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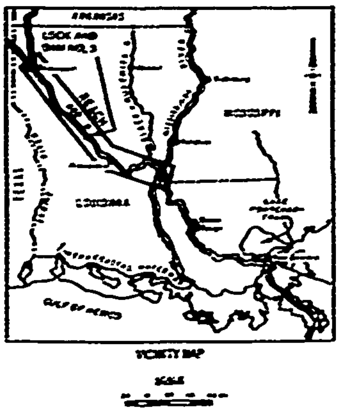

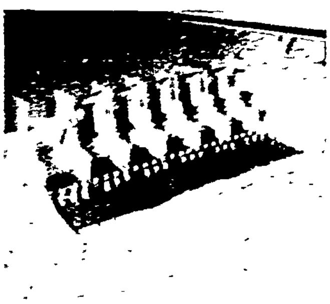

HYDRAULICS LABORATORY

TECHNICAL REPORT HL-91-10

RED RIVER WATERWAY, LOCK AND DAM NO. 3

Report 4
STILLING BASIN AND RIPRAP REQUIREMENTS

Spillway Hydraulic Model Study

by

Stephen T. Maynord

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Report 4 of a Series

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13. ABSTRACT *(Maximum 200 words)*

Tests were conducted on a 1:50-scale model of Lock and Dam No. 3, Red River Waterway, to develop a satisfactory energy dissipator and a stable riprap plan. The spillway consists of a 315-ft-long overflow weir and six 42-ft-high by 60-ft-wide tainter gates. The 84-ft-wide by 785-ft-long lock will be located on the left riverbank looking downstream and will have a maximum lift of 31 ft.

The recommended stilling basin provides adequate energy dissipation for normal flows and for a single gate fully opened with minimum tailwater. Stable riprap plans were developed for the upstream and downstream areas adjacent to the structure for both normal flows and for single gate conditions. Pressures measured on the spillway crest were sufficiently high to prevent cavitation problems on the downstream face of the crest. Water-surface profiles were measured for a wide range of conditions and are compared to computed spillway rating curves.

14. SUBJECT TERMS

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Riprap	

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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, Vicksburg (LMK). The study was conducted by personnel of the Hydraulics Laboratory (HL), US Army Engineer Waterways Experiment Station (WES), during the period January 1984 to September 1987 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 of the Hydraulic Structures Division (HSD), HL. The model tests were conducted by Messrs. J. V. Markussen and R. Bryant and Dr. S. T. Maynord under the supervision of Mr. N. R. Oswalt, Chief of the Spillways and Channels Branch, HSD. The model was constructed by the Model Shop, Mr. S. J. Leist, Chief, Engineering and Construction Services Division, WES. This report was prepared by Dr. Maynord and edited by Mrs. Marsha Gay, Information Technology Laboratory, WES.

During the course of the investigation, Mr. Bruce McCartney, HQUSACE; Mr. Larry Cook, LMVD; and Messrs. Phil Combs, Nolan Raphelt, David Biedenharn, Charles Shelton, and Rick Robertson, LMK, visited WES to observe model testing and discuss test results.

COL Larry B. Fulton, EN, was the Commander and Director of WES. Dr. Robert W. Whalin was the Technical Director.

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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
square feet	0.09290304	square metres

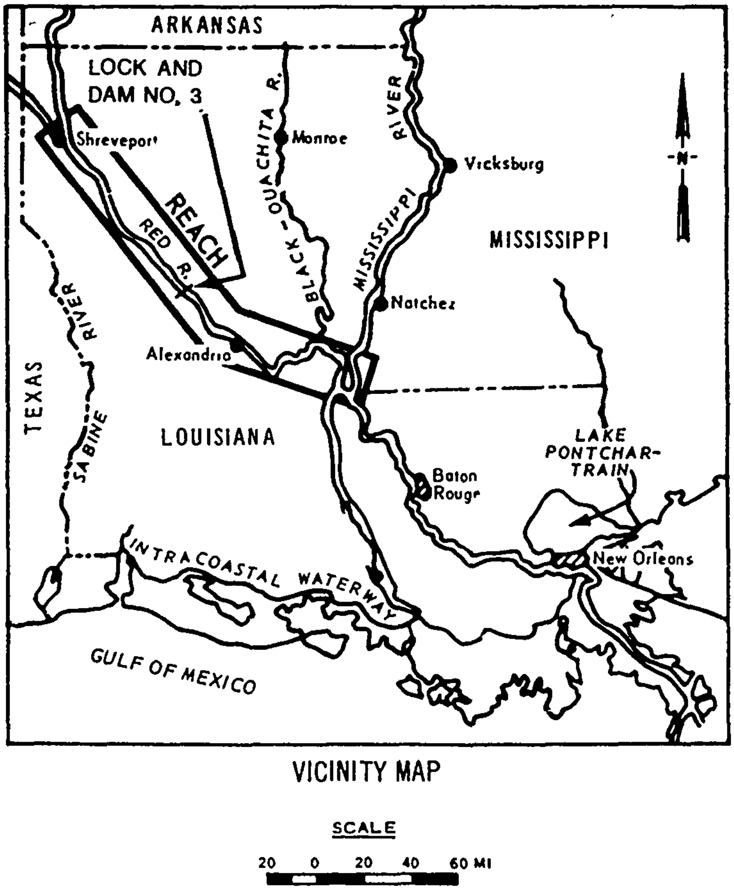

Figure 1. Vicinity map

RED RIVER WATERWAY, LOCK AND DAM NO. 3

STILLING BASIN AND RIPRAP REQUIREMENTS

Spillway Hydraulic Model Study

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 constructing a system of five locks and dams. Lock and Dam No. 3 will be located 38 miles upstream of Alexandria, LA, at 1967 river mile 141. The 1967 river mileage is based on preproject conditions, The location of the project is shown in Figure 1.

Pertinent Project Features

2. The principal structures associated with Lock and Dam No. 3 will consist of a navigation lock, a gated spillway, concrete abutment walls, and an overflow weir, with an optional hydropower facility. The lock, with nominal chamber dimensions of 84 by 785 ft, pintle to pintle, and usable chamber dimensions of 84 ft wide and 685 ft long, will be on the left riverbank looking downstream. The lift will vary up to a maximum of 31 ft.

3. The navigation dam will contain six 42-ft-high by 60-ft-wide tainter gates mounted between 9-ft-wide piers. The gate sill will be at el 55.0.** The tops of the gates, when closed, will be at el 97.0, which will provide a 2-ft freeboard above the normal upper pool elevation of 95.0. The net width

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

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

of the spillway is 360 ft, and the gross width of the abutments from face to face is 405 ft. Plate 1 shows the original (type 1) design spillway and stilling basin portion of the dam. The hydropower facility may be added to Lock and Dam No. 3 after construction of the lock and dam.

4. The postproject tailwater rating curve is shown in Plate 2. All references to normal tailwater are based on this curve. All references to minimum tailwater are based on a tailwater elevation of 64.0 at the downstream end of the model (sta 25+00).

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 when 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 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.

PART II: THE MODEL

Description

6. The investigation was conducted in a 1:50-scale model which reproduced the gated spillway, the navigation lock, upstream guard wall, downstream guide wall, and overflow weir, as shown in Figure 2. A 1,400-ft length of upstream and a 2,650-ft length of downstream topography were reproduced. The approach area was molded in pea gravel. The spillway weir, tainter gates, gate piers, lock, and overflow weir were fabricated of sheet metal. The stilling basin and its elements were of wood treated with a waterproofing compound to prevent expansion. Initially, the downstream area was molded in pea gravel 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. The 1:50-scale model reproduced all pertinent topography within the channel. Only a portion of the overbanks adjacent to the channel was reproduced in the model. Large discharges with significant overbank flow could not be accurately simulated in this model.

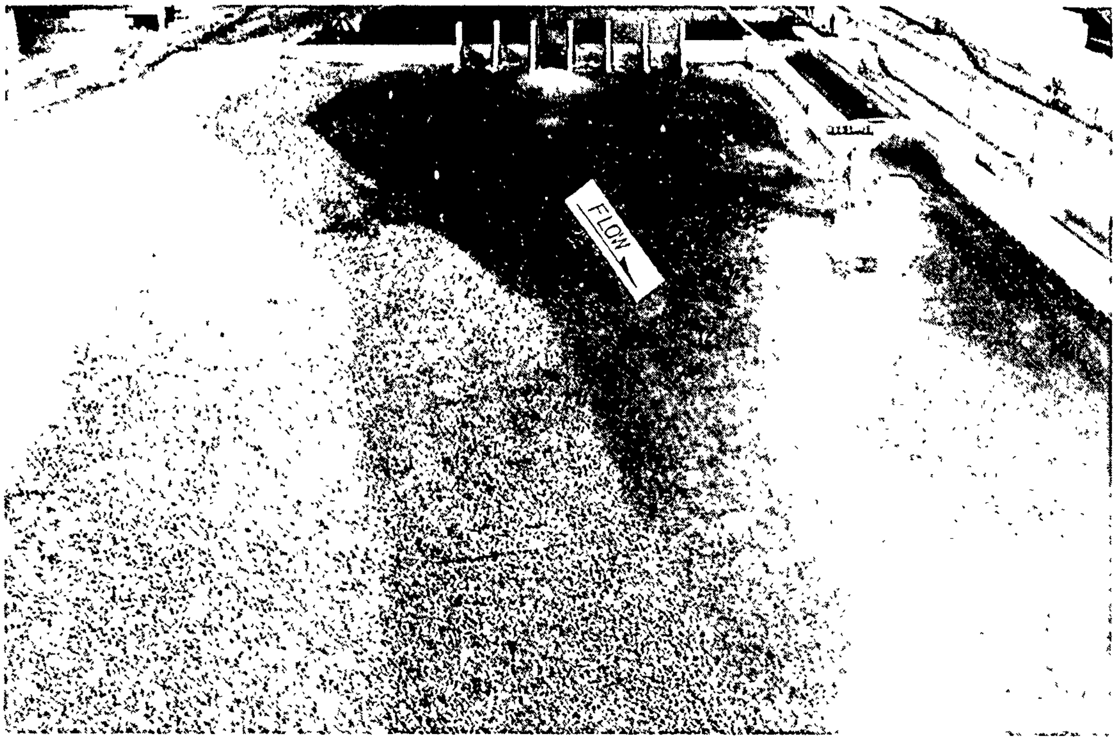
7. Discharges were measured with venturi meters, and water-surface elevations were measured with point gages. Sand and riprap scour depths were measured with point gages, and velocities were measured with a pitot tube or propeller meter. Steel rails set to grade along the sides of the flume provided 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 shown in the following tabulation.

9. Model measurements of discharge, water-surface elevation, and velocities can be transferred quantitatively to prototype equivalents by means of these scale relations.

a. Approach currents



b. Exit currents

Figure 2. Full-gate operation of gate 4, pool el 95, tailwater el 67, 14 sec (prototype)

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

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

PART III: TESTS AND RESULTS

10. The upper pool elevation, number of gates, stilling basin geometry, downstream sediment exclusion dikes, and hydropower options of Lock and Dam No. 3 have been modified. This report does not document every change made to the structure, but provides the information required to document performance of the recommended plan. Every plan that is presented in this report has the following common features:

- a. Six gates with spillway crest shown in Plate 1
- b. Normal upper pool el 95
- c. Minimum lower pool el 64
- d. Unless stated otherwise, headwater (HW) and tailwater (TW) elevations were measured at sta 12+00 upstream and sta 25+00 downstream, respectively

Many of the changes from the original design to the recommended design were not a result of findings in this spillway model. Studies were being conducted concurrently with this study in the sedimentation and the navigation models, and results from these studies brought about significant changes in the spillway model (Report 3 in this series (O'Neal, in preparation), and Report 2 in this series (Wooley, in preparation)).

Crest Pressures

11. Crest pressures were measured with the original (type 1) design (Plate 1) for half-opened and fully opened gates and pool elevations of 95 and 97. Results are shown in Plates 3-6. Pressures were sufficiently high to prevent cavitation problems on the downstream face of the crest.

Stilling Basin and Riprap Design

12. The following guidelines for stilling basin design are set forth in Engineer Manual (EM) 1110-2-1605, "Hydraulic Design of Navigation Dams" (Headquarters, US Army Corps of Engineers (HQUSACE), 1987):

- a. Uniform discharge through all spillway gates for a range of headwaters and tailwaters expected during project life.
- b. Single gate fully opened with normal headwater and minimum

tailwater. This condition would assume gate misoperation or marine accident. Minor damage to the downstream scour protection may occur as long as the integrity of the structure is not jeopardized. Single gate fully opened with above-normal pool (perhaps the 50- to 100-year pool) should also be given consideration. This condition would simulate loose barges that could block several gates causing above-normal pools as occurred at Arkansas River Lock and Dam No. 2 during December 1982.

c. **Single gate opened sufficiently wide to pass floating ice or drift at normal headwater and minimum tailwater. During preliminary design, a gate half opened can be assured to approximate ice- or drift-passing conditions. Final design usually requires model studies to determine the proper gate opening. No damage should occur for this condition. For most low-head navigation structures, conditions b and c result in free flow over the crest.**

The Lock and Dam No. 3 project was designed to meet all three guidelines. With the exceptions of riprap gradations B and C of the US Army Engineer Division, Lower Mississippi Valley (LMVD), riprap gradations come from Table 5-3 of EM 1110-2-1605 or Engineer Technical Letter (ETL) 1110-2-120 (HQUSACE 1971). The size used in the model for each gradation is shown in Table 1. Model sizes were chosen to reproduce the lower or minimum gradation curves.

13. The type 1 (original design) stilling basin was tested with the type 1 riprap plan (Plate 1) with a single gate. Results were as follows:

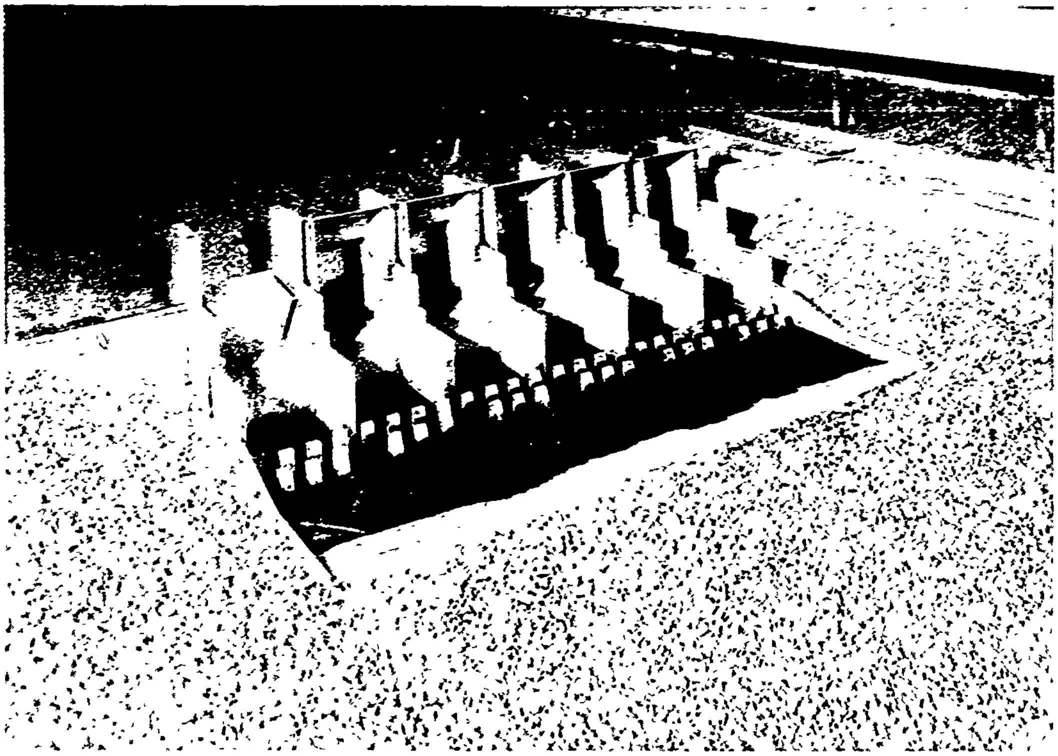
Gate Opening	Upper Pool El	Tailwater El	Test Duration (Prototype Time, hr)	Results
Half*	95	64.0	28	Stable
Full	95	66.8	28	Failed
Full	95	72.0	28	Stable
Full	95	66.8	2	Rock movement but no failure
Full	93	66.0	28	Stable
Half*	97	64.0	28	Stable

* 20 ft.

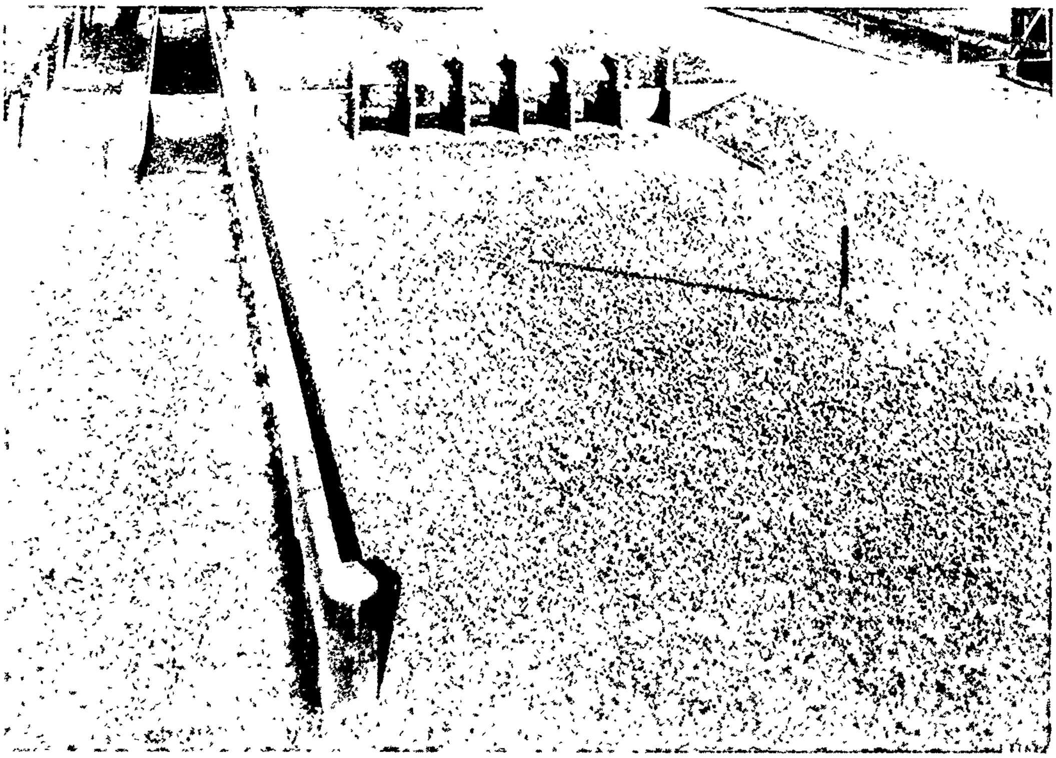
14. Since the type 1 stilling basin with the type 1 riprap design was not stable for extended runs of the single gate fully opened and minimum tailwater, modifications were required. Stilling basin modifications were necessary because the 81-in. riprap used in the type 1 design is the largest riprap that can reasonably be obtained. In the type 2 stilling basin (Plate 7), the basin apron elevation was lowered from el 31 to el 28 and the basin length was increased by 35 ft. The type 2 stilling basin with the type 2 riprap plan

(Plate 8) remained stable for extended runs of the single gate fully opened at normal upper pool and minimum tailwater. The riprap plan was changed from type 1 to type 2 because the longer and lower type 2 stilling basin required that changes be made to the exit channel and the riprap. The type 2 stilling basin is the recommended design.

15. Results from the sedimentation model (O'Neal, in preparation) showed that the exit channel required significant modifications to prevent the sedimentation problems that occurred at the Red River Lock and Dam No. 1 prototype. The recommended plan showing the realigned right bank, the sediment dikes, the extended downstream guide wall, and the recommended type 3 riprap design are shown in Plate 9 and Figure 3. Also shown is the replacement of the three separate 1V on 25H longitudinal slopes in the type 2 riprap plan with a single 1V on 25H longitudinal slope beginning at the downstream end of the stilling basin. The type 3 riprap plan shown in Plate 9 was stable for all uniform gate openings for the range of headwaters and tailwaters expected during the project life. The riprap plan was also stable for a single gate one-half open, normal upper pool, and minimum tailwater. The riprap plan sustained minor damage for a single gate fully open, normal upper pool, and minimum tailwater; but the integrity of the structure was not jeopardized. This damage occurred at (a) the top and toe of the right bank dike; (b) upstream end of the midchannel dike, and (c) upstream ends of 54- and 36-in. riprap. Damage to the dikes occurred with any of the six gates open. Damage to the 54- and 36-in. riprap was significant only when either gates 5 or 6 (numbered left to right looking downstream) were open. The riprap gradations in the model were chosen to reproduce the lower or minimum gradation curve. For the 54- and 36-in. ripraps, this was not possible, and the gradation used in the model was lower than the lower limit of the prototype gradation. The d_{50} in the model was 1.69 and 0.98 ft for the 54- and 36-in. ripraps, respectively. The $d_{50(min)}$ in the prototype is 1.75 and 1.17 ft, respectively, for the 54- and 36-in. ripraps. The smaller size used in the model means that the minor failure observed in the model will be less significant in the prototype. Test duration was 32 hr (prototype) with tailwater maintained at the minimum or lower pool elevation of 64. In the prototype, conditions will be less severe because the tailwater will build up to a normal tailwater elevation of 76.3. At this tailwater, no damage occurred to the riprap in the model for the single gate fully open and normal upper pool. An intermediate tailwater



a. Type 2 stilling basin



b. Approach channel

Figure 3. Recommended plan (Continued)

c. Exit channel

Figure 3. (Concluded)

elevation of 71.0 was tested with the single gate fully open and normal upper pool. Minor damage occurred on the top of the right bank dike and the upstream end of the midchannel dike. A cover layer of larger stone such as the 48-in. stone used in the nose of the right bank dike should be considered for these areas.

16. Although the model demonstrated that the standard LMVD C stone placed downstream of sta 21+00 was stable, results may not be valid because the model rock was smaller than the size normally used in riprap stability investigations. To address this problem, velocities were measured in the exit channel for discharges of 100,000 and 150,000 cfs at normal tailwater and 48,600 cfs at minimum tailwater. Results are shown in Plates 10-12. Based on these velocities and recent riprap research results, the C stone would fail with the single gate fully open, normal upper pool, and minimum tailwater. The C stone is borderline for a discharge of 150,000 cfs and stable with a discharge of 100,000 cfs with normal tailwater. An equal blanket thickness of a well-graded stone such as the 18-in.-thick gradation given in Table 5-2 of EM 1110-2-1605 (HQUSACE 1987) would be stable for the normal tailwater flows but unstable for the single gate fully open and minimum tailwater flow. As discussed in paragraph 15, tailwater buildup in the prototype will cause the single-gate condition to be less severe than in the model where the tailwater was held constant.

17. Rock immediately downstream of the overflow weir (Plate 9) remained stable for all normal headwater and tailwater combinations. Tests were run to evaluate the effects of flow over the overflow weir only; all gates were closed and the tailwater was at el 64. The minimum discharge that can be measured in the model, 5,000 cfs, was passed across the overflow weir. After passing over the weir, the flow concentrated and moved toward the channel. As the flow passed down the 1V on 3H slope, rock in the 81-in.-thick riprap began moving. A portion of the flow stayed on the overbank and passed downstream of the riprap protection below the overflow weir. This flow also concentrated at the point of return to the channel (sta 2+50) and caused failure of the 81-in. riprap at the top of the bank. These tests were conducted with the type 1 design overflow weir having a crest width of 16 ft. Results are considered adequate for use with the 4-ft-wide crest in the recommended plan. Free flow over the overflow weir could be minimized by making the top of gates 0.5 to 1.0 ft lower than the overflow weir. Although large amounts of flow over the

gates should not be allowed, flow observed resulting from a head of approximately 0.5 ft caused no problems at one of the Arkansas River lock and dam projects.

18. The 36-in.-thick riprap placed adjacent to the upstream ported guard wall was stable for all normal flow conditions as well as the following hinged pool conditions: a 100,000-cfs discharge and a headwater el of 88; and a 120,000-cfs discharge and a headwater el of 89. These tests were run with the berm and inflow distribution described in paragraphs 20 and 22.

19. The 81-in.-thick riprap placed upstream of the structure remained stable for all normal flows as well as for the two hinged pool conditions described in the preceding paragraph. Smaller sizes were not tested because large riprap upstream of the structure reduces the damage that can occur when barges break loose and impinge on the gate piers.

20. A disposal area dike was placed in the model as shown in Plate 9. Under overbank flow conditions, a concentration of flow existed at the upstream corner of the dike. Velocities on the overbank adjacent to the dike were 5-6 fps. Riprap protection for the portion of the dike adjacent to the channel should be considered. The standard LMVD C stone should be stable based on the observed velocities.

Approach Channel

21. The type 1 approach channel bottom was at el 59, as shown in Plate 1. Results from the sedimentation model (O'Neal, in preparation) showed the need to raise the channel bottom elevation to el 64. In addition, a berm with top elevation at 73 was placed along the left descending bank just upstream of the ported guard wall. Both the berm and the el 64 bottom were used in evaluating the recommended plan shown in Plate 9.

Flow Distribution Through Upstream Ported Guard Wall

22. Tests were conducted to determine the flow distribution through the upstream ported guard wall. The upstream end of the guard wall was at sta 9+83 with the top of the parapet wall at el 106.5. These tests were conducted with the approach channel bottom at el 64 and with the original top and bottom port elevations of 72 and 59, respectively. (The top and bottom port elevations in the recommended plan were changed to 78 and 64, respectively, based on results from the navigation study (Wooley, in preparation).) The

flow distribution tests were conducted to address the following:

a. Percent of total flow passing behind upstream guard walls Q_g, with and without the berm along the left descending bank.

b. Percent of Q_g passing through each port of the upstream guard wall with and without berm.

23. Before Q_g could be determined, the upstream baffling in the 1:50-scale structures model had to be adjusted to reproduce the correct flow distribution with and without the berm. The navigation model (Wooley, in preparation) provided the velocity distribution for the design with the berm. The floats used to determine velocities in the 1:100-scale navigation model were submerged to a depth of 8 ft. Velocities were measured at a position 8 ft below the water surface in the 1:50-scale structures model. Due to uncertainty about the velocity represented by the floats in the navigation model and the short distance available for flow development in the structures model, the magnitude of the velocities were not similar and only the shapes of the lateral water velocity distributions were compared. The comparison at sta 12+00 with the berm is shown in Plate 13. No navigation model velocity distributions were available for the plan without the berm. Velocities without a berm based on a numerical model of John H. Overton Lock and Dam (Copeland, in preparation) are compared to the flow distribution in the structures model for 145,000 cfs in Plate 14.

24. To determine Q_g and Q_r (flow in the main river channel) by means of the subject model, detailed velocity measurements were taken across the channel and discharge was computed using the corresponding area multiplied by the measured velocity. Results were as follows:

Berm	Q (Inflow) cfs	Q_g cfs	Q_r cfs	$\frac{Q_g + Q_r}{Q \text{ (Inflow)}}$	$\frac{Q_g}{Q_r + Q_g}$	$\frac{Q_r}{Q_r + Q_g}$
With	90,000	13,600	75,300	0.988	0.15	0.85
	125,000	21,400	99,500	0.967	0.18	0.82
	145,000	22,600	120,000	0.983	0.16	0.84
Without	90,000	20,500	68,500	0.989	0.23	0.77
	145,000	31,800	112,100	0.992	0.22	0.78

25. To determine the percent of flow through each port of the upstream guard wall, velocity measurements were used; but accurate definition of the mean velocity and the effective flow area was difficult due to the velocity distribution across the port. Velocities within the individual port cross

section varied in magnitude and direction from top to bottom and from side to side. Dye injections at the upstream ports showed that flow through the port was highly skewed with respect to a vertical plane normal to the face of the guard wall. Dye injections at the downstream ports showed that flow through the port was almost normal to the face of the guard wall. This was due to the decreased approach velocity to the downstream ports. The effective area in the upstream ports was relatively small; the opposite was true in the downstream ports. Velocities were measured at el 65.5 (average of top and bottom elevations of port) for two locations within the main flow through the port as shown in Plate 15. The flow lines shown in Plate 15 are typical of the middle ports of the guard wall. Dye injections were used to define the correct location and angle of placement for the velocity probe at each port. Dye injections were also used to estimate the effective flow area. For the downstream ports, 90 percent of the gross port area was used for the effective area because of the relatively uniform distribution of flow through the ports. A linear decrease in port area was used for the upstream ports as shown in Plate 16. The amount of decrease was varied until continuity of the flow was satisfied. This resulted in an effective area of 40 percent of the gross area for the upstream port. This effective area was reasonable based on the dye injections. Results are shown as follows:

		Percent of Q_g	
Port	Effective Port Area sq ft	With Berm Q = 125,000 cfs	Without Berm Q = 145,000 cfs
1*	218	3.0	3.4
2	239	3.4	3.8
3	259	4.1	4.5
4	281	4.6	4.9
5	302	5.0	5.5
6	323	5.8	6.1
7	344	6.4	6.8
8	365	7.1	7.4
9	386	7.7	8.0
10	407	8.5	8.5
11	433	9.1	9.1
12	449	9.6	9.3
13	470	10.0	9.3
14	491	10.3	9.0
14.5**	246	5.4	4.4

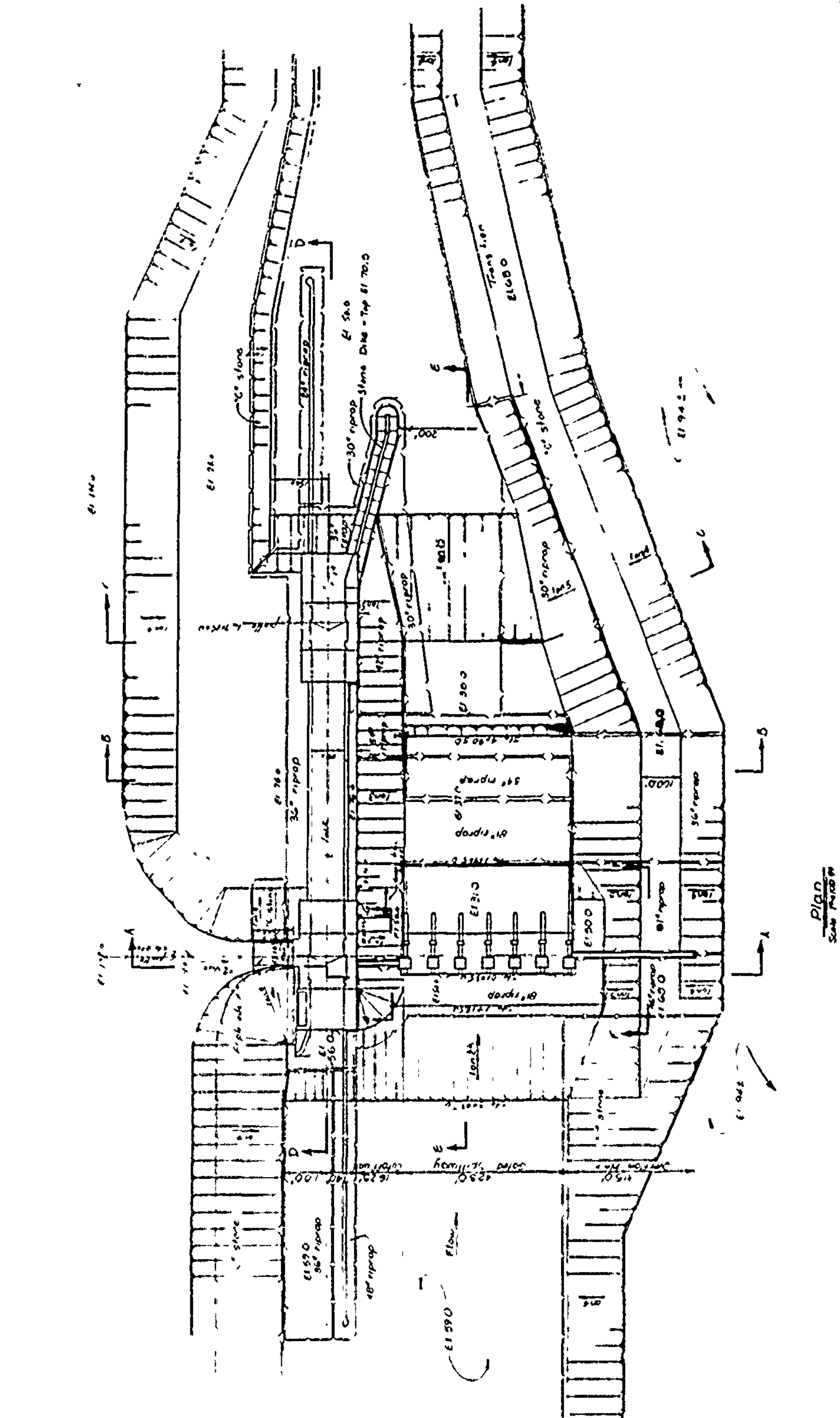
* Upstream.
** Downstream.

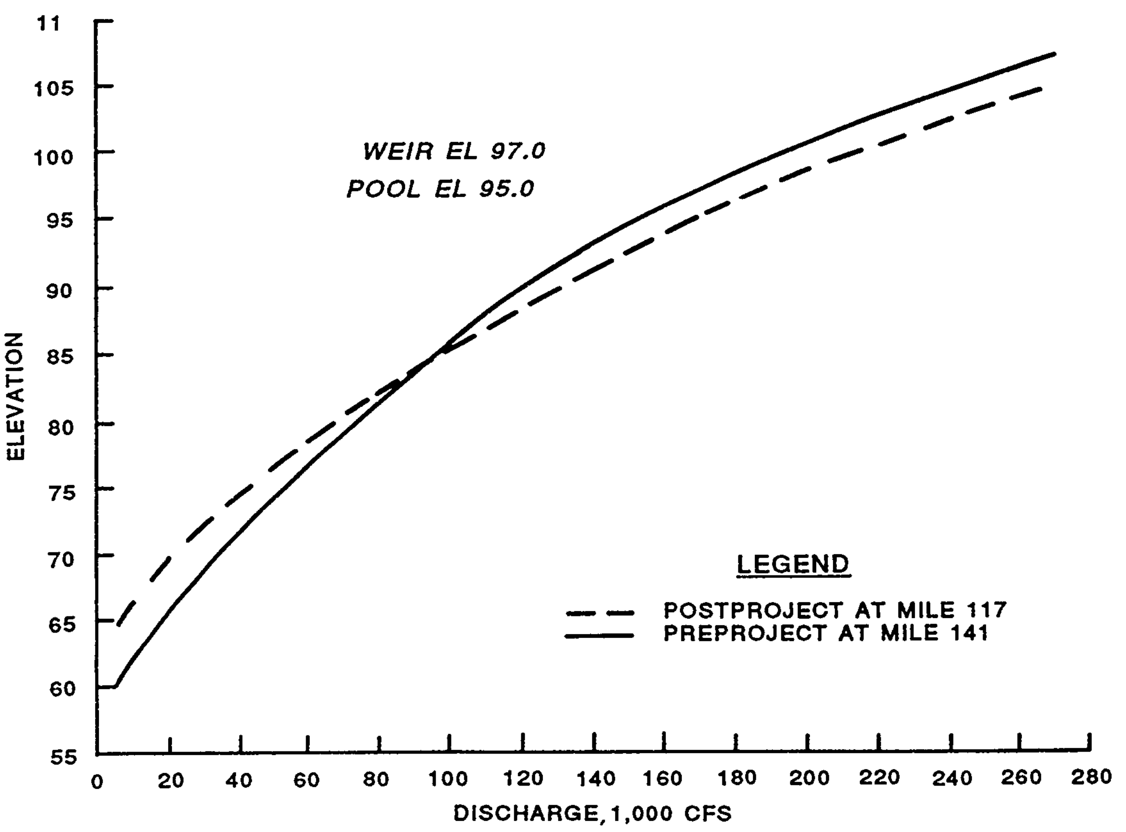
HW EL at Sta 12+00	TW El at Sta 25+00	Gate Opening ft	Q , cfs	Gate Lip El	Water-Surface El at Gate	Submergence, ft	Normal TW El
95	64	All - 2	24,200	57.0	64.9	7.9	71.0
95	64	All - 4	44,000	59.0	69.7	10.7	75.1

Note that at normal tailwater the submergence would be about 6 ft greater.

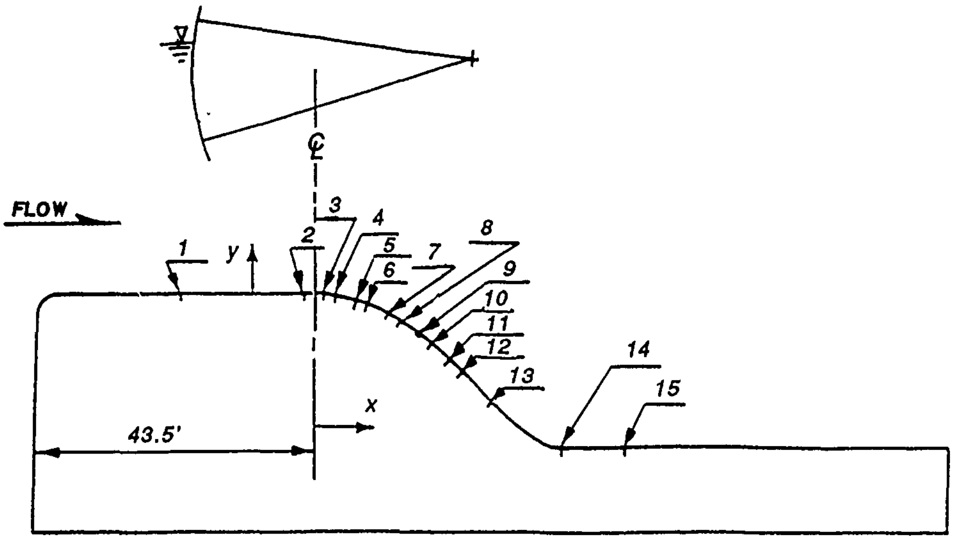
32. A plot of rating curves showing the relationship between discharge, tailwater, and gate opening for an upper pool elevation of 95 is shown in Plate 25. The solid lines are the ratings determined using the procedures set forth in EM 1110-2-1605 (HQUSACE 1987). The EM procedure is based on tailwater near the structure. The data points shown in Plate 25 were taken from the physical model (Table 1) but had to be adjusted for the difference between the tailwater near the structure and the tailwater at sta 25+00. Tailwater differences were obtained from Plates 19-24 and are shown in Plate 26. The model data were also adjusted for the difference in headwater due to the larger roughness in the model (see paragraph 28 and Plate 26). Some extrapolation and interpolation were required to compare all data for a pool elevation of 95.0.

ORIGINAL
(TYPE 1)
DESIGN





PREPROJECT AND POSTPROJECT
TAILWATER RATING CURVES

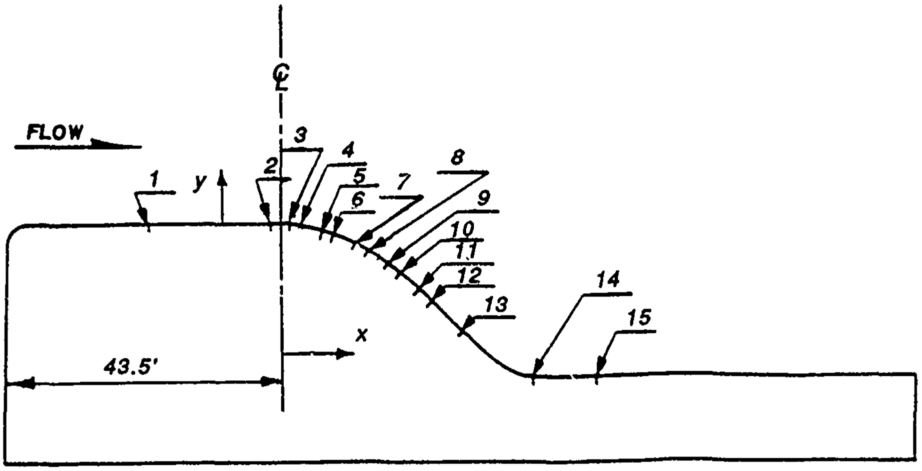


PIEZOMETER* NO.	EL	DISTANCE FROM CENTER LINE X, FT	PRESSURE FT OF WATER
1	55	-20.5	39.9
2	55	-1.5	3.5
3	54.8	1.5	-1.3
4	54.5	3.5	-5.4
5	53.7	6.5	-9.7
6	53.0	8.5	-8.1
7	51.6	11.5	-7.6
8	50.5	13.5	-6.1
9	48.5	16.5	-0.9
10	47.0	18.5	4.5
11	44.4	21.5	12.7
12	42.5	23.5	20.4
13	38.2	27.9	29.3
14	31.0	38.9	41.0
15	31.0	48.9	38.0

* PIEZOMETERS LOCATED ALONG GATE CENTER LINE

NOTE: TAILWATER ELEVATION ESTABLISHED AT TAILGATE
PIEZOMETERS 3-12 FOLLOW THE EQUATION
$$X^2 = -50y$$

SPILLWAY CREST PRESSURES
TYPE 1 DESIGN SPILLWAY
GATE NO. 6 ONE-HALF OPEN
DISCHARGE 33,000 CFS
POOL EL 95
TAILWATER EL 64.0 (MIN)

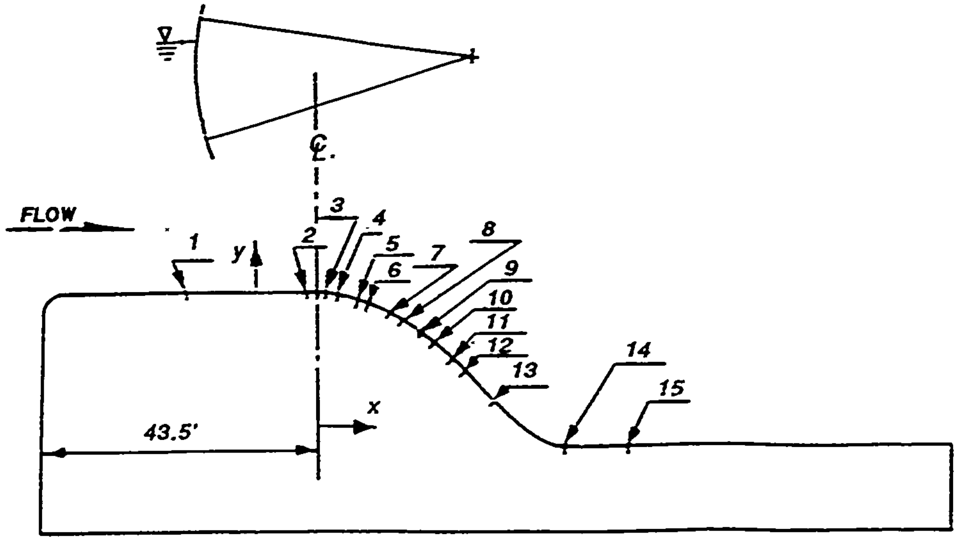


PIEZOMETER* NO.	EL	DISTANCE FROM CENTER LINE X, FT	PRESSURE FT OF WATER
1	55	-20.5	26.3
2	55	-1.5	7.9
3	54.3	1.5	2.8
4	54.5	3.5	-1.3
5	53.7	6.5	-5.7
6	53.0	8.5	-4.5
7	51.6	11.5	-4.7
8	50.5	13.5	-2.8
9	48.5	16.5	2.7
10	47.0	18.5	8.1
11	44.4	21.5	15.6
12	42.5	23.5	23.5
13	38.2	27.9	31.5
14	31.0	38.9	46.0
15	31.0	48.9	40.0

* PIEZOMETERS LOCATED ALONG GATE CENTER LINE

NOTE: CHANNEL CONTROL IN MODEL
PIEZOMETERS 3-12 FOLLOW EQUATION
$x^2 = -50y$

SPILLWAY CREST PRESSURES
TYPE 1 DESIGN SPILLWAY
GATE NO. 6 FULLY OPEN
DISCHARGE 49,000 CFS
POOL EL 95
TAILWATER EL 66.8 (MIN)

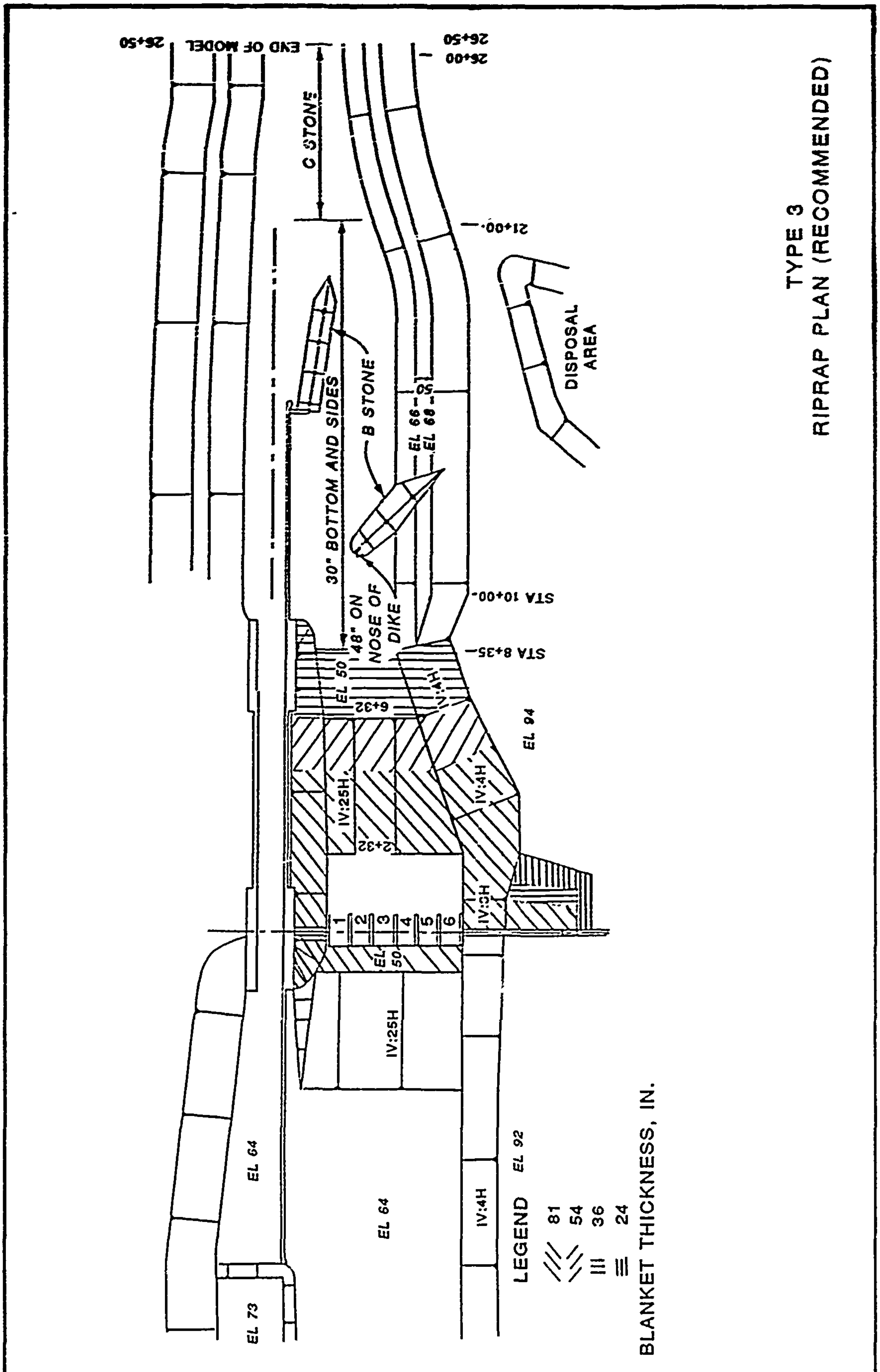


PIEZOMETER* NO.	EL	DISTANCE FROM CENTER LINE X, FT	PRESSURE FT OF WATER
1	55	-20.5	41.8
2	55	-1.5	0.60
3	54.8	1.5	-4.35
4	54.5	3.5	-8.90
5	53.7	6.5	-13.09
6	53.0	8.5	-12.15
7	51.6	11.5	-11.00
8	50.5	13.5	-9.85
9	48.5	16.5	-3.25
10	47.0	18.5	3.50
11	44.4	21.5	11.20
12	42.5	23.5	17.90
13	38.2	27.9	27.30
14	31.0	38.9	39.00
15	31.0	48.9	34.20

* PIEZOMETERS LOCATED ALONG GATE CENTER LINE

NOTE: CHANNEL CONTROL IN MODEL
PIEZOMETERS 3-12 FOLLOW THE EQUATION
$X^2 = -50y$

SPILLWAY CREST PRESSURES
TYPE 1 DESIGN SPILLWAY
GATE NO. 6 ONE-HALF OPEN
DISCHARGE 34,500 CFS
POOL EL 97
TAILWATER EL 64.1 (MIN)



TYPE 3
RIPRAP PLAN (RECOMMENDED)

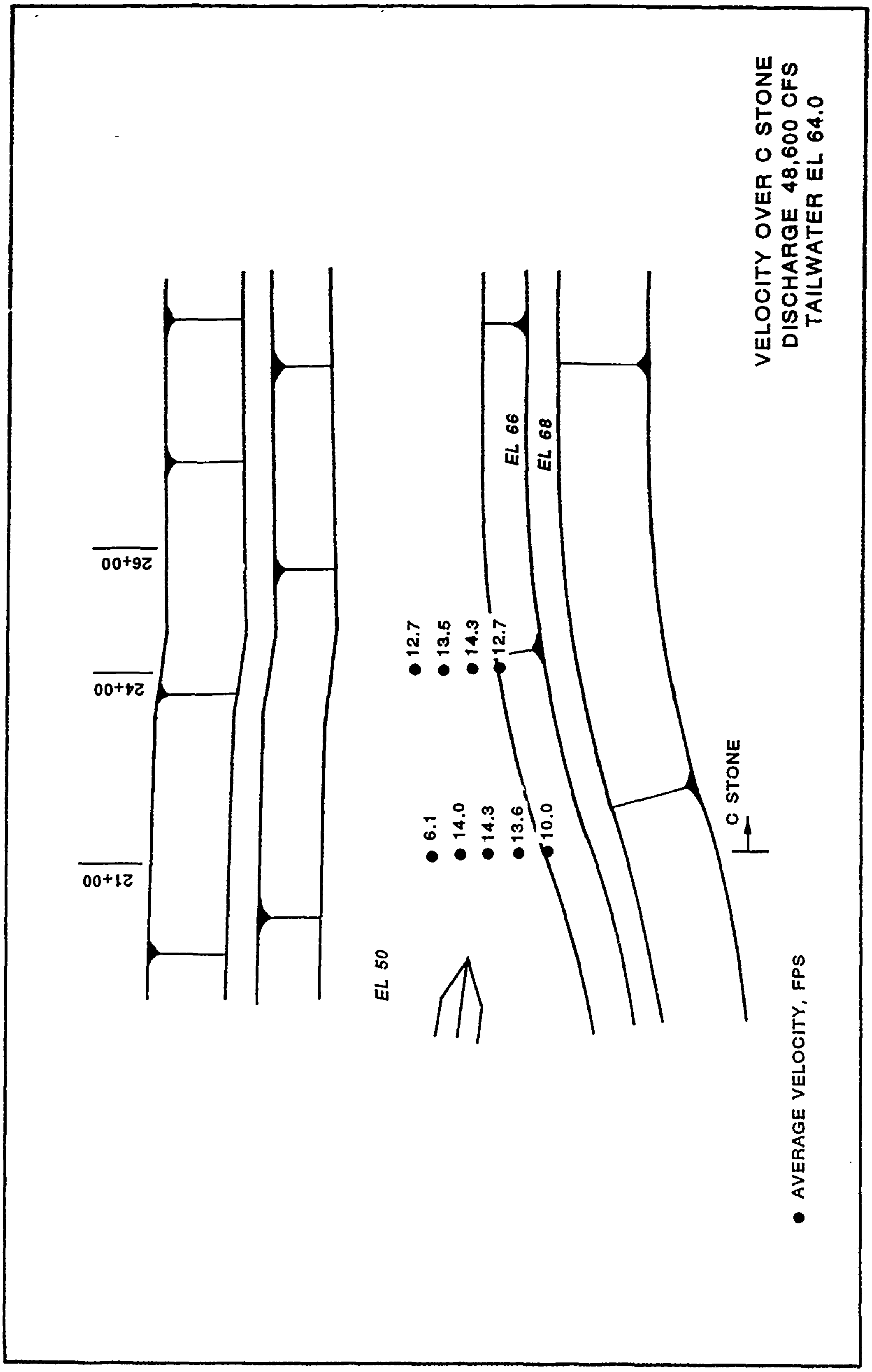
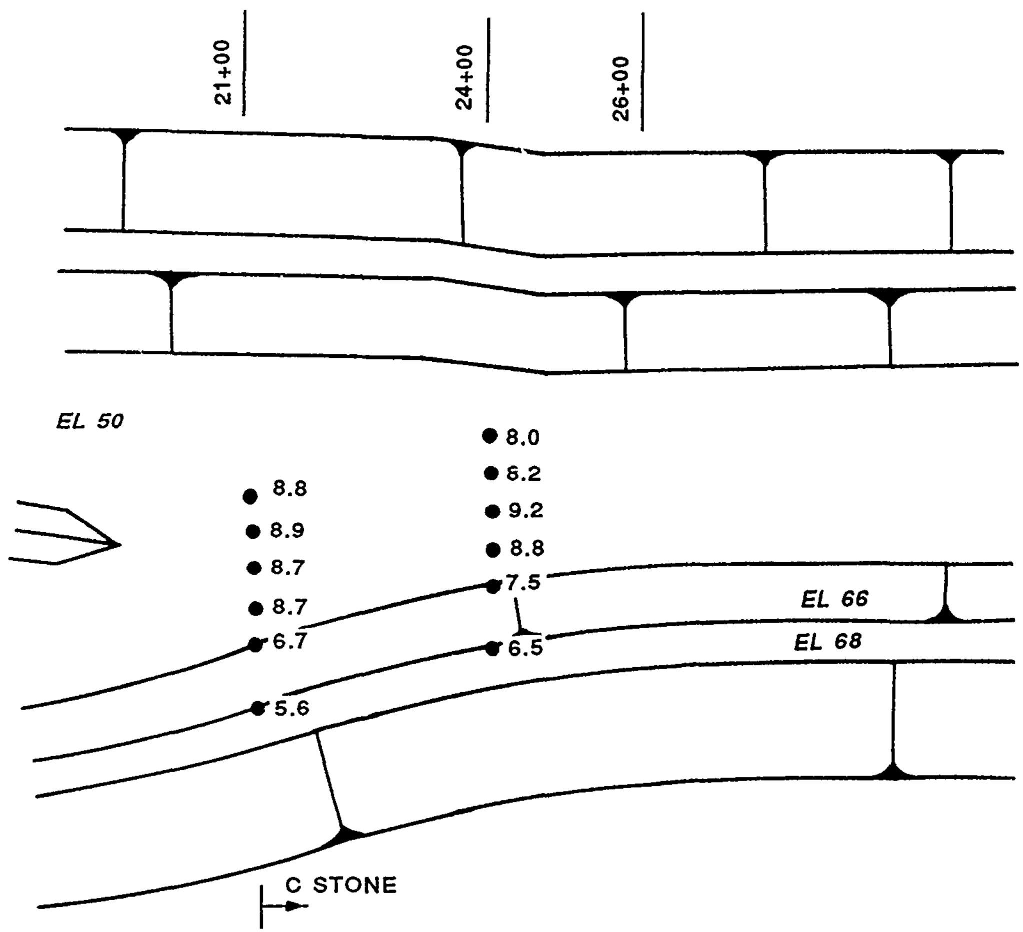

VELOCITY OVER C STONE
DISCHARGE 48,600 CFS
TAILWATER EL 64.0
21+00
24+00
26+00
EL 50
EL 66
EL 68
6.1
14.0
14.3
13.6
10.0
12.7
13.5
14.3
12.7
C STONE
● AVERAGE VELOCITY, FPS

PLATE 10



VELOCITY OVER C STONE
DISCHARGE 100,000 CFS
TAILWATER EL 85.1

● AVERAGE VELOCITY, FPS

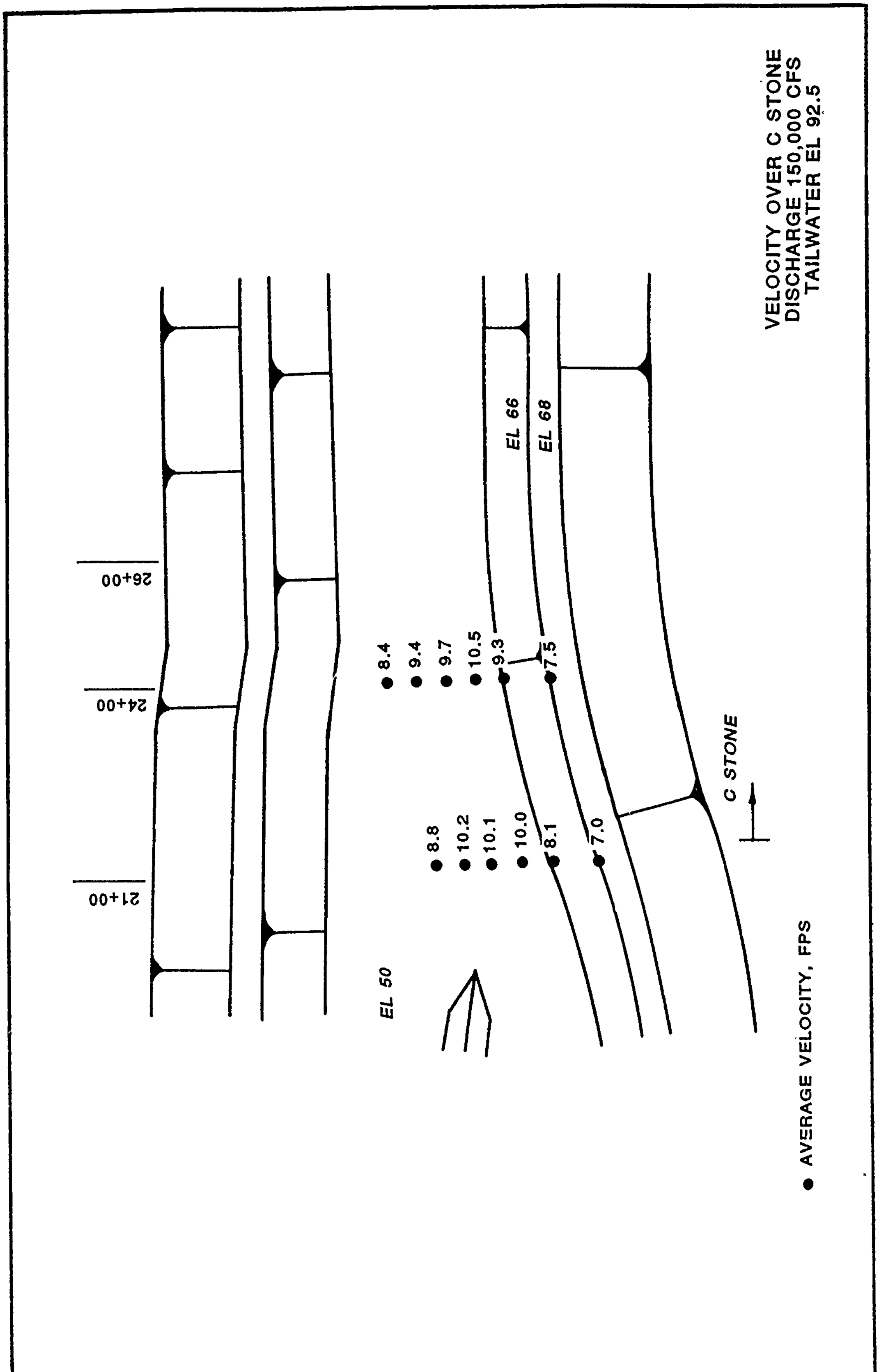
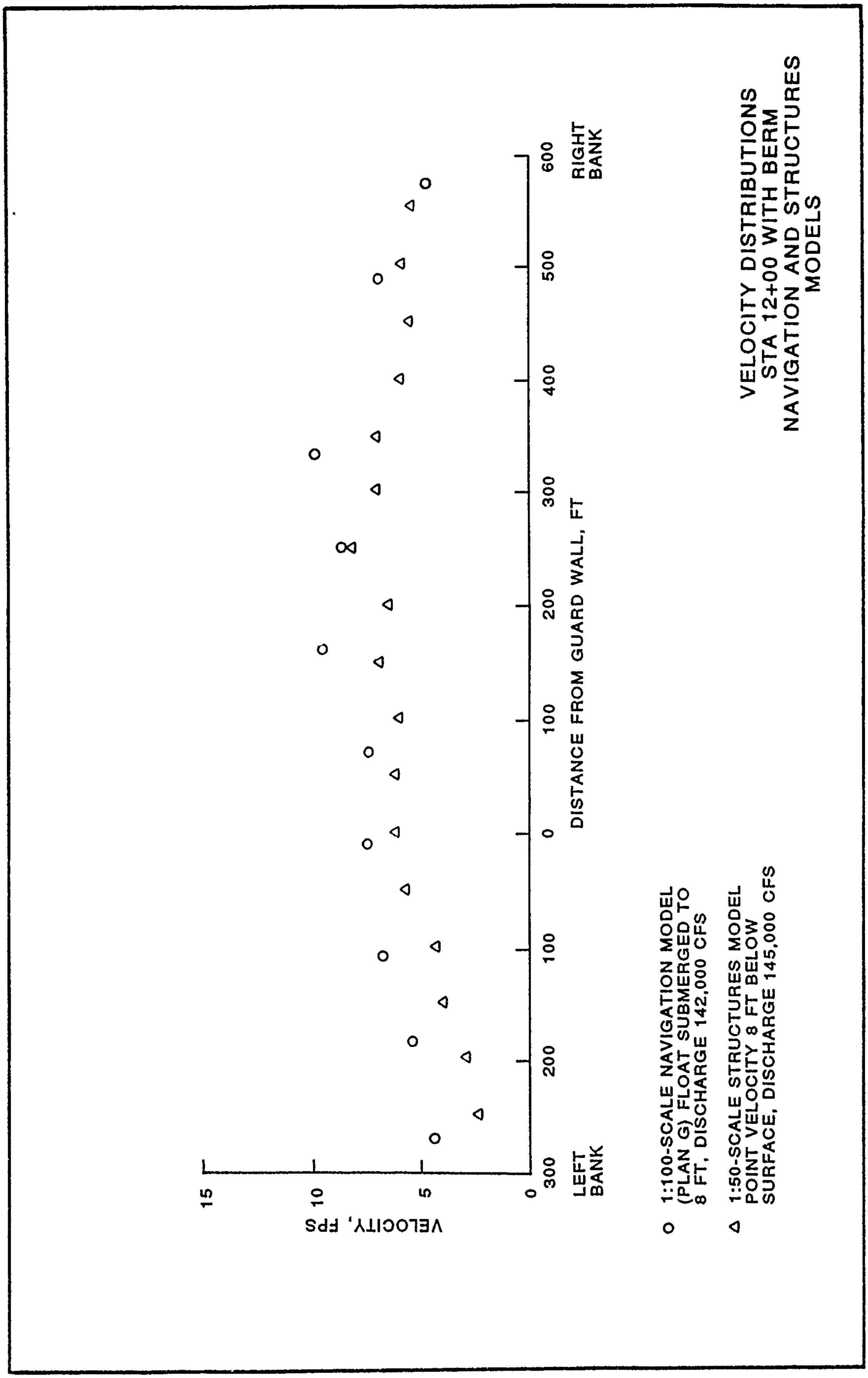
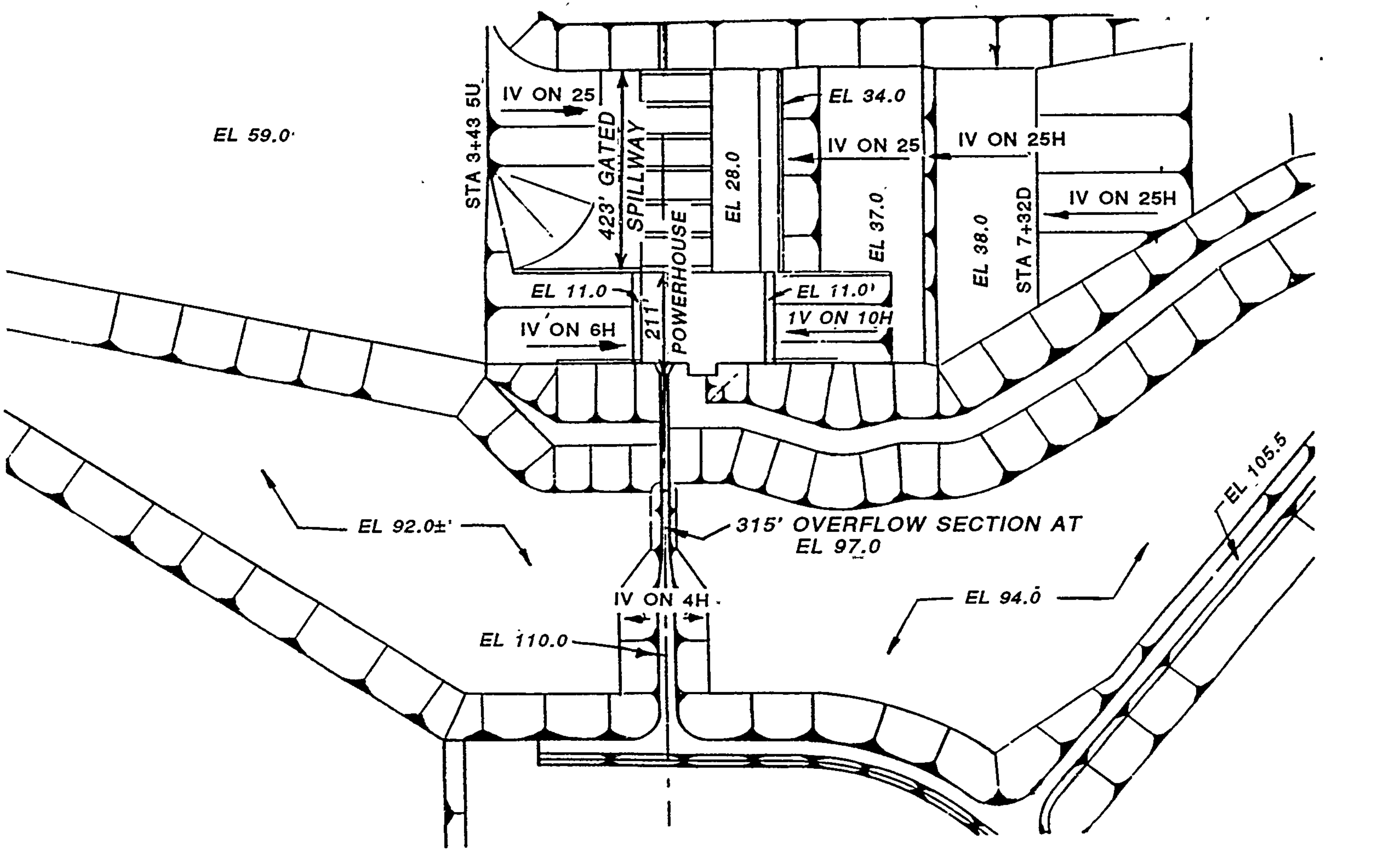
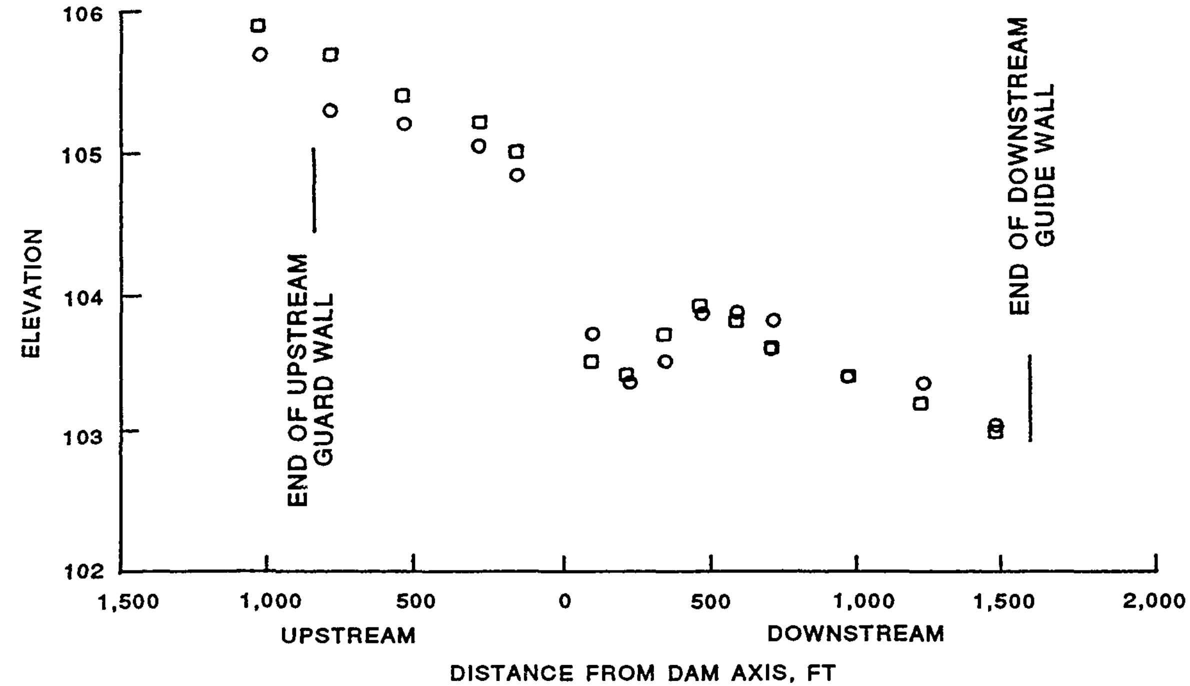

VELOCITY OVER C STONE
DISCHARGE 150,000 CFS
TAILWATER EL 92.5
EL 66
EL 68
EL 50
C STONE
21+00
24+00
26+00
8.4
9.4
9.7
10.5
9.3
7.5
8.8
10.2
10.1
10.0
8.1
7.0
● AVERAGE VELOCITY, FPS

PLATE 12


VELOCITY DISTRIBUTIONS
STA 12+00 WITH BERM
NAVIGATION AND STRUCTURES
MODELS
VELOCITY, FPS
15
10
5
0
DISTANCE FROM GUARD WALL, FT
300
200
100
0
100
200
300
400
500
600
LEFT BANK
RIGHT BANK
1:100-SCALE NAVIGATION MODEL (PLAN G) FLOAT SUBMERGED TO 8 FT, DISCHARGE 142,000 CFS
1:50-SCALE STRUCTURES MODEL POINT VELOCITY 8 FT BELOW SURFACE, DISCHARGE 145,000 CFS



PLAN WITH THREE-UNIT POWERHOUSE

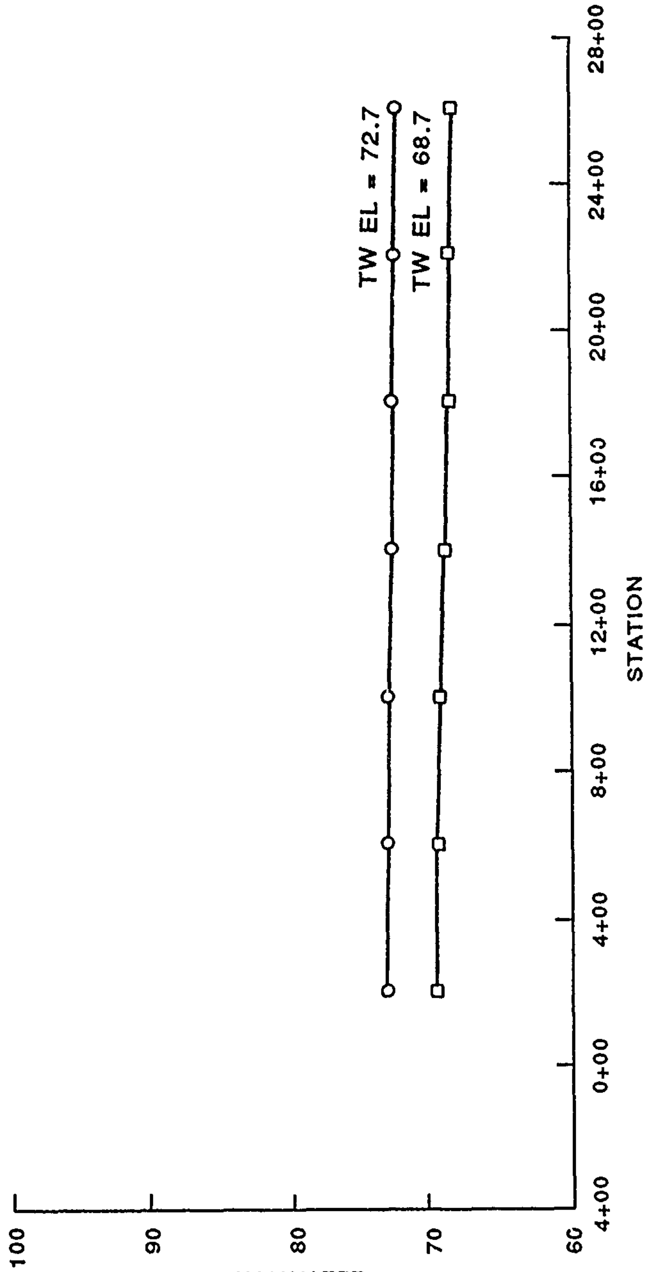


LEGEND

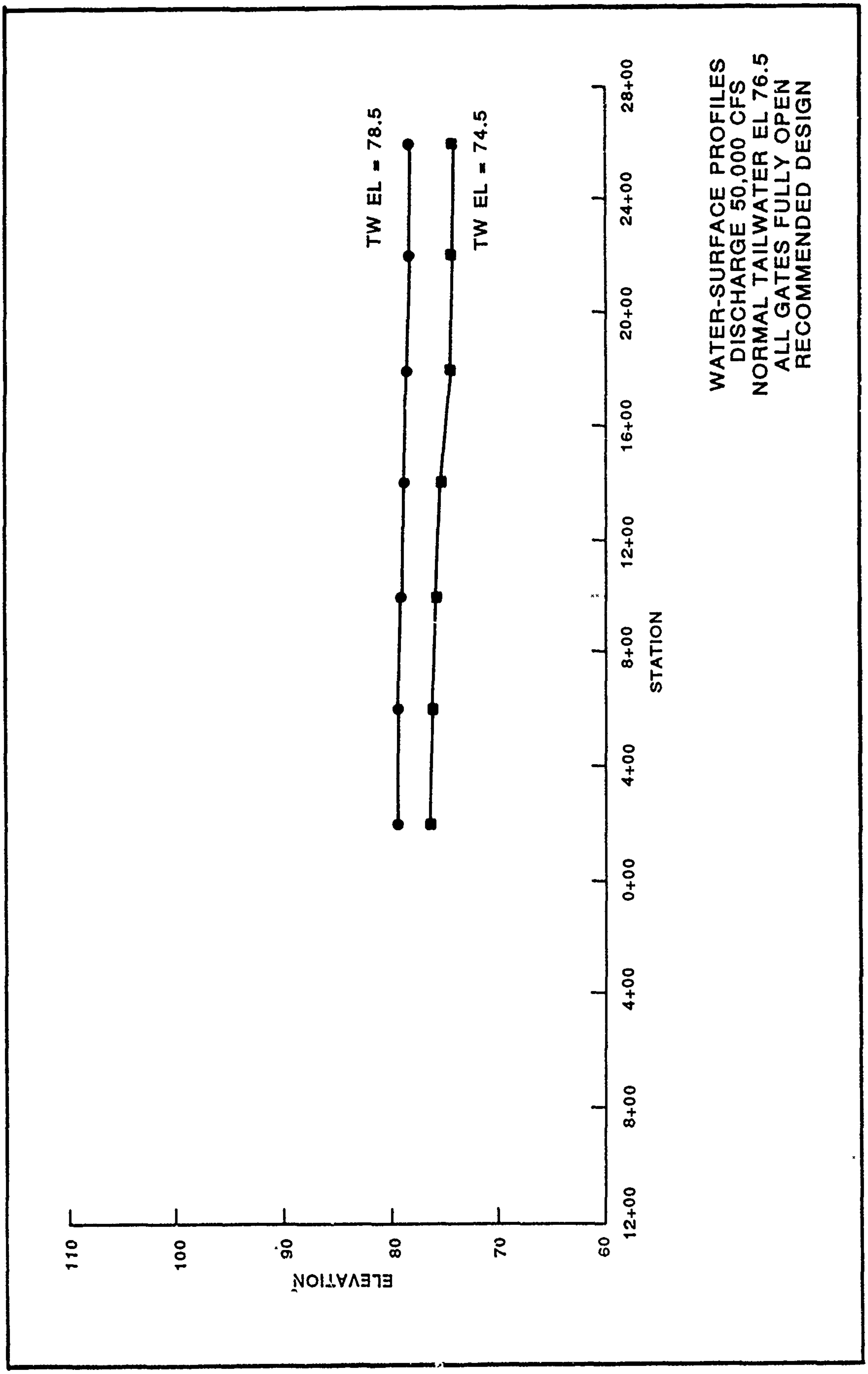
□ WITHOUT POWERHOUSE
○ WITH THREE-UNIT POWERHOUSE

NOTE: THE PROFILES WERE TAKEN BEFORE SEDIMENT DIKES WERE ADDED TO THE EXIT CHANNEL

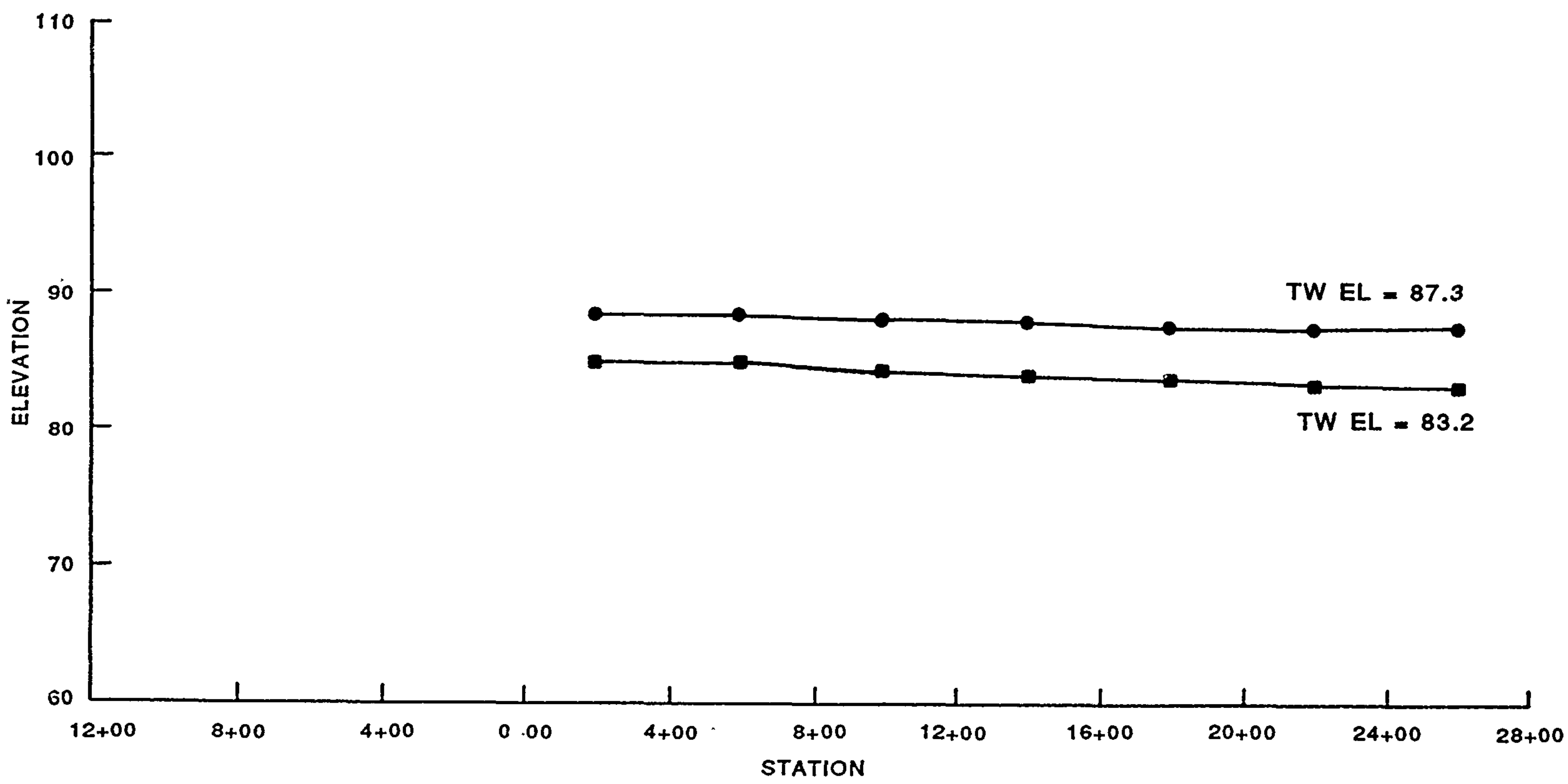
WATER-SURFACE PROFILES
DISCHARGE 248,600 CFS
FULLY OPENED GATES



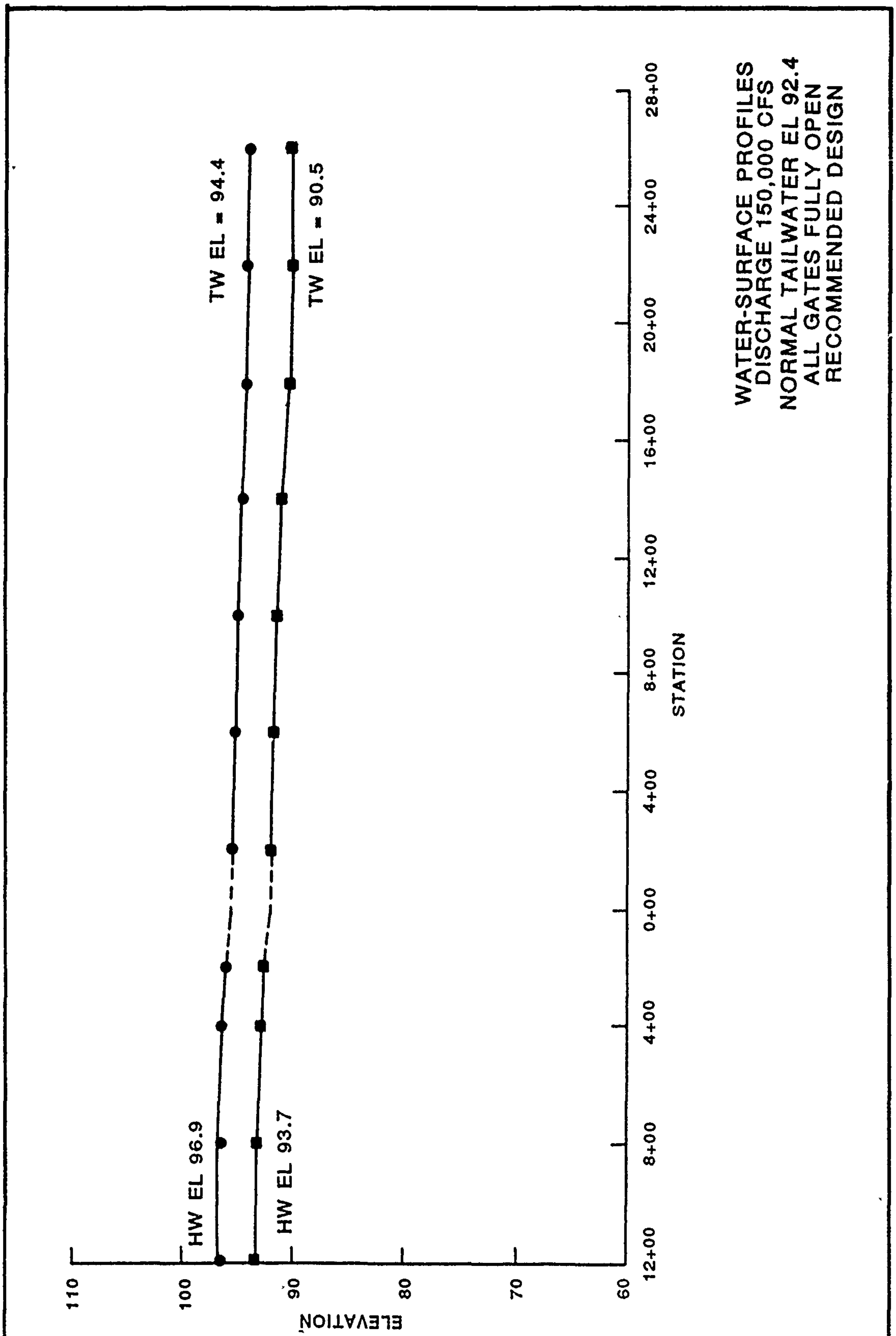
WATER-SURFACE PROFILES
DISCHARGE 25,000 CFS
NORMAL TAILWATER EL 70.7
ALL GATES FULLY OPENED
RECOMMENDED DESIGN



WATER-SURFACE PROFILES
DISCHARGE 50,000 CFS
NORMAL TAILWATER EL 76.5
ALL GATES FULLY OPEN
RECOMMENDED DESIGN

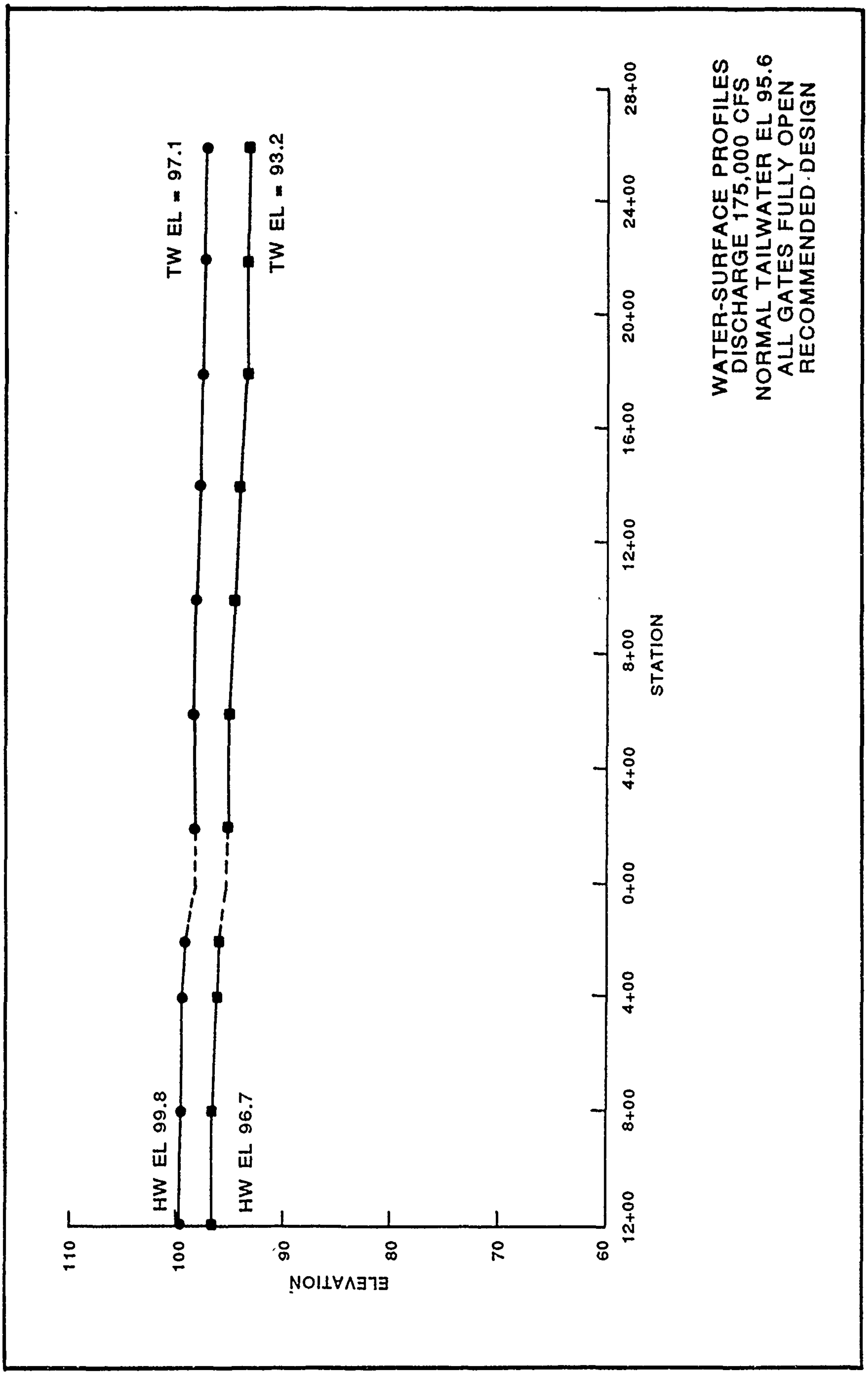


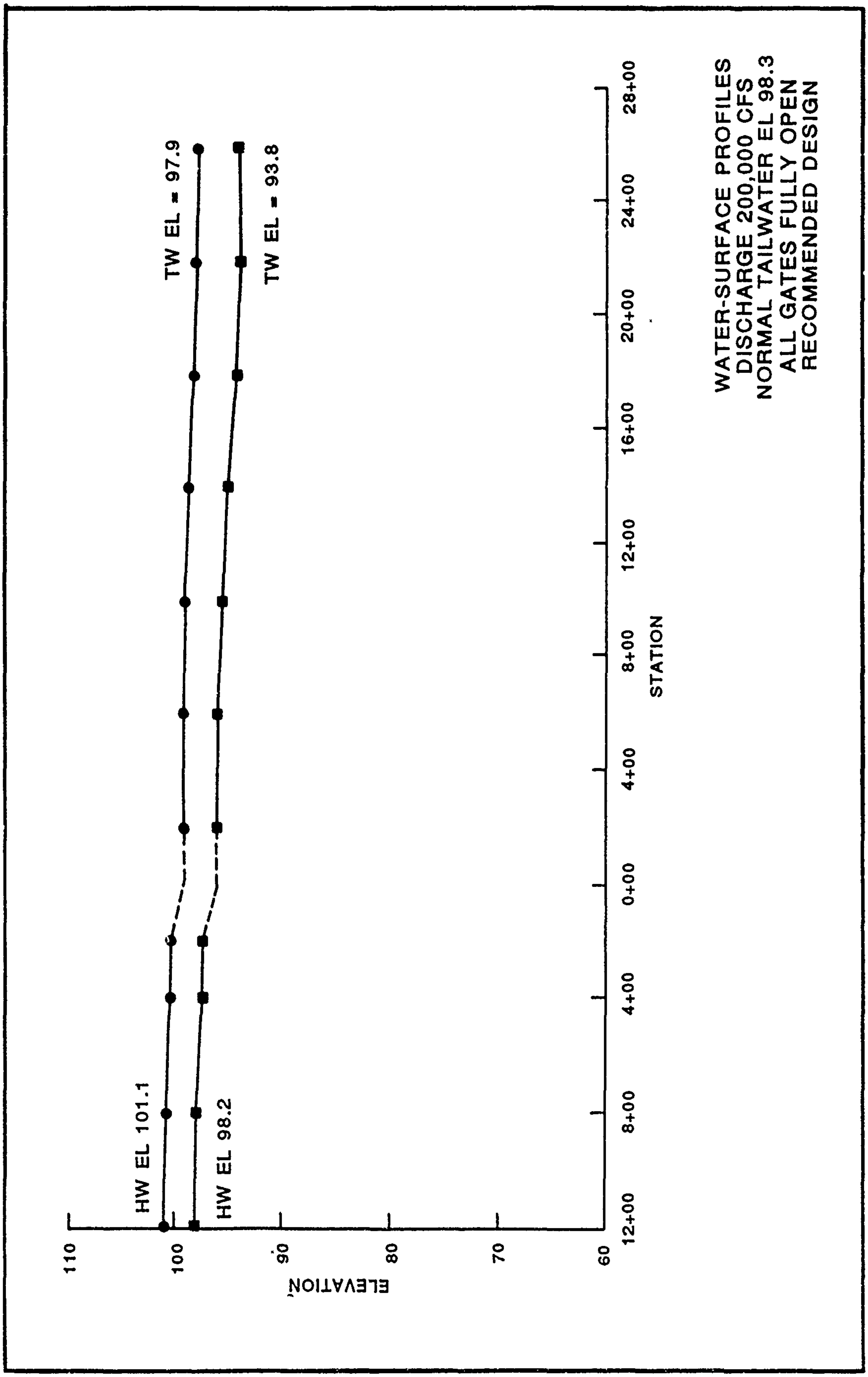
WATER-SURFACE PROFILES
DISCHARGE 100,000 CFS
NORMAL TAILWATER EL 85.1
ALL GATES FULLY OPEN
RECOMMENDED DESIGN

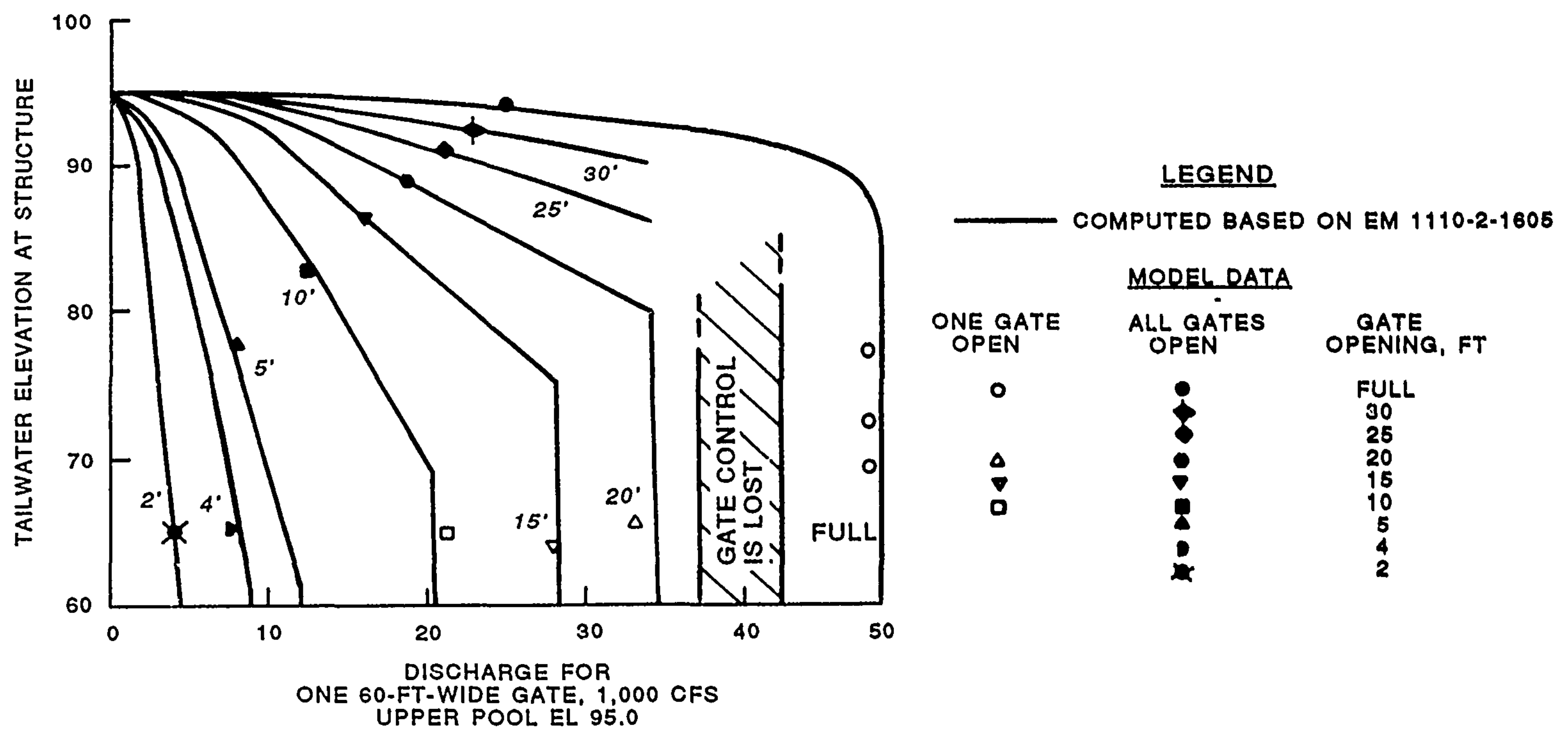


WATER-SURFACE PROFILES
DISCHARGE 150,000 CFS
NORMAL TAILWATER EL 92.4
ALL GATES FULLY OPEN
RECOMMENDED DESIGN

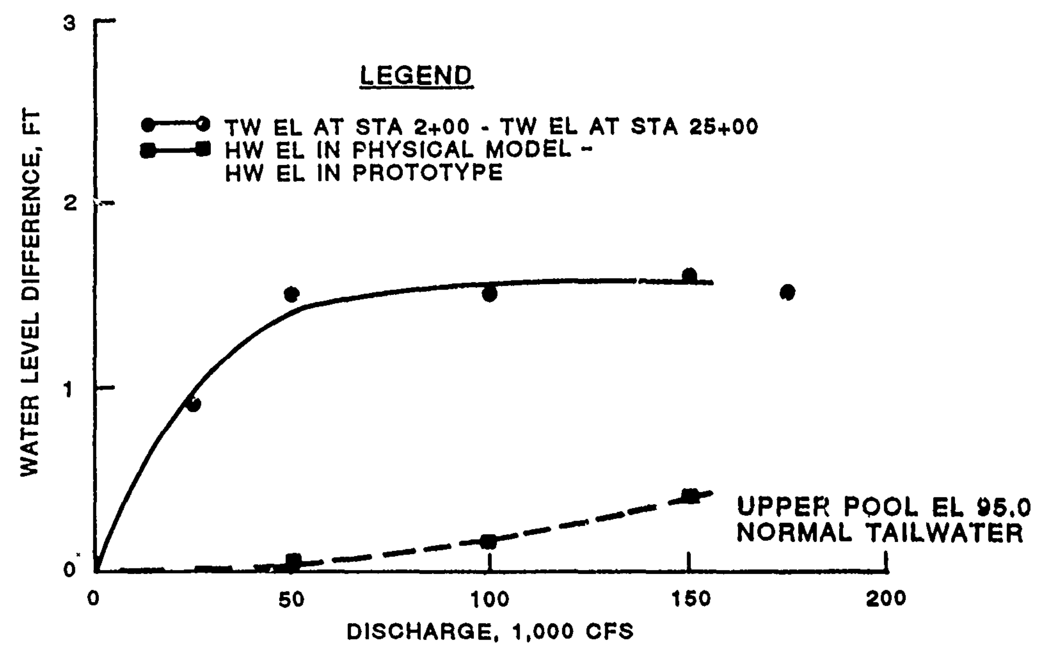
PLATE 22


WATER-SURFACE PROFILES
DISCHARGE 175,000 CFS
NORMAL TAILWATER EL 95.6
ALL GATES FULLY OPEN
RECOMMENDED DESIGN
HW EL 99.8
HW EL 96.7
TW EL = 97.1
TW EL = 93.2
ELEVATION
110
100
90
80
70
60
STATION
12+00
8+00
4+00
0+00
4+00
8+00
12+00
16+00
20+00
24+00
28+00


WATER-SURFACE PROFILES
DISCHARGE 200,000 CFS
NORMAL TAILWATER EL 98.3
ALL GATES FULLY OPEN
RECOMMENDED DESIGN
HW EL 101.1
HW EL 98.2
TW EL = 97.9
TW EL = 93.8
ELEVATION
STATION
110
100
90
80
70
60
12+00
8+00
4+00
0+00
4+00
8+00
12+00
16+00
20+00
24+00
28+00



RATING CURVES
FOR ONE TAINTER GATE



RELATIONSHIP BETWEEN DISCHARGE
AND DIFFERENCES IN WATER LEVEL