TECHNICAL REPORT H-70-14

SPILLWAY FOR KAW DAM
ARKANSAS RIVER, OKLAHOMA

Hydraulic Model Investigation

by

B. P. Fletcher, J. L. Grace, Jr.

December 1970

Sponsored by U. S. Army Engineer District, Tulsa

Conducted by U. S. 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, on 10 February 1967, at the request of the U. S. Army Engineer District, Tulsa.

The studies were conducted in the Hydraulics Division of the U. S. Army Engineer Waterways Experiment Station during the period February 1967 to May 1968 under the direction of Messrs. E. P. Fortson, Jr., Chief of the Hydraulics Division, and T. E. Murphy, Chief of the Structures Branch, and under the general supervision of Mr. J. L. Grace, Jr., Chief of the Spillways and Conduits Section. The engineer in immediate charge of the model was Mr. B. P. Fletcher, assisted by Messrs. H. H. Allen and A. C. Spivey. This report was prepared by Messrs. Fletcher and Grace.

During the course of the model investigations, Messrs. E. E. Hudson, W. T. Moore, A. M. Smith, H. K. Synder, and D. C. Helmberger of the Tulsa District and Mr. E. B. Madden of the Southwestern Division visited the Waterways Experiment Station to discuss results of the tests and to correlate these results with design studies.

Directors of the Waterways Experiment Station during the testing program and the preparation and publication of this report were COL John R. Oswalt, Jr., CE, COL Levi A. Brown, CE, and COL Ernest D. Peixotto, CE. Technical Directors were Messrs. J. B. Tiffany and F. R. Brown.
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<td></td>
</tr>
</tbody>
</table>
CONVERSION FACTORS, BRITISH TO METRIC UNITS OF MEASUREMENT

British units of measurement used in this report can be converted to metric units as follows:

<table>
<thead>
<tr>
<th>Multiply</th>
<th>By</th>
<th>To Obtain</th>
</tr>
</thead>
<tbody>
<tr>
<td>inches</td>
<td>2.54</td>
<td>centimeters</td>
</tr>
<tr>
<td>feet</td>
<td>0.3048</td>
<td>meters</td>
</tr>
<tr>
<td>miles (U. S. statute)</td>
<td>1.609344</td>
<td>kilometers</td>
</tr>
<tr>
<td>feet per second</td>
<td>0.3048</td>
<td>meters per second</td>
</tr>
<tr>
<td>cubic feet per second</td>
<td>0.02831685</td>
<td>cubic meters per second</td>
</tr>
</tbody>
</table>
SUMMARY

The spillway for Kaw Dam, which is expected to be subjected to discharges as large as 616,200 cfs, was studied on a 1:100-scale comprehensive model to determine flow conditions in the approach and exit channels and the performance of various elements of the structure.

Modifications to the left abutment were recommended to provide a more economical design without inducing excessive turbulence, surging, and/or drawdown in the gate bays adjacent to the left abutment. The capacity of the spillway as determined with the model was slightly less than computed. Modification of the left abutment did not alter the capacity.

Tests indicated that the original design stilling basin performed satisfactorily. However, in order to conform to the foundation, the basin was sloped laterally from left to right and the model indicated that this modification did not adversely affect stilling basin performance.

Tests also indicated that use of a dike to prevent return flow from the large expanse left of the exit channel and stilling basin would permit reducing the height and length of the left training wall and prevent excessive scour along the left portion of the exit channel. The designs of the right stilling basin training wall and the right tailrace training wall also were developed with the model.
Fig. 1. Vicinity map
The Prototype

1. Kaw Dam is a multipurpose project proposed for construction at mile 653.7 on the Arkansas River approximately 10 miles* east of Ponca City, Okla. (fig. 1). The reservoir will be in Kay and Osage Counties, Okla., and Cowley County, Kans., and will provide storage for flood control, water supply, water-quality control, recreation, and fish and wildlife purposes. The principal features of the project are an earth embankment approximately 8416 ft long with a maximum height of 121 ft above the streambed and an 8-gated overfall spillway.

2. The ogee weir section with a crest elevation of 997.5** is designed to pass the spillway design discharge of 616,200 cfs at a head of 54 ft. Flows over the spillway will be controlled by eight 50-ft-wide by 47-ft-high tainter gates. The piers supporting the gates will be 10 ft wide, making the gross length of the weir 470 ft. The weir profile is designed to conform to the shape of the lower nappe resulting from flow over a vertical-faced weir with a design head of 54 ft, which is essentially that required to pass the spillway design discharge. That portion of the weir upstream from the crest is formed by radii of 10.8 and 27.0 ft, and the downstream portion will follow the curve described by the equation, \( Y = 0.01684X^{1.85} \). The general plan, profile, and details of the spillway investigated in this study are shown in plates 1 and 2.

3. Static head water supply and low-flow releases will be made through a 48-in.-diam pipe in the right nonoverflow section and a

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* A table of factors for converting British units of measurement to metric units is presented on page vii.

** All elevations (el) cited herein are in feet referred to mean sea level.
3.5-ft-wide by 4.0-ft-high sluice located in the first pier from the right end of the spillway, respectively.

4. The recommended stilling basin (type 5) will consist of a horizontal apron about 205 ft long, with a uniform transverse slope from el 898.0 on the left to el 893.0 on the right, which is surmounted by two staggered rows of 12-ft-high baffle piers and a 10-ft-high end sill. The left training wall (type 4) will extend the full length of the basin with its top at el 950.0 and a dike (type 4), 250 ft long with top elevation of 974.0 and 1-on-3 side slopes normal to the embankment and parallel to the training wall, will be provided to reduce circulation of flow and excessive scour in the exit channel immediately downstream of the stilling basin. The right training wall (type 11) will extend the full length of the basin and its top will be stepped from el 971.0 to 950.0 and from el 950.0 to 940.0, respectively, at positions 111 ft and 55 ft upstream of the end of the basin.

5. An exit channel 850 ft long with a base width of 470 ft and 1-on-3 side slopes will be provided between the stilling basin and the downstream river channel. An adverse slope of 1 on 10 will be provided between the end sill (el 910.0) and the exit channel invert (el 932.0).

Purpose of Model Study

6. Although the design of the spillway for Kaw Dam was based on sound theoretical design practice, verification of the adequacy of the spillway and appurtenances was desired in view of the high unit discharges and heads involved. The model provided a means for checking the adequacy of approach flow conditions, weir capacity, stilling basin performance, length and height of training walls, and flow conditions in the exit channel and powerhouse tailrace.
PART II: THE MODEL

Description

7. The model (plate 1, fig. 2), constructed to an undistorted scale ratio of 1:100, reproduced all topography and structures in an area extending 2600 ft upstream and downstream from the axis of the spillway and 2200 ft to the left and 1100 ft to the right of the center line of the spillway. The approach, exit, and overbank areas were molded of cement mortar to sheet metal templates and were given a brushed finish. The weir, gate piers, tainter gates, powerhouse, and nonoverflow sections were fabricated of sheet metal. The stilling basin apron and training walls were constructed of plastic-coated plywood. The baffle piers and end sill were made of waterproofed wood.

Appurtenances

8. Water used in the operation of the model was supplied by pumps, and discharges were measured by means of venturi meters. Steel rails set to grade along the sides of the flume provided a reference plane for measuring devices. Water-surface elevations were measured by means of a point gage and velocities were measured with a pitot tube. Current patterns were determined by means of dye injected into the water and confetti sprinkled on the water surface. Tailwater elevations were regulated by a gate at the downstream end of the flume.

Scale Relations

9. The accepted equations of hydraulic similitude based on the Froudeian relations were used to express mathematical relations between dimensions and hydraulic quantities of the model and the prototype. General relations for transference of model data to prototype equivalents are as follows:

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Ratio</th>
<th>Scale Relations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>$L_r$</td>
<td>1:100</td>
</tr>
<tr>
<td>Area</td>
<td>$A_r = L_r^2$</td>
<td>1:10,000</td>
</tr>
<tr>
<td>Velocity</td>
<td>$V_r = L_r^{1/2}$</td>
<td>1:10</td>
</tr>
<tr>
<td>Discharge</td>
<td>$Q_r = L_r^{5/2}$</td>
<td>1:100,000</td>
</tr>
</tbody>
</table>
a. Looking downstream

b. Looking upstream

Fig. 2. The model
PART III: TESTS AND RESULTS

Presentation of Data

10. No attempt is made to present the model tests and results in the chronological order in which the tests were conducted. Instead, as each element of the structure is considered, all tests conducted thereon are described in detail. All model data are presented in terms of prototype equivalents.

Approach Flow Conditions

11. The general plan of the approach area is shown in plate 1 and fig. 2. The approach channel, with a base width of 470 ft and 1-on-3 side slopes, will be excavated to el 935.0 for a distance of about 1900 ft upstream of the spillway. Flow conditions in the approach were generally satisfactory as indicated by the magnitude and direction of currents presented in photograph 1. Lateral flow around the type 1 (original) design left abutment (plate 2) was sufficient to contract flow in the two adjacent gate bays as shown in fig. 3. Water-surface profiles through the first

![Fig. 3. Contraction of flow, type 1 (original) design left abutment; discharge 616,200 cfs](image)
gate bay (that immediately adjacent to the left abutment) resulting with
the design discharge of 616,200 cfs and the original design left abutment
are presented in plate 3.

12. Modifications of the left abutment were investigated to deter­
mine if the thickness of the nonoverflow section could be reduced for
 economical reasons without inducing objectionable drawdown and/or surging
of flow. Details of the left abutment designs investigated are shown in
plate 4. Flow conditions resulting with the design discharge and each
of the left abutment designs are presented in photograph 2. Although flow
in the first gate bay was contracted more with the types 2 and 3 left abut­
ments than with the original design, flow did not separate from either of
these designs and no significant adverse effects could be detected relative
to spillway capacity or stilling basin performance. Thus, adoption of
either type 2 or 3 design left abutment would permit economy to be realized
without impairing hydraulic performance.

Weir and Crest Piers

Description

13. The shape of the weir crest (el 997.5) was based on a design
head of 54 ft and consisted of an upstream portion formed by a compound
curve with radii of 10.8 and 27.0 ft and a downstream portion shaped to
the equation \( Y = 0.0168X^{1.85} \) (plate 2). Details of the crest piers are
also shown in plate 2. The weir will have a gross length of 470 ft and
flow will be controlled by eight 50-ft-wide by 47-ft-high tainter gates.
Since the maximum head anticipated is equal to the design head, negative
pressures should not exist on the weir crest.

Weir capacity

14. The capacity of the weir for anticipated tailwater elevations
(plate 5) as indicated by the model was slightly less than that computed
(plate 6), for an upper pool elevation of 1051.8 rather than 1051.5 was
required for passage of the maximum discharge of 616,200 cfs. The computed
rating curve was obtained by the weir formula \( Q = CH_e^{3/2} \), where \( Q \) is
the discharge in cubic feet per second, \( C \) is the discharge coefficient as
indicated by Corps of Engineers Hydraulic Design Chart (HDC) 111-3, \( H_e \) is the energy head above the weir crest in feet, and \( L \) is the effective length of spillway determined from the expression \( L = 400 - 2H_e K \), in which \( K \) is the summation of the pier and abutment contraction coefficients as shown in HDC 111-5 and 111-3/1, respectively.

**Free uncontrolled flow**

15. A free uncontrolled-flow rating curve is presented in plate 7. The equation presented in plate 7 is the best empirical fit of the free flow data by the method of least squares. For design head conditions, the model indicates an abutment contraction coefficient of 0.19, which is in close agreement with that indicated by HDC 111-3/2. This value was determined by substituting measured quantities of discharge and total energy head in the equation \( Q = C \left[ L - 2(N K_p + K_a H_e) H_e^{3/2} \right] \), where \( C \) is the discharge coefficient indicated by HDC 111-3, \( K_p \) is the pier contraction coefficient indicated by HDC 111-5, \( K_a \) is the abutment contraction coefficient indicated by HDC 111-3/2, \( L \) is the net length of weir, and \( N \) is the number of piers.

**Controlled free flow**

16. Free flow calibration data for various gate openings are presented and compared with computed rating curves in plate 8. Relations between the free controlled-flow discharge coefficient and gross head on the gate for various gate openings are presented in plate 9.

**Stilling Basin**

**Type 1 (original) design**

17. The original design stilling basin consisted of a horizontal apron 208 ft long and 470 ft wide at el 900.0, surmounted with two rows of 12-ft-high baffle piers and a 10-ft-high end sill (plate 3 and Fig. 4). The stilling basin training walls had a top elevation of 959.0 and the left training wall extended 50 ft downstream of the end sill to minimize lateral flow and eddy current erosion. With the maximum discharge of 616,200 cfs and minimum expected tailwater elevation of 971.0, the original design stilling basin provided a tailwater depth and length of basin of about 87 percent
and 2.5 times, respectively, the theoretical depth of tailwater required for a hydraulic jump. Hydraulic actions occurring in the stilling basin with discharges of 300,000 and 616,200 cfs are shown in photograph 3. Water-surface profiles through the first gate bay (adjacent to left abutment) and the stilling basin are presented in plate 3. Velocities observed over the end sill and in the exit channel 270 ft downstream of the end sill are presented in plate 10. Although the lateral flow that returned over the type 1 (original) design left training wall did tend to submerge the hydraulic jump somewhat, as shown in photograph 3, performance of the original design stilling basin was effective. The baffle piers and the vertical-faced end sill tended to contract flow somewhat more than desired, particularly with low and intermediate discharges.

Alternate basin designs

18. Performance of the original design stilling basin with only the first row of baffle piers (type 2 design stilling basin) and without baffle piers (type 3 design stilling basin) was observed and the velocities
measured over the end sill and 270 ft downstream of the end sill (plate 11) indicated the desirability and need for two rows of baffle piers.

19. In order to achieve economy by complying with actual foundation conditions, it was desired that the stilling basin apron be lowered and sloped uniformly from el 898.0 on the left to el 893.0 on the right. This apron with a minimum length of 203.1 ft surmounted with two rows of 12-ft-high baffle piers located 85.6 ft upstream of the end of a 10-ft-high, vertical-faced end sill was termed the type 4 design stilling basin. Hydraulic actions in this stilling basin, exit channel velocities, and flow patterns in the exit channel occurring with discharges of 300,000 and 616,200 cfs are shown in photograph 4, plate 12, and photograph 5, respectively. Sloping the apron from left to right caused no adverse effects on basin action and lowering the apron assisted in reducing the contraction of flow observed over the baffle piers and end sill of the original design stilling basin. Overall, the type 4 design stilling basin was effective in dissipating the energy of all anticipated discharges.

20. The type 5 (recommended) design stilling basin (plate 13) was similar to the type 4 design with the exception that the baffle piers were located 10 ft farther downstream and the face of the end sill was sloped 1 on 1 rather than vertical. These modifications reduced the contraction of flow over the baffle piers and end sill and should facilitate the removal of any debris that enters the stilling basin. Hydraulic actions, exit channel velocities, and exit channel flow patterns observed with the type 5 design stilling basin and the original training walls were essentially the same as those observed with the type 4 design shown in photograph 4, plate 12, and photograph 5, respectively. Hydraulic actions occurring in the type 5 design stilling basin equipped with a left training wall that eliminated lateral flow and submergence of the hydraulic jump (type 10, paragraph 24) for the maximum discharge of 616,200 cfs and tailwater elevations equal to and 13 ft below normal are shown in photograph 6. Performance characteristics of the type 5 design stilling basin for various discharges and a range of tailwater elevations are presented in plate 14. With the maximum discharge of 616,200 cfs, the tailwater had to be lowered 13 ft below normal to induce spray action in this basin.
21. During the course of the studies, it was decided, based upon more accurate foundation investigations, that the final design apron be set with the left side at el 899.0 and the right side at el 896.0. These changes in apron elevation were so small in the model that additional tests were not considered necessary for verification purposes.

Left Stilling Basin Training Wall

Type 1 (original) design

22. Details of the original and alternate designs of the left stilling basin training wall are presented in plate 15. The top of the original design wall was set at el 959.0 to contain the standard project flood discharge of 260,300 cfs. It was extended a distance of 50 ft downstream of the end sill so that the structure would not be endangered by eddy current erosion. However, qualitative sand scour tests indicated intense scour would occur along the left portion of the exit channel downstream of the structure with discharges in excess of 300,000 cfs (fig. 5, plate 16). Relatively uniform scour was observed across the exit channel with lesser discharges.

Alternate designs

23. Minor modifications to the length and height of the left training wall (types 2-8 and 15) had no noticeable effect on the scour pattern in the exit channel. However, the heights of the waves as well as velocities along the downstream left embankment were reduced as the height of wall was increased (photograph 7). Maximum wave heights observed along the left embankment with the types 2 (top el 959.0) and 4 (top el 950.0) left training walls are tabulated below:

<table>
<thead>
<tr>
<th>Q (cfs)</th>
<th>Distance from Spillway (ft)</th>
<th>Maximum Wave Height (ft)</th>
<th>Q (cfs)</th>
<th>Distance from Spillway (ft)</th>
<th>Maximum Wave Height (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>250,000</td>
<td>300</td>
<td>1.0</td>
<td>616,200</td>
<td>300</td>
<td>5.0</td>
</tr>
<tr>
<td>1200</td>
<td>1.0</td>
<td></td>
<td>1200</td>
<td>3.0</td>
<td></td>
</tr>
<tr>
<td>2200</td>
<td>0.5</td>
<td>(Continued)</td>
<td>2200</td>
<td>3.0</td>
<td></td>
</tr>
</tbody>
</table>

(Continued)
Fig. 5. Typical scour pattern observed with types 1-8 left stilling basin training walls after test of 1-hr duration (model) with discharge 616,200 cfs and tailwater el 971.0

<table>
<thead>
<tr>
<th>Q</th>
<th>Distance from 1/3 of Wave Spillway, ft</th>
<th>Maximum Wave Height, ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>250,000</td>
<td>300</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>1200</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>2200</td>
<td>1.0</td>
</tr>
<tr>
<td>616,200</td>
<td>300</td>
<td>7.0</td>
</tr>
<tr>
<td></td>
<td>1200</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td>2200</td>
<td>3.0</td>
</tr>
</tbody>
</table>

Velocities observed along the left embankment with the type 4 left training wall are presented in plate 17.

24. Since the unequal and intense scour of the exit channel was not reduced with walls that extended only to the end sill or 50 ft downstream of the end sill, additional designs (types 9-14) that extended as far as 300 ft downstream of the end sill were investigated. In general, the qualitative sand scour tests indicated that exit channel scour was reduced
as the size of the wall was increased; and the type 10 left training wall was about the minimum size wall that would prohibit unequal and intense scour across the exit channel (plate 18). Because of the estimated cost of the type 10 left training wall, it was desired that the practicality of providing a dike normal to the downstream left embankment in conjunction with a smaller and more conventional left training wall be investigated.

**Combined wall and dike design**

25. The initial (type 1) dike investigated was constructed with side slopes of 1 on 2.5 and a top elevation of 674.0 that extended 600 ft downstream of the end sill. A comparison of the pattern of scour that occurred with the type 1 dike in conjunction with the type 4 left training wall (plate 19) with those shown in plates 16 and 18 indicated that the combined wall and dike design was effective in reducing exit channel scour as well as velocities and wave action along the downstream embankment. The type 2 dike with 1-on-3 side slopes and a top elevation of 674.0 that extended 300 ft downstream of the end sill permitted a scour pattern (plate 20) similar to that obtained with the longer type 1 dike. The type 3 dike (plate 21) allowed greater circulation of flow and higher velocities around the end of the dike relative to that observed with the types 1 and 2 dikes and as a result increased scour along the left portion of the exit channel (compare plates 19, 20, and 21).

26. The type 4 (recommended) dike with top elevation of 674.0 that extended 250 ft downstream of the end sill permitted scour as shown in plate 22. Flow conditions along the type 4 dike and type 4 left training wall for various discharges are shown in photograph 8. Wave heights along the type 4 dike were measured to be approximately 5.0 ft with a discharge of 616,200 cfs, 2.0 ft with 285,000 cfs, and less than 1.0 ft for discharges equal to or less than 100,000 cfs. Wave heights behind the dike and along the left downstream embankment were less than 1.0 ft for all discharges. Water-surface profiles along the type 4 left training wall are presented in plate 23. All things considered, the combination of the type 4 left training wall with the type 4 dike appeared to be the most practical design.
Right Stilling Basin Training Wall

27. Initial tests were conducted with various right training walls (plate 24) to determine if the downstream portion of the wall could be lowered and a cutoff wall added to prevent flow around the powerhouse. A comparison of wave heights in photograph 9 shows the advantages of a cutoff wall. Wave heights with various walls are listed below:

<table>
<thead>
<tr>
<th>Type</th>
<th>Wave Height, ft</th>
<th>Type</th>
<th>Wave Height, ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6.0</td>
<td>4</td>
<td>3.5</td>
</tr>
<tr>
<td>2</td>
<td>6.0</td>
<td>5</td>
<td>4.0</td>
</tr>
<tr>
<td>3</td>
<td>7.0</td>
<td>6-11</td>
<td>3.0-4.0</td>
</tr>
</tbody>
</table>

28. Due to economical and structural considerations, the powerhouse was located farther upstream and the right stilling basin training wall was redesigned (type 6, fig. 6). The type 6 wall eliminated the return flow; however, the lower downstream portion of the wall did not contain the jump and allowed development of eddies in both the stilling basin and tailrace.

Fig. 6. Repositioned powerhouse and type 6 right training wall
Upstream currents caused sand to be deposited in the tailrace (photograph 10).

29. After testing various right stilling basin training walls, the type 11 was considered the most practical (fig. 7). Wave heights along the right bank of the exit channel with the design discharge were about 8 ft at a point 100 ft downstream from the end sill and only 3 ft about 200 ft downstream from the end sill. Waves in this area were about 2.0 ft high with a discharge of 285,000 cfs and 1.0 ft or less for 100,000 and 50,000 cfs. Velocities and current directions are indicated in photograph 11. Water-surface profiles along the type 11 right wall are indicated in plate 25.

Right Tailrace Training Wall

30. The type 2 right tailrace training wall (fig. 7) was installed when the powerhouse was repositioned. Hydraulic performance was satisfactory for all discharges (photograph 11).
31. Hydraulic model investigation of Kaw Dam revealed the adequacy of the overall scheme of the structure; however, minor modifications were proposed in the interest of economy and improved hydraulic performance. Performance of the approach channel was satisfactory. Flow was tranquil in the approach to the spillway and evenly distributed across the spillway crest. Some contraction of flow was observed in the two bays adjacent to the left abutment but was not serious enough to warrant modifications. In the interest of economy, the left abutment was modified to permit reduction of the thickness of the nonoverflow section. The capacity of the spillway was not affected by this modification. The capacity of the spillway was slightly less than expected. The design discharge (616,200 cfs) was passed at a pool elevation of 1051.8 rather than 1051.5.

32. Although the original stilling basin performed adequately, it was found that economics could be affected by sloping the basin laterally to conform to the existing rock strata. Also, the baffle piers were moved 10 ft farther downstream and the end sill was sloped 1 on 1 to allow self-cleaning of the basin.

33. The left stilling basin training wall was reduced in length 50 ft and in height 9 ft; however, it was necessary that a dike protected with rock be placed parallel to the left training wall to reduce circulation of flow and intense scour along the left portion of the exit channel. Results of scour, wave, and velocity tests indicated that this was an adequate design. The right stilling basin training wall was modified to prevent return flow along the upstream portion of the wall. Model tests indicated that the downstream segment of the wall could be reduced in height.
Photograph 1. Conditions in approach area
Photograph 2. Flow conditions at left abutment; discharge 616,200 cfs, pool el 1051.8
Photograph 3. Flow conditions in type 1 (original) design stilling basin
Photograph 4. Flow conditions in type 4 design stilling basin with original training walls
Photograph 5. Flow patterns in exit channel with type 4 design stilling basin and original training walls
Photograph 6. Flow conditions in type 5 (recommended) design stilling basin with type 10 left training wall
Photograph 7. Effect of training wall height on wave action along left embankment; discharge 616,200 cfs, tailwater el 971.0
Photograph 8. Flow conditions with type 4 dike and type 4 left training wall (sheet 1 of 2)
Photograph 9. Wave action along right stilling basin training wall; discharge 616,200 cfs, tailwater el 971.0
Photograph 10. Exit channel scour resulting from discharge of 616,200 cfs with type 6 right training wall, type 4 left training wall, and type 4 dike (1-hr model test duration)
Photograph 11. Flow conditions with type II right stilling basin training wall and type 2 right tailrace training wall (sheet 1 of 2)
WATER–SURFACE PROFILES
TYPE 1 (ORIGINAL) DESIGN LEFT ABUTMENT,
STILLING BASIN, AND LEFT TRAINING WALL
DISCHARGE 616,200 CFS
LEFT ABUTMENT DESIGNS

TYPE 1 (ORIGINAL)

TYPE 2

TYPE 3

PLATE 4
NOTE: MODIFIED TAILWATER CURVE BASED ON FREE FLOW DISCHARGE OVER VALLEY SPILLWAY WITH 8-50'x47' GATES AT CREST EL 997.5.
MODIFIED TAILWATER RATING CURVE BASED ON ULTIMATE DEGRADATION TO EL 909.0.
MODIFIED TAILWATER RATING CURVE IS APPLICABLE AT END SILL OF SPILLWAY STILLING BASIN (APPROXIMATE R.M. 653.8).

TAILWATER RATING CURVE AND SPILLWAY D₂ CURVE
SPILLWAY RATING CURVES
ORIGINAL DESIGN
DISCHARGE - HEAD
RELATION FOR FREE UNCONTROLLED FLOW
ORIGINAL DESIGN SPILLWAY

\[ Q = 220 L H^{1.64} \]
NOTE: GATE OPENING IS VERTICAL DISTANCE IN FEET BETWEEN GATE LIP AND GATE SEAT.
MODEL CURVES ARE BASED ON FLOW THROUGH EIGHT BAYS.
COMPUTATIONS FOR PARTIAL GATE OPENING DISCHARGES BASED ON HOC 311-1 TO 311-4.
LEGEND

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<th>SYMBOL</th>
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NOTE: $Q = CLG_0 \sqrt{2gH_g}$

$Q$ = Total Discharge, CFS
$L$ = Net Length of Spillway, FT
$g$ = Acceleration Due to Gravity
$G_0$ = Gate Opening, FT
$H_g$ = Head to Center of Gate Opening

Discharge Coefficients for Free Controlled Flow
EXIT CHANNEL VELOCITIES
TYPE I (ORIGINAL) DESIGN
STILLING BASIN

DISTANCE, FT
AT END SILL

NOTE: ALL SECTIONS SHOWN LOOKING DOWNSTREAM.
VELOCITIES ARE IN PROTOTYPE FEET PER SECOND.

DISCHARGE 616,200 CFS
POOL EL 1051.5
TAILWATER EL 971.0
NOTE: ALL SECTIONS SHOWN LOOKING DOWNSTREAM.
VELOCITIES ARE IN PROTOTYPE FEET PER SECOND.

DISCHARGE 616,200 CF/S
POOL EL 1051.8
TAILWATER EL 971.0

EXIT CHANNEL VELOCITIES
TYPES 2 AND 3 DESIGN
STILLING BASINS
## Exit Channel Velocities

Type 4 Design Stilling Basin

### 270 FT Downstream of End Sill

**At End Sill**

- **Discharge**: 300,000 CFS
- **Pool EL**: 1032.8
- **Tailwater EL**: 961.0

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### At End Sill

- **Discharge**: 616,200 CFS
- **Pool EL**: 1051.8
- **Tailwater EL**: 971.0

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<tr>
<th>Elevation, ft MSL</th>
<th>Channel Invert</th>
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**Note:** All sections shown looking downstream. Velocities are in prototype feet per second.
TWO STAGGERED ROWS OF BAFFLES, 8.5 FT WIDE AND 9.75 FT APART

SECTION A-A'

PROFILE

TYPE 5 (RECOMMENDED)
STILLING BASIN
NOTE: VELOCITIES WERE MEASURED 5 FT ABOVE AND 5 FT DOWNSTREAM FROM THE END SILL.

PERFORMANCE CHARACTERISTICS OF STILLING BASIN
TYPE 5 (RECOMMENDED) DESIGN
TYPE 1 (ORIGINAL) DESIGN

TYPE 2

TYPE 3

TYPE 4

TYPE 5

TYPE 6

TYPE 7

TYPE 8

TYPE 9

TYPE 10

TYPE 11

TYPE 12

TYPE 13

TYPE 14

TYPE 15

LEFT STILLING BASIN
TRAINING WALLS

PLATE 15
NOTE: DURATION OF SCOUR 1 HR MODEL (10 HR PROTOTYPE).
ELEVATIONS ARE IN FEET REFERRED TO MEAN SEA LEVEL.

TYPICAL SCOUR PATTERNS
TYPES 1-8 LEFT STILLING
BASIN TRAINING WALLS

VELOCITIES ARE IN PROTOTYPE FEET PER SECOND 1 FT OFF BOTTOM.

DISCHARGE 616,200 CFS
TAILWATER EL 971.0

VELOCITIES ALONG LEFT EMBANKMENT
TYPE 5 STILLING BASIN
TYPE 4 LEFT TRAINING WALL WITHOUT DIKE
NOTE: ELEVATIONS ARE IN FEET REFERRED TO MEAN SEA LEVEL.
EXIT CHANNEL MOLDED IN SAND TO EL 9090 PRIOR TO TESTING.

TEST CONDITIONS
DISCHARGE .... 616,200 CFS
POOL EL ....... 1051.6
TAILWATER EL .... 971.0

SCOUR PATTERN
TYPE 5 BASIN
TYPE 10 LEFT TRAINING WALL

PLATE 18
NOTE: ELEVATIONS ARE IN FEET REFERRED TO MEAN SEA LEVEL.
EXIT CHANNEL MOLDED IN SAND TO EL 9090 PRIOR TO TESTING.

TEST CONDITIONS
DISCHARGE......616,200 CFS
POOL EL......1051.8
TAILWATER EL......971.0

SCOUR PATTERN
TYPE 5 BASIN
TYPE 1 DIKE
TYPE 4 LEFT TRAINING WALL
TYPE 6 RIGHT WALL
TYPE 1 POWERHOUSE WALL
NOTE: ELEVATIONS ARE IN FEET REFERRED TO MEAN SEA LEVEL.
EXIT CHANNEL MOLDED IN SAND TO EL 909.0 PRIOR TO TESTING.

TEST CONDITIONS
DISCHARGE........616,200 CFS
POOL EL.............1051.8
TAILWATER EL........971.0

SCOUR PATTERN
TYPE 5 BASIN
TYPE 2 DIKE
TYPE 15 LEFT TRAINING WALL
TYPE 10 RIGHT WALL
TYPE 2 POWERHOUSE WALL
NOTE: ELEVATIONS ARE IN FEET REFERRED TO MEAN SEA LEVEL.
EXIT CHANNEL MOLDED IN SAND TO EL 909.0 PRIOR TO TESTING.

TEST CONDITIONS
DISCHARGE...616,200 CFS
POOL EL...........1051.8
TAILWATER EL....971.0

SCOUR PATTERN
TYPE 5 BASIN

TYPE 3 DIKE
TYPE 15 LEFT TRAINING WALL
TYPE 10 RIGHT WALL
TYPE 2 POWERHOUSE WALL

PLATE 21
NOTE: ELEVATIONS ARE IN FEET REFERRED TO MEAN SEA LEVEL.
EXIT CHANNEL MOLDED IN SAND TO EL 909.0 PRIOR TO TESTING.

TEST CONDITIONS
DISCHARGE: 616,200 CFS
POOL EL: 1051.8
TAILWATER EL: 971.0

SCOUR PATTERN
TYPE 5 BASIN
TYPE 4 DIKE
TYPE 4 LEFT TRAINING WALL
TYPE II RIGHT WALL
TYPE 2 POWERHOUSE WALL

PLATE 22
LEGEND
O RIGHT SIDE
A LEFT SIDE

WATER-SURFACE PROFILES
TYPE 5 BASIN
TYPE 4 LEFT TRAINING WALL
PLATE 24

POWERHOUSE

APRON EL 893.0 EL 903.0

TYPE I (ORIGINAL) DESIGN

APRON EL 893.0 EL 903.0

TYPE 2

EL 959.0 EL 959.0

TYPE 3

EL 959.0 EL 959.0

APRON EL 893.0 EL 903.0

TYPE 4

EL 959.0 EL 959.0

APRON EL 893.0 EL 903.0

TYPE 5

EL 971.0 EL 971.0

APRON EL 893.0 EL 903.0

TYPE 6

EL 971.0 EL 971.0

APRON EL 893.0 EL 903.0

TYPE 7

EL 971.0 EL 971.0

APRON EL 893.0 EL 903.0

TYPE 8

EL 971.0 EL 971.0

APRON EL 893.0 EL 903.0

TYPE 9

EL 971.0 EL 971.0

APRON EL 893.0 EL 903.0

TYPE 10

EL 971.0 EL 971.0

APRON EL 893.0 EL 903.0

RIGHT STILLING BASIN
TRAINING WALLS
WATER-SURFACE PROFILES

TYPE 5 BASIN

TYPE II RIGHT TRAINING WALL

PLATE 25
The spillway for Kaw Dam, which is expected to be subjected to discharges as large as 616,200 cfs, was studied on a 1:100-scale comprehensive model to determine flow conditions in the approach and exit channels and the performance of various elements of the structure. Modifications to the left abutment were recommended to provide a more economical design without inducing excessive turbulence, surging, and/or drawdown in the gate bays adjacent to the left abutment. The capacity of the spillway as determined with the model was slightly less than computed. Modification of the left abutment did not alter the capacity. Tests indicated that the original design stilling basin performed satisfactorily. However, in order to conform to the foundation, the basin was sloped laterally from left to right and the model indicated that this modification did not adversely affect stilling basin performance. Tests also indicated that use of a dike to prevent return flow from the large expanse left of the exit channel and stilling basin would permit reducing the height and length of the left training wall and prevent excessive scour along the left portion of the exit channel. The designs of the right stilling basin training wall and the right tailrace training wall also were developed with the model.
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