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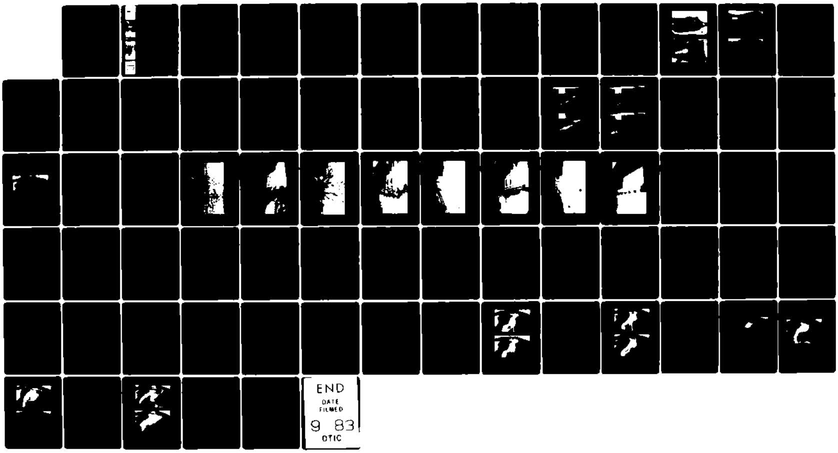

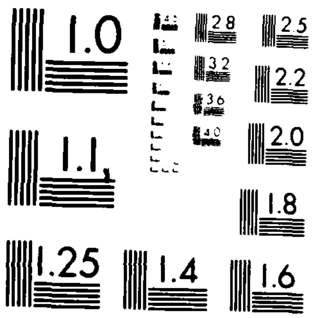
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US Army Corps of Engineers

TECHNICAL REPORT HL-83-9

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BLOOMINGTON SPILLWAY NORTH BRANCH POTOMAC RIVER MARYLAND AND WEST VIRGINIA

Hydraulic Model Investigation

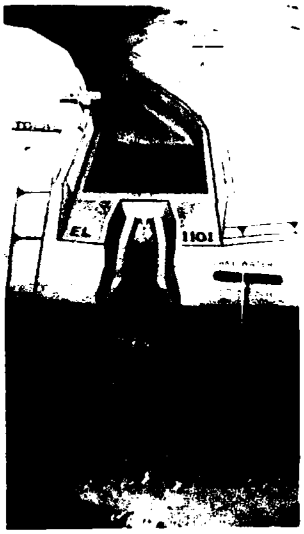
by

Bobby P. Fletcher

Hydraulics Laboratory

U. S. Army Engineer Waterways Experiment Station

P. O. Box 631, Vicksburg, Miss. 39180



June 1983

Final Report

HYDRAULICS LABORATORY

Prepared for U. S. Army Engineer District, Baltimore
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-Tests to investigate hydraulic performance of the gated spillway and the intensity of return flow on the downstream side of the saddle dike were conducted using a 1:60-scale model.

Flow approaching the spillway was satisfactory for all anticipated flow conditions. Eddies generated immediately upstream from the spillway had no significant effect on hydraulic performance. During controlled releases, no surging of flow at the abutments or tainter gates was observed.

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20. ABSTRACT (Continued).

Model tests of the exit area were conducted with the overburden fixed and then movable. Tests conducted with the fixed overburden indicated that the maximum spillway flow of 178,000 cfs generated only an insignificant (4-ft depth at toe) amount of return flow toward the downstream toe of the saddle dike. Tests conducted with the movable overburden indicated significant displacement caused by a spillway flow of 10,000 cfs. Flows as great as 178,000 cfs caused additional displacement of overburden. The overburden did not scour upstream to the toe of the dike. Test results indicated that the safety of the saddle dike was not jeopardized by return flow with either the fixed or movable overburden simulated in the model.

For any spillway flow condition, overburden material downstream from the spillway was transported through the swale and there was no tendency for a debris dam to form.

PREFACE

The model study of the Bloomington Spillway was authorized by the Office, Chief of Engineers (OCE), U. S. Army, on 4 August 1981, at the request of the U. S. Army Engineer District, Baltimore (NAB). The study was conducted during the period August 1981 to June 1982 in the Hydraulics Laboratory of the U. S. Army Engineer Waterways Experiment Station (WES) under the direction of Messrs. H. B. Simmons, Chief of the Hydraulics Laboratory, and J. L. Grace, Jr., Chief of the Hydraulic Structures Division. The tests were conducted by Messrs. B. P. Fletcher, Potong Bhramayana, and James Rucker. This report was prepared by Mr. Fletcher.

During the course of the study, Messrs. Sam Powell, Ben Kelly, and Rixby Hardy of OCE; Ed Lally, Frank Coppinger, and Andy Petallides of the U. S. Army Engineer Division, North Atlantic; and Dick Royer, Ed Palguta, Dennis Seibel, George Hufer, Jack Berezinak, and Dick Strong of NAB visited WES to discuss test results and assist in the formulation of plans for future model tests.

Commander and Director of WES during the conduct of the study and the preparation and publication of this report was COL Tilford C. Creel, CE. Technical Director was Mr. F. R. Brown.

A

CONTENTS

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

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

Multiply	By	To Obtain
acres	4046.857	square metres
cubic feet per second	0.02831685	cubic metres per second
feet	0.3048	metres
feet per second	0.3048	metres per second
miles (U. S. statute)	1.609344	kilometres

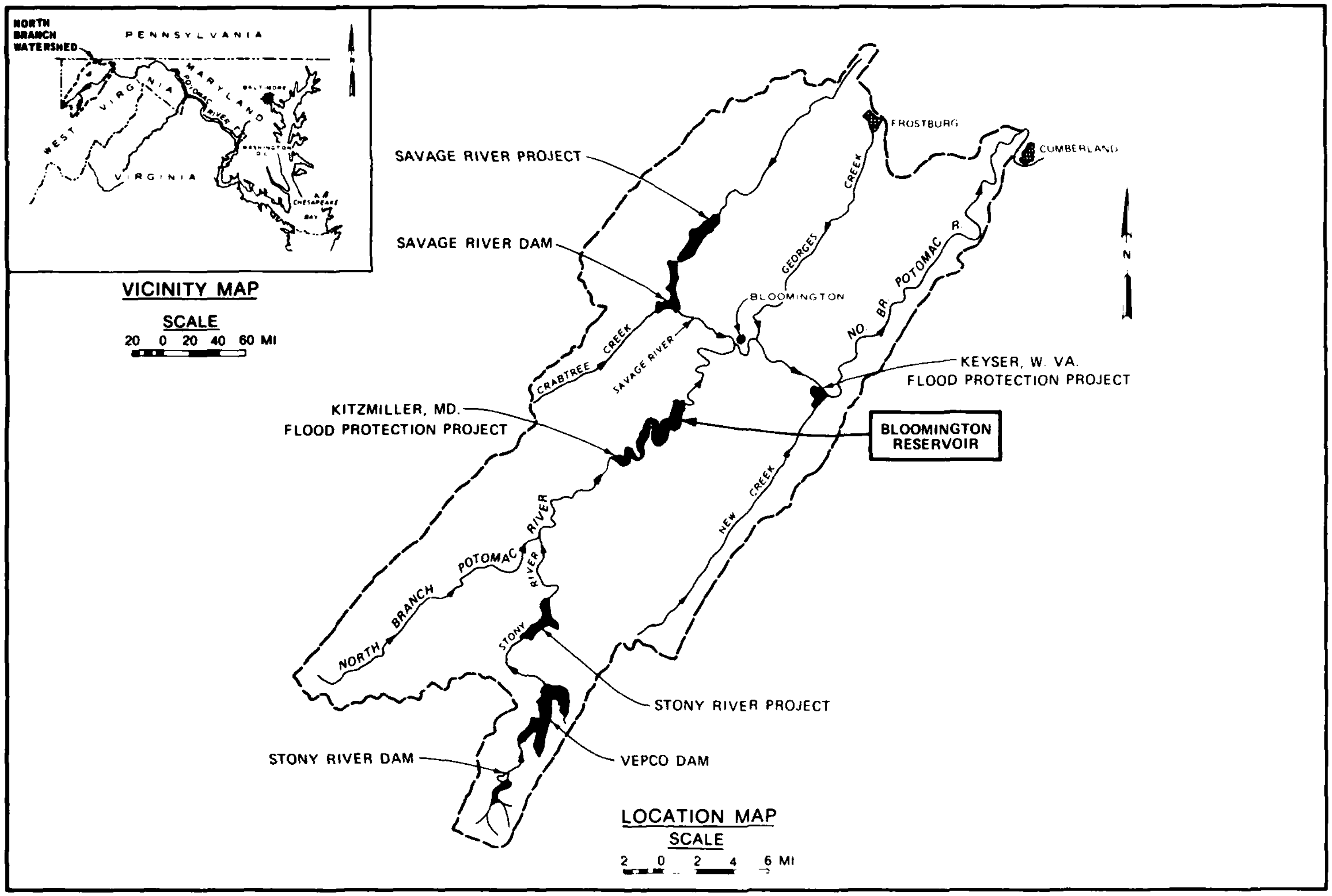

Figure 1. Location and vicinity maps of North Branch watershed

BLOOMINGTON SPILLWAY, NORTH BRANCH POTOMAC RIVER
MARYLAND AND WEST VIRGINIA

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. The Bloomington Spillway is located on the North Branch Potomac River which forms the State line between Garrett County, Maryland, and Mineral County, West Virginia (Figure 1). The spillway is approximately 7.9 miles* upstream of the confluence with the Savage River at Bloomington, Maryland. When the reservoir is at full conservation pool level, el 1466.0,** it will extend 5 miles upstream and will inundate about 952 acres. The Bloomington reservoir will be part of a system of major reservoirs that includes the existing Savage River reservoir, 2 proposed reservoirs on North Branch tributaries, and 13 reservoirs in other parts of the Potomac River basin.

2. The general arrangement of the spillway and appurtenant structures is shown in Plates 1-3. The spillway is designed to pass 178,000 cfs at the maximum pool elevation of 1508.9. Approach and discharge channels are 250 ft wide between concrete walls and have been excavated in earth and rock. The discharge channel (Figure 2c) is 270 ft wide beyond the retaining walls. Five 42-ft-wide gate bays, with tainter gates, are provided to furnish a crest length of 210 ft. Nonoverflow monoliths are concrete gravity sections with each end abutment making direct concrete-to-rock contact. Operating machinery for the tainter gates is located downstream from the service bridge, anchored to the top of the intermediate and end piers.

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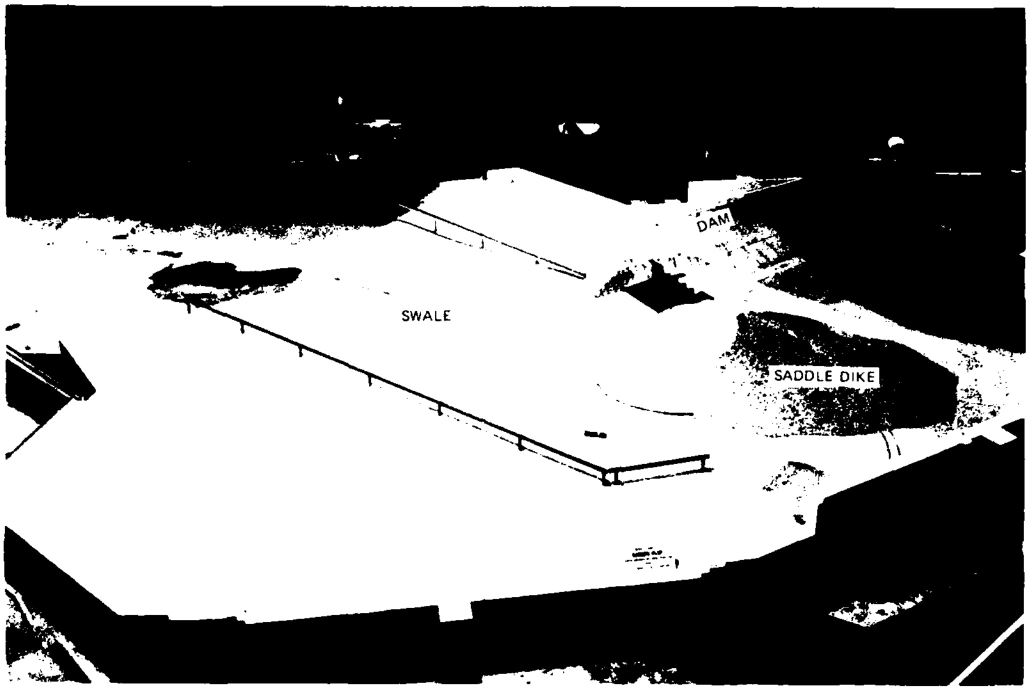
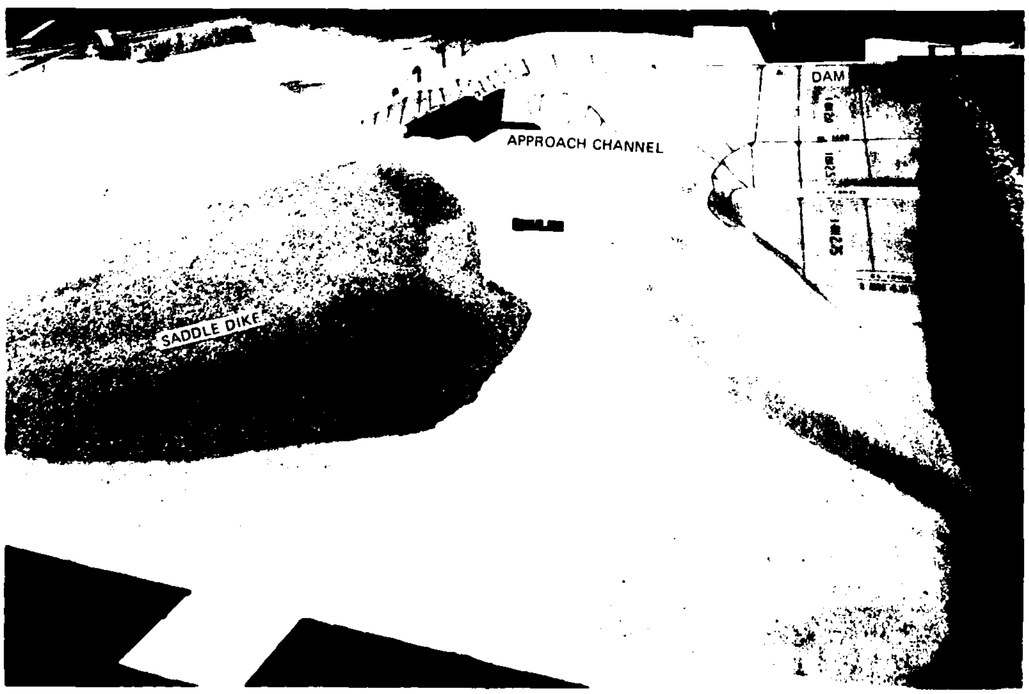
* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

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

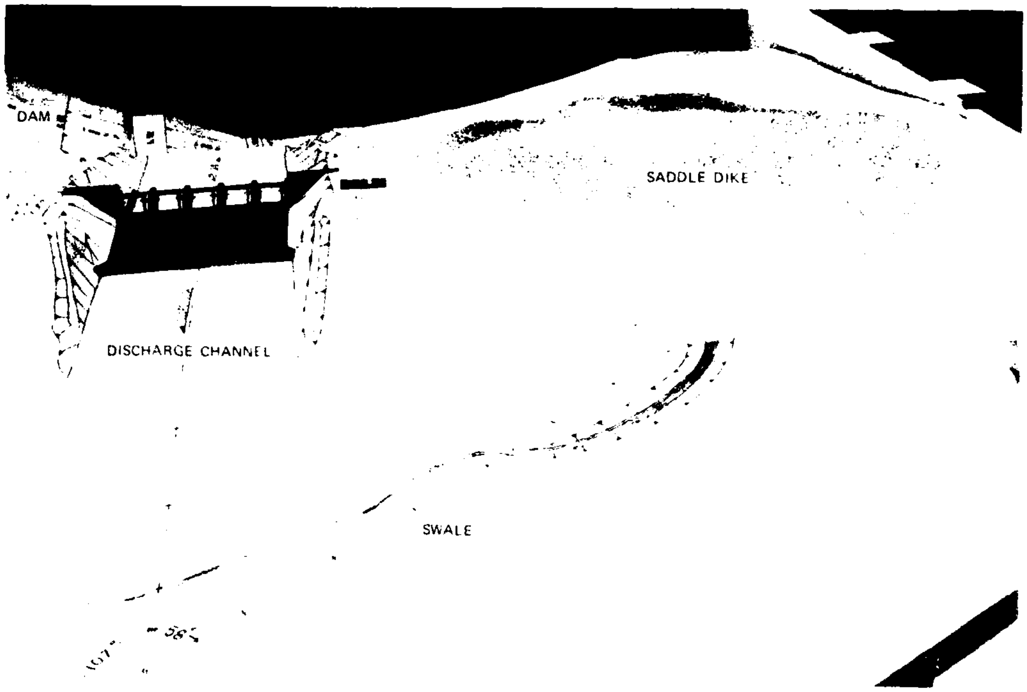


a. General view

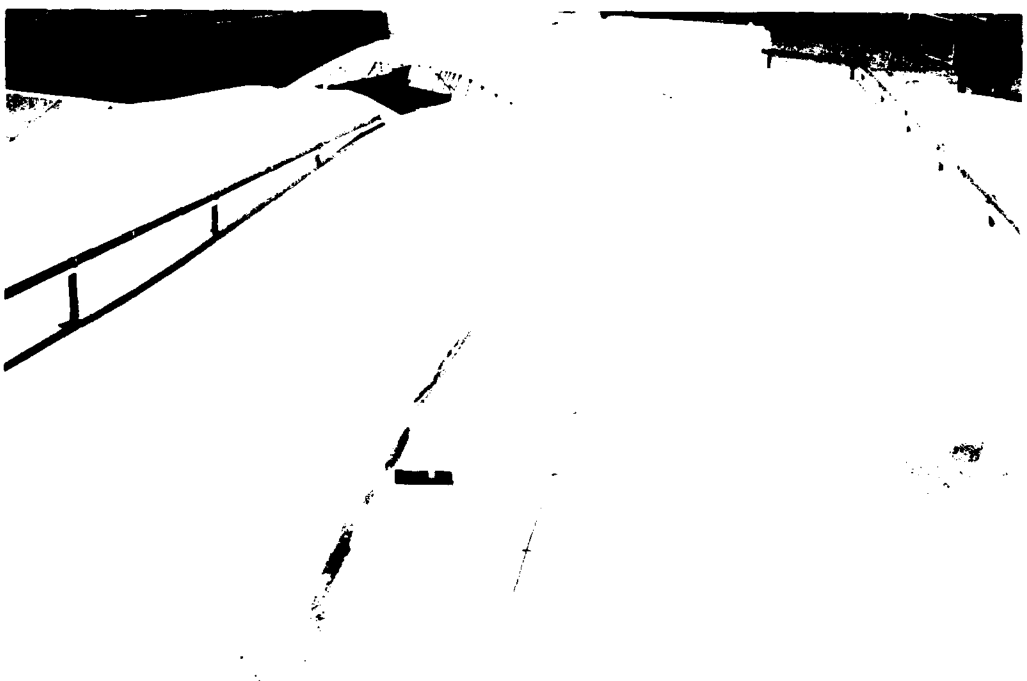


b. Spillway approach

Fig're 2. The model (Continued)



c. Spillway exit



d. Discharge channel

Figure 2 (Concluded)

3. The excavated portion of the approach channel (Figure 2b) is about 250 ft long with an unpaved channel of exposed sandstone. A 25-ft-long concrete apron is provided upstream of the ogee weir at el 1455.0. The approach channel bottom has a downward slope of 1 percent in the upstream direction. Approach channel sides are sloped 2V on 1H in the existing material. The earth-fill saddle dike (Figure 2a) has a top elevation of 1514.0 and its downstream toe is protected up to el 1430.0 by stone with a maximum diameter of 3 ft. The earth-fill dam has a top elevation of 1514.0 (Figure 2).

4. The 270-ft-long spillway structure, with a crest elevation of 1468.0 ft, includes four 10-ft-wide intermediate piers and two 10-ft-wide abutment piers. The spillway crest elevation of 1468.0 was located 2 ft above the maximum conservation pool in order to protect the steel tainter gates from corrosion by the acid reservoir water. No stop logs are provided for maintenance purposes since the gates will be above normal pool elevations.

5. The discharge channel is concrete-paved and has retaining walls on each side for a distance of 100 ft downstream of the weir section. The channel has a 4 percent downstream slope and is 270 ft wide beyond the walls and concrete apron. Spillway flow would be directed at a 90-deg angle into a natural swale (Figure 2a). The released flows will intersect the Potomac River approximately 1,700 ft downstream of the intersection of the swale and discharge channel. The toe of the saddle dike is approximately 300 ft upstream from the intersection of the discharge channel and swale. The swale consists of overburden material with depths up to 70 ft overlying bedrock. The earth-fill saddle dike is sitting on 65 ft of overburden. Basic data and assumptions are listed below:

	Elevation
Top of dam	1514.0
Top of saddle dike	1514.0
Spillway design flood (maximum pool prior to model study)	1508.9
Static full pool (top of closed crest gates)	1500.0
Spillway crest	1468.0
Conservation lake	1466.0

Purpose of Study

6. A model analysis was desired in the interest of good engineering practice to determine if spillway releases in the swale would induce surges, currents, and/or turbulence on the downstream side of the saddle dike sufficient to jeopardize the safety of the dike. The model was also used to verify satisfactory flow conditions in the approach to the spillway, at the abutments, through the spillway, and to identify areas of degradation and deposition of the overburden in the swale. Specific determination was made of any tendency for the erodible overburden material to form a debris dam in the swale.

PART II: THE MODEL

Description

7. A 1:60-scale comprehensive model (Figure 2 and Plate 1) was constructed to reproduce all topography and structures including a 900-ft length and 1,200-ft width of approach, spillway, dam, dike, discharge channel, and swale from the toe of the saddle dike to a point about 2,000 ft downstream. The approach 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, and nonoverflow sections of the dam were constructed of metal. The saddle dike (Figure 2) was constructed of loose stone simulating a maximum diameter (d_{100}) of 3.0 ft. The model was designed to permit tests with a fixed or movable overburden simulated in the exit area (swale).

8. Water used in the operation of the model was supplied by a pump and discharges were measured by means of venturi meters. Steel rails set to grade provided reference planes for measuring devices. Water-surface elevations were obtained with point gages. Velocities were measured by means of a pitot tube and stopwatch timing of dye injected into the water, and surface currents were determined by confetti sprinkled on the water surface.

Interpretation of Model Results

9. 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	1:60
Area	$A_r = L_r^2$	1:3,600

(Continued)

Dimension	Ratio	Scale Relation
Velocity	$V_r = L_r^{1/2}$	1:7.75
Discharge	$Q_r = L_r^{5/2}$	1:27,900
Time	$T_r = L_r^{1/2}$	1:7.75

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

PART III: TESTS AND RESULTS

Presentation of Data

10. Model tests and results are not presented in chronological order. As each element of the project is considered, all pertinent tests are described. All data are presented in terms of prototype equivalents.

Approach Area

11. The model reproduced the approach area for a width of 1,200 ft and a distance of 900 ft upstream from the spillway (Plate 1 and Figure 2a). Approach flows, controlled and uncontrolled, up to the design discharge of 178,000 cfs were evaluated.

12. Controlled flows in the approach to the spillway are shown in Photos 1-6. Uncontrolled approach flows are shown in Photos 7 and 8. Time-exposed photographs caused the paths traveled by confetti on the water surface to appear as white streaks. The magnitude and direction of approach bottom currents, for discharges from 10,000 to 178,000 cfs, are shown in Plates 4-9. Flow patterns indicate that flow contractions are generated at the upstream end of the rock cut. These contractions produce eddies on both sides of the approach to the spillway. The eddies partially dissipated approaching the crest and did not adversely affect spillway capacity or flow conditions at the spillway. Approach flows to the spillway were considered satisfactory for all anticipated discharges and gate openings.

Spillway

13. A plan and profile of the spillway are shown in Plates 2 and 3. The weir crest (el 1468.0) was designed for an approach depth of 13 ft and a maximum head, H_d, of 40.9 ft. The equation of the upstream quadrant is

$$\frac{x^2}{(0.228H_d)^2} + \frac{y^2}{(0.141H_d)^2} = 1$$

and the downstream quadrant is

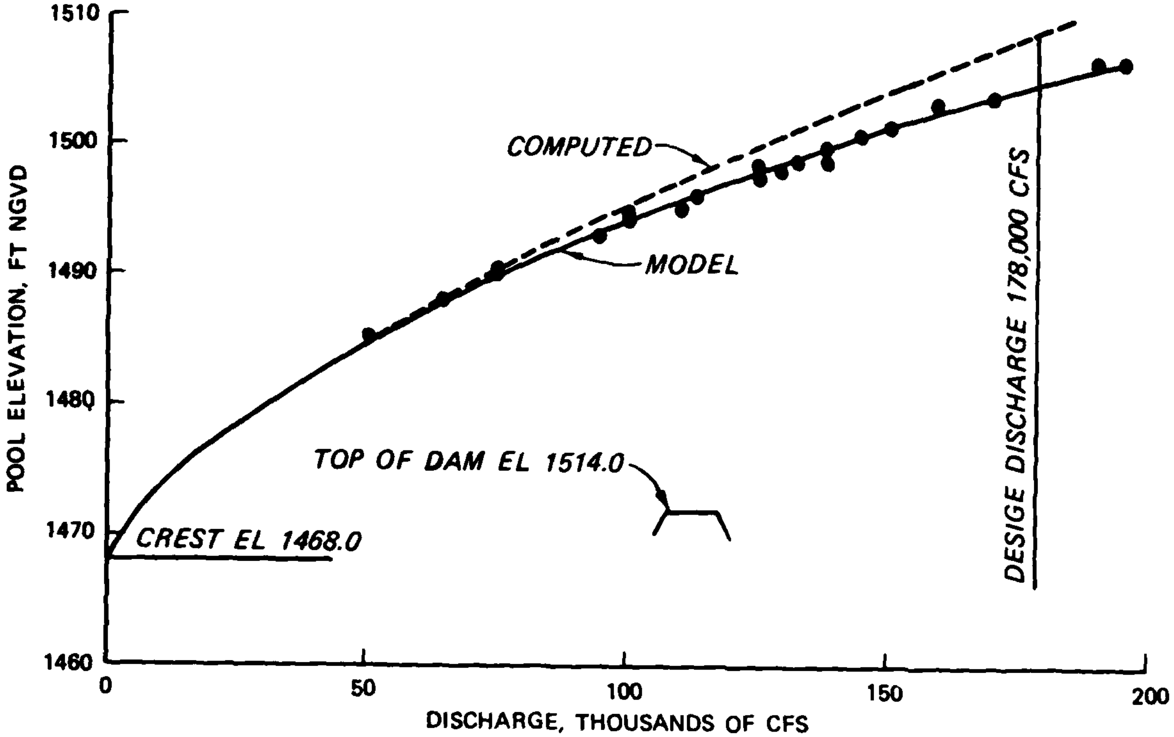
$$x^{1.85} = 2.0H_d^{0.85}y$$

The crest profile is shown in Plate 3. The computed and model flow rating curves and basic model data are shown in Figure 3. Coefficients used for computation of the computed curve are also listed in Figure 3. The model rating curve indicated that the spillway would pass the design flood of 178,000 cfs at a pool elevation 4.0 ft less than that computed. Controlled and uncontrolled flow rating curves are plotted in Figures 4 and 5. Controlled flow discharge coefficients, C_g , are plotted versus gross head on the gate, H_g , in Figure 6. Basic calibration data for controlled flows are tabulated in Table 1. No severe flow contraction at the spillway abutments or piers was observed. Vortices occurred occasionally immediately upstream from each tainter gate (Photo 2). These vortices had no significant effect on hydraulic performance. During uncontrolled or controlled flow releases, no random or periodic surging of flow was observed upstream of the spillway piers or abutments.

14. Water-surface profiles through the spillway were similar on each side of the spillway and are plotted for various discharges in Plates 10-13. At the maximum flow, the water surface contacted the gate trunnion and overtopped the upstream end of the downstream training walls (Plate 13). Flows above 150,000 cfs splashed over the top of the upstream end of each training wall and produced a flow in the downstream direction along the backside of each training wall. Water splashing over the walls at a discharge of 178,000 cfs produced a maximum insignificant flow rate of about 10 cfs along the backside of each wall at the downstream end of the wall.

Swale

15. Hydraulic performance of the prototype swale was evaluated



BASIC DATA

DISCHARGE CFS	POOL EL FT NGVD
50,000	1485.0
50,000	1485.0
64,000	1488.0
75,000	1490.0
94,000	1493.0
100,000	1494.1
100,000	1494.3
110,000	1495.1
113,000	1496.1
125,000	1498.4
125,000	1497.9
129,000	1498.2
132,000	1498.8
138,000	1498.9
138,000	1499.7
144,000	1500.7
150,000	1501.5
159,000	1503.1
170,000	1503.9
190,000	1506.4
195,000	1506.6

COEFFICIENTS FOR DESIGN HEAD OF 40.9 FT
- PIER CONTRACTION, K_p = 0.013
- ABUTMENT CONTRACTION, K_d = 0.17
- DISCHARGE, C = 3.56

Figure 3. Spillway rating curves for uncontrolled flow

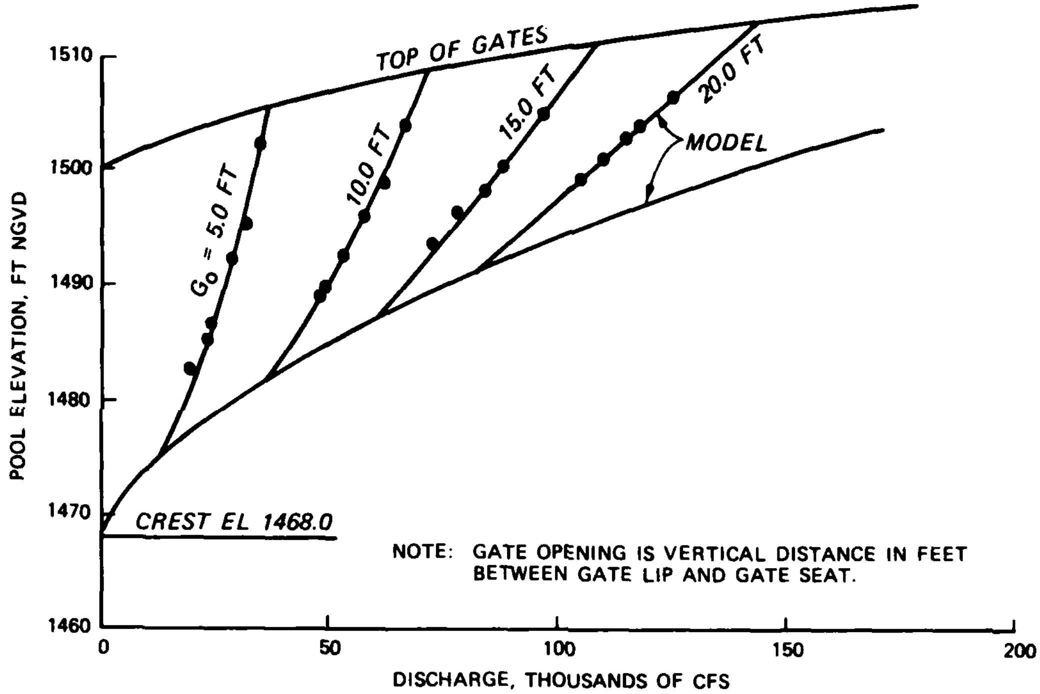


Figure 4. Spillway rating curves controlled and uncontrolled flows

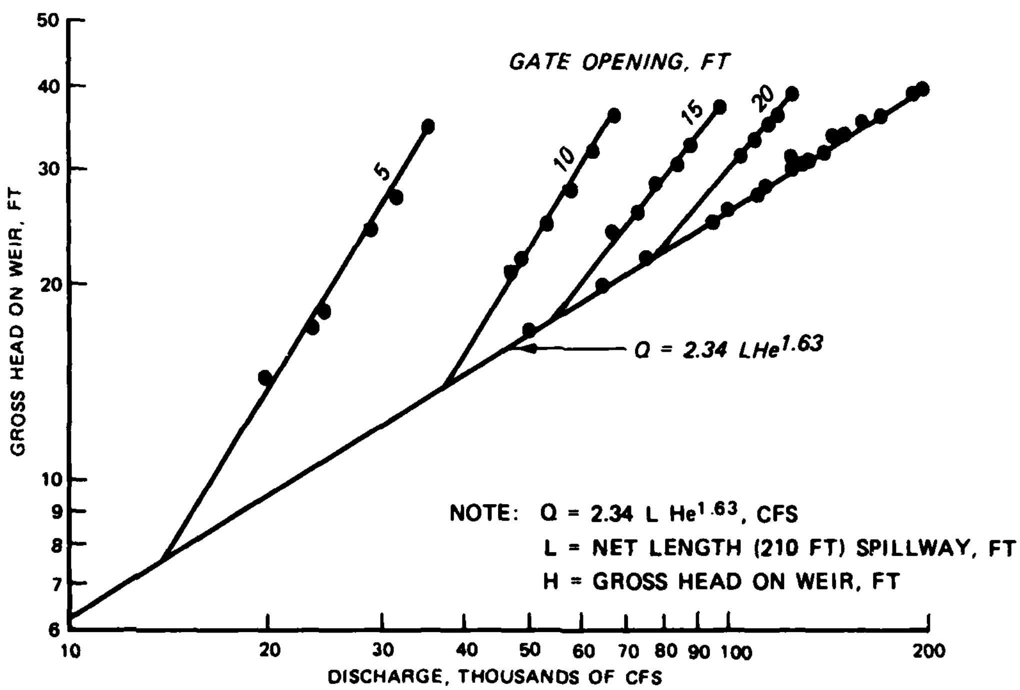


Figure 5. Discharge-head relation, controlled and uncontrolled flow

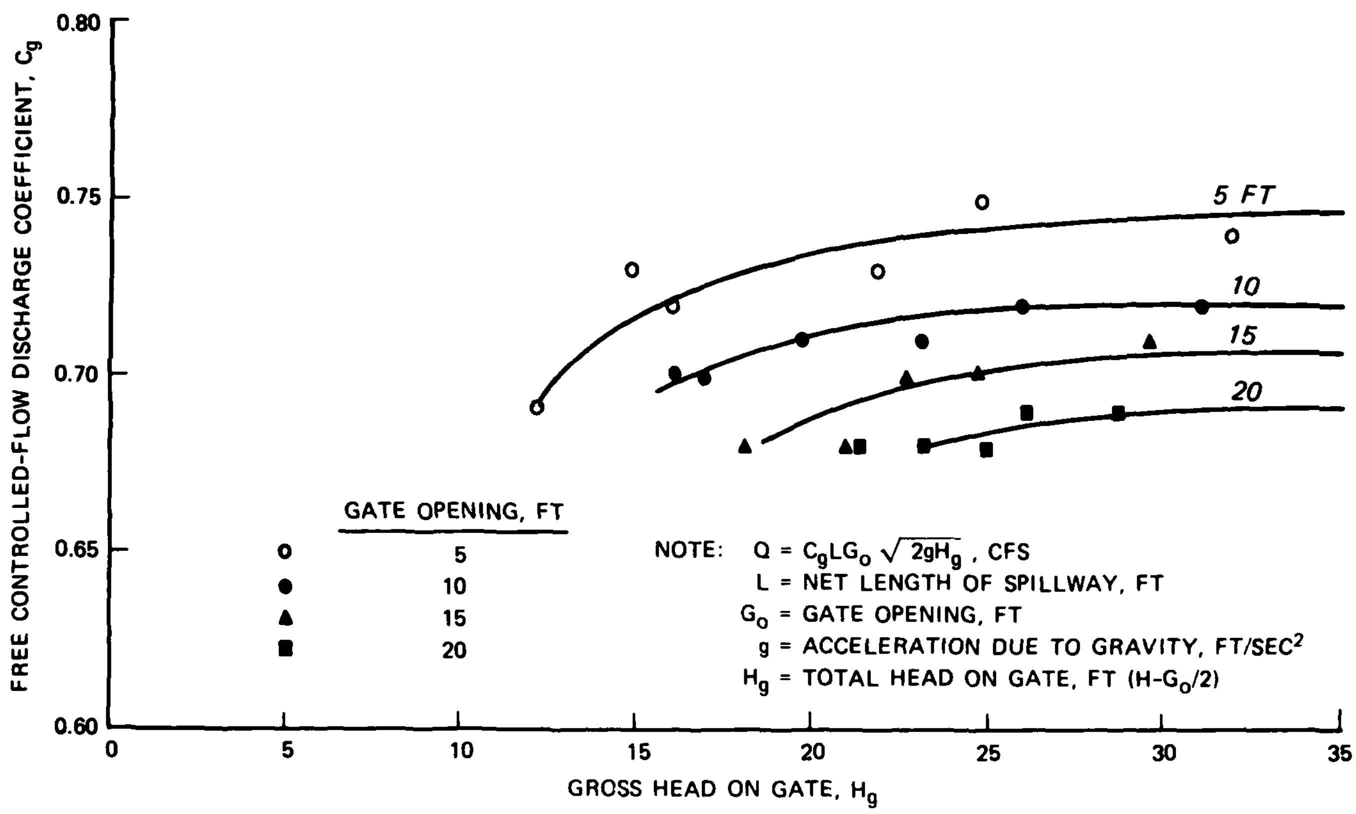


Figure 6. Discharge coefficients, controlled flow

by reproducing the bedrock surface geometry and simulating the overburden surface geometry, maximum stone size, and depth. Loose rock, having a maximum diameter, d_{100}, of 2.5 ft, was molded to the overburden contours. Flow characteristics in the swale were investigated with the overburden fixed and movable.

Fixed overburden

16. Tests were conducted with the fixed overburden simulated in the swale to determine the flow characteristics for various discharges in the approach, spillway, and swale.

17. Flow exiting the structure was supercritical with standing waves on the spillway apron and rock cut (Figure 7). The standing waves did not generate any adverse flow conditions. Return flow toward the dike is also illustrated in Figure 7. Current direction and magnitude in the swale for various flow conditions are shown in Plates 4-9; overburden contours are indicated by solid lines, and rock contours are indicated by dashed lines. As the spillway flow was increased, flow approached the toe of the dike as shown by the water-surface contours (Plates 4-9) and the plot in Figure 8. Water contacts the toe of the dike at a discharge of 150,000 cfs. Plate 9 and Figure 8 indicate that with a discharge of 178,000 cfs, water encroached on the dike to a maximum elevation of 1419.0 (a height of 4 ft above the toe of the dike). Figure 9 indicates a maximum wave amplitude at the toe of the dike of 6 ft. A maximum velocity of 2 fps near the toe of the dike is shown in Plate 9. The model indicated that with the fixed overburden, the stability of the dike would not be jeopardized by anticipated flows as high as 178,000 cfs.

18. Tests were conducted to determine if opening the spillway gates from left to right or right to left would affect the magnitude of the return flow toward the dike. The model indicated that fully opening the gates sequentially (normally gates are opened in increments) from left to right rather than right to left reduced the return flow toward the dike for all flows (Plate 14). Plates 15-25 depict the magnitude and direction of flow observed in the swale with the gates sequentially opened from left to right and right to left, respectively.

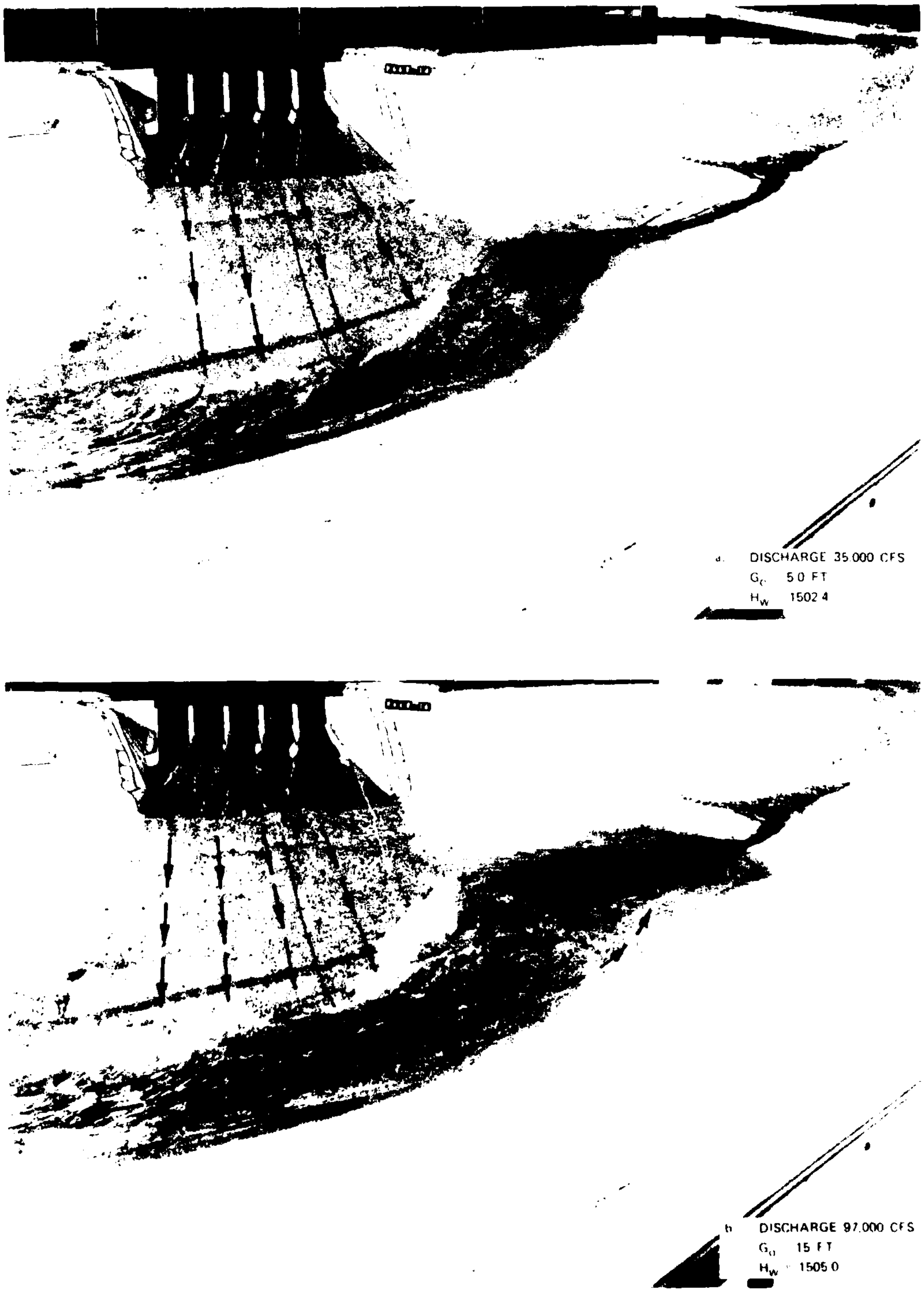
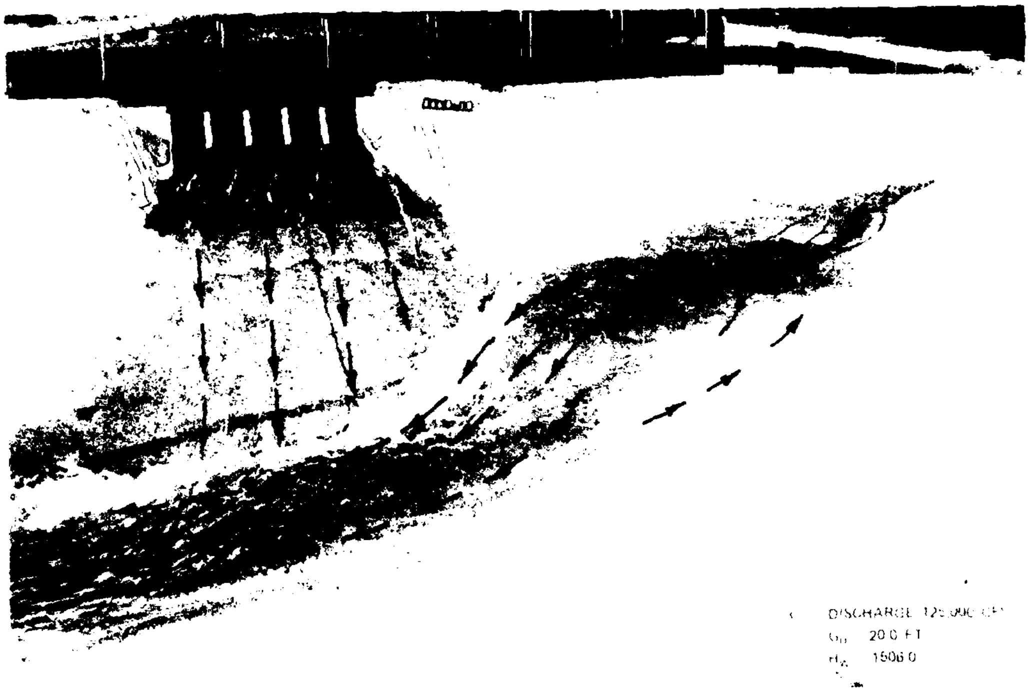


Figure 7. Flow exiting structures (Continued)



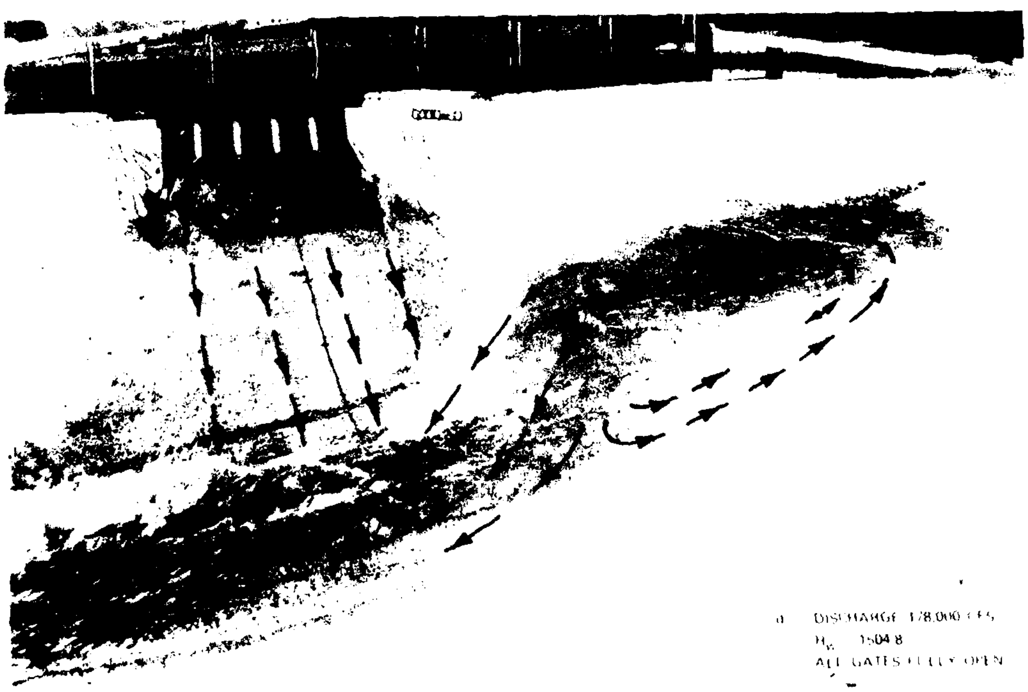


Figure 7. (Concluded)

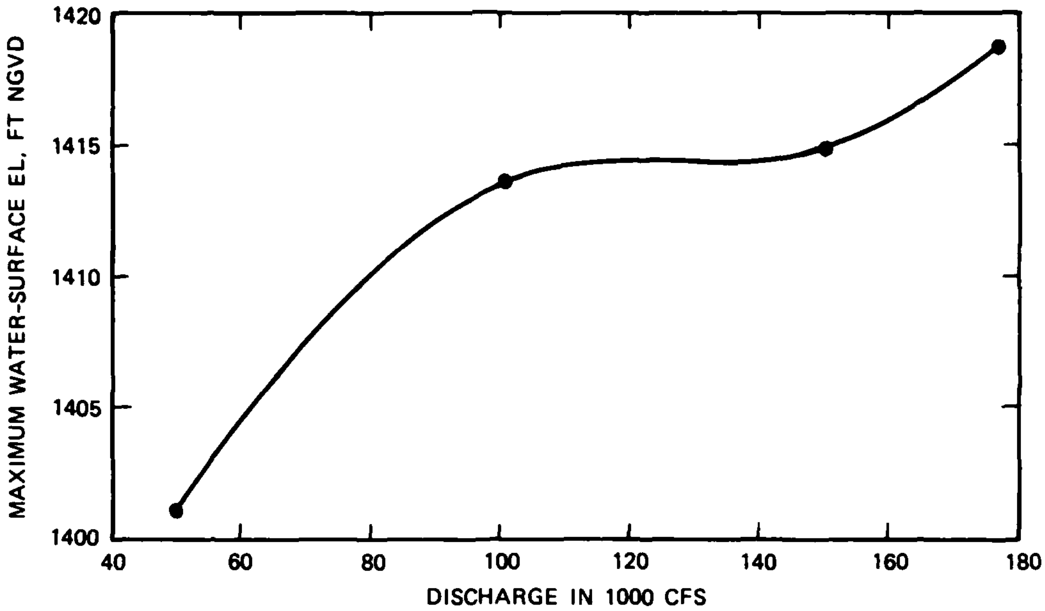


Figure 8. Fixed overburden, maximum water-surface elevation in return flow toward dike

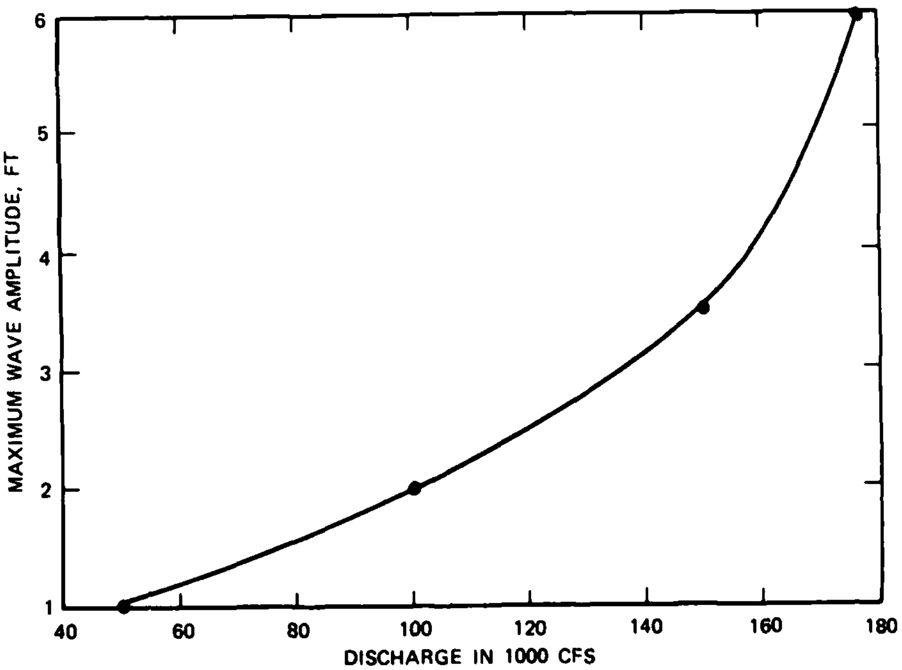


Figure 9. Fixed overburden, maximum wave amplitude in return flow toward dike

Movable overburden

19. Tests with the movable overburden simulated in the swale were conducted to qualitatively determine, for various discharges, if degradation would progress toward the toe of the saddle dike, where the overburden material would erode/deposit, and if a debris dam would form in the swale. Seismic tests and percussion drilling were conducted at the prototype site to determine the location of the top of rock contours. Also four test pit excavations were made to depths of approximately 20 ft to determine the size and distribution of the overburden material. The overburden was simulated by loose stone with a maximum diameter (d_{100}) of 2.5 ft. The bedrock contours are shown by dashed lines and the overburden contours by solid lines in Figure 10.

20. The testing procedure consisted of increasing the discharge in timed increments that approximated the project hydrograph. After each flow condition, the model was drained to permit observation and measurement of progressive rates of scour and deposition.

21. Figure 11 shows the dike and swale prior to flow. The dike consists of loose stone with a maximum diameter (d_{100}) of 3.0 ft. The downstream toe of the prototype dike is protected by stone with a d_{100} of 3.0 ft. Initial displacement of the overburden occurred at the downstream end of the spillway cut. Overburden displacement during and after a discharge of 10,000 cfs for a period of 2 hr (prototype) is shown in Plate 26. Resulting overburden displacement after discharges of 33,000, 65,000, 97,000, and 178,000 cfs is shown in Plates 27, 28, 29, and 30, respectively. A debris dam did not form for any discharge as the scoured overburden was transported through the swale.

22. With the movable overburden simulated, the model indicated that a debris dam would not form in the swale, the overburden would not scour near the saddle dike, and the safety of the dike would not be jeopardized by return flow from any anticipated spillway release.

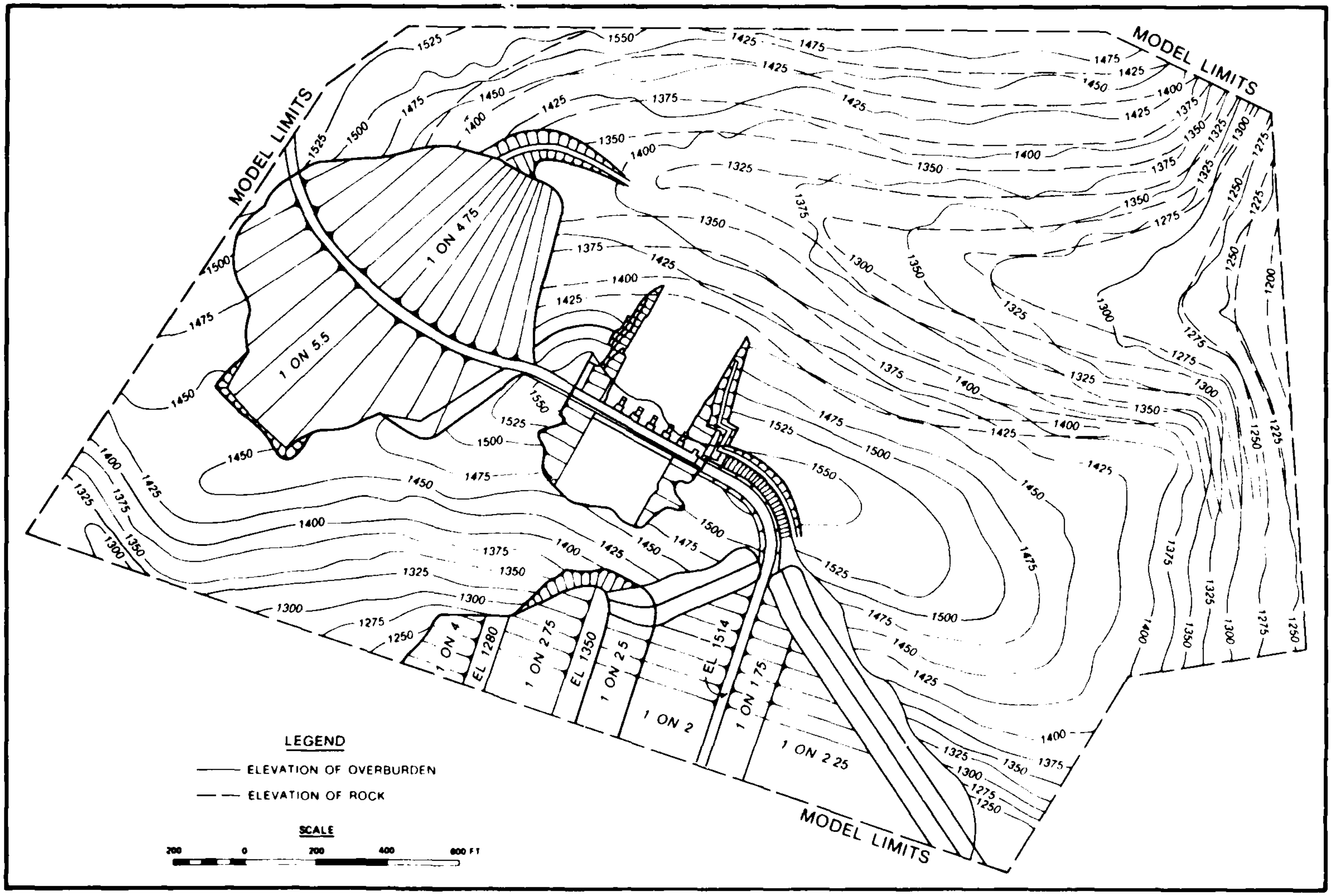


Figure 10. Overburden and bedrock contours

Figure 11. Overburden prior to flow

PART IV: DISCUSSION

23. Approach flows to the spillway were satisfactory for all spillway releases up to 178,000 cfs although eddies occurred upstream of both abutments. Flow contractions observed at the upstream end of the rock cut produced the eddies immediately upstream of the crest. The eddies partially dissipated prior to passing over the crest and had no significant effect on spillway flow characteristics.

24. The model indicated that the spillway could pass the design discharge of 178,000 cfs at a pool el 4.0 ft less than that computed. During controlled releases, vortices (which had no significant effect on hydraulic performances) occasionally developed immediately upstream from each tainter gate. No surging of flow at the abutments or tainter gates was observed for any anticipated flow condition. The water-surface profiles through the spillway were satisfactory for all anticipated flows.

25. Tests conducted with the fixed overburden in the swale indicated that the maximum spillway flow of 178,000 cfs generated only a relatively small amount of return flow toward the downstream toe of the saddle dike. The return flow encroached on the toe of the dike to a depth of 4 ft and produced maximum current velocities of 2 fps and waves with amplitudes as high as 6 ft. Test results with the fixed overburden revealed that return flow would not jeopardize the safety of the dike.

26. Tests conducted with the movable overburden simulated in the model also indicated that return flow toward the toe of the saddle dike, induced by spillway flows as high as 178,000 cfs, would not jeopardize the safety of the dike. The overburden was initially displaced by a discharge of 10,000 cfs at the downstream end of the rock cut. The model indicated that the overburden would not scour near the toe of the dike due to released flows not extending to the dike. Overburden material downstream from the spillway was scoured down to bedrock and transported through the swale. There was no tendency for the overburden material, with any flow rate, to form a debris dam in the swale.

Table 1

Basic Data, Controlled Flow

Crest El 1468.0

Gate Opening ft (G_o)	Discharge 1,000 cfs	Pool El ft NGVD	Gross Head on Weir ft (H)	Gross Head on Gate ft (H_g)	C_g
5.0	20.0	1482.5	14.5	12.0	0.69
	23.5	1485.2	17.2	14.7	0.73
	24.3	1486.4	18.4	15.9	0.72
	28.7	1492.2	24.2	21.7	0.73
	31.5	1495.1	27.1	24.6	0.75
	35.0	1502.4	34.4	31.9	0.74
10.0	47.0	1489.0	21.0	16.0	0.74
	48.2	1489.8	21.8	16.8	0.70
	53.0	1492.6	24.6	19.6	0.71
	57.5	1496.0	28.0	23.0	0.71
	62.1	1498.9	30.9	25.9	0.72
	67.3	1504.0	36.0	31.0	0.72
15.0	73.0	1493.5	25.5	18.0	0.68
	77.8	1496.3	28.3	20.8	0.68
	84.0	1498.1	30.1	22.6	0.70
	88.0	1500.2	32.2	24.7	0.70
	97.0	1505.0	37.0	29.5	0.71
20.0	105.0	1499.2	31.2	21.2	0.68
	110.0	1501.1	33.1	23.1	0.68
	115.0	1502.9	34.9	24.9	0.68
	118.0	1504.0	36.0	26.0	0.69
	125.0	1506.6	38.6	28.6	0.69

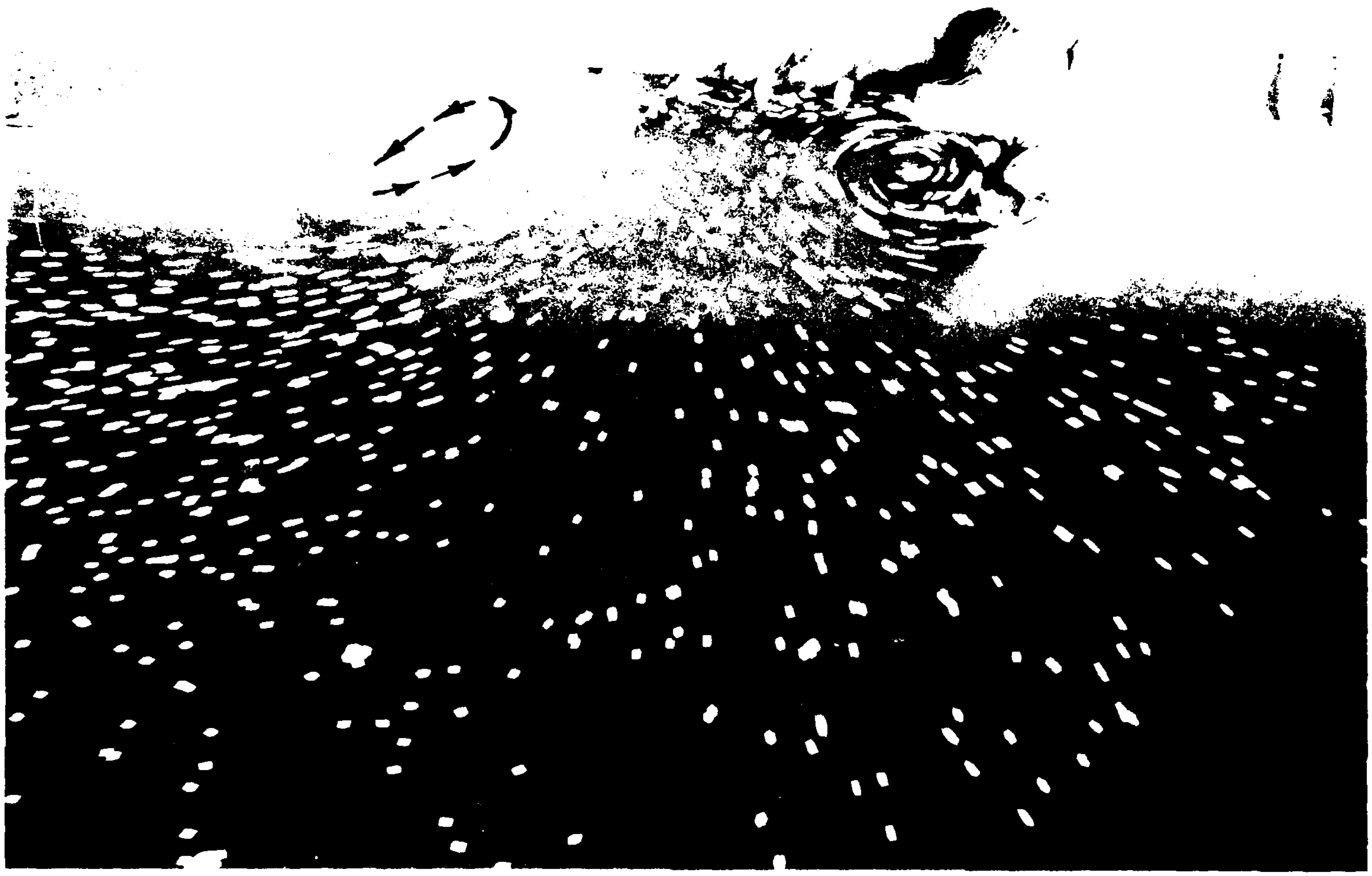


Photo 1. Approach flow conditions; discharge 33,000 cfs, pool el 1502.4, gate opening 5 ft, exposure time 15 sec

Photo 2. Spillway flow conditions; discharge 33,000 cfs, pool el 1502.4, gate opening 5 ft, exposure time 15 sec

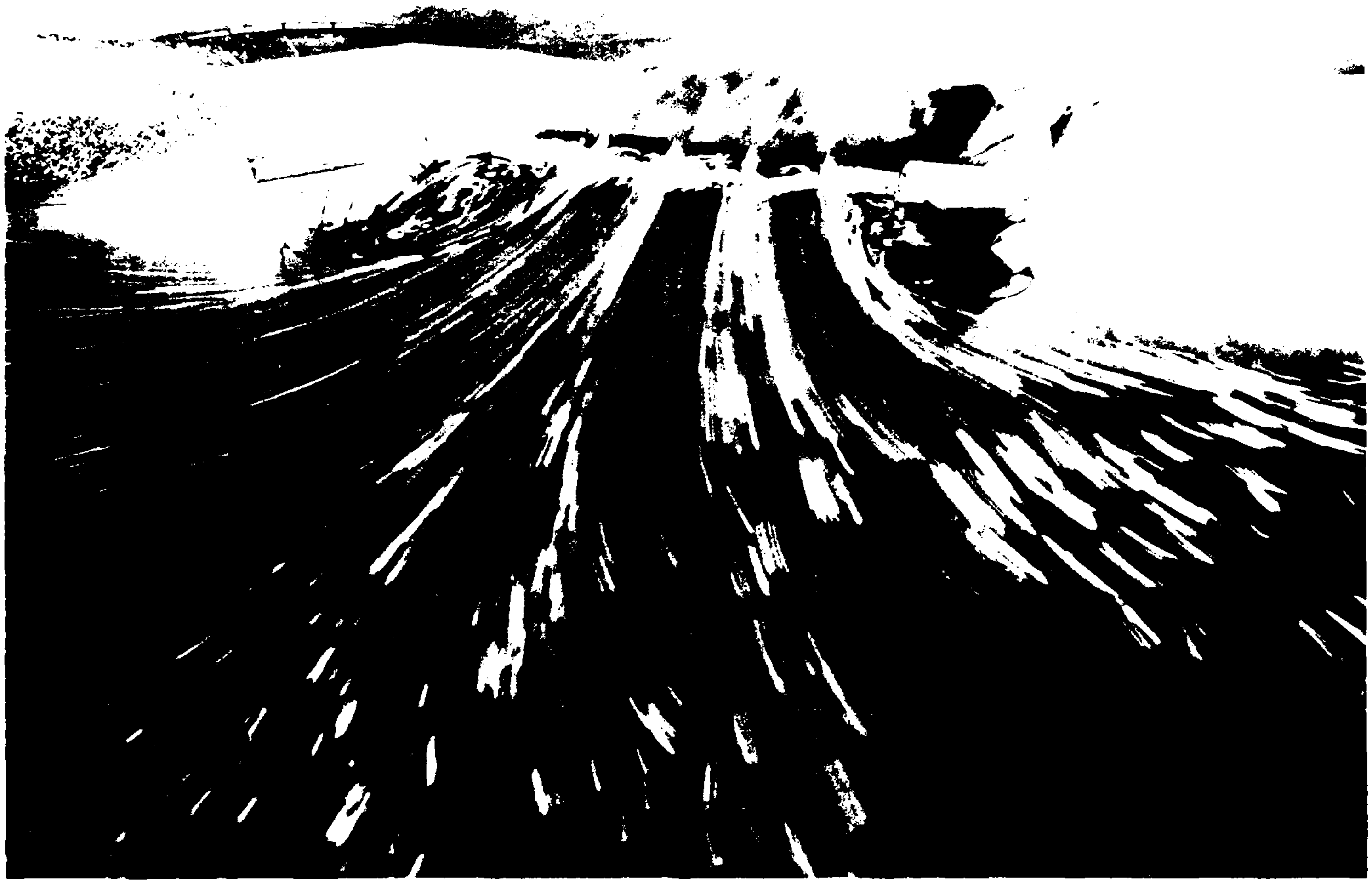


Photo 3. Approach flow conditions; discharge [illegible] cfs, pool el 1505.0,
[illegible]

Photo 4. Spillway flow conditions; discharge 97,000 cfs, pool el 1505.0, gate opening 15 ft, exposure time 15 sec

Photo 5. Approach flow conditions; discharge [illegible] cfs, pool el [illegible], gate opening [illegible] ft, [illegible]

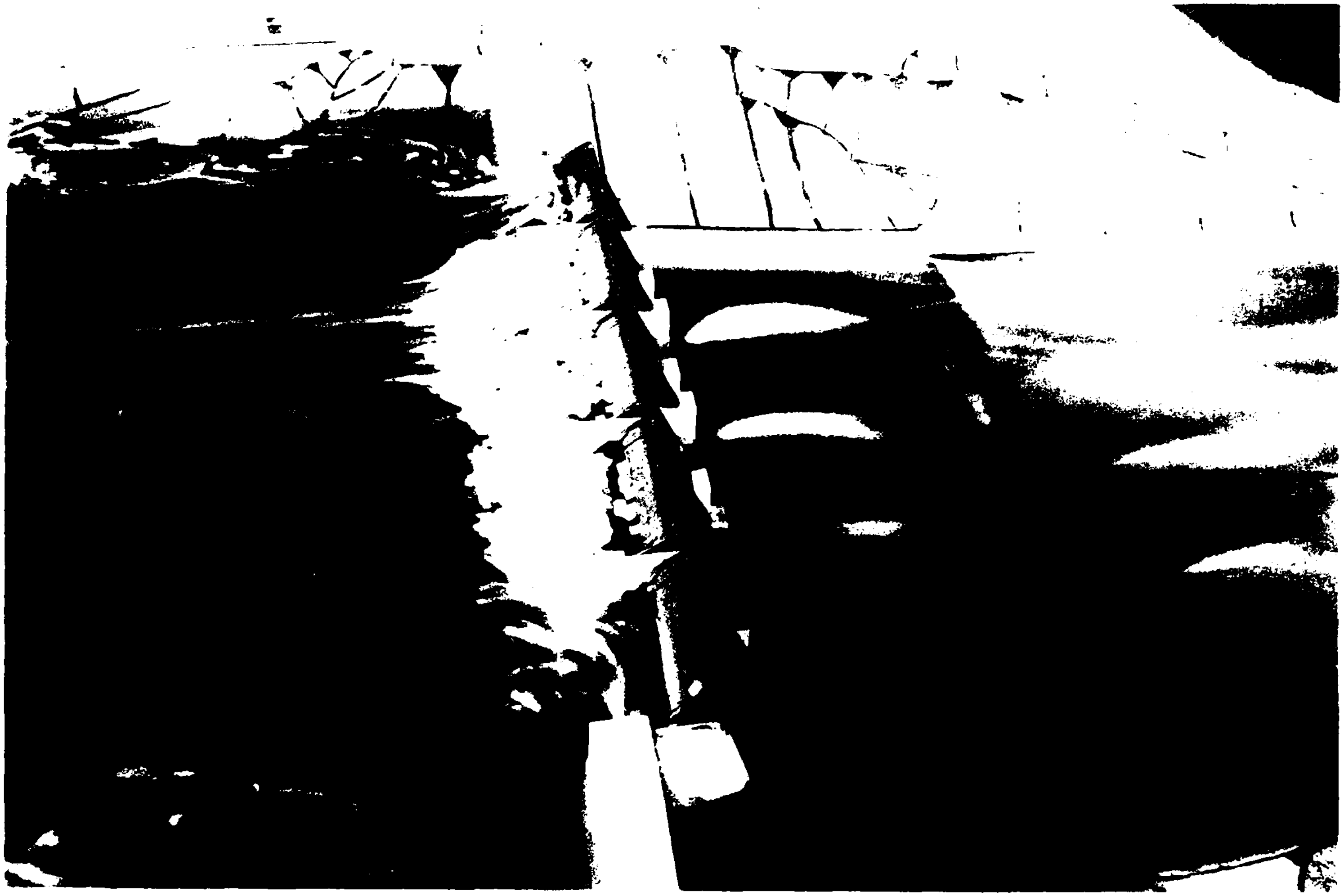


Photo 6. Spillway flow conditions; discharge 125,000 cfs, pool el 1506.0, gate opening 20 ft, exposure time 15 sec

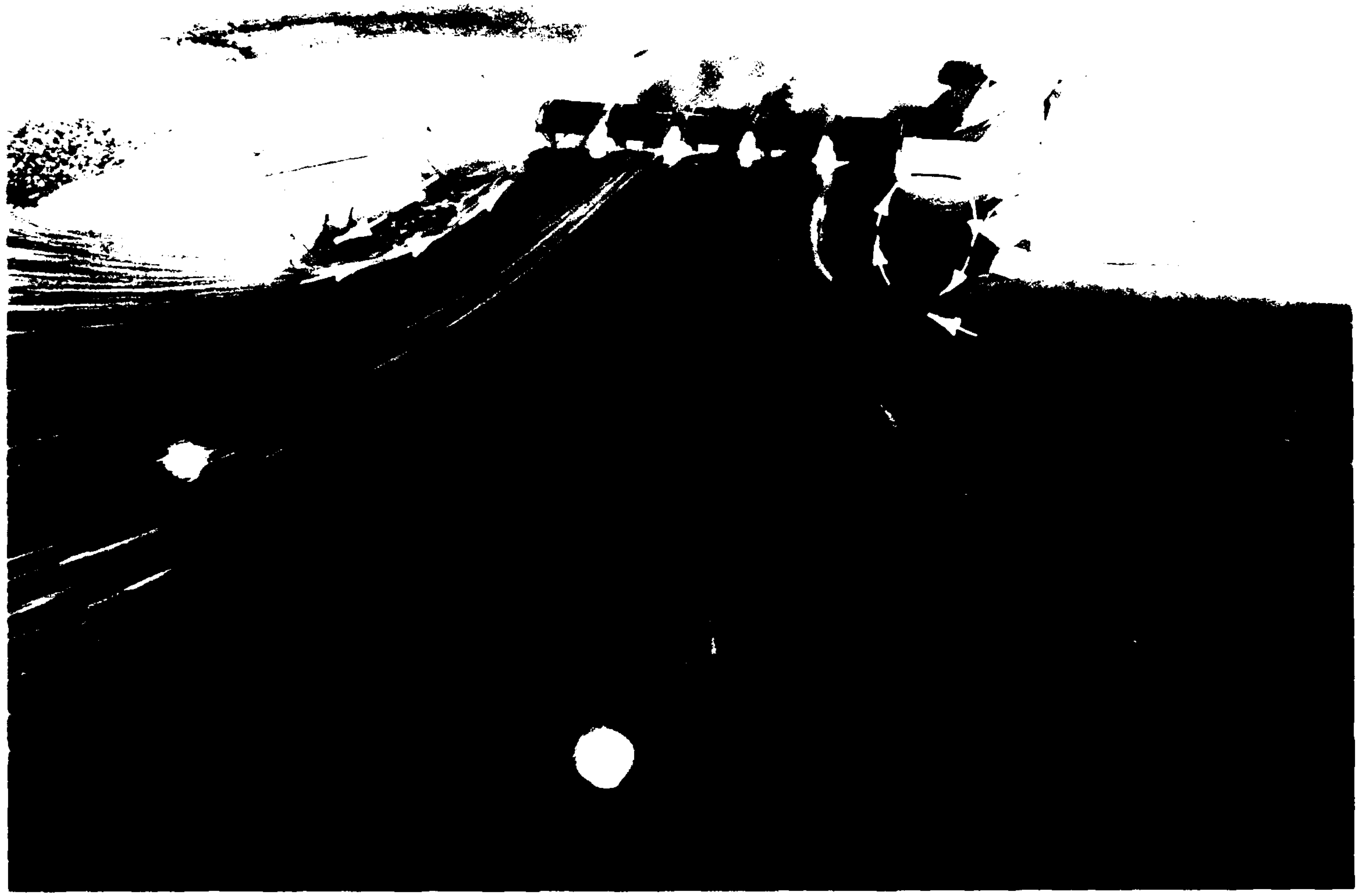


Photo [illegible]. Approach flow conditions; discharge 178,000 cfs, pool el 1504.8, gates fully open, exposure time 15 sec

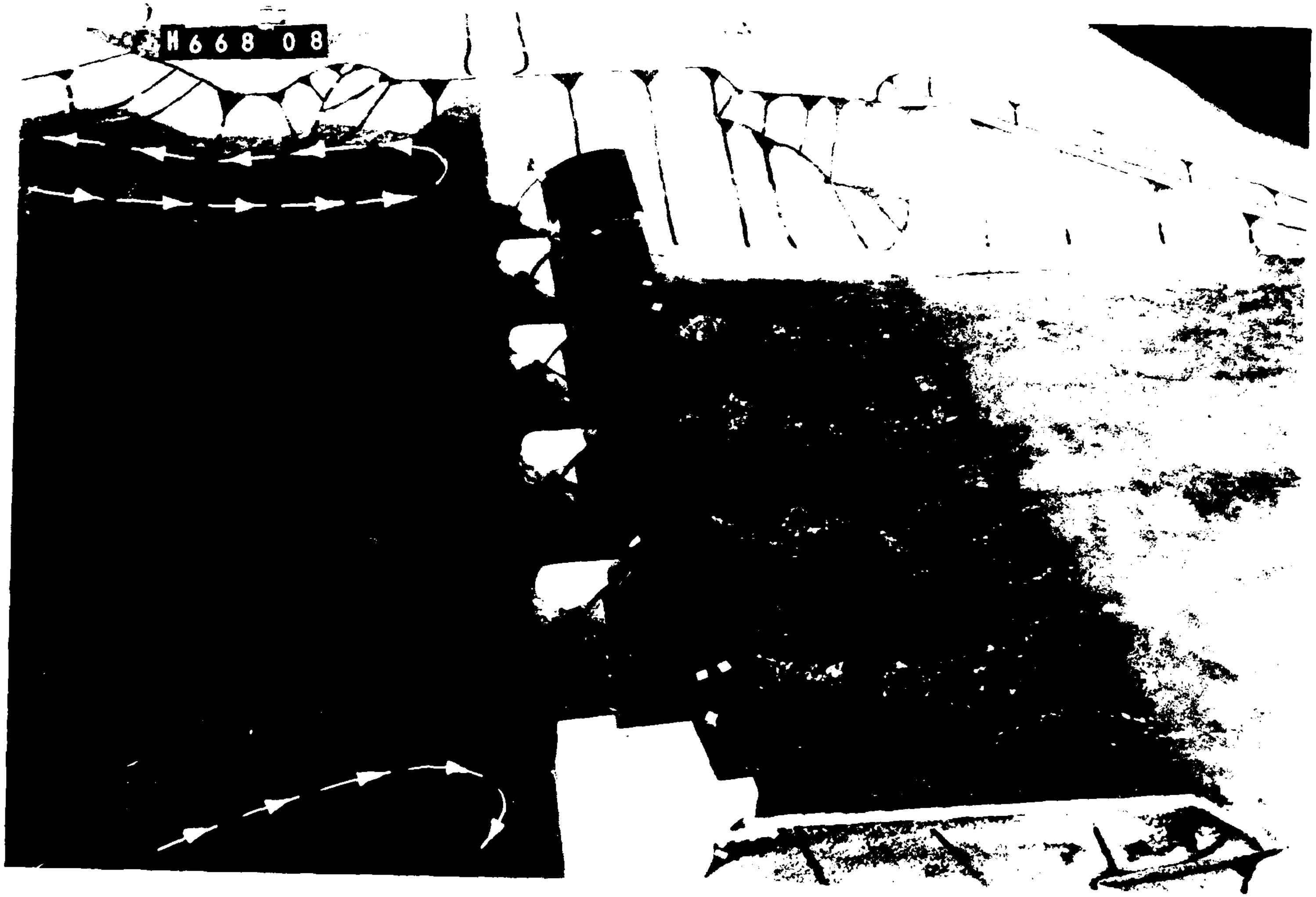


Photo 8. Spillway flow conditions; discharge 128,000 cfs, pool el 1504.8, gates fully open, exposure time [illegible] sec

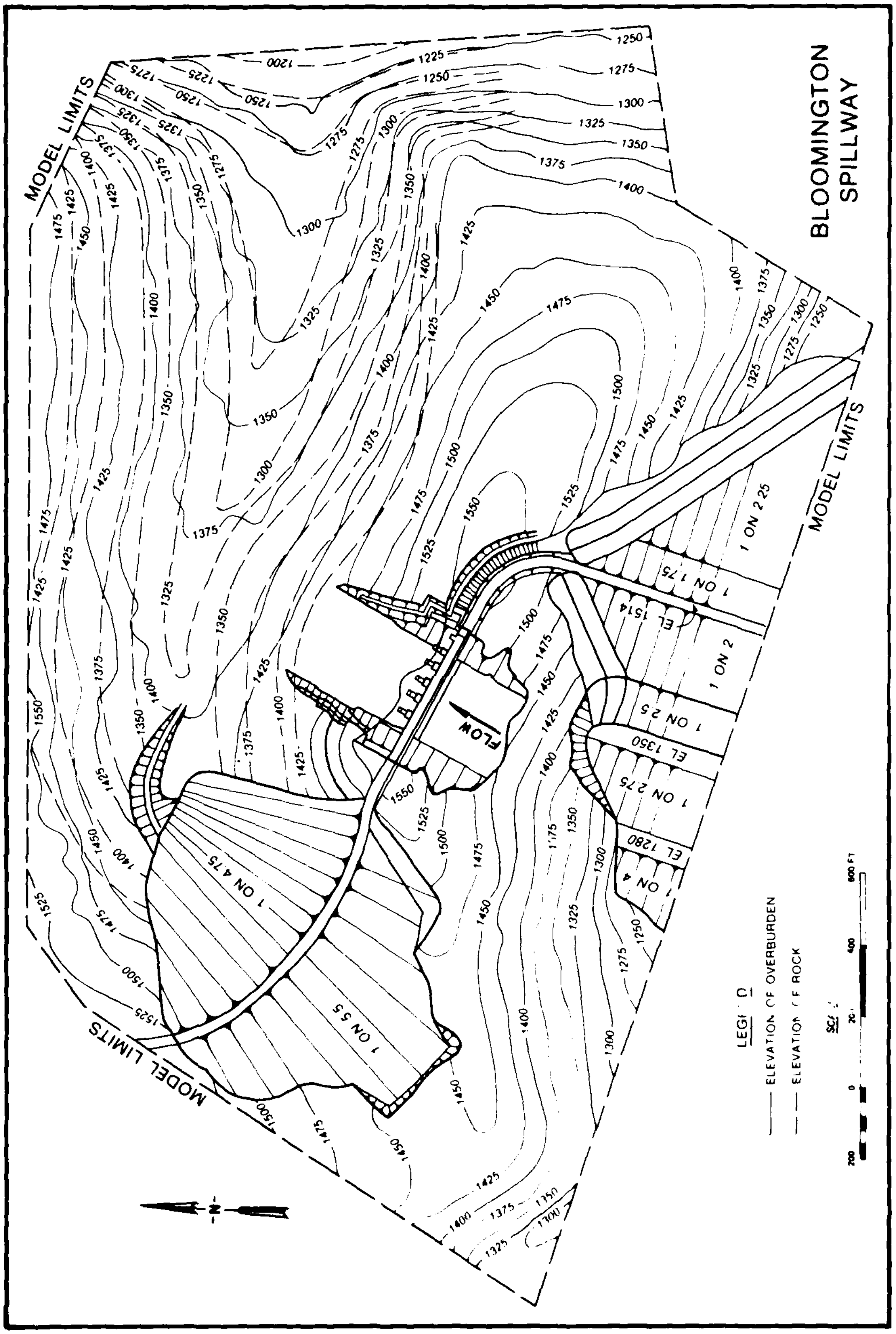

BLOOMINGTON SPILLWAY
MODEL LIMITS
FLOW
1 ON 2.25
1 ON 1.75
EL 1514
1 ON 2
1 ON 2.5
EL 1350
1 ON 2.75
EL 1280
1 ON 4
1 ON 4.75
1 ON 5.5
ELEVATION OF OVERBURDEN

PLATE 1

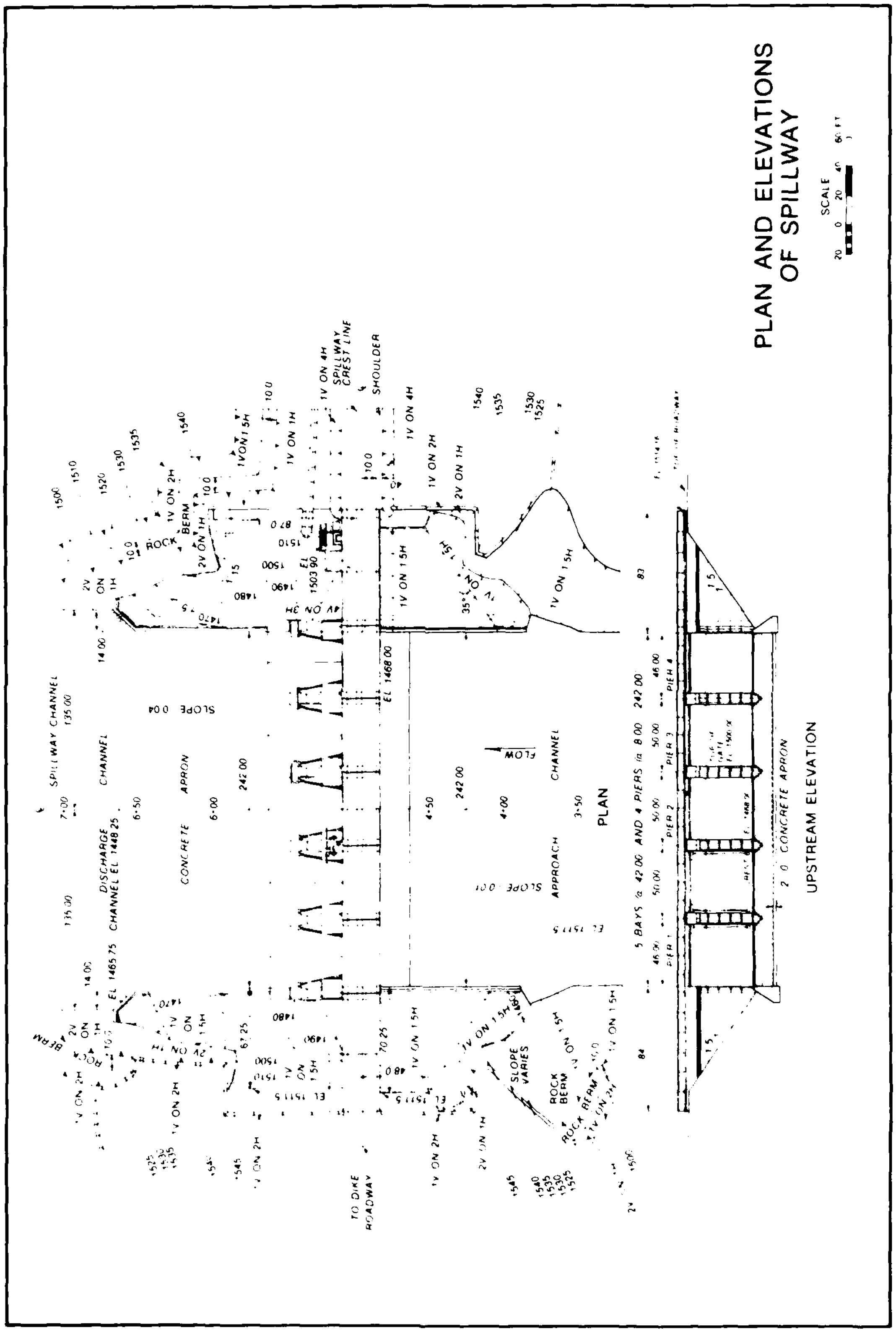
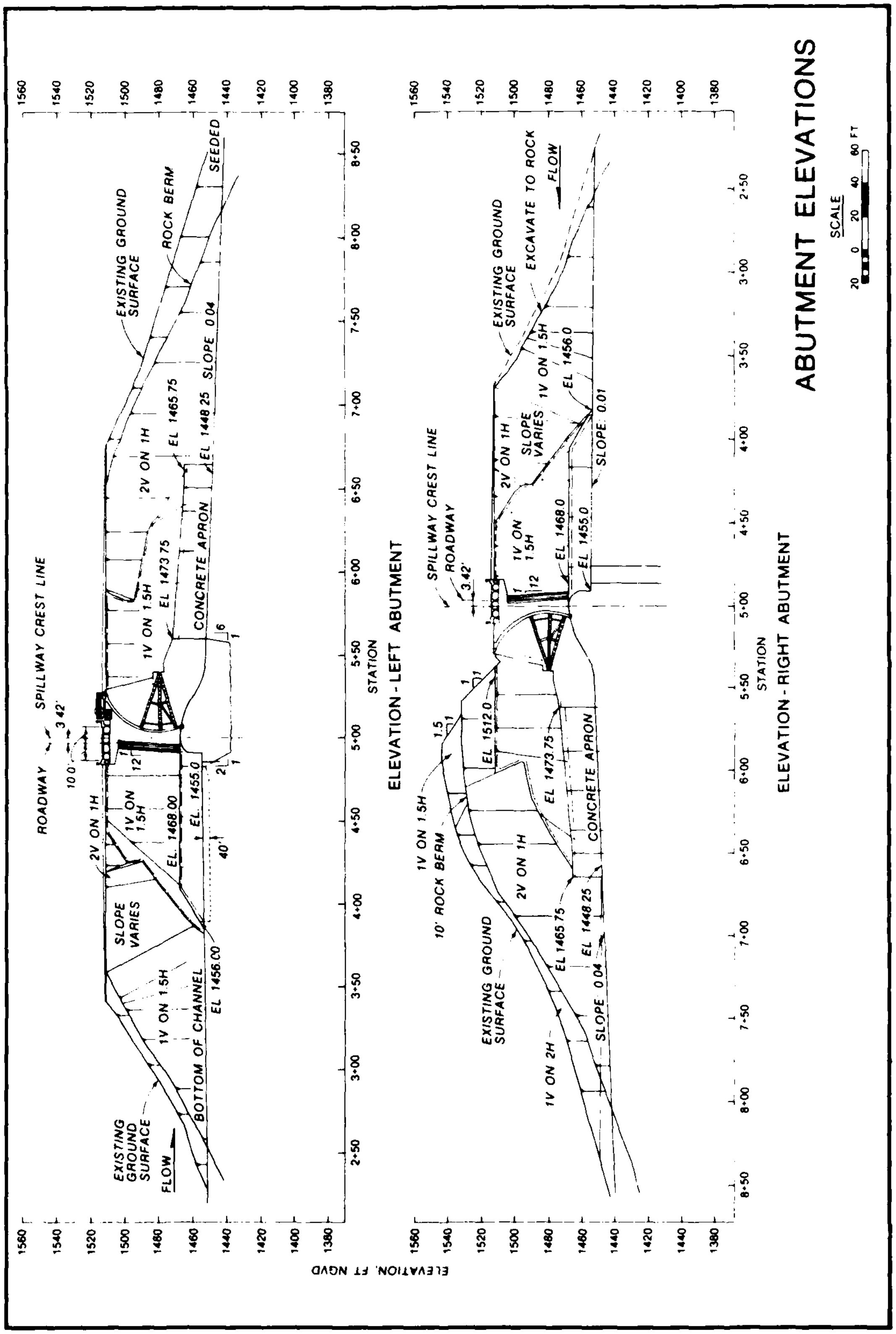

PLAN AND ELEVATIONS OF SPILLWAY
SCALE
PLAN
UPSTREAM ELEVATION
SPILLWAY CHANNEL
DISCHARGE CHANNEL EL 1448.25
CONCRETE APRON
APPROACH CHANNEL
FLOW
SLOPE 0.04
SLOPE 0.01
SPILLWAY CREST LINE
SHOULDER
ROCK BERM
SLOPE VARIES
TO DIKE ROADWAY
5 BAYS @ 42.00 AND 4 PIERS @ 8.00
2.0 CONCRETE APRON

PLATE 2


ABUTMENT ELEVATIONS
SCALE
20 0 20 40 60 FT
ELEVATION - LEFT ABUTMENT
ELEVATION - RIGHT ABUTMENT
STATION
ELEVATION, FT NGVD
FLOW
EXISTING GROUND SURFACE
BOTTOM OF CHANNEL
1V ON 1.5H
EL 1456.00
SLOPE VARIES
2V ON 1H
1V ON 1.5H
EL 1468.00
EL 1455.0
40'
ROADWAY
10.0'
3.42'
SPILLWAY CREST LINE
12
CONCRETE APRON
EL 1473.75
EL 1465.75
EL 1448.25
SLOPE 0.04
ROCK BERM
SEEDED
1V ON 2H
10' ROCK BERM
1V ON 1.5H
1.5
EL 1512.0
EXCAVATE TO ROCK
EL 1456.0
SLOPE 0.01
EL 1468.0

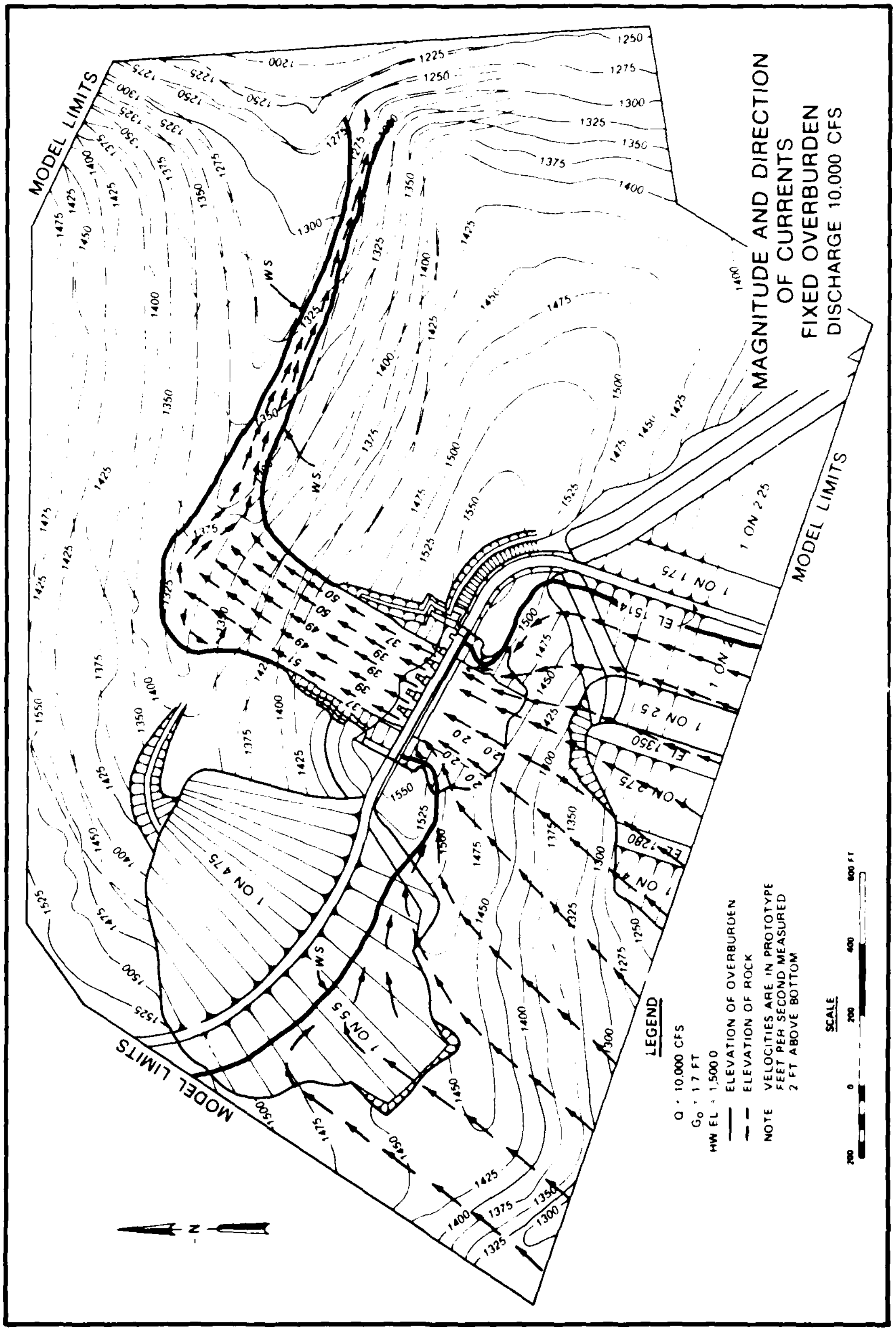
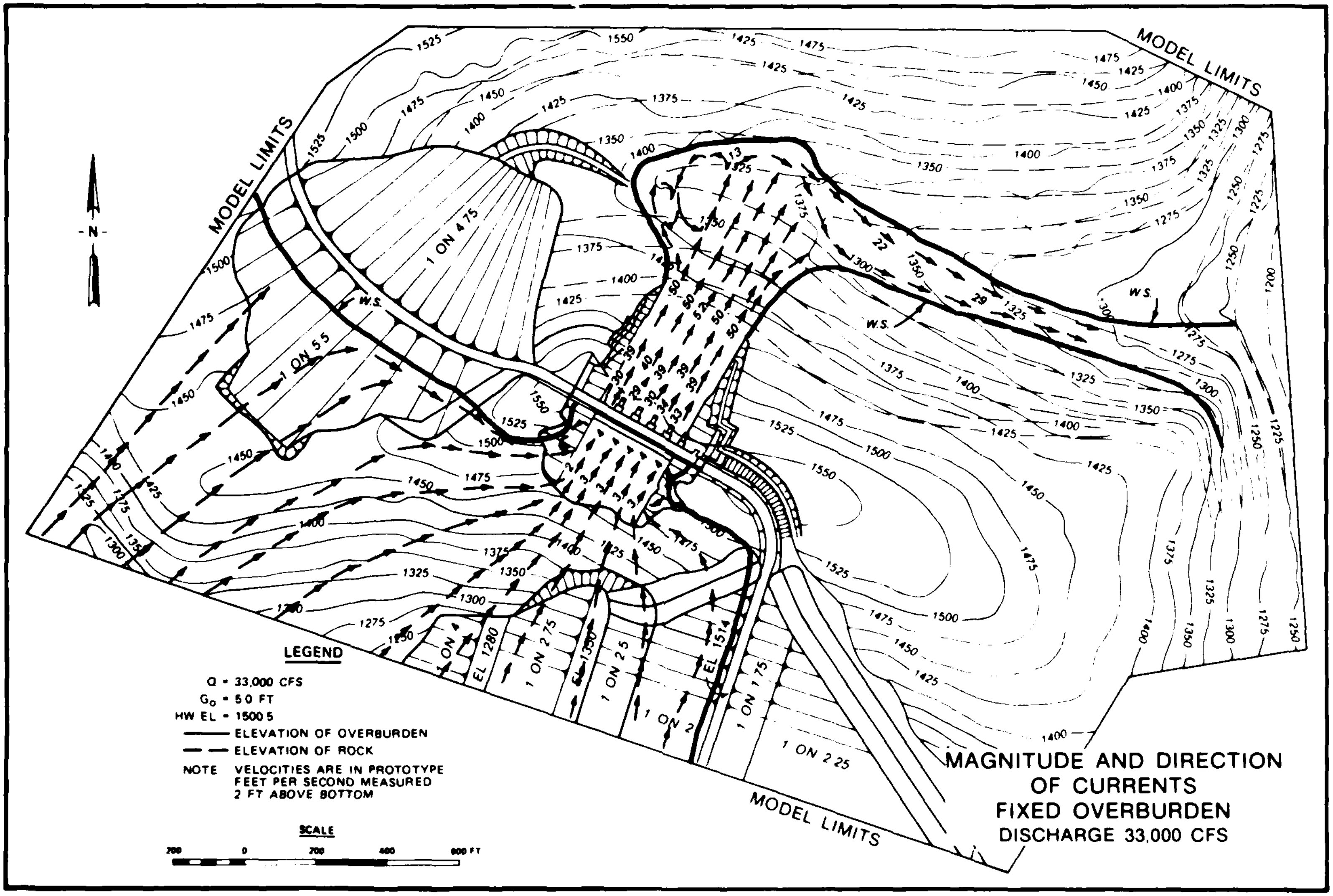

MAGNITUDE AND DIRECTION
OF CURRENTS
FIXED OVERBURDEN
DISCHARGE 10,000 CFS
LEGEND
Q = 10,000 CFS
G_o = 1.7 FT
HW EL = 1,500.0
ELEVATION OF OVERBURDEN
ELEVATION OF ROCK
NOTE VELOCITIES ARE IN PROTOTYPE FEET PER SECOND MEASURED 2 FT ABOVE BOTTOM
SCALE
200 0 200 400 600 FT
MODEL LIMITS
N

PLATE 4



MAGNITUDE AND DIRECTION
OF CURRENTS
FIXED OVERBURDEN
DISCHARGE 33,000 CFS

PLATE 5

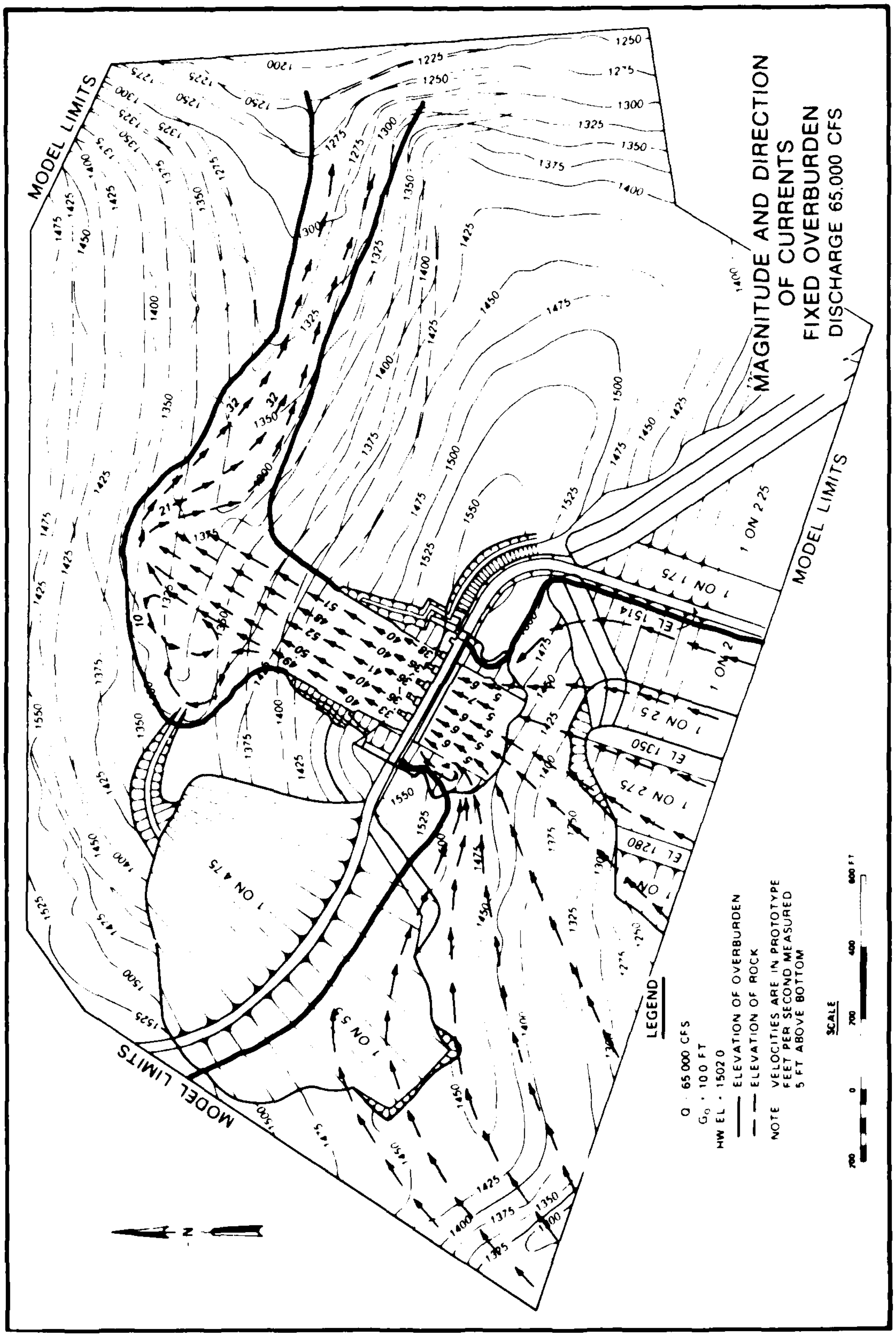

MAGNITUDE AND DIRECTION
OF CURRENTS
FIXED OVERBURDEN
DISCHARGE 65,000 CFS
MODEL LIMITS
LEGEND
Q = 65,000 CFS
G0 = 10.0 FT
HW EL = 1502.0
ELEVATION OF OVERBURDEN
ELEVATION OF ROCK
NOTE VELOCITIES ARE IN PROTOTYPE FEET PER SECOND MEASURED 5 FT ABOVE BOTTOM
SCALE

PLATE 6

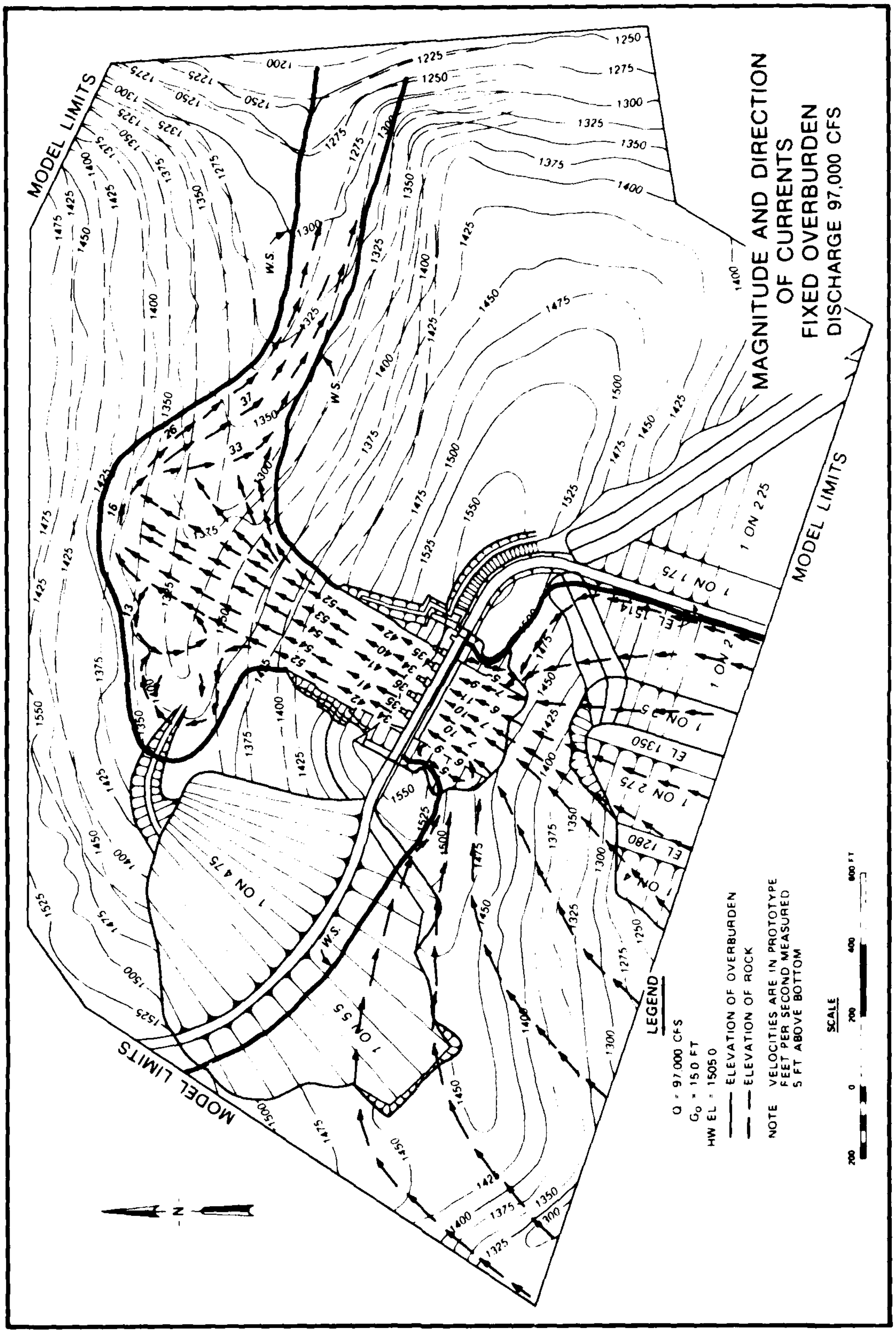

MAGNITUDE AND DIRECTION
OF CURRENTS
FIXED OVERBURDEN
DISCHARGE 97,000 CFS
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
W.S.
1 ON 4.75
1 ON 5.5
1 ON 2.75
1 ON 1.75
1 ON 2.25
EL 1350
EL 1280
LEGEND
Q = 97,000 CFS
Go = 15.0 FT
HW EL = 1505.0
ELEVATION OF OVERBURDEN
ELEVATION OF ROCK
NOTE VELOCITIES ARE IN PROTOTYPE FEET PER SECOND MEASURED 5 FT ABOVE BOTTOM
SCALE
N

PLATE 7

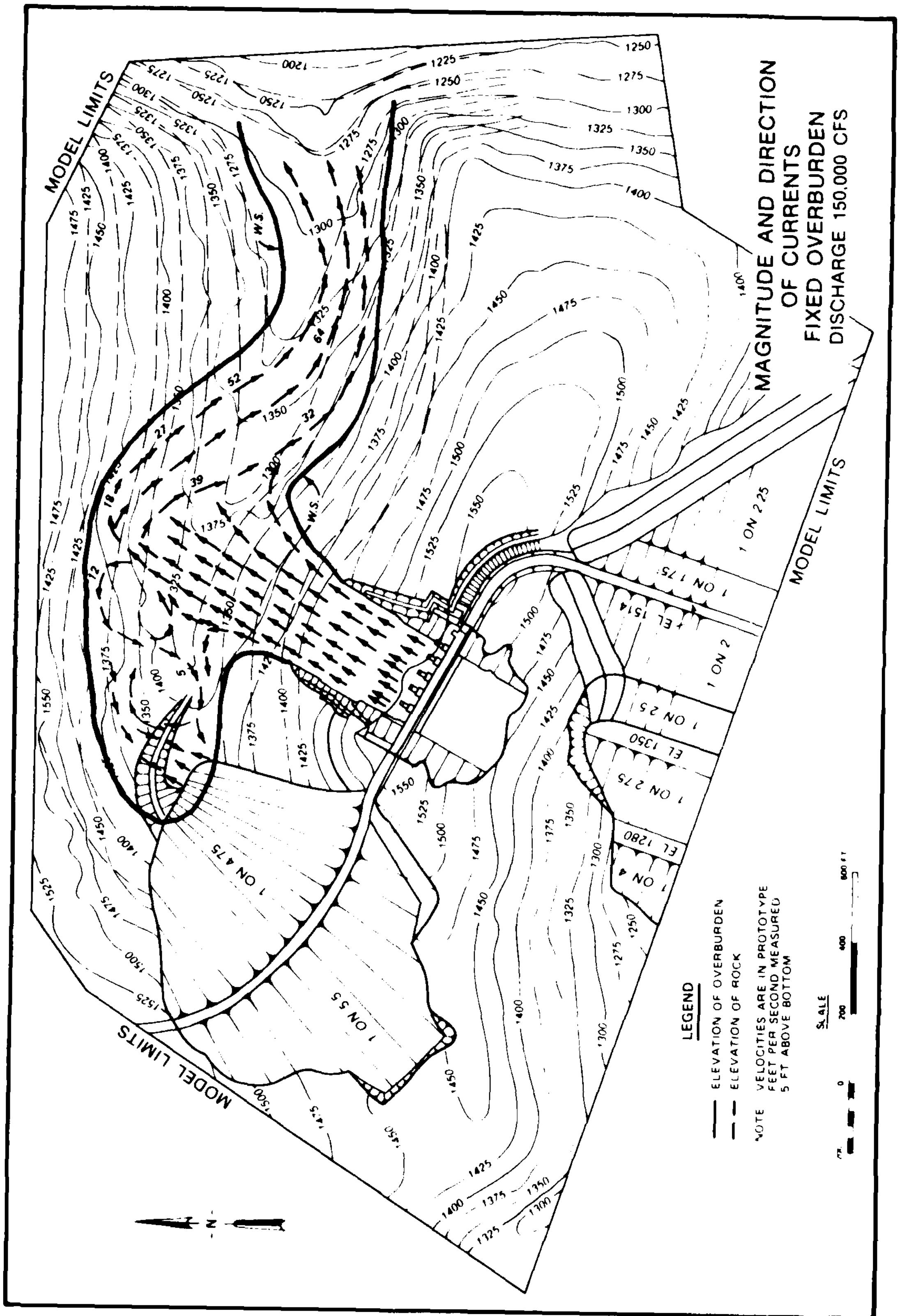

MAGNITUDE AND DIRECTION
OF CURRENTS
FIXED OVERBURDEN
DISCHARGE 150,000 CFS
LEGEND
ELEVATION OF OVERBURDEN
ELEVATION OF ROCK
NOTE VELOCITIES ARE IN PROTOTYPE FEET PER SECOND MEASURED 5 FT ABOVE BOTTOM
SCALE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
W.S.
W.S.
1 ON 4.75
1 ON 5.5
1 ON 2.25
1 ON 1.75
1 ON 2
1 ON 2.5
1 ON 2.75
1 ON 4
EL 1514
EL 1350
EL 1280

PLATE 8

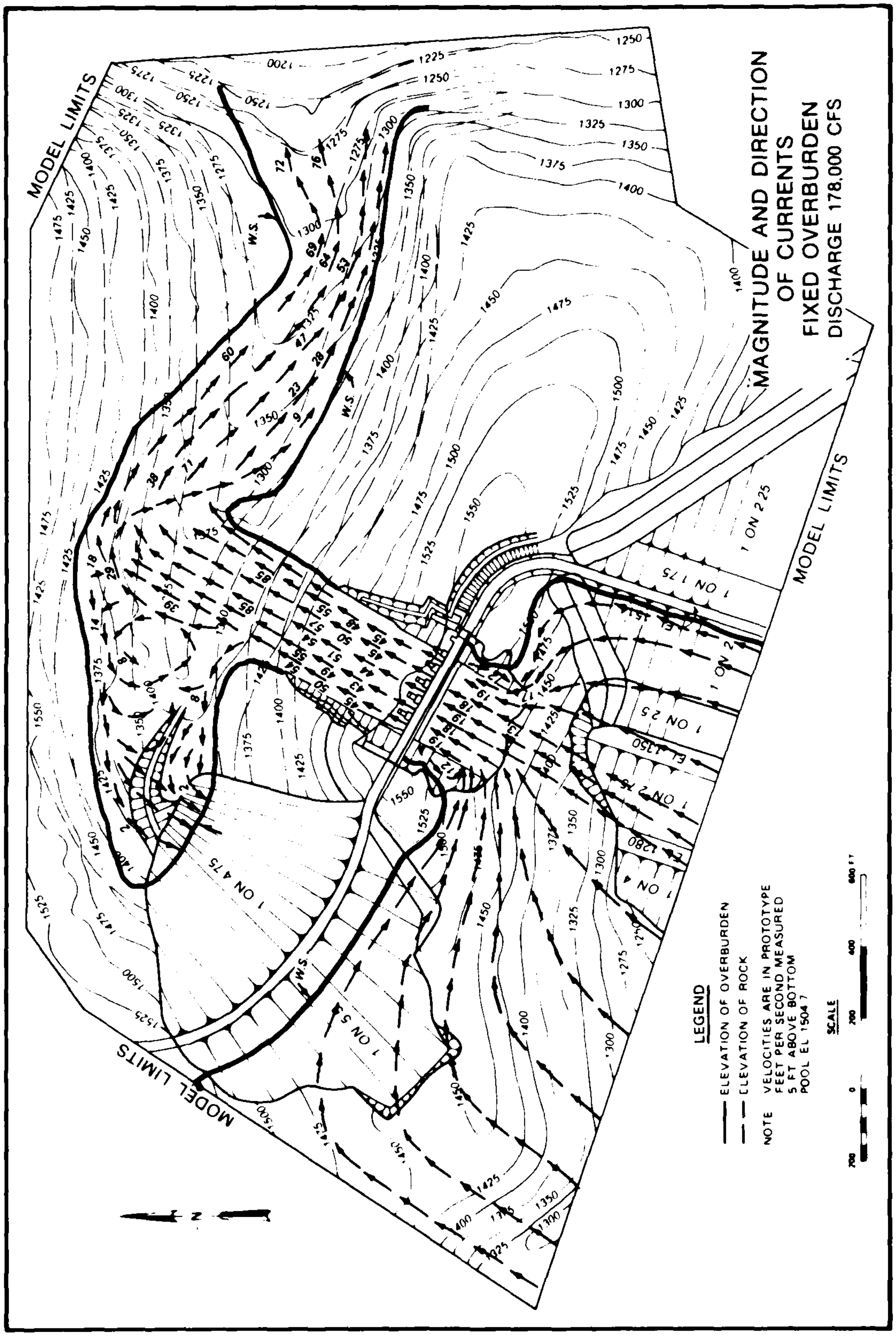
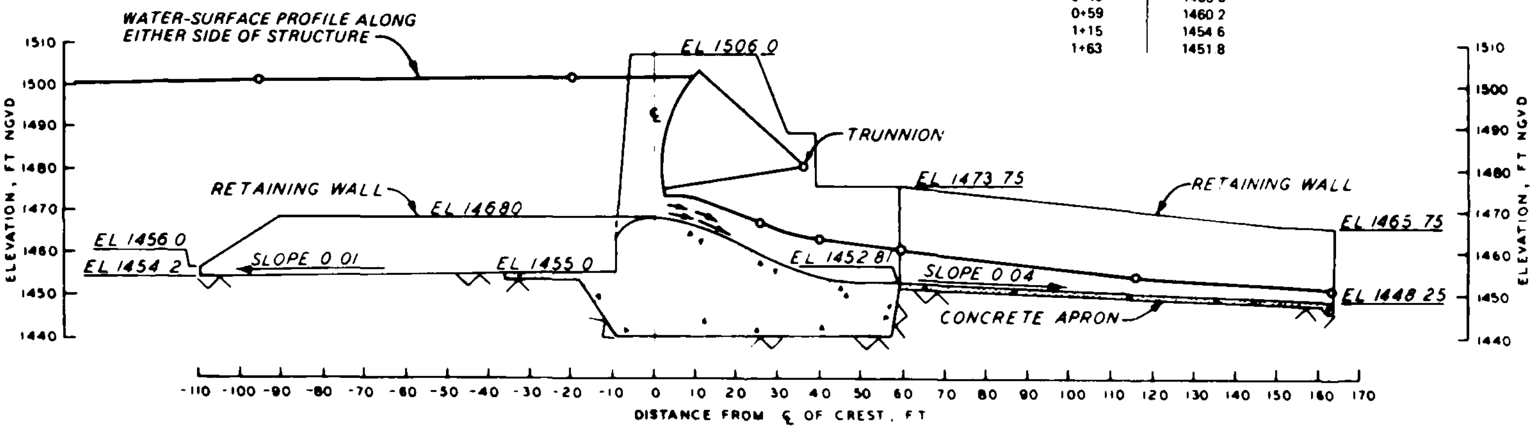

MAGNITUDE AND DIRECTION
OF CURRENTS
FIXED OVERBURDEN
DISCHARGE 178,000 CFS
LEGEND
ELEVATION OF OVERBURDEN
ELEVATION OF ROCK
NOTE VELOCITIES ARE IN PROTOTYPE FEET PER SECOND MEASURED 5 FT ABOVE BOTTOM POOL EL 1504.7
SCALE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
W.S.
W.S.
W.S.
1 ON 4.75
1 ON 5
1 ON 2.25
1 ON 1.75
1 ON 2
1 ON 2.5
EL 1350
1 ON 2.75

PLATE 9

BASIC DATA

DIST FROM ℄	EL, FT NGVD
-0+95	1500 5
-0+20	1500 5
0+25	1466.5
0+40	1463 0
0+59	1460 2
1+15	1454 6
1+63	1451 8



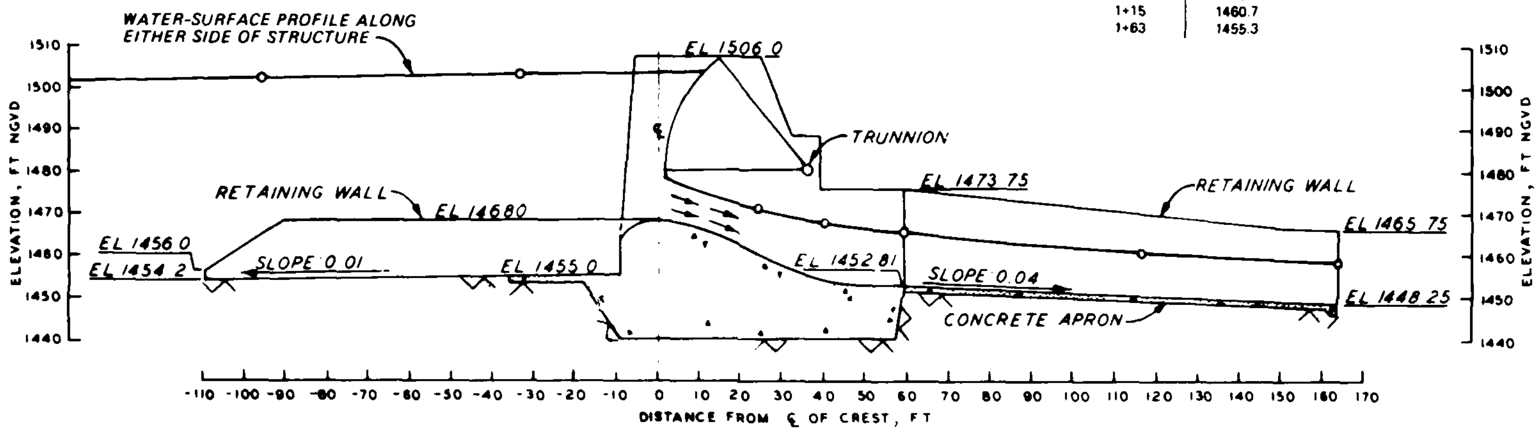
TEST CONDITIONS

Q = 33,000 CFS
G_o = 50 FT

WATER-SURFACE PROFILE
DISCHARGE 33,000 CFS

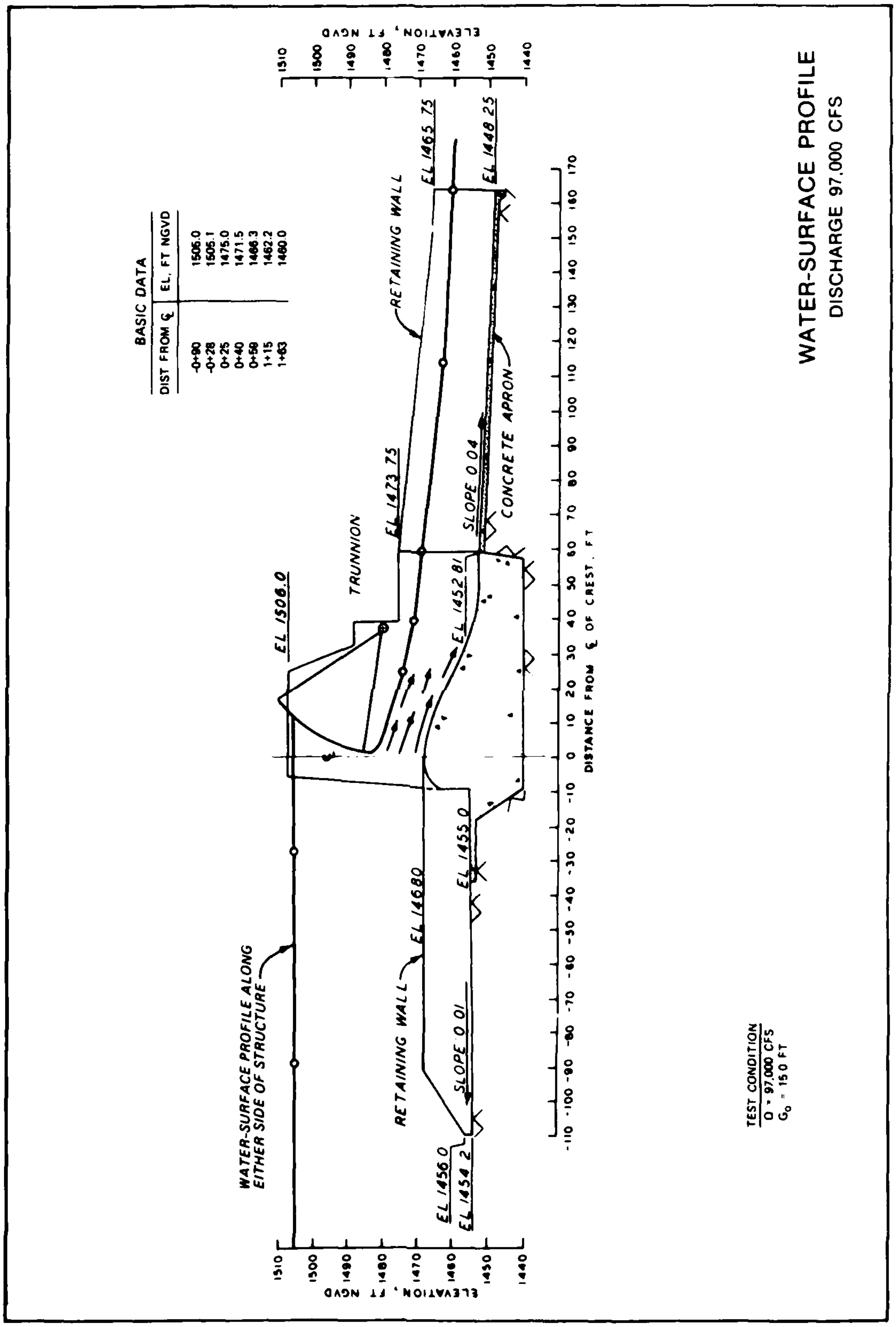
PLATE 10

BASIC DATA	
DIST FROM ℄	EL. FT NGVD
-0+95	1502.0
-0+35	1502.2
0+25	1471.4
0+40	1467.0
0+59	1465.3
1+15	1460.7
1+63	1455.3



TEST CONDITION
Q = 65,000 CFS
G_o = 10.0 FT

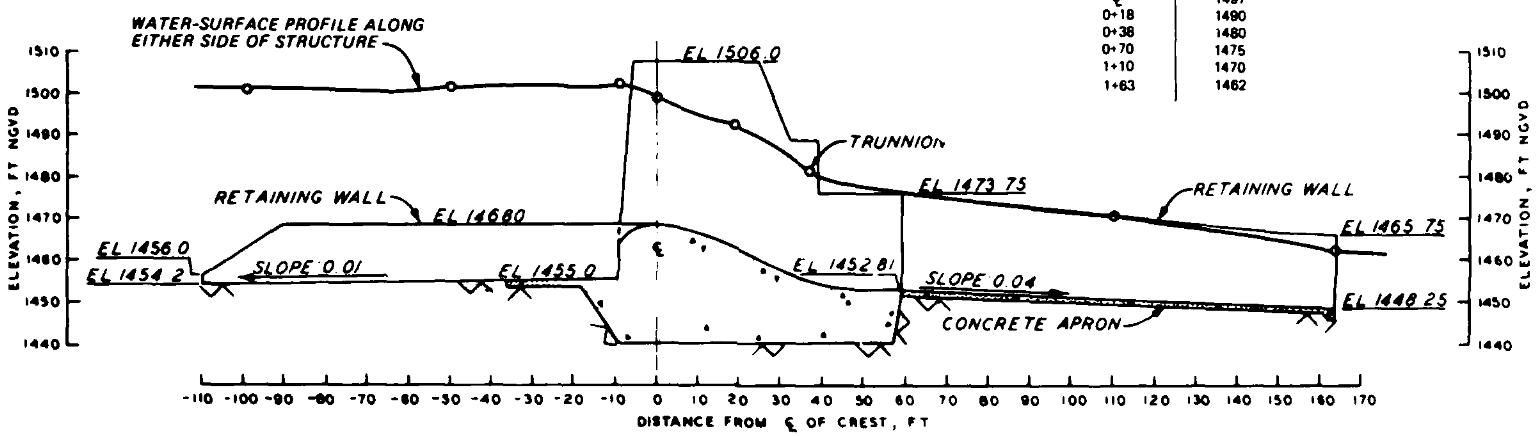
WATER-SURFACE PROFILE
DISCHARGE 65,000 CFS



DIST FROM ℄	EL, FT NGVD
-0+90	1505.0
-0+28	1505.1
0+25	1475.0
0+40	1471.5
0+58	1466.3
1+15	1462.2
1+63	1460.0

BASIC DATA

DIST FROM ℄	EL, FT NGVD
-1+00	1500
-0+50	1500
-0+10	1502
℄	1497
0+18	1490
0+38	1480
0+70	1475
1+10	1470
1+63	1462



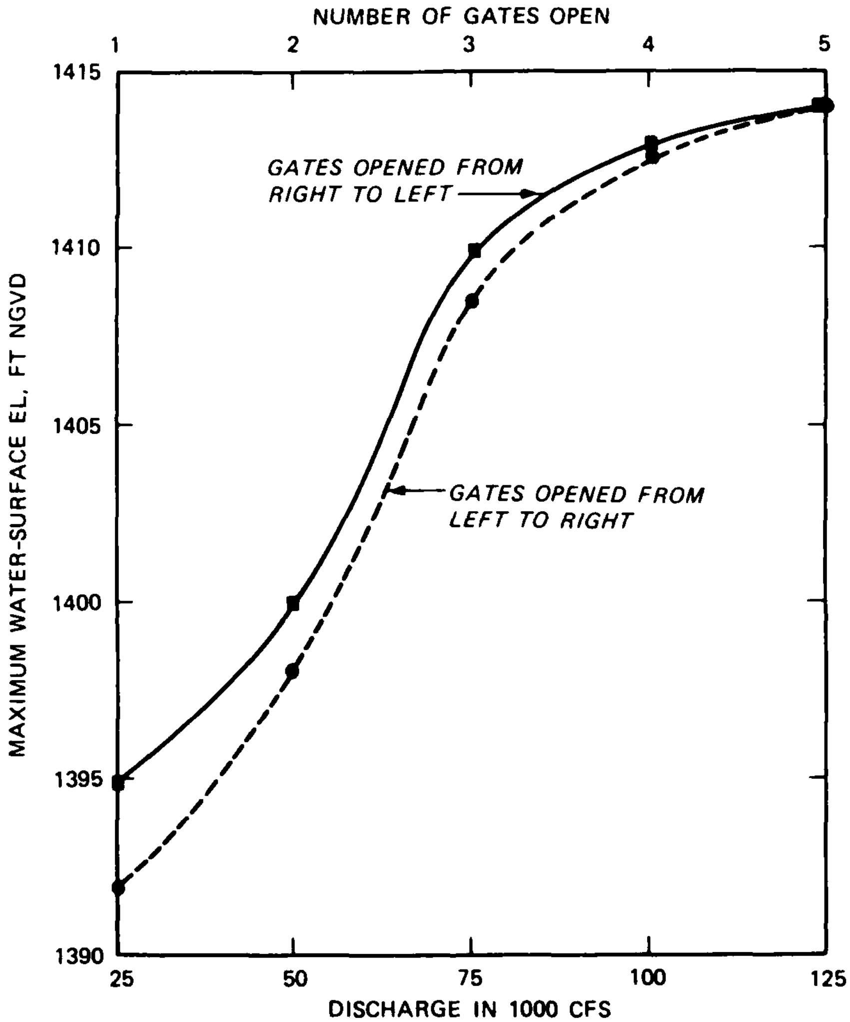
TEST CONDITION

Q = 178,000 CFS

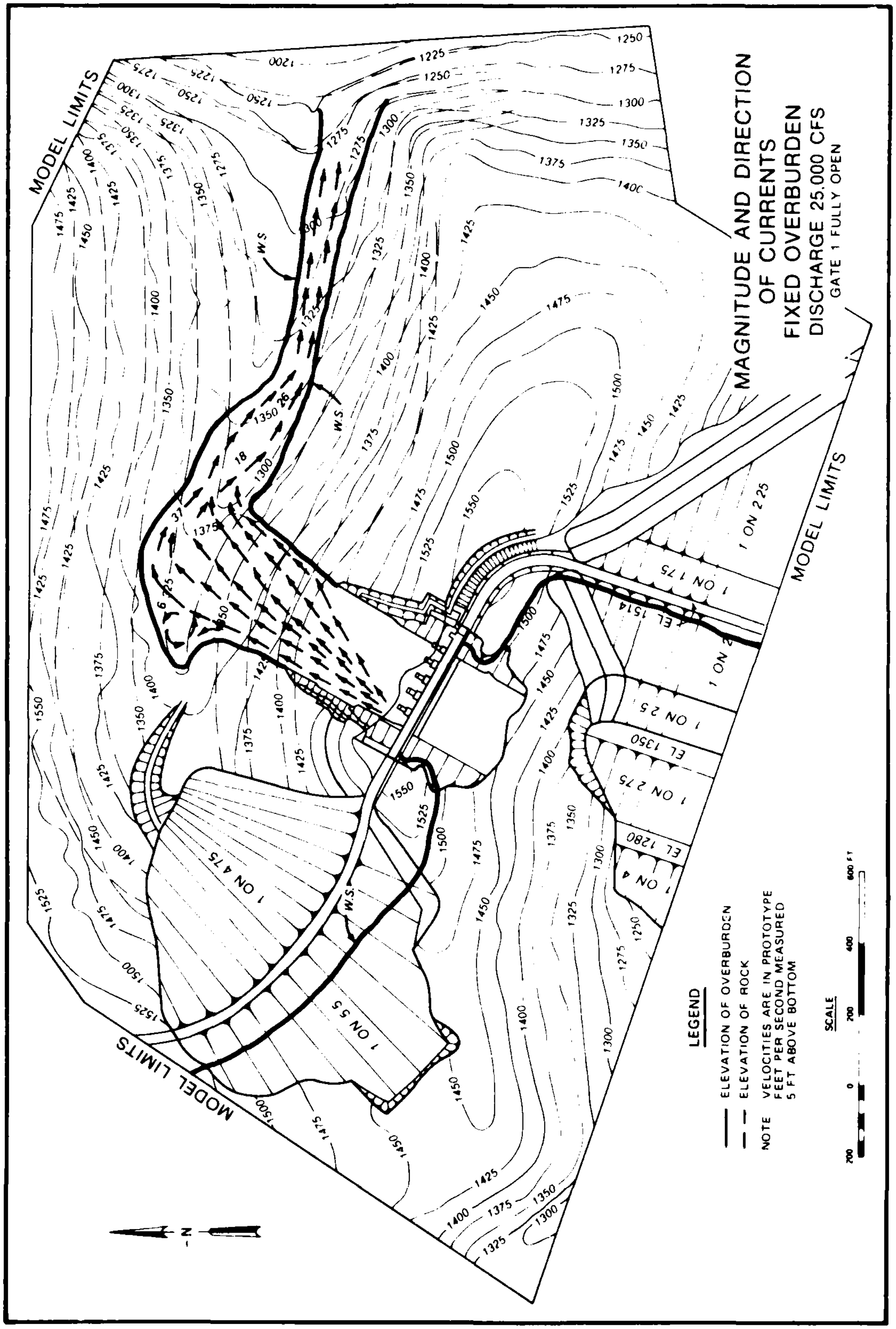
G_o = ALL GATES FULLY OPEN

WATER-SURFACE PROFILE

DISCHARGE 178,000 CFS



FIXED OVERBURDEN
TAINTER GATES OPENED IN SEQUENCE
MAXIMUM WATER-SURFACE ELEVATION IN RETURN FLOW TOWARD DIKE


MAGNITUDE AND DIRECTION
OF CURRENTS
FIXED OVERBURDEN
DISCHARGE 25,000 CFS
GATE 1 FULLY OPEN
MODEL LIMITS
LEGEND
ELEVATION OF OVERBURDEN
ELEVATION OF ROCK
NOTE VELOCITIES ARE IN PROTOTYPE FEET PER SECOND MEASURED 5 FT ABOVE BOTTOM
SCALE
1 ON 4.75
1 ON 5.5
1 ON 2.25
1 ON 1.75
1 ON 2.5
1 ON 2.75
1 ON 4
EL 1514
EL 1350
EL 1280
W.S.

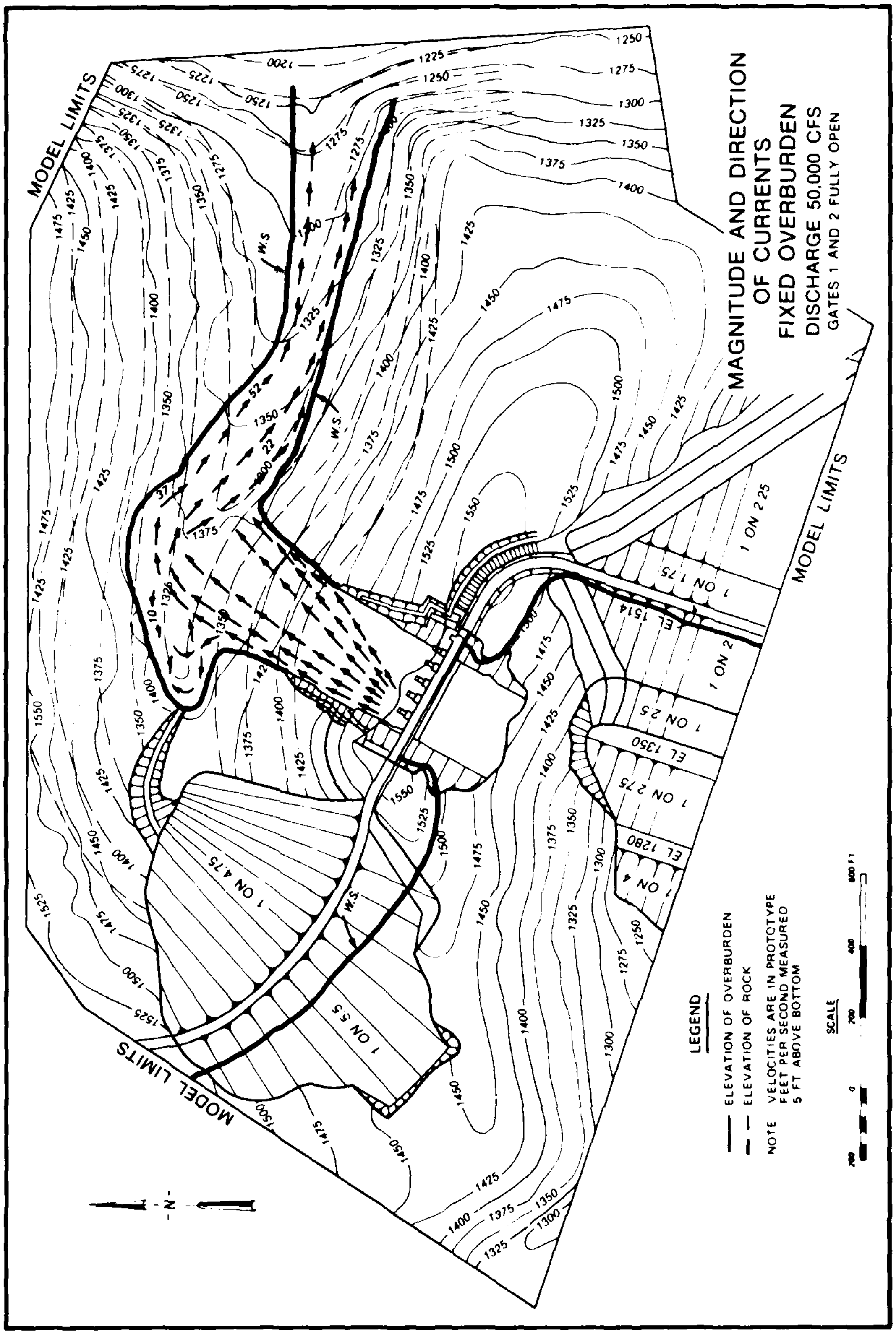
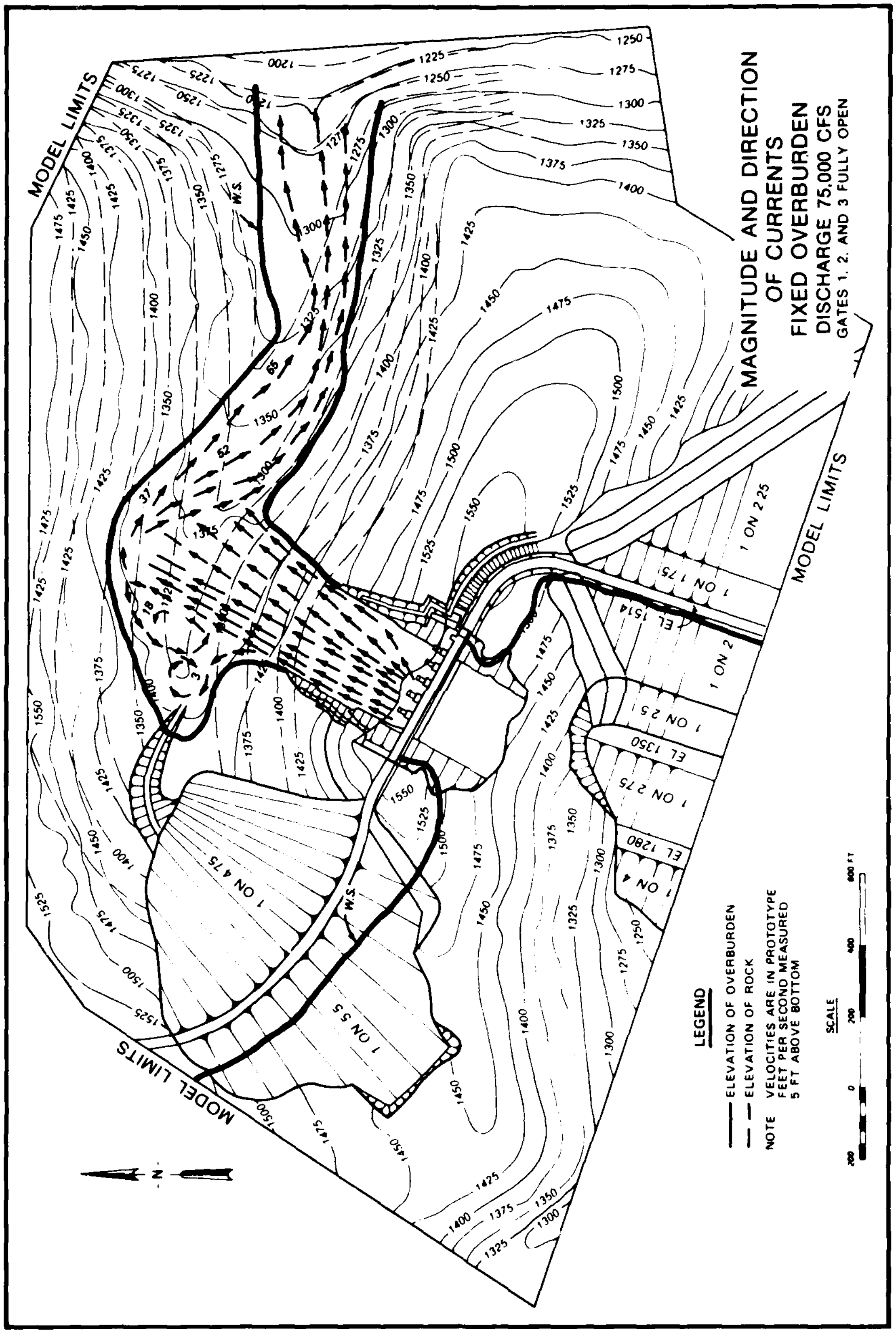

MAGNITUDE AND DIRECTION
OF CURRENTS
FIXED OVERBURDEN
DISCHARGE 50,000 CFS
GATES 1 AND 2 FULLY OPEN
LEGEND
ELEVATION OF OVERBURDEN
ELEVATION OF ROCK
NOTE VELOCITIES ARE IN PROTOTYPE FEET PER SECOND MEASURED 5 FT ABOVE BOTTOM
SCALE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
N
1 ON 2.25
1 ON 1.75
EL 1514
1 ON 2
1 ON 2.5
EL 1350
1 ON 2.75
EL 1280
1 ON 4
1 ON 4.75
1 ON 5.5
W.S.
W.S.
W.S.

PLATE 16


MAGNITUDE AND DIRECTION
OF CURRENTS
FIXED OVERBURDEN
DISCHARGE 75,000 CFS
GATES 1, 2, AND 3 FULLY OPEN
LEGEND
ELEVATION OF OVERBURDEN
ELEVATION OF ROCK
NOTE VELOCITIES ARE IN PROTOTYPE FEET PER SECOND MEASURED 5 FT ABOVE BOTTOM
SCALE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
1 ON 4.75
1 ON 5.5
1 ON 2.25
1 ON 1.75
1 ON 2
1 ON 2.5
1 ON 2.75
1 ON 4
EL 1514
EL 1350
EL 1280
W.S.
W.S.

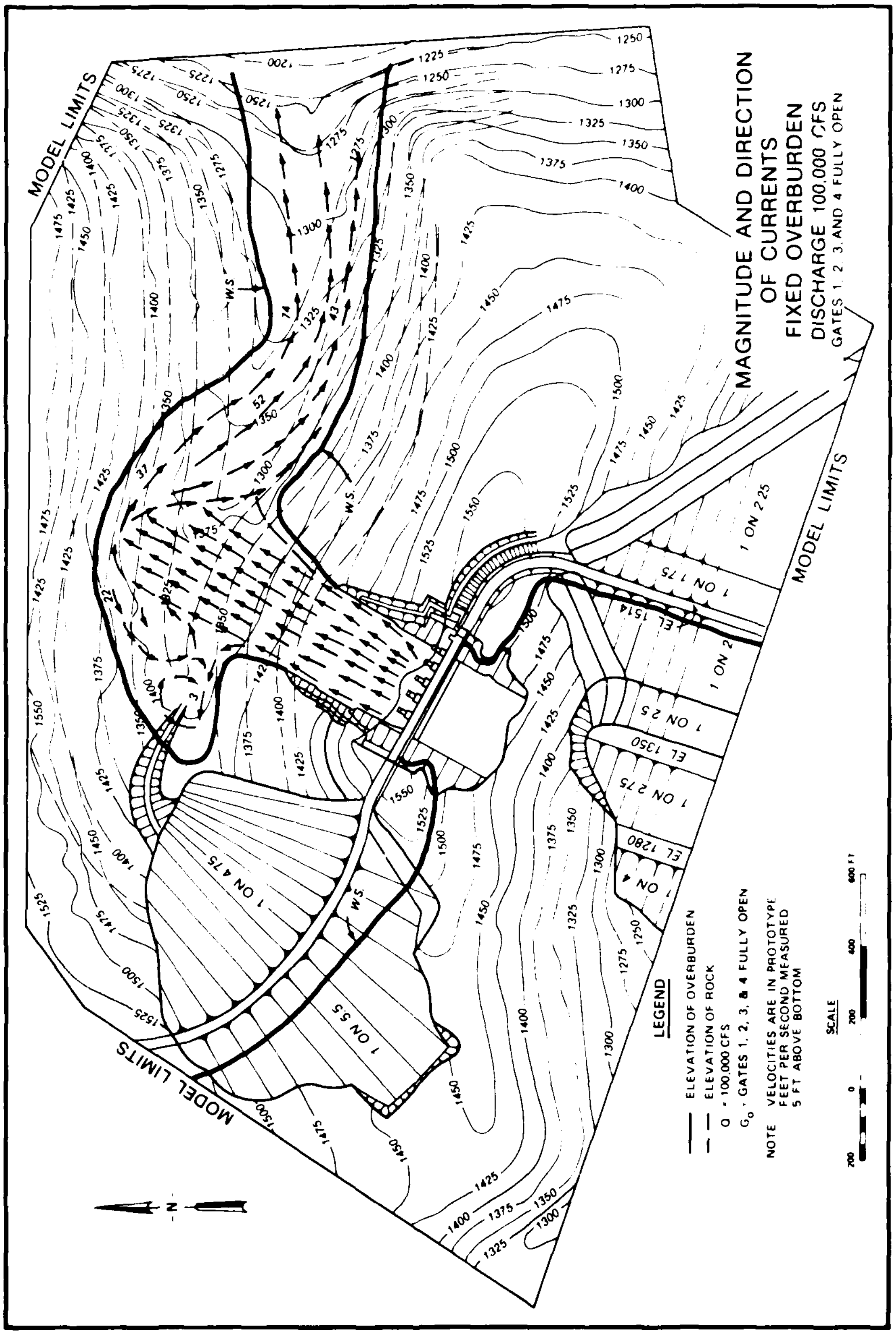
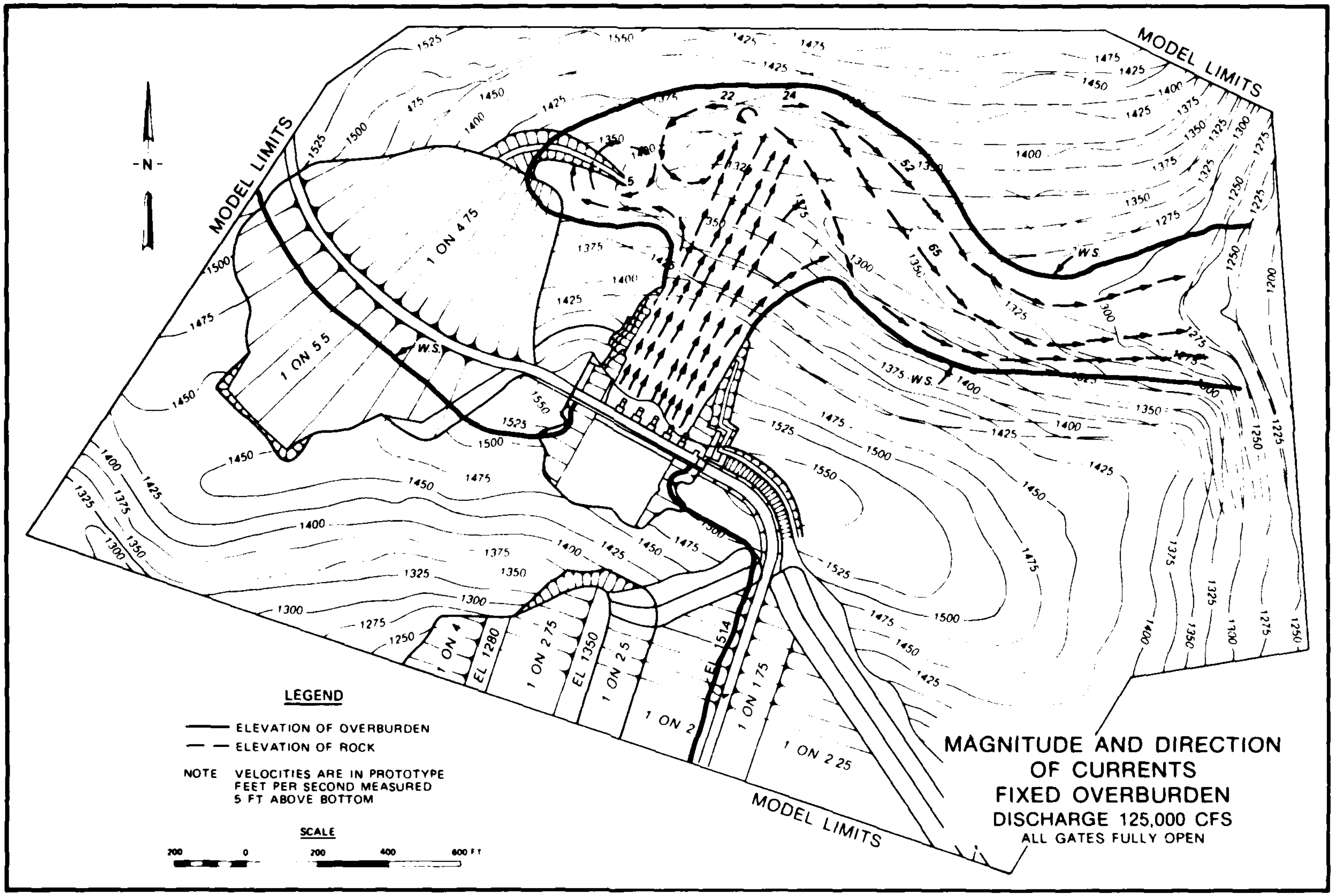

MAGNITUDE AND DIRECTION
OF CURRENTS
FIXED OVERBURDEN
DISCHARGE 100,000 CFS
GATES 1, 2, 3, AND 4 FULLY OPEN
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
W.S.
W.S.
W.S.
1 ON 4.75
1 ON 5.5
1 ON 2.25
1 ON 1.75
EL 1514
1 ON 2
1 ON 2.5
EL 1350
1 ON 2.75
EL 1280
1 ON 4
LEGEND
ELEVATION OF OVERBURDEN
ELEVATION OF ROCK
Q = 100,000 CFS
G_o - GATES 1, 2, 3, & 4 FULLY OPEN
NOTE VELOCITIES ARE IN PROTOTYPE FEET PER SECOND MEASURED 5 FT ABOVE BOTTOM
SCALE
N

PLATE 18



MAGNITUDE AND DIRECTION
OF CURRENTS
FIXED OVERBURDEN
DISCHARGE 125,000 CFS
ALL GATES FULLY OPEN

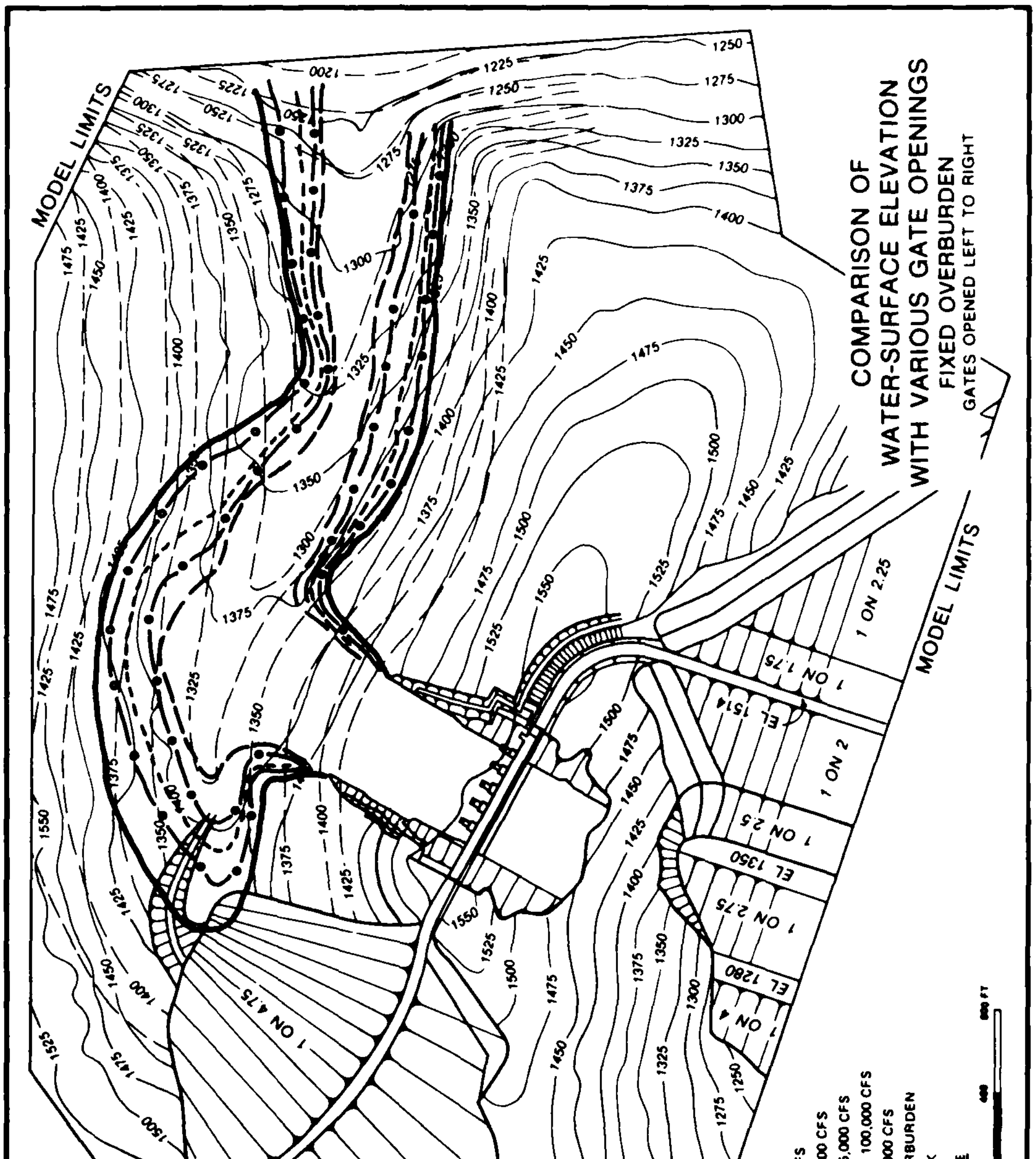

COMPARISON OF
WATER-SURFACE ELEVATION
WITH VARIOUS GATE OPENINGS
FIXED OVERBURDEN
GATES OPENED LEFT TO RIGHT
LEGEND
GATE 1, Q = 25,000 CFS
GATES 1 & 2, Q = 50,000 CFS
GATES 1, 2 & 3, Q = 75,000 CFS
GATES 1, 2, 3 & 4, Q = 100,000 CFS
ALL GATES, Q = 125,000 CFS
ELEVATION OF OVERBURDEN
ELEVATION OF ROCK
SCALE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
N
1 ON 4.75
1 ON 5.5
1 ON 2.25
1 ON 1.75
EL 1514
1 ON 2
1 ON 2.5
EL 1350
1 ON 2.75
EL 1280
1 ON 4

PLATE 20

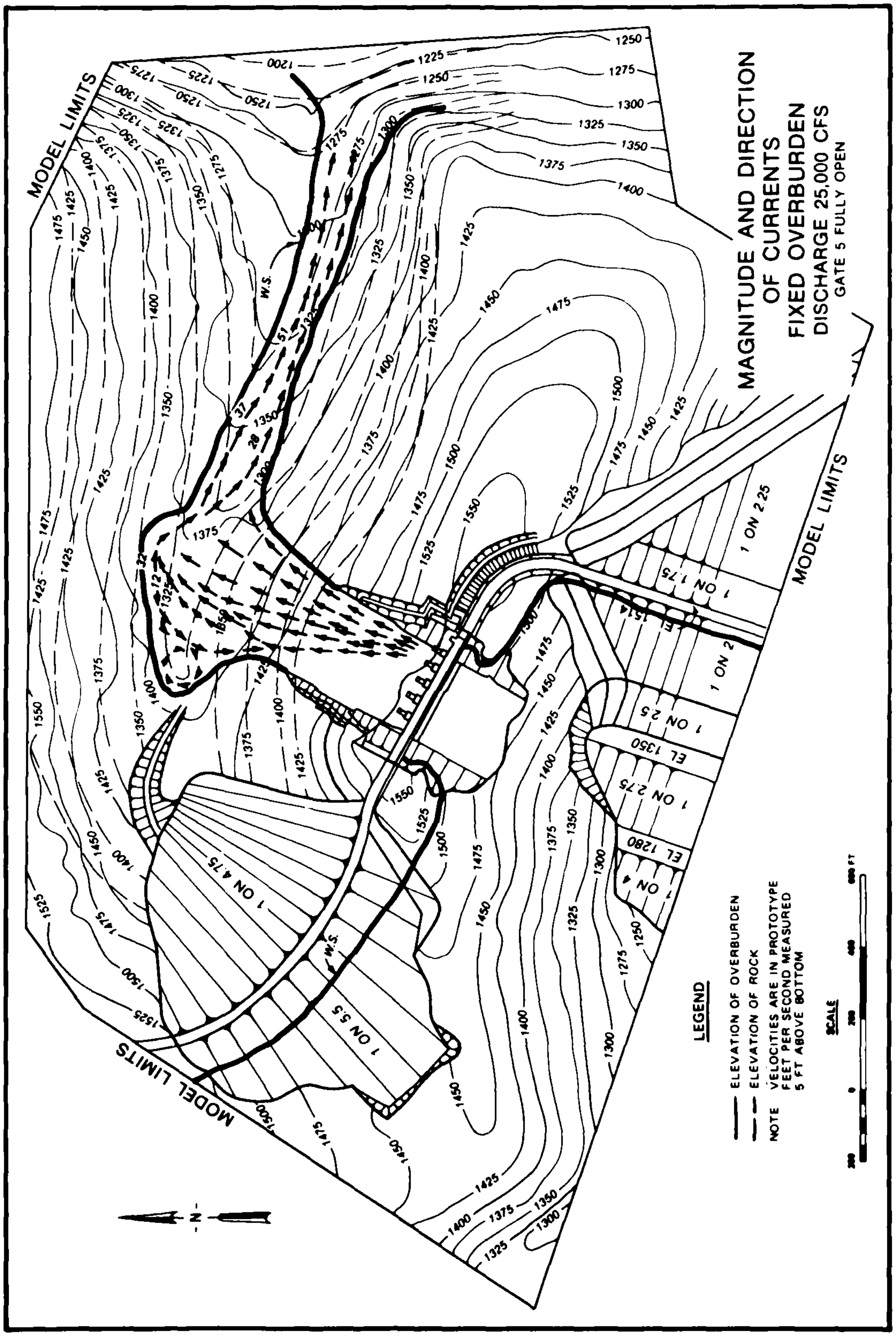
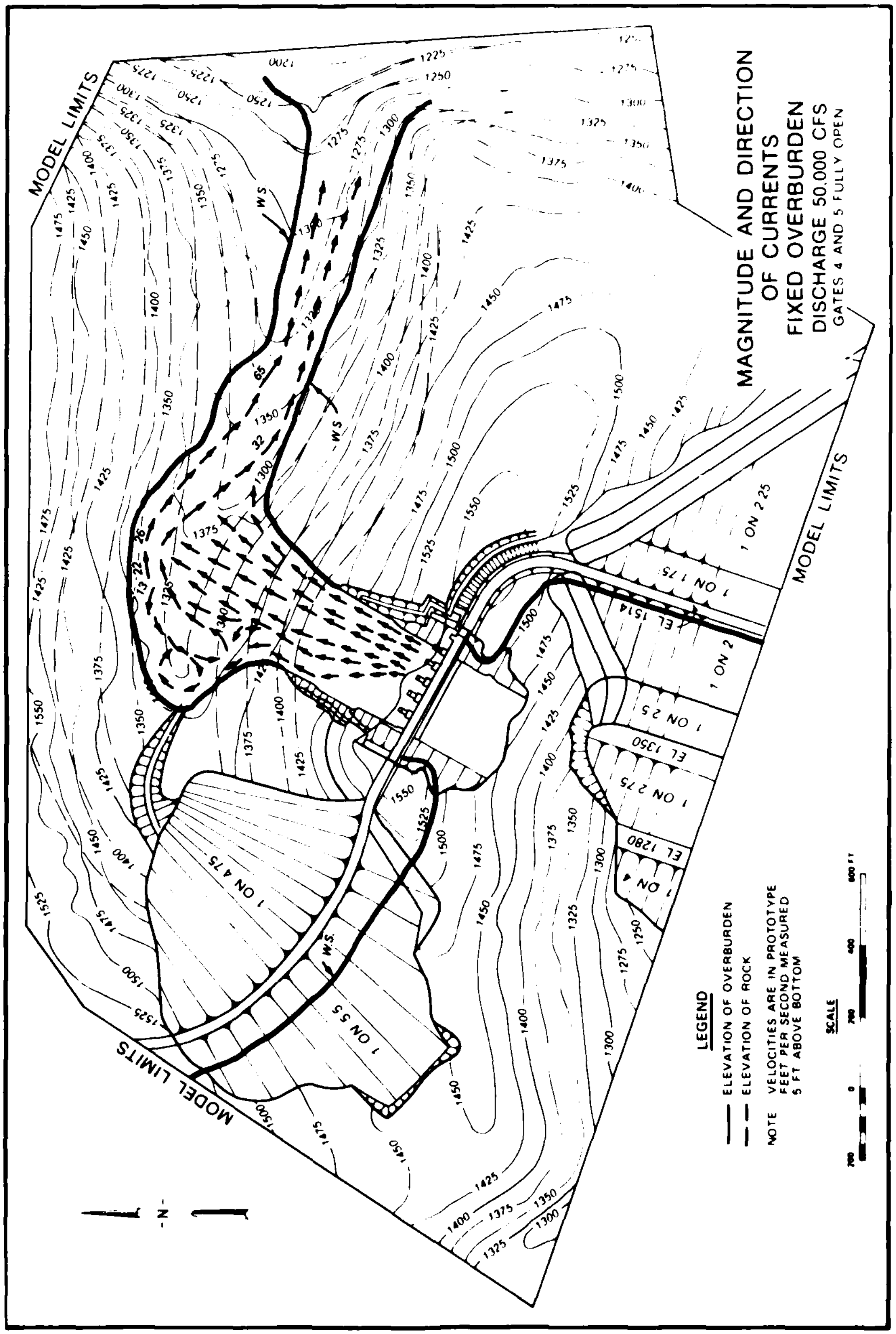

MAGNITUDE AND DIRECTION
OF CURRENTS
FIXED OVERBURDEN
DISCHARGE 25,000 CFS
GATE 5 FULLY OPEN
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
W.S.
W.S.
1 ON 4.75
1 ON 5.5
1 ON 2.25
1 ON 1.75
1 ON 2
1 ON 2.5
EL 1350
1 ON 2.75
EL 1280
1 ON 4
LEGEND
ELEVATION OF OVERBURDEN
ELEVATION OF ROCK
NOTE VELOCITIES ARE IN PROTOTYPE FEET PER SECOND MEASURED 5 FT ABOVE BOTTOM
SCALE
N

PLATE 21



MAGNITUDE AND DIRECTION
OF CURRENTS
FIXED OVERBURDEN
DISCHARGE 50,000 CFS
GATES 4 AND 5 FULLY OPEN
LEGEND
ELEVATION OF OVERBURDEN
ELEVATION OF ROCK
NOTE VELOCITIES ARE IN PROTOTYPE FEET PER SECOND MEASURED 5 FT ABOVE BOTTOM
SCALE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
W.S.
1 ON 4.75
1 ON 5.5
1 ON 2.25
1 ON 1.75
1 ON 2
1 ON 2.5
1 ON 2.75
EL 1514
EL 1350
EL 1280
-N-

PLATE 22

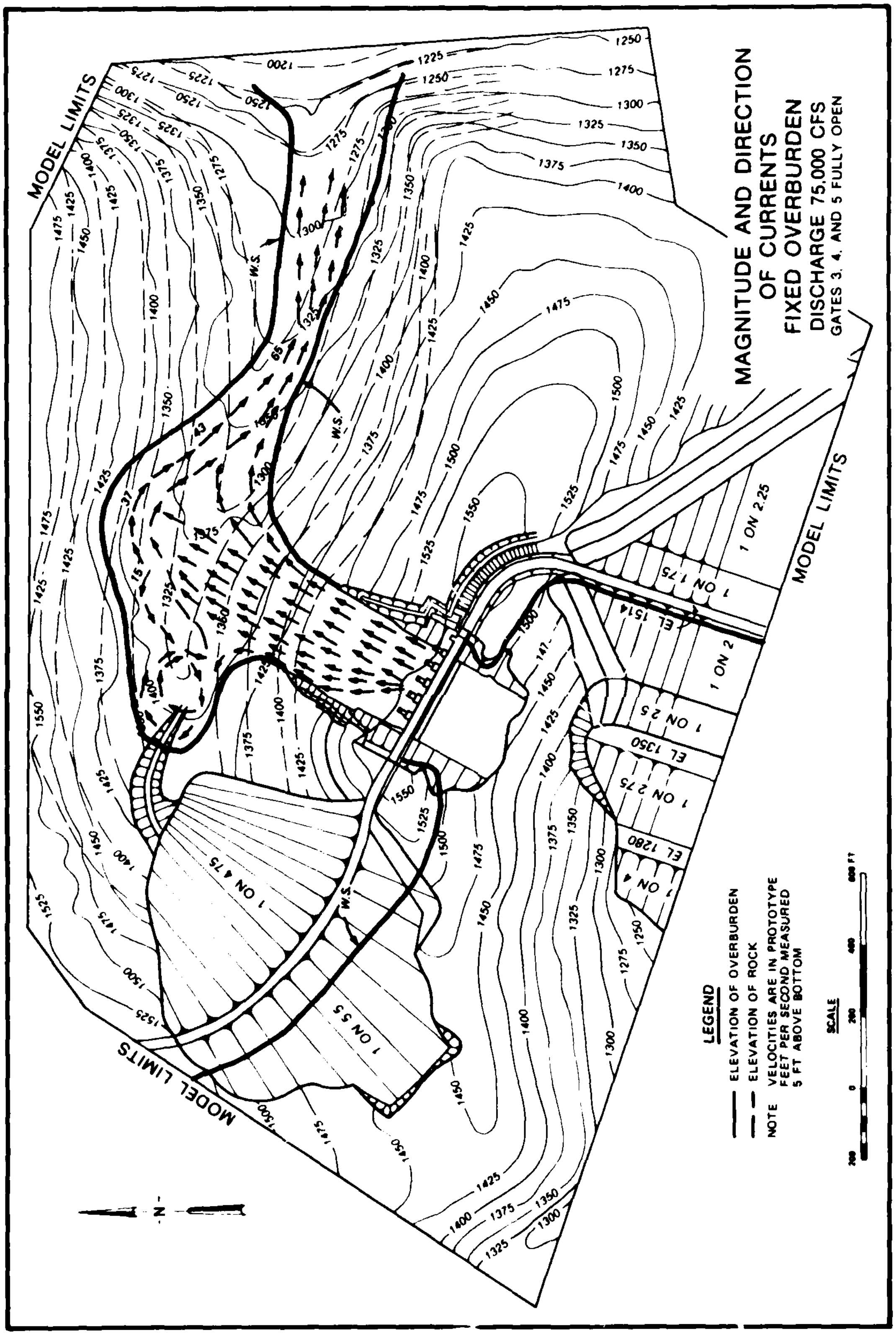

MAGNITUDE AND DIRECTION
OF CURRENTS
FIXED OVERBURDEN
DISCHARGE 75,000 CFS
GATES 3, 4, AND 5 FULLY OPEN
LEGEND
ELEVATION OF OVERBURDEN
ELEVATION OF ROCK
NOTE VELOCITIES ARE IN PROTOTYPE FEET PER SECOND MEASURED 5 FT ABOVE BOTTOM
SCALE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
1 ON 2.25
1 ON 1.75
EL 1514
1 ON 2
1 ON 2.5
EL 1350
1 ON 2.75
EL 1280
1 ON 4.75
1 ON 5.5
W.S.
N

PLATE 23

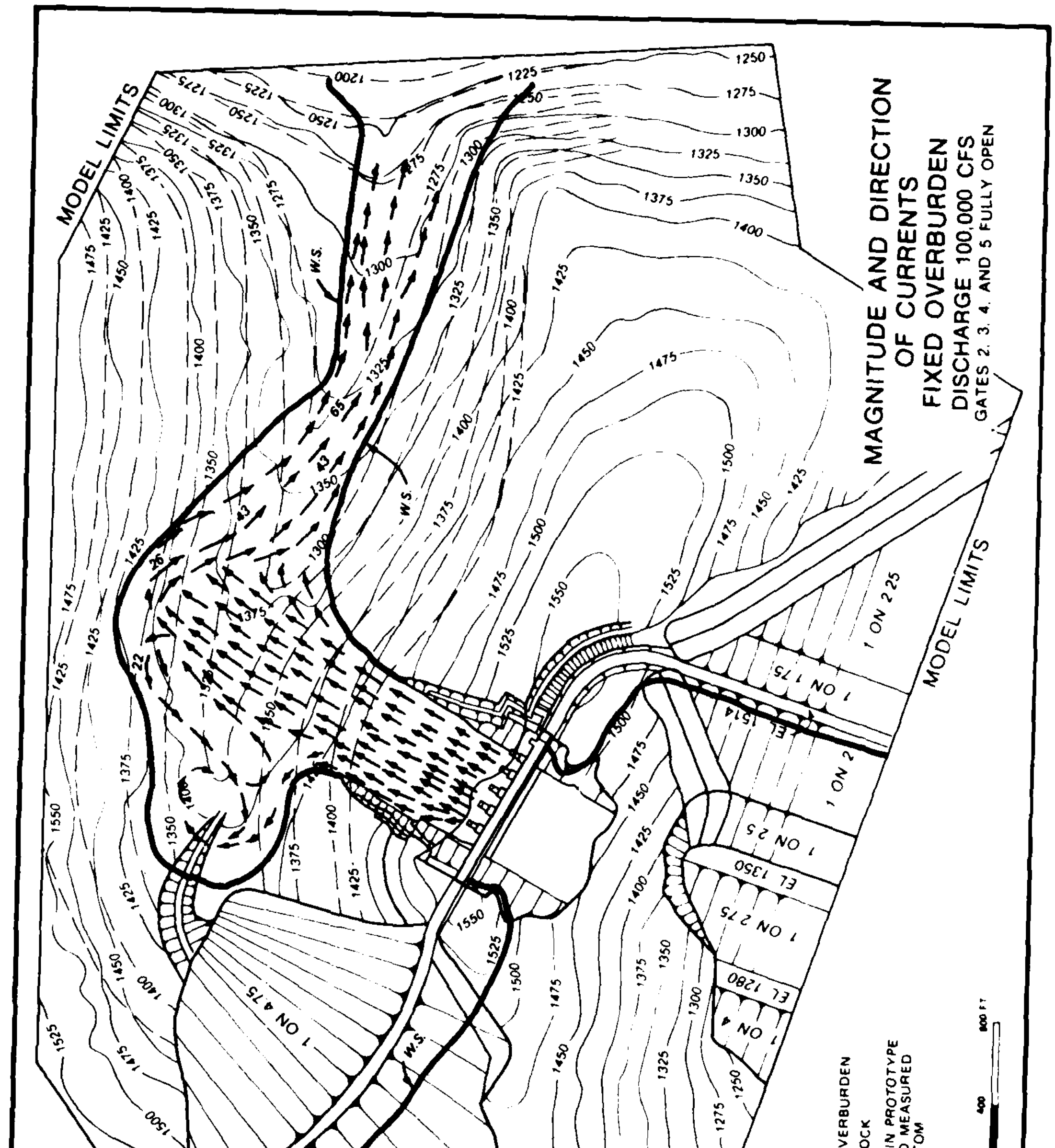

MAGNITUDE AND DIRECTION
OF CURRENTS
FIXED OVERBURDEN
DISCHARGE 100,000 CFS
GATES 2, 3, 4, AND 5 FULLY OPEN
LEGEND
ELEVATION OF OVERBURDEN
ELEVATION OF ROCK
NOTE VELOCITIES ARE IN PROTOTYPE FEET PER SECOND MEASURED 5 FT ABOVE BOTTOM
SCALE
200 0 200 400 800 FT
MODEL LIMITS
N
W.S.
1 ON 4.75
1 ON 5.5
1 ON 2.25
1 ON 1.75
1 ON 2
1 ON 2.5
1 ON 2.75
1 ON 4
EL 1514
EL 1350
EL 1280

PLATE 24

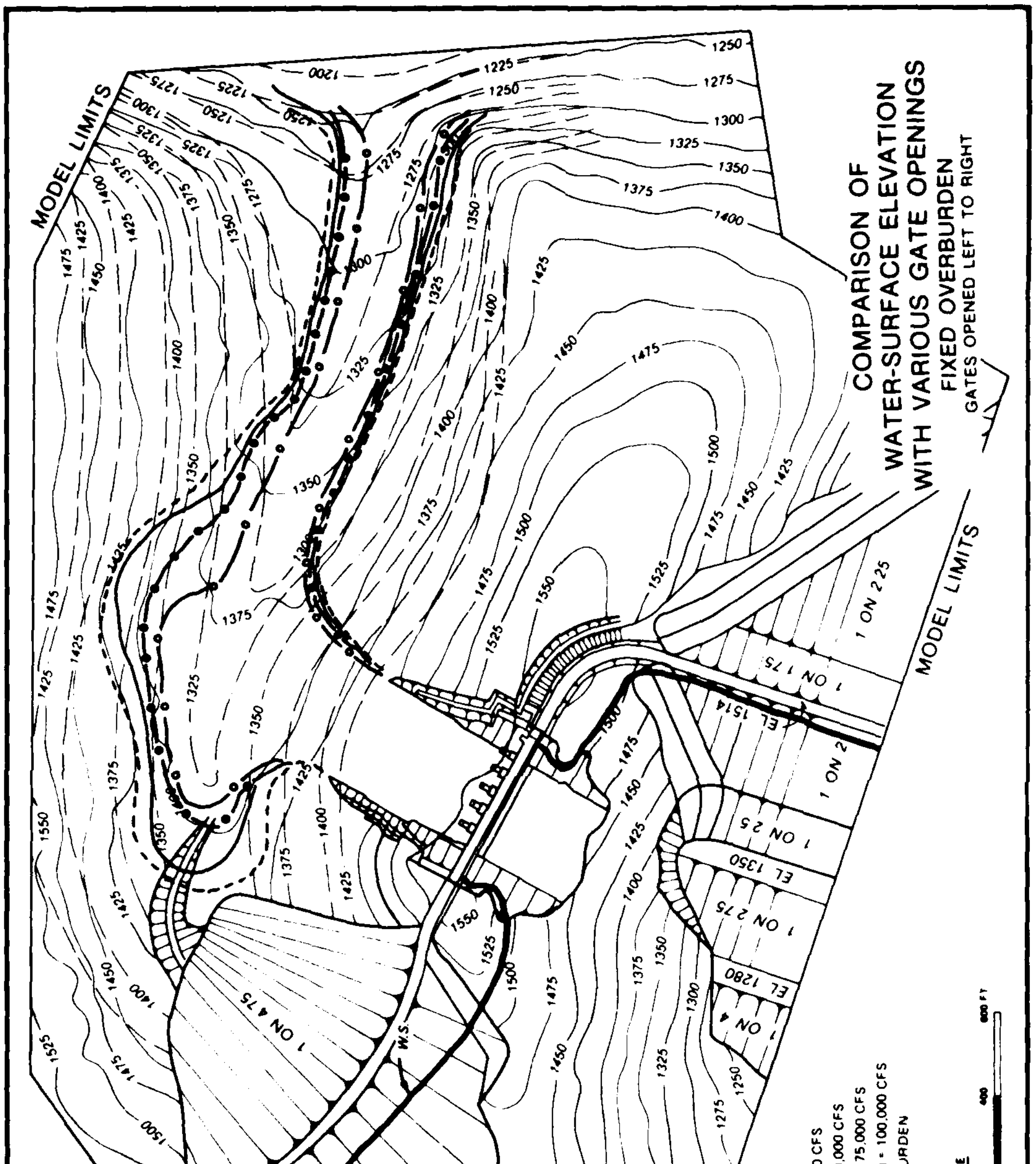

COMPARISON OF
WATER-SURFACE ELEVATION
WITH VARIOUS GATE OPENINGS
FIXED OVERBURDEN
GATES OPENED LEFT TO RIGHT
LEGEND
GATE 5 OPEN, Q = 25,000 CFS
GATES 4, 5 OPEN, Q = 50,000 CFS
GATES 3, 4, 5 OPEN, Q = 75,000 CFS
GATES 2, 3, 4, 5 OPEN, Q = 100,000 CFS
ELEVATION OF OVERBURDEN
ELEVATION OF ROCK
SCALE
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
MODEL LIMITS
1 ON 2.25
1 ON 1.75
EL 1514
1 ON 2
1 ON 2.5
EL 1350
1 ON 2.75
EL 1280
1 ON 4
1 ON 4.75
1 ON 5.5
W.S.
N

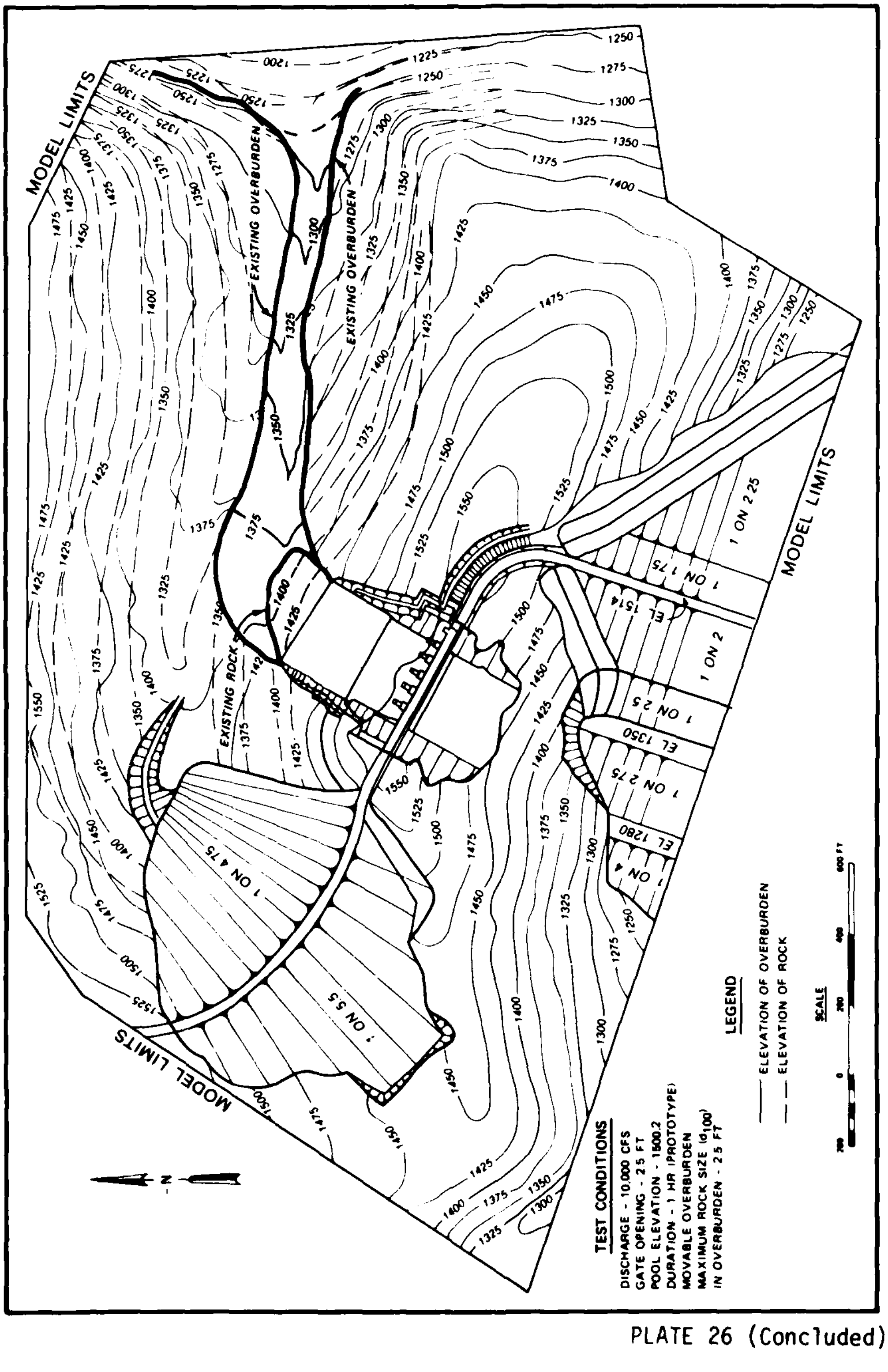
PLATE 25

a. Operating

b. Dry bed

Overburden displacement; discharge 10,000 cfs, G_o = 2.5 ft, H_w = 1500.2; operation time 2 hr

PLATE 26 (Continued)



c. Scour characteristics

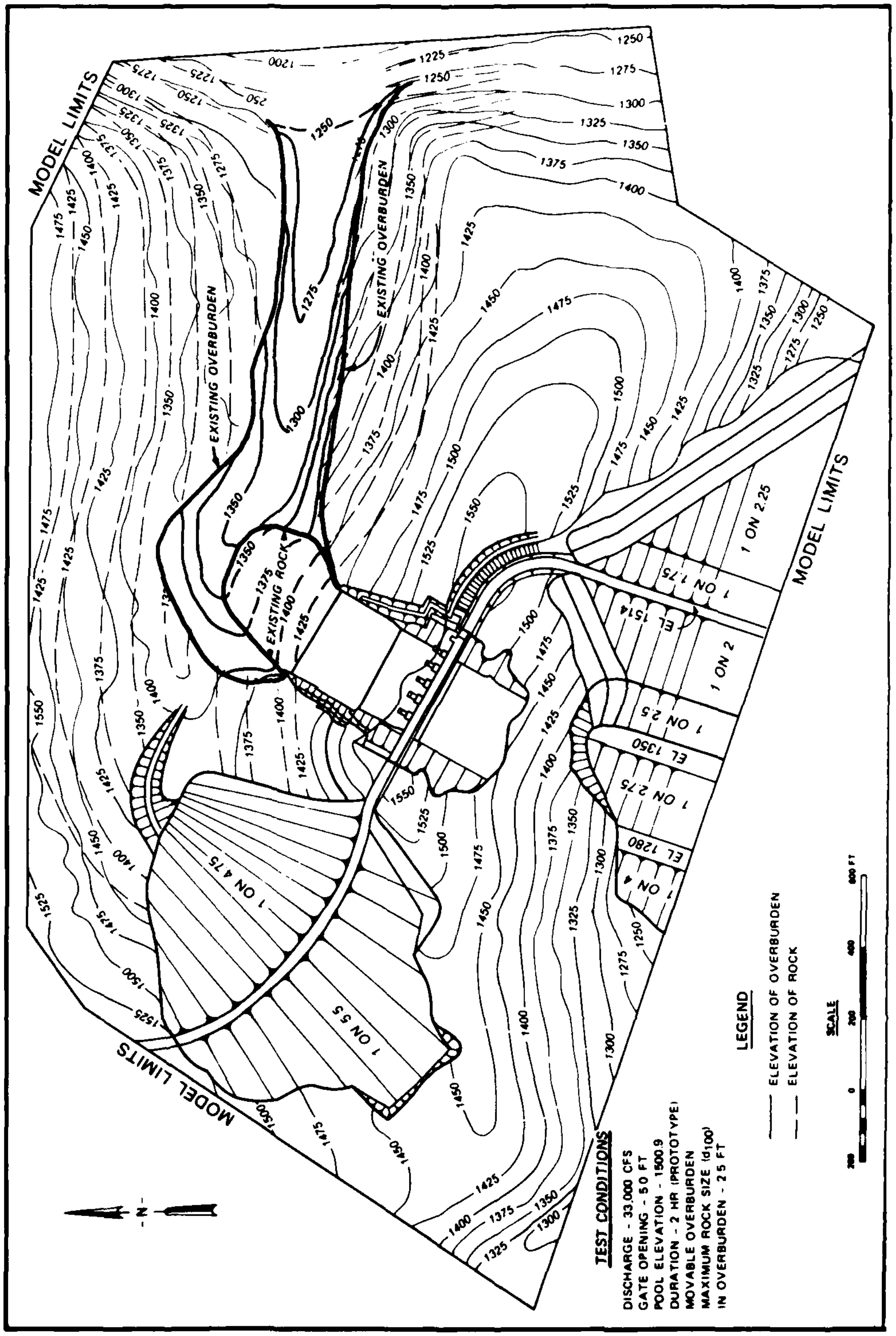
PLATE 26 (Concluded)

a. Operating

b. Dry bed

Overburden displacement; discharge 33,000 cfs, G_O = 2.5 ft, H_W = 1500.9; operating time, 2 hr at 10,000 cfs and 1 hr at 33,000 cfs

PLATE 27 (Continued)

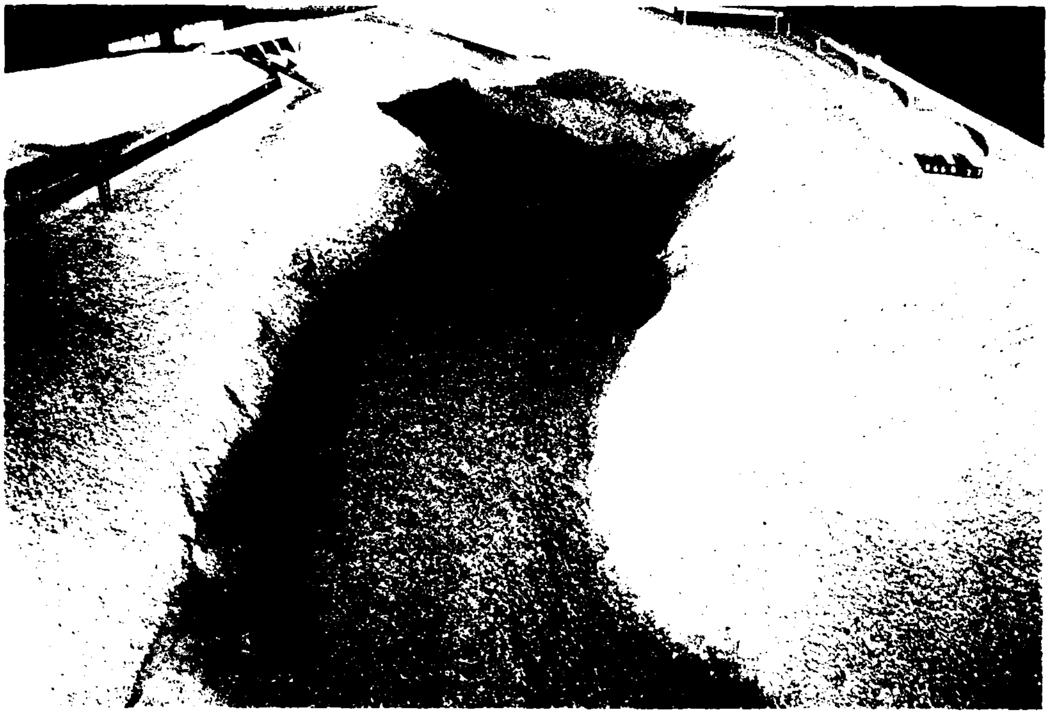


c. Scour characteristics

PLATE 27 (Concluded)

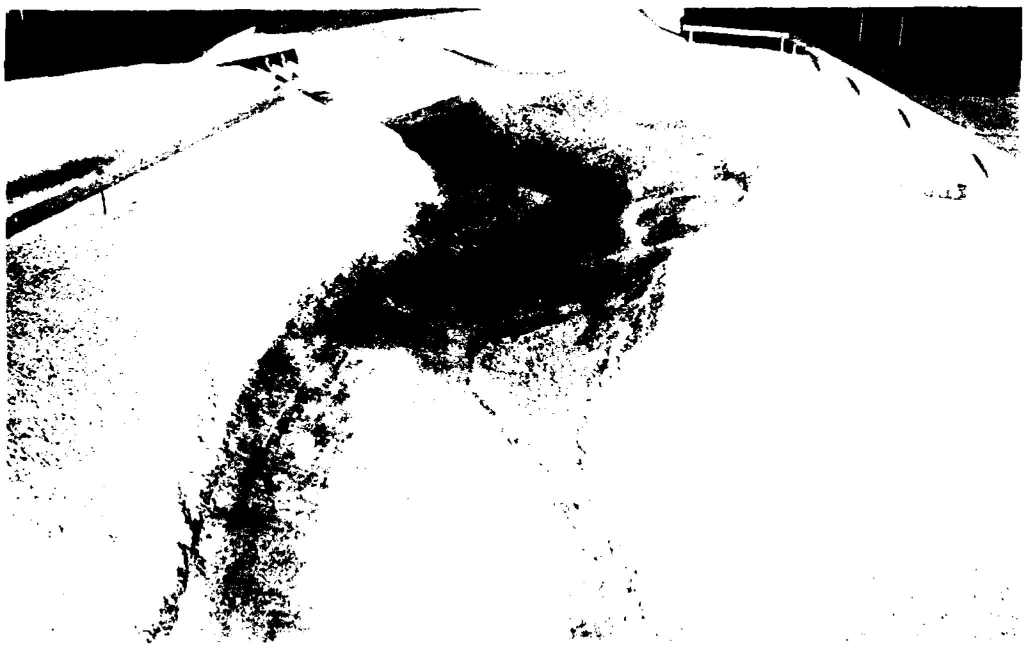
a. Operating

Overburden displacement; discharge 65,000 cfs, G_o - 10.0 ft, H_w - 1502.0; operating time, 2 hr at 10,000 cfs, 1 hr at 33,000 cfs, and 1 hr at 65,000 cfs



b. Dry bed

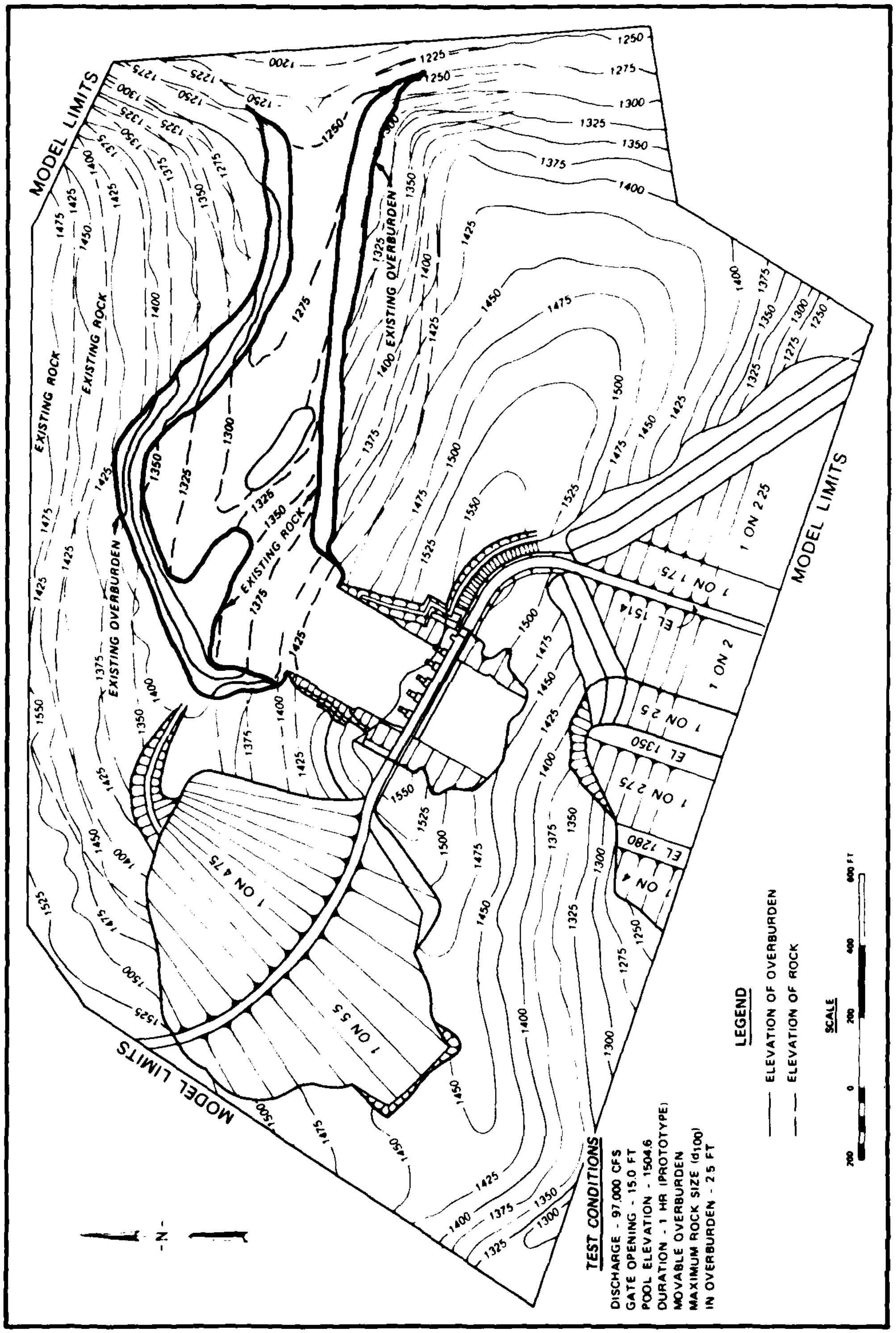
a. Operating



b. Dry bed

Overburden displacement; discharge 97,000 cfs, G_o = 15 ft, H_w = 1504.6; operating time, 2 hr at 10,000 cfs, 1 hr at 33,000 cfs, 1 hr at 65,000 cfs, and 1 hr at 97,000 cfs

PLATE 29 (Continued)



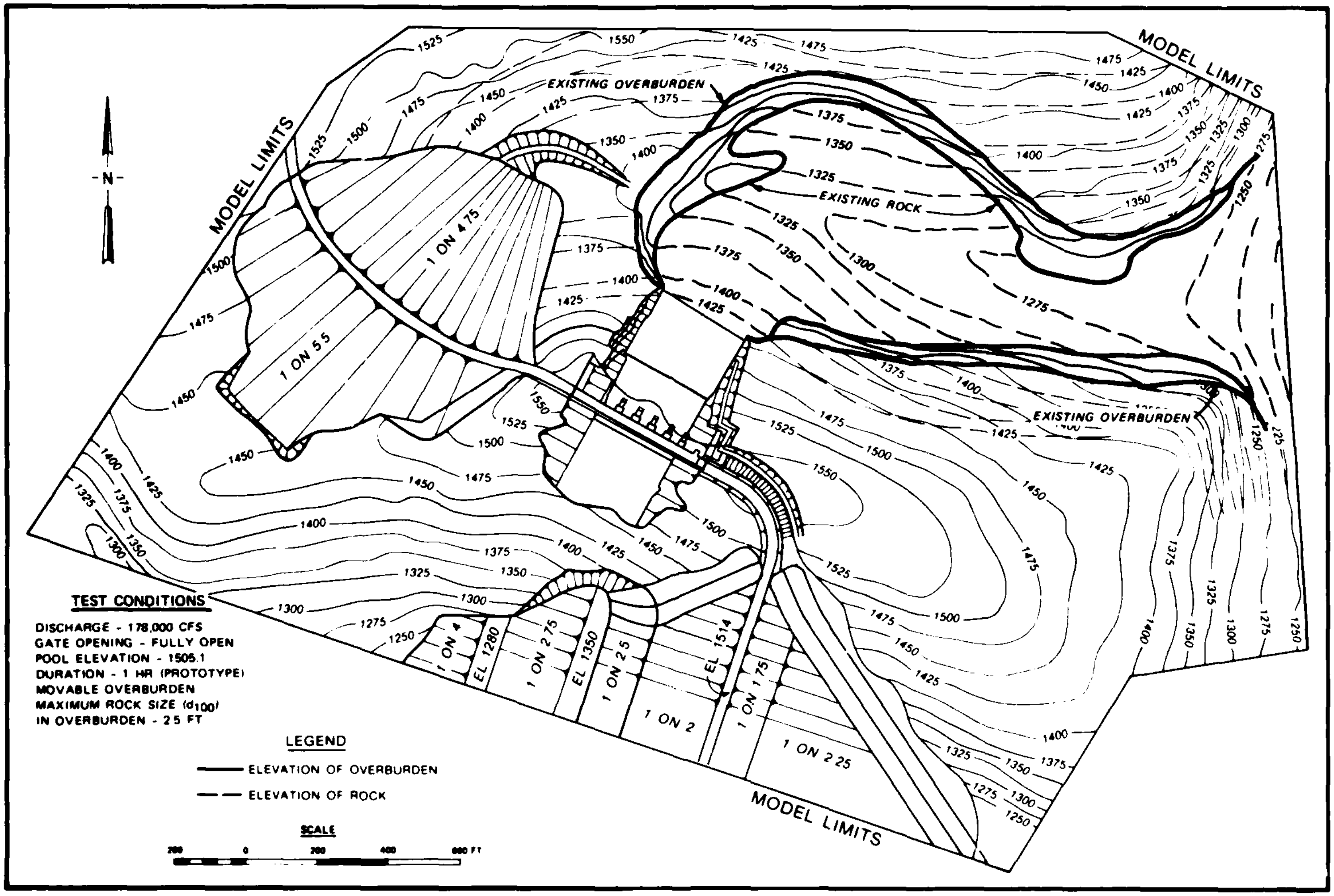
c. Scour characteristics

a. Operating

b. Dry bed

Overburden displacement; discharge 178,000 cfs, G_O = all gates fully open, H_W = 1504.7; operating time, 2 hr at 10,000 cfs, 1 hr at 33,000 cfs, 1 hr at 65,000 cfs, 1 hr at 97,000 cfs, and 1 hr at 178,000 cfs

PLATE 30 (Continued)



c. Scour characteristics

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

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