially at large discharges. It was impossible to predict what form this splash would take in the prototype, whether it would be as spray or as slugs of water. To show the extent of this splash, a study (Test 33) was made with all gates wide open and with the water surface in the reservoir at elevation 1535.0 to obtain maximum discharge. A strip of building paper was fastened to each wall and when a particle of water struck the paper, a stain resulted which could easily be seen. The splash over the right wall (Figure 20B) was not serious, although some spray struck the paper about 10 feet (prototype) above the wall. The splash along the left wall was more severe (Figure 20C). The particles of water were concentrated between Stations 9+30 and 2+30 with a maximum splash height of 40 feet (prototype) above the wall at Station 1+00. Such spray might be serious in the prototype only if the spillway were operated at maximum capacity for a long period of time. Since this was unlikely, the condition was not considered critical.

39. Discharge Coefficients of Final Design

The discharge coefficients of the final design (Test 34) were measured for a comparison with the previous tests to show the effect of the various changes upon the capacity of the spillway, made after the design described in Test 10. Briefly, these changes were, a long fillet in the channel downstream from the crest alongside the right wall, a fillet on the right wall immediately downstream from the pier to make the surface of the wall flush with the pier, the installation of a sea wall, modification of the sloping bank upstream from the left pier, and simplification of the bottom of the wing wall on the right end pier. These changes increased the coefficient (Figure 12B) slightly over that of Test 10 at the lower discharges, but had no material effect at the higher discharges.

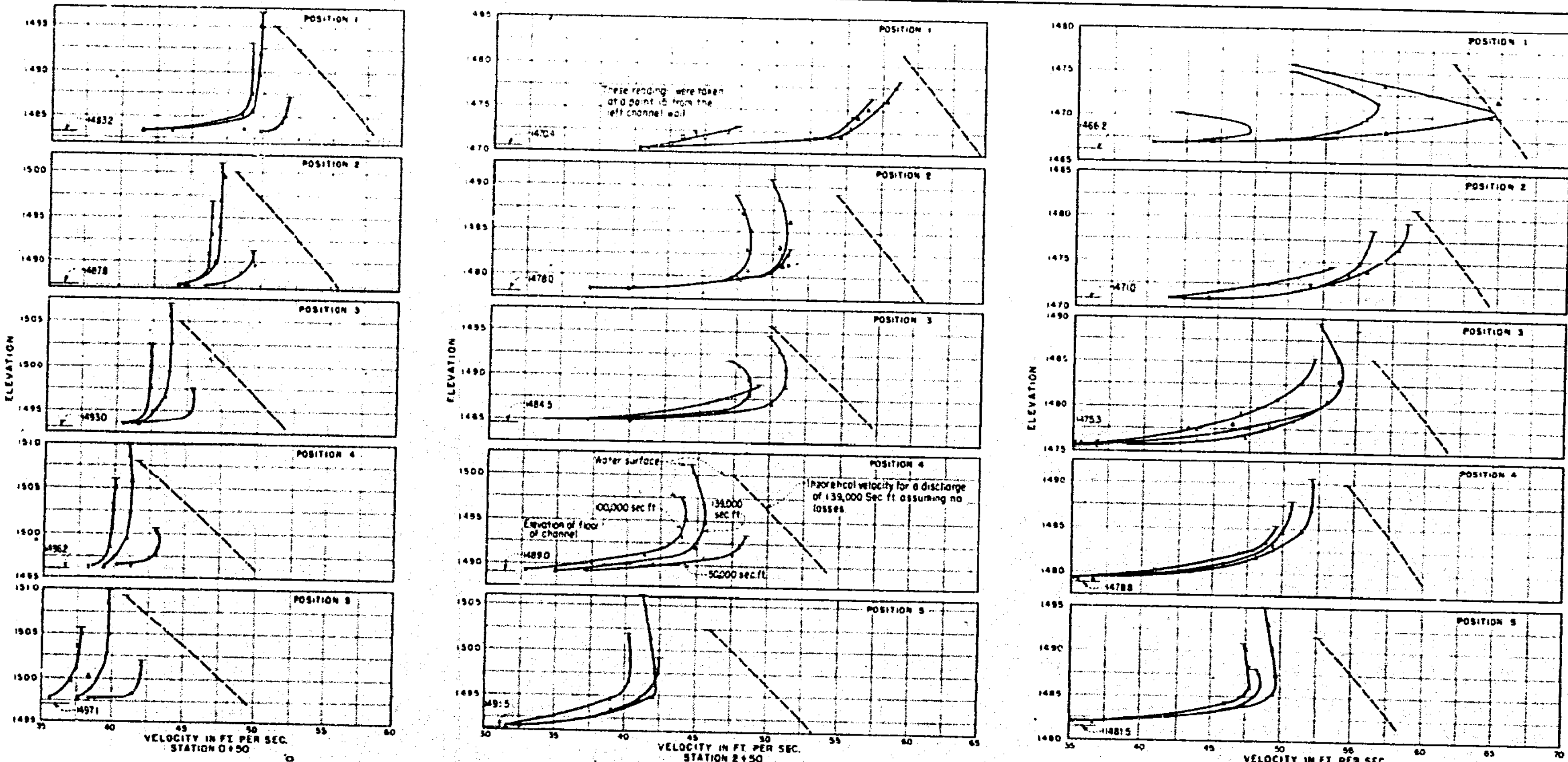
40. Discharge Coefficients With Spillway Channel Removed

To show conclusively the effect of the channel on the discharge capacity of the spillway, Test 35 was made with the spillway channel removed, and coefficients were measured and compared with those of Test 34. While no difference could be noted at the lower discharges, the discharge capacity was increased about 2 percent with the maximum flow, with the water surface at elevation 1535.0. Evidently, the flow over the crest was partially submerged by the channel at the high discharges, probably at the left side.

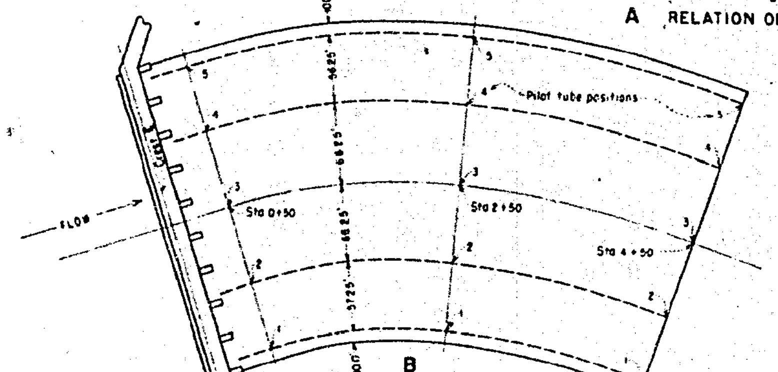
41. Discharge Over Crest With Piers and Channel Removed

To show the effect of the changes upstream from the crest, the piers and channel were removed so the model was similar to Test 2 of the preliminary studies. Figure 12B indicates a slight reduction in the coefficients over the final design. There is no logical explanation for this condition as the changes to the topography and conditions upstream should have tended to increase the capacity.

FIGURE 21



A RELATION OF VELOCITY TO DEPTH



B PLAN SHOWING PITOT TUBE LOCATIONS

Note For water surface profiles see figure 17

SALT RIVER PROJECT – ARIZONA
STEWART MOUNTAIN DAM
HYDRAULIC MODEL STUDIES OF SPILLWAY CHANNEL
VELOCITY MEASUREMENTS 1:50 MODEL