* SPILLWAY MODEL STUDIES *

TOLEDO BEND DAM

by

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and
Frank D. Masch

submitted under contract with
FORREST AND COTTON INC., CONSULTING ENGINEERS
Dallas, Texas

CENTER FOR RESEARCH IN WATER RESOURCES
Hydraulic Engineering Laboratory
Department of Civil Engineering
THE UNIVERSITY OF TEXAS

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Director:
EARNEST F. GLOYNA
Professor of Civil Engineering
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November 12, 1965

Mr. A. Martelli
Forrest and Cotton, Inc.
Consulting Engineers
600 Mercantile Continental Building
Dallas 1, Texas

Dear Mr. Martelli:

Transmitted herewith is our report on the model studies of the Toledo Bend Dam as authorized under contract with the Center for Research in Water Resources at The University of Texas.

This report represents a detailed study of the hydraulic performance characteristics of the spillway section of the dam for a range of discharges including the normal flood, design flood, and the maximum probable flood. Complete studies were made first on an original design based on Design Memorandum No. 2, August 1962, and then on a modified design dated February 1965.

Three design modifications have resulted from the model studies and include the construction of an upstream dike to improve the entering flow conditions, a change in the stilling basin design to improve the hydraulic jump stability, and a change in the thickness of the downstream rip-rap to provide better scour protection. A recommended gate operating sequence based on maintaining good flow distribution in the stilling basin is also included in the report.

Respectfully submitted,

Frank D. Masch

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Project Directors

PREFACE

This report contains the results of a model study on the spillway section of the Toledo Bend Dam, located on the Sabine River. The study was carried out in the Hydraulic Engineering Laboratory at the University of Texas during the period from January 1964 through September 1965. The principal objectives of the study were to determine the hydraulic performance characteristics of the spillway section of the dam, including stilling basin effectiveness, flow behavior in the upstream and downstream channels, and gate operating sequence for a range of discharges including normal flood, design flood, and maximum probable flood.

The model tests on the Toledo Bend Dam spillway section were carried out in two phases. The original model was constructed according to Design Memorandum No. 2, "Spillway," dated August 1962 and plans for the spillway of the Toledo Bend Dam as issued for construction. In February 1965 details of the new spillway profile and revised training walls were transmitted to the University of Texas. While complete studies were made on both the original and modified spillway sections, only that test data related to the modified spillway section is included in this report.

The work was initiated under an agreement between the Center for Research in Water Resources at the University of Texas and Forrest and Cotton Inc., Consulting Engineers in Dallas, Texas. Administrative details related to the study have been handled by the Bureau of Engineering Research at the University of Texas.

The authors wish to thank Mssrs. Tom Gebhard and Jim Avera for their very able assistance during the early planning stages and construction of the model. To them and to the other research assistants who have helped in carrying out various phases of the study, the authors wish to express their sincere appreciation. Special thanks are also due Mrs. E. S. Spencer who typed the report and to Mssrs. C. Y. Lee and C. L. Kuo who did most of the drafting.

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THE TOLEDO BEND PROJECT

The Toledo Bend Dam is a rolled earth-fill dam located at the Texas-Louisiana border on the Sabine River at river mile 156.5 in Newton County, Texas and Sabine Parish, Louisiana (Figure 1). The dam is a multi-purpose structure and provides for hydro-electric power, water conservation, navigation improvements and recreation. The drainage area above the dam site is 7,190 square miles or about 3/4 of the total area of 9,753 square miles drained at the mouth of the Sabine River. The reservoir capacity at an elevation 175.27 feet corresponding to the spillway design flood is 5,281,550 acre-feet. Development of design flood hydrographs, maximum reservoir water levels, and other pertinent hydrologic data have been set forth in Design Memorandum No. 1, "Hydrology," November 1960.

Description of Spillway Structures -- Original Design

The spillway structure for the Toledo Bend Dam and reservoir as originally designed was a gravity-type ogee weir section located on the Louisiana side of the dam. The weir crest was at elevation 145.0 feet and was controlled by eleven 40 ft. x 28 ft. tainter gates. The overall length of the spillway was 838 feet. The gross weir section was 530 feet long, with a net open length of 440 feet. The approach slab in the reservoir was at elevation 125 feet giving a weir height of 20 feet. The spillway design flood for the Toledo Bend Dam was one with a peak inflow of 554,000 cfs, a peak spillway discharge of 290,000 cfs, and a maximum pool elevation of 175.27 feet, or a 30.27 foot head upstream of the spillway crest. The shape of the spillway crest was based on a

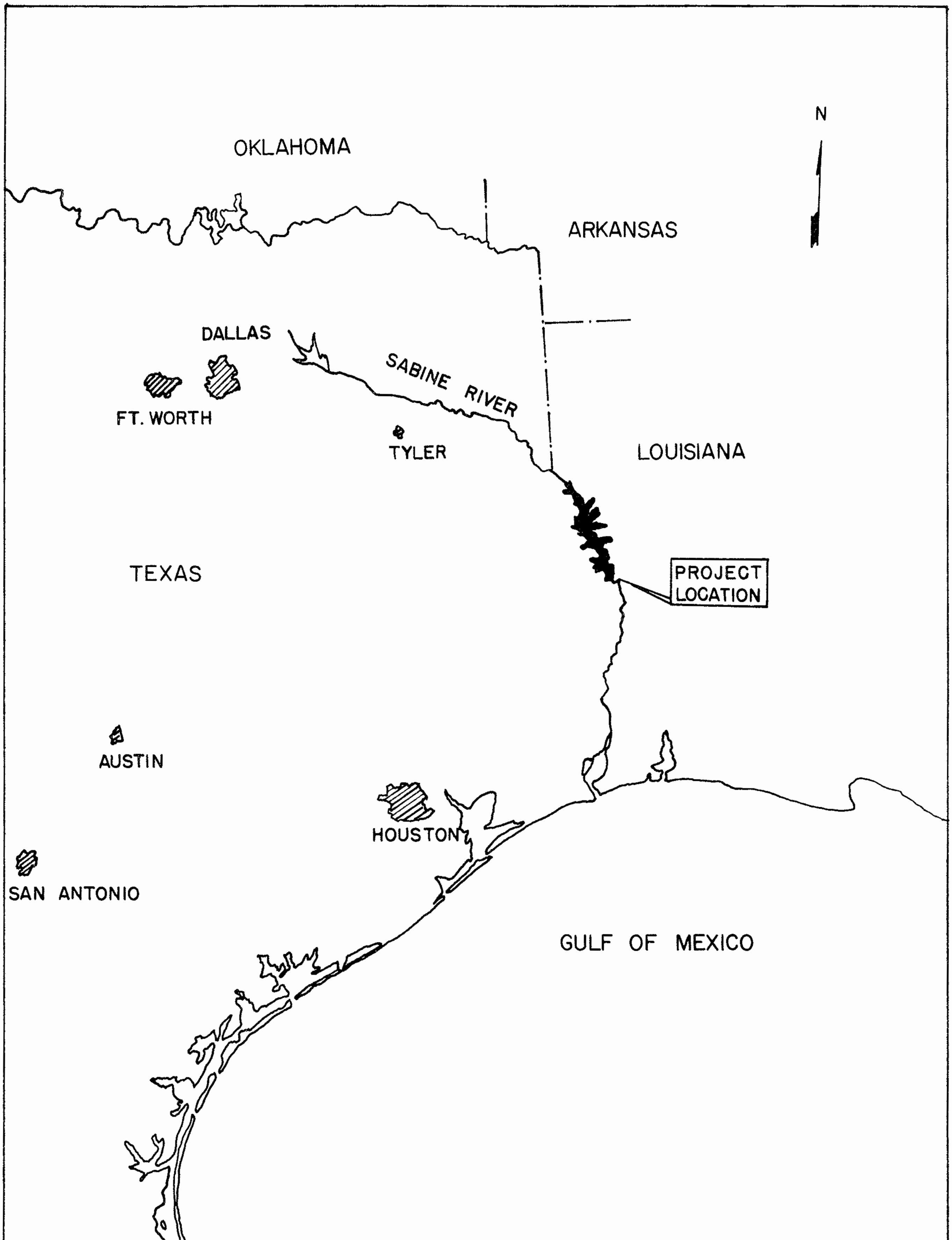


FIGURE I. LOCATION MAP

design head of 23.7 or about 78.5 percent of the maximum head anticipated during the passage of the design flood. The maximum probable flood had a peak inflow of 839,500 cfs, a peak spillway discharge of 329,000 cfs, and a maximum pool elevation of 177.86 feet, or 32.86 feet of head upstream of the spillway crest. Pertinent hydrologic data related to the spillway design is included along with design criteria, design analyses, and general information for the spillway in Design Memorandum No. 2, "Spillway," August 1962.

The stilling basin located immediately below the spillway had a horizontal floor at elevation 90.0 feet, with a width of 530 feet equal to the width of the gross weir section. Two rows of baffle blocks were placed in the stilling basin approximately 44.0 feet and 62.0 feet respectively from the downstream face of a five foot high end sill. Immediately downstream of the end sill was a horizontal discharge channel 100 feet long and 530 feet wide and covered with 48 inch rip-rap.

One hundred feet downstream of the end sill or at the end of the rip-rap, the discharge channel sloped upward on a slope of 1 on 10 for 100 feet to elevation 105.0 feet and then extended downstream with side slopes of 1 on 3 and a bottom width of 590 feet for a distance of about 3,500 feet at a slope of 0.0144 and terminated where it entered the natural flood plain of the Sabine River. A low flow pilot channel excavated in the center of the 590 foot wide discharge channel had a bottom elevation of 65.0 feet at the end of the rip-rap and extended downstream for about 8,200 feet on a bottom slope of 0.144 to its intersection with the Sabine River. The general alignment of these discharge channels may be seen in Figure 2.

In order to provide for low flow release through the spillway, an 8-1/3 ft. x 12 ft. sluiceway and control gate were located in a special low-flow

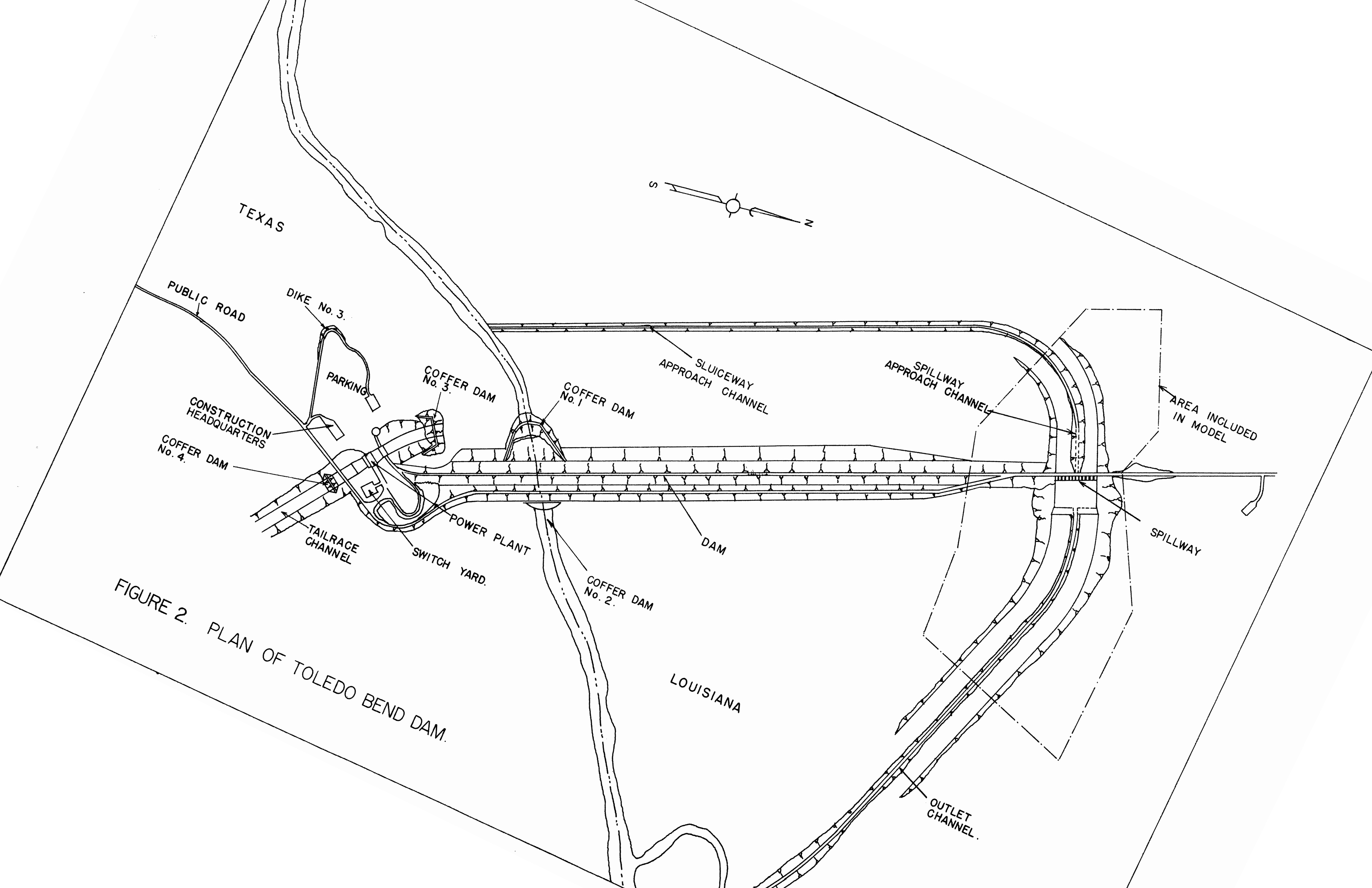


FIGURE 2. PLAN OF TOLEDO BEND DAM.

release pier near the center of the spillway. To release flows of less than 150 cfs, two 20-inch diameter conduits were also located in this low-flow pier.

Upstream of the spillway crest was an approach channel with a bottom width of 530 feet, side slopes of 1 on 3, and a horizontal bottom grade at elevation 125.0 feet. This channel extended from the upstream end of the spillway section for a distance of about 1,800 feet to natural grade. A sluiceway channel on a horizontal grade at elevation 100.0 feet with bottom width of 20 feet and 1 on 3 side slopes was provided to carry low flows. This low flow channel ran parallel to the dam, thence down the centerline of the spillway approach channel to the sluiceway in the low flow release pier. The alignment of the approach channels can also be seen in Figure 2.

Theoretical Performance of the Stilling Basin

The stilling basin below the spillway was designed to dissipate the energy of the spillway discharge through a hydraulic jump. The elevation of the stilling basin floor was computed from the hydraulic jump equation based on the conservation of linear momentum. The depth upstream from the hydraulic jump was determined from the energy equation for free-surface flow. Friction losses in the spillway chute were evaluated by application of the Manning formula, Memorandum No. 2, August 1962.

In order to induce additional energy dissipation and thus produce a shallower hydraulic jump, two rows of baffle blocks and an end sill were used. It was estimated from studies made by the Corps of Engineers that the downstream depth of the jump with the baffle blocks and end sill would be 90 percent of that previously computed from the hydraulic jump equation.

The theoretical water surface elevations versus discharge, with and without the use of baffle blocks and end sill, are plotted in Figure 3. Initial and probable minimum tailwater elevations versus discharge are also plotted in Figure 3. It is seen from these rating curves that for all discharges up to the spillway design discharge, the theoretical water surface produced from the jump is lower than the tailwater depth. This analysis, therefore, indicates that for all discharges up to the spillway design discharge, the jump will remain within the stilling basin.

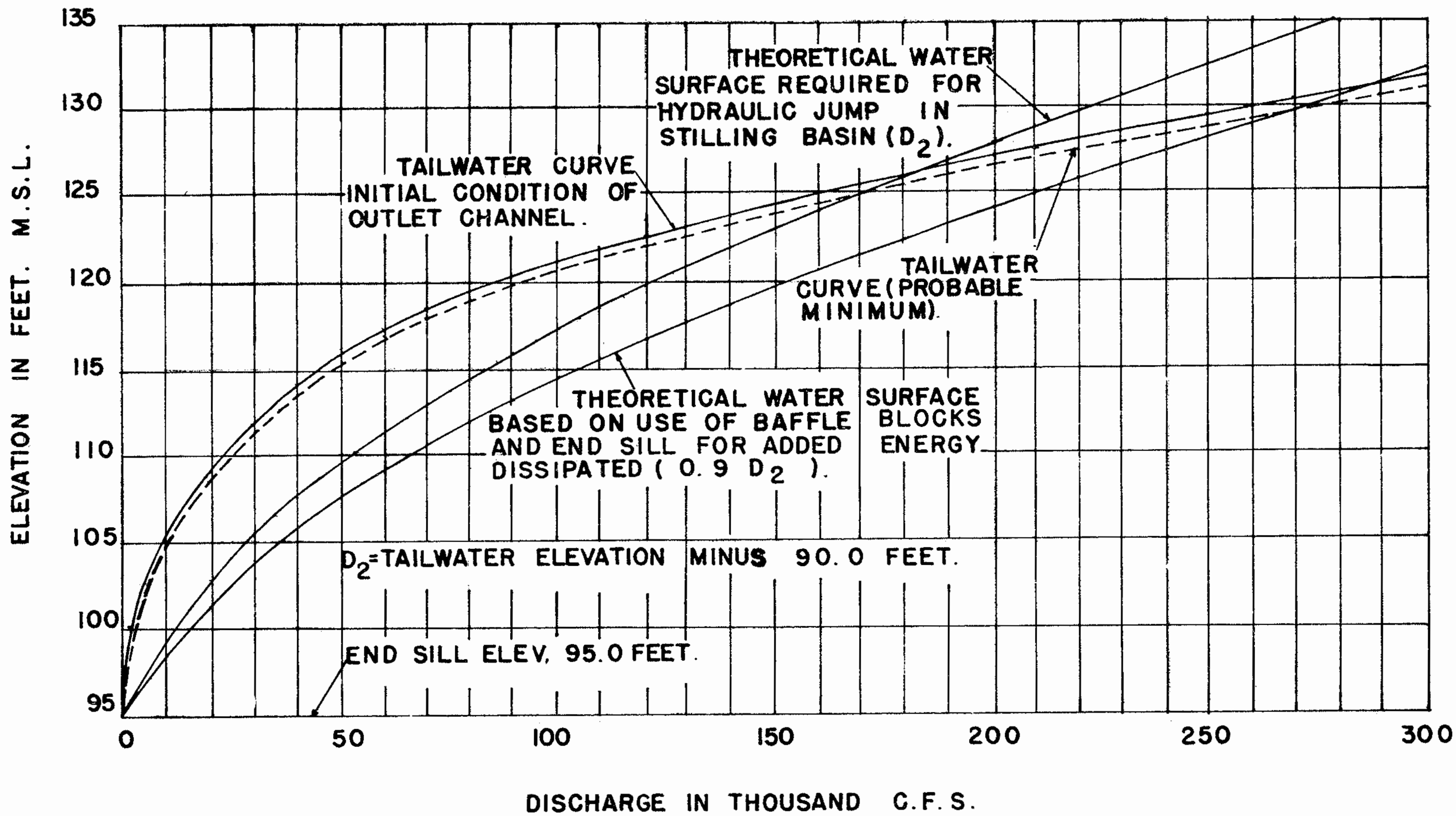


FIGURE 3. TAILWATER RATING CURVES AT STILLING BASIN.

OBJECTIVES OF MODEL STUDY

Due to the complexity of the flow pattern in the spillway area of a dam such as at Toledo Bend, the only feasible method to study the flow conditions in the approach channel, spillway section, stilling basin, and discharge channel is by means of a hydraulic model. Since it was desirable to investigate the hydraulic performance characteristics of the spillway section of the Toledo Bend dam, a hydraulic model was constructed in order to determine the following characteristics:

1. Water surface profiles through the spillway and stilling basin.
2. Effectiveness of the stilling basin of designed length.
3. Effect of curved approach channel.
4. Effect of curved outlet channel.
5. Operation with various floodgates open to develop gate operating sequence.
6. Operation of low-flow sluiceway alone and in combination with spillway.
7. Erosion of rip-rap in spillway discharge channel.
8. Bank erosion near wing walls.

REQUIREMENTS FOR SIMILITUDE

For a model to represent accurately a prototype in all respects, similitude requires that geometric, kinematic, and dynamic similarity be maintained between the model and prototype. An undistorted model is geometrically similar to a prototype when the ratios of corresponding lengths in the model and prototype are always equal to a constant, the scale ratio L_R. Kinematic similarity requires that the ratio between corresponding velocities and accelerations in the model and prototype also be equal to constants which can be written in terms of the scale ratio. Dynamic similarity will be attained if the forces which control the flow are in the same relative ratio in the prototype and in the model.

For spillways such as that used at the Toledo Bend Dam, inertia and gravity are the principal forces which affect the flow over the spillway and in the stilling basin. Although viscous and surface tension forces are present, they are very small in relation to inertia and gravity forces in the prototype and consequently have a negligible effect. If a model is very small, viscous and surface tension forces will become relatively more significant in the model than in the prototype, and there will be poor dynamic similarity. If a model is large enough the viscous and surface tension forces will be negligible and similarity will be achieved when the inertia and gravity forces are in the same ratio in model and prototype.

The relative importance of the inertia and gravity forces in the system can be determined from the numerical value of the ratio of these two forces as represented by the Froude number. In its usual form, the Froude number is

$$\mathbb{F} = V/\sqrt{gl}$$

where V is a velocity and l is some characteristic length. Since dynamic similarity requires that the relative magnitudes of the inertia and gravity forces be the same in the model and prototype, then

$$\mathbb{F}_m = V_m/\sqrt{gl_m} = V_p/\sqrt{gl_p} = \mathbb{F}_p$$

Since the acceleration of gravity is the same in model and prototype, the time scale can be related to the scale ratio as $T_R = L_R^{1/2}$. With geometric similarity represented by the scale ratio, the time ratio known, and dynamic similarity given by the Froude number, the ratios between the various flow conditions in the model and prototype can all be written in terms of the scale ratio. Table 1 summarizes the undistorted model-prototype relations applicable to this model study. In determining the relations in Table 1, it has been assumed that the same fluid (water) is used in both the model and prototype.

Apart from size and flow rate considerations, the scale ratio must be chosen so that the physical characteristics of the flow in the model are similar to those in the prototype. For large spillways, the Reynolds number is large and the flow is normally considered to be completely turbulent. For a similar condition to exist in a model, the Reynolds number in the model must be sufficiently high to insure that the flow is completely turbulent and that conventional free surface resistance equations are applicable.

Table 1. Summary of Model-Prototype Relations.

L_R	length ratio	L_R
T_R	time ratio	$L_R^{1/2}$
A_R	area ratio	L_R^{2}
v_R	volume ratio	L_R^{3}
V_R	velocity ratio	$L_R^{1/2}$
Q_R	discharge ratio	$L_R^{5/2}$
q_R	discharge/unit width ratio	$L_R^{3/2}$
p_R	pressure ratio	L_R
F_R	force ratio	L_R^{3}
$\mathbb{R}_R$	Reynolds number ratio	$L_R^{3/2}$
$\mathbb{F}_R$	Froude number ratio	1
g_R	gravity ratio	1
ρ_R	density ratio	1
μ_R	viscosity ratio	1

DESIGN AND CONSTRUCTION OF THE MODEL

Equality of Froude numbers was used as the basis for similitude in the Toledo Bend Dam spillway model. On this basis, the ratios of other characteristics such as length and flow rate were determined to establish a feasible scale ratio. After careful consideration of such factors as available space, existing maximum flow rate, minimum water depths, and model Reynolds numbers, a length scale ratio of 1:100 was chosen for an undistorted model of the Toledo Bend Dam spillway section. With this scale ratio, the principal variables of interest in the prototype are related to those in the model as shown in Table 2.

Table 2. Principal Model-Prototype Variables.

Lengths	$L_p = 100\ L_m$
Areas	$A_p = 10{,}000\ A_m$
Velocities	$V_p = 10\ V_m$
Discharge	$Q_p = 100{,}000\ Q_m$
Density	$\rho_p = \rho_m$

From the discharge relationship given in Table 2, it is found that the model design flood is 2.90 cfs, and the maximum probable flood is 3.29 cfs.

Also, since the flow in both the prototype and the model was assumed to be turbulent, the Reynold's number for both was checked in order to ascertain the validity of this assumption. It was found that the minimum Reynold's number in the model was greater than about 5×10^5, so that the operation of both the model and prototype were well in the range of turbulent flow.

Area and Location

The model of the Toledo Bend Dam spillway section covers an area which extends upstream of the spillway crest a distance of about 2,400 feet and downstream of the spillway crest about 2,800 feet. This made it possible to model both the spillway approach channel and the downstream spillway discharge channel including the curved-sections of these channels. The model also includes a section of the approaching sluiceway and low-flow release outlet channels and topography immediately adjacent to the spillway section. The outline of the area covered by the model is shown in Figure 2.

The model was constructed in the Hydraulics Laboratory on the Main Campus of The University of Texas, and located to make use of permanent equipment such as pumps, head tank, and piping for recirculating water flow.

Construction

The model was contained within concrete block walls approximately two feet high. The sides of the wall facing the model were lined with a polyethylene membrane and sealed to the floor to prevent leakage.

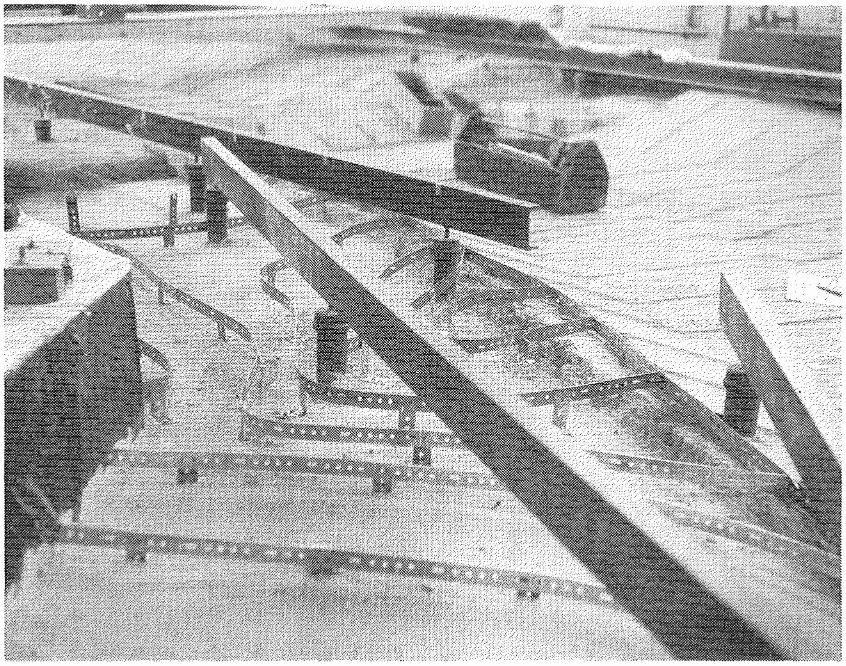
Before any of the topography or channel section was built, steel rails were set on both sides of the channel parallel to the channel centerline. These rails were supported on short steel pipe columns anchored to the laboratory concrete floor and were adjustable at each support column. The rails in turn served as reference points during construction of the model and later as supports for the instrument carriage from which water depths were measured.

The approach and discharge channel sections were then built by the use of templates made from sections of perforated aluminum strips cut and placed to conform to the channel section. These templates were placed at intervals of one foot along the channel centerline, which corresponded to 100 foot stations on the prototype. A dumpy level was used to set each template at its proper vertical position. Horizontal control was based on the steel rails used for the instrument carriage, and on the centers of the curves, all of which were set with a transit and tape. All horizontal and vertical control in the model was kept to an accuracy of 1/16 inch or better.

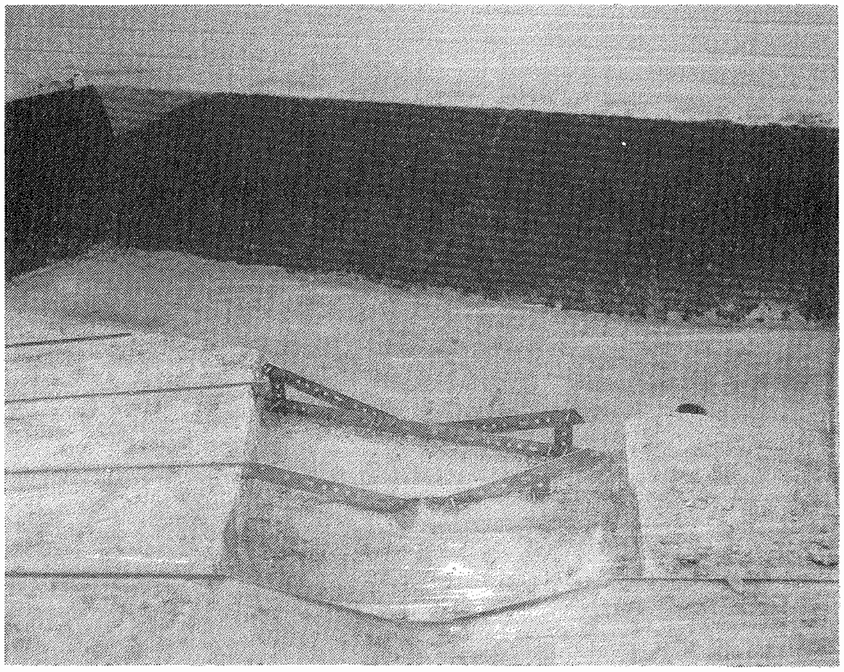
The channel section was molded to approximate grade by means of a weak mixture of concrete on a sand and gravel base. A space was left on both sides of each template and filled with a richer sand and cement mortar to hold the template permanently in its proper position. These channel templates can be seen in Figure 4. The channel sections were brought to final grade by filling the remaining 1/2 inch with White Portland plaster and troweling to a very smooth finish.

The topography was also built on a sand and gravel base topped with a concrete mixture at approximately 4 to 6 inches below finished grade. Contour lines were transferred from the plans to corresponding points on the model. A metal strip similar to those used in the channel section was then bent to conform to the contour line and fixed in its proper vertical and horizontal position to form each of the contour lines as seen in Figure 4a. Additional weak concrete was placed to about 1/2 inch below finished grade. The final surfacing was a sand and cement mortar screeded between contour lines and finished with a wood trowel.

For the ogee spillway section, aluminum templates were carefully machined to reproduce the profile of the overflow section and concrete was

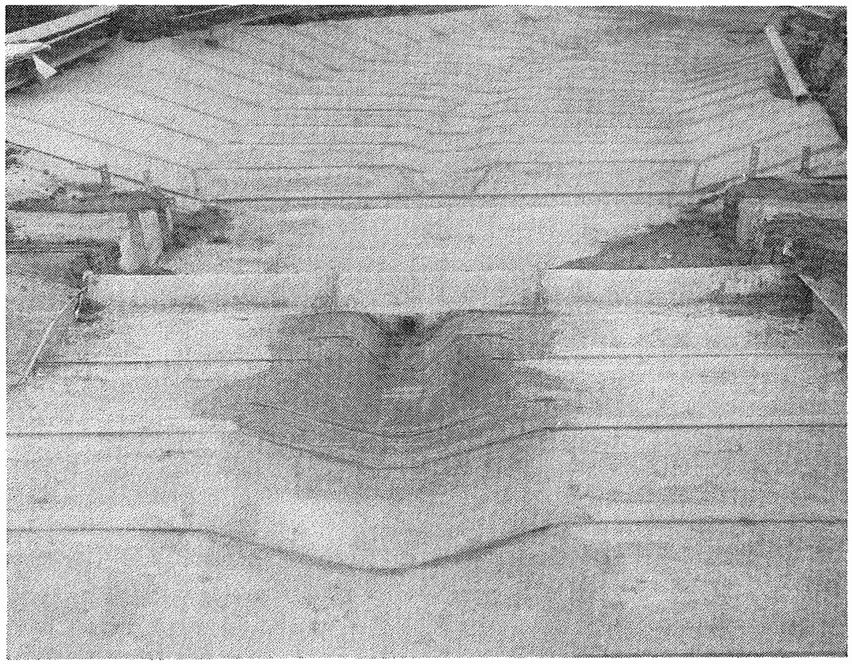


(a)

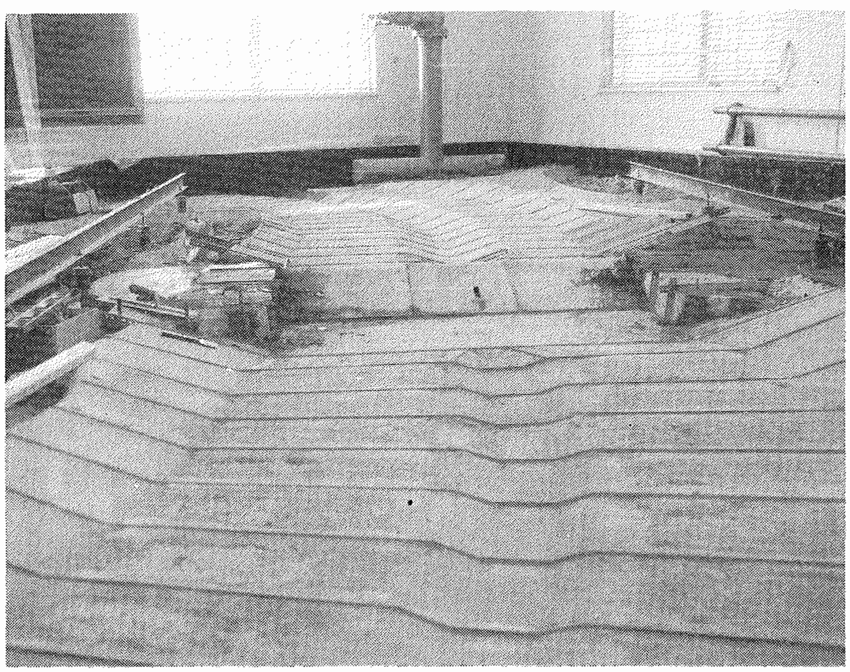


(b)

FIGURE 4. MODEL DURING CONSTRUCTION



(c)



(d)

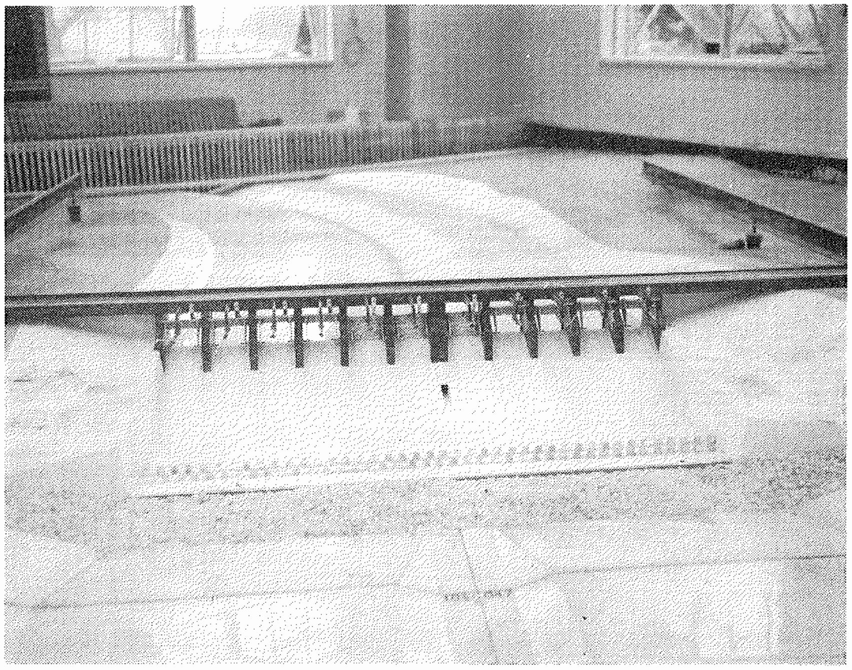
FIGURE 4. MODEL DURING CONSTRUCTION

carefully finished to this profile for the entire length of the overflow section. The piers were then made from wood, fitted to the profile of the overflow section and fastened in place at the proper locations. The wood piers were treated with resin to prevent wetting and swelling. Curved metal pieces of aluminum were mounted on pivots between the piers to reproduce the flow geometry of the tainter gates. A link connected each gate to an operating screw which was used to hold the gate in any desired position.

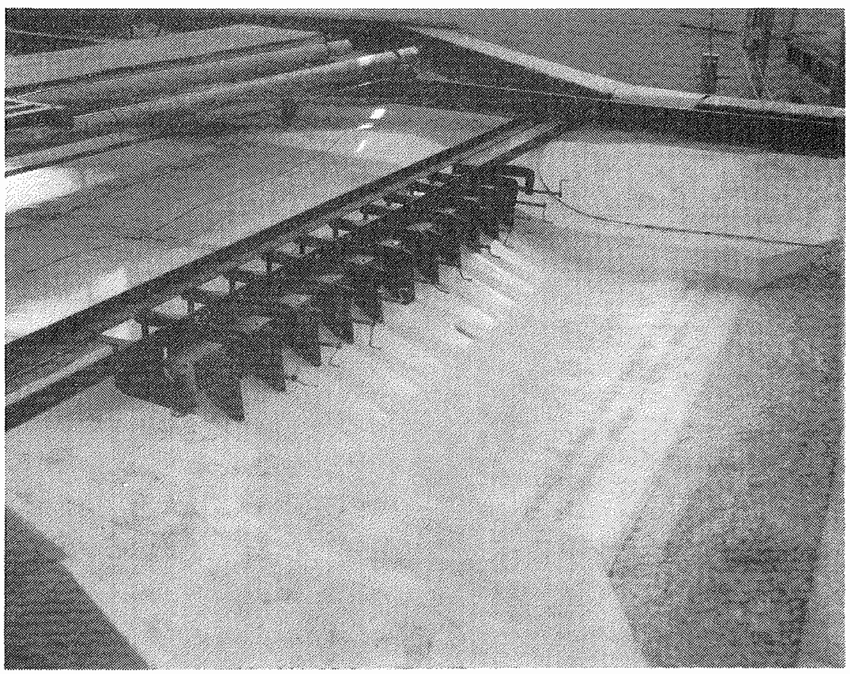
Baffle blocks in the stilling basin were made to scale from aluminum and attached to the floor of the stilling basin with screws. A solid aluminum strip formed the end sill of the stilling basin. For the original model the training walls were molded in concrete, but in the final model the training walls were made from wood and sealed with resin.

Downstream from the end sill of the stilling basin, the rip-rap was simulated by rock which was screened to reproduce the prototype size gradation specifications with a reduction by the linear scale of 1:100. No attempt was made to obtain rip-rap in the model such that the shape of the stones would be comparable to that in the prototype, and it was not intended that the model rip-rap would simulate the scour resistance of the prototype rip-rap. Rather, it was intended to reproduce only a generally similar roughness pattern in that section of the model. Two photographs of the completed model are shown in Figure 5.

Flow was brought into the model forebay through a pipe, manifold, and baffle arrangement at the upstream end of the model. To control the tailwater elevation downstream of the spillway, three independently adjustable tailgates were constructed across the downstream end of the model. Discharge waters passing over the tailgates were returned to the laboratory



(a)



(b)

FIGURE 5. COMPLETED MODEL

sump through a small channel.

Alterations to the Model

During the testing program it was necessary to make a number of alterations to the model. Some of the alterations were very minor; however, some due to major design changes were quite extensive. The same methods used in construction of the original model were in general used for the alterations.

Measuring Instruments

Flow into the forebay of the approach channel was metered with a U. S. Bureau of Reclamation combination Venturi-Orifice meter. A water manometer was used to measure head differentials for low flows and a mercury manometer was used for higher flows. Both manometers could be read to the nearest 0.001 foot.

Elevations of water surfaces were measured by Lory Type-A point gages mounted on an aluminum instrument carriage which rested across the parallel steel rails. The point gage was attached to a moveable platform on the carriage for determining average water surface elevation across the model. A neon signal light was used to indicate contact between the point gage and the water surface. Measurements were made to the nearest 0.001 foot. Steel tapes were mounted along the lengths of the carriage and the steel tracks, allowing the gage to be returned to any desired location.

Velocity measurements were taken with two instruments, a pitot tube and a Price pygmy current meter. The pitot tube was attached to the platform on the instrument carriage and connected to a water manometer, which could be read to the nearest 0.001 foot. The pitot tube was used for determining velocity distributions along cross-sections in the stilling basin area. The pygmy current meter was used to measure velocities downstream from the stilling basin. The meter used was obtained from the U. S. Geological Survey and was rated by the National Bureau of Standards (7-12-63). A stop watch was used for all time measurements.

TEST PROGRAM

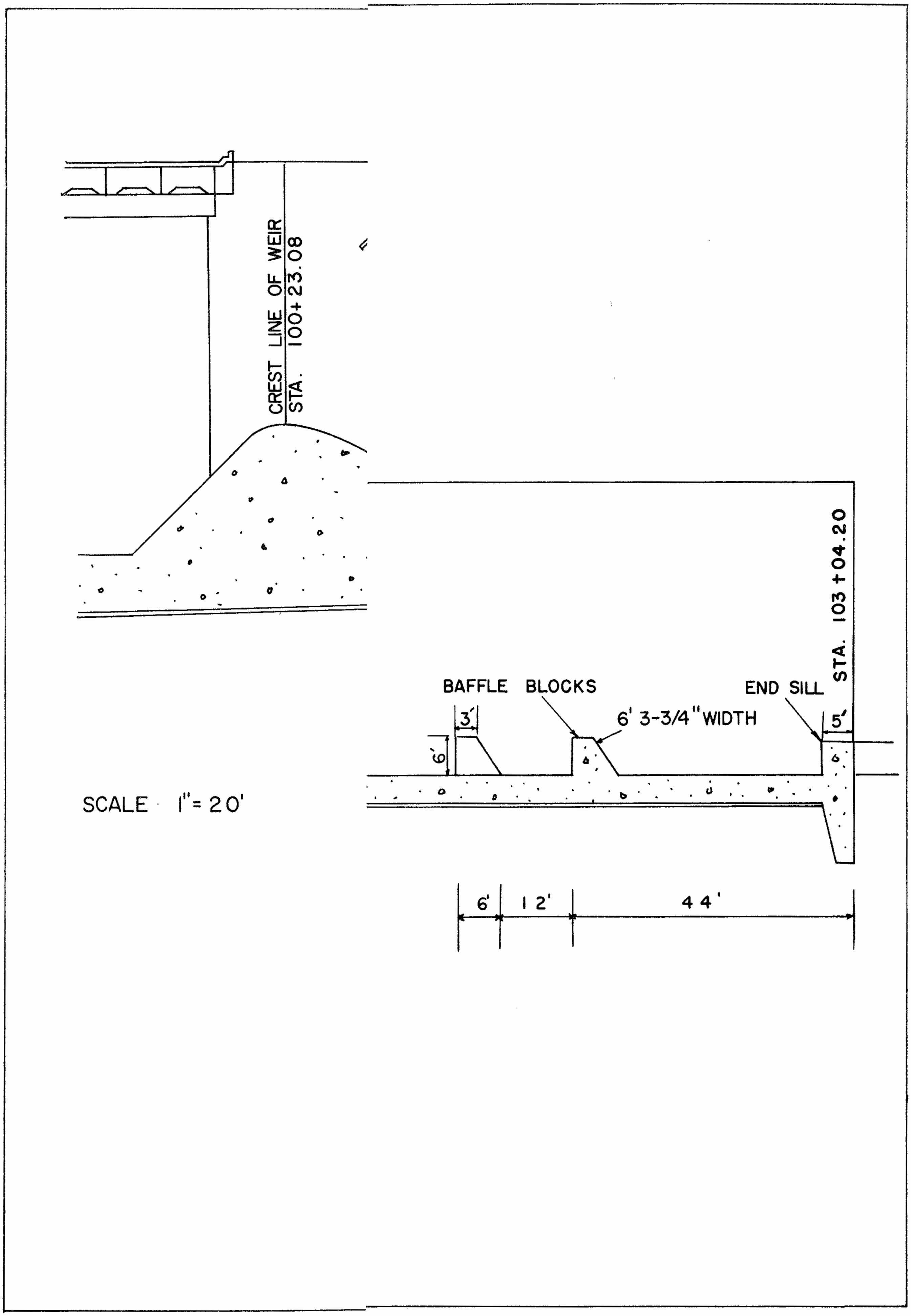
The general approach used in the model investigation was to operate the model at several critical discharges and observe points of potential difficulty. If modifications were necessary to improve flow conditions, the modifications were worked out experimentally on the model before the final measurements of performance were made.

Tests for Original Spillway Design

The original model was constructed according to Design Memorandum No. 2, "Spillway," dated August 1962, and a copy of the plans for the spillway of the Toledo Bend Dam as issued for construction and transmitted to the University of Texas in January 1964. As seen in Figure 6, water passing over the overflow section flowed down a chute on a slope of 1 on 3 and on to a horizontal stilling basin 130 feet long containing two rows of baffle blocks. In the original model the bottom of the trapezoidal downstream channel was at elevation 105, or 10 feet higher than the top of the stilling basin end sill. Also along the center of the downstream channel was a trapezoidal pilot channel with a 50 foot wide bottom at elevation 95 feet.

This original spillway model was operated at discharges corresponding to the spillway design flood and the test flood No. 2 as well as at lower discharges. Four critical areas were identified as follows:

1. At the spillway entrance there was a severe drawdown adjacent to end piers. The drawdown was more severe at the right abutment than at the left. The surface wave and an attendant longitudinal vortex caused water to pile up against the tainter gate pivot.


CREST LINE OF WEIR
STA. 100+23.08
STA. 103+04.20
BAFFLE BLOCKS
END SILL
6' 3-3/4" WIDTH
3'
6'
5'
SCALE 1"= 20'
6'
12'
44'

2. At discharges higher than 290,000 cfs and with the tailwater adjusted according to Figure 3, reproduced from Design Memorandum No. 2, "Spillway," the hydraulic jump produced waves which washed over the training walls and on to the excavated areas behind them.

3. The sloping banks of the trapezoidal channel downstream from the stilling basin were subject to attack by a vortex developing on each side of the channel just below the expansion from the rectangular stilling basin section to the trapezoidal channel section.

4. It was observed that velocities were high in the downstream channel so that scour would be anticipated below the rip-rap. It was felt that the scour might progress upstream and undermine the rip-rap.

After methods had been worked out to improve conditions in the four areas mentioned above, the model was operated at several discharges corresponding to prototype floods. Table 3 summarizes these discharges for both model and prototype.

Table 3. Test Floods.

Test Flood	Prototype Discharge	Model Discharge
1	60,000 cfs	0.6 cfs
2	132,000 cfs	1.3 cfs
3	290,000 cfs	2.9 cfs
4	330,000 cfs	3.3 cfs

For these discharges the water surface profiles and velocities were measured to describe the flow patterns in the model with all of the spillway gates open. In each case the tailwater in the model was adjusted by the tailgates at the downstream end of the model to agree with the tailwater rating curve supplied in Design Memorandum No. 2.

A series of tests were then made to determine the effect of gate operation on the performance of the stilling basin. During these tests the headwater was maintained at 172 feet by adjusting the flow rate to correspond to the gate openings. The tailwater was then set to the appropriate tailwater elevation corresponding to that discharge.

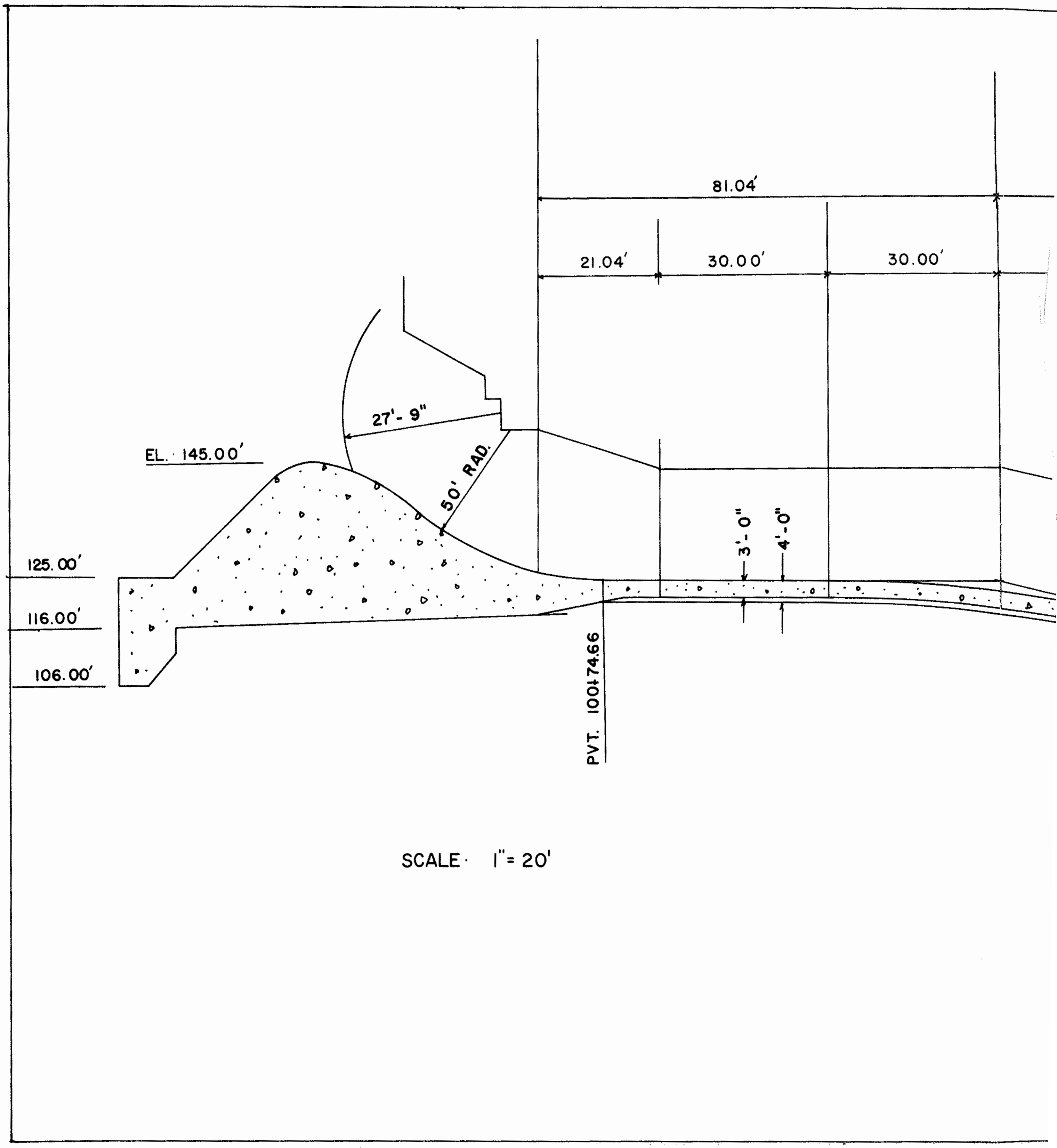
After this schedule of testing was completed the bottom of the downstream trapezoidal channel was lowered ten feet so that the entire bottom was level with the original pilot channel. This corresponded to an eroded downstream channel and completely eliminated the pilot channel. Several exploratory runs were made with the modified downstream channel. It was evident that the modification of the downstream channel had no effect on the performance of the stilling basin portion of the model other than to decrease the velocity in the downstream channel due to the increase in depth that resulted from lowering of the bottom. Data related to the tests on the original spillway have not been included in this report. However, this data is available through the Hydraulic Engineering Laboratory at the University of Texas.

Tests for Modified Spillway Design

In December 1964 it was learned that important modifications to the spillway profile were under consideration. On February 25, 1965 details of the new spillway profile and revised training walls were transmitted to the University of Texas. In the revised design the overflow section was unchanged, but immediately downstream from the overflow section was a section of channel 69.38 feet long with a total fall of 2.47 feet. At the end of this channel a vertical curve led into a chute on a slope of 1 on 4 which led down to a horizontal apron stilling basin 120 feet long with the apron at elevation 90, the same as for the original design. The two rows of baffle blocks were located 44.0 feet and 62.0 feet respectively from the downstream face of the end sill. This, however, placed the baffle blocks closer to the upstream end of the stilling basin. The elevation of Figure 7 shows the modified spillway design as well as the revised location of the baffle blocks, which was determined from the model operation.

The exploratory tests with the new design indicated that the hydraulic jump in the stilling basin was less stable than for the previous condition. A slight drop in the tailwater elevation would cause the jump to wash out of the stilling basin.

A modification to the stilling basin was developed by experiment. With the modified stilling basin, measurements were made of the water surface elevation and of velocities at various points in the model. Several gate operating sequences were then investigated to determine a convenient operating procedure that would also produce satisfactory flow conditions downstream. The results of the measurements and the gate operating studies are presented in the following section.



SCALE · 1" = 20'

fig 1

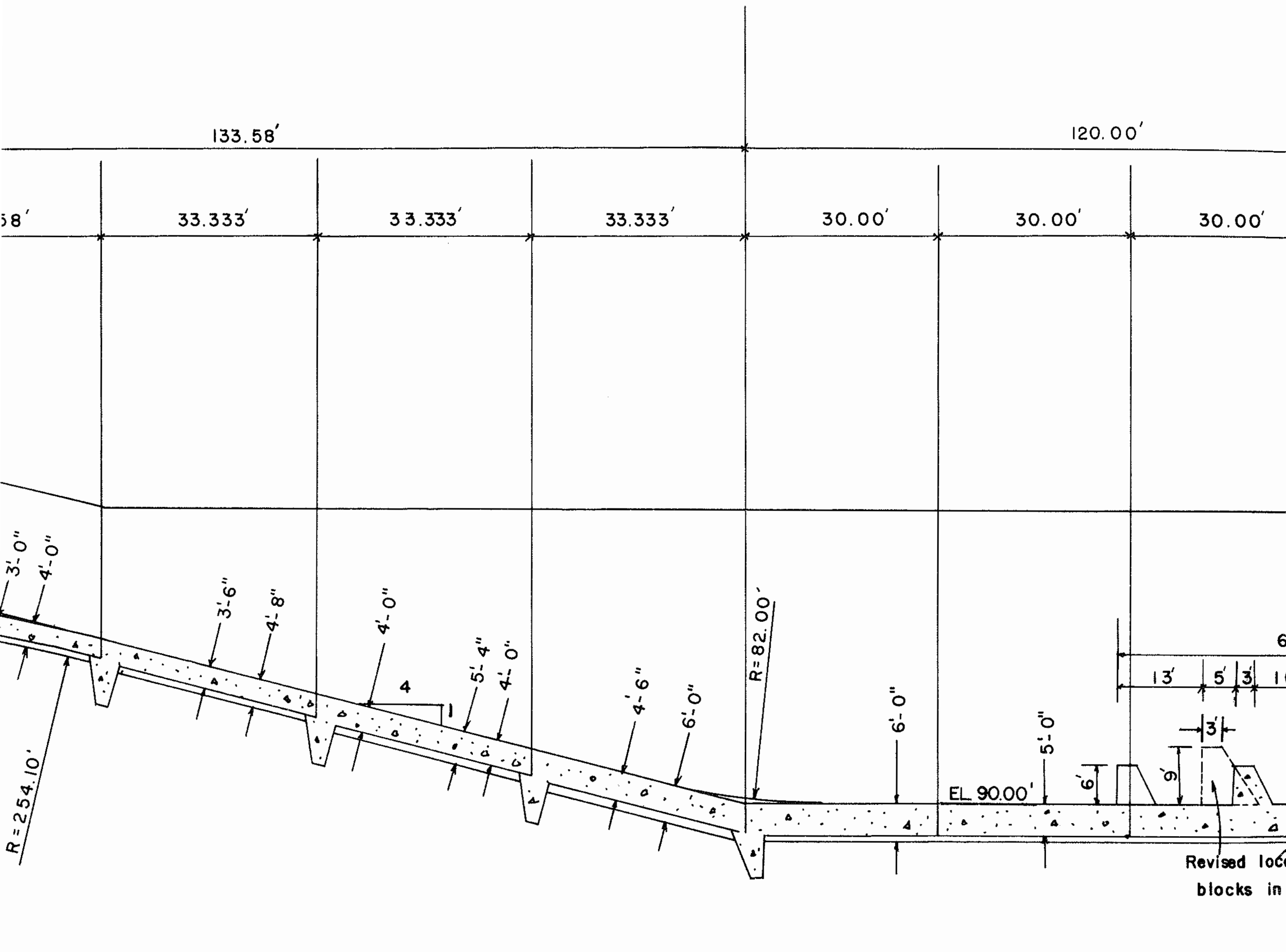


FIGURE 7. MODIFIED SPILLWAY DESIGN

fig 2

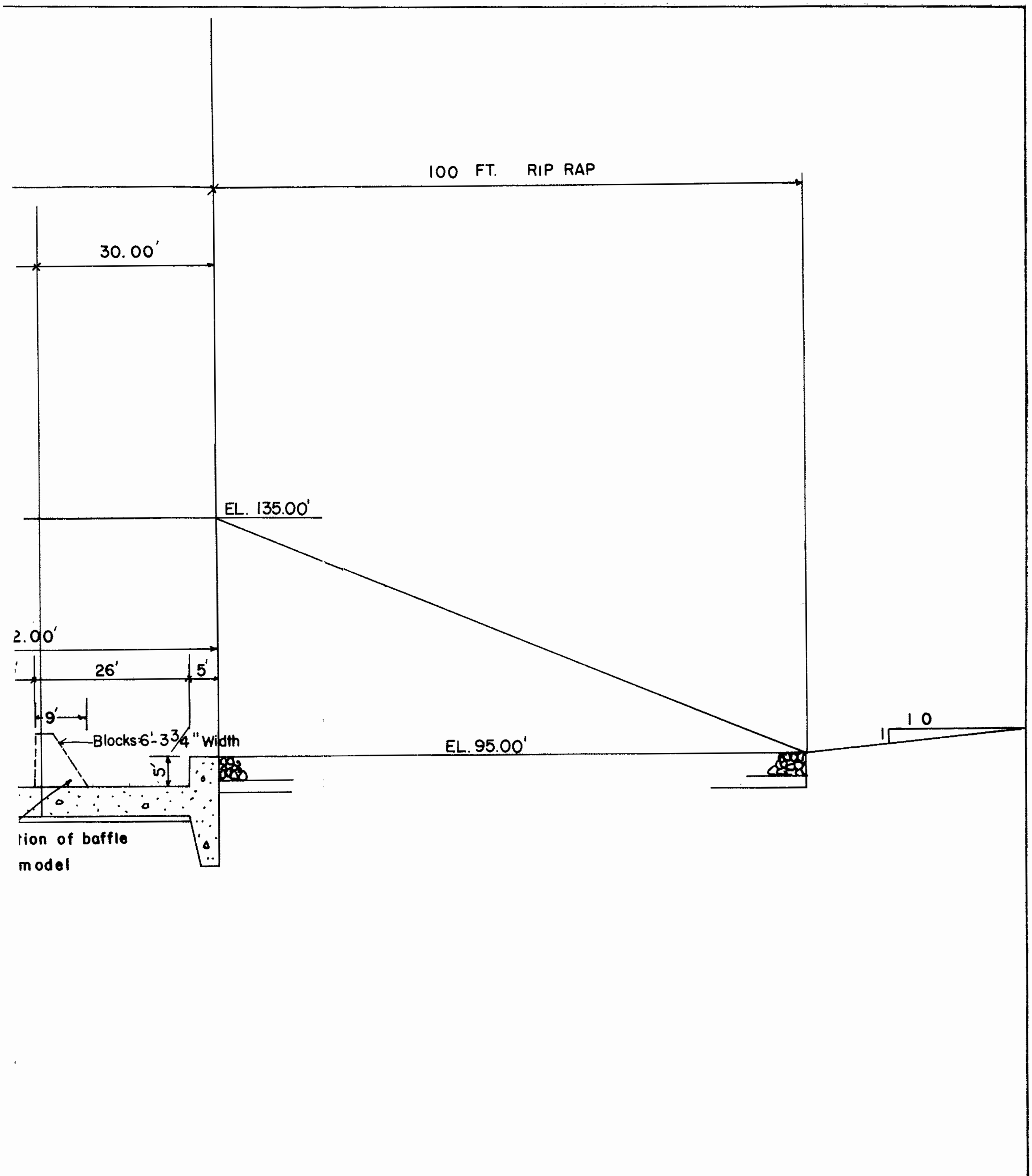

100 FT. RIP RAP
30.00'
EL. 135.00'
2.00'
26'
5'
9'
Blocks 6'-3 3/4" Width
5'
EL. 95.00'
1
10
tion of baffle
model

fig 3

EXPERIMENTAL RESULTS AND DISCUSSION

A complete schedule of the tests made on the model is included in the Appendix. This schedule also includes tests made on the original model before the major alterations to the spillway shape. Measurements of water surface elevation and velocity in the original chute, stilling basin, and downstream channel are of no significance for the final structure and are not included in the report. Measurements and observations made to improve the flow conditions upstream of the overflow section and near the right abutment were made on the original model. Since the alterations made below the overflow section had no effect on upstream conditions, these measurements are included in the report. The final recommendations include the modifications to the approach section developed in the original series of tests.

Spillway Rating Curve

The discharge into the model was measured as the flow passed through a calibrated combination Venturi-Orifice meter. This flow measurement was accurate within $\pm$ 2 percent. Figure 8 shows a comparison of the spillway rating curve taken from Design Memorandum No. 2 and the experimental rating curve as measured on the model. Reservoir elevations were measured at the centerline of the approach channel and 400 feet upstream of the spillway crest. The measurements indicated very good agreement between the calculated and the experimental rating curve.

When modifications were made to improve the flow conditions through the end gate openings near the abutments, a check on the rating curve showed that there were no measurable changes in the head-discharge relationship.

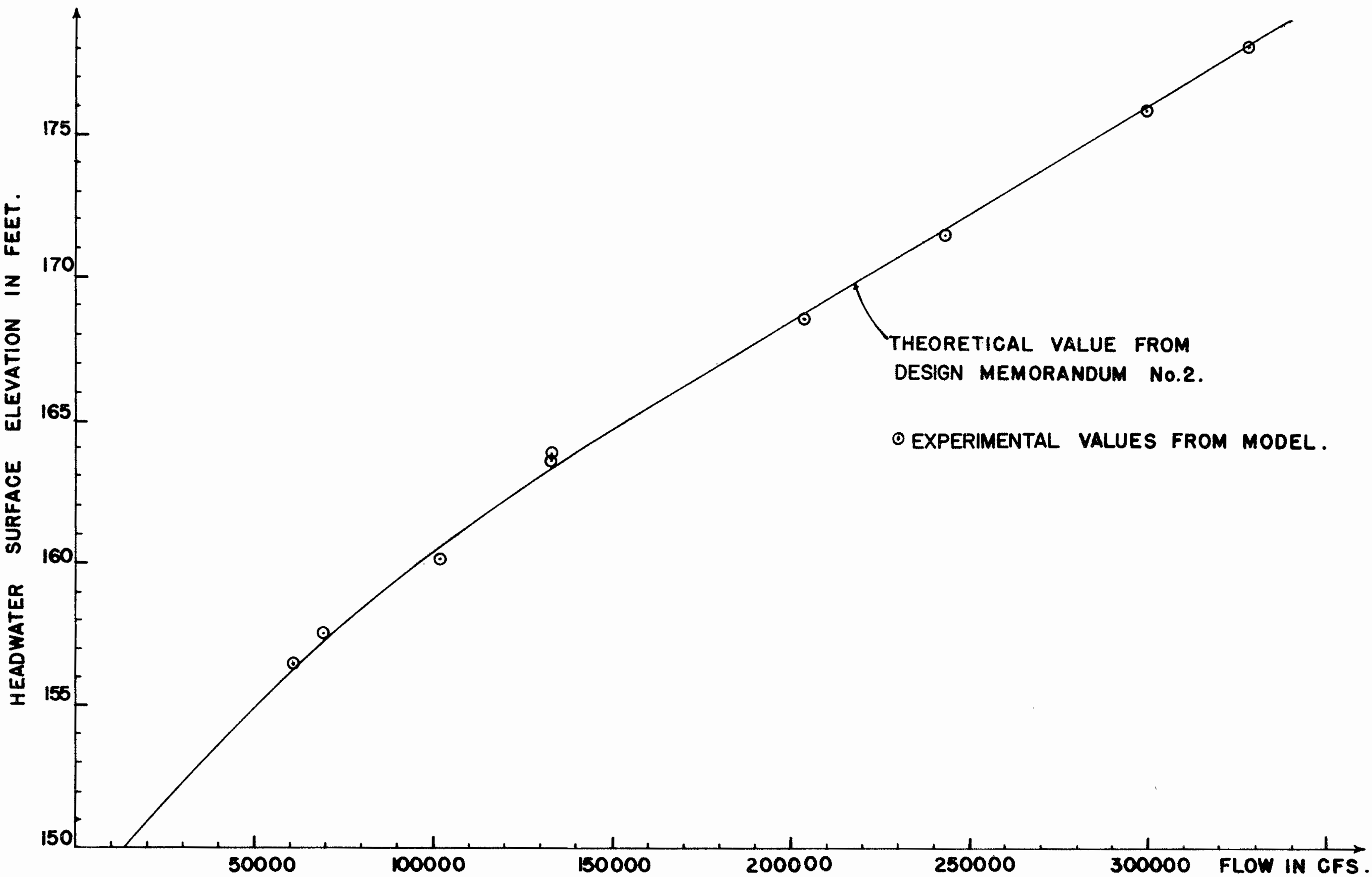


FIGURE 8. COMPARISON OF SPILLWAY RATING CURVES

Modification of Entrance to Spillway

At the entrance to the spillway there was a severe drawdown at the end piers on each side of the spillway. Figure 9 shows the profile of the water surface elevation taken along the spillway crest before any modifications were made. In Figure 9 a severe drawdown can be seen at the right abutment. A surface wave and an attendant longitudinal vortex which formed as the flow swept around the abutment caused water to pile up against the tainter gate pivot on the right side of the spillway training wall.

In order to correct the surface wave resulting from this severe drawdown, a number of different schemes were investigated including: a large increase in radius of the end pier, a training wall at 45° to the axis of the spillway, several training walls of different lengths normal to the spillway axis, and several dikes extending upstream from the main embankment and near the right spillway abutment. Satisfactory flow operation was obtained with the training wall located at 45° to the spillway axis, with the straight training wall at 90° to the spillway axis, and with two different designs for an upstream dike.

Because of the difficulties associated with the construction of the training walls, and because an upstream dike could also serve as an automobile turnout area, it was decided to use a dike extending upstream from the main dam embankment. It was observed that the design and location of the upstream dike was very critical. The dike functioned by creating a separation zone at its end which curved around to become tangent to the flow approaching the right abutment wall. Ideally the dike should have very steep sides. However, from a construction standpoint, steep side slopes are impractical and the design of the dike was finally selected as shown in Figure 10.

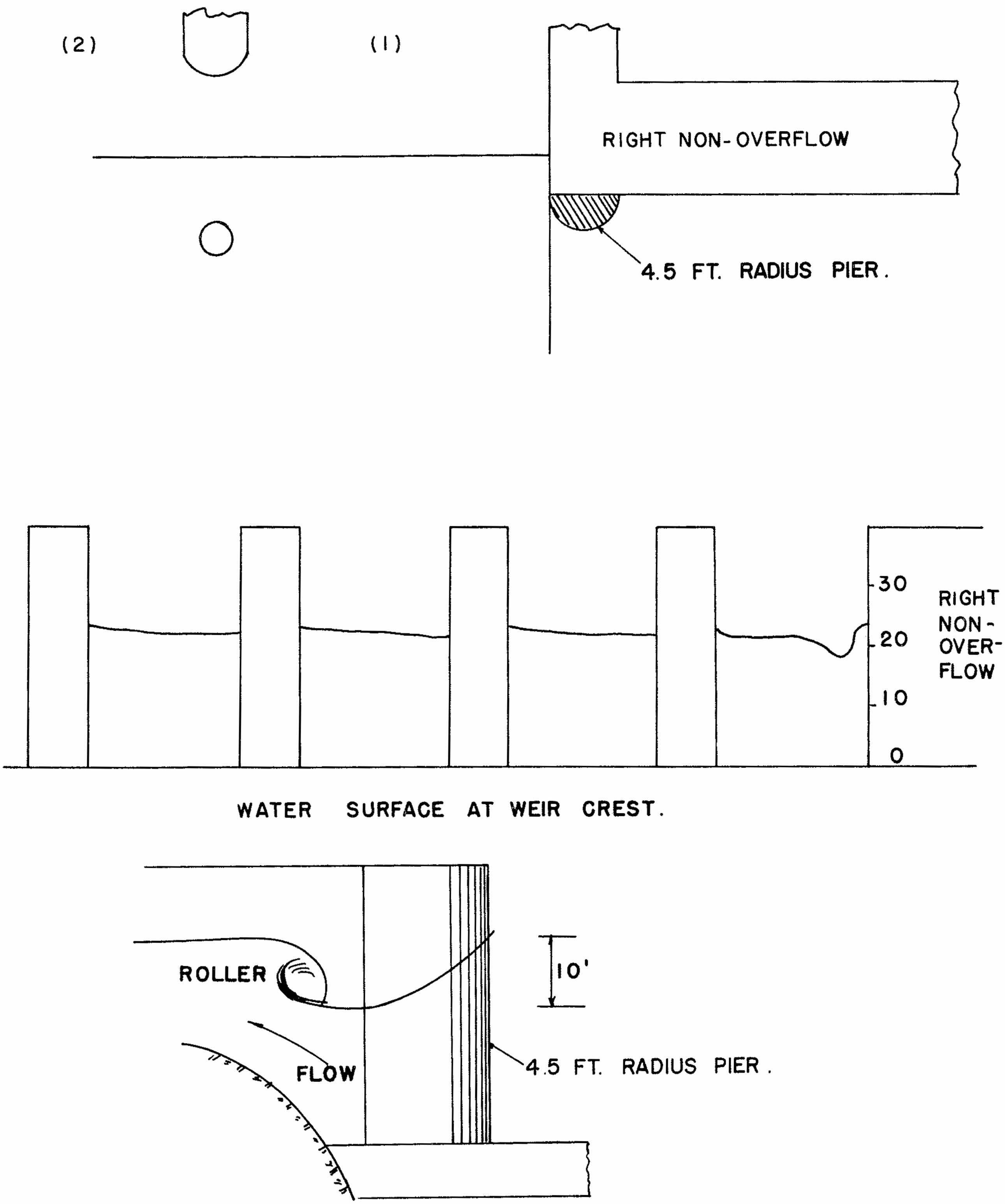


FIGURE 9. WATER SURFACE ELEVATION ALONG SPILLWAY CREST WITHOUT ENTRANCE MODIFICATION

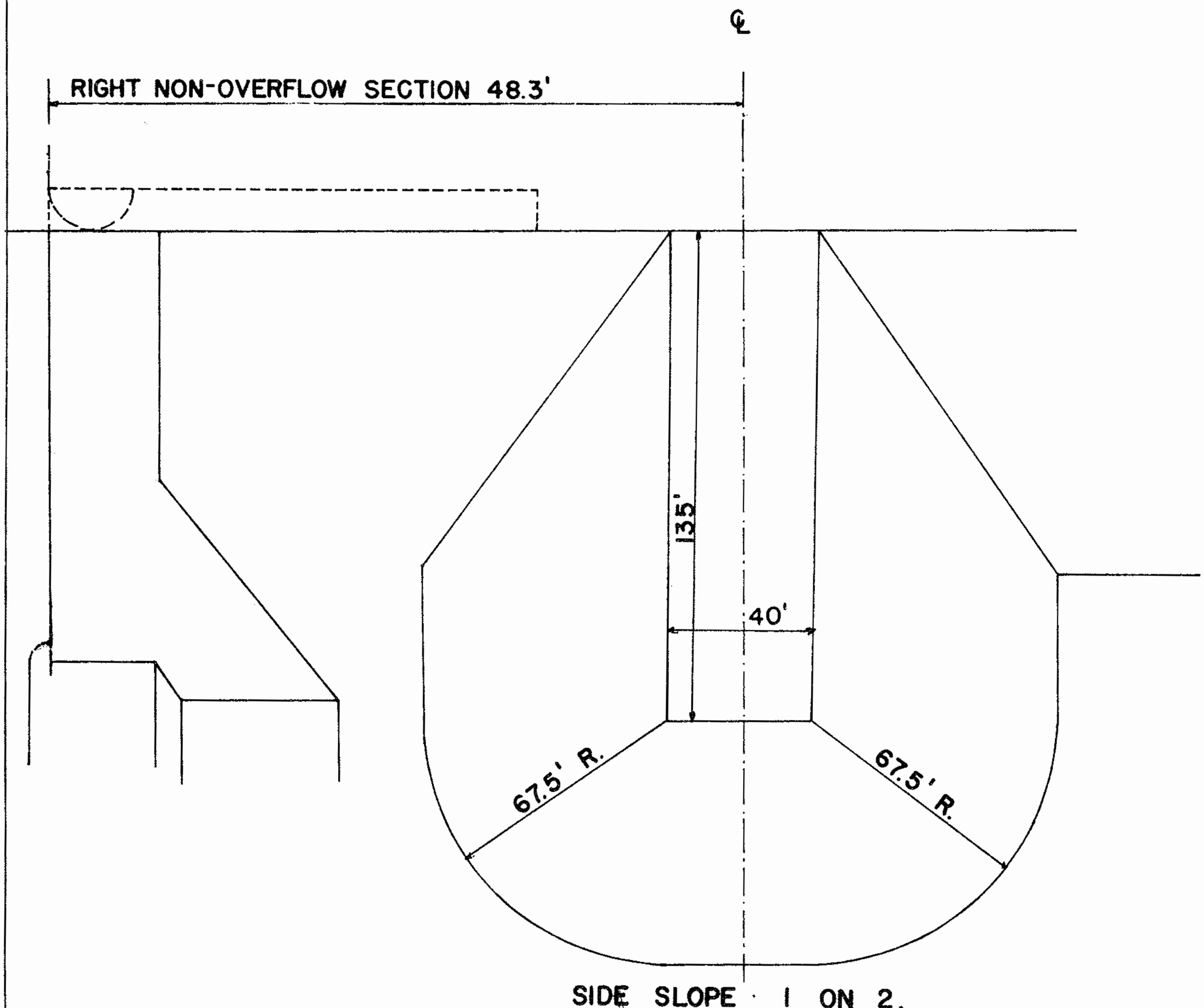


FIGURE 10. DIKE DESIGN

With the dike located as shown, only a moderate drawdown occurred at the right abutment, and the water surface elevation in the vicinity of the right tainter gate pivot was lowered sufficiently to eliminate the hazards of waves striking the tainter gate pivot at maximum discharge. At low discharges satisfactory flow conditions were obtained regardless of whether the dike was in place or not.

Modifications to the Stilling Basin

A major modification to the chute below the overflow section and to the stilling basin was necessary because of foundation problems and was made according to the dimensions shown in Figure 7. The important features of the modification were: a nearly horizontal chute leading away from the overflow section, the apron slope changed from 1:3 to 1:4, and the horizontal stilling basin length reduced to 120 feet.

With these modifications the spillway and stilling basin performed very satisfactorily up to the maximum discharge for Test Flood No. 2, see Table 3. At greater discharges, it was observed that the hydraulic jump in the stilling basin was less stable than for the originally designed spillway. A slight drop in tailwater elevation would allow the jump to wash out from the stilling basin. When the jump washed out of the stilling basin, it was always accompanied by very severe erosion of the downstream rip-rap. To improve the jump stability at the higher discharges, the stilling basin was modified by moving each of the two rows of blocks 13 feet downstream from their original position and increasing the height of each block to 9 feet. It was found that increasing the height of the end sill did not appreciably improve the jump stability. For this reason and because of the attendant changes involved, no modifications were made to the end sill. At the two highest discharges the operation of the stilling basin was observed visually for a range of tailwater elevations above and below the tailwater rating curve of Figure 3 taken from Design Memorandum No. 2. Table 4 shows for these discharges the rated tailwater, the minimum tailwater which would hold the hydraulic jump in the stilling basin, and the maximum tailwater used in the tests. The stilling action was satisfactory for the range of tailwater elevations between the minimum and the maximum used, and should be satisfactory for even higher tailwater elevations. At discharges less than 290,000 cfs the range of satisfactory tailwater elevations would increase.

Table 4. Range of Tailwater.

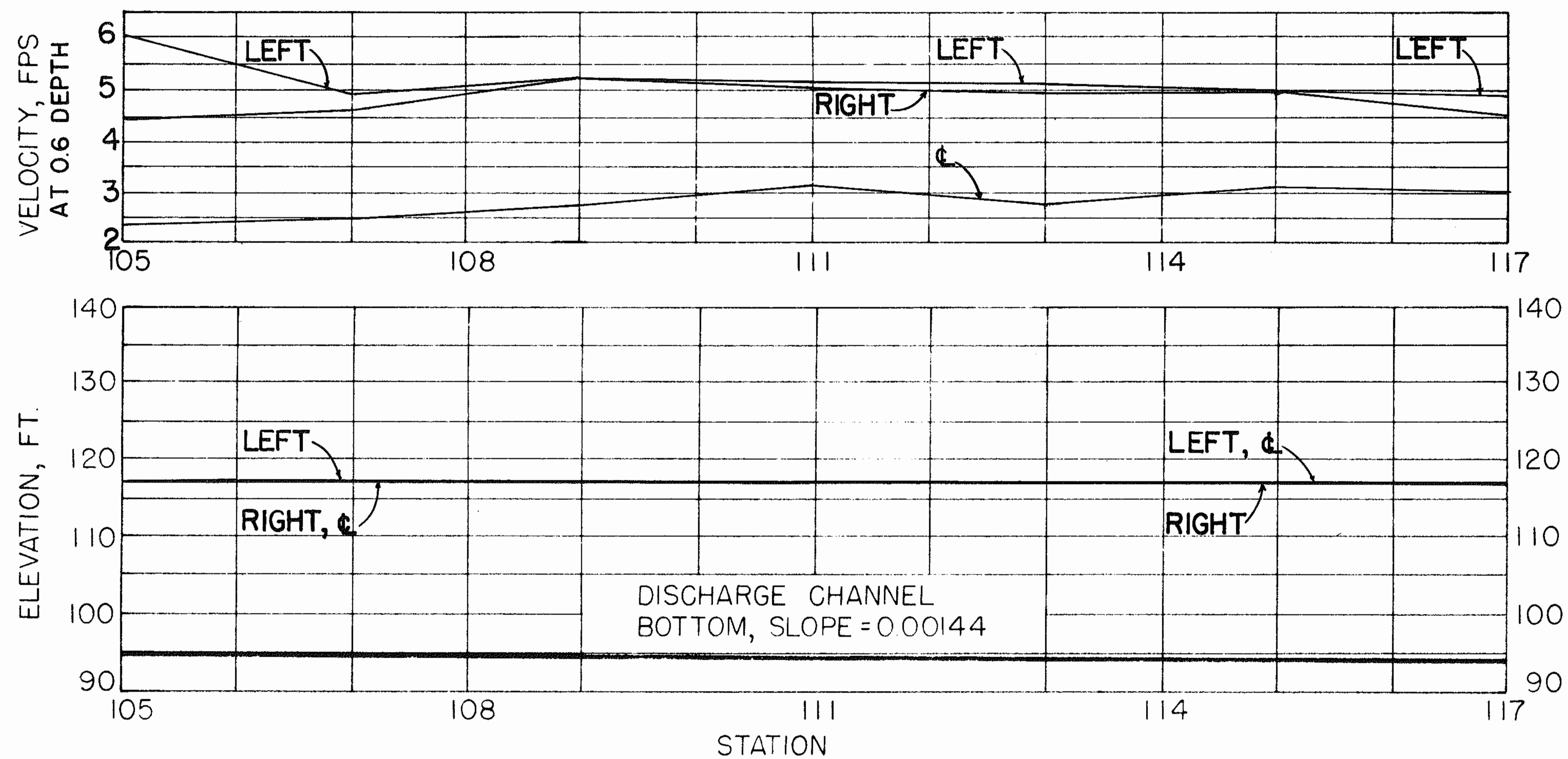
Discharge (cfs)	Tailwater Elevation (ft.)		
	Rated	Minimum	Maximum
290,000	131.0	126.5	134.0
330,000	132.0	129.0	135.0

Water Surface Elevation and Velocity Measurement

Measurements of the water surface elevation and velocity at selected points are presented in Figures 11, 12, 13 and 14 for the selected test floods of 60,000 cfs, 132,000 cfs, 290,000 cfs, and 330,000 cfs respectively. The upper part of the figure presents the water surface profile over the overflow section through the chute and stilling basin along a section cut through the centerline of gate no. 6. The lower part of this figure gives the velocities and water surface elevation in the downstream channel measured along the centerline of the channel and along lines 250 feet to the left and right of the centerline corresponding to the locations where the toe of the bank slopes intersect the channel bottom. These figures show that for all discharges the hydraulic jump stayed within the stilling basin. For the two lower discharges the super elevation due to the curve in the downstream channel was barely measurable. At the two higher discharges, however, super elevation of the water surface in the curve was evidenced by the higher elevation of the water surface near the left bank than near the right bank. The maximum velocity measured in the downstream channel at 6/10 depth was near the centerline and amounted to 14 feet per second at a discharge of 330,000 cfs. At this higher discharge the velocity in most of the downstream channel averaged about 12 feet per second. In general the velocities were higher near the right bank than the left bank as would be expected from the curvature of the downstream channel. This is compatible with the measured super elevation of the water surface. It should be emphasized that these velocities were measured with the entire downstream channel lowered 10 feet below the original elevation. Before the entire downstream channel erodes to this lower level, and when only the pilot channel bottom is at this low elevation, velocities will be greater. Model tests carried out on the original spillway before the entire downstream channel was lowered indicated average velocities could be as high as 17 feet per second.

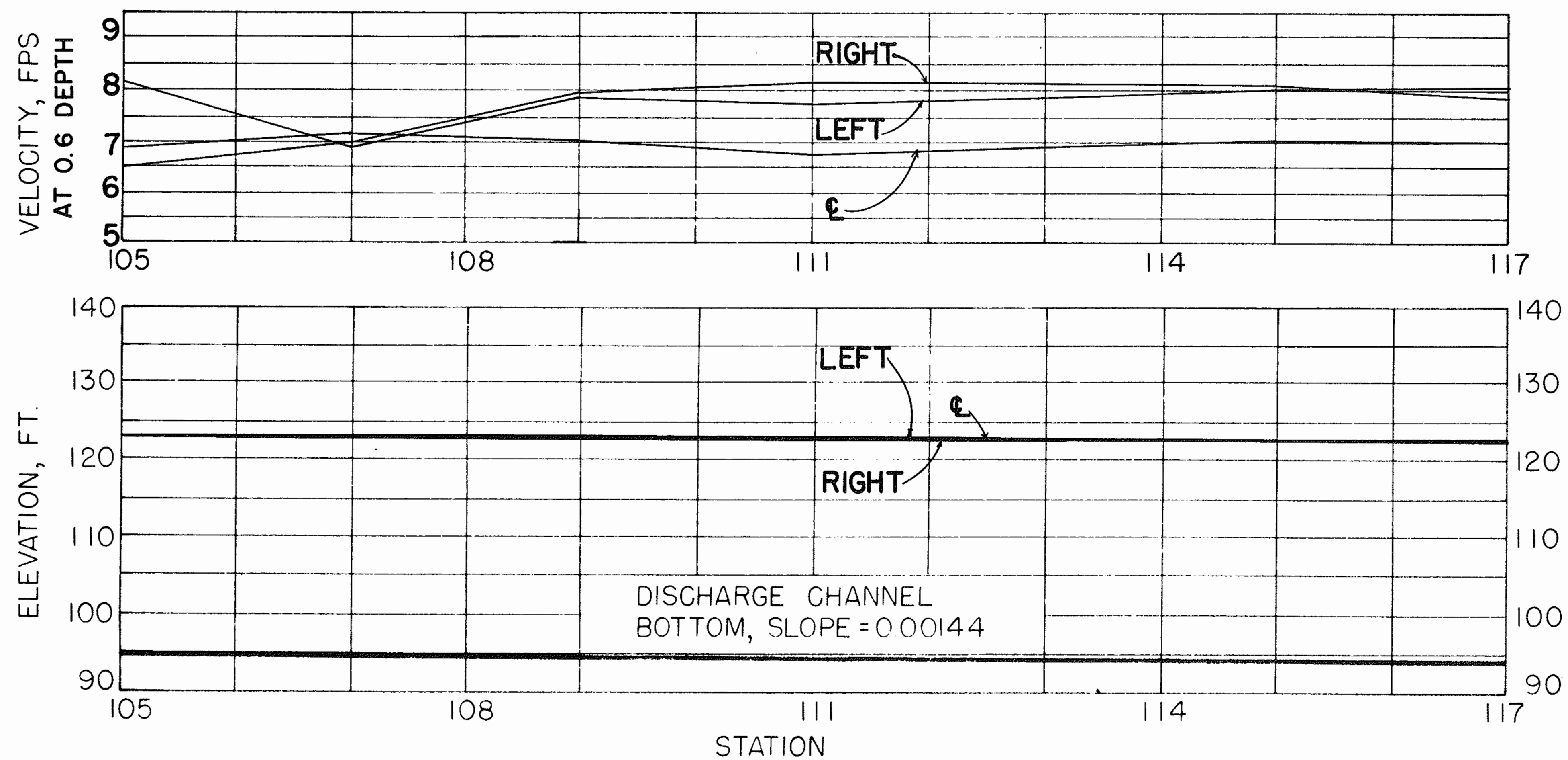
Gate Operation

A series of tests were run to explore various possible gate operating sequences and to observe their effect on the flow pattern in the chute and



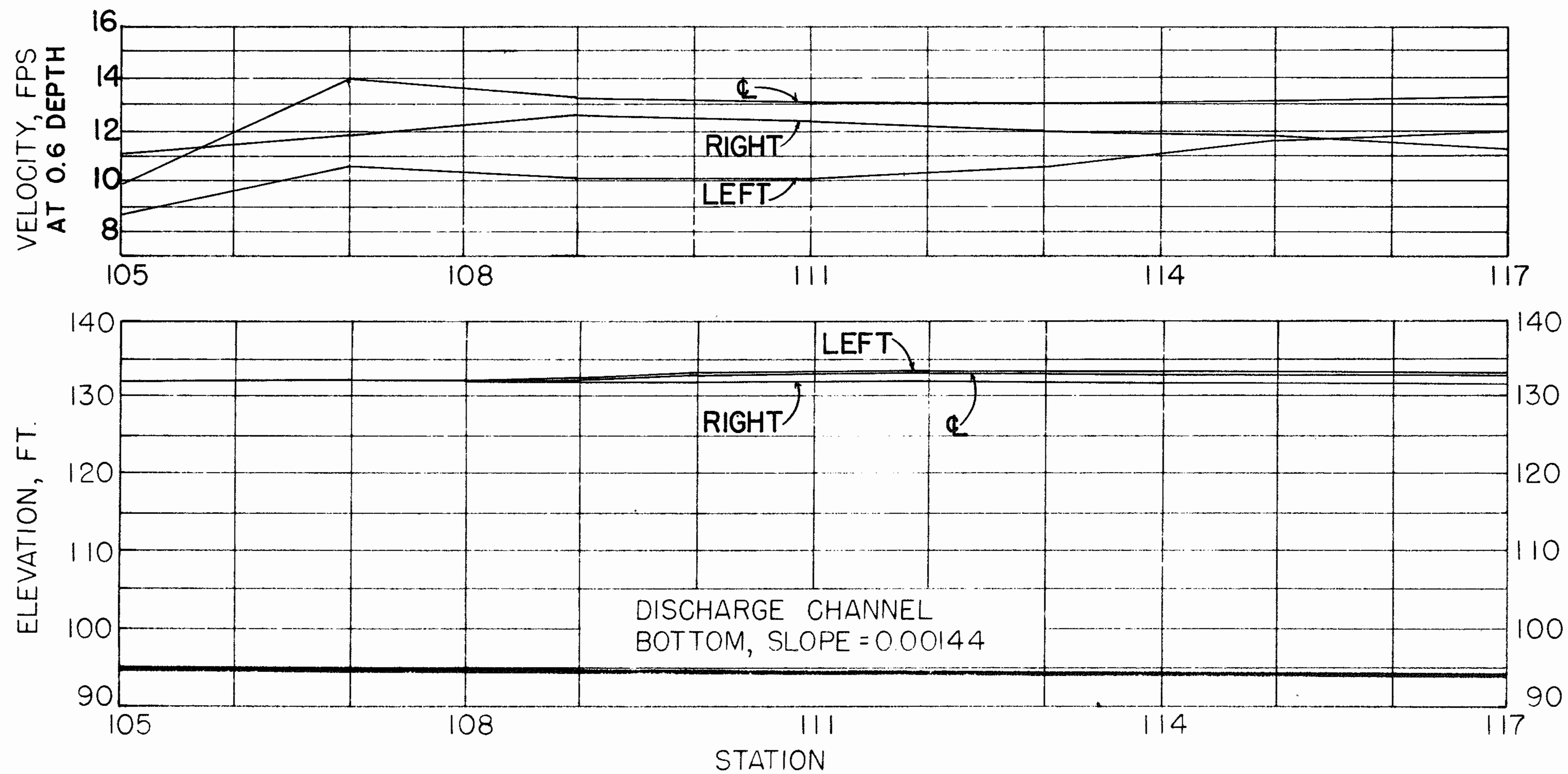
D. S. NO. 1367.

FIGURE 11. DOWNSTREAM VELOCITY AND ELEVATION PROFILES.



D.S. No. 1369.

FIGURE 12. DOWNSTREAM VELOCITY AND ELEVATION PROFILES.



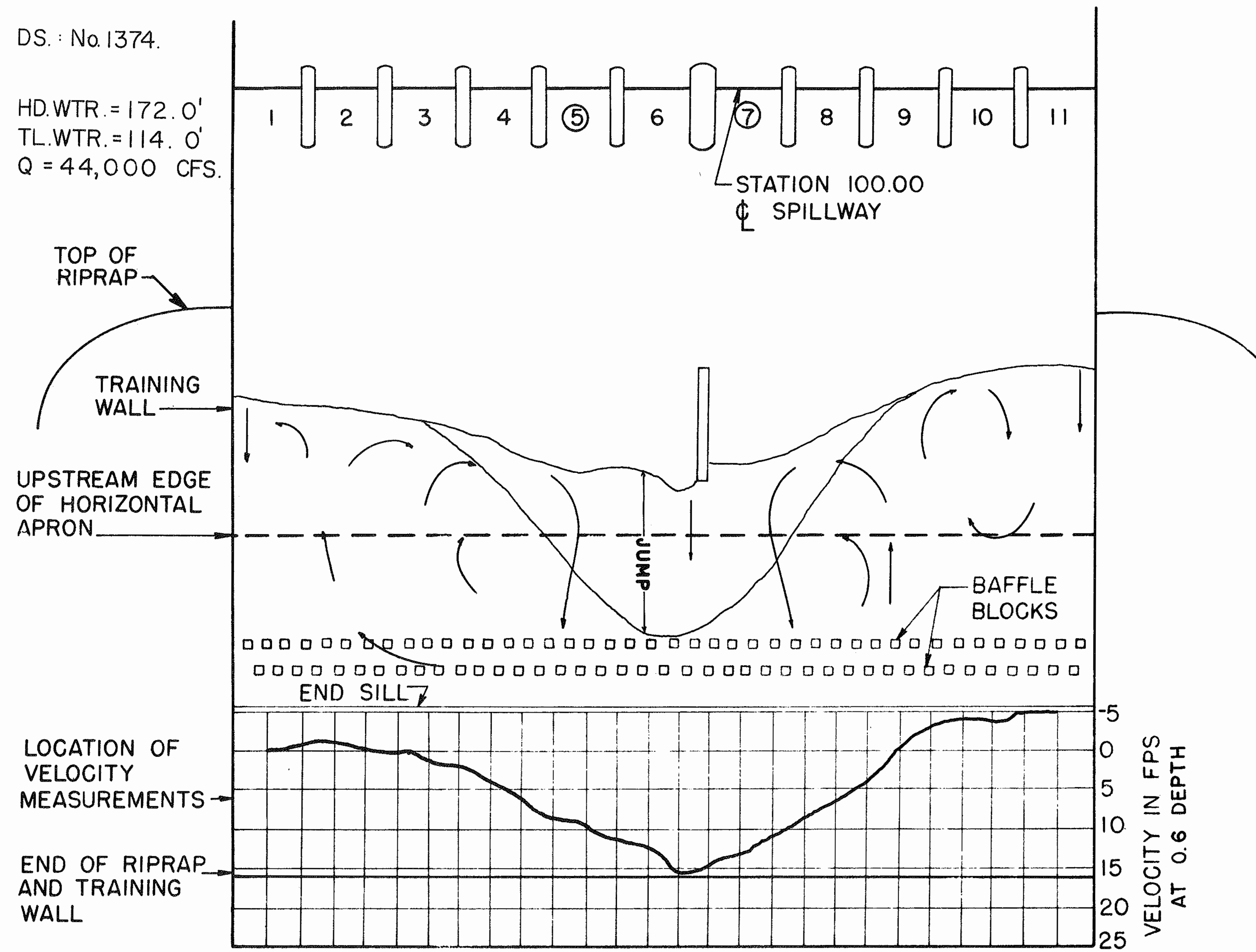
D.S. NO. 1371.

FIGURE 14. DOWNSTREAM VELOCITY AND ELEVATION PROFILES.

stilling basin. These tests were all made with the reservoir at elevation 172. As the gates were adjusted the rate of flow into the model was also adjusted to keep the reservoir at a constant elevation. The tailwater was adjusted for each setting to correspond with the tailwater rating curve at the particular discharge (Figure 3).

The first observations were made with a gate operating schedule which called for the gates to be opened in stages of several feet at a time and for the gates to be opened uniformly across the length of the spillway. Observation of the flow on the chute and in the stilling basin with this schedule suggested that it might be possible to utilize a gate sequence which would permit opening each gate fully before proceeding to the next one in sequence. In the first sequence to determine if the gates could be fully opened, the odd numbered gates were opened in pairs starting near the center with numbers 5 and 7 and progressed toward the ends of the spillway by opening pairs of odd gates. The even numbered gates were then opened starting with gate 6, at the center, and adding symmetrical pairs until all gates were open. The results from this gate operation are shown in Figures 15 through 20. Another sequence started with opening gate number 6 and then opening even numbered pairs of gates outward from the center. The odd numbered pairs of gates were then added outward from the center. The results of these observations are summarized in Figures 21 through 25.

When alternate gates were opened, as for example gates 1, 3, 5, 9, and 11, as shown in Figure 17, it was noted that the flow through the gates spread laterally meeting on a line downstream from the closed gates between adjacent open gates (for example gates 2, 4, 6, 8, and 10). This produced a concentration of flow and a rise in the surface elevation on the water



DS. : No. 1374.
HD. WTR. = 172.0'
TL. WTR. = 114.0'
Q = 44,000 CFS.
1
2
3
4
5
6
7
8
9
10
11
STATION 100.00
℄ SPILLWAY
TOP OF RIPRAP
TRAINING WALL
UPSTREAM EDGE OF HORIZONTAL APRON
JUMP
BAFFLE BLOCKS
END SILL
LOCATION OF VELOCITY MEASUREMENTS
END OF RIPRAP AND TRAINING WALL
-5
0
5
10
15
20
25
VELOCITY IN FPS AT 0.6 DEPTH

○ INDICATES OPEN GATES

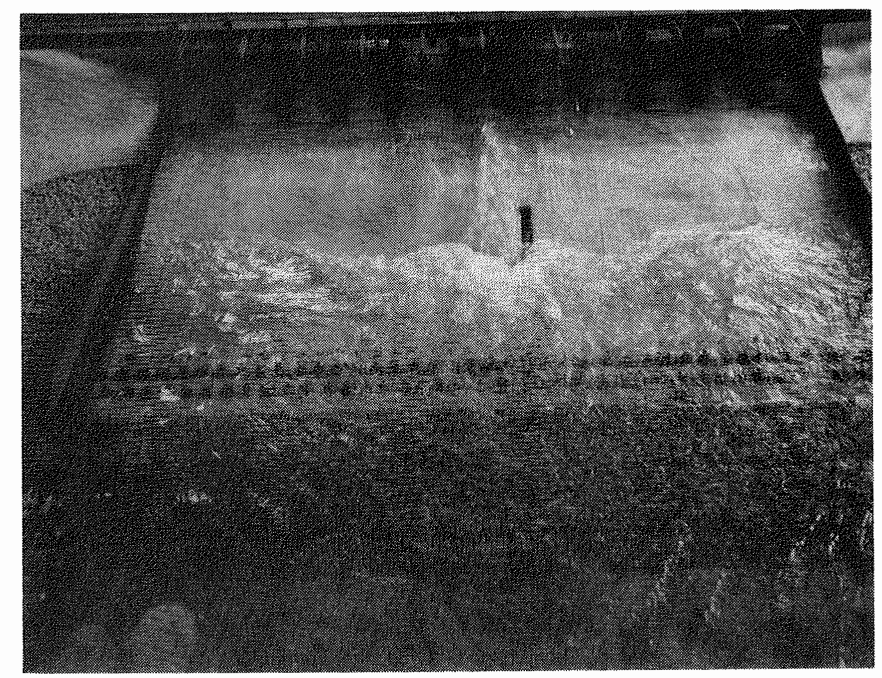
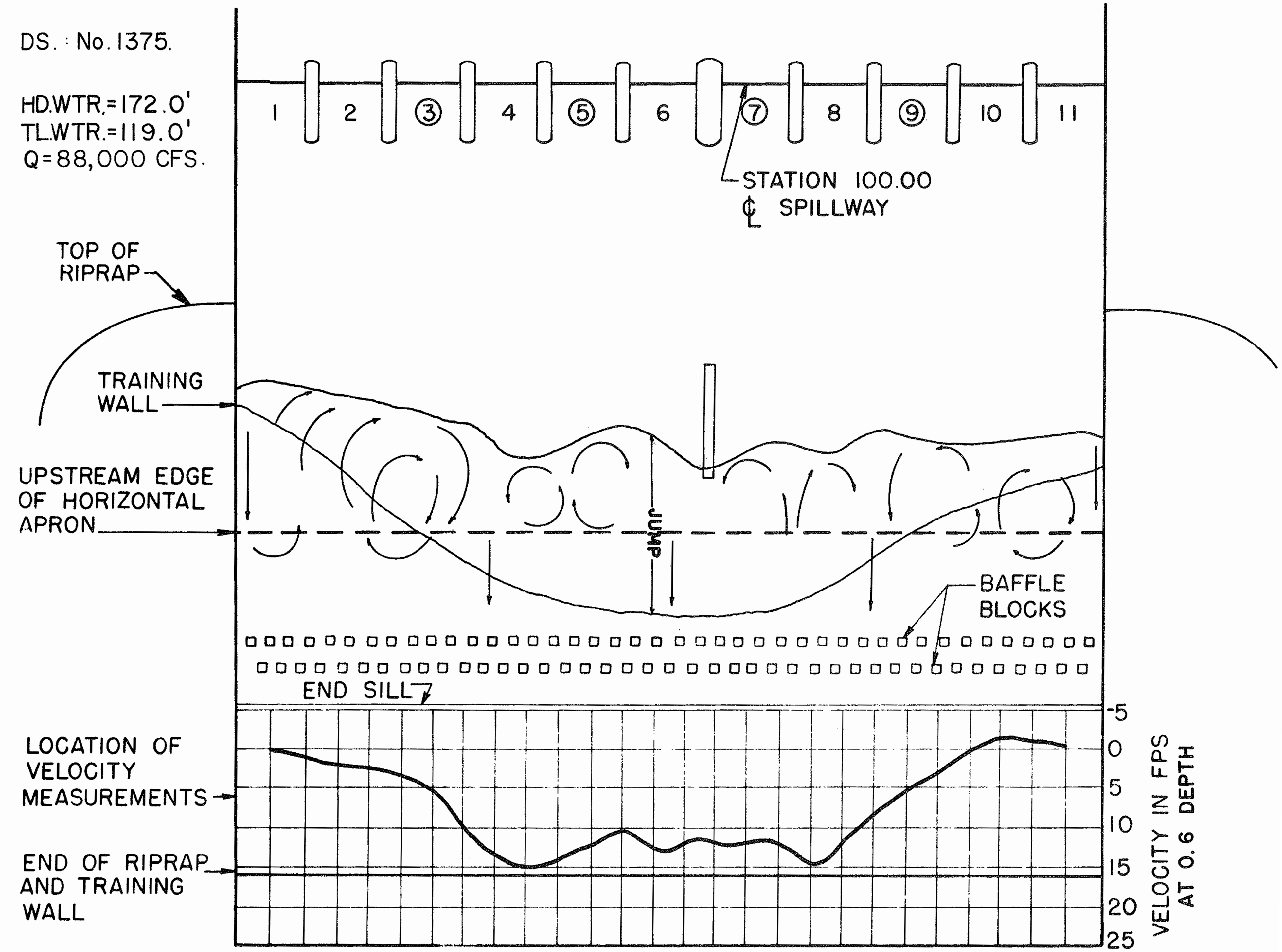


FIGURE 15. GATE OPERATION STUDY



DS. : No. 1375.
HD.WTR.=172.0'
TL.WTR.=119.0'
Q=88,000 CFS.
1
2
3
4
5
6
7
8
9
10
11
STATION 100.00
℄ SPILLWAY
TOP OF RIPRAP
TRAINING WALL
UPSTREAM EDGE OF HORIZONTAL APRON
JUMP
BAFFLE BLOCKS
END SILL
LOCATION OF VELOCITY MEASUREMENTS
END OF RIPRAP AND TRAINING WALL
-5
0
5
10
15
20
25
VELOCITY IN FPS AT 0.6 DEPTH

○ INDICATES OPEN GATES

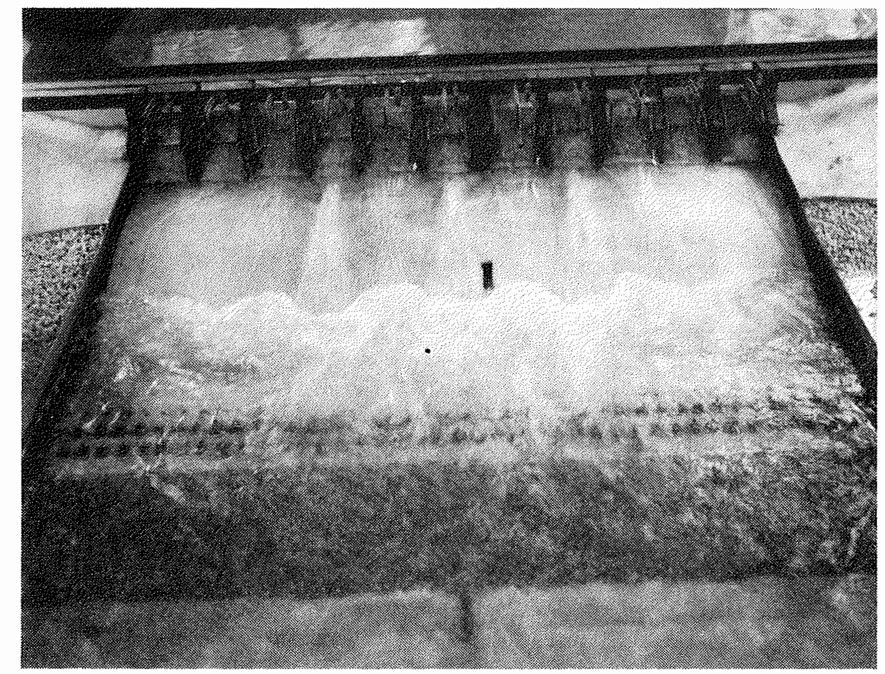
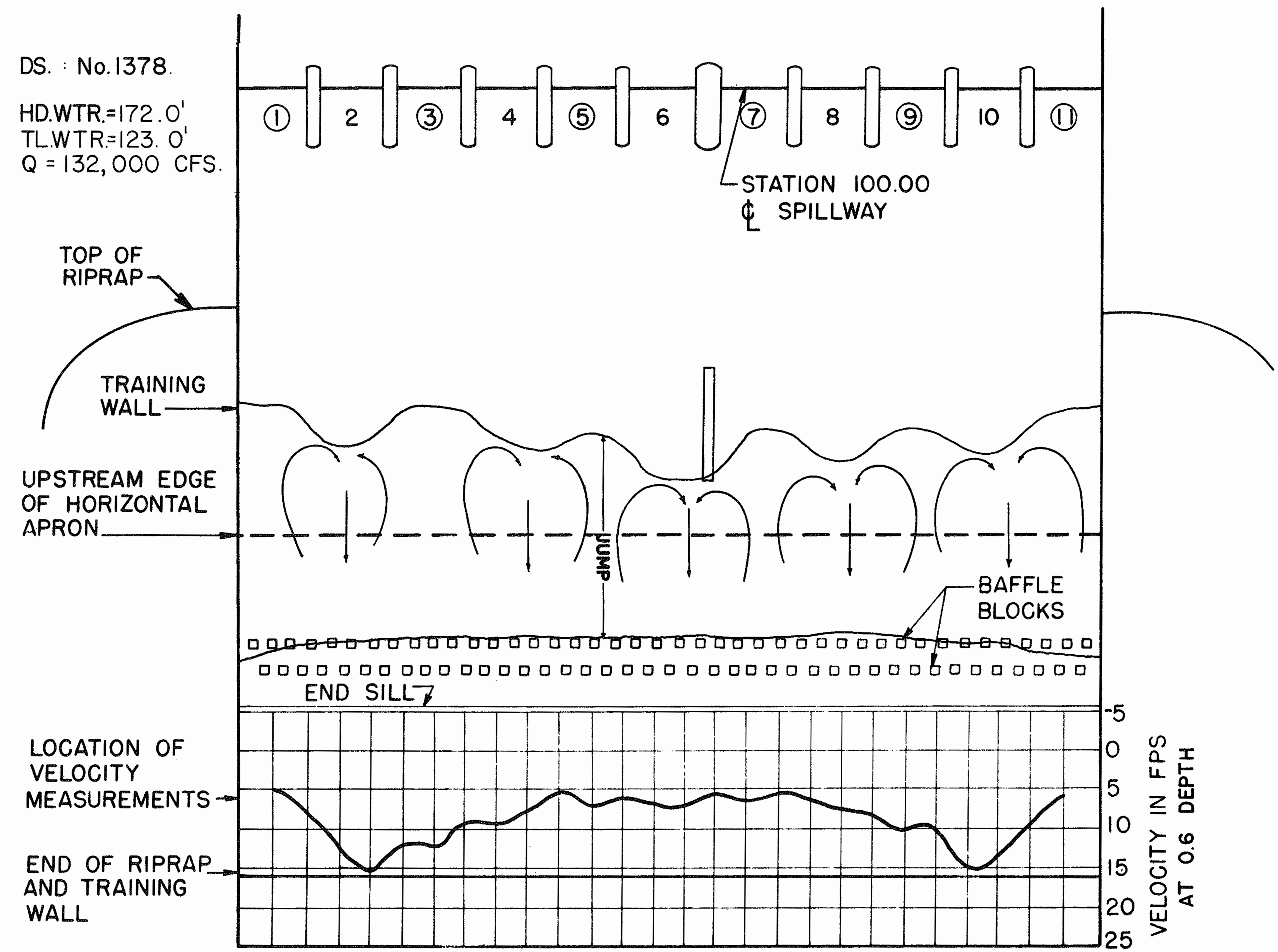


FIGURE 16. GATE OPERATION STUDY



DS. : No. 1378.
HD.WTR.=172.0'
TL.WTR.=123. 0'
Q = 132,000 CFS.
1
2
3
4
5
6
7
8
9
10
11
STATION 100.00
℄ SPILLWAY
TOP OF RIPRAP
TRAINING WALL
UPSTREAM EDGE OF HORIZONTAL APRON
JUMP
BAFFLE BLOCKS
END SILL
LOCATION OF VELOCITY MEASUREMENTS
END OF RIPRAP AND TRAINING WALL
-5
0
5
10
15
20
25
VELOCITY IN FPS AT 0.6 DEPTH

○ INDICATES OPEN GATES.

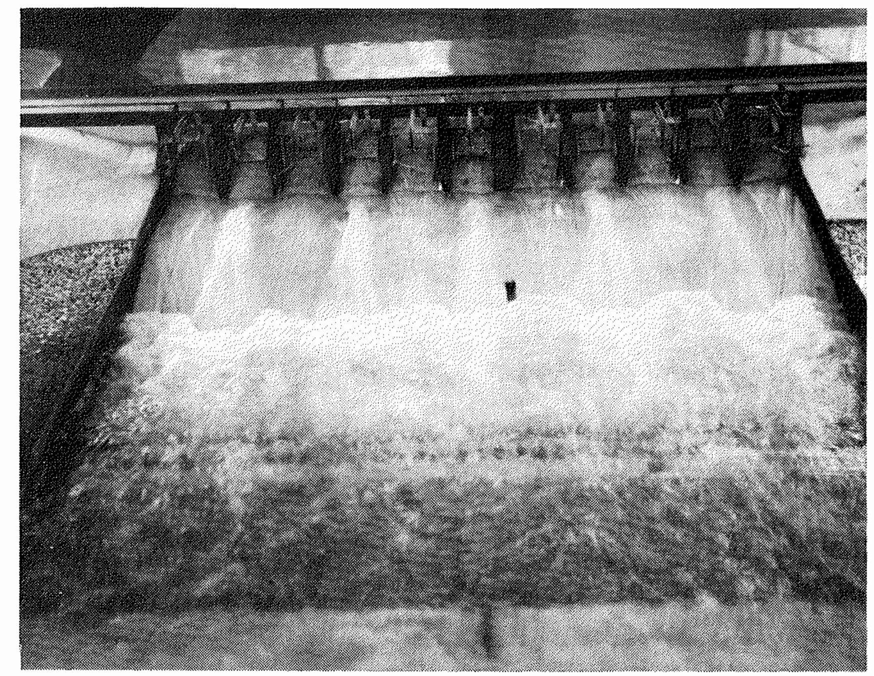
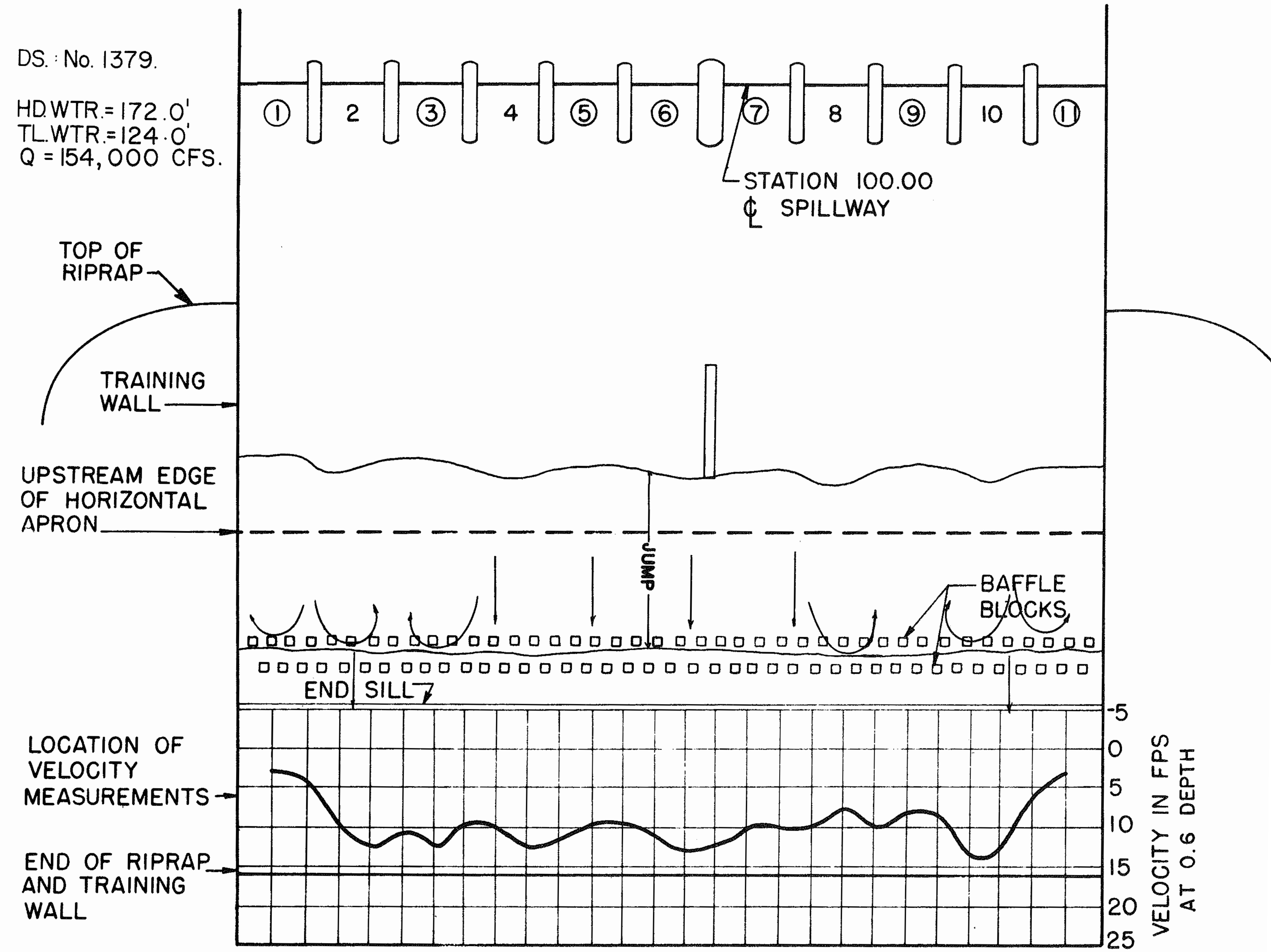


FIGURE 17. GATE OPERATION STUDY



DS. : No. 1379.
HD.WTR.= 172.0'
TL.WTR.= 124.0'
Q = 154,000 CFS.
1
2
3
4
5
6
7
8
9
10
11
STATION 100.00
℄ SPILLWAY
TOP OF RIPRAP
TRAINING WALL
UPSTREAM EDGE OF HORIZONTAL APRON
JUMP
BAFFLE BLOCKS
END SILL
LOCATION OF VELOCITY MEASUREMENTS
END OF RIPRAP AND TRAINING WALL
-5
0
5
10
15
20
25
VELOCITY IN FPS AT 0.6 DEPTH

○ INDICATES OPEN GATES.

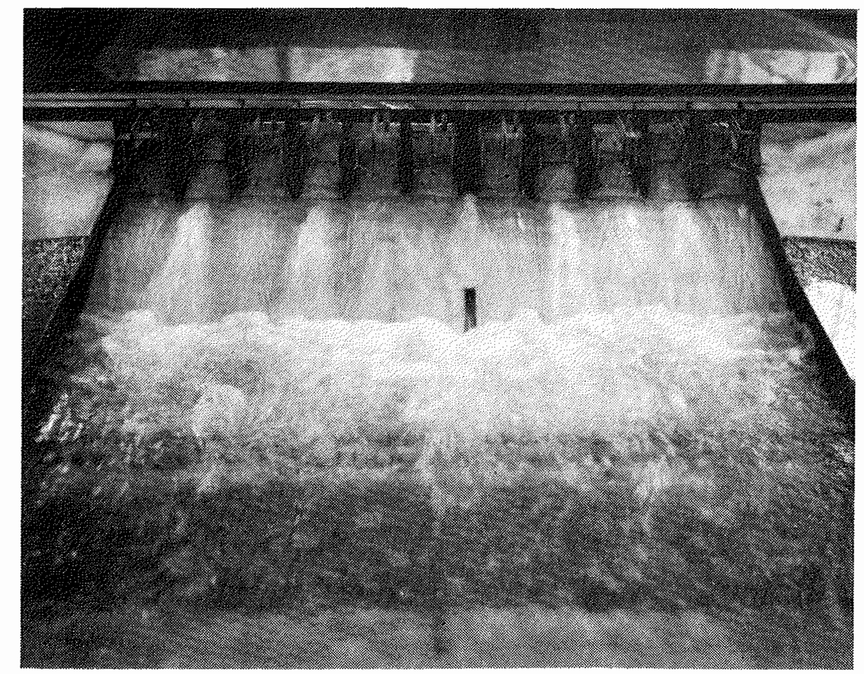
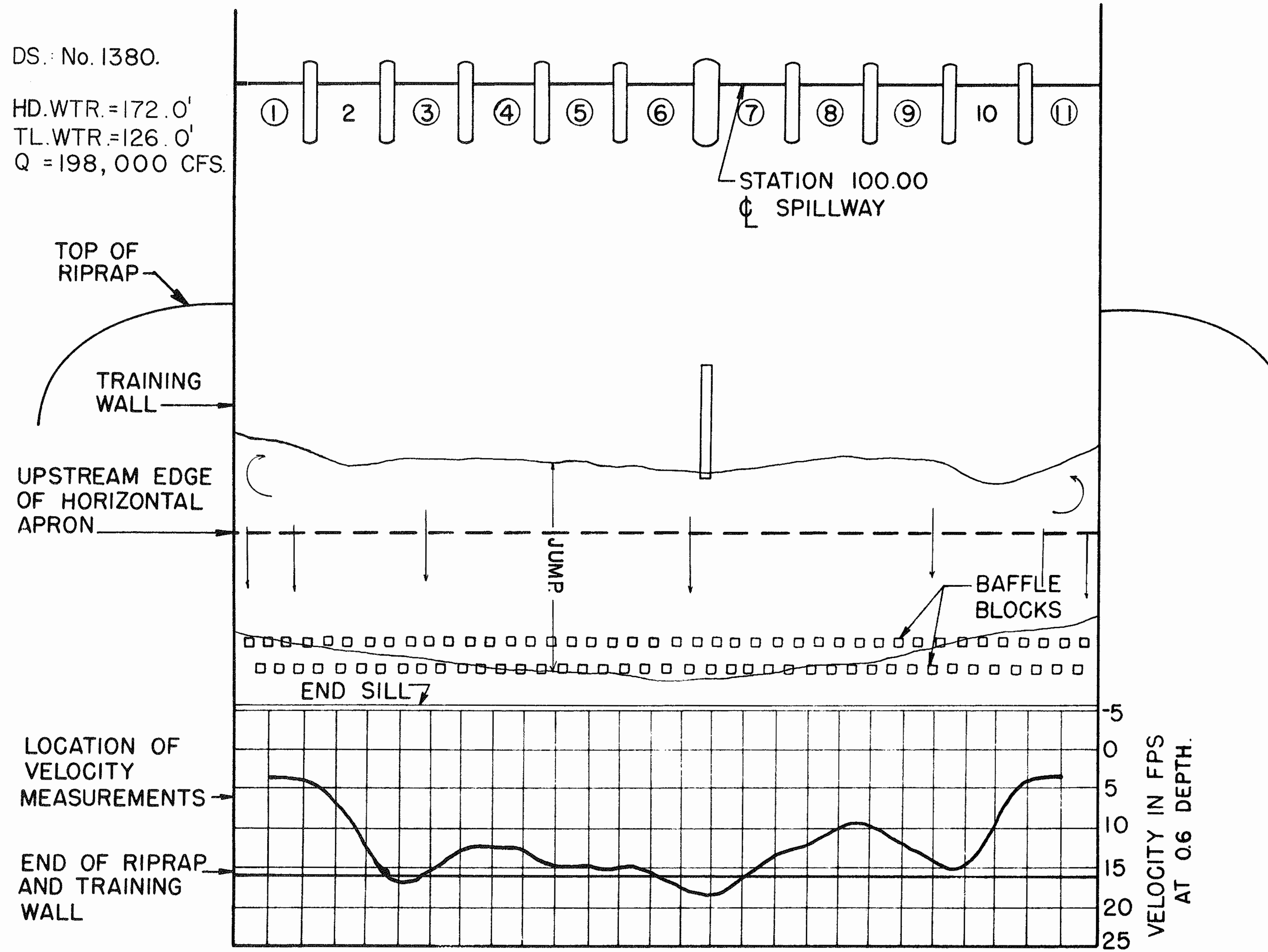


FIGURE 18. GATE OPERATION STUDY


DS.: No. 1380.
HD.WTR.=172.0'
TL.WTR.=126.0'
Q =198,000 CFS.
1 2 3 4 5 6 7 8 9 10 11
STATION 100.00
℄ SPILLWAY
TOP OF RIPRAP
TRAINING WALL
UPSTREAM EDGE OF HORIZONTAL APRON
JUMP
BAFFLE BLOCKS
END SILL
LOCATION OF VELOCITY MEASUREMENTS
END OF RIPRAP AND TRAINING WALL
-5
0
5
10
15
20
25
VELOCITY IN FPS AT 0.6 DEPTH.

○ INDICATES OPEN GATES

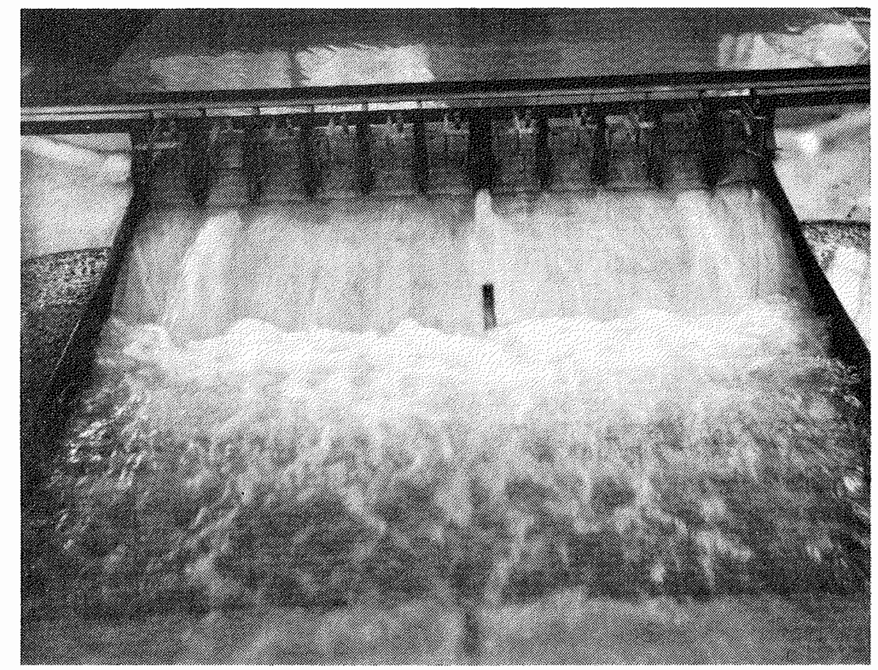
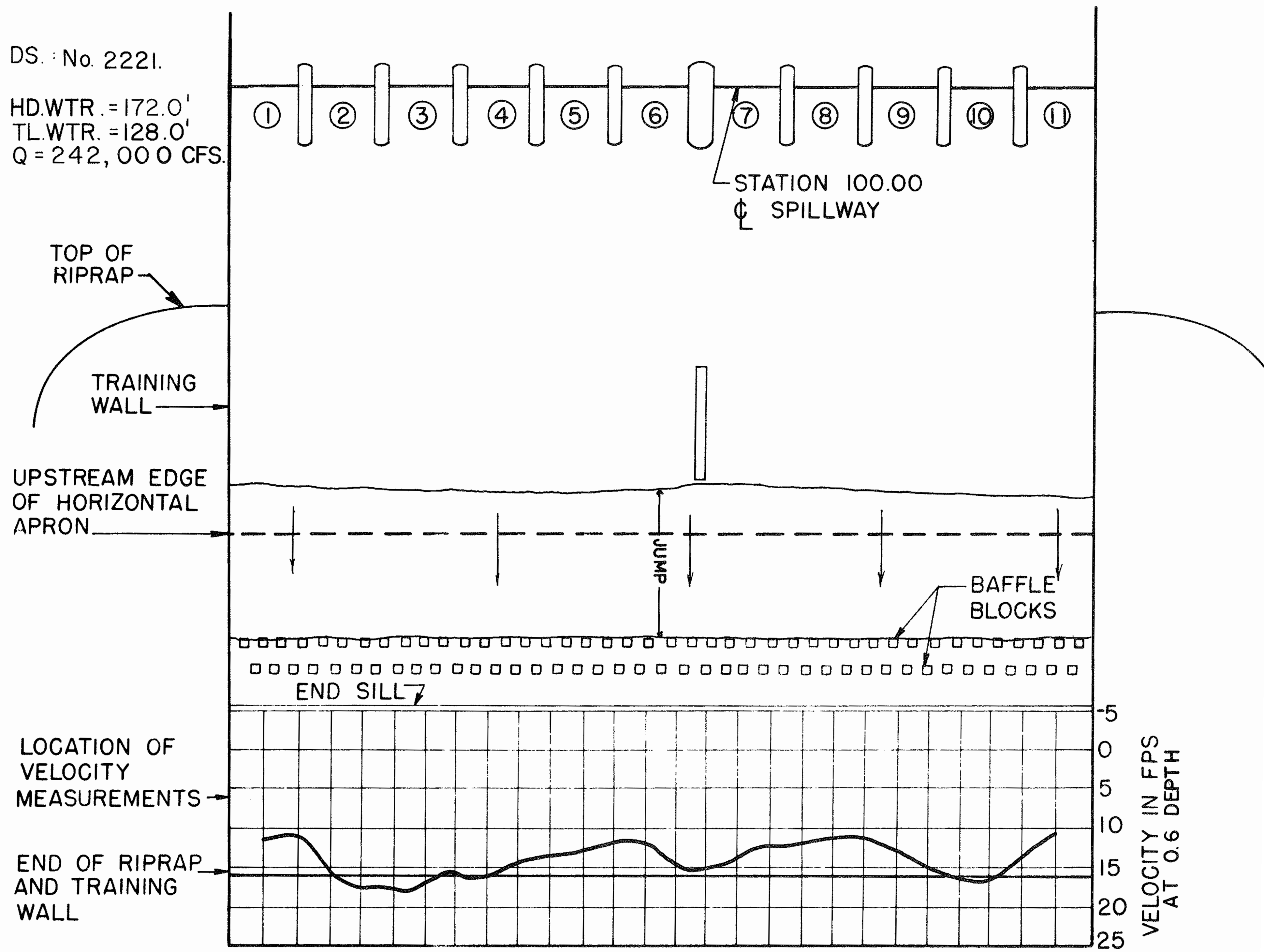


FIGURE 19. GATE OPERATION STUDY



DS. : No. 2221.
HD.WTR. = 172.0'
TL.WTR. = 128.0'
Q = 242,000 CFS.
1 2 3 4 5 6 7 8 9 10 11
STATION 100.00
℄ SPILLWAY
TOP OF RIPRAP
TRAINING WALL
UPSTREAM EDGE OF HORIZONTAL APRON
JUMP
BAFFLE BLOCKS
END SILL
LOCATION OF VELOCITY MEASUREMENTS
END OF RIPRAP AND TRAINING WALL
-5
0
5
10
15
20
25
VELOCITY IN FPS AT 0.6 DEPTH

INDICATES OPEN GATES

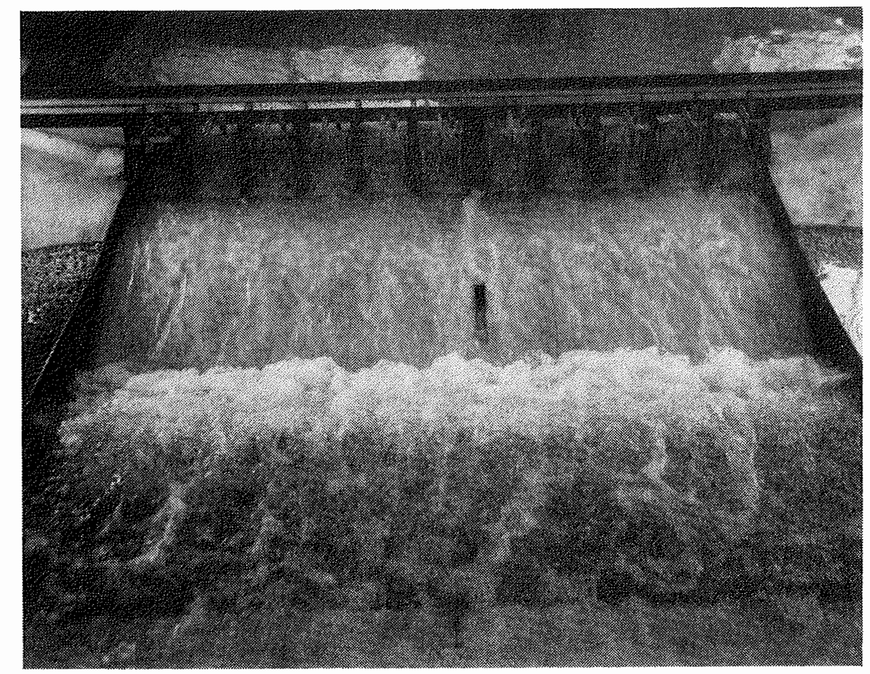
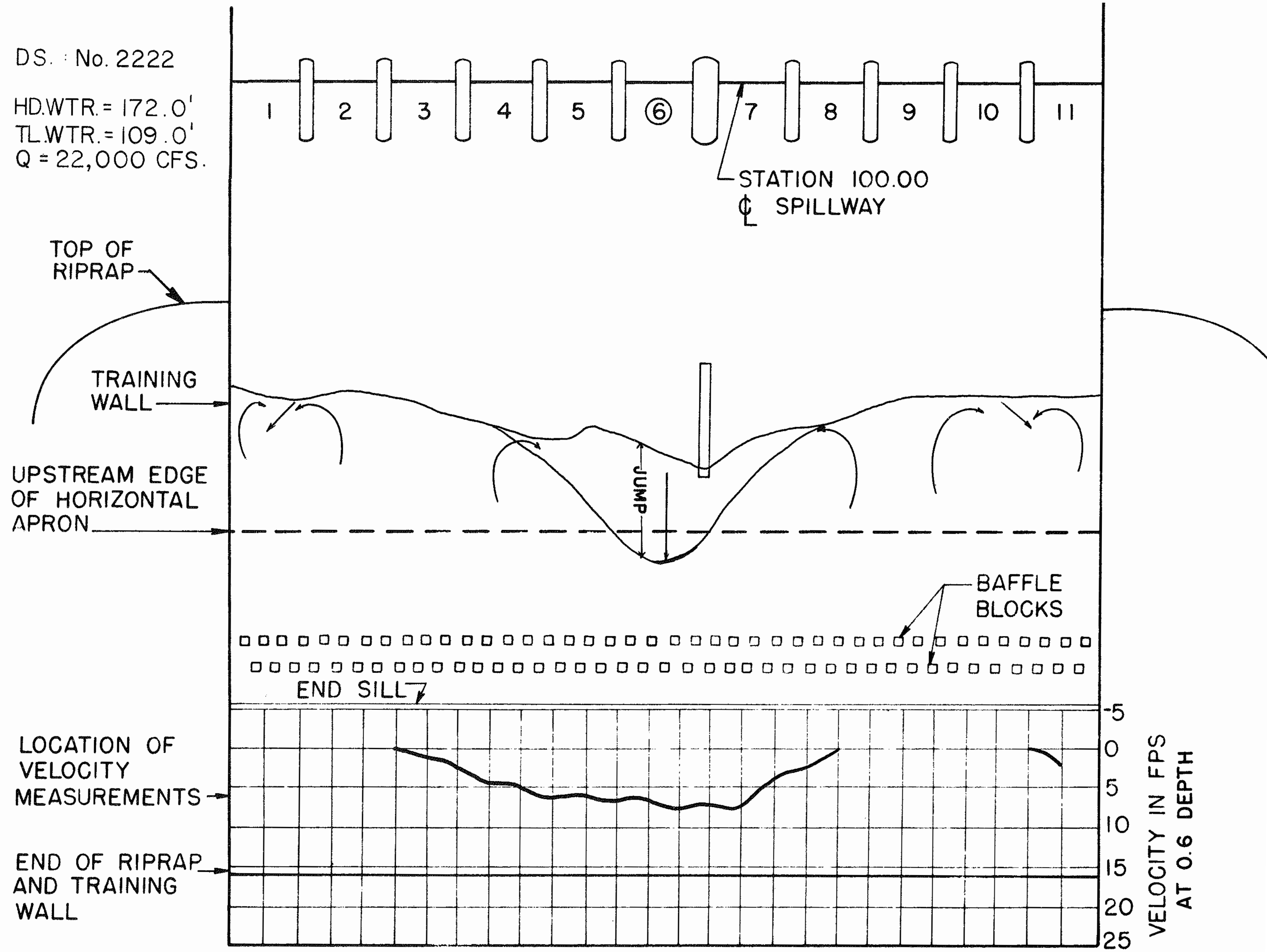


FIGURE 20. GATE OPERATION STUDY



D.S. : No. 2222
HD.WTR. = 172.0'
TL.WTR. = 109.0'
Q = 22,000 CFS.
1
2
3
4
5
6
7
8
9
10
11
STATION 100.00
℄ SPILLWAY
TOP OF RIPRAP
TRAINING WALL
UPSTREAM EDGE OF HORIZONTAL APRON
JUMP
BAFFLE BLOCKS
END SILL
LOCATION OF VELOCITY MEASUREMENTS
END OF RIPRAP AND TRAINING WALL
-5
0
5
10
15
20
25
VELOCITY IN FPS AT 0.6 DEPTH

○ INDICATES OPEN GATES

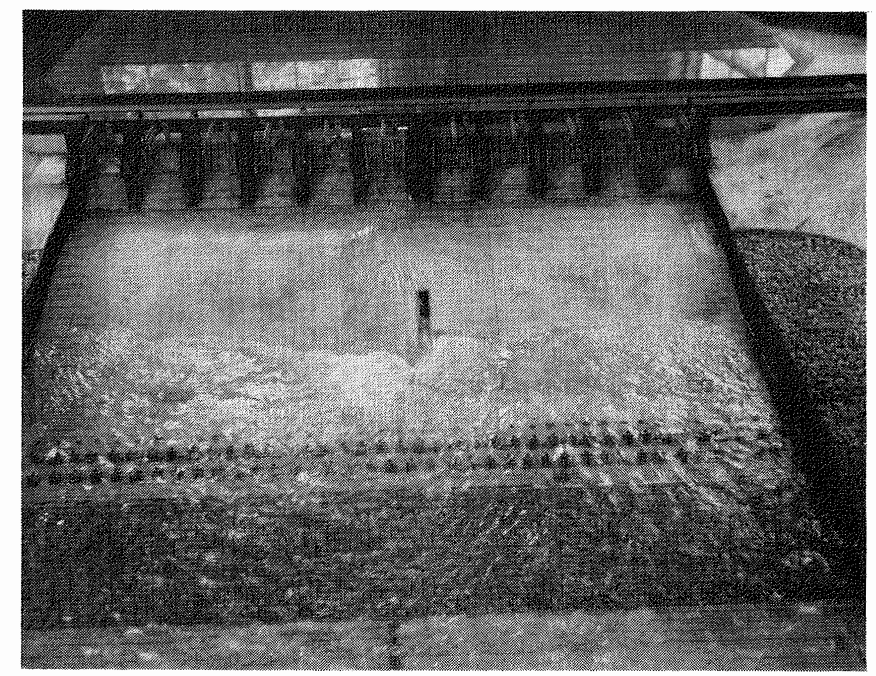
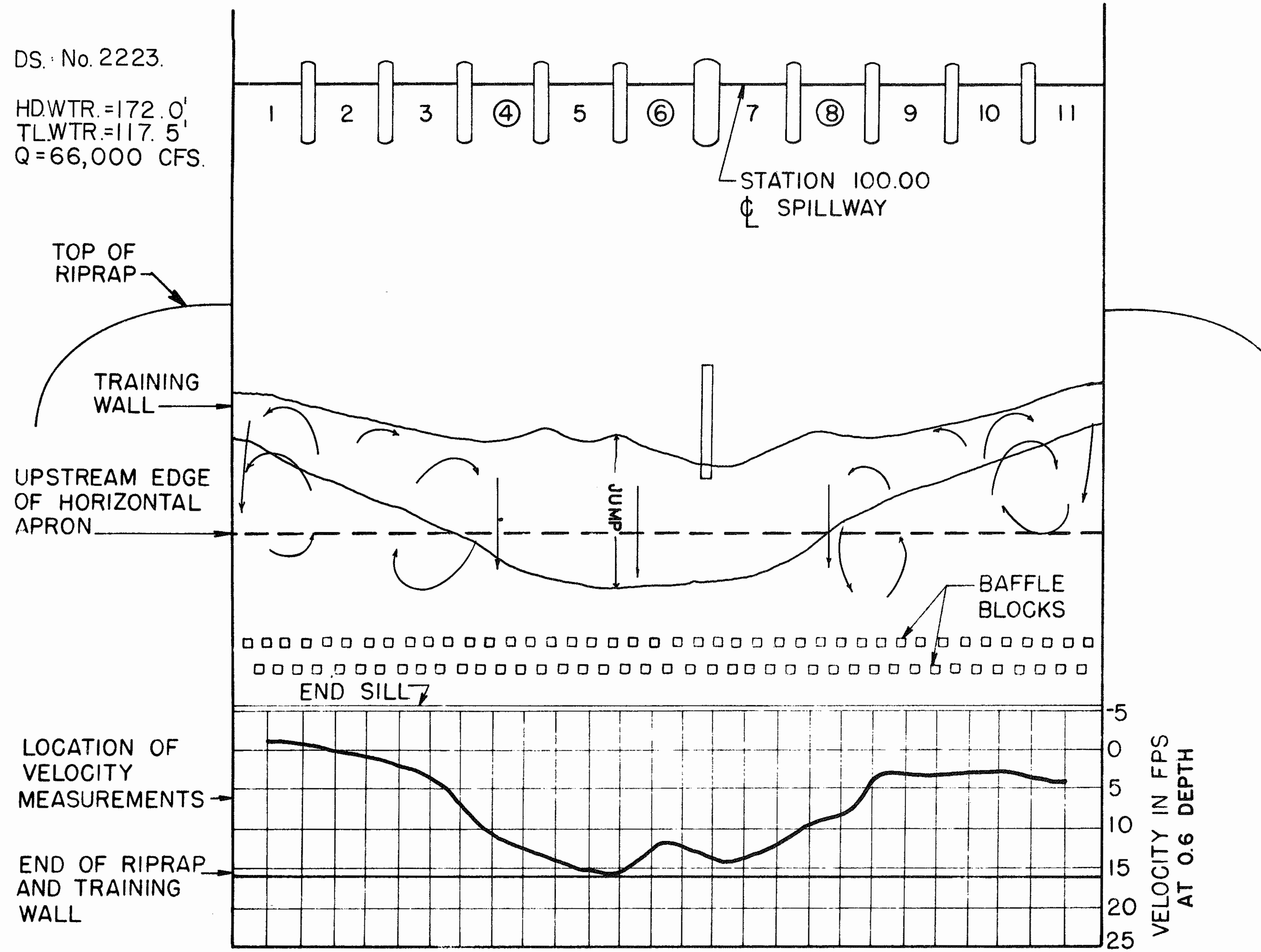


FIGURE 21. GATE OPERATION STUDY



DS. No. 2223.
HD.WTR.=172.0'
TL.WTR.=117.5'
Q=66,000 CFS.
1
2
3
4
5
6
7
8
9
10
11
STATION 100.00
℄ SPILLWAY
TOP OF RIPRAP
TRAINING WALL
UPSTREAM EDGE OF HORIZONTAL APRON
JUMP
BAFFLE BLOCKS
END SILL
LOCATION OF VELOCITY MEASUREMENTS
END OF RIPRAP AND TRAINING WALL
-5
0
5
10
15
20
25
VELOCITY IN FPS AT 0.6 DEPTH

○ INDICATES OPEN GATES.

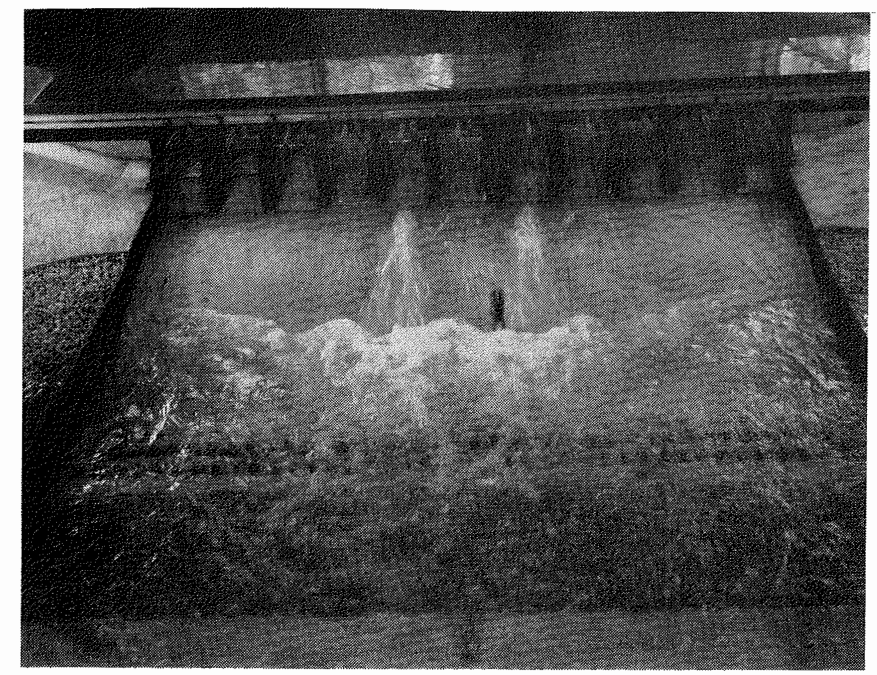
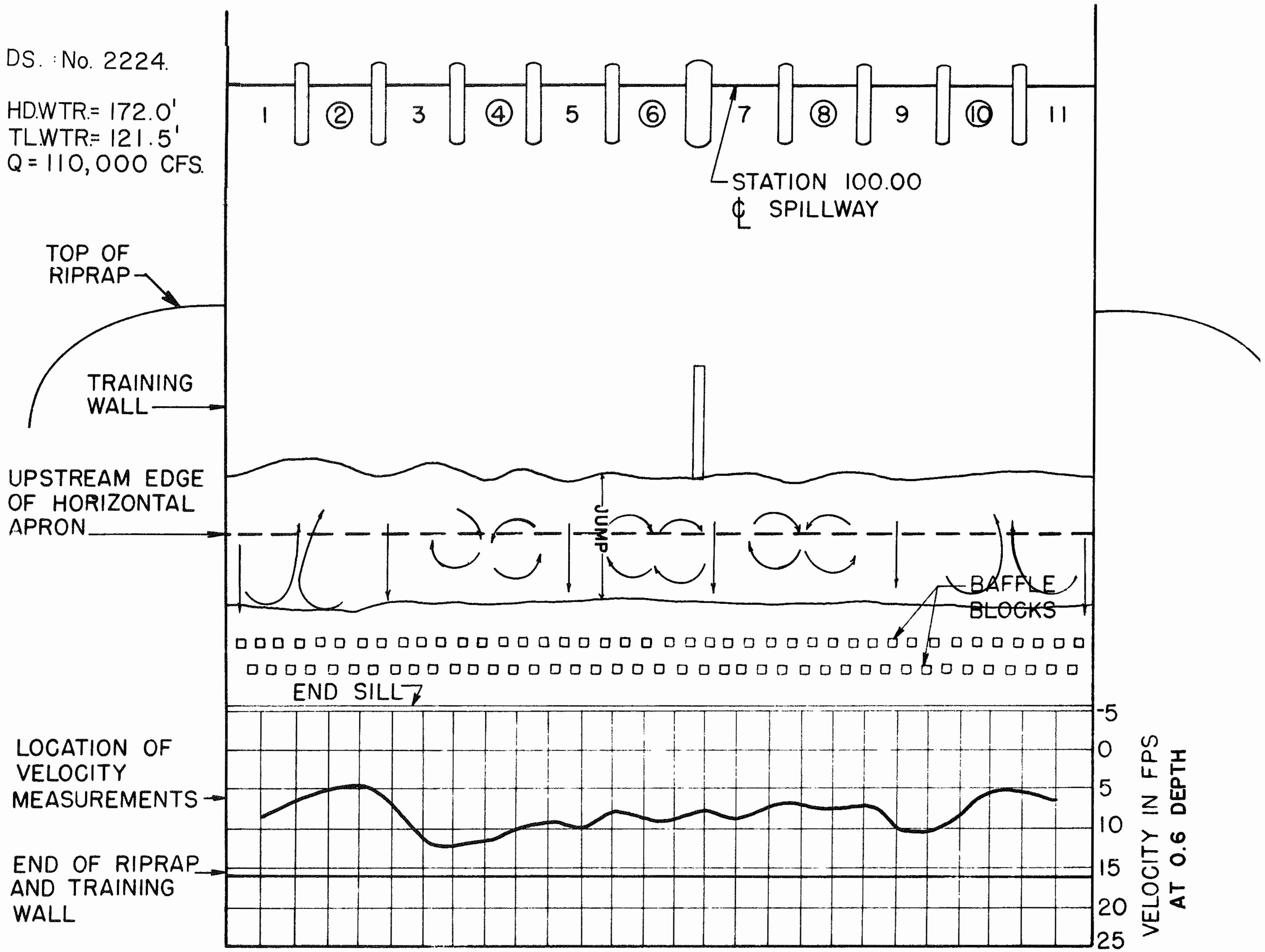


FIGURE 22. GATE OPERATION STUDY


DS. No. 2224.
HDWTR.= 172.0'
TLWTR.= 121.5'
Q = 110,000 CFS.
1
2
3
4
5
6
7
8
9
10
11
STATION 100.00
℄ SPILLWAY
TOP OF RIPRAP
TRAINING WALL
UPSTREAM EDGE OF HORIZONTAL APRON
JUMP
BAFFLE BLOCKS
END SILL
LOCATION OF VELOCITY MEASUREMENTS
END OF RIPRAP AND TRAINING WALL
-5
0
5
10
15
20
25
VELOCITY IN FPS
AT 0.6 DEPTH

○ INDICATES OPEN GATES

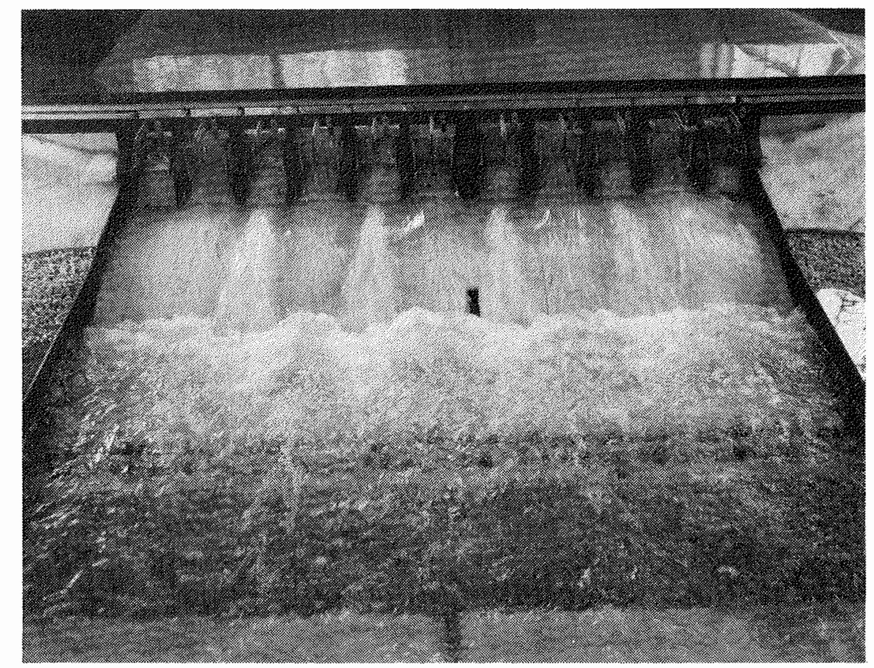
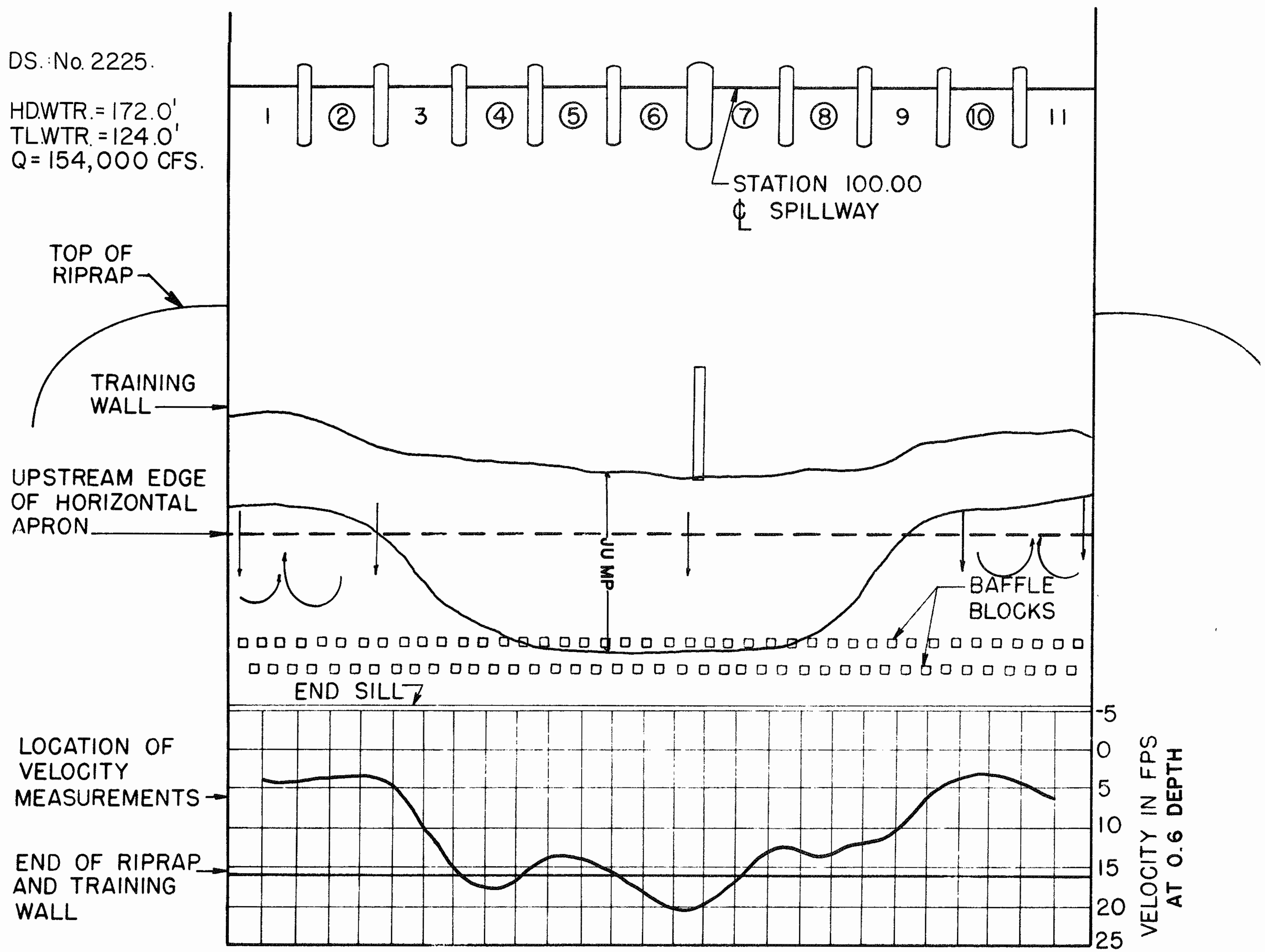


FIGURE 23. GATE OPERATION STUDY


DS. No. 2225.
HD.WTR. = 172.0'
TL.WTR. = 124.0'
Q = 154,000 CFS.
1
2
3
4
5
6
7
8
9
10
11
STATION 100.00
℄ SPILLWAY
TOP OF RIPRAP
TRAINING WALL
UPSTREAM EDGE OF HORIZONTAL APRON
JUMP
BAFFLE BLOCKS
END SILL
LOCATION OF VELOCITY MEASUREMENTS
END OF RIPRAP AND TRAINING WALL
-5
0
5
10
15
20
25
VELOCITY IN FPS AT 0.6 DEPTH

◯ INDICATES OPEN GATES

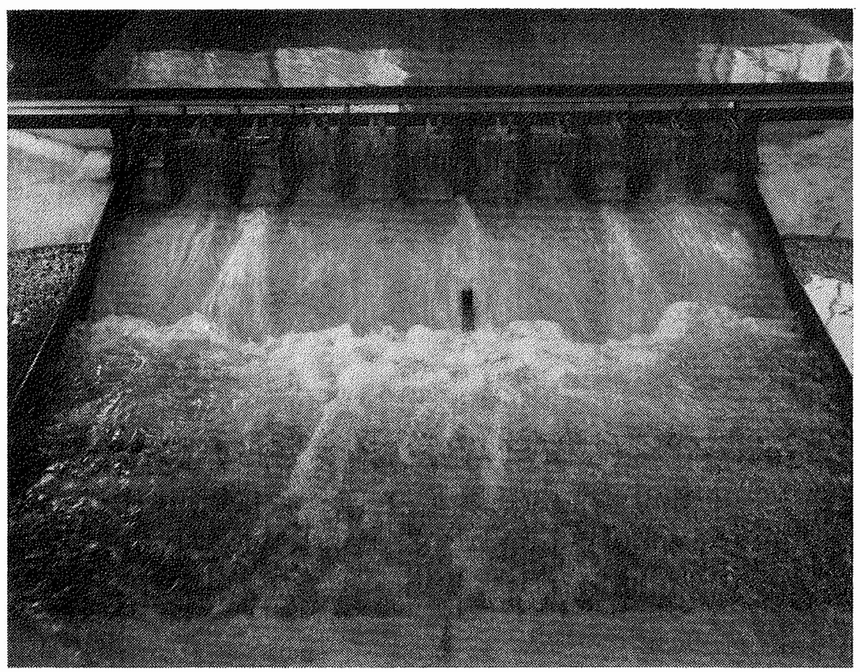
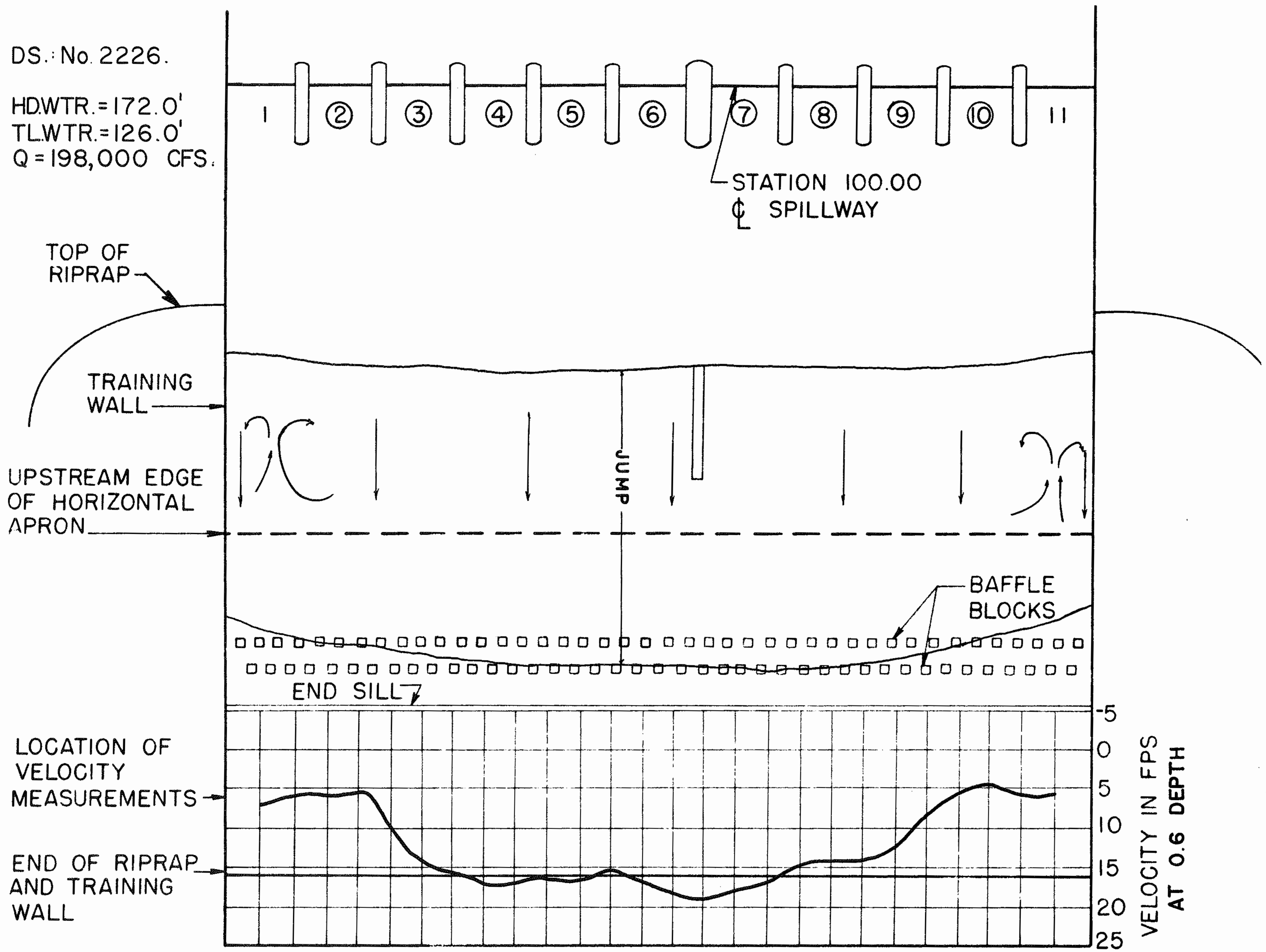


FIGURE 24. GATE OPERATION STUDY


DS.: No. 2226.
HDWTR.=172.0'
TLWTR.=126.0'
Q=198,000 CFS.
1
2
3
4
5
6
7
8
9
10
11
STATION 100.00
℄ SPILLWAY
TOP OF RIPRAP
TRAINING WALL
JUMP
UPSTREAM EDGE OF HORIZONTAL APRON
BAFFLE BLOCKS
END SILL
LOCATION OF VELOCITY MEASUREMENTS
END OF RIPRAP AND TRAINING WALL
-5
0
5
10
15
20
25
VELOCITY IN FPS AT 0.6 DEPTH

○ INDICATES OPEN GATES

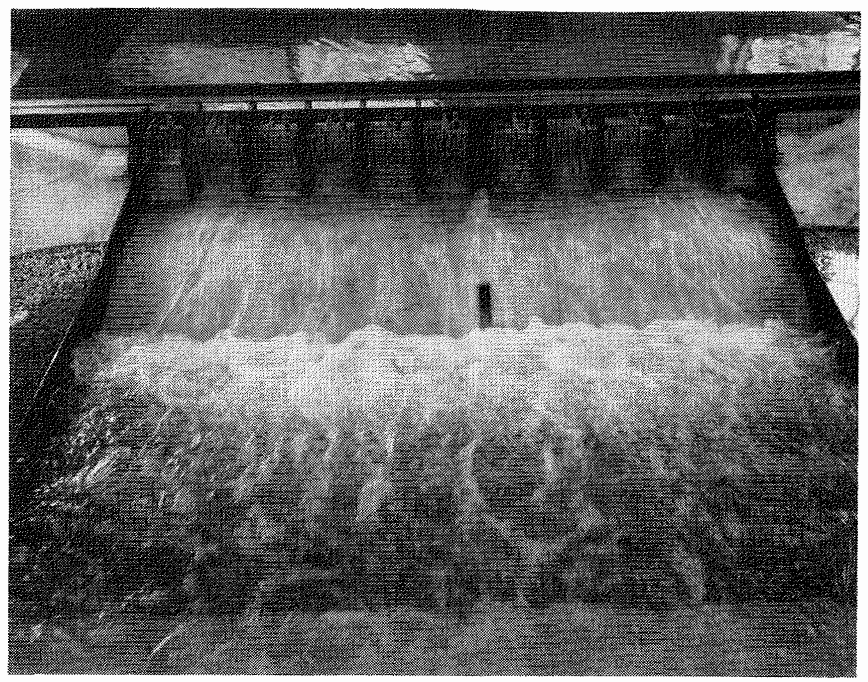


FIGURE 25. GATE OPERATION STUDY

downstream from the intermediate closed gates. This is clearly shown in the photographs, Figures 15-25, except for those cases where all gates were open, Figures 20 and 25. In all cases, the rise in the water surface was contained within the concrete stilling basin and was therefore considered of no significance to the gate operating schedule.

Observations of the performance of the chute and stilling basin led to the sequence of Table 5 as a suggested operating procedure which would produce satisfactory flow conditions and also be convenient for operation.

Table 5. Gate Opening Sequence.

Number of Open Gates Needed to Pass the Flow	Gate Opening Sequence
1	Open gate 6 as needed up to the fully opened position.
2	Open gates 5 and 7 maintaining approximately equal gate openings until both gates are fully opened.
3	Follow sequence for one gate opened and then that for two gates opened.
4	Follow the sequence for two gates opened and then open gates 3 and 9 maintaining approximately equal openings until they are fully opened.
5	Follow the sequence for one gate open and then the sequence for four gates open.
6	Follow the sequence for four gates open and then open gates 1 and 11 maintaining approximately equal openings until they are fully opened.
7	Follow sequence for one gate opened and then the sequence for six gates opened.
9	Follow the sequence for seven gates opened and then open gates 4 and 8 maintaining approximately equal openings until they are fully opened.
11	Follow the sequence for nine gates open and then open gates 2 and 10 maintaining approximately equal openings until the gates are fully opened.

This gate operating sequence was designed to maintain the concentration of flow near the center of the stilling basin and eliminate the tendency of the flow to pile up on the training walls when gates 2 and 10 are open while 1 and 11 are closed. When the prototype structure is in operation it would be well to follow the proposed sequence, at least initially. If there should be a tendency for the downstream channel to scour near the center of the channel below the spillway as a result of repeated passing of moderate flood discharges through the central spillway gates, it would be well to alter the sequence of gate operation and distribute the flow more completely across the channel by opening more gates to a partially opened position.

When the spillway was operated with all gates open (see Figure 20) there was a disturbance behind the pier between gate 6 and 7, the pier containing the control works for the low flow tunnel. The disturbance resulted from the flow closing in behind the wide pier and caused the flow in the chute to pile up considerably higher than the normal flow depth in the chute. This disturbance however was completely eliminated as the flow passed through the hydraulic jump and there is no reason to expect any harmful effects from the disturbance.

Operation of the Low Flow Sluiceway

During the development of the gate operating sequence, the low flow sluiceway was operated both with and without flow over the spillway. There was no tendency for this flow to separate from the face of the chute or to cause any noticeable disturbance. With flow over the spillway the sluiceway pier produced the "rooster tail" effect, seen in Figure 25. However all the existing disturbances were within the concrete stilling basin and should produce no adverse effects.

Considerations of Downstream Rip-Rap

With respect to the downstream rip-rap, it is undesirable to have the maximum size rip-rap the same as the thickness of the rip-rap blanket. Estimates based on approximate calculations indicate that rip-rap down to about one foot in size should be stable below the spillway. In the event there is movement of some of the smaller rip-rap so as to expose the large 48 inch stones, these large stones will then project up into the flow and will act as obstructions causing locally high velocities around them and intensifying the attack on the nearby rip-rap. It is recommended that the maximum size stone therefore be reduced from 48 inches to 36 inches. The thickness of the rip-rap blanket should be gradually increased from 4 feet to 8 feet over a 10 foot length at the downstream end of the rip-rap. In this way, if the stream bed in the downstream channel degrades leaving some local areas of deep scour, the downstream part of the rip-rap would tend to be washed into the scour holes. This extra depth of rip-rap at the downstream edge would help to prevent erosion from penetrating to the bed below the rip-rap.

RECOMMENDATIONS

As a result of the model tests and from a consideration of practical construction methods, the following recommendations are made for the construction and operation of the prototype structure.

1. A dike be constructed upstream from the embankment near the right abutment of the spillway as shown in Figure 10.

2. The stilling basin be modified by moving the two rows of blocks so that they will be 31 feet and 49 feet respectively from the downstream face of the end sill.

3. That the suggested gate operating sequence described in the previous section be used when passing floods through the spillway and that observations be made of the downstream channel after the passage of floods to see if any alteration of the gate operating sequence would provide better control over scour in the channel downstream from the stilling basin.

4. To obtain better scour protection from the rip-rap downstream from the stilling basin, it is recommended that the maximum size stone be reduced from 48 inches to 36 inches and that the thickness of the rip-rap be gradually increased from 4 feet to 8 feet over the last 10 foot length at the downstream end of the rip-rap. The stones should be approximately cubical in shape.

REFERENCES

1. Design Memorandum No. 1, "Hydrology," Toledo Bend Dam and Reservoir, Sabine River of Texas and Louisiana, Forrest and Cotton Inc., Consulting Engineers, Dallas, Texas, November 1960.

2. Design Memorandum No. 2, "Spillway," Toledo Bend Dam and Reservoir, Sabine River of Texas and Louisiana, Forrest and Cotton Inc., Consulting Engineers, Dallas, Texas, August 1962.

3. Design Memorandum No. 3, "Earthen Dams," Toledo Bend Dam and Reservoir, Sabine River of Texas and Louisiana, Forrest and Cotton Inc., Consulting Engineers, Dallas, Texas, October 1962.

4. Plans for Construction of Spillway, Embankment, Power Plant; Volume 1, Spillway, Forrest and Cotton Inc., Consulting Engineers, Dallas, Texas, November 1963.

APPENDIX

Summary of Tests

Table I. Summary of Tests - Original Design.

Approximate Date	Description of Model or of Test	Prototype Q (cfs)	Prototype Tailwater Elev. (ft.)	Original Data Page	Remarks
4-29-64	Depth vs. Q at Sta. 97 + 00.0	21,600~ 184,000		1294	Data plotted
5- 4-64	Surface Elev. at Sta. 96 + 00.0 and 105 + 04.2; velocity above spillway and 105 + 04.2	60,000		1295	
5- 5-64	Velocity at Sta. 105 + 04.2	60,000	117	1296	
5- 7-64	Velocity above spillway	211,000		1297	
5- 7-64	"	98,000		1298	
5- 7-64	W.S. Elev. and velocity at Sta. 97 + 00 and 105 + 00, tailwater varied	260,000	140 and 146	1299	W.S. Elev. = water surface elevation
5- 7-64	"	260,000- 270,000	119~136	1300	
5- 8-64	"	314,000- 317,000	120~136	1301	
5-11-64	"	268,000	117.5~120	1302	
5-13-64	W.S. Elev. from Sta. 97 + 00 to wier crest	284,000	130.5	1304	Data plotted
5-14-64	"	288,000	132	1305	"
5-16-64	W.S. Elev. and velocity downstream from Sta. 105 + 00	279,000	131	1306	"
5-16-64	"	280,000	136	1307	"
5-16-64	"	279,000	120	1308	"
5-23-64	Velocity at X-section over adverse slope	297,000	131	1309	"
5-26-64	W.S. Elev. and velocity profile	60,000	125	1310	"
5-26-64	"	60,000	117	1312	"
6- 5-64	"	132,000	123	1313	"
6-10-64	"	68,000	117	1314	"
6-13-64	"	278,000	131	1315	"
6-13-64	Velocity at section at end of rip-rap, 0.2 from bottom	278,000	131	1316	"
6-17-64	W.S. Elev. and velocity profile	330,000	133	1317	"

Table I. (continued).

Approximate Date	Description of Model or of Test	Prototype Q (cfs)	Prototype Tailwater Elev. (ft.)	Original Data Page	Remarks
6-17-64	Velocity at section at end of rip-rap	330,000	133	1318	Data plotted
6-18-64	"	60,000	117	1319	"
6-22-64	"	132,000	123	1320	"
6-24-64	"	132,000	123	1231	"
6-24-64	Velocity at section, middle of rip-rap	29,400	111	1232	
7- 6-64	"	94,000	120	1233	
7- 6-64	"	83,000	120	1234	
7- 6-64	"	25,000	112	1235	
7- 6-64	"	136,000	124	1236	
7- 6-64	W. S. profile from Sta. 99 + 50 to sill	60,000	117	1238	
7- 7-64	"	132,000	123	1239	
7- 9-64	"	279,000	131	1240	
7- 9-64	"	330,000	133	1241	
7-10-64	Velocity at section, middle of rip-rap, 0.6 depth	132,000	123	1242	
7-10-64	"	55,000	117	1243	
7-11-64	"	110,000	121	1245	
7-13-64	"	160,000	125	1246	
7-13-64	Flow patterns in stilling basin	103,000	121	1247	
7-13-64	"	113,050	121.5	1248	
7-13-64	Velocity at section, middle of rip-rap, 0.6 depth	170,000	126	1249	
7-14-64	W. S. Elev. and velocity profile from Sta. 105 + 00 to 117 + 00	279,000	131	1250	
7-15-64	Velocity at section, middle of rip-rap	180,000	126	1251	
7-16-64	"	155,000	125	1252	
7-17-64	"	205,000	125	1253	
7-19-64	"	220,000	128	1254	
7-24-64	"	200,000	127	1255	
7-24-64	Velocity at section at end of rip-rap, taken at 0.6 depth	132,000	123	1322	
11-10-64	Wing wall effects	279,000		1351	
11-10-64	"	279,000		1352	

Table I. (continued).

Approximate Date	Description of Model or of Test	Prototype Q (cfs)	Prototype Tailwater Elev. (ft.)	Original Data Page	Remarks
11-10-64	Wing wall effects	279,000		1353	
11-15-64	"	279,000		1354	
********BOTTOM OF DOWNSTREAM CHANNEL LOWERED TO ELEV. 95.0**					
12-23-64	Velocity at section at end of rip-rap, taken at 0.6 depth	279,000	131	1323	
12-23-64	"	279,000		1324	
12-23-64	Wing wall effects	279,000		1325	

Table II. Summary of Tests - Revised Design.

Approximate Date	Description of Model or of Test	Prototype Q (cfs)	Prototype Tailwater Elev. (ft.)	Original Data Page	Remarks
3-31-65	Hydraulic jump locations	330,000	125.0 ~ 144.5	1328	
3-31-65	"	290,000	123.1 ~ 143.5	"	
3-31-65	"	132,000	120.3 ~ 130.7	1329	
3-31-65	"	60,000	121.3 ~ 128.5	"	
4- 1-65	"	60,000	110.3 ~ 132.4	1330	
4- 1-65	Velocity at section at end of rip-rap	60,000	117.0	1331	
4- 2-65	Hydraulic jump locations	132,000	113.0 ~ 137.9	1333	
4- 2-65	Velocity at section at end of rip-rap	132,000	123.0	1334	
4- 2-65	W. S. Elev. from Sta. 105 + 00 to 120 + 00	132,000	123.0	1335	
4- 2-65	W. S. Elev. from Sta. 99 + 50 to end sill	132,000	123.0	1336	
4- 6-65	W. S. Elev. from Sta. 105 + 00 to 120 + 00	60,000	117.0	1337	
4- 6-65	W. S. Elev. from Sta. 99 + 50 to end sill	60,000	117.0	1338	
4- 6-65	W. S. Elev. from end sill to Sta. 110 + 00	60,000		1339	Jump location from wier crest 176 feet
4- 6-65	"	60,000		1340	" 184 feet
4- 7-65	"	60,000		1341	" 200 feet
4- 7-65	"	60,000		1342	" 211 feet
4- 7-65	Tailwater Elev. at which jump becomes unstable	60,000~ 330,000	Varied	1343	
4- 7-65	W. S. Elev. from end sill to Sta. 110 + 00	60,000		1344	" 240 feet
4- 7-65	"	132,000		1345	" 183 feet
4- 8-65	"	132,000		1346	
4- 8-65	"	132,000		1347	" 255 feet
4- 8-65	"	290,000		1348	" 220 feet
4- 9-65	"	290,000		1349	" 172 feet
4- 9-65	"	290,000		1350	" 210 feet

Table II. (continued).

Approximate Date	Description of Model or of Test	Prototype Q (cfs)	Prototype Tailwater Elev. (ft.)	Original Data Page	Remarks
4- 9-65	Velocity at section at end of rip-rap	290,000	131.0	1355	
4-14-65	Tailwater Elev. at which jump becomes unstable	330,000, 290,000	Varied	1356	
4-30-65	"	290,000, 330,000	Varied	1357	1 row baffle blocks, washer added
5- 3-65	"	290,000, 330,000	Varied	1358	2 rows baffle blocks, washer added
5- 5-65	"	290,000, 330,000	Varied	1359	2 rows baffle blocks, washer added, position changed; end sill raised 3 feet
5- 5-65	"	290,000, 330,000	Varied	1360	1 row or no baffle blocks; end sill raised 3 feet
5- 7-65	Velocity at section at end of rip-rap taken at 0.6 and 0.4 depth	290,000	131	1363	2 rows of elevated baffle blocks shifted downstream with 5 foot end sill (original design)
5- 7-65	W.S. Elev. from end sill to Sta. 116 + 00	290,000	131	1364	
5- 7-65	W.S. Elev. from Sta. 99 + 50 to wier crest + 230	290,000	131	1365	
5-10-65	Velocity at section at end of rip-rap taken at 0.6 and 0.4 depth	60,000	117	1366	
5-11-65	W.S. Elev. from Sta. 99 + 50 to 116 + 00	60,000	117	1367	
5-11-65	Velocity at section at end of rip-rap taken at 0.6 and 0.4 depth	132,000	123	1368	
5-12-65	W.S. Elev. from Sta. 99 + 50 to 116 + 00	132,000	123	1369	

Table II. (continued).

Approximate Date	Description of Model or of Test	Prototype Q (cfs)	Prototype Tailwater Elev. (ft.)	Original Data Page	Remarks
5-12-65	Velocity at section at end of rip-rap taken at 0.6 and 0.4 depth	330,000	132	1370	
5-12-65	W.S. Elev. from Sta. 99 + 50 to 116 + 00	330,000	132	1371	
6- 1-65	Velocity at section at center of rip-rap taken at 0.4 depth	44,000	114	1374	Gates closed except 5, 7; head water = 172 feet
6- 2-65	"	88,000	119	1375	Gates 3,5, 7,9 fully open, others closed; headwater = 172 feet
6- 8-65	Velocity at section at center of rip-rap taken at 0.4 depth	132,000	123	1378	Gates 1,3,5, 7, 9,11, fully open, others closed; headwater = 172 feet
6- 9-65	"	154,000	124	1379	Gates 1,3,5, 6,7,9,11, fully open, others closed; headwater = 172 feet
6-10-65	"	198,000	126	1380	Gates 2,10 closed, others fully open; headwater = 172 feet
6-11-65	"	242,000	128	2221	All gates fully open; headwater = 172 feet
6-14-65	"	22,000	109	2222	Gate 6 fully open, others closed; headwater = 172 feet

Table II. (continued).

Approximate Date	Description of Model or of Test	Prototype Q (cfs)	Prototype Tailwater Elev. (ft.)	Original Data Page	Remarks
6-14-65	Velocity at section at center of rip-rap taken at 0.4 depth	66,000	117.5	2223	Gates 4,6,8 fully open, others closed; headwater = 172 feet
6-14-65	"	110,000	121.5	2224	Gates 2,4,6,8, 10 fully open; others closed; headwater = 172 feet
6-15-65	"	154,000	124	2225	Gates 1,3,9,11 closed, others fully open; headwater = 172 feet
6-16-65	"	198,000	126	2226	Gates 1,11 closed, others fully open; headwater = 172 feet
6-21-65	Average velocity from Sta. 105 + 00 to 117 + 00 along left and right banks and centerline	60,000	117	2227	
6-22-65	"	132,000	123	2228	
6-22-65	"	290,000,	131,	2229	
6-22-65	"	330,000	132	"	
7- 8-65	Length and profile of hydraulic jump	60,000	117	2230	
7- 8-65	"	132,000	123	"	
7- 8-65	"	290,000	131	2231	
7- 8-65	"	330,000	132	"	