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RESEARCH REPORT H-75-3

PHYSICAL HYDRAULIC MODELS: ASSESSMENT OF PREDICTIVE CAPABILITIES

Report 3

MODEL STUDY OF SHOALING BRUNSWICK HARBOR, GEORGIA

by

Joseph V. Letter, Jr., William H. McAnally Jr.

Hydraulics Laboratory

U. S. Army Engineer Waterways Experiment Station

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

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Report 3 of a Series

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This study evaluates the sedimentation predictions of the fixed-bed model study of Brunswick Harbor, Ga. The predictions and subsequent prototype data analysis addressed the impacts of two closure dams in the vicinity of the harbor. The periods of analysis covered approximately 6.5 years prior to construction of the first closure dam, 5.5 years between the first and second closure dams, and 1 year after the completion of the second closure dam. The

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model study ~~employed~~ gilsonite (a noncohesive, finely ground asphaltic material) as a tracer, while the prototype sediment is highly cohesive clay.

The model correctly predicted the reduction and redistribution of shoaling with the first closure dam. However, with the second closure dam, the model erroneously predicted a large reduction in shoaling volumes, while the prototype experienced a slight increase in shoaling.

It is concluded that the prediction for the second closure dam was in error due to the difference in basic criteria for transport and deposition in the model and prototype (noncohesive versus cohesive, respectively) and to the drastic change in hydraulic conditions with the second closure dam relative to the shoaling verification and to the noncohesive critical velocity for transport of the model sediment.

It is recommended that cohesive sediments should be modeled by the smallest and lightest model tracer that can be realistically used and that when tracer transport is stopped by subcritical bed shear stresses, the results should be identified as potentially erroneous. Also, physical model tracer tests should be avoided or carefully qualified when testing for the effects of drastic changes in the basic parameters governing cohesive sedimentation.

PREFACE

The research described in this report was conducted at the U. S. Army Engineer Waterways Experiment Station (WES) with funding by the Coastal Engineering Research area of the General Investigation Research and Development category funding of the Office, Chief of Engineers (OCE), U. S. Army.

Personnel of the Hydraulics Laboratory of WES performed this study during the period 1976 through December 1977 under the direction of Messrs. H. B. Simmons, Chief of the Hydraulics Laboratory; F. A. Herrmann, Jr., Assistant Chief of the Hydraulics Laboratory; R. A. Sager, Chief of the Estuaries Division; G. M. Fisackerly, Chief of the Harbor Entrance Branch; and W. H. McAnally, Jr., Estuarine Research Projects Manager. Data reduction was performed by Mr. B. G. Moore, under the supervision of Mr. J. V. Letter, Jr., Project Engineer. Messrs. Letter and McAnally prepared this report.

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

CONTENTS

	Page
PREFACE	1
CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT	4
PART I: INTRODUCTION	5
Objectives	5
Background	5
Approach	6
Estuarine Sediment Transport	8
PART II: BRUNSWICK HARBOR	14
Description	14
Construction of Dams	19
Data Collection	22
PART III: THE MODEL	26
Description	26
Appurtenances	26
Verification	29
Tests Conducted	32
PART IV: RESULTS	34
Tidal Elevations	34
Current Velocities	36
Flow Predominance	46
Sedimentation	48
Postconstruction Verification	50
PART V: DISCUSSION	53
Prototype Data	53
Modeling Techniques	56
Hydraulic Results	60
Sedimentation Results	64
PART VI: CONCLUSIONS	68
Model Predictions	68
Recommendations	68
REFERENCES	70
TABLES 1-20	
PLATES 1-23	
APPENDIX A: SUPPLEMENTAL PROTOTYPE HYDRAULIC DATA	A1
PLATES A1-A7	
APPENDIX B: PROTOTYPE SHOALING	B1

Introduction B1
Prototype Surveys B1
Description of Analysis B1
Results B3

TABLES B1-B9

PLATES B1-B9

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
cubic feet per second	0.02831685	cubic metres per second
cubic yards	0.7645549	cubic metres
Fahrenheit degrees	5/9	Celsius degrees or Kelvins*
feet	0.3048	metres
feet per second	0.3048	metres per second
inches	25.4	millimetres
miles (U. S. statute)	1.609344	kilometres
pints (U. S. liquid)	0.4731765	cubic decimetres
square feet	0.09290304	square metres
square miles (U. S. statute)	2.589988	square kilometres

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

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PHYSICAL HYDRAULIC MODELS

ASSESSMENT OF PREDICTIVE CAPABILITIES

MODEL STUDY OF SHOALING, BRUNSWICK HARBOR, GEORGIA

PART I: INTRODUCTION

Objectives

1. The primary objective of this study is to define the accuracy with which the results of tests conducted in physical hydraulic models predict the changes induced by modifications to estuarine systems. A secondary objective is to improve understanding of modeling techniques such that the value of physical model studies may be increased. This is one of a series of reports concerning specific model studies conducted at the U. S. Army Engineer Waterways Experiment Station (WES).

2. The Brunswick Harbor model studies were approved by the Office, Chief of Engineers (OCE), on 22 June 1964 and were authorized by the U. S. Army Engineer District, Savannah, on 20 July 1964. The studies were conducted in the Hydraulics Laboratory of the WES during the period June 1965 to May 1968. The purpose of this report is to examine the model study of Brunswick Harbor, Georgia, navigation project, comparing the model predictions with observed prototype behavior.

Background

3. For many years physical models have been used to predict the response of estuarine systems and harbors to alterations in their boundary conditions such as dredging, landfills, structures, and flow regulation. However, little attention has previously been given to careful assessments of the accuracy of the model predictions after the proposed modification is constructed in the prototype. Physical model predictions of tidal elevations and phases, current velocities, circulation patterns, and salinity intrusion are considered reliable; yet little has

been done to quantify the degree of confidence to be placed in these predictions. Other phenomena are not considered to be as reliably reproduced in physical models. Sediment transport and pollutant transport and dispersion have particularly suffered from the lack of detailed evaluation of model performance.

4. In order to bridge the gap between the reliability of modeling purely hydraulic phenomena and the art of modeling the transport phenomena, in 1971 OCE authorized the WES to begin a series of studies to evaluate model predictions of estuarine phenomena. This report is the third of the series. The first two reports were on the Delaware River Model and the Galveston Harbor Model (Letter and McAnally 1975, 1977). For a more complete background of the research project, refer to those reports.

5. The Brunswick Harbor model study (Herrmann and Tallant 1972) was conducted at WES during the period of June 1965 through May 1968. The primary purpose of the model investigation was to develop the most practical and economic means of securing and maintaining the authorized 30-ft* channel depth in the area of Brunswick Harbor in East River (Figure 1). The study also evaluated means of reducing shoaling in the Intracoastal Waterway in the Jekyll Creek area.

6. Based on the conclusions and recommendations of the model study, a closure dam was built in the upper portion of the East River and later a smaller closure dam was built in Academy Creek. Analysis of the prototype behavior following each closure dam construction in view of the model predictions of behavior for those conditions is the subject of this report.

Approach

7. There are several ways of comparing and analyzing data in evaluating a model's predictions of the effects of proposed construction.

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 4.

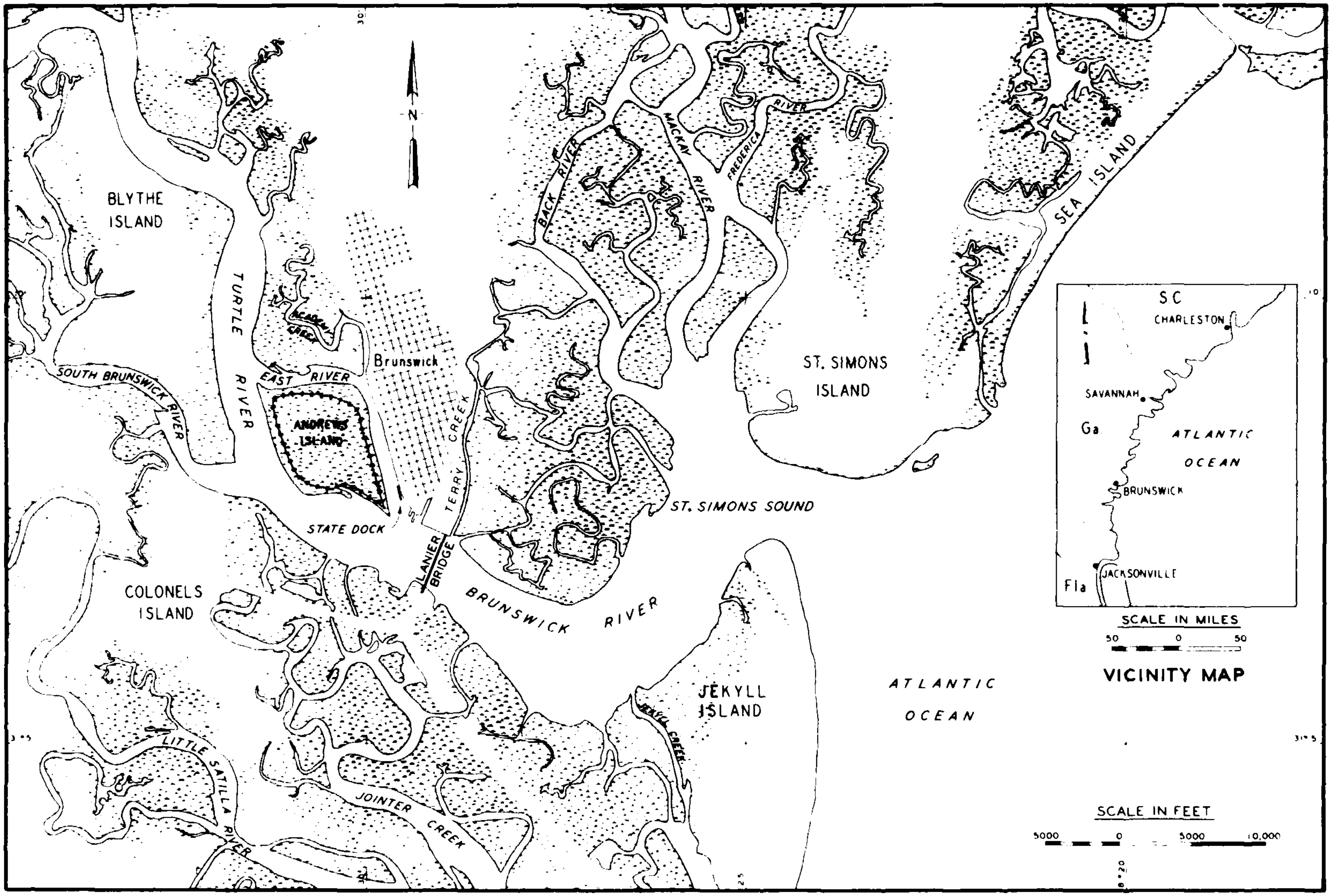

Figure 1. Location map

These methods are discussed fully in previous reports of this series.

8. The approach used in this study is to first directly compare hydraulic data collected in the prototype after construction with model hydraulic predictions. For tidal elevations, only direct comparisons of the tidal elevation will be presented and analyzed. For current velocity data, statistical analysis techniques are applied to the differences between model predictions and observed prototype velocities. The accuracy of model flow predominance compared with prototype flow predominance is also addressed.

9. Shoaling distributions within Brunswick Harbor for the model predictions and for the observed prototype shoaling are compared. Also, total shoaling in the harbor after construction relative to total shoaling before construction will be analyzed for both model and prototype. There will be no direct comparison of model and prototype shoaling volumes, as there is no means of scaling the fixed-bed model sediment volumes.

10. In order to provide a sound understanding of estuarine sediment transport and its modeling, the remainder of PART I of the report is devoted to that topic. The following PARTS II and III present descriptions of the prototype and model; PART IV presents the results of the study.

11. PARTS V and VI are devoted to a discussion of the results and to the conclusions of the study. In the discussion of the results, any peculiarities of the data are viewed with an eye toward the estuarine sediment transport processes to be discussed next.

Estuarine Sediment Transport

12. Estuarine sediment transport processes rank among the most poorly understood phenomena of the coastal zone and modeling them is certainly the least precise aspect of the modeler's art. A thorough discussion of estuarine sediment transport is well beyond the scope of this report; however, to provide a background against which to compare physical modeling methods, an outline of some important factors is

presented in the following paragraphs. For more detailed presentations, see Mehta and Partheniades (1973); Krone (1972); Partheniades (1971); Ippen (1966); and Krone (1962).

Transport of fine sediments

13. Sediment characteristics and their movement within an estuary are in part functions of their source. Potential sources of estuarine sediments include: (a) the upland drainage basin; (b) the estuary itself, through erosion of banks, bottoms, and marshes; (c) the ocean; (d) municipal and industrial wastes; (e) windborne sediment; and (f) organic materials. Different sediments will tend to predominate in different parts of the estuary.

14. Of concern here are the those fine-grained sediments--clays and fine silts--that dominate most estuaries. They are so small that they only occasionally deposit in the rivers that carry them as wash load to estuaries where water chemistry, estuary geometry, tidal currents, and density currents combine to trap fine sediments and cause their deposition. In some locations coarser sediments are found with the fines; however, fine sediments alone cause major shoaling problems in most estuaries, including Brunswick Harbor.

15. Individual clay particles will not settle under their own weight, even in still water, because their exceedingly small size allows thermal motion of water molecules to keep the particles in suspension. Only when the particles aggregate, forming porous composite particles of a number of individual particles, can settling begin. Surface electrical charges on the clay particles attract a layer of ions, making the particles mutually repulsive except at very short distances and preventing significant aggregation. If two particles collide in spite of this repulsion, short-range attractive forces bind them tightly together into aggregates. This aggregation process, called flocculation, increases with increasing numbers of particle collisions caused by higher sediment concentration, by increased turbulence in the flow, and by the presence of dissolved salts whose ions suppress electrical repulsion between particles. At some upper limit, turbulence may hinder flocculation by breaking aggregates as rapidly as they are formed.

Aggregates grow larger by colliding with individual particles and other aggregates when different settling velocities and flow velocity gradients permit them to be captured by faster moving ones. Other aggregation processes include agglomeration by filter feeding organisms and chemical cementation. When particle aggregates have grown to a size and weight sufficient to begin settling toward the bed, they become potentially depositable.

16. An aggregate approaching the bed can deposit if its shear strength due to interparticle bonds exceeds the shear stresses exerted on it by the shear gradient. If aggregate strength is exceeded and the bonds are broken, the resulting smaller pieces will probably be reentrained in the flow. If an aggregate survives the high shear zone near the bed and deposits, it forms bonds with particles in the bed and is shielded from the flow by surrounding particles; thus, it tends not to be eroded by the same flow conditions that allowed it to deposit. The above description of cohesive sediment behavior suggests that sediment beds that are undergoing deposition tend to do so without appreciable erosion, and those that are eroding tend to do so without appreciable deposition. This is in contrast to the live-bed concept of noncohesive sediments which is characterized by simultaneous interchange between material in transport and on the bed. Experimental evidence on this point is inconclusive, with studies by Krone (1962) supporting simultaneous erosion and deposition and studies by Mehta and Partheniades (1973) and others supporting exclusive erosion or exclusive deposition.

17. The behavior of a cohesive bed with depositable sediment is a function of the bed shear stress--below a certain minimum shear, available depositable sediment will quickly deposit; above that minimum and up to some maximum shear, available sediment will deposit at a rate dependent on bed shear; above that maximum, sediment will not deposit and erosion occurs at a rate dependent upon bed shear. The critical shear stress for erosion of a sediment layer increases if the weight of overlying sediment crushes the sediment structure, forming more particle bonds between aggregates.

18. Deposition of cohesive sediments is aided in several ways by

estuary water chemistry. As mentioned previously, dissolved salts encourage flocculation by suppressing electrical repulsion between particles; but the importance of this effect may have often been exaggerated, since only a few parts per thousand (ppt) salt concentration is necessary to initiate flocculation. Krone (1962) found in laboratory experiments that aggregate strength was independent of salt concentration above 1.2 ppt but that median aggregate settling velocities (and by implication, either size, shape, or density) increased with salinity up to 10 or 15 ppt. In static flocculation experiments, Sakamoto (1972) found that mean aggregate diameters increased with salinity as high as 30 ppt. He also found that illite and kaolinite aggregates exhibited increased densities in higher salinity waters by amounts several times that expected by the increased density of the entrained water. Other water chemistry effects on sediment transport may include bonding by organic constituents, water pH, and cementing of bed particles by precipitates.

19. Entrained water density may affect aggregate settling by the resulting slight changes in aggregate weight. Aggregates formed in low-salinity water will settle more slowly in high-salinity water until diffusion raises the entrained water salinity. Similarly, aggregates with high-salinity water entrained will more strongly resist suspension into lower salinity surface layers.

20. Although dissolved salts appear to affect flocculation and settling more with increasing salinity, the effect beyond a few ppt is relatively minor compared with circulation patterns caused by the density difference between fresh and salt water. Salinity-induced density currents--predominantly upstream flow in the lower layer and predominantly downstream flow in the upper layer--is one of the most important phenomena in estuarine sediment transport. Sediments traveling downstream near the bed encounter null points, where there is no net fluid transport in either direction, and tend to be concentrated there. Sediments settling downstream of the null point are trapped in a layer with net upstream transport and are carried upstream. Thus, suspended sediments are concentrated in a zone of little net transport, causing a turbidity maximum near the null point. The general area of the bottom

flow predominance null point is usually a zone of heavy deposition (Simmons 1965), though by no means is it the only zone of heavy deposition. Asymmetry of the flow's capacity to transport noncohesive sediments and erode cohesive sediments can cause the zone of heaviest deposition to be considerably upstream or downstream of the null point. Density currents also cause steep velocity gradients and flow turbulence resulting in accelerated growth of particle aggregates and therefore increased deposition.

21. Two additional factors--geometry and tidal flows--figure prominently in estuarine sedimentation. The most important geometric effect is the dramatic widening of the waterway that often occurs where the river enters the estuary. At this point current speeds drop considerably and much of the noncohesive sediment load may deposit. Tidal flows add to current speeds but, because of their oscillatory nature, also provide intervals of slack currents and rapid deposition. Due to geometry, multiple channels in an estuary divert sediment and discharge in uneven ratios and experience different phasing of tidal currents; and deep channels through shallow water create pools of quiet water that trap sediment or experience strong density currents. Nonuniform geometry and man-made structures create turbulence that increases the flocculation rate. During slack-water intervals, a substantial portion of suspended sediment may deposit, requiring vigorous flows to resuspend it. The relative scouring power of ebb versus flood flows is a determining factor in the direction of net transport at a location and in the supply of available sediment at adjacent locations.

22. From the preceding paragraphs, it can be summarized that estuarine sedimentation is dependent upon (a) the supply of depositable sediment and (b) flow conditions near the bed. The supply of depositable sediment is a catchall category, being a function of the character and amount of sediment, ambient water quality, and flow conditions throughout the water column. Flow conditions near the bed merely dictate whether deposition, erosion, or a stable bed will result, although they may limit what constitutes "depositable sediment." As an example of how these two criteria control sedimentation, consider first a zone of low

shear stress that does not experience significant deposition either because aggregation of sediments is not occurring or because nearby deposition has exhausted the supply of depositable sediment. However, a zone may have a high average bottom shear stress but have such an abundance of depositable sediment that all of the material deposited during slack-water periods cannot be eroded during strong flow periods. Thus, alteration of either sediment supply or flow conditions can significantly change patterns of deposition and erosion.

23. Transport of cohesive and noncohesive sediments shares this dependence upon the balance of bed shear against sediment supply but differs in that noncohesive sediment beds may experience simultaneous large-scale erosion and deposition while cohesive sediment beds may not. They also differ in that available noncohesive sediment tends to be depositable, while available cohesive sediment may not be, requiring a certain level of aggregation in order to become depositable.

PART II: BRUNSWICK HARBOR

Description

24. Brunswick Harbor is a saltwater embayment on the Atlantic coast 70 miles south of Savannah, Georgia (Figure 1 inset). It consists of St. Simon Sound, Brunswick River, Turtle River, East River, Back River, Mackay River, Academy Creek, and Terry Creek and is surrounded by numerous small creeks and marshes. The Altamaha River, branching to form Back River and Mackay River, discharges a portion of its flow into the north end of St. Simon Sound.

25. The existing navigation project consists of a channel 32 ft deep* by 500 ft wide across the bar; 30 by 400 ft from the entrance through St. Simon Sound, Brunswick River, and to the Georgia State Docks in East River; 27 by 350 ft from that point up East River to Academy Creek; 24 by 150 ft in Academy Creek; 30 by 300 ft in Turtle River to a point approximately 1000 ft below the Highway 303 Bridge; 20 by 150 ft in Back River; and 10 by 80 ft in Terry Creek. There are turning basins on the west side of the channel in East River and at the north end of the channel in Turtle River. The channel project prior to closure dams in East River and Academy Creek was completed in December 1960.

26. A 4350-ft-long stone jetty is at the entrance to East River on the southeast end of Andrews Island. Andrews Island is a dredged material disposal site that was completely diked in 1961.

27. Tides in Brunswick Harbor are of the semidiurnal type. Mean tide range is about 7 ft and spring tide ranges are about 8 ft. Tide ranges and mean tide levels for several locations in the harbor are shown in Table 1 (National Ocean Survey, 1975). The maximum phase lag of the tides from St. Simon Sound to the upper end of the navigation channel in Turtle River is about 50 minutes.

28. Freshwater discharges into Brunswick Harbor are usually negligible. The Altamaha River discharges an average flow of 12,600 cfs

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* Depths refer to mean low water (mlw).

(USGS 1975) on the north side of St. Simon Island but does not normally affect salinities in the harbor. During peak flows of the Altamaha, some freshwater flows enter the harbor through the Mackay River bringing suspended sediments and increasing shoaling rates (Neiheisel 1965). The maximum recorded flow of the Altamaha is 178,000 cfs gaged at Doctortown, Georgia (USGS 1975).

29. Currents in the harbor are essentially tidal. Maximum spring tide velocities are 4 to 5 fps in the Brunswick River and were 2.5 (flood flow) to 3.5 fps (ebb flow) in East River (Committee on Tidal Hydraulics 1971) prior to closure dam construction. Prior to construction of the closure dams (paragraph 6), tidal flow in East River had an overall net ebb discharge; however, flood flow volumes below 24 ft were substantially greater than ebb flow volumes at that depth (Harris 1963). This flood flow predominance in the lower water column plays an important role in the supply of sediment to the East River area.

30. Negligible freshwater discharges and vigorous tidal mixing combine to make Brunswick Harbor a well-mixed estuary with minimal vertical salinity gradients. The difference between surface and bottom salinities rarely exceeds 1 to 2 ppt (Committee on Tidal Hydraulics 1971). A survey in June 1963 (Harris 1963) found salinities ranging from 25 to 28 ppt in the upper portions of the harbor and 26 to 31 ppt in St. Simon Sound. Water temperature difference between the surface and bottom were within 1°F during the survey.

31. Sources of information on sediments in Brunswick Harbor are scarce and conflicting. Krone (1963) found that the clay fraction of samples provided to him by the Savannah District was approximately 80 percent of the samples by weight, and 72 percent of the sample was smaller than 1 micron, indicating extremely fine material. Neiheisel (1965) found substantially different results, with median sizes ranging from 0.10 mm in Turtle River and 0.24 mm in East River to 0.46 mm in St. Simon Sound. Neiheisel found that the sediments in East River were 22 percent of clay size, containing about 16 percent organic matter. There are several sources of these sediments--the Altamaha River by way of Mackay River and the tidal inlet between St. Simon and Jekyll

Islands and also from the Turtle River (Neiheisel 1965). The extensive marshes in the harbor area may serve to store sediments for later suspension and deposition in the channel in East River.

32. Bottom sediment samples were collected 19 October 1977 in East River by the Savannah District using a drag bucket sampler. The samples, about 3 pints each, were taken along the navigation channel center line approximately at sta 70+50, 87+00, and 105+00 (Figure 2) and forwarded to the WES for analysis. The samples were very similar, all dark gray plastic clays, with organic content varying from 8.9 to 10.0 percent. The specific gravities were 2.67, 2.66, and 2.69 for the above stations, respectively, increasing in the downstream direction. Results of the sieve analyses on the samples are plotted in Figure 3 along with the results found by Krone (1963) and Neiheisel (1965). Recent samples show approximately 95 percent of the material by weight being finer than 0.037 mm. Recent samples do not differ substantially from Krone's data, but are much finer than found by Neiheisel. Neiheisel's samples were collected during the period 12-20 June 1963 which was just after completion of dredging, while the recent samples were taken more than three months after completion of the last dredging operation. This factor could partially explain the differences in the observed data. Also, with the construction of the two closure dams in East River and Academy Creek, the East River harbor area is more like a slip than before, and a shift to a finer sediment supply would be expected in the recent samples.

33. Suspended sediment concentrations were measured during a survey on 5 June 1963 (Harris 1963). Concentrations of 0.01 to 0.034 g/ℓ were measured at the surface in the immediate harbor area with high-water concentrations generally larger than those at low water. Bottom concentrations showed more variability, ranging from about 0.02 to 0.01 g/ℓ. The tidal range for this period was about 7 ft.

34. The navigation channel was enlarged in several reaches in 1960 to its present dimensions. The channel over the bar was deepened 2 ft, the channel from St. Simons Sound to Brunswick Point and through Turtle River was deepened 3 ft, and the channel in East River was

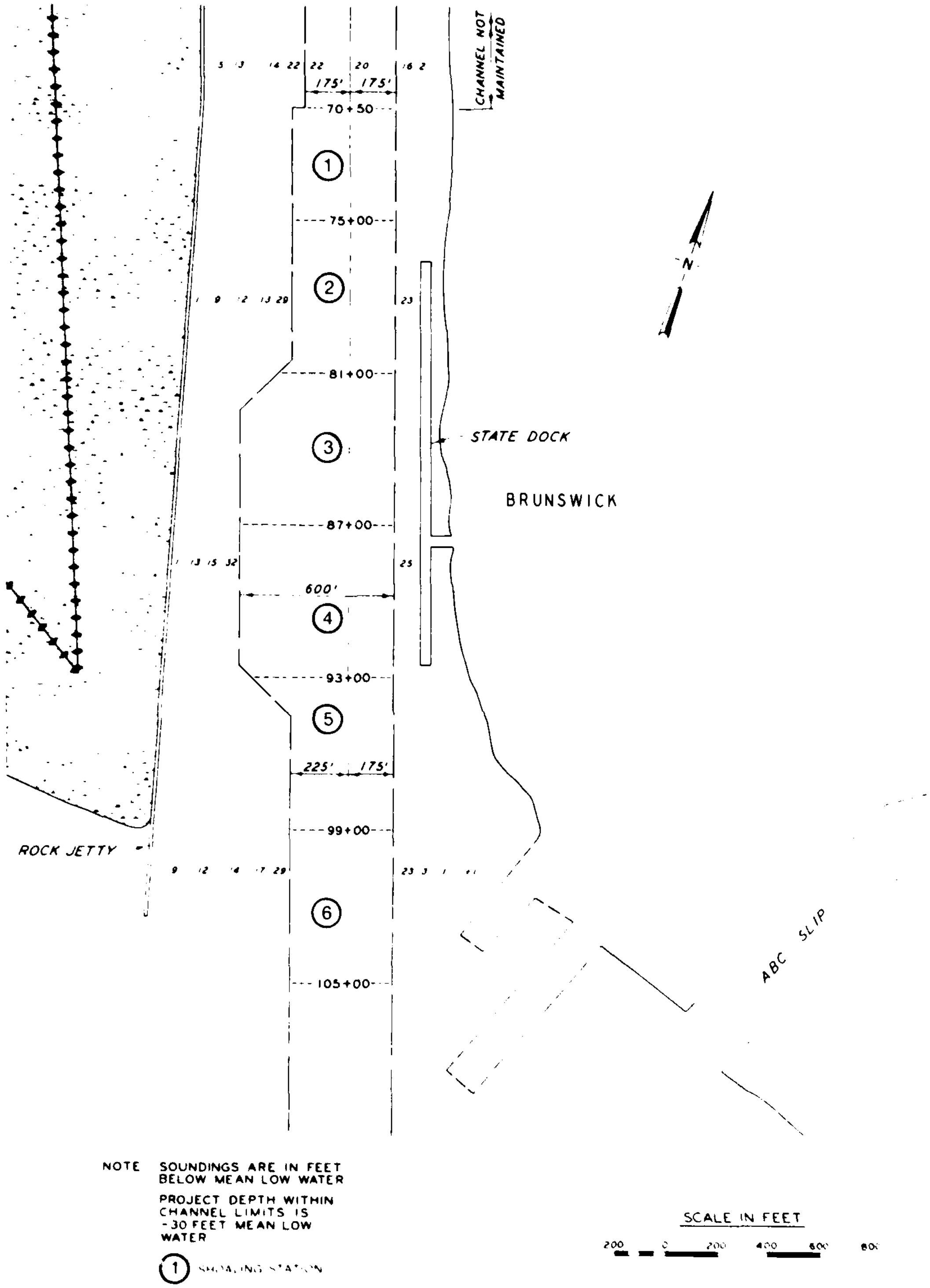

Figure 2. East River, port shoaling sections

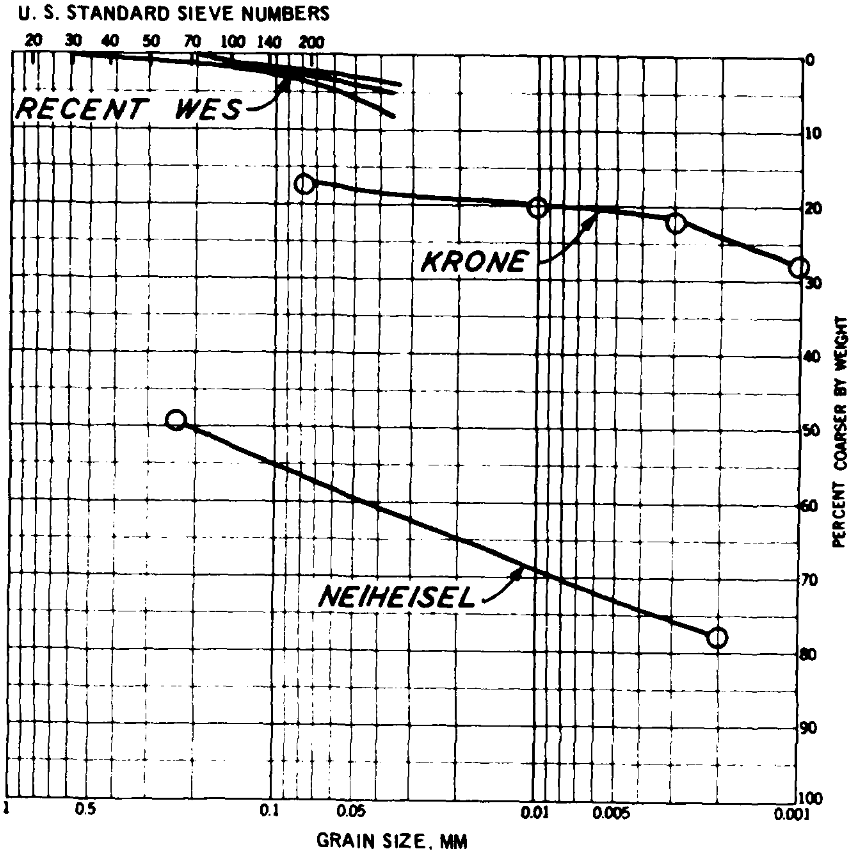

Figure 3. Grain sizes in Brunswick Harbor, East River

deepened 3 ft and widened 50 ft up to Second Avenue. Prior to the enlargement, dredged volumes in the inner portions of Brunswick Harbor were quite small as shown in Table 2 (OCE 1953-1974). From 1953 through 1959 less than 1 million cu yd were dredged from the navigation channel. After deepening, the dredged volumes in East River averaged 566,000 cu yd per year from 1961-1968. After the construction of a closure dam in 1969, the dredging volume in East River averaged 428,000 cu yd per year from 1970-1974, with occasional dredging in Terry Creek.

Construction of Dams

35. The project as it existed prior to the construction of the main closure dam in East River was completed in 1960. Construction of the main closure dam in East River (Figure 4) began 5 June 1969 and was completed 12 November 1969. That project was referred to as the partial closure plan during the model study.

36. The main closure dam, located approximately 700 ft upstream of the mouth of Academy Creek, was constructed from dredged material. The impermeable dam is about 600 ft long with a 20-ft-wide crest and side slopes at the natural angle of repose of the material. The top elevation of the dam is 14 ft above mlw. The period from the completion of the main closure dam in East River until the initiation of construction on the Academy Creek closure dam was about 5 years, November 1969 to March 1975.

37. The drainage canal from Academy Creek to the upstream side of the main closure dam in East River (Figure 5) was constructed under contract during the period 13 May 1974 to 19 June 1975. The canal is approximately 2700 ft long with a bottom width of 6 ft and elevation of 2 ft below mlw, and with side slopes of 1V on 3H. Constructed along with the canal was a dike of approximately the same length running adjacent to the canal. The small dike has a top width of 3 ft and a top elevation of 10 ft above mlw and is located a minimum of 30 ft from the top bank of the canal. The side slopes of the small dike are the natural angle of repose for the material.

38. The closure dam in Academy Creek (Figure 5), constructed during the period 21 March 1975 to July 1975, is located approximately 3800 ft upstream from the mouth of Academy Creek at East River. The center line of the dam is on the same alignment as the center line of the closure dam in East River. The dam has a top width of 10 ft at an elevation of 12 ft above mlw and is approximately 250 ft long. Side slopes for this dam are also the natural angle of repose of the material. The present project (Figure 5) was referred to as the alternate closure plan during the model study.

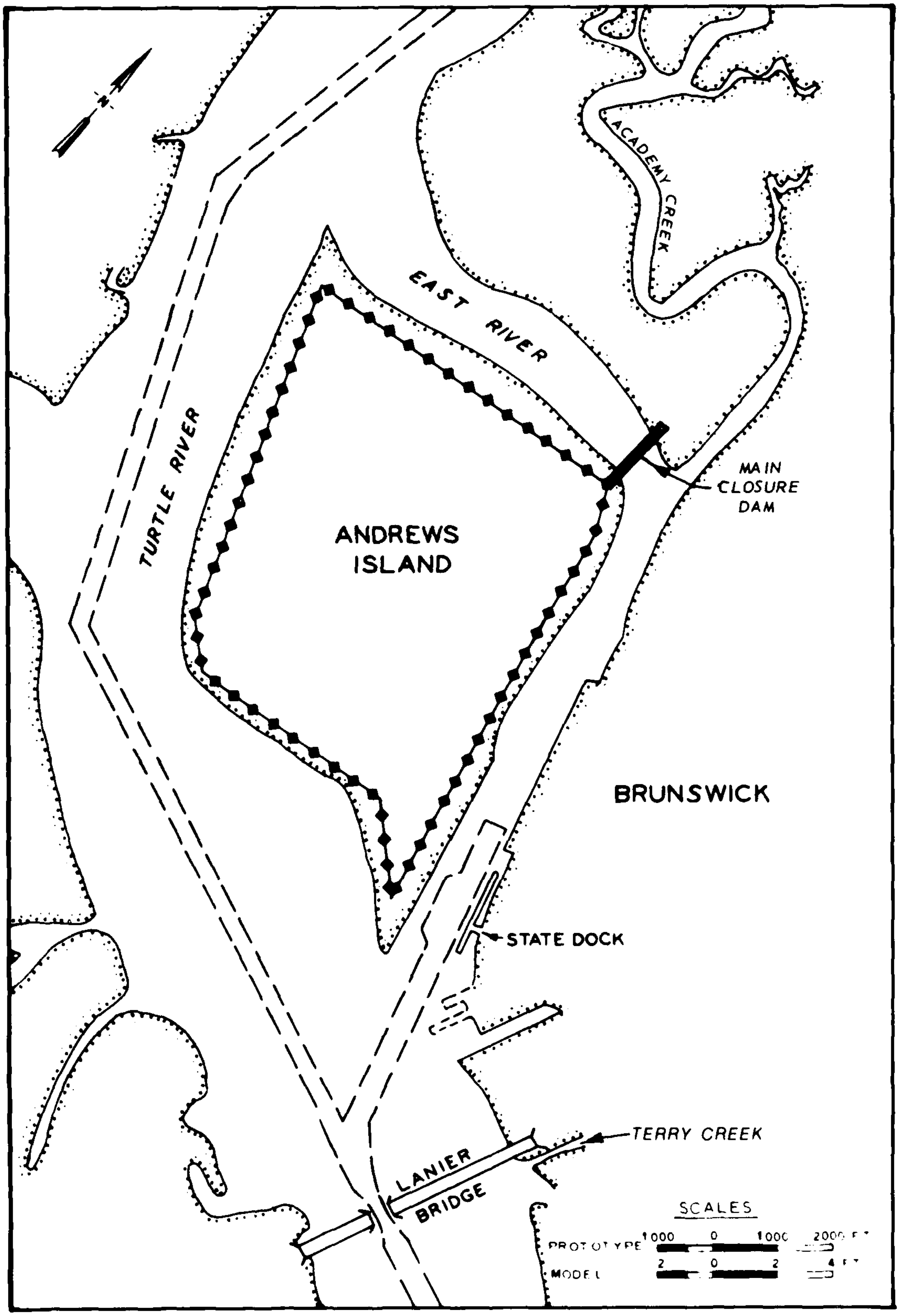

Figure 4. Partial closure plan

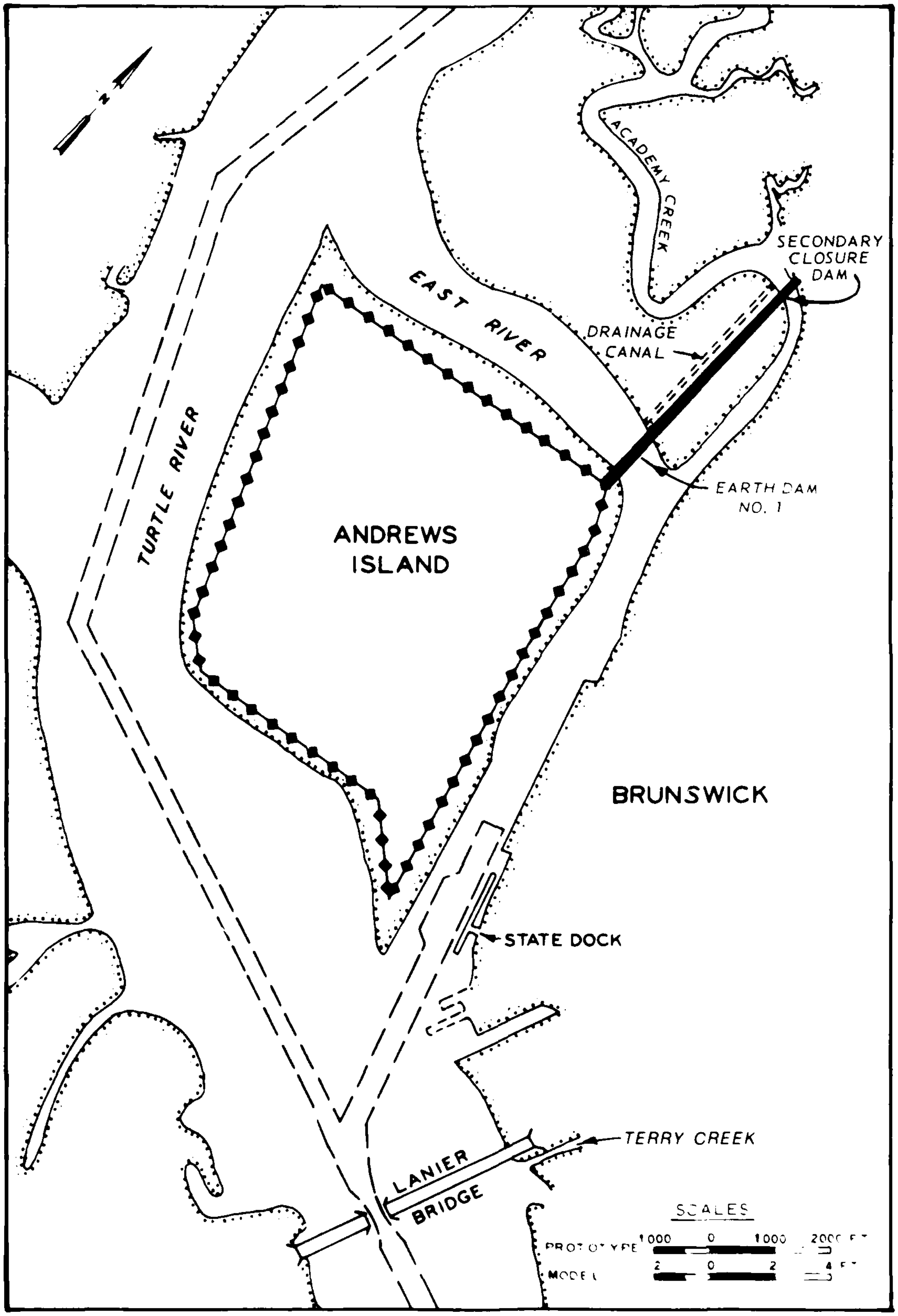

Figure 5. Alternate closure plan

Data Collection

39. The Savannah District provided WES with the required data for original verification of the physical model. These data consisted of current velocities; computations of flow volumes; float surveys; temperature, salinity, and suspended sediment concentrations; and information regarding the source of shoaling material for the harbor area based on Neiheisel's (1965) work.

40. Postconstruction prototype data collection consisted of a hydraulic data survey of current velocities and tidal elevations in the vicinity of the harbor on 20 March 1976 over a complete semidiurnal tidal cycle plus shoaling volumes and bathymetric surveys for the period 1969 to 1976. The prototype conditions for the hydraulic data survey corresponded to those of the alternate closure plan.

41. The postconstruction tidal survey data stations are located in Figure 6. The survey was scheduled for a period when the predicted tide at the East River port corresponded to the tide used in the model testing. During the survey, tides were recorded at the State Dock in East River (gage 3), at the State Highway Department dock in Terry Creek (gage HDD), in the mouth of South Brunswick River (gage 1). Tide data were collected at the Highway 303 Bridge (about 2-1/2 miles upstream of range 4, Figure 6), but the gage malfunctioned prior to the survey on 20 March 1977.

42. Tide data were recorded by punched paper-tape flotation level recorders of a spring counterbalance type. The frequency of the recordings was every 6 min. Tide recorder floats were suspended in a 4-in.-diam PVC pipe with a 3/8-in. orifice in the bottom. The precision of the recorder is to the nearest 0.01 ft; however, the accuracy of the overall gaging procedure is believed to be ±0.05 ft.

43. Current velocities were measured at the 12 stations shown in Figure 6 for a 13-hr period beginning 0800 hr EST on 20 March 1977. Measurements of current speed and direction were taken hourly at the surface, middepth, and bottom for each station. Current measurements were made with a speed sensor and a direction sensor suspended by a wire

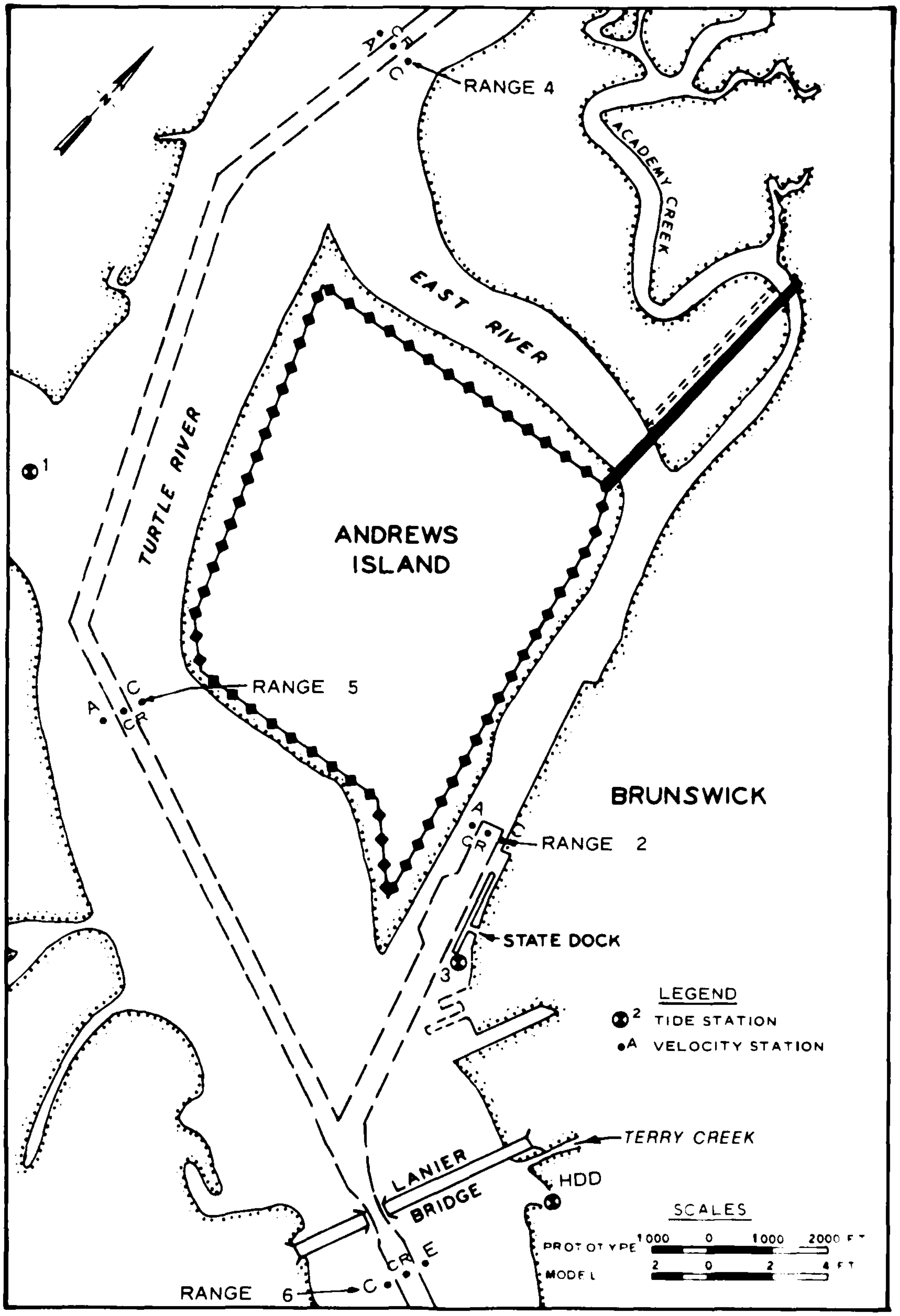

Figure 6. Postconstruction tidal survey data stations

rope from a support frame and winch located in a boat. The metering assembly was weighted by a streamlined weight.

44. The current speed indicator used for the survey was a vertical-axis-cup type with direct readout. Readout from the indicator is in feet per second with minimum scale graduations of 0.2 fps. The meter exhibits linearity of ± 5 percent from 0.2 to 7 fps and error due to temperature is approximately 0.05 percent per degree Fahrenheit deviation from 75°F. The threshold velocity is about 0.2 fps.

45. The direction indicator is a remote reading magnesyn compass designed by WES that indicates the magnetic north azimuth of the direction from which the current is flowing. The readout device has a precision of ± 2 deg, but accuracy is dependent upon the balance of the streamlined weight and the strength of the current available to turn it. For currents greater than 0.5 fps, the accuracy is ± 10 deg. For lower velocities, accuracy is reduced to ± 25 deg or worse when waves cause boat motion and when tidal currents slacken and turn.

46. Hydrographic sounding sheets for before- and after-dredging conditions in East River were supplied by the Savannah District office. All available sheets since 1962 were compiled. The soundings were made with a fathometer strip chart recorder for vertical control and the triangulation system for horizontal control. The accuracy of the control used during these surveys is dependent on many factors. Vertical control is affected by fathometer accuracy; vessel loading and attitude; wave action; vessel pitching, rolling, and heaving; correction for tidal fluctuation; and water temperature and salinity. Horizontal control accuracy is dependent on angular uncertainty, distance between the boat and transit, speed of the boat, and human error in reading the transit. With reasonable concern for these factors, the error in vertical control can be reduced to the effects of wave action and vessel motion. For the Brunswick Harbor area under average wave conditions, the vertical control may be assumed to be accurate within +0.5 ft. The various periods for which prototype hydrographic surveys were available for this study are indicated in the following tabulation:

Condition	Period of Prototype Data Used
Model Verification	Dec 1962 - Sep 1965
Preconstruction	Sep 1962 - Mar 1969
Partial Closure Plan	Nov 1969 - Apr 1975
Alternate Closure Plan	Jul 1975 - Sep 1976

PART III: THE MODEL

Description

47. The Brunswick Harbor model reproduced approximately 67 square miles of prototype area (Figure 7), from the ocean end of St. Simons Sound, north to the confluences of the Mackay and Back Rivers with St. Simons Sound, upstream along Brunswick River, and to the upper reaches of South Brunswick River and Turtle River. On the south side of St. Simons Sound, the Intracoastal Waterway from Brunswick River through Jekyll Creek to U. S. Highway 84 Bridge was reproduced. Throughout the modeled area the extensive system of saltwater creeks and marshes that affect tidal action was reproduced.

48. The fixed-bed model was constructed of concrete to linear scale ratios, model to prototype, of 1:500 horizontally and 1:100 vertically. The latest hydrographic surveys available at the time of model construction were used for molding the model bed. If shoals were observed within the navigation channels, the model was molded to full project dimensions in those areas. The model was approximately 195 ft long at its longest point and 40 ft wide at its widest point, covering an area of about 7500 sq ft. One prototype semidiurnal tidal cycle of 12 hr 25 min was reproduced in the model in 14.9 min.

Appurtenances

49. The model was equipped with the necessary equipment to reproduce and monitor hydraulic and shoaling phenomena. This equipment included a tide generator; recorder and controls; current meters; tide gages; and shoaling injection, recovery, and measuring equipment.

50. Tidal fluctuations in St. Simons Sound were generated by maintaining precise imbalances between a pumped inflow into the model and a gravity return to the sump, corresponding to the instantaneous tidal discharge at the ocean end of St. Simons Sound. This imbalance was adjusted over the tidal cycle by a mechanized valve on the gravity

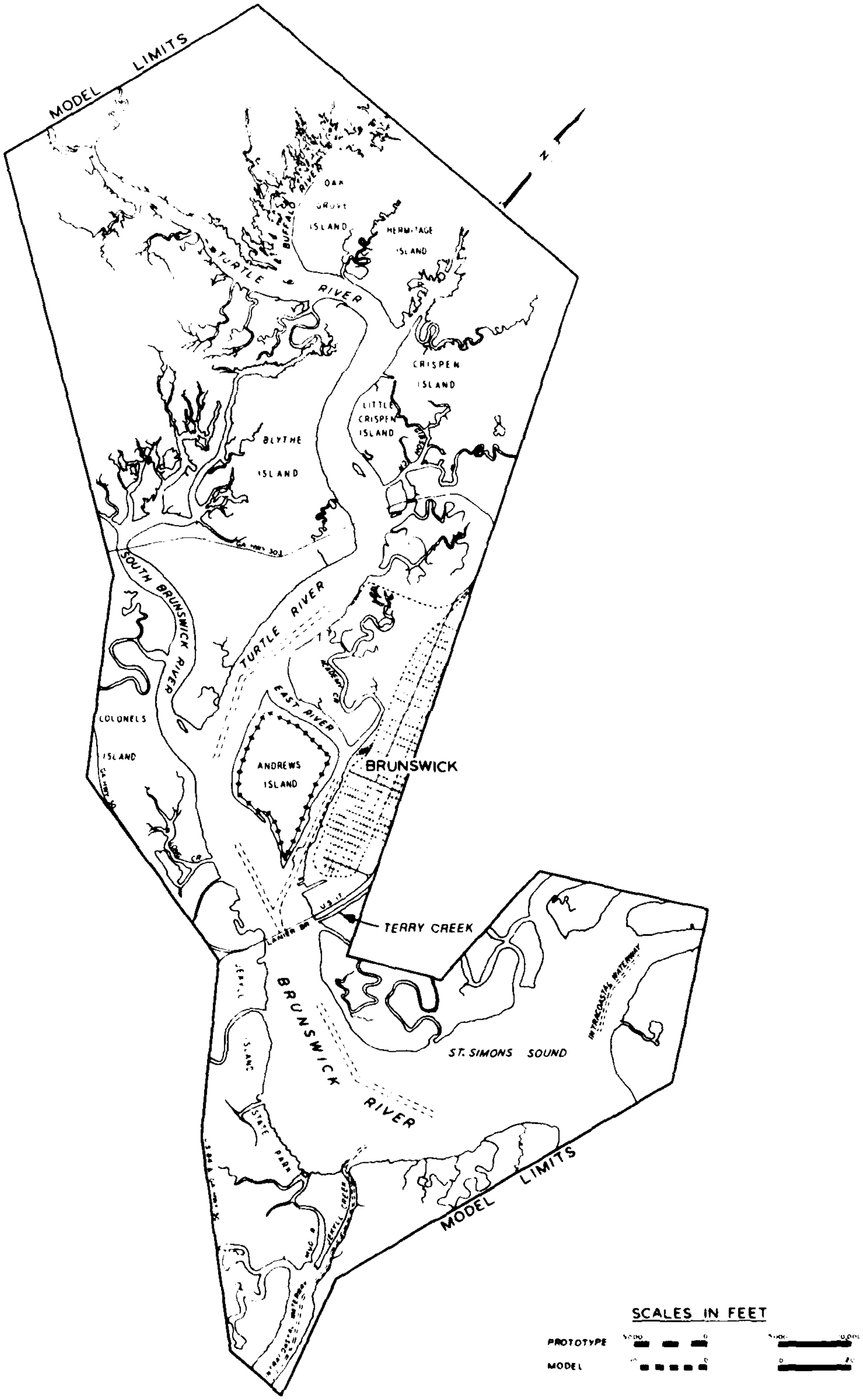

Figure 7. Model limits

return line until the required tide at State Dock was reproduced.

51. The tide control station at State Dock was equipped with a continuous recorder with which to visually check the accuracy of the tidal reproduction. The tide control was accomplished by controlling a reversible drive motor on the return line valve by adjustable cams through a system of mercury switches. The actual adjustment of the tide was made on the adjustable cams.

52. Measurement of current velocities was made with miniature Price-type meters (Figure 8) calibrated frequently to ensure accuracy.

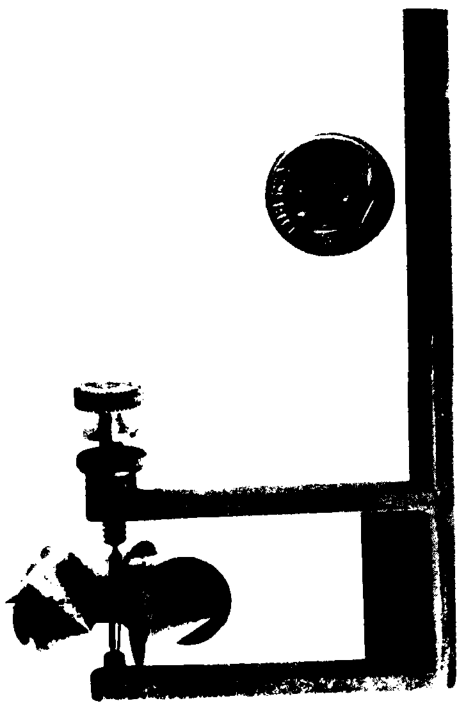

Figure 8. Miniature Price-type current meter

The meter cups were about 0.04 ft (4 ft vertically in prototype) in diameter and the mounting wheels about 0.11 ft (55 ft horizontally in prototype) in diameter. The center of the cups was about 0.045 ft (4.5 ft prototype) from the bottom of the meter frame. This miniature meter measures an average current speed over an area equivalent to 220 sq ft prototype. The threshold velocity for the meters was approximately 0.05 fps (0.5 fps prototype), and measurements are accurate above that value to ± 0.01 fps (0.1 fps prototype). Whenever the flow dropped below the threshold of the current velocity meters but motion of the water was still discernible, the flow was recorded as ebb or flood and a value of 0.2 to 0.3 fps assumed.

53. Model tidal elevations were recorded by permanently mounted surface-piercing point gages. The gages consisted of a pointed steel

rod attached to a vernier graduated to 0.001 ft (0.1 ft prototype). Measurements are considered to be accurate to the nearest 0.1 prototype foot.

54. The Brunswick Harbor model study used a tracer material--gilsonite--to simulate estuarine sediment transport. Gilsonite is a finely ground, noncohesive asphaltic material with a specific gravity of about 1.04. The grinding process produces a graded composition of grain sizes, but two size classes of material are used--a fine grade with a median grain size of about 0.3 mm and a coarse grade with medium grain size of about 0.6 mm. The choice is dependent upon model scales and currents in the model. For the Brunswick Harbor model the coarser material was used, with a size range from 0.4 to 0.7 mm.

55. There are two methods of injection of the gilsonite into the model. Both use a slurry of gilsonite and water. One method is to pump the slurry into a perforated pipe running along the axis of the estuary. The pipe is fixed just above the water surface and the gilsonite slurry flows through the perforations into the water. The other method of injection uses a trough with tubes extending below the water surface, which is placed at a channel cross section perpendicular to the flow. For either injection method, the gilsonite is transported at first in suspension and then mostly near the bed until it deposits on the model bed. The injection is made for fixed intervals during portions of the tidal cycle. At the end of the test, the gilsonite is retrieved from marked sections of the model, and the volume accumulated in each section is measured.

56. Retrieval of the material is accomplished through a suction tube with flared pickup head connected to an aspirator through a flexible hose. Measurements are recorded to the nearest increment of 5 cc; hence, an accuracy limit of ± 2.5 cc is inherent in the data. That accuracy is believed to include most other error factors.

Verification

57. The importance of model verification was summarized very well

by the authors of the original model study report (Herrmann and Tallant 1972): "It should be emphasized that the worth of any model study is wholly dependent on the proven ability of the model to produce with a reasonable degree of accuracy the results which can be expected to occur in the prototype under given conditions. It is essential, therefore, before any model tests are undertaken of proposed improvement plans, that the required similitude first be established between the model and prototype, and that all scale relations between the two be determined."

58. Verification of the Brunswick Harbor model consisted of two phases:

a. Hydraulic verification, ensuring that the model accurately reproduced all hydraulic phenomena of importance to the problems being studied.

b. Shoaling verification, which confirms the ability of the model to reproduce the location and distribution of prototype shoaling.

59. Hydraulic verification of the model consisted of first reproducing the prototype tidal elevations at State Dock for the prototype data periods. Then current velocities were measured at each survey station. Whenever discrepancies between model and prototype velocities were observed, the roughness near the station was adjusted. This procedure continued until reasonable agreement was obtained. The roughness consisted of 1/8-in.-thick by 1/2-in.-wide copper strips that extended from the bottom to just below mlw for the deep channels. The tidal marshes required greater resistance to flow, so a thin layer of stucco was placed on the marshes and roughened with a mason's float. The quality of the hydraulic verification will be discussed in PART IV.

60. Brunswick Harbor area, as previously discussed, has very little freshwater inflow and the vertical salinity gradients are very small, having a very minor role in the shoaling problems. Therefore, no freshwater flows were introduced into the model, and the model was operated with a constant salinity. Salt water was used rather than fresh water in order to reduce the effective density of the model sediment.

61. Shoaling verification for the Brunswick Harbor model was a

trial-and-error process to develop the test procedure that results in the most accurate reproduction of the location and distribution of shoaling during some prototype period.

62. Calibration of a model tracer procedure typically consists of attempting to reproduce typical hydrodynamic and shoaling conditions for a period of time for which prototype hydrographic or dredging data provide an estimate of deposition and erosion in the area of interest. Adjustments are made to the model and test procedures until the tracer distribution is similar to the shoaling volume distribution observed in the prototype. Adjustment may include changes in one or several of the following:

- a. Size of model sediment particles.
- b. Specific gravity of model sediment.
- c. Rate of tracer injection.
- d. Location of tracer injection.
- e. Times of tracer injection.
- f. Roughness element arrangement.
- g. Test duration.
- h. Tidal range.
- i. Freshwater discharge.
- j. Water salinity.

In addition, it may be necessary to simulate some unusual phenomena that exert a major influence on the sedimentation processes (e.g. the resuspension of sediments in the navigation channel resulting from ship passages). When model conditions and test procedures that satisfactorily reproduce available prototype sedimentation data are developed, the model is considered to be verified.

63. The final procedure developed to obtain shoaling verification used a 7.4-ft mean tide at State Dock. The injection procedure was the trough method, located at velocity range 6 on flood phase and range 4 on ebb phase. The model was operated with a constant 30-ppt salinity throughout to reduce the effective sediment density. Injection was accomplished over three tidal cycles using a 10 percent gilsonite slurry; the total amount injected was 12,000 cc of gilsonite. The model was

then operated for 20 cycles without injection. No appreciable sediment movement was observed after 10 cycles without injection.

64. Quality of the shoaling verification will be discussed in PART V.

Tests Conducted

65. Many plans were tested in the model study to determine their ability to reduce shoaling in the East River port area. Two approaches were generally taken toward remedying the problem, and all plans fall into one of these categories. The first approach was to place a closure dam in East River, varying in location. The closure was intended to alter the net flood flows near the bottom at the downstream end of East River, which were considered a source of shoaling material for the port. The other approach was to increase the velocities in East River to flush the fine material from the harbor. This flushing action was envisioned as a result either of dikes adjacent to the harbor in East River or by dredging a channel completely through East River to increase the volume of flow.

66. As with most model studies, base shoaling and hydraulic tests were conducted with the then existing project in order to develop a reference for comparison of the plans tested. Hydraulic and shoaling tests were then conducted for the plans considered, and a decision for construction was made based partly on the results of these tests. For the hydraulic tests of the base and plan conditions, the operating procedures used in the model were the same as those used in the model hydraulic verification. Likewise, in the shoaling tests, the test procedures for the base and plan conditions were identical with the procedures used during shoaling verification.

67. Shoaling tests also were conducted to determine the effects of the tide range on the shoaling in East River. Tide ranges of 5.2 ft (neap), 7.4 ft (mean), and 10.3 ft (spring) at State Dock control gage were tested toward that end. These tests showed that the total volume of shoaling increased with tide range.

68. The plan constructed in the prototype was one of the closure dams. The first phase of construction was a closure dam just upstream of the mouth of Academy Creek. This location was chosen over one downstream of Academy Creek in order to leave open the option of further harbor development in Academy Creek. This first phase of construction corresponded to the "partial closure plan" tested in the model study. Only shoaling and sewage effluent dispersion tests were conducted for this plan; thus, no model observations were made of tides or velocities. Because conditions in the model were changed prior to conducting these tests, shoaling tests of the partial closure plan were made only for a deepened (36 ft deep) navigation channel with a mean tide and with a spring tide.

69. The second phase of construction involved the additional closure dam in Academy Creek and the drainage canal, as discussed in paragraphs 37 and 38. This condition corresponds to the "alternate closure plan" tested in the model study. This plan was tested hydraulically in the model with the 36-ft-deep navigation channel and a mean tide. Hydraulic tests were not conducted with the 30-ft channel because of the similarity of this plan with a plan involving a dam across East River just downstream of Academy Creek. Shoaling tests of the alternate closure plan were conducted with a mean tide for both 30- and 36-ft-deep channels.

PART IV: RESULTS

70. Gaging stations used in the original model study are shown in Figure 9. Velocity data stations used for hydraulic verification of the model were: 1A, 2CR, 3A, 3CR, 4CR, 5A, 5CR, 5C, 6A, 6B, 6C, 6CR, 6E, 6F. Gage 3 (State Dock) was the only location verified for tidal elevations.

Tidal Elevations

71. The original verification of tidal elevations was accomplished only at sta 3 (Figure 9), the State Dock at Brunswick Harbor. Verification consisted of reproduction of four separate prototype tidal events; these four tides and their reproduction are shown in Plates 1 and 2.

72. Model tests included, however, the measurement of tidal elevations at a number of other gaging locations--some for comparison with model predictions, others for reference only. The postconstruction prototype data collection included two other gaging stations, one of which, sta 1 in the mouth of South Brunswick River, is used for postconstruction verification. Tidal data for the other station (sta HDD, Figure 6) are presented in Appendix A. Comparisons for model and prototype postconstruction tidal elevations at sta 1 and 3 are shown in Plate 3.

73. The prototype tide during the postconstruction prototype data collection program was very similar to but not identical to the model tide used during the testing program. At the model control gage (sta 3, State Dock), the time of high water was the same in model and prototype (about hour 8.0). The elevation at high water in the prototype (4.0 ft) was 0.2 ft higher than in the model. The time of the model low water occurred about 15 min before prototype low water (hour 2.0), and the prototype low water was 0.2 ft lower than the model low-water elevation (-4.2 ft). The prototype tide range was 8.4 ft compared with 8.0 ft for the model. The general shape of the tides is somewhat different. The

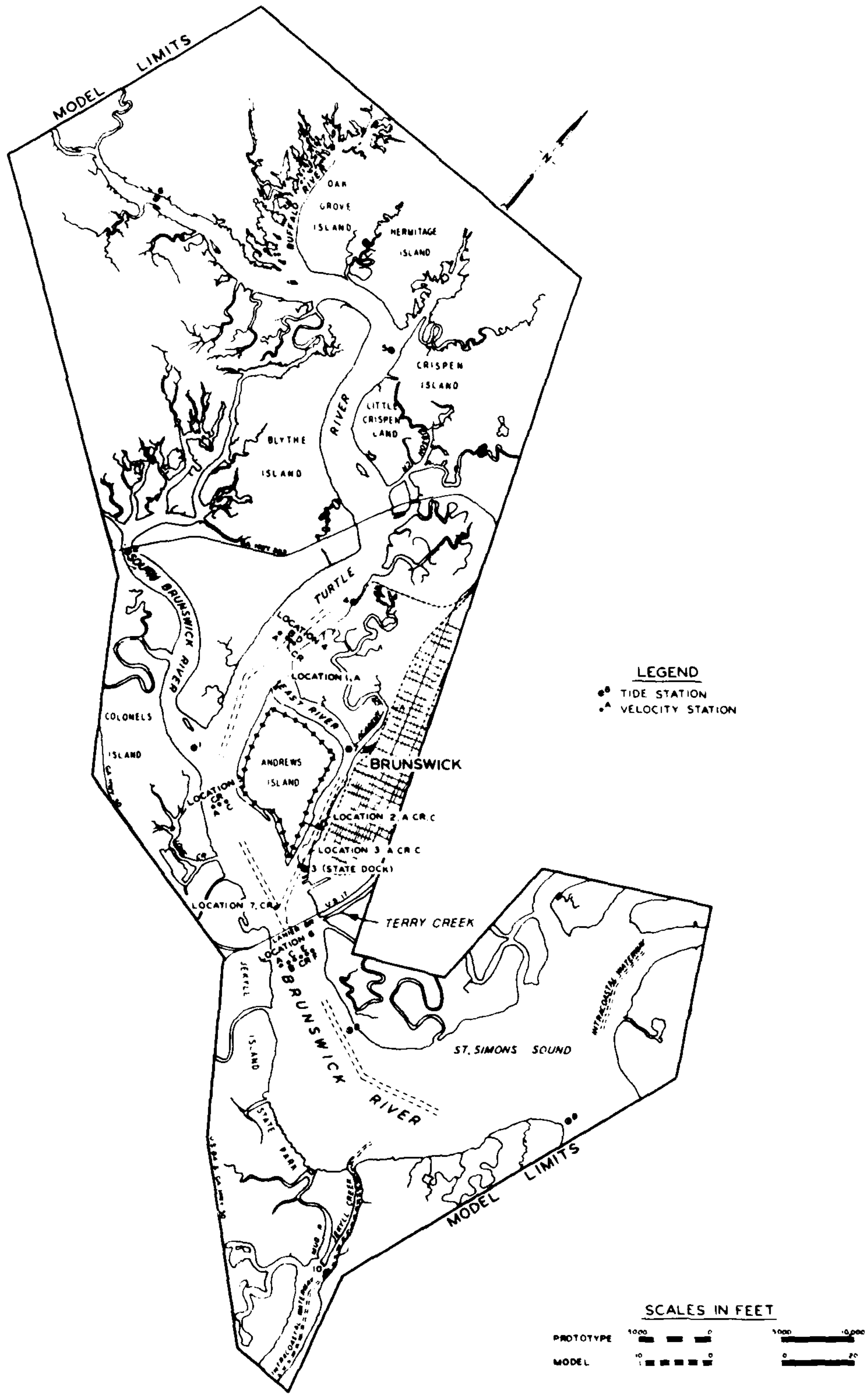

Figure 9. Location of velocity stations and tide gates

model goes into low water more quickly than the prototype and is slower into high water. This is evidenced by the model mean tide level (+0.22 ft msl) being higher than the prototype mean tide level (+0.11 ft msl). Because of the differences in tidal range and the shape of the tidal curves it would be anticipated that maximum prototype velocities should be slightly greater than corresponding model velocities. The agreement is considered to be adequate to directly compare tides, velocities, and flow predominance between model and prototype.

74. The time and the elevation of high water are the same for model and prototype at sta 1, in the mouth of South Brunswick River. The model low water is 0.6 ft higher than the prototype low water, but the times are the same. The model tide range of 8.1 ft is low relative to the prototype range of 8.7 ft. Because of the difference in shapes of the tides of model and prototype and the differences in the low-water elevations, the model mean tide level (+0.35 ft msl) at sta 1 is 0.5 ft higher than the prototype mean tide level (-0.15 ft msl).

Current Velocities

75. Before investigating the model predictions of current velocities it is appropriate to consider the quality of the original model verification of current velocities.

Original verification

76. Plates 4-11 present typical agreement obtained between model and prototype during original verification of current velocities. These stations were chosen for illustration because they are the stations used for postconstruction verification. Discussion of the current velocity verification will be based on all of the verification data, as presented by Herrmann and Tallant (1972). Comparison of mean current velocities on flood and ebb phases of the tidal cycle for model and prototype for all original verification data for postconstruction verification (PCV) stations is presented in Table 3.

77. The quality of the model original current velocity verification for the Brunswick Harbor is comparable to many other physical model

studies and could be considered good. However, it is important that any general deviations throughout the data be noted. Table 3 shows that at bottom the model current velocities are often stronger than the prototype velocities on both flood and ebb phases of the tidal cycle and are seldom lower on either (never on ebb). Middepth velocities in the model tended to be somewhat less than the prototype velocities on flood phase and greater on ebb phase. Surface velocities matched fairly well on both flood and ebb phases at most stations. Model flood velocities were generally less than prototype flood velocities, particularly at middepth. Model ebb velocities were generally greater than prototype ebb velocities, especially at bottom and at middepth.

78. Model slack waters (particularly after ebb flow) generally occurred slightly later than prototype slack waters. The model low water at State Dock in East River occurred slightly late for two of the four model verification tides, but the times of high waters coincided on each date. The 14-15 February 1963 prototype tide data at State Dock was only a partial record and did not include low water.

Postconstruction verification

79. The stations (Figure 6) used for the postconstruction verification of current velocities are 2CR, 4CR, 5A, 5CR, 5C, and 6CR. The prototype data were collected 20 March 1976, when an 8.4-ft tide was observed at the State Dock (remember that the model tide range was 8.0 ft). Other data collected during the prototype survey but not used in this analysis are presented in Appendix A.

80. Model data used for postconstruction verification come primarily from the Plan 1 tests. Current velocity data at sta 2CR in East River, however, is taken from the alternate closure plan mean tide test which had a 36-ft navigation channel. The relative effects of the alternate closure plan and Plan 1 are approximately the same for stations in Brunswick River and Turtle River, both plans having the tidal exchange for at least most of Academy Creek and its marshes through the upper end of East River. In East River at sta 2CR, however, the effects are somewhat different for the two plans, with the alternate closure plan providing tidal prism exchange for some distance up Academy Creek

that is cut off with Plan 1. Therefore, it was decided to compare the model alternate closure plan (36-ft channel) current data with the postconstruction prototype data (30-ft channel) at sta 2CR.

81. Plots showing the comparison of model (Plan 1 with 30-ft channel or alternate closure plan with 36-ft channel) with postconstruction prototype (alternate closure plan with 30-ft channel) velocities are presented in Plates 12-17. Mean current velocities of model and prototype are presented in Table 4, along with the ratios of model to prototype mean velocities, for both flood and ebb phases of the tidal cycle.

82. At sta 2CR (Plate 12), located in East River near the upper end of the 30-ft project, model velocities are extremely low relative to the prototype at all three depths. Model mean flood flows at sta 2CR were only 20 percent, 30 percent, and 35 percent of the prototype mean flood flows at surface, middepth, and bottom, respectively. Model ebb mean flows were also of a much smaller magnitude than in the prototype: 26 percent, 28 percent, and 27 percent of the prototype flows at surface, middepth, and bottom, respectively. Duration of the model flood flows was considerably (30 to 40 percent) longer than that observed in the prototype.

83. Data show the model flows to be much too small at sta 2CR. However, the prototype flows are also very small, and virtually all of the model flows during the tidal cycle are less than the threshold velocity of the model current meters of 0.5 fps. The technique of assuming a low-flow speed when the velocities are below threshold can be misleading. The flow could be just below threshold, on the order of 0.5 fps (prototype) but could be recorded as 0.2 or 0.3 fps. Prototype flows at 2CR vary between 0.4 and 0.8 fps on maximum flows, and a model maximum flow of 0.4 fps would be underestimated by 25 or 50 percent. Normally, the threshold of a meter is determined from new meters, and for older meters the actual threshold could be somewhat greater than 0.5 fps.

84. Sta 4CR was located in Turtle River upstream of the upstream end of East River, in the center of the navigation channel. The postconstruction comparison of model and prototype velocities is shown in

Plate 13. Due to problems with instrumentation during the prototype survey, a portion of the ebb phase currents was not measured at sta 4CR. The general agreement at this station is much better than that observed at sta 2CR. At surface on flood phase, the model mean velocity (1.47 fps) was 11 percent greater than the prototype mean flow (1.32 fps). At middepth the model flow (1.40 fps) was only 2 percent less than the mean prototype flow (1.43 fps). At the bottom the model flow (1.35 fps) was 23 percent greater than the prototype flow (1.09 fps). There was a difference in phase of about 30 min, with the model lagging the prototype at all three depths. There were not sufficient prototype ebb data to compute mean flows, but the approximately 3 hr of data agree fairly well if the phase shift is considered.

85. Range 5 is located in Turtle River, upstream of the entrance to Brunswick Harbor in East River and downstream of the mouth of the South Brunswick River. Except for range 2 in East River, this is the range for which the greatest changes in current velocities were expected due to the construction. At this range there are three stations for comparison. Postconstruction comparisons for sta 5A, 5CR, and 5C are shown in Plates 14, 15, and 16, respectively.

86. At sta 5A (Plate 14) the agreement is fairly good. There is a phase shift of about 30 min at all depths, with the model lagging. On flood phase the mean speeds of the model are higher than those of the prototype by 20 percent at surface and middepth and by 45 percent at bottom. On ebb phase, model flows are greater by 27 percent at the surface, by 4 percent at middepth, and by 89 percent at bottom.

87. At sta 5CR (Plate 15), there is still a 30-min phase shift between the model and prototype, but the general agreement remains good. At flood phase the model mean flows are low by 7 percent at surface, 2 percent at middepth, and are high by 39 percent at bottom. On ebb phase, model mean flows are greater than prototype mean velocity by 6 percent, 1 percent, and 49 percent at surface, middepth, and bottom, respectively.

88. At sta 5C (Plate 16) the model current velocities were in fair agreement with the prototype currents. In addition to the general phase shift between model and prototype slack waters observed at sta 5A

and 5CR, there is a somewhat longer flood phase (about 45 min) in the prototype than in the model, particularly at surface and bottom. The model mean flow velocity on flood phase is 41 percent greater than the prototype mean flow at the surface, 4 percent less at middepth, and 56 percent greater at bottom. On ebb phase the model mean flow is greater at all three depths by 23 percent, 10 percent, and 85 percent at surface, middepth, and bottom, respectively.

89. Agreement between model and prototype at range 5 is good. Model mean current velocities are generally greater than prototype mean velocities, particularly at the bottom.

90. Range 6 is located downstream of the study area and the Lanier (U. S. Highway 17) Bridge. Sta 6CR is located in the center of the navigation channel. The agreement observed at this station (Plate 17) is comparable to that seen at the other stations. At sta 6CR, however, the phase shift observed at slack waters at the other stations is not so evident. Low-water slacks are within 10 min for model and prototype at all depths. High-water slacks of the prototype occur sooner than the model, but by only about 20 min at each depth. Model mean flow velocities at sta 6CR are greater than prototype flows at all depths for both flood and ebb phases of the currents. For flood the model flows are greater by 30 percent at surface, 2 percent at middepth, and 122 percent at bottom. On the ebb phase, model mean flows are greater than prototype flows by 22 percent at surface and middepth both and by 61 percent at bottom.

Statistical analysis of current velocities

91. It is difficult to obtain a feeling for the overall quality of model predictions of current velocities without some scale by which to measure it. Plots, as presented in the previous section, are invaluable in obtaining a subjective feel for prediction accuracy. Comparison of mean current velocities is a step in the direction of quantification, but does not provide for quantitative comparisons of model predictions. A more quantitative measure of model accuracy is needed if the insight gained from this postconstruction verification study is to make a

contribution to the overall state of the art of physical modeling and to provide a basis for comparison of this model's accuracy with other model studies. For this reason, several statistical parameters were determined for the current velocity data of the model study. In order to put the accuracy of the postconstruction verification into perspective, the analysis was first performed on the original model verification; then the postconstruction analysis was compared with the original verification analysis.

92. The statistical analysis was first performed on each depth at each station separately, termed point statistics; then all the data at single stations, including all depths, were analyzed. Finally, the entire set of data was analyzed as one sample. The analysis was done for flood flow, ebb flow, and for the entire tidal cycle.

93. The data analyzed were the difference between the magnitudes (absolute values) of the model current velocities, V_m, and the prototype current velocities, V_p,

$$\Delta = \left|V_m\right| - \left|V_p\right| \tag{1}$$

These Δ values were computed for each half-hourly reading over the tidal cycle of model and prototype. The parameters computed in the analysis were the root-mean-square (RMS), the mean (M), and the standard deviation about the mean (S). These are computed in the usual manner as follow:

$$RMS = \left(\frac{1}{N}\sum_{i=1}^{N}\Delta_i^2\right)^{1/2} \tag{2}$$

$$M = \frac{1}{N}\sum_{i=1}^{N}\Delta_i \tag{3}$$

$$S = \left[\frac{1}{N - 1} \sum_{i=1}^{N} \left(\Delta_i - M \right)^2 \right]^{1/2} \tag{4}$$

94. The above three parameters alone were computed for the point statistics. However, for the larger samples of station statistics and all the data combined, two additional parameters were computed. These parameters are the coefficients of skewness, A3, and kurtosis, A4, based on the third and fourth moments about the mean, respectively. These are computed:

$$A3 = \frac{\left[\frac{1}{N} \sum_{i=1}^{N} \left(\Delta_i - M \right)^3 \right]}{A2^{3/2}} \tag{5}$$

$$A4 = \frac{\left[\frac{1}{N} \sum_{i=1}^{N} \left(\Delta_i - M \right)^4 \right]}{A2^2} \tag{6}$$

where

$$A2 = \frac{1}{N} \sum_{i=1}^{N} \left(\Delta_i - M \right)^2 \tag{7}$$

95. It is important that the significance of these parameters to model behavior be understood. The RMS is a measure of the differences in the model and prototype current velocities, regardless of which has the higher velocity. The mean difference is an indication of any net differences between model and prototype velocities over a phase (ebb or flood) of the tidal cycle, taking into account which has the greater velocity. Thus, a station could have a large RMS value for the differences on a phase of the tide, but have a very small mean difference if

the times when the model velocity is greater are offset by times when the prototype velocity is greater. A negative mean indicates the model flow magnitude was less than that of the prototype. If the mean is positive, the converse is true. The standard deviation is a measure of the scatter of the frequency distribution of the differences about the mean difference. A small value of standard deviation indicates that most of the differences measured were approximately the same (the mean difference). A large value of standard deviation indicates that there was a wide range of differences measured.

96. The coefficient of skewness is a measure of the asymmetry of the frequency distribution of differences about the mean. A negative skewness indicates that the difference of highest frequency is shifted toward the positive differences. A positive skewness has the maximum frequency toward the negative differences. Another way of viewing the coefficient of skewness is that a skewed distribution resembles a breaking wave form. A negative skewness has a distribution of a wave breaking toward the positive end of the real line and a positively skewed distribution seems to be breaking toward the negative end of the real line. A large value of the coefficient of skewness could mean that in addition to the normal random fluctuations in either the model or prototype velocity measurements, there is also some fluctuation or tendency toward either persistently faster or slower velocities caused by some external perturbation. This could be due to ship traffic, wind gusts, waves, or large oscillating gyres in the prototype. In the model this could be due to model oscillations, impeded movement of the cups in the current meter, an oscillating gyre, and others.

97. The coefficient of kurtosis is a measure of the peakedness of the frequency distribution of the differences. A lower value of kurtosis implies that the distribution is more peaked, with a large frequency of occurrence in a narrow band of differences. If the coefficient of kurtosis is large, the frequency distribution is more spread out over the range of differences with very little peak. The normal distribution has a coefficient of kurtosis of 3.0 and is helpful as a reference for the results. A low coefficient of kurtosis will

generally be found when the model and prototype current velocities at each depth have the same basic trends in time, whether in phase or not. If the trends are different in model and prototype, then a relatively high coefficient of kurtosis would be found.

Point statistics

98. Results of the statistical analysis of the original model verification at each depth at each station are presented in Table 5. Results of the postconstruction verification statistical analysis are presented in Table 6. The original and postconstruction verification statistical data are summarized in Table 7 for comparison.

99. RMS values for both the original and postconstruction verifications had the same general range of values for both phases of the tide and over the complete cycle. The original verification average RMS values were somewhat lower than those for the postconstruction verification on both phases of the tide, but in both original and postconstruction data the ebb phase RMS averages were greater than those in the flood phase.

100. Comparison of the magnitudes of mean differences at each point verified shows the same trends as does the RMS comparison.

101. For the standard deviation of the differences about the mean, the original verification and postconstruction verification had comparable ranges from minimum to maximum values on flood and ebb phases with the postconstruction range for the complete cycle being somewhat greater than for the original verification. Average standard deviations for the original and postconstruction verification, respectively, were 0.36 and 0.40 fps on flood phase, 0.45 and 0.66 fps on ebb phase, and 0.48 and 0.55 fps over the complete cycle.

102. Although the point statistics show that the postconstruction verification of current velocities was not as accurate as the model was originally verified, it shows that the accuracy is of the same order of magnitude. Ebb is worse than flood overall, and both the original and postconstruction verifications have the worst agreement at the bottom depth throughout the model.

Station statistics

103. Results of the statistical analysis of the original verification of current velocities taking each station as a whole including all depths are presented in Table 8. Results of the station statistical analyses for the postconstruction verification are presented in Table 9. Included in Tables 8 and 9 are the results of the statistical analyses of the entire sets of data as a single sample, including all stations. Results of the station statistics are summarized in Table 10 for original and postconstruction verifications for comparison.

104. Analysis of the differences taken at each station as a whole shows the same trends as did the statistics for each depth separately. Once again the ebb phase of the tide shows poorer agreement than on the flood phase.

Test statistics

105. Results of the statistical analyses of all the data in a verification are presented at the bottoms of Tables 8 and 9 for the original and postconstruction verifications, respectively.

106. For the test statistics, the postconstruction verification is again shown to be less accurate than the original verification. RMS errors for the postconstruction are approximately 40 percent greater than those for the original verification. The mean difference for the original verification is greater on ebb phase than on flood phase of the tide, but for the postconstruction data mean differences are the same on both phases. The standard deviations are greater in the postconstruction than in the original verification, with the ebb phase showing the largest variation.

107. The coefficients of skewness, A3, for the flood phase statistics were -0.07 for the original verification and 0.08 for the postconstruction verification. With mean differences of the same sign (-0.02 and 0.26 fps, respectively) in both cases as the coefficient of skewness, this places the differences of maximum frequency toward zero from the respective means for both verifications. This is also the case for both original and postconstruction verifications on ebb phase and over the complete cycle. These indicate that the mean differences are

due to a wide range of differences of the same sign as the mean difference.

108. The coefficients of kurtosis for the original verification were 3.15 on flood phase, 3.02 on ebb phase, and 3.32 over the complete cycle. For the postconstruction verification, the coefficient of kurtosis was 2.60 on flood phase, 2.10 on ebb phase, and 2.50 over the complete cycle. This indicates that in each case the original verification is closer to a normal distribution than the postconstruction verification, which has a more peaked distribution of differences. This is at least partly due to the fact that the size of the original verification sample is larger than for the postconstruction verification analysis.

Flow Predominance

109. As discussed in PART II flow predominance plays an important role in the supply of sediment to the Brunswick Harbor area. For this reason the quality of the reproduction of this phenomenon in the model was examined. Current velocity data for model and prototype were analyzed to determine flow predominance. This method of presenting current velocity data reduces magnitude, direction, and duration of the currents to a single expression that defines the predominant direction and percentage of total flow at any given point. This expression was derived from a conventional plot of velocity versus time at any given point. The area subtended by both ebb and flood portions of the curve was measured and summarized. The area subtended by the flood portion of the curve was then divided by the total area and multiplied by 100 to determine what percentage of the total flow was in the flood direction. A negative (-) sign and a positive (+) sign were designated to indicate ebb direction and flood direction, respectively. For simplification, the percentage of flow in the flood direction was calculated, then a value of 50 percent was subtracted from the calculation to determine predominant direction and magnitude. Using this method of analysis, a value of 0 percent indicates that flows in both the ebb and flood

direction are equally balanced, i.e., the area under the ebb and flood curves are equal. A value of +50 percent indicates that flow at that point is in the flood direction at all times during a tidal cycle, while a -50 percent value indicates flow in the ebb direction throughout a tidal cycle. Whenever the flow predominances are within ±5 percent of 0 percent (equal flows on flood and ebb phases), it is quite possible that the deviation can be due to random variation, rather than density effects or strong geometry effects.

110. Flow predominance calculations for the original model verification are presented in Table 11. Model and prototype flow predominances for each depth are given for each station used in the original verification. Flow predominance results for the postconstruction verification are presented in Table 12. Results for the six stations used for current velocity comparisons were used. Sta 4CR had insufficient prototype data for calculations of flow percentages.

111. Based on the 1963 prototype data, the mouth of East River (sta 3CR) exhibits a strong flood predominance at the bottom for neap and spring tides. Sta 2CR is about 2500 ft farther upstream and is beyond the zone where this flood predominance is pronounced. Sta 3CR data have not been formally included in the analysis because they do not correspond to one of the postconstruction prototype data stations. However, in the original verification of current velocities at sta 3CR (Plates 5 and 6) the prototype bottom flows were +37 percent and +25 percent flood predominance for neap and spring tides, respectively, while the model was almost exactly balanced for both these conditions. For a mean tide (not shown), the prototype flow at sta 3CR was almost exactly balanced, but the model exhibited an ebb predominance of +7 percent. The model flows show no net circulation in the mouth of East River as is found in the prototype.

112. A summary of the flow percentages for the original and the postconstruction verification data is given in Table 13. The frequency of times that the directional flow predominance of the model corresponded to the directional predominance of the prototype is given for each depth at each station for both original and postconstruction verifications.

This is indicated as either a correct or incorrect predominance and is so tabulated. In addition, the average deviation of the flow percentage is given.

113. From the data in Table 13, it appears that the accuracy of the original verification in terms of flow predominance had little bearing on the accuracy of the model predictions in terms of flow predominance. The worst station presented for original verification was in best agreement for postconstruction verification.

114. The summary of the two verifications at the bottom of Table 13 shows that the overall accuracy of flow predominance between model and prototype was better in the postconstruction data than for the original verification data. The average deviation for all the data in the original verification was 10 percent, compared with only 5 percent in the postconstruction verification. In the original verification, the direction of flow predominance was correct 69 percent of the time (27 out of 39 times). The postconstruction verification had the direction of flow predominance correct 92 percent of the time (11 out of 12 times).

115. Accuracy of both the original and the postconstruction verifications is better at surface and middepth than at bottom. On the bottom in the original verification, the direction of flow predominance was incorrect more frequently than it was correct. This was unfortunate, as it is the bottom flow predominance that is most important to sedimentation.

Sedimentation

Original verification

116. As with all phenomena of interest, when evaluating the predictive capabilities of a model the first item of interest is the verification. The shoaling verification for the East River-Port area is presented in Table 14. The volume of shoaling in each channel section is given, along with the percent of the total shoaling in that section. The data are also presented graphically in Plate 18. Channel shoaling

sections are defined in Table 14 and in Figure 2.

117. Comparison of the distribution of the total shoaling between model and prototype shows that the original verification was only fair. The centroid of the prototype shoaling in the port is at 3.61 (i.e., 61 percent of the distance from the center of section 3 to the center of section 4). The model shoal has its centroid at 3.83. The model shoaling was more evenly distributed over the port, while the prototype shoaling distribution had a peak in sections 3 and 4. The model distribution exhibits a trend from maximum shoaling at the downstream end of the port, gradually diminishing to the least amount of shoaling at the upstream end of the port. The average magnitude of the difference between the percentages of model and prototype shoaling in a given section was 5.3 percent. The prototype maximum shoaling was in channel shoaling sections 3 and 4, while the model maximum shoaling was in sections 5 and 6.

Model tests

118. The pertinent model shoaling tests are summarized in Table 15. There were four separate base tests: mean and spring tide tests for 30- and 36-ft navigation channels. The partial closure plan was tested only for 36-ft channel, but for both mean and spring tides. The alternate closure plan was tested for only a mean tide, but for both the 30- and 36-ft navigation channels. The table presents the volume of model shoaling in each channel section, beneath which is given the percentage of the total shoaling in sections 1-6 for that test. Also given at the bottom of the table is the centroid of the shoal.

Prototype shoaling

119. The average annual shoaling rates experienced in the prototype over the periods of interest to this study are summarized in Table 16. This table summarizes more extensive data presented in Appendix B on prototype shoaling volumes during these periods. The periods of concern to the study are the preconstruction period (4 September 1962 to 10 March 1969), the partial closure plan period (10 November 1969 to 21 April 1975), and the alternate closure plan period (11 July 1975 to 20 September 1976).

Postconstruction Verification

Partial closure plan

120. Shoaling distributions of model and prototype for the partial closure of East River are presented in Plate 19. Model tests were for a mean and a spring tide, both with the 36-ft channel.

121. Agreement between model and prototype distributions is very good in light of the original verification. The centroid of the prototype shoal is at 3.78 compared with a centroid of 3.90 for the model mean tide test and 4.13 for the model spring tide test. This yields an average model centroid of 4.01 and a difference in centroid location of 0.23 times the spacing between the centers of shoaling sections 3 and 4, comparing very closely with the difference in the original verification of 0.22.

122. The average magnitude of variation between model and prototype shoaling percentages in each section is 3.3 percent for the mean tide test and 4.7 percent for the spring tide test. This compares excellently with the original verification, which had a mean variation of 5.3 percent. In terms of shoaling distribution, the model partial closure plan test predictions were as close or closer to prototype results than the original verification.

123. Relative changes in the shoaling rates in each shoaling section from the preconstruction condition to the partial closure condition are presented in Table 17 and Plate 20. Changes in the shoaling rates are expressed as a percentage of the total preconstruction shoaling rates for all sections. Both the prototype and the model mean tide test show the maximum change in the shoaling rate in section 5. Both model tests agree with the prototype observations that essentially no change occurred in section 6, and that section 1 also had a reduction of less than 5 percent. The average magnitude of difference in the relative changes in each section between model and prototype was 4.9 percent for the mean tide test and 8.0 percent for the spring tide test. Both of these variations compare favorably with the variation of 5.3 percent observed in the original shoaling verification. Mean tide

model results are seen to be closer to prototype than spring tide results.

124. The overall effect of the partial closure on the total prototype shoaling was a 30 percent reduction. The model predicted a 44 percent reduction in the mean tide test and a 73 percent reduction in the spring tide test. These values are fairly good when one considers that the model was verified for shoaling with a mean tide and not a spring tide.

Alternate closure plan

125. Plate 21 presents the shoaling distributions for model and prototype for the alternate closure plan condition. The model tests were conducted with a mean tide for both the 30- and 36-ft channels. The centroid of the prototype shoal is located at 3.82 while the model centroid of shoal shifted to 5.59 for the 30-ft channel and 5.50 for the 36-ft channel. The average magnitude of variation between model and prototype shoaling percentages in each section is 22 percent for both the 30- and 36-ft channel tests. This compares quite poorly with the original verification, which had a mean variation of 5.3 percent.

126. Shoaling rate changes in each channel section induced by the alternate closure plan relative to the preconstruction condition are presented in Table 18 and Plate 22. Changes are expressed as percentages of the total preconstruction shoaling rates for all sections. The model changes are approximately three times as large as the prototype changes in sections 2 through 5. Both the prototype and the model 36-ft test showed virtually no change in section 6. The average magnitude of the difference in model and prototype relative changes in each section was 9.9 percent for the model 30-ft channel test and 8.6 percent for the 36-ft channel test. These are high relative to the original verification average magnitude of differences of 5.3 percent between model and prototype. These variations in the alternate closure plan data compared with postconstruction prototype data in each section are all in the same direction (more reduction in the model than in the prototype), resulting in a prediction of excessive shoaling reduction.

127. Overall, the prototype experienced a reduction in total

shoaling of 26 percent for the alternate closure plan relative to preconstruction. The model 30-ft channel test showed an overall reduction of 86 percent, and the 36-ft test had a reduction of 78 percent for all sections.

128. It is of interest to look into the relative change from the partial closure plan to the alternate closure plan. Table 19 and Plate 23 present the relative change based on a percentage of the total partial closure plan shoaling rate for all channel sections. The prototype experienced a minor increase in the shoaling rate overall of 6 percent. The model had an overall decrease in shoaling from the partial closure to the alternate closure of 60 percent. The average difference in the change in shoaling rate for the sections is 11 percent, but once again this difference is systematically excessive shoaling reduction predicted by the model.

PART V: DISCUSSION

129. In the previous sections, the Brunswick Harbor area has been briefly described, and the important factors having a bearing on sedimentation in the harbor have been mentioned. The model, its operating procedures, and adjustment techniques were described. Results of the postconstruction verification can now be discussed in view of the modeling procedures and the prototype phenomena; but it is first of value to discuss more fully the limitations of both the prototype data being used and the modeling techniques.

Prototype Data

130. Any model can only be expected to perform as accurately as it was originally verified. Often great pains are taken during verification testing to bring the model into closer agreement with the prototype. In the fervor to obtain the best possible verification the accuracy of the prototype data being used may sometimes be overlooked. The model, in turn, should not be expected to be verified any better than the accuracy of the prototype data used for verification, and prototype data are often unsatisfactory. They are typically either short-term data with the important boundary conditions vaguely defined or very long-term data that obscure the sequencing or severity of events that have produced the overall result.

131. In collecting prototype hydraulic data there is a trade-off between the amount of data that the modeler would like to have for hydraulic verification and the amount of data that funding and time will allow. These limits place restraints on both the quantity and quality of data collected and the collection technique utilized.

132. Prototype hydraulic conditions are constantly changing due to many influences. Tidal range variations, barometric tides, wind effects, wave conditions, and freshwater inflows all influence the hydraulic conditions within the estuary, and because of variations in these many factors, hydraulic conditions never reach a true equilibrium.

Hydraulic data measurements for model verification purposes are often obtained for a relatively short duration--typically one or two semi-diurnal tidal cycles. Tidal elevations are sometimes recorded for longer periods, but only recently have techniques been developed to make modeling long tide records practical. With the prototype system continually responding to these variations in boundary conditions, it is difficult to isolate influences and obtain representative prototype data for a model that does not have the capability of reproducing wind and wave effects. The modeler is usually forced to use the data collected without adjustment for these factors and lump variations into the data accuracy, which masks the capabilities and the limitations of the model.

133. There are many man-induced problems in the accuracy of prototype hydraulic data. An error in zeroing tide gages would have a significant effect on the hydraulic conditions reproduced in the model. Prototype current meters have a threshold velocity limitation (0.2 fps) below which the data are not accurate. A common occurrence in surveys is the passage of a large ship just prior to current measurement, which can cause erratic data. Factors such as this are monitored during most hydraulic surveys. Then, of course, the general types of human error in recording data arise; however, human errors often become obvious during data analysis and bad data values can be omitted. Equipment malfunction is unavoidable during data collection but can be minimized by proper maintenance. In addition, it is possible to introduce apparent errors through improper positioning (horizontally or vertically) of the prototype velocity meter.

134. The basic assumption used in collecting prototype hydraulic data to be used for verification is that the physical model has the capability to interpolate between stations that are verified. Using this principle, the modeler minimizes the number of stations at which data are required until the model's interpolative capacity is taxed. This will occur when the distance between successive stations along or across the model becomes great or when significant control sections are omitted. If this limit is exceeded, the circulation and transport processes occurring in the model may not be reliably modeled.

135. Prototype sedimentation data are normally generated from either hydrographic sounding data or dredging records. For either method there is some degree of accuracy or error associated with the volume of sedimentation. As discussed in paragraph 46, the accuracy of prototype hydrographic soundings is assumed to be ± 0.5 ft for average conditions in the Brunswick Harbor area. If this error is random, the cumulative error will not be of significance. However, if this involved some systematic error, such as improperly zeroed tide gage used for adjusting the soundings, then it could have a substantial effect on the results. For example, if a systematic error on the order of 0.5 ft occurred over the entire East River harbor area (1.62×10^6 sq ft) the error in the shoaling volume computed from hydrographic soundings would be about 30,000 cu yd. This is approximately 5 percent of the annual shoaling rate and is considered to be fairly accurate. If systematic errors are avoided, the volumes computed from hydrographic sounding data can be considered very accurate for the East River Port area.

136. The required sedimentation information is usually determined from hydrographic sounding data that cover only a small area of the estuary. The most plentiful data are those collected primarily for dredging purposes and are therefore normally confined to near the navigation channel. Important sources or sinks for shoaling material may easily be overlooked because of this. Comprehensive surveys of most estuaries are normally conducted many years apart.

137. The accuracy of volumes of sedimentation based on reported dredging volumes is much less accurate than that based on hydrographic surveys. This is particularly true for the Brunswick Harbor area, which has a silt and clay bottom. Reported dredging volumes are normally not a very good indication of the shoaling volumes. For example, in the preconstruction period pertinent to this report, the average annual dredging was reported as 566,000 cu yd, while the average annual shoaling volume based on hydrographic surveys was 781,100 cu yd. This kind of difference also appears in the postconstruction volumes, and is attributed to some effective agitation dredging during dredging. Variability in shoaling computations is also illustrated by the difference

in annual shoaling rates of 1,109,000 cu yd per year and 781,100 cu yd per year between the computations of the Savannah District and WES for this study. The Savannah District used seven shoaling periods and WES used fifteen periods between dredging surveys.

138. Dredging frequency and volumes are not always an indication of the shoaling rate, since dredging schedules are usually set long before the actual dredging operation. If an emergency situation develops, it is not uncommon in many harbors to perform emergency dredging with no record kept of the volume removed. Available funding, scheduling priorities, weather conditions, and available dredging plant can also affect dredging volumes without regard to the shoaling rate.

139. Aside from the problem of accuracy is the question of the prototype sedimentation data being characteristic of the total sedimentation picture for the harbor. Shoaling rates for the harbor are variable, depending on several boundary conditions. Rapid shoaling in the harbor is observed during spring tides and storms, when the water contains a large amount of sediment. There has also been a seasonal variation to the shoaling rate, with higher shoaling following high discharges of the Altamaha River. Another factor in the supply of shoaling material to the Brunswick Harbor area is wind waves, supplying the energy required to suspend fine material from nearby marsh areas.

Modeling Techniques

140. Physical hydraulic models are capable of reproducing most, but not all, important hydrodynamic phenomena influencing sediment transport. Tidal, freshwater, and salinity induced densimetric currents can be modeled. A model verified to accurately reproduce observed salinities is assumed to correctly model other dispersive transport in the salinity intrusion region. Geometric influences on current directions and magnitude are modeled. Phenomena usually not modeled include wind-induced currents and locally generated wind waves, though the latter can be simulated if they are known to be important and their effect can be defined. Occasionally, other influences such as ship transit

may have sufficient impact to require simulation.

141. The gilsonite technique cannot, of course, reproduce flocculation of clay sediments. It therefore will not directly model changes in the supply of depositable sediment due to increased flocculation rates caused by geometry, structures, and other shear-producing factors. These must be simulated by a sediment injection procedure that alters the supply available for deposition. A particular tracer grain size may satisfactorily replicate the settling velocity of a particular class of sediment aggregates, but finding the proper tracer is an empirical process and not easily subject to variation over the model.

142. Since the model tracer is not cohesive, erosional and depositional criteria are altered from the prototype. The model currents and sediment are such that much transport is in the bed-load mode, and the rate of transport is proportional to the excess shear stress (excess over the critical value for initiation of motion) in contrast to that described for cohesive sediments in the prototype in which particles, once eroded, are transported at a rate dependent only on their concentration and the current speed until they approach the bed and bed shear permits redeposition.

143. What the modeler must achieve is some correspondence between the ability of the model to transport and deposit available noncohesive sediment and the ability of the prototype to transport and deposit cohesive sediment. Obtaining that correspondence is intricately involved in the 10 adjustments listed in paragraph 62. Adjustments of sediment injection location, rate, time, and duration may be necessary to obtain the proper sediment supply. However, there is a danger that the model tracer supply can be arranged so as to compensate for inaccurate hydrodynamic reproduction. Knowledge of the modeled estuary sediment sources, suspended sediment concentration patterns, and estuarine sedimentation processes must be applied to ensure that the model tracer injection procedure does not force the model to reproduce prototype shoaling patterns without some correlation to the transport processes. For example, it would be reasonable to increase the injection rate in a region where a nearby flow constriction could be expected to increase

shear and thus the flocculation rate. It would not be reasonable to reduce the injection rate in an area where high current speeds in the prototype prevent deposition.

144. Careful attention to hydrodynamic verification is a prime requisite for avoiding errors that must be compensated for by tracer injection, but some adjustments to hydrodynamic verification conditions may be necessary to obtain shoaling verification. Changes may be necessary to obtain typical transport conditions, to improve hydrodynamic reproduction in areas between data stations, or to slightly alter the behavior of the model tracer. These changes can contribute to improve sedimentation simulation without sacrificing the model's faithful reproduction of prototype hydrodynamic behavior, but care is required. Just as with the tracer injection rate, deciding what constitutes valid hydrodynamic alterations requires knowledge of transport processes, estuary characteristics, and model behavior. For example, changing test duration, tidal range, and freshwater inflow within reasonable limits can be necessary to produce transport conditions in the model that represent typical transport conditions in the prototype. Changing ocean or inflow salinity can be used to slightly change tracer submerged weight, and thus settling velocity and erodibility. Rearrangement of roughness elements within a reach of the model is commonly necessary since hydrodynamic verification requires only that average energy dissipation between data stations be correct. Roughness redistribution may therefore be an extension of hydrodynamic verification; however, adding substantial amounts of additional roughness is not likely to be a valid extension.

145. Physical model reproduction of estuarine sediment transport is imprecise and is by necessity an empirical procedure requiring considerable knowledge and judgment of the modeler. Alterations to model conditions and procedures listed in paragraph 62 and described above can be meaningful and necessary to obtain adequate simulation of prototype behavior; but there is a real danger that they may force the model to produce desired depositional patterns without reproducing similar patterns of sediment transport. If this occurs, the model will be

unable to respond to alterations in the same way as the prototype, rendering it useless as a predictive tool. As model adjustment procedures become more extreme and as proposed estuary alterations have greater impact, the ability of the model to predict changes in sedimentation can become poorer. Determining the limits of reasonable model predictions is the object of this research program.

146. A consideration of importance for physical models using a fairly uniform size tracer material to represent a graded prototype material is the possible alteration in supplied sediment distribution caused by construction. If the distribution of sizes in the prototype shoaling material changes drastically with some modification to the estuarine system, the model will be taxed to adequately predict when verified to a fixed size fraction. There is not much that can be done to anticipate such a change and vary the model testing procedures accordingly, as the phenomena of sorting are very complex. This concept must, however, be kept in mind in interpretation of model results.

147. Certain assumptions made to make model operation and the tracing techniques simpler place limitations on the modeling capabilities. These limitations must be clearly understood before the results can be interpreted.

148. In designing the Brunswick Harbor model study, a trade-off was made between the inclusion of density effects caused by salinity gradients and model costs. To include salinity effects would have involved installing a skimming system in the model and operating costs would have been considerably greater. Because of the relatively well-mixed system and substantial evidence that inertial forces played a greater role than density gradients in diverting water into East River, the model was operated with a constant salinity throughout. This assumption of a totally mixed system may be valid over a wide range of conditions in the harbor and over the majority of the time. However, during periods of high discharge from the Altamaha River the assumption may break down.

149. The most rapid shoaling in the harbor area occurs after these higher discharge periods. Although a majority of the time density

currents are negligible, they may be important to the supply of shoaling material during the higher discharges of the Altamaha River. If the suspended sediment concentration in the prototype varies substantially over the water column, a slight increase to the density gradient could be present. When combined with a slight vertical salinity gradient as in the Brunswick Harbor area, this could influence hydraulic conditions.

150. Another important factor in the model's reproduction of the hydraulics in the harbor is the manner in which the tide was generated. The model headbay, located at the ocean end of St. Simon's Sound, provided the tidal exchange. The location of the headbay was not in an ideal location; it was at the narrow gap between St. Simon's Island and Jekyll Island, where the maximum tidal current velocities occur in the prototype. At that location in the prototype, there is a certain amount of kinetic energy associated with the tidal currents. Because the model has a fixed boundary there, the energy must be supplied as a difference in potential energy. Therefore, the model requires a greater range of tidal elevation fluctuation in the headbay than is experienced at that location in the prototype. This greater tidal range was experienced in the model. The prototype mean tide range on the bar is 6.5 ft and is 7.3 ft at the State Dock in East River. In order to generate the 7.3-ft range at State Dock in the model, a 7.9-ft tide was required at the limits of the model. In order to obtain a spring tide range of 10.3 ft at State Dock, the model required a range of 12.4 ft in the headbay. The prototype tide range on the bar corresponding to a 10.3-ft State Dock range is only 8.9 ft. This discrepancy in boundary conditions places a limit on how close to St. Simon's Sound testing could be conducted.

Hydraulic Results

Original verification - tidal elevations

151. Agreement obtained between model and prototype tides at State Dock in East River was generally very good for the prototype data

used for model verification. However, because State Dock was the only gaging station used in reproducing the tide in the model, it cannot be concluded that tidal propagation within the estuary was reproduced or verified.

Original verification - current velocities

152. The original verification of model current velocities was generally good. There is a tendency toward too great a magnitude of model flow at the bottom depth, particularly on the ebb phase of the tide. Surface velocities were reproduced fairly accurately, while model middepth currents tended to be slightly low on the flood phase and slightly high on the ebb phase.

153. Shapes of the velocity profiles were somewhat different in model and prototype. The prototype profile was fairly uniform between surface and middepth, then decreasing with depth with bottom velocities (3 ft above the bottom) approximately half as great as surface velocities. The model velocity profile tended to be more uniform over the entire depth with bottom velocities (4.5 ft above the bottom) generally about the same order as surface velocities.

154. The degree of the effects of salinity gradients in the Brunswick Harbor on currents is difficult to assess, as it is normally a well-mixed system. The effect can be pronounced along the longitudinal axis of the estuary with a moderate longitudinal salinity gradient. However, for the diversion of water preferentially into side channels due to a vertical salinity gradient, the effects are probably small compared with the effects of inertial forces. This preferential diversion being a key factor in the supply of sediment to the East River area, the impact of the absence of salinity gradients should have been minimal on the nearfield transport capacity into East River. However, the lack of the salinity gradient in Brunswick River could have changed the model sediment supply rate to the general area of Brunswick River, and thus affected the sediment supply to East River.

155. A second potential cause for dissimilar velocity profiles lies in measurement techniques. As described in paragraph 52, the model

current meters sensed an equivalent prototype area from about 3 ft above the bed to 7 ft above the bed. The average velocity of this area is virtually certain to be greater than a prototype point measurement 3 ft from the bed. Thus, the disagreement between model and prototype could be a result of measurement difference rather than profile dissimilarity.

156. With the model's tendency toward excessive bottom flow, particularly on the ebb phase, bottom ebb flow was predominant at all but one station in the model (and that station had almost equally balanced bottom flow). The prototype experienced bottom flood flow predominance at about half of the stations (but most of those also were essentially balanced bottom flow). This could be evidence of a slight difference in salinity gradient between the model and prototype. As mentioned, this could have had an impact on the sediment supply to the general area during the shoaling tests.

Postconstruction verification - alternate closure plan tidal elevations

157. Tidal elevations at the control station used in the model study (State Dock in East River, sta 3) matched fairly well for the 20 March 1976 prototype survey and the alternate closure plan hydraulic test in the model. Shapes of the model and prototype tides are somewhat different but the tide range was matched as accurately as the model range was originally adjusted.

158. Comparison of model and prototype tidal elevations at the mouth of South Brunswick River (sta 1) was fairly accurate in view of the fact that tidal propagation in the estuary was not verified. However, the South Brunswick River gage and the State Dock gage are in the same proximity (approximately 2 miles apart).

Postconstruction verification - alternate closure plan current velocities

159. The accuracy with which the physical model predicted the current velocities after the prototype construction is a direct reflection of the original verification of the model for current velocities.

The model study predicted too great a magnitude of bottom current velocities for the alternate closure plan at the stations located in Brunswick River, especially on ebb phase of the tide. Model velocities were below current meter threshold at sta 2CR in East River as a result of the closure dams and thus the degree of error cannot be specified.

160. The phase shift observed in the model original verification current velocities is also observed in the postconstruction model data relative to the observed prototype data. The model slack waters occur approximately 30 min after the prototype slack waters.

161. Results of the statistical analysis show that the model prediction of the effects of the alternate closure plan on current velocities was not quite as accurate as the model was originally verified. However, the accuracy of the prediction is of the same order of magnitude as the original verification as quantified by the RMS differences, mean differences, and standard deviations. Overall, the predictions had RMS differences approximately 40 percent larger than those of the original verification.

162. It is of interest to compare the statistical analysis of the original verification and predictions of the Brunswick Harbor model study with data from another model study. The only other model study for which this has been done is the Delaware River Model Study (Letter and McAnally 1975). For that study there was a slight difference in the statistical analysis in that the differences were taken as the algebraic difference between model and prototype velocities, with flood being positive and ebb being negative. For the present study, the differences in flow magnitude were used. The only discrepancies between the analysis techniques would occur near slack water when the model flow is in the opposite direction from the prototype. The frequency at which that occurs in the sampled data is very small (approximately 2 percent of time) because the sampling interval generally brackets that short period when the current directions are different in model and prototype. The change in analysis technique is primarily to simplify data interpretation. Table 20 compares the analogous statistical data from each of these studies. These two studies involved similar hydraulic

conditions, with tidal ranges in the study areas of approximately 8 ft and mean current velocities on the order of 1 to 2 fps. The two parameters compared are the average magnitude of mean differences and average standard deviation for the analyses of each depth at each station as a single sample. The parameters are compared for flood and ebb phases of the tidal cycle. From these data it is seen that the Brunswick Harbor model study original current velocity verification is comparable to the verification of the Delaware River Estuary model study. The original verification average magnitude of mean differences is less for the Brunswick study than for the Delaware River study on both flood and ebb phases of the tide. The Brunswick model prediction of current velocities was not as good as its original verification, but was approximately as accurate as the original verification of the Delaware River model. The average standard deviation for original verification was less in the Brunswick model than in the Delaware River model on both phases of the tide. The Brunswick model prediction had a standard deviation comparable to its original verification and to the Delaware model data on flood phase, but was somewhat high on the ebb phase. This could be explained by the model overchannelizing the tidal storage of the marsh areas as ebb flow begins. This can have a significant effect on the net flow, particularly at the bottom, and thus on sedimentation.

163. The flow predominance calculations for the postconstruction verification were only possible for the range 5 stations and sta 6CR. At these stations the predominances were in close agreement at both surface and middepth (especially considering that the model and prototype tides were not identical), but were not in as good agreement at bottom. This is once again indicative of the imprecise verification of bottom current velocities.

Sedimentation Results

Original verification

164. The original Brunswick Harbor model shoaling verification

in East River was only fair. This is primarily due to the fact that the criteria for transport and deposition in the model were different than the criteria in the prototype. Prototype geometry for the verification condition allowed through-flow around Andrew's Island and tidal prism exchange from Academy Creek through East River and there was sufficient erosion capacity to have a limiting effect on the amount of sedimentation. In the model tests, the transport rates for the noncohesive tracer material was also dependent on the transport capacity of the modeled flow and verification was thus obtained by adjusting the sediment supply until the shoaling distribution in the model corresponded to that in the prototype.

Partial closure plan

165. Model predictions for the partial closure plan were fairly good, especially relative to the model original verification. The closure dam in East River cut off the through-flow around Andrew's Island that existed in the verification condition, but the tidal exchange of Academy Creek and its large adjacent marsh areas was still through East River. For this condition, the prototype supply of sediment may have been reduced with the loss of flow discharge through East River; but the capacity of the flow to erode deposited sediments was also reduced. The result was a 30 percent shoaling reduction.

166. In the model, the partial closure plan had reduced velocities in East River, reduced sediment supply, and reduced deposition volumes. The model may have predicted too great a reduction in shoaling because the reduction in sediment supply was disproportionately high in comparison to reduction in the strength of flow. The prototype sediment supply was reduced because of the reduction in flow volume through East River, whereas the model sediment supply reduction occurred due to a reduction in transport capacity at the mouth. The prototype supply is influenced by flows external to East River that maintain or place material in suspension; thus even though the reduced velocity in the mouth of East River would result in a reduced influx of suspended sediment, a considerable amount of suspended sediment still would be transported into East River. A substantial portion of the sediment would be deposited at

slack water after the flood current, and the reduced ebb currents would be too low to resuspend all of the newly deposited sediment and remove it from the problem area. In contrast, for the model these local velocities at the mouth of East River were the primary control over the volume of tracer material transported into the harbor area as bed-load material. This difference in criteria for transport and deposition of material in model and prototype apparently was not too pronounced for the partial closure plan as indicated by the reasonably good agreement between the predicted and observed shoaling rates.

Alternate closure plan

167. With the addition of the closure dam in Academy Creek and the drainage canal, the hydraulics of East River in the vicinity of the harbor were changed enough to result in a significant difference between the model and prototype transport and deposition criteria.

168. The prototype suspended sediment supply to East River was once again reduced somewhat. However, the corresponding reduction in bed shear allowed a greater percentage of the sediment deposited during slack waters to resist erosion by peak currents. The reduced supply was offset by reduced erosion potential and thereby the shoaling rates remained essentially the same in the prototype for the first year.

169. On the other hand, in the model the supply of bed-load material to the harbor was reduced because the bed shear fell below that at which bed-load transport of model sediment could occur. Consequently, the model prediction of sedimentation was drastically low for the interior sections of East River.

170. Shoaling verification was accomplished by delicately balancing the many factors involved in the physical model shoaling process. Unfortunately, it is not possible to correctly model the prototype criteria for transport and deposition of cohesive material. Model verification of shoaling using drastically different sedimentation criteria than those controlling the prototype shoaling process will only be valid over a narrow range of the variable parameters that govern the shoaling process.

171. For the partial closure plan, shoaling verification was

taxed to near its limit, still giving reasonably good results. However, the alternate closure plan resulted in hydraulic conditions outside of the workable range for shoaling tests; hence, the predictions appear to have been in error. The short period of shoaling data for the alternate closure plan prevents a firm conclusion on this point. Subsequent to the data analysis and initial preparation of this report, more recent dredging records became available (see Table 2) and were subjected to a cursory examination. This examination indicated that the shoaling rate used for the detailed analysis of the alternate closure plan may actually be somewhat lower than the average rate over a period of several years. Thus, the general conclusion relating to the effectiveness of the sedimentation tests for the alternate closure plan seems to be valid.

PART VI: CONCLUSIONS

Model Predictions

172. The Brunswick Harbor model study predictions may be judged on the stated objectives of the model study as given in paragraph 6. It is concluded that:

- *a.* The model correctly predicted a reduction in the shoaling volume with the construction of the partial closure plan.
- *b.* The model correctly predicted a slight shift downstream of the centroid of the East River shoal with the partial closure plan.
- *c.* The model erroneously predicted a large reduction in shoaling volumes due to the alternate closure plan.

173. The model satisfied the stated objectives for the partial closure plan with the predictions of the shoaling reduction being of the correct order of magnitude. However, for the alternate closure plan the predictions were erroneous for the year following construction. It is concluded that these predictions were in error due to a combination of the following factors:

- *a.* The basic criteria for transport and deposition of shoaling material were different in the model than in the prototype (noncohesive versus cohesive, respectively). That is, the transport modes were different (bed load versus suspended load) in model and prototype.
- *b.* The model shoaling verification was overtaxed by a drastic change in the hydraulic conditions with the alternate closure plan.
- *c.* The sediment transporting capacity of the flow fell below the critical value for bed-load transport in the model; whereas in the prototype, the influx of suspended sediment was not reduced to such a great extent. The possibility remains that the period of prototype data for the one year of alternate closure plan condition is atypical and that subsequent years will more closely correspond to model predictions; but it is improbable.

Recommendations

174. The use of physical models for predictions of the effects of

proposed physical changes on cohesive sedimentation processes is very precarious. Realizing, however, the value of the physical model as a predictive tool, the following recommendations are made for designing and interpreting physical model results:

- a. The use of physical model tracer tests should be avoided or carefully qualified when testing for the effects of drastic changes in the basic parameters governing cohesive sedimentation (e.g. changes in aggregation rates or types of sediments).
- b. Cohesive sediments should be modeled by the smallest and lighest model tracer that can be realistically used in model operations, and when tracer transport is stopped by subcritical bed shear stresses the results should be identified as potentially erroneous.
- c. The hydraulic and salinity regimes of the model should be kept as close to the prototype system as possible.
- d. If the shoaling verification is feared overtaxed, currently available analytical techniques should be employed to assess the effects of the alteration on the sediment supply to the immediate problem area and adjustments to the far-field model tracer injection should be made to reproduce the expected local supply. It should be noted that such analytical techniques were not developed at the time of the Brunswick Harbor study.

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Table 1

Tidal Characteristics of Brunswick Harbor

Location	Mean Range ft	Average Spring Range ft	Mean Tide Level Above mlw ft
St. Simon Sound Bar	6.5	7.6	3.2
East River	7.3	8.5	3.6
Turtle River (upper end of harbor)	7.6	8.9	3.8
South Brunswick River	7.6	8.9	3.8

Table 2

Reported Maintenance Dredging Volumes Inner Brunswick Harbor

1953-1973

Fiscal Year	Location	Dredged Volume 1000 cu yd	
1953	Brunswick River	464	
1954		NR	
1955		NR	
1956		NR	
1957		NR	
1958	Turtle River	19	
1959	East River	375	
1960	Construction of deepened channel		
1961	East River	359	
1962	East River	607	
1963	East River	640	
1964	East River	599	
1965	East River	834	
1966	East River	419	
1967	East River	584	
1968	East River	483	
1969	East River	308	+ unspecified emergency dredging
1970	East River	295	
1971	East River	674	
1971	Terry Creek	57	(Dredging incomplete)
1972	East River	471	
1973	East River	476	
1973	Terry Creek	506	
1974	Total Harbor	1144	
1975	East River	315	
1976	East River	491	
1977	East River	885	
1977	Academy Creek	282	
1978	East River	479	
1978	Terry Creek	355	
1979	East River	950	
1979	East River and Turtle River	399	

Note: NR = None reported.

Table 3

Comparison of Mean Current Velocities Original Verification

			On Flood			On Ebb		
Date	Station	Depth	Model	Prototype	Model/Prototype	Model	Prototype	Model/Prototype
5/22/63	2CR	Surface	1.41	1.50	0.93	-2.05	-1.86	1.10
		Middepth	1.33	1.75	0.76	-2.10	-1.38	1.52
		Bottom	1.25	0.87	1.45	-1.96	-0.80	2.45
6/4/63	2CR	Surface	0.94	0.86	1.10	-1.12	-1.30	0.86
		Middepth	1.01	0.88	1.15	-1.20	-1.12	1.07
		Bottom	0.96	0.55	1.82	-1.17	-1.00	1.18
5/22/63	4CR	Surface	1.74	1.77	0.98	-2.11	-2.47	0.85
		Middepth	1.65	1.75	0.94	-1.96	-2.25	0.87
		Bottom	1.29	1.45	0.89	-1.80	-1.42	1.27
6/4/63	4CR	Surface	1.22	1.35	0.90	-1.78	-1.79	1.00
		Middepth	1.20	1.33	0.90	-1.76	-1.56	1.12
		Bottom	1.10	1.13	0.98	-1.45	-1.24	1.18
6/4/63	5A	Surface	1.26	0.99	1.27	-1.43	-1.40	1.02
		Middepth	1.24	0.93	1.33	-1.19	-1.31	0.91
		Bottom	0.91	0.69	1.32	-0.88	-0.86	1.03
2/14/63	5CR	Surface	1.14	1.25	0.91	-1.48	-1.14	1.30
		Middepth	1.05	1.42	0.75	-1.28	-0.93	1.39
		Bottom	0.66	0.94	0.70	-1.35	-0.94	1.43
5/22/63	5CR	Surface	2.49	2.41	1.03	-2.81	-2.71	1.03
		Middepth	1.73	2.14	0.81	-2.37	-2.44	0.97
		Bottom	1.96	1.73	1.13	-2.14	-1.84	1.16
6/4/63	5CR	Surface	1.83	1.67	1.10	-2.08	-1.73	1.20
		Middepth	1.32	1.35	0.98	-1.91	-1.64	1.16
		Bottom	1.34	0.97	1.39	-1.62	-1.09	1.47
2/14/63	5C	Surface	0.91	1.06	0.85	-1.21	-1.00	1.20
		Middepth	0.78	0.96	0.81	-0.97	-0.47	2.00
		Bottom	0.90	0.63	1.45	-0.86	-0.32	2.78
2/12/63	6C	Surface	1.22	1.32	0.93	-1.38	-1.50	0.93
		Middepth	1.25	1.37	0.91	-1.65	-1.31	1.27
		Bottom	1.15	1.02	1.12	-1.35	-0.94	1.43
2/12/63	6CR	Surface	1.67	1.47	1.14	-1.77	-1.72	1.03
		Middepth	1.22	1.50	0.81	-1.67	-1.35	1.23
		Bottom	1.05	1.04	1.01	-1.51	-1.05	1.45
2/14/63	6CR	Surface	0.91	0.86	1.06	-1.14	-1.56	0.73
		Middepth	0.88	1.45	0.61	-1.15	-0.71	1.64
		Bottom	0.71	1.01	0.70	-1.02	-0.33	3.13
5/22/63	6CR	Surface	2.56	2.79	0.92	-3.21	-3.21	1.00
		Middepth	1.90	2.53	0.75	-2.92	-3.02	0.96
		Bottom	1.67	1.90	0.88	-2.41	-2.19	1.10
6/4/63	6CR	Surface	1.88	1.86	1.01	-2.23	-2.13	1.04
		Middepth	1.38	1.65	0.84	-2.09	-1.99	1.05
		Bottom	1.25	1.10	1.14	-1.72	-1.58	1.09
2/12/63	6E	Surface	1.48	1.50	0.99	-1.80	-1.68	1.08
		Middepth	1.29	1.41	0.92	-1.46	-1.40	1.04
		Bottom	1.13	0.99	1.15	-1.28	-1.01	1.27

Table 4

Comparison of Mean Current Velocities for Postconstruction Verification

			On Flood			On Ebb		
Station	Plan	Depth	Model	Prototype	Model/Prototype	Model	Prototype	Model/Prototype
2CR	ACP	Surface	0.11*	0.54	--	-0.16*	-0.62	--
		Middepth	0.15*	0.50	--	-0.16*	-0.57	--
		Bottom	0.14*	0.39	--	-0.12*	-0.44	--
4CR	1	Surface	1.47	1.32	1.11	-2.20	**	--
		Middepth	1.40	1.43	0.98	-1.74	**	--
		Bottom	1.35	1.09	1.23	-1.68	**	--
5CR	1	Surface	2.06	2.21	0.93	-2.64	-2.47	1.06
		Middepth	1.75	1.78	0.98	-2.09	-2.08	1.01
		Bottom	1.88	1.36	1.39	-2.07	-1.38	1.49
6CR	1	Surface	1.97	1.51	1.30	-2.38	-1.96	1.22
		Middepth	1.39	1.36	1.02	-2.13	-1.74	1.22
		Bottom	1.41	0.64	2.22	-2.06	-1.29	1.61
5A	1	Surface	2.46	2.03	1.20	-2.85	-2.25	1.27
		Middepth	2.33	1.95	1.20	-2.62	-2.52	1.04
		Bottom	2.15	1.48	1.45	-2.47	-1.32	1.89
5C	1	Surface	2.28	1.62	1.41	-2.72	-2.19	1.23
		Middepth	1.55	1.61	0.96	-2.20	-2.00	1.10
		Bottom	1.51	0.96	1.56	-1.98	-1.07	1.85

Note: Prototype tide range = 8.4 ft. Model tide range = 8.0 ft.
* Below current meter threshold.
** Insufficient data to compute mean velocity.

Table 5

Statistical Data by Depth for Original Verification of Current Velocities

Date	Station	Depth	On Flood RMS	On Flood Mean	On Flood σ	On Ebb RMS	On Ebb Mean	On Ebb σ	Complete Cycle RMS	Complete Cycle Mean	Complete Cycle σ
5/22/63	2CR	Surface	0.26	-0.09	0.26	0.44	0.19	0.42	0.37	0.06	0.37
		Middepth	0.56	-0.39	0.42	0.90	0.70	0.59	0.77	0.20	0.76
		Bottom	0.51	0.39	0.34	1.38	1.16	0.78	1.04	0.77	0.71
6/4/63	2CR	Surface	0.28	0.13	0.26	0.47	-0.24	0.41	0.39	-0.07	0.39
		Middepth	0.31	0.10	0.30	0.42	0.03	0.44	0.37	0.06	0.38
		Bottom	0.41	0.36	0.20	0.42	0.24	0.35	0.41	0.30	0.28
5/22/63	4CR	Surface	0.56	-0.07	0.58	0.94	-0.30	0.93	0.77	-0.18	0.77
		Middepth	0.59	-0.12	0.61	0.70	-0.24	0.68	0.65	-0.18	0.64
		Bottom	0.40	-0.09	0.41	0.71	0.29	0.67	0.60	0.13	0.59
6/4/63	4CR	Surface	0.42	-0.05	0.44	0.63	0.01	0.66	0.55	-0.02	0.55
		Middepth	0.46	-0.07	0.48	0.61	0.10	0.62	0.55	0.02	0.56
		Bottom	0.50	-0.01	0.52	0.55	0.25	0.51	0.52	0.12	0.52
2/14/63	5A	Surface	0.57	0.54	0.20	0.36	-0.20	0.31	0.47	0.14	0.45
		Middepth	0.53	0.41	0.35	0.40	-0.29	0.29	0.46	0.03	0.47
		Bottom	0.35	0.28	0.22	0.30	-0.09	0.30	0.32	0.08	0.32
2/14/63	5CR	Surface	0.40	0.06	0.41	0.56	0.30	0.48	0.49	0.19	0.46
		Middepth	0.59	-0.35	0.50	0.39	0.11	0.39	0.49	-0.10	0.49
		Bottom	0.35	-0.21	0.29	0.63	0.40	0.51	0.52	0.12	0.52
5/22/63	5CR	Surface	0.46	0.10	0.46	0.78	0.28	0.75	0.64	0.19	0.62
		Middepth	0.56	-0.53	0.21	0.59	0.10	0.61	0.58	-0.22	0.55
		Bottom	0.21	0.12	0.18	0.60	0.46	0.41	0.44	0.28	0.35
6/4/63	5CR	Surface	0.35	0.12	0.34	0.80	0.41	0.72	0.62	0.26	0.57
		Middepth	0.33	-0.03	0.34	0.65	0.27	0.61	0.51	0.12	0.51
		Bottom	0.48	0.34	0.36	0.85	0.59	0.64	0.68	0.45	0.51
2/14/63	5C	Surface	0.46	-0.15	0.46	0.49	0.12	0.50	0.48	0.00	0.49
		Middepth	0.45	-0.32	0.34	0.49	0.35	0.35	0.47	0.04	0.48
		Bottom	0.48	0.28	0.41	0.68	0.57	0.39	0.60	0.43	0.42
2/12/63	6C	Surface	0.40	-0.10	0.40	0.20	-0.11	0.17	0.31	-0.11	0.30
		Middepth	0.33	-0.22	0.25	0.40	0.34	0.21	0.36	0.06	0.37
		Bottom	0.31	0.13	0.29	0.48	0.41	0.26	0.40	0.27	0.31
2/12/63	6CR	Surface	0.44	0.07	0.45	0.24	0.05	0.24	0.35	0.06	0.35
		Middepth	0.35	-0.20	0.30	0.32	0.24	0.23	0.34	0.02	0.34
		Bottom	0.25	0.01	0.26	0.58	0.55	0.22	0.45	0.28	0.36
2/14/63	6CR	Surface	0.40	0.05	0.42	0.57	-0.42	0.40	0.50	-0.20	0.47
		Middepth	0.67	-0.52	0.45	0.46	0.38	0.28	0.57	-0.03	0.58
		Bottom	0.40	-0.25	0.32	0.78	0.54	0.58	0.62	0.14	0.61
5/22/63	6CR	Surface	0.71	0.62	0.37	0.32	-0.19	0.27	0.55	0.21	0.52
		Middepth	0.80	-0.63	0.52	0.52	-0.11	0.53	0.68	-0.37	0.58
		Bottom	0.33	-0.23	0.24	0.31	0.22	0.23	0.32	-0.01	0.32
6/4/63	6CR	Surface	0.50	0.02	0.52	0.74	0.09	0.76	0.63	0.05	0.64
		Middepth	0.34	-0.26	0.23	0.51	0.10	0.52	0.43	-0.08	0.43
		Bottom	0.26	0.15	0.22	0.45	0.15	0.44	0.37	0.15	0.34
2/12/63	6E	Surface	0.45	-0.12	0.45	0.36	0.25	0.28	0.41	0.06	0.41
		Middepth	0.35	-0.22	0.29	0.27	0.17	0.22	0.31	-0.02	0.32
		Bottom	0.30	0.09	0.29	0.42	0.36	0.23	0.36	0.21	0.30

Table 6

Statistical Data by Depth for Postconstruction Verification of Current Velocities

Station	Depth	On Flood RMS	On Flood Mean	On Flood σ	On Ebb RMS	On Ebb Mean	On Ebb σ	Complete Cycle RMS	Complete Cycle Mean	Complete Cycle σ
4CR	Surface	0.48	0.28	0.41	0.63	-0.16	0.67	0.53	0.14	0.53
	Middepth	0.43	0.12	0.43	0.51	-0.12	0.54	0.46	0.04	0.47
	Bottom	0.52	0.37	0.38	0.67	0.24	0.68	0.58	0.32	0.50
5A	Surface	0.53	0.40	0.37	1.19	0.62	1.06	0.95	0.52	0.81
	Middepth	0.51	0.27	0.44	0.82	0.23	0.82	0.68	0.25	0.65
	Bottom	0.87	0.69	0.56	1.38	1.08	0.89	1.15	0.88	0.76
5CR	Surface	0.46	-0.26	0.39	0.88	0.12	0.90	0.70	-0.70	0.71
	Middepth	0.42	-0.14	0.41	0.95	0.14	0.98	0.73	0.00	0.75
	Bottom	0.66	0.52	0.43	0.86	0.64	0.59	0.77	0.58	0.51
5C	Surface	0.90	0.64	0.65	1.16	0.63	1.03	1.02	0.63	0.81
	Middepth	0.66	-0.03	0.69	1.02	0.40	0.97	0.84	0.17	0.84
	Bottom	0.56	0.50	0.26	1.33	0.88	1.04	0.99	0.67	0.74
6CR	Surface	0.70	0.58	0.41	0.63	0.42	0.49	0.67	0.50	0.45
	Middepth	0.44	0.13	0.44	0.58	0.38	0.45	0.51	0.26	0.45
	Bottom	0.90	0.85	0.29	0.67	0.53	0.42	0.78	0.67	0.40

Note: Prototype tide range = 8.4 ft. Model tide range = 8.0 ft.

Table 7

Summary of Statistical Analysis of Current Velocities by Depths

	On Flood		On Ebb		Complete Cycle	
	Original Verification	Postconstruction Verification	Original Verification	Postconstruction Verification	Original Verification	Postconstruction Verification
Root-Mean Square, fps						
Minimum	0.21	0.26	0.20	0.37	0.31	0.34
Average	0.43	0.56	0.55	0.81	0.51	0.70
Maximum	0.80	0.90	1.38	1.38	1.04	1.15
Mean Difference, fps						
Minimum magnitude	0.01	0.03	0.01	0.12	0.00	0.00
Average magnitude	0.21	0.37	0.29	0.43	0.16	0.38
Maximum magnitude	0.63	0.85	1.16	1.08	0.77	0.88
Standard Deviation, fps						
Minimum	0.18	0.12	0.17	0.14	0.28	0.16
Average	0.36	0.40	0.45	0.66	0.48	0.55
Maximum	0.61	0.69	0.93	1.06	0.77	1.23

Table 8

Statistical Data by Station for Original Verification of Current Velocities

Verification Prototype Date	Station	On Flood RMS	On Flood Mean	On Flood σ	On Flood A3	On Flood A4	On Ebb RMS	On Ebb Mean	On Ebb σ	On Ebb A3	On Ebb A4	Complete Cycle RMS	Complete Cycle Mean	Complete Cycle σ	Complete Cycle A3	Complete Cycle A4
5/22/63	2CR	0.47	-0.02	0.47	0.03	2.45	0.97	0.67	0.72	-0.38	2.40	0.78	0.34	0.70	0.22	2.30
6/4/63	2CR	0.34	0.21	0.28	-0.90	4.61	0.44	0.00	0.44	0.10	2.16	0.39	0.10	0.38	-0.41	2.66
5/22/63	4CR	0.53	-0.09	0.53	0.55	2.77	0.79	-0.06	0.80	0.32	2.07	0.68	-0.08	0.68	0.42	2.50
6/4/63	4CR	0.46	-0.04	0.47	-0.77	4.13	0.60	0.12	0.59	0.31	2.20	0.54	0.04	0.54	0.08	3.11
2/14/63	5A	0.49	0.41	0.28	1.28	7.03	0.35	-0.19	0.30	0.78	2.79	0.42	0.09	0.42	0.30	2.92
2/14/63	5CR	0.46	-0.17	0.43	-0.17	2.95	0.54	0.27	0.47	0.34	3.10	0.50	0.07	0.50	0.17	3.29
5/22/63	5CR	0.44	-0.10	0.43	0.47	3.08	0.66	0.27	0.61	-0.40	2.58	0.56	0.08	0.56	0.15	2.48
6/4/63	5CR	0.40	0.15	0.37	-0.10	1.74	0.77	0.42	0.65	0.29	2.72	0.61	0.28	0.54	0.60	3.55
2/14/63	5C	0.46	-0.06	0.47	0.12	3.19	0.56	0.35	0.45	0.06	1.90	0.52	0.16	0.50	0.02	2.59
2/12/63	6C	0.35	-0.06	0.34	0.39	1.98	0.38	0.21	0.32	-0.12	1.96	0.36	0.08	0.36	0.05	1.90
2/12/63	6CR	0.36	-0.04	0.36	0.58	2.54	0.41	0.28	0.31	-0.26	2.87	0.38	0.12	0.37	0.03	2.16
2/14/63	6CR	0.50	-0.24	0.45	-0.08	2.82	0.61	0.16	0.60	-0.26	1.91	0.56	-0.03	0.57	0.08	2.21
5/22/63	6CR	0.65	-0.08	0.65	0.05	2.75	0.40	-0.03	0.40	0.70	3.95	0.54	-0.05	0.54	0.12	3.49
6/4/63	6CR	0.38	-0.03	0.39	0.15	1.92	0.58	0.11	0.57	0.80	3.15	0.49	0.04	0.49	0.83	3.70
2/12/63	6E	0.37	-0.08	0.37	0.11	1.85	0.36	0.26	0.25	-0.75	3.17	0.36	0.08	0.36	-0.46	2.13
All data as a single sample		0.45	-0.02	0.45	-0.07	3.15	0.59	0.19	0.56	0.24	3.02	0.53	0.09	0.52	0.25	3.32

Table 9

Statistical Data by Station for Postconstruction Verification of Current Velocities

Station	On Flood					On Ebb					Complete Cycle				
	RMS	Mean	σ	A3	A4	RMS	Mean	σ	A3	A4	RMS	Mean	σ	A3	A4
2CR	0.35	-0.30	0.19	0.76	2.93	0.43	-0.40	0.16	1.08	4.05	0.41	-0.37	0.17	0.87	3.75
4CR	0.48	0.25	0.41	-0.59	3.49	0.61	0.00	0.63	0.30	1.99	0.52	0.17	0.50	-0.33	2.56
5A	0.66	0.45	0.49	-0.28	2.22	1.16	0.64	0.97	-0.49	2.00	0.95	0.55	0.78	-0.31	2.50
5CR	0.52	0.04	0.53	0.27	1.72	0.90	0.30	0.86	-0.51	2.10	0.73	0.17	0.72	-0.13	2.15
5C	0.73	0.38	0.63	-0.05	3.54	1.18	0.64	1.00	-0.56	2.22	0.95	0.49	0.82	-0.25	2.64
6CR	0.69	0.50	0.48	-0.29	2.37	0.63	0.45	0.45	-0.65	2.13	0.66	0.48	0.46	-0.45	2.31
All data as a single sample	0.60	0.26	0.55	0.08	2.60	0.86	0.25	0.82	0.21	2.10	0.74	0.25	0.70	0.18	2.50

Note: Prototype tide range = 8.4 ft. Model tide range = 8.0 ft.

Table 10

Summary of Statistical Analysis of Current Velocities for Stations

	On Flood		On Ebb		Complete Cycle	
	Original Verification	Postconstruction Verification	Original Verification	Postconstruction Verification	Original Verification	Postconstruction Verification
Root-Mean Square, fps						
Minimum	0.35	0.35	0.35	0.43	0.36	0.41
Average	0.45	0.57	0.56	0.82	0.52	0.70
Maximum	0.53	0.73	0.97	1.18	0.78	0.95
Mean Difference, fps						
Minimum magnitude	0.03	0.04	0.03	0.00	0.03	0.17
Average magnitude	0.20	0.32	0.23	0.41	0.11	0.37
Maximum magnitude	0.41	0.50	0.67	0.64	0.34	0.55
Standard Deviation, fps						
Minimum	0.28	0.19	0.25	0.16	0.36	0.17
Average	0.42	0.46	0.49	0.68	0.50	0.58
Maximum	0.65	0.63	0.80	1.00	0.70	0.82

Table 11

Flow Predominance for Original Verification

Date	Tide	Station	Depth	Flow Predominance Percent of Total Flow Model	Prototype
5/22/63	Spring	2CR	Surface	-13	-9
			Middepth	-15	+2
			Bottom	-11	+2
6/4/63	Mean	2CR	Surface	-4	-14
			Middepth	-6	-10
			Bottom	-1	-11
5/22/63	Spring	4CR	Surface	-9	-8
			Middepth	-8	-6
			Bottom	-12	-7
6/4/63	Mean	4CR	Surface	-9	-9
			Middepth	-10	-8
			Bottom	-3	-4
2/14/63	Neap	5A	Surface	+12	-14
			Middepth	+3	-12
			Bottom	+3	-9
2/14/63	Neap	5CR	Surface	-3	+2
			Middepth	+8	+7
			Bottom	-8	+15
5/22/63	Spring	5CR	Surface	-7	-5
			Middepth	-11	-3
			Bottom	-6	-1
6/4/63	Mean	5CR	Surface	-7	-1
			Middepth	-9	-5
			Bottom	-5	+1
2/14/63	Neap	5C	Surface	-9	-2
			Middepth	-10	+13
			Bottom	-3	+15
2/12/63	Neap	6C	Surface	-3	-3
			Middepth	-9	+1
			Bottom	-4	+2
2/12/63	Neap	6CR	Surface	-3	-4
			Middepth	-8	+3
			Bottom	-9	+2
2/14/63	Neap	6CR	Surface	-9	-18
			Middepth	-7	+14
			Bottom	-3	+25
5/22/63	Spring	6CR	Surface	-2	-1
			Middepth	-10	-4
			Bottom	-9	-4
6/4/63	Mean	6CR	Surface	-4	-3
			Middepth	-10	-5
			Bottom	-8	-9
2/12/63	Neap	6E	Surface	-9	-3
			Middepth	-7	0
			Bottom	-7	+3

Note: A negative sign (-) denotes flow predominance in the ebb direction; values with a positive sign (+) denote flood predominance.

Table 12

Flow Predominance for Postconstruction Verification

March 20, 1976

Station	Depth	Flow Predominance* Percent of Total Flow Model	Prototype
2CR	Surface	**	-21
	Middepth	**	-24
	Bottom	**	-26
4CR	Surface	-10	†
	Middepth	-5	†
	Bottom	-5	†
5A	Surface	-7	-6
	Middepth	-7	-6
	Bottom	-5	+1
5CR	Surface	-8	-3
	Middepth	-8	-4
	Bottom	0	+2
5C	Surface	-4	0
	Middepth	-12	-3
	Bottom	-9	+1
6CR	Surface	-5	-8
	Middepth	-11	-8
	Bottom	-11	-23

Note: Prototype tide range was 8.4 ft, while the model tide range was 8.0 ft.

* A negative sign (-) denotes flow predominance in the ebb direction; values with a positive sign (+) denote flood predominance.

** Velocities were below the threshold of the current meter.

† There was insufficient data to compute flow predominances.

Table 13

Summary of Flow Predominance Data

		Original Verification			Postconstruction Verification		
		Frequency			Frequency		
Station	Depth	Correct Direction	Incorrect Direction	Average Deviation Percent	Correct Direction	Incorrect Direction	Average Deviation Percent
2CR	Surface	2	0	7	--	--	--
	Middepth	1	1	10	--	--	--
	Bottom	1	1	12	--	--	--
4CR	Surface	2	0	1	--	--	--
	Middepth	2	0	2	--	--	--
	Bottom	2	0	3	--	--	--
5A	Surface	0	1	26	1	0	1
	Middepth	0	1	15	1	0	1
	Bottom	0	1	12	1	0	6
5CR	Surface	3	0	4	1	0	5
	Middepth	3	0	4	1	0	4
	Bottom	2	1	11	1	0	2
5C	Surface	1	0	7	1	0	4
	Middepth	0	1	23	1	0	9
	Bottom	0	1	18	0	1	10
6CR	Surface	4	0	3	1	0	3
	Middepth	2	2	11	1	0	3
	Bottom	2	2	11	1	0	12
All above		27	12	10	11	1	5

Note: Those cases which indicated an incorrect direction but for which both model and prototype flow predominance were between -5 and +5 percent (balanced flow) were considered to be correct direction.

Table 14

Results of Shoaling Tests

Model Verification

Area	Channel Shoaling Section	Prototype: Average Annual Shoaling cu yd	Prototype: Percent of Total	Model: Material Retrieved cc	Model: Percent of Total
East River - Port					
Sta 70+50 to 75+00	1	114,800	10.4	45	11.6
Sta 75+00 to 81+00	2	140,200	12.6	63	16.2
Sta 81+00 to 87+00	3	262,100	23.6	51	13.1
Sta 87+00 to 93+00	4	254,700	23.0	68	17.5
Sta 93+00 to 99+00	5	217,400	19.6	78	20.2
Sta 99+00 to 105+00	6	119,800	10.8	83	21.4
Total		1,109,000	100.0	388	100.0
Centroid (shoaling section)			3.61		3.83

Table 15

Model Shoaling Volumes (cc) East River Port

Channel Shoaling Section No.	Base Tests				Partial Closure 36 Ft		Alternate Closure Mean Tide	
	Mean Tide		Spring Tide		Mean Tide	Spring Tide		
	30 ft	36 ft	30 ft	36 ft			30 ft	36 ft
1	45	15	25	25	10	10	0	0
	(12%)	(8%)	(5%)	(5%)	(10%)	(7%)		
2	63	35	55	110	10	15	0	0
	(16%)	(19%)	(12%)	(21%)	(10%)	(11%)		
3	51	30	60	110	20	22	0	0
	(13%)	(17%)	(13%)	(21%)	(20%)	(16%)		
4	68	25	90	125	20	27	0	0
	(18%)	(14%)	(19%)	(24%)	(20%)	(20%)		
5	78	55	160	100	20	25	23	20
	(20%)	(31%)	(34%)	(19%)	(20%)	(18%)	(41%)	(50%)
6	83	20	75	45	20	37	33	20
	(21%)	(11%)	(16%)	(9%)	(20%)	(27%)	(59%)	(50%)
Total	388	180	465	515	100	136	56	40
Centroid (channel section)	3.83	3.72	4.15	3.58	3.90	4.13	5.59	5.50

Table 16

Average Annual Prototype Shoaling Rates for Brunswick

Harbor - East River Port

Shoaling Section	Preconstruction (1000 × cu yd)	Preconstruction Percent of Total	Partial Closure Plan (1000 cu yd)	Partial Closure Plan Percent of Total	Alternate Closure Plan (1000 cu yd)	Alternate Closure Plan Percent of Total
1	77.3	10	38.9	7	34.5	6
2	104.4	13	74.2	13	76.4	13
3	168.3	22	132.5	24	137.1	24
4	137.3	24	126.6	23	136.2	23
5	155.9	20	91.9	17	103.3	18
6	87.9	11	86.1	16	93.7	16
Total	781.1	100	550.2	100	581.2	100
Centroid	3.64		3.78		3.82	

Table 17

Summary of Relative Changes in Shoaling Volumes from Preconstruction to Partial Closure Plan

	Prototype				Model*							
					Base Test Volumes cc		Partial Closure Volumes cc		Relative Change cc		Percent of Base Total Volume	
Channel Section	Preconstruction (Base) Volumes 1000 cu yd	Partial Closure Plan Volumes 1000 cu yd	Relative Change 1000 cu yd	Percent of Preconstruction Total	Mean Tide	Spring Tide	Mean Tide	Spring Tide	Mean Tide	Spring Tide	Mean Tide	Spring Tide
1	77.3	38.9	-38.4	-4.9	15	25	10	10	-5	-15	-2.8	-2.9
2	104.4	74.2	-30.2	-3.9	35	110	10	15	-25	-95	-13.9	-18.4
3	168.3	132.5	-35.8	-4.6	30	110	20	22	-10	-88	-5.5	-17.1
4	187.3	126.6	-60.7	-7.8	25	125	20	27	-5	-98	-2.8	-19.0
5	155.9	91.9	-64.0	-8.2	55	100	20	25	-35	-75	-19.4	-14.6
6	87.9	86.1	-1.8	-0.2	20	45	20	37	0	-8	0.0	-1.6
Total	781.1	550.2	-230.9	-29.6	180	515	100	136	-80	-379	-44.4	-73.6

* 36-ft channel.

Table 18

Summary of Relative Changes in Shoaling Volumes from Preconstruction to Alternate Closure Plan

Channel Section	Prototype: Preconstruction (Base) Volumes 1000 cu yd	Prototype: Alternate Closure Plan Volumes 1000 cu yd	Prototype: Relative Change 1000 cu yd	Prototype: Percent of Preconstruction Total	Model*: Base Test Volumes cc 30 ft	Model*: Base Test Volumes cc 36 ft	Model*: Alternate Closure Volumes cc 30 ft	Model*: Alternate Closure Volumes cc 36 ft	Model*: Relative Change cc 30 ft	Model*: Relative Change cc 36 ft	Model*: Percent of Base Total Volume 30 ft	Model*: Percent of Base Total Volume 36 ft
1	77.3	34.5	-42.8	-5.5	45	15	0	0	-45	-15	-11.6	-8.3
2	104.4	76.4	-28.0	-3.6	63	35	0	0	-63	-35	-16.2	-19.4
3	168.3	137.1	-31.2	-4.0	51	30	0	0	-51	-30	-13.1	-16.7
4	187.3	136.2	-51.1	-6.5	68	25	0	0	-68	-25	-17.5	-13.9
5	155.9	103.3	-52.6	-6.7	78	55	23	20	-55	-35	-14.2	-19.4
6	87.9	93.7	+5.8	+0.7	83	20	33	20	-50	0	-12.9	0.0
Total	781.1	581.2	-199.9	-25.6	388	180	56	40	-332	-140	-85.5	-77.8

* Mean tide.

Table 19

Summary of Relative Changes in Shoaling Volumes from Partial Closure Plan to Alternate Closure Plan

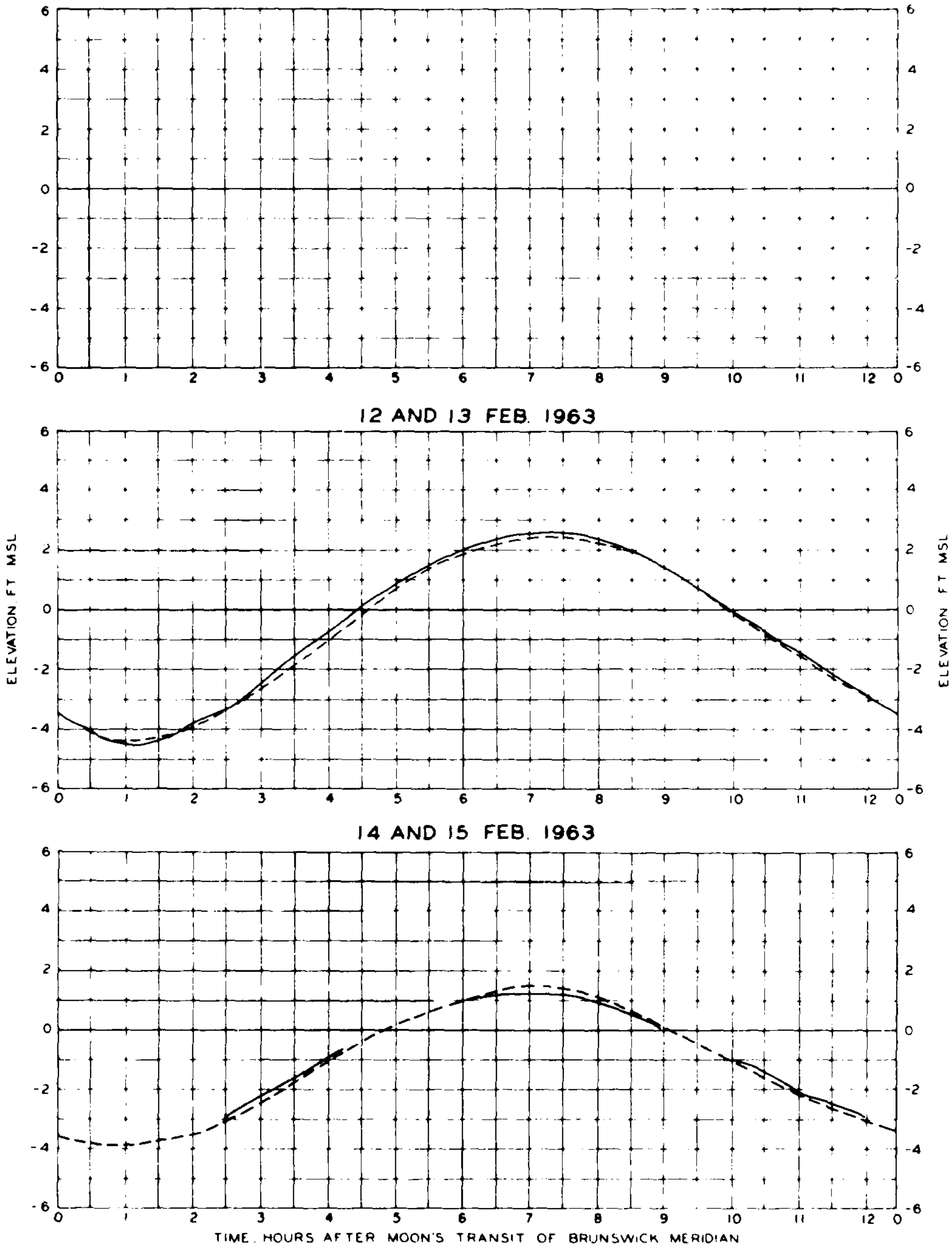
	Prototype				Model*			
Channel Section	Partial Closure Plan Volumes 1000 cu yd	Alternate Closure Plan Volumes 1000 cu yd	Relative Change 1000 cu yd	Percent of Total Partial Closure Plan Volume	Partial Closure Plan Volumes cc	Alternate Closure Plan Volumes cc	Relative Change cc	Percent of Total Partial Closure Plan Volume
1	38.9	34.5	-4.4	-0.8	10	0	-10	-10.0
2	74.2	76.4	+2.2	+0.4	10	0	-10	-10.0
3	132.5	137.1	+4.6	+0.8	20	0	-20	-20.0
4	126.6	136.2	+9.6	+1.7	20	0	-20	-20.0
5	91.9	103.3	+11.4	+2.1	20	20	0	0.0
6	86.1	93.7	+7.6	+1.4	20	20	0	0.0
Total	550.2	581.2	+31.0	+5.6	100	40	-60	-60.0

* Mean tide 36-ft channel test.

Table 20

Comparison of Statistical Analyses of Current Velocities for Two Model Studies

Statistical Parameter	Phase of Tidal Cycle	Delaware River Model Study: Original Verification	Delaware River Model Study: Postconstruction Verification	Brunswick Harbor Model Study: Original Verification	Brunswick Harbor Model Study: Postconstruction Verification
Average magnitude of mean differences, fps	Flood	0.36	0.31	0.21	0.37
	Ebb	0.38	0.24	0.29	0.43
Average standard deviation, fps	Flood	0.50	0.41	0.36	0.40
	Ebb	0.52	0.32	0.45	0.66



ORIGINAL

LEGEND

- - - - - - PROTOTYPE (12,13,14 & 15 FEB 1963)

——— MODEL

VERIFICATION OF TIDAL HEIGHTS

BRUNSWICK STATE DOCK

12-15 FEB. 1963

AD-A106 549 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS F/G 13/7
PHYSICAL HYDRAULIC MODELS: ASSESSMENT OF PREDICTIVE CAPABILITIE--ETC(U)
SEP 81 J V LETTER, W H MCANALLY
UNCLASSIFIED AEWES-RR-H-75-3-3 NL

2 OF 2
AD A
106549

END
DATE
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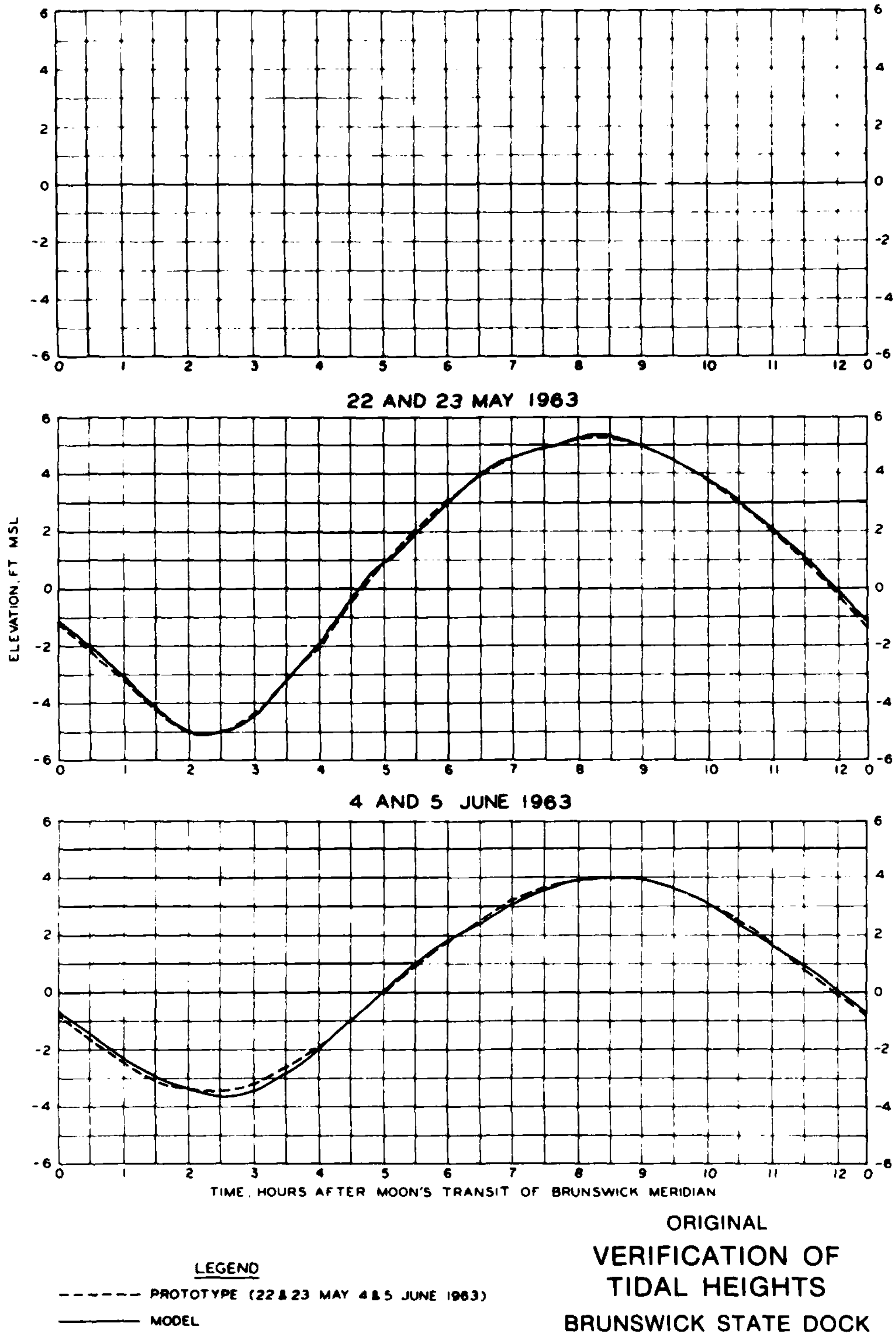
11-81
DTIC



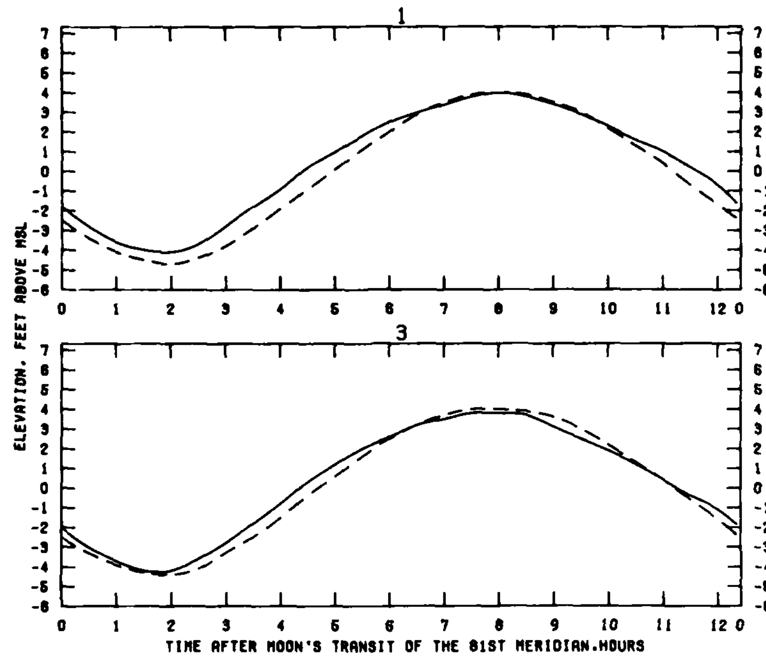
ORIGINAL

VERIFICATION OF TIDAL HEIGHTS

BRUNSWICK STATE DOCK

22 AND 23 MAY, 4 AND 5 JUNE 1963

PLATE 2



TEST CONDITIONS

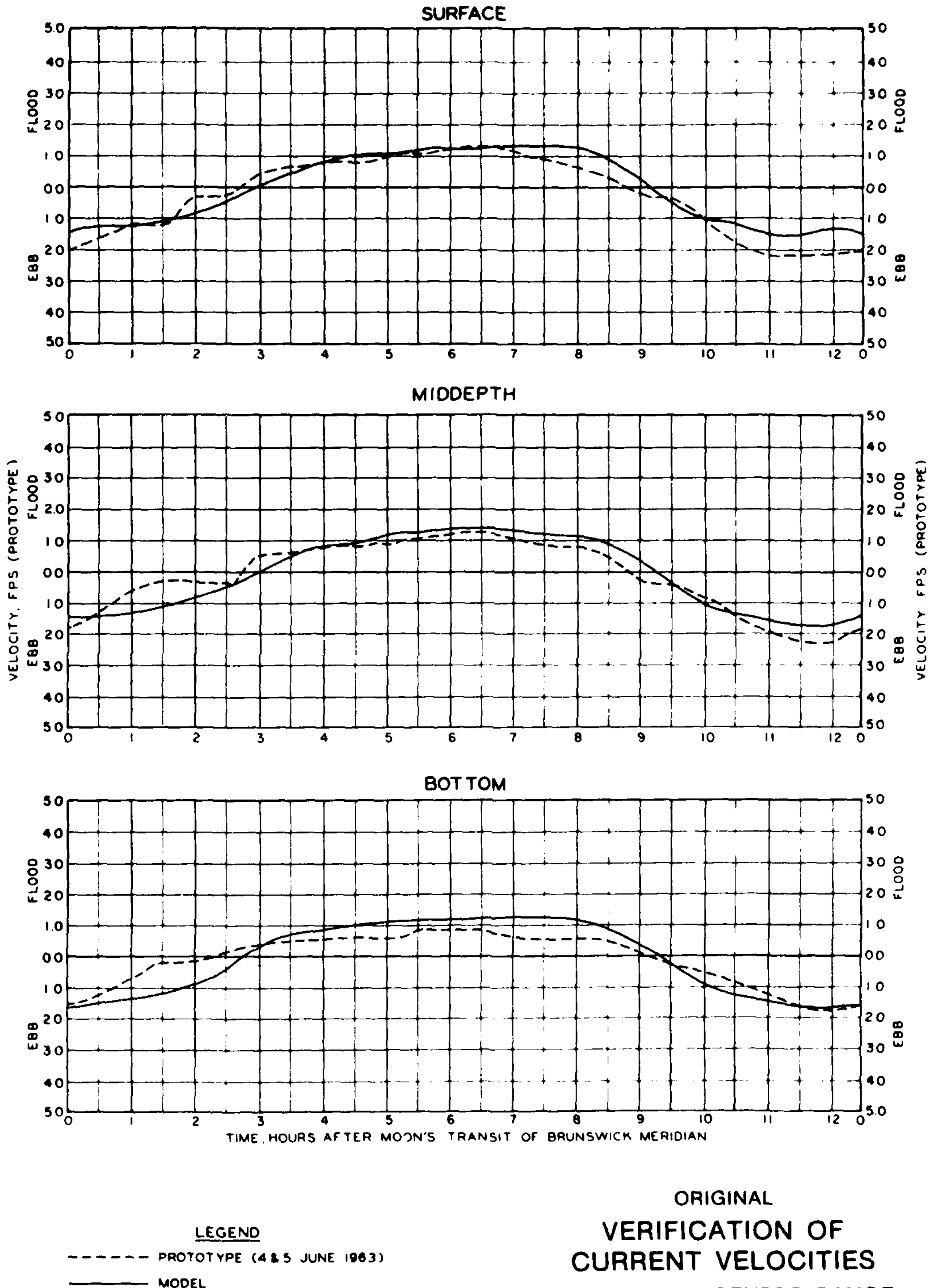
TIDE......................................MEAN
SOURCE SALINITY..........................30 PPT
FRESHWATER DISCHARGE.........................0
PROTOTYPE DATA COLLECTED......MARCH 20, 1978

LEGEND

PROTOTYPE — — —
MODEL (PLAN 1) ———

POSTCONSTRUCTION VERIFICATION OF TIDAL ELEVATIONS

STATIONS 1 AND 3



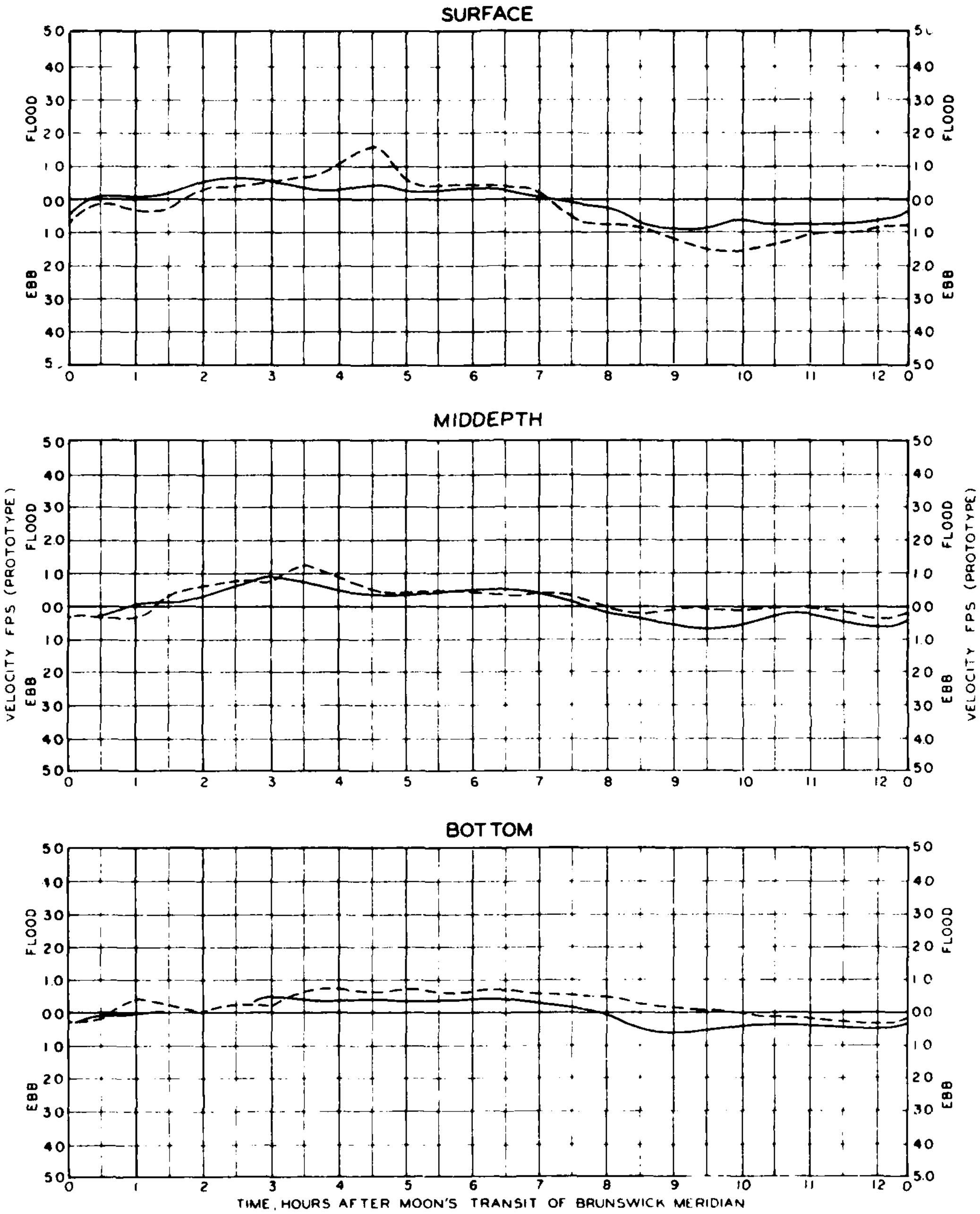
ORIGINAL

VERIFICATION OF CURRENT VELOCITIES

LOCATION 2, CENTER RANGE

4 AND 5 JUNE 1963

PLATE 4



LEGEND

------ PROTOTYPE (14 & 15 FEB 1963)

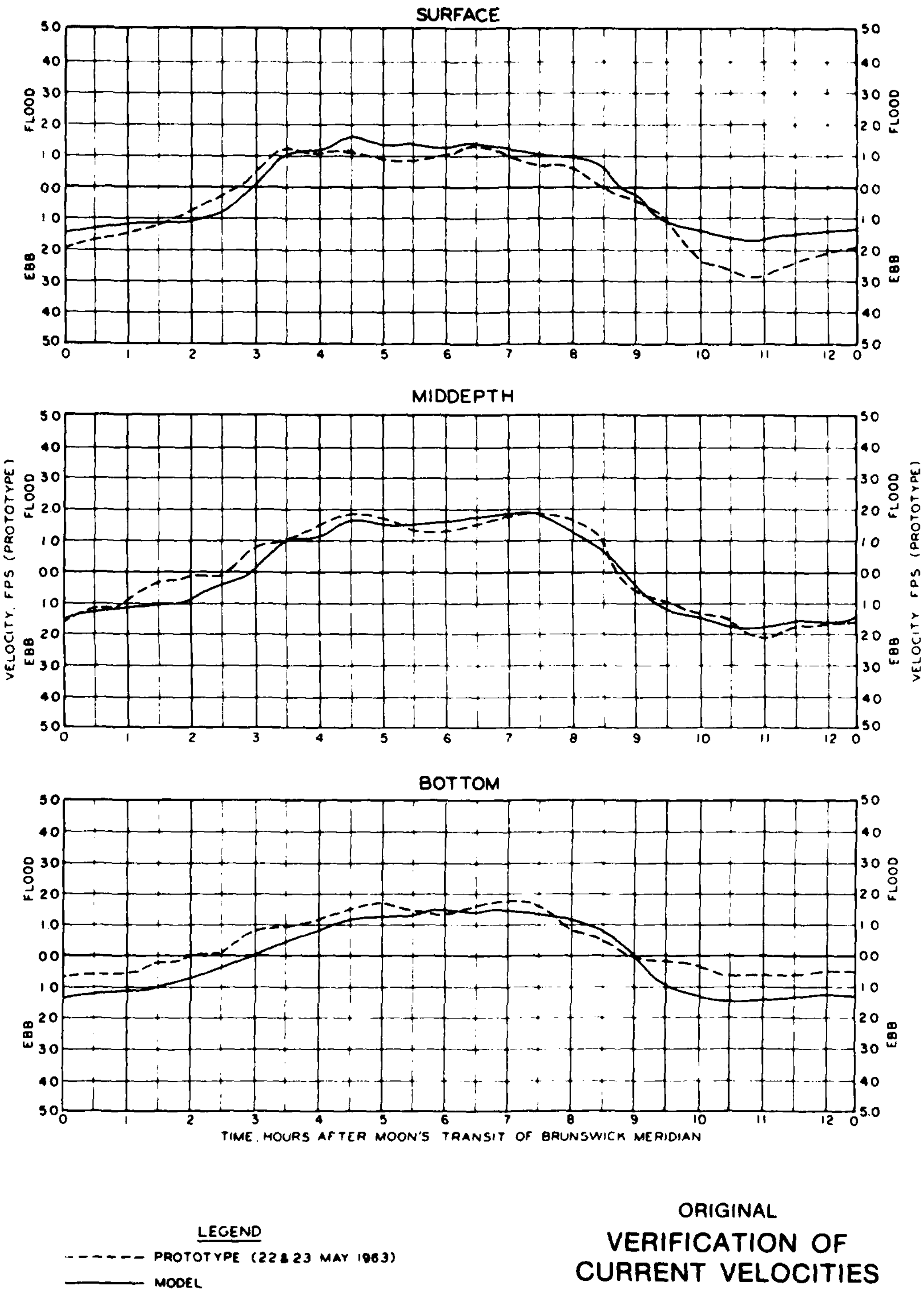
——— MODEL

ORIGINAL

VERIFICATION OF CURRENT VELOCITIES

LOCATION 3, CENTER RANGE

14 AND 15 FEB. 1963



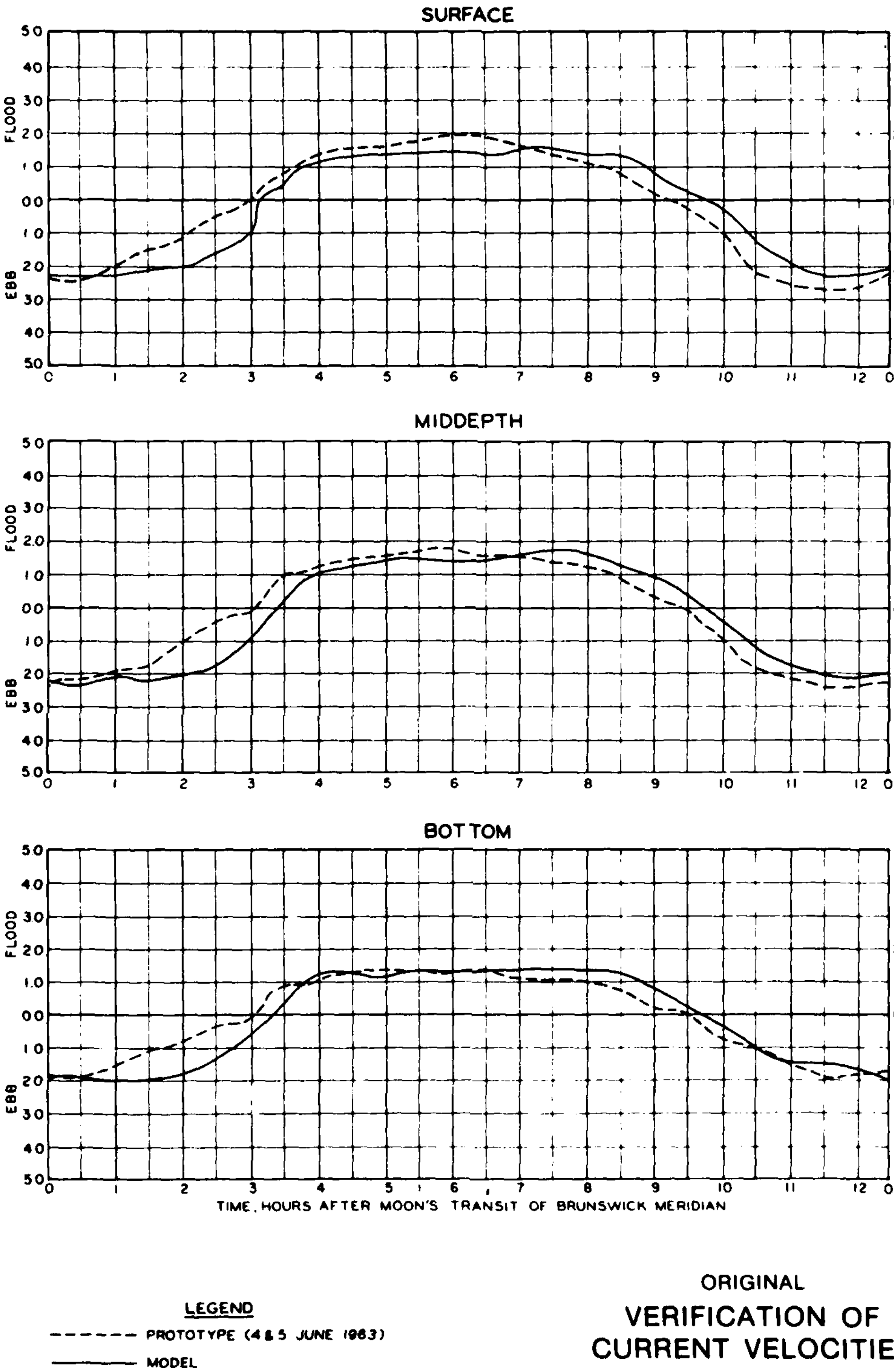
ORIGINAL

VERIFICATION OF CURRENT VELOCITIES

LOCATION 3, CENTER RANGE

22 AND 23 MAY 1963

PLATE 6

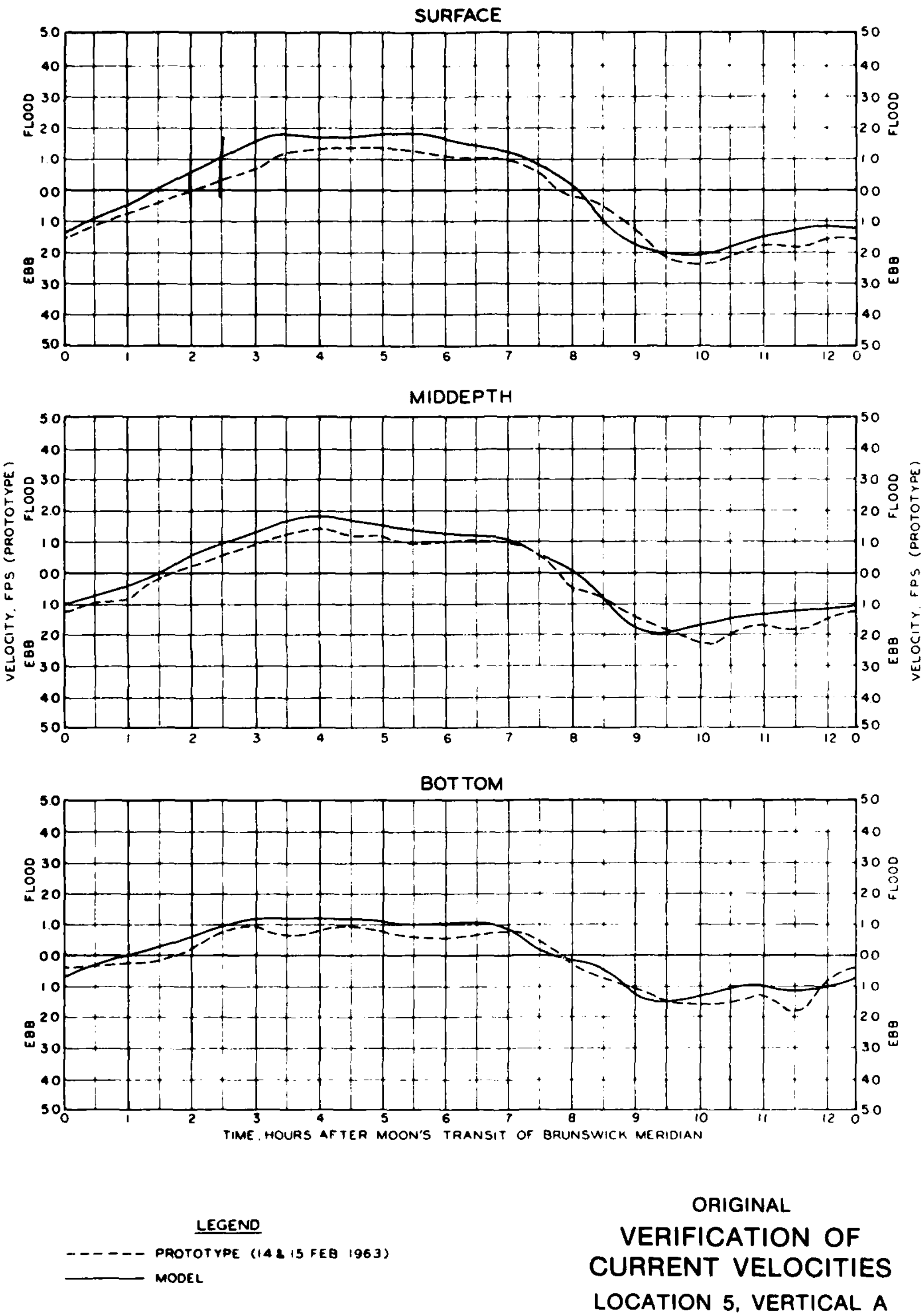


ORIGINAL

VERIFICATION OF CURRENT VELOCITIES

LOCATION 4, CENTER RANGE
4 AND 5 JUNE 1963

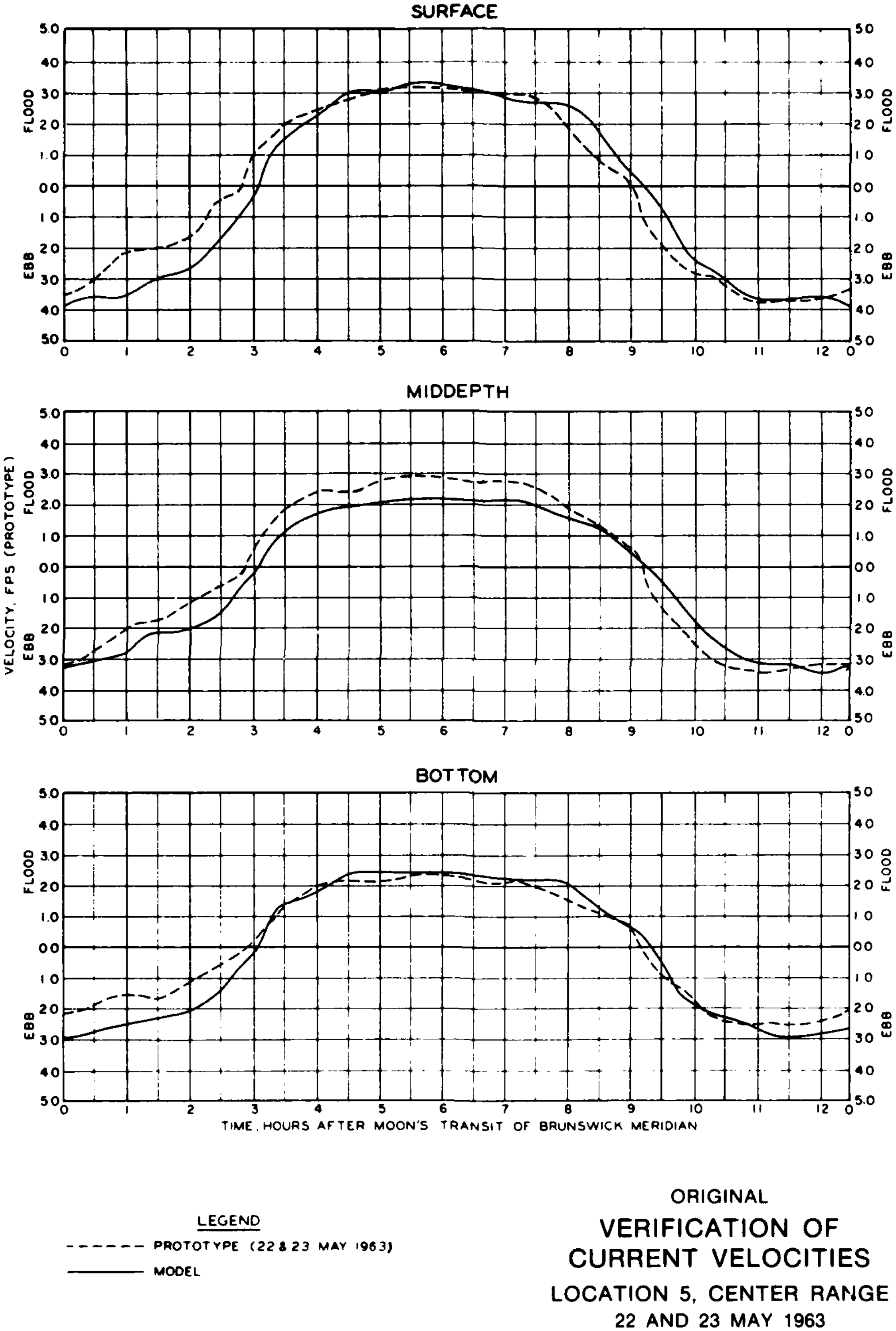
PLATE 7



ORIGINAL

VERIFICATION OF CURRENT VELOCITIES

LOCATION 5, VERTICAL A
14 AND 15 FEB. 1963

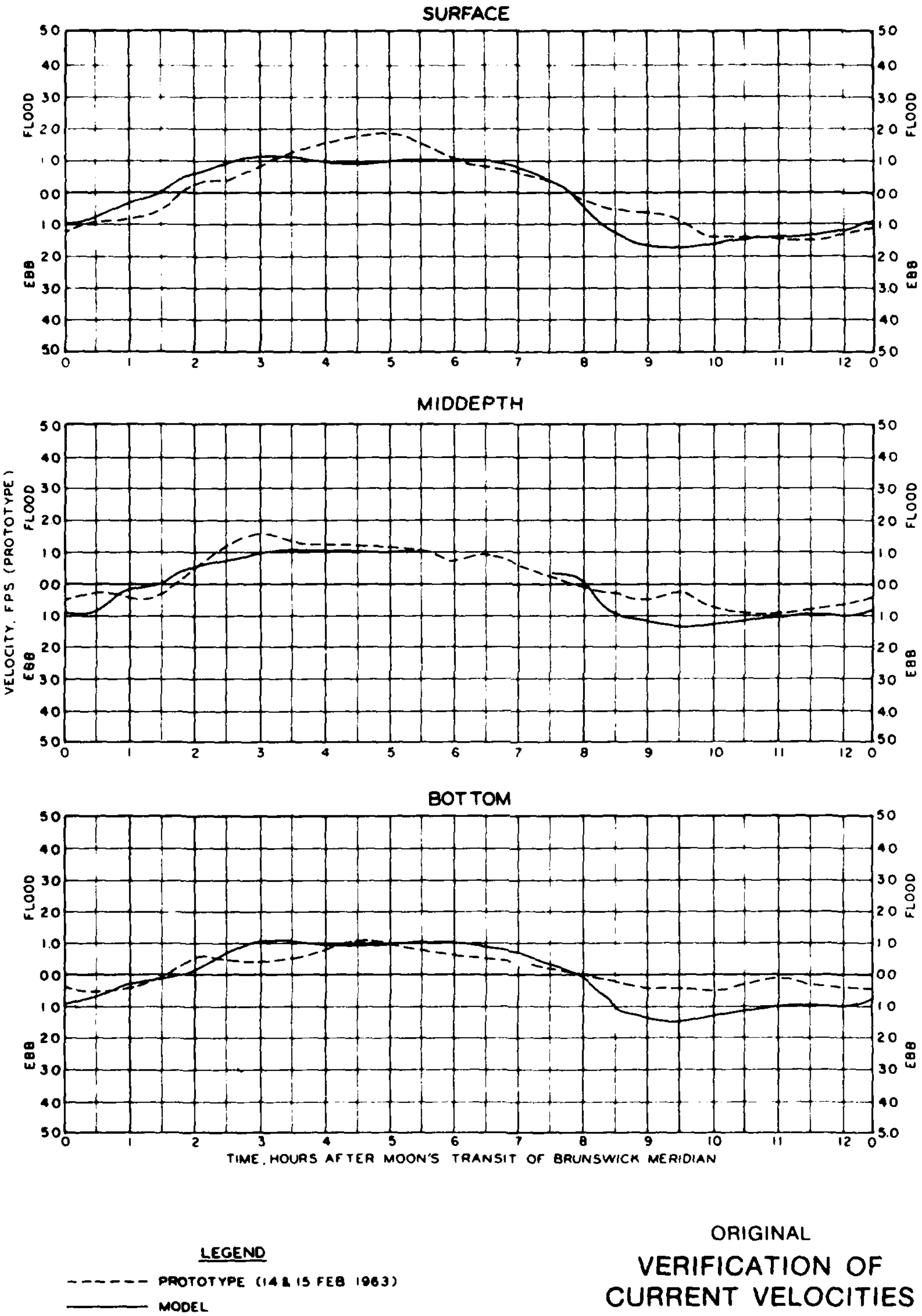


ORIGINAL

VERIFICATION OF CURRENT VELOCITIES

LOCATION 5, CENTER RANGE

22 AND 23 MAY 1963



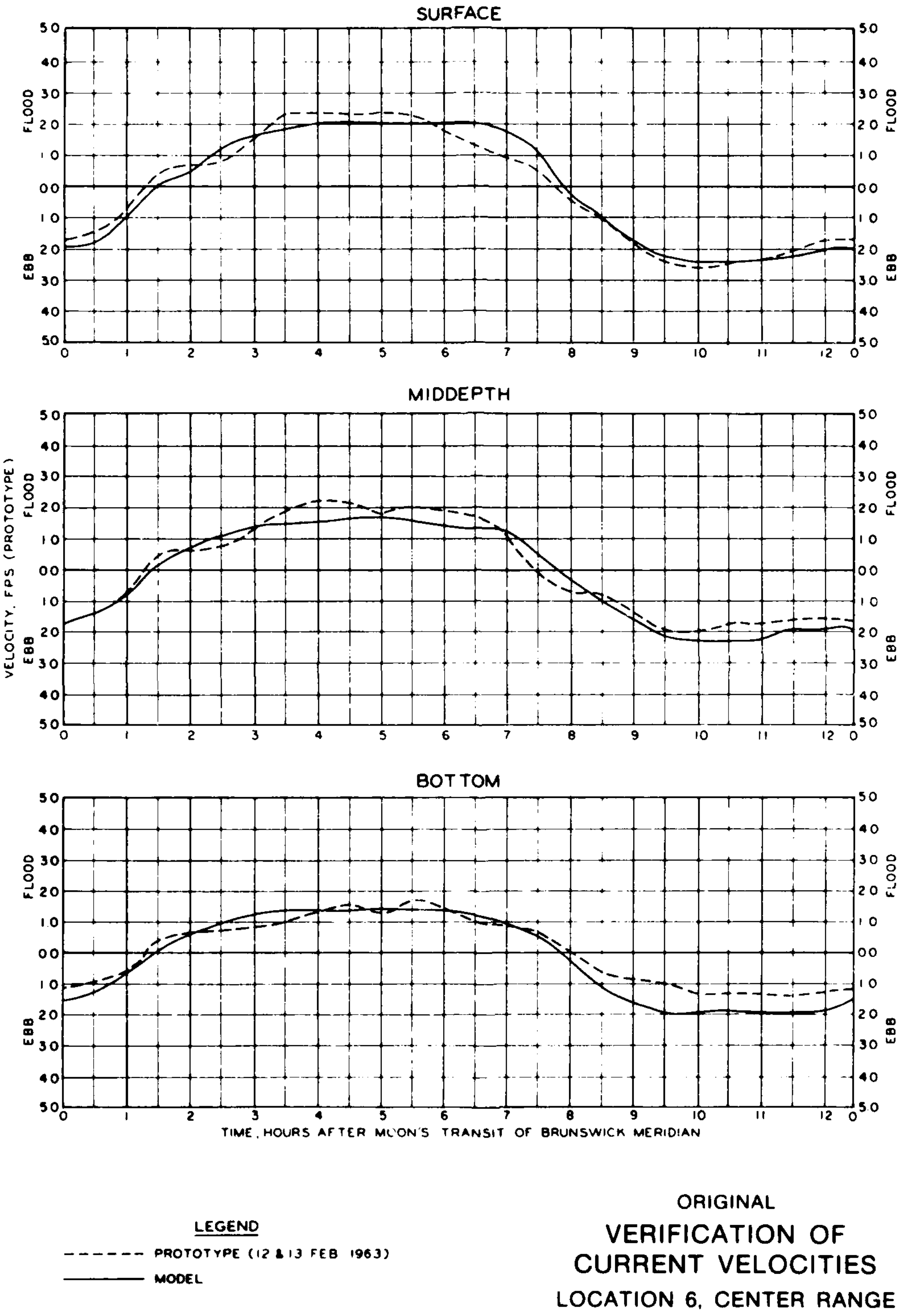
ORIGINAL

VERIFICATION OF CURRENT VELOCITIES

LOCATION 5, VERTICAL C

14 AND 15 FEB. 1963

PLATE 10

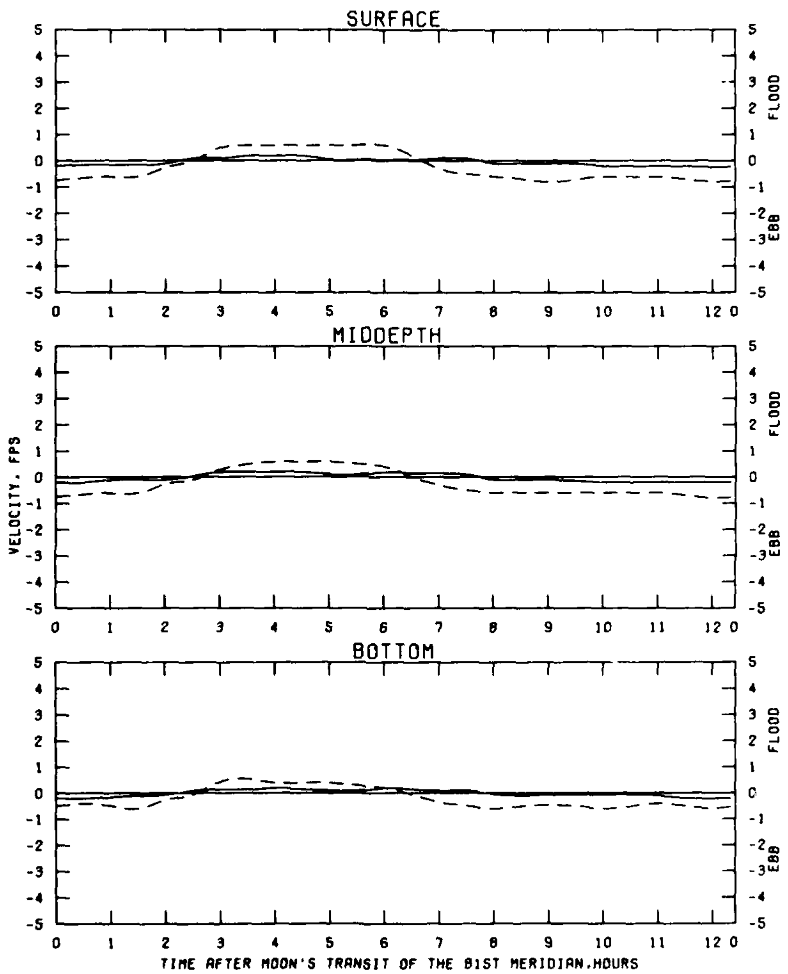


ORIGINAL

VERIFICATION OF CURRENT VELOCITIES

LOCATION 6, CENTER RANGE

12 AND 13 FEB. 1963



TEST CONDITIONS

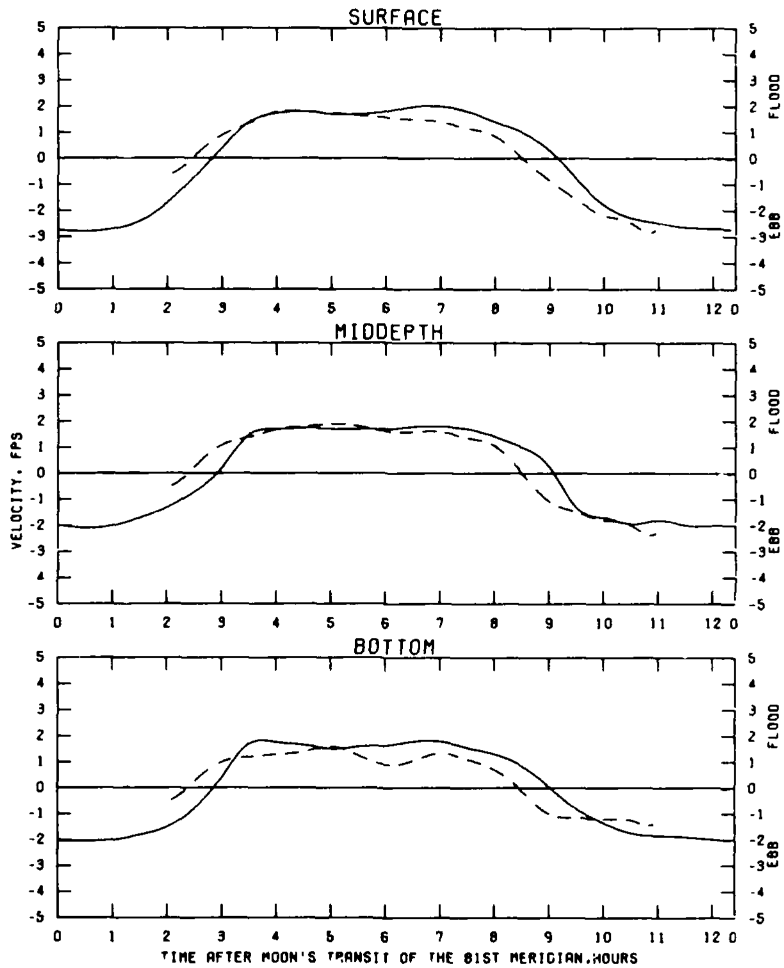
TIDE....................................MEAN
SOURCE SALINITY........................30 PPT
FRESHWATER DISCHARGE........................0
PROTOTYPE DATA COLLECTED......MARCH 20, 1976

LEGEND

PROTOTYPE – – –
MODEL (APC) ———

POSTCONSTRUCTION VERIFICATION OF CURRENT VELOCITIES

STATION 2CR



TEST CONDITIONS

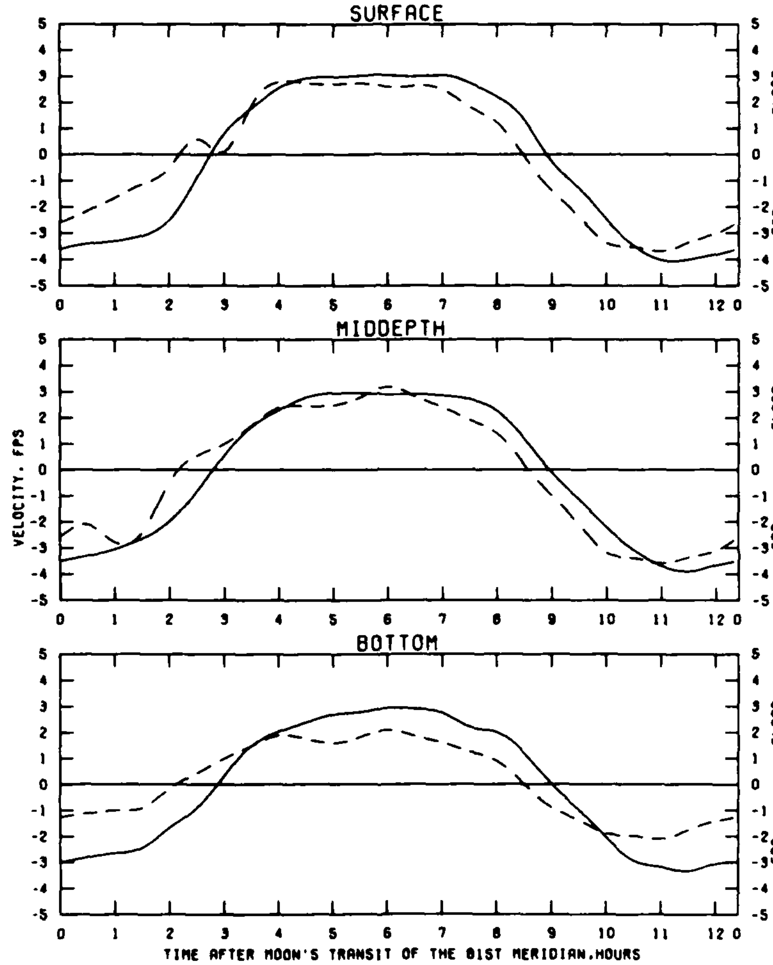
TIDE....................................MEAN
SOURCE SALINITY........................30 PPT
FRESHWATER DISCHARGE........................0
PROTOTYPE DATA COLLECTED......MARCH 20, 1976

LEGEND

PROTOTYPE — — —
MODEL (PLAN 1) ———

POSTCONSTRUCTION VERIFICATION OF CURRENT VELOCITIES

STATION 4CR



TEST CONDITIONS

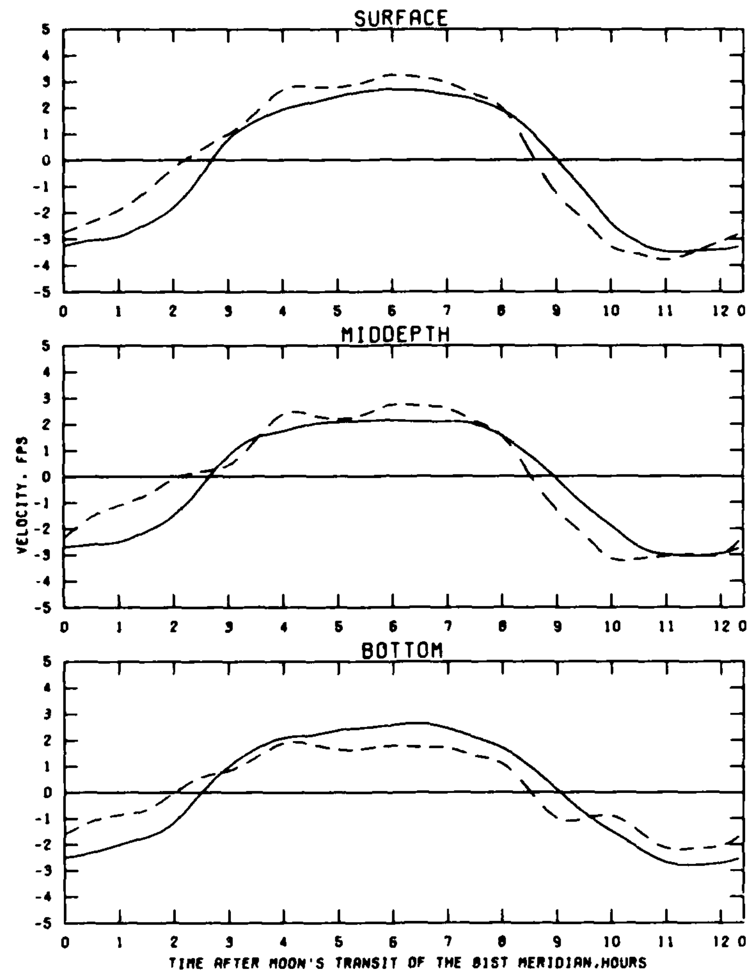
TIDE....................................MEAN
SOURCE SALINITY........................30 PPT
FRESHWATER DISCHARGE........................0
PROTOTYPE DATA COLLECTED......MARCH 20, 1976

LEGEND

PROTOTYPE — — —
MODEL (PLAN 1) ———

POSTCONSTRUCTION
VERIFICATION OF
CURRENT VELOCITIES

STATION 5A



TEST CONDITIONS

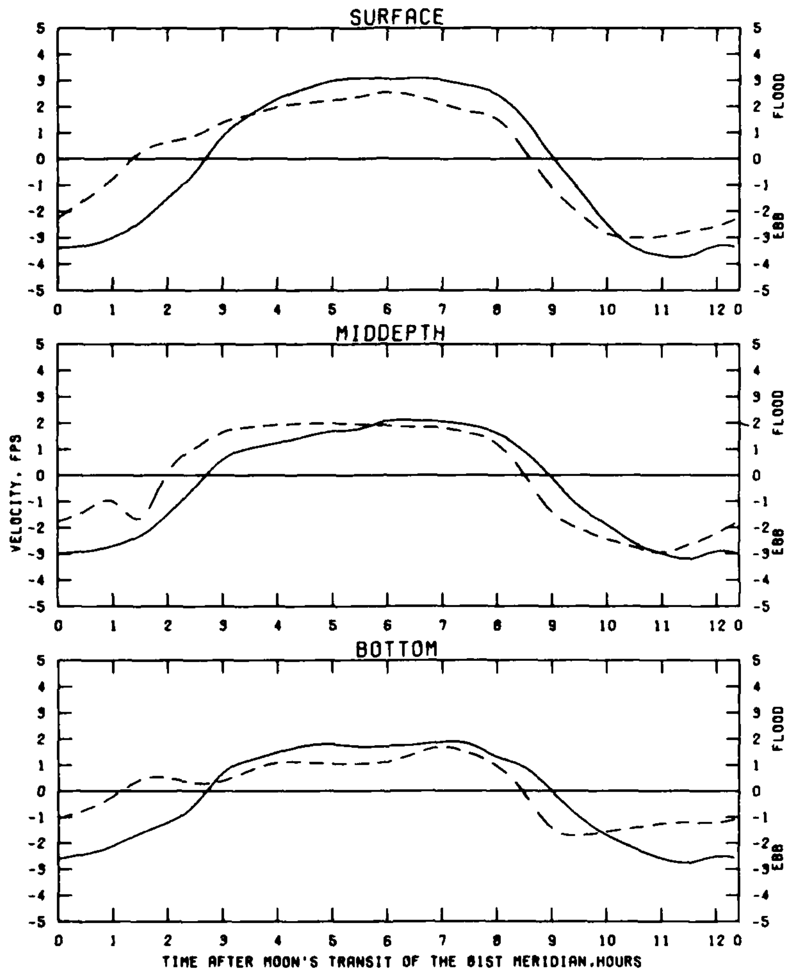
TIDE....................................MEAN
SOURCE SALINITY.......................30 PPT
FRESHWATER DISCHARGE........................0
PROTOTYPE DATA COLLECTED......MARCH 20, 1976

LEGEND

PROTOTYPE — — —
MODEL (PLAN 1) ———

POSTCONSTRUCTION VERIFICATION OF CURRENT VELOCITIES

STATION 5CR



TEST CONDITIONS

TIDE.....................................MEAN
SOURCE SALINITY........................30 PPT
FRESHWATER DISCHARGE.......................0
PROTOTYPE DATA COLLECTED......MARCH 20, 1976

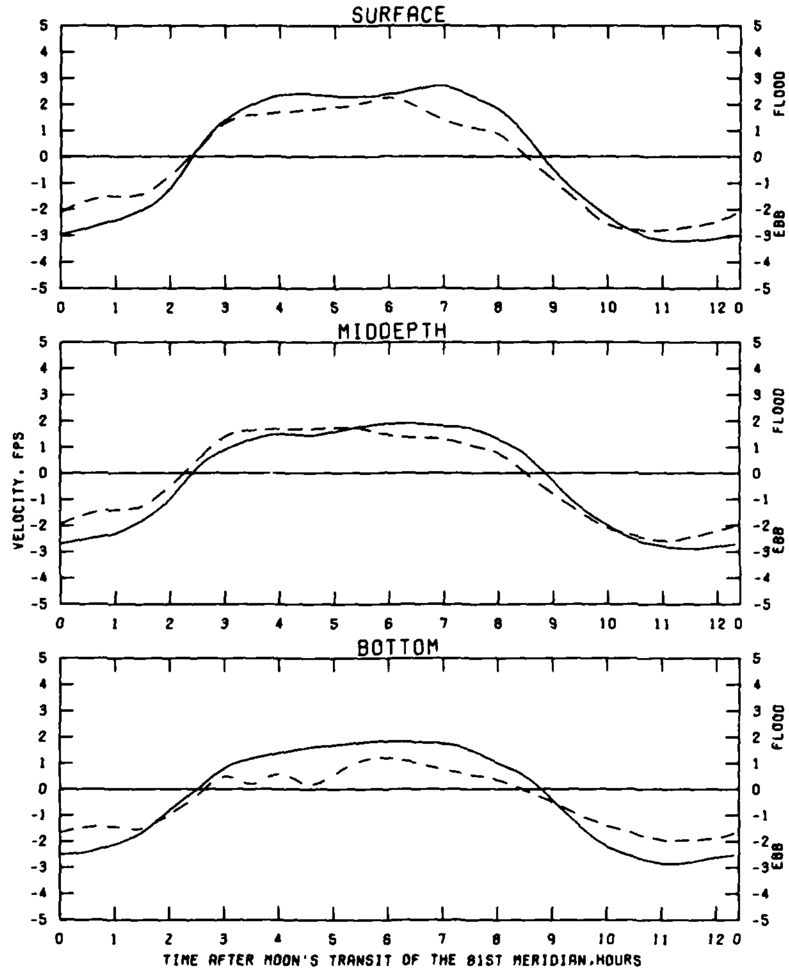
LEGEND

PROTOTYPE — — —
MODEL (PLAN 1) ———

POSTCONSTRUCTION
VERIFICATION OF
CURRENT VELOCITIES

STATION 5C

PLATE 16



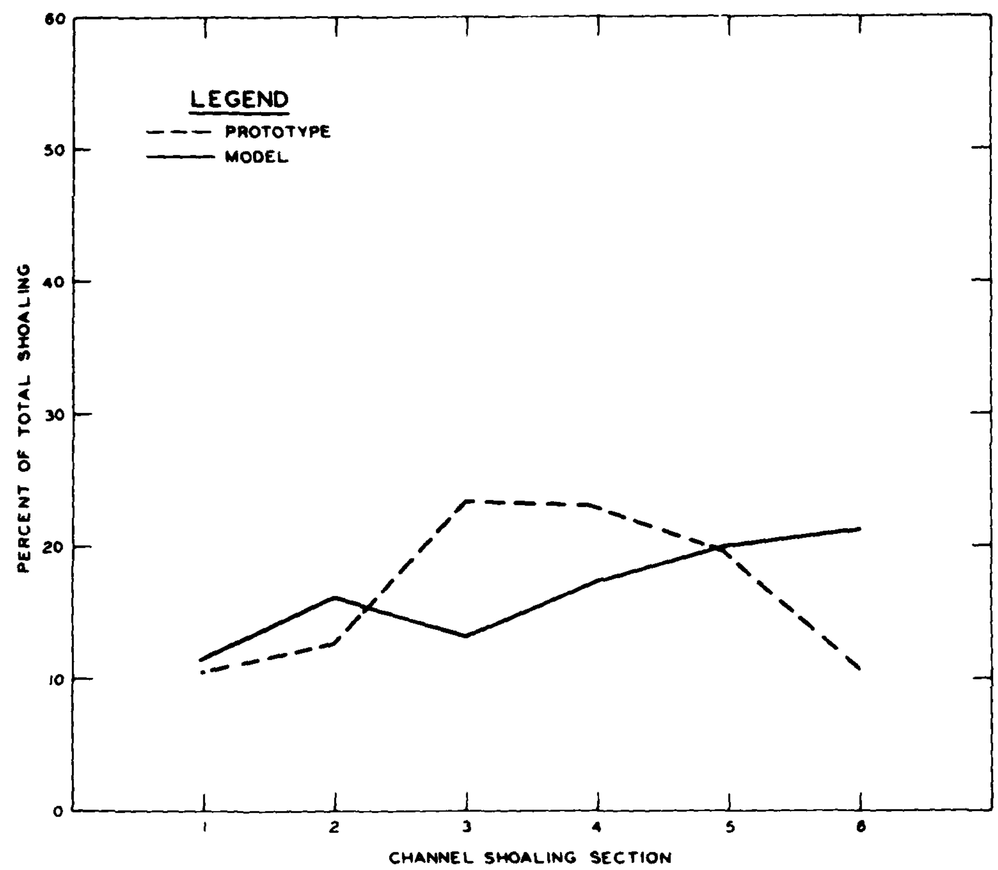
TEST CONDITIONS

TIDE....................................MEAN
SOURCE SALINITY........................30 PPT
FRESHWATER DISCHARGE........................0
PROTOTYPE DATA COLLECTED......MARCH 20, 1976

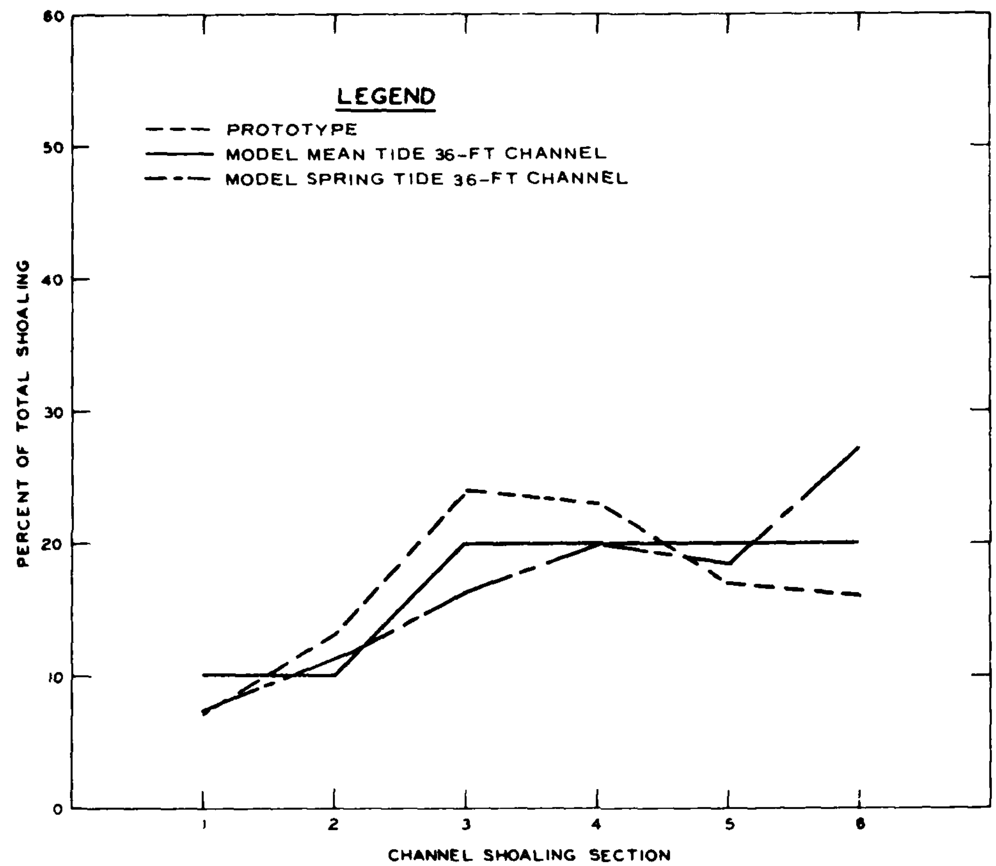
LEGEND
PROTOTYPE — — —
MODEL (PLAN 1) ———

POSTCONSTRUCTION
VERIFICATION OF
CURRENT VELOCITIES

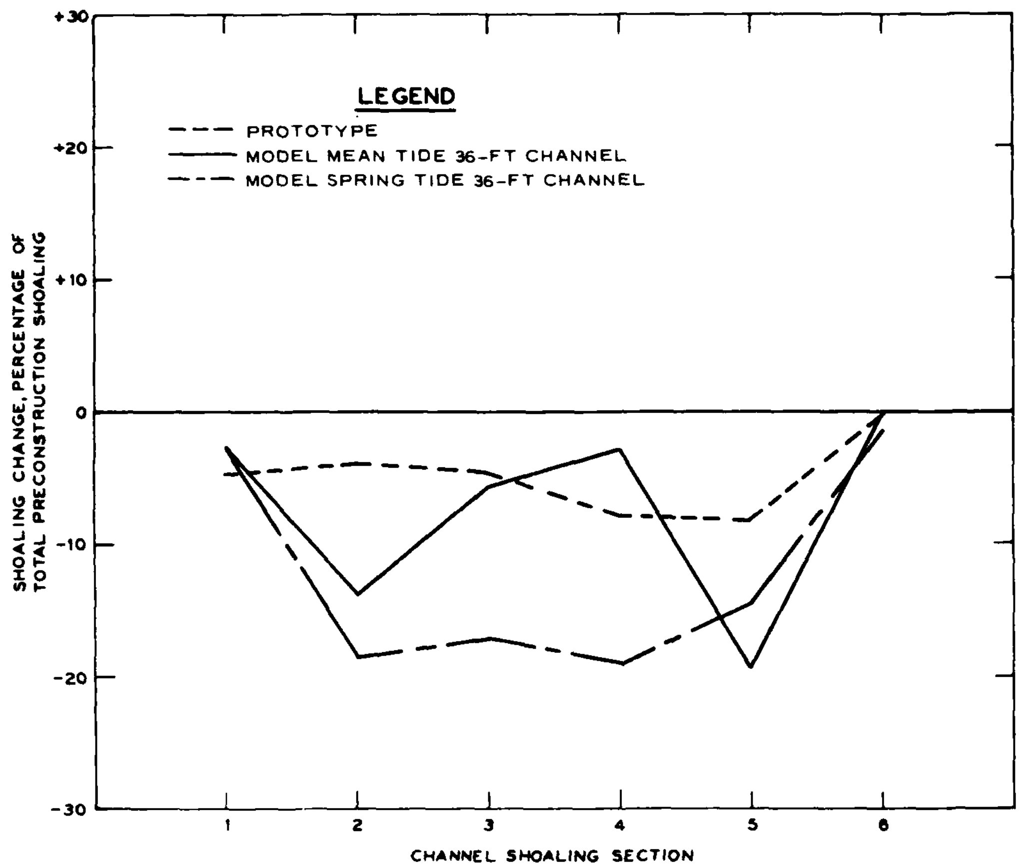
STATION 6CR



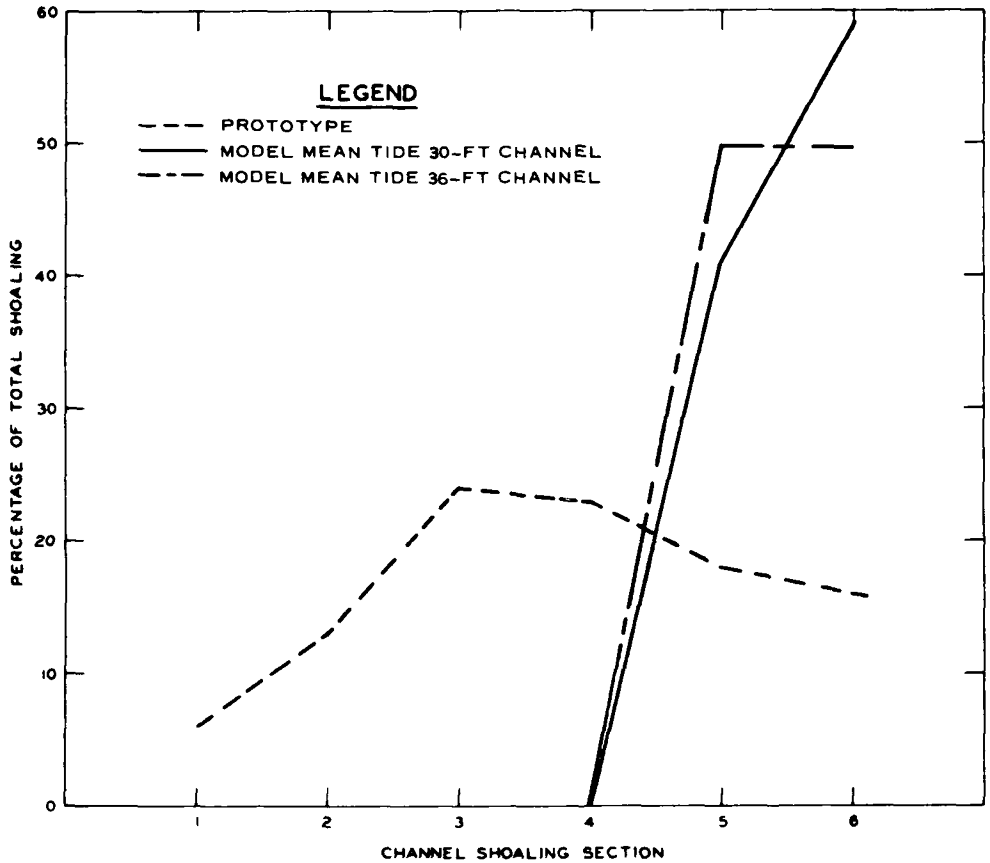
ORIGINAL SHOALING VERIFICATION



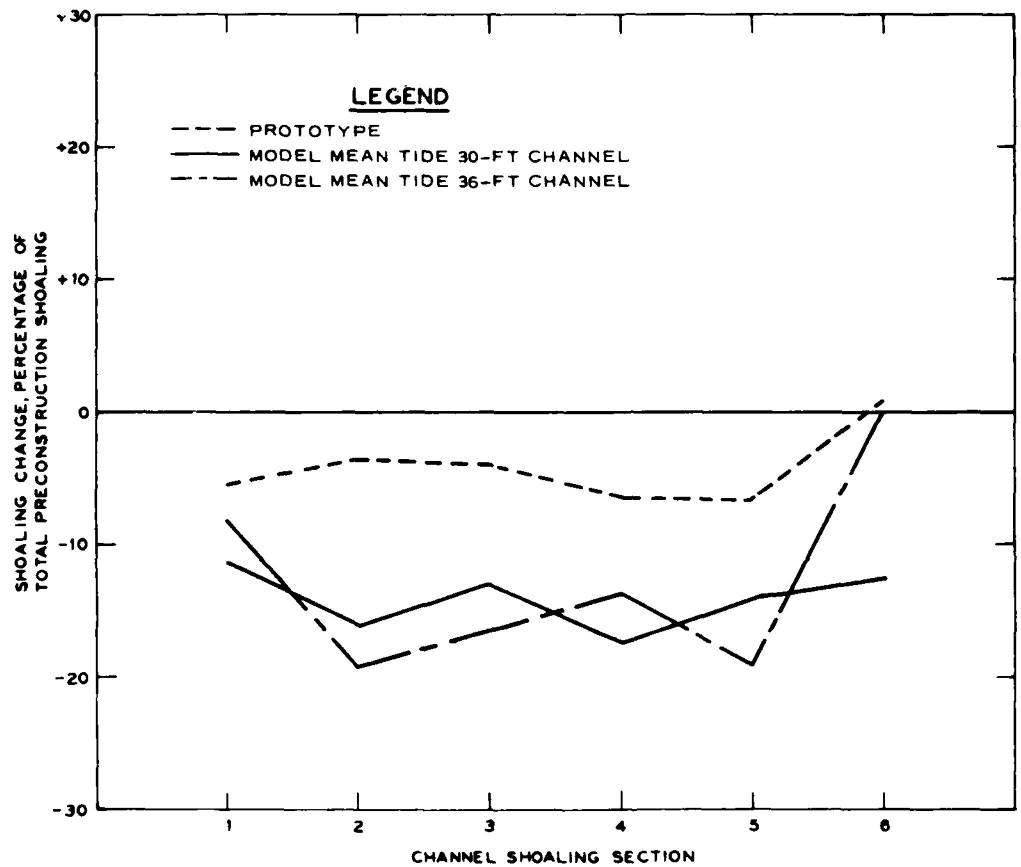
POSTCONSTRUCTION VERIFICATION
PARTIAL CLOSURE PLAN



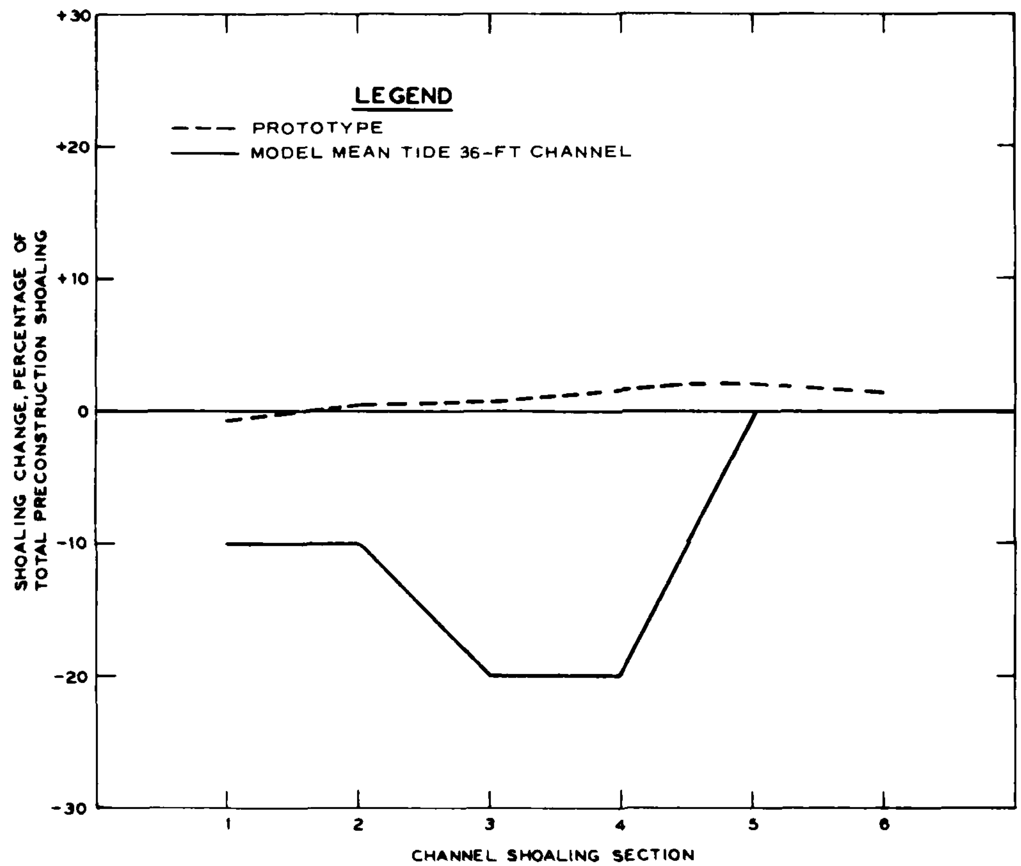
EFFECTS OF PARTIAL CLOSURE PLAN RELATIVE TO PRECONSTRUCTION



POSTCONSTRUCTION VERIFICATION
ALTERNATE CLOSURE PLAN



EFFECTS OF ALTERNATE CLOSURE PLAN
RELATIVE TO PRECONSTRUCTION



EFFECTS OF ALTERNATE CLOSURE PLAN
RELATIVE TO PARTIAL CLOSURE PLAN

APPENDIX A: SUPPLEMENTAL PROTOTYPE HYDRAULIC DATA

1. During the prototoype survey of Brunswick Harbor on 20 March 1976 several data collection stations were monitored that were not used for comparison with model measurements. This appendix presents that additional data. Details of the data collection procedures are given in PART II of the main report.

2. Figure A1 shows the locations of the single additional tide gage (sta HHD) and the six additional current velocity stations (2A, 2C, 4A, 6C, 6E).

3. Plate A1 presents the tidal elevations recorded at the highway department dock in Terry Creek (sta HDD), just downstream of the Lanier Bridge.

4. Plates A2 and A3 present the current velocities measured just outside the navigation channel lines (sta 2A and 2C) at the upper end of the 30-ft project in East River, about 100 ft downstream of Second Avenue.

5. Plates A4 and A5 present the measured current velocities at range 4 (sta 4A and 4C), outside the navigation channel in Turtle River upstream of the upper end of East River.

6. Range 5 current velocities outside the navigation channel (sta 6C and 6E) in Brunswick River, just downstream of the Lanier Bridge (Highway 17) are presented in Plates A6 and A7.

7. No detailed analysis has been performed for these stations and they are presented here for documentation and reference only.

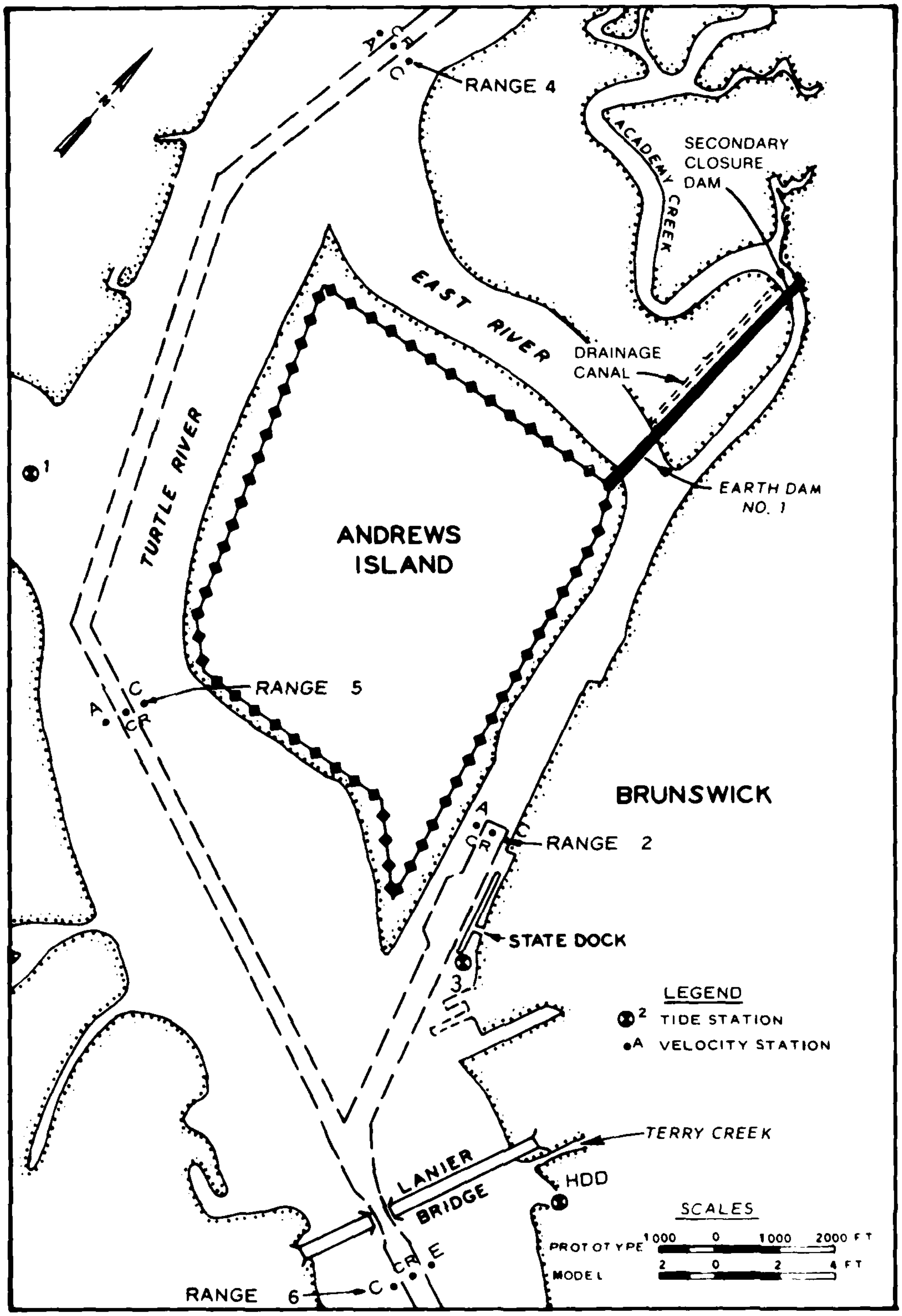
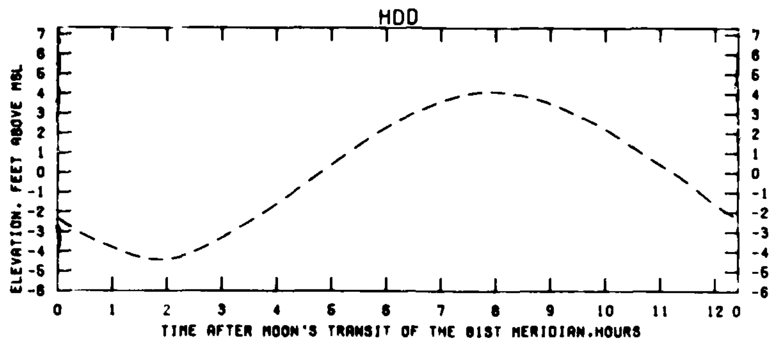


Figure A1. Additional alternate closure plan data stations



TEST CONDITIONS

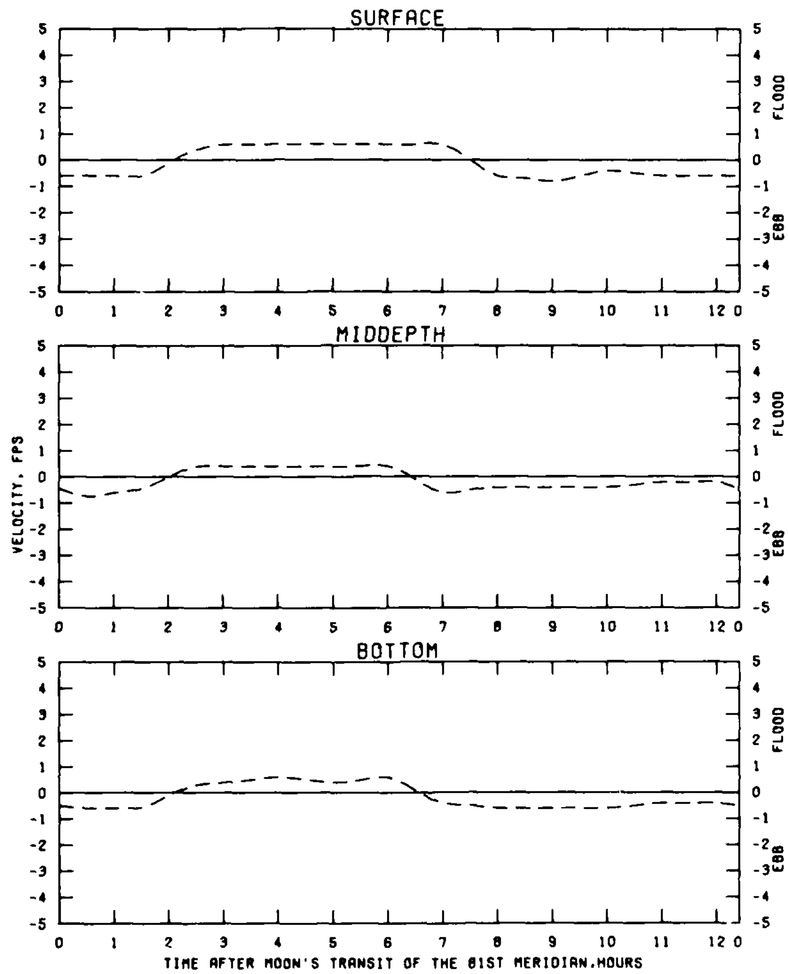
TIDE....................................MEAN
SOURCE SALINITY........................30 PPT
FRESHWATER DISCHARGE.........................0
PROTOTYPE DATA COLLECTED......MARCH 20, 1978

LEGEND

PROTOTYPE — — —

POSTCONSTRUCTION VERIFICATION OF TIDAL ELEVATIONS

STATION
HDD



TEST CONDITIONS

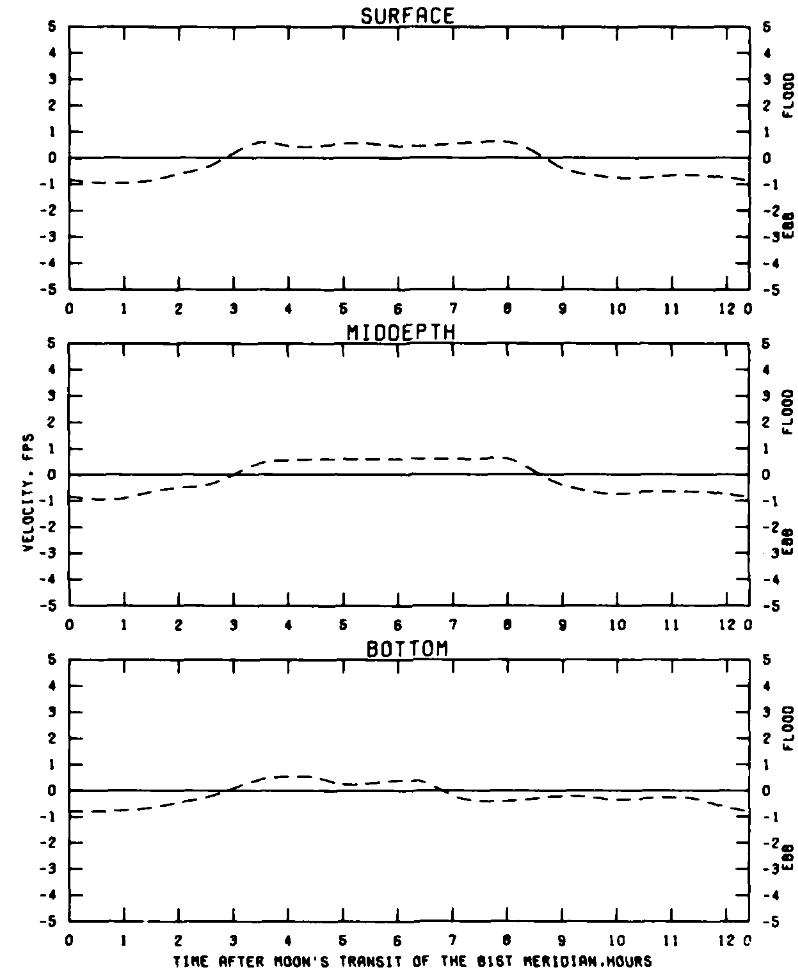
TIDE....................................MEAN
SOURCE SALINITY........................30 PPT
FRESHWATER DISCHARGE........................0
PROTOTYPE DATA COLLECTED......MARCH 20, 1976

LEGEND

PROTOTYPE — — —

POSTCONSTRUCTION
VERIFICATION OF
CURRENT VELOCITIES

STATION
2A



TEST CONDITIONS

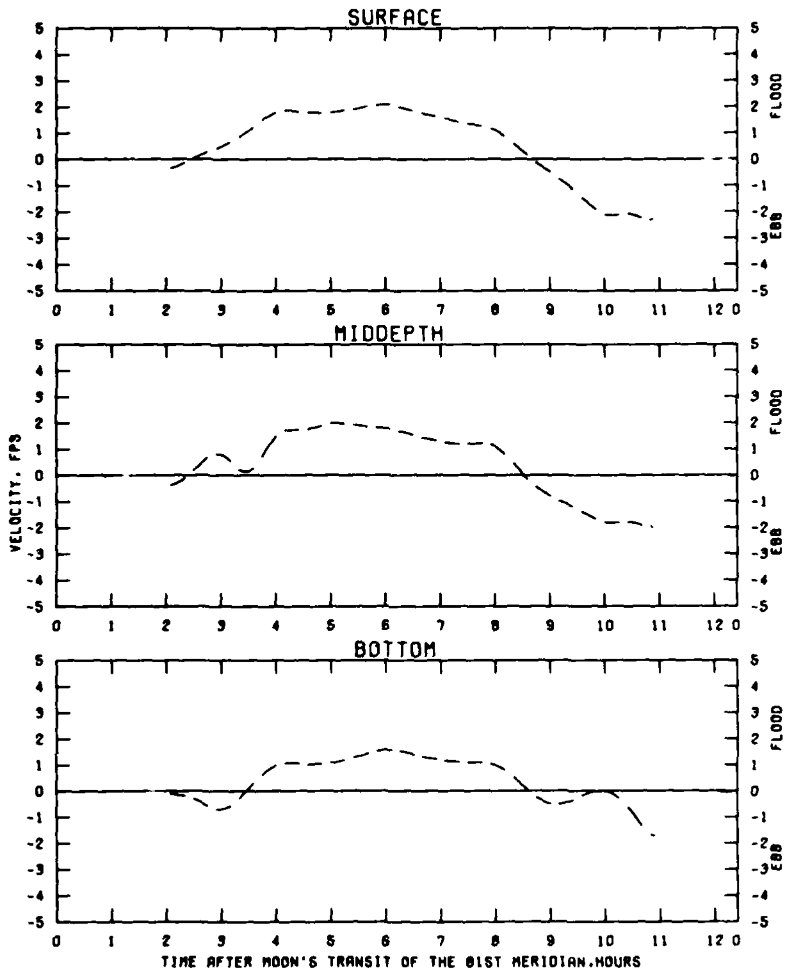
TIDE....................................MEAN
SOURCE SALINITY........................30 PPT
FRESHWATER DISCHARGE........................0
PROTOTYPE DATA COLLECTED......MARCH 20, 1976

LEGEND

PROTOTYPE — — —

POSTCONSTRUCTION
VERIFICATION OF
CURRENT VELOCITIES

STATION
2C



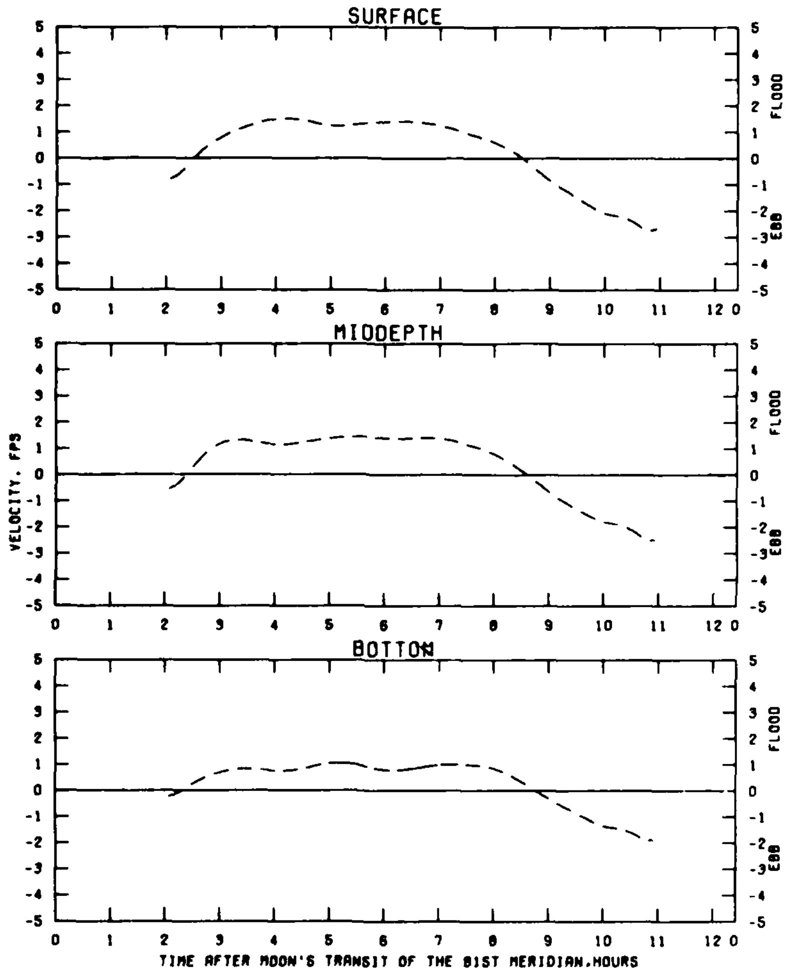
TEST CONDITIONS

TIDE....................................MEAN
SOURCE SALINITY........................30 PPT
FRESHWATER DISCHARGE........................0
PROTOTYPE DATA COLLECTED......MARCH 20, 1976

LEGEND
PROTOTYPE — — —

POSTCONSTRUCTION VERIFICATION OF CURRENT VELOCITIES

STATION 4A



TEST CONDITIONS

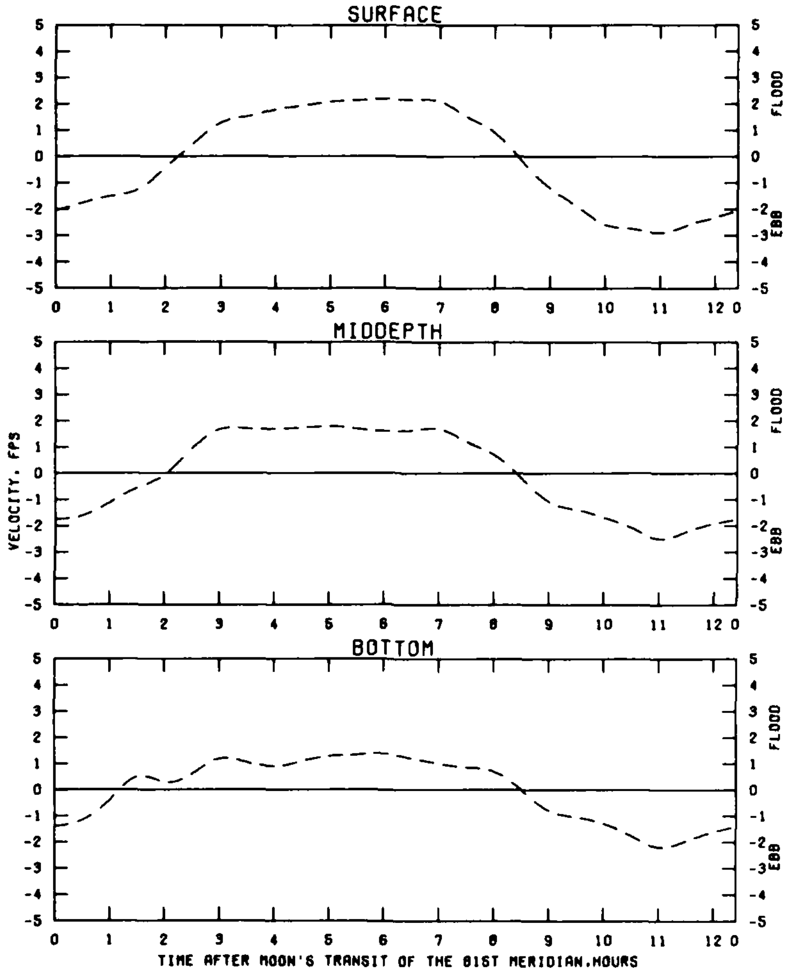
TIDE....................................MEAN
SOURCE SALINITY........................30 PPT
FRESHWATER DISCHARGE........................0
PROTOTYPE DATA COLLECTED......MARCH 20, 1976

LEGEND

PROTOTYPE — — —

POSTCONSTRUCTION VERIFICATION OF CURRENT VELOCITIES

STATION 4C



TEST CONDITIONS

TIDE....................................MEAN
SOURCE SALINITY.........................30 PPT
FRESHWATER DISCHARGE........................0
PROTOTYPE DATA COLLECTED......MARCH 20, 1976

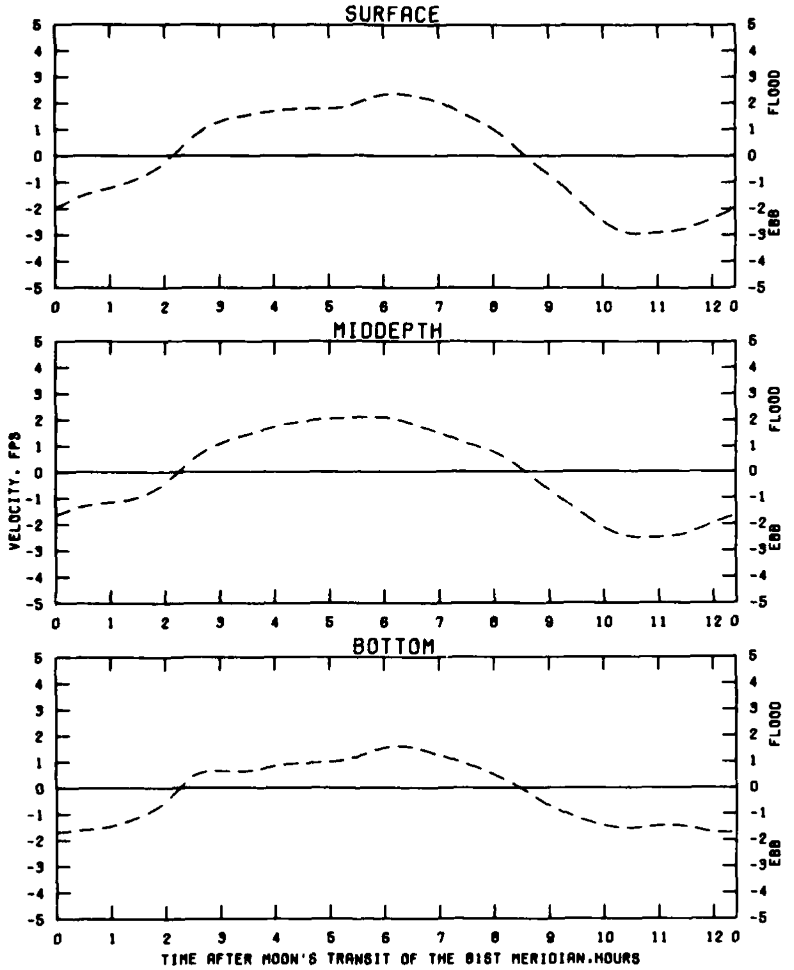
LEGEND

PROTOTYPE — — —

POSTCONSTRUCTION
VERIFICATION OF
CURRENT VELOCITIES

STATION
8C

PLATE A6



TEST CONDITIONS

TIDE....................................MEAN
SOURCE SALINITY........................30 PPT
FRESHWATER DISCHARGE........................0
PROTOTYPE DATA COLLECTED......MARCH 20, 1976

LEGEND
PROTOTYPE — — —

POSTCONSTRUCTION
VERIFICATION OF
CURRENT VELOCITIES

STATION
8E

APPENDIX B: PROTOTYPE SHOALING

Introduction

1. Comparison of model and prototype shoaling volumes is the climax of a long verification process. For most studies a great deal of effort is expended in a determination of reliable prototype shoaling volume and rates.

2. This appendix is provided as a documentation of the large volume of prototype data analysis performed in the determination of shoaling volumes for the East River portion of Brunswick Harbor, Georgia.

Prototype Surveys

3. The basic data used in this analysis were the periodic hydrographic surveys conducted by the Savannah District using a fathometer as recorder on sounding sheets. For a more detailed description of these sounding sheets, refer to paragraphs 46 and 135-136 of the main report.

4. The prototype hydrographic surveys used for the analysis were before-dredging (BD) and after-dredging (AD) surveys covering the period 4 September 1962 through 20 September 1976. Fifty-one surveys were used in the analysis.

Description of Analysis

5. The first step of the analysis of the prototype data was to digitize the hydrographic survey data into a computer-readable format. This digitization was performed manually by first transferring the depth reading to computer card coding sheets and then keypunching the data cards manually.

6. Locations of the depth readings were determined by means of a station location along the navigation channel and an offset distance from an arbitrary reference line paralleling the center line of the channel. These two values constituted coordinates of a Cartesian coordinate system.

7. The coordinates of the fathometer soundings from survey to survey generally were not coincident in space. Therefore, it was necessary to establish a consistent data grid from which to extract the volume change from survey to survey. This was accomplished by first establishing a desired data grid of locations within the area of interest (navigation channel) and then, for each survey, interpolating from the locations of measured data to those desired in the data grid. Thus, depth changes over the consistent grid could be computed between surveys.

8. The next step of the analysis was to compute the changes in cross-sectional areas along the navigation channel. These cross-sectional changes were computed in two methods. First, the positive area changes (deposition) were computed; that is, scour was not included in the integration. The net area changes were then computed by including both deposition and erosion within the integration. The integration technique used was trapezoidal.

9. Volume changes associated with one of the shoaling sections (in plan view) used in the model study were then computed by integrating the area changes along the navigation channel. Thus, for each shoaling section a net shoaling volume and a positive shoaling volume were computed for each period between surveys.

10. The numerical integrations for both area and volume were performed on the depth changes over the uniform data grid extracted from the random data coverage for each survey.

11. At this stage of the analysis the volumes computed were then converted to a rate of shoaling, based on the duration of the period between each survey. These rates were then averaged for all of the computation periods within each broader period (preconstruction, partial closure plan, and alternate closure plan). The averaging process was weighted by the duration of each computational period.

12. The end products of the analysis were average annual rates of shoaling (both net and positive) for each of the six shoaling sections used for the model study for the three construction conditions (preconstruction, partial closure plan, and alternate closure plan).

13. The approach to the shoaling determination was twofold.

First, the computation of the changes from each AD survey to the subsequent BD survey was made; then, the computations were made over each dredging period (BD to AD).

14. The definition of duration is straightforward for the AD to BD surveys, but not for the BD to AD. Since the dredging that occurred during that period was the result of deposition during the preceding AD to BD period as well, the duration associated with the BD to AD volume changes was the total period of time from the preceding AD survey to the AD survey used in the computations.

Results

15. Results of the shoaling analysis are presented in Tables B1-B9. Tables B1-B3 present the results of the analysis of the preconstruction shoaling rates. Positive shoaling rates computed from AD to BD surveys are presented in Table B1. Net shoaling rates computed from AD to BD surveys are presented in Table B2 and net shoaling rates computed from BD to AD surveys are presented in Table B3. Average rates for these tables are plotted graphically in Plates B1-B3.

16. Shoaling rates during the partial closure plan period (November 1969 to April 1975) are presented in Tables B4-B6. Positive shoaling volumes for period between AD and BD surveys are presented in Table B4. Net shoaling volumes computed for periods between AD and BD surveys are shown in Table B6. Average annual shoaling volumes are presented graphically in the corresponding Plates B4-B6.

17. For the alternate closure plan periods, shoaling volumes are similarly tabulated in Tables B7-B9 and Plates B7-B9.

18. For the preconstruction condition there were 15 periods both of AD to BD and BD to AD surveys. For the partial closure plan condition there were 8 periods each of AD to BD and BD to AD surveys. For the alternate closure plan condition there were 2 periods of AD to BD surveys and 1 period of BD to AD surveys analyzed.

19. Data used for the preconstruction verification of Brunswick Harbor were the net volumes as determined from AD to BD surveys, since no uncertainties due to dredging during the analysis periods are introduced.

Table B1

Positive Shoaling During Preconstruction Period Computed from After- to Before-Dredging Surveys

Period	Number of Days	Volume, cu yd, in Section (Distribution of Total)* 1	2	3	4	5	6	Total
4 Sep 62-13 Dec 62	100	8,400 (5%)	18,700 (12%)	35,100 (22%)	46,700 (30%)	31,700 (20%)	17,600 (11%)	158,200 (100%)
30 Dec 62-20 May 63	141	25,900 (12%)	21,500 (10%)	40,300 (18%)	56,500 (26%)	42,700 (20%)	31,700 (14%)	218,600 (100%)
11 Jun 63-9 Oct 63	120	26,400 (8%)	40,400 (12%)	70,600 (22%)	73,600 (22%)	76,500 (23%)	43,400 (13%)	330,900 (100%)
12 Nov 63-31 Mar 64	140	21,900 (9%)	31,000 (13%)	52,700 (22%)	52,600 (22%)	48,600 (20%)	32,500 (14%)	239,300 (100%)
24 Apr 64-20 Aug 64	118	44,300 (12%)	53,500 (15%)	83,200 (23%)	83,800 (24%)	61,300 (17%)	30,200 (9%)	356,300 (100%)
23 Sep 64-8 Jan 65	107	33,000 (10%)	46,400 (15%)	69,200 (22%)	77,100 (24%)	56,900 (18%)	33,600 (11%)	316,200 (100%)
4 Feb 65-21 May 65	110	29,100 (10%)	34,400 (12%)	62,900 (22%)	70,500 (24%)	59,900 (20%)	34,400 (12%)	291,200 (100%)
29 Jun 65-22 Sep 65	85	42,500 (16%)	42,800 (17%)	56,600 (22%)	55,600 (22%)	42,000 (16%)	19,000 (7%)	258,500 (100%)
1 Nov 65-29 Mar 66	148	19,000 (8%)	28,300 (13%)	44,300 (20%)	52,800 (24%)	45,800 (21%)	30,300 (14%)	220,500 (100%)
21 Apr 66-1 Aug 66	102	22,600 (8%)	36,600 (14%)	61,300 (23%)	66,400 (24%)	55,200 (20%)	30,100 (11%)	272,200 (100%)

(Continued)

* First value is volume (cu yd) in each section; the second value (in parentheses) is the percent distribution of total.

Table B1 (Concluded)

Period	Number of Days	Volume, cu yd, in Section (Distribution of Total) 1	2	3	4	5	6	Total
1 Sep 66-14 Dec 66	104	38,500 (13%)	57,500 (19%)	77,100 (25%)	69,300 (22%)	47,200 (15%)	17,600 (6%)	307,200 (100%)
6 Feb 67-8 Jun 67	122	20,800 (7%)	30,100 (11%)	58,100 (21%)	69,300 (25%)	64,700 (23%)	35,400 (13%)	278,400 (100%)
17 Jul 67-30 Oct 67	105	28,800 (9%)	40,300 (14%)	63,800 (22%)	70,300 (24%)	58,300 (20%)	32,700 (11%)	294,200 (100%)
7 Dec 67-12 Aug 68	249	31,100 (9%)	44,500 (13%)	64,200 (19%)	81,200 (24%)	75,100 (23%)	40,600 (12%)	336,700 (100%)
4 Sep 68-10 Mar 69	187	18,100 (7%)	28,100 (10%)	54,300 (20%)	68,700 (26%)	62,000 (23%)	37,400 (14%)	268,600 (100%)
Total	1938	410,400	554,100	893,700	994,400	827,900	466,500	4,147,000
Average annual shoaling rate (cu yd)		77,300	104,400	168,300	187,300	155,900	87,900	781,100
Distribution of total shoaling		(10%)	(13%)	(22%)	(24%)	(20%)	(11%)	(100%)

Table B2

Net Shoaling During Preconstruction Period Computed from After- to Before-Dredging Surveys

Period	Number of Days	Volume, cu yd, in Section (Distribution of Total)* 1	2	3	4	5	6	Total
4 Sep 62-13 Dec 62	100	5,900 (4%)	17,100 (11%)	34,500 (23%)	46,100 (31%)	30,800 (21%)	15,600 (10%)	150,000 (100%)
30 Dec 62-20 May 63	141	25,500 (12%)	19,300 (9%)	39,500 (18%)	55,800 (26%)	42,300 (20%)	31,300 (15%)	213,700 (100%)
11 Jun 63-9 Oct 63	120	26,000 (8%)	38,600 (12%)	70,400 (22%)	72,300 (22%)	76,400 (23%)	42,000 (13%)	325,700 (100%)
12 Nov 63-31 Mar 64	140	21,800 (10%)	29,700 (13%)	51,100 (22%)	49,700 (22%)	46,100 (20%)	30,100 (13%)	228,400 (100%)
24 Apr 64-20 Aug 64	118	43,900 (13%)	53,500 (15%)	83,200 (24%)	83,800 (24%)	60,800 (17%)	25,200 (7%)	350,400 (100%)
23 Sep 64-8 Jan 65	107	32,700 (11%)	45,100 (15%)	67,400 (22%)	73,900 (24%)	56,200 (18%)	31,800 (10%)	307,100 (100%)
4 Feb 65-21 May 65	110	29,100 (10%)	32,500 (11%)	62,300 (22%)	70,000 (25%)	59,500 (21%)	32,900 (11%)	286,300 (100%)
29 Jun 65-22 Sep 65	85	42,000 (17%)	42,000 (17%)	56,400 (22%)	54,000 (22%)	41,400 (16%)	15,700 (6%)	251,500 (100%)
1 Nov 65-29 Mar 66	148	19,000 (9%)	28,300 (13%)	42,200 (19%)	51,800 (24%)	45,500 (21%)	30,000 (14%)	216,800 (100%)
21 Apr 66-1 Aug 66	102	22,000 (9%)	32,500 (13%)	58,500 (23%)	63,500 (24%)	51,900 (20%)	28,900 (11%)	257,300 (100%)

(Continued)

* First value is volume (cu yd) in each section; the second value (in parentheses) is the percent distribution of total.

Table B2 (Concluded)

Period	Number of Days	Volume, cu yd, in Section (Distribution of Total)						Total
		1	2	3	4	5	6	
21 Apr 66–1 Aug 66	102	22,000 (9%)	32,500 (13%)	58,500 (23%)	63,500 (24%)	51,900 (20%)	28,900 (11%)	257,300 (100%)
1 Sep 66–14 Dec 66	104	38,500 (13%)	57,500 (19%)	77,100 (25%)	68,700 (23%)	46,700 (15%)	14,000 (5%)	302,500 (100%)
6 Feb 67–8 Jun 67	122	17,600 (6%)	28,900 (11%)	57,800 (21%)	68,000 (25%)	64,500 (24%)	34,600 (13%)	271,400 (100%)
17 Jul 67–30 Oct 67	105	28,800 (10%)	40,100 (14%)	63,800 (22%)	69,900 (24%)	56,400 (20%)	28,300 (10%)	287,400 (100%)
7 Dec 67–12 Aug 68	249	30,000 (9%)	39,300 (12%)	62,400 (20%)	77,900 (24%)	73,400 (23%)	37,100 (12%)	320,100 (100%)
4 Sep 68–10 Mar 69	187	18,000 (7%)	27,900 (11%)	53,000 (20%)	67,500 (26%)	60,900 (23%)	35,400 (13%)	262,700 (100%)
Total	1938	400,800	532,300	879,600	972,900	812,800	432,800	4,031,200
Average annual shoaling rate (cu yd)		75,500	100,200	165,600	183,300	153,100	81,500	759,200
Distribution of total shoaling		(10%)	(13%)	(22%)	(24%)	(20%)	(11%)	(100%)

Table B3

Net Shoaling During Preconstruction Period Computed from Before- to After-Dredging Surveys

Dredging Period	Number of Days: Since Prior Dredging	Number of Days: Dredging Period	Number of Days: Total Period	Volume, cu yd, in Section (Distribution of Total)*: 1	2	3	4	5	6	Total
13 Dec 62-30 Dec 62	100	17	117	-19,900 (8%)	-27,000 (11%)	-47,400 (19%)	-65,200 (27%)	-52,200 (22%)	-30,900 (13%)	-242,600 (100%)
20 May 63-11 Jun 63	141	22	163	-28,000 (14%)	-23,900 (12%)	-33,900 (17%)	-47,600 (24%)	-39,100 (20%)	-25,900 (13%)	-198,400 (100%)
9 Oct 63-12 Nov 63	120	34	154	-32,000 (9%)	-43,900 (13%)	-76,900 (23%)	-76,900 (23%)	-71,600 (21%)	-36,000 (11%)	-337,300 (100%)
31 Mar 64-24 Apr 64	140	24	164	-22,800 (8%)	-33,100 (12%)	-53,600 (20%)	-61,900 (23%)	-62,400 (23%)	-38,900 (14%)	-272,700 (100%)
20 Aug 64-23 Sep 64	118	34	152	-33,000 (11%)	-42,300 (14%)	-77,700 (27%)	-68,800 (24%)	-49,700 (17%)	-20,400 (7%)	-291,900 (100%)
8 Jan 65-4 Feb 65	107	26	133	-45,400 (13%)	-57,000 (16%)	-75,200 (21%)	-83,400 (24%)	-60,000 (17%)	-32,300 (9%)	-353,300 (100%)
21 May 65-29 Jun 65	110	39	149	-21,400 (10%)	-21,000 (9%)	-42,600 (19%)	-53,500 (23%)	-53,900 (24%)	-33,400 (15%)	-255,800 (100%)
22 Sep 65-1 Nov 65	85	40	125	-43,600 (16%)	-48,600 (18%)	-66,400 (24%)	-61,500 (22%)	-39,800 (14%)	-15,200 (6%)	-275,100 (100%)
29 Mar 66-21 Apr 66	148	23	171	-900 (0%)	-19,500 (10%)	-45,300 (24%)	-52,500 (28%)	-46,800 (25%)	-24,600 (13%)	-189,600 (100%)
1 Aug 66-1 Sep 66	102	31	133	-43,100 (13%)	-47,700 (15%)	-73,900 (23%)	-70,200 (22%)	-57,700 (18%)	-29,800 (9%)	-322,400 (100%)
14 Dec 66-6 Feb 67	104	54	158	-31,100 (11%)	-45,000 (17%)	-65,200 (24%)	-63,600 (23%)	-44,900 (17%)	-21,000 (8%)	-270,800 (100%)
8 Jun 67-17 Jul 67	122	39	161	-27,000 (10%)	-36,800 (13%)	-54,900 (20%)	-66,700 (24%)	-61,400 (22%)	-32,100 (11%)	-279,000 (100%)

(Continued)

* First value is volume (cu yd) in each section; the second value (in parentheses) is percent distribution of total.

Table B3 (Concluded)

Dredging Period	Number of Days: Since Prior Dredging	Number of Days: Dredging Period	Number of Days: Total Period	Volume, cu yd, in Section (Distribution of Total): 1	2	3	4	5	6	Total
30 Oct 67-7 Dec 67	105	38	143	-31,200 (10%)	-36,800 (12%)	-67,800 (23%)	-80,100 (27%)	-56,500 (19%)	-25,800 (9%)	-298,200 (100%)
12 Aug 68-4 Sep 68	249	23	272	-29,500 (9%)	-45,600 (13%)	-60,600 (18%)	-81,400 (23%)	-84,400 (24%)	-45,500 (13%)	-347,000 (100%)
10 Mar 69-11 Apr 69	187	32	219	-10,700 (4%)	-25,400 (10%)	-53,500 (22%)	-63,900 (26%)	-56,500 (23%)	-35,900 (15%)	-245,900 (100%)
Total	1938	476	2414	-419,700	-553,500	-894,900	-997,300	-836,700	-447,700	-4,149,800
Average annual shoaling rate (cu yd)				-63,500	-83,700	-135,300	-150,800	-126,500	-67,700	-627,500
Distribution of total shoaling				(10%)	(13%)	(22%)	(24%)	(20%)	(11%)	(100%)

Table B4

Positive Shoaling During Partial Closure Period Computed from After- to Before-Dredging Surveys

Period	Number of Days	Volume, cu yd, in Section (Distribution of Total)*						Total
		1	2	3	4	5	6	
10 Nov 69-30 Jun 70	232	23,200 (8%)	39,300 (13%)	64,300 (22%)	71,600 (24%)	51,000 (17%)	46,900 (16%)	296,300 (100%)
29 Jul 70-19 May 71	294	22,800 (7%)	40,400 (13%)	74,800 (23%)	70,700 (22%)	54,400 (17%)	55,900 (18%)	319,000 (100%)
8 Jun 71-31 Jan 72	237	30,500 (7%)	57,700 (14%)	94,200 (23%)	90,100 (22%)	73,400 (18%)	63,700 (16%)	409,600 (100%)
7 Mar 72-31 Oct 72	238	23,000 (6%)	45,800 (11%)	94,600 (24%)	101,300 (25%)	71,200 (18%)	65,900 (16%)	401,800 (100%)
1 Dec 72-29 Jun 73	211	28,300 (7%)	58,500 (15%)	92,900 (24%)	79,700 (21%)	67,500 (18%)	55,300 (15%)	382,200 (100%)
2 Aug 73-26 Nov 73	116	16,000 (8%)	27,000 (13%)	66,700 (31%)	49,400 (24%)	22,100 (11%)	26,800 (13%)	203,000 (100%)
15 Dec 73-24 May 74	160	16,600 (7%)	30,600 (12%)	55,700 (23%)	52,500 (22%)	43,000 (18%)	44,400 (18%)	242,800 (100%)
24 Jun 74-21 Apr 75	300	30,300 (7%)	64,000 (15%)	110,700 (25%)	104,800 (24%)	67,600 (15%)	62,900 (14%)	440,400 (100%)
Total	1788	190,700	363,300	648,900	620,100	450,200	421,900	2,695,100
Average annual shoaling rate (cu yd)		38,900	74,200	132,500	126,600	91,900	86,100	550,200
Distribution of total shoaling		(7%)	(13%)	(24%)	(23%)	(17%)	(16%)	(100%)

* First value is volume (cu yd) in each section; the second value (in parentheses) is the percent distribution of total.

Table B5

Net Shoaling During Partial Closure Period Computed from After- to Before-Dredging Surveys

Period	Number of Days	Volume, cu yd, in Section (Distribution of Total)* 1	2	3	4	5	6	Total
10 Nov 69-30 Jun 70	232	21,300 (7%)	37,900 (13%)	64,300 (22%)	71,500 (25%)	49,500 (17%)	46,900 (16%)	291,400 (100%)
29 Jul 79-19 May 71	294	22,200 (7%)	39,500 (13%)	72,000 (24%)	67,400 (22%)	50,600 (17%)	51,600 (17%)	303,300 (100%)
8 Jun 71-31 Jan 72	237	30,500 (8%)	57,700 (14%)	94,200 (23%)	90,100 (22%)	72,600 (18%)	61,600 (15%)	406,700 (100%)
7 Mar 72-31 Oct 72	238	22,200 (6%)	45,600 (12%)	93,900 (24%)	101,300 (25%)	70,500 (18%)	61,900 (15%)	395,400 (100%)
1 Dec 72-29 Jun 73	211	28,200 (7%)	58,500 (15%)	92,900 (25%)	79,700 (21%)	67,300 (18%)	54,000 (14%)	380,600 (100%)
2 Aug 73-26 Nov 73	116	15,500 (9%)	26,100 (15%)	59,400 (35%)	35,100 (20%)	9,700 (6%)	26,600 (15%)	172,400 (100%)
15 Dec 73-24 May 74	160	8,400 (4%)	27,800 (12%)	54,800 (24%)	51,500 (23%)	40,600 (18%)	44,300 (19%)	227,400 (100%)
24 Jun 74-21 Apr 75	300	30,000 (7%)	63,700 (15%)	109,700 25%)	103,500 (24%)	63,800 (15%)	61,200 (14%)	431,900 (100%)
Total	1788	178,300	356,800	641,200	600,000	424,600	408,100	2,609,000
Average annual shoaling rate (cu yd)		36,400	72,800	130,900	122,500	86,700	83,300	532,600
Distribution of total shoaling		(7%)	(14%)	(24%)	(23%)	(16%)	(16%)	(100%)

* First value is volume (cu yd) in each section; the second value (in parentheses) is the percent distribution of total.

Table B6

Net Shoaling During Partial Closure Period Computed from Before- to After-Dredging Surveys

Dredging Period	Number of Days: Since Prior Dredging	Number of Days: Dredging Period	Number of Days: Total Period	Volume, cu yd, in Section (Distribution of Total)*: 1	2	3	4	5	6	Total
30 Jun 70-29 Jul 70	232	29	261	-15,800 (6%)	-36,400 (13%)	-67,300 (24%)	-65,100 (24%)	-46,200 (17%)	-45,600 (16%)	-276,400 (100%)
19 May 71-9 Jun 71	294	20	314	-25,800 (7%)	-42,200 (11%)	-85,300 (22%)	-86,400 (23%)	-70,600 (19%)	-65,800 (18%)	-376,100 (100%)
31 Jan 72-7 Mar 72	237	36	273	-30,600 (7%)	-57,600 (14%)	-99,500 (24%)	-95,900 (23%)	-69,100 (17%)	-59,300 (15%)	-412,000 (100%)
31 Oct 72-1 Dec 72	238	31	269	-30,900 (7%)	-59,500 (14%)	-89,900 (21%)	-88,100 (21%)	-85,000 (20%)	-71,100 (17%)	-424,500 (100%)
29 Jun 73-2 Aug 73	211	34	245	-22,000 (6%)	-48,300 (14%)	-88,400 (26%)	-73,500 (21%)	-57,800 (17%)	-55,600 (16%)	-345,600 (100%)
26 Nov 73-15 Dec 73	116	19	135	-8,600 (3%)	-35,100 (14%)	-80,300 (33%)	-67,800 (28%)	-21,000 (9%)	-33,100 (13%)	-245,900 (100%)
24 May 74-24 Jun 74	160	31	191	-17,100 (7%)	-28,400 (12%)	-57,200 (24%)	-54,500 (23%)	-36,100 (15%)	-44,700 (19%)	-238,000 (100%)
21 Apr 75-11 Jul 75	300	51	351	-31,300 (8%)	-62,600 (16%)	-94,900 (24%)	-93,800 (24%)	-65,100 (17%)	-43,200 (11%)	-390,900 (100%)
Total	1788	251	2039	-182,100	-370,100	-662,800	-625,100	-450,900	-418,400	-2,709,400
Average annual shoaling rate (cu yd)				-32,600	-66,300	-118,600	-111,900	-80,700	-74,900	-485,000
Distribution of total shoaling				(7%)	(14%)	(24%)	(23%)	(17%)	(15%)	(100%)

* First value is volume (cu yd) in each section; the second value (in parentheses) is the percent distribution of total.

Table B7

Positive Shoaling During Alternate Closure Period Computed from After- to Before-Dredging Surveys

Period	Number of Days	Volume, cu yd, in Section (Distribution of Total)*						Total
		1	2	3	4	5	6	
11 Jul 75-7 Jan 76	210	24,000 (7%)	52,500 (15%)	78,000 (22%)	87,000 (25%)	61,600 (17%)	51,100 (14%)	354,200 (100%)
11 Feb 76-20 Sep 76	223	16,900 (5%)	38,100 (11%)	84,700 (26%)	74,500 (22%)	61,000 (18%)	60,100 (18%)	335,300 (100%)
Total	433	40,900	90,600	162,600	161,500	122,600	111,200	689,500
Average annual shoaling rate (cu yd)		34,500	76,400	137,100	136,200	103,300	93,700	581,200
Distribution of total shoaling		(6%)	(13%)	(24%)	(23%)	(18%)	(16%)	(100%)

* First value is volume (cu yd) in each section; the second value (in parentheses) is the percent distribution of total.

Table B8

Net Shoaling During Alternate Closure Period Computed from After- to Before-Dredging Surveys

Period	Number of Days	Volume, cu yd, in Section (Distribution of Total)* 1	2	3	4	5	6	Total
11 Jul 75-7 Jan 76	210	24,000 (7%)	52,500 (15%)	77,900 (22%)	87,000 (25%)	61,600 (17%)	48,000 (14%)	351,000 (100%)
11 Feb 76-20 Sep 76	223	16,500 (5%)	36,700 (11%)	84,700 (26%)	71,500 (22%)	59,700 (18%)	58,000 (18%)	327,100 (100%)
Total	433	40,500	89,200	162,600	158,500	121,300	106,000	678,100
Average annual shoaling rate (cu yd)		34,100	75,200	137,100	133,600	102,300	89,400	571,700
Distribution of total shoaling		(6%)	(13%)	(24%)	(23%)	(18%)	(16%)	(100%)

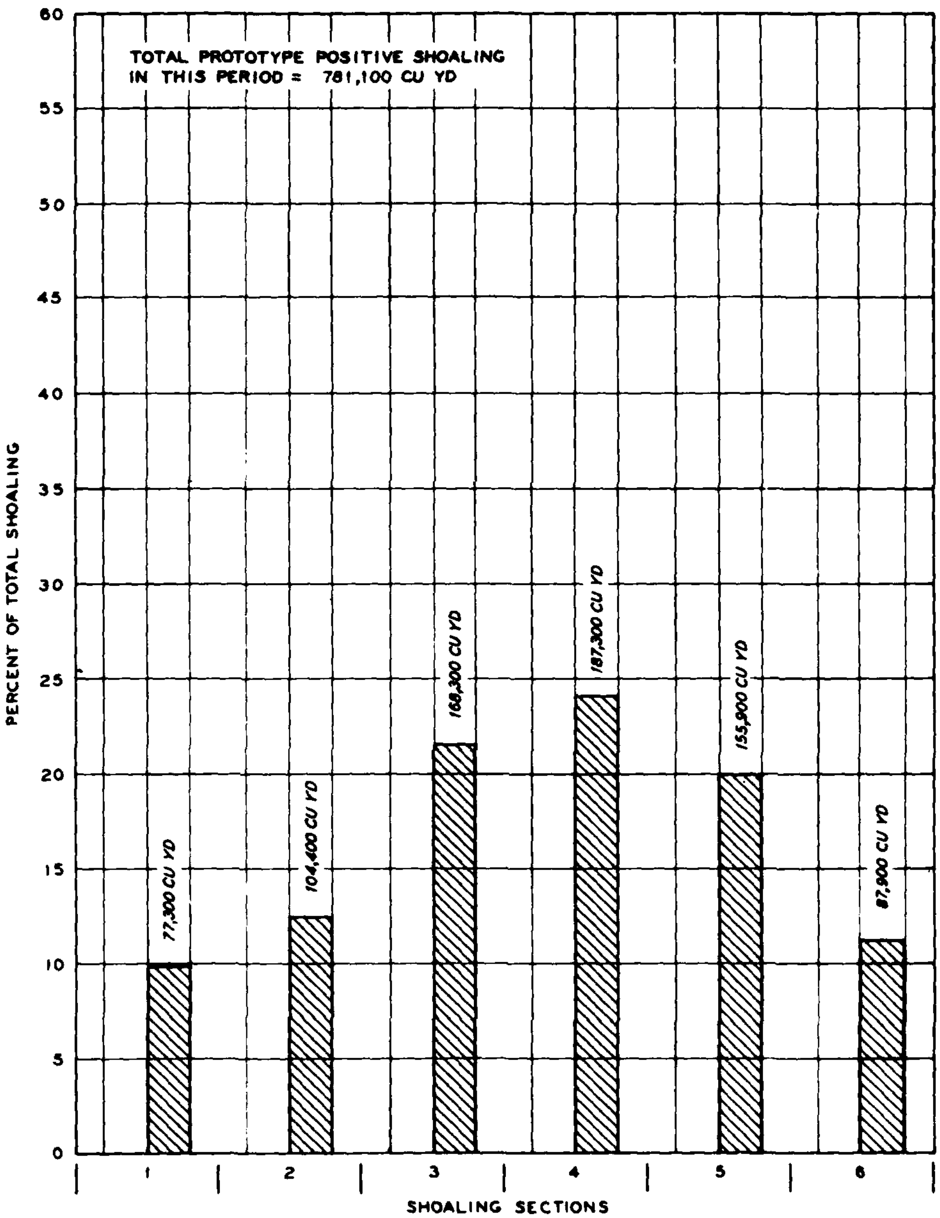
* First value is volume (cu yd) in each section; the second value (in parentheses) is the percent distribution of total.

Table B9

<u>Net Shoaling During Alternate Closure Period Computed from Before- to After-Dredging Surveys</u>

Dredging Period	Number of Days: Since Prior Dredging	Number of Days: Dredging Period	Number of Days: Total Period	Volume, cu yd, in Section (Distribution of Total)*: 1	2	3	4	5	6	Total
7 Jan 76-11 Feb 76	210	35	245	-21,500 (7%)	-47,100 (14%)	-79,600 (24%)	-69,500 (21%)	-57,600 (18%)	-52,300 (16%)	-327,600 (100%)
Average annual shoaling rate (cu yd)				32,100	70,100	118,600	103,600	85,700	78,000	488,100
Distribution of total shoaling				(7%)	(14%)	(24%)	(21%)	(18%)	(16%)	(100%)

* First value is volume (cu yd) in each section; the second value (in parentheses) is the percent distribution of total.



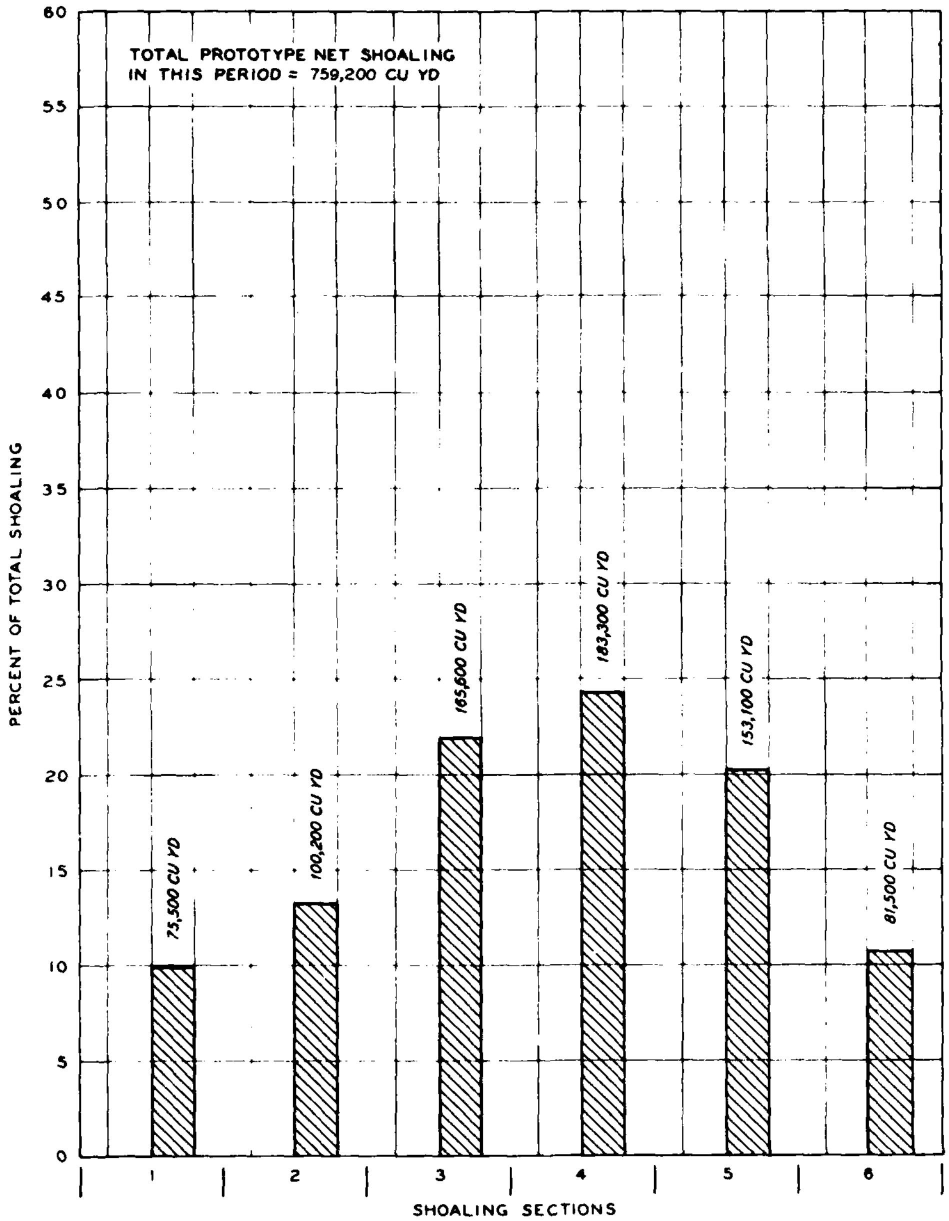
SECTION

1 STA 70+50 – 75+00
2 STA 75+00 – 81+00
3 STA 81+00 – 87+00
4 STA 87+00 – 93+00
5 STA 93+00 – 99+00
6 STA 99+00 – 105+00

LEGEND

MODEL
PROTOTYPE

AVERAGE ANNUAL POSITIVE SHOALING DISTRIBUTION
EAST RIVER-PORT
PRECONSTRUCTION
BASED ON HYDROGRAPHIC CHANGES AFTER- TO BEFORE-DREDGING



SECTION

1 STA 70+50 – 75+00
2 STA 75+00 – 81+00
3 STA 81+00 – 87+00
4 STA 87+00 – 93+00
5 STA 93+00 – 99+00
6 STA 99+00 – 105+00

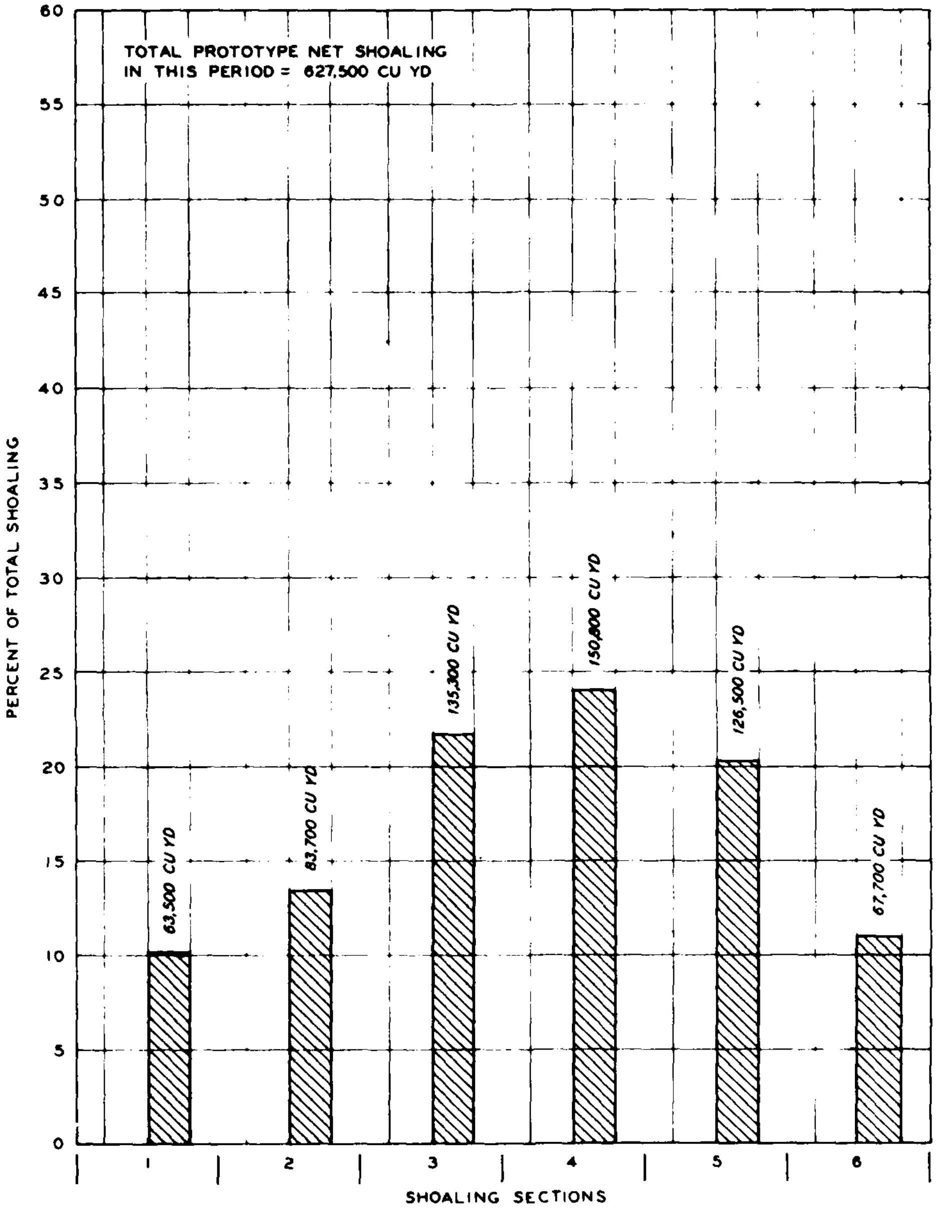
LEGEND

MODEL

PROTOTYPE

AVERAGE ANNUAL
NET SHOALING DISTRIBUTION
EAST RIVER-PORT
PRECONSTRUCTION
BASED ON HYDROGRAPHIC CHANGES
AFTER- TO BEFORE-DREDGING

PLATE B2



SECTION

1 STA 70+50 - 75+00
2 STA 75+00 - 81+00
3 STA 81+00 - 87+00
4 STA 87+00 - 93+00
5 STA 93+00 - 99+00
6 STA 99+00 - 105+00

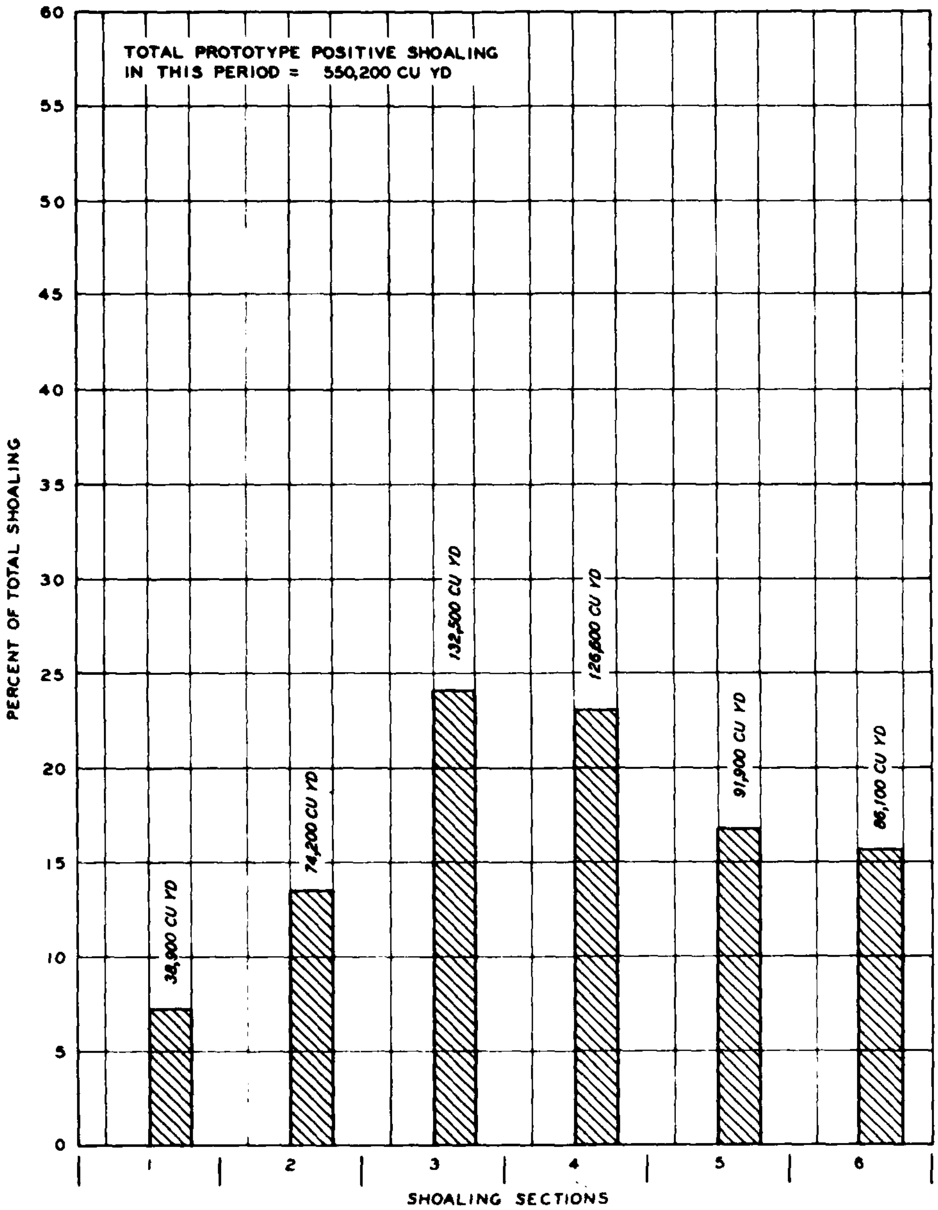
LEGEND

MODEL
PROTOTYPE

AVERAGE ANNUAL NET SHOALING DISTRIBUTION

EAST RIVER-PORT PRECONSTRUCTION

BASED ON HYDROGRAPHIC CHANGES BEFORE- TO AFTER-DREDGING



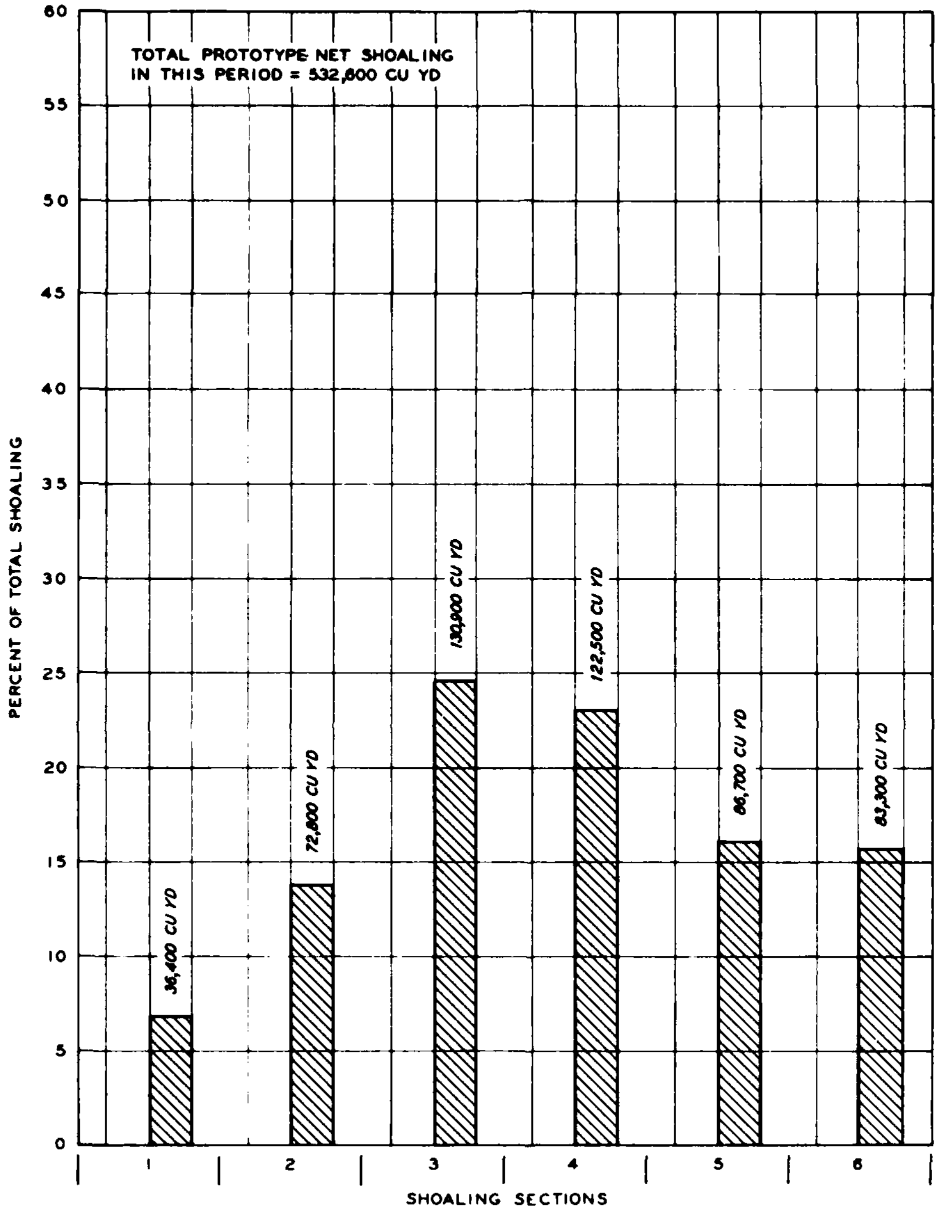
SECTION

1 STA 70+50 - 75+00
2 STA 75+00 - 81+00
3 STA 81+00 - 87+00
4 STA 87+00 - 93+00
5 STA 93+00 - 99+00
6 STA 99+00 - 105+00

LEGEND

MODEL
PROTOTYPE

AVERAGE ANNUAL POSITIVE SHOALING DISTRIBUTION
EAST RIVER-PORT
PARTIAL CLOSURE
BASED ON HYDROGRAPHIC CHANGES AFTER- TO BEFORE-DREDGING



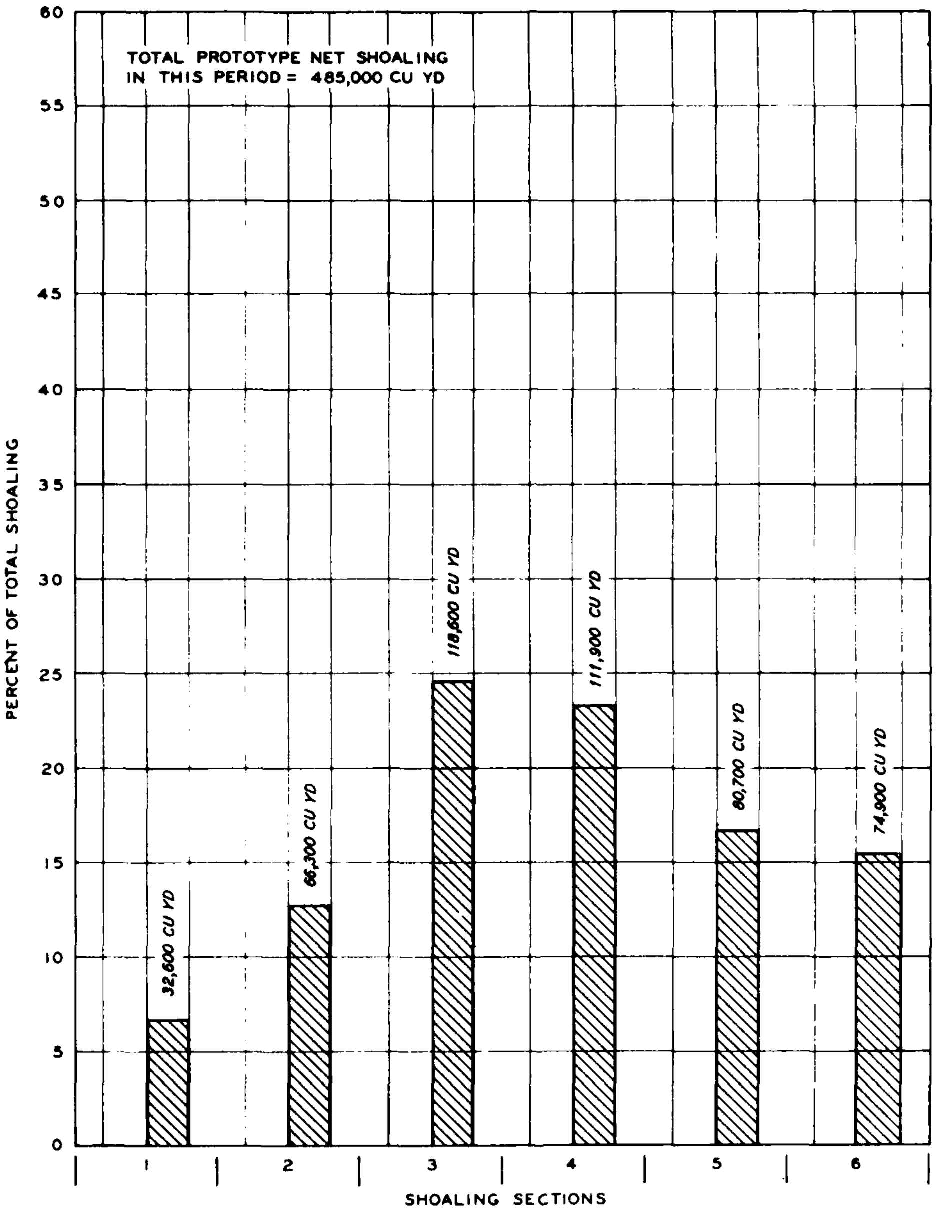
SECTION

1 STA 70+50 - 75+00
2 STA 75+00 - 81+00
3 STA 81+00 - 87+00
4 STA 87+00 - 93+00
5 STA 93+00 - 99+00
6 STA 99+00 - 105+00

LEGEND

MODEL
PROTOTYPE

AVERAGE ANNUAL
NET SHOALING DISTRIBUTION
EAST RIVER-PORT
PARTIAL CLOSURE
BASED ON HYDROGRAPHIC CHANGES
AFTER- TO BEFORE-DREDGING



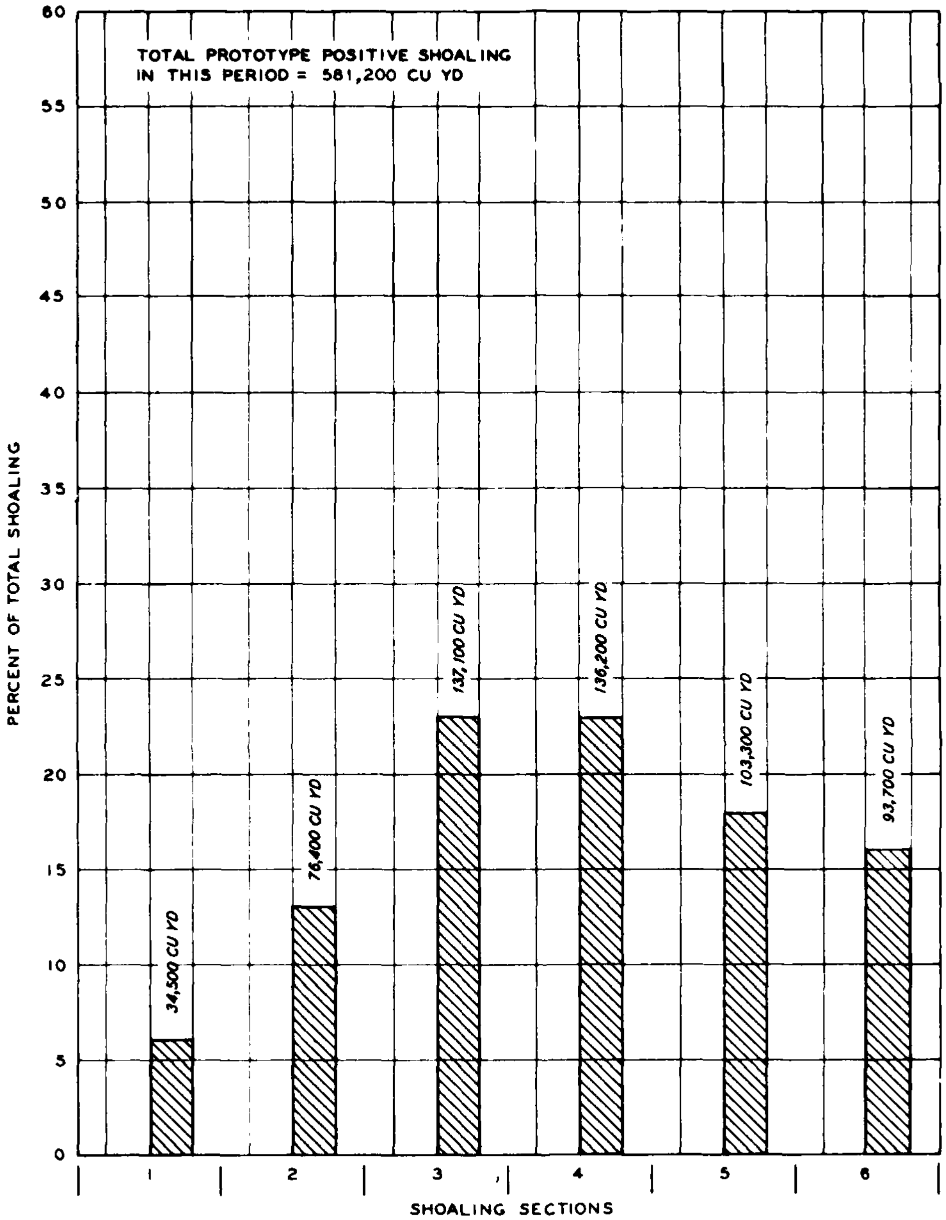
SECTION

1 STA 70+50 - 75+00
2 STA 75+00 - 81+00
3 STA 81+00 - 87+00
4 STA 87+00 - 93+00
5 STA 93+00 - 99+00
6 STA 99+00 - 105+00

LEGEND

MODEL
PROTOTYPE

AVERAGE ANNUAL
NET SHOALING DISTRIBUTION
EAST RIVER-PORT
PARTIAL CLOSURE
BASED ON HYDROGRAPHIC CHANGES
BEFORE- TO AFTER-DREDGING



SECTION

1 STA 70+50 - 75+00
2 STA 75+00 - 81+00
3 STA 81+00 - 87+00
4 STA 87+00 - 93+00
5 STA 93+00 - 99+00
6 STA 99+00 - 105+00

LEGEND

MODEL

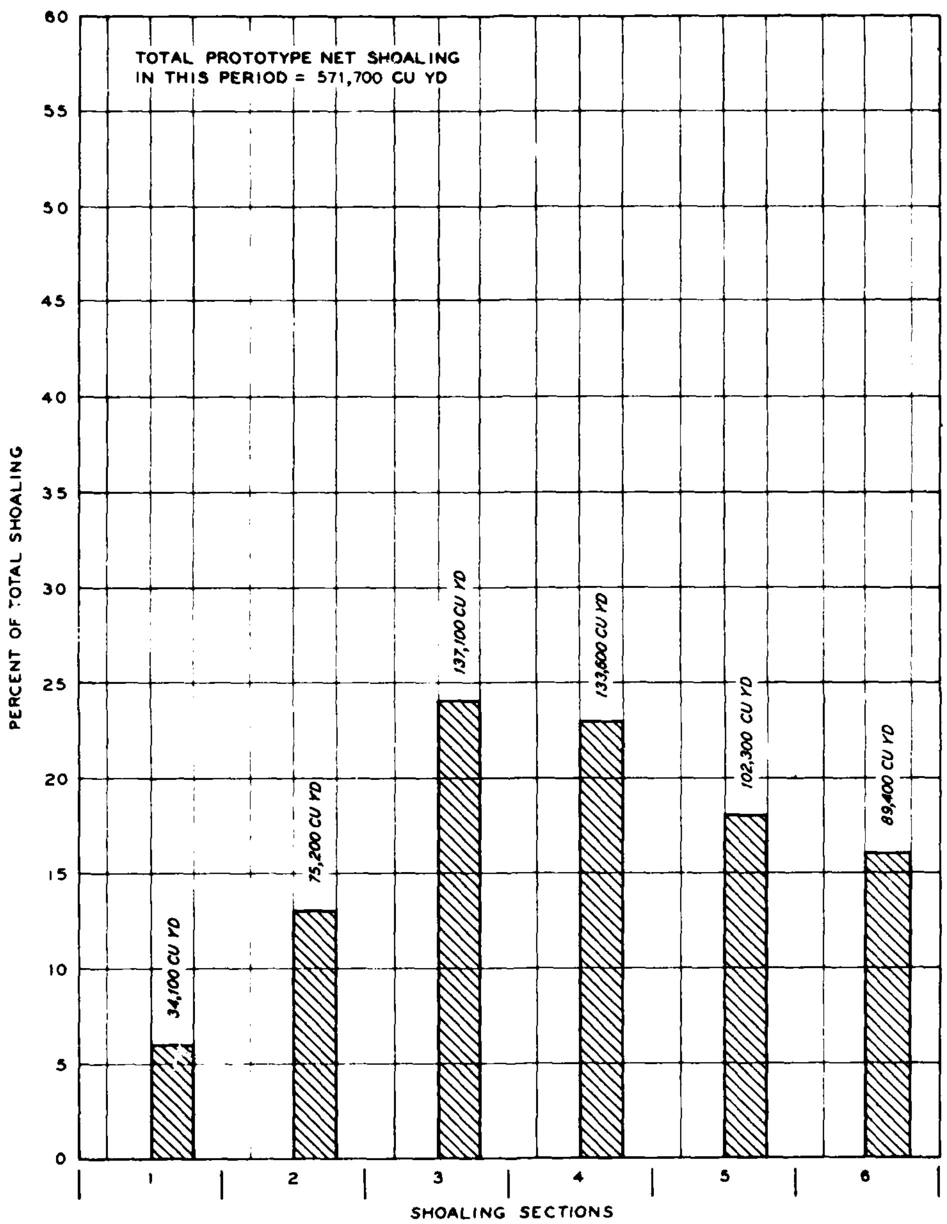
PROTOTYPE

AVERAGE ANNUAL POSITIVE SHOALING DISTRIBUTION

EAST RIVER-PORT

ALTERNATE CLOSURE

BASED ON HYDROGRAPHIC CHANGES AFTER- TO BEFORE-DREDGING



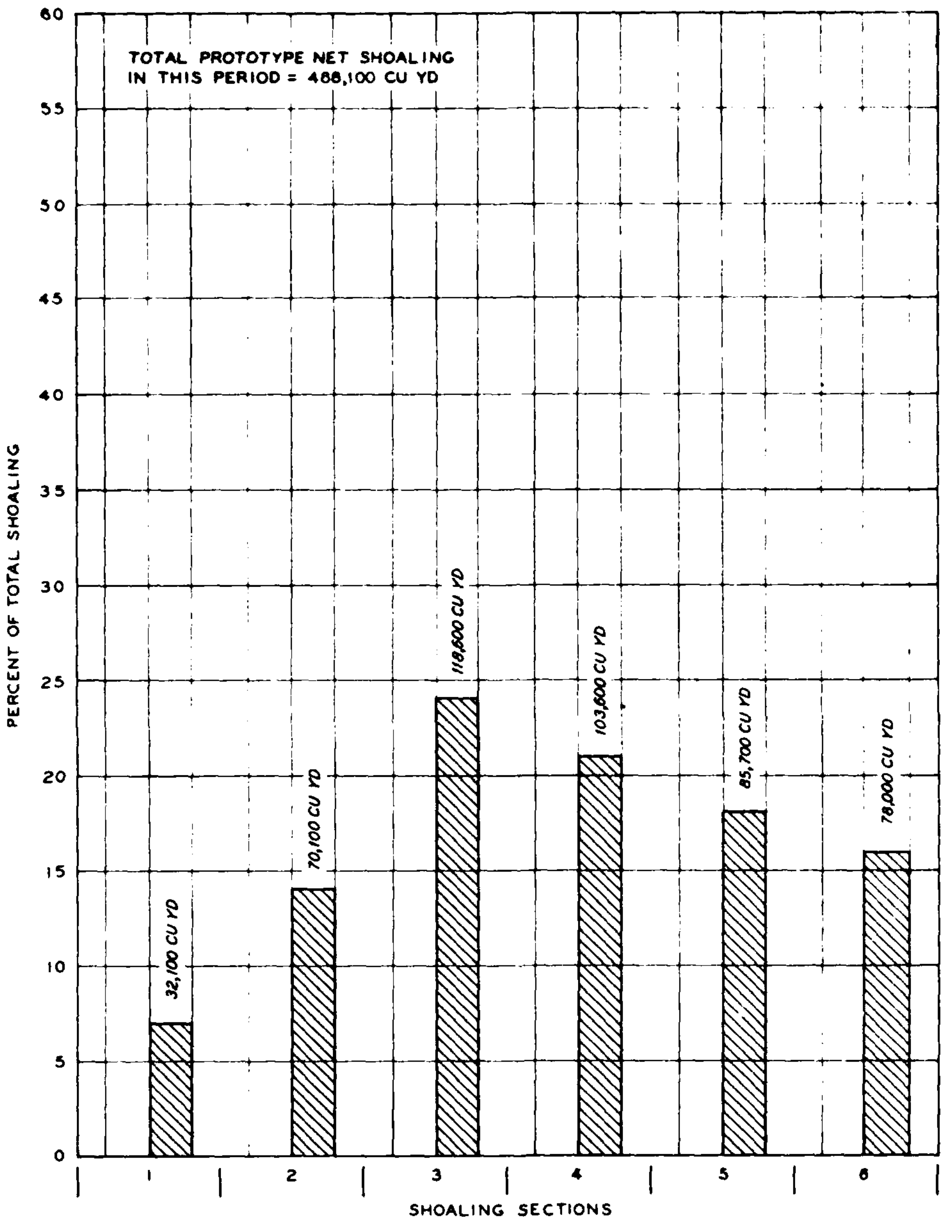
SECTION

1 STA 70+50 - 75+00
2 STA 75+00 - 81+00
3 STA 81+00 - 87+00
4 STA 87+00 - 93+00
5 STA 93+00 - 99+00
6 STA 99+00 - 105+00

LEGEND

MODEL
PROTOTYPE

AVERAGE ANNUAL
NET SHOALING DISTRIBUTION
EAST RIVER-PORT
ALTERNATE CLOSURE
BASED ON HYDROGRAPHIC CHANGES
AFTER- TO BEFORE-DREDGING



SECTION

1 STA 70+50 – 75+00
2 STA 75+00 – 81+00
3 STA 81+00 – 87+00
4 STA 87+00 – 93+00
5 STA 93+00 – 99+00
6 STA 99+00 – 105+00

LEGEND

MODEL
PROTOTYPE

AVERAGE ANNUAL NET SHOALING DISTRIBUTION

EAST RIVER-PORT ALTERNATE CLOSURE

BASED ON HYDROGRAPHIC CHANGES BEFORE- TO AFTER-DREDGING

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Letter, Joseph V., Jr.
Physical hydraulic models, assessment of predictive capabilities : Report 3 : Model study of shoaling, Brunswick Harbor, Georgia / by Joseph V. Letter, Jr., William H. McAnally, Jr. (Hydraulics Laboratory, U.S. Army Engineer Waterways Experiment Station). -- Vicksburg, Miss. : The Station ; Springfield, Va. : available from NTIS, [1981].
108 p. in various pagings, 39 p. of plates : ill. ; 27 cm. -- (Research report / U.S. Army Engineer Waterways Experiment Station ; H-75-3, Report 3)
Cover title.
"September 1981."
"Prepared for Office, Chief of Engineers, U.S. Army."
Bibliography: p. 70-71.

1. Brunswick Harbor (Georgia). 2. Hydraulic models. 3. Sedimentation and deposition. I. McAnally, William H., Jr. II. United States. Army. Corps of Engineers. Office of the Chief of Engineers. III. U.S. Army Engineer

Letter, Joseph V., Jr.
Physical hydraulic models, assessment of : ... 1981.
(Card 2)

Waterways Experiment Station. Hydraulics Laboratory. IV. Title V. Series: Research report (U.S. Army Engineer Waterways Experiment Station) ; H-75-3, Report 3.
TA7.W34r no.H-75-3 Report 3