TECHNICAL REPORT H-73-7

MODEL STUDY OF TROTTERS SHOALS SPILLWAY

Hydraulic Model Investigation

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

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Sponsored by U. S. Army Engineer District, Savannah

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ARMY-MRC VICKSBURG, MISS.

FOREWORD

The model investigation reported herein was authorized by the Office, Chief of Engineers, U. S. Army, on 30 December 1969, at the request of the U. S. Army Engineer District, Savannah.

The study was conducted in the Hydraulics Laboratory of the U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi, during the period February 1970 to January 1972, under the direction of Messrs. E. P. Fortson, Jr., and H. B. Simmons, Retired Chief and Chief, respectively, of the Hydraulics Laboratory, and Mr. T. E. Murphy, Chief of the Structures Division. The tests were conducted by Messrs. W. A. Walker and B. P. Fletcher under the supervision of Mr. J. L. Grace, Jr., Chief of the Spillways and Channels Branch. This report was prepared by Messrs. Fletcher and Grace.

During the course of the investigation, Messrs. G. H. Mittendorf, T. Abeln, and B. L. Kittle of the U. S. Army Engineer Division, South Atlantic, and Messrs. T. J. Durrence, E. H. Williams, and F. B. Mallette of the Savannah District visited the WES to discuss test results and correlate these results with design studies.

Directors of the WES during the conduct of the study and the preparation and publication of this report were COL Levi A. Brown, CE, and COL Ernest D. Peixotto, CE. Technical Directors were Messrs. J. B. Tiffany and F. R. Brown.

CONTENTS

CONVERSION FACTORS, BRITISH TO METRIC UNITS OF MEASUREMENT

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

Multiply	By	To Obtain
inches	2.54	centimeters
feet	0.3048	meters
miles (U. S. statute)	1.609344	kilometers
kips (force)	4.448222	kilonewtons
foot-kips	1.355818	meter-kilonewtons
feet per second	0.3048	meters per second
cubic feet per second	0.0283168	cubic meters per second

SUMMARY

Tests of the spillway for the Trotters Shoals Dam were conducted on a 1:80-scale model to investigate flow conditions in the approach and exit channel and the performance of various elements of the structure. Particular emphasis was placed on the development of an energy dissipator that would provide satisfactory energy dissipation and exit channel flow conditions.

Approach flow conditions were improved by revising the left abutment to prevent a severe drawdown of the water surface in the vicinity of the left abutment.

The model indicated that, after the left abutment was modified, the spillway capacity was equal to that computed. Nappe separation from the downstream quadrant of the crest was prevented by extending the crest piers upstream and shifting the gate slots downstream relative to their original positions. These modifications improved pressure conditions within the gate slots as well as along the spillway crest.

Model tests indicated that the type 3 stilling basin provided more appropriate energy dissipation than the flip buckets and that it reduced the maximum velocities and the concentration of flow along the left side of the exit channel.

Model tests were also conducted to investigate the instantaneous forces induced by the hydraulic jump which were exerted on the monoliths composing the left stilling basin wall. These instantaneous forces were measured electronically, and the measurements should provide information pertinent to the structural design of the stilling basin walls.

Fig. 1. Vicinity map

MODEL STUDY OF TROTTERS SHOALS SPILLWAY

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. The Trotters Shoals damsite (fig. 1) is located on the Savannah River, about 16 miles* southeast of Elberton, Georgia, 29.9 miles below Hartwell Dam, and 37.4 miles above Clark Hill Dam. The dam will have a total length of 6235 ft and a top elevation of 495.0** and will include a 404-ft-long powerhouse and a 590-ft-long concrete spillway surmounted by ten 50-ft-wide by 44-ft-high tainter gates. The spillway will be designed to pass a maximum discharge of 800,000 cfs at a pool elevation of 490.0. The underdesigned spillway shape is based on a design head of 40.5 ft, rather than on the maximum anticipated head of 54 ft. The spillway will have a crest elevation of 436.0 and a net length of 500 ft. The abutments will have a radius of 5 ft, and the 10-ft-wide crest piers will have a semicircular nose shape.

2. Preliminary plans of the spillway called for energy dissipation to be provided by a flip bucket. However, subsequent model tests indicated that a relatively short and high hydraulic-jump stilling basin would be more appropriate. A general plan of the project and the portion simulated in the model study is shown in plate 1. Details of the original design structures are shown in plate 2.

Purpose of the Investigation

3. Although the spillway and appurtenances were designed in accordance with the best guidance currently available, a model analysis

* A table of factors for converting British units of measurement to metric units is presented on page vii.
** Elevations are in feet referred to mean sea level.

was desired in the interest of economy, performance, and good engineering practice. The model investigation was particularly concerned with verification of satisfactory flow conditions in the approach channel, at the abutments, through the tainter gates, over the spillway, in the flip bucket energy dissipator and an alternate conventional stilling basin, and in the powerhouse tailrace and exit channel. The study was also conducted for the purpose of determining the magnitude and frequency of dynamic loads on the stilling basin walls induced by the hydraulic jump as well as the performance characteristics of the spillway and the energy dissipators.

PART II: THE MODEL

Description

4. The comprehensive model of the Trotters Shoals spillway (fig. 2) was constructed to an undistorted linear scale of 1:80, and it reproduced all topography and structures in an area extending 2000 ft upstream and 2000 ft downstream from the axis of the dam. The portions of the model representing the approach, exit, and overbank areas were molded of sand-cement mortar to sheet metal templates and were given a brushed finish. The weir crest, tainter gates, crest piers, flip bucket, and nonoverflow sections of the dam were constructed of metal.

5. Water used in the operation of the model was supplied by pumps, and discharges were measured by means of venturi meters. Steel rails set to grade along the sides of the flume provided reference planes for measuring devices. Water-surface elevations were measured by means of point gages. Velocities were measured by means of a pitot tube and a stopwatch timing of dye and flotage over measured distances. Current patterns were determined by means of dye injected into the water and confetti sprinkled on the water surface.

Interpretation of Model Results

6. The accepted equations of hydraulic similitude, based on the Froudian criteria, were used to express the mathematical relations 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 Relation
Length	L_r	1:80
Area	$A_r = L_r^2$	1:6,400
Velocity	$V_r = L_r^{1/2}$	1:8.94

(Continued)

a. General view looking upstream

b. General view looking downstream

Fig. 2. The 1:80-scale spillway model of Trotters Shoals Dam (original design)

Dimension	Ratio	Scale Relation
Discharge	$Q_r = L_r^{5/2}$	1:57,243
Time	$T_r = L_r^{1/2}$	1:8.94
Force	$F_r = L_r^3$	1:512,000
Frequency	$f_r = 1/L_r^{1/2}$	1:0.11

7. Measurements of each of the dimensions or variables can be transferred quantitatively from model to prototype equivalents by means of the above scale relations.

PART III: TESTS AND RESULTS

Presentation of Data

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

Approach Flow Conditions

Original design

9. The general plan of the approach area is shown in plate 1 and fig. 2b. Flow conditions in the approach area were generally satisfactory, except in the immediate vicinity of the abutments. Approach flow conditions with bottom and surface currents indicated by dye and confetti, respectively, are shown in photo 1.

10. Flow conditions observed in the vicinity of the type 1 (original) abutments and tainter gates with the design discharge of 800,000 cfs and a controlled flow of 528,000 cfs are shown in photo 2. With the design discharge, considerable contraction of flow was observed due to lateral flow around the abutments and adjacent crest piers and submergence of the gate trunnions on the right sides of bays 1-3 and on the left sides of bays 8-10. Severe drawdown of the water surface was observed in the vicinity of the left abutment to the extent that intermittently the weir crest was completely exposed.

Type 2 left abutment

11. The severe drawdown observed at the original left abutment was reduced by modifying the abutment as shown in fig. 3. A comparison of the flow conditions produced by the types 1 and 2 left abutments is shown in photo 3. Water-surface profiles in bay 1 produced by the types 1 and 2 left abutment designs are shown in plate 3, and the abutment contraction coefficients with these two abutments are presented in plate 4. These coefficients were computed based on discharges and

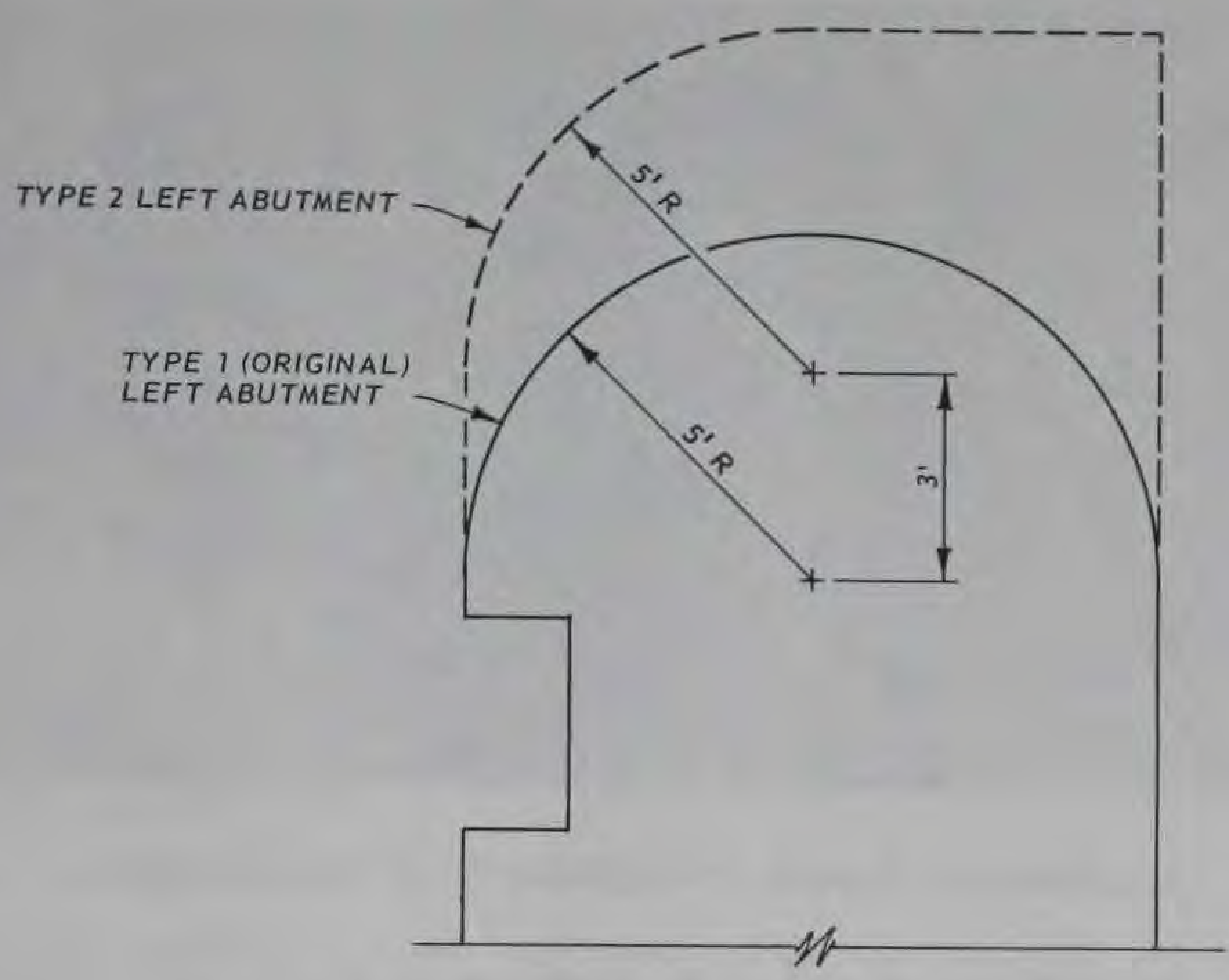

Fig. 3. Modification of left abutment

heads indicated by the model, weir discharge coefficients indicated by Corps of Engineers Hydraulic Design Chart (HDC) 111-3, and pier contraction coefficients indicated by HDC 111-5. The curve obtained with the type 2 left abutment (plate 4) is similar to the suggested design curve presented in HDC 111-3/1.

Alternate left
abutment terminal cones

12. Tests were conducted to determine the minimum clearance that could be permitted between the left abutment terminal cone and the spillway crest without adversely affecting the hydraulic capacity or flow conditions near the abutment and in the energy dissipator. The various cone designs tested are shown in plate 5. Tests of the types 1-4 cones with uncontrolled and gated flows indicated that neither the hydraulic capacity of the spillway nor the flow conditions in the stilling basin were affected by raising the elevation of the terminal cone. The type 4 left abutment terminal cone is shown in fig. 4. Velocities and current patterns observed along the types 1 (original), 2, and 4 terminal cones are presented in plates 6, 7, and 8, respectively. The velocities of

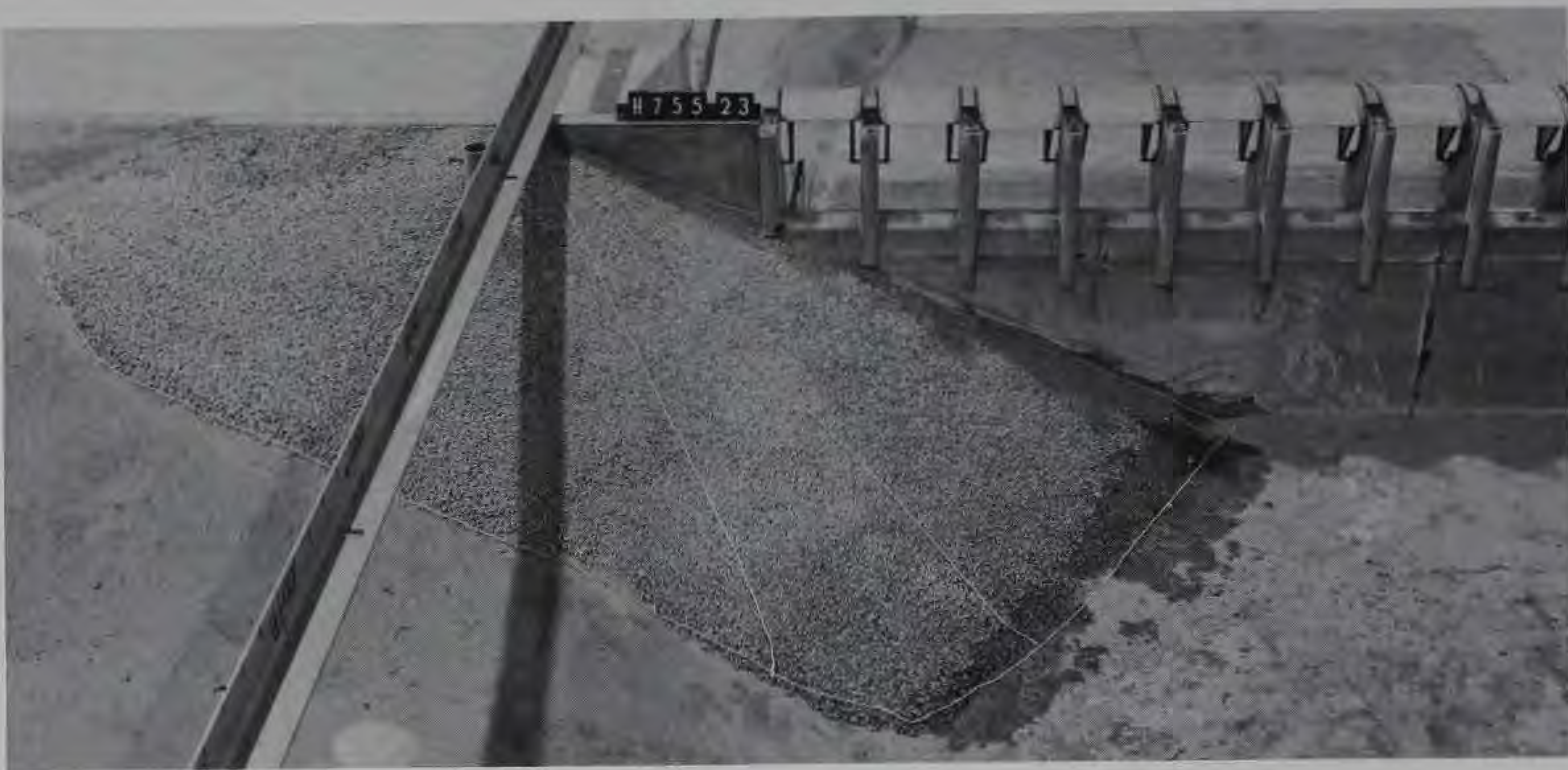

Fig. 4. Type 4 left abutment terminal cone

flows 5 ft above the cone and 5 ft upstream from the left abutment were less than 5 fps with the types 1 and 2 designs. However, as the terminal cone was raised and increased in size, this area was subjected to velocities as great as 16 fps (plate 8).

Weir and Crest Piers

Weir shape

13. The weir crest (elevation 436.0) was underdesigned; it was shaped for a design head H_d of 40.5 ft, although a maximum head of 54 ft is anticipated. The upstream quadrant was shaped to the curve described by the equation

$$Y = 0.724 \frac{(x + 0.270H_d)^{1.85}}{H_d^{0.85}} + 0.126H_d - 0.4315H_d^{0.375} (x + 0.270H_d)^{0.625}$$

and the downstream quadrant was formed to the curve described by the equation

$$x^{1.85} = 2.0H_d^{0.85}Y$$

Uncontrolled flow

14. The capacity of the original design weir for anticipated pool elevations was less than that computed (plate 9). The computed rating curve was obtained by the weir formula $Q = CLH_e^{3/2}$, where Q is the discharge in cubic feet per second, C is the discharge coefficient as indicated by HDC 122-1, H_e is the energy head above the crest in feet, and L is the effective length of the spillway determined from the expression $L = 500 - 2H_eK$, in which K is the summation of the pier and abutment contraction coefficients as shown in HDC 111-5 and HDC 111-3/1, respectively. Subsequent model tests revealed that the type 2 left abutment increased the capacity of the weir structure to that originally computed (plate 9).

15. The relatively short length of the original design crest piers and the position of the bulkhead slots upstream of the weir crest permitted the nappe to separate from the spillway with heads on the crest equal to or greater than 50 ft (photo 4). The nappe separation was eliminated by extending the piers 3 ft upstream and by shifting the center of the stop-log slots downstream to a position within 2.13 ft rather than 3.80 ft of the weir crest, as shown in fig. 5. The shifting of the stop-log slots will improve pressure conditions within the slots and along the crest and piers.*

Controlled flow

16. Flow conditions for all anticipated pool elevations and gate openings less than 21 ft were considered satisfactory. Flow conditions for the maximum gate opening of 30.6 ft and a pool elevation of 485.0 were considered tolerable. Intermittent vortices were observed upstream of all gates, and fixed vortices with an internal diameter of about 8 ft were observed at the abutments. Fluctuations of the water surface upstream of the tainter gates were less than 2 ft and of a random rather than a periodic nature. Flow conditions resulting with a pool elevation

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* See plate 11 of "Investigations of Various Shapes of the Upstream Quadrant of the Crest of a High Spillway; Hydraulic Laboratory Investigation," by E. S. Melsheimer and T. E. Murphy, Research Report H-70-1, Jan 1970, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

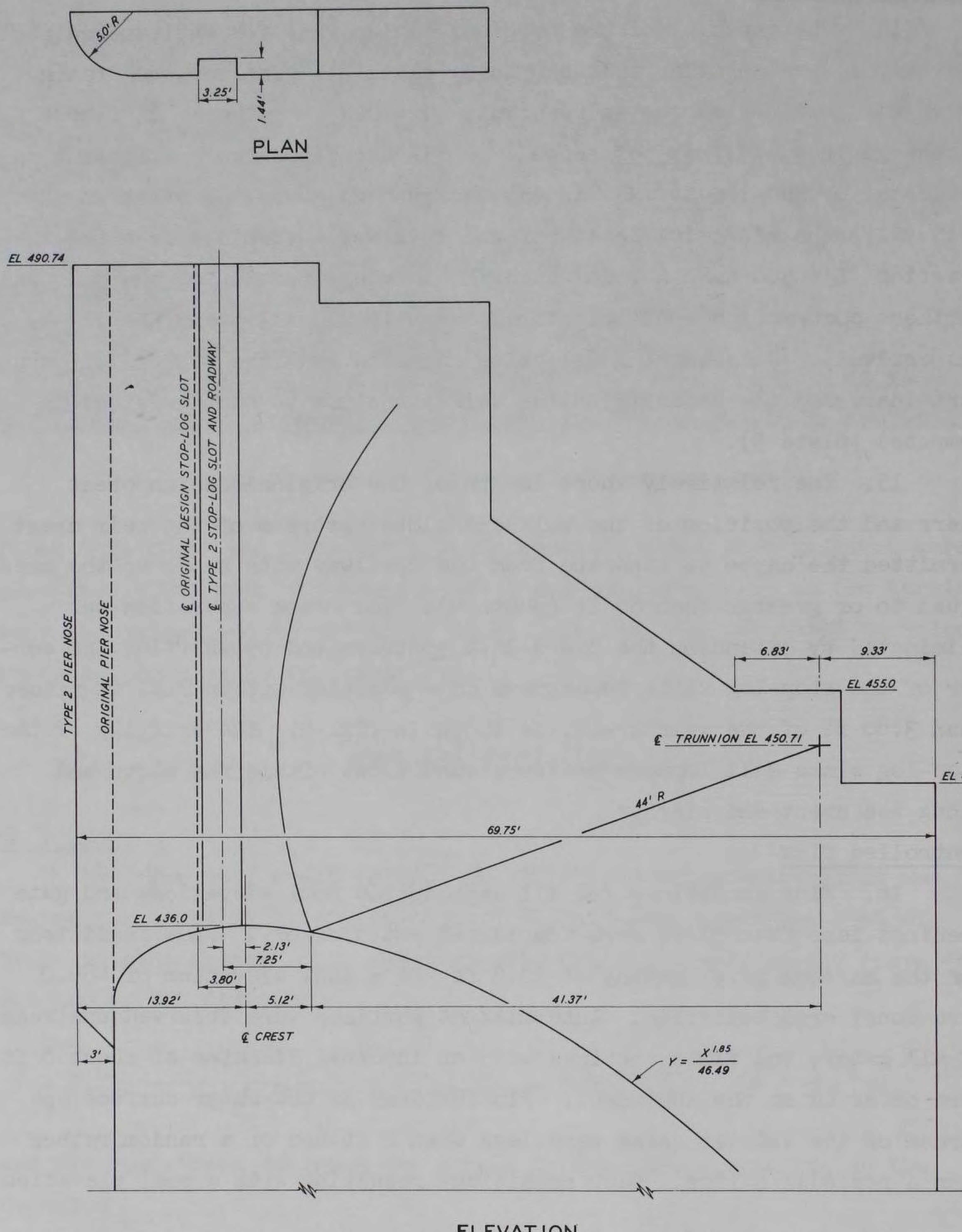


Fig. 5. Modifications of crest piers and stop-log slots

of 485.0 and all gates open 30.6 ft are shown in photo 2b.

Flip Bucket

Type 1 (original) flip bucket

17. The original design flip bucket consisted of a 50-ft-radius bucket with a lip that terminated at a 30-deg angle with a horizontal plane (plate 2). The lip of the bucket was set at elevation 353.0, 3 ft below the expected tailwater stage for the design discharge. The anticipated tailwater rating curves are shown in plate 10.

18. Flows above the standard project flood of 360,000 cfs when discharged from the flip bucket into the exit channel caused turbulent wave action and a concentration of flow along the left side of the channel. Performances of the flip bucket for two sets of flow conditions are shown in photo 5. The flip bucket provided a throw distance of 220 ft with the design discharge of 800,000 cfs. With controlled flows equal to or less than 40,000 cfs, the nappe adhered to the 4-ft horizontal lip of the flip bucket, as shown in photo 6a. The nappe was made to spring clear, as shown in photo 6b, by artificially venting the lip. General flow patterns and velocities observed in the exit channel with the type 1 flip bucket are shown in photo 7. Velocities measured in the exit channel for various discharges are shown as isovels in plates 11-14.

Type 2 (recommended) flip bucket

19. Flow conditions in the exit channel were improved by increasing the flip angle to 45 deg, terminating the lip of the flip bucket sharply, and raising the lip to elevation 357.0 (fig. 6). Various flow conditions with the type 2 (recommended) flip bucket are shown in photo 8. The bucket would not flip the jet at the minimum power pool elevation of 470.0 with discharges less than 33,000 cfs, due to the relatively large difference in elevation between the low point and the lip of the type 2 flip bucket. A splash apron should be provided at the toe of the structure to prevent erosion at low discharges, if the type 2 flip bucket is adopted for prototype construction. With flows in excess

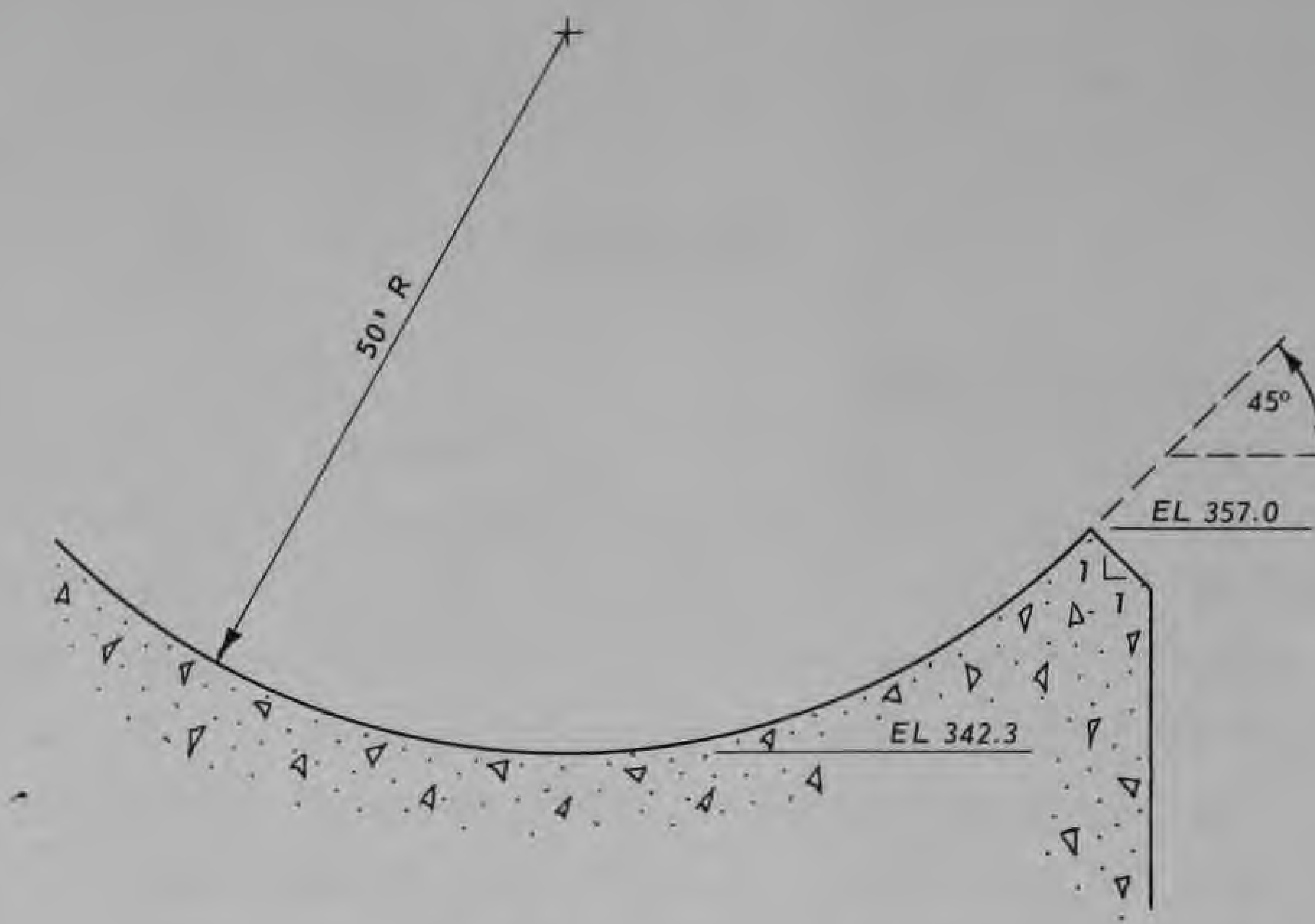


Fig. 6. Type 2 (recommended) flip bucket

of 33,000 cfs, sufficient energy was available to prevent submergence of the jet, and it sprang clear from the lip. This flip bucket provided a throw distance of 280 ft with the design discharge of 800,000 cfs. Velocities measured in the exit channel downstream of the type 2 flip bucket with various discharges are presented as isovels in plates 15-17.

Stilling Basin

20. Due to unsatisfactory energy dissipation with the flip bucket designs, tests were conducted to develop a relatively high and short stilling basin that would provide adequate energy dissipation.

Type 2 stilling basin

21. Pilot tests were conducted in a 1-ft-wide flume to obtain information pertinent to the design of an adequate stilling basin. The section model was constructed to a 1:120 scale, which permitted reproduction of a 120-ft length of spillway. The crest was surmounted by one 10-ft-wide pier. Tests with the apron at various elevations both with and without various arrangements of baffles and end sills resulted in the type 2 design stilling basin shown in fig. 7. Tests conducted in the general model with the type 2 basin and a discharge of

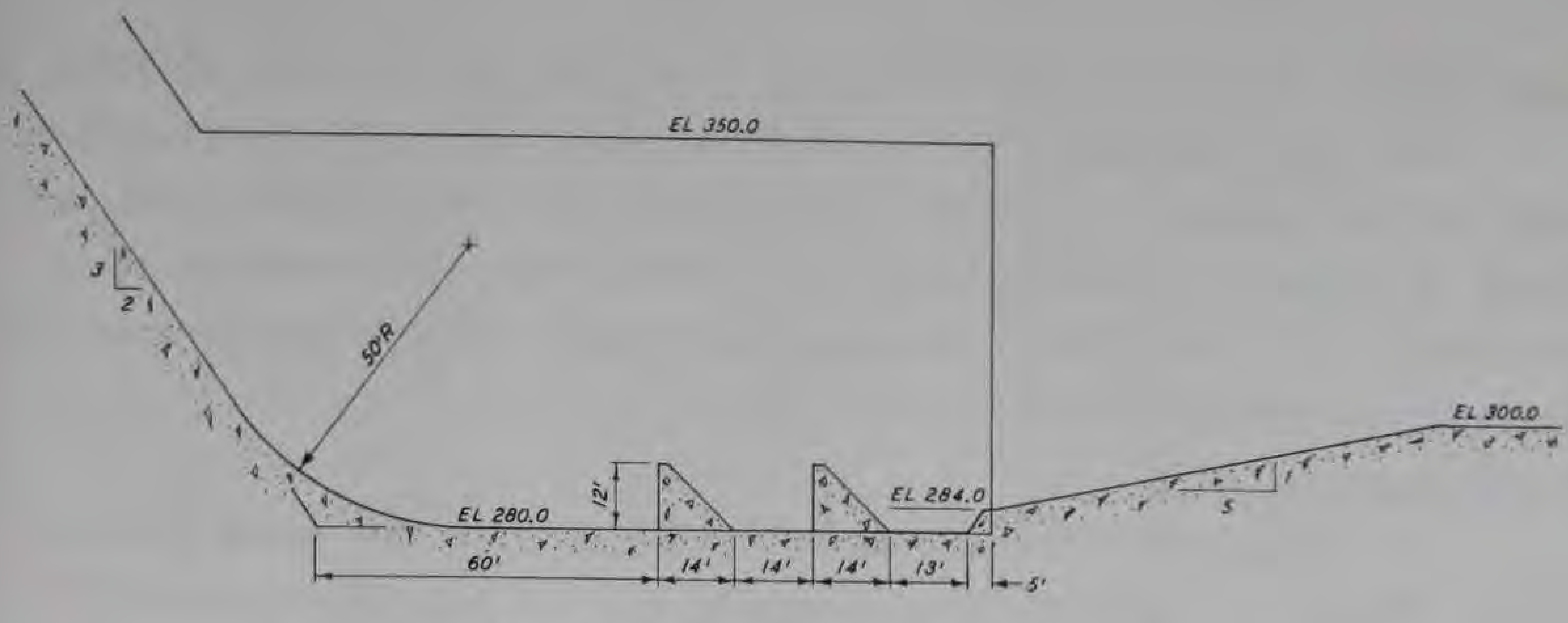


Fig. 7. Type 2 stilling basin

800,000 cfs indicated that the two rows of 12-ft-high baffle piers caused excessive contraction of flow in the basin and generated 15-ft-high standing waves in the exit channel (photo 9). With a discharge of 360,000 cfs and a tailwater elevation of 343.0, the type 2 stilling basin produced a roller action rather than a hydraulic jump action.

Type 3 (recommended) stilling basin

22. Additional tests in the general model resulted in a hydraulically favorable stilling basin design (type 3) whose details are shown in fig. 8. Performance of the type 3 basin was adequate for all expected flows. Basin actions for various discharges are presented in photo 10. At the design discharge, the basin maintained hydraulic jump action for tailwater elevations as low as 348.0. At tailwater elevations below 348.0, spray action occurred as shown in photo 11. The

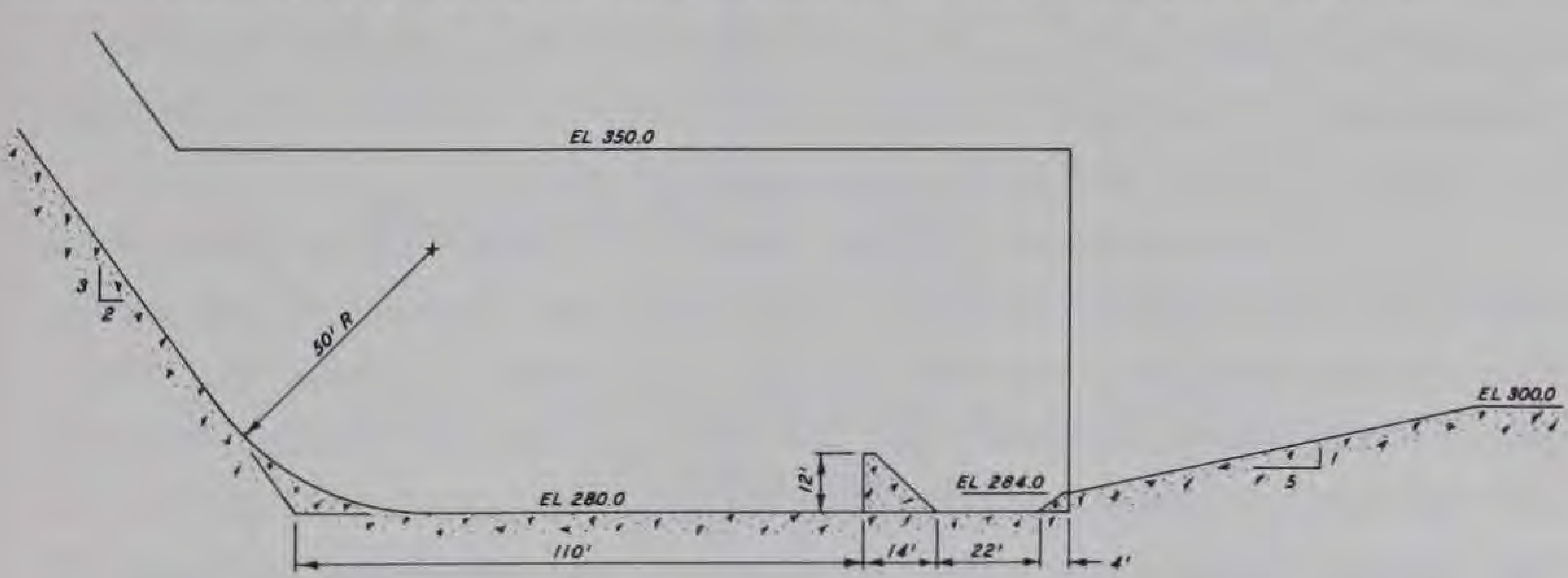


Fig. 8. Type 3 (recommended) stilling basin

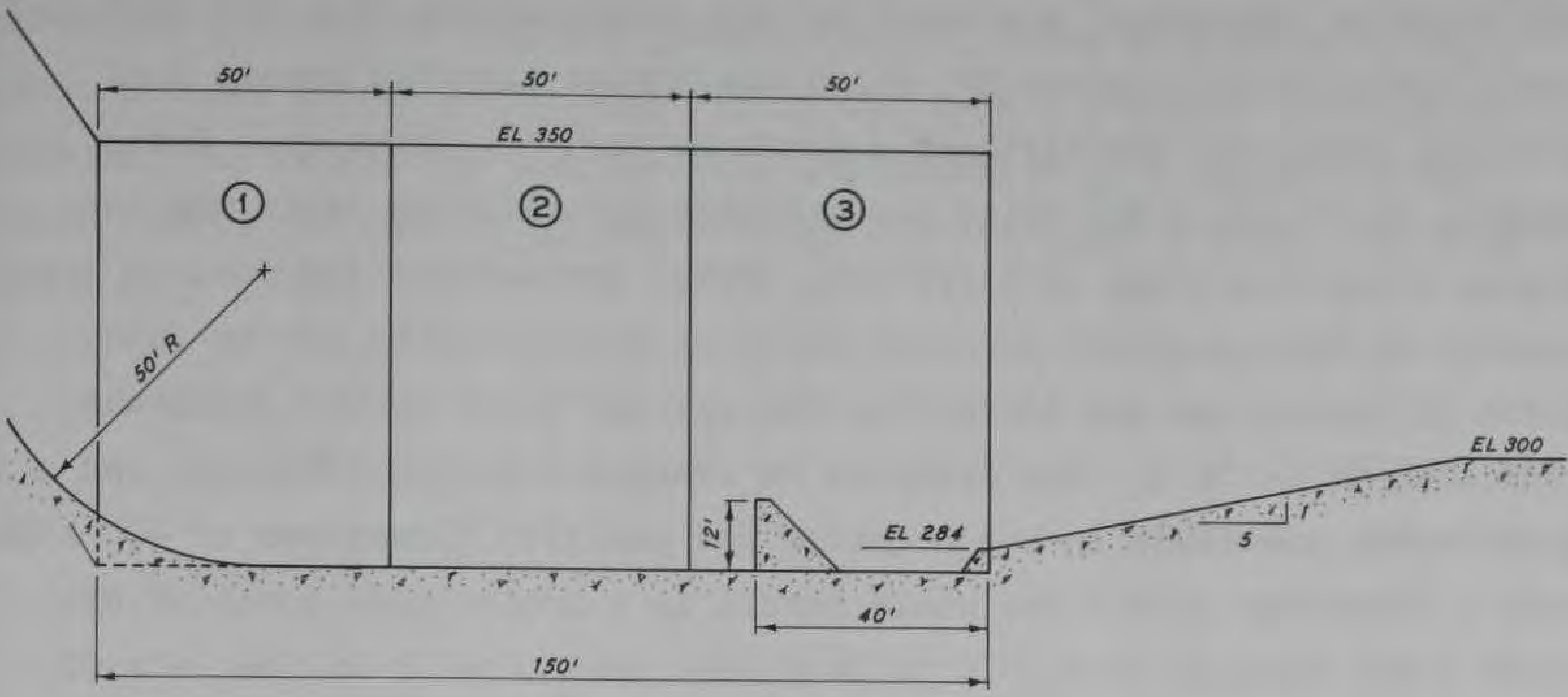
spray characteristics of the basin are described by the curve in plate 18.

23. Water-surface profiles observed along the left stilling basin wall and the maximum velocities in the vicinity of the baffle piers are shown in plate 19. Similar water-surface profiles were observed along the right wall. Velocities measured in the exit channel for various discharges are presented as isovels in plates 20-23.

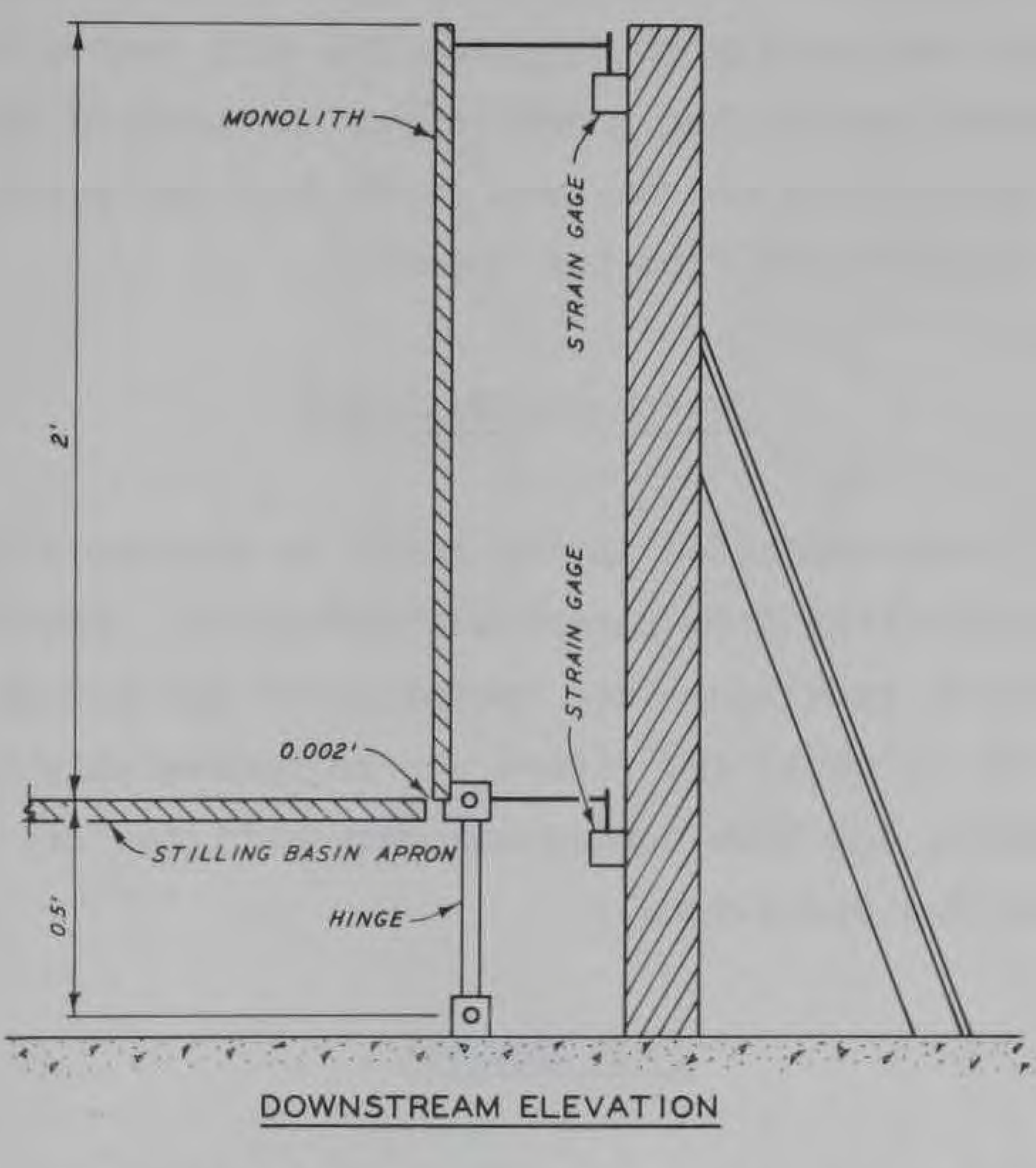
Stilling basin walls

24. Model tests were conducted to investigate the instantaneous forces induced on the monoliths composing the stilling basin walls resulting from the hydraulic jump. A section of the Trotters Shoals spillway was closely reproduced to a 1:100 scale in an existing 2.5-ft-wide flume that was being utilized for a general study of dynamic loads on stilling basin walls. The section model consisted of a 250-ft-long ogee weir, the type 3 basin, 80 ft of the exit channel, and a stilling basin wall divided into three 50-ft-long monoliths, as shown in fig. 9. The top elevation of the model monoliths was higher than that indicated in fig. 9 and did not permit return flow over the top of the wall at the maximum tailwater elevation of 356.0. It was considered that the prevention of return flow would indicate conservative or greater forces acting on the monoliths. A water level equal to and caused by the tailwater was permitted on the backside of the stilling basin wall to simulate that anticipated in the powerhouse tailrace. Each monolith was constructed of machined aluminum and was supported in a vertical plane by a hinge that would permit movement only in a direction normal to the face of the sidewalls (fig. 9). Each monolith was equipped with two strain gages, which have a maximum deflection of 0.001 in., to measure the dynamic forces induced by the hydraulic jump.

25. A typical record of the results of a test for a given set of hydraulic conditions to determine the direction, frequency, and magnitude of the externally applied forces is presented in plate 24. The data obtained were analyzed, and the values of the magnitude, frequency, and overturning moment, relative to the base of the wall at the top of the stilling basin apron, were computed and are tabulated in table 1. The values listed under the columns labeled "predominant amplitude,"



ELEVATION ALONG SIDEWALL



DOWNSTREAM ELEVATION

Fig. 9. Details of sidewall monoliths and experimental facilities

"maximum amplitude in the positive direction," and "maximum amplitude in the negative direction" are equal to the displacements from the average force datum shown in plate 24, which are always directed toward the stilling basin and are assigned a positive sign or direction. Forces tending to displace the walls outward from the stilling basin are assigned a negative sign or direction. Thus, predominant and maximum magnitudes of the resultant force or shear on the monoliths can be calculated by increasing and decreasing the average force by the amplitude indicated in table 1. For example, an average force of 3070 kips and a predominant amplitude in the negative and positive directions of 400 kips with a frequency of 0.3 cps would result in a predominant positive dynamic force ranging from 2670 to 3470 kips periodically at the rate of 0.3 cps and tending to displace the wall toward the stilling basin. An average force of 3070 kips and randomly occurring maximum amplitudes in the negative and positive directions of 1000 kips would result in a randomly occurring maximum force in the positive direction ranging from 2070 to 4070 kips and tending to displace the wall toward the stilling basin. The maximum overturning moment about the base of the monolith is obtained by multiplying the maximum force detected with the upper strain gage by its distance from the apron.

Powerhouse Tailrace

26. Tests were conducted in the model to observe flow conditions in the tailrace resulting from powerhouse operation. Bottom velocities and current patterns resulting from operation of the powerhouse at maximum flow are shown in photo 12. There was no appreciable turbulence below the structure, and flow conditions were satisfactory for all expected conditions and operations.

Exit Channel

27. About 2000 ft of the exit channel was reproduced in the model. Currents observed in the exit channel with the original and type 2 flip

bucket designs were excessive and flow was severely concentrated along the left side of the channel (plates 11-17). Flow conditions in the exit channel were greatly improved by the type 3 stilling basin which reduced the velocities considerably and provided essentially uniform distribution of flow along the left side of the exit channel, as shown in plates 20-23.

PART IV: DISCUSSION

28. The addition of a relatively small fillet to the left abutment was most effective in reducing the reentrant flow effect observed with a semicircular abutment and in reducing the resulting severe contraction and drawdown of flow. The importance of the positions of both the nose of the crest piers and the center of the bulkhead slots relative to the spillway crest to prevent adverse pressure conditions and separation of the nappe from the crest was reemphasized in this study. Care should also be taken to ensure that the crest piers are sufficiently long to extend downstream of the subatmospheric pressure zone on the spillway crest to prevent aeration of the nappe at the end of the crest pier, loss of capacity, and, even worse, an unstable nappe. A pier that extends a distance of $1.2H_d$ downstream of the spillway crest should be sufficiently long in most cases.*

29. The results of tests to investigate the tendency for surging on the tainter gates support the recommendations presented in Engineer Technical Letter 1110-2-51 for high overflow spillways. The maximum gate opening (30.6 ft) for which the tainter gates will control discharge is 0.625 times the 49-ft maximum head on the spillway crest with controlled releases. The ratio of 1.02 for the gate bay width to the maximum head on the spillway crest with controlled flows is sufficiently greater than the minimum of 0.8 recommended for the original pier-length to gate-bay-width ratio of 0.27 and is only slightly less than the recommended value of 1.05 for cases when the pier-length to gate-bay-width ratio is between 0.3 and 0.4. The revised pier-length to gate-bay-width ratio required to prevent nappe separation is 0.33.

30. Increasing the angle of the flip bucket to the optimum value of 45 deg permitted the nappe to be thrown 60 ft farther downstream;

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* See "Investigations of Various Shapes of the Upstream Quadrant of the Crest of a High Spillway; Hydraulic Laboratory Investigation," by E. S. Melsheimer and T. E. Murphy, Research Report H-70-1, Jan 1970, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

however, velocities as great as 90 fps can be expected in the exit channel. The additional height of the lip above the low point reduces the effectiveness of the 50-ft-radius flip bucket, since it drowns out all flows less than 33,000 cfs with a minimum power pool elevation of 470.0.

31. Although absolute magnitudes cannot be stated based upon present knowledge and the state-of-the-art, it is believed that flip bucket energy dissipators are not practical for nappes thicker than 10 ft or for unit discharges as large as 1000 cfs per foot of width. It is realized, however, that structural safety and economy will dictate which form of energy dissipator will be adopted for prototype construction. However, it is recommended that the reaeration characteristics of the flip bucket versus the relatively high and short stilling basin be considered in the final selection of the energy dissipator. This aspect is considered particularly pertinent for prevention and mitigation of adverse gas embolism effects in various species of fish.

Table 1

Amplitude and Frequency of Forces Acting on Stilling Basin Wall Monoliths

Discharge cfs	Pool Elevation ft, msl	Tailwater Elevation ft, msl	Monolith No.	Average Force* kips	Predominant Frequency cps	Predominant Amplitude kips	Maximum Amplitude + Direction kips	Maximum Amplitude - Direction kips	Maximum Overturning Moment About Apron ft-kips
360,000	469	343	1	3070	0.3	400	1000	1000	80,000
360,000	469	343	2	2800	0.3	400	700	700	52,000
360,000	469	343	3	1300	0.3	300	700	700	20,000
800,000	490	356	1	5700	0.3	600	1000	1000	200,000
800,000	490	356	2	6000	0.3	900	1500	1500	200,000
800,000	490	356	3	4000	0.3	1100	2000	2000	80,000

* All average forces are positive and directed toward stilling basin.

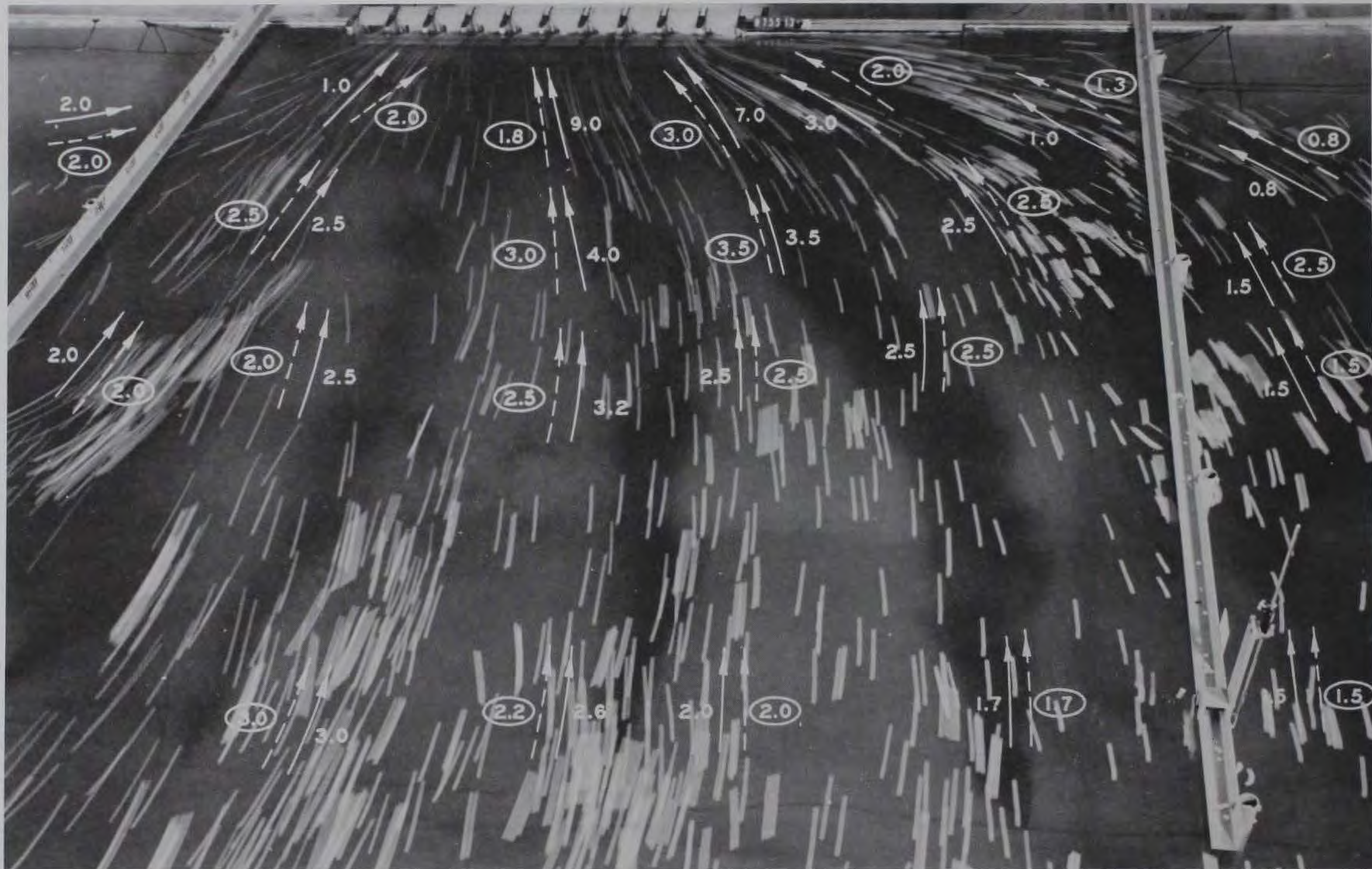
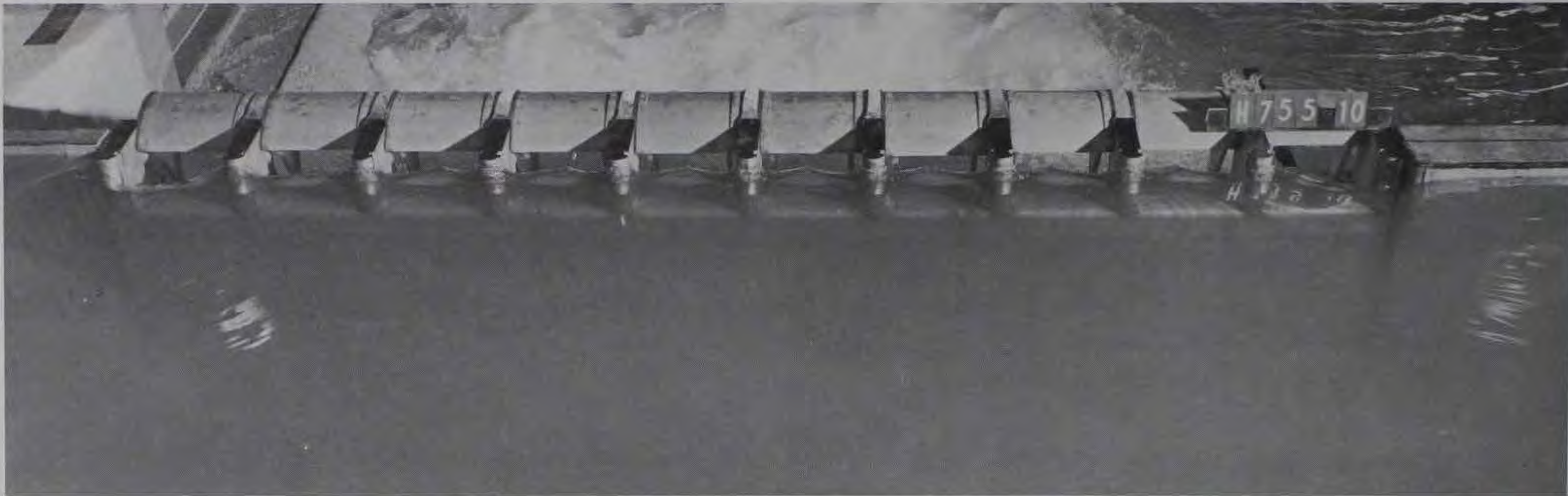


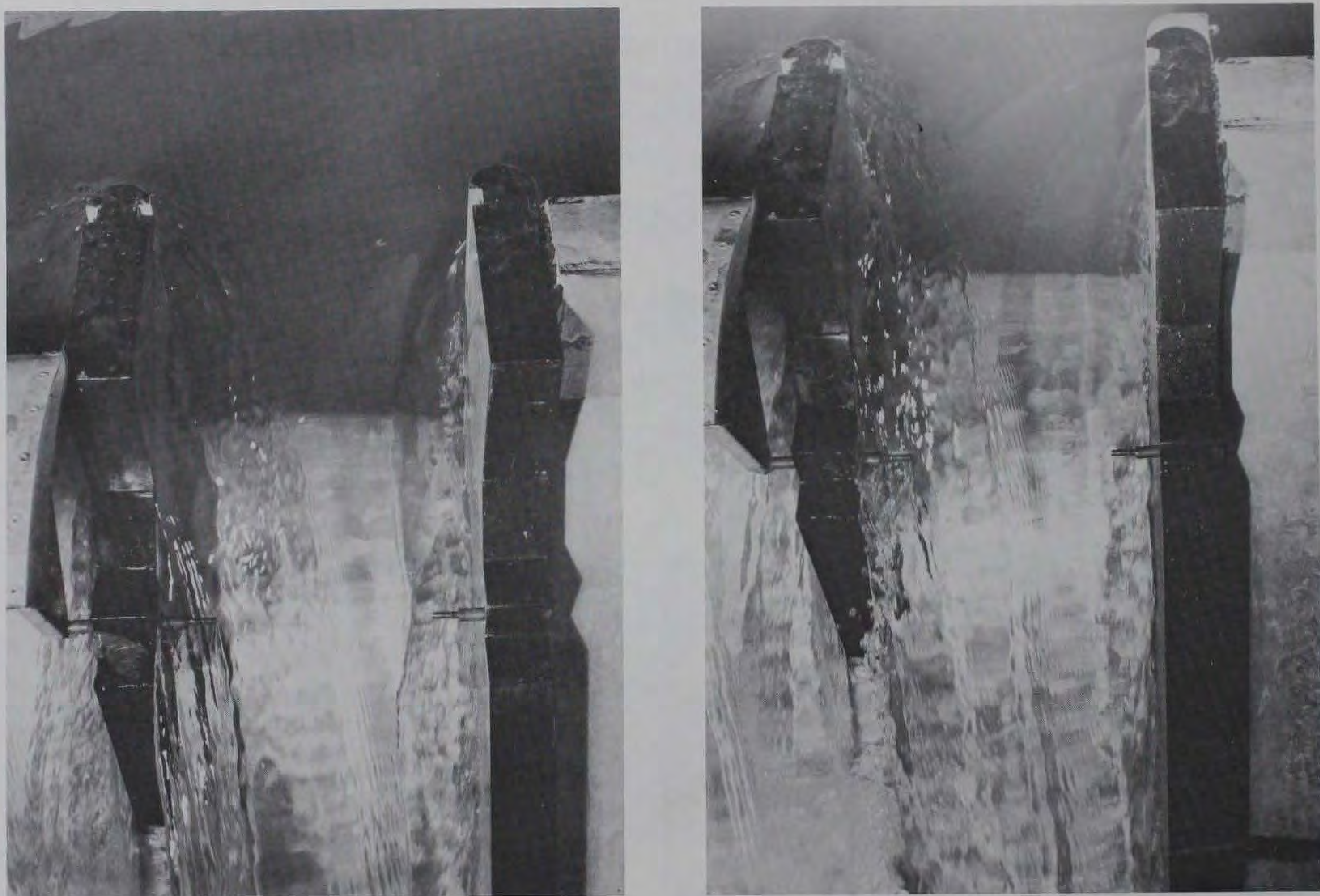
Photo 1. Approach flow conditions; discharge 800,000 cfs, pool el 490.0. Circled numbers denote bottom velocities in prototype feet per second



a. Discharge 800,000 cfs, pool el 490.0

b. Discharge 528,000 cfs, pool el 485.0, all gates open 30.6 ft

Photo 2. Flow conditions in vicinity of type 1 (original) abutments and tainter gates



a. Type 1 (original) left abutment

b. Type 2 left abutment

Photo 3. Flow conditions at left abutment; discharge 800,000 cfs

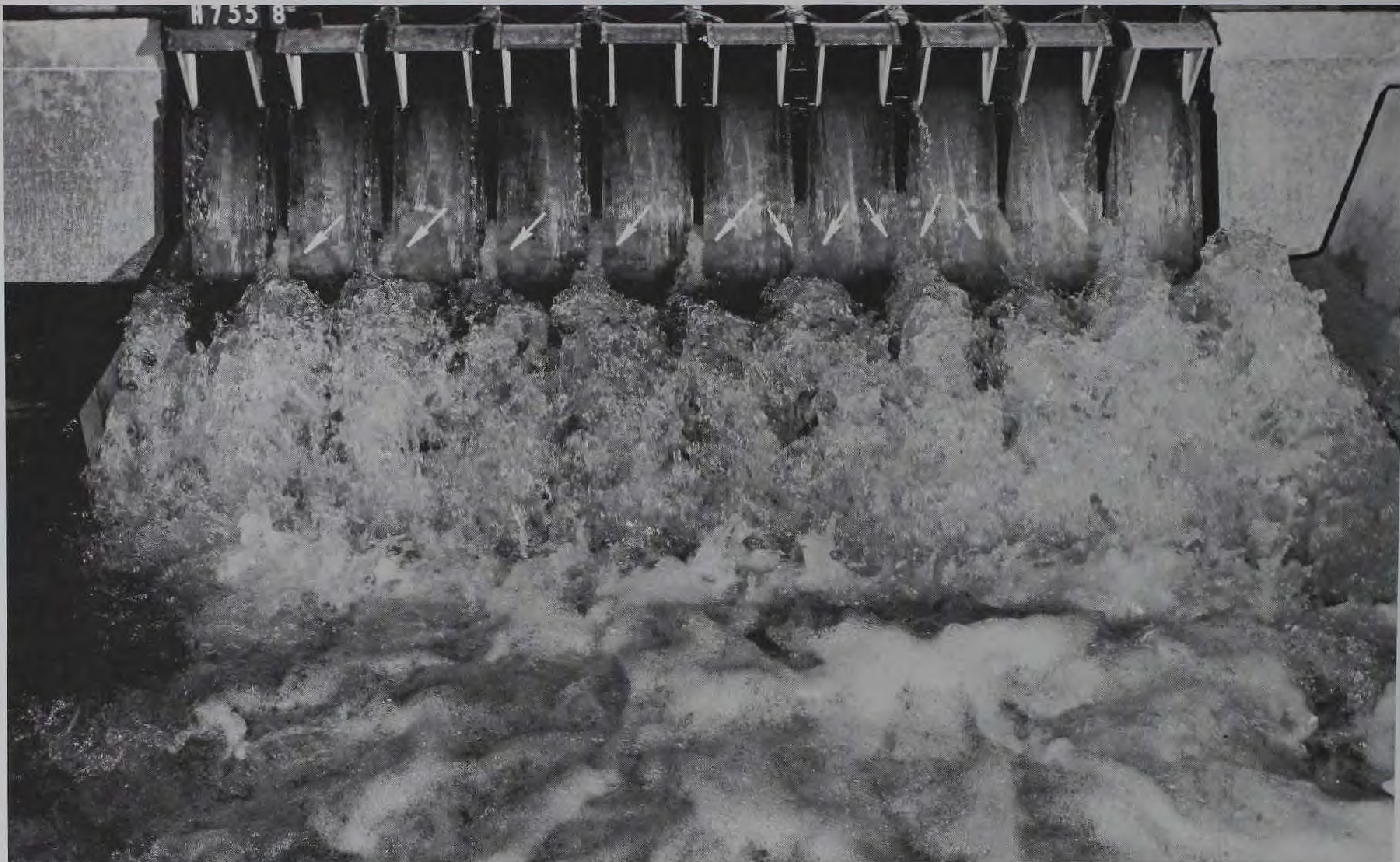
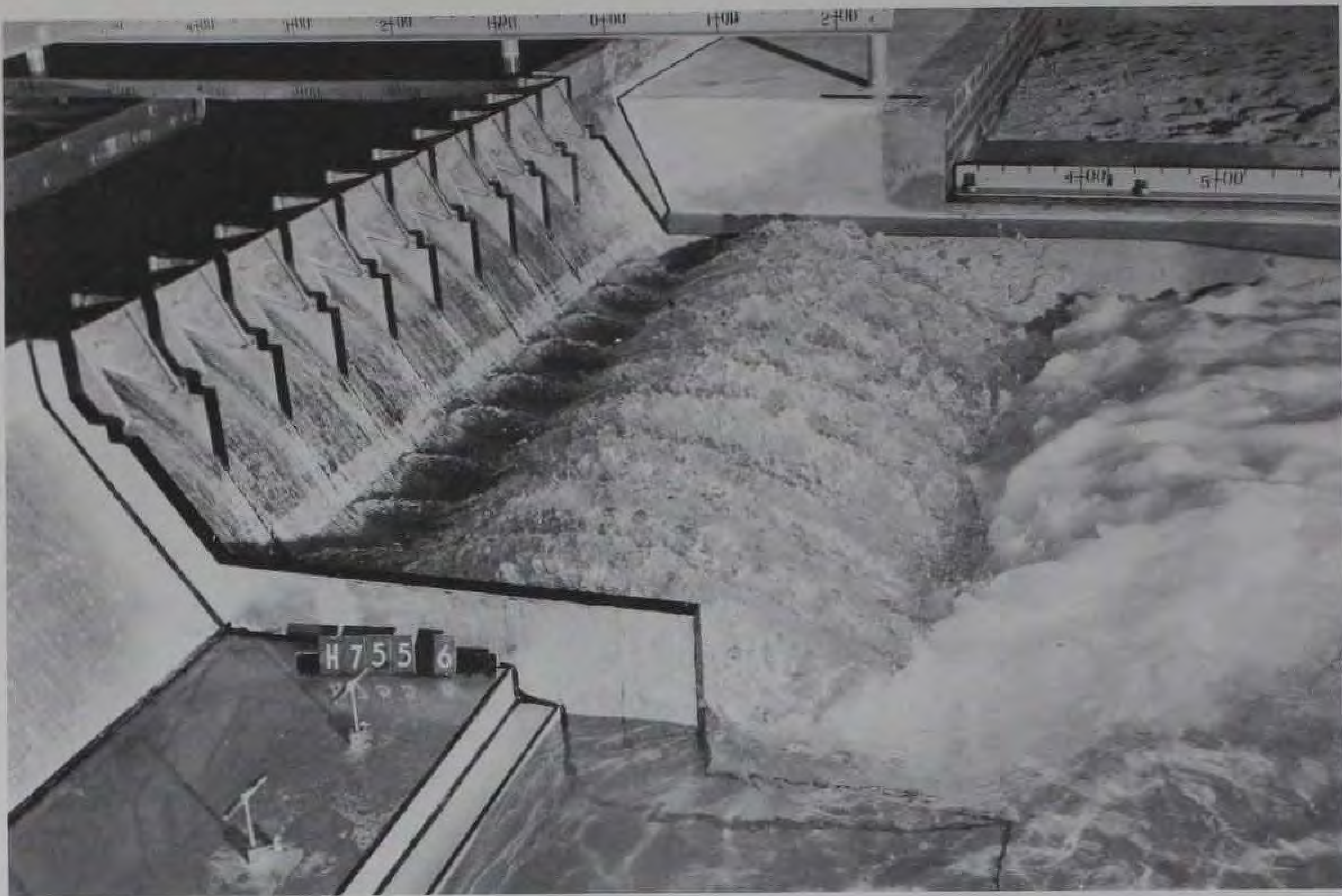


Photo 4. Flow conditions over crest with type 1 (original) piers; discharge 800,000 cfs. Arrows indicate zones of nappe separation



a. Discharge 360,000 cfs, pool el 485.0, tailwater el 343.0

b. Discharge 800,000 efs, pool el 490.0, tailwater el 356.0

Photo 5. Flow conditions in type 1 (original) flip bucket

a. Nappe clinging to lip of flip bucket

b. Nappe springing free from lip of flip bucket due to artificial venting

Photo 6. Conditions of nappe with type 1 (original) flip bucket; discharge 40,000 cfs

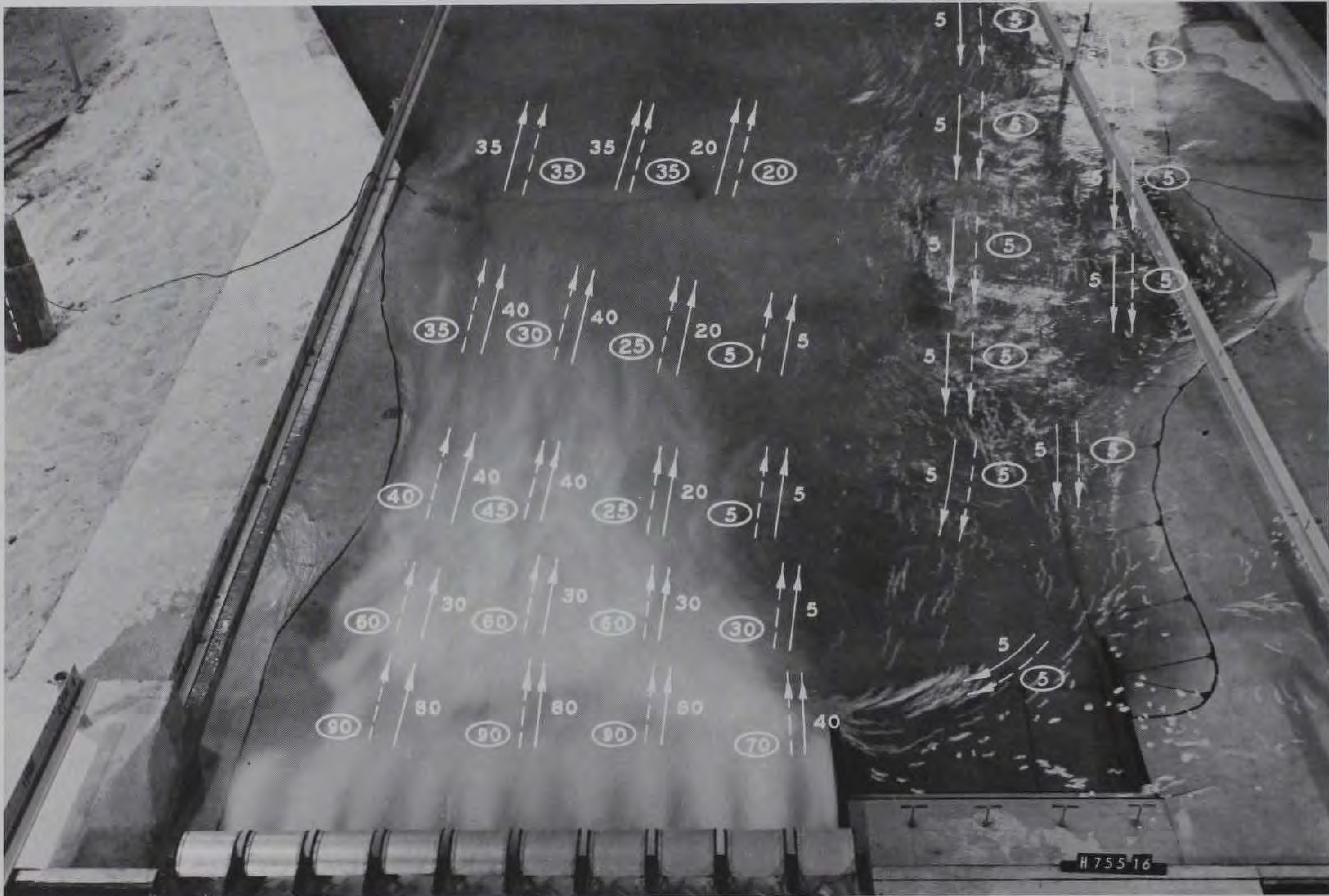
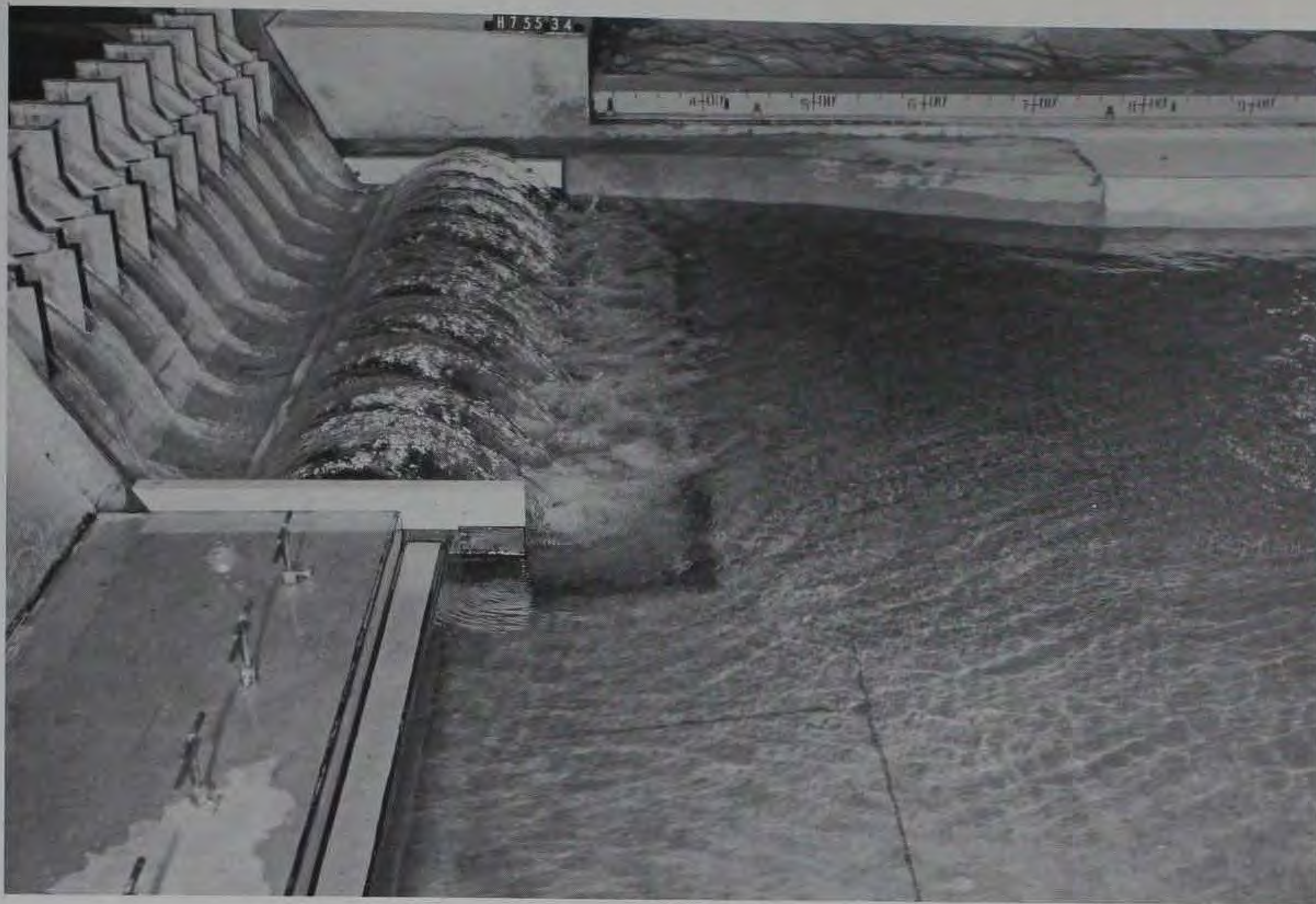
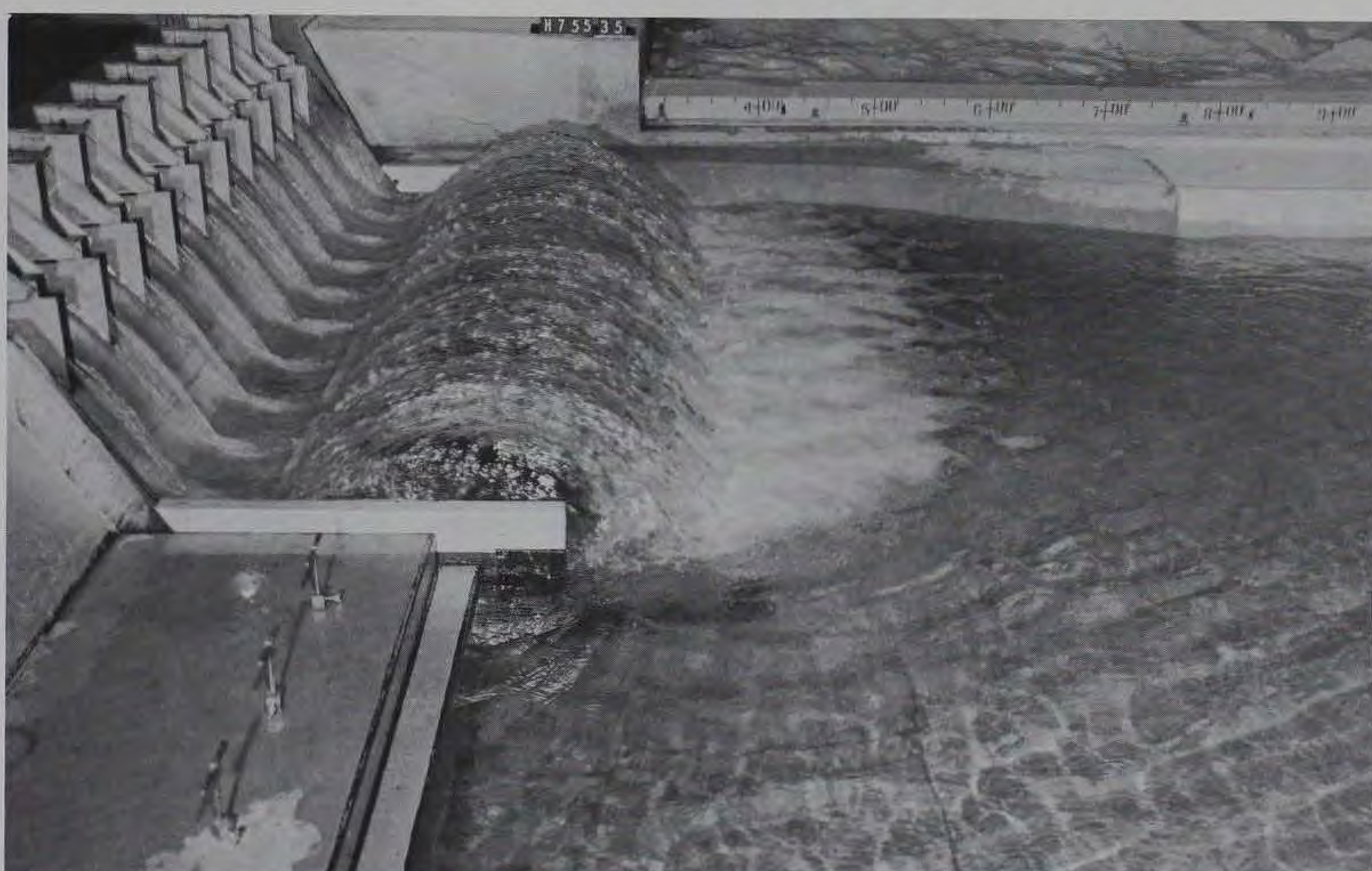


Photo 7. Flow conditions in exit channel with type 1 (original) flip bucket; discharge 800,000 cfs, pool el 490.0, tailwater el 356.0. Circled numbers (and dashed arrows) denote bottom velocities in prototype feet per second



a. Discharge 40,000 cfs, pool el 480.0, tailwater el 330.0



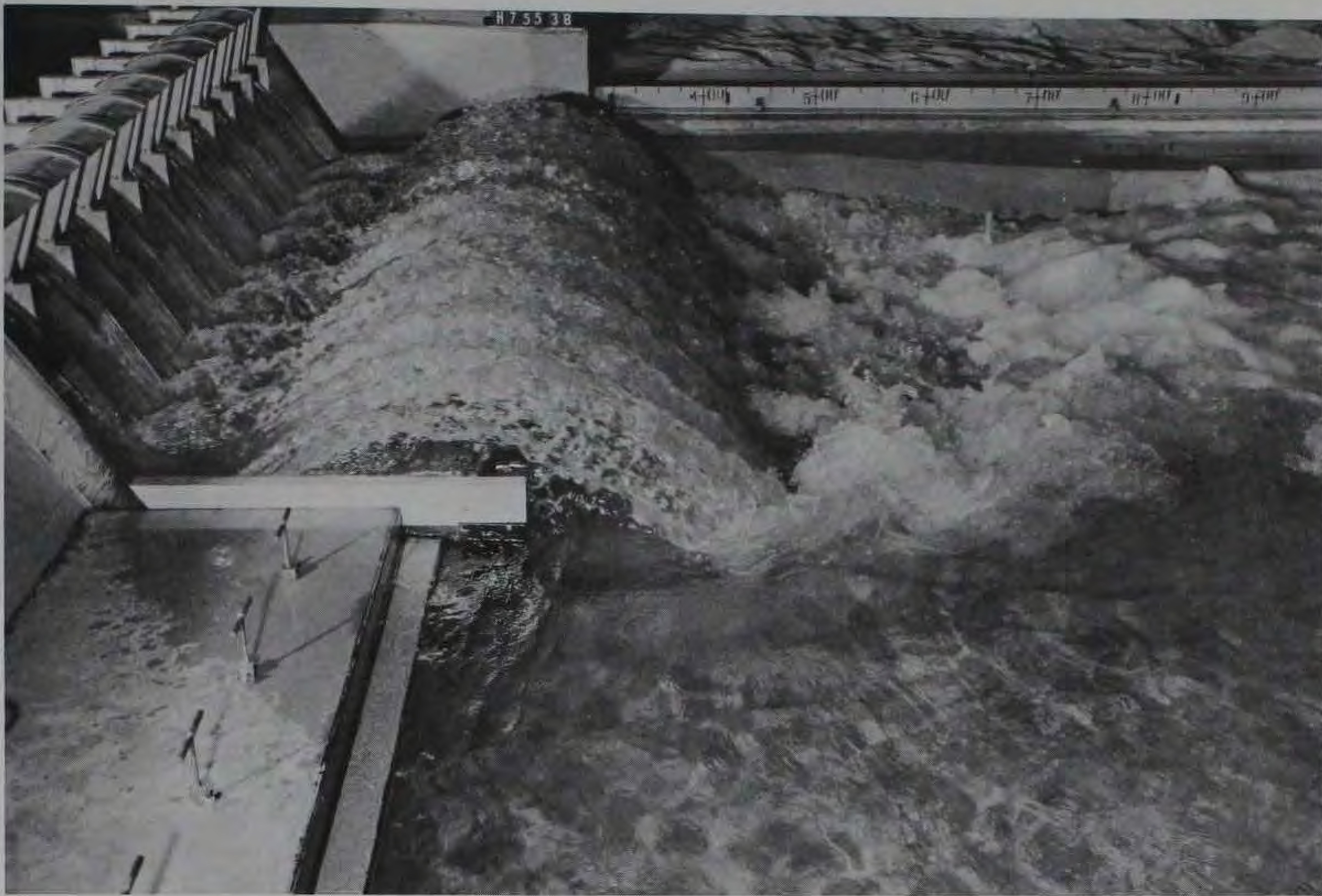
b. Discharge 100,000 cfs, pool el 482.0, tailwater el 335.0

Photo 8. Flow conditions with type 2 (recommended) flip bucket (sheet 1 of 3)

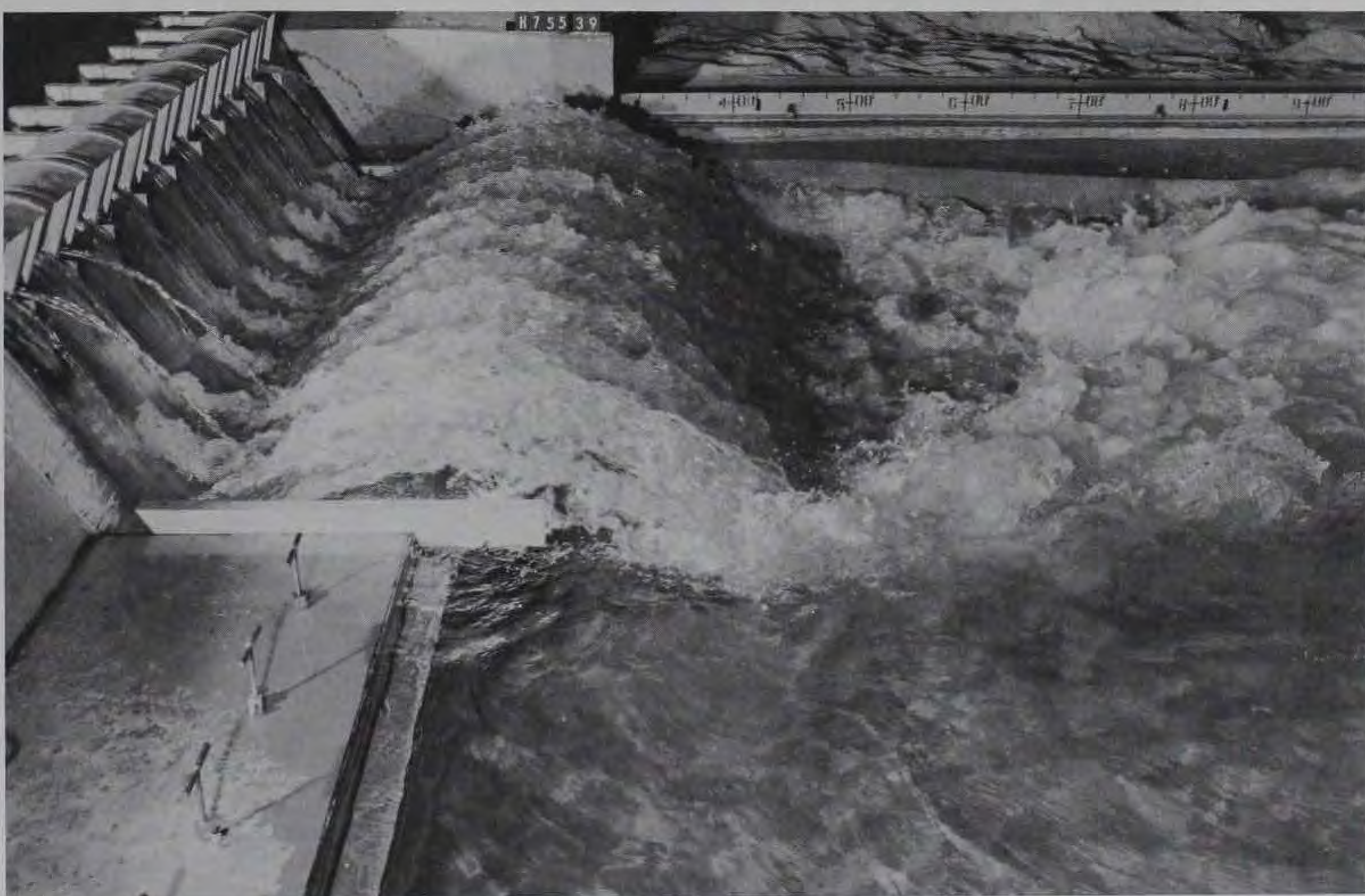
c. Discharge 200,000 cfs, pool el 484.0, tailwater el 337.0

d. Discharge 360,000 cfs, pool el 485.0, tailwater el 343.0

Photo 8 (sheet 2 of 3)



e. Discharge 528,000 cfs, pool el 478.0, tailwater el 346.0



f. Discharge 800,000 cfs, pool el 490.0, tailwater el 356.0

Photo 8 (sheet 3 of 3)

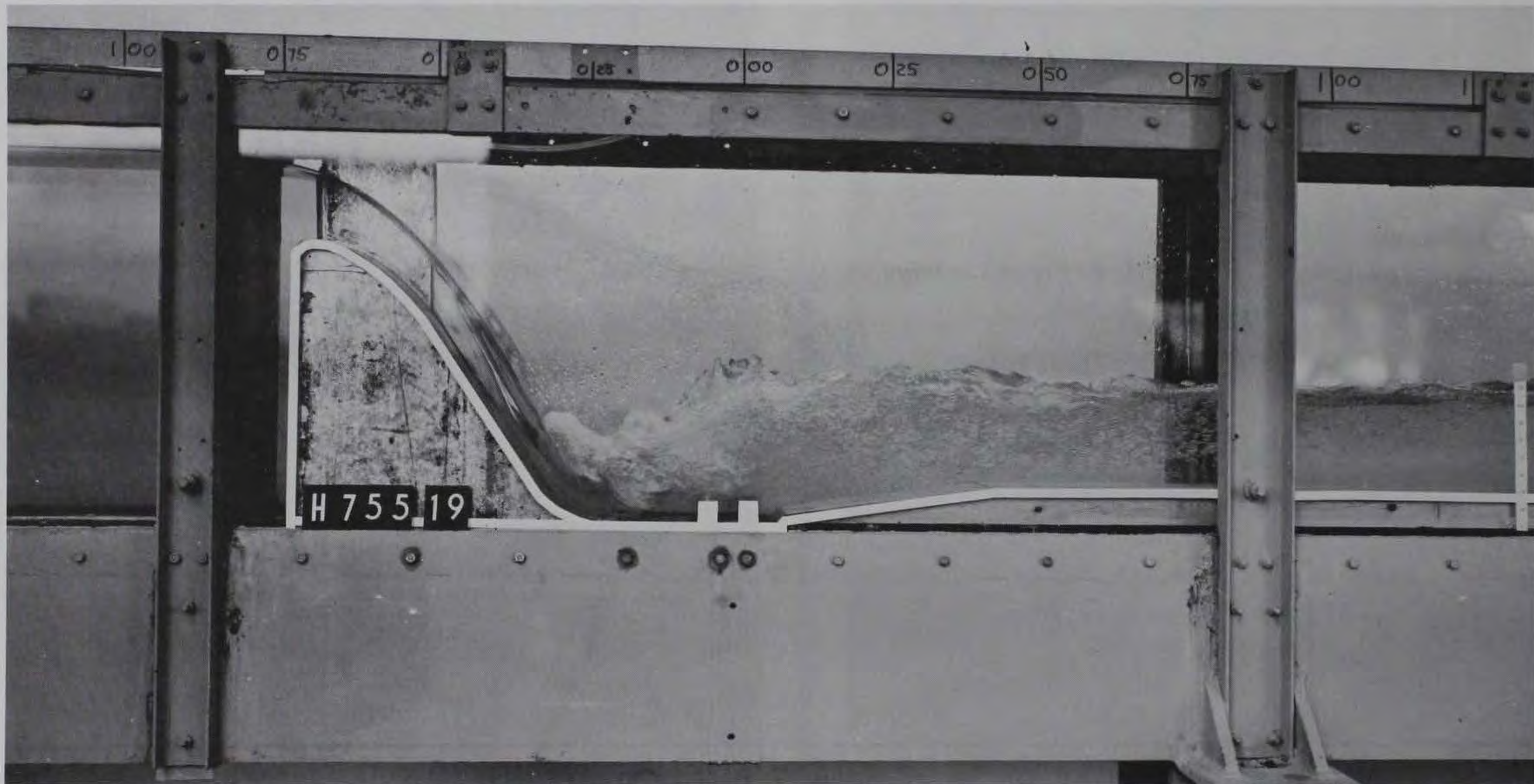
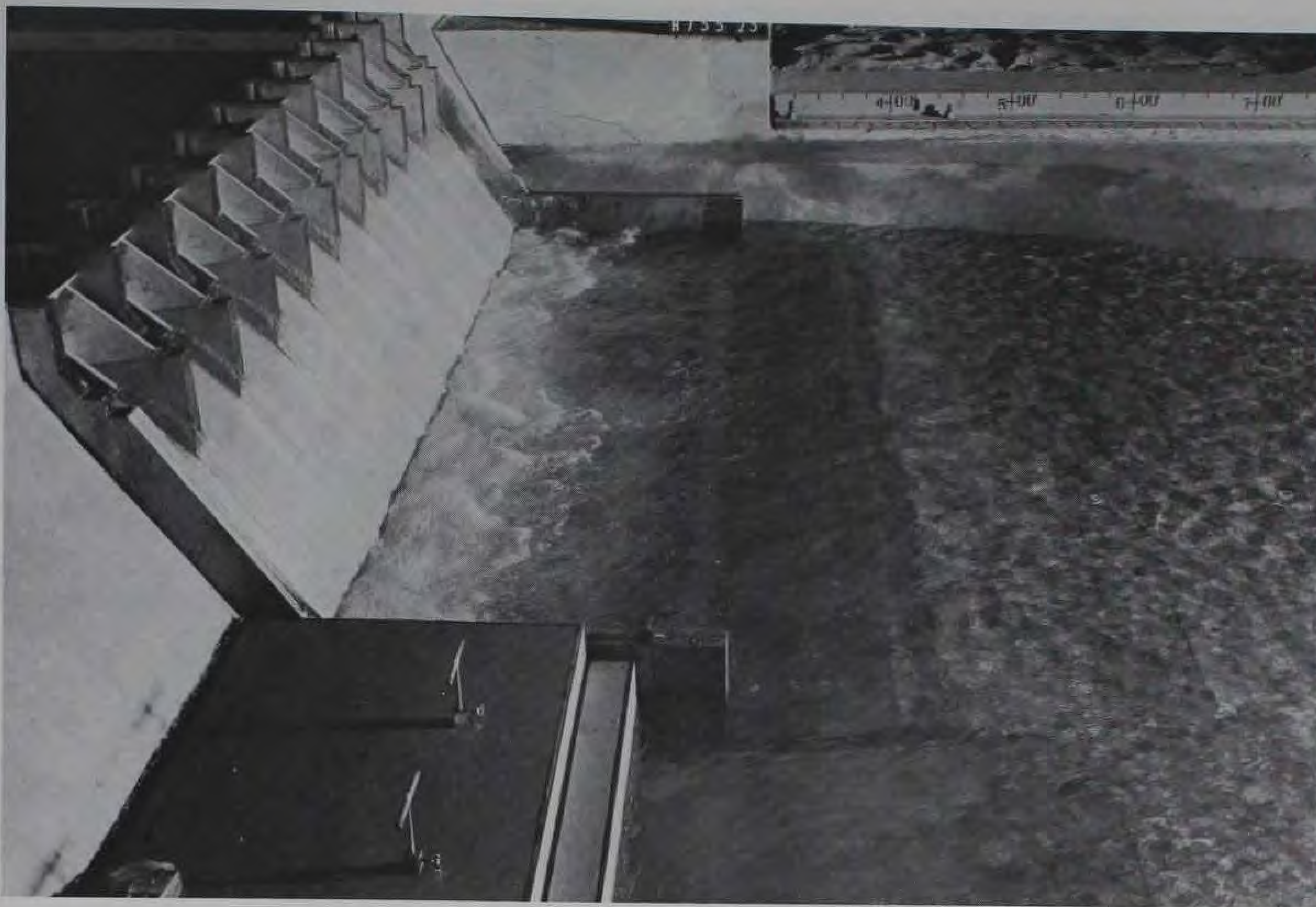
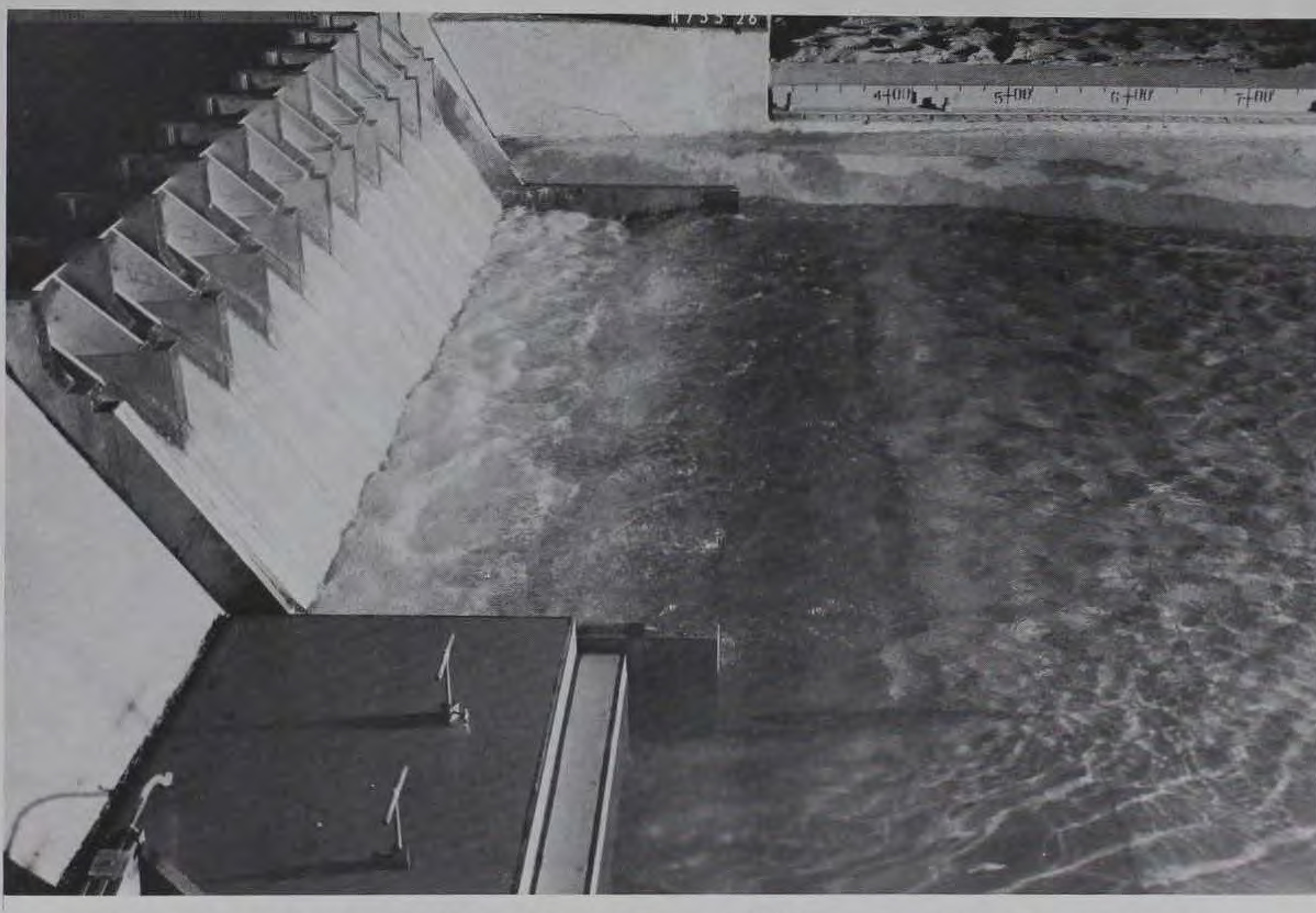


Photo 9. Flow conditions with type 2 stilling basin; discharge 800,000 cfs, pool el 490.0, tailwater el 356.0

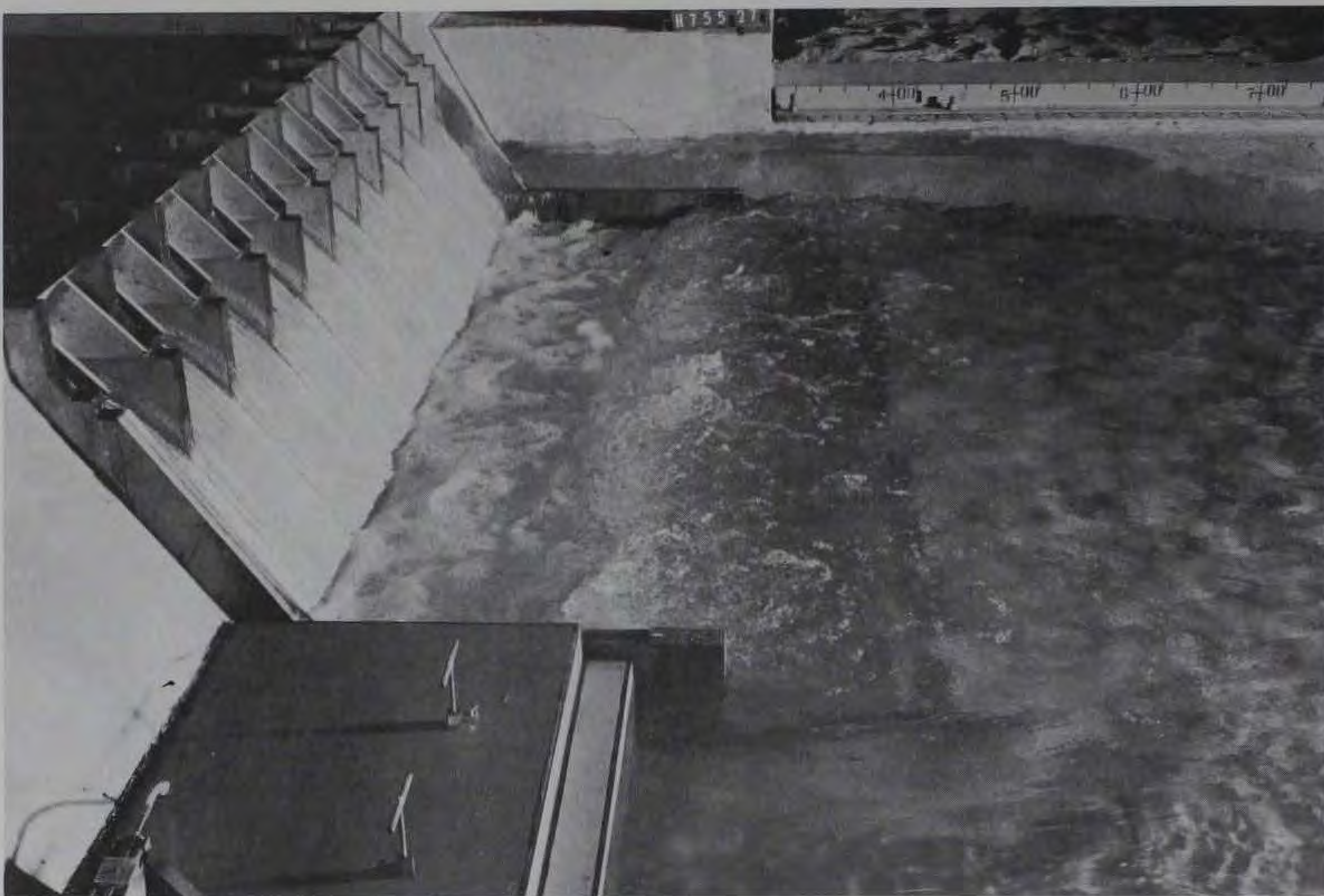


a. Discharge 40,000 cfs, pool el 480.0, tailwater el 330.0



b. Discharge 100,000 cfs, pool el 482.0, tailwater el 335.0

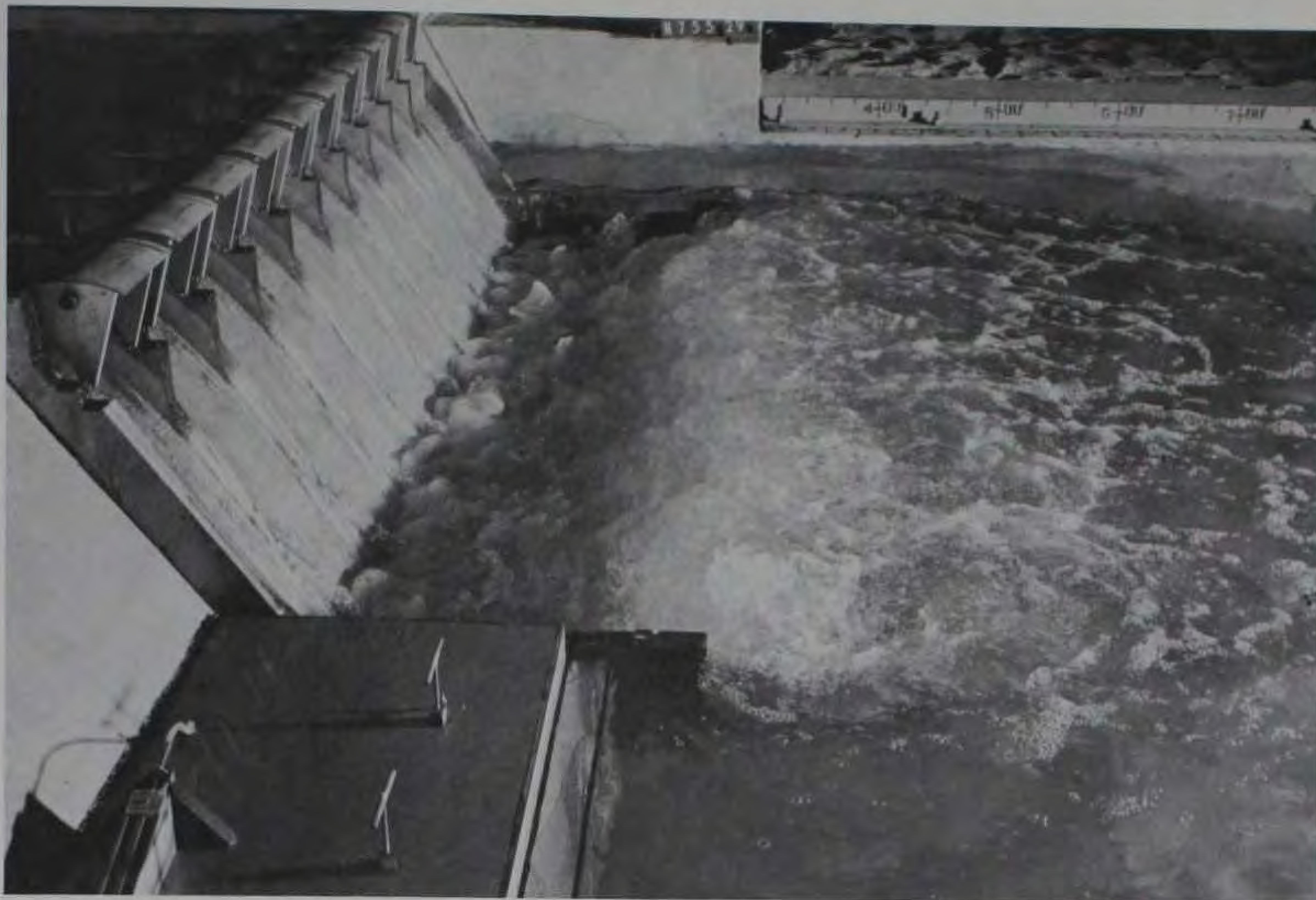
Photo 10. Flow conditions with type 3 (recommended) stilling basin (sheet 1 of 3)



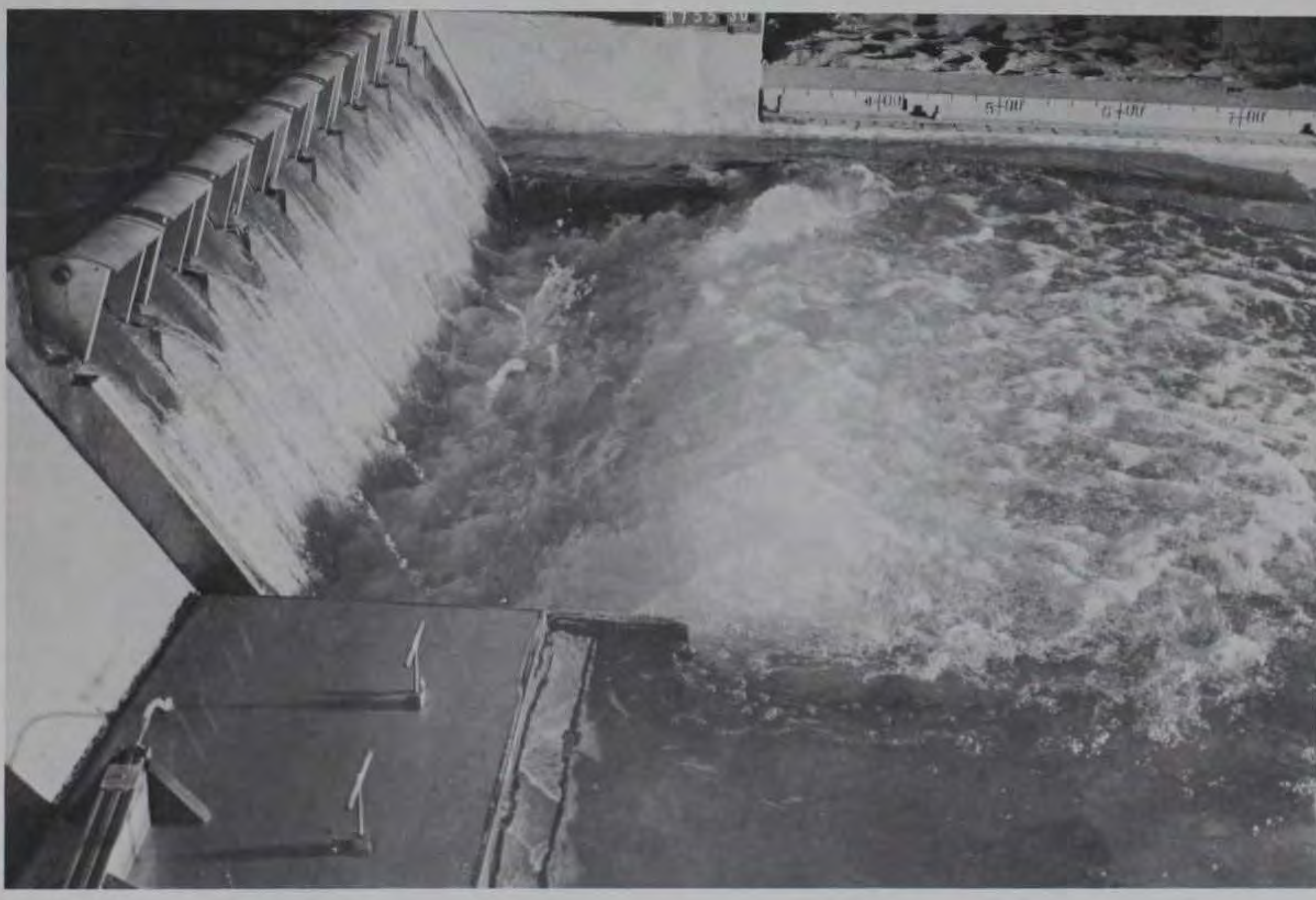
c. Discharge 200,000 cfs, pool el 484.0, tailwater el 337.0

d. Discharge 360,000 cfs, pool el 485.0, tailwater 343.0

Photo 10 (sheet 2 of 3)



e. Discharge 528,000 cfs, pool el 478.0, tailwater el 346.0



f. Discharge 800,000 cfs, pool el 490.0, tailwater el 356.0

Photo 10 (sheet 3 of 3)

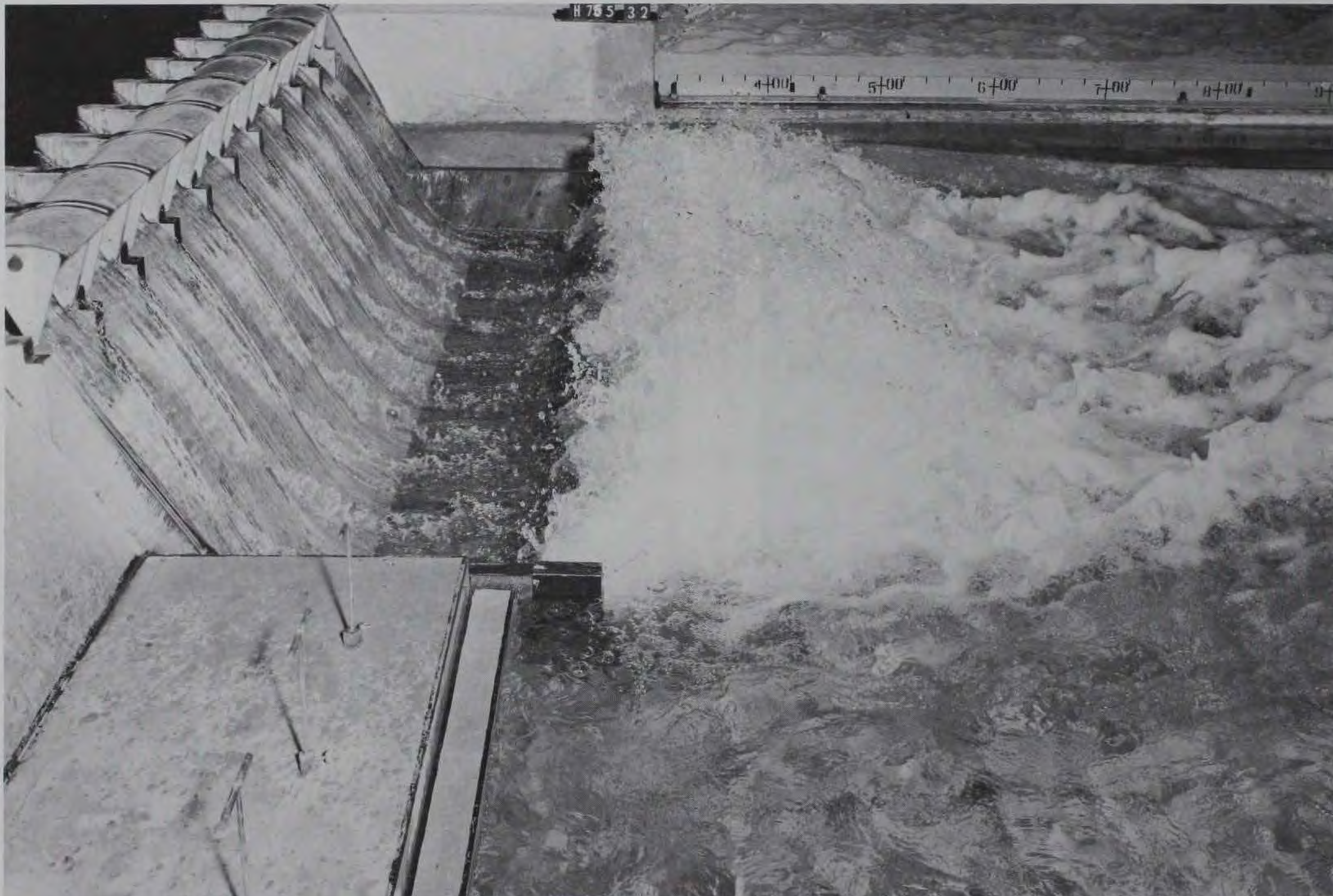
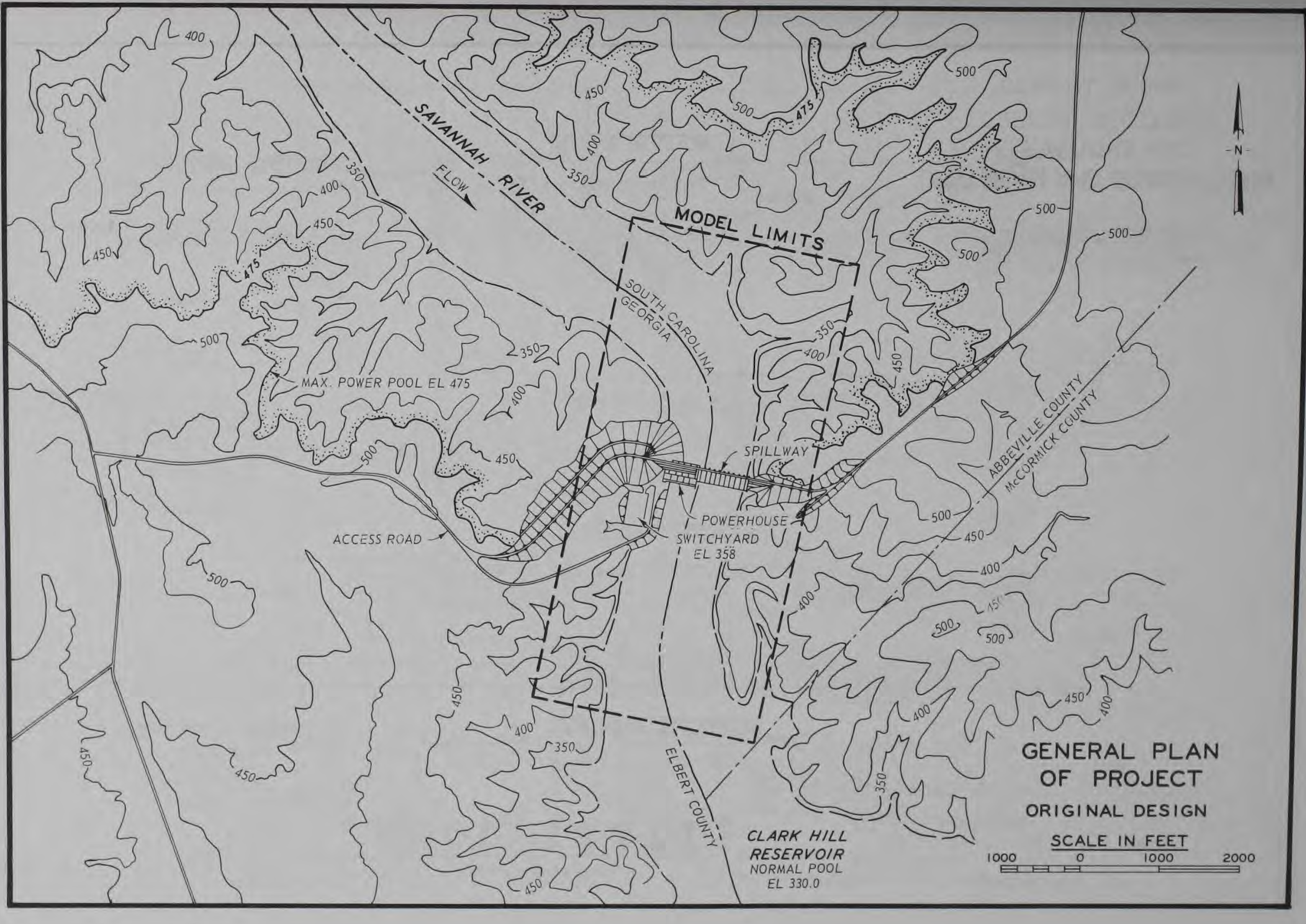


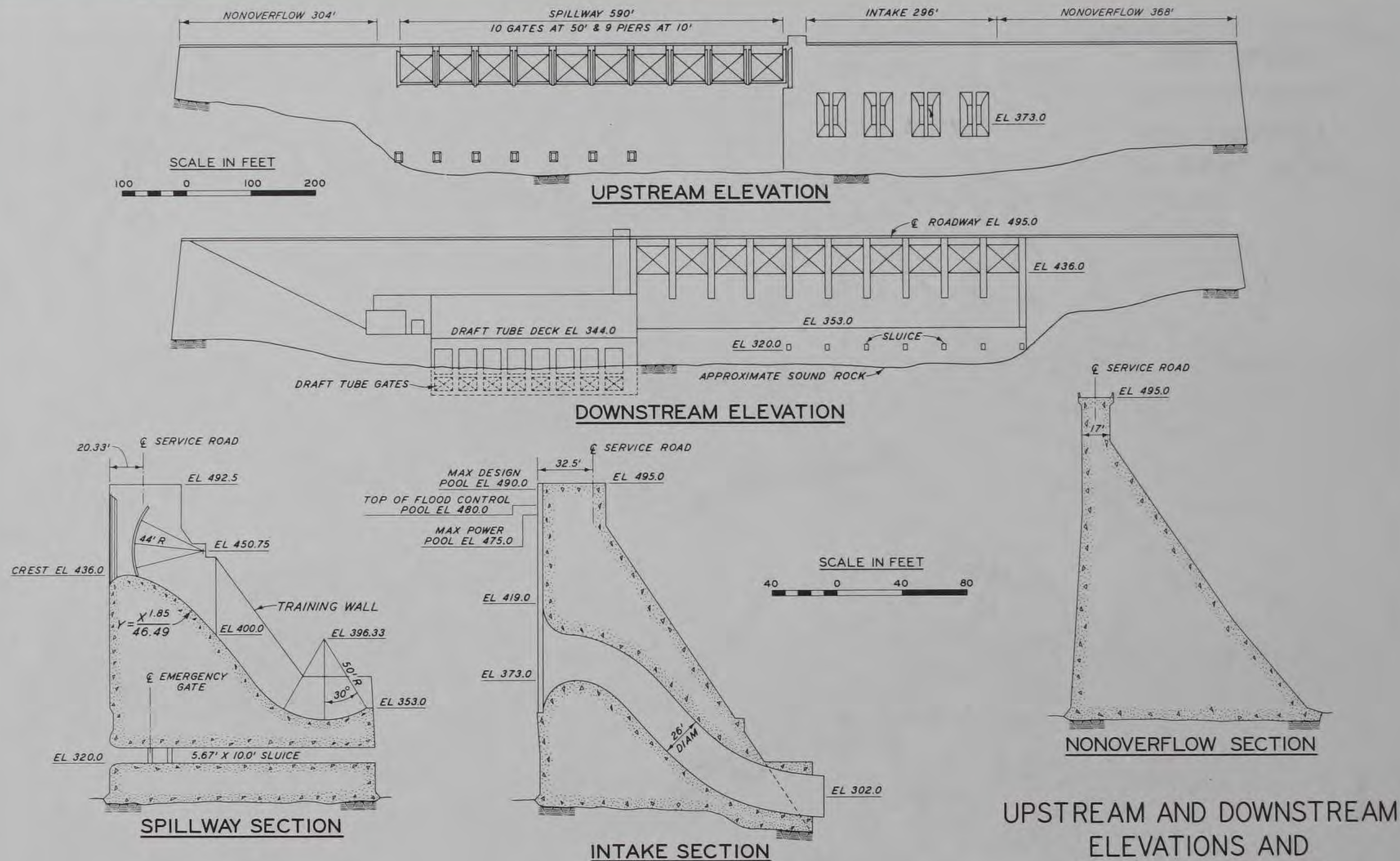
Photo 11. Spray action in type 3 (recommended) stilling basin; discharge 800,000 cfs, pool el 490.0, tailwater el 347.0

Photo 12. Flow conditions resulting from powerhouse release; discharge 28,000 cfs, tailwater el 345.0. Numbers denote bottom velocities in prototype feet per second



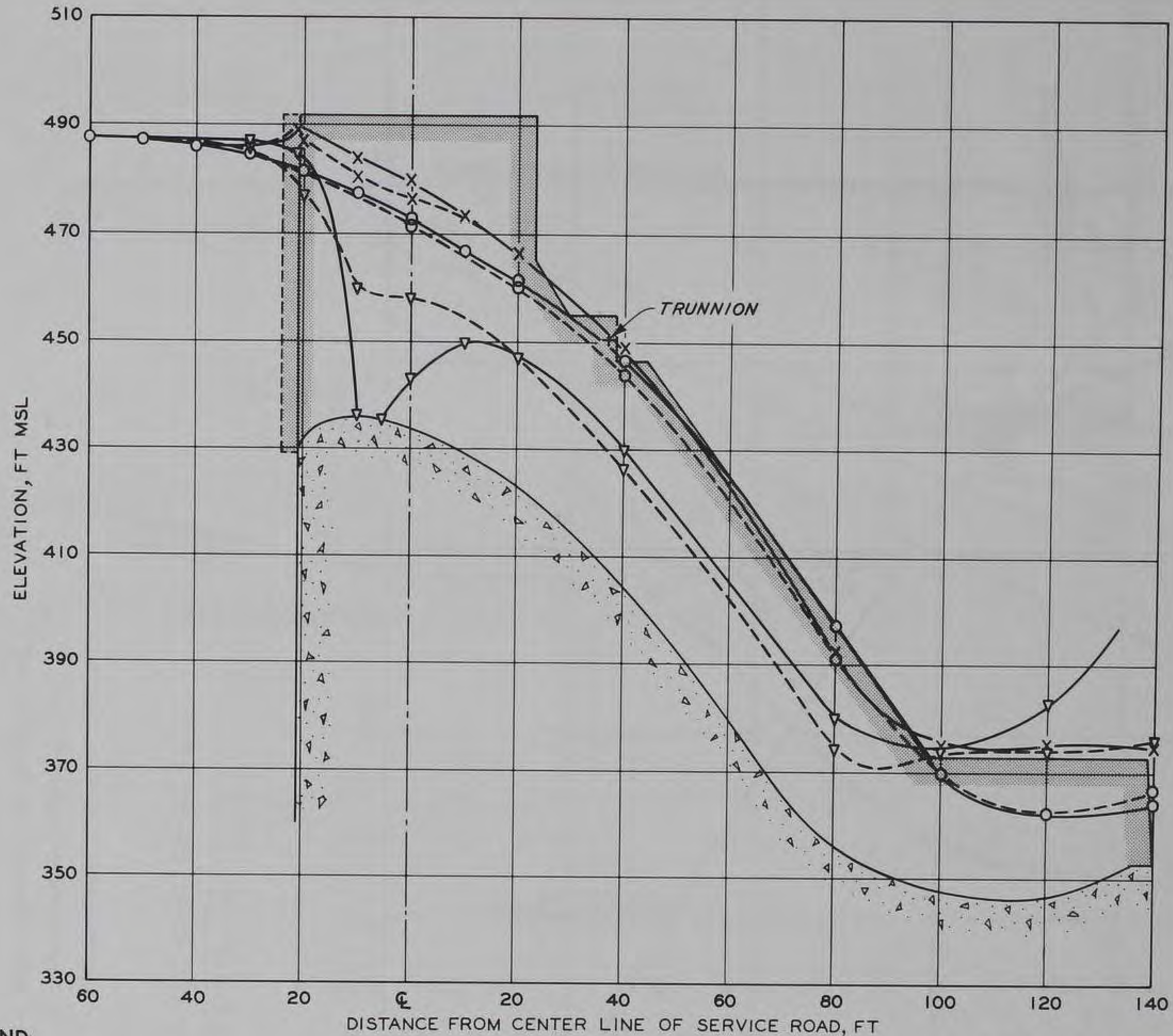
SAVANNAH RIVER
FLOW
MODEL LIMITS
SOUTH CAROLINA
GEORGIA
MAX. POWER POOL EL 475
SPILLWAY
POWERHOUSE
SWITCHYARD
EL 358
ACCESS ROAD
ABBEVILLE COUNTY
McCORMICK COUNTY
ELBERT COUNTY
CLARK HILL
RESERVOIR
NORMAL POOL
EL 330.0
N
GENERAL PLAN
OF PROJECT
ORIGINAL DESIGN
SCALE IN FEET
1000 0 1000 2000

PLATE 1



UPSTREAM AND DOWNSTREAM ELEVATIONS AND TYPICAL SECTIONS

ORIGINAL DESIGN



LEGEND

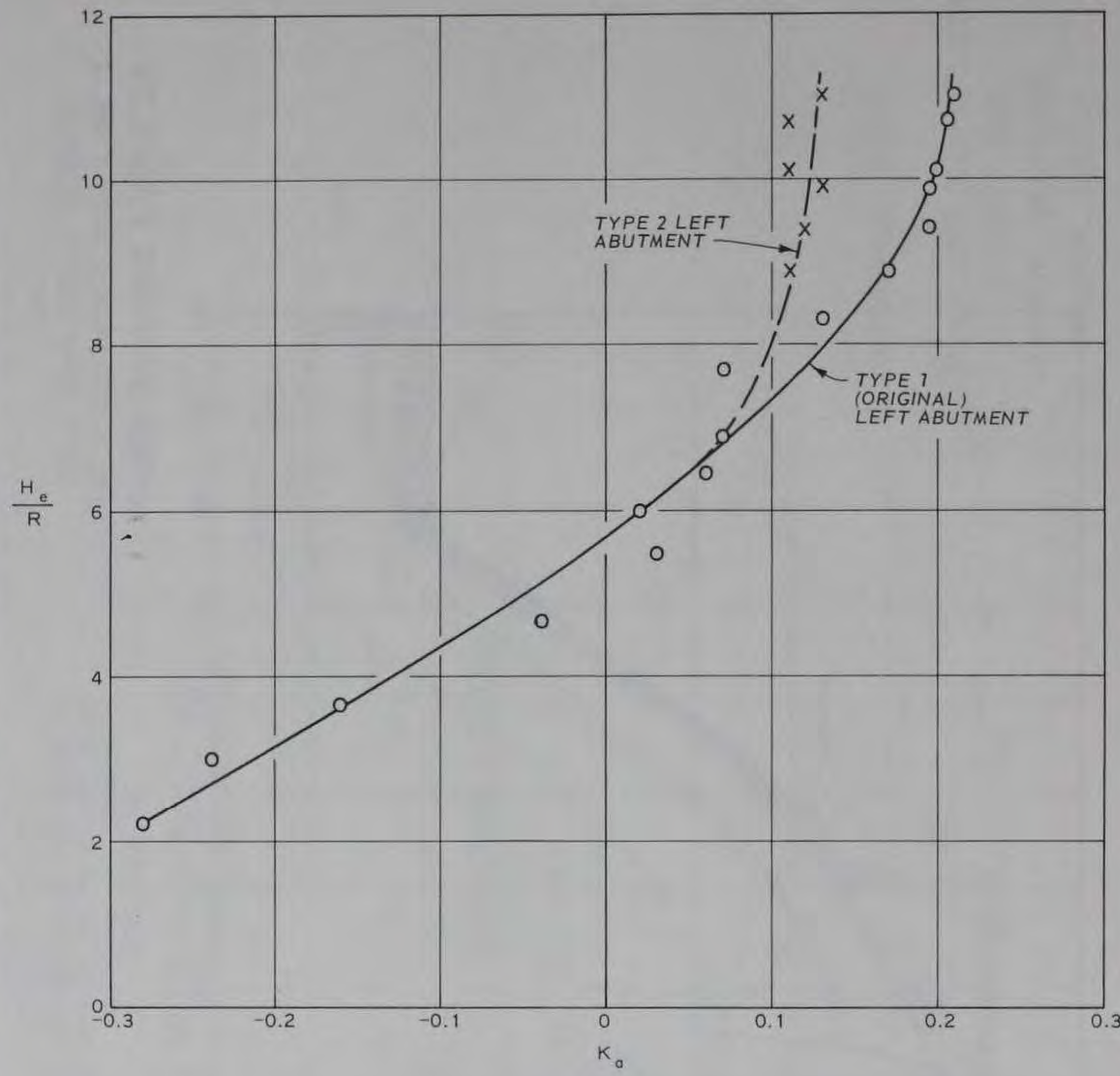
- - - - - - TYPE 2 LEFT ABUTMENT
———— TYPE 1 (ORIGINAL) LEFT ABUTMENT
O CENTER LINE
∇ LEFT WALL
X RIGHT WALL

TEST CONDITION

DISCHARGE . . . . 800,000 CFS

WATER-SURFACE PROFILES

TYPE 1 (ORIGINAL) AND TYPE 2
LEFT ABUTMENT DESIGNS
BAY 1



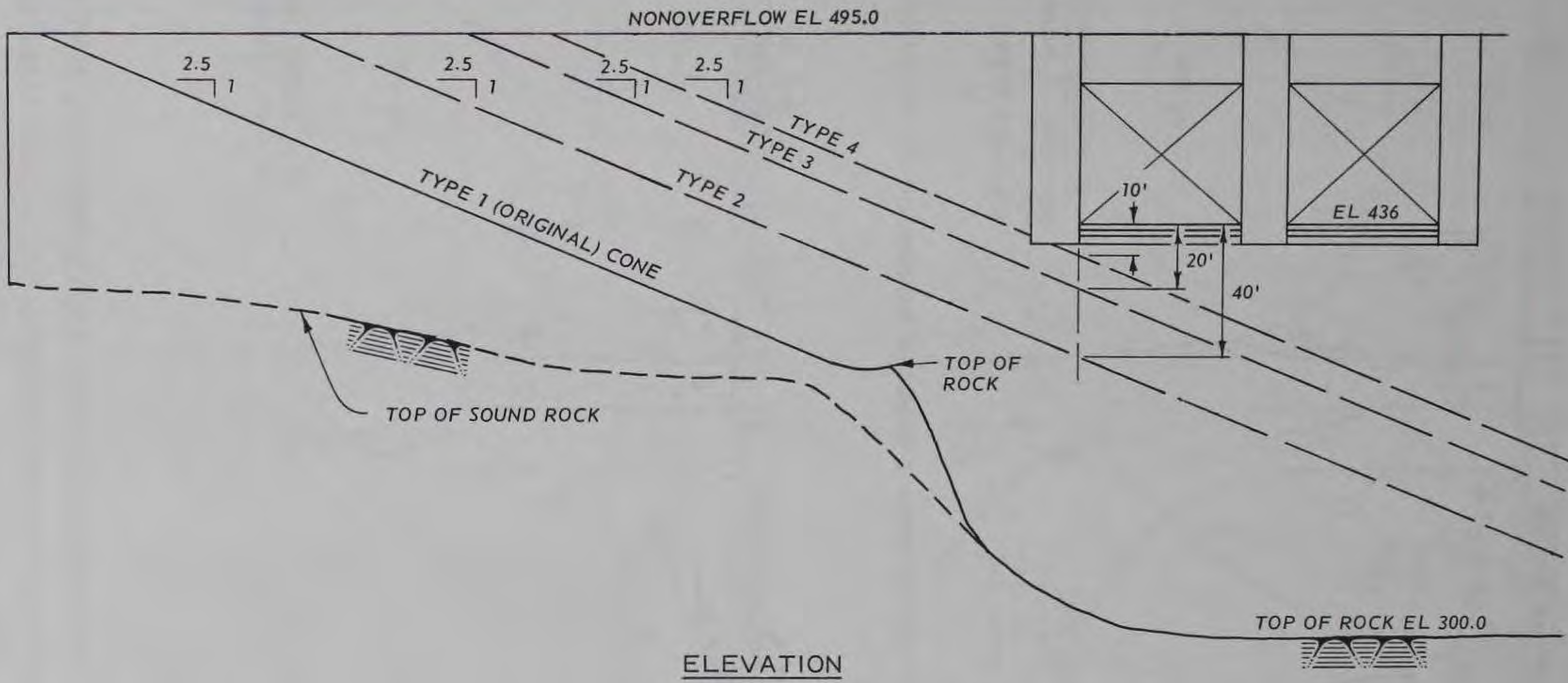
BASIC EQUATION

$$Q = C\left[L' - 2(NK_p + K_a)H_e\right]H_e^{3/2}$$

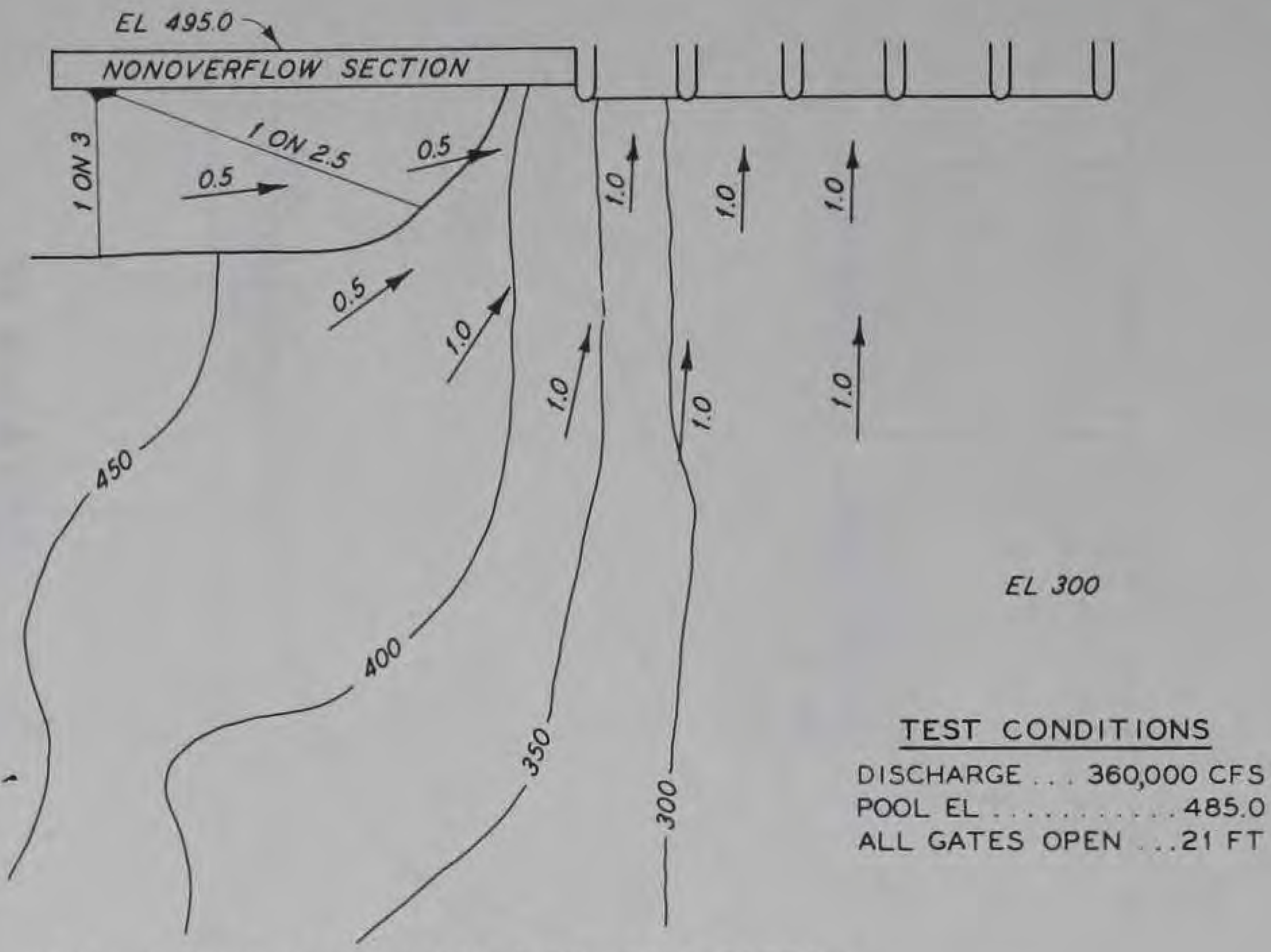
WHERE

- Q = DISCHARGE, CFS
- C = DISCHARGE COEFFICIENT
- L' = NET LENGTH OF CREST, FT
- N = NUMBER OF PIERS
- K_p = PIER CONTRACTION COEFFICIENT
- K_a = ABUTMENT CONTRACTION COEFFICIENT
- H_e = TOTAL HEAD ON CREST, FT

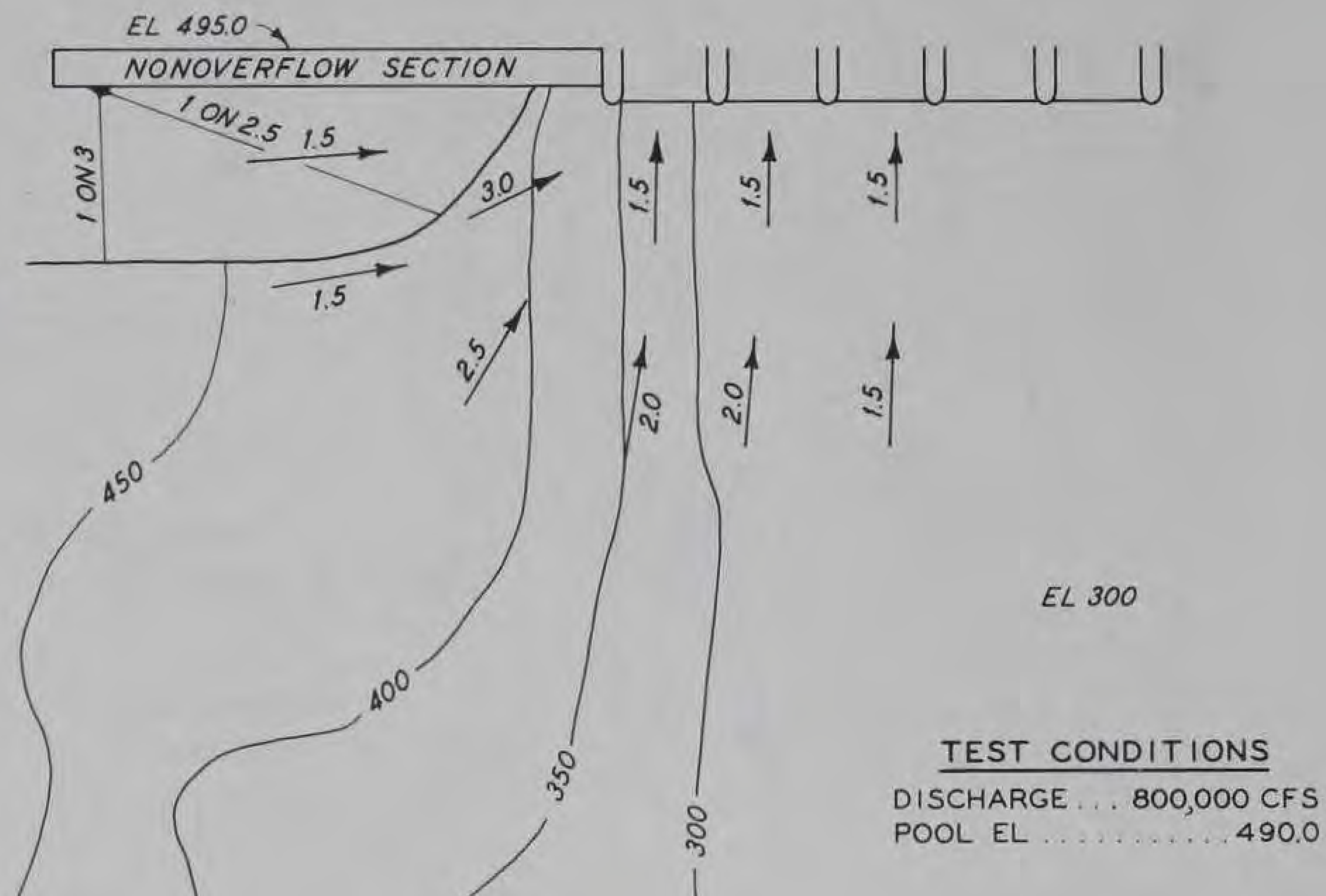
ABUTMENT CONTRACTION COEFFICIENTS FOR UNCONTROLLED FLOW



DETAILS OF
VARIOUS TERMINAL CONES
FOR LEFT ABUTMENT

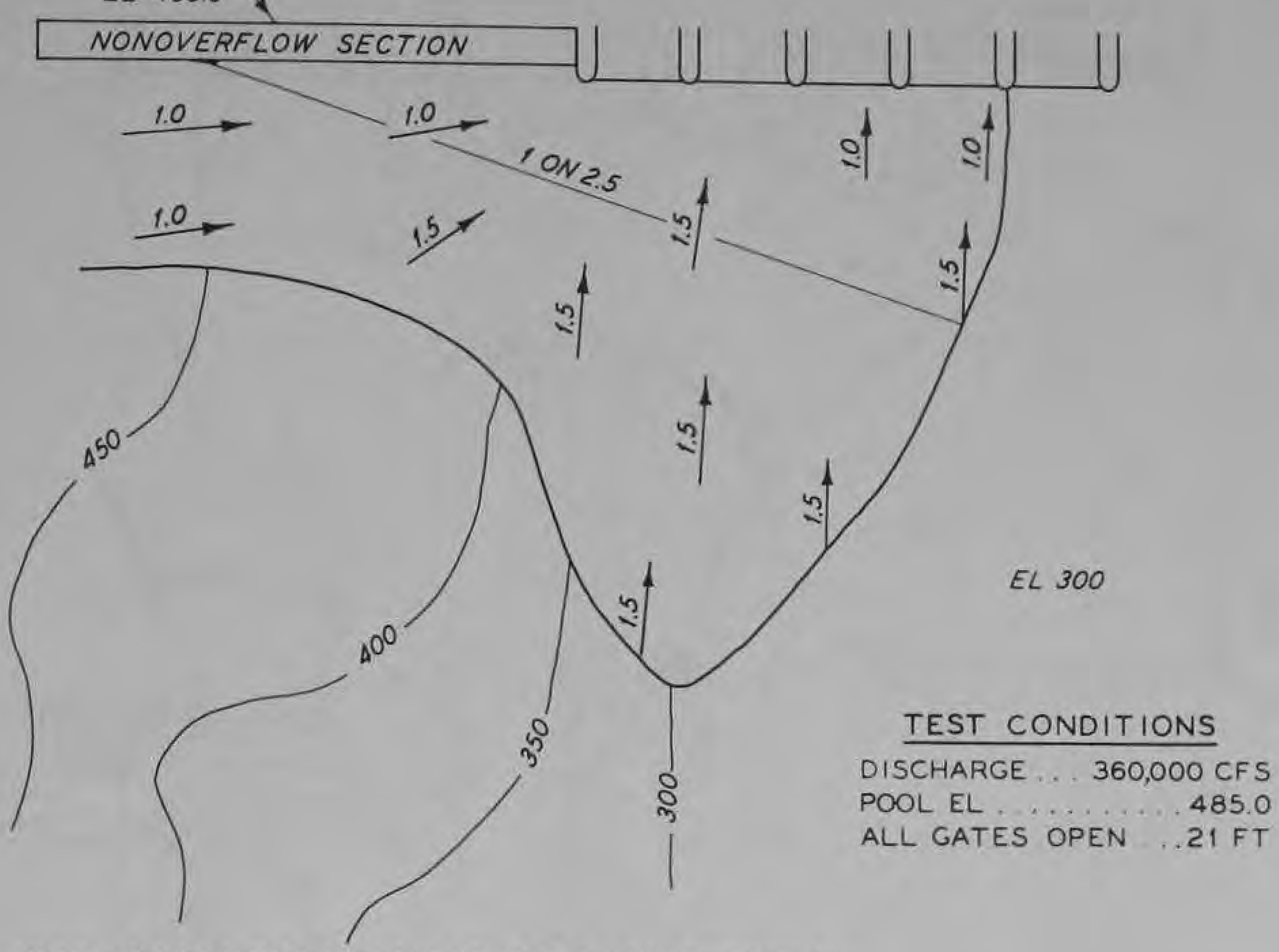


NOTE: VELOCITIES MEASURED 5 FT ABOVE BOTTOM.

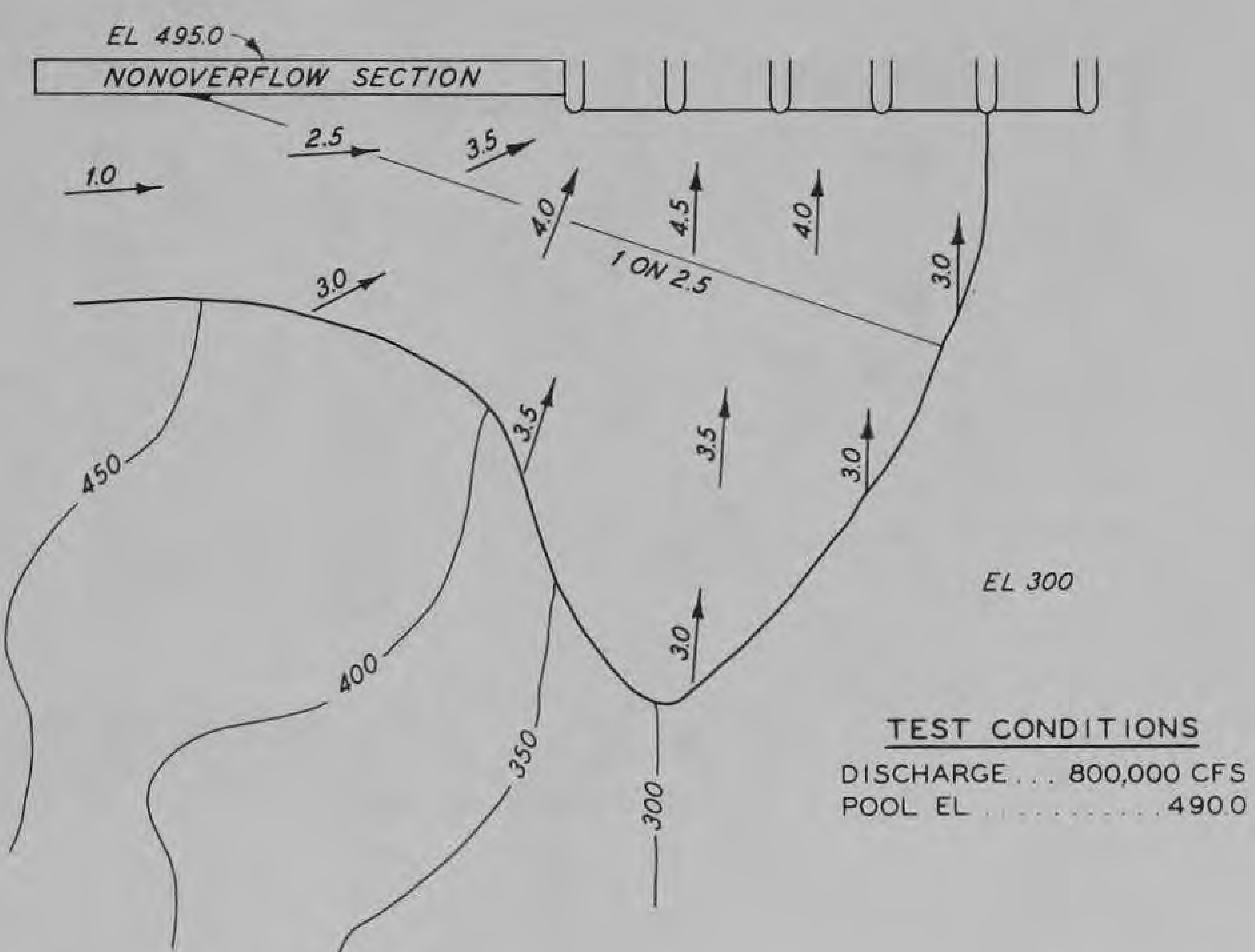


VELOCITIES AND CURRENT PATTERNS

TYPE 1 (ORIGINAL) LEFT ABUTMENT TERMINAL CONE



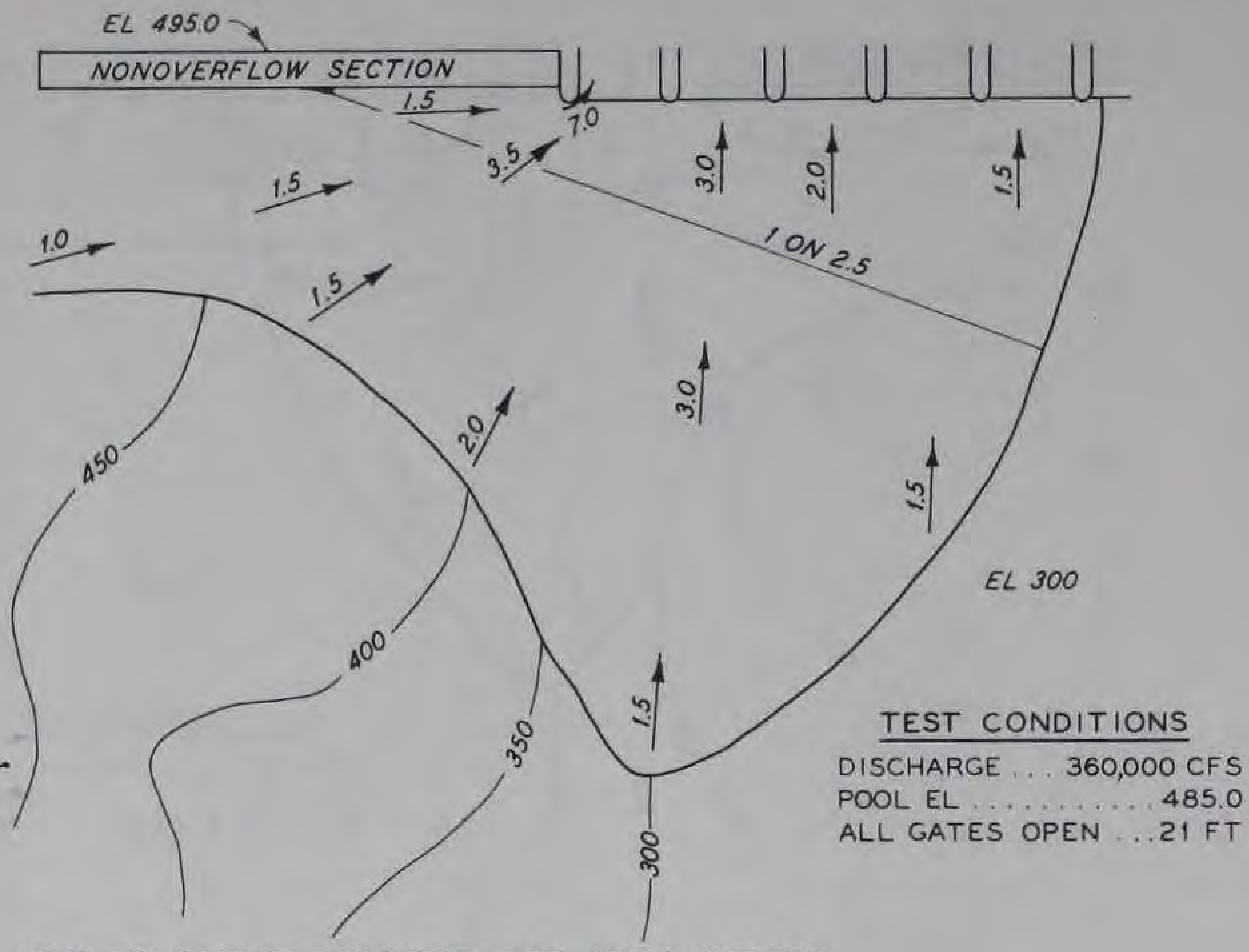
NOTE: VELOCITIES MEASURED 5 FT ABOVE BOTTOM.



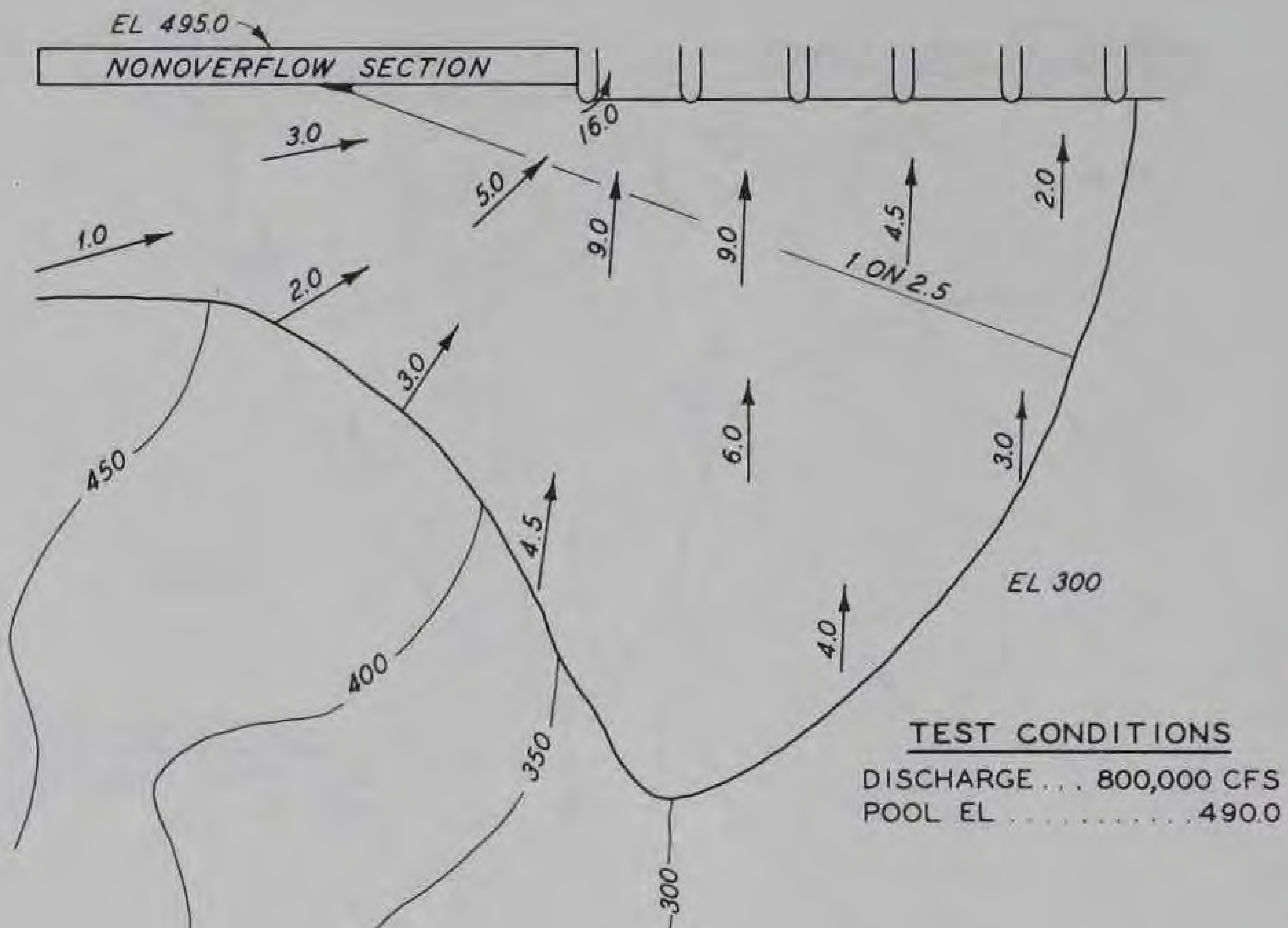
VELOCITIES AND CURRENT PATTERNS

TYPE 2 LEFT ABUTMENT

TERMINAL CONE

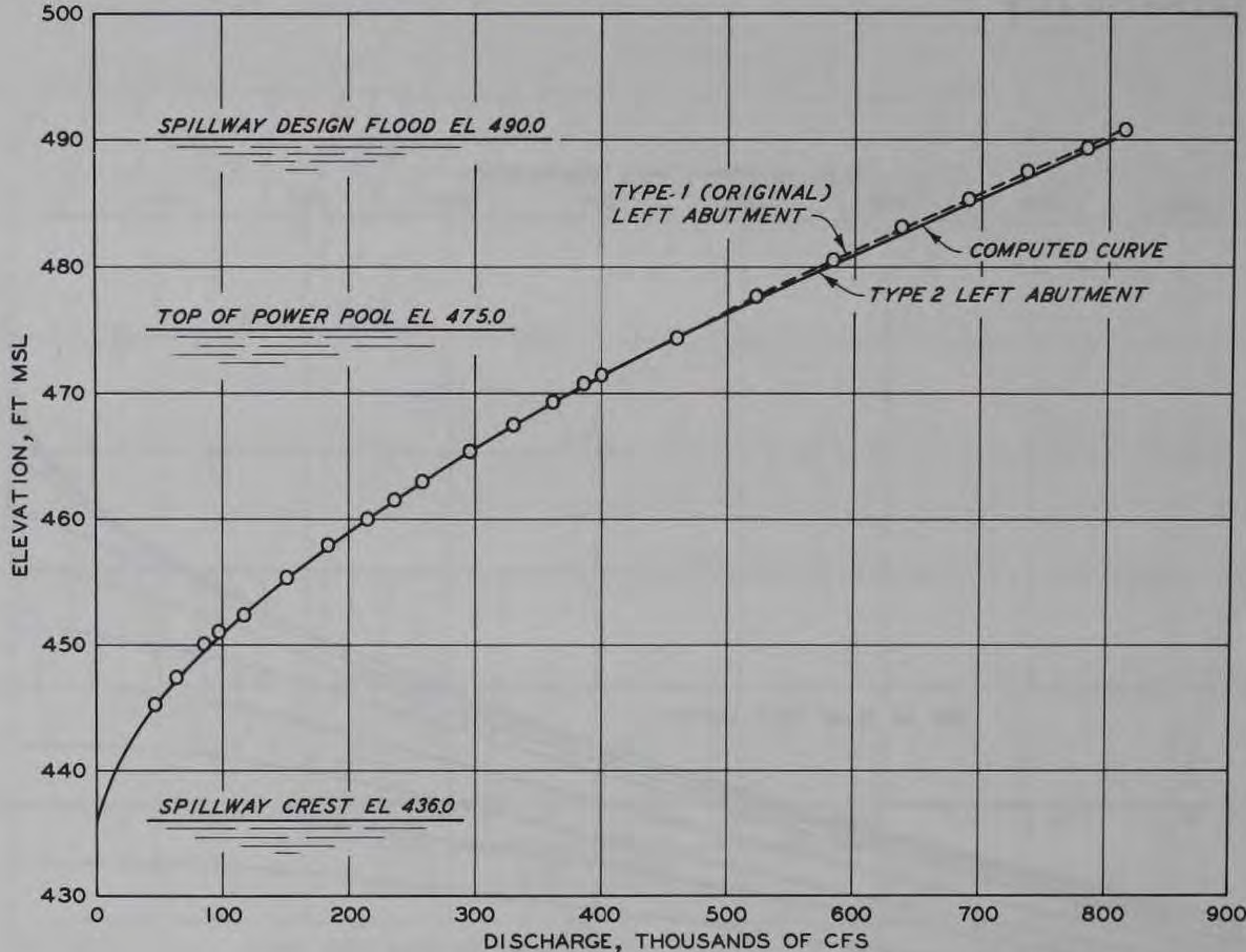


NOTE: VELOCITIES MEASURED 5 FT ABOVE BOTTOM.



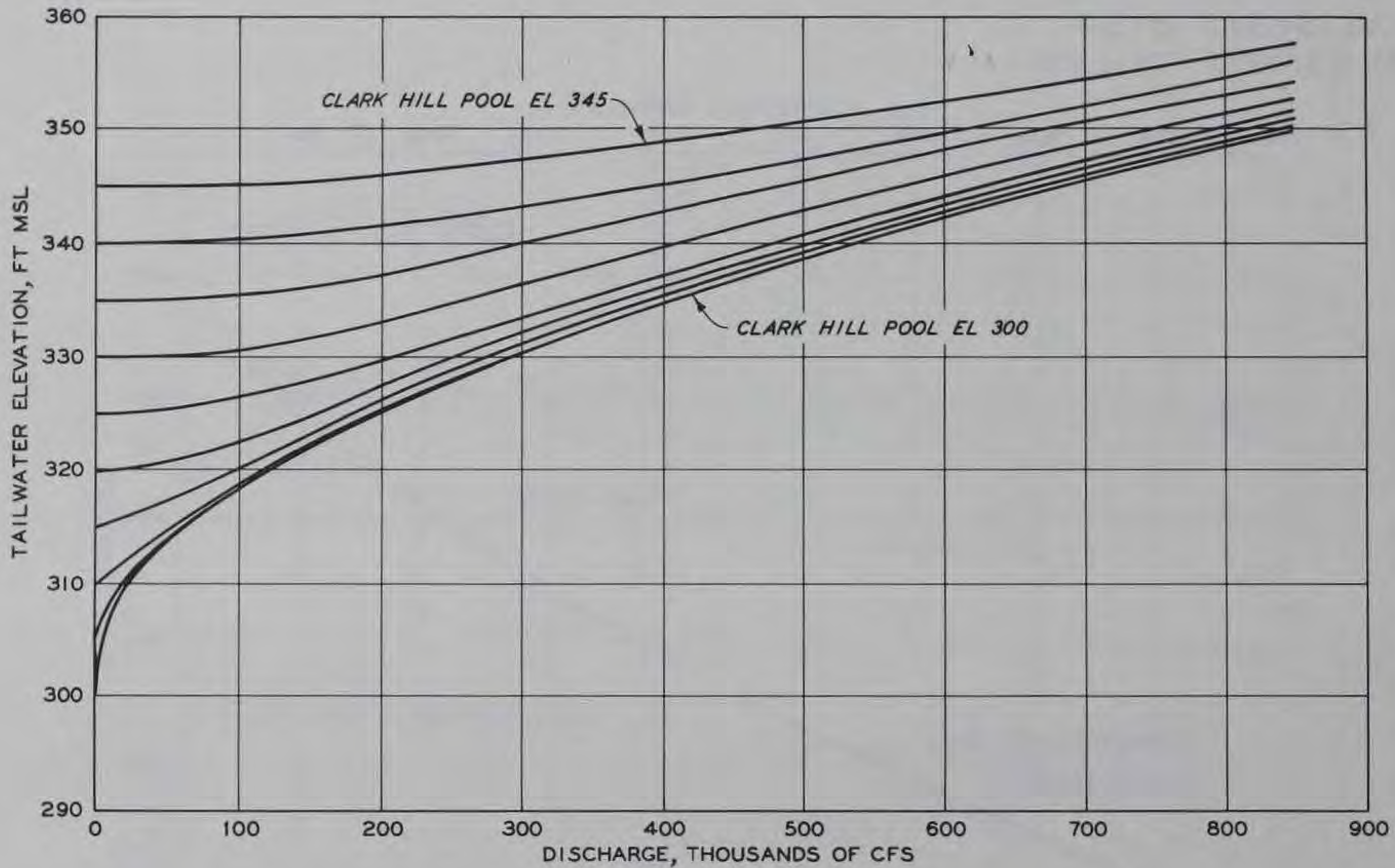
VELOCITIES AND CURRENT PATTERNS

TYPE 4 LEFT ABUTMENT TERMINAL CONE

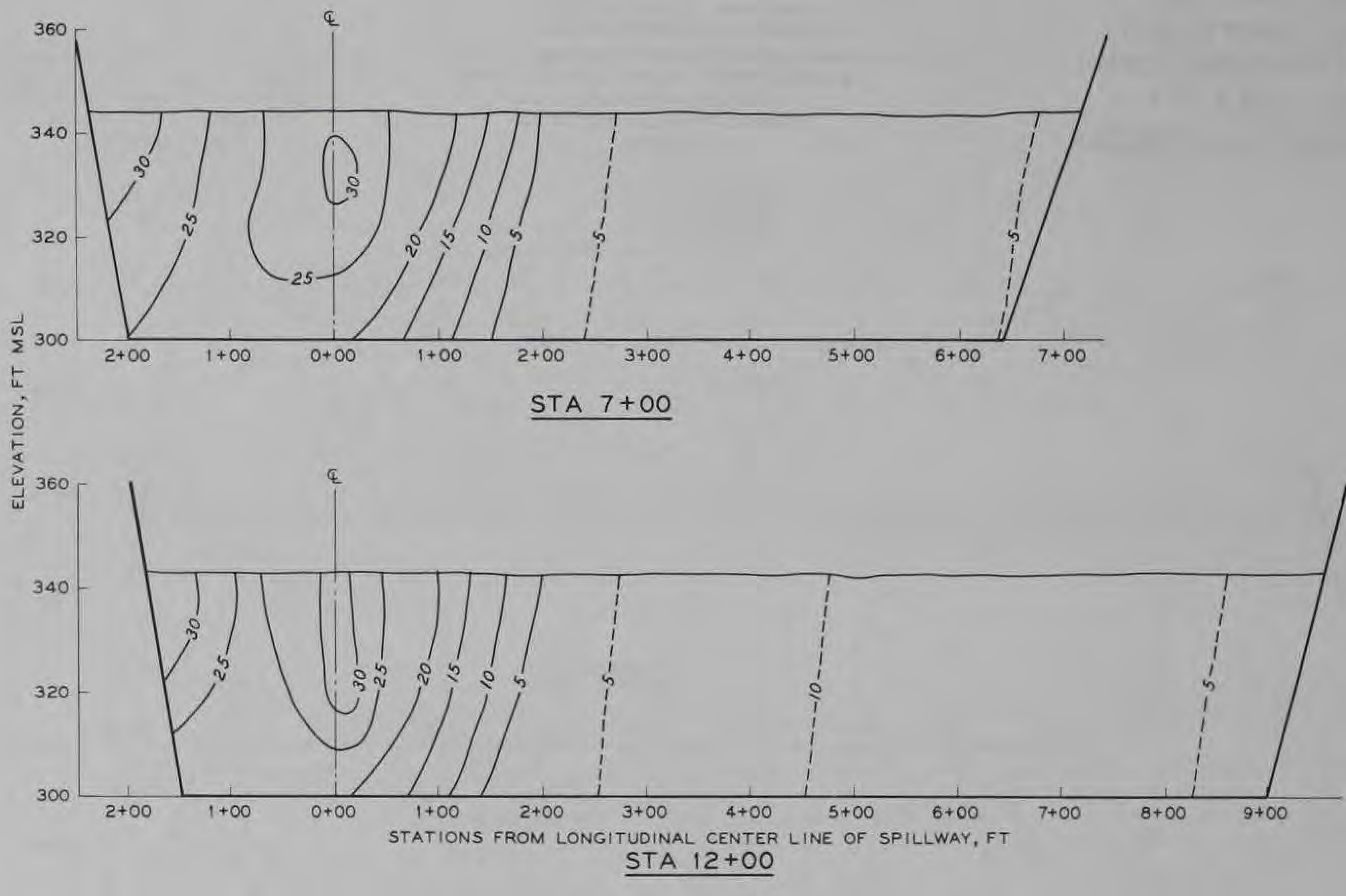


RATING CURVES OF WEIR CAPACITY

TYPES 1 AND 2 LEFT ABUTMENTS AND COMPUTED CURVE



ANTICIPATED TAILWATER
RATING CURVES



TEST CONDITIONS

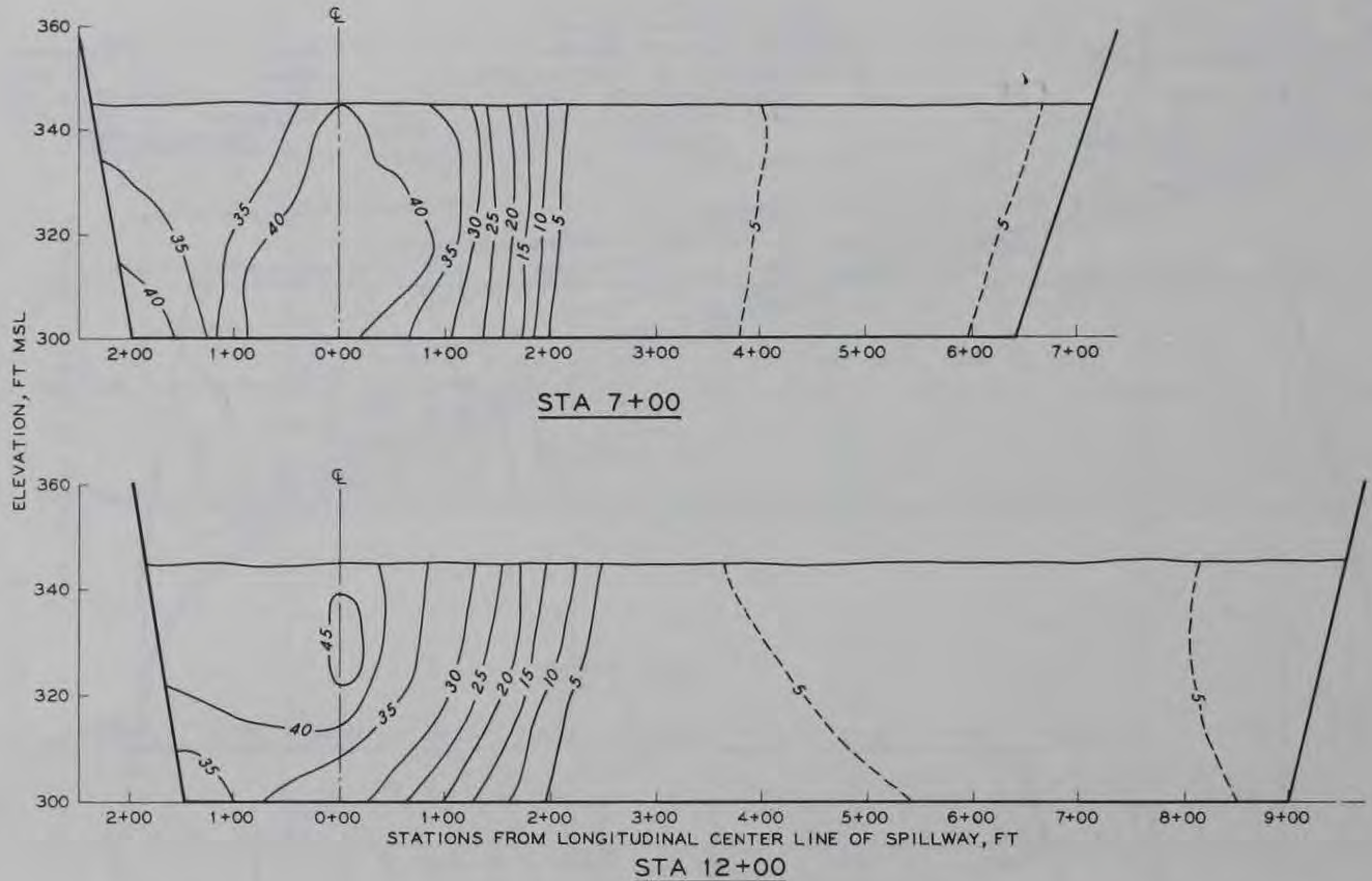
DISCHARGE .... 360,000 CFS
ALL GATES OPEN .... 21 FT
POOL EL 485.0
TAILWATER EL 343.0

LEGEND

——— DOWNSTREAM CURRENT
- - - UPSTREAM CURRENT

NOTE: ISOVELS ARE IN PROTOTYPE FT PER SEC LOOKING DOWNSTREAM.
STATION 0+00 IS TRANSVERSE CENTER LINE OF SPILLWAY.

VELOCITY DISTRIBUTIONS IN EXIT CHANNEL
TYPE 1 (ORIGINAL) FLIP BUCKET
DISCHARGE 360,000 CFS
STA 7+00 AND 12+00



TEST CONDITIONS

DISCHARGE .... 528,000 CFS
ALL GATES OPEN ...... 30.6 FT
POOL EL 485.0
TAILWATER EL 346.0

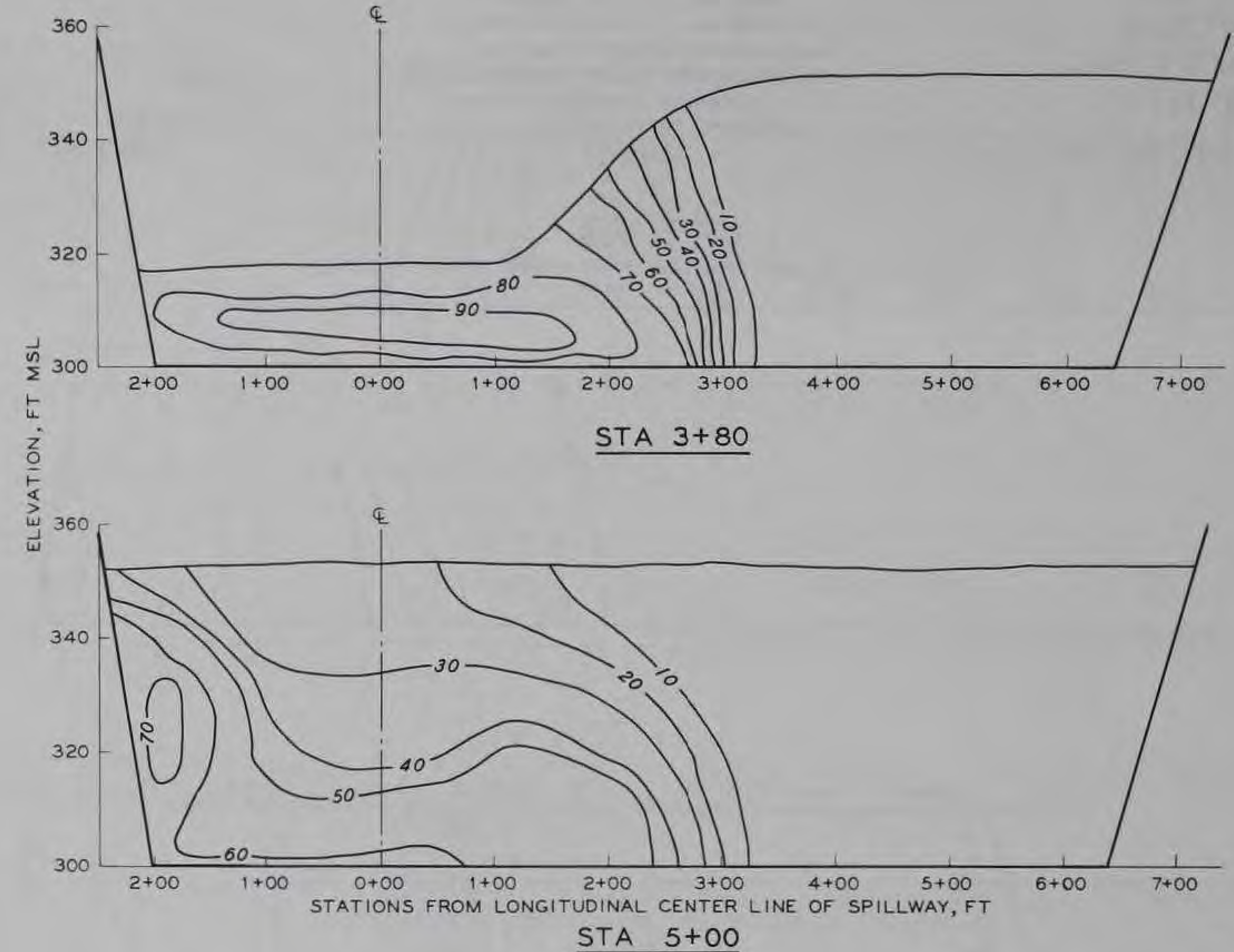
LEGEND

——— DOWNSTREAM CURRENT
– – – UPSTREAM CURRENT

NOTE: ISOVELS ARE IN PROTOTYPE FT PER SEC LOOKING DOWNSTREAM.
STATION 0+00 IS TRANSVERSE CENTER LINE OF SPILLWAY.

VELOCITY DISTRIBUTIONS IN EXIT CHANNEL

TYPE 1 (ORIGINAL) FLIP BUCKET
DISCHARGE 528,000 CFS
STA 7+00 AND 12+00



TEST CONDITIONS

DISCHARGE . . . . 800,000 CFS
FREE FLOW
POOL EL 490.0
TAILWATER EL 356.0

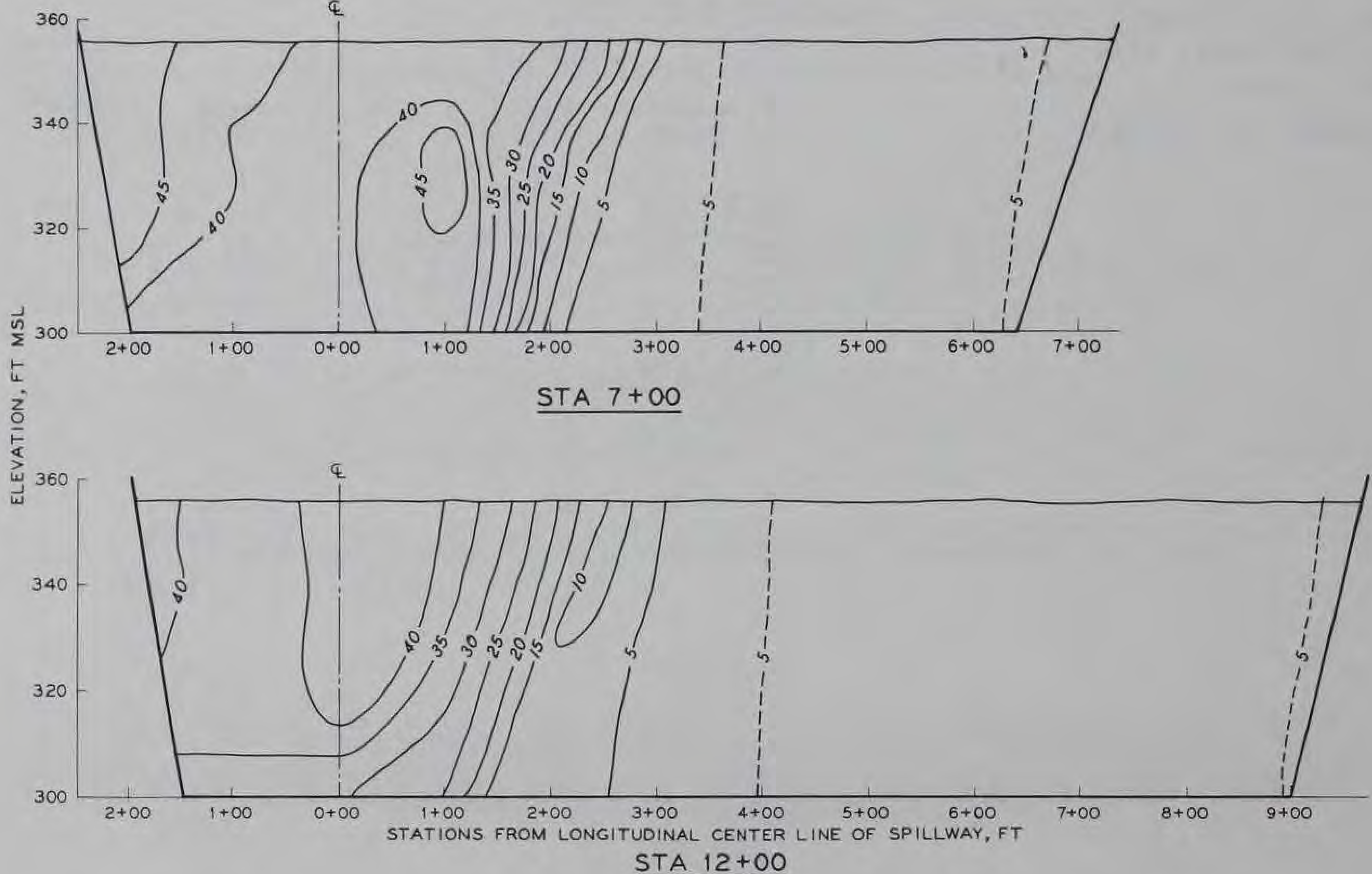
LEGEND

—— DOWNSTREAM CURRENT

NOTE: ISOVELS ARE IN PROTOTYPE FT PER SEC LOOKING DOWNSTREAM. STATION 0+00 IS TRANSVERSE CENTER LINE OF SPILLWAY.

VELOCITY DISTRIBUTIONS IN EXIT CHANNEL

TYPE 1 (ORIGINAL) FLIP BUCKET
DISCHARGE 800,000 CFS
STA 3+80 AND 5+00



TEST CONDITIONS

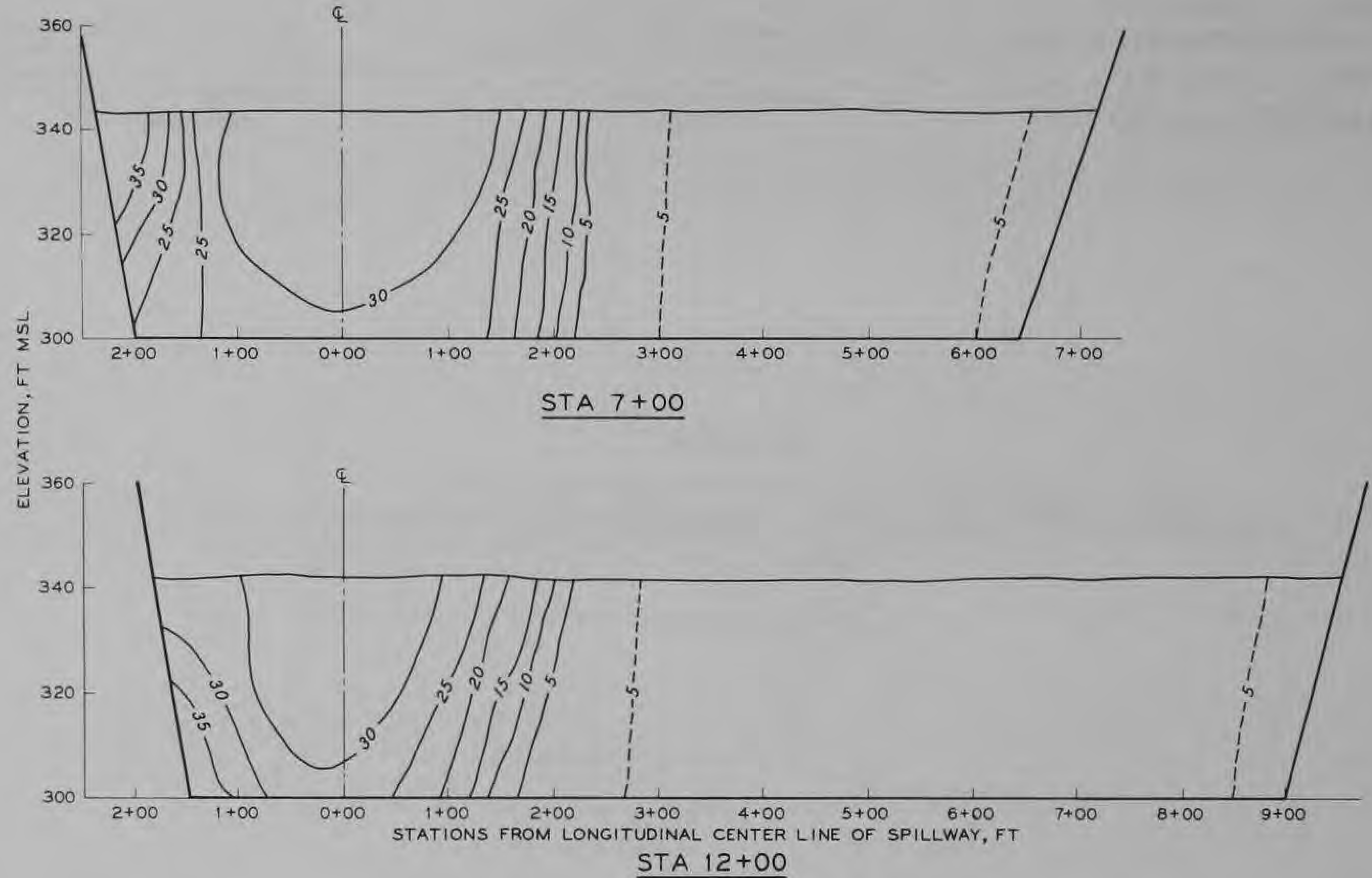
DISCHARGE . . . . 800,000 CFS
FREE FLOW
POOL EL 490.0
TAILWATER EL 356.0

LEGEND

——— DOWNSTREAM CURRENT
– – – UPSTREAM CURRENT

NOTE: ISOVELS ARE IN PROTOTYPE FT PER SEC LOOKING DOWNSTREAM
STATION 0+00 IS TRANSVERSE CENTER LINE OF SPILLWAY.

VELOCITY DISTRIBUTIONS IN EXIT CHANNEL
TYPE 1 (ORIGINAL) FLIP BUCKET
DISCHARGE 800,000 CFS
STA 7+00 AND 12+00



TEST CONDITIONS

DISCHARGE .... 360,000 CFS
ALL GATES OPEN .... 21 FT
POOL EL 485.0
TAILWATER EL 343.0

LEGEND

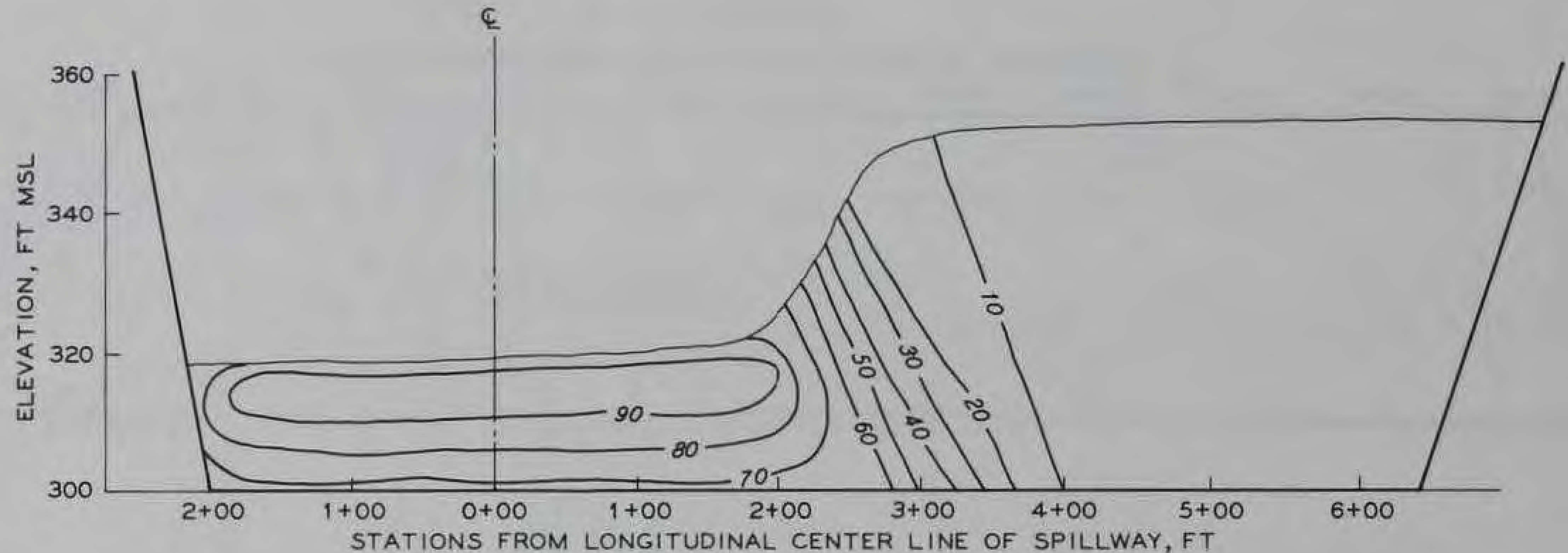
——— DOWNSTREAM CURRENT
– – – UPSTREAM CURRENT

NOTE: ISOVELS ARE IN PROTOTYPE FT PER SEC LOOKING DOWNSTREAM.
STATION 0+00 IS TRANSVERSE CENTER LINE OF SPILLWAY.

VELOCITY DISTRIBUTIONS IN EXIT CHANNEL

TYPE 2 (RECOMMENDED) FLIP BUCKET
DISCHARGE 360,000 CFS
STA 7+00 AND 12+00

PLATE 16



STA 4+40

TEST CONDITIONS

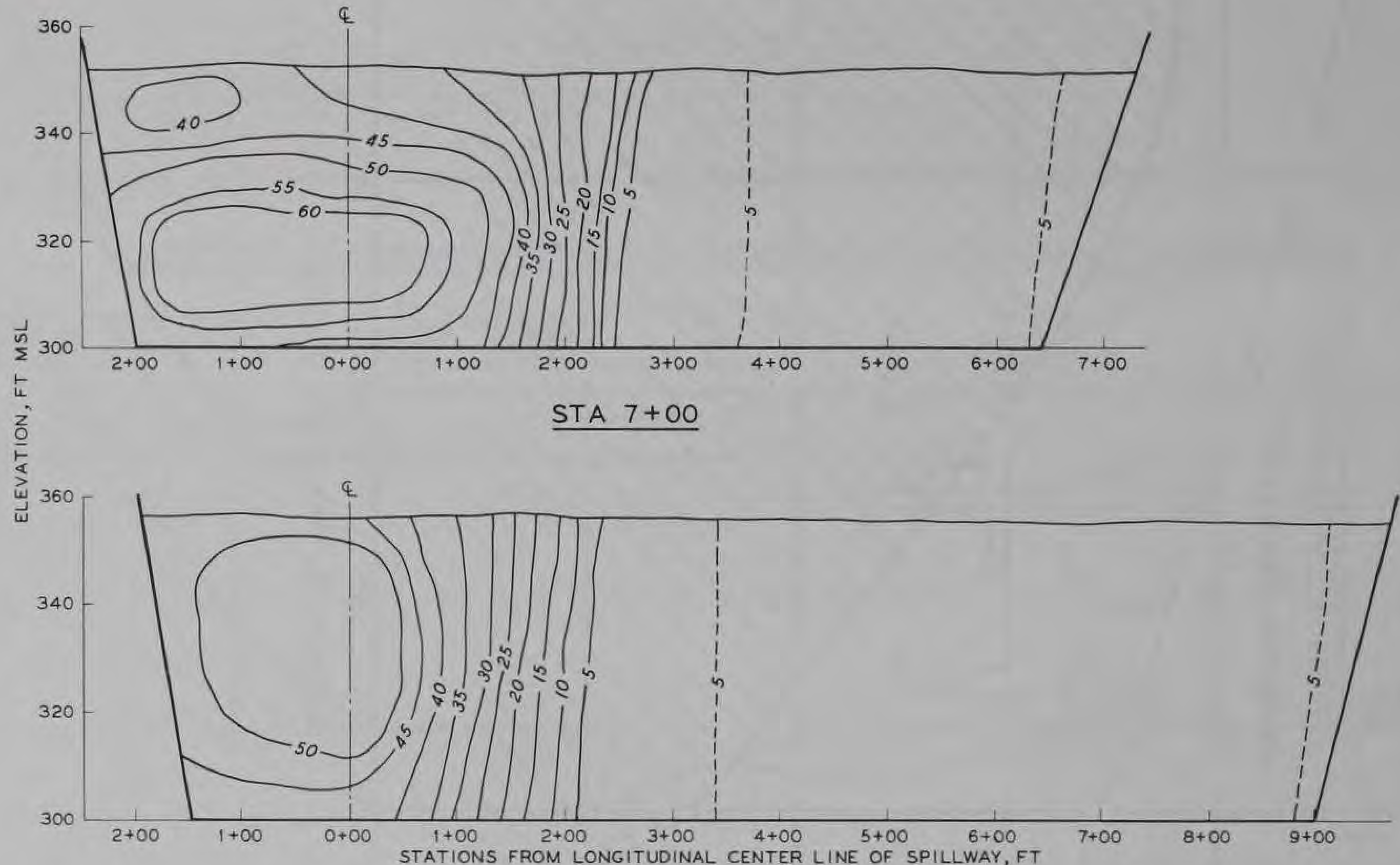
DISCHARGE .... 800,000 CFS
FREE FLOW
POOL EL 490.0
TAILWATER EL 356.0

LEGEND

—— DOWNSTREAM CURRENT

NOTE: ISOVELS ARE IN PROTOTYPE FT PER SEC LOOKING DOWNSTREAM. STATION 0+00 IS TRANSVERSE CENTER LINE OF SPILLWAY.

VELOCITY DISTRIBUTIONS
IN EXIT CHANNEL
TYPE 2 (RECOMMENDED) FLIP BUCKET
DISCHARGE 800,000 CFS
STA 4+40



STA 7+00

STA 12+00

TEST CONDITIONS

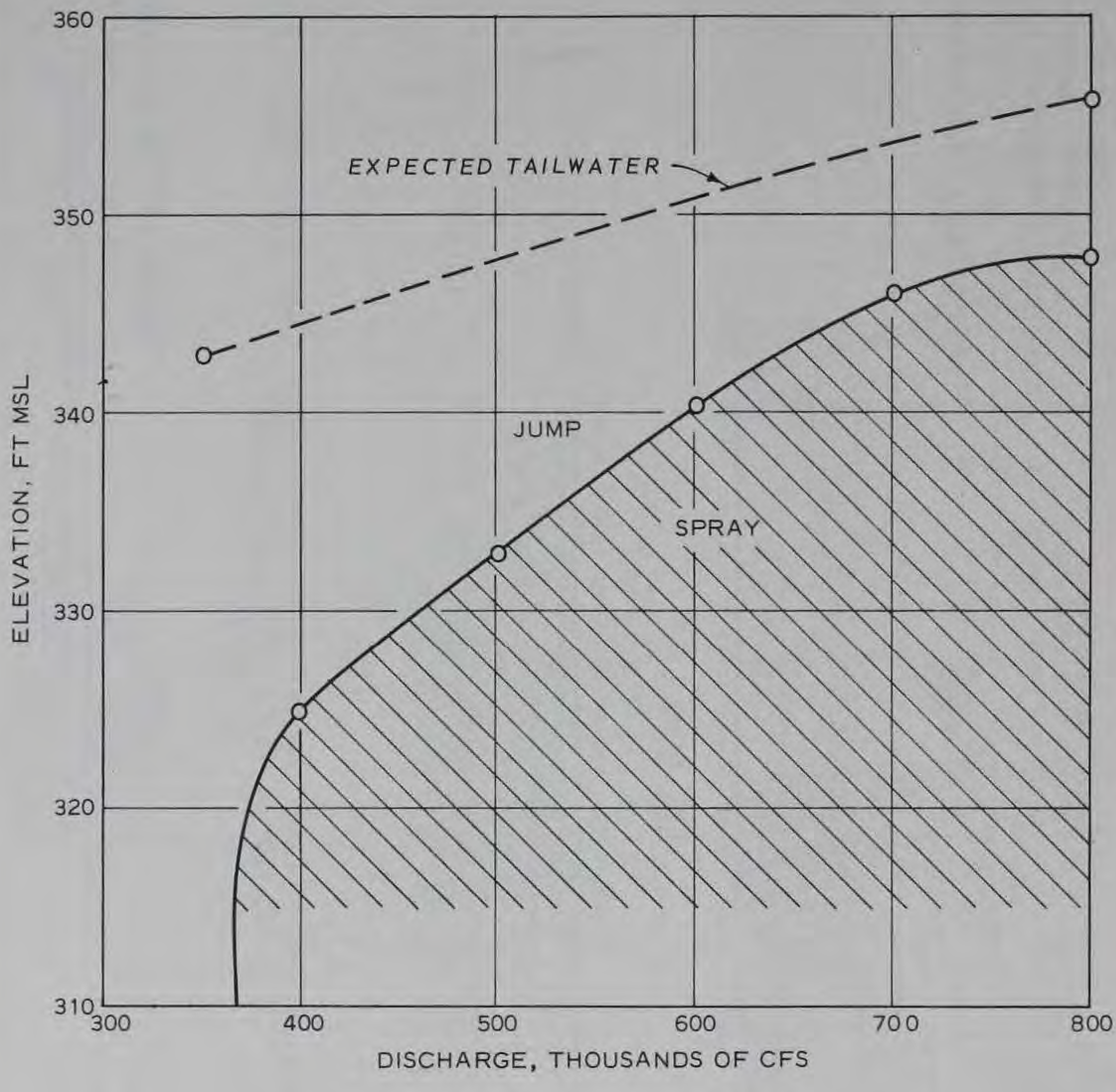
DISCHARGE .... 800,000 CFS
FREE FLOW
POOL EL 490.0
TAILWATER EL 356.0

LEGEND

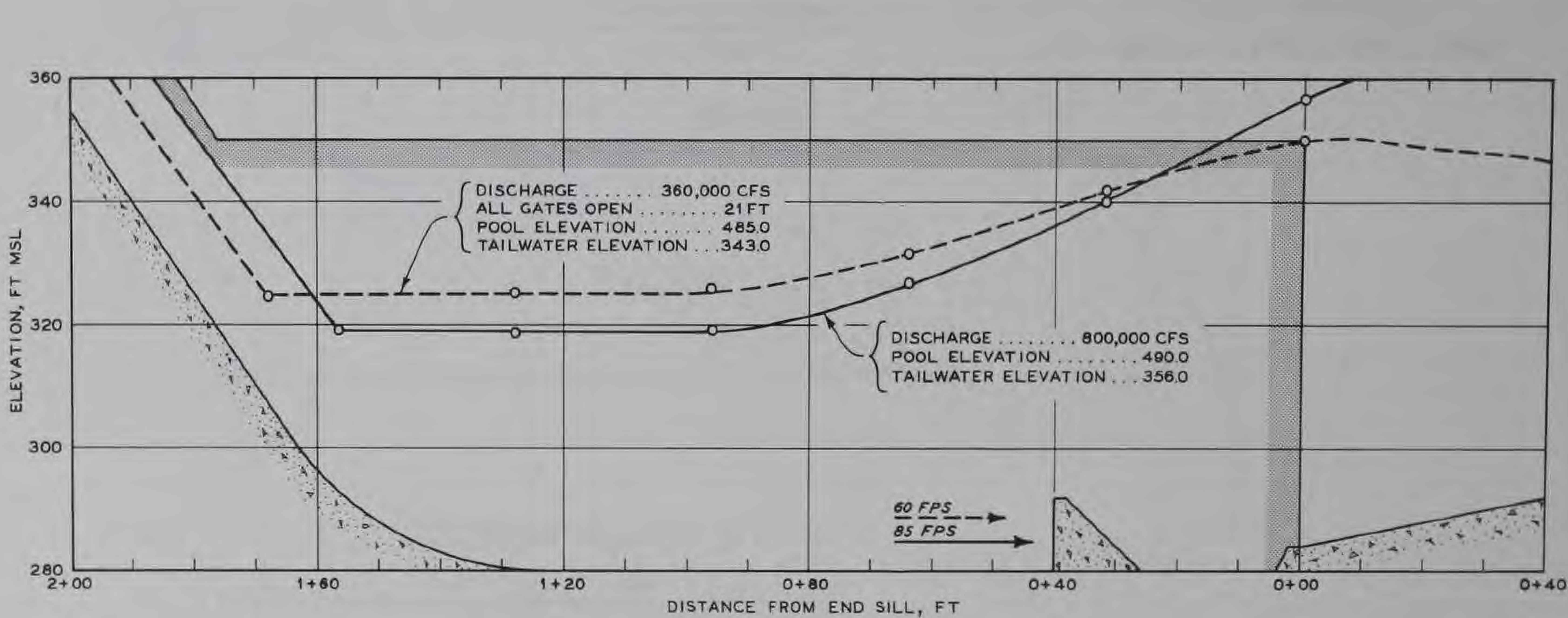
—— DOWNSTREAM CURRENT
– – – UPSTREAM CURRENT

NOTE: ISOVELS ARE IN PROTOTYPE FT PER SEC LOOKING DOWNSTREAM. STATION 0+00 IS TRANSVERSE CENTER LINE OF SPILLWAY.

VELOCITY DISTRIBUTIONS IN EXIT CHANNEL
TYPE 2 (RECOMMENDED) FLIP BUCKET
DISCHARGE 800,000 CFS
STA 7+00 AND 12+00



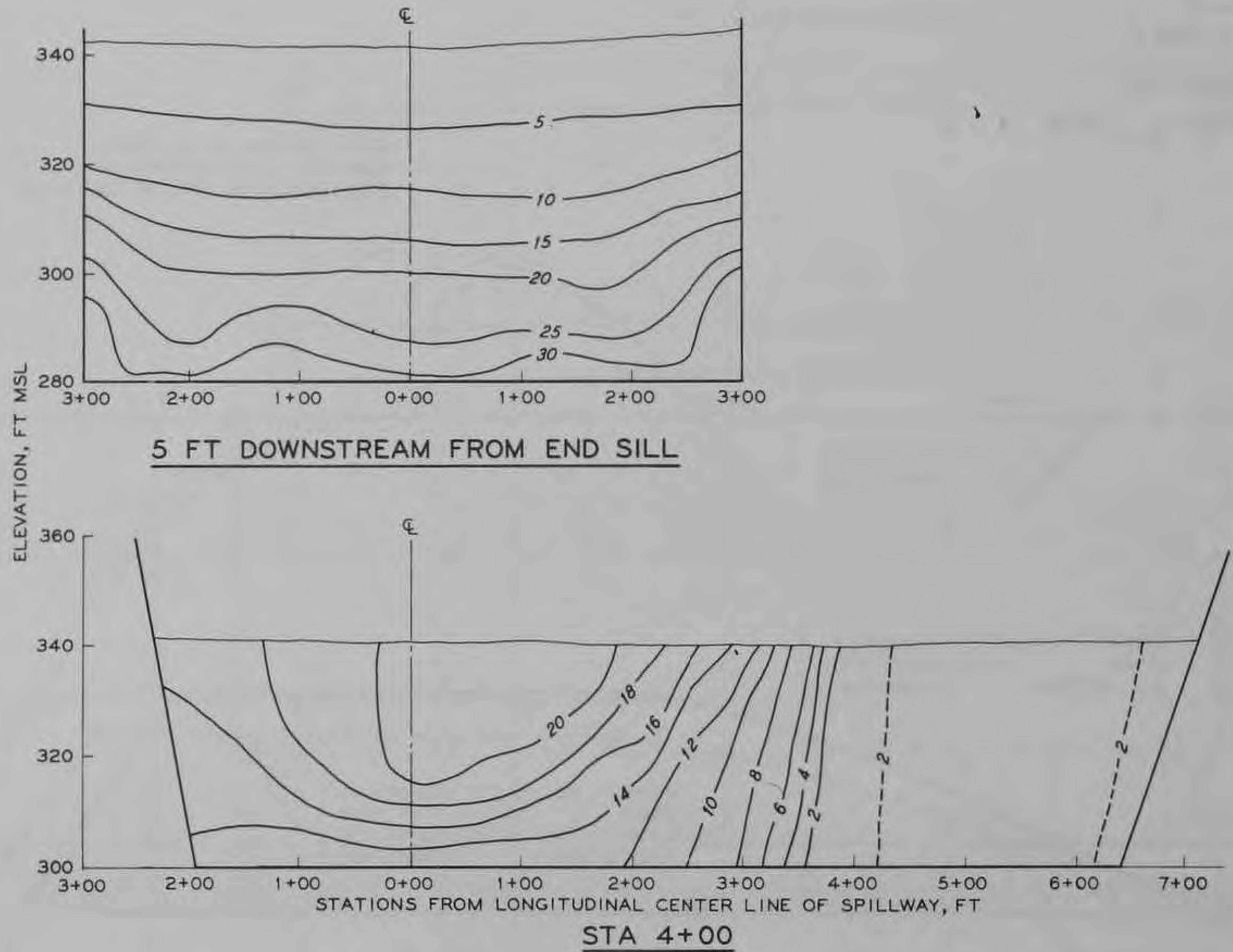
SPRAY CHARACTERISTICS OF TYPE 3 (RECOMMENDED) STILLING BASIN



NOTE: VELOCITIES INDICATED ARE MAXIMUMS THAT OCCURRED IN VICINITY OF BAFFLE PIERS

WATER-SURFACE PROFILES ALONG LEFT WALL

TYPE 3 (RECOMMENDED) STILLING BASIN



TEST CONDITIONS

DISCHARGE .... 360,000 CFS
ALL GATES OPEN .... 21 FT
POOL EL 485.0
TAILWATER EL 343.0

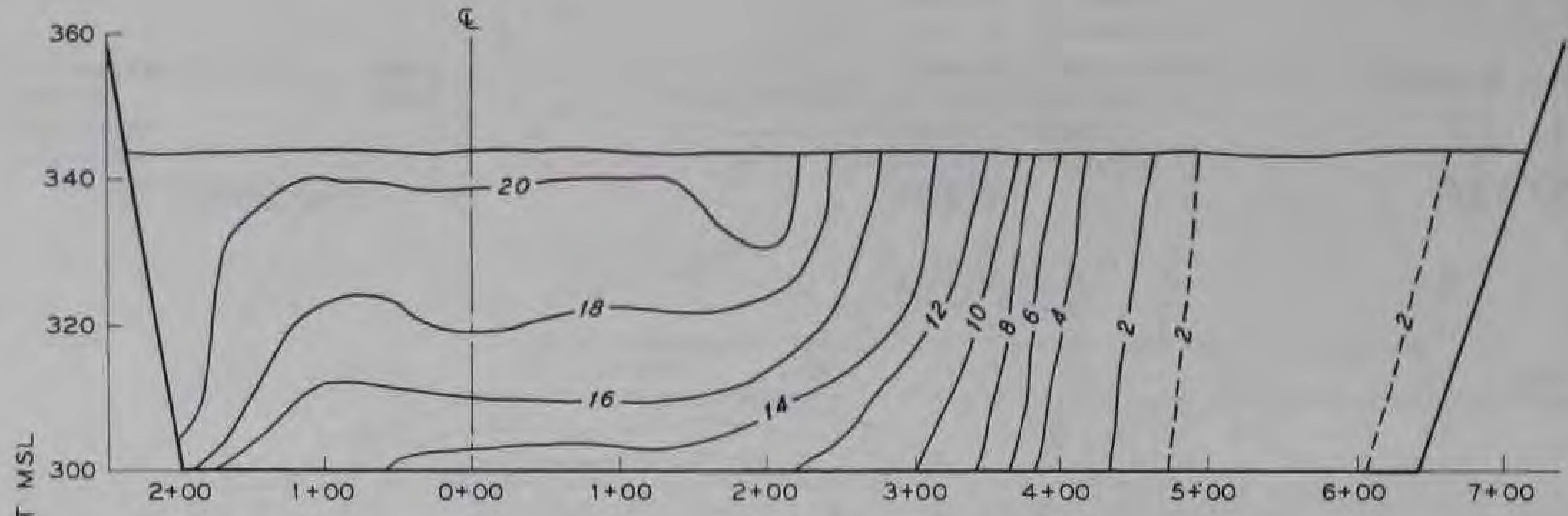
LEGEND

——— DOWNSTREAM CURRENT
– – – UPSTREAM CURRENT

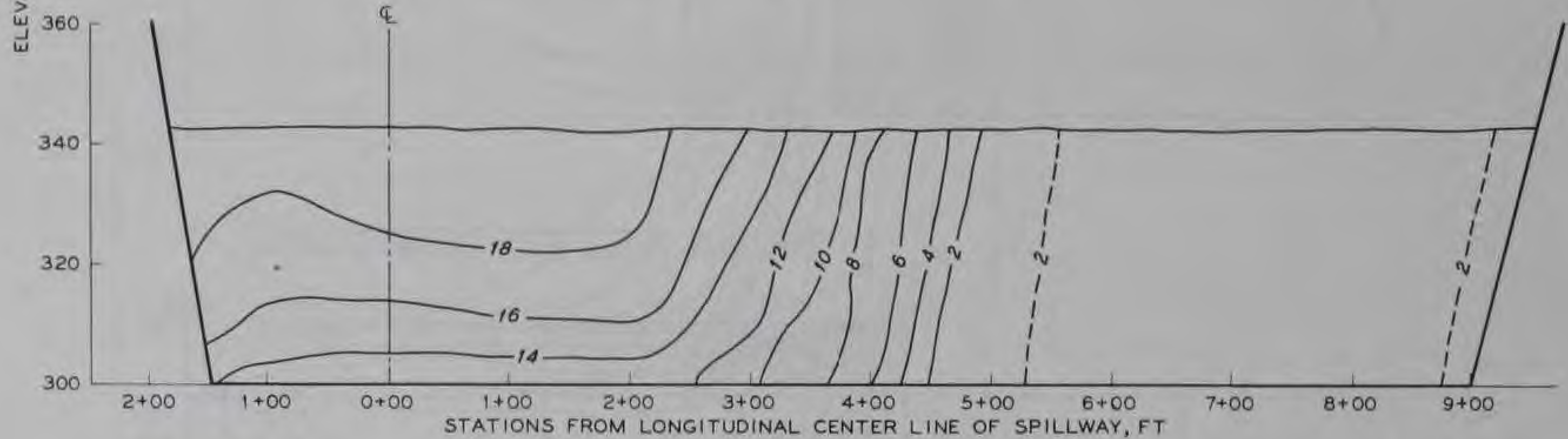
NOTE: ISOVELS ARE IN PROTOTYPE FT PER SEC LOOKING DOWNSTREAM
STATION 0+00 IS TRANSVERSE CENTER LINE OF SPILLWAY.

VELOCITY DISTRIBUTIONS IN EXIT CHANNEL

TYPE 3 (RECOMMENDED) STILLING BASIN
DISCHARGE 360,000 CFS
5 FT DOWNSTREAM FROM END SILL
AND STA 4+00



STA 7+00



STA 12+00

TEST CONDITIONS

DISCHARGE .... 360,000 CFS
ALL GATES OPEN .... 21 FT
POOL EL 485.0
TAILWATER EL 343.0

LEGEND

——— DOWNSTREAM CURRENT
– – – UPSTREAM CURRENT

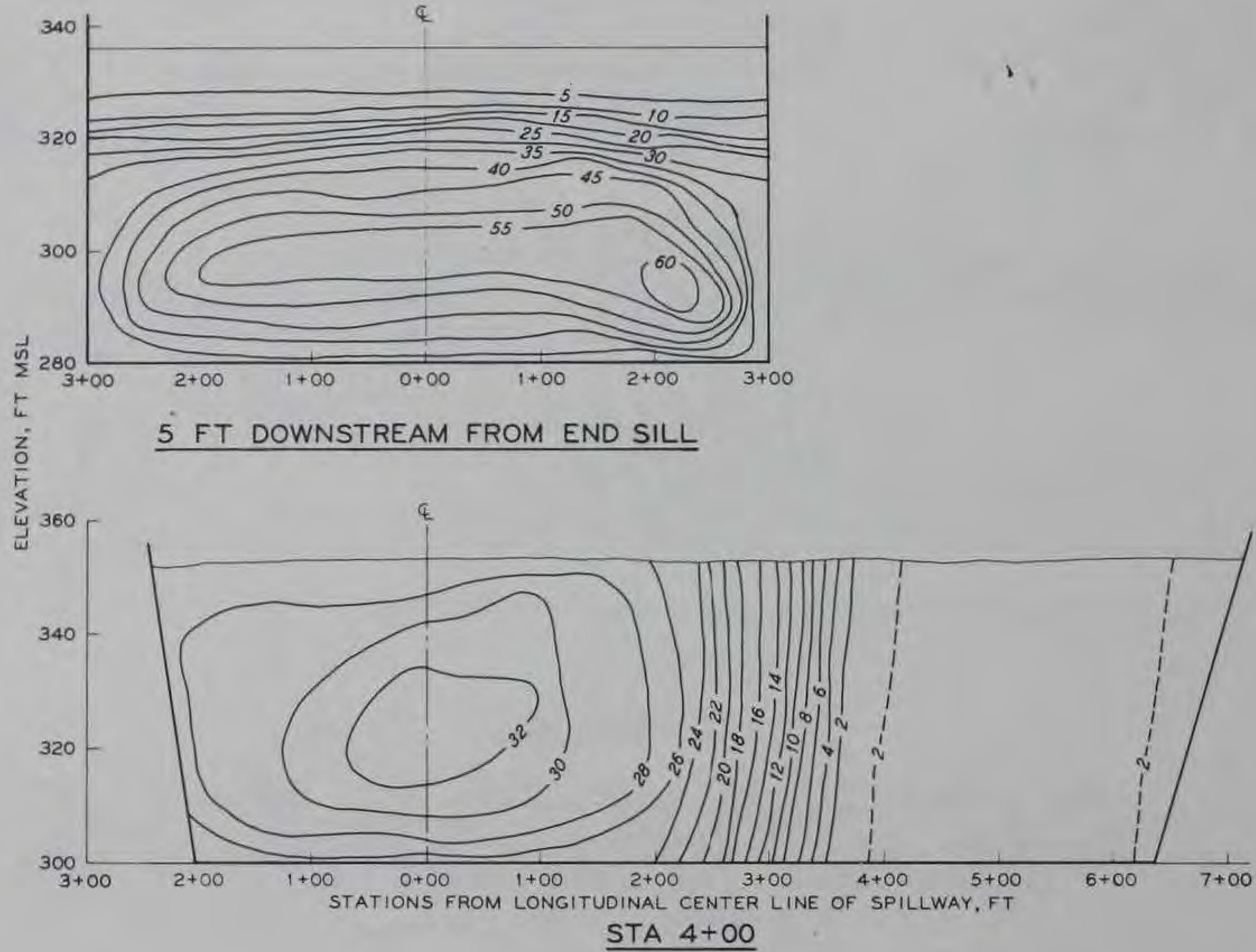
NOTE: ISOVELS ARE IN PROTOTYPE FT PER SEC LOOKING DOWNSTREAM. STATION 0+00 IS TRANSVERSE CENTER LINE OF SPILLWAY.

VELOCITY DISTRIBUTIONS IN EXIT CHANNEL

TYPE 3 (RECOMMENDED) STILLING BASIN

DISCHARGE 360,000 CFS

STA 7+00 AND 12+00



TEST CONDITIONS

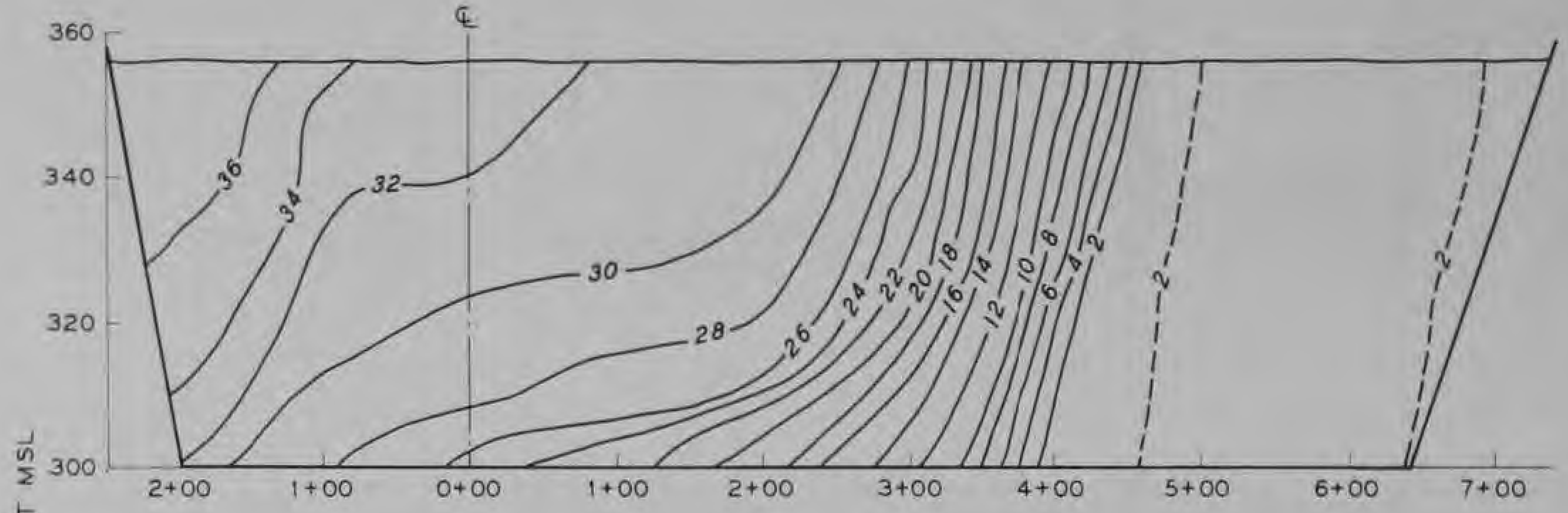
DISCHARGE .... 800,000 CFS
FREE FLOW
POOL EL 490.0
TAILWATER EL 356.0

LEGEND

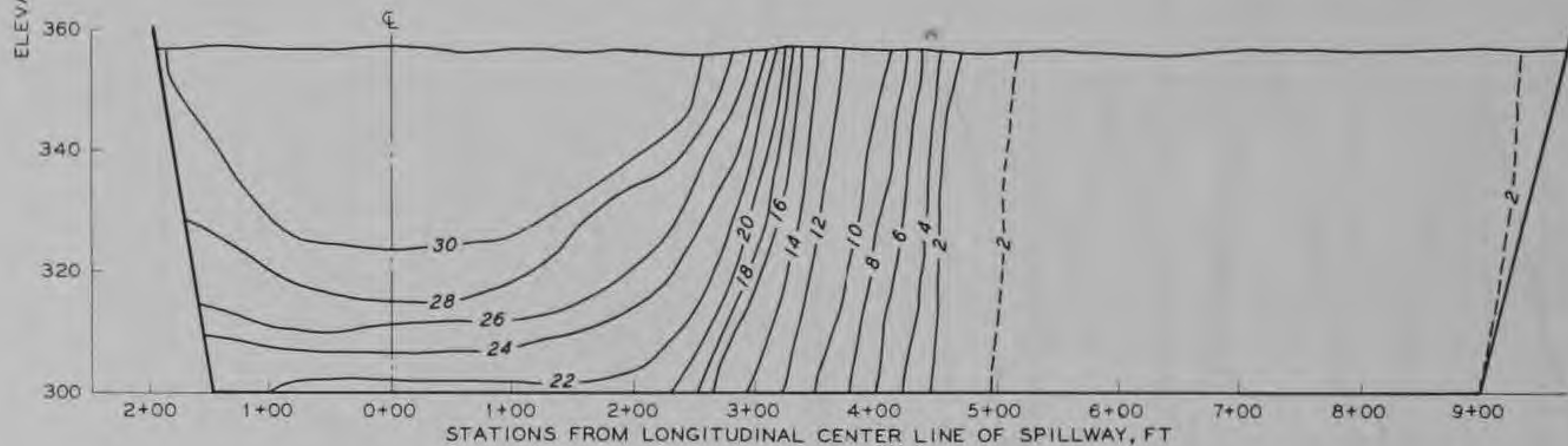
——— DOWNSTREAM CURRENT
- - - UPSTREAM CURRENT

NOTE: ISOVELS ARE IN PROTOTYPE FT PER SEC LOOKING DOWNSTREAM
STATION 0+00 IS TRANSVERSE CENTER LINE OF SPILLWAY.

VELOCITY DISTRIBUTIONS IN EXIT CHANNEL
TYPE 3 (RECOMMENDED) STILLING BASIN
DISCHARGE 800,000 CFS
5 FT DOWNSTREAM FROM END SILL
AND STA 4+00



STA 7+00



STA 12+00

TEST CONDITIONS

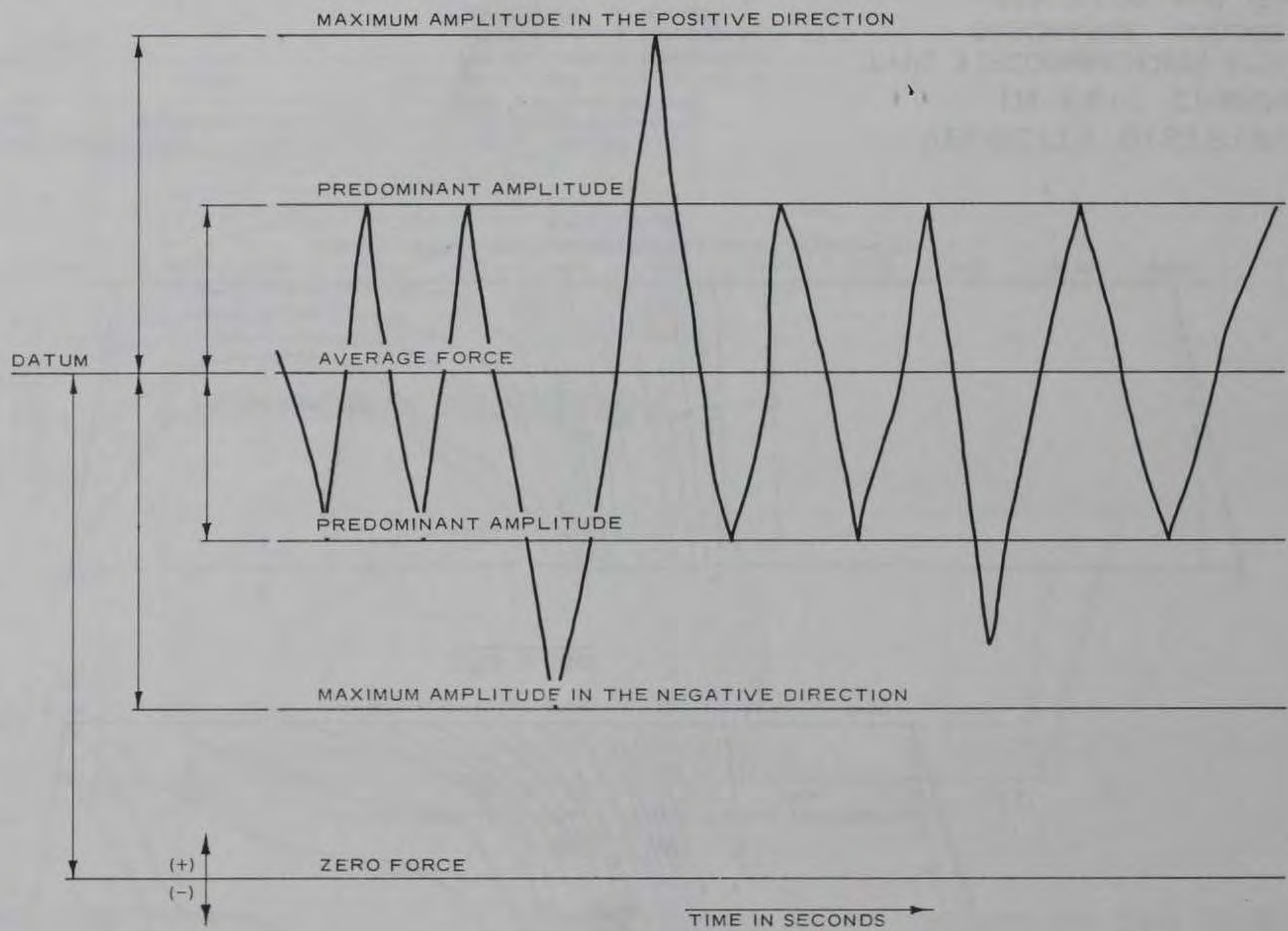
DISCHARGE .... 800,000 CFS
FREE FLOW
POOL EL 490.0
TAILWATER EL 356.0

LEGEND

——— DOWNSTREAM CURRENT
– – – UPSTREAM CURRENT

NOTE: ISOVELS ARE IN PROTOTYPE FT PER SEC LOOKING DOWNSTREAM
STATION 0+00 IS TRANSVERSE CENTER LINE OF SPILLWAY.

VELOCITY DISTRIBUTIONS IN EXIT CHANNEL
TYPE 3 (RECOMMENDED) STILLING BASIN
DISCHARGE 800,000 CFS
STA 7+00 AND 12+00



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Final report

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11. SUPPLEMENTARY NOTES	12. SPONSORING MILITARY ACTIVITY
	U. S. Army Engineer District Savannah, Georgia

13. ABSTRACT

Tests of the spillway for the Trotters Shoals Dam were conducted on a 1:80-scale model to investigate flow conditions in the approach and exit channel and the performance of various elements of the structure. Particular emphasis was placed on the development of an energy dissipator that would provide satisfactory energy dissipation and exit channel flow conditions. Approach flow conditions were improved by revising the left abutment to prevent a severe drawdown of the water surface in the vicinity of the left abutment. The model indicated that, after the left abutment was modified, the spillway capacity was equal to that computed. Nappe separation from the downstream quadrant of the crest was prevented by extending the crest piers upstream and shifting the gate slots downstream relative to their original positions. These modifications improved pressure conditions within the gate slots as well as along the spillway crest. Model tests indicated that the type 3 stilling basin provided more appropriate energy dissipation than the flip buckets and that it reduced the maximum velocities and the concentration of flow along the left side of the exit channel. Model tests were also conducted to investigate the instantaneous forces induced by the hydraulic jump that were exerted on the monoliths composing the left stilling basin wall. These instantaneous forces were measured electronically, and the measurements should provide information pertinent to the structural design of the stilling basin walls.

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14. KEY WORDS	LINK A		LINK B		LINK C	
	ROLE	WT	ROLE	WT	ROLE	WT
Energy dissipators						
Hydraulic models						
Spillways						
Stilling basins						
Trotters Shoals Dam						

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Security Classification