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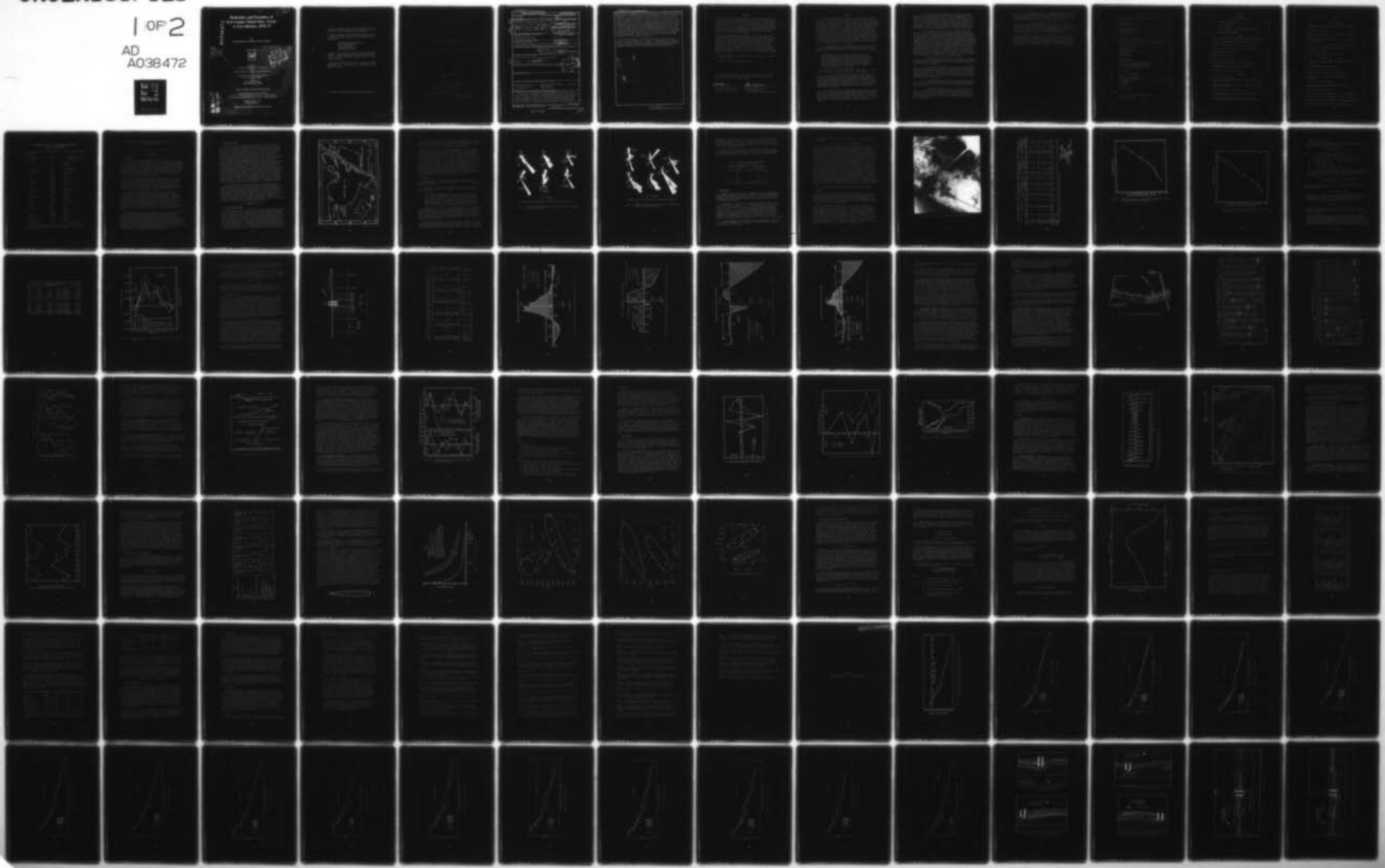

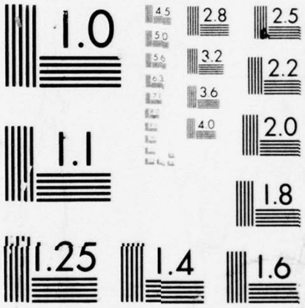

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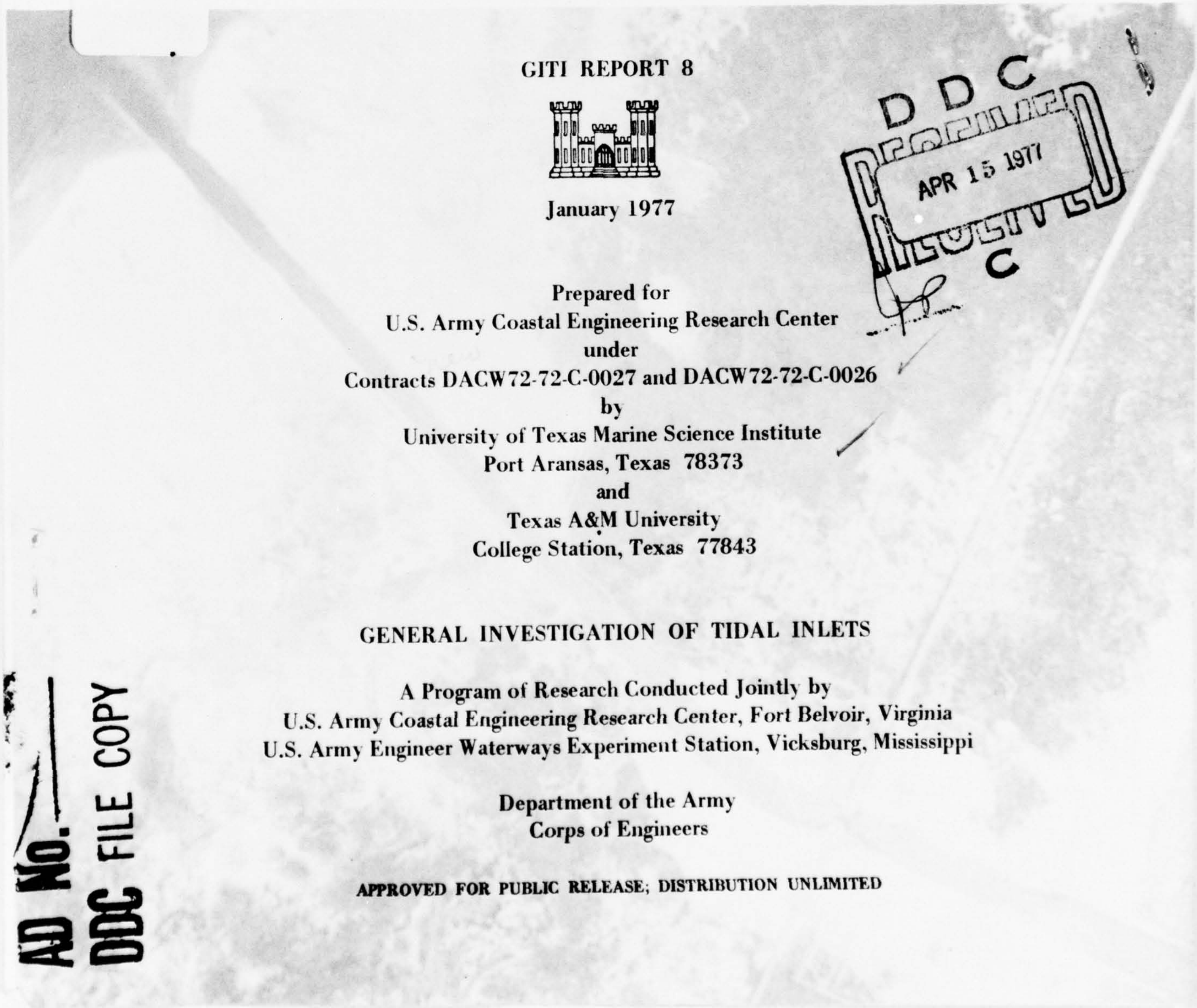
Hydraulics and Dynamics of New Corpus Christi Pass, Texas: A Case History, 1972-73

by

E.W. Behrens, R.L. Watson, and C. Mason



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January 1977

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under
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and
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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

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Cover Photo: Corpus Christi Water Exchange Pass, Texas, July 1973

U.S. ARMY COASTAL ENGINEERING RESEARCH CENTER
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Fort Belvoir, Virginia 22060

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HYDRAULICS AND DYNAMICS OF NEW CORPUS CHRISTI PASS, TEXAS: A CASE HISTORY: 1972-73

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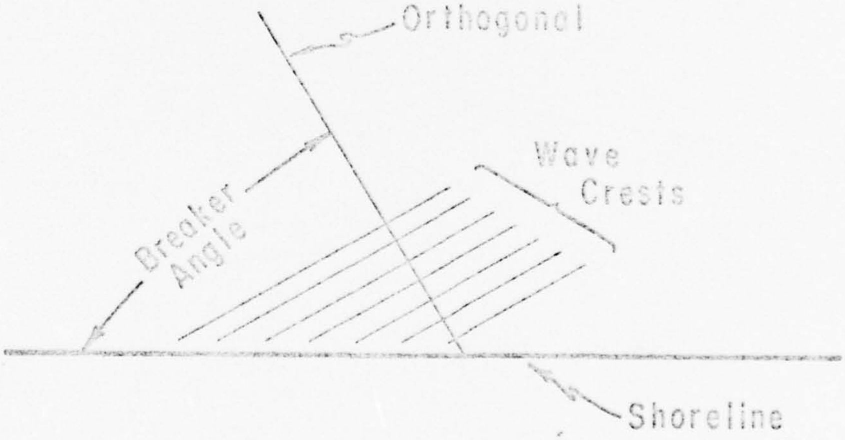

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Inlet hydraulics	Tidal hydraulics	

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

In 1972, a 2-mile channel was dredged through Mustang Island, Texas, to increase water exchange and fish migration between Corpus Christi Bay and the Gulf of Mexico. The pass' initial adjustment to tides, waves, and other forces was measured the first year following the opening. Hydraulic and sedimentary effects of the pass were studied by obtaining detailed bathymetric, topographic, and hydraulic surveys of the pass and adjacent gulf beaches; daily wave observations provided information on the seasonal → next page

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variability in wave height, period and direction. An estimated 1 million cubic yards of sand accumulated at the pass during construction of two gulf jetties. Thereafter, a loss of sand greater than the estimated net annual longshore transport rate occurred on beaches south (downdrift) of the jetties. Considerable sediment was deposited on shoals at the bay end of the pass with little accumulation in the pass. Hydraulic measurements indicate that channel frictional resistance increased by about 50 percent over the study period, although greater variability occurred during individual tidal cycles. Tidal discharge through the pass was highly dependent upon variations in the gulf tides, with equal volumes of ebb and flood flows during diurnal tides and strong flood predominance during mixed and semidiurnal cycles. The average discharge through the pass was only about 3 percent of the total tidal prism of Corpus Christi Bay, indicating that the bay tides, which partly control flow through the pass, result primarily from passage of the tide through Aransas Pass, the major bay-gulf connection.

The pass was marginally stable during the first year, but the wide range of climatic conditions in the region will probably cause the pass to be stable in some years and unstable in others. Although the pass undoubtedly influences bay water within the immediate vicinity, no significant effect on flushing of Corpus Christi Bay resulted from the pass construction.

FOREWORD

This report results from work done under contracts between the U.S. Army Coastal Engineering Research Center (CERC) and the University of Texas Marine Science Institute at Port Aransas, Texas (Contract DACW72-72-C-0027) and Texas A&M University at College Station, Texas (Contract DACW72-72-C-0026). It is one in a series of reports from the Corps of Engineers' General Investigation of Tidal Inlets (GITI), which is under the technical surveillance of CERC and is conducted by CERC, the U.S. Army Engineer Waterways Experiment Station (WES), other Government agencies, and by private organizations.

The report combines the final reports of the above contracts by Behrens and Watson (1973) and deFehr (1973), and was prepared by Curtis Mason, CERC technical monitor. The major part of this report is based on the Behrens and Watson report. Much of the hydraulic data were collected by deFehr. Assistance in data collection was provided by W.H. Sohl, H.S. Finkelstein, and W.N. Seelig. J.M. Kieslich analyzed the wave data. Some of the initial survey bench mark data were supplied by the Texas Department of Parks and Wildlife and by Urban Engineering Company of Corpus Christi, Texas. Dean Morrough P. O'Brien and Professor Robert L. Wiegel of the Coastal Engineering Research Board reviewed the report.

Technical Directors of CERC and WES were 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.

JOHN L. CANNON
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Commander and Director
Coastal Engineering Research Center

PREFACE

1. The Corps of Engineers, through its Civil Works program, has sponsored, over the past 23 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 routinely 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, by allowing for the ingress of storm surges and egress of flood waters, play an important role in the flushing of bays and lagoons.

2. A research program, the General Investigation of Tidal Inlets (GITI), 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: (a) inlet classification, (b) inlet hydraulics, and (c) inlet dynamics.

a. Inlet Classification. 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. Inlet Hydraulics. The objectives of the inlet hydraulics study are to define 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: (1) idealized inlet model study, (2) evaluation of state-of-the-art physical and numerical models, and (3) 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 part 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, North Carolina, 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. *Inlet Dynamics*. 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: (1) model materials evaluation, (2) movable-bed modeling evaluation, (3) reanalysis of a previous inlet model study, and (4) 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.

(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 1975, a report entitled, "Preliminary Report: Laboratory Study of the Effect of an Uncontrolled Inlet on the Adjacent Beach," 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. This report discusses the results of a field study on inlet dynamics and hydraulics performed at Corpus Christi Water Exchange Pass, Texas, during 1972-73. The data collected provide information on both the long- and short-term stability of the pass and on the wave and tidal forces which affect the dynamics of the pass. Another report (Watson and Behrens, 1976) presents more comprehensive results for the period 1973 to 1975.

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

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

Multiply	by	To obtain
inches	25.4	millimeters
	2.54	centimeters
square inches	6.452	square centimeters
cubic inches	16.39	cubic centimeters
feet	30.48	centimeters
	0.3048	meters
square feet	0.0929	square meters
cubic feet	0.0283	cubic meters
yards	0.9144	meters
square yards	0.836	square meters
cubic yards	0.7646	cubic meters
miles	1.6093	kilometers
square miles	259.0	hectares
acres	0.4047	hectares
foot-pounds	1.3558	newton meters
ounces	28.35	grams
pounds	453.6	grams
	0.4536	kilograms
ton, long	1.0160	metric tons
ton, short	0.9072	metric tons
degrees (angle)	0.1745	radians
Fahrenheit degrees	5/9	Celsius degrees or Kelvins[1]

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[1] To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use formula: C = (5/9) (F − 32). To obtain Kelvin (K) readings, use formula: K = (5/9) (F − 32) + 273.15.

HYDRAULICS AND DYNAMICS OF NEW CORPUS CHRISTI PASS, TEXAS: A CASE HISTORY, 1972-73

by

E.W. Behrens, R.L. Watson, and C. Mason

I. INTRODUCTION

1. Background.

In August 1972 the Texas Department of Parks and Wildlife constructed the Corpus Christi Water Exchange Pass, a 10,000-foot-long channel connecting Corpus Christi Bay with the Gulf of Mexico. The waterway (hereafter referred to as the pass) was built to enhance flushing of Corpus Christi Bay and to promote fish migrations between the bay and gulf. This report presents the results of a 1-year field study defining the hydraulic and sedimentary characteristics of the pass during initial adjustment, and evaluating the relative importance of waves, tides, and meteorological factors affecting the behavior of the inlet.

Inlet construction has been and continues to be advocated in many coastal areas for the following reasons: (a) Navigational, to provide routes for pleasure and commercial vessels between the ocean and the relatively protected bays, lagoons, and harbors; (b) biological, to allow marine life to migrate between estuarine breeding, nursery areas, and the sea; and (c) water quality, to allow seawater to flush inland tidal waters subjected to extremes in salinity or pollutants.

Prediction of the postconstruction behavior of an inlet and the adjacent beaches is difficult; many inlet improvements have had detrimental effects on both the inlet and the adjacent beaches. Therefore, the first objective of this study was to define the processes affecting sediment transport and deposition in and around a new inlet, and to correlate depositional patterns with discrete combinations of the processes identified. The second objective was to investigate the hydraulic characteristics of the pass, including tidal discharges, inlet friction, and bay and gulf tidal amplitude relationships. Evaluation of effects of the new pass on Corpus Christi Bay salinities was beyond the scope of this study.

A comprehensive data collection program was organized to meet the study objectives. Sequential bathymetric surveys provided information on deposition and erosion in the pass and adjacent bay and gulf regions. Visual wave observations provided a local wave climate for the study year. Tide records from existing and specially installed gages were combined with tidal current measurements to describe the hydraulic characteristics of the inlet-bay system. These data and meteorological summaries were used to describe the processes affecting inlet stability, deposition, and erosion in the study area. The results provide useful, rational inlet and inlet improvement design information with general application.

2. Literature Review.

Since Johnson (1919) recognized the importance of wave and current action on the formation and maintenance of tidal inlets, the study of inlets has been a topic of interest. Brown's (1928) comprehensive paper on detailed sedimentary characteristics of inlets as well as mathematical descriptions of tidal fluctuations and inlet current velocities was the first major work on the subject. O'Brien (1931, 1969) recognized a unique relationship between the minimum cross-sectional area of the inlet channel and the tidal prism of the enclosed bay for a number of stable inlets; Jarrett's (1976) subsequent investigation of a larger number of inlets revealed the widespread applicability of this relationship. Keulegan (1967) developed an improved method for predicting inlet hydraulic characteristics and ocean-bay tide relationships. However, there have been few detailed investigations of the nature and importance of the processes affecting inlet stability. The most complete studies are those of Bruun and Gerritsen (1960) and Bruun (1966) who assessed the importance of various environmental parameters to the problem. Graf (1971) stated that the design of stable channels in alluvial materials has been investigated over a long period of time, and that knowledge of shear stress values required for stability of such channels is fairly well established. However, Bruun and Gerritsen's (1960) application of selected streambed stability theories to the large-scale processes of natural inlets represented an original approach to the determination of stability criteria.

Detailed knowledge of inlet characteristics on the Texas coast is largely the result of Price (1951) who established the importance of north winds on the stability of many Texas inlets and the characteristic patterns exhibited by the ebb and flood current channels of Texas inlets. However, some engineering inlet design studies on the Texas coast have been conducted, mostly because of plans to alter the extreme salinities of Texas bays (Carothers and Innis, 1960); two reports on Texas inlet hydraulics and processes have been published (Mason and Sorensen, 1971; Prather and Sorensen, 1972). Other work pertaining to specific aspects of this study will be cited in the appropriate sections.

II. ENVIRONMENTAL SETTING

1. Barrier Island Geology.

Mustang Island is one of eight barrier islands and peninsulas comprising 290 miles of the Texas coast (Fig. 1). Fisk (1959), Bernard and LeBlanc (1965), and Behrens (1973) indicated that these barriers became emergent features restricting the exchange of waters between estuaries and the Gulf of Mexico about 4,500 years ago. The islands have grown from initially narrow sandbars to over 6 miles wide by a combination of processes; i.e., storm surge washovers, dune field formation and migration, and longshore and onshore sand movement. On Mustang Island these processes have produced (from the sea landward): An offshore region with depths exceeding about 15 feet; a surf zone normally 1,000 to 2,000 feet wide with two or three well-developed offshore bars; a beach consisting of a foreshore (swash zone),

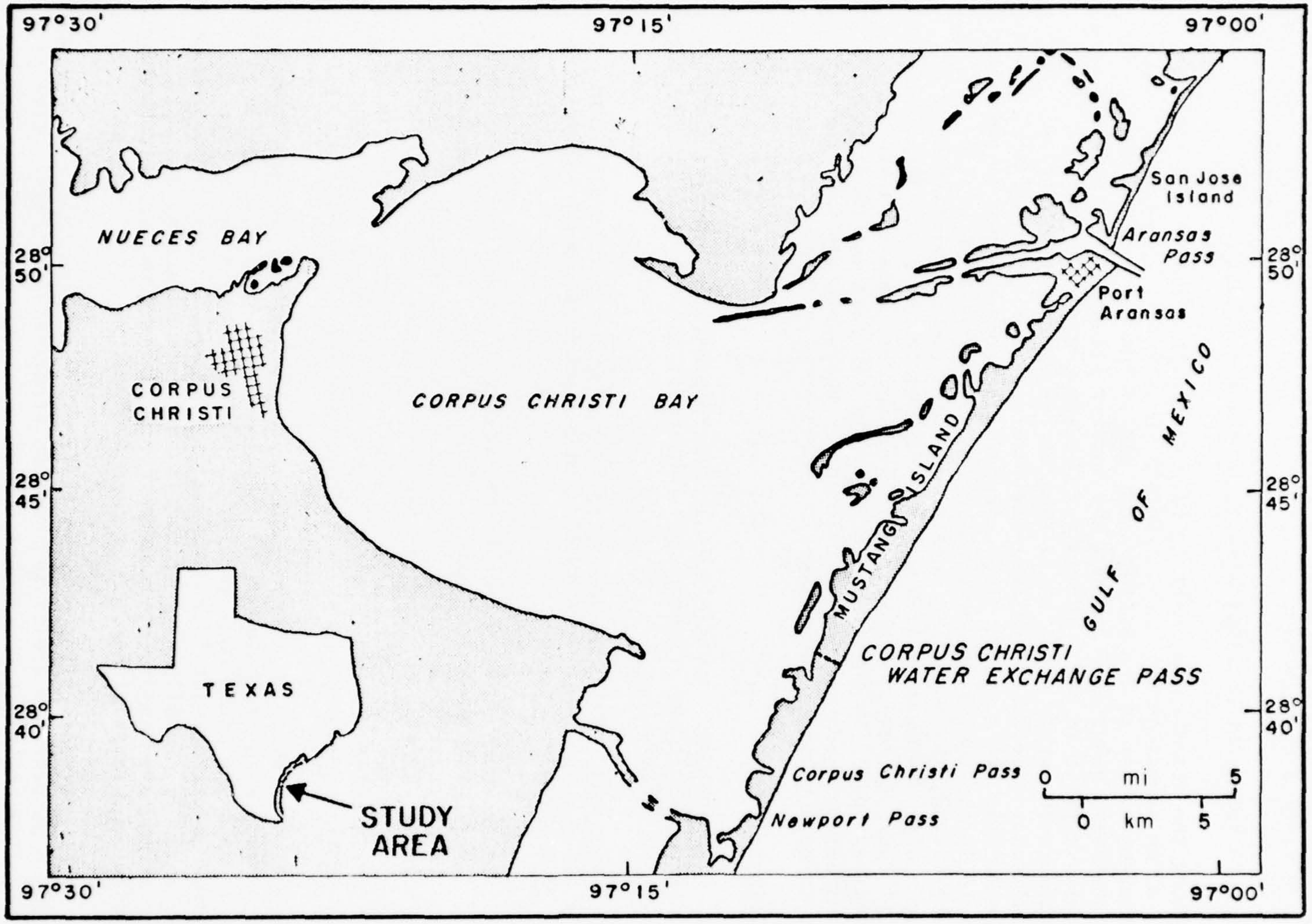


Figure 1. Location map of study area.

berm, and backshore; a generally well-vegetated dune ridge with elevations up to 30 feet; a barrier flat up to 2 miles wide with scattered, vegetated cross-island dune fields (occasionally active during drought periods); and a bay-shore subtidal flat with about seven sandbars.

Along the Texas coast, barrier islands typically cross the entire mouth of an estuarine bay such as Corpus Christi Bay. A natural tidal inlet often exists at the southernmost end of the bay and extends to the gulf southward diagonally between the barrier islands. These inlets typically migrate in the direction of the net longshore transport (southwestward in the study area) until they become excessively long and hydraulically inefficient. Then, the inlets either close or a new channel is opened near the original site, providing a shorter route to the gulf. There is little dune ridge development along the southern 6 miles of Mustang Island, the migration zone of the old Corpus Christi Pass. In the southern 4 miles of this zone, the shallow remnants of the old Corpus Christi Pass (Packery, Newport, and Corpus Christi channels) are irregularly opened by hurricane surges. The new pass was constructed at the northernmost edge of this 6-mile zone. Beach erosion near the pass has been negligible in recent years (Brown, et al., 1974).

2. Climate.

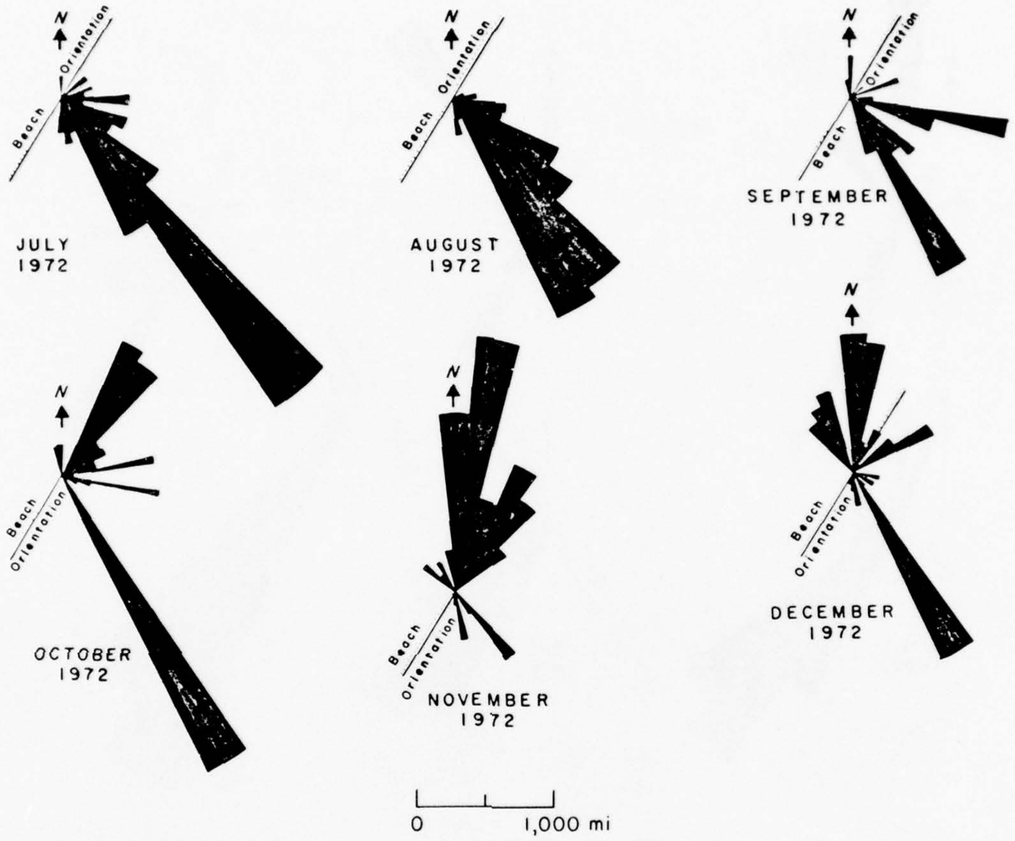
a. Temperature. Average monthly air temperatures range from 87° Fahrenheit in July and August to 59° Fahrenheit in January. Freezes occur during some winters.

b. Rainfall. Rainfall is extremely variable. Recorded 12-month totals range from less than 6 to 58 inches. Average annual rainfall is about 27 inches.

> "Four modes are apparent in the yearly distribution of rainfall. These are designated: Drought years, below 19 inches (20 percent); dry years, 19 to 24 inches (28 percent); normal years, 24 to 32 inches (35 percent); and wet years, above 32 inches (17 percent). The climate is characteristically dry with brief periods of heavy rain which may occur in any month but most commonly fall in May and September" (Behrens, 1966).

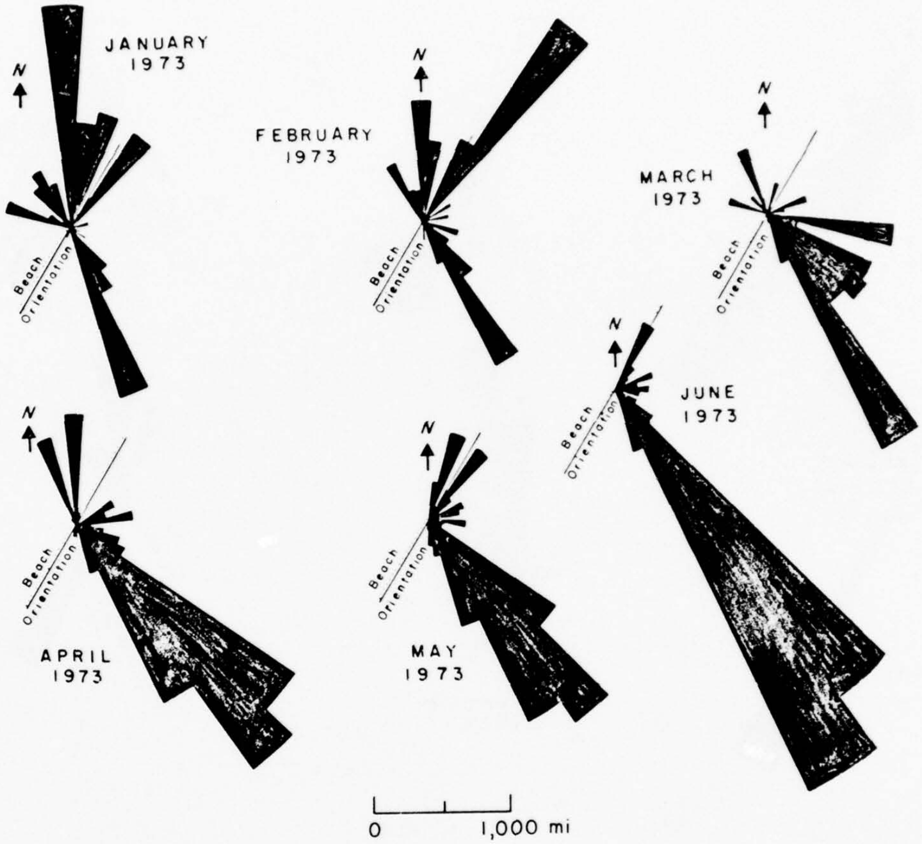
c. Winds. Winds in the area are strongly bimodal. The northern hemisphere Trade Wind System produces a southeasterly mode which has a strong onshore component and a moderate longshore component to the northeast. This wind is predominant from March or April through August or September (Figs. 2 and 3), usually with greater intensities in spring and fall. During midsummer, onshore winds often become lighter and are modified by a diurnal sea breeze effect.

A north-northeasterly mode results from the anticyclonic circulation of cold, high-pressure Arctic and Pacific airmasses (northers) which predominate from September or October through February or March. These winds have strong to weak offshore components and strong to moderate longshore



Miles per Month = ($\sum_{n=1}^{m}$ resultant daily windspeed, miles per hour) x 24 hours, where m = number of days in month

Figure 2. Monthly wind direction and intensity, Corpus Christi, Texas, July to December 1972.



Miles per Month = $\left(\sum_{n=1}^{m} \text{resultant daily windspeed, miles per hour} \right) \times 24$ hours, where m = number of days in month.

Figure 3. Monthly wind direction and intensity, Corpus Christi, Texas, January to June 1973.

components from the northeast. The high-pressure cells are preceded by low-pressure troughs or fronts which draw air from both directions. Thus, as a norther approaches the coast, strong onshore winds blowing into the trough build up until the front passes and are rapidly replaced by equally strong or stronger offshore winds.

Hurricanes strongly affect the Texas coast; a total of 27 hurricanes have crossed the coastline from 1900 to 1975. Table 1 shows the frequency of occurrence of hurricane surge heights on Mustang Island predicted by Bodine (1969) and Brown, et al. (1974).

Table 1. Predicted frequency of occurrence of hurricane surge heights.

Frequency of occurrence (yr)	Surge height (ft)
5	4
10	6
25	8
50	9
100	10

3. Hydrography.

a. Waves. Since waves are almost completely locally wind-generated, the longshore component of wave energy correlates well with the longshore component of wind (Watson and Behrens, 1970). A wave climatology based on visual observations during the study period was developed, and results are presented in Section IV.

b. Salinities. The salinity of gulf waters generally ranges between 30 and 35 parts per thousand ($^{o}/oo$). The bay water salinity range is much greater and highly dependent upon local weather conditions, with hypersaline water (35 to 40 $^{o}/oo$) occurring during some drought periods and almost freshwater (less than 10 $^{o}/oo$) following periods of high rainfall. Localized large salinity differences would affect the vertical velocity distribution, increasing flood velocities at the bottom, and therefore movement of bottom sediment. Salinities in the pass were not recorded regularly, but probably varied between 20 and 35 $^{o}/oo$ during the study period.

c. Water Temperature. Average monthly water temperatures ranged from 81° Fahrenheit in summer to 55° Fahrenheit in winter with daily extremes between 88° and 40° Fahrenheit.

d. Tides. Tidal fluctuations and currents are discussed in detail in Section VI.

III. PASS DESIGN AND CONSTRUCTION

Corpus Christi Water Exchange Pass was designed for the Texas Department of Parks and Wildlife by Turner, Collie, and Braden of Houston, Texas, and constructed by Brown and Root of Houston,Texas. The pass location at the southwest corner of Corpus Christi Bay was first proposed by Carothers and Innis (1960) to use the natural scouring capability of winter storms, as documented by Price (1951). The Department of Parks and Wildlife property boundaries necessitated a bend of 23° in the channel about 2,000 feet from the gulf (Fig. 4). Although the purpose of the pass was environmental, initial consideration of small-craft navigation requirements led to the following design channel dimensions: From the bay mouth to the landward ends of the jetties (10,000 feet), a bottom width of 60 feet, a top width of 120 feet, and a depth of 8 feet; between the jetties, which were 400 feet apart and extended 870 feet into the gulf, a bottom width of 100 feet, a top width of 150 feet, and a depth of 11 feet. Although the pass was intended to enhance flushing of the bay, the width of the pass was required to be a minimum to prevent tidal discharge from affecting the stability of Aransas Pass (Turner, Collie, and Braden, 1967). Thus, a basic conflict in design rationale existed. The inlet was constricted at the highway bridge where the top width was reduced to 100 feet. The jetties were completed by the summer of 1971, dredging began in October 1971, and breakthrough occurred in August 1972, after a postponement for bridge construction. Total cost of the project was approximately $3 million.

IV. WAVE OBSERVATIONS AND LONGSHORE SEDIMENT TRANSPORT RATES

1. Waves.

Daily visual wave observations were made on Mustang Island, Texas, between July 1972 and June 1973, at Port Aransas, Texas, about 10 miles north of the pass and 1 mile south of the Aransas Pass south jetty. Breaker height, period, and direction were recorded on Littoral Environment Observations (LEO) forms (Station 51600). Bruno and Hiipakka (1973) provides a description of the LEO program.

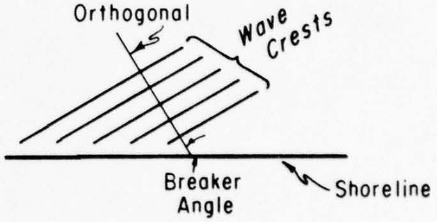
Monthly means and standard deviations of the wave data (Table 2) indicate that the average direction of wave approach is from the southeast between June and August, and from the east-southeast during other months. Mean wave periods of less than 6 seconds occurred between July and September, with maximum mean periods of about 7 seconds between December and March. Wave heights during the study period averaged 2.6 feet, with minimum heights associated with milder summer winds. However, anomalously large breakers which may have resulted from distant tropical storms were observed in July. The yearly cumulative frequency distribution of wave height and period is summarized in Figures 5 and 6.

Figure 4. Corpus Christi Water Exchange Pass, July 1973.

Table 2. Summary of visual wave observations, Port Aransas, Texas, July 1972 to June 1973.

Month	Observations (No.)	Breaker height (ft)		Breaker period (s)		Breaker angle[1] (degrees)	
		Mean	Standard deviation	Mean	Standard deviation	Mean	Standard deviation
1972							
July	16	3.1	0.8	6.0	1.2	95.4	15.6
Aug.	30	2.1	1.4	5.4	1.2	102.6	7.2
Sept.	28	2.4	0.9	5.0	0.6	86.2	18.0
Oct.	27	2.6	1.4	5.9	1.0	89.0	11.5
Nov.	18	2.8	1.3	6.8	1.3	84.1	4.6
Dec.	27	2.8	1.5	7.4	1.0	85.0	6.9
1973							
Jan.	28	3.0	1.5	6.9	0.7	86.4	9.7
Feb.	27	2.5	1.1	7.3	0.9	85.0	11.4
Mar.	28	2.6	0.7	7.0	0.7	85.1	9.5
Apr.	28	2.7	1.5	6.7	1.2	83.8	9.5
May	26	2.5	1.5	6.5	0.7	87.9	9.1
June	18	2.2	0.8	6.4	0.7	91.4	12.0
Total	301						
Yearly Weighted Means		2.6	1.2	6.4	1.2	88.4	12.0

[1] Angle of orthogonal to wave crest relative to shoreline.



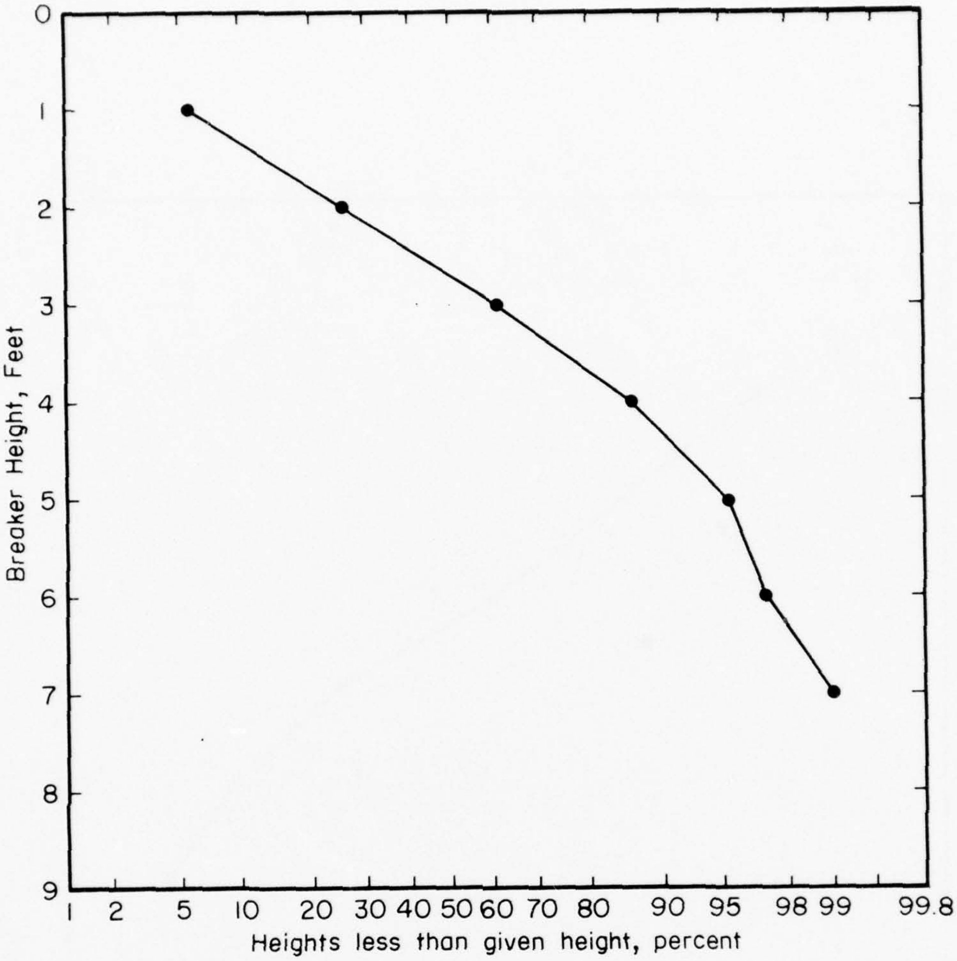


Figure 5. Cumulative frequency of occurrence of breaker heights, from 302 observations, July 1972 to June 1973.

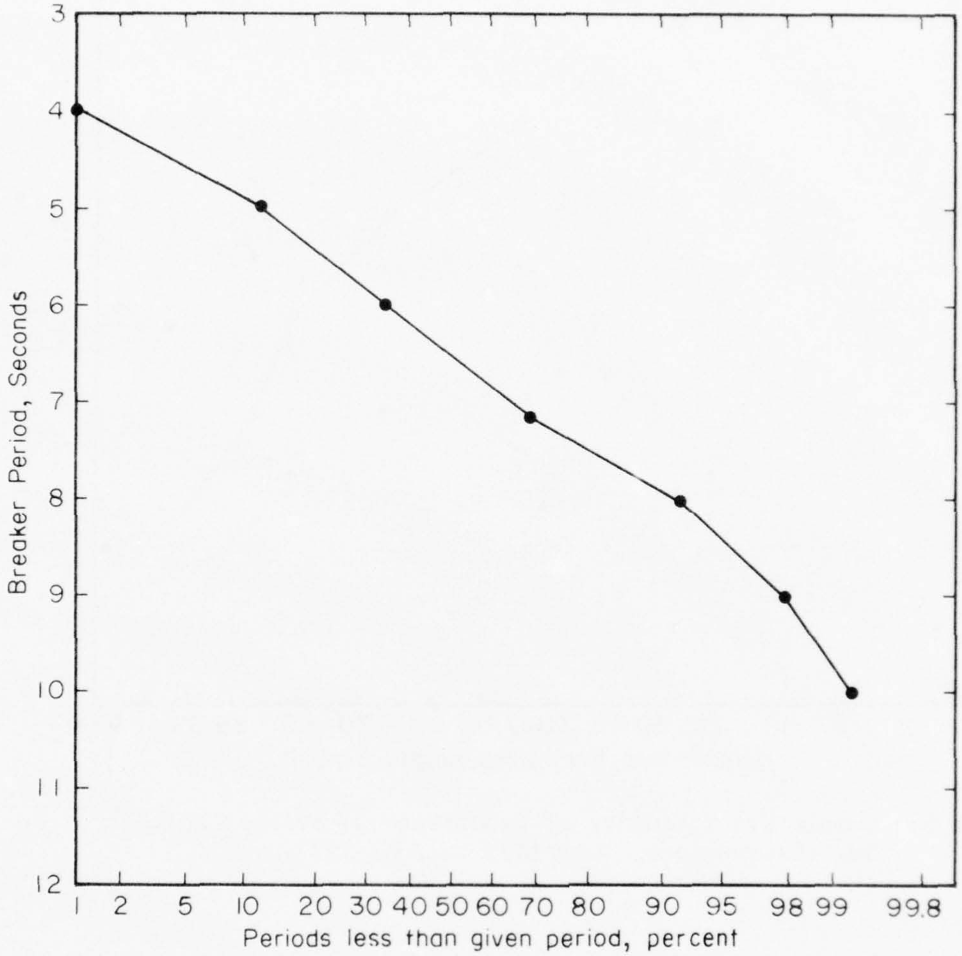


Figure 6. Cumulative frequency of occurrence of breaker periods, from 302 observations, July 1972 to June 1973.

2. Longshore Transport Rates.

Monthly longshore sediment transport rates were calculated using methods outlined by Das (1972) and by U.S. Army, Corps of Engineers, Coastal Engineering Research Center (1975):

$$P_{\ell s} = (\rho g/16)\, H_b^2\, (g d_b)^{1/2} \sin 2\alpha_b, \qquad (1)$$

where

$P_{\ell s}$ = longshore component of wave energy flux (foot-pounds per second per foot of beach)

ρ = density of seawater (two slugs per cubic foot)

g = gravitational acceleration (32.2 feet per second squared)

H_b = breaker height (feet)

d_b = water depth at breakpoint (feet)

α_b = angle of breaker incidence with the beach.

The water depth at breaking depends on the beach slope and incident wave steepness, but may be taken as approximately 1.3 times the wave height (Munk, 1949), so the equation for the longshore component of wave energy flux can be reduced to:

$$P_{\ell s} = 25.8\, H^{5/2} \sin 2\alpha_b \text{ (foot-pounds per second per foot of beach)}. \qquad (2)$$

The "immersed" weight transport rate, I_ℓ (pounds per second), is related empirically to $P_{\ell s}$ by:

$$I_\ell = k\, P_{\ell s} \text{ (pounds per second)}; \qquad (3)$$

in this study a k of 0.35 was used, as recommended by Das (1972). However, more recent data indicate that the transport rate may be over twice that predicted in this study (Coastal Engineering Research Center, 1974). The daily "volume" rate of transport, Q (cubic feet per day), is:

$$Q = I_{day}/[(\rho_s - \rho)\,(g)\,(1-p)], \qquad (4)$$

where ρ_s is sediment density (5.14 slugs per cubic foot), p is porosity of sand in place (taken to be 0.4), and I is (I_ℓ) (24 hours) (3,600 seconds per hour).

Monthly summaries of the gross and net longshore transport between July 1972 and June 1973 are given in Table 3. Daily values during each month were totaled and extrapolated to representative months of 30.4 days. Close correlation existed between the predicted longshore transport direction and the longshore component of wind (Fig. 7). During the summer, prevailing southeasterly winds produced waves which moved material northward along the

Table 3. Monthly longshore transport at Port Aransas, Texas.

Month	Longshore transport (yd^3)			
	Northward	Southward	Net	Gross
1972				
July	76,513	41,811	34,702 northward	118,324
Aug.	72,845	0	72,845 northward	72,845
Sept.	25,102	60,480	35,378 southward	85,583
Oct.	21,317	33,937	12,620 southward	55,255
Nov.	1,536	52,851	51,315 southward	54,388
Dec.	28,958	49,547	20,589 southward	78,506
1973				
Jan.	37,602	62,338	24,736 southward	99,940
Feb.	19,545	63,801	44,256 southward	83,347
Mar.	14,925	43,141	27,216 southward	57,066
Apr.	53,124	53,134	10 southward	106,259
May	40,980	21,166	19,814 northward	62,146
June	30,384	21,024	9,360 northward	51,409
Total	422,831	502,230	79,399 (southward)	925,061

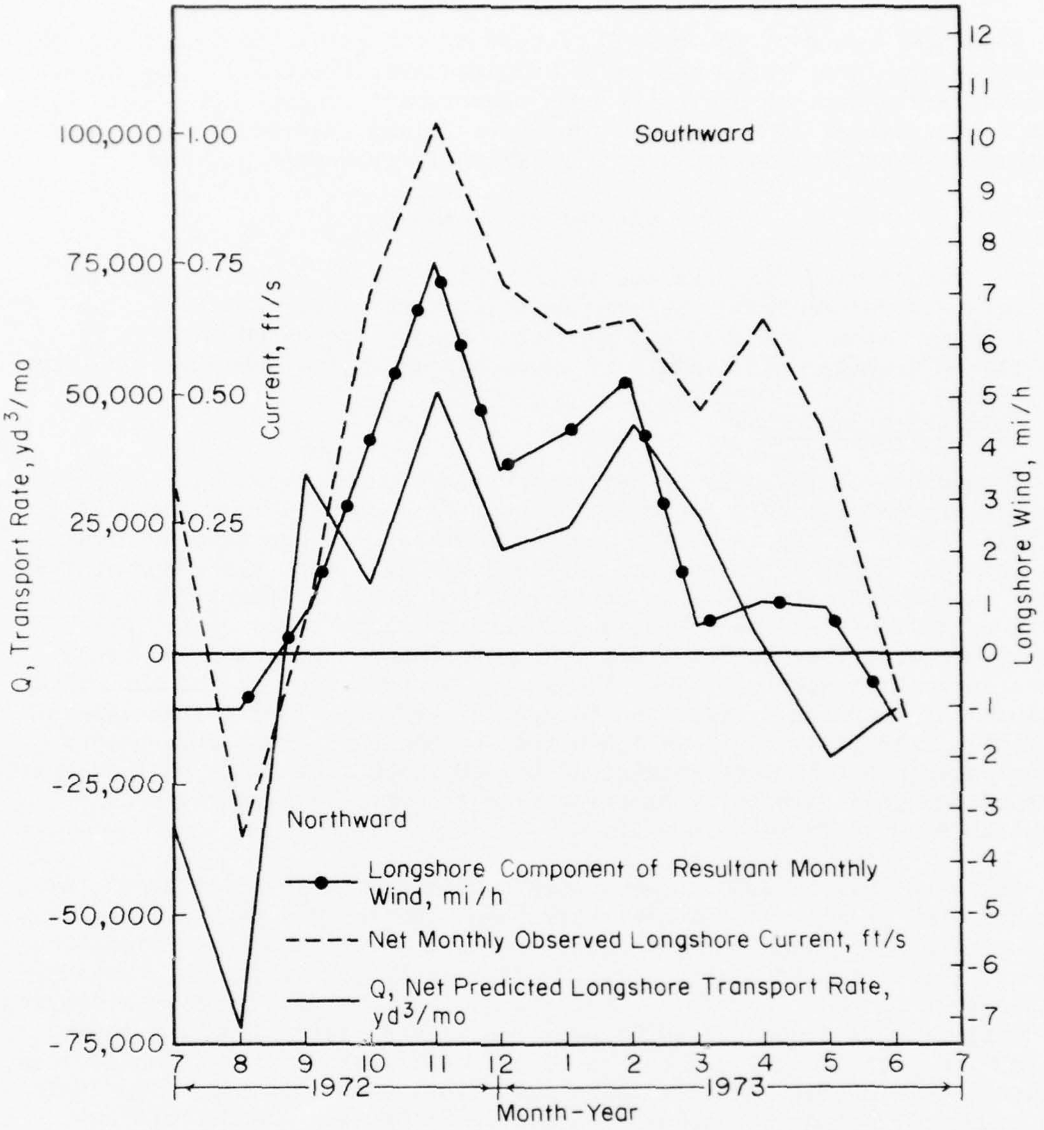


Figure 7. Monthly variation in longshore wind, current, and sediment transport rate.

coast. Strong north and northeast winds, which accompanied high-pressure cold fronts during the winter, resulted in net southward transport. This also generally holds true for longshore current direction (Fig. 7).

Since the net rate was only 8 percent of the gross, the northerly and southerly transport rates were well balanced over the year. This situation results from the pass' proximity to a convergence zone of zero "net" transport located about 35 miles south on Padre Island (Watson, 1971). Average monthly wave conditions may vary significantly from year to year.

V. BATHYMETRIC CHANGES

The response of the pass and adjacent beaches to waves, tides, and currents was documented by several detailed bathymetric surveys. The measurement techniques used and results obtained are based on four geographic areas: Gulf beach surf zone, bay mouth, gulf mouth, and channel.

1. Gulf Beach Surf Zone.

The effect of the pass on adjacent beaches and hydrographic changes in the vicinity of the gulf mouth of the pass were defined from the beach profiles shown in Figure 8; the measured data are in Appendix A, Figures A-2 to A-15. Most profiles were obtained at least four times during the study period and extended from about +14 feet mean sea level (MSL) to a depth of at least -14 feet MSL, a distance of about 2,200 feet. All elevations were tied to Texas State Highway Department temporary bench marks located at each profile. The survey accuracy (using the surf sled measurement techniques described in App. B) was ±0.1 foot vertically and ±5 feet horizontally. Within 1,300 feet of the base line, data points were spaced about 35 feet apart, and beyond 1,300 feet about 65 feet apart. Maps of the gulf bathymetry prepared from the profiles are given in Appendix A, Figures A-16 to A-21.

Beach changes between surveys were determined by subtracting the area between each profile and an arbitrary base line for the first survey from the corresponding area for each later survey. Since the beach profiling was not initiated until about 1 year after jetty construction, a standard preconstruction beach profile (App. A, Fig. A-1) was developed by averaging the October 1972 4000N and 4000S profiles with a preconstruction channel centerline profile obtained by the Texas Department of Parks and Wildlife. Although this average profile is not as accurate as those measured during the study, it permits reasonable estimates of erosion and deposition resulting from jetty construction. A summary of beach profile changes from preconstruction to June 1973 is given in Table 4. These data were plotted against distance north and south of each of the jetties and integrated to obtain the total volume changes between surveys (Figs. 9 to 12).

Erosion and deposition values represent the net change over both the subaerial and subaqueous parts of the profiles; definition of onshore and offshore sand movement would require further analysis of profile segments.

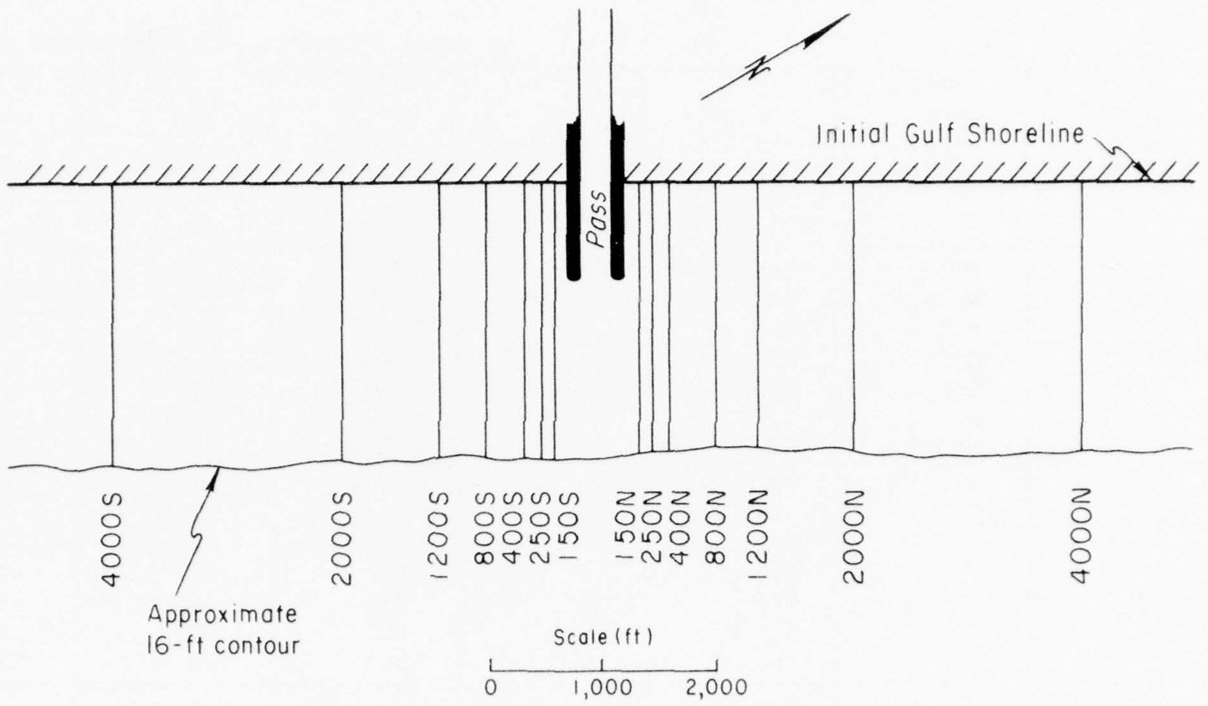


Figure 8. Beach profile locations.

Table 4. Erosion and deposition per linear foot of gulf beach (cubic feet).

Profile	0[1] to Oct. 1972	Oct. 1972 to Jan. 1973	Jan. 1973 to Apr. 1973	Apr. 1973 to June 1973	Oct. 1972 to June 1973	0[1] to June 1973
4000N	60	175	-565	290	-100	-40
2000N	-605	550	885	-1,315	120	-485
1200N	295	195	210	-346	65	360
800N	1,235	670	-660	-360	-350	885
400N	2,835	-315	-1,200	-195	-1,710	1,125
250N	2,420	--	-730	670	15	2,435
150N	2,767	1,020	-2,755	--	--	--
150S	2,190	655	-470	--	--	--
250S	2,285	695	-955	250	10	2,275
400S	1,610	1,070	-1,045	270	295	1,905
800S	1,150	1,025	-135	-250	-640	1,790
1200S	450	620	-140	30	510	960
2000S	85	-450	1,030	-1,280	-700	-615
4000S	-225	-950	-325	-2,695	-3,320	-3,545
			Longshore transport (ft^3 X 10^6)			
Northward	11.4	1.4	2.1	3.2	6.7	18.1
Southward	13.6	3.7	4.4	2.7	10.8	24.4
Net	2.2S	2.3S	2.3S	0.5N	4.1S	6.3S
Gross	25.0	5.1	6.5	5.9	17.5	42.5

[1] From composite preconstruction profile.

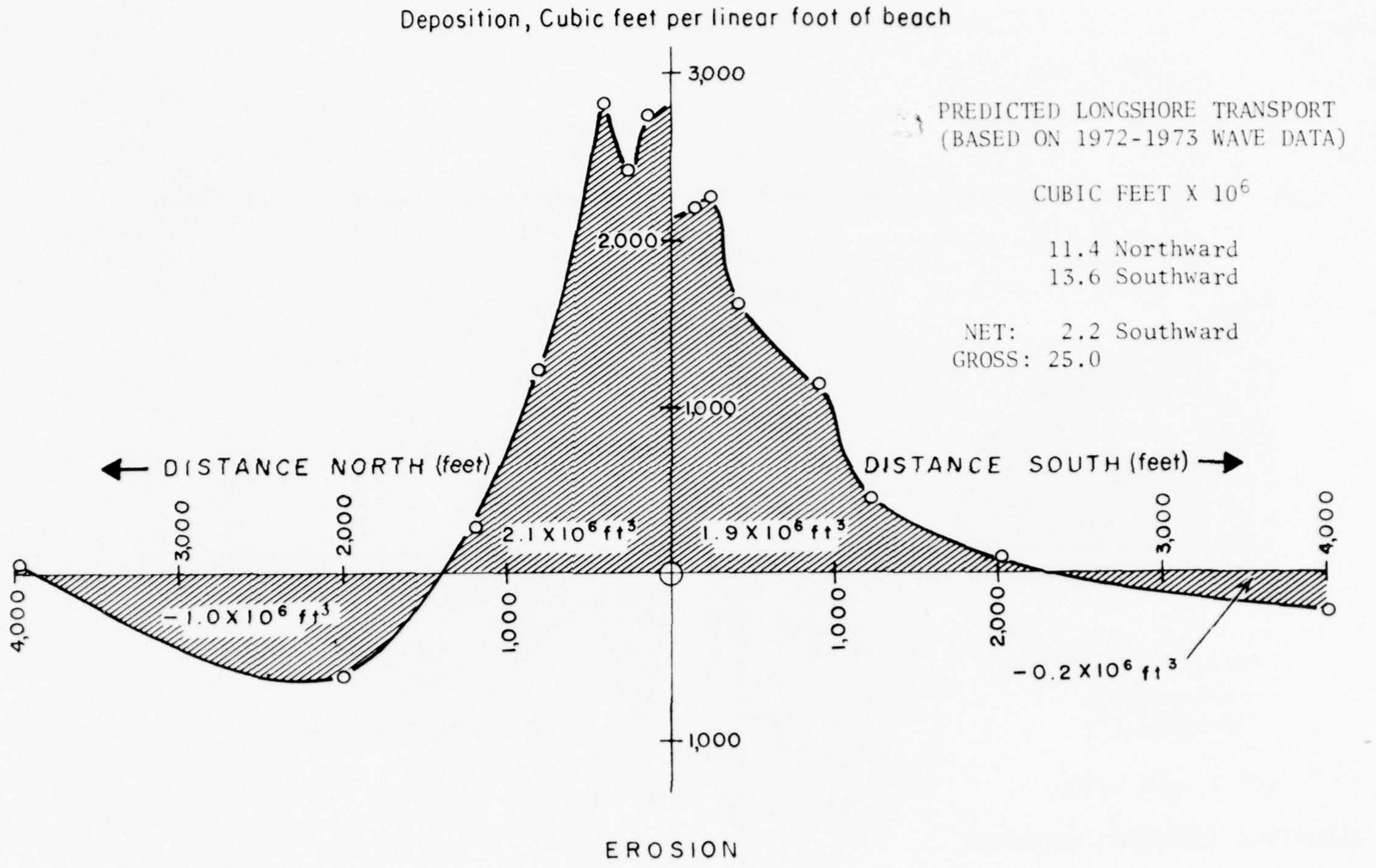


Figure 9. Gulf beach erosion-deposition, preconstruction to October 1972.

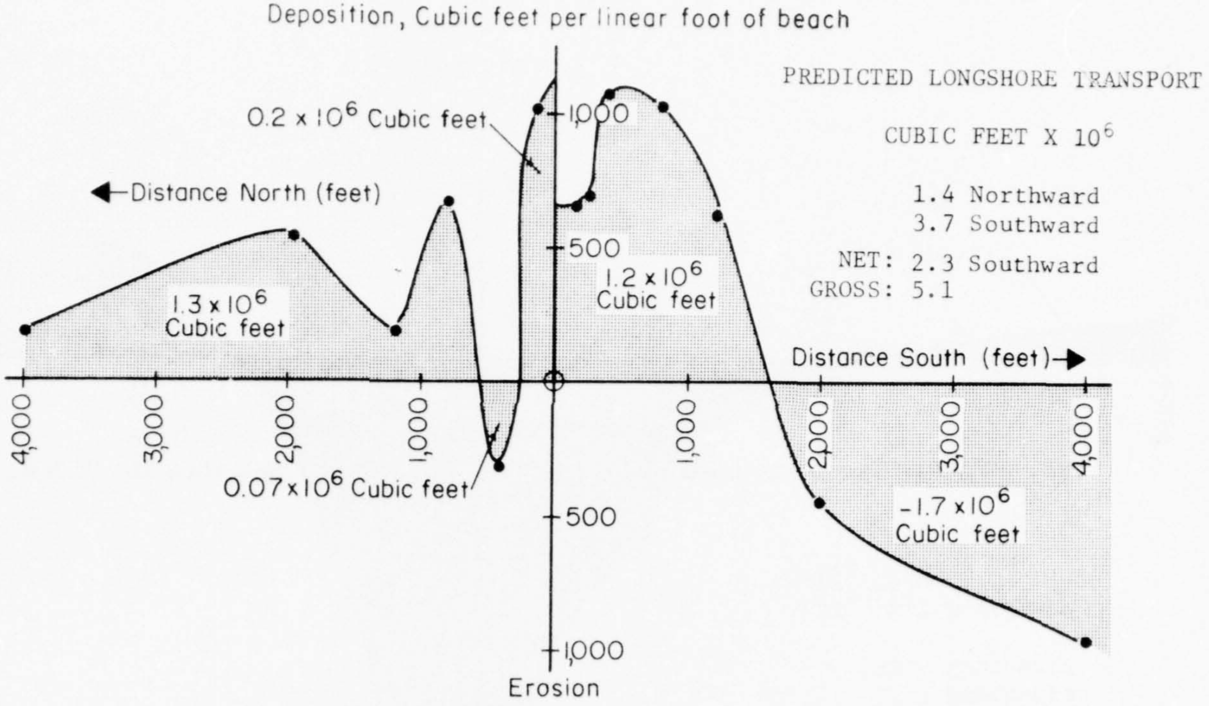
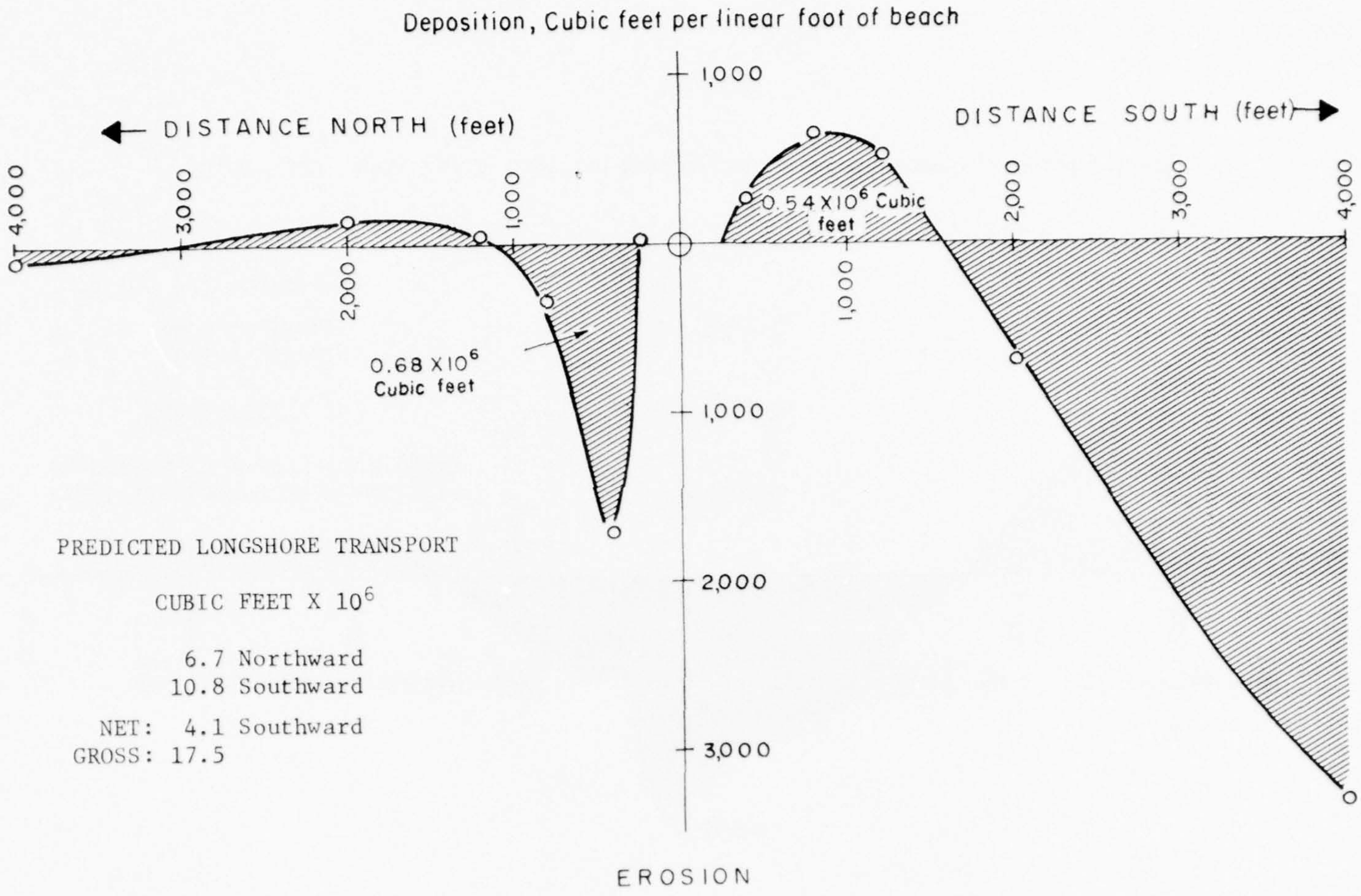


Figure 10. Gulf beach erosion-deposition, October 1972 to December 1972-January 1973.



PREDICTED LONGSHORE TRANSPORT

CUBIC FEET X 10^6

6.7 Northward
10.8 Southward

NET: 4.1 Southward
GROSS: 17.5

Figure 11. Gulf beach erosion-deposition, October 1972 to June 1973.

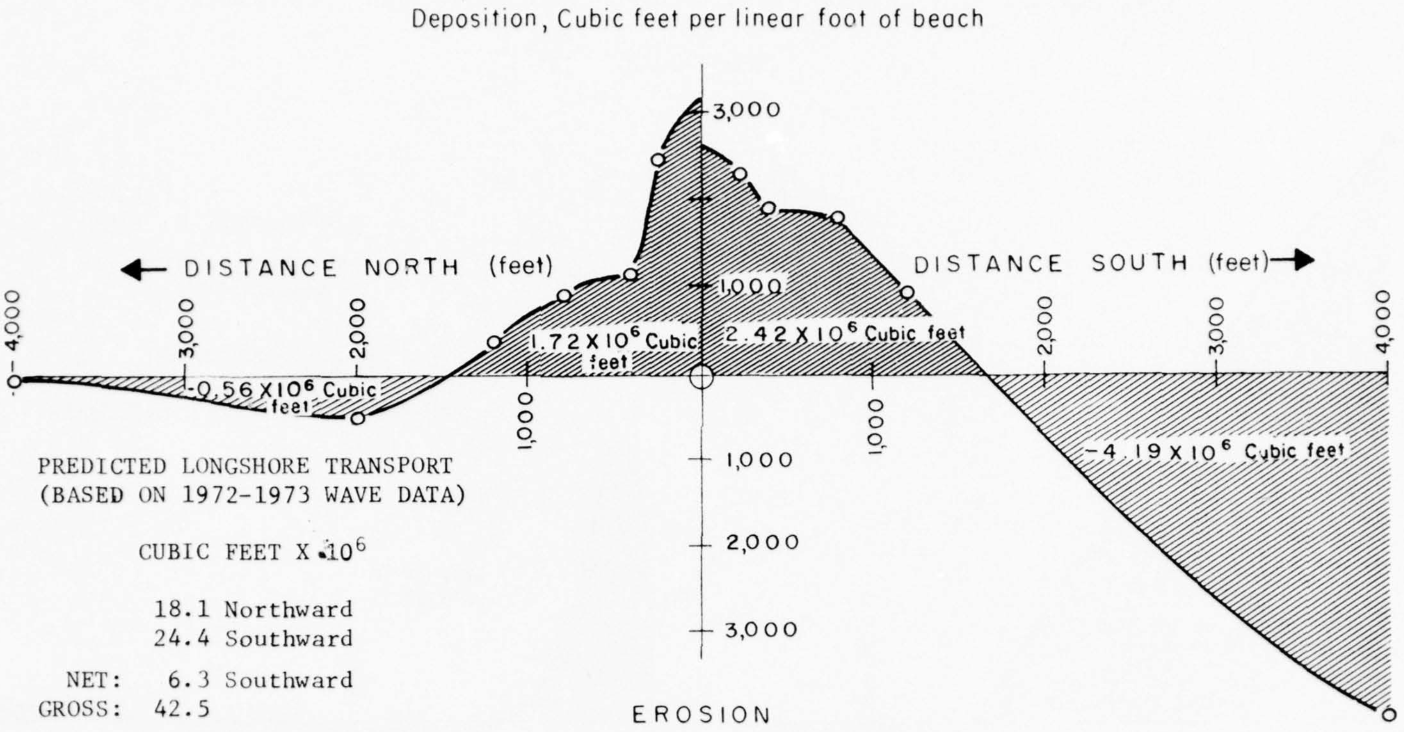


Figure 12. Gulf beach erosion-deposition, preconstruction to June 1973.

The following paragraphs summarize surf zone sand volume changes during the study period.

a. Preconstruction to October 1972 (Fig. 9). About 74,000 cubic yards of sediment was deposited within about 2,000 feet north and south of the jetties before the pass was opened. Within 4,000 feet of the pass, a net of about 41,000 cubic yards was deposited on the north side and 63,000 cubic yards was deposited on the south side.

b. October 1972 to December 1972 (North Side) and to January 1973 (South Side) (Fig. 10). During this period, there was a net deposition of 52,000 cubic yards on the north beach and a net erosion of 18,500 cubic yards on the south beach. The deposition at profile 150N resulted from growth of the seawardmost bar and associated deposition of almost 3 feet of sand over a large part of the profile. The slight erosion at profile 400N may have resulted from adjustment of dredged material released near this profile during dredging of the gulf mouth. During November the longshore current was usually to the south, and had the highest average monthly speed (over 1 foot per second). Increased deposition within 1,500 feet of the south jetty was caused by growth of the second and third offshore bars. Southerly waves and possible refraction and diffraction of northerly waves around the gulf mouth probably produced localized transport northward toward the jetty. Beyond 1,700 feet south of the pass, well out of the wave shadow zone, the loss of sand disturbed the balance between wave forces and longshore transport capacities, resulting in erosion.

c. October 1972 to June 1973 (Fig. 11). The dominant feature of surf zone volume changes during this period was extensive erosion (128,500 cubic yards) beyond profile 1500S. A general deepening of the 4000S profile indicated erosion throughout the offshore area. Erosion of the second and third bars at profile 400N, which increased depths as much as 4 feet, was the only significant change on the north side. Most of the winter deposition was removed during the spring months of predominantly northerly transport.

d. Summary (Preconstruction to June 1973) (Fig. 12). Since construction, the jetties were partial barriers to the longshore sediment transport, trapping material within 1,500 feet north and south of the pass in the first year. Between October 1972 and June 1973 the volume change within 1,500 feet of the jetties was negligible, the updrift beach beyond profile 1500N was stable, and the beaches south of profile 1500S eroded. The prevailing northward transport during the following summer would have probably replaced much of the sand lost on the southern beaches during the winter. Lack of significant accretion on the beaches just north of the pass during the winter months of southward sediment transport, and on the south beaches during summer when northward transport predominates, indicates that sediment bypassing was well established during the study period.

An additional consideration useful in defining the characteristics of beaches is the depth beyond which there is little net erosion or deposition. Although all beach profiles showed some changes at the maximum depth measured (16 feet), there was a characteristic depth at which no change occurred between October 1972 and June 1973, with the exception of two or three of the profile locations. For sites north of the inlet, this depth was 11.8 feet; for sites south it was 11 feet (see App. A, Figs. A-10 to A-13). Although the implications of this observation are significant, there is no explanation of the possible causative processes.

2. Channel.

To document the cross-sectional characteristics of the pass, including shape, area, and longitudinal variation, 22 cross sections were established and surveyed monthly (Fig. 13). A measuring rope (0.25-inch polypropylene) marked at 10-foot intervals was stretched across the channel and secured to pipes on either side. A small boat then drove slowly along the rope while one man sounded with a pole at 10-foot intervals and another man took notes. The profiles closest to the gulf (X21 and X22) were done less frequently because surf conditions between the jetties made boating hazardous. Cross-section X1 was discontinued after November 1972 because the bay shoreline had eroded past this point.

Profile data were automatically plotted for each cross section, and the cross-sectional area, wetted perimeter, and hydraulic radius were calculated. Summaries of the measured geometric parameters are given in Tables 5 and 6.

Erosion and deposition trends in the channel were obtained by computing changes in the volume of water contained between the bottom, the mean water level (MWL) plane, and the end points of each of the six zones of more or less distinct morphology (Fig. 13).

a. Zone 1 (X1 to X5), Bay End (Fig. 14, a). Bay end cross sections exhibited three forms: (a) An irregular shape from the initial dredging through November 1972; (b) trapezoidal with depths below MWL of about 7 feet, base widths from 92 to 120 feet, and surface widths from 180 to 215 feet from December 1972 to March 1973; and (c) generally U-shaped with maximum depths from 7 to 10 feet from April 1973 to June 1973. Minor deposition occurred during the initial adjustment period. However, with the onset of northers (storms), the volume of water below MWL increased to a maximum in October due to increased tidal discharge and wind-generated wave attack on the baymouth region. During midwinter (November to February), northerly winds were so frequent that regional water levels were generally depressed, tidal discharges reduced, and sediment deposition tended to produce flat-bottomed, trapezoidal cross sections. Yearly minimum cross-sectional areas were found in January at X3. From March to May 1973 increased tidal discharges eroded the channel into U-shaped sections throughout the zone.

b. Zone 2 (X5 to X9), Long Reach (Fig. 14, b). The initial breakthrough of the pass was made about 1 August 1972, but the gulf mouth continued to be dredged deeper and wider through most of August. Tidal flows were initially high through the constricted mouth and slower in the rest of the channel. The decreased currents deposited 17,000 cubic yards in zone 2 during early August. Most of this material was removed by

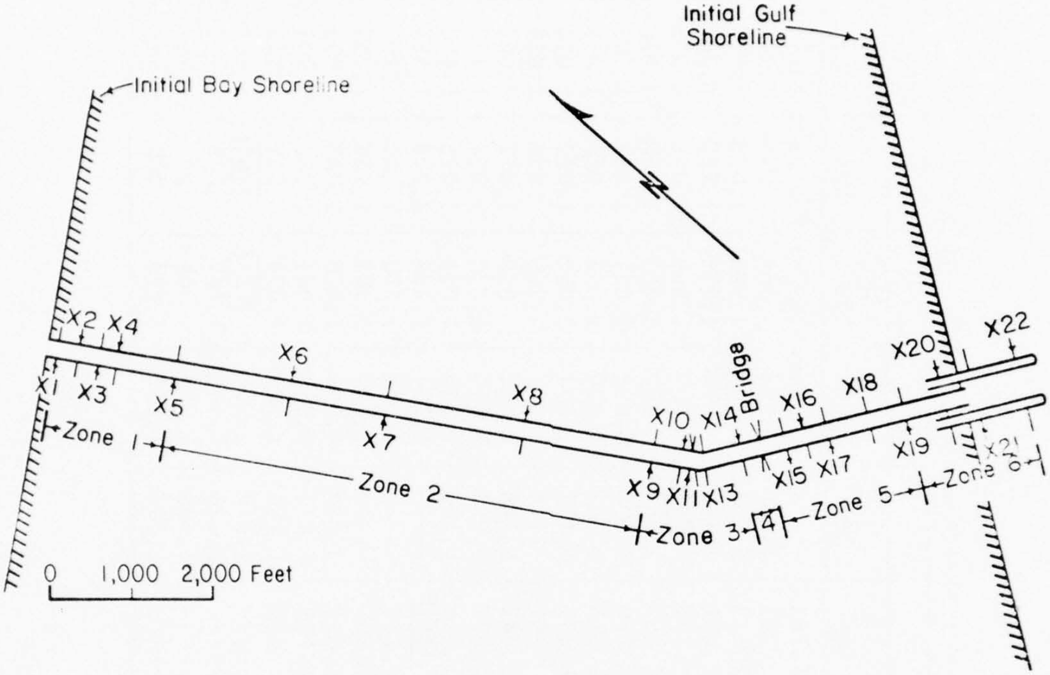


Figure 13. Channel cross-section profile locations.

Table 5. Channel cross-sectional area below +0.8 MSL.

Site	1972						1973					
	22 July	10 Aug.	8 Sept.	13 Oct.	13 Nov.	12 Dec.	15 Jan.	26 Feb.	5 Mar.	11 Apr.	25 May	22 June
X1	904	1,093	1,046	1,261	1,101	- - - bay shoreline eroded past this point - - - - -						
X2	1,264	1,227	1,089	1,387	1,242	1,146	988	064	948	1,102	1,059	1,135
X3	976	1,112	1,036	1,235	1,177	1,061	$\boxed{947}$[1]	1,002	1,018	1,089	1,118	1,193
X4	1,100	1,056	1,037	1,146	1,091	1,112	970	1,036	1,120	1,204	1,179	1,142
X5	893	908	$\boxed{913}$	968	894	$\boxed{991}$	957	1,003	1,046	1,073	1,082	1,078
X6	1,049	1,038	1,011	1,106	1,067	1,064	997	1,011	1,080	1,096	1,089	1,086
X7	1,228	1,100	976	1,075	1,080	1,099	1,028	1,021	1,084	1,103	1,077	1,189
X8	1,051	837	960	1,046	1,037	1,061	1,028	1,052	1,056	1,036	1,075	1,030
X9	1,108	829	994	$\boxed{929}$	1,052	1,116	1,074	1,022	1,022	1,014	972	1,121
X10	1,020	811	1,007	954	997	1,106	1,081	956	986	1,052	1,033	1,100
X11	1,142	798	1,000	957	1,042	1,108	1,052	929	1,072	1,062	1,114	1,215
X12	1,021	795	1,052	978	1,027	1,057	1,027	1,007	1,054	1,137	1,146	1,203
X13	966	874	1,047	1,030	1,016	1,106	1,045	1,054	1,076	1,160	1,189	1,307
X14	1,082	1,017	1,085	1,018	$\boxed{881}$	-----	1,374	1,465	2,200	1,351	1,315	-----
Bridge	$\boxed{800}$	$\boxed{589}$	-----	-----	1,182	-----	1,248	1,252	1,219	1,203	-----	-----
X15	-----	1,193	1,142	1,314	1,265	-----	1,440	1,101	1,404	1,155	1,067	945
X16	-----	1,242	1,109	1,045	1,091	1,083	1,094	1,020	1,083	996	1,010	967
X17	-----	1,130	1,055	1,110	1,128	1,071	1,068	1,041	1,002	994	969	930
X18	-----	-----	1,085	1,051	935	1,185	1,140	1,079	981	955	1,042	1,001
X19	-----	-----	1,120	-----	-----	1,019	954	$\boxed{914}$	$\boxed{961}$			$\boxed{818}$
X20	-----	-----	945	-----	-----	1,626	1,404	1,197	1,226	-----	-----	1,191
X21	-----	-----	1,748	-----	-----	1,495	1,340	961	1,044	-----	-----	1,321
X22	-----	-----	-----	-----	-----	1,344	1,372	1,293	1,249	-----	-----	-----

---

[1] Minimum area $\boxed{}$.

Table 6. Channel hydraulic radii in feet.

Site	1972						1973					
	22 July	10 Aug.	8 Sept.	13 Oct.	13 Nov.	20 Dec.	15 Jan.	26 Feb.	5 Apr.	11 May	25 May	22 June
X1	---	6.0	5.2	5.8	4.5	---	---	---	---	---	---	---
X2	---	5.7	4.9	5.9	5.4	5.7	6.2	5.1	4.7	5.6	5.9	4.4
X3	---	6.0	5.5	6.0	5.4	5.6	4.9	5.0	5.0	5.2	5.4	5.6
X4	---	6.0	$\boxed{3.8}$[1]	$\boxed{3.6}$	$\boxed{3.7}$	$\boxed{3.2}$	4.7	4.8	4.1	4.1	5.3	---
X5	$\boxed{4.3}$	5.9	5.7	5.4	5.0	6.4	6.2	6.0	6.0	6.1	6.8	5.9
X6	5.8	6.3	5.7	5.7	5.6	6.1	5.6	5.3	5.5	5.4	5.7	---
X7	6.0	6.3	5.2	5.7	5.7	6.5	6.0	5.7	6.2	6.0	6.1	5.7
X8	6.6	5.8	6.7	6.8	6.8	7.4	7.1	6.9	7.0	6.7	7.3	6.6
X9	6.8	5.4	6.2	5.6	5.8	5.7	5.2	4.6	4.3	4.1	4.0	4.3
X10	6.3	5.4	6.4	5.8	5.6	5.7	5.3	4.4	4.1	4.3	4.3	4.5
X11	6.5	$\boxed{5.0}$	6.0	5.6	5.8	5.7	5.3	4.2	4.5	4.4	4.6	4.9
X12	6.5	5.6	6.9	6.0	5.6	5.5	5.0	4.5	3.3	4.7	4.8	4.7
X13	5.8	6.0	6.7	6.1	5.5	5.5	4.8	4.6	4.3	4.7	4.7	5.0
X14	6.1	6.5	6.3	5.6	4.3	---	5.1	4.8	10.0	5.0	4.7	---
Bridge	---	5.5	---	---	10.8	---	11.1	10.8	10.6	---	---	---
X15	---	6.7	---	5.7	6.9	7.0	6.2	4.3	6.3	4.5	4.5	4.3
X16	---	7.1	6.3	6.0	5.9	5.6	5.3	4.6	4.4	4.0	---	---
X17	---	6.8	6.2	6.2	6.0	5.3	4.9	4.6	4.1	3.9	3.8	3.6
X18	---	---	6.2	5.3	4.3	5.1	4.6	4.4	3.7	3.6	3.9	3.8
X19	---	---	6.3	---	---	5.9	5.3	4.8	4.9	---	---	4.1
X20	---	---	6.2	---	---	4.8	$\boxed{4.0}$	3.7	$\boxed{3.0}$			$\boxed{2.3}$
X21	---	---	5.5	---	---	4.9	4.3	$\boxed{3.2}$	3.4	---	---	3.8
X22	---	---	---	---	---	4.9	4.6	4.7	4.4	---	---	---

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[1] Minimum hydraulic radius $\boxed{}$.

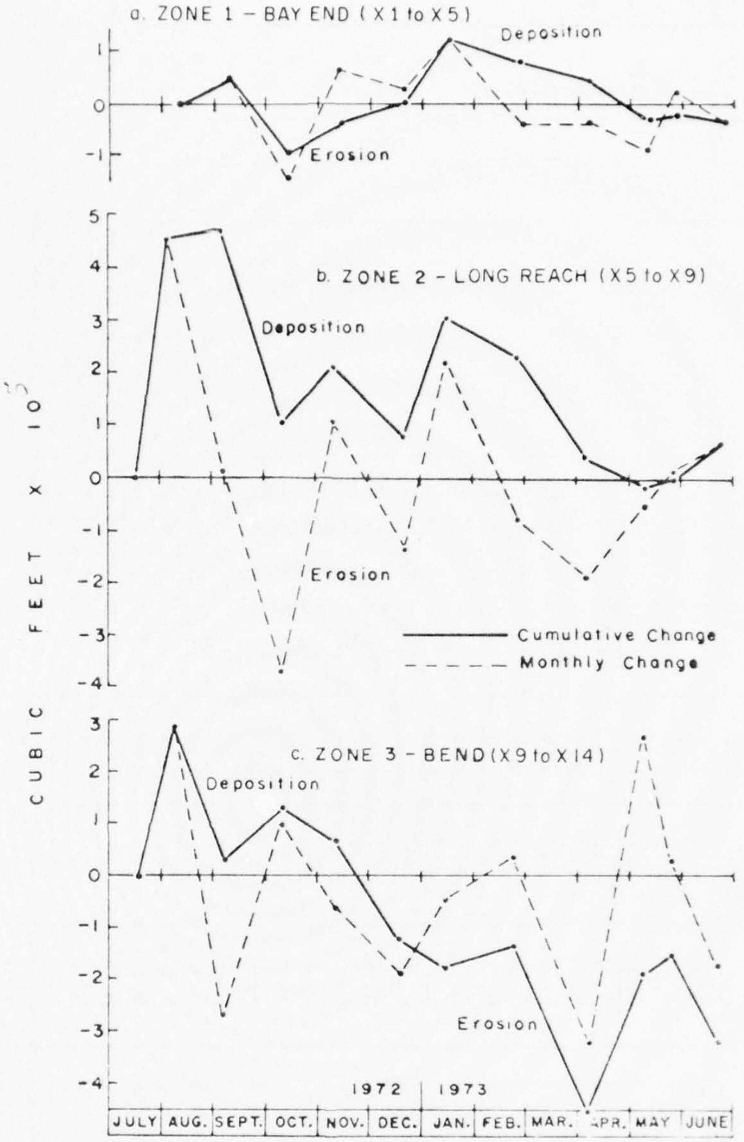


Figure 14. Channel volume changes, zones 1, 2, and 3.

November, and by December the sand remaining from the August filling had been molded to form a flat-bottomed, trapezoidal channel with steep walls in a cohesive shelly unit. Channel depths below MWL were from 6.5 to 8.5 feet, surface widths from 141 to 180 feet, and bottom widths from 105 to 150 feet. Following a short period of deposition in January, zone 2 continued to erode through May 1973, when it had scoured to its original dredged volume.

c. Zone 3 (X9 to X14), Bend (Fig. 14, c). Immediately after the pass opened, zone 3 filled in the same way as the straight reach. However, the new material was removed by November 1972, and the volume of the bend generally increased through June, except for heavy deposition during April. Erosion of the channel bend resulted from both ebb and flood flows. Ebb currents approaching the bend from the bay eroded the outer part of the bend in the vicinity of X13. The horizontal scour rate was about 0.3 foot per day, and a deep channel was formed next to the bank. Flood currents issuing as a jet from the narrow bridge cross section scoured a similar channel along the inside of the bend near X11. Development of these channels produced net erosion of 11,700 cubic yards.

d. Zone 4 (X Bridge) (Fig. 15, a). The channel under the highway bridge was originally dredged to the same depth (-8.0 feet MSL) as the adjoining parts of the channel, but was only 100 feet wide. The original cross-sectional area of about 800 square feet was reduced to 590 square feet by the early August flood sedimentation. Shortly thereafter, erosion started and by November 1972, the cross-sectional area approximated the average for the pass (1,070 square feet). By June, the maximum depth was 20 feet.

e. Zone 5 (X14 to X19), Short Reach (Fig. 15, b). The channel in zone 5 was influenced by the bridge constriction on the bay end and by the channel mouth effects at the gulf end. The deep jet emerging from under the bridge on ebb flows kept X15 relatively deep and narrow; the rest of the reach gradually shoaled to about 4.4 feet and widened about 80 feet.

In spite of the intervening bridge, a meander at the bend (zone 3) continued to develop on the north side of X15 in February 1973. This channel was not seen at X16, but appeared on the south side of the main channel at X17 and X18 from April to June 1973, and reappeared on the north side of the pass in zone 6 between X20 and X22.

Using the distance between bends in this channel as meander lengths, the corresponding meander width according to Zeller (1967) can be calculated from:

$$M_1 = 10\ B^{1.025} \tag{5}$$

and

$$M_\omega = 4.5\ B$$

or

$$M_\omega = 4.5\ (M_1/10)^{0.976} \tag{6}$$

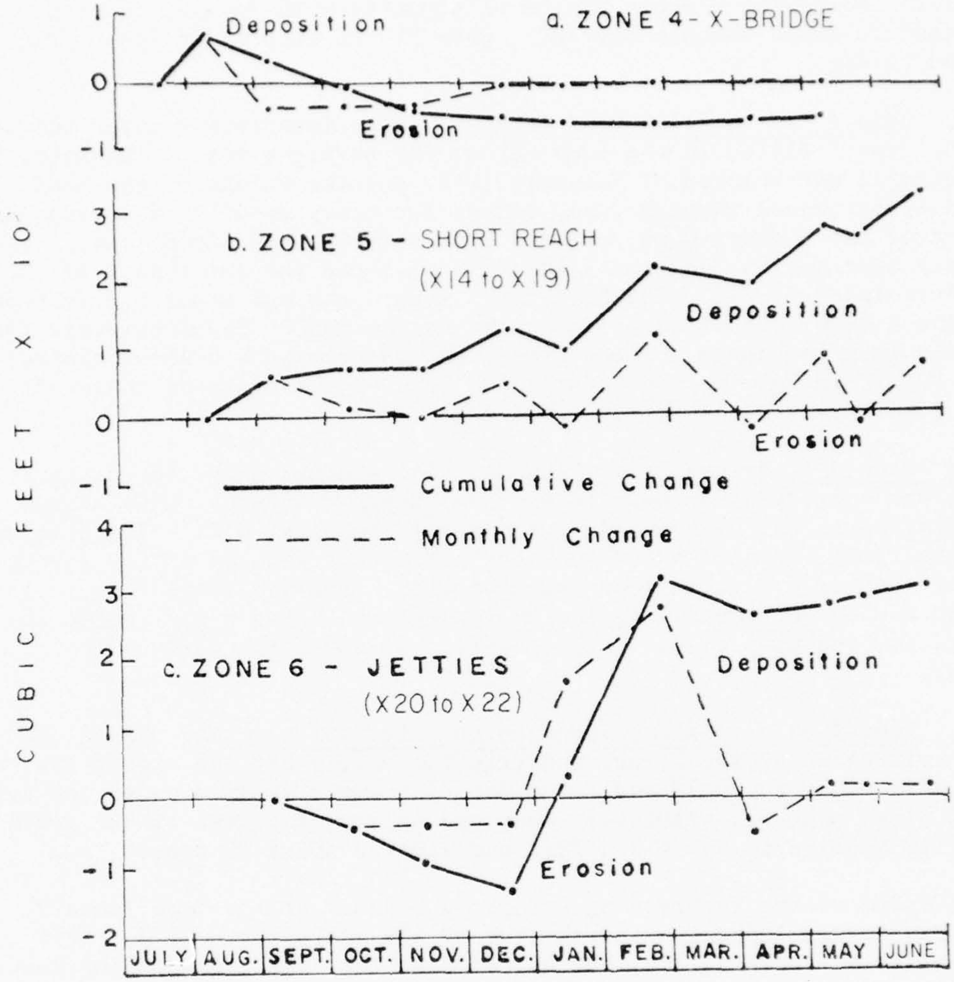


Figure 15. Channel volume changes, zones 4, 5, and 6.

where M_l = meander length 1,500 to 1,800 feet, M_w = meander width, and B = channel width. The actual meander width of the existing channel (200 to 300 feet) is less than one-half the equilibrium width calculated, using equation (6) (600 to 700 feet). The actual channel width (at least 200 feet) would correspond to both greater meander lengths and widths. Therefore, continued development of the meander might be expected.

f. Zone 6 (X20 to X22), Jetties (Fig. 15, c). Although this zone is only 400 feet long, changes to its outer and inner parts differed. The channel between the outer half of the jetties was dredged to almost twice the average channel cross-sectional area. Therefore, ebb velocities in the smaller part of the channel decreased by about one-half on entering the larger part; flood velocities were low in the outer section and increased on entering the smaller section. From August until December 1972, the inner half eroded more than the outer half filled. The net erosion resulted from flood current erosion of the inner part of this zone and, since the channel between the inner ends of the jetties was riprapped, this area was not initially dredged as large as the rest of the channel. From December until about March 1973, rapid deposition of 16,400 cubic yards of sand occurred in zone 6 with minor subsequent changes through June.

While little net scour or fill occurred between the jetties, a 6- to 8-foot-deep channel developed adjacent to the north jetty, and the southern half of the original channel shoaled almost to MSL. The north side channel existed to some extent immediately after the pass was opened and qualitative observations indicate that a channel existed even before the entrance was dredged. Therefore, it appears that before the pass was opened, waves approaching from the east-northeast induced a clockwise circulation of both sediment and water inward along the south jetty and outward along the north jetty. After the pass was opened, this circulation continued during floodflows; during ebb currents, breaking waves produced greater turbulence along the south jetty, decelerating the flow sufficiently to cause deposition. Since current observations were not made when waves approached from other directions, any circulation patterns during these times are unknown. Wave-induced circulation patterns have been observed by Seabergh and Sager (in preparation, 1976) in a model study of single-jettied Masonboro Inlet, North Carolina, and by Sato and Irie (1970) in model studies of breakwaters on open coasts, and may have an important effect on sediment deposition patterns at many improved inlets.

g. Entire Channel (X1 to X22). The volume changes in each zone were summed to determine the total channel response to selected environmental parameters. Monthly variability in these was plotted versus time (Fig. 16), and visual correlation was made between the variability in these parameters and the deposition and erosion history. Although flow velocity and shear stress controlled channel response, long-term velocity data were not available. Therefore, tide and wave data were used in the analysis.

Two tidal characteristics show the strongest qualitative correlation with changes in channel sedimentation—the monthly variability in mean gulf tide

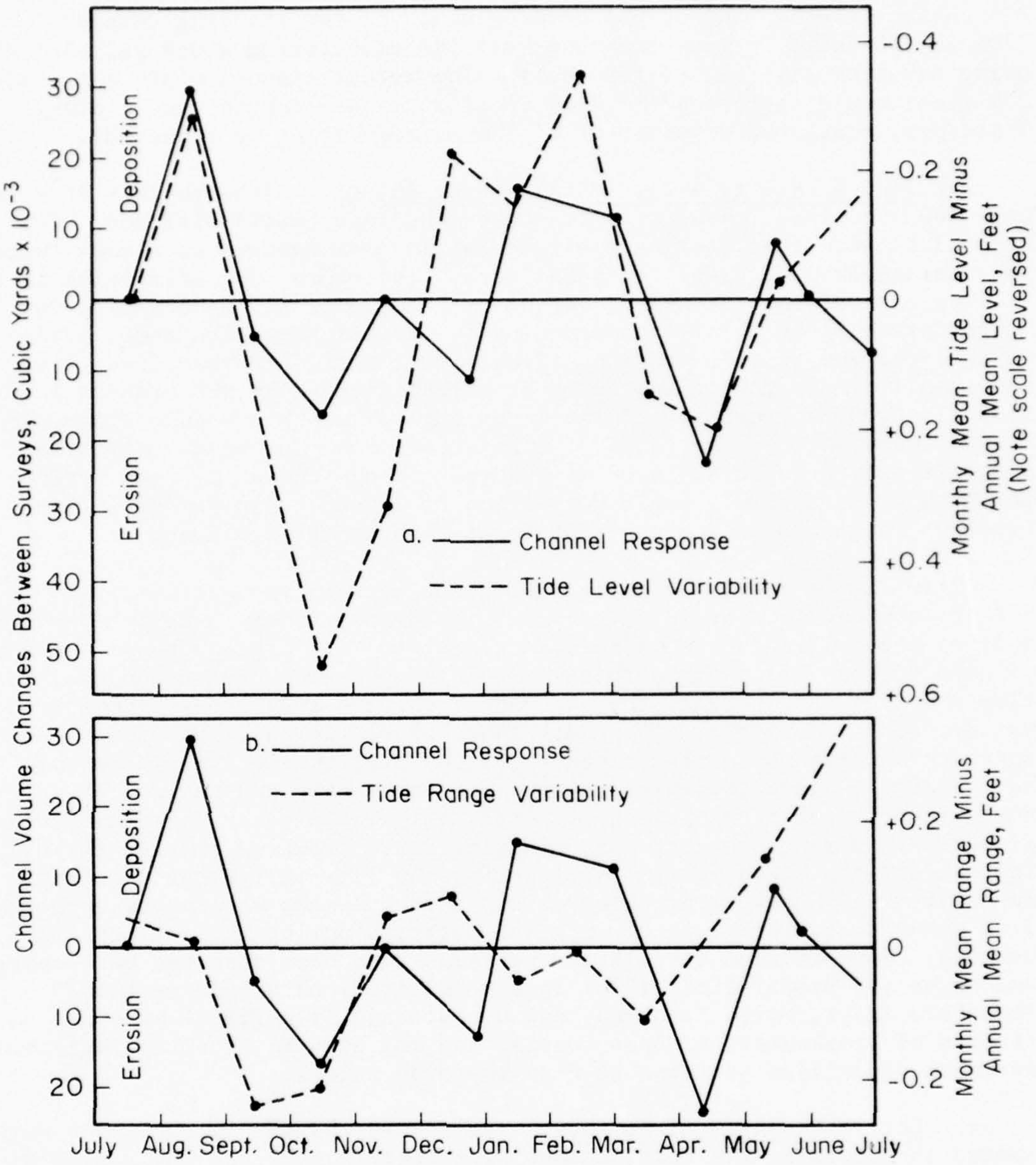


Figure 16. Comparison of monthly channel volume changes and tidal characteristics.

level, and the waxing and waning of seasonal gulf tidal range. It must be stressed that short-term effects such as storms are masked and can produce changes of comparable magnitude.

The monthly variability in mean gulf tide level (the difference between the mean monthly level and the mean annual level) is plotted against monthly channel volume changes in Figure 16(a). The mean monthly level was obtained from data collected by the National Ocean Survey (NOS) tide gage in Aransas Pass and calculated by averaging the daily higher high waters and lower low waters for each month. A strong correlation exists between deposition and low mean tide levels and erosion and high mean levels, except during December and June when erosion (primarily at the channel bend) coincided with low levels. Since flow velocity determines erosion or deposition, Figure 16(a) implies that during periods of high mean levels, flow velocities are high, a conclusion unsubstantiated due to lack of data.

Figure 16(b) graphs monthly variability in the tidal range (the difference between the monthly mean diurnal range and the annual mean diurnal range) and channel volume changes. The diurnal range is the difference between daily higher high waters and lower low waters. There is agreement between deposition and decreasing tidal ranges (e.g., August and September), and erosion and increasing ranges (e.g., October to December). This correlation probably exists because during increasing tidal ranges, discharge through the pass is also increasing; the channel adjusts to maintain equilibrium with an increase in cross-sectional area. Similarly, during waning ranges and discharges, flow scour potential decreases and sedimentation occurs. Deposition and erosion patterns may also be influenced by local or short-term processes which correlate in the following way:

Deposition:

(a) High gulf surf.

(b) Waning seasonal tidal ranges; decreasing discharges.

(c) Low seasonal tidal ranges; low discharges.

(d) Low lunar (monthly) tidal range (neap tides); low discharges.

(e) Low seasonal tide levels; low discharges.

Erosion:

(a) Waxing seasonal tidal range; increasing discharges.

(b) Alternating winds; maximum differentials when in phase with tides.

(c) High seasonal tidal range; large discharge.

(d) High lunar (monthly) tidal range (springtides); large discharges.

(e) High seasonal tide levels; large discharges.

(f) Steady offshore winds; low water levels, large ebb velocities.

3. Gulf Mouth.

Sequential surveys of the gulf entrance to the pass provided data for determining bottom changes between the ends of the jetty and on the ebb tidal delta. The surveys were obtained with an outboard motorboat equipped with a portable fathometer (Raytheon Model DE-719) which was run at a constant course and speed over 15 to 20 profiles. Boat position was determined by intersection of two simultaneous onshore transit sightings and coordinated by radio. Surf calm enough for accurate profiling was available less than once per month. Gulf mouth and beach profile data were used to develop the gulf mouth maps shown in Appendix C.

Estimates of volumetric changes in the gulf mouth region were made by determining water volumes between MSL and each bottom contour within the area shown in Appendix C, Figure C-1. Changes in volume between surveys constituted estimates of sediment added to or removed from the area. The dashline in Figure 17 summarizes erosion and deposition in the gulf mouth of the pass.

The trends shown in Figure 17 result primarily from the behavior of a scour hole across the channel mouth and adjacent to the north jetty (App. C, Fig. C-2). Since the configuration of this hole was controlled by tidal currents, longshore currents, and wave action, attempts to correlate bathymetric changes with one particular set of data were unsuccessful. However, erosion in this area probably resulted from the interaction of strong southward-moving longshore currents and tidal currents through the pass.

4. Bay Entrance.

The bay end of the channel and the flood tidal delta were surveyed with the same techniques used at the gulf mouth and in the rod and transit surveys of shorelines and berms. Aerial photos were used for qualitative mapping of bars and troughs in the extensive shallow sandflats adjacent to the channel. The resulting maps in Appendix D were used to document three types of changes at the bay mouth.

Using the same technique described for the gulf mouth, the contour maps were used to estimate changes in sediment volumes for the area shown in Appendix D, Figure D-1. A total of 63,000 cubic yards accumulated during the study period (Fig. 18). A comparison of the erosion and deposition patterns for the bay entrance and channel (Figs. 16 and 18) showed that the patterns were almost identical after an initial adjustment period (August to December). Apparently, much of the material deposited in the channel in August was transported to the bay during September and October, resulting in the first flood tidal delta depositional peak. Thereafter, erosion or deposition in the channel leads to erosion or deposition on the delta. Figure 19 shows that the controlling depth decreased at an average rate of about 0.6 foot per month and reached a minimum of 2 feet by May 1973, which equaled predredging depths in the area.

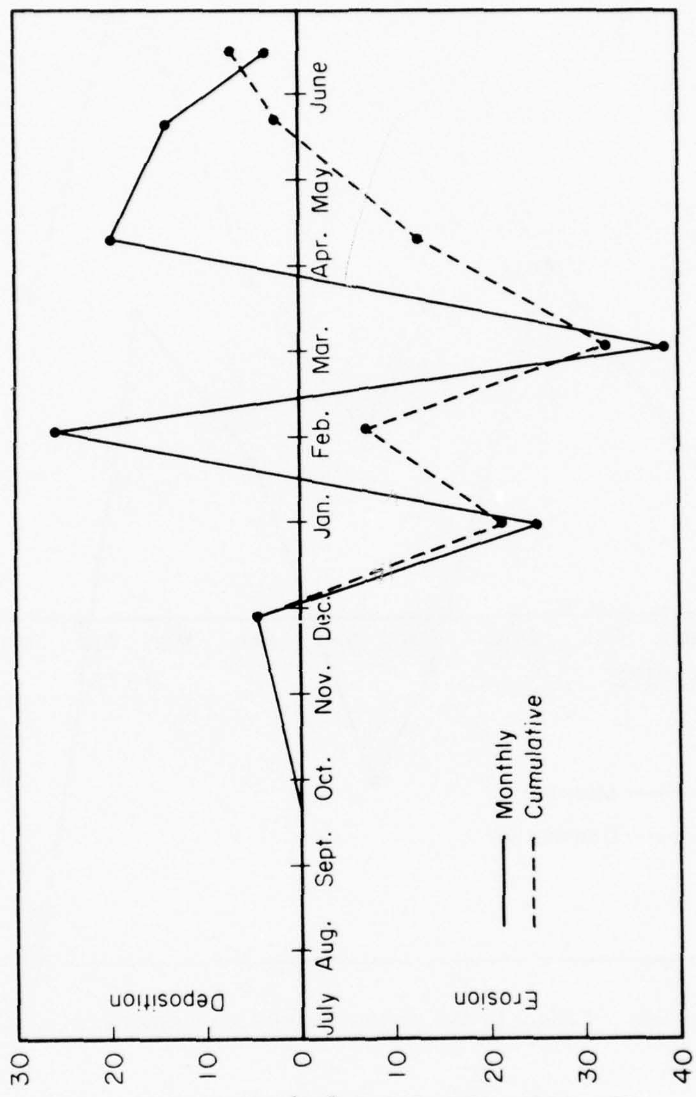


Figure 17. Gulf mouth deposition and erosion.

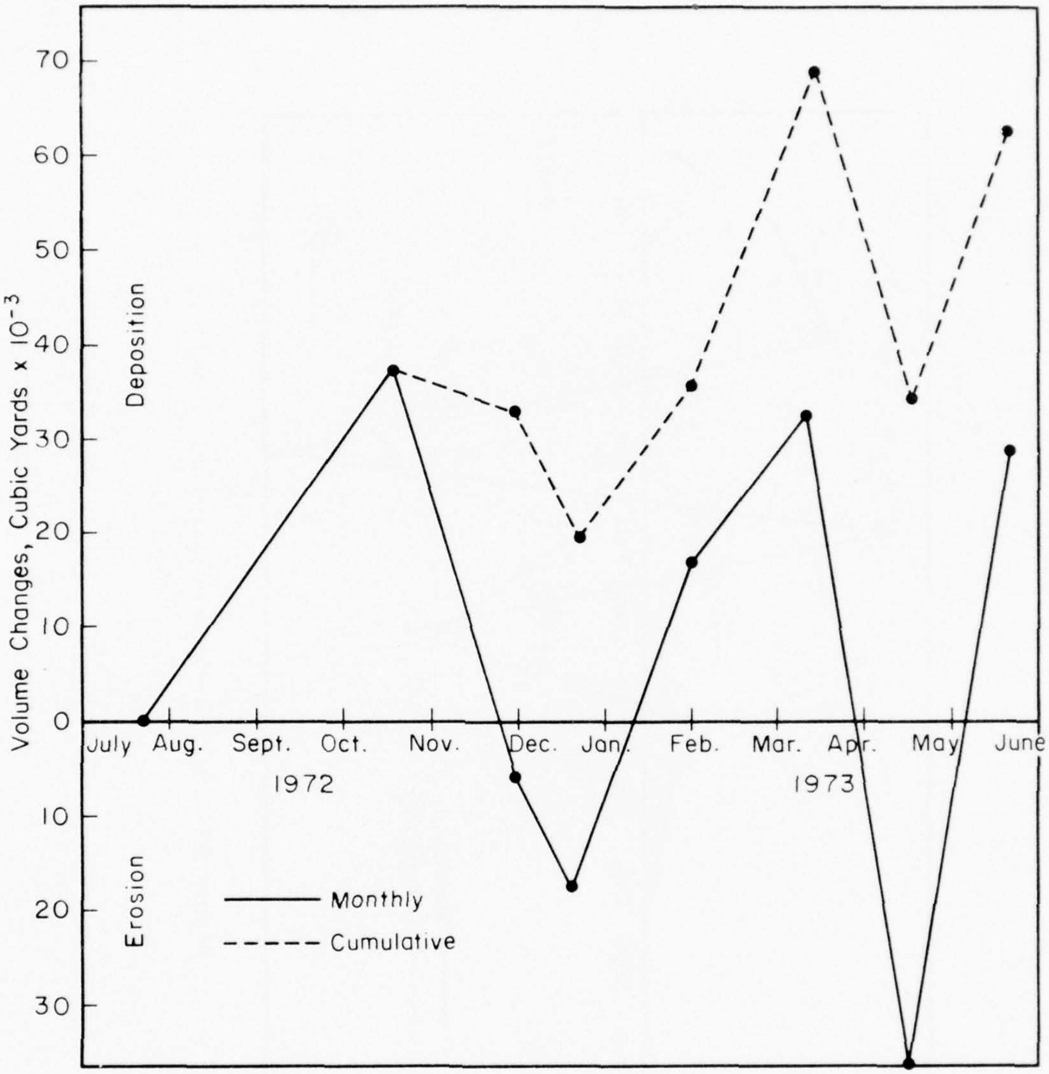


Figure 18. Flood tidal delta deposition and erosion.

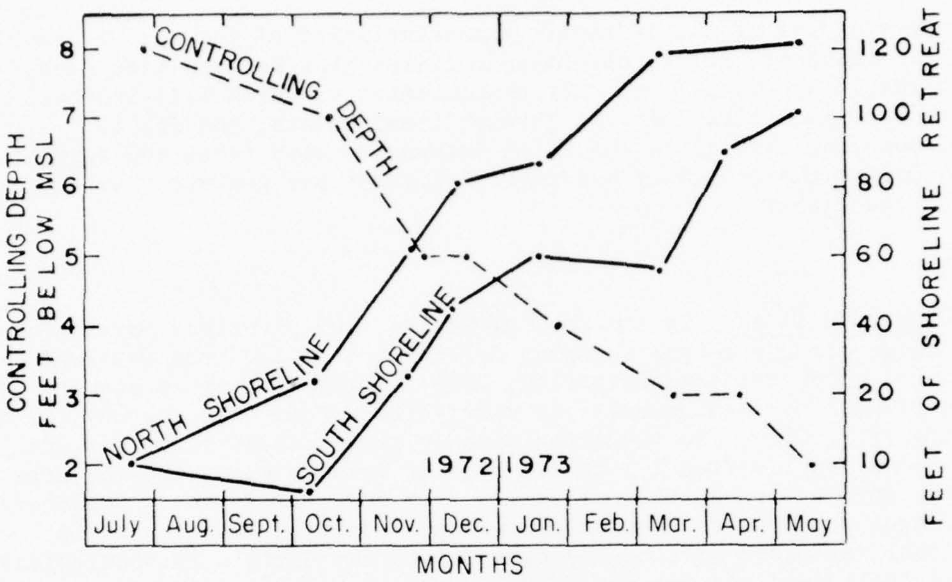


Figure 19. Variation in bay controlling depth and shorelines.

The final major change in the bay entrance was retreat of the shorelines on either side (Fig. 19). The shorelines 200 feet north and south of the channel centerline retreated over 100 feet. This retreat occurred mostly between October and December, and was probably caused primarily by waves generated in Corpus Christi Bay during northers; with the limited fetch and depths, significant wave heights and periods were probably in the order of 2 feet and 3 seconds, respectively.

VI. TIDAL HYDRAULICS

An understanding of the hydraulic characteristics of the pass is important for two reasons: (a) To determine relationships between tide elevations and resulting channel velocity characteristics which will improve techniques for predictions of flow through tidal inlets; and (b) to correlate observed changes in the inlet bathymetry with tides and currents (i.e., to define the effect of hydraulics on inlet bathymetric characteristics and stability).

1. Tides.

a. Predicted Tides. In the Gulf of Mexico, the principal variations in tidal range are due to the changing declination of the moon (National Oceanic and Atmospheric Administration, 1973). Diurnal tides occur at maximum declinations; semidiurnal and mixed tides occur when the moon is on the equator (Fig. 20). The predicted monthly spring tidal range (at new and full moons) varies from 2.1 to 2.8 feet at Aransas Pass, and the neap tidal range (at lunar quadratures) varies from 0.0 to 1.4 feet. Seasonally, maximum ranges occur at the summer and winter solstices when the sun's gravitational vector is more nearly parallel to the earth's in subtropical and higher latitudes; minimum ranges occur at the equinoxes when the sun's gravitational vector has a smaller vertical component.

The seasonal cycle in tidal ranges is not in phase with a similar seasonal cycle in tidal heights, which are a combination of astronomical and meteorological tides. Maximum water levels occur in October, April, and May when the strongest onshore winds generally occur. Minimum levels occur in January and February when strong offshore winds predominate. Low levels also occur during June and July when relatively light onshore winds are accompanied by significant diurnal sea breezes. This trend is subject to local variability in windspeed and direction, as well as in other factors influencing tide elevations.

b. Tide Measurements. Throughout the study period, tide data were collected from three gages located in the pass and throughout adjacent waters (Fig. 21) to define the tidal characteristics of the inlet-bay system, and to provide water surface slope measurements for hydraulic analyses. Two water level recorders (Stevens Type F) were installed in the pass and tied to the same bench mark to obtain data from which the water surface slope over a major part of the pass could be calculated. An existing NOS gage in Aransas Pass provided data approximating gulf tide levels. Since this gage was located inside the channel at the shore end of the south

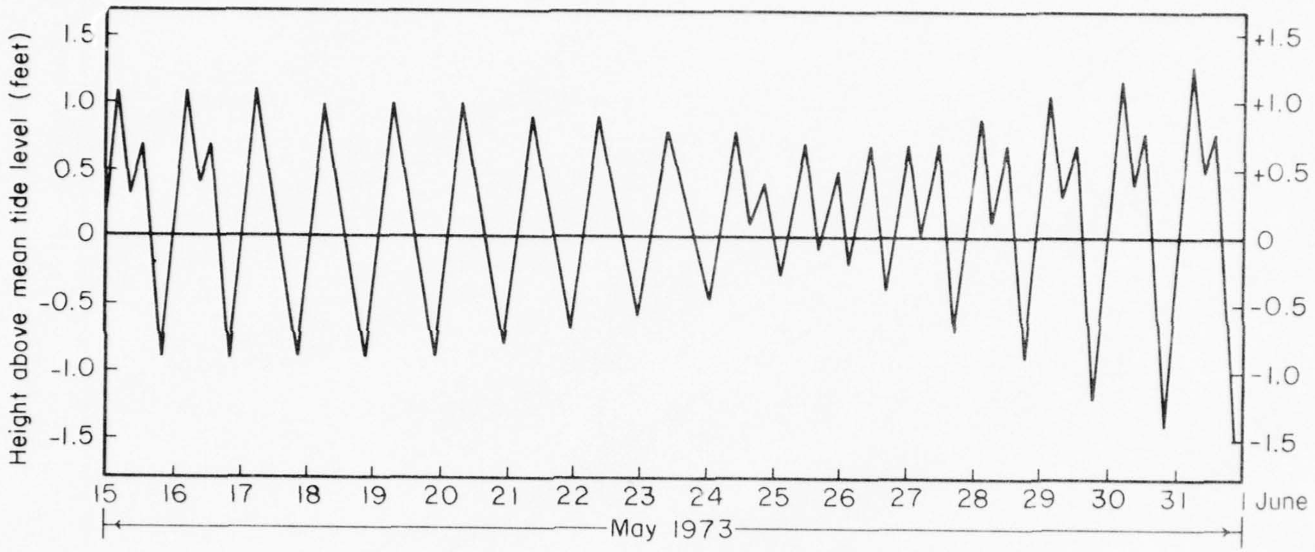


Figure 20. Predicted tides for Aransas Pass, Texas, 15 May to 1 June 1973 (after National Oceanic and Atmospheric Administration, 1973).

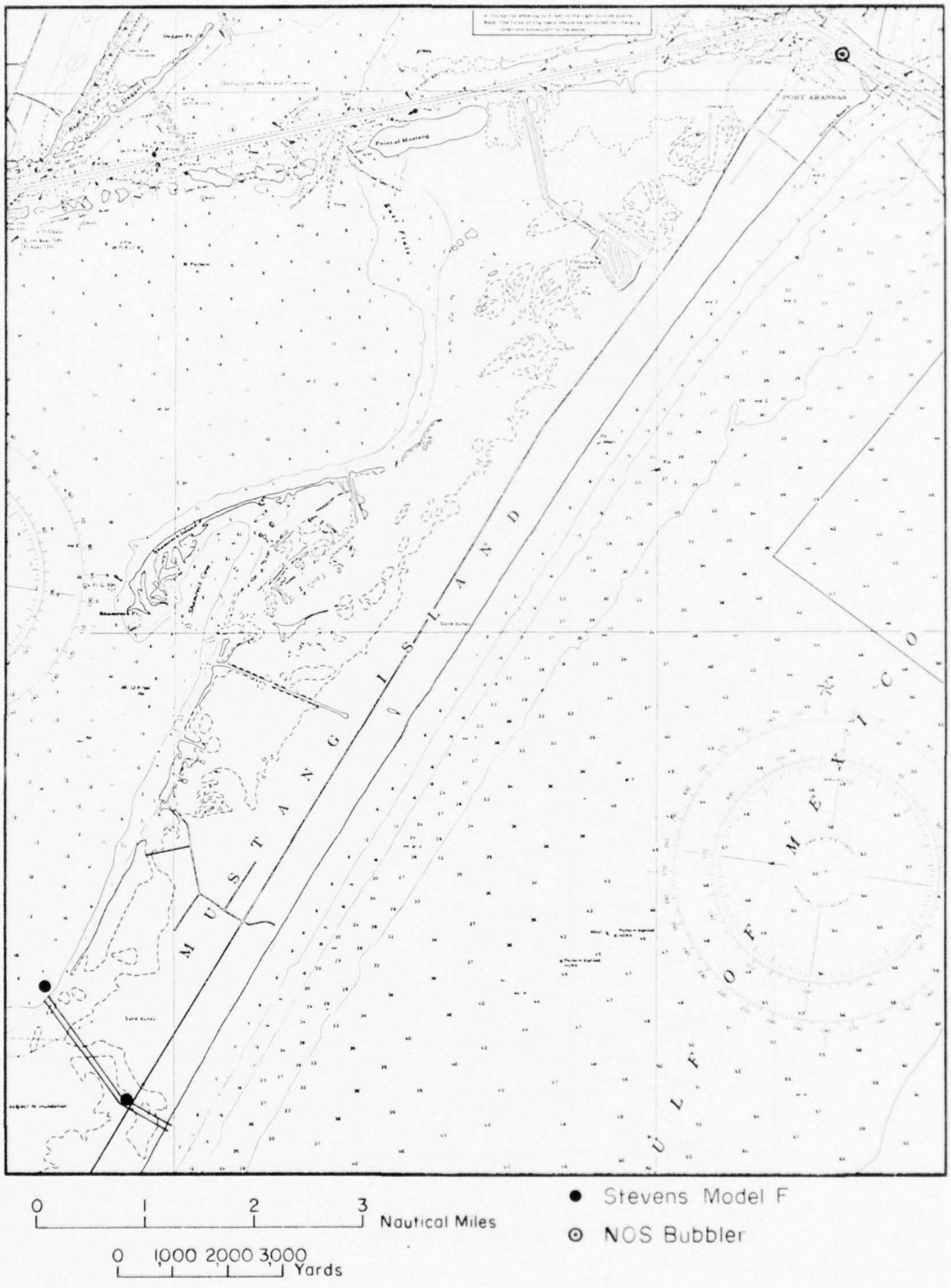


Figure 21. Location of recording tide gages (after NOS Chart 523).

jetty, it probably exhibited a tidal range somewhat less than that of the gulf. This difference was probably small, but unknown. The gages were geared to provide a 10-inch record for 24 hours of tide, with a vertical scale of 28.8 to 1 and a precision of ±0.01 foot. Water surface slopes calculated from these records should have errors less than 2×10^{-6}.

Seasonal variations in MWL and diurnal tidal range (Fig. 22), previously correlated with channel bathymetric changes, generally follow the trend of predicted tide characteristics, although significant deviations do occur.

2. Velocity and Discharge Measurements.

Nine tidal-cycle discharge surveys were completed, six near the bay mouth at cross-section X5 where the cross-sectional shape was simplest and most stable, and three at the bridge. A current meter (Gurley-Price cup-type) was used to obtain velocities at each range. Readings from the bridge were taken every 2 hours at depths of 0.2 and 0.8 times the maximum depth at each of 10 stations about 10 feet apart. At X5, hourly values at depths of 0.2, 0.6, and 0.8 times the maximum depth at each of nine stations were obtained except at the channel center, where readings were made every foot. Measurements were made from a small boat attached with a snatch block to a 0.75-inch nylon rope stretched tightly across the channel. The rope was held in place at each end with a large Danforth anchor. A measuring rope marked at 10-foot intervals attached to the nylon rope provided horizontal control. During the one tidal cycle when velocities were obtained simultaneously at both ranges, an average difference of only 2.5 percent was found in the calculated total discharge. Individual velocity readings probably erred no more that 0.1 foot per second (± one click of the meter). Three readings per vertical profile should give an accuracy of better than 2 percent (Rouse, et al., 1950). Assuming that a random error of 0.2 foot per second equals the standard deviation for a sample size of 30, the statistical parameter, t, is 3.65 at the 99.9-percent confidence level. Therefore, the calculated average velocity should be within 0.1 foot per second of the true value at the 99.9-percent confidence level.

a. Channel Geometry. The flow cross-sectional area during each velocity measurement was computed by adding the product of the channel width and the difference between low water and the instantaneous tide reading to the cross-sectional area below low water. The greatest potential source of error in defining the channel geometry at X5 was probably in the variability of the stretched measuring rope, which might lead to an error of up to 50 square feet in the cross-sectional area. Successive measurements during a discharge study should have relative errors of no more than ±20 square feet. With these variabilities for areas and wetted perimeter, maximum error in hydraulic radii is ±0.5 foot.

b. Discharge Characteristics. The amount of water flowing through the pass during each velocity measurement period was calculated as follows: A velocity for each vertical line was obtained by averaging two or more measured velocities on that line. The area defined by adjacent vertical profile lines, channel bottom, and the water surface was multiplied by the average of

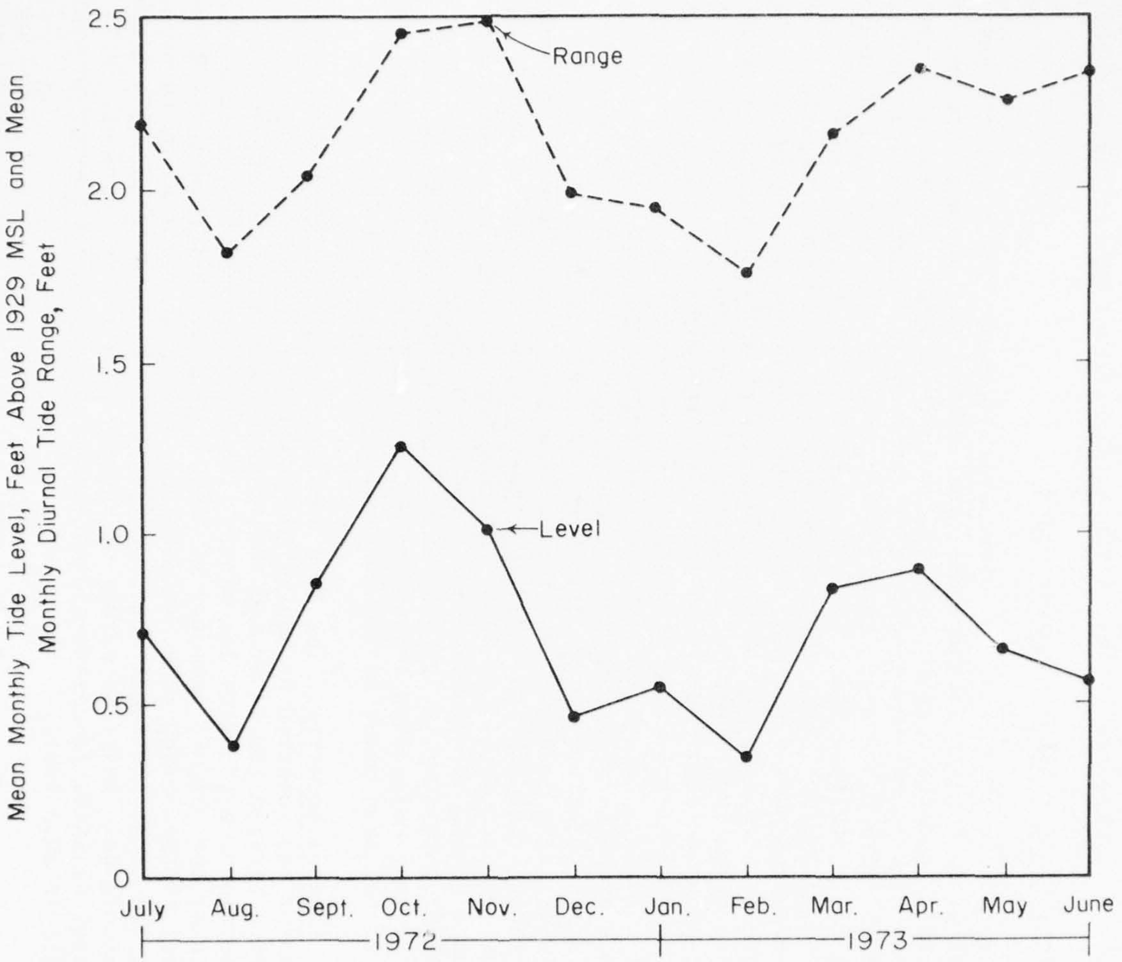


Figure 22. Summary of Aransas Pass tidal characteristics, July 1972 to June 1973.

the two profile lines to give discharge for that part of the cross section; total discharge was obtained as the sum of all sections. Average velocity for a cross section is total discharge divided by total area. Discharge, velocity, and geometric data are summarized in Table 7. Time histories of the discharges are shown in Appendix E.

Discharges through the pass over any one tidal cycle are highly dependent upon the tidal cycle characteristics and therefore are variable. Flood and ebb discharges were equal during diurnal tides; floodflow strongly predominated during mixed and semidiurnal tides (Table 7). The predicted tide curves in Figure 20 show that reduced tidal ranges during mixed tides result more from higher low waters than from lower high waters. Thus, during these periods, the flood duration and discharge are considerably greater than ebb. If the pass was the only entrance to Corpus Christi Bay, the system would tend to balance with an excess discharge at the beginning of diurnal tides. Although the 17 and 18 May discharges were obtained at such a time, only a slight excess ebb discharge was evident. Some water flowing through the pass probably returns to the gulf through Aransas Pass. However, measurements were not made during or shortly after northers which produce strong ebb flows through inlets on the Texas coast. Additional discharge measurements over a wider range of tide conditions are required to completely define the important relationship between tidal characteristics and discharge.

3. Channel Friction Characteristics.

An important factor affecting flow through tidal inlets is the channel friction. One of the most widely used measures of friction is the Manning's coefficient, n, which summarizes the resistance to flow resulting from small-scale surface roughness (material size and shape) and large-scale surface roughness (channel shape and bottom configuration). The following empirical equation equates the average steady uniform velocity in open channels to n:

$$V = \frac{1.49\ R^{2/3} S^{1/2}}{n} \qquad (7)$$

where V is average channel velocity (feet per second), R is channel hydraulic radius (feet), and S is slope of the energy gradient, approximately the slope of the water surface through the inlet. The rate of change in flow velocity through the channel is assumed to be small enough to approximate the steady-state conditions required for use of equation (7).

Although there has been much study of frictional characteristics of streams and rivers, less is known about values of n for tidal inlets. Values are given for some inlets by O'Brien and Clark (1973). To provide information on the frictional characteristics of a tidal channel, n values for the pass were computed at hourly intervals over selected discharge measurement periods. Since it is desirable to distinguish differences of at least 0.005 in n, and since measured n values averaged about 0.03, the

Table 7. Tidal cycle discharge data.

Data	1972		1973					
	17-18 Sept.	2-3 Dec.	20-21 Jan.	10-11 Mar.	18-19 Apr.	17-18 May	25-26 May	31 May to 1 June
Tidal range (ft) at Aransas pass gage	1.0 mean diurnal	1.5 mean diurnal	1.6 mean mixed	1.6 mean diurnal	0.9[1] neap semidiurnal	1.8 mean diurnal	0.8 neap mixed	2.3 spring diurnal
Discharge ($ft^3 \times 10^6$)								
Flood	61.2	69.9	109.0	90.1	118.7	43.4	48.4	113.0
Ebb	64.0	60.6	51.6	88.0	47.5	47.2	8.0	34.0
Duration (h)								
Flood	10.5	13.0	14.5	11.5	16.5	10.5	19.0	17.0
Ebb	14.4	12.0	10.5	11.0	8.5	11.5	6.0	8.0
Mean velocity (ft/s)								
Flood	1.98	1.53	1.64	1.76	1.64	1.13	0.68	1.73
Ebb	1.51	1.25	1.08	1.79	1.28	1.12	0.36	1.09
Maximum instantaneous velocity (ft/s)								
Flood	2.62	2.70	5.14	2.88	3.34	1.32	1.62	3.13
Ebb	2.76	2.54	3.12	3.07	1.78	1.54	0.73	1.55
Measurement range	bridge	bridge	bridge	X5	X5	X5	X5	X5
Mean cross-sectional flow area at measurement range (ft^2)	814.0	1,120.0	1,273.0	1,238.0	1,207.0	1,021.0	1,034.0	1,078.0
Minimum channel cross-sectional area below +0.8 MSL (ft^2)	780.0	980.0	950.0	930.0	940.0	900.0	866.0[1]	850.0[1]
Mean hydraulic radius at measurement range (ft)	8.1	11.2	12.7	6.6	6.0	6.2	6.0	6.1
Mean Manning's n	--------	0.019	--------	0.021	--------	0.029	--------	0.030

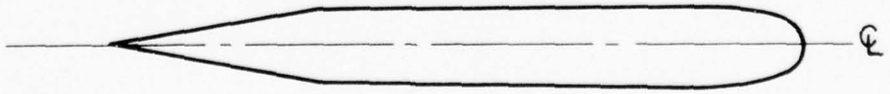
[1] Predicted or estimated; no data available.

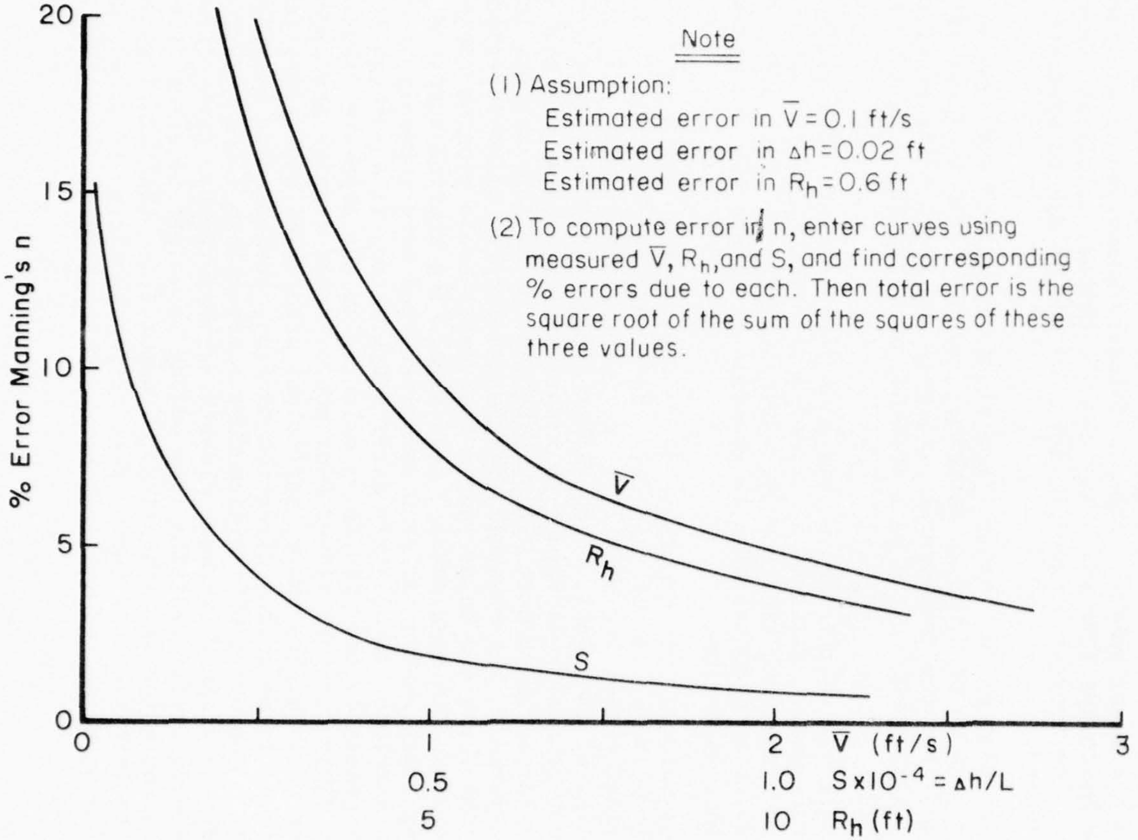
error of n must be kept below 15 percent. The effects of the estimated errors on n are computed in Appendix F and graphed in Figure 23. Hydraulic radii are all nearly 6 feet and thus produce a maximum error in n of 6 percent if measured to within 0.6 foot. Velocities below 0.6 foot per second and tidal differentials less than 0.2 foot do not give reliable values of n and were not computed for such times. Records from the water level recorders located at the bay mouth and the bridge provided data for calculating the water surface slopes. Velocity measurements provided discharge time histories from which the average channel velocities were determined. The average hydraulic radius of the bridge to baymouth channel section was used.

The mean value of n for the pass slowly increased through the study period from a value of 0.019 in September 1972 to 0.030 in April 1973. This resulted from the development of large- and small-scale bed forms in the channel subsequent to the pass opening, and shoaling at either end of the channel.

Short-term variability in n was also found during tidal cycle discharge measurements. Maximum n values were often twice the minimum values calculated for the same tidal cycle. Numerous scatter plots of n versus other variables were constructed to determine if there was a systematic cause for this variation.

The most informative were those of n versus time for a number of discharge periods (Figs. 24, 25, and 26). Note that for all tidal cycles except the March flood phase, the Manning's n increased regularly with the flood and ebb phases of the tidal cycle. After slack water the values of n were very low, but gradually increased to a maximum just before the next slack water. The factor which most likely affected n in this way was the existence of bed forms in the channel. Fathometer records and observations indicated that these bed forms ranged from small-scale ripples (less than 1 inch in amplitude) to sand waves 3 feet high. The ripples occurred throughout the long reach, while the sand waves occurred more commonly at either end of this section. Immediately after slack water, flow through the channel was generally over bed forms oriented in the opposite direction; i.e., early flood currents flowed over ebb-oriented ripples and dunes. During later parts of the phase, sufficient velocities would have developed to reorient bed forms in the direction of flow, perhaps also increasing their height. Hoerner (1965) listed drag coefficients for a number of shapes. For the shape which most closely represented bed forms (see sketch below), the drag coefficient for flow moving from left to right (i.e., for bed forms oriented with the flow) is about twice that for flow moving from right to left (for bed forms oriented in opposition to flow), due to the streamlining effect of the left side of the shape on the latter flow condition.





Note

(1) Assumption:
Estimated error in $\overline{V} = 0.1$ ft/s
Estimated error in $\Delta h = 0.02$ ft
Estimated error in $R_h = 0.6$ ft

(2) To compute error in n, enter curves using measured $\overline{V}$, R_h, and S, and find corresponding % errors due to each. Then total error is the square root of the sum of the squares of these three values.

Figure 23. Percent error in Manning's n.

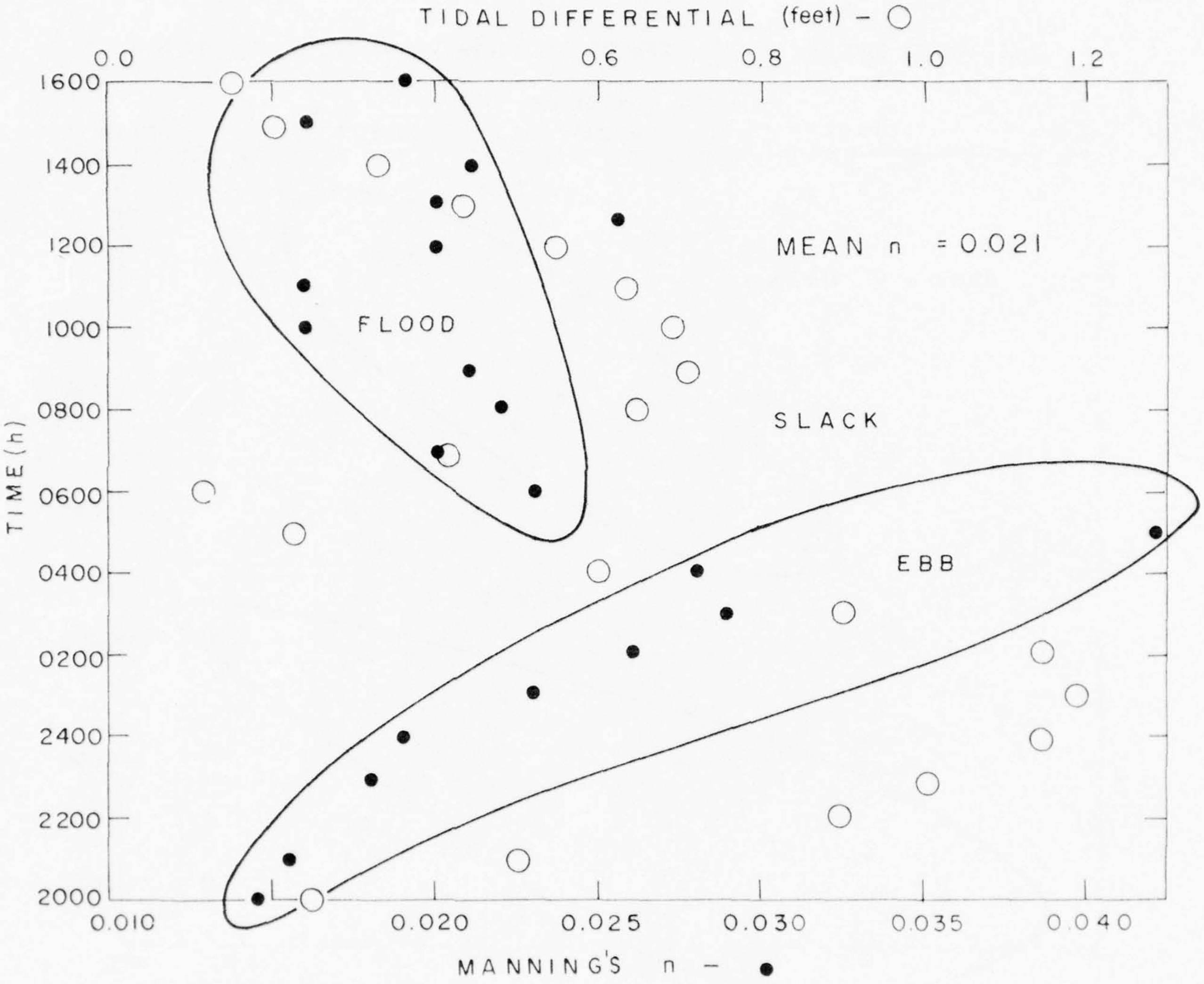


Figure 24. Tidal cycle variation in Manning's n, 10 and 11 March 1973.

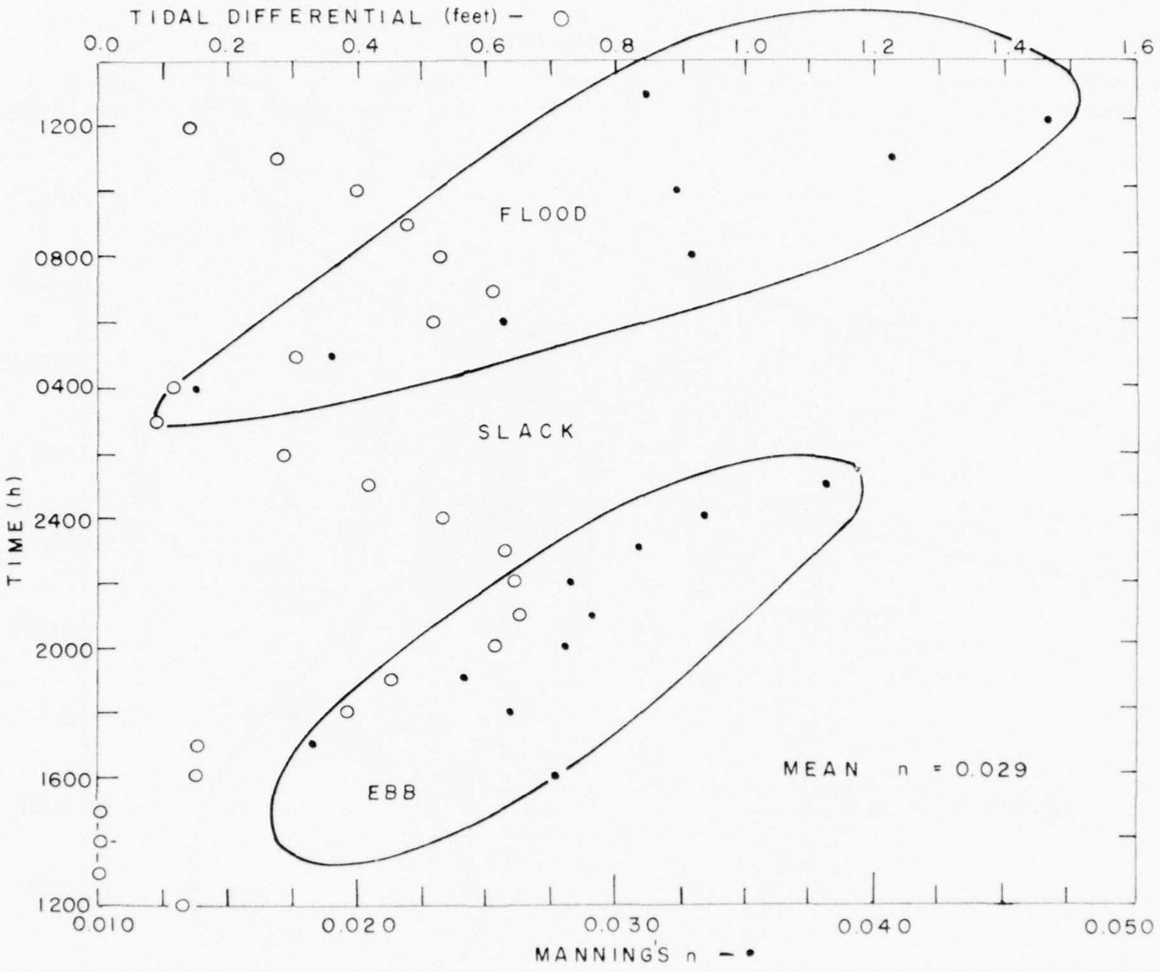
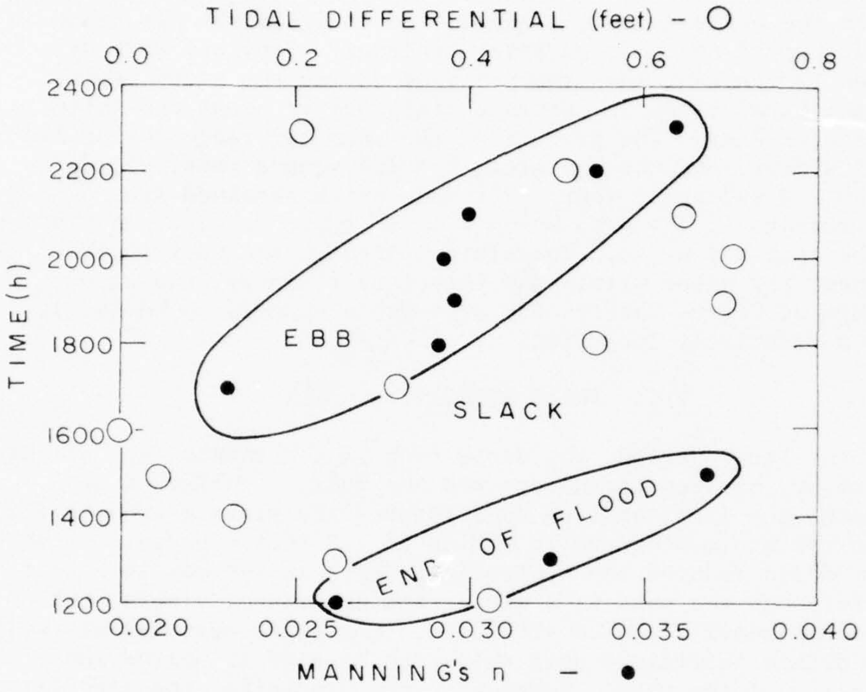


Figure 25. Tidal cycle variation in Manning's n, 17 and 18 May 1973.



MEAN n = 0.030

Figure 26. Tidal cycle variation in Manning's n, 31 May and 1 June 1973.

The ratio of mean maximum n to mean minimum n for the five phases (Figs. 24, 25, and 26) is 2:1, which agrees with Hoerner (1965). Therefore, the temporal variation in n is assumed to result primarily from the dependence of friction upon the orientation of channel bed forms with respect to flow direction, but may also be influenced by the growth of bed forms during a tidal cycle.

4. Inlet-Bay Hydraulic Relationships.

Since one of the objectives in building the pass was to enhance water exchange between the bay and gulf, a simplified analysis was performed to define the influence of the pass on water exchange. Previous work by Smith (1974) and independent measurements made during the study period indicate that the tidal range in Corpus Christi Bay is about one-third the range at Aransas Pass. The product of the mean bay range during the study period, 0.4 foot, and the bay area, 5×10^9 square feet, yields a mean tidal prism 2×10^9 cubic feet. The mean prism obtained from discharge measurements in the pass was 6.6×10^7 cubic feet, or only about 3 percent of the mean bay prism. Therefore, although the pass significantly influences bay water within the immediate vicinity, the effect on water exchange of Corpus Christi Bay as a whole appears to be small, occurring over a relatively long time.

VII. INLET STABILITY

Throughout the study period, the deposition in the channel was slight (Fig. 16). However, between the bridge and the gulf, significant deposition throughout the first half of 1973 reduced the minimum area to 818 square feet and the minimum hydraulic radius to 2.3 feet. Deposition on the flood tidal delta reduced the controlling depth in the bay to 2 feet by May 1973. Although the pass as a whole remained rather stable, the channel ends became heavily choked with sand. Continued measurements will be required to obtain sufficient data which can be used to define the long-term stability of the pass. However, since predicting the stability is important in properly designing an artificial inlet, a number of existing prediction methods applicable to the pass are discussed in the following paragraphs.

The relationship between tidal prism and inlet cross-sectional area has been used by O'Brien (1969) and others as an indication of inlet stability. The most recent and comprehensive analysis of this relationship was completed by Jarrett (1976), who found that for inlets on the gulf coast, the minimum inlet cross-sectional area, $A_{c_{min}}$, is related to the spring range tidal prism, P, by:

$$A_{c_{min}} = 5.02 \times 10^{-4} P^{\alpha 84} \qquad (8)$$

For the spring range tidal prism measured at the pass (9.9×10^7 cubic feet) the equilibrium cross-sectional area would be 2,612 square feet, which is over twice the actual minimum area of 950 square feet. However, this value falls within the 95-percent confidence limits of Jarrett's analysis. Jarrett points out that the gulf coast data exhibited more scatter than Pacific and Atlantic coast data, and attributes this in part to the wide variability in monthly tidal ranges. No conclusive prediction of the pass' stability can be made according to the area versus prism relationship.

A second prediction of inlet stability uses a method developed by Bruun (1966), who related inlet stability to the mean tidal prism, $\bar{P}$, and predominant (southward) annual longshore transport rate, M, at an inlet as follows:

$$\bar{P}/2M < 100 \text{ unstable}$$

$$\bar{P}/2M > 300 \text{ stable}$$

$$100 < \bar{P}/2M < 300 \text{ intermediate}$$

For the pass, the ratio of $\bar{P}/2M$ is only 2.4. Bruun's method predicts that the inlet is highly unstable with strong shoaling tendencies.

The final stability analysis to be applied to the pass is that of O'Brien and Dean (1972), which requires that the inlet-bay hydraulics be adequately described by the Keulegan (1967) method. The analysis utilizes a stability relationship between maximum velocity and cross-sectional area similar to that of Escoffier (1940). However, the O'Brien and Dean method is valid only for a single inlet to a bay and therefore does not strictly apply to the actual conditions. Nevertheless, it is instructive to investigate the stability of a hypothetical single entrance to Corpus Christi Bay.

Keulegan relates the ability of the inlet to fill the bay to a coefficient of repletion, K, given by:

$$K = \frac{TA_c}{\pi A_B} \sqrt{\frac{g}{2a_o \left[K_{en} + K_{ex} + \frac{fL}{4R}\right]}} , \tag{9}$$

where

T = gulf diurnal tide period, 89,000 seconds

A_c = inlet cross-sectional area, variable

A_B = bay area, 5×10^9 square feet

a_o = mean diurnal gulf tide amplitude, 0.9 foot

f = Darcy-Weisbach friction factor, 0.042

L = channel length, 10,000 feet

R = hydraulic radius, variable

$K_{en} + K_{ex}$ = entrance and exit energy loss coefficients = 1.0

g = acceleration of gravity, 32.14 feet per second squared

O'Brien and Dean (1972) described a dimensionless maximum velocity, V'_{max}, a function of K, which is given by:

$$V'_{max} = \frac{TA_c\ V_{max}}{2\pi\ a_o\ A_B} \ . \tag{10}$$

After determining K and V'_{max} for a given value of a cross-sectional area, a corresponding value of the average maximum velocity through the inlet, V_{max}, can be found using equation (10). However, an assumption concerning A_c and R must be made first to find K. Results of a study by Galvin, Kohler, and Tenney (1971) indicated that for seven Texas inlets the cross-sectional area at the minimum width is related to the average depth (≈ hydraulic radius) of that cross section by:

$$A_c = 162R^2 \ ; \tag{11}$$

thus, equation (9) becomes:

$$K = \frac{TA_c}{\pi A_B} \sqrt{\frac{g}{2a_o\left(K_{en} + K_{ex} + \frac{3.18fL}{\sqrt{A_c}}\right)}} \ .$$

Figure 27 is a plot of the V_{max} and A_c values. The coordinates of the peak of the curve (V_{max} = 2.5; A_c = 60,000) are the theoretical maximum velocity and the critical cross-sectional area of a single inlet having the same length and friction as the pass. The velocity of 2.5 feet per second is lower than the 3.5 feet per second velocity generally associated with stable inlets (Bruun, 1960; Carothers and Innis, 1960; O'Brien, 1969), indicating that at best, a single inlet would be marginally stable with a tendency to shoal. This description fits Aransas Pass, the existing major inlet to the bay, where considerable dredging is required. The initial cross-sectional area of the water exchange pass, 900 square feet, is much less than the optimum area. Therefore, if the pass was the only inlet to the bay, this method predicts that it would be highly unstable and close rapidly.

VIII. SEDIMENT CHARACTERISTICS

Sediment grain-size analyses were made with a modified Woods Hole Oceanographic Institution rapid sediment analyzer (Schlee, 1966). Grain

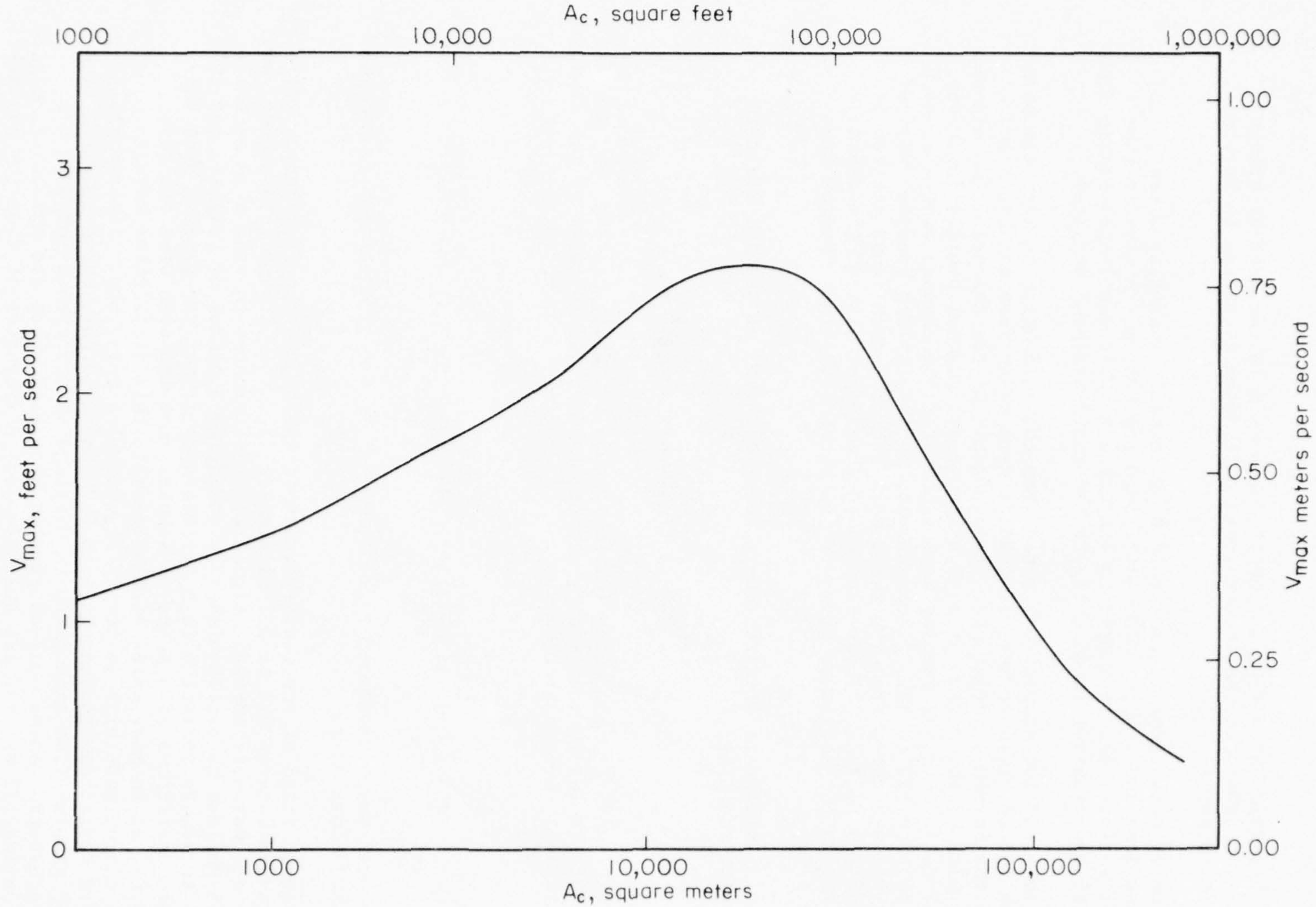


Figure 27. V_{max} versus A_c for single inlet to Corpus Christi Bay.

sizes were calculated from the modified Rubey's law derived by Watson (1969), and grain-size parameters were calculated according to Folk's (1964) equations.

Samples were taken in the field by vertically inserting a vial, 2 inches long and 1 inch in diameter into the bottom. A plastic tube (cutoff syringe) was then used to extract a 5-cubic centimeter sample from the vial and transfer it directly to the rapid sediment analyzer.

A total of 101 samples was taken from the gulf beach between the berm and the second offshore bar; 51 samples were taken from the beach and across the barred sandflat near the bay mouth of the channel. The channel was sampled by skindivers at each of the cross-channel profile locations from X2 to X22. Seven samples were taken from the channel center at each cross section (X2 to X8). Three samples in the thalweg (deepest part of the inlet) and three from the shoal were taken at each cross section in the bend (X9 to X14). Three samples in the deeper side of the channel and three in the shallower side were taken at each cross section from X15 to X22.

The significance of differences between groups of samples was determined with the t-test:

$$t = (\bar{X}_1 - \bar{X}_2) / s \, [(n_2 + n_1) / n_1 n_2]^{1/2} \tag{12}$$

where $\bar{X}_1$ and $\bar{X}_2$ are means for the groups 1 and 2, n_1 and n_2 are the number of samples in each group, and s is a pooled estimate of the standard deviations of the two groups;

thus,

$$t = [(n_1 - 1) \; s_1^2 + (n_2 - 1) s_2^2] / (n_1 - 1) \; (n_2 - 1)^{1/2}.$$

Differences were considered significant if t was greater than the 95-percent confidence level.

A small range of sedimentary textures existed in the study area; all sand ranged between 0.1 to 0.2 millimeter (3.2 to 2.4 phi) in diameter, and almost all was well sorted. Significant differences in sand size as small as 0.05 phi can be distinguished by comparing a number of samples, and if sample to sample variation in a certain area is not too great. When the standard deviations of a suite of samples from the same area are less than 0.05, as in many parts of the channel, this distinction between groups can be made with as few as five samples per group. Intersample standard deviation is often between 0.05 and 0.10 phi, which requires from 9 to 30 samples from each group to distinguish differences of 0.05 phi between groups. Where over 40 samples from each group are available, e.g., in comparing all bay and all gulf samples, differences of 0.05 phi can still

Table 8. Ranked sediment grain-size distribution parameters[1].

Means (phi)		Sorting (phi)		Skewness		Kurtosis	
G berm	3.02	G berm	0.33	G foreshore	+0.04	G second bar	1.12
C bend bar	2.96	C bend bar	0.34	C bend thalweg	+0.03	C bend thalweg	0.98
G second trough	2.96	C long reach	0.35	G berm	-0.01	B troughs	0.98
C long reach	2.92	G foreshore	0.37	C bend bar	-0.04	C long reach	0.97
B foreshore	2.92	B foreshore	0.37	ALL CHANNEL	-0.06	ALL CHANNEL	0.97
ALL CHANNEL	2.90	ALL CHANNEL	0.38	C long reach	-0.07	C bend bar	0.96
C short reach	2.88	C short reach	0.38	C short reach	-0.07	G berm	0.94
G foreshore	2.87	ALL BAY	0.39	B foreshore	-0.08	G second trough	0.94
ALL GULF	2.86	C bend thalweg	0.39	B bars	-0.08	ALL GULF	0.94
G first trough	2.86	G second trough	0.40	C first trough	-0.09	G foreshore	0.93
C bend thalweg	2.85	B bars	0.40	ALL BAY	-0.10	ALL BAY	0.93
ALL BAY	2.82	B troughs	0.41	ALL GULF	-0.11	G first bar	0.92
B troughs	2.82	ALL GULF	0.43	G first bar	-0.13	B foreshore	0.92
G second bar	2.79	G first trough	0.43	B troughs	-0.14	G first trough	0.92
G first bar	2.74	G first bar	0.47	G second trough	-0.19	C shore reach	0.91
B bars	2.72	G second bar	0.60	G second bar	-0.31	B bars	0.88

[1] G (gulf); C (channel); B (bay); bend bar (meander point); bend thalweg (thalweg adjacent to meander point bar); long reach (channel between X4 and X9); short reach (channel between X15 and X18).

be distinguished, but the larger samples usually represent more variable conditions of deposition and have larger intersample standard deviations.

Although small differences can be distinguished in groups of samples, these differences are rarely found in the study area (Table 8). Mean grain sizes of groups of samples have a range of only 0.30 phi (from 2.72 to 3.02 phi). The finest grain sizes (3.02 to 2.87 phi) occur where wave action is minimal; i.e., the gulf berm crest, the gulf and bay foreshores, and the channel. The coarsest samples (2.79 to 2.72 phi) are from the gulf and bay bars.

Except for the gulf and bay bars, sorting and skewness have ranges of only 0.1 and 0.2 phi, respectively. The finest samples are also the best sorted (0.33 to 0.38 phi sorting; +0.04 to -0.07 skewness) and include mostly channel samples. Except for the foreshore and berm samples, the gulf sands are the most poorly sorted and coarsely skewed.

Kurtosis also has a very small range (all groups except two are within 0.07 of each other) and generally has greater intersample standard deviation, so almost no differences are statistically significant. However, most channel samples are grouped closest to 1 (near normal distribution), and gulf and bay samples tend to be slightly platykurtic (a slight excess of very coarse material, probably shells).

As shown in Table 9, from gulf to bay to channel, all grain-size parameters seem to follow the general trend displayed by the mean values; i.e., where wave action is present, the coarsest and most poorly sorted material is deposited. The grain-size parameters also reflect differences in wave action in smaller depositional units such as the gulf and bar trough systems (Table 9). The same trend is weakly evident in the channel between the jetties where surf breaks on a shoal adjacent to the south jetty (mean = 2.85 phi); breakers are rare in the deeper channel adjacent to the north jetty (mean = 2.93 phi).

Table 9. Trend and differences in wave action of grain-size parameters.

Trend	Mean (phi)	Sorting (phi)	Skewness	Kurtosis
Bay bars	2.72	0.40	-0.08	0.88
Bay trough	2.82	0.41	-0.14	0.98
Gulf first bar	2.74	0.47	-0.13	0.92
Gulf first trough	2.86	0.43	-0.09	0.92
Gulf second bar	2.79	0.60	-0.31	1.12
Gulf second trough	2.96	0.40	-0.19	0.94
Intrajetty shoal	2.85	0.43	-0.09	0.92
Intrajetty channel	2.93	0.39	-0.13	0.95

The opposite trend occurs in the channel bend where a shoal formed as a meander point bar. This bar receives no wave action and less current than the adjacent thalweg; thus, its sand is finer and better sorted (Table 8).

Table 10. Comparison of grain-size parameters.

Study area	Mean (phi)	Sorting (phi)	Skewness	Kurtosis
Mustang Island (Mason and Folk, 1958)	2.82	0.31	+0.03	1.09
North Padre Island (Hayes, 1964)	2.80	0.35	-0.09	0.91
Corpus Christi Pass gulf foreshore	2.87	0.37	+0.04	0.93

Table 10 compares the grain-size parameters in the study area to the gulf beach grain-size parameters published by Mason and Folk (1958) and by Hayes (1964). Values obtained for the gulf foreshore in this study are slightly but significantly different. Sieve techniques used in previous studies were replaced in this study by the rapid sediment analyzer, but it is not known if this accounts for the differences in results.

The grain sizes in the study area are among the most difficult for predicting sediment transport from shear stress relationships. Estimates of critical shear stress for a grain diameter of 0.13 millimeter vary by an order of magnitude. However, the uniformity of sediment throughout the area makes it unlikely that differences in erosion, deposition rates, or distributions are due to differences in material-size characteristics.

IX. CONCLUSIONS AND RECOMMENDATIONS

1. Bathymetric Changes.

Before the pass was opened, the jetties acted strictly as groins and trapped about 100,000 cubic yards of sand within 1,500 feet of the pass, an amount roughly equal to the predicted net longshore transport rate for the study period. Erosion and deposition in the gulf areas adjacent to the pass responded to seasonal reversals in the longshore transport system. During the summer, the predominantly northerly transport caused accretion on the south beach and erosion to the north; the pattern reversed during 8 months of the year, due primarily to storms. Annually, the winter regime predominated and the net transport was to the south, causing erosion of the beaches south of the pass. The southward longshore transport predominance was not due to higher waves during the winter; the average monthly breaker heights (2.5 feet) were consistent throughout the year.

In the pass, monthly values of deposition and erosion sometimes exceeded 19,600 cubic yards, but the net change was only 6,000 cubic yards of deposition in the gulf end of the channel, which had been dredged deeper and wider than the rest of the channel. Most of the sediment entering the channel from the gulf was transported through to form an extensive flood tidal delta.

2. Hydraulics.

During diurnal gulf tides, the pass contributed an average of only 3 percent of the tidal prism of Corpus Christi Bay which did not appreciably enhance daily water exchange between the gulf and the bay as a whole. Nevertheless, while discharge through the pass was highly dependent upon tidal cycle characteristics, measured floodflows strongly predominated over ebb, indicating a possible long-term circulation from the pass through the bay to Aransas Pass. The measured mean flood current velocity of 1.5 feet per second transported sufficient quantities of sand to form an extensive flood tidal delta. Further studies to quantify long-term circulation patterns and the characteristics of tides and flow during northers are required to completely describe the hydraulics of the pass.

The average value of Manning's friction factor increased gradually during the study period from 0.019 to 0.030, due to bed form development and shoals at the ends of the pass. However, because of changing directions or dimensions of bed forms, instantaneous bed friction varied over a significantly greater range through a tidal cycle. Friction factors used to predict or calculate flow velocities should be determined from full tidal cycle data except during low flow velocities or small tide level differentials, when the error in calculating the friction factor is greatest.

Since realistic estimates of friction factors are required for accurate prediction of tidal flow through inlets, it is recommended that detailed field studies be performed to quantitatively document these effects. Observations and measurements of bed form size, orientation, and migration rates should be made concurrently with precise water level slope and velocity measurements.

3. Stability.

The small net change in channel volume and lack of diminution of tidal discharges suggest that the channel was relatively stable during the 10 months after opening. However, continued decreases in the minimum cross-sectional area and the low average velocities of 1.5 feet per second suggest unstable conditions. These conflicting results, lack of measurements during July and August, and extreme variability in climatic conditions in the study area make prediction of long-term stability difficult. Since the frequency of surveys was best suited to observe seasonal effects, these have been explained at least qualitatively. However, it is recommended that additional studies be made to correlate quantitatively the sedimentary changes with shorter term processes; i.e., storms and semimonthly tidal fluctuations.

4. General.

Although the long-term stability of the pass is not defined, several other short-term reactions of the pass provide criteria for qualitatively predicting behavior of channels of this type, and indicate several considerations which should be made when designing an inlet monitoring program. Examples of the criteria and indications are:

(a) Beach erosion is often a serious problem where jetties act as groins. Creation of an efficient natural or artificial bypassing system is usually needed to prevent downdrift beach erosion. However, when bypassing is proceeding effectively, there still can be significant beach erosion, such as that on the southwest side of the pass, since some of the longshore drift is carried into or through the pass to create permanent deposits (e.g., the flood tidal delta).

(b) Channeling through subaqueous shoals on a windward shore may increase wave attack and erosion on that shore. If wave erosion creates a funnel-shaped mouth to the channel, the shoreline constituting the sides of the funnel becomes oblique to almost all approaching waves, and is subject to increasing loss of material into the channel. This effect may be enhanced further by currents flowing through the marginal channels immediately adjacent to the shoreline (e.g., flood tidal channels on an ebb tidal delta). Some combinations of these processes resulted in more than 100 feet of shoreline retreat at the bay mouth of the pass. Therefore, smaller jetties at the bay ends of inlets should be considered when the bay fetch is as large as that of Corpus Christi Bay.

(c) In planning measurements of deposition and erosion at inlets, large, short-term variability (monthly or less) that may obscure longer term net effects should be considered. Variability in tidal characteristics and surf conditions is the major determinant of short-term erosion and deposition fluctuations. The tidal characteristics increase or decrease flow velocities and discharge, with velocity decreases corresponding to deposition and velocity, increases corresponding to erosion. Surf conditions control the amount of sediment suspended or transported by waves to the mouth of the channel making it available for transport by tidal currents. Both these factors are influenced by storms and hurricanes.

(d) Flow in channel bends is concentrated at the outside of the bend, producing channels which meander; the dimensions will depend on the channel width. In straight reaches, ebb and flood flows may use the same channel; in meanders, the flows may develop separate channels leading to erosion of a much larger total channel cross section.

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APPENDIX A

BEACH PROFILES AND GULF BATHYMETRIC MAPS

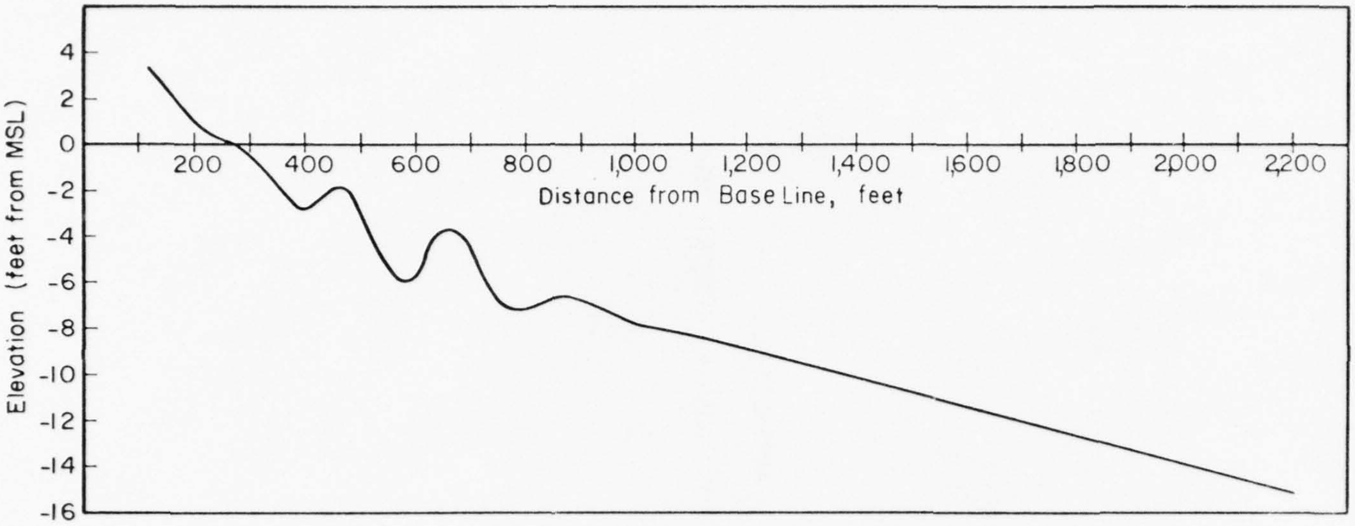

Figure A-1. Preconstruction profile, average of October 1972 4000N and 4000S profiles and a preconstruction centerline profile.

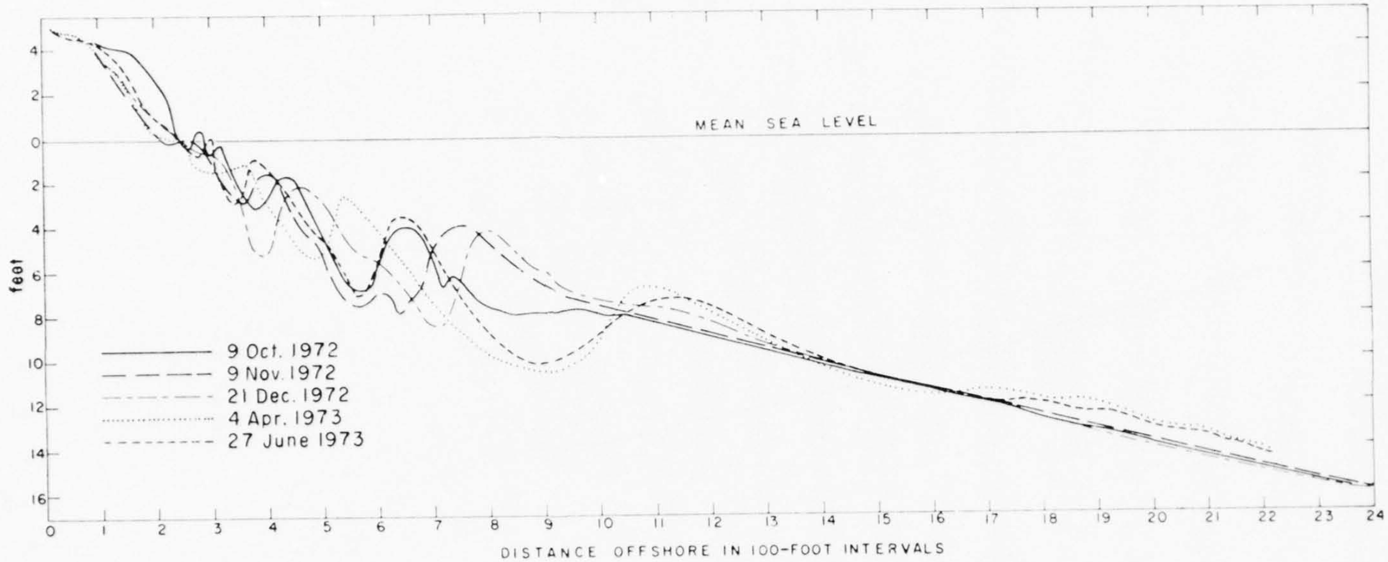

Figure A-2. Beach profile 4000N.

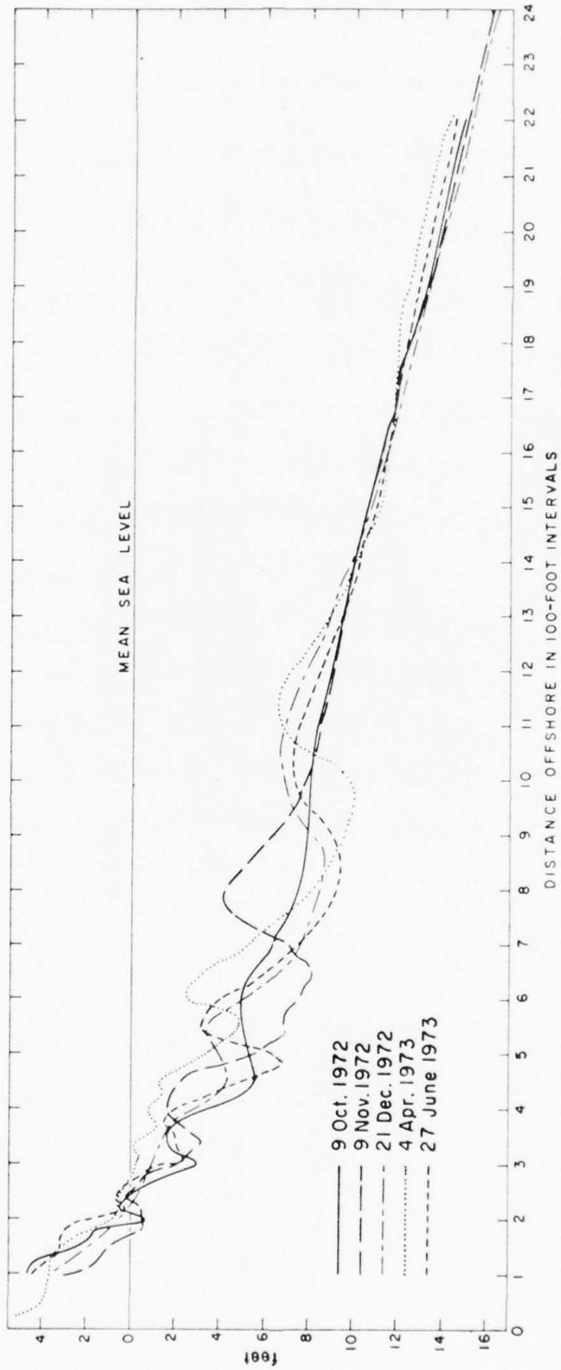

Figure A-3. Beach profile 2000N.

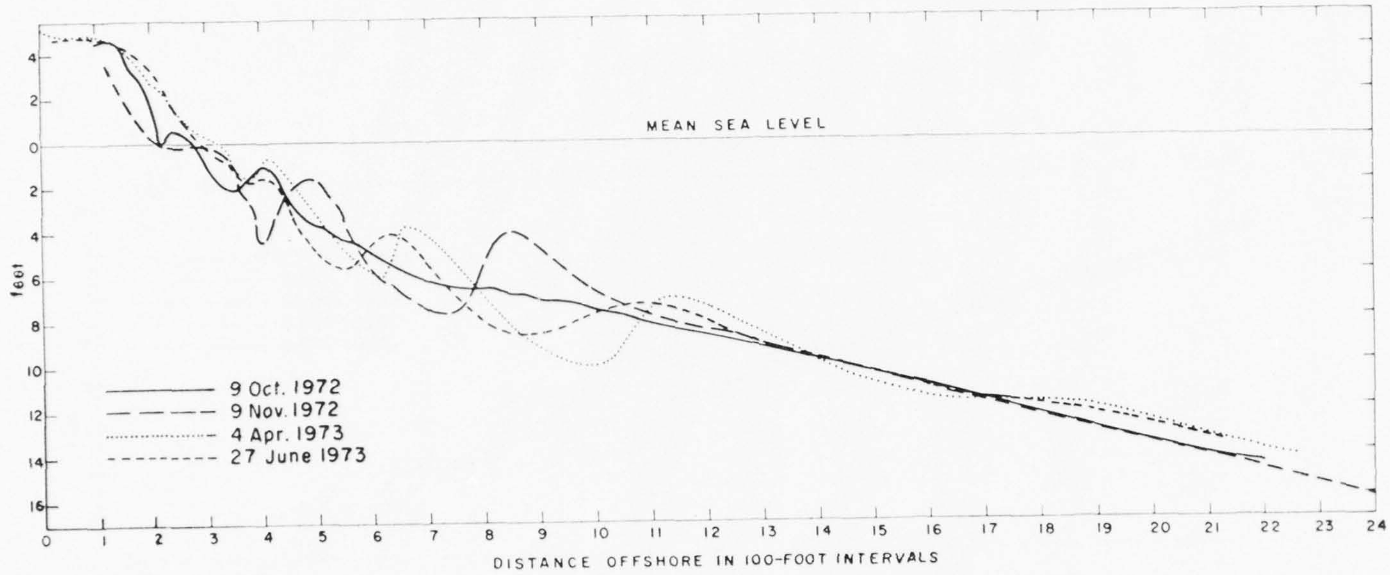

Figure A-4. Beach profile 1200N.

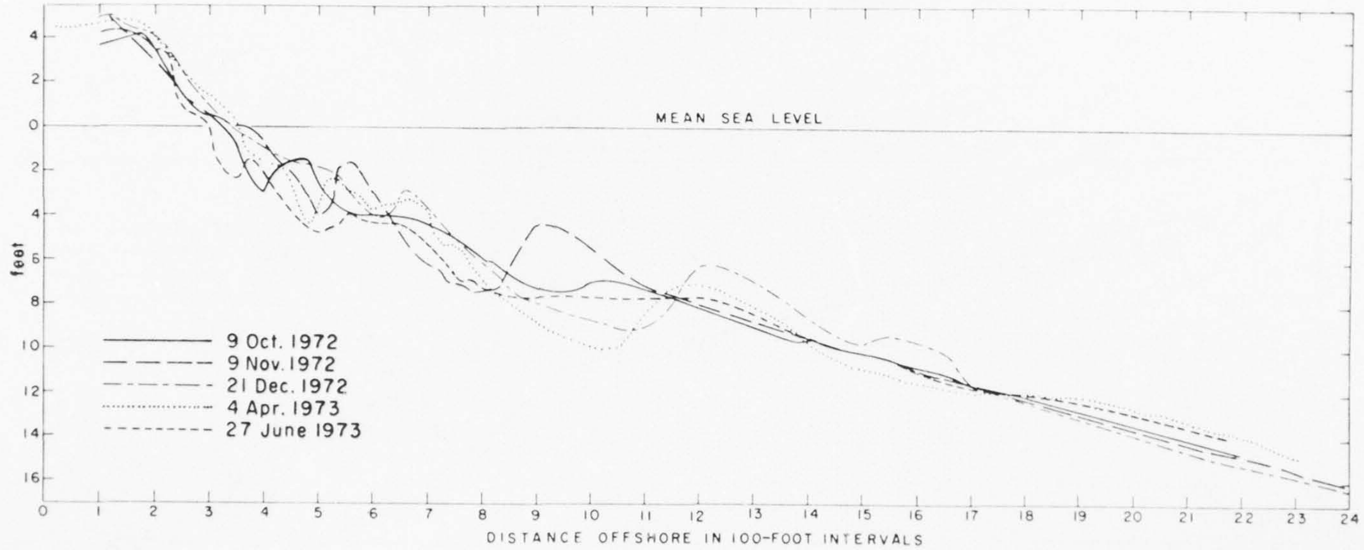

Figure A-5. Beach profile 800N.

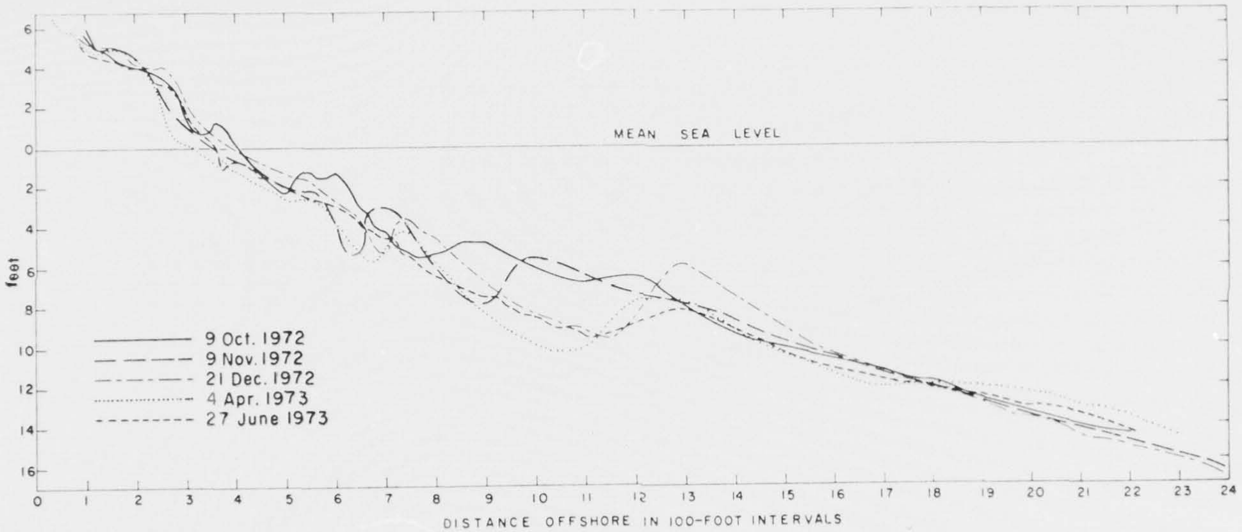

Figure A-6. Beach profile 400N.

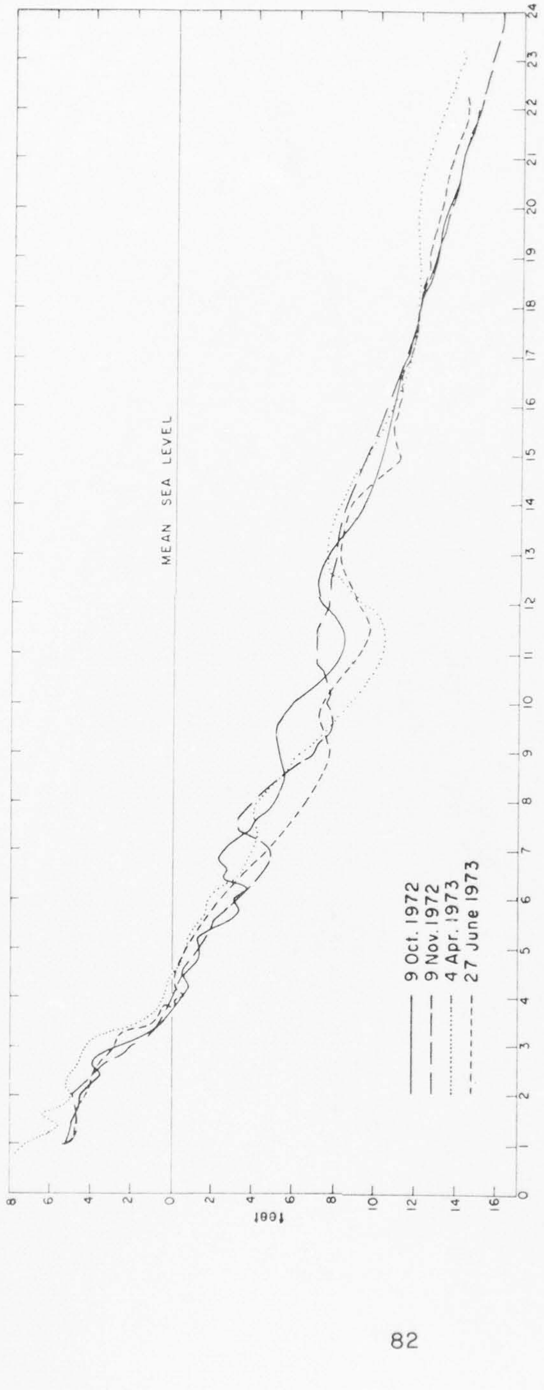

Figure A-7. Beach profile 250N.

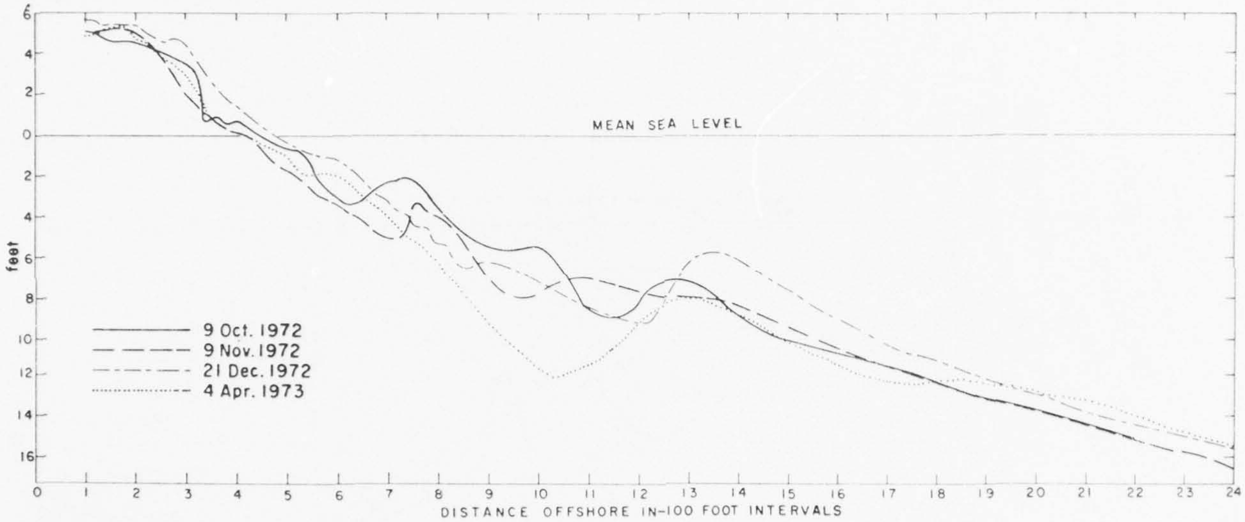

Figure A-8. Beach profile 150N.

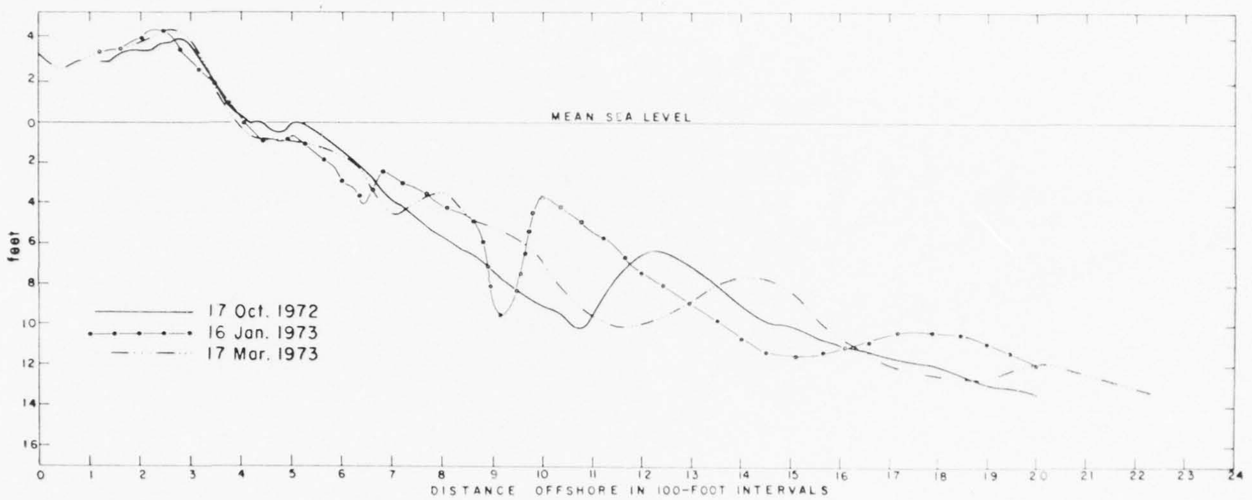

Figure A-9. Beach profile 150S.

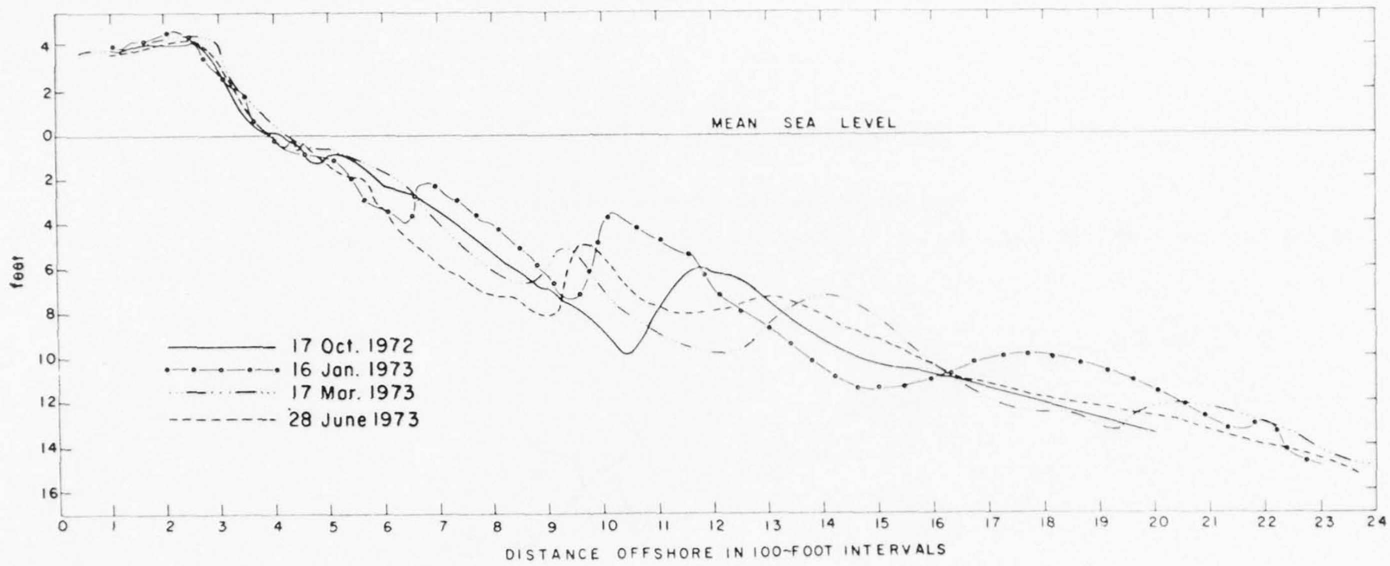

Figure A-10. Beach profile 250S.

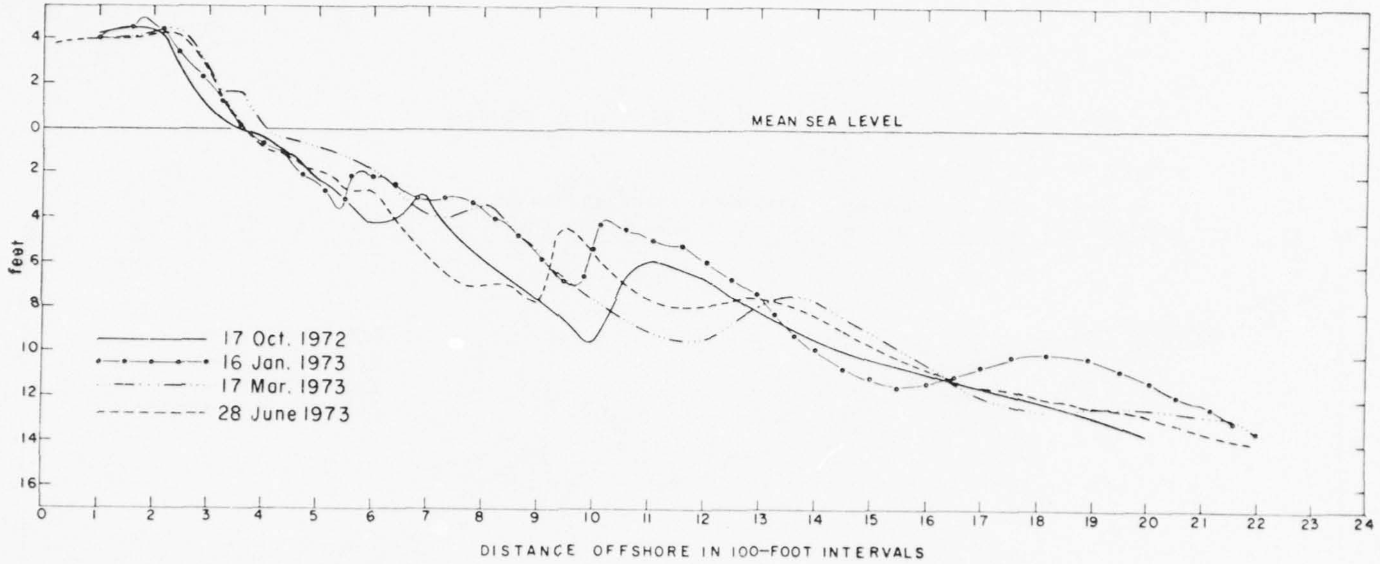

Figure A-11. Beach profile 400S.

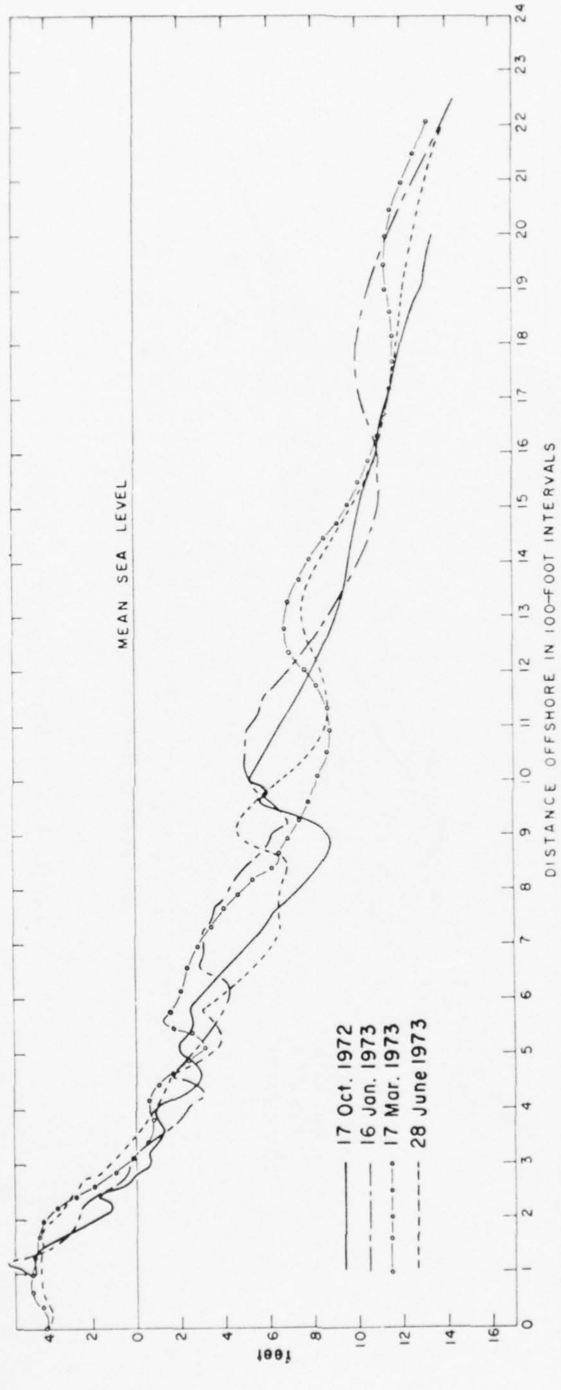

Figure A-12. Beach profile 800S.

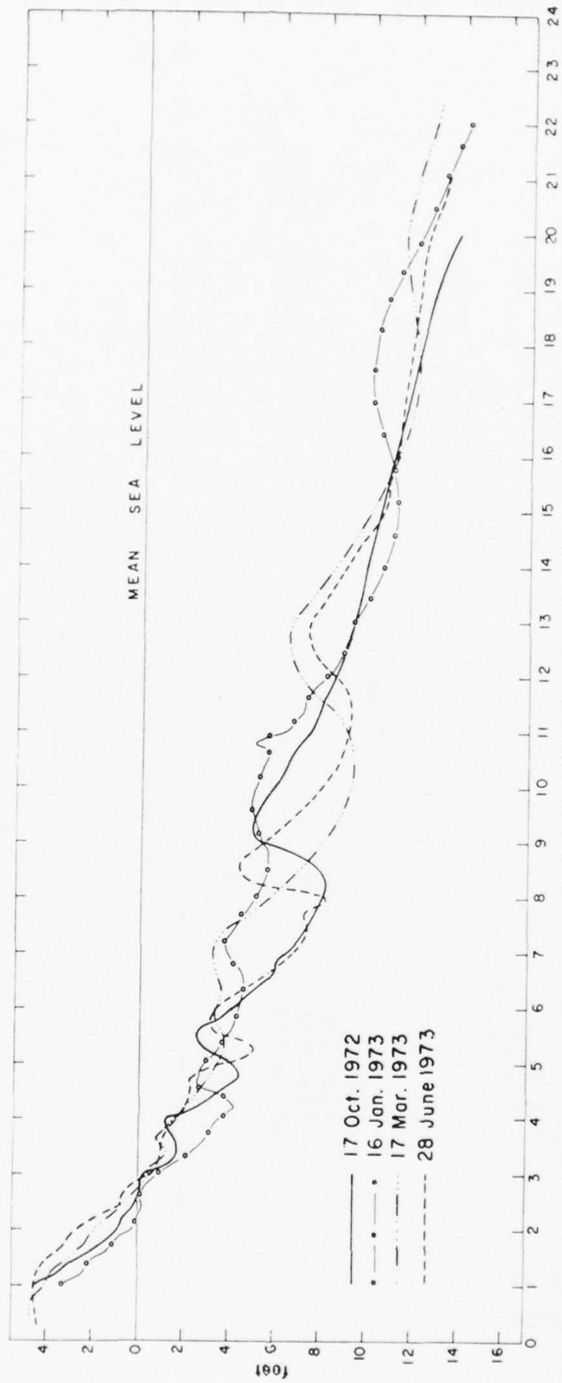

Figure A-13. Beach profile 1200S.

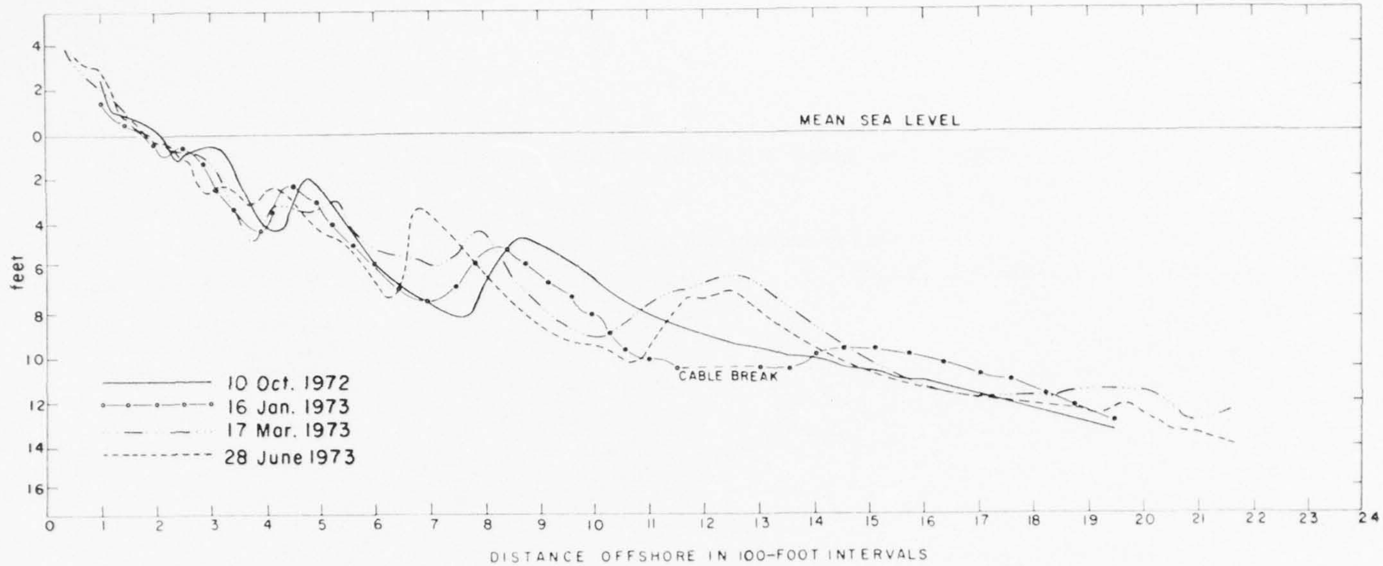

Figure A-14. Beach profile 2000S.

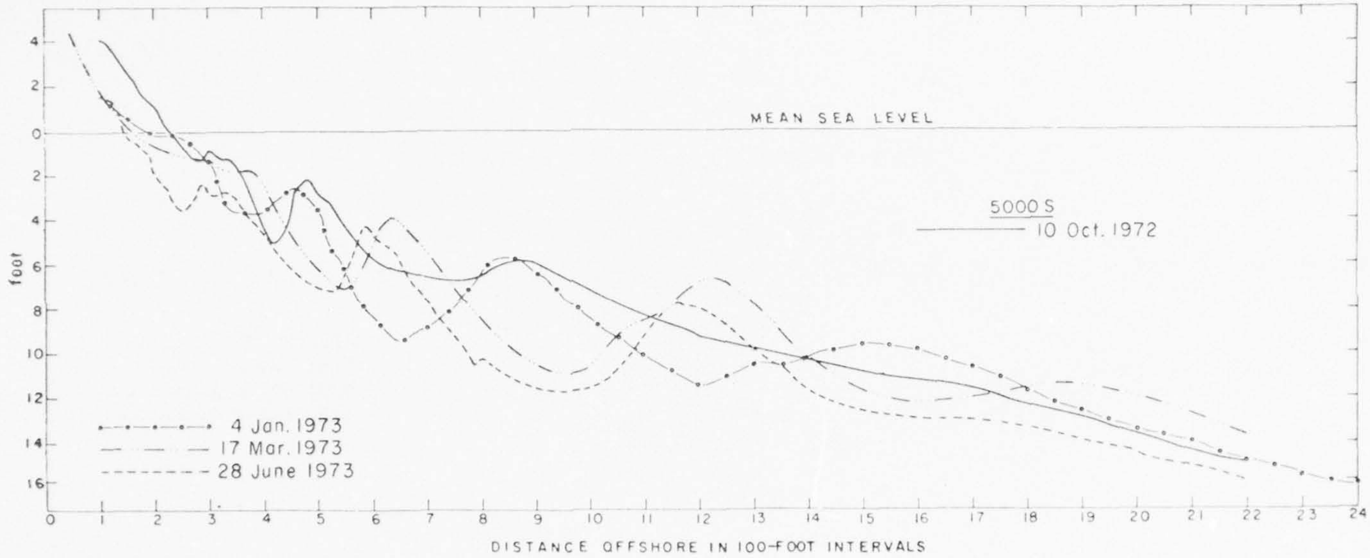

Figure A-15. Beach profiles 4000S and 5000S.

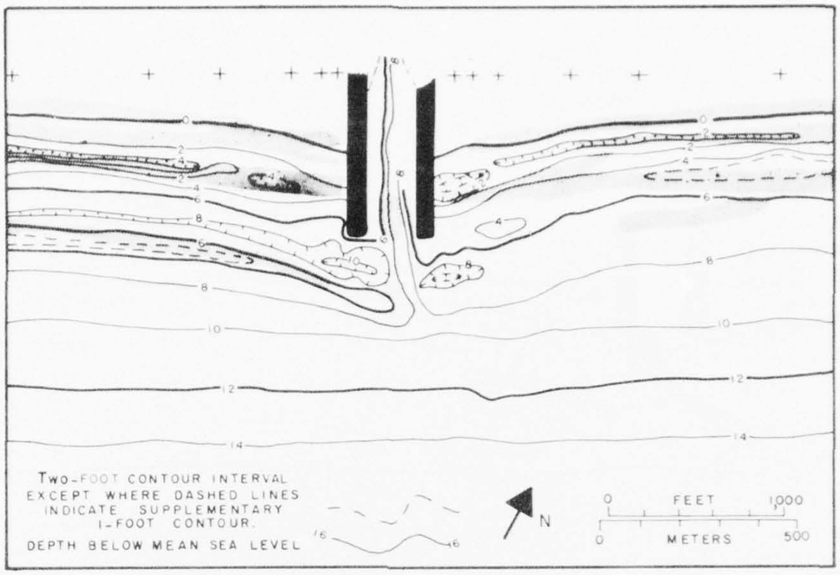

Figure A-16. Gulf survey, 15 September and 9 and 16 October 1972.

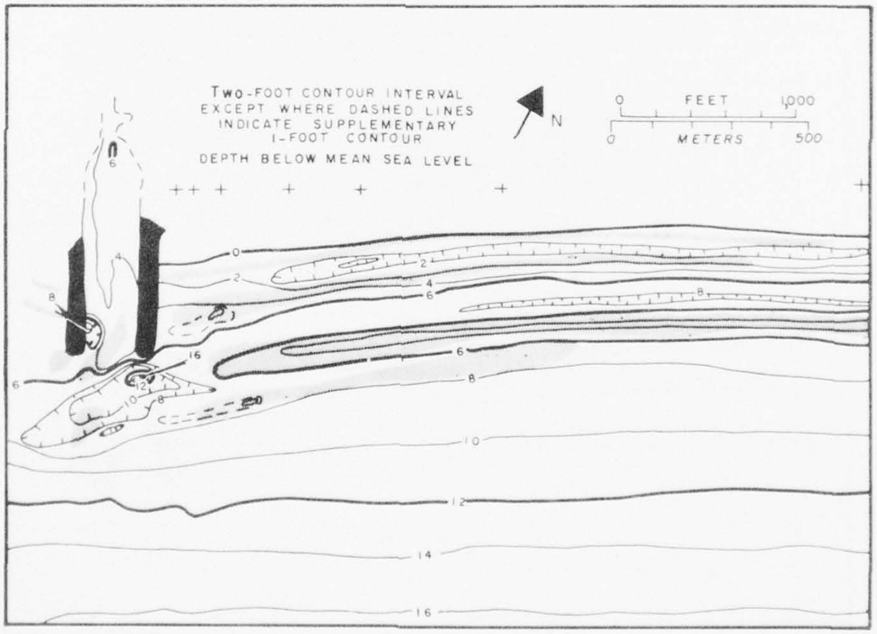

Figure A-17. Gulf survey, 9 November 1972.

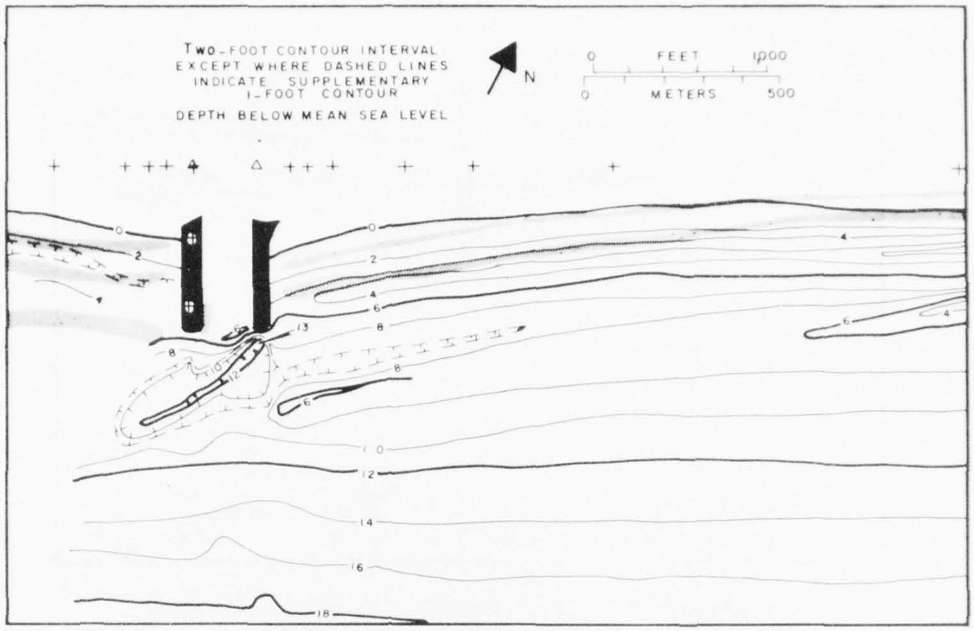

Figure A-18. Gulf survey, 21 and 23 December 1972.

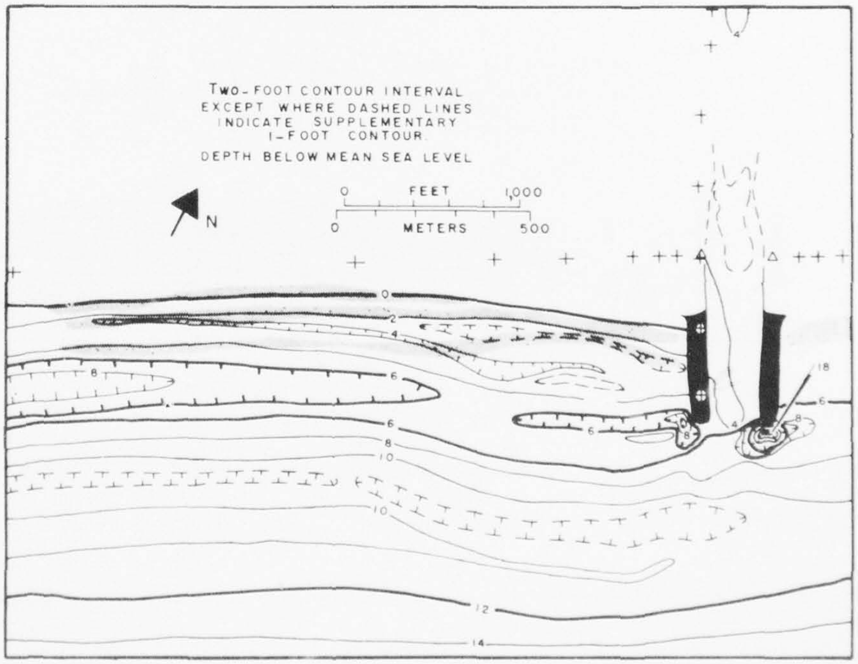

Figure A-19. Gulf survey, 16 and 29 January 1973.

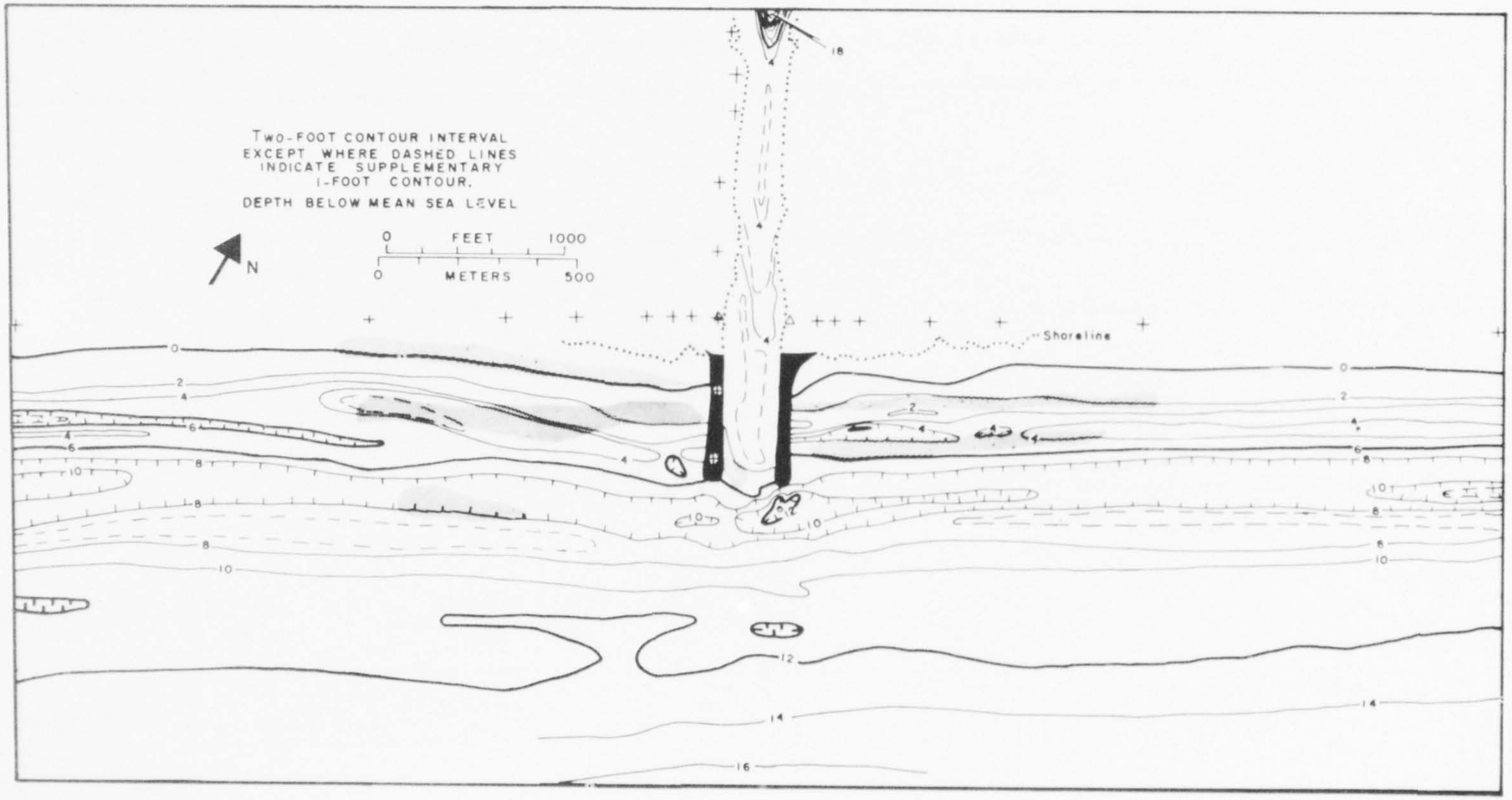

Figure A-20. Gulf survey, 17 March and 4 and 5 April 1973.

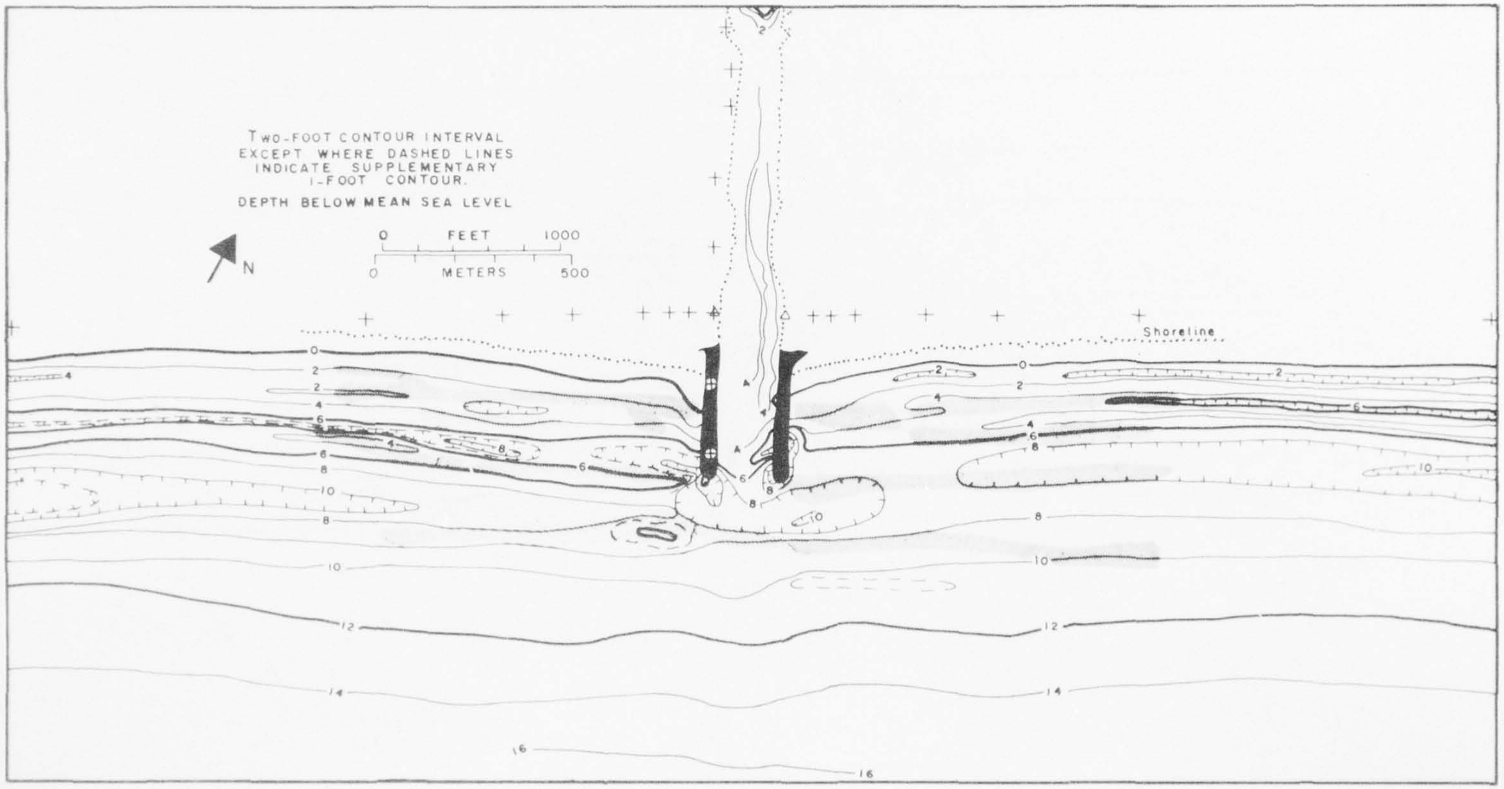

Figure A-21. Gulf survey, 7, 27, and 28 June 1973.

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HYDRAULICS AND DYNAMICS OF NEW CORPUS CHRISTI PASS, TEXAS: A CA--ETC(U)
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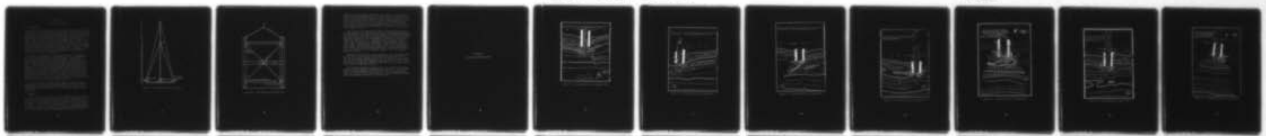

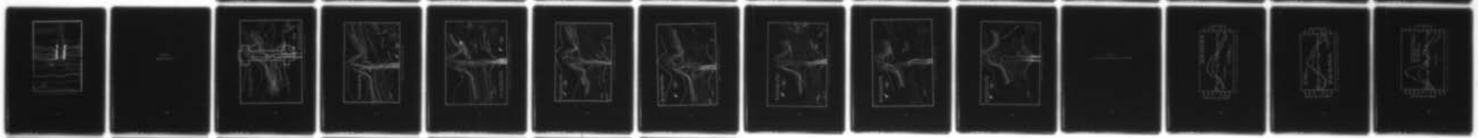

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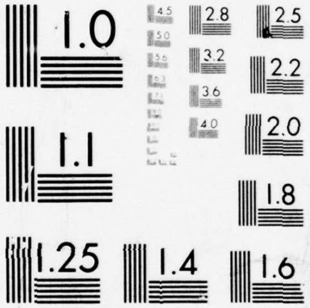


MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

APPENDIX B

SURF PROFILING SLED

1. Construction.

The sled is 12.5 feet long by 10 feet wide with a 30-foot mast. The runners are 3/16-inch steel plates, 1 foot wide, welded to the structure holding the cross supports with angle iron and 1/4-inch bar stock (Figs. B-1 and B-2). The original model used four steel crosspieces, but these were discarded to lighten the sled. The two crosspieces holding the runners apart are thick-walled 1/4-inch aluminum pipes, 3 inches in diameter. The crosspieces are held to the runners by four eyebolts (1/2-inch diameter) which are inserted in holes at the end of the pipes (Fig. B-2). A mast step built from two 2-inch by 6-inch by 10-foot pine planks laminated together with waterproof glue runs across the sled. A wooden box holds the base of the mast in the center of this crossmember. The mast step is bolted at the center of each runner to small platforms welded onto the runners for that purpose.

The mast is constructed of thin-walled (0.065 inch) aluminum pipe and is sleeved in the middle, allowing the mast to be disassembled into two 15-foot sections for ease of transport. Mast material is agricultural irrigation pipe; inserts of polyvinyl chloride pipe are used to strengthen the base and for the sleeve inside the mast at the joint. After 3 months of use, the lower mast section broke during one survey. This lower section should be made of stronger material or strengthened by inserting an appropriately ripped 2- by 4-inch board. Shrouds are 1/8-inch galvanized steel cables with turnbuckles on the bottom to adjust the angle of the mast. From each of the four corners of the assembled runners two sets of shrouds were used; one set connected to the mast just below the joint, and one set connected to the mast near the top. The mast was painted in alternating 1-foot-wide color bands of white, black, day-glow orange, and green.

The towing bridles were made of 5/16-inch wire rope with swivels attached for the towing rope to prevent kinking. The entire sled was disassembled and transported to the beach on a trailer. Cost of construction and maintenance of the sled for the first 6 months totaled about $1,500.

2. Operation.

A 36-foot trawler was used to tow the sled through the surf zone to the most seaward position on the profiles. Initial contact between the boat and the sled was usually achieved by sending a small boat as far shoreward as possible from the trawler (usually within about 600 feet of the beach). A light line was fired ashore with a line-throwing gun and the towline (1,200 feet long and 1/2 inch wide) was connected between the trawler and sled. The trawler then towed the sled seaward along the profile line using land-based range markers for navigation. After

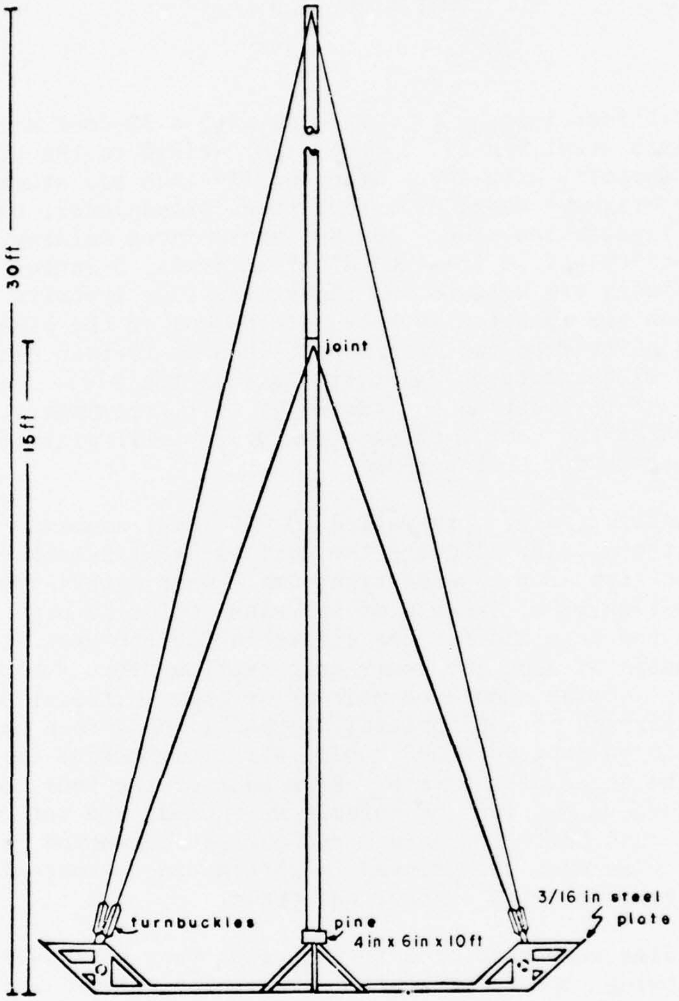


Figure B-1. Surf profiling sled, side view.

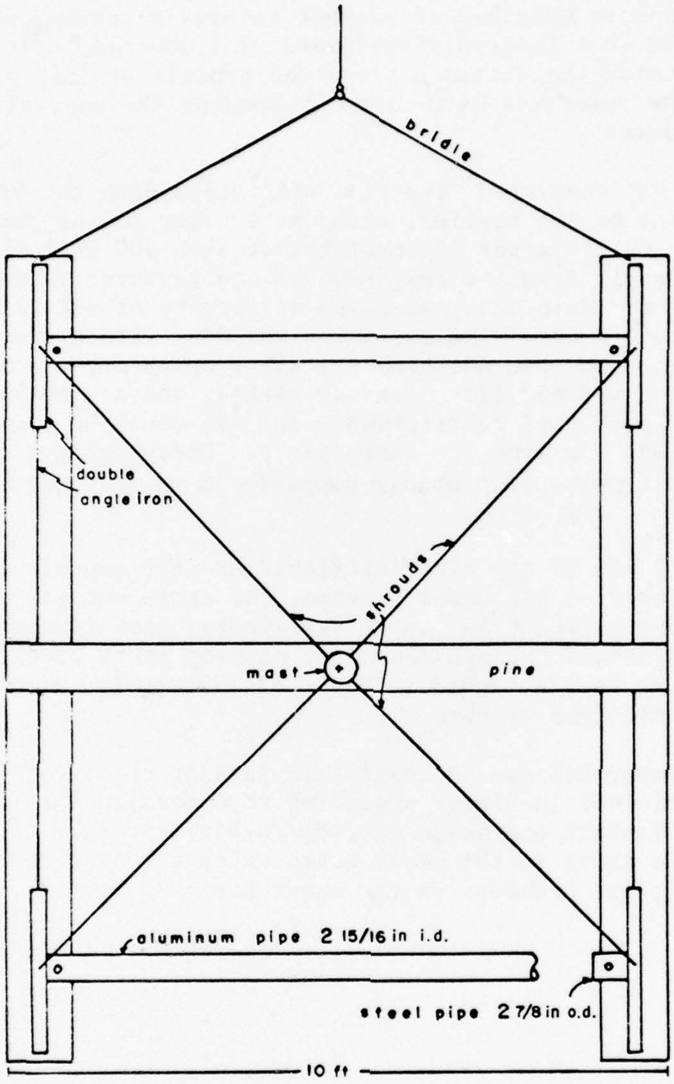


Figure B-2. Surf profiling sled, plan view.

reaching the seaward end of the profile, the sled was towed toward the beach by the trailer-mounted winch. Care was taken to maintain tension in the tow rope to prevent longshore currents from pulling the sled off range. Data points were obtained at 50-foot intervals seaward of 1,000 feet offshore and 20-foot intervals shoreward of 1,000 feet offshore. A meter wheel indicated the distance along the profile of each point; the elevation of the point was determined by reading the mast elevation with a surveyor's level.

Once a profile was completed, the sled was pulled down the beach to the next profile line by the trailer, with the trawler moving parallel to the sled offshore. On traverses closer together than 400 feet the trawler pulled the sled directly from the beach end of one traverse to the offshore end of the next. This eliminated the difficulty of establishing contact more than once a day. The operation was done with a crew of eight: One boat operator, two deckhands, a winch operator, a level man, a note keeper, a wire watcher (for distance marks), and a wire level wind watcher. With personnel restrictions, one man could be eliminated from the boat crew and one from the shore party. Under optimum conditions, an eight-man crew could probably complete 10 to 12, 2,500-foot profiles in 1 day.

During the first use of the sled, difficulties were experienced with the tow cable. As the sled was towed seaward, the cable cut its way into the ever-present sandbars and as it was winched back towards shore, the wire was pulled upward through the sand, causing it to break. Therefore, empty oil drums were attached to the cable 100 to 500 feet from the sled to keep the cable from sinking.

The operation described was successful in seas of up to 4 feet and could have been performed in higher waves (up to 6 feet). The limiting factor is not the offshore operation but the initial exchange of the towing line from the shore to the small boat, which was hazardous during high wave conditions and breakers on the outer bar.

APPENDIX C

GULF MOUTH BATHYMETRIC MAPS

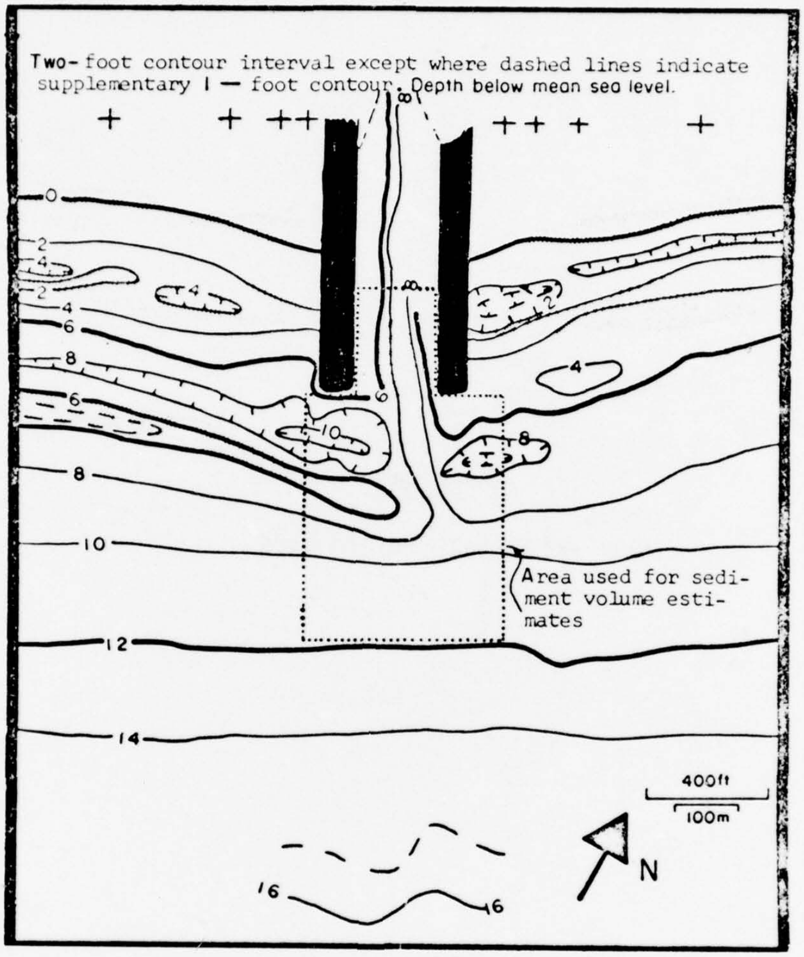
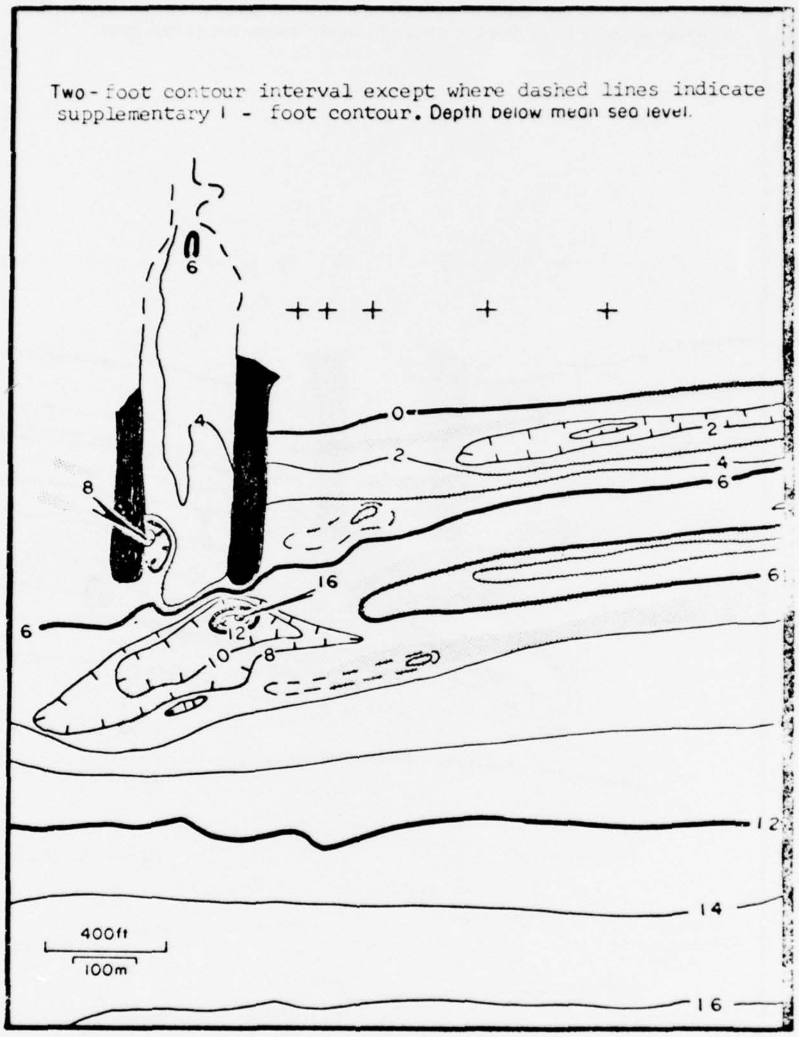


Figure C-1. Gulf mouth survey, 15 September 1972.



Two-foot contour interval except where dashed lines indicate supplementary 1 - foot contour. Depth below mean sea level.

Figure C-2. Gulf mouth survey, 26 November 1972.

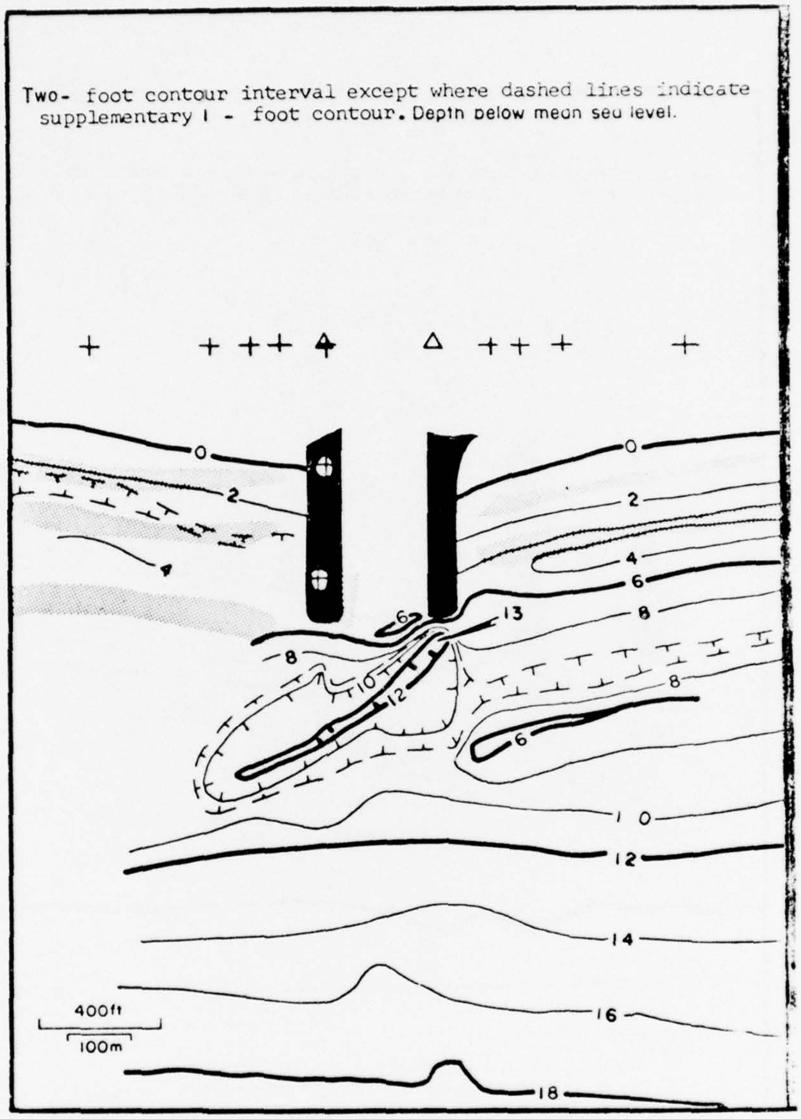
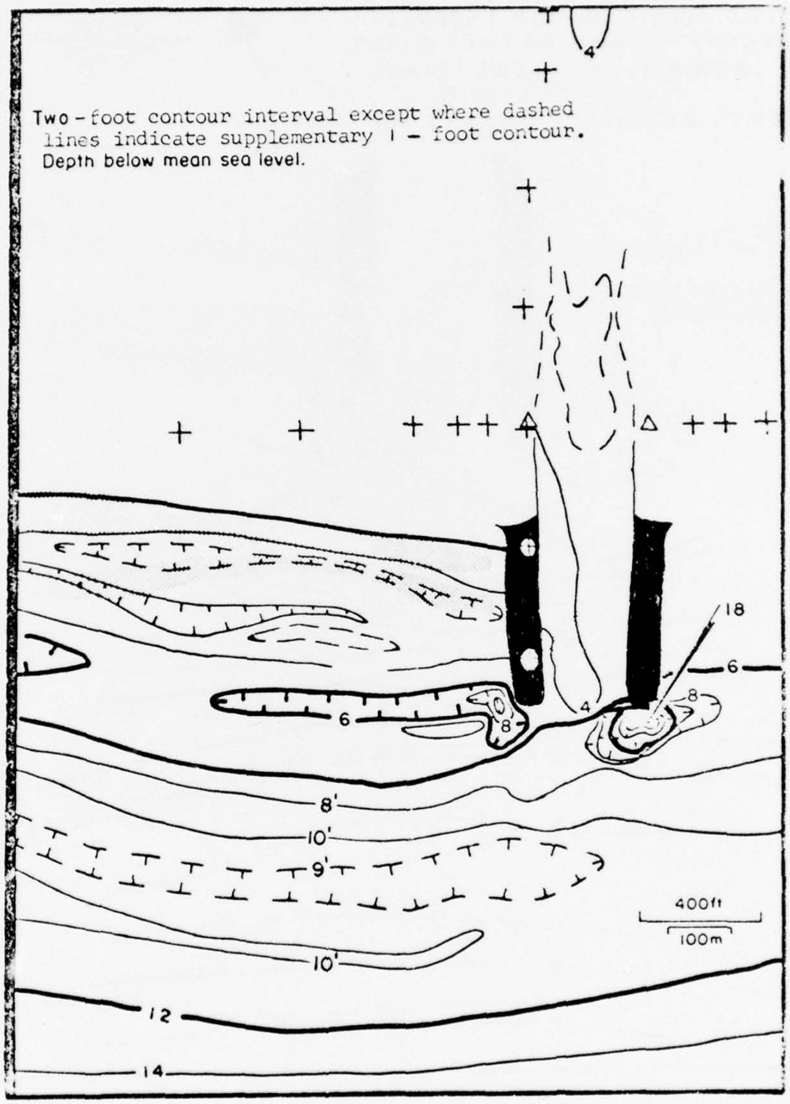


Figure C-3. Gulf mouth survey, 22 December 1972.



Two-foot contour interval except where dashed lines indicate supplementary 1-foot contour. Depth below mean sea level.

Figure C-4. Gulf mouth survey, 29 January 1973.

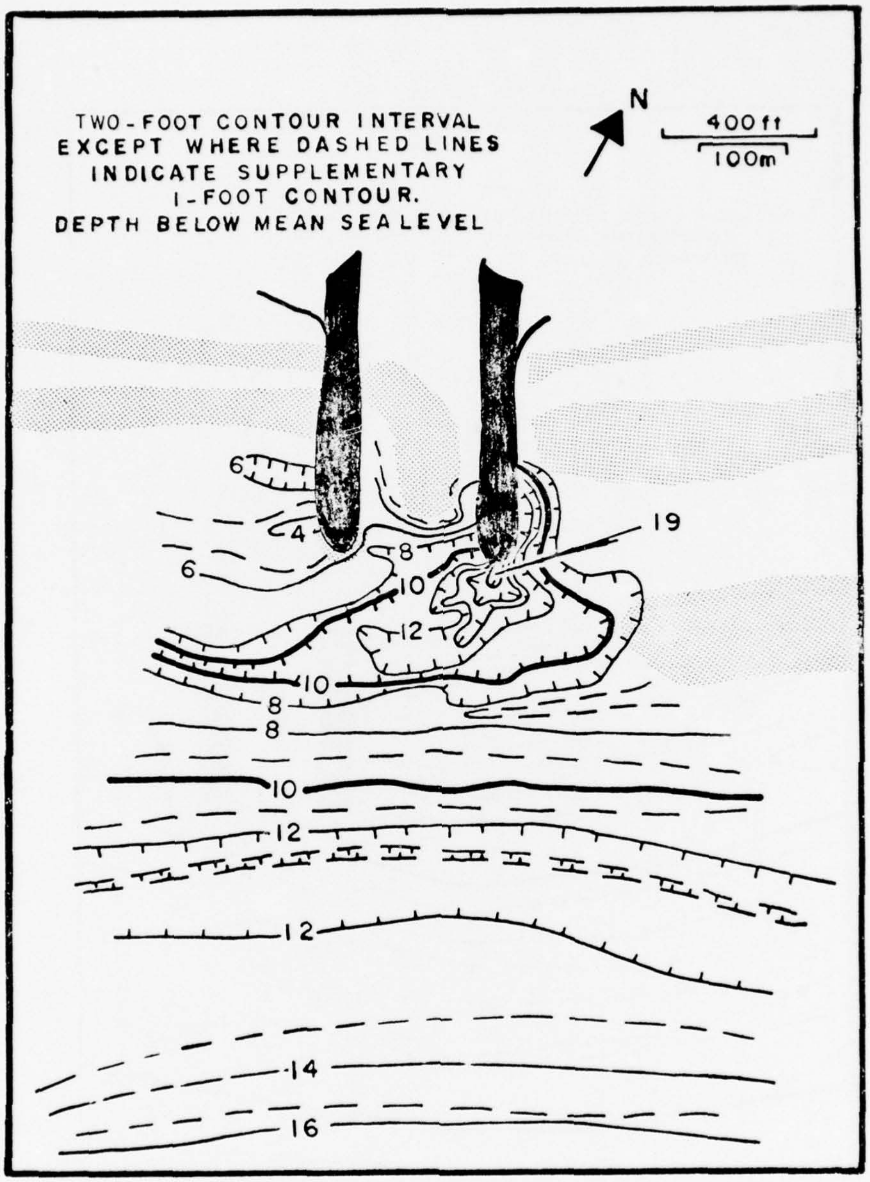


Figure C-5. Gulf mouth survey, 28 February 1973.

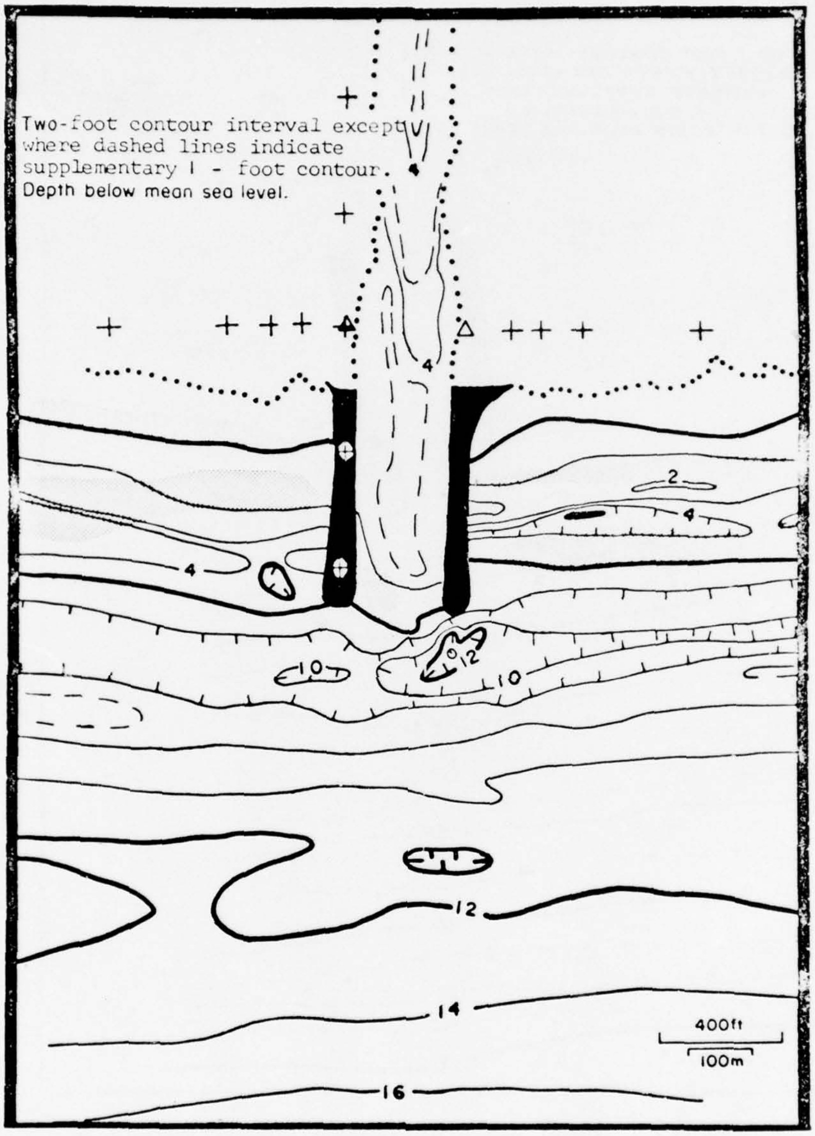


Figure C-6. Gulf mouth survey, 5 April 1973.

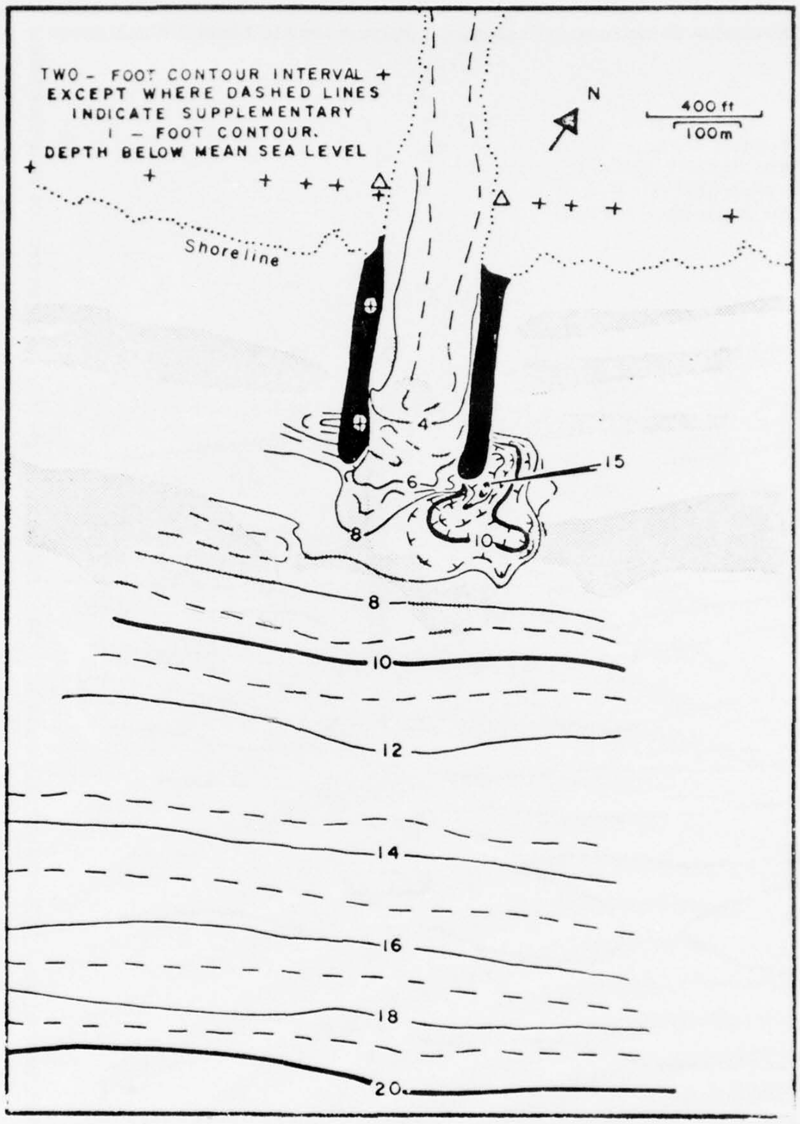


Figure C-7. Gulf mouth survey, 5 and 16 May 1973.

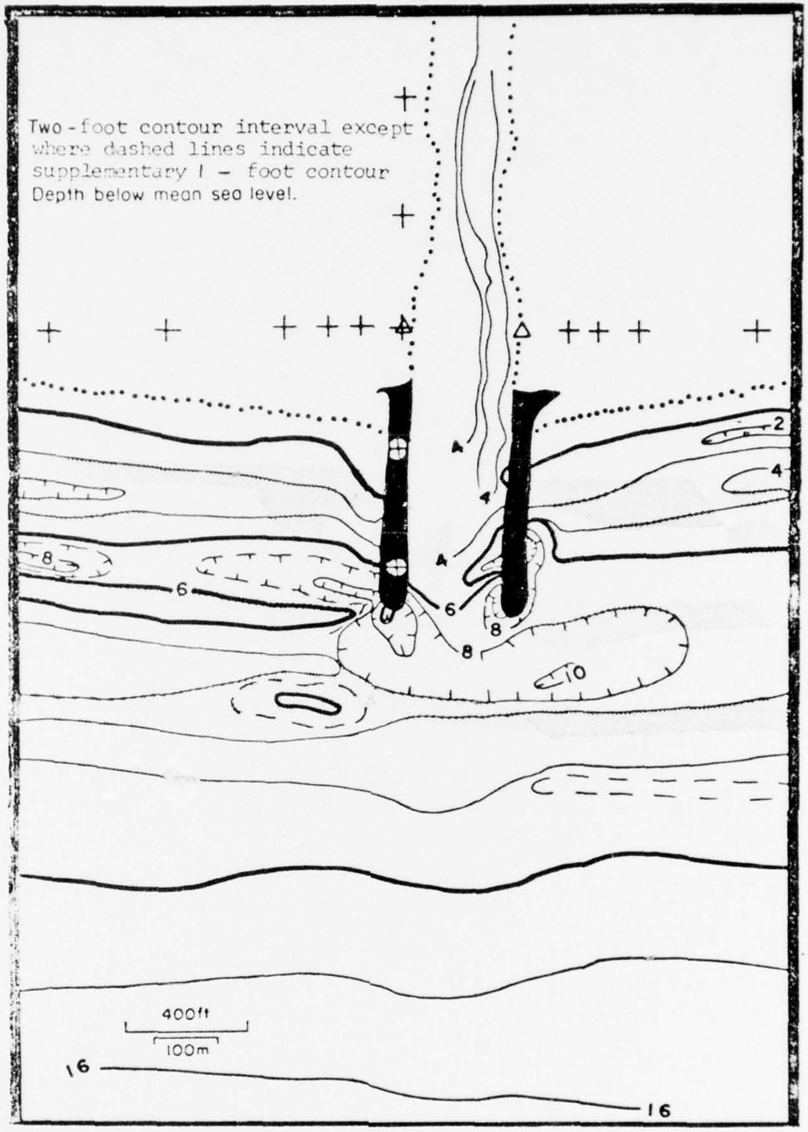


Figure C-8. Gulf mouth survey, 7 June 1973.

APPENDIX D

BAYMOUTH BATHYMETRY

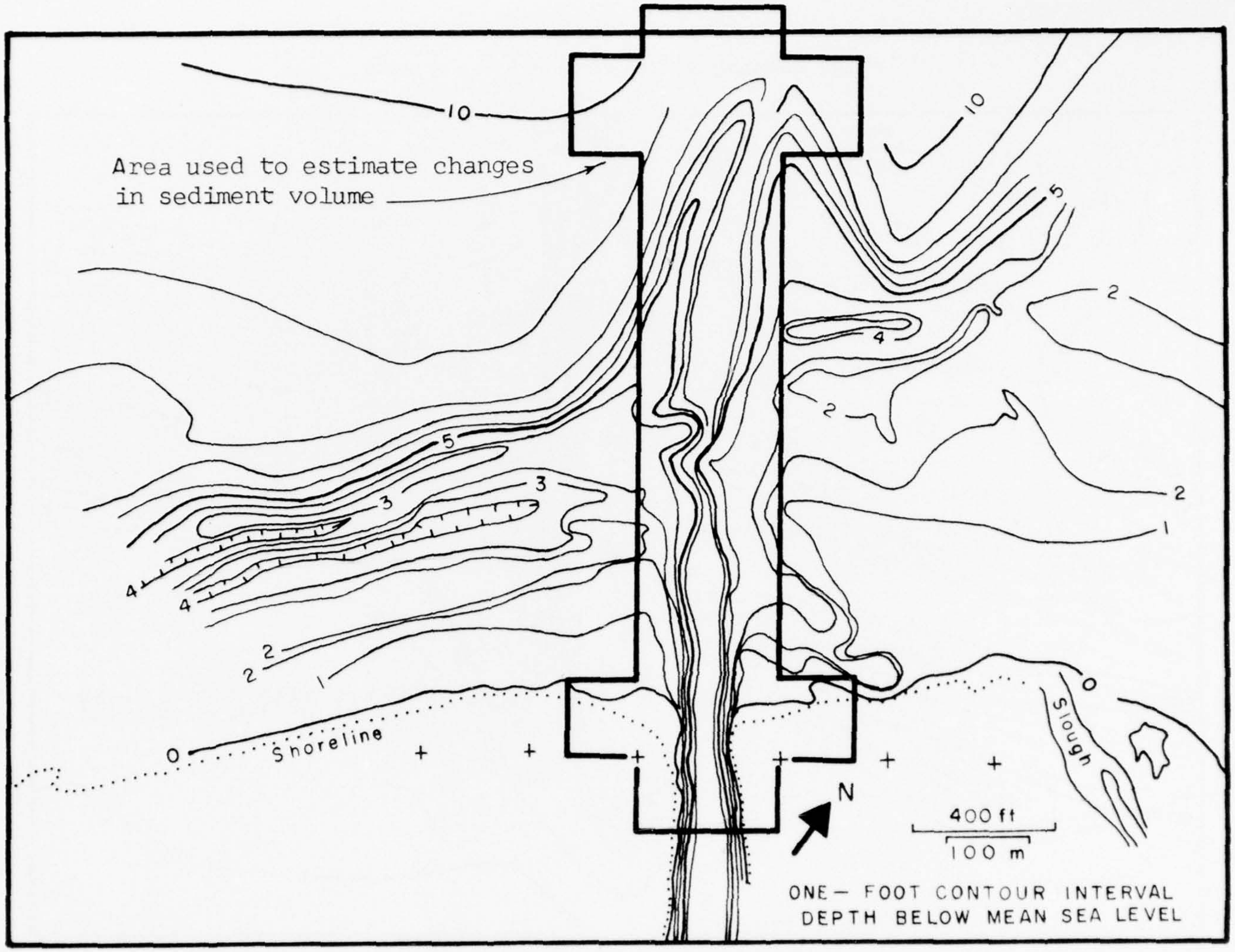


Figure D-1. Bay survey, 27 July 1972 (preopening condition).

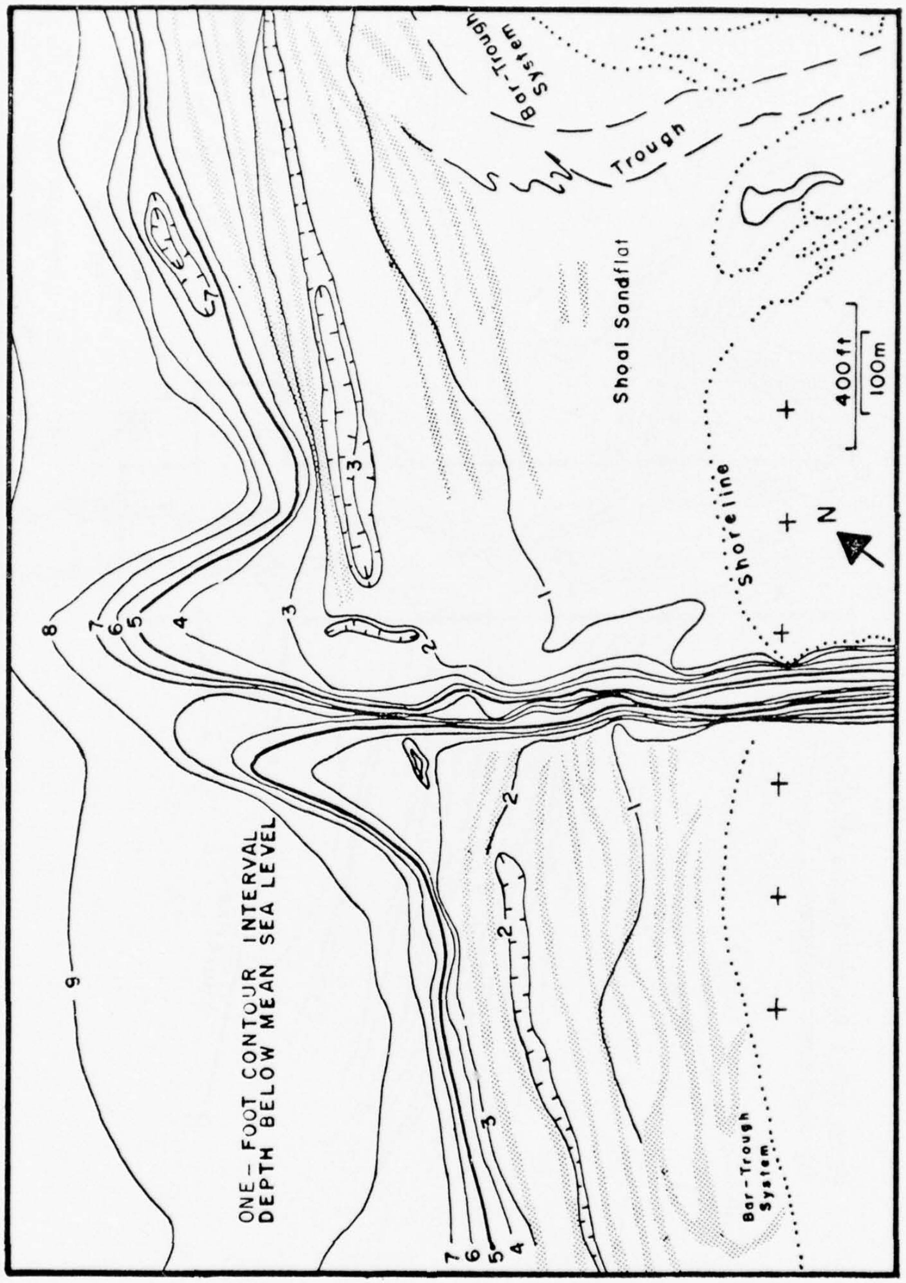


Figure D-2. Bay survey, 16 October 1972.

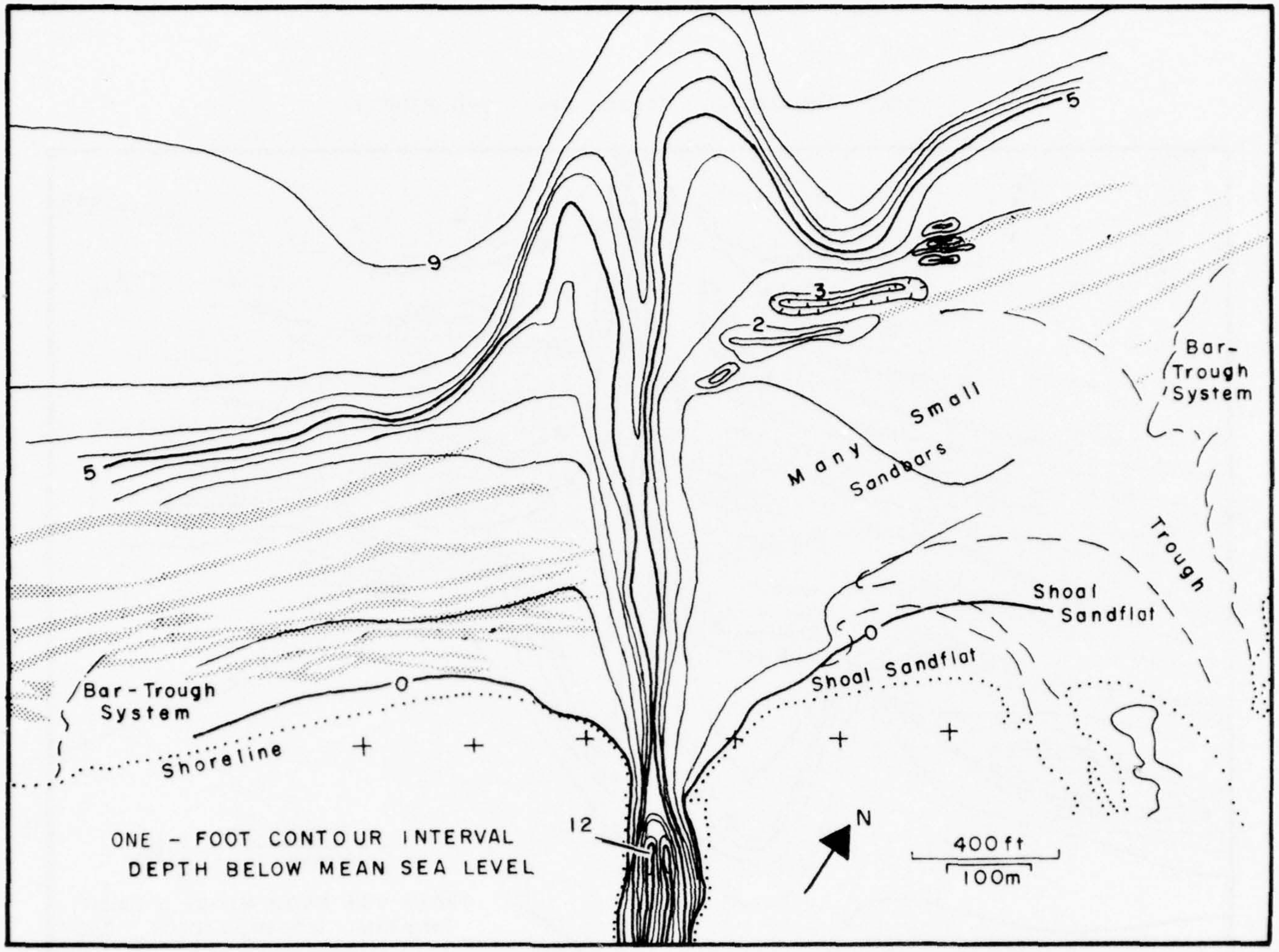


Figure D-3. Bay survey, 27 November and 1 December 1972.

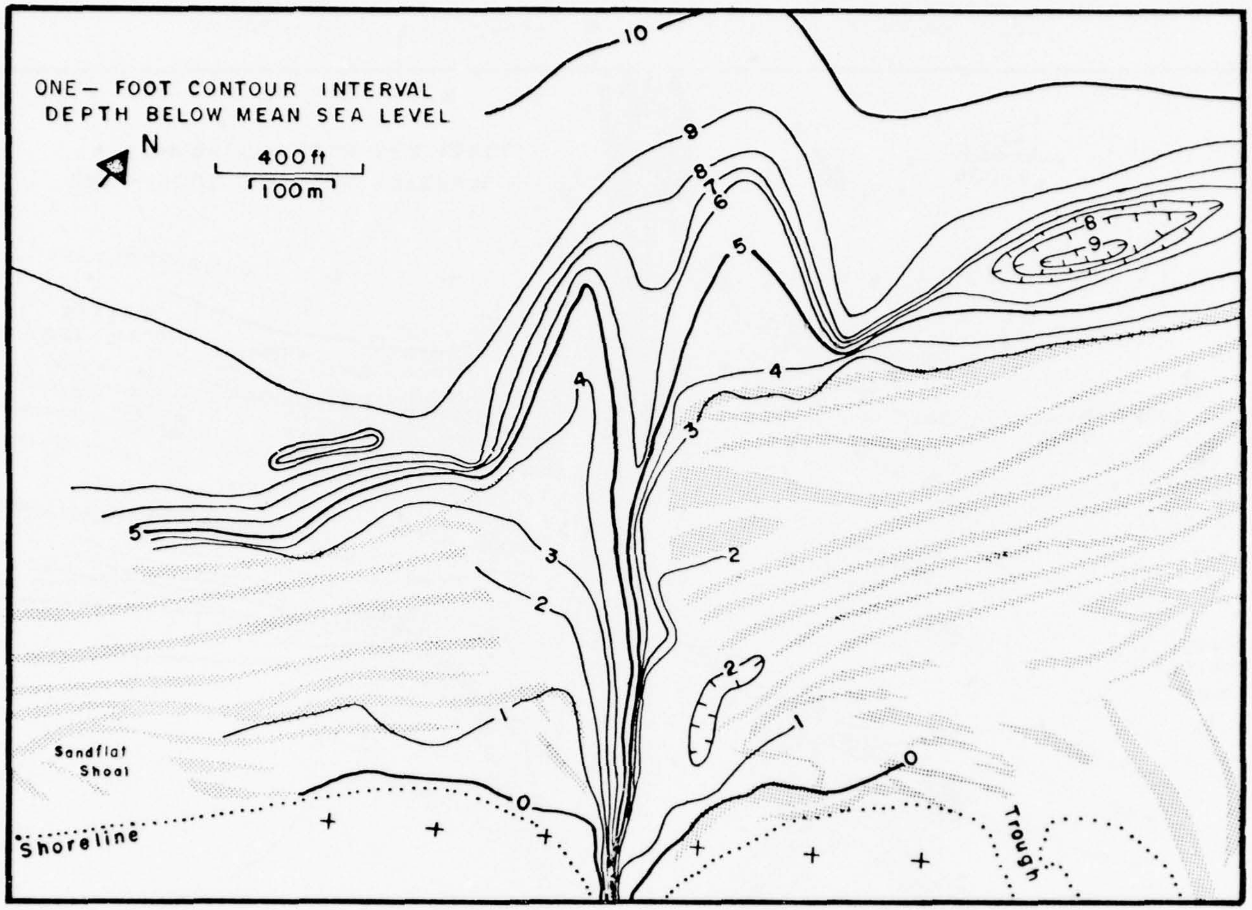


Figure D-4. Bay survey, 18 December 1972.

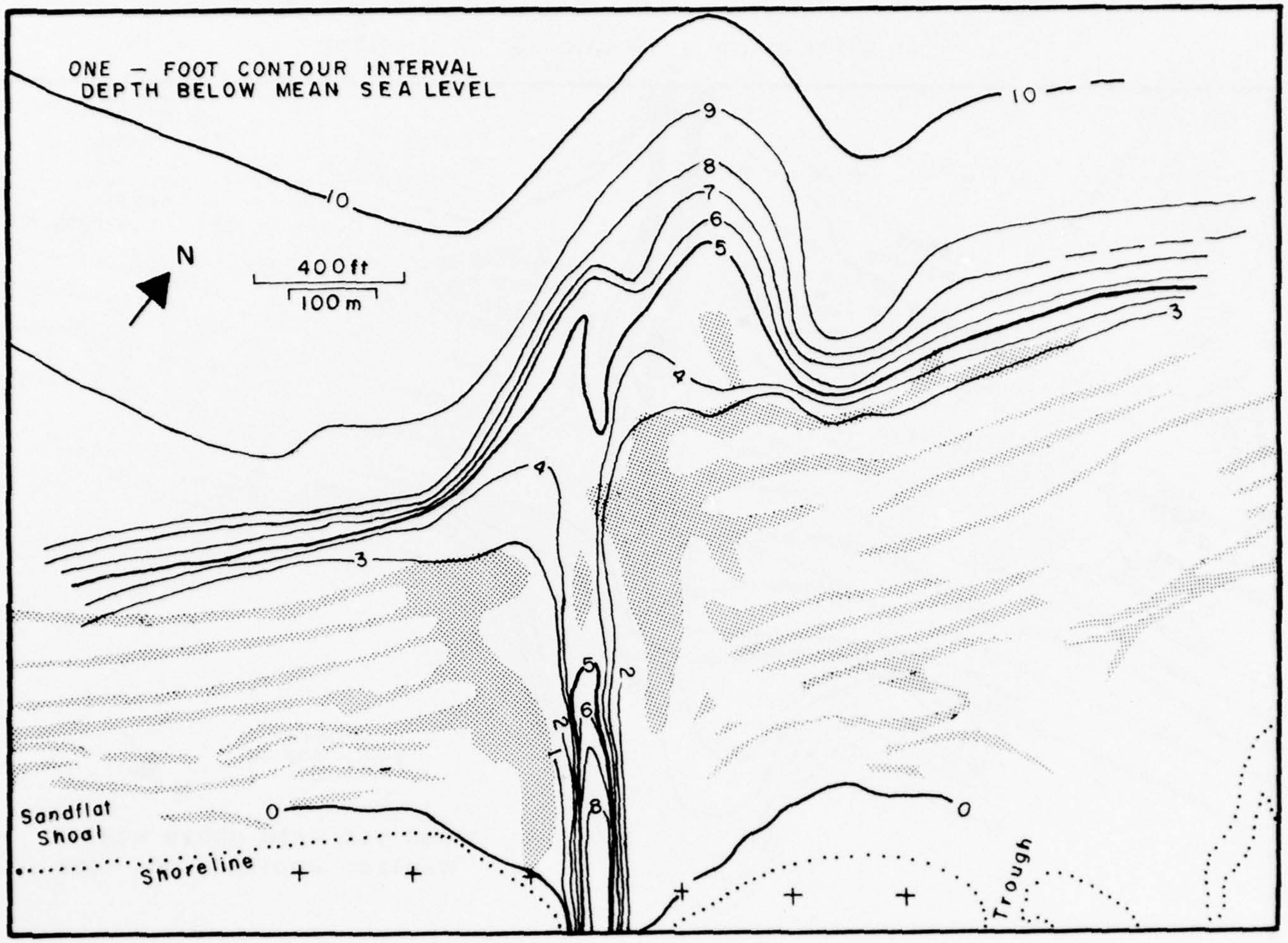


Figure D-5. Bay survey, 27 January 1973.

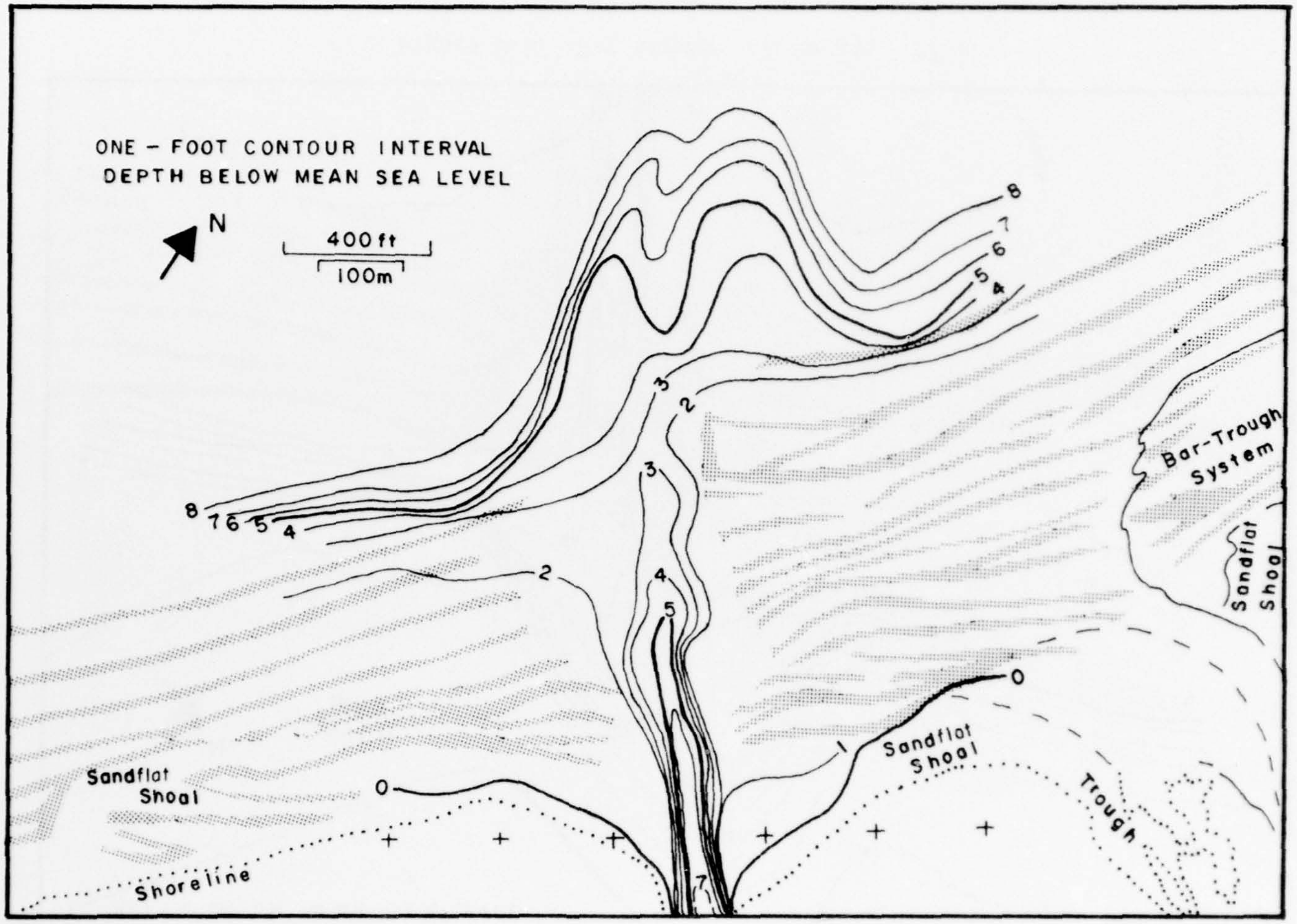


Figure D-6. Bay survey, 7 and 26 March 1973.

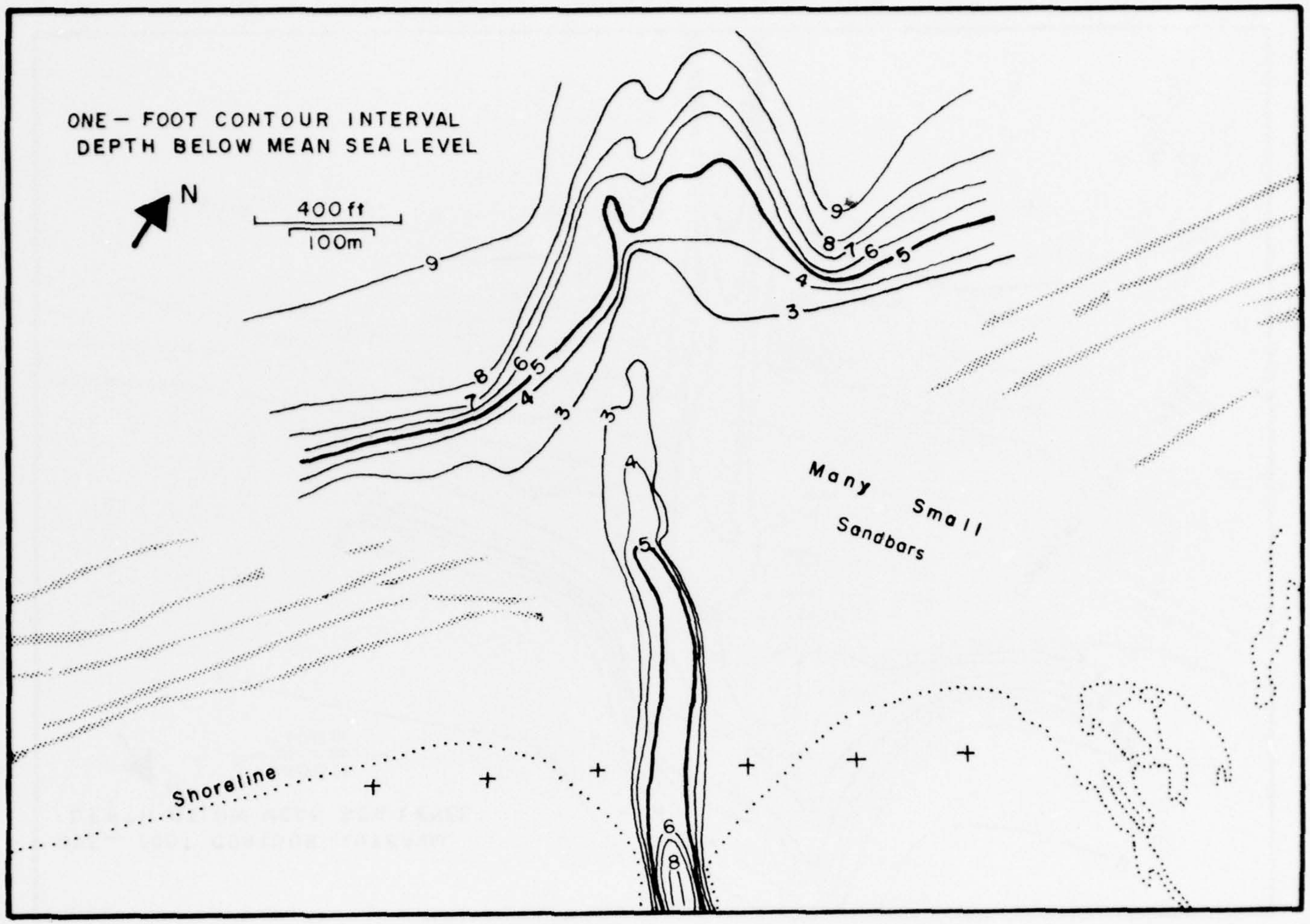


Figure D-7. Bay survey, 12 April 1973.

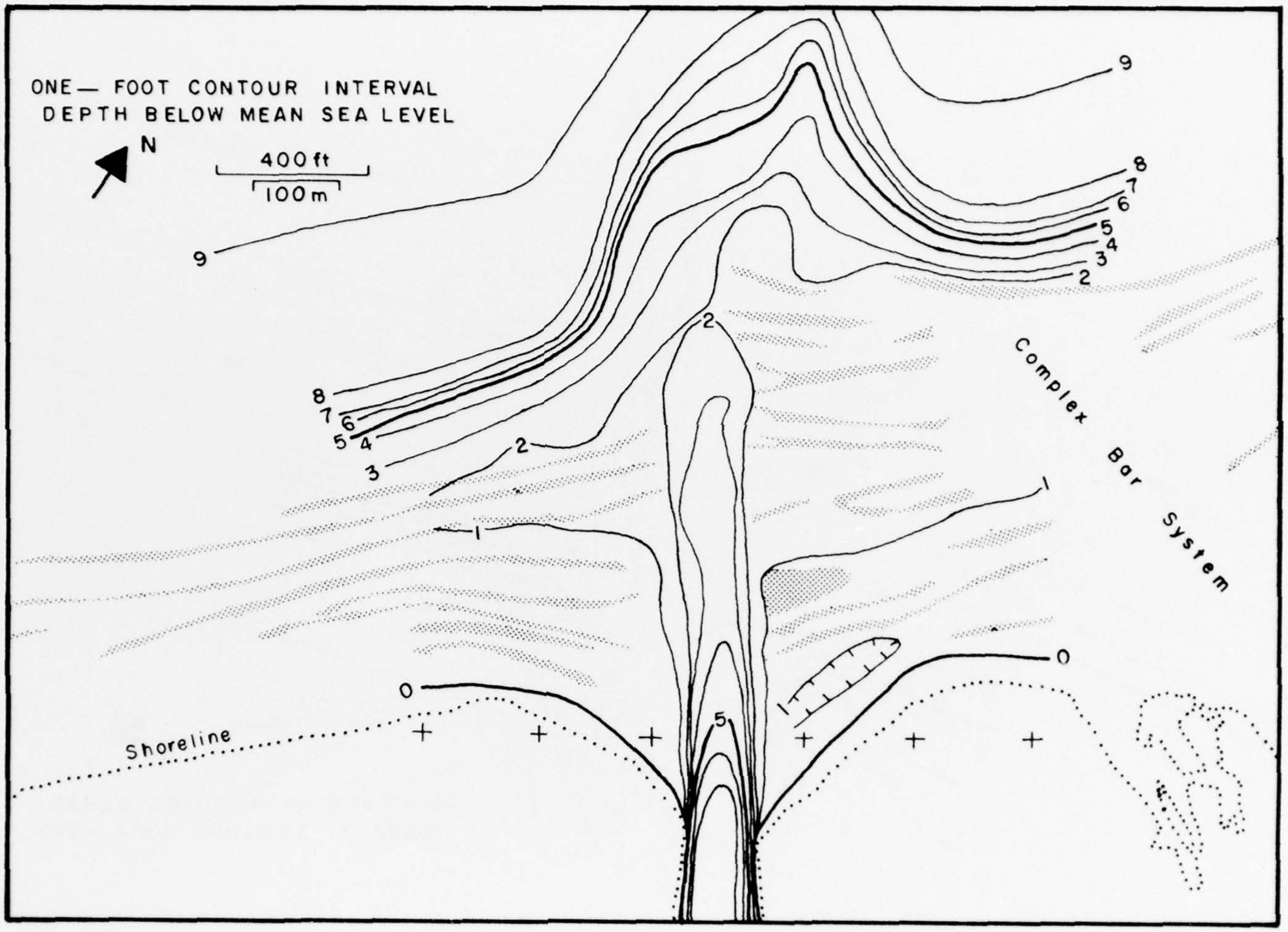


Figure D-8. Bay survey, 16 May 1973.

APPENDIX E

TIDAL DISCHARGE AND DIFFERENTIAL TIME HISTORIES

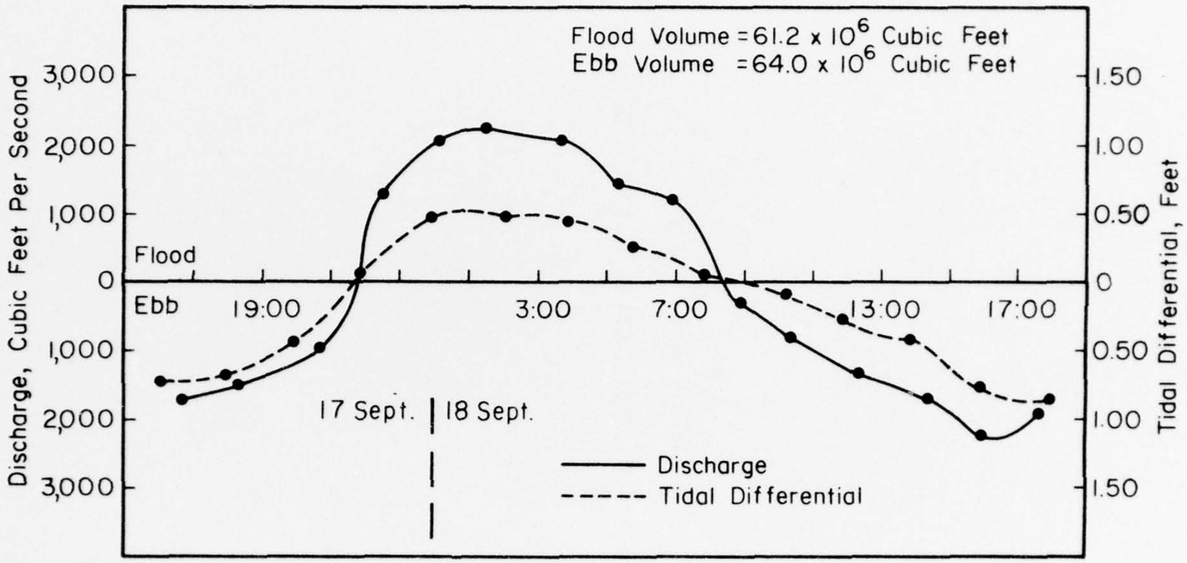


Figure E-1. Tidal differential and discharge, 17 and 18 September 1972.

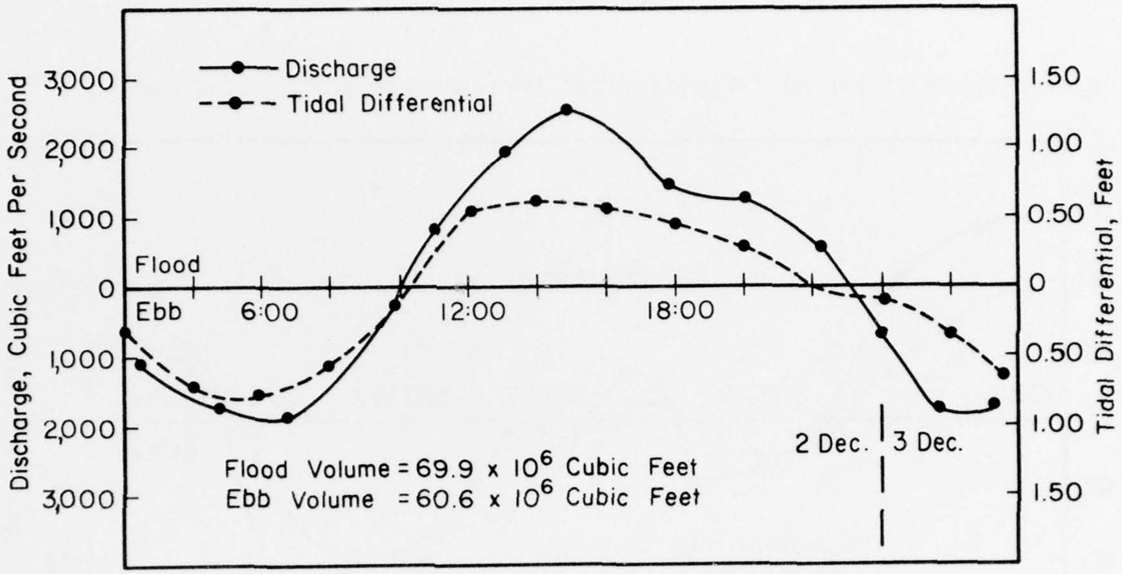


Figure E-2. Tidal differential and discharge, 2 and 3 December 1972.

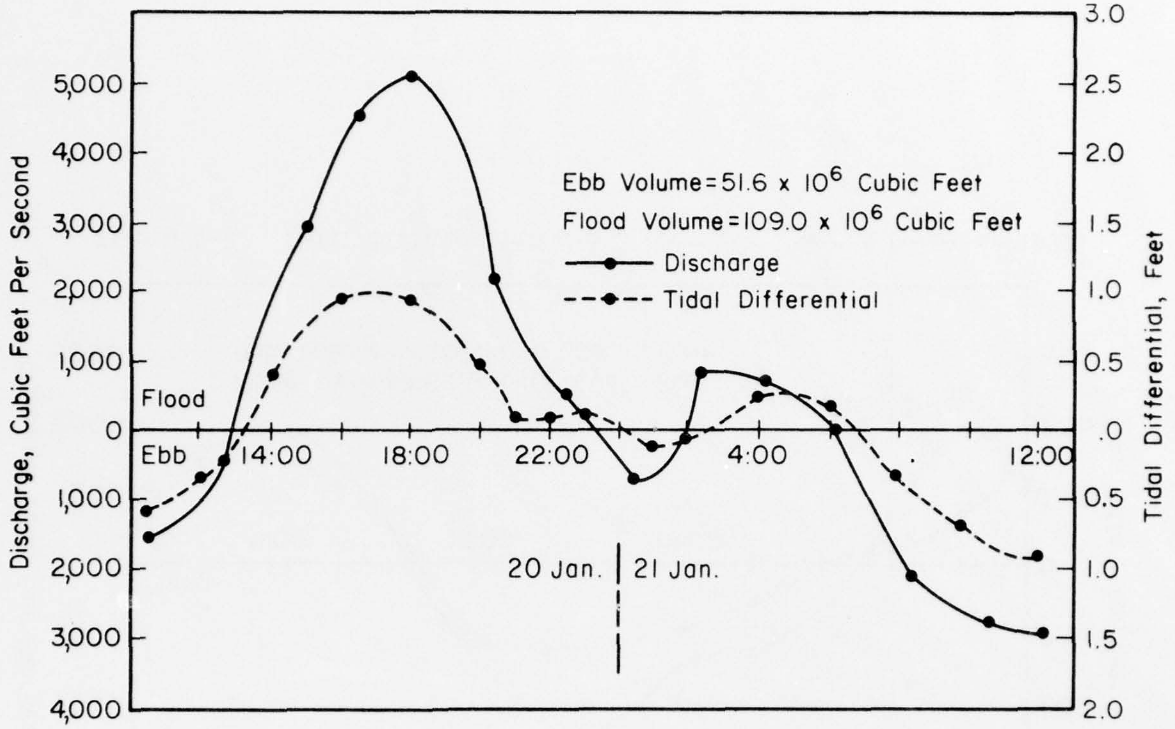


Figure E-3. Tidal differential and discharge, 20 and 21 January 1973.

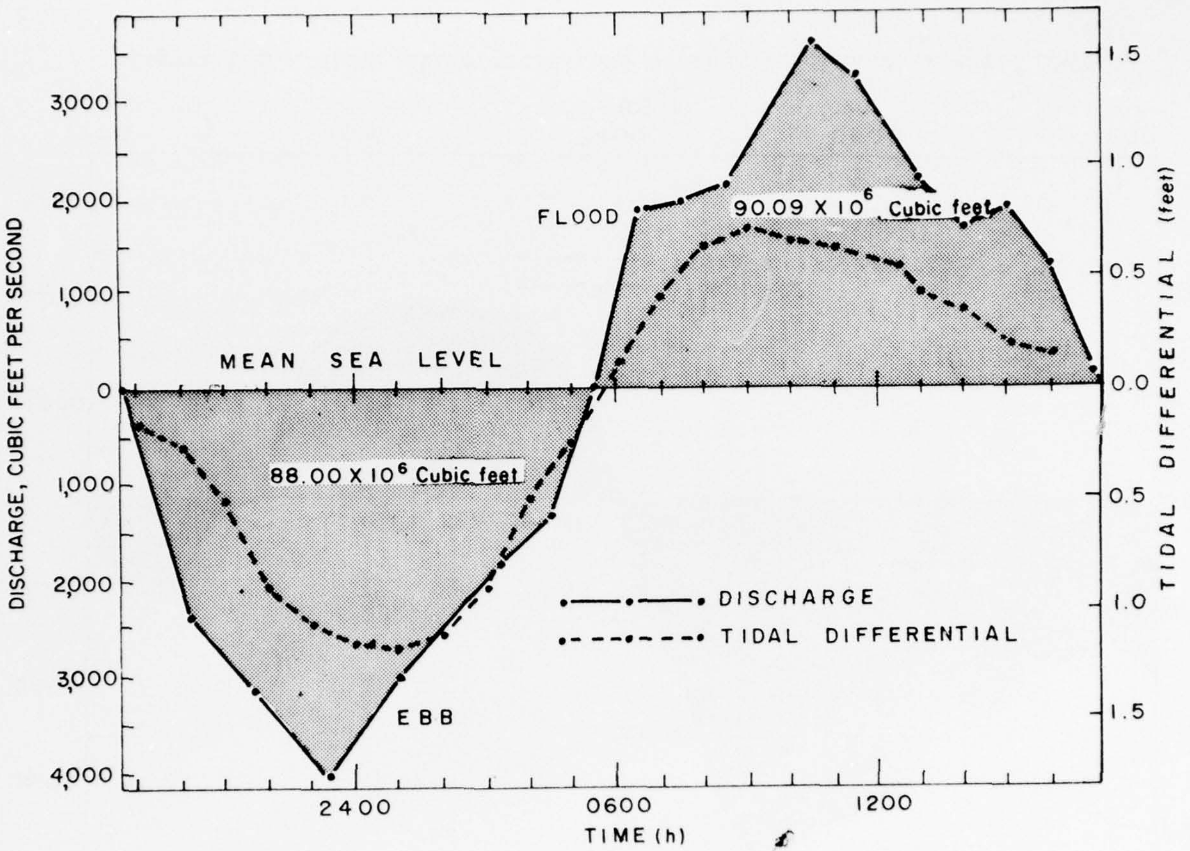


Figure E-4. Tidal differential and discharge, 10 and 11 March 1973.

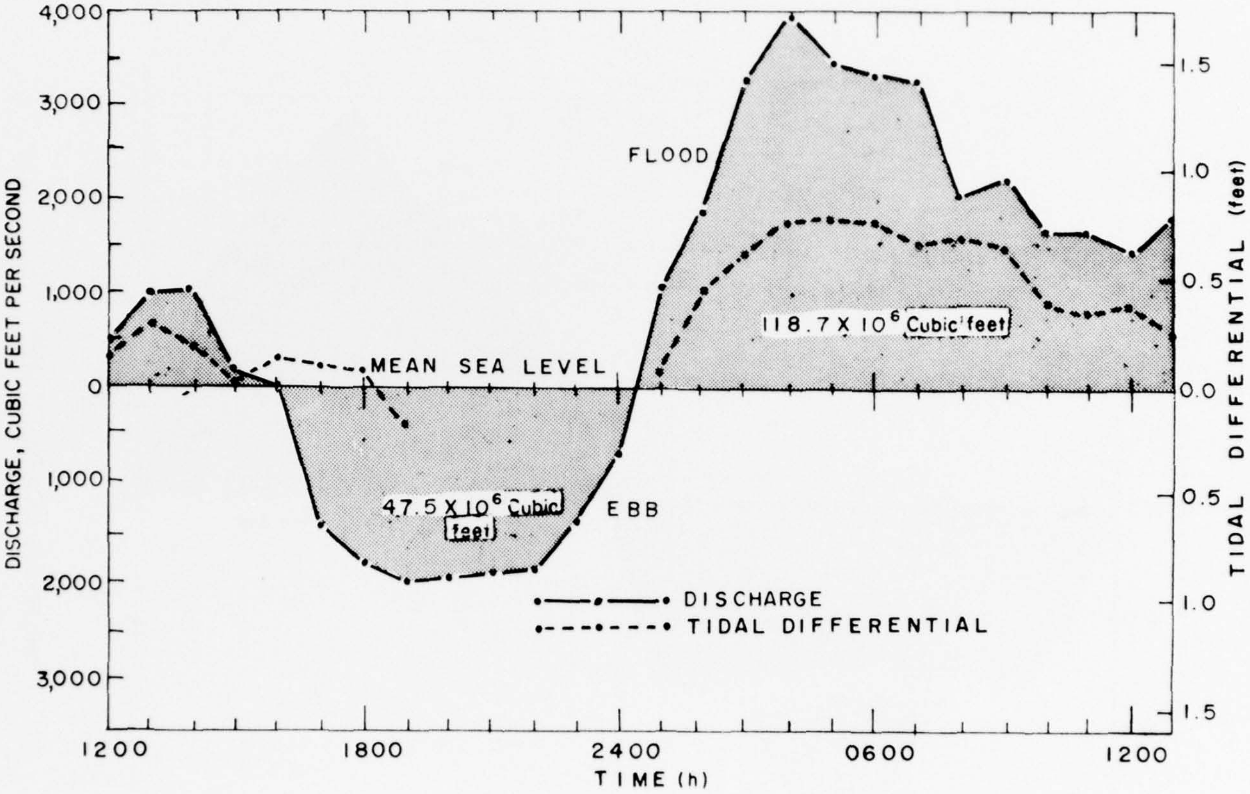


Figure E-5. Tidal differential and discharge, 18 and 19 April 1973.

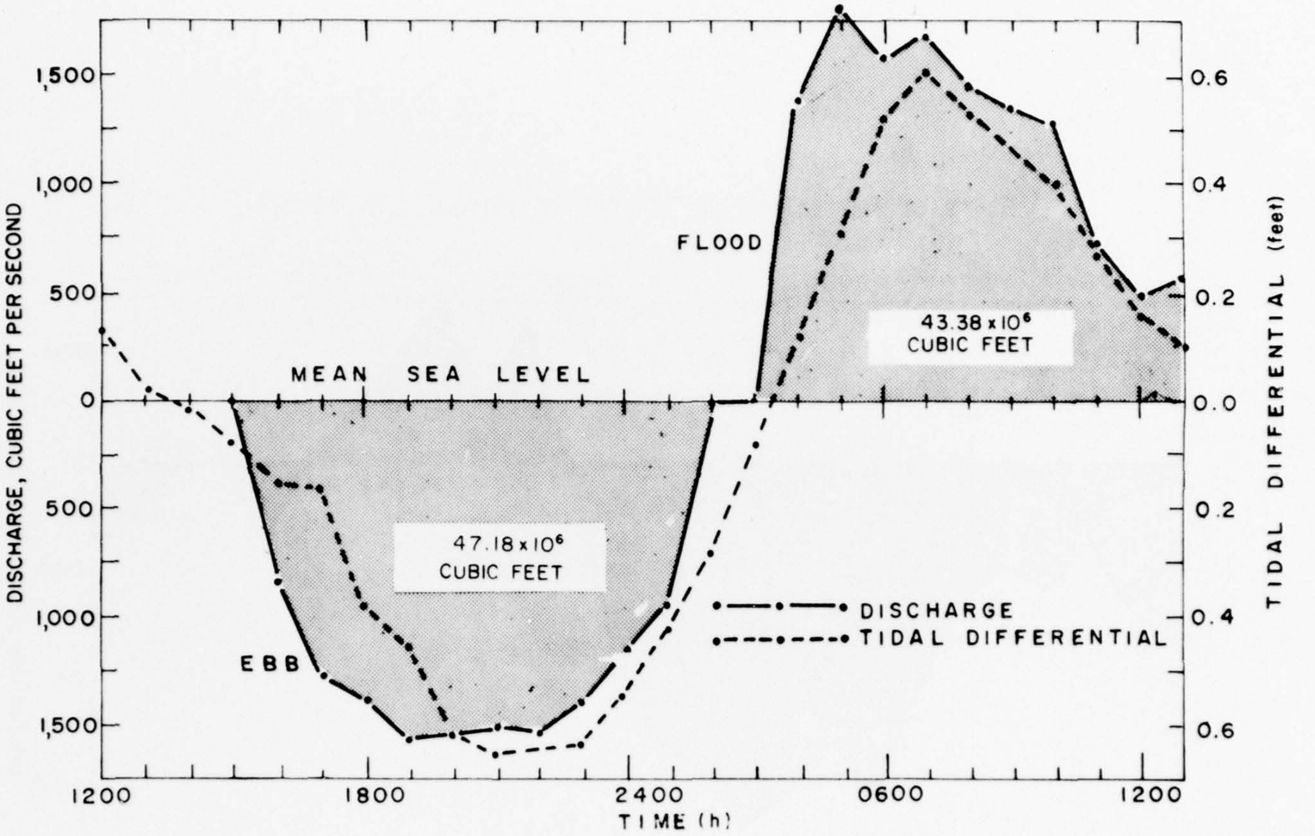


Figure E-6. Tidal differential and discharge, 17 and 18 May 1973.

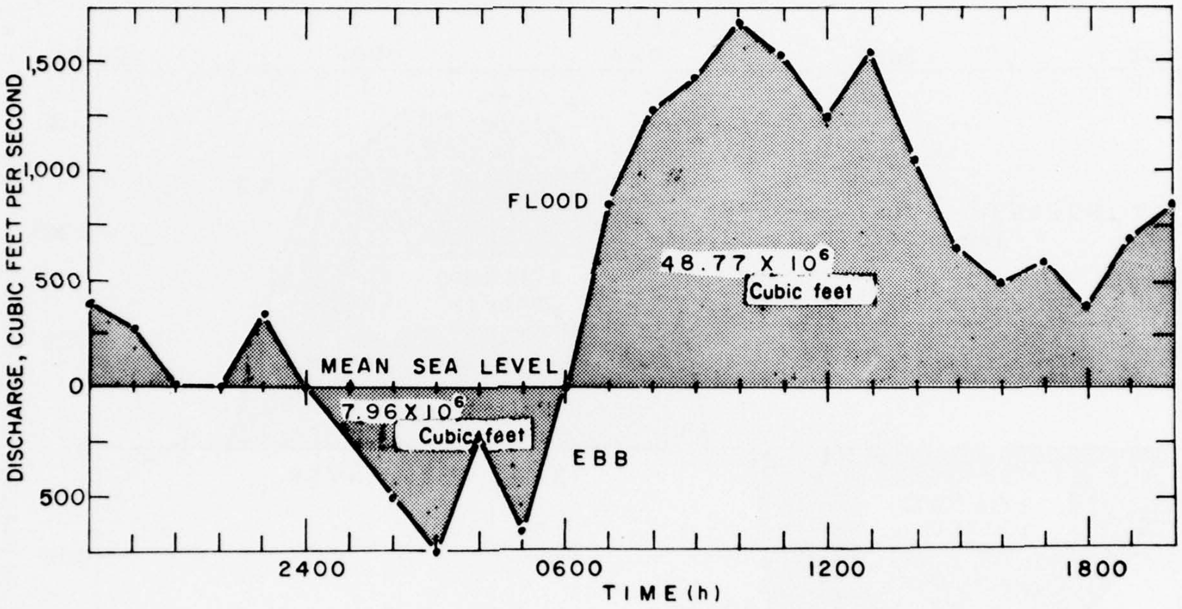


Figure E-7. Tidal discharge, 25 and 26 May 1973.

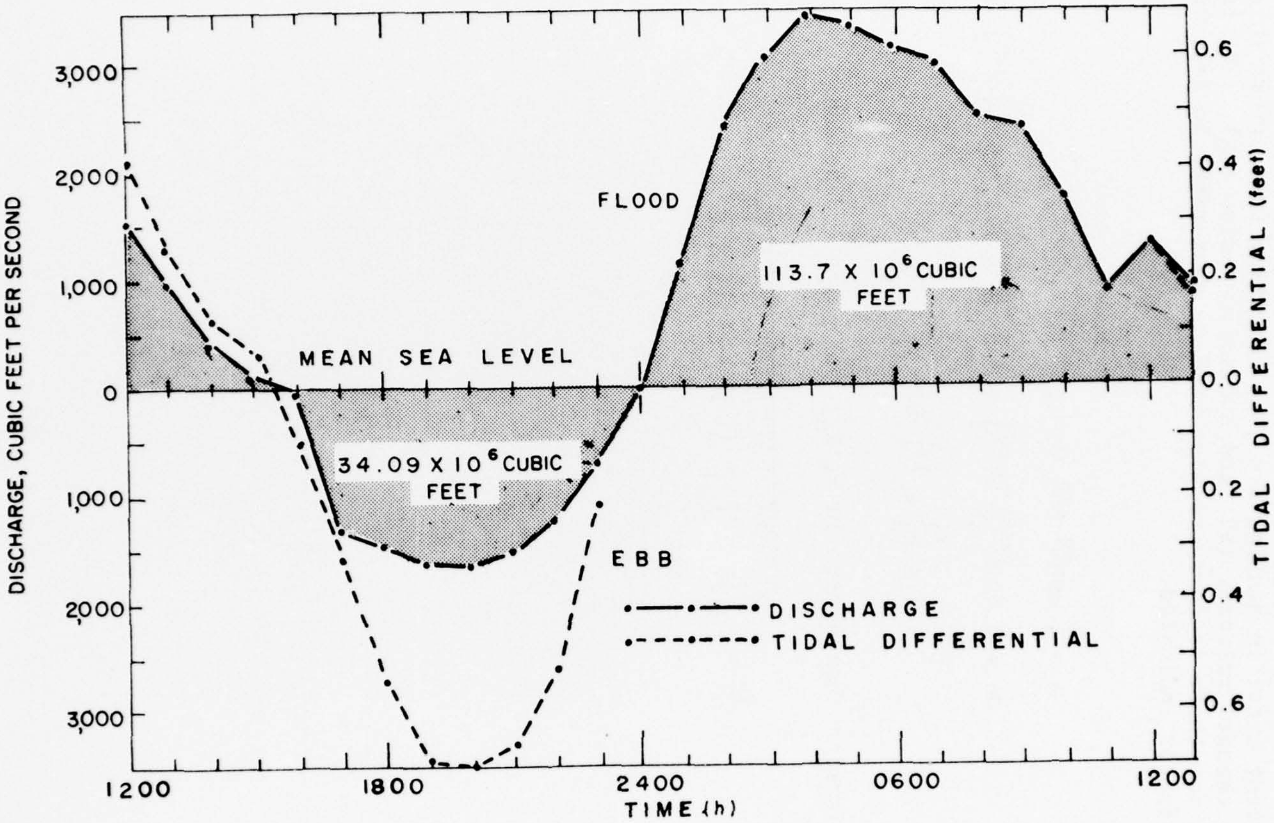


Figure E-8. Tidal differential and discharge, 31 May and 1 June 1973.

APPENDIX F

ERROR EQUATION DERIVATION

Percent error in Manning's n (%E) resulting from errors in slope (S), average channel velocity (V), and hydraulic radius (R_h). The letter symbols for these factors are also used as subscripts to (e), the error in the factor indicated.

$$n = 1.49R^{2/3}\, S^{1/2}\, V^{-1} \ ,$$

$$dn = 1.49R^{2/3}\left(\frac{1}{2}\, S^{-1/2}\right) V^{-1}\, dS + 1.49R^{2/3}\, S^{1/2}\left(-V^{-2}\right) dV$$

$$+\ 1.49\left(\frac{2}{3}\, R_h^{-1/3}\right) S^{1/2}\, V^{-1}\, dR_h$$

and, assuming that errors in S, R_h, and V are independent

$$\frac{dn}{n} = \sqrt{\left(\frac{1}{2}\,\frac{dS}{S}\right)^2 + \left(-\,\frac{dV}{V}\right)^2 + \left(\frac{2}{3}\,\frac{dR}{R}\right)^2}\ .$$

Behrens, E.W.

Hydraulics and dynamics of new Corpus Christi Pass, Texas: a case history, 1972-73 / by E.W. Behrens, R.I. Watson...[et al.]. – Fort Belvoir, Va. : U.S. Coastal Engineering Research Center, 1977.

126 p. : ill. (GITI report 8) Also (Contracts – U.S. Coastal Engineering Research Center ; DACW72-72-C-0026 and DACW 72-72-C-0027)

Bibliography : p. 70.

A case history of the hydraulics and sedimentation of the Corpus Christi Water Exchange Pass, Texas, from 1972-73 is presented. Qualitative data are given on longshore sediment transport, tidal differentials, flood and ebb tidal discharge, wind waves, and local winds to explain bathymetric changes in the Pass.

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