Hydraulics and Stability of Tidal Inlets

by

Francis F. Escoffier

GITI REPORT 13

August 1977

Prepared for
U.S. Army Coastal Engineering Research Center
under
Contract DACW72-74-C-0005

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

Department of the Army
Corps of Engineers

Limited free distribution within the United States of single copies of this publication has been made by this Center. Additional copies are available from:

National Technical Information Service
ATTN: Operations Division
5285 Port Royal Road
Springfield, Virginia 22151

Cover Photo: Drum Inlet, North Carolina, 13 March 1962
Courtesy of the U.S. Geological Survey

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER: GITI Report 13	2. GOVT ACCESSION NO.
3. RECIPIENT'S CATALOG NUMBER	
4. TITLE (and Subtitle): HYDRAULICS AND STABILITY OF TIDAL INLETS	5. TYPE OF REPORT & PERIOD COVERED: Final Report
	6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s): Francis F. Escoffier	8. CONTRACT OR GRANT NUMBER(s): DACW72-74-C-0005
9. PERFORMING ORGANIZATION NAME AND ADDRESS	10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS: F31019
11. CONTROLLING OFFICE NAME AND ADDRESS: Department of the Army, Coastal Engineering Research Center (CERRE-CS), Kingman Building, Fort Belvoir, Virginia 22060	12. REPORT DATE: August 1977
	13. NUMBER OF PAGES: 72
14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office)	15. SECURITY CLASS. (of this report): UNCLASSIFIED
	15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release, distribution unlimited.

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

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Functional design
Inlet hydraulics
Inlet stability
Tidal discharge
Tidal hydraulics
Tidal inlets

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

This report presents a summary of several of the important basic developments pertaining to analysis of the hydraulics and related stability of tidal inlets. In particular, it covers the work reported by Brown (1928) and Keulegan (1967) on inlet hydraulic calculations, and by O'Brien (1931, 1966), Jarrett (1976), Bruun (1966), Johnson (1973), O'Brien and Dean (1972), and Escoffier (1940) on the analysis of inlet channel stability. The original inlet stability concept proposed by Escoffier is extended in light of recent

(continued)

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered) work. The report also contains brief discussions on tidal inlet characteristics and functional design requirements as well as case studies of selected inlets on the U.S. coasts.

FOREWORD

This report results from work done under Contract No. DACW72-74-C-0005 between the author and the Coastal Engineering Research Center (CERC). It is one of 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 was prepared by Francis F. Escoffier (hydraulic engineer, retired, Corps of Engineers), presently engaged as a consultant in coastal engineering for private engineering firms and the Corps of Engineers.

Dr. R.M. Sorensen, Chief, Coastal Structures Branch, was the contract monitor for the report, under the general supervision of R.P. Savage, Chief, Research Division.

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 166th, 79th Congress, approved 31 July 1945, as supplemented by Public Law 172, 88th Congress, approved 7 November 1963.

JOHN L. CANNON
Colonel, Corps of Engineers
Commander and Director
Waterways Experiment Station

JOHN H. COUSINS
Colonel, Corps of Engineers
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 1957, 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 is published in the GITI report series because it presents a summary plus additional insight on some of the more important works on tidal inlet hydraulics and stability. It contributes to the inlet hydraulics and inlet dynamics parts (3b, 3c) of the GITI.

CONTENTS

TABLES

FIGURES

CONTENTS

FIGURES--Continued

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
knots	1.8532	kilometers per hour
acres	0.4047	hectares
foot-pounds	1.3558	newton meters
millibars	1.0197×10^{-3}	kilograms per square centimeter
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]

[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: $K = (5/9)(F - 32) + 273.15$.

SYMBOLS AND DEFINITIONS

A_b	water-surface area of bay
a'	initial value of a_c in an inlet undergoing change
a_b	amplitude of tide in bay
a_c	cross-sectional area in gorge of inlet (below mean sea level)
a_i	cross-sectional area in subreach of inlet
a_o	amplitude of tide at sea
a_p, a_E	limits of integration in equation 47
B', B''	coefficients in equation 8
b	coefficient in the O'Brien equilibrium formula
C	coefficient in the Keulegan method
C_1	coefficient in the O'Brien stability formula
D	water depth in bay
E_s	wave energy in foot-pounds per foot of beach per second
f	Darcy-Weisbach friction coefficient
g	acceleration of gravity
H_a	head due to acceleration in hypothetical rectangular bay
H_f	head due to bottom friction in hypothetical rectangular bay
h_b	elevation of water surface in bay
h_o	elevation of water surface in sea
i	subscript identifying a subreach in an inlet
j	exponent in equation 61
K	Keulegan's repletion coefficient
K'	initial value of K in an inlet undergoing change
K_1	lunisolar diurnal component of the tide

L	effective length of an inlet
L_b	length of a hypothetical rectangular bay
L_i	length of subreach in an inlet
M	gross annual littoral drift
M_2	principal lunar semidiurnal component of the tide
m	coefficient for combined entrance and exit losses
N	exponent in equation 44
N_2	larger lunar elliptic semidiurnal component of the tide
n	Manning's roughness coefficient
O_1	principal lunar diurnal component of the tide
P	wetted perimeter in an inlet cross section
Q	discharge of water through an inlet or bay cross section
Q_m	maximum discharge or discharge at strength of tide
R	hydraulic radius in an inlet gorge
R_i	hydraulic radius in subreach of an inlet
r	ratio used in the classification of tides
S_2	principal solar semidiurnal component of the tide
s	exponent in equation 60
s_a	component of the water-surface slope due to acceleration
s_f	component of the water-surface slope due to bottom friction
T	period of a tidal cycle
t	time
u	dimensionless measure of the velocity V_m, defined by equation 33
V	mean velocity of water in a cross section

V_m	maximum value of V at the strength of the tide
V_T	threshold velocity for sand transport
W	width of hypothetical rectangular bay
W_p	annual wave power
w	width of inlet entrance
x	distance in bay, measured landward from inlet
α	dimensionless measure of the cross-sectional area in an inlet gorge
β	measure of stability defined by equation 47
β_*	dimensionless number defined by equation 57
γ	unit weight of water; also phase angle defined by equation 70
ε	dimensionless number defined by equation 53
η	total loss of head in an inlet
η_e	combined entrance and exit losses of head in an inlet
η_f	loss of head due to bottom friction in an inlet
η_m	maximum value of η
η_s	value of η for which the quadratic and linear formulas agree
θ	angular measure of time in a tidal cycle
λ	dimensionless measure of stability defined by equation 59
ξ	dimensionless number defined by equation 5
ξ_E	equilibrium value of ξ
ξ'	initial value of ξ in an inlet undergoing change
π	3.1416
σ	dimensionless number defined by equation 49

τ	angular measure of the lapse of time from slack tide in an inlet to midtide in the sea
ν	dimensionless measure of the velocity V_m, defined by equation 54
ν_E	equilibrium value of ν
ϕ	tidal phase angle
Ω	tidal prism; i.e., the amount of water stored in a bay between high and low water
Ω_p	potential tidal prism; i.e., the value of the tidal prism if the full tidal range of the sea is admitted into a bay
ω	radian frequency of a component of the tide
ω'	radian frequency for the diurnal component of the tide
ω''	radian frequency for the semidiurnal component of the tide

HYDRAULICS AND STABILITY OF TIDAL INLETS

by
Francis F. Escoffier

I. INTRODUCTION

An inlet is a short, narrow waterway that connects a bay, a lagoon, or an estuary to a larger body of water, generally a sea. This report discusses those inlets where the larger body of water is tidal and where the inlets are effected by sedimentary processes. Many inlets are passages through barrier islands frequently found along shorelines of coastal plains.

Inlets frequently are entrances to harbors and are important to navigation. Inlets also contribute to the ecology of associated bays by causing an exchange of water with the sea. At times, this exchange of water plays a significant part in the control of the temperature and salinity of the water, in the dilution of industrial and municipal wastes, and in the migration of fish.

Inlets sometimes pose certain engineering problems; e.g., the depth of water in the channel may be inadequate for navigation or the channel may shift too much. The exchange of water between the sea and the bay may also be more or less than desired or the inlet may cause adjacent beaches to erode.

II. CHARACTERISTICS OF NATURAL INLETS

Some inlets are fairly permanent features of the coast whose existence may antedate historical records; other inlets are temporary, formed possibly by storms or floods, and subject to closure by natural forces. An inlet may also be artificial; i.e., dredged through a land barrier by man. Inlets are often found on coasts formed by the deposition of sediments during past geologic ages and have inner bays which are likely to be river valleys that have been submerged by the rise in sea level that occurred during the recent geologic epoch.

The actual effect of a hurricane or even of an ordinary windstorm on inlet formation or modification cannot be predicted with accuracy. The storm may cause the formation of a new inlet or the enlargement of an existing inlet by forcing water over a barrier beach. Also, the accumulated water in the bay caused by the wind tide or by the runoff from tributaries may breach the barrier beach and create or enlarge an inlet. However, littoral drift generated by the storm may block an inlet to an extent of closure either at that time or later. This subject is further discussed by Johnson (1919), Brown (1928), and Pierce (1970).

The rise and fall of the tide in the sea causes a flow of water into and out of the inlet which in turn causes the water level in the bay to rise and fall. The runoff which enters the bay from its tributaries also contributes to the flow through the inlet, but usually this is minor. The littoral drift which arrives at an inlet is carried into the inlet by the flood current, and partly deposited at the inner end of the channel to form a bay shoal. The ebb current then carries part of the sand back to the sea where some is deposited as an outer bar at the seaward end (Fig. 1). The sand which is permanently deposited on the bay shoal or the outer bar is sand that is lost from the longshore sediment transport. This loss is likely to cause some erosion of adjacent downdrift beaches.

At most inlets a part of the littoral drift passes the inlet and continues a course on the downdrift beach; this is called *bypassing*. Bypassing occurs as a continuous or intermittent process. Tidal currents carry a large quantity of sand back and forth through the inlet where more sand is constantly added by the littoral transport. On the downdrift side of the outer bar there is often an extensive shoal, and at each ebbtide the outgoing current deposits some sand on the shoal. The incoming waves drive the sand shoreward to the beach where it resumes a course as part of the longshore transport.

Bruun and Gerritsen (1960) distinguished between "bar bypassing" and "tidal-flow bypassing." They stated that most bypassing presents combinations of these two forms and that the form which predominates seems to depend on the ratio between the littoral drift and the tidal flow.

The rate at which sand bypasses an inlet is not ordinarily equal to the rate at which the littoral drift arrives at the inlet. The difference may be deposited on the bay shoal, on the outer bar, or offshore. However, the excess of incoming sand generally causes the inlet to migrate. Part of the incoming sand is deposited on the updrift side of the inlet while the downdrift side erodes, and as a result the inlet is displaced downdrift. Some inlets migrate updrift, such as Redfish Pass on the west coast of Florida. Apparently, in such cases, the rate at which sand is bypassed exceeds the longshore sediment transport.

The configuration or geometry of inlets varies, and these variations in updrift and downdrift shore configurations provide a basis for the classification of inlets. The following four groups were recognized by Galvin (1971) (Fig. 2):

(1) Overlapping. The updrift shore extends seaward and downdrift to overlap the entrance.

(2a) Significant offset on updrift side. The offset is on the downdrift side and exceeds the minimum width of the inlet.

(2b) Significant offset on downdrift side. The offset is on the downdrift side and exceeds the minimum width of the inlet.

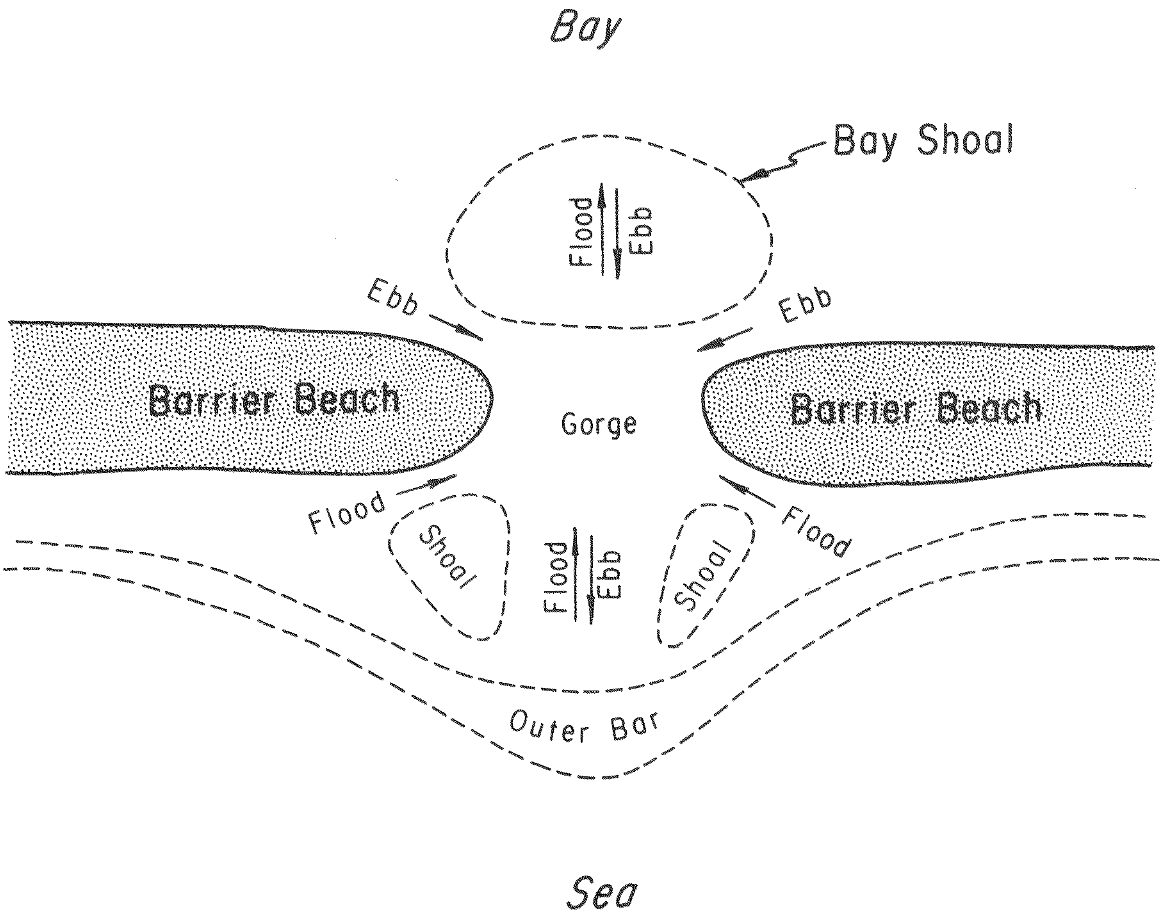

Figure 1. Typical barrier beach tidal inlet.

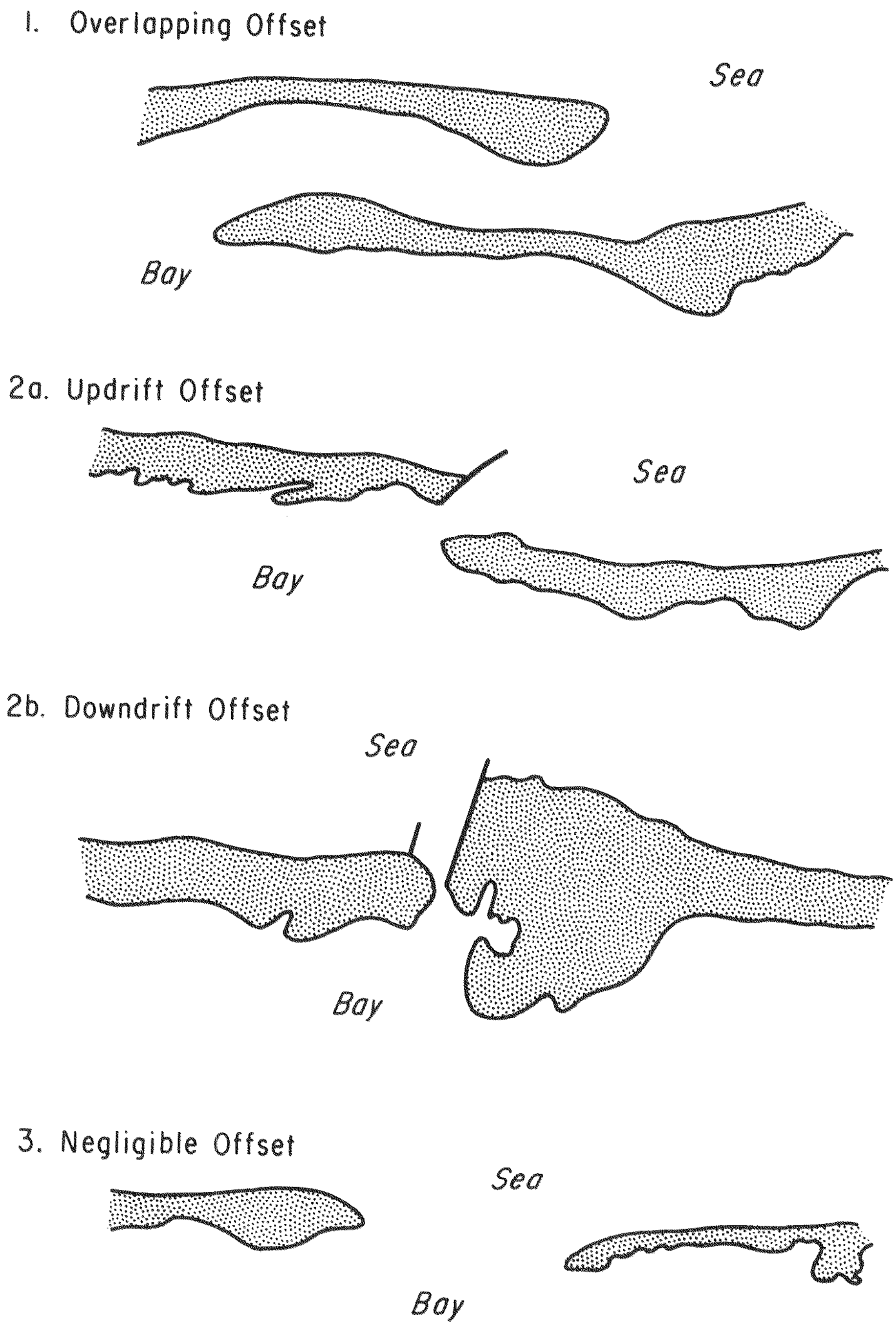

Figure 2. Offset classification of an inlet (Galvin, 1971).

(3) Negligible offset. The offset, if any, is less than the minimum width of the inlet.

Galvin concluded that group 1 inlets develop where the littoral drift is large and mostly from the updrift direction, and where tidal flow is strong enough to maintain the inlet. Groups 2a and 2b inlets occur under conditions similar to group 1 except that an appreciable littoral transport exists in a direction opposite to that of the predominant littoral transport. In group 2b inlets, the littoral transport rate is generally less than in group 2a inlets. Group 3 inlets occur on coasts where the littoral transport rates in the two directions are approximately equal.

The size of inlets varies widely. The entrance to Delaware Bay is an exceptionally large inlet with an opening about 12 miles wide. Philips Inlet, a small intermittent inlet 15 miles west of Panama City, Florida, is an example of an inlet at the opposite extreme. The size of the opening varies from about 100 feet to a completely closed inlet. A number of selected inlets arranged in the order of diminishing throat area, are given in Table 1 (O'Brien, 1966).

III. INLET HYDRAULICS

The flow of water through inlets is caused primarily by the tides; the flow from the tributaries to the bay is generally a minor factor. As the sea level rises and falls with the tide, the flow of water through an inlet causes the level of water in the bay to also rise and fall. At any instant the flow through the inlet depends primarily on the difference between the sea and bay water levels. The Manning formula is used in estimating this dependence because of familiarity to engineers and because information about values for Manning's coefficient, n, is generally available. However, the following limitations should be considered:

(a) The available values for Manning's n are based on observations made in uniform (prismatic) channels and their reliability in nonuniform channels such as those usually found in inlets is uncertain.

(b) The boundary resistance varies due to the presence or absence of ripples and other bed forms which are subject to variation during the tidal cycle.

(c) Little is known about the loss of head that takes place due to the contraction and expansion of the current passing through the inlet.

(d) The formula is for steady flow and thus ignores temporal accelerations.

Manning's formula for the loss of head due to hydraulic friction is:

$$\eta_f = \frac{Ln^2V^2}{(1.486)^2R^{4/3}} \quad , \qquad (1)$$

Table 1. Values of Ω and a_c.

Inlet	Location	Tidal prism on spring of diurnal tide, Ω (ft^3)	Minimum flow area at entrance channel below MSL a_c (ft^2)
		Without jetty	
Delaware Bay	Atlantic	1.25×10^{11}	2.5×10^6
Golden Gate	Pacific	5.1×10^{10}	9.38×10^5
Willapa	Pacific	2.50×10^{10}	3.94×10^5
North Edisto River	Atlantic	4.58×10^9	9.95×10^4
Tomales Bay	Pacific	1.58×10^9	3.6×10^4
Fire Island	Atlantic	2.18×10^9	3.56×10^4
Jones Inlet	Atlantic	1.5×10^9	2.89×10^4
Punta Banda	Pacific	2.99×10^8	5.46×10^3
		One jetty	
Rockaway	Atlantic	3.7×10^9	8.6×10^4
Tillamook	Pacific	2.11×10^9	3.69×10^4
East Rockaway	Atlantic	7.6×10^8	1.15×10^4
		Two jetties	
Columbia	Pacific	3.82×10^{10}	5.08×10^5
Grays Harbor	Pacific	2.43×10^{10}	2.85×10^5
Galveston	Gulf of Mexico	1.59×10^{10}	2.2×10^5
Charleston	Atlantic	5.75×10^9	1.44×10^5
Humboldt	Pacific	4.38×10^9	7.55×10^4
San Diego	Pacific	3.38×10^9	6.17×10^4
Coos Bay	Pacific	2.84×10^9	6.11×10^4
Umpqua	Pacific	2.20×10^9	4.62×10^4
Absecon	Atlantic	1.48×10^9	3.13×10^4
Morichee	Atlantic	1.57×10^9	2.04×10^4
Yaquina	Pacific	7.73×10^8	1.98×10^4
Nahalem	Pacific	6.0×10^8	1.12×10^4
Siuslaw	Pacific	4.64×10^8	1.10×10^4
Mission Bay	Pacific	4.2×10^8	1.04×10^4
Coquelle	Pacific	3.89×10^8	9.02×10^3
Newport Beach	Pacific	1.98×10^8	5.89×10^3
Pendleton Boat Basin	Pacific	1.14×10^7	4.64×10^2

(O'Brien, 1966)

where η_f is the loss in head due to hydraulic friction, L is the length of the channel, n is the Manning's roughness coefficient, V is the mean velocity of the water, and R is the hydraulic radius. This formula assumes that the unit of length is the foot; if the meter is used, the factor 1.486 should be deleted. In a uniform (prismatic) channel, L is simply the length of the channel; in a nonuniform channel it is convenient to divide the length of the channel into a number of subreaches denoted by the subscript i. The friction-loss term becomes the sum of the losses in the subreaches or

$$\eta_f = \Sigma \frac{L_i n^2 V_i^2}{(1.486)^2 R_i^{4/3}} ,$$

where it is assumed that the same value of n holds in all of the subreaches. This can be rewritten as

$$\eta_f = \Sigma \frac{L_i n^2 Q^2}{(1.486)^2 R_i^{4/3} a_i^2} ,$$

where Q is the discharge, and a_i is the cross-sectional area in the subreach. It is now assumed that the length, L, is an effective length that, when used in conjunction with the area, a_c, and the hydraulic radius, R, of the gorge, will yield the same value of η_f. Therefore, the following holds:

$$\frac{L n^2 Q^2}{1.486^2 R^{4/3} a_c^2} = \Sigma \frac{L_i n^2 Q^2}{1.486^2 R_i^{4/3} a_i^2} .$$

When this is solved for L, the result obtained is:

$$L = R^{4/3} a_c^2 \, \Sigma \frac{L_i}{R_i^{4/3} a_i^2} . \tag{2}$$

The entrance and exit losses should be added to the losses due to hydraulic friction. These two losses are combined into a single equation.

$$\eta_e = m \frac{V^2}{2g} , \tag{3}$$

where η_e is the combined entrance and exit losses, g is the acceleration of gravity, and m is an uncertain coefficient value which is often given the value one.

Equations (1) and (3) can be combined to yield:

$$\eta = \eta_f + \eta_e = \left(\frac{2gLn^2}{(1.486)^2 R^{4/3}} + m \right) \frac{V^2}{2g} . \tag{4}$$

Introducing the coefficient,

$$\xi = \left(\frac{2gLn^2}{1.486^2 R^{4/3}} + m \right)^{-1/2} , \tag{5}$$

so that

$$V = \xi \sqrt{2g\eta} \quad \text{for} \quad \eta > o ,$$

and

$$V = -\xi \sqrt{-2g\eta} \quad \text{for} \quad \eta < o , \tag{6}$$

where the sign of η must be changed when the direction of the flow is reversed. Because of the difficulties in evaluating n and m, there is some advantage in combining the effects into the one coefficient ξ which can, in some cases, be evaluated directly from observed data.

Table 2 gives values for the Darcy-Weisbach friction coefficient, f, and for Manning's n, and shows the wide range of values that are possible for these coefficients. For this reason it is preferable to base the value of n on actual measurements in the inlet channel. However, if this is not feasible, the value of $n = 0.025$ is suggested as a reasonable approximation. Similarly, for the coefficient m, the value one is suggested.

Although the tides in the sea are composed of many components, only a few are large enough to play a significant part in generating the tidal currents in inlets. These are the principal components, M_2, S_2, N_2, K_1, and O_1 (Pillsbury, 1940). The astronomical explanation of the components is given in Pillsbury (1940) and Dronkers (1964). Periods for the principal components in hours are:

Components	Hours
M_2	12.42
S_2	12.00
N_2	12.66
K_1	23.94
O_1	25.82

In classifying tides, the National Ocean Survey (NOS) of the National Oceanic and Atmospheric Administration (NOAA) uses the formula:

$$r = \frac{K_1 + O_1}{M_2} , \tag{7}$$

Table 2. Resistance coefficient as dependent on bed form for flume experiments.

Forms of bed roughness	0.28-mm sand		0.45-mm sand	
	f	n $(ft^{1/6})$	f	n $(ft^{1/6})$
Lower flow regime				
ripples	0.0635 to 0.1025	0.02 to 0.027	0.0521 to 0.1330	0.020 to 0.028
dunes	0.0612 to 0.0791	0.021 to 0.026	0.0489 to 0.1490	0.019 to 0.033
Transition	0.0250 to 0.0344	0.014 to 0.017	0.0415 to 0.0798	0.016 to 0.015
Upper flow regime				
plane	0.0244 to 0.0262	0.013 to 0.014	------ ------	----- -----
standing waves	------ ------	----- -----	0.0200 to 0.0406	0.011 to 0.015
antidunes	0.0281 to 0.0672	0.014 to 0.022	0.0247 to 0.0292	0.012 to 0.014

(Simons and Richardson, 1966)

where,

$r < 0.5$ = semidiurnal

$0.5 < r < 2.0$ = mixed

$2.0 < r$ = diurnal

In making calculations for inlets, the tide is represented as two components, a diurnal component having a period of 24.84 hours or 1 lunar day and a semidiurnal component having a period of 12.42 hours or one-half a lunar day. Where the tide is classified as diurnal or semidiurnal, a single component is used; where the tide is mixed, the two are combined. A suitable formula is:

$$h_o = B' \sin \omega' t + B'' \sin (\omega'' t + \phi) \tag{8}$$

where h_o is an approximation to the tide in the sea, B' and B'' are coefficients, ϕ is a phase difference between components, t is time, and ω' and ω'' are the radian frequencies for the diurnal and semidiurnal components of the tide, respectively. In any given case the coefficients, B' and B'', and the phase angle, ϕ, are selected to approximate the predicted tide (National Oceanic and Atmospheric Administration, 1976).

A number of methods have been developed for calculating the flows through a tidal inlet when that inlet is the only one connecting a bay otherwise closed to the sea. The more accurate methods require the use of a digital computer (e.g., Masch, Brandes, and Reagan, in preparation, 1977; Seelig, Harris, and Herchenroder, in preparation, 1977). Two methods which are adapted to hand calculation are considered in this study: Brown's (1928) linear method and Keulegan's (1967) nonlinear method. The following assumptions are made in both methods:

(a) The water surface in the bay remains horizontal throughout the tidal cycle.

(b) The walls of the bay are vertical so that the water-surface area remains constant.

(c) The tributary and surface runoff inflow to the bay is zero.

(d) No density currents are present.

(e) The tide in the sea is given by a simple sinusoidal curve.

(f) The depth variation in the inlet during the tidal cycle is small enough so that the cross-sectional area and hydraulic radius can be assumed to remain constant.

(g) The head due to acceleration in the inlet is negligible. This condition does not hold for many U.S. tidal inlets (King, 1974).

The two basic equations for both methods are:

$$Q = A_b \frac{dh_b}{dt} \quad , \tag{9}$$

and

$$h_o - h_b = \eta = \frac{|Q|Q}{2ga_c^{\,2}\xi^2} \quad , \tag{10}$$

where Q is the discharge through the inlet, reckoned positive on the floodtide and negative on the ebbtide; A_b is the water-surface area in the bay; t is time; and h_b and h_o are the water-surface elevations in the bay and in the sea, respectively. Equation (9) is an equation of continuity which states that the inflow to the bay is equal to the rate at which water is stored in the bay. Equation (10) equates the difference in water level between the sea and the bay to the head losses in the inlet channel.

In the linear method, the right-hand side of equation (10) is replaced with

$$\frac{\sqrt{\eta_s}\, Q}{a_c \xi \sqrt{2g}} \quad ,$$

and that equation becomes:

$$h_o - h_b = \frac{\sqrt{\eta_s}\, Q}{a_c \xi \sqrt{2g}} \quad , \tag{11}$$

where η_s is the value of η for which the quadratic and the linear formulas yield the same discharge (Fig. 3). The assumption is made that the tide is either diurnal or semidiurnal, not mixed. The tides in the bay and in the sea are represented by

$$h_b = -a_b \cos\theta \quad , \tag{12}$$

and

$$h_o = a_o \sin(\theta - \tau) \tag{13}$$

where,

$$\theta = \omega t \tag{14}$$

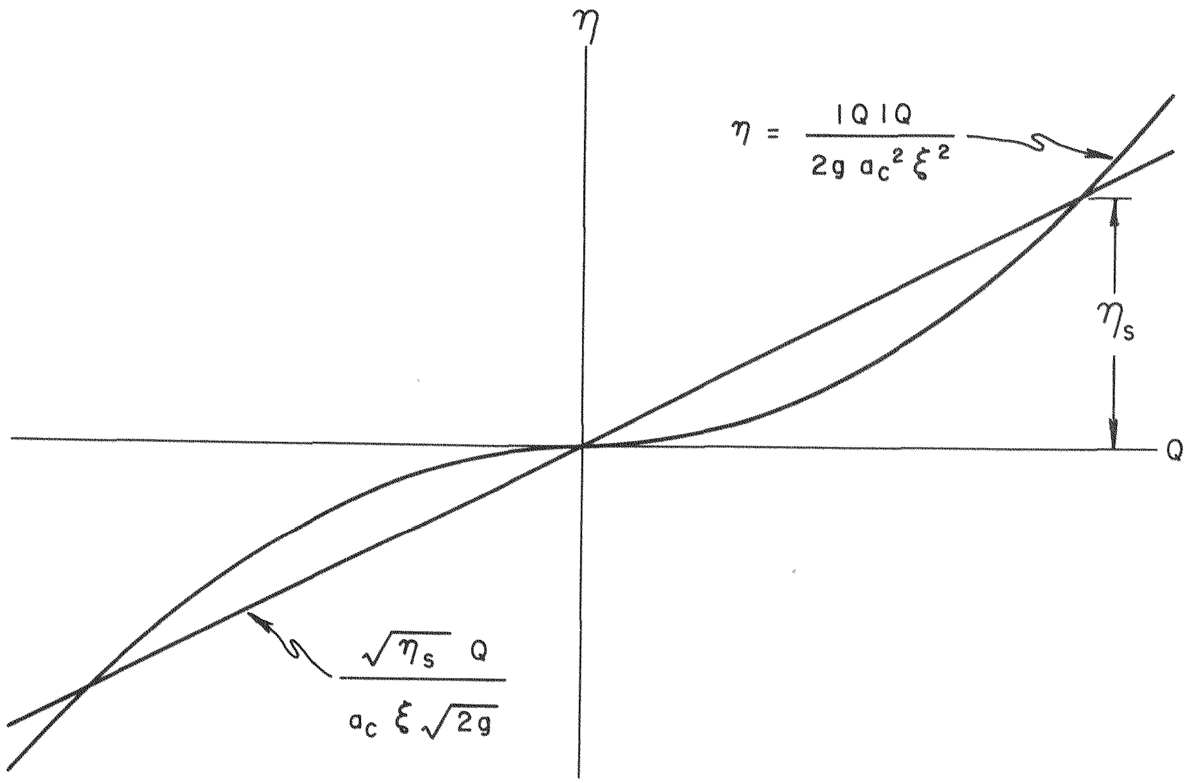

Figure 3. Comparison of quadratic and linear formulas.

a_o and a_b are the amplitudes of the tide in the sea and in the bay, respectively; ω is the radian frequency of the tide; and τ is the value of θ at midtide on a floodtide. When equation (12) is substituted into equation (9), the equation

$$Q = A_b a_b \omega \sin \theta \tag{15}$$

is obtained. Equations (12), (13), and (15) are now substituted into equation (11) to obtain:

$$a_o \sin (\theta - \tau) + a_b \cos \theta = \frac{\sqrt{\eta_s} A_b a_b \omega}{a_c \xi \sqrt{2g}} \sin \theta \ .$$

When the identity,

$$\sin (\theta - \tau) = \sin \theta \cos \tau - \cos \theta \sin \tau \ , \tag{16}$$

is introduced and the terms are rearranged, this becomes:

$$\left(a_o \cos \tau - \frac{\sqrt{\eta_s} A_b a_b \omega}{a_c \xi \sqrt{2g}}\right) \sin \theta - (a_o \sin \tau - a_b) \cos \theta = 0 \ .$$

As this equation must be true for all values of θ, the two expressions in parentheses must be separately equated to zero; this yields:

$$a_o \cos \tau = \frac{\sqrt{\eta_s} A_b a_b \omega}{a_c \xi \sqrt{2g}}$$

and

$$a_o \sin \tau = a_b \ .$$

These are then solved to yield:

$$\tan \tau = \frac{a_c \xi \sqrt{2g}}{\sqrt{\eta_s} A_b \omega} \tag{17}$$

and

$$\sin \tau = \frac{a_b}{a_o} \ . \tag{18}$$

Brown (1928) assumed η_s to be equal to the maximum value of $h_o - h_b$; i.e., the value when the velocity of the water in the inlet reaches a maximum. If equation (12) is subtracted from equation (13) the result is:

$$h_o - h_b = a_o \sin (\theta - \tau) + a_b \cos \theta \ , \tag{19}$$

which on appropriate substitution from equations (16) and (18) becomes:

$$h_o - h_b = a_o \cos \tau \sin \theta \ . \tag{20}$$

Equating the maximum value of this expression to η_s yields:

$$a_o \cos \tau = \eta_s \ ;$$

when this is substituted into equation (17),

$$\tan \tau = \frac{a_c \xi \sqrt{2g}}{A_b \omega \sqrt{a_o \cos \tau}} \ ,$$

which can be rewritten

$$\frac{\sin \tau}{\sqrt{\cos \tau}} = K \ , \tag{21}$$

where

$$K = \frac{a_c \xi \sqrt{2g}}{A_b \omega \sqrt{a_o}} \ , \tag{22}$$

is known as the repletion coefficient. Eliminating τ between equations (18) and (21) yields the equation:

$$\left(\frac{a_b}{a_o}\right)^2 + \left(\frac{a_b}{Ka_o}\right)^4 = 1 \ , \tag{23}$$

which can be solved for a_b/a_o with the result:

$$\frac{a_b}{a_o} = \frac{K^2}{\sqrt{2}} \sqrt{\sqrt{1 + \frac{4}{K^4}} - 1} \ . \tag{24}$$

The following expression for Q_m, the maximum discharge or discharge at the strength of the tide, is obtained from equation (15):

$$Q_m = \omega A_b a_b \ , \tag{25}$$

similarly for V_m, the value of the mean velocity in the gorge at the time of the strength of the tide, is

$$V_m = \frac{\omega A_b a_b}{a_c} \ . \tag{26}$$

A sample problem illustrating the use of the Brown method is discussed below.

The nonlinear method developed by Keulegan (1967) is more accurate than the Brown method. The sea level in this method (as in the Brown method) is assumed to follow a simple sinusoidal curve; however, the level in the bay does not. Numerical methods are used to solve the appropriate differential equation and the results are presented in both tabular and graphical form. The important and fundamental repletion coefficient, K, is introduced and it is shown that many of the characteristics of an inlet can be expressed in terms of that number. The expression for K is:

$$K = \frac{a_c \xi \sqrt{2g}}{A_b \omega \sqrt{a_o}} .$$

A set of tidal curves obtained by Keulegan's method is shown in Figure 4. Details on how such curves can be calculated are given in Keulegan (1967).

The two dimensionless numbers, $\sin \tau$ and C, are given as functions of K in Table 3 and graphically in Figure 5. These numbers are used to obtain other quantities needed in inlet studies by means of the following equations:

$$a_b = a_o \sin \tau \tag{27}$$

$$\Omega = 2A_b a_o \sin \tau \tag{28}$$

$$Q_m = \frac{\pi \Omega C}{T} . \tag{29}$$

In these equations, Ω is the tidal prism or the amount of water stored in the bay between high and low water in the bay, and T is the tidal period.

The relationship between ω and T is given by the equation:

$$\omega = \frac{2\pi}{T} . \tag{30}$$

The elimination of Ω between equations (28) and (29) yields the equation:

$$Q_m = \frac{2\pi A_b a_o}{T} C \sin \tau ,$$

which in view of equation (30), can be rewritten as:

$$Q_m = \omega A_b a_o C \sin \tau . \tag{31}$$

The corresponding expression for the maximum velocity, V_m, is:

$$V_m = \frac{\omega A_b a_o}{a_c} C \sin \tau . \tag{32}$$

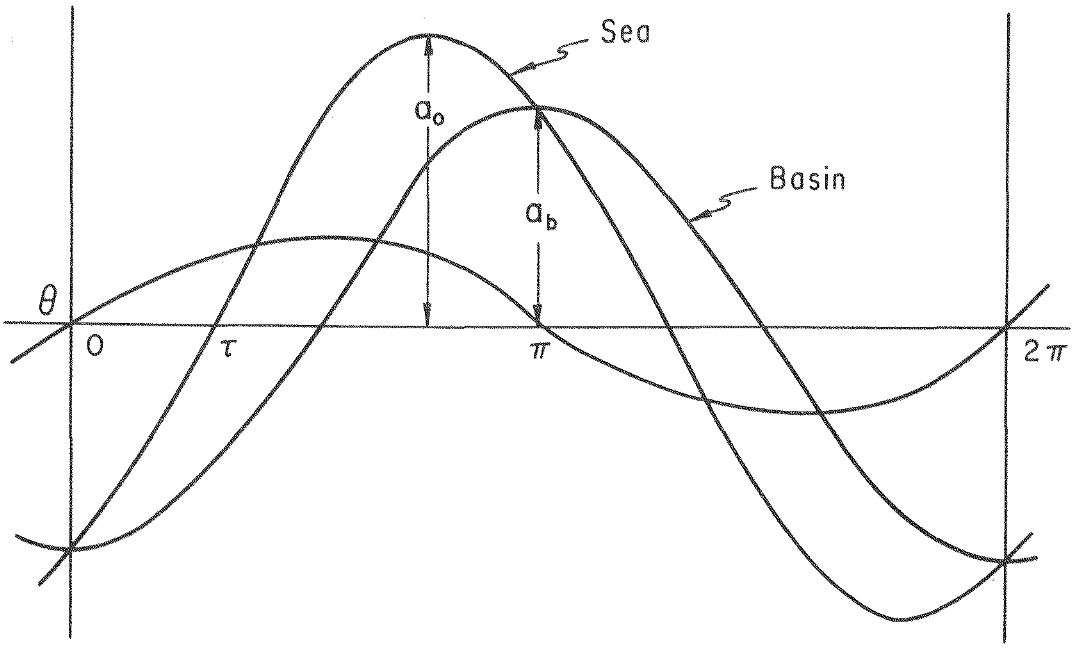

Figure 4. Surface level fluctuations of sea and basin.

Table 3. Coefficient C in tidal prism formula and ratio of bay to ocean tidal range.

K	$\sin \tau$ ($= a_b/a_o$)	C
0.1	0.1158	0.8106
0.2	0.2293	0.8116
0.3	0.3387	0.8128
0.4	0.4414	0.8153
0.5	0.5359	0.8184
0.6	0.6209	0.8225
0.7	0.6955	0.8288
0.8	0.7592	0.8344
0.9	0.8165	0.8427
1.0	0.8555	0.8522
1.2	0.9168	0.8751
1.4	0.9536	0.9016
1.6	0.9745	0.9267
1.8	0.9861	0.9484
2.0	0.9926	0.9650
3.0	0.9996	0.9950

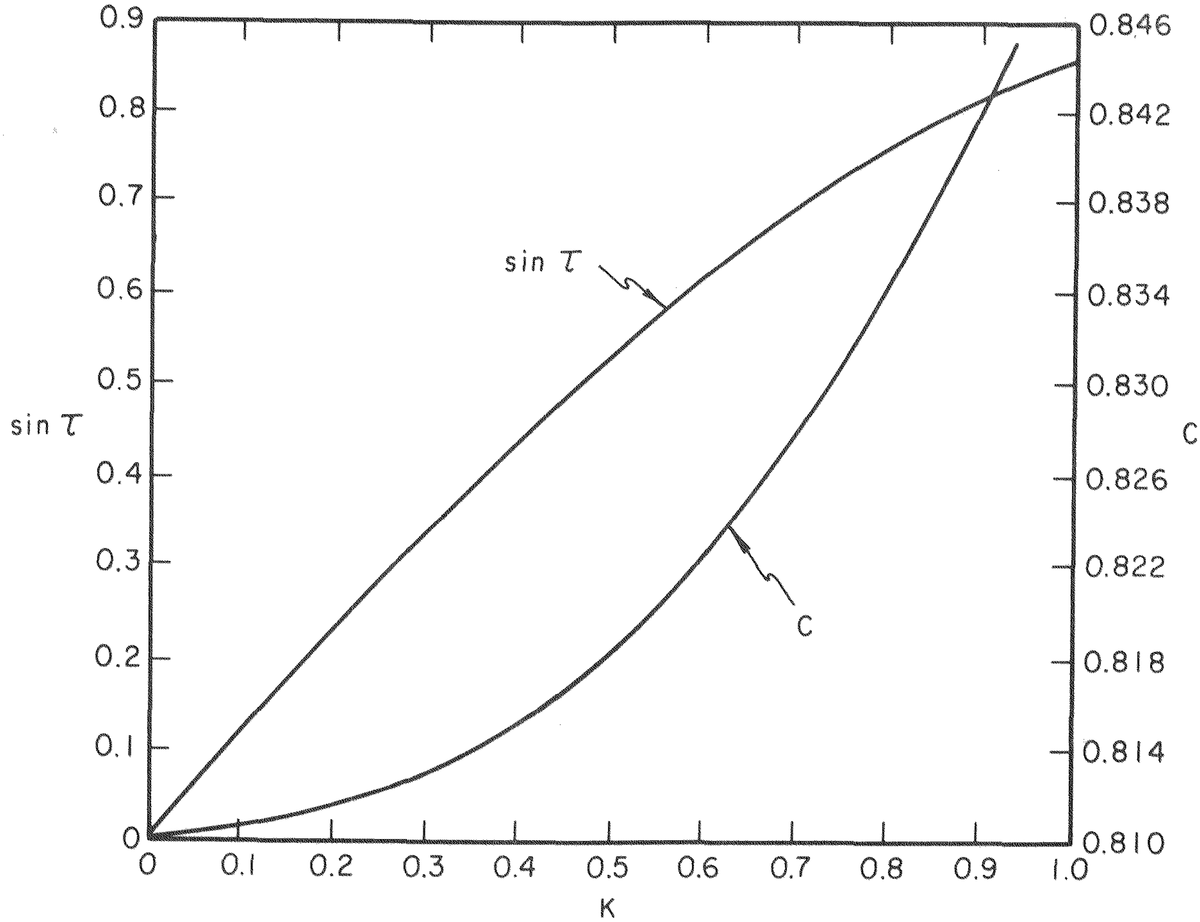

Figure 5. Sin τ and C versus the coefficient of repletion.

To facilitate the use of the Brown and Keulegan methods and to permit a comparison of the two methods, it is convenient to represent the results in the form of curves of dimensionless numbers plotted as functions of the repletion coefficient, K. This has been done for the ratio a_b/a_o in Figure 6. Values for the Brown method were calculated with equation (24) and those for the Keulegan method were taken from Table 2, while keeping in mind that

$$\sin \tau = \frac{a_b}{a_o} .$$

An examination of the two curves shows that the Keulegan method yields larger tidal amplitudes in the bay than the Brown method. To present the maximum velocity, V_m, the dimensionless number,

$$u = \frac{a_c}{\omega \Lambda_b a_o} V_m , \tag{33}$$

is introduced. From equation (26) the Brown method shows

$$u = \frac{a_b}{a_o} , \tag{34}$$

and from equation (32) the Keulegan method shows

$$u = C \sin \tau . \tag{35}$$

Curves representing these two values of u in Figure 7 show that the Brown method yields larger velocities than the Keulegan method. To arrive at curves representing the lag of the bay tide behind the sea tide, the following relationship was used:

$$\tau + \phi = \frac{\pi}{2} , \tag{36}$$

where ϕ is the lag under consideration. Substituting this into equation (18) yields

$$\cos \phi = \frac{a_b}{a_o} , \tag{37}$$

which is the equation used to arrive at the values of ϕ plotted in Figure 8. In the Keulegan method the bay tide is not truly sinusoidal and there can be no question about a lag for the curve as a whole. The curve in the figure shows the lag for the point of maximum discharge.

The following calculation illustrates the use of the Brown and Keulegan methods. An inlet is assumed to be described by the values:

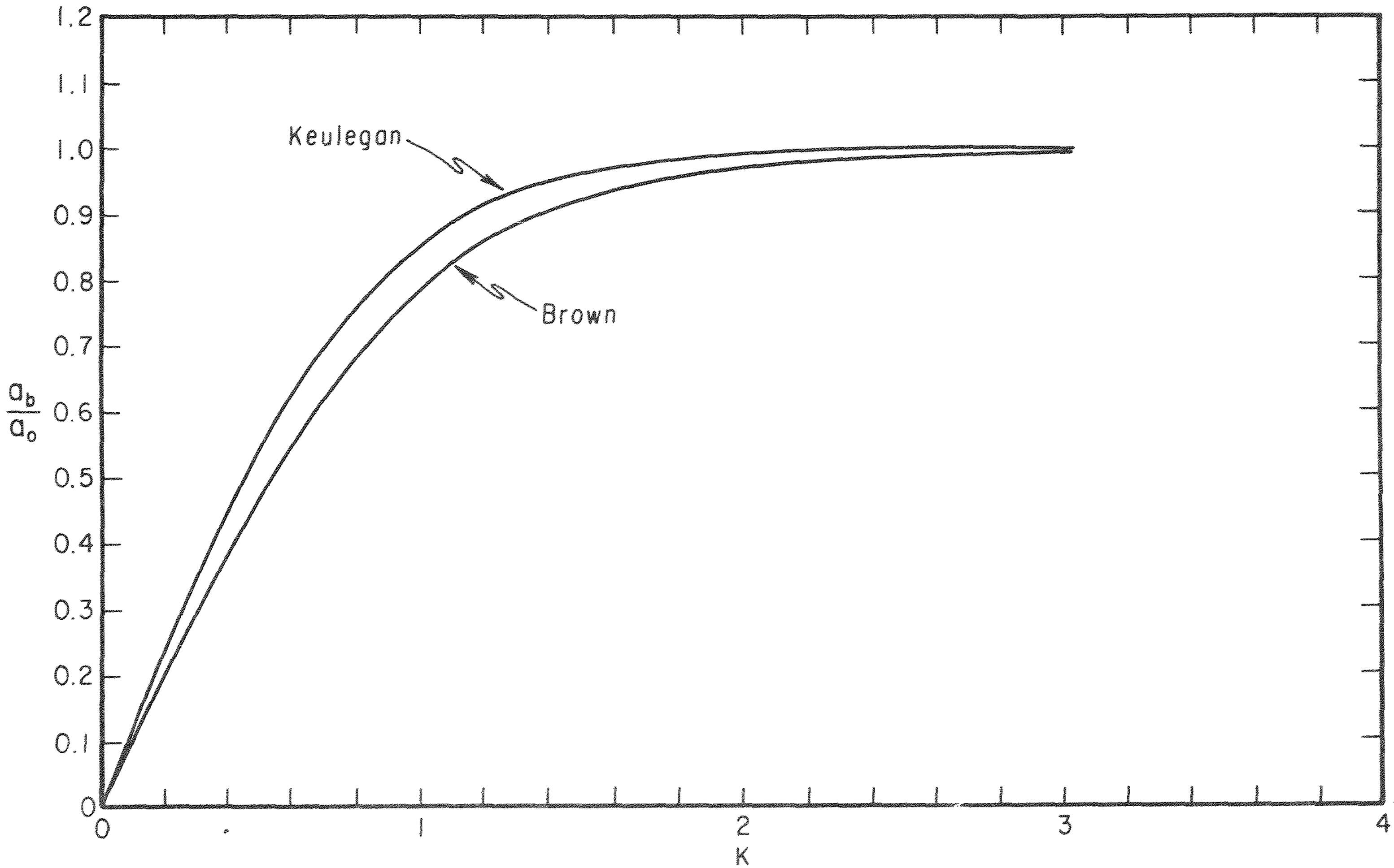

Figure 6. a_b/a_o from Brown (1928) and Keulegan (1967) versus the coefficient of repletion.

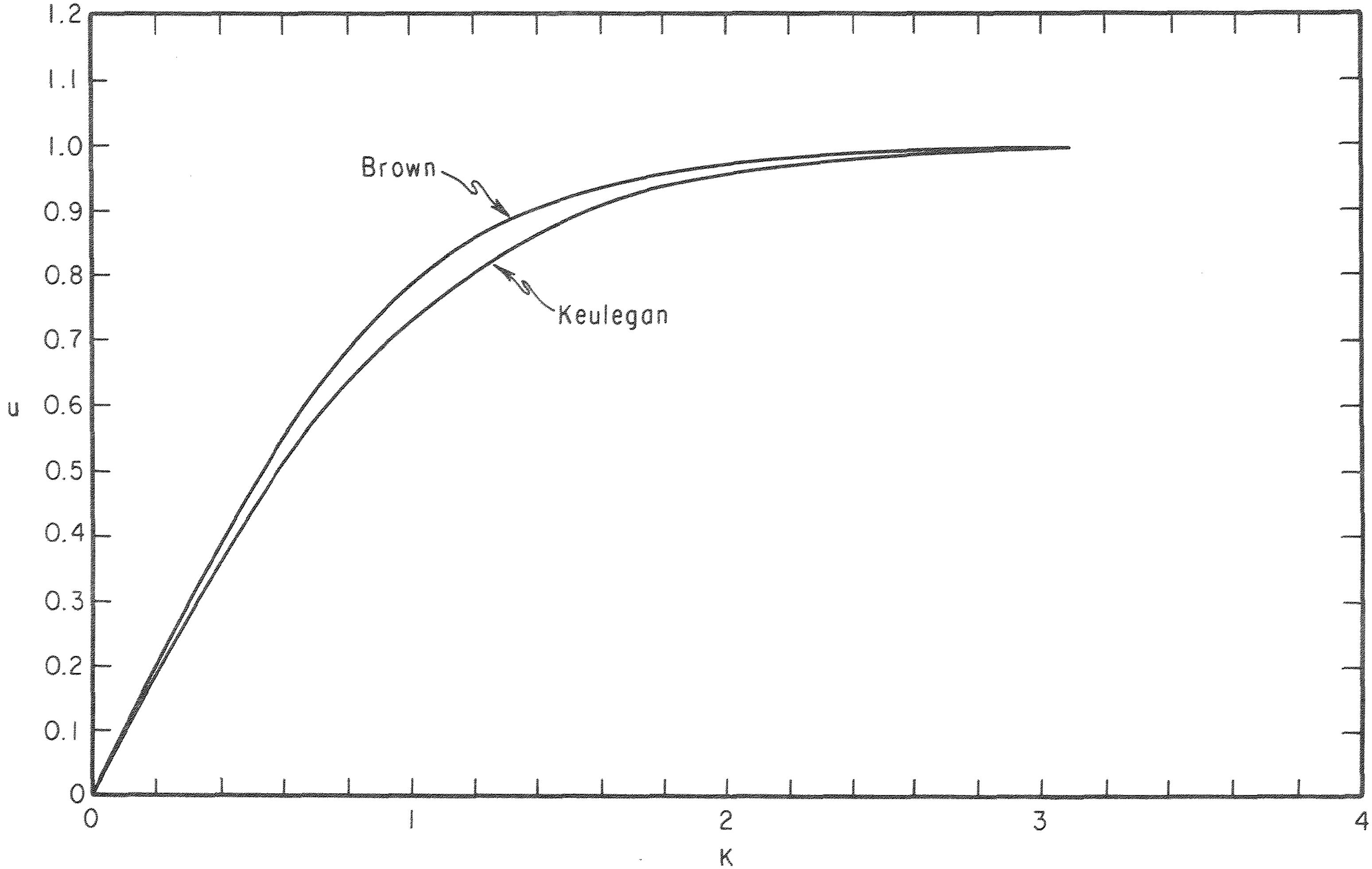

Figure 7. u from Brown (1928) and Keulegan (1967) versus the coefficient of repletion.

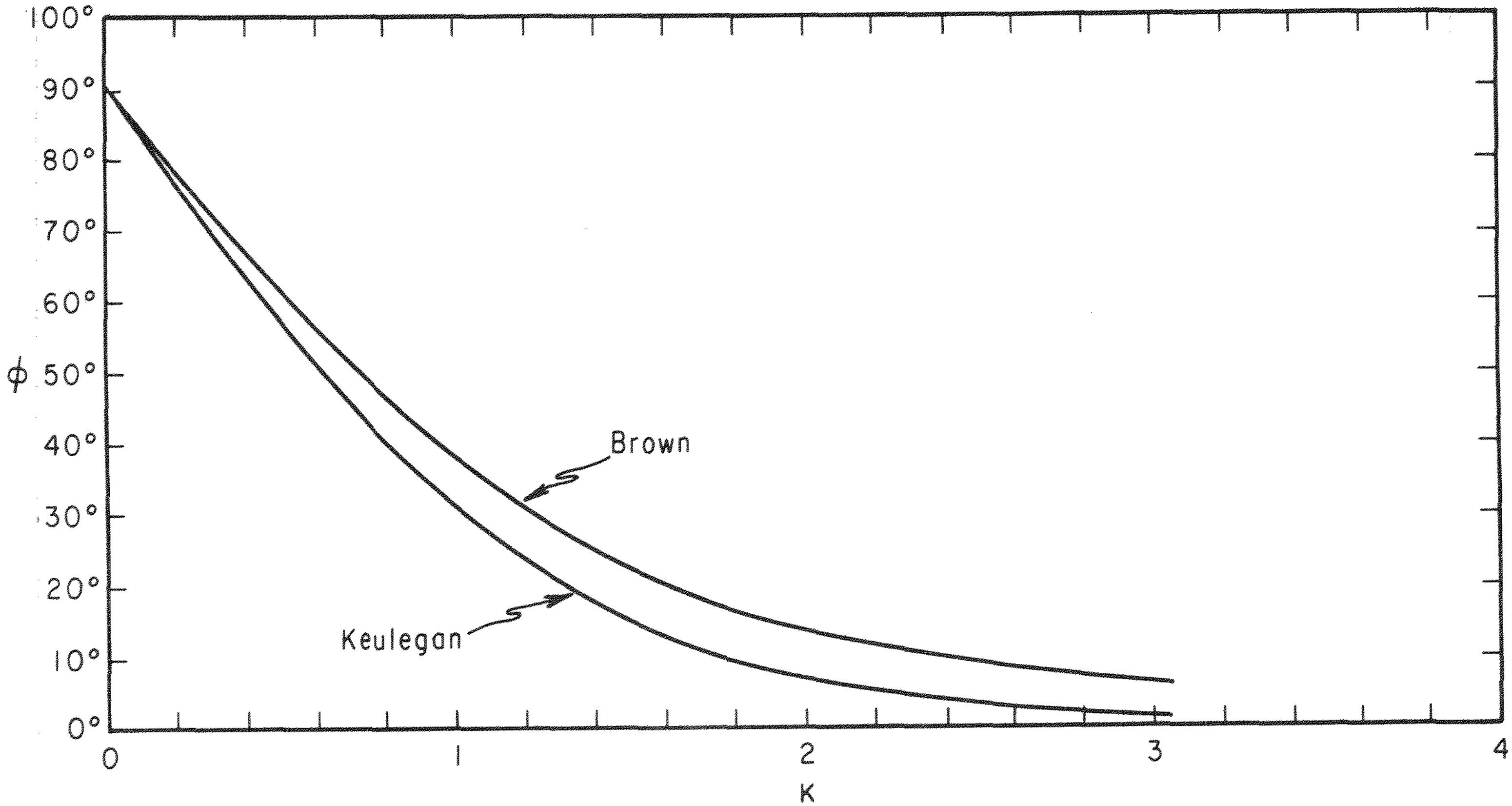

Figure 8. Phase lag from Brown (1928) and Keulegan (1967) versus the coefficient of repletion.

$A_b = 3 \times 10^8$ square feet

$a_c = 1.5 \times 10^4$ square feet

$P = 10^3$ feet (wetted perimeter)

$R = a_c/P = 1.5 \times 10^4/10^3 = 15$ feet

$L = 4 \times 10^3$ feet

$a_o = 2$ feet

$n = 0.025$

$m = 1$

$g = 32.14$ feet per second squared

$T = 12.42 \times 3{,}600 = 44{,}712$ seconds (semidiurnal tide)

$\omega = 2\pi/T = 6.2832/44{,}712 = 1.405 \times 10^{-4}$ second^{-1}

From these values are obtained:

$$\xi = \left(\frac{2gLn^2}{1.486^2 R^{4/3}} + m\right)^{-1/2} = \left(\frac{64.28 \times 4 \times 10^3 \times 0.025^2}{1.486^2 \times (15)^{4/3}} + 1\right)^{-1/2} = 0.58$$

$$K = \frac{a_c \xi \sqrt{2g}}{A_b \omega \sqrt{a_o}} = \frac{1.5 \times 10^4 \times 0.58 \times \sqrt{64.28}}{3 \times 10^8 \times 1.405 \times 10^{-4} \times \sqrt{2}} = 1.17$$

$$\frac{\omega A_b a_o}{a_c} = \frac{1.405 \times 10^{-4} \times 3 \times 10^8 \times 2}{1.5 \times 10^4} = 5.62 \text{ feet per second.}$$

Values for a_b/a_o, u, and ϕ are taken from Figures 6, 7, and 8; values for a_o, V_m, Ω, and Q_m are calculated with the formulas:

$$a_b = a_o \times \frac{a_b}{a_o} = 2 \times \frac{a_b}{a_o}$$

$$V_m = \frac{\omega A_b a_o}{a_c} u = 5.62 u$$

$$\Omega = 2A_b a_b = 2 \times 3 \times 10^8 \times a_b = 6 \times 10^8 \times a_b$$

$$Q_m = a_c V_m = 1.5 \times 10^4 \times V_m .$$

The results are:

	Brown method	Keulegan method
a_b/a_o	0.85	0.91
a_b	1.70 feet	1.82 feet
Ω	1.02×10^9	1.09×10^9
u	0.85	0.79
V_m	4.78 feet per second	4.44 feet per second
Q_m	7.17×10^4 cubic feet per second	6.66×10^4 cubic feet per second
ϕ	32°	25°

For many bays the assumption that the water surface remains horizontal throughout the tidal cycle is not satisfactory. The amplitude and phase of the tide vary from place to place, and the amplitude may in some places actually exceed that on the outside. Methods of calculation are needed which will take these variations into account. Such methods depend on the theory of tidal oscillations in open bodies of water and in bays with irregular shapes and varying depths that generally require the use of finite-difference methods of calculations. The theory of two-dimensional tidal motion and the application to such motion of finite-difference methods are developed in Dronkers (1964).

An idea of the effects of inertia and bottom friction on the tidal motions in a bay can be formed by considering the simplified case of a narrow rectangular bay having a uniform depth. It is assumed that for a satisfactory first approximation, the depth remains constant and the water level remains horizontal. The tidal motion obtained in this way is then used to arrive at second approximations in which the slopes due to inertia and bottom resistance are estimated. The elevation of the bay tide, h_b, and the discharge, Q, are related by the equation of continuity:

$$W \frac{\partial h_b}{\partial t} + \frac{\partial Q}{\partial x} = 0 , \qquad (38)$$

where W, the width of the bay, is a constant; h_b, the elevation of the water surface, is a function of t only; Q, the discharge is considered positive when flowing bayward from the inlet, and x, the distance from the inlet (Fig. 9). As h_b is a function of t only, the equation can be integrated with respect to x with the result:

$$Q = W (L_b - x) \frac{\partial h_b}{\partial t} ,$$

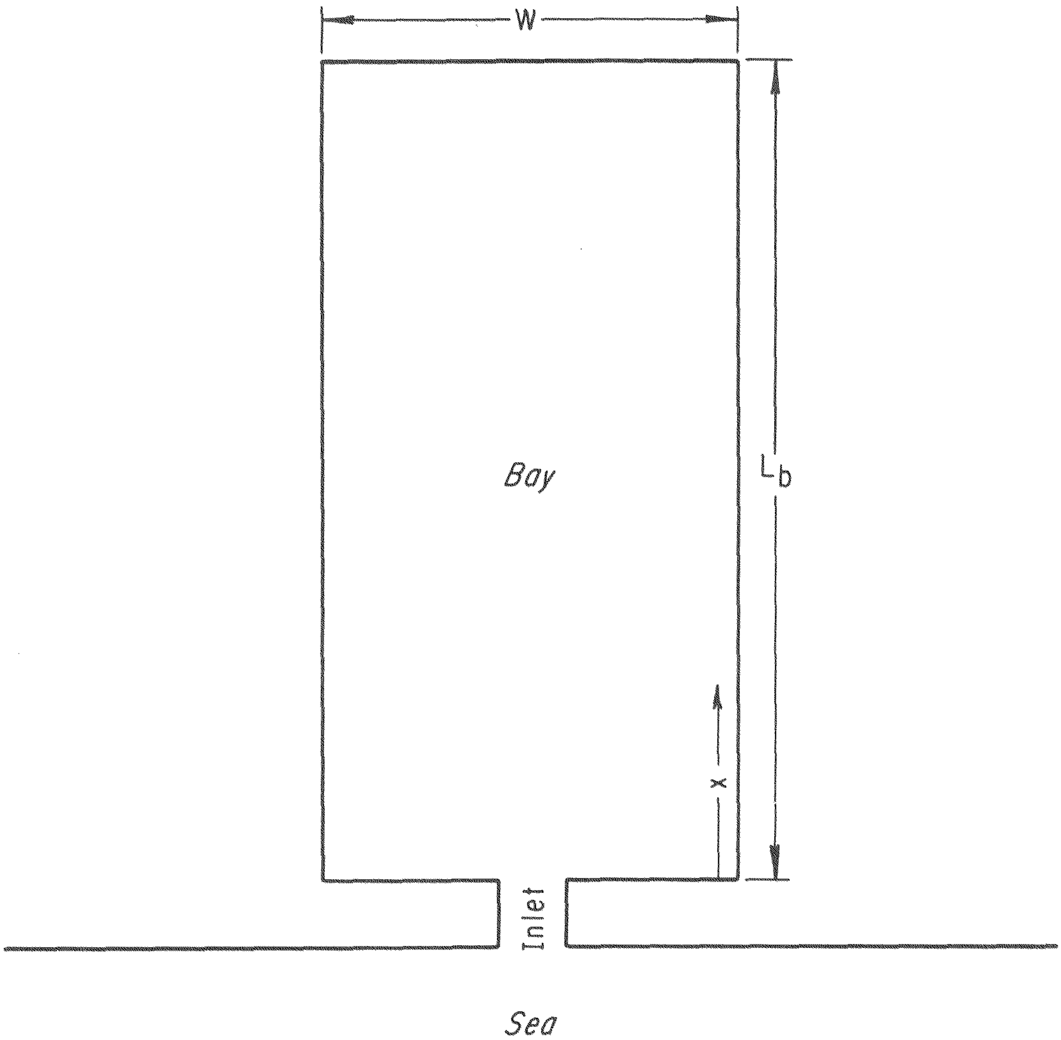

Figure 9. Schematic inlet and narrow rectangular bay with uniform depth.

where L_b is the length of the bay. It is then assumed that the tide follows the simple sinusoidal function,

$$h_b = a_b \sin \omega t \, . \tag{39}$$

Therefore,

$$Q = W a_b \omega \, (L_b - x) \cos \omega t \tag{40}$$

and

$$V = \frac{Q}{WD} = \frac{a_b \omega}{D} \, (L_b - x) \cos \omega t \, , \tag{41}$$

where D is the depth of water in the bay. The water-surface slope due to acceleration is:

$$S_a = \frac{1}{g} \frac{\partial V}{\partial t} = - \frac{a_b \omega^2}{gD} \, (L_b - x) \sin \omega t \, .$$

At high water this slope reaches the maximum absolute value of

$$|S_a|_{max} = \frac{a_b \omega^2}{gD} \, (L_b - x) \, ,$$

and H_a, the difference in level between the two extremities of the bay at that time is

$$H_a = \frac{a_b \omega^2}{gD} \int_0^{L_b} (L_b - x) \, dx = \frac{a_b \omega^2 L_b^2}{2gD} \, . \tag{42}$$

The water-surface slope due to bottom friction expressed in terms of Manning's roughness coefficient is

$$S_f = \left(\frac{nv}{1.486 D^{2/3}} \right)^2 \, ,$$

which in view of equation (41) becomes:

$$S_f = \left(\frac{n a_b \omega}{1.486 D^{5/3}} \right)^2 (L_b - x)^2 \cos^2 \omega t \, .$$

At midtide this slope reaches the maximum absolute value of

$$|S_f|_{max} = \left(\frac{n a_b \omega}{1.486 D^{5/3}} \right)^2 (L_b - x)^2 \, ,$$

and the difference in level between the two extremities of the bay at that time is:

$$H_f = \left(\frac{n a_b \omega}{1.486 D^{5/3}}\right)^2 \int_0^{L_b} (L_b - x)^2 \, dx = \frac{1}{3}\left(\frac{n a_b \omega}{1.486 D^{5/3}}\right)^2 L_b^3 \quad . \qquad (43)$$

The conclusion is that an assumed horizontal water surface in the bay is a reasonable approximation only if the values of H_a and H_f, as estimated by equations (42) and (43), are both small in comparison with the value of a_b. Although these equations were derived for a rectangular bay having a uniform depth, they can be used with judgment for irregular bays having varying depths.

A number of published studies have considered aspects of inlet hydraulics that go beyond those considered in the Brown and Keulegan methods. Baines (1957) accounted for the effect of acceleration in the inlet and pointed out that in some inlets this results in a bay range greater than that in the sea; Kreeke (1967) accounted for the freshwater inflow to the bay. The calculations of Shemdin and Forney (1970) included the effect of acceleration in the inlet and some of the harmonics of the tide. Oliveira (1970) developed a method which allowed both the inlet cross section and the water-surface area of the bay to vary with the progress of the tidal cycle. A bibliography of hydraulics and other aspects of tidal inlets is presented by Barwis (1976).

IV. INLET STABILITY

Some inlets are permanent and remain open with little change; others are ephemeral, opening or closing in response to natural forces. It is a matter of considerable engineering importance to be able to determine the hydraulic characteristics which govern the stability of inlets. Some of the current theories that relate to stability of inlets are discussed in this section; however, these theories are only approximations, and an inlet which is stable during ordinary weather may be unstable during a severe storm. Furthermore, these theories are subject to revision as additional information is accumulated.

A relationship that defines the stability of inlets was presented by O'Brien (1931, 1966). He found a close relationship between the cross-sectional area in the gorge of an inlet and the tidal prism in its bay by the formula:

$$a_c = b\Omega^N \qquad (44)$$

where a_c is the cross-sectional area in the gorge measured below mean sea level, and Ω is the tidal prism or volume of water stored in the bay between the high and the low waters corresponding to the diurnal or the spring range of tide. The values of b and N by O'Brien are:

	b	N
Inlets with two jetties	4.69×10^{-4}	0.85
Inlets without jetties	2.00×10^{-5}	1.00

The unit of length for values of b is the foot; if the meter is used, the corresponding values of b become 9.01×10^{-4} and 6.56×10^{-5}. The data used by O'Brien are given in Table 1 and are shown graphically in Figure 10.

Jarrett (1976) studied the O'Brien formula in which data relating to many other inlets were analyzed. He attempted to determine if differences existed between the tidal prism cross-sectional area relationships for inlets on the Atlantic, gulf, and Pacific coasts of the United States. Jarrett's conclusions were:

(a) Unjettied and single-jettied inlets on the Atlantic, gulf, and Pacific coasts exhibited different Ω versus a_c relationships as a result of differences in the tidal and wave characteristics between these coasts.

(b) The available data did not appear to warrant any modification in the Ω versus a_c relationship for jettied inlets as originally determined by O'Brien (1931, 1966).

The values of b and N obtained by Jarrett are given in Table 4.

Table 4. Values of b and N for the O'Brien (1931, 1966) equilibrium formula (Jarrett, 1976).

Inlets	b	N
Atlantic, gulf, Pacific coasts		
All	5.74×10^{-5}	0.95
One or no jetty	1.04×10^{-5}	1.03
Two jetties	3.76×10^{-4}	0.86
Atlantic coast		
All	7.75×10^{-6}	1.05
One or no jetty	5.37×10^{-6}	1.07
Two jetties	5.77×10^{-5}	0.95
Gulf coast		
All	5.02×10^{-4}	0.84
No jetty	3.51×10^{-4}	0.86
Two jetties[1]	---- ---[1]	----[1]
Pacific coast		
All	1.19×10^{-4}	0.91
One or no jetty	1.91×10^{-6}	1.10
Two jetties	5.28×10^{-4}	0.85

[1]Insufficient data.

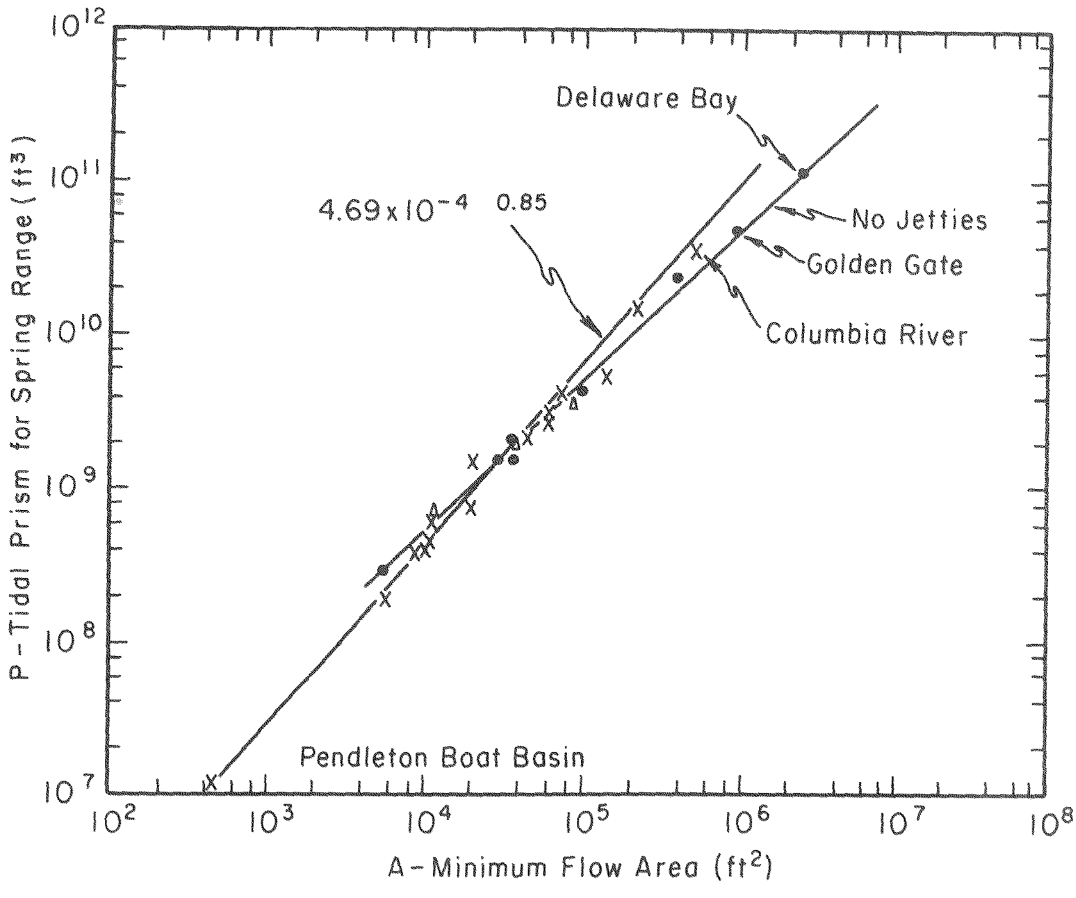

Figure 10. Tidal prism-inlet area relationship.

Various theories relating to the stability of inlets have been proposed. Bruun and Gerritsen (1960) and Bruun (1966,1973) proposed the ratio Ω/M as a measure of stability where M is the annual gross littoral drift. Bruun (1973) observed that:

$$\Omega/M > 200 \text{ gave good stability}$$

$$200 > \Omega/M > 100 \text{ gave fair stability}$$

$$100 > \Omega/M \quad \text{gave poor stability}$$

Bruun and Gerritsen (1960) reported that for a diurnal tide the ratio Ω/M should be replaced with the ratio $\Omega/2M$. Later reports by Bruun (1966, 1973) do not refer to ratio $\Omega/2M$ and the impression is that it should be used in all cases.

Carothers and Innis (1960) discussed the stability of inlets and offered an explanation of how the processes of accretion and erosion on the outer bar provide ". . .the automatic mechanism to maintain dynamic balance between the various rates of sediment and intercepted littoral transport." According to the authors this means that when the littoral drift brings an excess of sediment into an inlet, the sediment is deposited on the outer bar during the floodtide and is removed by the ebbtide, and returned to the littoral drift. However, Carothers and Innis do not propose a formula to determine whether a given inlet is stable or unstable.

Johnson (1973) studied inlets on the coasts of Washington, Oregon, and California and concluded that, "Wave power appears to be the most important factor affecting the stability of tidal inlets." He quotes O'Brien (1931, 1966) as proposing for a closure criterion, C_1, the ratio of the wave energy to the tidal energy per tidal cycle or

$$C_1 = \frac{E_S T_W}{\Omega(2a_o)\gamma} , \qquad (45)$$

". . . in which E_s = wave energy, in foot pounds per foot of beach per second; T = duration of tidal cycle, in seconds; w = width of entrance, in feet; a_o = tide amplitude, in feet; and γ = unit weight of water, in pounds per cubic foot." O'Brien reasoned that there exists a critical value of this ratio and that if C_1 exceeds the critical value, the inlet remains open but if C_1 is less than the critical value, the inlet closes if the storm duration exceeds a certain time. However, Johnson found that insufficient data were available to apply this criterion and used the simplified procedure of comparing the estimated annual wave power with the potential tidal prism. The potential tidal prism is the tidal prism that would be obtained if the full tidal range of the ocean were admitted into the bay. Johnson (1973) gave the results of his study in graphic form by plotting the annual wave power against the potential tidal prism and drawing a line to separate the inlets that have closed from those that have remained open. That line can be represented by the formula:

$$W_p = 7.32 \times 10^{11}\ \Omega_p^{0.20} \tag{46}$$

where W_p is the annual wave power (sum of sea and swell) in deep water near each inlet in foot pound per foot per year and Ω_p is the potential tidal prism.

O'Brien and Dean (1972) have proposed a stability index, β, defined as:

$$\beta = \int_{a_p}^{a_E} (V_m - V_T)^3\ da_c \tag{47}$$

where V_m is the maximum velocity in the gorge of the inlet, V_T is the threshold velocity for sand transport, a_c is the cross-sectional area in the gorge, a_p is the value of a_c at the peak of the V_m curve, and a_E is the value of a_c at the intersection of that curve with the O'Brien equilibrium velocity curve (Fig. 11) as discussed below. The stability index, β, represents the capacity of an inlet to resist closure under conditions of deposition.

A method for investigating the stability of inlets was suggested by Escoffier (1940). He proposed a diagram in which the velocity, V_m, is plotted against the cross-sectional area, a_c, as the latter is assumed to vary over a wide range of values. It was assumed that equilibrium required a particular value for V_m and accordingly, that value was plotted as a straight line parallel to the a_c axis. However, the use of Keulegan's and O'Brien's formulas permits the construction of a better diagram, and the equilibrium value for V_m is not a constant but varies with the repletion coefficient, K.

It is convenient to express K as:

$$K = \sigma\alpha\xi \tag{48}$$

where

$$\sigma = \frac{T\sqrt{2ga_o}}{2\pi a_o} \tag{49}$$

$$\alpha = \frac{a_c}{A_b} \tag{50}$$

and (eq. 5)

$$\xi = \left(\frac{2gLn^2}{1.486^2 R^{4/3}} + m \right)^{-1/2} .$$

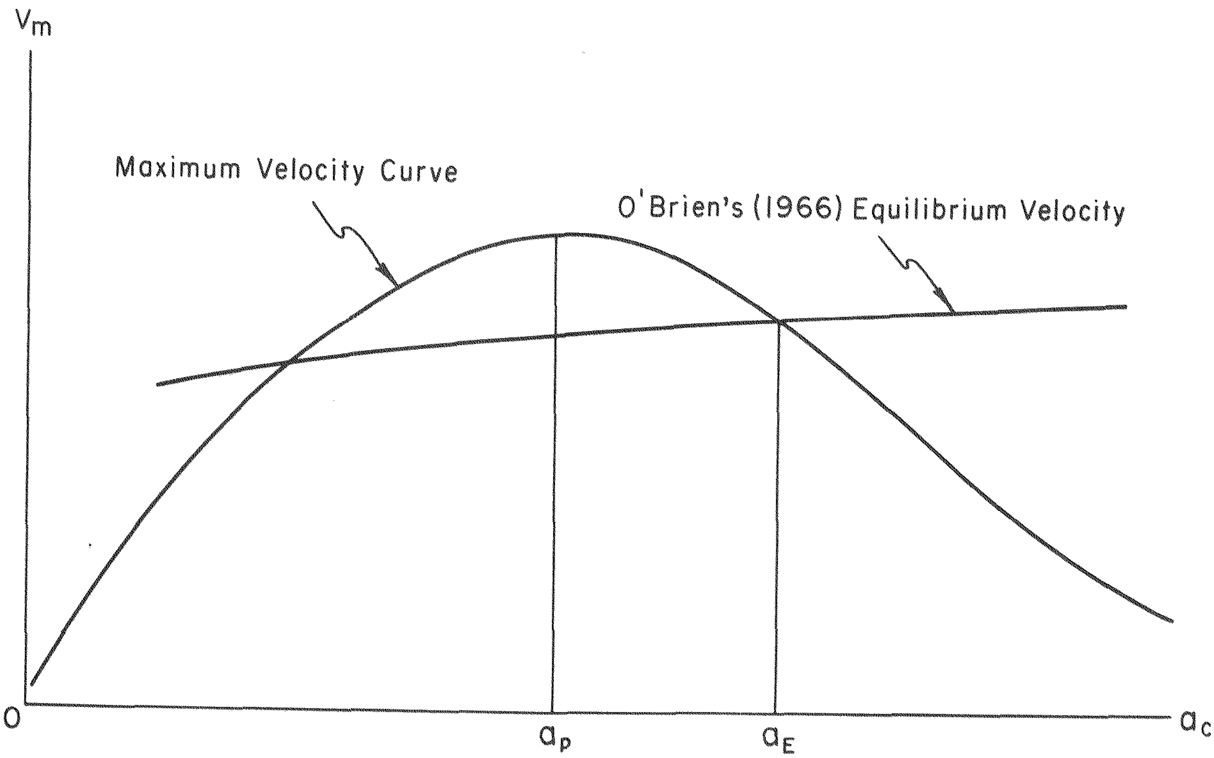

Figure 11. Graphic description of integration limits a_p and a_E.

Equations (30), (32), and (50) are used to arrive at the following expression for V_m:

$$V_m = \frac{2\pi a_o}{T\alpha} C' \sin \tau \ , \tag{51}$$

and equations (48), (49), and (50) are then used to obtain

$$V_m = \sqrt{2ga_o}\ \varepsilon\ \xi \tag{52}$$

where

$$\varepsilon = \frac{C \sin \tau}{K} \ . \tag{53}$$

Conveniently this introduces the dimensionless velocity:

$$\nu = \frac{V_m}{\sqrt{2ga_o}} = \varepsilon\ \xi \ . \tag{54}$$

Equation (28) is used to eliminate Ω from equation (44) to yield:

$$a_c = b(2A_b a_o \sin \tau)^N \ . \tag{55}$$

Equations (48) and (50) are then used to eliminate a_c from equation (55) and to obtain

$$\xi_E = \frac{1}{\beta_*} K (\sin \tau)^{-N} \tag{56}$$

where

$$\beta_* = \frac{bT\sqrt{2ga_o}}{\pi} (2A_b a_o)^{N-1} \tag{57}$$

The subscript E has been added to indicate the equilibrium value for ξ. Accordingly, the equilibrium value for the dimensionless velocity ν becomes:

$$\nu_E = \varepsilon \xi_E \ . \tag{58}$$

Figure 12 shows ν and ν_E plotted as functions of K. The intersections A and B in the figure are points of equilibrium with A an unstable point and B a stable one; this can be seen by considering that a small deviation from the conditions represented by point A sets into operation forces that tend to increase or reinforce the deviation, whereas a similar deviation at point B sets into operation forces that tend to restore the inlet to its equilibrium point. If the deviation at point A is an increase in the size of the inlet the velocity of the water will increase and the consequent erosion will cause the inlet to increase further in size. A similar deviation at point B will cause

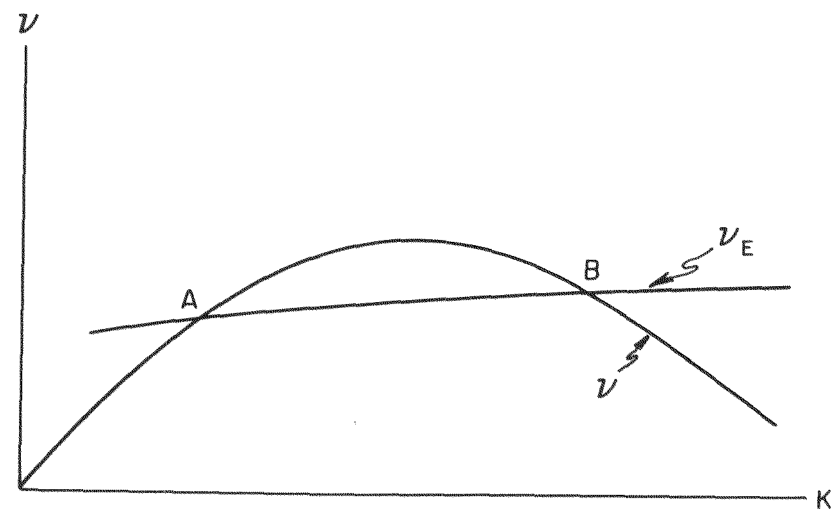

Figure 12. ν_E and ν versus repletion coefficient, K.

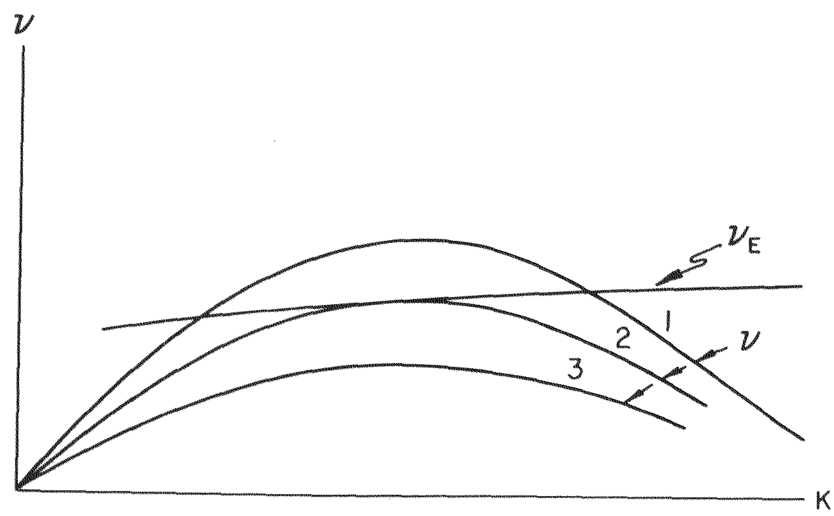

Figure 13. ν_E and various values of ν versus repletion coefficient, K.

the velocity to diminish and the consequent accretion will cause the inlet to diminish in size and therefore to move back toward the point B. A decrease in the inlet area at point A will diminish the flow velocity to cause a further area decrease, etc., until closure. A decrease in area at point B increases the flow velocity causing scour and a return to point B. Figure 13 shows three possible relative positions between the two curves. The first ν curve plots high enough to intersect the ν_E curve in two points, one unstable and the other stable. The second ν curve has only a point of tangency with the ν_E curve which point must be classed as an unstable one. The third ν curve fails to touch the ν_E curve and consequently stability is not possible.

All of the stability formulas that have been mentioned are empirical and subject to revision with the progress of research. The O'Brien formula and the stability diagrams (Figs. 12 and 13) which are based on it do not explicitly contain the littoral drift or the wave power as a variable. However, it can be assumed that either or both of these quantities play a part in determining the position of the curve and in this way influence the equilibrium size and the stability of an inlet; e.g., the assumption is that the high position of the first ν curve in Figure 13 is the reflection of a small littoral drift or a small wave power. The lower positions of the other two curves might then be reflections of greater littoral drifts or greater wave powers. The highest curve can thus be regarded as the most stable which suggests that the height of the curve might, in some way, be translated into a measure of stability. An examination of equation (47) and Figure 11 indicates that to some extent the stability index, β, as defined by equation (47), is such a measure; the limits of integration, a_p and a_E, prescribe a range that varies roughly with the height of the ν curve above the O'Brien equilibrium curve. This observation is not offered as the sole justification for the formula under consideration but simply as an indication of how the size of the littoral drift or of the wave power may enter into the determination of the stability index, β.

The idea of translating the height of the ν curve into a measure of stability can also be carried out as follows. Figure 14 shows an ν and an ν_E curve. The ratio of ν to ν_E for the value of K, which makes ν a maximum, yields the dimensionless number,

$$\lambda = \left(\frac{\nu}{\nu_E}\right)_{max\ \nu} \quad . \tag{59}$$

The condition $\lambda > 1$ indicates stability, but the value of λ can be taken as a measure of the degree of stability. However, to convert this concept into a usable form it will be necessary to adopt an appropriate expression for ν as a function of the repletion coefficient, K. It is here assumed that as erosion or accretion causes the dimensions of an inlet to change, the quantities, a_c and ξ, will vary according to the formulas:

$$a_c = a' \left(\frac{R}{R_o}\right)^s \tag{60}$$

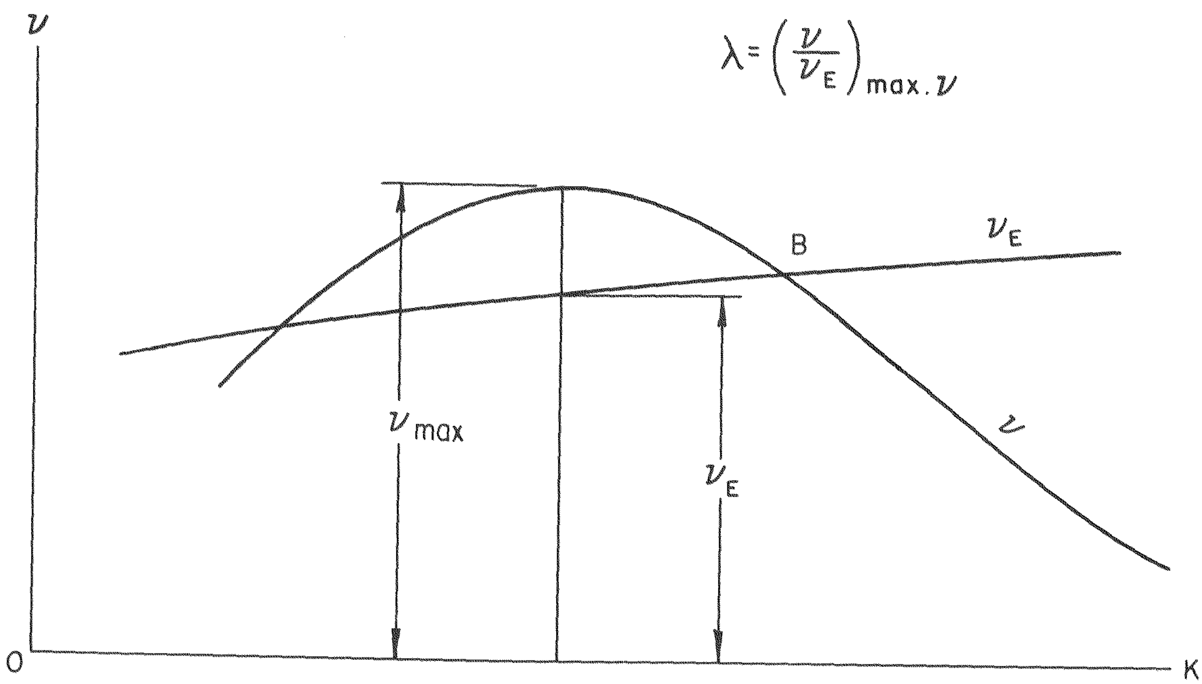

Figure 14. Definition diagram for the stability number λ.

and

$$\xi = \xi' \left(\frac{R}{R_o}\right)^j \tag{61}$$

where the subscript o identifies the initial or unchanged values. From equations (60) and (61) the following equations are derived:

$$K = K' \left(\frac{R}{R_o}\right)^{s+j} \tag{62}$$

$$\xi = \xi' \left(\frac{K}{K_o}\right)^{\frac{j}{s+j}} \tag{63}$$

$$\nu = \varepsilon\xi = \varepsilon\xi' \left(\frac{K}{K'}\right)^{\frac{j}{s+j}} . \tag{64}$$

For the exponents, the values $s = 5/2$ and $j = 2/3$ are chosen. The first value is suggested by the regime theory (Henderson, 1966) which expresses that the ratio of width to depth is generally greater in large channels than in small ones. The second value is derived from equation (5) where it is assumed that the influence of the coefficient, m, is negligible. The exponent in equations (63) and (64) becomes, to a sufficient degree of accuracy, 0.21; equation (64) then becomes:

$$\nu = \xi' \; K'^{-0.21} \; \varepsilon \; K^{0.21} , \tag{65}$$

as the relationship between ε and K is known (established by Keulegan's method in equation 53), it is possible to determine the value of K for which ν is a maximum. At that point it is found that

$$K = 0.64$$

$$\sin\tau = 0.651$$

$$\varepsilon = 0.840$$

$$\varepsilon K^{0.21} = 0.765 .$$

From equation (56) is obtained

$$\xi_E = \frac{1}{\beta_*} K \,(\sin\tau)^{-N} = \frac{1}{\beta_*} \times 0.64 \times 0.651^{-N}$$

and therefore,

$$\lambda = \frac{\nu}{\nu_E} = \frac{0.765 \; \xi' \; K'^{-0.21}}{0.840 \times 0.64 \times 0.651^{-N} \times \frac{1}{\beta_*}}$$

$$= 1.423 \times 0.651^N \; \beta_*\xi' \; K'^{-0.21}$$

$$= 0.988 \; \beta_*\xi' \; K'^{-0.21} \quad \text{for} \quad N = 0.85$$

$$= 0.926 \; \beta_*\xi' \; K'^{-0.21} \quad \text{for } N = 1.00 ,$$

or to a sufficient degree of accuracy,

$$\lambda = \beta \xi'_* K'^{-0.21} \quad . \tag{66}$$

When this formula is used it must be understood, that the favorable condition, $\lambda > 1$, does not imply that the point at $K = K'$ is a stable point but only a stable solution exists; i.e., the one represented by the point B in Figure 14. If a point of stable equilibrium exists it will correspond to a value of K larger than the one for which ν is a maximum, i.e., larger than about $K = 0.64$.

Problems arise when a bay communicates with the sea through two or more inlets. If the water surface in the bay remains horizontal throughout the tidal cycle, the following conclusions can be drawn from the theories of stability:

(a) The effective repletion coefficient, K, for the system as a whole is the sum of the values of that coefficient for the individual inlets.

(b) A single value of ξ_E, which depends on the system K, holds for all of the inlets and equilibrium for the system requires that the value of ξ for each inlet shall be equal to that value.

(c) The system is unstable. If the value of ξ for any inlet is slightly greater than the values for the other inlets, that inlet will develop a higher velocity than the others and will enlarge until it captures the entire tidal prism.

However, observation indicates that some bays which communicate with the sea through two or more inlets are stable. For example, the bay formed by Gasparilla Sound, Charlotte Harbor, and Pine Island Sound on the west coast of Florida has six inlets and the system appears to be stable. However, since the bay has a large expanse and is mostly shallow, it can be assumed that there are significant variations throughout the bay in the the amplitude and in the phase of the tide. A greater phase lag may occur at a small inlet and cause larger velocities in that inlet than at the other inlets. The resulting erosion would cause the small inlet to enlarge until the increase in flow through the inlet causes enough of a reduction in the phase lag to establish a condition of equilibrium.

To determine the stability of one of several inlets serving a bay, calculate the amplitudes and phase lags of the tide in the bay, using a finite-difference method such as the one developed by Dronkers (1964). This is done twice for two assumed sizes of the inlet, the initial size and a larger or smaller hypothetical size that represents the result of assumed erosion or accretion. After the amplitudes and the phase lags for the two assumed sizes of the inlet have been determined, the maximum velocity, V_m, for each case is calculated. If V_m is found to decrease when the size of the inlet increases or to increase when the size of the inlet decreases the inlet is judged to be stable. Otherwise, it is unstable.

V. FUNCTIONAL DESIGN REQUIREMENTS

In planning the improvement of an inlet, possible objectives which may be involved are:

(a) Improving the sediment flushing capacity of the inlet. This means increasing the capacity of the ebbtide to carry out the sand brought into the inlet by the floodtide.

(b) Stabilizing the inlet. An inlet may not be in a condition of stability because the channel has too small a cross-sectional area or too great a length (see Section IV). Then appropriate steps to enlarge the cross section or to shorten the channel should be taken. Also, the inlet may be migrating too rapidly or the part of the channel over the outer bar may be shifting excessively. Steps to arrest these processes may then be desirable.

(c) Providing a channel of adequate dimensions for navigation.

Before discussing means for improving the flushing capacity of an inlet, some of the natural factors that enter into the determination of that flushing capacity should be mentioned. These include:

(a) The freshwater that enters the bay from its tributaries. This inflow causes the ebb flow through the inlet to exceed the floodflow and thus favors the outward transport of sand.

(b) At many inlets the tidal flow has a long runout; i.e., higher high water is followed by lower low water and the outward flow of the tidal prism takes place in a shorter period of time than the inward flow. Consequently, the transporting capacity of the ebb current tends to be greater than that of the flood current. At a few inlets, the tidal sequence is reversed, a long run-in exists, and the flood current tends to have the greater transporting capacity.

(c) The depth of water in the inlet channel is less on the ebbtide than on the floodtide. This gives rise to higher velocities and a greater transporting capacity on the ebbtide.

(d) The water-surface area of a bay is greater at high tide than at low tide which causes the velocities and the transporting capacity to be greater on the ebbtide than on the floodtide.

Items (c) and (d) have been discussed by Oliveira (1970). These factors become important when a bay is large and shallow because bottom friction causes the tidal variations in the distant parts of the bay to lag behind those near the inlet. This causes the water depth in the inlet to be less and the water-surface area in the bay to be greater at the time of the strongest ebb current. Consequently, the outward transport of sand is favored over the inward transport. Another factor that has a bearing on the transporting capacity of a current is the presence of acceleration or deceleration in that current. The transporting capacity of an accelerating current is greater than that of a decelerating current. If the banks of an inlet converge in the direction of the ebb current, that current will have a greater transporting capacity than the flood current.

The flushing capacity of an inlet can be increased by shortening the channel and thereby reducing the resistance to the tidal flow. This can sometimes be done by changing the alinement of the channel or by dredging part of the inner shoal. The flushing capacity can also be increased by using jetties and bank paving to give the inlet an alinement that is convergent for the ebb current.

The means available for improving an inlet include: (a) dredging, (b) bank protection, (c) jetties, and (d) artificial bypassing. In inlets with a high degree of natural stability it is sometimes possible to achieve the desired improvement with dredging only. Dredging is used initially to obtain the desired channel dimensions and is then used periodically to remove the sand deposited in the channel by the waves and the currents. Bank protection by revetments or walls can be used to hold the alinement of the inlet and to reduce shoaling in the inlet by preventing bank erosion.

Generally, jetties are necessary to stabilize a channel across the outer bar and to prevent the migration of the inlet. Jetties serve as a nozzle to direct the ebb current across the outer bar at the point where a channel is desired and as breakwaters to reduce wave action in the harbor, but exclude, to some extent, wave-driven sand from the inlet. By giving the jetties and the inlet banks an alinement that converges toward the sea, the gorge is made to occupy a position at or near the outer ends of the jetties. The maximum velocity will then occur at that point and the scouring action of the ebb current on the outer bar will be improved.

Ordinarily, the outer ends of the jetties are extended to the point where the natural depth is equal to the project depth of the navigation channel. To determine the spacing between the outer ends of the jetties, two considerations are the needs of navigation and the desirability of having a gorge area that satisfies, at least approximately, the O'Brien equilibrium formula. However, some variation in the value of the area is possible when conforming to the O'Brien formula. By increasing ξ, i.e., by shortening or deepening the channel, the tidal prism and the cross-sectional area can both be increased. A limitation to this is that the tidal prism often cannot be increased beyond the value that corresponds to the admission into the bay of the full tidal range in the sea. If an entrance with satisfactory dimensions for navigation cannot be achieved within the limits imposed by the O'Brien formula, it is possible to disregard that formula by adopting a nonscouring inlet. The theory of nonscouring inlets is discussed later in this section. The determination of the dimensions of a navigable inlet channel is beyond the scope of this report. This subject is discussed in Waugh (1971), Kray (1973), and Dunham and Finn (1974).

Most natural inlets withdraw sand from the adjacent beaches and deposit the sand on the outer bars or on the bay shoals. To the extent that a jetty system reduces these losses it benefits the beaches. However, unless the longshore transport in one direction is approximately equal to that in the opposite direction the jetty system will ordinarily cause accretion on one side of the inlet and erosion on the other. Since the amount of longshore transport bypassed by the currents and waves is rarely equal to the net longshore transport, it is usually desirable to provide for artificial bypassing (see U.S. Army, Corps of Engineers, Coastal Engineering Research Center, 1975).

The previous discussion assumed that the inlet to be improved is naturally stable, that the gorge area will remain approximately the same, and that artificial bypassing will only be necessary to supplement the natural bypassing of the inlet. However, in some cases a navigable passage may be desired into a harbor or a bay that is too small to provide either a stable channel or one that meets the needs of navigation. It is possible in such a case to obtain a channel of the desired dimensions by converting the inlet to a nonscouring inlet. The jetties are extended into exceptionally deep water to completely exclude the littoral drift. The channel is then dredged to the desired dimensions but must be deep enough to preclude the movement of sand in the channel by either the tidal currents or the waves. Under these conditions it is possible to obtain a gorge area larger than the equilibrium area given by the O'Brien formula. Since no bypassing of sand by the tidal currents and wave will occur, the entire net littoral drift must be passed artificially. The combination of such an oversized inlet with a small inner basin may lead to the admission of excessive wave action into the bay; consequently, consideration should be given to the possible need for wave absorbers or spending beaches in the basin. Four nonscouring inlets on the Pacific coast in southern California are Marina del Rey, Alamitos Bay, Newport Beach, and Mission Bay. Mission Bay Inlet is discussed in Section VI; a graph of several nonscouring inlets plotted on an O'Brien chart is shown in Figure 15.

VI. CASE STUDIES

The three inlets discussed in this section were selected because each represents a kind of problem that can arise in the improvement of

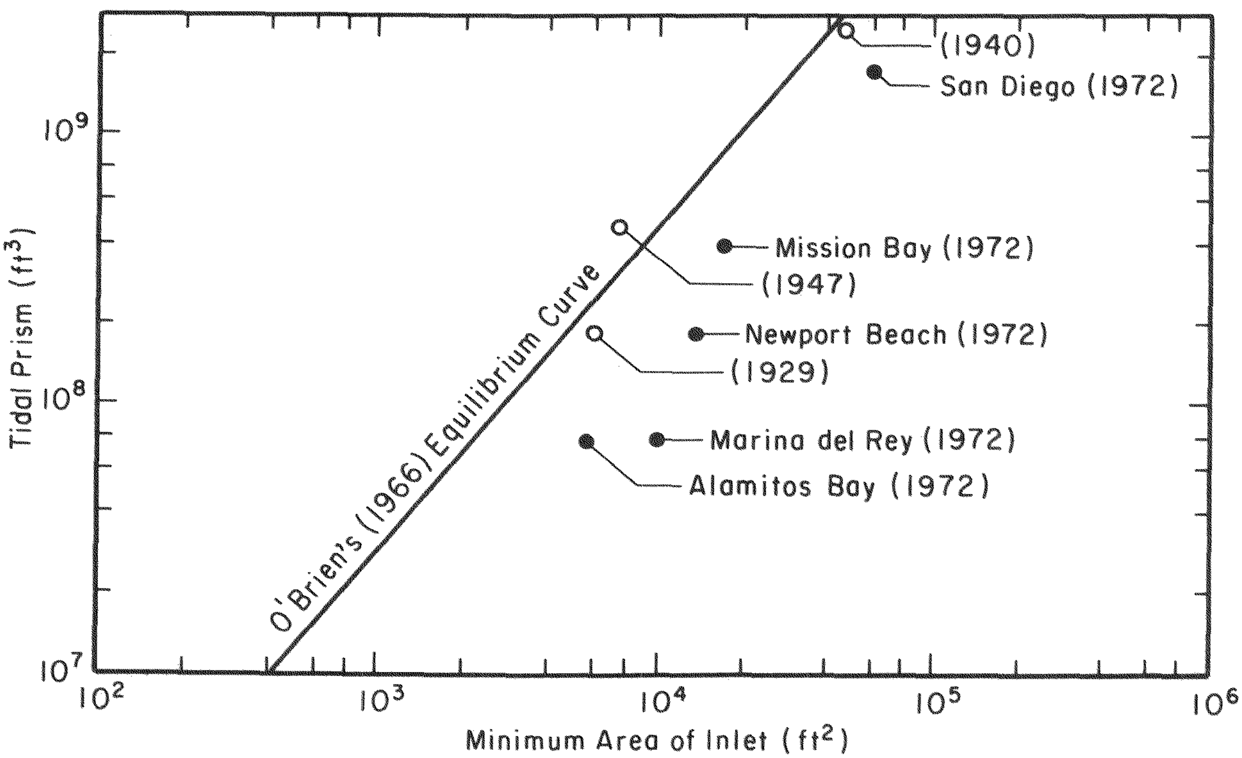

Figure 15. Nonscouring channels, southern California.

inlets. Masonboro Inlet, North Carolina, was the first inlet to be equipped with a jetty containing an artificial sand-passing weir; Rollover Fish Pass, Texas, answers some questions about stability when an artificial inlet is connected to a bay already served by an existing inlet; and Mission Bay Inlet, California, is a good example of a nonscouring inlet.

1. Masonboro Inlet, North Carolina.

Experience with the natural sand-passing weir provided by a rock reef at Hillsboro Inlet, Florida, furnished much of the guidance for the design of the artificial weir at Masonboro Inlet. Details are given by Magnuson (1965, 1967), Rayner and Magnuson (1966), Watts, Vallianos, and Jachowski (1973), and Dean and Walton (1975). A general plan of the inlet is shown in Figure 16.

The existing navigation project provides for a channel 14 feet deep at mean low water (MLW) and 400 feet wide across the outer bar, and thence 12 feet deep and 90 feet wide to the Atlantic Intracoastal Waterway. The mean tidal range in the ocean is 3.8 feet and the spring range is 4.5 feet. The range in the bay near the inlet is about 0.5 foot less than that in the ocean. The longshore transport rate is estimated to be 120,000 cubic yards per year northward and 220,000 cubic yards per year southward. The net longshore transport rate is the difference between these two or about 100,000 cubic yards per year southward.

One jetty constructed on the north side of the inlet is 3,600 feet long, contains a sand-passing weir 1,000 feet long, and has been in operation since July 1966. The elevation of the crest of the weir (made of concrete sheet piling) is 2 feet above MLW. A deposition basin dredged southwest of the weir is about 1,300 feet long, 400 feet wide, 16 feet deep at MLW, and is a minimum distance of 200 feet from the weir. About 376,300 cubic yards of sand was removed from the deposition basin and placed on Wrightsville Beach north of the inlet.

Since completion of the north jetty, the channel has moved toward the jetty and has encroached on the deposition basin. Therefore, it has been necessary to place side-slope protection along the lee side of the jetty to prevent undermining. Accretion has also caused the south bank of the inlet to advance northward. Three factors are believed to have played a part in causing the channel to move northward:

(a) The momentum of the ebb current carries it against the jetty which then acts as a training wall to deflect the current toward the southeast.

(b) The jetty shelters the south outer shoal from waves that would naturally tend to drive the sand southward but leaves the shoal exposed to waves that tend to drive it northward.

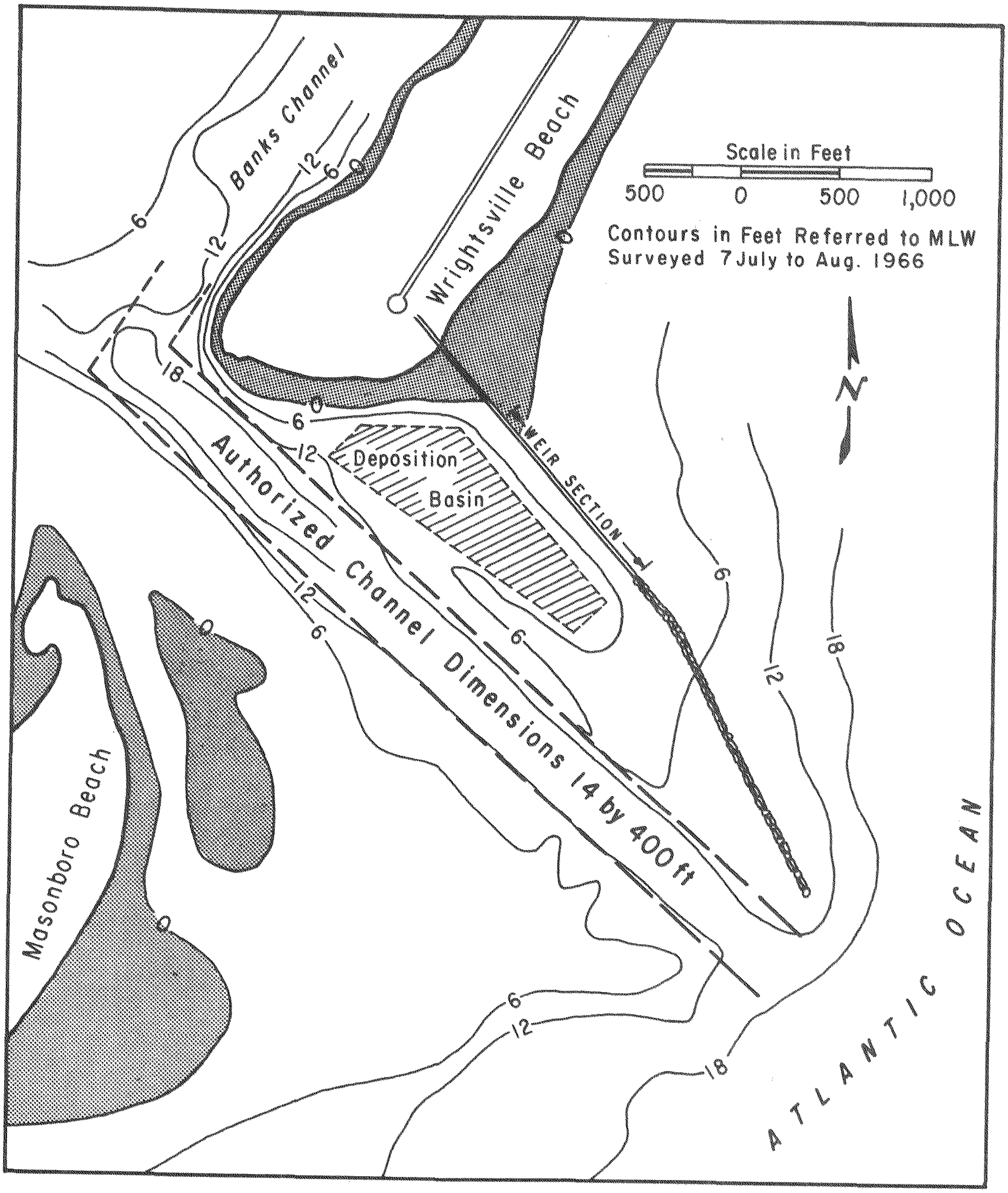

Figure 16. Masonboro Inlet, North Carolina.

(c) The greater length of the deposition basin in a northwest to southeast direction provided a path of less resistance for the tidal currents.

Construction of a south jetty is currently planned. Model studies have been made at the U.S. Army Engineer Waterways Experiment Station (WES) to determine the alinement and other features of that jetty. Experience at Masonboro Inlet indicates that sand passes over the weir and that the weir and the outer part of the jetty offer sufficient protection for a pipeline dredge. It can be anticipated that when the south jetty is built and the other improvements indicated by the model study are incorporated into the inlet, the position of the channel will be more stable.

2. Rollover Fish Pass, Texas.

During 1954-55, the Texas Game and Fish Commission (now the Parks and Wildlife Department) excavated an artificial inlet from the Gulf of Mexico into East Bay, an arm of Galveston Bay (Fig. 17). Galveston Bay was already served by two inlets, Galveston (or Bolivar) Inlet and San Luis Pass. However, San Luis Pass is located in West Bay, another arm of Galveston Bay, and is not believed to play a significant part in determining the tidal characteristics of Galveston Bay because of the shallowness of the water that connects the two sections of the bay. Therefore, it can be assumed for this study that the main body of Galveston Bay and East Bay, Galveston Inlet, and the newly excavated Rollover Fish Pass, constitute a system that can be considered as a unit.

Shortly after the Rollover Fish Pass (Fig. 18) was excavated, the tidal currents began to erode the channel and it was necessary to drive a wall of sheet piling across the channel to stop the flow of water. Later, some of the sheet piling was driven low enough to convert the wall into a weir; the inlet has since remained stable.

The theory of inlet stability based on an assumed horizontal water surface in a bay leads to the conclusion that if the bay is served by two or more inlets, the inlet with the largest value of ξ will gradually capture the entire tidal prism of the bay and the other inlets will close (see Section IV). However, a number of bays served by more than one inlet are known to exist and are obviously stable. Apparently, the reason for this is that the water surface in these bays does not remain horizontal but varies from place to place in both amplitude and phase.

A simple formula can be derived to show the effect of the amplitude and phase of the inner tide on the difference in water levels between the sea and the bay if a simple sinusoidal tide is assumed in both bodies of water. Thus,

$$\eta = a_o \sin \omega t - a_b \sin (\omega t - \phi) \qquad (67)$$

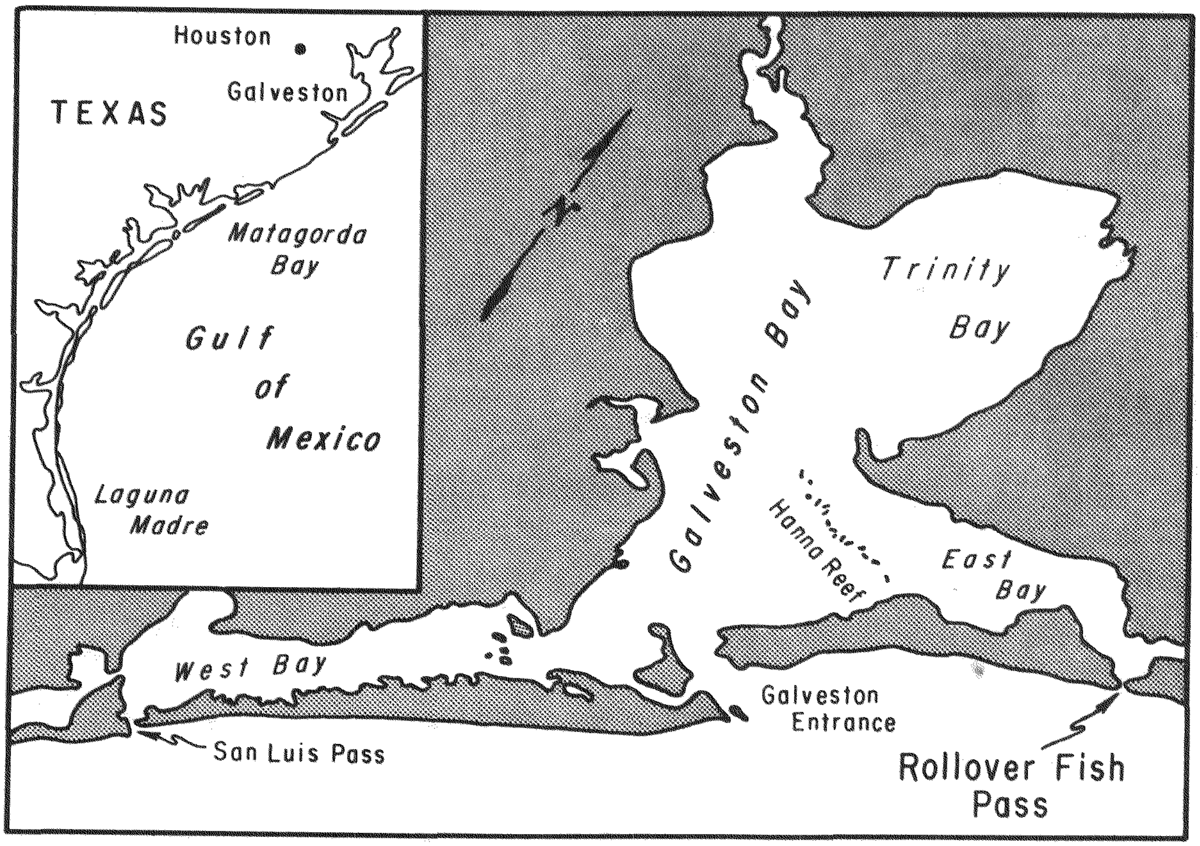

Figure 17. Area map showing location of Rollover Fish Pass, Texas.

Figure 18. Rollover Fish Pass, Texas, November 1970.

where ϕ is the phase angle of the tide in the bay near the inlet. Equation (67) can be rewritten as

$$\eta = \eta_m \sin (\omega t + \gamma) \ , \tag{68}$$

where

$$\eta_m = \sqrt{(a_o - a_b \cos \phi)^2 + (a_b \sin \phi)^2} \ , \tag{69}$$

and

$$\tan \gamma = \frac{a_b \sin \phi}{a_o - a_b \cos \phi} \ . \tag{70}$$

The maximum tidal difference on the inlet is η_m and equation (69) can be used to show the influence of the amplitude, a_b, and phase angle, ϕ, on that difference.

The maximum head differential, η_m, plays a significant part in determining whether an inlet erodes or accretes. The amplitudes and lags shown in Table 5 were taken from a study of Rollover Fish Pass by Prather and Sorensen (1972) and from tide tables (National Oceanic and Atmospheric Administration, 1976). Amplitudes and phases for a number of points near Galveston Inlet were used to derive representative values.

Table 5. Tidal amplitudes, phase lags, and maximum head differential at Galveston Bay, Texas.

Location	a_o (ft)	a_b (ft)	Phase lag (h)	ϕ	η_m (ft)
Galveston Inlet	0.9	0.63	1.0	14°	0.33
Rollover Fish Pass	0.9	0.85	3.5	51°	0.75
Rollover Fish Pass	0.9	0.63	3.5	51°	0.70

The amplitude of the bay tide at Rollover Fish Pass is larger than that at Galveston Inlet and is almost as large as the amplitude in the gulf. This large value is believed to be caused by the convergence of the shorelines in East Bay. However, to show the effect of the phase lag on the differential η_m, a second calculation was made in which the amplitude at Rollover Fish Pass was assumed to be equal to that at Galveston Inlet. The results show that in either case the value of η_m at Rollover Fish Pass is significantly greater than that at Galveston Inlet and that this is due primarily to the greater phase lag at Rollover Fish Pass.

Values of ξ at Galveston Inlet and at Rollover Fish Pass are estimated to be 0.4 and 0.5, respectively. In deriving the value for Rollover Fish Pass it was assumed that the sheet-piling weir was not in place. A comparison of the two values of ξ suggests that even if the water surface in the bay remained horizontal there would be a tendency for Rollover Fish Pass to enlarge at the expense of Galveston Inlet. This tendency is magnified by the large value of η_m at Rollover Fish Pass.

It does not necessarily follow that the smaller inlet, if permitted to continue enlarging, would have captured the entire tidal prism. The increasing tidal flow through that inlet would have caused a reduction in its phase lag and a condition of stable equilibrium may possibly have been reached.

Therefore, equilibrium and stability formulas based on an assumed horizontal water surface in a bay with more than one inlet are only rough approximations. For a more reliable evaluation of the equilibrium and stability of the inlets, the actual calculation of the bay tides using the methods discussed in Section IV are necessary. Unless the inlets can be shown in this way to be stable, the designer should recognize the possibility of instability and the possible need for controlling structures; e.g., side walls, bottom sills, and weirs.

3. Mission Bay Inlet, California.

Mission Bay Inlet (Fig. 19) is unusual for two reasons: (a) It is one of a small group of inlets in southern California designed as nonscouring inlets (others are Marina del Rey, Alamitos Bay, and Newport Harbor); and (b) in common with Alamitos Bay, it is equipped with three jetties. The passage between the north and the middle jetties is the harbor entrance; the passage between the middle and the south jetties is the channel of the San Diego River to the ocean, carrying a sediment load to the outer shore rather than depositing it in the bay (Herron, 1972).

The bay has a surface area of about 2,000 acres. Originally, the San Diego River discharged into the bay and deposited a load of sediments. The inlet between the bay and the ocean was a small shifting channel with a controlling depth of about 6 feet at mean lower low water (MLLW). Mission Beach is confined between the two headlands of La Jolla and Point Loma. The principal source of sand for the beach is the San Diego River. The longshore transport is northward in the summer and southward in the winter, the two movements being approximately in balance.

The mean ocean tide at the inlet is 3.8 feet and the springtide is 5.4 feet. Mean sea level is 2.8 feet above MLLW. The tides have a diurnal inequality with a long runout which, under natural conditions, causes the ebb current to have a greater sediment transporting capacity than the flood current.

In the theory of nonscouring inlets, an inlet is given a cross-sectional area large enough to prevent significant movement of sand by the tidal currents and deep enough to prevent the breaking of waves. In

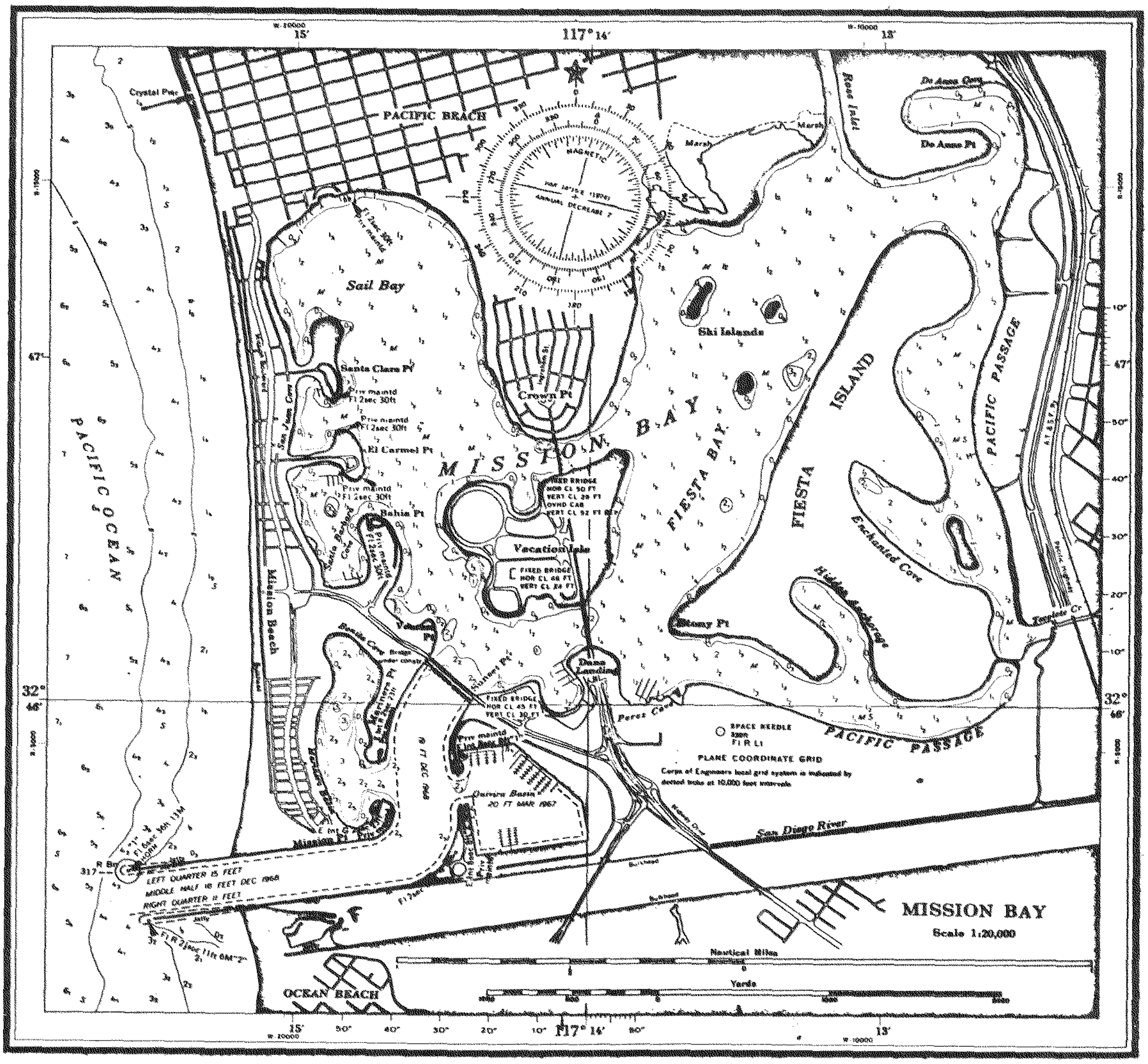

Figure 19. Mission Bay Inlet, California.

Mission Bay a limiting velocity of 2 feet per second was adopted as a nonscouring velocity and a depth of 20 feet was adopted as one which would allow the 16-foot design wave to enter without breaking. The cross-sectional area is more than twice that required by the O'Brien equilibrium formula. The north and the middle jetties were extended to the 25-foot depth contour to prevent any movement of sand around the ends of the jetties from the adjacent beaches. The jetties were spaced about 9,000 feet apart and the channel in between was dredged to a depth of 20 feet below MLLW to provide the desired cross-sectional area of 19,800 square feet below mean tide level. A number of nonscouring inlets, including Mission Bay, have been plotted on an O'Brien chart in Figure 15.

However, even with a depth of 20 feet below MLLW the waves in the entrance were so steep as to constitute a serious hazard to small boats. Accordingly, the outer 1,000 feet of the channel were dredged to a 25-foot depth which greatly improved the navigability of the outer part.

The unusually large entrances at Mission Bay, Alamitos Bay, Newport Harbor, and Marina del Rey admit much wave energy which must be disposed of in some way. At Mission Bay, problems were experienced in the two deepwater anchorages, Quivera Basin and Mariners Basin, because of waves that were as high as 2.5 feet. A model study made at WES concluded that excessive wave energy was reaching the inshore end of the entrance channel where that channel makes a 90° left turn into the main channel. The shore at this point is protected by a semicircular rock-revetted slope which reflects too much energy and tends to focus it on the entrance to Mariners Basin. Of the corrective measures tested, the most promising was to convert the semicircular revetment to a series of straight-revetted sections in echelon that would tend to reflect this energy back out the entrance channel to the ocean.

The advantage of a nonscouring inlet is that it provides a safer and a more reliable entrance channel. The greater channel depth allows deeper-draft vessels to use the harbor. The greater width of channel, the lower water velocities, and the absence of breaking waves facilitate the maneuvering of boats, particularly sailboats. The disadvantages are the excessive wave energy that may be admitted into the harbor and the possible need for artificially bypassing a larger amount of littoral drift than would otherwise be necessary.

LITERATURE CITED

BAINES, W.D., "Tidal Currents in Constricted Inlets," *Proceedings of the Sixth Conference on Coastal Engineering*, 1957, pp. 545-561.

BARWIS, J.H., "Annotated Bibliography on the Geologic, Hydraulic, and Engineering Aspects of Tidal Inlets," GITI Report 4, U.S. Army, Corps of Engineers, Coastal Engineering Research Center, Fort Belvoir, Va., and U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Jan. 1976.

BROWN, E.I., "Inlets on Sandy Coasts," *Proceedings of the American Society of Civil Engineers*, Vol. LIV, 1928, pp. 505-553.

BRUUN, P., "Tidal Inlets and Littoral Drift," Universitetsforlaget, Norway, 1966.

BRUUN, P., "Means of Controlling Littoral Drift to Protect Beaches, Dunes, Estuaries, and Harbour Entrances," XXIII International Navigation Congress, Ottawa, 1973.

BRUUN, P., and GERRITSEN, F., *Stability of Coastal Inlets*, North Holland, Amsterdam, 1960.

CAROTHERS, H.P., and INNIS, H.C., "Design of Inlets for Texas Coastal Fisheries," *Journal of the Waterways and Harbors Division*, Vol. 86, No. WW3, Sept. 1960, pp. 103-128.

COMMITTEE ON TIDAL HYDRAULICS, "Evaluation of Present State of Knowledge of Factors Affecting Tidal Hydraulics and Related Phenomena," Report No. 3, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., May 1965.

DEAN, R.G., and WALTON, T.L., Jr., "Sediment Transport Processes in the Vicinity of Inlets with Special Reference to Sand Trapping," *Estuarine Research*, Vol. II, Academic Press, New York, 1975, pp. 129-149.

DRONKERS, J.J., *Tidal Computations in Rivers and Coastal Waters*, Wiley, New York, 1964.

DUNHAM, J.W., and FINN, A.A., "Small-Craft Harbors: Design, Construction and Operation," SR-2, Stock No. 0822-00091, U.S. Government Printing Office, Washington, D.C., Dec. 1974.

ESCOFFIER, F.F., "The Stability of Tidal Inlets," *Shore and Beach*, Vol. 8, No. 4, 1940, pp. 114-115.

GALVIN, C.J., Jr., "Inlets and Wave Direction, Wave Climate and Coastal Processes," *Symposium on Water, Environment and Human Needs*, Massachusetts Institute of Technology, Oct. 1971, pp. 44-78.

HAY, D., "Harbour Entrances, Channels and Turning Basins," *The Dock and Harbour Authority*, Vol. 4, No. 567, 1968, pp. 269-276.

HENDERSON, F.M., *Open Channel Flow*, Macmillian, New York, 1966.

HERRON, W.J., Jr., "Case History of Mission Bay Inlet, San Diego, California," *Proceedings of the 13th Coastal Engineering Conference*, 1972, pp. 801-822.

JARRETT, J.T., "Tidal Prism - Inlet Area Relationships," GITI Report 3, U.S. Army, Corps of Engineers, Coastal Engineering Research Center, Fort Belvoir, Va., and U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Feb. 1976.

JOHNSON, D.W., *Shore Processes and Shoreline Development*, Wiley, New York, 1919.

JOHNSON, J.W., "Characteristics and Behavior of Pacific Coast Tidal Inlets," *Journal of the Waterways, Harbors and Coastal Engineering Division*, Vol. 99, No. WW3, Aug. 1973, pp. 325-339.

KEULEGAN, G.H., "Tidal Flow in Entrances Water-Level Fluctuations of Basins in Communication with Seas," Technical Bulletin No. 14, Committee on Tidal Hydraulics, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., July 1967.

KING, D.B., "The Dynamics of Inlets and Bays," Technical Report 22, Coastal and Oceanographic Engineering Laboratory, University of Florida, Gainesville, Fla., 1974.

KRAY, C.J., "Design of Ship Channels and Maneuvering Areas," *Journal of the Waterways, Harbors and Coastal Engineering Division*, Vol. 99, No. WW1, Feb. 1973, pp. 89-110.

KREEKE, J., "Water-Level Fluctuations and Flow in Tidal Inlets," *Journal of the Waterways and Harbors Division*, Vol. 93, No. WW4, Nov. 1967, pp. 97-106.

MAGNUSON, N.C., "Planning and Design of a Low-Weir Section Jetty at Masonboro Inlet, North Carolina," *Coastal Engineering Specialty Conference*, Santa Barbara, Calif., 1965, pp. 807-820.

MAGNUSON, N.C., "Planning and Design of a Low-Weir Section Jetty," *Journal of the Waterways and Harbors Division*, Vol. 93, No. WW2, May 1967.

MASCH, F.D., BRANDES, R.J., and REAGAN, J.D., "Numerical Simulation of Hydrodynamics (WRE)," App. 2, GITI Report 6, *Comparison of Numerical and Physical Models, Masonboro Inlet, North Carolina*, U.S. Army, Corps of Engineers, Coastal Engineering Research Center, Fort Belvoir, Va., and U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., (in preparation, 1977).

NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION, "Tide Tables, East Coast of North and South America Including Greenland," National Ocean Survey, Rockville, Md., 1976.

O'BRIEN, M.P., "Estuary Tidal Prism Related to Entrance Areas," *Civil Engineering*, Vol. 1, No. 8, 1931, pp. 738-739.

O'BRIEN, M.P., "Equilibrium Flow Areas of Tidal Inlets on Sandy Coasts," *Proceedings of the 10th Coastal Engineering Conference*, Vol. I, Sept. 1966, pp. 676-686.

O'BRIEN, M.P., and DEAN, R.G., "Hydraulics and Sedimentary Stability of Coastal Inlets," *Proceedings of the 13th Coastal Engineering Conference*, 1972, pp. 761-780.

OFFICE OF THE CHIEF OF ENGINEERS, "Tidal Hydraulics, EM 1110-2-1067," Washington, D.C., Aug. 1965.

OLIVEIRA, I.B., "Natural Flushing Ability in Tidal Inlets," *Proceedings of the 12th Coastal Engineering Conference*, Vol. III, Sept. 1970, pp. 1827-1845.

PIERCE, J.W., "Tidal Inlets and Washover Fans," *Journal of Geology*, Vol. 78, No. 2, Mar. 1970, pp. 230-234.

PILLSBURY, B., *Tidal Hydraulics*, U.S. Government Printing Office, Washington, D.C., 1940.

PRATHER, S.R., and SORENSEN, R.M., "A Field Investigation of Rollover Fish Pass, Bolivar Peninsula, Texas," Coastal and Ocean Engineering Division, Texas A&M University, College Station, Tex., 1972.

RAYNER, A.C., and MAGNUSON, N.C., "Stabilization of Masonboro Inlet, North Carolina and Adjacent Shores," *Shore and Beach*, Vol. 34, No. 2, Oct. 1966.

SEELIG, W., HARRIS, D.L., and HERCHENRODER, B., "A Spatially Integrated Numerical Model of Inlet Hydraulics," GITI Report 14, U.S. Army, Corps of Engineers, Coastal Engineering Research Center, Fort Belvoir, Va., and U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., (in preparation, 1977).

SHEMDIN, O.H., and FORNEY, R.M., "Tidal Motion in Bays," *Proceedings of the 12th Coastal Engineering Conference*, Vol. III, Sept. 1970, pp. 2225-2242.

SIMONS, D.B., and RICHARDSON, E.V., "Resistance to Flow in Alluvial Channels," Professional Paper 442J, U.S. Geological Survey, Washington, D.C., 1966.

TASK FORCE ON FRICTION FACTORS IN OPEN CHANNELS, "Friction Factors in Open Channels," *Journal of the Hydraulics Division*, Vol. 89, No. HY2, Mar. 1963, pp. 97-143.

U.S. ARMY, CORPS OF ENGINEERS, COASTAL ENGINEERING RESEARCH CENTER, *Shore Protection Manual*, 2d ed., Vols. I, II, and III, Stock No. 008-022-00077-1, U.S. Government Printing Office, Washington, D.C., 1975, 1,160 pp.

WATTS, G.M., VALLIANOS, L., and JACHOWSKI, R.A., "The Importance of Controlling Littoral Drift," Paper for Sec. 11, Subject 2, XXIII International Navigation Congress, Ottawa, Canada, 1973, pp. 254-257.

WAUGH, R.G., Jr., "Water Depths Required for Ship Navigation," *Journal of the Waterways, Harbors, and Coastal Engineering Division*, Vol. 97, No. WW3, Aug. 1971, pp. 455-474.

WICKER, C.F., "Design of Navigation Channels," unpublished memorandum, U.S. Army, Corps of Engineers, Coastal Engineering Research Center, Washington, D.C., Feb. 1967.

APPENDIX

SUGGESTIONS FOR INLET DESIGN

Information should be obtained concerning the needs of the people for whose benefits the inlet is to be developed. If navigation is involved, the size of the largest vessel that is to use the channel should be determined. The needs that may exist for the control of temperature, salinity, and pollution in the bay should be ascertained.

Available information concerning winds, waves, tides, tidal currents, and littoral drift should be collected. To the extent necessary this information will have to be supplemented by field observations.

Available maps and other forms of both early and recent surveys should be collected. It is largely by comparing successive surveys that the past performance of an inlet and the extent of past erosion or accretion of adjacent shores are determined.

The size of the sediments in the inlet and on the adjacent beaches should be ascertained. It should be determined whether there is any rock exposed or likely to be exposed in the inlet or on the adjacent shores.

People familiar with the inlet, the bay, and the adjacent shores should be questioned on past activity. A search should be made for available written accounts, such as newspaper articles, engineering reports, etc.

Inlet design procedure. Define the desired objectives of the design study. If the purpose of the improvement is to serve navigation, the dimensions of the channel and its alinement should be determined. If the purpose is to provide for the control of temperature, salinity, pollution, or other ecological factors, the inlet flows most advantageous should be determined.

Determine the equilibrium size of the inlet under consideration and evaluate its stability. Past performance of the inlet is important. However, the conclusions drawn from past performance can be supplemented by consideration of the various measures of equilibrium and stability described in Section IV.

Determine whether the desired improvement can be achieved by dredging alone. If this has already been tried, the results should be carefully evaluated. In general, an inlet can be maintained by dredging alone only if its equilibrium size is adequate and it is stable.

Determine whether jetties are necessary. If the past performance of the inlet indicates that it migrates rapidly, is subject to repeated closure, or shoals excessively, it is probably desirable to stabilize the inlet with jetties.

Determine whether a nonscouring inlet is needed. In general, the equilibrium size of an inlet with jetties is about the same as that of the same inlet without jetties. The jetties serve to stabilize the channel but not to enlarge the inlet cross-sectional area. If a cross-sectional area larger than the equilibrium area is desired, then a nonscouring inlet is the appropriate development.

Determine whether artificial bypassing is needed. If there exists an approximate balance between the littoral drift in one direction and that in the opposite direction so that the resultant or net drift is quite small, it may be that the natural bypassing by the waves and currents will be adequate. However, such a condition is exceptional and some form of artificial bypassing is usually necessary.

Escoffier, Francis F.

Hydraulics and stability of tidal inlets / by Francis F. Escoffier. – Fort Belvoir, Va. : U.S. Coastal Engineering Research Center, 1977.
72 p. : ill. (GITI report 13) Also (Contract - U.S. Coastal Engineering Research Center ; DACW72-74-C-0005)
Bibliography: p. 66.
Report summarizes several important basic developments pertaining to analysis of the hydraulics and related stability of tidal inlets. The original inlet stability concept proposed by Escoffier is extended in light of recent work. Tidal inlet characteristics and functional design requirements as well as case studies of selected inlets on the U.S. coasts are briefly discussed.
1. Tidal inlets. 2. Tidal hydraulics. 3. Stability. I. Title. III. Series: U.S. Coastal Engineering Research Center. Contract DACW72-74-C-0005.

Escoffier, Francis F.

Hydraulics and stability of tidal inlets / by Francis F. Escoffier. – Fort Belvoir, Va. : U.S. Coastal Engineering Research Center, 1977.
72 p. : ill. (GITI report 13) Also (Contract - U.S. Coastal Engineering Research Center ; DACW72-74-C-0005)
Bibliography: p. 66.
Report summarizes several important basic developments pertaining to analysis of the hydraulics and related stability of tidal inlets. The original inlet stability concept proposed by Escoffier is extended in light of recent work. Tidal inlet characteristics and functional design requirements as well as case studies of selected inlets on the U.S. coasts are briefly discussed.
1. Tidal inlets. 2. Tidal hydraulics. 3. Stability. I. Title. III. Series: U.S. Coastal Engineering Research Center. Contract DACW72-74-C-0005.

GB454 .I5 .U581r no. 13 551.4

Escoffier, Francis F.

Hydraulics and stability of tidal inlets / by Francis F. Escoffier. – Fort Belvoir, Va. : U.S. Coastal Engineering Research Center, 1977.
72 p. : ill. (GITI report 13) Also (Contract - U.S. Coastal Engineering Research Center ; DACW72-74-C-0005)
Bibliography: p. 66.
Report summarizes several important basic developments pertaining to analysis of the hydraulics and related stability of tidal inlets. The original inlet stability concept proposed by Escoffier is extended in light of recent work. Tidal inlet characteristics and functional design requirements as well as case studies of selected inlets on the U.S. coasts are briefly discussed.
1. Tidal inlets. 2. Tidal hydraulics. 3. Stability. I. Title. III. Series: U.S. Coastal Engineering Research Center. Contract DACW72-74-C-0005.

GB454 .I5 .U581r no. 13 551.4

Escoffier, Francis F.

Hydraulics and stability of tidal inlets / by Francis F. Escoffier. – Fort Belvoir, Va. : U.S. Coastal Engineering Research Center, 1977.
72 p. : ill. (GITI report 13) Also (Contract - U.S. Coastal Engineering Research Center ; DACW72-74-C-0005)
Bibliography: p. 66.
Report summarizes several important basic developments pertaining to analysis of the hydraulics and related stability of tidal inlets. The original inlet stability concept proposed by Escoffier is extended in light of recent work. Tidal inlet characteristics and functional design requirements as well as case studies of selected inlets on the U.S. coasts are briefly discussed.
1. Tidal inlets. 2. Tidal hydraulics. 3. Stability. I. Title. III. Series: U.S. Coastal Engineering Research Center. Contract DACW72-74-C-0005.

GB454 .I5 .U581r no. 13 551.4