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LAKE CHICOT PUMPING PLANT OUTLET STRUCTURE, ARKANSAS; HYDRAULIC--ETC(U)
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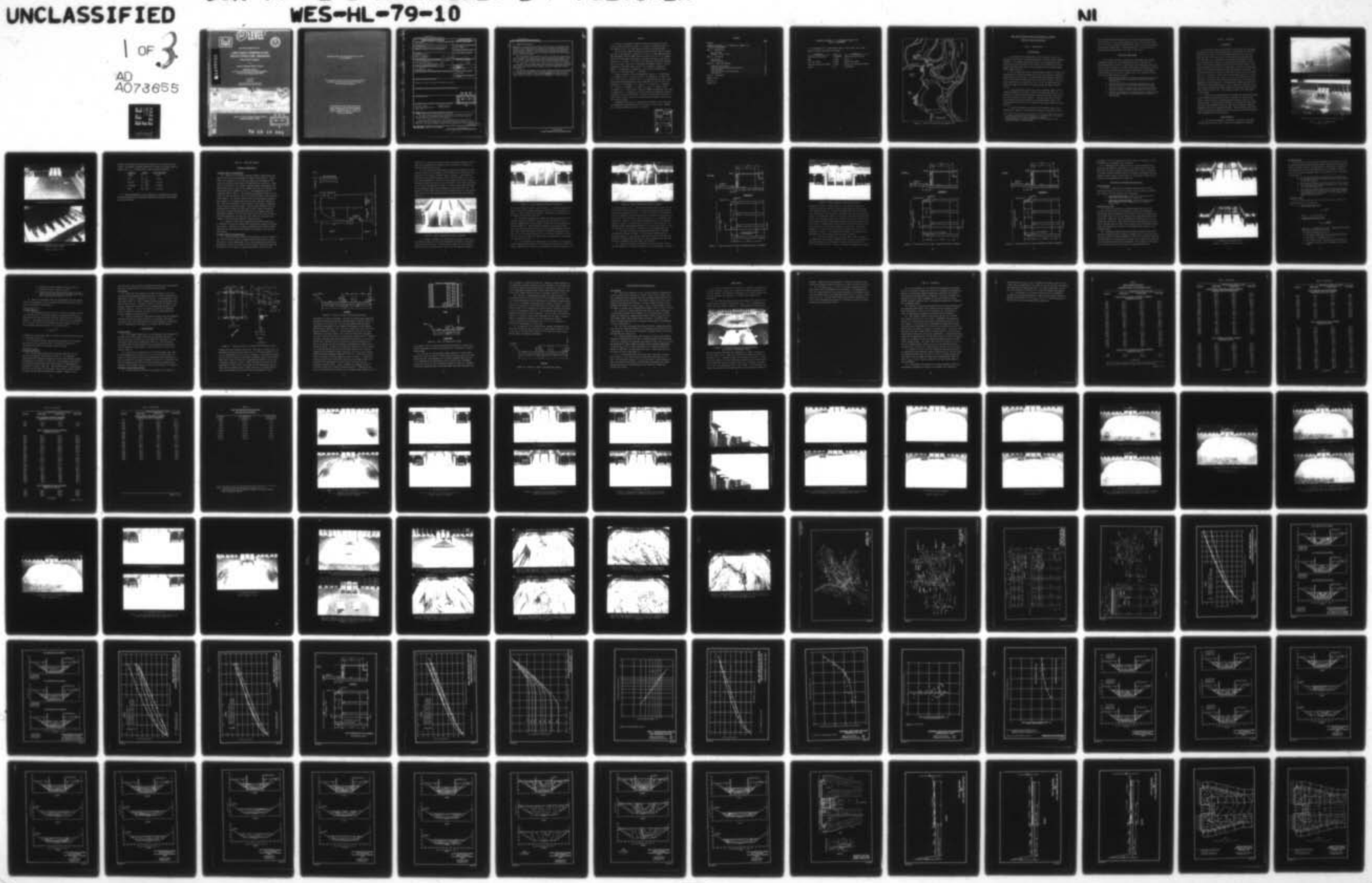

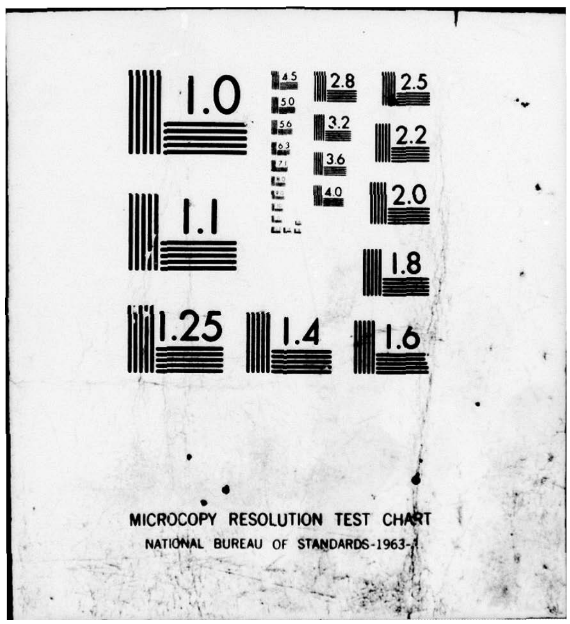
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TECHNICAL REPORT HL-79-10

LAKE CHICOT PUMPING PLANT OUTLET STRUCTURE, ARKANSAS

Hydraulic Model Investigation

by

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Final Report

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Stilling basins

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The model study reported herein was conducted to evaluate the gravity-flow and pump outlet discharge characteristics and the size and extent of stone protection required in the exit channel.

The study indicated the need for minor modifications to improve flow characteristics in the approach to the gravity-flow section and in the stilling basin. The major problems encountered were generated by adverse entrance
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20. ABSTRACT (Continued).

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conditions to the gravity-flow section with both free and submerged flows. Satisfactory approach flows were obtained by excavating a portion of the approach channel immediately upstream from the structure and streamlining the gravity-flow abutments. The improved flow conditions eliminated the adverse drawdown at the abutments and provided a more uniform flow distribution in the stilling basin.

The stilling basin was modified to permit more efficient energy dissipation by moving the original row of baffles downstream and adding a second row of baffles. Tests also indicated that the apron length could be reduced by 12 ft without significantly affecting stilling basin performance.

The model tests indicated that an 84-ft length of riprap with a maximum stone weight of 292 lb located downstream from the stilling basin would remain stable under expected flow conditions with the recommended stilling basin design.

The hydraulic performance of the pump discharge outlets was satisfactory for the range of anticipated flow conditions. Riprap protection (d_{100} = 18 in., maximum stone weight = 292 lb) downstream from the pump outlets remained stable for various tailwaters and combinations of pumps operating.

PREFACE

The model investigation reported herein was authorized by the Office, Chief of Engineers (OCE), U. S. Army, on 27 February 1976, at the request of the U. S. Army Engineer District, Vicksburg (LMK).

The study was conducted during the period February 1976 to September 1977 in the Hydraulics Laboratory of the U. S. Army Engineer Waterways Experiment Station (WES) under the direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory, and under the general supervision of Messrs. J. L. Grace, Jr., Chief of the Hydraulic Structures Division, and N. R. Oswalt, Chief of the Spillways and Channels Branch. Project Engineer for the model study was Mr. E. D. Rothwell, assisted by Messrs. B. Perkins and E. Jefferson. This report was prepared by Messrs. E. D. Rothwell and B. P. Fletcher.

During the course of the investigation, Messrs. J. S. Robertson, S. B. Powell, and R. L. Kinsel of OCE; J. R. McCormick, J. Harze III, W. R. Hill, and H. E. Walker of the U. S. Army Engineer Division, Lower Mississippi Valley/Mississippi River Commission; R. Lucius, COL G. E. Galloway, E. G. McGreggor, R. T. Miller, R. C. Randall, J. T. Knight, R. O. Smith, L. E. Banks, J. O. Ward, Jr., and P. G. Combs of LMK; P. Erekson, L. L. Pruitt, and P. Sharp of Stanley Consultants, Inc.; and T. Nakato and J. F. Kennedy of the Iowa Institute of Hydraulic Research, University of Iowa, visited WES to discuss the program and results of model tests, observe the model in operation, and correlate these results with design studies.

Commander and Director of WES during the conduct of the study and the preparation and publication of this report was COL John L. Cannon, 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
cubic feet per second	0.02831685	cubic metres per second
feet	0.3048	metres
feet per second	0.3048	metres per second
feet per second per second	0.3048	metres per second per second
inches	25.4	millimetres

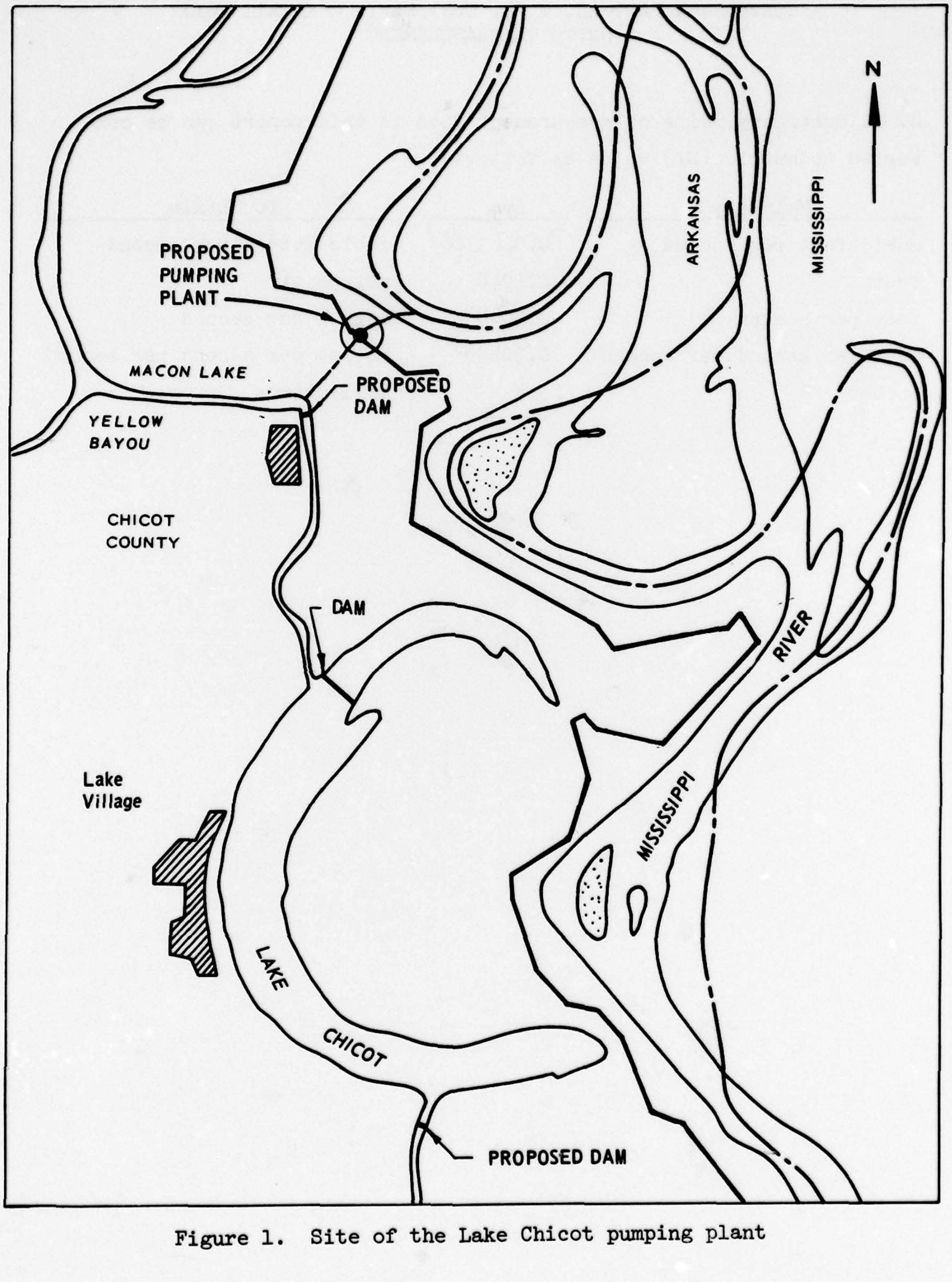

Figure 1. Site of the Lake Chicot pumping plant

LAKE CHICOT PUMPING PLANT OUTLET STRUCTURE, ARKANSAS

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. The Lake Chicot pumping plant will be located in the right bank main-line levee of the Mississippi River in Chicot County, Arkansas (Figure 1). The pumping plant will consist of 12 pumping bays and a gravity-flow section located at the center of the structure, an inlet channel from Macon Lake, and an outlet channel to the old Mississippi River channel (Plate 1).

2. The 12 pumps will have a total discharge capacity of 6,500 cfs,* consisting of 10 identical pumps rated at 600 cfs each and 2 identical pumps rated at 250 cfs each. The pump intake sump effective widths are 23 ft for the 600-cfs pumps and 16 ft for the 250-cfs pumps. The sump floor elevation for all sump bays is 93.0 ft.** The pump outlet bay widths are equal to the width of the pump intake bay sumps (Plates 2 and 3).

3. The gravity-flow section will have a spillway length of 88 ft between abutments and consists of three 26-ft-wide gate bays, Nos. 1-3 from left to right looking downstream, separated by piers, and a spillway crest elevation of 93.0 (Plate 4). The gate bays will be fitted with vertical-lift gates operated by an overhead gantry crane. The stilling basin will consist of a horizontal apron with baffle piers and end sill.

4. The inlet channel will convey flows from Macon Lake to the pumping plant forebay. The forebay will provide a transition from the

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

** All elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum.

150-ft bottom width inlet channel to the 454-ft-wide pumping plant as shown in Plate 1. The outlet channel will have a 130-ft bottom width at el 80.0 and will convey pump and gravity flows to the old Mississippi River channel (Rowdy Bend). Additional details of the structure are described where appropriate in the text of this report.

Purpose of Model Study

5. The model tests were conducted to investigate the hydraulic performance relative to the pumping plant outlet structures for uncontrolled-flow operations. Specifically, the model study would provide the data necessary to evaluate and develop a satisfactory stilling basin design, upstream approach channel configuration, and adequate riprap protection in the outlet channel area. The following information was obtained during the study:

a. Modifications relative to improving the approach flow conditions and hydraulic performance through the gravity-flow section.

b. Flow characteristics and hydraulic performance of the gravity-flow section with uncontrolled-flow operations.

c. Guidance relative to design of the gravity-flow stilling basin and riprap protection downstream of the end sill.

d. Hydraulic performance of the pumping plant discharge outlet and the riprap protection required below the pump discharge bays.

PART II: THE MODEL

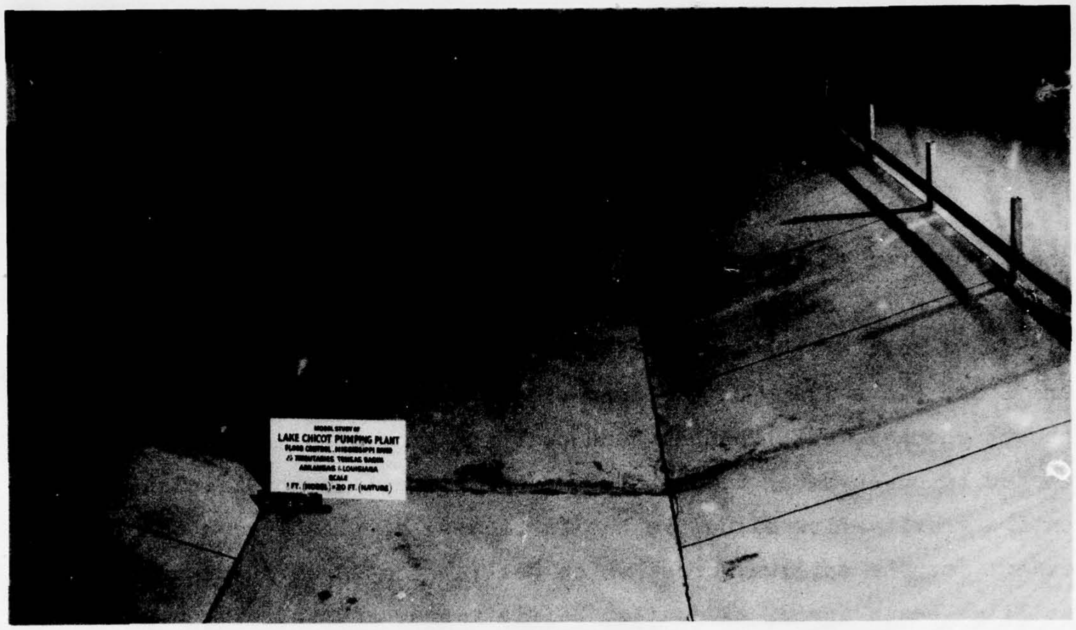
Description

6. A 1:20-scale model was constructed to reproduce all topography and structures in an area extending 420 ft upstream and 1,100 ft downstream from the center line of the pumping plant and 330 ft to the right and left of the center line of the gravity-flow spillway (Figure 2 and Plate 1). The portions of the model representing the approach channel, exit channel, and overbank area were molded of cement mortar to sheet-metal templates and were given a brushed finish. The entire gravity-flow structure was fabricated of plastic-coated plywood except for the spillway crest which was fabricated of sheet metal. The stilling basin apron, sidewalls, baffle piers, and end sill were fabricated of wood material treated with a waterproofing compound to prevent expansion. The pumping bay intake piers and the discharge outlets were fabricated of plastic-coated plywood and treated with a waterproofing compound to prevent expansion. The pumping bay intake pier noses were fabricated of transparent plastic. The 12 pumps were simulated by two 20-in.-diam pressurized manifolds symmetrical about the center gravity-flow structure (Figure 2d). The discharge conduits were fabricated of sheet metal.

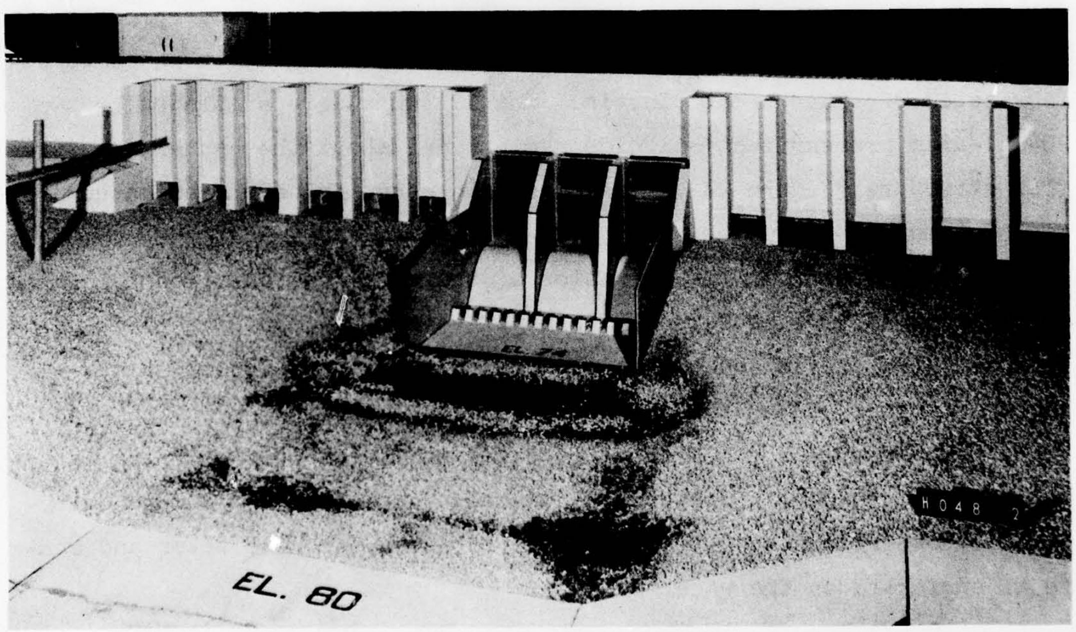
7. Water used in the operation of the model was supplied by pumps, and discharges were measured by means of venturi and orifice plate meters. Steel rails set to grade provided reference planes for measuring devices. Water-surface elevations were obtained by point gages. Velocities were measured with pitot tubes and by stopwatch timing of movement of dye over a measured distance. Current patterns were determined by observing the movement of dye injected into the water and confetti sprinkled on the water surface.

Scale Relations

8. The accepted equations of hydraulic similitude, based upon Froudian criteria, were used to express the mathematical relations

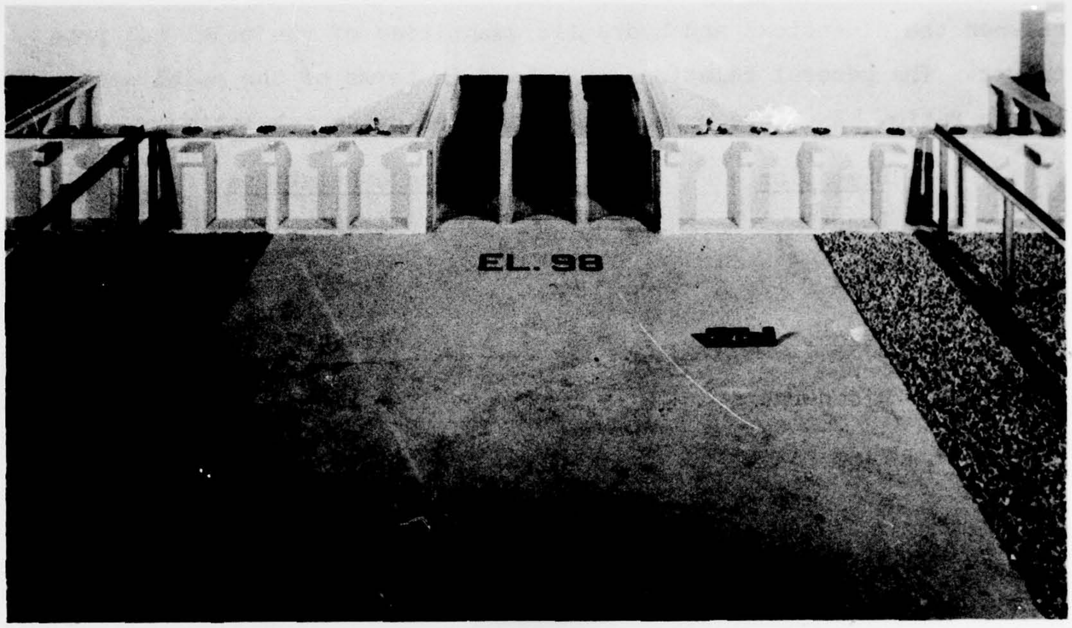


a. Looking upstream toward pumping plant

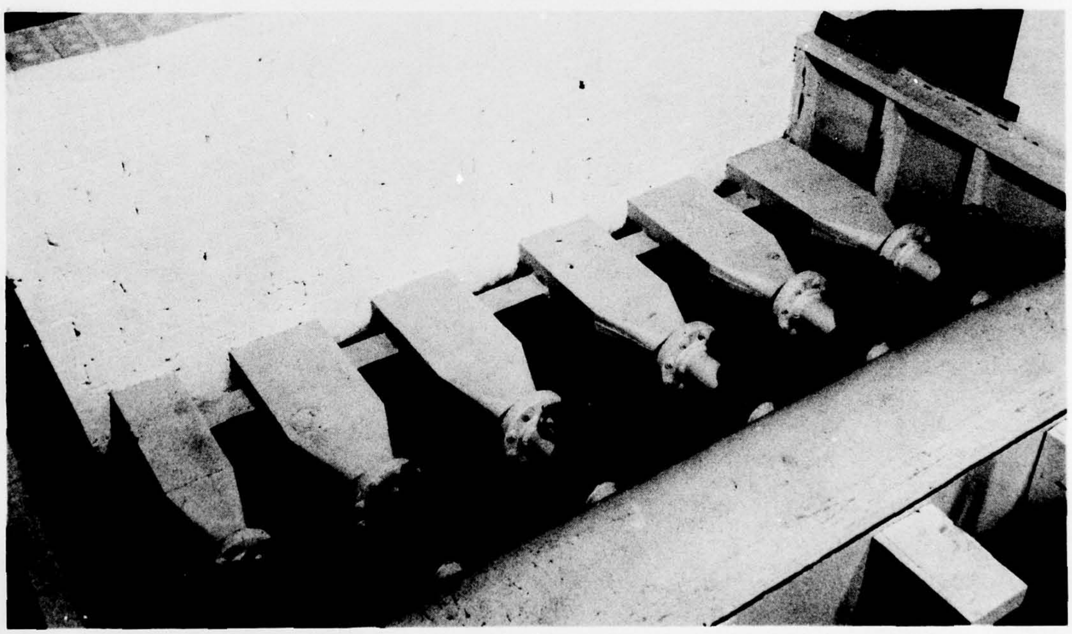


b. Original stilling basin and riprap protection (type 1)

Figure 2. The 1:20-scale model
(sheet 1 of 2)



c. Upstream approach area with riprap protection



d. Pumping discharge conduits (one 250-cfs pump and five 600-cfs pumps)

Figure 2 (sheet 2 of 2)

between the dimensions and hydraulic quantities of the model and prototype. The general relations expressed in terms of the model scale or length ratio, L_r , are presented in the following tabulation:

Dimension	Ratio	Scale Relations
Length	L_r	1:20
Area	$A_r = L_r^2$	1:400
Velocity	$V_r = L_r^{1/2}$	1:4.472
Discharge	$Q_r = L_r^{5/2}$	1:1788
Time	$T_r = L_r^{1/2}$	1:4.472

9. Model measurements of each dimension or variable can be transferred quantitatively to prototype equivalents by means of the preceding scale relations.

PART III: TESTS AND RESULTS

Approach Configuration

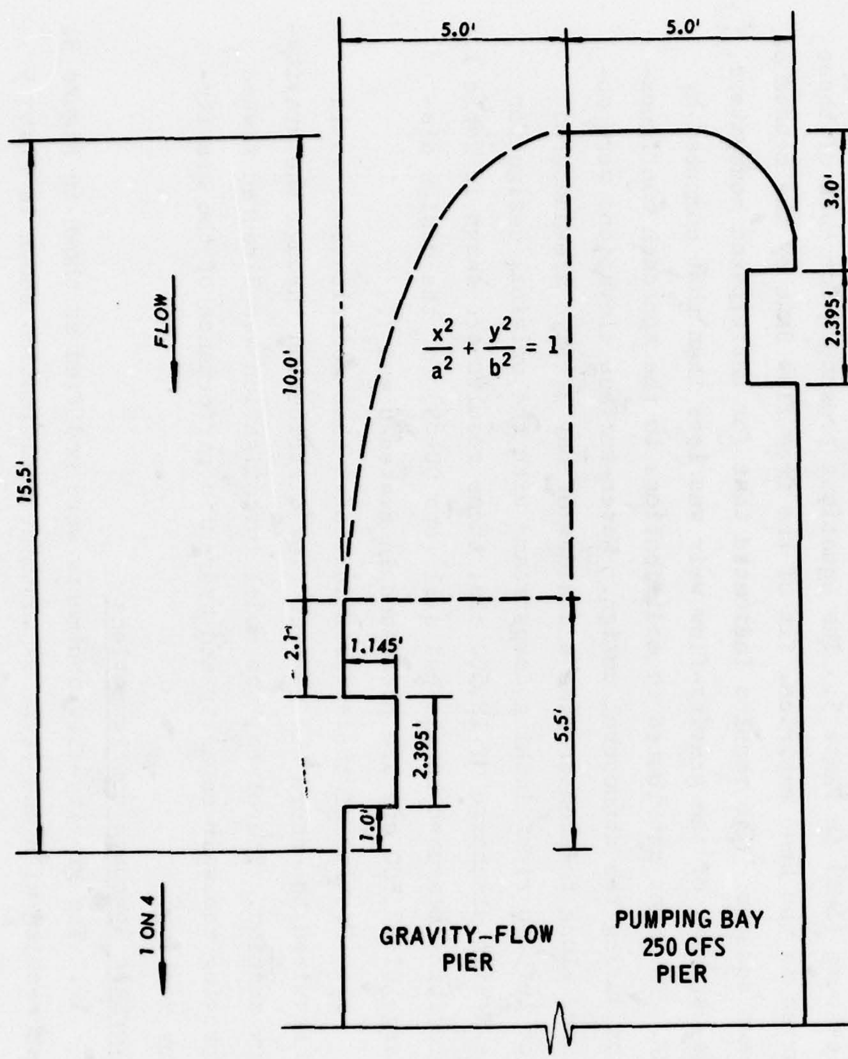
Original (type 1) configuration

10. Details of the original approach channel configuration and gravity-flow section are presented in Figure 2c and Plate 4. Initial tests were conducted in the 1:20-scale model with the gravity-flow section to determine and evaluate the approach flow conditions. Model results obtained with the original approach channel configuration and gravity-flow section (type 1) are compared with the free uncontrolled-flow rating curves computed by the U. S. Army Engineers District, Vicksburg (LMK) in Plate 5. The equations presented for each of these curves is the best empirical fit of the free flow data by the method of least squares. The results indicated that for anticipated headwaters, the capacity of the gravity-flow weir was less than that computed by LMK. This was attributed to modifications to the approach configuration during the structural design. Water-surface elevations were obtained along the center line and sides of bay 3 (bays are numbered from left to right looking downstream) with the original design for the design discharge of 12,500 cfs; these results are shown in Table 1. Velocities measured at the end sill (sta 00+85) for the design discharge of 12,500 cfs are presented in Plates 6 and 7.

11. Results of these tests indicated that modifications would be required to ensure the desired flow distribution through the gravity-flow section. Therefore, the model investigation was directed toward improving the weir capacity and hydraulic performance of the gravity-flow section.

Alternate approach configurations

12. The gravity-flow abutments were modified as shown in Figure 3. Test results with the (type 2) abutment modification shown in Plate 5 reveal only a slight improvement in the free uncontrolled rating curve. An analysis of the data indicated that an unequal flow distribution was associated with the (type 2) abutment modification and that it was



ELLIPSE EQ. $\frac{x^2}{a^2} + \frac{y^2}{b^2}$

y	x
0	5
1	4.97
2	4.89
3	4.77
4	4.58
5	4.33
6	4.00
7	3.57
8	3.00
9	2.18
10	0

Figure 3. Details of gravity-flow section abutment type 2 modification

ineffective in reducing the water-surface contraction around the abutments and eliminating the nonuniform flow distribution through the three-bay gravity-flow section.

13. The original upstream approach channel was modified by excavating a portion of the channel to a IV-on-4H slope to obtain the type 2 approach with the type 2 abutment shown in Figure 4. Water-surface elevations were obtained along the center line and sides of bay 3 for the design discharge of 12,500 cfs; these results are shown in Table 1. Velocities measured at the end sill (sta 00+85) for the design discharge of 12,500 cfs are presented in Plate 6. Photo 1 shows flow conditions in the approach channel with discharges of 5,000 and 12,500 cfs. An analysis of these data indicates that discharge capacity of the gravity-flow section with the modified approach is greater than that computed (Plate 8).

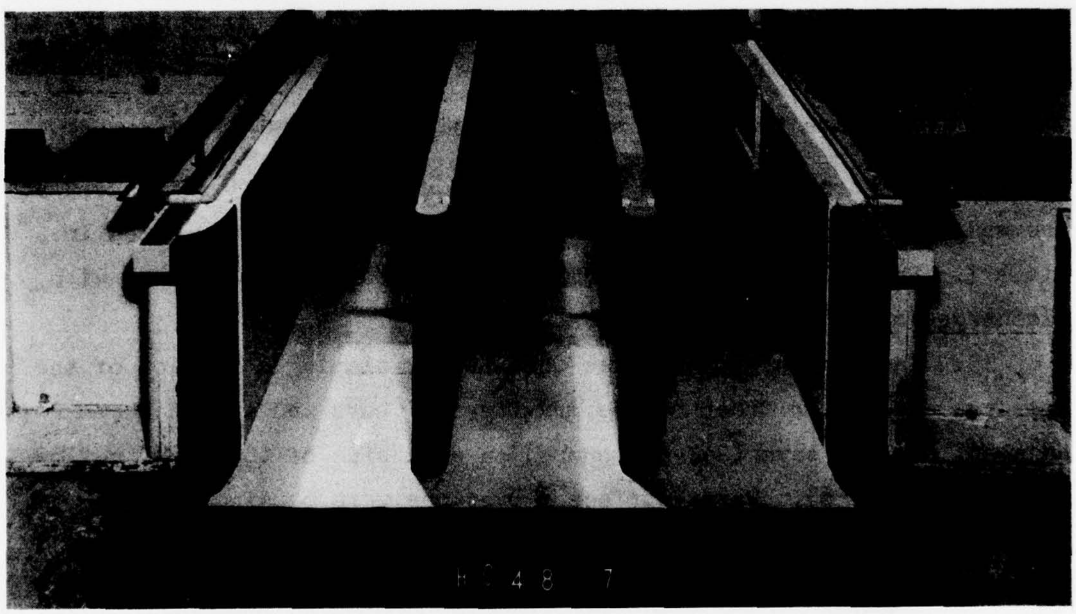


Figure 4. Type 2 approach with type 2 abutment

14. The type 3 approach was constructed by sloping a portion of the approach channel in front of the gravity-flow bays to a IV-on-10H-slope as shown in Figure 5. Water-surface elevations were obtained along the center line and sides of bay 3 with the design discharge of

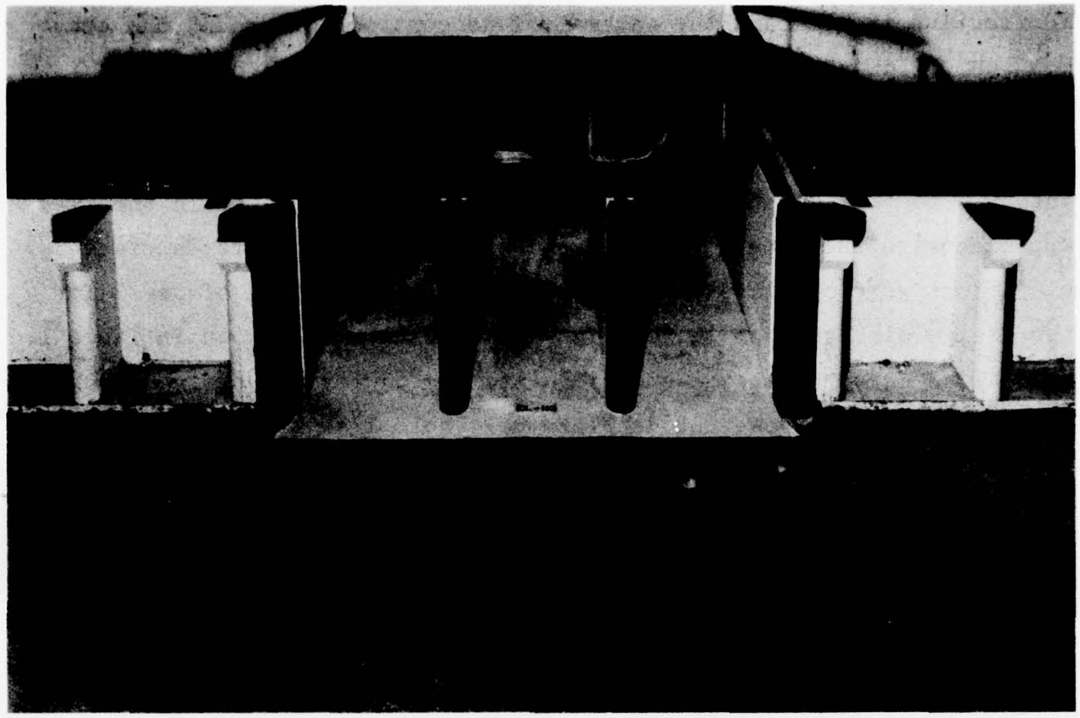


Figure 5. Type 3 approach with type 2 abutment

12,500 cfs; these results are shown in Table 1. Velocities measured at the end sill for the design discharge of 12,500 cfs are presented in Plate 7. Photo 2 shows flow conditions observed in the approach with discharges of 5,000 and 12,500 cfs.

15. The type 4 approach was obtained by sloping a portion of the approach channel in front of the gravity-flow bays and two of the adjacent pumping bays (250-cfs pumps) on each side of the gravity-flow section (Figure 6). Water-surface elevations were obtained along the center line and sides of bay 3 with the design discharge of 12,500 cfs; these results are shown in Table 1. Velocities measured at the end sill for the design discharge of 12,500 cfs are presented in Plate 7. Photo 3 shows flow conditions observed in the approach with discharges of 5,000 and 12,500 cfs.

16. It is apparent that modification of a portion of the approach channel immediately upstream of the gravity-flow section will effectively improve approach flow conditions and increase the discharge capacity of

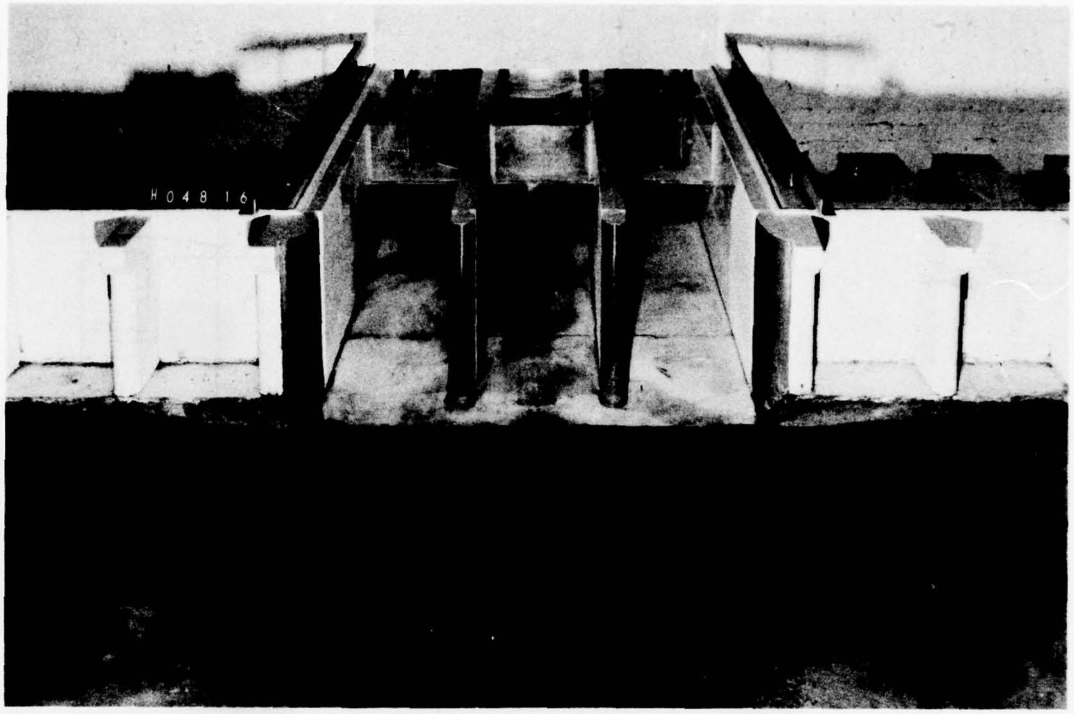


Figure 6. Type 4 approach with type 2 abutment

the gravity-flow section. The free uncontrolled rating curves for the gravity-flow section with each of the various approach channel modifications are presented in Plate 8. Results indicate that reduced velocities and improved flow distribution can be obtained in the vicinity of the end sill (sta 00+85) with either the type 2, 3, or 4 approach configuration. However, it was required that excavation in the approach channel be kept to a minimum and be restricted to that portion of the approach channel immediately upstream of the gravity-flow bays to avoid interfering with the approach conditions to the adjacent pump bays.

17. The type 5 approach configuration and the type 3 abutment modification is shown in Figure 7. Test results indicate that the discharge capacity of the gravity-flow section was less than that computed for discharges greater than 9,000 cfs and greater than that computed for discharges less than 8,000 cfs (Plate 9).

18. The type 6 approach configuration, which consisted of excavating a portion of the approach channel to a IV-on-6H slope, and type 3

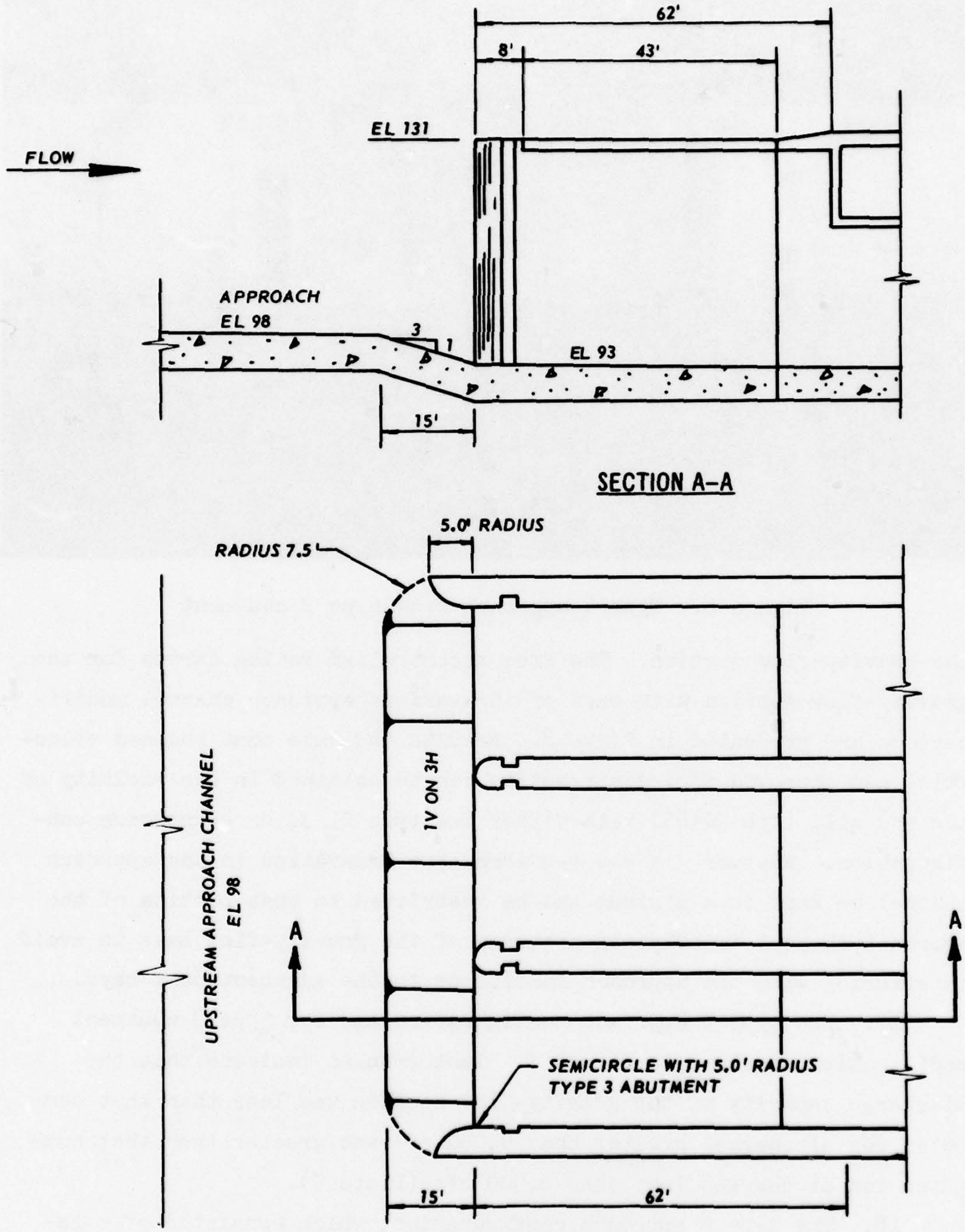


Figure 7. Plan and section of type 5 approach and type 3 abutment

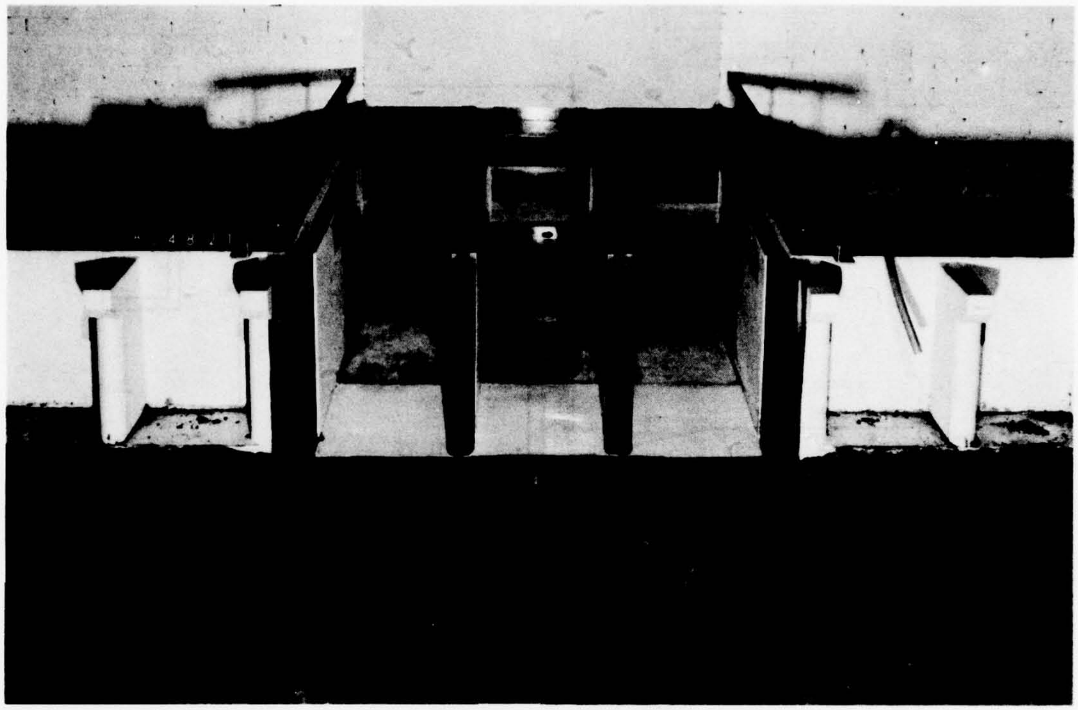


Figure 8. Type 6 approach with type 3 abutment

abutment are shown in Figure 8 and 10. Model results indicate a slightly larger flow capacity through the gravity-flow section than that computed (Plate 11). However, the magnitude of flow instability which occurred at the gravity-flow abutments, with previous configurations, was sufficiently reduced. This scheme would also minimize excavation in the approach channel and would be restricted to that portion of the approach channel immediately upstream of the three-bay gravity-flow section. Water-surface elevations were obtained along the center line of bay 3 with the design discharge of 12,500 cfs. Velocities measured at the end sill for the design discharge of 12,500 cfs are presented in Plate 7. Flow conditions observed in the approach with discharges of 5,000 and 12,500 cfs are shown in Photo 4.

19. Tests were conducted with both the types 7 and 8 approach configurations and the type 3 abutment which consisted of modifying the gravity-flow section by raising the spillway crest and floor to el 95.0 ft (Figures 9 and 10). Results with these configurations indicate

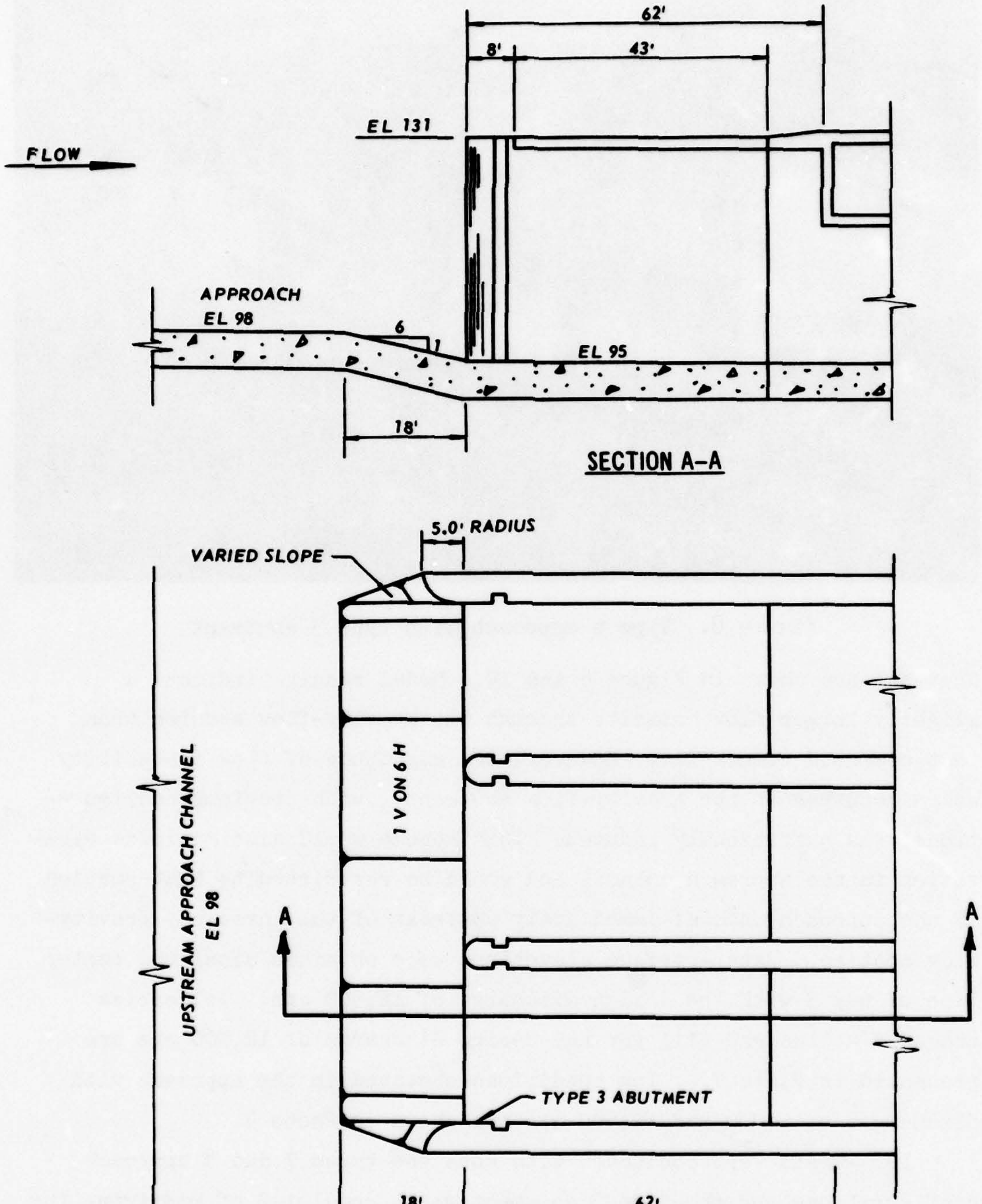


Figure 9. Plan and section of type 7 approach and type 3 abutment

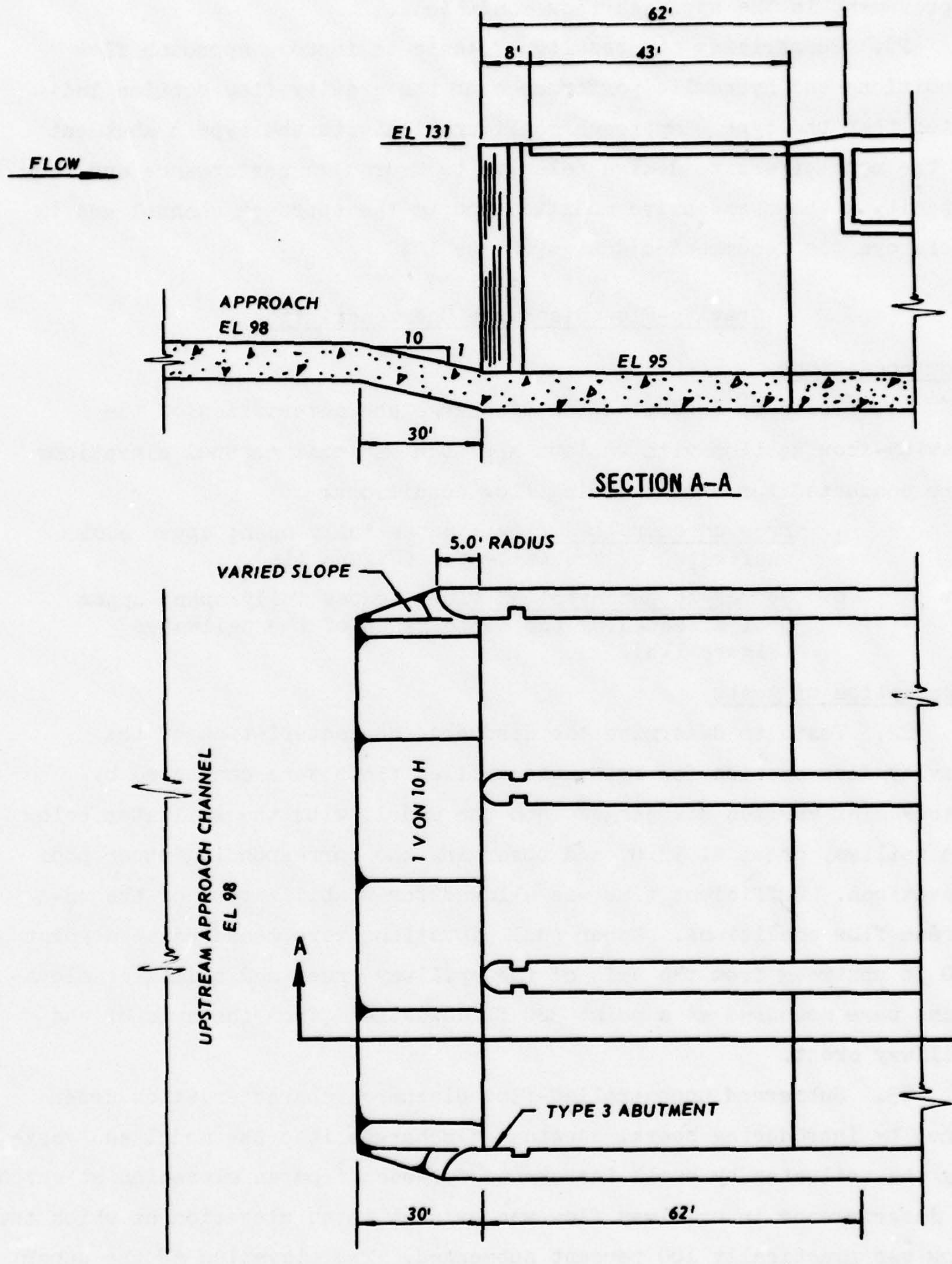


Figure 10. Plan and section of type 8 approach and type 3 abutment

a decrease in the flow capacity as shown in Plate 10 and only a slight improvement in the approach flow conditions.

20. Summarizing the results of tests to improve approach flow conditions and hydraulic performance of the gravity-flow section indicates that the type 6 approach configuration with the type 3 abutment is the most effective design relative to hydraulic performance and flow capacity without extensive modification to the approach channel and is therefore the recommended design (Plate 10).

Gravity-Flow Discharge Characteristics

Flow conditions

21. Tests to determine the discharge characteristics of the gravity-flow section with various approach and exit channel elevations were conducted for the following flow conditions:

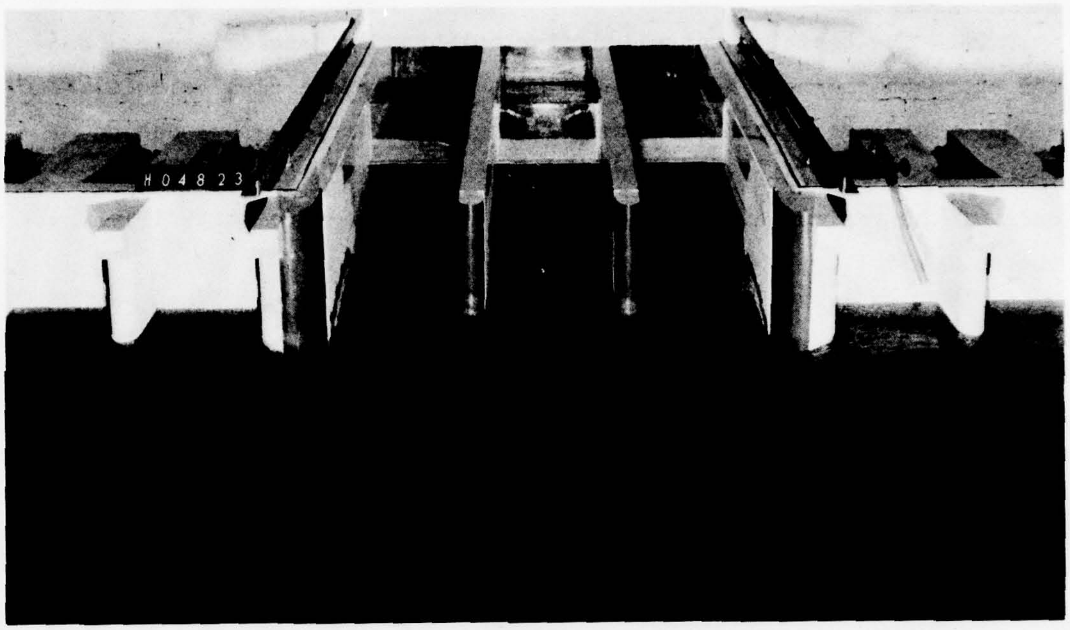
a. Free uncontrolled flow: gates fully open; upper pool unaffected by the tailwater (Figure 11a).

b. Submerged uncontrolled flow: gates fully open; upper pool affected by the submergence of the tailwater (Figure 11b).

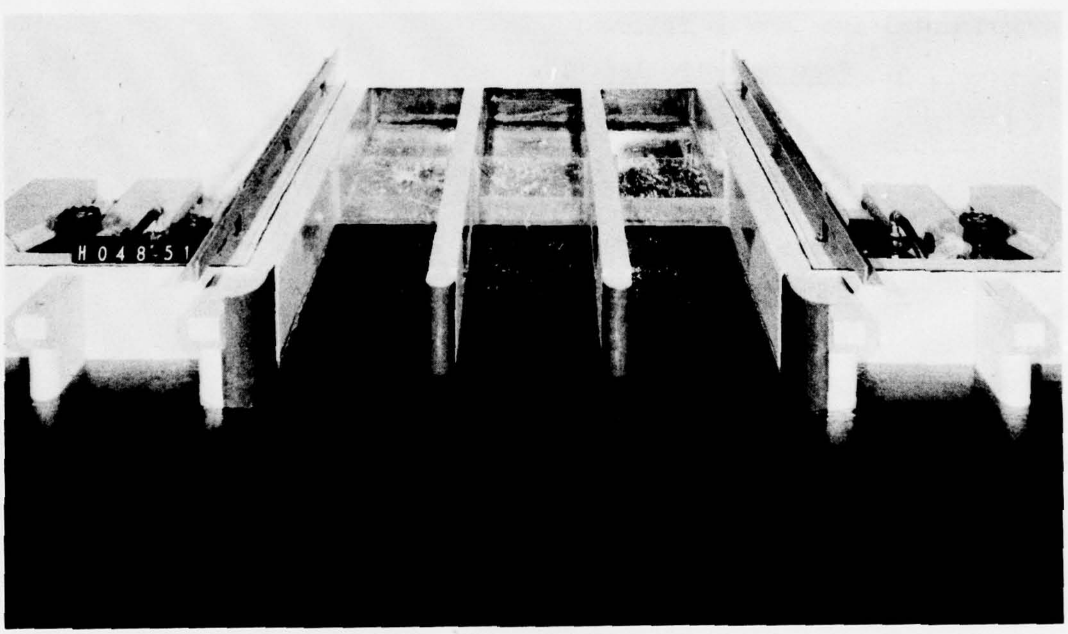
Description of tests

22. Tests to determine the discharge characteristics of the gravity-flow section for free uncontrolled flows were conducted by introducing various discharges into the model, with the tailwater below the spillway crest el 93.0, and observing the corresponding upper pool elevations. Sufficient time was allowed for stabilization of the upstream flow conditions. Upper pool elevations were measured at a point 310 ft upstream from the axis of the spillway crest and tailwater elevations were measured at a point 320 ft downstream from the axis of the spillway crest.

23. Submerged uncontrolled-flow discharge characteristics determined by introducing seeral constant discharges into the model and varying the tailwater by small increments for each from an elevation at which no interference in spillway flow was evident to an elevation at which the flow was practically 100 percent submerged. The elevation of the upper pool was noted at each of the respective tailwater elevations.



a. Free uncontrolled flow



b. Submerged uncontrolled flow

Figure 11. Approach flow conditions

Calibration data

24. The basic uncontrolled-flow calibration data (Plate 12) show the approach channel energy elevation (water surface plus velocity head based on average velocity) corresponding to a particular elevation of the tailwater for a given discharge observed in the model. Data for each of the various discharges shown in the respective plate illustrate the following:

- a. The relation between the elevation of the energy flow in the approach channel and the elevation of the tailwater in the exit channel.
- b. The range of tailwater elevations at which the elevation of the approach flow energy is constant, i.e., the range of free uncontrolled flow.
- c. The range of tailwater elevations at which the elevation of the approach flow energy is controlled by the submergence effects of the tailwater, i.e., the range of submerged uncontrolled flow.

Analyses of data

25. The flow conditions and the equations used to satisfy the experimental data are as follows:

a. Free uncontrolled flow:

$$Q = CLH^{3/2}$$

where C is a function of H .

b. Submerged uncontrolled flow:

$$Q = C_s Lh\sqrt{2g\Delta H}$$

where C_s is a function of h/H . Symbols used in these equations are defined as follows:

Q = total discharge per bay, cfs

C = discharge coefficient for free uncontrolled flow

L = net length of spillway crest, ft

H = total head on weir (including velocity head), ft

C_s = discharge coefficient for submerged uncontrolled flow

h = tailwater elevation referred to weir crest, ft

g = acceleration due to gravity, ft/sec^2

ΔH = difference between total energy of flow in the approach channel and elevation of tailwater with reference to the spillway crest (H - h), ft

26. Quantities determined from the experimental data were substituted in the equations, and the discharge coefficients for the respective flow conditions were computed.

Uncontrolled flow--spillway capacity

27. The recommended head on the crest (based on the depth of flow and velocity head in the approach channel) to discharge relation for uncontrolled flow was determined from basic data obtained in the model (Table 2). The equation presented for this curve is the best empirical fit of the free flow data by the method of least squares (Plate 13). The following equation satisfies this calibration data:

$$Q = 436.4H^{1.39}$$

A comparison of the actual model and computed uncontrolled rating curves is presented in Plate 14.

28. The model indicated satisfactory performance of the gravity flow section for the expected range of discharges; therefore no alterations were made in the design during the study.

Uncontrolled flow--discharge coefficient

29. The free uncontrolled-flow discharge coefficient for the gravity-flow spillway is presented in Plate 15. The submerged uncontrolled-flow discharge coefficient resulting from various degrees of submergence is presented in Plate 16. The submerged uncontrolled-flow discharge coefficients varied considerably for submergences of 85 percent or greater (Plate 16). This variation may be contributed to the fact that the degree to which model scale effects vitiate calibration data at small head differentials is unknown. However, it is

suspected that viscous effects predominate with small head differentials and yield increased values of the discharge coefficients.

Flow regime

30. Model data were analyzed to define the limits of the flow regime and corresponding discharge equations in terms of dimensionless quantities in order to generalize the results. An investigation of the basic data (Plate 12), with a constant discharge and uncontrolled flow, reveals that there is a tailwater elevation at which the energy of the approach channel flow increases with a corresponding increase in the tailwater elevation. This is the elevation at which the tailwater begins to submerge or control the flow, and free uncontrolled flow becomes submerged uncontrolled flow.

31. Results of analyses to distinguish between free and submerged uncontrolled flows are shown in Plate 17. In general, this plate illustrates that free uncontrolled flow becomes submerged uncontrolled flow for submergences (h/H) equal to or greater than 60 percent.

Stilling Basin

Original design

32. The original design (type 1) stilling basin was provided with a single row of baffle piers and an end sill (Figures 2b and 12). Isovels obtained at the end sill (sta 00+85) for discharges of 5,000, 10,000 and 12,500 cfs are presented in Plate 18. Flow conditions and surface flow patterns observed with the original stilling basin and exit channel for discharges of 5,000, 10,000 and 12,500 cfs are shown in Photo 5.

33. Location of the single row of 4.0-ft-high baffle piers permitted unimpeded flow to pass over the top of the baffles, which resulted in concentrated velocities and nonuniform flow distribution in the exit channel at the design discharge of 12,500 cfs. Therefore, the model investigation was directed toward improving the hydraulic performance of the original basin by varying the positions of the baffles.

Alternate stilling basin designs

34. Several alternate stilling basin designs were obtained by

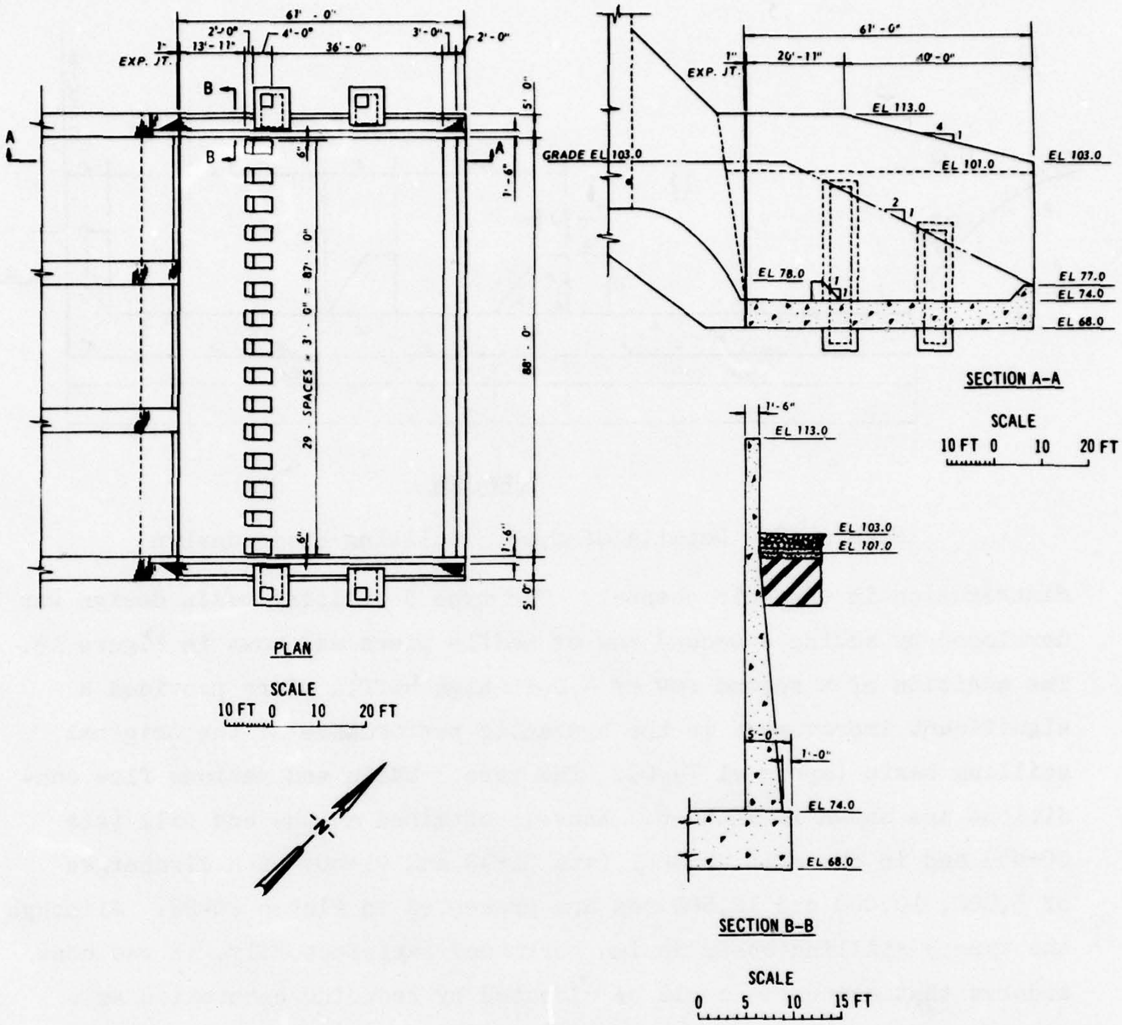
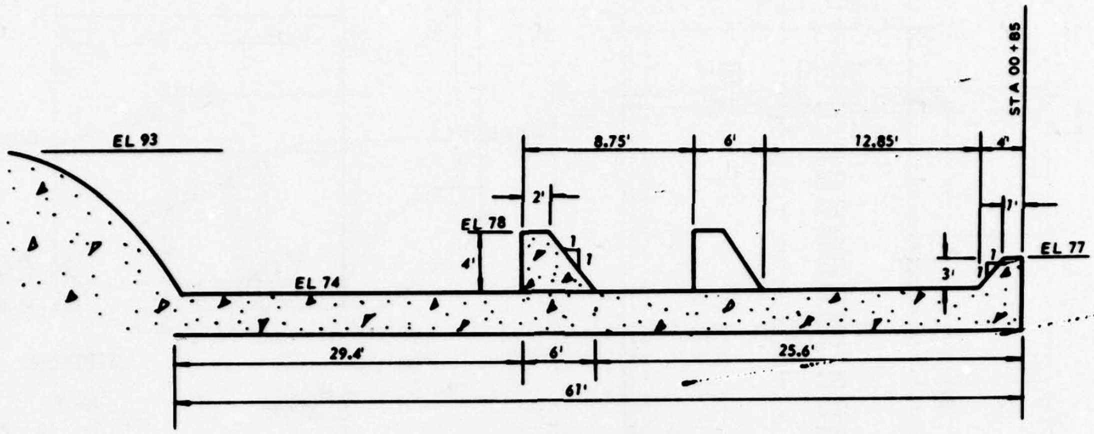


Figure 12. Plan and sections of original stilling basin (type 1)

varying the apron elevation and length. The type 2 stilling basin was developed by positioning the single row of 4.0-ft-high baffles 49.4 ft (1.5 D_2) downstream of the toe of the spillway. The theoretical depth (D_2) is the flow required to maintain a hydraulic jump in the stilling basin with a flow of 14,000 cfs, a headwater elevation of 110.3, and a tailwater elevation of 92.8. Isovels obtained at the end sill for discharges of 5,000, 10,000, and 12,500 cfs are presented in Plate 19. These results indicated only a slight reduction in the magnitude of velocities measured at the end sill and no improvement in flow



ELEVATION

Figure 13. Details of type 3 stilling basin design

distribution in the exit channel. The type 3 stilling basin design was developed by adding a second row of baffle piers as shown in Figure 13. The addition of a second row of 4.0-ft-high baffle piers provided a significant improvement in the hydraulic performance of the original stilling basin (apron el 74.0). The type 3 basin and various flow conditions are shown in Photo 6. Isovels obtained at the end sill (sta 00+85) and in the exit channel (sta 01+40 and 01+60) with discharges of 5,000, 10,000 and 12,500 cfs are presented in Plates 20-22. Although the type 3 stilling basin design performed satisfactorily, it was considered that economies could be effected by reducing excavation as a result of raising the apron elevation or decreasing the length of the horizontal apron. The type 5 stilling basin (Figure 14), which consists of 4.0-ft-high baffle piers and a 2-ft-high sloped end sill, was developed for a theoretical sequent depth (D_2) of 18.0 ft with a flow of 14,000 cfs and a headwater elevation of 110.3 ft. Various flow conditions in the type 5 stilling basin are shown in Photo 7. Isovels obtained at the end sill (sta 00+72) and in the exit channel (sta 01+40 and 01+69) with discharges of 5,000, 10,000, and 12,500 cfs are presented in Plates 23-25. Isovels obtained for submerged flows of 5,000 and 12,000 cfs are shown in Plates 26 and 27. Analysis of the test data indicated that reduction of the original apron length (61.0 ft)

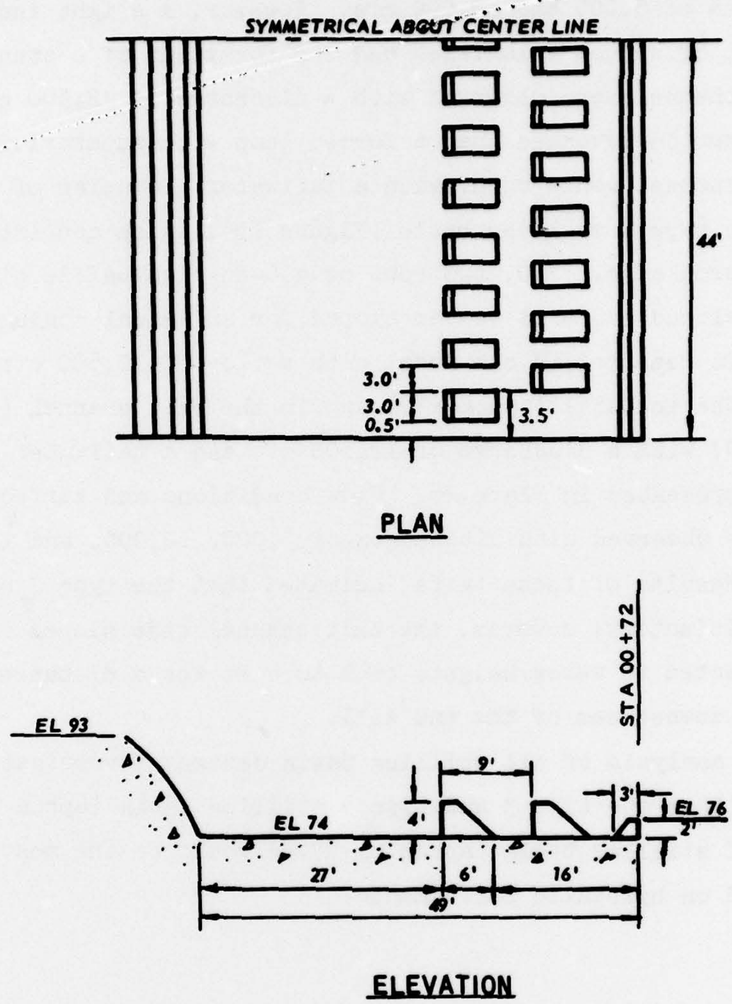


Figure 14. Type 5 stilling basin design

to a length of 49.0 ft would not significantly affect the overall hydraulic performance of the stilling basin and is therefore an adequate design length.

35. The type 6 stilling basin, which consists of a 49-ft-long apron at el 80.0, two rows of 4.0-ft-high baffle piers, and a 2-ft-high sloped end sill, was developed for an actual conjugate depth (D_1) of 4.1 ft measured in the model with a flow of 12,500 cfs, a headwater elevation of 108.9 ft, and a tailwater elevation of 92.8 ft. Analysis of the data indicated that the stilling basin performed satisfactorily

for discharges of 5,000 and 10,000 cfs. However, a slight increase in the magnitude of bottom velocities and the formation of a standing wave in the exit channel were observed with a discharge of 12,500 cfs. Further investigation revealed that a forced jump with supercritical flow in the exit channel would occur with a tailwater elevation of 90.8.

36. The type 7 stilling basin (Figure 15), which consists of a 61-ft-long apron at el 77.0, two rows of 4.0-ft-high baffle piers, and a 2-ft-high sloped end sill was developed for an actual conjugate depth (D_1) of 4.0 ft measured in the model with a flow of 12,500 cfs. Isovels obtained at the end sill (sta 00+79) and in the exit channel (sta 01+40 and sta 01+69) with a discharge of 12,500 cfs and a tailwater elevation of 92.8 are presented in Plate 28. Flow conditions and surface flow patterns were observed with discharges of 5,000, 10,000, and 12,500 cfs (Photo 8). Results of these tests indicated that the type 7 stilling basin is satisfactory; however, the exit channel side slopes will be subjected to waves heights of 1 to 2 ft for a distance of about 250 ft downstream of the end sill.

37. An analysis of all stilling basin designs investigated indicated that either the type 3 and type 5 stilling basin (apron el 74.0) or the type 7 stilling basin (apron el 77.0) would be the most satisfactory based on hydraulic performance.

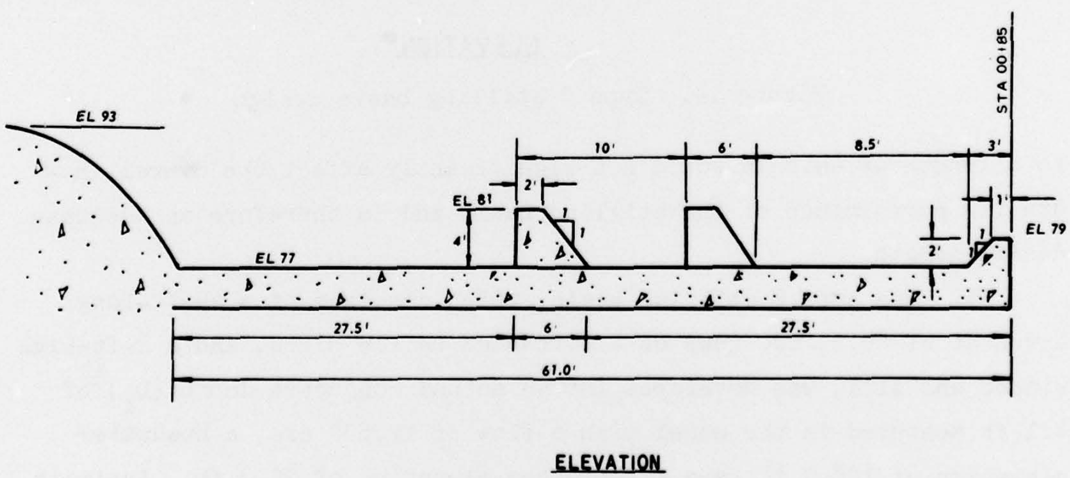


Figure 15. Details of type 7 stilling basin design

Stone Protection (Stilling Basin)

Exit channel

38. Tests were conducted in the 1:20-scale model to determine the riprap protection plan relative to various stilling basin designs with the recommended type 6 approach and type 3 abutment modifications to the three-bay gravity-flow section. The original riprap protection type 1 (Figure 2b and Plante 29) consists of a 30-ft length of riprap, with a maximum stone weight of 2,333 lb (d_{100} = 36 in.), and followed by a 54-ft length of riprap, with a maximum stone weight of 691 lb (d_{100} = 24 in.).

39. Results of riprap stability tests with the type 3 and type 5 stilling basin designs (apron length 61.0 ft) indicated that an 84-ft length of riprap with a maximum stone weight of 292 lb (d_{100} = 18 in.) will remain stable under expected flow conditions. Details of the type 5 stilling basin and type 2 riprap protection plan 1 are presented in Plate 30 and Photo 9.

40. The type 6 stilling basin design and type 3 riprap protection plan (Plate 31) indicated failure of the proposed riprap protection for a discharge of 12,500 cfs with a tailwater elevation of 90.8, 2 ft below minimum tailwater (Photo 10).

41. Riprap stability tests with the type 7 stilling basin design (Figure 15) indicated that the type 4 riprap protection plan consisting of 40 ft of riprap, with a maximum stone weight of 691 lb (d_{100} = 24 in.), and followed by a 49-ft length of riprap, with a maximum stone weight of 292 lb (d_{100} = 18 in.) was essential for exit channel protection. Details of the type 7 stilling basin and type 4 riprap protection plan are shown in Plate 32 and Photo 11.

42. It is considered that either the type 3 and type 5 stilling basin (apron el 74.0) and the type 2 riprap protection plan or the type 7 stilling basin (apron el 77.0) and the type 4 riprap protection plan would provide satisfactory stilling basin performance and stable riprap protection for the anticipated flow conditions through the gravity-flow section.

Pump Outlets

43. Tests were conducted to evaluate the hydraulic performance of the pump discharge outlets, exit channel, and riprap requirements for the exit channel. The hydraulic performance of the pump discharge outlets was satisfactory throughout the range of anticipated flow conditions.

44. The area downstream from the pump outlets was protected and tested with the original riprap design (d_{100} = 18 in., maximum stone weight 292 lb) as shown in Figure 16. Flow conditions resulting from

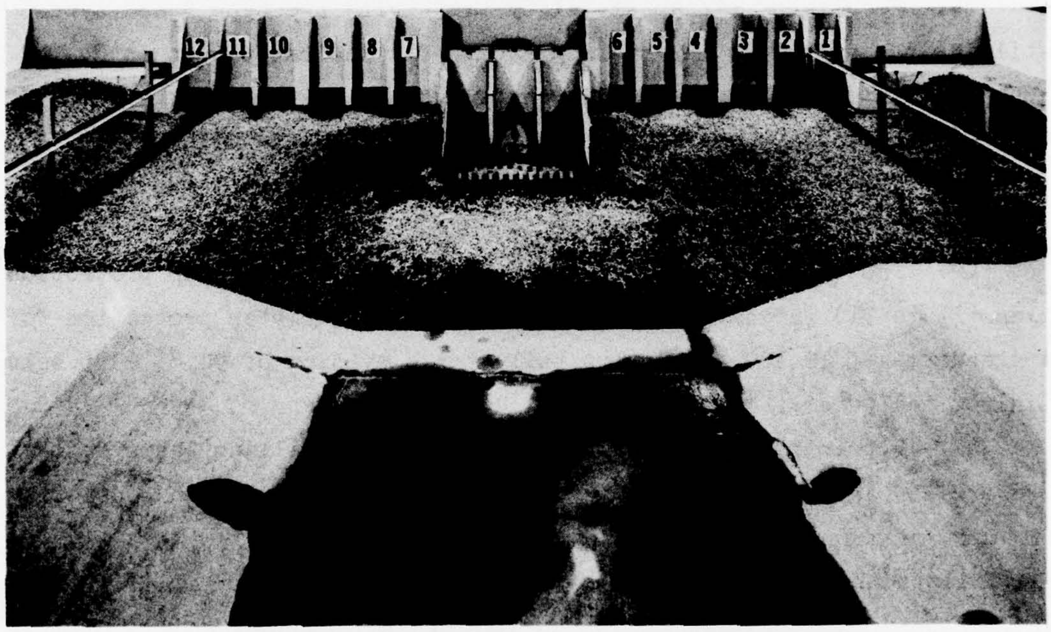


Figure 16. Original riprap design downstream from pump outlets; recommended design

various tailwater elevations and combinations of pumps operating are shown in Photos 12-17. Pumps are identified by numbers in Figure 16 and Photos 12-17. Surface currents are indicated by confetti. Photos 12-17 were taken at a 13-sec (prototype) exposure and the route and direction that each piece of confetti travels are indicated by streaks with a dot at the downstream end of each streak. As the tailwater elevation was increased, flows emerging from the pump outlets became more tranquil. Maximum velocities measured 1 ft above the bottom for the minimum anticipated tailwater elevation are shown in Plates 33 and 34.

45. The riprap (d_{100} = 18 in.) was stable downstream from the pump outlets for all anticipated flow conditions. To determine the point of riprap failure with all pumps operating, the tailwater elevation was lowered below the minimum anticipated (110.0 ft msl) in increments of 0.5 ft. Initial riprap failure was observed immediately in front of the discharge pump outlets at a tailwater elevation of 108.0, indicating a slight safety factor.

PART IV: DISCUSSION

46. Flow instability at the gravity-flow abutments was improved and the flow capacity through the section was increased by excavating a portion of the approach channel immediately upstream of the gravity-flow section and streamlining the abutments. The type 6 approach configuration with the type 3 abutment was the most effective design without extensive modification to the approach channel and is therefore the recommended design.

47. Results of tests and data analysis for both free uncontrolled and submerged uncontrolled flows indicate that the discharge coefficients applicable to these two flow conditions can be described in terms of dimensionless parameters involving head and tailwater.

48. The original design stilling basin had one row of baffle piers located near the toe of the crest. Unimpeded flow passed over the top of the baffle piers and created concentrated currents in the exit channel. The stilling basin performance was improved by moving the baffle piers downstream (27 ft from toe of spillway to face of the baffle piers) and adding a second row of baffle piers. Tests also indicated that the apron length could be reduced from 61 to 49 ft without significantly affecting stilling basin performance. Although the type 5 design was recommended based on these model tests, the type 3 design was chosen by the sponsor for the prototype construction because of other design considerations. The type 3 basin was 61 ft long at el 74.0, as was the original design, having two rows of 4-ft-high baffle piers located 29.4 ft and 38.15 ft, respectively, from the toe of the crest.

49. Results of tests with flow through the gravity-flow section to determine the most feasible riprap protection plan in the exit channel downstream from either the type 3 or type 5 stilling basins indicated that an 84-ft length of riprap with a maximum stone weight of 292 lb (d_{100} = 18 in.) would be stable for the anticipated range of flow conditions.

50. The hydraulic performance of the pump discharge outlets was satisfactory throughout the range of anticipated flow conditions.

Riprap protection (d_{100} = 18 in., maximum stone weight = 292 lb) downstream from the pump outlets was stable for various tailwater elevations and combinations of pumps operating. The riprap failed when the tailwater was lowered to el 108.0, 2.5 ft below the minimum anticipated.

51. An analysis of the results obtained with the recommended gravity-flow and pumping discharge riprap protection plans indicates a maximum stone weight of 292 lb (d_{100} = 18 in.) to ensure stability of the structure for the anticipated flow conditions.

Table 1

Water-Surface Elevations

Discharge 12,500 cfs, Tailwater El 92.8

Station	Elevations of Bay 3, ft msl Right Side	Center Line	Left Side
	Original Type 1 Approach and Type 1 Abutment Headwater El 112.04		
1+90	112.0	112.0	112.0
1+80	112.0	111.9	111.9
1+70	111.8	111.7	111.7
1+60	111.4	111.2	111.3
1+50	109.5	110.1	110.7
1+40	103.2	108.5	110.9
1+30	101.4	106.0	105.5
1+20	100.6	98.1	98.7
1+10	100.8	97.6	97.7
1+00	99.6	97.8	97.2
0+90	98.7	98.9	96.9
0+80	98.2	99.5	98.0
0+70	98.4	98.3	98.0
0+60	98.4	97.7	100.5
0+50	98.2	97.9	100.9
0+40	--	--	--
0+30	97.9	100.1	97.8
0+20	99.4	98.4	97.9
0+10	99.3	96.2	96.8
0+00	94.3	89.5	90.6
0+10	87.3	87.3	87.2
0+20	88.6	88.6	88.8
0+30	89.6	90.2	91.1
0+40	90.2	90.9	91.1
0+50	92.0	92.2	92.4
	Original Type 1 Approach and Type 2 Abutment Headwater El 112.04		
1+90	111.8	111.7	111.7
1+80	111.5	111.4	111.5

(Continued)

Note: Sta 0+00 is located at center line of spillway crest.

Table 1 (Continued)

Station	Elevations of Bay 3, ft msl Right Side	Center Line	Left Side
	Original Type 1 Approach and Type 2 Abutment Headwater El 112.04 (Continued)		
1+70	111.4	111.4	111.4
1+60	111.0	110.9	110.9
1+50	109.3	109.9	110.4
1+40	104.1	108.2	110.6
1+30	102.1	104.0	105.5
1+20	100.1	101.5	101.7
1+10	99.8	98.2	99.0
1+00	100.4	98.0	98.0
0+90	99.5	98.0	97.5
0+80	--	--	--
0+70	98.2	99.4	97.5
0+60	98.4	98.4	98.5
0+50	98.5	97.9	100.2
0+40	98.4	98.0	99.5
0+30	--	--	--
0+20	98.4	99.0	98.0
0+10	98.5	98.0	97.5
0+00	98.7	96.7	97.0
0+10	93.9	89.7	91.6
0+20	87.2	87.5	87.2
0+30	88.4	88.5	88.5
0+40	89.6	89.8	90.3
0+50	90.5	90.8	91.3
	Type 2 Approach and Type 2 Abutment Headwater El 108.7		
1+90	108.0	102.9	102.8
1+80	107.8	102.6	102.5
1+70	107.2	101.9	101.8
1+60	106.4	101.2	101.2
1+50	102.8	100.3	102.0
1+40	98.3	99.7	100.0
1+30	97.2	98.8	98.1
1+20	97.4	95.9	96.5
1+10	96.7	95.0	95.5
1+00	95.9	95.9	94.9
0+90	95.4	95.9	94.7
0+80	--	--	--
0+70	95.5	94.9	96.3

(Continued)

Table 1 (Continued)

Station	Elevations of Bay 3, ft msl Right Side	Center Line	Left Side
	Type 2 Approach and Type 2 Abutment Headwater El 108.7 (Continued)		
0+60	95.3	95.4	95.9
0+50	95.5	96.1	94.9
0+40	96.1	96.0	95.0
0+30	--	--	--
0+20	100.2	100.8	101.2
0+10	99.5	100.6	100.4
0+00	97.0	98.4	98.3
0+10	91.2	91.1	91.8
0+20	87.3	87.4	87.4
0+30	87.5	87.5	87.5
0+40	89.9	90.0	90.1
0+50	91.6	91.6	91.6
0+60	92.0	92.2	92.2
	Type 3 Approach and Type 2 Abutment Headwater El 107.6		
1+90	107.1	107.0	107.2
1+80	107.2	107.0	107.1
1+70	107.0	107.0	107.0
1+60	106.7	106.6	106.7
1+50	104.4	105.7	106.4
1+40	100.6	104.7	102.0
1+30	99.6	98.6	97.5
1+20	98.6	97.1	96.7
1+10	96.9	96.4	96.7
1+00	96.6	96.8	97.0
0+90	98.2	97.7	98.6
0+80	--	--	--
0+70	98.5	98.7	99.8
0+60	98.6	98.5	97.9
0+50	98.0	97.5	96.9
0+40	97.0	97.2	97.1
0+30	--	--	--
0+20	101.8	101.7	101.6
0+10	100.3	100.6	100.4
0+00	98.1	98.4	98.5
0+10	92.8	92.4	93.3
0+20	90.3	89.9	89.5
0+30	90.1	89.9	90.0

(Continued)

Table 1 (Continued)

Station	Elevations of Bay 3, ft msl Right Side	Center Line	Left Side
	Type 3 Approach and Type 2 Abutment Headwater El 107.6 (Continued)		
0+40	91.4	92.2	91.9
0+50	92.6	92.6	92.3
0+60	92.9	92.9	92.9
	Type 4 Approach and Type 2 Abutment Headwater El 107.9		
1+90	107.5	107.5	107.4
1+80	107.4	107.4	107.4
1+70	107.2	107.2	107.2
1+60	106.8	106.8	106.8
1+50	104.9	105.9	106.6
1+40	103.6	104.3	106.6
1+30	102.2	103.2	102.8
1+20	103.3	100.8	101.4
1+10	102.1	100.5	100.6
1+00	100.8	101.8	100.3
0+90	100.5	101.4	101.0
0+80	--	--	--
0+70	104.3	104.2	103.3
0+60	104.2	104.9	104.3
0+50	103.0	102.7	103.6
0+40	102.0	102.0	102.2
0+30	--	--	--
0+20	101.4	101.5	101.4
0+10	100.1	100.6	101.2
0+00	98.4	98.5	98.3
0+10	91.5	91.3	91.0
0+20	87.5	87.6	87.6
0+30	88.2	88.2	88.0
0+40	89.8	89.9	90.2
0+50	91.4	91.5	91.5
0+60	92.2	92.2	92.3
	Type 6 Approach and Type 3 Abutment Headwater El 108.92		
1+90	108.3	108.3	108.1
1+80	108.1	107.9	107.8
1+70	107.7	107.4	107.2
1+60	106.9	106.7	106.7

(Continued)

Table 1 (Concluded)

Station	Elevations of Bay 3, ft msl Right Side	Center Line	Left Side
	Type 6 Approach and Type 3 Abutment Headwater El 108.92 (Continued)		
1+50	103.8	105.3	106.5
1+40	98.5	102.8	107.3
1+30	100.4	103.0	102.9
1+20	102.2	100.6	100.9
1+10	102.2	99.5	99.7
1+00	101.1	99.8	99.0
0+90	99.9	100.8	98.9
0+80	--	--	--
0+70	99.7	99.3	101.3
0+60	99.9	99.5	101.3
0+50	100.0	100.5	100.1
0+40	101.1	101.0	99.3
0+30	--	--	--
0+20	100.5	100.1	101.3
0+10	98.9	100.5	101.1
0+00	97.0	98.4	98.1
0+10	90.7	92.7	91.3
0+20	85.5	85.6	85.9
0+30	87.0	87.1	87.0
0+40	90.8	90.8	90.6
0+50	93.1	93.1	93.1
0+60	93.7	93.5	93.7

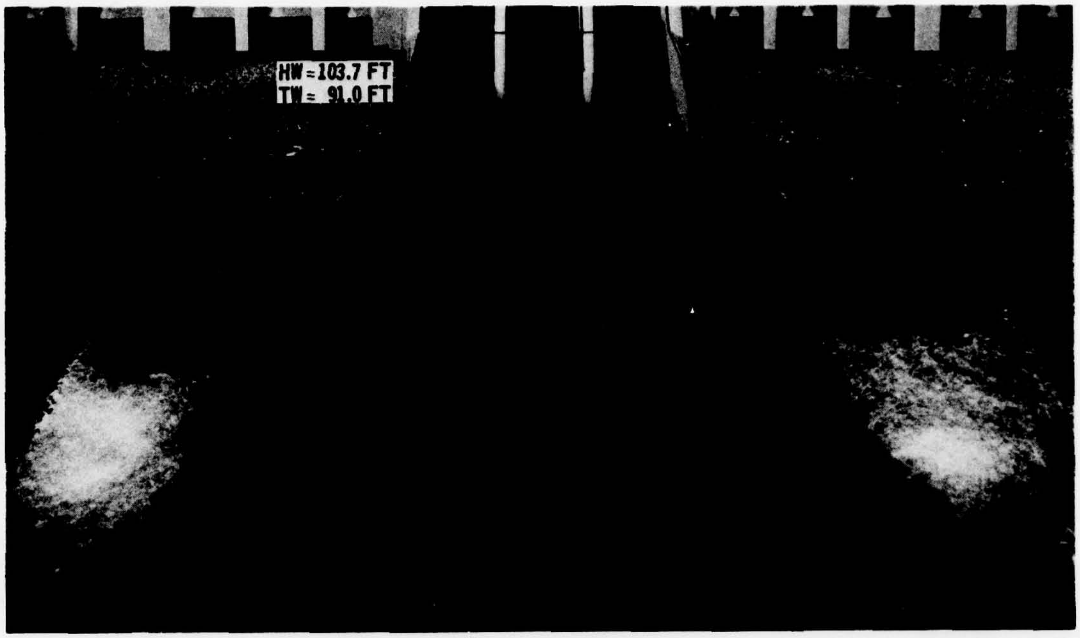
Table 2

Basic Uncontrolled Spillway Rating

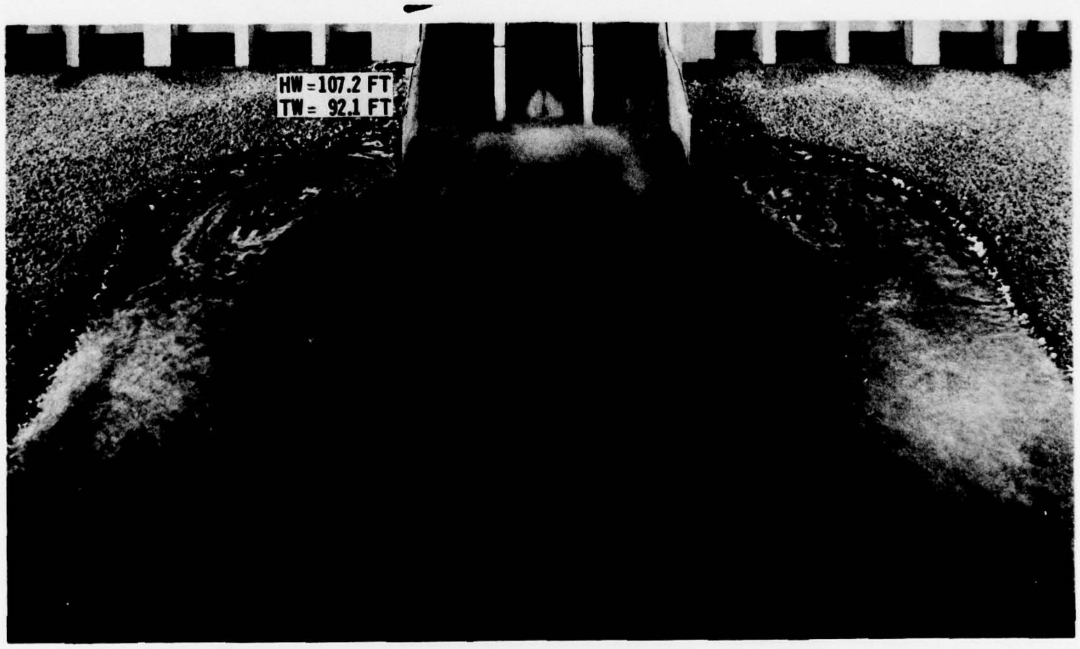
Data Obtained from Model

Discharge cfs	Pool Elevation ft msl	Total Head on Crest, ft*
3,000	102.04	4.04
5,000	103.78	5.78
7,000	105.27	7.27
9,000	106.49	8.49
10,000	107.29	9.29
11,000	107.80	9.80
12,000	108.70	10.70
13,000	109.50	11.50

Note: Approach el 98.0 and gravity-flow control el 98.0.
* Total head on crest, ft (based on depth of flow and velocity head in approach channel).



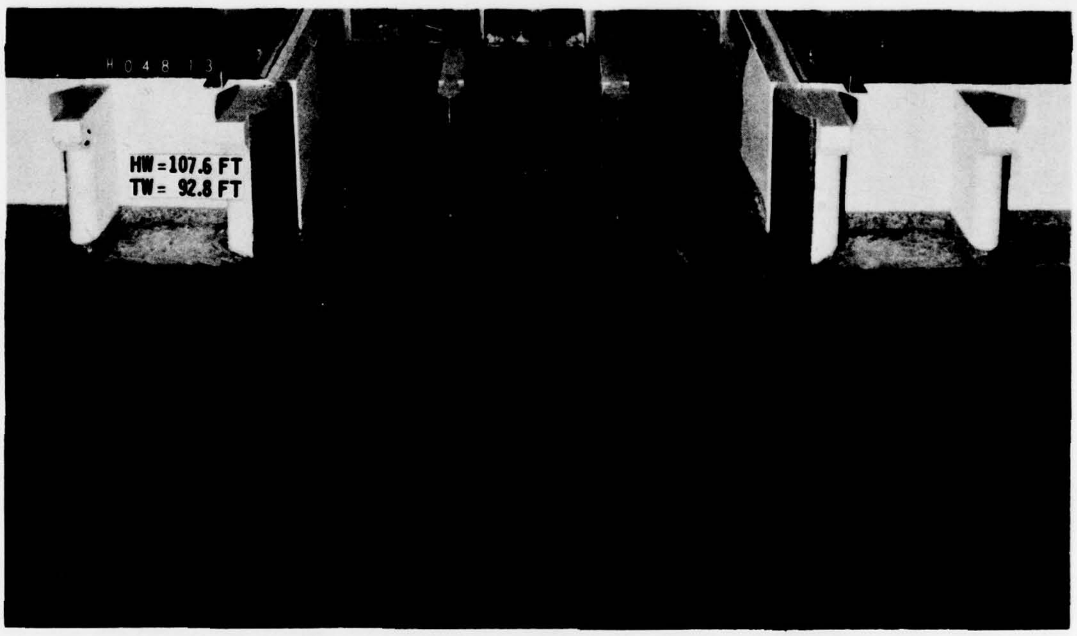
a. Discharge 5,000 cfs



b. Discharge 12,500 cfs

Photo 1. Approach flow conditions with type 2 approach and type 2 abutment

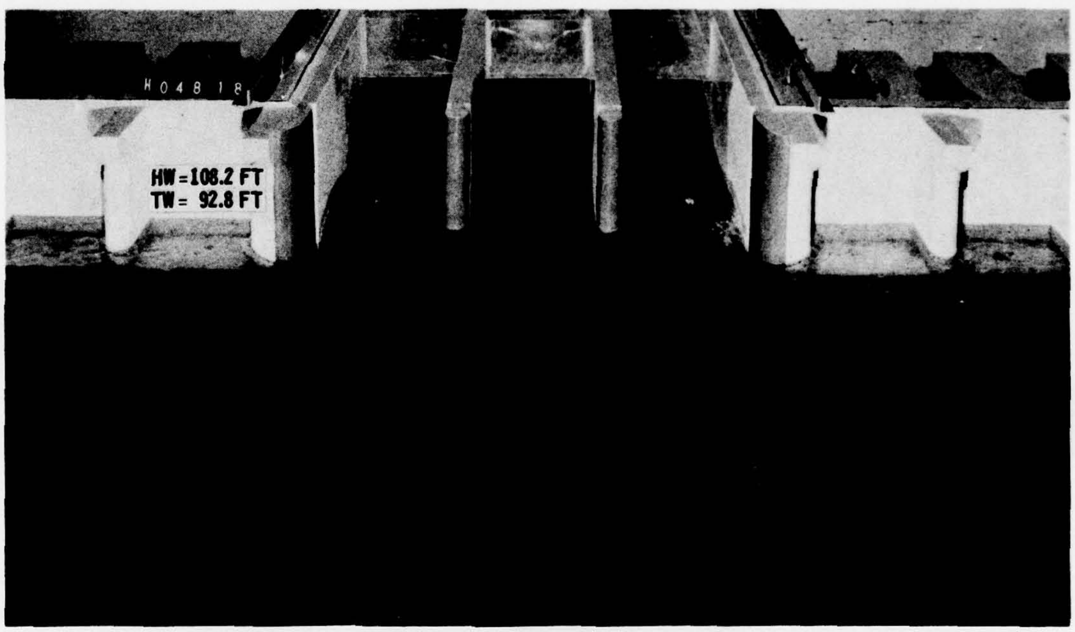
a. Discharge 5,000 cfs



b. Discharge 12,500 cfs

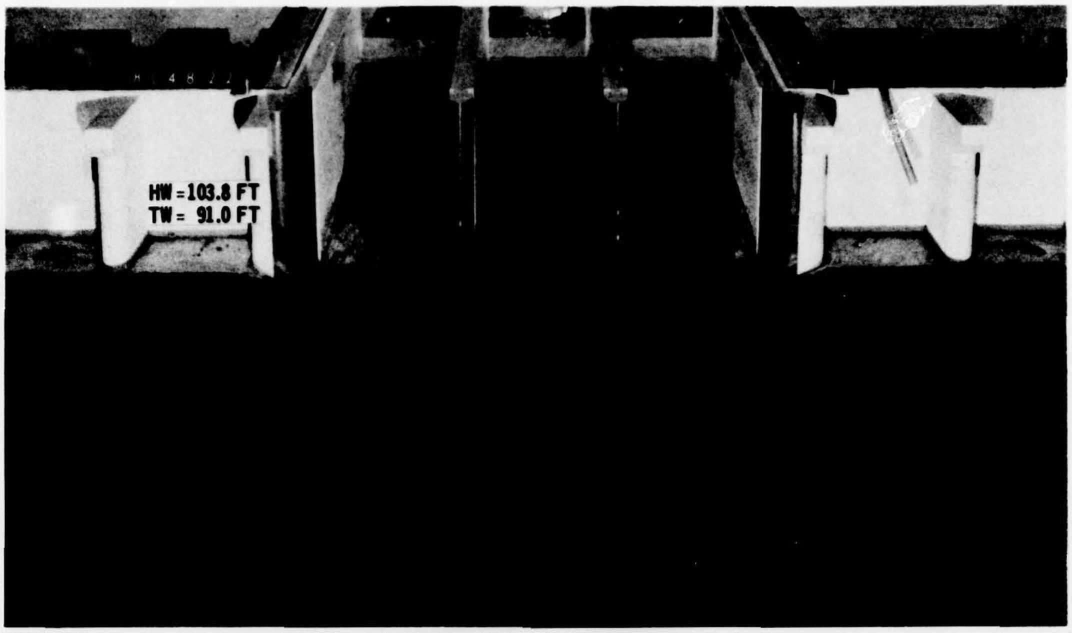
Photo 2. Approach flow conditions with type 3 approach and type 2 abutment

a. Discharge 5,000 cfs

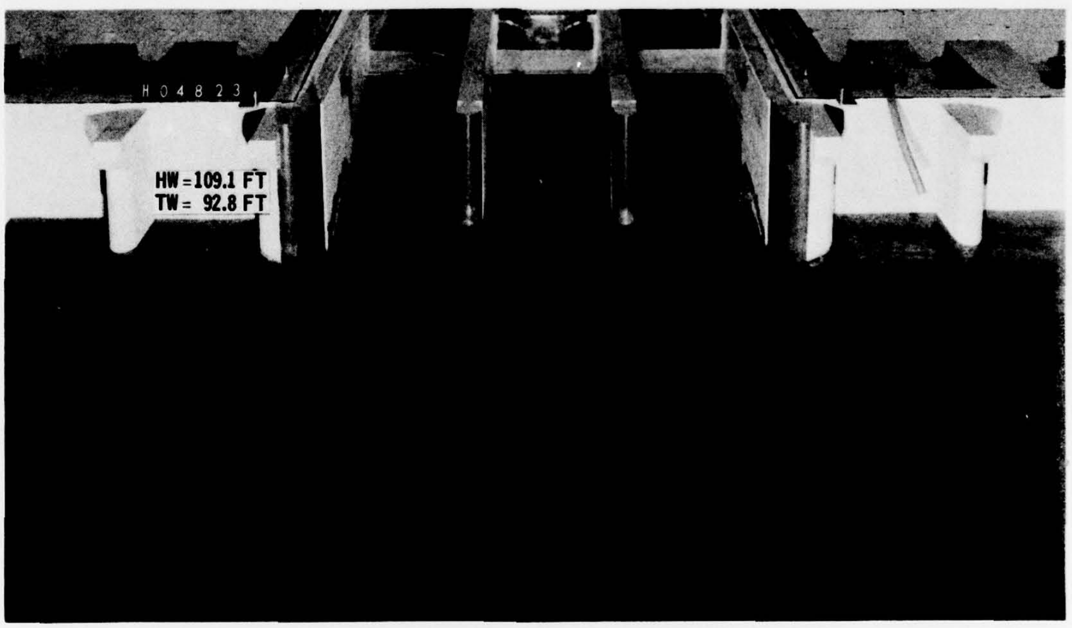


b. Discharge 12,500 cfs

Photo 3. Approach flow conditions with type 4 approach and type 2 abutment



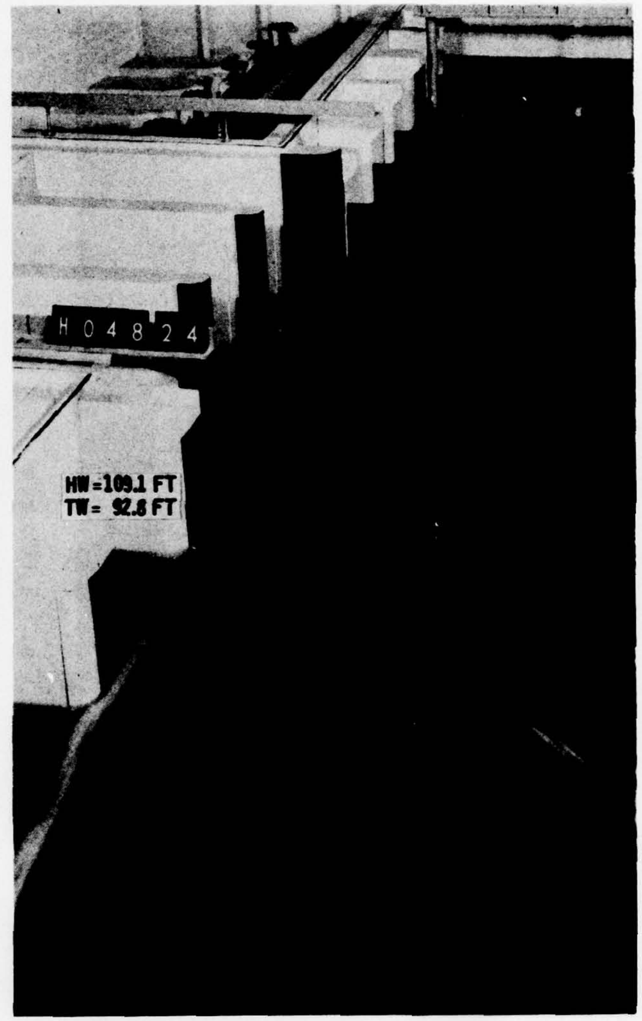
a. Discharge 5,000 cfs



b. Discharge 12,500 cfs

Photo 4. Approach flow conditions with type 6 approach and type 3 abutment; recommended design (sheet 1 of 2)

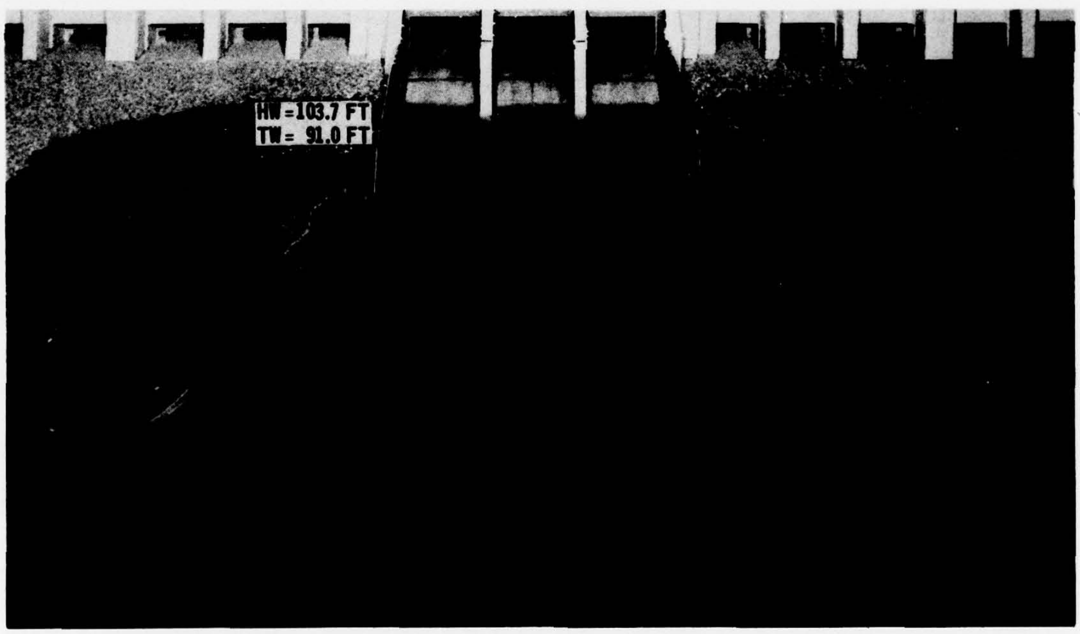
c. Discharge 5,000 cfs, 9-sec exposure



d. Discharge 12,500 cfs, 9-sec exposure

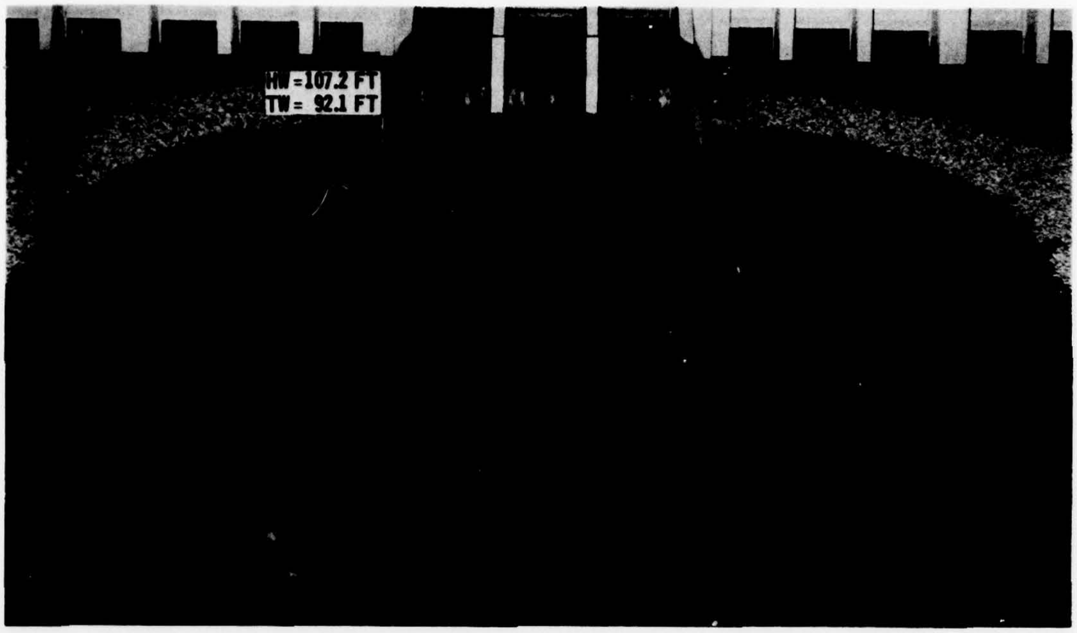
Photo 4 (sheet 2 of 2)

a. 5,000 cfs

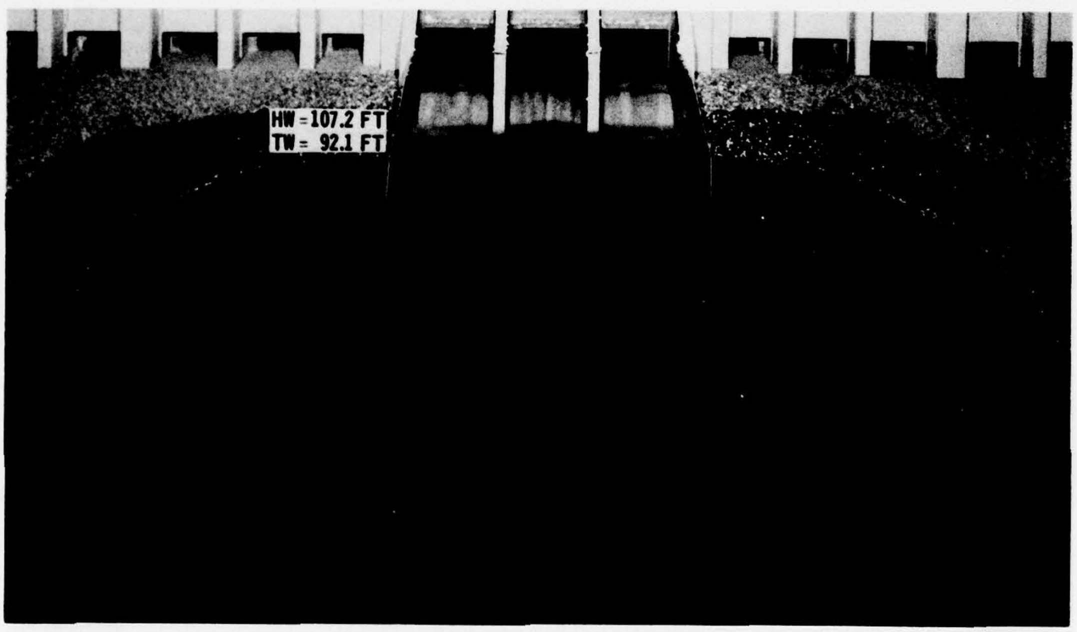


b. 5,000 cfs, 9-sec exposure

Photo 5. Flow conditions in the exit channel with the original stilling basin, type 6 approach, and type 3 abutment (sheet 1 of 3)

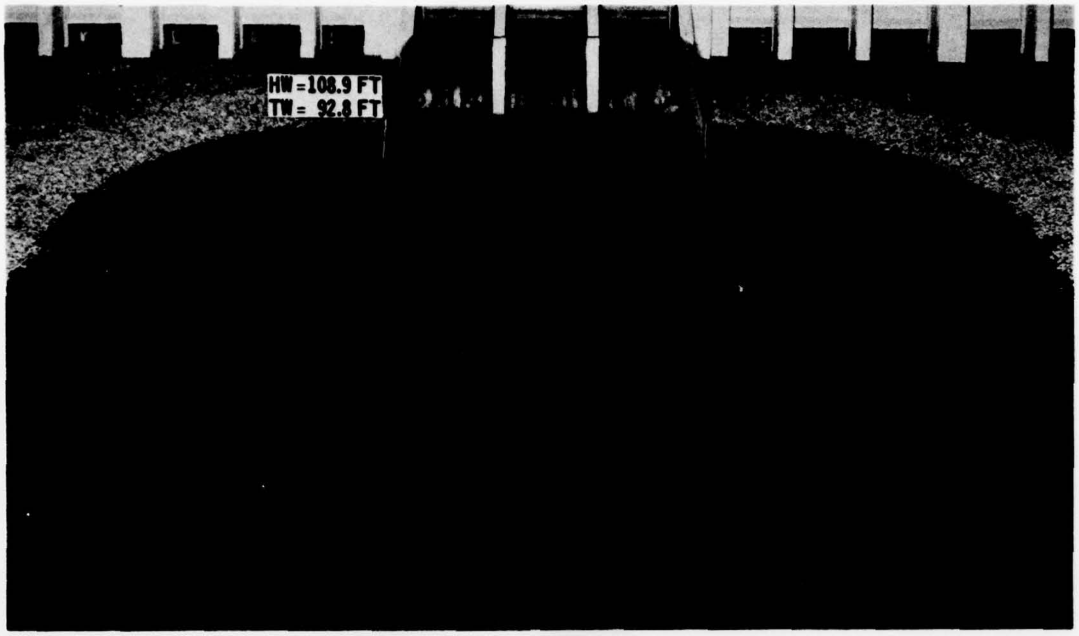


c. 10,000 cfs

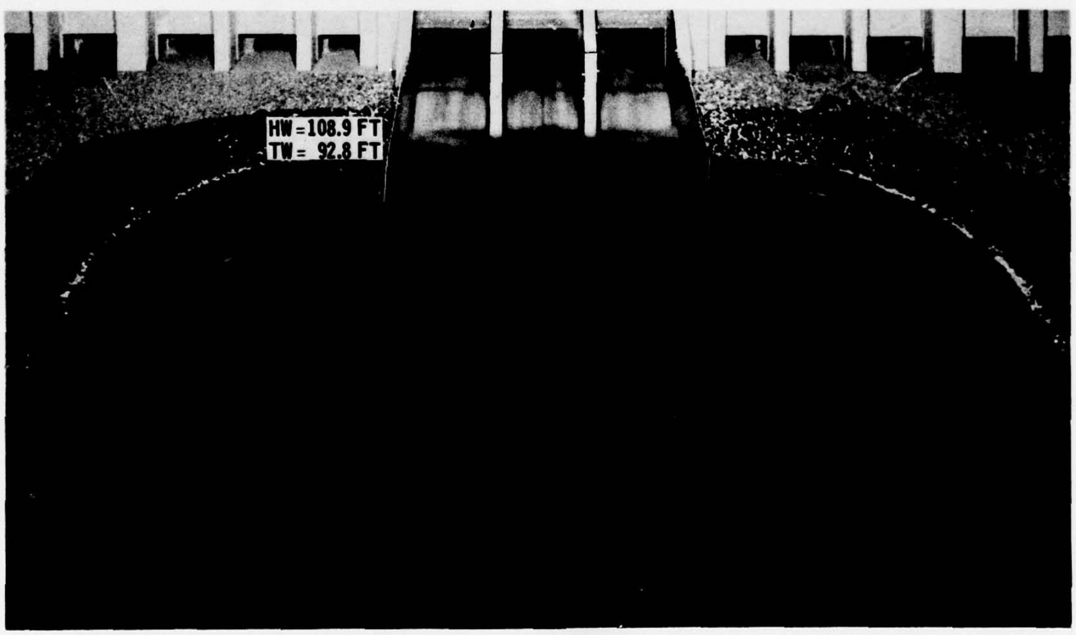


d. 10,000 cfs, 9-sec exposure

Photo 5 (sheet 2 of 3)

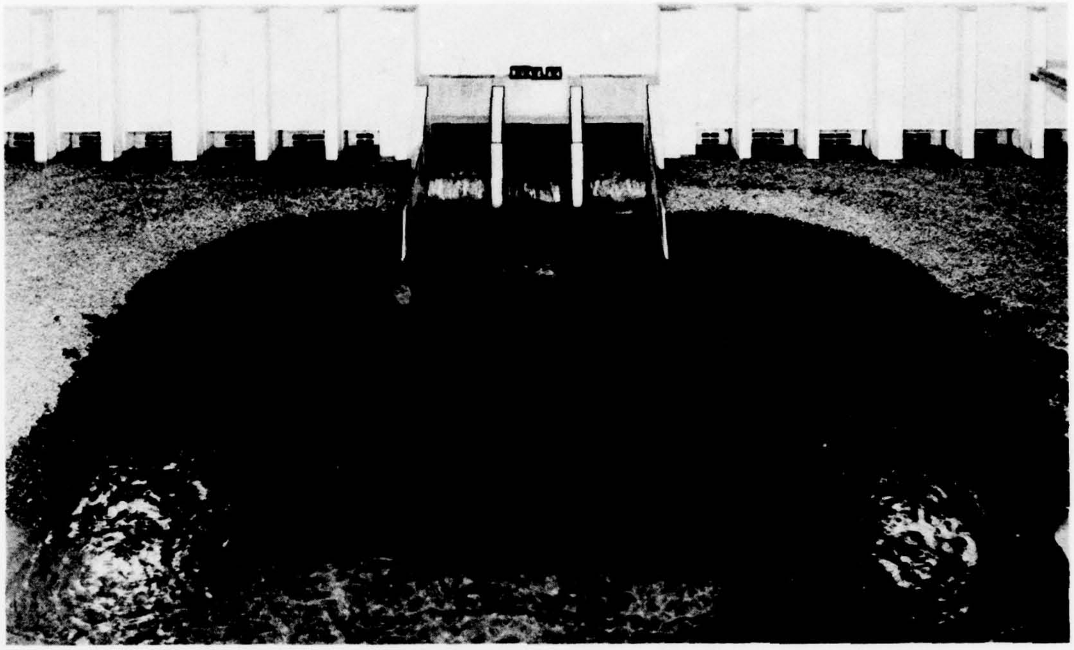


e. 12,500 cfs

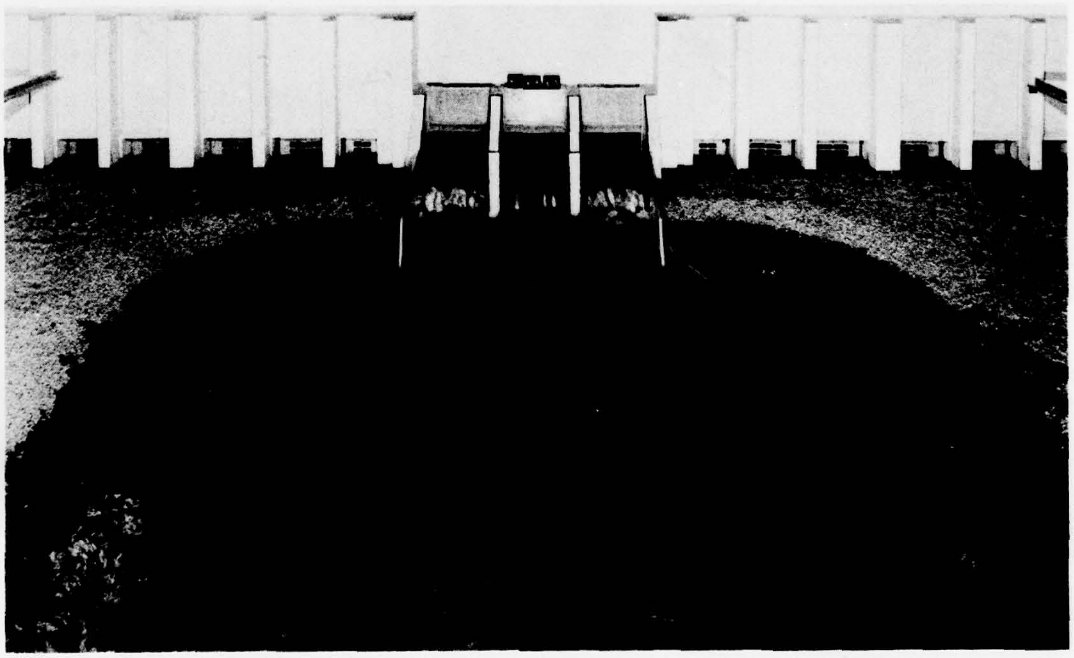


f. 12,500 cfs, 9-sec exposure

Photo 5 (sheet 3 of 3)

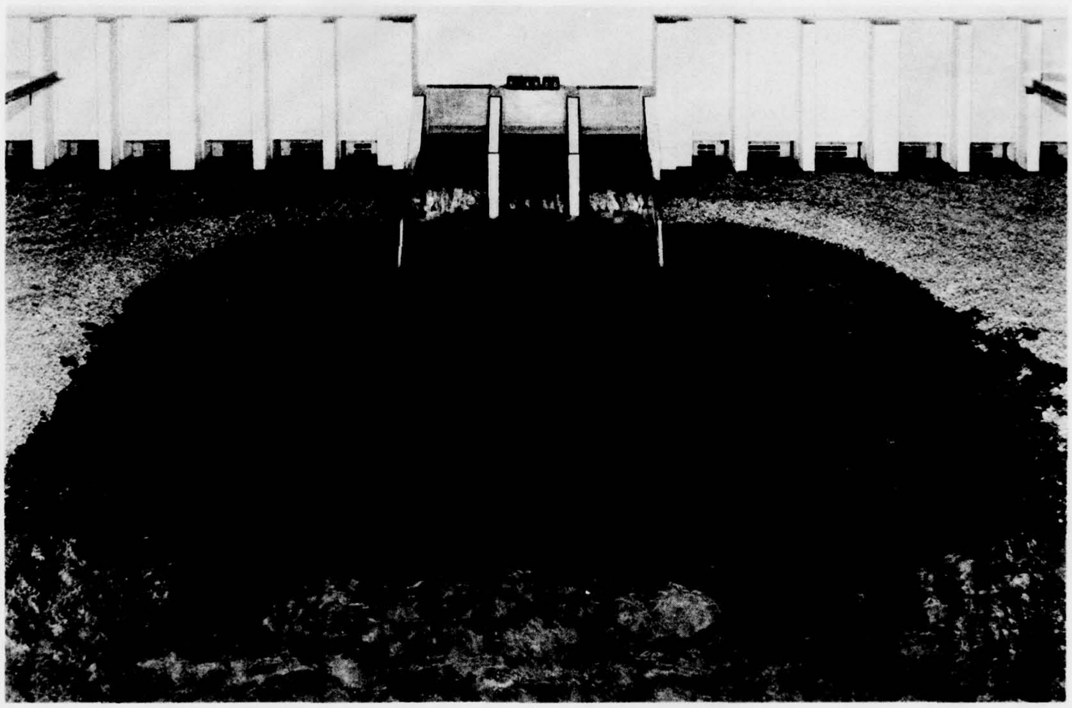


a. Discharge 5,000 cfs, tailwater el 91.0



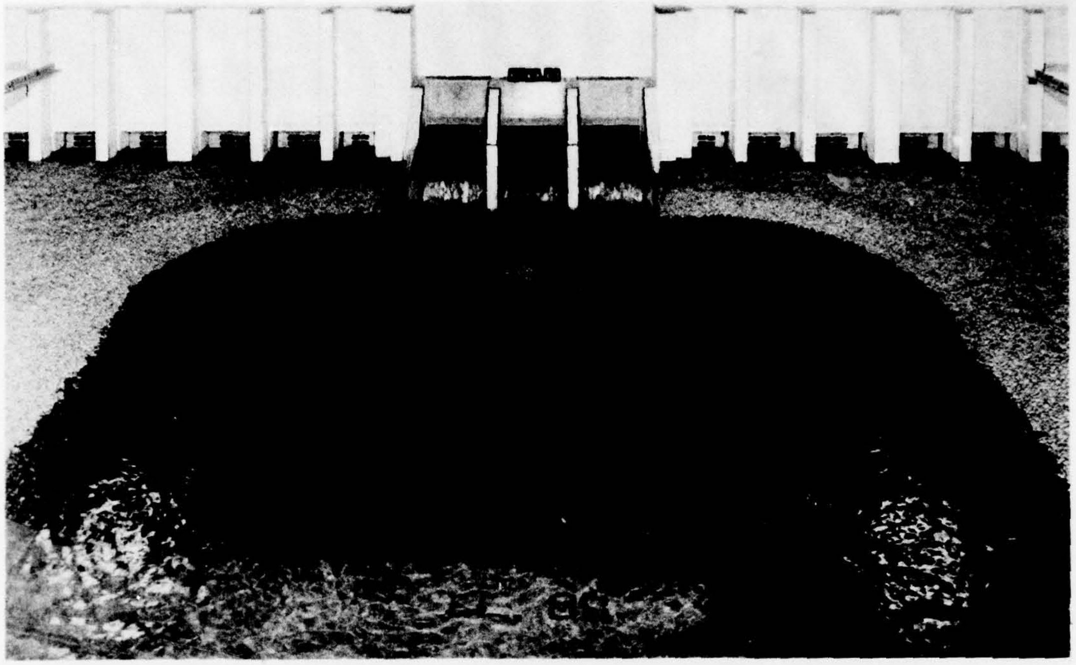
b. Discharge 10,000 cfs, tailwater el 92.1

Photo 6. Flow conditions in the exit channel with type 6 approach, type 3 abutment, and type 3 stilling basin (sheet 1 of 2)

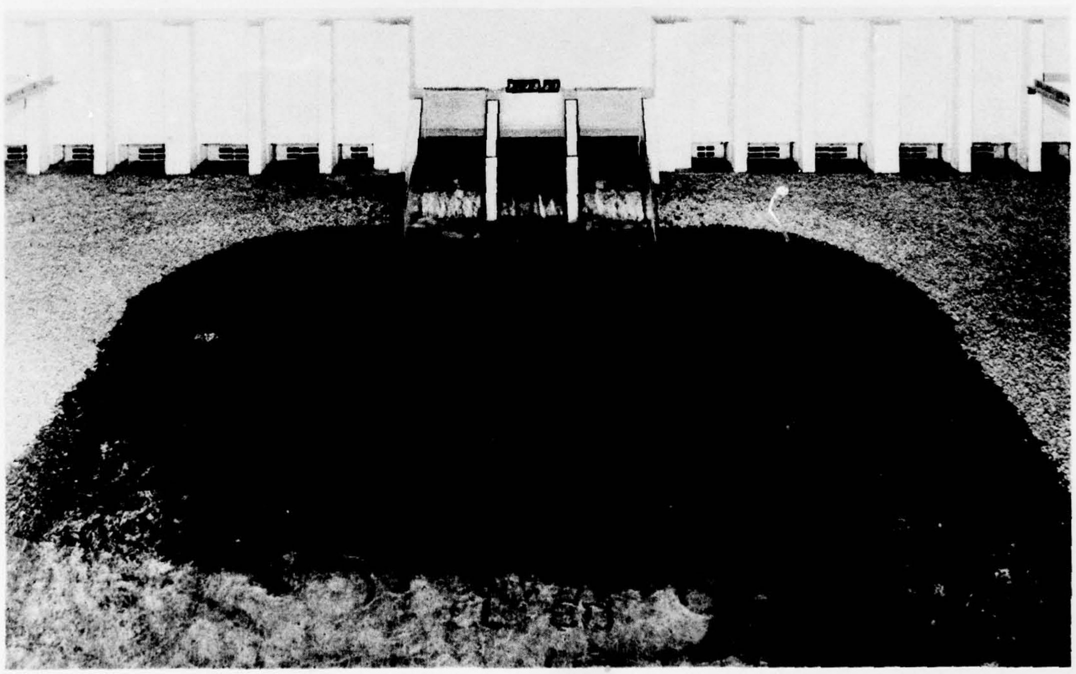


c. Discharge 12,500 cfs, tailwater el 92.8

Photo 6 (sheet 2 of 2)

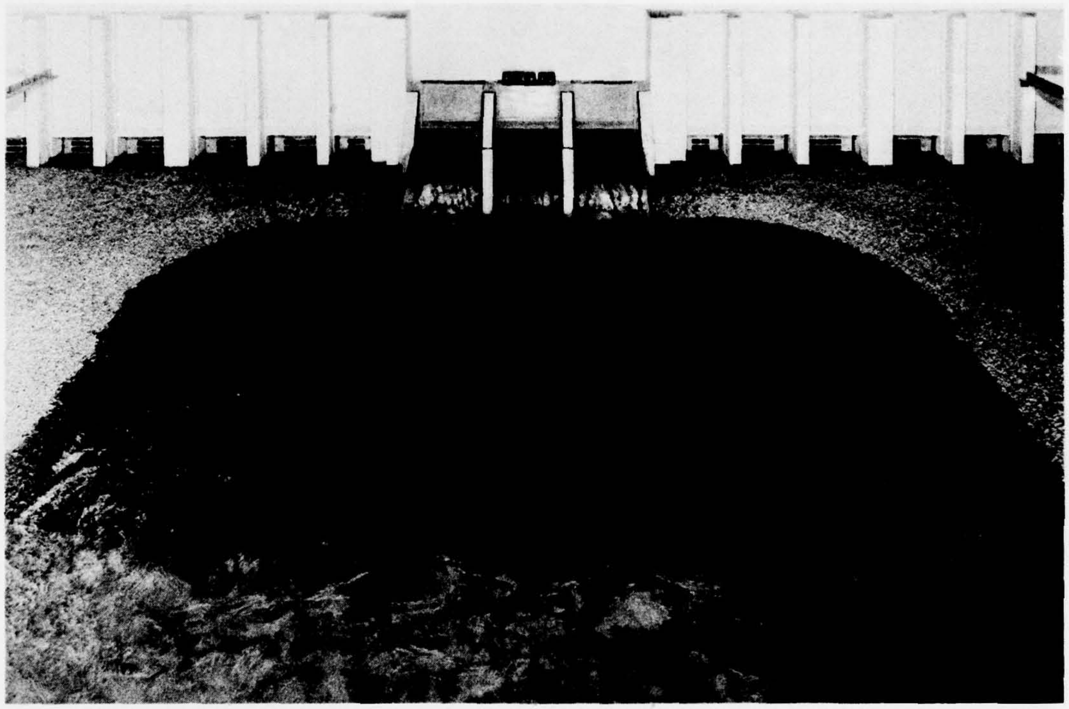


a. Discharge 5,000 cfs, tailwater el 91.0



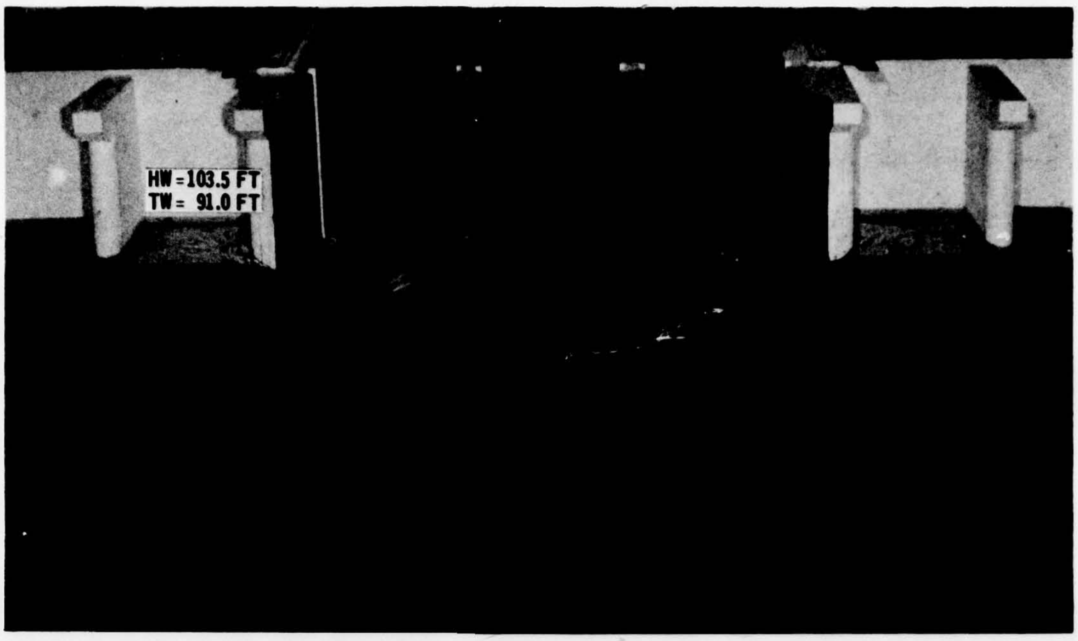
b. Discharge 10,000 cfs, tailwater el 92.1

Photo 7. Flow conditions in the exit channel with type 6 approach, type 3 abutment, and type 5 stilling basin (sheet 1 of 2)

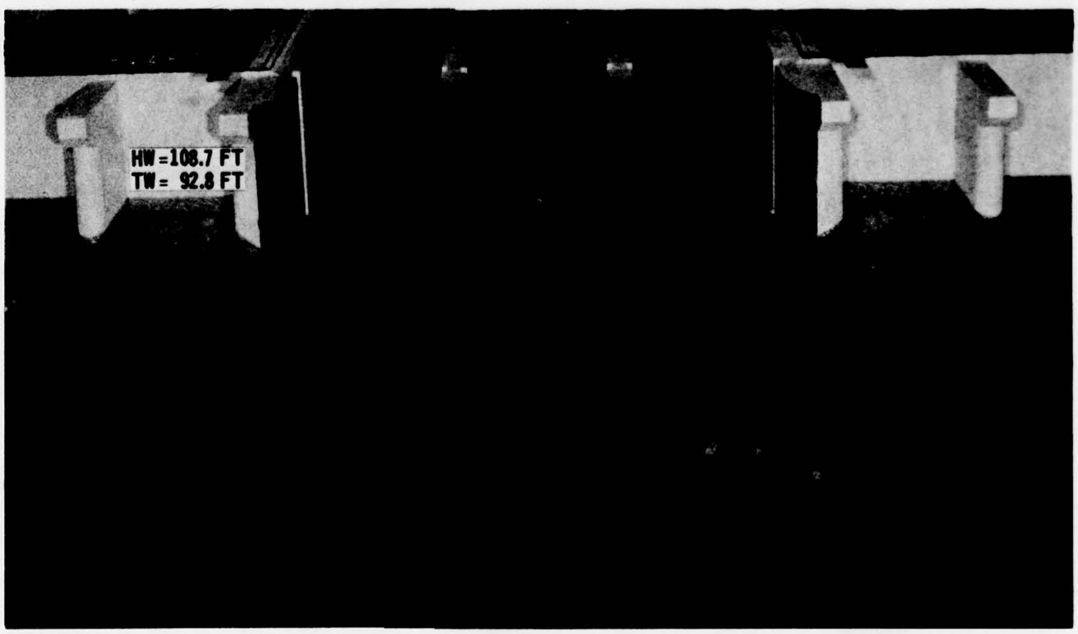


c. Discharge 12,500 cfs, tailwater el 92.8

Photo 7 (sheet 2 of 2)

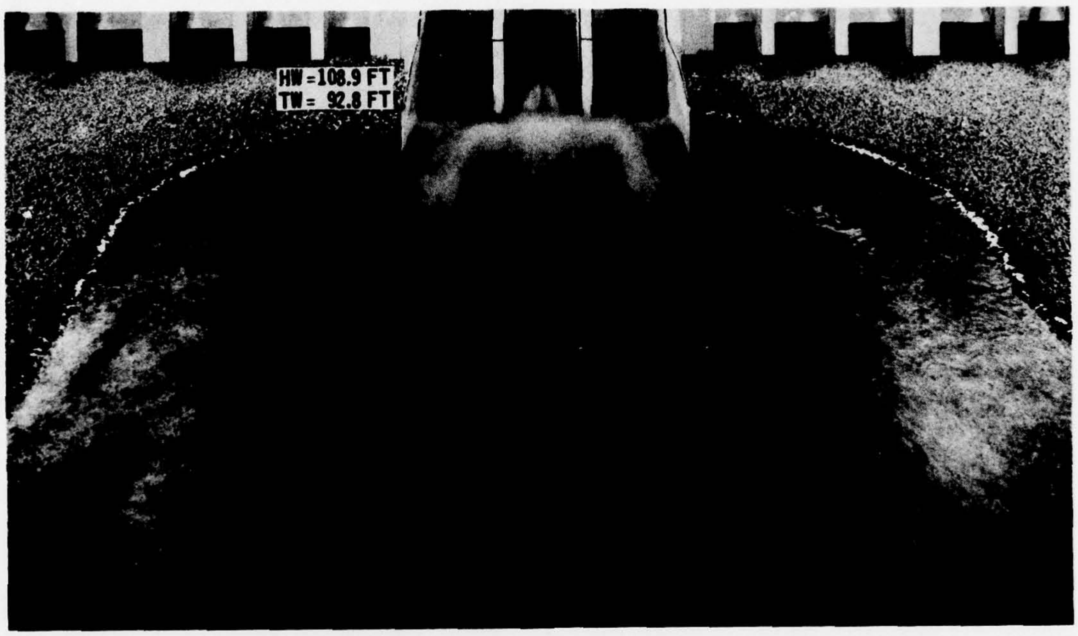


a. Discharge 5,000 cfs



b. Discharge 10,000 cfs

Photo 8. Flow conditions with type 7 stilling basin and type 4 riprap protection plan (sheet 1 of 2)



c. Discharge 12,500 cfs

Photo 8 (sheet 2 of 2)

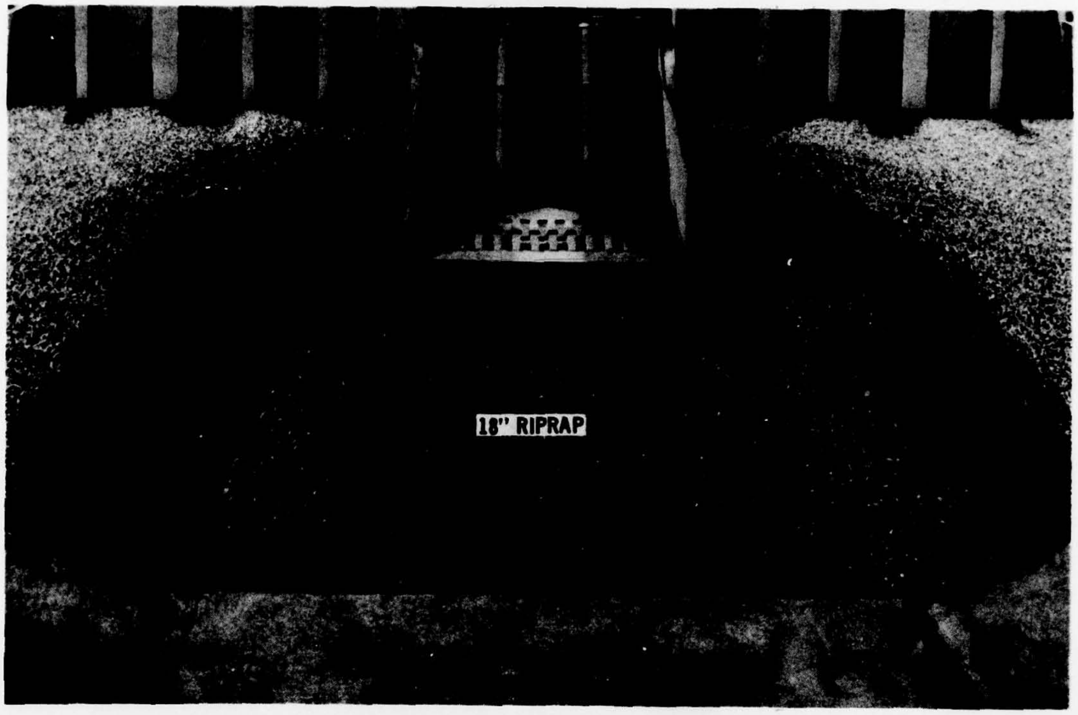


Photo 9. Type 5 stilling basin and type 2 riprap protection plan for gravity-flow section; recommended design

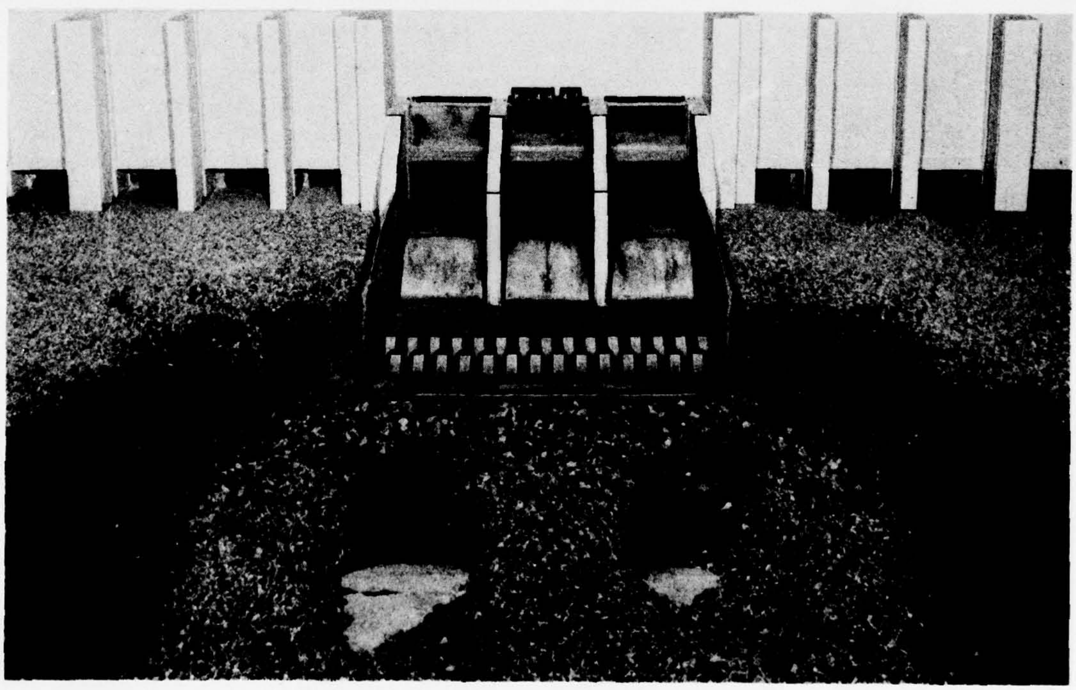


Photo 10. Failure of type 6 stilling basin and type 3 riprap protection plan for gravity-flow section, discharge 12,500 cfs, tailwater el 91.0 ft

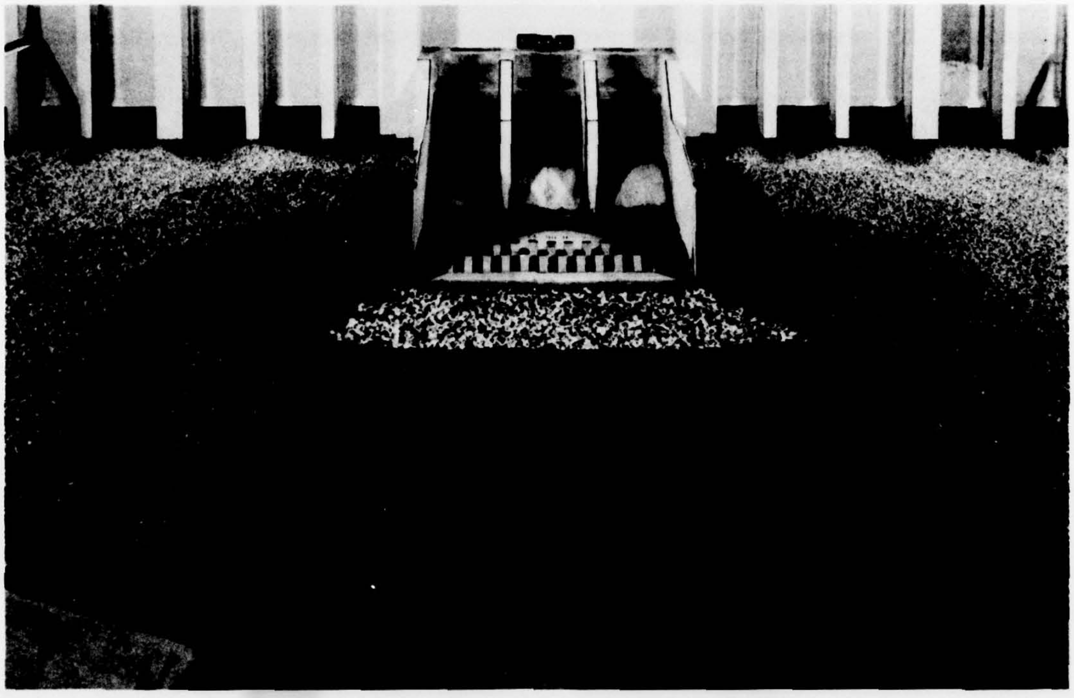


Photo 11. Type 7 stilling basin and type 4 riprap protection plan for gravity-flow section

Photo 12. Pumps operating: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12. Discharge per pumps: 6 and 7, 250 cfs; 1, 2, 3, 4, 5, 8, 9, 10, 11, 12, 600 cfs; tailwater el 110 ft

Photo 13. Pumps operating: 7, 8, 9, 10, 11, 12. Discharge per pump 7, 250 cfs; pumps 8, 9, 10, 11, 12, 600 cfs; tailwater el 110 ft

Photo 14. Pumps operating: 2, 4, 6, 7, 9, 11. Discharge per pumps 6 and 7, 250 cfs; pumps 2, 4, 9, 11, 600 cfs; tailwater el 110 ft

Photo 15. Pumps operating: 2, 4, 6, 7, 9, 11. Discharge per pumps 6 and 7, 250 cfs; pumps 2, 4, 9, 11, 600 cfs; tailwater el 620 ft

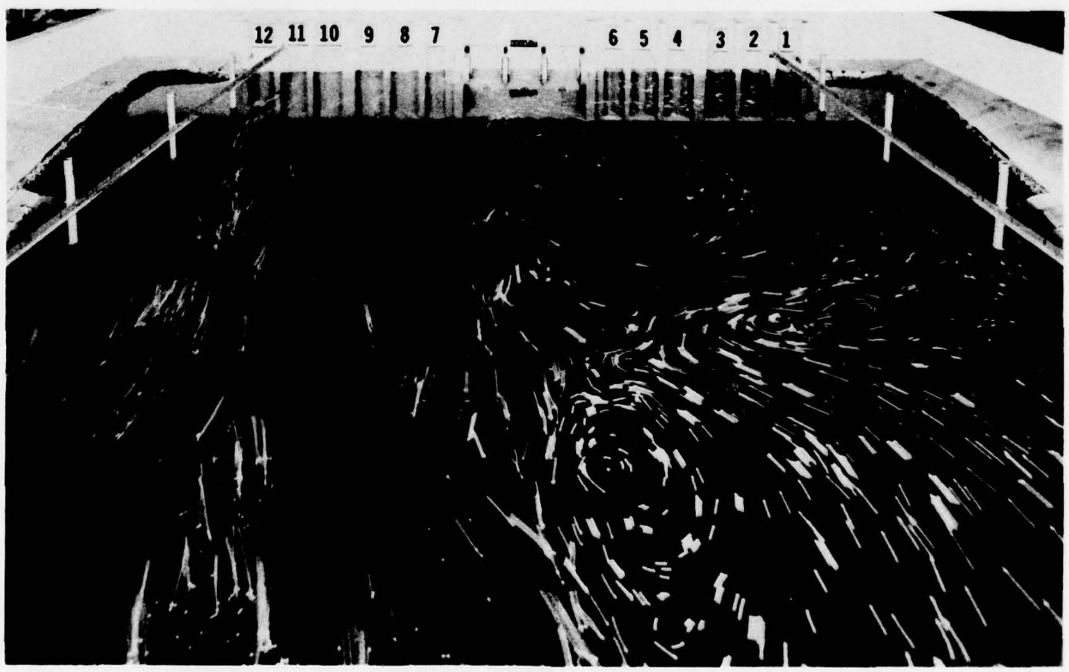


Photo 16. Pumps operating: 7, 8, 9, 10, 11, 12. Discharge per pump 7, 250 cfs; pumps 8, 9, 10, 11, 12, 600 cfs; tailwater el 620 ft

Photo 17. Pumps operating: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12. Discharge per pumps 6 and 7, 250 cfs; pumps 1, 2, 3, 4, 5, 8, 9, 10, 11, 12, 600 cfs; tailwater el 120 ft

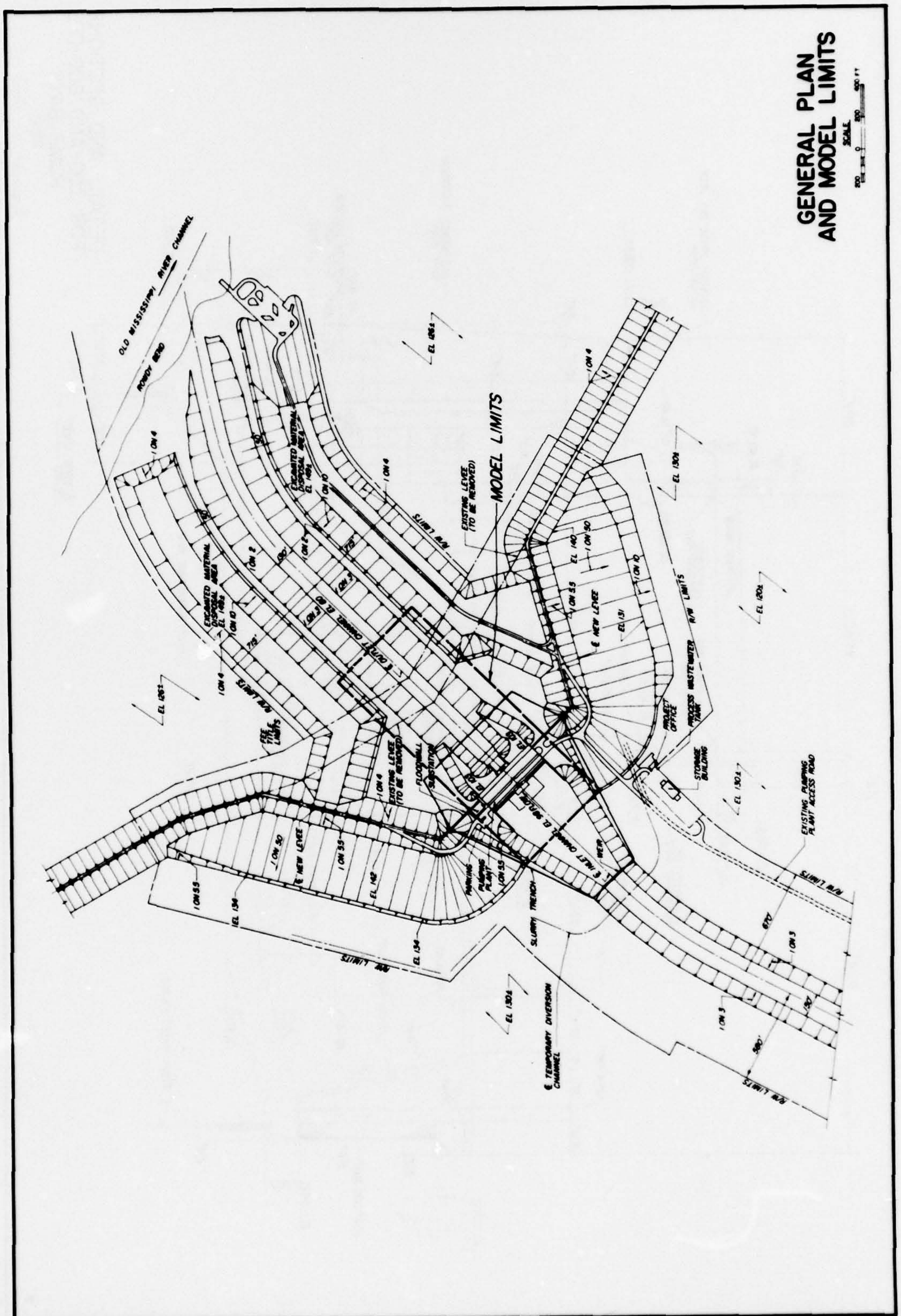

GENERAL PLAN
AND MODEL LIMITS
SCALE
MODEL LIMITS
OLD MISSISSIPPI RIVER CHANNEL
ROWDY BEND
EXISTING LEVEE (TO BE REMOVED)
EXCAVATED MATERIAL DISPOSAL AREA
PROJECT OFFICE
STORAGE BUILDING
EXISTING PUMPING PLANT ACCESS ROAD
SLURRY TRENCH
℄ TEMPORARY DIVERSION CHANNEL
R/W LIMITS
℄ NEW LEVEE

PLATE 1

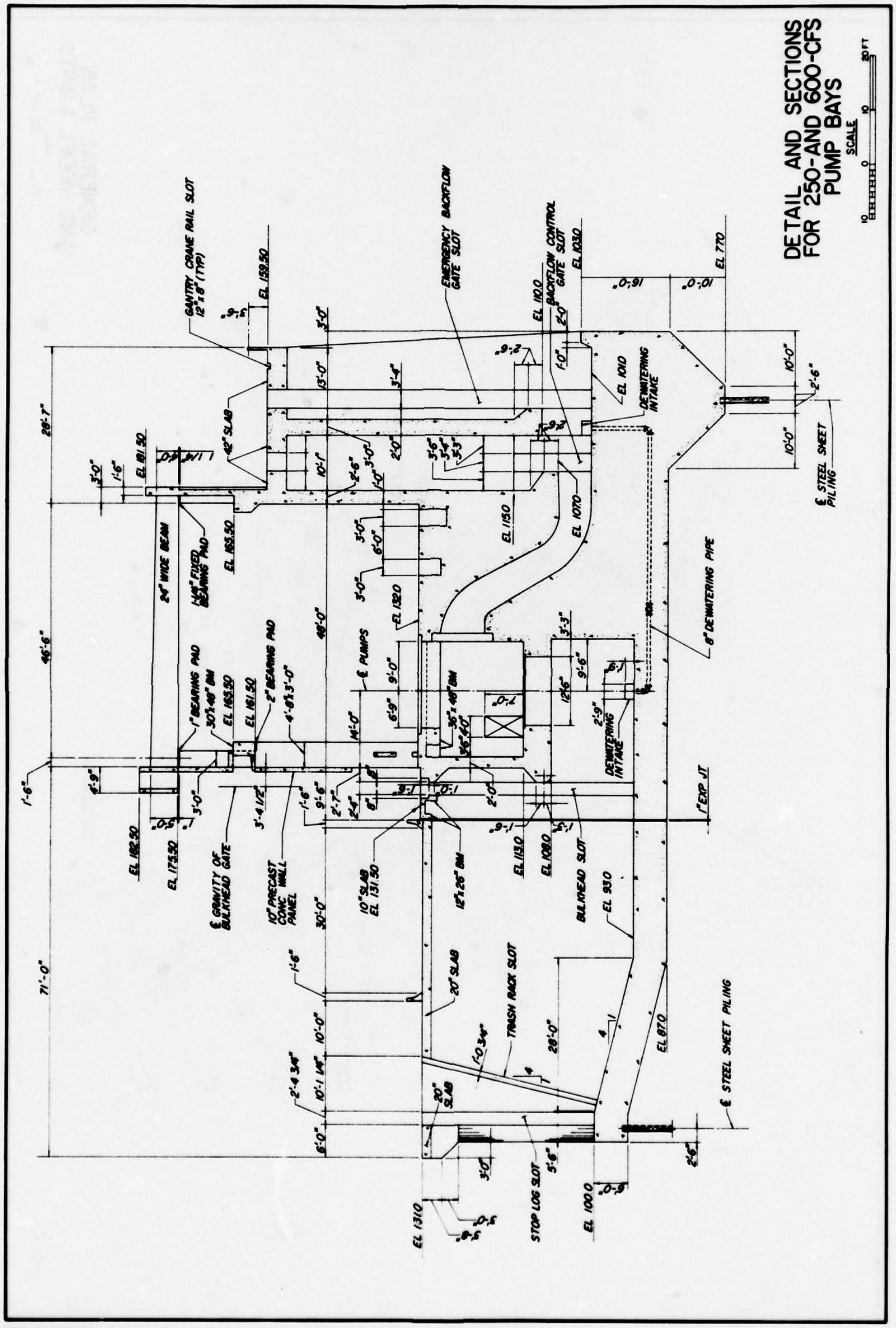
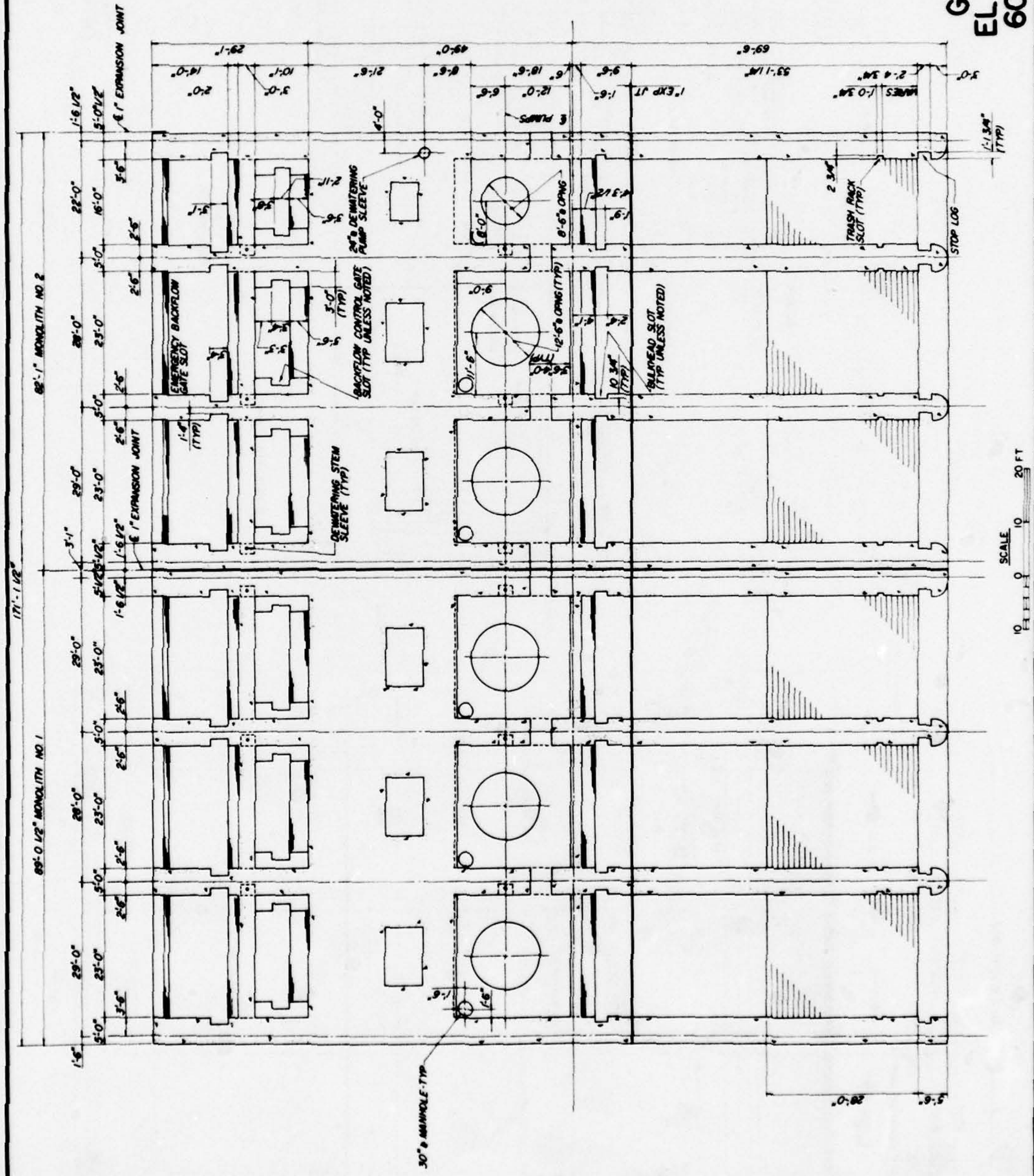

DETAIL AND SECTIONS
FOR 250-AND 600-CFS
PUMP BAYS
SCALE
10 0 10 20 FT

PLATE 2

GENERAL PLAN AT EL 118.0 FOR 250-AND 600-CFS PUMP BAYS



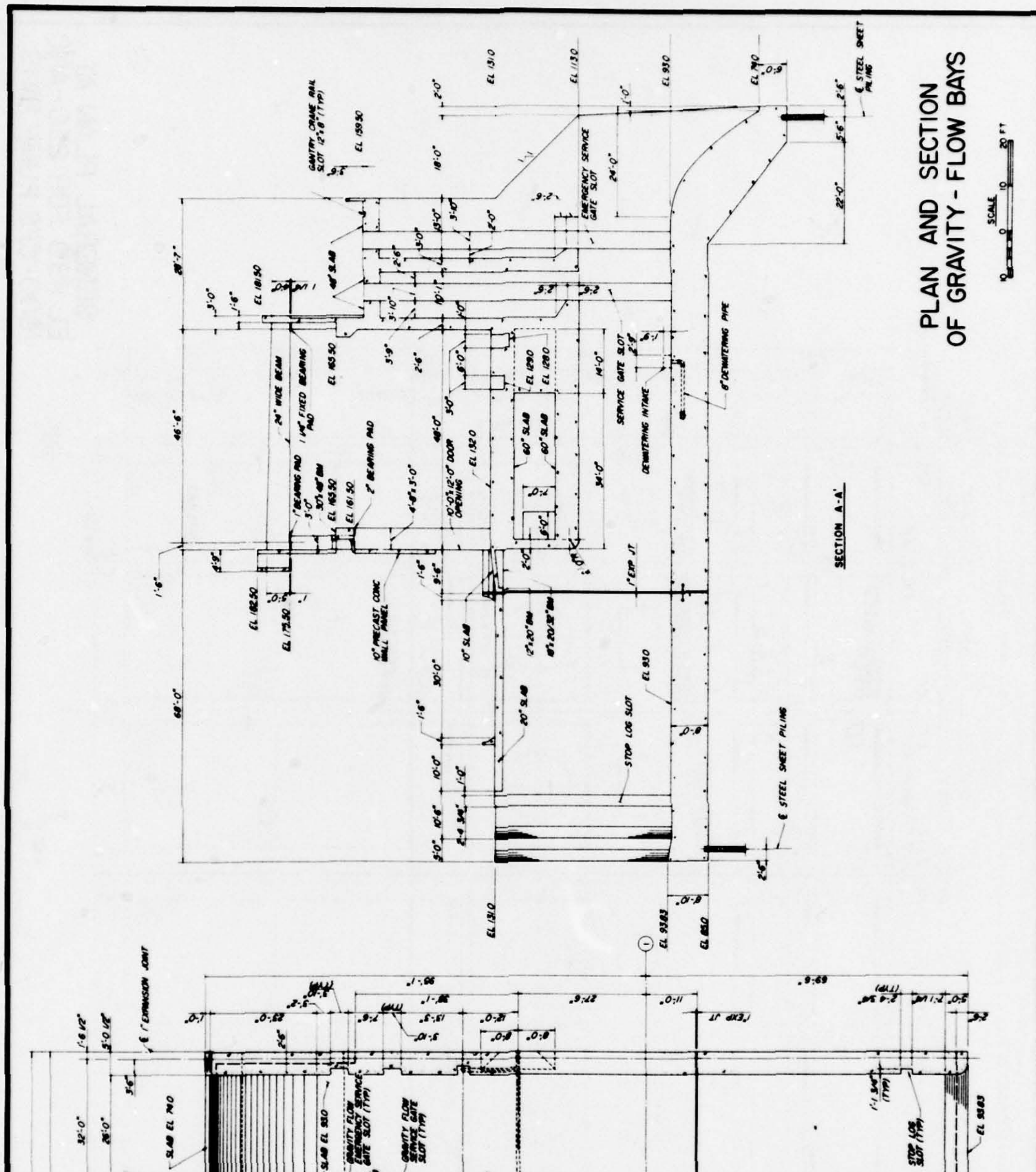
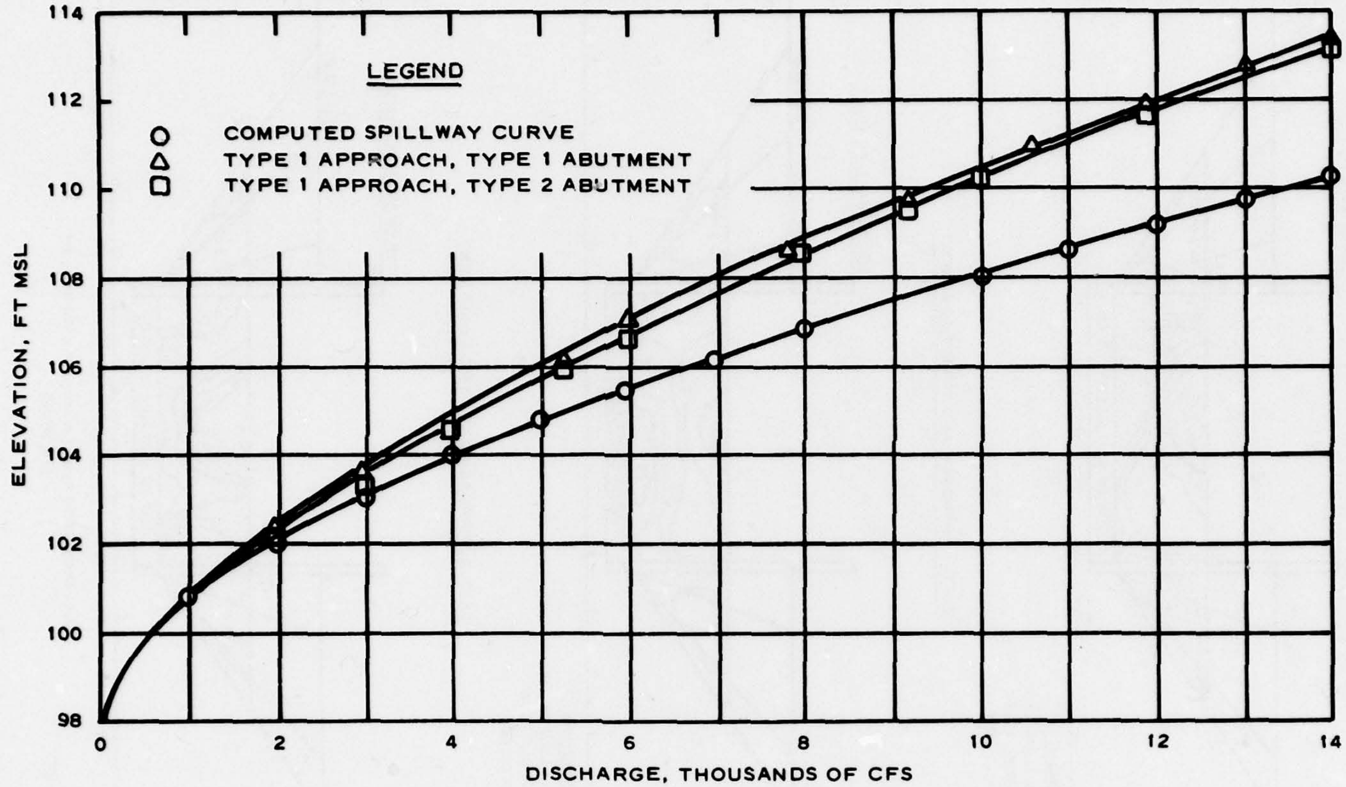


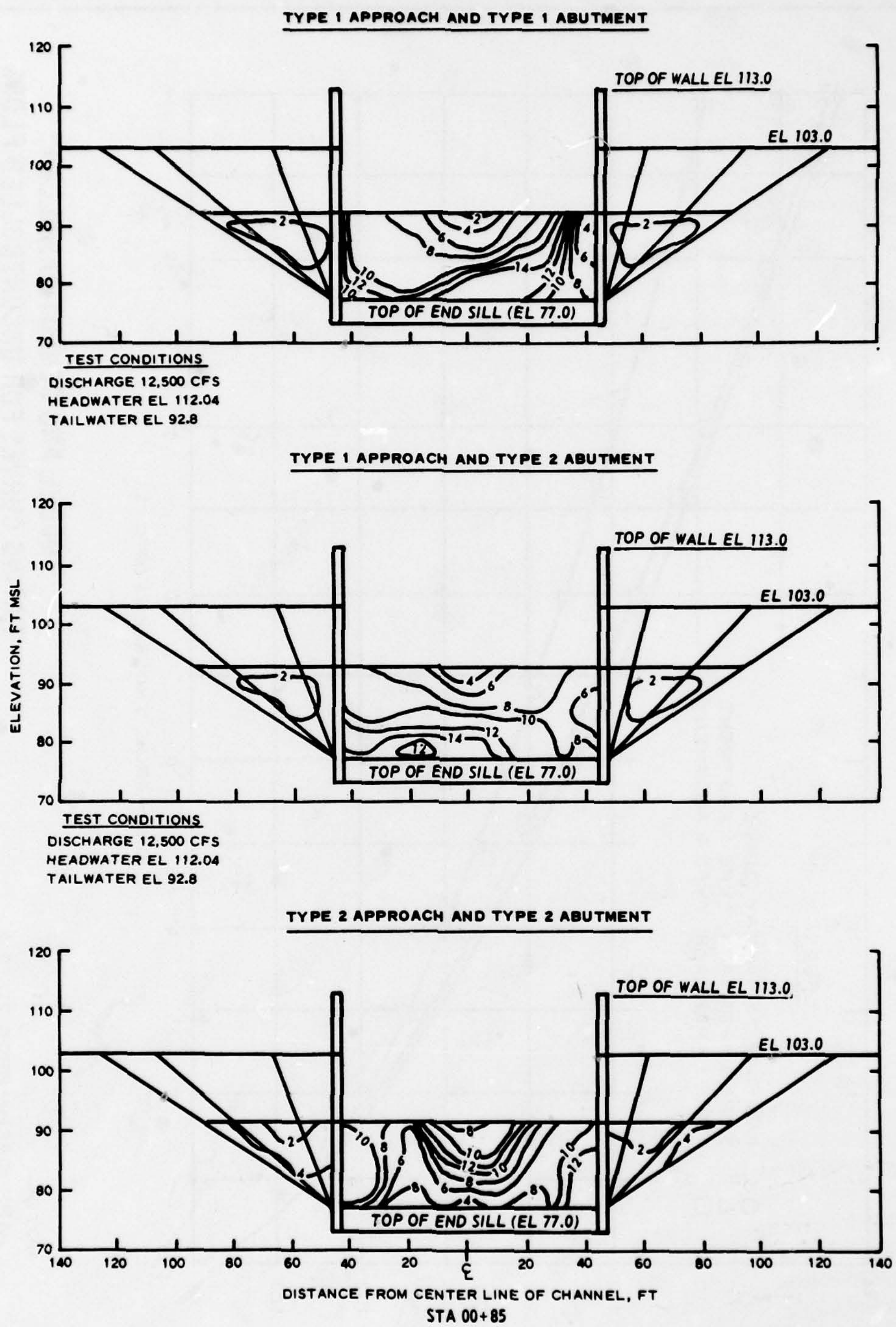
PLATE 4



NOTE:
APPROACH APRON EL 98.0
SPILLWAY CREST EL 93.0

MODEL AND COMPUTED SPILLWAY RATING CURVES FOR UNCONTROLLED FLOWS

TYPE 1 APPROACH

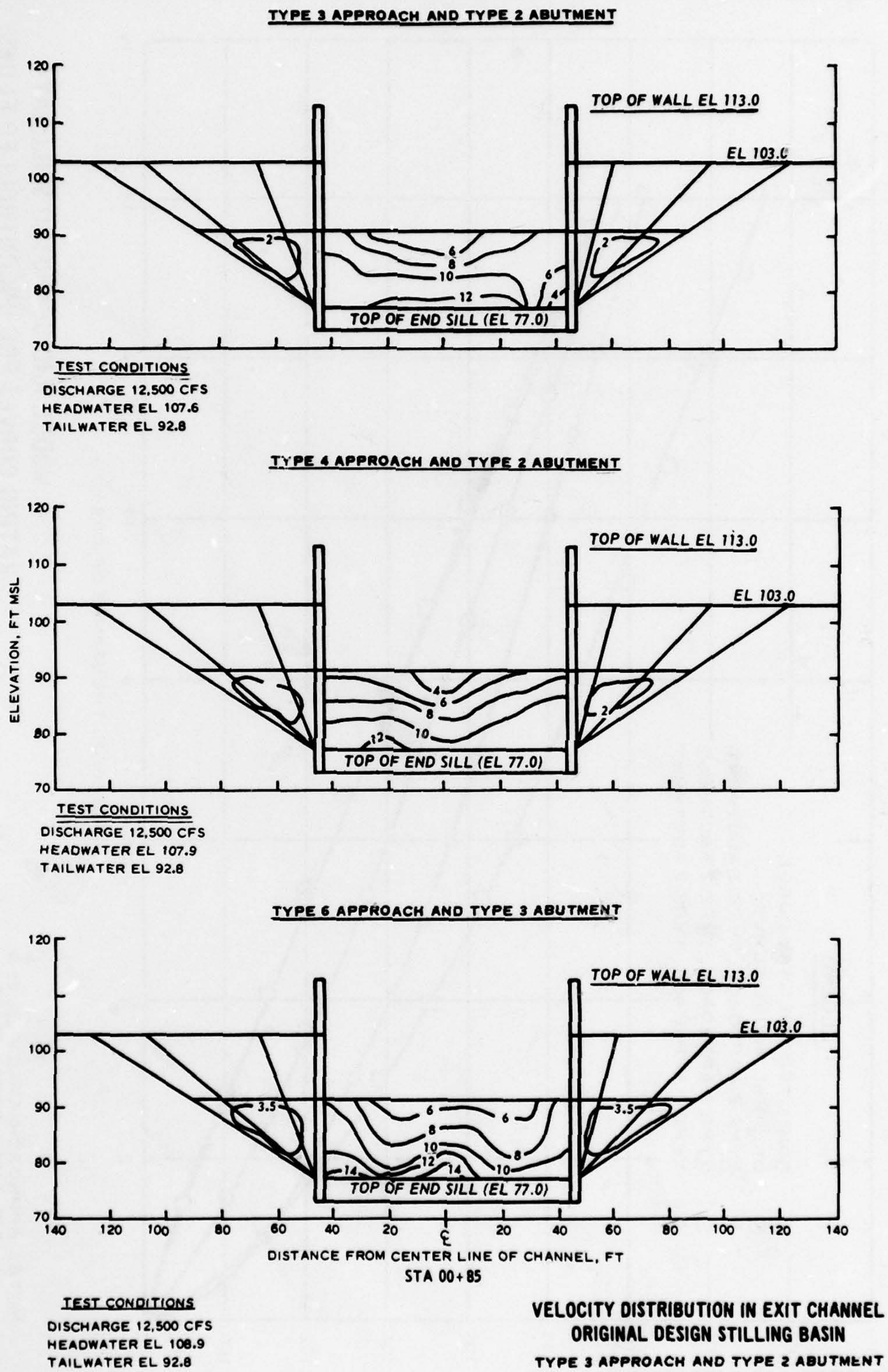


TEST CONDITIONS

DISCHARGE 12,500 CFS
HEADWATER EL 108.5
TAILWATER EL 92.8

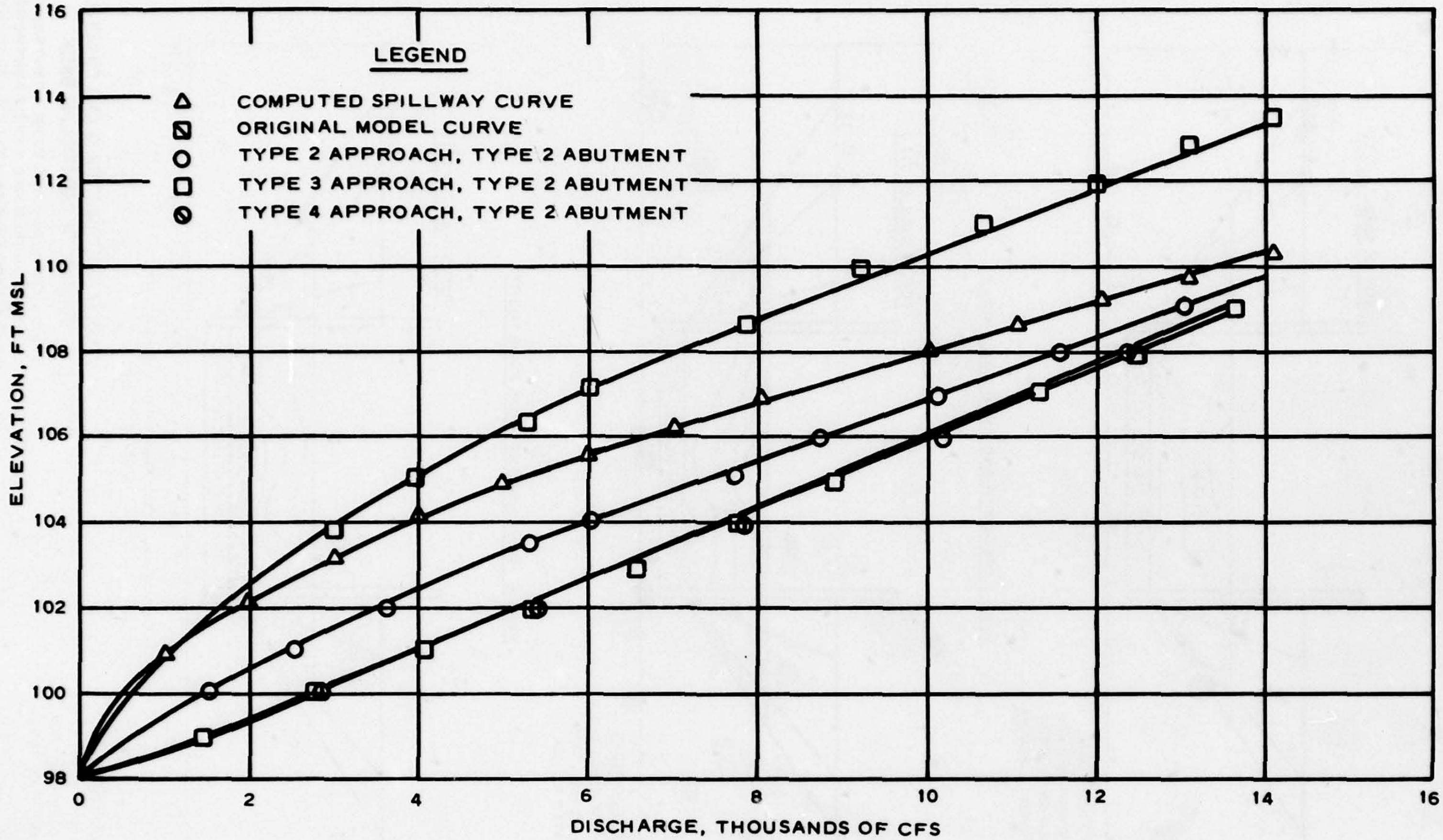
VELOCITY DISTRIBUTION IN EXIT CHANNEL
ORIGINAL STILLING BASIN DESIGN

TYPE 1 APPROACH AND TYPE 1 ABUTMENT
TYPE 1 APPROACH AND TYPE 2 ABUTMENT
TYPE 2 APPROACH AND TYPE 2 ABUTMENT



VELOCITY DISTRIBUTION IN EXIT CHANNEL
ORIGINAL DESIGN STILLING BASIN

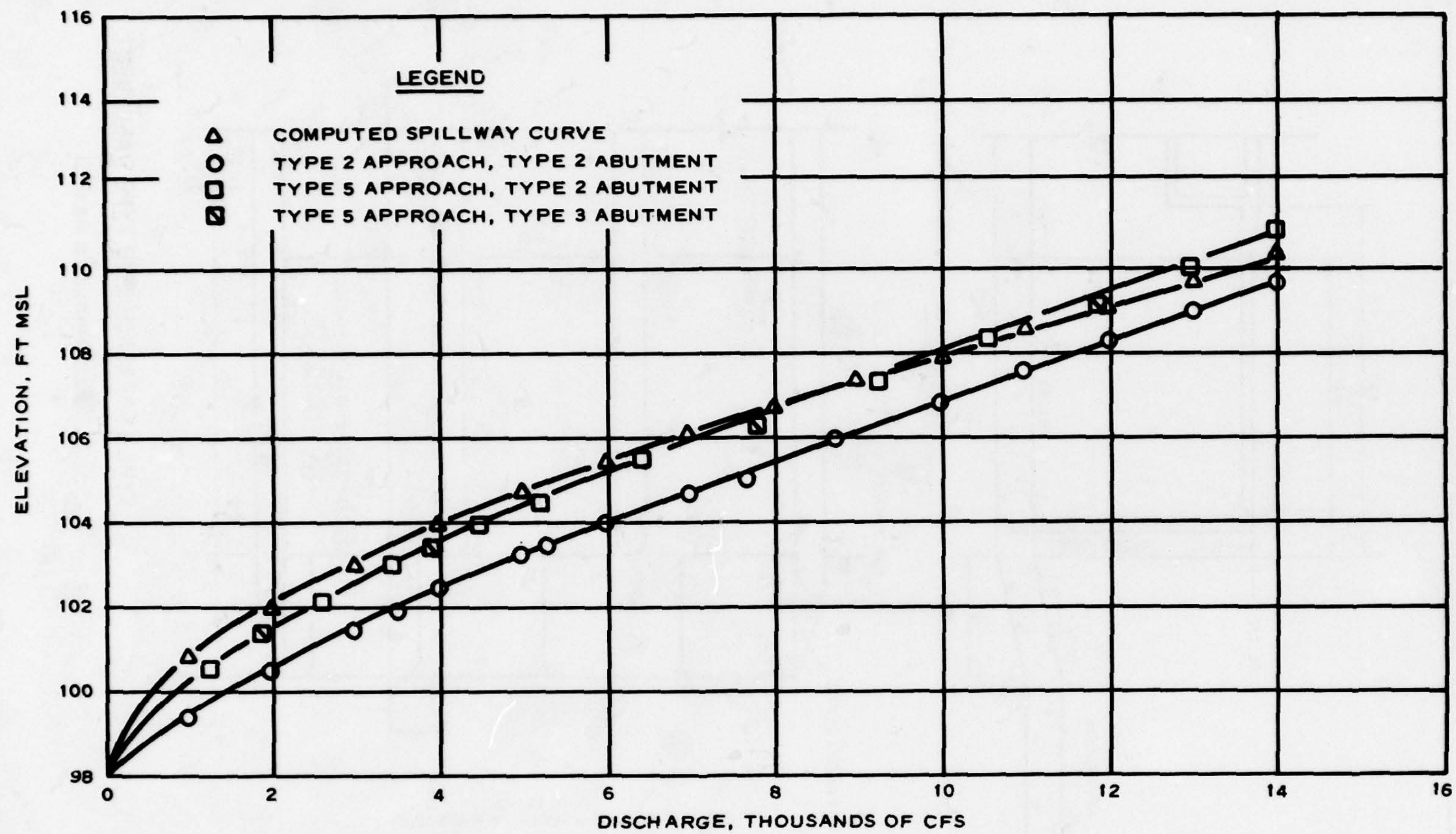
TYPE 3 APPROACH AND TYPE 2 ABUTMENT
TYPE 4 APPROACH AND TYPE 2 ABUTMENT
TYPE 6 APPROACH AND TYPE 3 ABUTMENT



NOTE: APPROACH APRON EL 98.0
SPILLWAY CREST EL 93.0

MODEL AND COMPUTED SPILLWAY RATING CURVES FOR UNCONTROLLED FLOWS

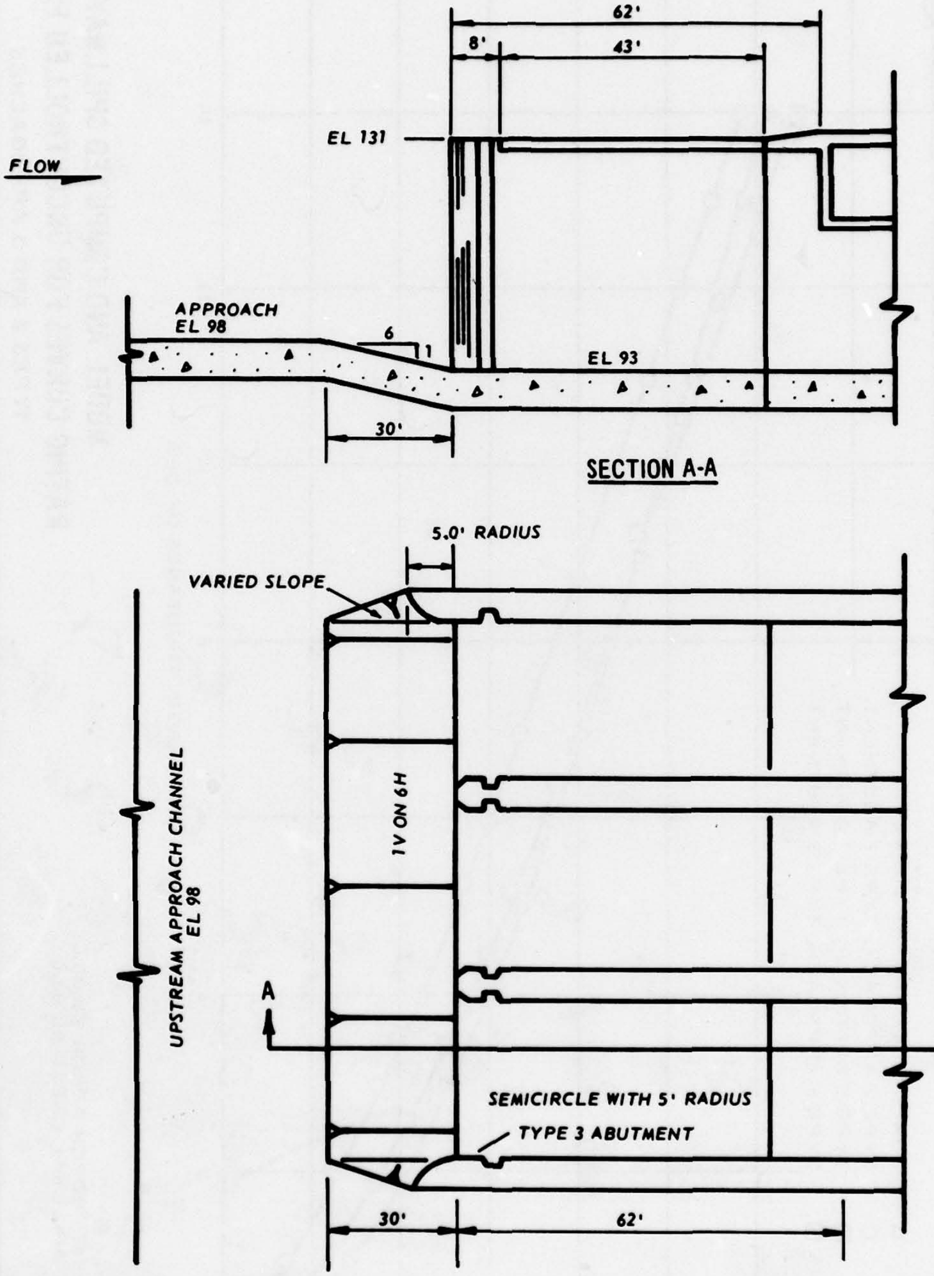
TYPES 2, 3, AND 4 APPROACHES



NOTE: APPROACH APRON EL 98.0
SPILLWAY CREST EL 93.0

MODEL AND COMPUTED SPILLWAY RATING CURVES FOR UNCONTROLLED FLOWS

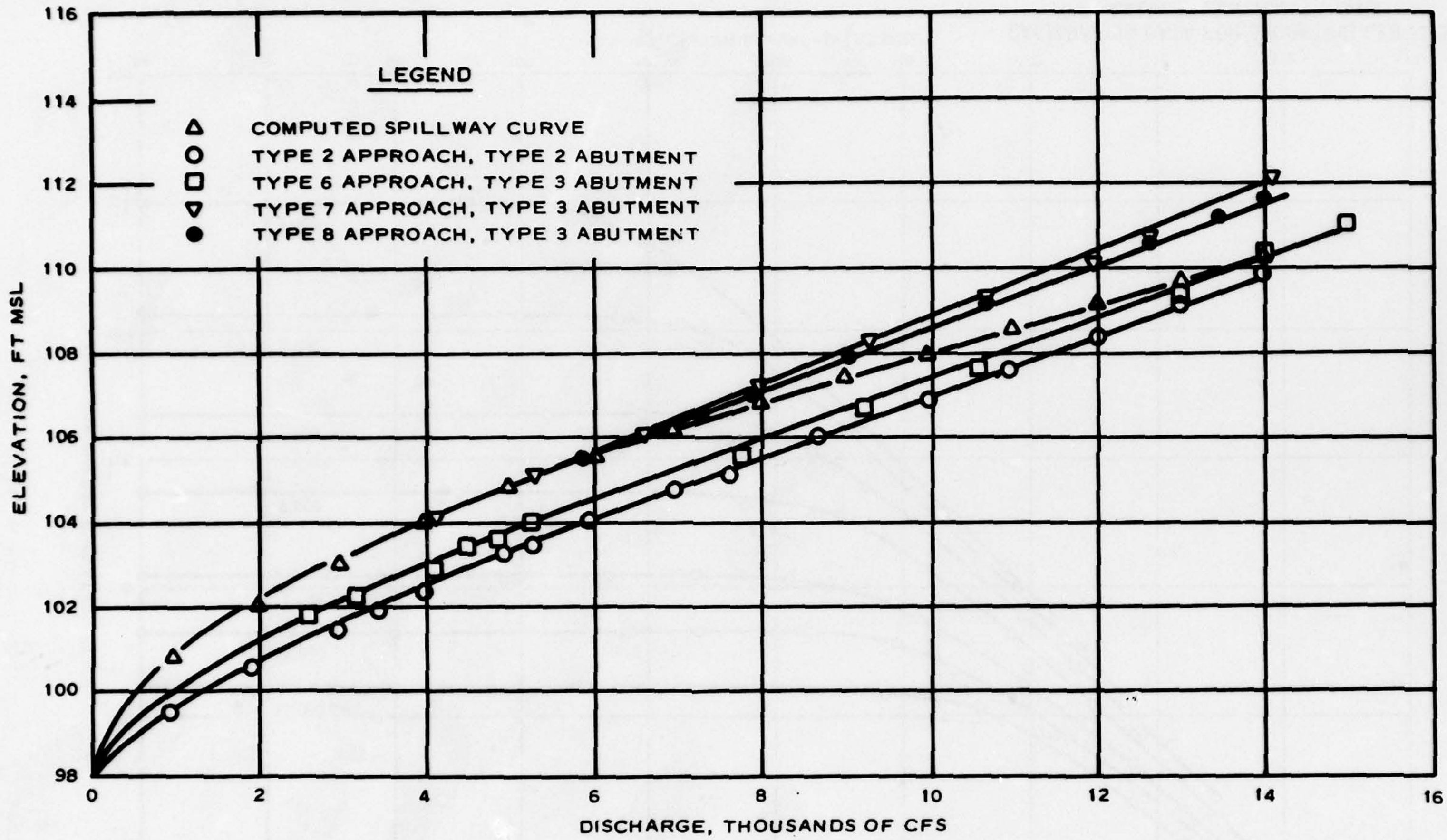
TYPES 2 AND 5 APPROACHES



TYPE 6 APPROACH WITH TYPE 3 ABUTMENT

RECOMMENDED DESIGN

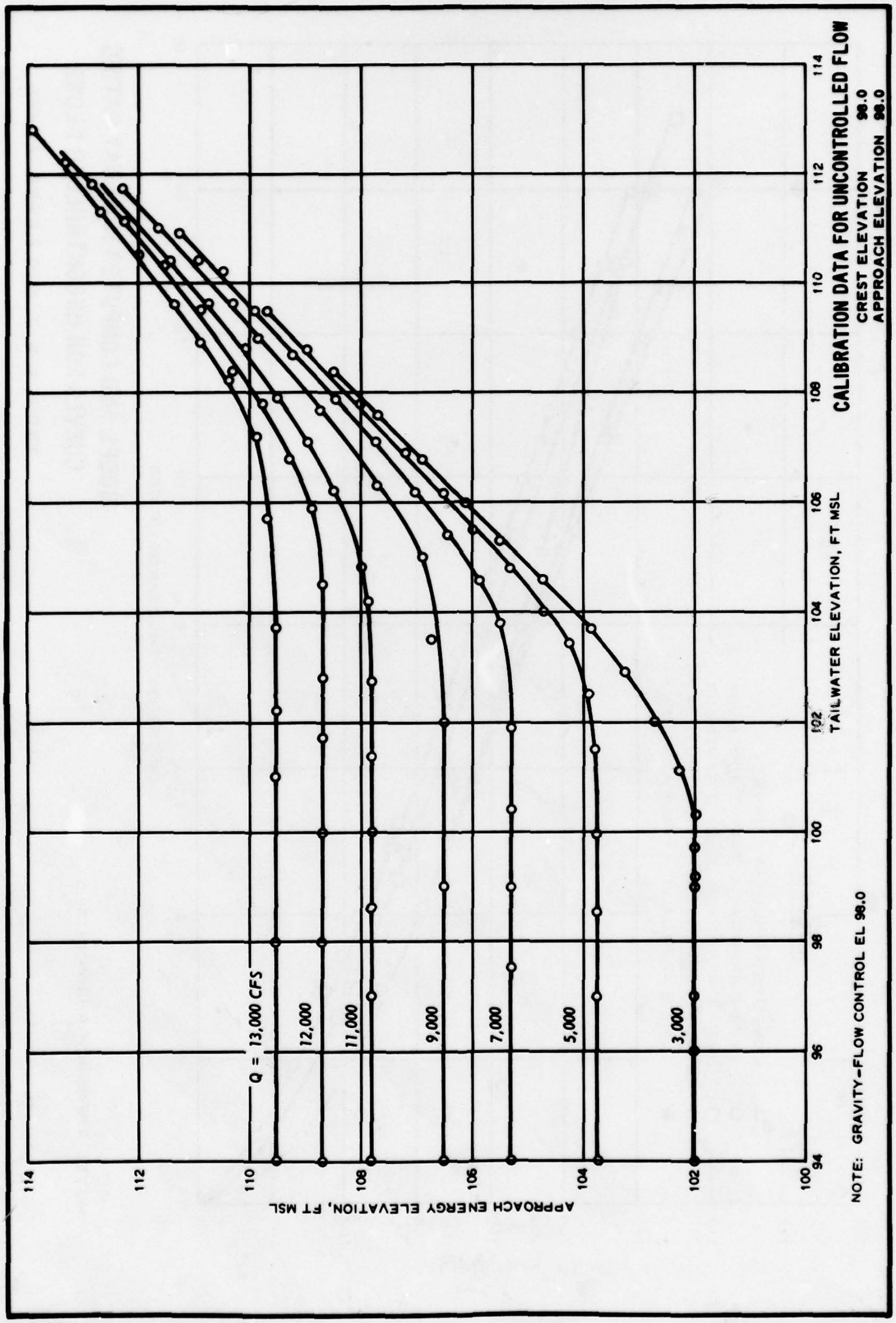
PLATE 10



NOTE: APPROACH APRON EL 98.0

MODEL AND COMPUTED SPILLWAY RATING CURVES FOR UNCONTROLLED FLOWS

TYPES 2, 6, 7, AND 8 APPROACHES

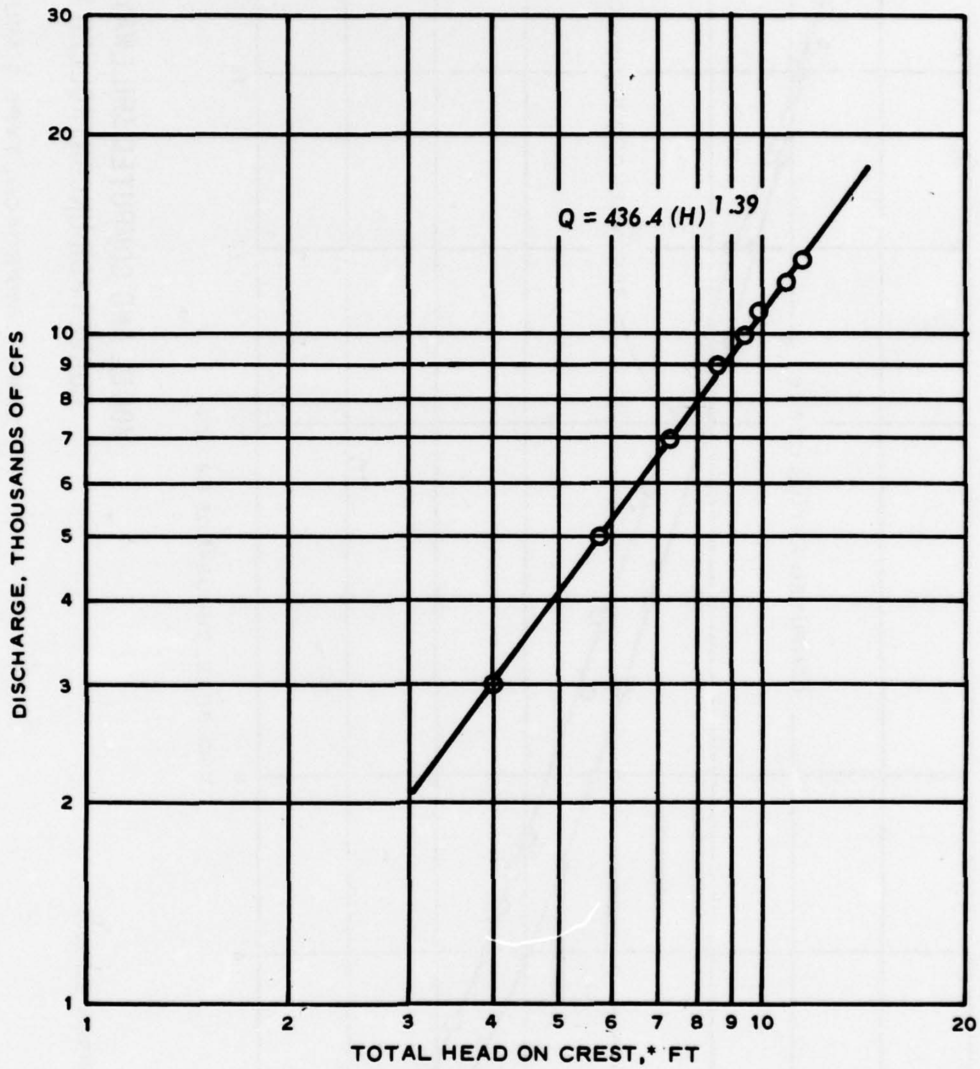


CALIBRATION DATA FOR UNCONTROLLED FLOW

CREST ELEVATION 98.0

APPROACH ELEVATION 98.0

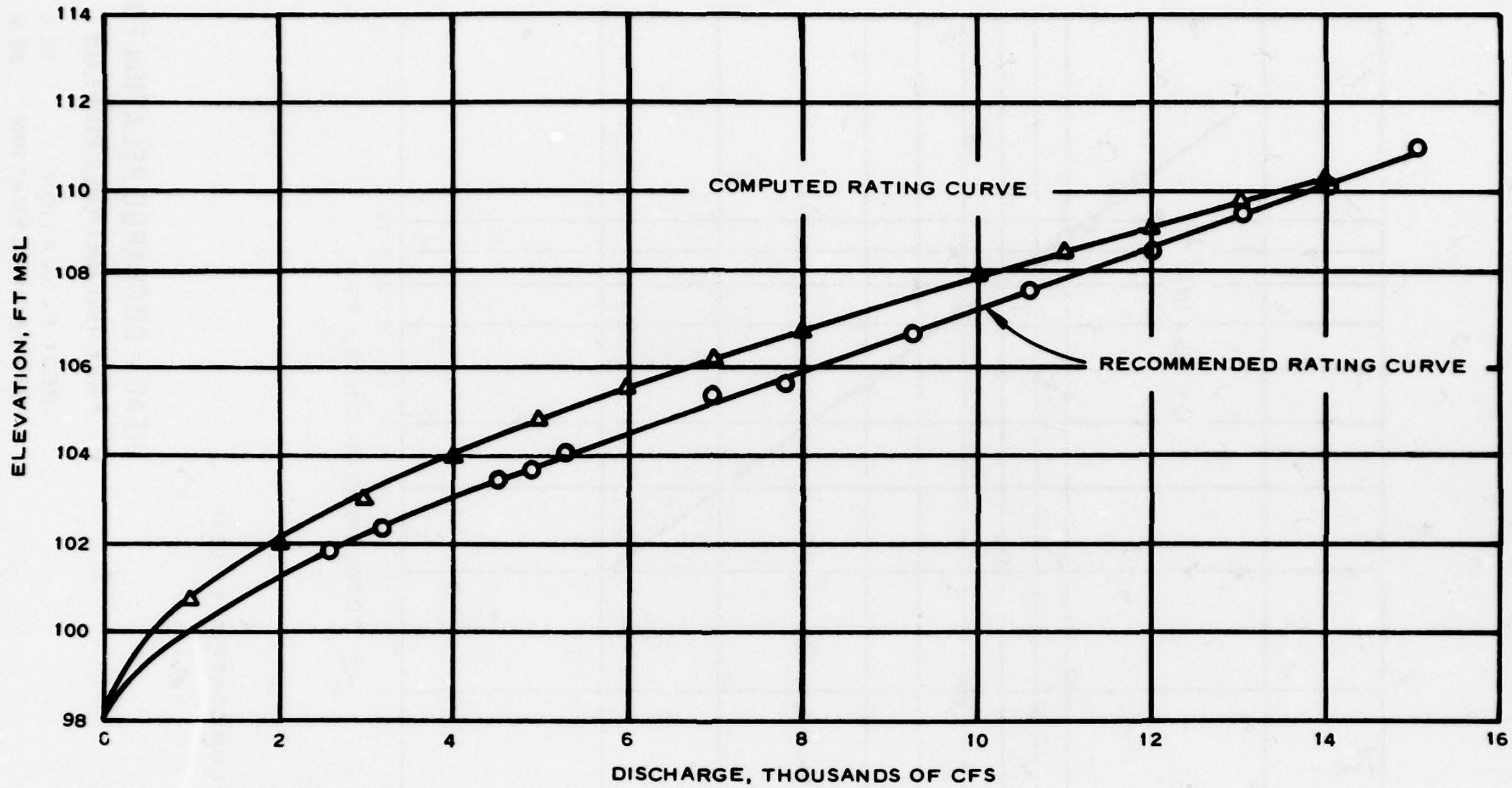
PLATE 12



* GRAVITY-FLOW CONTROL EL 98.0

HEAD – DISCHARGE RELATION FOR FREE UNCONTROLLED FLOW

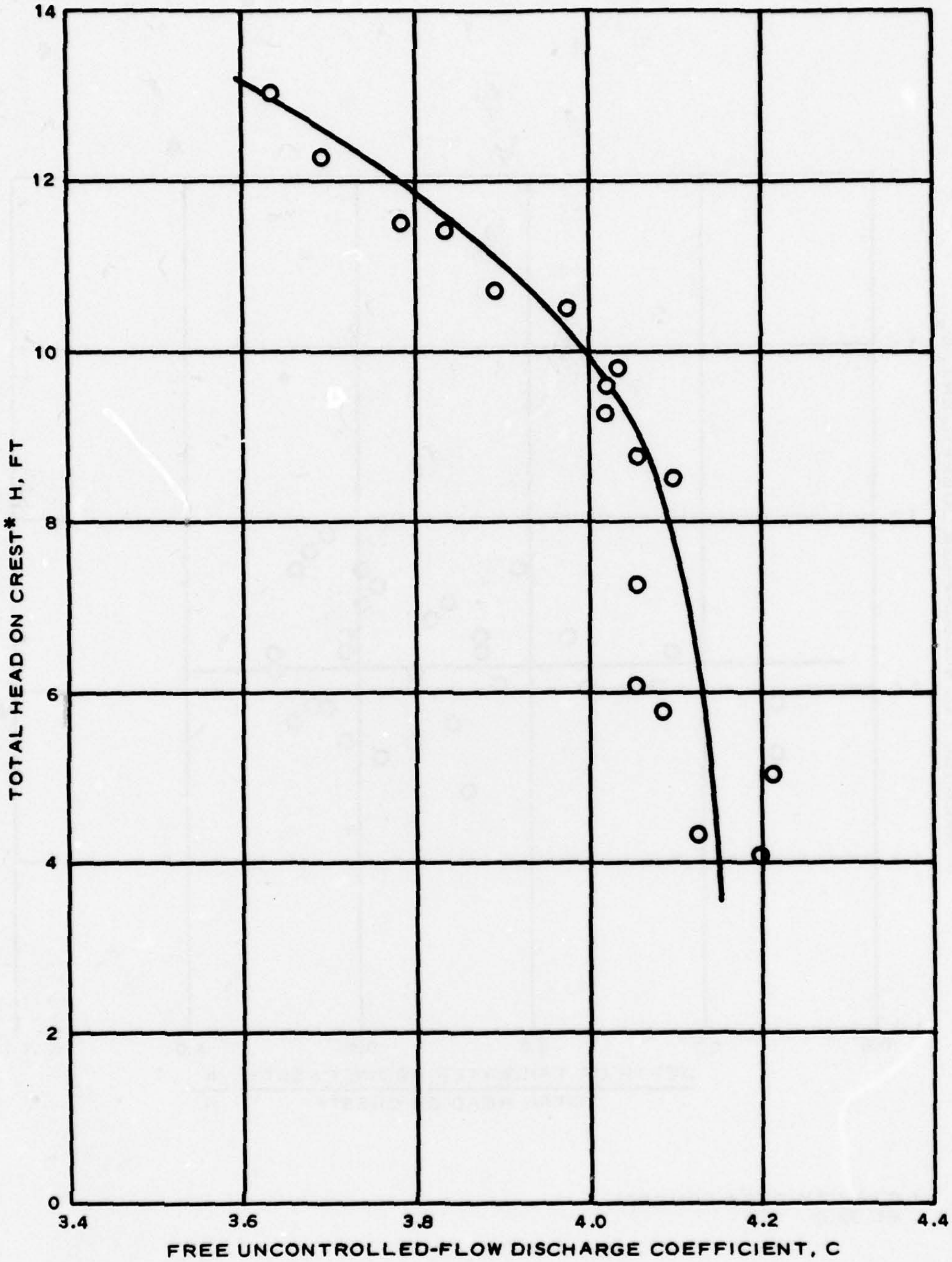
CREST ELEVATION	98.0
APPROACH ELEVATION	98.0



NOTE: APPROACH APRON EL 98.0

MODEL AND COMPUTED SPILLWAY RATING CURVES FOR UNCONTROLLED FLOW

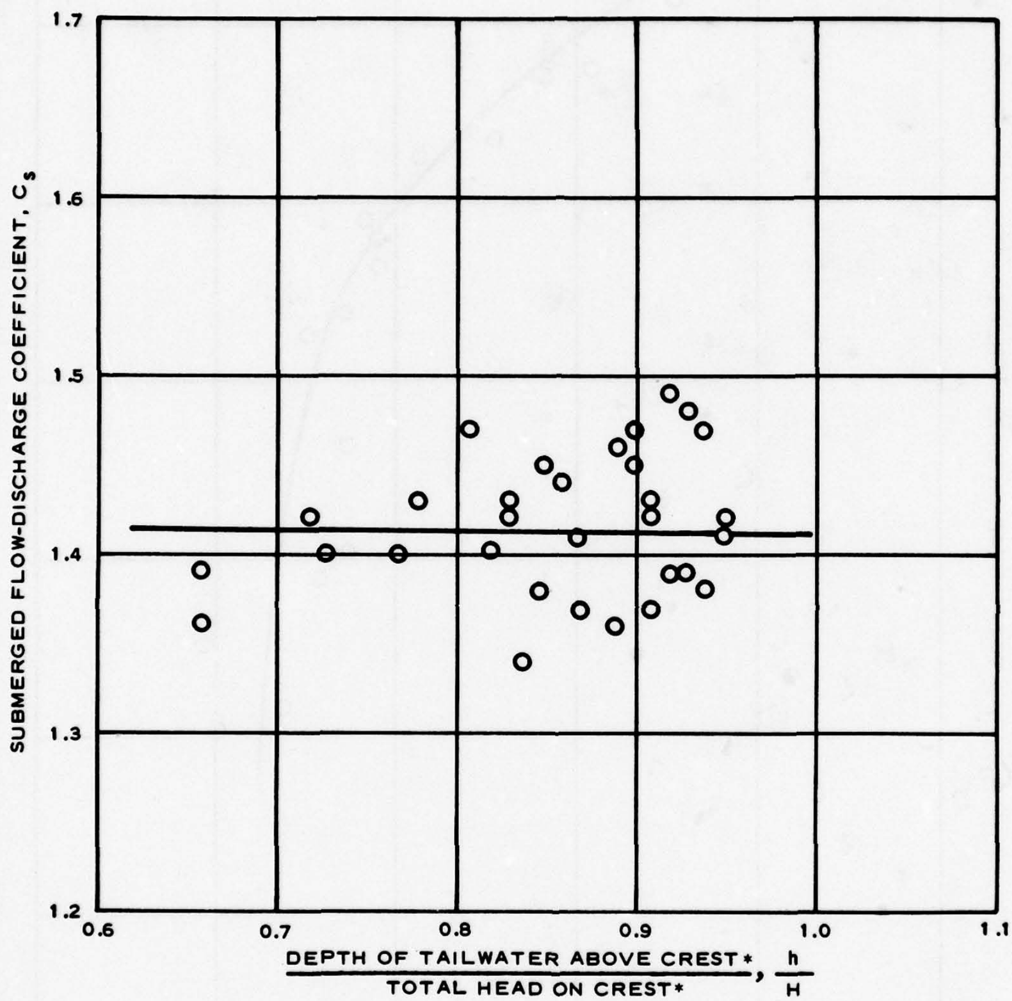
TYPE 6 APPROACH, TYPE 3 ABUTMENT



* GRAVITY-FLOW CONTROL EL 98.0

DISCHARGE COEFFICIENT FOR FREE UNCONTROLLED FLOW

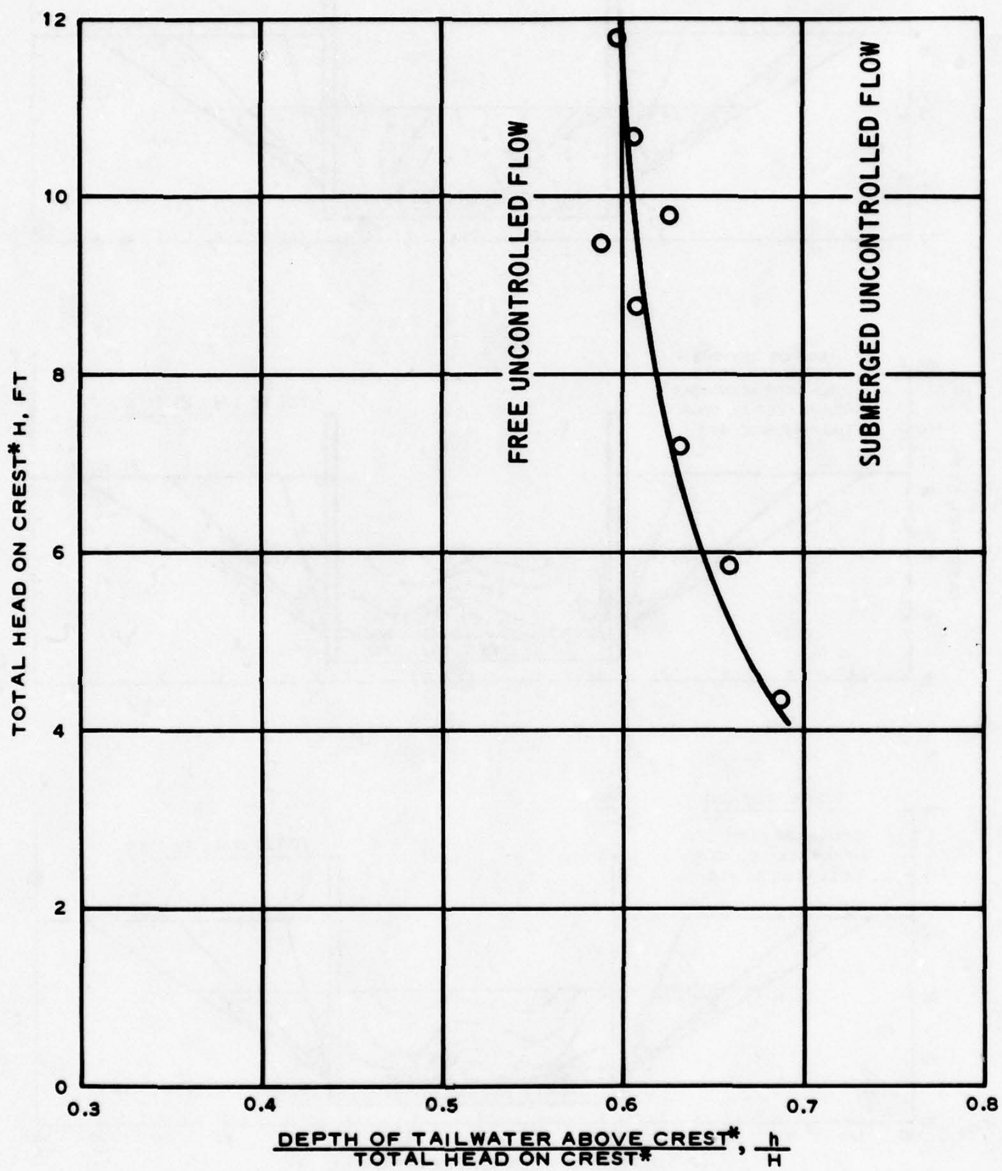
CREST ELEVATION	98.0
APPROACH ELEVATION	98.0



* GRAVITY-FLOW CONTROL
EL 98.0

DISCHARGE COEFFICIENT FOR SUBMERGED UNCONTROLLED FLOW

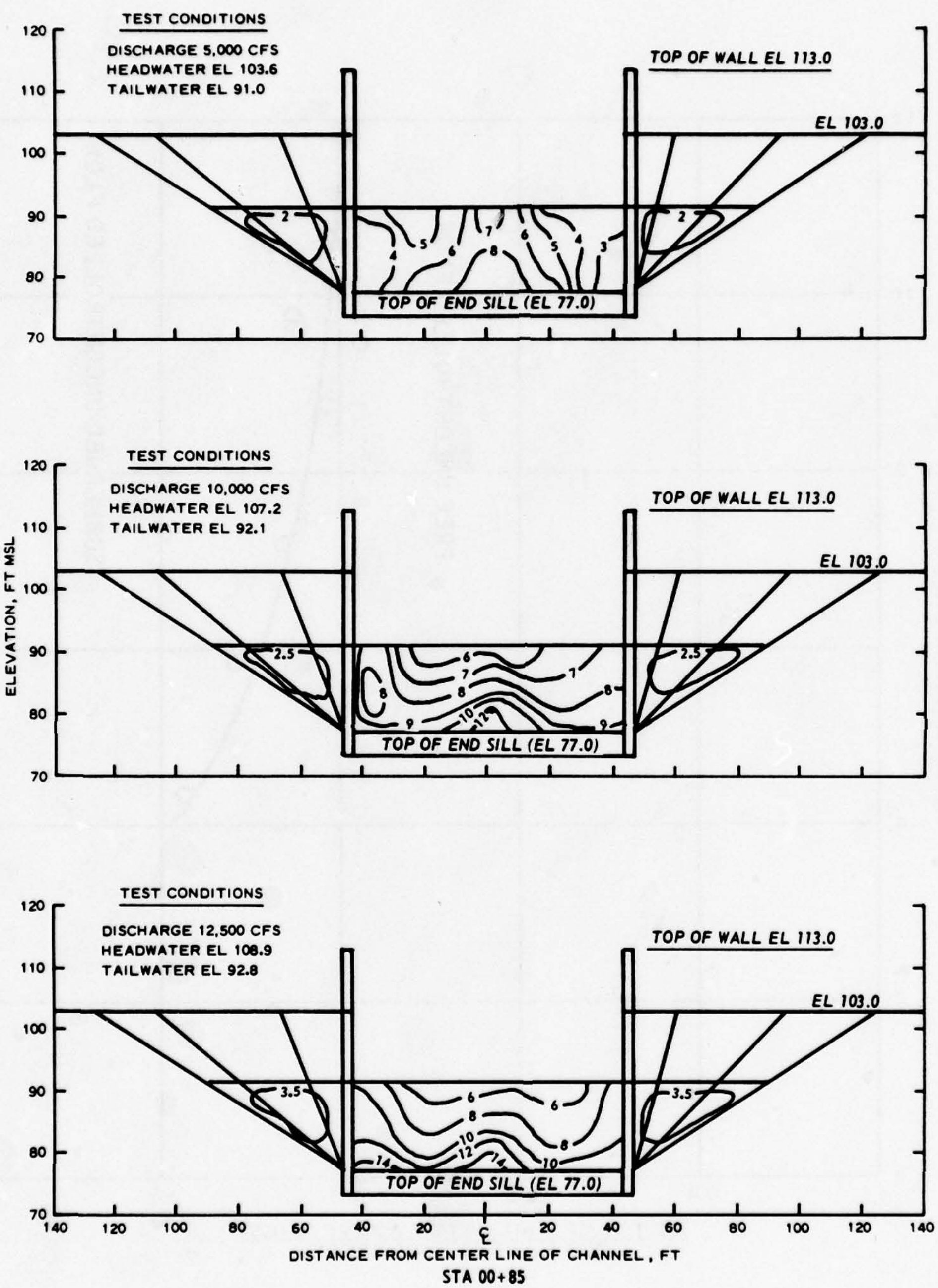
CREST ELEVATION	98.0
APPROACH ELEVATION	98.0



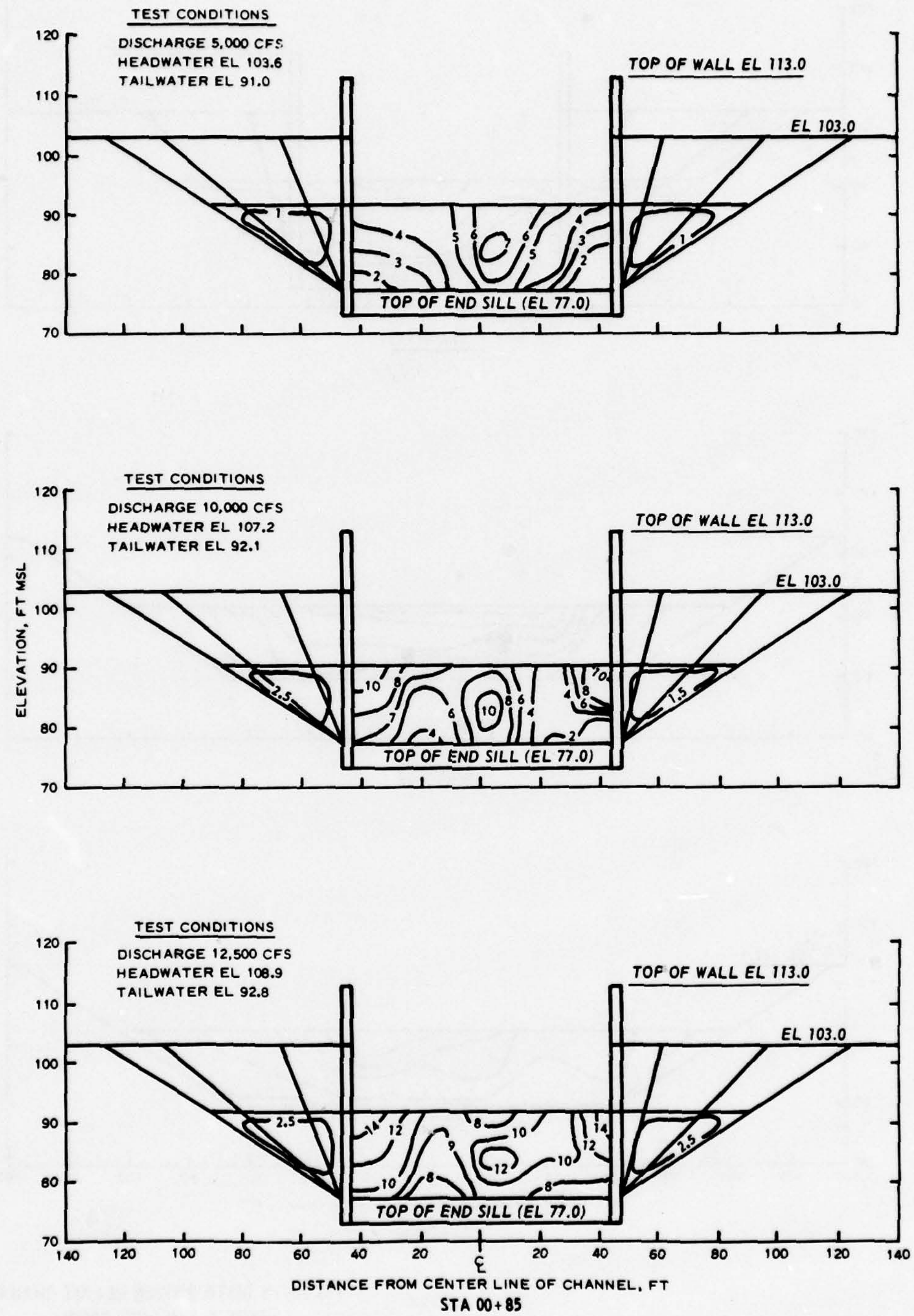
NOTE: H = TOTAL HEAD ON CREST IN FEET
(BASED ON DEPTH OF FLOW AND
VELOCITY HEAD IN APPROACH CHANNEL)

* GRAVITY-FLOW CONTROL EL 98.0

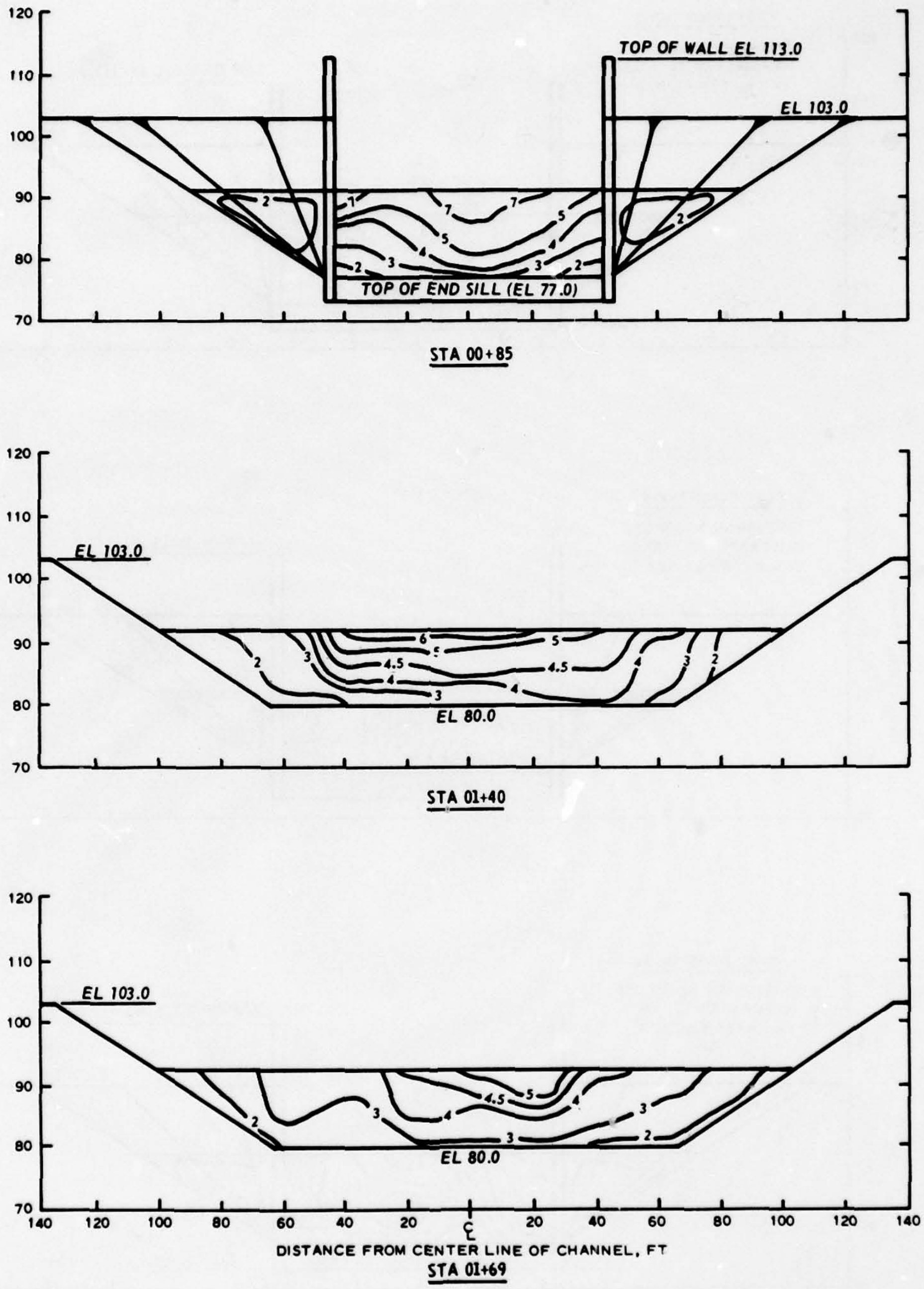
UNCONTROLLED-FLOW REGIMES



VELOCITY DISTRIBUTION IN EXIT CHANNEL
ORIGINAL STILLING BASIN DESIGN
TYPE 6 APPROACH AND TYPE 3 ABUTMENT
ISOVELS

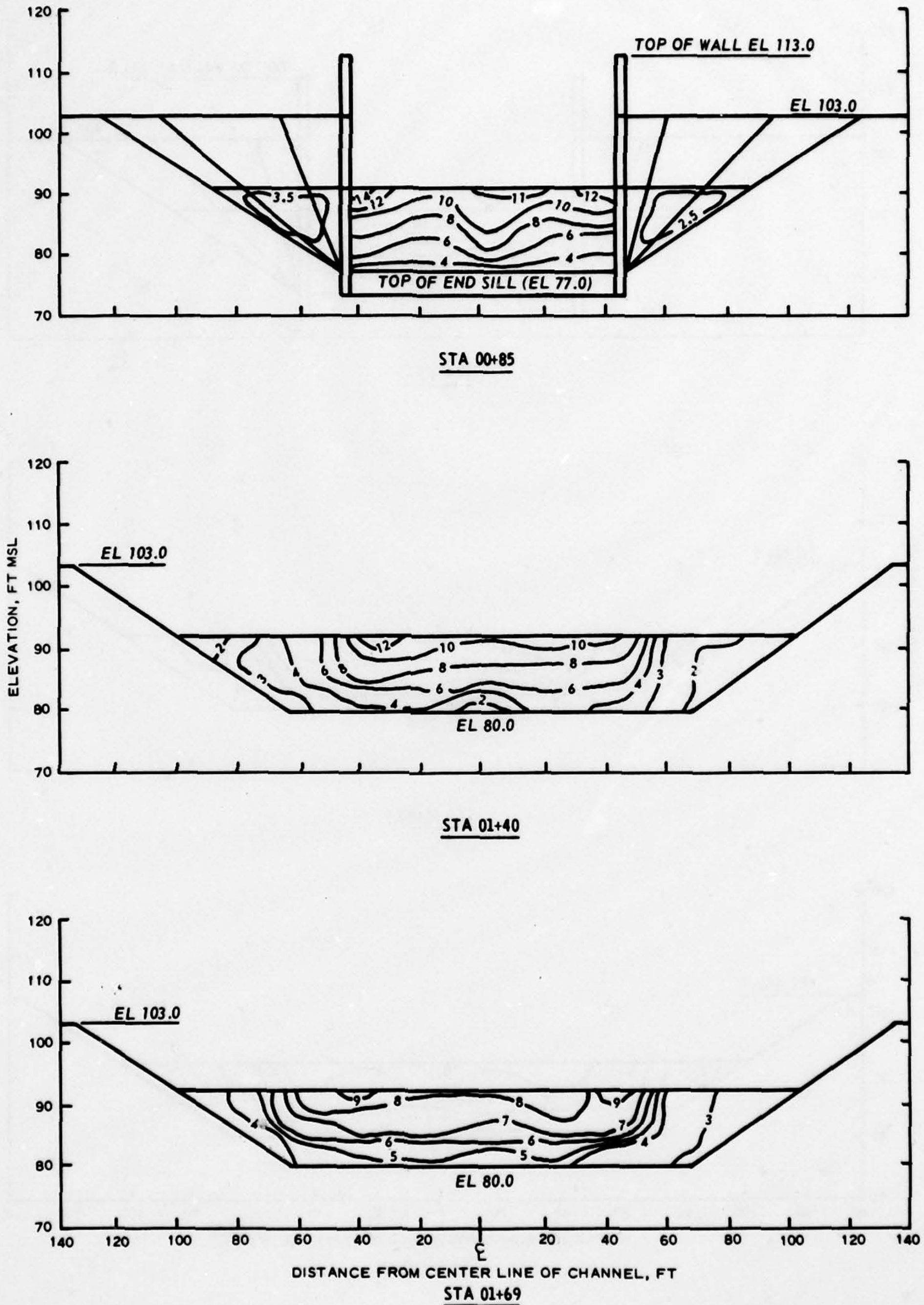


VELOCITY DISTRIBUTION IN EXIT CHANNEL
TYPE 2 STILLING BASIN
TYPE 6 APPROACH AND TYPE 3 ABUTMENT
ISOVELS



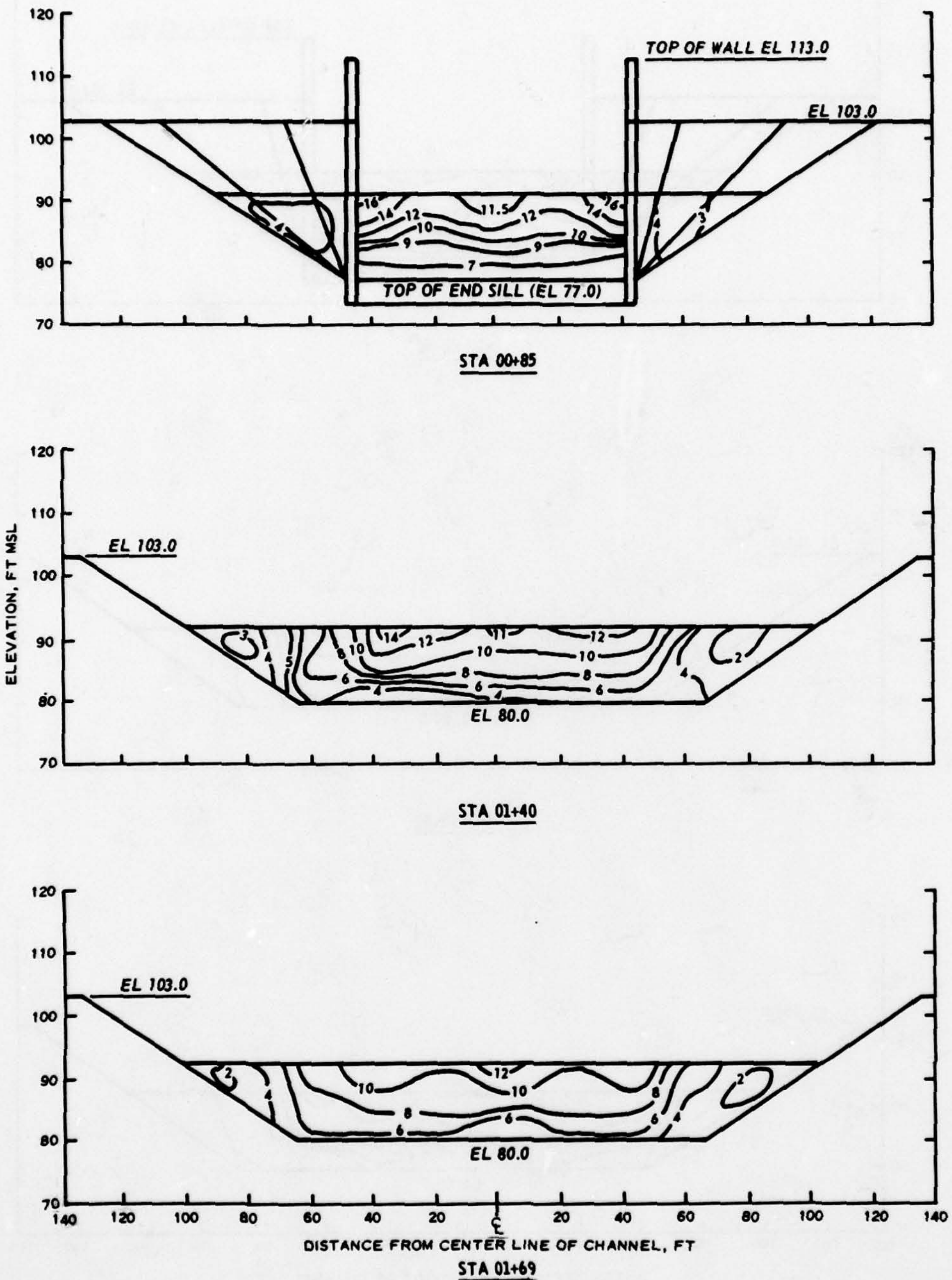
VELOCITY DISTRIBUTION IN EXIT CHANNEL
TYPE 3 STILLING BASIN
TYPE 6 APPROACH AND TYPE 3 ABUTMENT
ISOVELS

DISCHARGE 5000 CFS
HEADWATER EL 103.6
TAILWATER EL 91.0



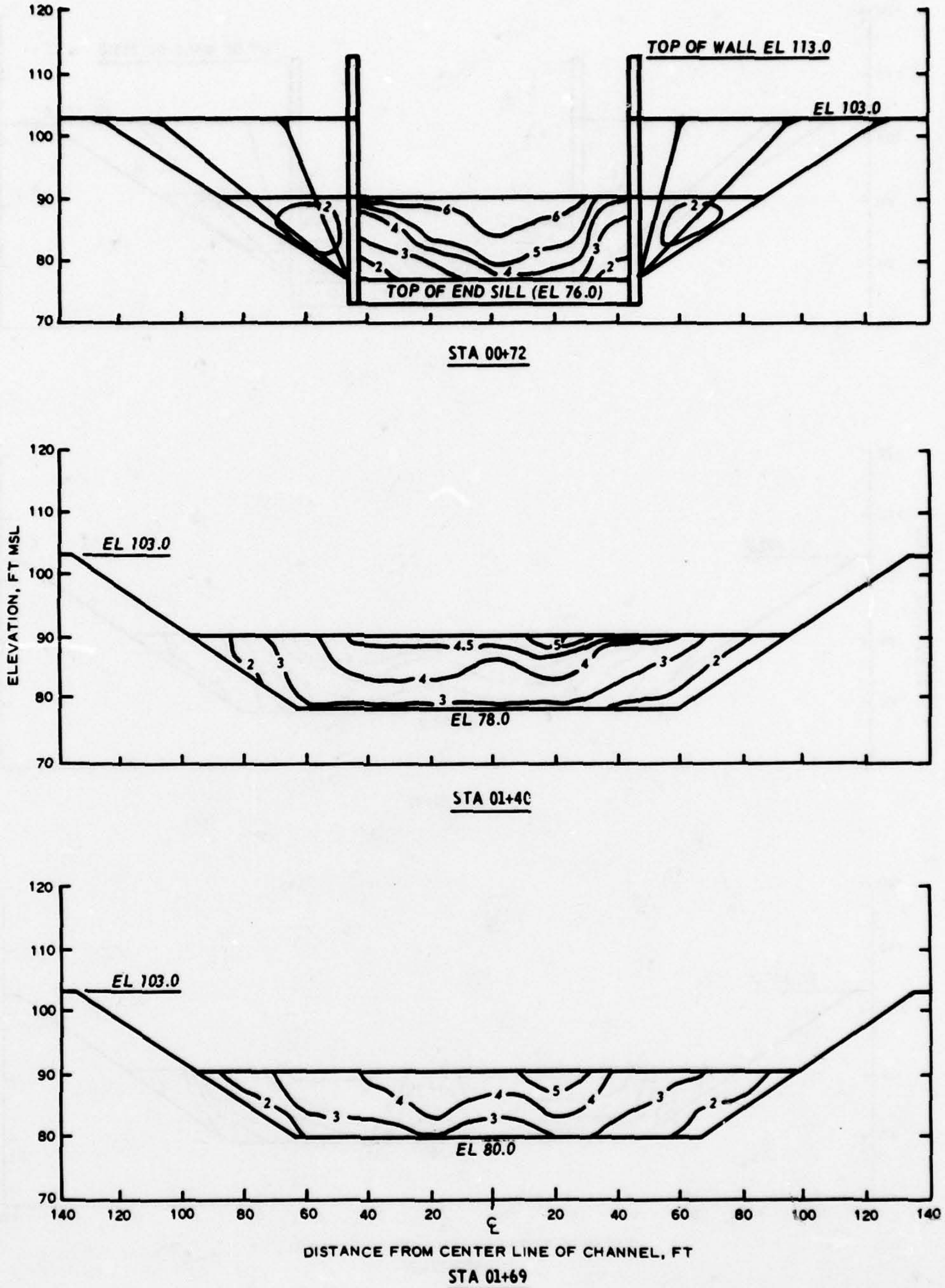
VELOCITY DISTRIBUTION IN EXIT CHANNEL
TYPE 3 STILLING BASIN
TYPE 6 APPROACH AND TYPE 3 ABUTMENT
ISOVELS

DISCHARGE 10,000 CFS
HEADWATER EL 107.2
TAILWATER EL 92.1



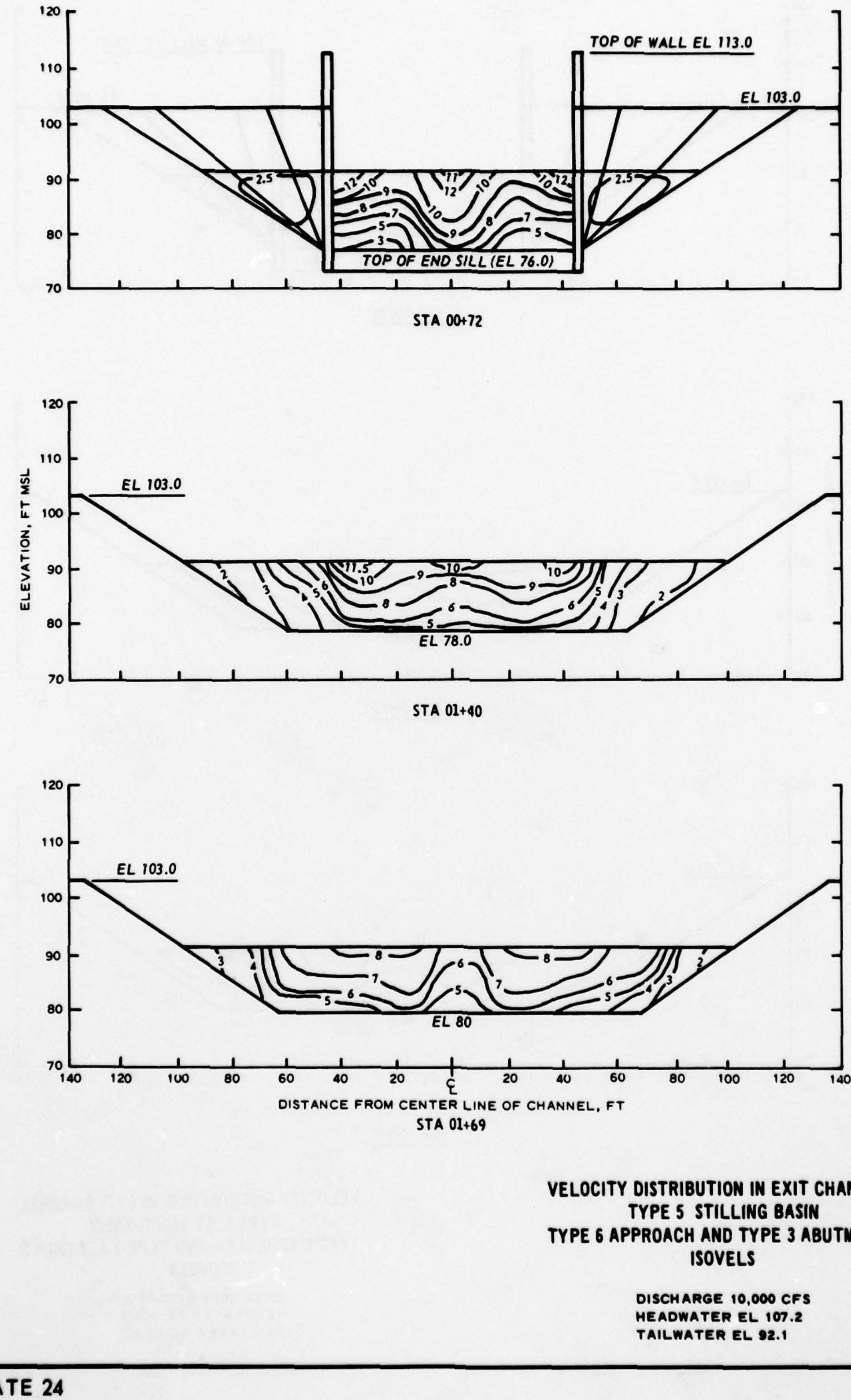
VELOCITY DISTRIBUTION IN EXIT CHANNEL
TYPE 3 STILLING BASIN
TYPE 6 APPROACH AND TYPE 3 ABUTMENT
ISOVELS

DISCHARGE 12,500 CFS
HEADWATER EL 108.9
TAILWATER EL 92.8



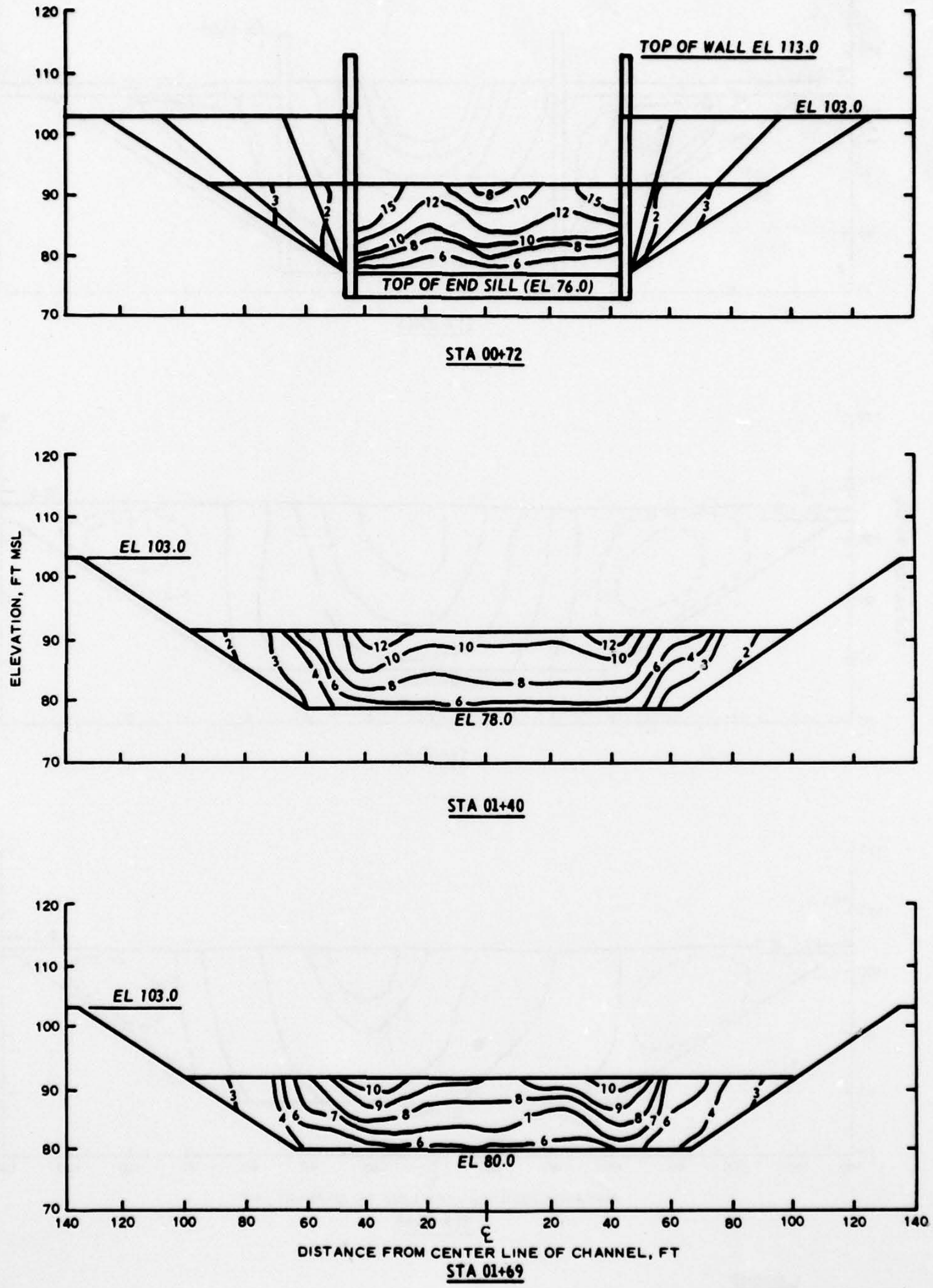
VELOCITY DISTRIBUTION IN EXIT CHANNEL
TYPE 5 STILLING BASIN
TYPE 6 APPROACH AND TYPE 3 ABUTMENT
ISOVELS

DISCHARGE 5000 CFS
HEADWATER EL 103.6
TAILWATER EL 91.0



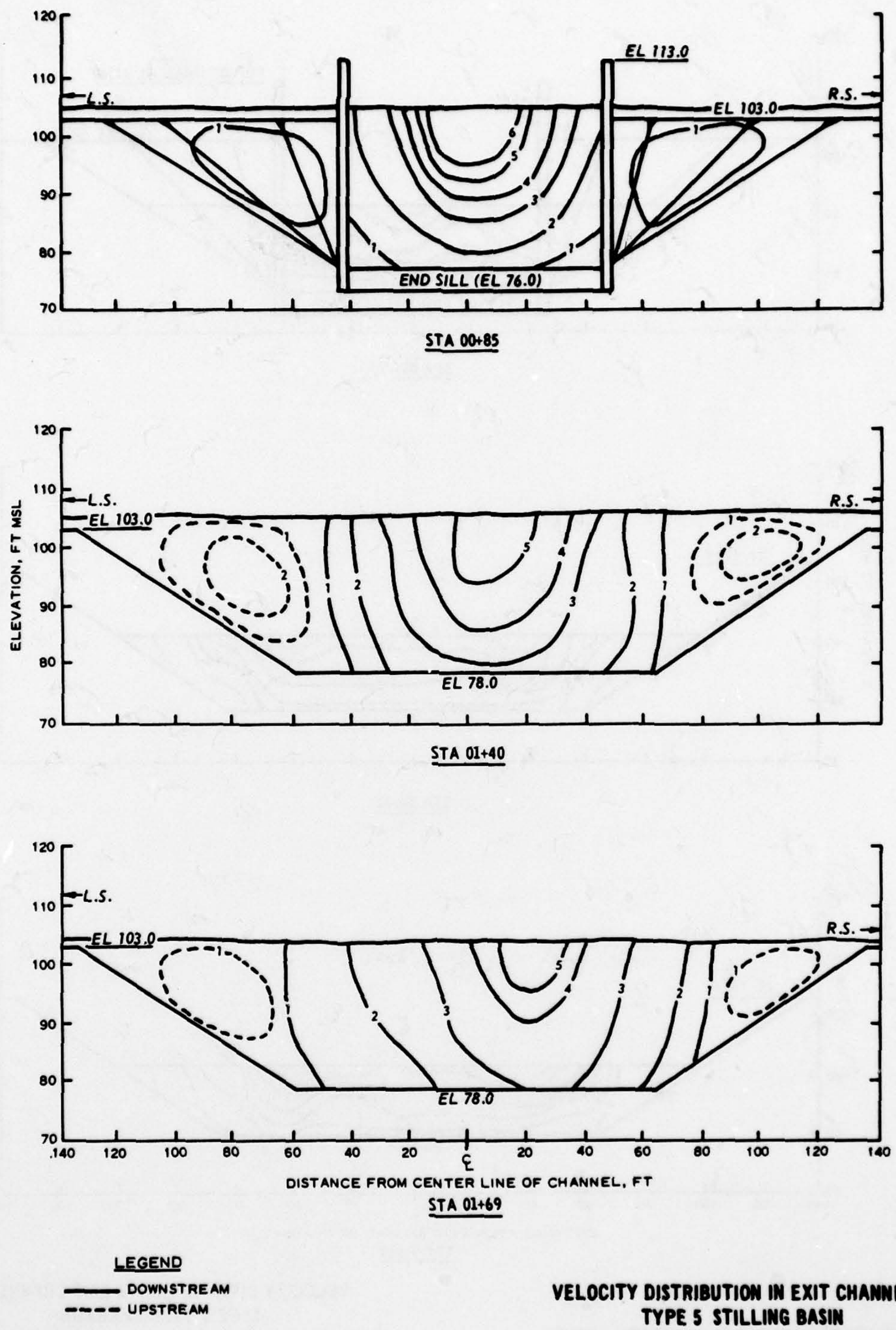
VELOCITY DISTRIBUTION IN EXIT CHANNEL
TYPE 5 STILLING BASIN
TYPE 6 APPROACH AND TYPE 3 ABUTMENT
ISOVELS

DISCHARGE 10,000 CFS
HEADWATER EL 107.2
TAILWATER EL 92.1



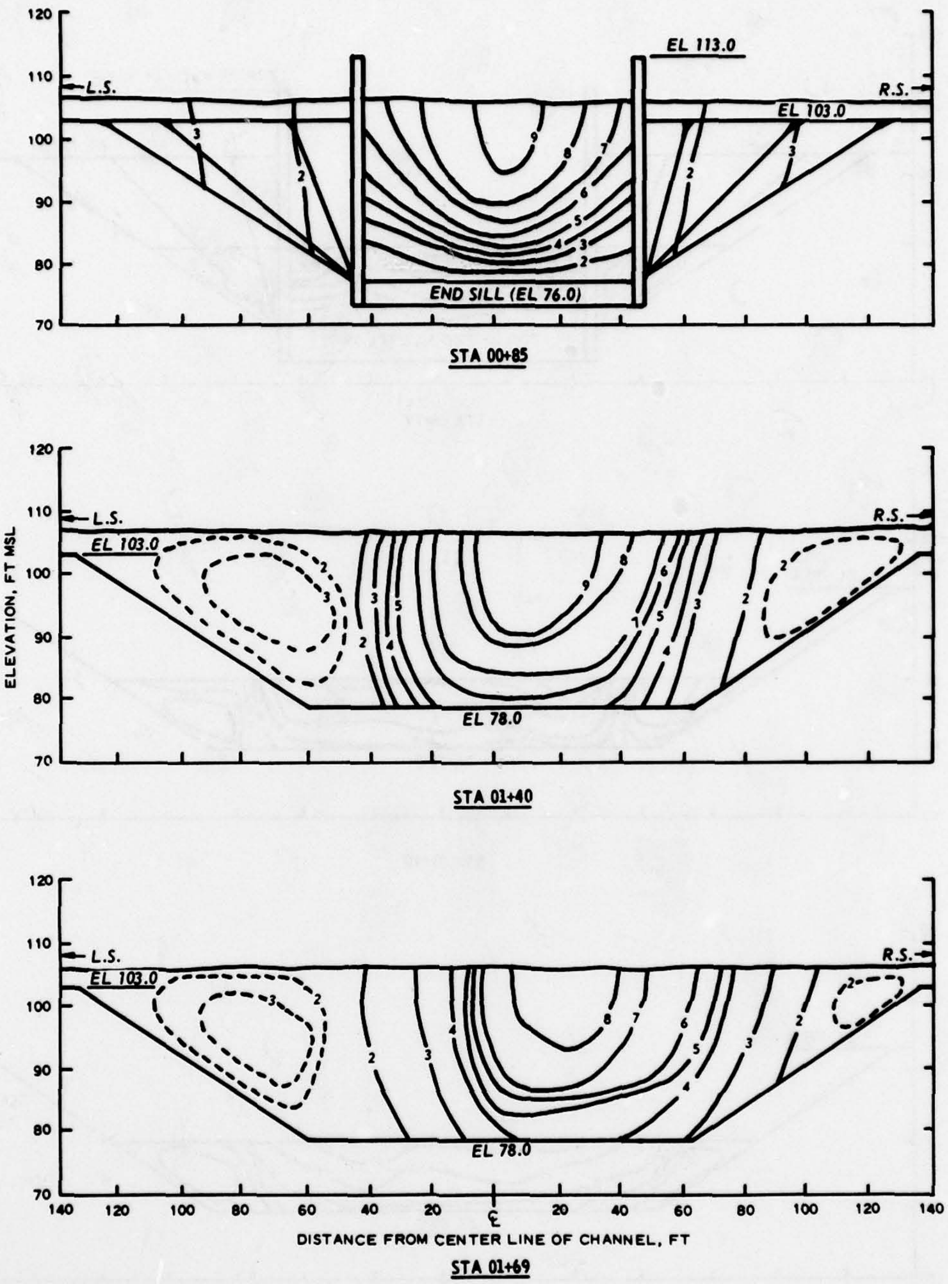
VELOCITY DISTRIBUTION IN EXIT CHANNEL
TYPE 5 STILLING BASIN
TYPE 6 APPROACH AND TYPE 3 ABUTMENT
ISOVELS

DISCHARGE 12,500 CFS
HEADWATER EL 108.9
TAILWATER EL 92.8



VELOCITY DISTRIBUTION IN EXIT CHANNEL
TYPE 5 STILLING BASIN
TYPE 6 APPROACH AND TYPE 3 ABUTMENT
ISOVELS

DISCHARGE 5,000 CFS
HEADWATER EL 104.6
TAILWATER EL 104.0



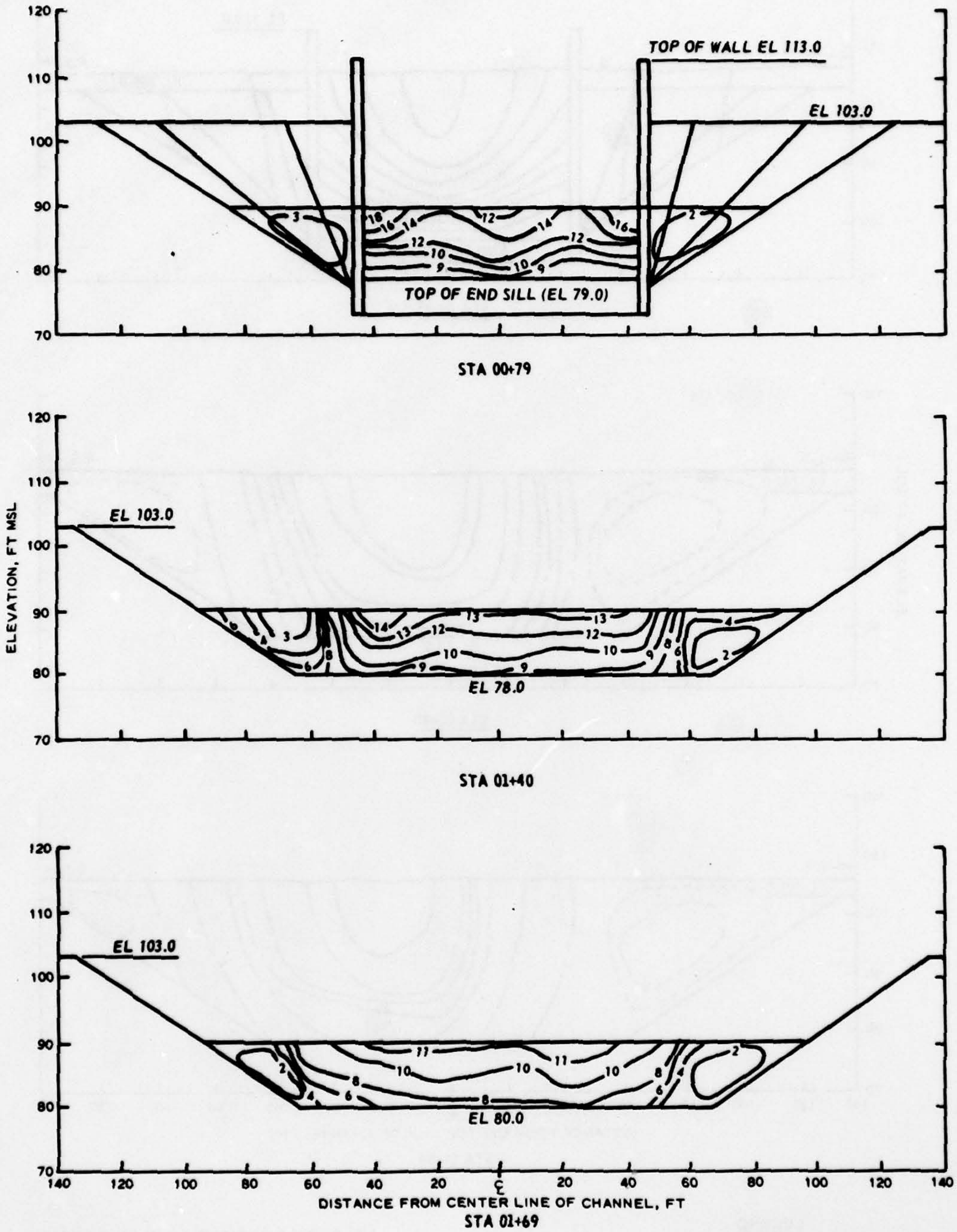
LEGEND

—— DOWNSTREAM

- - - UPSTREAM

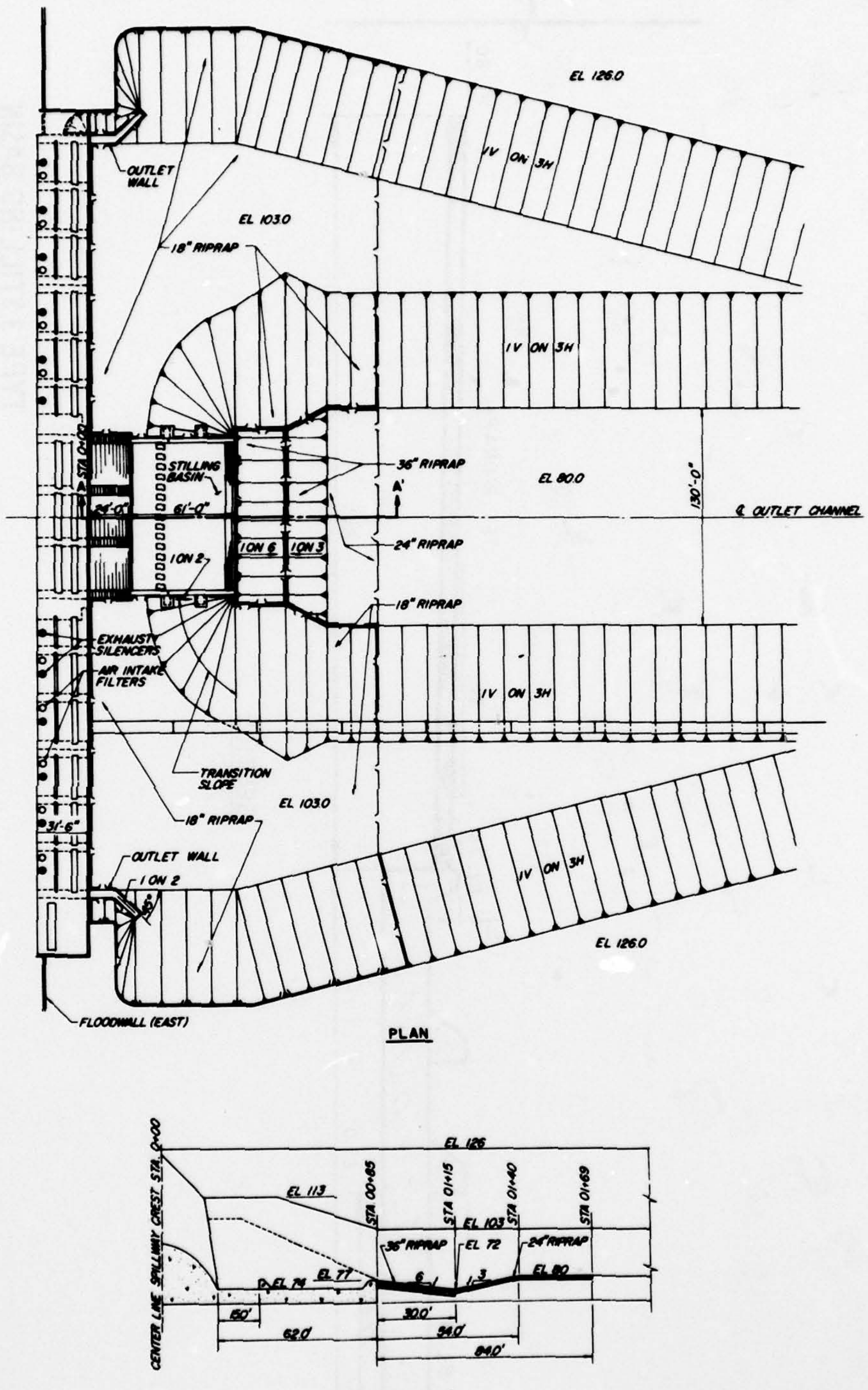
VELOCITY DISTRIBUTION IN EXIT CHANNEL
TYPE 5 STILLING BASIN
TYPE 6 APPROACH AND TYPE 3 ABUTMENT
ISOVELS

DISCHARGE 12,500 CFS
HEADWATER EL 108.9
TAILWATER EL 106.0

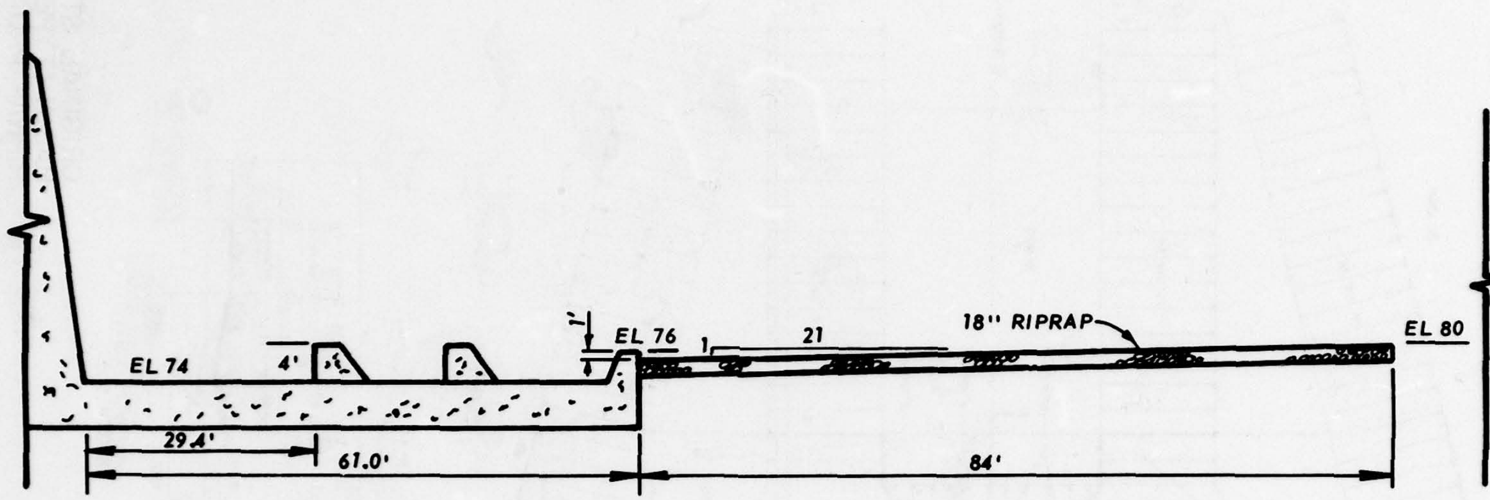


VELOCITY DISTRIBUTION IN EXIT CHANNEL
TYPE 7 STILLING BASIN
TYPE 6 APPROACH AND TYPE 3 ABUTMENT
ISOVELS

DISCHARGE 12,500 CFS
HEADWATER EL 108.9
TAILWATER EL 92.8

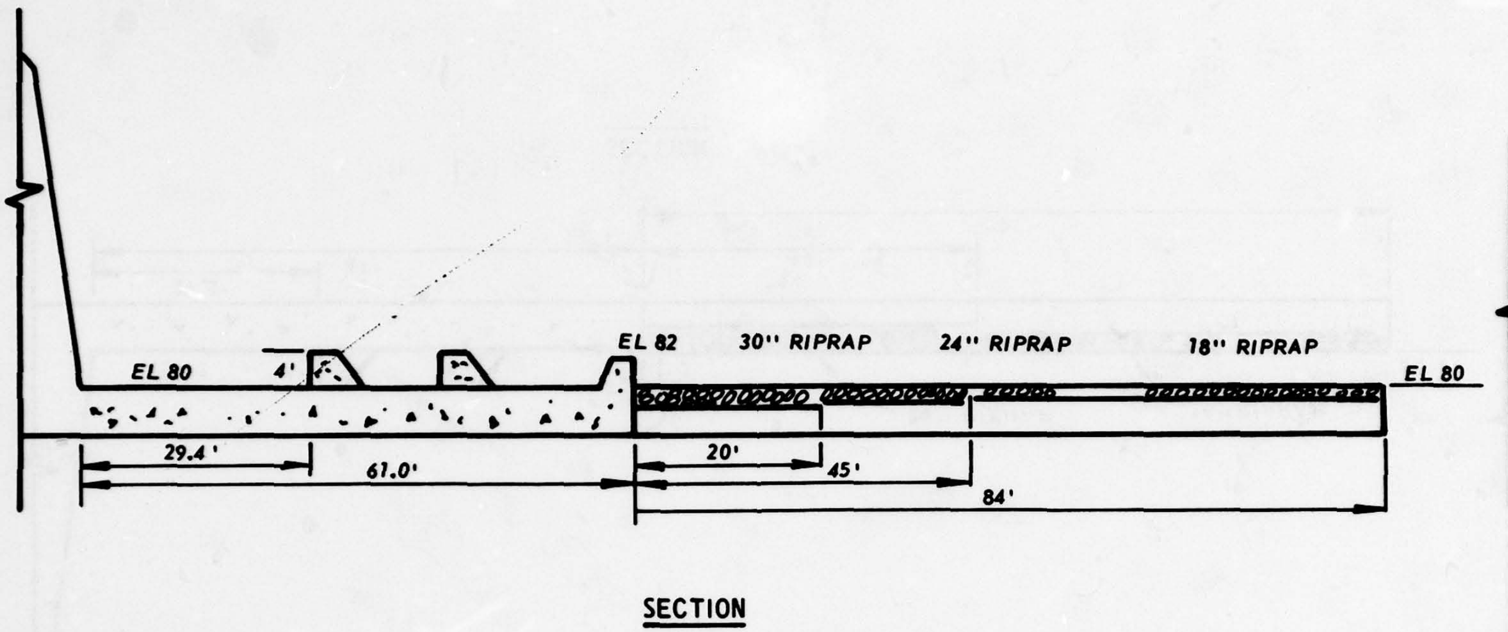


ORIGINAL STILLING BASIN (TYPE I) AND RIPRAP PROTECTION

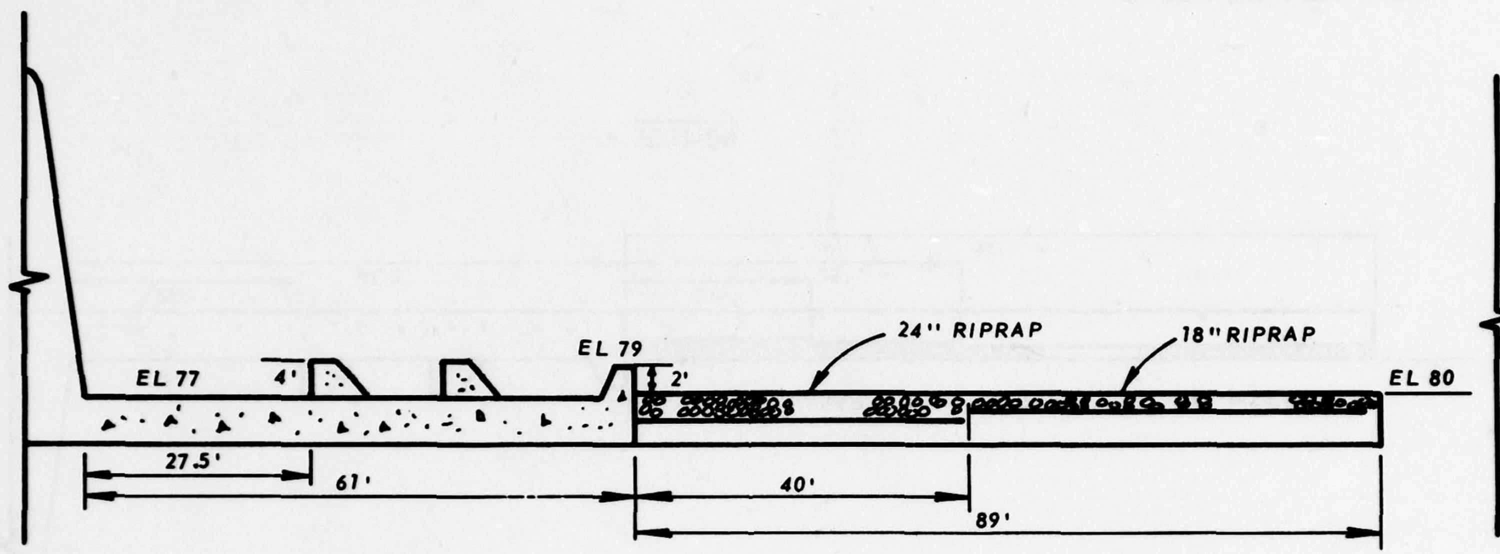


SECTION

**TYPE 3 STILLING BASIN
AND
TYPE 2 RIPRAP PROTECTION PLAN**

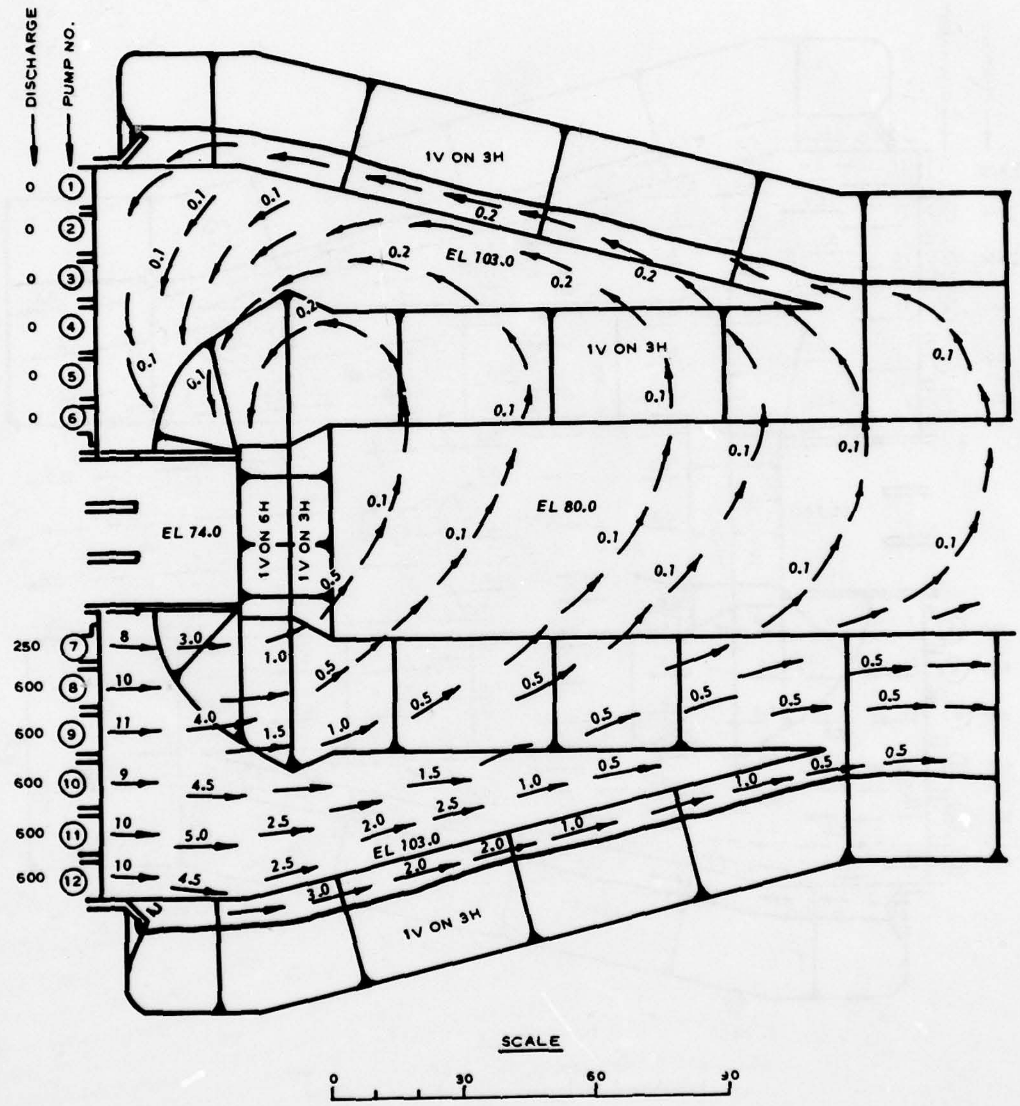


TYPE 6 STILLING BASIN
AND
TYPE 3 RIPRAP PROTECTION PLAN



SECTION

TYPE 7 STILLING BASIN
AND
TYPE 4 RIPRAP PROTECTION PLAN

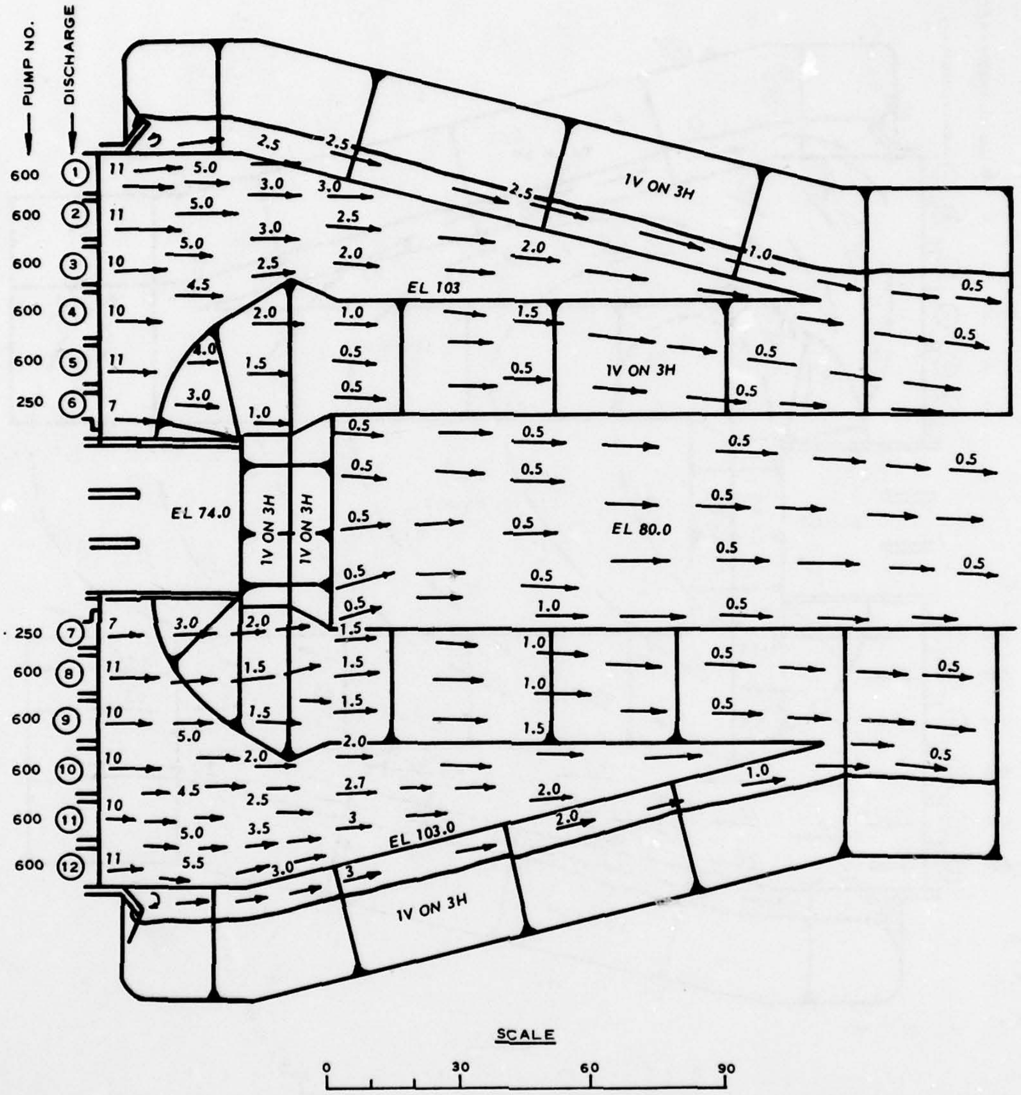


MAGNITUDE AND DIRECTION OF CURRENTS IN EXIT CHANNEL FOR PUMPED FLOWS

DISCHARGE 0, 250, AND 600 CFS
TAILWATER EL 110.0

NOTE: PUMPS OPERATING AND DISCHARGE PER PUMP IN CFS IS INDICATED AT OUTLETS.

ALL VELOCITIES (FPS) ARE 1 FT (PROTOTYPE) ABOVE BOTTOM



NOTE: PUMPS OPERATING AND DISCHARGE PER PUMP IN CFS IS INDICATED AT OUTLETS.

ALL VELOCITIES (FPS) ARE 1 FT (PROTOTYPE) ABOVE BOTTOM

MAGNITUDE AND DIRECTION OF CURRENTS IN EXIT CHANNEL FOR PUMPED FLOWS

DISCHARGE 250 AND 600 CFS
TAILWATER EL 110.0

PLATE 34

AD-A073 655 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS F/G 13/2
LAKE CHICOT PUMPING PLANT OUTLET STRUCTURE, ARKANSAS; HYDRAULIC--ETC(U)
JUN 79 E D ROTHWELL, B P FLETCHER
UNCLASSIFIED WES-HL-79-10 NI

2 OF 3
AD A073655

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DATE
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2 OF 3

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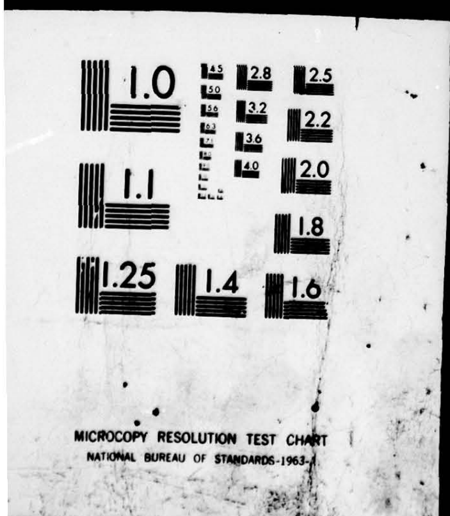

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Rothwell, Edward D
Lake Chicot pumping plant outlet structure, Arkansas; hydraulic model investigation / by Edward D. Rothwell and Bobby P. Fletcher. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1979.
33, [25] p., 34 leaves of plates : ill. ; 27 cm. (Technical report - U. S. Army Engineer Waterways Experiment Station ; HL-79-10)
Prepared for U. S. Army Engineer District, Vicksburg, Vicksburg, Mississippi.

1. Hydraulic models. 2. Lake Chicot pumping plant. 3. Outlet works. 4. Pumping stations. 5. Stilling basins. I. Fletcher, Bobby P., joint author. II. United States. Army. Corps of Engineers. Vicksburg District. III. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Technical report ; HL-79-10.
TA7.W34 no.HL-79-10

S-HL-79

END

DATE
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0-79

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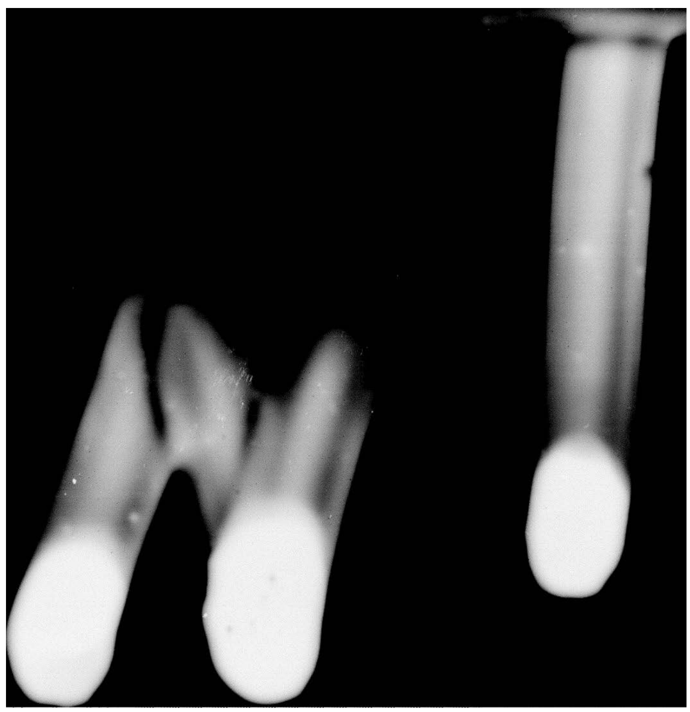

AD-A073 655 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS F/G 13/2
LAKE CHICOT PUMPING PLANT OUTLET STRUCTURE, ARKANSAS; HYDRAULIC--ETC(U)
JUN 79 E D ROTHWELL, B P FLETCHER
UNCLASSIFIED WES-HL-79-10 NL

3 OF 3

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SUPPLEMENTARY

INFORMATION

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3 OF 3

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1.0 1.1 1.25 1.4 1.6 1.8 2.0 2.2 2.5 2.8 3.2 3.6 4.0 4.5 5.0 5.6

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

SUPPLEMENTARY

INFORMATION

DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS
P. O. BOX 631
VICKSBURG, MISSISSIPPI 39180

IN REPLY REFER TO: WESHS

13 November 1979

Errata Sheet

No. 1

LAKE CHICOT PUMPING PLANT
OUTLET STRUCTURE, ARKANSAS

Hydraulic Model Investigation

Technical Report HL-79-10

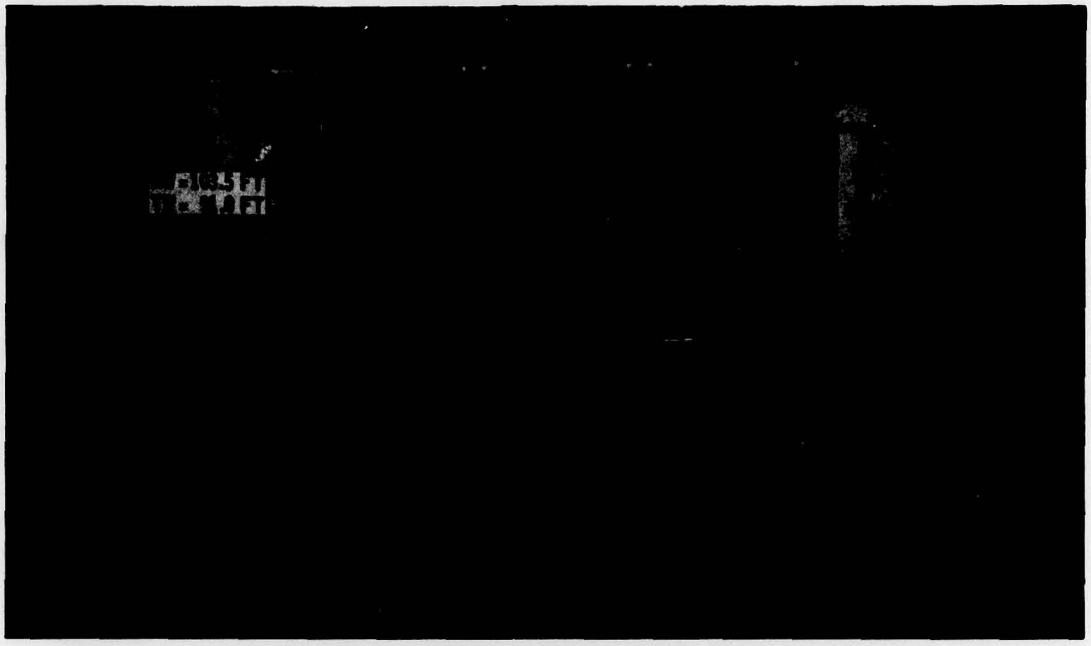
June 1979

1. Photos 1a and 1b:

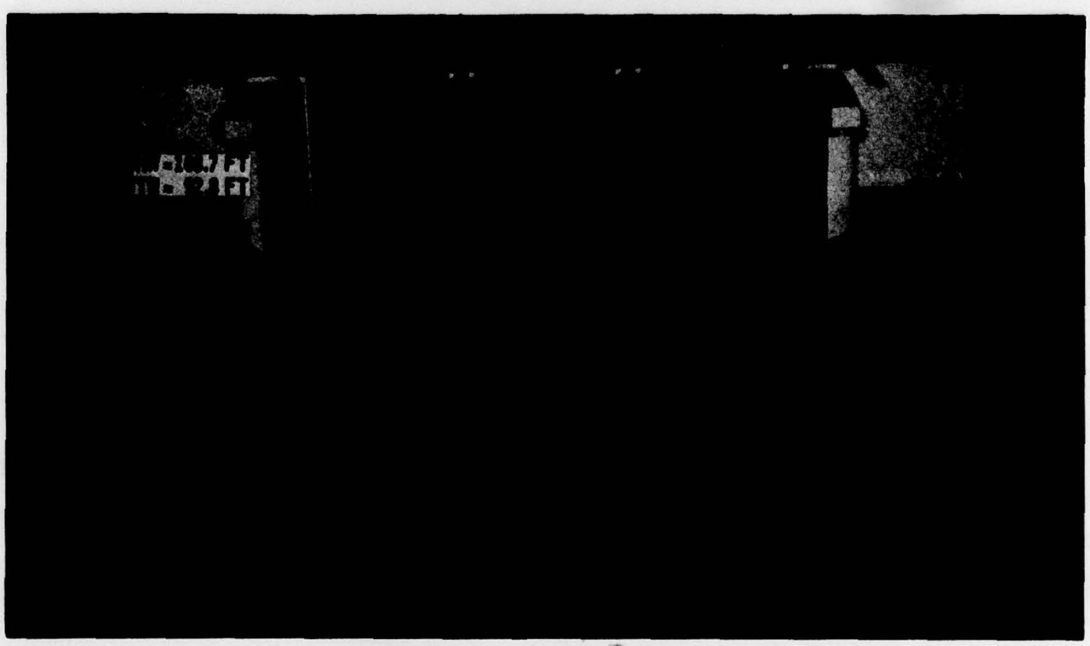
   Replace these photographs with the inclosed corrected page.

2. Photos 8a and 8b:

   Replace these photographs with the inclosed corrected page.



a. Discharge 5,000 cfs



b. Discharge 12,500 cfs

Photo 1. Approach flow conditions with type 2 approach and type 2 abutment

a. Discharge 5,000 cfs

b. Discharge 10,000 cfs

Photo 8. Flow conditions with type 7 stilling basin and type 4 riprap protection plan (sheet 1 of 2)