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TECHNICAL REPORT HL-84-3

RIPRAP PROTECTION ON NAVIGABLE WATERWAYS

by

Stephen T. Maynord

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March 1984

Final Report

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Prepared for Office, Chief of Engineers, U. S. Army
Washington, D. C. 20314

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER
Technical Report HL-84-3

2. GOVT ACCESSION NO.

3. RECIPIENT'S CATALOG NUMBER

4. TITLE (and Subtitle)
RIPRAP PROTECTION ON NAVIGABLE WATERWAYS

5. TYPE OF REPORT & PERIOD COVERED
Final report

6. PERFORMING ORG. REPORT NUMBER

7. AUTHOR(s)
Stephen T. Maynord

8. CONTRACT OR GRANT NUMBER(s)

9. PERFORMING ORGANIZATION NAME AND ADDRESS
U. S. Army Engineer Waterways Experiment Station
Hydraulics Laboratory
P. O. Box 631, Vicksburg, Miss. 39180

10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS

11. CONTROLLING OFFICE NAME AND ADDRESS
Office, Chief of Engineers
U. S. Army
Washington, D. C. 20314

12. REPORT DATE
March 1984

13. NUMBER OF PAGES
54

14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)

15. SECURITY CLASS. (of this report)
Unclassified

15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)
Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)
Shore protection (LC)
Rock mechanics (LC)
Channels (Hydraulic engineering) (LC)
Embankments (LC)
Riprap (WES)

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

Three separate physical model investigations concerning riprap protection in navigable waterways are reported herein. The first investigation was a site-specific study of riprap failure and repair along a reach of the Sacramento River Deep Water Ship Channel. The second study was a general investigation to address the required riprap size when placed in an environment subject to propeller wash from relatively large horsepower towboats. The third study was a site-specific study of the required riprap size in the bottom of

(Continued)

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Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

20. ABSTRACT (Continued).

the proposed Seabrook Lock. Towboats entering and exiting the Seabrook Lock are usually much smaller than those used in the general investigation.

PREFACE

The model investigation of the Sacramento River Deep Water Ship Channel and the general investigation of Effects of Propeller Wash on Riprap Stability were conducted as part of the Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32 Program. The model investigation of the Seabrook Lock Bottom Riprap was authorized by the U. S. Army Engineer District, New Orleans, on 6 November 1980. These studies were conducted by personnel of the Hydraulics Laboratory, U. S. Army Engineer Waterways Experiment Station (WES), during the period 1978 to 1980 under the direction and supervision of Messrs. H. B. Simmons, Chief of the Hydraulics Laboratory, and J. L. Grace, Jr., Chief of the Hydraulic Structures Division and Manager of the Section 32 Program Hydraulics Laboratory Research. The tests were conducted by Messrs. S. T. Maynord and H. R. Smith under the supervision of Mr. N. R. Oswalt, Chief of the Spillways and Channels Branch. This report was prepared by Mr. Maynord.

Commanders and Directors of WES during this testing program and the preparation and publication of this report were COL John L. Cannon, CE, COL Nelson P. Conover, CE, and COL Tilford C. Creel, CE. Technical Director was Mr. F. R. Brown.

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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
feet	0.3048	metres
feet per second	0.3048	metres per second
feet per second per second	0.3048	metres per second per second
inches	25.4	millimetres
knots (international)	0.514444	metres per second
miles per hour (U. S. statute)	1.609344	kilometres per hour
pounds (force)	4.448222	newtons
pounds (mass)	0.4535924	kilograms
horsepower	1.014	horsepower (metric)

RIPRAP PROTECTION ON NAVIGABLE WATERWAYS

PART I: MODEL INVESTIGATION OF SACRAMENTO RIVER DEEP WATER SHIP CHANNEL

Introduction

1. Under the Section 32 program,* limited hydraulic research was conducted to study the effects of navigation on bank stability in a confined waterway. This effort included a review of literature and both site-specific model testing and model study of certain problems applicable to a wide range of conditions. This site-specific study was funded under the Section 32 program to better understand failure mechanisms in confined waterways for application to other confined waterways located nationwide.

The Prototype

2. A site-specific 1:30-scale model study was conducted of a reach of the Sacramento River Deep Water Ship Channel (SRDWSC) which has experienced several riprap failures along the levees of the channel. The SRDWSC has been in operation since 1963. Table I-1 shows a sampling of vessels using the SRDWSC. Also shown in the last column of this table is the value of the cross-section ratio, η , which is the ratio of the waterway cross-sectional area to the submerged cross-sectional area of the ship. Many of the vessels using the SRDWSC result in a cross-section ratio as low as 4 with the average ratio being 4.8. Feuerhake et al. (1969) reports "tests with ship speeds up to 15 km/hr (9.3 mph**) showed that the cross-section ratio, η , should be at least 7. Economic bank revetments then provide protection against forces."

3. A map of the SRDWSC is shown in Plate I-1. The particular area of concern is along the east levee from about mile 18.6 to mile 21.0. The as-built (1963) channel cross section in this reach is shown in Plate I-2. In May 1979, five cross sections were surveyed within the study reach and increases in cross-sectional area ranged from 13 to 35 percent with the average

---

* Public Law 93-251, The Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, as amended by Public Law 94-587, Sections 155 and 161, October 1976.

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

increase being 23 percent. Significant bank protection maintenance work has been required in this reach since 1967 as listed below:

Specification No.	Issue Date	Quarrystone Thickness in.	Quarrystone Max Size lb	Bedding Layer Thickness	Embankment Material	Replaced Under Later Specs.
3418	1967	12	300	6 in.	Uncompacted	Yes
3572	1969	12	300	6 in.	Uncompacted	Yes
4010	1971	18	300	9 in.	Uncompacted	Yes
4284	1972	18	300	9 in.	Uncompacted	Yes
4851	1974	18	400	9 in.	Compacted	No
4958	1976	18	350	9 in.	Compacted	No
5296	1977	18	350	Filter cloth	Compacted	No

Note: From Jones (1980).

4. During spring inspections of 1979, damage was observed at the sites constructed in 1974 and 1976. According to Jones (1980), three possible failure mechanisms are indicated.

a. Improper gradation of quarrystone versus filter material. This allows wave action to remove filter material and expose the embankment to wave and seepage erosion which leads to stone failure.

b. Saturation of uncompacted embankment material. Saturation by waves and tidal action resulting in subsequent seepage moving embankment material through the filter material which leads to stone protection failure.

c. Inadequate design wave. If stone protection is being subjected to larger waves than presently designed for (design wave $\cong$ 4.0 ft), then the stone layer may not be adequate to withstand the wave force.

5. In the summer of 1979, the U. S. Army Engineer Waterways Experiment Station (WES) was asked to review the bank protection design relative to:

a. Quarrystone gradation and layer thickness.

b. Filter material gradation and layer thickness.

c. Embankment material selection and compaction.

After review by WES and discussion with the U. S. Army Engineer District, Sacramento, a design was developed and construction was completed in 1981. This design consists of a compacted embankment overlaid with 6 in. of masonry

sand which in turn is overlaid with a 12-in.-thick granular filter. The entire system is covered with a 27-in.-thick stone protection layer all placed on a 1V-on-3H slope. The maximum size stone was increased to 1,300 lb. The design was developed using the following design guidance:

(1) EM 1110-2-2300, 1 Mar 1971, updated by ETL 1110-2-222 dated 10 July 1978.

(2) EM 1110-2-1901, Part CXIX, Chapter 1, February 1952.

(3) Shore Protection Manual, 1977, Volume 2.

The gradation limits for the recommended replacement stone and for specification No. 4851 (placed in 1974) which experienced damage are as follows:

	Weight lb	Finer by Weight Percent
Specification No. 4851	400	100
	200	70-90
	100	30-70
	50	20-50
	20	10-30
	5	0-10
Replacement gradation	1,300	100
	1,000	80-90
	500	50-70
	100	10-30
	50	0-10

6. The existing speed limit with the study reach for all oceangoing craft is as follows:

a. When going against a current of 2 knots or more, the maximum speed over the bottom shall not exceed 5 knots (5.8 mph).

b. When going with the current, in slack water, or against a current of 2 knots or less, the maximum speed through the water shall not exceed 7 knots (8.05 mph).

Past speed surveys have shown that the average speed of all vessels was 8.4 mph.

Purpose of the Model Study

7. The purpose of the model study was to determine the mode of failure of the existing riprap along the SRDWSC and to evaluate the adequacy of the rock to be used in repairing the damaged sections. A side benefit of this study is the opportunity for future model-prototype correlation of results. Results of this study are applicable specifically to the SRDWSC and generally to similar confined waterways.

Pertinent Literature

8. The study of navigation effects such as drawdown, surges, and waves created by ships in a confined waterway has received considerable attention in the literature. Cases similar to the SRDWSC having a horizontal berm have received limited attention, but much of the research based on trapezoidal channels without berms can yield information pertinent to the berm situation.

9. Passage of large ships in confined channels results in significant drawdown levels but relatively small waves compared with the smaller but much faster vessels that induce very little drawdown. Prototype measurements on the St. Lawrence Seaway are reported (Gelencser 1977) which confirm and quantify these observations. A plot of the time-history of the water surface for a relatively large ship is shown in Plate I-3. The cross-section ratio for the trapezoidal channel and vessel was 5.3 and the speed of the vessel was 7.6 mph. Maximum drawdown below the static water level was 2.1 ft.

10. Model experiments were also reported (Gelencser 1977) which were directed at bank stabilization. "... the cost studies preconcluded have shown that the riprap protection is the cheapest one and therefore became the only protection investigated in the model." These tests showed that a design vessel of 730-ft length traveling at 14 to 15 mph would fail riprap as large as 5 ft in diameter. They also found that by limiting speeds to 10 mph, riprap material of 30 in. (maximum size) would not be damaged. The cross-section ratio for these tests was 7.0.

11. Various studies have addressed the drawdown that occurs with ship passage (Gelencser 1977, Balanin et al. 1977, Dand and White 1977, Van de Kaa 1978, and Lee and Bowers 1947). Most relate drawdown as a function of channel depth, ship speed, and cross-section ratio.

12. Research connected with the design of the Kiel Canal in Europe revealed that the final channel size was more dependent on bank and bed stability than on navigability requirements (Wiedemann 1978).

13. Dand and White (1977) report on surge waves that result from drawdown effects in the Suez Canal. The following excerpt is from Dand and White (1977):

> The present western side of the Suez Canal has a horizontal berm which runs out from the bank at a level between 1 m and 2 m below water level. One disadvantage of a horizontal berm [see cross section, Plate I-4] is that under certain circumstances surge waves can be created by vessels in transit. Conditions which increase the likelihood of surge waves include shallow depths of water over a berm, h_b, high speeds of transit, and significant drawdown of the water surface caused by the passing ship.
>
> At slow speeds of transit there is a gradual and small fall in water level as the bow of the vessel passes and a gradual increase as the stern reaches the point under consideration. As the speed of transit increases, the amount of drawdown increases and at some stage a weak undular disturbance is initiated over the berm. At even higher speeds this weak undular disturbance is transformed into a surge wave which travels along the berm roughly in line with the stern of the vessel.
>
> It is unwise to design new banks which will induce damaging surge waves under the operating conditions anticipated for the new canal. Hence it was desirable to be able to predict when these effects would occur and to develop design criteria which would avoid them.
>
> A semiempirical approach to the problem, utilizing the observed drawdown characteristics, indicated a general relationship between the Froude number (based on the speed of the ship and the undisturbed depth over the berm), the blockage ratio and the type of wave disturbance. The plot is given in nondimensional form in [Plate I-4].

Dand and White (1977) also verify "bank erosion is caused by drawdown from large vessels and free waves from small vessels moving at a higher speed."

14. Lee and Bowers (1947) report on restricted channel tests for the Panama Canal. Extensive drawdown measurements were made for a wide range of channel depths, widths, ship speed, and relative position in the cross section.

15. Helm and Wöltinger (1953) state that:

> The speed of return-flow gives us an idea of the attack

on the bottom to be expected. It should be restricted by imposing a speed-limit on navigation, according to the nature of the soil. For the present report, the speed of the return flow has been fixed at 1 m/sec [3.3 ft/sec].

Helm and Wöltinger (1953) state that the limiting speed of a ship in a canal is a function of the speed of the translation wave and the ratio of the waterway area to the submerged ship area (η).

16. Jansen and Schijf (1953) present curves in Plate I-5 for determining the water-surface drawdown and the speed of return flow as a function of the reciprocal of the cross-section ratio and the ship Froude number.

17. Tenaud (1977) reports on the damaging effects of waves in a navigation channel. Extensive model testing was conducted over a wide range of channel shapes, rock sizes, ship sizes, and ship speeds. Techniques are presented for estimating the wave heights as well as the protective stone required.

The Model Study

Description

18. A straight 0.5-mile-long reach of the SRDWSC was reproduced in the model at a scale of 1:30 (Figure I-1). The as-built cross section (Plate I-2) was used throughout the model study with the exception that the replacement rock was tested at a 1V-on-3H side slope as proposed for the prototype. Only the east berm (left side of channel) was reproduced in the model. The correct channel area was maintained by a small adjustment of the bottom width of the channel. The channel used in the study was a slack-water channel; therefore the small tidal-induced velocities that occur in the prototype were not reproduced in the model. The berm and side slopes on the east bank were molded in concrete with sand used in the bottom of the channel.

19. The 1:30-scale model ship used in the study (Figure I-2) represented a tanker having a prototype length of 660 ft, beam of 102.6 ft, and drafts up to 40 ft. The ship is self-propelled with an operator on board. The draft of the model ship was varied to represent different ship displacements. This allowed variation of the ratio η (channel cross-section area/submerged ship cross-section area) during the test program. The model ship was capable of prototype speeds up to 11 mph. Ship speeds were determined by

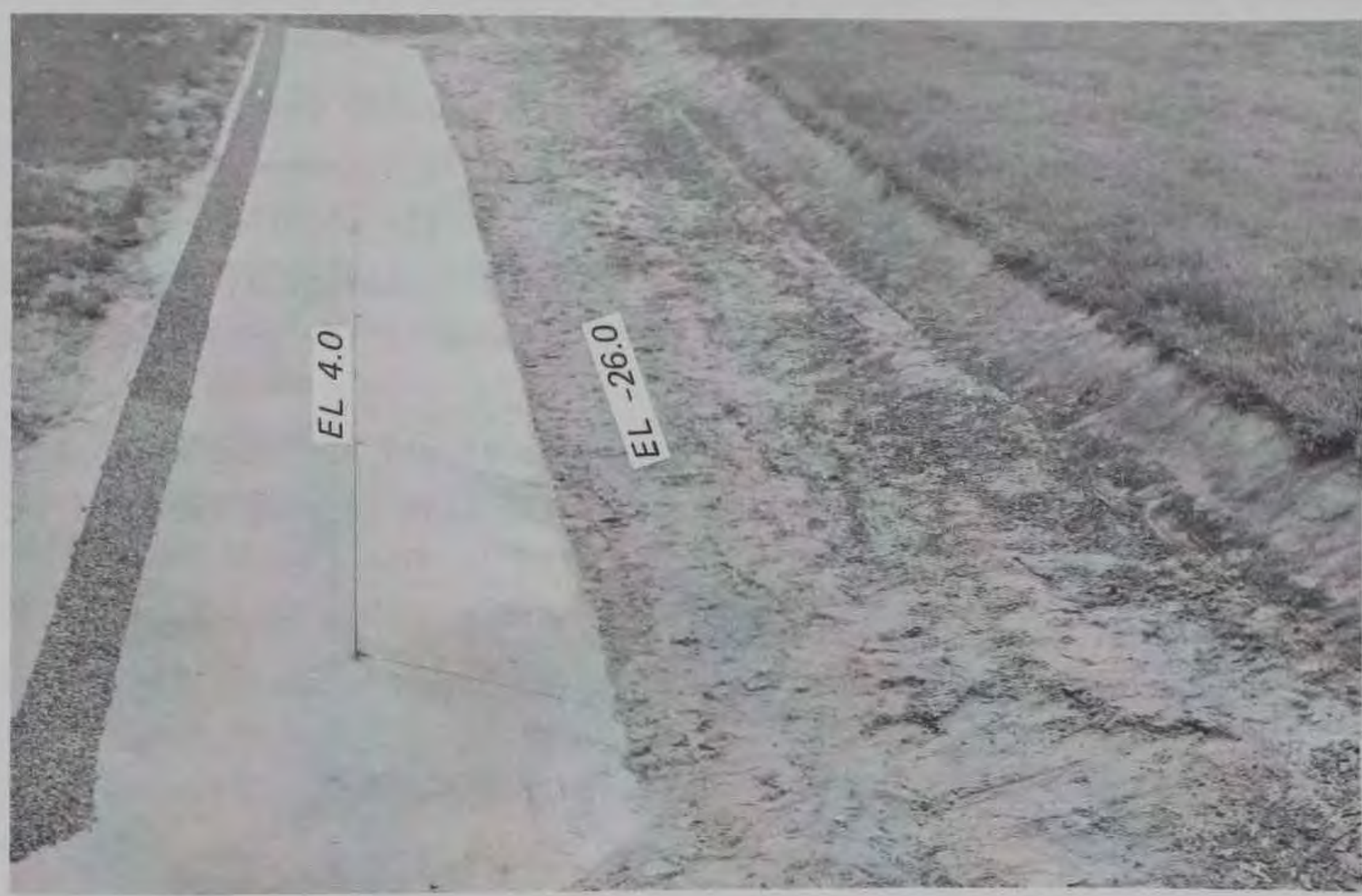

Figure I-1. 1:30-scale model of 0.5-mile reach of SRDWSC

Figure I-2. 1:30-scale ship in SRDWSC model

measuring the time required for the ship to traverse the middle 0.25-mile length of the test reach.

20. Crushed limestone was sieved and mixed to the proper gradation to simulate the prototype riprap. The specific gravity of the crushed limestone used in the model was 2.67. Movement of riprap in the model was determined by inspection during and immediately following each passage of the model ship. The W_{50}'s simulated in the model were 90 lb and 340 lb for gradation No. 4851 and the replacement riprap, respectively.

21. A continuous recording water-level detector was used to monitor the water level over the berm at the midpoint of the test section as shown in Figure I-2.

Scaling relations

22. The equations of similitude based on Froude's law

$$\text{Froude No. Model} = \text{Froude No. Prototype} = \frac{V}{\sqrt{gL}}$$

where

V = velocity, ft/sec

g = acceleration due to gravity, ft/sec^2

L = characteristic length, ft

were used to express mathematical relations between the dimensions and the hydraulic quantities of the model and prototype. The following relations were used:

Dimension	Ratio	Scale Relation
Length	L_r	1:30
Time	$T_r = L_r^{1/2}$	1:5.48
Velocity	$V_r = L_r^{1/2}$	1:5.48
Weight	$W_r = L_r^3$	1:27,000

However, frictional resistance of ships is dependent on Reynolds number

$$R = \frac{VL}{\nu}$$

where

R = Reynolds number

V = velocity, ft/sec

L = characteristic length, ft

ν = kinematic viscosity, ft^2/sec

and the model and prototype Reynolds numbers are different when the same fluid is common to both model and prototype and the Froude criteria are used as the basis of similitude. Greater relative thrust must be applied in the model to overcome the greater friction in the model. The drag coefficient as a function of ship Reynolds number is shown in Figure I-3. Also shown in this figure is the point of the curve for a typical prototype and the point on the curve for the 1:30-scale model used in this investigation. Drag coefficients in the model are relatively close to those of the prototype, and only a small increase in thrust in the model was required. Water-level drawdown and return surge or a classical bore are the most likely failure mechanisms in the SRDWSC. Drawdown is a function of the cross-section ratio and the ship speed. The increase in thrust required in the model to simulate a given prototype speed should not affect the similarity of the drawdown phenomenon.

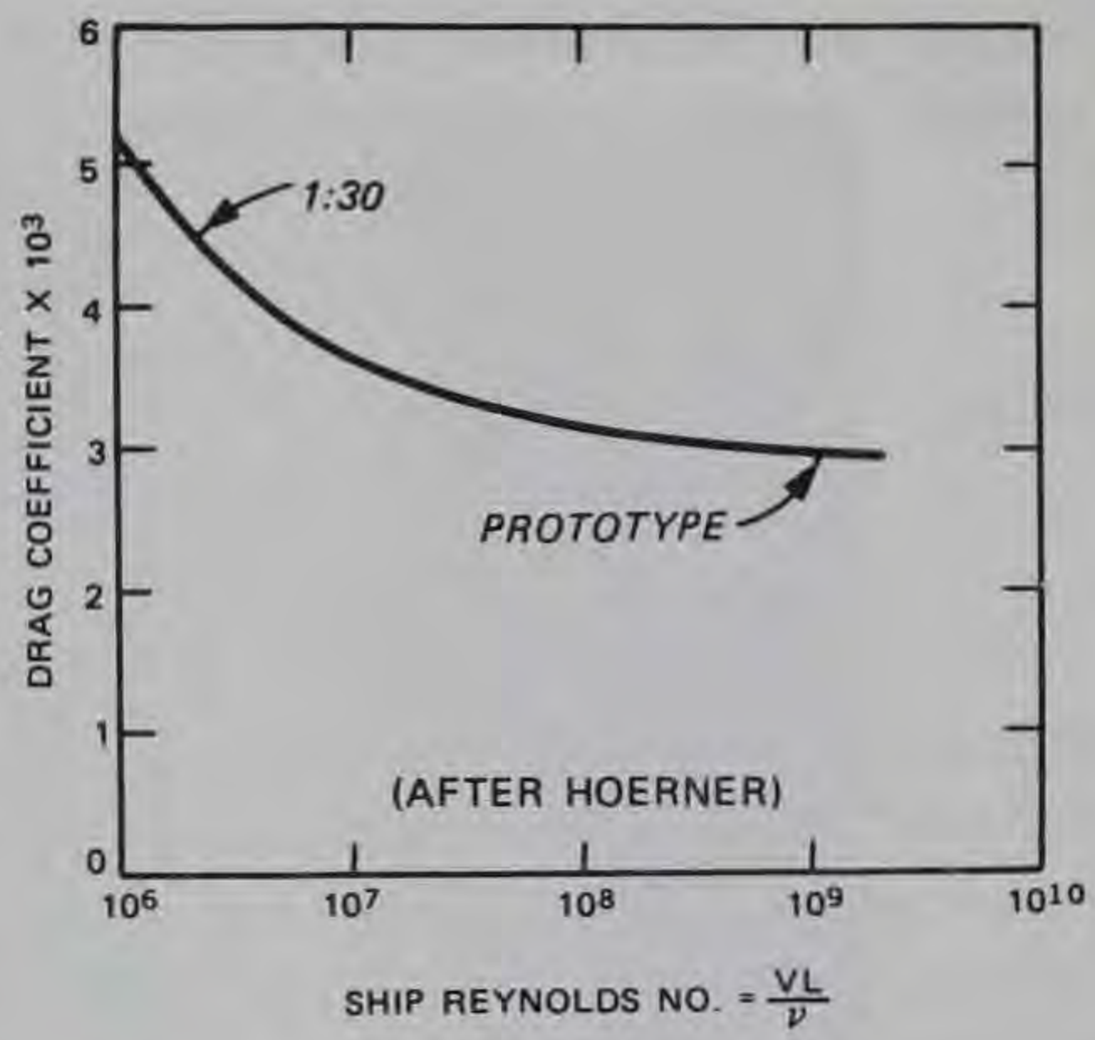

Figure I-3. Drag coefficient as a function of ship Reynolds number (from Huval and Pickering 1978)

23. Other scale effects are present relative to the rock movement resulting from the return surge or wave occurring on the berm. Dai and Kamel (1969) compared rock stability of rubble-mound breakwater models constructed with a wide range of model Reynolds numbers. Their tests indicated that scale effects due to viscous forces were significant below a certain Reynolds number. Unlike the case of the breakwater, the speed of the surge on the berm affects the stability of the riprap. The mechanics of a moving surge or bore departs considerably from that of a wave train. The forces generated by the surge moving parallel to the bank line are more analogous to forces generated by flow over a channel boundary than to wave-generated forces. Unfortunately, certain viscous scale effects are present when testing riprap stability in a

channel flow environment in models not having sufficiently large Reynolds number. The particle Reynolds number is defined as

$$R = \frac{Vd_{50}}{\nu}$$

where

V = average velocity, ft/sec

d_{50} = 50 percent riprap size, ft

ν = kinematic viscosity, ft^2/sec

O'Loughlin et al. (1970) recommend a particle Reynolds number greater than 2.5×10^3 to minimize Reynolds number scale effects. The particle Reynolds number for the model using a surge speed of 8 mph to represent the average velocity and the d_{50} for gradation No. 4851 is 7×10^3, indicating minimal Reynolds number scale effects.

Test Results

Existing design

24. The riprap plan based on gradation No. 4851 was placed in the model for the initial test series. Two values of the cross-section ratio, η, were tested for each of three different water depths over the berm. For each test, the speed of the ship was varied and the drawdown of water over the berm was monitored and recorded. This drawdown was the difference between the static water level and the minimum water level that occurred during the passage of the ship. Results are shown in Plates I-6 and I-7 for cross-section ratios of 6.1 and 4.3, respectively. A typical trace of the water level as a function of time for a 4-ft depth over the berm and a cross-section ratio of 4.3 is shown in Plate I-8. The speed of the ship for this test was 8.8 mph. The speed at which the water surface falls can be a significant factor in the stability of the riprap on the levee. For the condition shown in Plate I-8, a fall of 3.5 ft occurs in approximately 1 min in the prototype. This rapid drawdown can result in removal of bank material through the revetment if adequate filters are not installed. The drawdown over the berm is shown in Figure I-4.

25. During these tests, the ship speed at which the riprap on the levee began moving was observed for each water depth over the berm and each cross-section ratio for gradation No. 4851. Results of these rock movement

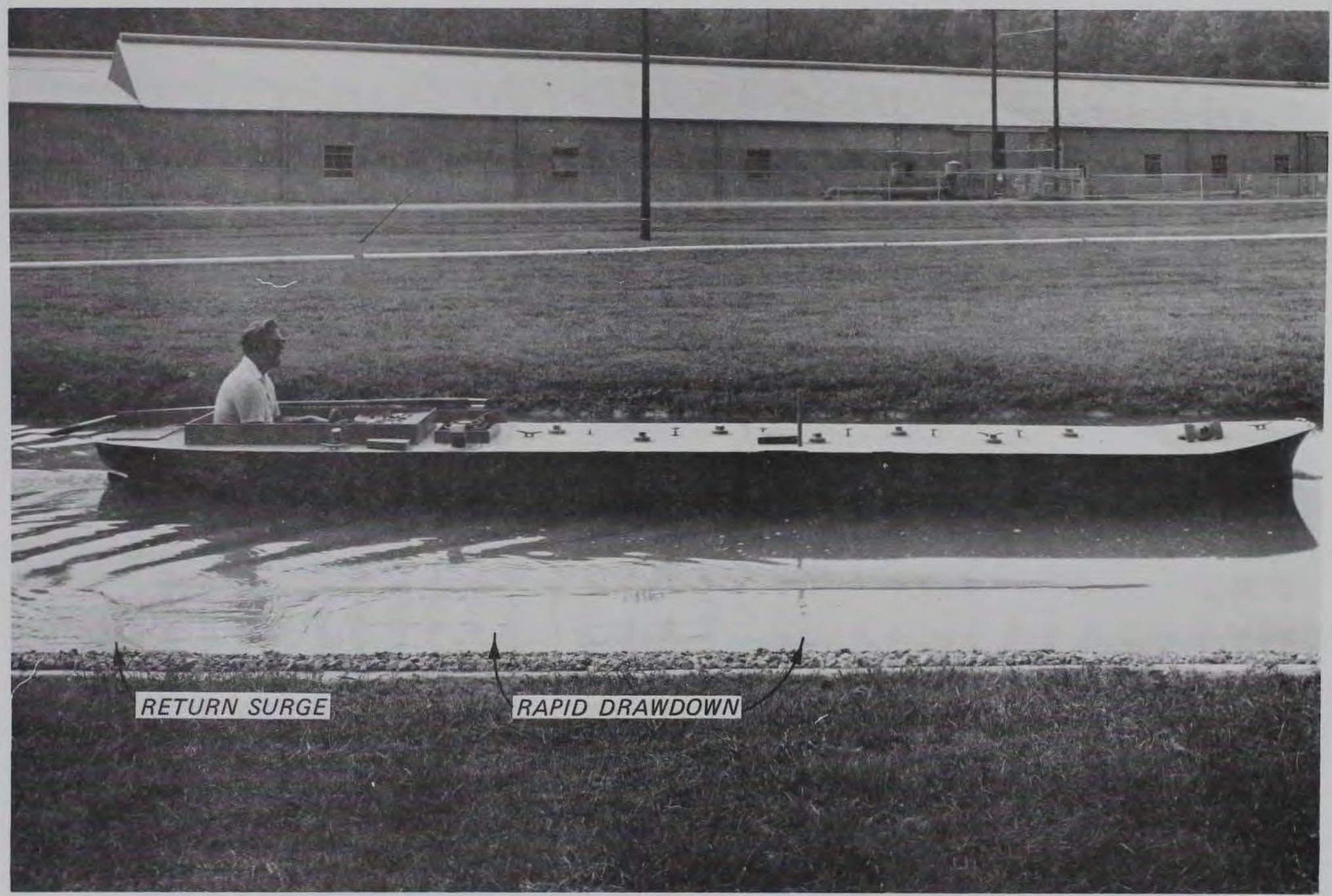

Figure I-4. Rapid drawdown followed by return surge

observations are shown in Plate I-9. The surge or bore which caused the rock movement is shown in Figure I-4.

Alternate designs

26. Limited testing was conducted to evaluate the effects of gabion dikes on the levee riprap stability. Gabion dikes were placed along the berm (Plate I-10) at 150-ft intervals. These dikes were 3 ft high and 40 ft in length. Tests were conducted for an η ratio of 4.3 and a 4-ft depth over the berm. Results of the drawdown measurements are shown in Plate I-11. Only a small decrease in drawdown was observed with the gabion dikes. However, the ship speed at which rock movement occurred was increased from approximately 7.8 to 9 mph with the 4-ft depth over the berm and the cross-section ratio of 4.3

27. The gabion dike spacing was increased to 300 ft with the length and height remaining 40 ft and 3 ft, respectively. Rock movement tests were conducted with this design, and the ship speed at which rock movement began remained unchanged from the original design without dikes.

28. The third alternate design tested was an attempt to reduce the rapid drawdown occurring over the berm and particularly on the levee. A gabion fence was constructed in the model as shown in Plate I-12. This design stopped the drawdown at the top elevation of the gabion fence and none of the rock was moved at speeds up to 9.8 mph. However, this design reduced the effective channel area and the cross-section ratio and resulted in more adverse conditions out in the channel, particularly at the toe of the gabion fence. One engineer observing tests noted that a gabion levee revetment could be constructed with the same amount of gabions required to construct the gabion fence and thus avoid any reduction in the channel area.

29. The fourth alternative tested was a 20 percent increase in channel area which changed the cross-section ratio with the largest ship from 4.3 to 5.2. The resulting drawdown plot as a function of ship speed is shown in Plate I-13. Rock movement tests indicated that only a small increase from 7.8 mph with the original design to $\cong$ 8.5 mph with the increased channel area could be achieved. These tests were conducted with a 4-ft depth of water over the berm.

Proposed replacement design

30. The riprap design proposed for repair of the prototype was then placed in the model. Rock movement tests were conducted for a cross-section

ratio of 4.3 with water depths over the berm of 2, 4, and 6 ft. The replacement rock was moved at ship speeds of ≅ 1.0 mph faster than the speeds for the original No. 4851 gradation (Plate I-9) for all depths over the berm.

Discussion of Results and Conclusions

31. Model tests show that surging and rapid drawdown in the SRDWSC are caused by the low ratio of waterway cross-sectional area to submerged ship cross-sectional area in conjunction with the typical speed of the using vessels. The average cross-section ratio for the channel and all ships sampled was 4.8. Model tests showed rock movement began along the levee with ship speeds as low as 8 mph for a cross-section ratio of 6.1. Model results are valid for the original as-built cross section. Results of model tests with a 20 percent increase can be used to estimate the effects of the altered prototype cross section (paragraph 3). Based on these tests, rock movement on the levee with the enlarged section will occur at ship speeds of 0.5 to 1.0 mph faster than with the as-built section.

32. The failure mechanisms observed in the model study were similar to those stated by Jones (1980). The rapid drawdown that occurs as the ship passes can lead to riprap failure if adequate filters are not provided beneath the revetment. At the highest ship speeds and ship displacements, the drawdown can be equal to the depth of water over the berm. The surge or bore that follows the rapid drawdown leads to rock revetment failure. The surge or bore height always exceeded the bow or stern waves coming off the ship. The surge moved along the berm approximately equal to the location of the stern of the ship. The rock movement curves approximate the point at which rock moved off the levee and onto the berm. For determining the speed at which revetment failure should not occur, a safety factor should be incorporated by selecting a speed less than the speed at which initial rock movement occurred.

33. The gabion dike (150-ft spacing) alternative did not solve the rapid drawdown problem but was effective in allowing an increase of ship speed at which rock movement was initiated. The gabion fence alternative solved both the rapid drawdown and the rock movement due to the surge or bore but reduced the cross-sectional channel area in an already critically confined channel.

34. The proposed larger replacement riprap will help solve the problem

of riprap failure if past failures have been caused by the action of the surge or bore. If past failures have been caused by rapid drawdown, the filters proposed for the replacement riprap may solve the problem of failure due to rapid drawdown.

35. Only a small increase (≅ 1.0 mph) in ship speed above the speeds shown in Plate I-9 for gradation No. 4851 can be tolerated without movement of the replacement riprap. This is surprising since the average diameter of the replacement stone is 50 percent greater than the existing riprap. The reason for this small increase is the small cross-section ratio of the SRDWSC to the using vessels. The drawdown curves shown in Plates I-6 and I-7 show a large increase in drawdown (and therefore surge or bore height) for a relatively small increase in ship speed. Only an increase in channel area or a decrease in ship speed can result in favorable conditions within the channel and along the riprapped levees with the using vessels and the proposed replacement riprap.

36. According to research conducted on the Suez Canal a horizontal berm can result in severe surging if the depth over the berm is shallow or ships travel at high speeds. These breaking surge waves or bores occur in the SRDWSC and may result in rock failure along the levee at the higher ship speeds.

37. Drawdown, surge or bore height, wave action, and rock failure would be reduced by enforcement of longer travel times and/or lower speed limits.

38. Results of this study are valid quantitatively to only the specific channel dimensions of the SRDWSC. Results are valid qualitatively to other confined channels. Qualitative application of these results to large navigable waterways that cannot be considered as confined channels is not valid.

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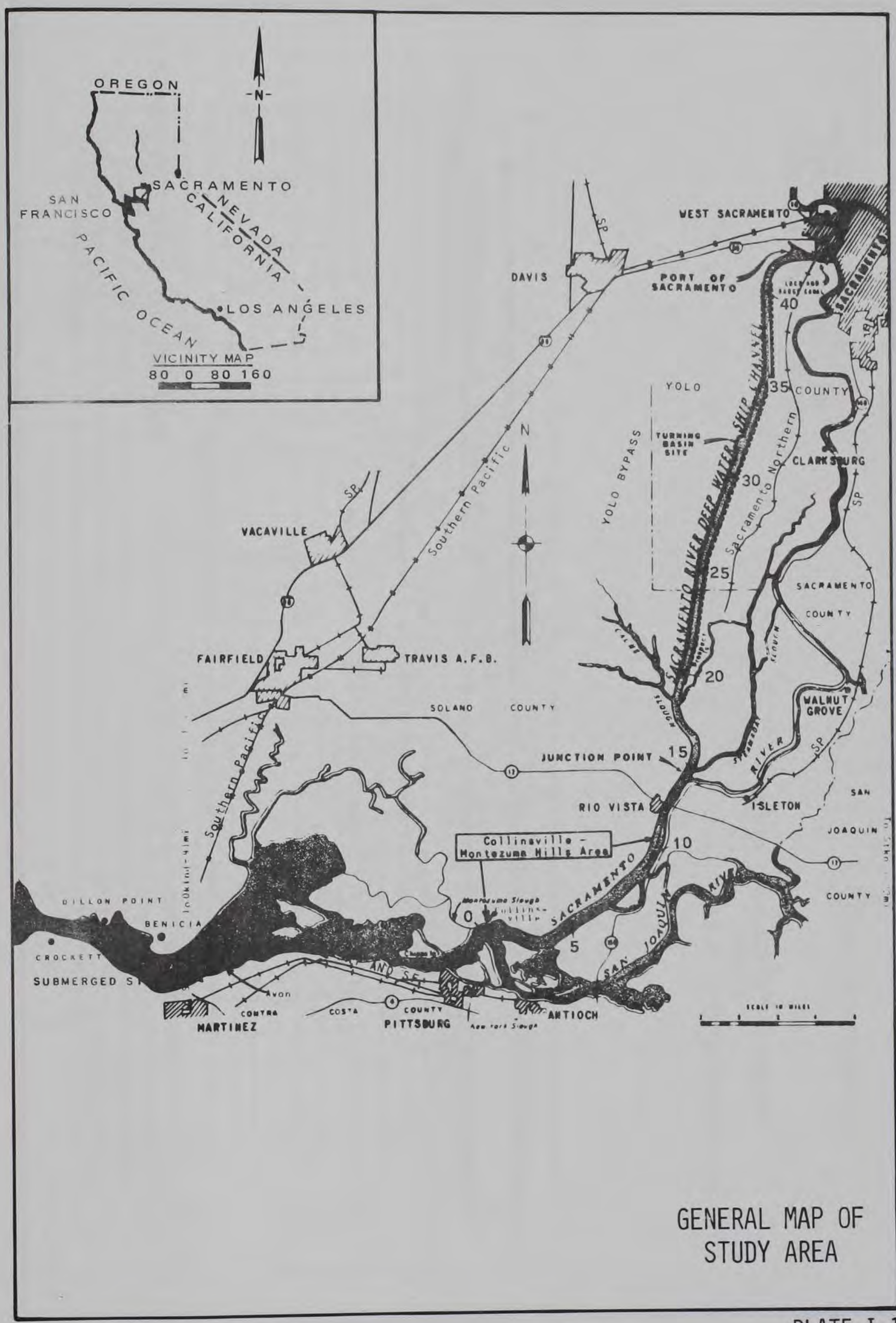
Table I-1

Sample of Vessels Loading at the Port of Sacramento

1976-79

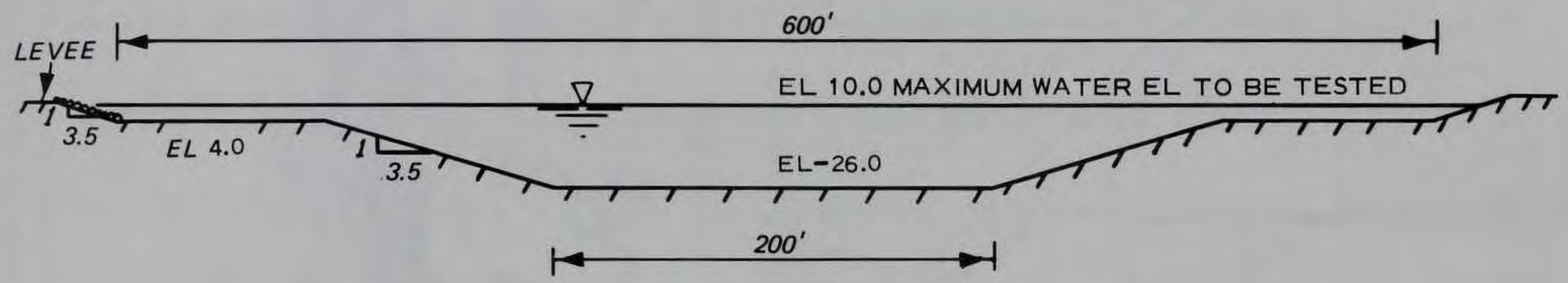
dwt	Length ft	Length in.	Beam ft	Beam in.	Design Draft ft	Design Draft in.	Sailing Draft ft	Sailing Draft in.	Tons Loaded	η Ratio
25,040	585	8	75	1	34	1	30	6	20,235	5.1
22,593	539	2	75	2	32	4	30	6	21,041	5.1
35,657	655	5	91	5	36	10	31	5	30,166	4.1
16,588	557	10	86	0	29	10	30	6	16,421	4.1
52,225	700	0	96	2	41	10	30	0	33,075	4.0
26,900	581	4	75	0	34	3	31	0	23,681	5.0
38,711	656	0	88	9	36	7	28	0	27,305	4.7
20,203	518	4	76	11	31	3	30	4	18,682	5.0
26,600	600	6	74	8	34	5	30	6	23,422	5.1
60,740	736	3	106	0	41	4	29	2	39,225	3.8
25,604	591	7	75	1	33	6	29	10	20,944	5.2
34,602	604	9	85	9	36	3	28	9	24,182	4.7
52,733	716	9	102	2	39	5	30	10	37,449	3.7
39,796	623	4	90	9	37	3	29	8	29,848	4.3
30,668	623	4	75	6	35	1	30	0	24,556	5.2
19,030	506	0	74	10	30	2	30	4	19,711	5.1
29,168	593	2	91	3	35	1	28	9	22,046	4.4
22,697	544	5	75	1	34	0	30	4	16,204	5.1
16,061	474	6	67	10	30	4	31	0	16,017	5.6
20,520	520	0	74	3	30	1	31	2	19,996	5.0
16,230	534	4	66	7	30	10	30	4	16,436	5.8
40,347	669	11	90	7	38	7	30	5	29,423	4.2
27,306	597	1	75	2	34	11	30	5	21,600	5.1
51,658	655	11	105	8	40	9	30	10	36,904	3.6
29,202	593	2	75	11	35	0	30	3	24,595	5.1
27,593	577	0	75	0	36	1	31	0	21,934	5.0
23,625	555	11	80	10	32	6	31	2	21,197	4.6
37,836	615	9	93	2	35	2	30	6	32,554	4.1
41,035	602	3	90	8	39	4	29	7	29,944	4.4
27,890	564	0	85	2	34	2	30	4	--	4.5
29,709	564	0	85	2	35	4	31	0	--	4.4
27,890	564	0	85	2	34	2	29	6	--	4.6
27,890	564	0	85	2	34	2	29	6	--	4.6
28,939	593	10	95	11	34	4	27	8	--	4.4
24,090	534	4	75	2	34	2	29	6	--	5.3
29,623	574	7	85	5	33	9	30	8	20,609	4.5
19,418	512	4	74	4	31	3	30	5	15,064	5.2
18,820	508	8	75	0	29	4	30	6	14,512	5.1
20,009	502	0	77	6	30	11	30	6	15,804	4.4
18,546	469	10	75	0	30	0	29	2	13,244	5.3
16,549	465	9	71	7	29	10	30	2	13,769	5.4
19,297	512	4	74	4	31	3	31	0	14,603	5.1
25,401	576	11	83	5	31	2	31	6	20,684	4.4
20,000	555	10	75	0	33	8	31	1	17,236	5.0

* Ratio of waterway cross-sectional area to submerged cross-sectional area of the ship.



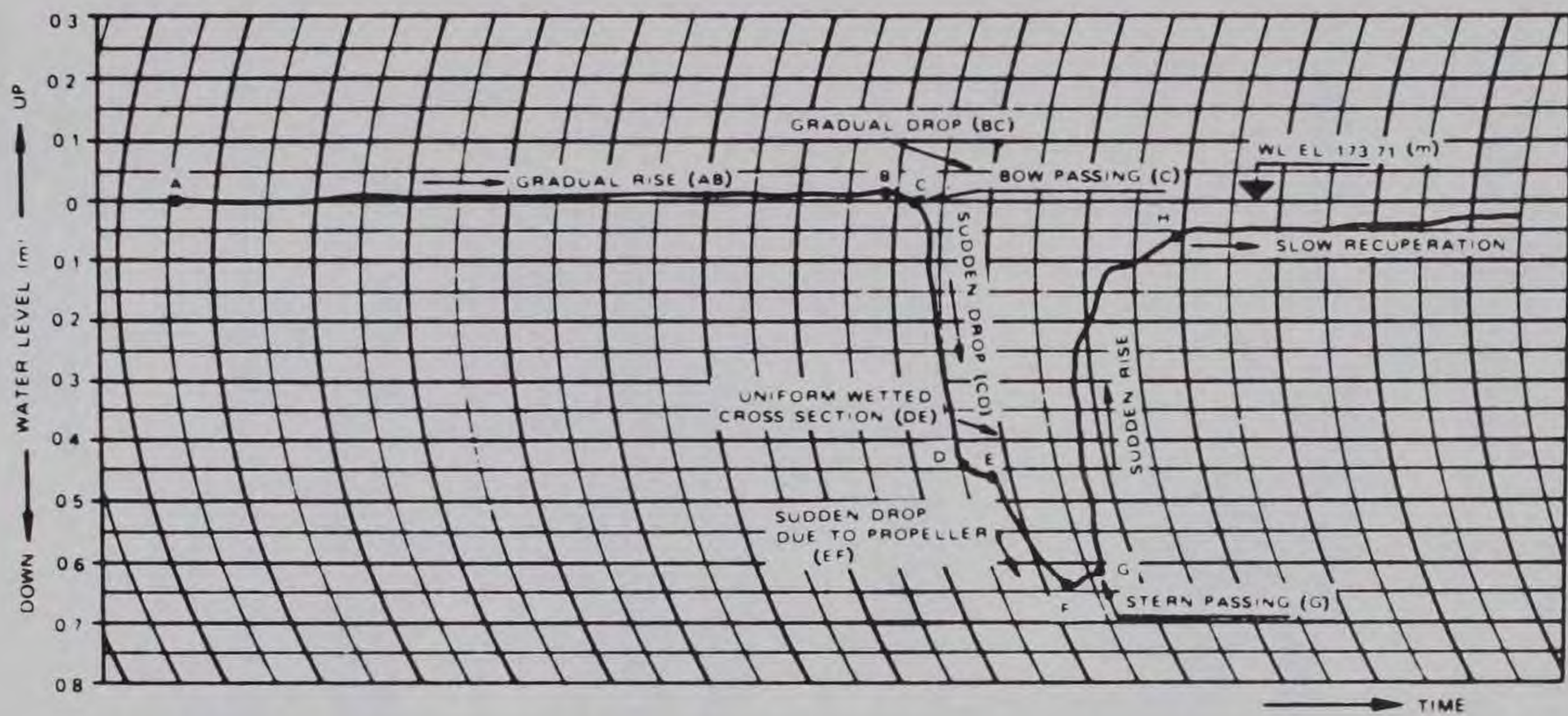
GENERAL MAP OF STUDY AREA

PLATE I-1



NOTE: ELEVATIONS IN FEET NGVD

SACRAMENTO RIVER DEEP WATER
SHIP CHANNEL
TYPICAL SECTION
MILE 18.6 TO 21.0



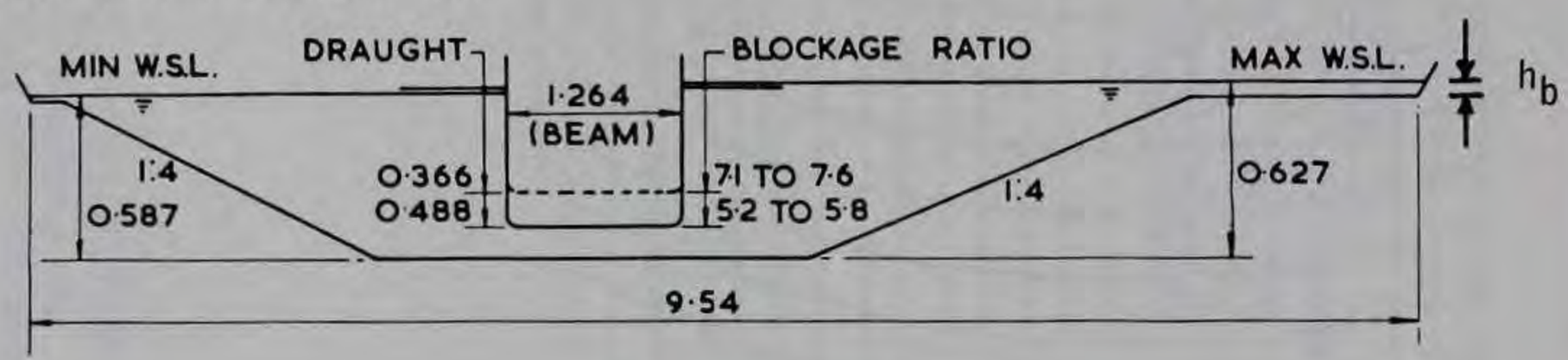
NOTE: SHIP SPEED = 7.6 MPH
η = 5.3
MAXIMUM DRAWDOWN = 2.1 FT

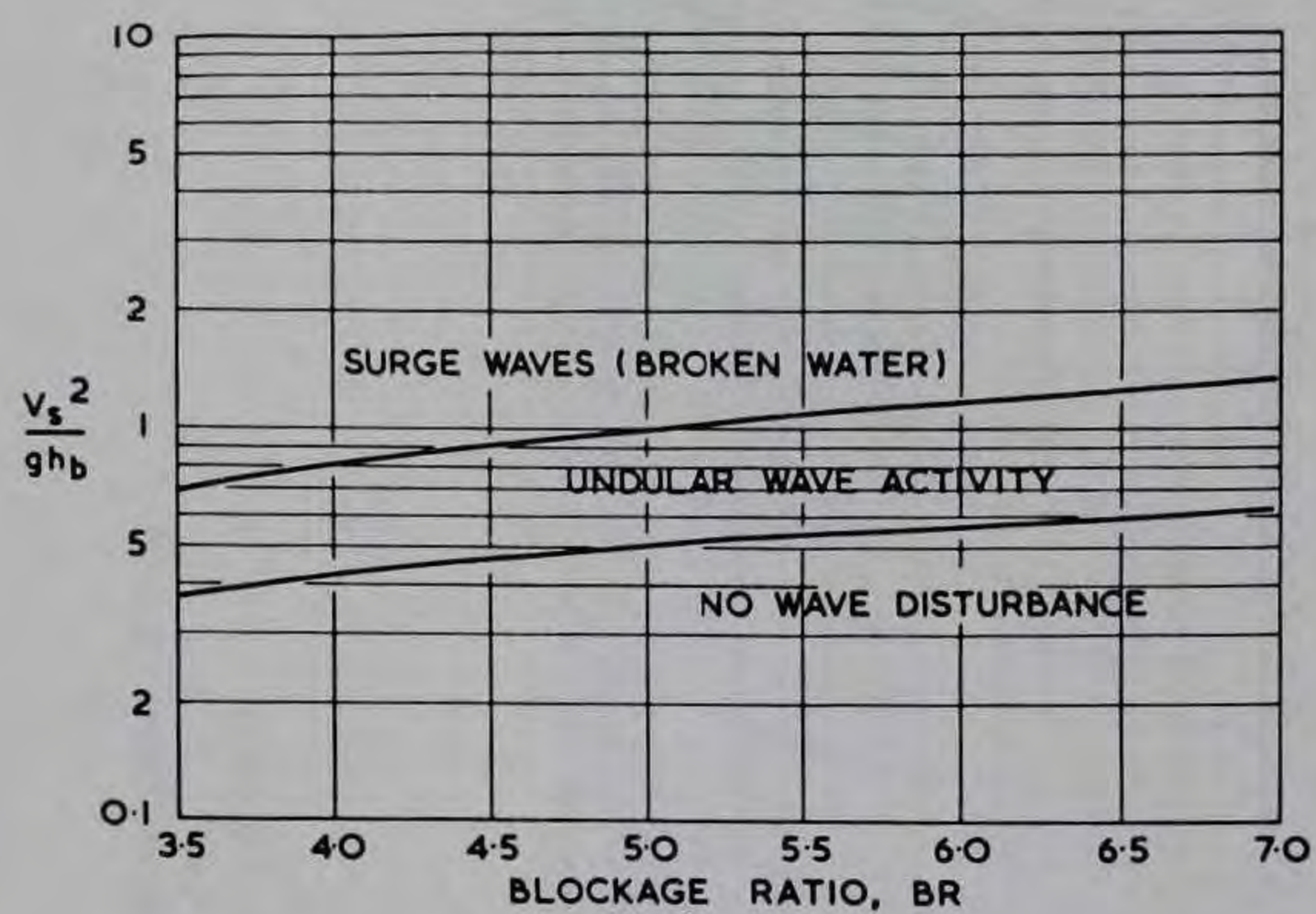
FROM GELENCSER (1977)

WATER-LEVEL DRAWDOWN
ST. LAWRENCE SEAWAY
PROTOTYPE MEASUREMENT

1. ALL DIMENSIONS IN METRES (MODEL)
2 VERTICAL EXAGGERATION 2:1

CANAL CROSS-SECTION No.1





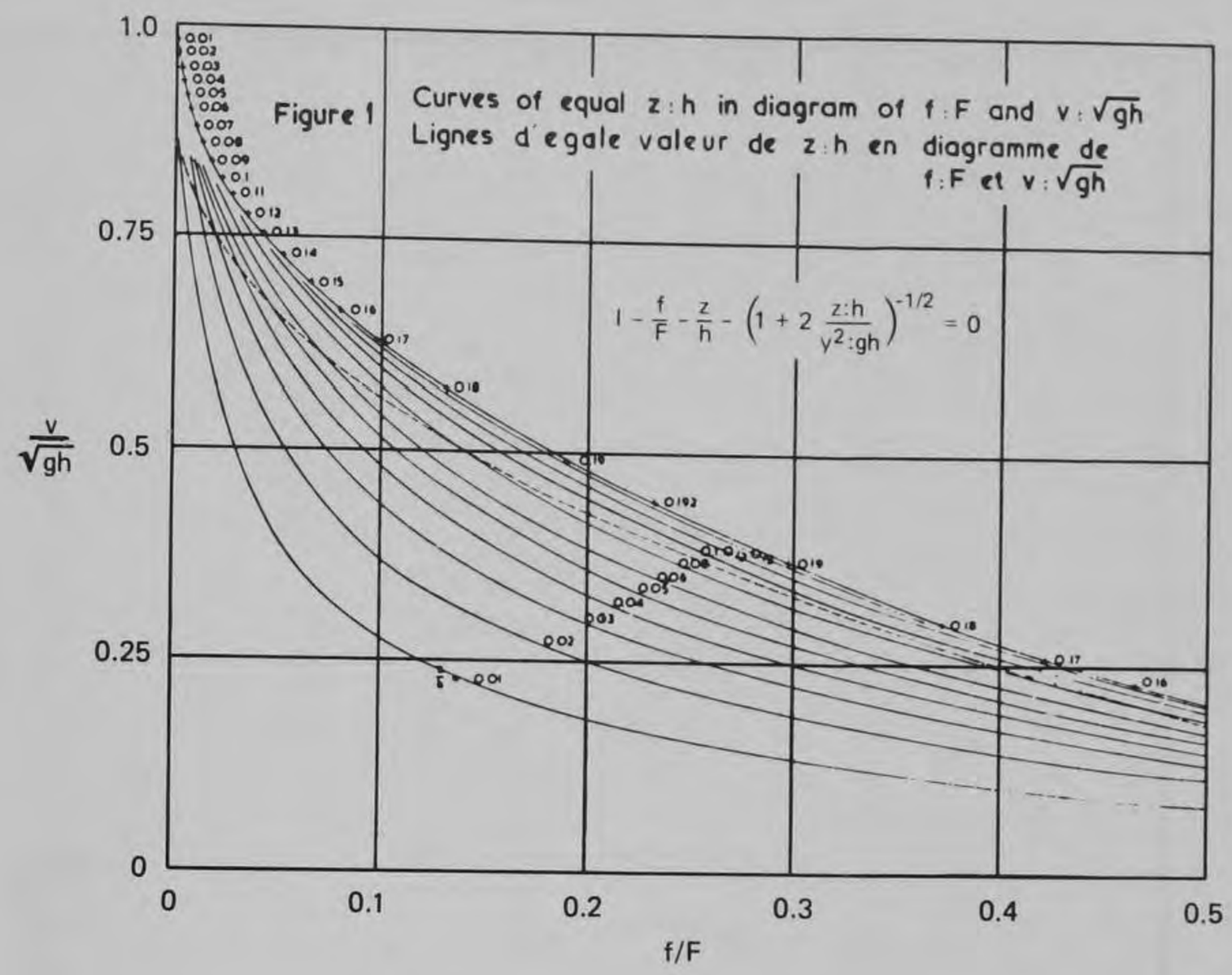
WAVE ACTIVITY ASSOCIATED WITH DRAWDOWN AT THE BERM

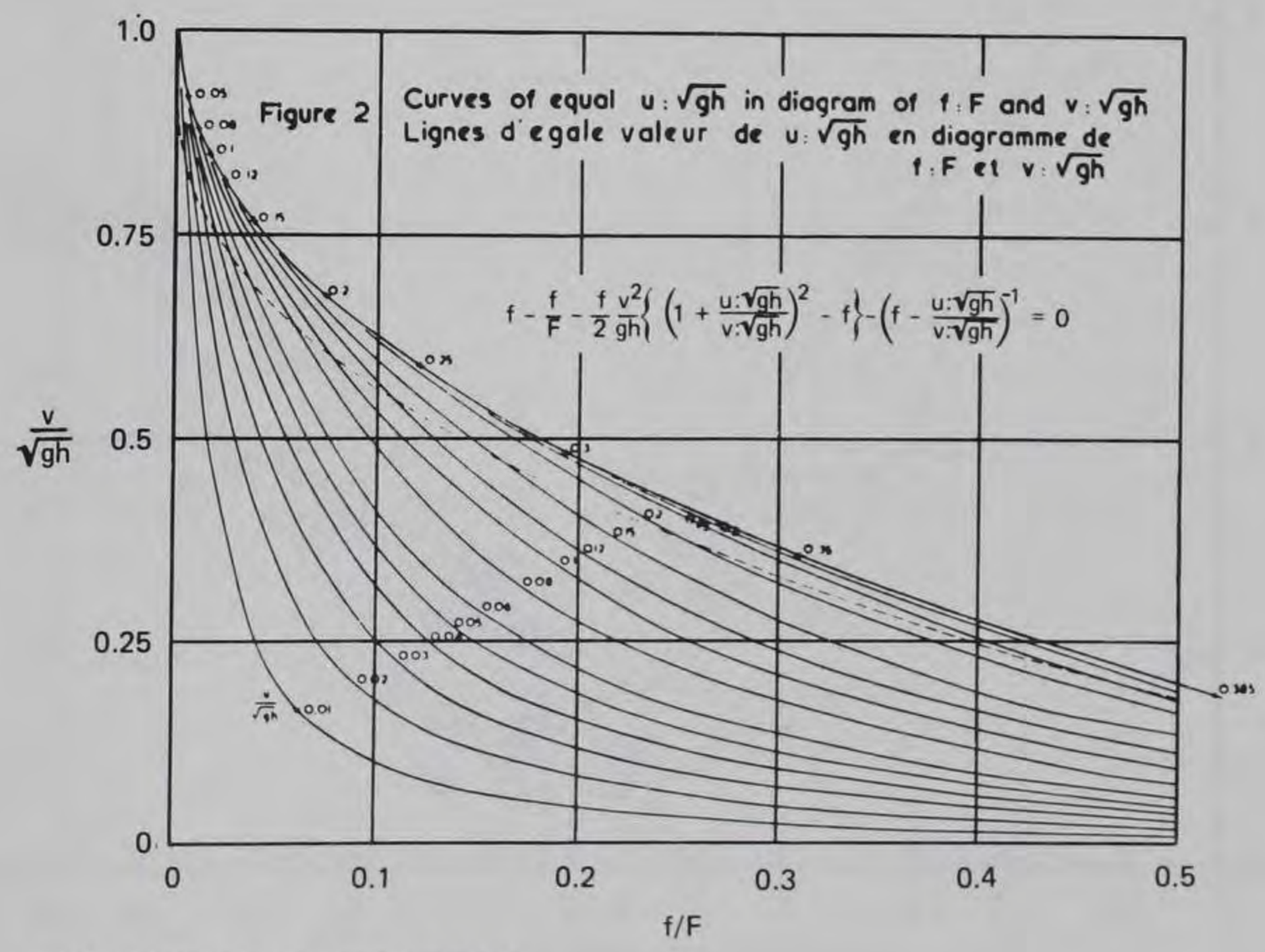
BR= η

Vs = SHIP SPEED

h_b = DEPTH OVER BERM

DRAWDOWN EFFECTS IN THE
SUEZ CANAL
(FROM DAND AND WHITE (1977))





z = WATER-SURFACE DRAWDOWN
u = SPEED OF RETURN FLOW
f/F = $1/\eta$
h = CHANNEL DEPTH
v = SHIP SPEED

WATER-SURFACE DRAWDOWN AND SPEED OF RETURN FLOW (FROM JANSEN AND SCHIJF (1953))

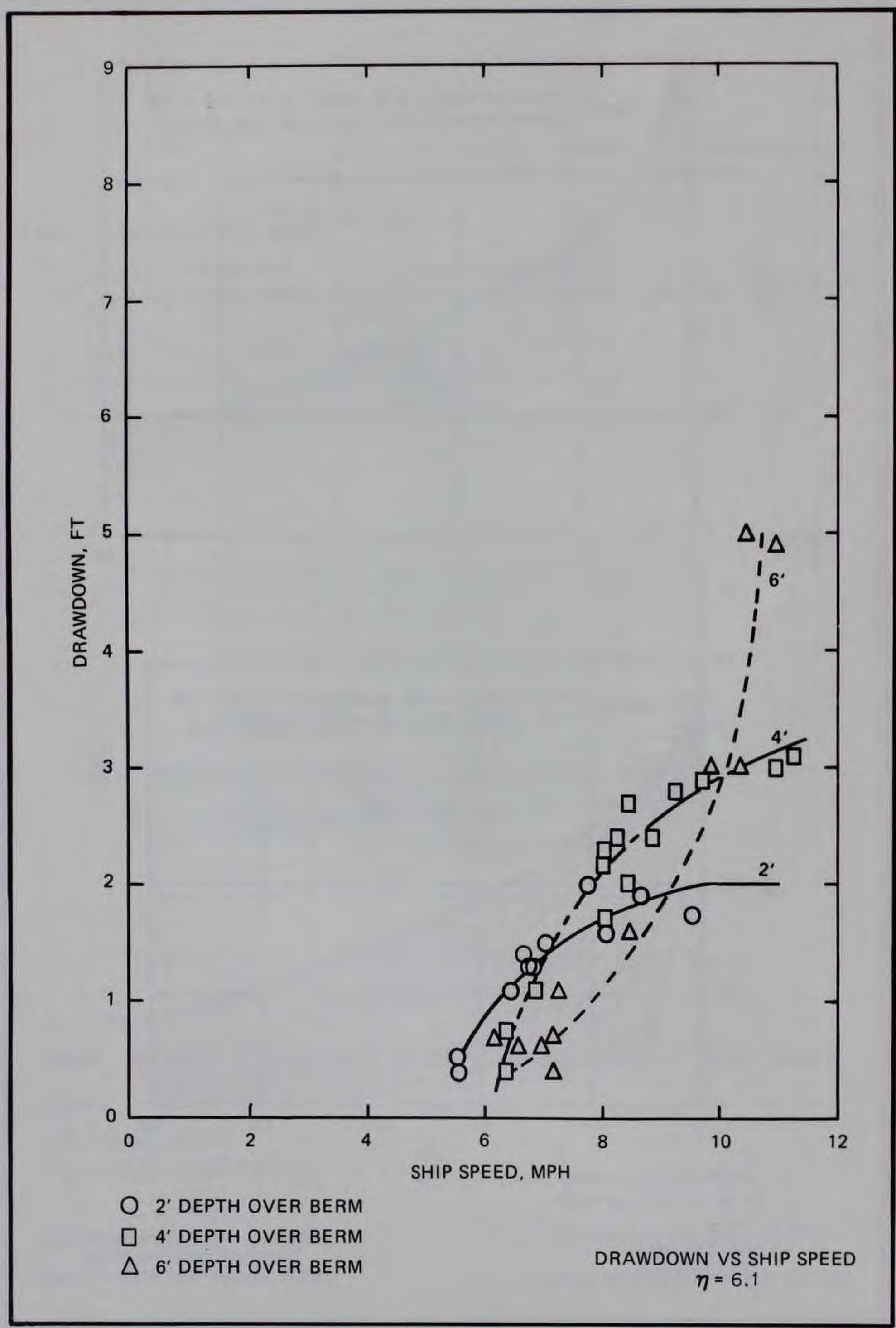
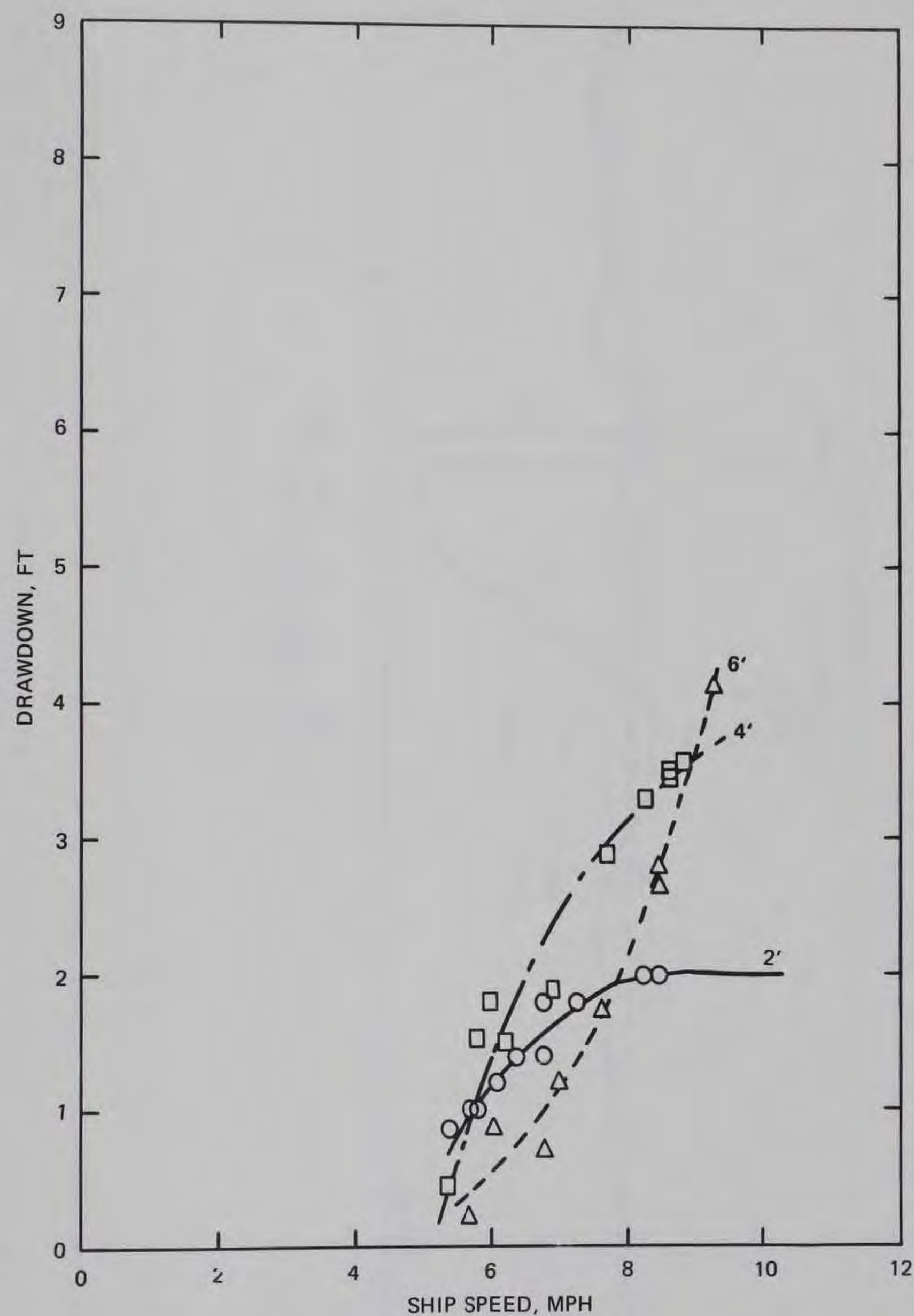
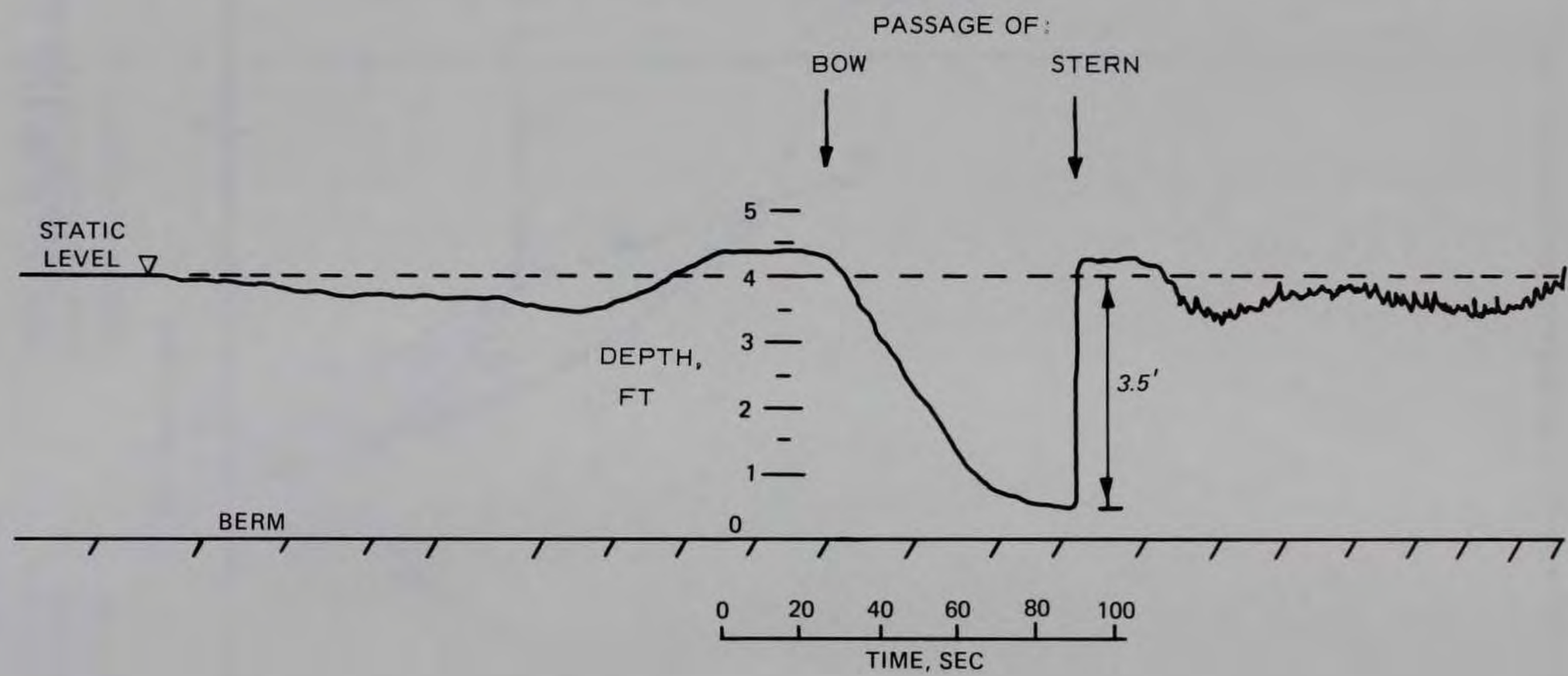

DRAWDOWN, FT
SHIP SPEED, MPH
0
1
2
3
4
5
6
7
8
9
0
2
4
6
8
10
12
6'
4'
2'
2' DEPTH OVER BERM
4' DEPTH OVER BERM
6' DEPTH OVER BERM
DRAWDOWN VS SHIP SPEED
η = 6.1

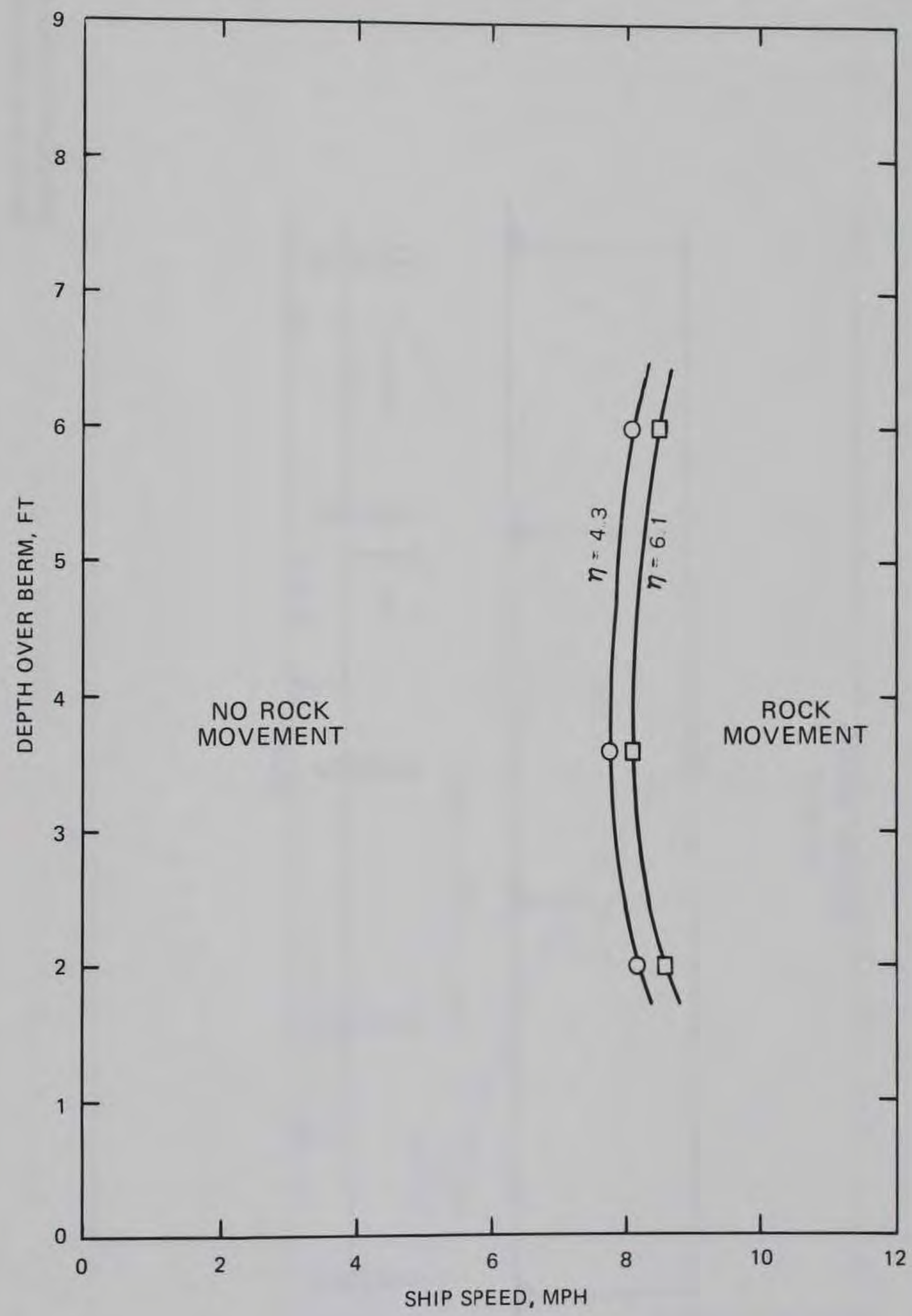
PLATE I-6



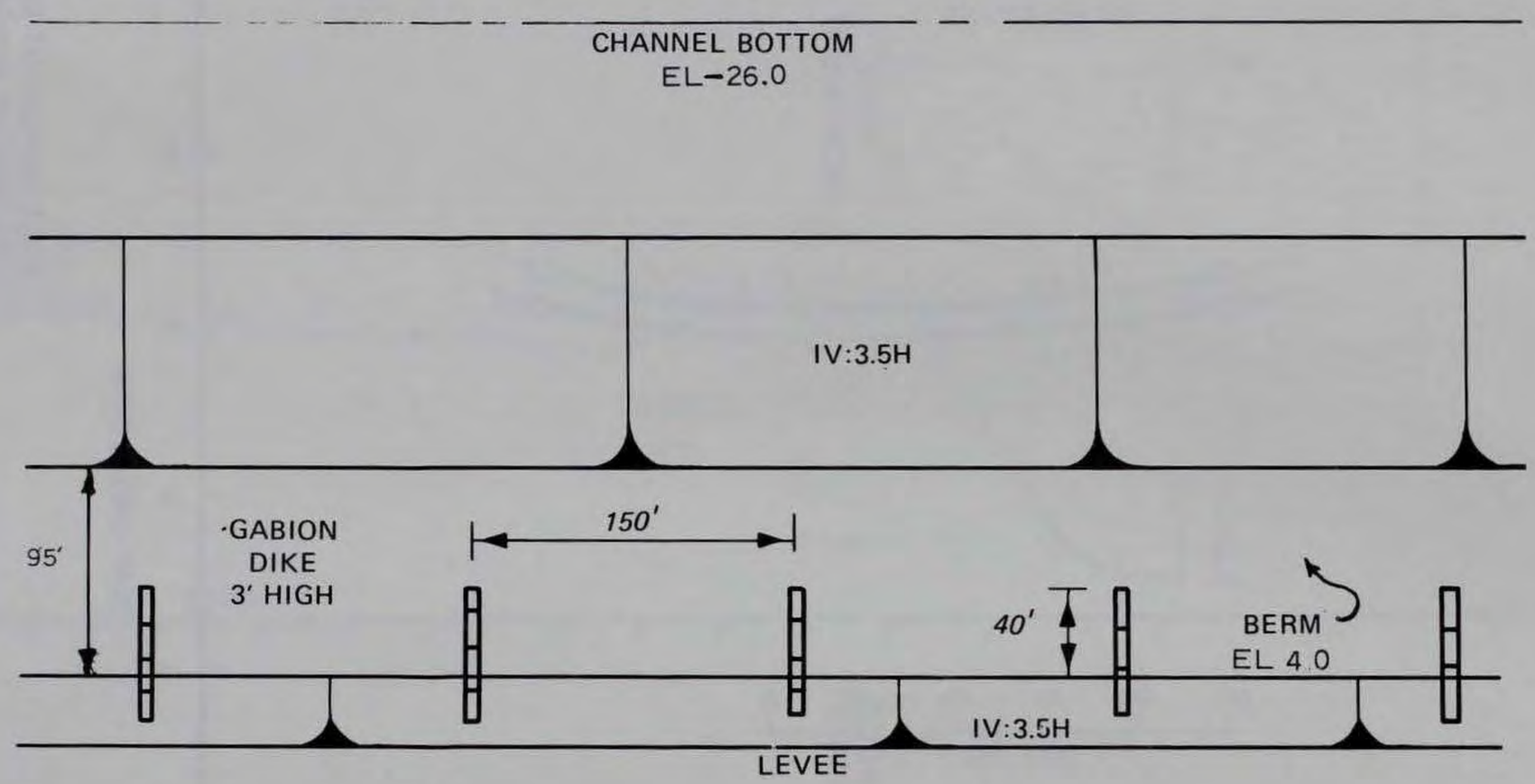
DRAWDOWN VS SHIP SPEED
$\eta = 4.3$



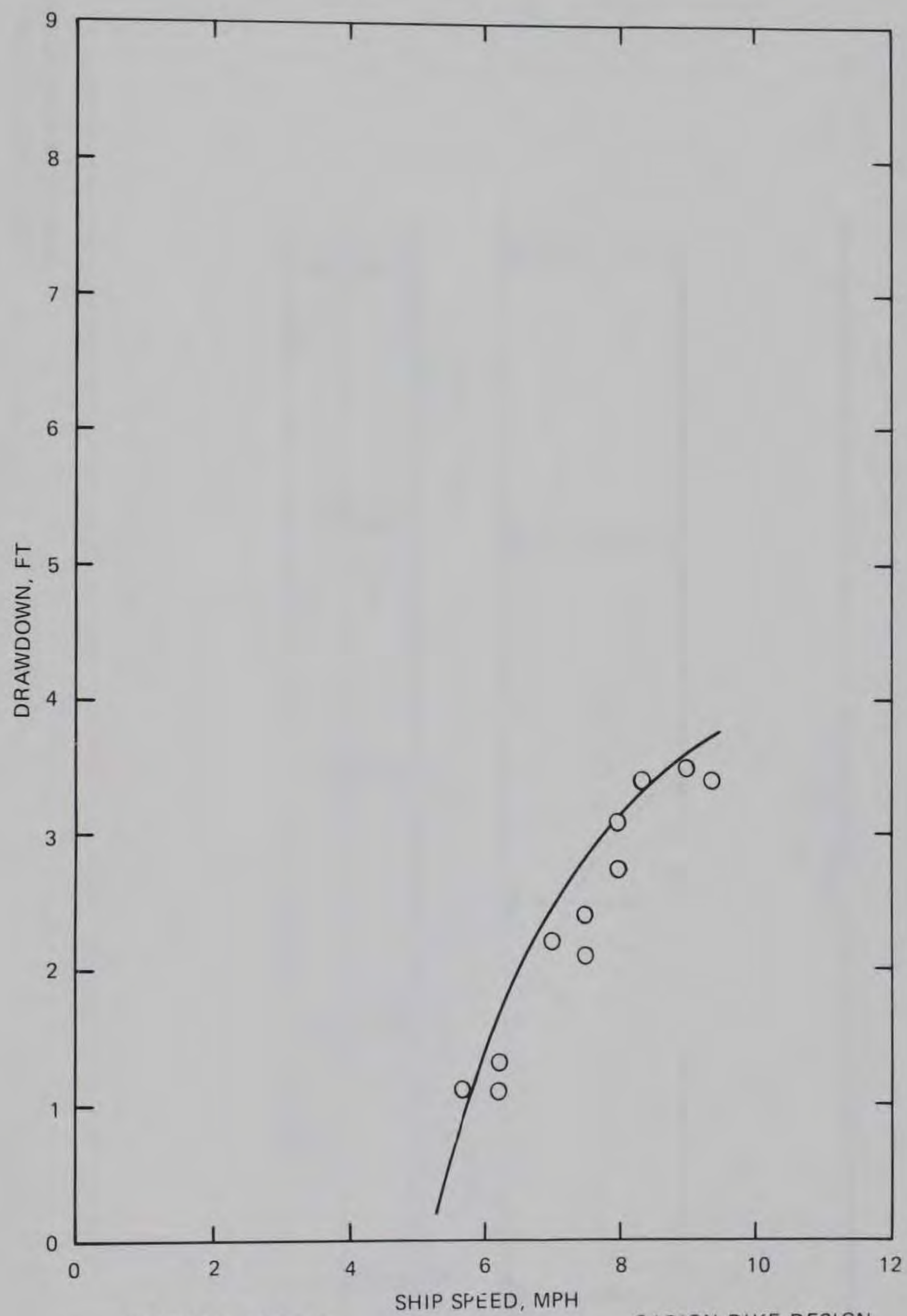
TIME-HISTORY
WATER LEVEL OVER BERM
4-FT DEPTH OVER BERM
SHIP SPEED = 8.8 MPH
η = 4.3



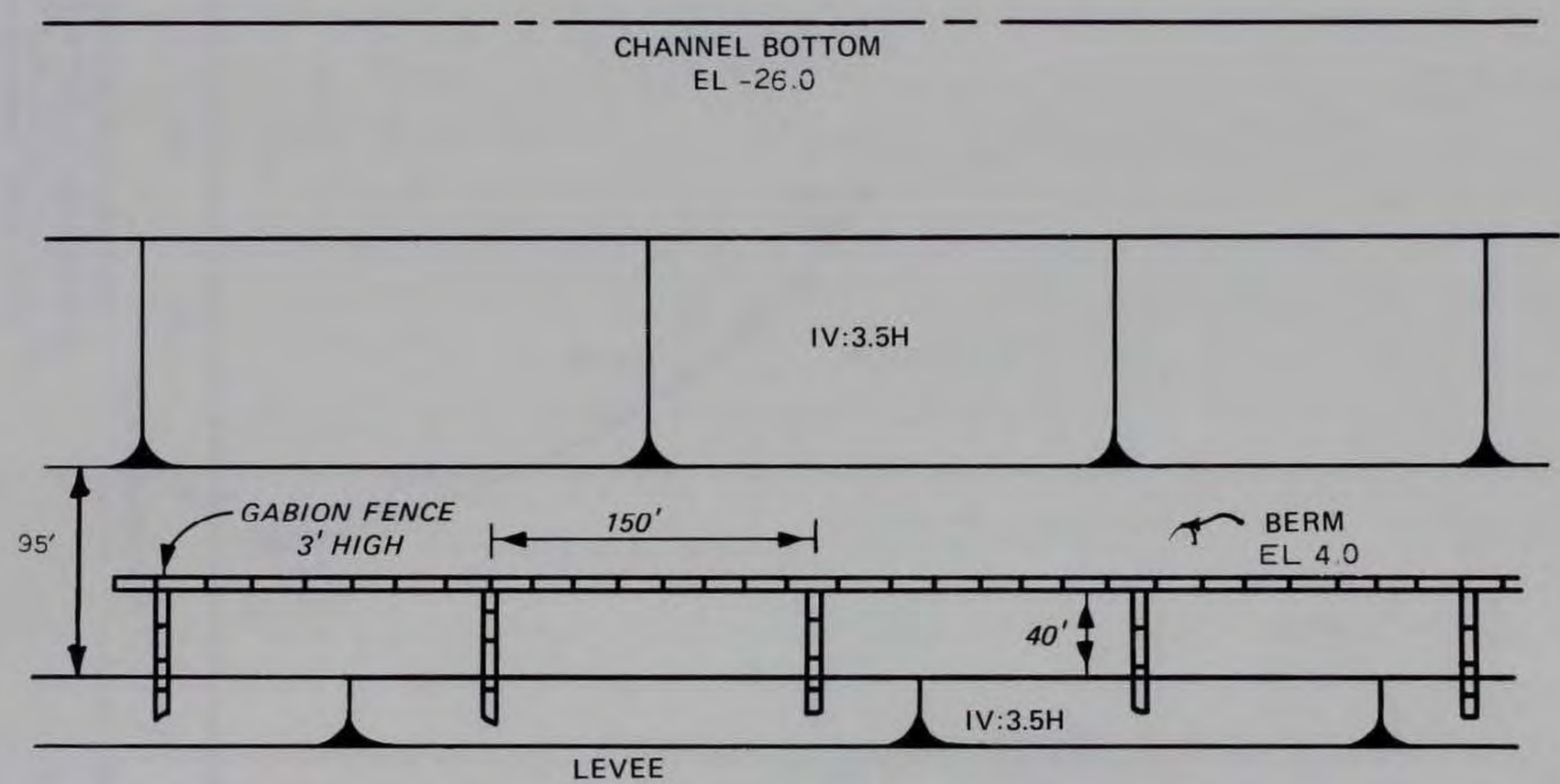
ROCK MOVEMENT DUE TO RETURN SURGE GRADATION #4851



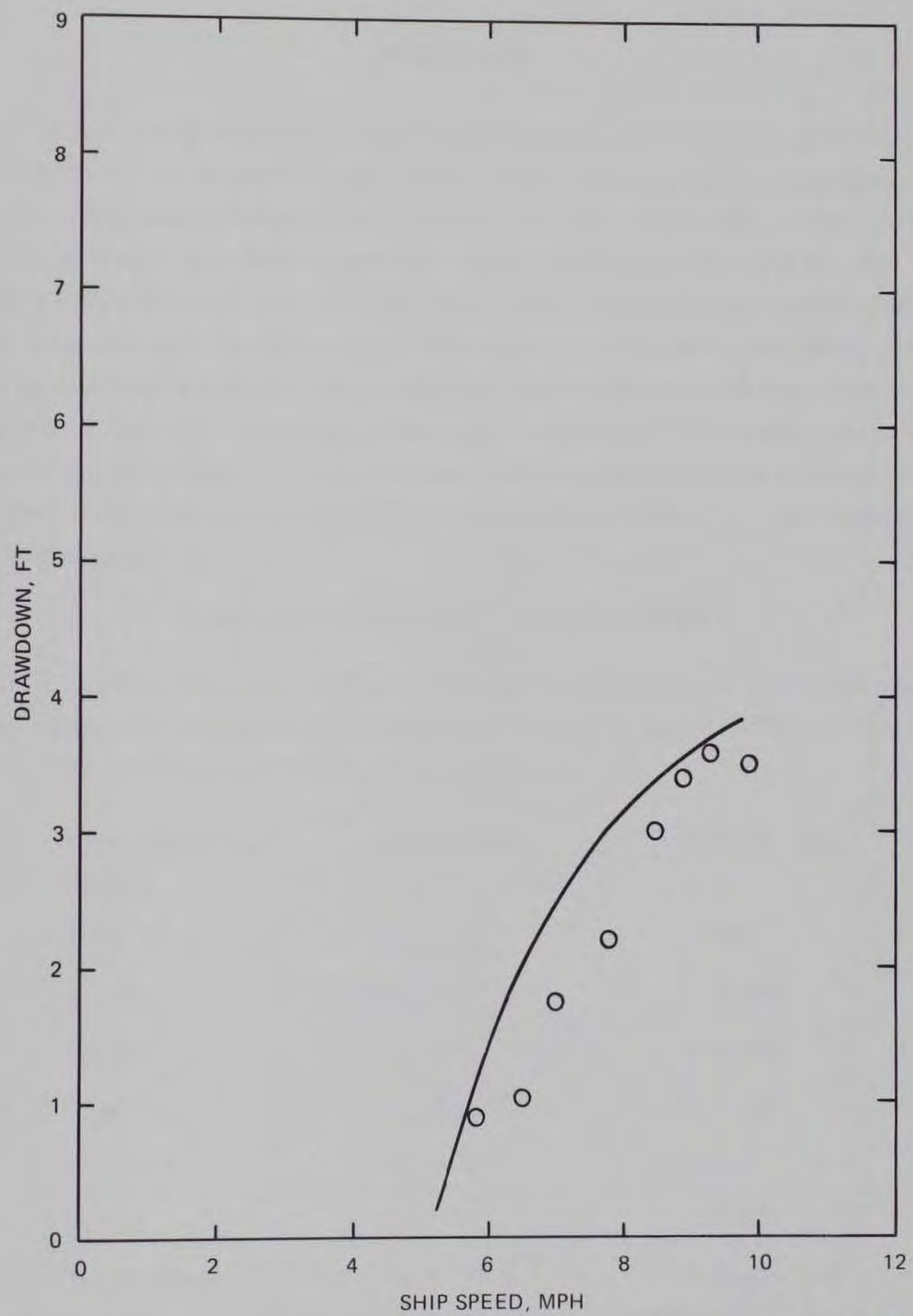
GABION DIKE DESIGN
150-FT DIKE SPACING



GABION DIKE DESIGN
DRAWDOWN VS SHIP SPEED
η = 4.3 SHIP DISPLACEMENT
4-FT DEPTH OVER BERM
150-FT GABION DIKE SPACING



GABION FENCE DESIGN
150-FT DIKE SPACING



——ORIGINAL DESIGN

O 20% INCREASE IN CHANNEL AREA

INCREASED CHANNEL AREA
DRAWDOWN VS SHIP SPEED
η = 5 2 SHIP DISPLACEMENT
4-FT DEPTH OVER BERM

PART II: MODEL INVESTIGATION OF EFFECTS OF PROPELLER WASH ON RIPRAP STABILITY

Introduction

1. Under the Section 32 Program,* hydraulic research was conducted to study the effects of propeller wash from inland navigation on channel bottom stability. This research addressed the riprap size required in maneuvering areas such as docks and lock approaches where vessel speeds are low but the energy of propeller wash can be high. The increasing size of vessels and vessel horsepower has exposed inland waterways to increased hydraulic forces and previously stable maneuvering areas are experiencing problems with scour of the channel bottom. Engineers planning and designing rehabilitation of existing or construction of future inland navigation facilities requiring bottom protection can use the results of this research within the limits stated in paragraph 10.

Model Appurtenances and Test Procedures

2. A 1:20-scale model was used for the investigation and model quantities were converted to prototype quantities based on the Froudian similarity criteria. The scaling relations are as follows:

Characteristic	Dimension	Model:Prototype
Length	L_r	1:20
Area	$A_r = L_r^2$	1:400
Volume	$Vol_r = L_r^3$	1:8,000
Weight	$W_r = L_r^3$	1:8,000
Time	$T_r = L_r^{1/2}$	1:4.47
Velocity	$V_r = L_r^{1/2}$	1:4.47
Thrust	$Th_r = L_r^3$	1:8,000
Revolutions	$R_r = 1/L_r^{1/2}$	1:0.224

* Public Law 93-251, The Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, as amended by Public Law 94-587, Sections 155 and 161, October 1976.

An outdoor slack-water channel with depths up to 25 ft (prototype) was used to represent the maneuvering areas. All references to sizes refer to the prototype unless stated otherwise. The channel bottom was sand having a medium diameter of approximately 0.5 mm (model dimension) and the side slopes of the channel were covered with filter fabric (prototype). To form the model riprap test sections, filter fabric was placed over the horizontal sand bed and riprap was placed on the filter fabric to simulate 300-ft-long by 100-ft-wide (prototype) test sections. Riprap used in the model was crushed limestone having a specific gravity of 2.67, and d_{50} sizes used in the investigation simulated prototype stone with diameters up to 2.92 ft. Gradations of the prototype stone simulated in the different model riprap test sections are shown in Plate II-1.

3. The 1:20-scale model tow (Figure II-1) used in the investigation

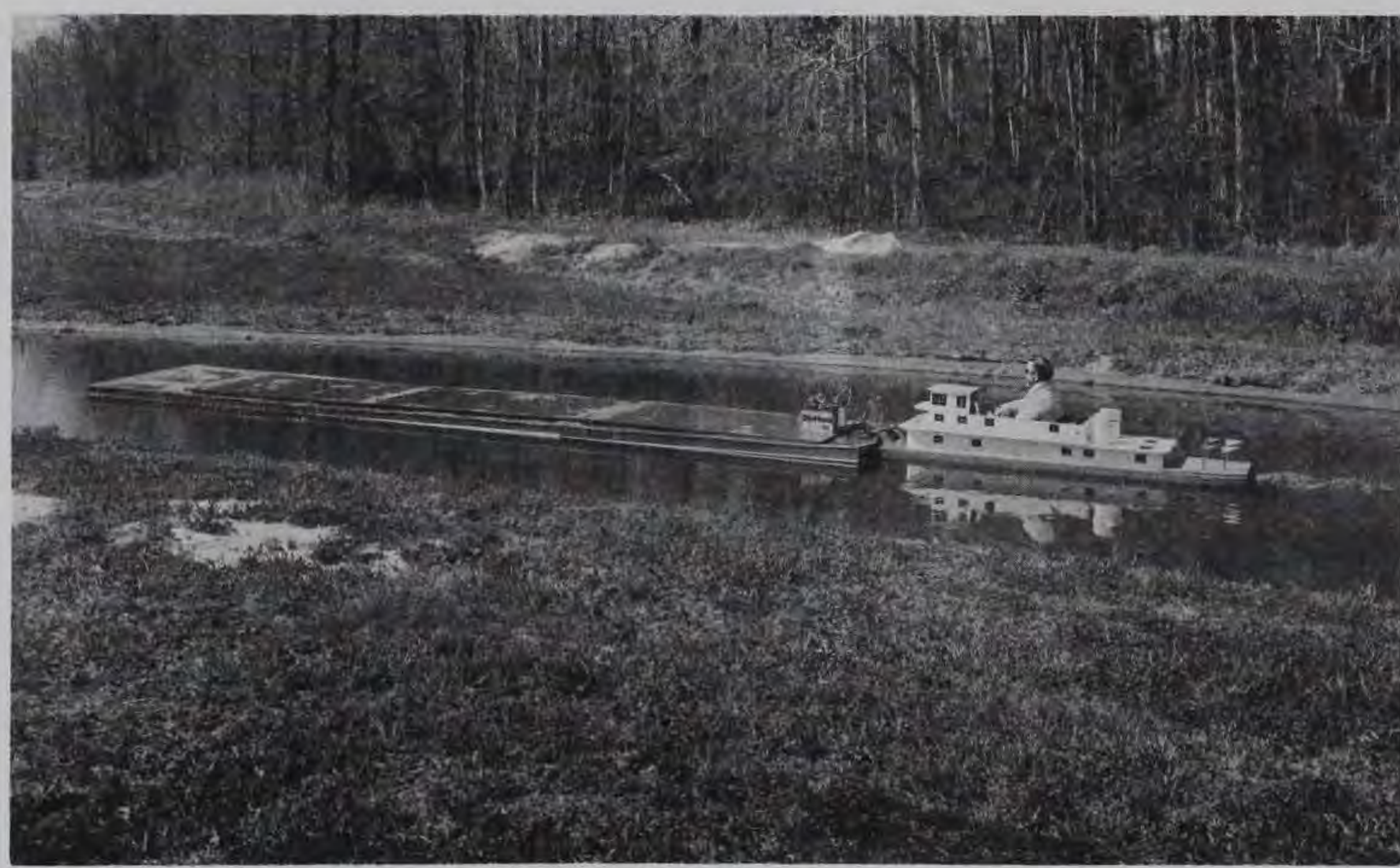


Figure II-1. 1:20-scale model tow

represents an inland waterway vessel having twin screws, main and flanking rudders for each screw, tunnel stern with twin rudder gear, and without Kort nozzles. Dimensions and other pertinent data for the simulated towboat are as follows:

Length = 208.8 ft

Width = 45.6 ft

Draft = 9 ft

Horsepower = 5,600

No. of propellers = 2

Propeller diameter = 10 ft

No. of blades = 4

Propeller rpm = 190

K_T , thrust coefficient at zero ship speed = 0.36

4. Each test was conducted with the vessel held in a stationary position over the riprap test section in the slack-water channel and the required propeller speed was established in the model. Depths were gradually lowered until failure of the bottom riprap was detected. Each riprap size was subjected to a 9-min (prototype) duration of the full thrust of the towboat at each depth before the test section was inspected for failure. Depths were measured with staff gages and riprap failure was determined by observing either rock movement or exposure of the underlying filter fabric. Tests were conducted with both forward and backward thrust and attack of the rock was similar in the slack-water channel.

Test Results

5. A summary of tests conducted and results is shown in Table II-1. A plot of rock size as a function of channel depth for the 5,600-hp towboat is shown in Plate II-2. These results are applicable to attack on the channel bottom without the effects of any lateral walls. Details of the rock gradations simulated and investigated are shown in Plate II-1. These gradations represent very uniform riprap or capstone and do not address energy absorption or filter requirements between the riprap and soil. A rock gradation with a wide variation in sizes contains fine material that may be transported by the propeller wash and deposited in undesirable areas within the maneuvering zones.

Comparison of Model Results with Engineering Literature

6. A search of the literature was conducted to evaluate existing design information regarding bottom protection against propeller wash. A recent article by Fuehrer, Römisch, and Engelke* gives an excellent review of past work and presents a design procedure for protecting both the bottom and side slopes of navigation canals. This procedure requires computing the induced jet velocity, Vo , defined as

$$Vo = 1.6nDKT \tag{1}$$

where

Vo = induced jet velocity at ship speed = 0, m/sec
n = propeller speed, rev/sec
D = propeller diameter, m
KT = thrust coefficient, at ship speed = 0

For the 5,600-hp towboat, the induced jet velocity is Vo = 9.27 m/sec = 30.4 ft/sec. Many times the thrust coefficient may not be known and Blaauw and Van de Kaa** present an equation for estimating Vo based on horsepower and propeller diameter

$$Vo = 1.48\left(\frac{P_D}{D^2}\right)^{1/3} \tag{2}$$

where

P_D = installed engine power, kw (1 horsepower = 0.746 kw)
D = propeller diameter, m

---

* M. Fuehrer, K. Römisch, and G. Engelke. 1981. Criteria for Dimensioning the Bottom and Slope Protections and for Applying the New Methods of Protecting Navigation Canals," Permanent International Association of Navigation Congresses, 25th Congress, Section I, Volume I.

** H. G. Blaauw and E. J. van de Kaa. 1978. "Erosion of Bottom and Sloping Banks Caused by the Screw Race of Maneuvering Ships," Delft Hydraulics Laboratory, Publication No. 202.

Based on Equation 2 for the 5,600-hp towboat, Vo = 9.0 m/sec = 29.4 ft/sec which is close to the value obtained by Equation 1. Next, the bottom velocity is determined as a function of Vo , propeller diameter, and depth by Fuehrer as

$$V_B,max = Vo \cdot E \cdot (hp/D)^{-1.0} \tag{3}$$

V_B,max = maximum botton velocity at zero ship speed, m/sec

E = a coefficient depending upon the stern shape and type of rudder arrangement: 0.25 for inland ship, tunnel stern, single screw, with twin rudder gear

hp = distance from center of propeller to bottom, m

D = propeller diameter, m

This value of E was determined by Fuehrer using single screw vessels whereas the model vessel used in this investigation was a twin screw vessel. At the shallower depths, the propeller jet may attack the bottom before the jets intersect. At deeper depths, the jets may intersect before attacking the bottom and result in greater attack than with the single screw vessel. Comparison of the twin screw model results with the results of Fuehrer's design procedure should help resolve the difference between single screw-double screw ships. The final step is relating the maximum bottom velocity to the required stone size by Fuehrer's equation for V_B,max defined as

$$V_B,max = B \sqrt{d_{50} g \left(\frac{\rho_s - \rho}{\rho} \right)} \tag{4}$$

where

B = a coefficient depending upon the type of stern and type of rudder arrangement: 0.9 for inland ship, tunnel stern, and twin rudder gear

d_{50} = average stone diameter, m

g = acceleration due to gravity = 9.81 m/sec^2

ρ_s = stone density

ρ = water density

This value of B is the limiting condition or point at which rock movement would be incipient. For safe design the d_{50} size should be increased by an appropriate factor.

7. A comparison of the 1:20-scale model data and the Fuehrer, Römisch, and Engelke technique is shown in Plate II-3 for the 5,600-hp towboat. The curve represents incipient motion for bottom riprap protection without the effects of walls or flowing water which inhibit spreading of the flow and concentrate the attack.

8. According to Fuehrer, a significant reduction in the maximum bottom velocity occurs for normal navigation, i.e. navigation that is under way at a constant rate of speed. The maximum bottom velocity (V_B,max) for normal navigation is given by the relation

$$V_B,max = Vo \cdot E \cdot (hp/D)^{-1.0} \left(1 - \frac{V}{nD}\right) \tag{5}$$

where

V = ship speed, m/sec

Discussion of Results and Conclusions

9. The relation between rock size and water depth developed from the model tests is as expected: large rock required for small depths and small rock with large depths. Further, it appears that asymptotic limits of depth exist such that the size of stone required for stability increases and/or decreases at an infinite rate. For example, with a depth of 12.5 ft a significant increase in rock size does not permit any decrease in the depth allowed. This is not unexpected because at this condition there exists a jet of water approximately 10 ft in diameter with a velocity of about 30 ft/sec at a distance of only 3.5 ft from the riprap. The energy dissipation and velocity reduction at the boundary will be small for this condition. One preliminary conclusion from these tests is that riprap should not be used as protection with these small depths for the towboat size tested in this investigation. Stated differently, a greater depth of water would be necessary for use of riprap to protect the bottom of a berthing area or navigation channel, lock approach, etc.

10. Good correlation was found between the design procedure recommended by Fuehrer and the results of this investigation. These results, although more conservative than Fuehrer's, show that Fuehrer's design procedure for single screw vessels is applicable to the twin screw vessel used in this

investigation. The curve shown in Plate II-3 represents incipient motion of the bottom riprap protection and rock size should be increased to provide a stable design. This curve and the model results should not be used where adjacent lock or training walls limit spreading of the jet or in flowing waters. This occurs mainly when propeller thrust is angled toward a wall or upstream against flowing water which results in concentrated attack on the bottom.

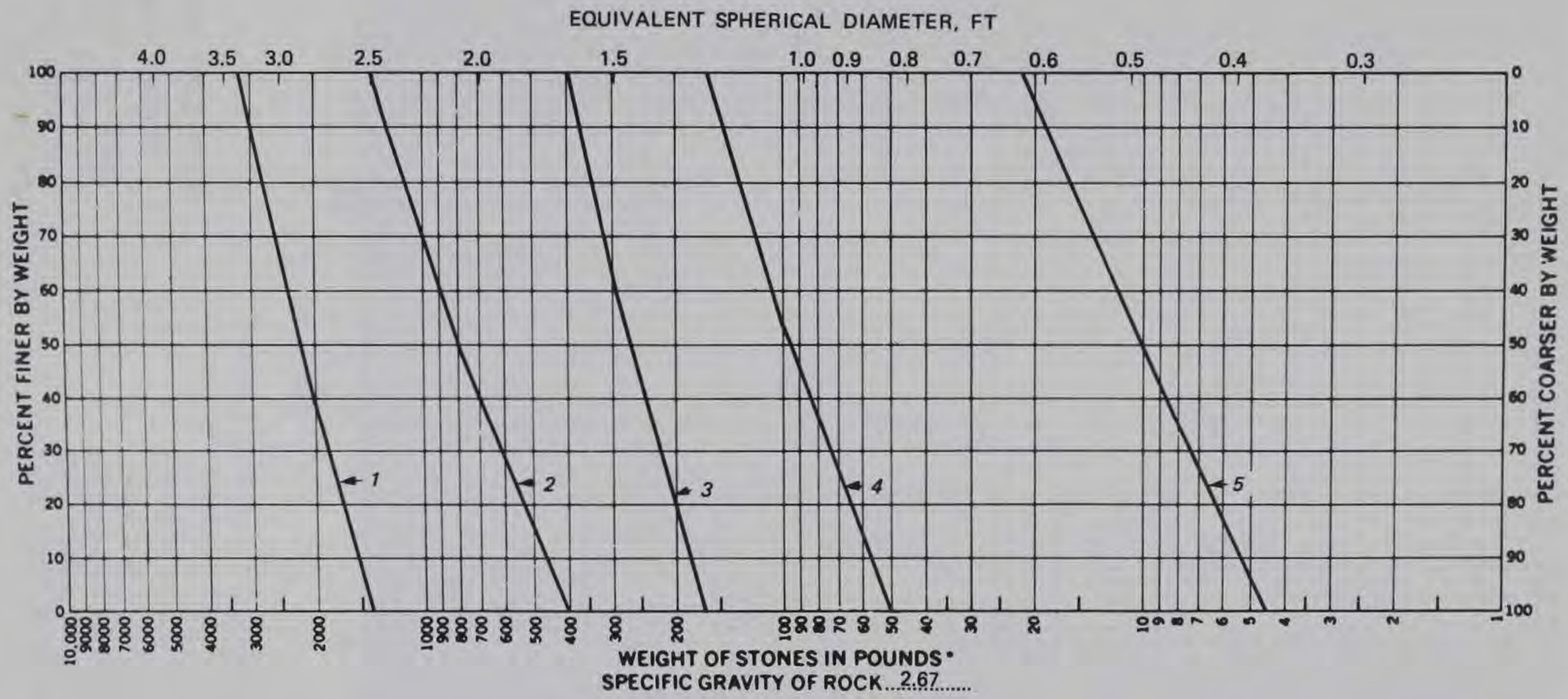
11. Fuehrer's design procedure can be used to estimate the rock size required in maneuvering areas for various towboat sizes and water depths. Additional research is needed to determine the individual and collective effects on walls, angle of attack, depth, draft, horsepower, and velocity of vessel relative to riverflow. The capability to do such research experiments has been demonstrated and such additional R&D would result in improved guidance and criteria for plan, design, operation, and maintenance of the Nation's waterways.

Table II-1

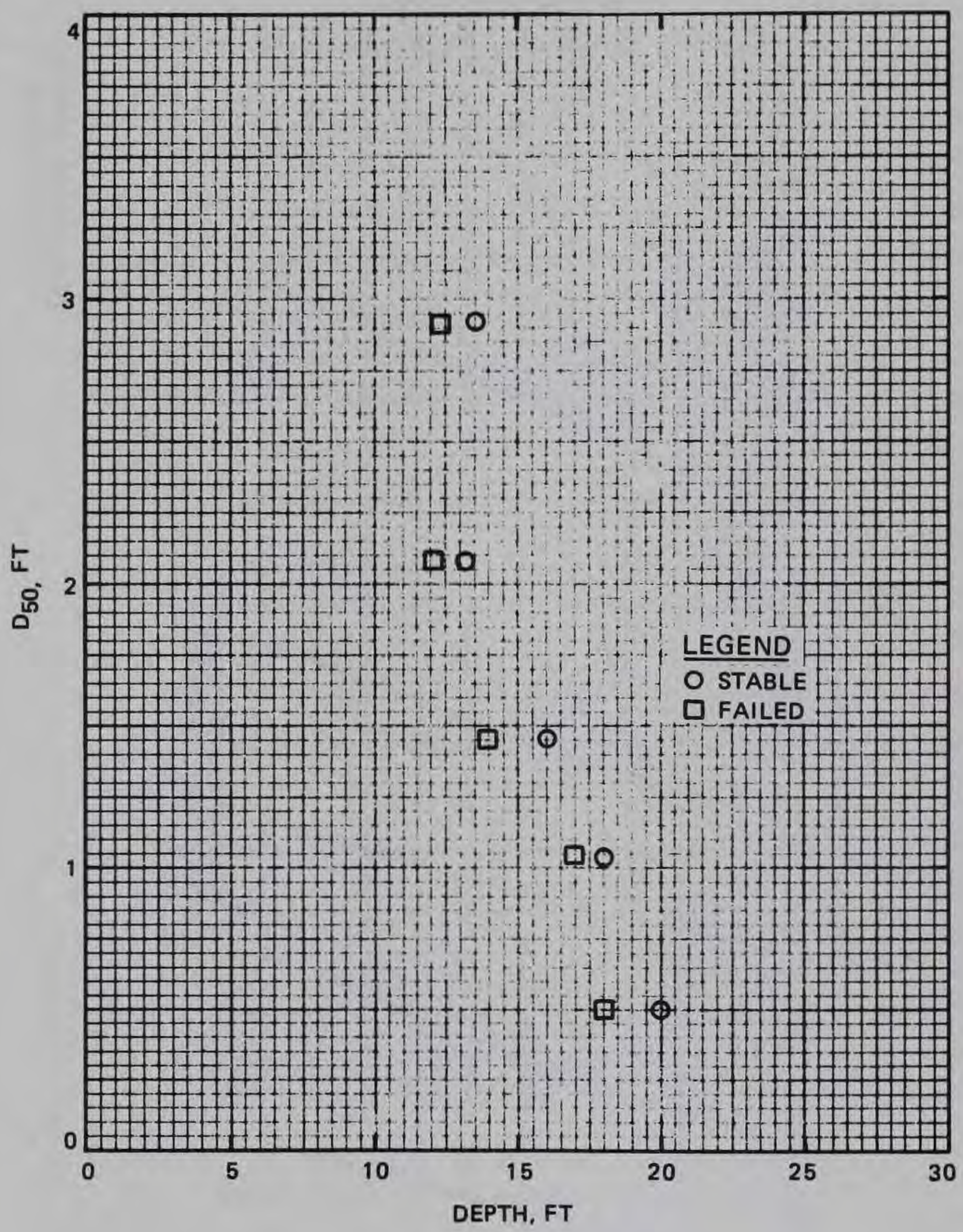
Summary of Test Results

Gradation No.	d_{50} ft	Depth ft	hp ft	hp/D*	Test Result
1	2.92	15.6	11.6	1.16	Stable
	2.92	14.6	10.6	1.06	Stable
	2.92	13.6	9.6	0.96	Stable
	2.92	12.4	8.4	0.84	Failed
2	2.08	15.3	11.3	1.13	Stable
	2.08	14.2	10.2	1.02	Stable
	2.08	13.2	9.2	0.92	Stable
	2.08	12.2	8.2	0.82	Failed
3	1.46	18.0	14.0	1.40	Stable
	1.46	17.0	13.0	1.30	Stable
	1.46	16.0	12.0	1.20	Stable
	1.46	14.0	10.0	1.00	Failed
4	1.04	19.0	15.0	1.50	Stable
	1.04	18.0	14.0	1.40	Stable
	1.04	17.0	13.0	1.30	Failed
5	0.50	24.0	20.0	2.00	Stable
	0.50	22.0	18.0	1.80	Stable
	0.50	20.0	16.0	1.60	Stable
	0.50	18.0	14.0	1.40	Failed

* hp is the distance from the center of the prop to the channel bottom; D is the prop diameter.



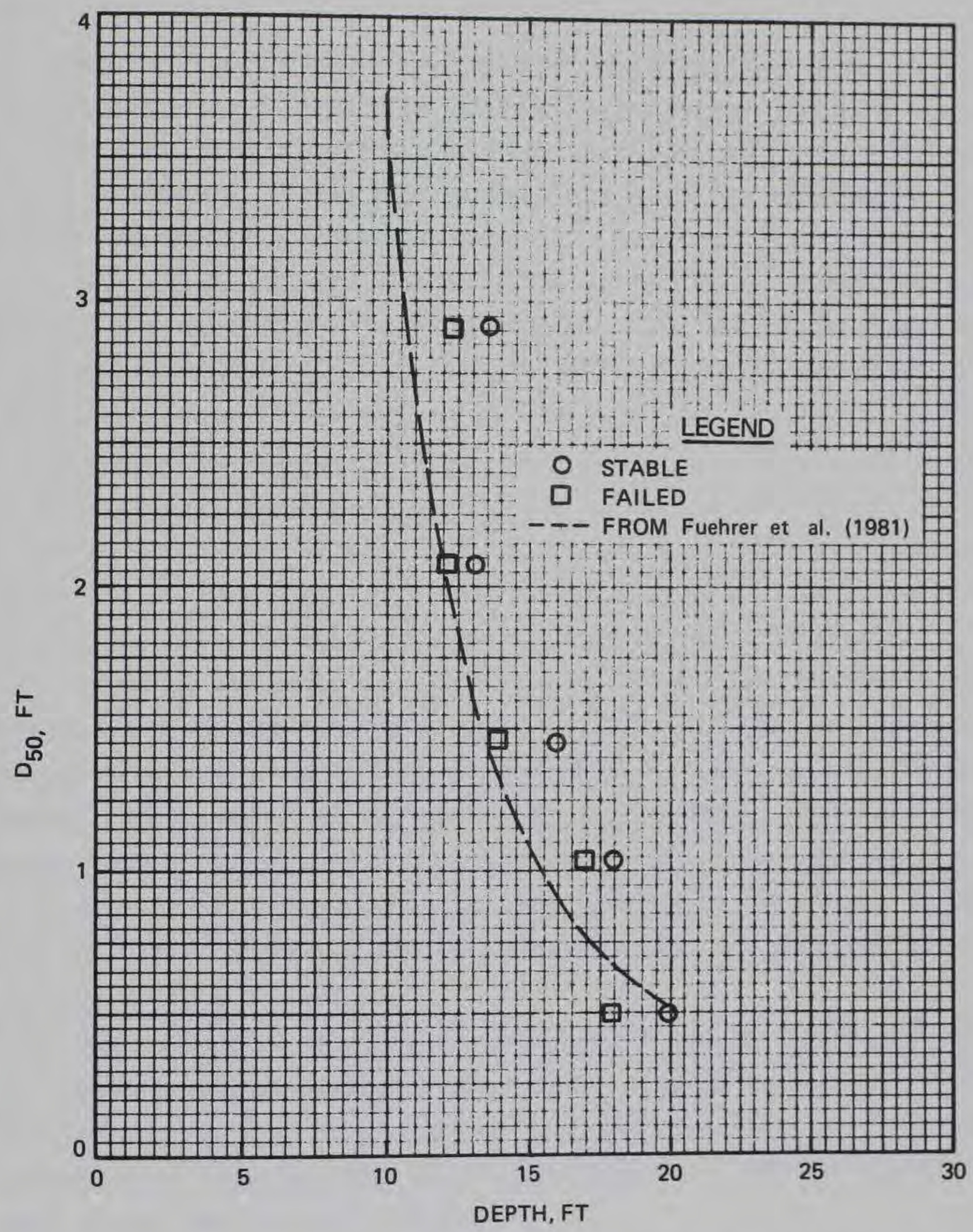
SIMULATED RIPRAP
GRADATIONS



ROCK SIZE VS DEPTH

MODEL RESULTS

5600–HP TOWBOAT
STRAIGHT ATTACK
W/O WALL EFFECTS
SLACK–WATER CHANNEL



ROCK SIZE VS DEPTH COMPARISON OF MODEL RESULTS WITH FUEHRER'S DESIGN TECHNIQUE

5600–HP TOWBOAT
STRAIGHT ATTACK
W/O WALL EFFECTS
SLACK–WATER CHANNEL

PART III: MODEL INVESTIGATION OF SEABROOK LOCK BOTTOM RIPRAP

1. The proposed Seabrook Lock will be located on the south shore of Lake Pontchartrain at its junction with the Inner Harbor Navigation Canal (IHNC). The IHNC provides access from Lake Pontchartrain to the Mississippi River and, indirectly, to the Gulf of Mexico through the Mississippi River Gulf Outlet and Gulf Intracoastal Waterway.

2. The navigation lock (Plate III-1) will be 84 ft wide, 860 ft center to center of the operating sector gate pintles, with an 800-ft usable length and a minimum submergence of 14.8 ft. Two reinforced concrete gate bays with hydraulically operated steel sector gates will be located at each end of the lock. The connecting chamber will consist of steel sheet pile cells topped with a concrete wall supported on bearing piles within the cells. Instead of reinforced concrete, riprap will serve as the bottom of the lock chamber and protect the soils from severe erosive actions of displaced water and propeller wash due to the small vessels entering and exiting the lock chamber.

3. Based on information supplied by the U. S. Army Engineer District, New Orleans, and a limited telecon survey of towboat owners and operators using Lake Pontchartrain and vicinity, towboats that will use Seabrook Lock will not exceed 1,400 hp with the average being in the range of 800 to 1,000 hp. The design towboat selected for the study was 1,400 hp. Barges on the waterways around Seabrook Lock are generally 250 by 50 ft or 195 by 35 ft with draft up to 10 ft. Therefore the maximum width tow will not generally exceed 70 ft in the 84-ft-wide lock.

4. The purpose of this site-specific study was to assess the effects of propeller wash on the stability of the proposed riprap for the bottom of the Seabrook Lock and test alternate sizes, if required. This research was confined to the size vessels given in paragraph 6 and the operating conditions furnished by the New Orleans District.

5. The model of the Seabrook Lock was constructed to a scale ratio of 1:10. Five hundred feet of the length of the lock and the full 84-ft width were reproduced in the model (Plate III-1). The model lock was installed in a slack-water channel having a sand bed with a median diameter of 0.5 mm. The riprap was originally placed in the center of the lock and later extended to the lock sidewall to test both straight attack without wall effects and angled

attack where the propeller was directed into the wall at an angle of 8 deg. The original design type 1 riprap gradation limits and the gradation represented in the model are shown in Plate III-2. Rock used in the model was crushed limestone having a specific gravity of 2.65. The thickness of the original gradation was 3 ft and the rock underlayment or filter material was 1.5 ft thick. Filter cloth was used in the model to represent the underlayment.

6. The 1:10-scale model towboat and barges are shown in Plate III-3 and dimensions represented by the model tow are as follows:

<u>Towboat:</u> Length = 104.4 ft
Width = 22.8 ft
Draft = 6.5 ft
No. of propellers = 2
Propeller size = 5 ft
No. of blades = 4
Design thrust = 27,400 lb at 4.5 knots
Rated horsepower = 1,400 hp
Kort nozzles = without
rpm = 350
Propeller thrust coefficient at zero speed of advance = 0.36

<u>Barges:</u> Length = 292.5 ft
Width = 52.5 ft
Maximum draft = 10 ft

7. Water depths in the model were measured with staff gages and riprap failure was determined by observation after each run of the towboat.

8. Model quantities were scaled to prototype by means of the Froudian similarity criteria. The following characteristics are expressed in terms of the length ratio, L_r:

Characteristic	Dimension	Model:Prototype
Length	L_r	1:10
Area	$A_r = L_r^2$	1:100
Volume	$Vol_r = L_r^3$	1:1,000

Characteristic	Dimension	Model:Prototype
Weight	$W_r = L_r^3$	1:1,000
Time	$T_r = L_r^{1/2}$	1:3.16
Velocity	$V_r = L_r^{1/2}$	1:3.16
Thrust	$Th_r = L_r^3$	1:1,000
Revolution	$R_r = 1/L_r^{1/2}$	1:0.316

9. The original design type 1 riprap was tested in the model at the minimum depth of 14.8 ft. The full thrust of the towboat (27,400 lb) was exerted on the riprap test section for 5 min (prototype) with the boat held in a stationary position. A schematic of the test setup is shown in Plate III-4. Table III-1 summarizes tests conducted and results. For the minimum depth of 14.8 ft and towboat draft of 6.5 ft, no rock movement occurred for straight or angled attack with the type 1 riprap. Based on the limited surveys of towboat owners and operators, towboat drafts of up to 10 ft can be expected in the Seabrook Lock. With a depth of 14.8 ft and towboat draft of 10 ft, h_p becomes 7.3 ft (Plate III-4). This $h_p = 7.3$ ft was simulated with the 6.5-draft towboat by decreasing the water depth to 11.3 ft. This condition was tested in the model with full thrust for 5 min (prototype) and no rock movement occurred for either straight or angled attack. These tests were conducted with the tow directed both into and out of the lock and the most severe attack is with the tow directed into the lock. For this condition, the propellers actually "pumped" water out of the lock, lowering the actual depth of water below 14.8 ft. Also, thrust of the propellers was directed both forward and reverse and attack on the bed was similar for both conditions.

10. To determine the relative safety factor of the type 1 rock, tests were conducted at values of h_p down to 3.0 ft and no rock movement was observed for straight attack but failure occurred at $h_p = 4.4$ ft for angled attack.

11. With the original type 1 riprap having a relatively high degree of safety, tests were conducted with a smaller size of riprap. Type 2 riprap gradation limits and the gradation represented in the model are shown in Plate III-5. The thickness of the type 2 riprap was 1.6 ft which corresponds to the approximate d_{100} of the stone. The minimum depth of 14.8 ft and draft of 6.5 ft was tested and the type 2 riprap remained stable for both

straight and angled attack. When the 10-ft-maximum towboat draft was simulated by lowering the depth to 11.3 ft, the type 2 riprap failed for straight attack. The type 2 riprap failed at a depth of 12.9 ft for angled attack.

12. After failure of the type 2 riprap design, tests were conducted with the type 3 riprap design. The gradation is shown in Plate III-6 and the thickness was 2.3 ft. For the minimum depth of 14.8 ft and towboat draft of 6.5 ft, no rock movement occurred for straight or angled attack with the type 3 riprap. When the 10-ft-maximum towboat draft was simulated by lowering the depth to 11.3 ft, the type 3 riprap remained stable for straight and angled attack. Failure occurred when the depth was lowered to 9.4 ft for angled attack and 7.9 ft for straight attack.

13. Summarizing, the type 1 riprap proved to have a high degree of stability when compared with the forces generated by the 1,400-hp towboat. Failure of the riprap occurred only when the depth was lowered far below that now possible in the prototype. The type 2 riprap was not stable for the anticipated towboat sizes and water depths in the Seabrook Lock. The type 3 riprap remained stable for the design 1,400-hp towboat and maximum towboat draft of 10 ft and minimum lock depth of 14.8 ft. In selecting the riprap required to protect the lock bottom, consideration should be given to future trends of vessel sizes, horsepower, lock operating conditions, and the desired safety factor. Should vessels larger than the design 1,400-hp tow begin using the lock, type 1 and 3 riprap would be subject to failure. Due to the site-specific testing conditions and the small vessels using Seabrook Lock, these results should not be extrapolated to other locks of apparent similarity.

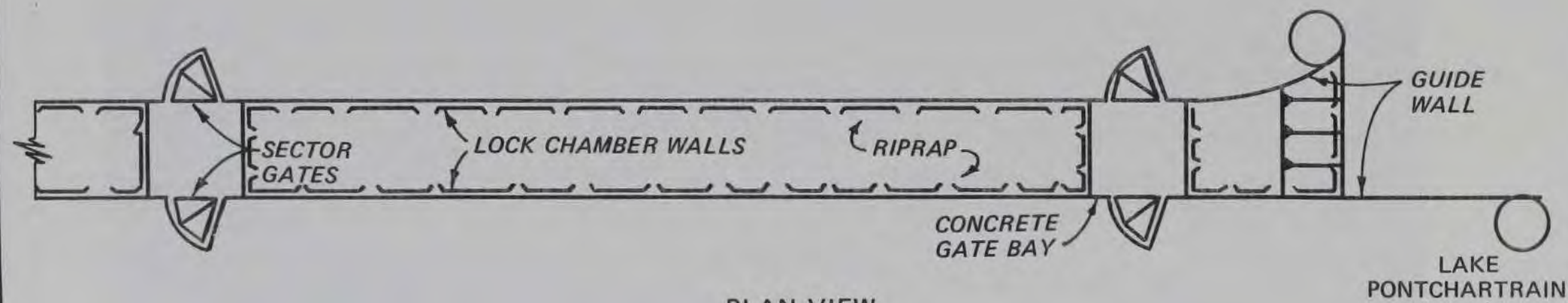
14. In placement of the riprap in the prototype, care should be taken to eliminate any sizes smaller than the minimum shown on the gradation curve. These sizes would be easily moved by the propwash and might deposit in areas that would hinder operation of the lock.

15. Although not tested in the model, the need for an adequate filter or underlayment cannot be overstressed. The high degree of turbulence generated by the propeller wash can extend through the cover riprap and remove the bottom material if an adequate filter is not provided in the lock chamber and approach to the lock.

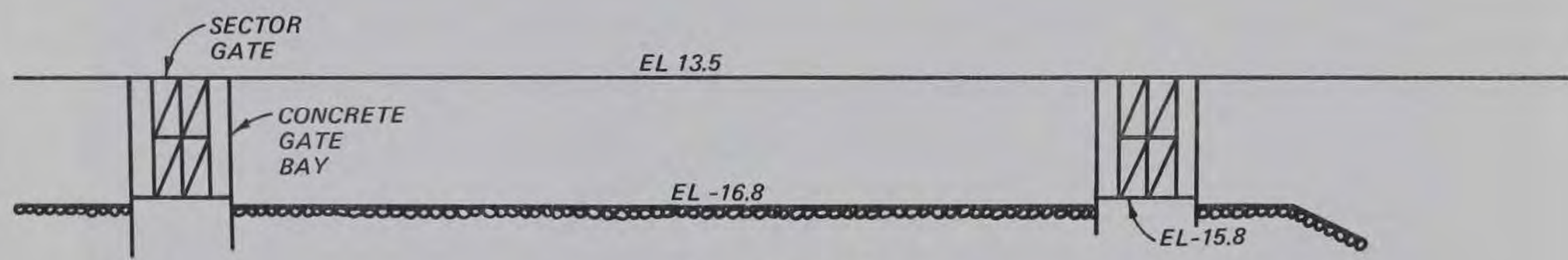
Test III-1

Test Summary

Riprap Design Type	Direction Of Attack	d_{50} ft	Towboat Draft ft	Depth ft	h_p ft	Thrust lb	Test Result
1 (Original)	Straight	1.8	6.5	14.8	10.8	27,400	Stable
	Angle	1.8	6.5	14.8	10.8	27,400	Stable
	Straight	1.8	6.5	11.3	7.3	27,400	Stable
	Straight	1.8	6.5	7.0	3.0	27,400	Stable
	Angle	1.8	6.5	11.3	7.3	27,400	Stable
	Angle	1.8	6.5	8.4	4.4	27,400	Failed
2	Straight	1.0	6.5	14.8	10.8	27,400	Stable
	Straight	1.0	6.5	11.3	7.3	27,400	Failed
	Angle	1.0	6.5	14.8	10.8	27,400	Stable
	Angle	1.0	6.5	12.9	8.9	27,400	Failed
3	Straight	1.5	6.5	14.8	10.8	27,400	Stable
	Straight	1.5	6.5	11.3	7.3	27,400	Stable
	Straight	1.5	6.5	7.9	3.9	27,400	Failed
	Angle	1.5	6.5	14.8	10.8	27,400	Stable
	Angle	1.5	6.5	11.3	7.3	27,400	Stable
	Angle	1.5	6.5	9.4	5.4	27,400	Failed



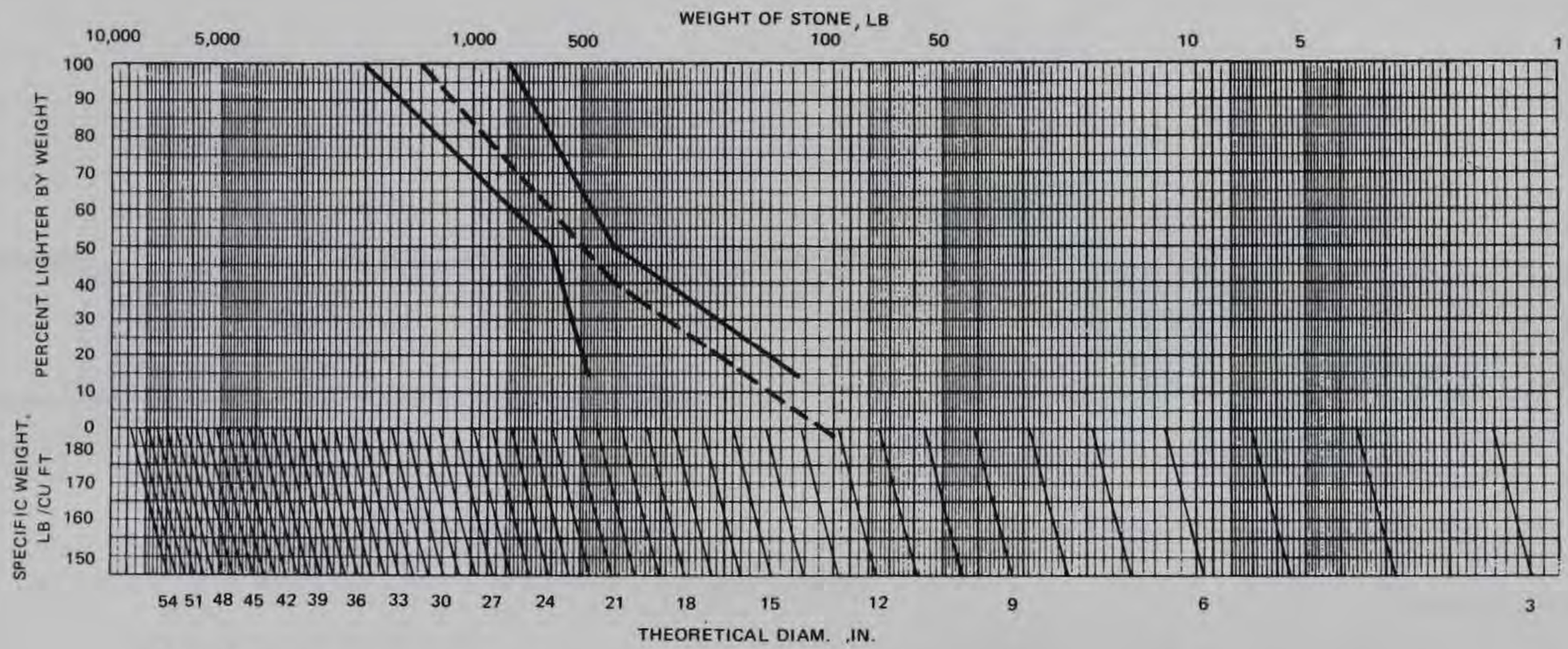
PLAN VIEW



ELEVATION VIEW

NOTE: ELEVATIONS IN FEET ABOVE NGVD

LOCK DETAILS



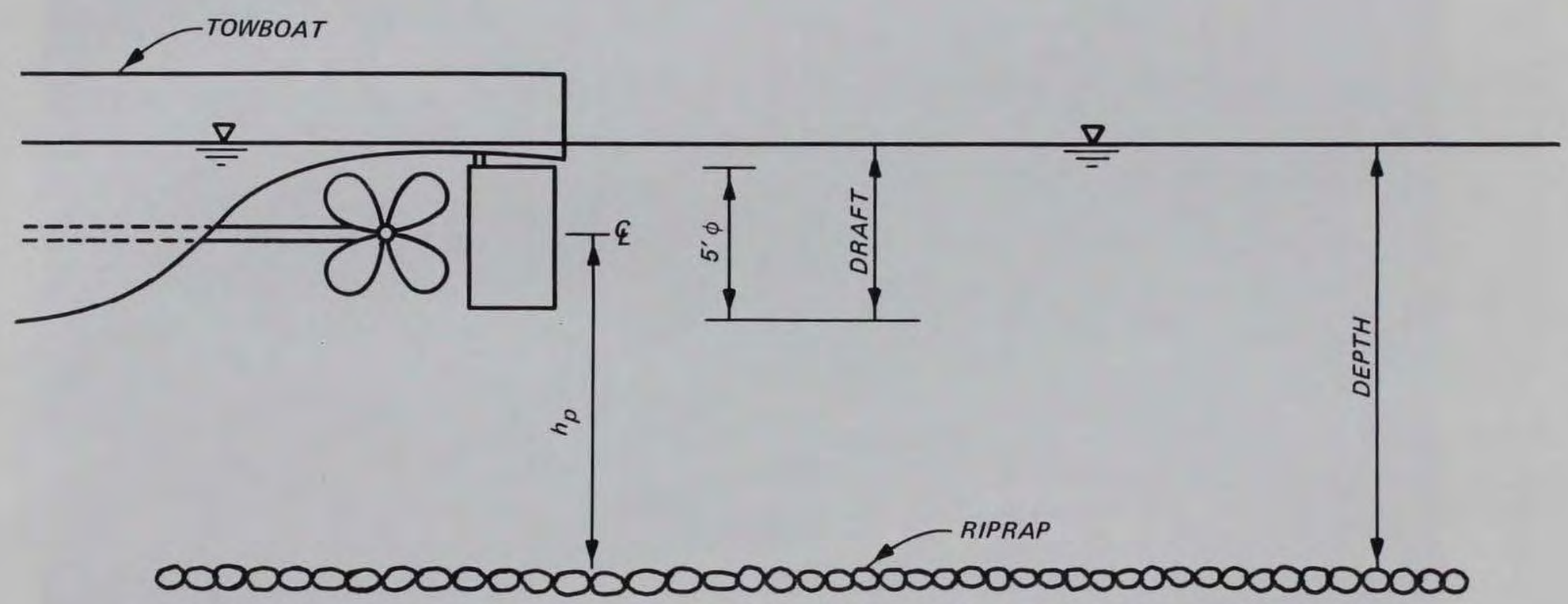
LEGEND

——— GRADATION LIMITS
– – – MODEL GRADATION

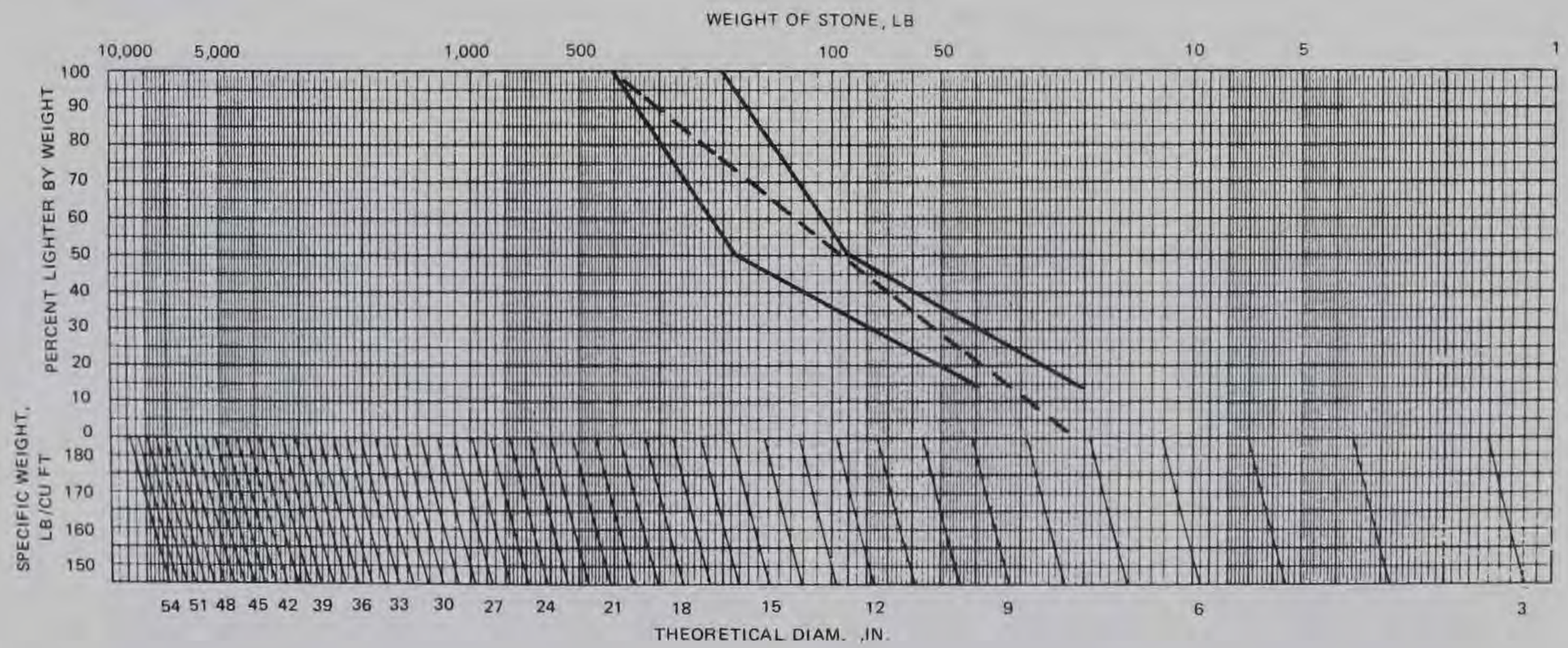
SPECIFIC GRAVITY 2.65
THICKNESS 1.5 ft

RIPRAP GRADATION CURVES
SEABROOK LOCK
TYPE 1 ORIGINAL

1:10-SCALE MODEL TOWBOAT



SEABROOK LOCK
SCHEMATIC

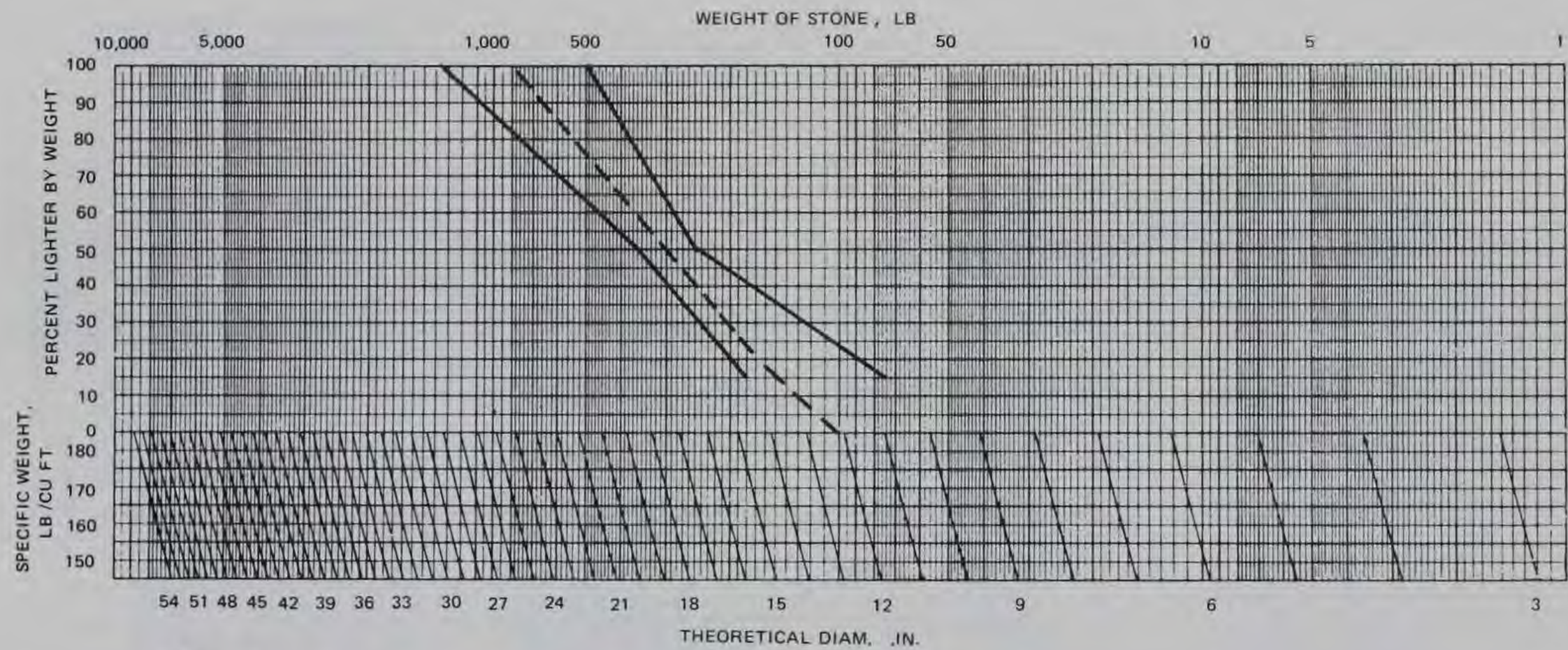


LEGEND

——— GRADATION LIMITS
— — — MODEL GRADATION

SPECIFIC GRAVITY 2.65
THICKNESS 1.6 ft

RIPRAP GRADATION CURVES
SEABROOK LOCK
TYPE 2



LEGEND

——— GRADATION LIMITS
– – – MODEL GRADATION

SPECIFIC GRAVITY 2.65
THICKNESS 2.3 ft

RIPRAP GRADATION CURVES
SEABROOK LOCK
TYPE 3