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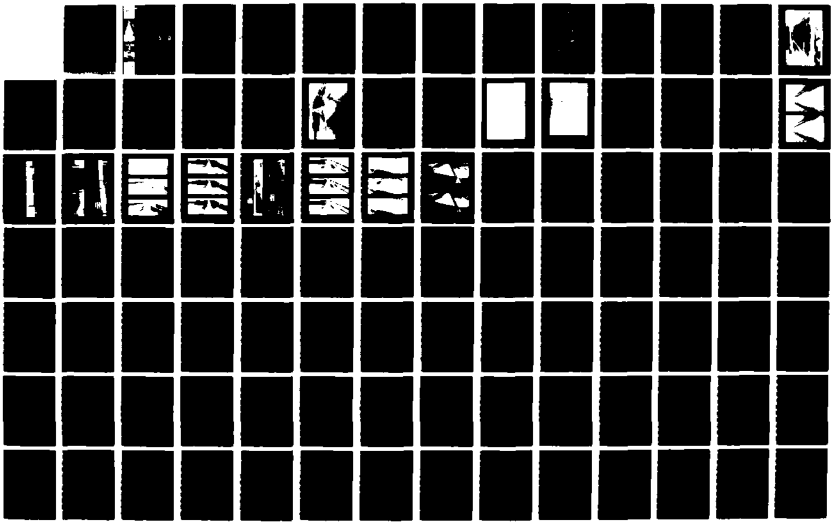

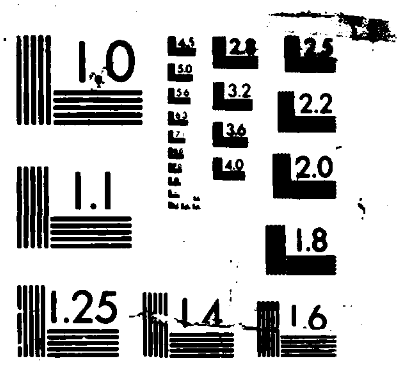

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AD-A181 888

SHELDRAKE RIVER TUNNEL MAMARONECK, NEW YORK

Hydraulic Model Investigation

by

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It is proposed to permanently divert the Sheldrake River, in southeast New York, from upstream of the confluence with the Mamaroneck River through a concrete-lined tunnel into the West Basin of the Mamaroneck Harbor. Included in the proposed plan are improvement of the existing channel, construction of an uncontrolled ogee drop structure and converging approach to a tunnel inlet portal, construction of 3,550 ft of concrete-lined tunnel, including a 16.25-ft-square section and a 17.5-ft horseshoe section with an intermediate transition, and construction of a standard stilling basin energy dissipator at the juncture with the West Basin. A 1:25-scale model was used to determine the flow conditions in the proposed project for Manning's n values of 0.013 and 0.010 up to the Standard Project Flood (SPF), which peaked at 4,039 cfs. Tide effects were investigated for tides of −2.7 ft, 6.7 ft, 10.0 ft, and 16.8 ft, National Geodetic Vertical Datum. Flow conditions were improved in the model by

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19. ABSTRACT (Continued).

modifying the tunnel alignment, the shape of the tunnel, and transitions near the middle of the tunnel. The stilling basin was modified to improve flow conditions for the varying tide effects. Riprap protection downstream of the stilling basin was verified as being adequate for the SPF. Stage-discharge relations were determined as were flow velocities and water-surface profiles in the tunnel approach. Hydraulic grade lines were determined for the tunnel for several flows. Flow velocities at the stilling basin end sill are presented as are velocities in the West Basin of the Mamaroneck Harbor.

PREFACE

The model investigation reported herein was authorized by the Office, Chief of Engineers, US Army, on 8 August 1984 at the request of the US Army Engineer District, New York (NAN). The studies were conducted by personnel of the Hydraulics Laboratory (HL), US Army Engineer Waterways Experiment Station (WES), during the period September 1984 to February 1986. All studies were conducted under the direction of Messrs. F. A. Herrmann, Jr., Chief, HL, and J. L. Grace, Jr., Chief of the Hydraulic Structures Division. Tests were conducted by Messrs. C. H. Tate, Jr., J. Cessna, L. East, and N. Ford of the Locks and Conduits Branch under the supervision of Messrs. G. A. Pickering, former Chief of the Locks and Conduits Branch, and J. F. George, Acting Chief of the Locks and Conduits Branch. This report was prepared by Mr. Tate and edited by Mrs. Marsha Gay, Information Technology Laboratory.

During the course of the investigation, Messrs. J. Rosen, J. Urbelis, and R. Schembri, and Ms. L. Koeth of NAN visited WES to discuss model results and correlate these results with concurrent design work.

COL Allen F. Grum, USA, was the previous Director of WES. COL Dwayne G. Lee, CE, is the present Commander and Director. Dr. Robert W. Whalin is Technical Director.

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 per second	0.02831685	cubic metres per second
feet	0.3048	metres
feet per second	0.3048	metres per second
inches	25.4	millimetres
miles (US statute)	1.609344	kilometres
pounds (mass)	0.4535924	kilograms
square miles	2.589988	square kilometres

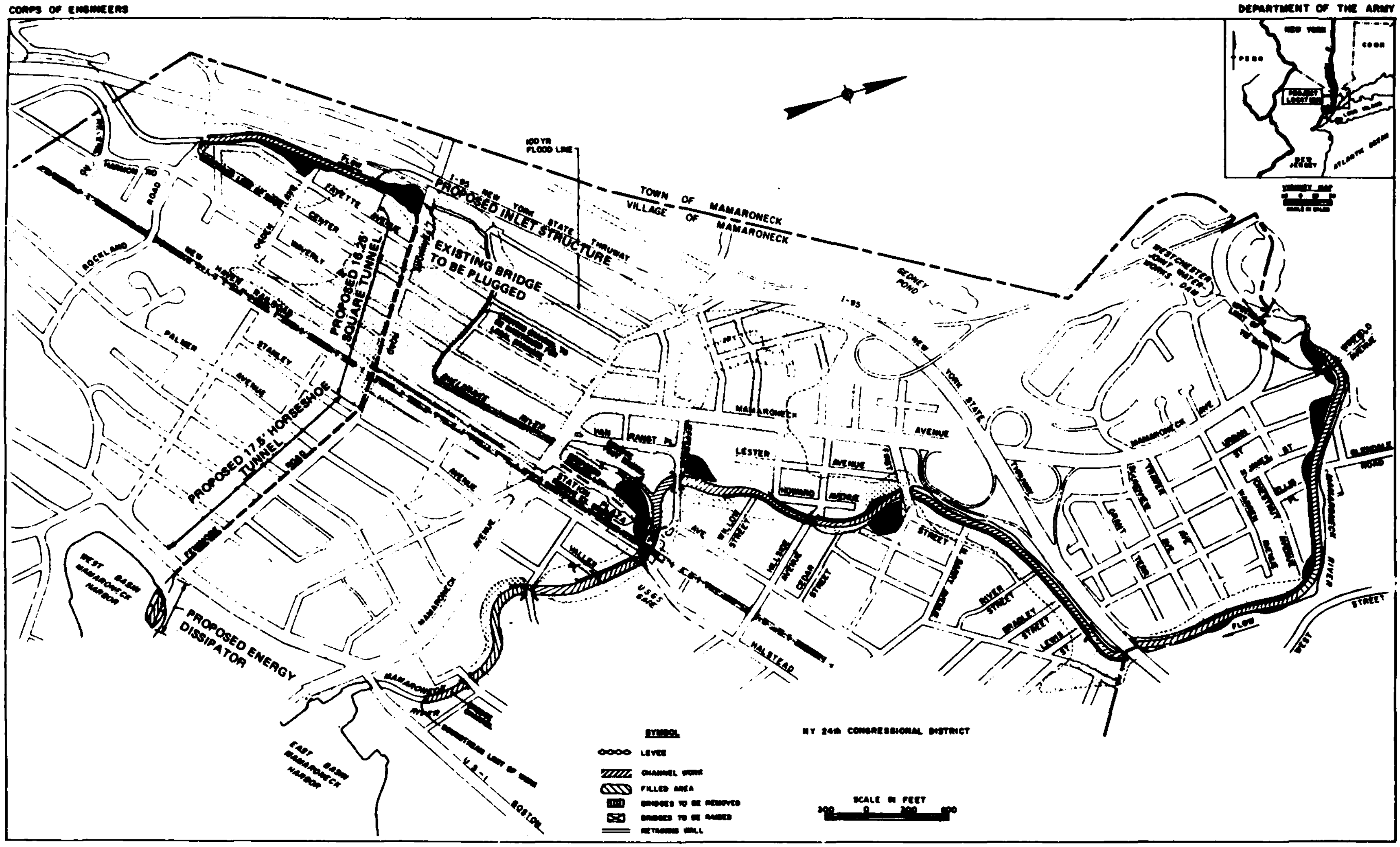

Figure 1. Project location (courtesy of US Army Engineer District, New York)

SHELDRAKE RIVER TUNNEL, MAMARONECK, NEW YORK

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. The Sheldrake River Tunnel is located in the Village of Mamaroneck, N. Y., on the north coast of Long Island Sound, northeast of New York City (Figure 1). The Sheldrake River joins with the Mamaroneck River approximately 3,000 ft* upstream of the mouth of the Mamaroneck River at the East Basin of the Mamaroneck Harbor. Between the confluence and the harbor, the Mamaroneck River channel is confined by steep rock banks, the tops of which have been heavily developed. This existing channel will not adequately convey the flood flows from both the Sheldrake and the Mamaroneck basins, which subjects large areas to flooding upstream of the confluence.

2. The proposed Sheldrake River Tunnel will intercept the Sheldrake River immediately upstream of Fenimore Road and permanently divert the river through a tunnel under Fenimore Road to the West Basin of the Mamaroneck Harbor. Channel improvements along the Mamaroneck River and the diversion of flow from approximately 5.57 square miles of the Sheldrake River basin will render the Mamaroneck River capable of passing the flood flows from the remaining drainage basin.

3. The proposed Sheldrake River Tunnel, with a peak discharge of 4,039 cfs, is designed to accommodate the Standard Project Flood (SPF). Improvements to the existing channel will result in a trapezoidal approach channel with a 40-ft-wide base upstream of a 60-ft-wide ogee drop structure that is located 230 ft upstream of the tunnel entrance. The sides of the channel between the ogee drop structure and the 16.25-ft-wide tunnel entrance converge on a 1V on 8H slope. Downstream from the tunnel entrance, the tunnel will turn and follow Fenimore Road to the south. The tunnel will change shape to accommodate the subsurface conditions with the upper cut-and-cover section

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

being 16.25 ft square and the drill-and-blast sections being a 17.5-ft horseshoe shape (Plate 1). Flow will enter the West Basin of the Mamaroneck Harbor through a standard stilling basin designed for 4,039 cfs and a tide elevation of -2.7 ft NGVD.*

Purpose of Model Investigation

4. Due to the unusually complex design and the many changes in flow control, a hydraulic model study was considered necessary to verify the adequacy of and to develop desirable modifications to the project design. Specifically, the model study was to

- a. Ensure that the design water levels are not exceeded.
- b. Determine if undesirable eddies or wave patterns develop at the tunnel inlet.
- c. Ensure the hydraulic capacity of the tunnel.
- d. Observe and define flow conditions within the tunnel to ensure that undesirable flow conditions do not develop.
- e. Develop an energy dissipator at the tunnel outlet that will function over a long range of discharges and tailwater (tide) elevations.
- f. Determine the size, extent, and thickness of riprap required to prevent scour downstream from the energy dissipator.
- g. Determine the current patterns and velocities in the West Basin to identify possible problems to existing structures and the Federal navigation project.

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

PART II: THE MODEL

5. The 1:25-scale model (Figure 2) reproduced approximately 400 ft of the channel approaching the drop structure upstream of the tunnel, the drop structure and converging approach to the tunnel, 3,550 ft of tunnel in various cross sections, the stilling basin at the downstream end of the tunnel, and the West Basin of the Mamaroneck Harbor to a point seaward of the permanent pier located on the south side of the harbor (Plate 1). Concrete was used to form the approach channel, and artificial roughness was added to the channel to develop the approach depth at the entrance to the model. Galvanized sheet metal and polyethylene-coated plywood were used to construct the drop structure and converging tunnel inlet. The tunnel was constructed of hand-molded acrylic plastic. Polyethylene-coated plywood and sheet aluminum were used to construct the stilling basin with minor portions being made with plastic fillers or wood. The harbor seawall was built with brick and the outlying topography formed with concrete. Conditions within the seawall were simulated with molded sand and scaled rock to simulate the riprap protection downstream from the stilling basin. Piling for the floating docks was installed using steel rods, and the permanent pier was built of wood dowels and plywood.

6. The coefficient of roughness of the conduit model surface had previously been determined to be approximately 0.009 (Manning's n). Basing similitude on the Froudian relation, this n value would be equivalent to a prototype n of 0.0154. The n value used in the design and analysis of the prototype channel varied from 0.010 to 0.013; therefore, supplementary slopes were added to the model to correct for this difference in the n values of the model and prototype.

Model Appurtenances

7. Flow to this model was supplied through a circulating system. Discharges were measured and controlled through a feedback system using a mechanized rotating disk valve and a venturi meter equipped with a differential pressure cell. Control voltages were compared with the output from the differential pressure cell, and the valve position was adjusted by the control system as required to satisfy the control input. This system allowed the use of a varying control voltage to reproduce a hydrograph. Constant flows could

Figure 2. Dry bed view of model looking upstream

also be set, and the control system compensated for changes in supply pressure to maintain a constant flow to the model.

8. Tide elevations were controlled using a manually operated tailgate. Tide elevations were set in the harbor without flow through the tunnel, and the water surface was allowed to fluctuate with the flow in the tunnel.

9. Velocities were measured in the model with pitot-static tubes and with propeller meters with a minimum measurable velocity of approximately 0.4 fps prototype. Point gages and piezometers were used to measure water-surface elevations throughout the model. Flow conditions were observed for all designs tested, with the original designs and the potentially usable designs and associated flow conditions being recorded photographically.

Scale Relations

10. 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 transference of model data to prototype equivalents are as follows:

Characteristic	Dimension*	Model:Prototype
Length	L_r	1:25
Area	$A_r = L_r^2$	1:625
Velocity	$V_r = L_r^{1/2}$	1:5
Discharge	$Q_r = L_r^{5/2}$	1:3,125
Volume	$V_r = L_r^3$	1:15,625
Weight	$W_r = L_r^3$	1:15,625
Time	$T_r = L_r^{1/2}$	1:5
Roughness coefficient	$N_r = L_r^{1/6}$	1:1.710

* Dimensions are in terms of length.

Model measurements of discharge, water-surface elevations, and velocities can be transferred quantitatively to prototype equivalents by means of the scale relations. Experimental data also indicate that the model-to-prototype scale ratio is valid for scaling stone in the sizes used in this investigation. Evidences of sand scour are considered only qualitatively reliable, since it is not yet possible to reproduce quantitatively in the model the resistance to erosion of fine-grained prototype bed material.

PART III: TESTS AND RESULTS

11. Tests were conducted to determine the general flow conditions into and through the tunnel and in the harbor area downstream of the tunnel. Depending on the quality of the tunnel construction and finish work, the Manning's n value was expected to range between 0.013 and 0.010. Tests were first conducted with the slope adjusted to reproduce the energy gradient for the n value of 0.013.

Manning's n = 0.013

Approach channel and drop structure

12. The original design for the tunnel approach and drop structure (Plate 2) was tested, and discharge rating curves were determined for sta 40+80, 39+15, and 38+00 at tide el -2.7 and 6.7 (Plate 3). Velocity cross sections were collected at sta 40+80 (Plate 4) and 39+15 (Plate 5) for discharges of 512 cfs (1 year), 1,208 cfs (10 year), 2,551 cfs (100 year), and 4,039 cfs (SPF) for tide elevations -2.7 and 6.7. Flows approaching the drop structure were satisfactory through the converging wing walls and over the ogee crest. Surface currents at discharges of 2,551 cfs and 4,039 cfs are shown in Photos 1 and 2 for tide el of 6.7 and -2.7, respectively. Backwater effects caused by the tunnel submerged the ogee crest at approximately 3,500 cfs. Conditions at flows higher than this tended to be rougher due to the presence of standing waves near the crest. A hydraulic jump formed at the base of the ogee crest for lower flows. Flow downstream of the ogee crest was generally subcritical due to the tunnel backwater. Water-surface profiles between sta 42+00 and 37+00 for tide el -2.7 and 6.7 are shown in Plate 6. Tide elevations are important for some flow conditions because the entrance portal invert is at el 2.2, and thus subjected to changing backwater effects.

13. The New York District (NAN) provided a modified crest design designated as type 2 (Plate 7) for testing in an attempt to reduce the water level upstream of the drop structure. This design eliminated the raised crest and used a smooth curve to drop from the channel invert. Water levels upstream of the drop structure were lowered for low flows, but due to the effects of the tunnel entrance, the water levels were not significantly different at the design flow of 4,039 cfs (Plate 8). Additionally, flow conditions were

rougher with the type 2 design crest (Plate 9). This design was not considered acceptable and was not used for the remainder of the study. Instead, the original design ogee crest was returned to the model for the rest of the study.

Tunnel

14. Flow conditions in the tunnel as originally designed with discharges greater than 3,700 cfs were unsatisfactory. Flows between the two shape transitions (Plate 1) tended to fill the tunnel due to the flow resistance and backwater effects of the transitions and the curve located between the transitions. The downstream transition, located between sta 18+35 and 18+10, was designed to converge the 17.5-ft-wide flat invert to a point invert over 25 ft. This sharp convergence and the decreased area in the lower half of the tunnel produced a backwater effect that reached through the curve immediately upstream of the transition and the intermediate shape tunnel to the upstream transition located between sta 20+75 and 20+50. With the losses associated with the shape change at the upstream transition added to the backwater effect of the curve and downstream transition, the flow tended to fill the conduit at less than the design discharge of 4,039 cfs. Flow exiting the downstream transition accelerated forming an undular condition that continued through the remainder of the tunnel (Photo 3). Hydraulic grade lines through the tunnel are shown in Plates 10-13.

15. An additional flow feature was observed immediately downstream of the tunnel entrance portal at approximately sta 36+40. Flows above 3,600 cfs produced a standing wave at this location after the rapid drawdown through the entrance portal (Photo 4). For flows above approximately 3,600 cfs, the standing wave impacted the top of the tunnel.

16. The original design was not satisfactory primarily due to the flow conditions in the vicinity of the two transitions. The conduit design was revised by NAN based on the previous observations. The intermediate shape was deleted and a single transition was installed between sta 20+75 and 19+67 which joined the 16.25-ft-square section to the 17.5-ft horseshoe. The curve originally located between sta 18+83 and 18+35 was redesigned with a longer radius (268.85 ft) and constructed between sta 19+17 and 18+00 with the horseshoe shape (Plate 14). The new curve in the type 2 design conduit shortened the tunnel by 1.04 ft. In order to keep common references for identifying points, the type 2 design conduit has a 1.04-ft break in

stationing at the downstream end of the redesigned curve.

17. Flow conditions in the type 2 design conduit for the Manning's n of 0.013 were significantly improved over the original design. Discharge rating curves for the original drop structure and the type 2 conduit are shown in Plates 15 and 16. As can be seen in Plate 16, the tide elevation of 16.8 ft limited the structure to less than the design flow. At the design flow of 4,039 cfs, the short straight section downstream of the transition and the new curve would occasionally flow full; however, this condition was temporary and the conduit downstream of the transition would return to open channel flow (Photo 5). During the periods that the conduit was flowing full, the hydraulic grade line was within 1 ft of the roof of the tunnel for the design discharge and tide elevations of 6.7 and below. Conditions near the entrance portal remained the same as with the original tunnel design. Hydraulic grade lines in the tunnel approach and through the tunnel are shown in Plates 17-23 for flows of 512 cfs, 1,208 cfs, 2,551 cfs, and 4,039 cfs and tide elevations -2.7 ft, 6.7 ft, 10.0 ft, and 16.8 ft.

Stilling basin

18. The design of the original stilling basin (Plate 24) was based on using a length of approximately $6D_2$ (D_2 equals the tailwater depth) for the basin length. Observation of the action in the basin indicated that the flow was not spreading with the apron flare and that eddies were forming in the basin that extended beyond the end sill. These conditions resulted in unstable jump action even at the design condition of the SPF of 4,039 cfs and a -2.7-ft tide elevation. Some of the flow instability was due to the flow not spreading on the apron flare. This appeared to be caused by the absence of a curved transition from the parallel tunnel walls to the 1:6 flare. The remaining instability appeared to be due to excess width and length in the basin design. Velocity cross sections at the end sill are shown in Plates 25 and 26. Stilling basin action and surface currents are shown in Photos 6-8.

19. An extended flare was installed to limit the basin width at the location of the jump (type 2 stilling basin design, Plate 27). This type of design is usually avoided in confined exit channels because the expanding exit flow impacts the banks and can cause significant scour problems. The harbor, however, allows the exit flow to spread without impacting the banks. Some improvement was realized with this design, but eddies still formed downstream of the jump with the potential to bring material into the basin. Due to the

width of the stilling basin, the jump was unstable.

20. Parallel walls were added to the type 2 stilling basin at the intersection of the downstream end of the trajectory and the flaring sidewalls to form the type 3 stilling basin design (Plate 27). This was done to try to stabilize the jump action and to confine the flow to inhibit the formation of eddies in the basin. Strong eddies formed at intermediate flows and caused highly unstable jump conditions. In an attempt to stabilize the hydraulic jump for various flow conditions, a single row of baffle blocks, 9 ft high, was installed in the type 3 stilling basin design to form the type 4 stilling basin design (Plate 27). Because minimal improvement was observed, a second row of baffle blocks was installed (type 5 stilling basin design, Plate 27). Flow conditions were not significantly improved with the type 5 stilling basin and the addition of baffle blocks caused severe drawdown over the end sill.

21. The type 6 stilling basin design modified the original stilling basin from a width of 55 ft to 45 ft and from a length of 72 ft to 47 ft with a 3-ft-high sloping end sill. Baffle blocks were not used since the tailwater was sufficient to cause a hydraulic jump to occur in the basin and NAN indicated that they could cause a maintenance problem. The riprap immediately downstream from the stilling basin was offset 2 ft below the top of the end sill and sloped 1V on 10H upward to el -10. The riprap and channel downstream from that point remained as originally designed. The type 6 design stilling basin is shown in Figure 3 and Plate 28.

22. Performance of the type 6 stilling basin was superior to the performance of the other designs tested. Some eddy action still occurred with this design; however, the eddies were less severe and were confined to the stilling basin. Flow conditions for this design and the tunnel n simulated at 0.013 are shown in Photos 9-11, and velocities at the end sill are shown in Plates 29-30.

<u>Harbor</u>

23. Harbor circulation and velocities were checked with the tunnel Manning's n simulated at 0.013, but the major effort in studying the flow conditions was reserved for the simulation of the 0.010 value for n. The greater energy level entering the stilling basin and harbor for the 0.010 n value was expected to have a greater impact than the simulation of the rough n value.

Figure 3. Dry bed of type 6 design stilling basin

Manning's $n = 0.010$

Approach channel and drop structure

24. The original drop structure was used in these tests based on the results of the tests simulating $n = 0.013$. For the smoother tunnel, the flow conditions and the discharge rating curves did not change upstream of the drop structure for flows up to approximately the design flow of 4,039 cfs. For flows higher than the design condition, the water surface was slightly lower due to reduced backwater effects from the tunnel. Water-surface elevations between the entrance portal and the drop structure were lower throughout the flow range. Stage-discharge relations are shown in Plate 31 for tide el -2.7 and 6.7. Tide elevations above 6.7 could not be simulated in the approach area for a Manning's n of 0.010 due to the excess slope distortion required.

Tunnel

25. Flow conditions were satisfactory throughout the type 2 design tunnel (Photo 12). Open channel flow was maintained up to the design discharge and tide el 8.7. The undular condition at the tunnel entrance that was observed for the rough n value did not exist for the smoother condition (Photo 13). Hydraulic grade lines for flows of 512 cfs, 1,208 cfs, 2,551 cfs, and 4,039 cfs are shown in Plates 32-35 for tide el -2.7 and 6.7 and for tide el 10.0 for the design discharge.

Stilling basin

26. The type 6 stilling basin performed as expected for the smooth n value. Flow conditions were acceptable and very similar to the conditions observed for the rough n value. Surface currents are shown in Photos 14-16, and velocities over the end sill are shown in Plates 36-38. Water-surface elevations through the stilling basin are shown in Plate 39 for the -2.7-ft and 6.7-ft tide and the design flow.

Harbor

27. For various flow conditions, several circulation patterns or eddies were set up in the harbor area. These eddies were generally fairly large with low velocities. Surface circulation patterns are shown in Photos 17-21 for several flow conditions. Surface, middepth, and bottom velocities in the harbor are shown in Plates 40-75. Flow exiting the stilling basin tended to bend slightly to the east and cross the harbor to the south shore and then

flow generally east past the permanent pier toward Long Island Sound. The outflow velocities at the permanent pier did not exceed 3.5 fps for the flow conditions observed as shown in Plates 76-79. These velocity cross sections are shown as Section A-A on Plate 75. The major circulation cell existed in the southwest portion of the harbor where clockwise flow was established by the outflow from the stilling basin. Harbor water-surface elevations for 4,039 cfs and tide elevations of -2.7 ft, 6.7 ft, and 10.0 ft were level at -1.55 ft, 7.75 ft, and 11.30 ft, respectively.

28. The design hydrograph (Plate 80) was used to test the stability of the riprap protection downstream of the stilling basin and to investigate the scour conditions downstream of the riprap. Initial bottom conditions are shown in Figure 4 prior to the application of the design hydrograph. Harbor bottom conditions are shown in Figure 5 after the hydrograph. This figure shows that although minor changes can be seen in the channel bottom, the riprap remained in place. These tests were conducted with the tide elevation set at -2.7 ft.

Figure 4. Dry bed view of area downstream from stilling basin prior to test

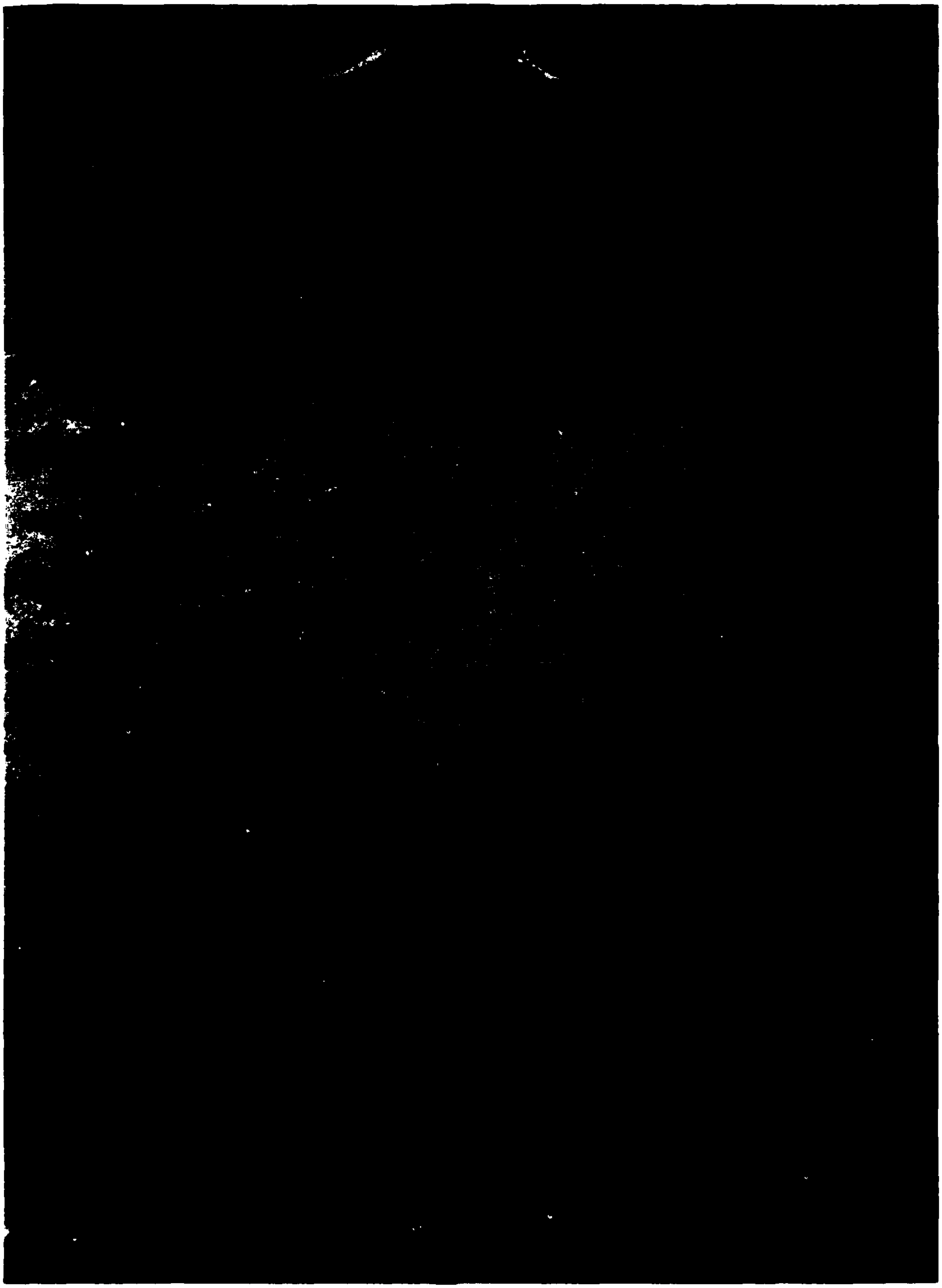


Figure 5. Dry bed view of area downstream from stilling basin after test with design hydrograph

PART IV: CONCLUSIONS AND RECOMMENDATIONS

29. Tests to determine the adequacy of the Sheldrake River Tunnel to convey the design flow conditions indicated that the project with certain modifications would perform satisfactorily. After these modifications to the conduit transitions, the maximum capacity of the system was 4,750 cfs with a tide elevation of 6.7 and the tunnel Manning's n at 0.013. At this discharge, flow overtopped the walls at the tunnel entrance portal. With the smoother n of 0.010, the capacity was 5,200 cfs. Table 1 lists stage-discharge relations.

30. Flows approaching the drop structure were relatively smooth and were below the level specified by NAN (el 25 at sta 40+80) for the design flow. Flow over the crest of the drop structure was controlled by the crest until the backwater effects of the tunnel submerged the crest. At this point the upstream water-surface elevations were controlled primarily by downstream conditions.

31. Minor surface waves were observed downstream of all the curved sections in the tunnel. These waves did not have any significant adverse effects on the flow conditions in the tunnel.

32. For the rough n value tested, flow tended to fill the transition between the square and the horseshoe-shaped conduit. This flow pattern indicates that the tunnel is barely capable of passing the design flow without causing the tunnel to flow full. The pressures exerted on the conduit when the flow caused the tunnel to flow full were very low and were definitely less than the pressures exerted on the conduit when the tide backed the flow into the tunnel.

33. The type 6 stilling basin design provides an acceptable flow transition from the tunnel to the harbor in spite of the fact that this type of energy dissipator is designed to have an increasing tailwater with an increasing flow. Because this stilling basin is located in a tidal basin, the downstream water surface is largely independent of the flow through the stilling basin. NAN studies indicate that tidal stages are usually, but not always, above the astronomical tide levels during flood flow periods. The stilling basin design must therefore be based on the design flow and a minimum tide. All other flow combinations necessarily result in degraded stilling basin performance. Riprap protection downstream of the stilling basin was

stable for the conditions tested. Based on NAN's determination that this riprap protection plan contains the minimum diameter rock that would be installed, smaller diameter rock was not tested.

34. Flow conditions in the harbor generally consisted of low-velocity flow. Areas near the stilling basin and directly in line with the outflow from the stilling basin were subjected to the high velocities at higher discharges. These areas include the floating dock facilities in the northwest portion of the harbor. Presently many small craft moor with anchors in the southwest portion of the harbor. With the construction of the project these boats would be anchored sideways to the eddy flow. This condition should be evaluated.

35. Scour problems seem to be minimal based on sand movement in the model. The sediments in the harbor may be considerably finer than that represented by the sand in the model. Consequently, further studies should be considered to address the scour question.

36. The testing program resulted in the following changes.

- a. Elimination of the intermediate tunnel section (circular roof with vertical sides (Plate 1)) and the substitution of an elongated transition directly from the box shape to the horseshoe shape (Plate 14).

- b. Revision of the original stilling basin (Plate 24) to that shown in Plate 28.

Table 1

Water-Surface Elevations for Various

Tide Elevations and Flows

Tide Elevation	Station	Flow: 1 year	10 years	100 years	SPF
		n = 0.013			
-2.7	40+80	16.9	18.2	19.9	22.3
	39+15	16.9	18.1	19.5	21.8
	38+00	7.1	11.1	16.6	21.6
6.7	40+80	16.9	18.2	19.9	22.3
	39+15	16.9	18.1	19.5	21.8
	38+00	7.8	11.1	16.6	21.6
10.0	40+80	16.9	18.2	19.9	22.4
	39+15	16.9	18.1	19.5	22.0
	38+00	11.2	13.1	17.0	21.8
16.8	40+80	17.5	19.3	25.4	x
	39+15	17.5	19.3	25.2	x
	38+00	17.5	19.3	25.2	x
		n = 0.010			
-2.7	40+80	16.9	18.2	19.9	22.3
	39+15	16.9	18.1	19.5	21.7
	38+00	6.8	10.7	16.3	21.4
6.7	40+80	16.9	18.2	19.9	22.3
	39+15	16.9	18.1	19.5	21.7
	38+00	7.4	10.7	16.3	21.4

Photo 1. Discharge 2,561 cfs, tide el 6.7

Photo 2. Discharge 4,039 cfs, tide el -2.7

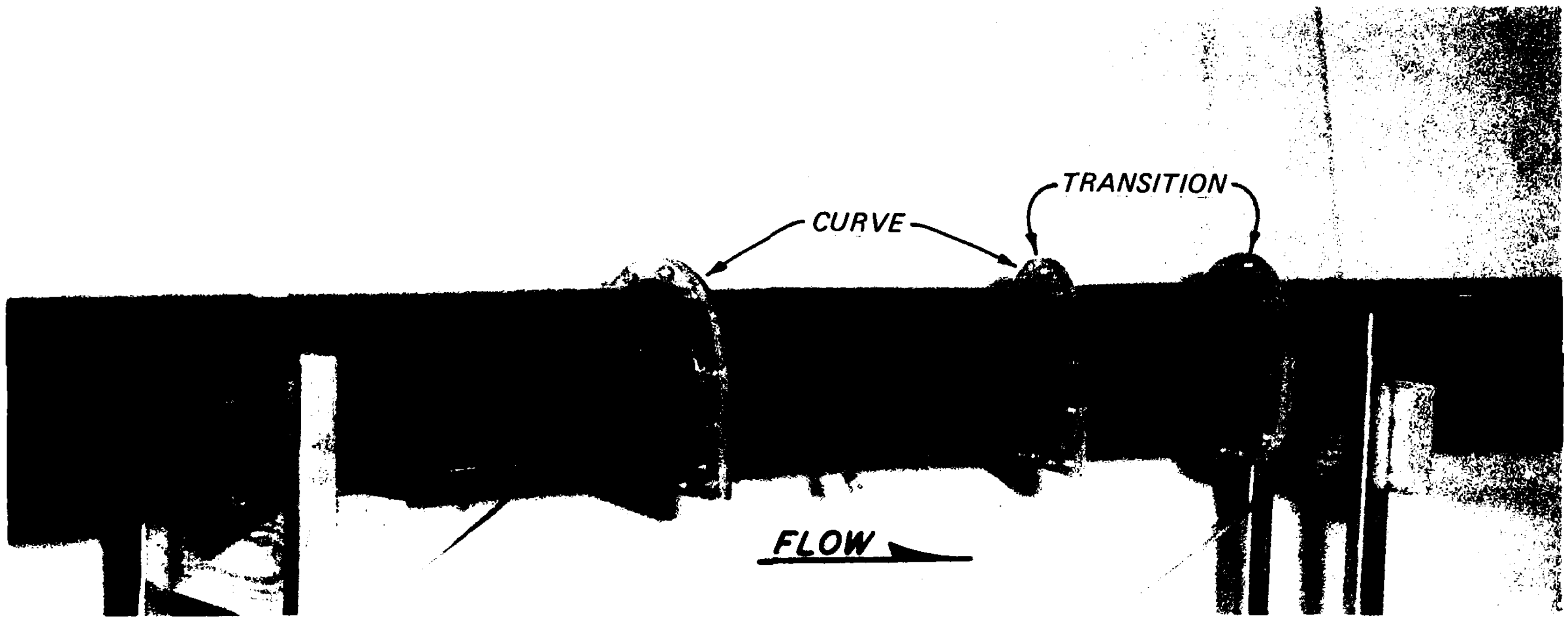


Photo 3. Flow conditions in type 1 design tunnel transition.
Discharge 4,039 cfs, tide el 6.7, n 0.013

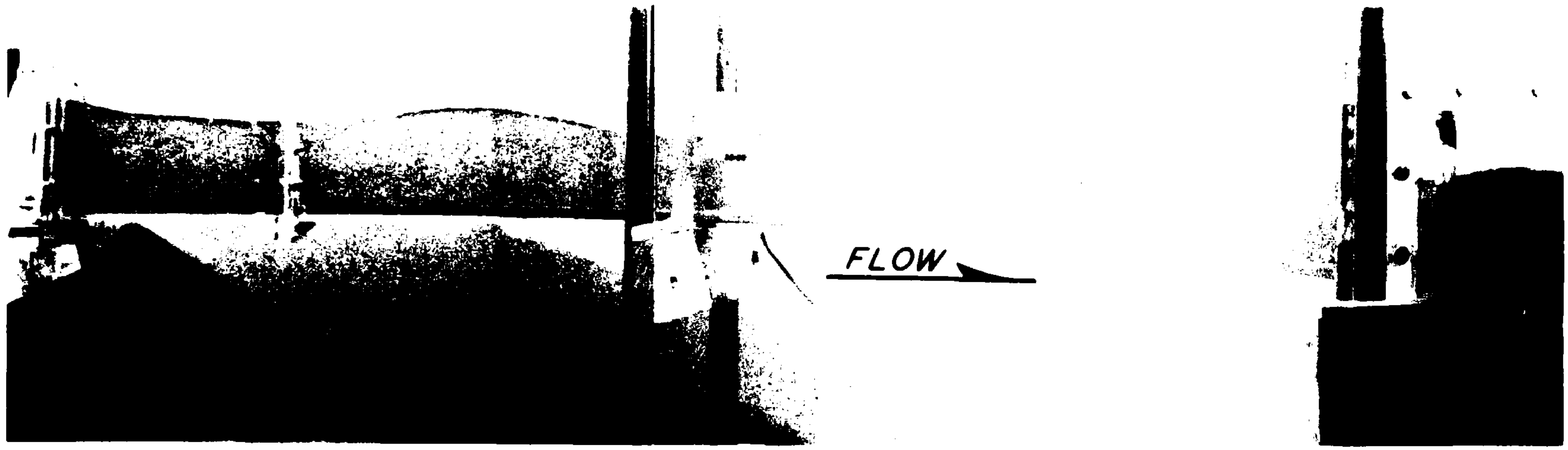


Photo 4. Flow conditions at tunnel entrance, type 1 design tunnel, discharge 3,600 cfs, tide el 6.7, n 0.013

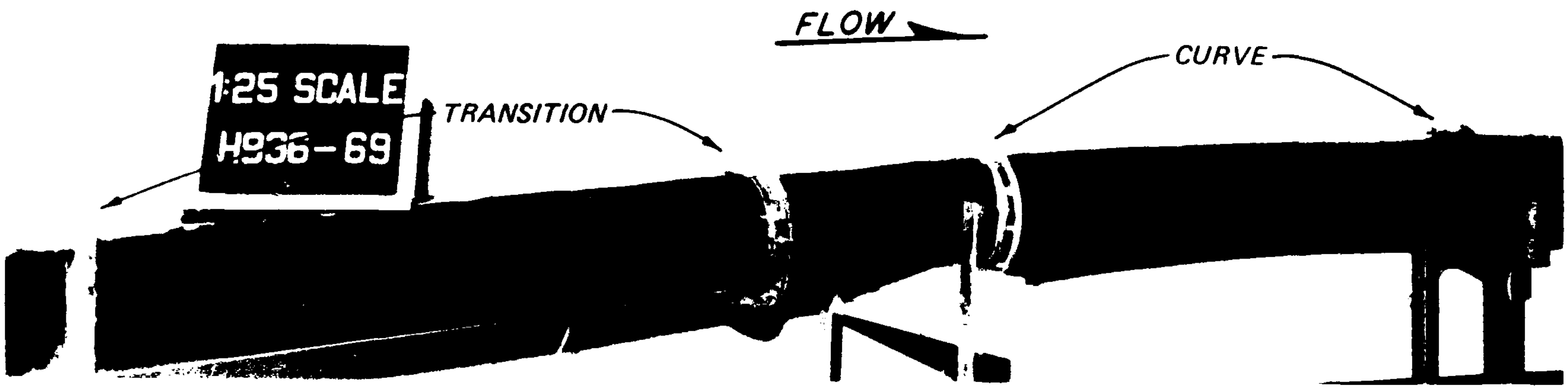


Photo 5. Flow condition in type 2 design transition, discharge 4,039 cfs, tide el 6.7, n 0.013

Photo 6. Stilling basin action and surface currents, discharge 512 cfs, tide el 6.7

Photo 7. Stilling basin action and surface currents, discharge 1,208 cfs, tide el 6.7

Photo 8. Stilling basin action and surface currents, discharge 4,039 cfs, tide el -2.7

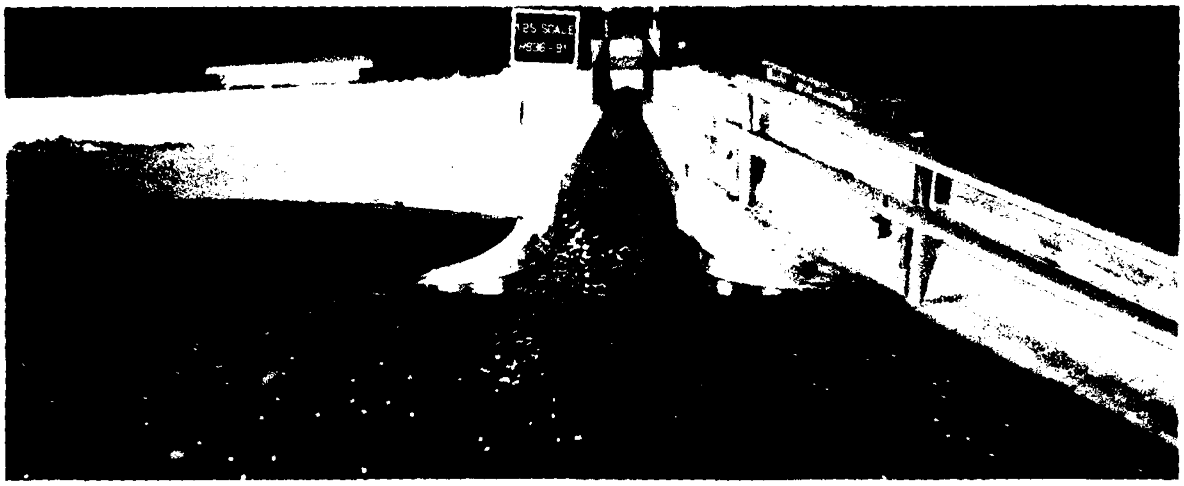


Photo 9. Flow conditions, type 6 design, n 0.013, discharge 512 cfs, tide el 6.7

Photo 10. Flow conditions, type 6 design, n 0.013, discharge 1,208 cfs, tide el 6.7

Photo 11. Flow conditions, type 6 design, n 0.013, discharge 4,039 cfs, tide el -2.7

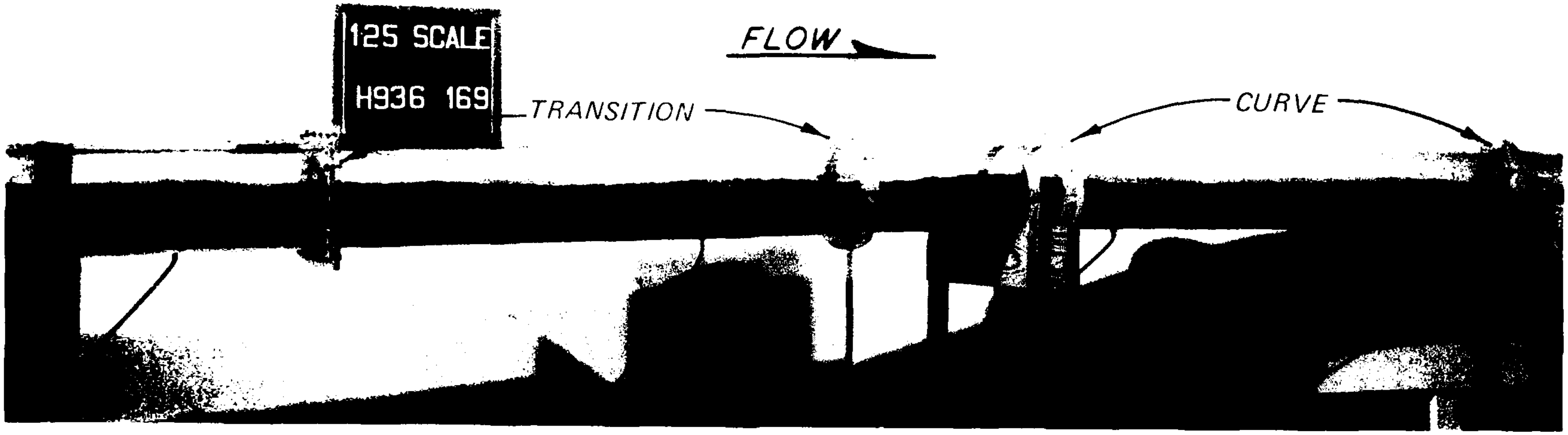


Photo 12. Flow conditions in type 2 design tunnel, discharge 4,039 cfs, tide el 6.7, n 0.010

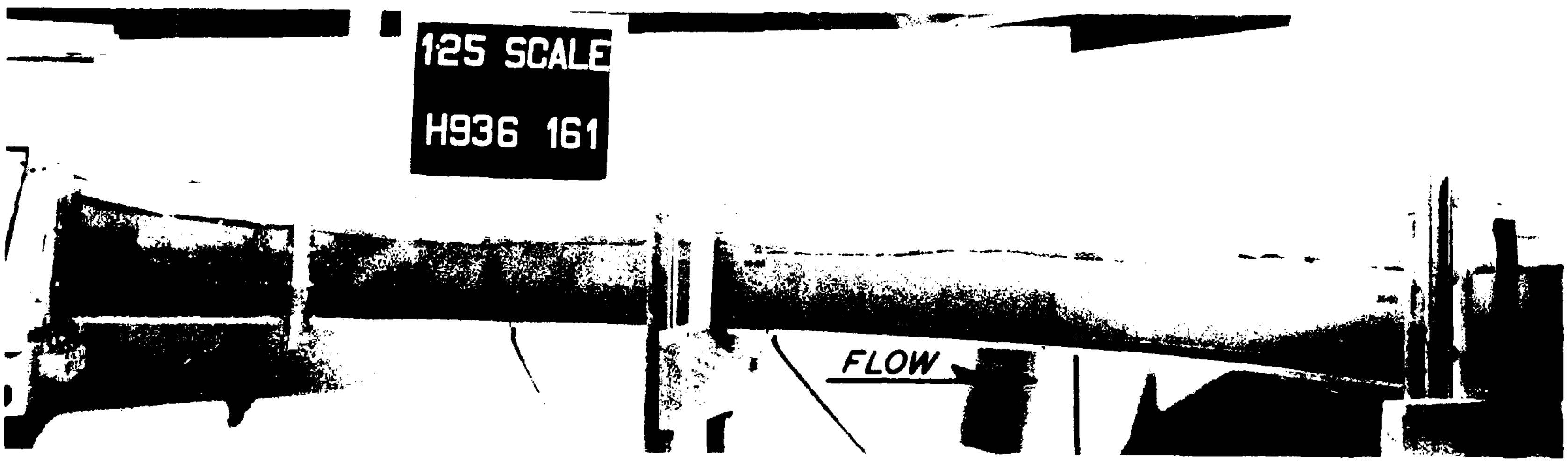


Photo 13. Flow conditions at tunnel entrance, type 2 design tunnel, discharge 4,039 cfs, tide el 6.7, n 0.010

Photo 14. Surface conditions, type 6 stilling basin, discharge 512 cfs, tide el 6.7

Photo 15. Surface conditions, type 6 stilling basin, discharge 1,208 cfs, tide el 6.7

Photo 16. Surface conditions, type 6 stilling basin, discharge 4,039 cfs, tide el -2.7

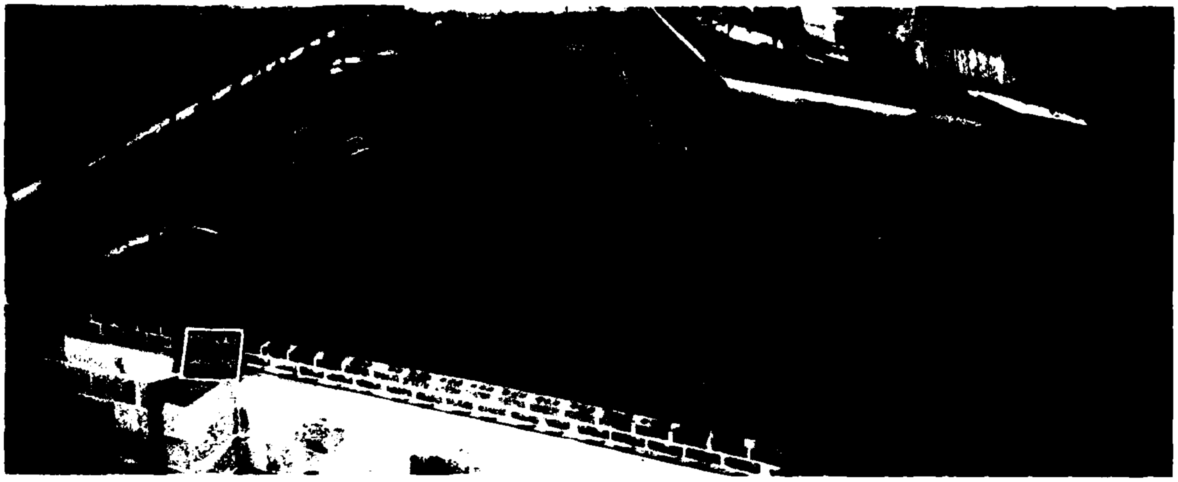


Photo 17. Surface circulation patterns, discharge 512 cfs, tide el 6.7

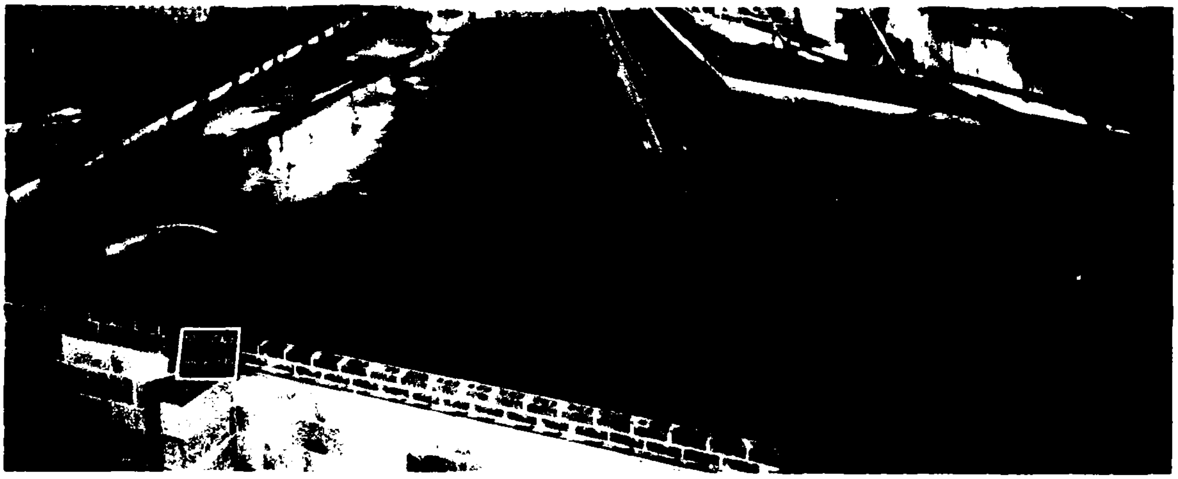


Photo 18. Surface circulation patterns, discharge 1,208 cfs, tide el 6.7

Photo 19. Surface circulation patterns, discharge 4,039 cfs, tide el -2.7

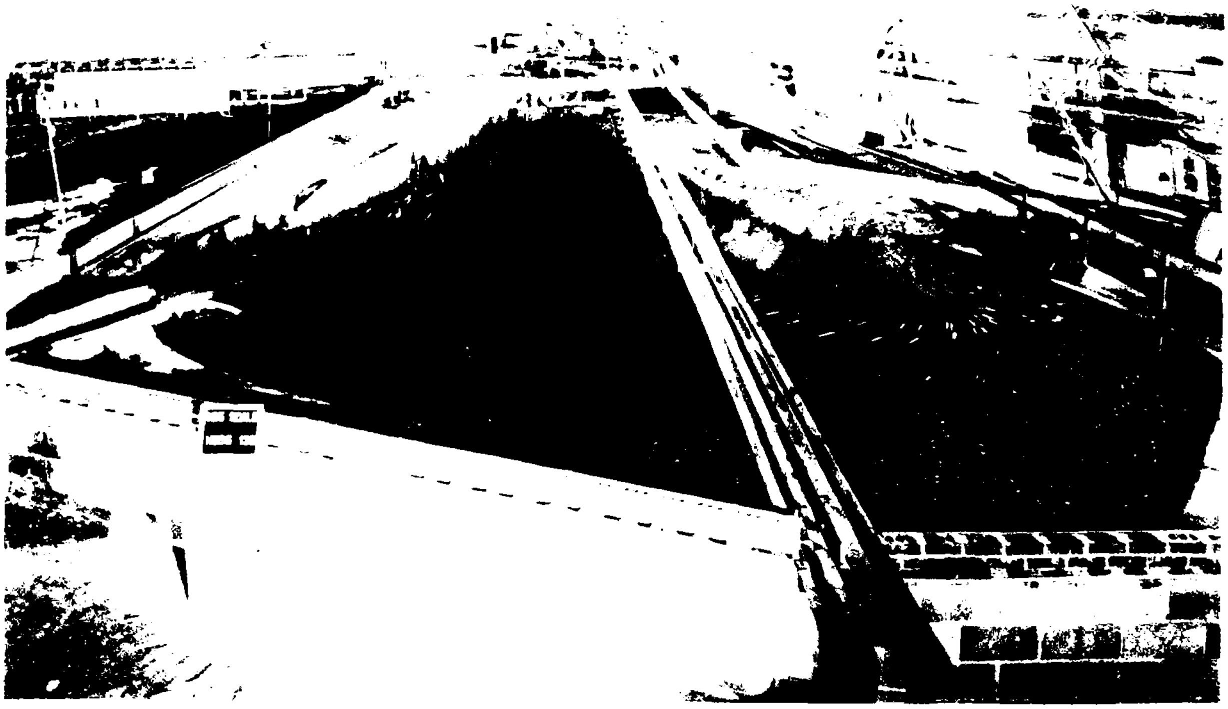
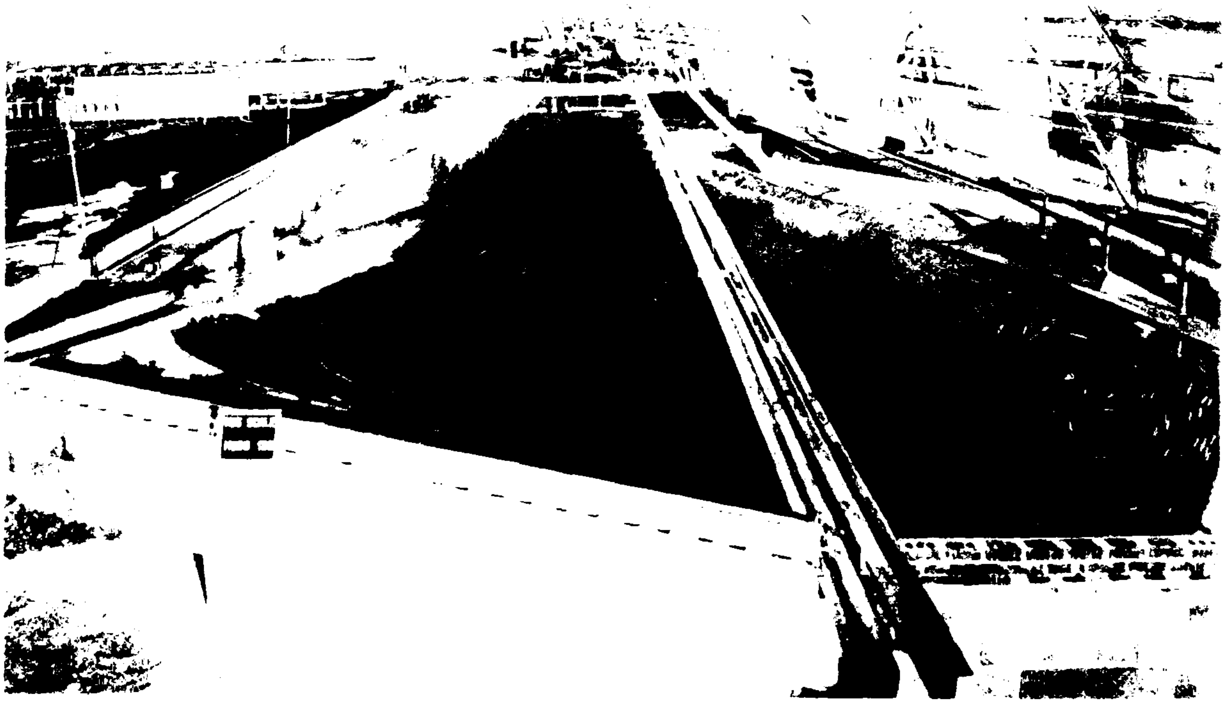
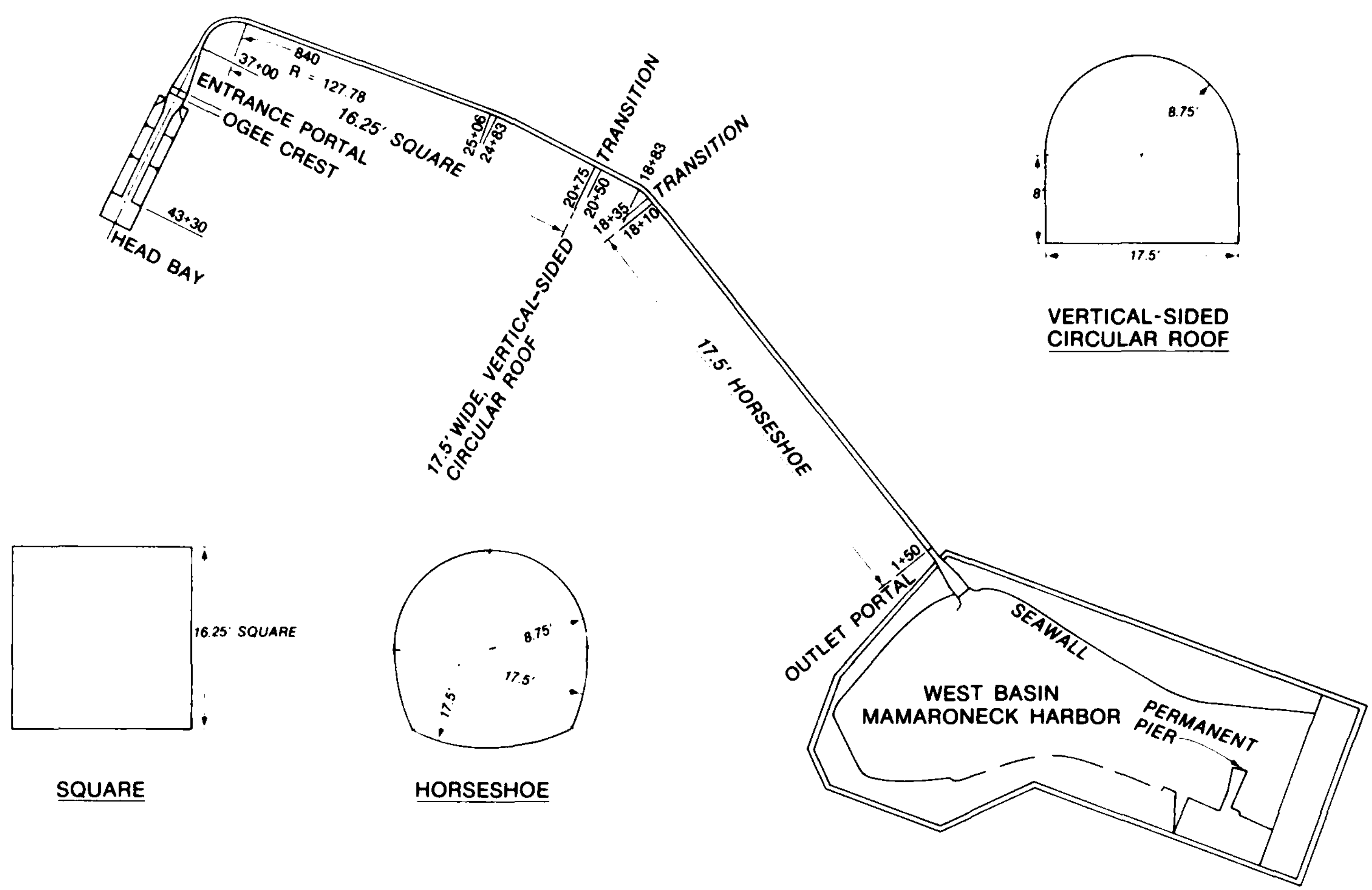


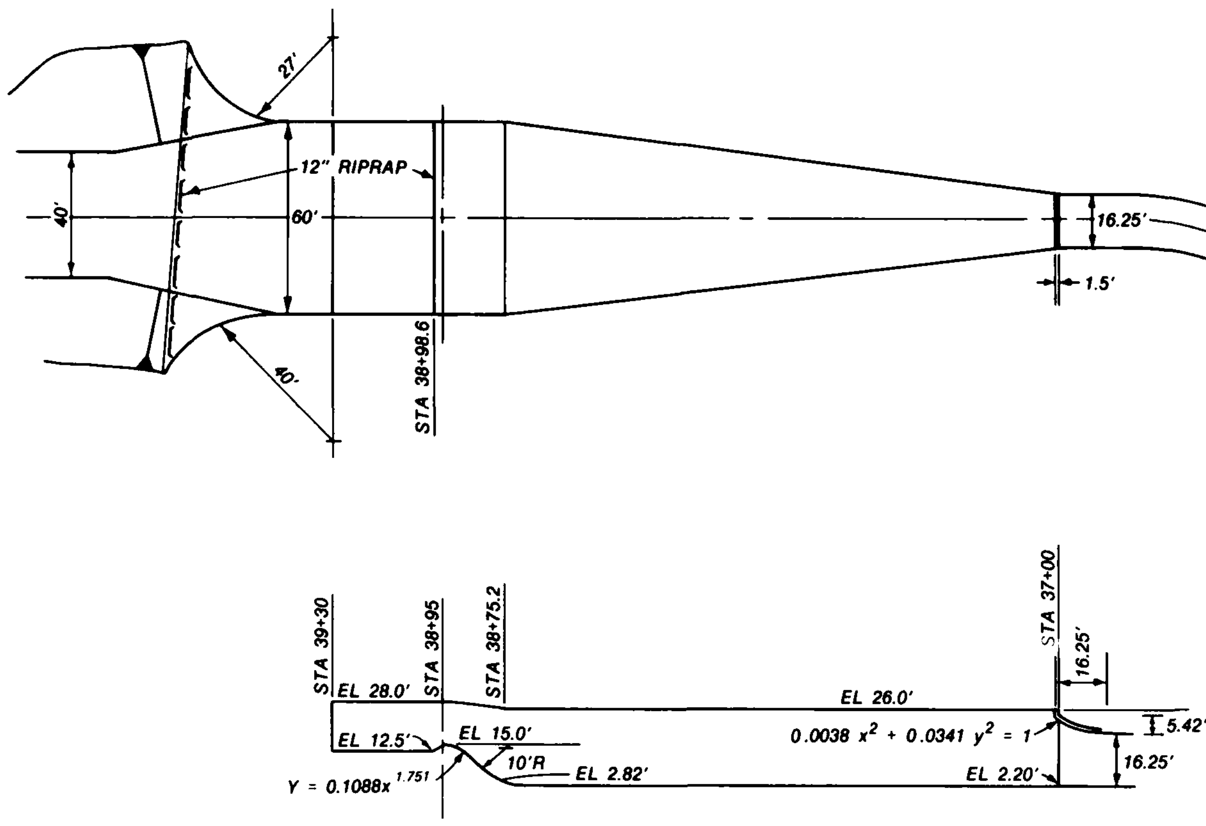
Photo [illegible]. Surface circulation patterns, discharge [illegible] cfs, tide el 6.7



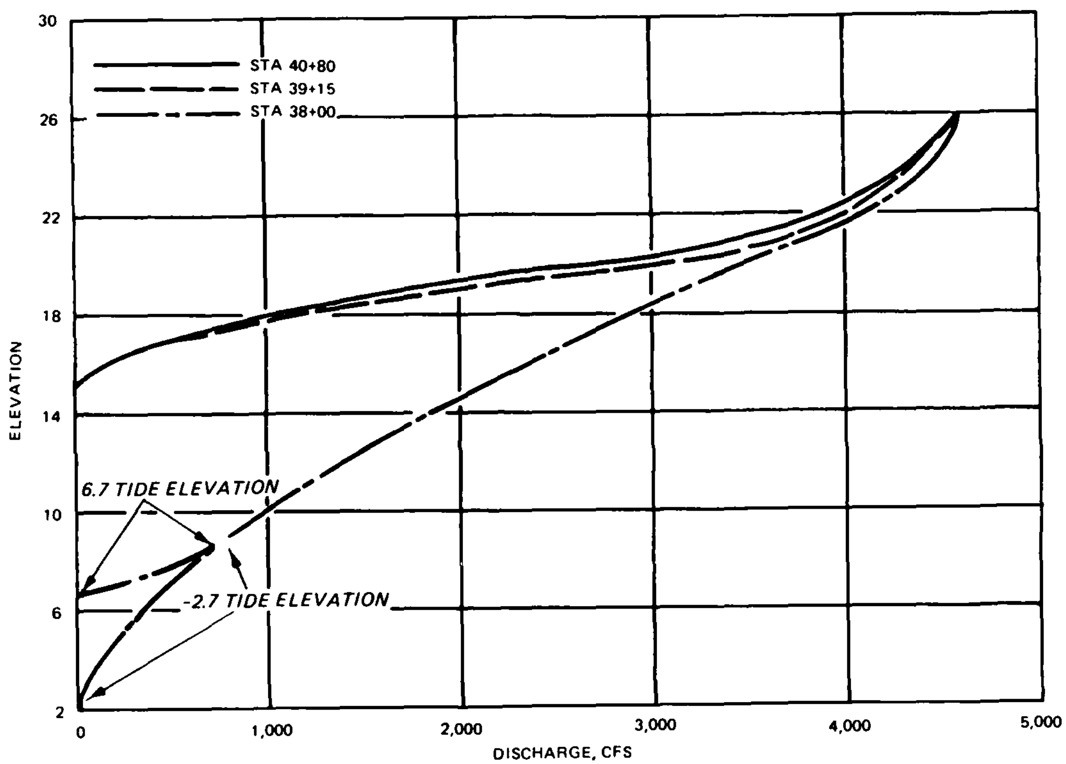
[illegible]



PLAN VIEW
SHELDRAKE RIVER TUNNEL MODEL



SHELDRAKE RIVER TUNNEL APPROACH
ORIGINAL DESIGN

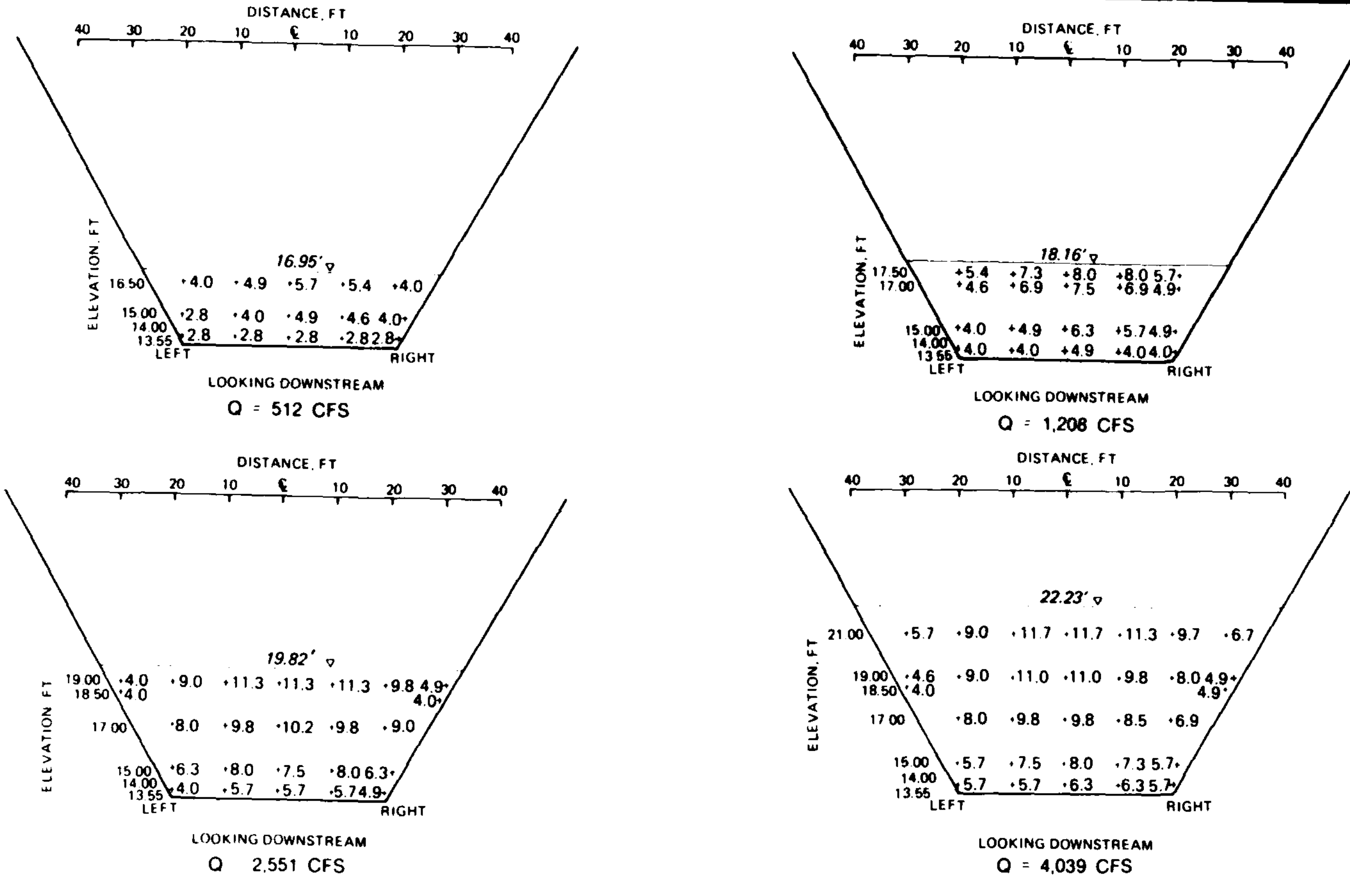


DISCHARGE RATING CURVES
SHELDRAKE RIVER TUNNEL

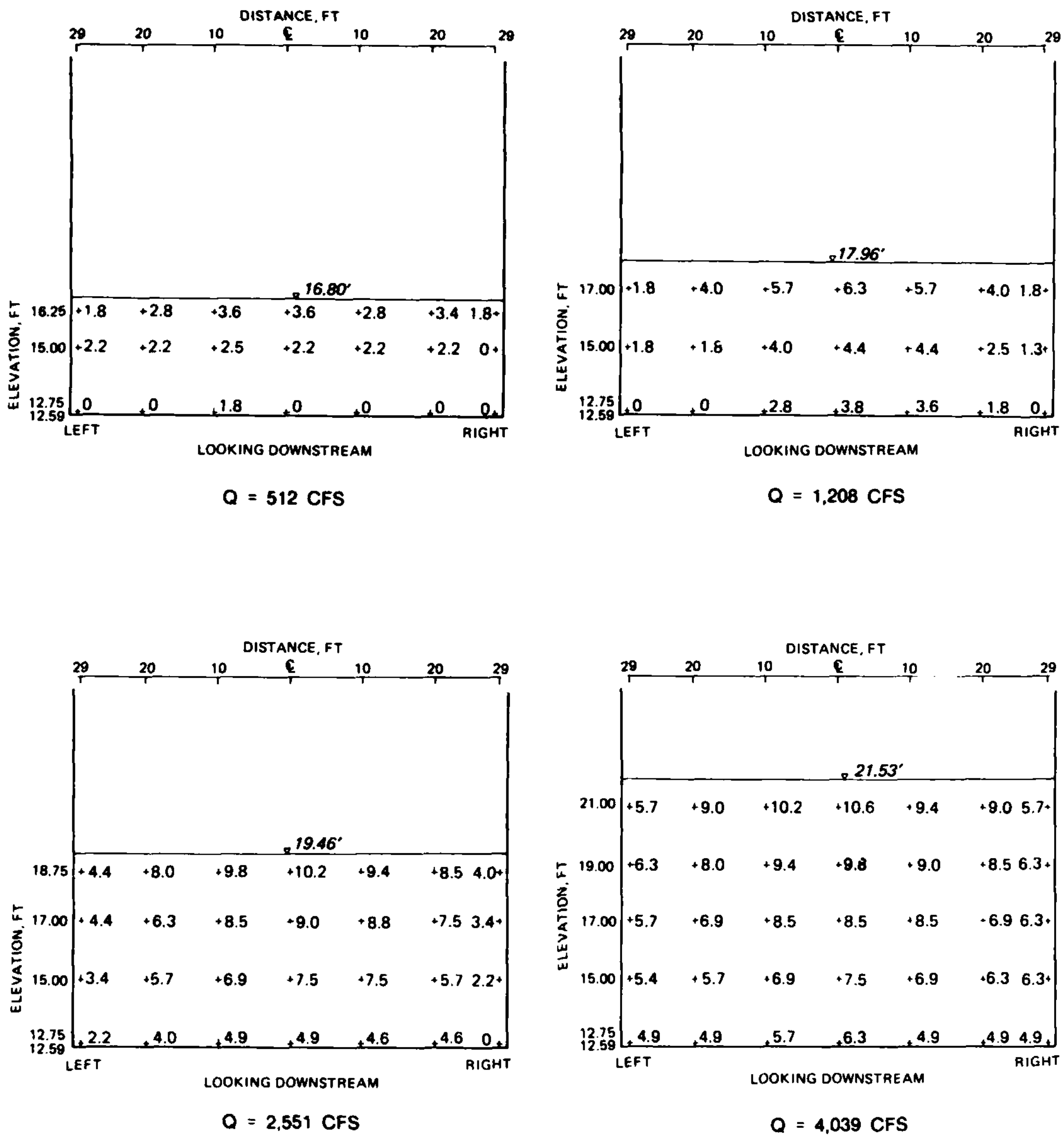
TIDE EL = 6.7 AND -2.7

DESIGN: ORIGINAL CREST & CONDUIT

TUNNEL $n = 0.013$

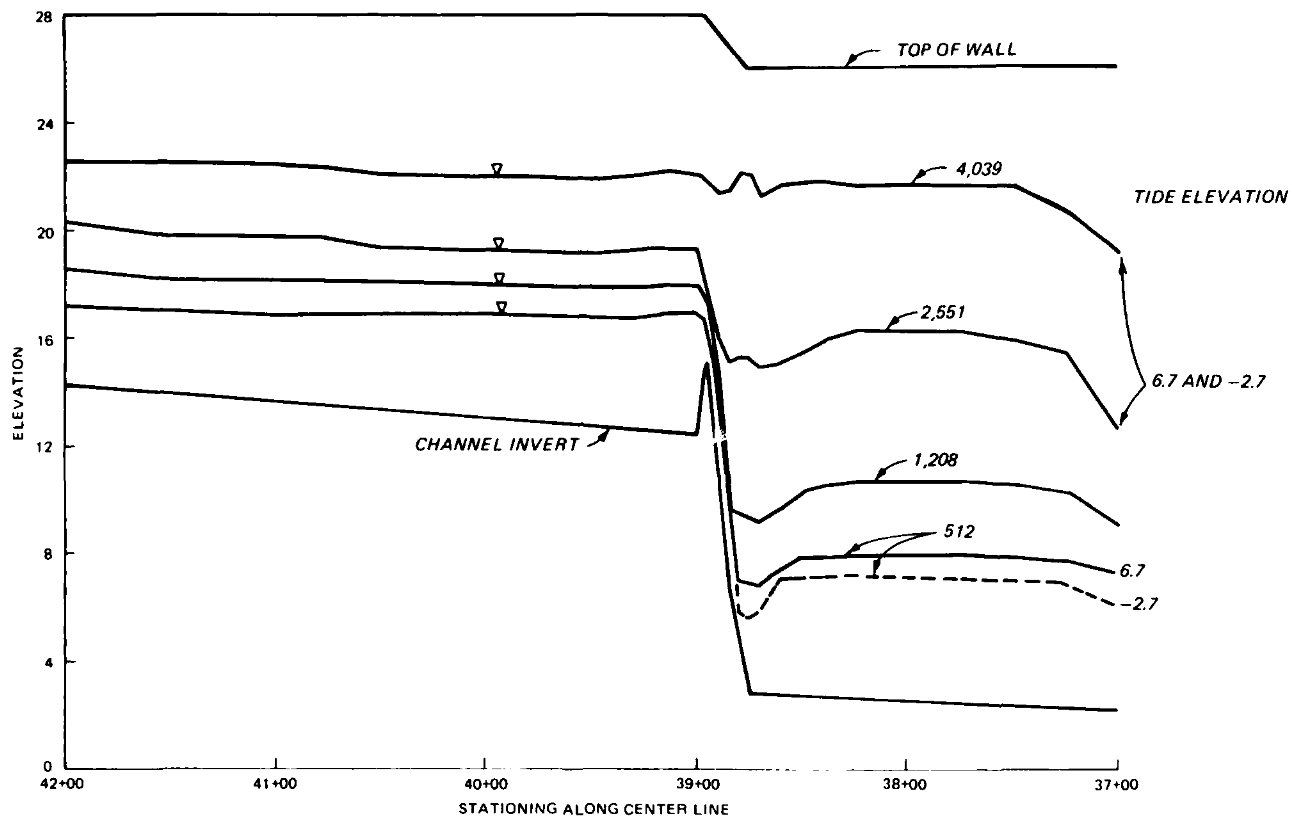


VELOCITY CROSS SECTION
SHELDRAKE RIVER TUNNEL APPROACH
STA 40+80
TIDE EL = 6.7
DESIGN: ORIGINAL CREST & CONDUIT
TUNNEL n = 0.013

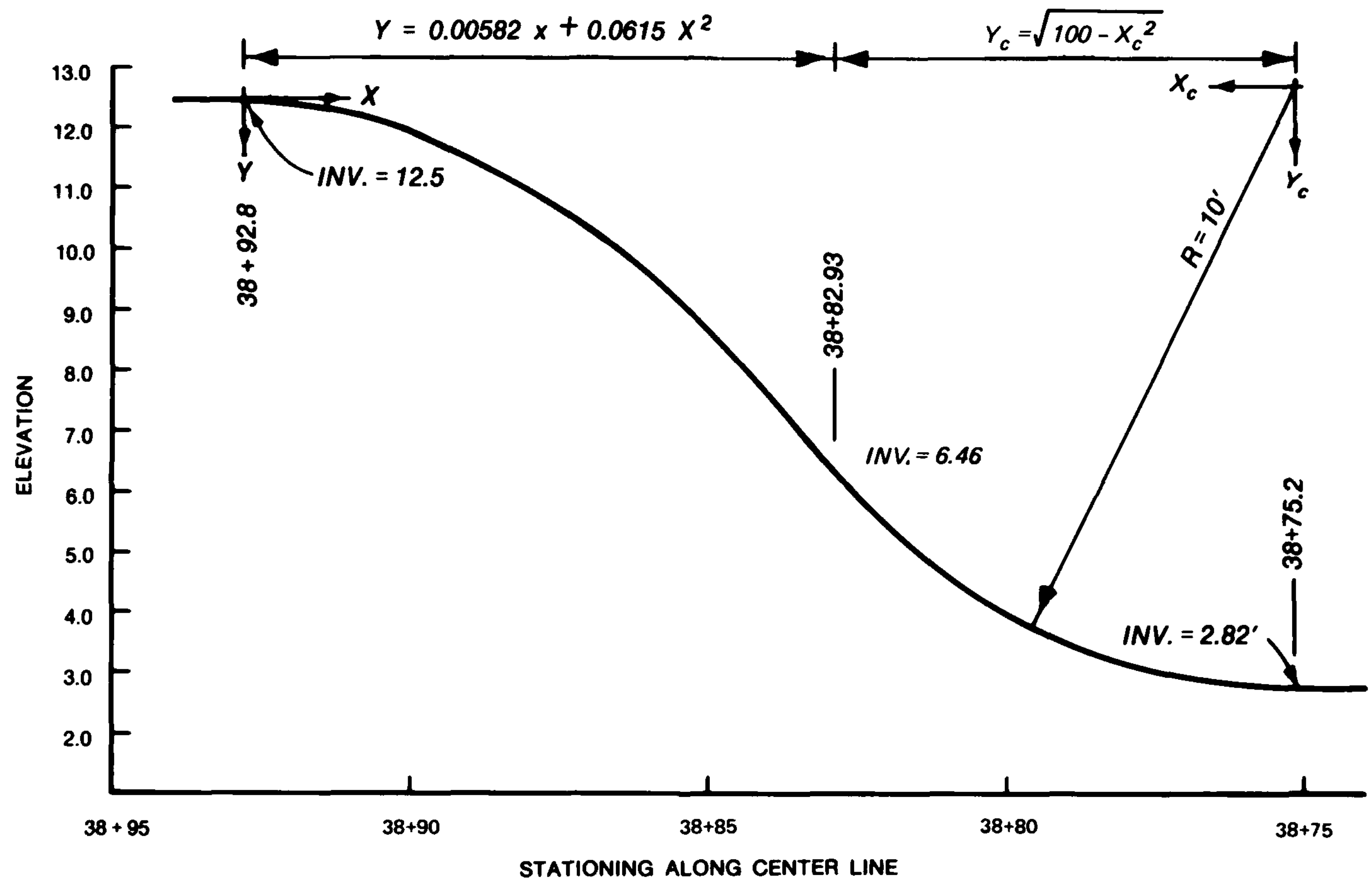


VELOCITY CROSS SECTION
SHELDRAKE RIVER TUNNEL APPROACH

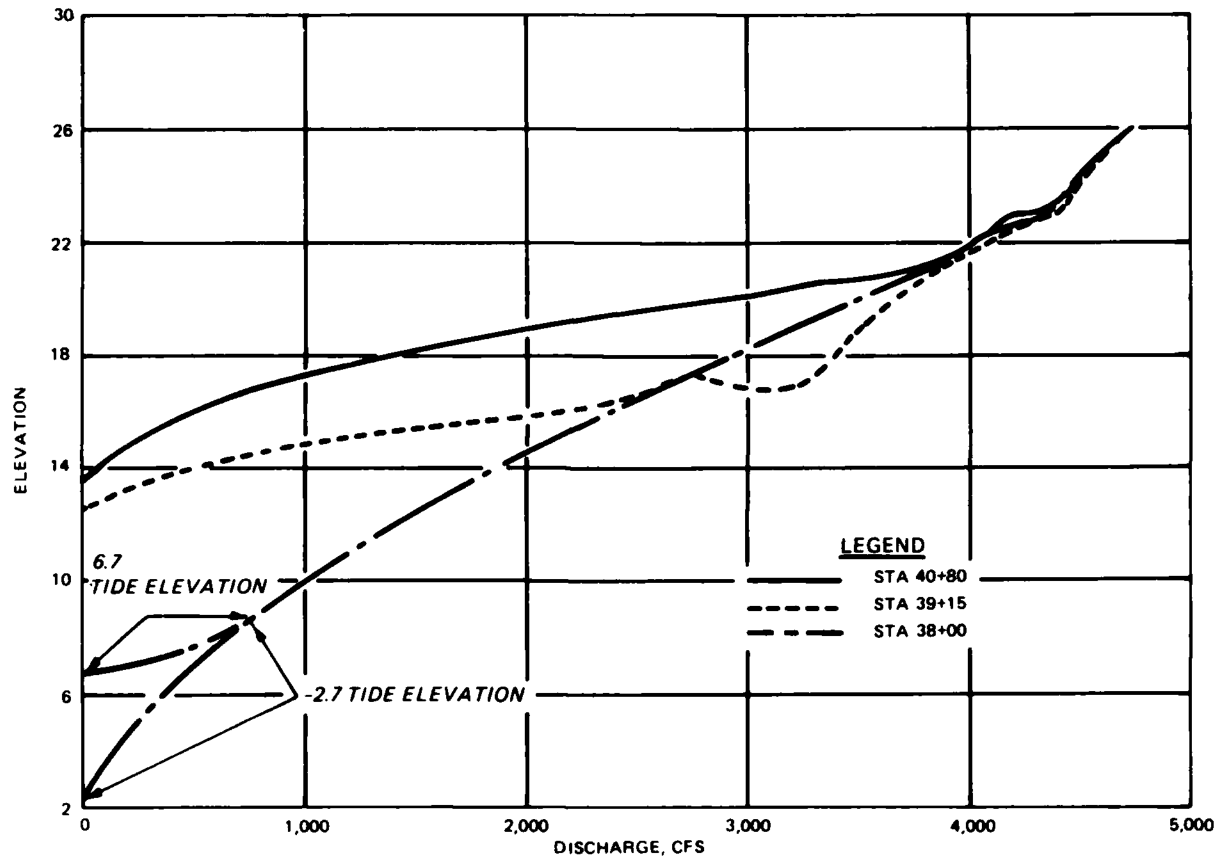
STA 39+15
TIDE EL = 6.7
DESIGN: ORIGINAL CREST & CONDUIT
TUNNEL n = 0.013



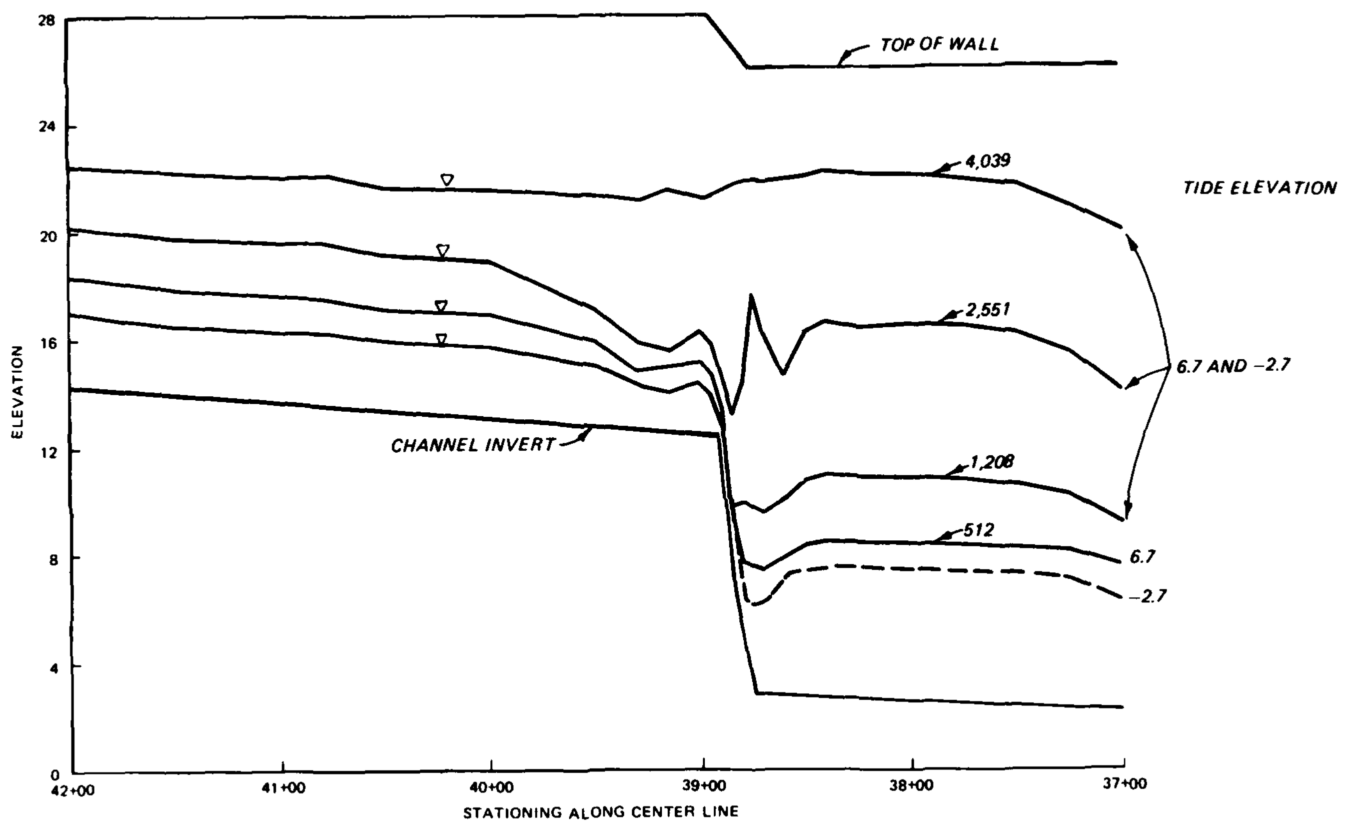
WATER-SURFACE PROFILE
SHELDRAKE RIVER TUNNEL APPROACH
TIDE EL = 6.7 AND -2.7
DESIGN: ORIGINAL CREST & CONDUIT
TUNNEL n = 0.013



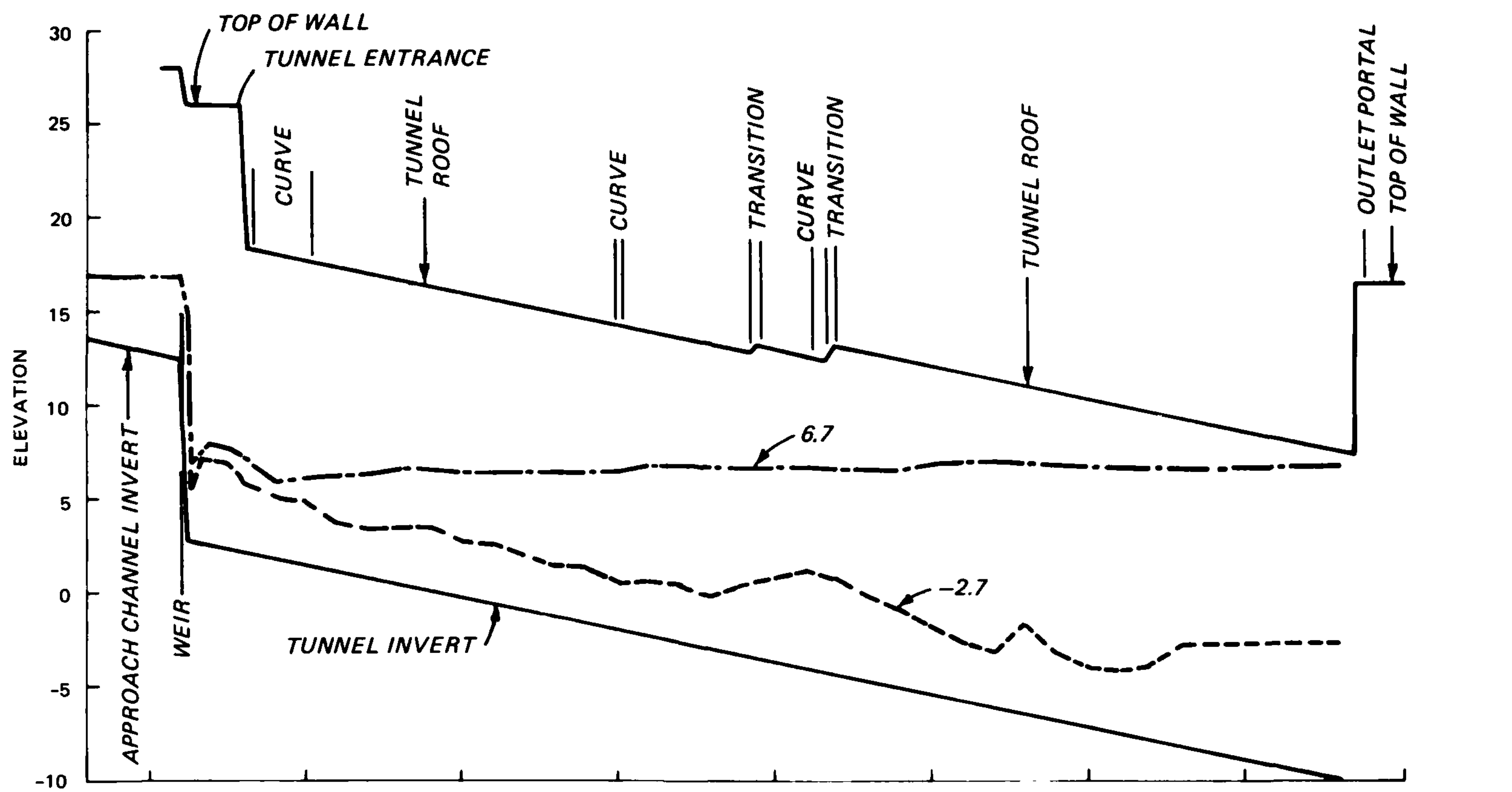
MAMARONECK & SHELDRAKE TUNNEL INLET
DESIGN: TYPE 2 CREST



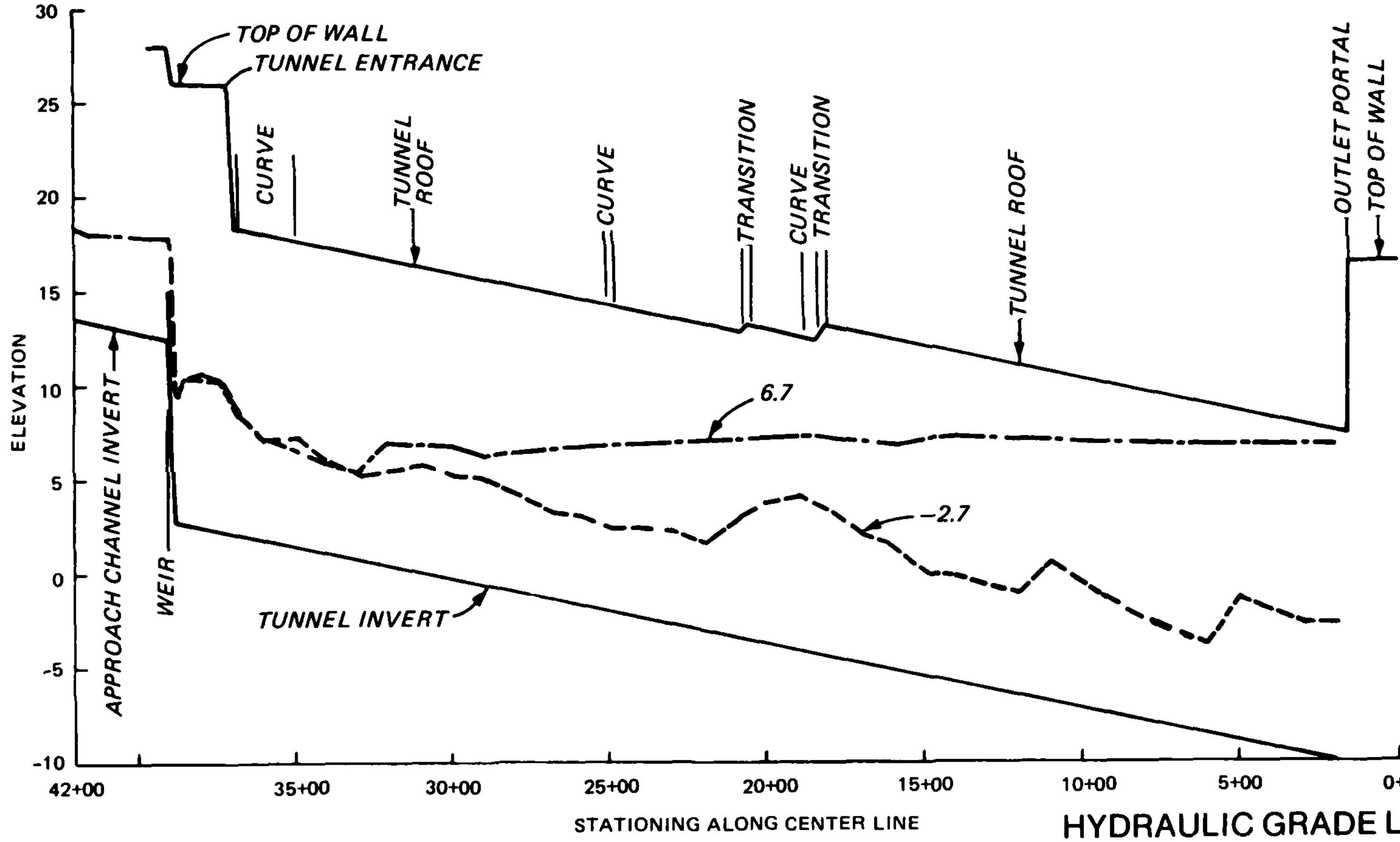
DISCHARGE RATING CURVES
SHELDRAKE RIVER TUNNEL
TIDE EL = 6.7 AND -2.7
DESIGN: TYPE 2 CREST
MANNING'S n = 0.013



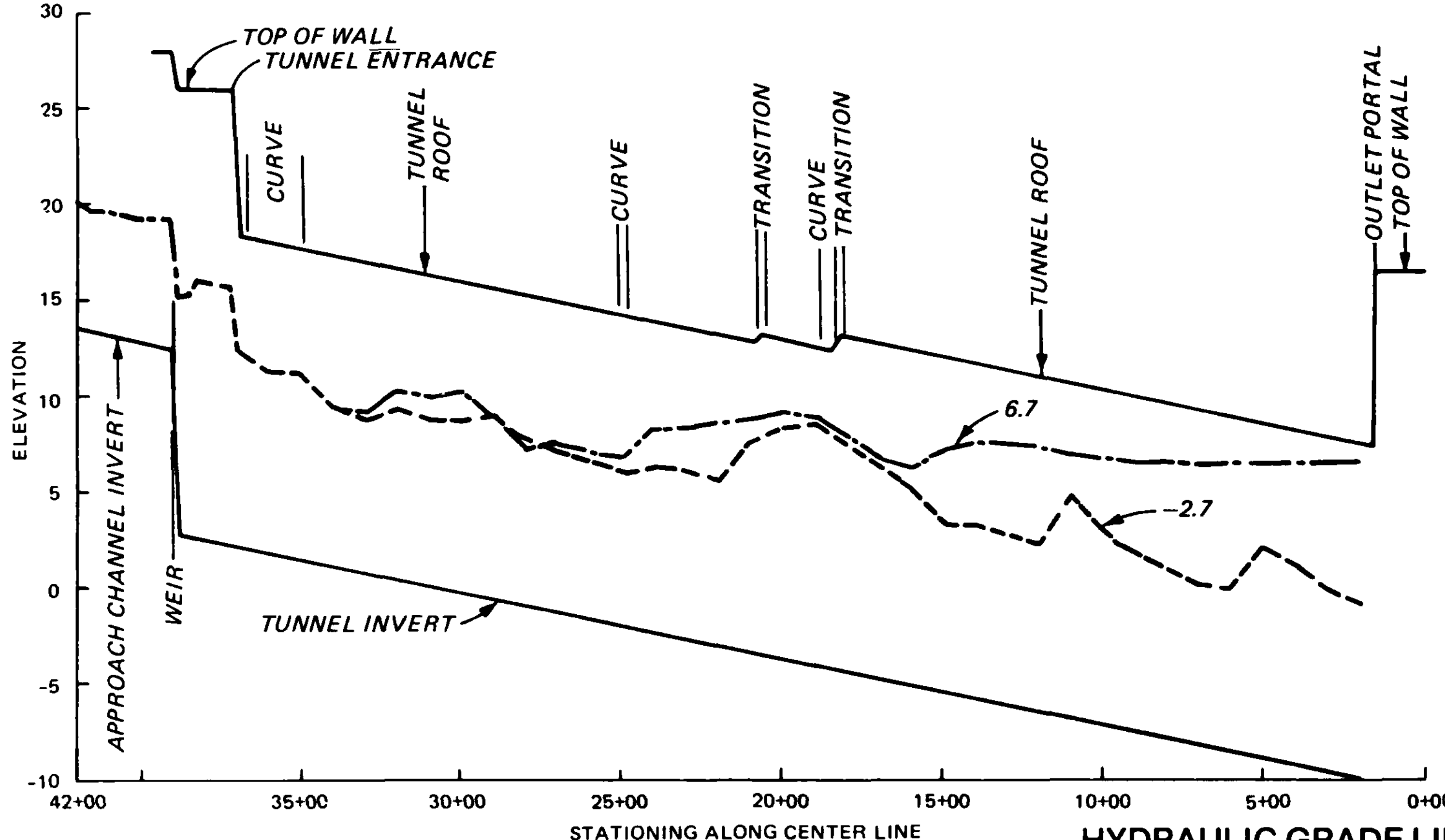
WATER-SURFACE PROFILE
SHELDRAKE RIVER TUNNEL APPROACH
TIDE EL +6.7 AND -2.7
DESIGN: TYPE 2 CREST
TUNNEL n = 0.013



HYDRAULIC GRADE LINE
SHELDRAKE RIVER TUNNEL
TIDE EL = 6.7 AND -2.7
DESIGN: ORIGINAL CONDUIT
TUNNEL n = 0.013
DISCHARGE: 512 CFS

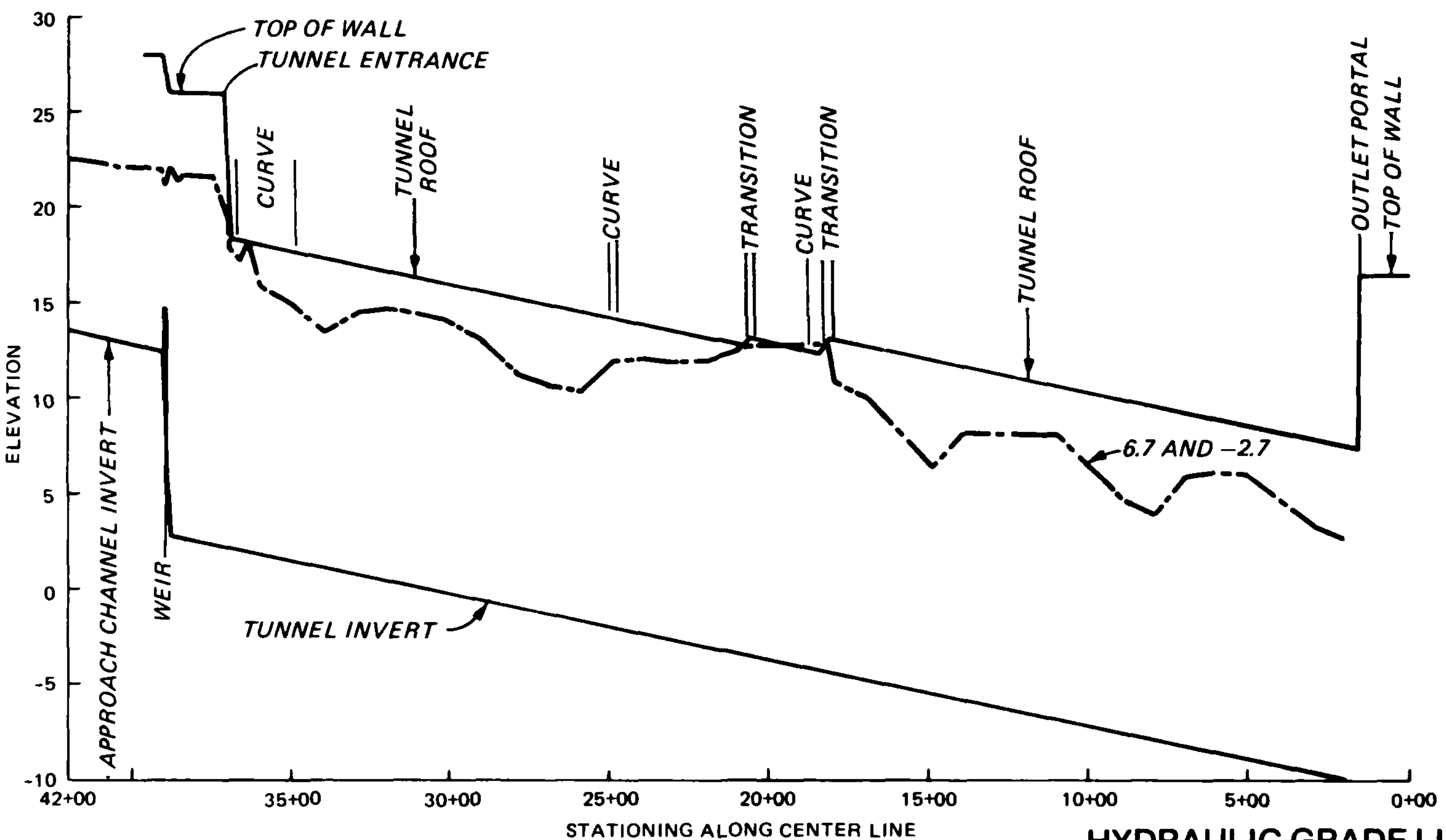


HYDRAULIC GRADE LINE
SHELDRAKE RIVER TUNNEL
TIDE EL = 6.7 AND -2.7
DESIGN: ORIGINAL CONDUIT
TUNNEL n = 0.013
DISCHARGE: 1,208 CFS

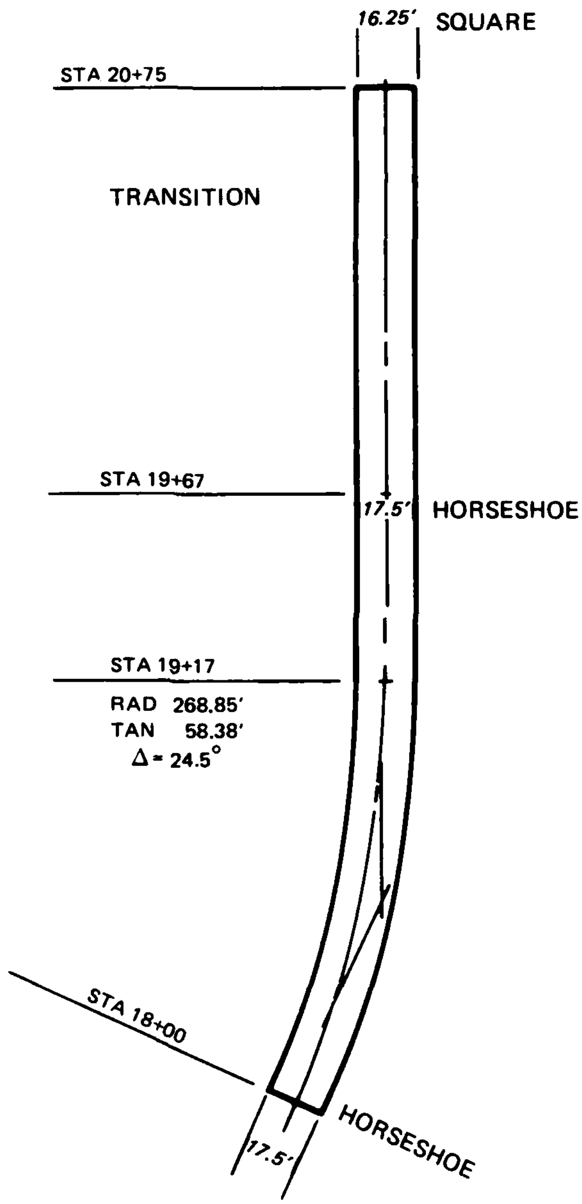


HYDRAULIC GRADE LINE
SHELDRAKE RIVER TUNNEL
TIDE EL = 6.7 AND -2.7
DESIGN: ORIGINAL CONDUIT
TUNNEL n = 0.013
DISCHARGE: 2,551 CFS

PLATE 12

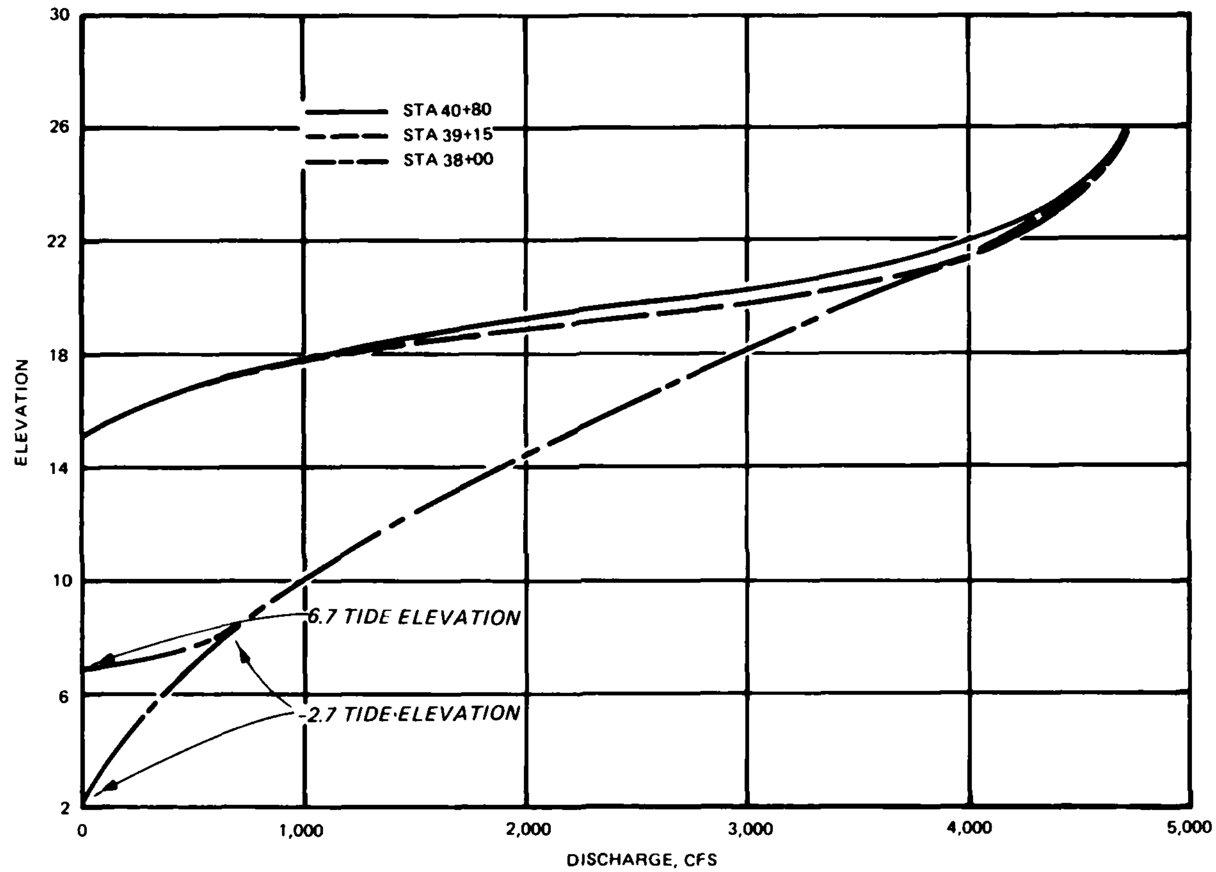


HYDRAULIC GRADE LINE
SHELDRAKE RIVER TUNNEL
TIDE EL = 6.7 AND -2.7
DESIGN: ORIGINAL CONDUIT
TUNNEL n = 0.013
DISCHARGE: 4,039 CFS

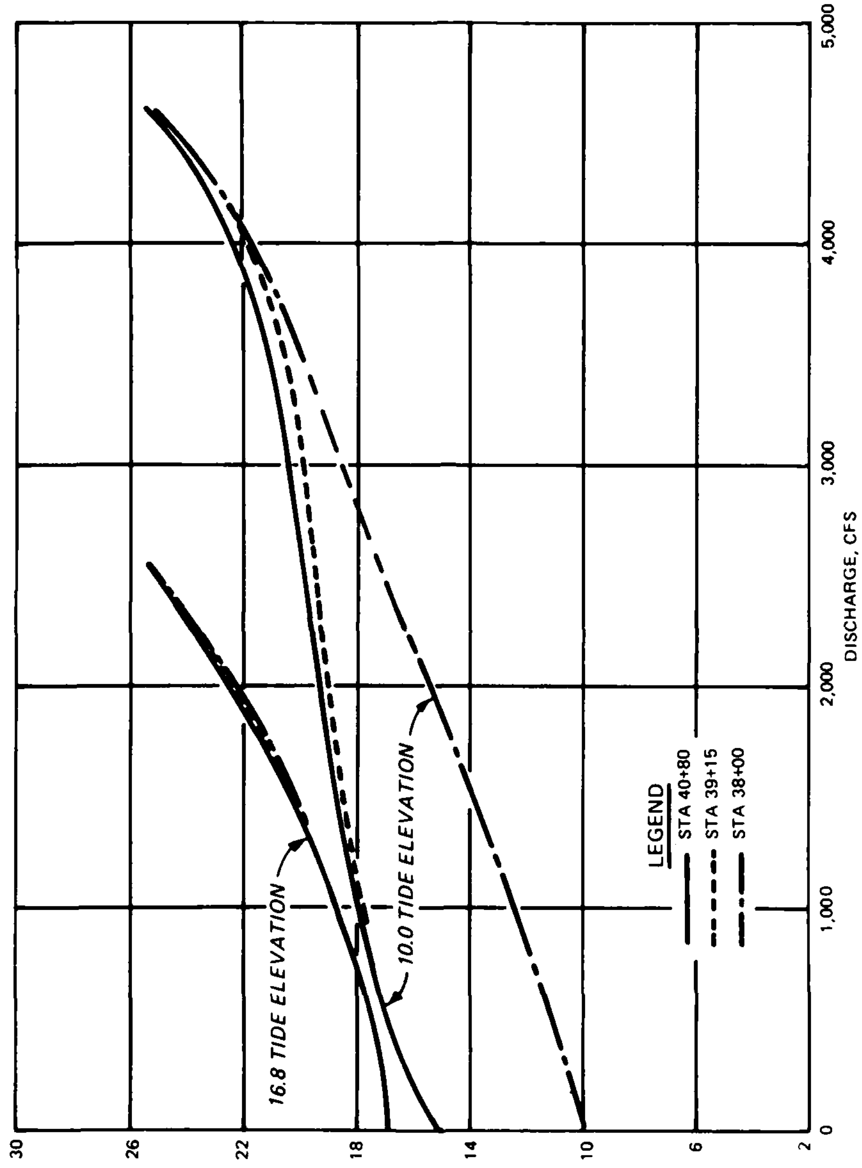


SHELDRAKE RIVER TUNNEL
DESIGN: TYPE 2 CONDUIT

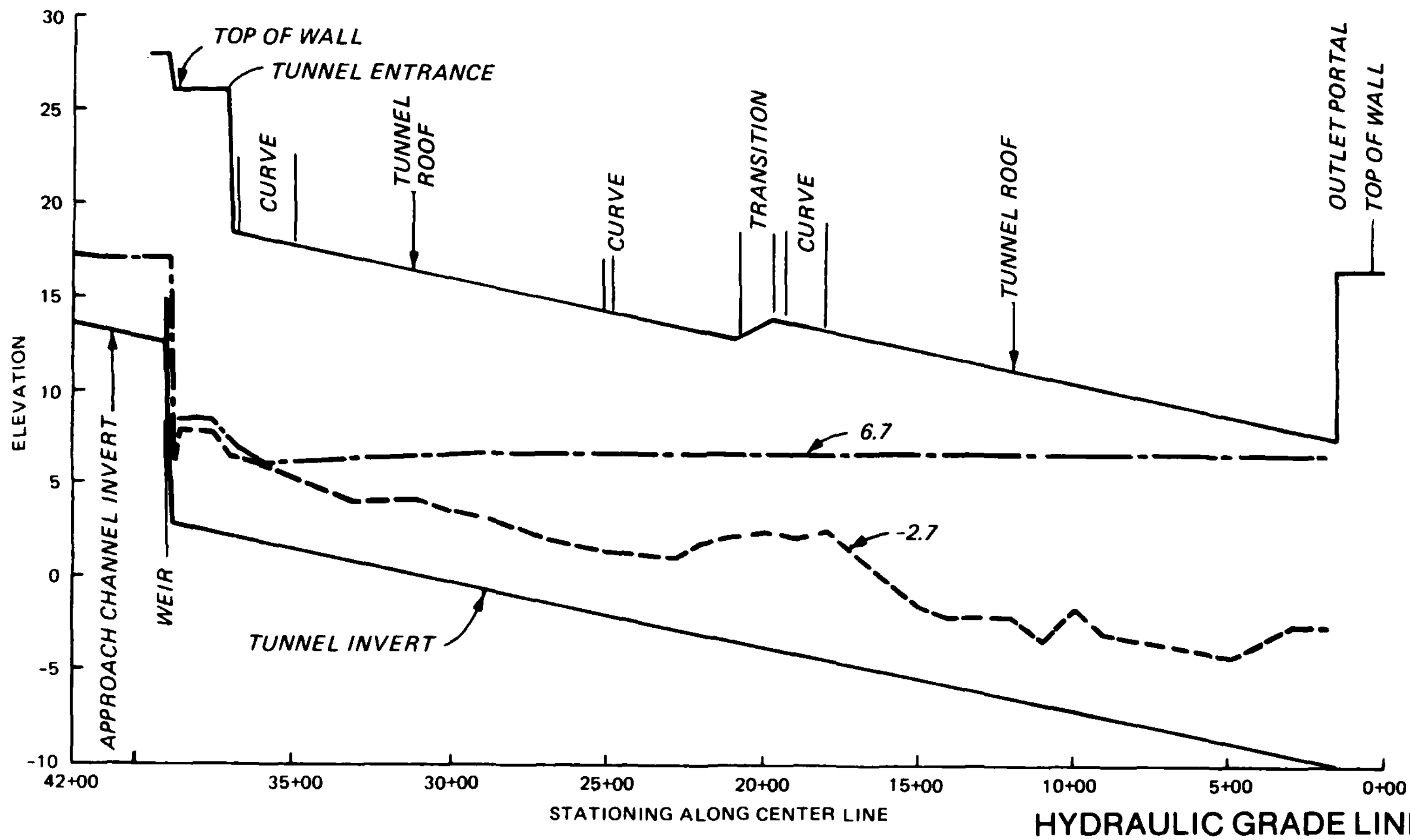
PLATE 14



DISCHARGE RATING CURVES
SHELDRAKE RIVER TUNNEL
TIDE EL = 6.7 AND -2.7
DESIGN: TYPE 2 CONDUIT
MANNING'S n = 0.013



DISCHARGE RATING CURVES
SHELDRAKE RIVER TUNNEL
TIDE EL = 10.0 AND 16.8
DESIGN: TYPE 2 CONDUIT
MANNING'S n = 0.013



HYDRAULIC GRADE LINE
SHELDRAKE RIVER TUNNEL

TIDE EL = 6.7 AND -2.7
DESIGN: TYPE 2 CONDUIT
TUNNEL n = 0.013
DISCHARGE: 512 CFS

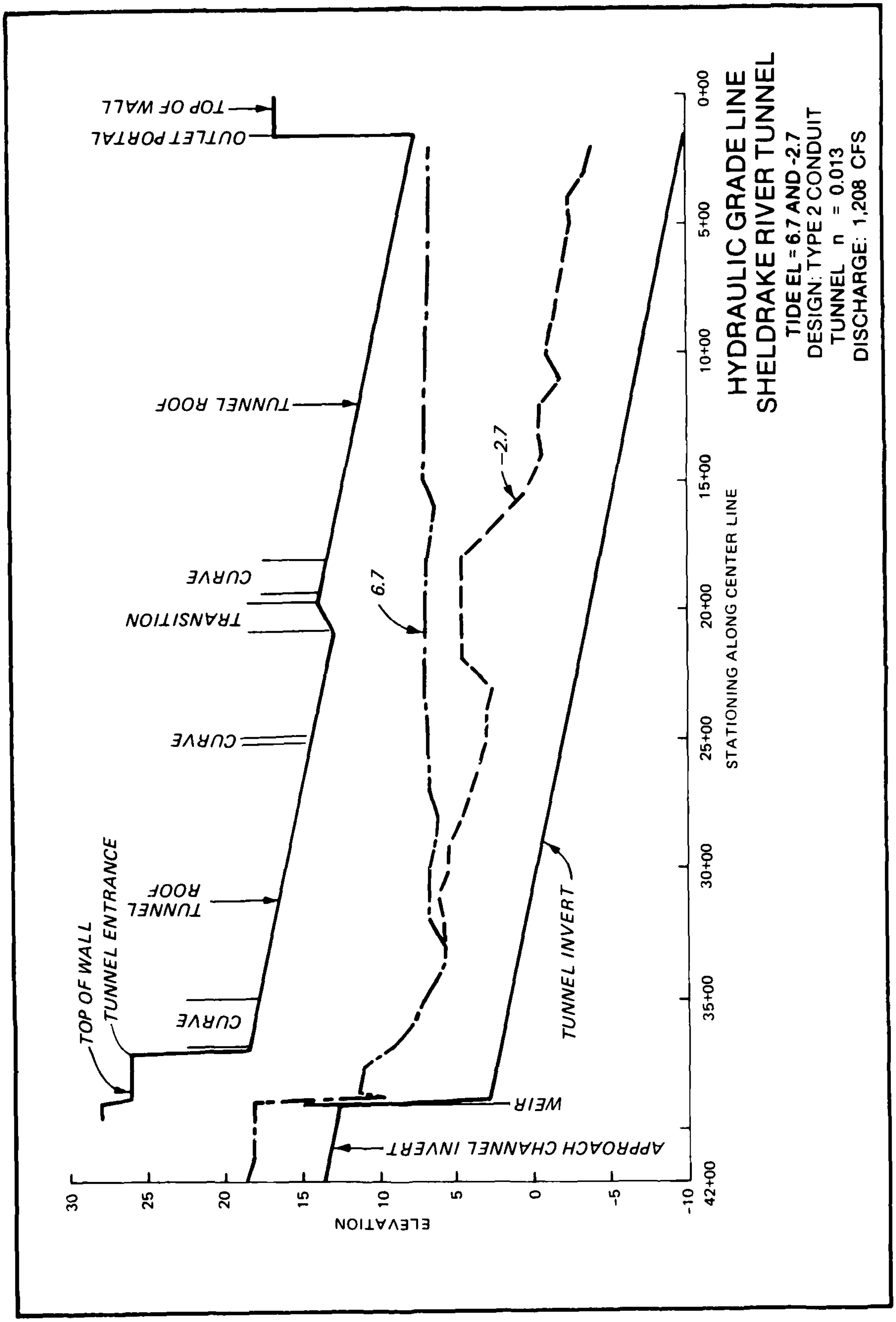
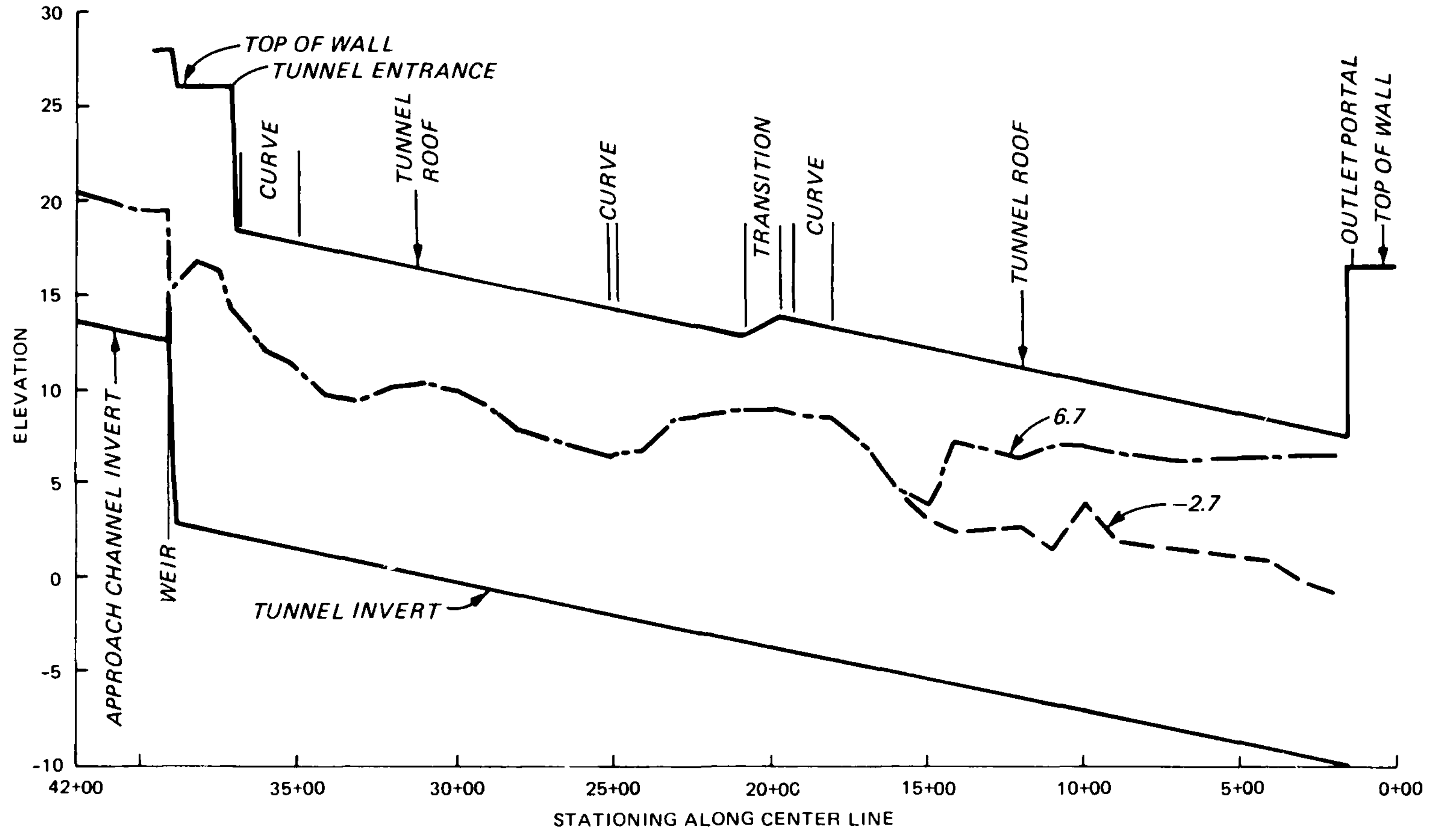
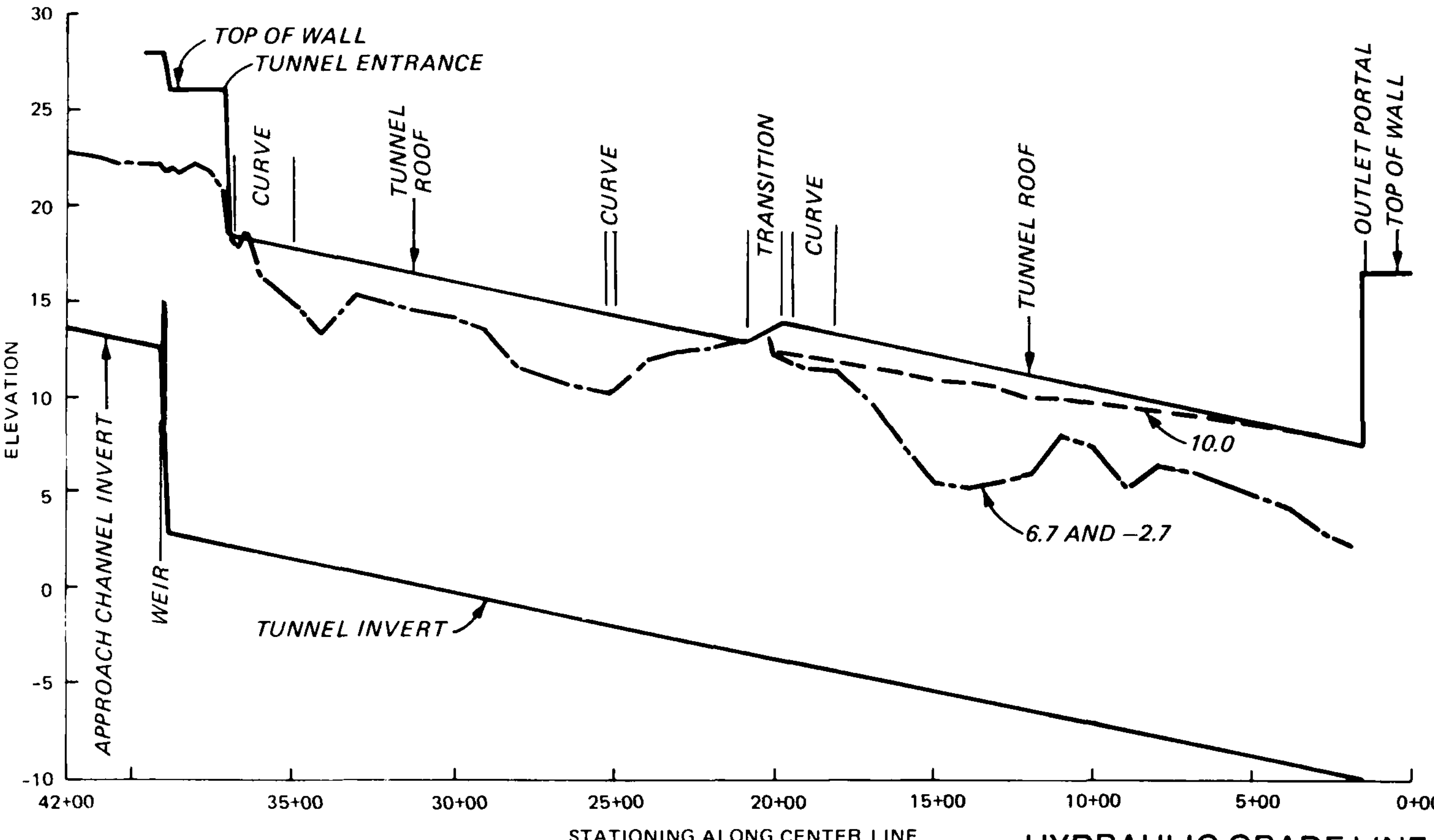

HYDRAULIC GRADE LINE
SHELDRAKE RIVER TUNNEL
TIDE EL = 6.7 AND -2.7
DESIGN: TYPE 2 CONDUIT
TUNNEL n = 0.013
DISCHARGE: 1,208 CFS
ELEVATION
STATIONING ALONG CENTER LINE
30
25
20
15
10
5
0
-5
-10
42+00
35+00
30+00
25+00
20+00
15+00
10+00
5+00
0+00
TOP OF WALL
OUTLET PORTAL
TUNNEL ROOF
CURVE
TRANSITION
CURVE
TUNNEL ROOF
TOP OF WALL
TUNNEL ENTRANCE
CURVE
WEIR
APPROACH CHANNEL INVERT
TUNNEL INVERT
6.7
-2.7

PLATE 18

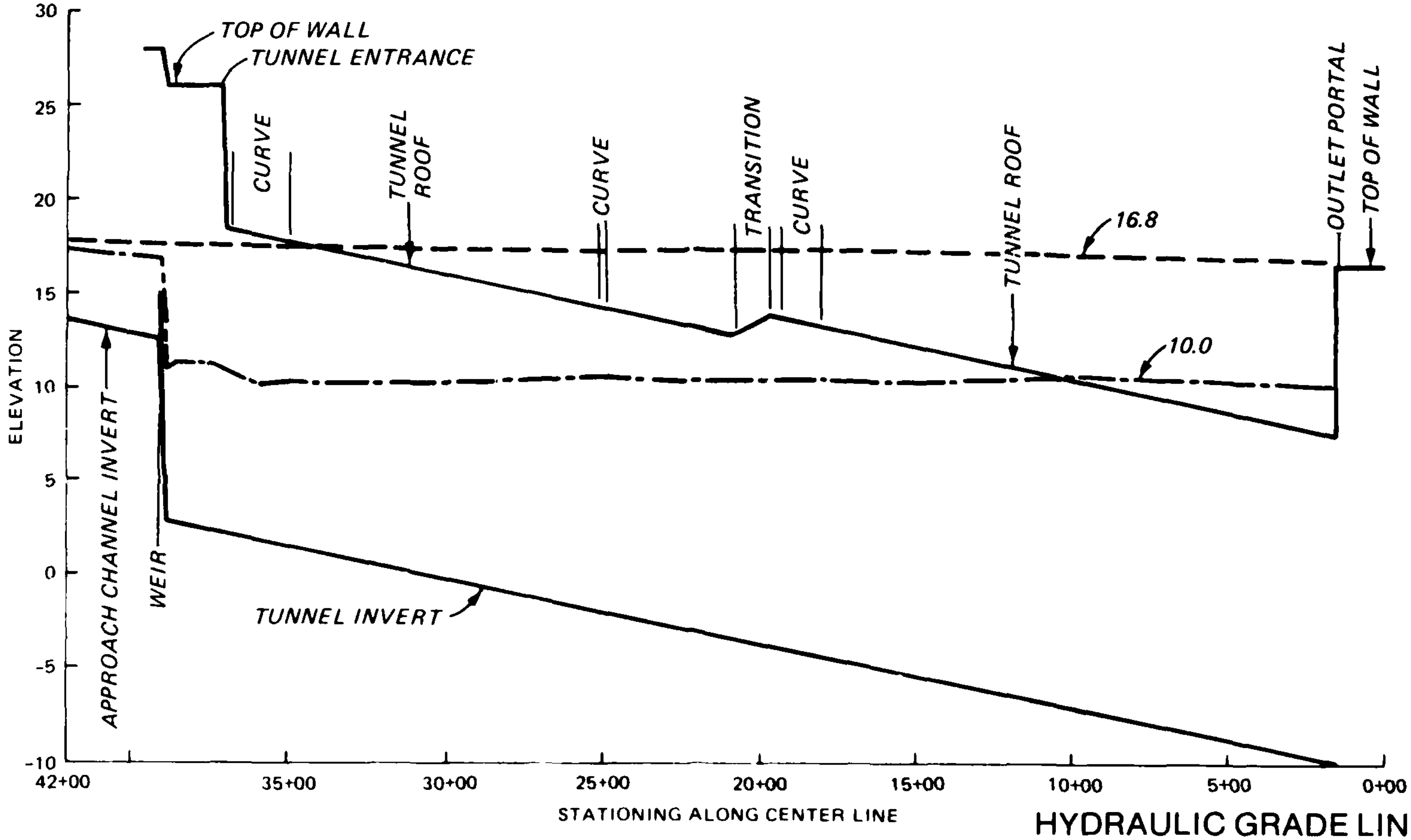


HYDRAULIC GRADE LINE
SHELDRAKE RIVER TUNNEL
TIDE EL = 6.7 AND -2.7
DESIGN: TYPE 2 CONDUIT
TUNNEL n = 0.013
DISCHARGE: 2,551 CFS

PLATE 19



HYDRAULIC GRADE LINE
SHELDRAKE RIVER TUNNEL
TIDE EL = 6.7, -2.7, AND 10.0
TUNNEL n = 0.013
DISCHARGE: 4,039 CFS



HYDRAULIC GRADE LINE
SHELDRAKE RIVER TUNNEL
TIDE EL = 10.0 AND 16.8
DESIGN: TYPE 2 CONDUIT
TUNNEL n = 0.013
DISCHARGE: 512 CFS

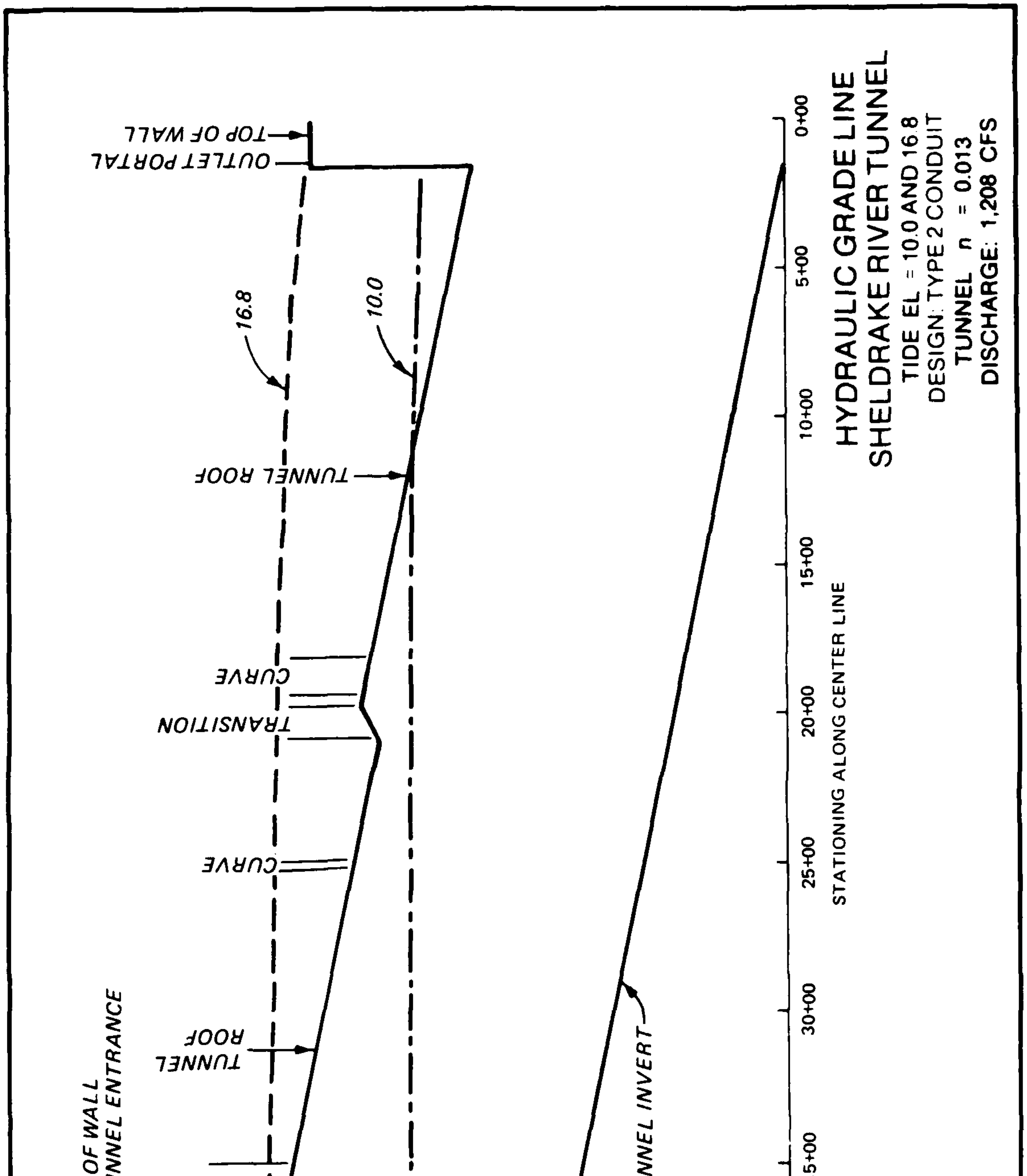
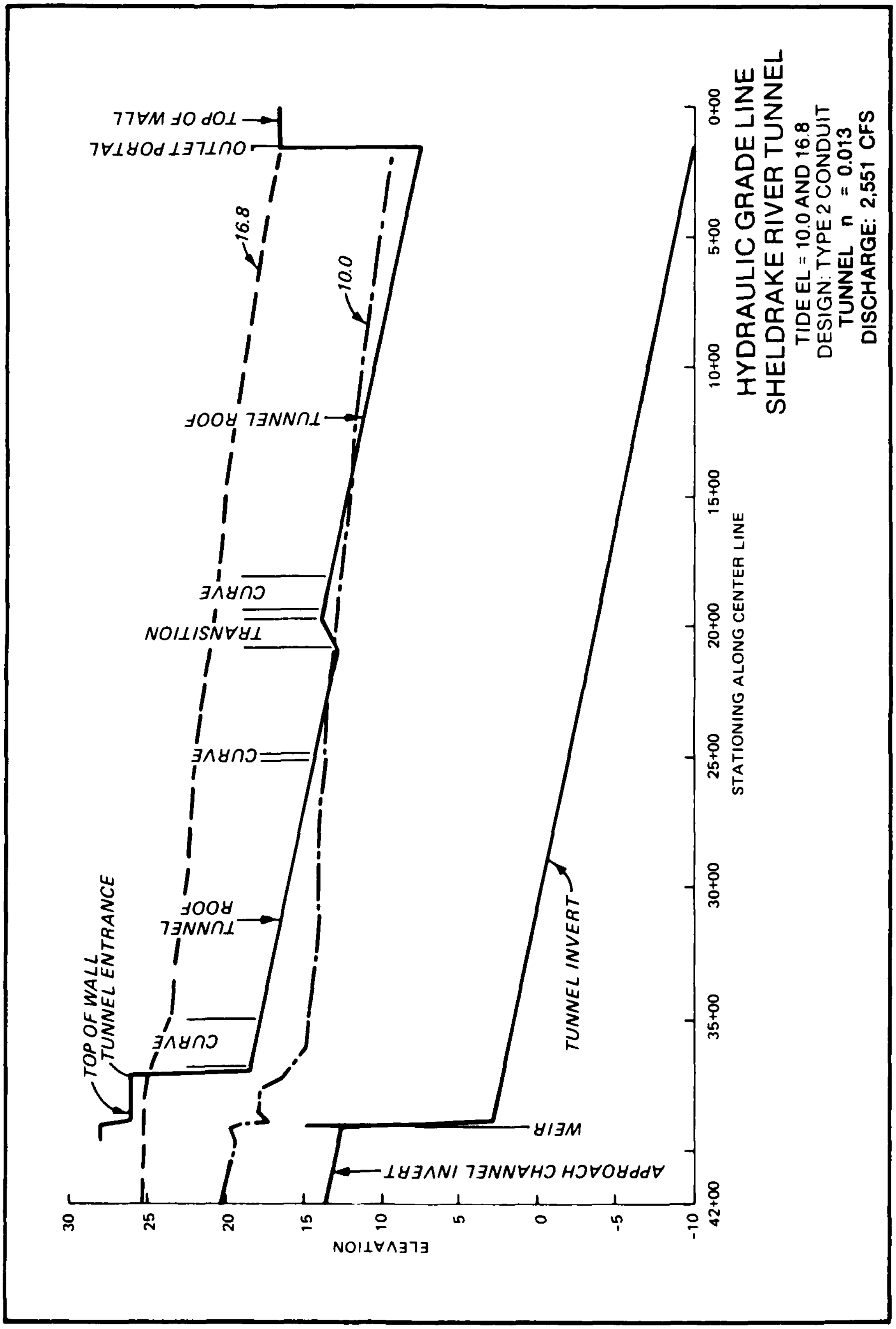
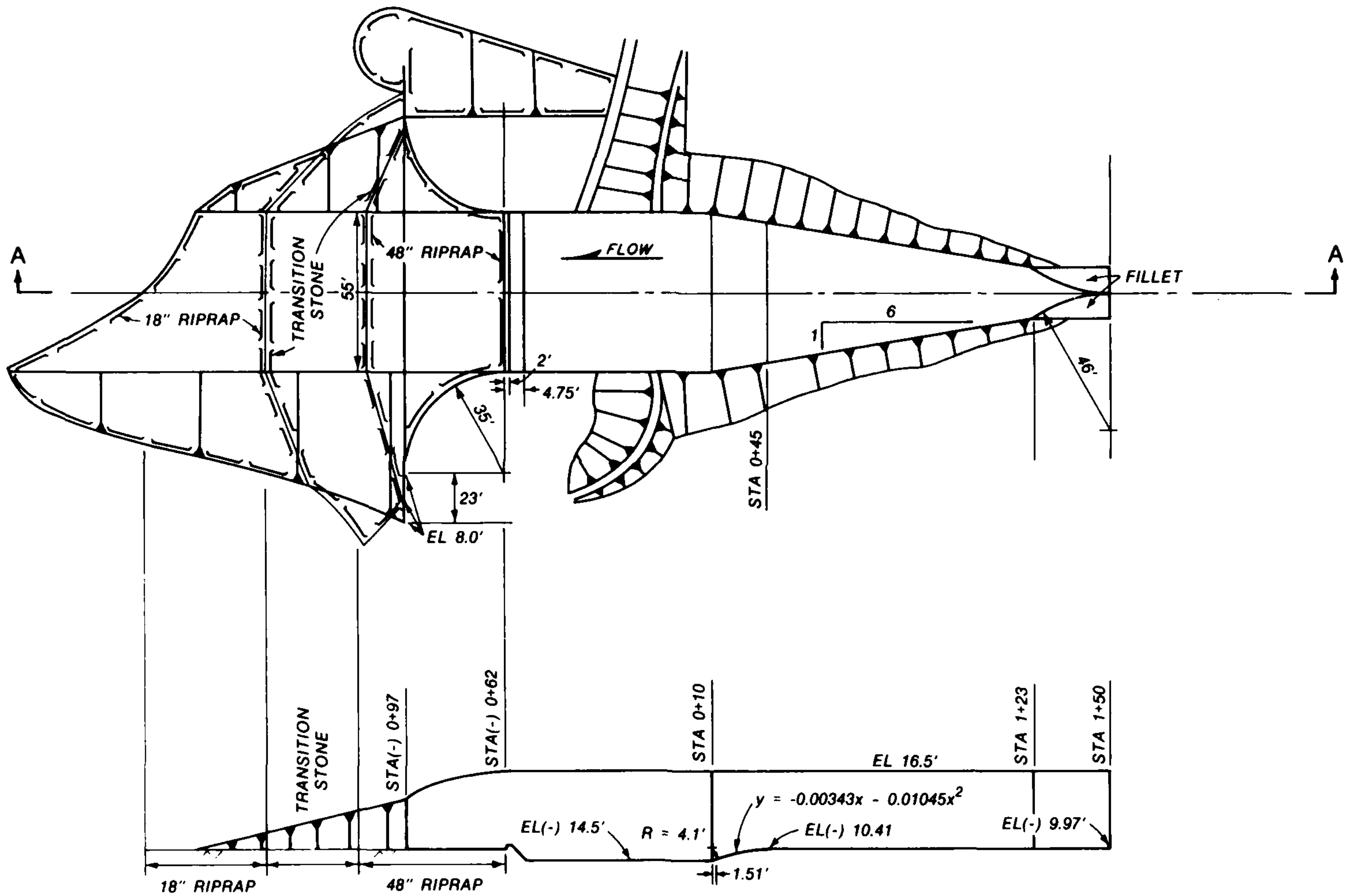

HYDRAULIC GRADE LINE
SHELDRAKE RIVER TUNNEL
TIDE EL = 10.0 AND 16.8
DESIGN: TYPE 2 CONDUIT
TUNNEL n = 0.013
DISCHARGE: 1,208 CFS
ELEVATION
STATIONING ALONG CENTER LINE
TOP OF WALL
OUTLET PORTAL
16.8
10.0
TUNNEL ROOF
CURVE
TRANSITION
CURVE
TUNNEL ROOF
TOP OF WALL
TUNNEL ENTRANCE
CURVE
TUNNEL INVERT
WEIR
APPROACH CHANNEL INVERT
30
25
20
15
10
5
0
-5
-10
42+00
35+00
30+00
25+00
20+00
15+00
10+00
5+00
0+00

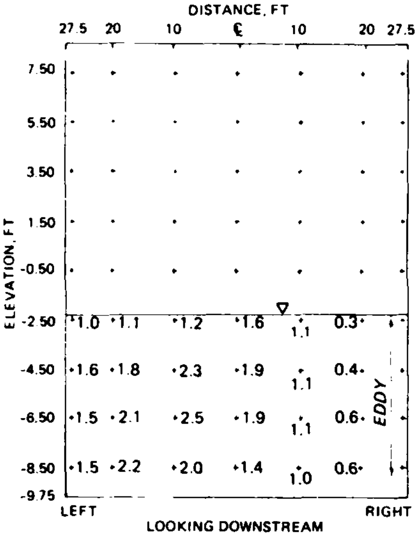
PLATE 22


HYDRAULIC GRADE LINE
SHELDRAKE RIVER TUNNEL
TIDE EL = 10.0 AND 16.8
DESIGN: TYPE 2 CONDUIT
TUNNEL n = 0.013
DISCHARGE: 2,551 CFS
ELEVATION
STATIONING ALONG CENTER LINE
30
25
20
15
10
5
0
-5
-10
42+00
35+00
30+00
25+00
20+00
15+00
10+00
5+00
0+00
TOP OF WALL
TUNNEL ENTRANCE
CURVE
TUNNEL ROOF
CURVE
TRANSITION
CURVE
TUNNEL ROOF
16.8
10.0
OUTLET PORTAL
TOP OF WALL
WEIR
APPROACH CHANNEL INVERT
TUNNEL INVERT

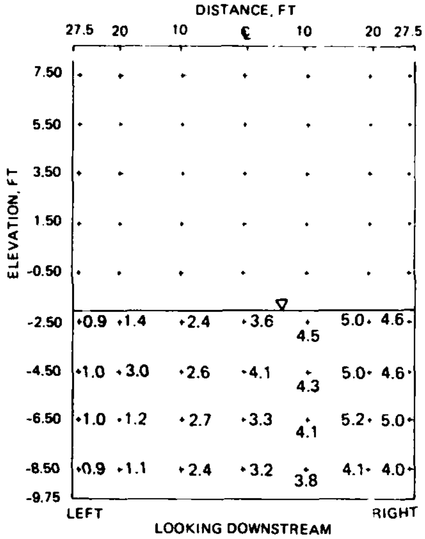


SECTION A-A

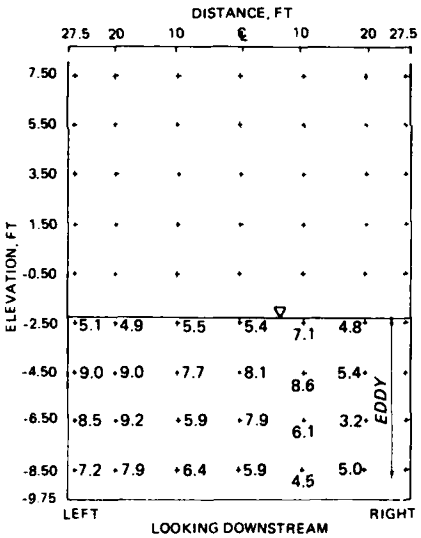
SHELDRAKE RIVER TUNNEL
ORIGINAL STILLING BASIN



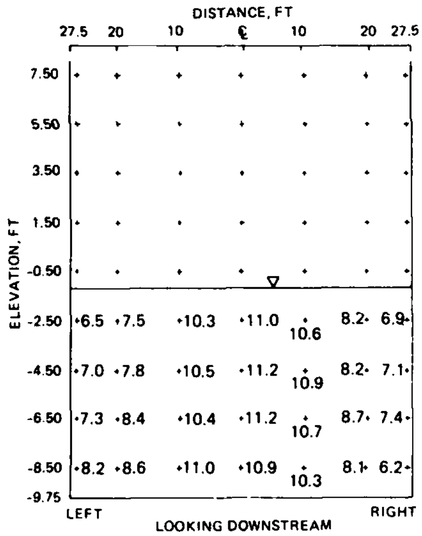
Q = 512 CFS



Q = 1,208 CFS



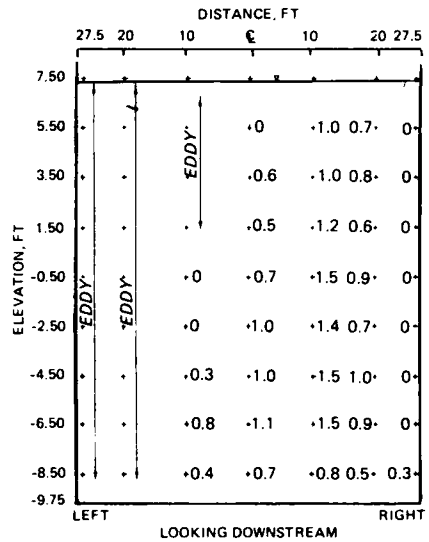
Q = 2,551 CFS



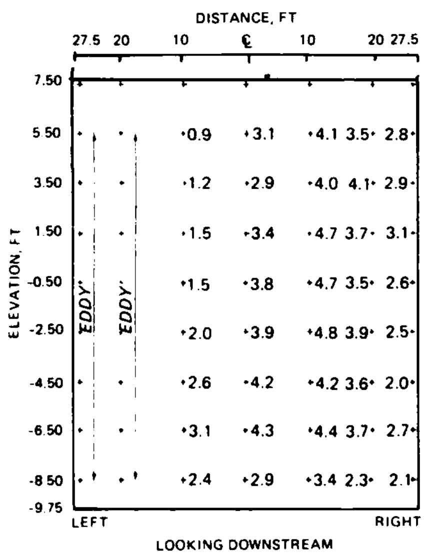
Q = 4,039 CFS

VELOCITY CROSS SECTION
SHELDRAKE RIVER TUNNEL STILLING BASIN

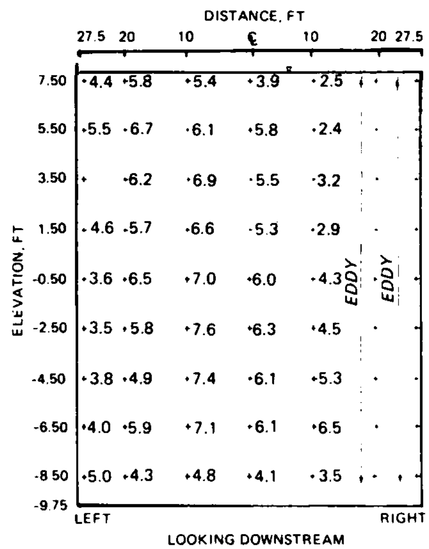
STA -00+62
TIDE EL = -2.7
DESIGN: TYPE 1 STILLING BASIN
TUNNEL n = 0.013



Q = 1,208 CFS



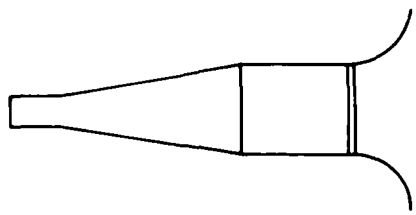
Q = 2,551 CFS



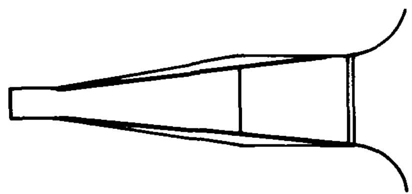
Q = 4,039 CFS

VELOCITY CROSS SECTION
SHELDRAKE RIVER TUNNEL STILLING BASIN

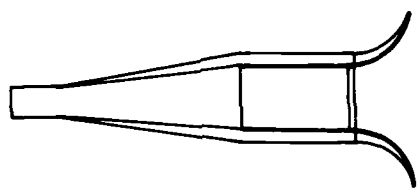
STA -00+62
TIDE EL = 6.7
DESIGN: TYPE 1 STILLING BASIN
TUNNEL n = 0.013



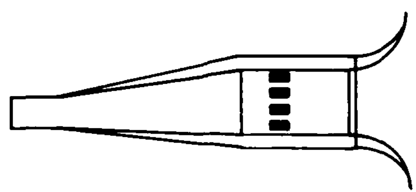
ORIGINAL STILLING BASIN DESIGN



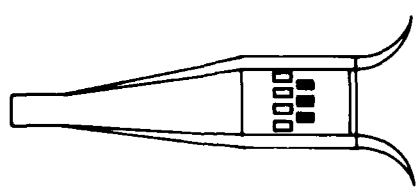
TYPE 2 STILLING BASIN



TYPE 3 STILLING BASIN

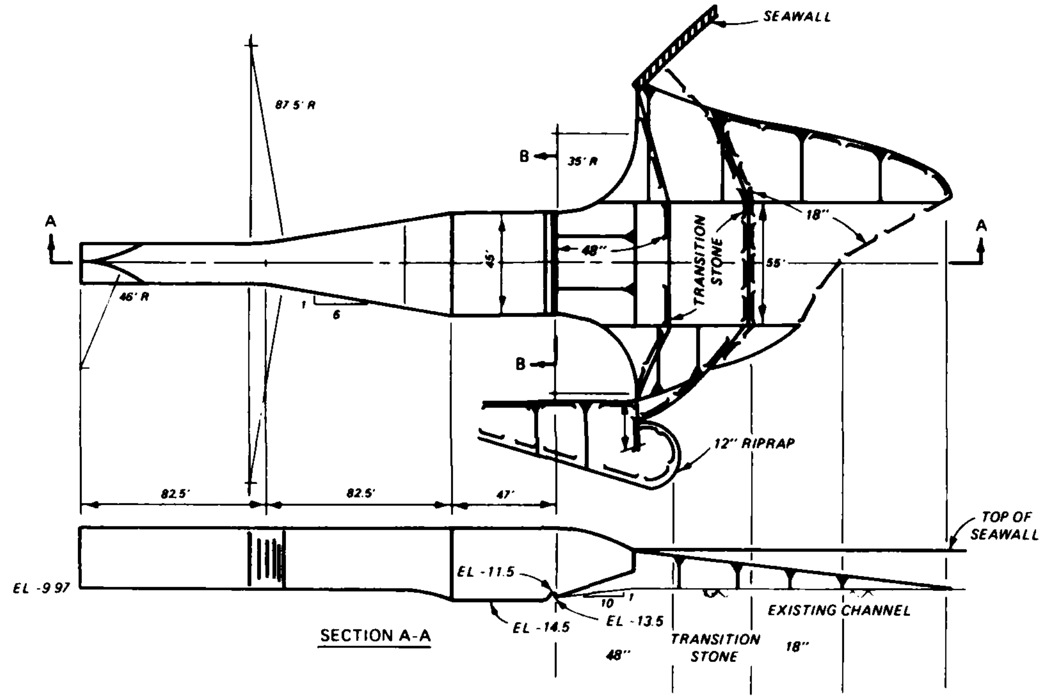


TYPE 4 STILLING BASIN



TYPE 5 STILLING BASIN

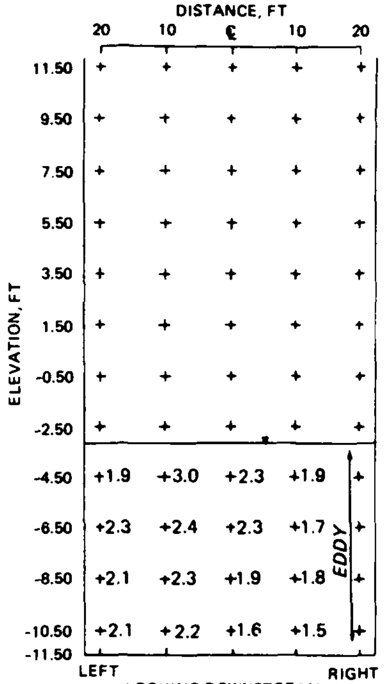
STILLING BASIN MODIFICATIONS
SHELDRAKE RIVER TUNNEL



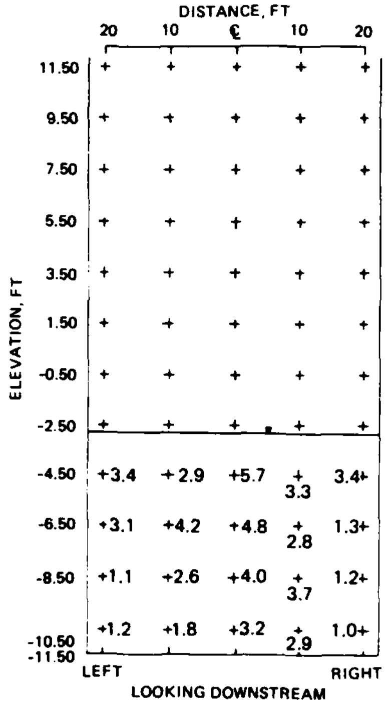
SPECIFIC WEIGHT = 165 LB/CU FT

% LIGHTER BY WEIGHT	LIMITS OF STONE WEIGHT, LB — THICKNESS 12 IN.		18 IN.		48 IN.	
100	26	10	86	35	1,638	655
50	8	5	26	17	485	328
15	4	2	13	5	243	102

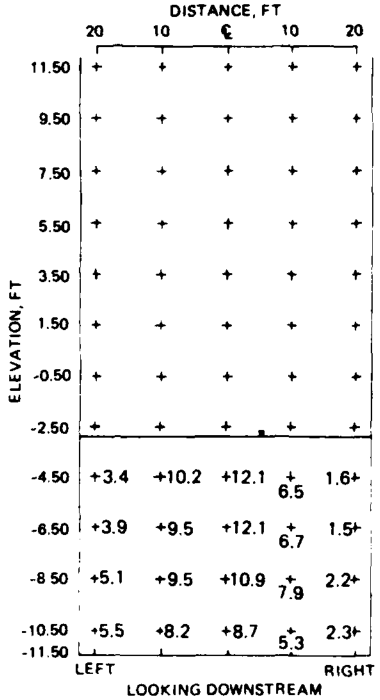
SHELDRAKE RIVER TUNNEL
STILLING BASIN
DESIGN: TYPE 6 STILLING BASIN



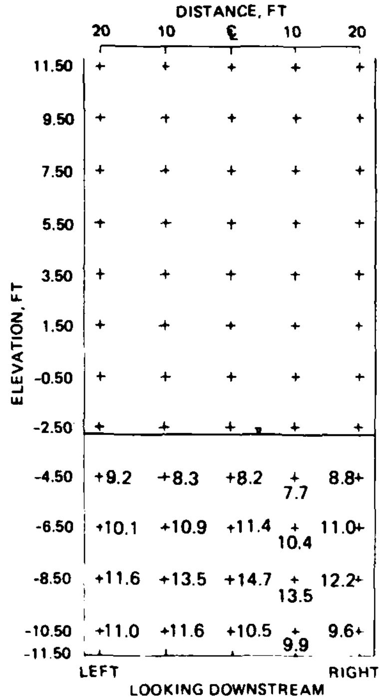
Q = 512 CFS



Q = 1,208 CFS



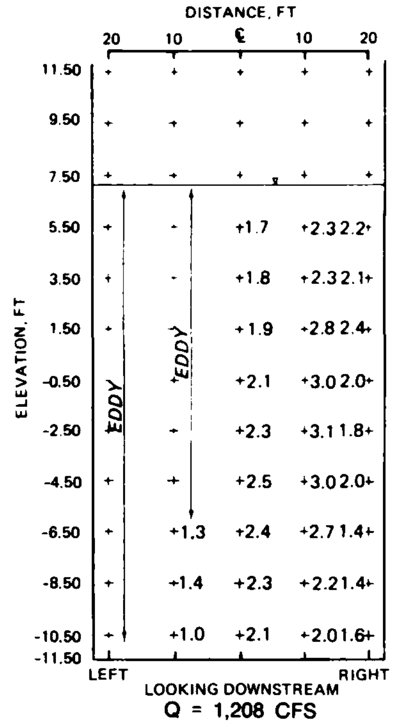
Q = 2,551 CFS

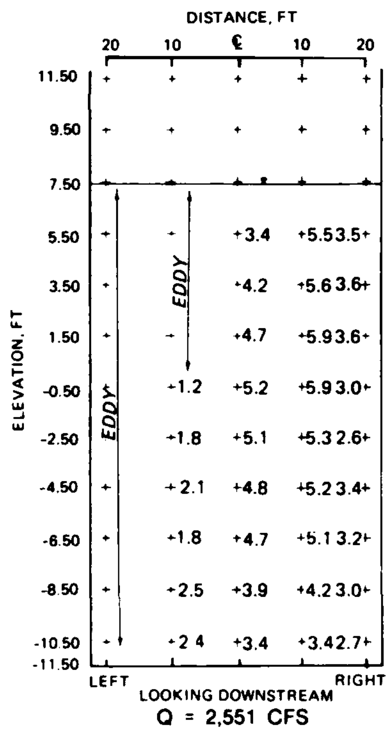


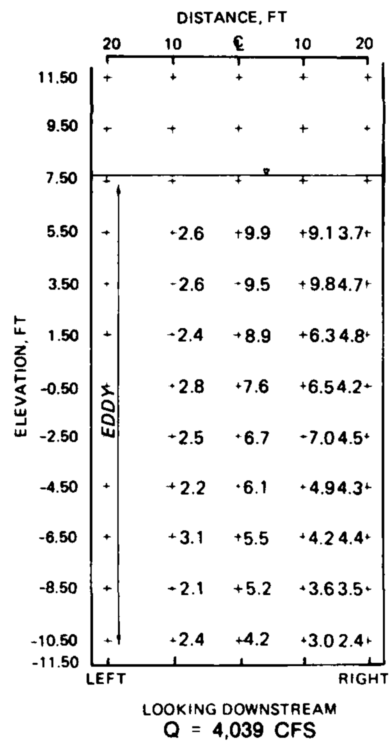
Q = 4,039 CFS

VELOCITY CROSS SECTION
SHELDRAKE RIVER TUNNEL STILLING BASIN

STA -00+62
TIDE EL = -2.7
DESIGN: TYPE 6 STILLING BASIN
TUNNEL n = 0.013

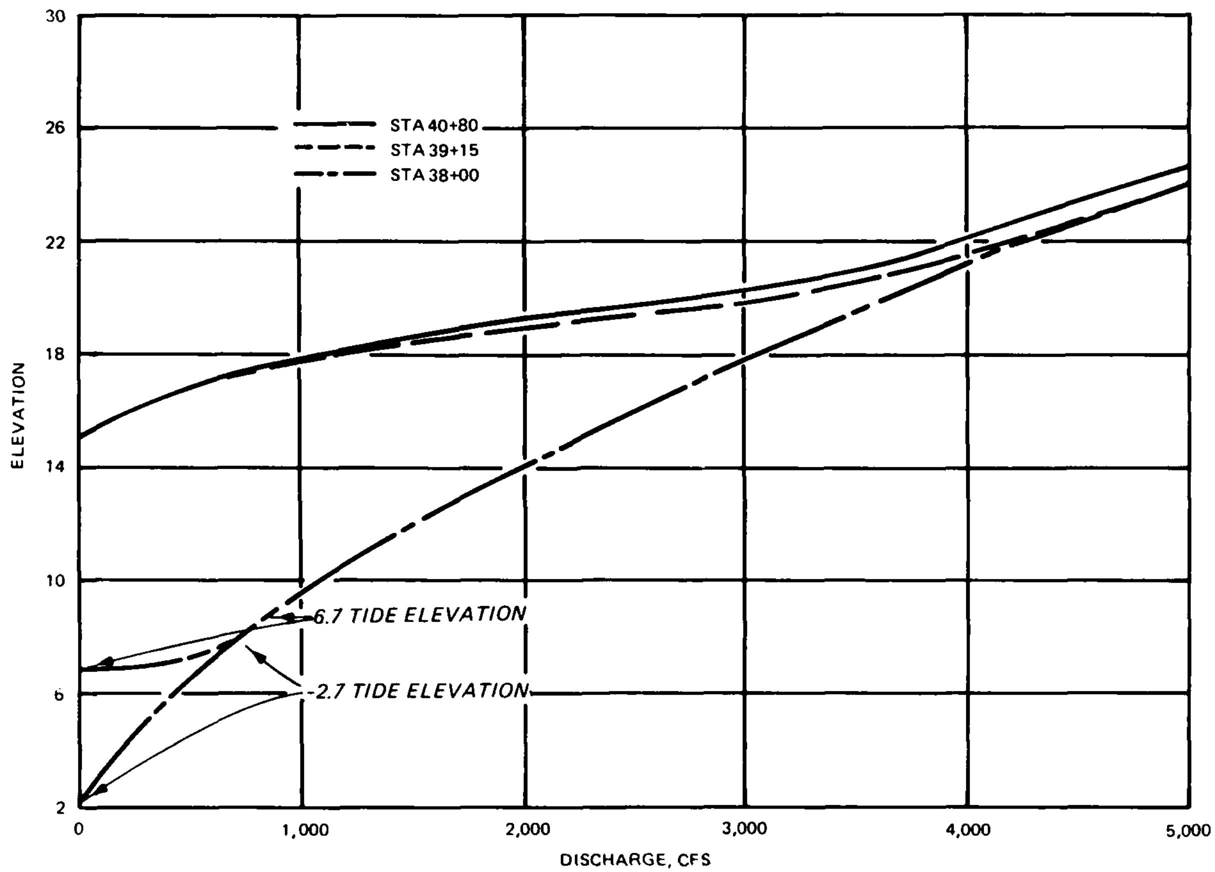






VELOCITY CROSS SECTION
SHELDRAKE RIVER TUNNEL STILLING BASIN

STA -00+62
TIDE EL = 6.7
DESIGN: TYPE 6 STILLING BASIN
TUNNEL n = 0.013



DISCHARGE RATING CURVES
SHELDRAKE RIVER TUNNEL
TIDE EL = 6.7 AND -2.7
DESIGN: TYPE 2 CONDUIT
MANNING'S n = 0.010

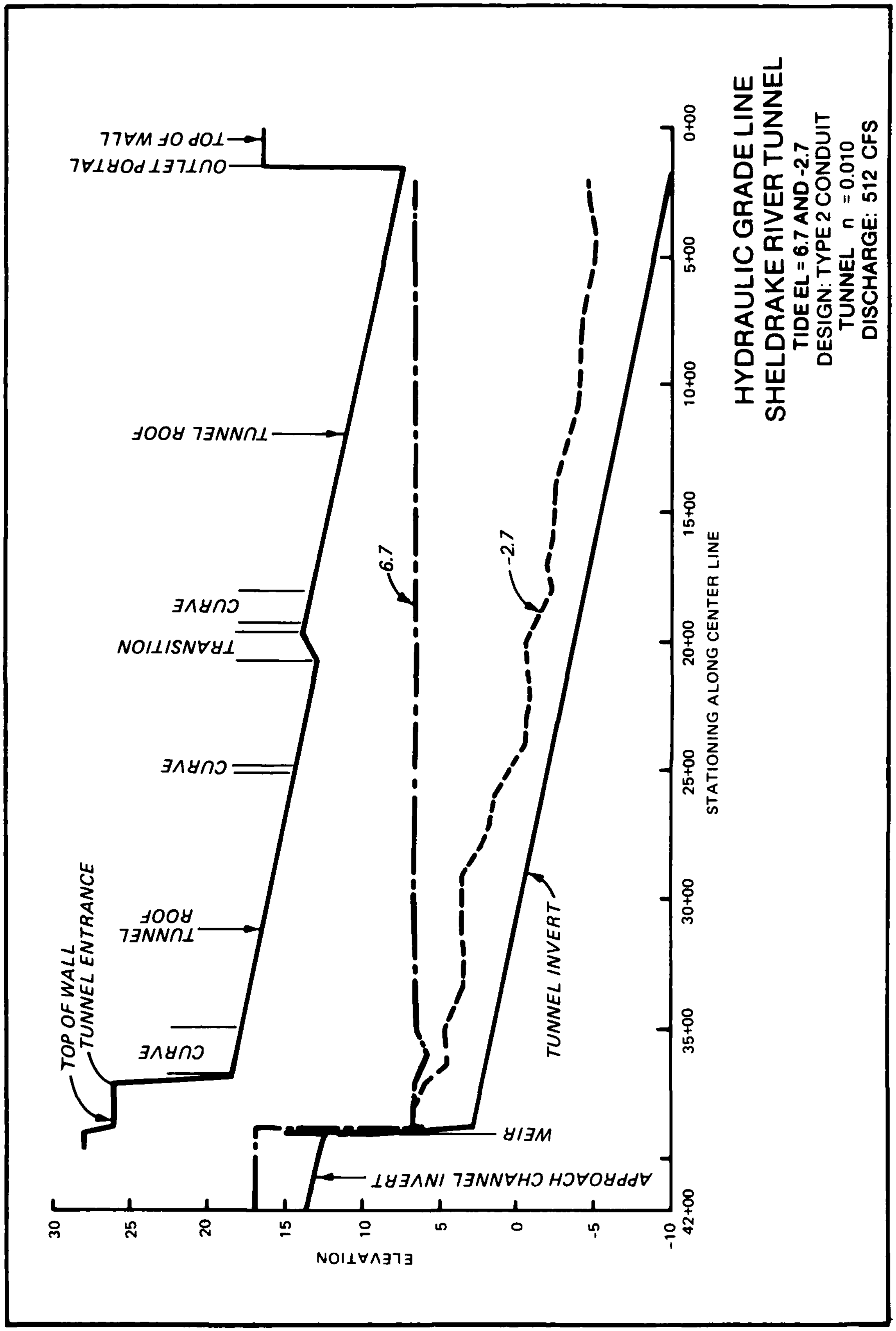
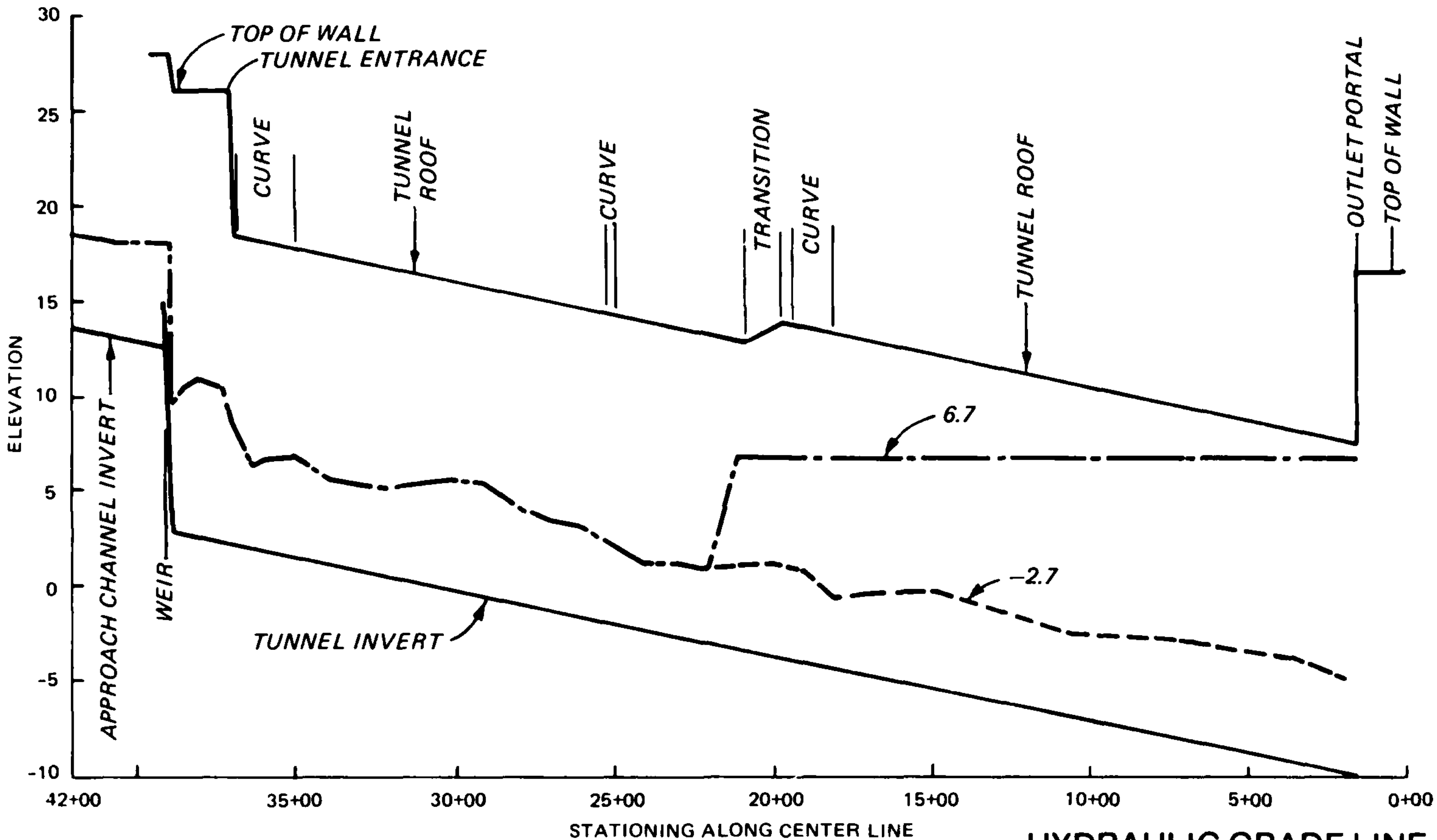
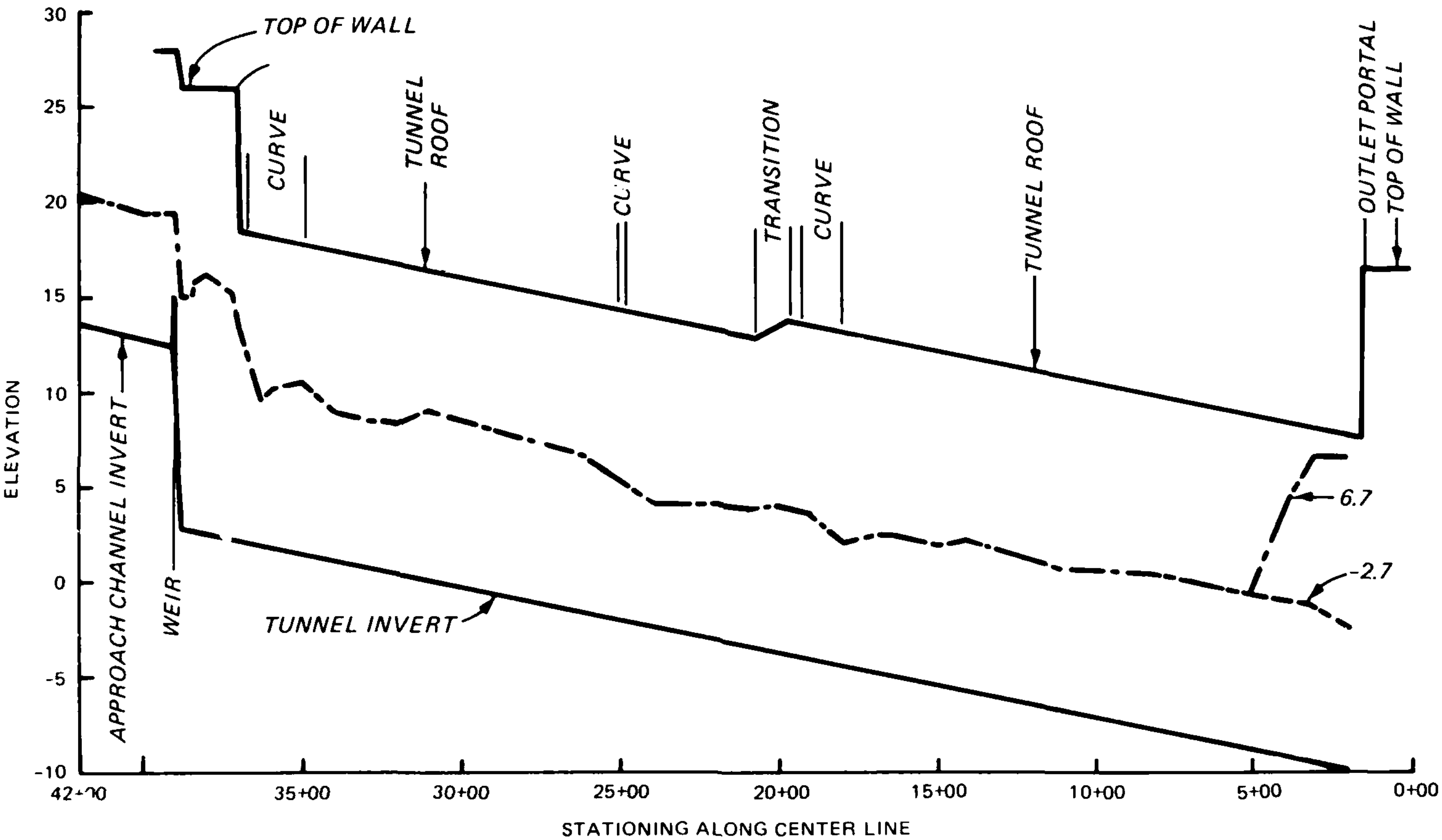

HYDRAULIC GRADE LINE
SHELDRAKE RIVER TUNNEL
TIDE EL = 6.7 AND -2.7
DESIGN: TYPE 2 CONDUIT
TUNNEL n = 0.010
DISCHARGE: 512 CFS
TOP OF WALL
OUTLET PORTAL
TUNNEL ROOF
CURVE
TRANSITION
CURVE
TUNNEL ROOF
TOP OF WALL
TUNNEL ENTRANCE
CURVE
WEIR
APPROACH CHANNEL INVERT
TUNNEL INVERT
6.7
-2.7
ELEVATION
STATIONING ALONG CENTER LINE
30
25
20
15
10
5
0
-5
-10
42+00
35+00
30+00
25+00
20+00
15+00
10+00
5+00
0+00

PLATE 32

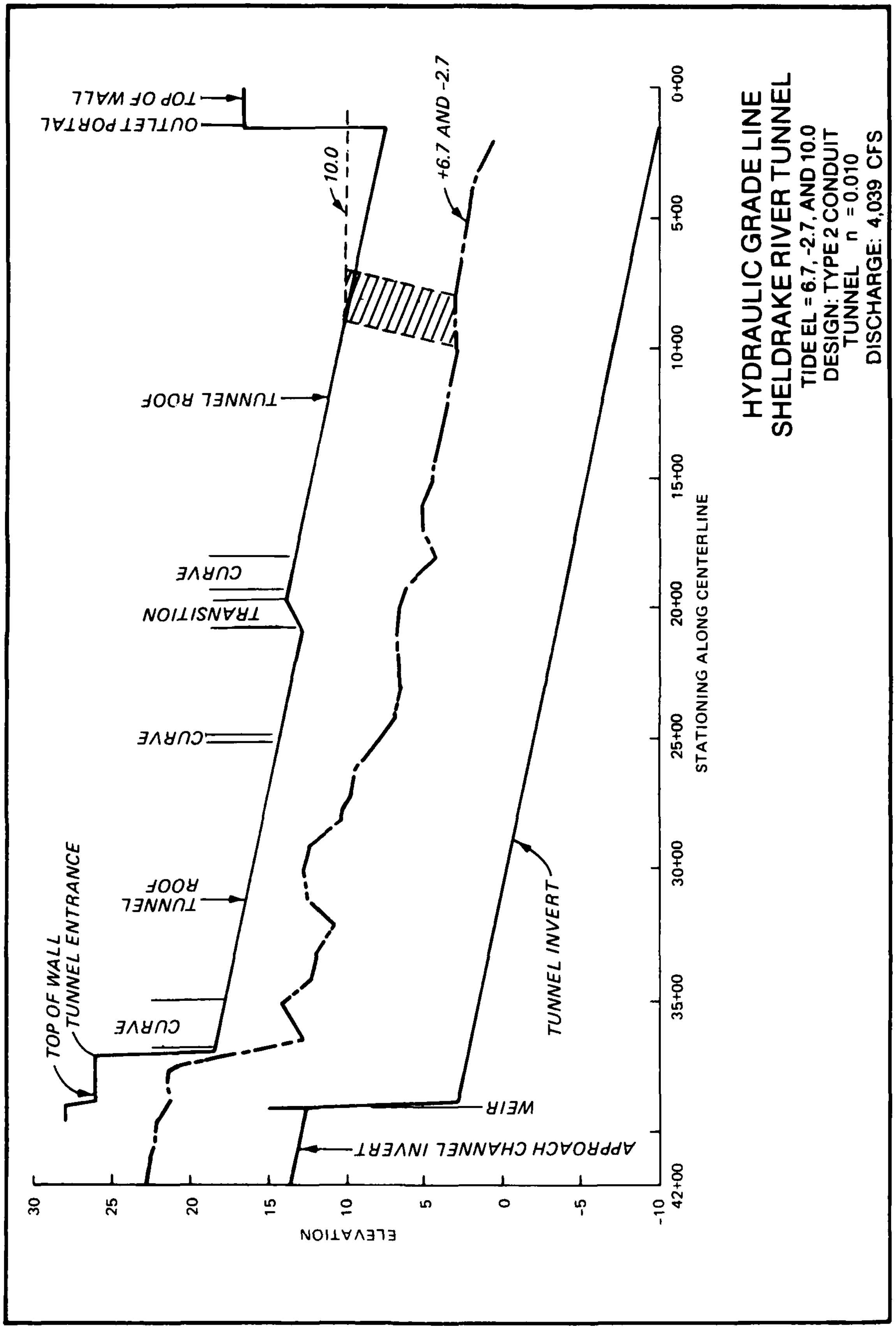


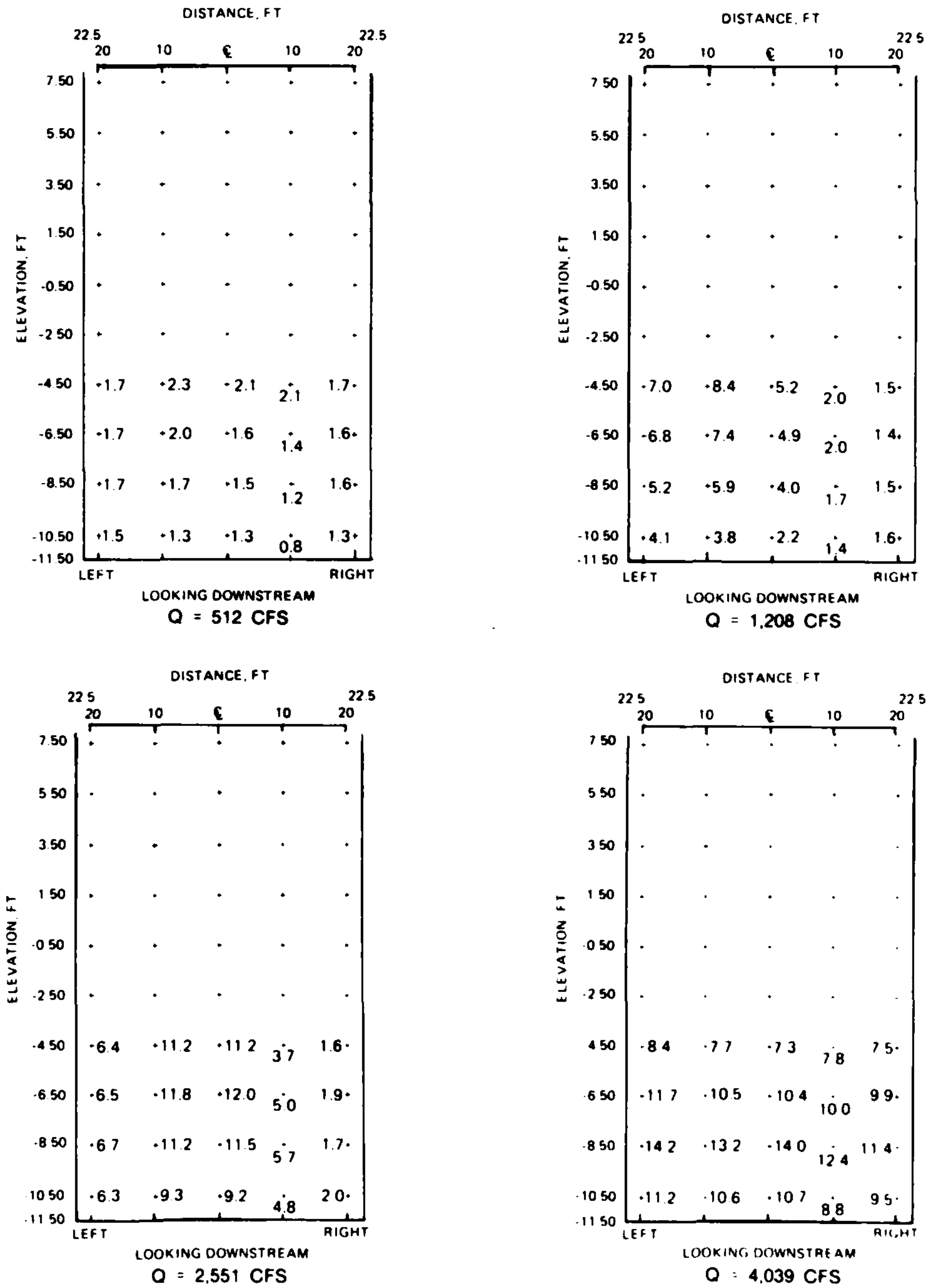
HYDRAULIC GRADE LINE
SHELDRAKE RIVER TUNNEL
TIDE EL = 6.7 AND -2.7
DESIGN: TYPE 2 CONDUIT
TUNNEL n = 0.010
DISCHARGE: 1,208 CFS



HYDRAULIC GRADE LINE
SHELDRAKE RIVER TUNNEL
TIDE EL = 6.7 AND -2.7
DESIGN: TYPE 2 CONDUIT
TUNNEL n = 0.010
DISCHARGE: 2,551 CFS

PLATE 34


HYDRAULIC GRADE LINE
SHELDRAKE RIVER TUNNEL
TIDE EL = 6.7, -2.7, AND 10.0
DESIGN: TYPE 2 CONDUIT
TUNNEL n = 0.010
DISCHARGE: 4,039 CFS
ELEVATION
30
25
20
15
10
5
0
-5
-10
42+00
35+00
30+00
25+00
20+00
15+00
10+00
5+00
0+00
STATIONING ALONG CENTERLINE
TOP OF WALL
TUNNEL ENTRANCE
CURVE
TUNNEL ROOF
CURVE
TRANSITION
CURVE
TUNNEL ROOF
OUTLET PORTAL
TOP OF WALL
10.0
+6.7 AND -2.7
TUNNEL INVERT
WEIR
APPROACH CHANNEL INVERT



VELOCITY CROSS SECTION
SHELDRAKE RIVER TUNNEL STILLING BASIN

STA -00+62
TIDE EL = -2.7
DESIGN TYPE 6 STILLING BASIN
TUNNEL n = 0.010

DISTANCE, FT — LOOKING DOWNSTREAM

ELEVATION, FT	20 (LEFT)	10	℄	10	20 (RIGHT)
7.50					
5.50	1.1	1.4	1.2	0	
3.50	0.9	1.4	1.2	0	
1.50	1.1	1.6	1.3	0.4	
-0.50	1.2	1.7	1.6	0.5	
-2.50	1.1	1.7	1.5	0.6	EDDY
-4.50	1.1	1.7	1.5	0	
-6.50	1.2	1.5	1.4	0	
-8.50	0.9	1.4	1.2	0	0.4
-10.50	0.9	1.1	0	0	0.4
-11.50					

Q = 512 CFS

DISTANCE, FT — LOOKING DOWNSTREAM

ELEVATION, FT	20 (LEFT)	10	℄	10	20 (RIGHT)
7.50					
5.50	1.7	2.4	1.8	0.6	0.6
3.50	1.9	2.6	2.0	0.7	0.5
1.50	2.2	2.9	2.1	0.6	0.6
-0.50	1.9	3.0	2.3	0.7	0.6
-2.50	1.9	3.1	2.7	1.4	0.5
-4.50	1.9	3.0	2.7	1.5	0.7
-6.50	1.8	2.8	2.9	1.6	0.7
-8.50	1.7	2.2	2.1	1.4	0.8
-10.50	1.5	2.0	1.7	1.2	0.9
-11.50					

Q = 1,208 CFS

DISTANCE, FT — LOOKING DOWNSTREAM

ELEVATION, FT	20 (LEFT)	10	℄	10	20 (RIGHT)
7.50					
5.60	0.9	1.3	3.5	4.6	2.7
3.50	0.9	1.3	4.2	5.5	2.9
1.50	0.9	1.5	4.9	5.9	3.1
-0.50	1.0	1.9	5.3	5.8	2.9
-2.50	1.1	2.8	4.8	5.6	3.3
-4.50	1.7	2.6	5.0	5.1	3.8
-6.50	1.3	2.6	4.6	4.3	3.1
-8.50	1.3	3.1	4.3	4.4	2.9
-10.50	2.0	2.5	3.2	3.7	2.3
-11.50					

Q = 2,551 CFS

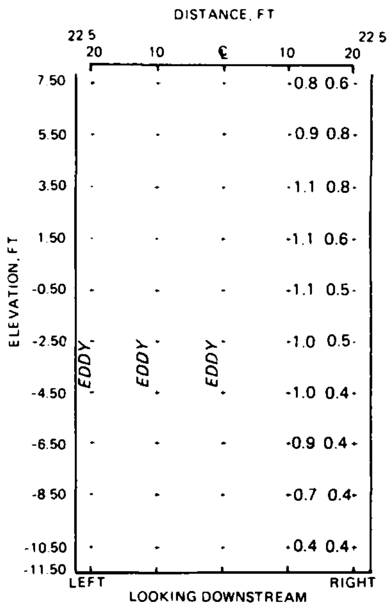
DISTANCE, FT — LOOKING DOWNSTREAM

ELEVATION, FT	20 (LEFT)	10	℄	10	20 (RIGHT)
7.50					
5.50	4.2	10.9	11.3	6.4	1.7
3.50	6.3	10.8	10.2	5.3	2.4
1.50	5.6	10.2	9.9	4.1	3.3
-0.50	3.8	8.4	8.1	4.9	2.6
-2.50	4.2	15.2	7.7	4.1	1.8
-4.50	2.6	14.5	6.0	2.9	1.9
-6.50	2.3	14.0	5.0	3.1	1.7
-8.50	2.2	12.2	3.3	2.3	1.4
-10.50	1.6	11.2	3.2	2.2	1.4
-11.50					

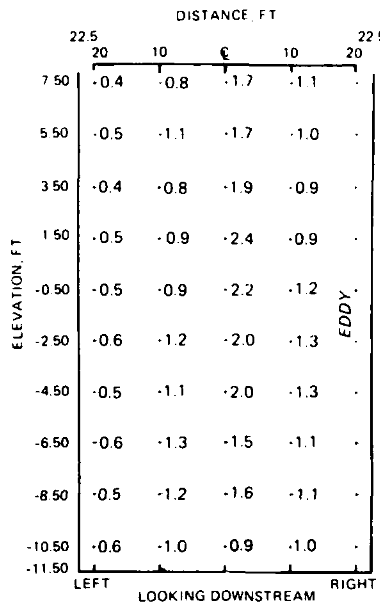
Q = 4,039 CFS

VELOCITY CROSS SECTION
SHELDRAKE RIVER TUNNEL STILLING BASIN

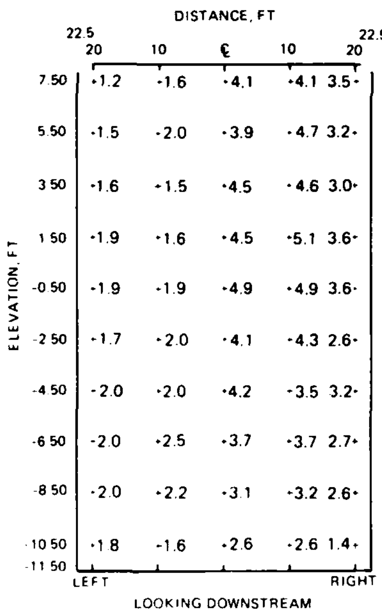
STA -00+62
TIDE EL = 6.7
DESIGN TYPE 6 STILLING BASIN
TUNNEL n = 0.010



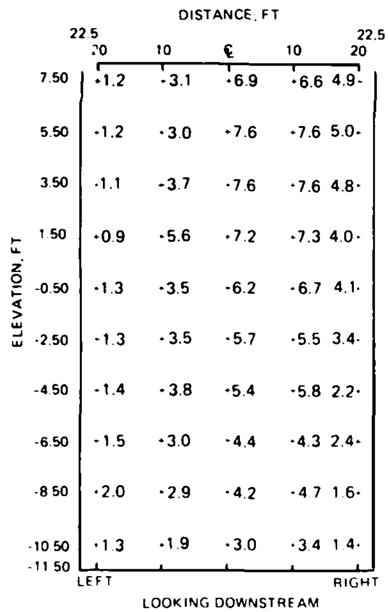
Q = 512 CFS



Q = 1,208 CFS



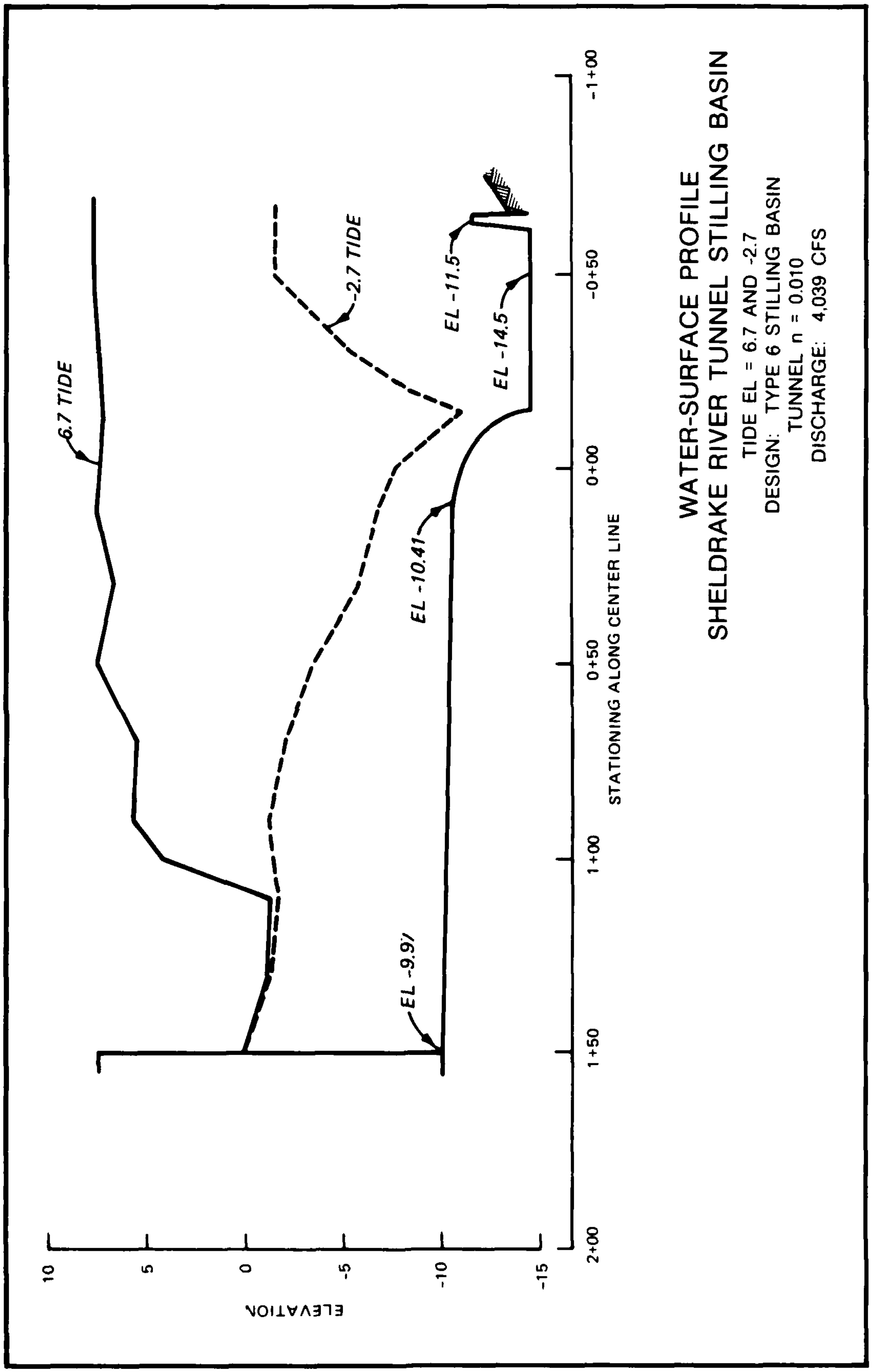
Q = 2,551 CFS



Q = 4,039 CFS

VELOCITY CROSS SECTION
SHELDRAKE RIVER TUNNEL STILLING BASIN

STA -00+62
TIDE EL = 10.0
DESIGN TYPE 6 STILLING BASIN
TUNNEL n = 0.010



WATER-SURFACE PROFILE
SHELDRAKE RIVER TUNNEL STILLING BASIN
TIDE EL = 6.7 AND -2.7
DESIGN: TYPE 6 STILLING BASIN
TUNNEL n = 0.010
DISCHARGE: 4,039 CFS

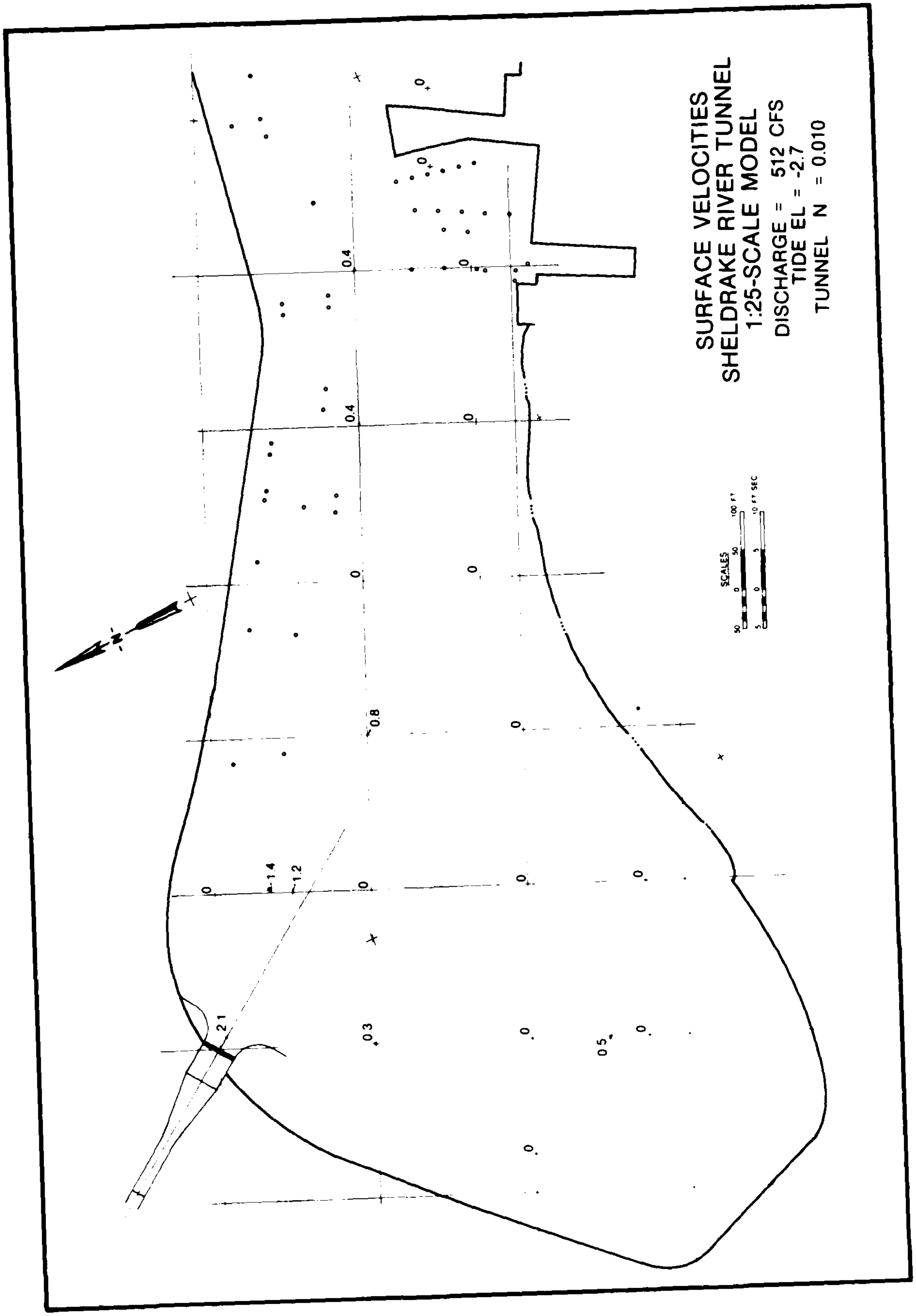
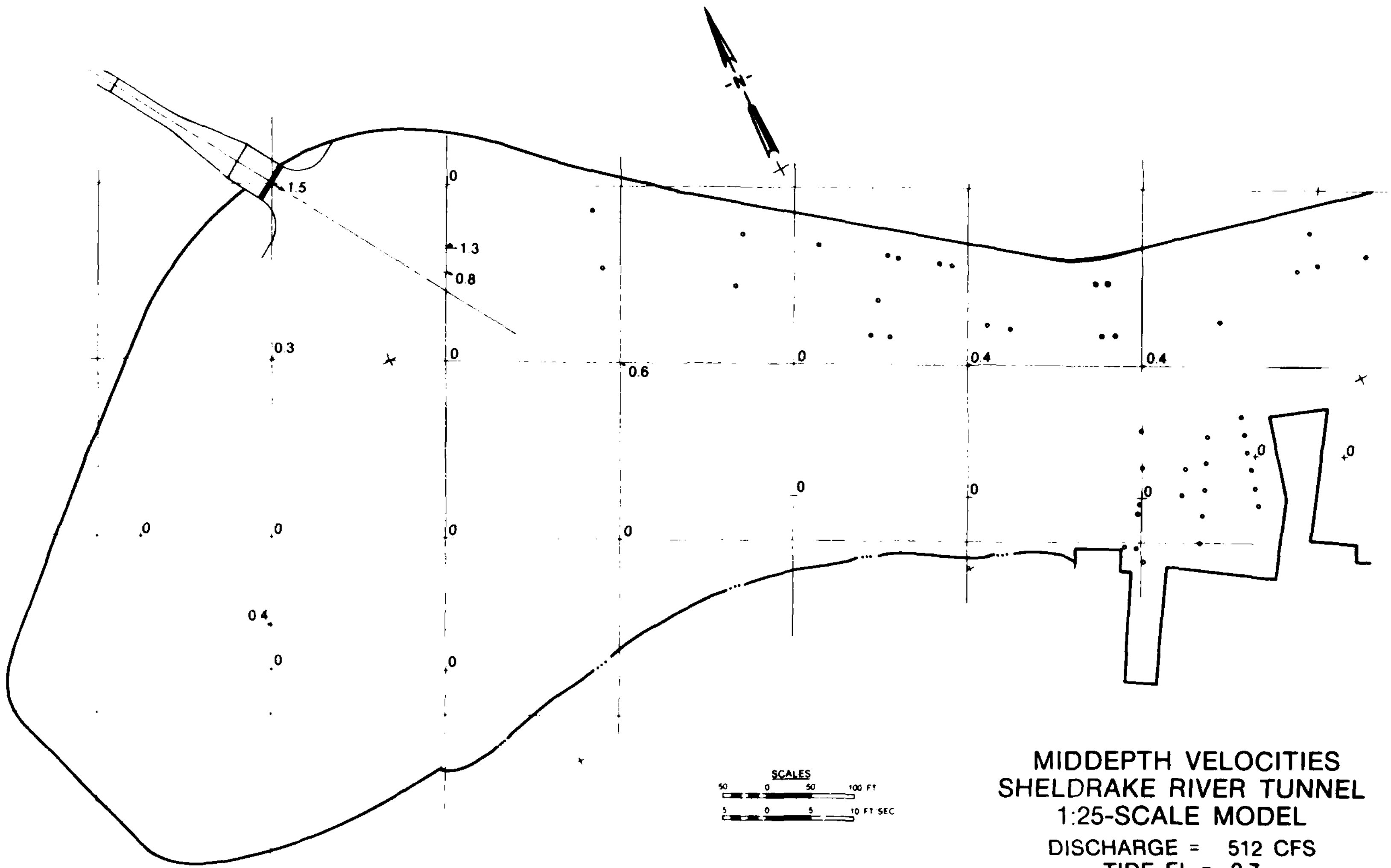
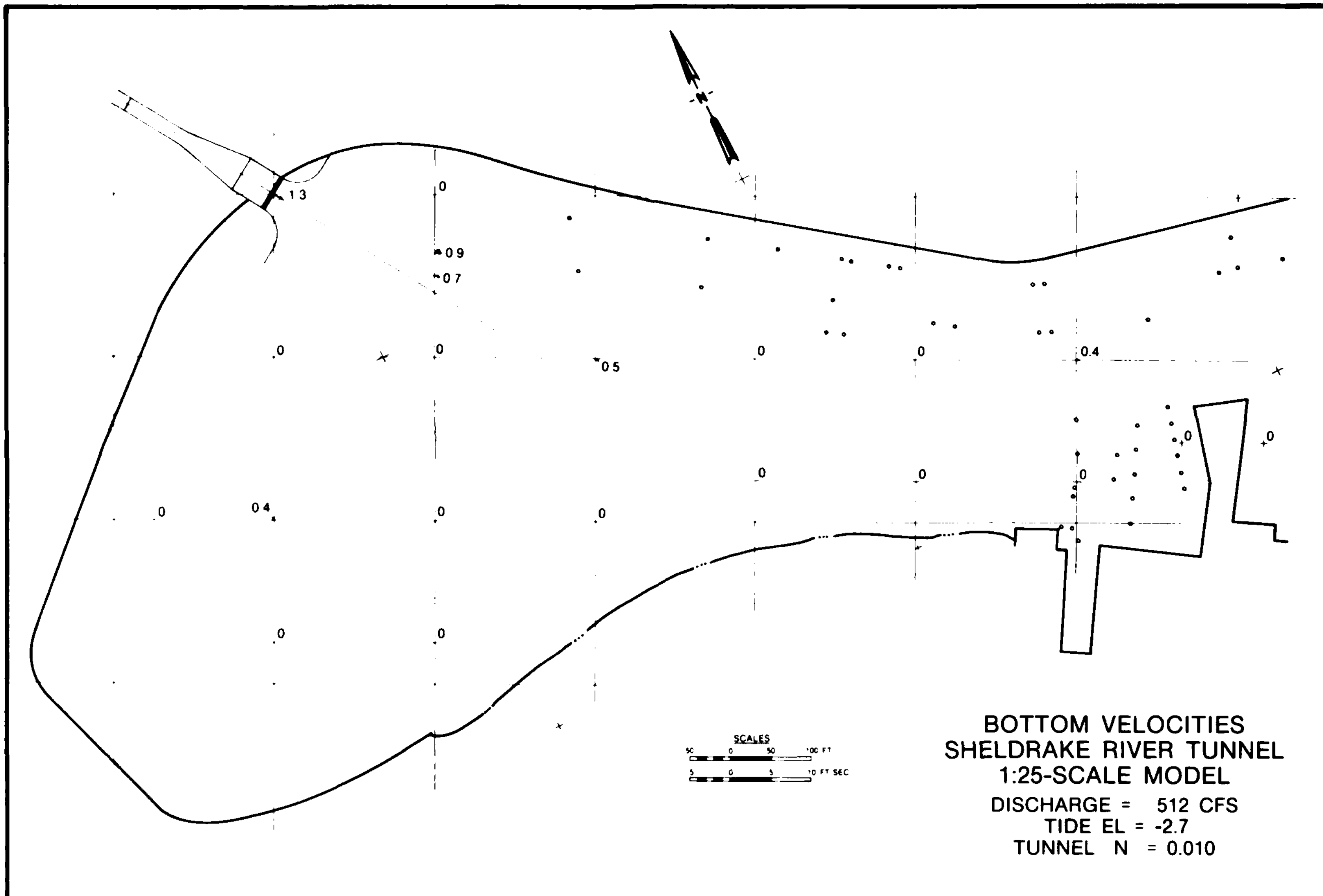

SURFACE VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 512 CFS
TIDE EL = -2.7
TUNNEL N = 0.010
SCALES

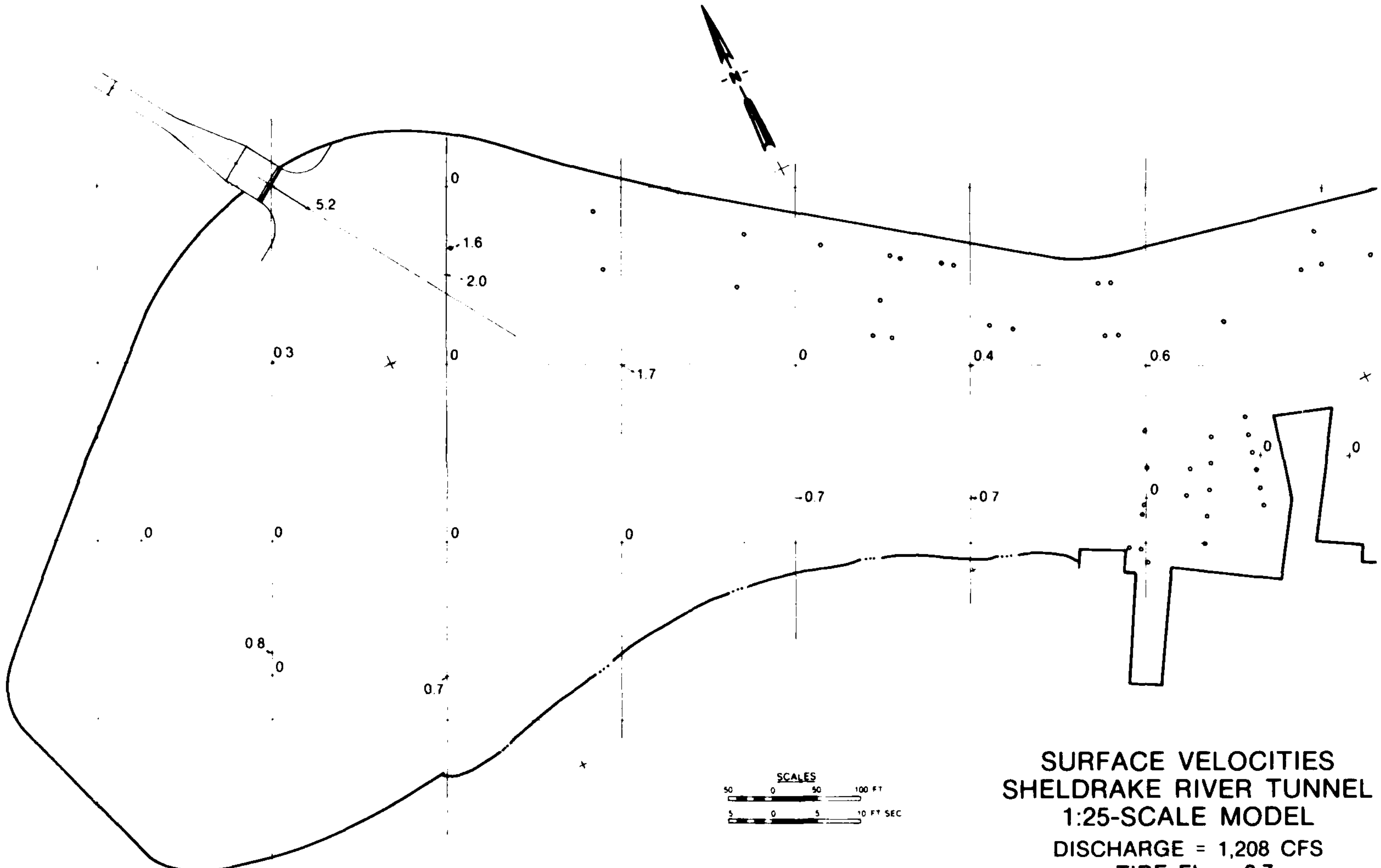
PLATE 40



MIDDEPTH VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 512 CFS
TIDE EL = -2.7
TUNNEL N = 0.010



BOTTOM VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 512 CFS
TIDE EL = -2.7
TUNNEL N = 0.010



SURFACE VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL

DISCHARGE = 1,208 CFS
TIDE EL = -2.7
TUNNEL N = 0.010

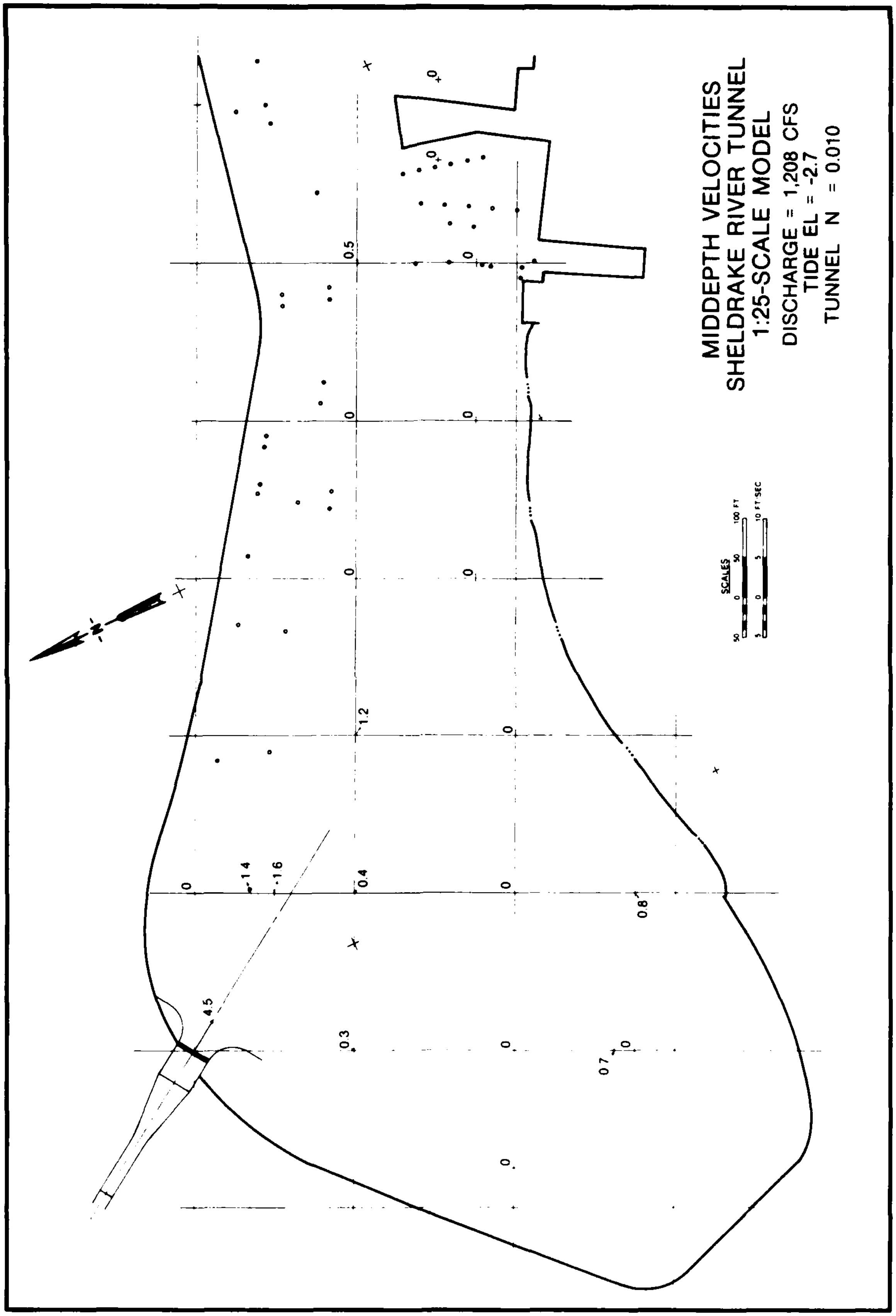
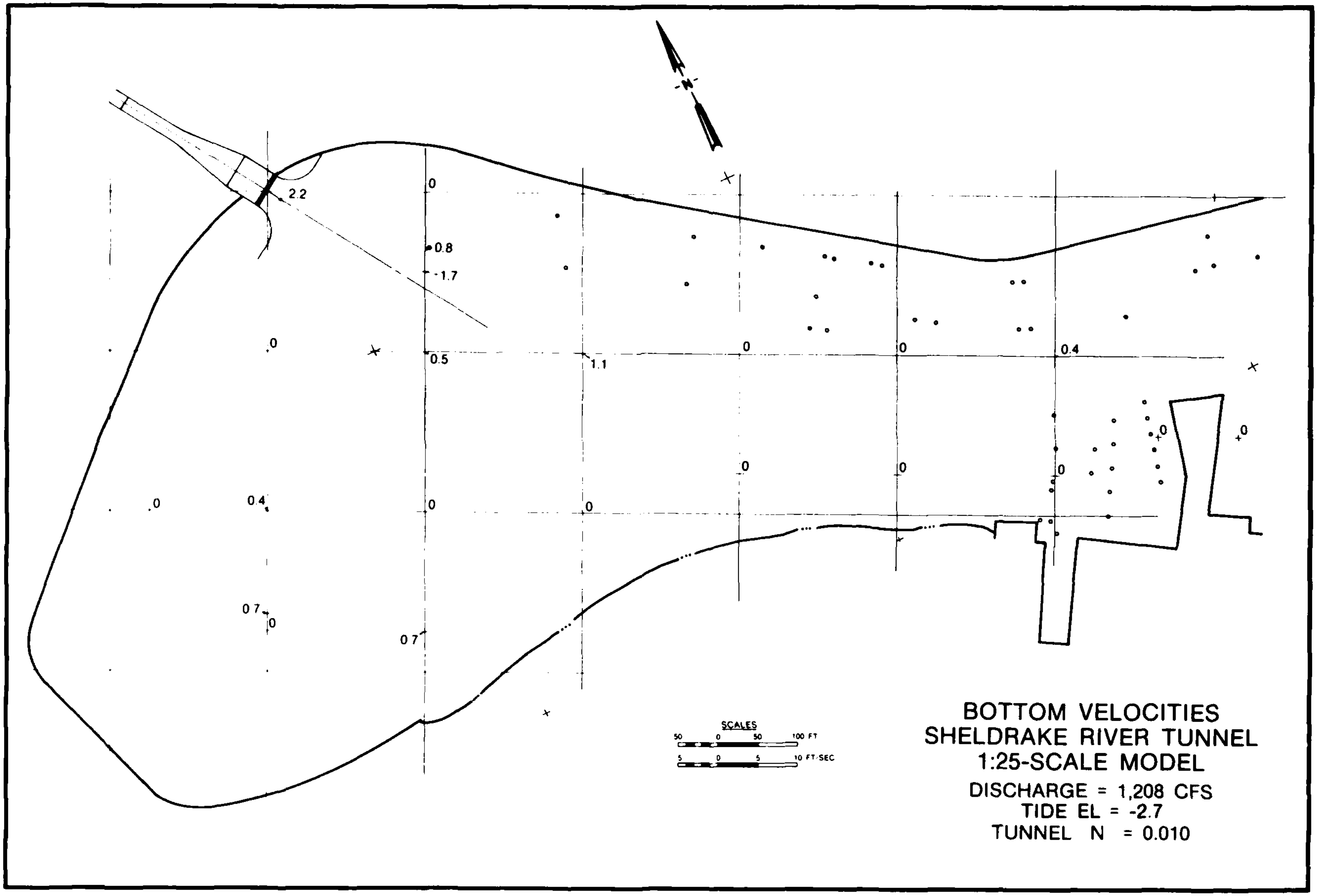
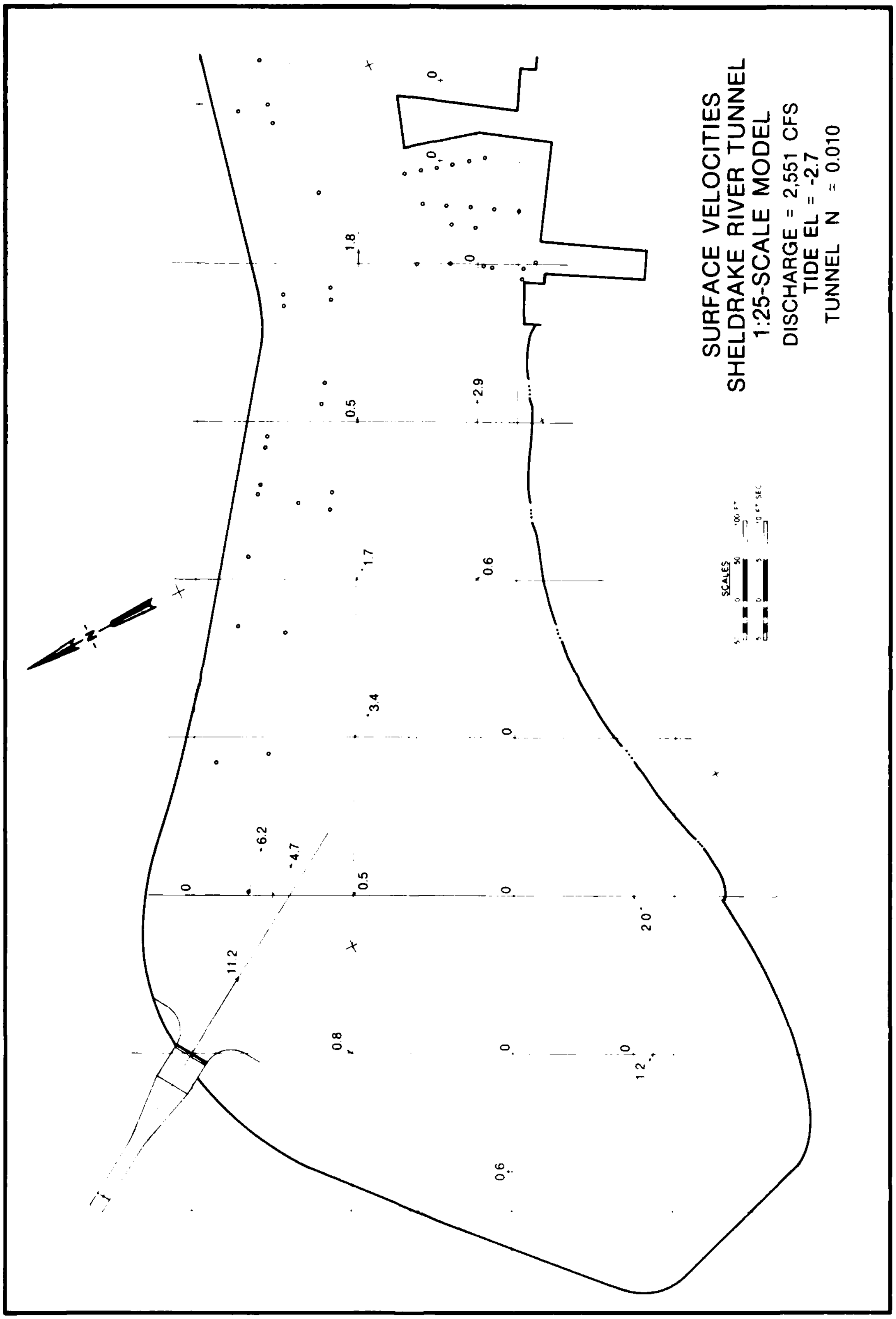

MIDDEPTH VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 1,208 CFS
TIDE EL = -2.7
TUNNEL N = 0.010
SCALES
100 FT
10 FT/SEC

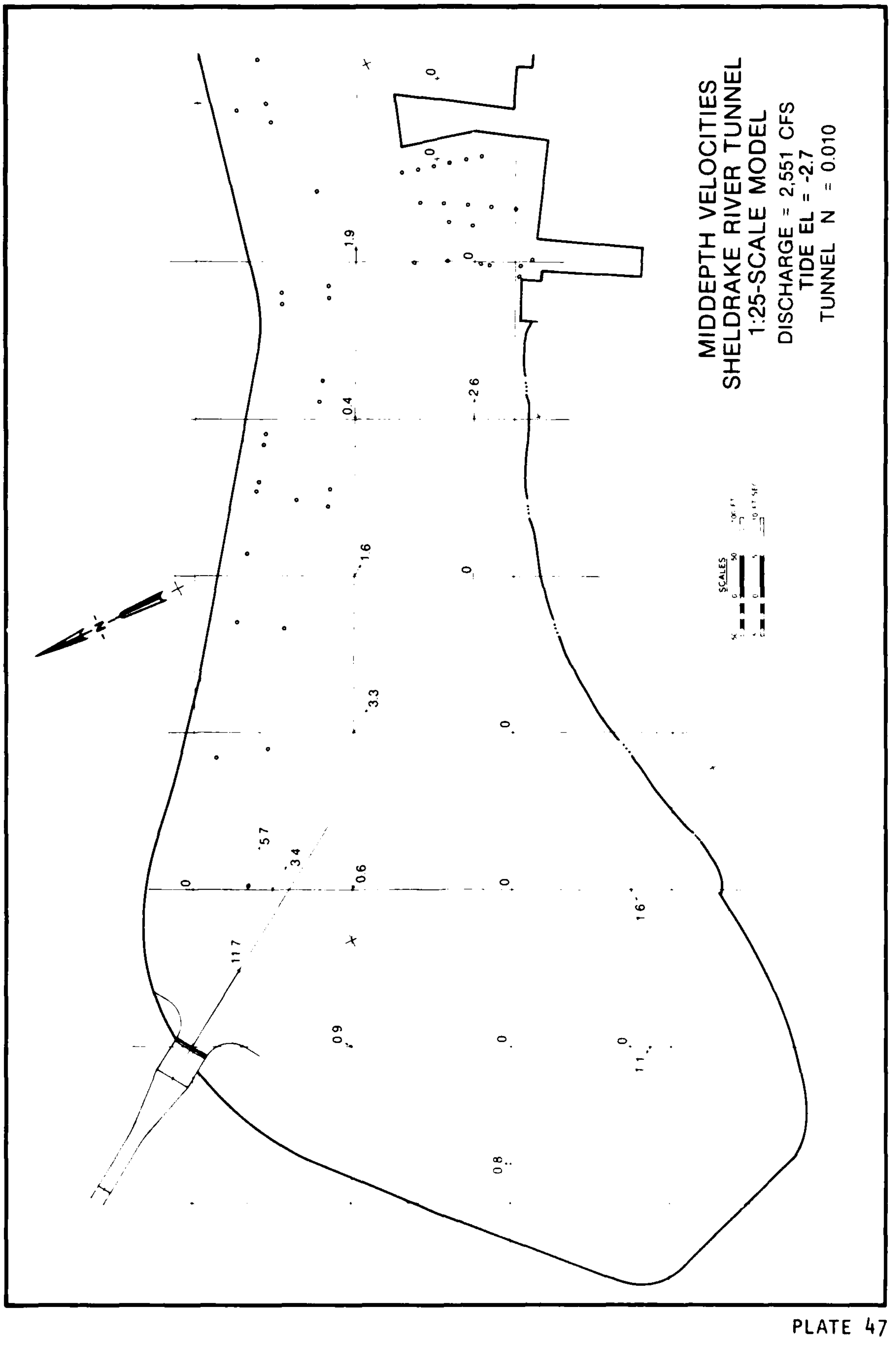
PLATE 44



N
2.2
0
0.8
-1.7
0.5
1.1
0.4
0.7
SCALES
50 0 50 100 FT
5 0 5 10 FT/SEC
BOTTOM VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 1,208 CFS
TIDE EL = -2.7
TUNNEL N = 0.010



SURFACE VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 2,551 CFS
TIDE EL = -2.7
TUNNEL N = 0.010



MIDDEPTH VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 2,551 CFS
TIDE EL = -2.7
TUNNEL N = 0.010

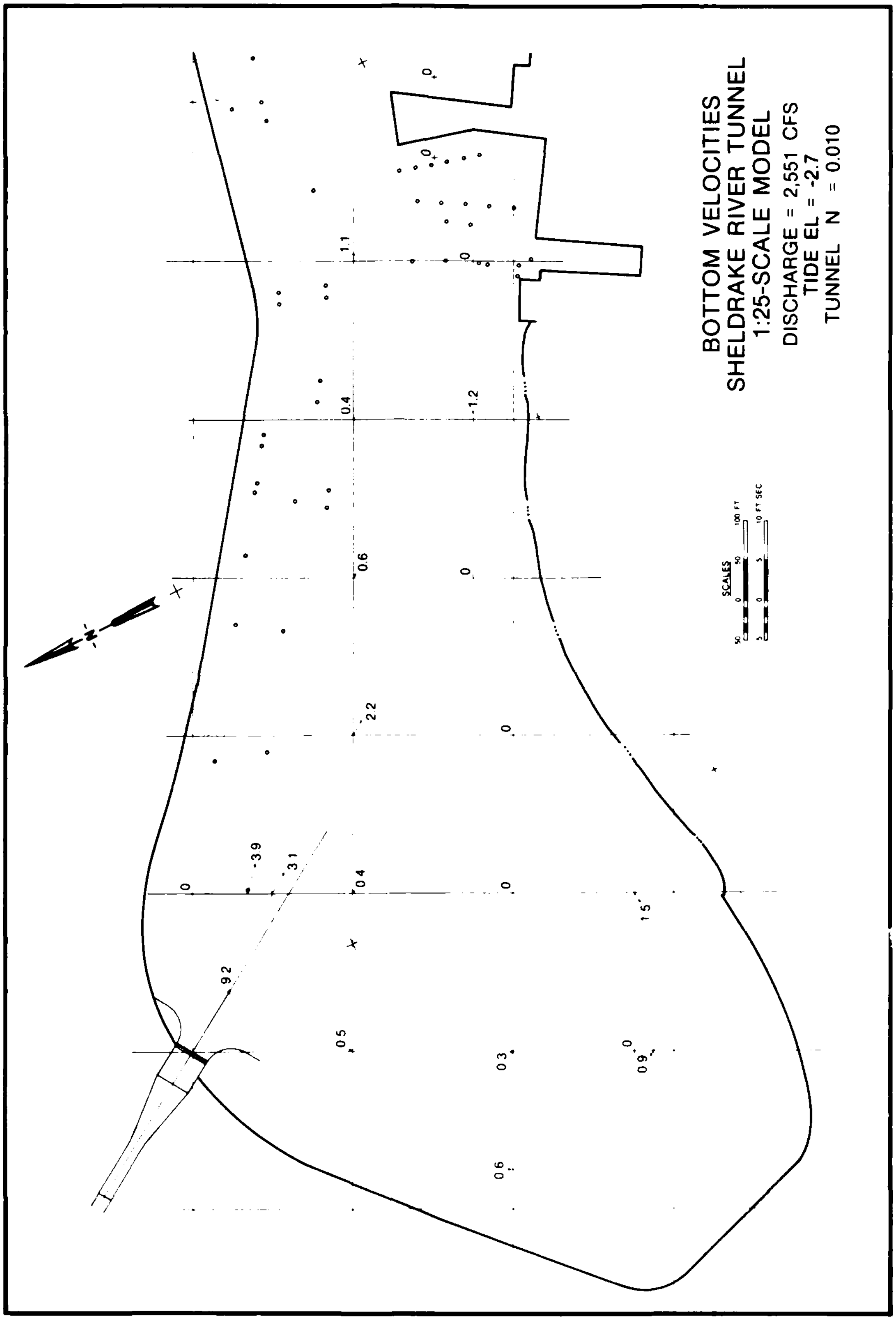
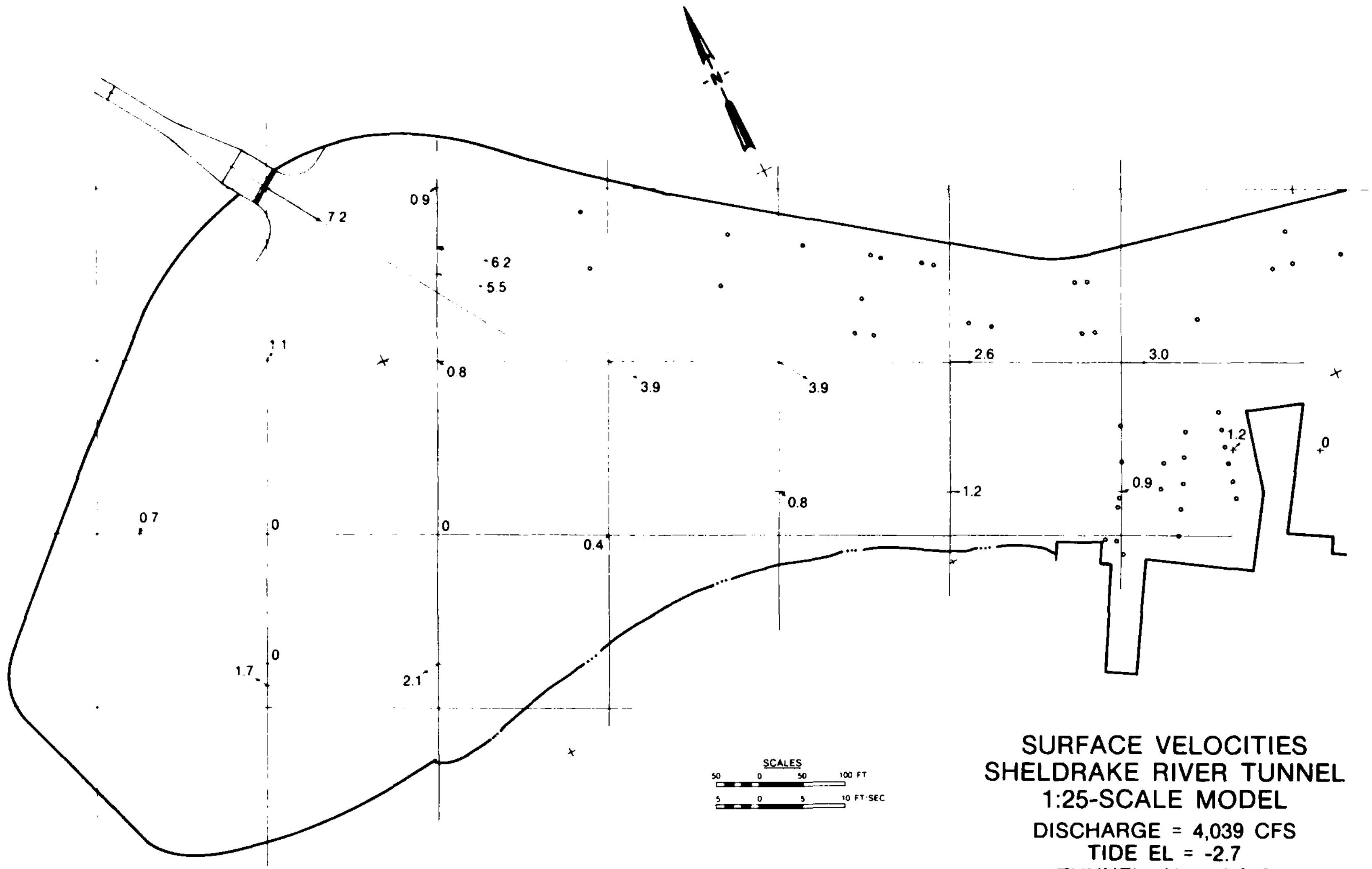
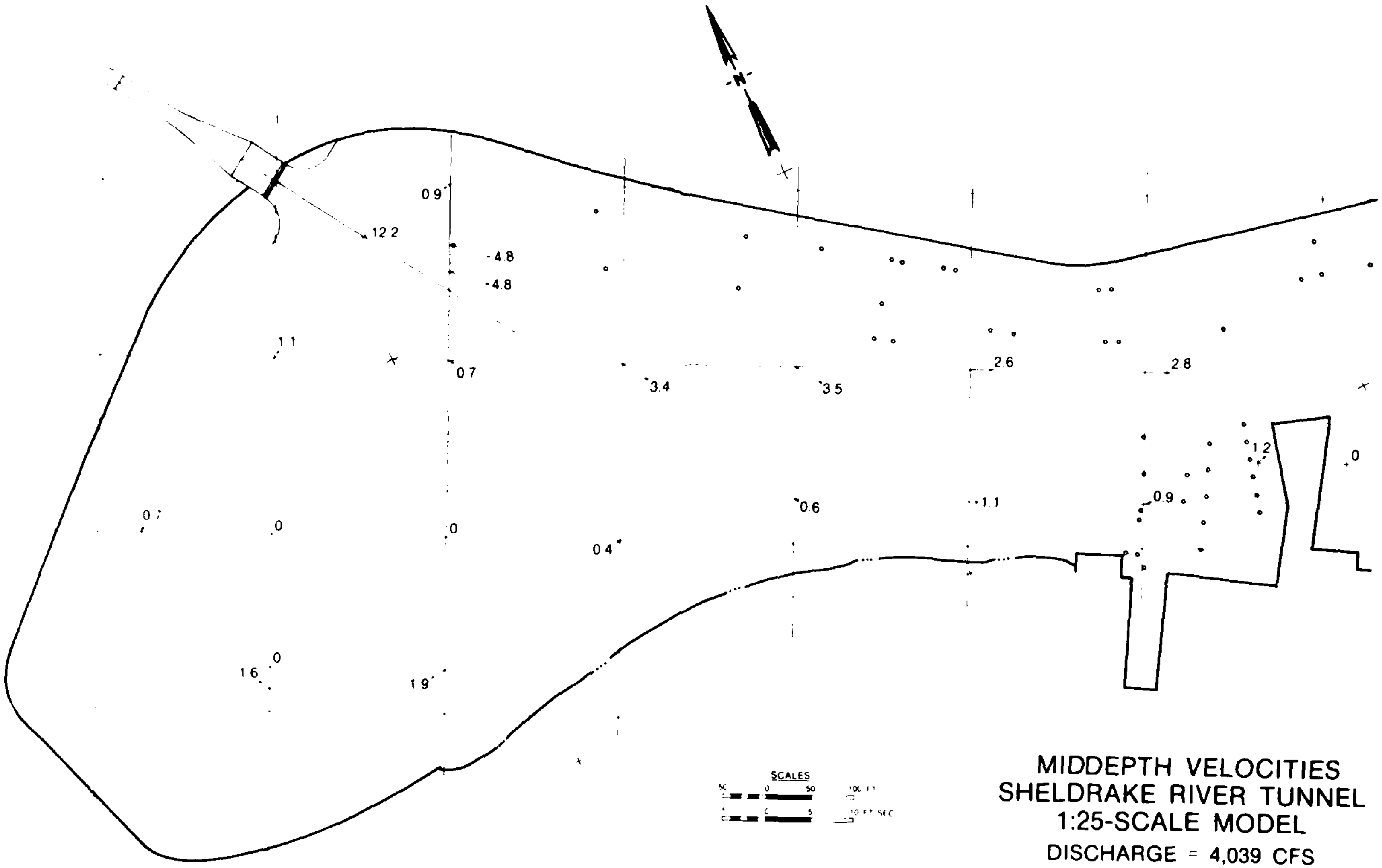

BOTTOM VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 2,551 CFS
TIDE EL = -2.7
TUNNEL N = 0.010
SCALES
100 FT
10 FT SEC
N

PLATE 48

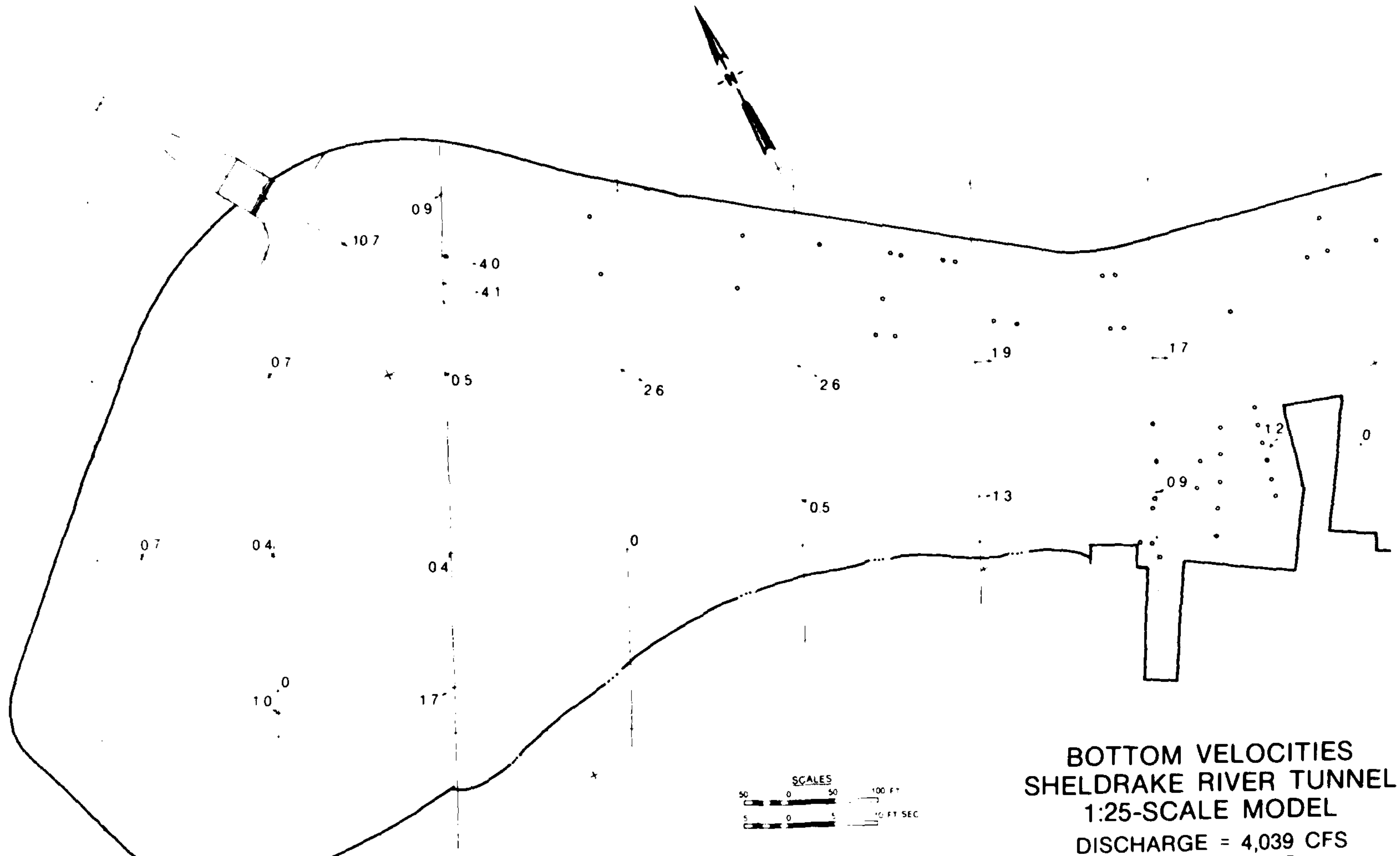


SURFACE VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 4,039 CFS
TIDE EL = -2.7
TUNNEL N = 0.010



MIDDEPTH VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL

DISCHARGE = 4,039 CFS
TIDE EL = -2.7
TUNNEL N = 0.010



BOTTOM VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 4,039 CFS
TIDE EL = -2.7
TUNNEL N = 0.010

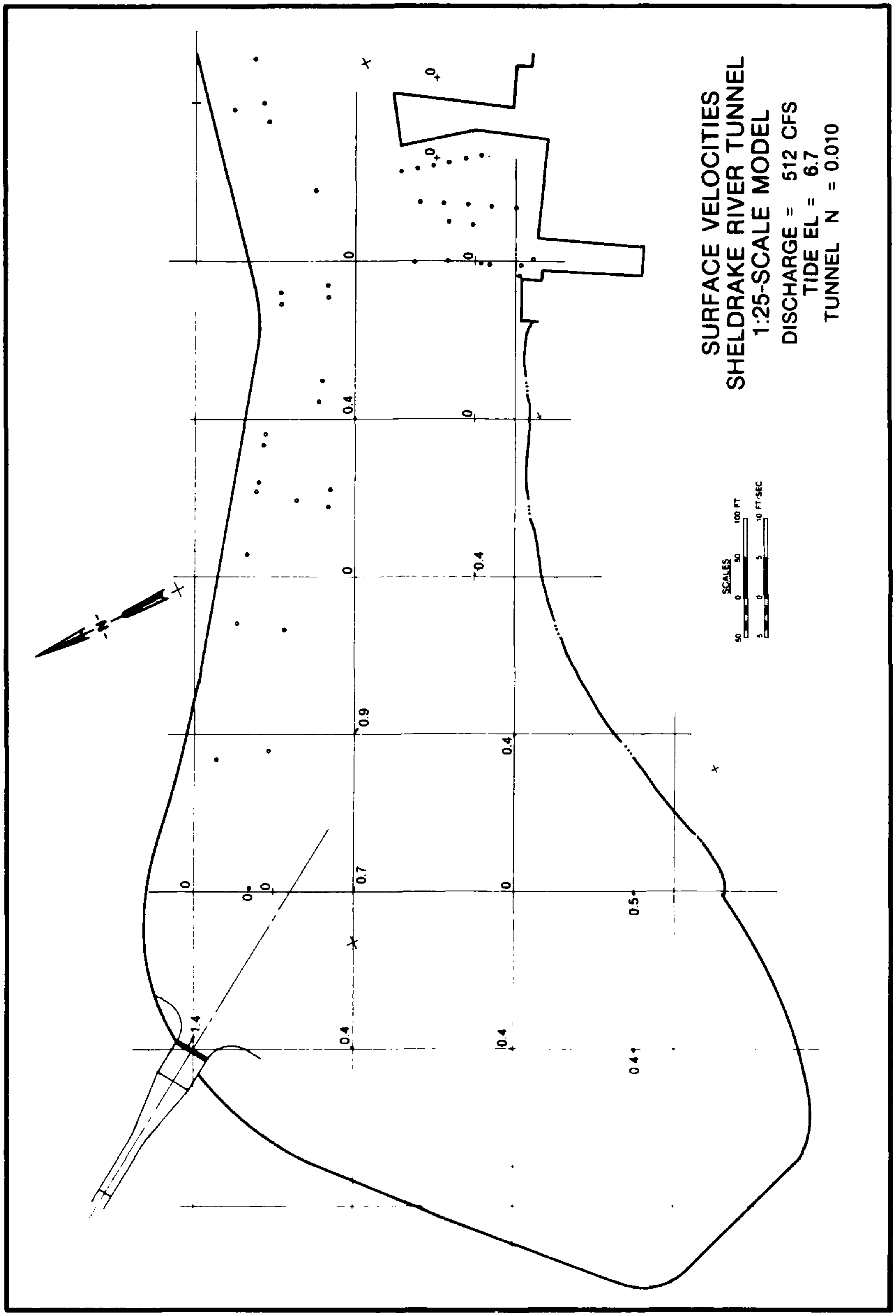
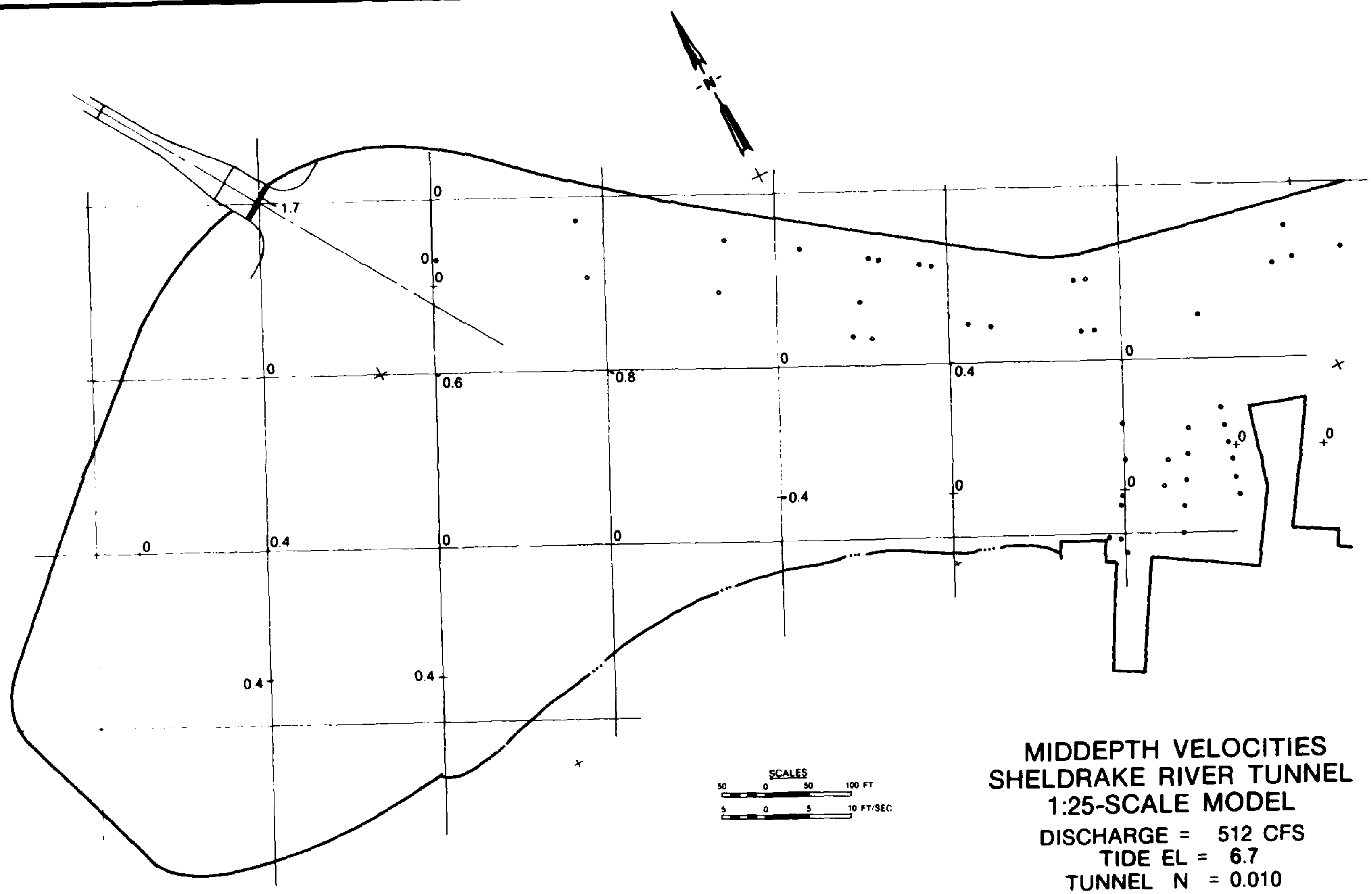
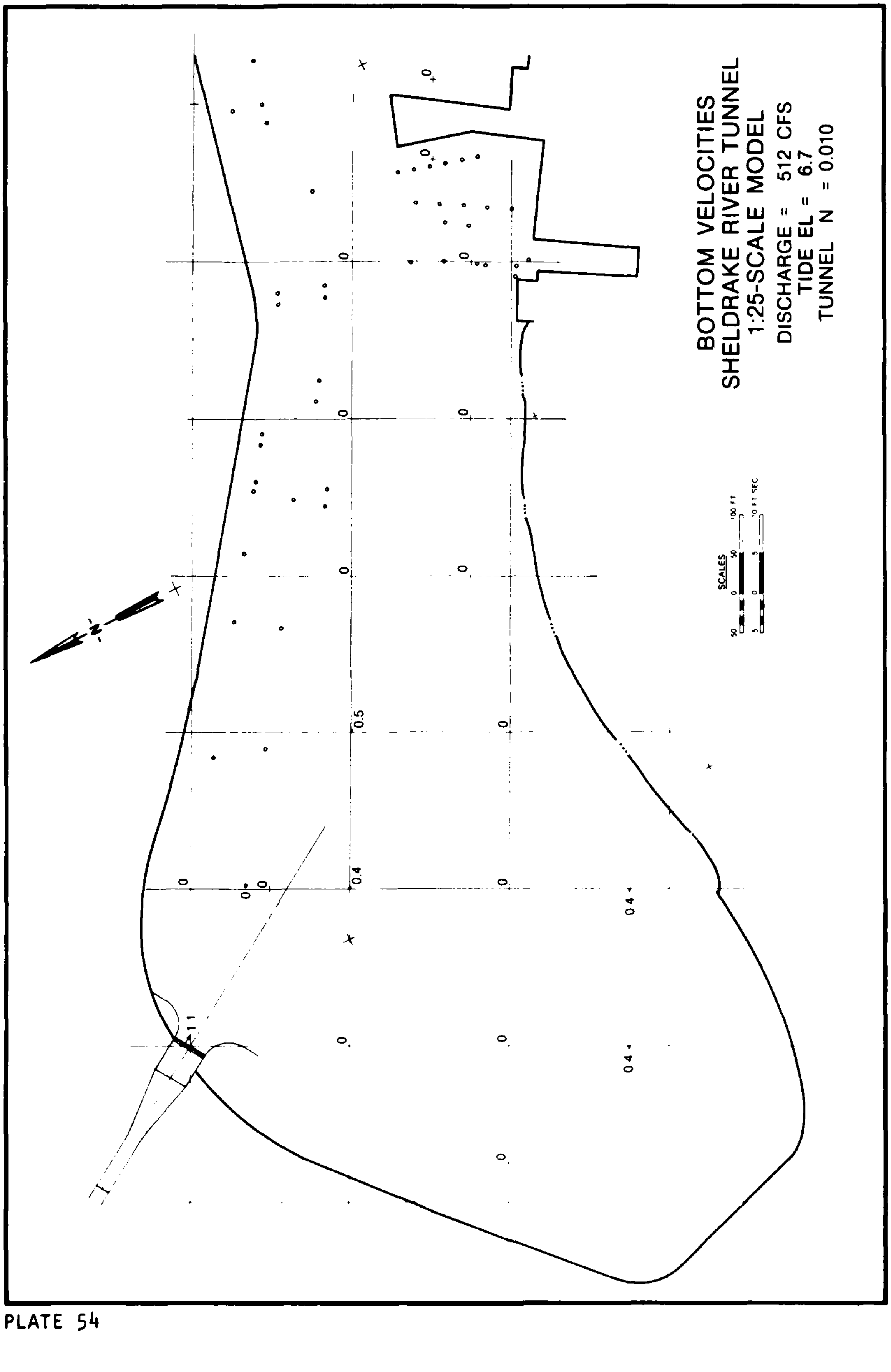

SURFACE VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 512 CFS
TIDE EL = 6.7
TUNNEL N = 0.010
SCALES

PLATE 52



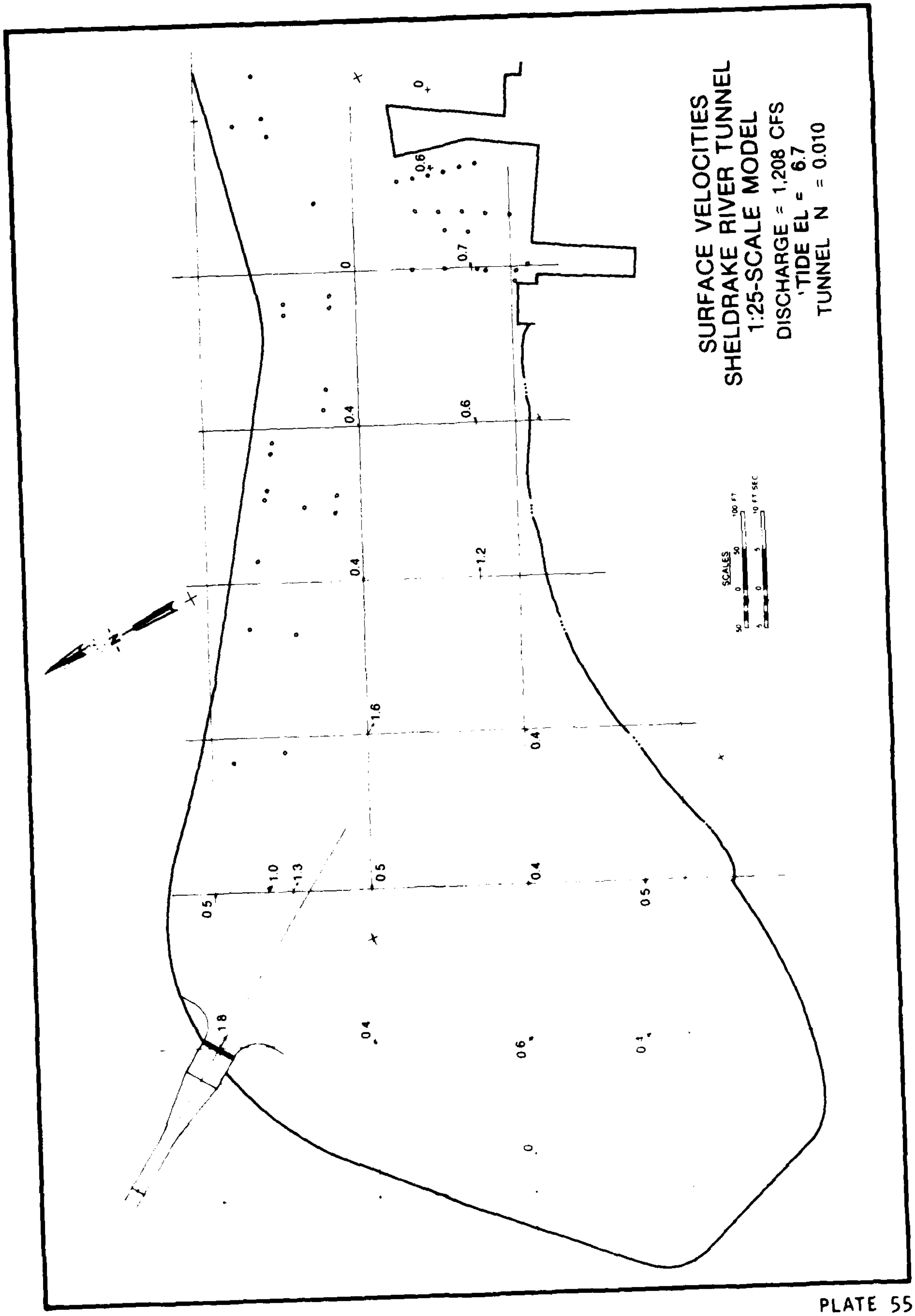
MIDDEPTH VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL

DISCHARGE = 512 CFS
TIDE EL = 6.7
TUNNEL N = 0.010



BOTTOM VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 512 CFS
TIDE EL = 6.7
TUNNEL N = 0.010
SCALES

PLATE 54


SURFACE VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 1,208 CFS
TIDE EL = 6.7
TUNNEL N = 0.010
SCALES

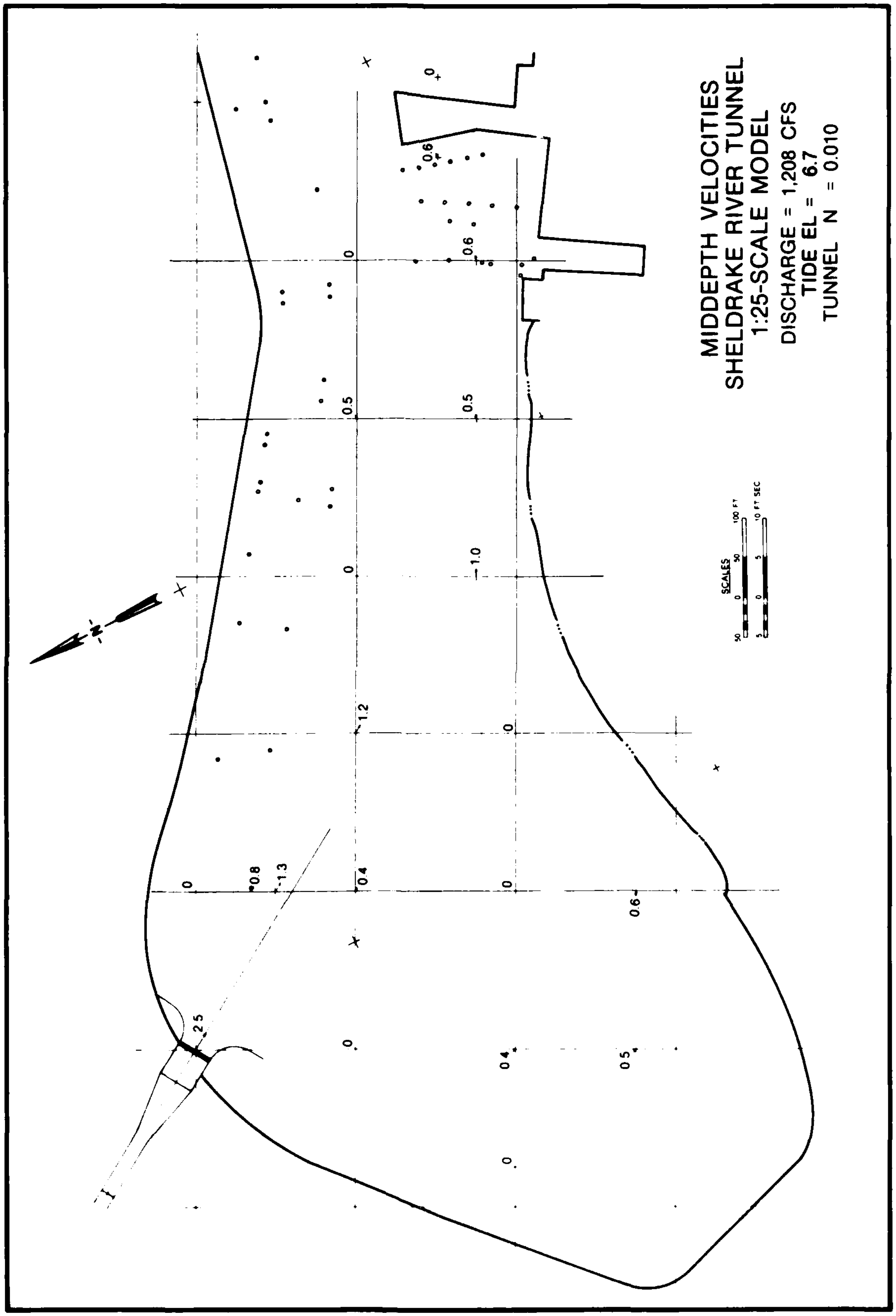
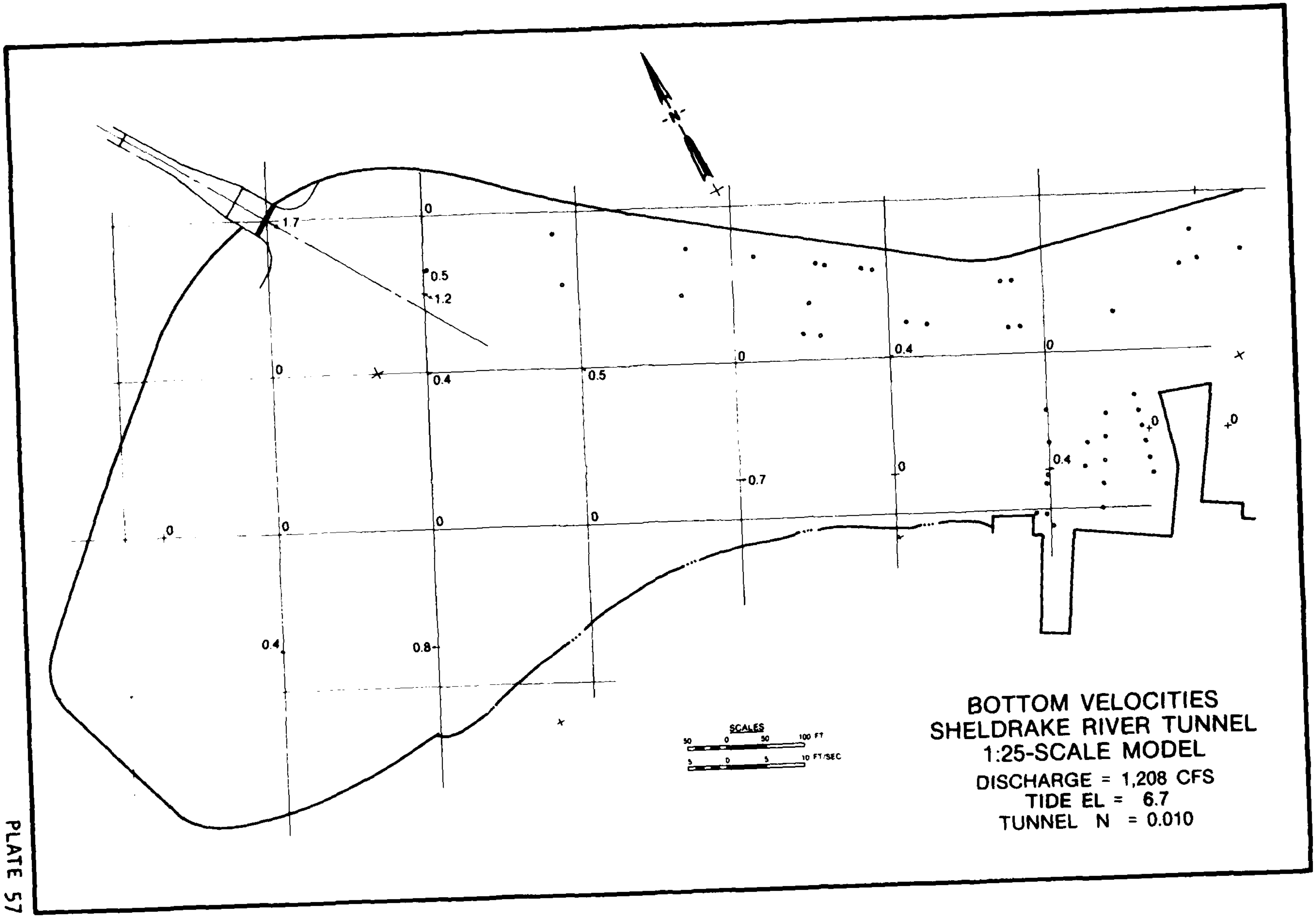

MIDDEPTH VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 1,208 CFS
TIDE EL = 6.7
TUNNEL N = 0.010
SCALES
50 0 50 100 FT
5 0 5 10 FT SEC

PLATE 56



BOTTOM VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 1,208 CFS
TIDE EL = 6.7
TUNNEL N = 0.010

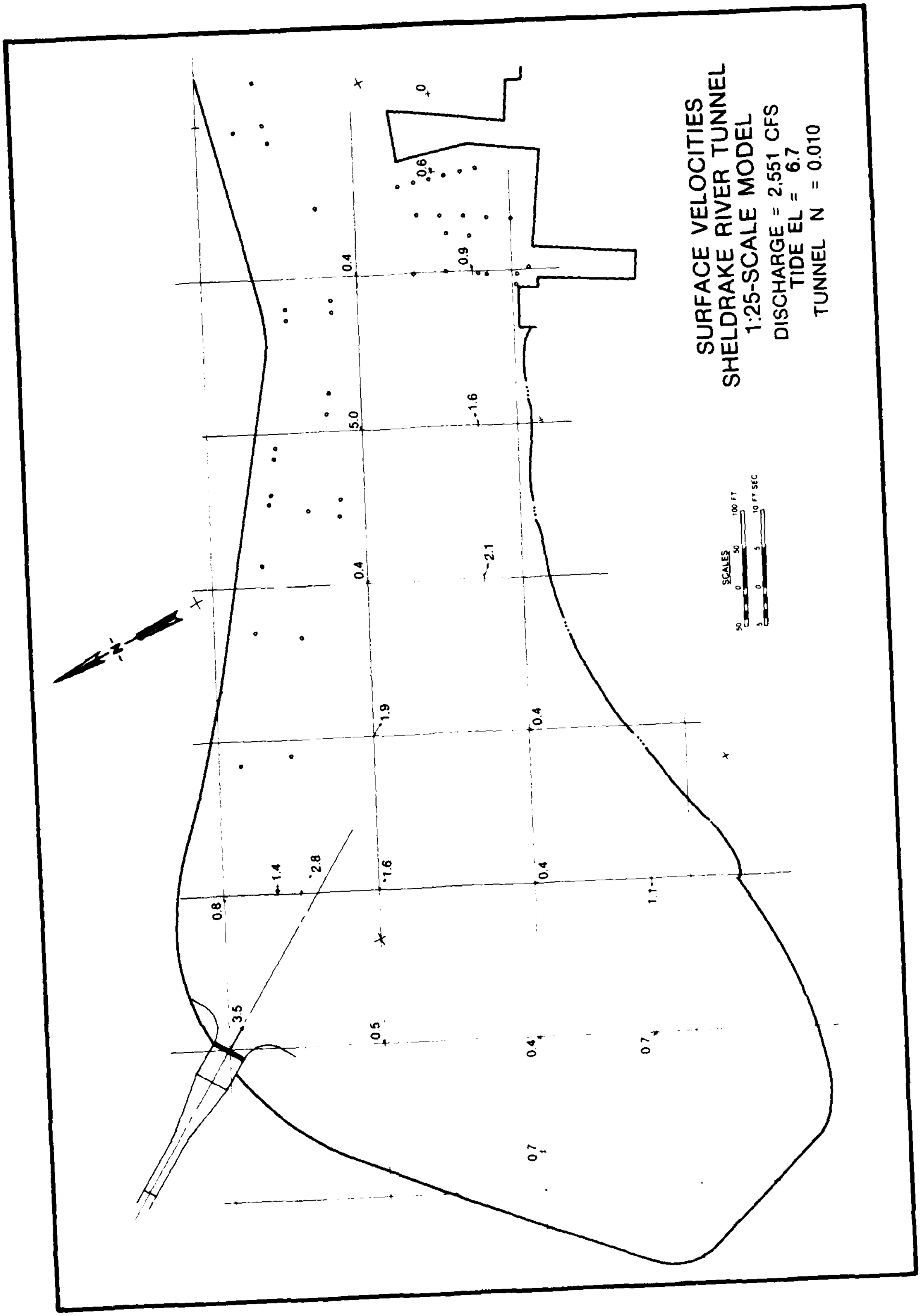
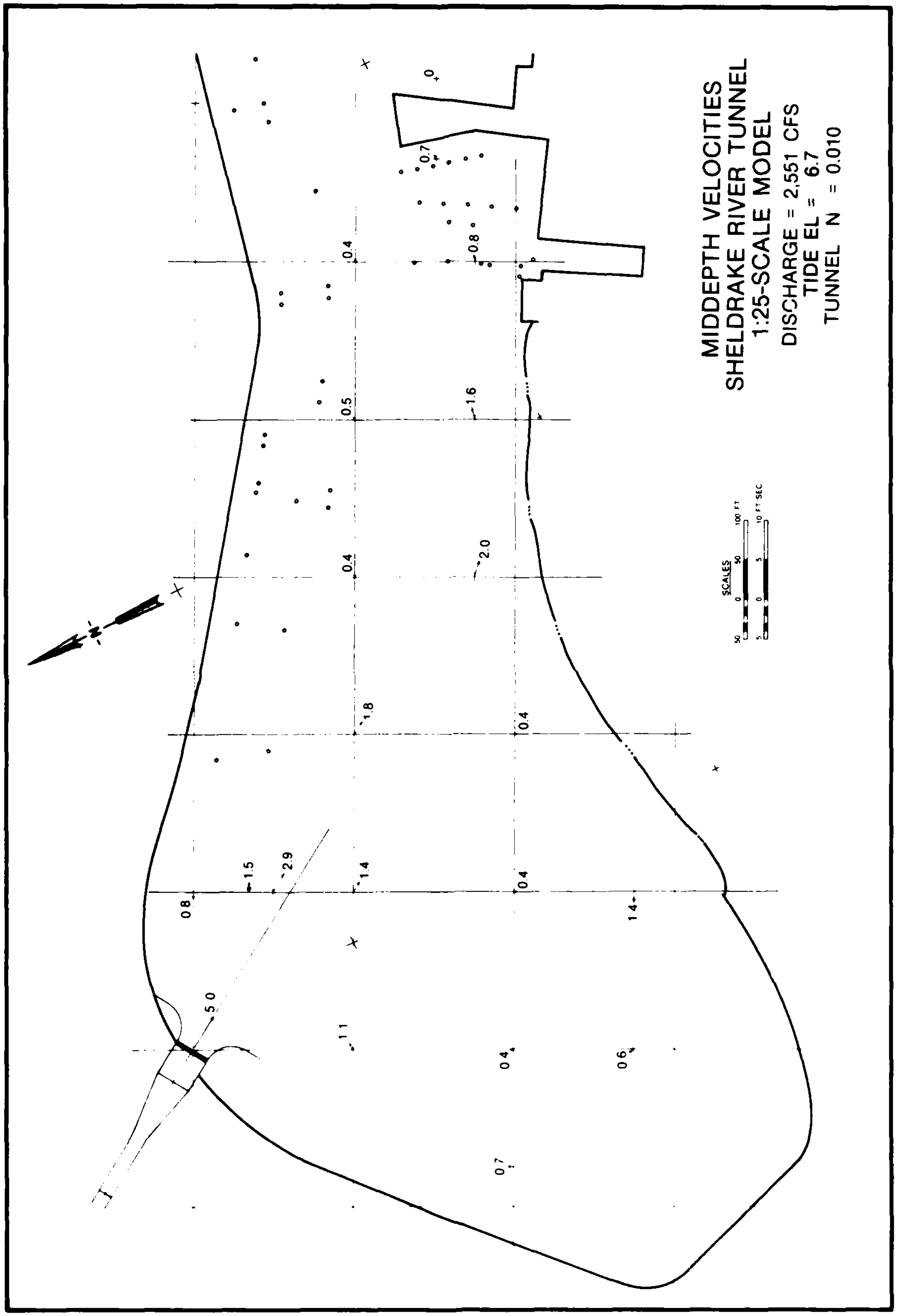

SURFACE VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 2.551 CFS
TIDE EL = 6.7
TUNNEL N = 0.010
SCALES
50 0 50 100 FT
5 0 5 10 FT SEC
N

PLATE 58


MIDDEPTH VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 2,551 CFS
TIDE EL = 6.7
TUNNEL N = 0.010
SCALES
50 0 50 100 FT
5 0 5 10 FT/SEC
N
0.7
0.8
0.4
0.5
1.6
0.4
2.0
1.8
0.4
1.5
2.9
1.4
0.4
0.8
1.4
5.0
1.1
0.4
0.6
0.7

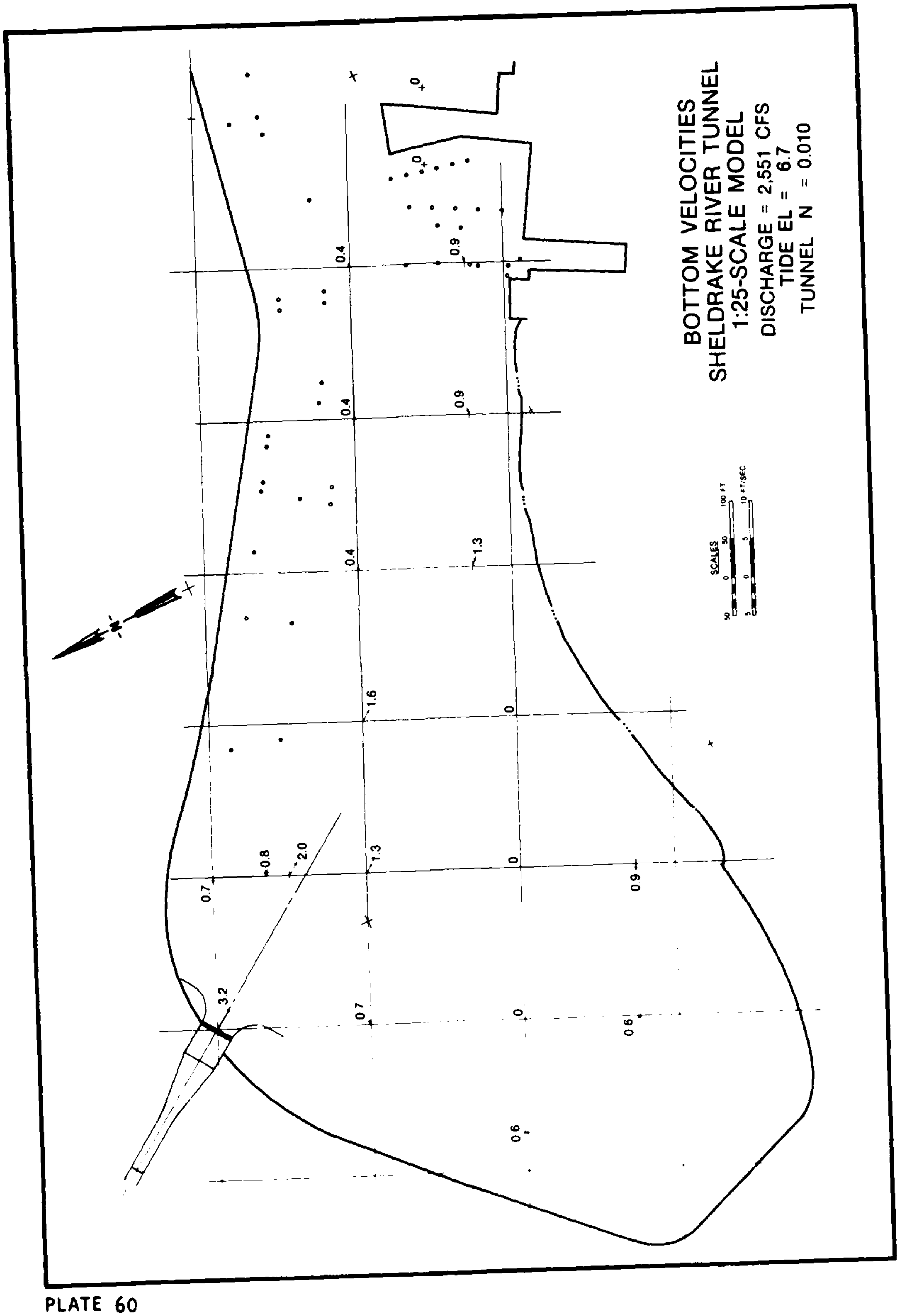
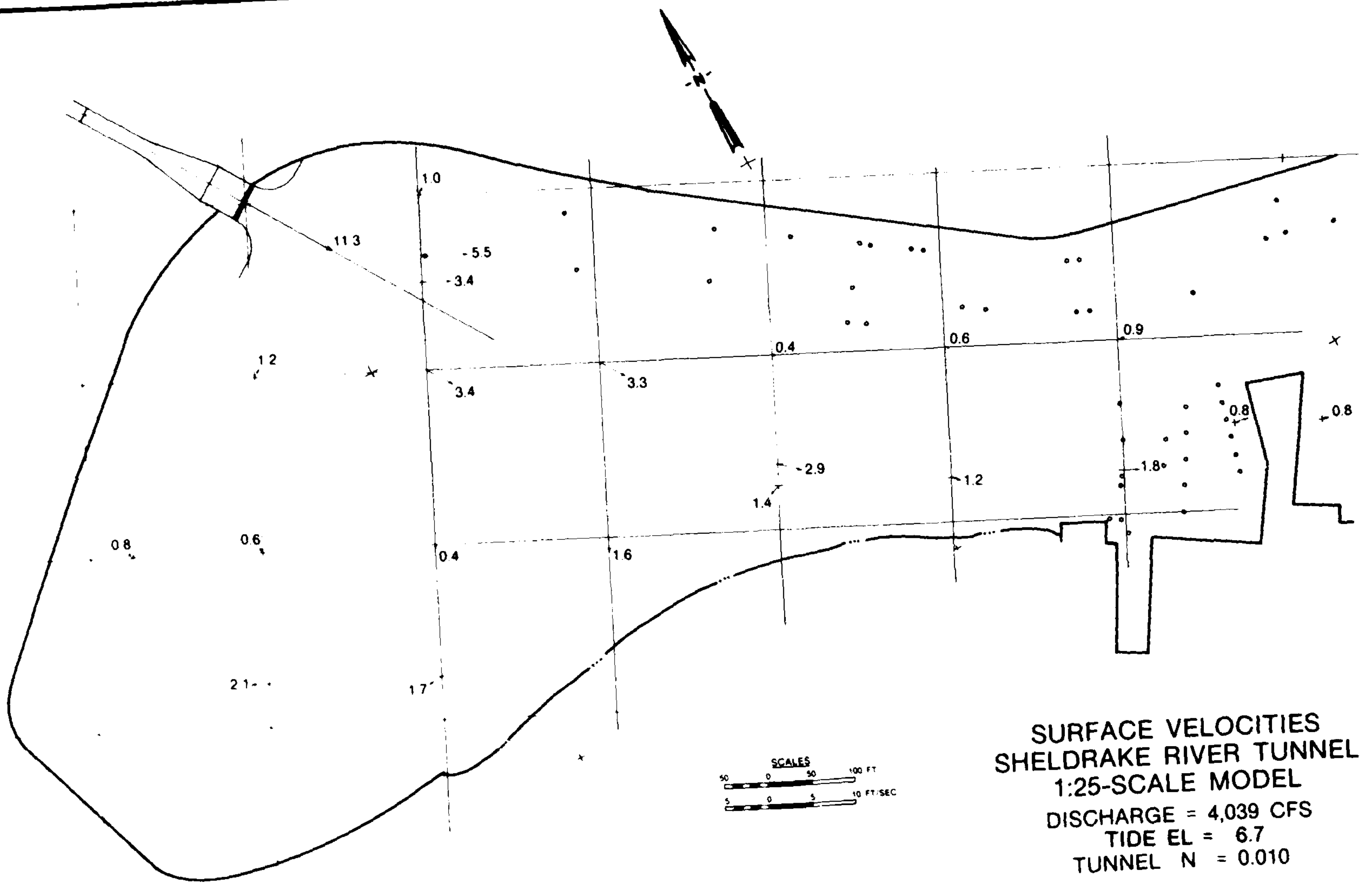

BOTTOM VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 2,551 CFS
TIDE EL = 6.7
TUNNEL N = 0.010
SCALES
50 0 50 100 FT
5 0 5 10 FT/SEC

PLATE 60



SURFACE VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 4,039 CFS
TIDE EL = 6.7
TUNNEL N = 0.010

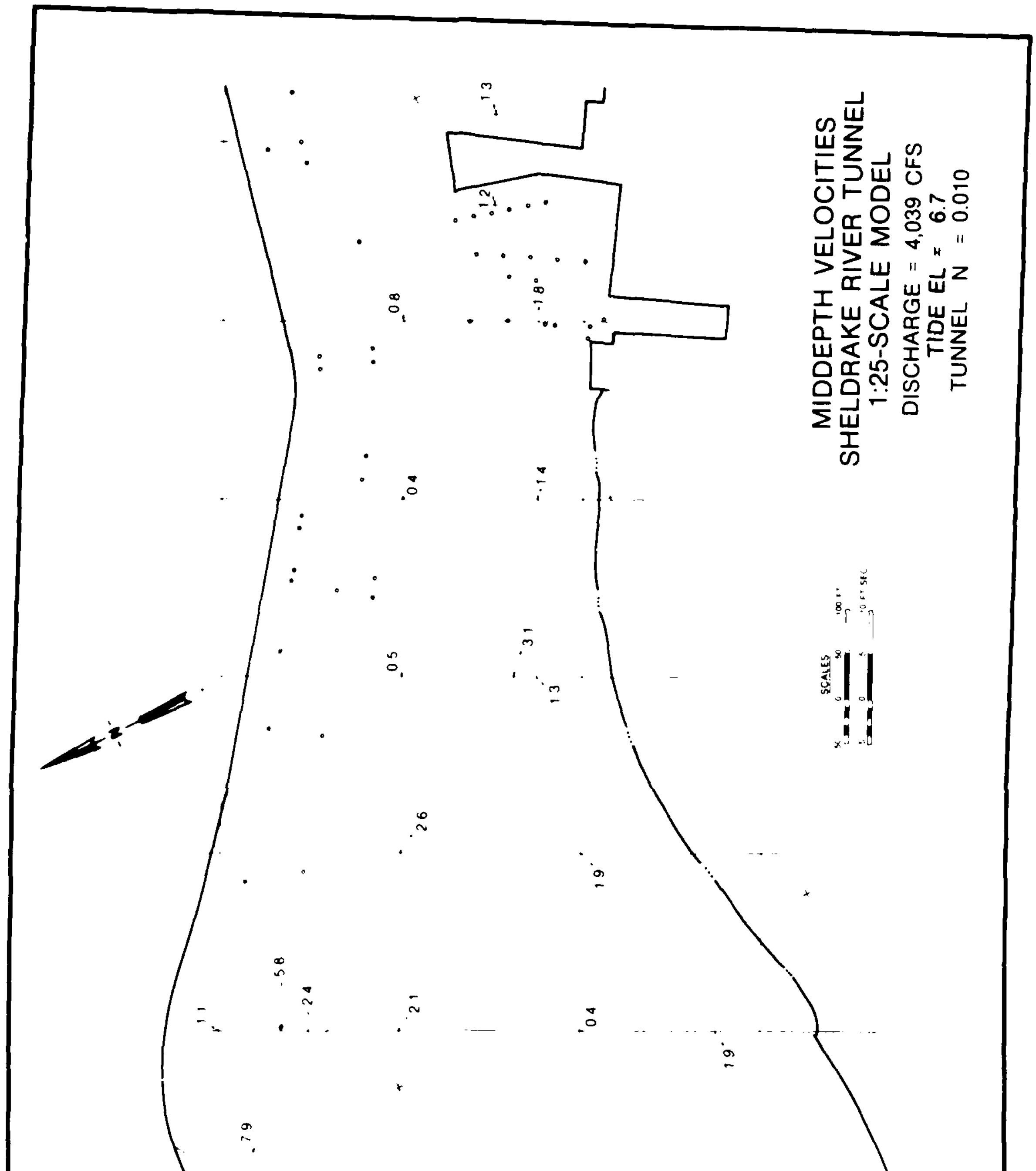

MIDDEPTH VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 4,039 CFS
TIDE EL = 6.7
TUNNEL N = 0.010
SCALES

PLATE 62

AD-A181 888 SHELDRAKE RIVER TUNNEL MAMARONECK NEW YORK HYDRAULIC MODEL INVESTIGATION(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS HYDRAULICS LAB 2/2

UNCLASSIFIED C H TATE DEC 86 WES/TR/HL-86-7 F/G 13/2 NL

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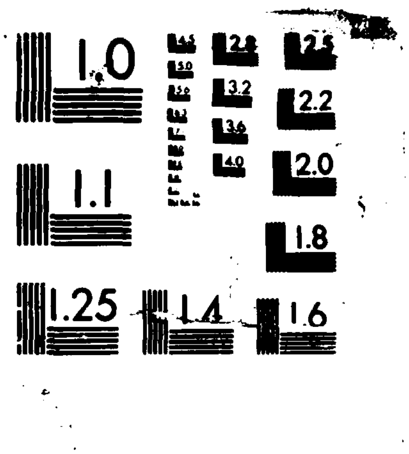
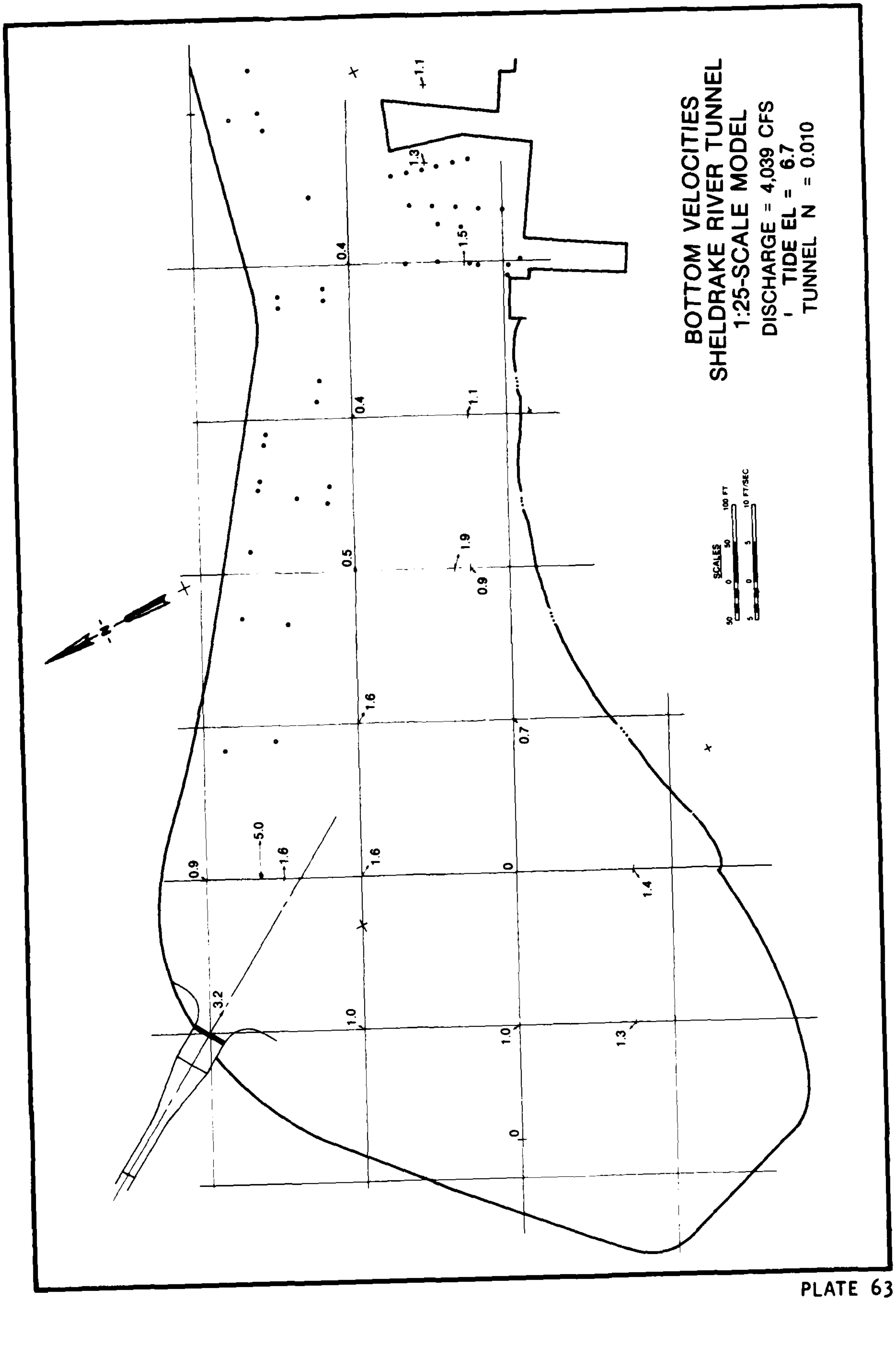


MICROCOPY RESOLUTION TEST CHART


BOTTOM VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 4,039 CFS
TIDE EL = 6.7
TUNNEL N = 0.010
SCALES
100 FT
10 FT/SEC
N

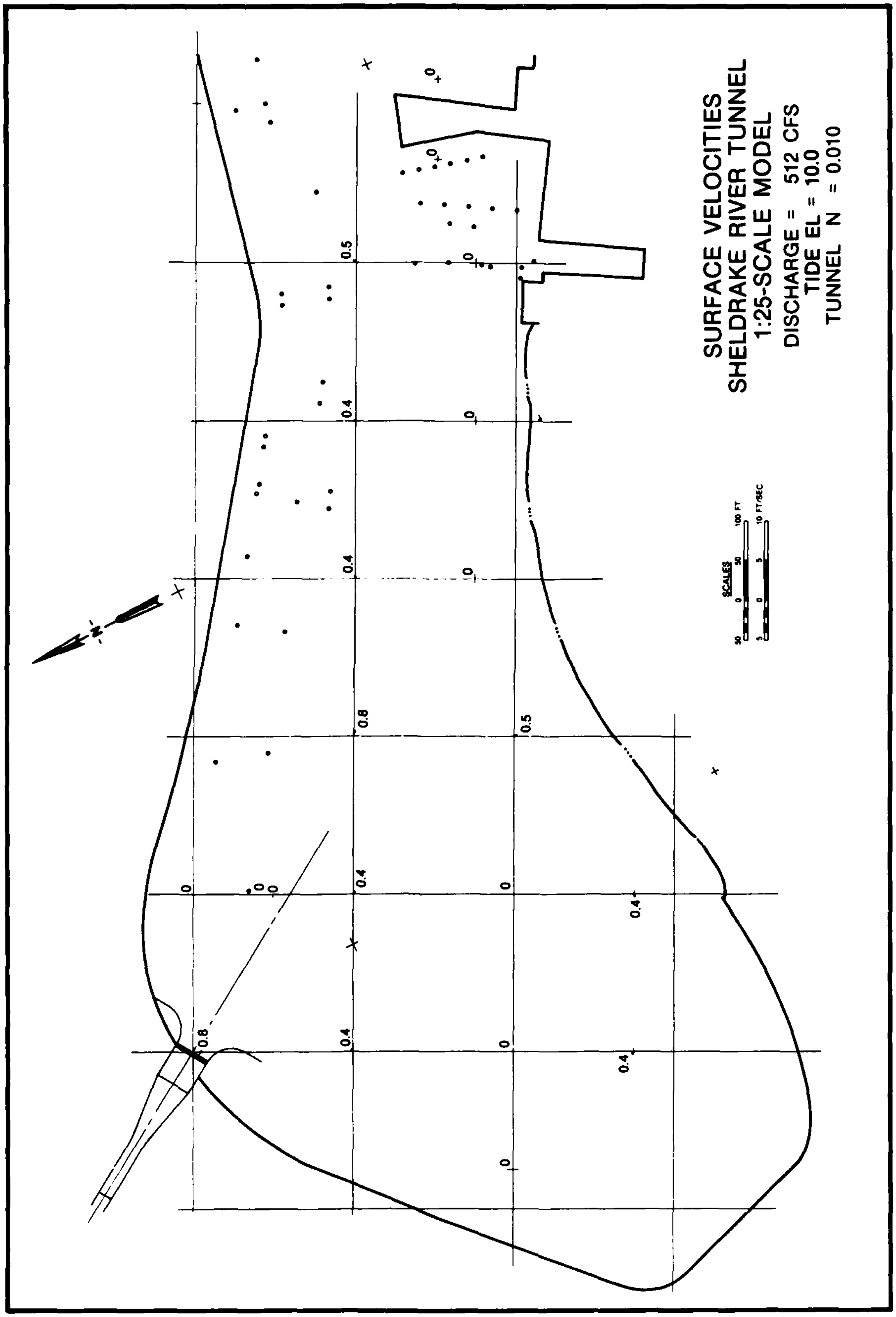
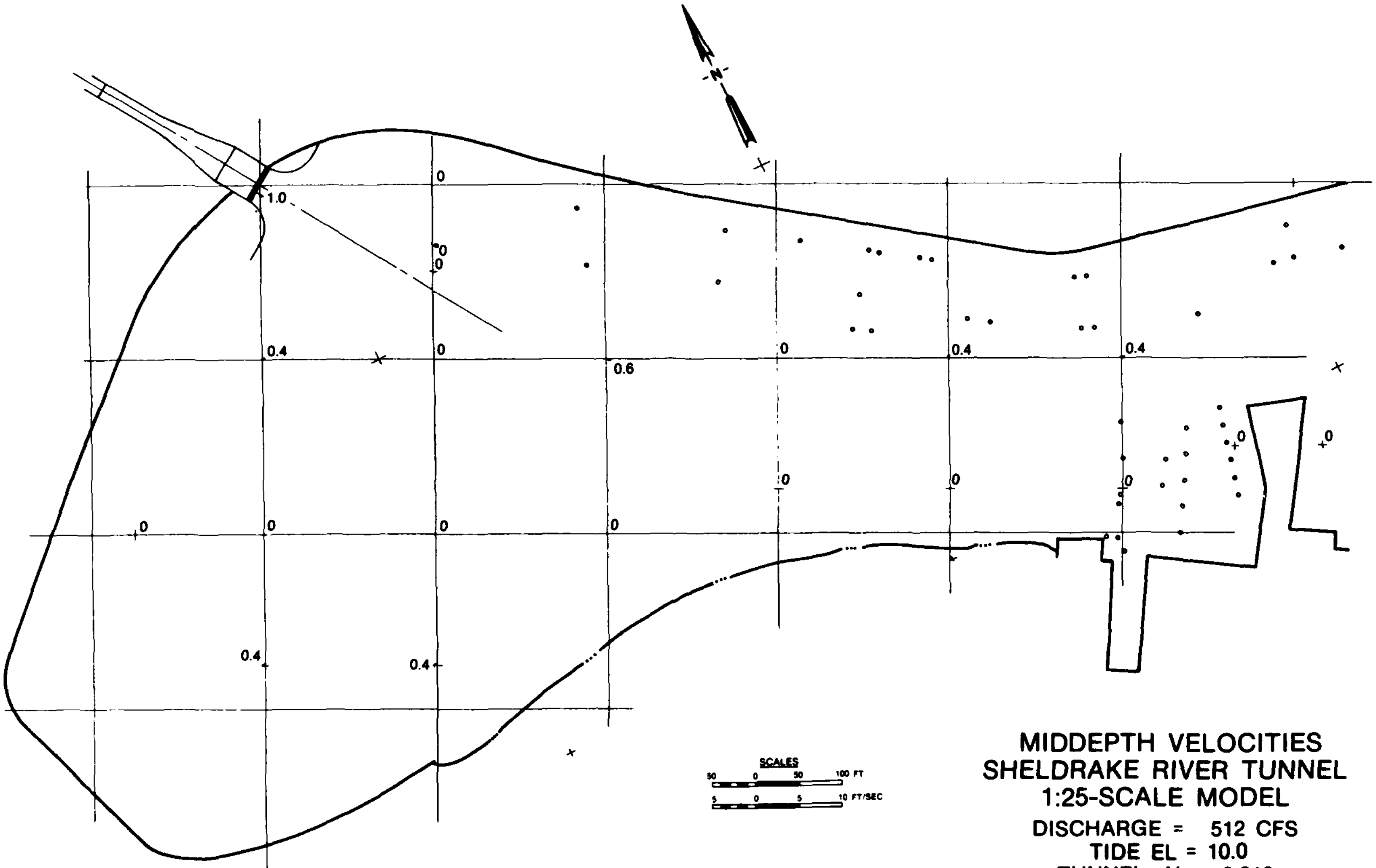

SURFACE VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 512 CFS
TIDE EL = 10.0
TUNNEL N = 0.010
SCALES
100 FT
10 FT/SEC

PLATE 64



MIDDEPTH VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL

DISCHARGE = 512 CFS
TIDE EL = 10.0
TUNNEL N = 0.010

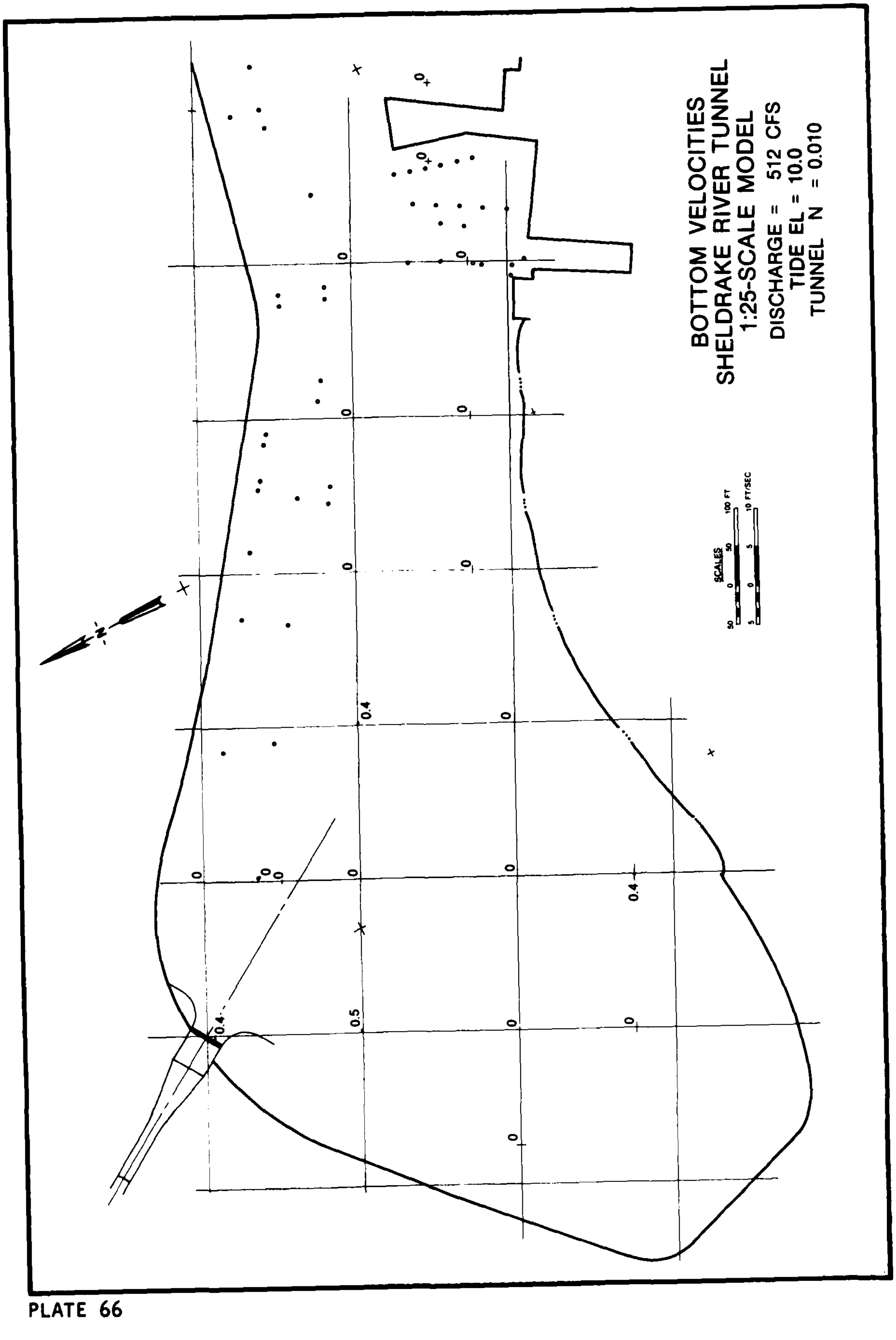
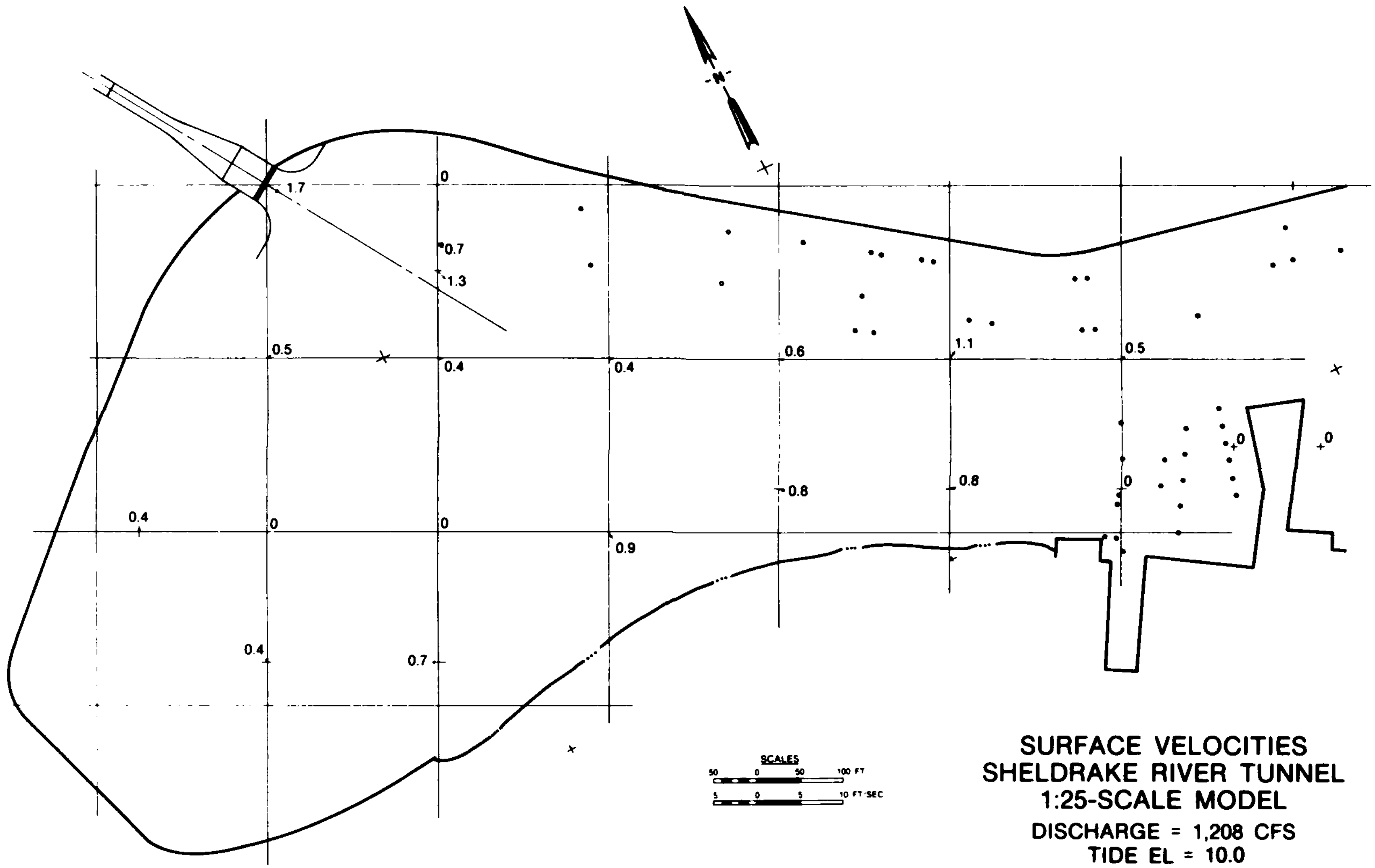

BOTTOM VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 512 CFS
TIDE EL = 10.0
TUNNEL N = 0.010
SCALES
50 0 50 100 FT
5 0 5 10 FT/SEC
N

PLATE 66



SURFACE VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 1,208 CFS
TIDE EL = 10.0
TUNNEL N = 0.010

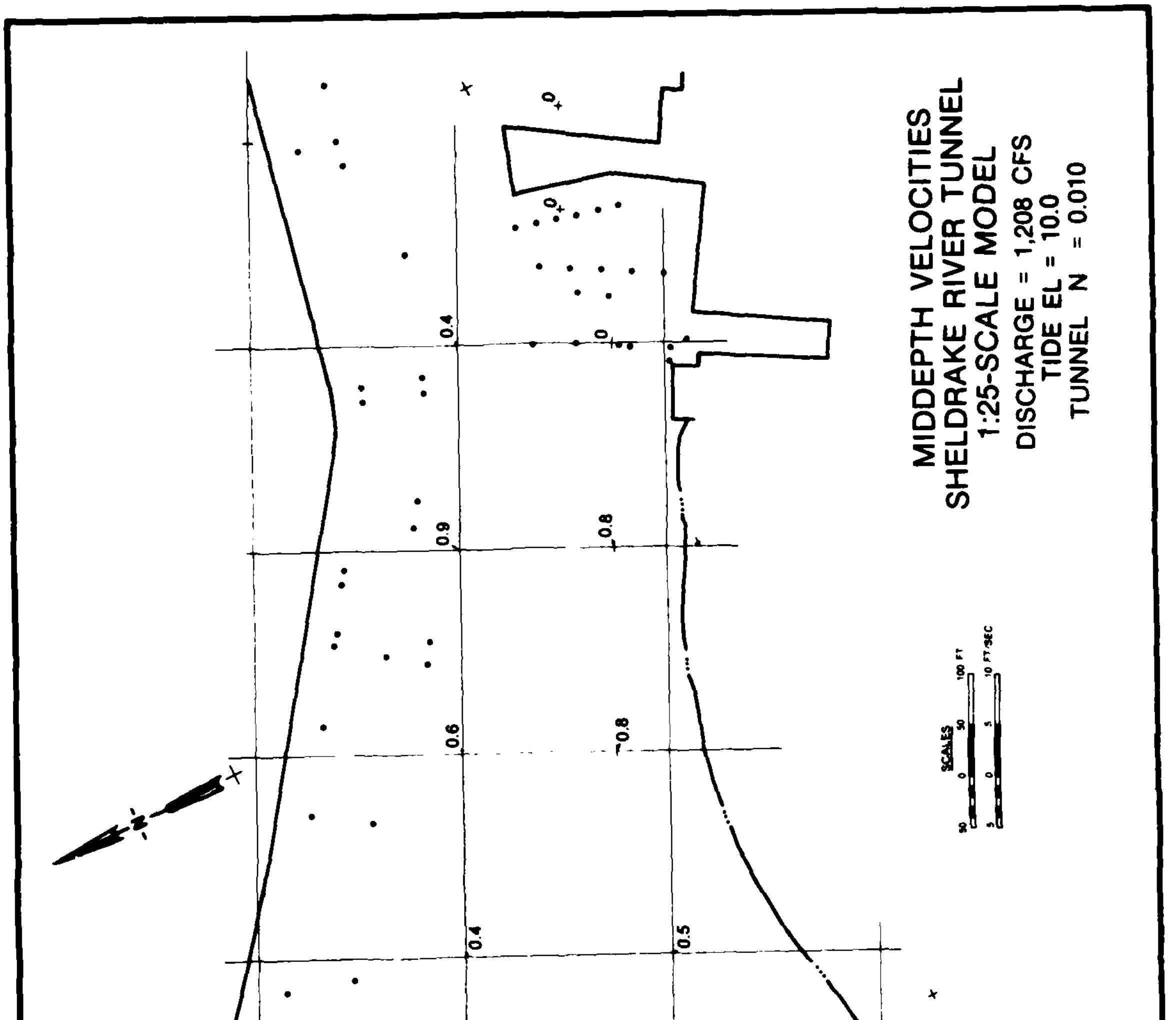
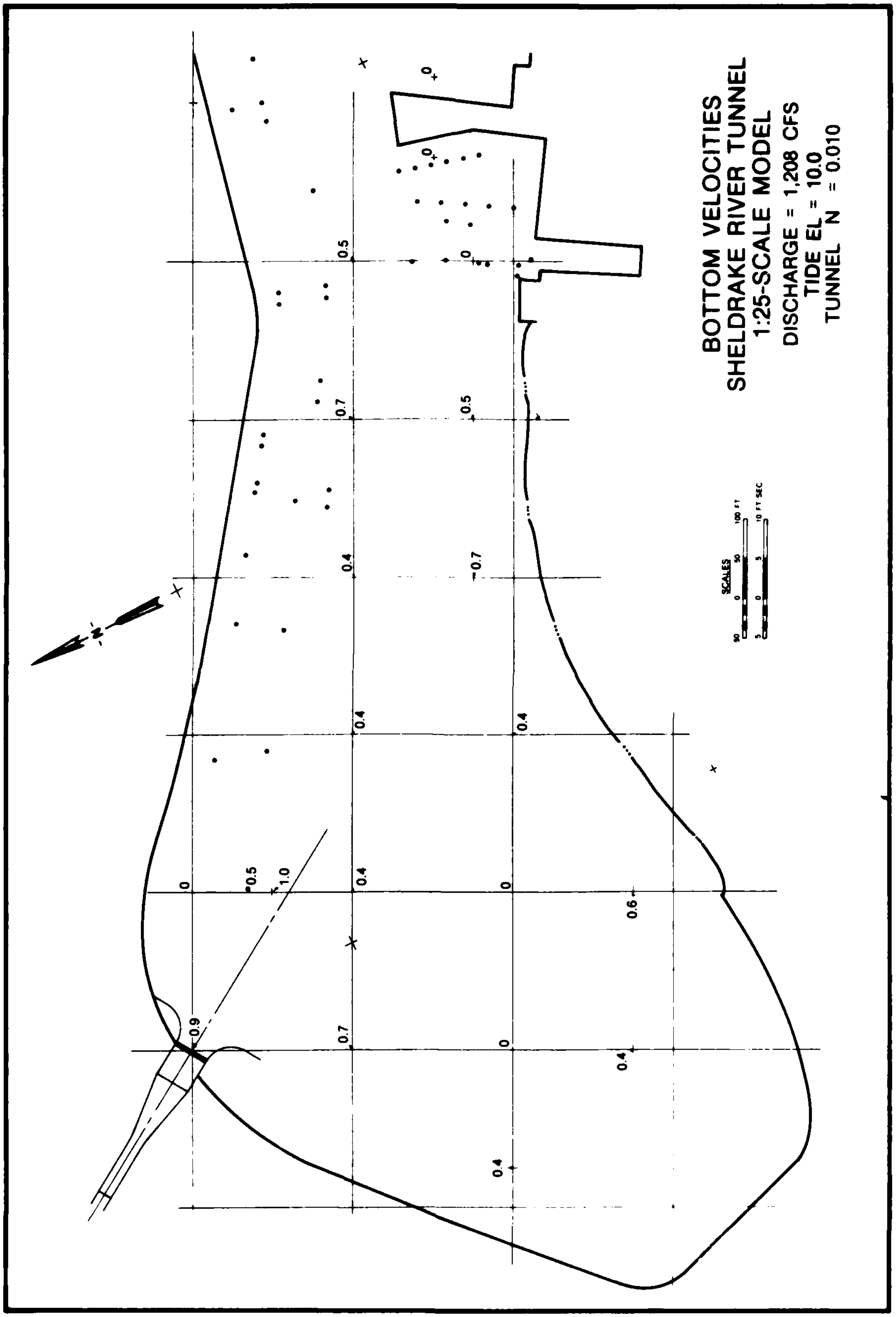

MIDDEPTH VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 1,208 CFS
TIDE EL = 10.0
TUNNEL N = 0.010
SCALES

PLATE 68



BOTTOM VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 1,208 CFS
TIDE EL = 10.0
TUNNEL N = 0.010
SCALES

PLATE 69

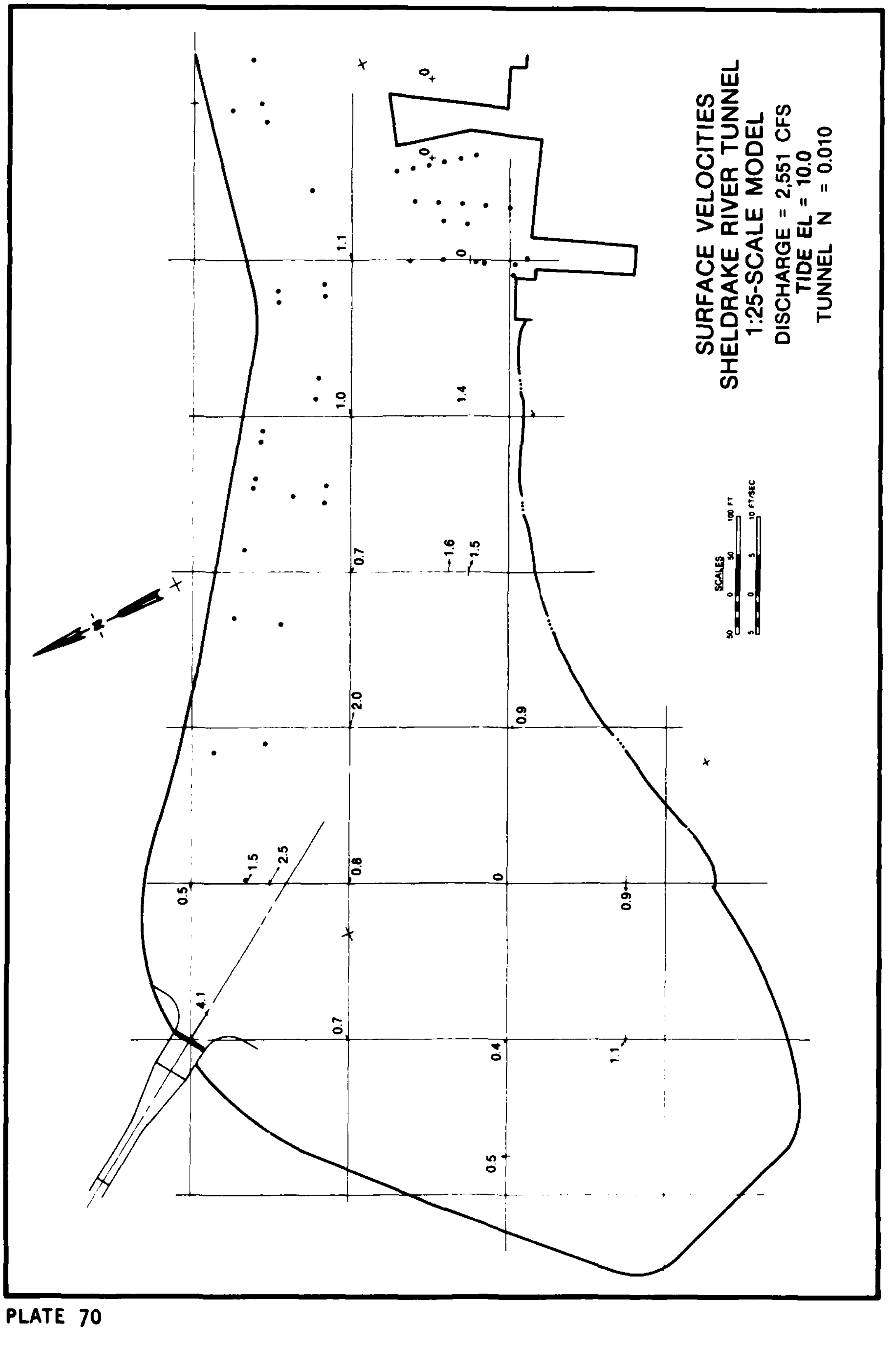
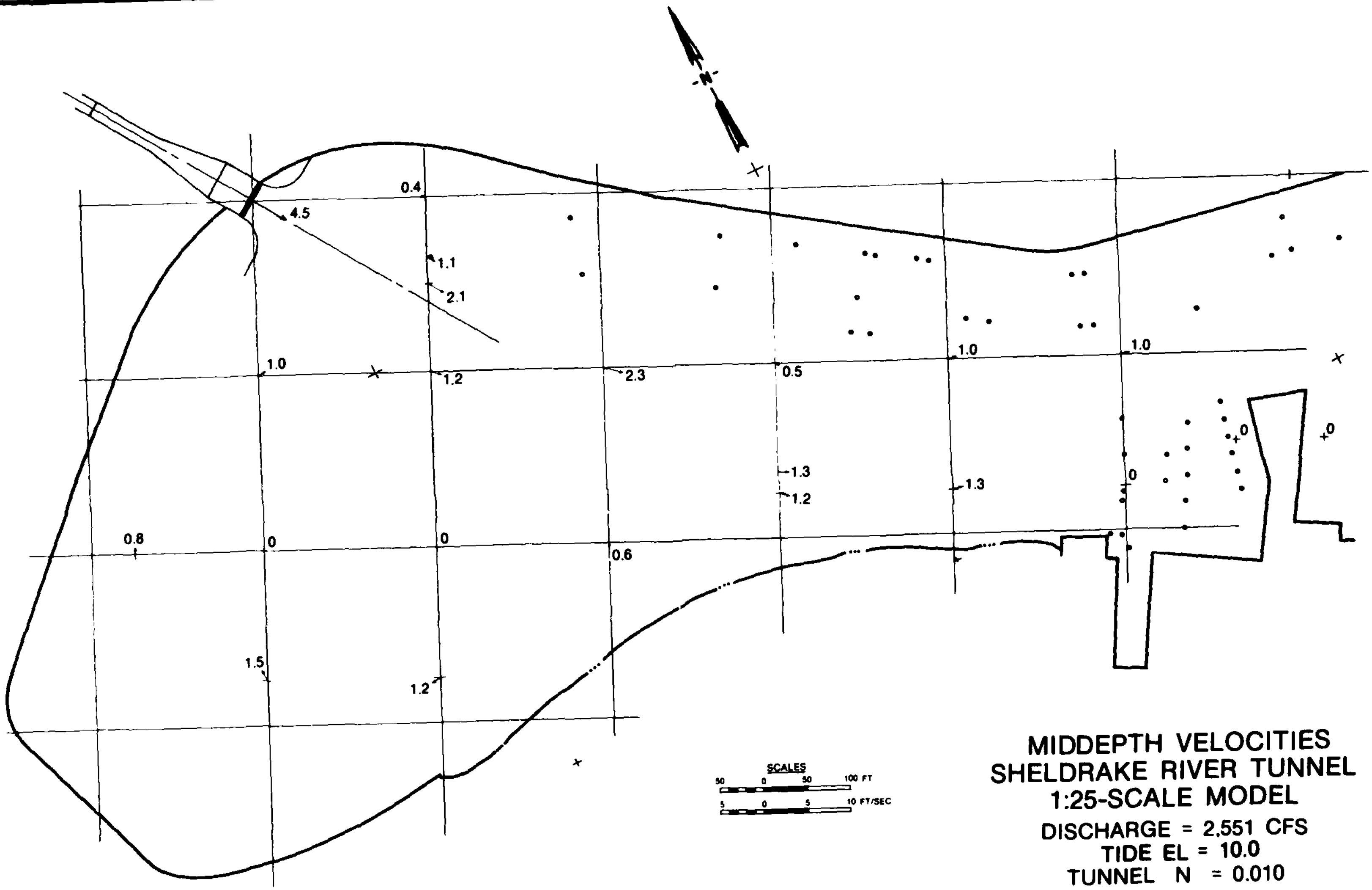

SURFACE VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 2,551 CFS
TIDE EL = 10.0
TUNNEL N = 0.010
SCALES
50 0 50 100 FT
5 0 5 10 FT/SEC
N

PLATE 70



MIDDEPTH VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 2,551 CFS
TIDE EL = 10.0
TUNNEL N = 0.010

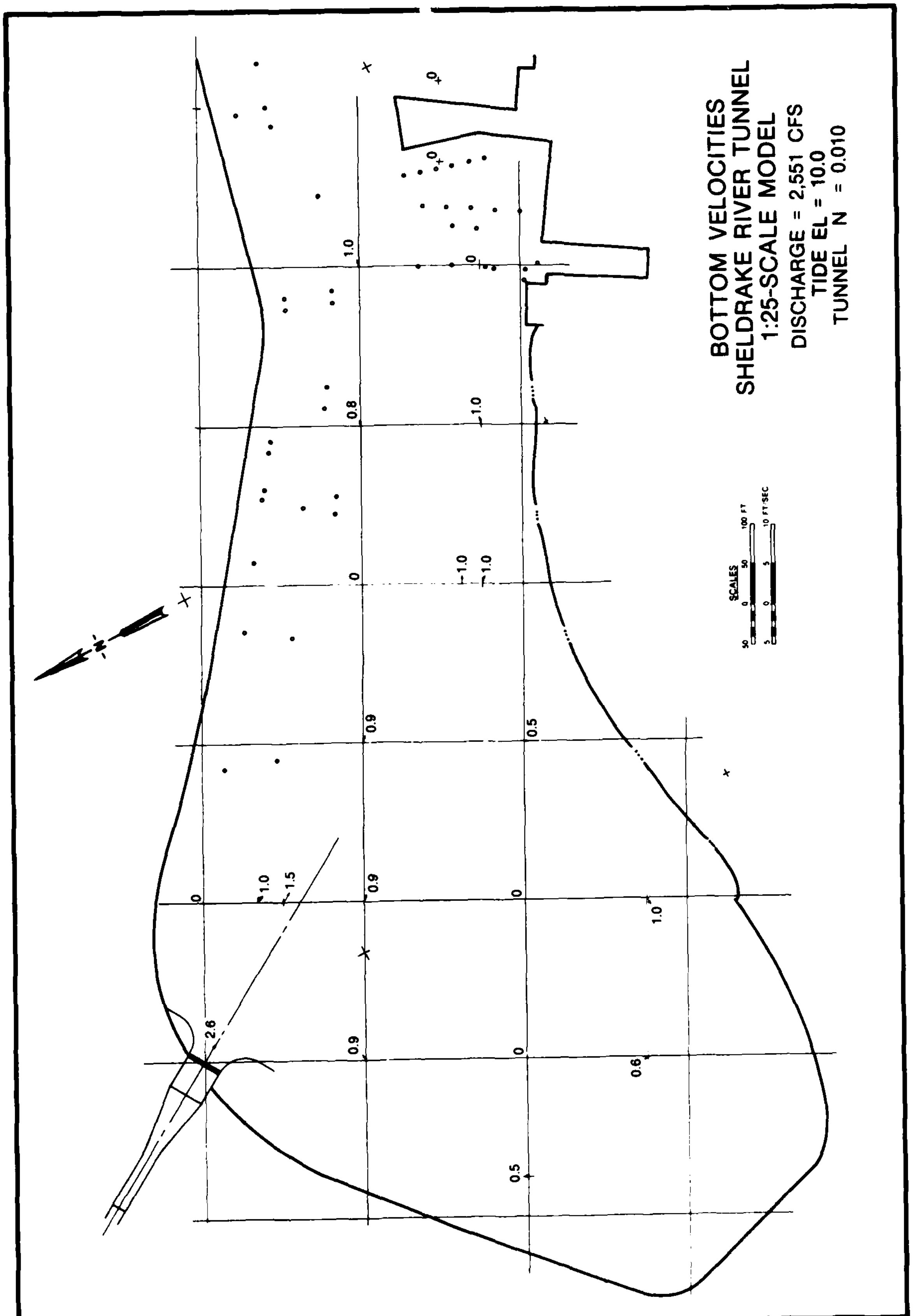
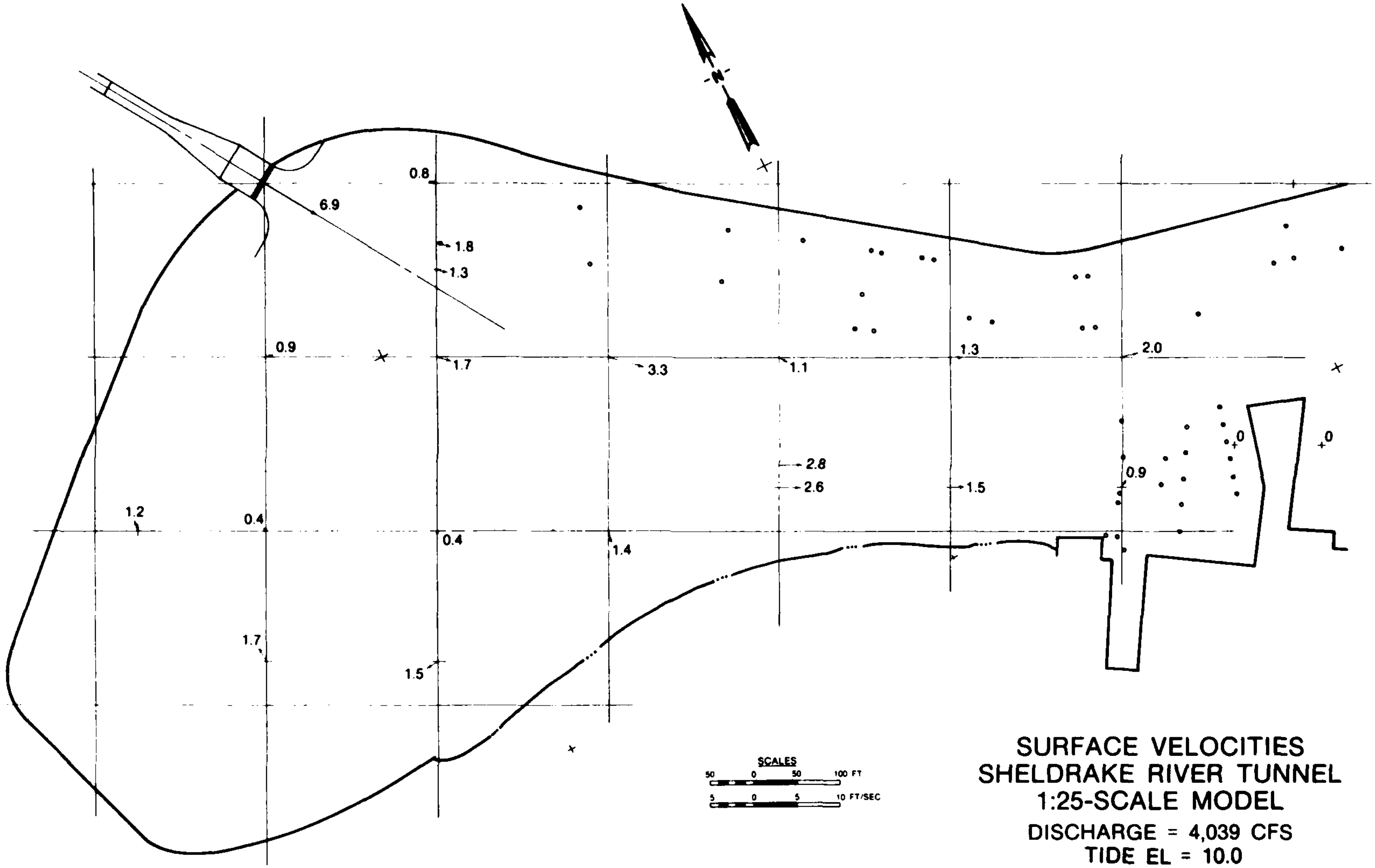
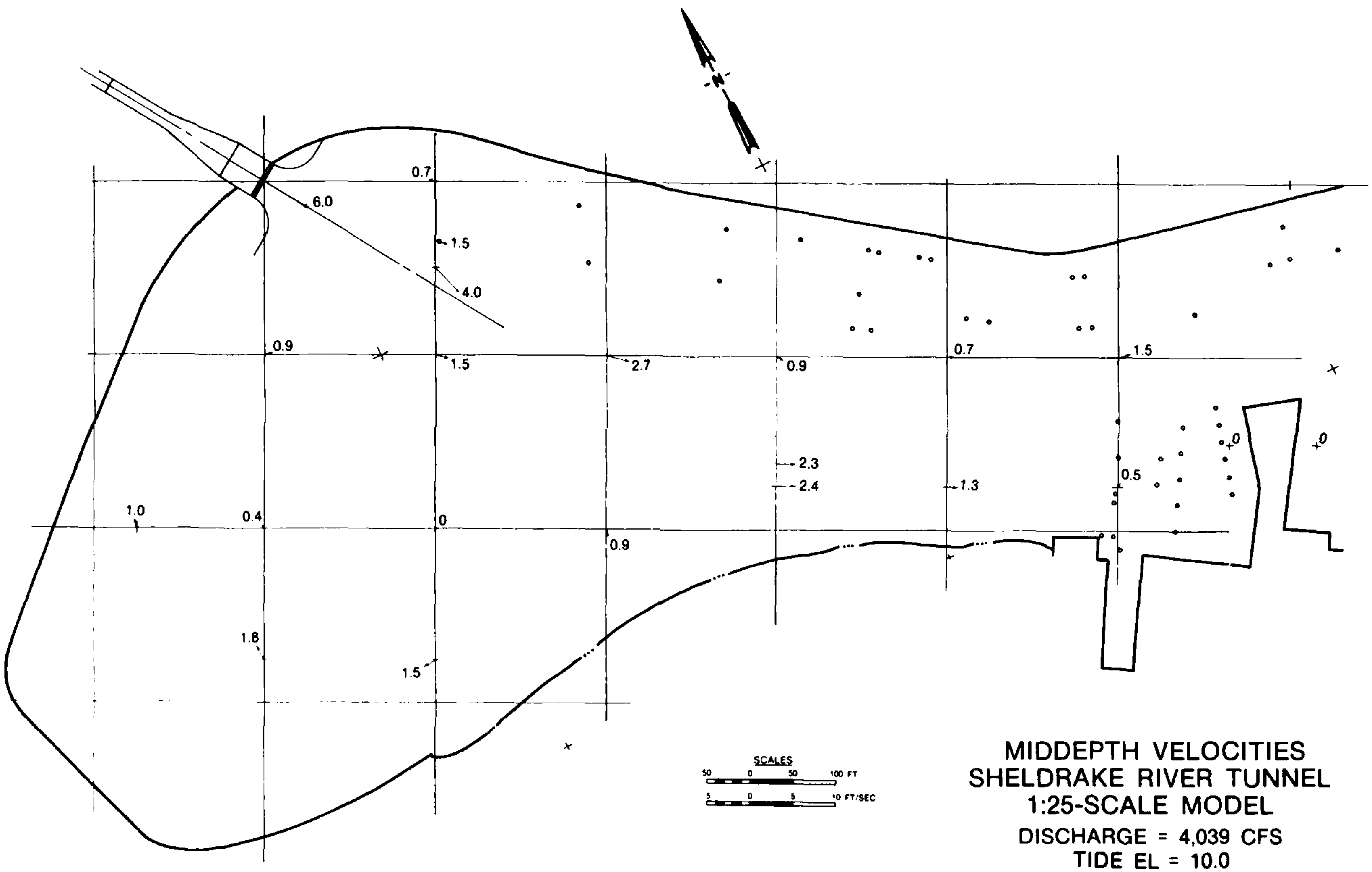

BOTTOM VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 2,551 CFS
TIDE EL = 10.0
TUNNEL N = 0.010
SCALES

PLATE 72

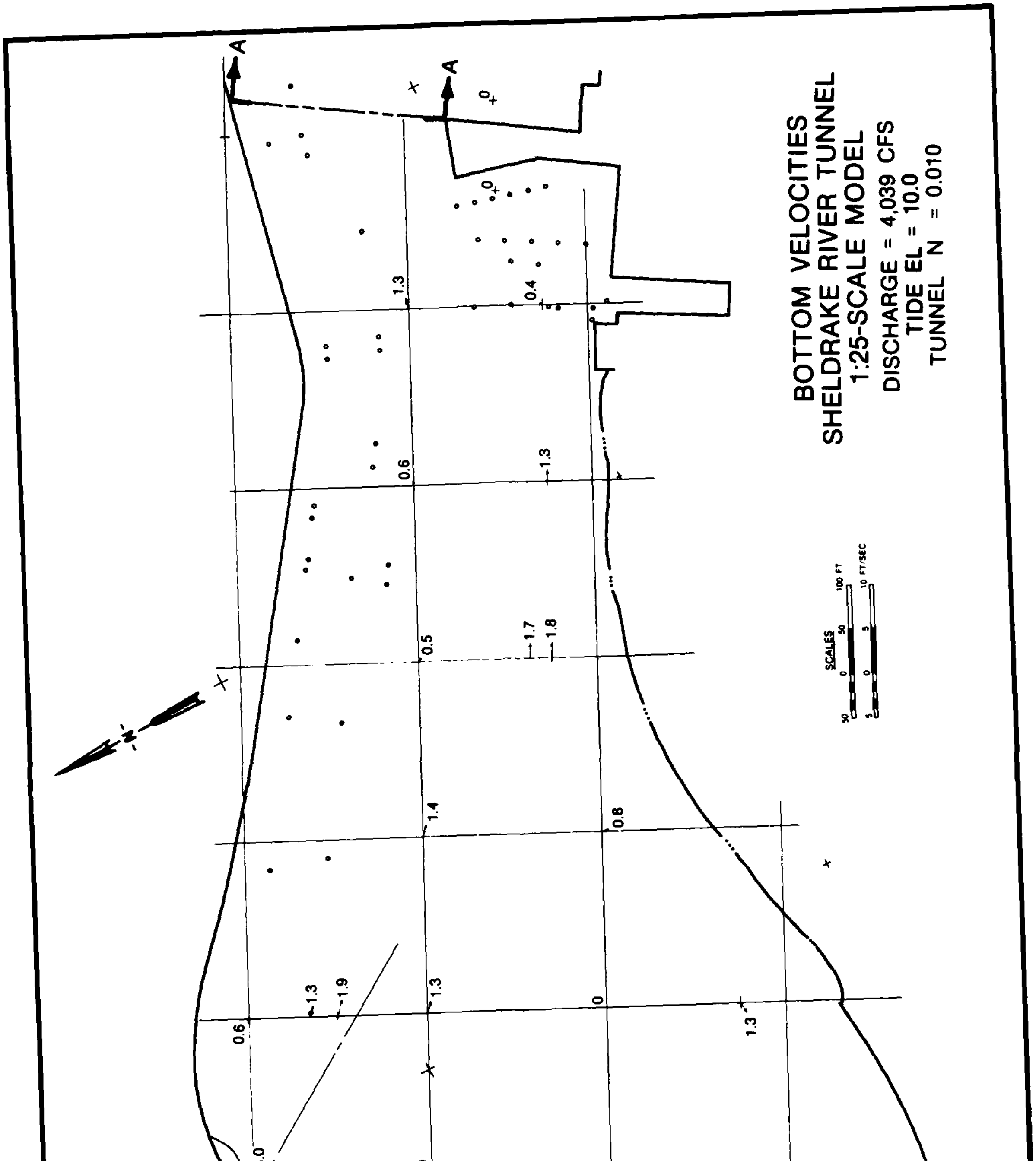


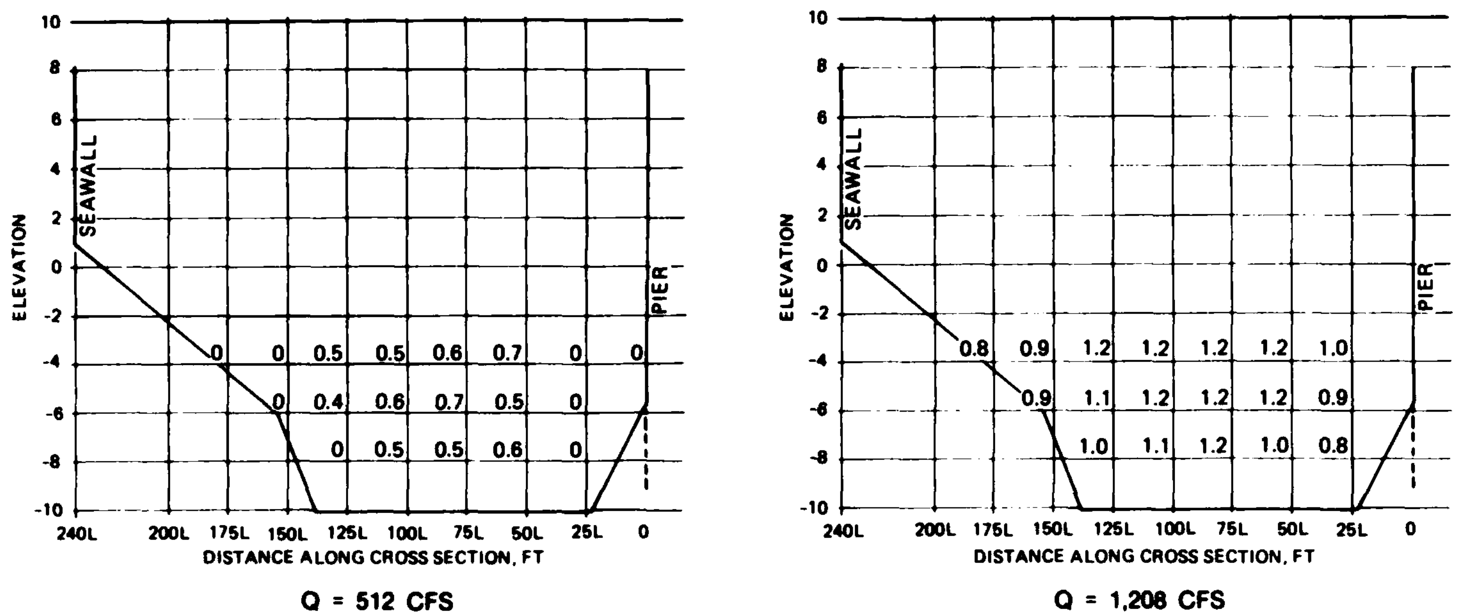
SURFACE VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL

DISCHARGE = 4,039 CFS
TIDE EL = 10.0
TUNNEL N = 0.010

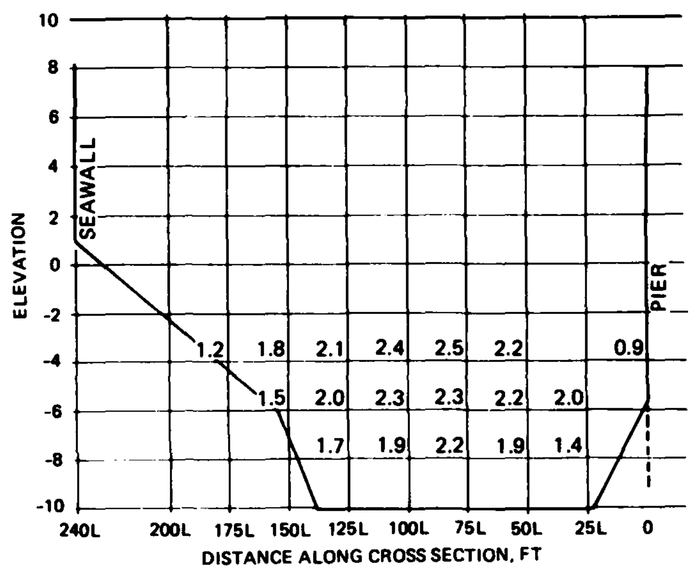


MIDDEPTH VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 4,039 CFS
TIDE EL = 10.0
TUNNEL N = 0.010

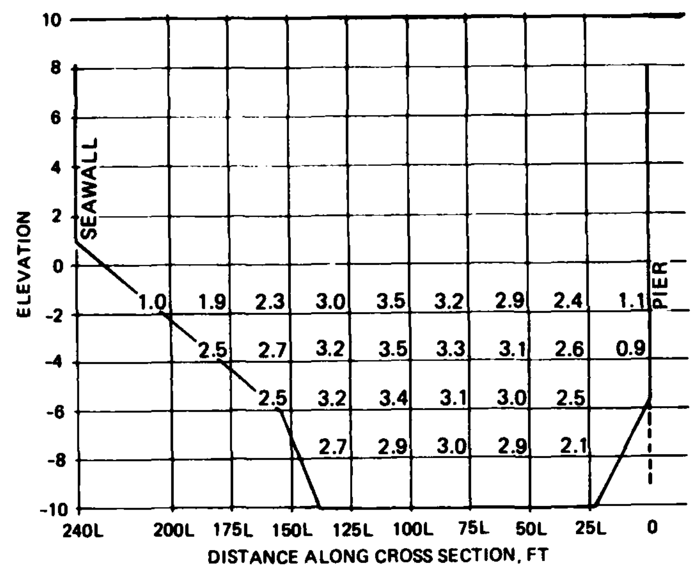

BOTTOM VELOCITIES
SHELDRAKE RIVER TUNNEL
1:25-SCALE MODEL
DISCHARGE = 4,039 CFS
TIDE EL = 10.0
TUNNEL N = 0.010
SCALES
50 0 50 100 FT
5 0 5 10 FT/SEC
A
A
N



VELOCITY CROSS SECTION AT HARBOR PIER
SHELDRAKE RIVER TUNNEL
TIDE EL = -2.7
TUNNEL n = 0.010
DISCHARGE = 512 AND 1,208 CFS

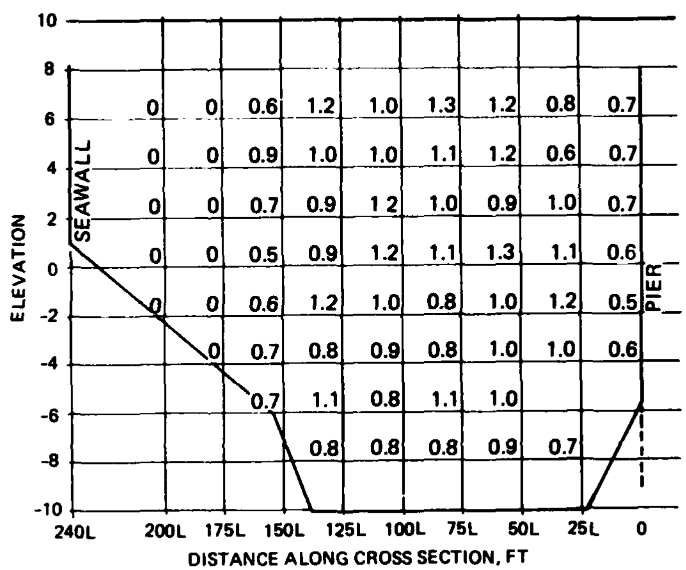


Q = 2,551 CFS

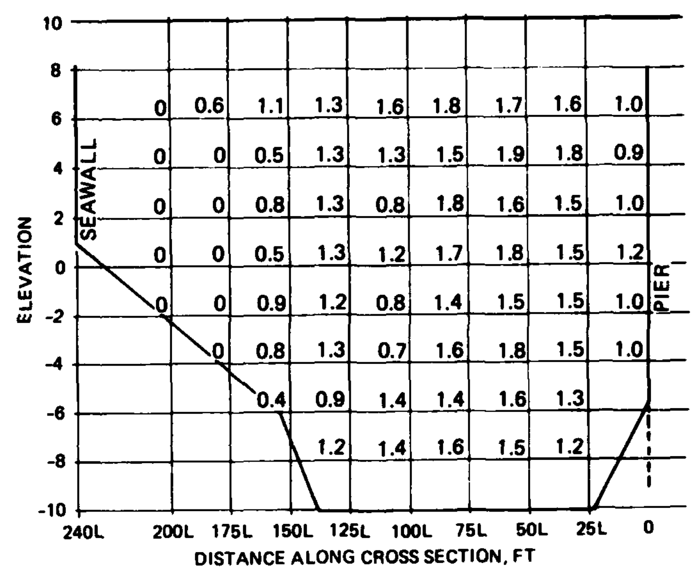


Q = 4,039 CFS

VELOCITY CROSS SECTION AT HARBOR PIER
SHELDRAKE RIVER TUNNEL
TIDE EL = -2.7
TUNNEL n = 0.010
DISCHARGE = 2,551 AND 4,039 CFS

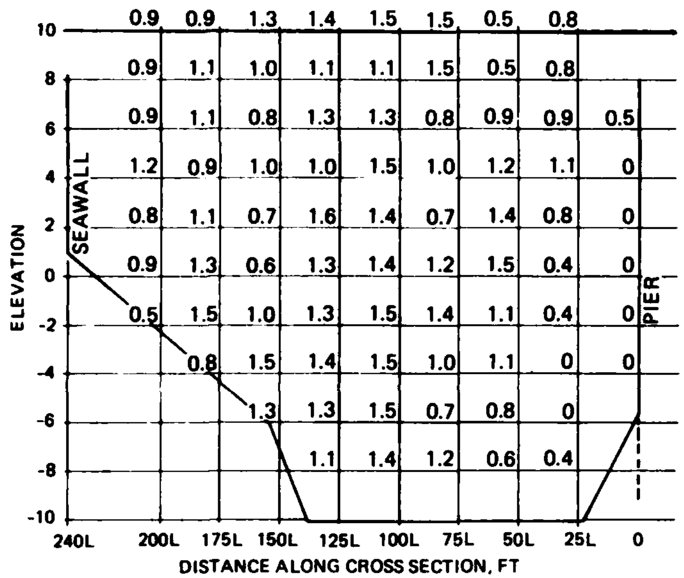


Q = 2,551 CFS

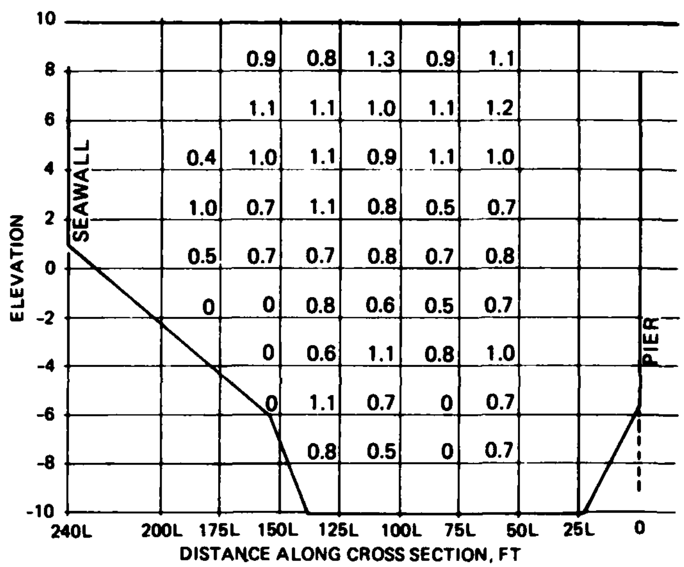


Q = 4,039 CFS

VELOCITY CROSS SECTION AT HARBOR PIER
SHELDRAKE RIVER TUNNEL
TIDE EL = 6.7
TUNNEL n = 0.010
DISCHARGE = 2,551 AND 4,039 CFS

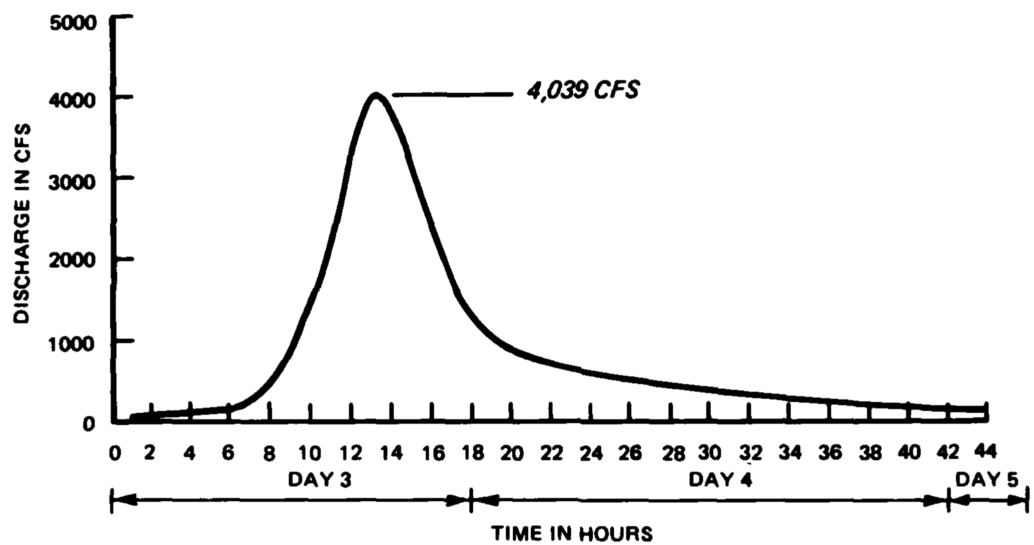


Q = 2,551 CFS



Q = 4,039 CFS

VELOCITY CROSS SECTION AT HARBOR PIER
SHELDRAKE RIVER TUNNEL
TIDE EL = 10.0
TUNNEL n = 0.010
DISCHARGE = 2,551 AND 4,039 CFS



STANDARD PROJECT FLOOD SPF
SHELDRAKE RIVER TUNNEL

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