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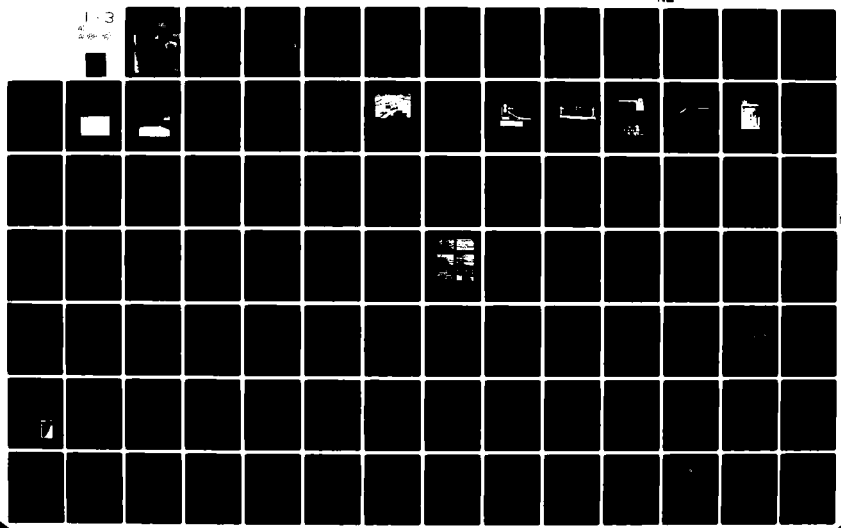
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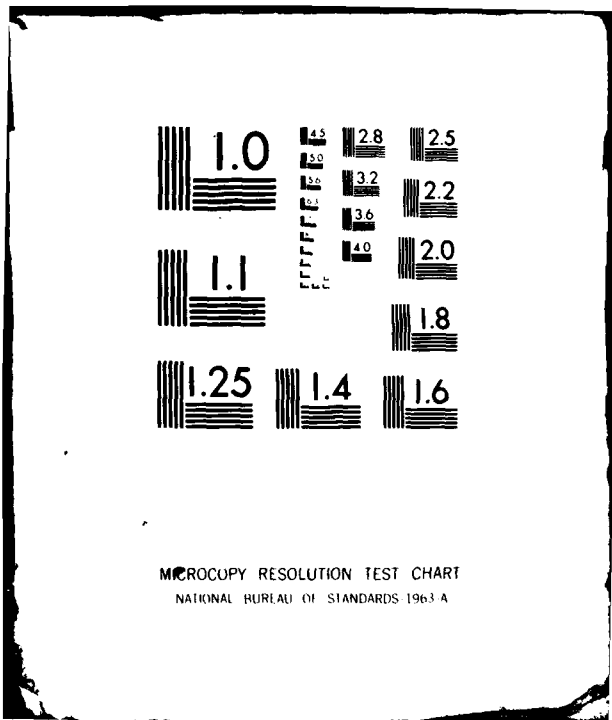
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# SUPPLEMENTARY TESTS OF MASONBORO INLET FIXED-BED MODEL

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## HYDRAULIC MODEL INVESTIGATION

by

# LEVEL

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AD A088761

GITI REPORT 18



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### GENERAL INVESTIGATION OF TIDAL INLETS

A Program of Research Conducted Jointly by

Army Corps of Engineers Research Station, Fort Belvoir, Virginia

and Army Engineer Waterways Experiment Station, Vicksburg, Mississippi

Department of the Army  
Corps of Engineers

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Cover Photo: Masonboro Inlet, North Carolina, 25 July 1974



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20. ABSTRACT (Continued)

Jetties. The third study examines the use of a tracer material and closely parallels the hydraulic testing sequence discussed in the previous Masonboro Inlet report. The tracer tests include verification of the 1969-1971 shoaling trends, testing of the prejetty 1964 condition, testing of the single jetty plan with the 1964 bathymetry, and testing of the postjetty construction 1966 condition. These tests were performed to evaluate the effectiveness of using sediment tracer materials in inlet model studies.

Results indicate the closure of any of the three interior channels in Masonboro Inlet produces a significant change in the inlet hydraulics and would most likely produce a significant change in morphology as illustrated by the prototype case history. The important changes in flow patterns at the inlet entrance occur for the ebb flow, with flood flow patterns at the entrance staying fairly constant. Keulegan K values show decreases for each closure condition, with closure of Banks Channel (the largest interior channel) providing the greatest decrease. These decreases in K are the result of a reduction in flow area and an increase in the effective length of the inlet for each closure condition. This in turn increases the bay superelevation because of the reduction in K. Shinn Creek, the interior channel perpendicular to the coast, aids in directing ebb flow oceanward, perpendicular to the coast as seen in evaluating flow volumes through range 1 for the various conditions. This indicates that the initial scour along the outer portion of the north jetty could have been caused by this ebb flow effect. This result must also be coupled with wave effects which probably contributed to filling the dredged channel location.

Weir jetty testing indicated the effect of a south jetty is to centralize the flood flow through the inlet gorge, and the presence of a weir on the south jetty does not change this basic flow pattern. As a result of this flow pattern flow increases through Shinn Creek, the channel directly behind the inlet gorge. Flow velocities over the 0.0 mlw weirs are slightly greater than those over the +2.0 weir jetties (0.2 to 0.4 fps greater). However, the flow volume coming over the 0.0 mlw weir sections is substantially greater, which slightly reduces flow entering between the jetties at their oceanward end. Ebb velocities seem to be better centralized between the jetties for the +2.0 mlw weirs and the non-weir south jetty conditions than for the 0.0 mlw weirs. Flow over the +2.0 mlw weirs cuts off very early during ebb flow since the tide level has dropped to just below the mtl once ebb flow has begun to reach faster velocities. For the 0.0 mlw weir condition, however, ebb flow over the weirs continues until nearly low water. The continued drawdown of ebb flow over the 0.0 mlw weirs aids in dispersing the flow over the entire region between the jetties and seemingly could aid currents to flow along the inner walls of the jetties rather than concentrating in the central region between the jetties where the navigation channel is located. Plan 3 (dual weirs at 0.0 mlw) produced a much greater ebb velocity predominance between the jetties than did plan 2 (dual weirs at +2 mlw)

Results of the sediment traces testing indicate that short term fill and scour trends can be predicted qualitatively. Also the reduction in model distortion to values less than this study's 5:1 scale distortion would appear to improve fill and scour tests, especially in areas of steep slopes such as the inlet's gorge.

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## FOREWORD


This report was prepared by the Estuaries and Wave Dynamics Divisions of the Hydraulics Laboratory at the U. S. Army Engineer Waterways Experiment Station (WES) as one in a series of reports on the General Investigation of Tidal Inlets (GITI). The GITI research program is under the technical surveillance of the U. S. Army Coastal Engineering Research Center (CERC), and is conducted by CERC, WES, and other Government and private organizations.

The Masonboro Inlet fixed-bed model tests were conducted by Messrs. W. C. Seabergh, Project Engineer, E. F. Lane, and L. M. Neal. The report was prepared by Messrs. Seabergh and R. A. Sager (WES). The study and report preparation were supervised by E. C. McNair, MAJ F. C. Perry, and C. L. Vincent (former WES GITI Program Managers), R. A. Sager, Chief of the Estuaries Division, R. W. Whalin, Chief of the Wave Dynamics Division, and H. B. Simmons, Chief of the Hydraulics Laboratory. CERC technical surveillance was conducted by Messrs. C. Mason and R. M. Sorensen. Civilian members of the Coastal Engineering Research Board, Dean Morrrough P. O'Brien, Professor Robert G. Dean, and Professor Arthur T. Ippen (former member, deceased), were intimately involved in both the planning and review of this report, and Professor Robert L. Wiegel contributed review comments. Technical Directors of CERC and WES were Messrs. T. Saville, Jr., and F. R. Brown, respectively.

Comments on this publication are invited.

Approved for publication in accordance with Public Law 166, 79th Congress, approved 31 July 1945, as supplemented by Public Law 172, 88th Congress, approved 7 November 1963:

  
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## PREFACE

1. The Corps of Engineers, through its Civil Works program, has sponsored, over the past twenty-three years, research into the behavior and characteristics of tidal inlets. The Corps' interest in tidal inlet research stems from its responsibilities for navigation, beach erosion prevention and control, and flood control. Tasked with the creation and maintenance of navigable U. S. waterways, the Corps dredges millions of cubic yards of material each year from tidal inlets that connect the ocean with bays, estuaries, and lagoons. Design and construction of navigation improvements to existing tidal inlets are an important part of the work of many Corps offices. In some cases, design and construction of new inlets are required. Development of information concerning the hydraulic characteristics of inlets is important not only for navigation and inlet stability but also because inlets control the daily exchange of water between bay and ocean. Accurate predictions of the effects of storm surges and runoff also require an understanding of inlet hydraulics during extreme conditions.

2. A research program, the General Investigation of Tidal Inlets program, was developed to provide quantitative data for use in design of inlets and inlet improvements. It is designed to meet the following objectives:

To determine the effects of wave action, tidal flow, and related phenomena on inlet stability and on the hydraulic, geometric, and sedimentary characteristics of tidal inlets; to develop the knowledge necessary to design effective navigation improvements, new inlets, and sand transfer systems at existing tidal inlets; to evaluate the water transfer and flushing capability of tidal inlets; and to define the processes controlling inlet stability.

3. The GITI is divided into three major study areas: inlet classification, inlet hydraulics, and inlet dynamics.

a. The objectives of the inlet classification study are to classify inlets according to their geometry, hydraulics, and stability, and to determine the relationships that exist among the geometric and dynamic characteristics and

the environmental factors that control these characteristics. The classification study keeps the general investigation closely related to real inlets and produces an important inlet data base useful in documenting the characteristics of inlets.

b. The objectives of the inlet hydraulics study are to define the tide-generated flow regime and water-level fluctuations in the vicinity of coastal inlets and to develop techniques for predicting these phenomena. The inlet hydraulics study is divided into three areas: idealized inlet model study, evaluation of state-of-the-art physical and numerical models, and prototype inlet hydraulics.

- (1) The idealized inlet model. The objectives of this model study are to determine the effect of inlet configurations and structures on discharge, head loss, and velocity distribution for a number of realistic inlet shapes and tide conditions. An initial set of tests in a trapezoidal inlet was conducted between 1967 and 1970. However, in order that subsequent inlet models are more representative of real inlets, a number of "idealized" models representing various inlet morphological classes are being developed and tested. The effects of jetties and wave action on the hydraulics are included in the study.
- (2) Evaluation of state-of-the-art modeling techniques. The objectives of this portion of the inlet hydraulics study are to determine the usefulness and reliability of existing physical and numerical modeling techniques in predicting the hydraulic characteristics of inlet/bay systems, and to determine whether simple tests, performed rapidly and economically, are useful in the evaluation of proposed inlet improvements. Masonboro Inlet, N. C., was selected as the prototype inlet which would be used along with hydraulic and numerical models in the evaluation of existing techniques. In September 1969 a complete set of hydraulic and bathymetric data was collected at Masonboro Inlet. Construction of the fixed-bed physical model was initiated in 1969, and extensive tests have been performed since then. In addition, three existing numerical models were applied to predict the inlet's hydraulics. Extensive field data were collected at Masonboro Inlet in August 1974 for use in evaluating the capabilities of the physical and numerical models.
- (3) Prototype inlet hydraulics. Field studies at a number of inlets are providing information on prototype inlet/bay tidal hydraulic relationships and the effects of friction, waves, tides, and inlet morphology on these relationships.

c. The basic objective of the inlet dynamics study is to investigate the interactions of tidal flow, inlet configuration, and wave action at tidal inlets as a guide to improvement of inlet channels and nearby shore protection works. The study is subdivided into four specific areas: model materials evaluation, movable-bed modeling evaluation, reanalysis of a previous inlet model study, and prototype inlet studies.

(1) Model materials evaluation. This evaluation was initiated in 1969 to provide data on the response of movable-bed model materials to waves and flow to allow selection of the optimum bed materials for inlet models.

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(2) Movable-bed model evaluation. The objective of this study is to evaluate the state-of-the-art of modeling techniques, in this case movable-bed inlet modeling. Since, in many cases, movable-bed modeling is the only tool available for predicting the response of an inlet to improvements, the capabilities and limitations of these models must be established.

(3) Reanalysis of an earlier inlet model study. In 1957, a report entitled "Preliminary Report: Laboratory Study of the Effect of an Uncontrolled Inlet on the Adjacent Beaches" was published by the Beach Erosion Board (now CERC). A reanalysis of the original data is being performed to aid in planning of additional GITI efforts.

(4) Prototype dynamics. Field and office studies of a number of inlets are providing information on the effects of physical forces and artificial improvements on inlet morphology. Of particular importance are studies to define the mechanisms of natural sand bypassing at inlets, the response of inlet navigation channels to dredging and natural forces, and the effects of inlets on adjacent beaches.

4. As a part of the inlet hydraulics portion of the GITI, this report is the second concerned with testing in the Masonboro Inlet fixed-bed model. This report brings together three separate test series: (a) the effects of closing various bay channels on the inlet's hydraulics; (b) an examination of weir jetties; and (c) sediment tracer testing under the action of tides and wind waves.

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

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

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
cubic feet	0.02831685	cubic metres
feet	0.3048	metres
feet per second	0.3048	metres per second
feet per second per second	0.3048	metres per second per second
inches	25.4	millimetres
miles (U. S. statute)	1.609344	kilometres
square feet	0.09290304	square metres
square miles (U. S. statute)	2.589988	square kilometres

## SUPPLEMENTARY TESTS OF MASONBORO INLET FIXED-BED MODEL

### Hydraulic Model Investigation

#### PART I: INTRODUCTION

1. This report describes supplemental tests of the Masonboro Inlet fixed-bed model not reported in "Physical Model Simulation of the Hydraulics of Masonboro Inlet, North Carolina," GITI Report 15.<sup>1</sup> The supplemental tests consist of three separate studies performed in the Masonboro Inlet fixed-bed model. The first study, reported in PART II, examines the effects of the closing of various bay channels on the inlet's hydraulics. The second study, reported in PART III, examines the effects of the addition of a south jetty to the existing condition which has a single north jetty and examines the resulting hydraulics for various weir elevations on both jetties. The third study, reported in PART IV, examines the use of a tracer material and closely parallels the hydraulic testing sequence discussed in the previous Masonboro Inlet report.<sup>1</sup> The tracer tests include verification of the 1969-1971 shoaling trends, testing of the prejetty 1964 condition, testing of the single jetty plan with the 1964 bathymetry, and testing of the postjetty construction 1966 condition. These tests were performed to evaluate the effectiveness of using sediment tracer materials in inlet model studies.

#### Background

2. Masonboro Inlet, a natural inlet through the coastal barrier beach of North Carolina, is located in the southern portion of the state, 8 miles\* northeast of Wilmington, North Carolina (Figure 1). Briefly outlining the recent history of the prototype, a single weir jetty, a navigation channel, and a sand deposition basin were constructed

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\* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 7.

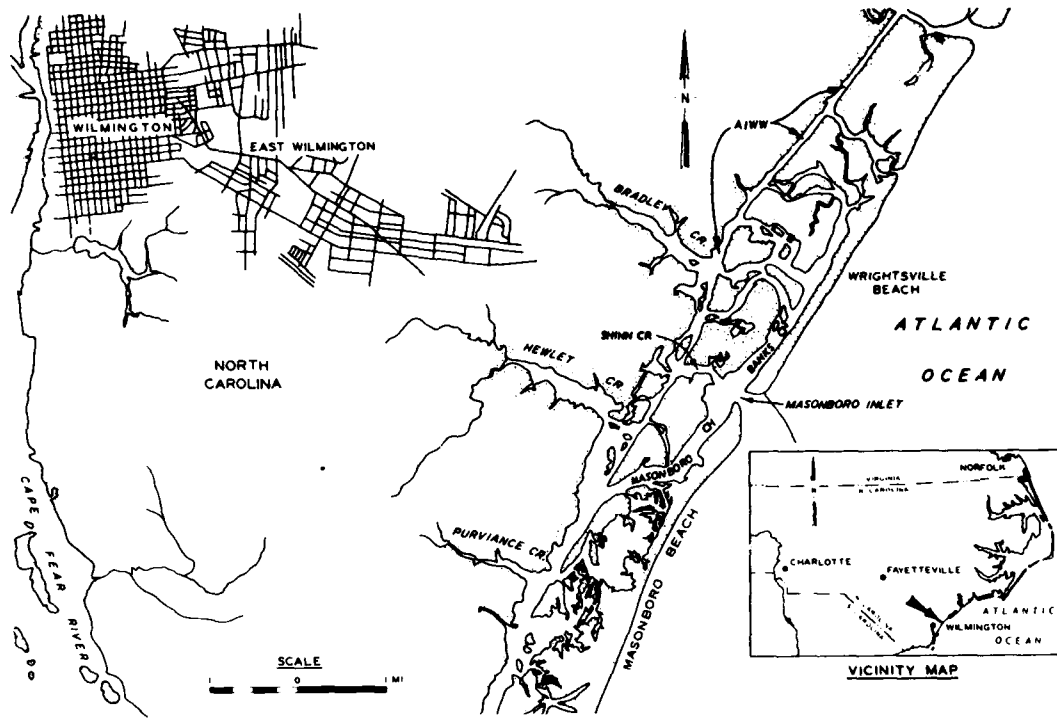
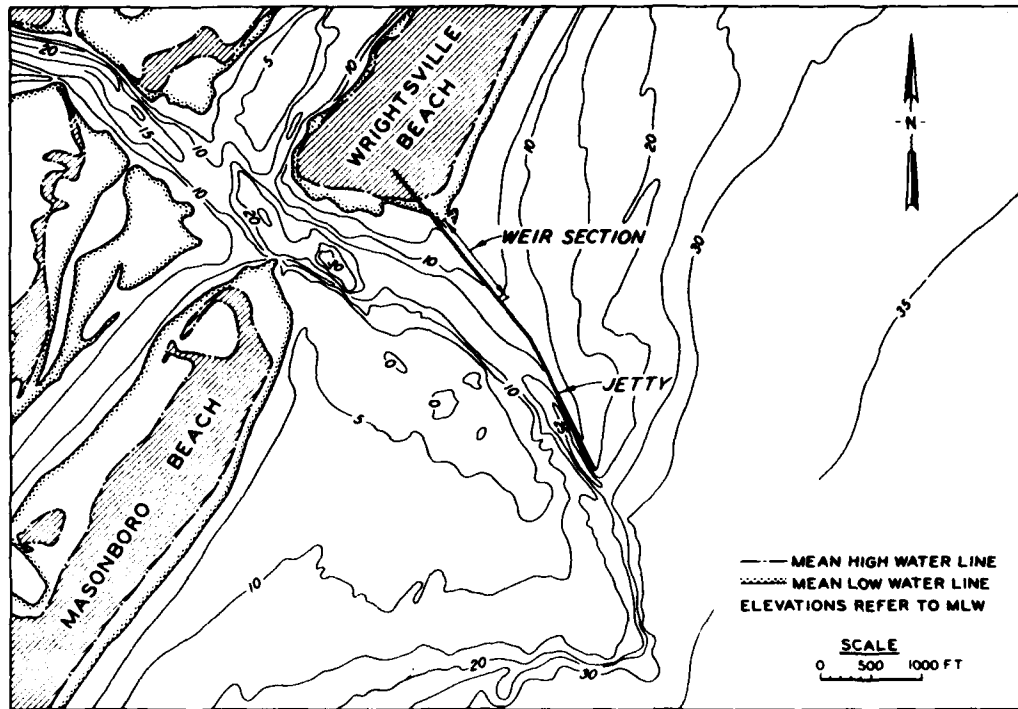


Figure 1. Location map

at the natural inlet beginning in 1965. It is noted that no model study was done prior to construction of the project. By July 1966 construction was complete. The inlet channel migrated northward into the deposition basin and alongside the outer end of the jetty between 1966 and 1969. These changes are documented by bathymetric surveys (see Figure 2). Figure 3 illustrates the movement of the channel thalweg from 1964 to 1973; Figure 4 shows the inlet in 1973. The only records of tidal heights and velocities in the inlet for this time period were obtained in 1969, four years after construction of the project; thus the model had to be verified to the 1969 postimprovement condition.

3. After verification to the 1969 condition, the model was modified to the 1964 preimprovement condition. A bathymetric survey of a limited region in and around the inlet throat was available for this period. The modeled bay area of the inlet remained the same as the original 1969 condition since there was very little change in the bay during the 1964-1969 period. No hydraulic data were available to



a. Hydrographic condition of September 1969



b. April 1969

Figure 2. Relocation of original channel

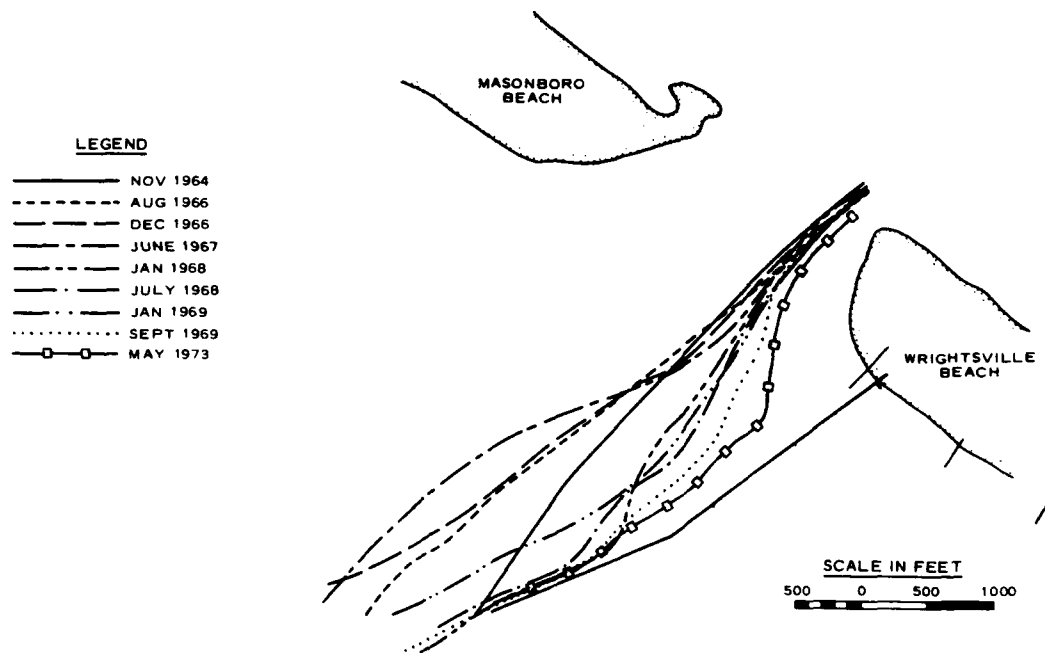


Figure 3. Channel thalwegs from 1964-1973



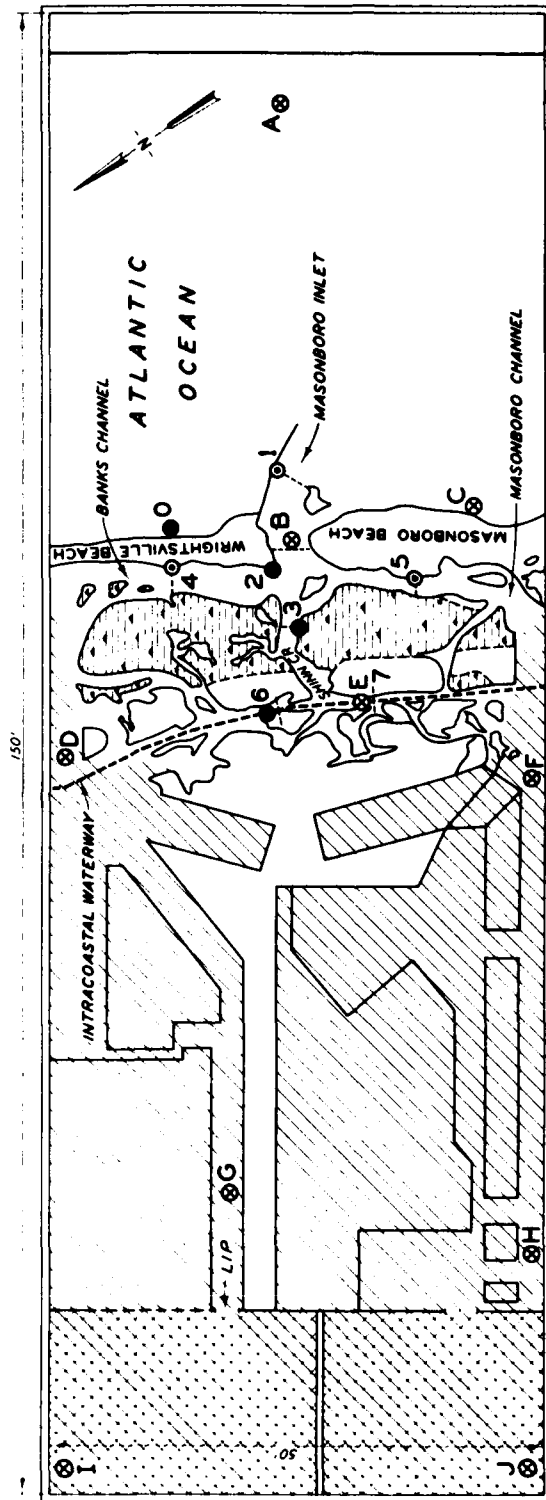
Figure 4. Masonboro Inlet in 1973

verify this preimprovement condition, but the area that was changed from the 1969 to the 1964 condition is small relative to the entire model. It was believed that experience with the 1969 verification provided sufficient information in the adjustment of roughness to allow the 1964 hydraulic conditions to be simulated accurately in the model. Model data were collected from the 1964 condition. A similar set of data was obtained with the plan installed in the model with the 1964 bathymetry. Finally, data were collected for the June 1966 postconstruction bathymetry. These hydraulic data are reported in Reference 1.

#### The Model

4. The Masonboro Inlet model reproduced approximately 14.5 square miles of the prototype including the bay area to the limits of the inlet's influence and offshore to the -45 ft mlw (mean low water) contour. The model was constructed in a 50-ft-wide by 150-ft-long facility which was completely enclosed to protect the model and necessary appurtenances from inclement weather and to allow uninterrupted operation. However, due to the limitation of the lateral extension of the facility, artificial bending of the bay areas north and south of the inlet was necessary. Distant areas of the bay were schematized. Since the principal area of interest was the immediate vicinity of the inlet, this primarily schematic reproduction of the bay allowed significant savings to be achieved in model construction. In the prototype, the bay extended north and south from the inlet entrance parallel to the coast. In the model, both outer limits of the bay were folded back toward the rear of the model facility, as shown in Figure 5. As a result, the wetland areas of the bay were maintained in the correct model-to-prototype proportion and thus should have provided the proper tidal prism storage.

5. The model was constructed to linear scale ratios, model to prototype, of 1:300 horizontally and 1:60 vertically. From these linear ratios, the following scale relations were computed based on the Froudian law of similitude:



- LEGEND**
- ▨ TIDAL AREA ADDED DURING VERIFICATION
  - ▩ AREAS REPRODUCED SCHEMATICALLY
  - PROTOTYPE AUTOMATIC RECORDING GAGE
  - PROTOTYPE STAFF GAGE
  - VELOCITY RANGE
  - ⊗ ADDITIONAL MODEL GAGES



Figure 5. General model layout

<u>Characteristic</u>	<u>Scale Relations</u>
Horizontal	$L_H = 1:300$
Vertical	$L_V = 1:60$
Volume	$L_H L_H L_V = 1:5,400,000$
Velocity	$L_V^{1/2} = 1:7.746$
Discharge	$L_V^{3/2} L_H = 1:139,428$
Time--tides	$L_H / L_V^{1/2} = 1:38.73$
Slope	$L_V / L_H = 5:1$
Time--wind wave (modeling refraction)	$L_V / L_V^{1/2} = 1:7.746$

6. North Carolina grid coordinates were used for horizontal control, and the mlw Beaufort datum (1.88 ft below 1929 mean sea level (msl)) was used for vertical control during model construction. The model was molded from metal templates spaced approximately 2 ft apart, secured to and graded from the permanent facility floor. Since the model was of distorted scale, it was necessary to add artificial roughness to achieve the proper relation of model-to-prototype resistance.

7. Three types of roughness were installed in the model in order to obtain a higher friction loss than could be achieved from the relatively smooth concrete model bed.

8. While the molded concrete was still soft, 1/2-in.-wide flat metal strips were inserted in the main flow channel regions perpendicular to the expected flow directions. The strips extended from the bottom of the channel to just below the mlw level. Spacing of the strips was governed by previous experience with roughness adjustment; and during the model construction, more of the strips were placed in the model than would be required for proper flow reproduction. The second type of roughness was created by raking the soft concrete in shallow areas of the model where roughness strips could not be used because of their low efficiency in such areas. This was done primarily on the regions in which marsh grass or shallow sandy shoals existed in

the prototype. The third means of applying roughness was to place stucco on the model and trowel it to a rough finish. Each type of roughness is shown in Figure 6.



Figure 6. The model, looking from bay to ocean

#### Model Appurtenances

9. The model was equipped with the necessary appurtenances to reproduce and measure all pertinent phenomena including tidal elevations, current velocities, waves, and sediments used in shoaling tests. Apparatus used in connection with the reproduction and measurement of these phenomena include a tide generator and recorder, velocity meters, wave generators, tidal gages, and sediment tracer injection and recovery apparatus. This equipment is described in detail in subsequent paragraphs.

### Tide generator

10. The reproduction of tidal action in the model was accomplished by means of a tide generator that maintained a differential between a pumped inflow of water to the model and a gravity return flow to the supply sump as required to reproduce all characteristics of the prototype tides at the ocean control tide gage. A schematic drawing of this system is shown in Figure 7.

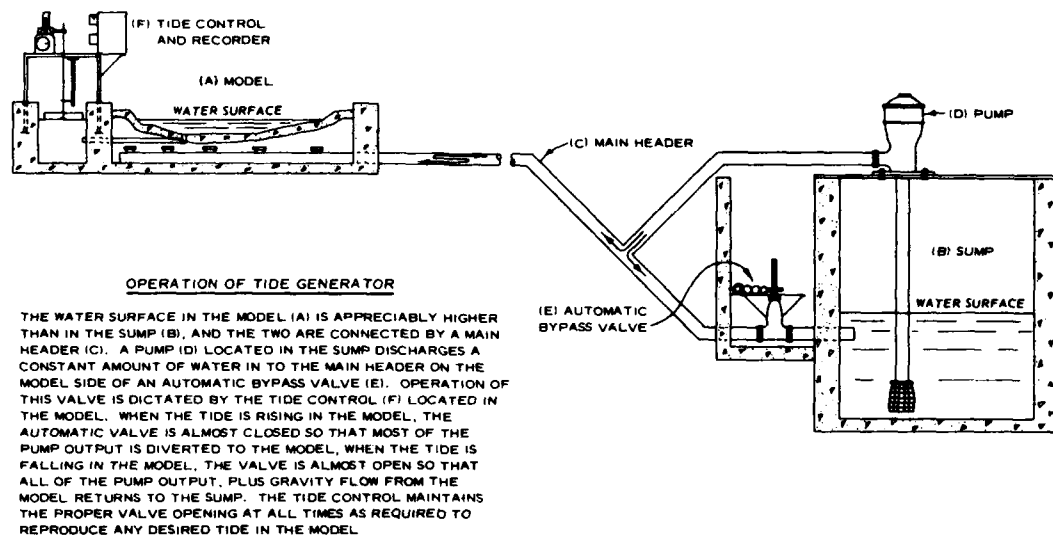


Figure 7. Operation of the tide generator

### Current meters

11. Current measurements were made in the model with miniature Price-type current meters (Figure 8). Each meter cup was about 0.04 ft in diameter, representing 2.4 ft vertically in the prototype. The center of the cup was about 0.045 ft from the bottom of the frame, representing 2.7 ft in the prototype. In a vertical plane the entire meter occupied a space of about 2 by 30 ft when scaled to the prototype. The meters were calibrated frequently to ensure their accuracy of  $\pm 0.02$  fps ( $\pm 0.15$  fps prototype) and were capable of measuring velocities as low as 0.05 fps (0.4 fps prototype). Some meters were equipped with fiber optics of which half of the strands introduced a light source that was reflected from the meter's cups. The reflected light pulse was received

on the other half of the fiber optic strands and counted electronically over a 10-sec time interval. A small digital computer controlled and recorded data collection.

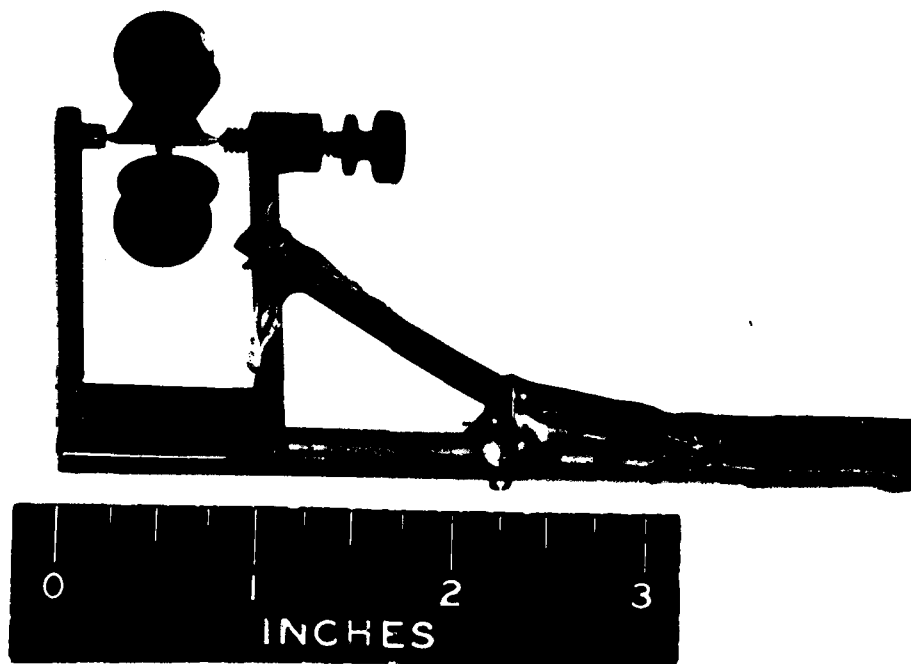


Figure 8. Miniature current meter

### Wave generators

12. Prototype wave action was reproduced in the model for shoaling tests with 20-ft-long wave generators located in such a manner as to be normal to the direction of prevailing waves. Two vertical plunger-type wave generating machines were used, and either could be quickly adjusted to generate the desired wave height, length, and period required. A section of the vertical plunger wave machine is shown in Figure 9.

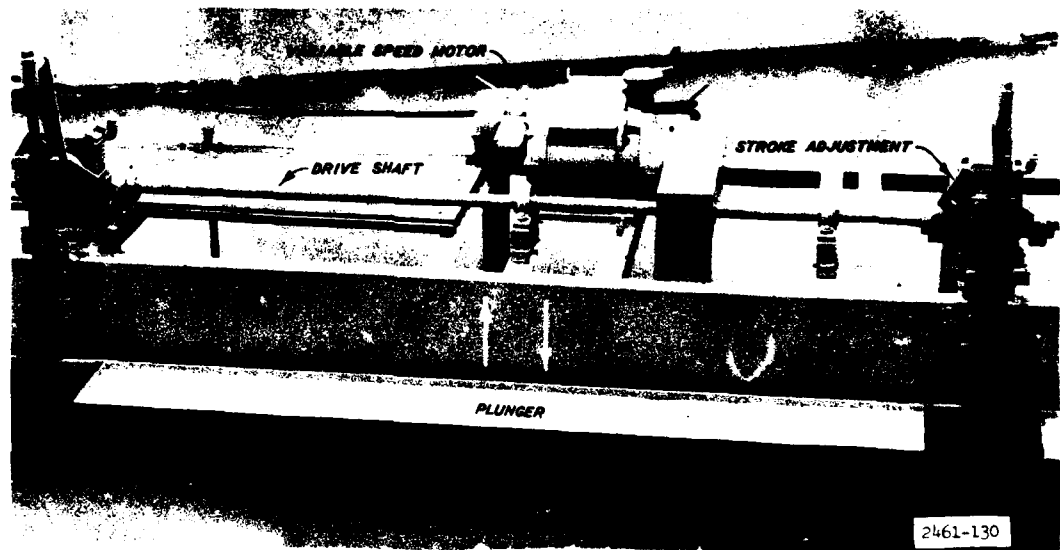


Figure 9. Wave generator

### Tide gages

13. The tidal-stage-time history was measured by the use of an electronic system consisting of a transmitter (Figure 10) and a recorder (Figure 11) with a telemetering circuit consisting of two selsyn motors, one in the transmitter and the other in the recorder, connected by an electrical cable. The tidal stage transmitter, located over the desired data gathering point, measures the water-surface elevation by means of an electronic sensing probe and transmits this elevation to a recorder located in a control or instrument house. An ink pen continually records the water-surface elevation on a chart that is turned automatically at a preset rate to give a plot of water-surface elevation

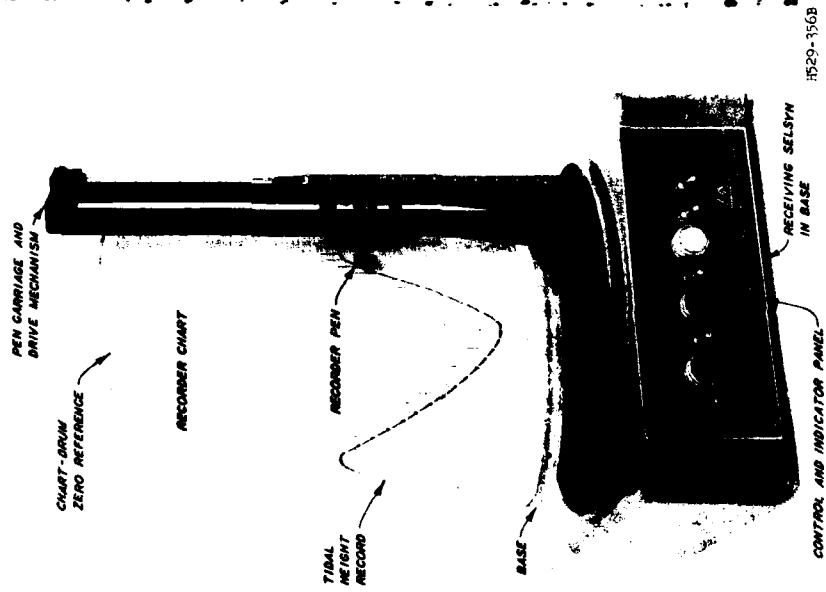


Figure 11. Water-height recorder

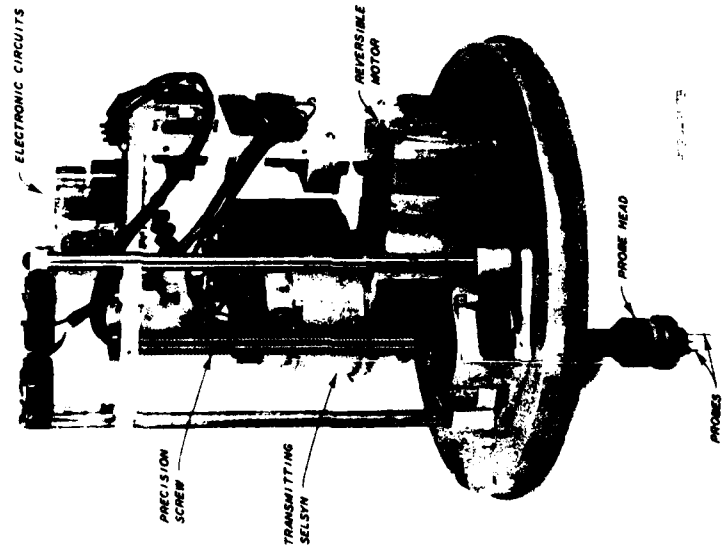


Figure 10. Water-height transmitter with cover removed

versus time. Portable point gages were also used to measure tidal elevation at other points as required.

Tracer injection  
and recovery apparatus

14. Tracer material injection and recovery were accomplished in the model by placing the premeasured amounts of material by hand, then after each test the material was picked up with a jet pump wand (Figure 12). The lightweight wand offers a high suction-to-supply flow ratio and is supplied from the city waterlines. The material is carried along



Figure 12. Jet pump wand

with the discharge water into a hydrocyclone concentrator (Figure 13) where it is separated by gravitational and centrifugal forces and deposited in a measuring cylinder.

Photographic system

15. In order to take surface current photographs, 8- by 10-in. cameras were placed about 15 ft above the water surface of the model. Their shutters were tripped simultaneously by an electronic timer for a 4-sec exposure. An electronic strobe light was flashed near the end of the exposure so that a bright spot was recorded on the tip of the confetti streak, indicating the direction of movement. Velocities can be determined by measuring the length of a streak and comparing it with a scale included on each photograph.

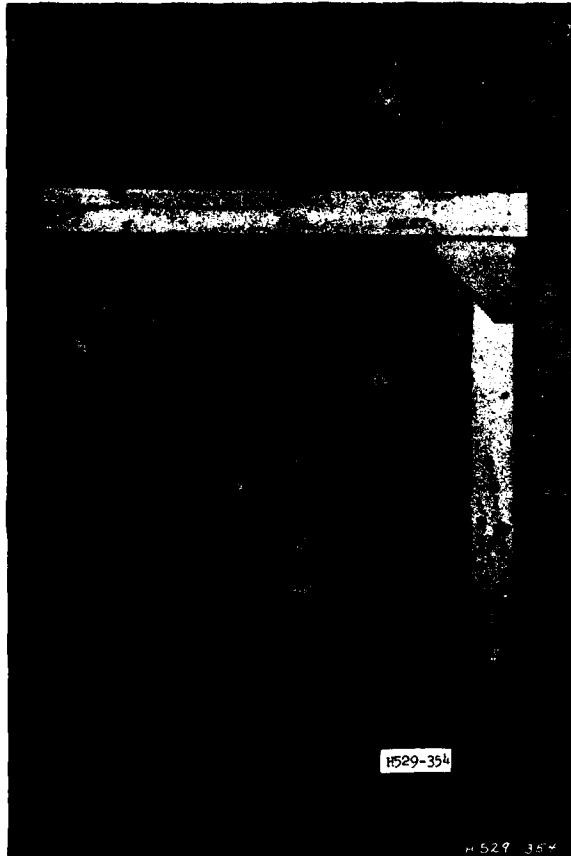


Figure 13. Hydrocyclone concentrator

## PART II: CHANNEL CLOSURE TESTS

16. The bay system of Masonboro Inlet is composed of three distinct channels which diverge from the inlet's gorge and eventually intersect the Atlantic Intracoastal Waterway (AIWW) at three different locations (Figure 1). The channels convey flow both directly to the AIWW and laterally into adjacent marshlands. The channel closure tests were designed to investigate the effect on the tidal hydraulics of closing one of the three channels for each of the three possible cases. Location of the barrier for each channel and locations of tide gage stations and velocity ranges at which data were collected are shown in Figure 14. The following identification and abbreviations are used for the various curves and discussions:

- Base = Natural 1973 conditions
- Plan 1 = Banks Channel closed = BCC
- Plan 2 = Shinn Creek closed = SCC
- Plan 3 = Masonboro Channel closed = MCC

### Tidal Heights

17. The tidal height hydrographs are shown in Plates 1-4 and a summary of the tidal data is tabulated in Table 1 showing high- and low-water elevations, tide ranges, and mean tide levels. The greatest changes occur in the region directly behind a barrier. For example, when Banks Channel was closed, the tide range bayward of the barrier of gage 4 was reduced 36 percent from the base condition. With Shinn Creek closed, the tide range on the bay side of the barrier at gage 3 was reduced 26 percent. The tide ranges at gages 3 and 4 did not vary over 6 percent when the channel in which they were located was open. The tide range at gage 5 in Masonboro Channel was reduced from 12 to 15 percent when each of the other two channels was individually closed. With Masonboro Channel closed, gage 5, located oceanward of the barrier, showed a 0.1-ft increase in tide range over the base condition. Gages 6 and E, located in the AIWW north and south, respectively, of

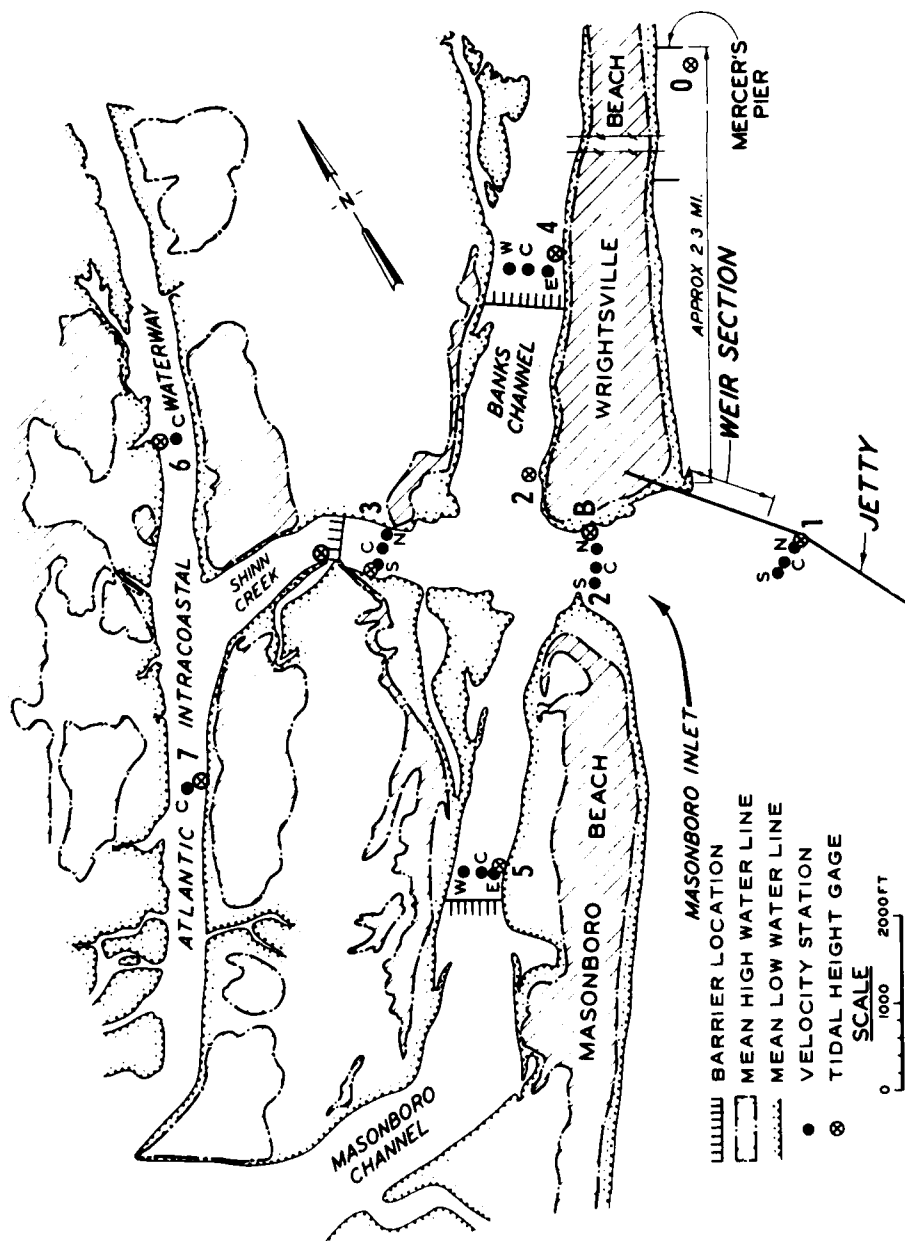


Figure 14. Range, gage, and barrier locations

the intersection of Shinn Creek with the AIWW, showed tide range changes for each case of channel closure. The greatest change for these gages resulted when Shinn Creek was closed--a 21 to 24 percent reduction in range occurred. When Banks Channel was closed, gage 6 showed a greater reduction in range than gage E since flow to the north side of the estuary was reduced and thus the tide range was decreased a greater amount than at gage E. When Masonboro Channel was closed just the opposite was true; that is, gage E showed a greater reduction in range than gage 6.

#### Mean Tide Levels

18. Mean tide levels at bay gages 3-6 and E showed increases from 0.02 to 0.32 ft over the base condition for various closure conditions except for two cases at gage 5, where the gage is located oceanward of the Masonboro Channel barrier. With this barrier closed, the mean tide level at gage 5 dropped 0.20 ft. A discussion of these superelevations will be continued after an examination of the velocity data.

19. The greatest phase changes in the tidal height data (Table 2) occurred at gages directly behind the barriers. At gage 4 with a barrier in Banks Channel, the high water was delayed 1.0 hr and low water 1.5 hr from the base condition. A barrier in Shinn Creek delayed high water 0.5 hr and low water 1.5 hr from the base condition at gage 3 (located bayward of the barrier). Gage 1 oceanward of the entrance showed no phase changes. Gage B, in the entrance, showed only a 0.5 hr early arrival of high water for Banks Channel closed. Gage 2 had no changes in high-water arrival times, but had low waters 0.5 to 1.0 hr earlier due to faster falling tides or lower elevation of high water (see Plate 1). Plate 2 indicates an earlier rise and fall at gages 3-5 whenever the channel in which one of these gages is located is open.

#### Velocities

20. Velocities were measured at ranges 1-7 (Figure 14). Graphs of velocities during the tidal cycle are given in Plates 5-19. Ranges 1-5

had three stations across the channel whereas ranges 6 and 7 had only center-line stations. Velocities were measured at three depths (surface, middepth, and bottom) if the location was sufficiently deep; otherwise, surface and bottom, or just middepth, measurements were made at shallower locations.

21. Velocities at range 1 (Plates 5-7) showed reductions in the average of the maximum velocities for the entire range for all the channel closures when compared with the base condition. The average of the maximum flood velocities was reduced from the base condition value of 1.5 fps to 1.2 for BCC, 1.0 for SCC, and 1.4 fps for MCC. The average of the maximum ebb velocities was reduced from a base value of 2.9 fps to 2.6 for BCC, 2.5 for SCC, and 2.8 fps for MCC. Individual maximum velocities at ranges 1-5 are shown in Table 3. At range 2 (Plates 8-10) the average of the maximum velocities at each point velocity location was reduced for all the channel closure conditions. An average maximum for the base condition flood flow was 2.8 fps, compared with 2.2 for BCC, 2.3 for SCC, and 2.3 for MCC. On ebb flow, average maximum velocities were reduced from a base condition value of 3.1 fps to 2.5 for BCC, 2.5 for SCC, and 2.9 fps for MCC. Velocities at range 3 (Plates 11-13) in Shinn Creek always showed increases for the closures of its complementary channels. The greatest changes in peak velocities for BCC were from 0.5 to 0.9 fps for flood flow and from 0.3 to 1.4 fps for ebb flow. Increases in peak velocities for MCC were smaller--0.2 to 0.8 fps during flood flow, 0.1 to 0.4 fps during ebb flow. There was a lag of about 1 hr in the change from ebb to flood flow at range 3, when closure conditions were in effect.

22. At range 4 the greater change in velocities (Plates 14-16) occurred for the closing of Shinn Creek which generally increased peak velocities by about 0.5 fps. The east station showed the greatest velocity increases at the range for SCC. A delay in the change from ebb to flood occurred at this range similar to that at range 3.

23. At range 5 (Plate 17) the velocities for the BCC condition appeared to be low. When compared with velocities observed on the hourly surface current photographs, they were confirmed to be too low.

These velocities could not be checked because the data analysis was performed two years following the test; the model facility was not currently operable and was not molded to the 1973 bathymetry. The error is likely due to an incorrect calibration of the computer terminal--velocity meter interface. Later analyses of flows through range 5 for the BCC condition were based on ratios of surface currents (from the photographs) to known correct point velocity readings, e.g., at a given time. Maximum velocities determined in this way show increases over the base condition. At range 5, for the SCC condition, velocities were generally increased by about 0.5 fps. Also a lag of about 1 hr occurred for the change from ebb to flood flow when compared with base conditions.

24. Ranges 6 and 7 in the AIWW (Plates 18 and 19) show marked changes due to the various closure conditions. At range 6 the base condition showed a long duration (about 11 hr) flood flow. This is due to the manner in which Shinn Creek joins the AIWW. During ebb flow (for the base condition), the currents through range 7 dominated flow into Shinn Creek and only a small amount of flow ebbed through range 6. When Banks Channel was closed, this channel was the only route for flow to and from the northern portion of the bay. The highest velocities at range 6, up to 2.2 fps, were experienced on both ebb and flood flows. When Shinn Creek was closed, however, the flow direction was altered considerably. During the flooding tide at the entrance, the flow at range 6 was toward range 7 (the ebb direction). During the ebbing tide at the entrance, flow at range 6 was in what was normally the flood direction. This flow pattern was in response to the filling of the south bay. Masonboro Channel is relatively small and cannot handle a sufficient amount of flow to fill the south bay. The flow along the AIWW fills this portion of the bay. When Masonboro Channel was closed, the velocities at range 6 on flood were reduced because essentially all the flood flow through Shinn Creek was required to fill the south bay area.

25. The greatest velocity increase at range 7 (Plate 19) occurred when Masonboro Channel was closed, with changes up to +1.5 fps. The greatest decrease in velocities occurred when Shinn Creek was

closed and flow was fed through range 6 to range 7 on the flood, and which returned through range 6 and Banks Channel on the ebb flow. Closure of Banks Channel caused slight overall reductions in velocities at range 7 because the flows through range 6 increased greatly to feed the north portion of the bay. The greater head difference drew some of the range 7 normal flow through range 6.

#### Isovels

26. The manner in which the channel closures affected velocities at ranges 1 and 2 is illustrated in Plates 20-23 which show isovels (lines of constant velocity) for the actual maximum velocities. The maximum flows shown were determined by choosing the time of the maximum average velocity over the channel cross section and then plotting the individual point velocities. These point velocities were normalized by dividing by the cross-sectional average velocity and are seen in Figure 15. Regions with values greater than 1.0 are shaded. The isovels are for periods of maximum flow through the cross section. The point velocities were normalized by dividing by the average velocity over the cross section. Figure 15a shows that the flood flow distribution at range 1 stayed about the same for all cases with the closure of Shinn Creek producing the greatest concentration of flood flow away from the jetty.

27. Ebb isovels at range 1 (Figure 15b) show that the base condition has the highest concentration of flow along the shoal portion of the cross section, away from the north jetty. When Banks Channel was closed, the highest flow region was shifted to the surface layer across the width of the channel, due to the effect of ebb flow from Masonboro Channel forcing the mainstream of ebb flow closer to the jetty. The reason that the higher flows occurred near the surface was most likely due to the fact that Masonboro Channel is shallower (-6 ft) than Shinn Creek and contributes its flow to the upper portion of the water column. With Shinn Creek closed, the ebb flow has two high concentration regions. The higher flow is concentrated toward the bar side of the channel, due

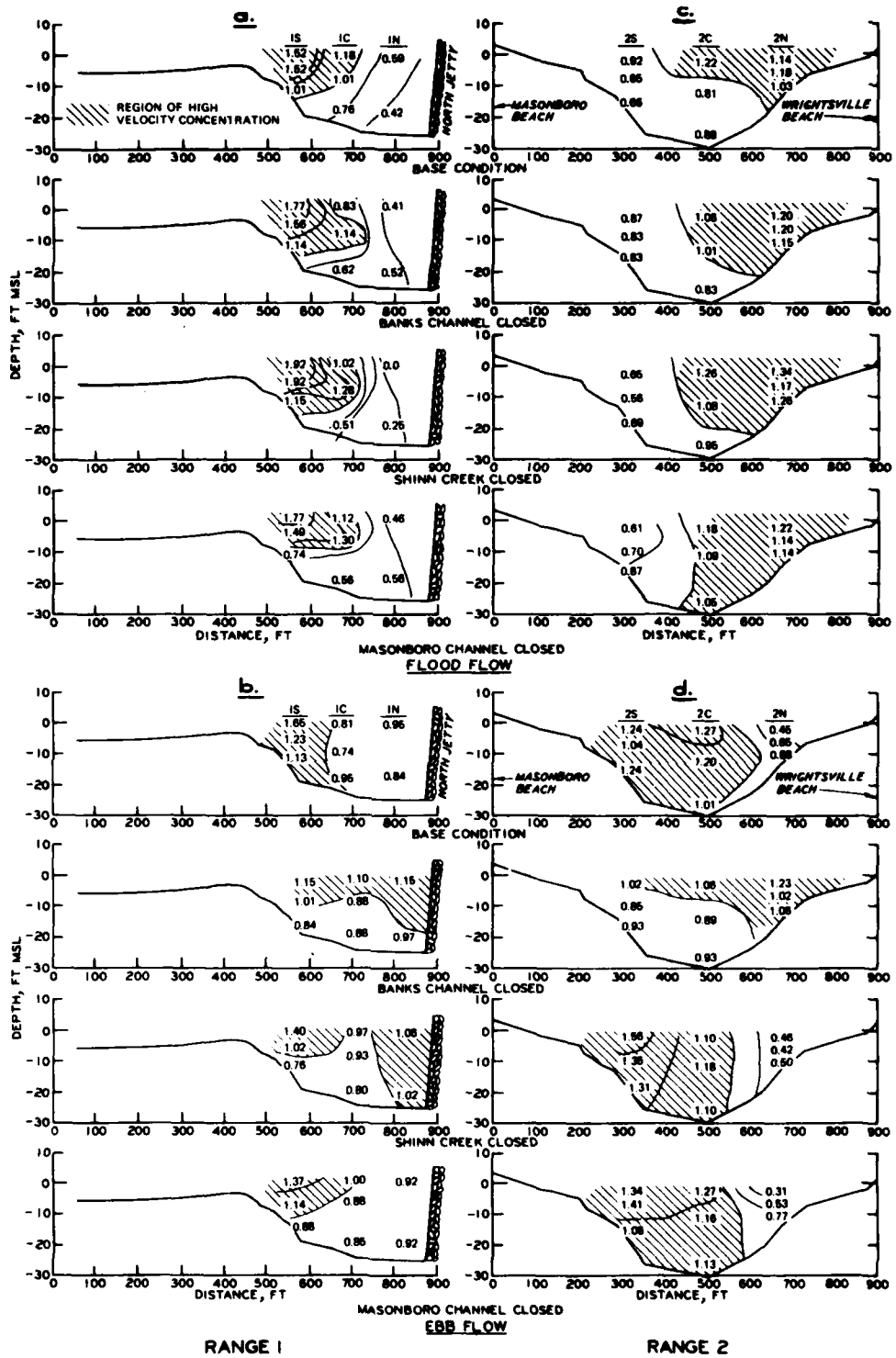


Figure 15. Normalized isovels for flood and ebb flow at ranges 1 and 2

to the dominance of the ebb flow of Banks Channel over that of Masonboro Channel. With Masonboro Channel closed, the ebb flow concentrated toward the shoal region, very similar to the base condition.

28. Figure 15c shows the maximum flood isovels as one looks bayward at the channel cross section at range 2. For all four conditions tested, the flood velocities were concentrated toward the Wrightsville Beach side, the location of the larger bay channels--Banks Channel and Shinn Creek--and also the side where flow over the weir enters the bay. Flow over the shallow bar at the entrance approaches the inlet throat (range 2) from the south, thus concentrating flow on the northern (Wrightsville Beach) side of the inlet. In each case of closure there was an increase of flow concentration on the Wrightsville Beach side of the inlet throat.

29. Maximum ebb isovels at range 2 are shown in Figure 15d. For the base condition the dominance of Banks Channel is noted by the concentration of current on the Masonboro Beach side of the inlet. With Banks Channel closed the maximum currents were much lower and were located on the Wrightsville Beach side. With Shinn Creek closed the currents from Banks Channel dominate and the higher velocity region occurred along the Masonboro Beach side of the inlet. When Masonboro Channel was closed the isovel pattern was similar to that of the base condition except a slightly higher flow concentration occurred along the Masonboro Beach side.

#### Discharges and Flow Volumes

30. By averaging the flow velocities over the cross section at a given time and multiplying by the channel cross-sectional area at that time, discharge curves have been prepared for ranges 1-7 (Plates 24-30). These curves were then integrated over the flood and ebb portions of the tidal cycle to calculate ebb and flood flow volumes through the range. These are included on the discharge curves and are also shown in Table 4 and diagrammed in Figure 16.

31. The easiest way to examine these data would be through the

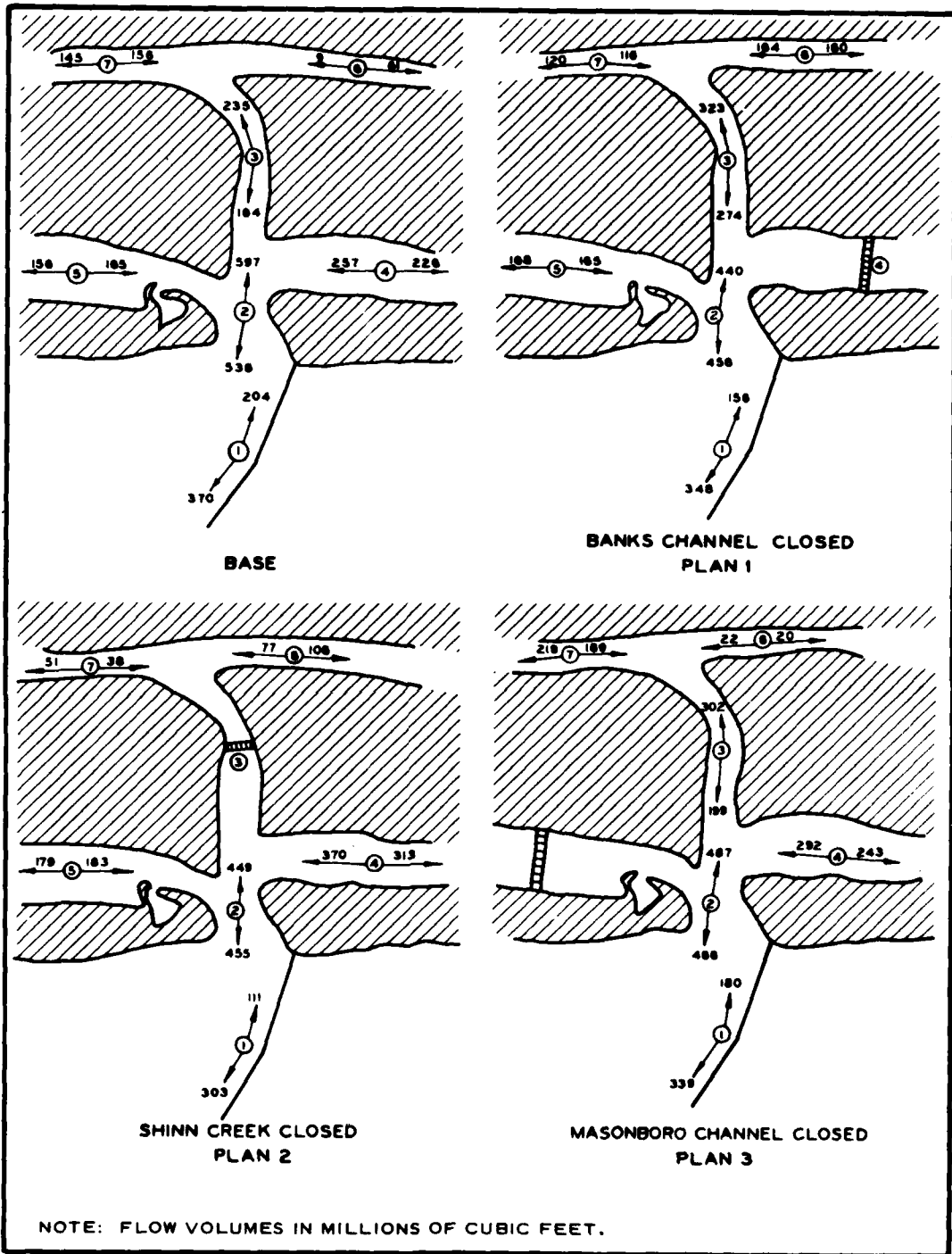


Figure 16. Ebb and flood flow volumes through ranges 1-7

flow volumes, which are shown in their respective locations in Figure 16. At range 1 there is a decrease of flood and ebb volumes for each closure. This is due to reduction of the tidal prism (which can be represented by the average of the flood and ebb volumes at range 2) in each case. The closures caused a 16 to 21 percent reduction in tidal prism (Table 4).

32. Because there were no velocity stations on the shallow area south of the channel on range 1, a substantial portion of the total flows past range 1 is unaccounted for in the discharge calculation (about 30 percent). It is, therefore, difficult to evaluate the effects of the closure by direct comparison of the discharge computation. The flows at range 1 for each plan can be expressed as the percentage of the flow at range 2. For the base condition 69 percent of the ebb flow at range 2 passes through range 1, 76 percent for BCC, 66 percent for SCC, and 73 percent for MCC. Inspection of these percentages shows that for BCC, proportionally 7 percent greater flow ebbed through the metered portion of range 1 than the base condition, for SCC there was a 3 percent reduction, and for MCC there was a 4 percent increase. Shinn Creek then must have an important role in directing ebb flows out along the jetty channel through range 1. With BCC there was a higher proportion of flow along the jetty. With SCC there was a considerable decrease in percent and magnitude of the flow through range 1. For MCC an increase in percent of flow through range 1 was somewhat surprising, but examination of Table 4 shows that the flow increased 25 percent through Shinn Creek, while it increased only 11 percent for Banks Channel. This enabled the ebb flow from Shinn Creek to dominate that through Banks Channel and caused a slight increase over the base condition for the flow through range 1 without the aid of the ebb flow from Masonboro Channel. The closure of Shinn Creek also had the greatest effect on flood flows through range 1. Taking the percentage of flood flow through range 1 of the flood flow measurement through range 2, it is found that for the base, flood flow through range 1 was 34 percent of the flood flow through range 2, 35 percent for BCC, 25 percent for SCC, and 37 percent for MCC. This shows a 9 percent decrease in flood

flow through range 1 for SCC, and the greatest increase--3 percent-- coming for MCC. For BCC, the change is only 1 percent. Thus the flow in Shinn Creek had a noticeable effect on flood flows through range 1.

33. Table 4 indicates that with BCC, Shinn Creek almost entirely absorbed excess flow diverted by the closure, with a 49 percent increase in flow volume compared with a 4 percent increase in Masonboro Channel. With SCC, Banks Channel flow increased 42 percent and Masonboro Channel increased 12 percent. Masonboro Channel took more flow in this case due to its closer proximity to Shinn Creek than Banks Channel. With MCC, Shinn Creek showed the greatest increase, 25 percent, with Banks Channel flow volume increasing 11 percent.

#### Surface Current Photographs

34. Photographs during early flood (hour 4), late flood (hour 7), early ebb (hour 10), and late ebb (hour 0) are identified as Photos 1-16 for the base and three closure conditions.

35. Early flood flow patterns (hour 4, Photos 1, 5, 9, and 13) look similar oceanward of the inlet gorge. Inside the inlet they vary with the closure conditions. Two interesting points are noticed. First the effect of whether there is flow into Shinn Creek or not on flow lines into Banks Channel can be seen by comparing Photos 1, 9, and 13. With Shinn Creek closed (Photo 9) the radius of curvature of the flow lines is smaller for the streaks entering Banks Channel. With either the base condition (Photo 1) or Masonboro Channel closed (Photo 13), the flow lines follow a broader arc. In Photo 9 the region behind the barrier is at slack while for the other conditions there are significant flows.

36. Peak flood flow photographs (hour 7, Photos 2, 6, 10, and 14) indicate patterns similar to early flood except there is flow over the north jetty weir section. Velocities over the north weir were slightly increased over the base at 1.5 fps for each closure compared with 1.3 fps for the base.

37. Early ebb photos (hour 10, Photos 3, 7, 11, and 15) show

variations for the different test conditions primarily at the inlet throat. Photo 7 compared with Photo 3 (base condition) shows how the flow is directed toward the Wrightsville Beach side of the inlet. Photo 11 shows the thrust in the opposite direction with SCC. The Banks Channel ebb flow is greater than the Masonboro Channel ebb flow. Photo 15 compared with Photo 11 illustrates the straightening effect Shinn Creek has on the flow exiting through the throat.

38. Late ebb flow photographs (hour 0, Photos 4, 8, 12, and 16) show interesting variations from the base (Photo 4). Photo 8 shows very high velocities exiting the AIWW from the north bay into Shinn Creek due to Banks Channel closure while the base condition shows the flow from the south bay dominating AIWW flow. Late in the ebb, Photos 8 and 16 show that flows out along the jetty are similar to the base. The discharge through range 3 has just passed its maximum value and more of the flow is directed similar to the base conditions. Also, in each of the above cases (BCC and MCC) range 3 has received the greater increase in flows than the other open channel, so it has more power to direct flows along the existing channel. Photo 12 shows chaotic con-fetti streaks at the inlet gorge, with the Banks Channel flow still dominant and perhaps forcing more flow over the bar region than the other conditions do at this time of the tidal cycle.

#### Keulegan's K

39. Masonboro Inlet is a complex inlet and its hydraulic characteristics are not necessarily fully described by the method of Keulegan<sup>2</sup> but it is possible to use his method to make comparisons due to the changes in inlet hydraulics caused by the various channel closures. Keulegan's analysis, among other simplifying conditions, considered a channel cross section that did not vary with the tidal elevation and a bay with vertical walls which did not vary in surface area. Obviously, Masonboro Inlet does not meet these criteria but relative comparisons can be made. Keulegan defined a coefficient K, called the coefficient

of repletion, to represent the relative filling of the bay.  $K$  is defined as follows:

$$K = \frac{T}{2\pi a_o} \cdot \frac{A_C}{A_B} \cdot \sqrt{\frac{2gRa_o}{\lambda L + mR}}$$

in which

$T$  = tidal period, sec

$a_o$  = ocean tidal amplitude, ft, half-range

$A_C$  = cross-sectional area of inlet,  $\text{ft}^2$

$A_B$  = surface area of bay,  $\text{ft}^2$

$g$  = acceleration due to gravity,  $\text{ft}/\text{sec}^2$

$R$  = hydraulic radius of channel, ft

$\lambda$  = coefficient of friction, defined as follows:  $\sqrt{\lambda} = \frac{n\sqrt{2g}}{1.486R^{1/6}}$   
with  $n$  = Manning's coefficient

$L$  = length of channel, ft

$m$  = coefficient resulting from velocity distribution

Dean<sup>3</sup> has shown that  $K$  is related to the ratio of bay to ocean tidal amplitude and the phase lag  $\epsilon$  as shown in Figures 17 and 18,

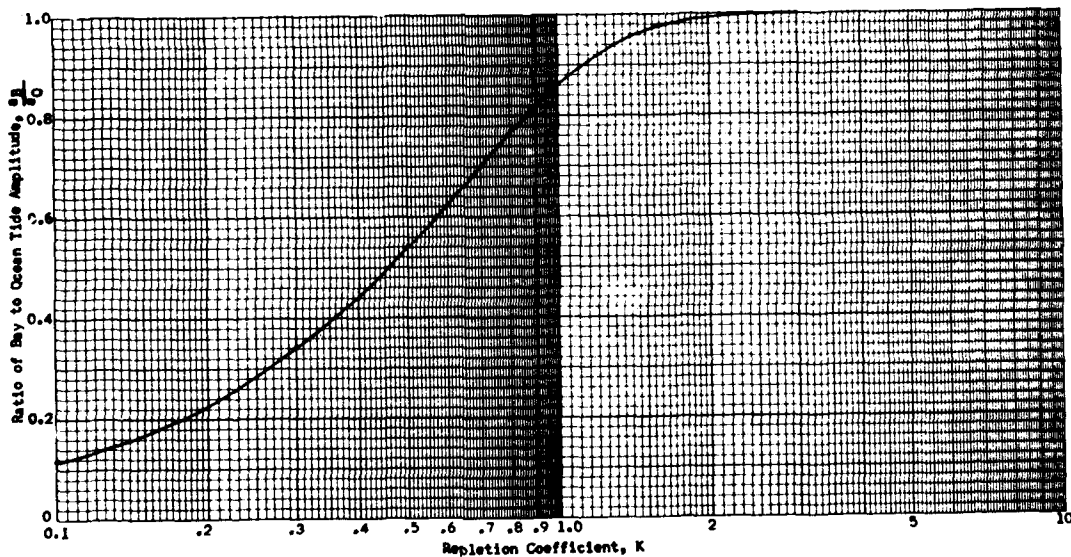


Figure 17. Ratio of bay tide to ocean tide amplitude as a function of the coefficient of repletion,  $K$

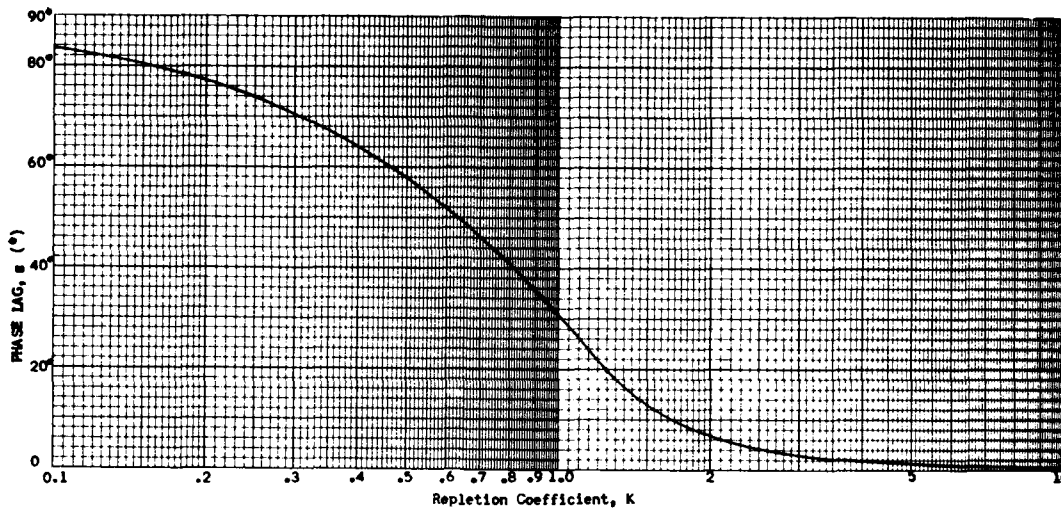


Figure 18. Phase angle as a function of the coefficient of repletion, K

respectively. The phase lag  $\epsilon$  may be found by the "current slack method" as first used by O'Brien and Dean<sup>4</sup> and by Jarrett.<sup>5</sup> In this method the time lags between high and low water in the ocean and slack water in the inlet entrance are found and then the duration of the flood and ebb flows determined. The phase lag for the flood,  $\epsilon_F$ , and ebb,  $\epsilon_E$ , are then found by dividing the time lag by the duration of the flood and ebb flows, respectively, and multiplying by  $180^\circ$ . These two values may then be averaged,  $\epsilon_{avg}$ , to give an average phase lag for the inlet. Figure 18 is then used to determine the K value. Jarrett<sup>5</sup> found that K for Masonboro Inlet, as determined by the current slack method, was 0.93 for the year 1969.

40. Current slack in the inlet may be determined in two ways. First would be the time the actual velocity curve reverses from ebb to flood or vice versa at the inlet throat. If velocity data are not available, the intersection of a bay tide curve for a tide gage near the entrance and the ocean tide curve could be used as a measure of slack water since there is no head difference across the inlet. However, this method should not be relied upon, since inertial effects often allow ebb or flood flows to continue after the time of zero head difference. Also, the intersection of the tide curves occurs when both

bay and ocean curves are very flat so that head differences are very small, which causes some error in determining the actual time of intersection of the curves. This variation can be one-half hour or greater. In terms of velocities it appears that the slack time can be estimated within 15 min, especially when data are taken at half-hour intervals.

41. The 1973 model data were used to determine  $K$  for the various closure conditions. Table 5 shows the times of low-water and high-water slack at range 2, the inlet entrance, and the calculated phase lags  $\epsilon_F$ ,  $\epsilon_E$ ,  $\epsilon_{avg}$  and the  $K$  value determined from Figure 18. Table 5 shows that  $K$  was reduced most for the BCC condition, i.e. from a base condition value of 0.74,  $K$  was reduced to 0.46. The SCC value was 0.51, and with MCC  $K = 0.63$ .

42. An examination of the expression for  $K$  indicates that the variables that could effect a change are  $A_C$ ,  $L$ ,  $m$ , and  $R$ . As noted by the variations in isovels,  $m$  does change, and the average  $R$  would change if a deeper or shallower channel is cut off; but  $m$  and  $R$  variations are small compared with the possible variations of  $L$  or  $A_C$ . A rule of thumb sometimes used to define channel length and cross-sectional area is to limit the bayward end of the channel at the point at which the main channel subdivides and to use an average area over this length. If this rule is applied in this case, there would be no change in the variables defining  $K$ . Therefore it is necessary to extend the inlet entrance region into the individual bay channels where a simple analysis by proportion shows that the changes in  $K$  are caused by the changes in channel cross-sectional area since  $K \propto A_C$ . Finding that

$$\begin{aligned} A_C \text{ (Masonboro Channel)} &= 4,300 \text{ sq ft} \\ A_C \text{ (Banks Channel)} &= 9,900 \text{ sq ft} \\ A_C \text{ (Shinn Creek)} &= \underline{8,300 \text{ sq ft}} \\ A_C \text{ (Total)} &= 22,500 \text{ sq ft} \end{aligned}$$

Then knowing  $K_{Base}$ , by proportion for  $K_{BCC}$ ,

$$K_{BCC} = (K_{Base}) \frac{A_C \text{ (Total)} - A_C \text{ (Banks Channel)}}{A_C \text{ (Total)}} = 0.41$$

In a similar manner,  $K_{MCC} = 0.60$  and  $K_{SCC} = 0.47$ . These values compare closely with the measured values determined from the model data in Table 5. It would also appear that the effective channel length might increase due to a channel closure as a result of the more circuitous route the flow must travel to fill a region behind a barrier. This is indicated in a later section.

43. The K value as determined from the ratio of bay tide range to ocean tide range (Figure 17) is shown in Table 6 and compared with the K's as determined by the current slack method. The average tide range of gages 6 and E was used to represent the entire bay range since these gages are somewhat midway between the inlet entrance and the bay extremes. The difference between the K values for the two methods is about the same for each test except for SCC, and in each case the K determined by the current slack method is smaller than that determined by the ratio of bay to ocean tide ranges. These differences are evidently due to the more channelized nature of the inlet when compared with the "ideal" inlet which is an open surface area with a uniform rise and fall of tide. The closure of a channel makes the model inlet's bay rise and fall even less uniform. Also, gages 6 and E are closer to the entrance than to the rear of the bay, where there would probably be a smaller tide range, though no data were taken at the far reaches of the bay for these tests.

#### Bay Superelevation

44. Bay superelevation, or bay setup as it may be identified, is defined here as the difference in the mean water level of the bay and the mean ocean level. For the case under study the setup is defined over the interval of one tidal cycle. Bay setup can be produced in a number of ways. Wave action which directs a shoreward component of momentum flux may cause a bay setup.<sup>1,6</sup> Freshwater inflow into the bay will cause a higher mean bay level than that of the ocean.<sup>2,7,8</sup> The variation of the entrance channel area during the tidal cycle can also produce a setup in the bay,<sup>2,7</sup> with the greater the difference in

cross-sectional area between high water and low water, the greater the setup. Wind also can cause bay setup in the bay or over a part of the bay.<sup>2</sup> Another and perhaps primary reason for the change in bay setup in this study would be the observed increase in superelevation with a reduction in  $K$ , which is caused by an absolute reduction in channel area and a lengthening of the channel. This effect has been observed in model studies by Mayor-Mora<sup>9</sup> and in numerical studies by Oliveira<sup>10</sup> and also by King.<sup>7</sup> Data from Mayor-Mora and Oliveira are plotted with results of this study in Figure 19. The reduction in  $K$  caused an

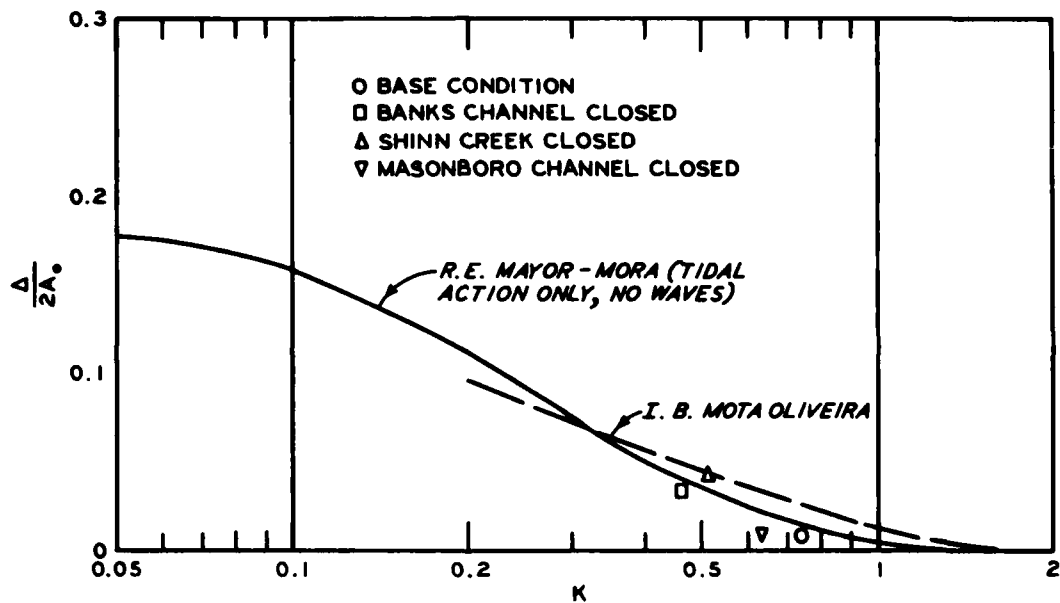


Figure 19. Ratio of bay superelevation to ocean tide range as a function of the coefficient of repletion,  $K$

increased bay superelevation because as  $K$  decreases, flood flow occurs during higher water levels and ebb flow at lower water levels. This can be seen in Figure 20 where the times of maximum head difference and thus the time of maximum currents are noted for various  $K$  values. The head difference between ocean and bay,  $Z$ , is evaluated from  $Z = a_1 \sin \theta + a_1 b_3 (\cos \theta - \cos 3\theta) + a_1 a_3 \sin 3\theta$  which is from Reference 2 where the various coefficients are defined for various  $K$  values. As  $K$  decreases, maximum currents occur nearer high and low water. For a balanced ebb and flood tidal prism there must be a longer ebb duration

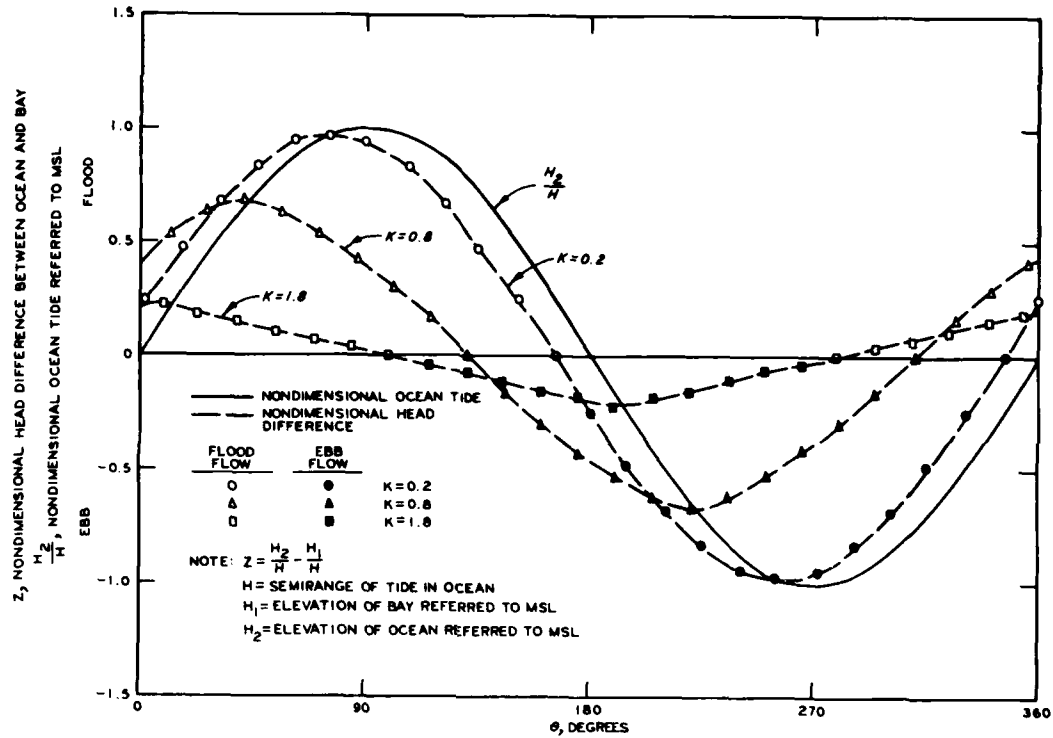


Figure 20. Ocean-bay head difference as a function of time for varying coefficient of repletion

to move the water mass through the smaller cross-sectional area occurring during ebb flow. Figure 21 shows ebb flow duration versus  $K$  from Mayor-Mora, Oliveira, and these tests. The relation between the longer ebb duration (caused by the decrease in  $K$ ) and the increased setup in the bay is shown in Figure 22, taken from Keulegan. An expression derived by Keulegan relating inflow duration to the bay superelevation is

$$\frac{\Delta}{H} = \frac{\sin \sigma t_i}{1 - \cos \sigma t_i}$$

where

$\Delta$  = superelevation, ft

$H$  = half-tide range, ft

$\sigma = 2\pi/T = 360^\circ/12.42 \text{ hr}$

$t_i$  = duration of inflow, hr

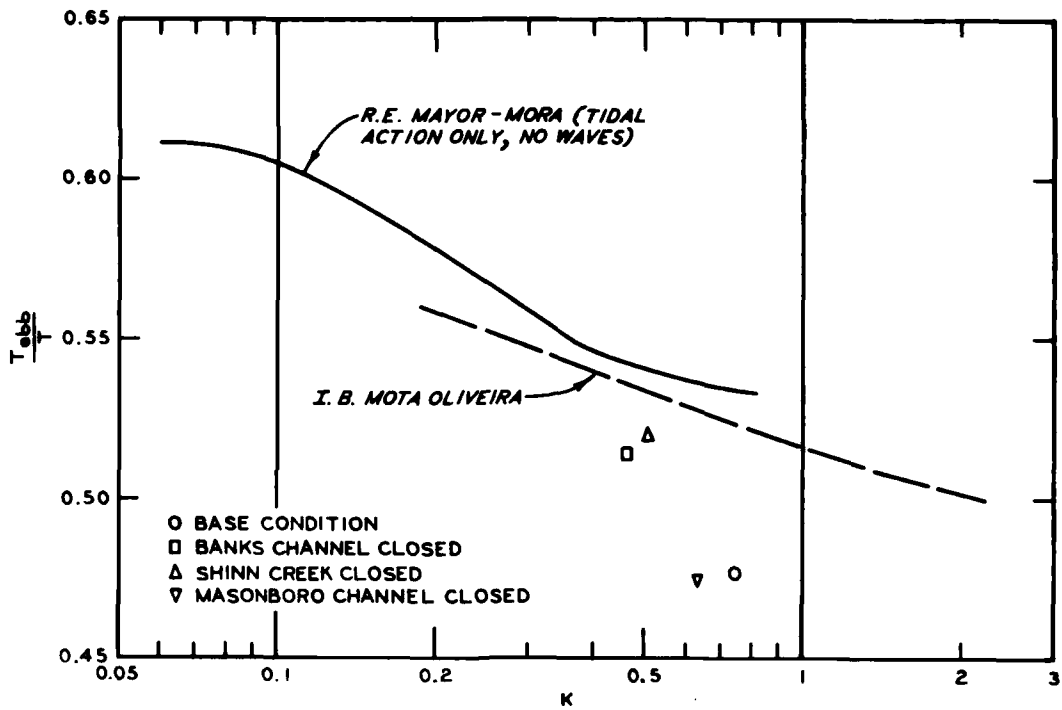


Figure 21. The ratio of the duration of ebb flow to tidal period as a function of the coefficient of repletion

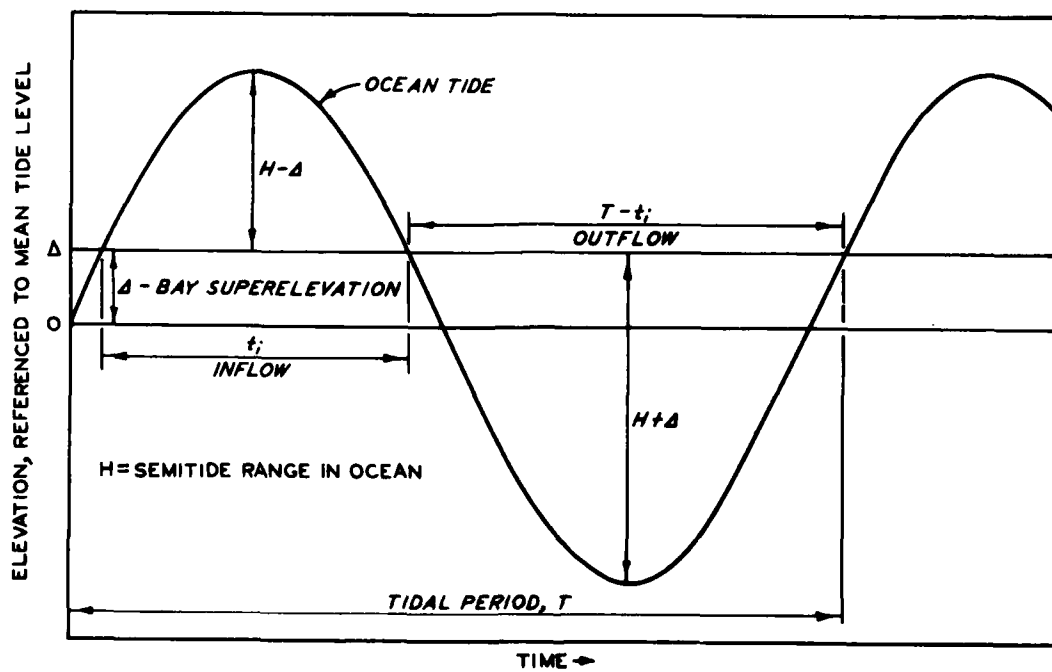


Figure 22. Relation between duration of outflow and bay super-elevation

Table 7 shows the calculated  $\Delta$  for each condition compared with the measured values using the average of the average tide elevation for the bay tide stations. From Table 7 the calculated superelevations were less than the measured ones, but were very close for BCC and SCC. The negative calculated superelevations for the base and for MCC were due to the fact that flood duration is greater than ebb duration for these conditions and the formula used then produces a setdown. The reason for a longer inflow for the base condition is probably based on the fact that due to the variation in bay area with tidal elevation, the greater the variation over the tidal cycle the longer the flood duration.<sup>11</sup> The variation of bay area with tide elevation at Masonboro Inlet is of such magnitude that this could be expected.

#### Inertial Effects

45. If an inertial effect is present there will be a difference in bay water elevation and ocean elevation when there is slack water at the entrance. Keulegan<sup>2</sup> derives an expression to determine this head difference from a one-dimensional equation of motion:

$$\frac{dV}{dt} = -g a_o \frac{dh}{dx} - \lambda \frac{V|V|}{r}$$

Integrating this along the length of channel  $L_x$ , and evaluating when  $V = 0$ , the differential head due to inertia reduces to:

$$H_B - H_O = \frac{V_{\max} \pi L_x}{gT}$$

where

$H_B - H_O$  = head due to inertia, ft

$V_{\max}$  = average maximum velocity in inlet, fps

$g = 32.2 \text{ ft/sec}^2$

$T$  = tidal period, sec

To evaluate the differential head, a channel length  $L_x$  has to be defined. Since  $V_{\max}$  was taken as the velocity at the inlet gorge

(range 2), the area at range 2 was used in the equation defining K and the other parameters were chosen as follows:

$$T = 44,712 \text{ sec}$$

$$a_o = 1.9 \text{ ft}$$

$$A_C = 12,450 \text{ ft}^2$$

$$A_B = 2.066 \times 10^8 \text{ ft}^2$$

$$R = 14.6 \text{ ft}$$

$$\lambda = 0.00743 \text{ (based on } n = 0.025)$$

Solving the equation for Keulegan's K in terms of L, an expression for the effective channel length is:

$$L = \frac{12,251}{.K^2} - 1965$$

Evaluating for the various K values, the calculated lengths are: for the base condition, L = 20,407 ft; for BCC, L = 55,931 ft; for SCC, L = 45,136 ft; for MCC, L = 28,902 ft (Table 8). The effective length with BCC and MCC more than doubled the base condition and with MCC the effective length increased by 42 percent. Using these effective lengths of channel,  $H_B - H_o = 0.13$  ft for the base condition, 0.28 ft for BCC, 0.24 ft for SCC, and 0.16 for MCC. These values are compared with observed model values of 0.15 ft for the base, 0.15 ft for BCC, 0.23 ft for SCC, and 0.20 for MCC (Table 9). The tidal elevations at gages 6 and E were averaged again to represent the bay tide. The measured values and calculated values compare fairly well. An interesting effect noted was that for the channel closure conditions the observed head difference between ocean and bay at high water was higher than that at low water by at least a factor of 3 (Table 9). The above expression for the inertial head as derived by Keulegan is based on a sinusoidal velocity curve, so it does not illustrate the difference in the inertial head difference at high water and at low water. After examining the model velocity curves at range 2 (Plates 8-10), it was noted that the difference in slopes of the velocity curves when the velocity is zero is less during the ebb to flood velocity transition than in the flood to ebb transition. However, an analysis for the inertial head difference

for these two conditions using the expression

$$(H_b - H_o) = \frac{L}{ga_o} \left( \frac{dV}{dt} \right)_{v=0}$$

proved to be less accurate than the previous method (Table 9). It was seen that  $L$  is the critical parameter for this determination and the necessity of evaluating it for each channel closure condition was noted because the base condition values of  $(dV/dt)_{v=0}$  were greater than for the channel closure condition values. Therefore  $L$  must be increased for the closure conditions if the calculated results are to agree with the model observations since  $g$  and  $a_o$  are constant values. Therefore the effective length of the inlet was increased for each channel closure condition based on the observation of increased inertial head and analysis of the velocity curves.

46. From Table 9 it is noted that the inertial head difference is increased for the channel closure conditions when compared with the base. This is possibly related to the later occurrence of high water in the bay, i.e., the reduction in  $K$ , at which time the ocean level has fallen a greater amount than for the earlier high-water base condition, thus creating a greater head difference for the channel closure conditions. From Table 9 it is seen that the inertial head of the channel closure condition is lower than the base condition at low water. Due to the phase lag in low water for the channel closure conditions, the bay low water occurs later than the base condition with respect to the time of the ocean low water, with the result that the rising ocean tide is at a higher level for the channel closure conditions and thus the head difference between bay and ocean is smaller.

#### Prototype Channel Closure

47. In 1947 a sand dike was built across Shinn Creek at its junction with the AIWW as a part of a program to alleviate erosion along the Wrightsville Beach shoulder of the inlet and to prevent the growth of a spit into Masonboro Channel as seen in Figure 23a, a photograph



a. 23 JAN 1945



b. 15 NOV 1949



c. 31 MAY 1953



d. 30 NOV 1954



e. 25 MAR 1956



f. 16 AUG. 1959

Figure 23. Masonboro Inlet 1945-1959

of the inlet in January 1945. The effect of this dike is seen in Figure 23b, a November 1949 photograph. There was a widening of the inlet on the Masonboro Beach side, a buildup on the Wrightsville Beach shoulder, and an inner bar buildup inside the entrance. By May 1953 (Figure 23c) the inlet actually bifurcated into two separate inlets. In 1954 Hurricane Hazel washed open the sand dike (Figure 23d). By March 1956 (Figure 23e) Shinn Creek had cut through the inner bar. In 1959, when the main channel was dredged, the dredged material was placed across the Masonboro Channel inlet (Figure 23f).

48. The effects of closing Shinn Creek as seen in the model pointed out that ebb flow would be directed toward Masonboro Beach (Plates 22 and 23) and flows through Masonboro Channel would be increased (Plate 28). Plate 31 shows the head differences throughout a tidal cycle in the model between the channel entrance (gage B) and gages 3-5. The greatest change in head difference occurs in Masonboro Channel between gages B and 5 for the Shinn Creek closed condition. Figure 23b shows that increases in the effect of flow through Masonboro Channel on the entrance region probably increased with time as the inner bar built up and deflected Banks Channel ebb flow in a direction more nearly perpendicular to the beachline, perhaps even reducing flood flow into Banks Channel through Masonboro Channel. A stability analysis as performed by Escoffier<sup>12</sup> was applied to Masonboro Inlet and it was found that  $K_{\text{CRITICAL}}$ , i.e. the critical Keulegan K value, was 0.64. This means that when the K value falls below this value the inlet is an unstable state and shoaling will probably occur. As noted from the testing, closing either Banks Channel or Shinn Creek will cause the K value to fall below 0.64. Thus instability is expected for either of these closures and the prototype closure of Shinn Creek bears this out.

49. Having examined the results of the closure of Shinn Creek, it might be of interest to hypothesize the effects of closing either Banks Channel or Masonboro Channel on the inlet and bay morphology. For example, with Banks Channel closed, material would enter the Banks Channel entrance and fill in the immediate vicinity of the mouth. This

fill probably would not have as much effect on changing current patterns as the middle ground shoal that formed when Shinn Creek was closed (Figure 23b). The region near gage 6 in the AIWW would be scoured out by faster currents rather than being a depository for material due to the eddylike currents that exist there now over a major portion of the tidal cycle. At the inlet entrance, the highest ebb and flood currents for BCC exist at the north shoulder, so possibly some scour would occur there. A reduction in velocity at the south shoulder might permit greater accumulation of sand there. With Masonboro Channel closed, there would probably be a buildup somewhat like Figure 23a with a spit building across the channel. The inlet throat would probably then orient itself at a downcoast angle with the beachline.

#### Conclusions

50. The closure of any of the three interior channels in Masonboro Inlet produces a significant change in the inlet hydraulics and would most likely produce a significant change in morphology as illustrated by the prototype case history. The important changes in flow patterns at the inlet entrance occur for the ebb flow, with flood flow patterns at range 2 staying fairly constant. Keulegan K values show decreases for each closure condition, with closure of Banks Channel (the largest interior channel) providing the greatest decrease. These decreases in K are the result of a reduction in flow area and an increase in the effective length of the inlet for each closure condition. This in turn increases the bay superelevation because of the reduction in K. Also the inertial head increases at high water and decreases at low water with respect to base condition for each channel closure which is associated with the increased bay tide phase lag because of lower K values.

51. Shinn Creek, the interior channel perpendicular to the coast, aids in directing ebb flow oceanward, perpendicular to the coast as seen in evaluating flow volumes through range 1 for the various conditions. This indicates that the initial scour along the outer portion of the

north jetty (Figure 24 for the June 1967 to January 1968 period) could have been caused by this ebb flow effect. This result must also be coupled with wave effects which probably contributed to filling the dredged channel location, also shown in Figure 24. It is possible that considering the effect of Shinn Creek on ebb flow orientation, it would be preferable if the north jetty was oriented perpendicular to the shoreline, rather than having a dogleg in the downcoast direction. This flow orientation was also noted in testing the 1964 prejetty inlet as seen in Photo 17 showing surface current patterns during ebb flow. However, it is almost certain that even with a perpendicular orientation, the channel would have migrated to the jetty, which Kieslich and Mason<sup>13</sup> documented and which they attribute to wave-induced processes.

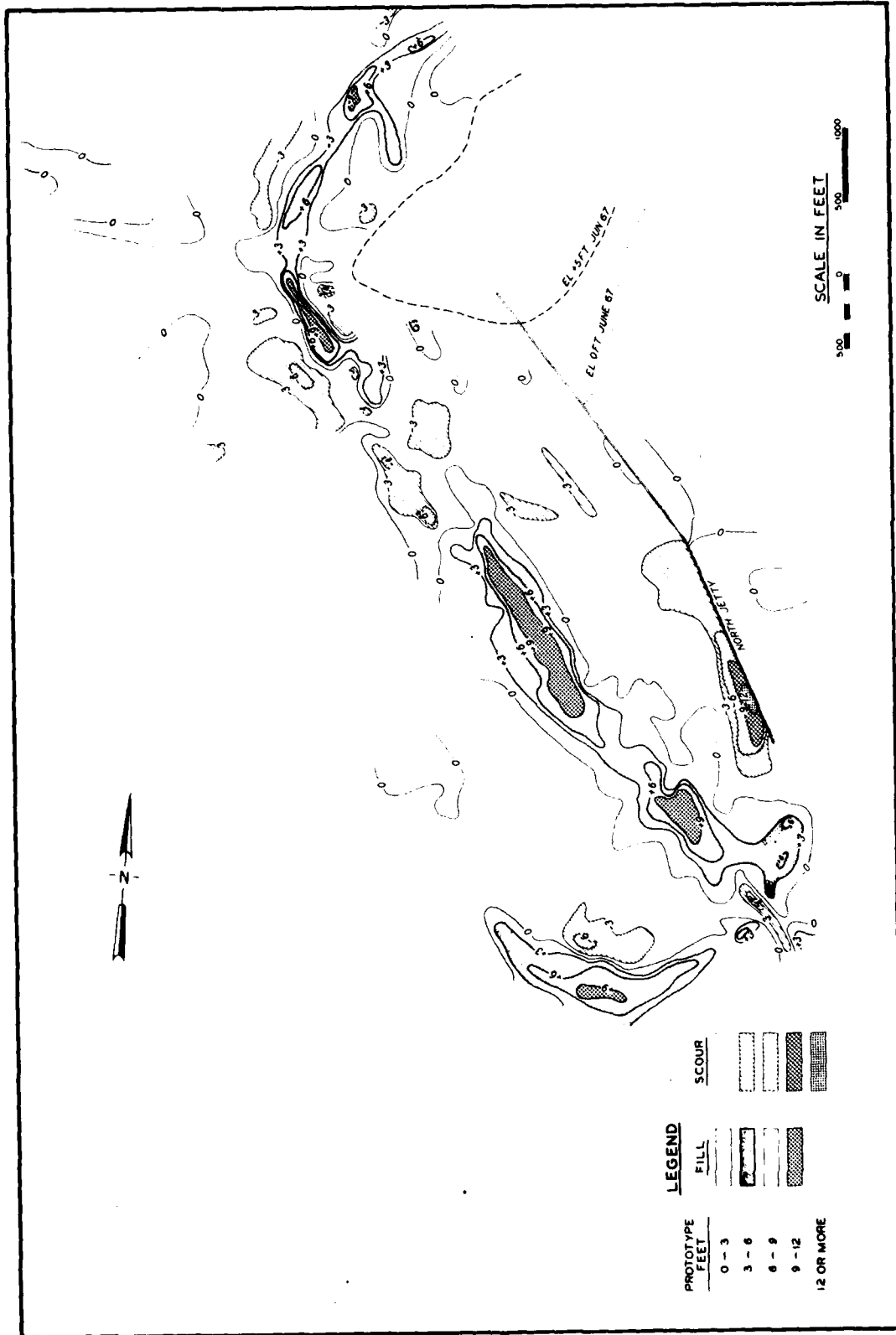


Figure 24. Scour and fill, June 1967 to January 1968

PART III: WEIR JETTIES TESTS

Introduction

52. These tests were designed to examine the effects of weir jetties on the inlet's hydraulics. The existing condition in the prototype has a north jetty with a weir section at el +2.0 mlw. A test program for the Wilmington District was performed to select the best alignment for a future jetty on the south side of the inlet (Figure 25). Since the south jetty did not include a weir, results of the

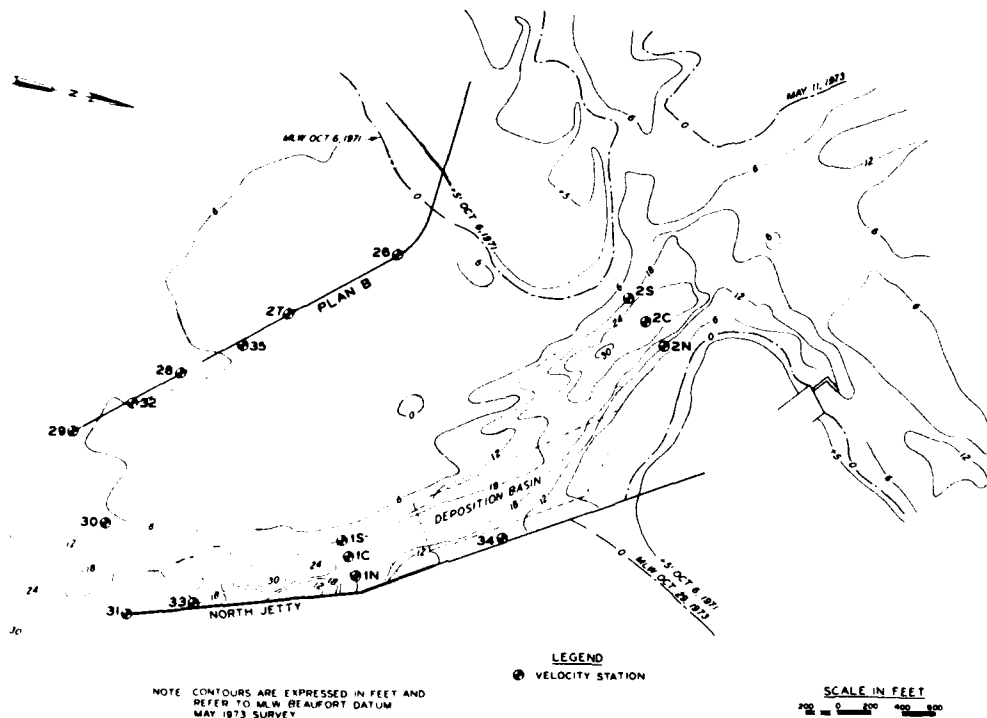


Figure 25. Masonboro Inlet with south jetty

Wilmington District tests could be compared with the data taken for various weir configurations in the supplementary tests. First a +2.0 mlw weir was added to the south jetty, then the weirs on both jetties were lowered to 0.0 mlw. For each of these cases a set of tidal

heights, velocities, discharges and flow volumes, and surface current photographs were collected. Gaging locations are shown in Figures 14 and 25.

#### Background

53. Weir jetties were originally designed for hydraulic purposes. Flood currents flowed over two weirs on either side of the inlet between the shoreline and offshore rubble-mound sections and were elevated slightly above the bottom. The weirs confined ebb flow largely to the channel, producing a predominant ebb flow between the jetties which aided in maintaining the navigation channel. This design at Charleston Harbor Entrance, South Carolina, produced average maximum ebb velocities 60 percent greater than flood, and the channel was naturally maintained between 1896 and 1936, when increased depths for navigation were required. Later designs of weir jetties for sand bypassing originated from observation of a natural condition existing at Hillsboro Inlet, Florida, where sand was washed over a reef adjacent to the inlet entrance and deposited in protected waters where it could be easily dredged. This concept was extended to Masonboro Inlet and later to others, such as Ponce de Leon Inlet, East Pass, Perdido Pass (all in Florida) and Rudee Inlet, Virginia. At Masonboro Inlet the weir section in the north jetty is nearly perpendicular to the shoreline and extends approximately 1000 ft oceanward from the mlw line. It appears that most material transport in the vicinity of the weir is along the shoreward end, but the extra length of weir is necessary to prevent a "landlocked" condition from occurring. The viability of the weir system is not universally accepted. At Masonboro Inlet it can be shown that the weir worked well initially, but migration of the channel into the deposition basin eliminated it as a depository for sand. It should be noted that the channel migration into the basin was most likely a phenomenon of the single jettied system as discussed in PART II rather than an effect of the weir and/or basin.

### Testing

54. The model was run for only one tidal condition, the mean tide, with a 3.8-ft range. The tidal height and velocity plots show four sets of data and each curve is identified as follows:

- a. Base = Natural 1973 model condition (including north weir jetty, with weir at el +2.0 mlw)
- b. Plan 1 = Base plus south jetty (no weir on south jetty)
- c. Plan 2 = Base plus south jetty with a +2.0 mlw weir
- d. Plan 3 = North and south jetties with 0.0 mlw weirs

### Tidal Heights

55. Plates 32-35 indicate that tide heights at gages 1, B, 2, 3, 6, and E showed the most change for these tests. These gages are along the middle of the inlet area. Gages 4 and 5 in the flanking channels did not show as much change. Gage 1, which is between the oceanward tip of the jetties and the weir section, showed reductions in high and low water for plans 1 and 2, then with the 0.0 mlw weirs of plan 3 the tide curve returned to near that of the base condition. Comparison of gages 3-5 (Plate 34) shows that gage 3 (Shinn Creek) elevations were higher for each plan. This illustrates the increased concentration of flow by the jetties toward the center of the entrance channel and then into Shinn Creek.

56. Tables 10 and 11 summarize the effect of each plan on tide range and mean tide level (mtl). In Table 10 the average tide range for all gages within the confines of the jetties shows that plan 1 resulted in the smallest average tidal range, 3.43 ft, a reduction of 0.8 ft from the base condition. When a weir was added to the south jetty (plan 2), the average range increased to 3.46 ft, then with 0.0 mlw weirs the range approached the base condition with a range of 3.49 ft. The average mtl shows an interesting change also. For the base condition the average mtl is +0.05 ft msl; for plan 1, 0.00 ft; plan 2, +0.06 ft; and for plan 3, +0.14 ft. Once the weir section on the south jetty

is installed, the average level increases and for the 0.0 msl weir section the increase is 0.09 ft above the base. This last increase is most likely due to the fact that the 0.0 mlw weirs permit the greatest flood flow into the jetties-bay system, while restricting the ebb flow to the channel between the jetties. Since the jetties increase the effective length of the inlet during ebb flow, an increased bay superelevation results. Table 11 shows the change of mtl's from the base for each plan condition.

#### Velocities, Discharges, and Flow Volumes

57. Velocity measurements are presented in Plates 36-51, the results of discharge and flow volume computations are presented in Plates 52-56, and cross-sectional plots of maximum ebb and flood isovels at ranges 1 and 2 are presented in Plates 57-60. Velocities at range 1, near the north jetty (Plates 36-38) and discharges (Plate 52) indicate that ebb flows were increased over the base condition for all cases because of the confinement of flow by the addition of a south jetty. Flood flows at this location increased over the base because of confinement of flood currents for both the nonweir south jetty and the +2.0 mlw weir on both jetties; but when weirs on both jetties were reduced in elevation to mlw, flood currents and discharges dropped to or below the level of the base condition due to the increased flood flow over the weirs. Ebb isovels at range 1 (Plate 57) show higher velocities at the surface stations of C and S for all the south jetty conditions. Flood isovels (Plate 58) show increases for plans 1 and 2, but for the 0.0 mlw weirs (plan 3), the velocity distribution becomes somewhat similar to the base condition. At range 2, velocities (Plates 39-41) showed the greatest variation from the base at sta 2N. Discharges and flow volumes (Plate 53) show that the average tidal prisms were relatively close for each condition: base condition, 566 million cubic feet; plan 1, 556 million cubic feet; plan 2, 587 million cubic feet; plan 3, 546 million cubic feet; all within the 10 percent possible error of measurement. Ebb isovels (Plate 59) indicate a slight increase

of flow at sta 2N surface, especially with the 0.0 mlw weir. Flood isovels (Plate 60) remain very similar for all conditions.

58. Because of a failure in the data acquisition system which was not discovered until after completion of these tests, there are no reliable velocity data at range 3 (Shinn Creek) for plan 3. Bay velocities at ranges 3-5 (Plates 42-49) and discharge and flow volume data at these locations (Plates 54-56) indicate that plans 1 and 2 increased velocities, discharges, and flow volumes through range 3 (Shinn Creek). All plans reduced flow through range 4 (Banks Channel), with range 5 (Masonboro Channel) showing little change. Plan 3 velocities at range 3, determined from surface current photographs, were on the average slightly higher for the plan conditions than for the base.

59. Velocities at sta 29-31 and 35 near the outer ends of the jetties (see Figure 25 for locations) are shown in Plates 50 and 51. No base data were taken at these locations. At sta 29 (the south jetty tip) surface ebb velocities were significantly increased by addition of the weir section to the south jetty. Surface ebb velocities at sta 30 and 31 showed little change, but sta 35 showed slightly higher bottom ebb velocities, although between hours 2 and 9 this is actually an eddy region during flood flow.

#### Keulegan's K

60. K values were computed by the current-slack method and are shown in Table 12. The range of K was very small, from 0.74 (base condition) to 0.70 (plan 3). A reduction in K would be expected since the confining effect of a south jetty would tend to make the effective length of the inlet greater, and K is inversely proportional to the length. Among the three dual jettied conditions it might seem surprising that the plan 3 system, which has the greatest flow area (including flow over the mlw weirs), might have the smallest K value. However, the reduction in K for plan 3 is supported by the increased bay superelevation (Table 10) for this plan. This increased superelevation is most likely caused by the variation in flow cross section

during the tidal cycle (see Keulegan<sup>2</sup> or King<sup>7</sup>). Plan 3 probably has the greatest effective flow area variation because of the 0.0 mlw weirs. The word "effective" is applied here because though the flow area at the weirs and at the entrance between the jetties is constant at a given water level for plan 3, only the flow area between the jetties is the primary flow region during ebb flow due to the concentrated ebb jet. Thus plan 3 would have the greatest flow area into the inlet during flood flow and roughly the same flow area as plan 2 for the ebb flow.

#### Surface Current Photographs

61. Surface current pattern photographs for the base condition and plans 1-3 are Photos 1-4, 18-23, 24-29, and 30-35, respectively. Times of the photographs for plans 1-3 are as follows: hour 4, early flood flow; hour 7, peak flood flow; hour 9, start of ebb flow; hour 10, early ebb flow; hour 11, peak ebb flow; hour 0, late ebb flow. Base condition photographs are presented for hours 4, 7, 10, and 0.

62. At hour 4 (Photos 18, 24, and 30) the main difference between the plans is that for plan 3 there is flow over the 0.0 mlw weirs with velocities of about 1.5 fps. At hour 7 (Photos 19, 25, and 31) flood flow toward the inlet over the weirs is occurring for all plans. Velocities over the 0.0 mlw weirs (plan 3) are about 2 fps and for the +2 mlw weirs, 1.6 to 1.8 fps. Thus, there is not a significant difference in maximum flood velocities over the weir sections for the different weir elevations. These velocities over the weir are somewhat greater than the base condition (Photo 2) which has just the single north weir jetty, and is thus open to the ocean on both sides of the weir. With two jetties there is a greater drawdown over each weir on flood flow due to a greater head difference between the ocean and the confined region between the jetties.

63. At hour 9, during the initiation of ebb velocities, there is ebb flow over the jetties in every case (Photos 20, 26, and 32). One hour later, at hour 10 (Photos 21, 27, and 33), flow over the weirs at +2.0 mlw (plans 1 and 2) has stopped since the water has almost dropped

to this level, but for plan 3 (the 0.0 mlw weir) flow over the weirs is still occurring. At hour 11 (Photos 22, 28, and 34) there is still flow over the 0.0 mlw weirs of plan 3. The existence of ebb flow over the 0.0 mlw weirs appears to concentrate the flow that remains between the jetties adjacent to the inner walls of both north and south jetties (compare Photo 34 with Photos 28 and 22). Detailed velocities at range 1 indicate this primarily at sta 1N, surface. Also the trajectories of the flow lines in the middle of the jetties toward the oceanward end are not exiting as straight for plan 3 as plans 2 or 1, though the existence of a shoal in the midregion between the jetties and its deflection of flow lines make this comparison difficult to evaluate. At hour 0 (Photos 23, 29, and 35) the flow appears to be concentrated closer to the north jetty for plan 3 (Photo 35) than for the others, similar to hour 11 photographs.

#### Conclusions

64. The effect of a south jetty is to centralize the flood flow through the inlet gorge, and the presence of a weir on the south jetty does not change this basic flow pattern. As a result of this flow pattern, flow increases through Shinn Creek, the channel directly behind the inlet gorge. Flood velocities over the 0.0 mlw weirs are slightly greater than those over the +2.0 weir jetties (0.2 to 0.4 fps greater). However, the flow volume coming over the 0.0 mlw weir sections is substantially greater, which slightly reduces flow entering between the jetties at their oceanward end. Mean tide range measurements in the bay show that the 0.0 mlw weir sections bring the average tide range closest to the base condition; however, the reduction in tide range for the most constricted condition, the nonweir south jetty, is only 0.08 ft.

65. Ebb velocities seem to be better centralized between the jetties for the +2.0 mlw weirs and the nonweir south jetty conditions than for the 0.0 mlw weirs. Flow over the +2.0 mlw weirs cuts off very early during ebb flow since the tide level has dropped to just below

the mtl once ebb flow has begun to reach faster velocities. For the 0.0 mlw weir condition, however, ebb flow over the weirs continues until nearly low water. The continued drawdown of ebb flow over the 0.0 mlw weirs aids in dispersing the flow over the entire region between the jetties and seemingly could aid currents to flow along the inner walls of the jetties rather than concentrating in the central region between the jetties where the navigation channel is located.

66. Plan 3 (dual weirs at 0.0 mlw) produced a much greater ebb velocity predominance between the jetties than did plan 2 (dual weirs at +2 mlw). At range 1 and sta 30 the ratio of maximum ebb velocity to maximum flood velocity for plan 3 was about 2.1, while for plan 2 this ratio was only 1.2 at range 1 and 1.6 at sta 30. Since sediment transport is an exponential function of velocity, an excellent potential for natural scour of the channel between the jetties exists for plan 3, similar to the Charleston Harbor Entrance condition previously mentioned. However, maintenance dredging cannot be completely eliminated since sediment transported over the weirs is likely to be deposited in quiescent interior regions of the inlet and bay.

67. There was a slight reduction in Keulegan K values for both the dual weir jetty conditions when compared with the base condition.

68. Under actual prototype conditions, other factors would need to be considered in evaluating low-water weir elevations against mtl weirs. For example, inlets with very small K values would have different time relations of when flow would occur over the weir if it was at a mtl elevation. Also, the effect of waves and the amount of protection needed from wave activity within the jetty system would be of importance in determining the proper weir elevation.

#### PART IV: SEDIMENT TRACER TESTING

69. Fixed-bed shoaling tests or sediment tracer tests, as they can also be identified, have been used in tidal model studies for many years. Essentially they involve the placement of some small-grained material on the concrete model bed which is subjected to the action of tidal currents, and/or wind waves. These tests have been carried out using a variety of modeling techniques depending upon whether the study region of interest is in a sheltered bay where tidal and density currents are the dominant transport mechanism and the prototype sediment might be lightweight silt, or in a tidal inlet entrance where the prototype sediment, sand and shell fragments, are transported by both tidal currents and waves, or along a beach where waves are the dominant transporting force.

70. In order to conduct fixed-bed shoaling tests, many variables must be evaluated. These include shape, size, and specific weight of the modeling sediment tracer; method, location, duration, and quantity of sediment injection; height, period, direction, and duration of waves, if used; magnitude of tide range, spring or neap, for example, or perhaps artificially distorted; and duration of the test. Some examples of previous types of studies follow.

71. In a study of Brunswick Harbor and Jekyll Creek, Georgia,<sup>14</sup> an interior estuarine channel, gilsonite (an asphaltic material with a specific gravity (sp gr) of 1.035) was injected uniformly along a channel through overhead perforated pipes as a slurry of 10 percent gilsonite and 90 percent water. A total of 12,000 cc was injected over 3 tidal periods, then the model was operated for 20 additional tidal cycles, permitting the tidal currents time to move and deposit the sediment. Quantities of material accumulating in different regions of the channel were measured and the percent of shoaling material in these regions was calculated to determine shoaling trends.

72. In a study of the Grays Harbor<sup>15</sup> estuary entrance the entire region between the two jetties was subdivided by a grid system and the sediment tracer, 100,000 cc of a nylon material (sp gr 1.14,  $D_{50} \sim 3$  mm),

was introduced along a line at various times over five tidal cycles across the entrance channel and shoals. The material was then subjected to tidal currents and waves and its distribution on the model bed determined by measuring the quantity of material accumulated in each grid block. Also in model studies of the Columbia River<sup>16</sup> a lightweight plastic (sp gr 1.08) was placed as a thin layer over the bottom in the entrance area before model operation, then during operation additional material was injected at point locations. One other type tracer study was performed on the Umpqua River Entrance<sup>17</sup> in which the model was molded about 0.02 ft lower than the natural bathymetry, then a veneer of coal was placed on this concrete base bringing the bottom up to grade. This type of test was almost a three-dimensional movable-bed test.

73. The previously discussed tests which combined tides and waves were confined to regions between the jetties. The Masonboro Inlet test, due to the existence of only a single jetty on one side of the inlet, was a more open system which permitted the influx of longshore currents into the tidal current system, so that these tests might be considered a variation of previous sediment tracer testing.

74. The sediment tracer tests were initiated in order to "observe movement patterns" of material introduced on the fixed-bed model surface subjected to wave and tidal current forces--in effect, to trace its movement, hence - tracer tests. This test series evolved into a "verification" of filling and scouring regions which were identified from the prototype by preparing fill and scour maps from successive historical survey maps covering the inlet throat oceanward to the extent just beyond the north jetty. It was anticipated that short-term trends in fill and scour might be predictable with the technique, and if nothing else, the patterns of movement of material during ebb and flood tidal currents with the influence of ocean wind waves could be examined to define mechanisms of sediment circulations in the vicinity of a tidal inlet--a region of dynamic complexity.

75. Certain considerations must be kept in mind for testing of this nature. For example, it is not possible to simultaneously model refraction and diffraction in a distorted-scale model. Refraction is

depth-dependent and as a result wavelength is scaled by the vertical scale. Diffraction effects are correctly scaled by the horizontal dimension and so wavelength would be scaled horizontally. For this study, refraction was correctly scaled and it is believed that effects of diffraction distortion are small.

76. Since only a thin veneer is placed on the model surface, sediment movement trends can probably only be predicted in a qualitative manner. For example, since there is only a limited amount of movable material in a region, if the region scours until it reaches the fixed-model bed, only this amount of material can cause fill in another region. In nature, the initial scouring region may degrade a considerable amount more and thus cause a larger fill in another region. Also, the fixed-bed maintains existing wave refraction patterns with only minor changes at regions of sediment tracer accumulation.

77. Tracer tests were initiated to determine patterns of material movement in and around the inlet. This portion of the study evolved into an effort to duplicate shoaling patterns for a two-year prototype period while reproducing tidal currents and a monochromatic wave field.

#### Introductory Testing

78. Four materials were tested to determine if patterns of movement could be simulated in the model. They were:

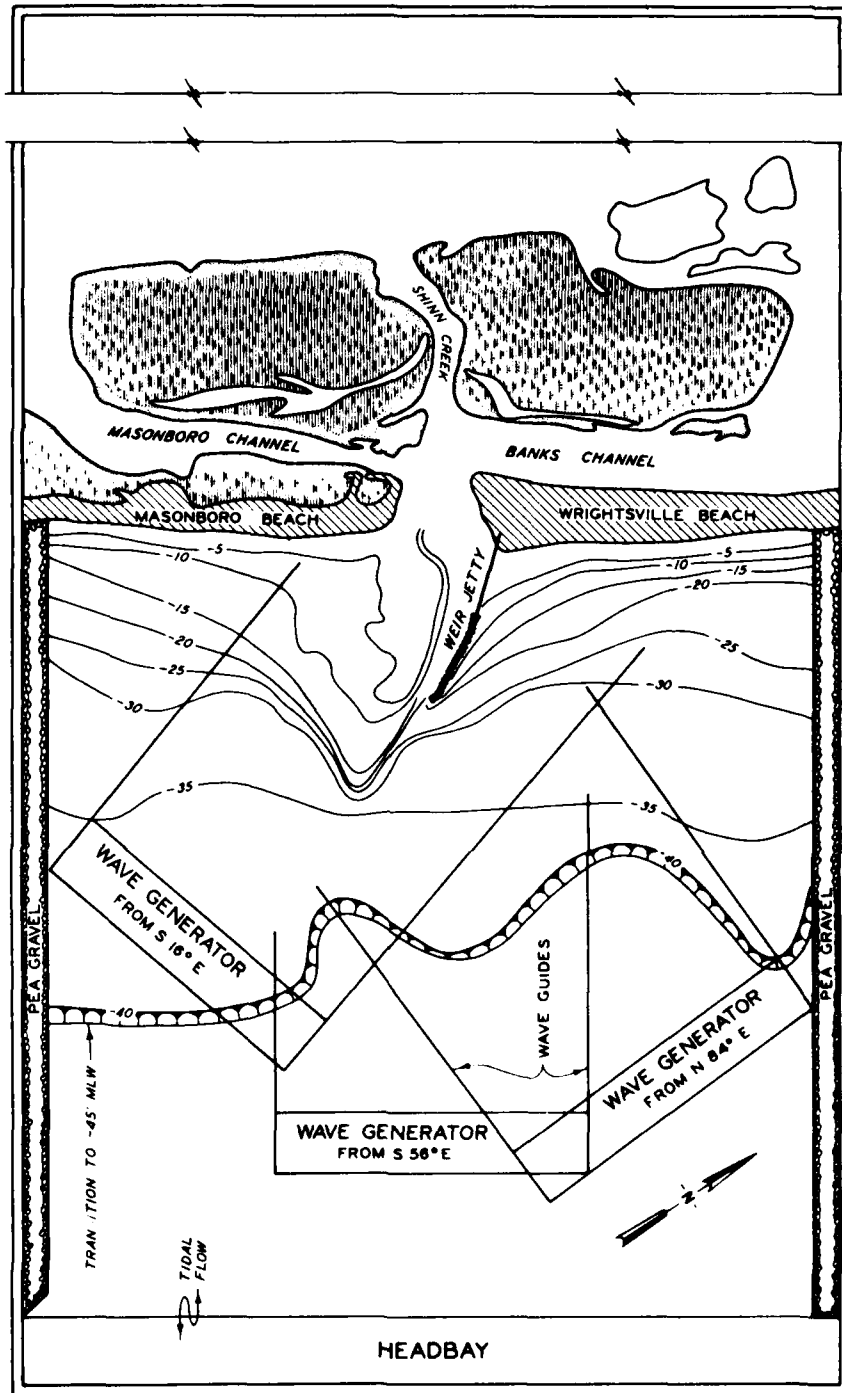
<u>Material</u>	<u>Specific Gravity</u>	<u>Shape of Grains</u>	<u>Median Diameter Size, mm</u>
Quartz sand	2.65	Angular	0.23
Expanded shale	2.05	Rounded	1.5
Naturalite (ground pumice)	1.7	Angular	0.95
Tenite butyrate plastic	1.18	Cubical	3 mm on a side

A detailed summary of the properties of these materials is shown in Appendix A, including gradation curves, fall velocities, and angle of repose.

79. A 3-ft-high, 7.4-sec-period wave generated from S56°E (for locations of wave machine, see Figure 26) was selected for the first test. Movement of the sediment on the model bed resulting from waves without effects of tides was observed, and similar results were obtained with all four materials; however, with sand and expanded shale, higher waves and a longer time were needed to obtain results similar to that of the plastic. Material moved toward the inlet and along Masonboro Beach over a 1500-ft-long section adjacent to the inlet and along Wrightsville Beach over a 600-ft-long section adjacent to the inlet. Beyond these points, material moved away from the inlet on both beaches. Movement of material over the landward edge of the weir into the north side of the inlet gorge occurred with each material.

80. Next, two series of tests were conducted with a mean tide (3.8 ft) and 3-ft-high, 7.4-sec-period waves from S56°E. Expanded shale (sp gr 2.05) was used for the first test series. This material was placed uniformly over the ocean bar at high water of the tidal cycle. Waves were then introduced. After five tidal cycles, patterns of material movement were observed and found to be similar to patterns in the tests without the tide in that some material moved upbeach, some moved downbeach, and some moved toward the gorge of the inlet. The ocean shoal was increased by material brought in from areas farther oceanward.

81. For the second test, the Tenite butyrate plastic (sp gr 1.18) was used as the bed material. Because of its rapid movement, the plastic material was placed uniformly on the surface of the model at high-water slack, and the resulting movement was observed until low-water slack. The plastic material was then uniformly placed on the model surface at low-water slack and the resulting movement observed through high-water slack. Results of these tests indicated that, during the flood flow, material moved from Masonboro Beach to the south side of the inlet gorge; material from the south shoal moved to the south side of the jetty channel; and material from Wrightsville Beach moved over the shoreward end of the weir to the north side of the inlet gorge. During the ebb phase, material did not move toward the gorge of the inlet but built up on the south shoal.



NOTE. CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MLW  
 VERTICAL SCALE OF MODEL 1:80  
 HORIZONTAL SCALE OF MODEL 1:300  
 THERE IS ADEQUATE CLEARANCE BETWEEN WAVE GENERATOR  
 AND MODEL FLOOR TO PERMIT TIDAL FLOW

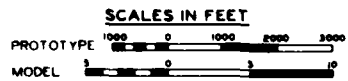


Figure 26. Wave generator locations, facility B

82. Waves were next introduced from a  $S16^{\circ}E$  direction. Sediment movement tests with waves and no tide were conducted using the plastic (sp gr 1.18) material. Results showed a net movement of material into the channel and over the weir to the north side of the inlet.

83. For the sediment tests with waves from  $S16^{\circ}E$  and a 3.8-ft tide, plastic and the pumice material known as naturalite (sp gr 1.7) were used. Results with plastic were similar to the tests using plastic with a 3.8-ft tide and waves from  $S56^{\circ}E$ , except that there was more movement of material into the channel along the jetty and less movement of material over the weir into the north side of the inlet. Tests using naturalite with the waves and a 3.8-ft tide showed a net movement of material from the south shoal into the channel. Enough material moved into the channel to significantly affect the tide control mechanism, which indicated that the tidal prism was decreased. A qualitative indication of fill and scour patterns is shown in Plate 61.

84. The next test was conducted to determine patterns of material movement in and around the inlet with waves approaching from  $N84^{\circ}E$  and using naturalite. After insertion of the material at slack water, the model was operated for eight tidal cycles with 3-ft-high, 7.4-sec-period waves used continuously. Results showed a movement pattern similar to that produced by the  $S16^{\circ}E$  waves. The quantity of material was less, but it was moved toward the channel. The main deposition area for the material on the south shoal was farther oceanward along the channel than for the  $S16^{\circ}E$  test. A qualitative fill and scour map (Plate 62) shows results of this test.

#### Prototype Data

85. At this point in the shoaling test program, it was decided to use a semiquantitative analysis procedure and to try to examine sediment movement trends for a two-year prototype period.

86. The verification period selected for the fill and scour adjustment of the model was the two years between prototype surveys of September 1969 and August 1971 (Figure 27). To facilitate a direct



Figure 27. Prototype scour and fill, September 1969 to August 1971

comparison for the verification tests, the model grid (described in paragraph 88) was superimposed on the prototype scour and fill map. Since each block may have some fill and some scour, it was decided to delineate blocks of significant scour or fill and blocks with approximately equal amounts of scour or fill. A map showing this delineation for the prototype for the 1969-1971 time period is shown in Plate 63. Small revisions to this map will be given in descriptions of tests to follow.

#### Procedure and Testing

87. It was necessary to choose one of the four sediment materials that would move and deposit under the influence of model forces in the same manner that the natural sediments move and deposit under the influence of natural forces. Significant variables expected to be encountered during verification of this model were: (a) shape, size, gradation, and specific weight of the artificial sediment; (b) method, location, duration, and quantity of artificial sediment injection; (c) height and direction of waves; and (d) length of model operation. Each of these variables had to be resolved.

88. The model grid system was redesigned as shown in Figure 28 to follow the natural contours of the inlet region, so that each block is an area of essentially all scour or fill (with respect to the prototype). Each block has an identifying number in the upper right corner. The amount of material initially placed in each block was directly proportioned to its area so that volume of material per unit of model area was constant. Material quantities (Figure 28) are given in millilitres, and the average depth of material spread over the study area was 0.018 ft. Placement of the material was done by hand, as in previous tests, during slack high or low water and with the wave generator turned off. After placing the material, wave action was initiated and continued through a predetermined schedule; then the model operation was stopped and the material from each section was recovered and measured volumetrically (Figure 29 shows the study area with material in place before testing).

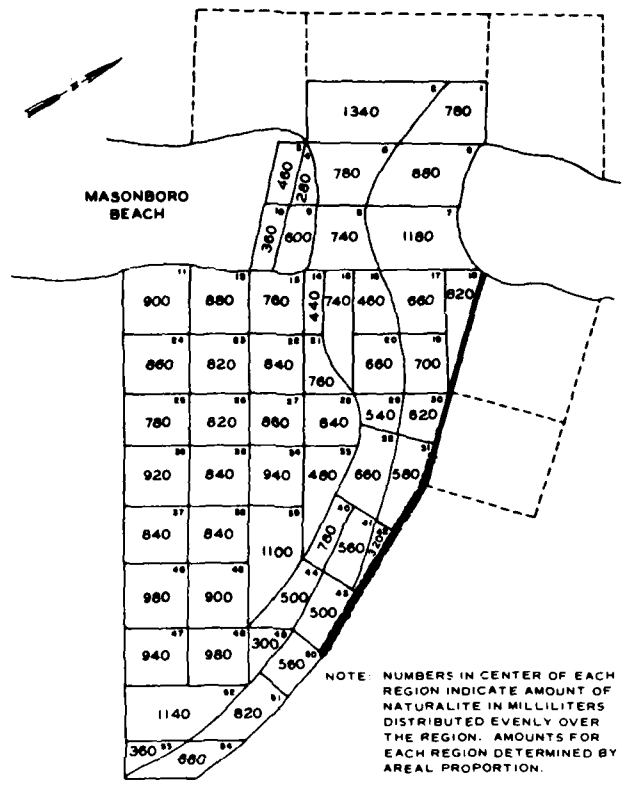


Figure 28. Material placement for tests 1-5

Figure 29. Typical placement of naturalite material before testing



89. The procedure described in paragraph 88 involved 30 tests. Tests 6, 7, and 8 concerned south jetty tests and are described in Reference 18; tests 15, 29, and 30 were tests to check repeatability of the model test results; test 22 conditions were altered during the test because of a malfunction in machinery before the test could be completed as planned. Each test is described in detail below and is summarized in Table 13.

#### Tests 1-5

90. Naturalite was the bed material used for tests 1-5. Tests 1-3 were run with the mean tide (3.8-ft range) and a 3-ft-high wave with a 7.4-sec period; the duration of each test was eight tidal cycles. The only variable in these three tests was the wave direction: S16°E, test 1; N84°E, test 2; and S56°E, test 3. As the wave was rotated from S16°E through S56°E to N84°E, it was noted that the ratio of total mass of material moved was approximately 3:2:1, with respect to the given directions. Test 1 (Plate 64) showed good movement into the shoal region, but material that moved into the channel alongside the jetty was not carried out by the ebb currents. Tests 2 and 3 (Plates 65 and 66) showed lesser material movement toward the channel; thus scour took place in the channel but not enough to compare favorably with the prototype. Also, material movement in the inlet gorge was very low.

91. In order to increase the tidal current scouring action, the range of ocean tide was increased to 6.1 ft. The S56°E wave was used as an average direction; and test 4 was conducted with wave parameters held constant as before. When compared with test 1 which had a 3.8-ft ocean tidal range, results of test 4 showed twice as much material movement (Plate 67); however, most of the material moved was at the outer tip of the jetty where ebb currents were strongest, and no increase in scouring at the inlet gorge took place.

92. Test 4 was conducted with a 6.1-ft tide which had a 0.0 msl mean tidal level. Test 5 was planned to increase ebb velocities at the inlet gorge so it was believed that if the tide range was kept the same but the mean level lowered 1.0 ft to a -1.0 ft msl mean tidal

level, an increase in velocity could be gained. Results of test 5 (Plate 68) did not indicate an increase of material movement; in fact, the total amount of material moved in test 5 was about 20 percent less than the material moved during test 4. The major cause of this decrease appears to be that lowering the tidal plane resulted in less penetration of wave action.

93. Tests using naturalite as the bed material gave reasonable indications of shoal scour patterns on the shallow bar areas and near the beach; but in deeper water where wave motion was weaker and not sufficient to initiate sediment motion, the shoal and scour trends indicated by the naturalite material did not appear reasonable. The plastic material (sp gr 1.18), which is somewhat lighter than the naturalite, was selected for use as modeling material. A more detailed version of the prototype scour and fill map for 1969-1971 is shown in Plate 69. On this map, some blocks were broken into smaller units to make better distinction between fill and scour regions.

#### Tests 9 and 10

94. Tests 9 and 10 were conducted without waves in order to observe the movement caused by tidal velocities alone. Test 9 showed very limited movement of the plastic across the ocean shoal and less than desirable movement in the gorge area (Plate 70). Test 10 was conducted with a higher ocean tidal range (4.5 ft) and a greater movement of the material was experienced as shown in Plate 71.

95. Some problems noted during tests 9 and 10 (Plates 70 and 71) related to both the characteristics of the model and the plastic material used. The plastic is placed in the model during a slack-water period while the model is in operation; as a result, there is limited time to accomplish the placement before current action begins. Because the plastic is very light, it is difficult to get a uniform placement of material, especially in areas of steep slope. The model has a distortion factor of 5, i.e., slope is five times steeper in the model than it is in the prototype, so the material tends to slide down slopes before it comes to rest on the surface of the model.

96. The tendency of the plastic material to shed water rapidly

and dry out became another problem during shoaling verification. In the shoal areas that became exposed during low water, the material would dry in the short time of exposure and as the water began to rise, some of the plastic would fail to break the surface tension and would remain on the surface of the water and float off as the surface velocity increased.

97. This problem was solved by thoroughly wetting the plastic in a concentrated detergent solution prior to each test. The water-softening capabilities of the detergent served to break the surface tension, thereby allowing the material to remain in place until covered with water. Detergent was also sprinkled lightly over the shoal areas as they became exposed throughout the test. Out of a total of 46,530 ml of plastic material used for a test, the amount of material lost averaged 207 ml, or a 0.4 percent loss.

98. After test 10, two 20-ft-long plunger-type wave machines were installed in the model ocean, providing the capability to simulate two wave directions during a given test. The tests following test 10 were then designed to use wave machines, thus giving the more realistic capability of having wave seasons for a test. The wave directions for which the machines were located were, as before, S16°E and N84°E.

99. Although movement of the plastic material by tidal currents alone was not as successful as was hoped for, a series of tests (11-30) with waves plus tidal action was initiated with the expectation that the turbulence from wave action would improve material movement.

100. A short series of experiments was run in order to determine what type of wave would best move the plastic material. Plastic material was placed over the model surface, and waves of various heights and periods were run and their effect on movement of plastic at various depths was observed. Generally, for a given wave height, the longer the wave period, the greater its ability to move material in deeper regions and also the greater its ability to reach and move sediment near the inlet throat after crossing the ocean shoals. Another guideline in the selection of waves was based on prototype observations which were: higher, steeper, shorter period waves approached from a northeasterly direction in the fall and winter; smaller, longer period swell

approached from the southwest in the spring and summer.

101. In order to achieve a balance between tidal and wave action, an intermittent type wave was used for tests 11-15, i.e., waves were reproduced for certain time increments throughout the tidal cycle. One prototype hour equals approximately 92 sec in the model; and waves were reproduced for the first 10 sec of this 92 sec throughout the tidal cycle. Although this procedure aided in suspending sediment and initiating sediment motion, the circulation pattern due to waves, as discussed in a previous report,<sup>1</sup> was probably not maintained.

#### Tests 11-14

102. Before discussing tests 11-14, a short outline of the parameter changes made between tests will aid in interpretation of results. Periods of the waves used in tests 11 and 12 were longer than periods of the waves used in tests 13 and 14. Also, the time duration of northerly waves was twice that for southerly waves for tests 11 and 13, while the time duration for southerly waves was twice that for northerly waves for tests 12 and 14. A spring tide with a range of 4.5 ft was used in all of these tests. Tests 11-14 were run for six tidal cycles each. The wave direction sequence was two tidal cycles of the predominant wave, then one tidal cycle of the other wave, two more tidal cycles of the predominant wave, and one tidal cycle of the other wave.

103. The initial distribution of material injected into the model during tests 11 and 12 is shown in Plate 72 while that for tests 13 and 14 is shown in Plate 73. In general, results from tests 11 and 12 (Plates 74 and 75) compared better with the prototype on the beachward half of the shoal area, and those from tests 13 and 14 (Plates 76 and 77) gave better results on the oceanward half. The longer period waves of tests 11 and 12 moved more material from prototype scour blocks 39 and 33 into prototype fill blocks 29 and 32. The shorter period waves of tests 13 and 14 moved material into block 33, an area of scour in the prototype, but not into block 32, an area of fill in the prototype. Blocks 37, 38, 45, 46, and 47 (areas of fill in the prototype) were heavily scoured in the model by the longer period waves of tests 11 and 12. The shorter period waves did not cause as much scouring in the

deeper water areas of blocks 37, 38, 45, 46, and 47, and did help obtain some fill in those areas. That region in the prototype most likely receives some material from suspended sediments during ebb flow, a phenomenon which is not reproduced in the model. In the shoreward regions (blocks 9 and 10), material was not moved into the inlet area by the shorter period waves during tests 13 and 14 nor by the longer period waves used in tests 11 and 12.

104. All four tests satisfactorily reproduced the scour along the outer reaches of the north jetty and deposited this material farther oceanward. In the inlet throat, however, block 7 always accumulated an undesired fill because the material would pile up in the bottom of the gorge due to the steep side slope.

105. Comparison of the effects of wave predominance for the longer period waves indicated that the 2:1 northerly wave predominance of test 11 gave slightly better results than did the 2:1 southerly wave predominance of test 12. However, with the shorter period waves, the scouring action of the 2:1 southerly wave predominance of test 14 gave significantly better sedimentation patterns than did the 2:1 northerly wave predominance of test 13. An overall comparison of tests 11, 12, 13, and 14 indicates that a 2:1 ratio of southerly wave duration to northerly wave duration produces the better model shoaling patterns.

#### Test 15

106. Test 15 was a repeat of test 14. No changes were made in any parameters. Results (Plate 78) were compared with those of test 14 to determine if the tests are repeatable. The general pattern repeated very well, with most changes from fill to scour or vice versa occurring in areas of low material movement. In comparing volumes measured in each test for each block, those pairs of blocks of which one block had less than a 10 percent volume change were eliminated from the analysis. It was then determined that, between the two tests, 49 percent of the blocks agreed within  $\pm 10$  percent, 71 percent of the blocks agreed within  $\pm 20$  percent, and 80 percent of the blocks agreed within  $\pm 30$  percent. In other words, this is a comparison of the areas of significant material movement. When all blocks were used for comparing how well volume

changes repeated, i.e., including those areas in which there is only small movement, it was found that 66 percent of the blocks agreed within +10 percent, 85 percent of the blocks agreed within +20 percent, and 90 percent of the blocks agreed within +30 percent.

#### Tests 16 and 17

107. Test 16 was conducted using the wave burst technique but with a change in timing of the bursts. The burst from the north wave was increased from 10 to 20 sec per model hour. Results of test 16 (Plate 79) showed less movement of material on the outer shoal than the previous test, and an undesirable change from scour to fill in the area of blocks 48 and 54. After further analysis of test 16, it was determined that the wave burst technique does not develop the proper circulation patterns.

108. Test 17 was conducted with a 2-ft-high, 10-sec-period wave from S16°E and a 2-ft-high, 5.8-sec-period wave from N84°E. A 4.5-ft tide (average spring) with a 0.0-ft mtl was used. Waves were reproduced from the south throughout the first two cycles then from the north throughout cycle 3. This sequence was repeated once. The necessary circulation pattern created by continuous wave action existed during the test, but the south waves resulted in excessive material movement. Results of test 17 are shown in Plate 80.

#### Test 18

109. For test 18, a S16°E wave height was reduced to 1 ft with a 10-sec period. This change gave an improvement in desired patterns, especially in the outer area of the south shoal; however, the lack of movement in many locations indicated that too great a reduction in wave height was made to the southerly wave. Results are shown in Plate 81.

#### Test 19

110. For test 19, the S16°E wave height was increased to 1.5 ft. In addition, the N84°E wave was reduced in height from 2 to 1.5 ft and the period increased from 5.8 to 9 sec in order to produce more movement along Masonboro Beach. Results of test 19 (Plate 82) indicate slight improvement, especially in the inner shoal area (blocks 33, 23, 24, 25, and 26); but the outer shoal area continued to show excessive scour.

#### Tests 20 and 21

111. Tests 20 and 21 were conducted to determine the effects of waves from a single direction. For test 20, a  $S16^{\circ}E$ , 2-ft-high, 6.7-sec-period wave was used throughout the six-cycle testing period. For test 21, a  $N84^{\circ}E$ , 2-ft-high, 6.5-sec-period wave was used throughout the six-cycle testing period. Time-lapse motion pictures were made of both tests, with an exposure obtained every 2 sec. This was done to allow detailed analysis of developments during the tests and proved helpful in analyzing test results. Results of test 20 (Plate 83) indicated that waves from the south caused excessive scour on the south shoal. Results of test 21 (Plate 84) indicated that waves from a northerly direction produce proper but limited movement of material toward the main channel to allow simulation of the fill adjacent to the channel in blocks 32 and 40.

112. Analysis of results obtained to this point in the test program indicated that waves from both directions contributed equally to obtaining the desired prototype shoaling and scouring patterns; therefore, equal time was allotted waves from both directions during the next test.

#### Test 22

113. For test 22, a  $N84^{\circ}E$ , 2-ft-high, 5.8-sec-period wave and a  $S16^{\circ}E$ , 1-ft-high, 10-sec-period wave were used. The sequence of wave operation was to have been N-S, N-S, N-S; but an equipment malfunction resulted in a modification of test plans. The test was completed as follows: (a) first cycle with waves from  $N84^{\circ}E$ , (b) second cycle with waves from  $S16^{\circ}E$ , (c) third cycle with no wave, and (d) fourth, fifth, and sixth cycles with 1-ft, 13-sec-period waves from  $S16^{\circ}E$ . Although results (Plates 85) were considered questionable because of the malfunction, it was determined that a shorter period wave from  $S16^{\circ}E$  would improve the model verification.

#### Test 23

114. Test 23 was conducted using a wave sequence of N-S, N-S, N-S. A  $N84^{\circ}E$ , 2-ft-high, 5.8-sec-period wave and a  $S16^{\circ}E$ , 1-ft-high, 10-sec-period wave were used. These were the same wave characteristics used

for test 18; however, the sequencing was changed. Results (Plate 86) showed an improved transport of material toward the channel area (from blocks 33 and 34 to block 32) but insufficient movement of material in the shoal area (blocks 59, 60, 61, and 65).

#### Test 24

115. For test 24, the period and height of the S16°E wave were increased to 13 sec and 1.25 ft, respectively. The north wave remained the same (2 ft high, 5.8-sec period). A procedure for injecting material was initiated for test 24, because, in test 23, areas along Masonboro Beach and from the end of Masonboro Beach to the main channel (blocks 11A, 12A, 63A, 64A, 9, 13, 14, 21, and 22) showed excessive scour. The total deficiency of material in these blocks at the end of test 23 was 3440 ml. This amount was arbitrarily increased by 50 percent to determine the amount of material to be fed to the beach. Because waves from the south moved the material along Masonboro Beach toward the inlet more than waves from the north, it was decided to deposit material at the beginning of each cycle when waves from the south were reproduced. One third of the material was fed into the model at the beginning of tidal cycles 2, 4, and 6. Results of the test (Plate 87) showed a shorter period wave was needed from S16°E to decrease the amount of material movement.

#### Test 25

116. Test conditions for test 24 were maintained for test 25, except the wave period for waves from S16°E was decreased from 13 to 10 sec and the feeding procedure was modified. It was determined from observations of test 24 that material fed to Masonboro Beach was more likely to move when deposited in small amounts in the breaker zone rather than in one large amount at the beginning of each cycle. Thus, for test 25, material was deposited on the beach during each cycle as was needed to maintain the desired conditions in blocks 64A, 63A, 11A, and 12A, i.e., as the blocks scoured out they were fed again. Results of test 25 (Plate 88) show that significant improvement in reproduction of shoaling near the south side of the inlet throat was achieved. Shoaling of the prototype along the jetty channel (blocks 32 and 40) was not reproduced by the model.

### Test 26

117. After analyzing films of test 25, it was concluded that an increase in the period of the waves from the north would improve movement of material along Masonboro Beach. Since the results of test 21 showed substantial material movement, the north wave of test 21 (2 ft high with a period of 6.5 sec) was used during test 26. The rate at which material was fed to Masonboro Beach was increased for test 26 because of the longer period north wave. Results for test 26 (Plate 89) showed minor improvement in fill and scour patterns. It was noted that the longer period north wave moved some of the material fed to Masonboro Beach southward along the beach.

### Test 27

118. Test 27 was conducted with a decreased tidal range (3.8-ft mean tide) in order to reduce scouring in the channel along the jetty and especially to obtain increased shoaling in blocks 32 and 40. Results (Plate 90) showed an improvement of desired shoaling for blocks 32, 29, and 20. This was the only significant change in results from test 26.

### Test 28

119. Conditions for test 28 remained the same as test 27, except that the mean tidal level was raised to +0.5 ft msl from 0.0 ft msl. This was done to increase movement of material toward the throat of the inlet by the penetration of wave action across the south shoal. Results of test 28 (Plate 91) show acceptable agreement with the prototype data.

## Accuracy of Tests

### Tests 29 and 30

120. Tests 29 and 30 were undertaken to examine the repeatability of identical tests; therefore the conditions of test 28 were repeated for tests 29 and 30. Results of tests 29 and 30 are shown in Plates 92 and 93, respectively. The differences in percentages of change between tests 28 and 29, tests 28 and 30, and tests 29 and 30 are shown in Table 14. Analysis shows that for results of tests 29 and 30 when

compared with test 28, 67 percent of the blocks had less than 10 percent volume difference, 83 percent had less than 20 percent difference, 92 percent had less than 30 percent difference, and 95 percent had less than 40 percent difference. The average percent volume difference for a block was 11 percent.

Comparison with the prototype

121. Results of the model shoaling verification tests (average of 28, 29, 30) were also compared semiquantitatively with prototype volume changes that occurred during the 1969-1971 verification period (Table 15). The word semiquantitative is used because in the model material scoured is limited to that amount placed on a block at the start of the test, which in turn may affect the amount of fill in an adjacent block. In the prototype there may be much more scour in one area and, as a result, much more fill in another area. The percentages for both model and prototype data are determined by dividing the fill (or scour) of a block by the total fill (or scour) of the entire study area. These percentages are then weighed by the ratio of the area of the given block to the sum of the block areas. The differences in percentages are presented first in bar graph form (Figures 30 and 31) to allow for comparison of model with prototype results. These percentages are then grouped and shown symbolically to represent each percentage group as shown in Plate 94. Identical procedures and operating techniques developed on

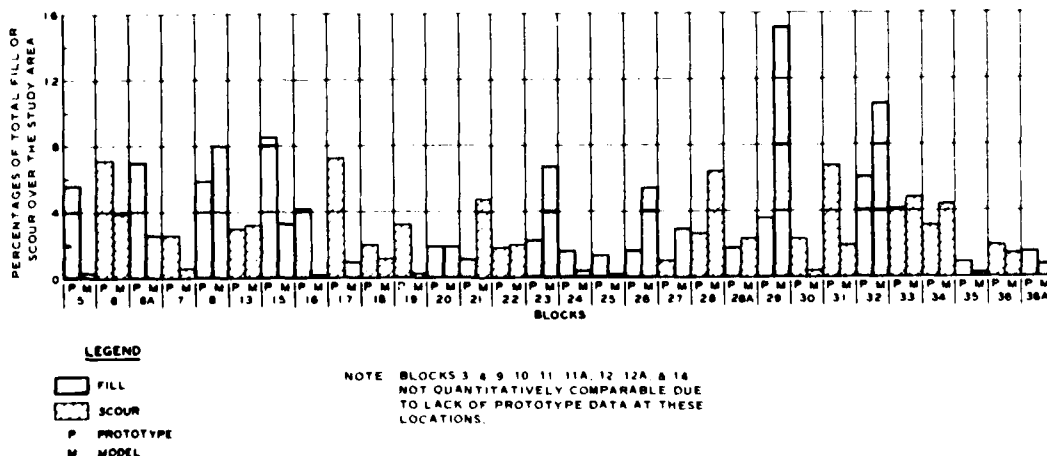
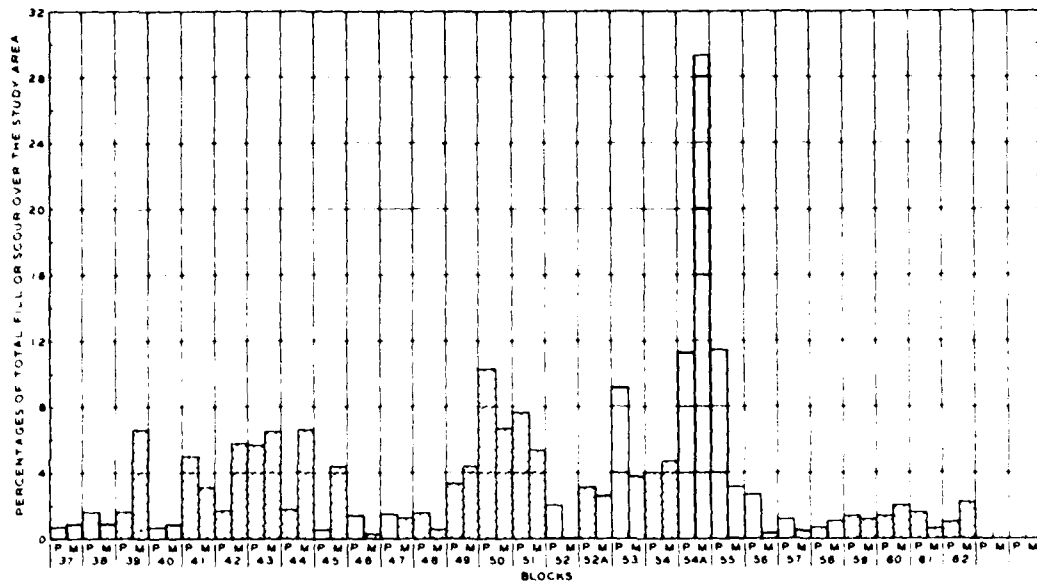


Figure 30. Comparison of model and prototype percentages of total fill or scour, blocks 5-36A



**LEGEND**  
 [Solid Bar] FILL  
 [Dotted Bar] SCOUR  
 P PROTOTYPE  
 M MODEL

NOTE: BLOCKS 53, 63A, 64, & 64A NOT QUANTITATIVELY COMPARABLE DUE TO LACK OF PROTOTYPE DATA AT THESE LOCATIONS.

Figure 31. Comparison of model and prototype percentages of total fill or scour, blocks 37-62

the model to obtain shoaling verification for the desired verification period of September 1969 to August 1971 were followed in all tests for different conditions or plans of improvement.

General Observations

122. During the verification of shoaling, some interesting observations were made with respect to sediment transport. On the flood phase there was a continual movement of sediment toward the inlet. North of the inlet, sediment moved along the beach, over the weir at its intersection with the beach, then along the north side of the inlet toward Banks Channel. From the south, material moved along the beach and along the south side of the inlet with the aid of the current and refracting waves. Some material also moved from the shoals toward the inlet. A summary of movement is shown in Figure 32.

123. During ebb flow, a more complex sediment movement system

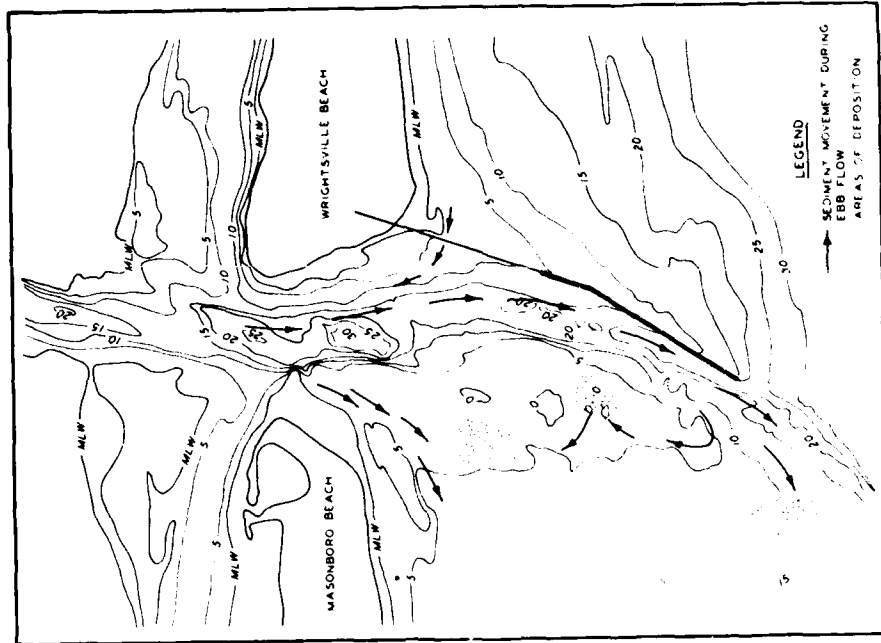


Figure 32. September-October 1969 hydrography, flood flow

Figure 33. September-October 1969 hydrography, ebb flow

exists (Figure 33). Material is swept out on the south side of the inlet over the south shoal area near the inlet. Also, the strong currents along the jetty channel move material at a high rate. At the tip of the jetty, material moved laterally from its original direction because of the approaching waves. This was particularly true of waves approaching from the south but not as severe for waves approaching from a northerly direction. In the lee of the shoal area (with respect to the ebb currents), the wave action captures some of the laterally moving material and carries it up to the south shoal and deposits some material there while some is carried laterally again toward the south. Thus there is sediment movement directly counter to the ebb flow.

124. On the north side of the inlet gorge, there is no movement of material oceanward since this area lies in an eddy generated by flow coming out of Banks Channel; if the wave power is high enough, it is possible to obtain movement bayward during the ebb flow.

#### Conclusions for 1969 Shoaling Tests

125. A successful shoaling verification was completed for the September 1969-August 1971 period. This is to say that regions of fill or scour were verified in a semiquantitative manner. It cannot be explicitly stated that the forcing parameters (tides, waves, etc.) adjusted for this verification are applicable for another time frame, but due to their rational selection, e.g. shorter period, steeper waves from N84°E and longer period, low waves from the S16°E direction used in the verification which are also typical of the prototype, it is hoped that roughly correct short-term predictions can be made for other bathymetries as discussed in the following sections.

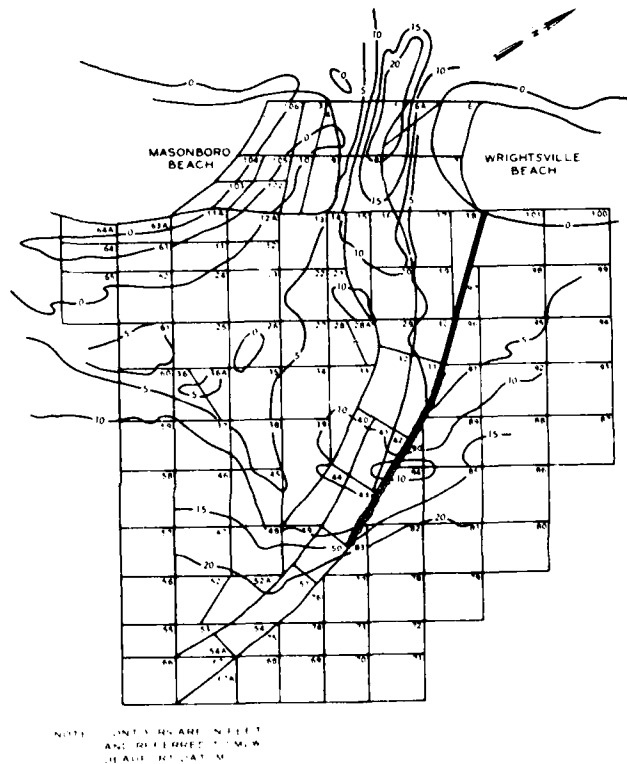
#### 1964 Condition Fill and Scour Tests

126. A later phase of the model study involved remolding the model to the November 1964 prejetty condition to collect hydraulic and fill and scour data (see Reference 1 for hydraulic data). The purpose of

testing with this bathymetry was to provide base data for comparison with tests of the plan as constructed in 1965.

127. With the 1964 prejetty bathymetry installed in the model (see Reference 1 for details) and prior to conducting the fill and scour tests, the area of the inlet from the throat oceanward was segmented into a pattern of blocks (Figure 34) as initially established for the

Figure 34. November 1964 contours, located on shoaling test grid with jetty plan



verification tests. The quantity of material placed in each block at the beginning of test to assure a similarity of material uniformly placed over the test area was proportional to the plan areas of the blocks. The material was then placed in the appropriate block and the test conducted. The tide, waves, shoaling material, injection procedure, and test duration developed in the scour and fill verification were duplicated during all subsequent tests. For information, the test conditions are listed in the plates presenting the results of each scour and fill test. After the test, the material in each block was measured.

The percentage change from the original amount of material was then determined. This percentage is shown in the center of each block in the plates presenting the results of the tests. Block numbers are identified by the small numbers in the upper right corner of each block.

128. Base tests for fill and scour patterns were conducted and were identified as tests 1964-1 and 1964-2. Two tests were conducted to allow the repeatability of the tests to be defined. The quantities of material placed in each of the grid blocks used for the tests are shown in Plates 95-97 for the base and subsequent tests. A contour map for the 1964 hydrographic condition and the location of each block are shown in Plate 98. Results of both base tests are shown in Plates 99 and 100.

129. In order to quantify the difference between the results of tests 1964-1 and 1964-2 (Plates 99 and 100), the percentage change for each block of test 1964-2 was subtracted from the corresponding changes for test 1964-1. Results are shown in Plate 101. In this procedure, changes which indicated scour were assigned positive values (increase in depth), whereas changes indicating fill were assigned negative values (decrease in depth). A positive difference between the changes observed in the two base tests (Plate 101) thus indicates a greater depth in that block for test 1964-1 than for test 1964-2.

130. Several blocks in both base tests were swept clean of material (blocks 24, 25, 61, and 35 on the south offshore shoal and blocks 15 and 16 in the main channel). These blocks thus experienced "100 percent scour" as shown in Plates 99 and 100. Since "100 percent scour" is impossible in the prototype, the significance of this occurrence in the model is difficult to evaluate. The apparent duplication of this occurrence in the two base tests is also difficult to evaluate, since if more material had been initially available the scour rates for the two tests may have been different.

131. Plate 101 shows no particular consistency of the error between the two tests on the basis of broad regions within the entrance area. For example, almost the same number of positive and negative differences occurred along each beach, in the natural offshore channel, and over each offshore shoal. However, several groupings of contiguous

blocks exhibited either positive or negative differences, e.g., blocks 3, 4, 10, 102, 103, 12A, 12, 13, 23, and 26 showed positive differences while blocks 5, 6A, 8, 9, and 14 and blocks 104, 105, and 106 showed negative differences. The latter two groups are immediately adjacent to the first. It is thus apparent that relatively small areas were subjected to more scour in one test than in the other, and that the material was probably deposited in one or more nearby small areas. In both tests, the percentage change in most blocks oceanward from the north and south shoals was minimal; therefore, the differences between the two tests are small. Although local differences are evident, the overall results of tests 1964-1 and 1964-2 are considered to be similar.

132. Results of tests 1964-1 and 1964-2 for each block were averaged, and the average data are provided in Plate 102. In addition, three photographs taken at the end of test 1964-2 provide additional information on the test results. Views of the model and pattern of the material at the end of test 1964-2 as seen from the bay, from directly overhead, and from the ocean are shown in Photos 36, 37, and 38, respectively. The average results for each block (Plate 102) show that the gorge of the inlet experienced scour for most blocks (blocks 5, 6, 6A, and 8 show scour, but block 7 shows fill); however, inspection of Photo 37 shows that material existed in only the Wrightsville Beach end of blocks 6 and 7 with minimum or no material in the portions in the gorge. Movies showed that flood flow at the throat of the inlet caused material to move a short distance into the bay and that ebb flow caused the material to be transported back through the inlet to a position farther oceanward than the original location of the material at the start of the flood phase. Thus there was a net oceanward movement of material originally placed in the throat. The data in Plate 102 indicate a major accumulation of material at the end of Masonboro Beach (blocks 102-106). The data show a net scour for the approach portion of Wrightsville Beach (blocks 100, 101, and 18) and a trend for fill along Masonboro Beach (blocks 63 and 64 show fill; block 11 shows a small amount of scour). Both the shallow shoals oceanward from Masonboro and Wrightsville Beaches and the channel area immediately oceanward from

the throat showed pronounced scour, whereas the offshore portion of the natural channel exhibited pronounced deposition. In the model, the wave action on both shoals was intense and easily put the shoaling material into suspension. Currents then moved the material off the shoal. As the water depths over the shoal increased, material was deposited particularly in the channel between the shoals. As deeper waters were encountered offshore from the shoals, the tides and wave action did not cause the material to move.

#### Summary - Base Tests

133. The following observations are made, based on the results of the base tests:

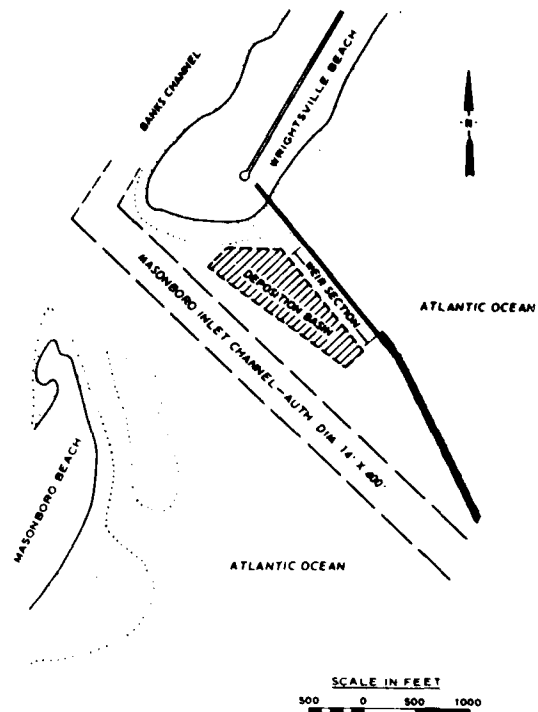
- a. The fill and scour tests indicate that the waves are important in the elongation of the shoals oceanward from Wrightsville and Masonboro Beaches. The results appear to overemphasize the removal of material from the shallower portions of the shoals.
- b. The same patterns of fill or scour were repeated for identical tests. There are cause-effect variations from test to test though, as a slightly increased scour in one location will probably produce an increased fill elsewhere.

#### 1964 Hydrography Plan Tests

134. After the 1964 base data for the natural inlet were collected, the 14-ft-deep by 400-ft-wide navigation channel, the 16-ft-deep deposition basin, and the 3400-ft-long weir jetty (Figure 35) were constructed in the model.

135. Tests 1964-3 and 1964-4 were conducted with the jetty, basin, and navigation channel installed in the model. The quantity of material placed in each block of the grid prior to conducting the tests is shown in Plate 96. The only difference between the initial injection for the plan tests and that for the base tests (Plate 95) was that the initial injection was further subdivided in those blocks through

Figure 35. Inlet improvements



which the navigation channel or deposition basin passed. This was necessary since construction of the plan resulted in very sharp depth variations in blocks 14-21, 27-30, 33, 34, 39, and 44. The plan test results also show the subdivision of these blocks. A third test (1964-5, Plate 97) was conducted with the jetty, basin, and navigation channel installed in the model. In this test, no material was initially placed north of the jetty in order to assess the importance of material placed in that area. Material placement quantities were identical with those of tests 1964-3 and 1964-4, except for those blocks that were omitted. A contour map superimposed over the grid block layout is shown in Plate 103.

136. The changes that occurred in each block during the tests are shown in Plates 104-106. Photos 39-41 show various views of the deposition patterns at the conclusion of test 1964-3. The differences between the results of tests 1964-3 and 1964-4 are shown in Plate 107. Results of the average of tests 1964-3 and 1964-4 are presented in Plate 108. The differences between the average of tests 1964-3 and 1964-4 and

test 1964-5 are shown in Plate 109. The differences between the 1964 base data (tests 1964-1 and 1964-2) and the 1964 plan data (tests 1964-3 and 1964-4) are presented in Plate 110. An indication of the significance of the change that occurred in a particular block was also determined. This consisted of determining the amount of fill or scour per unit area for each block of the grid for the average data for both the 1964 base tests and the 1964 plan tests. The total amount of fill or scour was then determined for the test area, and the percentage of total fill or scour for the blocks was determined. Results are presented in a plan of the test area (Plate 111) and in bar charts (Plates 112-115).

137. The differences between the results of tests 1964-3 and 1964-4 (Plate 107) were determined as for the two base tests (see paragraph 129). In general, the same comments made about this comparison for the base test data (see paragraphs 130 and 131) can be made about the comparison for the plan tests. It is apparent that relatively small areas were subjected to more scour in one test than in the other, and the material was probably deposited nearby in one or more small areas. Although local differences are evident, the overall results of tests 1964-3 and 1964-4 are similar.

138. Results of the test to determine the influence of lack of material placed northeast of the jetty (Plate 106) are compared with the average of tests 1964-3 and 1964-4 in Plate 109. In this case, the 1964-5 results were subtracted from the 1964-3 and 1964-4 average results. If the lack of material available for transport over the weir section and around the outer end of the jetty significantly influenced the results, appreciable change should occur to the results. In particular, a large number of blocks near the jetty with appreciable positive differences would indicate an insufficient inflow of material from the area north of the jetty. A general review of the data does not show a significant difference of this nature. The differences observed are in the general range of differences observed in the comparison between the results of test 1964-3 and 1964-4 (Plate 107).

139. The average results of the plan tests (Plate 108) show heavy scour in the throat, on the south shoal, between the channel and the jetty seaward of the deposition basin, and immediately north of the weir section. Heavy shoaling occurred in the navigation channel, in the deposition basin, and along Masonboro Beach.

140. Compared with the base tests, the effects of the jetty, deposition basin, and navigation channel are shown in Plate 110. A marked increase in scour occurred along Wrightsville Beach at the throat of the inlet and in the immediate vicinity of the deepest portion of the gorge. Approximately halfway between the gorge and Masonboro Beach the trend reversed to increased fill. The data for the area at the end of Masonboro Beach showed a major reduction of filling with the plan installed. The overall trend for the data at the throat of the inlet was a deepening of the gorge and a widening of the inlet, especially toward Wrightsville Beach (see Photos 36 and 39).

141. The difference between the base and plan data (Plate 110) for the area of the deposition basin showed that the plan reversed significant scour to heavy fill and decreased the rate of scour in the area between the deposition basin and the weir section of the jetty. In the navigation channel opposite the jetty weir section, scour at the inner end was reduced and scour at the outer end of this section was reversed to fill. Heavy shoaling that occurred in the base test between the navigation channel and the jetty from the outer end of the deposition basin to the outer end of the jetty was reversed to scour. Block 20 and that part of block 16 outside the navigation channel were the only blocks on a line from the gorge to the outer end of the jetty that did not indicate increased scour or reversal from fill to scour resulting from the plans. The contour map with the plan installed (Plate 103) shows that block 20 is located in a shallow area with sharp slopes to deep water immediately bayward and into the navigation channel. The data for both the base test (Plate 102) and the plan test (Plate 108) show that all material was removed ("100 percent scour") from the block immediately bayward (block 16). Since the bed of the model is fixed

and erosion cannot be simulated beyond the removal of the material placed in the blocks, the material in block 20 may be artificially protected by the inability of the fixed-bed model to fully simulate scour in block 16. If the capability to fully simulate scour in block 16 had existed in the model, the data for the test with the plan may have shown scour (or at least less fill) for block 20. In general, the model results show that compared with base conditions, the plan caused definite filling of the deposition basin and inner half of the navigation channel, and scour between the navigation channel and deposition basin at the inner end of the channel and between the navigation channel and the jetty at the outer end.

142. The difference between the base and plan data (Plate 110) for the south shoal shows that the plan resulted in increased scour (or reversal of fill to scour) adjacent to the central portion of the navigation channel and increased filling or reversal of scour to fill adjacent to the outer end of the navigation channel. The area just south of the crown of the south shoal (blocks 24, 25, 61, and 35) shows no change; however, this can be misleading for two reasons. Again, the data for the base (Plate 102) and plan (Plate 108) both show "100 percent scour" in these blocks. Had more material been initially placed in these blocks, the scour rate for the plan might have been different from that of the base. The second reason data may be misleading is that no material was initially placed immediately outside the blocks; therefore, material was not available for transport into these blocks. The hydraulic tests with the  $N84^{\circ}E$  waves and  $S16^{\circ}E$  waves (see Reference 1) showed that the current patterns were such that transport of material from outside of the test area was a reasonable assumption; the same is true of base test conditions. The general trend for the south shoal area was for the area immediately adjacent to the navigation channel to scour and for the shoal to elongate into the ocean. The area immediately northeast of the jetty showed that scour at the beach was reversed to fill, and scour along the seaward face of the jetty was increased. The remaining portions of the study area did not experience any appreciable movement for either test.

143. Results of the computations of percentages of total fill and scour based on the fill and scour per unit area are shown in Plates 111-115. The data tend to emphasize those areas of the inlet in which the most significant changes of fill or scour occurred. At the throat of the inlet, the influence of the plan on scour is emphasized at and near the gorge (blocks 6 and 7). The filling of the deposition basin (blocks 7-IB, 17-IB, 18-IB, 19-IB, 20-IB, 29-IB, and 30-IB) and scour (or reduction of fill) on both sides of the outer end of the navigation channel (blocks 33-OC, 34-OC, 39-OC, 40-OC, and 44-OC) are evident.

144. Based on the analysis of the data resulting from the fill and scour tests, predictions for the improvement of the inlet are:

- a. Both the navigation channel and the deposition basin will fill.
- b. The area between the navigation channel and the deposition basin and along the outer end of the jetty will scour.
- c. The shoal located oceanward from Masonboro Beach will elongate.
- d. The throat of the inlet will increase in depth and width and shift toward Wrightsville Beach.

145. The more significant predictions are:

- a. The navigation channel can be expected to shift toward the jetty.
- b. The shoal located oceanward from Masonboro Beach can be expected to elongate.
- c. The deposition basin can be expected to fill.

#### 1966 Hydrography Tests

146. The 1964 hydrography was removed from the region of the inlet entrance and replaced with the 1966 hydrography. The July 1966 prototype hydrographic survey was made immediately after construction of the jetty, channel, and deposition basin. Thus the 1964 plan data

could be compared with data from a postconstruction condition.

147. Figure 36 shows the 1966 hydrography. Comparison of Figures 34 and 37 shows that depths were generally shallower for the 1966 condition, and that at velocity range 2 location, there was a significant shift in the channel to the north (about 150 ft). Also, the actual channel was not as wide as the plan, probably due to the natural sloping of sand under wave and tidal current action subsequent to completion of dredging.

148. Locations of the blocks of the grid for the fill and scour tests with respect to the 1966 hydrography are shown in Plate 116. The quantity of material placed in each block prior to conducting the tests is shown in Plate 95. Results of tests 1966-1 and 1966-2 are presented in Plates 117 and 118. The deposition pattern at the conclusion of test 1966-1 is shown in Photo 42. The differences between the two tests are plotted in Plate 119. The averages of the percentage change for each block for tests 1966-1 and 1966-2 are shown in Plate 120. The differences between the average 1964 plan data and the average 1966 data are recorded in Plate 121. The data in Plate 122 are the percentage of the total scour and total fill for the average of the 1964 and 1966 tests. The procedure used to determine the percentage of fill and scour is discussed in paragraph 136. The same data are presented in bar graph form in Plates 123-126.

149. The differences between the results of tests 1966-1 and 1966-2 (Plate 119) were determined by the same procedure used for the two base tests (see paragraph 129). In general, the comments presented about the base test data (paragraphs 126 and 127) also apply to the 1966 tests. The data indicate that scour which appears for small areas to be more for one test resulted in increased deposition in one or more of the small areas in the immediate area.

150. The average results of the 1966 tests show areas of significant fill along Masonboro Beach, in most of the navigation channel, in the deposition basin, between the deposition basin and the landward half of the weir section, on the south side of the outer end of the jetty, and from the seaward edge of the south shoal extending due east to the

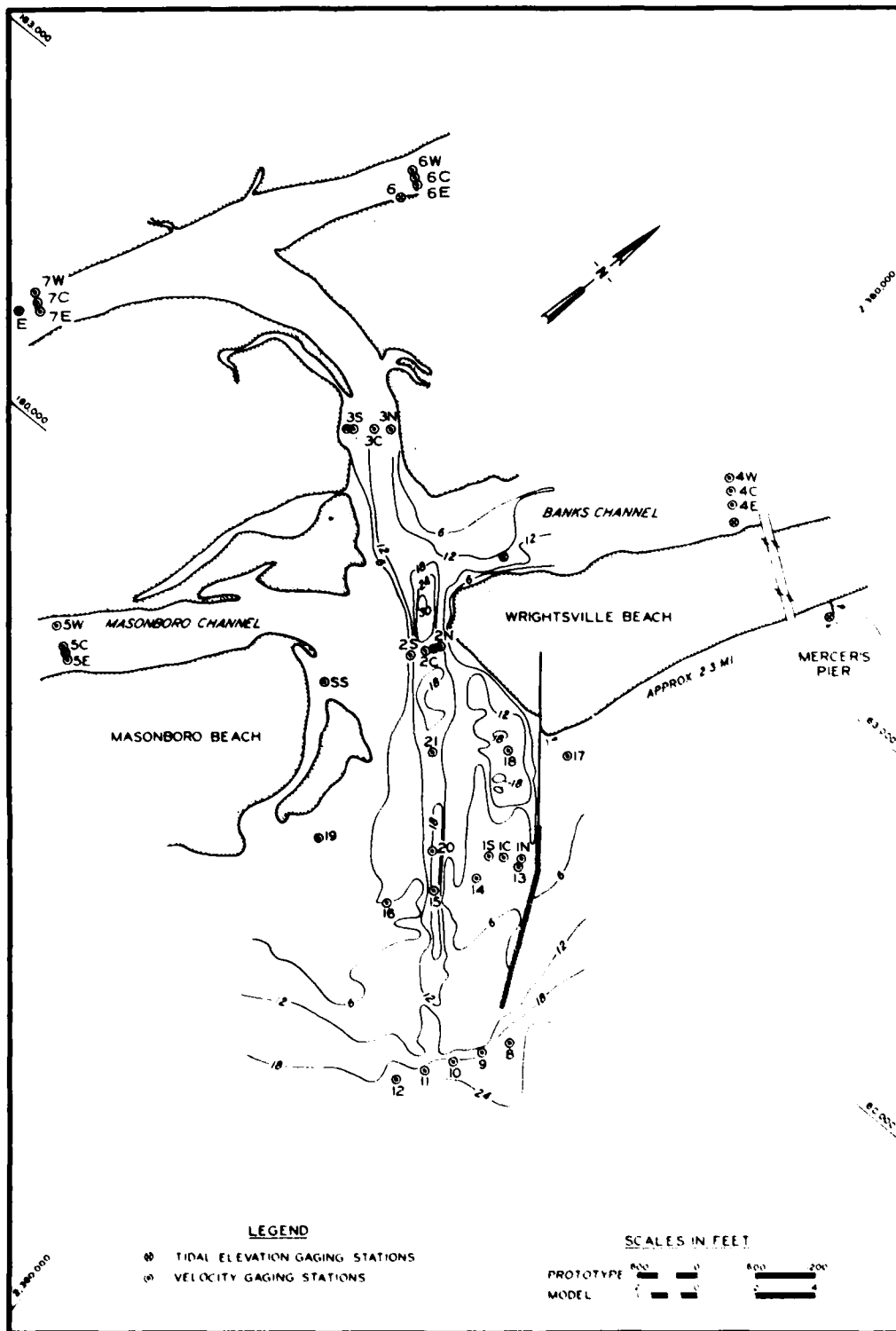
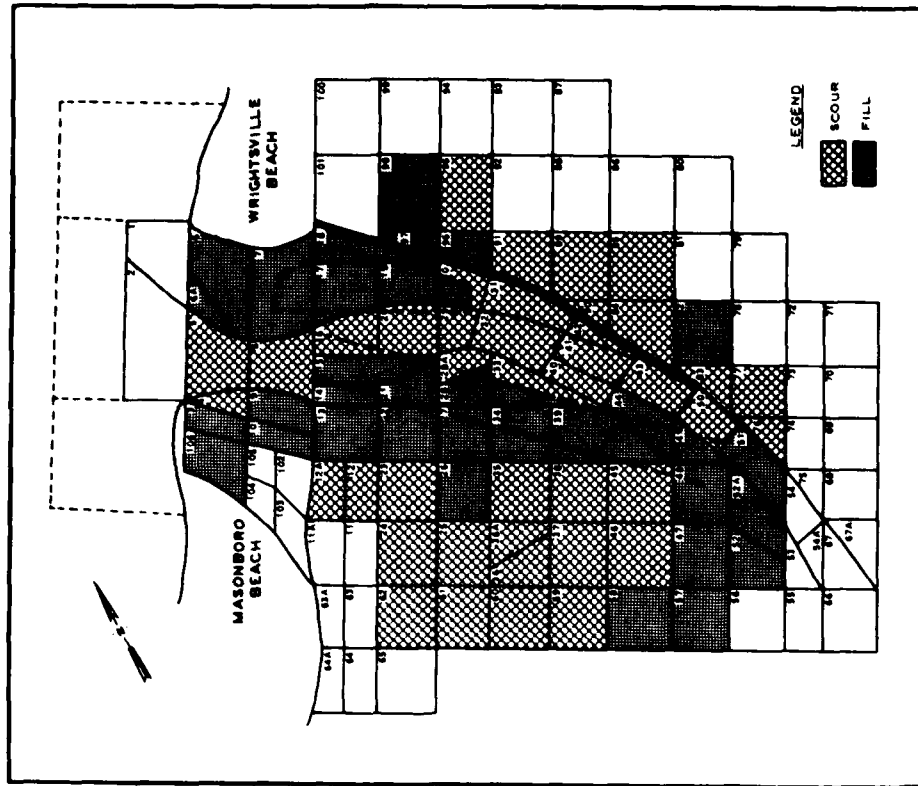
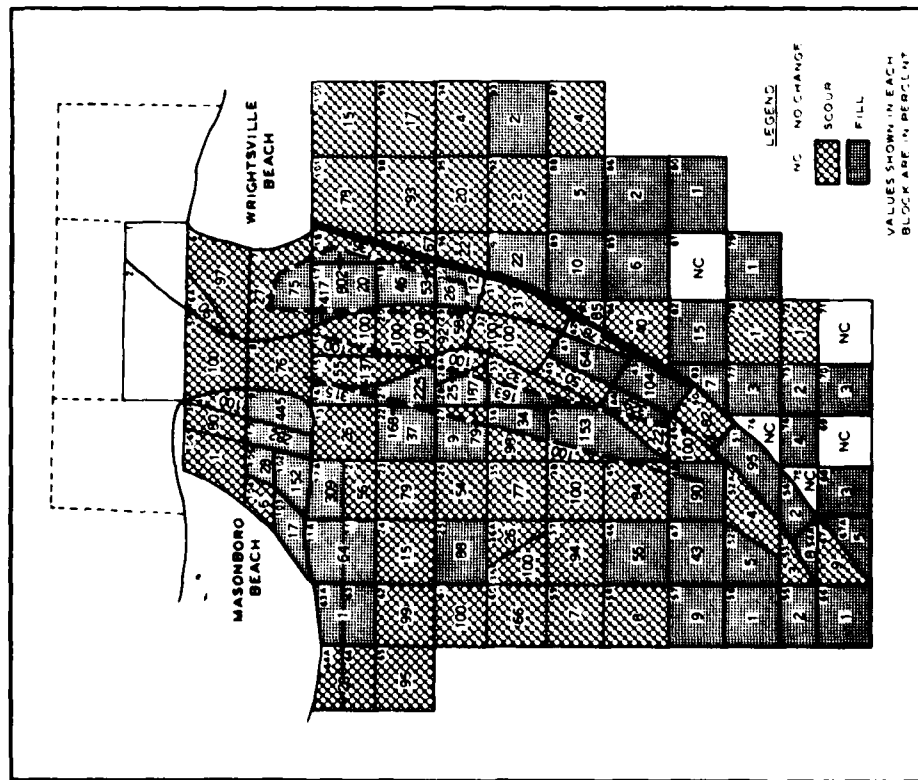


Figure 36. 1966 hydrography



a. 1966 shoaling test (model)



b. Scour and fill for the period July 1966-July 1968 (prototype)

Figure 37. Comparison of prototype fill and scour with model shoaling

extension of the navigation channel alignment. Areas of significant scour occurred in the throat, between the channel and the deposition basin, between the channel and the outer leg of the jetty, on the south shoal, and offshore of Wrightsville Beach at the weir section of the jetty. In general terms, these scour and fill patterns are very similar to those of the 1964 plan tests (Plate 108).

151. The differences between the average 1964 plan tests and the average 1966 tests (Plate 121) were determined by subtracting the 1964 plan results from the 1966 results. At the throat of the inlet, the 1964 tests overemphasized the trend for deepening and reducing the width of the gorge compared with the 1966 results. Along Masonboro Beach, the 1966 tests showed much less fill. On the south shoal, the 1966 tests showed considerably less scour or, in a few blocks, reversal from scour to fill. As the depths of the shoal increased (to the south), the 1966 data showed a trend for increased scour or reversal from fill to scour. The 1966 test data for the ocean end of the south shoal show considerably more fill (elongation of the shoal). In the navigation channel, the 1966 data showed more fill in the deeper portion of the cut through the bar (blocks 28, 34A, and 39) with less fill or more scour at the inner end of the channel. The 1966 test showed more filling at the inner end of the deposition basin but less filling at the outer end. The area (ridge) between the outer half of the deposition basin and the navigation channel and the area between the channel and the jetty seaward of the deposition basin were distinctly areas of more scour for the 1966 results. Near the outer end of the jetty, however, the 1966 test data showed a relatively small area (blocks 41 and 43) with more fill or a reversal from scour to fill. Along the northeast side of the jetty, the 1966 test data showed more scour or reversal from fill to scour near Wrightsville Beach and at the outer end of the jetty, whereas less scour or reversal from scour to fill occurred at the bend in the jetty. Oceanward from these areas, the movement of material was minimal (see Plates 108 and 120).

152. Results of percentage of total fill or scour on a unit area basis for the 1964 plan and 1966 test data (Plate 122) show that the

areas of most significant deviation between the two tests were the tip of Masonboro Beach, the seaward tip of the south shoal, the area between the channel and the outer half of the basin, the area just north of the channel and seaward of the basin, and a short section of channel adjacent to the outer end of the basin (all of which indicated greater depths for 1966 conditions), and the south shoal, the areas just south of the inner end of the channel, and two isolated sections of the navigation channel (all of which indicated lesser depths for 1966 conditions).

#### Discussion of 1964 Plan Predictions

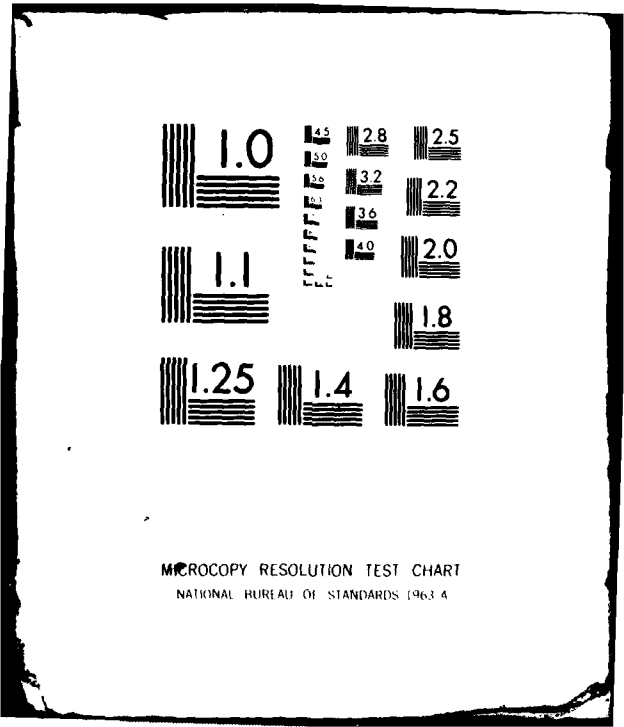
153. Predictions based on the results of the tests with the 1964 hydrography with and without the plan installed are presented in paragraphs 144 and 145. In general, the results of the tests with the 1966 hydrography installed in the model confirm these predictions or show that the 1964 results are conservative.

154. Results of the fill and scour tests show:

- a. The 1966 tests indicate the bayward portion of the deposition basin receives more of the material than was indicated by the 1964 plan tests and less material for the oceanward end. Both conditions, however, indicated filling throughout the basin. The 1966 tests indicate lesser depths in the outer half of the navigation channel and greater depths in the inner half than the 1964 plan tests. Both conditions, however, show scour in the inner one third of the channel and fill in the outer two thirds.
- b. The 1966 data showed a significantly more pronounced scour pattern than the 1964 plan data in the area between the navigation channel and the deposition basin with a transition to a pronounced fill (rather than continued scour) along the outer end of the jetty.
- c. The 1966 data compared with the 1964 plan data show a more pronounced trend for the south shoal located oceanward from Masonboro Beach to elongate.
- d. The 1966 data show less of an emphasis for the throat to increase in depth and more of a trend to increase in width when compared with the 1964 data.

155. It is generally concluded that the fill and scour tests





MICROCOPY RESOLUTION TEST CHART  
NATIONAL BUREAU OF STANDARDS 1963 A

tended to be more conservative when the 1964 hydrography was used than for the tests with the 1966 hydrography.

156. Based on the analysis of the data resulting from the fill and scour tests, predictions for the period after 1966 for the inlet are:

- a. The entire deposition basin will fill but at a lower rate in the oceanward end.
- b. The navigation channel will scour in the area immediately oceanward from the throat and fill in the outer portions.
- c. The channel can be expected to shift to the areas between the existing navigation channel and the deposition basin and between the navigation channel and the jetty in the area farther oceanward. Near the end of the jetty, fill between the jetty and the navigation channel is predicted.
- d. The south shoal can be expected to elongate.
- e. The throat can be expected to move toward Wrightsville Beach. This is contrary to the prediction for this area based on hydraulic test results.

157. The data indicate:

- a. The navigation channel can be expected to migrate closer to the jetty.
- b. The south shoal can be expected to elongate.

#### Comparison of Model Predictions with Prototype Data

##### Prototype data

158. This section contains an evaluation of the list of predictions based on the 1966 tests as outlined in paragraphs 156 and 157 and generally of the 1964 plan predictions, since the 1966 and 1964 plan results were very similar. Prototype velocity and tidal data were collected in September 1969 and semiannual bathymetric surveys were made after construction of the project so that hydraulic and shoaling test predictions could be inspected.

##### General trends

159. An examination of the 1969 bathymetry in Figure 2a when compared with the 1966 bathymetry in Figure 37 indicates that the general

trends predicted in paragraph 156 did occur. The navigation channel migrated closer to the jetty and the south shoal elongated oceanward a significant amount.

Fill and scour test predictions

160. In order to compare the 1966 fill and scour test predictions of paragraph 156 with the prototype results, a prototype fill and scour map for the July 1966-July 1968 period is shown in Figure 37b with the 1966 shoaling test results of the model (Figure 37a). The July 1968 bathymetry is shown in Figure 38. The two-year period is used since it

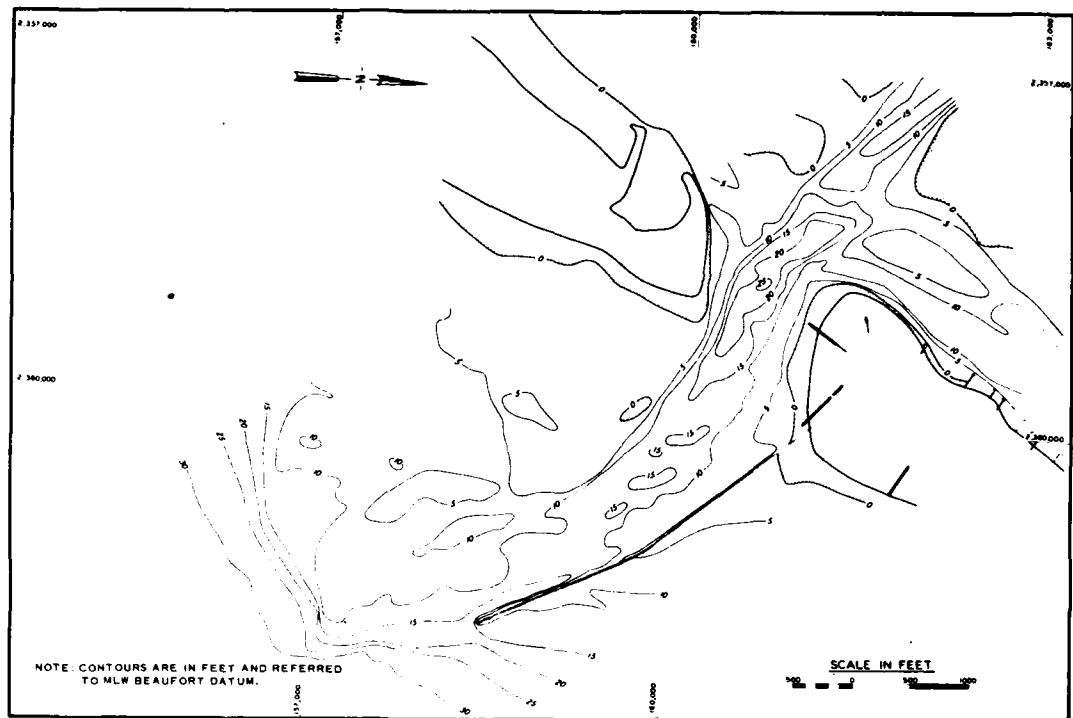


Figure 38. Hydrographic survey for July 1968

is the predictive period as determined from the verification shoaling tests. Plates 127-130 also aid in the comparison.

161. The prototype data show fill in the deposition basin as predicted by the model in paragraph 156c (blocks 17, 18, and 19). Scour occurred at the inner portion of the navigation channel (block 8) and

fill in the outer portion (blocks 28, 34, 39, and 21) following the prediction in 156b. In paragraph 156c it was said that the channel would shift toward the jetty. Agreement with the prototype to this effect is shown in blocks 16, 20, 29, 31, 32, 40, and 42. Blocks 41 and 43 of the model indicate fill compared with prototype scour, but oceanward of this location, agreement is again seen in blocks 50 and 51. Figure 38 shows shifting of the channel by the location of the 15-ft contours (compare with Figure 37). Also, the prediction of shoal elongation in paragraph 156d can be seen in Figure 38 when compared with the 1966 condition in Figure 37. There is no noticeable movement of the inlet throat toward Wrightsville Beach as predicted by the tests and stated in paragraph 156e, but the fill areas of blocks 9, 10, 3, and 106 (Figure 38b) in the prototype indicate a trend for pushing the throat toward Wrightsville Beach, though 6, 6A, and 7 also show fill along the Wrightsville Beach side.

#### Summary

162. The 1966 predictions are substantiated by prototype fill and scour trends. Motion pictures were of significant value in analysis of the fill and scour test results, and movies should be obtained for all tests.

#### Conclusions

163. Comments concerning interpretation of sedimentation test results are:

- a. The fill and scour tests did show a trend, near the navigation channel and throat of the inlet, for material to accumulate at the base of steep slopes. The reduction of the model distortion to a lower value than the 5:1 distortion used in this study would appear to improve the results of fill and scour tests in areas of steep slopes.
- b. It was noted in this and other model studies that the steepened side slopes of navigation channels, deposition basins, etc., resulting from model scale distortion can

apparently adversely affect the results of a fixed-bed model sedimentation test. It may be beneficial to conduct investigations to determine if it is feasible, or necessary, to adjust slopes in such areas to achieve better simulation of scour and fill characteristics.

- c. As plans of improvement are installed in the model, unrealistic ridges and simulation of dredging required in the prototype to get into an isolated area such as a deposition basin must be considered to achieve adequate simulation of proposed changes to the inlet.
- d. In the course of the development of the size and shape of the grid to be used for fill and scour tests, consideration of possible areas of significant fill and scour as the result of possible additions or changes to the model should be defined as well as possible. This is necessary to avoid difficulty in evaluating test results for blocks that experience scour in one part and fill in the other part. It may be necessary to conduct one or more preliminary tests before finalizing the grid system. This information should then be used to establish small blocks in critical areas, especially where substantial scour is expected for future tests. The grid system also should be designed around scour and fill areas for verification conditions.
- e. The grid system for scour and fill tests should be larger than the model test area in order to provide a source of sediments to the perimeter of the test area. Changes in the amount of material in the area located outside the test area should be determined at the conclusion of the tests; however, these data should not be considered in analysis of the test area.

164. Comments on model predictions are:

- a. When significant bed changes to the inlet are predicted based on the results of fixed-bed model tests, it should be emphasized that the predictions are short term in nature. After the relatively short period of time necessary to effect such changes in the field, subsequent changes could be quite different than the short-term predictions.
- b. When model results indicate major scour or fill, it would be quite beneficial to modify the bed of the model and conduct another test with these major changes incorporated in the model. This should provide better information on the intermediate- to long-term changes.

165. A second objective of the study was to define the effectiveness of distorted-scale, fixed-bed models in predicting fill and scour

changes to an inlet when significant artificial changes are made to the inlet. This purpose was achieved. The fill and scour tests and hydraulic tests were quite effective in allowing short-term fill and scour trends to be predicted qualitatively. However, major changes in hydrography again preclude quantitative long-range predictions.

#### Recommendations

166. Based on the results of the study, recommendations concerning model tests of inlets to evaluate basic plans of improvement are:

- a. Fixed-bed, distorted-scale models should be used where economically justified to evaluate the changes in hydraulics that result from possible plans of improvement. Although this investigation considered only a single-jettied system, this recommendation also is valid for dual-jetty systems.
- b. Fill and scour tests should be used to develop data for short-term predictions of hydrographic changes to an inlet; however, care should be taken in attempting to project the predictions of extensive changes over a long period of time.

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Table 1  
Summary of Tidal Data

Gage	High-Water Elevation ft msl			Low-Water Elevation ft msl			Tide Range, ft			Mean Tide Level, ft		
	Base	BCC*	MCC†	Base	BCC	MCC	Base	BCC	MCC	Base	BCC	MCC
B	1.8	1.7	1.8	1.8	1.9	1.9	3.7	3.6	3.7	-0.04	-0.08	-0.05
1	1.9	1.9	1.9	1.8	1.8	1.9	3.7	3.7	3.8	+0.08	+0.07	+0.02
2	1.8	1.8	1.7	1.8	1.8	1.8	3.6	3.6	3.5	+0.06	+0.01	-0.05
3††	1.6	1.7	1.8	1.9	1.8	1.7	3.5	3.5	3.5	-0.06	-0.04	+0.14
3‡	--	--	1.5	--	--	-1.1	--	--	2.6	--	--	+0.29
4‡	1.6	1.3	1.7	1.8	-0.90	-1.7	3.4	2.2	3.2	+0.01	+0.33	+0.18
5††	1.7	1.5	1.6	1.7	-1.5	-1.3	3.4	3.0	2.9	+0.12	+0.04	-0.08
6	1.7	1.6	1.5	1.7	-1.1	-1.6	3.4	2.7	2.7	+0.09	+0.32	+0.16
E	1.7	1.7	1.5	1.6	-1.3	-1.4	3.4	3.0	2.6	+0.12	+0.30	+0.18

\* BCC = Banks Channel closed.

\*\* SCC = Shinn Creek closed.

† MCC = Masonboro Channel closed.

†† Tide gage oceanward of barrier location.

‡ Tide gage bayward of barrier location.

Table 2  
Phase Changes in Tidal Height Data

Gage	Time of High Water, hr				Time of Low Water, hr			
	Base	BCC*	SCC**	MCC†	Base	BCC	SCC	MCC
B	7.5	7.0	7.5	7.5	1.0	1.0	1.0	1.0
1	7.0	7.0	7.0	7.0	1.0	1.0	1.0	1.0
2	7.5	7.5	7.5	7.5	1.5	1.0	0.5	1.0
3††	7.5	7.5	7.0	7.5	1.5	1.5	1.0	1.0
3‡	7.5	--	8.0	--	1.5	--	3.0	--
4††	7.5	8.5	7.0	7.0	1.5	3.0	1.5	1.5
5	7.5	7.0	8.0	7.0	1.5	1.5	2.0	1.0
6	8.0	8.0	8.0	7.5	1.5	2.0	2.0	1.5
E	7.5	7.0	8.0	7.5	1.5	2.0	2.5	1.5

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- \* BCC = Banks Channel closed.
  - \*\* SCC = Shinn Creek closed.
  - † MCC = Masonboro Channel closed.
  - †† Tide gage oceanward of barrier location.
  - ‡ Tide gage bayward of barrier location.

Table 3  
Maximum Velocities at Ranges 1-5

Station	Depth	Maximum Flood Velocities, fps				Maximum Ebb Velocities, fps			
		Base	BCC*	SCC**	MCC†	Base	BCC	SCC	MCC
<u>Range 1</u>									
N	Surface	1.2	0.9	0.7	1.0	2.8	3.0	2.7	2.7
	Bottom	1.0	0.7	0.5	1.0	2.5	2.3	2.8	2.5
C	Surface	1.6	1.4	1.3	1.5	2.6	2.6	2.3	2.7
	Middepth	1.4	1.4	1.2	1.6	2.1	2.2	2.4	2.0
	Bottom	1.3	0.6	0.4	1.2	2.7	2.4	1.9	2.4
S	Surface	1.9	1.7	1.6	2.0	3.8	3.1	3.5	4.0
	Middepth	1.8	1.5	1.5	1.7	3.5	2.8	2.4	3.5
	Bottom	1.4	1.2	0.9	1.1	3.2	2.1	1.8	2.6
<u>Range 2</u>									
N	Surface	3.1	2.6	3.1	2.9	1.3	3.2	1.2	1.0
	Middepth	3.2	2.7	2.7	2.7	2.3	2.8	1.2	1.6
	Bottom	2.8	2.5	2.9	2.6	2.7	2.7	1.2	2.4
C	Surface	3.4	2.3	2.9	2.7	4.0	2.5	2.9	3.6
	Middepth	3.0	2.3	2.5	2.5	3.7	2.1	2.8	3.3
	Bottom	2.4	1.8	2.2	2.4	3.3	2.2	2.6	3.3
S	Surface	2.5	1.9	1.8	1.5	3.8	2.4	4.0	4.1
	Middepth	2.4	1.8	1.4	1.8	3.2	2.4	3.5	4.0
	Bottom	2.0	1.8	1.6	2.0	3.8	2.2	3.1	3.0
<u>Range 3</u>									
N	Surface	2.3	2.8	--	3.1	2.3	3.7	--	2.6
	Middepth	1.9	2.6	--	2.7	1.9	2.2	--	2.0
	Bottom	1.6	2.5	--	1.8	1.4	2.4	--	1.7
C	Surface	2.1	2.6	--	2.4	1.7	3.0	--	2.0
	Middepth	1.9	2.5	--	2.4	1.5	2.4	--	1.9
	Bottom	1.8	2.6	--	2.5	1.5	2.2	--	1.6
S	Middepth	1.9	2.7	--	2.2	1.3	2.0	--	1.7
<u>Range 4</u>									
E	Surface	1.0	--	1.5	1.0	1.9	--	2.6	2.6
	Middepth	0.8	--	1.2	0.8	1.9	--	2.3	2.1
	Bottom	0.5	--	1.0	0.6	2.0	--	2.4	2.2
C	Surface	1.4	--	2.2	1.6	1.6	--	2.1	1.7
	Middepth	1.1	--	1.7	1.6	1.4	--	1.8	1.5
	Bottom	1.0	--	1.5	1.2	1.2	--	1.6	1.6
W	Surface	2.1	--	1.8	1.7	1.4	--	2.3	1.6
	Middepth	1.6	--	1.6	1.6	1.0	--	1.7	1.3
	Bottom	1.4	--	1.3	1.5	0.9	--	1.5	1.1
<u>Range 5</u>									
E	Surface	2.0	2.3	2.8	--	2.9	3.5	2.8	--
C	Surface	2.1	2.5	2.6	--	2.7	3.3	3.1	--
W	Surface	2.0	2.3	2.3	--	2.8	3.4	3.2	--

\* BCC = Banks Channel closed.

\*\* SCC = Shinn Creek closed.

† MCC = Masonboro Channel closed.

Table 4  
Flow Volumes\* (in millions of cubic feet) and  
Percent Change from the Base Condition

<u>Range</u>	<u>Base</u>	<u>BCC</u>	<u>Percent Change</u>	<u>SCC</u>	<u>Percent Change</u>	<u>MCC</u>	<u>Percent Change</u>
2	566	449	-21	452	-20	477	-16
3	200	299	+49	--	--	251	+25
4	241	--	--	342	+42	268	+11
5	161	167	+4	181	+12	--	--

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\* Average of flood and ebb flow volumes.

Table 5  
K Value for 1973 Condition Channel Closure Tests  
by the Current Slack Method

	<u>Time of HW Slack in Inlet</u>	<u>Time of LW Slack in Inlet</u>	<u>Slack Lag from Ocean HW*</u> hr	<u>Slack Lag from Ocean LW**</u> hr
Base	8.75	2.26	1.50	1.51
BCC†	9.08	3.06	1.83	2.31
SCC††	8.94	2.98	1.69	2.23
MCC‡	8.98	2.46	1.73	1.71

	<u>Duration of Ebb Flow</u>	<u>Duration of Flood Flow</u>	<u><math>\epsilon_f</math></u>	<u><math>\epsilon_e</math></u>	<u><math>K_f</math></u>	<u><math>K_e</math></u>	<u><math>K_{avg}</math></u>
Base	5.93	6.49	41.6°‡‡	45.8°	0.78	0.70	0.74
BCC	6.40	6.02	54.7°	65.0°	0.55	0.36	0.46
SCC	6.46	5.96	51.0°	62.1°	0.61	0.43	0.51
MCC	5.91	6.51	47.8°	52.1°	0.67	0.59	0.63

- \* Time of ocean high water = 7.25.  
 \*\* Time of ocean low water = 0.75.  
 † BCC = Banks Channel closed.  
 †† SCC = Shinn Creek closed.  
 ‡ MCC = Masonboro Channel closed.  
 ‡‡ Sample calculation:  $1.50/6.49 \times 180^\circ = 41.6^\circ$ .

Table 6  
K Evaluated by  $a_b/a_o$

	$a_o$	$a_b$	$a_b/a_o$	$K_{a_b/a_o}$	$K_\epsilon$	$\Delta K (K_{a_b/a_o} - K_\epsilon)$
Base	3.8	3.40	0.89	1.10	0.74	0.36
BCC*	3.8	2.85	0.75	0.78	0.46	0.32
SCC**	3.8	2.65	0.70	0.70	0.51	0.19
MCC†	3.8	3.15	0.83	0.94	0.63	0.31

Note:  $a_b$  = average tide range of gages 6 and E.

\* BCC = Banks Channel closed.

\*\* SCC = Shinn Creek closed.

† MCC = Masonboro Channel closed.

Table 7  
Bay Superelevations,  $\Delta$

	Duration of Flood Flow <u>hr</u>	$\sigma\tau_i$ <u>deg</u>	Calculated $\Delta$ , ft	Measured* $\Delta$ , ft
Base	6.49	188.1	-0.13	+0.04
BCC**	6.02	174.5	0.09	+0.13
SCC†	5.96	172.8	0.12	+0.16
MCC††	6.51	188.8	-0.15	+0.04

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- \* Averaging the average tide elevation for all gaging stations in bay.
  - \*\* BCC = Banks Channel closed.
  - † SCC = Shinn Creek closed.
  - †† MCC = Masonboro Channel closed.

Table 8  
Effective Channel Length, L

	<u>K</u>	<u>K<sup>2</sup></u>	<u>L*, ft</u>	<u>Percent Change from Base</u>
Base	0.74	0.548	20,407	--
BCC**	0.46	0.212	55,931	+174
SCC†	0.51	0.260	45,136	+122
MCC††	0.63	0.397	28,902	+42

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\*  $L = \frac{12,251}{K^2} - 1965$  .

\*\* BCC = Banks Channel closed.

† SCC = Shinn Creek closed.

†† MCC = Masonboro Channel closed.

Table 9  
Inertial Effects

	Head Differences Between Bay and Ocean when Inlet Velocity is Zero				Slope of Velocity Curve at $V = 0$		
	Measured at HW ft	Average Measured ft	Calculated Keulegan's Method, ft	Calculated at HW Measuring $dV/dt$ , ft	Calculated at LW Measuring $dV/dt$ , ft	$dV/dt$ at HW, ft/sec <sup>2</sup>	$dV/dt$ at LW, ft/sec <sup>2</sup>
	Base	0.15	0.15	0.13	0.29	0.17	0.00088
BCC*	0.25	0.05	0.28	0.57	0.33	0.00062	0.00036
SCC**	0.35	0.10	0.24	0.42	0.33	0.00057	0.00044
MCC†	0.30	0.10	0.16	0.14	0.18	0.00076	0.00039

\* BCC = Banks Channel closed.

\*\* SCC = Shinn Creek closed.

† MCC = Masonboro Channel closed.

Table 10  
Tidal Data

Gage	Tide Range, ft				Mean Tide Level ft msl			
	Base*	Plan 1**	Plan 2†	Plan 3††	Base	Plan 1	Plan 2	Plan 3
1	3.7	3.6	3.6	3.7	+0.08	-0.07	-0.10	+0.06
B	3.7	3.6	3.7	3.6	-0.04	-0.09	-0.02	+0.04
2	3.6	3.4	3.5	3.4	+0.06	-0.11	-0.02	+0.08
3	3.5	3.4	3.4	3.5	-0.06	0.00	+0.07	+0.12
4	3.4	3.4	3.5	3.4	+0.01	+0.03	+0.05	+0.11
5	3.4	3.3	3.2	3.5	+0.12	+0.11	+0.13	+0.17
6	3.4	3.4	3.4	3.4	+0.09	+0.07	+0.24	+0.27
E	3.4	3.3	3.4	3.4	+0.12	+0.08	+0.15	+0.27
Avg	3.51	3.43	3.46	3.49	+0.05	0.00	+0.06	+0.14

- \* Base = natural 1973 model condition (including north weir jetty).  
 \*\* Plan 1 = base plus south jetty (no weir on south jetty).  
 † Plan 2 = base plus south jetty with a +2.0 mlw weir.  
 †† Plan 3 = north and south jetties with 0.0 mlw weirs.

Table 11  
Change of Mean Tide Levels from Base  
Condition in Feet Above msl

<u>Gage</u>	<u>Plan 1*</u>	<u>Plan 2**</u>	<u>Plan 3†</u>
1	-0.15	-0.18	-0.02
B	-0.05	+0.02	+0.08
2	-0.17	-0.08	+0.02
3	+0.06	+0.13	+0.18
4	+0.02	+0.04	+0.10
5	-0.01	+0.01	+0.05
6	-0.02	+0.15	+0.18
E	-0.04	+0.07	+0.15

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\* Plan 1 = base plus south jetty (no weir on south jetty).

\*\* Plan 2 = base plus south jetty with a +2.0 mlw weir.

† Plan 3 = north and south jetties with 0.0 mlw weirs.

Table 12

K Value for Weir Jetties Test by the Current-Slack Method

	<u>Time of HW Slack in Inlet</u>	<u>Time of LW Slack in Inlet</u>	<u>Slack Lag from Ocean HW*, hr</u>	<u>Slack Lag from Ocean LW**, hr</u>	<u>Avg Lag hr</u>	<u><math>\epsilon_{avg}^{\dagger}</math></u>	<u><math>K_{avg}</math></u>
Base <sup>††</sup>	8.75	2.26	1.50	1.51	1.505	43.6°	0.74
Plan 1 <sup>‡</sup>	8.74	2.27	1.49	1.52	1.505	43.6°	0.74
Plan 2 <sup>‡‡</sup>	8.78	2.31	1.53	1.56	1.545	44.8°	0.72
Plan 3 <sup>§</sup>	8.80	2.34	1.55	1.59	1.57	45.5°	0.70

\* Time of ocean high water = 7.25.

\*\* Time of ocean low water = 0.75.

$\dagger \epsilon_{avg} = \frac{\text{avg lag time (hr)}}{1/2 \text{ tidal cycle (6.21 hr)}} \times 180^\circ.$

†† Base = natural 1973 model condition (including north weir jetty).

‡ Plan 1 = base plus south jetty (no weir on south jetty).

‡‡ Plan 2 = base plus south jetty with a +2.0 mlw weir.

§ Plan 3 = north and south jetties with 0.00 mlw weirs.

Table 13  
Scour and Fill Verification Tests, 1969 Conditions  
Tidal Wave Period, 1:38.76, Wind Wave Period, 1:7.75

Test No.	Material Used Specific Gravity	Tidal Range Proto ft	Mean Plane Proto ft mal	Test Duration in Tidal Cycles	Wave Direction	Wave Period Proto sec	Wave Height	Wave Machine Operation		Duration of Model sec	Remarks
								Partial Time	Full Time		
1	Naturalite (1.7)	3.8	0.0	8	S16°E	7.4	3.0		X	9,224	
2	Naturalite (1.7)	3.8	0.0	8	N84°E	7.4	3.0		X	9,224	Same as test 1 except for wave direction
3	Naturalite (1.7)	3.8	0.0	8	S56°E	7.4	3.0		X	9,224	Same as test 1 except for wave direction
4	Naturalite (1.7)	6.1	0.0	8	S56°E	7.4	3.0		X	9,224	Same as test 3 except tide range increased
5	Naturalite (1.7)	6.1	-1.0	8	S56°E	7.4	3.0		X	9,224	Same as test 4 except mean tidal plane lowered
6, 7, 8											South jetty tests--omitted from this report
9	Plastic (1.18)	3.8	0.0	6	--	--	--	--	--	--	Studying velocity effects on material movement
10	Plastic (1.18)	4.5	0.0	6	--	--	--	--	--	--	Same as test 9 except tide range increased
11	Plastic (1.18)	4.5	0.0	6	N84°E S16°E	15 10	1.5 2.0	X X		520 260	N84°E wave machine operated 10 sec each prototype hour for 2 cycles then S16°E wave machine operated next cycle in same way. Repeat above once after first 3 cycles
12	Plastic (1.18)	4.5 4.5	0.0	6	N84°E S16°E	15 10	1.5 2.0	X X		260 520	S16°E wave machine operated 10 sec each prototype hour for 2 cycles then N84°E wave machine operated next cycle in same way. Repeat above once after first 3 cycles
13	Plastic (1.18)	4.5	0.0	6	N84°E S16°E	5.8 7.5	2.0 1.0	X X		520 260	Same as test 11 except using shorter wave periods
14	Plastic (1.18)	4.5	0.0	6	N84°E S16°E	5.8 7.5	2.0 1.0	X X		260 520	Same as test 12 except using shorter wave periods
15	Plastic (1.18)	4.5	0.0	6	N84°E S16°E	5.8 7.5	2.0 1.0	X X		260 520	Same as test 14 checking repeatability of tests
16	Plastic (1.18)	4.5	0.0	6	N84°E S16°E	5.8 7.5	2.0 1.0	X X		1,040 520	Same as tests 14 and 15 except duration of wave burst from N84°E wave machine was doubled in duration to 20 sec
17	Plastic (1.18)	4.5	0.0	6	N84°E S16°E	5.8 10.0	2.0 2.0	X X		2,306 4,612	Continuous wave action--first 2 cycles have waves from S16°E. Repeat sequence once again
18	Plastic (1.18)	4.5	0.0	6	N84°E S16°E	5.8 10.0	2.0 1.0	X X		2,306 4,612	Same as test 17 except S16°E wave reduced to 1.0-ft height
19	Plastic (1.18)	4.5	0.0	6	N84°E S16°E	9.0 10.0	1.5 1.5	X X		2,306 4,612	Wave sequence same as tests 17 and 18 north wave: period lengthened, height reduced. South wave: height increased
20	Plastic (1.18)	4.5	0.0	6	S16°E	6.7	2.0		X	6,918	Using only 1 wave from south
21	Plastic (1.18)	4.5	0.0	6	N84°E	6.7	2.0		X	6,918	Using only 1 wave from north
22	Plastic (1.18)	4.5	0.0	6	N84°E S16°E	5.8 10.0 13.0	2.0 1.0 1.0	X X		1,153 4,612	Malfunction of north wave machine after 1st cycle. Test run as follows: 1st cycle, N84°E wave; 2nd cycle, S16°E wave; 3rd cycle, no wave; 4th, 5th, 6th cycles used S16°E wave 1-ft height, 13-sec period
23	Plastic (1.18)	4.5	0.0	6	N84°E S16°E	5.8 10.0	2.0 1.0	X X		3,459 3,459	1st cycle wave from N84°E, 2nd cycle wave from S16°E, then repeat the same 2 times
24	Plastic (1.18)	4.5 4.5	0.0 0.0	6 6	N84°E S16°E	5.8 13.0	2.0 1.25	X X		3,459 3,459	Same wave sequence as test 23 increased period from south. Feeding plastic at beginning of each cycle with south wave
25	Plastic (1.18)	4.5	0.0	6	N84°E S16°E	5.8 10.0	2.0 1.25	X X		3,459 3,459	South wave period reduced. Feeder material added as needed
26	Plastic (1.18)	4.5	0.0	6	N84°E S16°E	6.5 10.0	2.0 1.25	X X		3,459 3,459	North wave period increased
27	Plastic (1.18)	3.8	0.0	6	N84°E S16°E	6.5 10.0	2.0 1.25	X X		3,459 3,459	Reduced tide range
28	Plastic (1.18)	3.8	0.5	6	N84°E S16°E	6.5 10.0	2.0 1.25	X X		3,459 3,459	Raised mean tidal plane 0.5-ft verification achieved
29	Plastic (1.18)	3.8	0.5	6	N84°E S16°E	6.5 10.0	2.0 1.25	X X		3,459 3,459	Repeat of test 28
30	Plastic (1.18)	3.8	0.5	6	N84°E S16°E	6.5 10.0	2.0 1.25	X X		3,459 3,459	Repeat of test 28

Table 14  
Evaluation of Repeatability of Tests 28, 29, and 30

Block No.	Percent Fill			Percent Scour			Percent Change Test 28 to Test 29	Percent Change Test 28 to Test 30	Percent Change Test 29 to Test 30
	Test 28	Test 29	Test 30	Test 28	Test 29	Test 30			
3	4	38	30				+34	+26	-8
4	57	89	61				+32	+4	-28
5		1	18	8			+9	+26	+17
6				21	22	44	-1	-23	-22
6A	14	29	62				+15	+48	+33
7				8	13	8	-5	NC	+5
8	86	126	101				+40	+15	-25
9	25	3				7	-22	-32	-10
10	89	100	111				+11	+22	+11
11-11A	5	8				2	+3	-7	-10
12	45	93	111				+48	+66	+18
12A				54	57	57	-3	-3	NC
13				32	61	50	-29	-18	+11
14				89	86	95	+3	-6	-9
15	26	55	49				+29	+23	-6
16	NC	NC	4	NC	NC		NC	+4	+4
17	12	17	12				+5	NC	-5
18				18	16	15	+2	+3	+1
19				1	3	3	-2	-2	NC
20	26	20	24				-6	-2	+4
21				70	66	78	+4	-8	-12
22				31	29	31	+2	NC	-2
23	93	79	95				-14	+2	+16
24	5	6	2				+1	-3	-4
25	4	3	NC			NC	-1	-4	-3
26	61	61	87				NC	+26	+26
27	44	36	37				-8	-7	+1
28				98	95	95	+3	+3	NC
28A				36	36	29	NC	+7	+7
29	200	191	194				-9	-6	+3
30				5	10	3	-5	+2	+7
31				31	14	40	+17	-9	-26

(Continued)

Note: Percent fill or scour is the percentage change of material in a given block from the amount of material originally placed in the block. NC denotes no change, i.e., no net material movement in a given block.

Table 14 (Concluded)

Block No.	Percent Fill			Percent Scour			Percent Change Test 28 to Test 29	Percent Change Test 28 to Test 30	Percent Change Test 29 to Test 30
	Test 28	Test 29	Test 30	Test 28	Test 29	Test 30			
32	95	182	138				+87	+43	-44
33				46	88	81	-42	-35	+7
34				66	72	59	-6	+7	+13
35	2	2	4				NC	+2	+2
36				35	13	15	+22	+20	-2
36A	15	4	9				-11	-6	+5
37	13	11	12				-2	-1	+1
38	8	12	17				+4	+9	+5
39				100	99	99	+1	+1	NC
40	19	14	4				-5	-15	-10
41				4	82	55	-78	-51	+27
42				84	94	94	-10	-10	NC
43				100	100	100	NC	NC	NC
44				100	100	100	NC	NC	NC
45				68	58	69	+10	-1	-11
46				2	6	7	-4	-5	-1
47				28	20	13	+8	+15	+7
48				2	8	15	-6	-13	-7
49				57	73	63	-16	-6	+10
50				100	100	100	NC	NC	NC
51				85	78	79	+7	+6	-1
52	2		3		7		-9	+1	+10
52A	42	36	26				-6	-16	-10
53	67	39	47				-28	-20	+8
54				70	79	60	-9	+10	+19
54A	331	423	385				+92	+54	-38
55		2	3	3			+5	+6	+1
56	7	2	3				-5	-4	+1
57	14	3	2				-11	-12	-1
58	18	13	11				-5	-7	-2
59				14	16	24	-2	-10	-8
60				28	30	33	-2	-5	-3
61				8	10	9	-2	-1	+1
62				34	33	31	+1	+3	+2
63-63A				42	37	42	+5	NC	-5
64-64A				27	20	23	+7	+4	-3
65				9	11	7	-2	+2	+4
66	2	2				5	NC	-7	-7
67	241	227	250				-14	+9	+23

Table 15  
Comparisons of Percentages of Quantitative Fill and Scour

Model with Prototype

Block No.	Prototype		Model		Difference	Block No.	Prototype		Model		Difference
	Fill	Scour	Fill	Scour			Fill	Scour	Fill	Scour	
1					*	35	0.9		0.2		0.7
2					*	36		1.9		1.4	0.5
3					*	36A	1.5		0.7		0.8
4					*	37	0.7		0.9		0.2
5	5.6		0.3		5.3	38	1.6		0.9		0.7
6		7.1		3.9	3.2	39		1.6		6.6	5.0
6A	7.0		2.6		4.4	40	0.7		0.9		0.2
7		2.6		0.6	2.0	41		5.0		3.1	1.9
8	5.9		8.0		2.1	42		1.7		5.8	4.1
9					*	43		5.7		6.5	0.8
10					*	44		1.8		6.6	4.8
11					*	45	0.6			4.4	5.0
11A					*	46	1.4			0.3	1.7
12					*	47	1.5			1.3	2.8
12A					*	48		1.6		0.6	1.0
13		3.0		3.2	0.2	49		3.3		4.4	1.1
14					*	50		10.3		6.7	3.6
15	8.6		3.3		5.3	51		7.7		5.4	2.3
16	4.2		0.1		4.1	52	2.0		0.0	0.0	2.0
17		7.2	1.0		8.2	52A		3.1	2.6		5.7
18		2.0		1.1	0.9	53	9.2		3.8		5.4
19		3.2		0.2	3.0	54		4.0		4.7	0.7
20	1.9		1.7		0.2	54A	11.3		29.3		18.0
21	1.1			4.8	5.9	55	11.5		0.0	0.0	11.5
22		1.8		2.0	0.2	56	2.7		0.4		2.3
23	2.2		6.8		4.5	57	1.2		0.5		0.7
24	1.6		0.4		1.2	58	0.7		1.1		0.4
25	1.3		0.2		1.1	59		1.4		1.2	0.2
26	1.7		5.4		3.7	60		1.4		2.0	0.6
27		1.0	2.9		3.9	61		1.6		0.6	1.0
28		2.7		6.4	3.7	62		1.0		2.2	1.2
28A	1.7			2.3	4.0	63					*
29	3.6		15.1		11.5	63A					*
30		2.3		0.4	1.9	64					*
31		6.8		1.9	4.9	64A					*
32	6.0		10.5		4.5	65					*
33		4.1		4.8	0.7	66					*
34		3.1		4.4	1.3	67					*

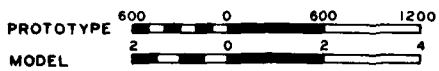
\* Not quantitatively comparable.



**VELOCITY SCALE**



**SCALES IN FEET**



**SURFACE CURRENTS**  
1973 CONDITIONS  
MEAN TIDE  
HOUR 4

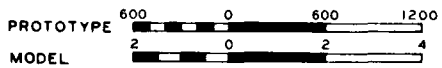
PHOTO 1



VELOCITY SCALE



SCALES IN FEET

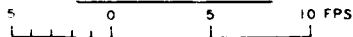


SURFACE CURRENTS  
1973 CONDITIONS  
MEAN TIDE  
HOUR 7

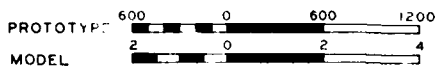
PHOTO 2



VELOCITY SCALE

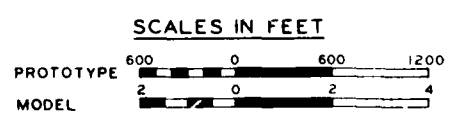
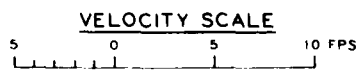


SCALES IN FEET



**SURFACE CURRENTS**  
 1973 CONDITIONS  
 MEAN TIDE  
 HOUR 10

PHOTO 3



**SURFACE CURRENTS**  
 1973 CONDITIONS  
 MEAN TIDE  
 HOUR 0

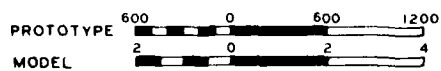
PHOTO 4



VELOCITY SCALE



SCALES IN FEET



**SURFACE CURRENTS**  
 1973 CONDITIONS  
 MEAN TIDE WITH  
 BANKS CHANNEL CLOSED  
 HOUR 4

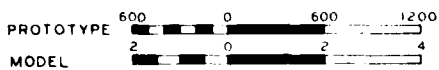
PHOTO 5



VELOCITY SCALE



SCALES IN FEET



SURFACE CURRENTS  
 1973 CONDITIONS  
 MEAN TIDE WITH  
 BANKS CHANNEL CLOSED  
 HOUR 7

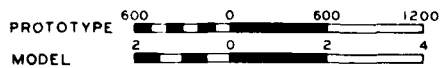
PHOTO 6



VELOCITY SCALE



SCALES IN FEET



SURFACE CURRENTS  
1973 CONDITIONS  
MEAN TIDE WITH  
BANKS CHANNEL CLOSED  
HOUR 10

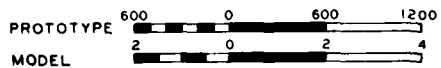
PHOTO 7



VELOCITY SCALE



SCALES IN FEET



**SURFACE CURRENTS**  
 1973 CONDITIONS  
 MEAN TIDE WITH  
 BANKS CHANNEL CLOSED  
 HOUR 0

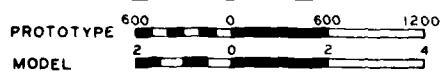
PHOTO 8



VELOCITY SCALE



SCALES IN FEET



SURFACE CURRENTS  
1973 CONDITIONS  
MEAN TIDE WITH  
SHINN CREEK CLOSED  
HOUR 4

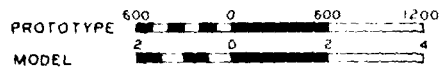
PHOTO 9



VELOCITY SCALE



SCALES IN FEET



**SURFACE CURRENTS**  
1973 CONDITIONS  
MEAN TIDE WITH  
SHINN CREEK CLOSED  
HOUR 7

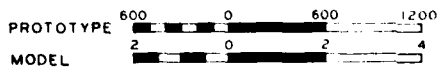
PHOTO 10



VELOCITY SCALE



SCALES IN FEET



SURFACE CURRENTS  
 1973 CONDITIONS  
 MEAN TIDE WITH  
 SHINN CREEK CLOSED  
 HOUR 10

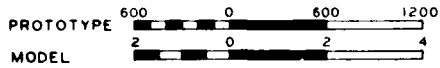
PHOTO 11



VELOCITY SCALE



SCALES IN FEET



SURFACE CURRENTS  
1973 CONDITIONS  
MEAN TIDE WITH  
SHINN CREEK CLOSED  
HOUR 0

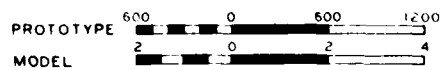
PHOTO 12



VELOCITY SCALE



SCALES IN FEET



SURFACE CURRENTS  
1973 CONDITIONS  
MEAN TIDE WITH  
MASONBORO CHANNEL CLOSED  
HOUR 4

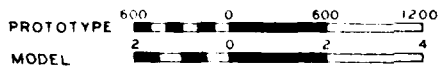
PHOTO 13



VELOCITY SCALE



SCALES IN FEET



**SURFACE CURRENTS**  
**1973 CONDITIONS**  
**MEAN TIDE WITH**  
**MASONBORO CHANNEL CLOSED**  
**HOOR 7**

PHOTO 14



VELOCITY SCALE



SCALES IN FEET



**SURFACE CURRENTS**  
1973 CONDITIONS  
MEAN TIDE WITH  
MASONBORO CHANNEL CLOSED  
HOUR 10

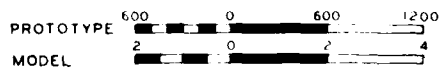
PHOTO 15



VELOCITY SCALE



SCALES IN FEET

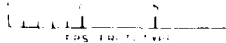


**SURFACE CURRENTS**  
1973 CONDITIONS  
MEAN TIDE WITH  
MASONBORO CHANNEL CLOSED  
HOUR 0

PHOTO 16



VELOCITY SCALE



SCALE S IN FEET



SURFACE CURRENTS  
 1964 CONDITIONS  
 MEAN TIDE  
 HOUR 10

PHOTO 17

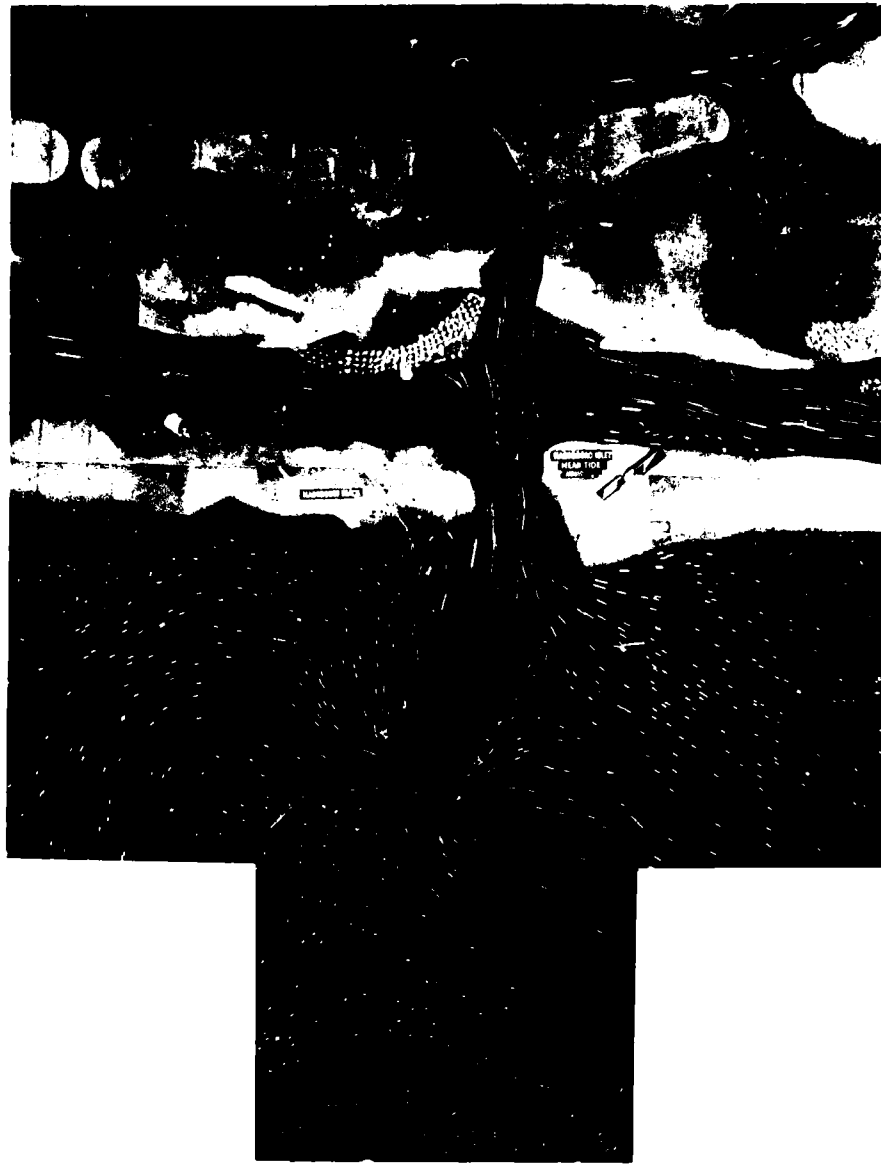


VELOCITY SCALE  
 5 0 5 10  
 FT. PROTOTYPE

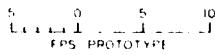
SCALES IN FEET  
 PROTOTYPE 800 0 800 1200 1800  
 MODEL 2 0 2 4 6

SURFACE CURRENTS  
 1973 CONDITIONS  
 WITH SOUTH JETTY  
 PLAN B INSTALLED  
 MEAN TIDE  
 HOUR 4

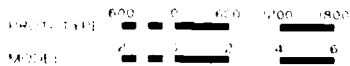
PHOTO 18



VELOCITY SCALE



SCALES IN FEET



**SURFACE CURRENTS**

**1973 CONDITIONS  
WITH SOUTH JETTY  
PLAN B INSTALLED**

**MEAN TIDE**

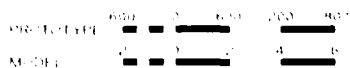
**HOUR 7**



VELOCITY SCALE



SCALE S IN FEET



SURFACE CURRENTS

1973 CONDITIONS  
WITH SOUTH JETTY  
PLAN B INSTALLED

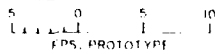
MEAN TIDE

HOUR 9

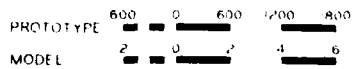
PHOTO 20



VELOCITY SCALE



SCALES IN FEET



**SURFACE CURRENTS**

1973 CONDITIONS  
WITH SOUTH JETTY  
PLAN B INSTALLED

MEAN TIDE

HOOR 10

PHOTO 21



VELOCITY SCALE  
 5 0 5 10  
 1 1 1 1 1 1 1 1 1 1  
 FPS. PROTOTYPE

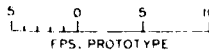
SCALES IN FEET  
 PROTOTYPE 600 0 600 1200 1800  
 MODEL 2 0 2 4 6

SURFACE CURRENTS  
 1973 CONDITIONS  
 WITH SOUTH JETTY  
 PLAN B INSTALLED  
 MEAN TIDE  
 HOUR 11

PHOTO 22



VELOCITY SCALE



SCALES IN FEET

	600	0	600	200	400
PROTOTYPE	—	—	—	—	—
MODEL	2	0	2	4	6

**SURFACE CURRENTS**

1973 CONDITIONS  
WITH SOUTH JETTY  
PLAN B INSTALLED

MEAN TIDE

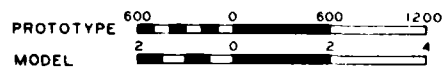
HOUR 0



VELOCITY SCALE



SCALES IN FEET

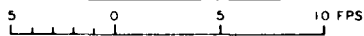


**SURFACE CURRENTS**  
**1973 CONDITIONS**  
**MEAN TIDE WITH**  
**NORTH AND SOUTH JETTY**  
**WIER ELEVATION 2.0 FT MLW**  
**HOUR 4**

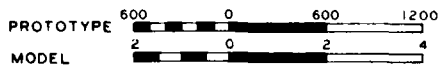
PHOTO 24



VELOCITY SCALE



SCALES IN FEET



**SURFACE CURRENTS**  
**1973 CONDITIONS**  
**MEAN TIDE WITH**  
**NORTH AND SOUTH JETTY**  
**WIER ELEVATION 2.0 FT MLW**  
**HOUR 7**

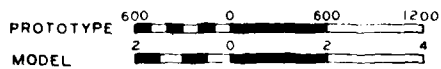
PHOTO 25



VELOCITY SCALE



SCALES IN FEET



SURFACE CURRENTS  
 1973 CONDITIONS  
 MEAN TIDE WITH  
 NORTH AND SOUTH JETTY  
 WIER ELEVATION 2.0 FT MLW  
 HOUR 9

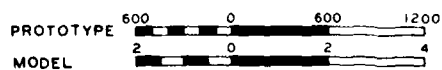
PHOTO 26



VELOCITY SCALE



SCALES IN FEET



SURFACE CURRENTS  
1973 CONDITIONS  
MEAN TIDE WITH  
NORTH AND SOUTH JETTY  
WIER ELEVATION 2.0 FT MLW  
HOUR 10

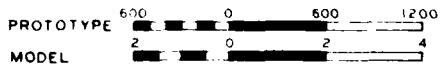
PHOTO 27



VELOCITY SCALE



SCALES IN FEET



SURFACE CURRENTS  
1973 CONDITIONS  
MEAN TIDE WITH  
NORTH AND SOUTH JETTY  
WIER ELEVATION 2.0 FT MLW  
HOUR 11

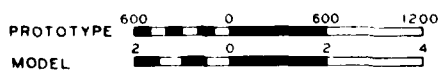
PHOTO 28



VELOCITY SCALE



SCALES IN FEET



**SURFACE CURRENTS**  
1973 CONDITIONS  
MEAN TIDE WITH  
NORTH AND SOUTH JETTY  
WIER ELEVATION 2.0 FT MLW  
HOUR 0

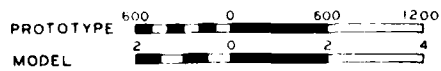
PHOTO 29



VELOCITY SCALE



SCALES IN FEET



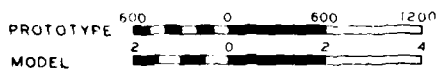
**SURFACE CURRENTS**  
 1973 CONDITIONS  
 MEAN TIDE WITH  
 NORTH AND SOUTH JETTY  
 WIER ELEVATION 0.0 FT MLW  
 HOUR 4



VELOCITY SCALE



SCALES IN FEET



**SURFACE CURRENTS**  
 1973 CONDITIONS  
 MEAN TIDE WITH  
 NORTH AND SOUTH JETTY  
 WIER ELEVATION 0.0 FT MLW  
 H J U R 7

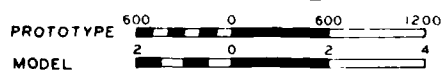
PHOTO 31



VELOCITY SCALE



SCALES IN FEET



**SURFACE CURRENTS**  
1973 CONDITIONS  
MEAN TIDE WITH  
NORTH AND SOUTH JETTY  
WIER ELEVATION 0.0 FT MLW  
HOUR 9

PHOTO 32



VELOCITY SCALE

5 0 5 10 FPS

SCALES IN FEET

PROTOTYPE	0	600	1200
MODEL	0	2	4

**SURFACE CURRENTS**  
 1973 CONDITIONS  
 MEAN TIDE WITH  
 NORTH AND SOUTH JETTY  
 WIER ELEVATION 0.0 FT MLW  
 HOUR 10

PHOTO 33



**VELOCITY SCALE**



**SCALES IN FEET**



**SURFACE CURRENTS**  
 1973 CONDITIONS  
 MEAN TIDE WITH  
 NORTH AND SOUTH JETTY  
 WIER ELEVATION 0.0 FT MLW  
 HOUR 11

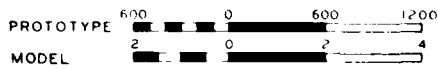
PHOTO 34



VELOCITY SCALE



SCALES IN FEET



SURFACE CURRENTS  
 1973 CONDITIONS  
 MEAN TIDE WITH  
 NORTH AND SOUTH JETTY  
 WIER ELEVATION 0.0 FT MLW  
 HOUR 0

PHOTO 35



TEST CONDITIONS

MATERIAL USED PLASTIC SP. GR. = 1.18

TIDE 3.8 FT RANGE

MEAN TIDAL LEVEL = 0.5 FT MSL

WAVES USED N 84° E (2.0 FT - 6.5 SEC)

S 16° E (1.25 FT - 10 SEC)

TEST DURATION DURING EACH CYCLE CONTINUOUS

WAVE DIRECTION SEQUENCE PER CYCLE NS NS NS

MATERIAL FEEDING YES TOTAL AMT 8000 ML

POST TEST 1964-2  
INLET AND OCEAN SHOALS

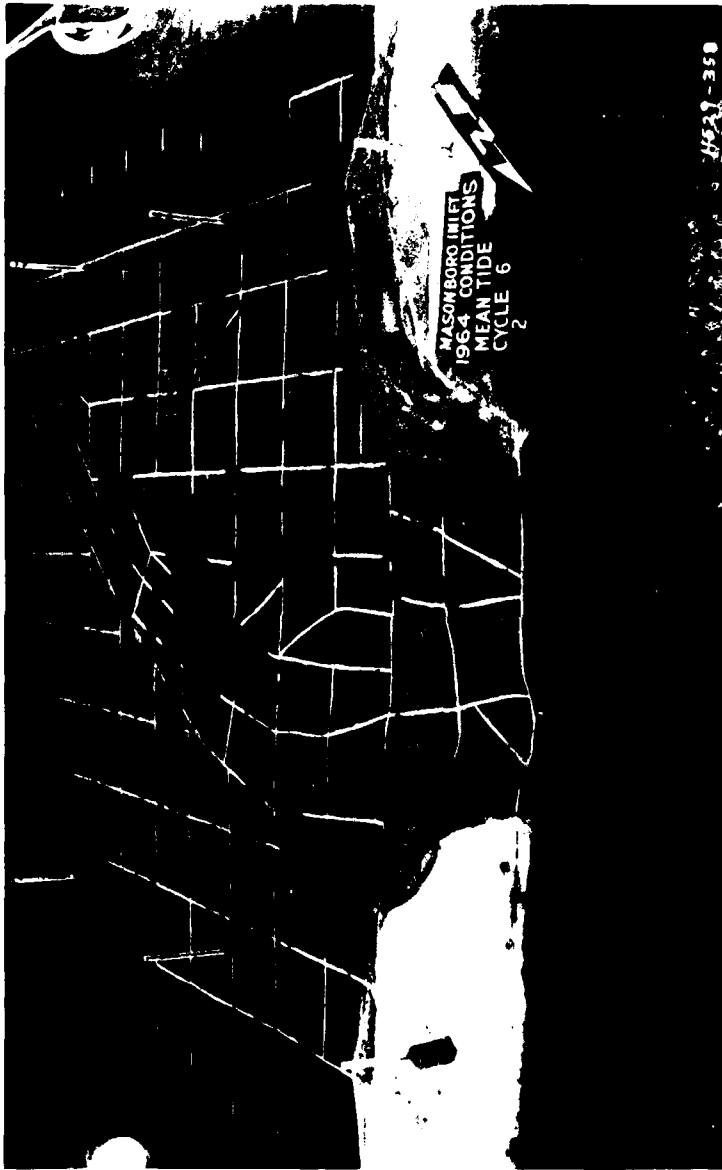


POST TEST 1964-2  
LOOKING BAYWARD

TEST CONDITIONS

MATERIAL USED PLASTIC SP. GR. = 1.18  
TIDE 3.8 FT RANGE  
MEAN TIDAL LEVEL = 0.5 FT MSL  
WAVES USED N 84° E (2.0 FT - 6.5 SEC)  
S 16° E (1.25 FT - 10 SEC)  
TEST DURATION DURING EACH CYCLE CONTINUOUS  
WAVE DIRECTION SEQUENCE PER CYCLE NS NS NS  
MATERIAL FEEDING YES TOTAL AMT 8000 ML

PHOTO 37



TEST CONDITIONS

MATERIAL USED PLASTIC SP. GR. = 1.18

TIDE 3.8 FT RANGE

MEAN TIDAL LEVEL 0.5 FT MSL

WAVES USED N 84° E (2.0 FT - 6.5 SEC)

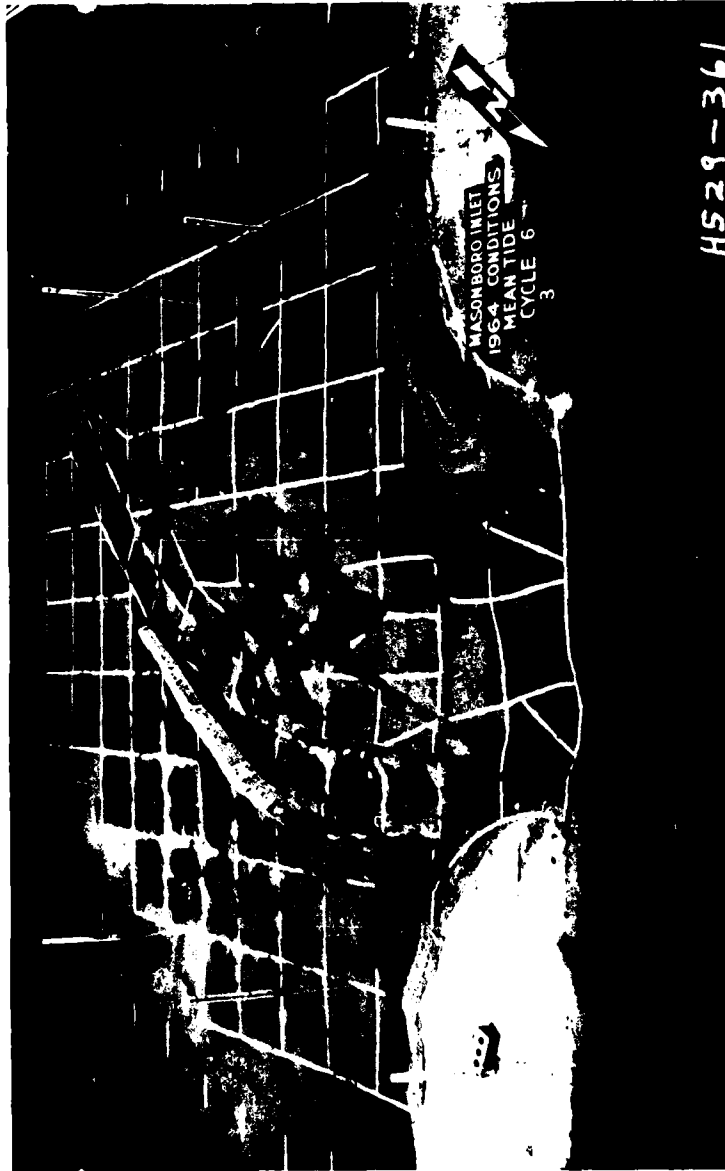
S 16° E (1.25 FT - 10 SEC)

TEST DURATION DURING EACH CYCLE CONTINUOUS

WAVE DIRECTION SEQUENCE PER CYCLE NS NS NS

MATERIAL FEEDING YES TOTAL AMT 8000 ML

POST TEST 1964 - 2  
SHOALING RESULTS



TEST CONDITIONS

MATERIAL USED PLASTIC SP. GR. = 1.18

TIDE 3.8 FT RANGE

MEAN TIDAL LEVEL = 0.5 FT MSL

WAVES USED N 84° E (2.0 FT - 6.5 SEC)

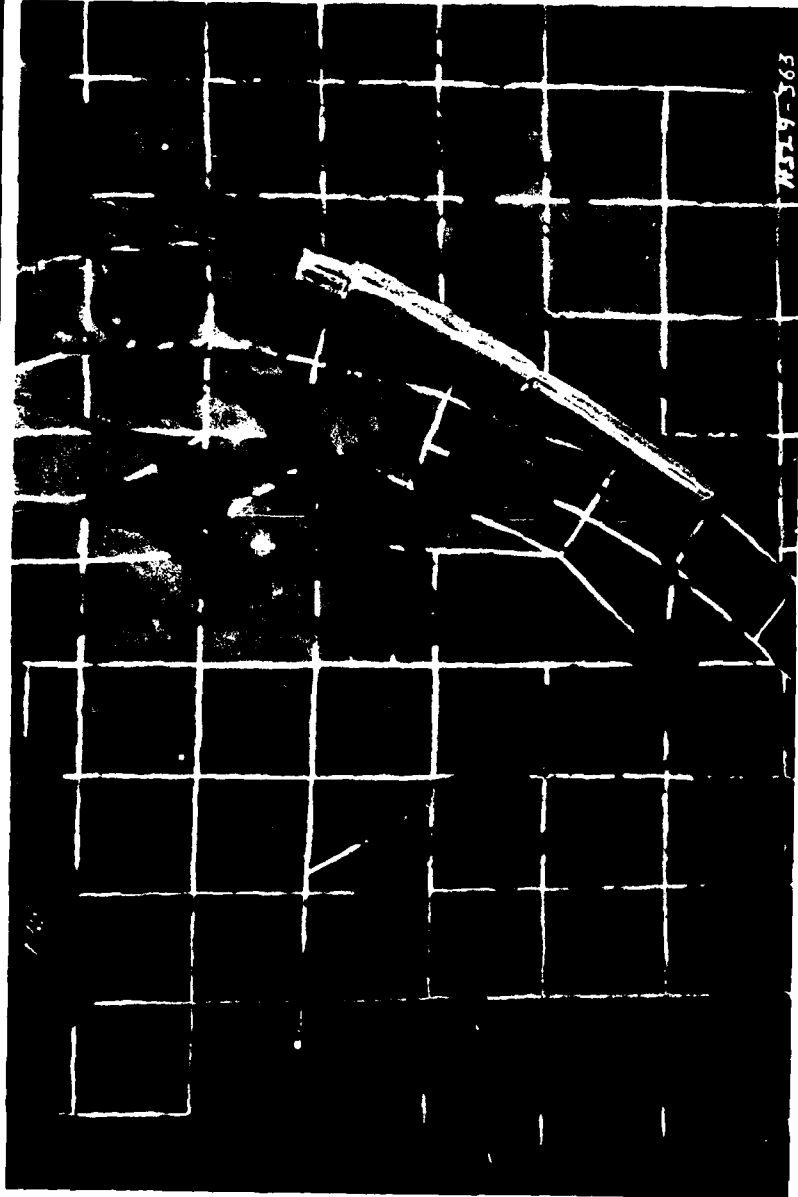
S 16° E (1.25 FT - 10 SEC)

TEST DURATION DURING EACH CYCLE CONTINUOUS

WAVE DIRECTION SEQUENCE PER CYCLE NS NS NS

MATERIAL FEEDING YES TOTAL AMT 8000 ML

POST TEST 1964 - 3  
SHOALING RESULTS



TEST CONDITIONS

MATERIAL USED PLASTIC SP. GR. = 1.18

TIDE 3.8 FT RANGE

MEAN TIDAL LEVEL = 0.5 FT MSL

WAVES USED N 84° E (2.0 FT - 6.5 SEC)

S 16° E (1.25 FT - 10 SEC)

TEST DURATION DURING EACH CYCLE CONTINUOUS

WAVE DIRECTION SEQUENCE PER CYCLE NS NS NS

MATERIAL FEEDING YES TOTAL AMT 8000 ML

POST TEST 1964 - 3  
CHANNEL AND DEPOSITION BASIN



TEST CONDITIONS

MATERIAL USED PLASTIC SP. GR. = 1.18

TIDE 3.8 FT RANGE

MEAN TIDAL LEVEL = 0.5 FT MSL

WAVES USED N 84° E (2.0 FT - 6.5 SEC)

S 16° E (1.25 FT - 10 SEC)

TEST DURATION DURING EACH CYCLE CONTINUOUS

WAVE DIRECTION SEQUENCE PER CYCLE NS NS NS

MATERIAL FEEDING YES TOTAL AMT 8000 ML

POST TEST 1964-3  
LOOKING BAYWARD

PHOTO 41



TEST CONDITIONS

MATERIAL USED PLASTIC SP GR = 1.18

TIDE 3.8 FT RANGE

MEAN TIDAL LEVEL = 0.5 FT MSL

WAVES USED N 84 E (2.10 FT - 6.5 SEC)

S 16 E (1.25 FT - 10 SEC)

TEST DURATION DURING EACH CYCLE CONTINUOUS

WAVE DIRECTION SEQUENCE PER CYCLE NS NS NS

MATERIAL FEEDING YES TOTAL AMT 8000 ML

POST TEST 1966-1  
SHOALING RESULTS

PHOTO 42

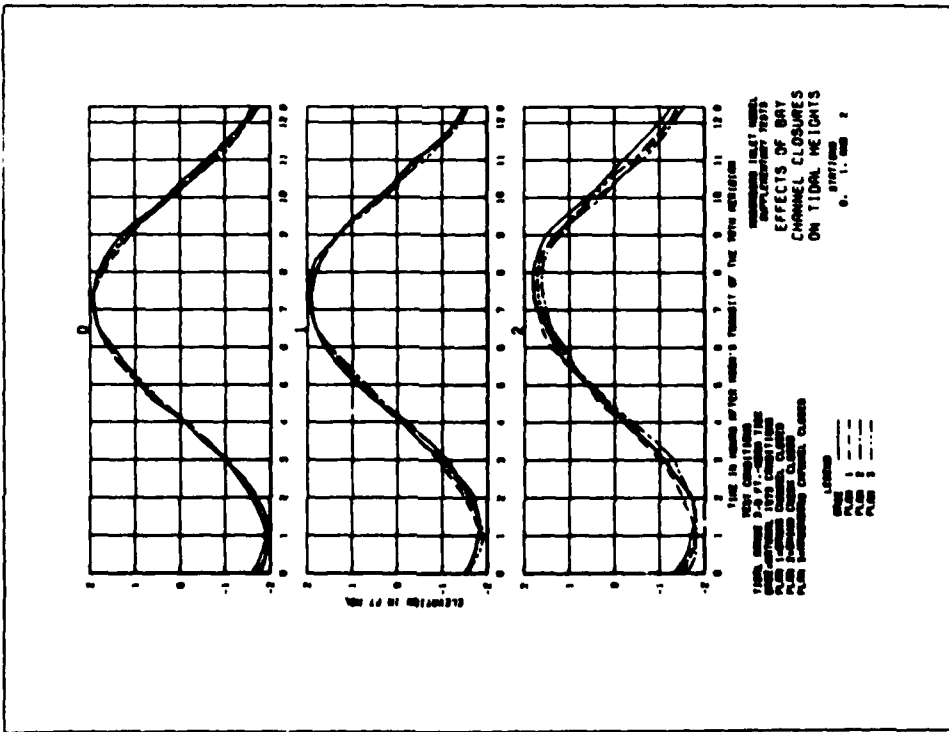


PLATE 1

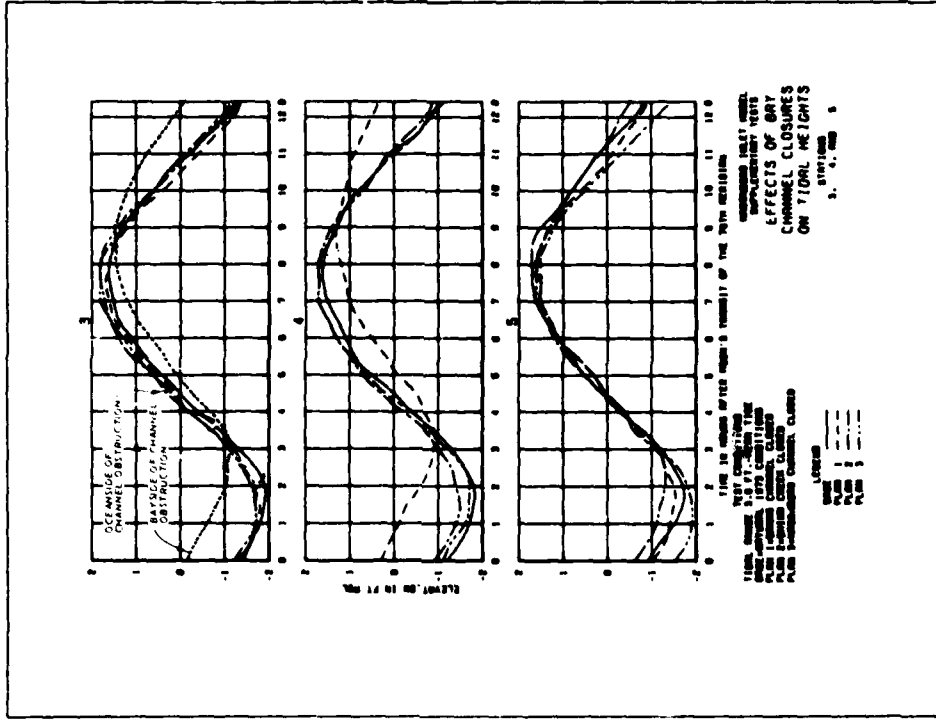
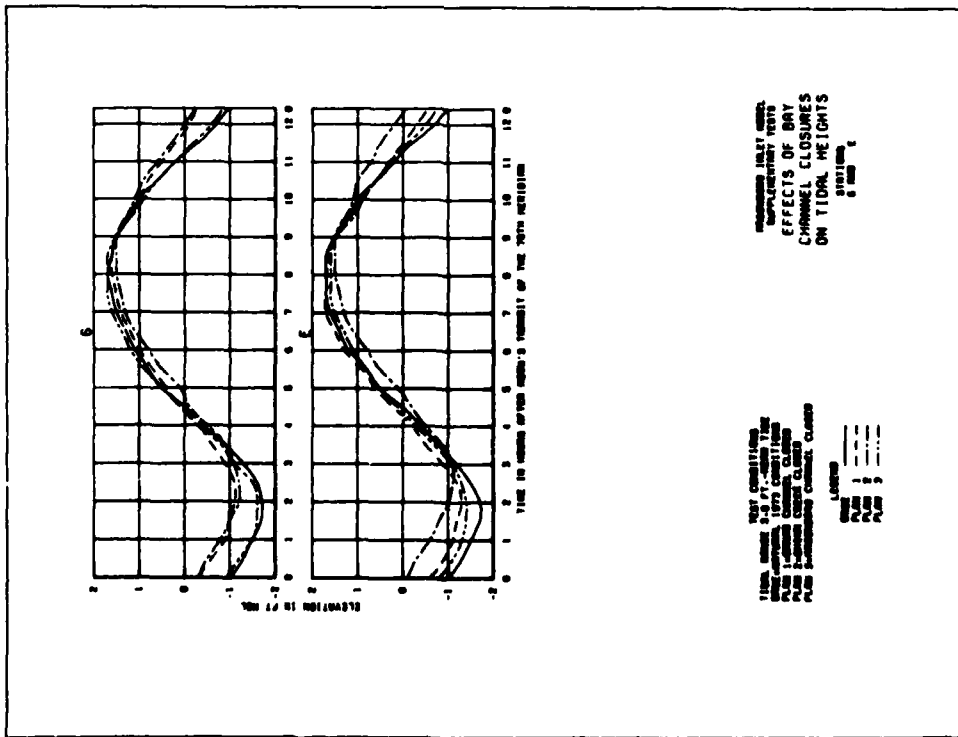
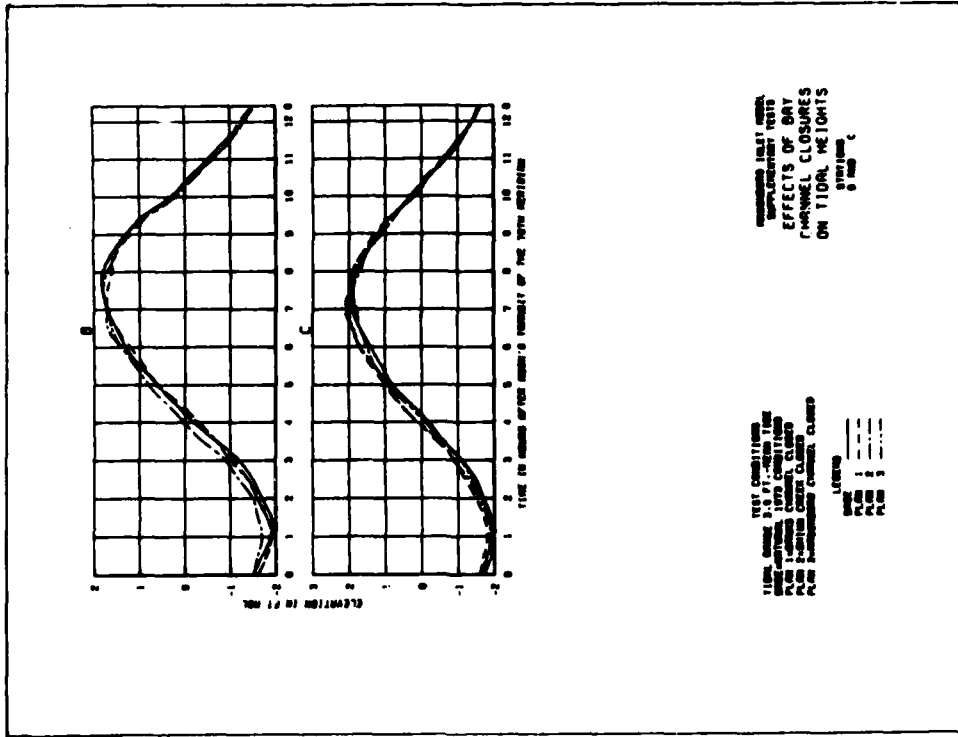


PLATE 2



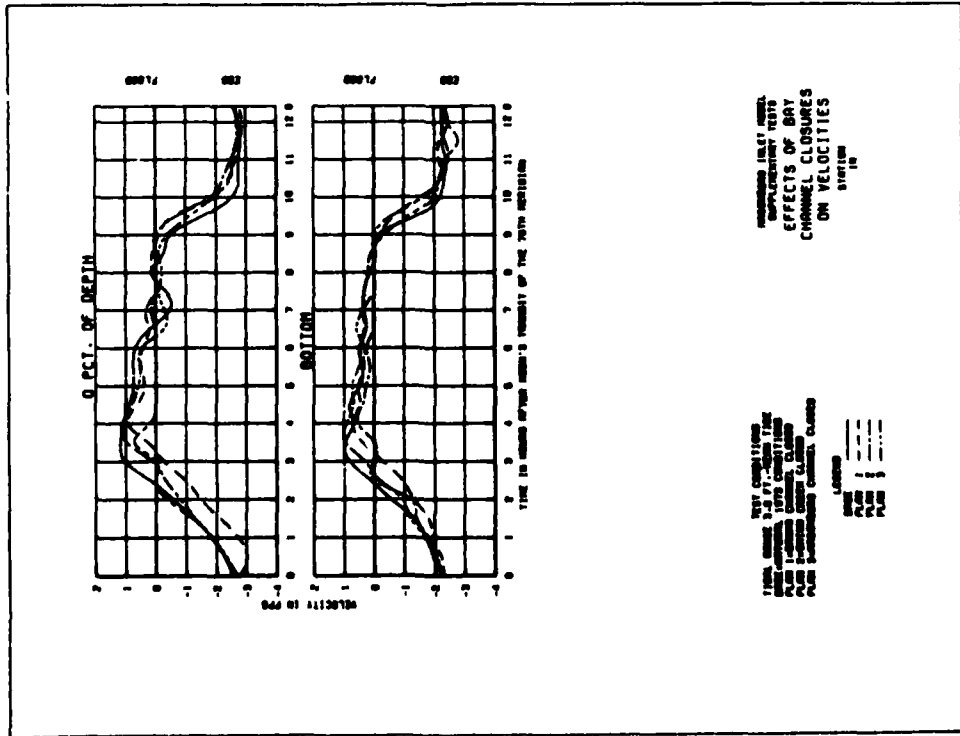


PLATE 5

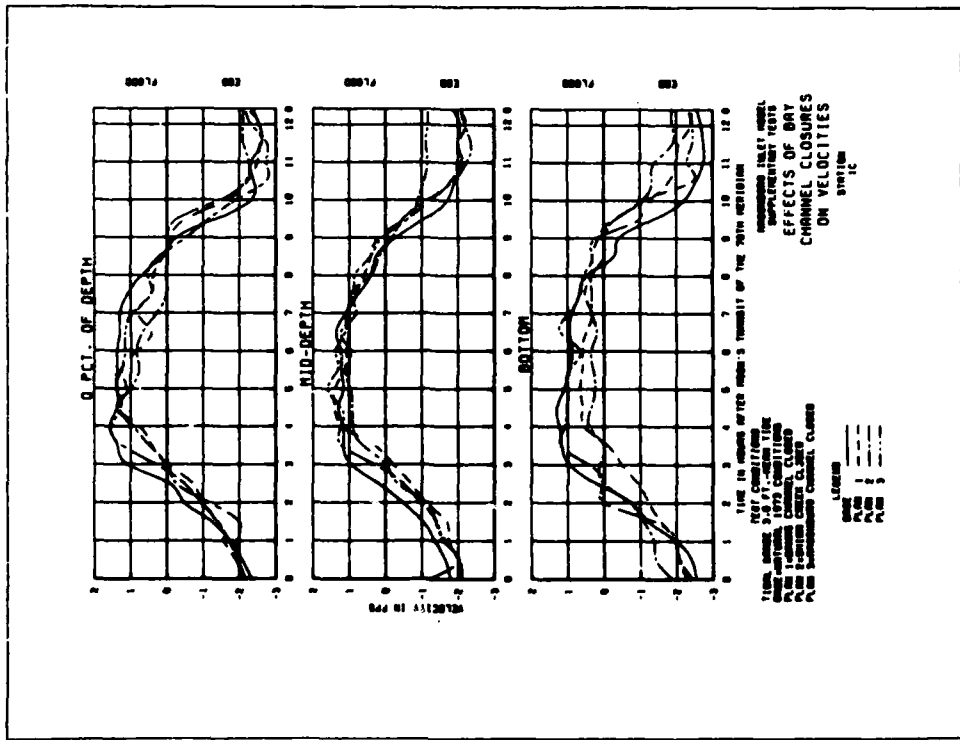


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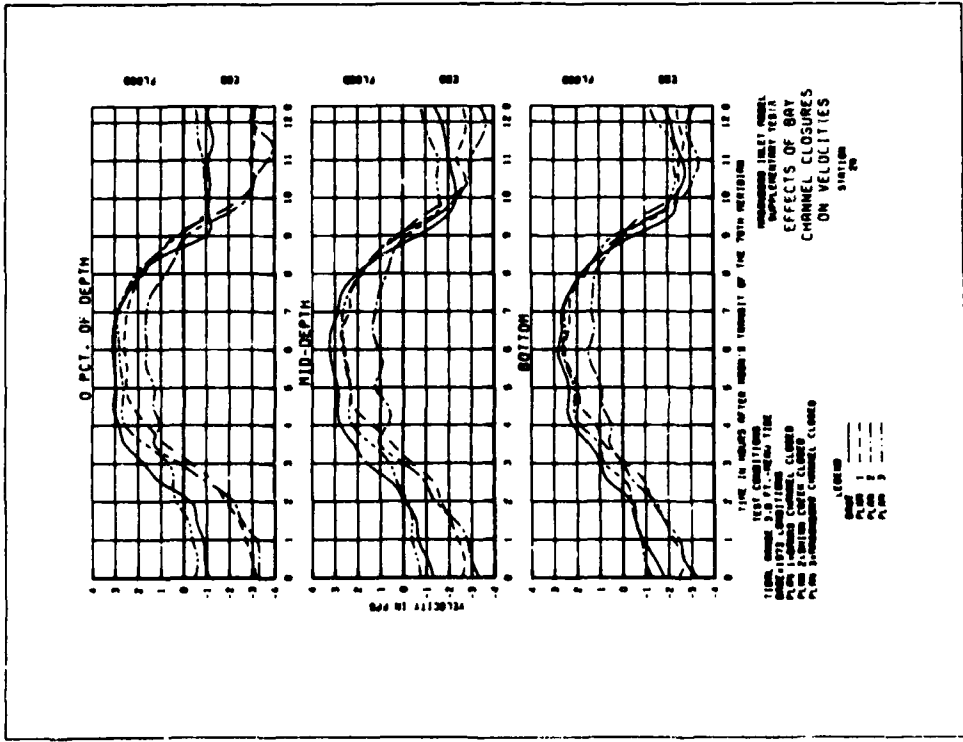


PLATE 8

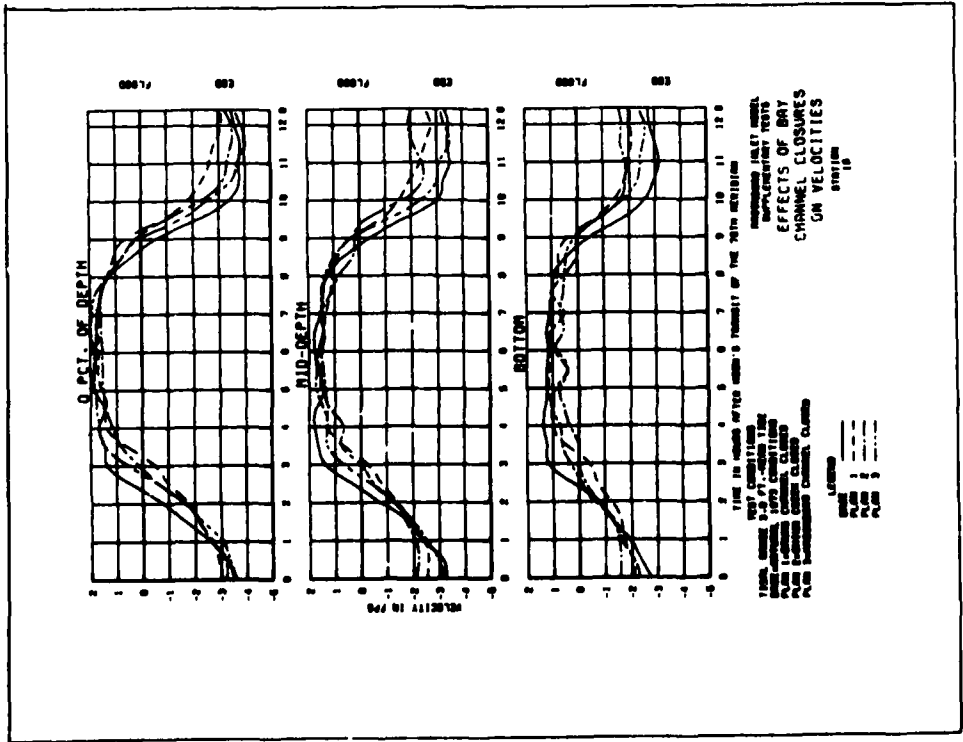


PLATE 7

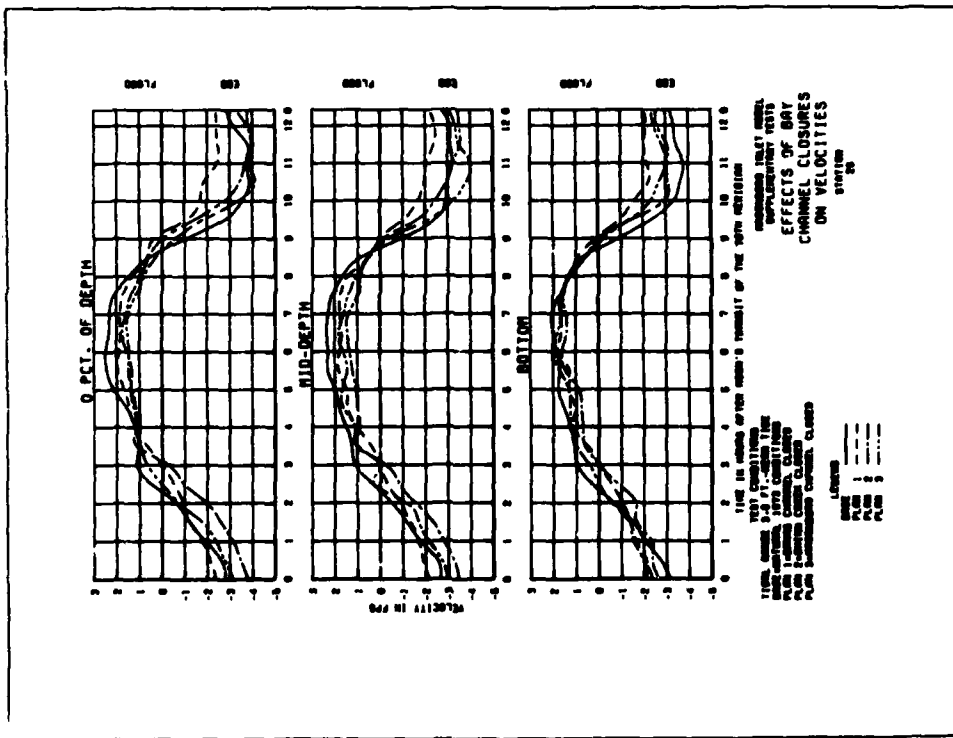


PLATE 10

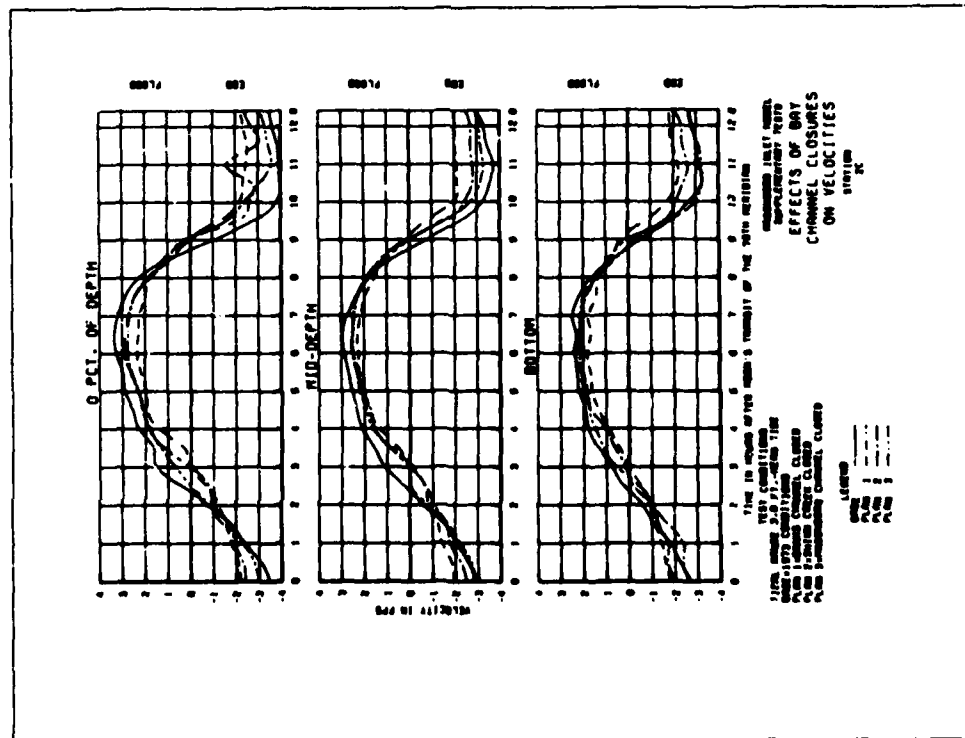


PLATE 9



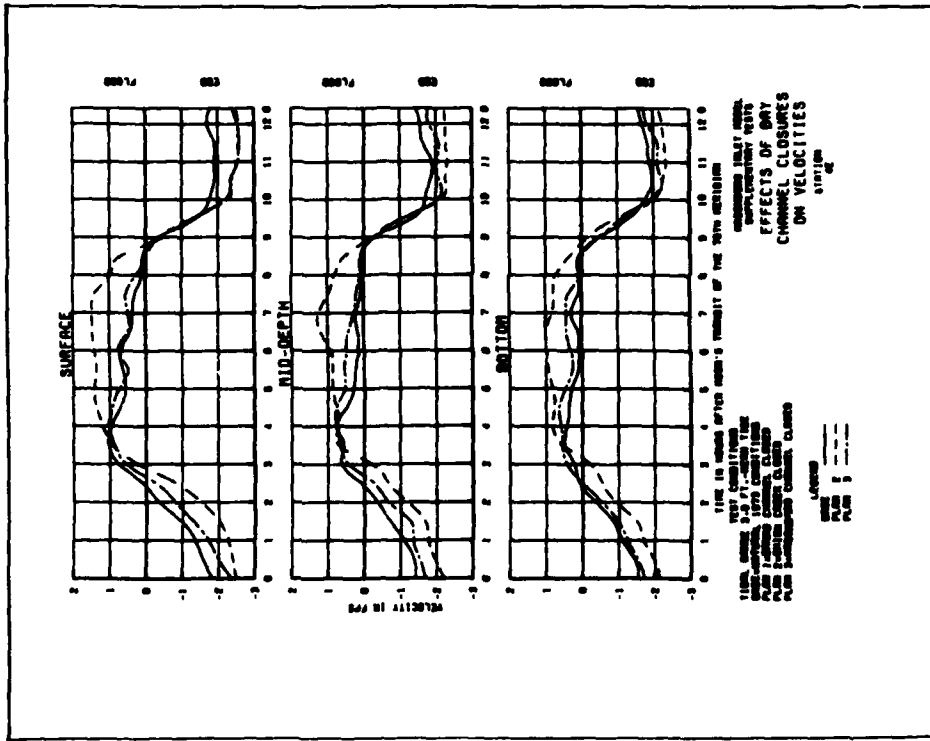


PLATE 14

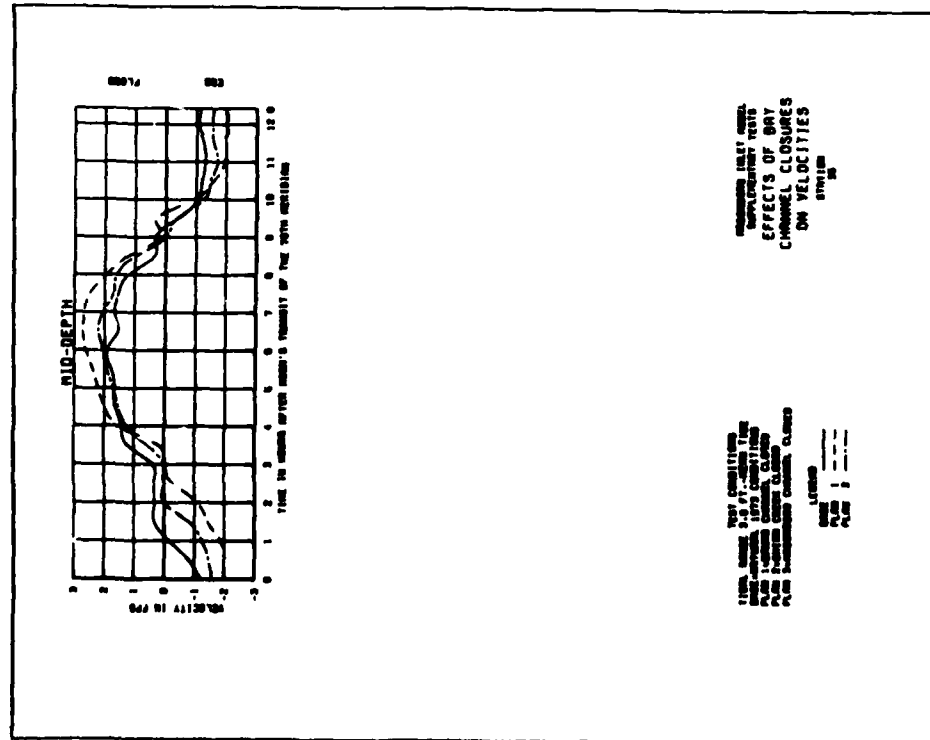


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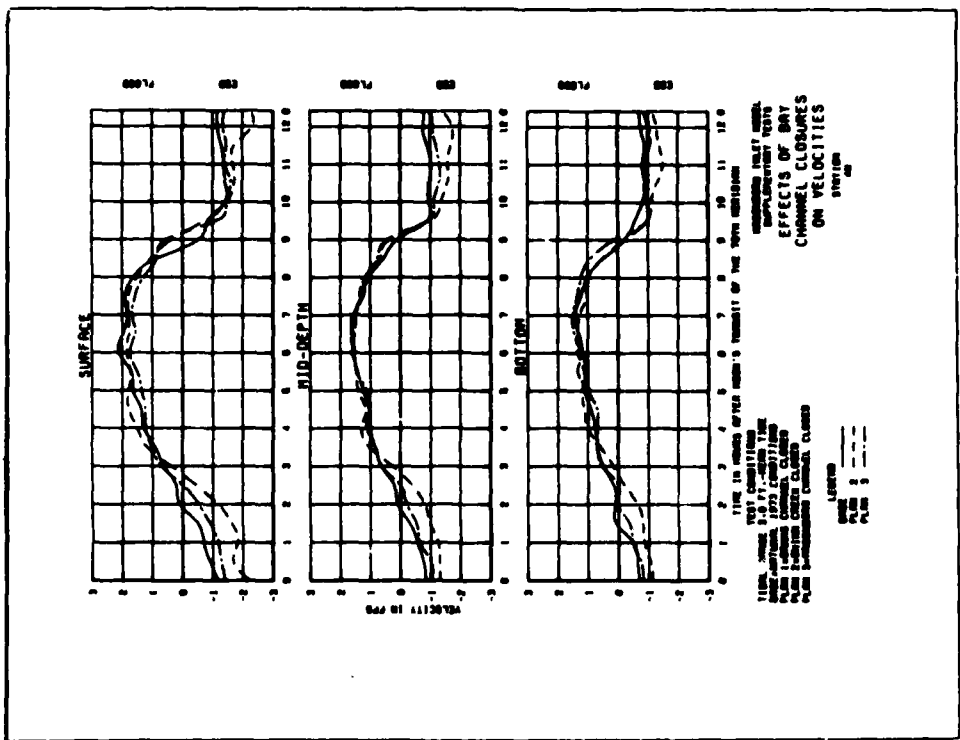


PLATE 15

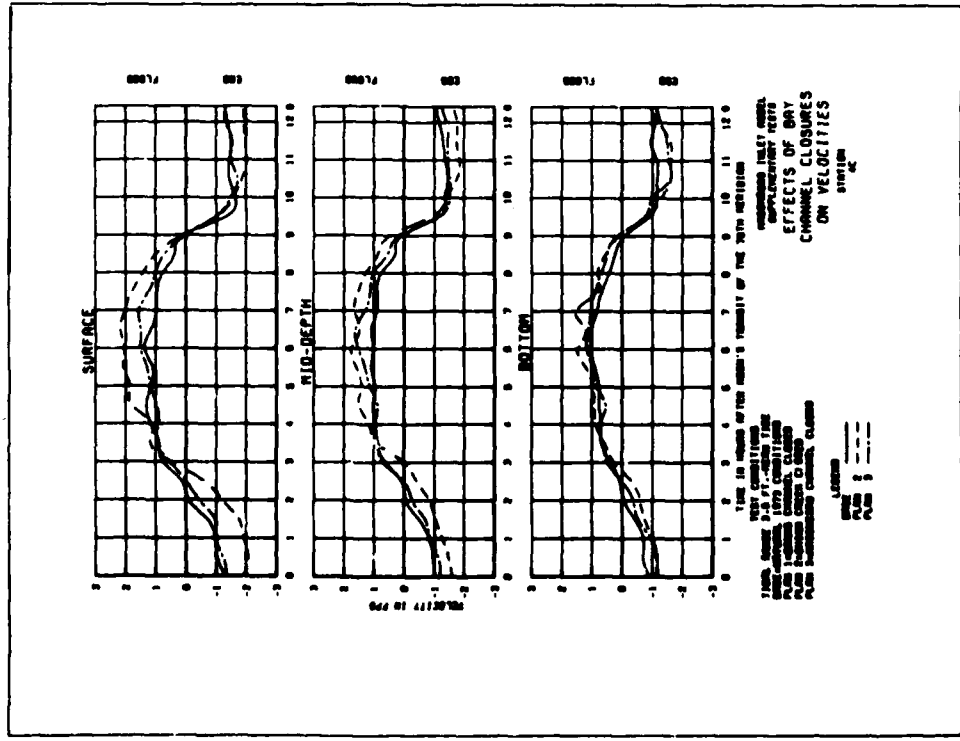


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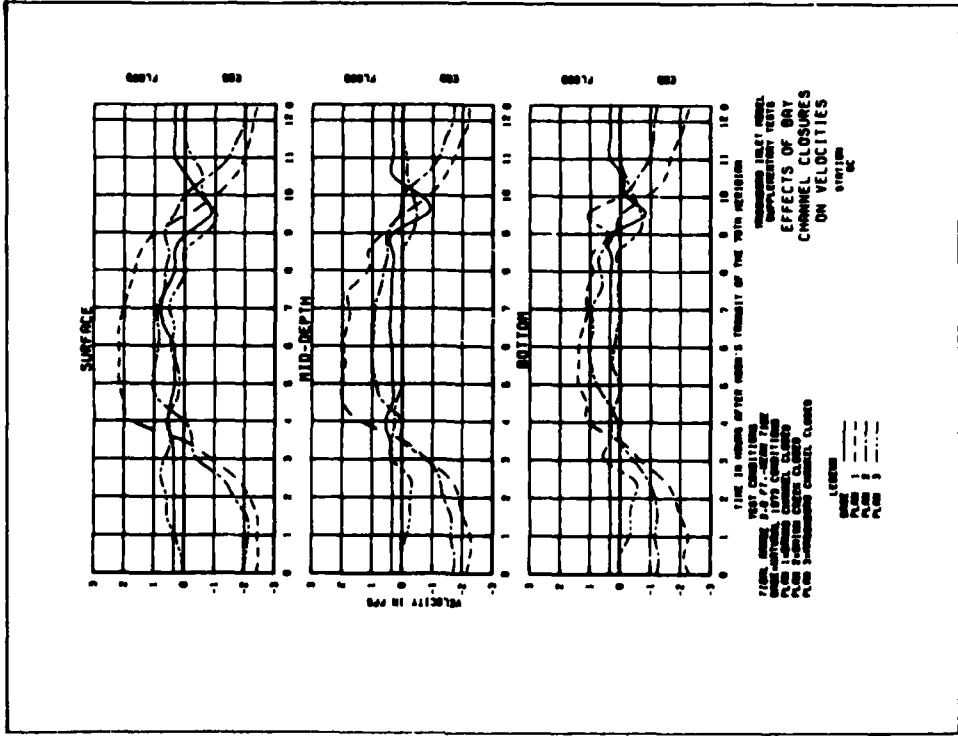


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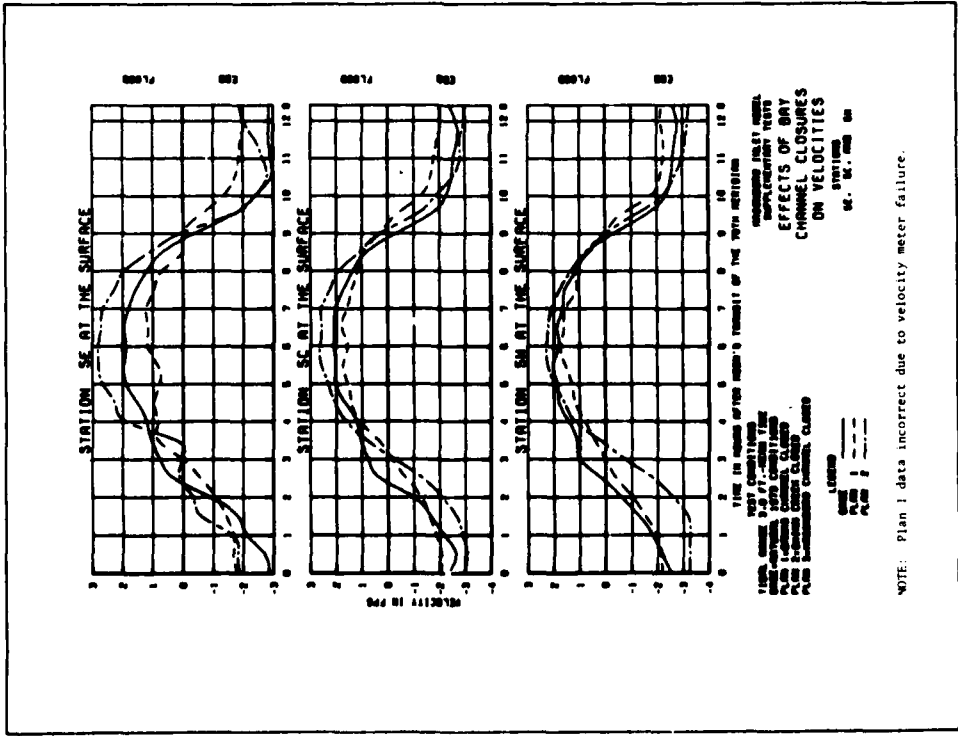


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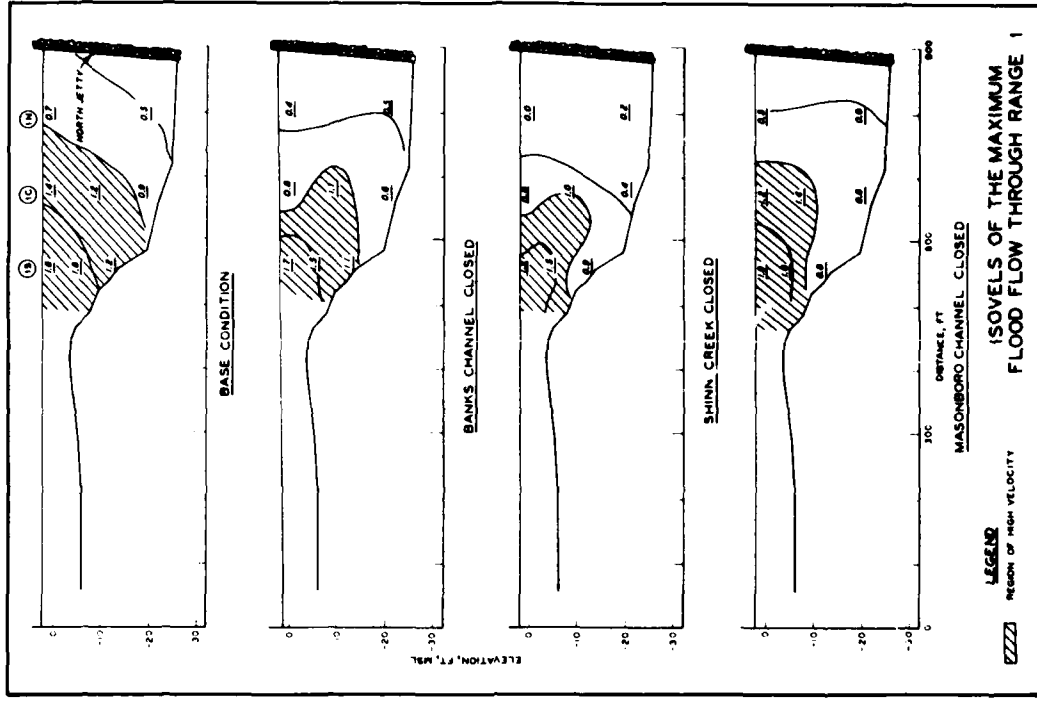


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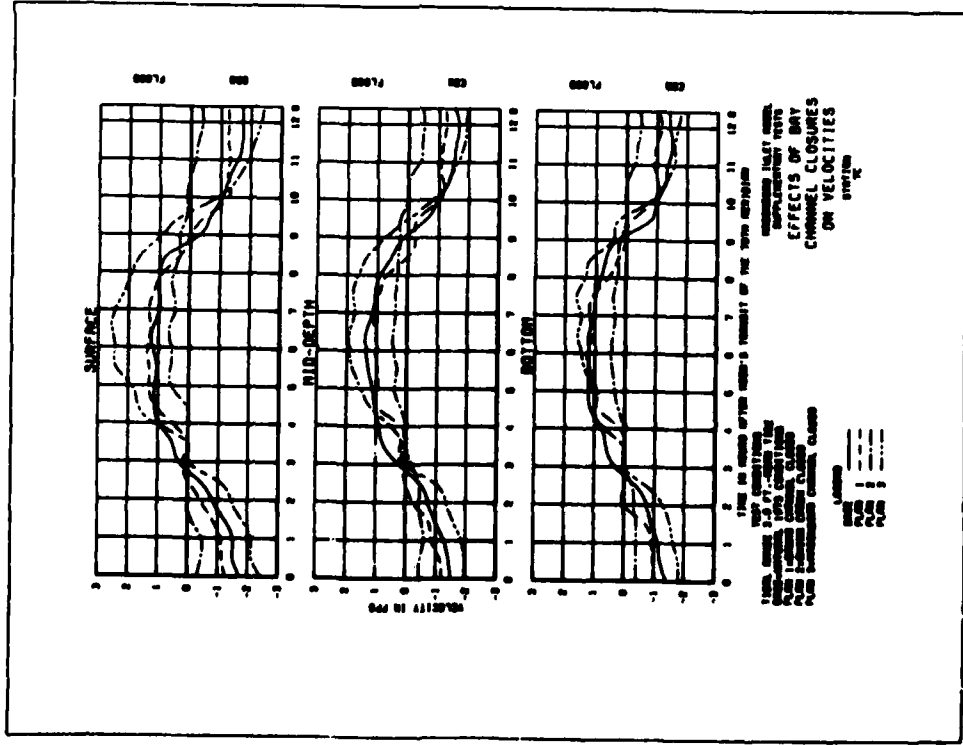
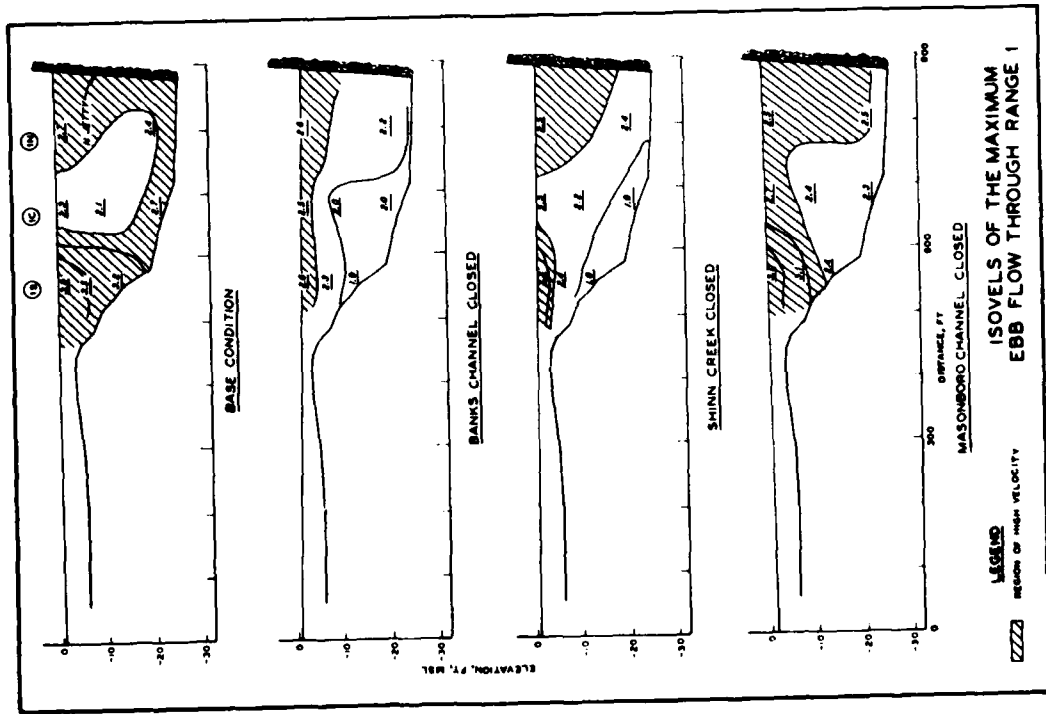
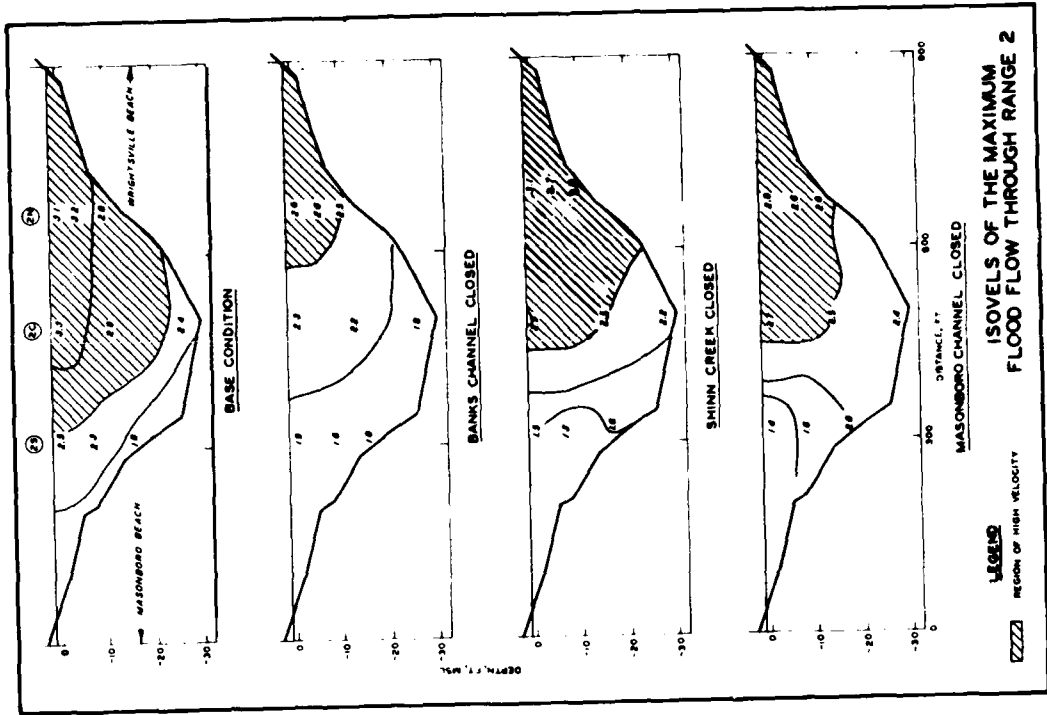
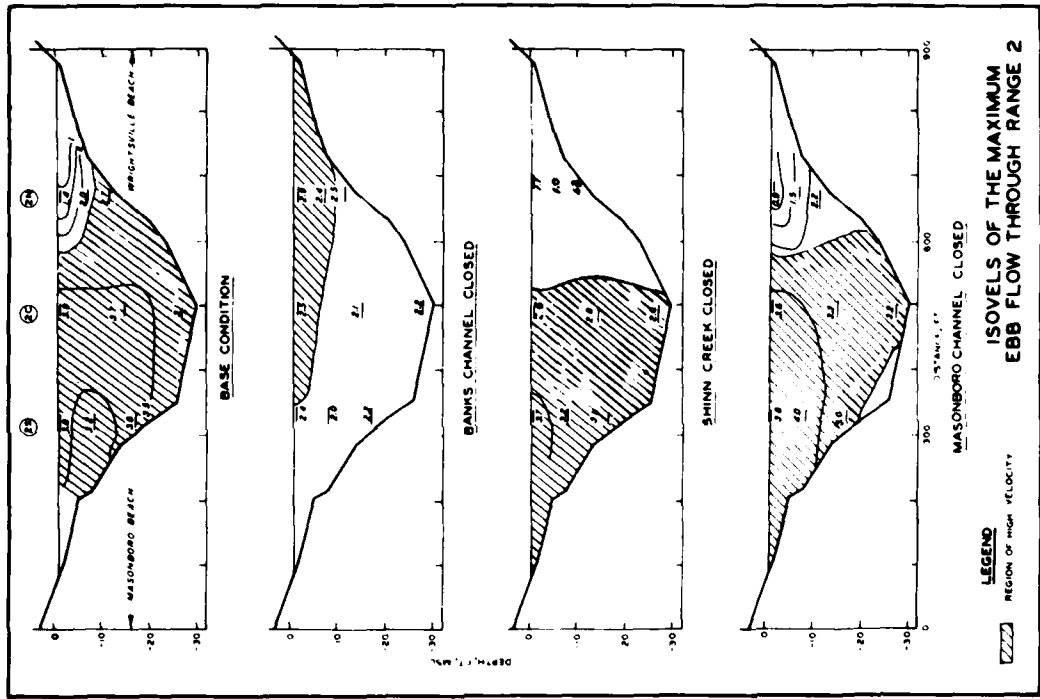


PLATE 19





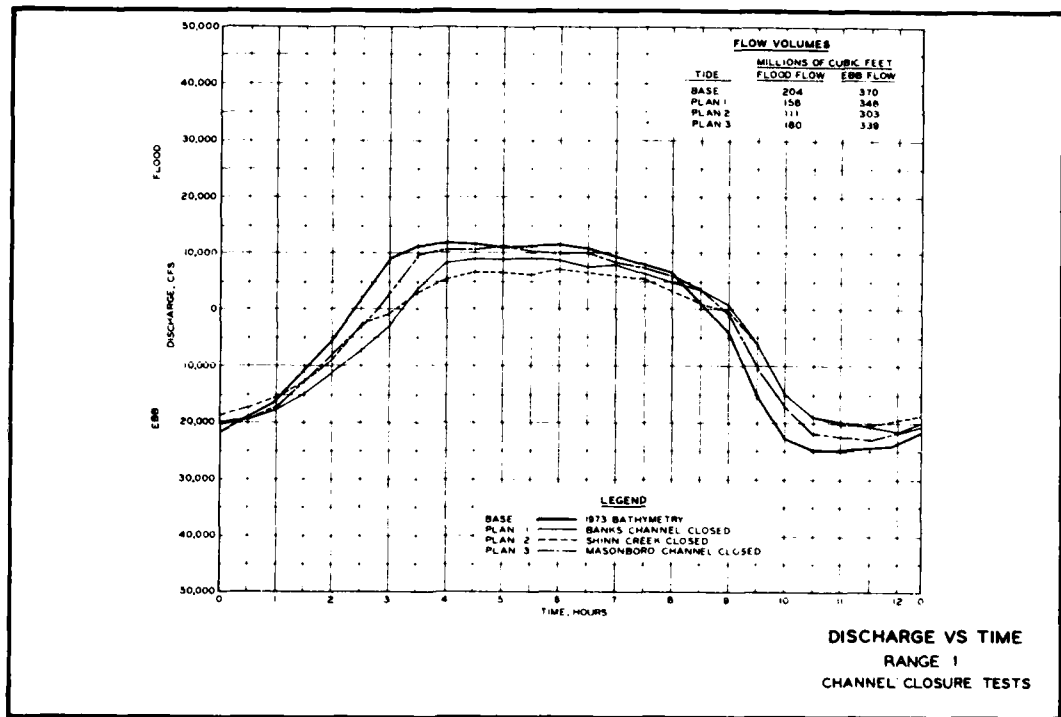


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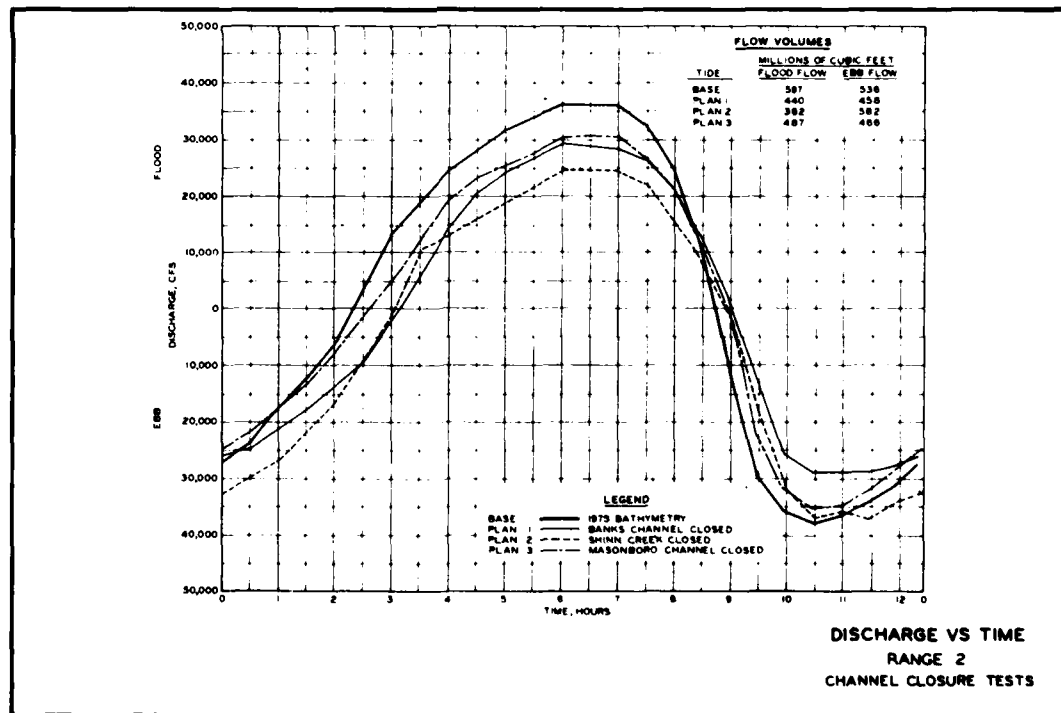


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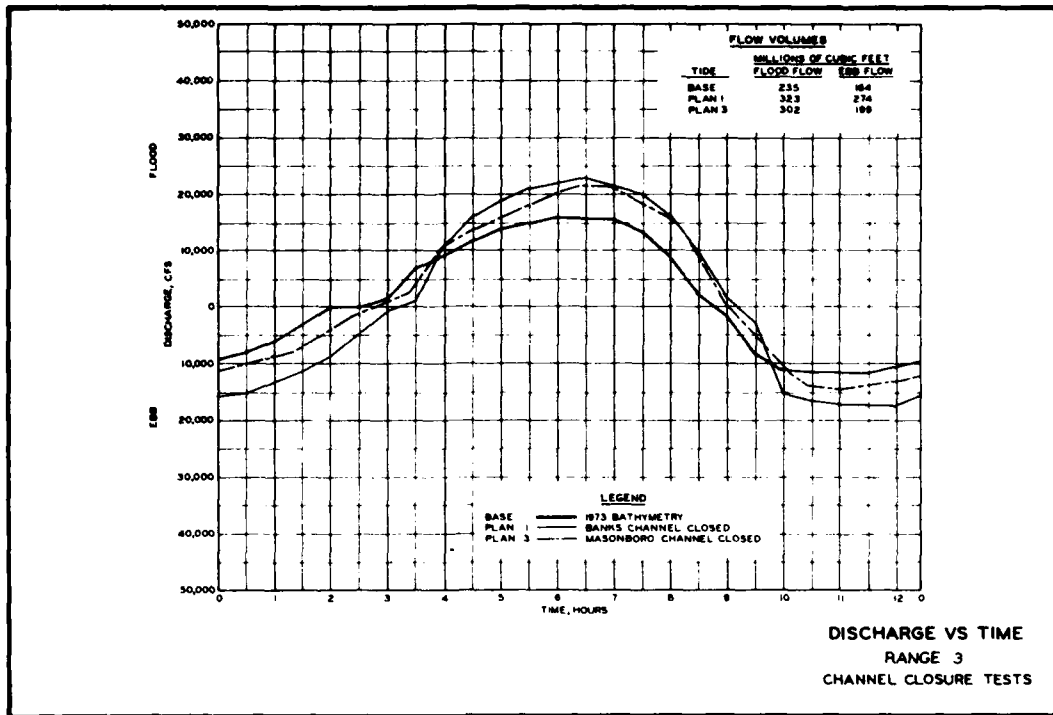


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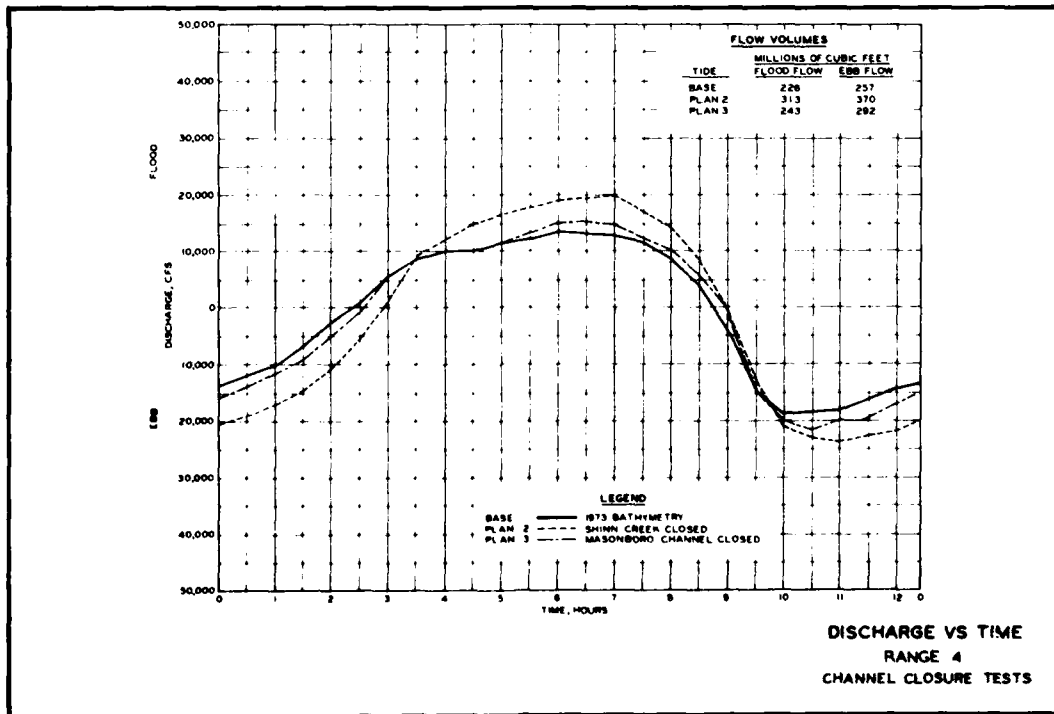


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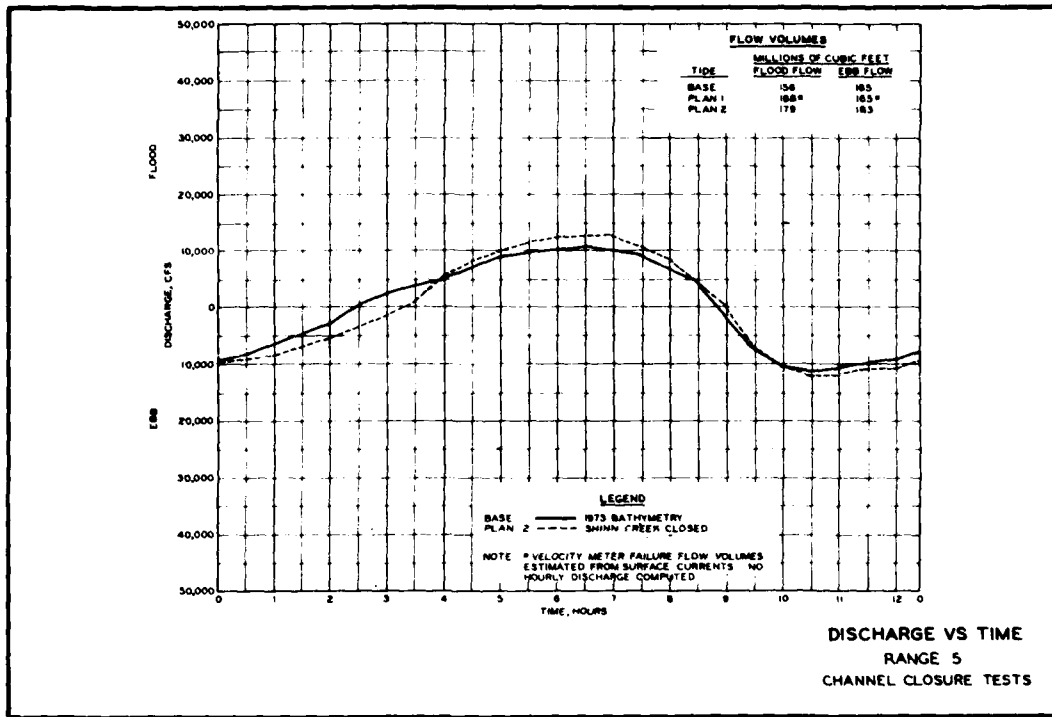


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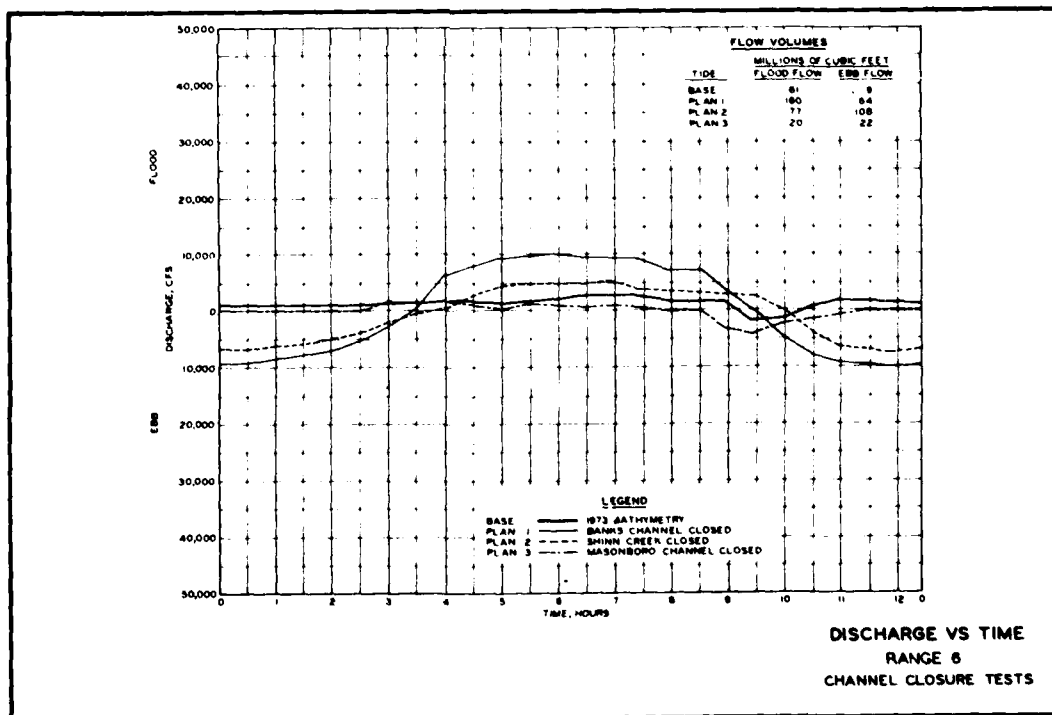


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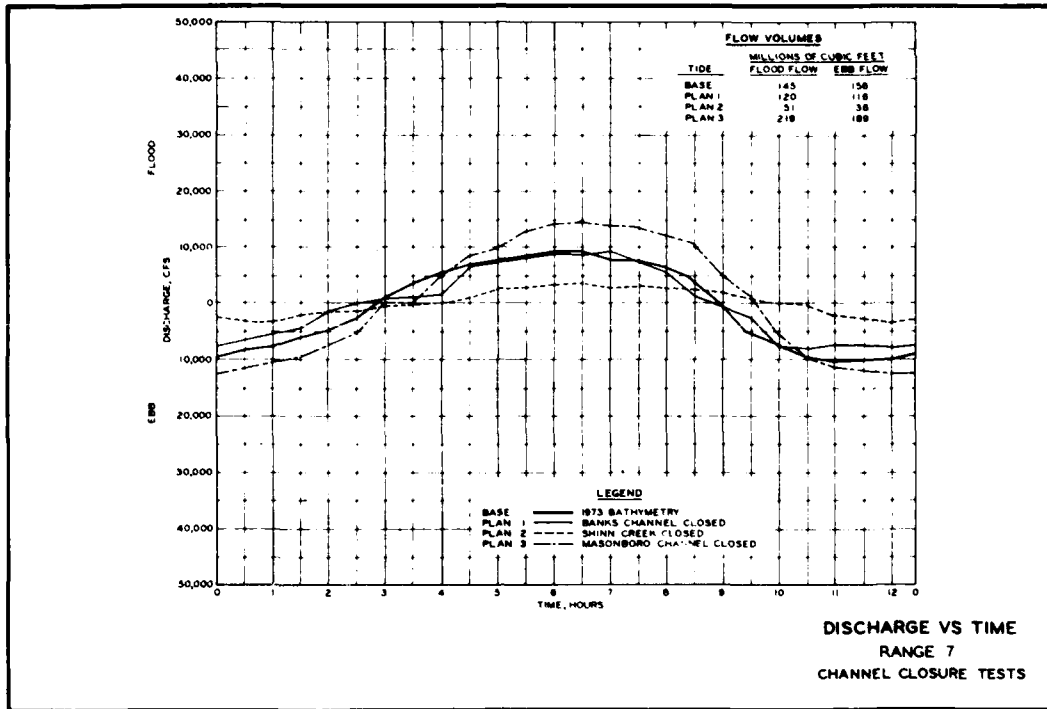


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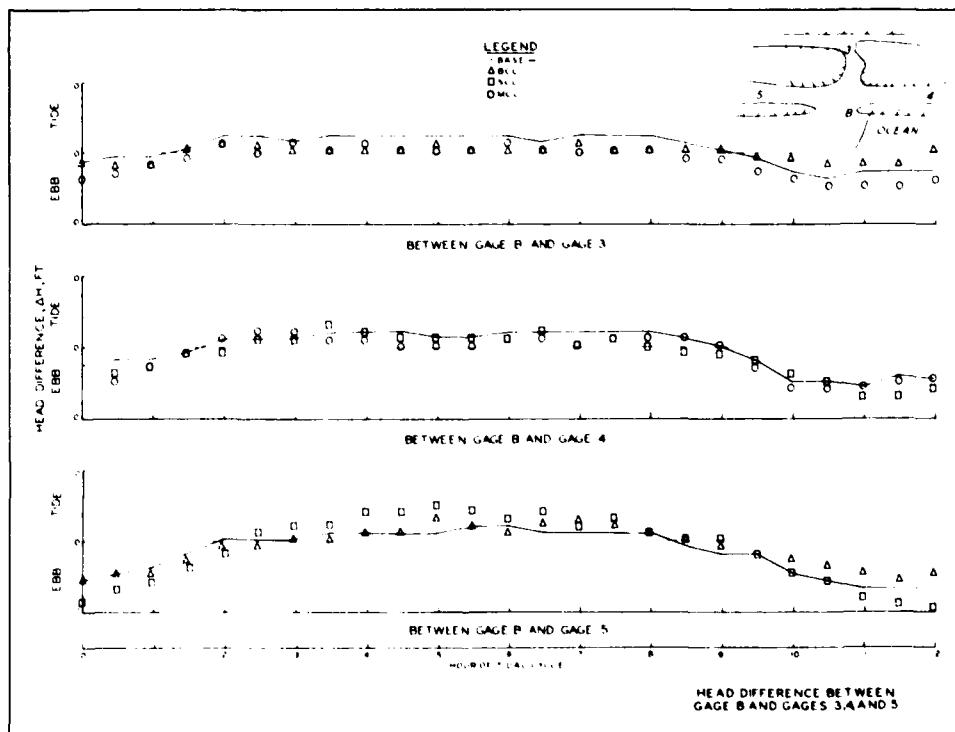
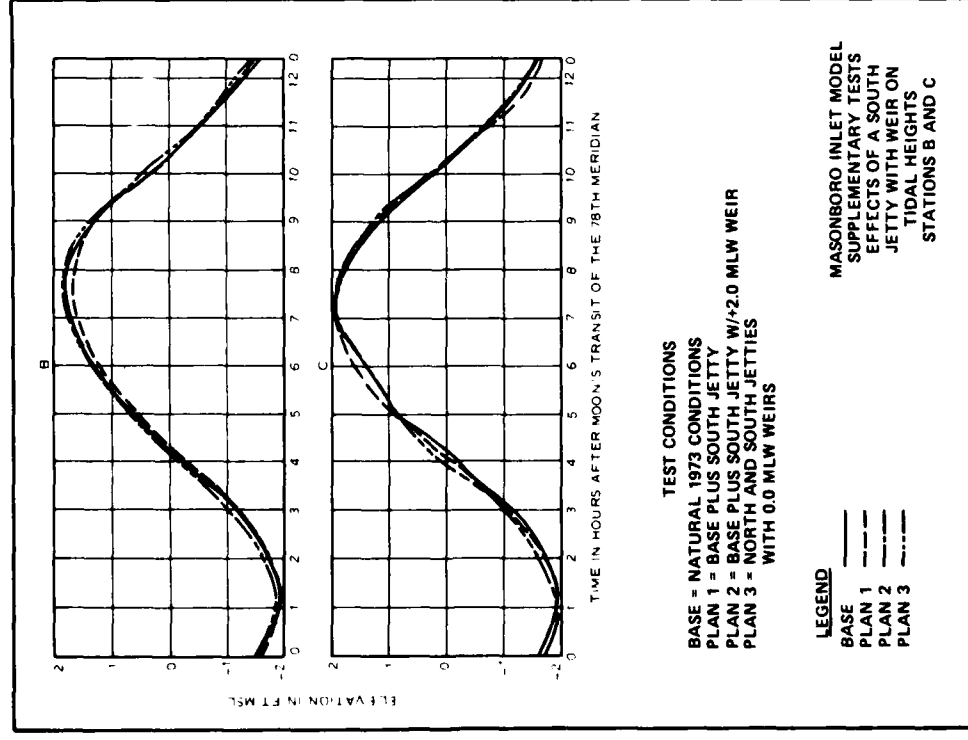
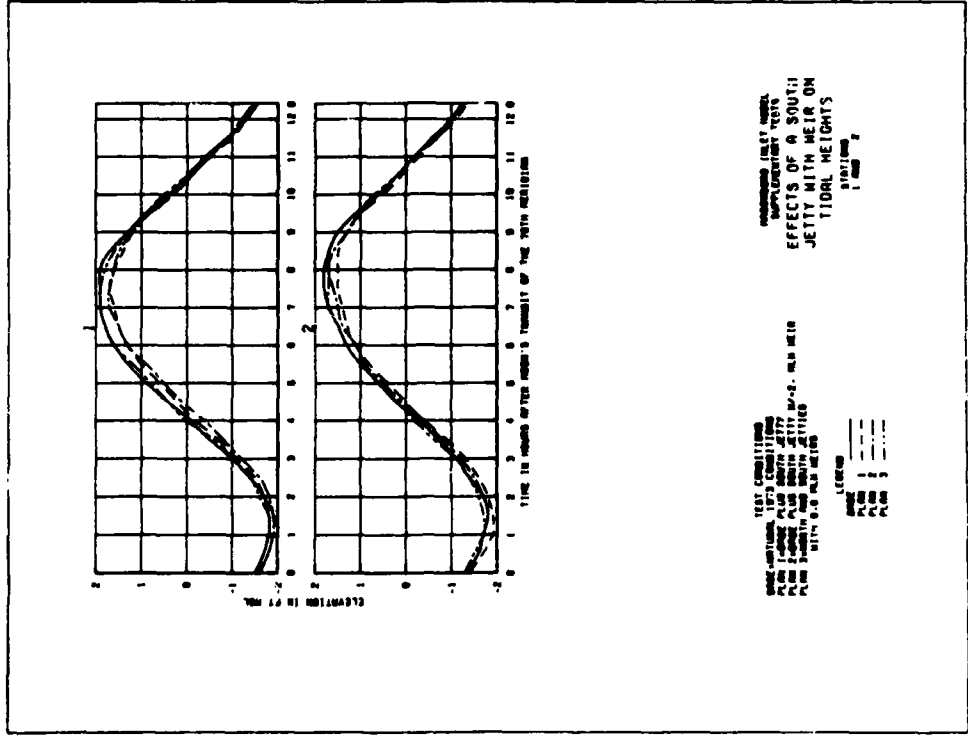


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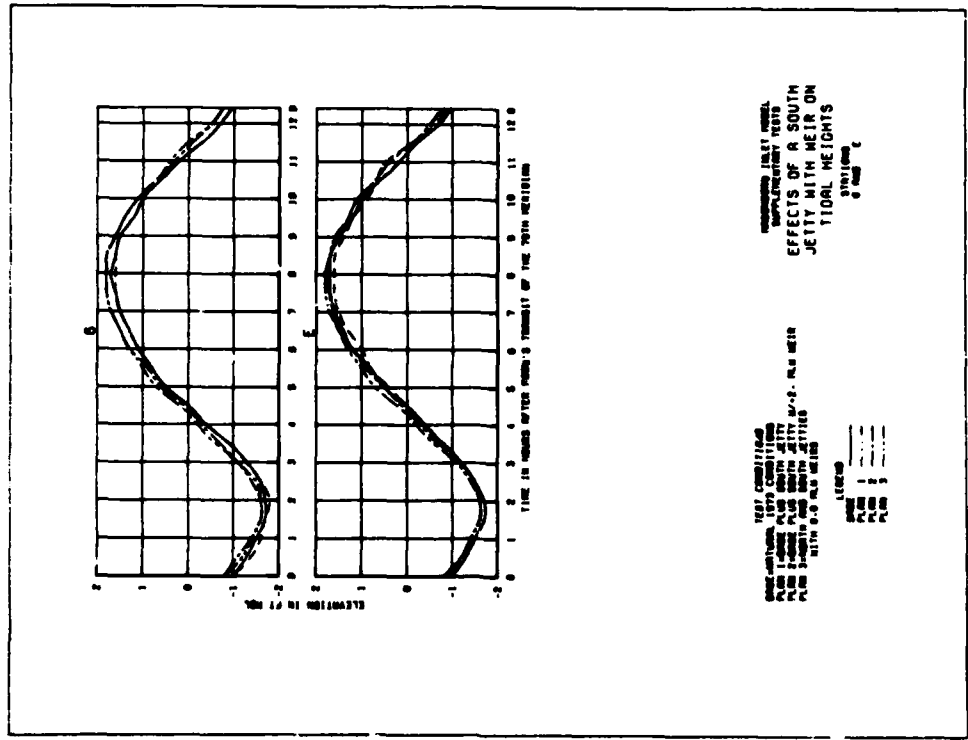


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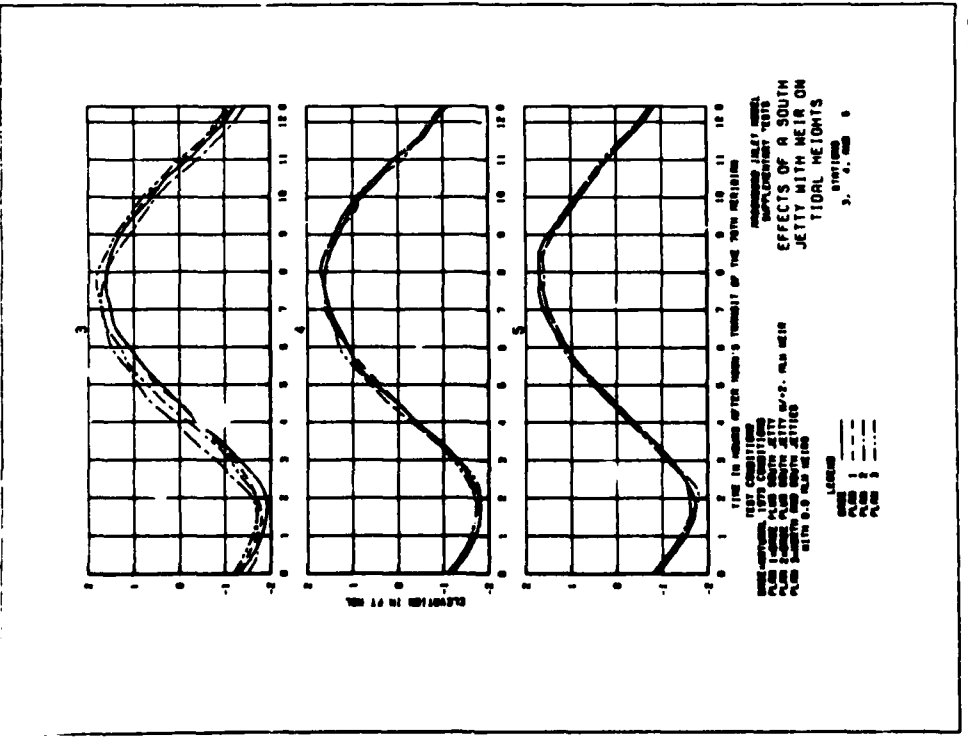


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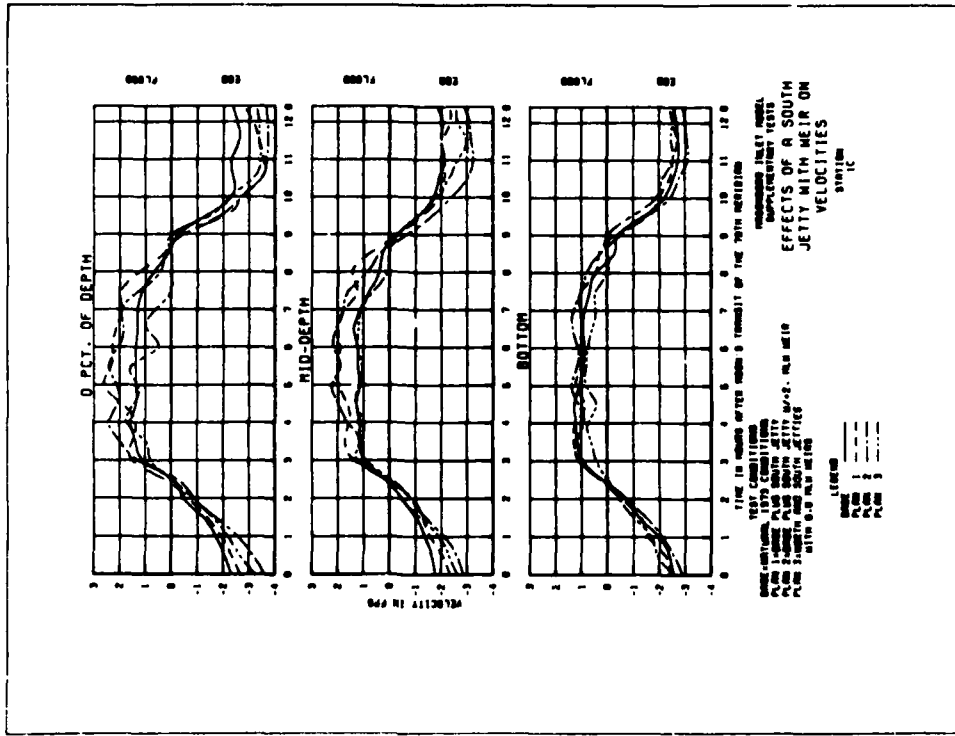


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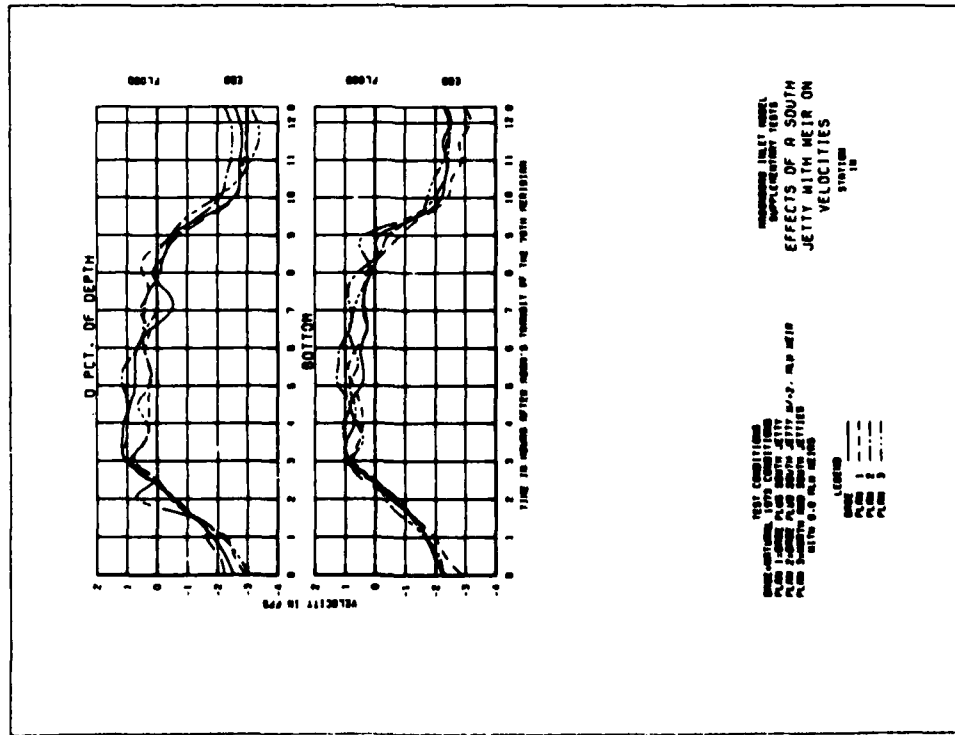


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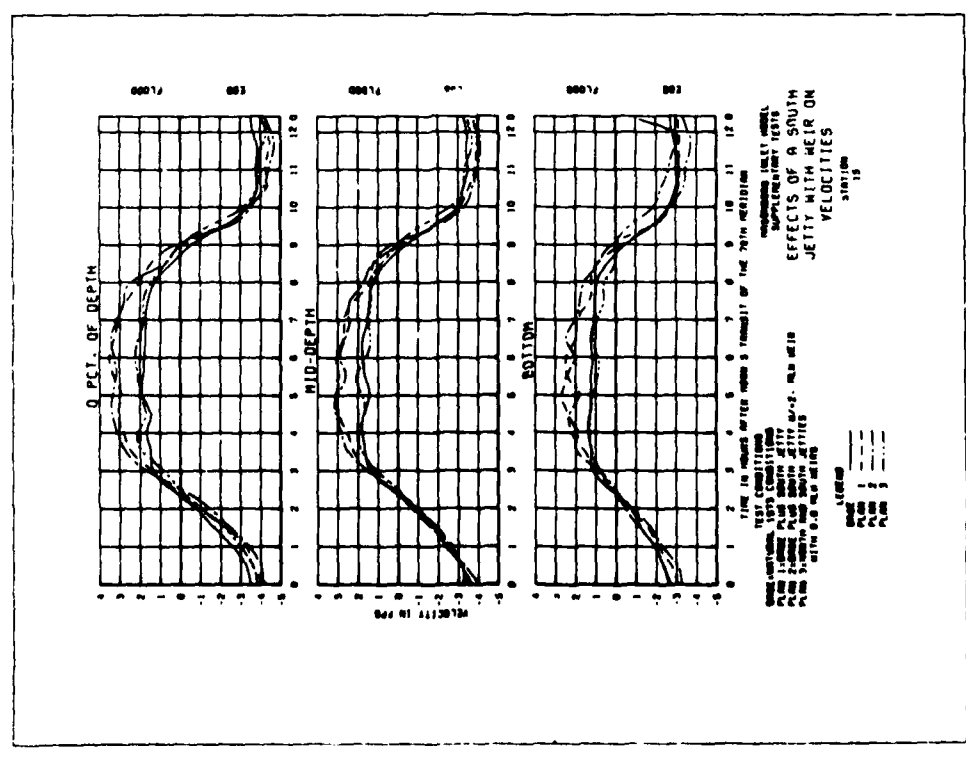
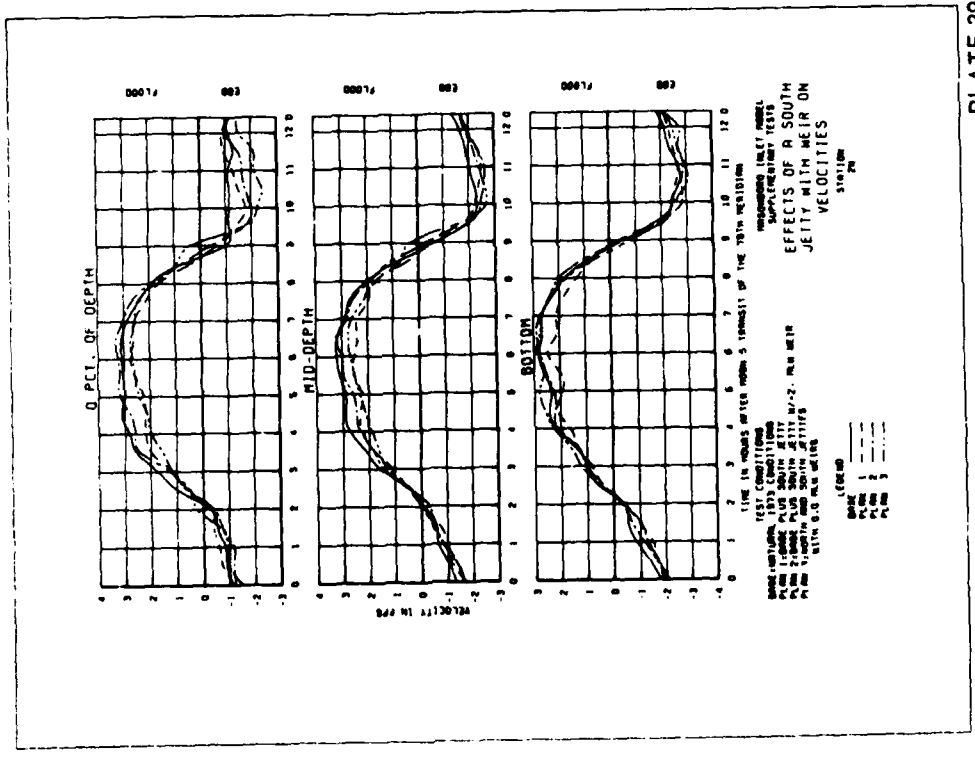


PLATE 39

PLATE 38

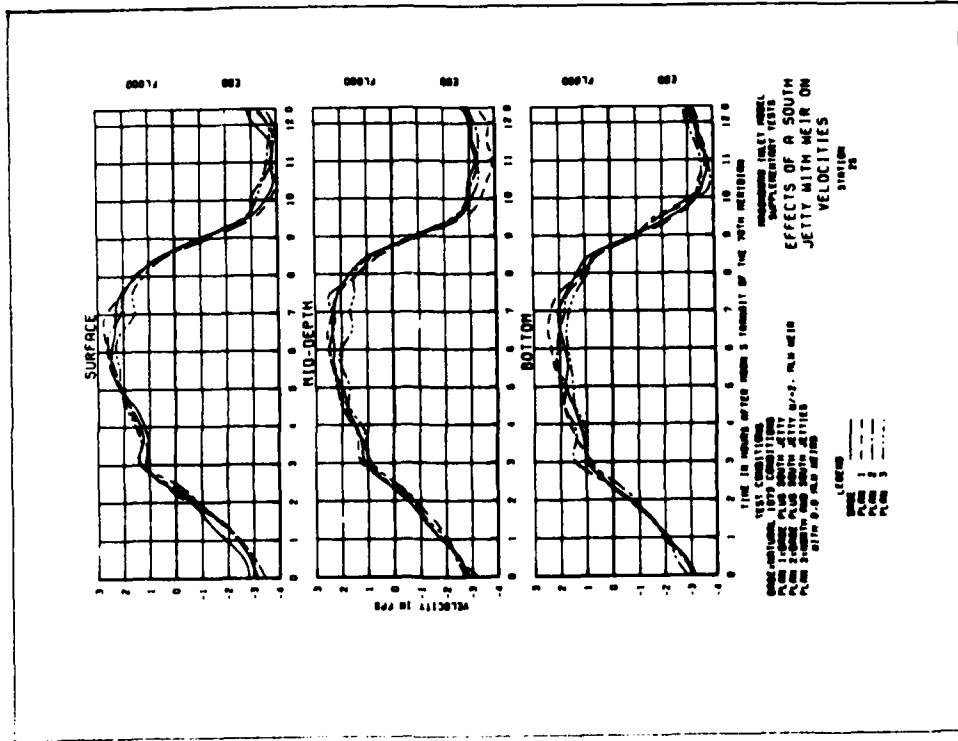


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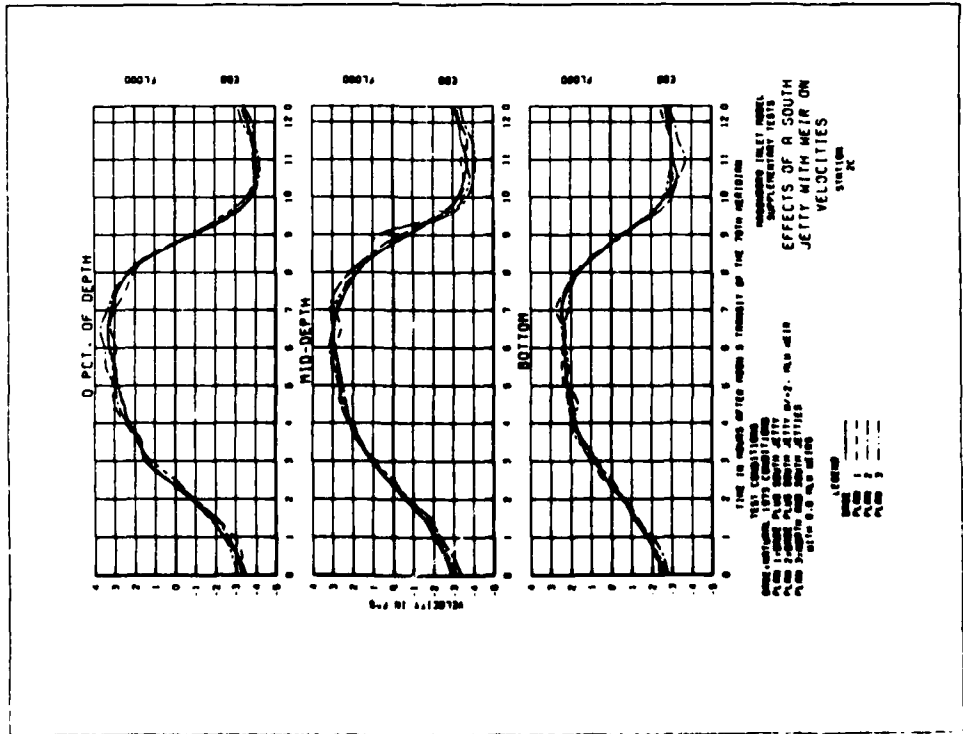


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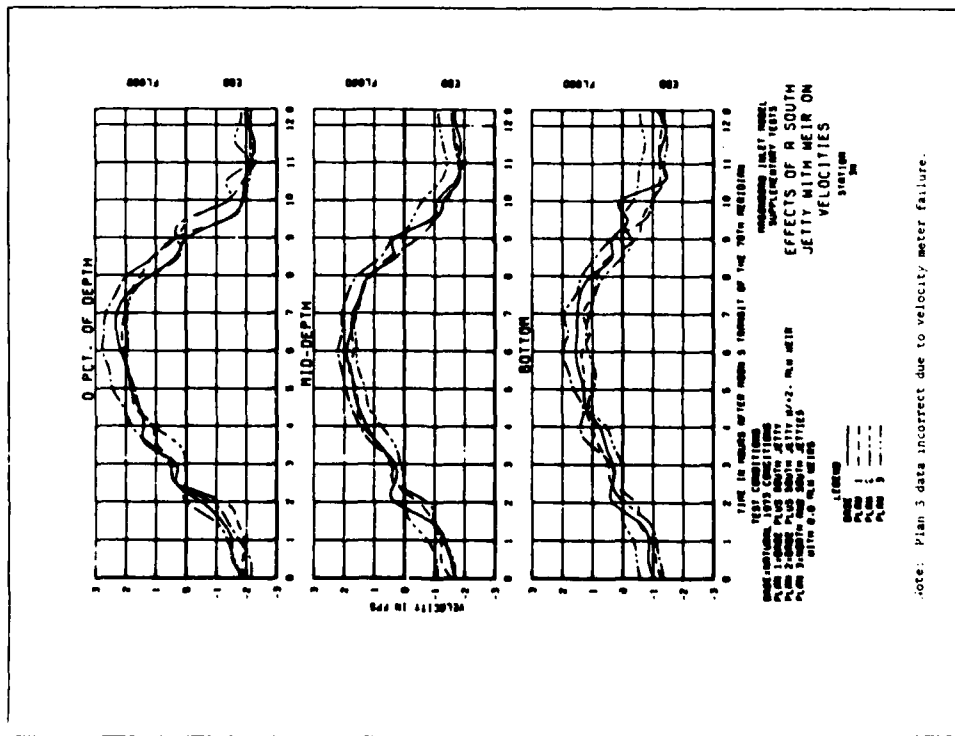


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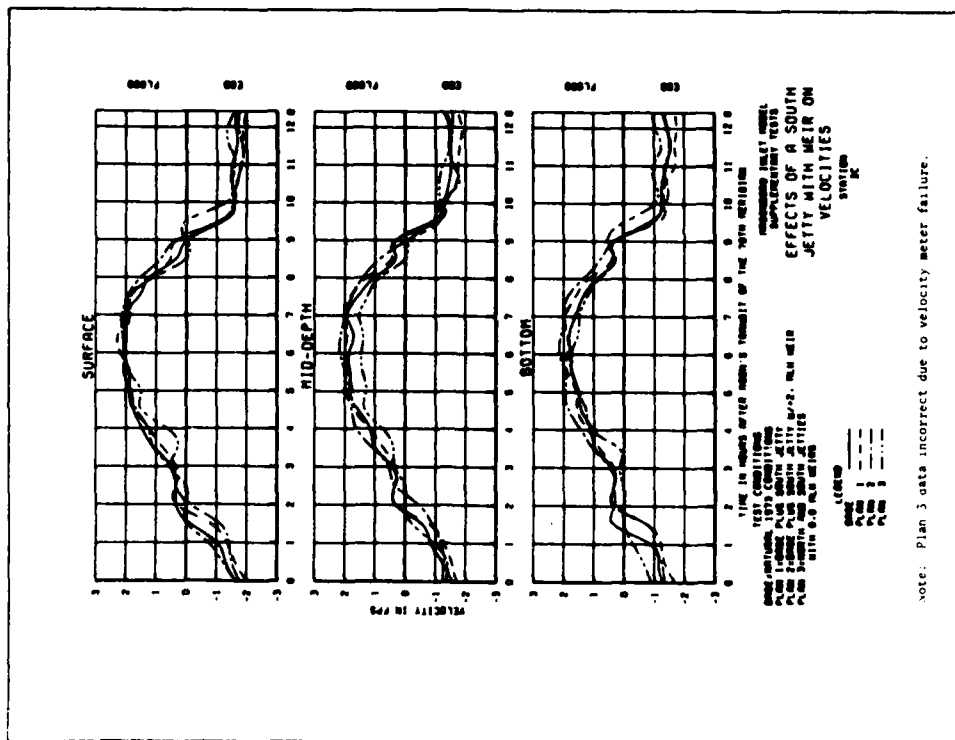


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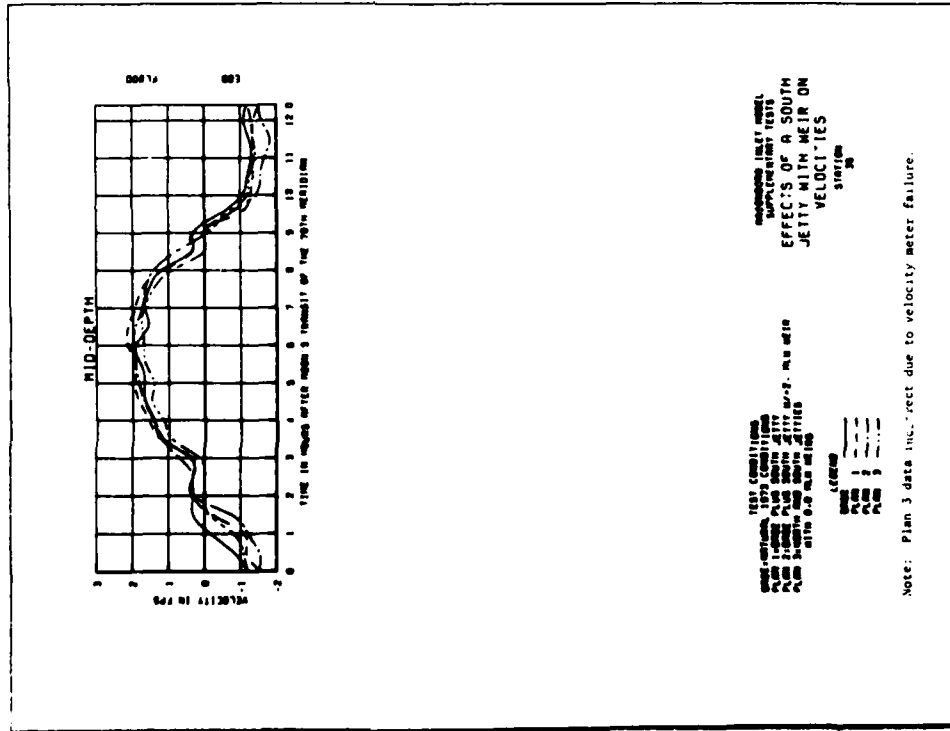


PLATE 44

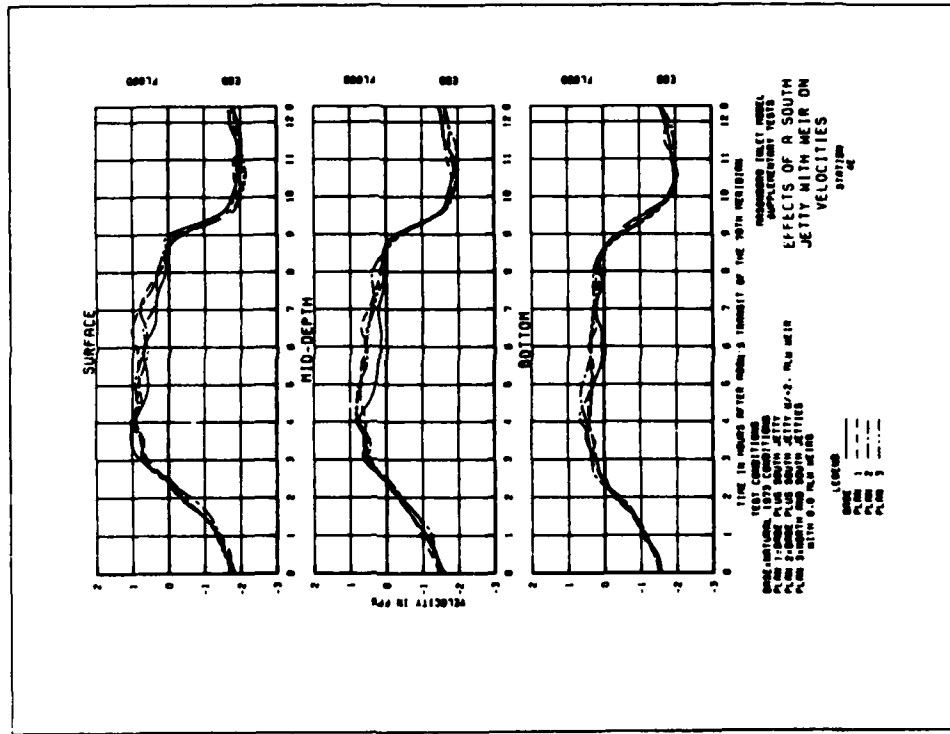


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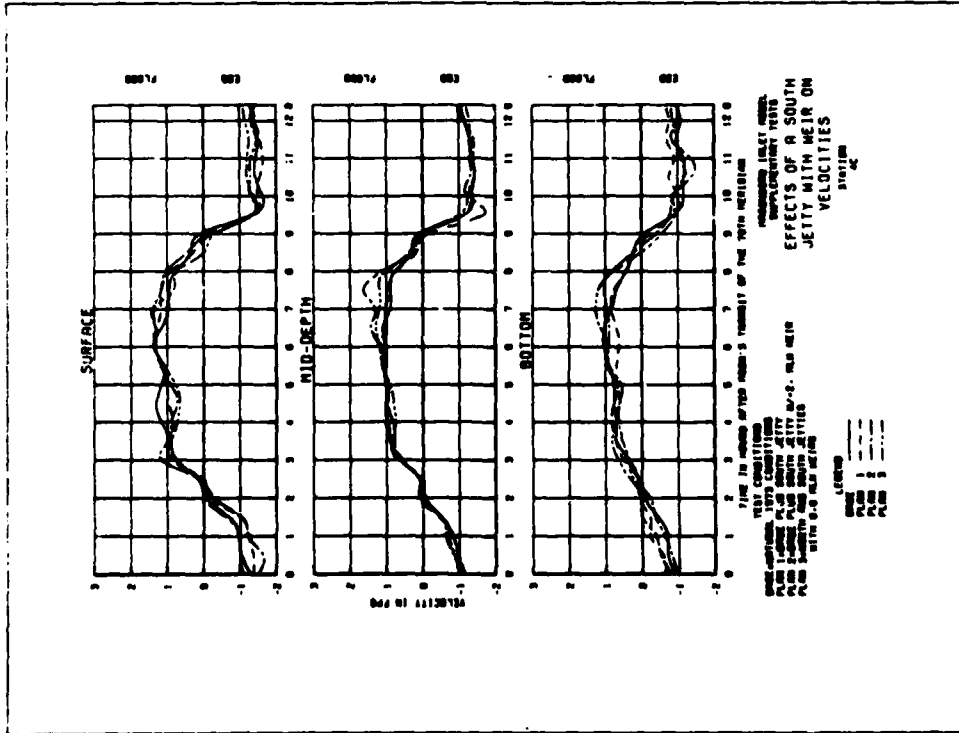


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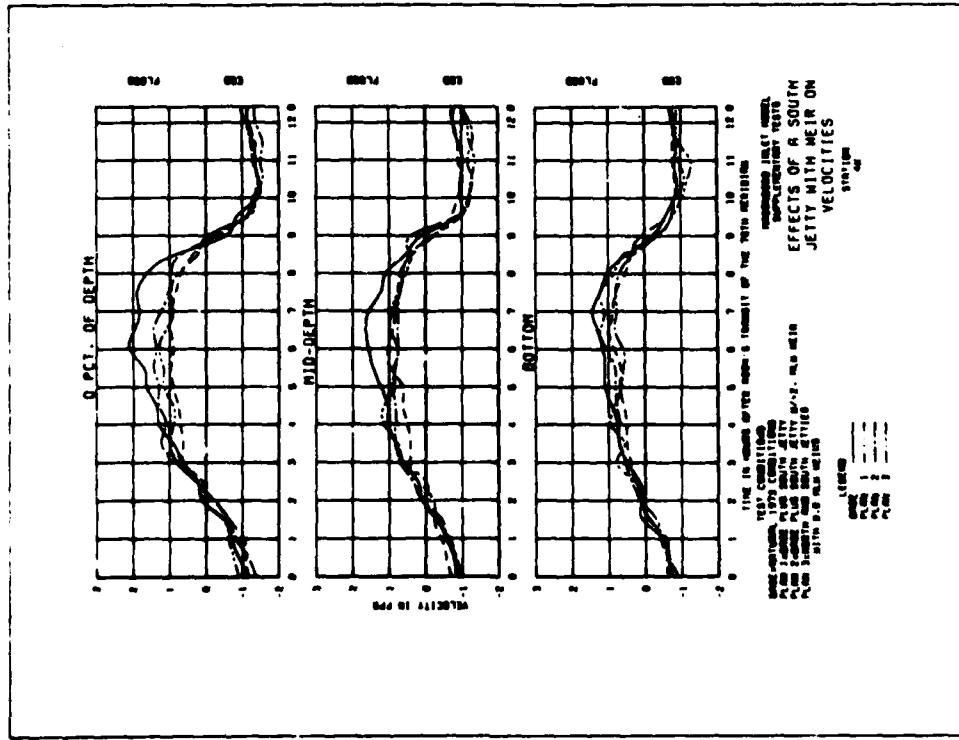


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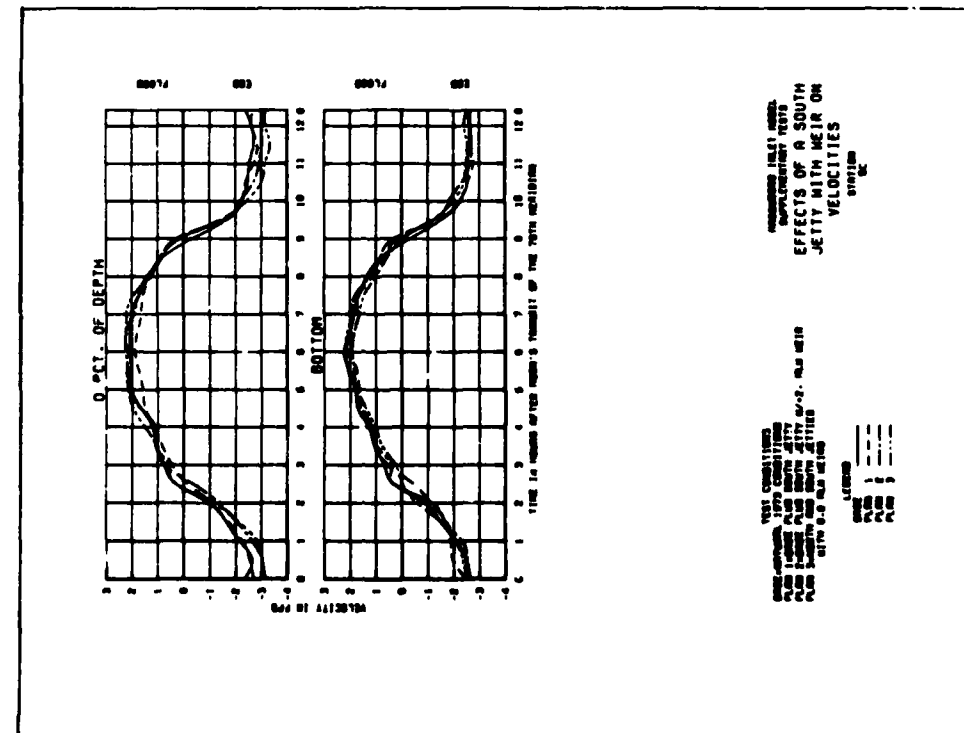


PLATE 48

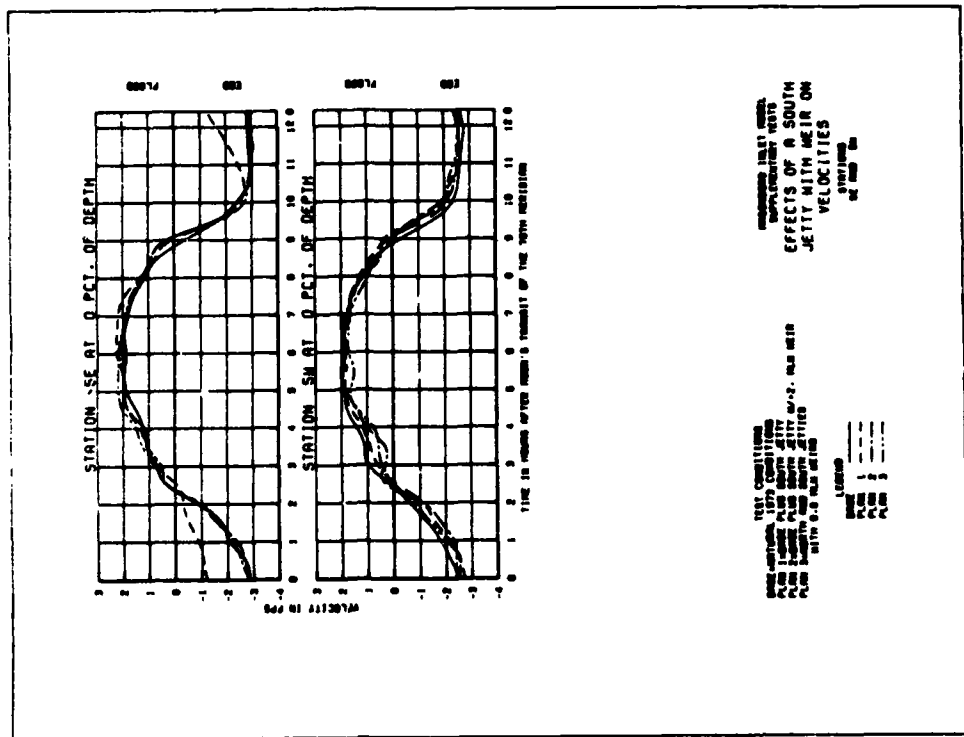
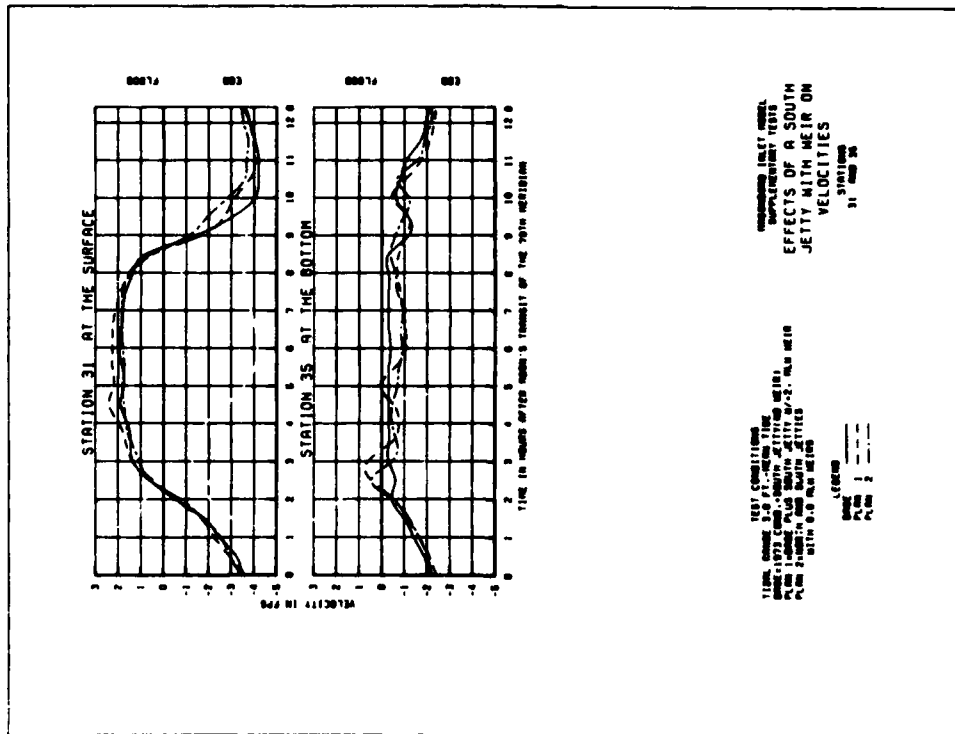
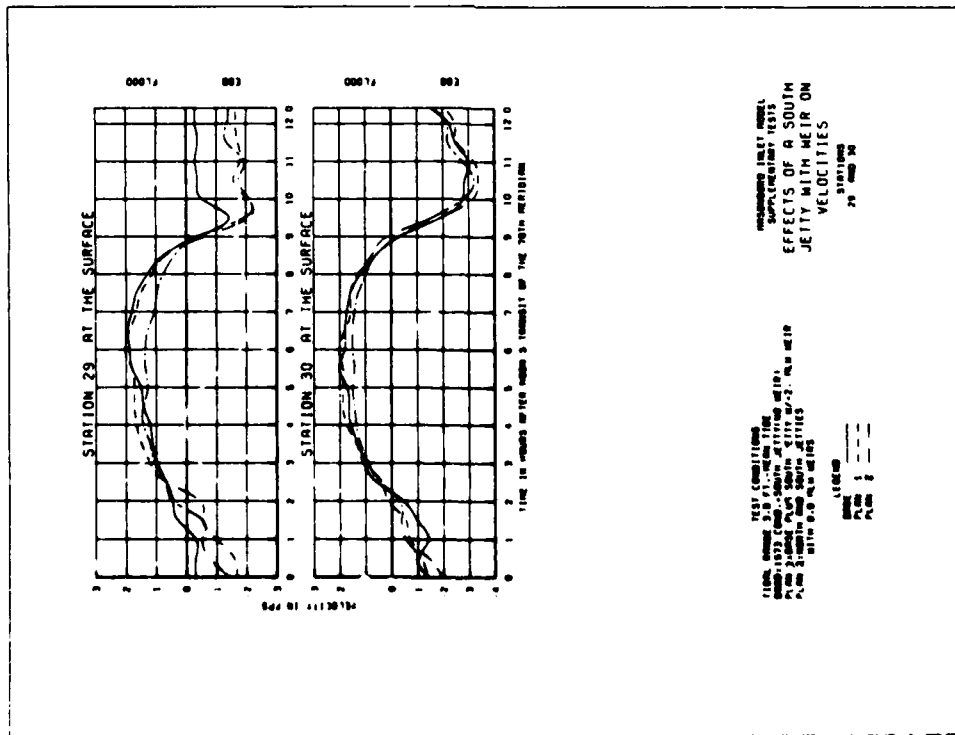


PLATE 49



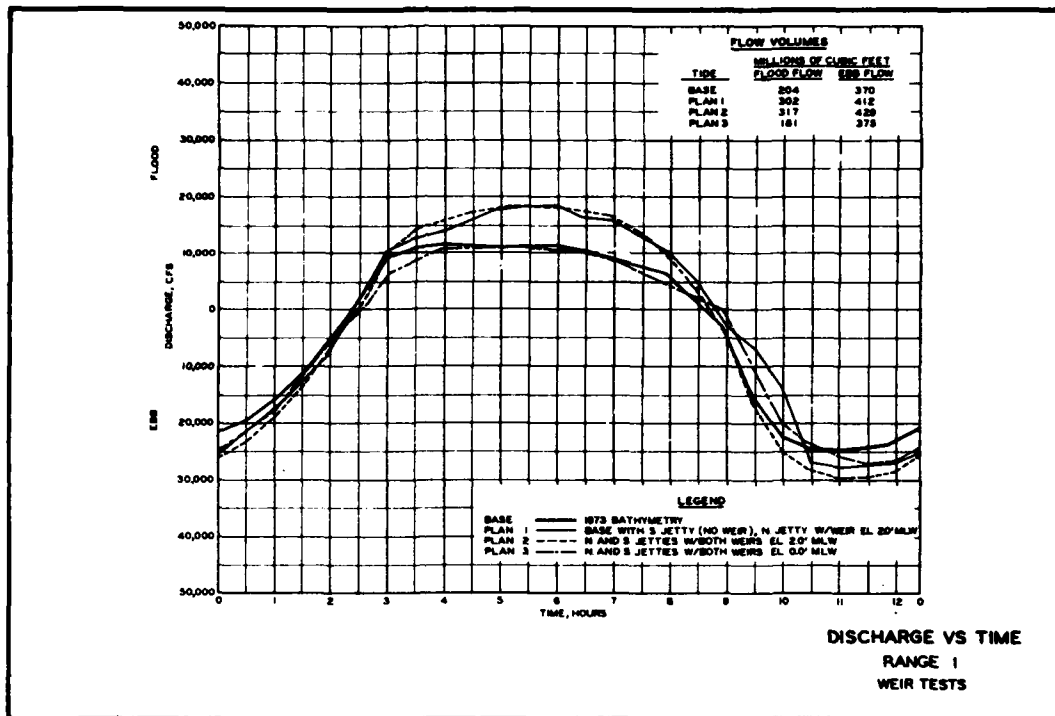


PLATE 52

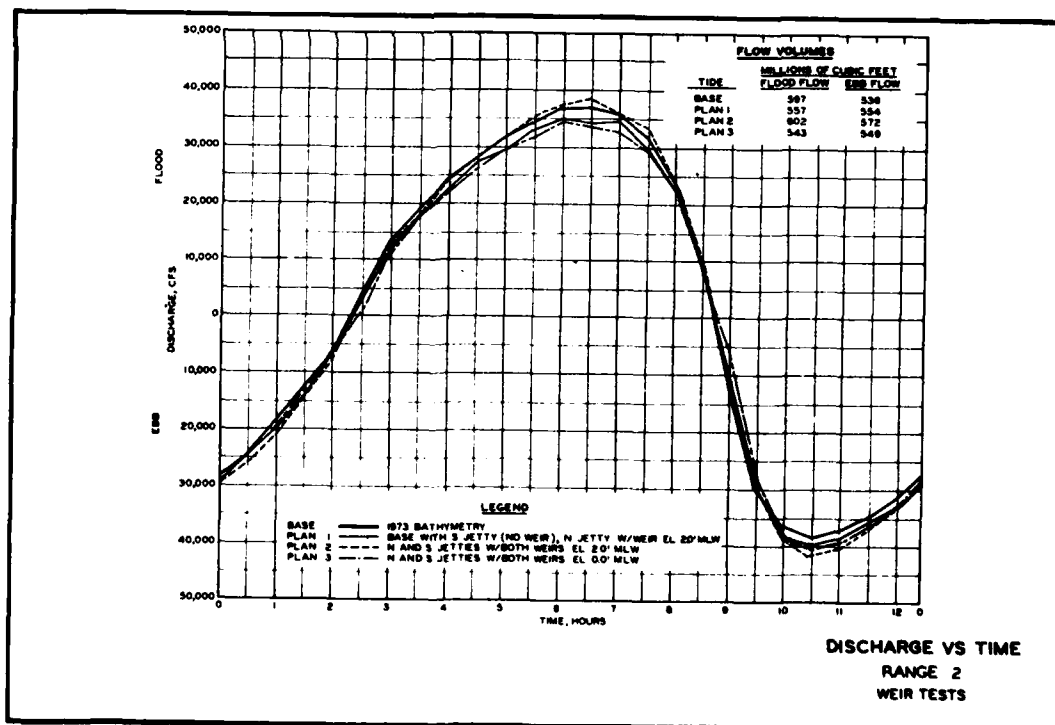


PLATE 53

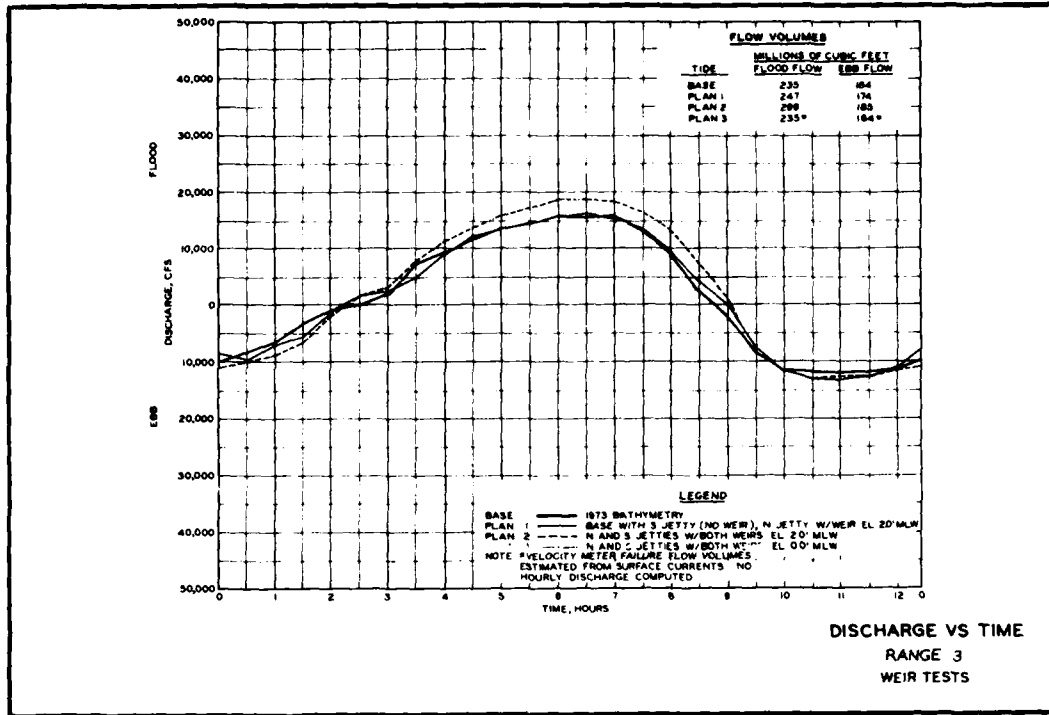


PLATE 54

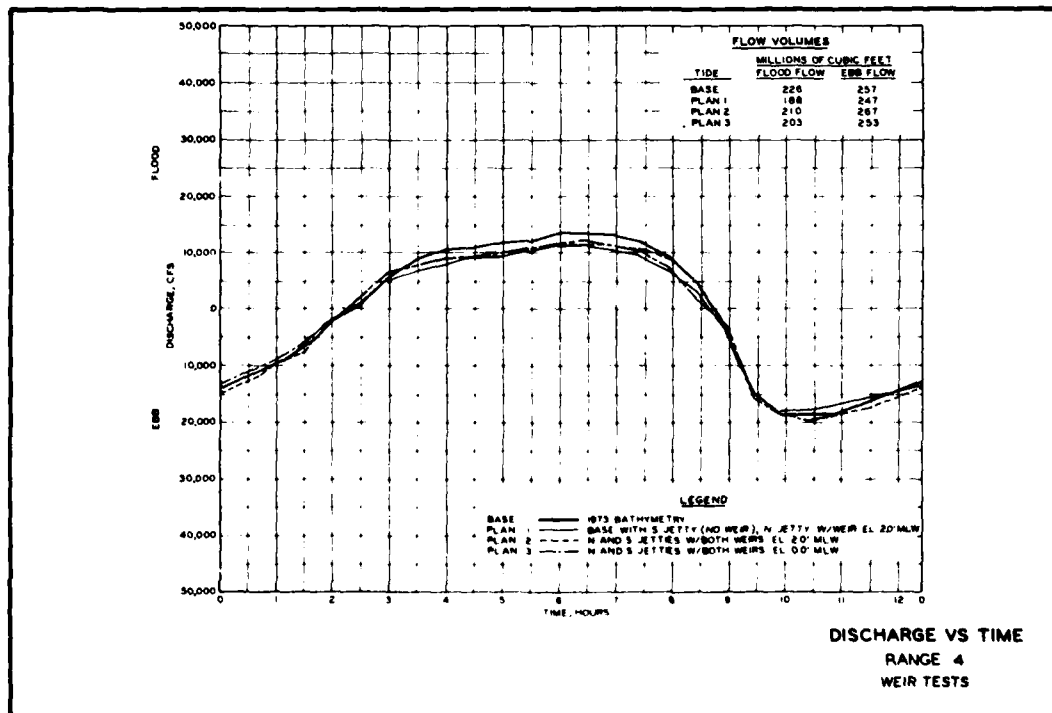
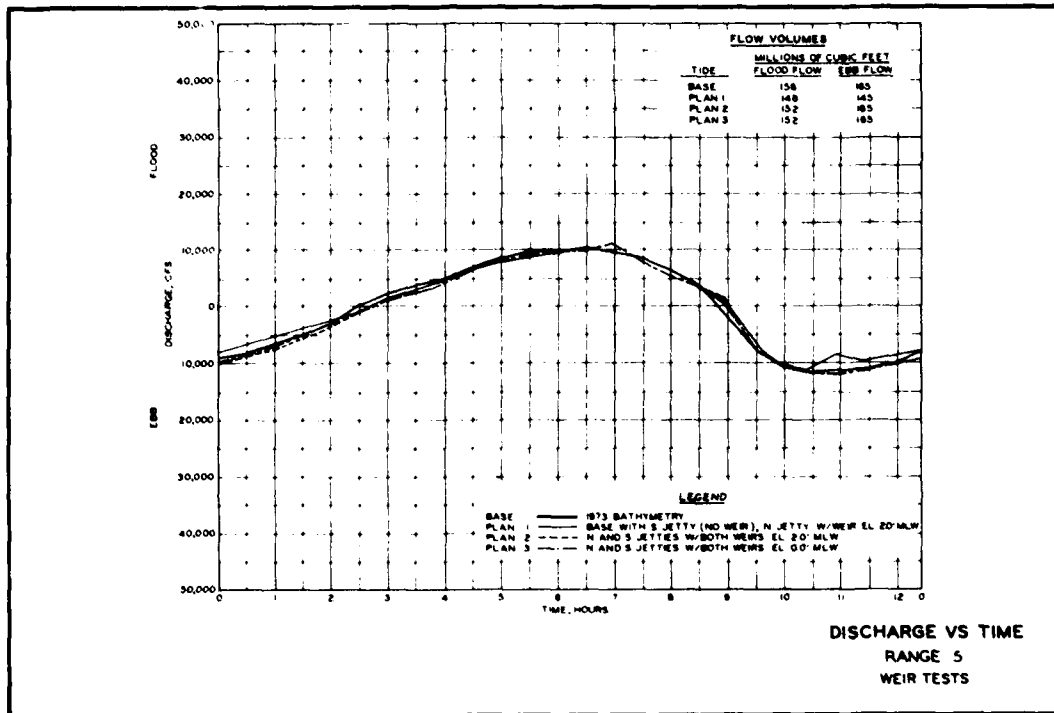


PLATE 55



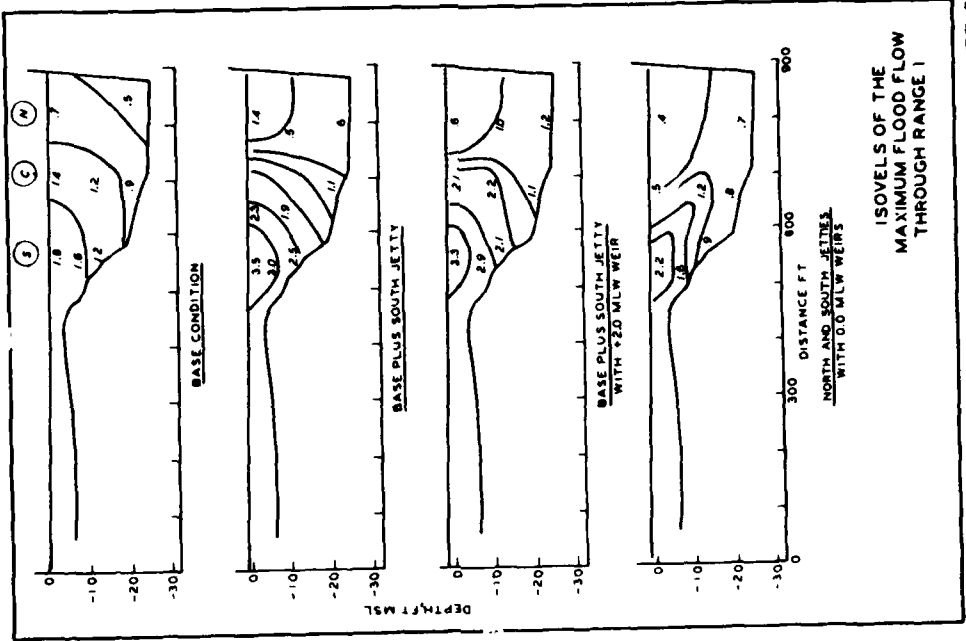


PLATE 58

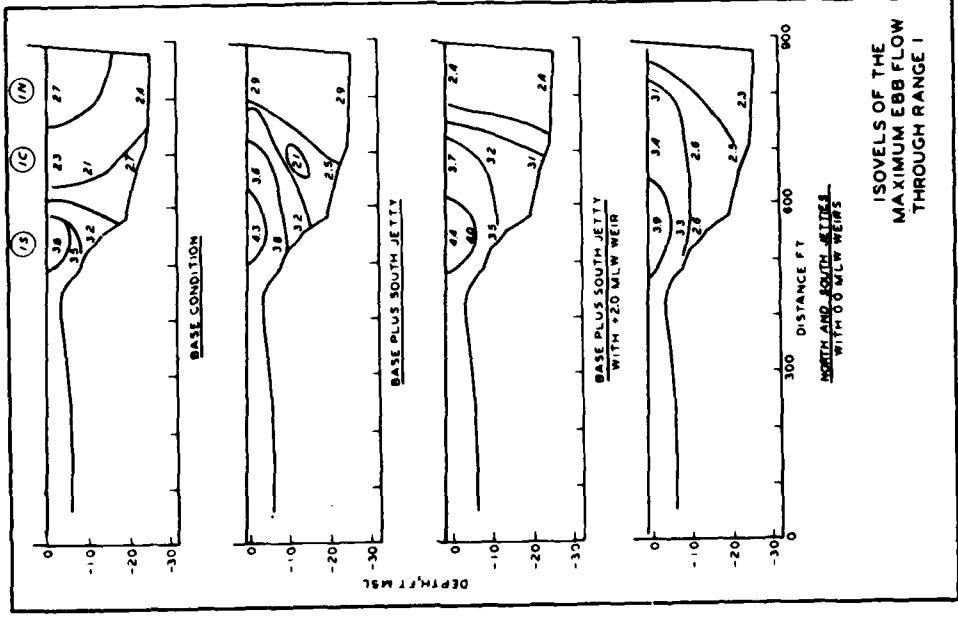


PLATE 57

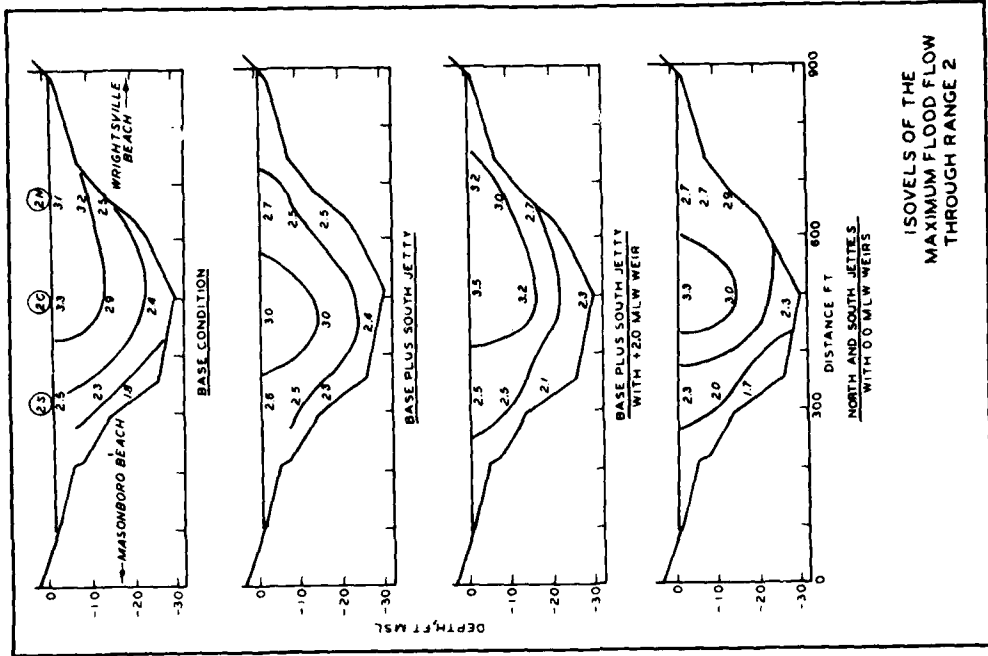


PLATE 59

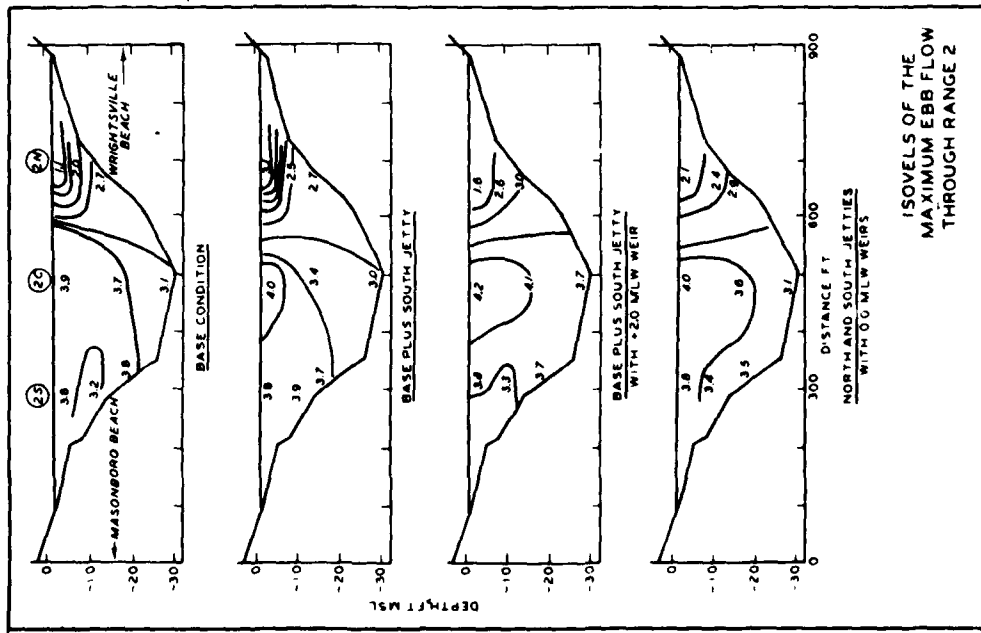


PLATE 60

AD-A088 761

ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS F/6 8/3  
SUPPLEMENTARY TESTS FOR MASONBORO INLET FIXED-BED MODEL. HYDRAU--ETC(U)  
MAY 80 W C SEABERGH, R A SAGER

UNCLASSIFIED

WES-GITI-18

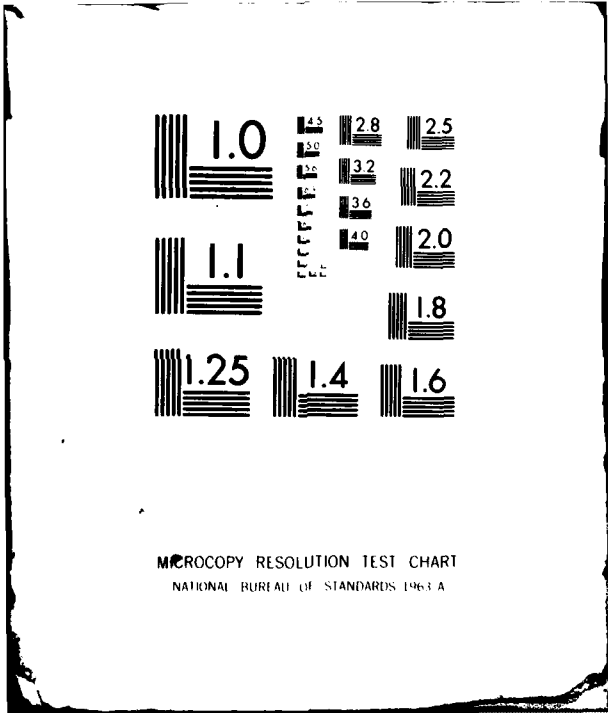
NL

3-3

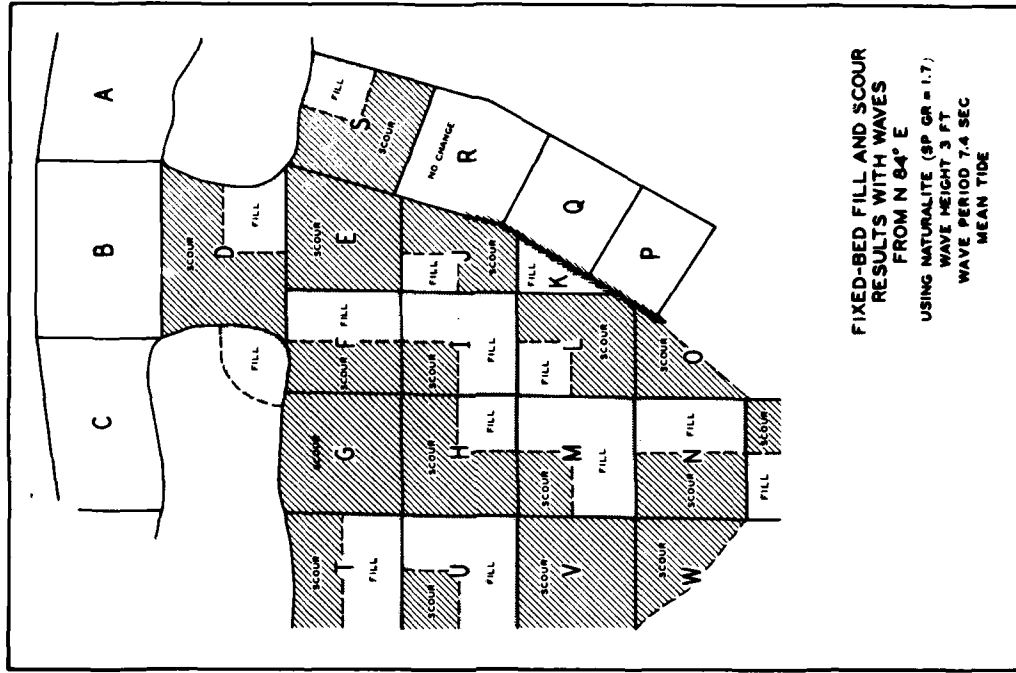
AD-A088 761

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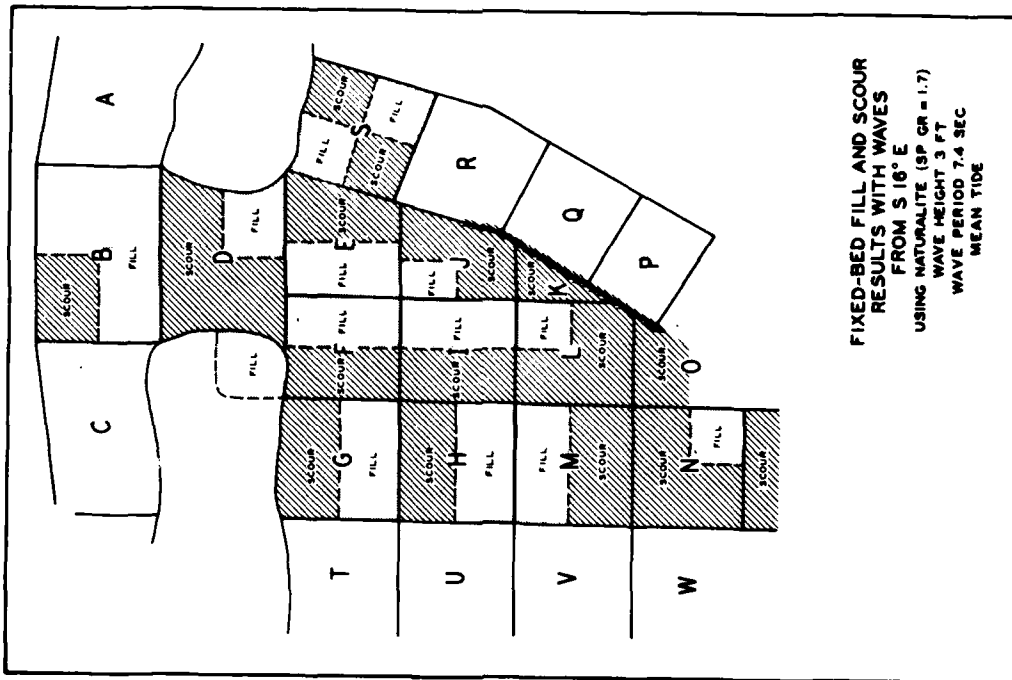


MICROCOPY RESOLUTION TEST CHART  
NATIONAL BUREAU OF STANDARDS 1963 A



FIXED-BED FILL AND SCOUR  
RESULTS WITH WAVES  
FROM N 84° E  
USING NATURALITE (SP GR = 1.7)  
WAVE HEIGHT 3 FT  
WAVE PERIOD 7.4 SEC  
MEAN TIDE

PLATE 62



FIXED-BED FILL AND SCOUR  
RESULTS WITH WAVES  
FROM S 16° E  
USING NATURALITE (SP GR = 1.7)  
WAVE HEIGHT 3 FT  
WAVE PERIOD 7.4 SEC  
MEAN TIDE

PLATE 61

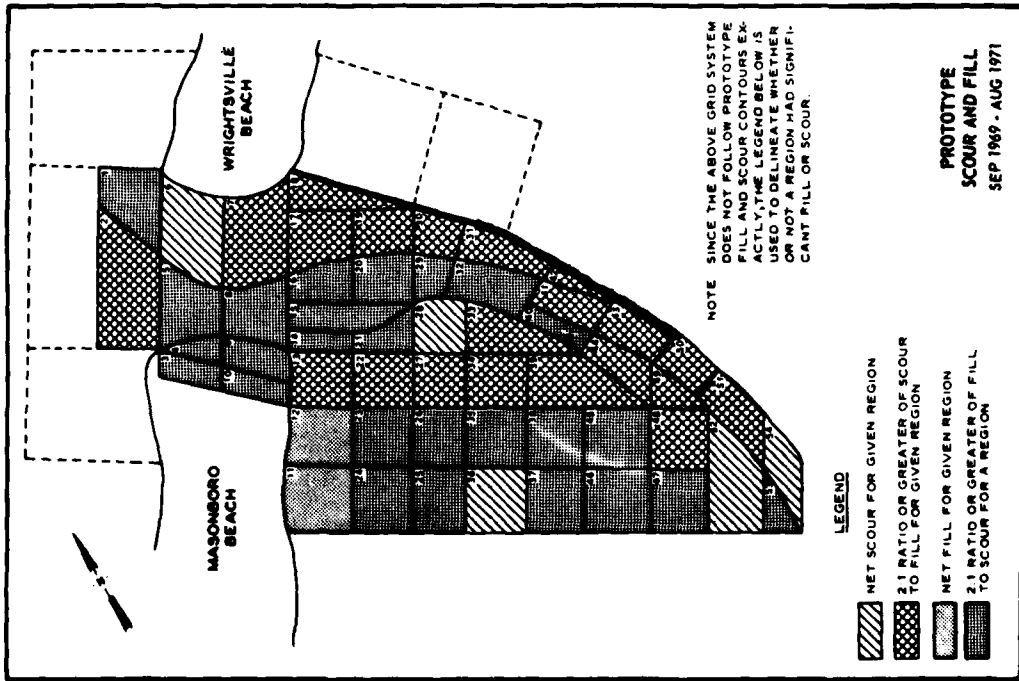


PLATE 63

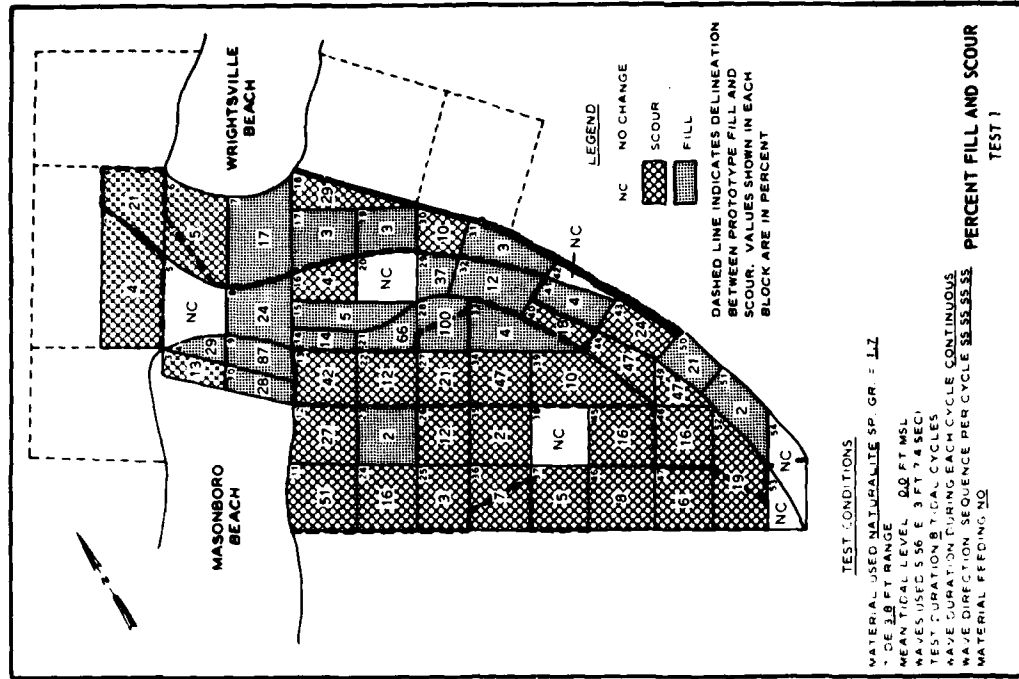


PLATE 64

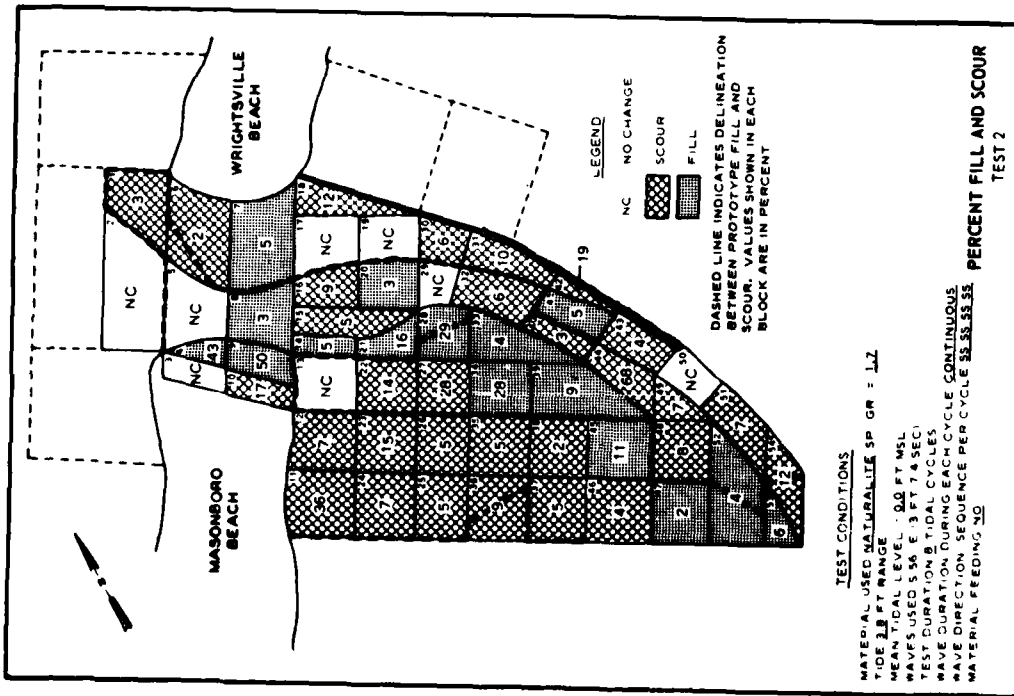


PLATE 65

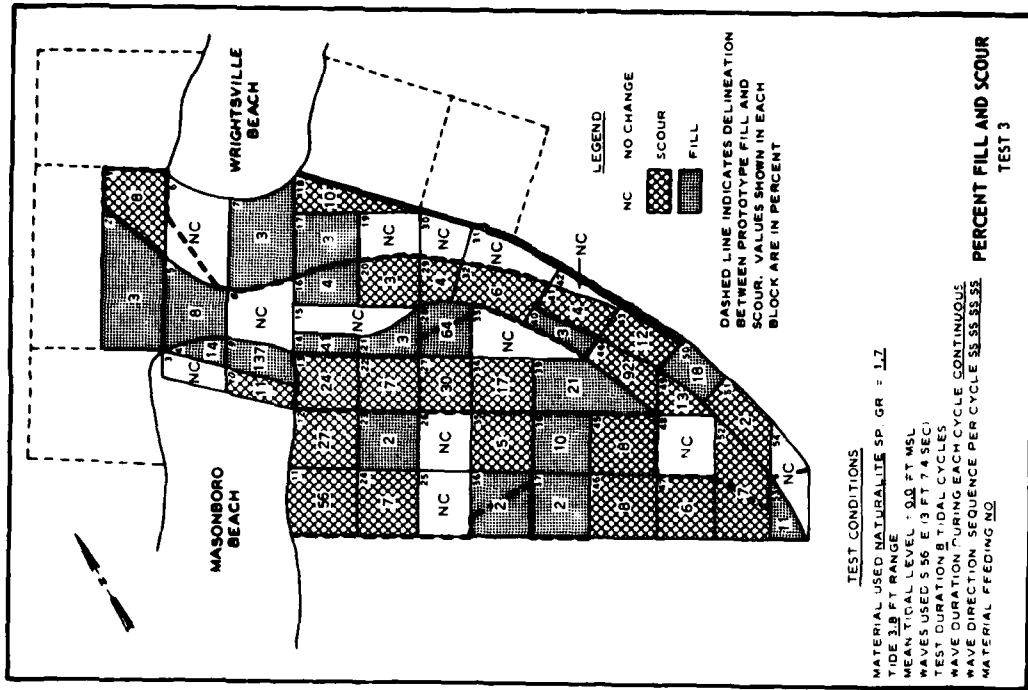


PLATE 66

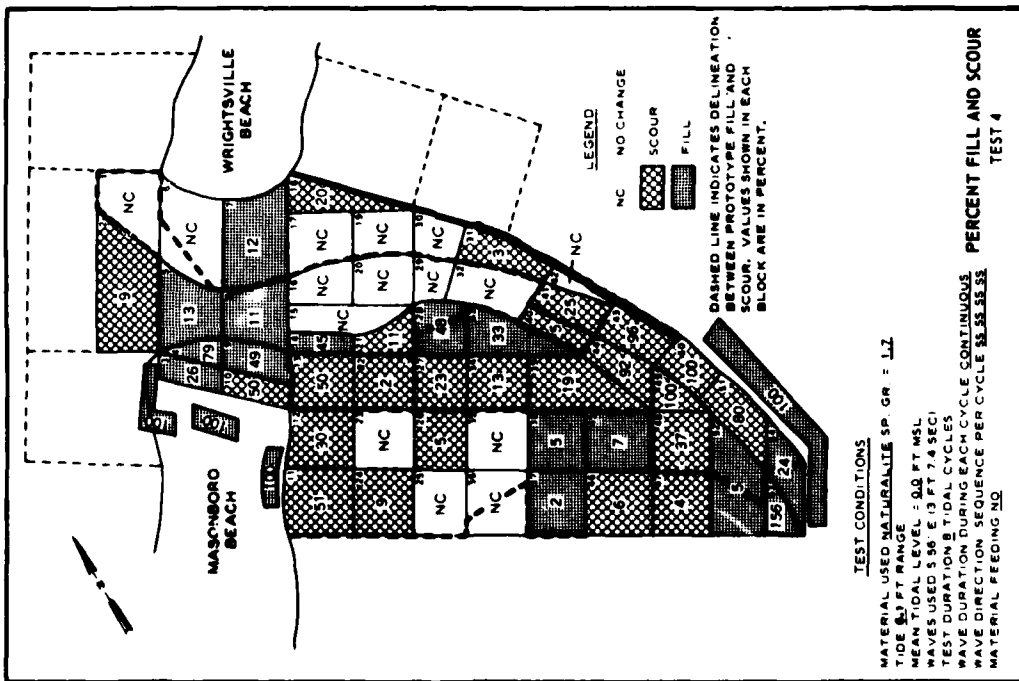


PLATE 67

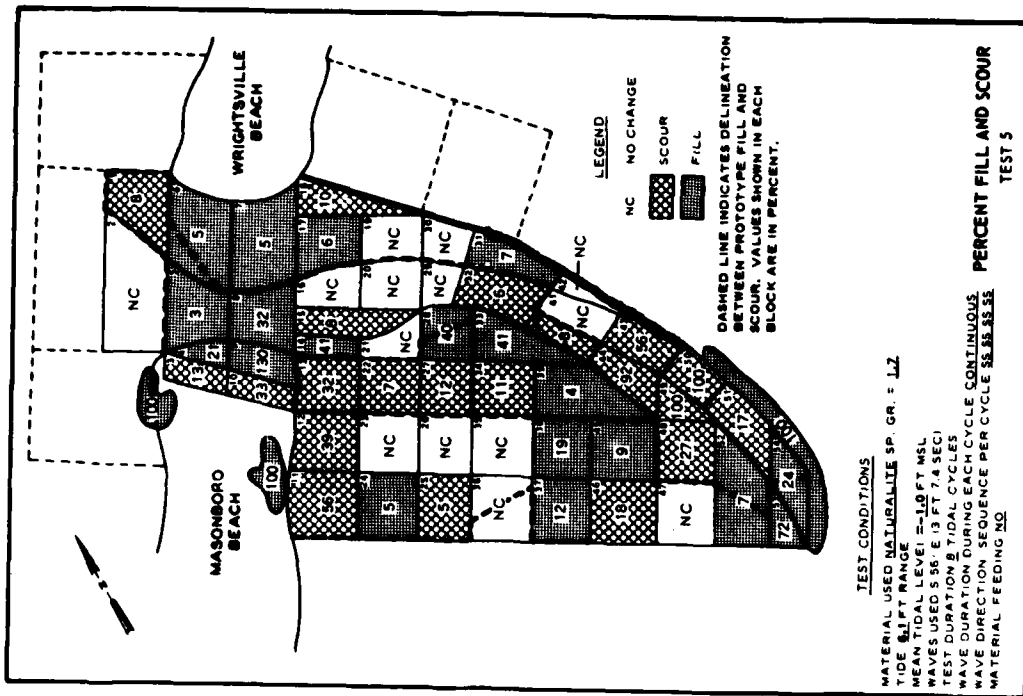


PLATE 68

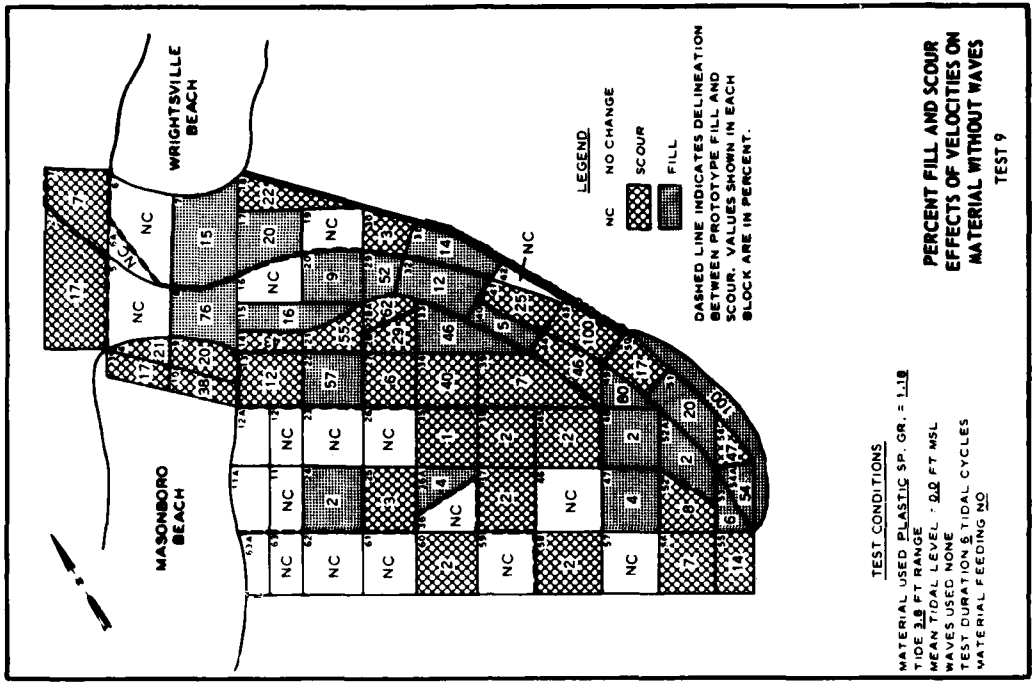


PLATE 70

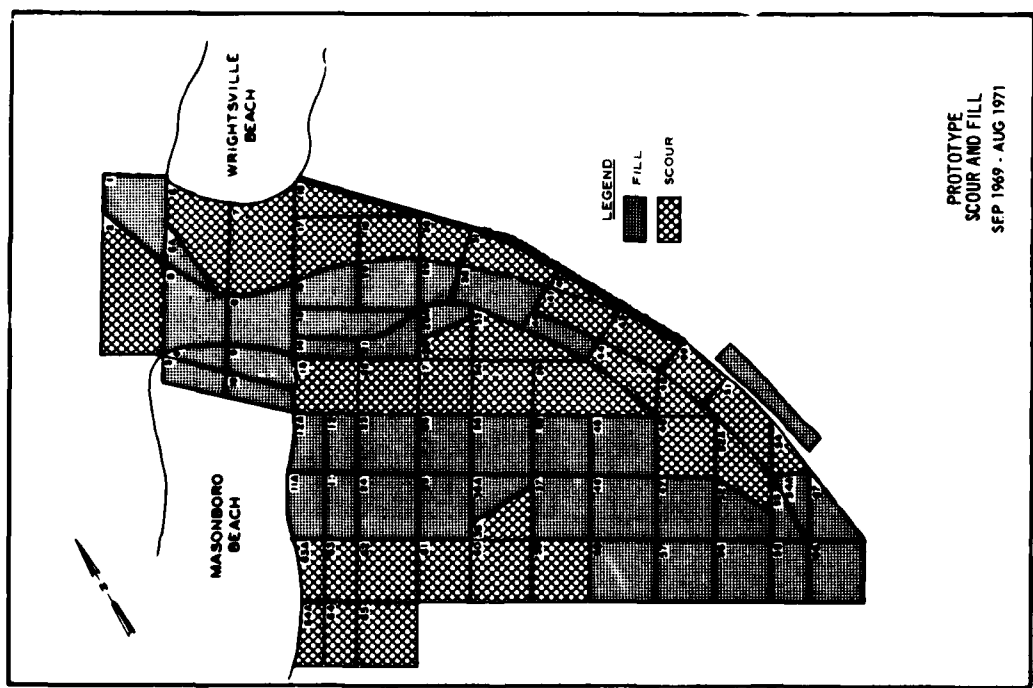


PLATE 69

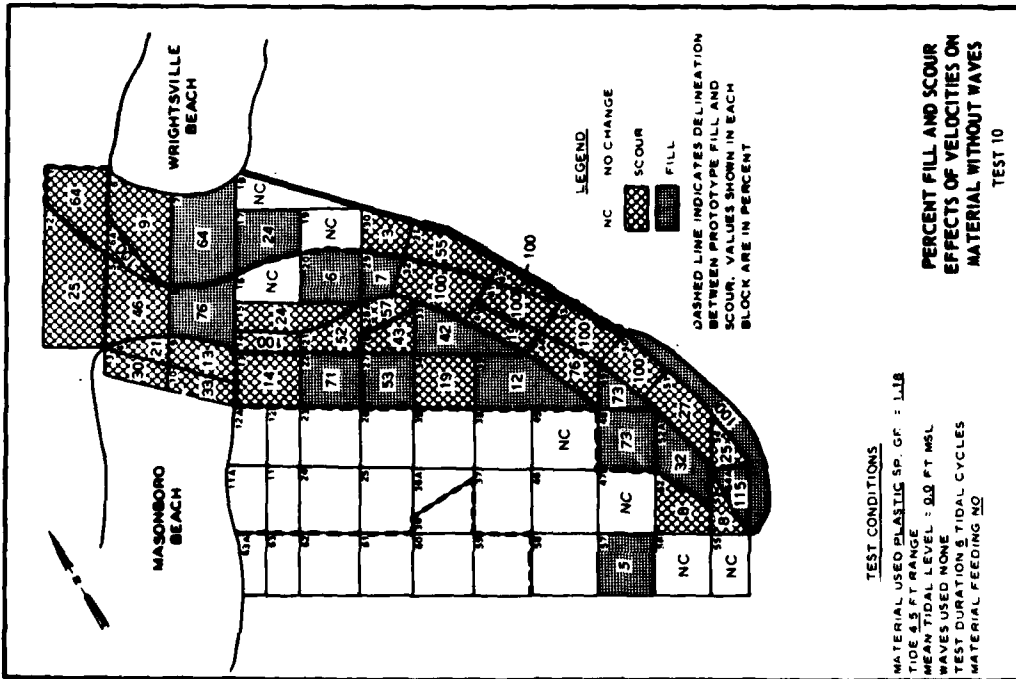


PLATE 71

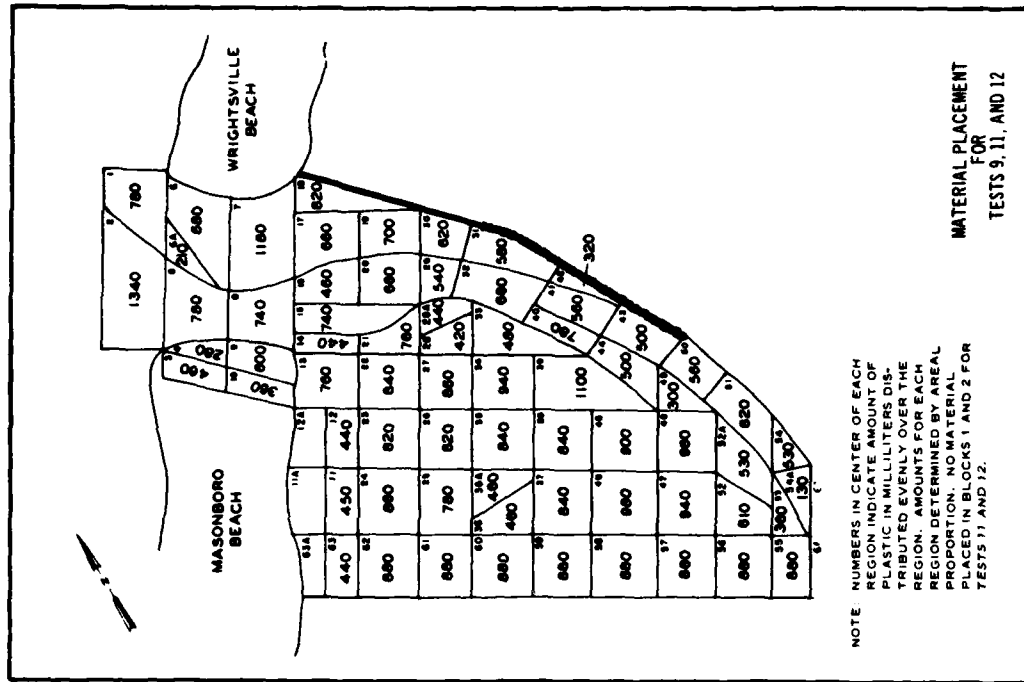


PLATE 72

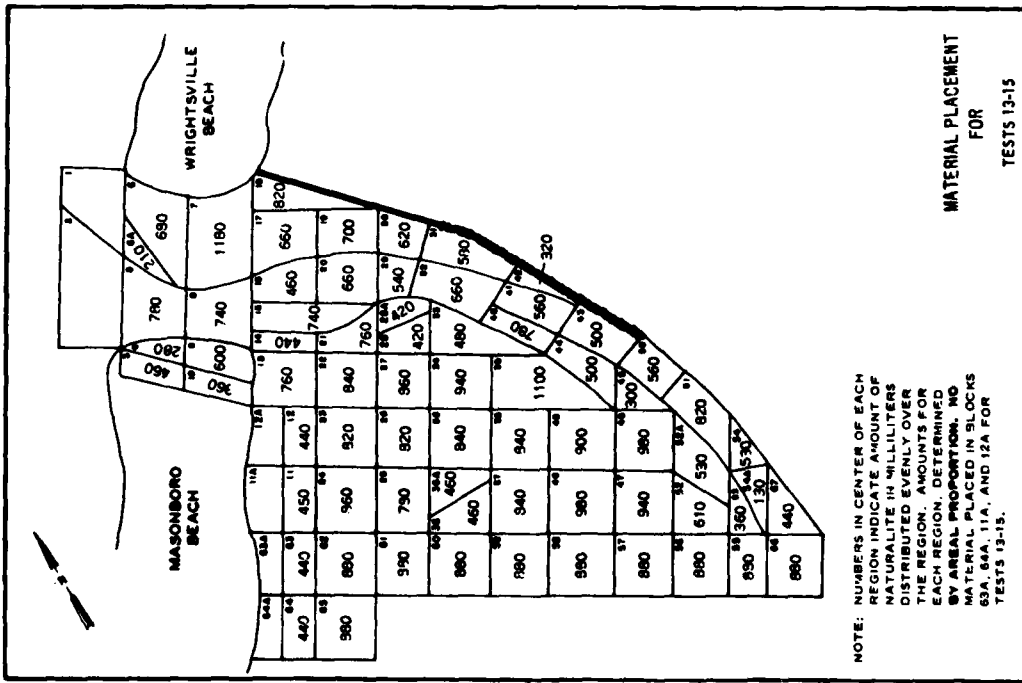


PLATE 73

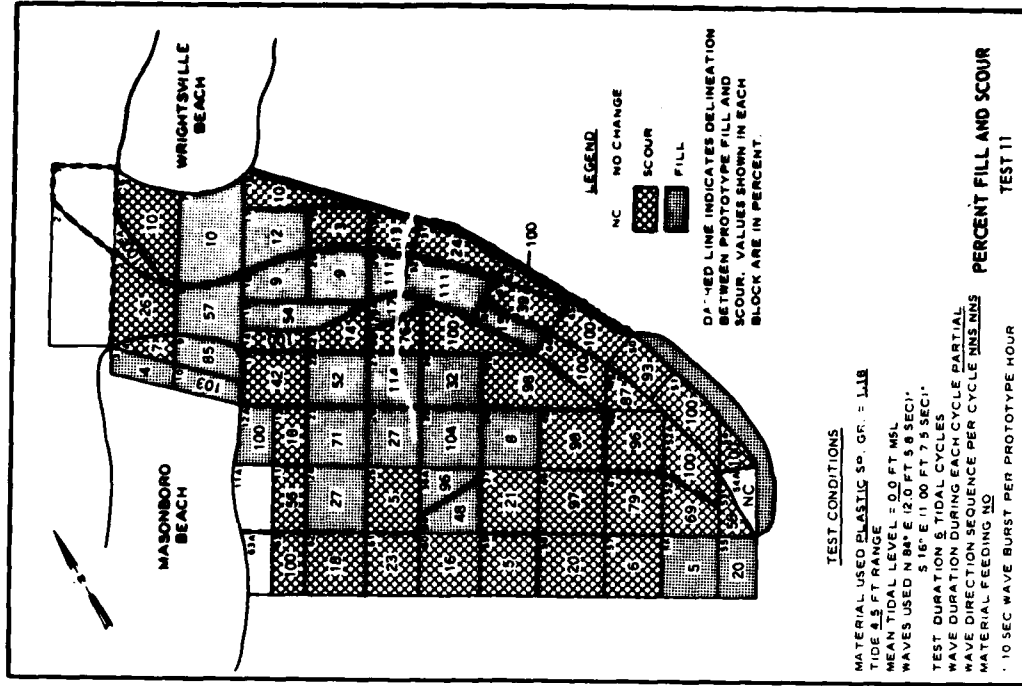


PLATE 74



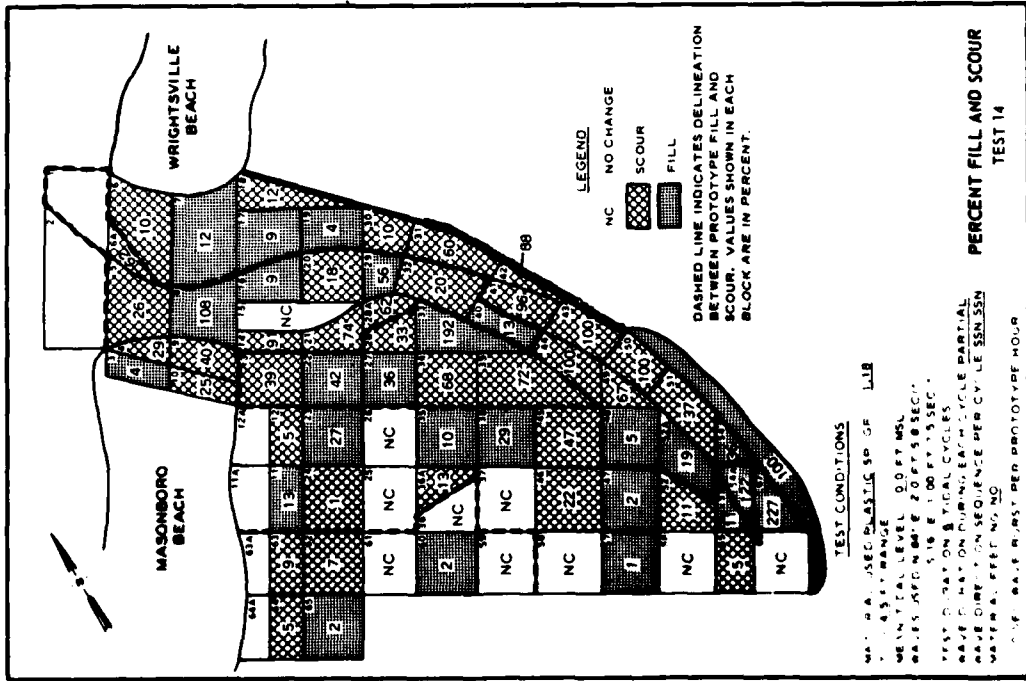


PLATE 77

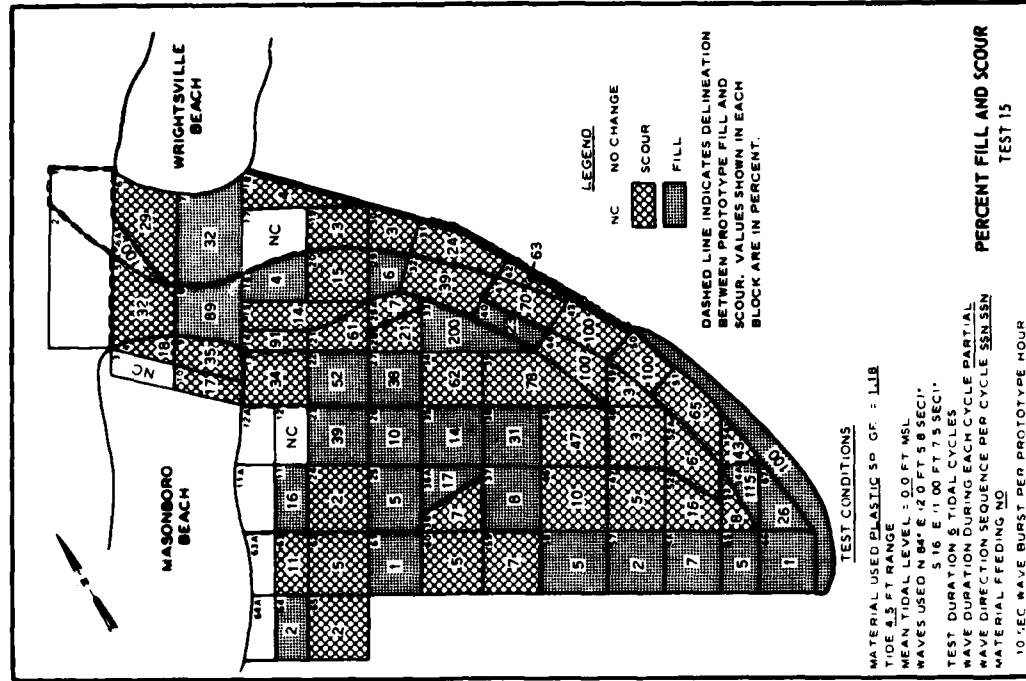


PLATE 78

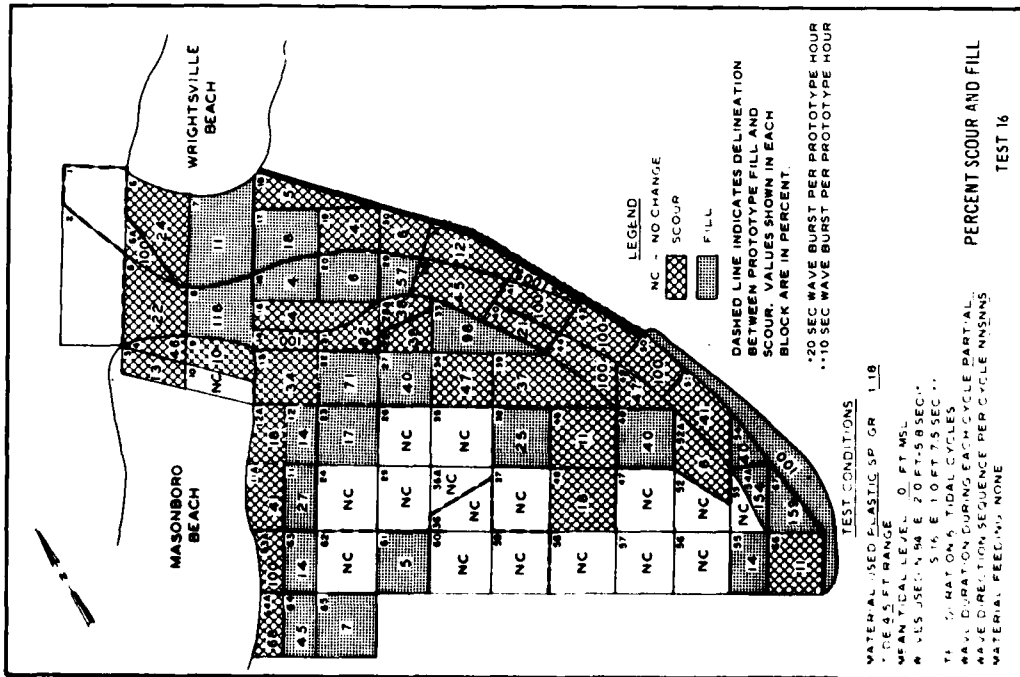


PLATE 79

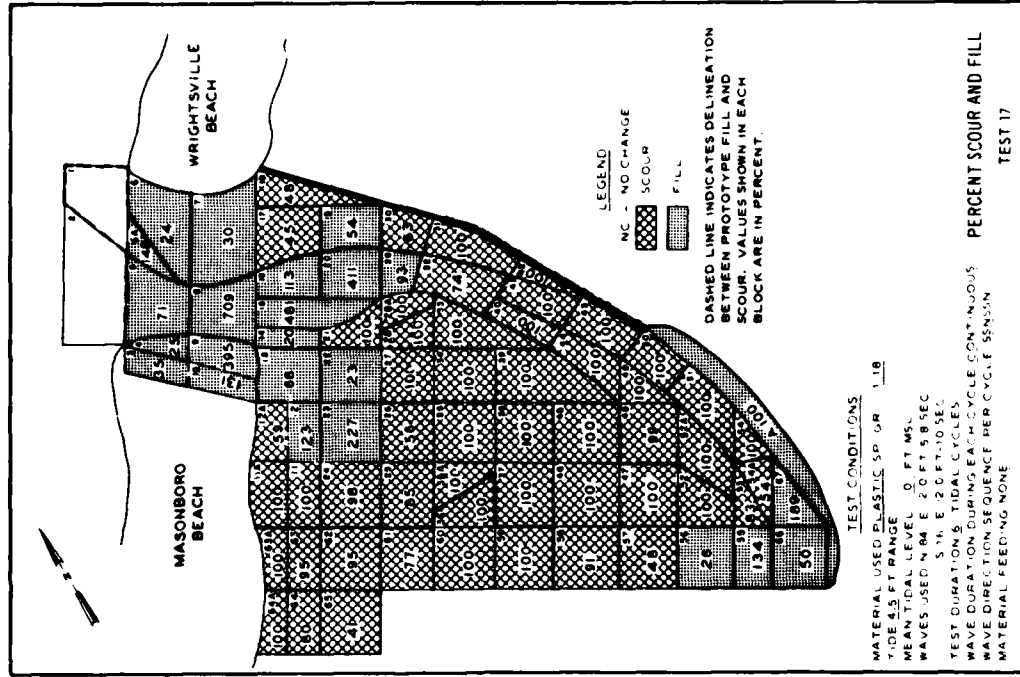


PLATE 80

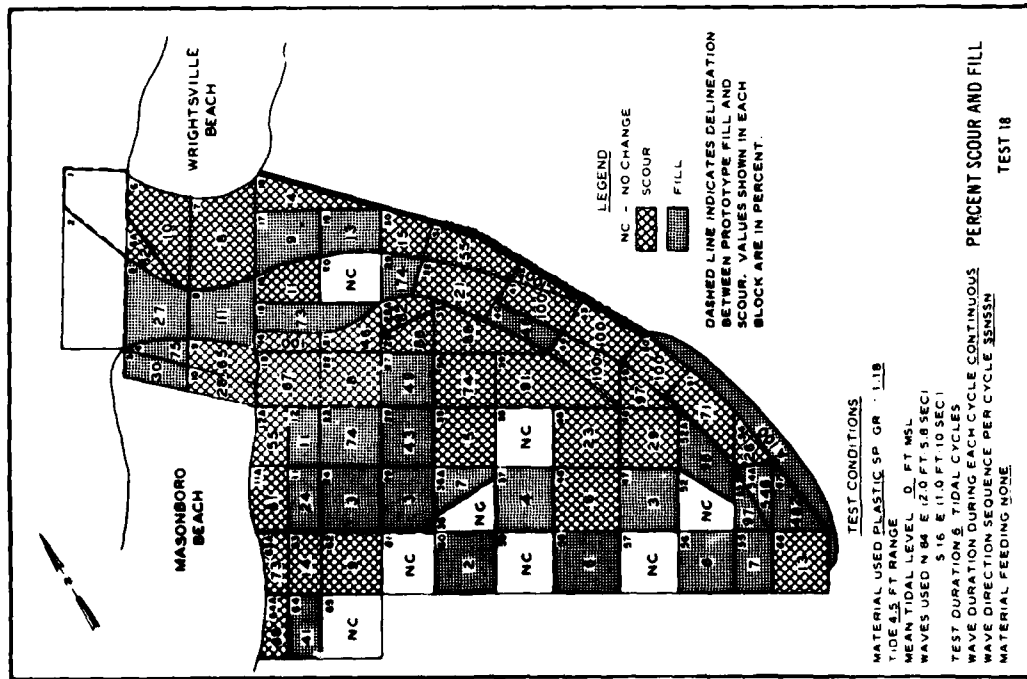


PLATE 81

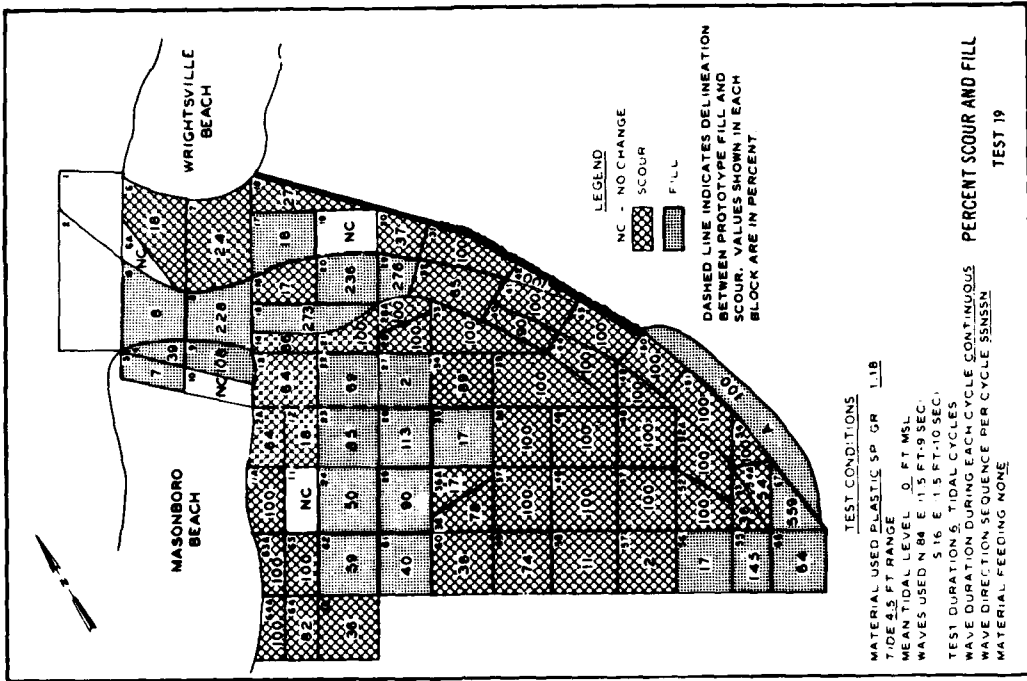


PLATE 82

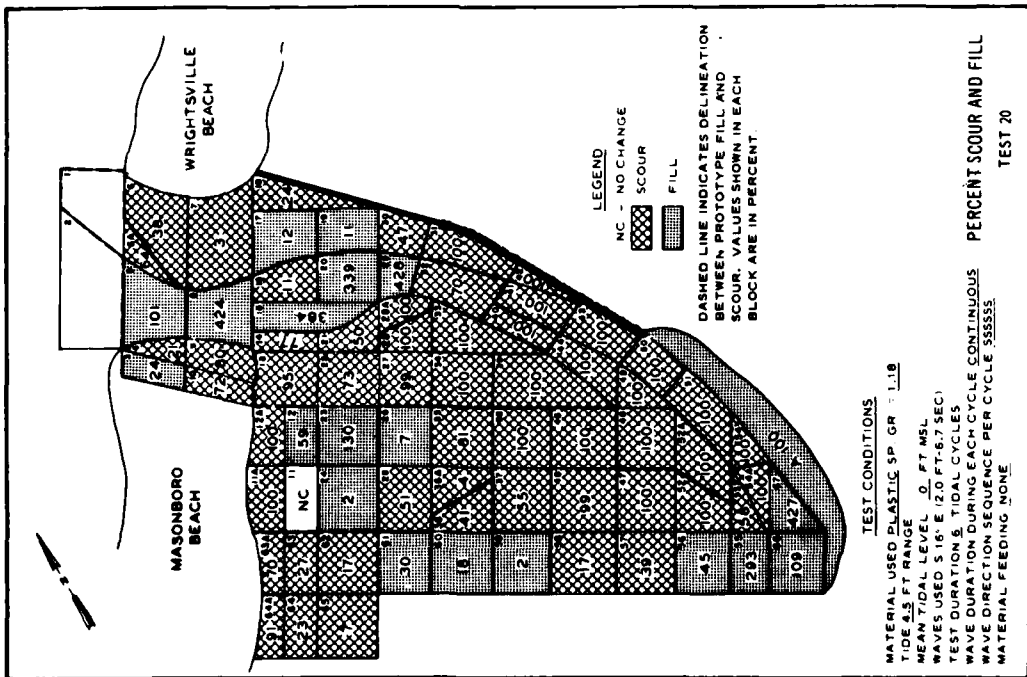


PLATE 83

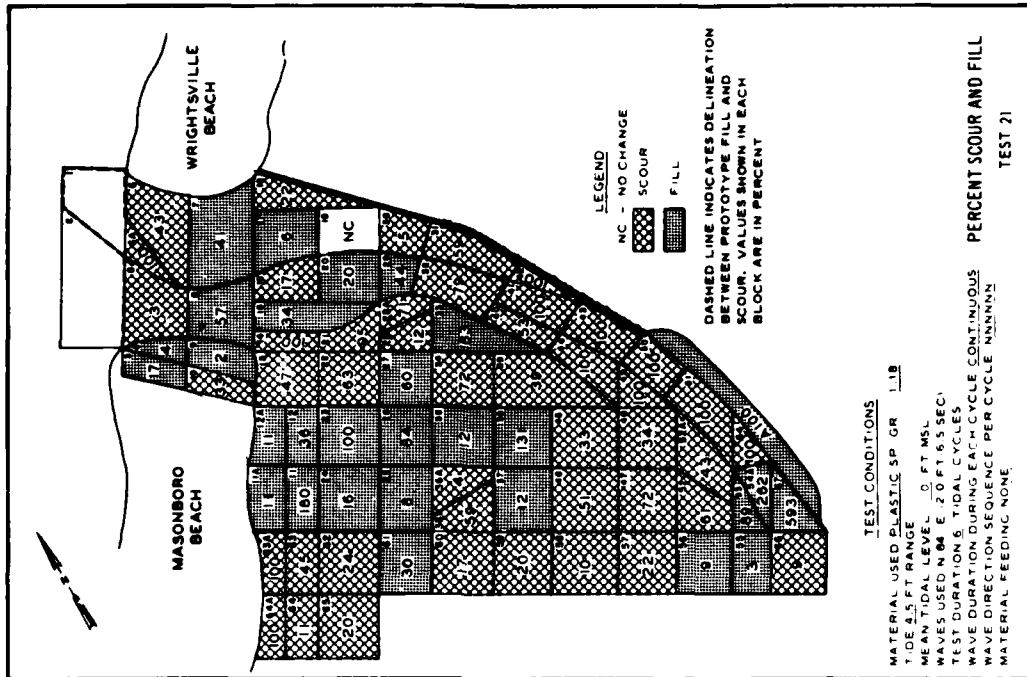


PLATE 84

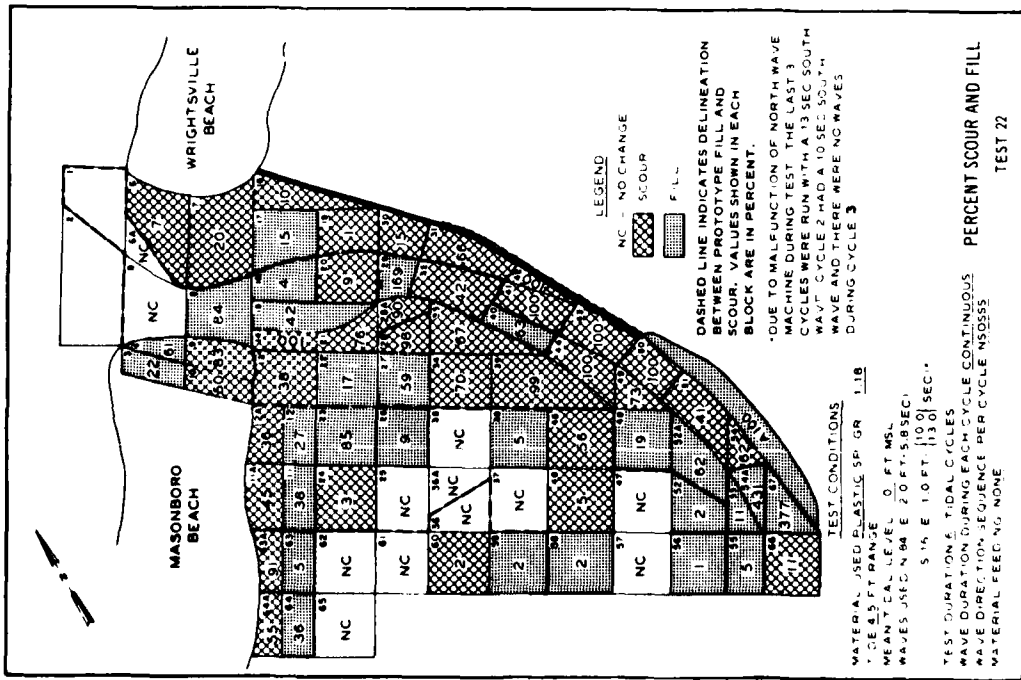


PLATE 85

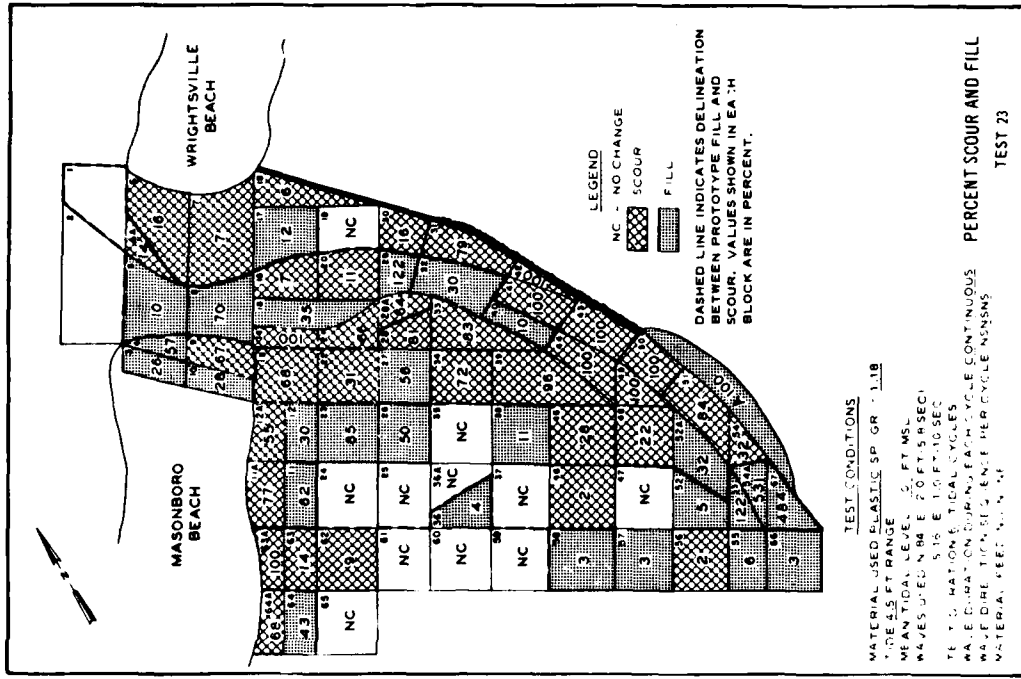


PLATE 86

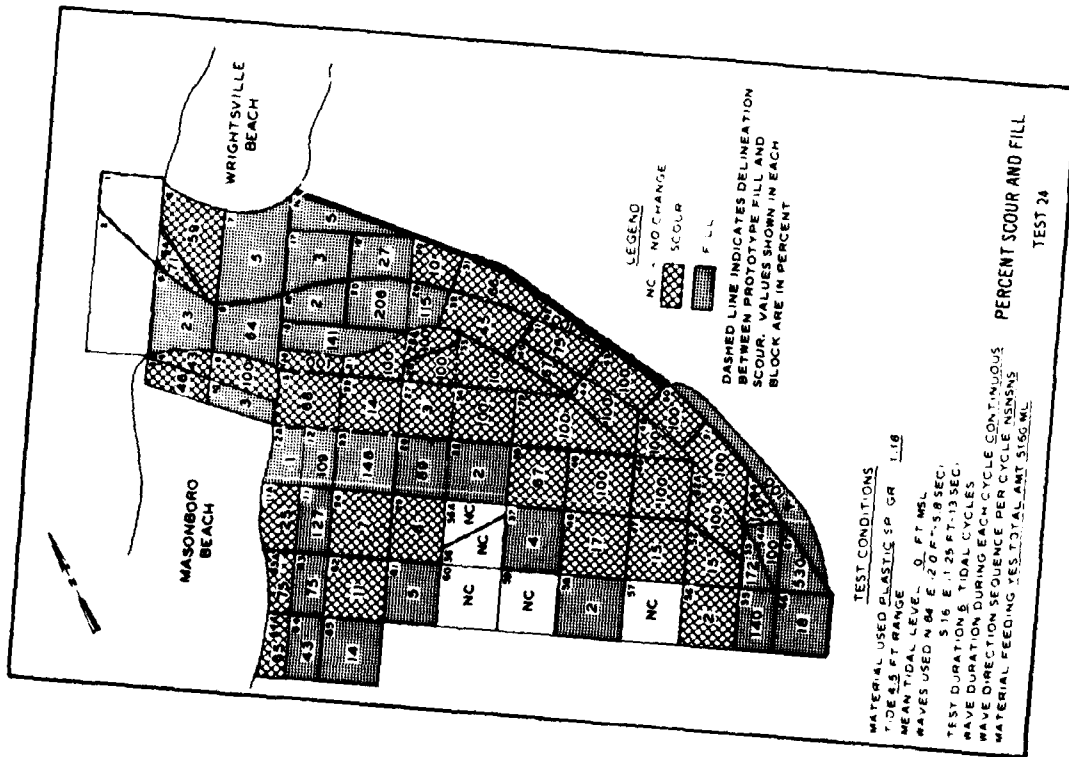


PLATE 87

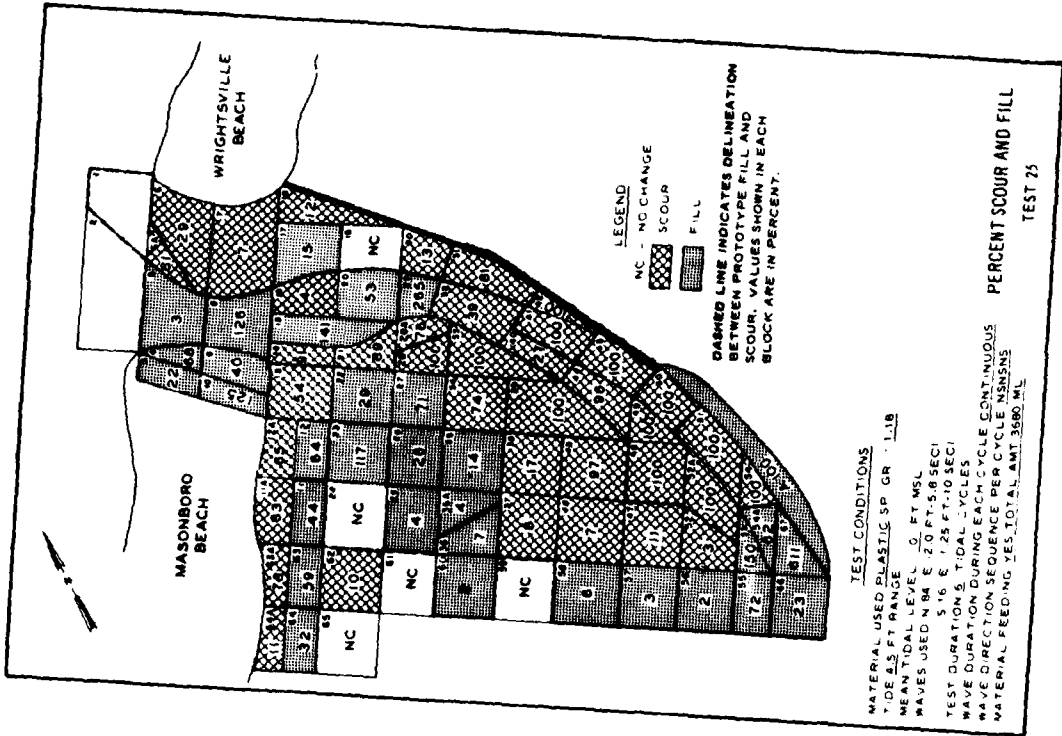


PLATE 88

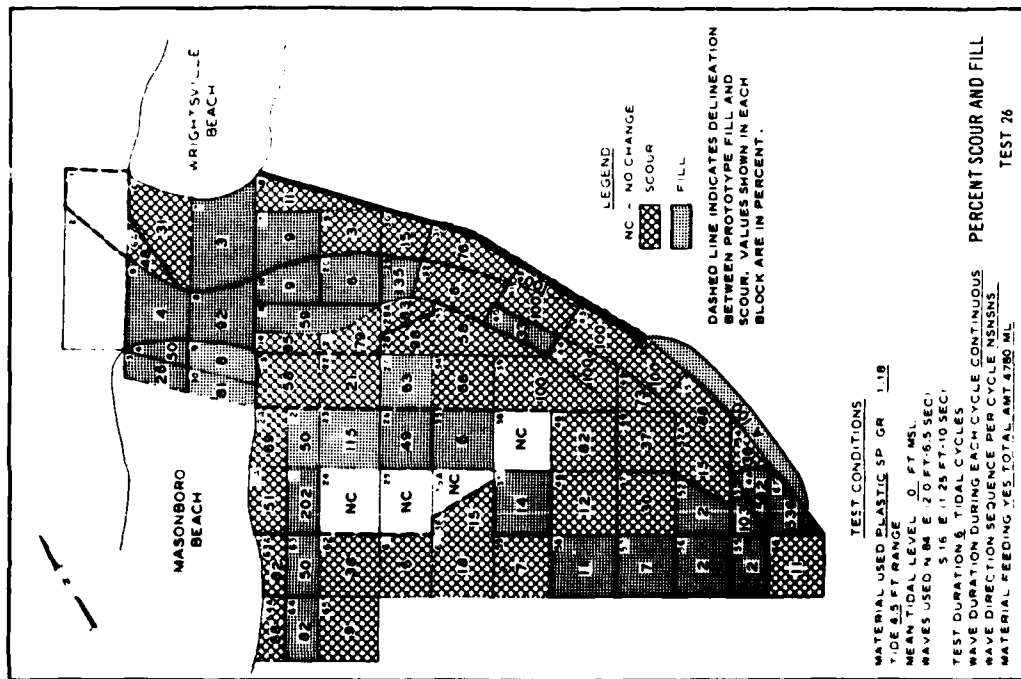


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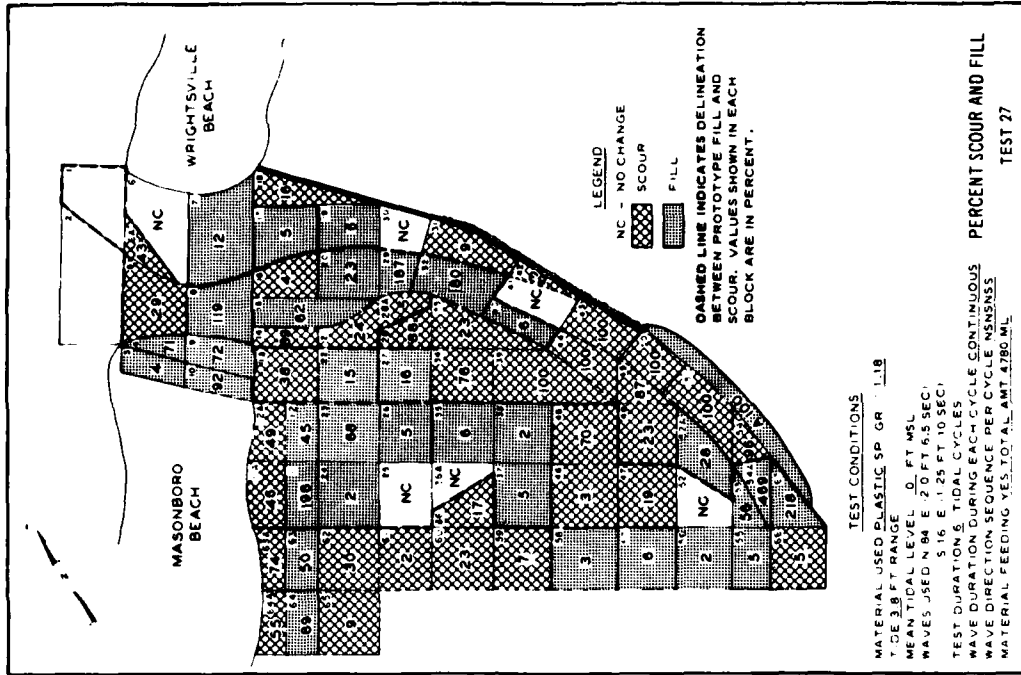


PLATE 90

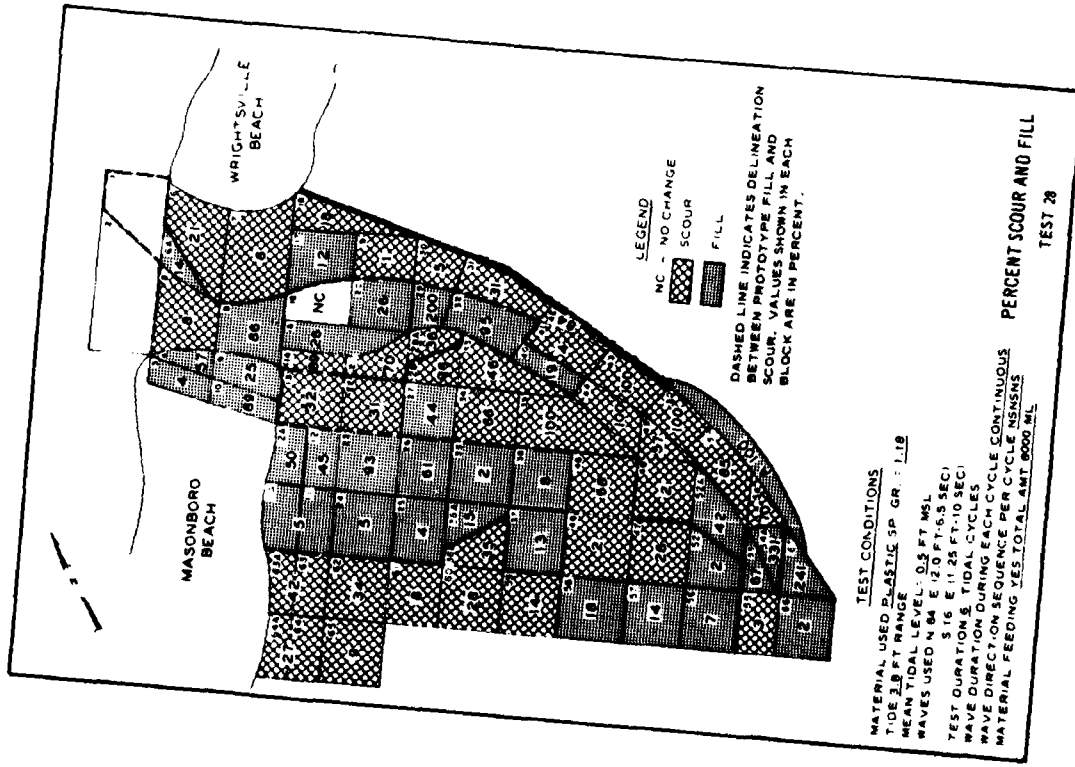


PLATE 91

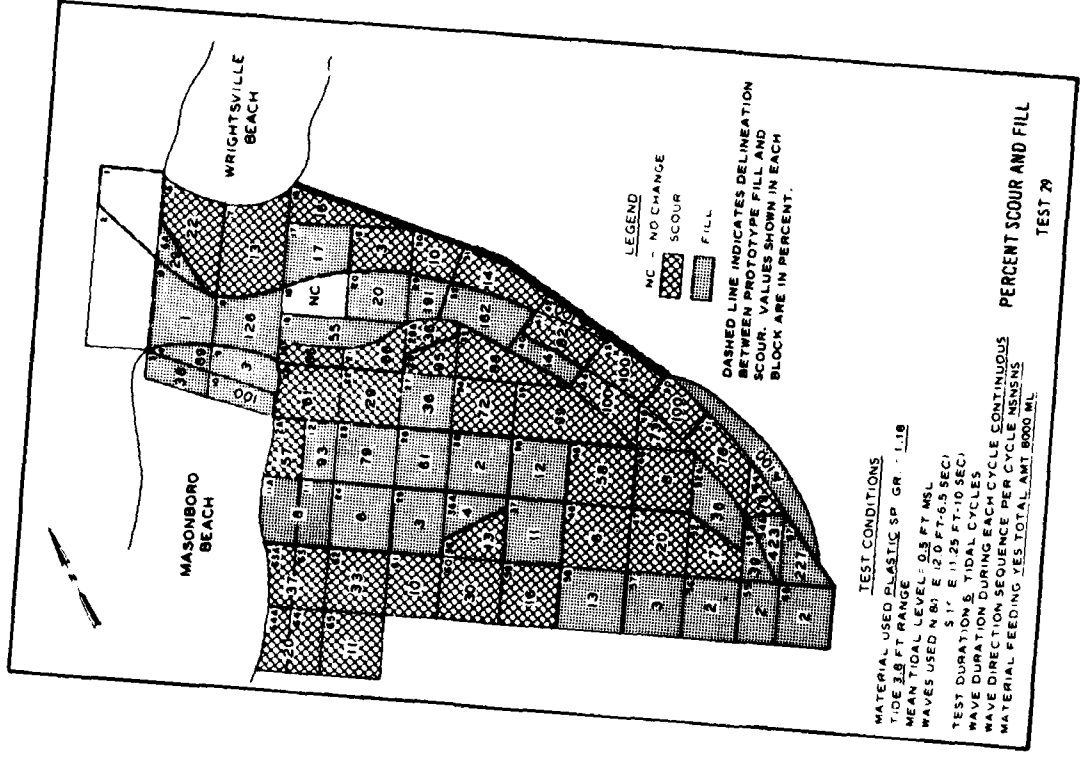


PLATE 92

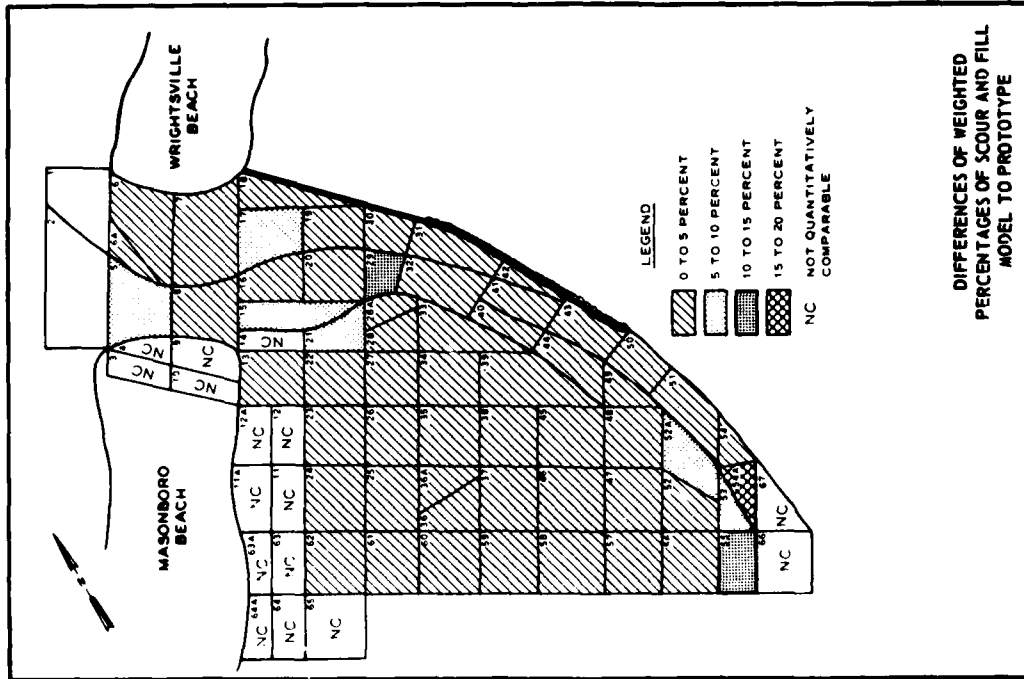


PLATE 94

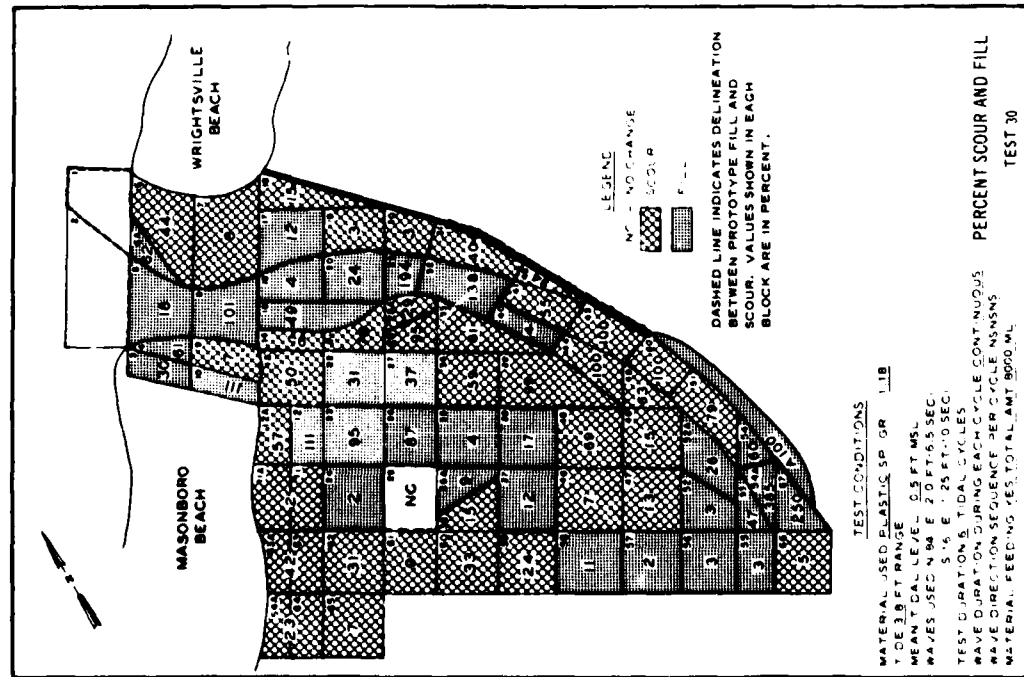


PLATE 93









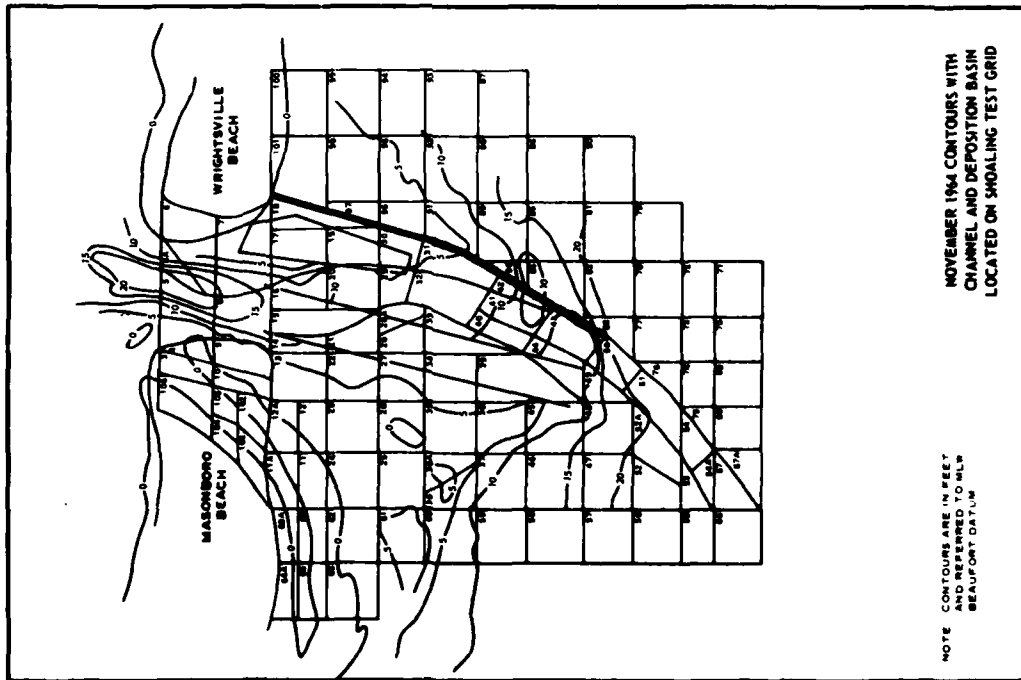


PLATE 103

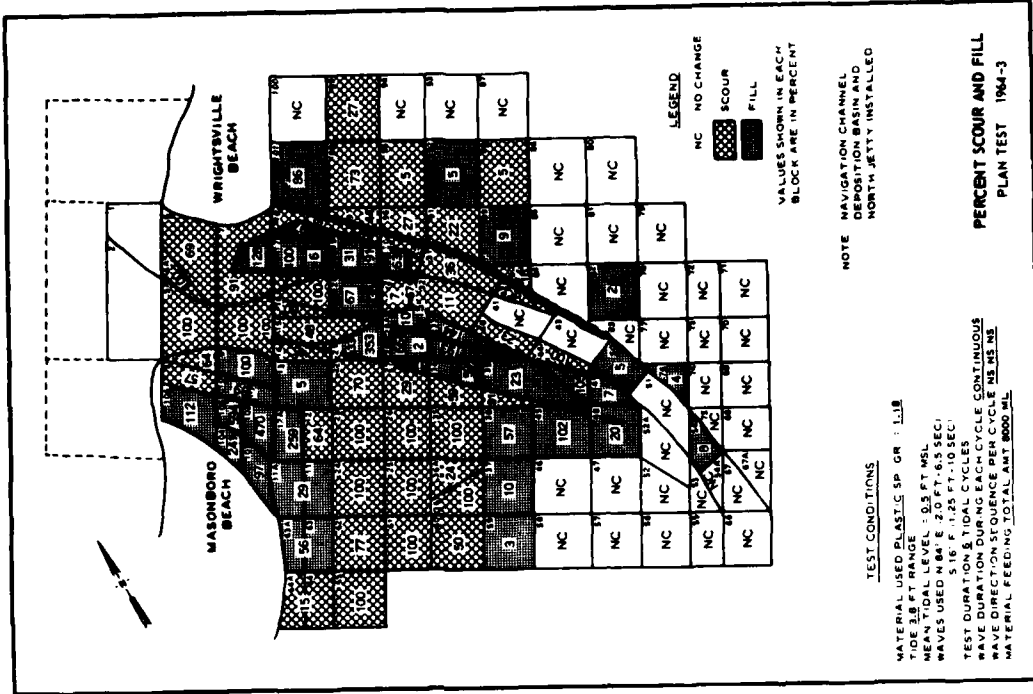


PLATE 104

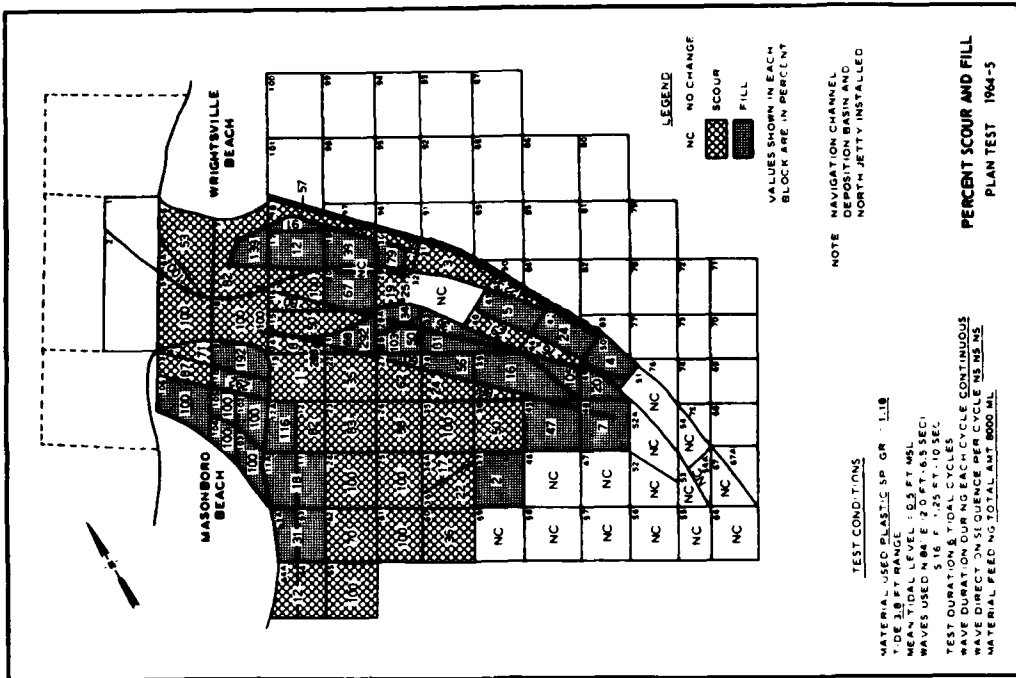


PLATE 106

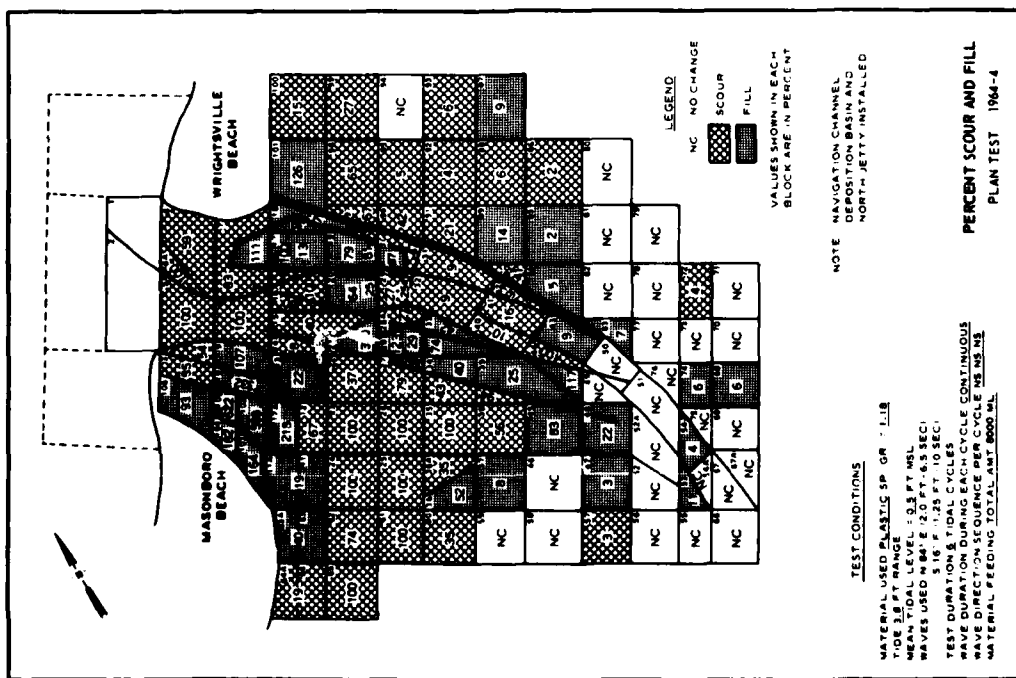


PLATE 105

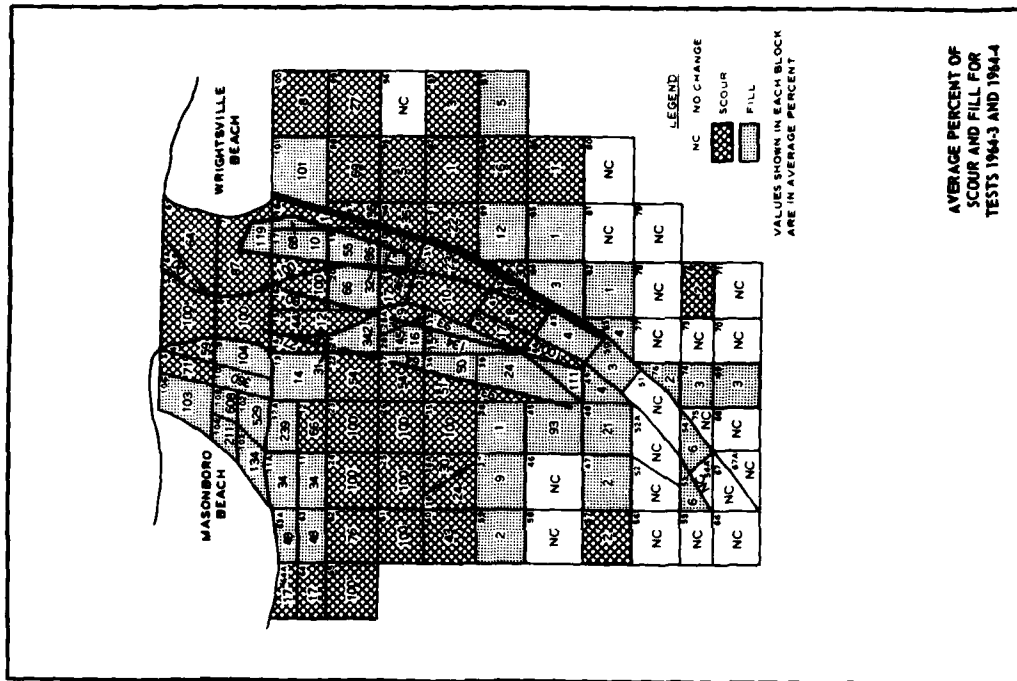


PLATE 108

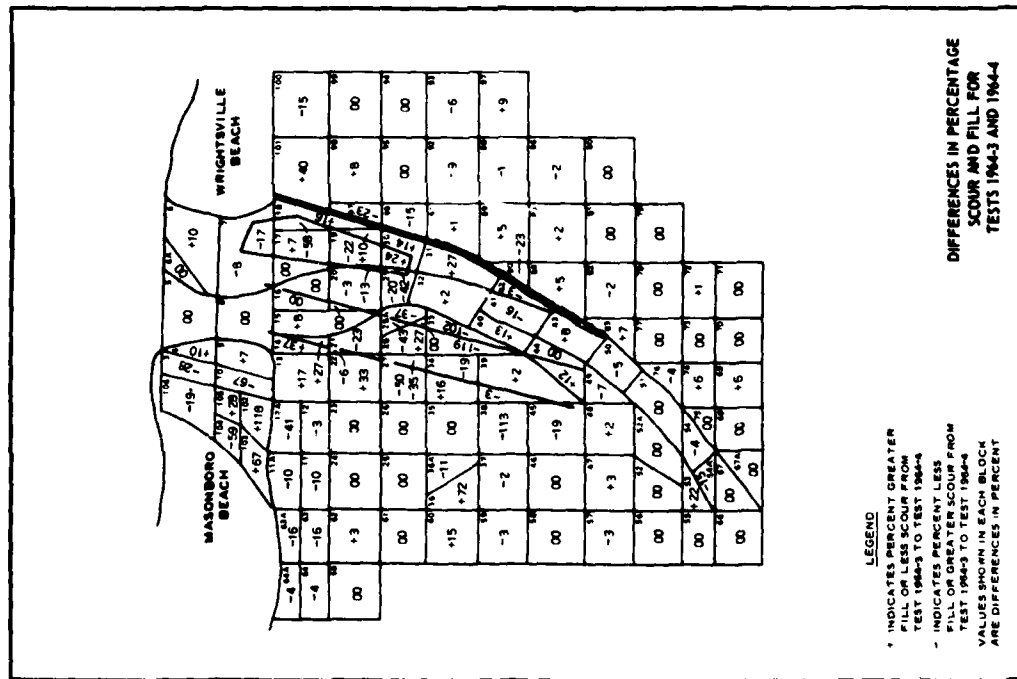
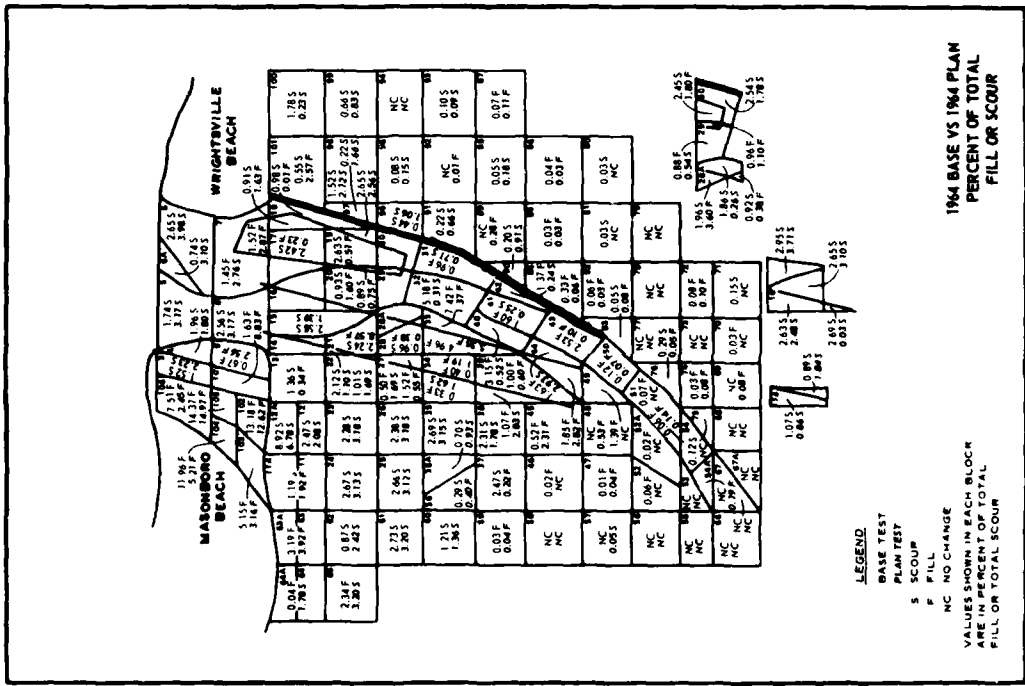


PLATE 107





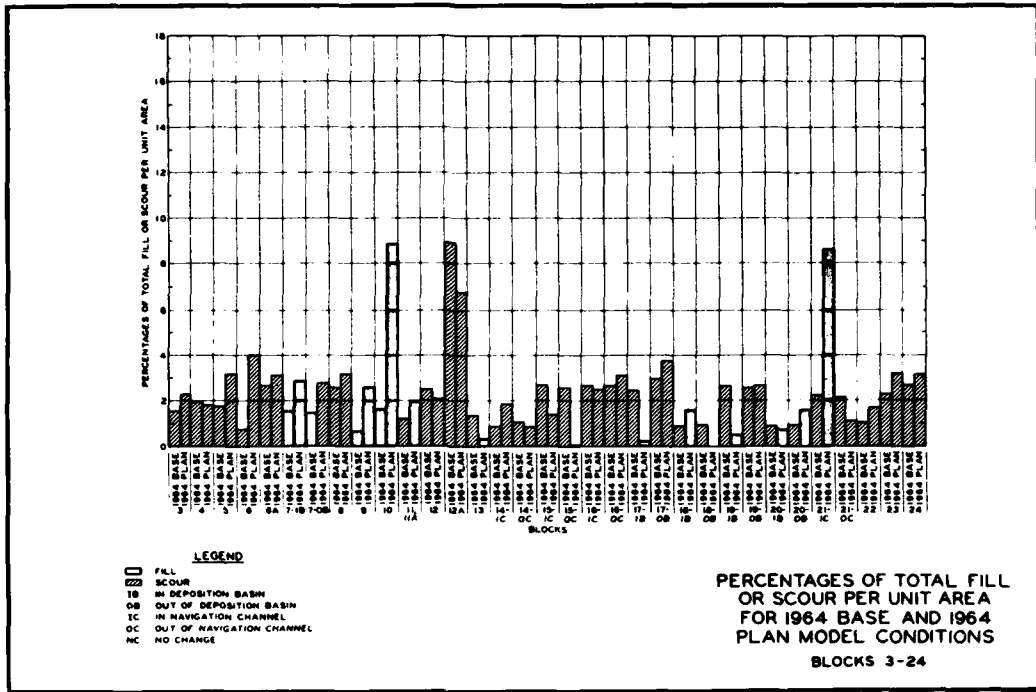


PLATE 112

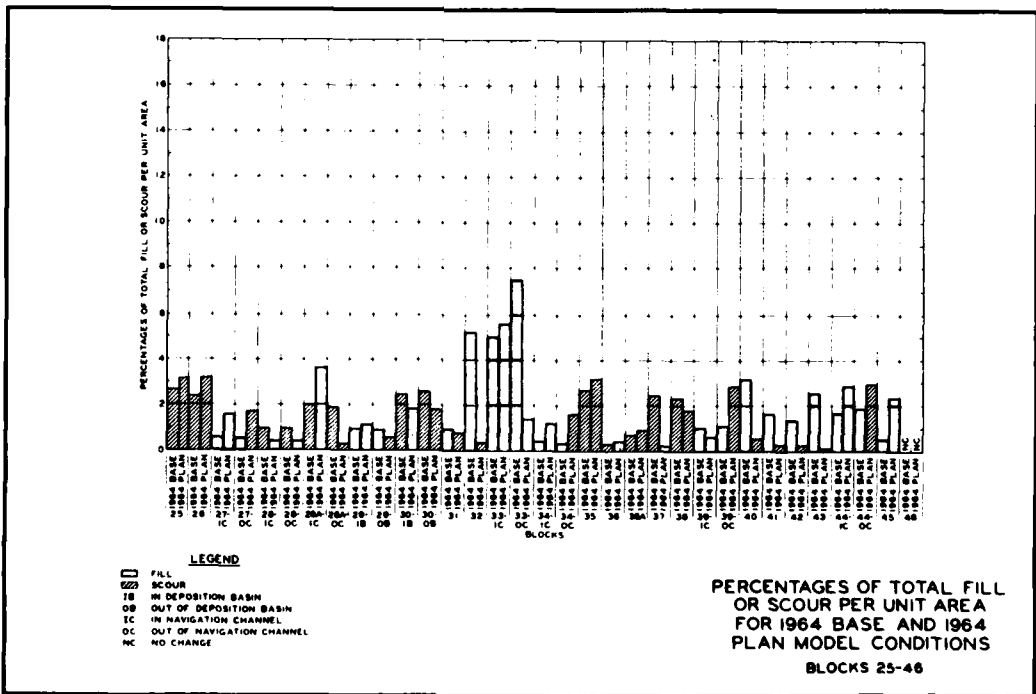


PLATE 113

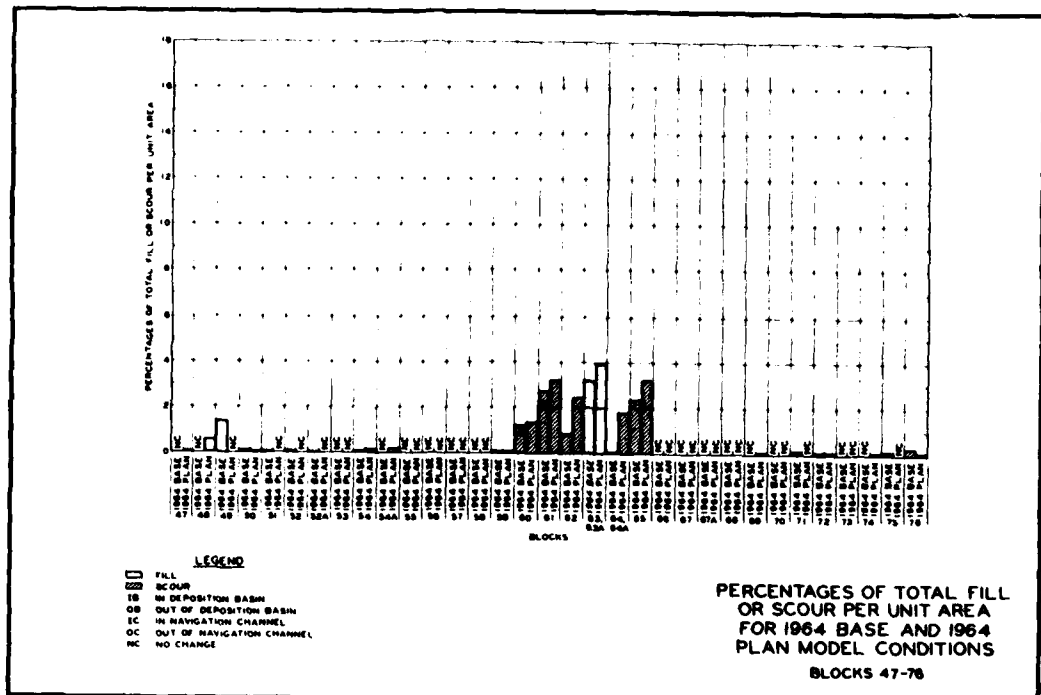


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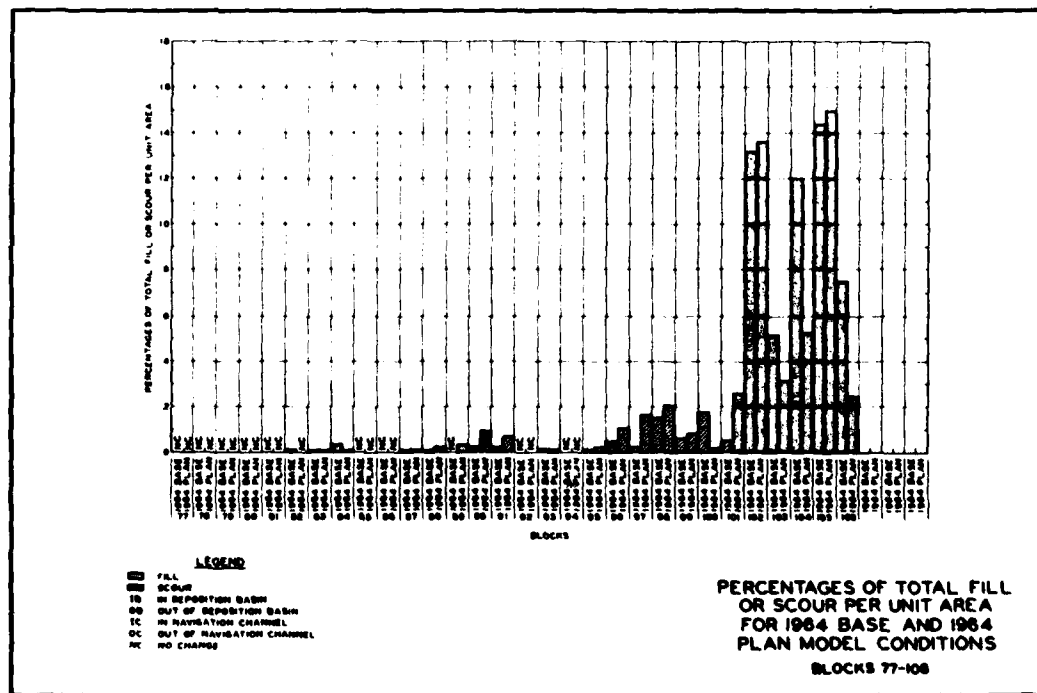


PLATE 115

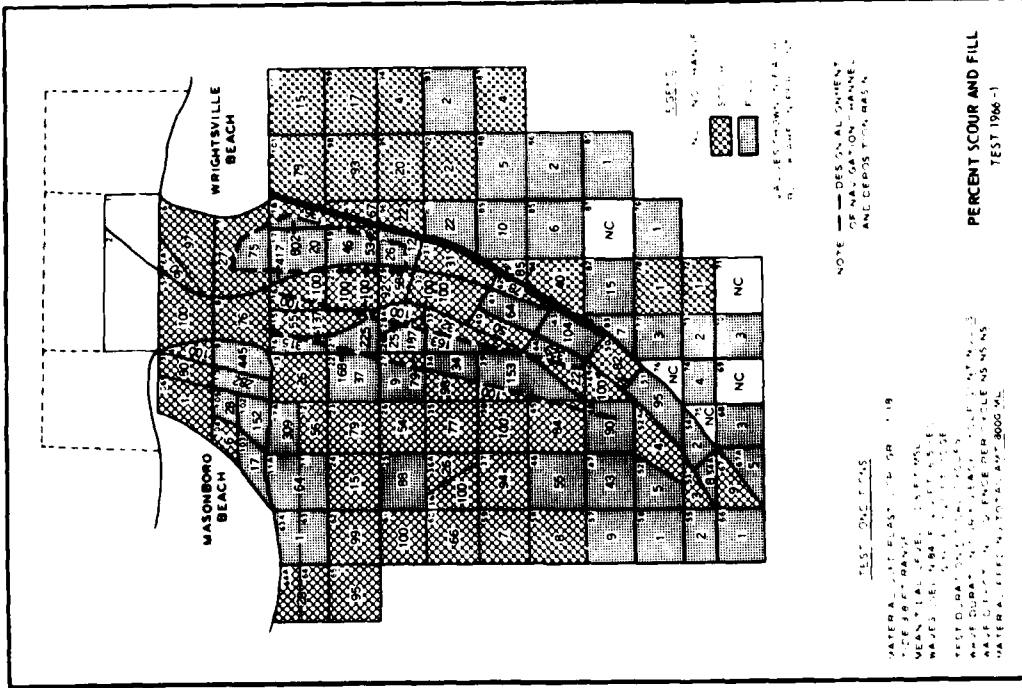


PLATE 117

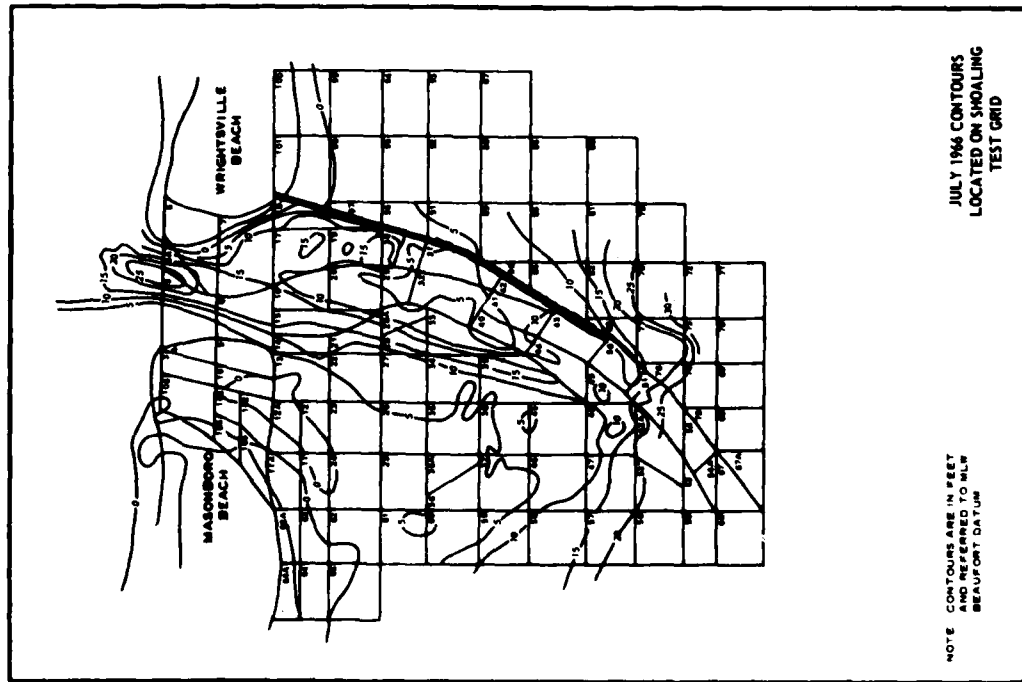


PLATE 116

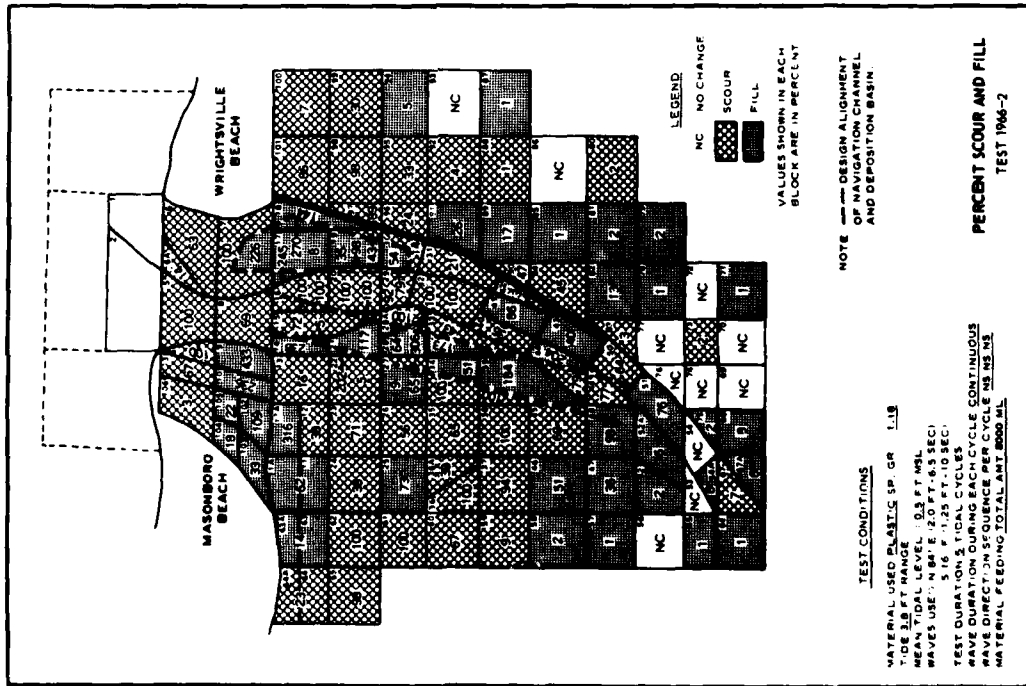


PLATE 118

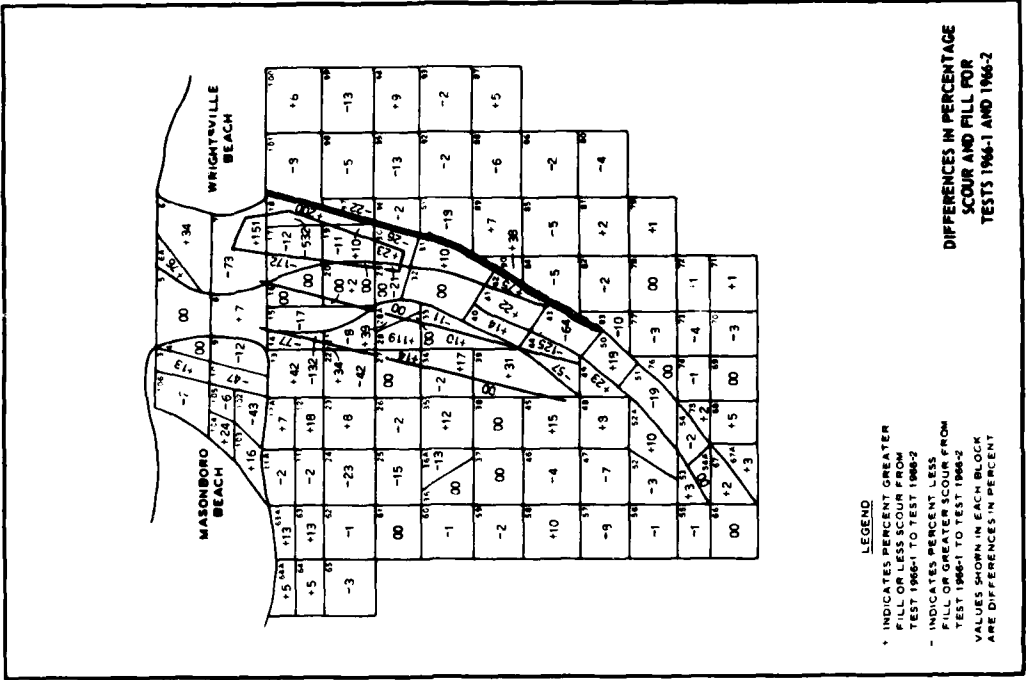
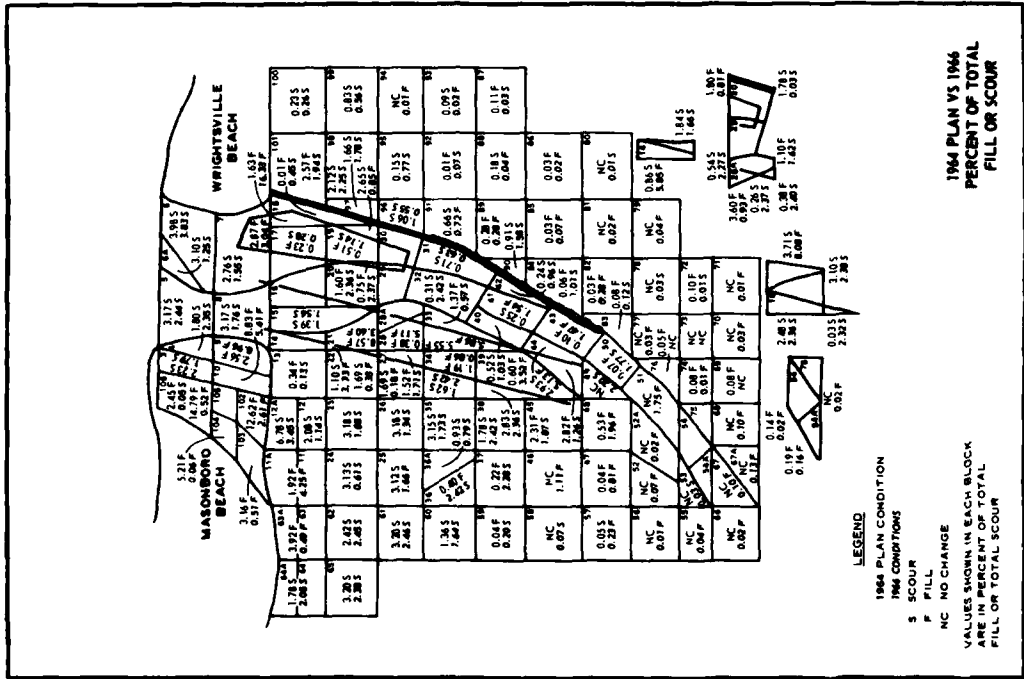


PLATE 119





1964 PLAN VS 1966 PERCENT OF TOTAL FILL OR SCOUR

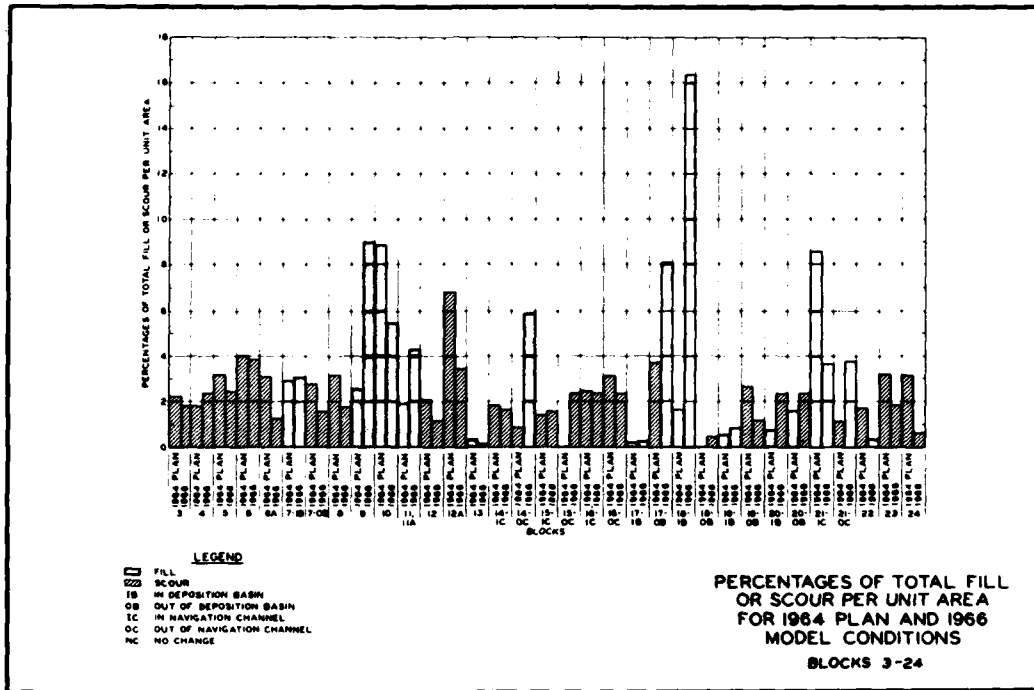


PLATE 123

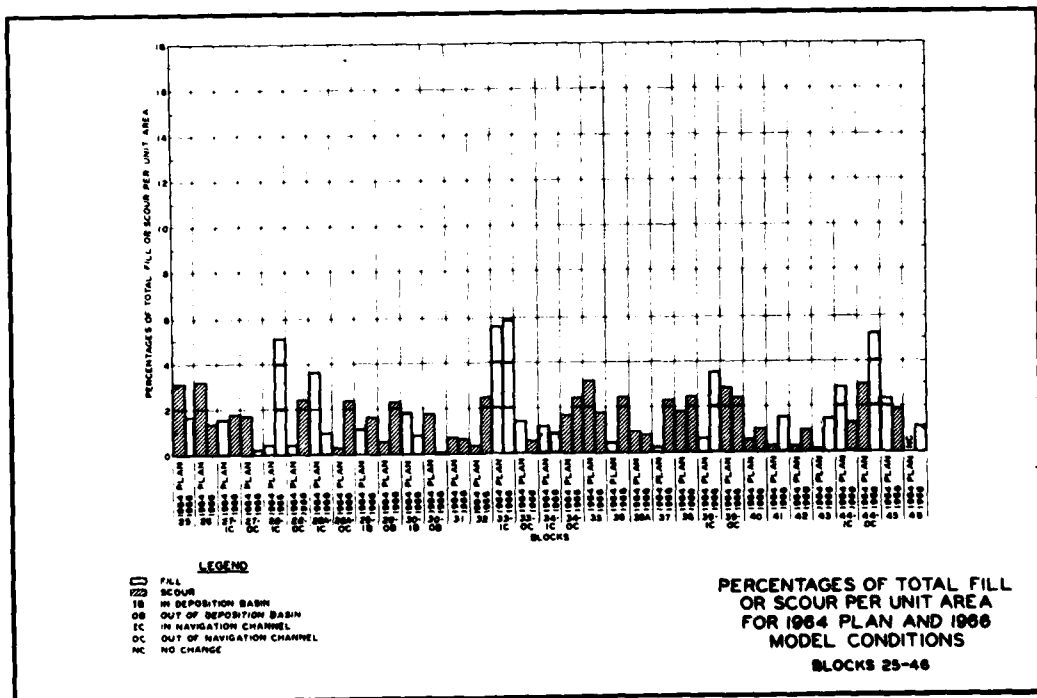


PLATE 124

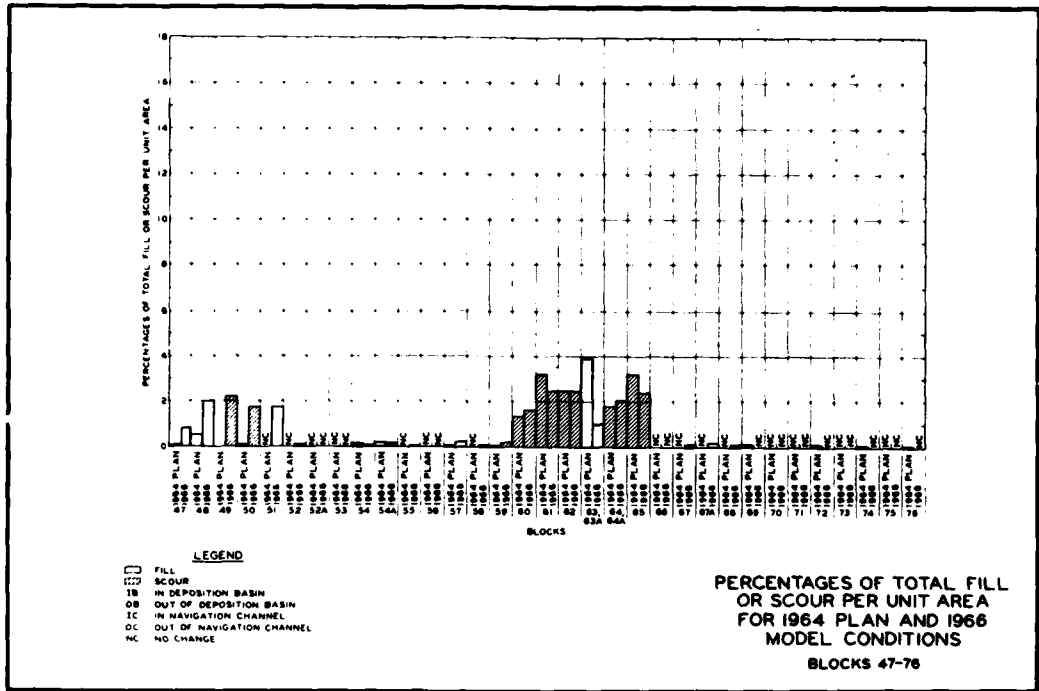


PLATE 125

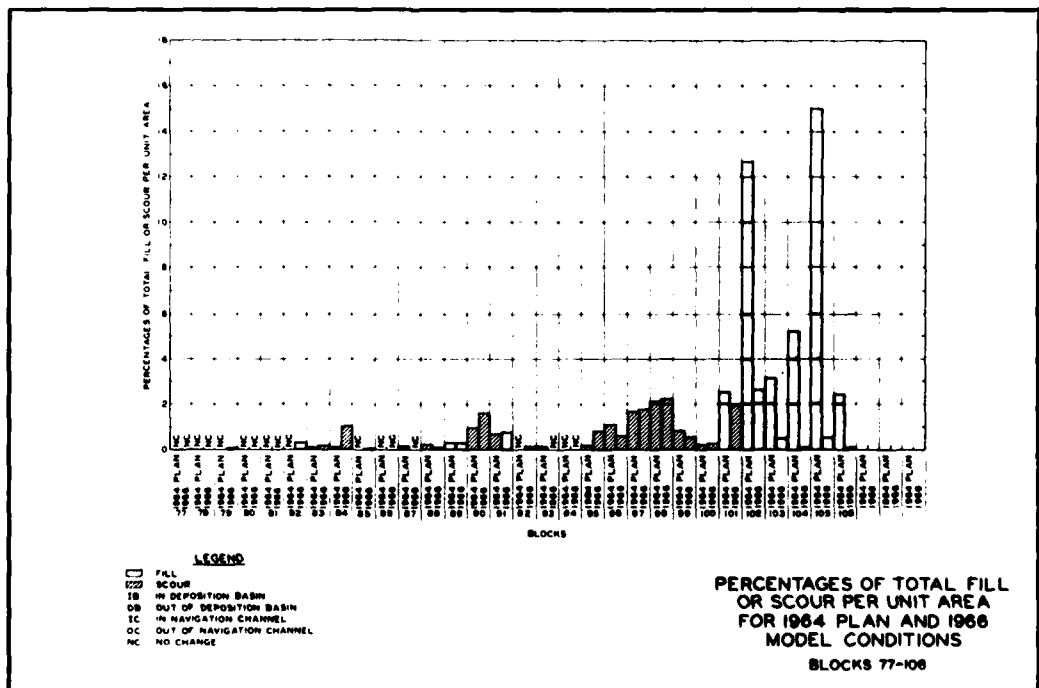


PLATE 126

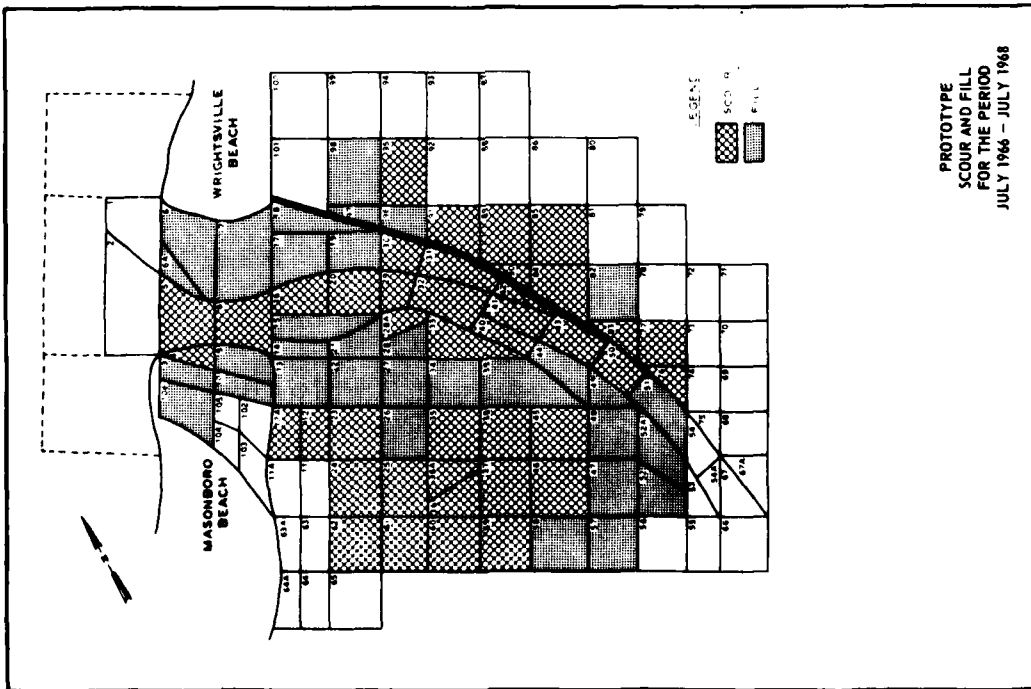


PLATE 127

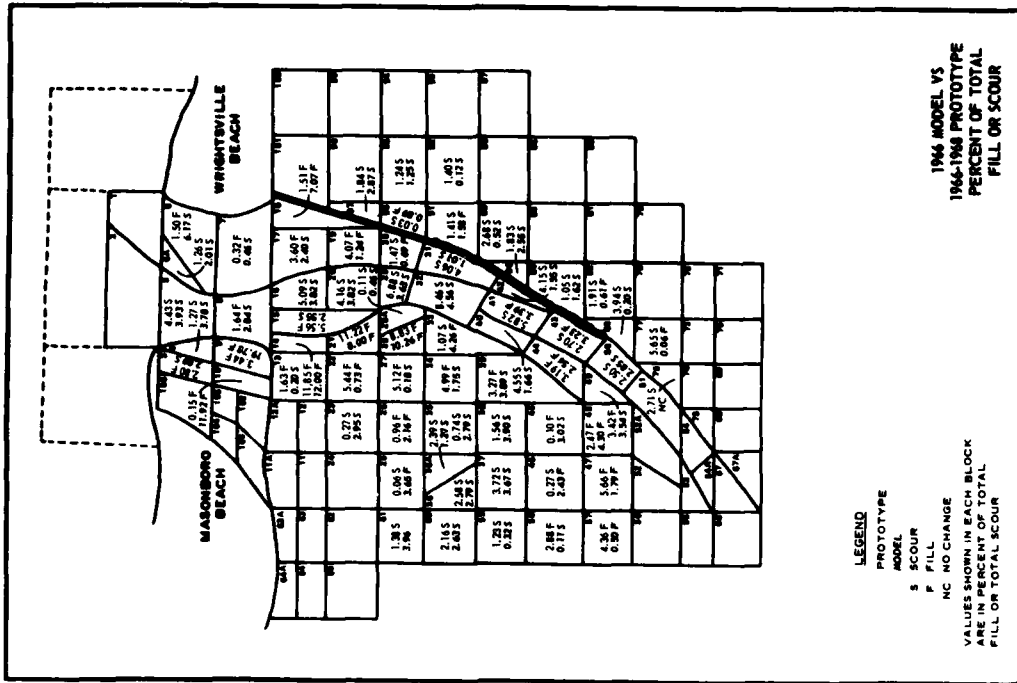


PLATE 128

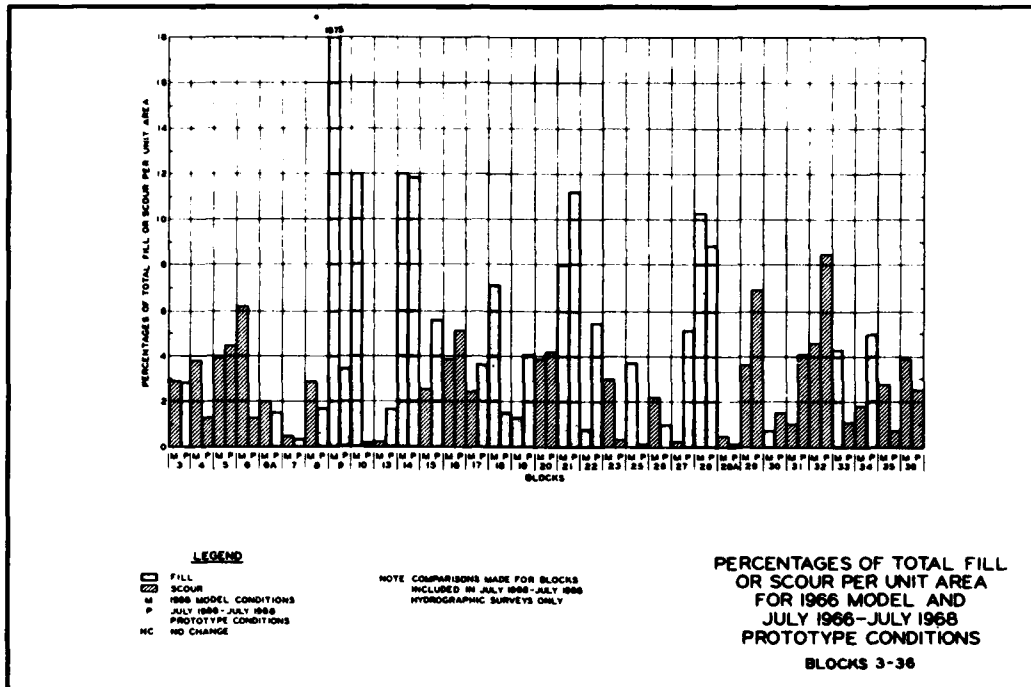


PLATE 129

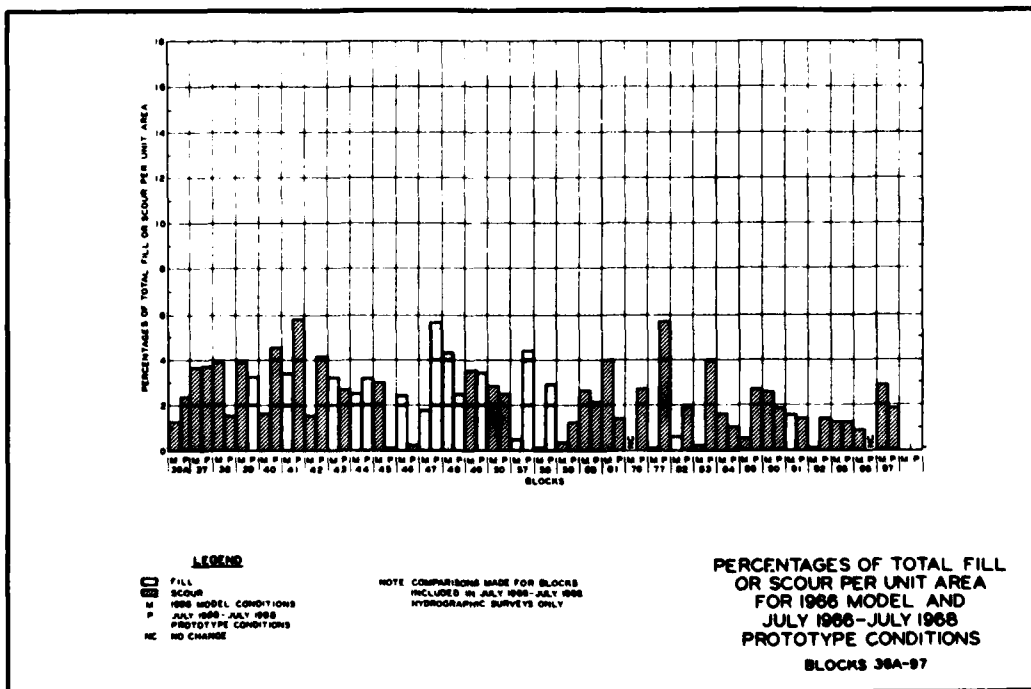


PLATE 130

## APPENDIX A: PHYSICAL CHARACTERISTICS OF SHOALING TEST MATERIALS

1. Throughout the Masonboro Inlet fixed-bed model shoaling studies, four materials were used: sand (sp gr = 2.65), expanded shale (sp gr = 2.1), naturalite (a ground pumice, sp gr = 1.17), and plastic (Tenite butyrate, sp gr = 1.18). Test procedures to define the physical parameters are discussed in the subsequent two paragraphs.

### Test Procedures

2. The angle of repose was measured by means of a clear plastic box, 30 cm on a side. The material was first stirred to assure a uniform mixture of particle sizes, then submerged in water and molded into one half of a cone against one side of the box. Material at the base of the cone was pulled away so that the material in the cone could stabilize at the steepest slope possible. The angle between the horizontal plane and the side of the cone (Plate A1) was then measured. This procedure was repeated four times for each material. A summary of the measured angles is shown in Plate A1.

3. The fall velocity measurements were completed using a 2000-ml graduated cylinder which is 8 cm in diameter and 48 cm deep. Ideally, a larger diameter and taller vessel is needed for the larger particle sizes; but since most of the materials were of relatively small dimensions, the graduated cylinder was used. The fall velocities are standard fall velocities, i.e., standard conditions were maintained, the medium being quiescent distilled water at 24°C. Velocities were measured over a distance of 29.25 cm, and the timing device was an electric timer reading to hundredths of a second. An individual piece of material was held with tweezers, submerged in the water, shaken to assure that no air bubbles were attached, and then released. A check of the system was made with the sand sample of 0.25-mm-diam grain size and compared with the fall velocity as determined from a curve developed by

Rouse.<sup>6\*</sup> Velocities compared within 0.2 cm/sec. Results of the fall velocity measurements are shown in Plate A2.

#### Test Results

4. The sand tested was similar to that used in the Model Materials Evaluation program. Fall velocity versus sand diameter curves have been completed previously by others, and one such curve (for 24°C, interpolated from Rouse's curves) has been plotted in Plate A2. There are also particle diameter versus angle of repose curves for sand by the U. S. Bureau of Reclamation (USBR). The measured angle of repose for the WES sand was 31 degrees. This agreed closely with the angle of repose obtained by the USBR for moderately angular to very angular shaped sand. An examination of the WES sand showed the material to be angular; the gradation curve of the sand is shown in Plate A3.

5. The expanded shale was a moderately rounded material, varying in size from 0.15 to 4 mm, with a mean size of approximately 1 mm. The gradation curve is shown in Plate A4. Data points for fall velocities fell in a relatively smooth curve. The angle of repose was 33 degrees.

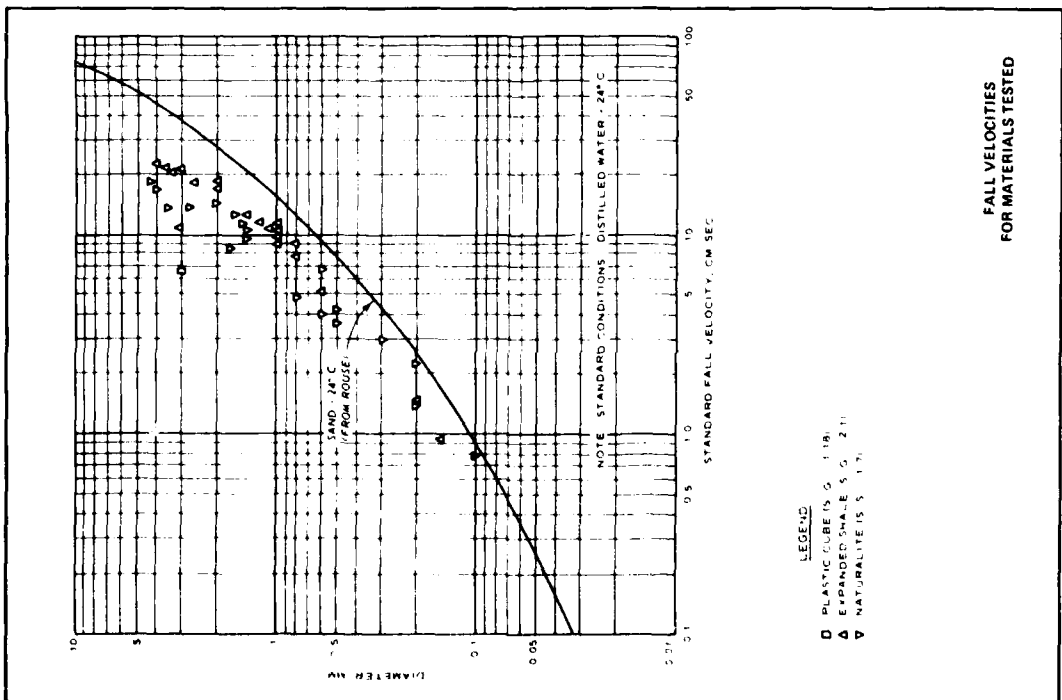
6. Naturalite is a very angular material, and this most likely accounts for the scatter of fall velocity data points. No attempt was made to classify the fall velocities by shape factors (shape factor, S.F., is equal to the length of the shortest of three mutually perpendicular axes, divided by the square root of the product of the length of the other two axes, i.e.,  $S.F. = c/\sqrt{ab}$ ). Shape factors varied from 0.4 to 0.9 for some of the larger sized particles. The mean size for this material was approximately 0.7 mm, with sizes ranging from almost a powder to 4 mm; the gradation curve is shown in Plate A5. The angle of repose was 35 degrees.

7. The plastic material was approximately a 3-mm cube, with slight variations in size and shape, usually toward a form with rhomboidal

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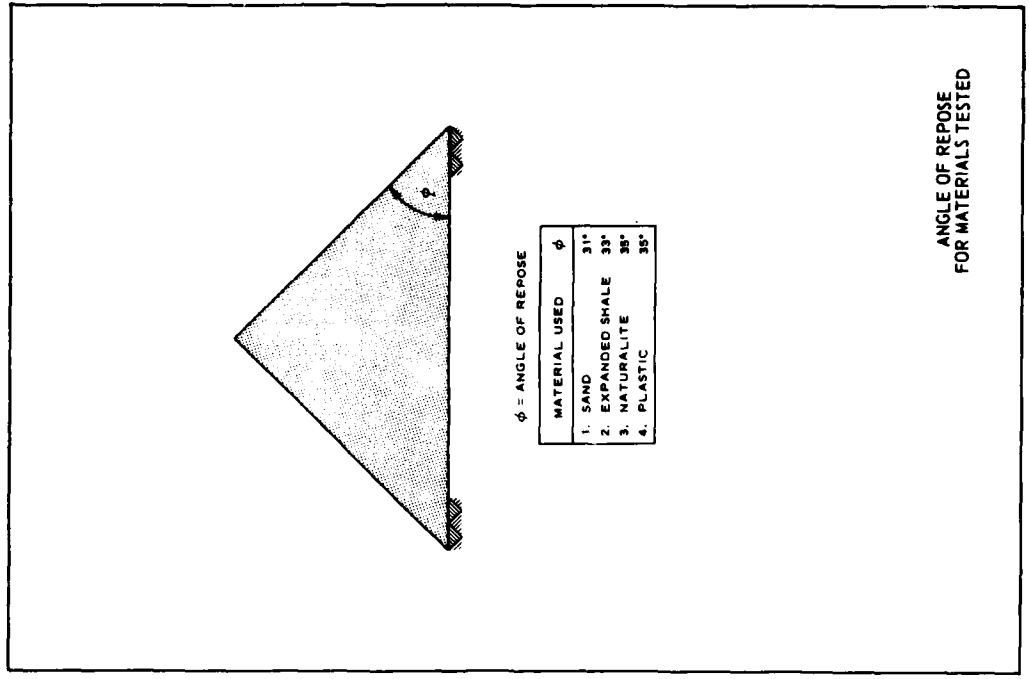
\* Raised number refers to similarly numbered item in "References" at end of main text.

sides. No attempt was made to delineate geometric differences between each piece of material tested, but for the 19 samples tested, the average fall velocity was 6.5 cm/sec with a standard deviation of 0.6 cm/sec. The angle of repose was 35 degrees.



FALL VELOCITIES FOR MATERIALS TESTED

PLATE A1



ANGLE OF REPOSE FOR MATERIALS TESTED

PLATE A2





Seabergh, William C.  
Supplementary tests of Masonboro Inlet fixed-bed model; hydraulic model investigation / by William C. Seabergh and Richard A. Sager. - Fort Belvoir, Va. : U.S. Coastal Engineering Research Center ; Springfield, Va. : available from National Technical Information Service, 1980. [233] p. : ill. : 27 cm. - (GITI report 18)  
"General Investigation of Tidal Inlets - a program of research conducted jointly by U.S. Army Coastal Engineering Research Center, Fort Belvoir, Virginia, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi."  
Includes bibliographical references.  
Appendixes.  
This report describes supplemental tests (three separate studies) performed in the Masonboro Inlet fixed-bed model; these were not reported in GITI Report 15.  
1. Tidal inlets. 2. Hydraulic models. 3. Masonboro Inlet.  
I. Title. II. Sager, Richard A. III. Series: U.S. Army, Corps of Engineers. GITI report ; 18.  
.I18 .U581r no. 18 551.4  
GB454

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