TECHNICAL REPORT H-69-11

WEBBERS FALLS LOCK AND DAM
ARKANSAS RIVER NAVIGATION PROJECT

Hydraulic Model Investigation

by

J. J. Franco
J. E. Glover

August 1969

Sponsored by

U. S. Army Engineer District
Tulsa

Conducted by

U. S. Army Engineer Waterways Experiment Station
CORPS OF ENGINEERS
Vicksburg, Mississippi

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US ARMY ENGINEER WATERWAYS EXPERIMENT STATION
VICKSBURG, MISSISSIPPI
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FOREWORD

The model investigation reported herein was authorized by the Office, Chief of Engineers, in the 2d indorsement dated 11 May 1964 to a letter from the U. S. Army Engineer District, Tulsa, dated 28 April 1964. The study was conducted for the U. S. Army Engineer District, Tulsa, in the Hydraulics Division of the U. S. Army Engineer Waterways Experiment Station during the period August 1964 to December 1966.

The investigation was conducted under the general supervision of Mr. E. P. Fortson, Jr., Chief of the Hydraulics Division, and under the direct supervision of Mr. J. J. Franco, Chief of the Waterways Branch. The engineer in immediate charge of the model was Mr. J. E. Glover, Chief of the Potamology Section, assisted by Messrs. E. E. Moorhead, and R. K. Anglin. This report was prepared by Messrs. Franco and Glover.

During the course of the model study, the Tulsa District was kept informed of the progress of the study through monthly progress reports and interim reports of tests. Messrs. E. E. Hudson of the Tulsa District and E. B. Madden of the Southwestern Division visited the Waterways Experiment Station at intervals to observe model tests and discuss test results. Visits were also made by COL G. A. Rebh, Messrs. M. W. DeGeer, LeRoy Crabbe, R. W. Lane, John L. Bush, George C. Kelley, W. T. Moore, and A. N. Steele of the Tulsa District.

Directors of the Waterways Experiment Station during the conduct of the tests and preparation and publication of this report were COL Alex G. Sutton, Jr., CE; COL John R. Oswalt, Jr., CE; and COL Levi A. Brown, CE. Technical Directors were Messrs. J. B. Tiffany and F. R. Brown.
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TABLES 1 and 2
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British units of measurement used in this report can be converted to metric units as follows:

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Webbers Falls Lock and Dam, proposed for construction on the Arkansas River about 432 miles above the junction of the Mississippi and White Rivers, will provide a navigable pool for 33 miles upstream to Lock and Dam No. 17 on the Verdigris River. The project comprises a nonnavigable gated dam with twelve 50-ft-wide by 41-ft-high tainter gates, a 110- by 600-ft lock on the left bank with a maximum lift of 30 ft, and a three-unit powerhouse with a discharge capacity of 35,500 cfs on the right bank.

A 1:120-scale fixed-bed model, reproducing approximately 3.1 miles of the Arkansas River, was used to determine flood stages at the dam, navigation conditions in the lock approaches, tendency for sediment deposition in the lower lock approach, and magnitude of surges generated in the upper lock approach canal by lock filling, and to develop modifications required to provide satisfactory conditions.

The investigation has resulted in the development of modifications to the original design required to produce satisfactory navigation conditions in the approaches to the lock and improve flow conditions through the spillway. In general, the results have indicated the following:

a. Realignment of the entrance to the upper lock approach canal would be required to eliminate hazardous conditions for downbound tows attempting to enter the canal. Ports would be required in the upper guard wall to reduce crosscurrents near the entrance to the upper lock approach canal and to reduce surges resulting from lock filling operations.

b. Surges would develop in the upper lock approach canal from lock filling operations. The magnitude of the surge could be reduced to the point that it would have no serious effect on navigation with ports in the upper guard wall and a surge basin near the lock.

c. Water-surface elevations at the dam embankment to the left of the lock could be lowered by structures designed to eliminate overbank flow toward the embankment and improve the alignment of currents approaching the spillway.

d. Sediment below the dam would be moved into the lower approach channel and could produce shoaling.

e. The size of the surge basin and the length of the upper guard wall could be reduced without any appreciable effect on navigation.

f. The amount of excavation in the approach to the spillway could be reduced appreciably without affecting flow through the spillway.
PART I: INTRODUCTION

Present Development Plan for the Arkansas River

1. Although the Arkansas River is considered a navigable stream from its mouth to the mouth of the Verdigris River, during periods of low water the controlling depths of the river are about 2 ft* from its mouth to Little Rock, and about 1 ft from Little Rock to the mouth of the Verdigris River. In this section, the slope of the stream averages 0.9 ft per mile above Little Rock and 0.7 ft per mile between Little Rock and the Mississippi River. Water-surface elevations and slopes in the lower river are affected by backwater from the Mississippi; at times these effects extend as far upstream as the vicinity of Pine Bluff (mile 111).

2. As presently authorized, the Arkansas River multipurpose project provides for the improvement of the Arkansas River and its tributaries in Arkansas and Oklahoma through the construction of coordinated developments to serve navigation, produce hydroelectric power, afford additional flood control, and provide related benefits such as public facilities for recreation and conservation of fish and wildlife. The navigation feature of the project provides for a 9-ft-deep channel from Catoosa, Okla., on the Verdigris River, 52 miles downstream to the Arkansas River at mile 458, thence down the Arkansas River to Arkansas Post, about 46 miles from its mouth (fig. 1). From this point the Arkansas Post Canal will connect the Arkansas River with the White River. The navigation channel will then continue down the White River for about 10 miles to its junction with the Mississippi River. The 9-ft-deep channel will be provided by a system of

* A table of factors for converting British units of measurement to metric units is presented on page vii.
locks and dams, some of which will be used not only for navigation but also for the production of hydroelectric power. Lock chambers will be 110 by 600 ft on the Arkansas River and in the Arkansas Post Canal, and 83 by 600 ft on the Verdigris River. A minimum channel width of 150 ft is proposed for the Verdigris River section, 250 ft for the Arkansas and White River sections, and 300 ft for the Arkansas Post Canal. Bank stabilization and channel rectification works such as training dikes, cutoffs, and revetments are included in the multipurpose plan and are part of the proposed overall development of the Arkansas River.

Description of Structures and Improvements

3. Webbers Falls Lock and Dam is one unit of the plan for the development of the Arkansas River. It is a multipurpose navigation and hydroelectric power project located on the Arkansas River about 432.2 river miles above the junction of the Mississippi and White Rivers. The dam will provide a normal upper pool elevation of 490.0,* which will provide a 9-ft navigable depth for about 33 miles upstream to Lock and Dam No. 17 on the Verdigris River. The lock will be 110 by 600 ft with a maximum lift of 30 ft, and the powerhouse will have three turbines with a generating capacity of 44,000 kw. The dam will consist of an ogee-shaped overflow section with a crest at el 450.0 containing twelve 50-ft-wide by 41-ft-high tainter gates for control of flow between thirteen 10-ft-wide piers.

Purpose of Model Study

4. The original design of Webbers Falls Lock and Dam was based on sound theoretical design practice and experience with similar structures. However, navigation conditions vary with location and flow conditions upstream and downstream of a structure, and an analytical determination of the hydraulic effects that can reasonably be expected to result from a

* All elevations (el) cited herein are in feet referred to mean sea level.
particular design is not only difficult to achieve, but also uncertain in its findings. Thus, a comprehensive model study to investigate these effects was considered necessary. Specifically, the purposes of the model study were to:

a. Determine navigation conditions in the lock approaches with various flows.

b. Develop modifications required to eliminate any undesirable navigation conditions in the lock approaches.

c. Obtain an indication of the effects of sediment movement on channel maintenance.

d. Determine flow conditions that could be expected through the spillway, and determine water-surface elevations along the dam embankments with the maximum design discharge.
PART II: THE MODEL

Description

5. The model, which reproduced a short reach of the Arkansas River extending from 10,000 ft above to 6300 ft below the proposed site for Webbers Falls Lock and Dam (fig. 2), was of the fixed-bed type with the channel and overbank areas molded in sand-cement mortar to sheet metal templates. The dam, lock, guard walls, and powerhouse were fabricated of sheet metal. The lock and dam gates were simulated schematically with sheet metal slide type gates. The powerhouse was also fabricated from sheet metal and was provided with small rectangular weirs that could be adjusted to regulate flow through the powerhouse.

6. The model was molded in accordance with a 1964 hydrographic and topographic survey except for areas proposed for dredging that were molded to after-dredged conditions. Sufficient overbank area was included in the model to give an accurate reproduction of flow conditions up to the spillway design discharge of 1,200,000 cfs.

Scale Relations

7. The model was built to an undistorted scale ratio of 1:120, model to prototype, to effect an accurate reproduction of velocities, crosscurrents, and eddies that would affect navigation. Other scale ratios resulting from the linear scale ratio were as follows:

| Area       | 1:14,400 |
| Velocity   | 1:10.95  |
| Time       | 1:10.95  |
| Discharge  | 1:157.743|
| Roughness (Manning's n) | 1:2.22  |

Model quantities have been converted to prototype equivalents by means of these ratios.

Appurtenances

8. Water was supplied to the model by a 10-cfs pump operating from a
central sump, and discharge was measured by means of venturi meters. Watersurface elevations were measured by 13 piezometers located in the channel and overbank areas adjacent to the dam, and connected to a centrally located gage pit. Special continuous recording gages were used to measure surges and head on the lower lock gate. Upper pool elevations were controlled by opening and closing the dam gates; tailwater elevations were controlled with a tailgate located at the lower end of the model.

9. Velocities and current directions were determined by means of wooden cylindrical floats weighted on one end to simulate the maximum permissible draft for barges using the waterway (9 ft prototype, the proposed project depth). The paths of the floats were traced by means of ranges placed across the model, and the velocities were determined by timing the travel of floats between ranges. A model towboat and tow (fig. 3) were used to determine and demonstrate the effects of currents on tows navigating the lock approaches. The overall length and width of the towboat with tow simulated the largest tow for which the lock was designed, 104 by

Fig. 3. Remote-controlled towboat and tow
600 ft. The towboat used was somewhat larger than would normally be expected to operate on the water, but the power of the boat was adjusted to provide for the maximum speed that could be developed by most towboats using the waterway. The towboat was equipped with twin screw-type propellers, each powered by a small electric motor operating from batteries located in the tow. The towboat was remote-controlled and could be made to run forward or reverse at any one of four speeds; the rudders could be set at any desired position.

Model Adjustment

10. Inclusion of the proposed plan in the initial model construction precluded adjustment to existing prototype conditions. This type of adjustment was not considered necessary since the proposed improvements would involve radical changes from existing conditions. The model surface was of brushed concrete, providing a roughness (Manning’s n) of about 0.013, which corresponds to a prototype roughness of about 0.029. Based on experience with other models of this type, brushed concrete gives a close approximation of roughness required to reproduce prototype conditions. Because of the short reach of river included in the model, any errors in simulation of prototype roughness would have little effect on test results.
PART III: TESTS AND RESULTS

11. Tests were concerned primarily with the study of navigation conditions approaching the lock as indicated by current patterns and velocities and the behavior of the model tow in the lock approaches with various river flows. Also included in the testing program were studies to develop modifications to reduce stages with flood discharges, determine the height of surge waves generated in the upper approach canal by lock filling, and determine the tendency for sediment to deposit in the lower lock approach. Most of the modifications in the original design were developed during preliminary tests. The evaluation of the effectiveness of these modifications was based mostly on observation of conditions in the model and little or no data were obtained.

Test Procedure

12. Tests to determine navigation conditions consisted of reproducing selected, representative flows and determining current velocities and directions and their effect on the model tow. All navigation flows were controlled flows and were obtained by introducing the proper discharge, manipulating the tailgate until the computed tailwater was obtained, setting the movable weirs in the powerhouse to previously calibrated settings, and manipulating the dam gates until the proper upper pool elevation was obtained. Uncontrolled river flows were obtained by introducing the proper discharge with the powerhouse intake closed and dam gates fully open and manipulating the tailgate until the computed tailwater elevation was obtained. All flows were reproduced as constant flows and permitted to stabilize before data were obtained. Representative flows selected and used during most of the tests were as follows:

a. Flow of 35,500 cfs with all flow passing through the powerhouse. This is the design discharge through the powerhouse and a river discharge that will be equaled or exceeded about 15 percent of the time.

b. An intermediate flow of 80,000 cfs, which is equaled or exceeded about 6 percent of the time. This flow was tested
with 35,500 cfs through the powerhouse and 44,500 cfs through the spillway.

c. Maximum navigable discharge of 150,000 cfs, which is equaled or exceeded about 1 percent of the time. This flow was tested with 35,500 cfs through the powerhouse and the remainder through the spillway.

d. Flow of 1,200,000 cfs, which is the spillway design discharge.

13. Velocities were determined by timing the travel of floats (described in paragraph 9) over a measured distance. Current directions were ascertained concurrently by plotting the paths of the floats with respect to ranges established for that purpose. In plots of turbulent areas where crosscurrents existed, only the main trends are shown in the interest of clarity.

14. The radio-controlled model towboat and tow were observed to determine the effects of currents on its behavior and the maneuvering required to overcome the effect of adverse currents and to demonstrate the effectiveness of plans tested to interested observers.

**Navigation Conditions**

**Original design**

15. **Description.** The features of the original design proposed for the investigation, shown in plates 1 and 2, included the following:

a. A nonnavigable gated spillway with a crest at el 450.0 which contained twelve 50-ft-wide by 41-ft-high tainter gates with 10-ft-wide piers to control flow. A dredged approach to the spillway at el 436.0 and a spillway exit at el 443.0. A 3000-ft-long earth dam with top elevation at 521.0 extending from the lock esplanade to high ground.

b. A 110- by 600-ft lock to the left of the river channel with top of walls at el 500.0.

c. A 600-ft-long upper guard wall with top elevation at 500.0 containing nine 15-ft-high by 20-ft-wide ports, a 12.5-ft-wide by 15-ft-high port near the upstream end of the wall, and a 2.5-ft-wide by 15-ft-high port near the downstream end of the wall. The tops of the ports were at el 480.0.

d. A 4400-ft upper lock approach canal dredged to el 477.0 with fill on both sides with top elevation at 500.0. The fill on the right side of the canal extended from the
entrance to the end of the upper guard wall. The fill along the left side of the canal was terminated at a point opposite the end of the upper guard wall to provide a surge basin on the land side. The surge basin area was dredged to el 477.0.

e. A 600-ft lower guard wall with top elevation at 476.0.

f. A lower lock approach dredged to el 447.0.

g. A three-unit powerhouse to the right of the gated spillway.

16. Results. A large eddy formed below the spillway and extended into the lower lock approach with the powerhouse in operation and no flow through the spillway (plate 3). Upstream currents with velocities of from 0.5 to 1.7 fps were measured along the left bank of the approach channel producing outdraft near the end of the guard wall. Because of the upstream currents within the channel and the set of the currents near the end of the guard wall, upbound tows could experience some difficulties in approaching the lower guard wall. The effects of these currents could be easily overcome by tows moving into the approach at a slight angle from along the right side of the approach channel. With flow through the spillway, eddies would form in the lower lock approach (plates 4 and 5). The size and intensity of the eddies were not sufficient to seriously affect the movement of tows approaching the lower guard wall. Currents with spillway flow would move across the approach channel toward the left side of the channel. These currents would tend to move tows toward the left side of the channel and to resist the movement of the head of a downbound tow riverward after leaving the lock. However, tows proceeding at near normal speed should have little difficulty in maintaining satisfactory alignment.

17. The velocity of currents moving across the entrance to the upper lock approach varied from about 0.7 to about 2.8 fps (plates 3-5). Because of the alignment of the entrance to the upper lock approach canal with respect to the river currents, downbound tows would have difficulty making the turn to enter the canal, especially at the higher flows. In order to enter the canal a downbound tow would have to make a turn by moving its stern riverward near the entrance, exposing its side to river currents moving toward the spillway. River currents on the side of the tow would tend to rotate it counterclockwise as the head moved into the canal entrance, causing the
18. Results of tests to determine the effects of surges resulting from lock filling are discussed later in this report.

**Plans A and A-1**

19. **Description.** Plan A was developed during preliminary tests conducted to determine modifications that could be made to improve navigation conditions at the entrance to the upper lock approach canal. The plan was the same as the original design except for realignment of the left side of the approach canal and the left bank fill to provide a better approach angle for downbound tows entering the canal (plate 1). Plan A-1 was the same as plan A except for the removal of the fill along the right side of the navigation canal.

20. **Results.** Current directions and velocities obtained with plans A and A-1, shown in plate 6, indicate that most of the currents moving across the entrance to the canal were eliminated. Downbound tows moving close along the left bank would experience little difficulty in making a satisfactory entrance into the canal. Removal of the fill along the right side of the navigation channel (plan A-1) increased the velocity of currents near the entrance. Also, flow over the right bank of the canal above the end of the guard wall produced crosscurrents that could cause considerable difficulty for tows approaching the guard wall, particularly with the maximum navigable flow.

**Plans B and B-1**

21. **Description.** Plan B was the same as plan A except that the upper guard wall was reduced to 450 ft in length with a reduction in the top elevation of the fills along the lock approach canal to 493.0, and the spillway approach dredging was revised (plate 1). This plan also included a dike extending from the end of the fill along the left bank of the approach canal to high ground. This dike was installed to reduce water-surface elevations along the dam embankment and would have no effect on navigable flows. Plan B-1 was the same as plan B except for the removal of the lower 3000 ft of the left lock approach fill.

22. **Results.** Reduction of the length of the upper guard wall (plan B) had little effect on currents and navigation conditions at the
entrance to the approach canal. There was a small reduction in the velocity of currents in the canal, which could be attributed to the reduction in the number of ports in the guard wall, but the reduction in flow in the canal was not enough to cause a crosscurrent at the entrance to the canal (plate 7). Removal of the downstream 3000 ft of the left lock approach fill (plan B-1) would have no effect on navigation conditions within the approach canal (plate 7). Without the fill, channel markers would be required in low areas to delineate the channel.

**Plans C and C-1**

23. **Description.** Plan C was the same as plan B except that the upper guard wall was reduced to 300 ft in length and the number of 20-ft-wide ports was reduced to three. Plan C-1 was the same as plan C except that the ports in the upper guard wall were eliminated.

24. **Results.** Reduction of the length of the guard wall and the number of ports (plan C) would reduce flow in the approach canal to such a degree that there would be some crosscurrent at the entrance to the canal (plate 8). With this condition, downbound tows moving close along the left side of the channel should encounter no serious difficulty in entering the canal. Downbound tows attempting to turn into the canal from some distance out in the channel and tows having limited power could experience difficulty entering the canal. Eliminating the ports in the upper guard wall (plan C-1) would cause all flow along the left bank of the river to move across the entrance to the canal (plate 8). Because of these crosscurrents, tows would experience difficulty in making the turn into the approach canal.

**Powerhouse releases**

25. **Results of tests with no flow through the spillway indicated** that a rapid change in powerhouse release from 0 to 35,500 cfs in 6 sec prototype would produce an initial surge wave about 2.0 ft high near the end of the lower guard wall; the water level immediately decreased about 1 ft, then rose gradually until the normal tailwater was reached. A normal change in release from 0 to 35,500 cfs in 18.3 min prototype caused a gradual increase in water-surface elevation with no surges indicated. With the rapid increase in powerhouse discharge, a tow immediately downstream of the guard wall would be moved first toward the left bank then in an upstream
direction. The eddy in the lower lock approach during and for a short time after either type of powerhouse start tended to be smaller than with stabilized conditions, but velocities in the eddy did not appear to be excessive with either condition (photographs 1 and 2).

Head on lower lock gate

26. The maximum head on the lower lock gate with the lock emptying valve open was 0.5 ft with the 150,000-cfs discharge and decreased to 0.3 and 0.0 ft with the 80,000- and 35,500-cfs flows, respectively.

Sediment Movement Below the Dam

27. The study of sediment movement downstream of the dam was conducted by introducing below the dam a granular plastic material with a specific gravity of 1.2 and by tracing its movement with selected flows. These currents indicate that sediment would tend to move toward the left bank across the lock approach channel and could produce shoaling in the channel (plate 9). The alignment of the bottom currents was generally the same for all flows studied except that the crossing of bottom currents with powerhouse flow only would occur farther downstream.

Lock Filling Surges

28. The results of tests made to determine the surges that would be developed in the upper approach canal from lock filling are shown in table 1. The model was adjusted to reproduce the design lock filling curve for both the 25-ft lift 8-min lock filling time and the 30-ft lift 9.6-min lock filling time with a normal upper pool elevation of 490.0 and no river discharge. The magnitude of the surges was measured by means of continuous recording gages located near the lock entrance (sta 2+90) about midpoint of the canal (sta 24+80) and at the entrance to the canal (sta 46+80). Base tests with the original design indicated the maximum surge developed from lock filling would be about 0.4 ft trough to peak (table 1). Without ports in the guard wall, surges would be increased to about 0.8 ft. Maximum surges would be about 1.6 ft with a solid guard wall and no surge basin.
(fill along the left side of the canal extended to the earth dam). Lock filling would produce maximum surges of about 0.5 ft with a 300-ft guard wall containing five ports and the surge basin (plan C). Progressive reduction in the length of the surge basin indicated that the length of the basin could be reduced to 650 ft without any appreciable effect on the magnitude of the surge; eliminating the basin would increase the magnitude of surge to about 1.0 ft. The surge period was affected by lock filling time, ports in the upper guard wall, surge basin, and location within the approach canal. The magnitude of the surge with the surge basin and 300-ft ported guard wall did not appear to have any serious effects on navigation in the canal.

**Spillway Design Discharge**

**Head on dam embankment**

29. Water-surface elevation measured with the original design upstream of the left dam embankment was 518.9 with the spillway design discharge of 1,200,000 cfs. The value obtained in the model was higher than the computed value. This was attributed at least partly to the alignment of currents approaching the spillway along each side and to the currents moving along the left overbank toward the embankment. The water-surface elevation upstream of the spillway was 5.6 ft lower than the elevation along the embankment. A series of tests was made to determine the effectiveness of various plans designed to lower the water-surface elevations along the embankment. Plans tested were as follows:

a. Plan 1 was the same as plan A, except for a dike extending from the land-side lock approach dike to high ground (plate 10).

b. Plan 2 was the same as plan 1 except for a 600-ft breach in the river-side lock approach fill upstream of the end of the guard wall.

c. Plan 3 was the same as plan 2 except for the breach in the river-side lock approach fill which was lengthened to 900 ft.

d. Plan 4 was the same as plan 3 except for a 500-ft angled dike with top elevation of 520.0 about 1000 ft upstream of the powerhouse (plate 10).
Plan 5 was the same as plan 3 except for a 200-ft curved wall with top elevation at 520.0 between the powerhouse and the spillway (plate 10).

Plan 6 was the same as plan 1 except for an improved transition between the powerhouse and spillway (plate 10).

Plan 7 was the same as plan 1 except for an improved transition between the lock and spillway (plate 10).

Plan 8 was the same as plan 1 except for improved transitions between the spillway and lock, and between the spillway and powerhouse.

Plan 9 was the same as plan B (paragraph 21), which was the same as plan 1 except that the length of the upper guard wall was reduced to 450 ft, elevations of the fills along the approach canal were reduced, and dredging in the spillway approach was modified.

Plan 10 was the same as plan C (paragraph 23), which was the same as plan B except that the length of the upper guard wall was reduced to 300 ft and the number of 20-ft-wide ports was reduced to three.

Plan 11 was the same as plan 10 except that a section of the proposed cofferdam with top elevation of 480.0 extending into the channel upstream of the powerhouse was left in place (plate 10).

Plan 12 was the same as plan 11 except that the top elevation of the cofferdam was raised to el 520.0.

Plan 13 was the same as plan 10 except that the dredged approach to the spillway was reduced in length to 500 ft above the spillway (plate 10).

Plan 14 was the same as plan 11 except that an angled dike 1000 ft upstream of the powerhouse with top at el 517.0, a waste fill along the right bank upstream to sta 20+20A, and a 300-ft breach in the river-side lock approach dike were added (plate 10).

Plan 15 was the same as plan 14 except that the 300-ft breach in the lock approach dike was closed.

Results

30. The results of tests of these plans which are summarized in table 2 indicate the following:

a. Elimination of flow along the left overbank toward the approach canal (plan 1) reduced stages at the embankment by about 1.4 ft.
b. A 600-ft breach in the river-side fill along the lock approach canal (plan 2) reduced stages an additional 0.4 ft, a total reduction of 1.8 ft. Increasing the length of the breach to 900 ft (plan 3) had no effect on stages near the embankment. Leaving an opening in the fill along the river side of the canal would produce crosscurrents that would be hazardous to tows moving along the canal.

c. Placement of a dike with top elevation of 520.0 along the right bank about 1000 ft above the powerhouse, with other conditions the same as plan 3 (plan 4), reduced stages an additional 0.6 ft, a total reduction of 2.4 ft. Replacement of the angled dike with a curved wall with top elevation at 520.0 between the powerhouse and spillway (plan 5) reduced stages 0.3 ft more than the dike (plan 4), a total reduction of 2.7 ft.

d. Installation of a more streamlined transition between the powerhouse and spillway (plan 6), and between the lock and spillway (plan 7) with other conditions the same as those of plan 1 reduced stages 0.3 and 0.2 ft, respectively, or 0.5 ft with both transitions installed (plan 8) below that obtained with the plan 1, resulting in a total reduction of 1.9 ft. A check of the water-surface elevation on the right side of the powerhouse near the nonoverflow section of the dam (between the powerhouse and the right bank) with plan 8 indicated it to be about 0.8 ft higher than the water-surface elevation at the embankment on the opposite side of the river (gage 7).

e. Modification of plan 9 had no effect on stages near the left embankment; the reduction in stage was the same as plan 1 (1.4 ft). Reduction of the length of the upper guard wall (plan 10) lowered stages 0.2 ft for a total reduction of 1.6 ft.

f. The section of cofferdam with a top elevation at 480.0 extending into the channel above the powerhouse (plan 11) reduced stages only about 0.1 ft below that obtained with plan 10. Raising the top elevation of the cofferdam (plan 12) to 520.0 reduced the stage at the embankment 1.0 ft below that obtained with plan 10, for a total reduction of 2.6 ft. Reduction of the dredged area at el 436.0 to within 500 ft of the spillway (plan 13) had no effect on stage near the embankment.

g. Combining the cofferdam section of plan 11 and the angled dike 1000 ft upstream of the powerhouse of plan 3 with a 300-ft breach in the fill along the river side of the lock approach canal (plan 15) raised stages 0.2 ft, making a total reduction of 2.1 ft and a water-surface elevation of 516.8.
31. Tests with a 37,000-cfs powerhouse discharge at pool el 487.0, the most adverse condition expected, to determine current pattern and velocities approaching the powerhouse intake indicated that the cofferdam section and dike extending into the channel (plan 14) would not seriously affect the powerhouse intake (plate 11). Water-surface measurements made with the same conditions indicated no measurable drawdown near the powerhouse intake.
PART IV: SUMMARY OF RESULTS AND CONCLUSIONS

Limitation of Model Results

32. Analysis of the results of this investigation is based principally on a study of current directions and velocities in the upper and lower approaches to the lock, the behavior of the model towboat and tow, and the effects of modifications on water-surface elevations. In evaluating test results, it should be borne in mind that small changes in the direction of flow or in velocities are not necessarily changes produced by a change in plan, since several floats introduced at the same point may follow slightly different paths and move at slightly different velocities. Because of the small model scale, it was difficult to reproduce the hydraulic characteristics of the prototype structures or to measure water-surface elevations within an accuracy greater than \( \pm 0.1 \text{ ft} \) (prototype). Also, the model was of the fixed-bed type and was not designed to reproduce sediment movement. The results of sediment tests were based on an evaluation of bottom current patterns as determined from the movement of lightweight material on the bottom of the channel which was not correlated with the material found in the prototype.

Conclusions

33. The following conclusions and indications were developed during the investigation:

a. Downbound tows would encounter considerable difficulty entering the lock approach canal during the higher flows with the original plan because of the velocity and direction of currents moving across the path of tows attempting to make the turn into the canal. Satisfactory navigation conditions could be obtained by realigning the left side of the canal and fill near the entrance to provide a better angle of approach. With this modification, downbound tows would have to approach the entrance to the canal from along the left side of the river channel to avoid the high-velocity currents moving toward the spillway during high flows.

b. A continuous fill or dike would be required along the right
side of the lock approach channel to eliminate crosscurrents within the channel. The fill along the left side of the canal would have little effect on currents affecting navigation and need not be continuous. Without the fill along the left side channel markers would be required to prevent tows from being grounded in areas where depths are inadequate.

c. Ports should be provided in the upper guard wall to eliminate the effects of an eddy that would form near the guard wall and to reduce crosscurrents near the entrance to the lock approach canal. Ports would also tend to reduce the magnitude of surges developed in the canal from lock filling operation. The minimum number of ports required would be that provided with the 300-ft upper guard wall of plan C; conditions would be better with the 450-ft guard wall of plan B-1.

d. Surges would develop in the upper lock approach canal during lock filling. The magnitude of surge would be affected by ports in the upper guard wall and the surge basin. Without a surge basin and ports in the upper guard wall, the magnitude of the surge peak-to-trough would be about 1.6 ft for a single lock filling operation. With the surge basin and ports in the upper guard wall, the magnitude of surge would be reduced to 0.4 or 0.5 ft. The length of the upper guard wall with ports could be reduced to 300 ft and the size of the surge basin could be reduced about 60 percent without any significant effect on the magnitude of surge. A surge having a magnitude of 0.5 ft did not appear to have any serious effects on navigation. The magnitude of the surge could be affected by successive filling operations.

e. A large eddy would form in the lower lock with flow through the powerhouse and no flow through the spillway. The velocity and direction of currents in the eddy could cause some difficulties to upbound tows approaching the lower guard wall at reduced speed. The effects of these currents could be easily overcome by tows moving into the approach at a slight angle from along the right side of the approach channel. The size and intensity of the eddy would be reduced with flow through the spillway.

f. Without spillway discharge, a normal start of powerhouse flow with discharge increased from 0 to 35,500 cfs in 18.3 min would produce no unusual effect on navigation in the lower lock approach. A rapid increase in powerhouse flow from 0 to 35,500 cfs in 6 sec could produce waves about 2 ft or greater in height which could be hazardous to navigation.

g. Currents with spillway flow would move across the approach channel toward the left bank. These currents would tend to move tows toward the left bank and would tend to resist the
movement of the head of a downbound tow riverward after leaving the lock. Tows moving at near normal speed should encounter little difficulty in overcoming the effects of these currents even during high flows.

h. Sediment from the area immediately downstream of the spillway or from the powerhouse tailrace could be moved into the lock approach channel.

i. Water-surface elevation along the left dam embankment with the original design would be higher than the computed value. A reduction in water-surface elevation of as much as 2.6 ft can be effected by eliminating overbank flow toward the embankment and improving the alignment of currents approaching the spillway.

j. Excavation in the spillway approach channel can be reduced to within 500 ft of the spillway without affecting flow through the spillway.

k. The maximum head on the lower lock gate with the lock emptying valve open would be about 0.5 ft with the maximum navigable flow (150,000 cfs) and decrease with decrease in flow.
Table 1

Results of Surge Tests in Upper Lock Approach Channel During Lock Filling

<table>
<thead>
<tr>
<th>Test Conditions</th>
<th>Lock Lift Valve Opening</th>
<th>Lock Filling Time, min</th>
<th>Magnitude of Surge Sta 2+90* Trough Time** Elt Peak Time** Elt</th>
<th>Magnitude of Surge Sta 24+80* Trough Time** Elt Peak Time** Elt</th>
<th>Magnitude of Surge Sta 46+80* Trough Time** Elt Peak Time** Elt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plan</td>
<td>ft</td>
<td></td>
<td>Trough Time** Elt Peak Time** Elt</td>
<td>Trough Time** Elt Peak Time** Elt</td>
<td>Trough Time** Elt Peak Time** Elt</td>
</tr>
<tr>
<td>Original design</td>
<td>30</td>
<td>5.5</td>
<td>9.6</td>
<td>3.8 -0.3 8.3 +0.1</td>
<td>5.8 -0.3 8.6 -0.1</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>3.5</td>
<td>8.0</td>
<td>2.7 -0.4 6.1 0.0</td>
<td>4.2 -0.3 7.2 0.0</td>
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<tr>
<td>Solid guard wall</td>
<td>30</td>
<td>5.5</td>
<td>9.6</td>
<td>3.8 -0.6 17.8 +0.2</td>
<td>4.8 -0.5 9.5 0.0</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>3.5</td>
<td>8.0</td>
<td>2.6 -0.6 16.3 +0.1</td>
<td>4.7 -0.6 8.7 0.0</td>
</tr>
<tr>
<td>Solid guard wall; no surge basin (plan C-1)</td>
<td>30</td>
<td>5.5</td>
<td>9.6</td>
<td>4.3 -1.0 11.8 +0.6</td>
<td>6.1 -1.0 11.3 +0.3</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>3.5</td>
<td>8.0</td>
<td>3.2 -0.9 10.3 +0.5</td>
<td>4.9 -0.8 9.4 +0.4</td>
</tr>
<tr>
<td>300-ft ported guard wall (plan C)</td>
<td>30</td>
<td>5.5</td>
<td>9.6</td>
<td>3.6 -0.5 9.9 0.0</td>
<td>5.7 -0.3 9.9 +0.1</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>3.5</td>
<td>8.0</td>
<td>2.9 -0.5 8.2 0.0</td>
<td>5.2 -0.2 9.9 +0.1</td>
</tr>
<tr>
<td>300-ft ported wall; 1300-ft surge basin</td>
<td>30</td>
<td>5.5</td>
<td>9.6</td>
<td>3.8 -0.5 11.9 0.0</td>
<td>5.6 -0.4 9.9 +0.1</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>3.5</td>
<td>8.0</td>
<td>3.5 -0.4 10.2 0.0</td>
<td>4.9 -0.4 9.2 +0.1</td>
</tr>
<tr>
<td>300-ft ported wall; 650-ft surge basin</td>
<td>30</td>
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<td>9.6</td>
<td>3.8 -0.5 9.9 0.0</td>
<td>5.4 -0.4 9.7 0.0</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>3.5</td>
<td>8.0</td>
<td>3.4 -0.4 8.2 0.0</td>
<td>5.1 -0.4 9.2 +0.1</td>
</tr>
<tr>
<td>300-ft ported wall; no surge basin</td>
<td>30</td>
<td>5.5</td>
<td>9.6</td>
<td>4.6 -0.7 11.3 0.0</td>
<td>5.9 -0.7 11.2 +0.2</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>3.5</td>
<td>8.0</td>
<td>4.3 -0.6 11.9 +0.1</td>
<td>5.1 -0.7 10.1 +0.3</td>
</tr>
</tbody>
</table>

* From upper miter gate pintle.
** Time from start of valve opening, min.
† Elevation in feet referred to normal upper pool, 490.0.
## Table 2

**Effect of Modifications on Upper Pool Stages**

**Discharge 1,200,000 cfs**

<table>
<thead>
<tr>
<th>Plan</th>
<th>Water-Surface Elevation at Gage 7</th>
<th>Reduction in Stage* ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original design</td>
<td>518.9</td>
<td>--</td>
</tr>
<tr>
<td>1</td>
<td>517.5</td>
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<td>1.7</td>
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</tr>
<tr>
<td>15</td>
<td>516.8</td>
<td>2.1</td>
</tr>
</tbody>
</table>

* Compared with the original design.
Photograph 1. Original design: paths of floats (submerged 9 ft) during period 1-12 min after start of powerhouse release. Powerhouse discharge increased from 0 to 35,500 cfs in 6 sec (prototype). Numbers indicate average velocity in feet per second during the 11-min period.
Photograph 2. Original design; paths of floats (submerged 9 ft) during period 25-36 min after start of powerhouse release. Powerhouse flow increased from 0 to 35,500 cfs in 18.3 min (prototype).

Numbers indicate the average velocity in feet per second during the 11-min period.
LEGEND

- CONTOURS ABOVE NORMAL POOL ELEVATION
- CONTOURS BELOW NORMAL POOL ELEVATION
- GAGE LOCATIONS

NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN SEA LEVEL.
LEGEND
- VELOCITY IN FEET PER SECOND
- VELOCITY LESS THAN 0.5 FEET PER SECOND
NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOATS SUBMERGED TO DRAFT OF LOADED BARGES (9 FT).

VELOCITIES AND CURRENT DIRECTIONS
ORIGINAL DESIGN
RIVER DISCHARGE 80,000 CFS
POWERHOUSE DISCHARGE 35,500 CFS
TAILWATER EL 465.7 FT MSL
LEGEND

-xx- VELOCITY IN FEET PER SECOND
-xx- VELOCITY LESS THAN 0.5 FEET PER SECOND

NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOATS SUBMERGED TO DRAFT OF LOADED BARGES (9 FT).

VELOCITIES AND CURRENT DIRECTIONS
ORIGINAL DESIGN

RIVER DISCHARGE 150,000 CFS
POWERHOUSE DISCHARGE 35,500 CFS
TAILWATER EL 471.4 FT MSL

SCALE IN FEET

PROTOTYPE

MODEL
PLATES 6

PLAN A

PLAN A-I

LEGEND

- VELOCITY IN FEET PER SECOND
- VELOCITY LESS THAN 0.5 FEET PER SECOND
- NORMAL POOL

NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOATS SUBMERGED TO DRAFT OF LOADED BARGES (9 FT).

PLANS A AND A-I

RIVER DISCHARGE 150,000 CFS
POWERHOUSE DISCHARGE 35,500 CFS
TAILWATER EL 471.0 MSL

SCALES IN FEET

<table>
<thead>
<tr>
<th>Prototype</th>
<th>0</th>
<th>500</th>
<th>1000</th>
<th>1500</th>
<th>2000</th>
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<tbody>
<tr>
<td>Model</td>
<td>0</td>
<td>10</td>
<td>20</td>
<td>30</td>
<td>40</td>
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</table>
LEGEND

- VELOCITY IN FEET PER SECOND
- VELOCITY LESS THAN 0.5 FEET PER SECOND
- NORMAL POOL

NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOATS SUBMERGED TO DRAFT OF LOADED BARGES (9 FT).

VELOCITIES AND CURRENT DIRECTIONS
PLANS B AND B-1
RIVER DISCHARGE 150,000 CFS
POWERHOUSE DISCHARGE 35,500 CFS
TAILWATER EL 471.0 MSL

SCALES IN FEET

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<th>Prototype</th>
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<th>500</th>
<th>1000</th>
<th>1500</th>
<th>2000</th>
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<tbody>
<tr>
<td>Model</td>
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<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
</tbody>
</table>
LEGEND
- VELOCITY IN FEET PER SECOND
- VELOCITY LESS THAN 0.5 FEET PER SECOND
- NORMAL POOL

NOTE: VELOCITIES AND CURRENT DIRECTIONS OBTAINED WITH FLOATS SUBMERGED TO DRAFT OF LOADED BARGES (9 FT).

VELOCITIES AND CURRENT DIRECTIONS
PLANS C AND C-I
RIVER DISCHARGE 150,000 CFS
POWERHOUSE DISCHARGE 35,500 CFS
TAILWATER EL 471.0 MSL

SCALES IN FEET

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<tr>
<th>Prototype</th>
<th>500</th>
<th>1000</th>
<th>1500</th>
<th>2000</th>
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</thead>
<tbody>
<tr>
<td>Model</td>
<td>0</td>
<td>5</td>
<td>10</td>
<td>15</td>
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</table>

TOP BANK
SURFACE CURRENT DIRECTIONS

BOTTOM CURRENT DIRECTIONS

LEGEND

\[ \begin{align*}
\text{2.0} & \rightarrow \text{VELOCITY, FPS} \\
\rightarrow & \text{LESS THAN 0.5 FPS}
\end{align*} \]

VELOCITIES AND CURRENT DIRECTIONS

PLAN 14

DISCHARGE \quad 37,000 CFS
POOL EL \quad 487.0 FT MSL

PLATE II
WEBBERS FALLS LOCK AND DAM, ARKANSAS RIVER NAVIGATION PROJECT; Hydraulic Model Investigation

Final report

John J. Franco
James E. Glover

August 1969

Technical Report H-69-11

Realignment of the entrance to the upper lock approach canal would be required to eliminate hazardous conditions for downbound tows attempting to enter the canal. Ports would be required in the upper guard wall to reduce crosscurrents near the entrance to the upper lock approach canal and to reduce surges resulting from lock filling operations. Magnitude of the surge could be reduced so that it would have no serious effect on navigation with ports in the upper guard wall and a surge basin near the lock. Water-surface elevations at the dam embankment to the left of the lock could be lowered by structures designed to eliminate overbank flow toward the embankment and improve the alignment of currents approaching the spillway. Sediment below the dam would be moved into the lower approach channel and could produce shoaling. Size of the surge basin and length of the upper guard wall could be reduced without any appreciable effect on navigation. The amount of excavation in the approach to the spillway could be reduced appreciably without affecting flow through the spillway.
<table>
<thead>
<tr>
<th>KEY WORDS</th>
<th>LINK A</th>
<th>LINK B</th>
<th>LINK C</th>
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<td>Arkansas River</td>
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<td>Channel flow</td>
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<td>Hydraulic models</td>
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<td>Webbers Falls Lock and Dam</td>
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