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Tests were conducted on a 1:36-scale model of Cooper Lake Spillway to investigate the hydraulic performance of the structure for various flow conditions. The model reproduced approximately a 1,300-ft-long approach area, the entire service spillway including nonoverflow sections, and 800 ft of exit channel.

Approach conditions immediately upstream of the spillway created a buildup of the water surface along the left and right sides of the spillway.

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20. ABSTRACT (Continued).

Modifications to the approach wing walls eliminated the buildup of the water surface, but did not significantly improve the velocity distribution or flow conditions throughout the structure. The modified wing walls were considerably more expensive to construct; therefore, the original wing wall design was reinstalled.

Satisfactory flow conditions were observed on the trajectory and in the stilling basin with all tainter gates opened 3 ft and greater. However, unsatisfactory flow conditions occurred in the stilling basin when using equal tainter gate openings of 2 ft and less or sluice gates to regulate low discharges. The depth of tailwater did not allow the low discharges to spread properly across the lower parabolic trajectory, which resulted in unstable flow conditions in the stilling basin. Modifications consisting of chute blocks and deflectors positioned on the trajectory and sluice outlets, respectively, were ineffective in improving flow conditions in the stilling basin. Optimum combinations of sluice gate operations for regulating various discharges were determined with normal pool elevation (440) and different tailwater elevations. The same optimum arrangements were determined for tainter gate operations.

Riprap requirements were determined for the area immediately downstream from the stilling basin and for the outlet channel.

PREFACE

The model investigation reported herein was authorized by the Office, Chief of Engineers, U. S. Army, on 1 February 1978, at the request of the U. S. Army Engineer District, New Orleans (LMNED). The studies were conducted by personnel of the Hydraulics Laboratory, U. S. Army Engineer Waterways Experiment Station (WES), during the period May 1978 to April 1979. Reassignment of responsibilities for the Cooper Dam and Spillway from LMNED to the Fort Worth District became effective 1 September 1979. All studies were conducted under the direction of Messrs. H. B. Simmons, Chief of the Hydraulics Laboratory, and J. L. Grace, Jr., Chief of the Hydraulic Structures Division. The tests were conducted by Messrs. J. F. George, J. H. Riley, and A. Allen under the supervision of Mr. G. A. Pickering, Chief of the Locks and Conduits Branch. This report was prepared by Mr. George.

Representatives from the Lower Mississippi Valley Division and LMNED visited WES during the study to discuss test results and to correlate these results with concurrent design work.

Commanders and Directors of WES during the testing program and the preparation and publication of this report were COL John L. Cannon, CE, and COL Nelson P. Conover, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
acre-feet	1233.482	cubic metres
cubic feet per second	0.02831685	cubic metres per second
feet	0.3048	metres
feet per second	0.3048	metres per second
inches	25.4	millimetres
pounds (mass)	0.4535924	kilograms

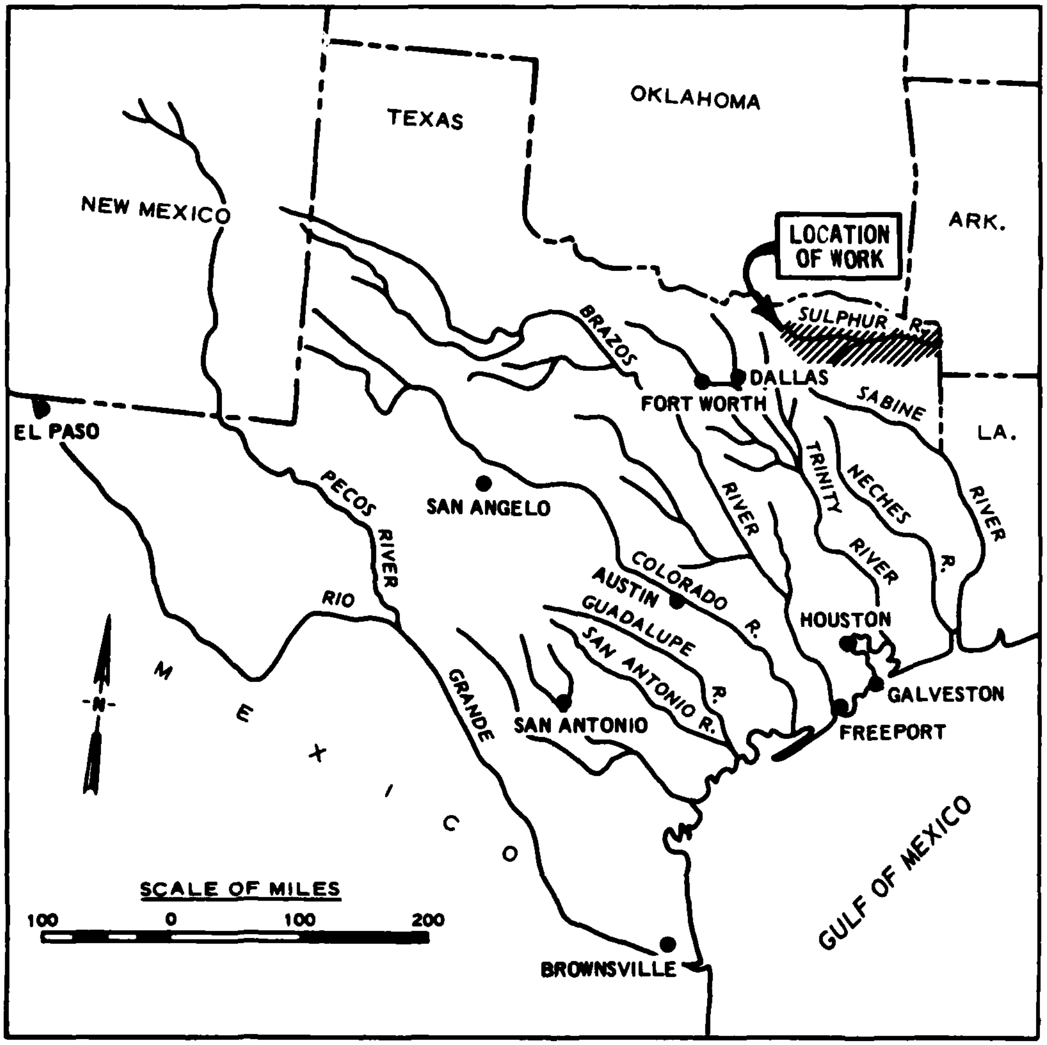

Figure 1. Location map

SPILLWAY FOR COOPER DAM, SULPHUR RIVER, TEXAS

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. The Cooper Dam Spillway (Figure 1) which will be located on South Sulphur River, Texas, is designed to provide flood-control protection, a municipal and industrial water supply, and an area for recreation for the surrounding area in Delta and Hopkins Counties. The flood-control feature of the project is directly related to Wright Patman Lake, in that in addition to providing protection to the floodplain between the two lakes the flood-control storage of 131,000 acre-ft* in Cooper Lake will make available for conversion to water supply storage 120,000 acre-ft of the storage in Wright Patman Lake presently reserved for flood control.

2. The service spillway, to be located on the south abutment approximately 5,300 ft from the main channel of South Sulphur River, will consist of a weir controlled by five 40-ft-wide by 20-ft-high tainter gates separated by 11-ft-wide piers. The weir will have an ogee shape with crest elevation at 426.2** and a 1V-on-1H sloping upstream face. The weir, classified as a low weir, is based on a design head of 26.6 ft which is the maximum head on the spillway crest resulting from the spillway design flood.

3. The spillway chute will be 244 ft wide between parallel walls and will consist of a bucket curve with a 35-ft radius, a 14.55-ft-long 1V-on-20H slope, a 45.6-ft-long parabolic curve, a 34-ft-long 1V-on-3.5H slope, and a bucket curve with a 40-ft radius.

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* A table of factors for converting metric (SI) units of measurement to U. S. customary units is presented on page 3.

** All elevations (el) cited herein are in feet referred to mean sea level (msl).

4. The stilling basin, also 244 ft wide, will be 114.5 ft long from the point of tangency of the spillway chute to the apron. The stilling basin will contain two rows of 7-ft-high by 6-ft-wide baffle blocks that will be spaced approximately 6 ft apart. The upstream row of baffle blocks will be placed 43.5 ft from the downstream end of the stilling basin.

5. In each of the four intermediate service spillway piers a 6-ft by 6-ft conduit with intake invert at el 398.0 will be provided. The conduits, controlled by vertical slide gates, will be approximately 70 ft long at a one percent grade and will discharge on the service spillway chute. One pier will also contain two 24- by 36-in. sluices to allow for selective withdrawal during low-flow periods. Details of the structure are shown in Plates 1 and 2.

Project Design Flood

6. The maximum design discharge in determining the stilling basin requirements was 110,000 cfs with the reservoir at el 452.8. The design discharge consisted of approximately 103,000 cfs for the service spillway and approximately 7,000 cfs for the total capacity of the four conduits at the reservoir elevation of 452.8. The combined discharge was considered to have the energy equivalent to a spillway discharge of the same amount for developing the stilling basin design. At normal upper pool (water supply pool) el 440.0 the service spillway discharge is approximately 36,000 cfs and the combined capacity of the conduits is 6,000 cfs.

Purpose of Model Study

7. A model was considered necessary to verify the adequacy of the original design and develop, if necessary, a design that would provide satisfactory flow conditions throughout the structure. Specifically, the model study was to determine:

a. Approach conditions to the spillway.

b. Performance of the spillway and stilling basin for various flow conditions.

c. Spillway rating curves with partial and full gate openings.

d. The optimum riprap protection plan for the exit channel downstream from the stilling basin.

PART II: THE MODEL

Description

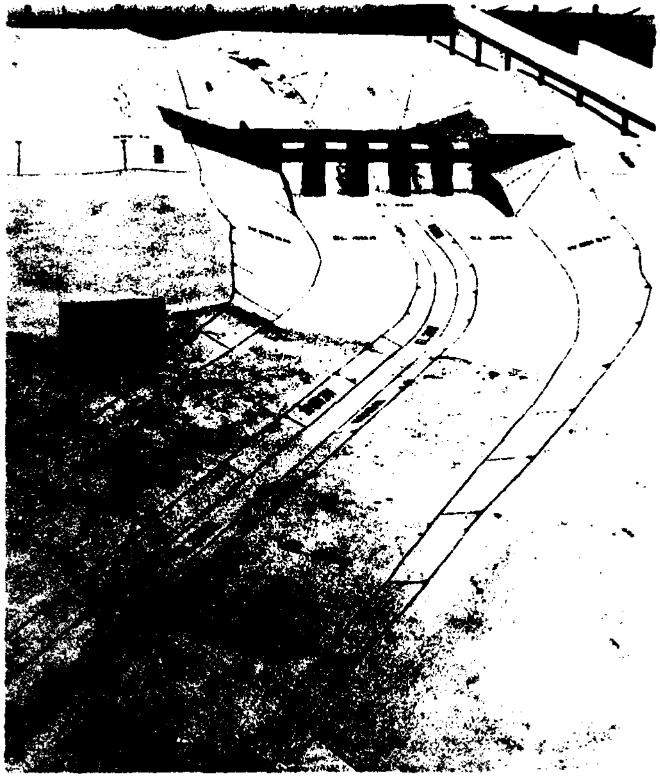
8. The 1:36-scale model reproduced approximately a 1,300-ft-long approach area, the entire service spillway including nonoverflow sections, and 800 ft of exit channel (Figure 2). The proposed spillway and tainter gates were constructed of sheet metal, and all piers and vertical slide gates were constructed of transparent plastic. The stilling basin, baffle blocks, end sill, and stilling basin walls were constructed of plastic-coated plywood. The approach channel, nonoverflow sections, and exit channel were molded in sand and cement mortar to sheet-metal templates initially to observe the performance of the spillway with various flow conditions. The cement mortar was replaced with riprap in later tests to determine the optimum channel protection downstream of the structure. Filter cloth was placed between the sand and graded riprap for all channel protection tests.

Model Appurtenances

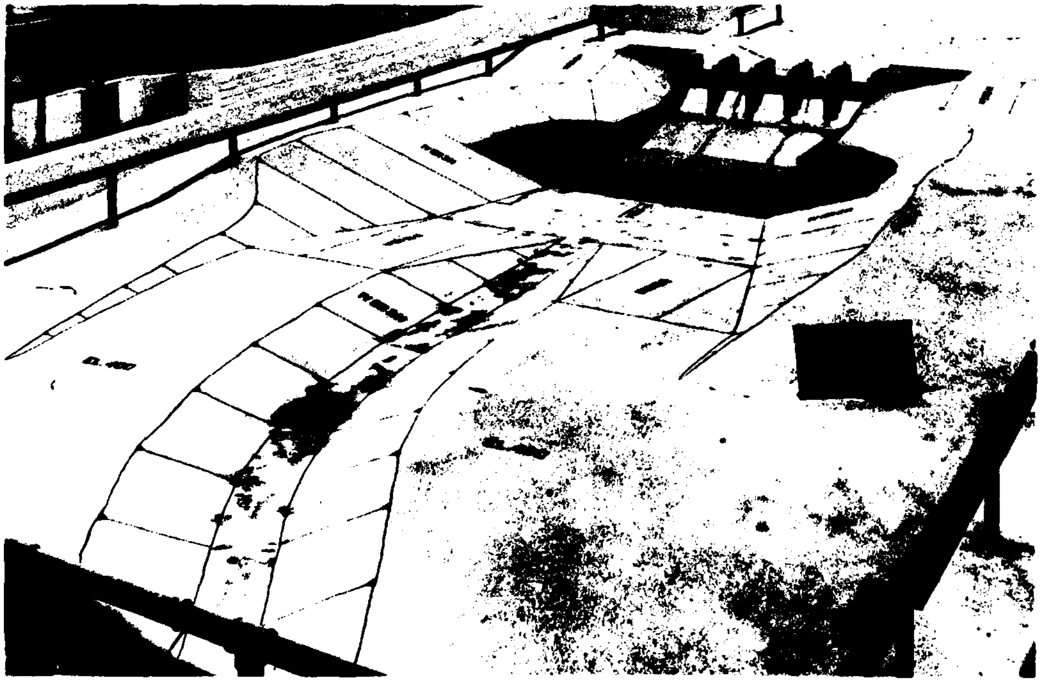
9. Water used in the operation of the model was supplied by a circulating system. Discharges were measured by means of venturi meters installed in the flow lines and were baffled when entering the model. Velocities were measured with pitot tubes that were mounted to permit measurement of flow from any direction and at any depth. Water-surface elevations were measured with point gages. Different designs, along with various flow conditions, were recorded photographically.

Scale Relations

10. The accepted equations of hydraulic similitude, based on the Froudian criteria, were used to express mathematical relations between the dimensions and hydraulic quantities of the model and prototype. General relations for the transference of model data



a. Looking downstream



b. Looking upstream

Figure 2. General view of model

to prototype equivalents are presented below:

Characteristic	Dimension*	Model:Prototype
Length	L_r	1:36
Area	$A_r = L_r^2$	1:1,296
Velocity	$V_r = L_r^{1/2}$	1:6
Discharge	$Q_r = L_r^{5/2}$	1:7,776
Volume	$V_r = L_r^3$	1:46,656
Weight	$W_r = L_r^3$	1:46,656
Time	$T_r = L_r^{1/2}$	1:6

* Dimensions are in terms of length.

Model measurements of discharge, water-surface elevations, and velocities can be transferred quantitatively to prototype equivalents by means of the scale relations. Experimental data also indicate that the prototype-to-model scale ratio is valid for scaling riprap in the sizes used in this investigation.

PART III: TESTS AND RESULTS

Approach Area

11. The model reproduced an 800-ft-wide approach for a distance of approximately 1,300 ft upstream from the spillway (Plate 1, Figure 2). Although satisfactory flow conditions were observed in the approach area upstream of the wing walls for all discharges up to the spillway design flood of 110,000 cfs, buildup of the water surface along the piers at the left and right sides of the spillway became significant with the design discharge (Photo 1). However, velocities measured on the crest and in the stilling basin indicated that fairly uniform velocity distribution was present throughout the structure with the spillway design flood (Plate 3).

12. Several modifications to the approach wing walls upstream of the dam were tested in an attempt to reduce the buildup of the water surface along each side of the spillway and to further improve flow conditions throughout the structure. The type 2 design incorporated the original design wing walls and curved walls positioned on the 1V-on-3H slopes on each side of the spillway as shown in Plate 4. Test results showed that the type 2 design was not effective in reducing the buildup of flow along the walls. Velocities measured in the left bay (looking downstream) with the maximum discharge are shown in Plate 5.

13. The type 3 wing wall design consisted of 56-ft-high walls on each side of the spillway as shown in Plate 6 and Photo 2. This modification significantly reduced the buildup of flow along the walls (Photo 2), but velocities measured in the left bay (Plate 5) indicated little change in velocity distribution relative to the types 1 and 2 designs previously tested.

14. The height of the type 3 wing wall was reduced 29 ft (type 4) so that the top of the wall was at el 425.0. Results of tests conducted with this modification showed little difference in flow conditions than were observed with the types 1 and 2 approach wing wall designs.

15. Flow conditions were observed with rock dikes located at

several locations on the 1V-on-3H slopes with types 1 and 4 wing wall designs. Test results indicated that the dikes would have to be as high as the maximum water-surface elevation to be effective in reducing the buildup of flow along the walls.

16. The type 3 approach wing wall design reduced the buildup of the water surface along the left and right walls of the spillway more effectively than the other designs tested. However, little improvement was obtained in the velocity distribution relative to the other designs tested. Also, little change was observed in the discharge calibration data obtained with the type 3 design as compared with the first two approach wing wall designs tested (Plate 7). The expense of the additional wall height is probably not justifiable since the only benefit derived from this design was a reduction in the water surface along the sides of the spillway. Therefore, the original design (type 1) approach wing walls were reinstalled.

Spillway and Stilling Basin

Tainter gate operations

17. Performance of the spillway and stilling basin was observed and documented with discharges ranging from 8,000 cfs to 110,000 cfs and various upper pool and tailwater elevations. At the normal upper pool elevation of 440.0, flow conditions were satisfactory throughout the spillway and stilling basin for the majority of gate openings and tailwater elevations observed. Velocities measured on the spillway crest and at several locations in the stilling basin with a discharge of 40,000 cfs, the normal upper pool elevation, and all gates fully open indicated uniform velocity distribution throughout the structure as shown in Plate 8. Various flow conditions with normal upper pool are shown in Photo 3.

18. With the design discharge of 110,000 cfs, cross waves were created by the piers in each gate bay (Photo 1). The cross waves were confined to the surface since uniform velocity distributions were observed throughout the depth of flow at the crest and in the stilling

basin (Plate 3). Flow conditions downstream of the stilling basin were considerably rough with significant wave action occurring along the side slopes immediately downstream from the structure (Photo 4).

19. Calibration data obtained with the original design indicated that the structure was slightly more efficient in passing large flows than was anticipated (Plate 9). Gate openings used in calibration tests were set using the shortest distances measured from the spillway to the gate lip.

20. Flow conditions were observed with equal tainter gate operations for discharges ranging from 6,000 cfs to 16,500 cfs at normal upper pool. Satisfactory flow conditions were observed with all gates opened 3 ft and greater. However, unsatisfactory flow conditions occurred in the stilling basin with 1- and 2-ft gate openings. With the lower discharges, the depth of tailwater did not allow the flow to spread properly across the lower parabolic trajectory, which resulted in unstable flow conditions involving concentrated jets and eddies in the stilling basin as shown in Photo 5. These concentrated jets and eddies were present for all tests with tailwater elevations ranging from normal tailwater elevations (Plates 10 and 11) to the minimum possible in the model due to the flow control by the downstream channel (channel control depths, Plate 12). Tainter gate operations listed in Table 1 are gate arrangements producing the weakest eddies in the stilling basin. Combinations using four tainter gates were not listed since any four-gated arrangement produced small intense eddies that could possibly move rocks. The location and numbering system used to identify the tainter gates are shown in Plate 13.

21. Tests were conducted to evaluate the effectiveness of placing chute blocks on the spillway trajectory to spread and distribute the flow entering the stilling basin. Several types of chute blocks (Plate 14) were tested in an effort to improve flow distribution across the lower parabolic trajectory of the spillway with gated discharges of 10,000 cfs and less. Little improvement in flow distribution and stilling basin performance was observed with these modifications and normal tailwaters. Since modifications on the trajectory were ineffective in

improving flow conditions, gate combinations furnished in Table 1 should be considered in regulating low discharges with the tainter gates.

Sluice gate operations

22. Tests were conducted to determine satisfactory gate operations using sluice intake gates to regulate low discharges and avoid adverse flow conditions in the stilling basin. Tests were conducted with different combinations of sluice gates fully open to regulate discharges between 1,500 cfs and 6,000 cfs with normal upper pool elevation of 440.0 and tailwater elevations ranging from normal to channel control depths.

23. With normal tailwater depths, unsatisfactory flow conditions were observed for any and all combinations of sluice gate operations ranging from a single sluice gate to all sluice gates open (Photo 6). The unsatisfactory flow conditions resulted because the excessive tailwater depths concentrated the discharge from the sluices rather than spreading or distributing the flow properly across the lower parabolic trajectory. The unstable flow conditions created small intense eddies that could possibly move small rock and result in abrasive damage to the stilling basin. Although mild eddies occurred in the basin, satisfactory flow conditions were observed when tests were conducted with tailwater depths resulting from channel control. Tests results indicated that the maximum tailwater elevation for satisfactory flow conditions with sluice gate operations was approximately 394.0. Flow conditions with various sluice gate operations and a tailwater elevation of 394.0 are shown in Photo 7.

24. Additional tests were conducted to determine the best arrangements for operations with one, two and three sluice gates for both normal and channel control tailwater depths. Eddies developed in the basin when using up to three sluice gates to regulate low discharges, regardless of the combination used or the elevation of the tailwater. However, the sluice gate combinations listed in Table 1 produced the weakest eddies in the stilling basin. The location and numbering system used to identify the sluices are shown in Plate 13.

25. Deflectors with heights of 1, 2, and 3 ft (Plate 15) were

placed on the invert of the sluice outlets in an attempt to spread and distribute the flow entering the stilling basin. The deflectors were found to be effective in spreading flow over a greater width of the spillway trajectory. However, they did not improve stilling basin performance with sluice discharge operations.

26. Various size chute blocks (Plate 14) were placed immediately downstream of the sluice outlets to aid in spreading the flow entering the stilling basin. The chute blocks were found to be ineffective in spreading releases from the sluices across the trajectory with normal tailwater depths.

27. Since the sluice deflectors and chute blocks did not produce significant beneficial improvement of flow conditions on the spillway trajectory or in the stilling basin, it was recommended that the spillway be operated using gate arrangements furnished in Table 1. Tests were not conducted with combined sluice and tainter gate operations.

Riprap Protection Plans

28. Various riprap plans were tested to determine optimum riprap protection for the area immediately downstream of the stilling basin and for the outlet channel. Velocities were measured 1 ft above the channel invert downstream of the stilling basin for discharges of 40,000 cfs (normal pool el 440.0) and 110,000 cfs (design discharge) with all gates fully open (Plates 16 and 17). Velocities measured approximately 500 ft downstream from the stilling basin were higher than those in the area immediately downstream from the basin due to the exit channel configuration in that vicinity. Riprap tests conducted in an area that extended from the stilling basin to 287 ft downstream were not affected since this increase in velocities occurred 213 ft farther downstream.

Area immediately downstream from stilling basin

29. The type 1 riprap plan (Plate 1) consisted of riprap with an average diameter (D_{50}) of 27 in. on the channel invert from the end sill to 20 ft downstream, 18-in. (D_{50}) riprap from 20 ft to 126.67 ft

downstream, and 12.-in. (D_{50}) riprap on the side slopes from the stilling basin to 126.67 ft downstream. The blanket thickness for each size riprap was equivalent to one and one half times the average diameter of the stones. Tests were conducted with flow conditions resulting from discharges of 40,000 cfs and 110,000 cfs with all gates fully open and tailwater elevations of 406.0 and 410.5, respectively. Failure in the type 1 riprap plan occurred almost immediately along the slopes and on the channel invert as the discharge was increased to 110,000 cfs.

30. In the type 2 riprap plan, 27-in. (D_{50}) stone was placed on the entire channel invert with a blanket thickness of 54 in. The riprap on the side slopes was increased to 18 in. (D_{50}) with a blanket thickness of 36 in. The riprap protection for the channel invert and the side slopes was extended from the stilling basin to 287 ft downstream (Photo 8). After a 36-hr (prototype) test with a discharge of 110,000 cfs, failure occurred just downstream of the end sill (Photo 8).

31. The size of the riprap on the channel invert was increased to an average diameter of 32 in. for a distance of 127 ft downstream from the end sill (type 3, Photo 9). Failure occurred just downstream of the end sill after a 36-hr test with a discharge of 110,000 cfs.

32. The 32-in. (D_{50}) riprap was replaced with 36-in. (D_{50}) riprap with the 27-in. (D_{50}) and 18-in. (D_{50}) riprap protection covering the same areas as in the previous test (type 4, Photo 10). Again, failure was observed just downstream of the end sill after a 48-hr test with a discharge of 110,000 cfs (Photo 10).

33. In the type 5 riprap plan (Photo 11) the small dip in the channel invert immediately downstream of the end sill was eliminated, and the channel invert was modified to a 1V-on-45H upward slope from the end sill to a location 90 ft downstream. The channel invert from the end sill to a location 127 ft downstream was protected with 36-in. (D_{50}) riprap, and the reach from 127 ft to 287 ft downstream was covered with 27-in. (D_{50}) riprap. The side slopes were protected with 18-in. (D_{50}) riprap. During a 36-hr test with the design discharge of 110,000 cfs, failure occurred in an area that extended from the end sill to approximately 20 ft downstream.

34. The type 6 riprap plan (Plate 18, Photo 12) consisted of basically the same riprap plan as the type 5 plan except for the area on the channel invert from the end sill to 30 ft downstream. In this area the size of the riprap was increased from 36-in. (D_{50}) riprap to a uniform gradation of stones varying from 36 in. to 45 in. in diameter (D_{50} = 40 in.). No failure was observed after several 12-hr tests with a discharge of 110,000 cfs. From the results of these tests, the type 6 riprap plan is recommended for the prototype for the area immediately downstream of the stilling basin.

Outlet channel

35. Tests to determine the riprap requirements for the outlet channel were conducted with discharges ranging from 8,000 cfs, bank-full capacity of the outflow channel, to 110,000 cfs, the spillway design discharge. The area immediately and for a distance of 287 ft downstream of the stilling basin was protected with the type 6 riprap protection, while tests were conducted to determine riprap requirements farther downstream in the outlet channel.

36. Initially, the outlet channel in a reach from 287 ft to 867 ft downstream of the end sill was protected with riprap with an average diameter (D_{50}) of 27 in. and a blanket thickness of 54 in. placed on the invert and side slopes (type 7, Photo 13). The type 7 riprap protection plan remained stable throughout a series of tests consisting of a 6-hr duration with a discharge of 8,000 cfs, 6 hr of 40,000 cfs, and 12 hr of 110,000 cfs. The size of the riprap was reduced to an average diameter of 18 in. and a blanket thickness of 36 in. (type 8, Photo 14). No failure was observed in the type 8 riprap protection plan after being subjected to the same series of flow conditions previously described.

37. In the type 9 riprap protection plan 12-in. (D_{50}) riprap with a blanket thickness of 24 in. was placed on the invert and side slopes. This riprap protection plan also remained stable throughout a series of tests consisting of 12 hr of 8,000 cfs, 12 hr of 40,000 cfs, and 12 hr of 110,000 cfs. Based on these tests and results, the type 9 riprap protection plan (Plate 19, Photo 15) was considered to be adequate and was recommended for prototype construction. Smaller riprap was not

tested, since it was deemed impractical. The gradations of the riprap tested in the model are shown in Plates 20-23.

PART IV: DISCUSSION OF RESULTS

38. Flow conditions in the approach area were satisfactory for the range of anticipated discharges. Approach conditions immediately upstream of the spillway created a buildup of the water surface along the piers at the left and right sides of the spillway which was significant with the spillway design discharge. Velocities measured on the crest and in the stilling basin, however, indicated fairly uniform velocity distribution throughout the structure.

39. Modifications to the approach wing walls reduced the buildup of the water surface along the spillway walls, but little improvement was obtained in the velocity distribution throughout the structure. The expense of the additional wall height incorporated in the wing wall modifications is probably not justifiable since the only benefit derived was a reduction in the water surface along the sides of the spillway. Therefore, the original design approach wing walls were recommended for the prototype.

40. Satisfactory flow conditions were observed on the spillway and in the stilling basin with all tainter gates opened 3 ft and greater with normal upper pool el 440.0. With low and intermediate discharges, the hydraulic jump occurred on the spillway chute in the vicinity of the lower parabolic curve. As the discharge increased to flood conditions, the jump moved downstream into the stilling basin where satisfactory energy dissipation occurred. With the design discharge of 110,000 cfs, flow conditions downstream of the stilling basin were considerably rough with significant wave action occurring along the side slopes immediately downstream from the structure.

41. Calibration data obtained with the original design indicated that the structure was slightly more efficient in passing large flows than was anticipated.

42. Unsatisfactory flow conditions were observed with all five tainter gates opened 2 ft and less with normal upper pool el 440.0. With discharges of 10,000 cfs and less, the depth of tailwater did not allow the flow to spread properly across the lower parabolic trajectory.

This resulted in unstable flow conditions involving concentrated jets and eddies in the stilling basin. These unstable flow conditions were present with tailwater elevations ranging from normal tailwater elevations to the minimum possible in the model due to flow control by the downstream channel.

43. Various size chute blocks were placed on the trajectory to spread and distribute the flow entering the stilling basin. Little improvement in flow conditions was observed with these modifications. Since any tainter gate operations in regulating low discharges resulted in unstable flow conditions, regardless of modification tested, gate arrangements producing the weakest eddies in the stilling basin were determined.

44. Unsatisfactory flow conditions were also observed when using sluice intake gates to regulate low discharges with normal upper pool and tailwater elevations. The unsatisfactory flow conditions resulted because the excessive tailwater depths concentrated the discharge from the sluices rather than spreading or distributing the flow properly across the lower parabolic trajectory. The unstable flow conditions created small intense eddies that could possibly move small rock and result in abrasive damage to the stilling basin. However, satisfactory flow conditions were observed when tailwater elevations were at el 394.0 and lower.

45. Different size chute blocks were placed on the trajectory immediately downstream of the sluice outlets to aid in spreading the flow entering the stilling basin, but little improvement in flow conditions was obtained. Various size deflectors were then placed on the invert of the sluice outlets in an attempt to spread and distribute the flow entering the stilling basin. The deflectors were found to be effective in spreading flow over a greater width of the spillway trajectory, but they did not improve stilling basin performance. Since modifications on the trajectory and at the sluice outlet did not produce significant improvement of flow conditions on the spillway or in the stilling basin, sluice gate arrangements producing the weakest eddies in the stilling basin were determined.

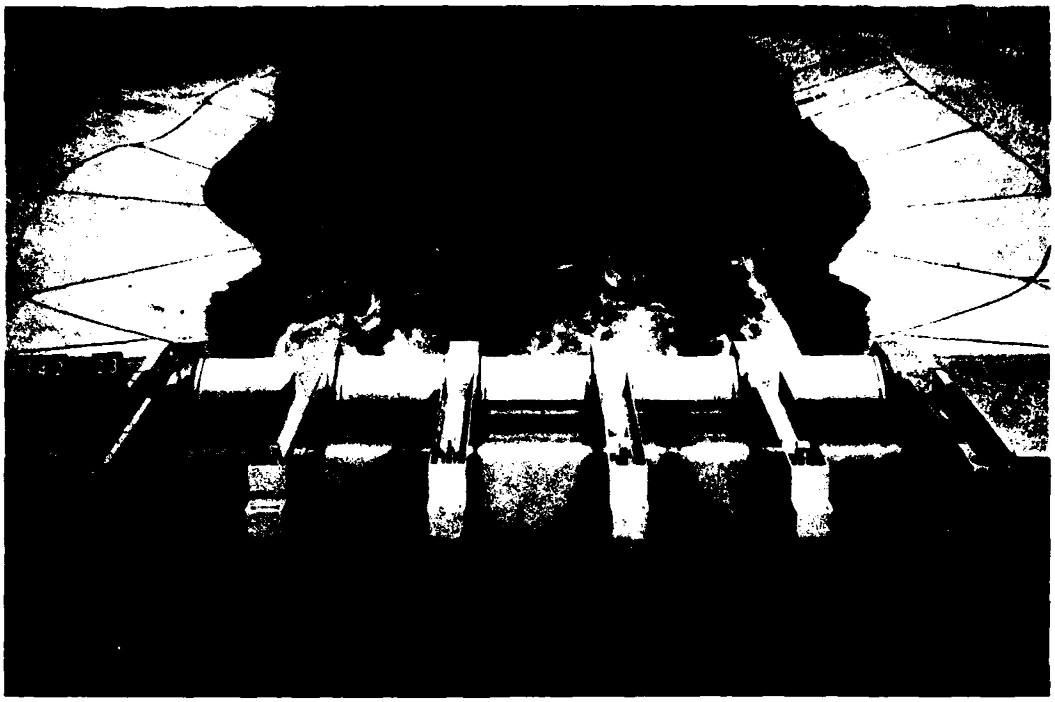
46. Riprap protection plans were developed for the area immediately downstream of the stilling basin (type 6) and for the outlet channel farther downstream (type 9). The minimum riprap protection requirements established were adequate for the full range of expected flows and tailwaters.

Table 1

Cooper Lake Spillway Gate Operations

Gate Opening ft	No. of Gates	Recommended Combination of Gates
	Tainter Gates	
1	1	1 or 5
1	2	1 & 2 or 4 & 5
1	3	1, 2, & 3 or 3, 4, & 5
1	4	--------------------
2	1	1 or 5
2	2	1 & 2 or 4 & 5
2	3	1, 2, & 3 or 3, 4, & 5
2	4	--------------------
	Sluice Gates	
6	1	A or D
6	2	B & C
6	3	A, B, & C or B, C, & D

PHOTOGRAPHS



a. Gate opening 4 ft, discharge 16,000 cfs, tailwater el 402.0

b. Gate opening 6 ft, discharge 23,000 cfs, tailwater el 403.5

Photo 1. Looking downstream at flow conditions immediately upstream of spillway; upper pool el 440.0 (Sheet 1 of 3)

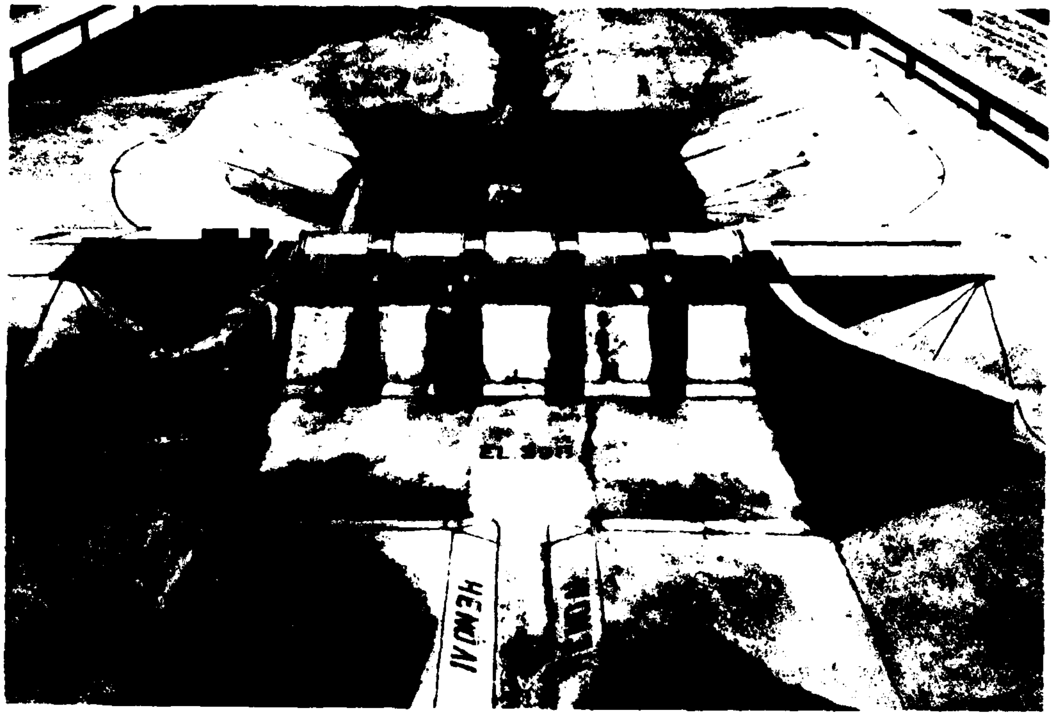
c. Gate opening 8 ft, discharge 30,000 cfs, tailwater el 404.5

d. All gates fully open, discharge 40,000 cfs, tailwater el 406.0

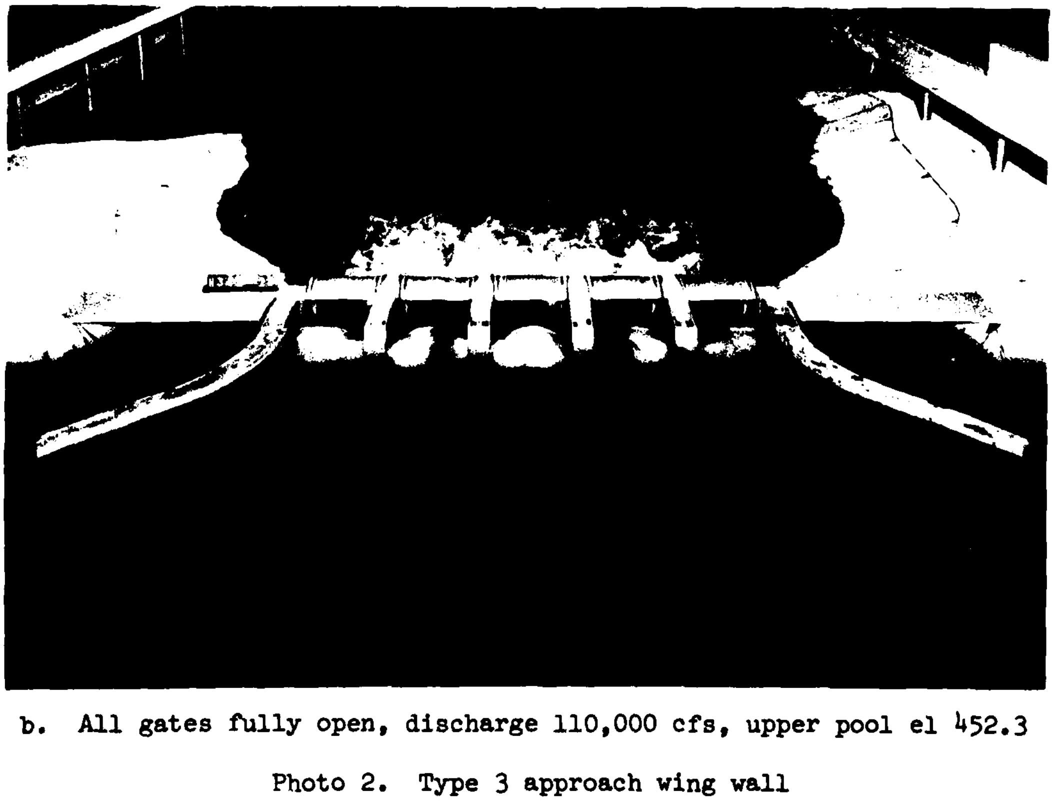
Photo 1. (Sheet 2 of 3)

e. All gates fully open, discharge 110,000 cfs, tailwater el 410.5

Photo 1. (Sheet 3 of 3)



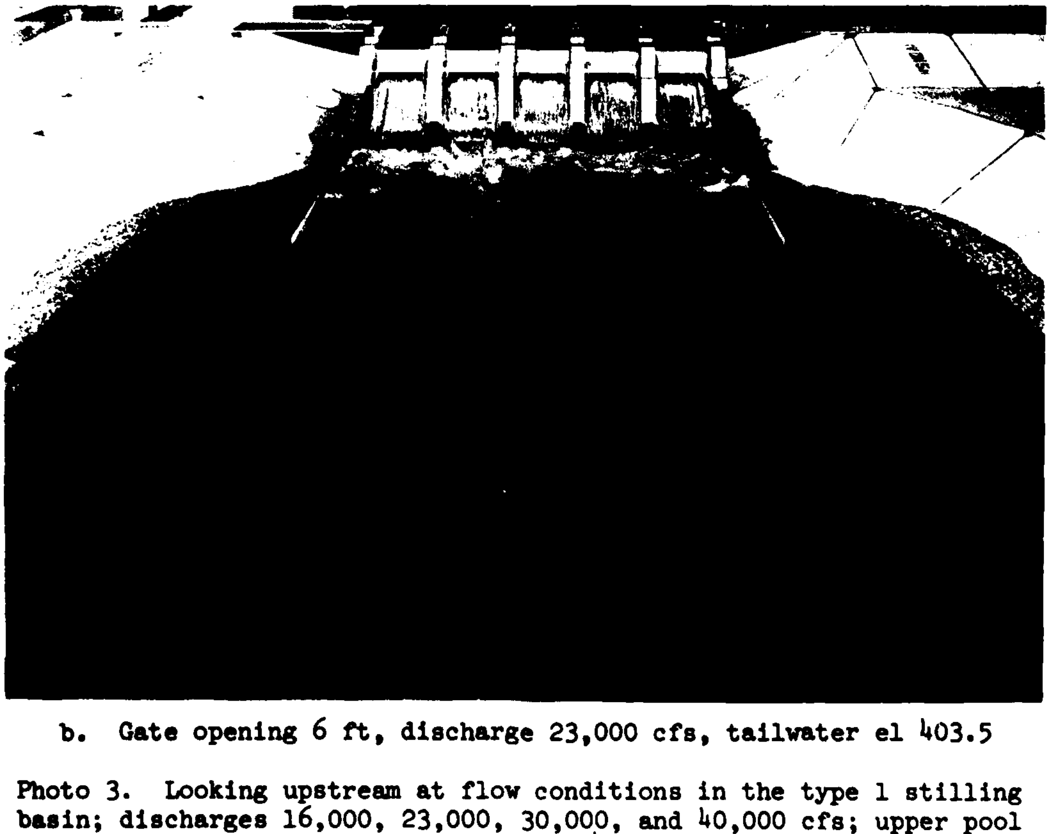
a. Dry bed



b. All gates fully open, discharge 110,000 cfs, upper pool el 452.3

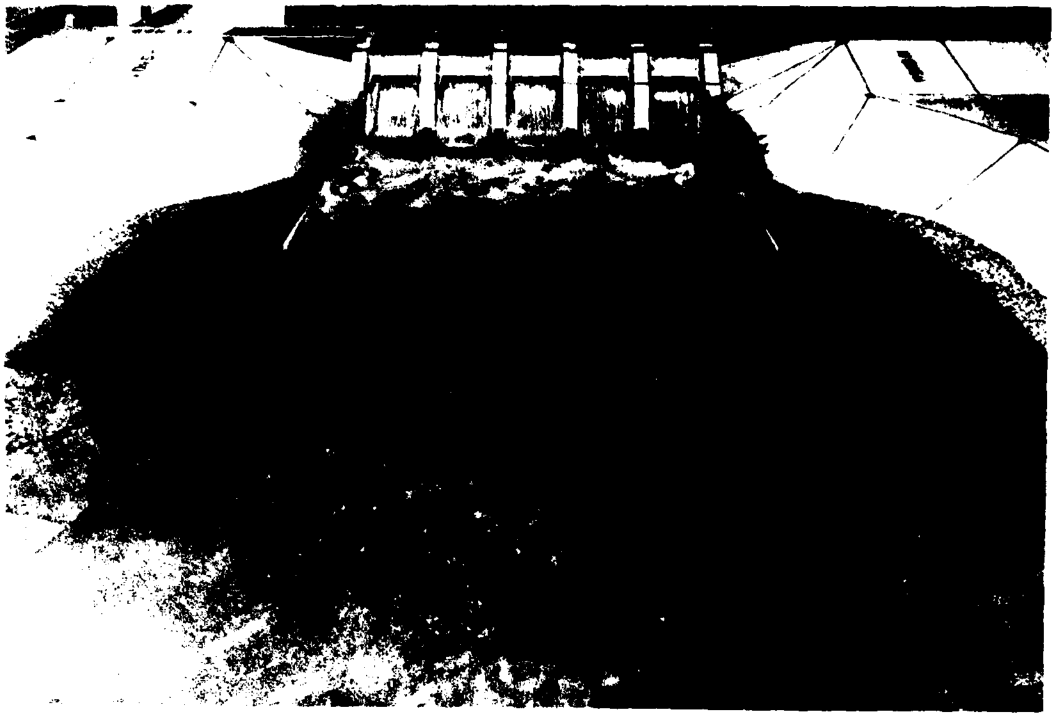
Photo 2. Type 3 approach wing wall

a. Gate opening 4 ft, discharge 16,000 cfs, tailwater el 402.0

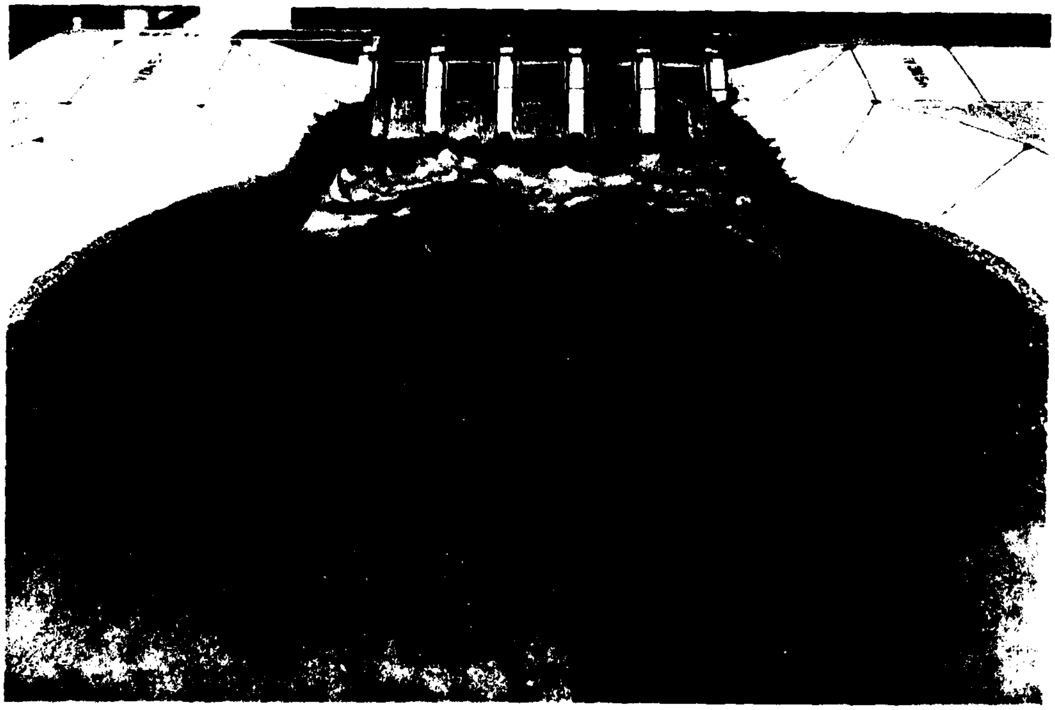


b. Gate opening 6 ft, discharge 23,000 cfs, tailwater el 403.5

Photo 3. Looking upstream at flow conditions in the type 1 stilling basin; discharges 16,000, 23,000, 30,000, and 40,000 cfs; upper pool el 440.0 (Sheet 1 of 2)



c. Gate opening 8 ft, discharge 30,000 cfs, tailwater el 404.5



d. All gates fully open, discharge 40,000 cfs, tailwater el 406.0

Photo 3. (Sheet 2 of 2)

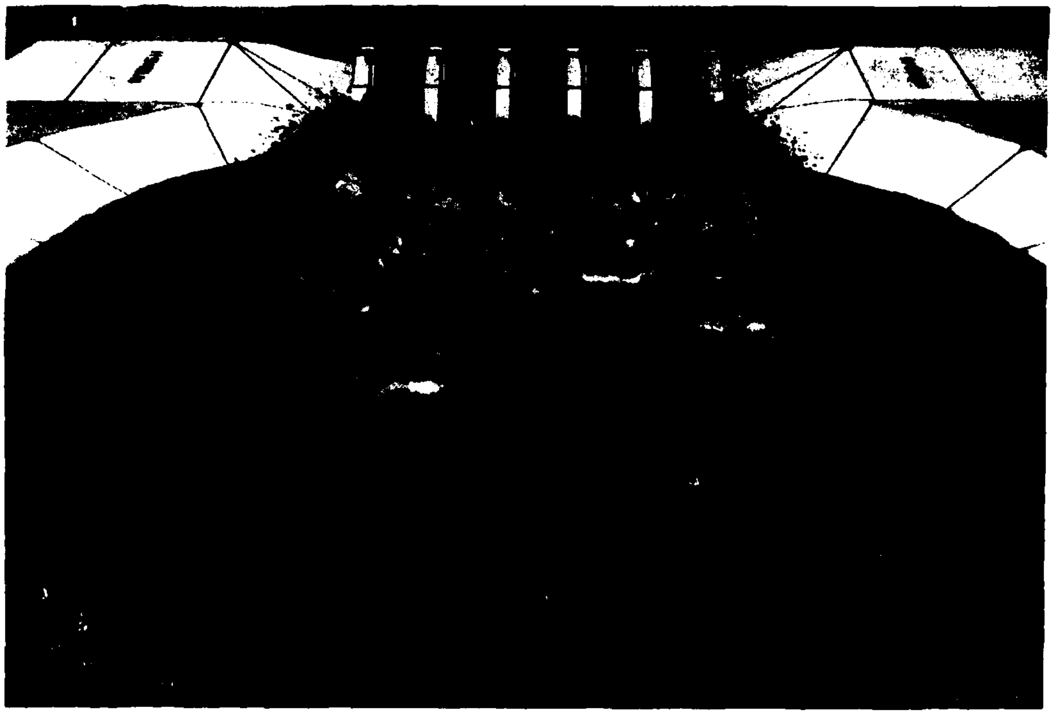
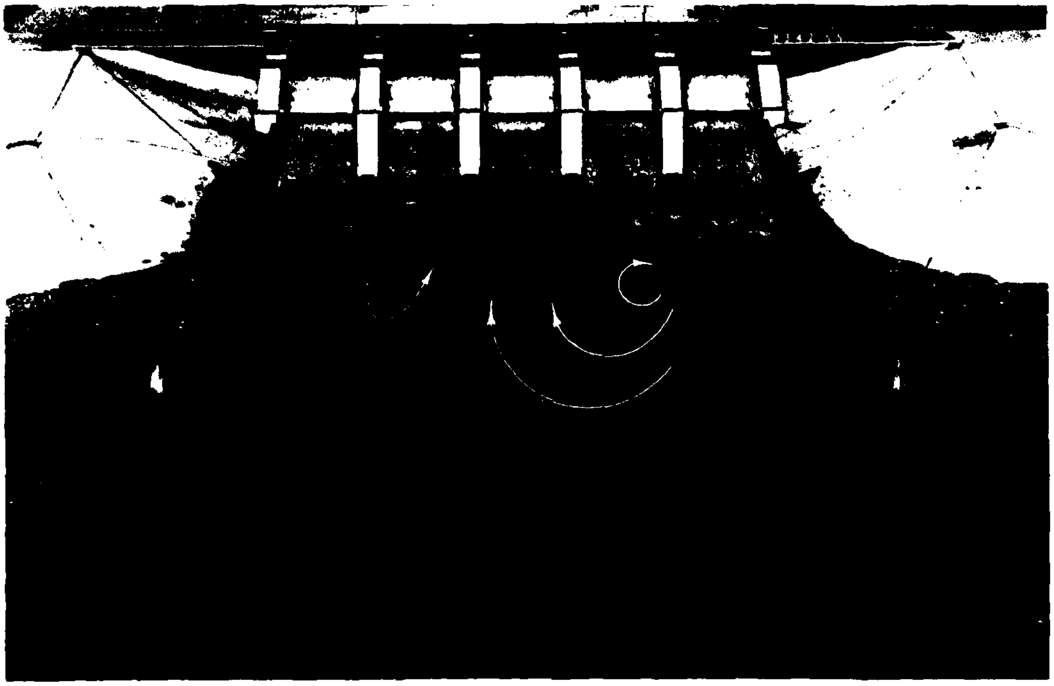


Photo 4. Looking upstream at flow conditions in the type 1 stilling basin. All gates fully open, discharge 110,000 cfs, upper pool el 452.3, tailwater el 410.5

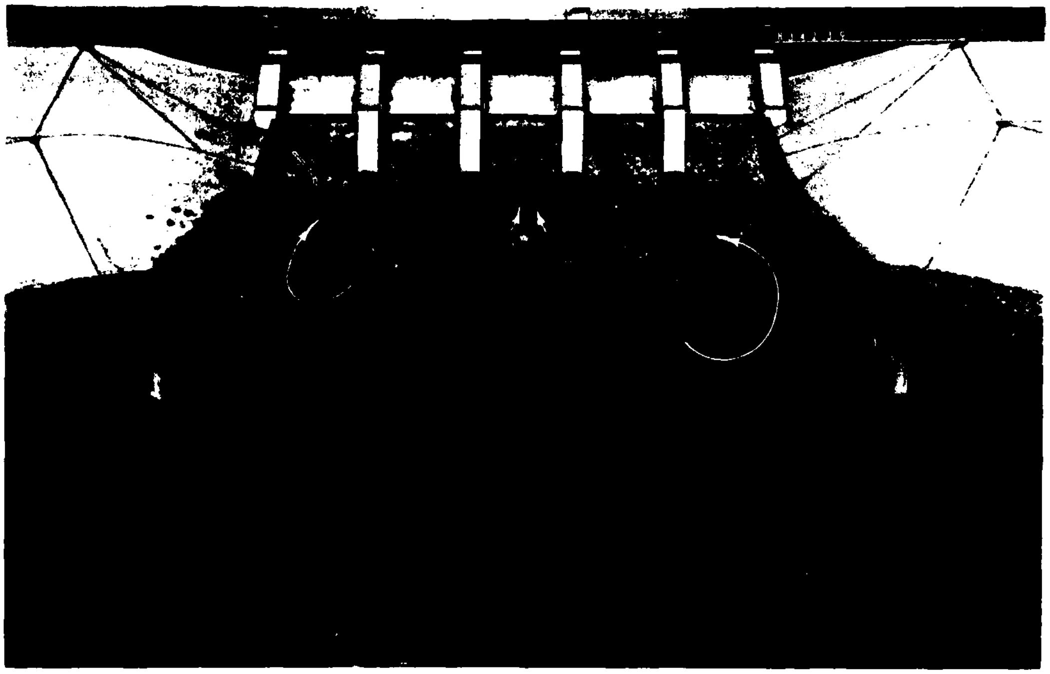
a. Gate opening 1 ft, discharge 6,000 cfs, tailwater el 399.2



b. Gate opening 2 ft, discharge 10,000 cfs, tailwater el 400.5

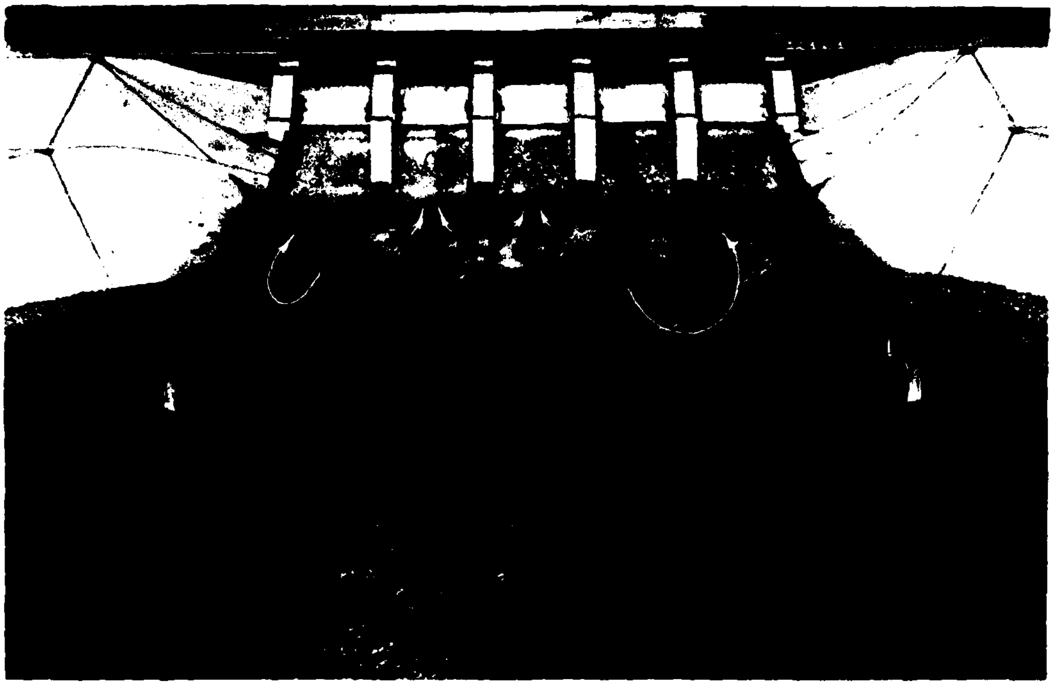
Photo 5. Looking upstream at flow conditions in the type 1 stilling basin; discharges 6,000 and 10,000 cfs; upper pool el 440.0

a. Discharge passing through sluice D; discharge 1,500 cfs, tailwater el 397.3

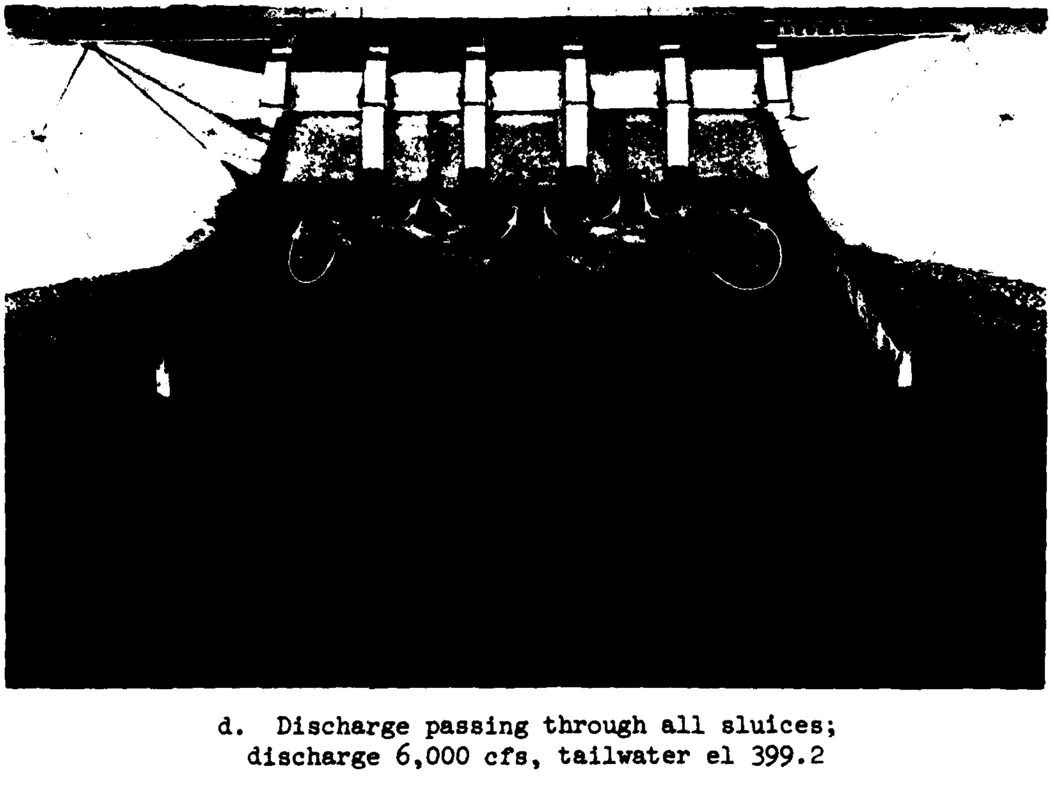


b. Discharge passing through sluices B and C; discharge 3,000 cfs, tailwater el 398.0

Photo 6. Looking upstream at flow conditions in the type 1 stilling basin; discharges 1,500, 3,000, 4,500, and 6,000 cfs; upper pool el 440.0 (Sheet 1 of 2)

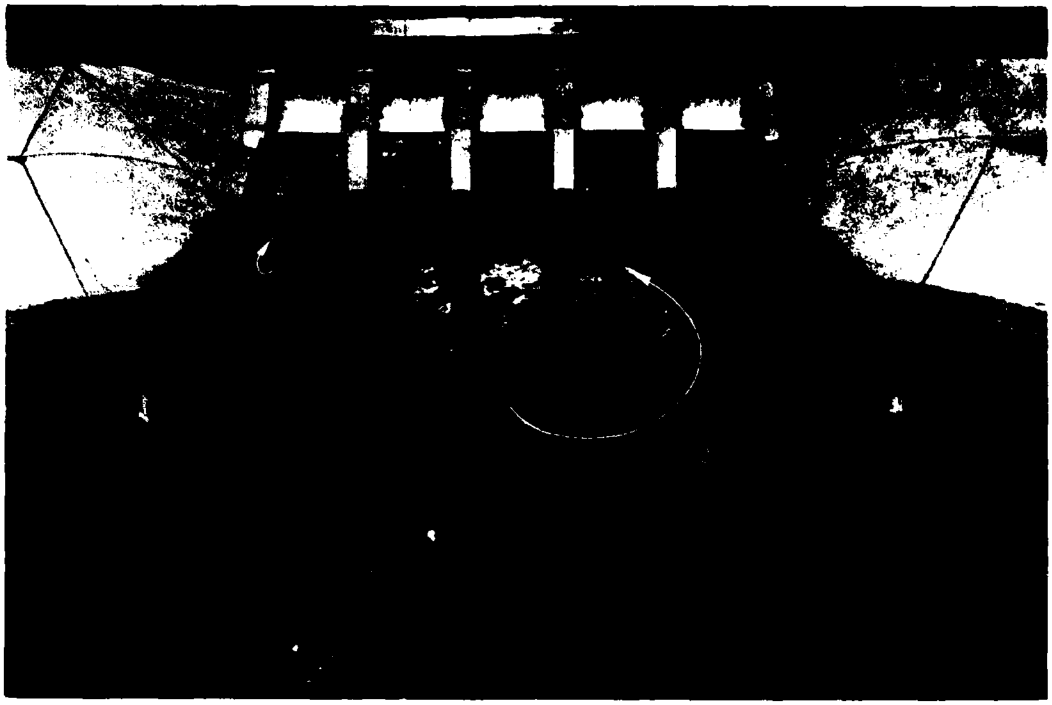


c. Discharge passing through sluices B, C, and D; discharge 4,500 cfs, tailwater el 398.5

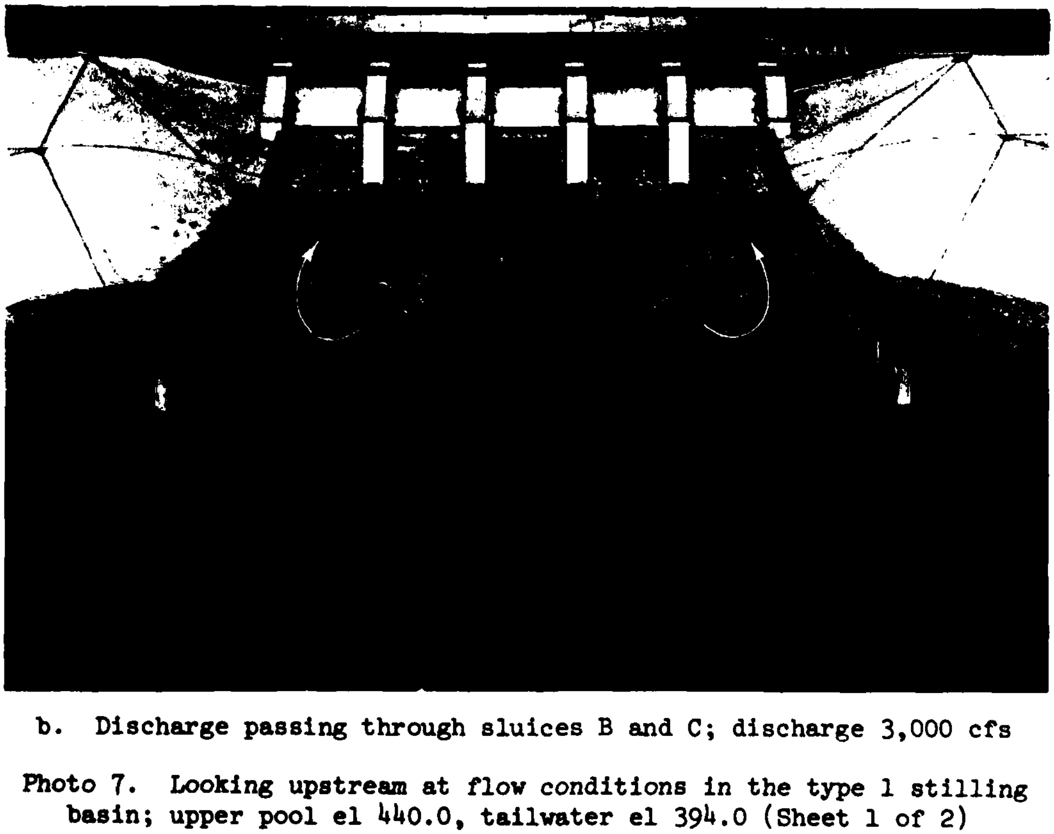


d. Discharge passing through all sluices; discharge 6,000 cfs, tailwater el 399.2

Photo 6. (Sheet 2 of 2)

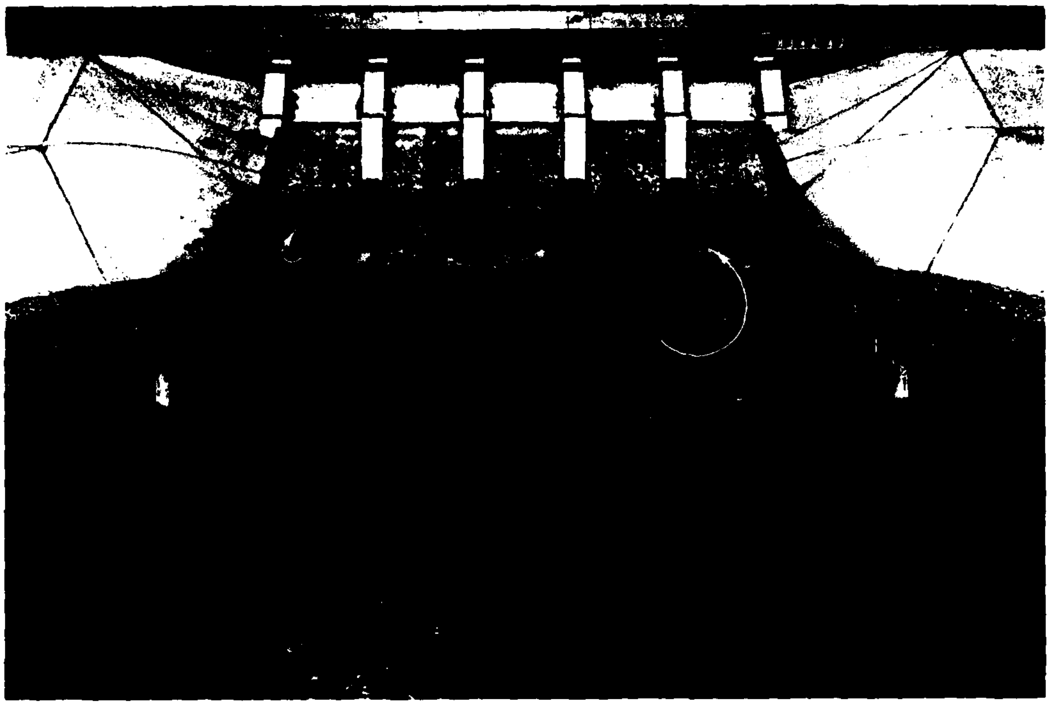


a. Discharge passing through sluice D; discharge 1,500 cfs

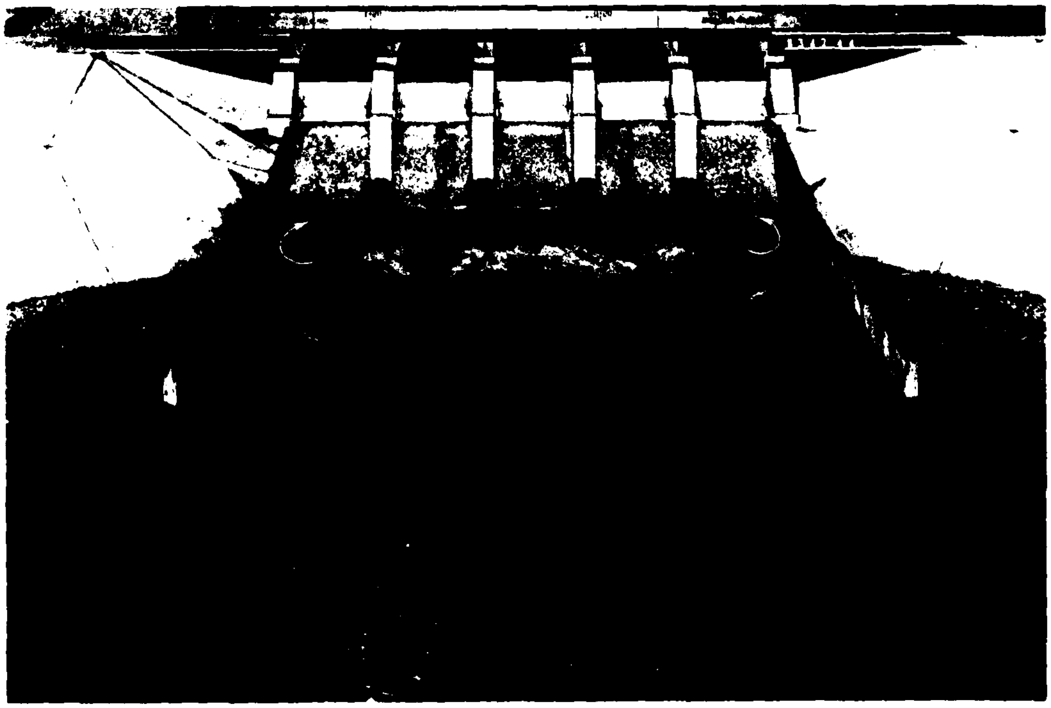


b. Discharge passing through sluices B and C; discharge 3,000 cfs

Photo 7. Looking upstream at flow conditions in the type 1 stilling basin; upper pool el 440.0, tailwater el 394.0 (Sheet 1 of 2)

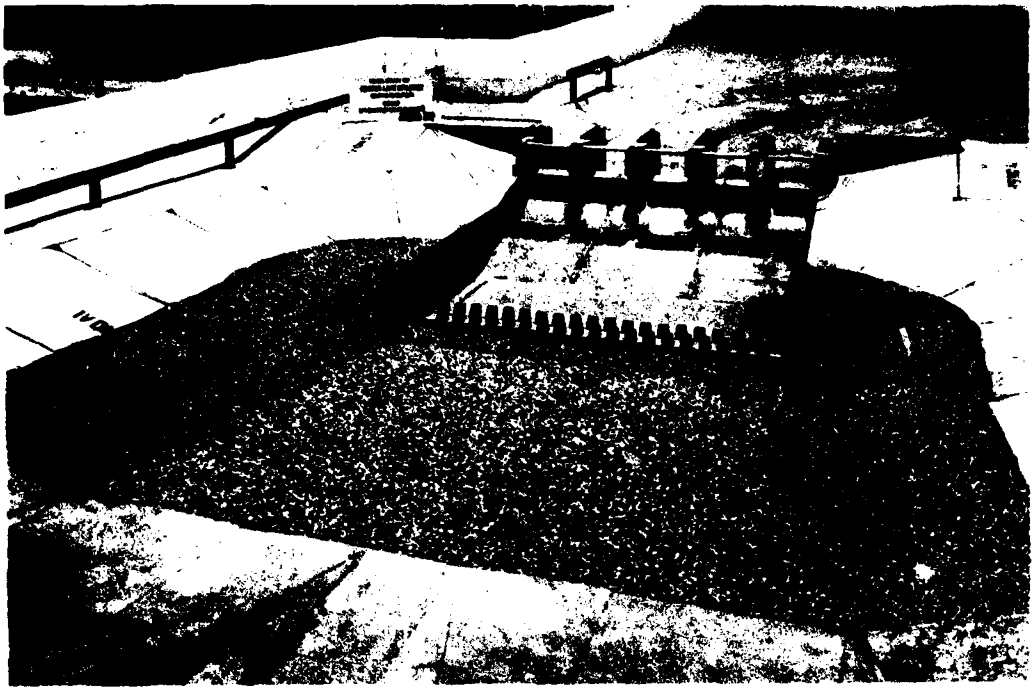


c. Discharge passing through sluices B, C, and D; discharge 4,500 cfs

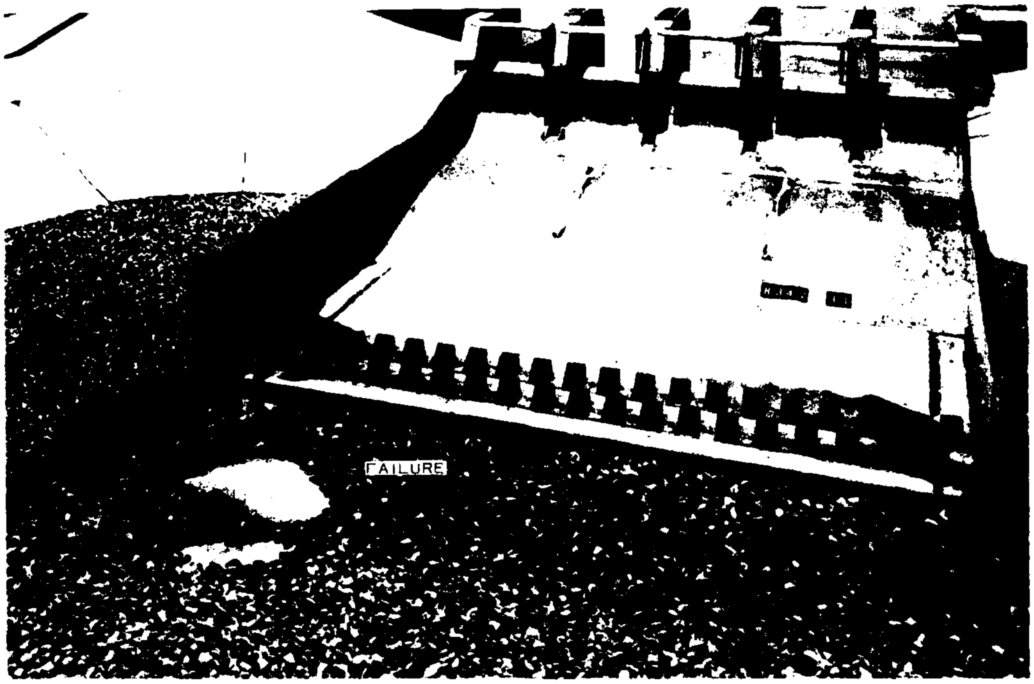


d. Discharge passing through all sluices; discharge 6,000 cfs

Photo 7. (Sheet 2 of 2)



a. Dry bed



b. Dry bed showing failure after a 36-hr (prototype) test with discharge 110,000 cfs, upper pool el 452.3, and tailwater el 410.5

Photo 8. Type 2 riprap protection plan

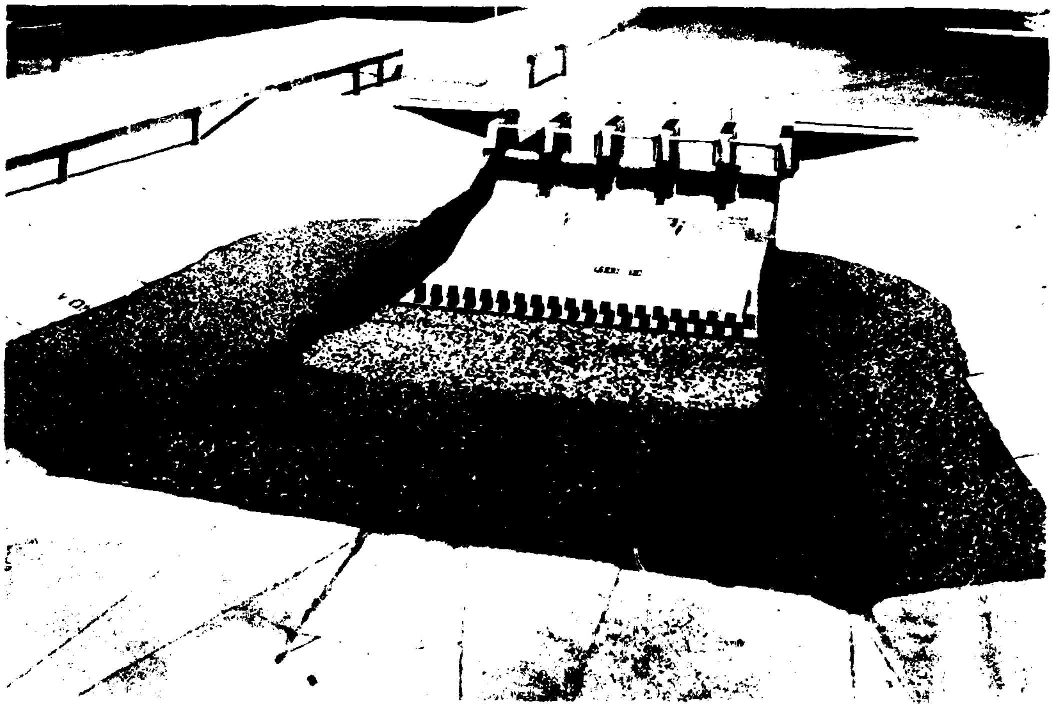
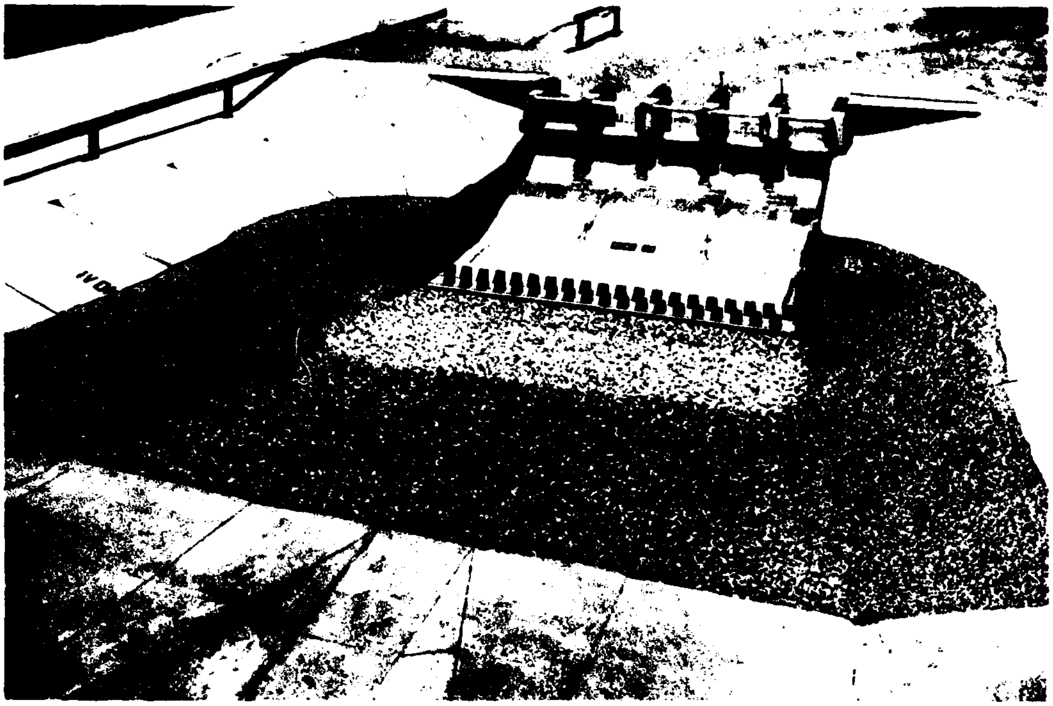
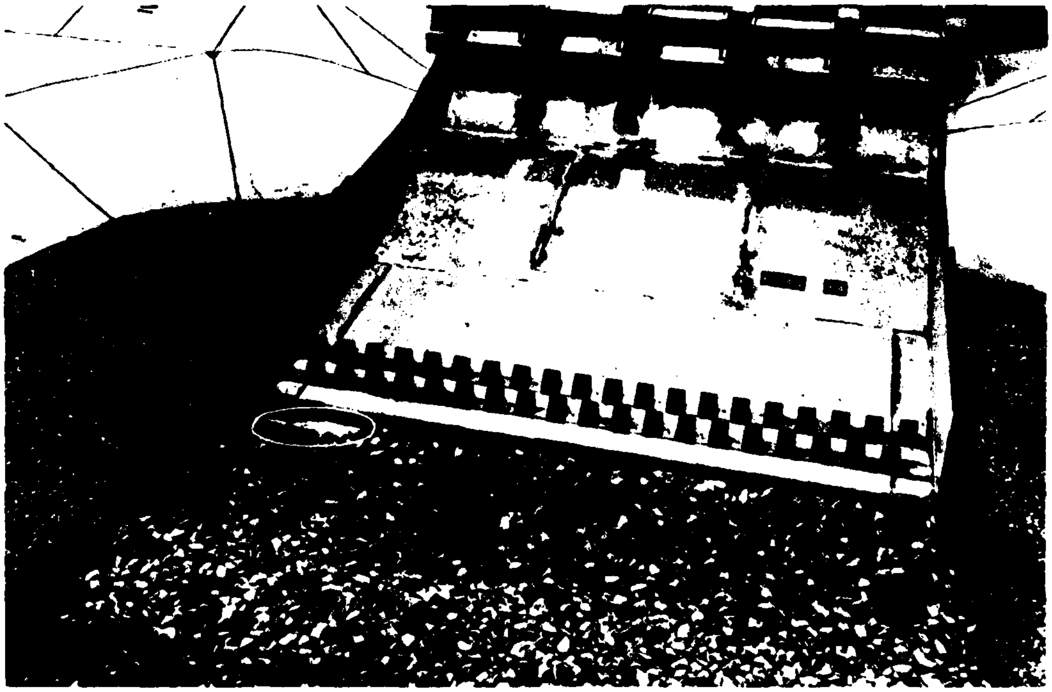


Photo 9. Type 3 riprap protection plan



a. Dry bed



b. Dry bed showing failure after a 48-hr (prototype) test with discharge 110,000 cfs, upper pool el 452.3, and tailwater el 410.5

Photo 10. Type 4 riprap protection plan

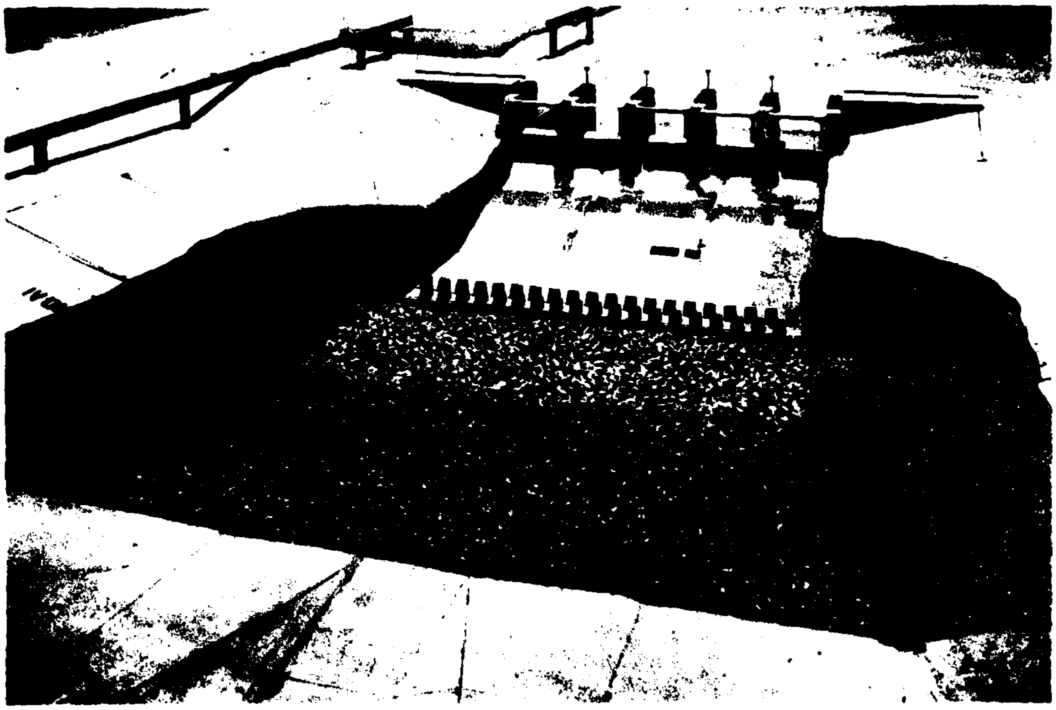


Photo 11. Type 5 riprap protection plan

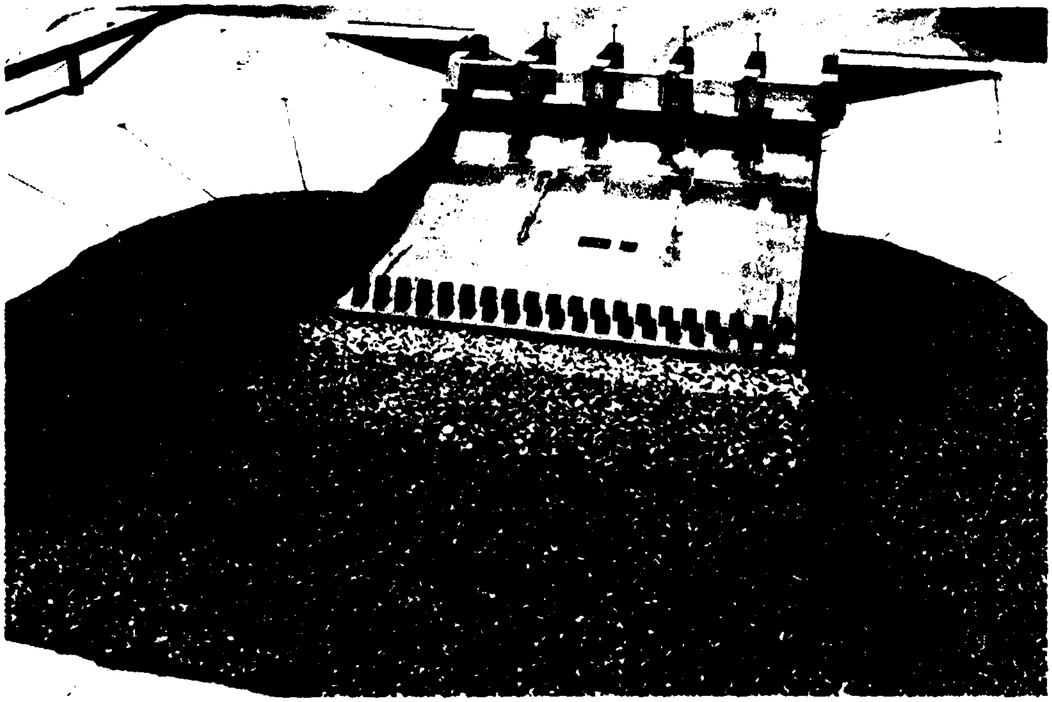


Photo 12. Type 6 riprap protection plan

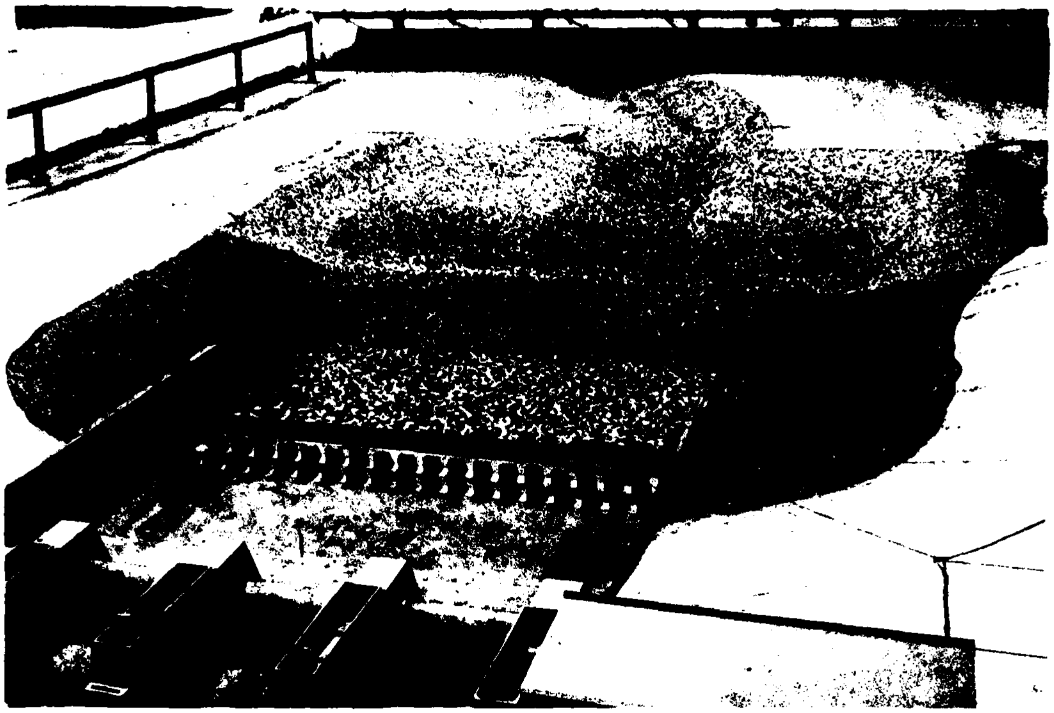


Photo 13. Type 7 riprap protection plan

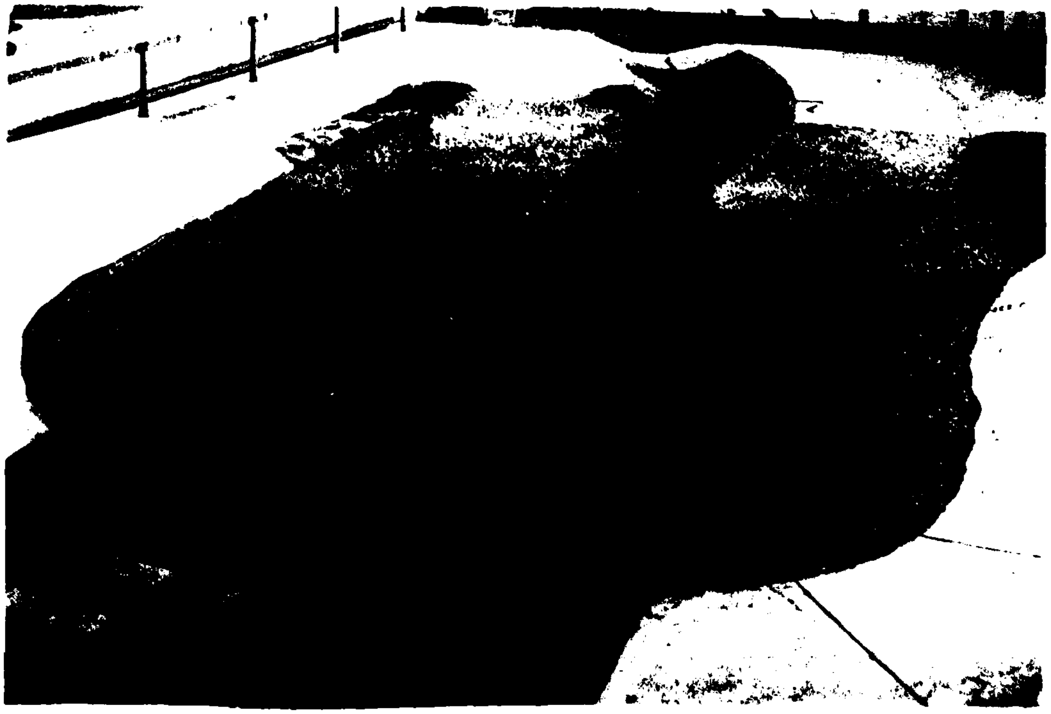


Photo 14. Type 8 riprap protection plan

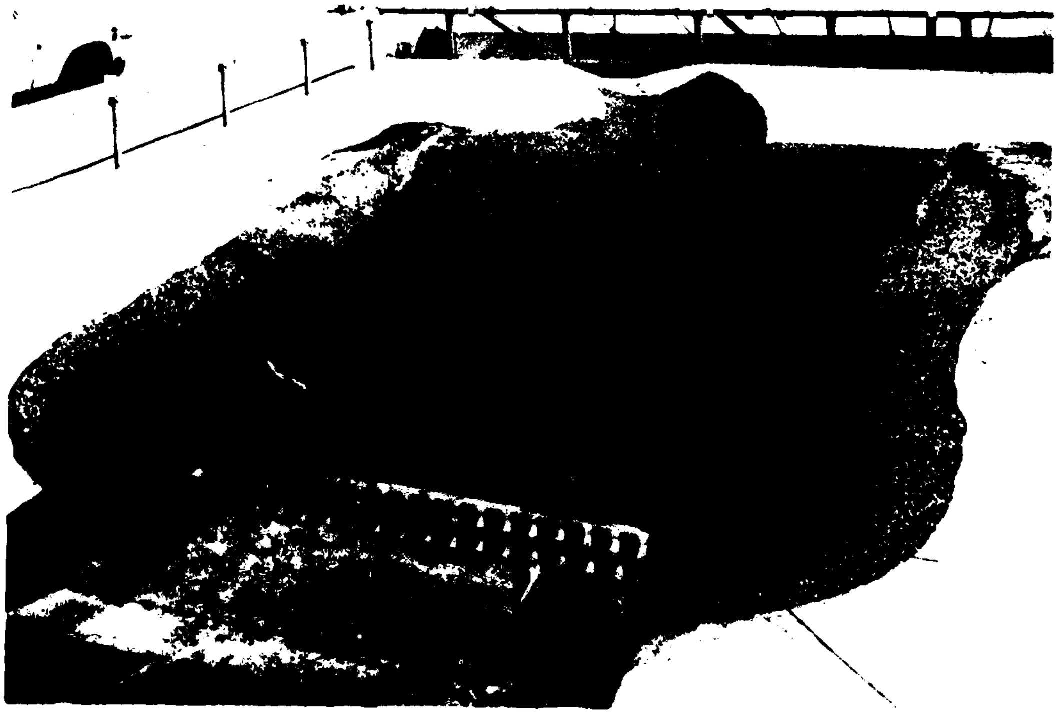
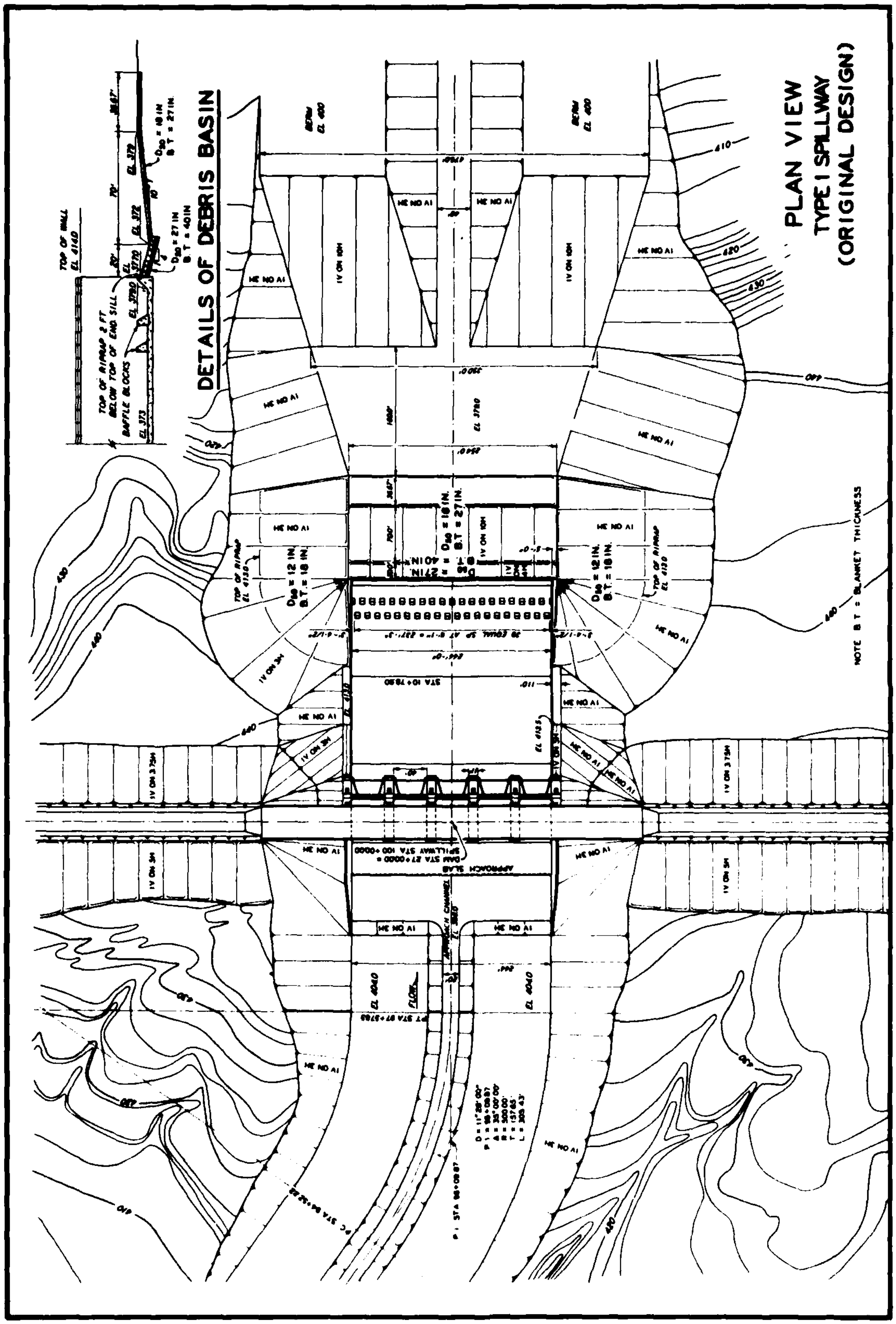


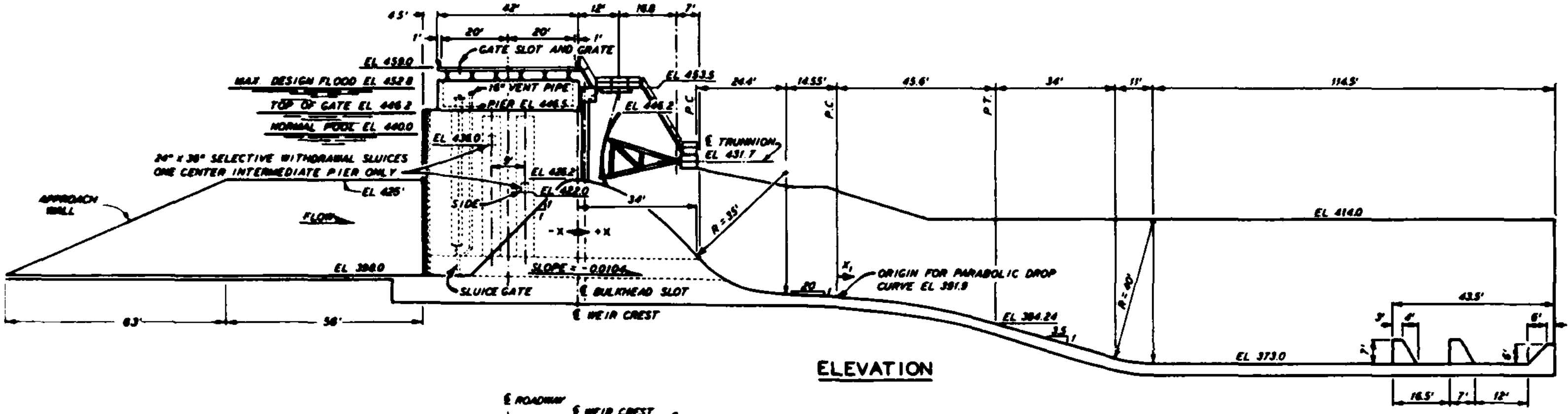
Photo 15. Type 9 riprap protection plan



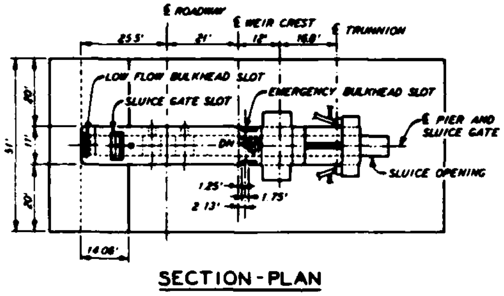
PLAN VIEW
TYPE 1 SPILLWAY
(ORIGINAL DESIGN)

PLATE 1

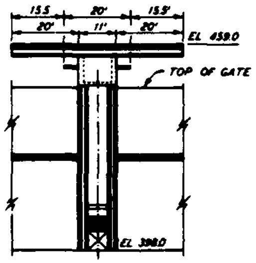
PLATE 2



ELEVATION



SECTION - PLAN



UPSTREAM ELEVATION

WEIR PROFILE COORDINATES

-X	EL	X	EL
5.33'	425.00	3'	425.89
4.75	425.47	4	425.68
3.17	425.88	7	424.82
2.08'	426.03	12	422.67
1	426.16	20	417.55
0	426.28	32.50'	405.00
1	426.15	34	404.33
2	426.05		

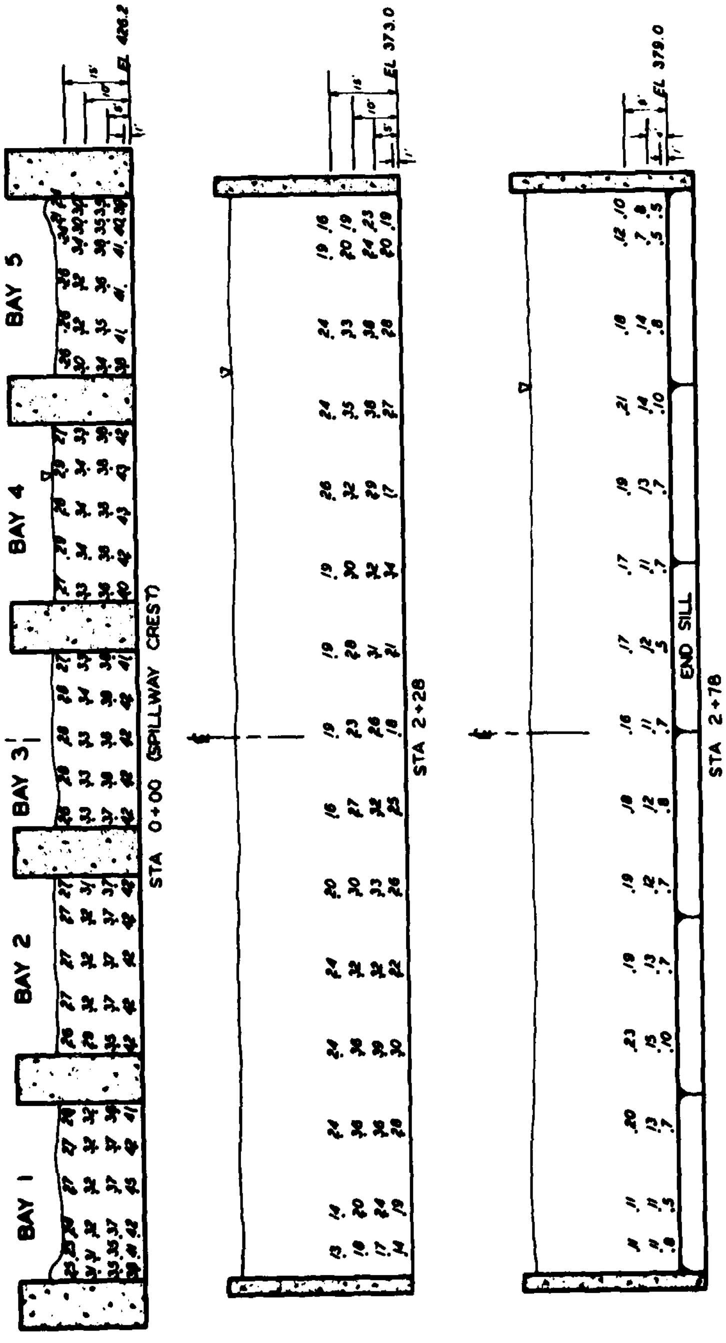
DOWNSTREAM QUADRANT Y = 0.0[illegible]X 1.85

PARABOLIC DROP COORDINATES

X_1	EL	X_1	EL
0	391.90	20'	389.87
2'	391.79	24'	389.21
4'	391.66	30'	388.07
6'	391.51	35'	386.98
8'	391.33	40'	385.76
10'	391.14	45.65'	384.23
14'	390.69		

$(Y = -(0.05X + 0.002587X^2))$

CENTER-LINE PROFILE
ORIGINAL SPILLWAY DESIGN



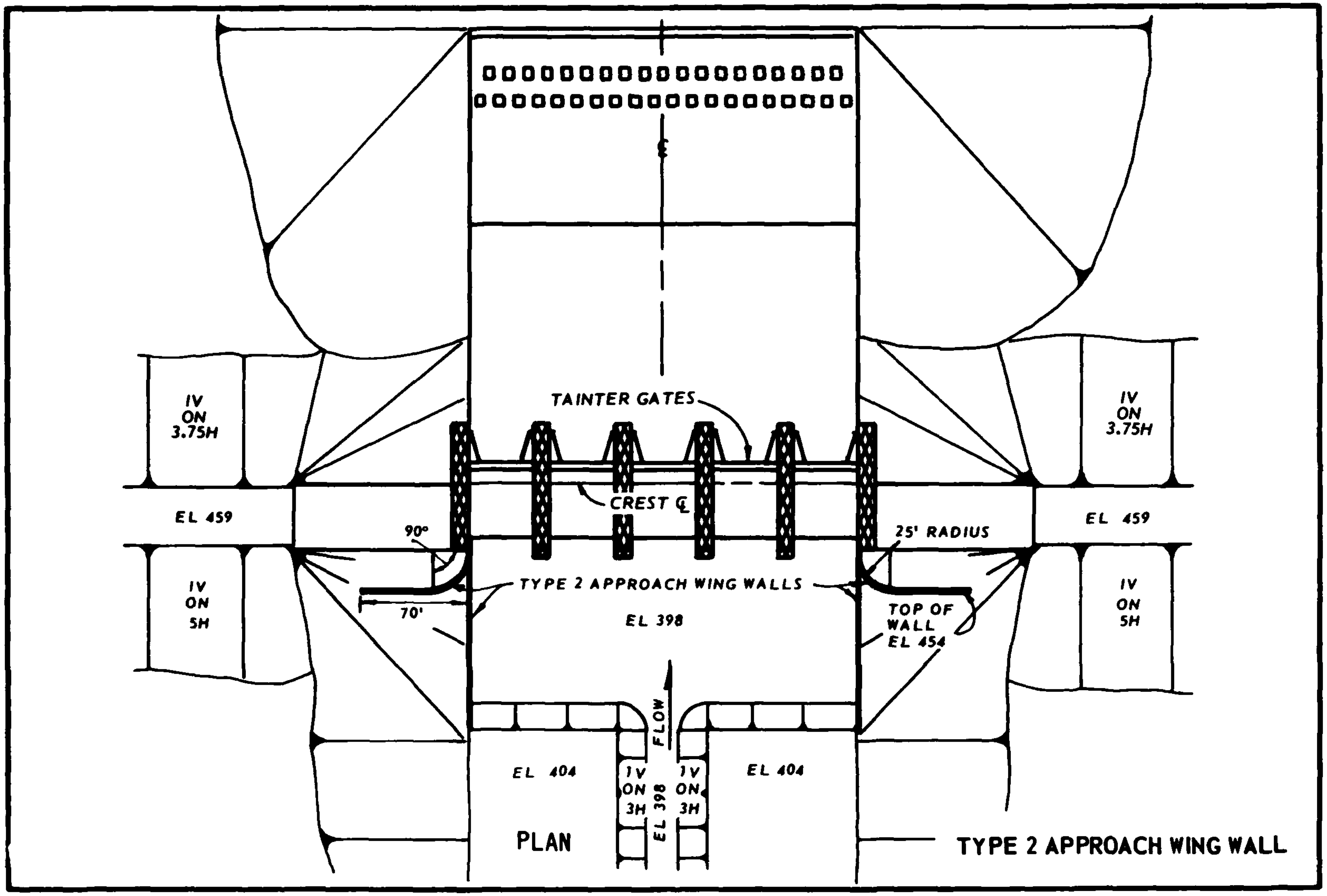
MAXIMUM VELOCITIES
ORIGINAL SPILLWAY DESIGN

DISCHARGE 110,000 CFS
UPPER POOL EL 452.5
TAILWATER EL 410.5

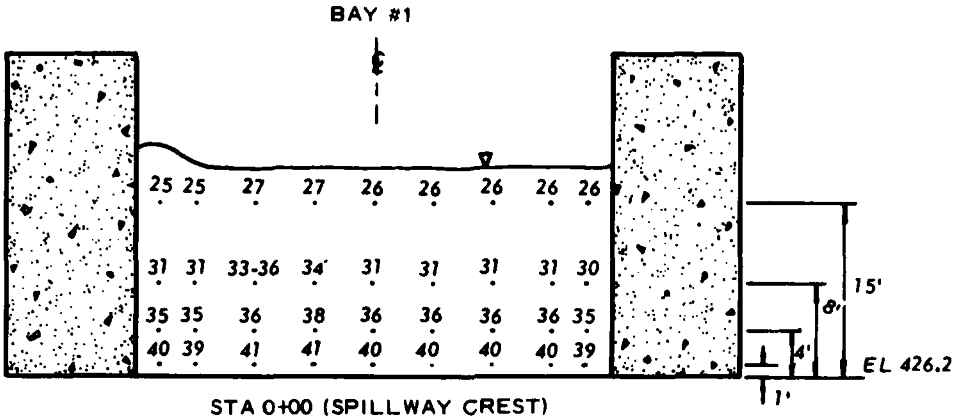
NOTE: VELOCITIES ARE IN PROTOTYPE FEET PER SECOND. LOOKING DOWNSTREAM.

PLATE 3

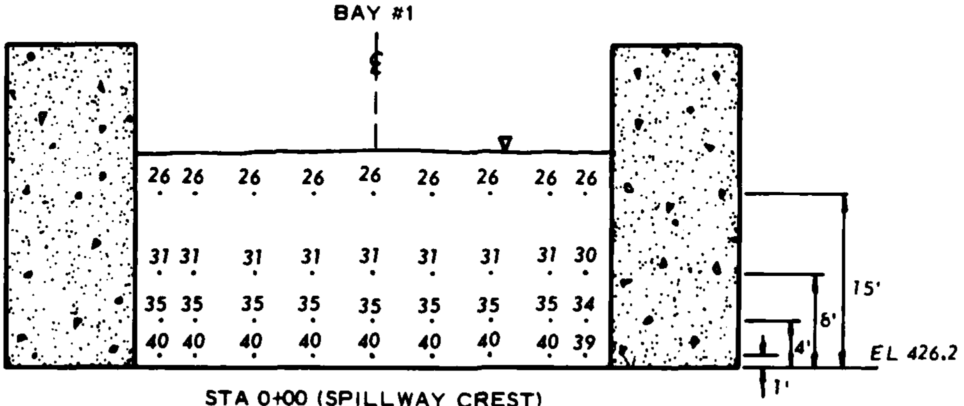
PLATE 4



TYPE 2 APPROACH WING WALL



STA 0+00 (SPILLWAY CREST)
TYPE 2 APPROACH WING WALL

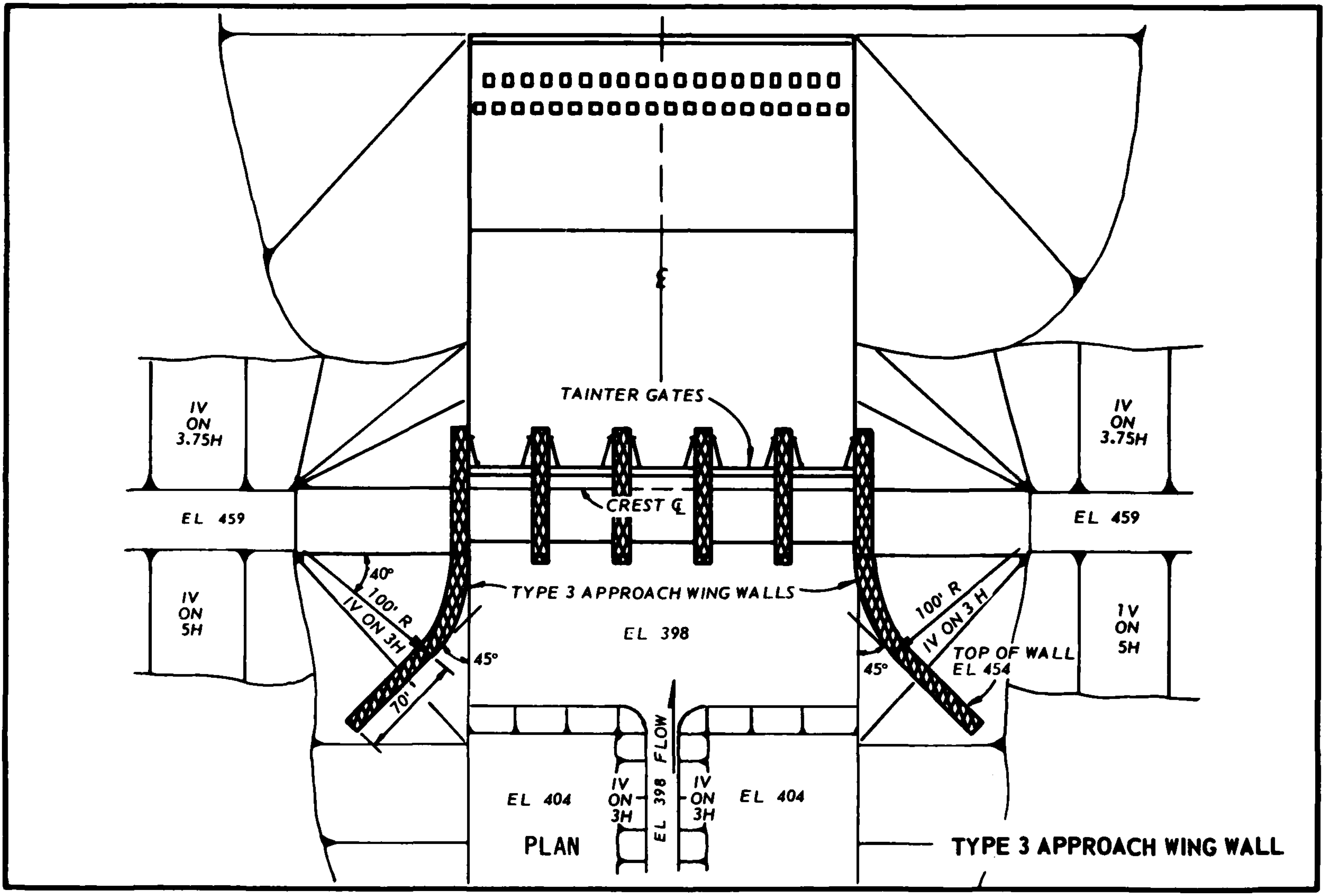


STA 0+00 (SPILLWAY CREST)
TYPE 3 APPROACH WING WALL

MAXIMUM VELOCITIES
TYPES 2 AND 3 APPROACH WING WALLS

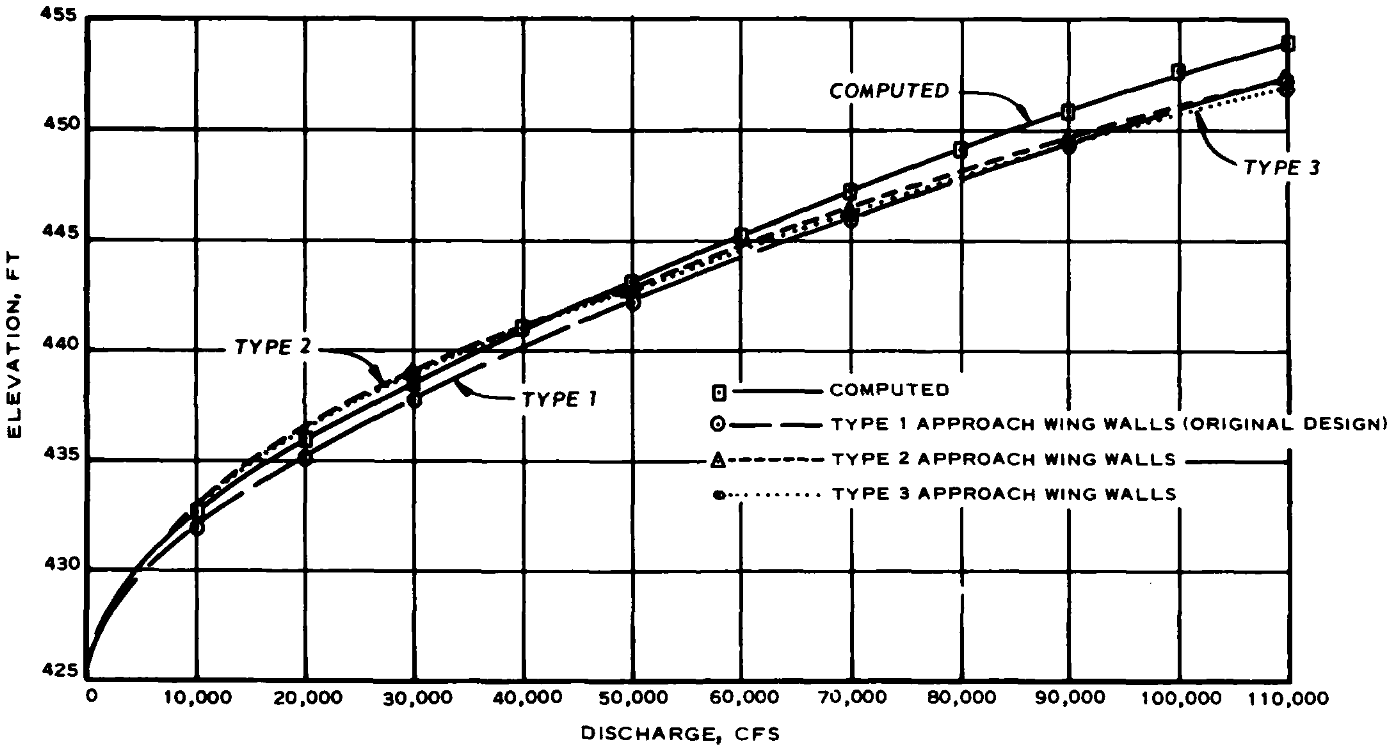
DISCHARGE	110,000 CFS
UPPER POOL EL	452.2
TAILWATER EL	410.6

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

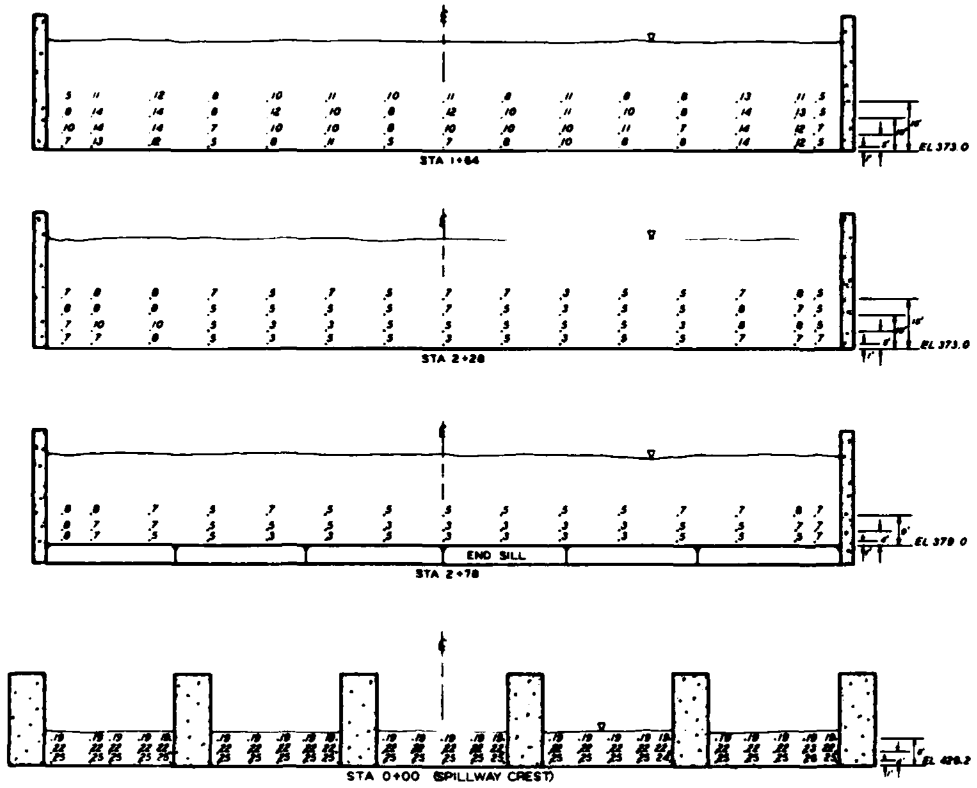


TYPE 3 APPROACH WING WALL

PLATE 6



COMPARISON OF
DISCHARGE RATING CURVES
WITH ALL GATES FULLY OPEN

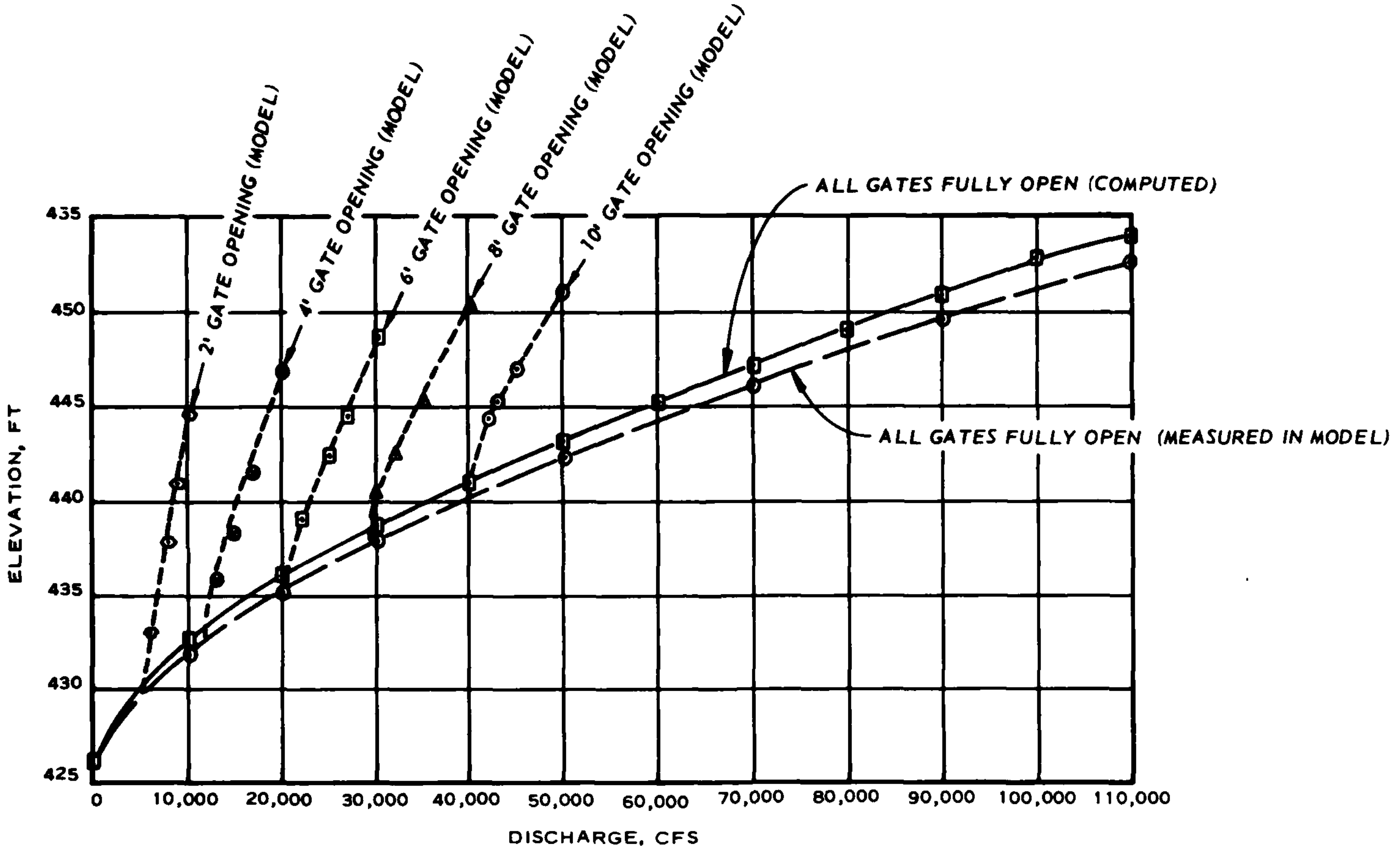


MAXIMUM VELOCITIES

ORIGINAL SPILLWAY DESIGN
DISCHARGE 40,000 CFS
UPPER POOL EL 440.0
TAILWATER EL 406.0

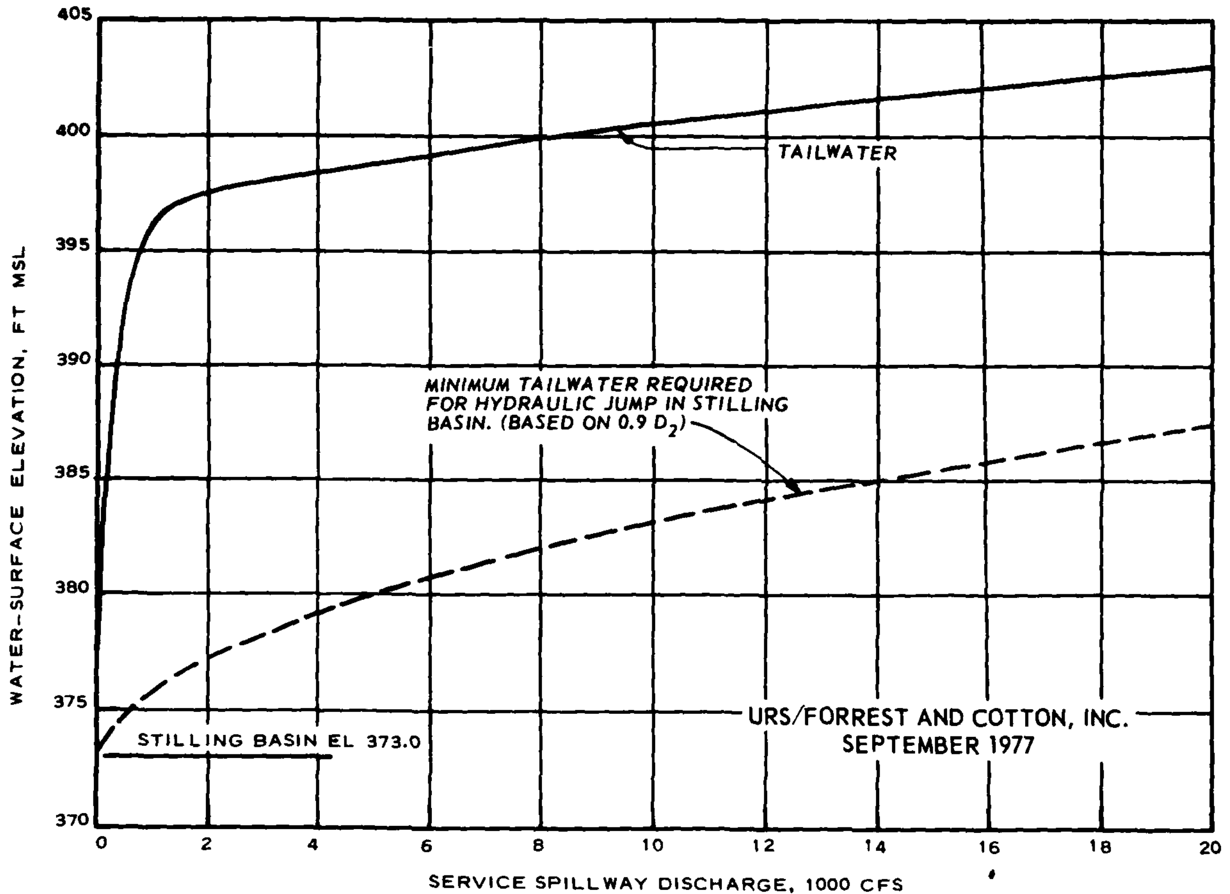
NOTE VELOCITIES ARE IN PROTOTYPE FEET PER SECOND
LOOKING DOWNSTREAM

PLATE 8



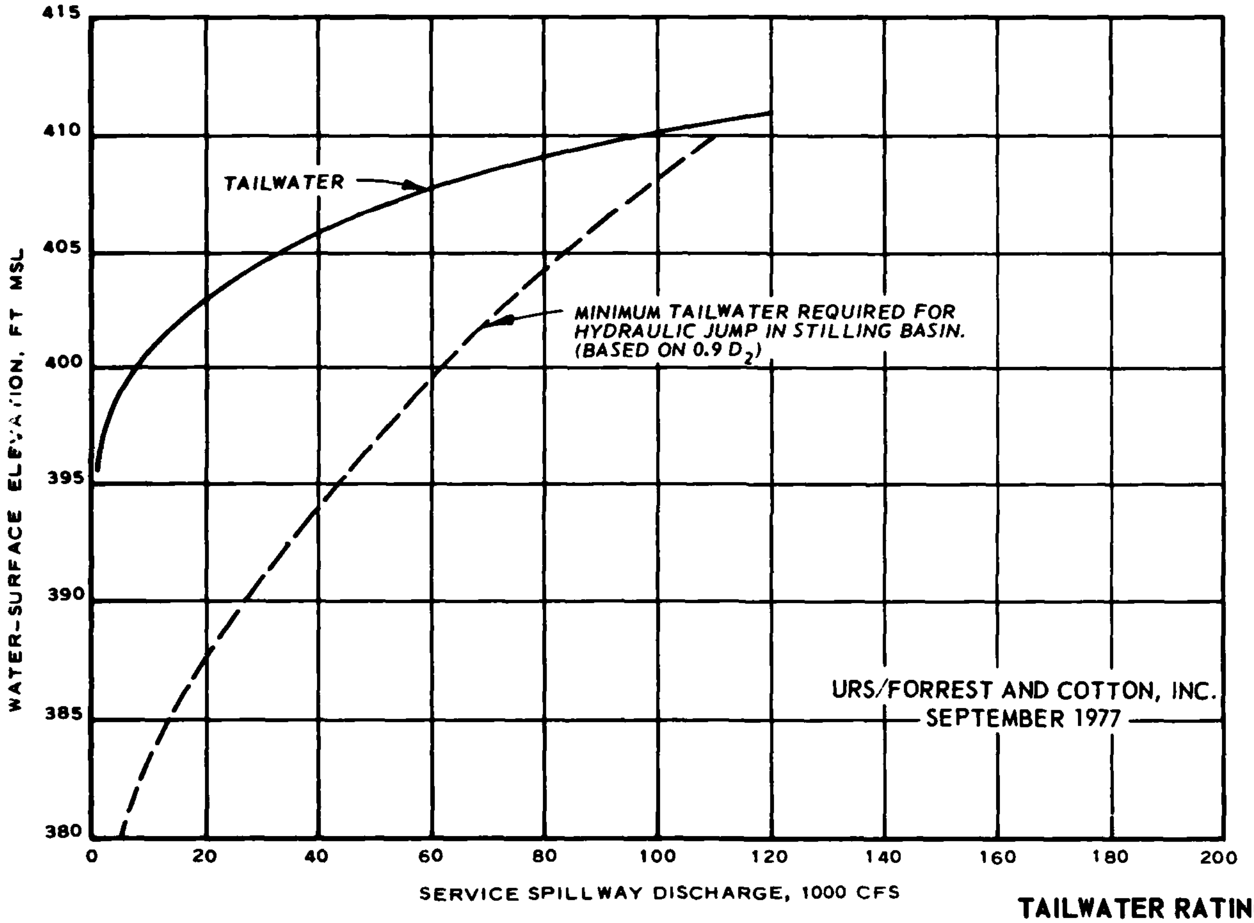
DISCHARGE RATING CURVES

PLATE 10



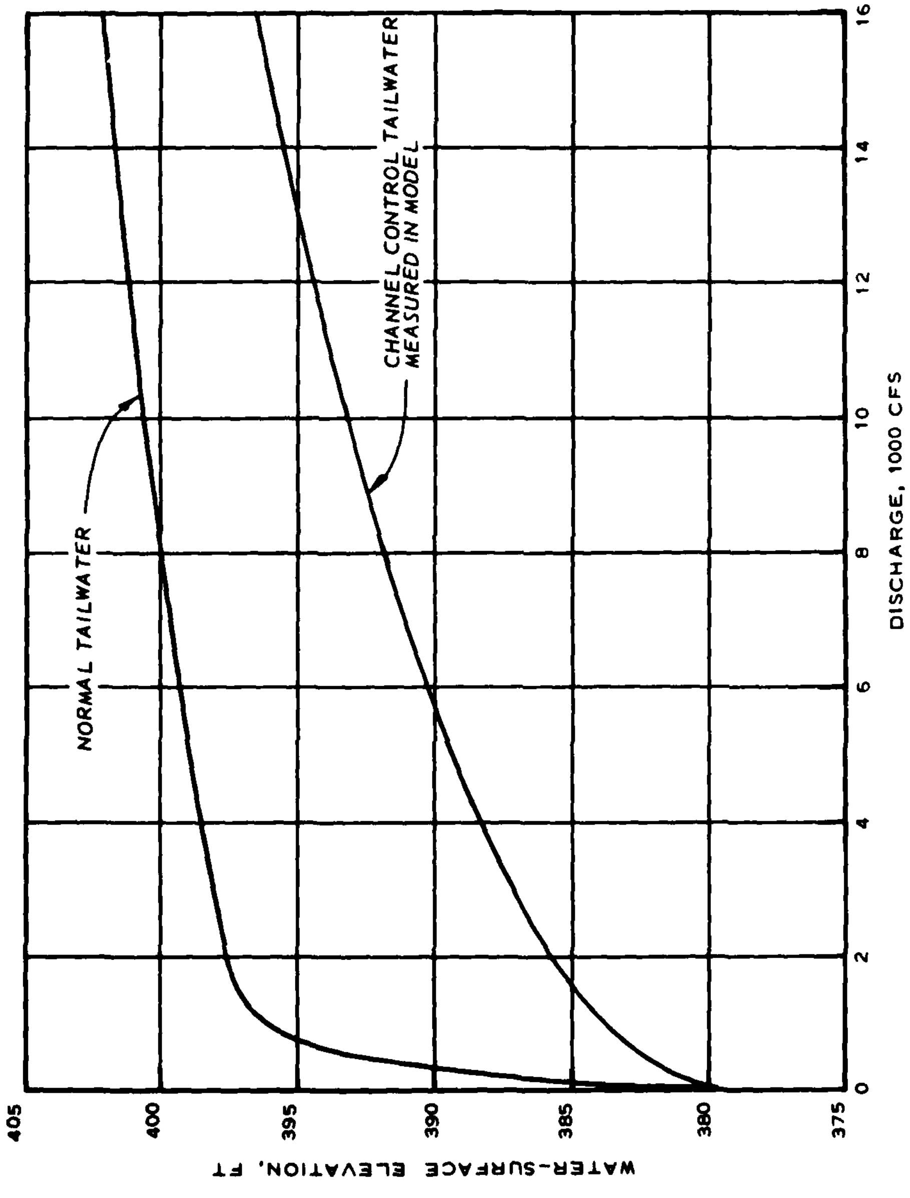
URS/FORREST AND COTTON, INC.
SEPTEMBER 1977

TAILWATER RATING CURVE
AT SERVICE SPILLWAY STILLING BASIN
0 TO 20,000 CFS



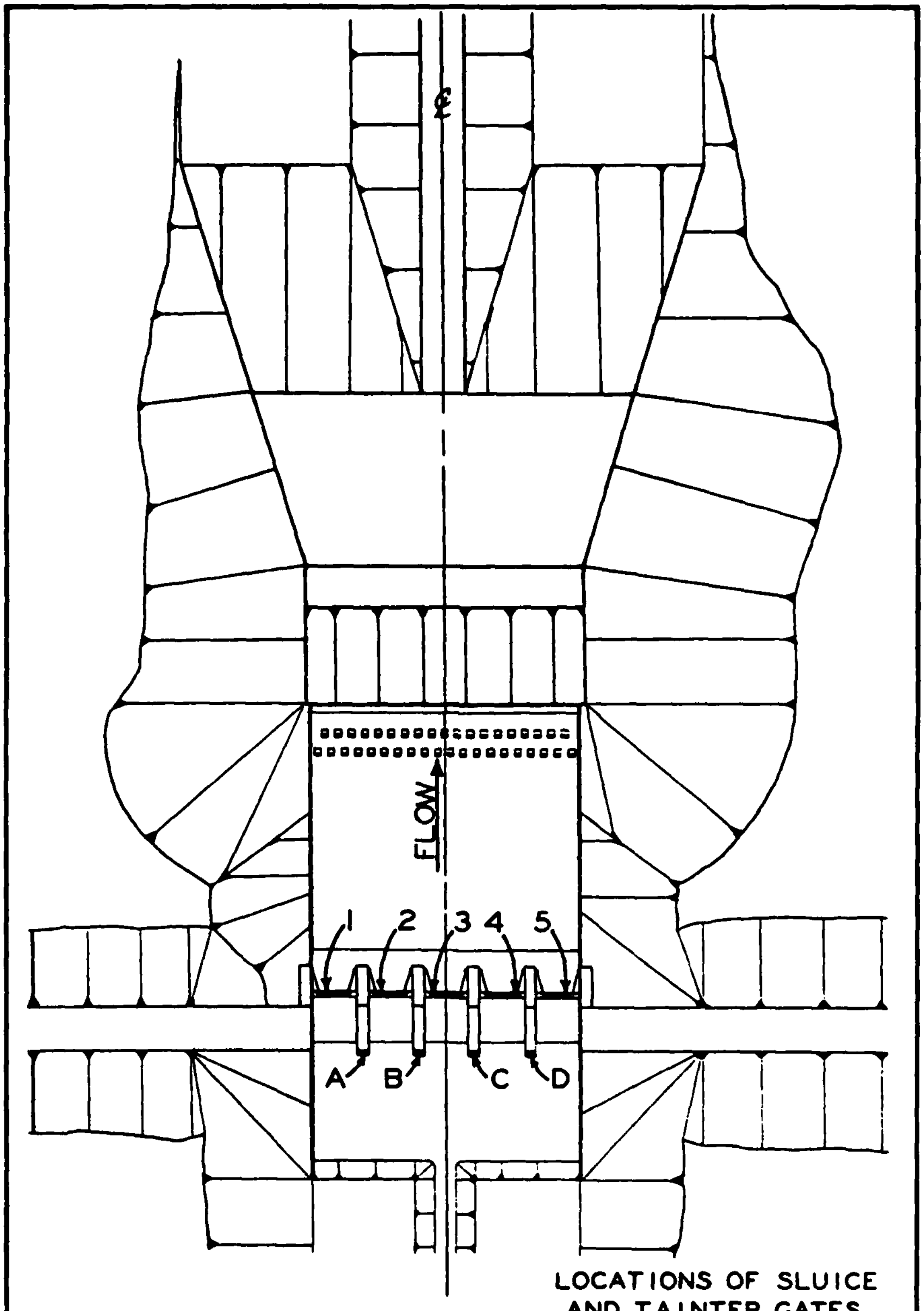
URS/FORREST AND COTTON, INC.
SEPTEMBER 1977

TAILWATER RATING CURVE
AT SERVICE SPILLWAY STILLING BASIN
1,000 TO 120,000 CFS



TAILWATER RATING CURVES

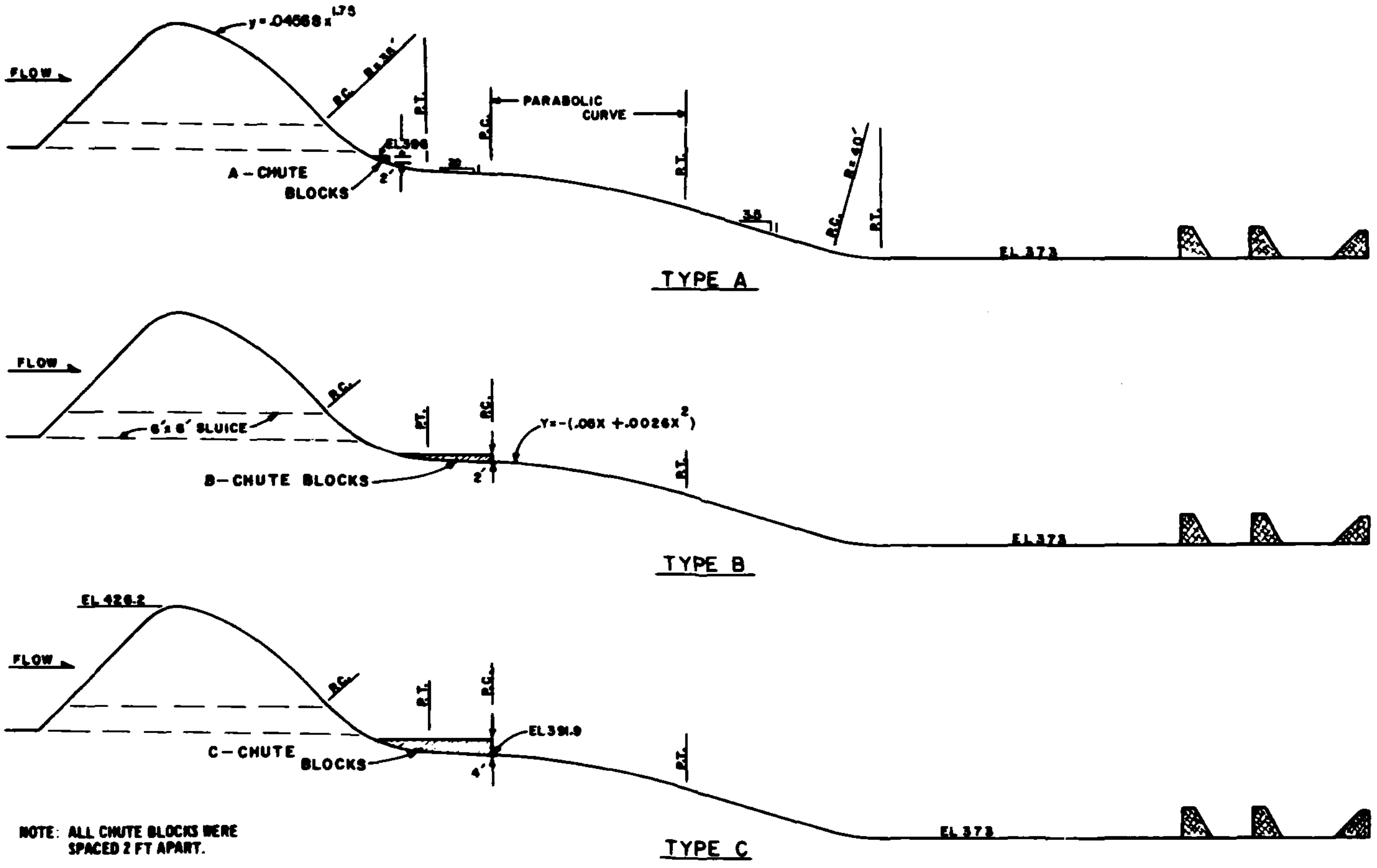
PLATE 12



LOCATIONS OF SLUICE AND TAINTER GATES

PLATE 13

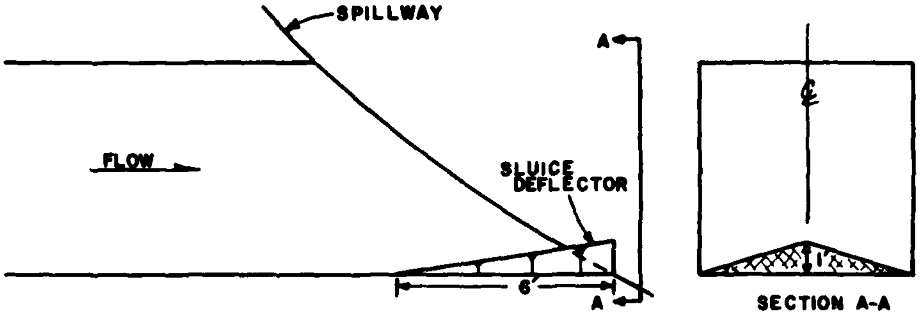
PLATE 14



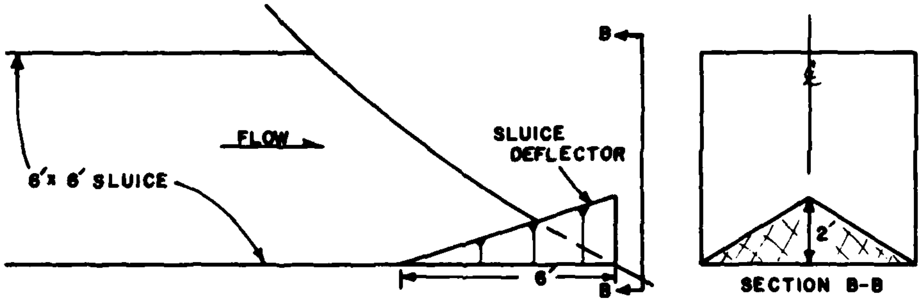
NOTE: ALL CHUTE BLOCKS WERE SPACED 2 FT APART.

ELEVATION

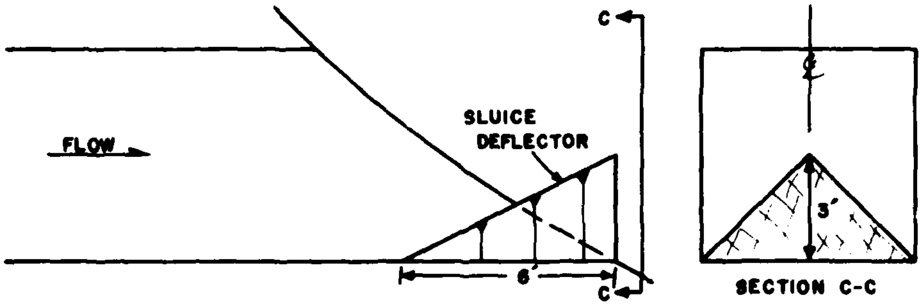
CHUTE BLOCKS
TYPES A, B, AND C



TYPE 1



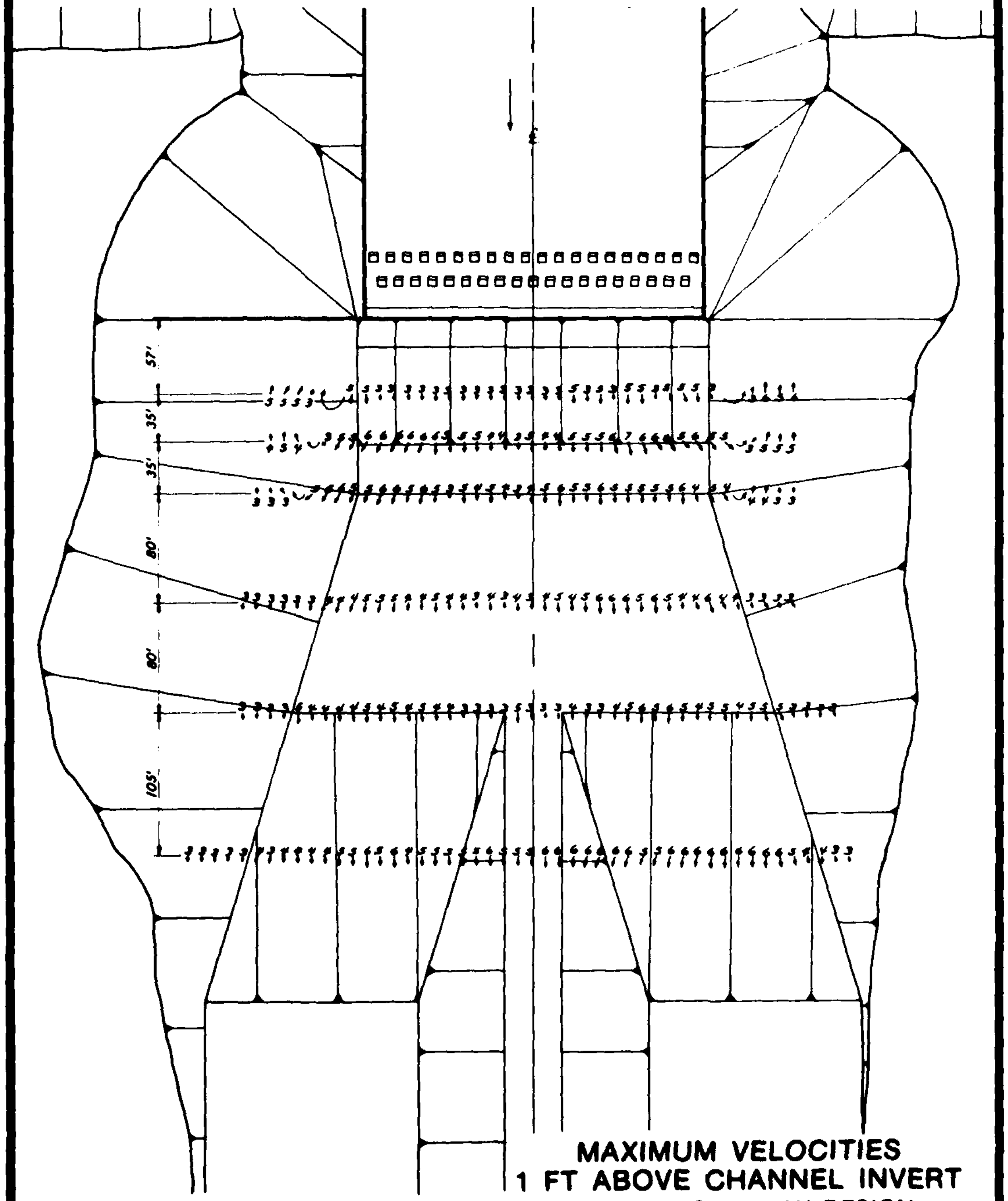
TYPE 2



TYPE 3

ELEVATION

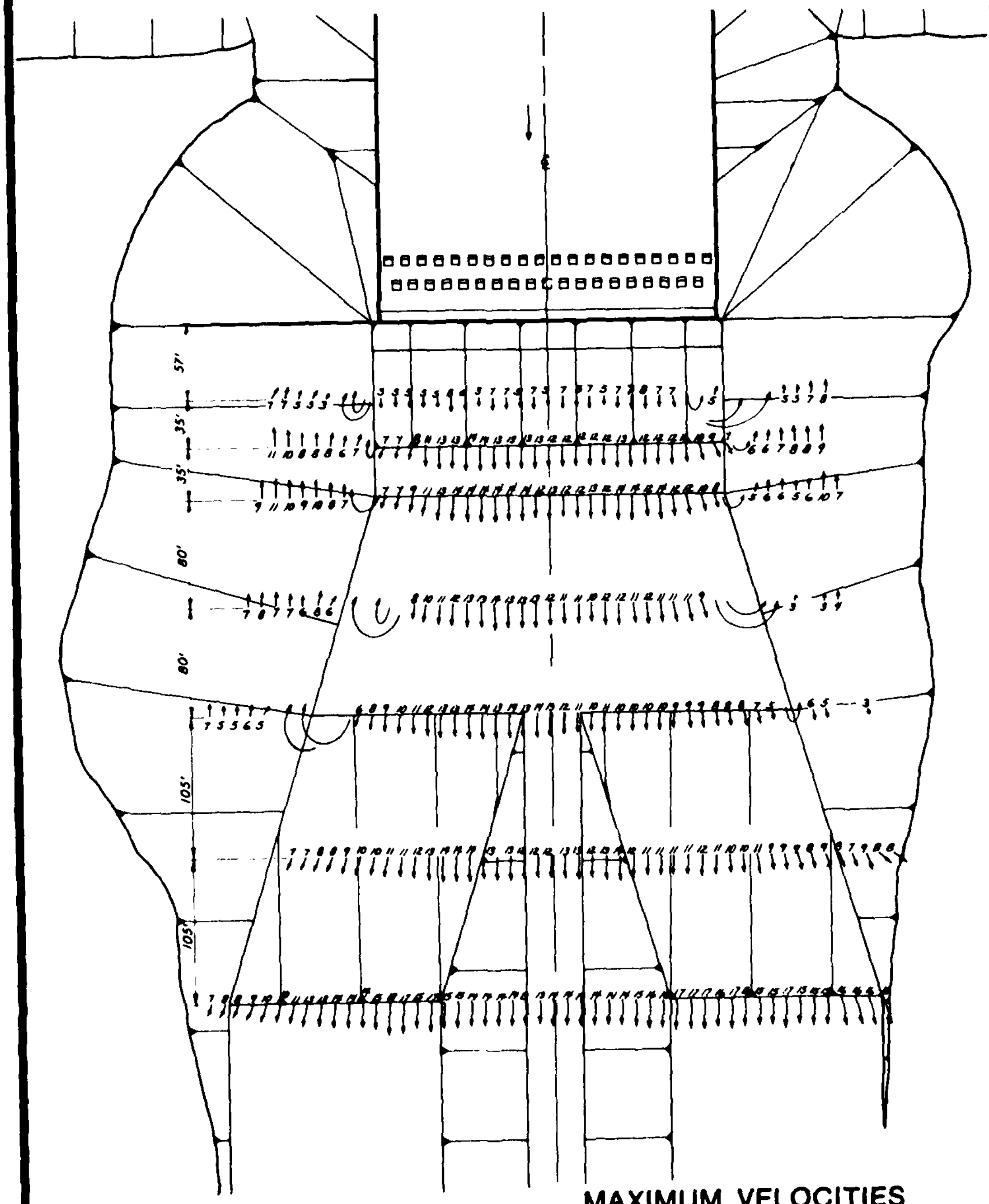
SLUICE DEFLECTORS
TYPES 1, 2, AND 3



MAXIMUM VELOCITIES
1 FT ABOVE CHANNEL INVERT
ORIGINAL SPILLWAY DESIGN

DISCHARGE 40,000 CFS
UPPER POOL EL 440.0
TAILWATER EL 406.0

NOTE: VELOCITIES ARE IN PROTOTYPE FEET PER SECOND

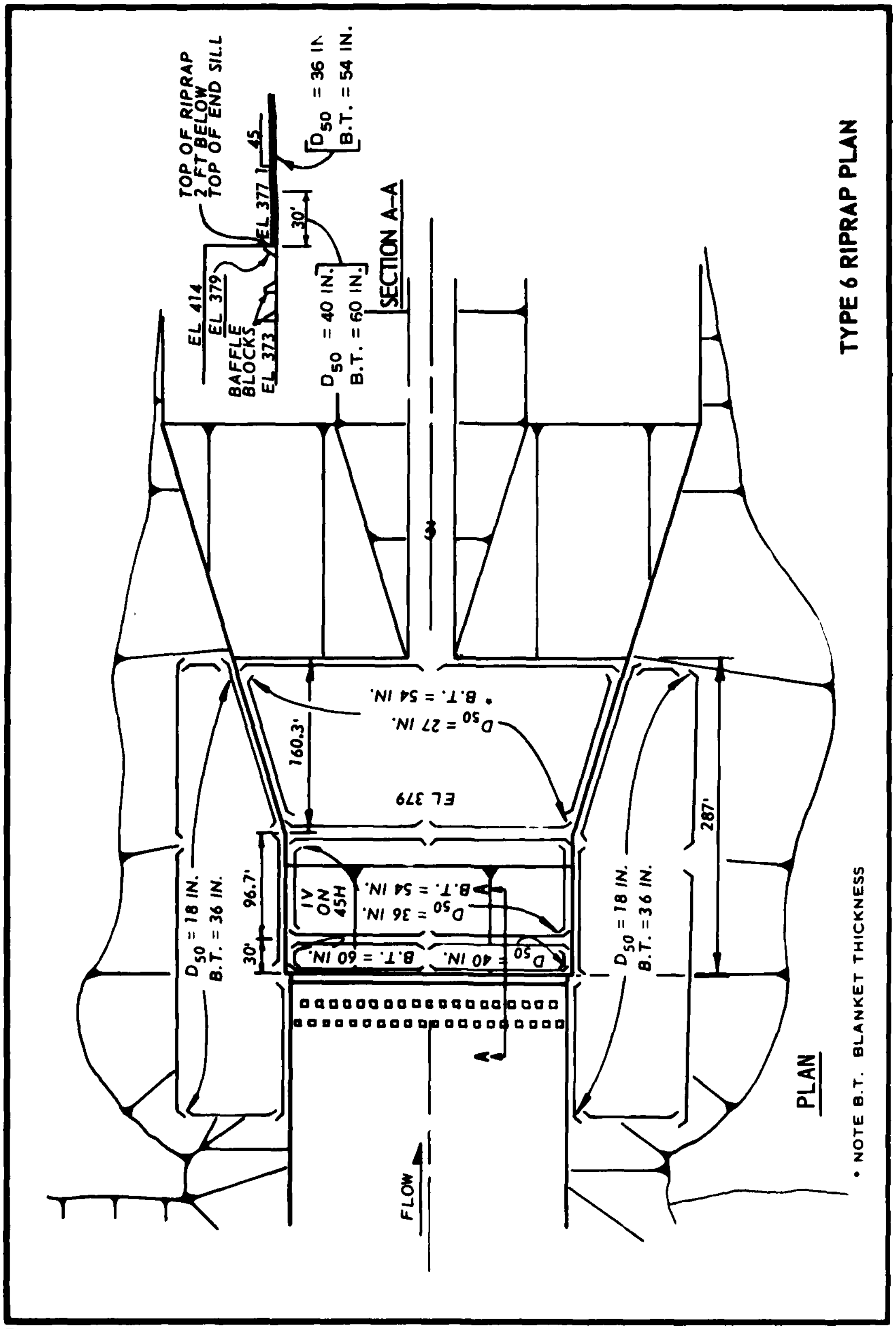


MAXIMUM VELOCITIES 1 FT ABOVE CHANNEL INVERT

ORIGINAL SPILLWAY DESIGN

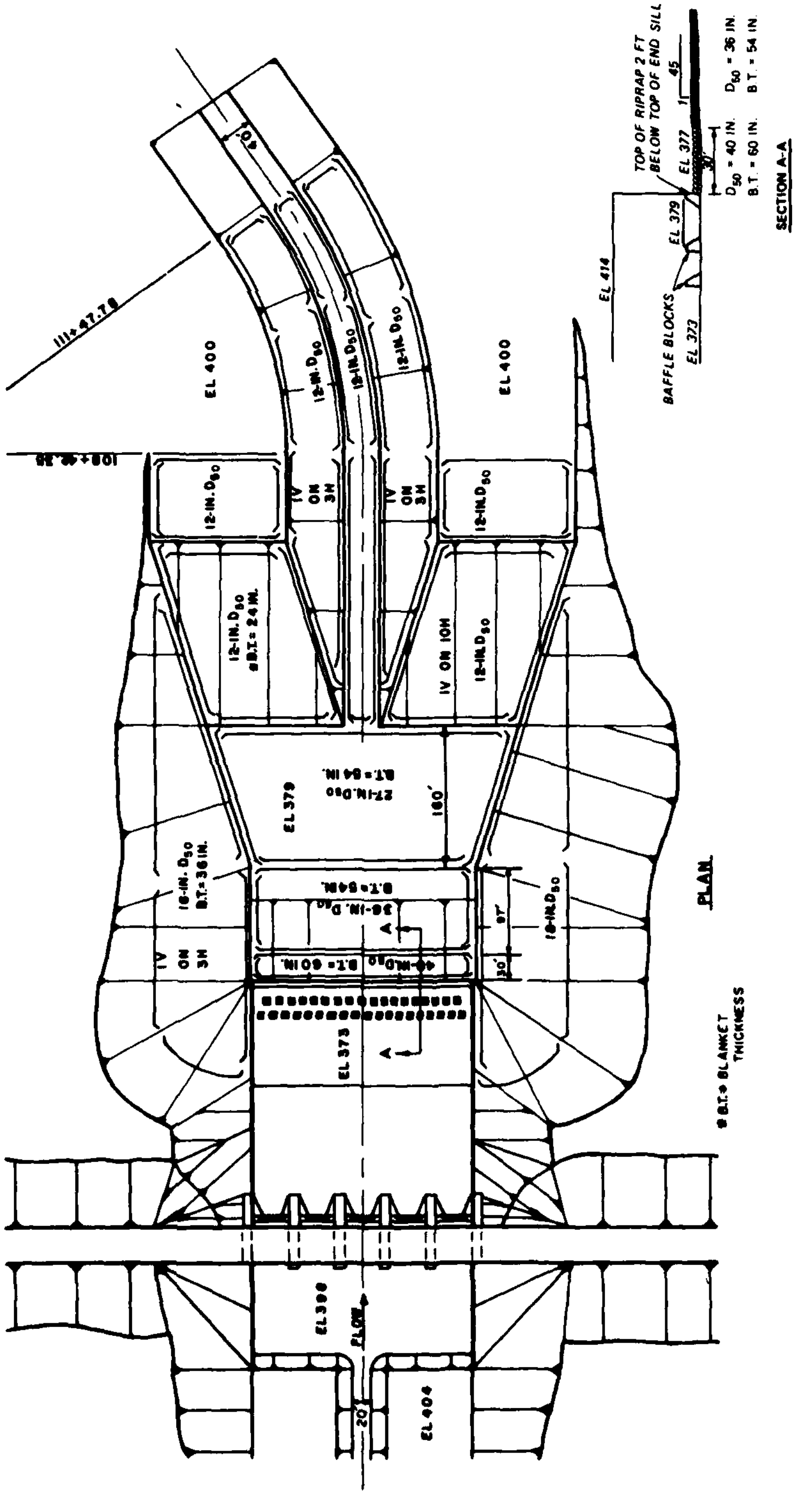
DISCHARGE	110,000 CFS
UPPER POOL EL	452.5
TAILWATER EL	410.5

NOTE VELOCITIES ARE IN PROTOTYPE FEET PER SECOND

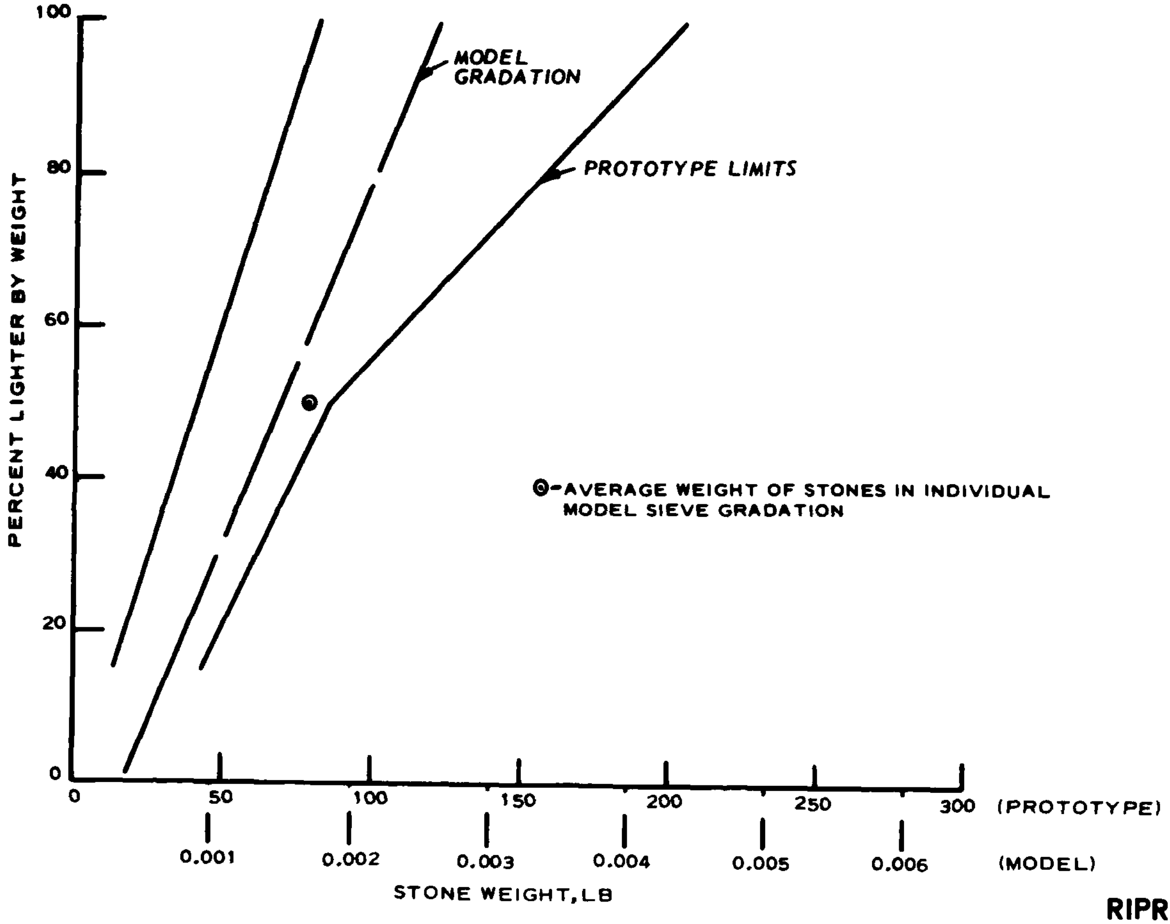


TYPE 6 RIPRAP PLAN

• NOTE B.T. BLANKET THICKNESS



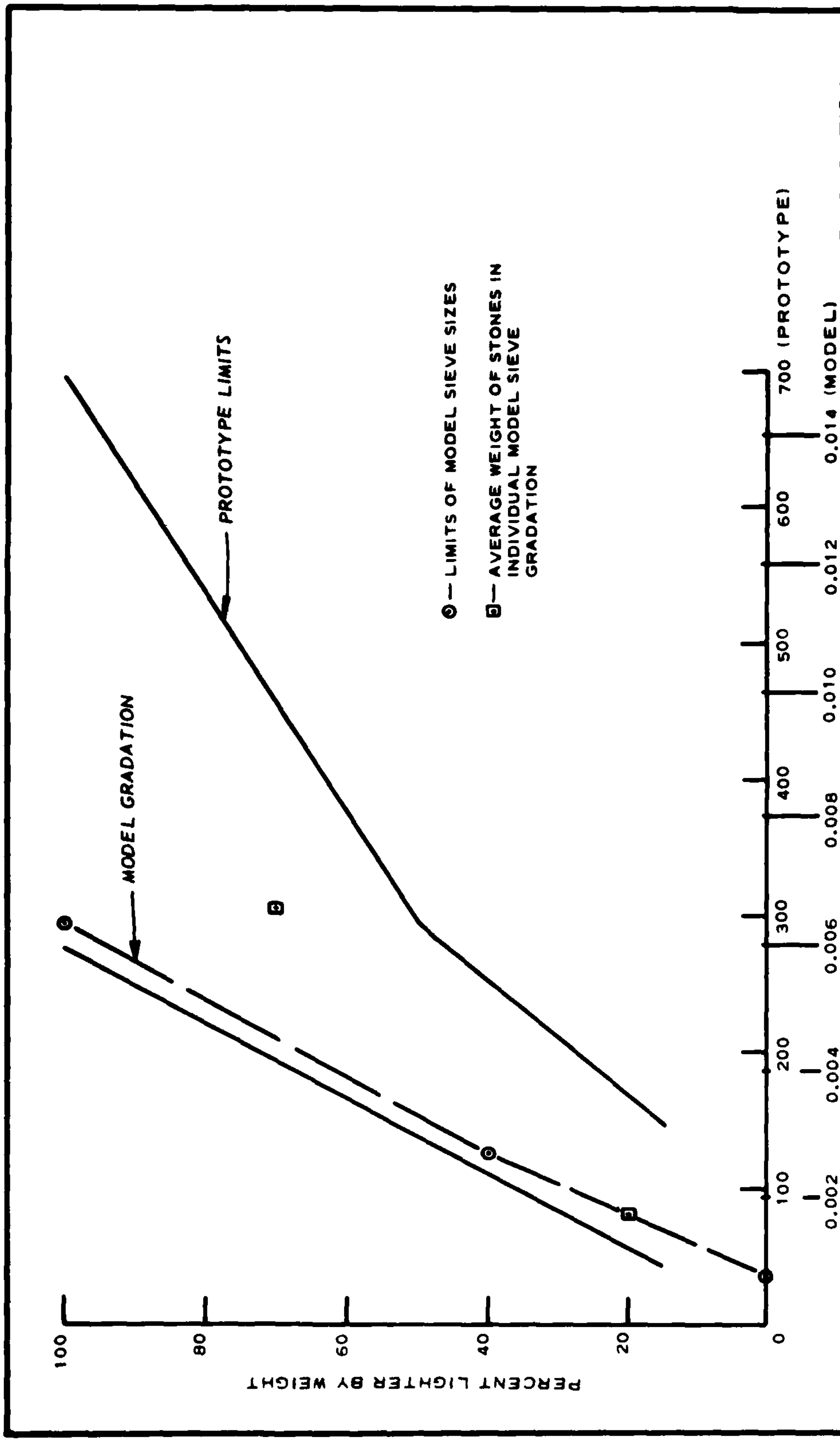
TYPE 9 RIPRAP PLAN



RIPRAP GRADATION

BLANKET THICKNESS = 24 IN.

$D_{50} = 12$ IN.

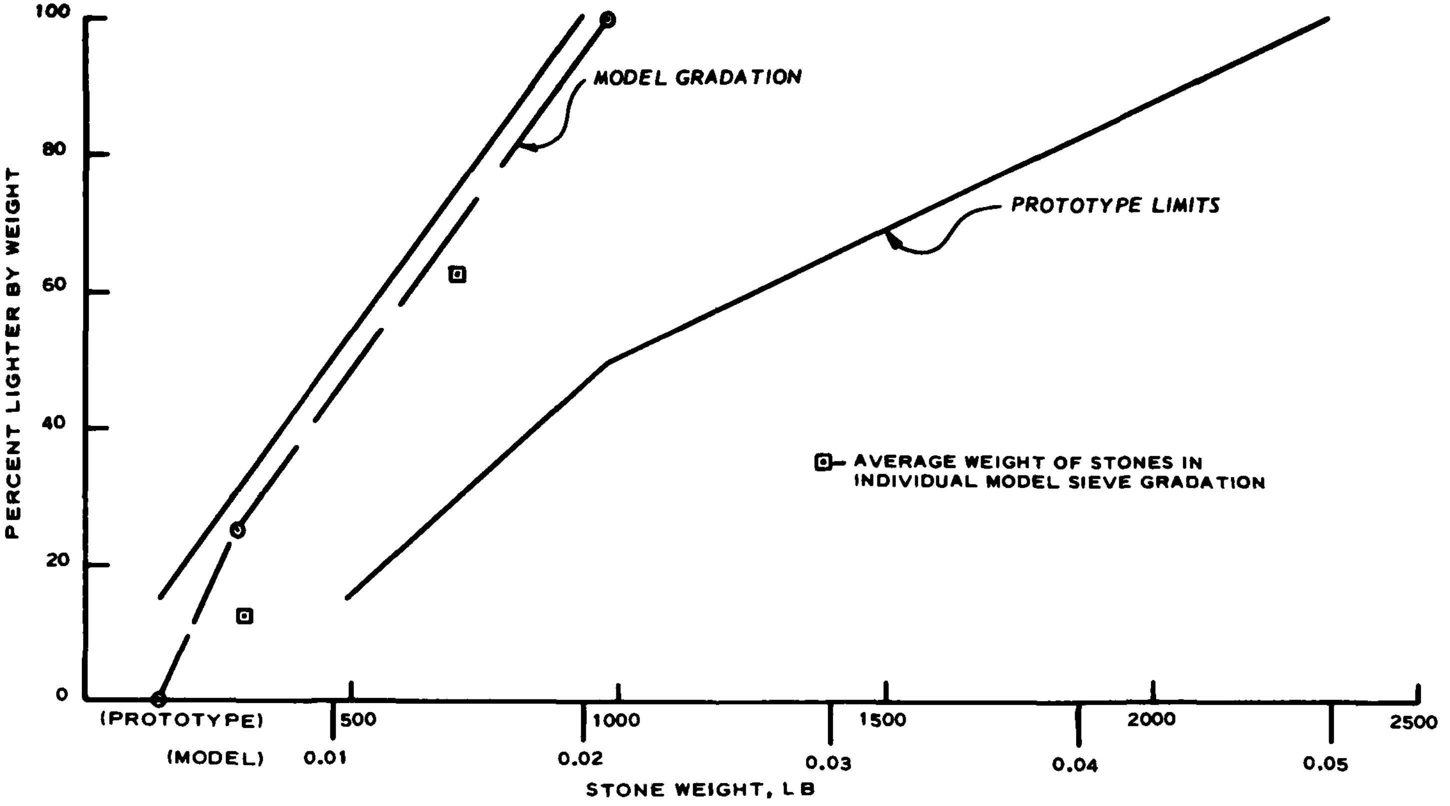


RIPRAP GRADATION

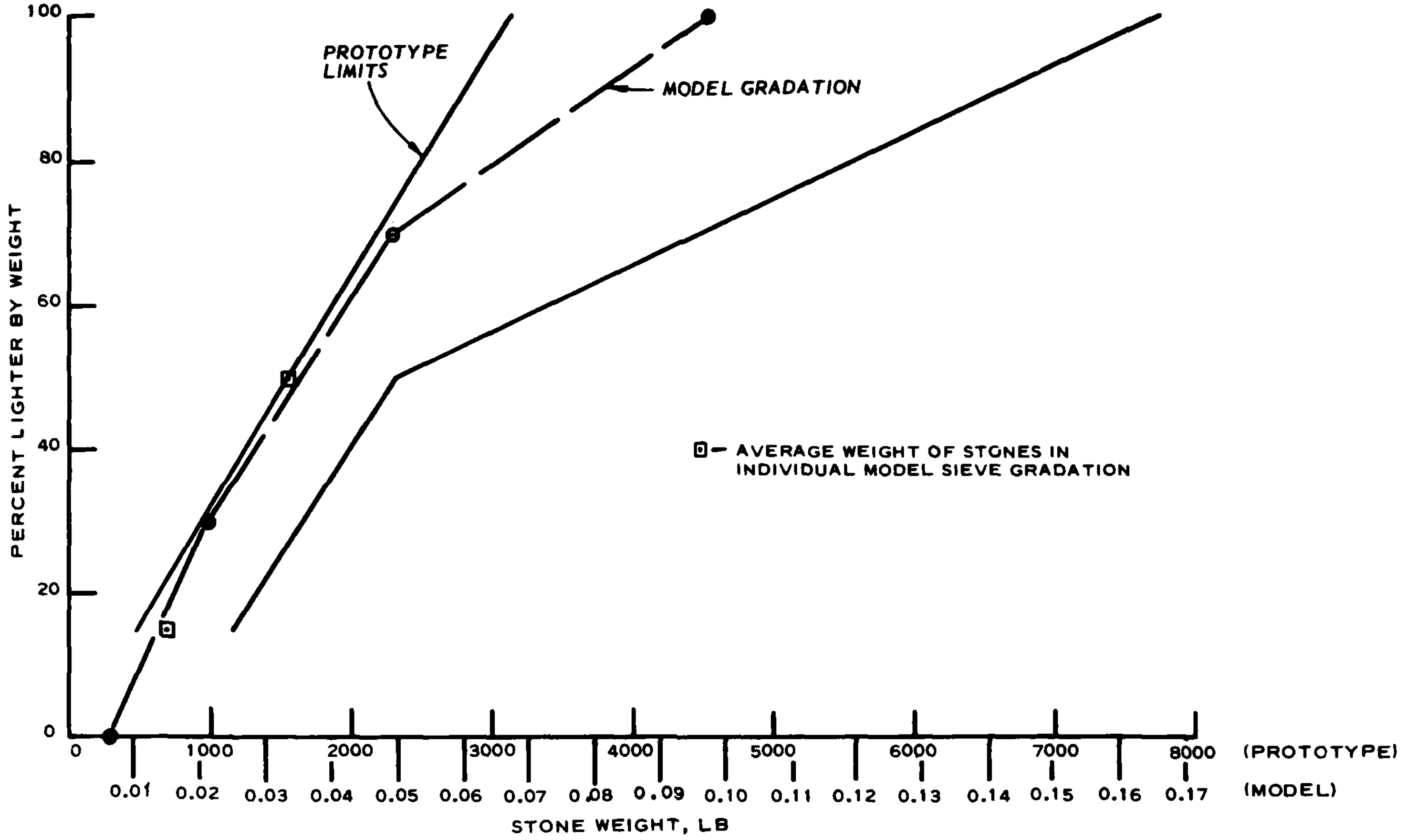
BLANKET THICKNESS = 36 IN.

D_{50} = 18 IN.

PLATE 21



RIPRAP GRADATION
BLANKET THICKNESS = 54 IN.
D_{50} = 27 IN.



RIPRAP GRADATION
BLANKET THICKNESS = 54 IN.
D_{50} = 36 IN.

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

George, John F
Spillway for Cooper Dam, Sulphur River, Texas; hydraulic model investigation / by John F. George. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1980.
21, [19] p., [12] leaves of plates : ill. ; 27 cm. (Technical report - U. S. Army Engineer Waterways Experiment Station ; HL-80-15)
Prepared for U. S. Army Engineer District, Fort Worth, Fort Worth, Texas.

1. Cooper Dam. 2. Flow characteristics. 3. Hydraulic models. 4. Spillways. 5. Sulphur River. I. United States. Army. Corps of Engineers. Fort Worth District. II. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Technical report ; HL-80-15.
TA7.W34 no.HL-80-15