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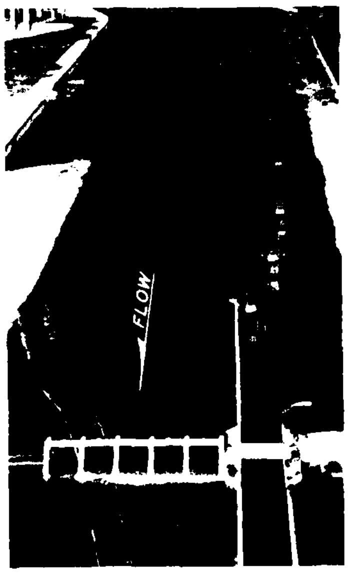

HYDRAULICS LABORATORY

TECHNICAL REPORT HL-89-16

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RED RIVER WATERWAY
JOHN H. OVERTON LOCK AND DAM

Report 4
STILLING BASIN, RIPRAP, AND HYDROPOWER REQUIREMENTS

Spillway and Hydropower Model Investigation

by

Stephen T. Maynord, Jerry V. Markussen

Hydraulics Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

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Tests were conducted on a 1:50-scale model of John H. Overton Lock and Dam, Alexandria, LA, to develop a satisfactory spillway, stilling basin, and riprap plan for the proposed project. Tests were conducted with and without a proposed hydropower plant. The recommended stilling basin and riprap designs provided stable conditions for single gate operation with normal upper pool and minimum tailwater. Tests were conducted with four- and three-unit powerhouses having various approach and exit configurations.

During construction of John H. Overton Lock and Dam, sediment deposition problems occurred in the lower lock approach of Red River Lock and Dam No. 1. Additional sedimentation studies were conducted to ensure that these problems would not occur at John H. Overton. Dikes placed in the downstream channel were proposed as a solution to the sedimentation problem. The stability of the riprap forming these dikes and the impact of the dikes on the water-surface profiles were also studied in this investigation.

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PREFACE

The model investigation reported herein was authorized by the Headquarters, US Army Corps of Engineers (HQUSACE), and the US Army Engineer Division, Lower Mississippi Valley (LMVD), at the request of the US Army Engineer District, New Orleans (LMN). During the course of the investigation, responsibility for John H. Overton Lock and Dam was transferred from LMN to the US Army Engineer District, Vicksburg (LMK). In addition to this hydraulic model investigation, a numerical model study and two other physical model studies of John H. Overton Lock and Dam were conducted at WES: a fixed-bed navigation study (Report 2); a movable-bed sedimentation study (Report 3); and a numerical model investigation (Report 5). This is Report 4 of the series. Report 1, to be published later, will summarize all of the physical and numerical modeling studies.

The study was conducted by personnel of the Hydraulics Laboratory (HL), US Army Engineer Waterways Experiment Station (WES), during the period January 1980 to February 1986 under the general supervision of Messrs. H. B. Simmons and F. A. Herrmann, Jr., former and present Chiefs, HL, respectively; and J. L. Grace, Jr., and G. A, Pickering, former and present Chiefs, Hydraulic Structures Division (HSD), HL, respectively. The model tests were conducted by Messrs. J. V. Markussen, R. Bryant, and S. T. Maynord, Spillways and Channels Branch, HSD, under the supervision of Mr. N. R. Oswalt, Chief, Spillways and Channels Branch. The model was constructed by the Model Shops, Engineering and Construction Services Division, WES, Mr. S. J. Leist, Chief. The report was prepared by Messrs. Maynord and Markussen and edited by Mrs. Marsha C. Gay, Information Technology Laboratory, WES.

During the course of the investigation, Mr. Bruce McCartney, HQUSACE; Messrs. Estus Walker and Larry Cook, LMVD; Messrs. Cecil Soileau, Willie Shelton, Don Theriot, Marcial Facio, Philip Ziegler, Mike Sanchez-Barbudo, Dennis Strecher, Paublo Raman, Reynold Brossard, James Miles, and Arthur Laurent, LMN; Messrs. Tom Quigley, Bob Hughey, Tom Mudd, and Billy Arthur, US Army Engineer District, St. Louis; and Messrs. Phil Combs, Nolan Raphelt, and Rick Robertson of LMK visited WES to observe model testing and discuss test results.

Commander and Director of WES during preparation of this report was COL Larry B. Fulton, EN. Technical Director was Dr. Robert W. Whalin.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
feet	0.3048	metres
inches	2.54	centimetres
miles (US statute)	1.609347	kilometres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre

RED RIVER WATERWAY, JOHN H. OVERTON LOCK AND DAM

STILLING BASIN, RIPRAP, AND HYDROPOWER REQUIREMENTS

Spillway and Hydropower Model Investigation

PART I: INTRODUCTION

Location of Project

1. The Red River Waterway Project consists of four distinct reaches: (a) Mississippi River to Shreveport, LA; (b) Shreveport, LA, to Daingerfield, TX; (c) Shreveport, LA, to Index, AR; and (d) Index, AR, to Denison Dam, TX. Only the first reach (Figure 1) is pertinent to this report. Within the first reach, the plan provides for establishing a navigable channel approximately 236 miles* long and 9 ft deep by 200 ft wide from the Mississippi River to Shreveport via the Old and Red Rivers and construction of a system of five locks and dams. The lock dimensions (usable chamber) will be 84 ft wide and 685 ft long. John H. Overton Lock and Dam (JHO) will be located 16 miles downstream of Alexandria, LA, at 1967 river mile 87. The location of the project is shown in Figure 1.

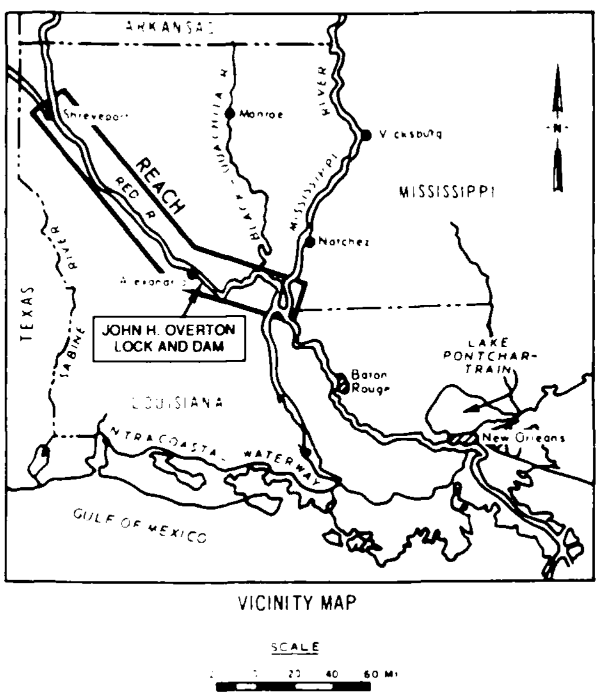

Figure 1. Vicinity map

Pertinent Project Features

2. The principal structures associated with JHO will consist of a navigation lock, a gated spillway, concrete abutment walls, an overflow weir, and an optional hydropower facility within the overflow weir (Plate 1). The lock, with nominal chamber dimensions of 84 by 785 ft, pintle to pintle,

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

will be on the left riverbank looking downstream. The lift will vary up to a maximum of 24 ft.

3. The navigation dam will contain five 38-ft-high by 60-ft-wide tainter gates mounted between 8-ft-wide piers (Plate 2). The gate sill will be at el 28.0,* and the tops of the gates, when closed, will be at el 66.0 and will provide a 2-ft freeboard above the normal upper pool elevation of 64.0. The net width of the spillway is 300 ft and the gross width of the abutments from face to face is 332 ft. Plate 2 shows the original (type 1) design spillway and stilling basin portion of the dam.

4. The hydropower facility may be added to JHO after construction of the lock and dam. As presently configured, the facility consists of three bulb turbines having a maximum discharge of 8,000 cfs per turbine.

Purpose of Model Investigation

5. Hydraulic model tests were conducted to assist in the development of satisfactory stilling basin designs and riprap protection plans for the conditions of one gate one-half and fully open and subject to normal pool and minimum tailwater elevations. The model provided a means for checking discharge characteristics of the spillway. Tests were conducted to develop satisfactory flow conditions approaching and exiting the hydropower facilities and to develop a stable riprap plan for the downstream sediment dikes. These dikes were added to the project after sedimentation problems occurred in the lower lock approach of the Red River Lock and Dam No 1 prototype.

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

PART II: THE MODEL

Description

6. The investigation was conducted in a 1:50-scale model that reproduced the gated spillway, the navigation lock, upstream guard wall, downstream guide wall, and overflow weir, as shown in Figures 2 and 3. A 1,400-ft length of upstream and a 2,700-ft length of downstream topography were reproduced. The approach area was molded in cement mortar to sheet metal templates. The spillway weir, tainter gates, gate piers, lock, and overflow weir were fabricated of sheet metal. The stilling basin and its elements were constructed of wood treated with a waterproofing compound to prevent expansion. Initially, the downstream area was molded in cement mortar to sheet metal templates, but this area was replaced with a blanket of crushed limestone to permit study and development of the plan of riprap protection required.

7. Discharges were measured with venturi meters, and water-surface elevations were measured with point gages. Sand and riprap scour depths were measured with point gages, and velocities were measured with a pitot tube or propeller meter. Steel rails set to grade along the sides of the flume provided a reference plane for measuring devices. Tailwater elevations were regulated by a flap gate at the downstream end of the flume.

Scale Relations

8. The accepted equations of hydraulic similitude, based on the Froudian criteria, were used to express mathematical relations between the dimensions and hydraulic quantities of the model and prototype. General relations for the transfer of model data to prototype equivalents are as follows:

Characteristic	Ratio*	Scale Relation Model:Prototype
Length	L_r	1:50
Area	$A_r = L_r^2$	1:2,500
Velocity	$V_r = L_r^{1/2}$	1:7.07
Discharge	$Q_r = L_r^{5/2}$	1:17,678
Force or weight	$F_r = L_r^3$	1:125,000

* Dimensions are in terms of length, time, and mass.

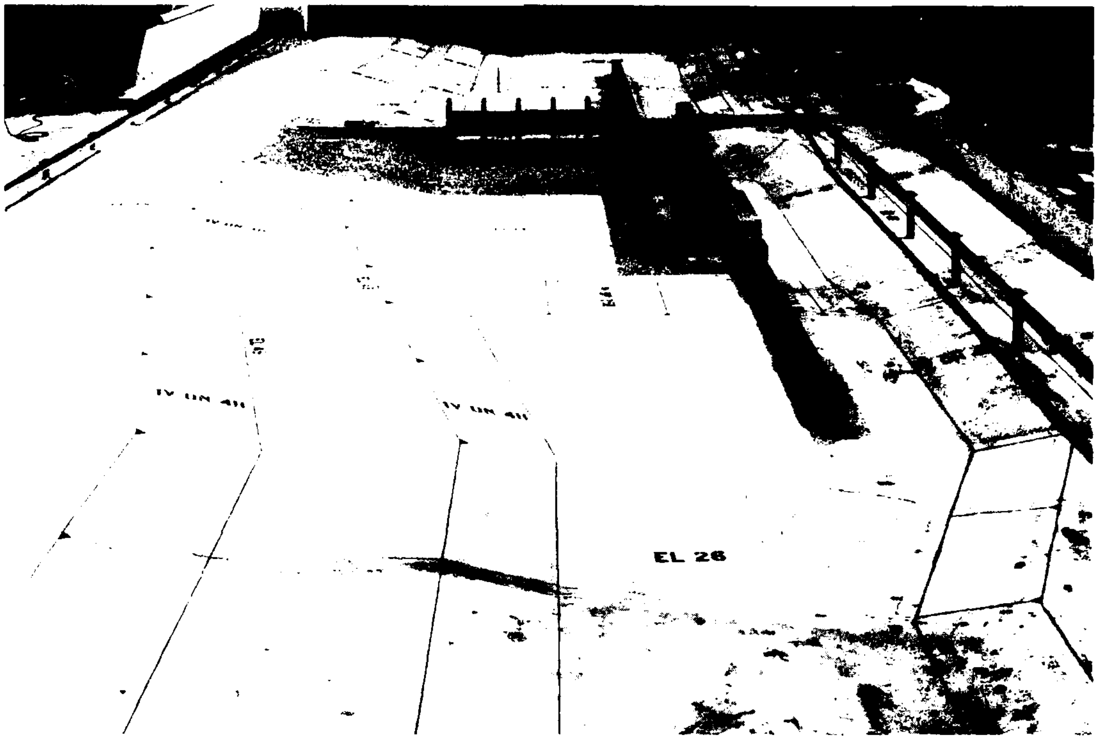

Figure 2. General view of 1:50-scale model, looking upstream

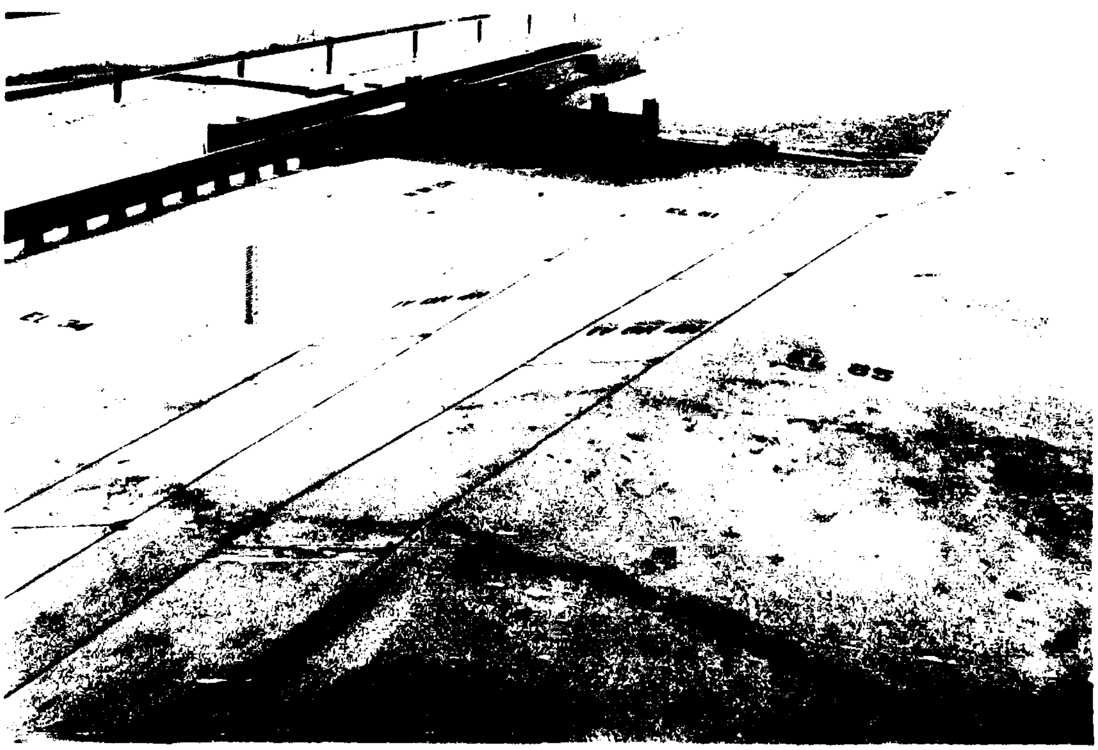

Figure 3. General view of 1:50-scale model, looking downstream

9. Model measurements of discharge, water-surface elevation, and velocities can be transferred quantitatively to prototype equivalents by means of these scale relations. Evidences of sand scour, however, are considered only qualitatively reliable, since it is not possible to reproduce quantitatively in a model the same ratio of flow depth to the diameter of bed material representative of the prototype.

PART III: TESTS AND RESULTS

Project Without Hydropower and Without Downstream Sediment Dikes

10. The initial portion of this investigation was directed at developing a satisfactory plan for the project without a hydropower facility. The sedimentation problems at Lock and Dam No. 1 prototype had not yet occurred and the downstream sediment dikes were not a part of this portion of the study. The overall plan of the project, the type 1 (original) design, is shown in Plate 1. The tailwater rating curve is shown in Plate 3.

Stilling basin design

11. One of the primary purposes of this study was to develop an adequate stilling basin design. The requirements for stilling basin performance set forth by the US Army Engineer District, New Orleans, were as follows:

Condition	Performance
Normal gate openings, normal pools	Good energy dissipation, no standing waves
One gate half open, normal upper pool, lower pool el 45	Good energy dissipation, occasional standing waves
One gate half open, normal upper pool, minimum lower pool, el 40	Some standing waves and minor riprap damage allowable, project integrity not threatened
One gate fully open, normal upper pool, lower pool el 46	Occasional standing waves and riprap blanket movement, minor damage allowable but project integrity must be maintained
One gate fully open, normal upper pool, minimum lower pool, el 40	Some standing waves and riprap blanket movement, minor damage allowable but project integrity not threatened

The New Orleans District also required that the stilling basin apron be no lower than el 12.

12. Tests were conducted in the model to determine the hydraulic performance of the type 1 (original) design stilling basin. The type 1 design stilling basin (Plate 2) consisted of a 100-ft-long apron (2.5 d_2 where d_2 is the sequent depth of hydraulic jump) at el 12 with two rows of 5.5-ft-high and 4.25-ft-wide baffle piers spaced 4.25 ft apart, and a 1V on 1H sloping end sill. This design was tested according to the criteria established by the

New Orleans District using the single-gate emergency operating conditions given in paragraph 11. Test results indicated that this design provided poor energy dissipation as evidenced by excessively high velocities over the end sill and the formation of standing waves in the lower pool extending along the full length of the lock. Considerable damage to the riprap protection downstream of each operating gate also occurred due to the poor energy dissipation and short length of apron.

13. With the stilling basin apron at el 12, the length of basin, size and location of baffle blocks, and size of end sill were varied to obtain satisfactory stilling basin performance. The recommended (type 13) design stilling basin (Plate 4) incorporated 20-ft-long by 8-ft-wide pier extensions and a 142-ft-long apron at el 12 with two rows of 9-ft-high and 8.5-ft-wide baffle piers spaced 8.5 ft apart and terminated with a 1V on 5H sloping end sill 7 ft high. The baffle piers located just downstream of each gate pier extension were omitted since model tests indicated that these baffle piers did not contribute to the overall performance of the stilling basin. The type 13 design stilling basin provided satisfactory hydraulic performance for the single-gate emergency operating conditions. The velocities over the end sill were reduced significantly from those measured with the type 1 (original) design stilling basin due to the reduction of flow returning to the basin as a result of the addition of the pier extensions. The 7-ft-high, 1V on 5H sloping end sill also appeared to help spread the flow, thereby reducing flow concentrations and formation of standing waves in the exit channel. Using zero energy loss between the upper pool and stilling basin, upper pool el 64, and tailwater el 40, the recommended basin has the following hydraulic characteristics with one gate open:

Gate Opening	Q, cfs	q cfs/ft	d_1, ft	V_1 ft/sec	d_2, ft	(Tailwater El - Apron El)/d_2
Fully open	40,000	667	13.38	49.90	39.30	0.71
Gate half open (18.0-ft gate opening)	28,300	471	8.97	52.70	35.10	0.80

Note: Q = total discharge
q = unit discharge
d_1 = initial depth before hydraulic jump
V_1 = velocity before hydraulic jump
d_2 = sequent depth after hydraulic jump
Apron el 12.0

Riprap stability

14. Stability of the riprap below the stilling basin was developed based on New Orleans District guidance given in paragraph 11. The recommended plan without hydropower facilities is the type 10 shown in Plate 5. The type 10 design riprap provided adequate riprap protection for the full range of open river and gated operating conditions. The most severe test of the stilling basin riprap is when a single gate is half or fully opened with normal upper pool and minimum tailwater. In the model investigation, the tailwater was held constant at the minimum. In the prototype, conditions will be less severe due to buildup of tailwater. The relationship between time required for gate opening and tailwater buildup determined analytically by the New Orleans District is shown in Plate 6.

15. Riprap stability downstream of the el 66 overflow weir was based on open river conditions only. The recommended plan is the type 10 shown in Plate 5.

16. The stability of riprap placed immediately upstream of the structure was based on a single gate fully open, normal upper pool, and minimum tailwater. A 66-in. blanket thickness failed for the single gate fully open. A 78-in. blanket thickness (36-in. D_{50}) remained stable and is included in the recommended type 10 riprap design shown in Plate 5.

17. Recommended riprap sizes adjacent to the upstream guard wall and upstream lock approach are also shown in Plate 5. These riprap gradations are from Engineer Technical Letter (ETL) 1110-2-120.*

Debris passage tests

18. Tests were conducted to determine the tailwater and gate opening combination required for passage of floating debris under the tainter gates. The debris size simulated in the model was 1 ft in diameter by 10 ft in length. Test results are shown in Plate 7.

Gate submergence tests

19. Tests were conducted to determine the gate opening at which the tailwater is swept away from the lip of the gate. These tests were conducted for normal upper pool and all gates opened an equal amount. Results show that the gate lip remains submerged for even the minimum tailwater possible in the

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* Headquarters, US Army Corps of Engineers. 1971 (14 May). "Additional Guidance for Riprap Channel Protection," Change 1, ETL 1110-2-120, US Government Printing Office, Washington, DC.

model. This minimum tailwater was below the predicted tailwater given in Plate 3. The amount of gate lip submergence and the corresponding minimum tailwater are given in Plate 8.

Water-surface profiles and discharge rating

20. The water-surface profile measured downstream of gate 1, adjacent to the navigation lock, is given in Plate 9. The profile was measured along the center line of the bay with the gate half open, normal upper pool el 64, and minimum lower pool el 40.

21. Water-level differences ΔUD, commonly referred to as swellhead, were measured between the upstream end of the upstream guard wall and the downstream end of the downstream guide wall. These differences were used to evaluate the impact of various modifications to the project. The measured differences for uncontrolled flows based on the project without hydropower are as follows:

Q, cfs	Headwater El (sta 9+25)	Tailwater El (sta 15+00)	ΔUD
100,000	66.1	64.5	1.6
150,000	73.7	72.0	1.7
200,000	79.7	77.5	2.2
256,000	86.2	83.5	2.7

These water-level differences are not representative of the as-built prototype because of the effects of the sedimentation dikes. Because overbank areas were not accurately modeled above el 75, swellhead for headwater and tailwater elevations above 75 may not be accurate (model values are probably higher than prototype because of the limited flow area available in the model).

22. Discharge rating curves will be discussed in a later paragraph and will include the effects of the downstream sedimentation dikes.

Velocities and flow patterns

23. Velocities were measured above the stilling basin floor and the end sill with the recommended type 13 design stilling basin. Results are shown in Plate 10 for normal upper pool, minimum tailwater, and an 18-ft gate opening. A comparison of velocities over the end sill for gate bays 1 and 3 is shown in Plate 11. Flow patterns with the recommended design (type 13 stilling basin and type 10 riprap) are shown in Photos 1 and 2.

Project with Hydropower and Without Downstream Sediment Dikes

24. The second portion of this study was the development of a project plan that provided hydropower facilities with satisfactory approach and exit flow conditions. As in the initial portion of the study without hydropower, the sedimentation problem had not yet occurred at Red River Lock and Dam No. 1 prototype and the downstream sediment dikes were not a part of this portion of the study.

Cofferdam studies

25. Tests were conducted to evaluate several cofferdam schemes that were proposed for use in construction of the hydropower facilities. The type 1 design cofferdam is shown in Plate 12 and Figure 4. Types 3 and 4 design cofferdams are shown in Plate 13. Riprap stability tests were conducted with the type 1 design cofferdam. Results indicate that the type 10 design riprap with 36-in.-thick riprap around the cofferdam will remain stable for the full range of open river conditions, including those discharges overtopping the cofferdam. However, some riprap movement at the base of the cofferdam occurred when overtopping of the cofferdam occurred with gated

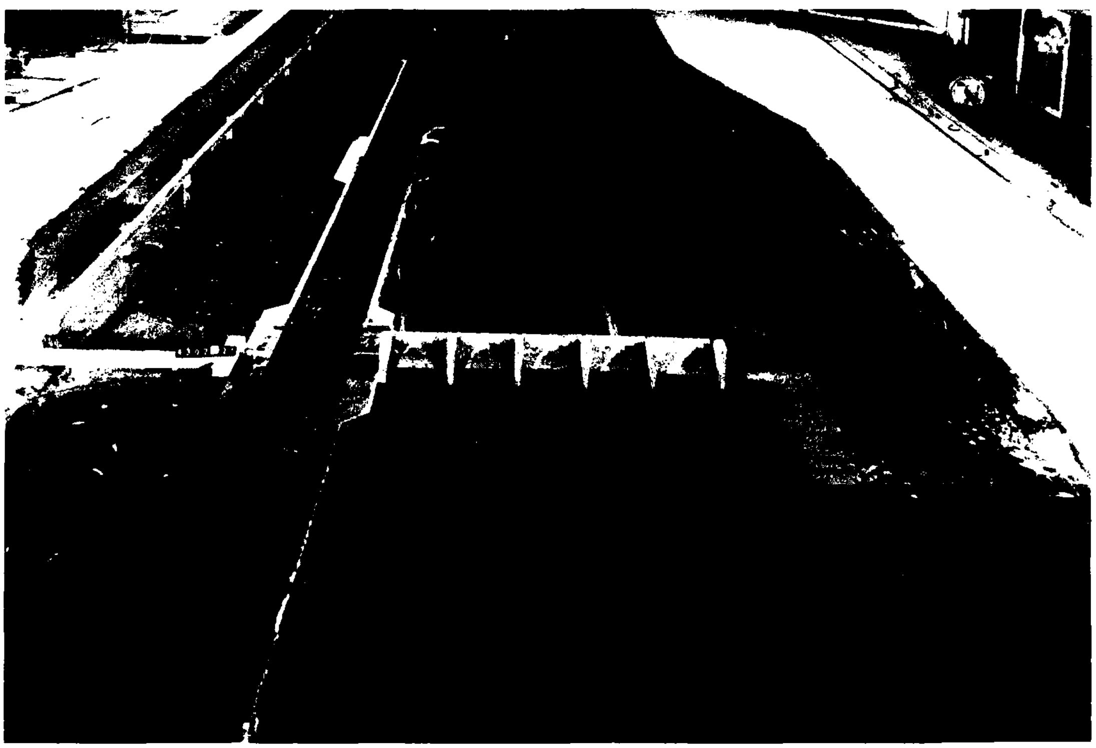

Figure 4. Type 1 hydropower cofferdam

operating conditions. Additional stability tests with this operating condition were not conducted because operation of the project with the cofferdam in place would be with fully opened gates to minimize the occurrence of overtopping of the cofferdam.

26. Water-level differences were determined for several of the cofferdam schemes for comparison with water-level differences determined without the cofferdam. Results were as follows:

Cofferdam Type	Q cfs	Headwater El	Tailwater El	ΔUD ft
1	100,000	66.1	64.6	1.5
	150,000	73.7	72.0	1.7
	200,000	80.0	77.8	2.2
3,4	100,000	65.9	64.6	1.3
	150,000	73.6	72.0	1.6
	200,000	79.8	77.8	2.0
4	256,000	85.8	83.5	2.3

Measured ΔUD was not significantly different from values measured without the cofferdam (paragraph 21), and differences between the two do not represent any definite trends. As stated before, these values are not representative of the as-built prototype because of the absence of the downstream sediment dikes in this portion of the study.

Hydropower approach and exit flow conditions

27. The type 1 hydropower design, shown in Plate 14, has a nonovertopping four-unit powerhouse with each unit having a maximum capacity of 6,000 cfs. Surface flow patterns shown in Photo 3 indicate unsatisfactory approach flow conditions because flow lines indicated by the confetti streaks are not normal to the face of the powerhouse.

28. In the type 2 hydropower design, the approach to the powerhouse was changed as shown in Plate 15. Surface flow patterns as indicated by confetti streaks in Photo 4 show an eddy in front of the structure and flow across the face of the powerhouse, both of which are undesirable. The eddy increases the severity of the vortices at the inlets to the hydropower unit.

29. In the type 3 hydropower design, a berm separating the power plant and the spillway was placed in the model (Plate 16) similar to the berm used in the type 1 hydropower design. The type 3 also included a change from the four-unit powerhouse to a three-unit powerhouse with each unit having a

maximum discharge of 8,000 cfs. Approach flow conditions with the type 3 design were not satisfactory.

30. In the type 4 hydropower design, the berm was removed as shown in Plate 17. Flow conditions approaching the type 4 hydropower design were improved but vortices still formed just above the intakes to the powerhouse. The 78-in. riprap placed downstream of the outlets in the type 4 design failed when the powerhouse was operated with the maximum head differential of 16 ft. The displacement consistently occurred just downstream of the outlets at the beginning of the tailrace.

31. In the type 5 hydropower design, a sloping lip was added to the upstream face of the powerhouse as shown in Plate 18 and Figure 5. This modification minimized vortex formation above the hydropower intakes. Flow conditions approaching the type 5 hydropower design are shown in Photo 5. A 20-ft horizontal concrete apron was added to the downstream end of the powerhouse as shown in Plate 19. This apron eliminated the riprap failures that occurred in the type 4 design and allowed a reduction in riprap size from 72-in. to 60-in. blanket thickness. The gradation tested in the model is also shown in Plate 19.

Effect of increased
upper pool on riprap stability

32. Upper pool elevations 2 ft higher than the normal upper pool el of 64 are being considered to increase the amount of power generation. This increased pool raised questions about the performance of the stilling basin and the integrity of the downstream riprap. Testing of a single gate fully open and minimum tailwater resulted in significant displacement of the downstream riprap. The hydraulic jump was no longer maintained in the basin.

33. When operated with a single gate half opened (18 ft), the stilling basin performed satisfactorily and the riprap remained stable when subjected to approximately 30 hr (prototype) of operation.

Project Without Hydropower and With Downstream Sediment Dikes

34. After the first two portions of this investigation were completed, prototype sedimentation problems were experienced at the lower lock approach to Red River Lock and Dam No. 1 prototype. Testing of a movable-bed sedimentation model was conducted to determine a project plan that would prevent

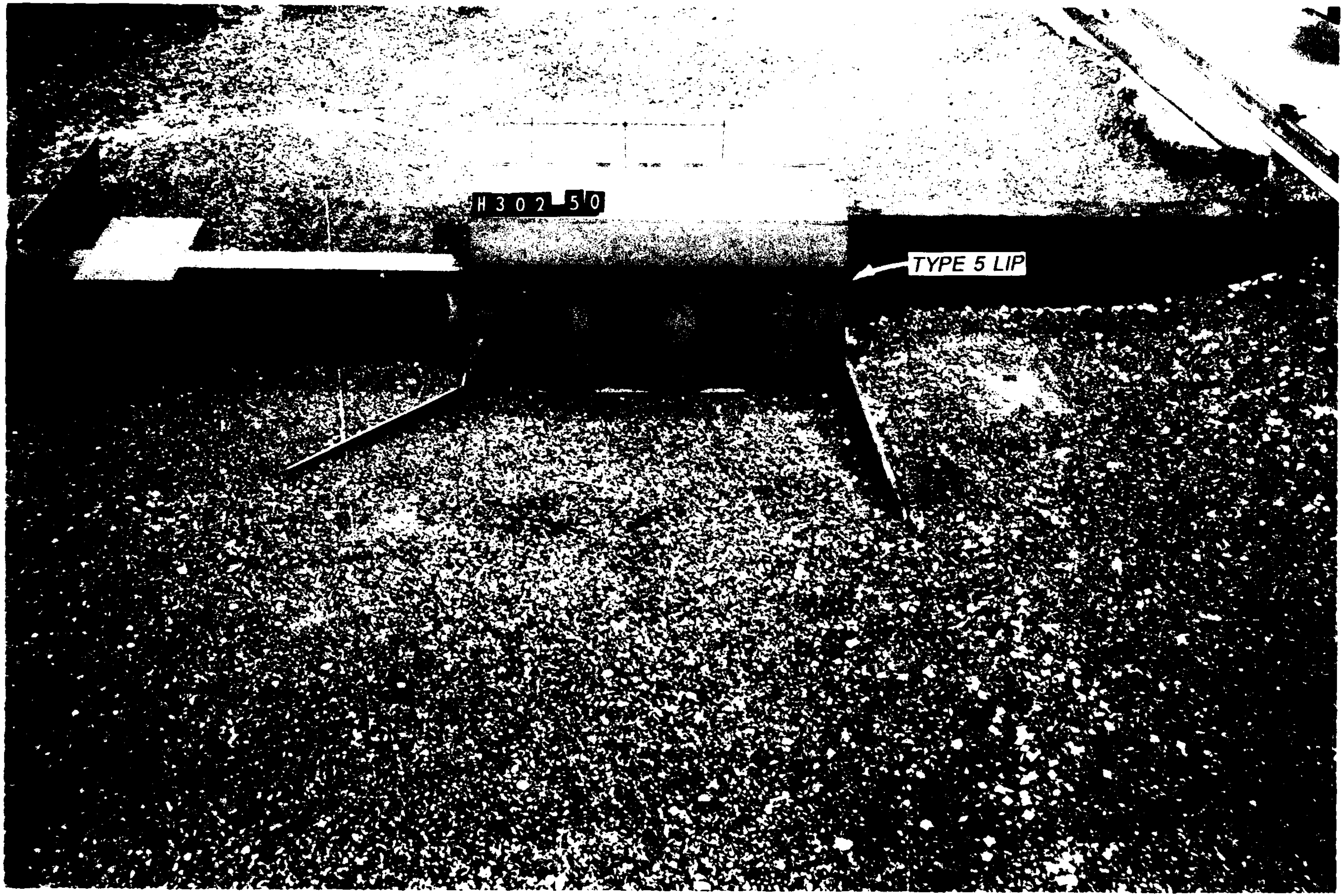

Figure 5. Type 5 hydropower design, dry bed

similar sedimentation problems at the lower lock approach of JHO.* The resulting recommended plan was designated C-81 and is shown in Plate 20. The following components are included:

- a. Angled, longitudinal stone dike, top el 74.5, attached at riverside lock wall by a retaining wall.
- b. Wing dike parallel to navigation channel at el 50 at the connection with the longitudinal stone dike.
- c. Three repelling dikes along the right bank side of the spillway outlet channel.
- d. Carefully positioned right bank alignment from the spillway to the existing Red River Channel.
- e. Carefully positioned left bank alignment from the spillway axis extended to the existing Red River Channel.
- f. Riprap pavement along the channel bed from the spillway through the zone of most intense turbulence.
- g. Three spur dikes on the left bank berm from the end of the guide wall through the zone of eddy formation.
- h. Outlet channel cross-sectional shape and size.

35. Several features of Plan C-81 required testing in the physical model. However, the spillway and hydropower model of JHO had been dismantled and replaced by the spillway model of Red River Lock and Dam No. 3. Plan C-81 was modeled by modifying the 1:50-scale spillway model of Lock and Dam No. 3.** The five-gated JHO structure was simulated by closing off one of the six gates proposed for the Lock and Dam No. 3 structure. The overflow weir length of 250 ft was placed in the model and the upstream ported guard wall was placed adjacent to the left tainter gate. Lock and dam wall elevations and overbank areas were correctly modeled up to the elevation corresponding to a discharge of 145,000 cfs. The "new" JHO spillway model is shown in Figure 6.

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* J. L. MacGregor and C. W. O'Neal. "Red River Waterway, John H. Overton Lock and Dam, Sedimentation Conditions; Hydraulic Model Investigation" (in preparation), Report 3, Technical Report HL-89-16, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

** S. T. Maynord. "Red River Waterway, Lock and Dam No. 3, Stilling Basin and Riprap Requirements; Spillway Hydraulic Model Investigation" (in preparation), Report 4, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Figure 6. 1:50-scale Red River Lock and Dam No. 3 model modified to simulate JHO Plan C-81

Flow distribution

36. Flow distribution for open river or uncontrolled flow conditions through the five-gated structure was determined for both the Lock and Dam No. 3 configuration, in which the upstream ported guard wall is 116 ft from the gated structure, and the JHO configuration, in which the upstream ported guard wall is adjacent to the left tainter gate. Results were as follows:

Bay (Left to Right)	Proportion of Total Flow Through Gated Structure for Each Bay: Q = 145,000 cfs	Q = 90,000 cfs
Lock and Dam No. 3 Configuration		
1	0.157	0.157
2	0.213	0.218
3	0.218	0.217
4	0.216	0.221
5	0.196	0.187
JHO Configuration		
1	0.168	0.172
2	0.206	0.206
3	0.221	0.219
4	0.209	0.211
5	0.196	0.192

Results show a decrease in flow through the outside bays (1 and 5) for both guard wall configurations. Moving the upstream guard wall 116 ft away from the left tainter gate caused a small reduction in flow through bay 1. Both tests had the downstream lock wall located adjacent to the stilling basin. as shown in Figure 6.

Water-surface profiles

37. Water-surface profiles were measured in the exit channel for discharges of 30,000, 90,000, and 145,000 cfs. Results (Plate 21) show a relatively steep water surface in the vicinity of the three repelling dikes. Measurements of water-surface elevations were taken at points A, B, and C shown in Plate 22 for the same three discharges. The discharge outlets for the lock are at points A and C. Results (also given in Plate 22) show a differential of up to 1.5 ft. The differentials are important in evaluating potential lock operational problems. Large differentials can cause operational difficulties during lock emptying.

Velocities

38. Velocities were taken to assist in evaluating the stability of the riprap in the exit channel for discharges of 90,000 and 145,000 cfs and are shown in Plates 23 and 24, respectively. These velocities were taken at a distance of 0.6 times the depth of flow below the water surface.

Riprap stability and bottom scour

39. Riprap stability tests were conducted to evaluate Plan C-81 for normal and emergency operation. For normal operation (uniform gate openings), the proposed riprap sizes (Plate 25) were stable except for the nonovertopping dike just downstream of the retaining wall. Flows of 30,000, 90,000, and 145,000 cfs moved the riprap on the right side just downstream of the retaining wall. Stable conditions were achieved when the riprap size in this area was increased to a 48-in. blanket thickness for a distance of 50 ft. At a discharge of about 175,000 cfs, flow overtopped the retaining wall and moved some of the riprap on the top and left side of the nonovertopping dike. This required increasing the riprap size in this area to a 48-in. blanket thickness.

40. For riprap stability tests under emergency gate operation, a single gate was fully opened with normal upper pool and minimum tailwater conditions. Minimum tailwater in these tests was the lower normal pool el of 40. Each gate was tested for an emergency operation with the following results:

a. With gate 1 open, riprap (B stone) was removed from the right side of the nonovertopping dike just downstream of the retaining wall.

b. With gates 2 and 3 open, only minor rock movement was produced on the right side of the nonovertopping dike.

c. With gates 4 and 5 open, riprap was moved off the top and nose of the upstream repelling dike.

In all five cases, the amount of rock moved was rather small. The proposed plan actually increased the stability of the riprap just downstream of the stilling basin because the repelling dikes, retaining wall, and nonovertopping dike increased tailwater in the exit channel.

41. The model indicated significant channel bottom scour just downstream of the end of the riprap adjacent to the right bank. The velocities presented in Plates 23 and 24 show concentrated flow along the right bank. Bottom profiles are not presented because although the 1:50-scale model is an excellent indicator for the area to be scoured, it cannot be used to predict actual prototype scour depths, which will be greater than those indicated by the model.

Sedimentation studies

42. The potential for sediment deposition in the corner of the lock wall and retaining wall was addressed by using plastic beads and coal to simulate sediment movement in the model. Based on a comparison of fall velocity, the plastic beads were equivalent to a prototype quartz particle diameter of 3-4 mm. The coal was equivalent to a prototype diameter of roughly 7-8 mm. Initially coal and plastic were introduced upstream of the corner area for a discharge of 145,000 cfs. The coal moved along the bed and bypassed the corner area. The plastic moved predominantly as bed load with some suspended load. The plastic moving as bed load also bypassed the corner area. The plastic moving as suspended load entered but quickly passed through the corner area. Next, coal and plastic were dumped into the corner area to see if the flow (145,000 cfs) was capable of moving the material out of the corner. After an extended run, most of the coal and all of the plastic were removed from the corner area. Coal and plastic were again placed in the corner and tested with a flow rate of 90,000 cfs. The coal would not move out of the corner, but the plastic was removed from the corner. Plastic was again placed in the corner and tested at a flow rate of 30,000 cfs. The plastic did not move out of the corner, but the flow was capable of causing the particles to

move around on top of the scour slab. When a lighter weight plastic representing a prototype size of approximately 2 mm was placed in the corner, the 30,000-cfs discharge moved the plastic off the scour slab except for the particles located within 10 ft of the lock wall. Considering the relatively large particles represented by the plastic compared to the prototype sand (0.06 mm), deposition in the corner should not be a significant problem.

Effects of downstream sediment dikes on project features developed without dikes

43. The addition of the five dikes (items a, b, and c in paragraph 34) to the exit channel of JHO increased flow resistance, which increased tailwater at the structure. This was a local increase that occurred only between the structure and the downstream end of the wing dike. The amount of tailwater increase was determined by comparison of with- and without-dike water-surface profiles. Water-surface slopes without the dikes were mild. The tailwater increase was as follows:

Q, cfs	Tailwater El	Without Dike El (sta 3+00)-El (sta 20+00)	With Dike El (sta 3+00)-El (sta 20+00)	Tailwater Increase ft
30,000	49.9	0.1	1.1	1.0
90,000	63.1	0.1	1.0	0.9
145,000	71.6	0.2	1.4	1.2

The without-dike elevation differences were based on a previous study of JHO, which had six tainter gates instead of five. The with-dike elevation differences were based on Plate 21. These results show that tailwater can be expected to increase approximately 1 ft for flows up to 145,000 cfs. Above this discharge, overbank flows occur and the influence of the dikes will probably be less significant. This could not be tested in the model because the overbank areas were not simulated.

44. The increase in tailwater had a positive influence on riprap stability downstream of the structure for the conditions tested in the model. The higher tailwater generally improved basin performance and reduced average velocity, both of which increased riprap stability.

45. The increased tailwater had a negative influence on the water-level difference (swellhead) caused by the structure. For example, in paragraph 21, the following conditions existed for the five-gated structure without Plan C-81: discharge 100,000 cfs; tailwater el 64.6 at sta 3+00 and 64.5 at

sta 15+00; headwater el 66 1; and uncontrolled flow. Using the d'Aubuisson equation for estimating ΔUD

$$Q = KLh \sqrt{[2g(\Delta UD) + V^2]}$$

where

K = d'Aubuisson coefficient

L = net length = 5(60) = 300 ft

h = tailwater el − sill el = 64.6 − 28 = 36.6

g = gravitational acceleration

ΔUD = 1.5 ft between headwater and sta 3+00

Approach depth = 66.1 − 23 = 43.1 ft

Approach width = 500 ft

V = approach velocity = 100,000/[(500)(43.1)] = 4.64 ft/sec

Solve for d'Aubuisson coefficient

$$K = \frac{Q}{Lh\sqrt{2g(\Delta UD) + V^2}} = \frac{100{,}000}{(300)(36.6)\sqrt{64.4(1.5) + (4.64)^2}} = 0.84$$

Use this d'Aubuisson coefficient K to determine the influence of a 1-ft tailwater rise for a discharge of 100,000 cfs that results from the addition of Plan C-81 to the five-gated structure:

Exit Channel Configuration for Five-Gated Structure	Headwater El	Tailwater El at sta 3+00	Tailwater El Downstream of Structure Influence
Without dikes	66.1	64.6	64.5
With dikes	66.9	65.5	64.5

The result is that the addition of the downstream sediment dikes causes an 0.8-ft increase in water-level difference for the five-gated structure. Computed water-level differences instead of model measurements were used in this comparison because the "new" JHO model did not reproduce all pertinent features of the prototype that would influence water-level differences.

46. Discharge ratings for use in operation of JHO should be developed using guidance in Engineer Manual (EM) 1110-2-1605.* These ratings should be dependent on the location of the tailwater measuring gage and the tailwater changes discussed in paragraph 43.

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* Headquarters, US Army Corps of Engineers. '987 (12 May). "Hydraulic Design of Navigation Dams," EM 1110-2-1605, US Government Printing Office, Washington, DC.

PART IV: DISCUSSION OF RESULTS AND CONCLUSIONS

47. Tests with the original design stilling basin showed that a longer basin was required to meet the stilling basin performance requirements established by the sponsor. The recommended type 13 stilling basin design provides satisfactory energy dissipation for normal flows and for the single-gate emergency operating conditions.

48. Satisfactory riprap plans were developed for the upstream and downstream areas adjacent to the structure for both normal flows and the single-gate emergency operating conditions. Stable riprap plans were also developed for the overflow weir and the upstream guard wall.

49. Debris passage curves were developed to determine the tailwater and gate opening combinations required to pass floating debris under the tainter gates.

50. Water-level differences and riprap stability tests were conducted for several cofferdam schemes that are proposed for construction of the powerhouse.

51. The original hydropower design resulted in unsatisfactory approach flow conditions. The type 5 hydropower design resulted in a significant improvement in approach flow conditions and reduced the occurrence of vortices at the powerhouse intakes.

52. A 2-ft increase in upper pool elevation to increase power generation resulted in significant riprap failure for the single gate fully opened and minimum tailwater condition. The riprap remained stable for the 2-ft pool increase, gate opening of 18 ft, and minimum tailwater.

53. The various elements of Plan C-81 locally increased tailwater at the structure by about 1 ft for discharges ranging from 30,000 to 145,000 cfs. This was based on a comparison of water levels at the dam and at a point just downstream of the dikes used in Plan C-81. This increased tailwater increased the stability of the stilling basin riprap by reducing velocities and improving stilling basin performance. The increased tailwater increased the water-level differences (swellhead) caused by the project.

54. A stable riprap plan for the various elements of Plan C-81 was developed for normal operating and single-gate emergency conditions.

55. Sedimentation studies in the 1:50-scale model were conducted using crushed coal and lightweight plastic beads. These qualitative studies

indicated that the corner area between the lock wall and the retaining wall will remain free of significant sediment deposition.

56. Discharge ratings for JHO should be developed with consideration given to the location of tailwater measurement.

Photo 1. Surface flow patterns for gate 1, one-half opened, headwater el 64, tailwater el 40, all other gates closed; type 13 stilling basin and type 10 riprap

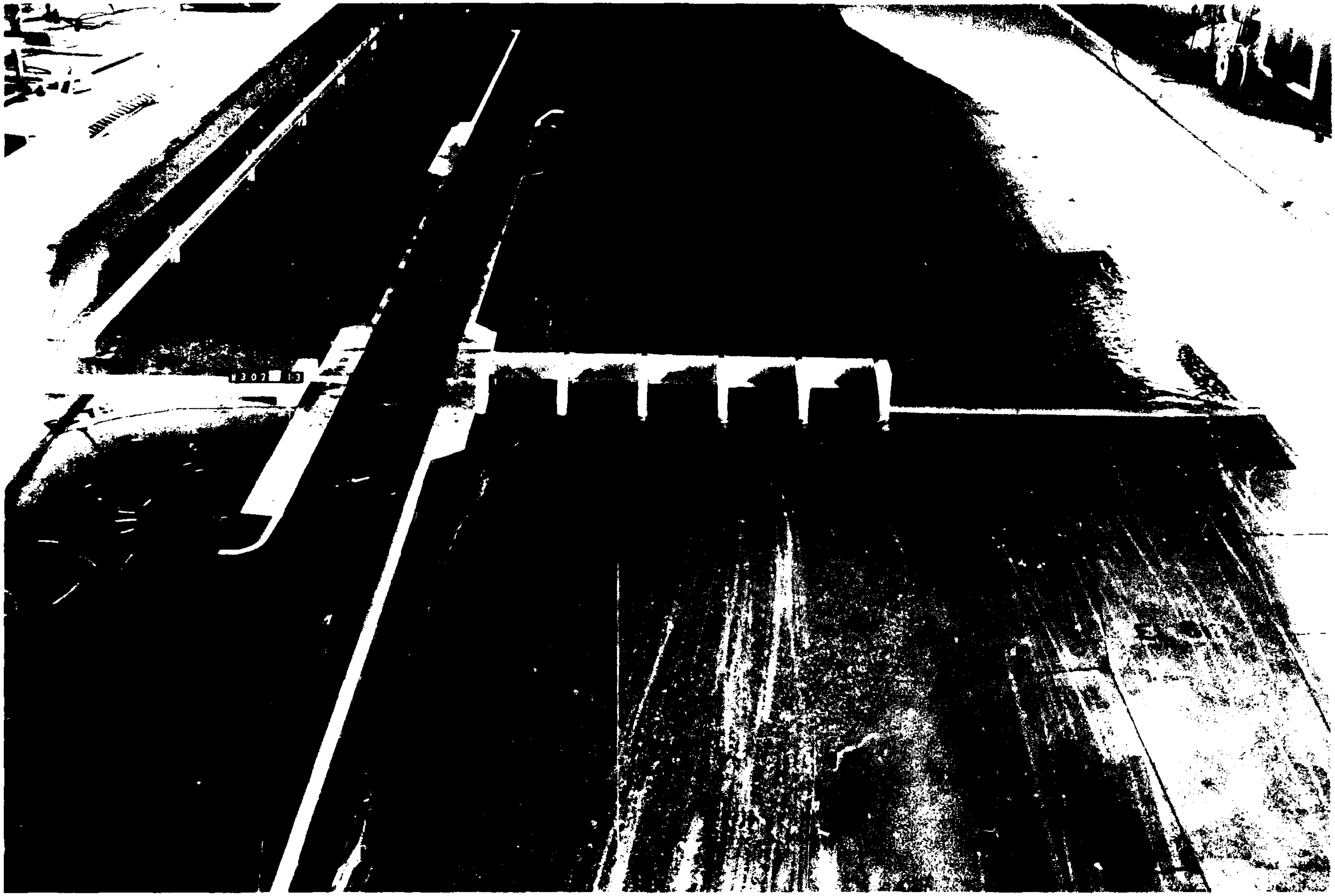

Photo 2. Surface flow patterns for open river conditions, discharge 150,000 cfs, tailwater el 72, all gates fully opened

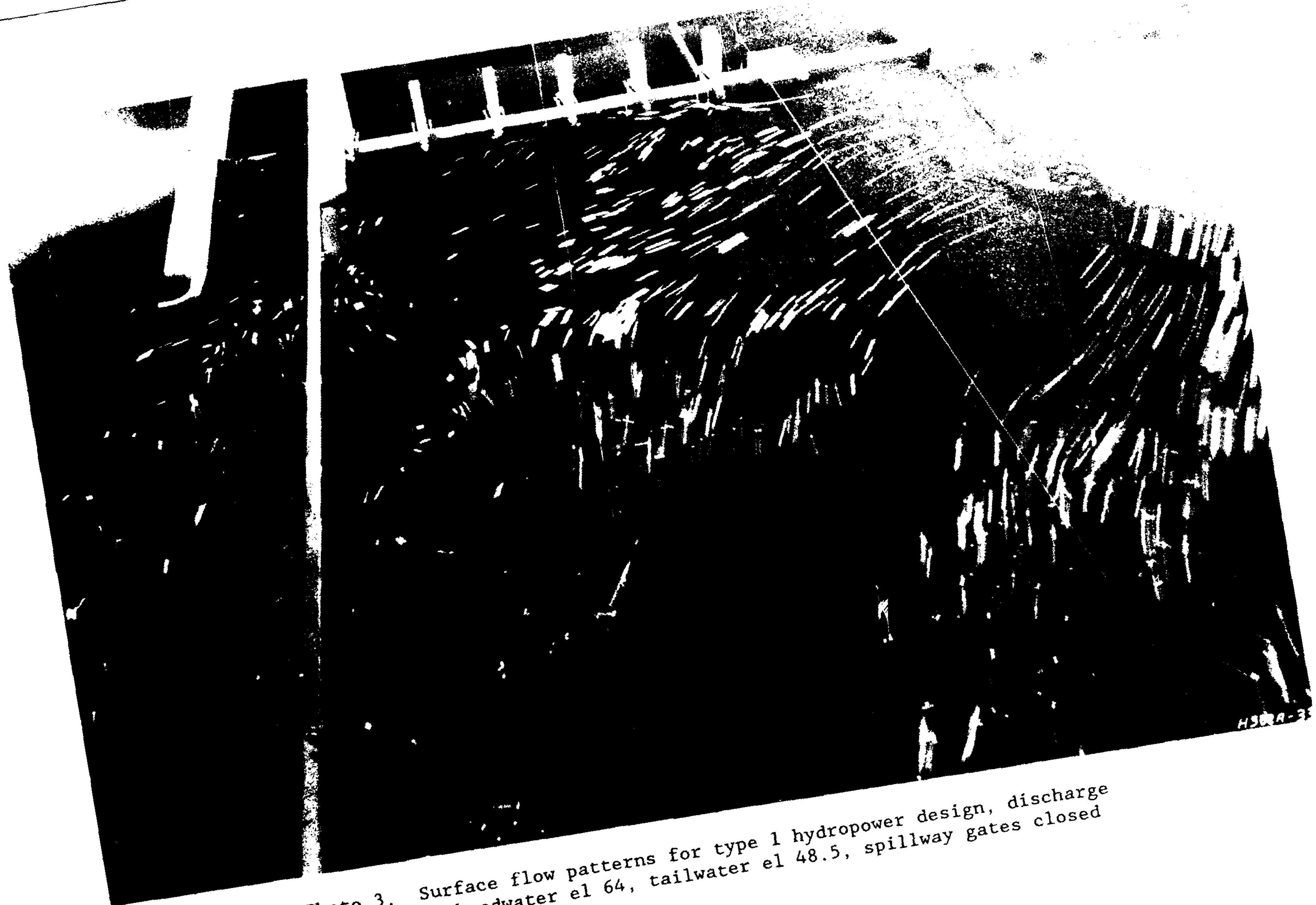

Photo 3. Surface flow patterns for type 1 hydropower design, discharge 24,000 cfs, headwater el 64, tailwater el 48.5, spillway gates closed

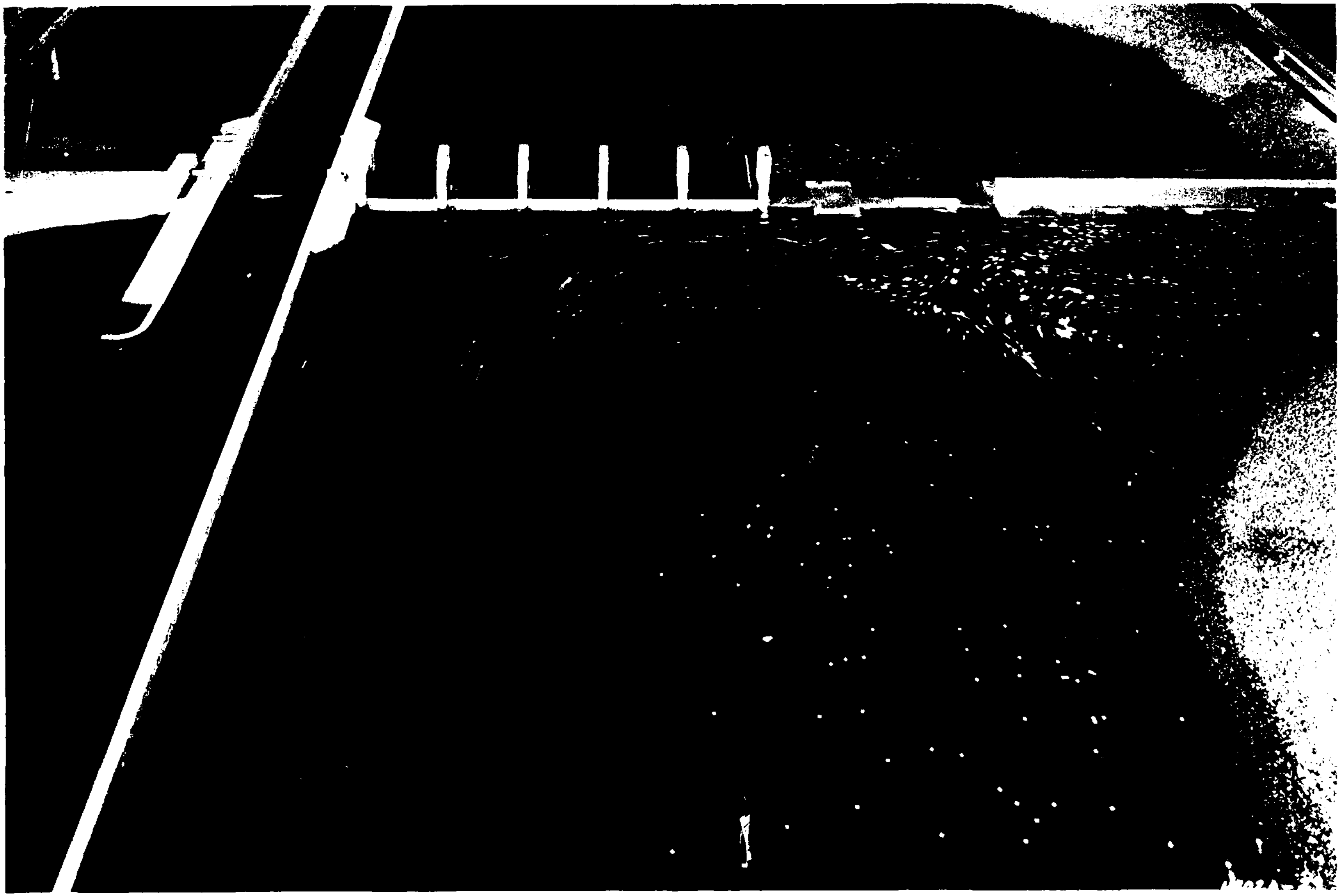

Photo 4. Surface flow patterns for type 2 hydropower design, discharge 24,000 cfs, headwater el 64, tailwater el 48.5, spillway gates closed

46. Discharge ratings for use in operation of JHO should be developed using guidance in Engineer Manual (EM) 1110-2-1605.* These ratings should be dependent on the location of the tailwater measuring gage and the tailwater changes discussed in paragraph 43.

* Headquarters, US Army Corps of Engineers. 1987 (12 May). "Hydraulic Design of Navigation Dams," EM 1110-2-1605, US Government Printing Office, Washington, DC.

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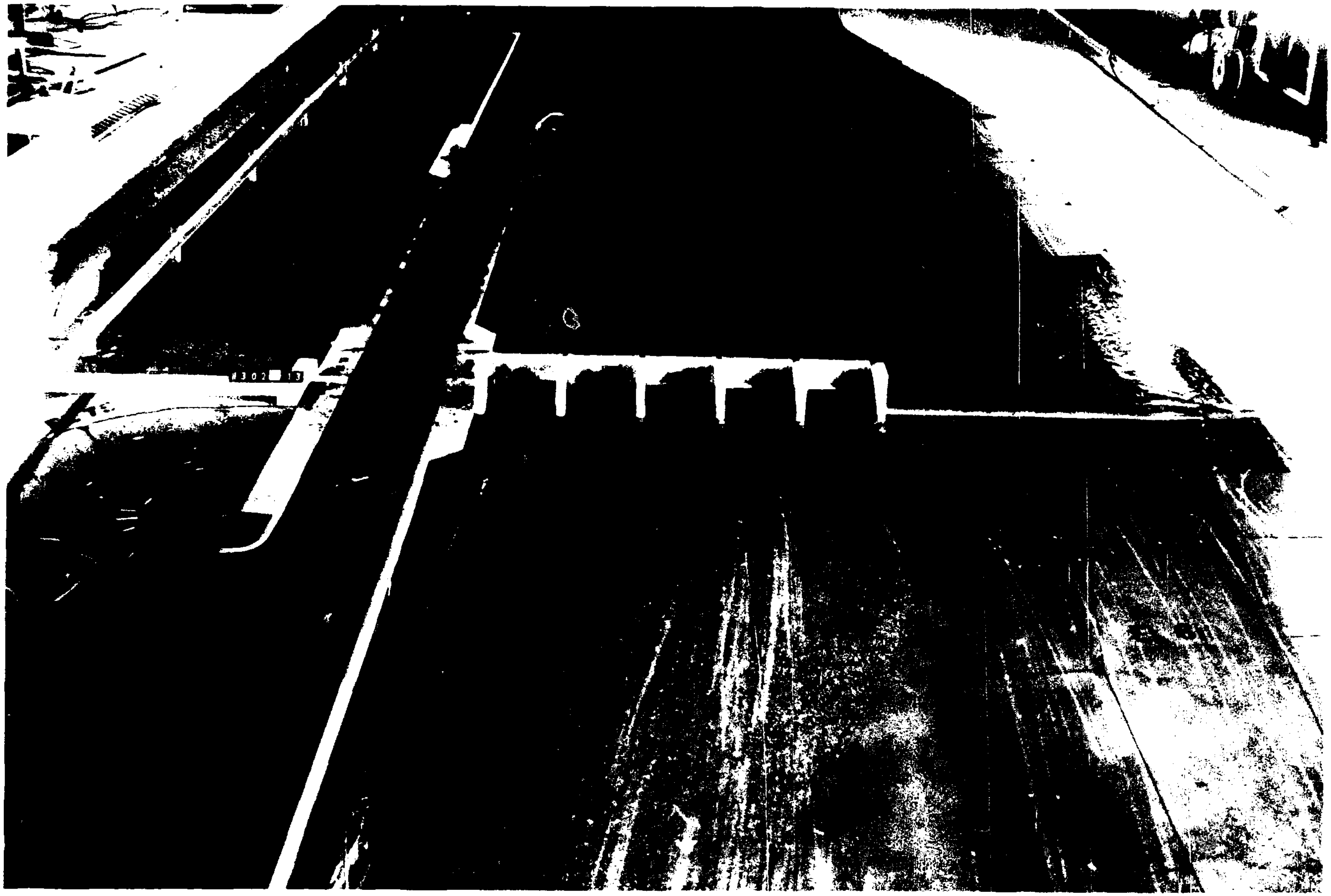


Photo 2. Surface flow patterns for open river conditions, discharge 150,000 cfs, tailwater el 72, all gates fully opened

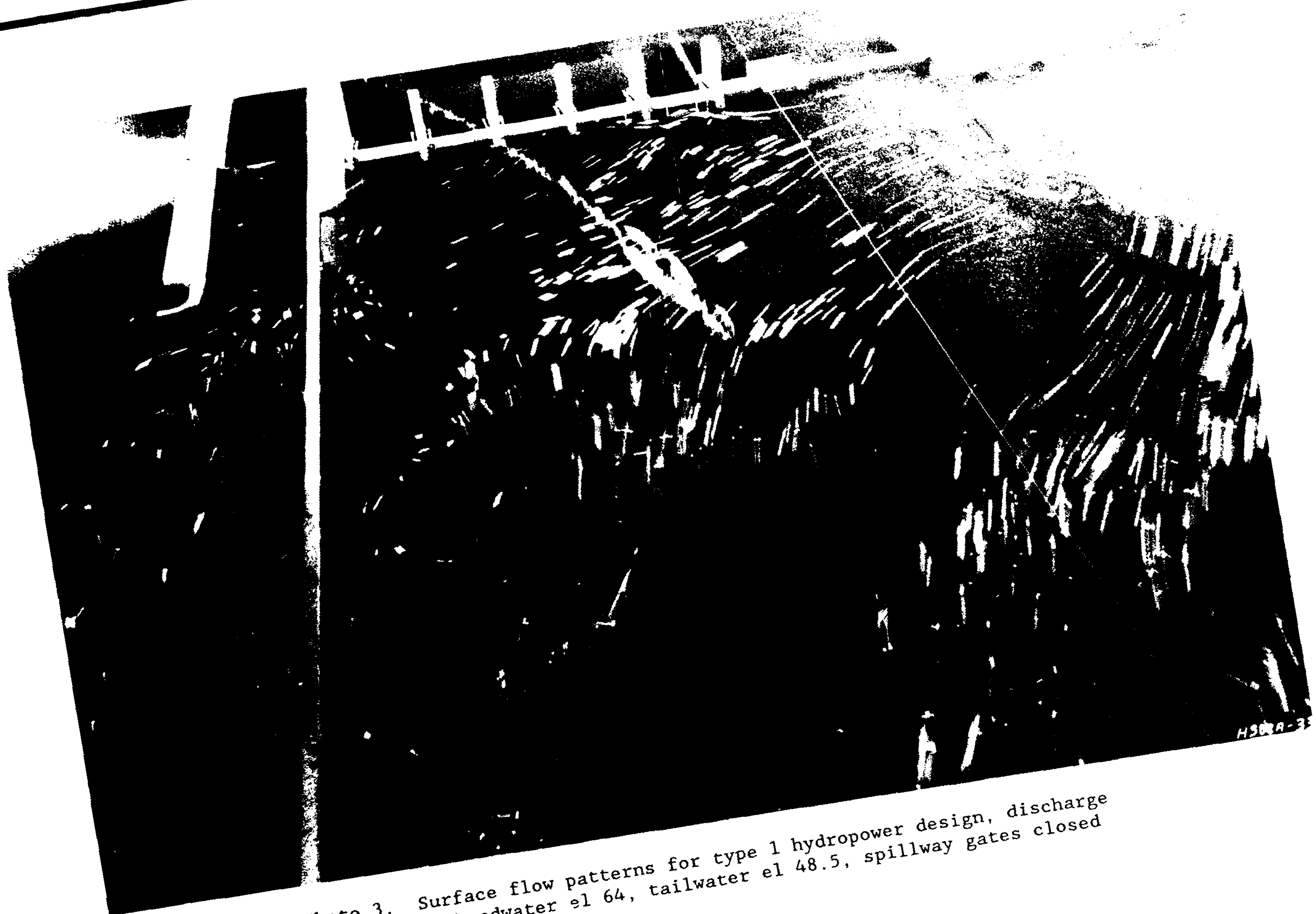


Photo 3. Surface flow patterns for type 1 hydropower design, discharge 24,000 cfs, headwater el 64, tailwater el 48.5, spillway gates closed

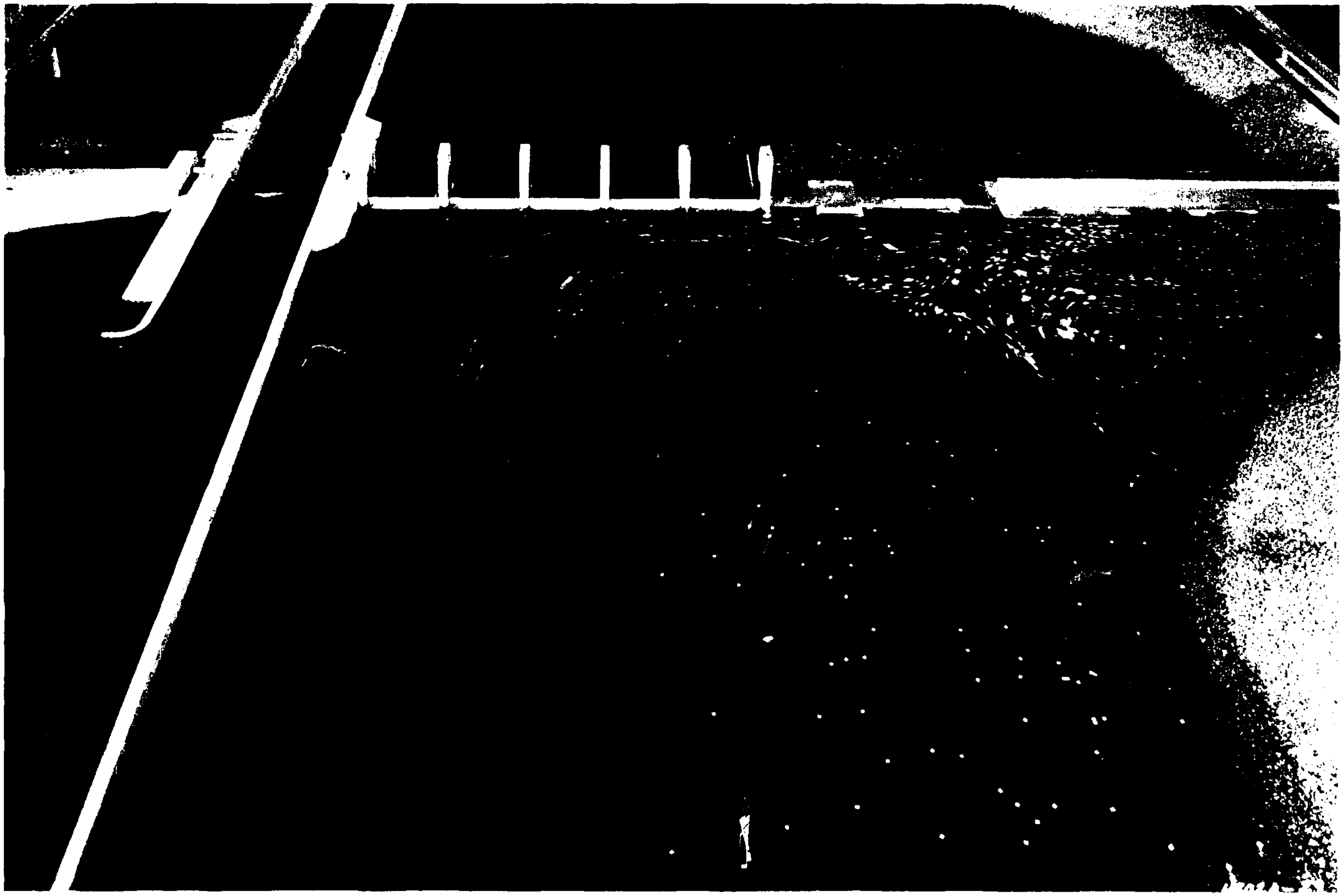
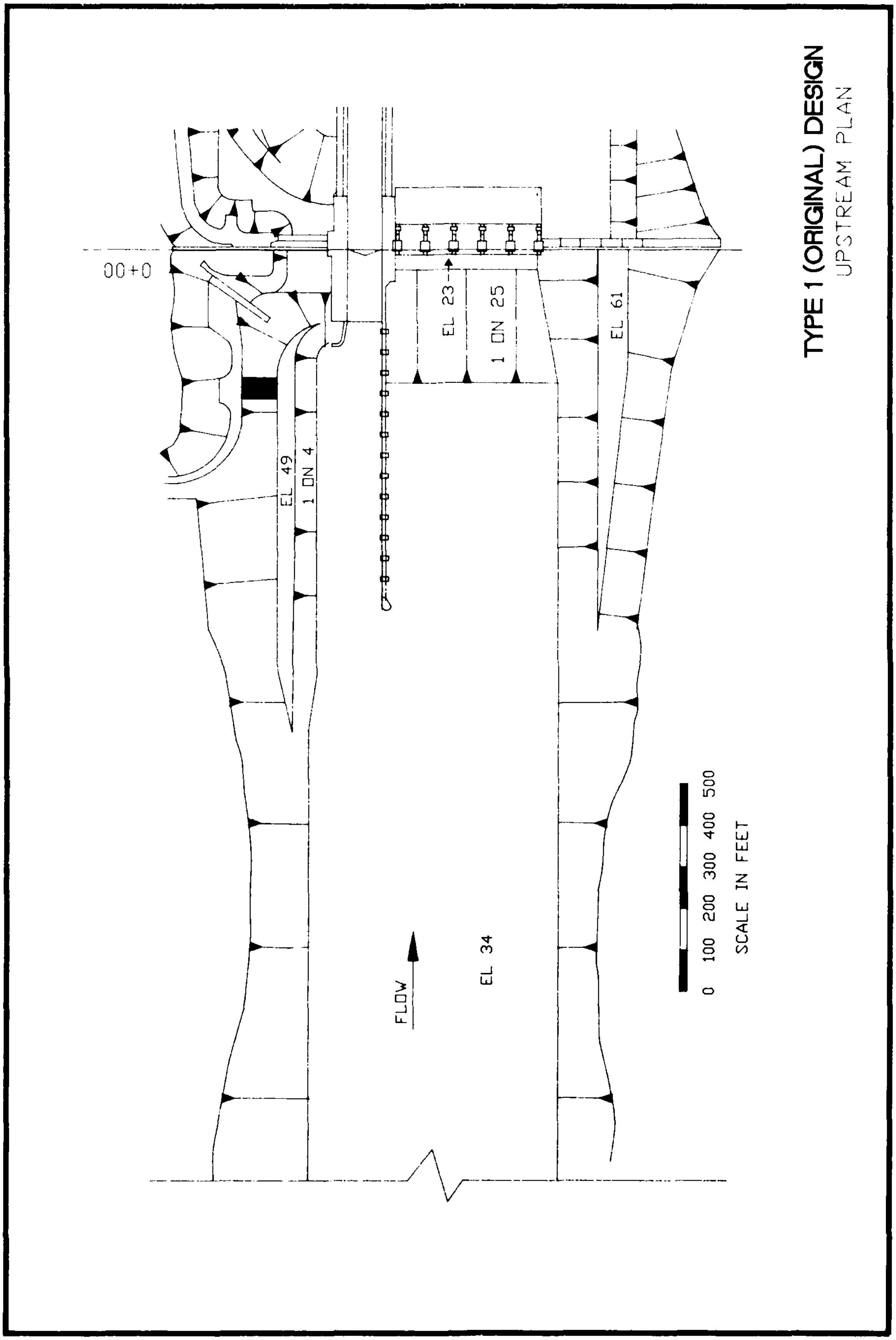


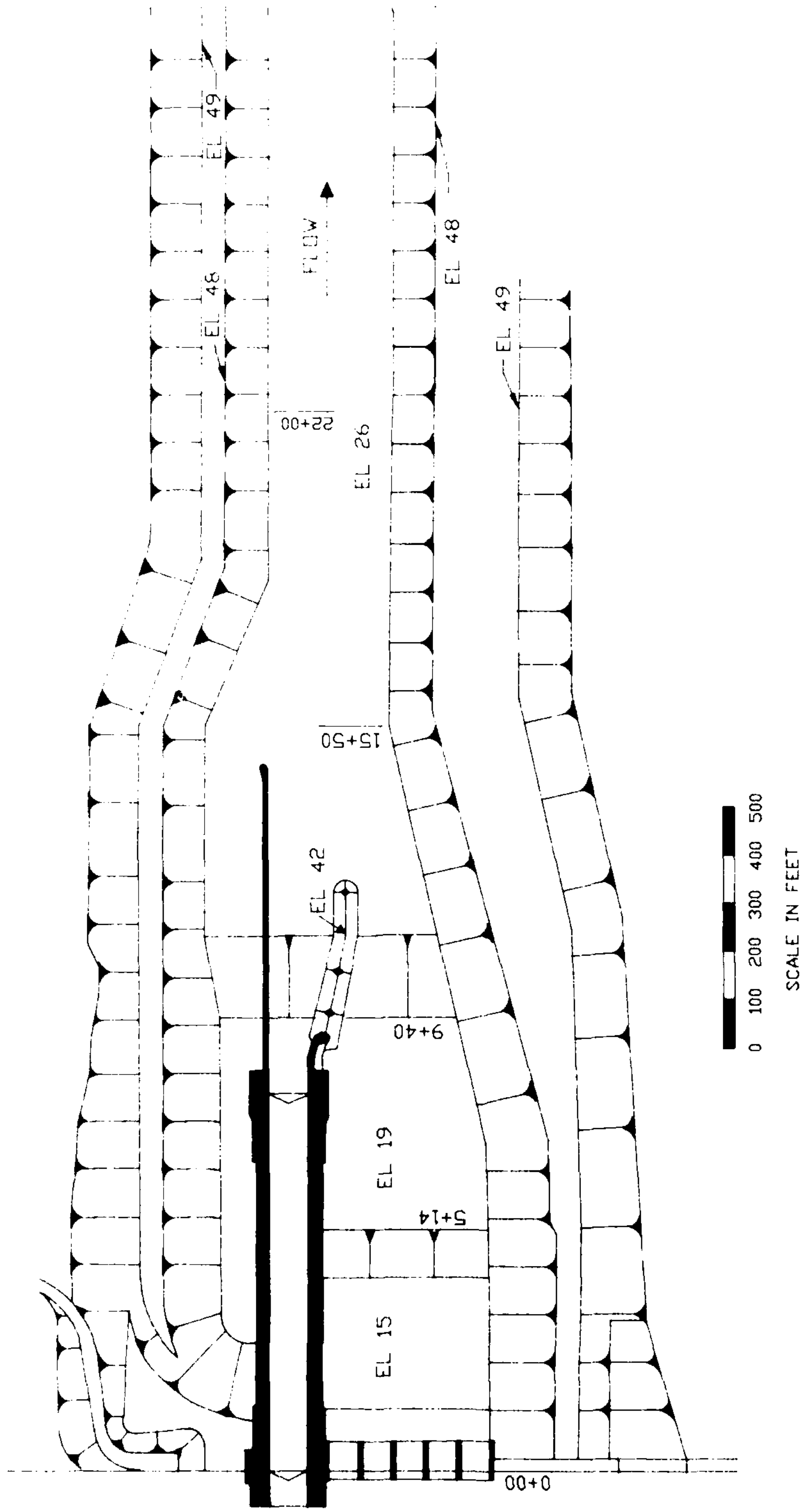
Photo 4. Surface flow patterns for type 2 hydropower design, discharge 24,000 cfs, headwater el 64, tailwater el 48.5, spillway gates closed

Photo 5. Type 5 hydropower design, discharge 24,000 cfs



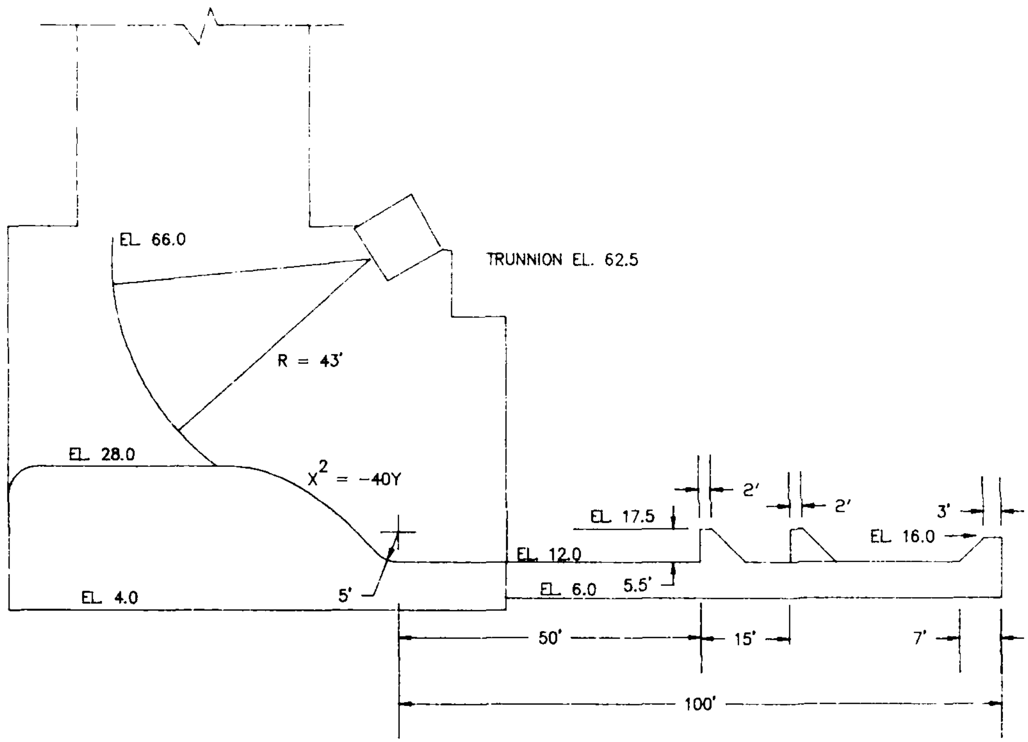
TYPE 1 (ORIGINAL) DESIGN
UPSTREAM PLAN

PLATE 1
(SHEET 1 OF 2)

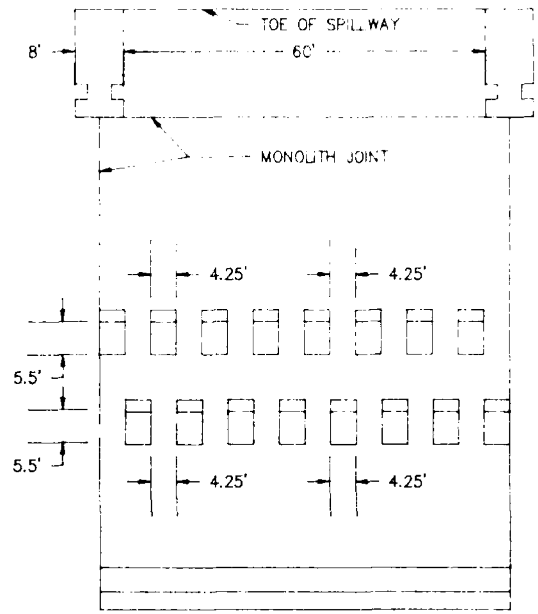


TYPE 1 (ORIGINAL) DESIGN
DOWNSTREAM PLAN

PLATE 1
(SHEET 2 OF 2)



ELEVATION VIEW - SPILLWAY AND STILLING BASIN



PLAN VIEW - STILLING BASIN

TYPE 1 (ORIGINAL) DESIGN
SPILLWAY AND STILLING BASIN

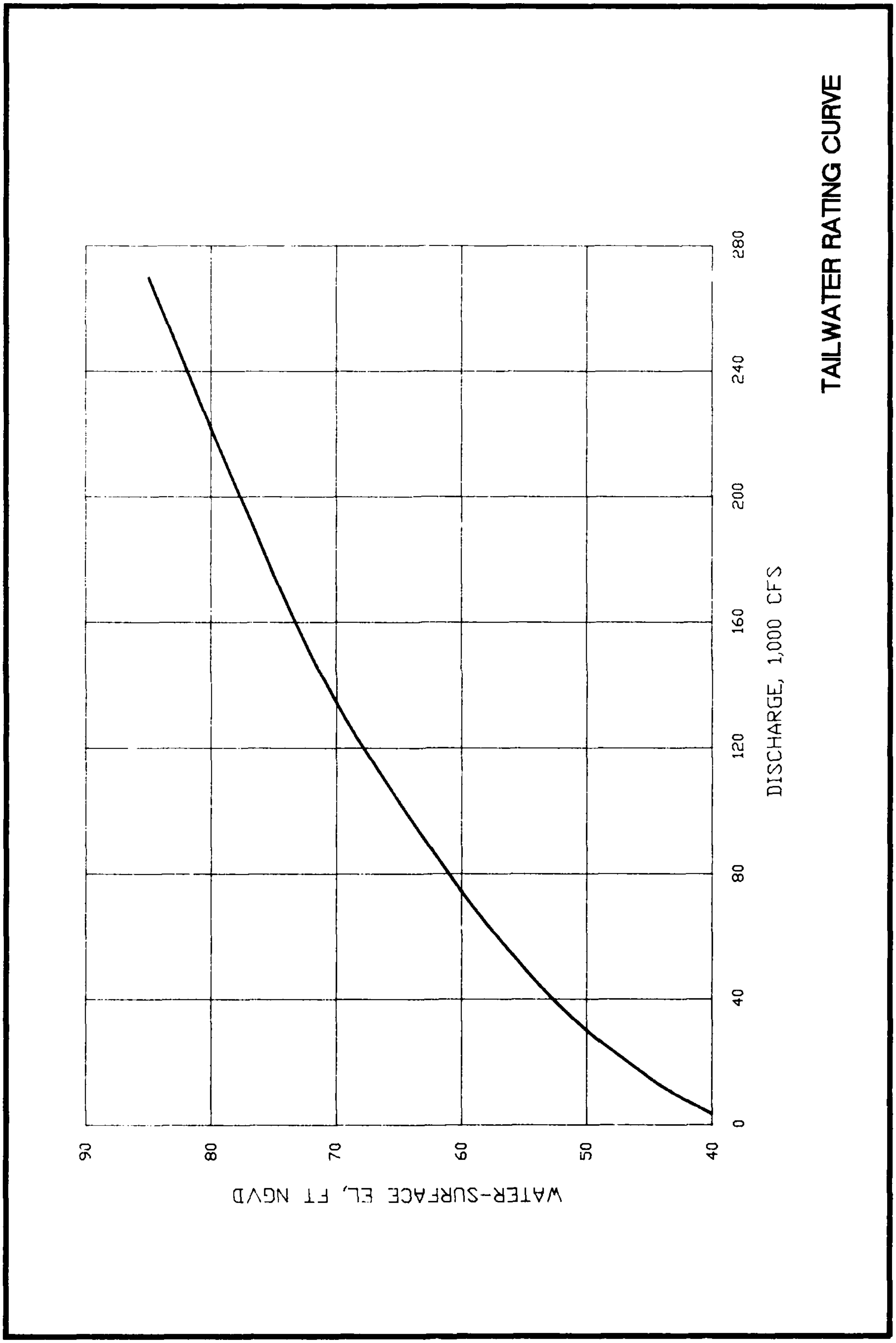
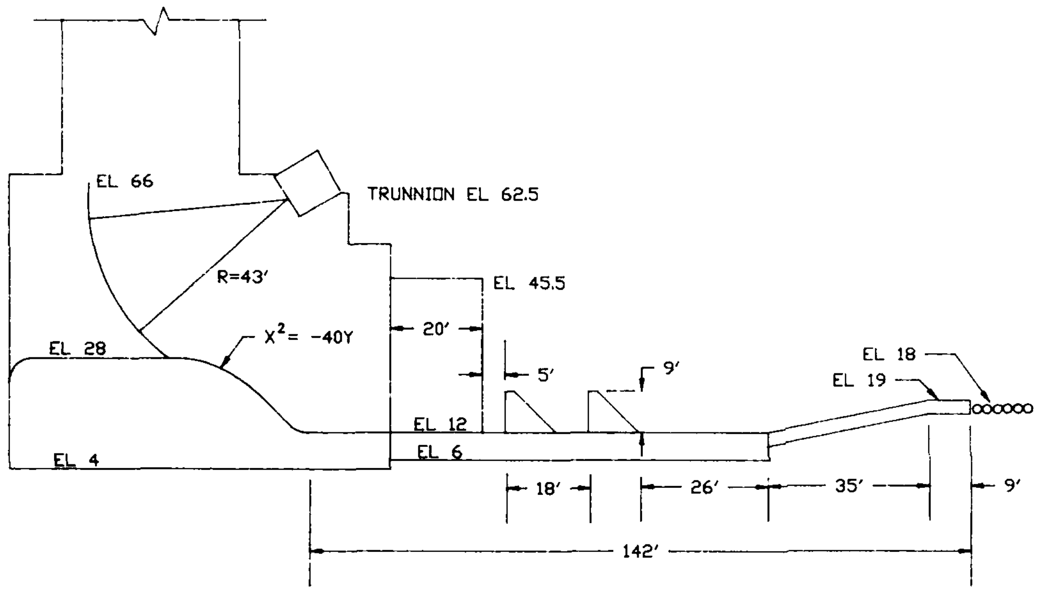
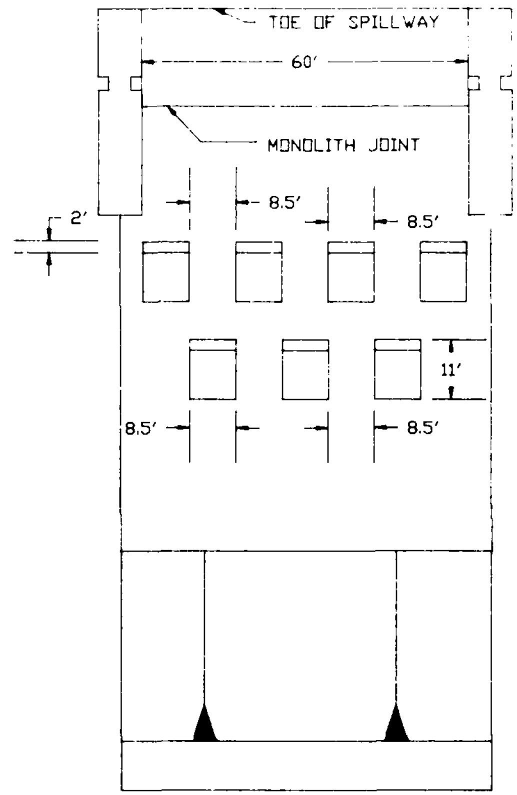

TAILWATER RATING CURVE
DISCHARGE, 1,000 CFS
0
40
80
120
160
200
240
280
WATER-SURFACE EL, FT NGVD
40
50
60
70
80
90

PLATE 3



ELEVATION VIEW- SPILLWAY AND STILLING BASIN



PLAN VIEW- STILLING BASIN

TYPE 13 DESIGN STILLING BASIN

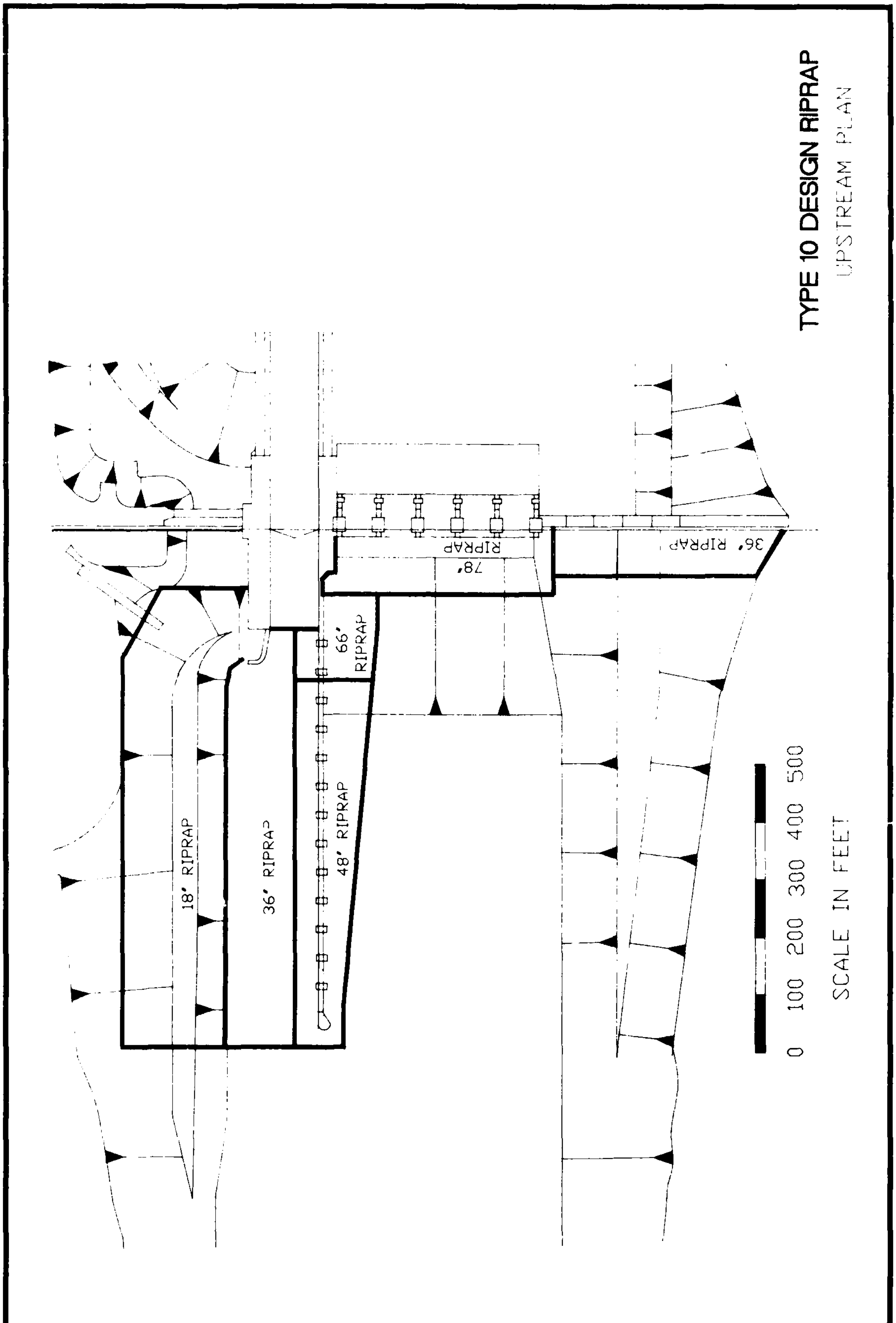
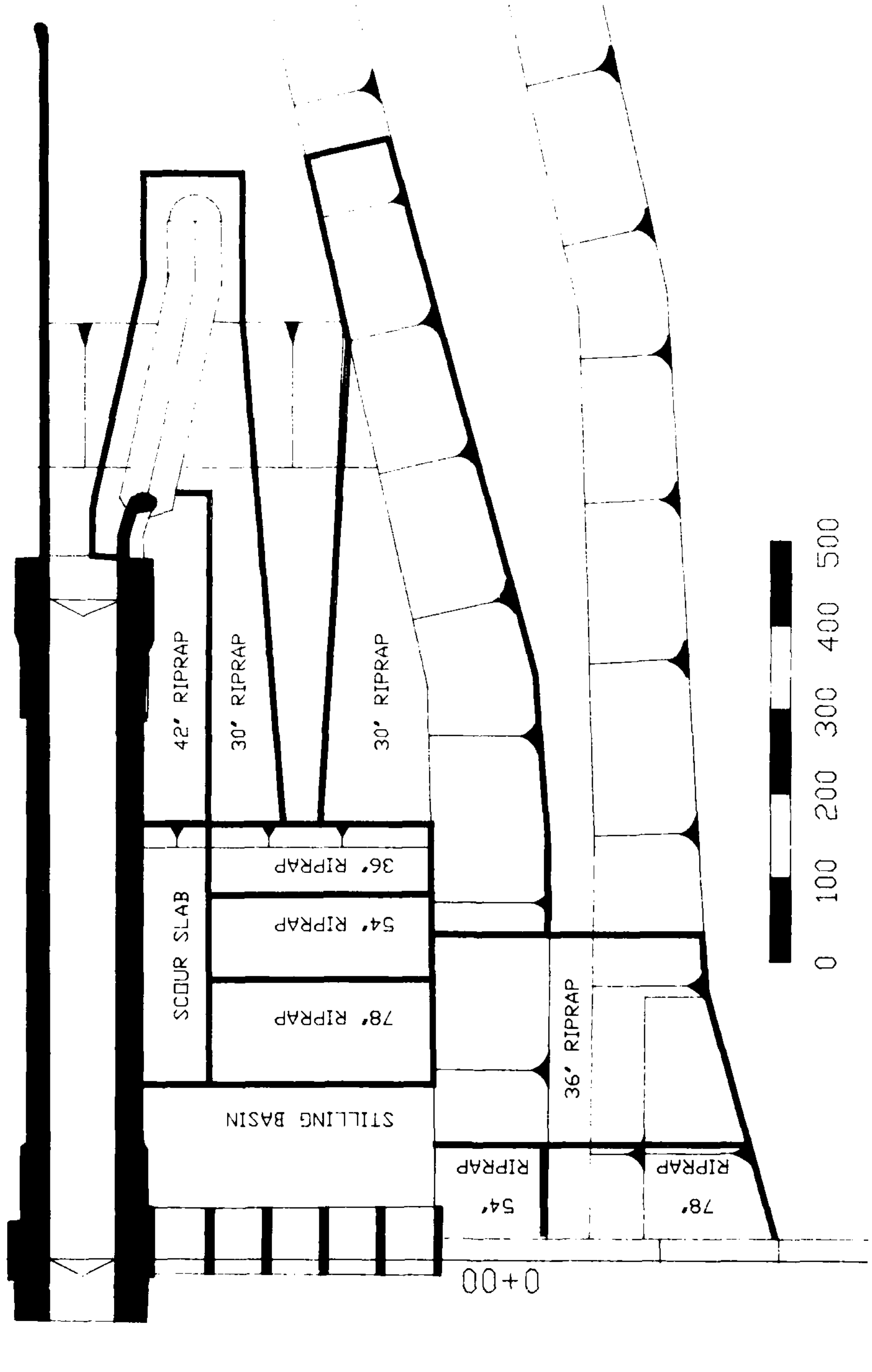
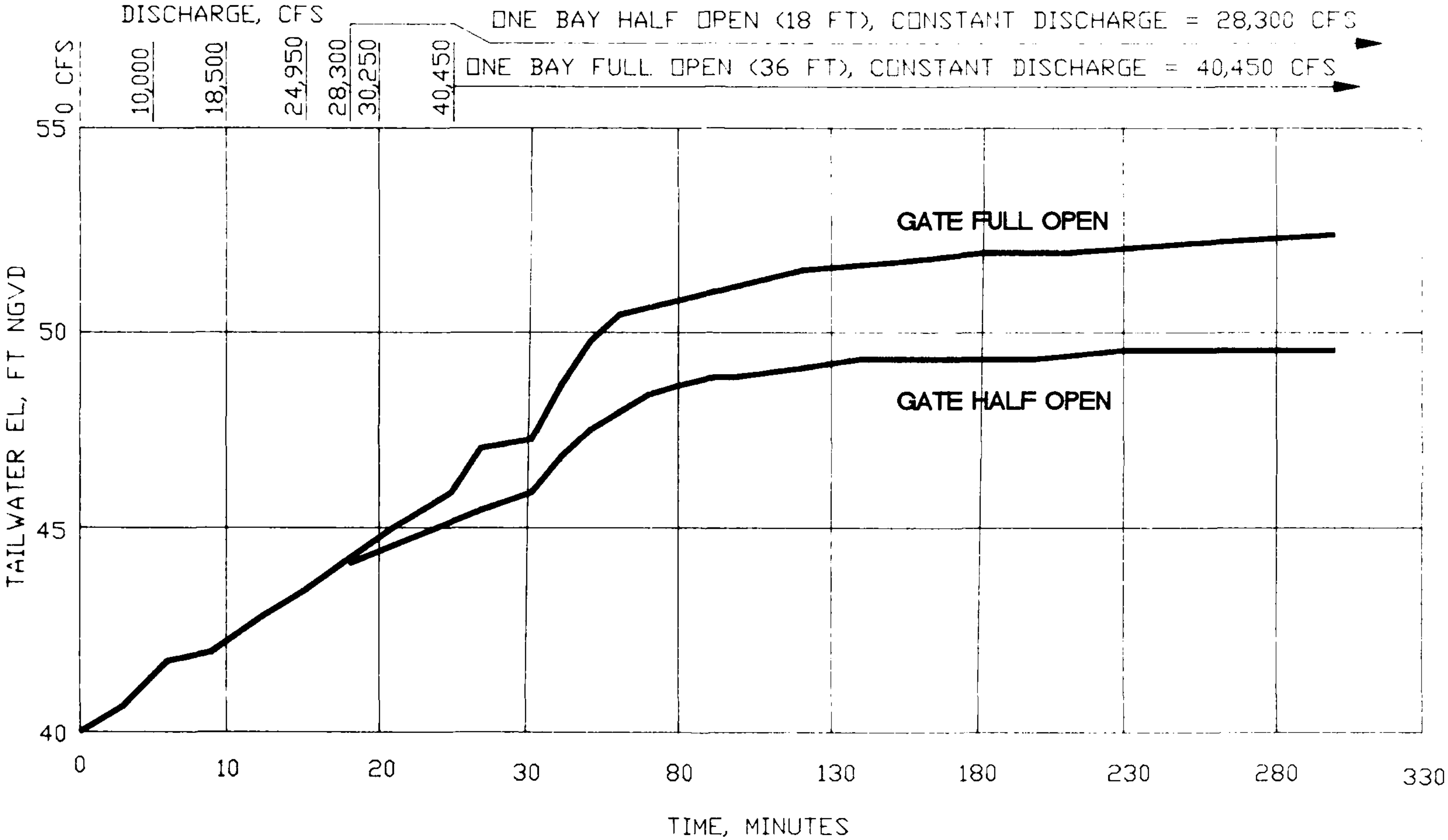


PLATE 5
(SHEET 1 OF 2)



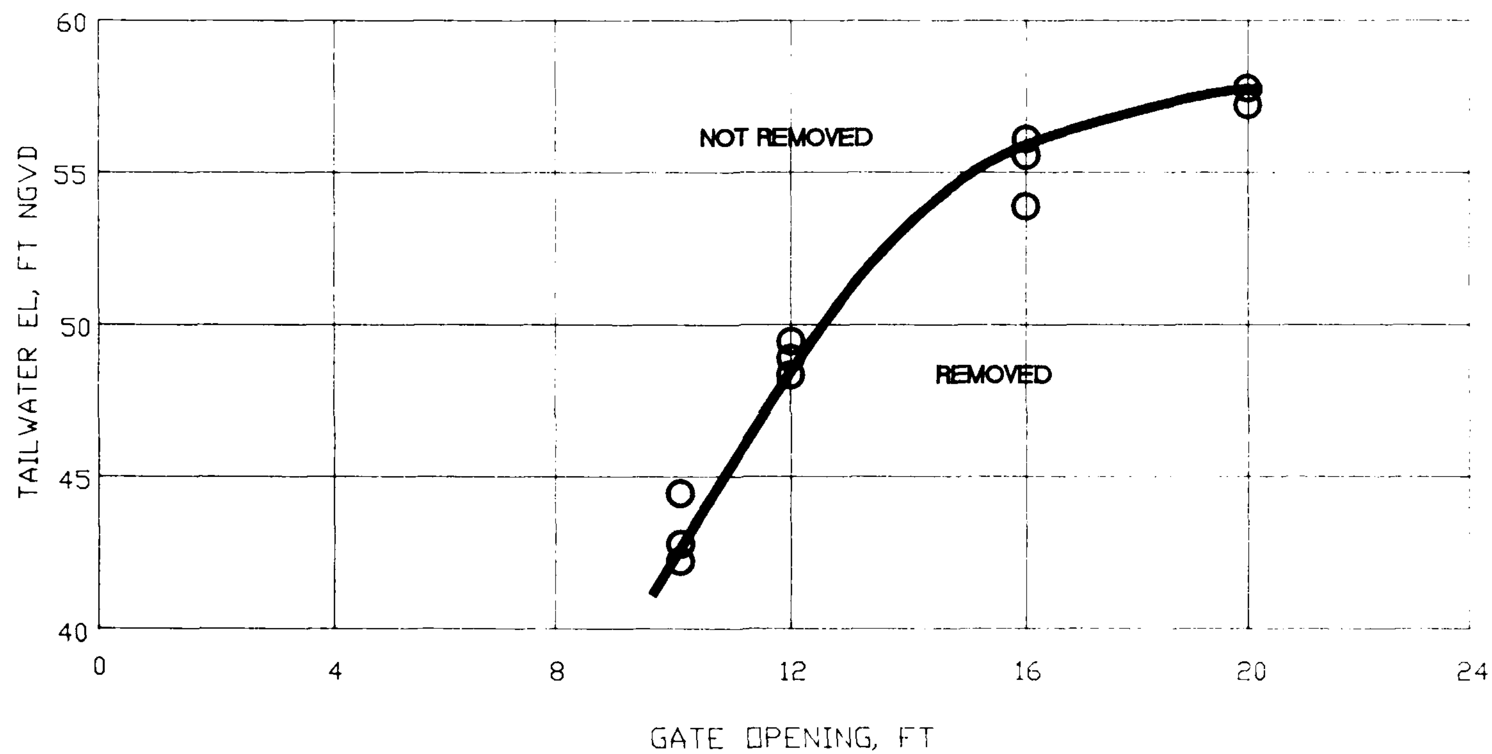
TYPE 10 DESIGN RIPRAP

DOWNSTREAM PLAN



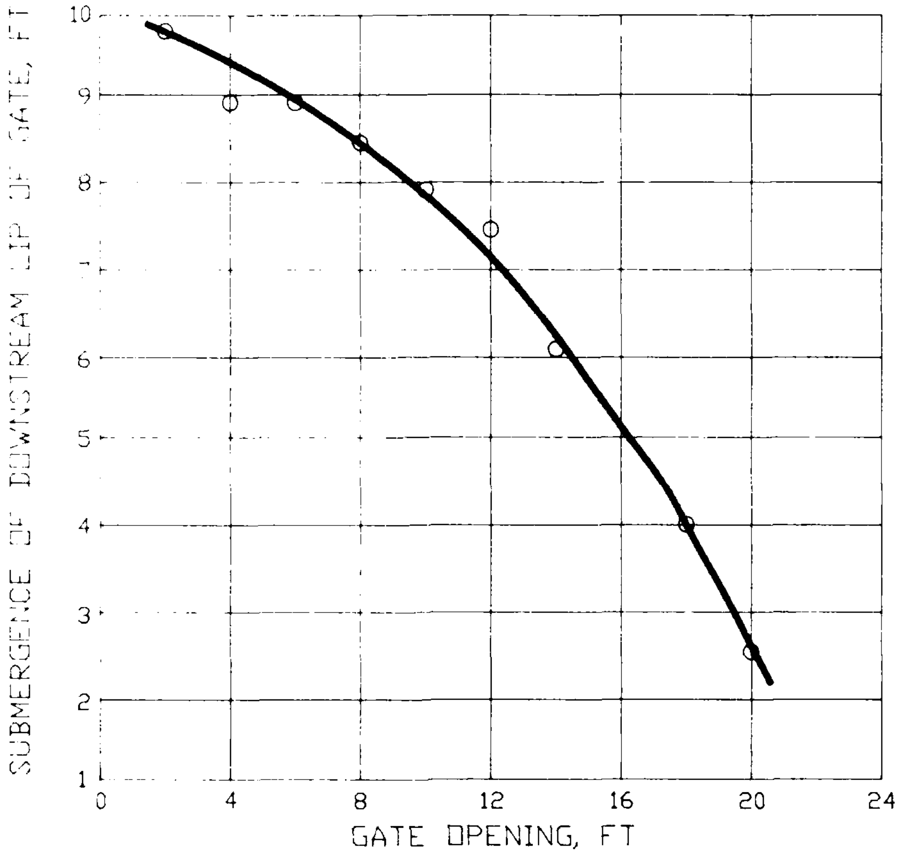
NOTE: TIME SCALE CHANGES AT 30 MINUTES
RATE OF GATE OPENING = 1 FT/MIN

TAILWATER RISE

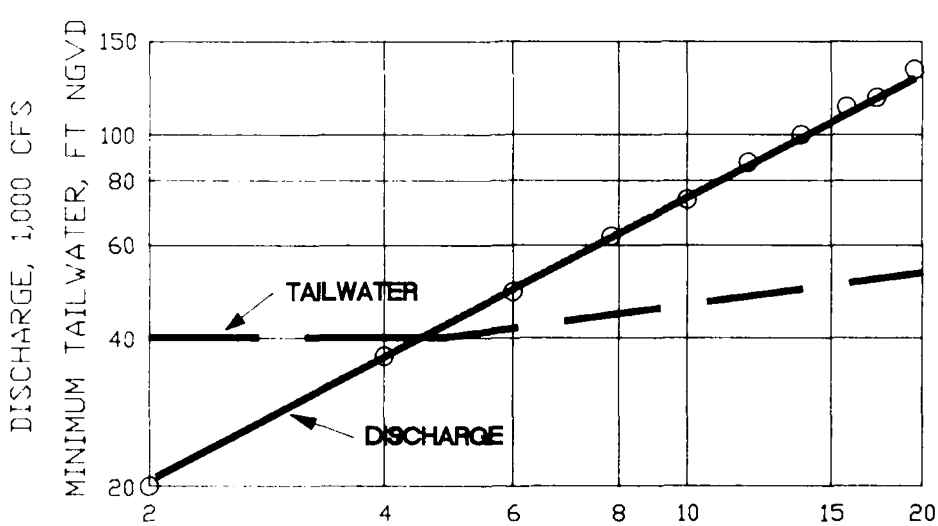


NOTE: SINGLE GATE OPERATION
NORMAL UPPER EL 64
DEBRIS SIZE = 1-FT DIAMETER
10-FT LENGTH

DEBRIS REMOVAL CURVE



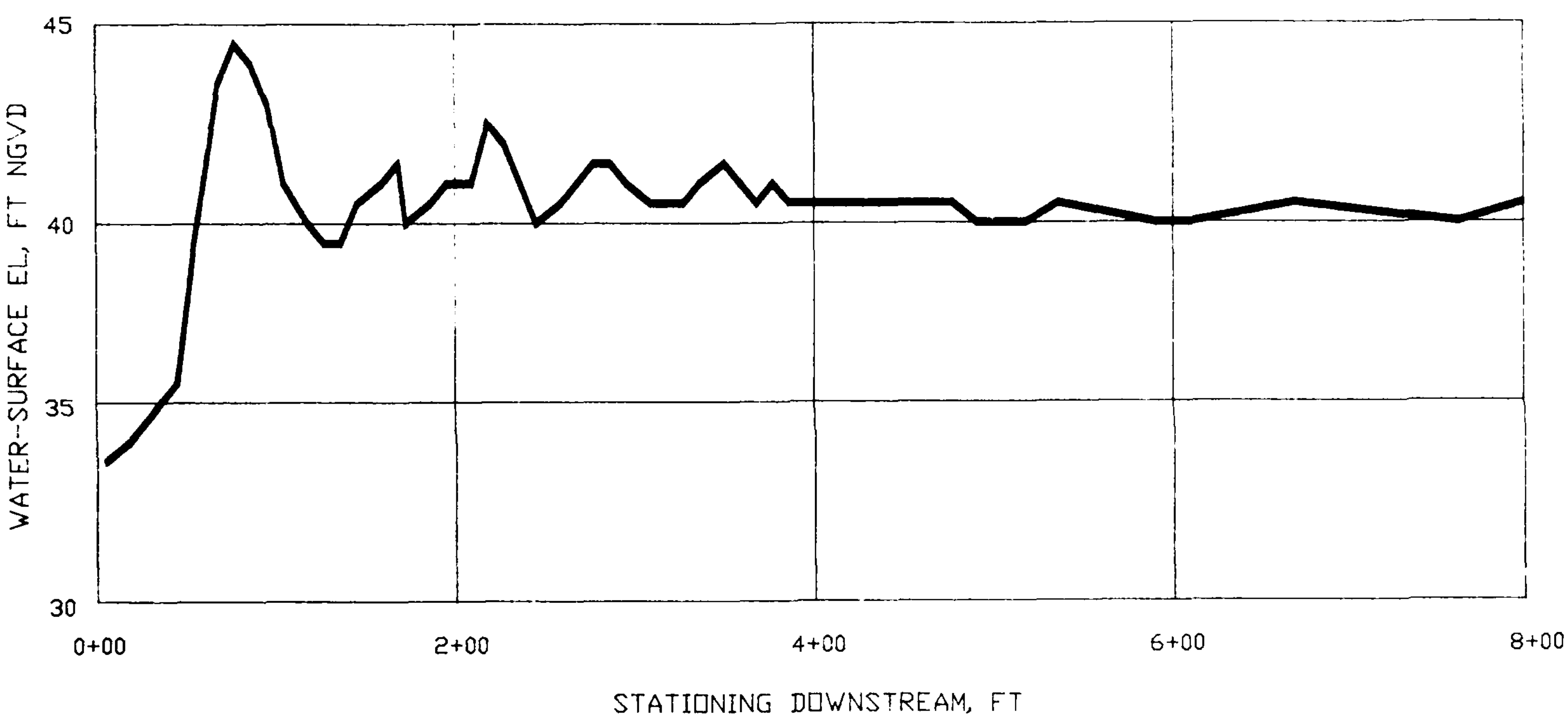
SUBMERGENCE VERSUS GATE OPENING



DISCHARGE AND TAILWATER USED IN SUBMERGENCE TESTS

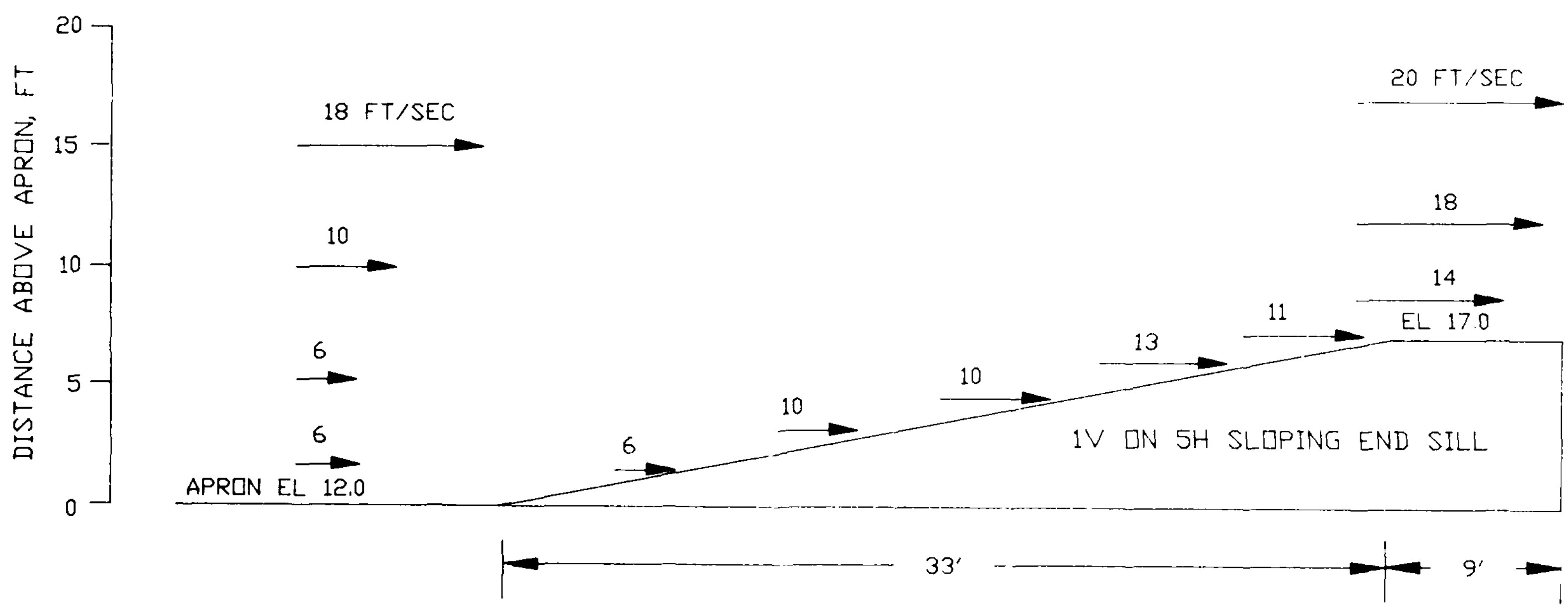
NORMAL UPPER EL 64.0
EQUAL GATE OPERATION

SUBMERGENCE OF GATE LIP VERSUS GATE OPENING



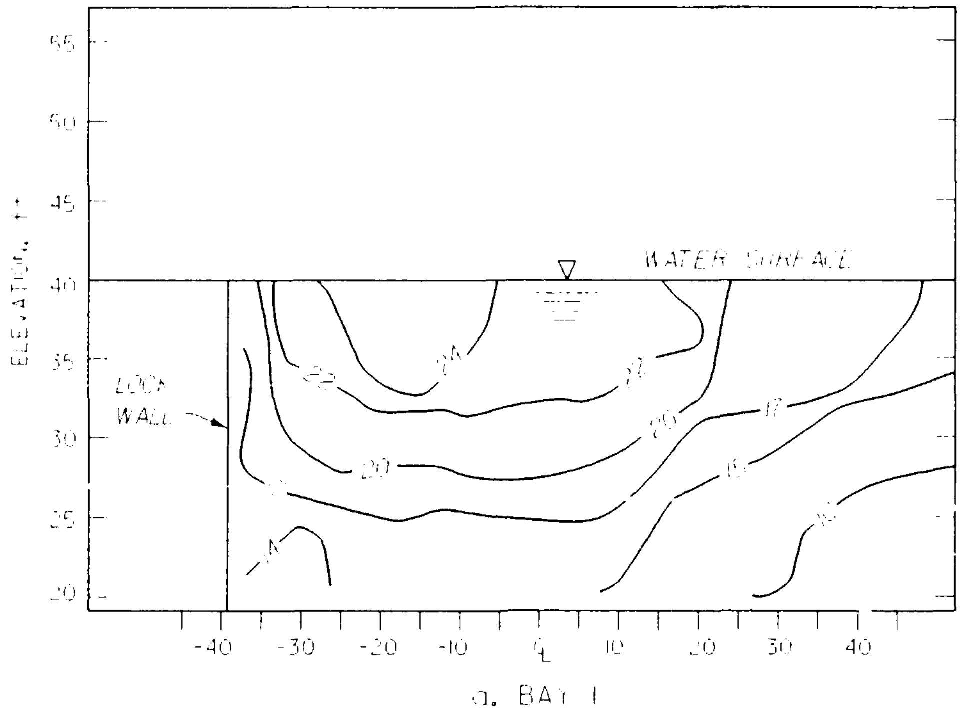
GATE NO. 1
TYPE 13 DESIGN STILLING BASIN
HEADWATER EL 64.0
TAILWATER EL 40.0
GATE OPENING = 18 FT

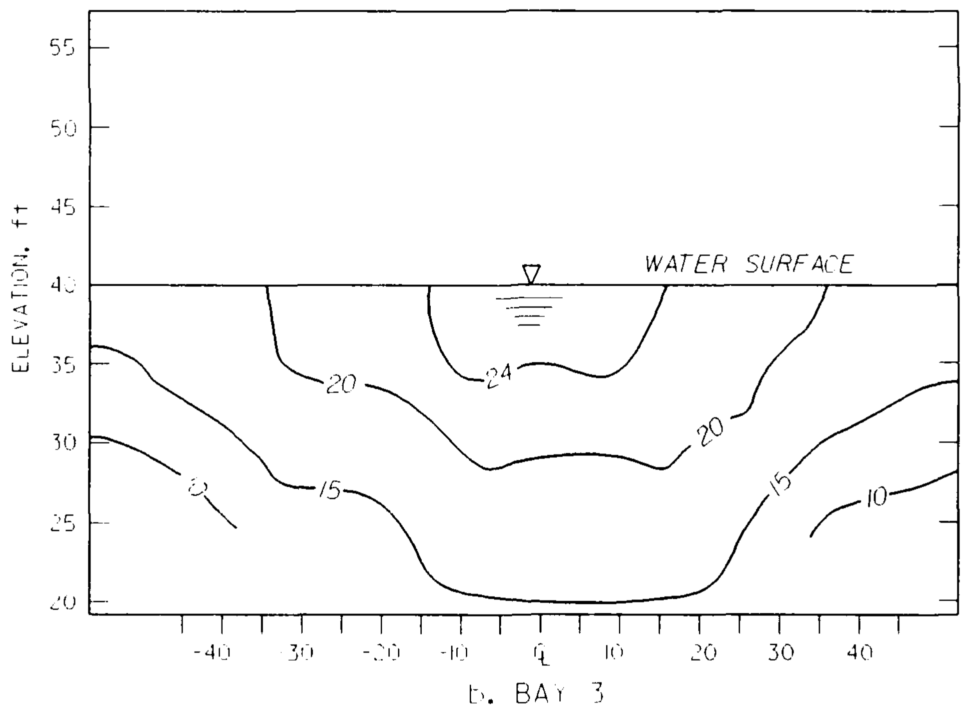
DOWNSTREAM WATER-SURFACE PROFILE



VELOCITIES TAKEN AT CENTER LINE OF
GATE NO. 3
DISCHARGE = 28,300 CFS
HEADWATER EL 64.0
TAILWATER EL 40.0
GATE OPENING = 18 FT
TYPE 13 DESIGN STILLING BASIN

END SILL VELOCITIES



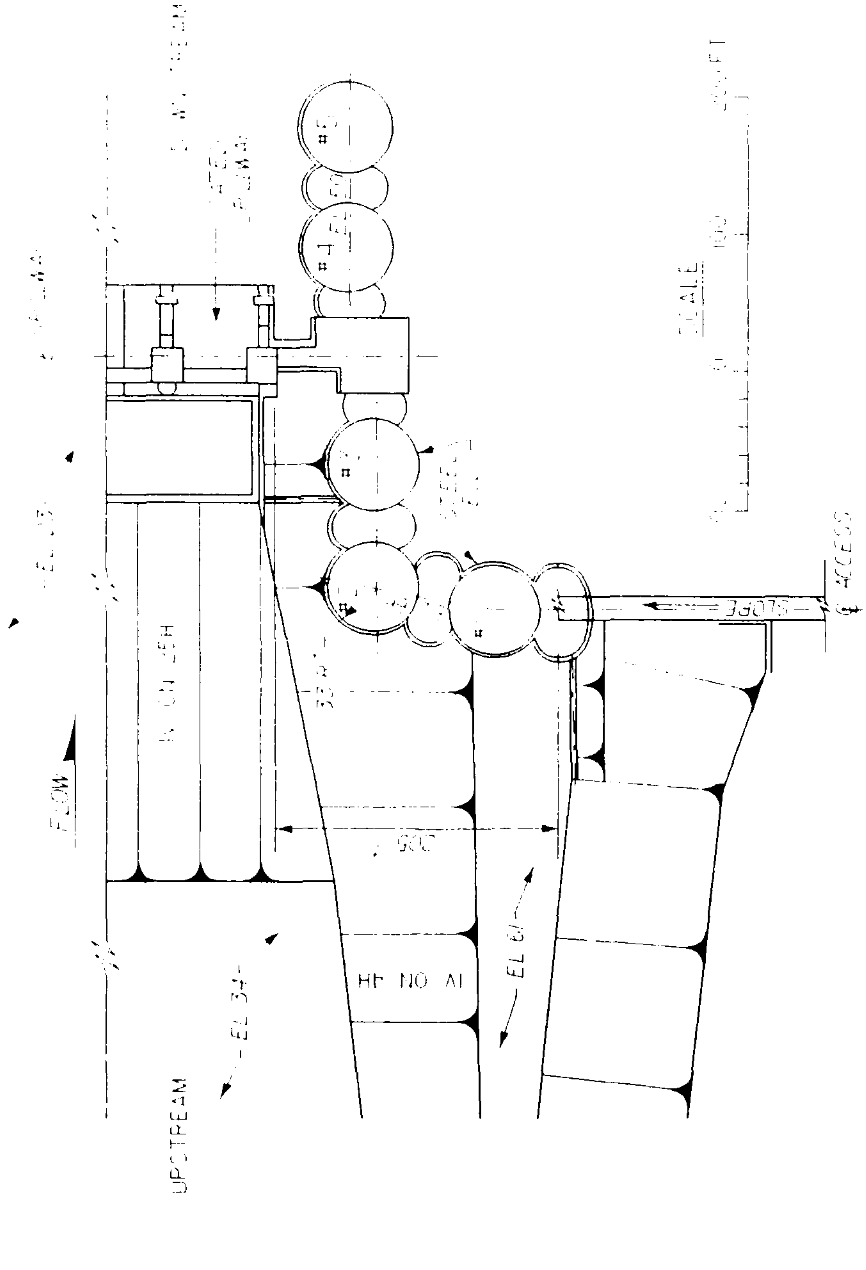


TEST CONDITIONS

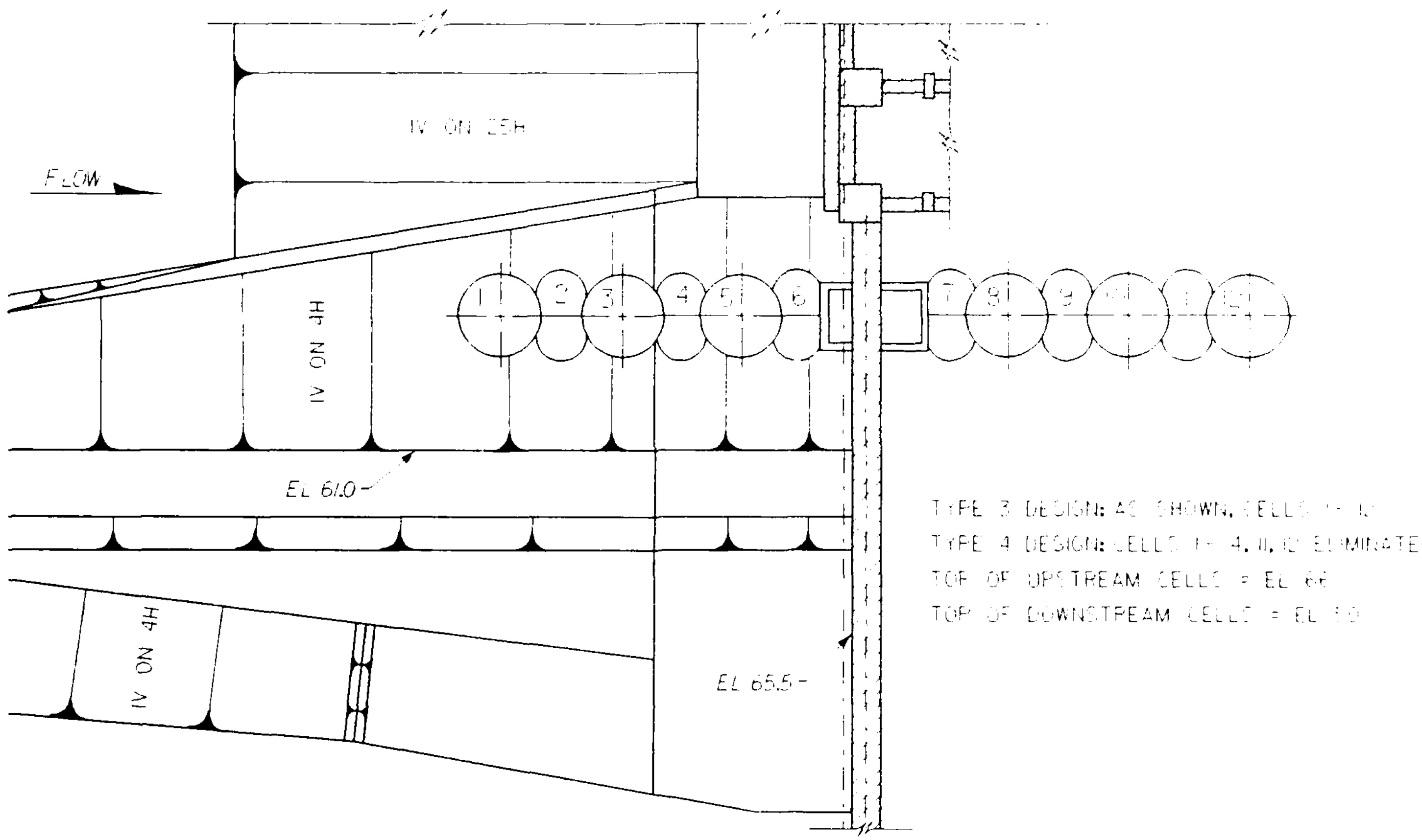
Q = 28,300 cfs
NORMAL UPPER POOL 64 ft
HALF GATE CONDITION G_0 = 18 ft
TAILWATER EL 40 ft
VELOCITIES MEASURED ABOVE END SILL
LOOKING DOWNSTREAM

ISOVELS

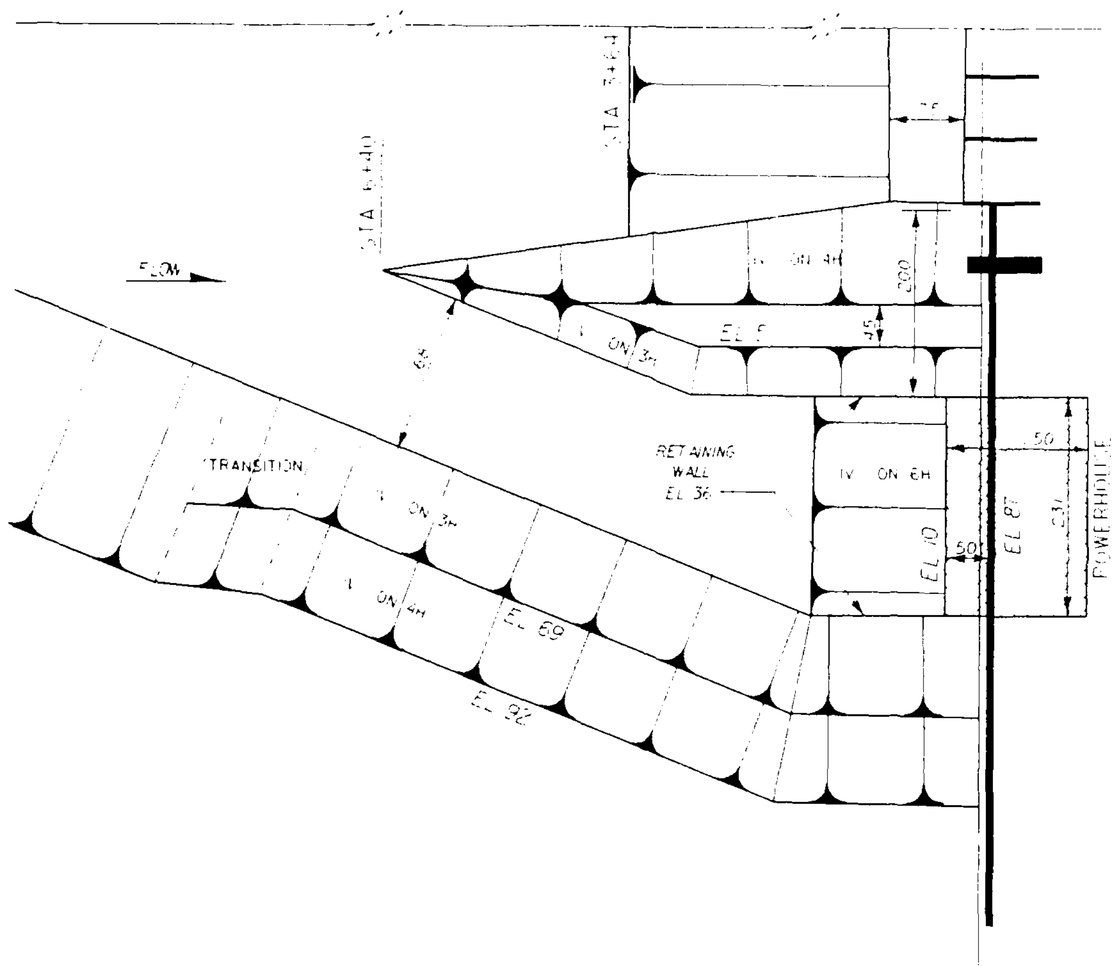
TYPE 13 STILLING BASIN
UNIT DISCHARGE 471 cfs/ft



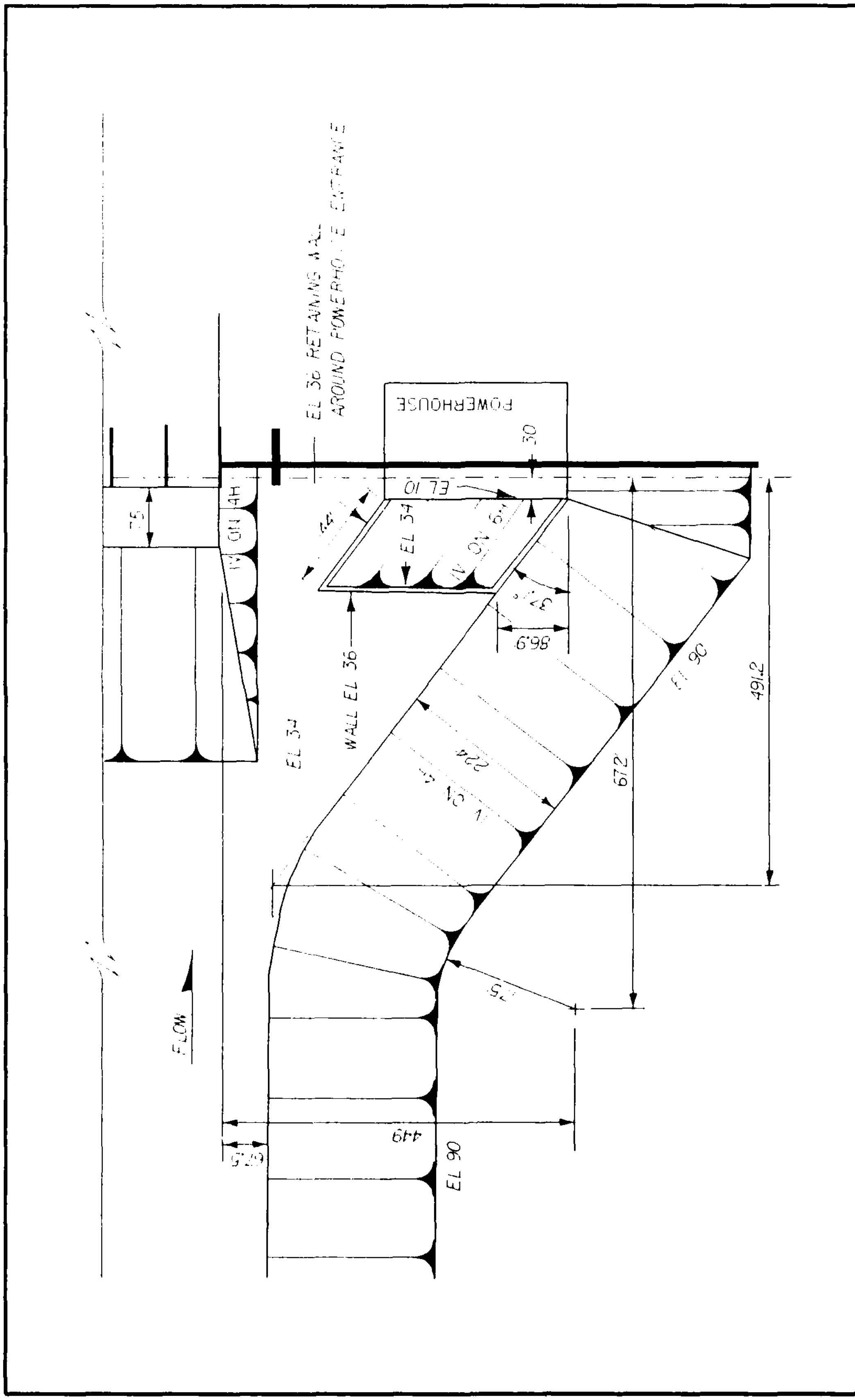
TYPE I COFFERDAM DESIGN



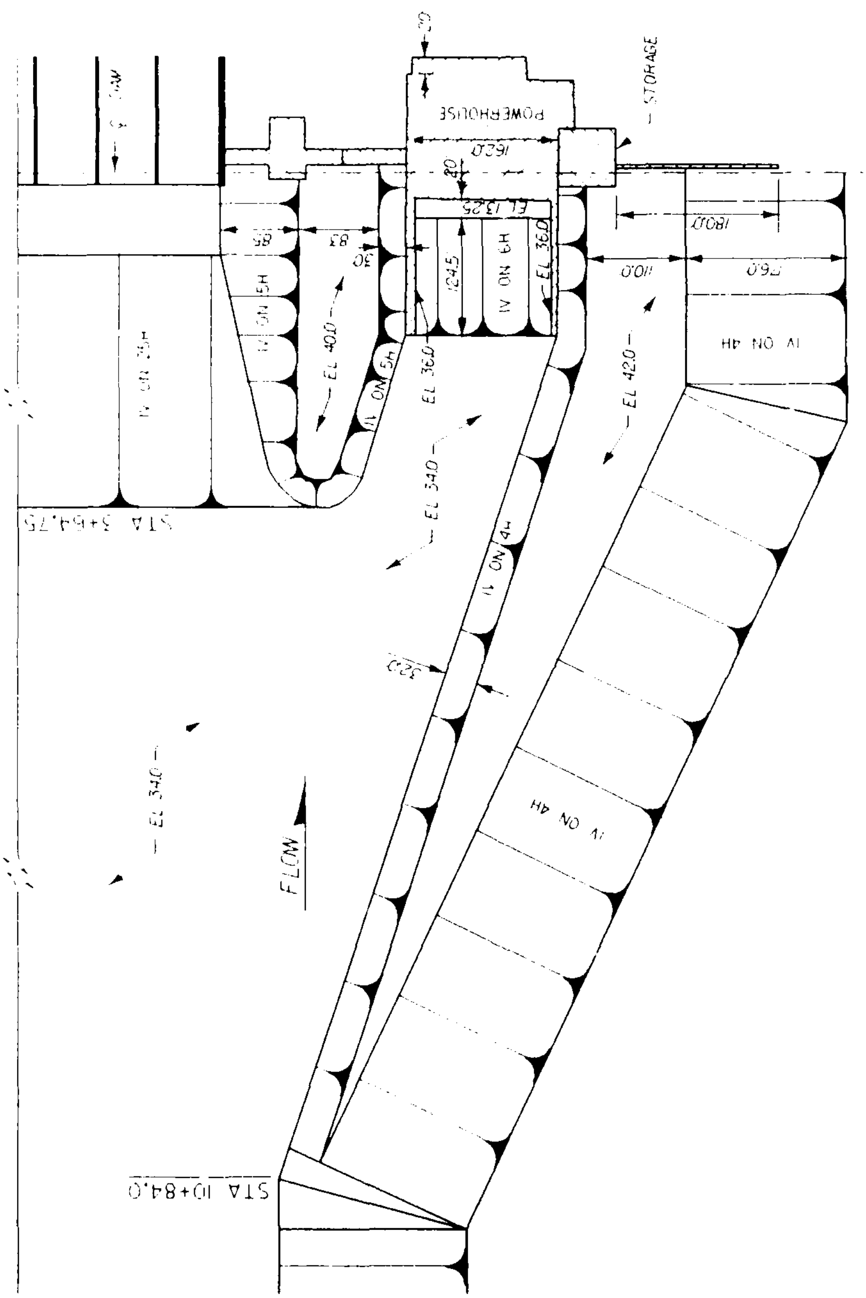
TYPE 3 AND 4
DESIGN COFFERDAMS



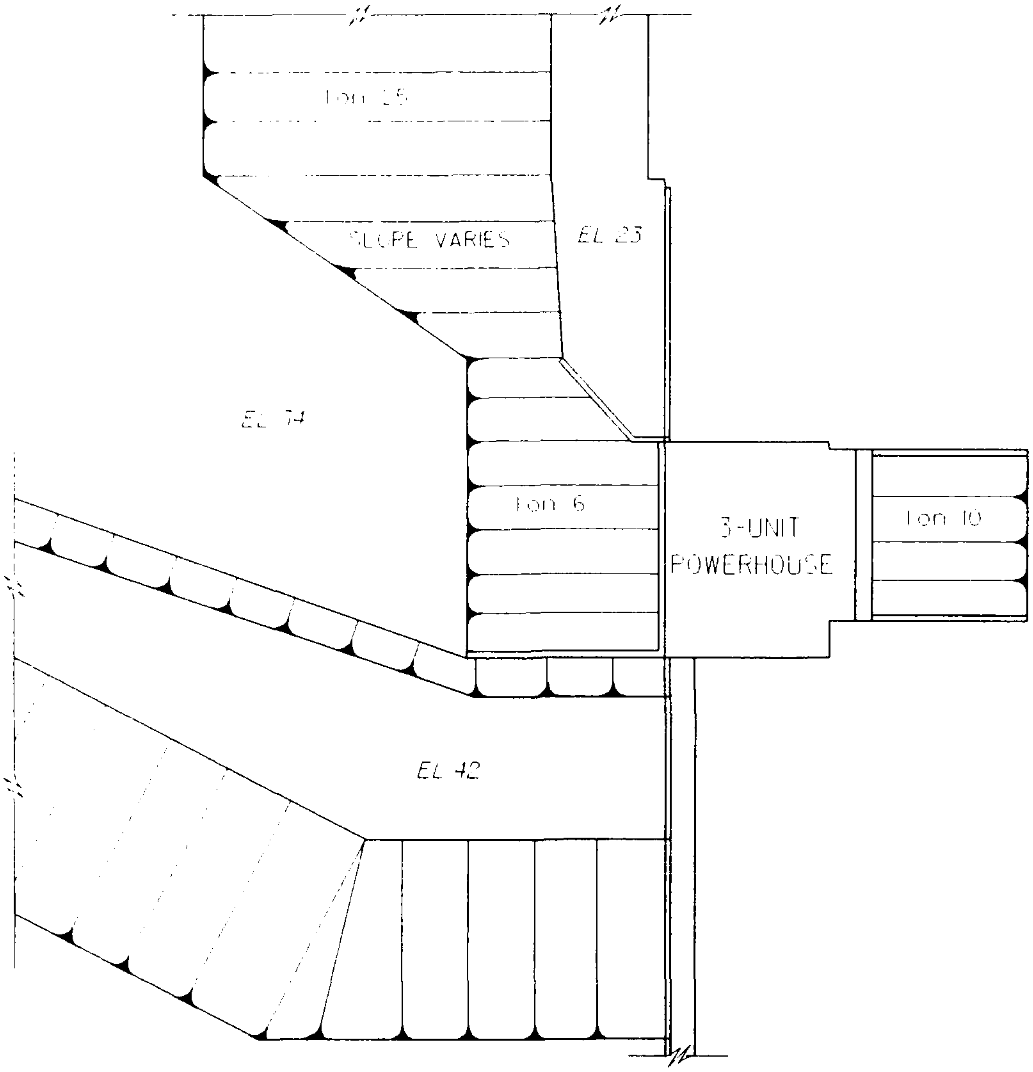
TYPE I HYDROPOWER DESIGN



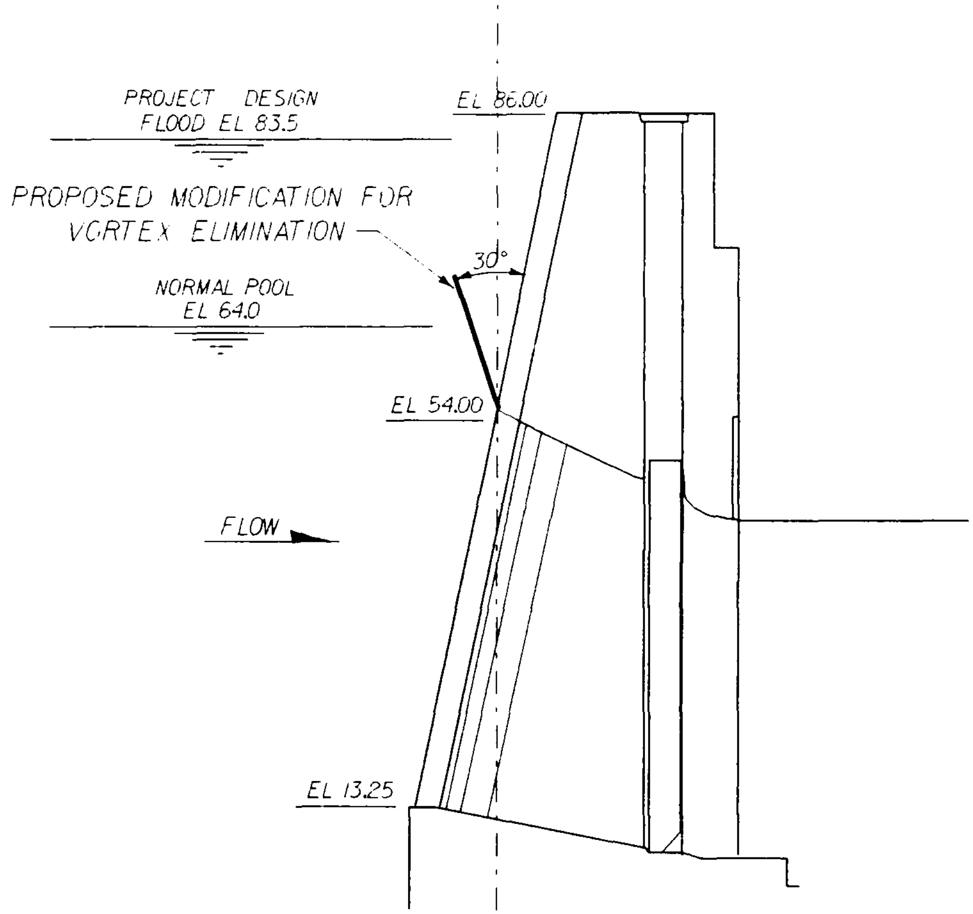
TYPE 2 HYDROPOWER DESIGN



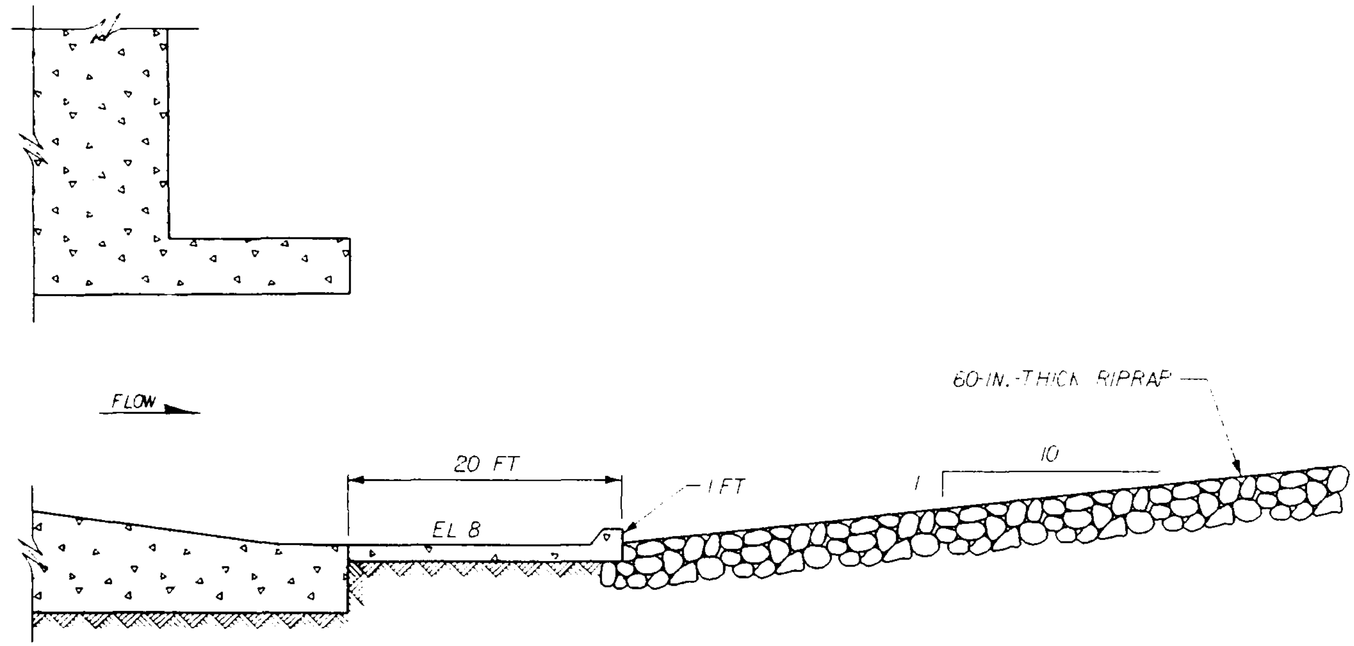
TYPE 3 HYDROPOWER DESIGN



TYPE 4
HYDROPOWER DESIGN



TYPE 5 HYDROPOWER DESIGN



TRANSVERSE SECTION THRU TYPICAL TURBINE

MODEL GRADATION TESTED

PERCENT FINER BY WEIGHT, %	WEIGHT STONE, lb
100	2640
50	1350
15	1100

TYPE 5 HYDROPOWER DESIGN
EXIT DESIGN

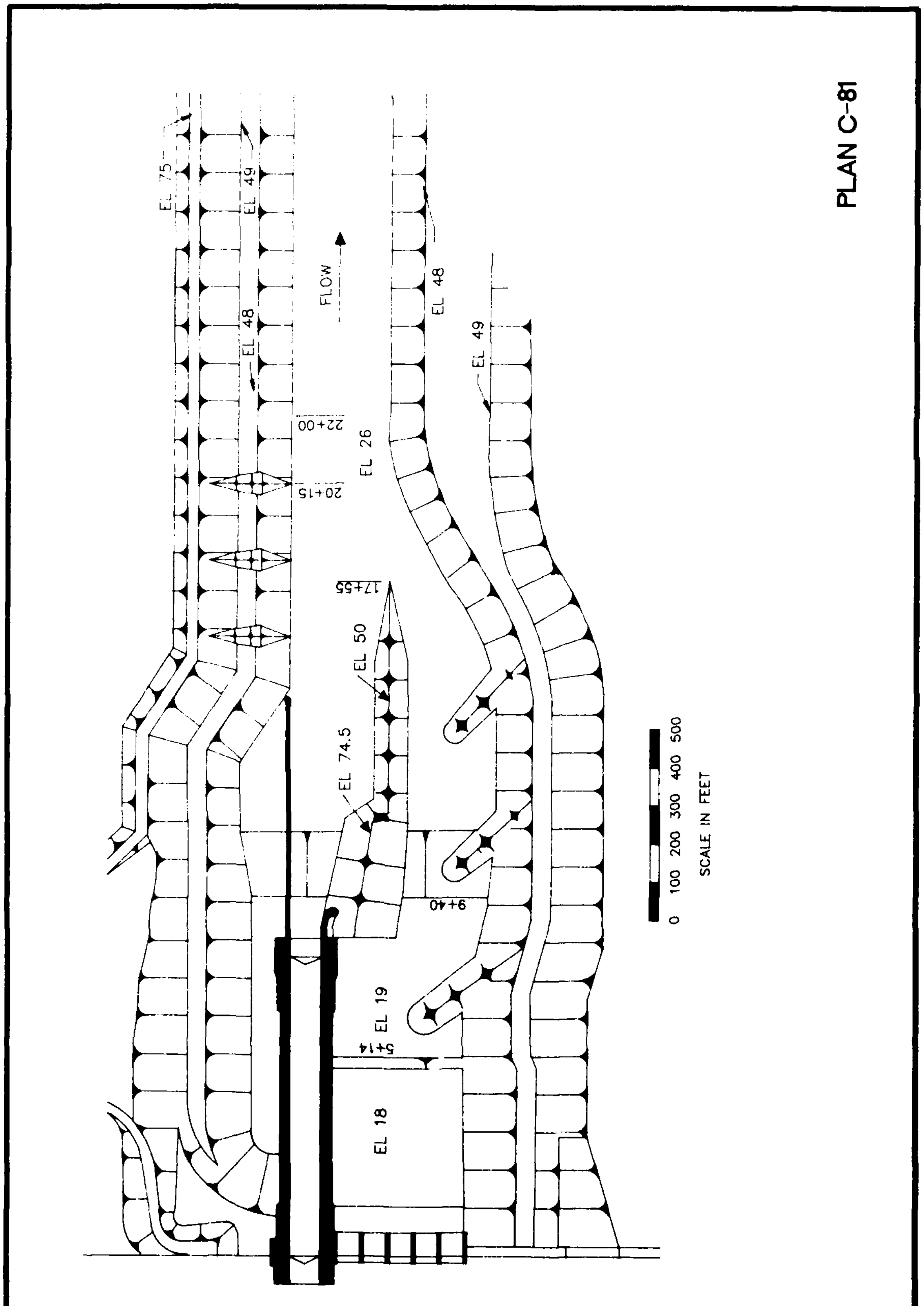
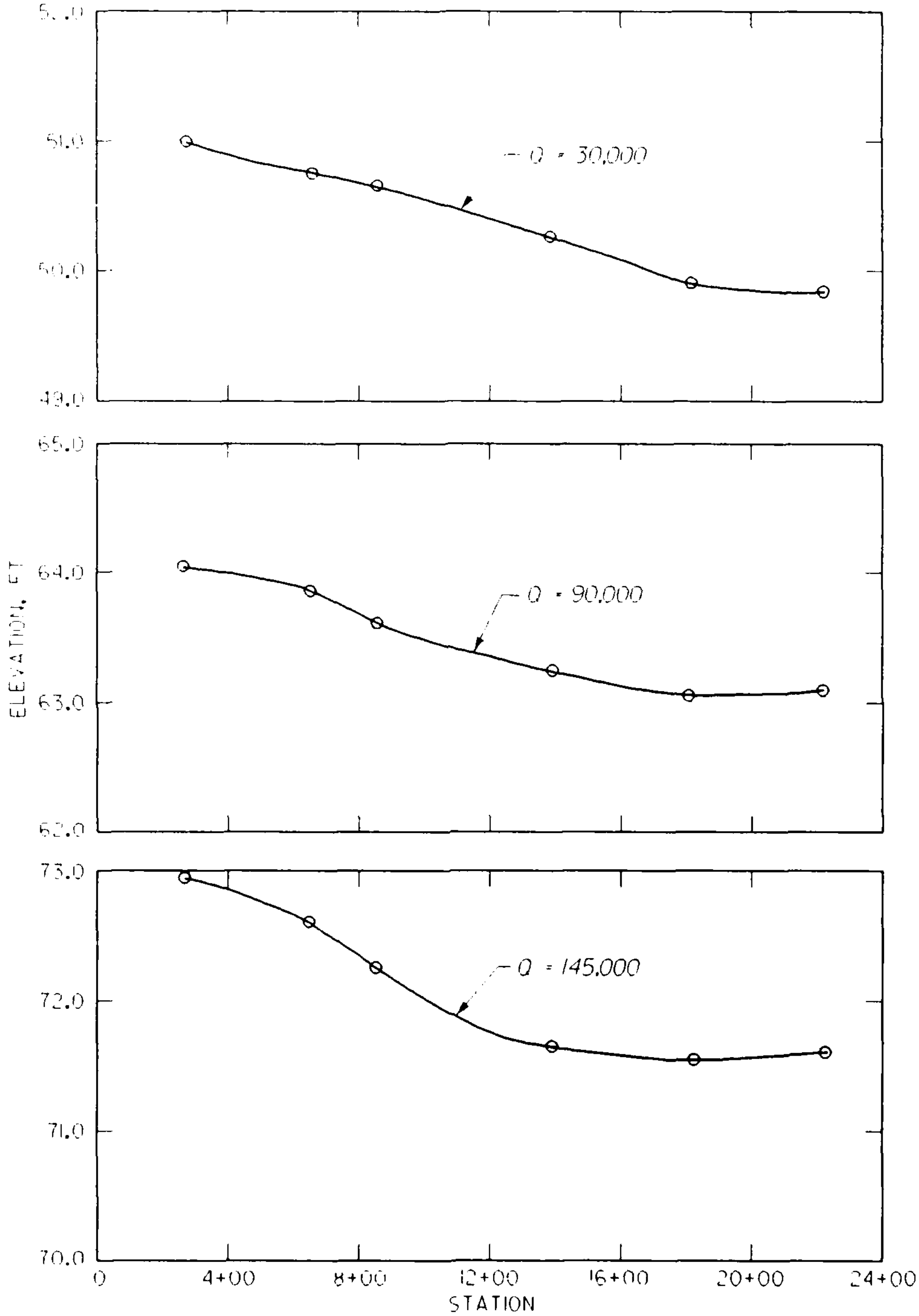
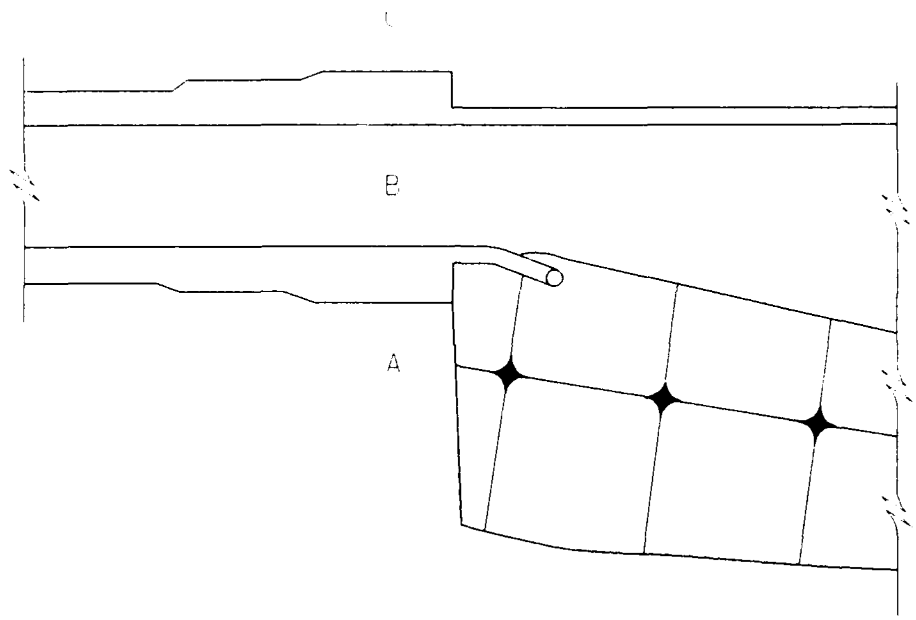

PLAN C-81
EL 75
EL 49
EL 48
FLOW
EL 48
EL 49
22+00
EL 26
20+15
17+55
EL 50
EL 74.5
9+40
EL 19
5+14
EL 18
0 100 200 300 400 500
SCALE IN FEET

PLATE 20

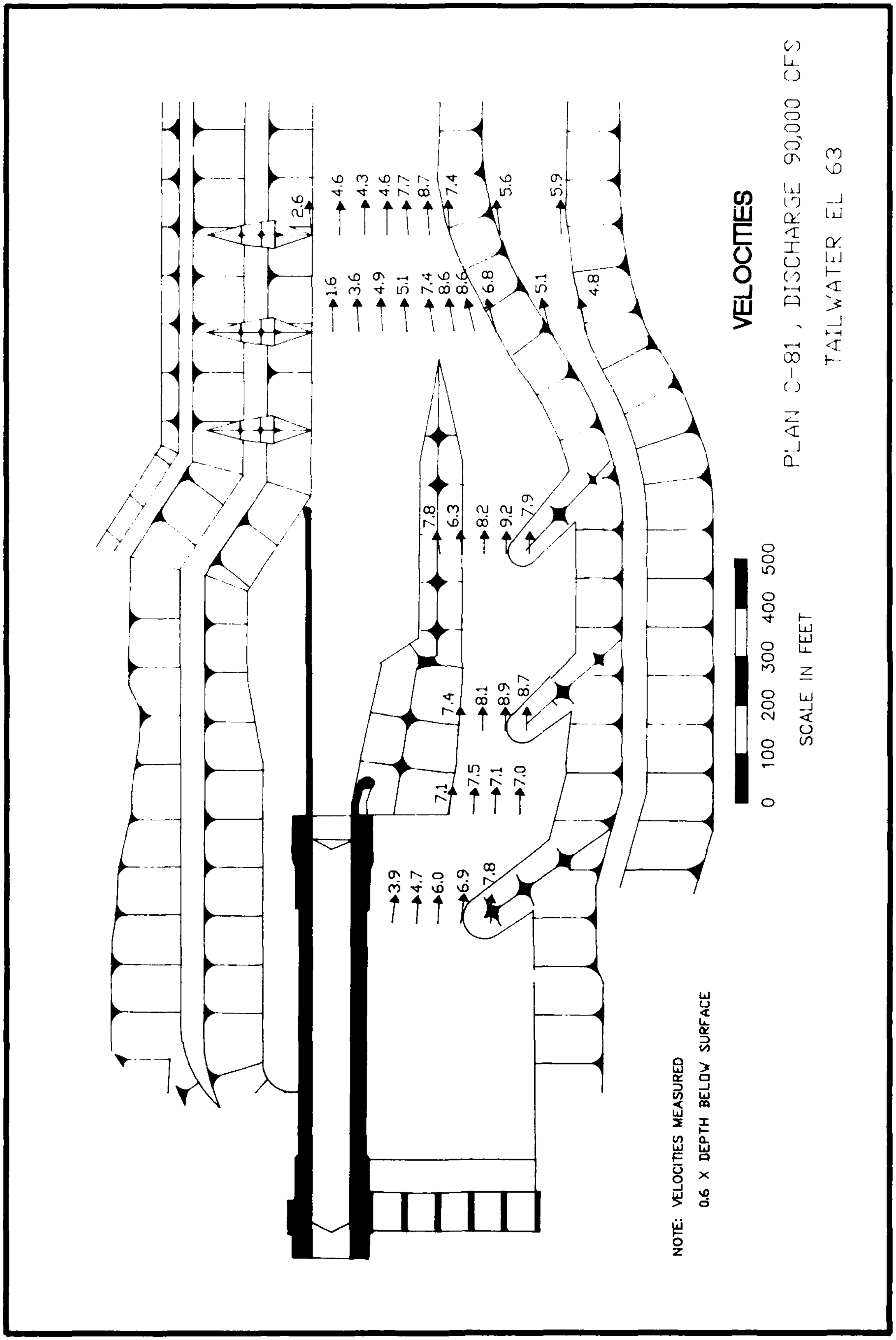


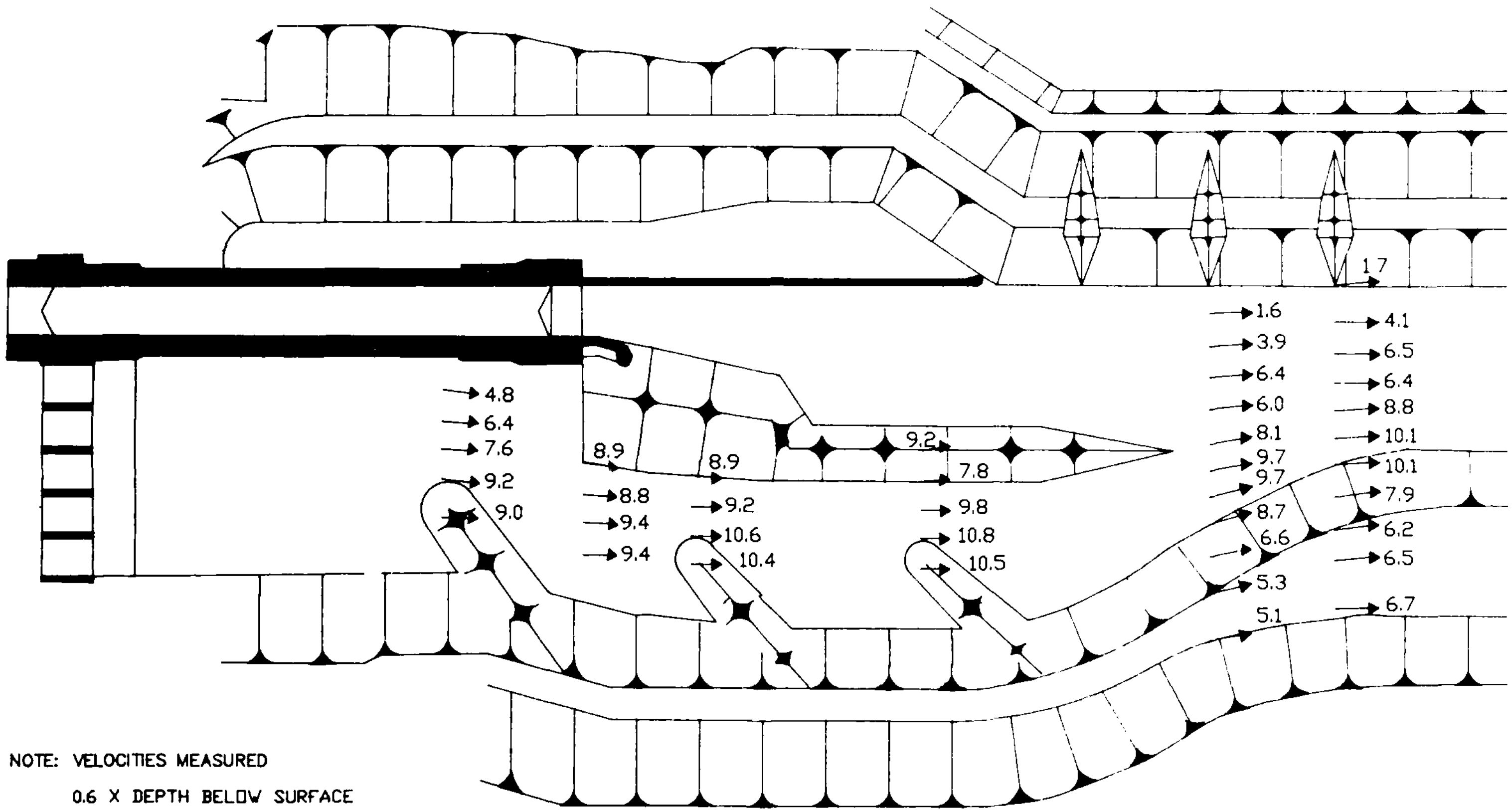
DOWNSTREAM
WATER-SURFACE PROFILES
PLAN C-81



Q, cfs	WATER-SURFACE ELEVATION		
	A	B	C
30,000	50.9	49.9	49.9
90,000	64.1	63.1	63.1
145,000	73.0	71.5	71.5

WATER-SURFACE ELEVATIONS
PLAN C-81

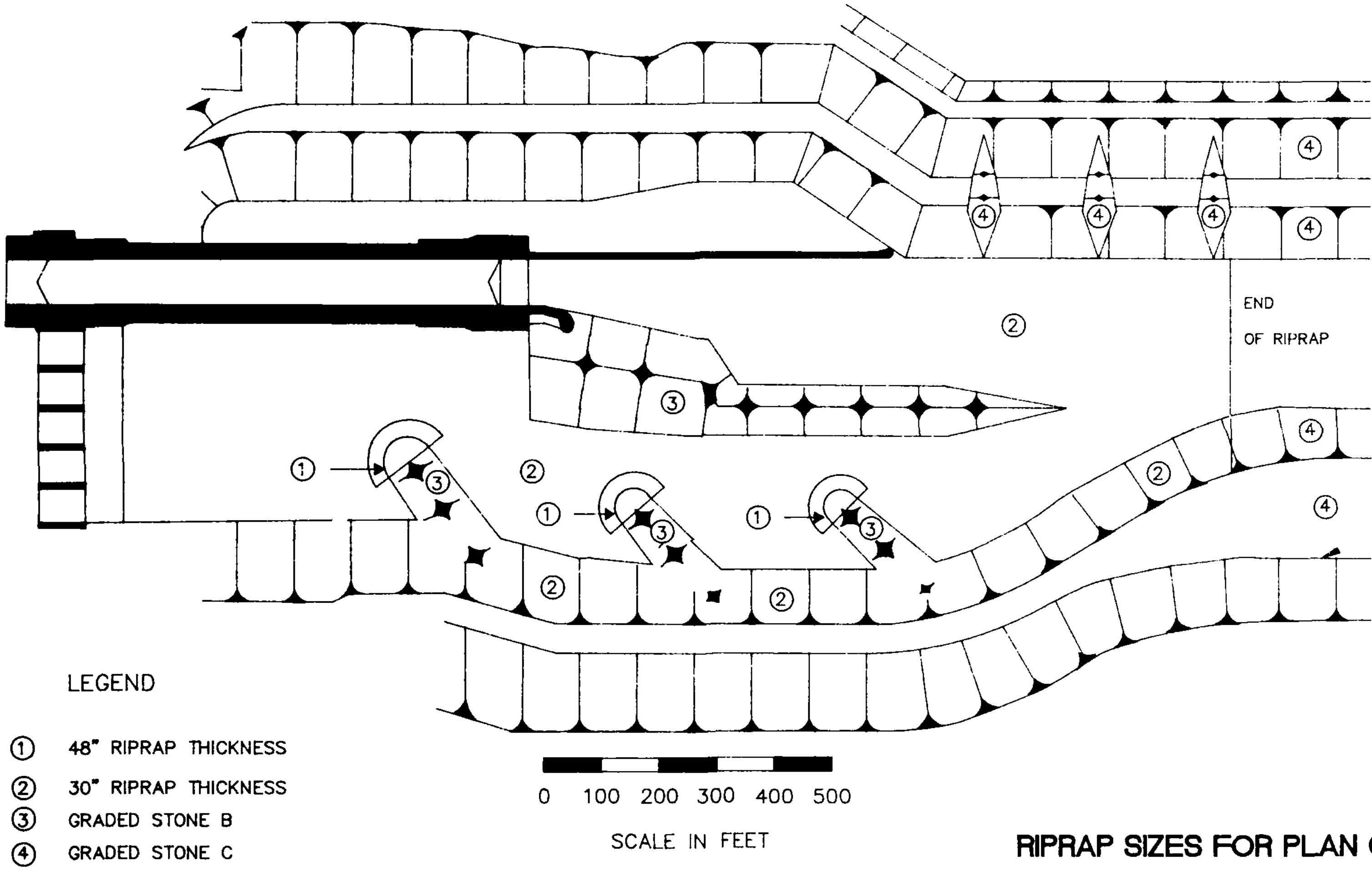

VELOCITIES
PLAN C-81, DISCHARGE 90,000 CFS
TAILWATER EL 63
0 100 200 300 400 500
SCALE IN FEET
NOTE: VELOCITIES MEASURED
0.6 X DEPTH BELOW SURFACE



VELOCITIES

PLAN C-81, DISCHARGE 145,000 CFS

TAILWATER EL 71.5



RIPRAP SIZES FOR PLAN C-81

R2P24.DWG