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# Symposium 3.—Sedimentation in Estuaries, Harbors, and Coastal Areas

## INTRODUCTION

Most papers presented in this and the 1947 sedimentation conference were concerned primarily with the erosion, transportation, and deposition of sediment in streams and reservoirs. Of equal importance, particularly in coastal areas, is the behavior of stream-borne sediments upon entering the ocean. Such sediments may enter directly on an open coast or they may enter into an estuary. The practical problems involved in these two processes differ greatly and are discussed briefly as follows.

A stream discharging directly on to an open coast will drop its sediment load upon entering the relatively quiet water of the ocean. The bed materials load will be deposited in the form of a delta, whereas the wash load will be carried offshore into deep water either as a density current or in suspension by the rather feeble coastal currents. Whether or not the delta composed of the coarser sediment fractions exists as a pronounced bulge on the coastline depends on the size of the sediment, the range of tide, but principally on the intensity of wave attack. If the wave action is relatively light and the material is relatively coarse, then the delta may be of a rather permanent form. On the other hand, if the intensity of wave attack is high and the material fine, then the stream-borne sediments are resorted by the high turbulence of the breaking waves and moved along the coast by the littoral currents, and no evidence of a delta in the usual sense exists.

In many localities along the coastlines of the world, streams often enter a bay or estuary before the river waters reach the ocean. The amount of sediments transported by such streams that may reach the ocean depends on a number of factors, such as stream discharge, size of the bay or estuary, and character of the sediments. In case of large bays or estuaries and moderately sized streams, almost all the bed material load will be deposited in the bay and only some of the washload is eventually transported to the ocean by streamflow and tidal currents. A great percentage of the washload actually may be deposited in the bay or estuary as a result of flocculation, which occurs when the sediment-laden fresh water meets the saline bay waters.

Although streams appear to be the major source of sediments supplied to our coastlines, such factors as cliff erosion, slides, onshore movement by wave action, and wind action may be of importance in many regions. In addition to the supply of sand, the problem of the loss of sand may also be important. Such losses may result from the movement of sediment offshore into deep water, losses into submarine canyons, accretion against littoral barriers, removal of sand for construction purposes, wind action, and abrasion by wave action. While all of these factors are not discussed in this morning's session, it is gratifying to see the amount of time that has been alloted to this subject.

# IMPORTANCE OF MARINE INFLUENCES IN ESTUARINE SEDIMENTATION

[Paper No. 64]

By Robert E. Burns, *research oceanographer, Office of Research and Development, U.S. Coast and Geodetic Survey* [1]

The important early settlements in this country were in the estuaries of major rivers. Consequently, it is not surprising that many investigations of estuarine sedimentation have been made in these same estuaries. Under these conditions, it is important to realize that most contemporary concepts of estuarine sedimentation have been strongly influenced by the pres-

[1] Field work and analysis of sediment in Port San Juan, British Columbia, was conducted at the University of Washington under Office of Naval Research Contract Nonr-477(10), Project NR 083 012.

ence of an overwhelming dominant river load. Although initially there was a tendency to consider the estuary merely as a continuation of the river environment, it was recognized early that marine processes must also be considered. Since the estuary is an environment in which the marine forces are at least in equilibrium with the river forces (as contrasted with, for example, the deltaic environment), the importance of these marine forces should be considered at some length.

Although the role of tides and tidal currents has been recognized for some time, only in recent years has the importance of salinity and the saline-wedge intrusions been examined in detail. The role of the salinity distribution is now the subject of many investigations, and in general terms its effect on both the net-circulation and on flocculation of suspended clay mineral load is reasonably well understood.

Unfortunately, because so much of the research has been conducted in estuaries of major rivers, examination of the behavior of river-introduced load has tended to overshadow examination of other sources of sediment and of the nonriver processes. The saline wedge is important in any estuary, but its impact upon sedimentation becomes relatively less important as the magnitude of the suspended clay mineral load of the river decreases.

In many nonmajor river estuaries, the problem of evaluation of sedimentation is extremely complex because of the secondary importance of the river-related source of sediment and the absence of a major river flow. In these areas, an understanding of the various sources of sediment other than a major river and an evaluation of the importance of the processes of nonriver origin are necessary to understand the overall sedimentation regime.

One of the simplest possible models for sedimentation is that in which the characteristics of the sediment are determined by the interaction between those processes tending to remove sediment and those processes tending to add sediment. If the sediment is characterized as a function of the mineralogy, diameter, shape, and lattice of the component grains, this interaction can be expressed in the form:

$$S_o = (S_i + S_s) - S_r$$

Where $S$ = A function (mineralogy, shape, diameter, lattice);
$_o$ = Observed at time of survey;
$_i$ = Initial condition (presurvey);
$_s$ = Added from source;
$_r$ = Removed from area.

Despite its simplicity, this relation is the basis for more elaborate models, and in itself is of value in determining changes in sediment over periods of time. Although the model is rarely expressed in these specific terms, it has been used by many investigators of estuarine sedimentation. Since adequate evaluation of even this basic model is generally beyond the capabilities of the investigator, attempts to examine natural situations usually involve one of two basic approaches.

(1) The more common approach assumes a steady state in which the observed sediment type is unchanged through time, i.e., $S_o = S_i$ and $S_r$ must equal $S_s$. The sediment found within the estuary is assumed to be in equilibrium with environmental processes, and an attempt is made to determine the physical processes controlling this equilibrium.

Geologists examining lithified samples from the geologic column are invariably limited to these steady-state assumptions by the nature of their data. Further, many studies of contemporary conditions follow this line of reasoning because of the extreme difficulties in conducting more adequate sampling programs.

(2) The second general use of the basic model involves a system in which $S_o \neq S_i$. Evaluation of this assumption involves at least two surveys of the area and an attempt to determine representative values of both $S_s$ and $S_r$ for the period between the surveys. A comparison of charts based on past surveys with more recent data is frequently used to evaluate $S_s$ and $S_r$.

However, in this simple form the model gives no insight into the various processes acting within the area of investigation nor is it useful in determining the particular processes responsible for additions and removals of sediment. A further development of the model is necessary in order to consider the behavior of sediment within the estuary and to evaluate the various processes responsible for sediment distribution.

One of the simpler models of estuarine sedimentation that can be developed is presented in figure 1. This indicates that all sediment introduced into an estuary, from whatever source, has one of five possible behaviors:

(1) The sediment can move through and out of the estuary without ever being deposited (a process of bypassing).

(2) The sediment can move through and out of the estuary in a series of steps involving alternate deposition and erosion.

(3) The sediment can be permanently deposited after initial deposition.

(4) The sediment can be subjected to alternating deposition and erosion through one or more cycles and can then be permanently deposited.

(5) The sediment can be recycled continually without ever being moved out of the estuary or being permanently deposited.

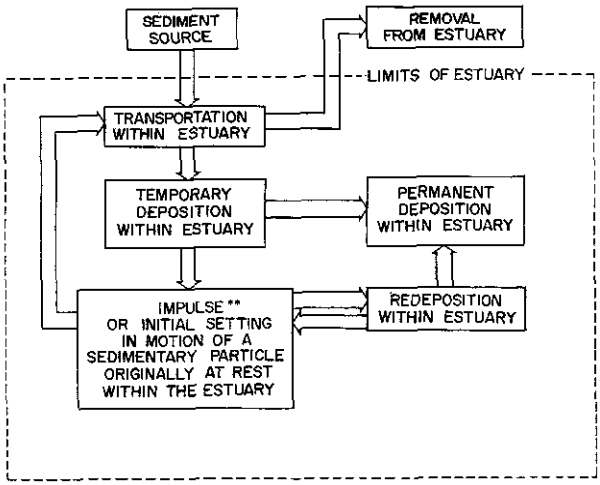


FIGURE 1.—Block model for evaluating sedimentation in an estuary.

Each of these five behaviors may vary in importance through time, and all of them may be operating simultaneously but on different sedimentary particles. The observed bottom sediment distribution is, then, the result of the summation through time of all the processes acting in the area.

The application of this model to a natural situation involves the evaluation of the various sources of sediment and the processes of transportation and erosion within the area. In particular, we must consider the processes that may be capable of giving impulse to particles which for one reason or another are at rest on the bottom.

Figure 2 is a generalized presentation of the various sources from which sediment may be introduced into an estuary.

The surface source is responsible for the

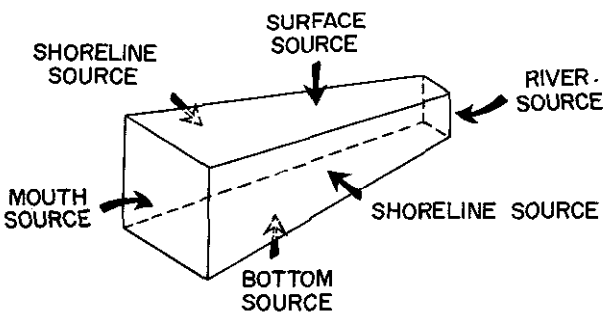


FIGURE 2.—Generalized sources of sediment entering an estuary.

[2] WANG, F. H. RECENT SEDIMENTS IN PUGET SOUND AND PORTIONS OF WASHINGTON SOUND AND LAKE WASHINGTON. 160 pp. 1955. Ph.D. thesis. On file at the library, University of Washington.

introduction of such material as volcanic ash, rafted sediment, windblown particles, etc. Although some specific incidents of importance have been reported (*11*), such sources are usually minor and are not generally considered in the evaluation of most natural situations.

The bottom of the estuary may be a source of sediment introduced into an estuary by processes related to turbidity currents. Although such processes are sometimes important in redistributing sediment already within the estuary, it is unlikely that the bottom is normally an important source of new sediment in estuaries.

Without a river source the estuary cannot exist, but the importance of the sediment added from this source both varies greatly with time in any single estuary and certainly from one estuary to another. In the estuaries of large rivers the river is usually a dominant source of sediment, and several investigators (*2*, *4*) have emphasized processes that distribute the river-introduced sediments. When the river is a dominant source — specifically when the river is a large one with an extensive drainage basin — the problem of flocculation of silt- and clay-sized particles is of major importance. However, in areas of limited drainage basins and relatively little weathering to form clay minerals, the problem of flocculation becomes considerably less important.

An additional factor affecting the sedimentary-laying importance of the river source is the general circulation related to the intrusion of the saline water along the bottom. This has been covered in detail from the hydrodynamic viewpoint by various investigators (*9*). The effectiveness of this circulation in trapping river-introduced sediment was discussed by Schultz and Simmons (*12*). When the river flow is diminished, as in the case of many of the nonmajor river estuaries, downstream movement is confined to the upper layers or may be completely overbalanced by the tidal currents. In these cases, all but the very finest sediment is trapped within the limits of the estuary by some initial deposition. In keeping with the model (fig. 1), further resuspension and transport must then be related to processes that are not directly attributable to river action.

The shoreline source exists in the nearshore water within the estuary, and it contributes sediment from along the beaches and shoreline, including that produced by slumping of the shore. The importance of sediment from shoreline sources has been minimized in many of our investigations of major river estuaries, but it is considered of primary importance in the north European estuaries (*3*) and in the Puget Sound area.[2]

The final, and most frequently overlooked, source of sediment is the seaward end of the estuary. Sediment brought to the mouth of the estuary by longshore movement and coastal currents can be carried into the area by tidal currents, wave-induced movement, the salt-replacing bottom current, and other elements of the general circulation. In some areas the mouth source of sediment is the dominant one and overshadows all others. In the major east coast estuaries it has been observed that the sand content increases from the head to the mouth, and this may be the result of introduction of sediment from a mouth source.

The importance of the mouth source is difficult to evaluate in any except special cases. Ippen[3] suggests that the salt-replacing bottom current may be of major importance in moving sands headward in some of the east coast major estuaries. In any area with abundant longshore and coastal current along the open coast outside the estuary, the importance of the mouth source must be considered.

The phenomena contributing to the transport of sediment that give some insight into the possible magnitudes of fluvial and marine processes are:

(1) River-associated flow;
(2) Tidal influence —
  (a) Vertical movement (tides), and
  (b) Horizontal movement (tidal currents);
(3) Wave influence —
  (a) Longshore and littoral transport or littoral transport,
  (b) Bottom movement under shoaling waves, and
  (c) Suspended movement under shoaling waves;
(4) General circulation —
  (a) Related to salt budget,
  (b) Related to tides,
  (c) Related to waves, and
  (d) Related to winds.

Seaward movement of river-introduced sediment by river current alone is of limited extent in any estuary. Without those aspects of turbulence generally associated with processes other than river flow, even the fine sediment that normally moves seaward in the surface layers would not be maintained in suspension long enough to bypass the estuary in any quantity. Bedload is characteristically deposited near the point of entry of the river into the estuary or near the point of maximum intrusion of saline bottom water.

The influence of tidal action is important both as a direct cause of sediment movement and in many indirect ways. The periodic diminishing and reversal of tidal currents allow much of the sediment in suspension to be deposited. If subsequent maximum velocities are too small to give impulse to these particles, their transport directly as a result of tidal currents is ended. In addition, the geometry of many of the nonmajor river estuaries is such that the tidal currents are generally oscillatory and result in little net movement. Those net movements that have been observed appear to be more closely related to the indirect effect of the tidal current on the general circulation (*1*).

Most other effects of tidal action are also indirect. The rise and fall of the water has no direct effect except that due to the periodic covering and uncovering of the nearshore bottom. The effect of the tidal action as a mechanism for mixing fresh and saline water exerts a control on the general circulation of the estuary.

One of the major marine influences that controls sedimentation in the estuary (and, specifically, in the nearshore areas) is the effect of wave action. The process of longshore transport is generally related either to tidal currents or to net movements under the influence of wave action. In contrast to the larger scale oscillatory movement associated with tidal currents, there will generally be, at any point that is under wave influence, a more pronounced unidirectional movement of the bottom sediment associated with wave action.

The direction of this transport associated with wave action is related to the direction of dominant wave propagation. Actual sediment movement may be generally with, or counter to, the direction of wave movement, depending upon factors associated both with wave and sediment particle dimensions and the depth of water.[4]

All the factors mentioned above are individually important, but they also contribute to the general circulation in the estuary. One element of the general circulation is the headward movement of bottom water resulting from continuity requirements imposed by the removal of salt (*7*, *10*). The interaction of the tidal currents with the bottom results in a net mass transport as described by Abbott (*1*). Other mass transports that result from the net velocity profiles associated with shoaling waves (*8*) and from motion due to the effect of wind stress (*5*) contribute to the general circulation.

The general circulation, as defined here, is a low velocity net transport that should be of little direct importance in the transportation of

[3] IPPEN, A. T. SEDIMENTATION IN ESTUARIES. Mass. Inst. Technol. Class notes for course in estuarine environment. 1960. (Mimeo.)

[4] VINCENT, G. CONTRIBUTION TO THE STUDY OF SEDIMENT TRANSPORT ON A HORIZONTAL BED DUE TO WAVE ACTION. Sixth Conf. on Coastal Engin., Miami, Proc., pp. 326-355. 1958. Council on Wave Research, Univ. of Calif. (Berkeley).

sediment. However, combined with processes capable of giving impulse to, of resuspending or of maintaining sediment in suspension, the general circulation becomes a major factor in controlling the distribution and sorting of sediment within the estuary. Although an evaluation of the general circulation in any estuary is an almost impossible undertaking (*6*), the interaction of river flow, tidal currents, wave-related movement, and wind-induced current appears to be the basic control.

A brief discussion of some of the features of Port San Juan, British Columbia, may be used as an example of the importance of the mouth source and the marine processes. Although this is a small estuary of the Gordon and San Juan Rivers, fresh-water discharge appears to be adequate to allow development of a two-layered system. An evaluation of the possible sources of sediment has been conducted and indicated that the shoreline, bottom, and surface sources may be considered negligible.[5] The introduction of a characteristically micaceous sediment by the rivers is indicated by a very localized occurrence associated with the extreme head of the estuary. On the other hand, the mouth source is considered of primary importance, since the net longshore movement of sediment along the coast outside Port San Juan provided material of the same general characteristics as that collected from the bottom over most of the area. This sediment is mineralogically distinct from the river-related micaceous sediment mentioned above.

In Port San Juan, it is impossible to separate the processes of initial transportation and deposition from processes of reworking. The unimodal, well-sorted sediment and the lack of indications of permanent deposition imply that a reworking after initial deposition is an important and continuing process.

The major contribution of the river to the transport of sediment appears to be in relation to the role of fresh-saline water mixing in influencing the general circulation. This general circulation appears inadequate to give impulse to any sediment particles on the bottom, but should act as an initial trap for any sediment introduced from the river source.

The effect of longshore movement and sediment sorting under wave action appears to be of primary importance in Port San Juan. Measurements conducted by the National Research Council of Canada indicate that a swell having an 8- to 10-second period enters the mouth of the estuary throughout most of the year.[6] Waves of this magnitude are capable of giving impulse to sedimentary particles over the whole of the estuary bottom because of its relatively shallow depth (18 meters and less).

The estuary acts as an effective trap for much of the river-introduced sediment, but the absence of the finer sizes raises the possibility of bypassing of the fine sediment as the result of interaction between the wave net-velocity profile and the sediment-suspension gradients of the finer particles.

With the addition of sediment from both the mouth and the river, there remains the problem of satisfying the continuity requirements. Because the waves are capable of giving impulse to bottomed particles, the mode of movement induced by wave action is a function of the size of the wave and the normal daily and seasonal variations in wave dimensions will impose varying effects. The overall bed movement will always be in the direction of the dominant wave propagation, headward in this estuary. The suspended load will be affected differently as the wave dimension modifies with passage over a shoaling bottom.

Under maximum wave conditions there may even be a net removal of sediment from the area. It appears more probable, however, that the amount of bedload entering from the mouth is compensated by a continuing removal of the finer sediment of stream origin and a periodic removal of some of the sand sizes from the bottom. If it can be assumed that the amount of river-introduced sediment is a function of the river discharge, the rate of addition of sediment by streams is greater for the winter months during the very time the larger waves are generally steeper but less persistent. During the summer, the amount of stream-introduced sediment is sharply decreased at the same time that the waves entering the area are most persistent. The winter waves appear to be capable of removing greater peak loads and coarser sediment during the times when they are effective in the area; the summer waves, because of their greater persistence, probably remove a greater overall volume of sediment, but without the coarse extremes.

Additional field testing of the general model of sedimentation in some nonmajor river estuaries has verified the importance of the nonriver sources of sediment. Further, there is good evidence that most sediment introduced into estuaries, from whatever source, is subject to an initial deposition within the limits of the area. An understanding of the processes capable of providing impulse to these bottomed particles is

[5] BURNS, R. E. A MODEL OF SEDIMENTATION IN SMALL, SILL-LESS, EMBAYED ESTUARIES OF THE PACIFIC NORTHWEST. 117 pp. 1962. Ph.D. thesis. On file at the library, Univ. of Wash.

[6] NATIONAL RESEARCH COUNCIL OF CANADA. PRELIMINARY (WAVE) STUDIES. 4 pp. and 4 figs. 1961. Division of Mechanical Engineering. Unpublished manuscript on Port San Juan.

necessary in order to evaluate the eventual distribution of sediment. In the nonmajor river estuaries these processes are rarely related directly to river flow, but appear to be predominantly of marine origin.

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# HOPPER DREDGE DISPOSAL TECHNIQUES AND RELATED DEVELOPMENTS IN DESIGN AND OPERATION

[Paper No. 65]

By LOUIS J. MAURIELLO, *chief, Marine Division, U.S. Army Engineer Division, North Atlantic, New York, N.Y.*, and LEWIS CACCESE, *chief, Operations Division, U.S. Army Engineer District, Philadelphia, Pa.*

## Synopsis

A considerable amount of dredging by the Corps of Engineers to improve and maintain navigation channels in various regions of the United States is accomplished by seagoing hopper dredges. Conventional methods of operation are described. Various disposal techniques employed, particularly recent developments, and related modifications in design and operation are discussed.

## Introduction

A significant amount of dredging performed by or under the jurisdiction of the Corps of Engineers, U.S. Army, requires the use of seagoing hopper dredges. The Corps presently owns and operates in the various coastal and Great Lakes regions of the United States 15 dredges of this type with hopper capacities ranging from 500 cubic yards to over 8,000 cubic yards. Physical and other descriptive data of this hopper dredge fleet is shown in table 1. The annual total cost of operating the dredges amounts to nearly $19,000,000, with costs of individual dredges ranging from approximately $700,000 to $2,400,000 a year. In recent years, the total volume of material removed annually by these dredges is approximately 65 million cubic yards, as measured in place within project limits.

Basically, the seagoing hopper dredge is a self-propelled vessel resembling the modern ocean tanker in appearance except for a larger amount of deck equipment (figs. 1 and 2). In lieu of the cargo tanks of the ocean tanker, the hopper dredge is provided with hoppers or bins used to load and carry material dredged hydraulically from the bottom. It is a completely self-contained dredging plant equipped with all necessary dredging equipment (i.e., centrifugal pumps, dragarms or trailing suction assemblages, etc.). The significant characteristic of the hopper dredge is that it operates while underway and requires no anchors or other mooring devices. Modern hopper dredges reflect about a century of development. They usually work in channels or harbors in which wave action or heavy traffic makes a stationary dredging plant undesirable or intolerable; also, in many other waterways the location of the shoal and the disposal area are so far removed from each other that the mobility of the hopper

TABLE 1.—*Specifications of Corps of Engineers hopper dredges*

| Name | Year built | Length, beam, and depth | Maximum hopper capacity | Maximum draft loaded | Propulsion, All twin screw | | Dredge pumps | | | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Horsepower | Type [1] | Number | Size | Horsepower | |
| | | *Feet* | *Cu. yd.* | *Ft. In.* | | | | *Inches* | | |
| Essayons... | 1950 | 525x72x40 | 8,270 | 31-0 | 8,000 | TE | 2 | 32 | 1,850 | |
| Goethals... | 1938 | 476x68x36 | 6,422 | 29-0 | 4,500 | TE | 2 | 30 | 1,300 | Equipped for sump rehandling. |
| Biddle..... | 1947 | 352x60x30 | 3,060 | 24-3¾ | 6,000 | TE | 2 | 28 | 1,150 | |
| Comber.... | 1947 | 352x60x30 | 3,422 | 24-3¾ | 6,000 | TE | 2 | 28 | 1,150 | Equipped for direct pump-out. |
| Gerig..... | 1947 | 352x60x30 | 3,060 | 24-3¾ | 6,000 | TE | 2 | 28 | 1,150 | |
| Langfitt.... | 1947 | 352x60x30 | 3,060 | 24-3¾ | 6,000 | TE | 2 | 28 | 1,150 | |
| Harding ... | 1939 | 308x56x30 | 2,682 | 20-3 | 4,240 | D | 2 | 20 | 650 | |
| Markham.. | 1960 | 339x62x28 | 2,681 | 20-0 | 5,300 | DE | 2 | 23 | [2] 650 | Equipped for direct pump-out. |
| Mackenzie . | 1924 | 268x46x22 | 1,656 | 21-0 | 2,400 | DE | 1 | 26 | 900 | |
| Hains..... | 1942 | 216x40x15 | 885 | 13-0 | 1,400 | DE | 1 | 20 | 410 | Equipped for direct pump-out and side-casting. |
| Hoffman... | 1942 | 216x40x15 | 920 | 13-0 | 1,400 | DE | 1 | 20 | 410 | |
| Hyde..... | 1945 | 216x40x15 | 720 | 13-0 | 1,400 | DE | 1 | 20 | 410 | |
| Lyman.... | 1945 | 216x40x15 | 920 | 13-0 | 1,400 | DE | 1 | 20 | 410 | |
| Davison ... | 1945 | 216x40x15 | 720 | 13-0 | 1,400 | DE | 1 | 20 | 410 | |
| Pacific..... | 1937 | 180x38x14 | 500 | 11-3 | 1,200 | D | 1 | 18 | 340 | |

[1] TE = Turbo-electric; DE = Diesel-electric; D = Diesel direct drive.
[2] Also equipped for operation at 1,000 hp.

FIGURE 1.—Hopper dredge *Essayons*.

dredge makes it by far the most efficient dredging plant.

As in any dredging operation, the location of the dredging site, the character of the bottom material, the regimen of the waterway, and the availability of suitable disposal areas are important factors that affect the economics of hopper dredging and dictate the methods and techniques employed. The principal intent of this paper is to discuss the influence of such factors on hopper dredge disposal techniques and design.

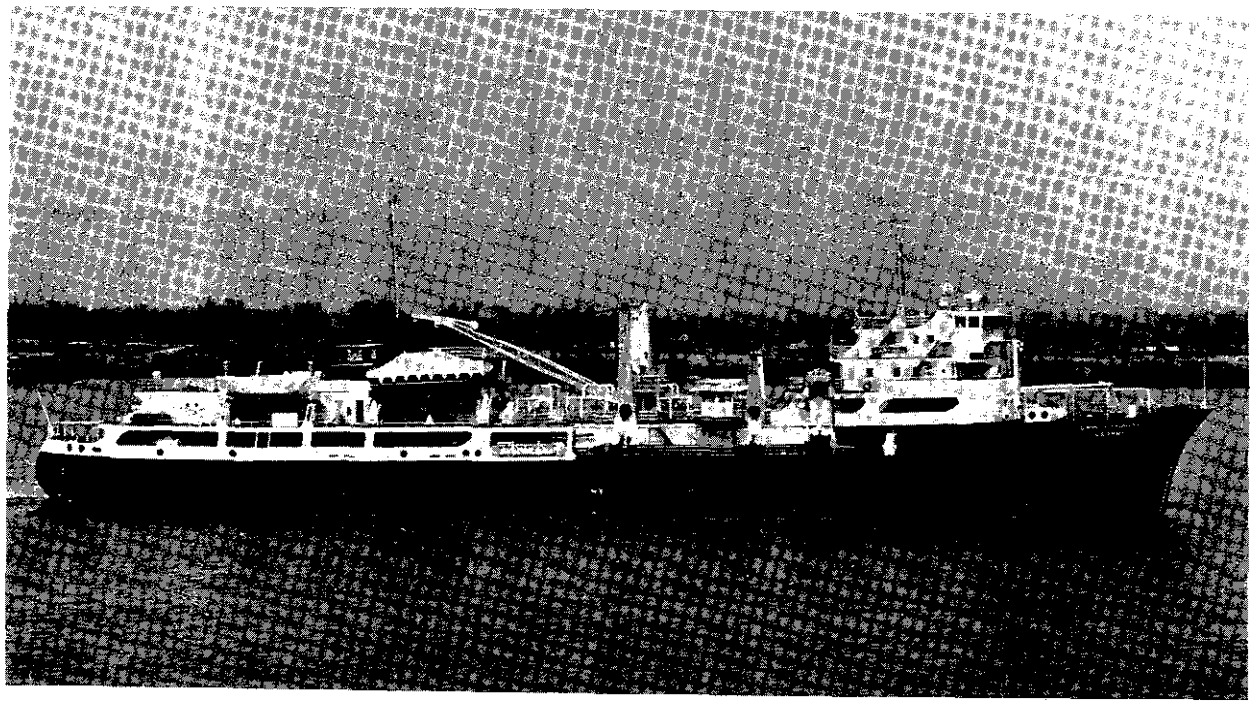

FIGURE 2. — Hopper dredge *Markham*.

## How Hopper Dredges Usually Work

Although this paper will deal particularly with the disposal phase of hopper dredge operations, a brief discussion of how hopper dredges normally work and certain elements involved in their operation is appropriate.

Hopper dredges are designed primarily to dredge materials hydraulically, to load or retain the solids dredged in their hoppers, and then to haul the spoil to a disposal area or dump, where the material is disposed of by dumping through doors in the bottom of the hoppers (fig. 3). Loading is accomplished by sucking the bottom material through the drags into the hoppers while making one or more cuts (or passes) through the dredging area. The quantity or volume pumped during the loading operation depends primarily upon the character of the material and the amount of pumping time involved as well as the hopper capacity and the pumping and propulsive capability of the dredge being used. The length of haul also affects the amount of time that may be spent pumping. During the dredging (or pumping) process the bottom material is pumped in a diluted state and the hoppers are equipped with overflows to allow excess water to be discharged overboard. In the case of such materials as very heavy coarse sand and gravel that readily settle in the hoppers, most, if not all, of the solids are retained in the hoppers and only minor quantities of solids are lost in the overflow. Conversely, in certain materials such as very light and fine silt, most of the solids pumped after the hoppers are filled with the dredged mixture may be overflowed if pumping is continued. Generally, the preponderance of materials encountered in dredging problems fall in between these extreme categories. For illustrative purposes, figure 4 shows theoretical hopper dredge loading curves for materials of various degrees of retainability between 0 and 100 percent, if a uniform solids pumping rate is assumed. The retainability factor obviously dictates to a large measure the time involved in loading and the hopper load obtained. In addition the length of haul and the time spent in making the round trip to and from the disposal area and dumping the load are important in determining when to terminate loading so as to attain the so-called "economic load" and "economic pumping time." The economic load and economic pumping time are determined by a series of economic load tests conducted periodically during normal dredging and hauling operations. During these test loads, measurements of the hopper load are made when it reaches overflow level and thereafter at least at every turn; the economic load is dredged when the retained cubic yards per minute of total cycle time (i.e. pumping, turning, running to and from dump, and dumping) equals the incremental retained

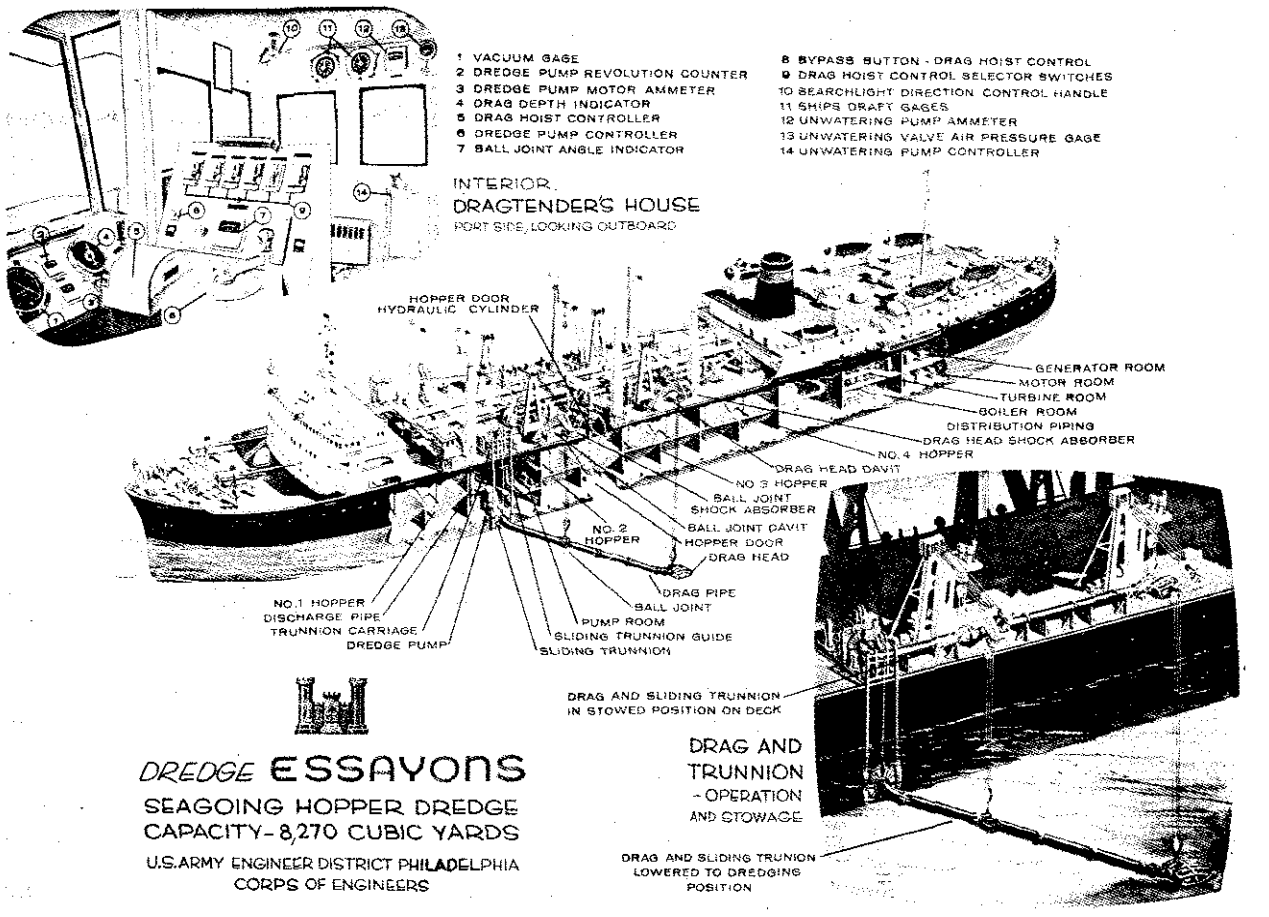


FIGURE 3.—Cutaway view of hopper dredge *Essayons*.

cubic yards per pumping minute or expressed in mathematical form when:

$$\frac{\Delta Y}{\Delta t} = \frac{Y}{T+t} \qquad \text{where}$$

$Y$ = Hopper load measured in cubic yards of material comparable in density to that in situ;

$\Delta Y$ = Increase in hopper load between measurements, in cubic yards;

$t$ = Pumping time, in minutes;

$\Delta t$ = Pumping time to obtain increase in hopper load ($\Delta Y$) between measurements, in minutes;

$T$ = Nonpumping time (i.e., turning, running to and from dump, and dumping), in minutes.

During initial stages of loading prior to reaching an economic load, $\frac{\Delta Y}{\Delta t}$ invariably exceeds $\frac{Y}{T+t}$ and this is indicative of underpumping. As loading progresses $\frac{Y}{T+t}$ normally increases and $\frac{\Delta Y}{\Delta t}$ decreases and approach each other in value. If pumping continues after an economic load is obtained, then $\frac{\Delta Y}{\Delta t}$ becomes less than $\frac{Y}{T+t}$, which indicates overpumping.

In practice the point of economic loading generally is not exactly defined for each and every load dredged, since it may well occur at a point other than exactly at the end of a dredging cut. Therefore, overpumping to a slight degree usually results. This is generally true except in cases where loading by pumping only to overflow is found to be economical or otherwise desirable. In such cases, since essentially nonretainable materials are involved, the hoppers are normally dewatered before commencement of loading.

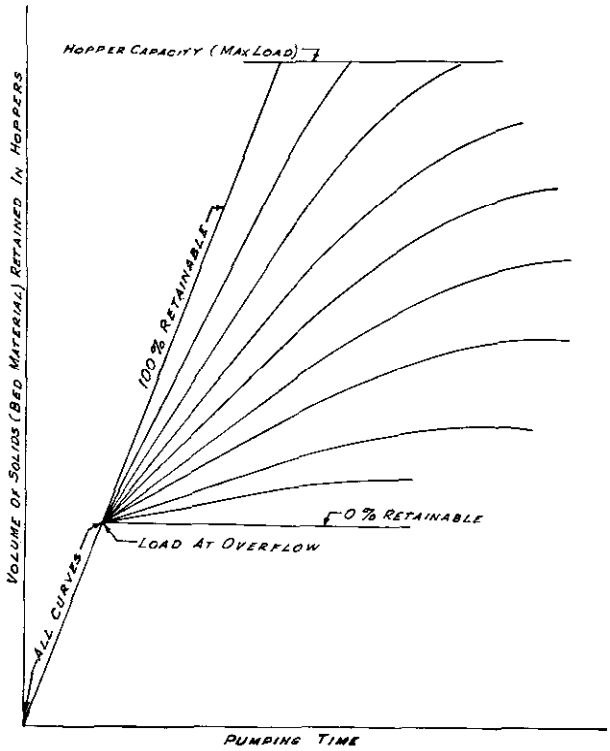


FIGURE 4. — Retention in hoppers of varying grain-sized materials.

Upon completion of loading, the dredge proceeds at full speed to a designated disposal area to dump the load. After disposal, the dredge returns to the dredging area to repeat the loading.

## Disposal by Conventional Dumping

As indicated above, hopper dredge spoil is normally disposed of by dumping through doors in the bottom of the hoppers. Emptying the hoppers by this operation takes about 3 to 15 minutes, depending principally on the volume and nature of the material in the hoppers. Generally, relatively fluid materials are dumped readily, whereas sticky clays and certain granular materials are more difficult to dump and washout and hopper jetting may be required. In this regard, it has been found that the dumping of heavy medium sand of essentially uniform grain size existing in various coastal regions may induce severe hull vibrations, unless the dumping sequence of individual hoppers is carefully controlled and large amounts of flushing and jetting water are used.

In most cases, hopper dredge spoil is presently being disposed of by dumping in designated areas in deep water at sea or outside navigation channel limits in open bays or rivers. Dump areas usually are selected with a view toward effecting permanent (within practical limits) disposal of the dredged material while maintaining the length of haul to a minimum. Application of these criteria results in compromises based on practical considerations of the particular job conditions and the economics involved. In general, the hauling distance for Corps of Engineers hopper dredging averages from 3 to 4 miles. The degree of harbor waterfront industrialization in certain waterways makes it necessary, however, to haul spoil considerably greater distances. For example, all spoil dredged by hopper dredge from principal channels in New York Harbor is dumped in a designated area of deep water in the Atlantic Ocean, involving a haul of more than 30 miles from certain locations.

Practically all developments leading to the use of hopper dredge disposal techniques differing from conventional dumping in deep water, have stemmed primarily from efforts to reduce the length of haul required for disposal.

## Dumping in Rehandling Basins

In operating hopper dredges in the Delaware River from Philadelphia to the sea, a haul of 50 miles or more would be required to effect disposal in deep water at sea. The necessity for precluding such long hauls was realized as early as 1880, when provisions were made for hopper dredges to dump in prepared basins from which a pipeline dredge redredged the material and discharged it ashore behind retaining dikes. The basin was in effect a previously dug hole in the riverbed into which materials were dumped by the hopper dredge. The rehandling dredge then pumped this material from the hole to the shore disposal area. This scheme, with improvements from time to time to reduce losses of spoil dumped by the hopper dredge before it could be picked up by the rehandling dredge, was used extensively in the Delaware River for over a half century. The rehandling basins were usually located in pairs so that one could be used by the hopper dredge to dump into while the other was being emptied by the rehandling pipeline dredge. A typical arrangement is illustrated in figure 5. Maneuvering the hopper dredge in extremely restricted basins requires considerable care and greater skill by navigating personnel than in conventional dumping in open deep water. Therefore, the time required for dumping in such operations usually was greater than in the latter case.

The bottom material comprising recurring shoals in the lower part of the Delaware River is a light silt, about 1,300 grams per liter in density, with approximately 90 percent by weight of the solid grains finer than 0.06 mm. in diameter. The retention of such material in rehandling basins presents a problem. In the Delaware River it was found that at least half the volume of material dumped by the hopper

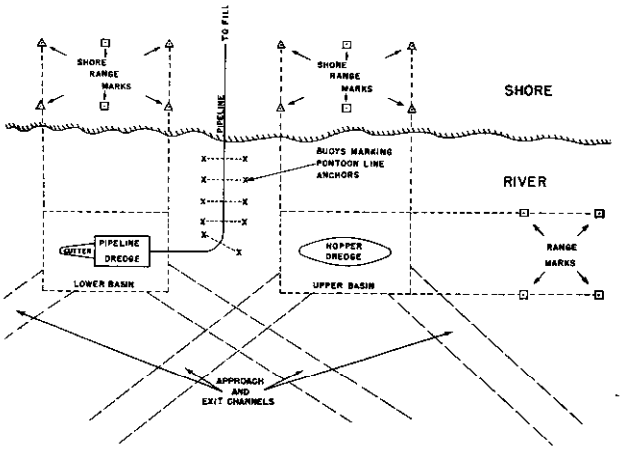


FIGURE 5. — Typical rehandling scheme.

dredge was carried away by tidal currents before it could be picked up by the pipeline dredge. As a result this scheme was discarded in 1954 by adoption of the sump rehandling technique, as described hereafter in this paper. It is considered, however, that the method of rehandling spoil by dumping into open water basins has merit when the material is heavy and consists of relatively coarse grains that readily settle after dumping and are not significantly affected by prevailing currents.

## Agitation Dredging

During the loading of hopper dredges, as described above, the overflow contains varying amounts of solids that are returned to the waterway. A significant part of this overflowed material, particularly when fine-grained and slow-settling, may be carried and distributed by currents over wide areas in the vicinity of the dredging site. In a strict sense, this constitutes "agitation dredging." Some degree of agitation exists in most types of dredging and it is fundamentally inherent but not intentional in most hopper dredge operations. However, by definition, agitation dredging by hopper dredge is a process that intentionally discharges overboard large quantities of dredged material with the objective that a major part will be transported and deposited outside channel limits by tidal, river, or littoral currents. This method of dredging with direct disposal or discharge in the immediate vicinity of the area being dredged may be advantageous when most of the material handled does not resettle in the channel and is not deposited where it can become a potential source of detrimental shoaling that may require future excavation. A fairly thorough knowledge of prevailing currents in the waterway and the composition and characteristics of the sediment to be dredged is essential to determine the feasibility of employing agitation dredging of any kind. Conditions may be favorable for agitation dredging when the shoal material consists of fine grains in the silt and clay categories with extremely slow settling rates, and dispersal of the dredging material over a large area is affected by prevailing currents. A chart developed and used by the Corps of Engineers to determine the largest grain size that may be expected to reach a deposit area of given depth at a given distance from the dredge is shown in figure 6. In any event, in specific cases, although favorable field conditions for this method of dredging may be indicated, the effectiveness of the operation also must be evaluated not only by the immediate and lasting improvements in the particular channel dredged but also with the full consideration of possible adverse effects in adjacent areas as well.

In agitation dredging, the hopper dredge normally continues to make repetitive cuts through the dredging area while pumping well beyond economic loading limits. The hauling and dumping phases of operation are ordinarily eliminated except for occasional trips to the dump to avoid consolidation of material in the hopper that may eventually result in undue difficulty in dumping. In some cases, to avoid consolidation, the dump doors may be kept cracked open and lower (emergency) overflow gates may be used for discharging the material overboard. Also, at certain locations where agitation dredging is not feasible during parts of the tidal current cycle (i.e. slack water, floodtide), dredging and hauling on an economic basis are employed intermittently.

Corps of Engineers' hopper dredges have been used with mixed success for agitation dredging in practically all regions of the United States since the early 1900's. In some cases, it has been found to be an expedient and economical means of keeping navigation channels open. There has been a decided decline in the use of this dredging technique by hopper dredges since 1955. The big advantage in agitation dredging is the ability to move large masses of material from shoal areas quickly and cheaply. It therefore cannot be overlooked as a dredging technique where it can be successfully employed. In some cases, particularly where frequent recurring dredging requirements are present regardless of the dredging methods and type of plant used, any disadvantages of this procedure can be tolerated with good judgment for the economic considerations involved. At present, agitation dredging is occasionally used in certain entrance channels along the Gulf coast, such as the Southwest Pass of the Mississippi River in Louisiana, and the Mississippi Sound channels leading into Gulfport and Pascagoula

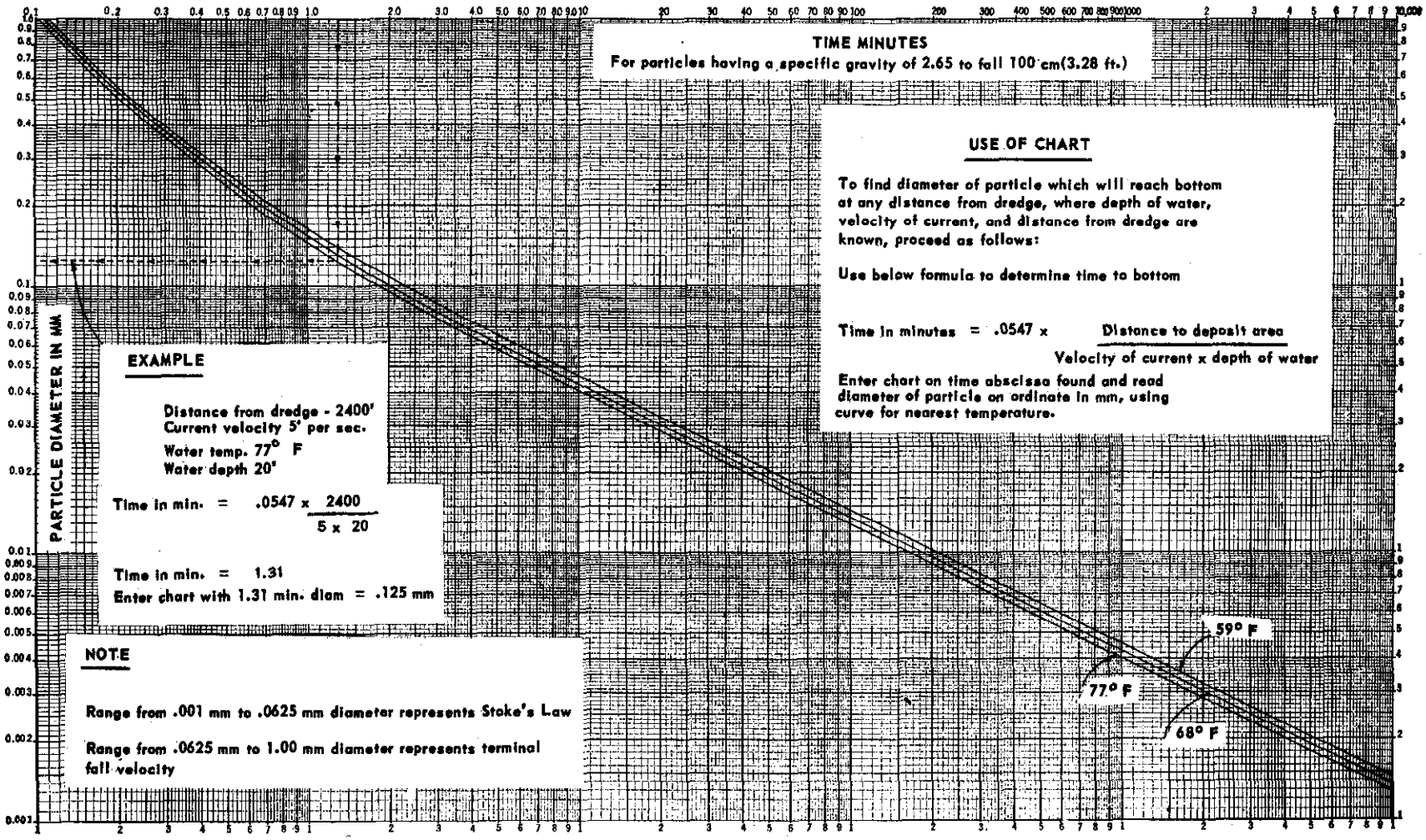


FIGURE 6. — Chart for determining largest grain size to reach deposit area.

Harbors in Mississippi. In these projects, the dredge *Langfitt* has agitated and removed a million or more cubic yards of sediment from within channel limits within a 1-week dredging assignment. The annual total volumes agitated by hopper dredges in the past 10 years has ranged from 1.3 to 11.3 million cubic yards.

## Sump Rehandling Operations

A review of the amount of dredging effort required to maintain navigation projects dredged by hopper dredges was made in 1950-52. This review revealed the need for developing new methods for the disposal of hopper dredge spoil. The Delaware River, below Philadelphia, was an example of a location where costly maintenance was resulting from material that was being lost in the dredging process. Engineering studies indicated that approximately 20 to 25 million cubic yards of material were being dredged each year in the maintenance of this channel, whereas only approximately one-third of this amount was entering the estuary as new material. After studying the problem, it was concluded that the difference between the total volume of material being dredged each year and the estimated sedimentation or natural shoaling represented the quantity of material that was escaping from open water disposal areas after the dredged materials had been dumped from hopper dredges.

Based on this conclusion, it was decided that a method of providing for the positive capture, retention and permanent disposal of dredged material was required to effect a significant reduction in the annual dredging workload. Experience had demostrated that agitation dredging, bottom dumping in open waters, or bottom dumping in rehandling basins for removal by pipeline dredges did not offer any degree of reasonable success in reducing the annual volume of maintenance dredging by hopper dredges. Various possible solutions were studied, and from this study there evolved the idea of unloading a hopper dredge through a self-contained and elevated discharge line into a rehandling barge or vessel, located at selected suitable mooring stations, from which the material could then be pumped ashore. Since the material to be dredged in the Delaware River was predominantly light silt susceptible to being transported extended distances by the river currents to possibly form new shoals, it was also decided as a general rule, that the hopper dredges would load by pumping only to the overflow level and then haul and dispose of the material into the rehandling vessel.

Conversion of a hopper dredge to unload into a rehandler included the installation of a collecting system in the hopper area to transport the dredged materials in the hoppers for delivery into the rehandler through an elevated pipe (referred to as a "snorkel") that is extended over the dredge side (fig. 7). This system, including the necessary valves, permits the discharge of material into the rehandler by a hopper dredge, using the same pumps utilized for dredging. For example, on the dredge *Goethals*, two 32-inch collecting mains with suction inlets into the hoppers extend longitudinally from the rear to the forward end of the hoppers. These lines are connected to the suction side of the two 32-inch dredge pumps. The material is then pumped into a single 42-inch pipeline at the main deck level and through the discharge snorkel into the rehandler. A sump rehandling vessel was provided by converting an obsolescent hopper dredge (the *New Orleans* built in 1912) and using its hopper area as the storage compartment. The propulsion and original dredging machinery was removed and a 6,000-hp. pump installed for discharging the material from its hoppers to shore disposal areas in the same manner as a pipeline dredge (figs. 8 and 9).

Since December 1954 the sump rehandling dredging system has been used with considerable success to maintain a major part of the 96-mile channel of the Delaware River between Philadelphia and the sea. It has also been used to improve and maintain the Norfolk Harbor channel. From December 1954 through June 1962 the following quantities have been handled utilizing this technique.

| | Cubic yards |
|---|---|
| Delaware River | 77,390,700 |
| Norfolk Harbor | 10,908,200 |
| Other areas | 2,782,000 |

It has been found that positive disposal and retention of material which has formed shoals can be obtained at reasonable costs. The material when placed behind shore disposal area dikes is not only removed as a potential source of future shoaling but also frequently enhances land values.

The measure of success of the sump rehandling system with its criteria for positive disposal of all dredged material is manifested by the Delaware River maintenance history. Prior to sump rehandling, as much as 25 million cubic yards of material were being handled annually by hopper dredges to maintain the Delaware River channel. This workload required as many as 22 hopper dredge months of operation per year, together with the full-time use of several pipeline dredges to rehandle material ashore. Presently, the lower Delaware River channel is being maintained by removing approximately 7½ million cubic yards annually, involving the use of about 10 hopper dredge months. Despite this reduced effort, a better channel exists. In brief, the development of the sump rehandling

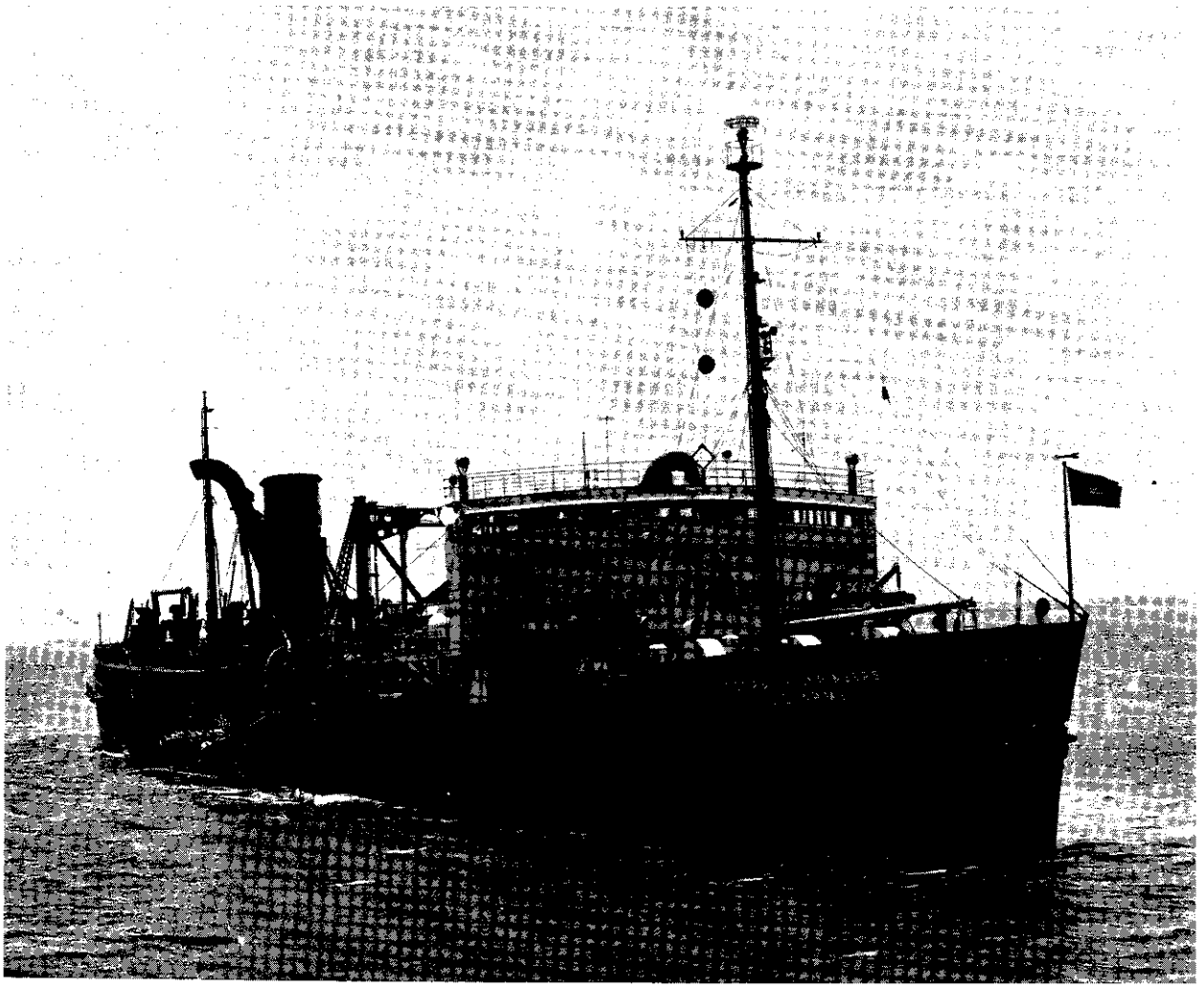

FIGURE 7. — Hopper dredge *Goethals*, showing 42-inch discharge pipe for sump rehandling.

system and the positive disposal of dredged material has reduced the required dredging effort to approximately one-half of its former level.

It was found that the sump rehandler was the answer where:

(1) The particle sizes of the sediment are predominantly in the silt category or finer. (In the Delaware River it is most common that less than 10 percent of the shoal material is in the sand category.)

(2) Enclosed disposal areas can be located at a distance of within 3 miles of the sump rehandler.

(3) A mooring for sump rehandler can be provided in 30 feet of water to permit the hopper dredge to maneuver and moor for unloading.

(4) It is determined that modern pipeline dredges cannot be used effectively because of any of the following conditions:

(a) Distance between the shoal and disposal area exceeds a reasonable discharge pipeline length;

(b) Water and weather conditions are too extreme or severe; or

(c) Floating pipeline is too obstructive to channel navigation.

The above criteria all existed in the lower Delaware River and Philadelphia Harbor and as a result the sump rehandler was successfully operated.

A significant factor in employment of the sump rehandler is that by the nature of the operation the maximum economy results when both converted hopper dredges — that is, the *Comber* and the *Goethals* — are used in conjunction with the sump rehandler *New Orleans*. This results in large production where as much

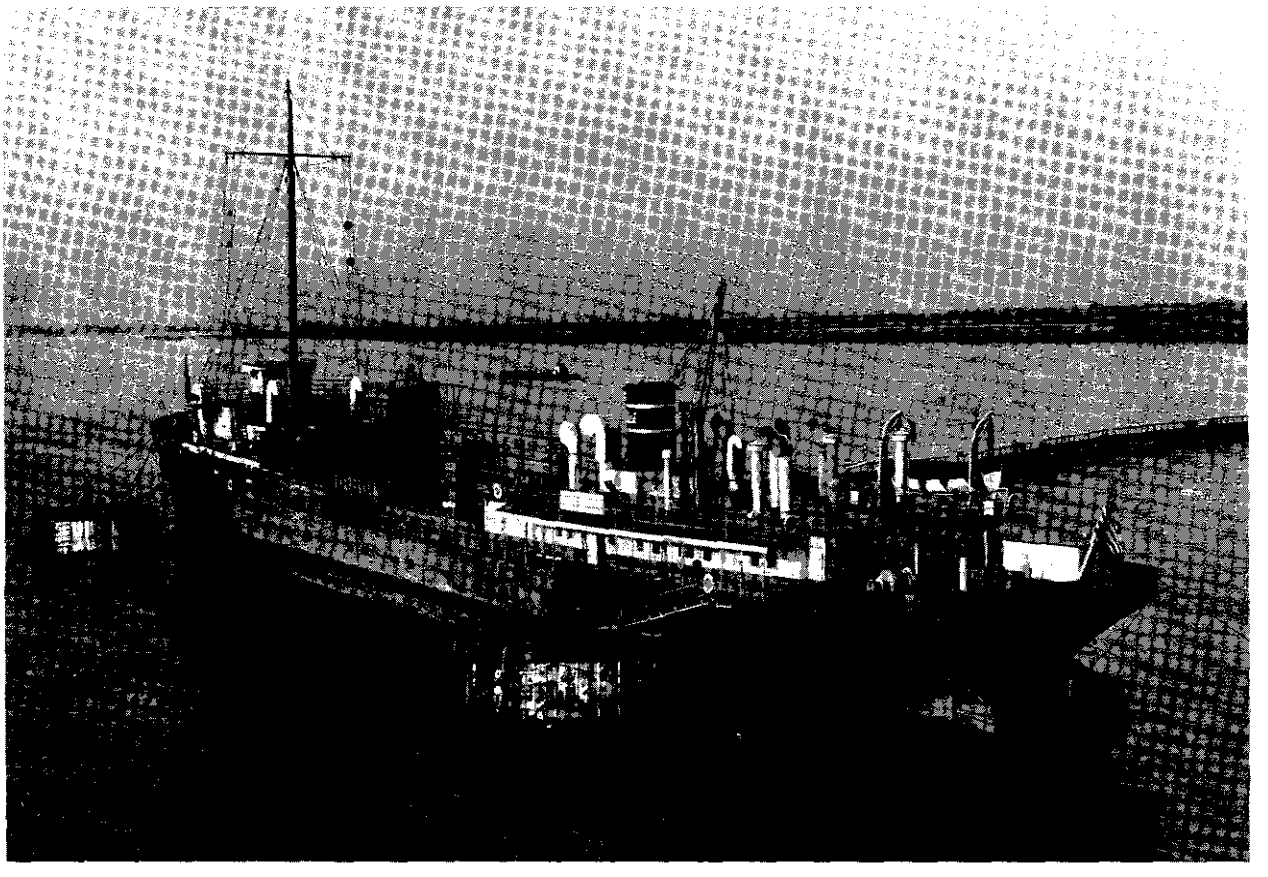

FIGURE 8. — Sump rehandler *New Orleans.*

as 2 million cubic yards of shoal material are placed ashore per month and positively retained behind banks.

The sump rehandling team of two hopper dredges and a sump rehandler dredges mass quantities of material efficiently and effectively; however, three major pieces of dredging plant are required. Not many projects either have the requirement for so much capacity or are able to justify the mobilization and operating cost of three major pieces of plant. This has been the major limitation that prevents the widespread application of this technique beyond the principal waterways of Philadelphia and Norfolk.

From the sump rehandling experience it was apparent that positive retention of dredged material is the best solution to many dredging requirements. However, the fact that the relatively high operating cost of the sump rehandler system precluded its use on projects with limited dredging requirements or workload necessitated further studies to provide additional improvements in disposing of hopper dredge spoil. As a result the design of the direct pumpout system for hopper dredges was developed to overcome this limitation. This direct pumpout system was devised to accomplish the same positive retention of dredged material as the sump rehandling system but with only one major item of plant. It is anticipated that direct pumpout capability will bring the technique of positive disposal of hopper dredge spoil within the financial capacity of many more projects.

## Direct Pump Ashore Operations

The sump rehandling system experience established that the positive disposal of all dredged spoil is an excellent solution in projects with dredging requirements such as the Delaware River. It was also evident that this dredging system was compatible with all campaigns for cleaner waters and most consistent with the Corps of Engineers responsibilities for assuring that solids are not indiscriminately introduced into the navigable waters of the United States. These observations, together with the concern for reducing the high operating cost of the sump rehandling operation, led to consideration of equipping the hopper dredge to con-

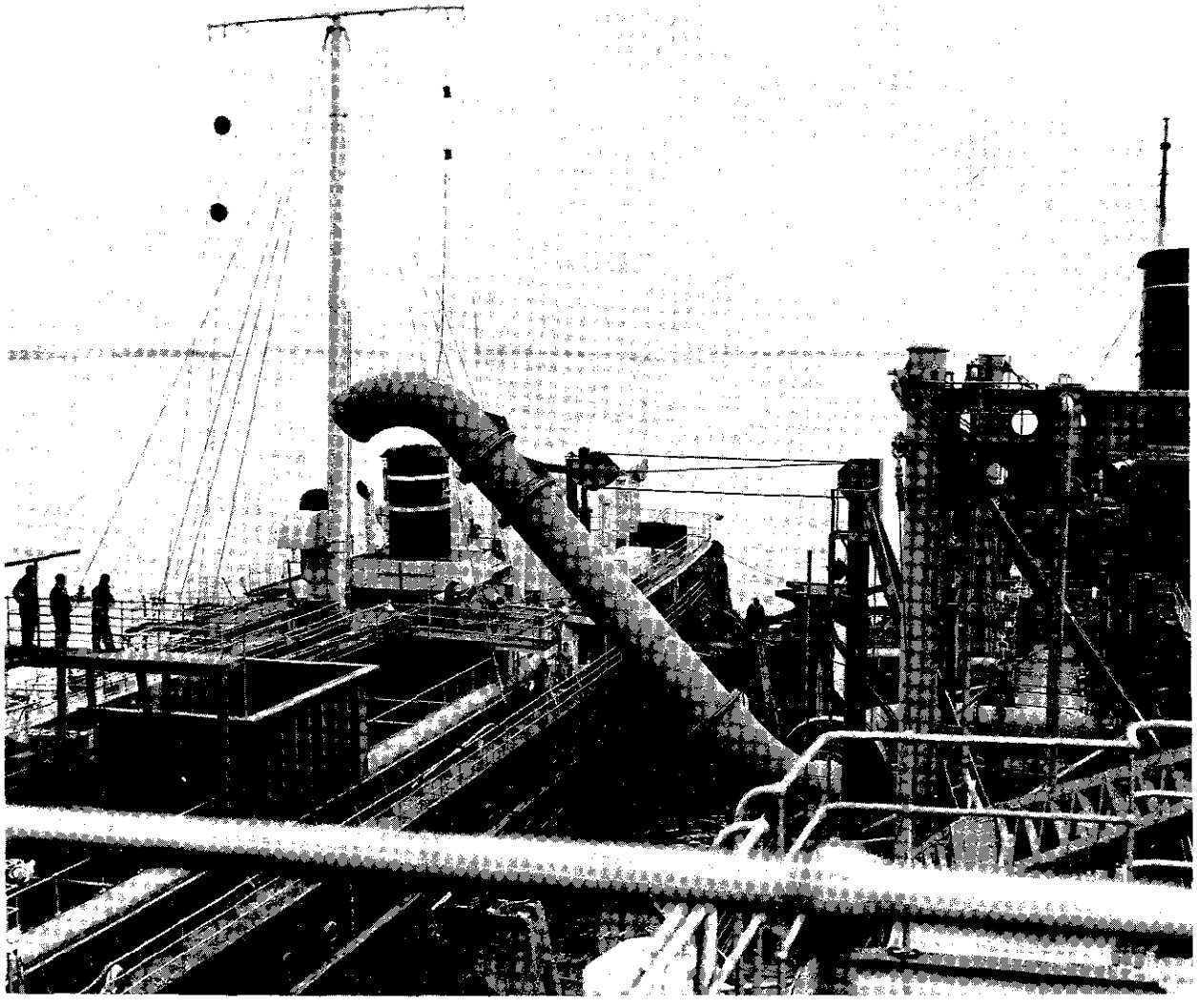

FIGURE 9. — Hopper dredge discharge being set into receiving hopper of sump rehandler *New Orleans.*

nect directly to a pipeline and pump itself out. Obviously this procedure would slow down the disposal operation of the hopper dredge, but a compensating feature would be the elimination of a high-cost rehandling plant required in the sump rehandling process.

Direct pumpout of Government hopper dredges had not been previously used to any appreciable extent. The hopper dredge *New Orleans* was equipped with a direct pumpout system when originally constructed, but no record of the use or effectiveness of this system is available. The hopper dredge *San Pablo* did do some direct pumpout dredging in 1923 and 1924 at Mare Island Strait with an improvised system, but again the records do not indicate any evidence of the degree of its success.

The first modern effort to pump out directly a Government hopper dredge was with the *Hains* in 1959. The direct pumpout capability on the *Hains* was obtained by improvised modifications to the piping system that is normally used to de-water the hoppers. This system normally removes all water that remains in the hoppers after dumping, so that the dredging may commence with completely empty bins. The dewatering system was modified so that the dredged material could be sucked from the hopper and then pumped through a pipe connected to shore. This first effort was on the Calumet River, where it was formerly necessary to haul dredged material 13 miles into Lake Michigan. More recently, direct pump ashore with the *Hains* also has been utilized in dredging the Rouge River. An area of 75 acres at Grassy Island in the Detroit River has been diked and

riprapped and mooring facilities have been provided for direct pumpout operation. During the first two seasons of operation, over 300,000 cubic yards of material have been directly pumped ashore by the *Hains*. The average haul to the diked area is 6 miles, whereas the nearest alternate location for open water disposal in Lake Erie would have involved a haul of 23 miles. This success has prompted the Corps to make plans to procure and dike an area at Holland Harbor, Mich., to replace the long haul now necessary from the inner end of the project.

The trend toward more extensive use of this method of positive disposal of dredged material is next evident in the *Markham*. The original design and construction of this 2,600-cubic-yard hopper dredge, which was completed in 1960, included a direct pumpout capability. The dredge can place 2,000 hp. of its total available 5,300 hp. for use in the pumpout operation. This is in contrast to the dredge *Hains*, which is limited to only the 410-hp. pumping power available for normal dredging operations.

One pump ashore project was done by *Markham* in 1961 at Toledo, Ohio, where approximately 416,000 cubic yards of maintenance dredging material was pumped about 1,400 feet into a diked area provided by the city of Toledo on park property. A second shore disposal area is being developed at Toledo known as Buckeye basin, an inland area about 3,500 feet distant. The discharge line will be laid in an 8-foot storm sewer, which discharges into the Maumee River close to present moorings. Use of storm sewer is necessary because the area to be traversed is in the metropolitan part of the city. The Buckeye basin will hold about 5,000,000 cubic yards and it is estimated it will be adequate for about 20 years of maintenance dredging in the river section. A third shore disposal area in Toledo is being created just outside the river mouth by a new work contract requiring diking of a disposal area in Maumee Bay for pipeline dredging. This will be available in future years for hopper dredge disposal from the inner bay channel and lower river channel.

In a short time an area in Lake St. Clair known as Gull Island, a former open water pipeline dump when channels in the vicinity were dug, will have to be diked for confined disposal of maintenance dredging from channels in Lake St. Clair. The island is about 30 acres of firm ground with much additional bottom just awash around it.

From the knowledge that direct pump ashore operations have been performed successfully, coupled with an indicated need for replacing the existing sump rehandler *New Orleans* in the near future, it has been determined that a more efficient program would result from adoption of the direct pumpout technique on hopper dredges utilized in the Delaware River and Norfolk Harbor. A hopper dredge that can also rehandle material directly ashore has the obvious advantage of more mobility and less fixed operating cost than the system that requires a hopper dredge and a sump rehandler. As stated earlier, these latter features were the ones that were proving a limitation on a greater use of sump rehandling. In view of this, it was concluded that the method of direct pumpout was generally preferable and the decision was made to convert the *Comber* and *Goethals* for self-unloading directly into shore disposal areas and to eliminate the sump rehandler.

In summary, the decision for direct pumpout in lieu of sump rehandling was arrived at because:

(1) Direct pumpout plant (by conversion of existing hopper dredges) can be provided at less initial cost than sump rehandling plant.

(2) The volumes of material that can be placed through a sump rehandling facility indicates it may become limited to an inefficient part-time use.

(3) The lesser operating cost of a direct pumpout dredge than a hopper dredge plus a sump rehandler will place the means of taking advantage of a positive retention dredging system more within the budget limitation of smaller projects, which have not been able to afford the sump rehandler.

Sump rehandling ceased in February 1963 when the *New Orleans* was deactivated for eventual disposal. The hopper dredge *Comber* was converted for direct pumpout. This conversion alters the dredge so that her full propulsion power of 6,000 hp. is utilized for direct pumpout. The *Comber* pumpout power for sump rehandling was limited to the dredge pump power of 1,150 hp. per pump or 2,300 hp. total. This alteration provides the *Comber* with the ability to pump through 20,000 feet of pipeline. As a result, direct pumpout operations in the Delaware River were initiated in May 1963 and hereafter it is anticipated that the 9,000,000 cubic yards of material previously dredged annually with sump rehandling will be dredged, utilizing the direct pumpout technique instead. In order to provide the productive capability required to handle this volume of material, the dredge *Goethals* will also be similarly converted to include pump ashore facilities in the spring of 1964.

There is presently under design a new hopper dredge to replace the *Mackenzie* in the Southwestern Division. This proposed 3,100-cubic-yard dredge, to be named *McFarland*, will be the most modern that our present technology can produce. A feature of this dredge will be its

ability to utilize as much as 5,600 hp. for direct pumpout operations. This will enable the dredge to pumpout through as much as 20,000 feet of pipeline. This capability for direct pumpout ashore through a pipeline is expected to be of considerable value in certain areas of the Sabine-Neches Waterway and the Houston Ship Channel, particularly where a shortage of disposal areas sufficiently accessible for conventional pipeline dredges is developing as a result of waterfront industrialization.

## Side-Casting (or Boom-Discharging) Operations

As indicated in the foregoing discussion on agitation dredging, hopper dredges are sometimes used to perform this type of dredging where certain conditions exist, making the overboard discharge of essentially all pumpings advantageous. Since 1957, private interests have developed self-propelled seagoing dredges of a type similar to Corps hopper dredges, but designed principally as more effective, high-capacity agitation dredging tools. To date, these dredges have been used only in Mexico and Venezuela, mainly in the latter country.

D. K. Ludwig, president of National Bulk Carriers, Inc., owners and operators of some of the world's largest oil tankers and ore carriers, reportedly conceived the idea of such dredges for dredging the bar at the mouth (called Boca Grande) of the Orinoco River that extends seaward over 25 miles. In 1956, this company converted a T-2 tanker to an experimental side-casting dredge. The conversion essentially consisted of installing two steam-turbine driven 36-inch suction, 32-inch discharge dredge pumps, with associated trailing suction dragarm assemblages of conventional hopper dredge design. However, no hoppers were provided, and the discharges from the two pumps are carried through individual 25-inch diameter overboard discharge pipes that form integral parts of a fixed (boom) structure extending 250 feet outboard of the starboard side of the vessel.

The material comprising the Boca Grande bar is a consolidated clayey silt with an in-place density averaging over 1,600 grams per liter. The controlling depth prior to commencement of work by the *Sealane* in December 1956 was approximately 12 ft. By May, 1961, the *Sealane* had removed over 100,000,000 cubic yards of this sediment through the entire length of the offshore bar, thereby providing a navigation channel for 60,000-d.w.t. ore carriers loaded to 33-ft. draft. Also, it is significant that since the method of operation involved resulted in gradual deepening over the entire length of the channel, interim use of the channel was permitted during the course of the work with attendant increasing benefits. The *Sealane's* success became apparent in its first year's operation. This prompted Venezuelan authorities to enter into a long-term charter agreement in 1958 with National Bulk Carriers for the construction of a specially designed 4-pump combination hopper and boom-type dredge and its use in deepening the Maracaibo Bar and the channel within Lake Maracaibo to permit navigation by the largest deep draft oil tankers.

This dredge named *Zulia* was built in Kure, Japan, in 1959 (figs. 10 and 11). It is a steam turbine vessel 548 feet long, 95 feet in beam (molded), and 40 feet deep, with 8,500-cubic-yard capacity hoppers and four 32-inch dredge pumps, each fitted with a suction dragarm assemblage (2 outboard or side drags and 2 inboard in ladder wells). The discharge piping is arranged so that the hoppers can be loaded in the conventional manner or the spoil can be pumped directly overboard through a single 57-inch boom-supported discharge line extending 328 feet from the ship's side. The boom can be rotated 180° from one side to the other, to permit discharge on either side. The *Zulia* undoubtedly is the largest and most productive earth-moving machine afloat. In its first year's operation it pumped about 57,000,000 cubic yards, mostly from the bar channel. The dredgings were thrown to either side of the channel, and reportedly most of the material was carried away by littoral currents and did not return to the channel.

Another dredge similar to the *Zulia* in size and pumping capability, except that it is diesel-powered and has a hopper capacity of 3,500 cubic yards, was built in 1961 by National Bulk Carriers. This dredge named *Icoa* was also constructed in Kure, Japan, for long-term charter by the Orinoco Mining Co., a subsidiary of U.S. Steel Corp., and is presently working in the Orinoco River to deepen further and maintain both the Boca Grande bar and river sections of the channel.

The Corps of Engineers has followed the development of the *Sealane, Zulia,* and *Icoa* with considerable interest. It is recognized that under certain conditions that the side-casting method of agitation dredging provides a practical means of removing large quantities of material relatively inexpensively and rapidly. In addition, when controlling depths do not permit conventional loading of hopper dredges during initial stages of work, side-casting may be employed economically and expediently to obtain an interim improvement in channel depths. Such conditions were encountered by the Corps in 1959 when it became necessary to dredge the entrance channel to the small harbor of Ontonagon, Mich., on Lake Superior. A very uniform controlling depth of between 9 and 10 feet ex-

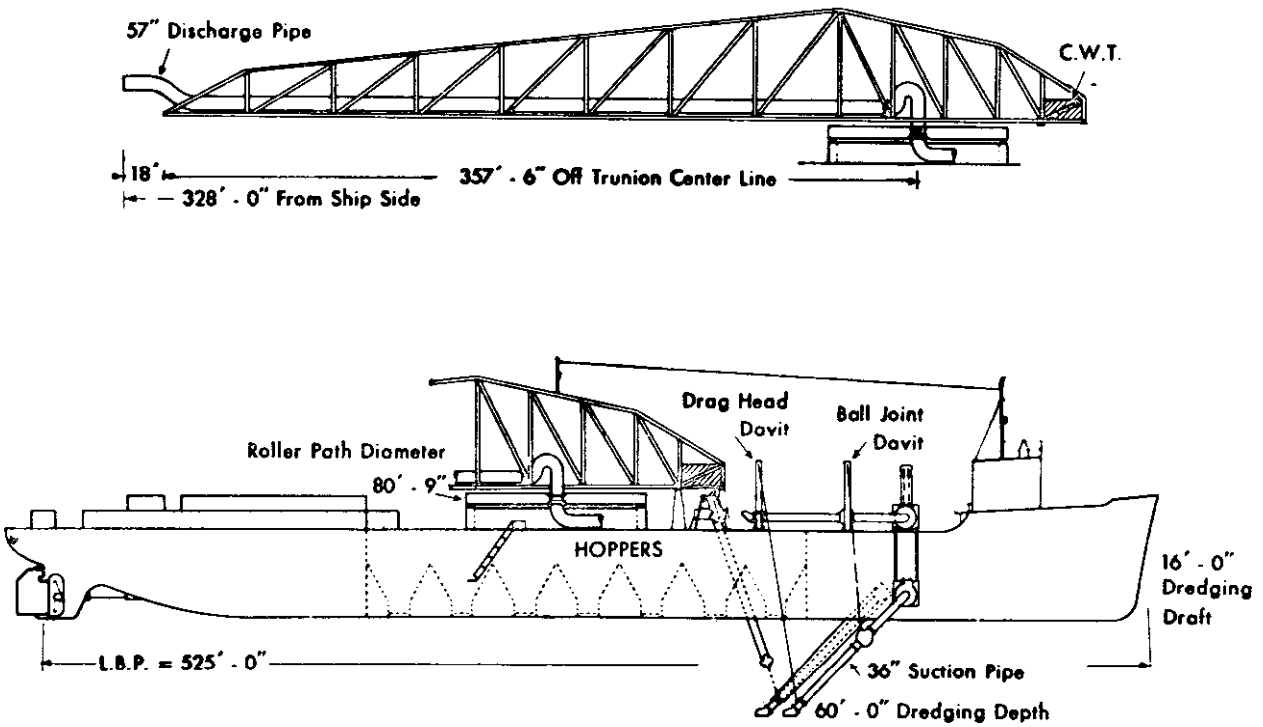


FIGURE 10. — Dredge *Zulia:* Elevations showing arrangement of boom discharge pipe.

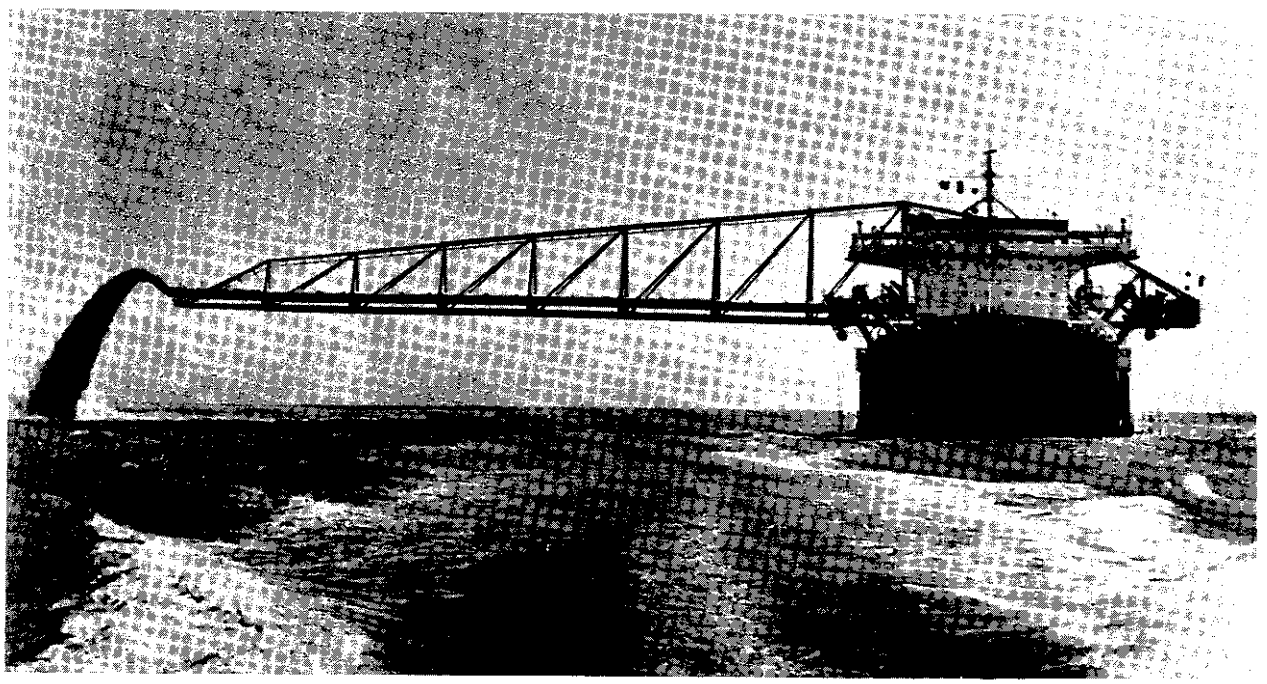

FIGURE 11. — Dredge *Zulia* during side-casting operations.

isted in the channel section between pier breakwaters, which owing to draft requirements, precluded conventional use of the hopper dredge *Hains*. Therefore, this dredge was fitted with a discharge pipe extending approximately 20 feet from the ship's side to permit the dredging of a pilot channel by discharging directly overboard. The outer 10-foot section of pipe was elevated at a 20° angle, and at full pump power the point of discharge impact was approximately 45 feet from the dredge. By using this improvised setup, the critical part of the channel was deepened to 14 feet — sufficient for flotation required for normal loading and hauling operations. The material dredged was mostly a uniform fine sand with an in-place density of approximately 2,000 grams per liter. It settled rapidly despite prevailing current velocities of about 3 miles per hour, and after dredging soundings showed most of the side-boomed material remained where deposited. After this first experience with side-casting in the Great Lakes, the *Hains* has been equipped with a discharge pipe of 70-foot outreach (with impact about 100 feet off the ship's side) as shown in figure 12 and presently it is being used in selected projects in the Detroit district, particularly where the material may be deposited behind breakwater piers. Production in such cases is reportedly from 60 to 160 percent greater than when hopper dredging is done conventionally and the spoil is hauled a mile or more to a bottom dump.

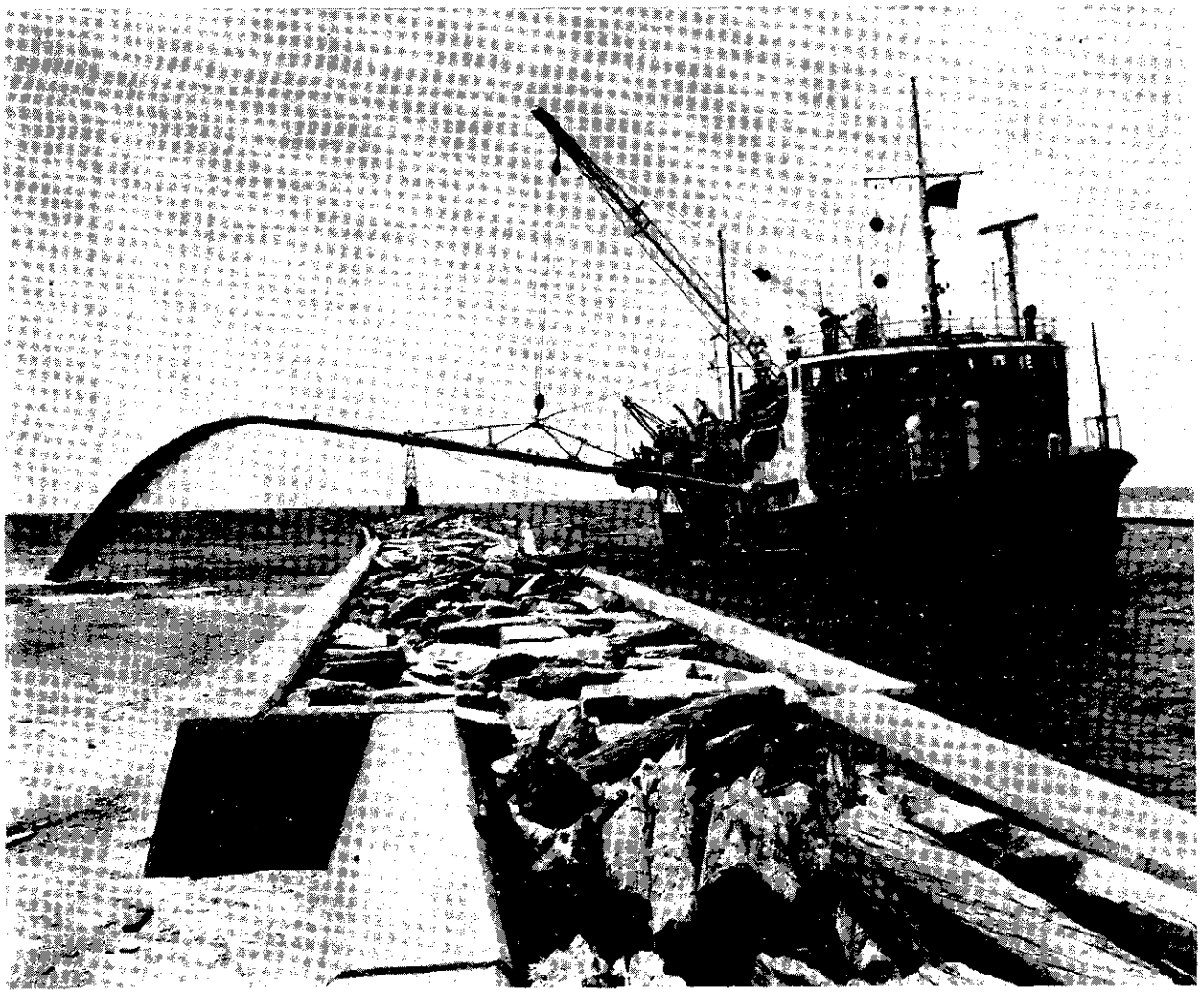

FIGURE 12. — Hopper dredge *Hains* during side-casting operations at White Lake Harbor, Mich.

Side-casting capability also is being incorporated in the design of the proposed new dredge *McFarland*. This will be the first Corps of Engineers hopper dredge equipped with a revolving boom-supported discharge line for this purpose. The proposed vessel will be 300 feet long and will accommodate a fabricated aluminum boom 174 feet, 2 inches long (measured from the center of rotation to the discharge end), which will extend 138 feet, 2 inches beyond the nearest side of the dredge in the 90° working position. Side-casting dredging operations with this dredge in selected offshore channel sections of Gulf coast projects where prevailing cross-channel littoral currents occur are expected to reduce dredging costs. These projects include the Sabine-Neches outer bar, Freeport Harbor, Galveston entrance channel, Port Aransas, Brazos Island harbor, and Matagorda ship channel.

In addition, the Corps of Engineers is presently planning to convert two small surplus Navy vessels (one YSD and one YF) for exclusive use (no hoppers will be provided) as side-casting dredges in various shallow coastal inlet projects along the North Carolina coast and in similar exposed channels along the Louisiana coast, respectively.

## Summary

In most hopper dredging work, spoil disposal is dumped in deep water. However, other methods of disposal listed have been or are being used under certain conditions:

(1) Dumping in rehandling basins (for subsequent disposal ashore by pipeline dredges);
(2) Agitation dredging;
(3) Sump rehandling (self-unloading);
(4) Direct pump ashore (self-unloading);
(5) Side-casting (boom discharge).

It is noteworthy that the two principal unorthodox hopper dredge disposal methods being given increasing attention involve widely different basic philosophies. On the one hand, the development of direct pumpout emphasizes the permanent removal of sediment so that it can never become a source of further shoaling or have any other detrimental effect on water resources development. At the other extreme, side-casting involves returning all the dredged material to the waterway directly adjacent to the shoal. This contrast becomes sharper when it is noted that the newest hopper dredges are equipped to dredge both ways; that is, they can pump directly ashore or side-cast, in addition to being able to operate conventionally by filling hoppers and dumping through bottom gates.

This increase in versatility of the hopper dredge manifests that there is no single solution to fit all dredging problems. Studies of harbors and estuaries are required to provide for an intelligent selection of spoil disposal techniques. These studies must include sedimentation and current analyses. From such analyses it can be concluded if the low cost and high-volume side-casting operation is advantageous, or if the slower and more expensive direct pumpout method should be utilized for the greatest long-range economy, or if the intermediate more conventional techniques are more appropriate.

The significant factor is that the modern hopper dredge is being designed and constructed to suit a wide range of requirements, and the task for the engineer is becoming one of giving greater emphasis to a truly considered discrimination of the various dredging and disposal methods that are available to him.

# SOME ENGINEERING ASPECTS OF DISPOSAL OF SEDIMENTS DREDGED FROM BALTIMORE HARBOR

[Paper No. 66]

By Michael A. Kolessar, *chief, Navigation Reports Section, U.S. Army Engineer District, Baltimore*

## Synopsis

Disposal of sediments dredged from channels and harbors becomes increasingly difficult as waterfront property is developed commercially and disposal areas become scarce. This paper presents a method for obtaining more effective use of a limited area. It describes laboratory tests made to determine the bulking characteristics of the sediments when placed in a confined area by hydraulic pipeline dredge.

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It has been said that great ports are usually geographic accidents. They achieve their early prominence by virtue of their location with respect to centers of population and overland transportation networks. Interestingly, however, the further development of the port quite often depends, to a large degree, on the geological history of the region. The ease with which channel dimensions can be increased and maintained is closely related to the geomorphology of the area and particularly to sedimentation processes that for centuries have contributed to the makeup of the river bottom.

The port of Baltimore is a striking example of the strong influence these processes have on

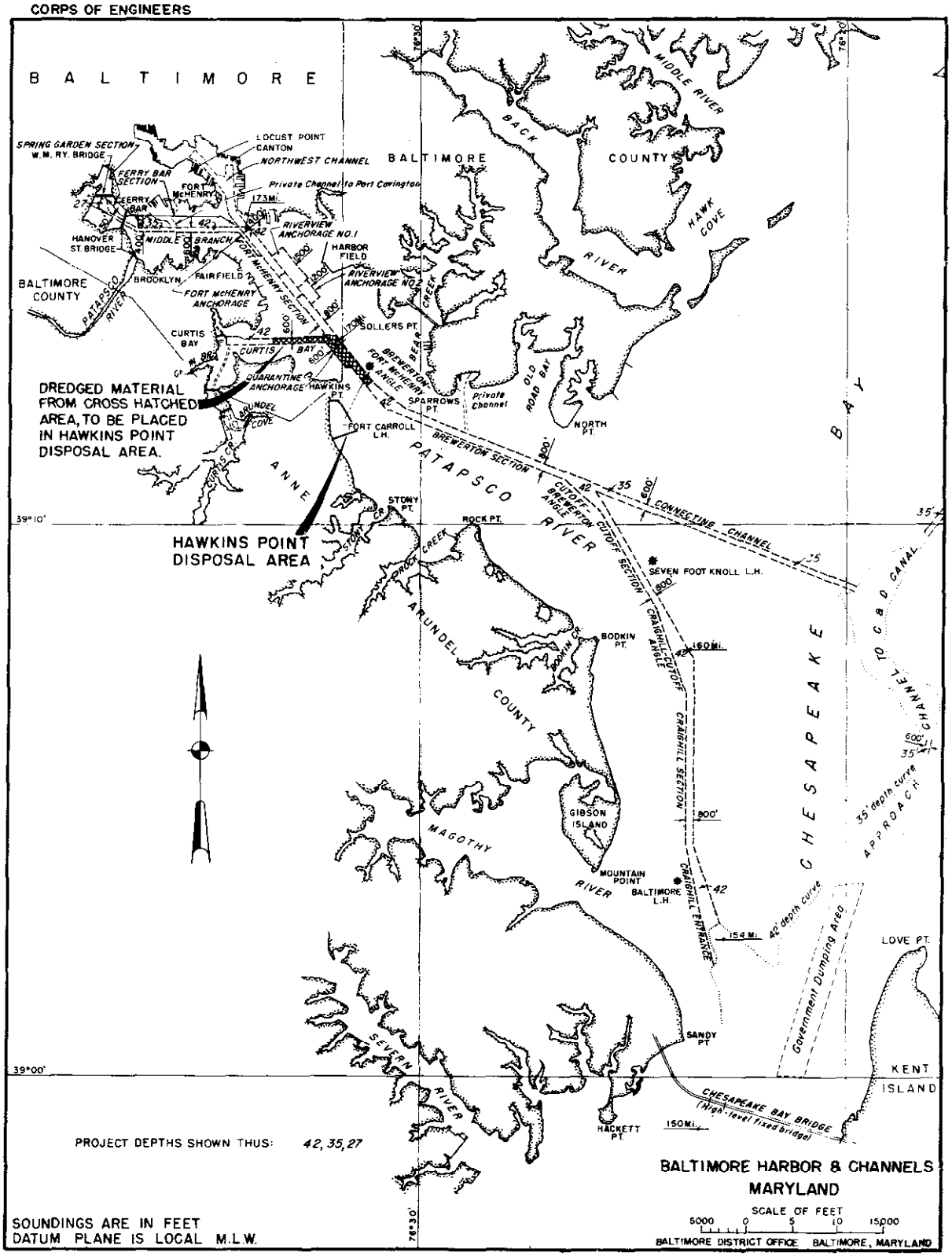



FIGURE 1. — Upper reaches of Baltimore Harbor and channels, Maryland.

the development of channels for deep-draft traffic (fig. 1). Baltimore is a fall-line city, being on the boundary between the Atlantic Coastal Plain and the Piedmont Plateau, and is located at the head of the navigable part of the Patapsco River, which is a tidal estuary on the west side of Chesapeake Bay. Chesapeake Bay, actually a large tidal estuary, represents an ancient drainage system, which was drowned toward the end of the Pliocene epoch. Further modification was introduced by the rising and falling sea level upon a presumable stable land surface during the following Pleistocene epoch.

The valleys of the ancient drainage system in unconsolidated Coastal Plain deposits of pre-Pleistocene Age were subsequently filled by alluvium eroded from highlands and by deposits of Recent Age after inundation by marine waters. It is in these recent deposits of silt, clay, and sand that the navigation channels in the Patapsco River are dredged and maintained. The existence of these soft sediments has contributed largely to the growth of the Baltimore port. The material is relatively easy to dredge, and a minimum of maintenance is required because the shoaling rate is quite small. Another advantage is that the soft bottom of the channel presents far less danger of damage to heavily laden vessels operating with little clearance under the keel. The presence of these fine-grained materials, which in the past have made it feasible to keep pace with the ever-increasing demands for channels to accommodate larger and larger carriers, now presents one of the most difficult problems in further development of the port.

The problem is one of disposal of the dredged material. Waterfront property on the periphery of the harbor is highly industrialized and valued, in some cases, as high as $15,000 per acre. Large land areas for dumping are simply not available within economic distances from the channels to be dredged. Water areas adjacent to the shoreline are not often usable because depths of water and poor foundation conditions make the cost of necessary retaining dikes prohibitive. The dredged material, composed of silt and clay with a dry weight of about 35 pounds per cubic foot, is not ideal fill, and property owners are not eager to obtain such material for enhancement of their lands. In view of these circumstances, the efficient use of the limited areas available for disposal of spoil is not only desirable but becomes a very important factor in determining the method of dredging that will be used.

This paper presents an approach to the design of an optimum-size disposal area to be used for disposal of material dredged by a hydraulic pipeline dredge. The author, who was one of the engineers who formulated the plan, will readily admit that methods used may appear cumbersome to those more familiar with sedimentation processes. It is his hope, however, that if any fundamental concept has been missed or improperly applied, informed readers will bring the matter to his attention.

One of the disposal areas made available by local interests as part of the local cooperation required by the authorizing Act for the Baltimore Harbor project consisted of about 130 acres of water area immediately channelward and adjacent to lands owned by two different corporations. One of the riparian owners, in giving permission to use the offshore area, specified that the final elevation of the fill would not exceed 9 feet above mean low water. The other riparian owner requested that fill be not less than 4 feet above mean low water and not higher than adjacent fast lands that have an elevation of about 35 feet. The average depth of water in the area was about 6 feet but a submarine bar roughly parallel to the shore line and about 2,000 feet therefrom rose to within 3 feet of the water surface. Subsurface explorations confirmed the belief that the bar was composed of hard material and would provide adequate foundation for a moderately high dike. The borings also showed that, within the disposal area, there were pockets of sand and other granular materials which could be used for construction of a dike.

A rule of thumb in general use in the dredging industry for determining size of a disposal area needed for a continuous hydraulic dredging operation is that the volume of the enclosure should be a minimum of 2 to 3 times the in situ volume of material to be removed from the cut.

In this particular instance, however, it was believed that considerable refinement was necessary to assure that the final elevation of the fill would be within the limits specified without having to resort to intermittent work stoppages while the fill decanted. Information was also needed on the retention time that would be required in the disposal area to assure that the effluent over sluiceways would not exceed by more than the specified 20 grams per liter the weight of ambient water in the channel. It was decided to simulate the mixture that might be expected in the dredge pipe and observe the rate of sedimentation in transparent vertical stand pipes.

Five transparent plastic tubes of about 4 inches in diameter were erected in a freight elevator shaft at the Baltimore District Office of the Corps of Engineers. Two of the tubes were slightly over 6 feet in length, two were a few inches longer than 12 feet, and one was

about 24 feet long. The 12-foot lengths corresponded to the probable protoype dimensions of the column of mixture that would occur in the disposal area. The 24-foot tube was erected to simulate the placing of material in a disposal area containing 6 feet of water.

Earlier observations of various mixtures in 1,000 cc. graduates showed, as might be expected, that the rate the interface between clear water and the turbid mixture lowered in the graduate varied in proportion to the amount of mixing water. But since there was some question as to whether the consolidated characteristics of the sediment in the small test could be correlated with the higher columns in the disposal area, it was decided to make direct observation of a full-size column under conditions more nearly resembling the actual operation.

It was not possible to find general agreement among dredging contractors regarding the density of material that would be transported through the proposed 2-mile long dredge pipeline. Most agreed that about 25 percent of the mixture in the pipeline would be solids, but they were not certain if the measurement was by weight or volume.

Laboratory analysis of 14 samples taken in the area to be dredged showed that the in situ density of the material as measured by a Bico-Braun Mudwate hydrometer averaged about 1,300 grams per liter (table 1). Specific gravity of solids ranged from 2.70 to 2.79 and void ratio ranged from 3.413 to 6.063. Mechanical analysis showed that the median diameter of grains was about 0.0022 mm.

TABLE 1.—*Soils data for Curtis Bay channel obtained November 1960*

[Weight of water=62.355 lb. at 60 F°.]

[Water content, $w\% = \frac{\text{weight of water}}{\text{weight of solids}} \times 100$]

| Hole No. | Sample No. | Depth below water surface | Water content w% | In situ density | | Specific gravity of solids | Void ratio | Grain size | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | G./liter | Lb./cu. ft. | | | $D_{50}$ | $D_{10}$ |
| | | *Feet* | *Pct.* | | | | | | |
| C-1 | 2 | 30–35 | 193.3 | 1,258 | 78.4 | 2.728 | 5.856 | ..... | ..... |
| C-1 | 3 | 35–40 | 164.6 | 1,285 | 80.1 | ..... | 5.075 | 0.0014 | 0.00015 |
| C-1 | 4 | 40–45 | 150.5 | 1,319 | 82.2 | 2.696 | 4.413 | ..... | ..... |
| C-2 | 1 | 20–25 | 189.0 | 1,274 | 79.4 | ..... | 5.693 | ..... | ..... |
| C-2 | 2 | 25–30 | 199.9 | 1,260 | 73.6 | 2.794 | 6.063 | .0047 | .00040 |
| C-2 | 3 | 30–35 | 158.1 | 1,313 | 81.9 | ..... | 4.585 | ..... | ..... |
| C-2 | 4 | 35–40 | 161.9 | 1,311 | 81.7 | ..... | 4.622 | .0025 | .00025 |
| C-2 | 5 | 40–45 | 158.8 | 1,319 | 82.2 | 2.716 | 4.478 | ..... | ..... |
| C-3 | 1 | 23–28 | 111.0 | 1,397 | 87.1 | ..... | 3.413 | .0020 | .00010 |
| C-3 | 2 | 29–30 | 25.3 | ..... | ..... | 2.693 | ..... | .28 | .0090 |
| C-4 | 1 | 25–30 | 144.4 | 1,337 | 83.4 | 2.683 | 4.079 | ..... | ..... |
| C-4 | 2 | 30–35 | 159.5 | 1,317 | 82.1 | ..... | 4.406 | ..... | ..... |
| C-4 | 3 | 35–40 | 169.2 | 1,295 | 80.7 | ..... | 4.934 | .0014 | .00025 |
| C-4 | 4 | 40–45 | 165.0 | 1,309 | 81.6 | ..... | 4.705 | .0011 | .00030 |

It was decided to place mixtures of 1 part of 1,300-gram-per-liter material and 4 parts of water by volume in one of the 6-foot tubes and in one of the 12-foot tubes. To obtain an approximation of the effect of a denser mixture, mixtures of 1 part 1,300-gram-per-liter material and 1 part of water by volume were placed in the other 6- and 12-foot tubes. Mixing water was water taken from the site of the proposed dredging and had a specific gravity of 1.003. These proportions were selected as representing the probable upper and lower limits of the density of material in the dredge pipe line.

The 24-foot tube was charged with 6 feet of brackish water and 6 feet of 1-to-1 mixture was added. After 4 days the supernatant liquid was drawn off and another 6-foot lift of 1-to-1 mixture was added. The process was repeated on the 27th day. The elevation of the interfaces was recorded and plotted against elapsed time. The results of the tests are shown on figures 2 and 3. Numbers along the curves represent the relation of the volume of the mixture containing sediments in suspension as compared to the original in situ volume of material of 1,300 grams-per-liter density contained in the mixture. This relation is referred to later in this paper as the bulking factor.

In computing the volume required in the disposal area, the following assumptions were made:

1. Dredging would be by hydraulic dredge with 27-inch discharge that would deposit about 6,350 cubic yards of mixture in the disposal area per hour.

2. Pumping would be in progress 22 hours per day for 25 days per month.

3. Discharge over spillways would not exceed 3 inches in depth, and density of effluent over spillways would not exceed 1,020 grams per liter.

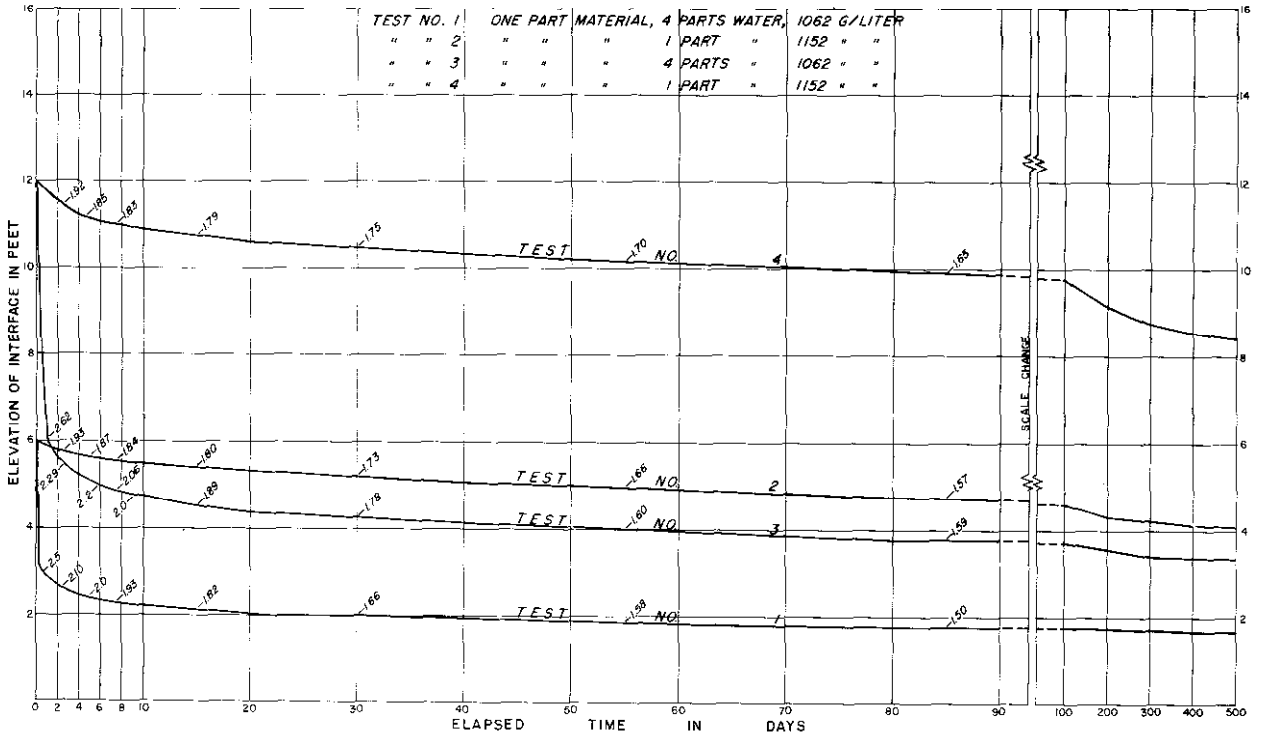



FIGURE 2. — Relationship between elevation of interface with elapsed time: Tests 1 through 4.

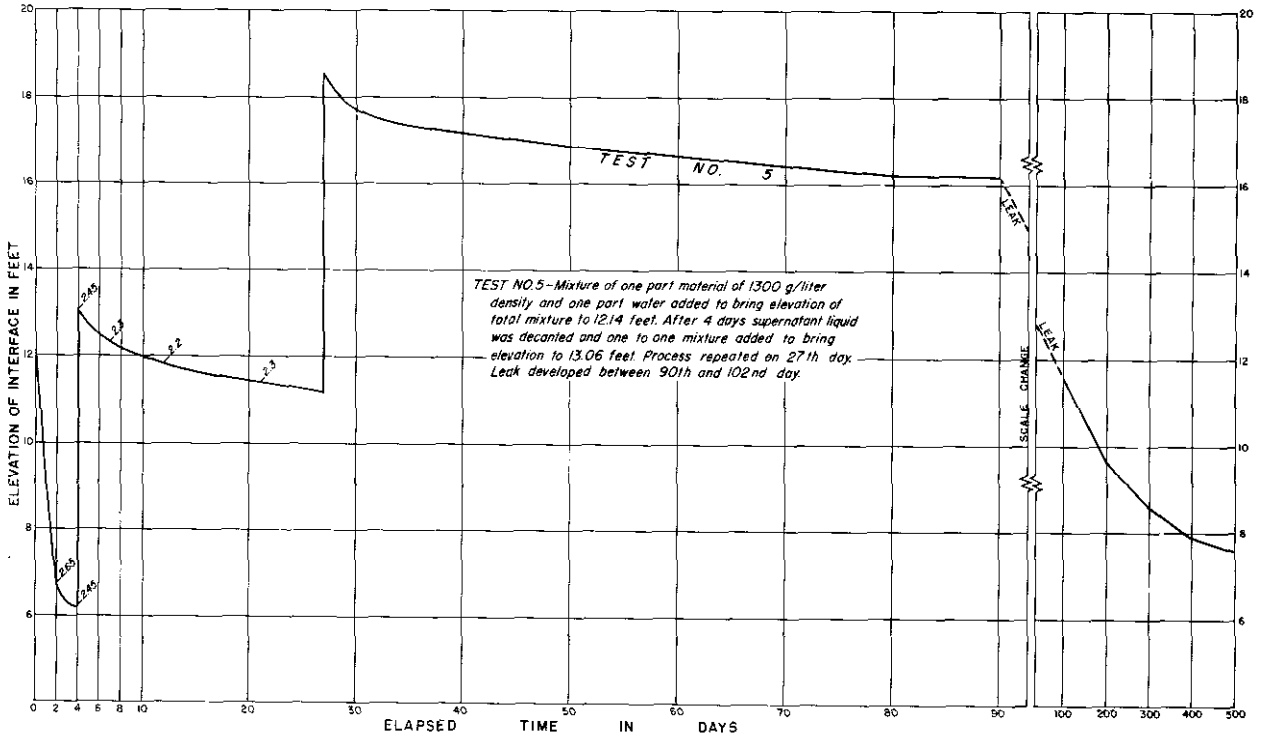



FIGURE 3. — Relationship between elevation of interface with elapsed time: Test 5.

4. Losses over the spillway would be approximately equal to excess overdepth dredging.

5. The density of material in the discharge pipe would be between 1,060 and 1,152 grams per liter.

6. A maximum of 1 million cubic yards in place in the channel would be dredged and placed in the disposal area during 1 month, and the entire pumping operation would be complete in 100 days.

TABLE 2.—*Computed volume of fill after 100 days of pumping*

(In thousands of cubic yards)

| Portion of job (percent) | Volume cut | Days in disposal | Bulking factor test[1] | | | | Volume of fill test | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | 1 | 2 | 3 | 4 | 1 | 2 | 3 | 4 |
| 30 | 960 | 70–100 | 1.50 | 1.57 | 1.59 | 1.65 | 1,440 | 1,507 | 1,526 | 1,584 |
| 30 | 960 | 40–70 | 1.58 | 1.66 | 1.60 | 1.70 | 1,517 | 1,594 | 1,536 | 1,632 |
| 20 | 640 | 20–40 | 1.66 | 1.73 | 1.78 | 1.75 | 1,062 | 1,107 | 1,140 | 1,120 |
| 10 | 320 | 10–20 | 1.82 | 1.80 | 1.89 | 1.79 | 582 | 576 | 605 | 573 |
| 5 | 160 | 5–10 | 1.93 | 1.84 | 2.06 | 1.83 | 309 | 294 | 330 | 293 |
| 5 | 160 | 0–5 | 2.10 | 1.93 | 2.29 | 1.92 | 336 | 309 | 367 | 307 |
| 100 | 3,200 | ..... | ..... | ..... | ..... | ..... | 5,246 | 5,187 | 5,504 | 5,509 |
| Ratio of volume in fill to volume in cut | | | | | | | 1.63 | 1.62 | 1.72 | 1.72 |

[1] Test No. 1 was 6-foot tube of 1 to 4 mix (1,062 g./l.), test No. 2 was 6-foot tube of 1 to 1 mix (1,152 g./l.), test No. 3 was 12-foot tube of 1 to 4 mix (1,062 g./l.), test No. 4 was 12-foot tube of 1 to 1 mix (1,152 g./l.).

Table 2 presents a summary of the computation of the volume required in the disposal area to accommodate 3.2 million cubic yards of material to be removed from the cut. Bulking factors as determined by tests 1 through 4 are shown for increments of time varying from 5 days to 30 days, the shorter times being selected for the periods when the curves showed the greater rates of change. The totals at the bottom of each of the last 4 columns represent the maximum volume the fill will attain and the volume that must be made available in the disposal area.

The density of the initial mix had no appreciable effect on the volume of the fill at the end of the pumping time. Also, the volume of fill as determined by the 12-foot tests indicated a higher relationship of fill to initial volume than did the tests using 6-foot tubes.

The estimated volume of the fill computed on the basis of the test data is believed to be somewhat higher than would actually occur because the test did not take into account the probability that a substantial part of the material dredged would be transported to the disposal area in the form of balls and would not go into suspension. It was considered safe to assume, therefore, that the maximum volume of fill would be not more than about 70 percent greater than the volume in the cut. This narrowing down of the range of bulking that could be expected made it possible to make a better determination of the economical height of the dike and the optimum amount of material to be placed in the disposal area.

The disposal area at Hawkins Point was constructed in September 1962. The contractor pumped up dikes to about mean low water, using borrow areas within the disposal area itself. By dragline he brought material from the inside slope to bring the top of the dike to about 12 feet above mean low water. By excavating material from within the disposal area for dikes, he increased the capacity below mean low water. A hydrographic survey was made after dikes were constructed but before filling operations began. Further surveys after the fill is complete will be made to compare the actual prototype conditions with those predicted by the laboratory tests. From initial observations, close agreement is expected.

# CHANNEL SEDIMENTATION AND DREDGING PROBLEMS, MISSISSIPPI RIVER AND LOUISIANA GULF COAST ACCESS CHANNELS

[Paper 67]

By AUSTIN B. SMITH, *Chief, Navigation and Dredging Branch, U.S. Army Engineer Division, Lower Mississippi Valley and Mississippi River Commission, Vicksburg, Miss.*

## Synopsis

Sediment has been a major navigation problem at the Mississippi River Passes since Sieur de la Tour completed the plans for New Orleans. Channel shoaling in the Mississippi River between Cairo, Ill., and New Orleans, La., has

been a major navigation problem since 1870. The solution to the Passes problem, where the salt-water wedge and the Mississippi River sediment meet, is being resolved by a combination of jetties, contraction works, and dredging. The annual channel shoaling on the 200 crossings between Cairo, Ill., and Baton Rouge, La., has been battled by the dustpan dredge since the development of this unique equipment by the Mississippi River Commission in 1895. Today this problem is being combated by channel stabilization works and dredging.

Silting in the Atchafalaya River, including the basin, the lake section, and the Gulf access channel below Morgan City, is being attacked by a major dredging program. Channel silting in the Calcasieu River 40-foot ship channel is being met by new construction dredging, including retention dikes, enlargement of the existing waterway, and maintenance dredging. Channel sedimentation on the new 36- by 500-foot gulf outlet channel is expected to be heavy. Experience on similar channels and that gained during current construction indicates that channel siltation in this new tidewater channel to New Orleans will be heavy from wave wash of passing vessels and sea action, particularly in the seabar section where the littoral drift is athwart the entrance channel.

## Channel Sedimentation — Historic Problem

During 27 to 30 May 1962, 28 motor vessels with 272 barges were delayed a combined total of 831 hours and 25 minutes due to the loss of project depth in Grand Lake Crossing (mile 504) on the Mississippi River. Until dredged, the controlling depth in this river crossing was 7 feet, whereas the drafts of the loaded barges and towing vessels ranged from 8 to 10 feet. Again, on 15, 21, and 22 August 1962 the navigation channel was blocked by sediment at three other separate shoaled crossings. These channel blocks occurred as the Mississippi River fell rapidly from middling stages and before the Corps of Engineers dredge plant could deepen a number of heavily shoaled crossings. These 1962 low-water navigation difficulties attest to the fact that sediment in the channel remains the major problem in providing a dependable navigation channel on the Lower Mississippi River, the congressionally assigned task of the Mississippi River Commission, Corps of Engineers, U.S. Army.

River engineers have been concerned with Mississippi River sediment since Sieur de la Tour completed the plan of New Orleans and Sieur de Paugér constructed the post called Balize (the buoy) at the mouth of Southeast Pass (Pass a l'Outre), circa 1724, to serve as a guide to ships wishing to make passage over the 10-foot-deep seabars and ascend the river. They tried various schemes to combat the silt and deepen the selected Passes, but it remained for Eads and Corthell in 1875 to design and construct the first successful channel control works which, with dredging, provided a deep-draft channel via South Pass.

In his 1821 examination of the Mississippi River, Captain H. Young of the U.S. Topographic Engineers found the navigation channel infested with logs, snags, and sawyers deeply embedded in Mississippi River silt. Captain Henry M. Shreve removed the snags, but it remained for the Mississippi River Commission to develop, in 1895, the unique dustpan dredge, which has provided a dependable method of maintaining the project navigation channel by removing the annual high-water siltation from those crossing channels that do not erode as the river falls to low stages.

On 24 July 1862, during the War Between the States, Adm. David G. Farragut moved his Union fleet of war vessels from siege before Vicksburg, Miss., downstream to the deeper waters in the Baton Rouge-New Orleans sector to prevent these overdraft vessels from being trapped in hostile territory behind shallow crossings as the river fell to low stages.

By 1870 larger and deeper draft steamboats were being constructed for economy of transportation. During years of extreme low water, overdraft steamboats were forced to lay by for 1 to 3 months of the year because of shoaled crossings. This critical situation led the U.S. Congress to direct that extensive studies be made by the Army Corps of Engineers of the Mississippi River navigation problems and to establish the Mississippi River Commission in 1879.

This Commission gave immediate consideration to the channel siltation problem. It made extensive studies and examined all phases of the Mississippi River regimen, including the movement of sand waves on the riverbed and other siltation phenomena associated with river meander, such as bank caving and the enlargement of the meander loops by alluvial deposits. First attempts to cope with the problem were: (1) regulation of the low-water channel by the construction of permeable dikes, and (2) closure of secondary channels at Plum Point, Tenn., and Lake Providence, La. The dikes and closure dams, as then constructed, were inadequate to withstand the forces of the river. After exhaustive experiments, the Commission adopted dredging as the positive method of maintaining a dependable navigation channel against the cyclic siltation in the Mississippi River. In combating the silt problem by dredging, the

Commission was fortunate in the early development of the dustpan-type dredge, which remains the optimum plant for dredging Mississippi River crossings.

## Mississippi River Channel Shoaling Problem — Cairo to Baton Rouge

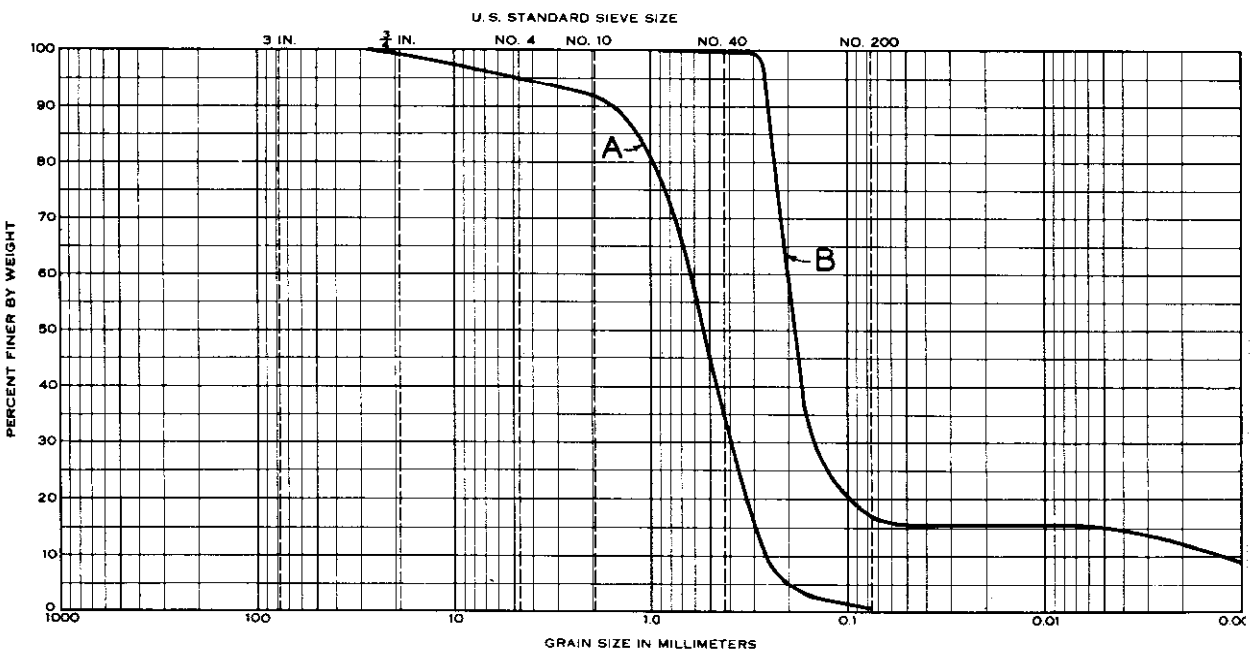
In the analysis of the Mississippi River dredging problems, sediment is considered to be any material (silt, sand, or clay) that is deposited in the channel and has to be removed by dredging. Figure 1 shows the mechanical analysis of a sample taken from the bed of the Mississippi River at a point 813 miles above the Head of Passes and of a sample taken in Southwest Pass near the mouth of the Mississippi River.

The two major maintenance requirements are (1) removal of high-water shoaling from the Mississippi River crossings, and (2) removal of highwater shoaling in the "shabby" reaches where the river's flow is bifurcated and trifurcated. Figure 2 shows a typical Mississippi River meander and the crossings that usually shoal at high water and require dredging at low stages.

Bank caving has been a major source of the material that is deposited in river crossings and "shabby" reaches during high-river stage. In his 1892 study and analysis of the 1877-83 and the 1892 hydrographic and caving bank surveys of the Mississippi River, J. A. Ockerson, member, Mississippi River Commission, found that the total length of caving banks between Cairo, Ill., and Donaldsonville, La., was then 921 miles, and that the average annual volume of caving per mile from Cairo to Donaldsonville was then 1,003,579 cubic yards.

From 1945 prototype studies of Mississippi River bank caving, occurring between the 1931 and 1941 hydrographic surveys, Austin Smith and Edgar McCleave, staff engineers of the Mississippi River Commission, reported as follows:

> Without stabilization there will be continued caving of the bank. The amount of annual bank caving on the Mississippi River between Cairo, Illinois, and the mouth of Old River near Angola, Louisiana, a river distance of approximately 770 miles, is some 800,000,000 cubic yards. While the magnitude of this bank caving, when indicated only in figures, might not seem excessive, it would require about seventy-five 30-inch dredges working constantly to move this volume of material annually. The Mississippi River is the world's biggest dredge.




Sample A obtained from bed of Mississippi River 813 miles above Head of Passes.
Sample B obtained from Southwest Pass near the mouth of Mississippi River.

FIGURE 1. — Grain-size distribution at Mississippi River crossing below Cairo and at mouth of Mississippi River.

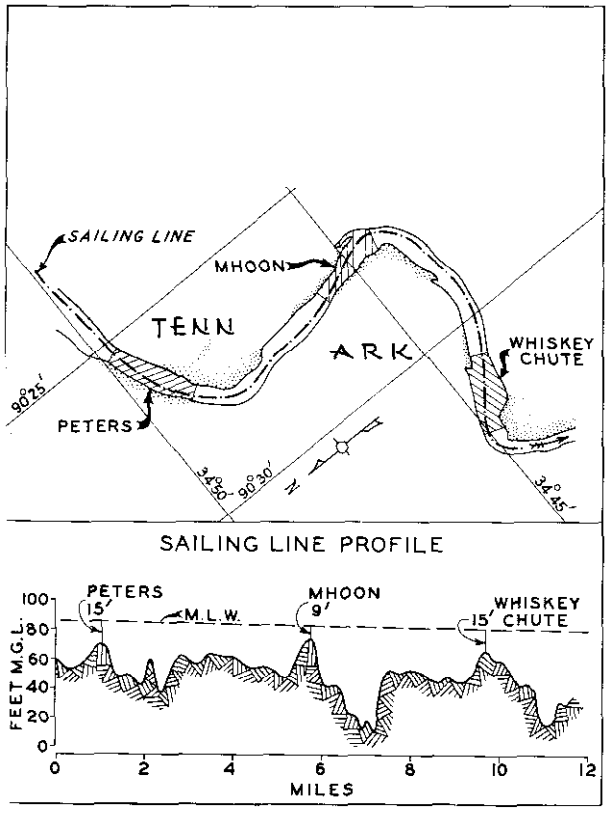



FIGURE 2. — Typical crossing.

Figure 3 shows the 1931-41 caving bank history between Cairo and Old River, as described above.

These two caving bank studies, some 50 years apart, show that in its natural regimen annual bank caving on the Mississippi River between Cairo and Donaldsonville averaged over 1,000,000 cubic yards per mile. Both reports show that the heaviest bank caving occurred between the mouth of the Arkansas River and Natchez, Miss. The influence of this source of channel sediment on the Mississippi River regimen is marked.

The Mississippi River Commission in its study and review report on the Mississippi River between Cairo and Baton Rouge, dated 14 February 1944, House Document No. 509 (78th Congress, 2d Session), noted that the source of sediment from caving banks would be markedly reduced when the river is stabilized, and stated:

> Stabilization of the banks will reduce the sand in motion, will fix the location of crossings and reduce the amount of dredging required to maintain the 9-foot channel of the existing project. It is estimated that the existing authorization for maintenance dredging will therefore be ample to secure a minimum low-water depth of 12 feet after stabilization is accomplished.

In concluding its report the Commission stated:

> The Commission is of the opinion that stabilization of the river is necessary to retain reduction in floods obtained by channel rectification and is advisable for the purpose of safeguarding the main Mississippi River levees and protecting the investment which they represent. The Commission is of the further opinion that such stabilization may materially increase the flood-carrying capacity of the river channel and, together with maintenance dredging already authorized, will provide a minimum depth of 12 feet at low water for navigation.

With the advance of the stabilization program there has been some reduction in the annual dredging requirements to maintain the interim 9- by 300-foot navigation channel.

## Channel Crossing Shoaling — Cairo to Baton Rouge

Between Cairo and Baton Rouge there are some 200 river crossings, including a number of "shabby" reaches where there is divided flow. As the river falls to low stages, some 40 to 85 crossings do not scour and thus require annual dredging. The annual dredging estimates are based on the assumption that at least 60 crossings will require dredging during normal low-water seasons.

The number of crossings requiring dredging in the fiscal years 1953-61, inclusive, is shown on table 1. The number of crossings having a controlling depth of less than 12 feet in selected years between 1936 and 1960 is shown on table 2.

TABLE 1.—*Number of crossings dredged between Cairo, Ill., and Baton Rouge, La., 1953–62*

| Fiscal year | Vicksburg lowest flow | Number of crossings dredged at— | | | Total |
|---|---|---|---|---|---|
| | | Memphis district | Vicksburg district | New Orleans district | |
| | *C.f.s.* | | | | |
| 1953 | 132,000 | 60 | 14 | 1 | 75 |
| 1954 | 136,000 | 54 | 31 | 1 | 86 |
| 1955 | 145,000 | 37 | 16 | ..... | 53 |
| 1956 | 128,000 | 35 | 29 | ..... | 64 |
| 1957 | 133,000 | 33 | 23 | ..... | 56 |
| 1958 | 230,000 | 30 | 14 | ..... | 44 |
| 1959 | 198,000 | 33 | 15 | ..... | 48 |
| 1960 | 162,000 | 33 | 11 | ..... | 44 |
| 1961 | 205,000 | 35 | 9 | ..... | 44 |
| 1962 | 246,000 | 37 | 5 | ..... | 42 |

TABLE 2.—*Number of crossings with less than 12-foot depth, Cairo, Ill., to Baton Rouge, La., for selected years, 1936–60*

| Year | Crossings with less than 12 feet | Vicksburg low flow |
|---|---|---|
| | *Number* | *C.f.s.* |
| 1936 | 130 | 94,000 |
| 1938 | 71 | 157,000 |
| 1939 | 153 | 100,000 |
| 1942 | 41 | 254,000 |
| 1943 | 74 | 161,000 |
| 1956 | 77 | 128,000 |
| 1960 | 54 | 187,000 |

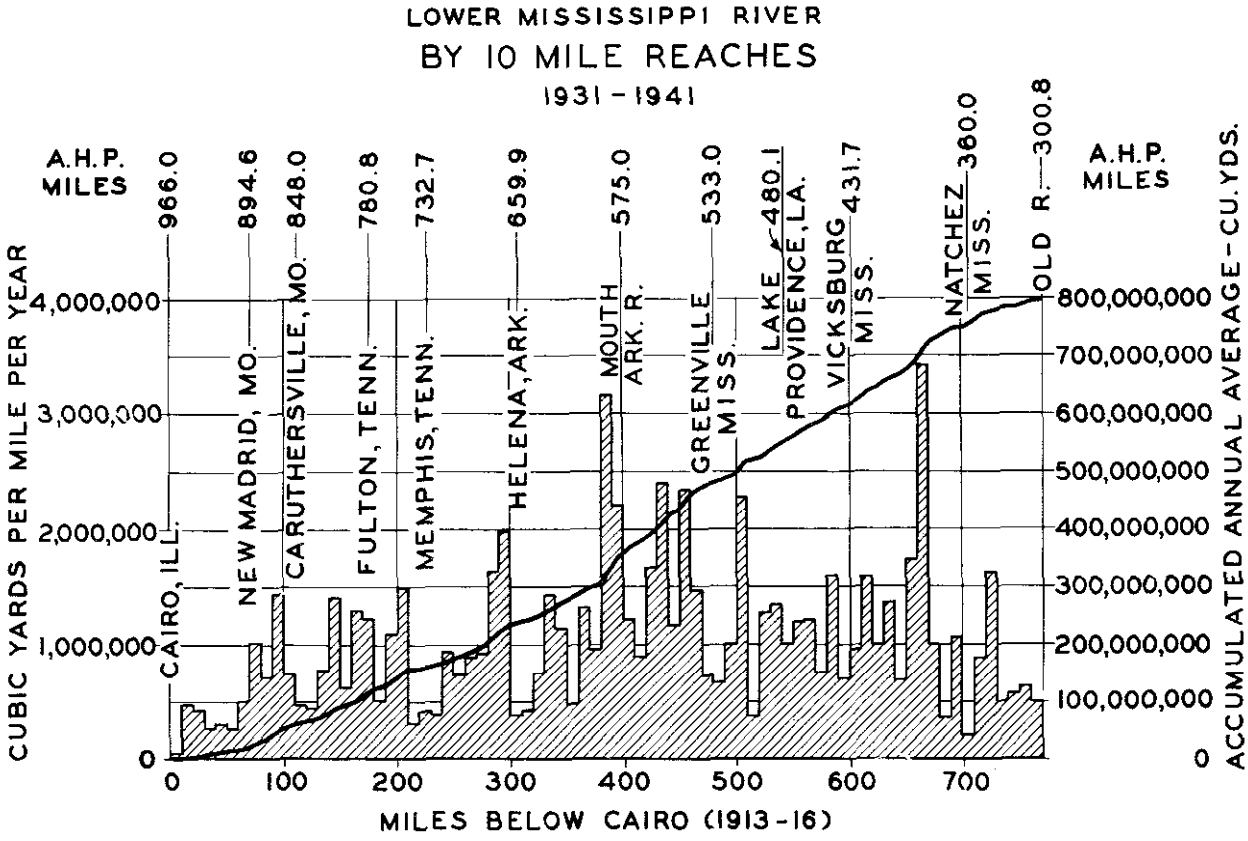



FIGURE 3. — Caving bank history.

Since the crossing bars and divided flow reaches build up during high-water stages and about one-third of them do not scour out satisfactorily of their own accord, steps must be taken at the beginning of each low-water season to (1) determine which crossings will require attention and mark the best channel available until improvements can be made, (2) make "before dredging" surveys for getting the dredging work properly laid out, (3) move dredges to the jobs and deepen the shallow crossings at the proper time, and (4) check all crossings regularly during the low-water season and dredge if necessary.

## Annual Maintenance Dredging Requirements — Cairo to Baton Rouge

The annual maintenance dredging required to provide a dependable 9- by 300-foot channel from Cairo to Baton Rouge, a distance of approximately 737 miles, ranges from an extreme low of 20,000,000 to a high of about 45,000,000 cubic yards. The quantity of maintenance dredging required during the fiscal years 1930 through 1962 is shown on table 3. This large amount of maintenance dredging is accomplished annually by four large dustpan dredges, three of which are located in the Memphis district and one in the Vicksburg district. This dredging work is performed annually between May and January at an annual cost of about $4½ million. The schematic diagram of the operation of the dustpan dredge is depicted on figure 4.

The dustpan dredging requirements listed in table 3 show an increase during 1932-45, reflecting a channel rectification program that included 16 cutoffs and other major channel realinements. While channel stabilization accomplished since 1945 has had a salutary influence on the annual dredging requirements, there are other factors that must be considered in estimating the dredge capacity needed to maintain a dependable 12-foot channel between Cairo and Baton Rouge. These factors are (1) the increased vessel draft that is depicted on figure 5, (2) increase in lost dredging time due to passing navigation, ranging up to 16 hours in some situations, and (3) outsize tows that require greater than the project-width channel for safe passage during low river stages.

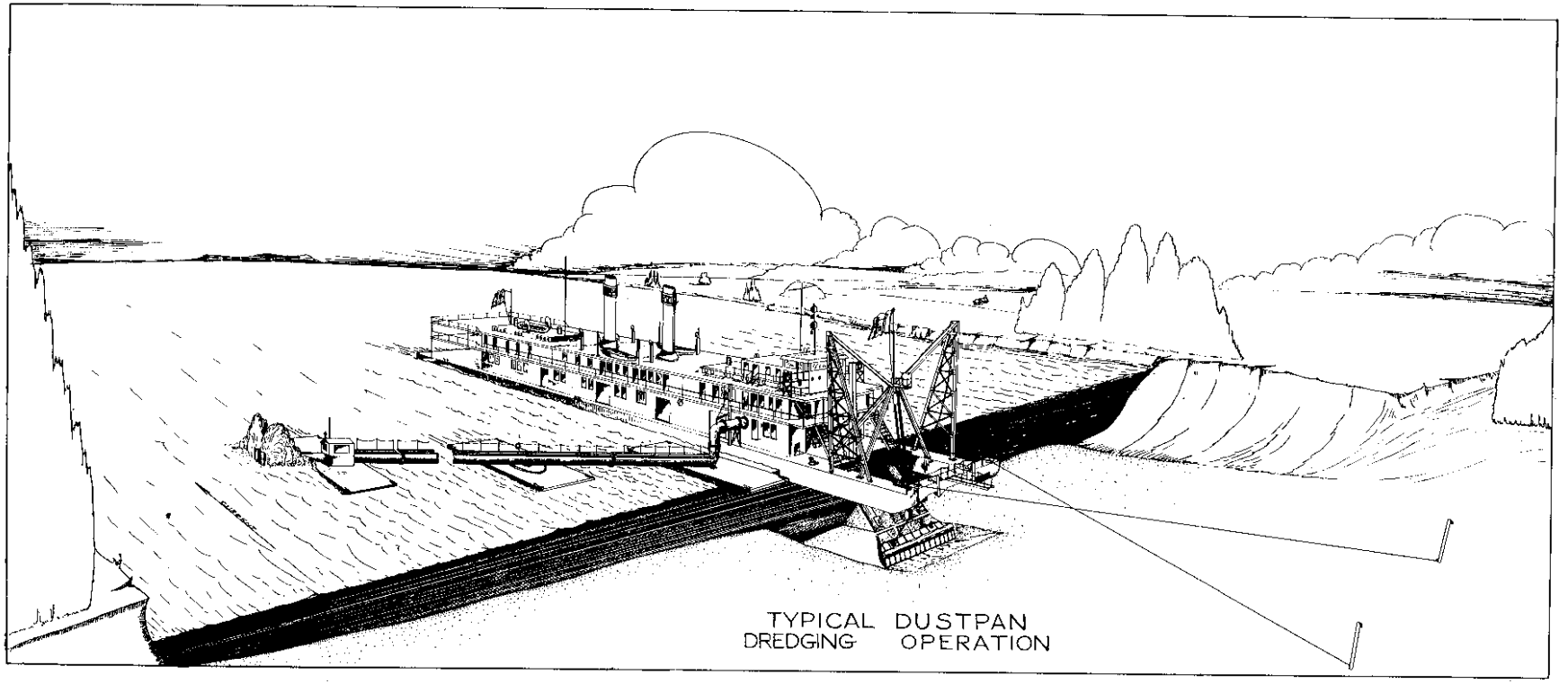



FIGURE 4. — Schematic diagram of dustpan dredge operation.

TABLE 3.—*Maintenance dredging between Cairo, Ill., and Baton Rouge, La., 1930–62*

| Fiscal year | Quantity excavated in— | | | Total |
|---|---|---|---|---|
| | Memphis district | Vicksburg district | New Orleans district | |
| | *Cu. yd.* | *Cu. yd.* | *Cu. yd.* | *Cu. yd.* |
| 1930 | 5,044,507 | 0 | 0 | 5,044,507 |
| 1931 | 7,579,131 | 0 | 0 | 7,579,131 |
| 1932 | 8,916,679 | 0 | 84,000 | 9,000,679 |
| 1933 | 12,127,300 | 184,363 | 258,653 | 12,570,316 |
| 1934 | 29,673,173 | 437,690 | 208,215 | 30,319,078 |
| 1935 | 27,554,524 | 183,310 | 97,300 | 27,835,134 |
| 1936 | 17,596,668 | 918,744 | 0 | 18,515,412 |
| 1937 | 31,069,025 | 2,625,898 | 0 | 33,694,923 |
| 1938 | 26,108,704 | 1,483,501 | 99,415 | 27,691,620 |
| 1939 | 19,624,409 | 3,105,420 | 0 | 22,729,829 |
| 1940 | 31,168,225 | 4,262,679 | 1,086,660 | 36,517,564 |
| 1941 | 25,814,861 | 6,201,059 | [1] 8,116,124 | 40,132,044 |
| 1942 | 19,713,591 | [2] 6,200,000 | [1] 11,407,272 | 37,320,863 |
| 1943 | 15,466,388 | [2] 6,150,000 | [1] 13,930,593 | 35,546,981 |
| 1944 | 27,037,000 | [2] 8,450,000 | [1] 5,478,000 | 40,965,000 |
| 1945 | 25,162,000 | 7,686,000 | [1] 9,174,000 | 42,022,000 |
| 1946 | 20,278,277 | 5,420,075 | [1] 6,924,800 | 32,632,152 |
| 1947 | 17,143,575 | 10,373,460 | 551,775 | 28,068,810 |
| 1948 | 30,407,310 | 10,956,235 | 1,138,141 | 42,501,686 |
| 1949 | 27,901,570 | 9,324,614 | 1,762,142 | 38,988,326 |
| 1950 | 29,242,773 | 9,223,574 | 432,294 | 38,898,641 |
| 1951 | 25,987,195 | 6,991,611 | 0 | 32,978,806 |
| 1952 | 14,271,086 | 2,850,564 | 1,490,285 | 18,611,935 |
| 1953 | 35,163,231 | 5,358,374 | 132,545 | 40,367,307 |
| 1954 | 39,830,162 | 6,156,021 | 373,852 | 46,403,493 |
| 1955 | 29,839,459 | 7,288,930 | 0 | 37,128,389 |
| 1956 | 33,286,998 | 8,706,476 | 0 | 41,993,674 |
| 1957 | 34,390,837 | 5,806,922 | 0 | 34,390,837 |
| 1958 | 22,204,396 | 6,777,131 | 0 | 28,981,527 |
| 1959 | 26,928,000 | 7,230,000 | 0 | 34,158,000 |
| 1960 | 25,335,247 | 4,942,497 | 50,300 | 30,328,044 |
| 1961 | 24,709,000 | 4,943,000 | 218,000 | 29,870,000 |
| 1962 | 28,213,000 | 7,502,000 | 199,000 | 35,914,000 |

[1] Under channel conditions after major rectification of channel trace.

[2] Quantities are approximate and are adjusted to reflect only maintenance dredging work, excluding all improvement dredging.

## Baton Rouge to the Head of Passes

In this sector of the Mississippi River, where a 35- by 500-foot channel is maintained, there are some six crossings that may require dredging, depending upon the magnitude and duration of the high water. Here, as the river falls to low stages, high-water shoaling is removed by dredging, presently with the use of a hopper dredge. Recent prototype experiments with a dustpan dredge have proved successful at crossings at Red Eye and Bayou Goula, La. Early consideration will be given by the Corps of Engineers to the development of a dustpan dredge capable of efficient excavation to 60-foot depth for use in maintaining the recently authorized 40- by 500-foot channel in this sector of the Mississippi River.

## Special Maintenance Dredging Problems

In addition to the maintenance of a dependable channel on the Mississippi River, the Corps of Engineers project provides for the maintenance of access to the off-river harbor channels at Memphis, Helena, Greenville, Vicksburg, and Baton Rouge. The siltation of the Vicksburg Harbor that followed the 1876 Centennial cutoff has been satisfactorily solved by diverting the Yazoo River past the Vicksburg city front via the lower arm of the abandoned Centennial Band. High-water siltation in the approach channels to the other four harbors requires annual dredging.

The port of Baton Rouge requires no dredging to maintain the authorized 500-foot channel through the harbor. At New Orleans, annual channel sedimentation is heavy along the convex side of the channel. Dredging is required to maintain the 35-foot- by 1,500-foot-wide channel, which is parallel to and 100 feet riverward of the wharf line.

## Mississippi River Passes

The Mississippi River brings to its mouth sediment loads of 1,000,000 tons daily. As shown in figure 6, this alluvian silt advances the 35-foot-deep contour at the mouth of the

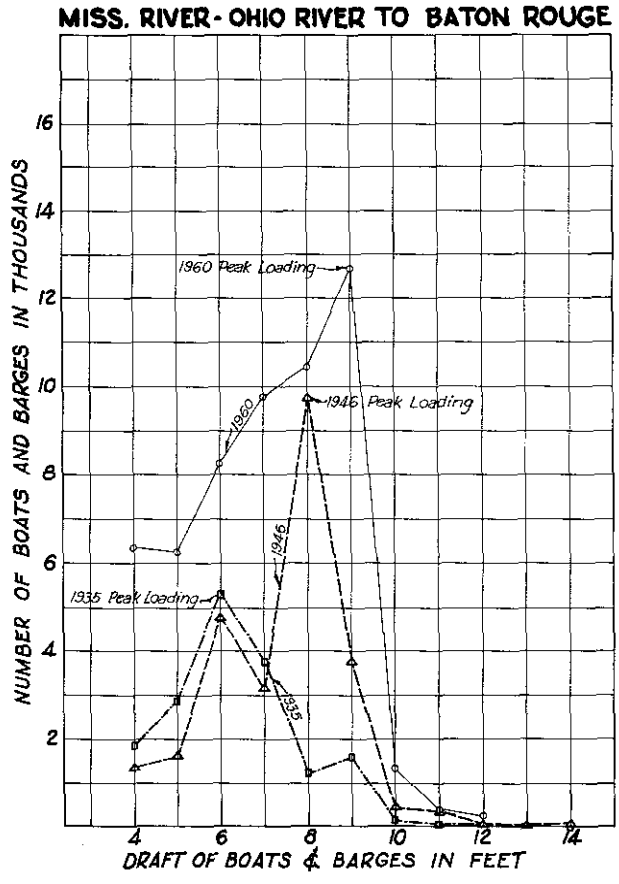



FIGURE 5. — Vessel draft increase.

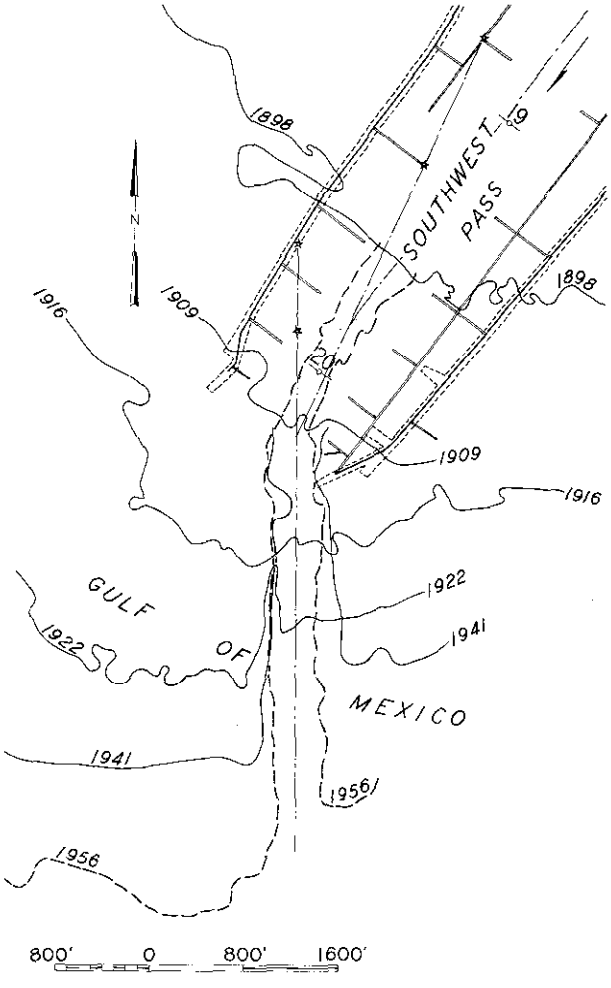



FIGURE 6. — Advance of 35-foot contour, Mississippi River delta.

Mississippi River seaward about 100 feet per year. Consequently, silt is the major factor to be considered in the planning, construction, and maintenance of a dependable navigable channel through the Passes. The planning and construction work by Eads and Corthell in 1879 at South Pass remains an outstanding example of overcoming the silt-salt water consequence. Only a nominal amount of dredging is required to maintain the 30- by 600-foot channel.

Annual siltation is important to the maintenance of the batture at the Mississippi River Passes. Where the batture becomes narrow from wave wash of passing vessels and winds, it is frequently necessary to construct outlet channels to supply siltation in the open bay in order to strengthen the batture banks on the bay side and prevent serious crevassing during high-river stages.

The Eads design of the South Pass jetties and the subsequent relocation of the entrance range across the seabar largely eliminated the silt-salt water intrusion problem at South Pass. At Southwest Pass improvements are in progress to provide a dependable 40-foot channel based on a design channel layout developed from model and prototype studies. The improvements at Southwest Pass consist of pile dikes to collect the silt and constrict the channel and realinement of the lower jettied channel and the approach channel across the seabar, to conform generally with the channel alinement at South Pass. The purpose is to provide, insofar as possible, a self-maintained 40-foot channel throughout Southwest Pass without increasing the hopper-dredge requirements resulting from high-water sedimentation at the mouth. High-water deposition in the Mississippi River Passes requires the removal of some 8,000,000 cubic yards annually.

## Atchafalaya River Basin Shoaling

Deposition of sediment in Atchafalaya Basin east and west floodways has become a serious threat to the flood-carrying capacity of the floodways. The development of an enlarged central channel is in progress in order to confine the flow of the Atchafalaya River through the basin and close off side-diversion channels that are carrying silt into the floodways. Since 1954 some 115,000,000 cubic yards has been dredged or placed under contract to be dredged in developing and enlarging the central channel. About 140,000,000 cubic yards is scheduled to be dredged to complete the central enlargement and distributary closures from Alabama Bayou, mile 54.0, to Stouts Pass, a distance of 61 miles. High-water siltation in Grand Lake requires annual maintenance dredging to provide the 12- by 125-foot navigation channel.

In the sector of the Atchafalaya River between Morgan City and the Gulf, siltation in the 15-mile seabar and approach channel requires annual dredging.

## Calcasieu River

The 35-mile deep-water channel (30 by 250 feet) between Lake Charles and the Gulf of Mexico was completed in 1941, and at that time maintenance was assumed by the United States. In 1953, deepening of the channel to 35 feet was completed. The project channel crosses three large lakes and courses along the margin of Calcasieu Lake for a distance of 14 miles. Siltation from stream runout, wave wash of passing ships, and extreme weather conditions has been heavy, particularly through Calcasieu Lake. Since 1953 some 29,900,000 cubic yards of material has been removed in maintaining the project channel.

The present narrow 250-foot channel is being enlarged to 40 by 100 feet to accommodate supertankers. In addition to enlargement of the existing channel, the project will provide for constructing parallel dikes through the lake sections to lessen the heavy siltation in these sectors of the waterway.

## Gulf Outlet Channel

It is anticipated that siltation from wave wash of passing vessels, sea action, and the bedload movement from the littoral currents, which are westward athwart the seabar channel, will require extensive maintenance dredging to maintain the 70-mile tidewater channel now under construction from New Orleans to the Gulf of Mexico via Chandeleur Islands. The bulk of the shoaling is expected to occur in 20 miles of open-water channel through Breton Sound and across the seabar channel to the Gulf of Mexico.

## Conclusions

The river engineer is concerned with two ultimate problems in the maintenance of our modern waterways: (1) the reduction in annual channel sedimentation and (2) the prevention of bank erosion by the commerce it carries. These vexing problems are aggravated by rising dredge costs and the increased volume and speed of traffic carried by the waterway.

On the Lower Mississippi River annual channel and harbor sedimentation should lessen with the completion of upstream reservoirs and stabilization of the channel below Cairo. Here the yardsticks of increasing traffic and accelerated industrial growth are excellent measures of the economic benefits being derived from the dredging required to remove the annual channel and harbor siltation.

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# PHENOMENA AFFECTING IMPROVEMENT OF THE LOWER COLUMBIA ESTUARY AND ENTRANCE

[Paper No. 68]

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## Synopsis

This paper describes work undertaken to improve the lower Columbia estuary and entrance for navigation and discusses past concepts of phenomena controlling the regimen of this area as related to these improvements. Recent advances in technology have given birth to a new concept of tidal hydraulic phenomena that emphasizes the relation of salinity intrusion and littoral movement to the degree of shoaling experienced in estuarine areas. The findings of prototype measurements undertaken in 1959 along the lower Columbia and other endeavors to expand the knowledge of controlling phenomena, including statistical wave studies, analyses of offshore changes, studies of attrition

and accretion of adjacent shorelines, and comprehensive investigations of the distribution of Columbia River sediments are reviewed in the light of this new concept. Authorization, construction, and verification of the comprehensive lower Columbia estuary hydraulic model are discussed, as well as tests proposed for identification of controlling phenomena and for development of the most effective and economical plan of improvement. Finally, the paper summarizes the extent of present knowledge and outlines considerations for the future under conditions of controlled upland discharge resulting from contemplated headwater reservoir operation.

## Introduction

### *The River*

#### *Physical characteristics*

Columbia River rises in Columbia Lake, British Columbia, Canada, and flows in a circuitous course for 462 miles to cross the International Boundary, from where it flows generally southward 435 miles across the State of Washington to the Washington-Oregon State line, thence generally westward for 310 miles through the Cascade and Coast Ranges to empty into the Pacific Ocean near Astoria, Oreg., as shown in figure 1. The river and its tributaries drain an area of 259,000 square miles, consisting of rugged mountain ranges separated by valley troughs, trenches, and plateaus, of which 39,500 square miles lie in Canada. Elevations in the basin range up to nearly 14,000 feet above sea level in the Rocky Mountain system on the east. The total fall of the Columbia River from its source to the Pacific Ocean is 2,652 feet.

#### *Runoff*

The normal annual runoff of the Columbia River is 180,100,000 acre-feet, and the discharge pattern of the lower river is quite regular with low flows occurring from September through March and high flows from April through August. At The Dalles, Oreg., 192 miles above the entrance, the mean annual runoff is 133,700,000 acre-feet and extremes in mean daily discharge at this point have ranged from 36,000 to 1,240,000 c.f.s.

#### *Role in economic development*

Water resource development has been of basic significance in the economic evolution of the Pacific Northwest, which has in recent years emerged from an almost complete dependency on its basic resources of agriculture and timber to diversification and broadening of economic activities by production of more and more finished products. Water transportation by inland rivers, the Pacific Ocean, and the Puget Sound area has played a major role in the economic expansion of this great region. With suitable harbors that are strategically located, a flourishing trade has developed with Alaska, Canada, the Far East, California, and the east coast of the United States. The Columbia River, the most important inland waterway, provides a route for ocean-going traffic from its mouth to Vancouver, Wash., and, through the connecting Willamette River, to Portland, Oreg. Improvements are being undertaken to extend deep-water navigation with authorized channel depth of 27 feet to The Dalles. Barge traffic extends up the Columbia to Pasco, Wash., by a combination of open-river regulation and slackwater navigation provided by Bonneville, The Dalles, John Day, and McNary Dams. With completion of four authorized dams on the Lower Snake River, slackwater navigation for barge traffic will then extend to Lewiston, Idaho. Progressive improvement of the Columbia River has been accompanied by significant increases in ocean-going commerce as shown by traffic trends of major commodity groups in table 1.

TABLE 1.—*Ocean-borne commerce on Columbia River*

| Item | 1925 | 1940 | 1950 | 1955 | 1960 |
|---|---|---|---|---|---|
| | *Thousand tons* | *Thousand tons* | *Thousand tons* | *Thousand tons* | *Thousand tons* |
| Agricultural products | 733 | 812 | 695 | 1,153 | 4,417 |
| Animal products | 41 | 49 | 61 | 20 | 69 |
| Minerals | 31 | 86 | 323 | 71 | 989 |
| Forest products | 1,998 | 1,962 | 1,372 | 1,775 | 1,041 |
| Petroleum products | 1,720 | 3,050 | 5,971 | 7,090 | 5,551 |
| Miscellaneous | 713 | 965 | 792 | 820 | 426 |
| Total | 5,236 | 6,924 | 9,214 | 10,929 | 12,493 |

### *The Lower Estuary*

#### *Extent*

The Columbia Estuary, as affected by tidal variations, extends during periods of low river flows up to Bonneville Dam on the Columbia River, 140 miles above the entrance, and to Willamette Falls at Oregon City on the Willamette River. At higher river stages the sections of the river thus affected by tidal conditions progressively move downstream. The reach below Oak Point, Wash. (river mile 52), shown on figure 2, forms the lower Columbia Estuary, and tidal conditions in this section generally have more effect on stages than river flows. In this reach the main river channel traverses a meandering course between generally widening bank lines, past many low islands and shoals, to eventually join the Pacific Ocean between the rugged headlands of Cape Disappointment on the north and the low-lying Point Adams on the south.

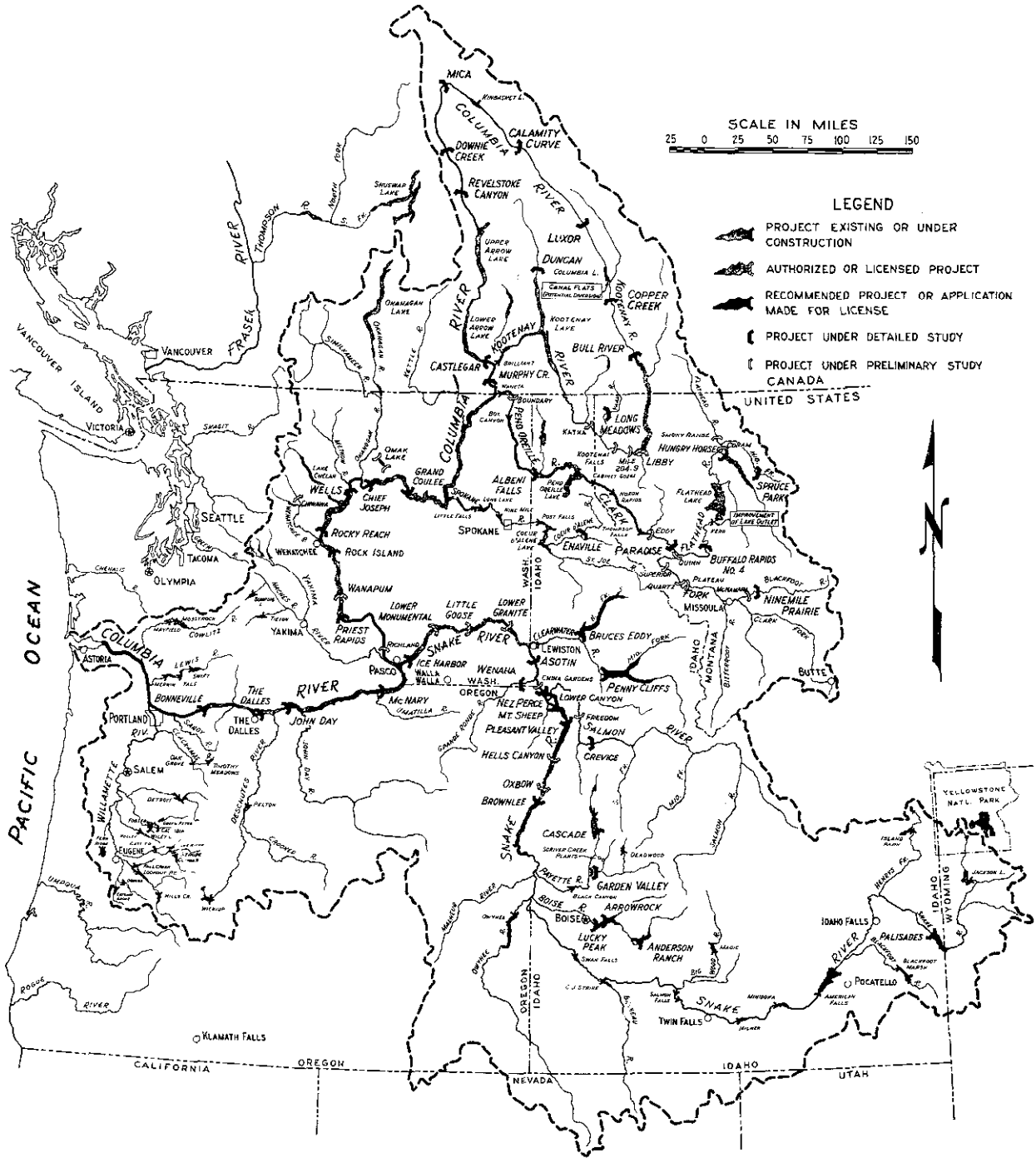



FIGURE 1. — Columbia River Basin.

### *Tidal influence*

Normally, reversals of current direction occur throughout the lower Columbia Estuary as the result of tidal fluctuations of the Pacific Ocean. At Fort Stevens, river mile 8, the average daily tidal variation is about 8 feet. The lowest tide observed at this point was estimated to be 3.0 feet below mean lower low water and the highest stage, which resulted from a combination of storm activity and tidal action,

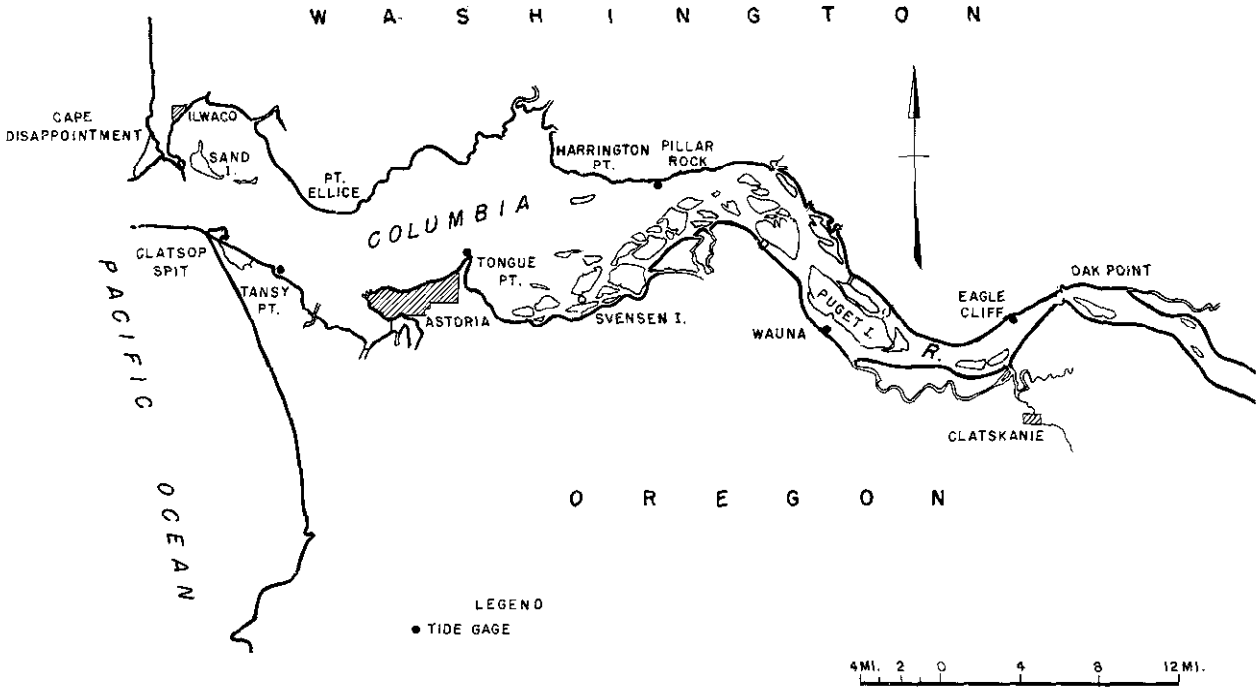



FIGURE 2. — Lower Columbia Estuary.

was 11.6 feet above mean lower low water. The tidal prism of the Columbia Estuary has been estimated at 732 square-mile feet.

### *The Entrance*

#### *General*

Perhaps the most consistent characteristic of the Columbia entrance, figure 3, is its continually changing regimen that strives to create a balance between the forces of the ocean and those of the river. This balance is quite delicate, as any modification of the river forces in the entrance area, no matter how minor, is accompanied by a corresponding change in ocean forces. As a consequence, throughout the period of man's endeavor to improve conditions for navigation, the entrance has undergone many changes and has assumed many different complexions.

#### *Tides*

Tides at the Columbia River entrance have the diurnal inequality typical of the Pacific coast, with a long runout from higher high to lower low water. The mean range is 6.5 feet, the range from mean lower low water to mean higher high water is 8.5 feet, and extreme tides range from minus 2.6 feet to plus 11.6 feet, mean lower low water.

#### *Winds and storms*

Two seasons having wide variance in wind and sea conditions generally occur each year at the entrance. The season of severe storms usually extends from October to April, and storm winds, beginning generally in the south quadrant, gradually veer to the southwest and finally blow themselves out as they move around to the west or northwest. Storms are often continuous for several days and are accompanied by extremely heavy seas mostly from the southwest. During the summer season, the conditions of wind and sea are much less severe than those of the winter season. From May through September, winds from the north or northeast begin in the morning, increase in force during the day, and diminish at night. Occasionally during the winter and spring, there are several days when an easterly wind predominates and the weather is cold.

## History

### *Prior to Improvement*

A chart (fig. 4) made under the direction of Admiral Vancouver clearly indicated the existence in 1792 of a single channel at the Columbia entrance with a depth of 4½ fathoms between Cape Disappointment and Point Adams. In 1839 Sir Edward Belcher revealed the existence of two channels at the entrance separated by a large shoal area which forced the deeper channel northward against Cape Disappointment to turn sharply southward and join a generally shallower channel along Point Adams, as shown in figure 5. This shoal area, called the

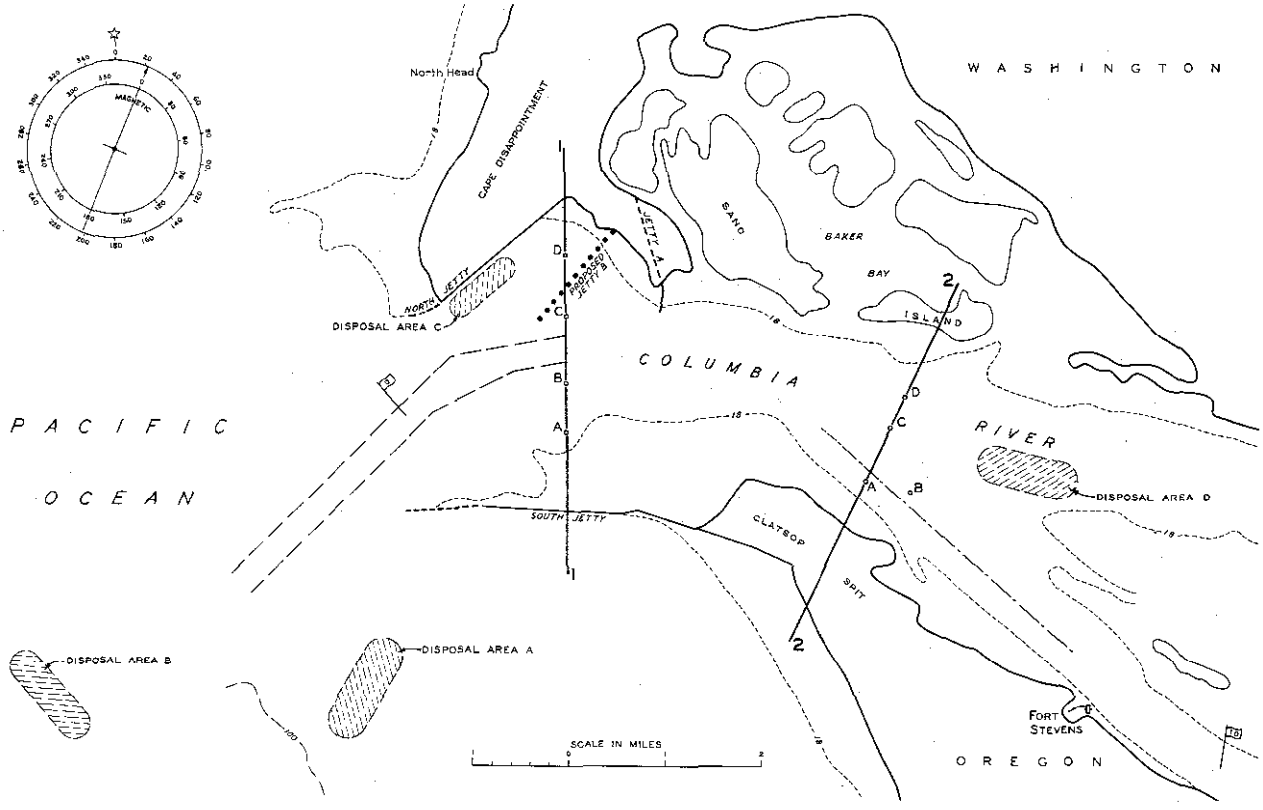



FIGURE 3. — Columbia Entrance.

Middle Sands, grew in size until its 18-foot contour in 1874 embraced a very large area seaward and south of Cape Disappointment and covered almost all of the area now occupied by the present entrance. A part of the Middle Sands formed an island inside the entrance that later separated from the main shoal area and, migrating to the north to form Sand Island, restored a single channel to the entrance in 1885. At that time the main part of the Middle Sands had moved westward to create a westward extension of Cape Disappointment and minimum depths of 20 feet prevailed on the ocean bar, as shown in figure 6.

### *30-Foot Entrance Project*

Early attempts to improve the entrance, prior to the adoption by Congress in 1882 of the initial permanent navigation project, consisted of sporadic dredging in combination with the construction of temporary training structures. It was soon realized even in those early days that such limited work could not be effective in controlling, or even influencing to any appreciable degree, the outsized forces at work in the entrance and that improvement of more permanent character would be required. The first project was adopted on August 2, 1882, pursuant to the recommendations of the board of engineers for the permanent improvement of the mouth of the Columbia River.[1]

This project provided for the construction of a South Jetty, 4½ miles in length, lying in a generally northwesterly direction from its base at Point Adams, to secure a depth of 30 feet over the entrance bar. Construction of the rubble-mound structure was initiated in April 1885 and completed, with four groins along its northerly side, in October 1895. The crest of the completed jetty sloped from 12 feet above mean lower low water at its shore end to plus 10 feet at a point 1⅛ miles from the shore, thence to plus 4 feet at its outer end. A total of 946,000 tons of stone were placed in the structure at a total cost of $1,969,000.

Although shoals marking the beginning of Clatsop Spit began to form along the north side of the jetty during its construction, a general improvement of depths over the bar was soon evident. By 1895 depths of 31 feet were

[1] Report of the Board of Engineers, dated 13 October 1882, printed as part of Senate Exec. Doc. 13, 47th Cong., 2d Sess.

available on an alinement that, under the influence of the jetty, had migrated to the north about 3 miles. These favorable conditions were only temporary, however, as the continued

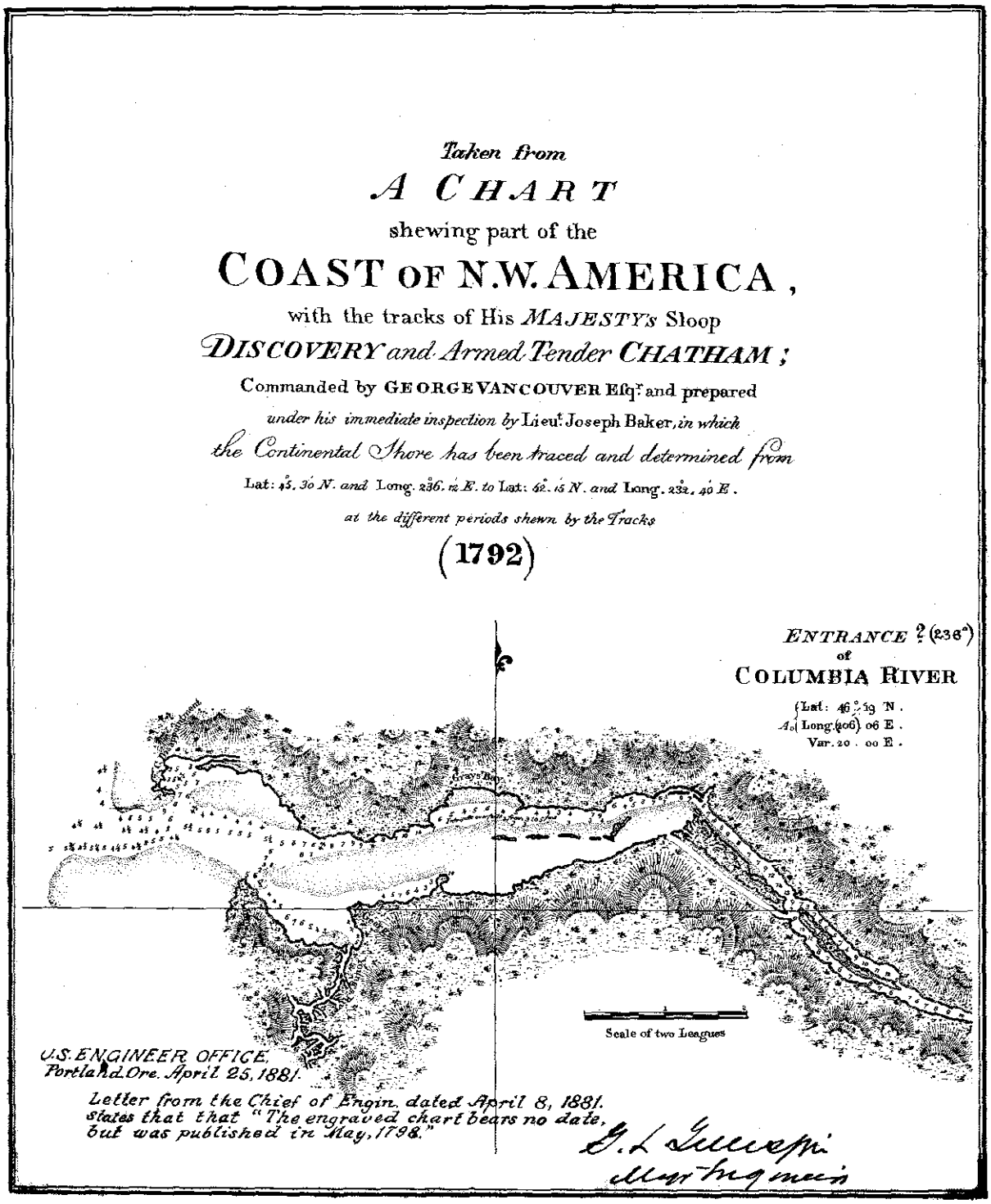



FIGURE 4. — Chart by Admiral Vancouver.

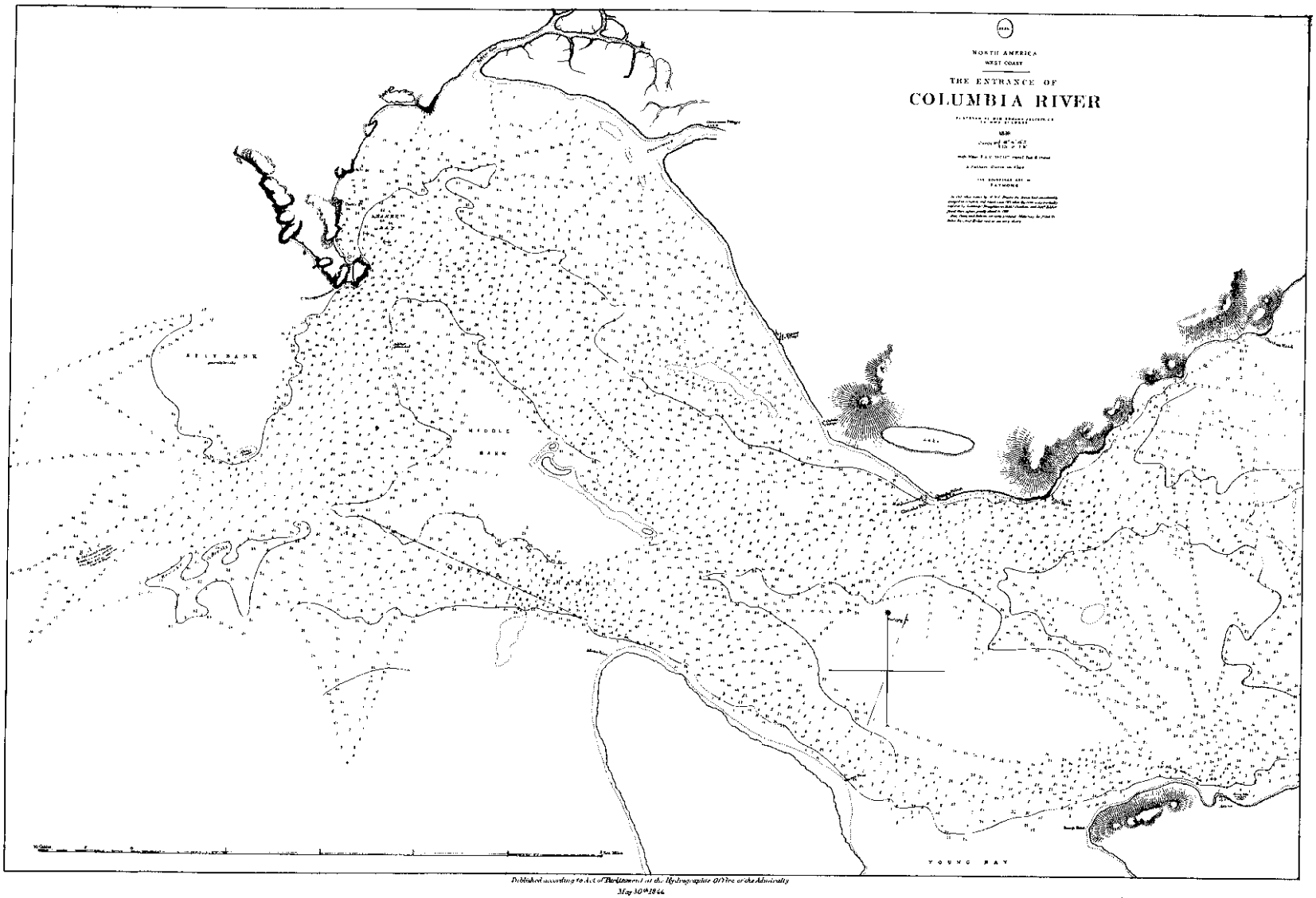



FIGURE 5. — Chart by Sir Edward Belcher.

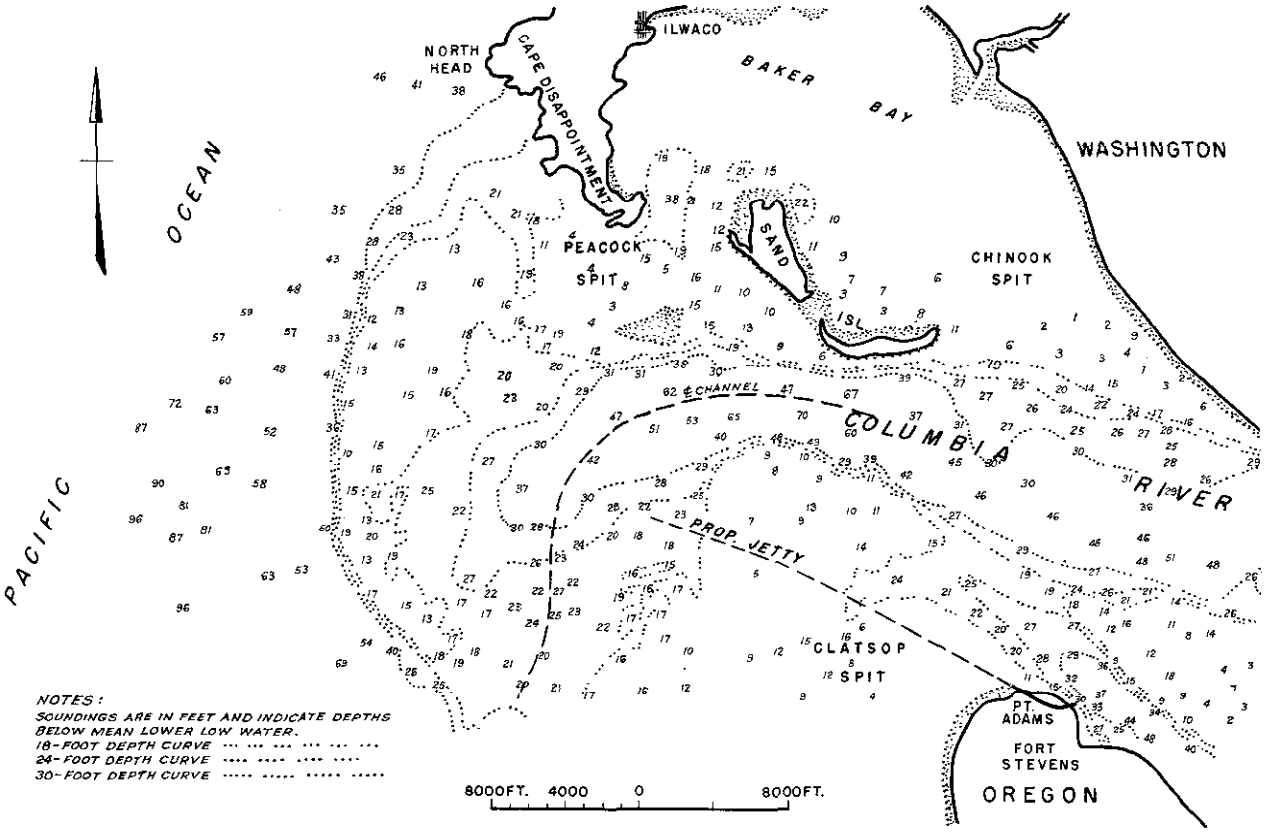



FIGURE 6. — Columbia entrance, 1885.

northward migration of the channel by 1898 reduced depths to 29 feet and by 1901 caused further deterioration of the entrance channel until in 1902, as shown in figure 7, there existed three entrance channels, each only about 22 feet deep. In the meantime Clatsop Spit had grown to form a hook-shaped island through the South Jetty at its midlength, which later extended to create a solid land connection with Point Adams.

### *40-Foot Entrance Project*

#### *Construction phase*

In view of these deteriorating conditions and the need for increased entrance depths to furnish a margin of safety to navigation, Congress, in the River and Harbor Act of March 3, 1905, adopted the recommendations of the board of engineers on project for improvement of the mouth of Columbia River[2] that provided for extension of the existing South Jetty and construction of a North Jetty and groins, in combination with dredging, to secure an entrance channel one-half mile wide and 40 feet deep. As the South Jetty was being extended seaward, a single entrance was restored and by 1911 a depth of 24 feet was again available across the ocean bar. As completed to a total length of 6.6 miles in August 1913, the South Jetty had an average top width of 25 feet and an elevation of plus 10 feet from the shore to the "knuckle" end of the original project, from whence the height was increased by stages to an elevation of plus 24 feet at the outer end.

Construction of the North Jetty began in 1913 and by the next year, when only about 1 mile in length, entrance depths had increased to 30 feet. As constructed, the North Jetty had a top width of 25 feet, sideslopes of 1 on 1½, and crest elevation of 28 to 32 feet above mean lower low water. Completion of the jetty to its full length of 2.4 miles in August 1917 was accompanied by further improvement of entrance depths to 40 feet through a narrow bar channel. In addition to 410,000 tons of stone used for rebuilding the outer end of the original South Jetty, a total of 4,427,000 tons of stone was placed in the extension of the South Jetty, at a total cost of $5,657,000. Nearly 3 million

[2] Report of the Board of Engineers, dated 24 January 1903, printed as part of appendix 20 of Annual Report, Chief of Engineers, U.S. Army, 1903, Pt. 3, v. 3.

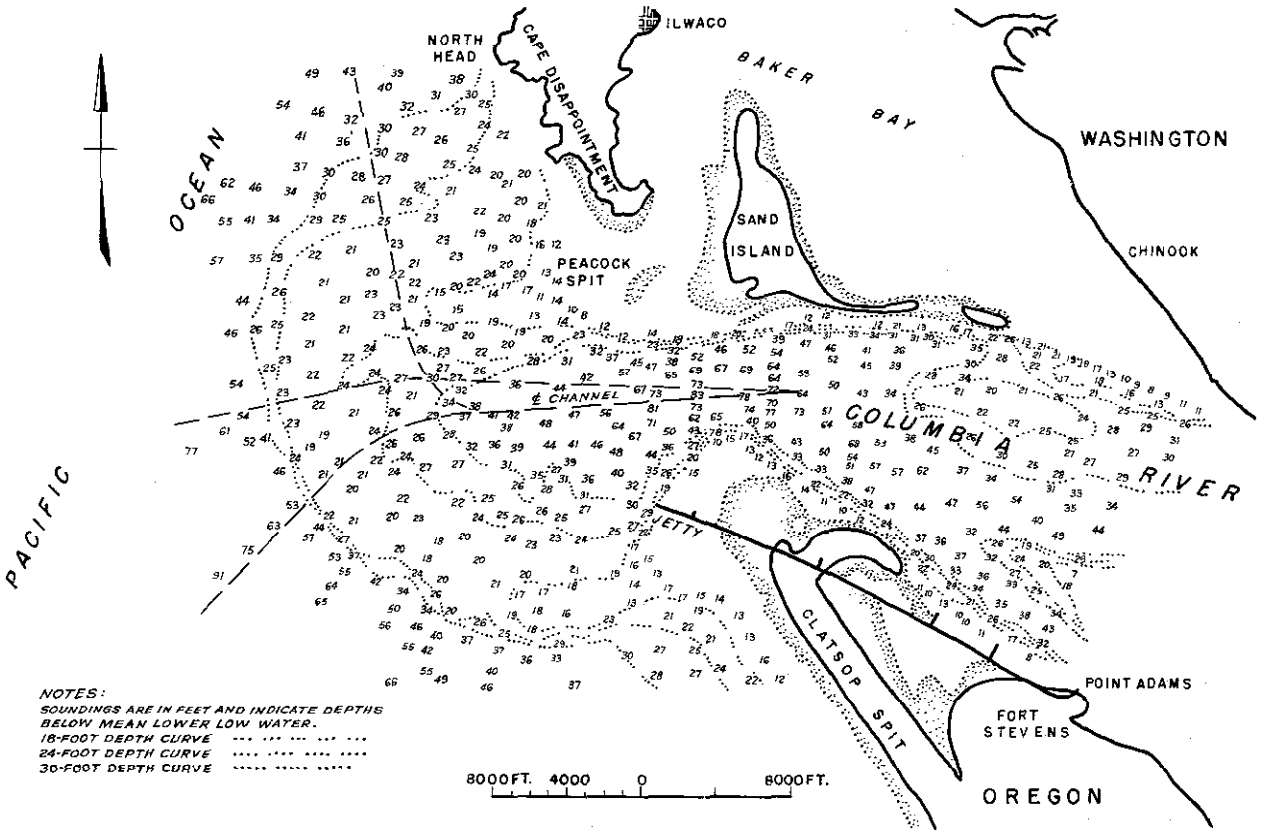



FIGURE 7. — Columbia entrance, 1902.

tons of stone were used in the construction of the North Jetty at a cost of $4,319,000.

*Adjustment phase*

Completion of the North Jetty, with its positive control on flows, marked the beginning of the achievement of the long-sought goal of securing dependable depths through the entrance for navigation. By 1920 the 40-foot depth contours had retreated considerably, providing an entrance channel about 1 mile wide and, during the next 5 years, further deepening occurred to make a minimum entrance depth of 45 feet available in 1925. By 1927 minimum depths of 47 feet were available over the bar, as shown in figure 8. For the first time, depths in excess of 40 feet then prevailed along the channel side of the North Jetty. However, concurrently with improvement of navigation conditions, shoals developed immediately north of the North Jetty and formed a solid land mass extending almost to its full length. Clatsop Spit continued to grow until by 1931 its westward and northward extension had reduced entrance depths to about 43 feet.

*Refinement phase*

Rehabilitation of the South Jetty was begun in 1931 and completed in 1936. Later in 1936, the outer 500 feet of the structure was impregnated with a hot asphaltic mix in an attempt to prevent disintegration at this point by heavy seas. This expedient did not prove successful, and a solid concrete terminal was constructed at the outer end in 1941. Further growth of Clatsop Spit was accompanied by movement of the navigation channel closer to the North Jetty and by development of increased depths along the channel side of the outer end of that structure. In an effort to prevent the continued movement of the channel to the north, four permeable dikes were constructed along the south shore of Sand Island and jetty A, extending southward from Cape Disappointment, was completed in 1939. In spite of this, the continued westward movement of Clatsop Spit was accompanied by removal of more and more of the shoal area protecting the North Jetty and some damage was caused by undercutting along the channel face of the structure.

In 1939 the North Jetty was rehabilitated and a concrete terminal block placed at its outer end. Continued advances of the Spit through 1951 coincided with definite movement of the 40-foot depth contour along the north side of the entrance channel to the east and south. In 1952, for the first time, an inner bar

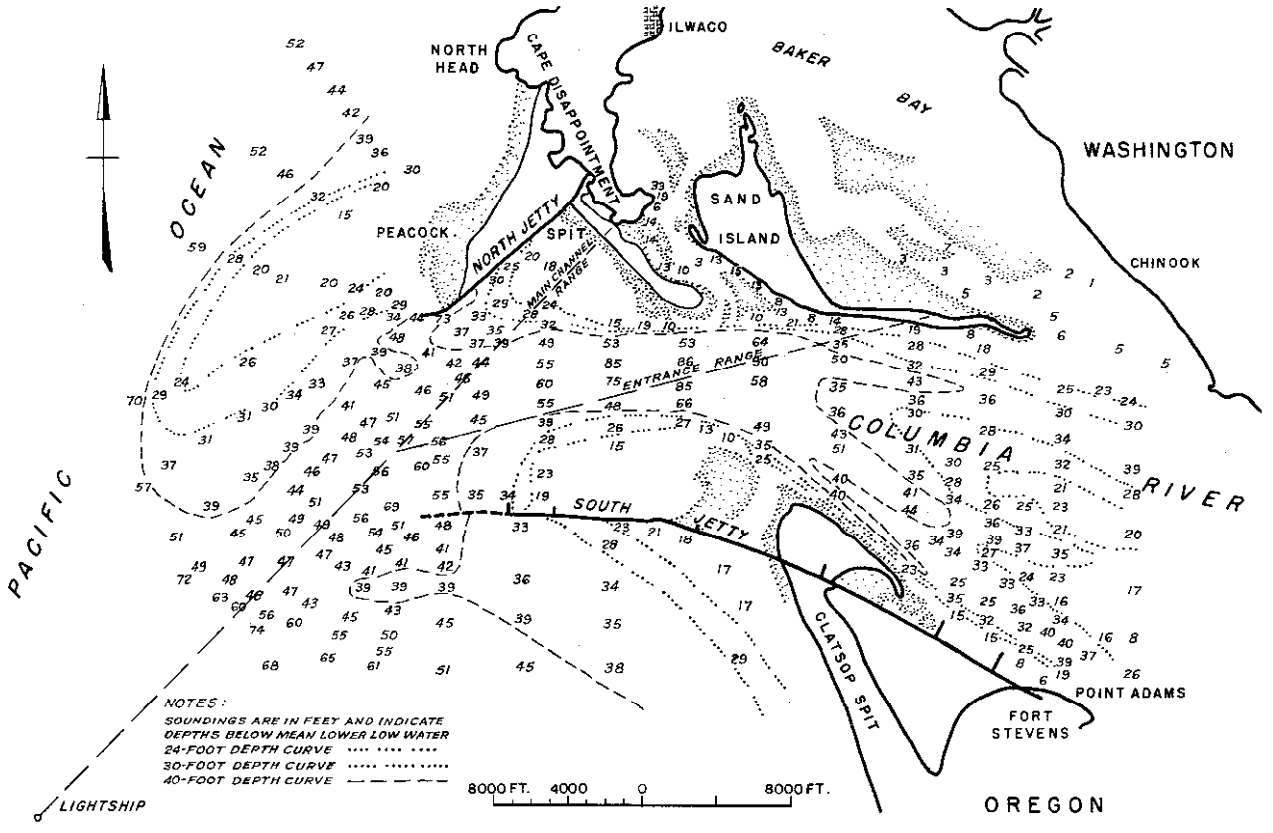



FIGURE 8. — Columbia entrance, 1927.

in extension of Clatsop Spit had formed between the jetties. Reconstruction of the South Jetty involved the placement of 4,288,000 tons of stone in the superstructure at a cost of $4,930,000. Rehabilitation of the North Jetty required the placement of 244,000 tons of stone and 234,000 tons of stone were used in the construction of jetty A, all at a total cost of $1,271,000.

### *Dredging*

As ample depths prevailed over the ocean bar, dredging under the 40-foot entrance project was confined to the face of Clatsop Spit. Amounts dredged since 1939 under that project are shown in table 2; the full project width was obtained only during the last 3 fiscal years listed.

TABLE 2.—*Dredging on ocean bar since 1939; 40-foot project*

| Fiscal year | Amount dredged | Fiscal year | Amount dredged |
|---|---|---|---|
| | *Cubic yards* | | *Cubic yards* |
| 1939 | 501,300 | 1949 | 1,023,800 |
| 1940 | 1,319,800 | 1950 | 923,200 |
| 1941–44 | 0 | 1951 | 1,000,300 |
| 1945 | 393,100 | 1952 | 1,267,400 |
| 1946 | 186,900 | 1953 | 2,796,800 |
| 1947 | 483,000 | 1954 | 2,141,700 |
| 1948 | 1,030,200 | 1955 | 1,749,300 |

## *48-Foot Entrance Project*

### *Project progress*

To meet the needs of modern ocean navigation, Congress, in 1954, modified the entrance project to provide for minimum depths of 48 feet throughout the half-mile channel width to be secured initially by dredging and, subsequently if experience so warranted, by construction of spur jetty B along the north shore (fig. 9), which also shows conditions prevailing in 1962. Dredges were successful in 1956, the initial year of dredging on the modified project, in securing by the end of the season in October depths of 48 feet over the full channel width, in spite of heavy amounts of erratic shoaling and scouring that accompanied the dredging operations.

Extremely heavy shoaling of the newly dredged channel during the following winter months, together with lack of sufficient dredging plant, resulted in the decision to concentrate the 1957 dredging effort to obtain project depths throughout a channel of only 1,500-foot width. Dredging in 1957 was also accompanied

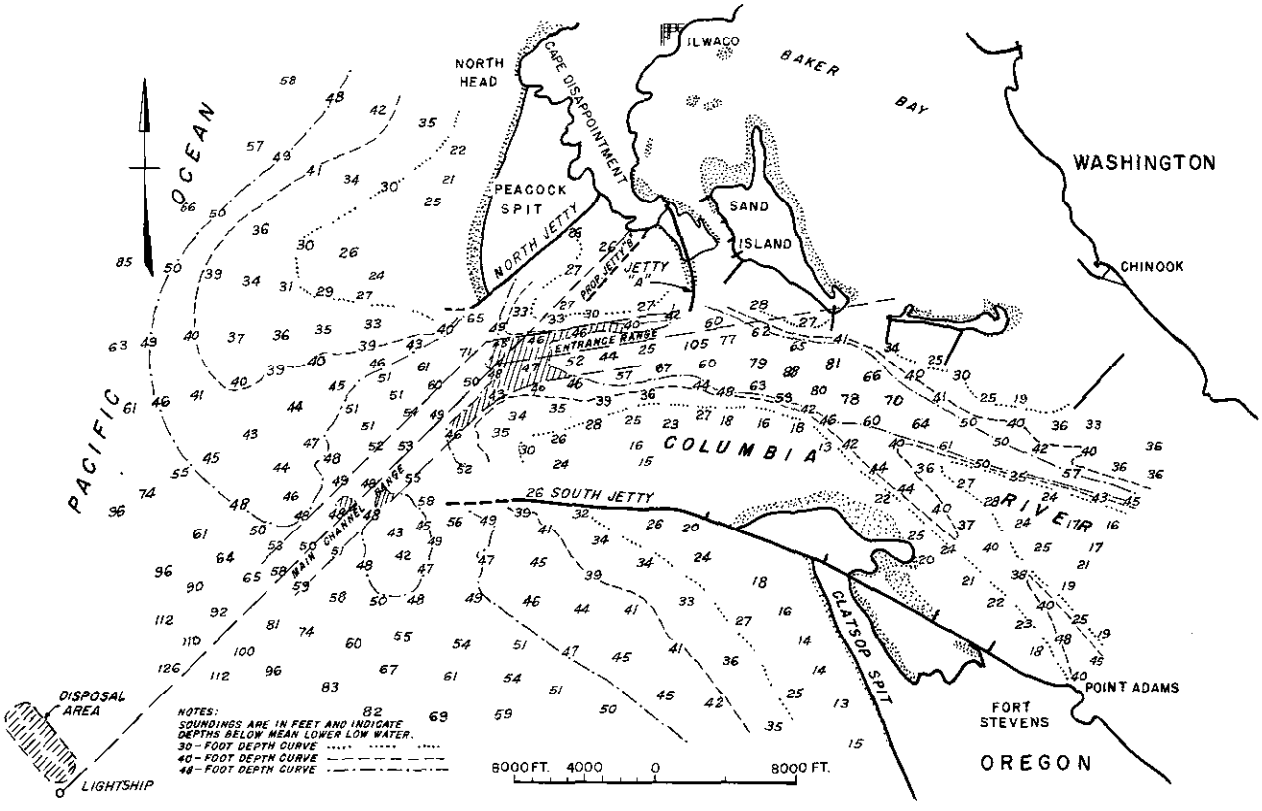



FIGURE 9. — Columbia entrance, 1962.

by erratic shoaling and scouring over the 1,500-foot channel and was similarly followed by proportionately heavy shoaling during the ensuing winter season. Approximately 75 percent of the material dredged in 1956 and 1957 was disposed in an offshore area approximately 1 mile south of the extreme end of the South Jetty (area A), as shown on figure 3, with the balance disposed in deep water along the entrance range (area B), adjacent to the inside of the North Jetty (area C), and upriver in the vicinity of Chinook Point (area D). Quantities dredged during these and subsequent years and the amounts of total apparent shoaling and scouring noted within the authorized channel width by frequent condition surveys are given in table 3.

TABLE 3.—*Dredging, shoaling and scour; 48-foot project*

| Calendar year | Quantity dredged | Amount of— | | Net | |
|---|---|---|---|---|---|
| | | Shoaling | Scouring | Shoal | Scour |
| | *Thousand cubic yards* | *Thousand cubic yards* | *Thousand cubic yards* | *Thousand cubic yards* | *Thousand Cubic yards* |
| 1956 | 14,436 | 8,492 | 6,263 | 2,229 | ..... |
| 1957 | 3,909 | 7,279 | 7,436 | ..... | 157 |
| Total, 1956 57 | 18,345 | 15,771 | 13,699 | 2,229 | 157 |
| 1958 | 2,603 | 3,216 | 5,942 | ..... | 2,726 |
| 1959 | 2,289 | 6,146 | 3,361 | 2,785 | ..... |
| 1960 | 2,286 | 5,008 | 4,048 | 960 | ..... |
| 1961 | 2,054 | 3,559 | 3,007 | 552 | ..... |
| Total, 1958–61 | 9,232 | 17,929 | 16,358 | 4,297 | 2,726 |

### *Modification of spoil disposal areas*

Tests of pilot equipment designed to measure subsurface current directions and velocities, conducted early in 1958 in preparation for the general prototype measurement program of the following year, revealed the fact that spoil disposed in areas A and C returned to the dredged entrance channel. As a consequence, disposal practices were subsequently modified so as to concentrate disposal of dredged materials in deeper water offshore (area B) and to eliminate as much as possible the disposal of these materials south of the South Jetty and near the North Jetty. Although since that date shoaling and scouring has generally followed the previously noted pattern, the magnitude of such occurrences has been reduced somewhat from that experienced in previous years as shown above.

*Rehabilitation of entrance structures*

Rehabilitation of jetty A in 1961 involved the placement of about 214,000 tons of stone at a cost of about $814,000. In that same year, 213,460 tons of stone were used in rehabilitation of the upper part of the South Jetty between the land connection with Point Adams and the "knuckle," at a cost of $2,328,000. Rehabilitation of the South Jetty seaward of the "knuckle," involving the placement of 268,000 tons of stone, is scheduled for completion in 1963.

## Past Concepts of Controlling Phenomena

### *Approach to Past Concepts*

Any discussion of past concepts of phenomena controlling the regimen of the lower Columbia Estuary and Entrance, as related to the constructed improvements, should recognize the state of knowledge prevailing at that time. Engineers had little but readily observable conditions and apparent reactions of surface currents to obstacles to guide their thinking, planning, and design, of improvements, since estuarine instrumentation as it is known today did not then exist. Hence it is small wonder that some works constructed under these circumstances failed to yield the desired results. The surprising thing is that many of these works did somehow accomplish their mission, perhaps not in the exact manner predicted, and have nevertheless survived to this day as monuments to the integrity and ingenuity of their planners. Accordingly, as we review these past accomplishments in the light of present knowledge, engineers of today should feel quite grateful and, indeed humble, for the advancement in technology that alone is responsible for their more enlightened position.

### *Past Experience*

With little to guide their efforts other than the observed conditions in the natural estuary and entrance, it was apparent to engineers charged with planning the Columbia Entrance structures that in order to provide a dependable channel, the waters of the entrance area should be confined to an area smaller than then existed in nature. To effect such an accomplishment, a South Jetty was constructed with its base at Point Adams which, while temporarily improving entrance conditions during the subsequent period of estuarine adjustment, did not fulfill the needs of then-existing navigation, as depths over the ocean bar had not been permanently improved by this expedient.

In view of this, engineers then studied the bankline geography of the lower estuary and concluded that further confinement of entrance flows could be effected by opposing jetties spaced about 2 miles apart at their ends, roughly the minimum distance between banklines in the lower estuary.[3] Accordingly, the South Jetty was extended and a North Jetty constructed. Although some improvement was effected by the extension of the South Jetty, this betterment of conditions was relatively minor and of a tentative nature only.

It was not until the North Jetty had been constructed that a truly dependable channel was made available in the Columbia River entrance. Although ever-changing controlling depths over the entrance bar were achieved in different phases of estuarine adjustment after the completion of the North Jetty, these depths were much greater than those that previously existed. As a result, by confinement of entrance flows to a relatively smaller area, engineers achieved a breakthrough in their efforts to control depths over the entrance bar.

### *Past Concept as Developed by Experience*

Thus the success gained by confinement of entrance flows supported the conclusion that, in order to achieve greater entrance depths, it was only necessary to confine flows to the extent required. Although recognizing that there existed a number of observed phenomena such as formation of shoals in unexpected places, which somehow did not conform to this concept, these engineers were primarily concerned, and justifiably so, with net results as measured by past accomplishments. As a consequence, the theory of confinement of flow was accepted by these engineers as a basic principle in the planning, design, and construction of estuarine improvements.

*1932 current measurements*

Notwithstanding the apparent validity of the theory of flow confinement, there prevailed some question whether this theory properly integrated all factors and forces in estuarine areas. Consequently, in 1932 the Corps of Engineers undertook a program of prototype measurements at five stations across a range located some 5 miles above the ends of the jetties.[4] More than 5,600 individual current meter observations were made at several depth levels at these stations during three different stages of upland discharge. Although no instruments were specifically employed for the purpose of determining subsurface current directions, the measurements taken were interpreted to indicate that mean ebbtide velocities were greater than floodtide velocities for all stages of the upper river and, during freshet stages, were

[3] See footnote 2.

[4] Mouth of Columbia River Current Survey, 1932. On file in the Office of the District Engineer, U.S. Army Engineer District, Portland, Oreg.

much stronger. Also at high river stages, ebb bottom velocities were much higher than flood bottom velocities for all percentages of time. At intermediate river stages, bottom velocities were lower than at high stage, with ebb bottom velocities predominating for about half the measurements. At low river stages, flood bottom velocities were generally predominant.

If the magnitude of ebb bottom velocities as compared to flood bottom velocities and their assumed greater transporting power are considered, it appeared evident at that time that the ebb flow was sufficient to maintain a channel between the jetties of at least 40 feet deep at mean lower low water and no further contraction of the entrance appeared necessary for satisfactory channel maintenance. Thus, the results of the 1932 current measurement program were interpreted as confirming the theory of flow confinement as applied to improvements in the Columbia Estuary and Entrance.

*Jetty A*

Notwithstanding the indication revealed by the 1932 current measurements that further contraction works would not be necessary to secure project depths, Clatsop Spit continued to advance to the northwest, crowding the entrance channel against the North Jetty and reducing the available depths in a channel, which was at that time assuming an undesirable alinement. To correct this condition jetty A was constructed in 1938 and 1939 to confine further flows in the entrance area. Although jetty A provides a degree of protection to vessels using the west Baker Bay channel against winter winds and waves, this structure, even in combination with the concurrently constructed Sand Island dikes, has had little beneficial influence on navigation depths or conditions through the lower Columbia Estuary or Entrance.

*Jetty B*

Although the failure of jetty A to produce the desired channel control could not be explained at that time, the validity of the theory of flow confinement was still accepted, although to a somewhat lesser degree, by engineers as late as the early 1950's when planning for the 48-foot entrance channel was initiated. Accordingly, as part of the 48-foot project, jetty B, to be located between the North Jetty and jetty A, was recommended and authorized in 1954 as a structure to supplement and enhance the confining influence of jetty A, if initial operations to secure the 48-foot entrance channel proved impracticable. Subsequent events and almost explosive expansion of knowledge regarding estuarine phenomena have cast considerable doubt on the value of the proposed jetty B as a regulating structure, and, consequently, no further action has been undertaken by the Corps of Engineers leading toward its construction.

## Modern Concepts of Controlling Phenomena

### *Approach to Modern Concepts*

Much credit should be given to the Corps of Engineers' Committee on Tidal Hydraulics and to the Waterways Experiment Station for their pioneering efforts and studies that have led to a broadening of engineering knowledge of character and magnitude of forces controlling the regimens of estuaries and their entrances. Early hydraulic model studies at the Waterways Experiment Station included a model of the Savannah Estuary and Entrance (Georgia) and, as was the practice at that time, fresh water was employed throughout the model system. Considerable difficulty was initially experienced in attempting to verify current measurements made in the model with those observed in the prototype.

All efforts to obtain accurate verification of the model failed until someone suggested that salt be added to the ocean portion of the model in the amount necessary to simulate salinity conditions in the prototype ocean. When this was done, verification was a relatively easy task, as the model, thus operated, completely and accurately reproduced observations of current direction and velocity noted in the prototype. This "breakthrough" in model techniques led to study and definition by many engineers and scientists of the important role that salinity intrusion plays in the development of forces controlling estuarine environment. These studies and companion investigations of related phenomena by others since 1948 have produced a new concept of estuarine phenomena, which has given the waterways engineer a more complete and accurate understanding to the end that it is no longer necessary to resort to a trial and error approach to estuarine improvements.

Although this new concept has completely revolutionized all past thinking and has given birth to the embryonic science of tidal hydraulics, investigations of all estuarine phenomena are being pursued at an ever-expanding rate under the favorable technological climate prevailing in the world today. Hence, new discoveries will be made and new techniques will be developed that will further assist the engineer of the future to solve estuarine problems more completely than is now possible.

### *Salinity Intrusion*

Although of extreme importance in the development of the estuarine regimen, the role that salinity intrusion plays in this development has to date been only broadly defined. Schultz

and Simmons [5] point out that engineers concerned with the solutions of problems in estuarine environments have become keenly aware in recent years that the amount of fresh water discharged into an estuary, and the degree to which it mixes therein with the salt water of the sea, are major factors in establishing the hydraulic and shoaling regimens of the estuary. The presence in estuaries of water of variable density causes marked differences in the magnitudes, distributions, and durations of the currents, as compared to those of a single density system.

As a result of the density difference between the heavier salt water at the seaward end of the estuary and the fresh water at the upstream end, each type of water tends to assume a rough wedge shape with the base of the wedge at the source. The interface of (or line of demarcation between) the salt and fresh water may vary from well defined to almost obscure, depending on the degree of mixing of the salt and fresh water in any given estuary. Where mixing is slight the transition from fresh to salt water is well defined and occurs within a small percentage of the channel depth. On the other hand, where the mixing is appreciable no definite interface of the salt and fresh water exists except in isolated instances.

For convenience, the degree of mixing may be classed into three broad categories of highly stratified, partly mixed, and well mixed, with the transition from one type of mixing to another being gradual instead of well defined. The most significant effect of salinity intrusion, thus defined, is the creation of density currents in estuaries that cause the bottom flood currents to predominate over the bottom ebb currents by increasing the velocity and duration of the former and decreasing the velocity and duration of the latter. The resulting net upstream movement of bottom currents within the saline region of the estuary constitutes an effective trap for sediments on and near the bottom, preventing their movement to the sea and causing the bottom to be shoaled and unstable. From their studies Schultz and Simmons arrived at the following general conclusions:

1. The degree of mixing of salt and fresh water in estuaries plays an important role in the establishment of their hydraulic regimens. As the shoaling regimens of some estuaries are related directly to their hydraulic regimens, it follows that the degree of mixing also plays an important role in establishing their shoaling regimens.

2. Because of incomplete mixing of salt and fresh water in estuaries, the predominance of flow in the bottom strata is almost always upstream while that in the surface strata is downstream; the degree of such predominance is dependent on the degree of mixing, being most prominent in the highly stratified estuary and least prominent in the well-mixed type.

3. Changes in upland discharge, tidal prism, and physical configurations of estuaries will frequently change the degree of mixing of salt and fresh water therein and thus affect such important features as the vertical distribution of current velocities, the direction and degree of flow predominance, the amount of shoaling, and the location of major shoal areas.

4. As lightweight sediments are supplied to estuaries principally through the medium of upland discharge, it follows that such discharge plays a dual role in estuarine sedimentation.

5. The magnitude of changes in upland discharge into estuaries usually far exceeds that of changes in tidal prism of physical configuration. For this reason and because upland discharge is of primary importance, it follows that major changes in upland discharge should be accomplished only after consideration of all probable effects.

### *Continuing Research on Salinity Intrusion*

An analytical investigation of salinity intrusion and related phenomena was initiated by the Corps of Engineers Committee on Tidal Hydraulics in January 1954. This general investigation was designed to determine for conditions of open channels subject to salt water intrusion and tidal oscillations from the sea the following four aspects of the phenomena: (1) The extent of salinity intrusion and the mean salinity distribution, (2) the vertical mixing of fresh and salt water and the resulting vertical salinity distribution, (3) the vertical distribution of current velocities as affected by salinity distribution, and (4) the movement and deposition of sediments as affected by density-current phenomena. On the basis of studies undertaken on phase (1) of this general investigation, Ippen and Harleman [6] indicate that it is possible to make quantitative predictions regarding salinity intrusions in partially or well-mixed estuaries of essentially uniform cross section without prior knowledge of any existing salinity conditions. Further, the factors that cause changes in salinity intrusions have, for the first time, been quantitatively evaluated. Thus by means of intrusion equations, the effect of changes in the fresh water discharge, channel depth, etc., can be predicted. Of even greater importance is

[5] SIMMONS, H. B., and SCHULTZ, E. A. FRESH WATER-SALT WATER DENSITY CURRENTS, A MAJOR CAUSE OF SILTATION IN ESTUARIES. Corps of Engineers Committee on Tidal Hydraulics, Tech. Bul. 2. April 1957.

[6] IPPEN, A. T., and HARLEMAN, D. R. F. ONE-DIMENSIONAL ANALYSIS OF SALINITY INTRUSION IN ESTUARIES. Corps of Engineers Committee on Tidal Hydraulics, Tech. Bul. 5. June 1961.

the possibility that the diffusion and tidal parameters developed in this study will have important significance in understanding the intrusion mechanics of estuaries of nonuniform geometry. Analyses will be made of phases (2), (3), and (4) of this investigation in the light of findings of studies of phase (1) as may be applicable.

### *Shoaling Processes*

Much remains to be learned regarding the processes involved in the formation and stabilization of shoals in estuarine channels. In 1957 the Committee on Tidal Hydraulics launched a broad program of studies considered necessary to develop essential knowledge of these processes. The different phases of work contemplated under this program are summarized as follows.

#### *Basic laws for movement and deposition of muddy sediments*

Since most estuarine shoals in the United States are composed of muds rather than sands, a contract has been made with the University of California to study the rheological properties of consolidating sediments. Natural sediments from a number of estuaries are being furnished to the university as a basis for the studies.

#### *Effects of a repetitive scour and deposition on sedimentation*

As it appears that successive scour and deposition of sediments may play an important role in shoaling processes, early flume tests to determine the significance of these factors are planned.

#### *Techniques for radioactive tracing of sediment action*

These techniques offer a great potential in the development of knowledge in shoaling processes, making it mandatory that their use be encouraged in connection with specific investigations and that consultation and advice be extended to those engaged in this work. Field tests planned by the district engineers of Galveston and Norfolk will be closely followed.

#### *Development of in-place turbidity meter*

Because of the great areas embraced by most estuaries and the constantly changing conditions of tide and fresh-water inflow, it is apparent that complete and simultaneous coverage of such systems for measurements of suspended sediment concentrations, with only approximately accurate results, will yield much more valuable information than will long-term coverage and highly accurate results. Available evidence indicates that an instrument operating on the basis of light extinction as a measure of turbidity may fulfill these needs. Development of such an instrument is under way at Johns Hopkins University.

#### *Flocculation*

Flocculation of suspended and dissolved solids plays an important role in shoaling processes, as sea water is an efficient flocculating agent, but little work of scientific value has yet been accomplished to evaluate its significance. A comprehensive literature survey of this subject has been made and a report thereon completed. Recommendations for further work in this field are now under review, and it is probable that further research will shortly be initiated.

#### *Stabilization of deposits*

The physical and chemical changes that occur in sediments after deposition appear important to an overall appraisal of the matter of shoaling processes. A literature survey of this subject has been completed and additional research is forthcoming.

#### *Analysis and correlation of prototype data*

Proper analysis and correlation of existing prototype hydraulic, hydrographic, and other data will reveal certain relations among estuaries that should lead to a better understanding of the overall subject of shoaling processes. Data on important estuaries are now being assembled and analyzed, and it is anticipated that direct comparisons and correlations should reveal definite characteristics.

#### *Shoaling in slips and tributary channels*

It is anticipated that test facilities for general investigations of hydraulic and shoaling phenomena in slips and tributary channels will shortly be constructed as additions to existing hydraulic models at the Waterways Experiment Station and tests initiated to determine the factors involved.

#### *Classification of sediments*

Proper classification of the sediments that contribute to shoaling of the different estuaries in the United States is of importance from the viewpoint of a comprehensive understanding of shoaling processes. This effort is being carried out concurrently with the analysis and correlation of prototype data previously described.

### *Littoral Processes*

Waves of all character and magnitude breaking on a coastline generate movements of beach materials in the alongshore component direction of the generating waves. These movements establish the littoral regime of the shoreline, and the intensity of these movements determine the quantity of material or littoral drift moving past a point on the shoreline. As, during the course of an extended period of time, waves attack a shore generally from one predominant direction, so too will the littoral movements of material passing a point on that shore move in

one predominant direction parallel to the coast. Just as waves also attack this shore from other directions over an extended period of time, though less predominately from those directions, so too will there be littoral movements past the point that may be in a direction opposite that of the predominant littoral drift.

Thus, from a point along a shoreline, such as the entrance to an estuary, the total quantity of littoral material in movement is the sum of all littoral movements in both the up-coast and down-coast directions. Both of these material movements contribute to shoaling in major estuarine entrances. They also affect the delicate balance between erosion and accretion of the shoreline. In instances where littoral drift is intercepted by coastal structures such as groins or jetties, the shore updrift of these structures will grow seaward to a point, fed by the cessation of the littoral movement, while the shore downdrift from the structures will recede, being starved by the intercepted littoral drift.

## Prototype Measurement Program of 1959

### *Background*

In view of difficulties experienced in maintenance of the 40-foot entrance project resulting from the continual encroachment of Clatsop Spit shoals on the adopted channel alinement, the problem at the Columbia River Entrance was re-approached in 1956 in the light of new concepts and expanding knowledge of tidal hydraulic phenomena concurrently with initiation of work on the newly authorized 48-foot entrance project. In making this new approach, the advice and guidance of the Committee on Tidal Hydraulics formed the basis for an intensive investigation of several aspects of the problem.

As the initial step in this investigation, the committee recommended that a program of prototype measurements of current velocity and direction, as well as salinity, be undertaken to define broadly the density current regime established by environmental river, ocean, and hydrographic conditions prevailing in the lower estuary. With effects of density currents thus defined it was the view of the committee that any additional structural measures designed to reduce shoaling in the entrance area, such as jetty B, could be properly analyzed and evaluated in a hydraulic model study.

### *General Scope of Program*

In order to achieve the above goal, it was necessary that the program provide data sufficient to meet the design and verification needs of a hydraulic model study. To this end the program required 3 cycles of measurements to obtain data for conditions of low, normal, and high river discharge. Each cycle was to include observations of current direction and velocity for a continuous full tidal period of about 25 hours at 23 stations located along 7 ranges across the lower 52 miles of the estuary (fig. 10). At each station current velocity and direction measurements were taken at 30-minute intervals at the surface and near the bottom and at the intervening quarter-points of depth.

Initially, simultaneous observations were made at one station on each range for a continuous period of about 25 hours to establish the relationship between the ranges. This was followed by simultaneous observations at all stations on each range until all observations had been obtained. Salinities and temperatures were also observed throughout the vertical at each station concurrently with the measurements of currents and directions. This program, due to its magnitude and the complexities involved in instrumentation, administration, and operation, not to mention the trying conditions of weather and heavy seas, represented a monumental task. A total of approximately 26,000 observations were taken during the 3 cycles of measurements. However, in spite of the magnitude of work involved and the difficulties experienced, reliable measurements were obtained.

### *Program Instrumentation*

#### *Velocity–azimuth–depth assembly*

Equipment being tested by the U.S. Geological Survey to measure tidal flow was used as a guide in the prototype measurement program for the development of an instrument designed to measure subsurface current velocity and direction, as well as depth of the instrument above the bottom. This instrument consisted of a transducer for a Raytheon Fathometer, mounted in the bottom of 140-pound streamline brass weight with fins, suspended on the bottom of a Price current meter hanger bar. Also mounted in the weight was a Magnesyn compass transmitter that monitored on a remote panel the direction in which the assembly was facing as deflected by the current. A seven-conductor, pancake swivel was mounted between the current meter hanger bar and the seven-conductor, ¼ inch diameter, suspension cable. The assembly as finally developed and used in the program is shown in figure 11.

#### *Salinity–temperature assembly*

Although consideration was given to the advisability of taking water samples at the five elevations of each station and determining salinity by laboratory tritration procedures, it was felt that, in the interest of reducing the workload, a less exact method of measuring salinity would suffice for the needs of the data collection and analysis program. It was also considered

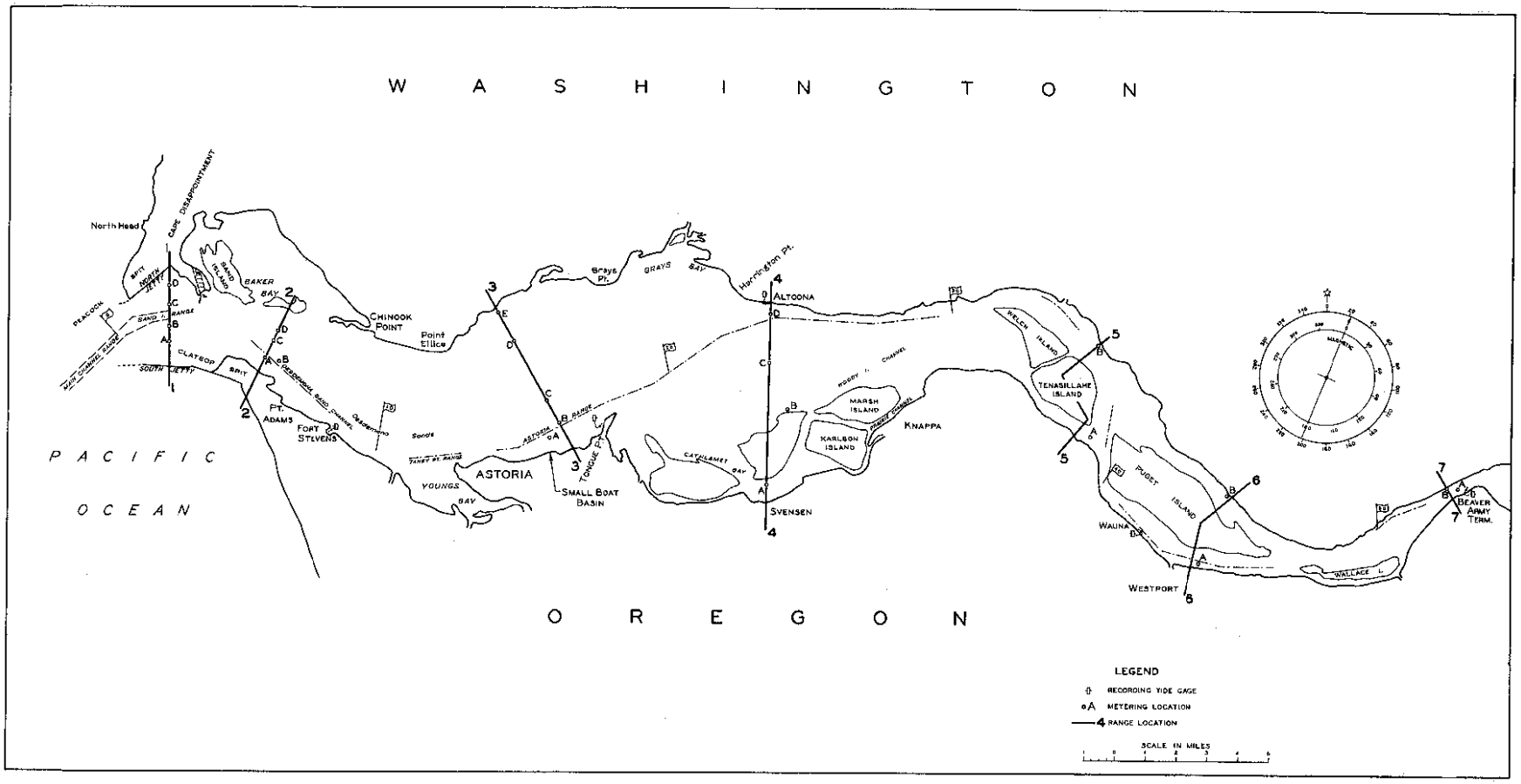



FIGURE 10. — Prototype measurement ranges and stations.

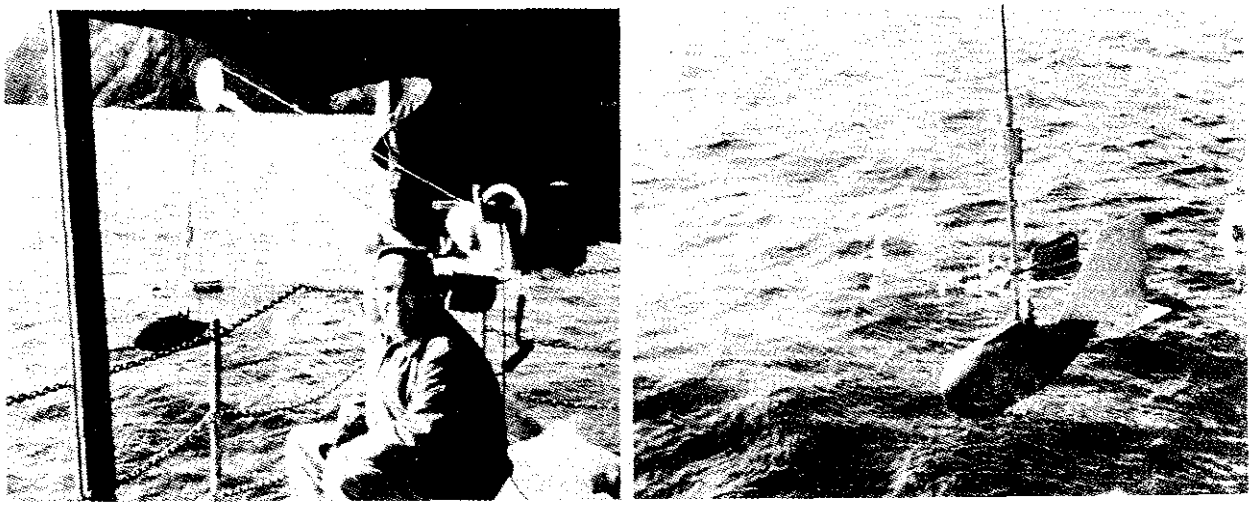


FIGURE 11. — Velocity — Azimuth — depth assembly.

advisable to obtain concurrent measurements of water temperature. To meet these requirements a conductivity cell with remote-reading resistance indicator was used to measure salinity, and a temperature cell, together with Wheatstone-balanced bridge, was used to obtain temperatures. The conductivity and temperature cells were mounted on a bracket above a 140-pound brass weight, and short leads from the cells were connected to leads from the suspension cable inside a short stainless steel hanger that was clamped to the lower end of the suspension cable, as shown in figure 12. The soldered connections were embedded in a hard, waterproof plastic filler. A seven-conductor suspension cable identical to that used for the velocity-azimuth-depth assembly was used.

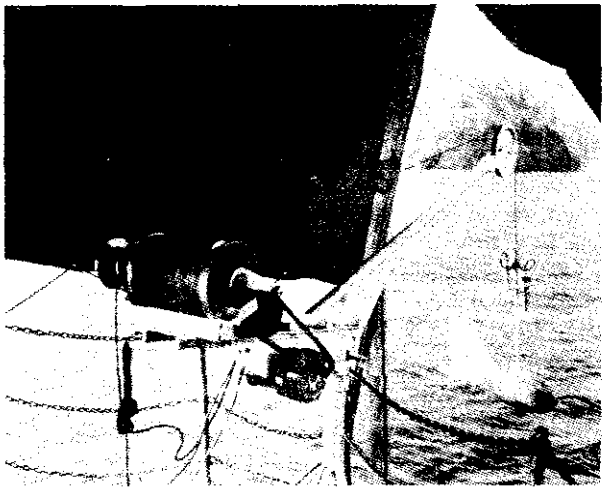


FIGURE 12. — Salinity — temperature assembly.

*Pillar crane*

As the metering and salinity-temperature assemblies were to be used over the sides of small boats, a special crane was designed and fabricated to raise the bottom of the assembly weights 30 inches above the boat deck and far enough away from the side of the vessel so that the danger of striking the assembly instruments against the boat hull would be minimal. Pipe was used for the main structural members of the pillar crane shown in figure 13.

*Other equipment*

In addition to 12-volt batteries for each metering vessel, one complete set of current metering and salinity-temperature assemblies and a pillar crane were kept in readiness for use. Automatic tide gage recorders were maintained at strategic points in the lower estuary to record tidal stage variations during the data measurement periods.

## *Program Administration*

*General*

Administrative tasks associated with the program included the procurement of necessary equipment, rental of vessels, training of equipment operators, and coordination of program details with fishery and navigation interests.

*Procurement of equipment*

Assemblies and pillar cranes previously described were obtained by contract with local manufacturers. Small items, including repair parts and flashing lights for buoys, as well as battery chargers, were obtained by direct purchase. Storage batteries were rented for each of the measurement periods. All equipment was assembled at the Government moorings in Port-

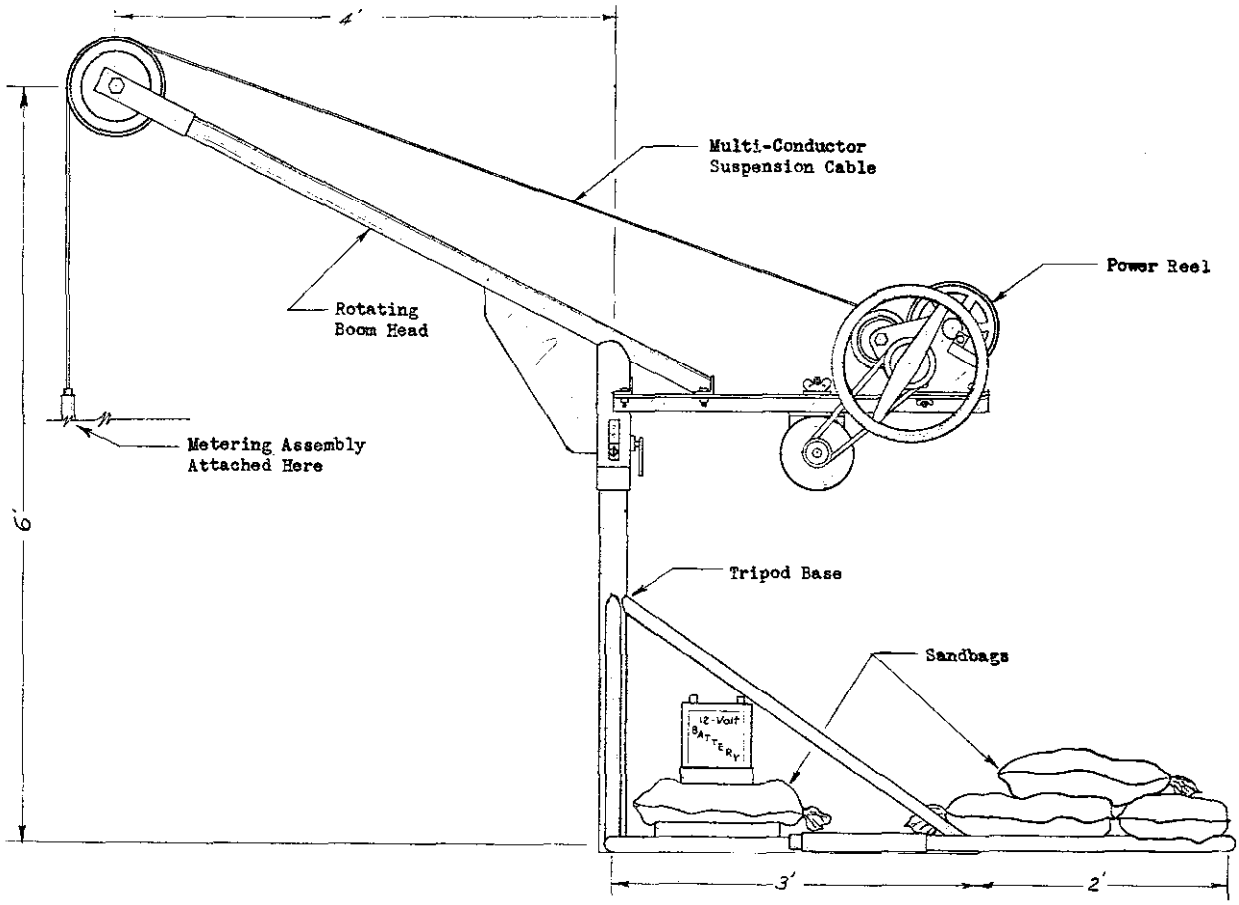



FIGURE 13. — Pillar crane.

land prior to movement by truck to the small-boat basin at Astoria, which served as the field headquarters for the program. Specifications were prepared to meet the particular requirements of the control boat, the master metering boat, and four metering boats. These vessels were hired through normal contractual procedures on an hourly rental basis.

*Operator training*

Professional engineers and engineer technicians from the engineering division of Corps of Engineers, Portland District, were selected as equipment operators and indoctrinated by means of a special training program. Each operator was furnished a 20-page brochure describing the purpose of the prototype measurement program and related administrative and technical details, prior to a general orientation session to insure a thorough understanding by all of their specific duties. This was followed by a 4-hour session in which each operator was taught how to assemble, operate, and maintain each item of measuring equipment.

*Coordination*

Discussions were held with the United States Coast Guard to determine the type of buoy most desirable to mark the location of each measurement station in the lower estuary, and to insure the availability of such buoys and needed appurtenant equipment. At the request of the Columbia River Fisherman's Protective Association, arrangements were made to forego the use of buoys at stations C and D on Range 2, to eliminate interference with fishermen's nets on established drifts and to remove and replace buoys at stations D and E on Range 3 during and after the August fish runs.

Some minor shifting of station locations in the back channels was made at the suggestions of tug and barge operators to avoid accidental removal of buoys by log tows. The Columbia River Bar Pilots Association was notified of proposed activities, and the Coast Guard was furnished the latitude and longitude of each buoy for publishing in its notice to mariners well in advance of the placement of buoys.

### *Program Operation*

Field operations, with the attendant requirement for extreme accuracy and timing of observations, occasionally in the face of adverse weather and wave conditions, represented the most difficult phase of the measurement program. Initial operations involved the placement of first class can buoys by the Coast Guard early in April 1959 to mark the location of the measurement stations.

#### *Distribution of vessels*

Four equipment operators were assigned to each of the six vessels providing two 2-man operator crews for alternate 12-hour shifts throughout each 8-day measurement cycle. The control boat, the *Sujan,* in addition to representing an immediate source of supply of spare equipment and the means for supervision and control of operations, was also used to ferry operator crews to and from shore bases established at Fort Stevens, Astoria, Svenson, Knappa, and Westport for the purpose of reducing the time required for changing shifts. Station A, Range 3, the master-metering station, was occupied by the metering boat, *Catana,* continuously throughout each 8-day measurement cycle. The four other metering boats, the *Morning Star, Rosie, Mary K,* and *My Boat,* were phased among the remaining 22 stations to obtain combinations of 25-hour measurements at each station as shown in table 4.

TABLE 4.—*Distribution of metering boats*

| 25-hour metering period | Boat and station occupied | | | | |
|---|---|---|---|---|---|
| | CATANA | MORNING STAR | ROSIE | MY BOAT | MARY K |
| 1 | 3–A | 3–C | 3–B | 3–D | 3–E |
| 2 | 3–A | 1–A | 1–B | 1–C | 1–D |
| 3 | 3–A | 2–A | 2–B | 2–C | 2–D |
| 4 | 3–A | 4–A | 4–B | 4–C | 4–D |
| 5 | 3–A | 5–B | 5–A | 6–A | 6–B |
| 6 | 3–A | 5–B | 6–A | 7–A | 7–B |
| 7 | 3–A | 4–D | 2–C | 5–B | 7–B |

#### *Observations*

Measurements of current velocity and direction, salinity, and temperature at five different levels were made every half hour at each station. These levels were located 3 feet below the water surface, 2 feet above the bottom, and at the intervening ¼-, ½-, and ¾-depth levels.

#### *Cycles of measurement*

The initial cycle of measurement was undertaken from May 5 through May 13, 1959, a period of normal river flow, when the discharge ranged from 365,000 to 404,000 c.f.s. in the lower estuary. The second cycle of measurement was accomplished between June 16 and June 24, 1959, a period of high river flow, when the discharge ranged between 532,000 and 577,000 c.f.s. The final cycle of measurement was made between September 15 and September 23, 1959, a period of low river flow, when the discharge ranged between 153,000 and 214,000 c.f.s.

### *Analysis of Prototype Data*

#### *General*

An office analysis of observed prototype data was undertaken by the district engineer, Portland, and a record of all observations obtained and his analyses of these data are contained in a four-volume manuscript report [7] that provides the most complete source of observed data relating to the current, salinity, and temperature regimes of the lower estuary and entrance. Due to weather and sea conditions, which precluded the undertaking of a program of measurements near the point where Clatsop Shoals encroach upon the entrance channel, all observations were made upstream from this problem area.

While the measurement program did not reveal conditions prevailing in that area, it did show the vagaries of currents and other measured phenomena throughout other parts of the lower estuary and, thus, provided data for competent verification of a hydraulic model of the entrance and lower estuary. In such a model, physical phenomena contributing to shoaling in the problem area are now in the process of being observed, understood, and analyzed to the extent feasible by tests of possible structural expedients.

#### *Flow predominance*

In analyzing observations of current direction and velocity, plots similar to that shown in figure 14 were prepared for each station. Changes in current direction between flood and ebb and velocity were noted at each depth level throughout each observation cycle of approximately 25 hours' duration. From these plots it was possible, by computing the subtended areas above and below the zero velocity line, representing the volumes of flood and ebb flows, to determine the predominant direction of flow and the degree of predominance of such flows at those levels.

Figure 15 shows changes in flow predominance noted at all stations and levels during each river discharge cycle of measurement on Range 1, and figure 16 shows similar information obtained from observations taken on Range 2. From similar data developed along each measurement range it was possible to obtain flow predominance profiles along the northerly

[7] DISTRICT ENGINEER, U.S. ARMY ENGINEER DISTRICT, PORTLAND. INTERIM REPORT ON 1959 CURRENT MEASUREMENT PROGRAM, COLUMBIA RIVER AT MOUTH, OREGON AND WASHINGTON. 1960.

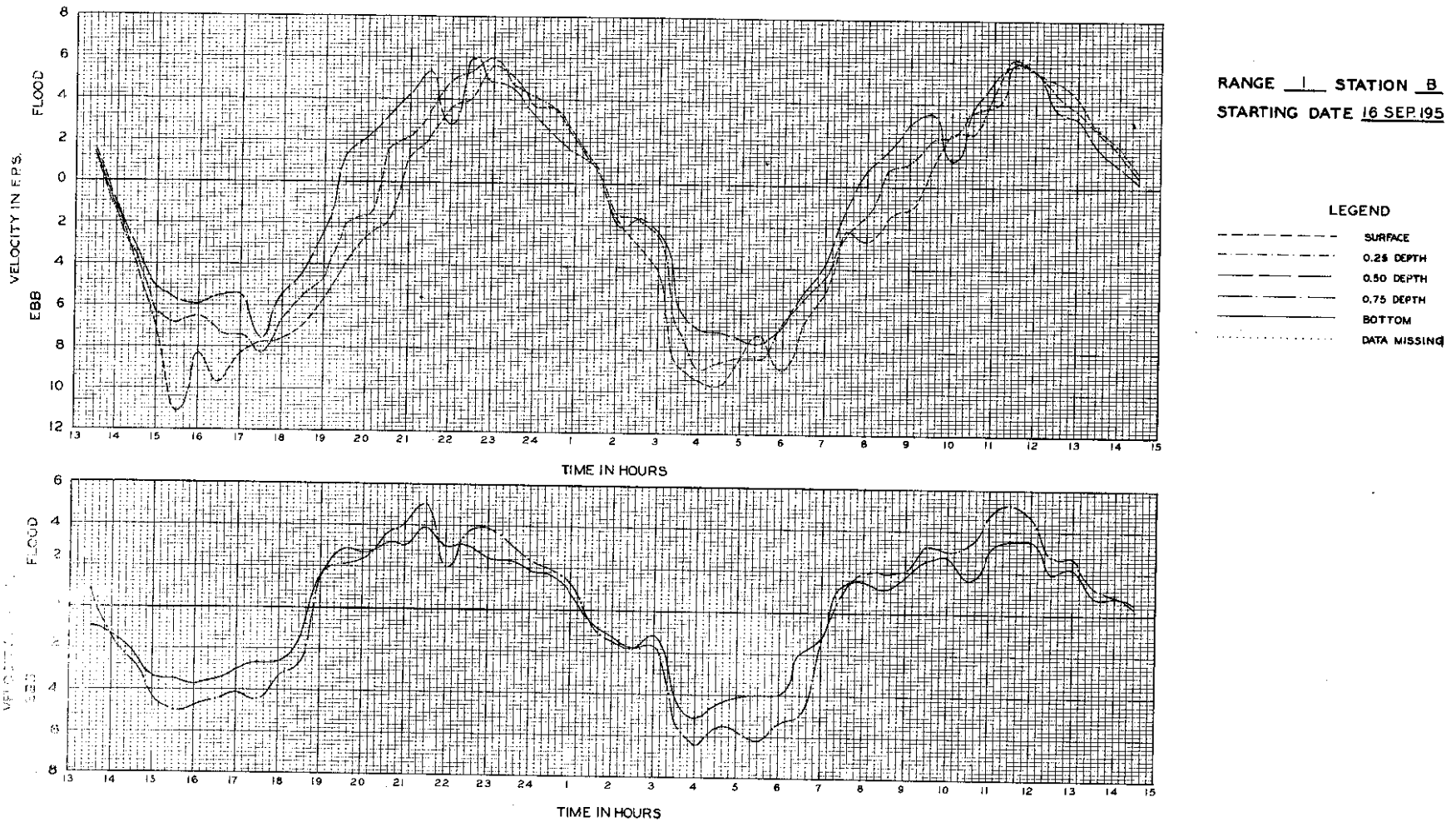



FIGURE 14. — Typical velocity — time — direction curve.

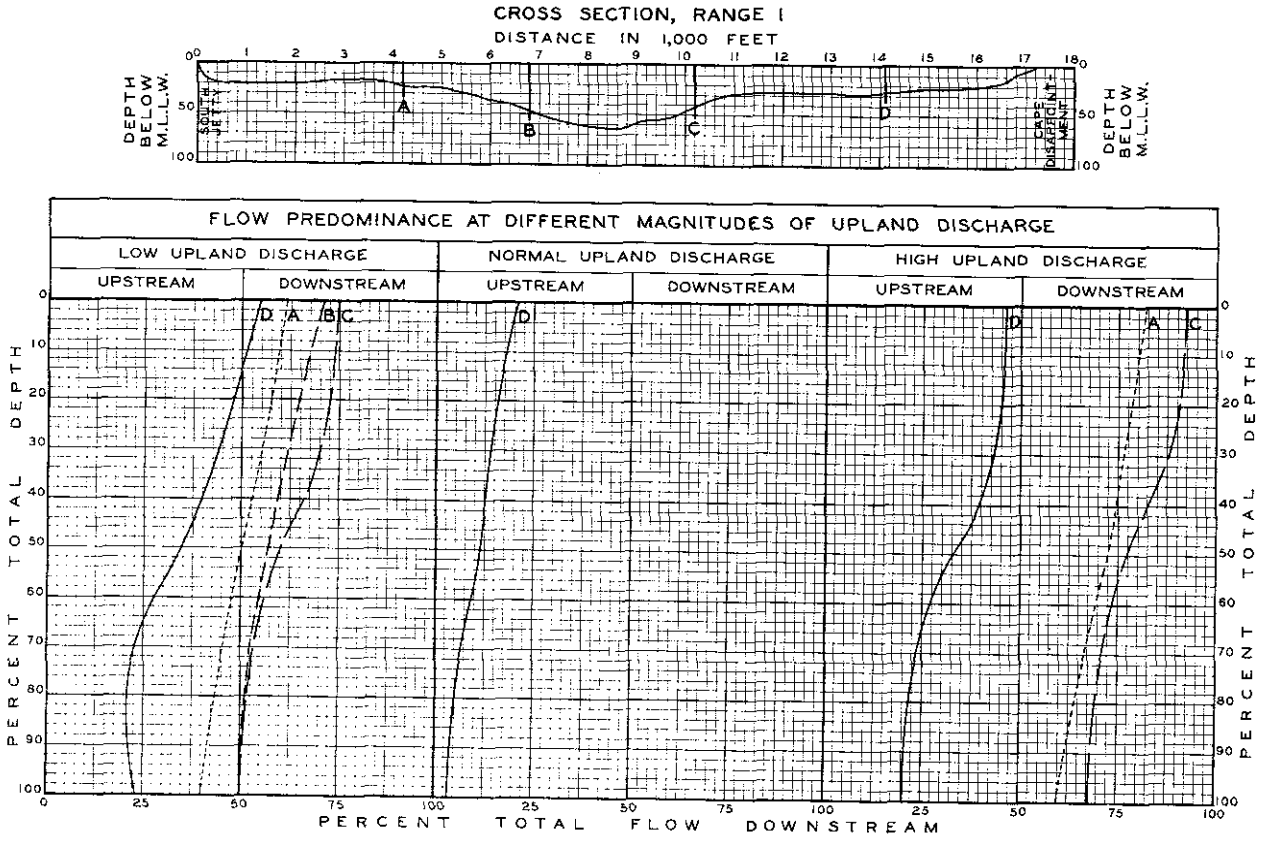



FIGURE 15. — Flow predominance, range 1.

part of the lower estuary, as shown in figure 17, as well as along the adopted ship channel alinement shown in figure 18.

***Salinity intrusion***

Measurements of salinity, taken concurrently with those of velocity and direction, revealed significant intrusion by salinity within the lower 20 miles of the estuary, shown in figure 19.

***Temperature***

Although a general trend of correlation between salinity and temperature was noted, this correlation was not conclusive, owing possibly to the time lag between the taking of the salinity and the temperature observations.

### *Findings of the Program*

The prototype measurement program confirmed the view that the Columbia Estuary, like any other major estuary, is influenced by tidal forces to an extent governed by geography and the complex interaction of these forces with density, littoral, and perhaps other forces significant to the various problems experienced in the estuary. The program definitely identified the normal estuarine characteristic of upstream flow predominance along the bottom levels of the lower estuary in contrast to the downstream predominance noted in the upper levels. This revealed existence of a generally defined pattern of circulation within that region, which has governed the deposition of sediments forming the shoals obstructive to navigation. It has also given rise to the view that the estuary now acts as a vast sediment trap, in which the phenomenon of density currents preclude, except during rare periods of extreme upland discharge, the movement of bottom sediments to the sea.

In consideration of these findings, the Committee on Tidal Hydraulics concluded that while it would be possible to obtain, by extensive and costly analytical analyses of observed data together with collection of additional data, an element of success in determining the cause or causes of the existing shoaling and other problems, such analyses could not accurately predict the effects of improvements works.

Analyses of anticipated future problems would be even more difficult, if not impossible. Also, the effects of regulatory works that are

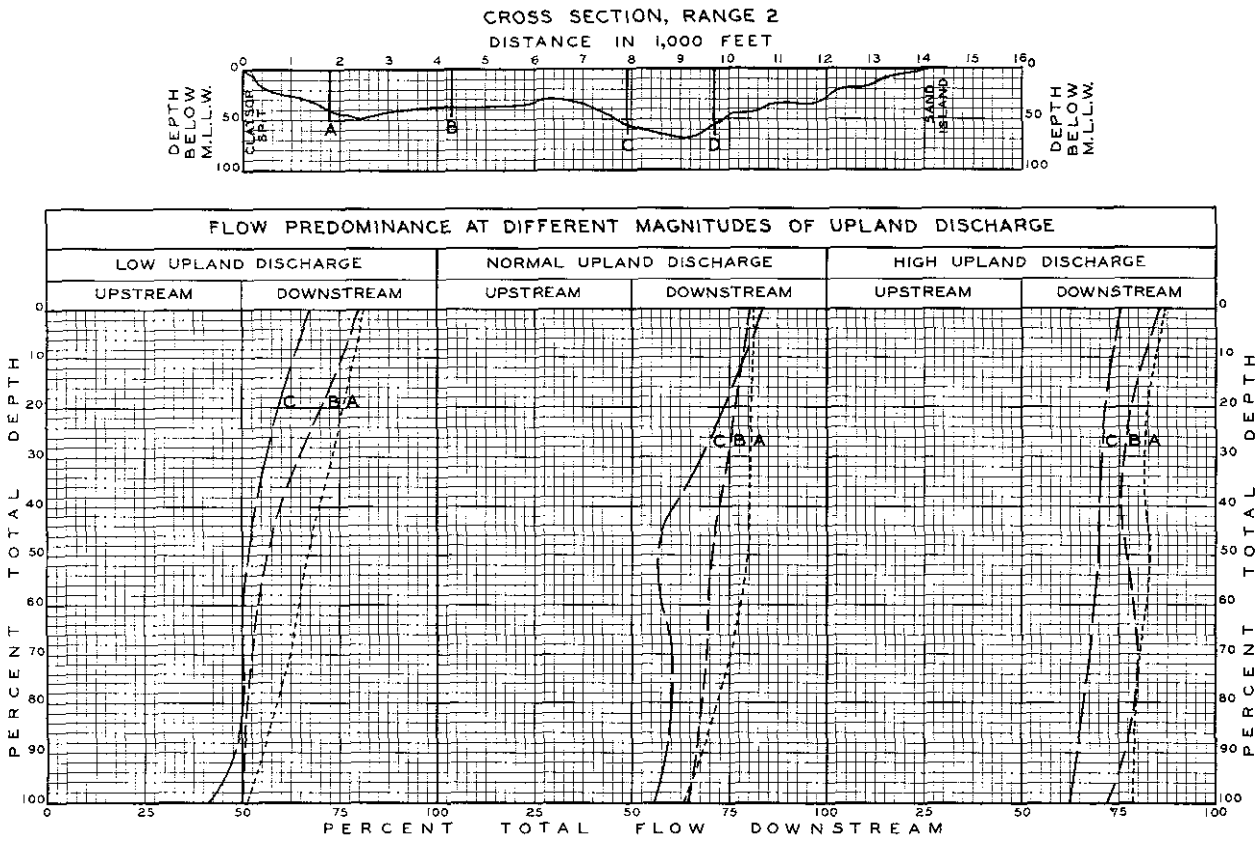



FIGURE 16. — **Flow predominance, range 2.**

completed, in progress, or planned for the Upper Columbia River and its tributaries will be very significant in the estuarine section of the river.

In the light of all these conditions, the committee felt that more must be learned regarding the physical factors governing the behavior of the waterway and that a comprehensive hydraulic model of the lower estuary represented the most important and urgently needed tool to aid in the development of this knowledge. Accordingly, the committee recommended that immediate steps be taken toward construction of a suitable hydraulic model of the Lower Columbia Estuary from Oak Point to the sea as the initial phase of a comprehensive model investigation.

## Offshore and Estuarine Scour and Shoal

### *Long-Term Trends*

Concurrently with the general analysis of observed data, studies were made of the general shoaling patterns since 1945 reflected by condition surveys made of the immediate offshore area, the entrance area, and two shoal areas upstream therefrom (Desdemona and Flavel shoals). These latter shoal areas are located in the vicinities of miles 8.8 and 10.7, respectively. In the review of these particular studies it should be recognized that they show only the changes occurring within the limits of the authorized channel, and, consequently, such studies are not capable of showing the character or magnitude of changes occurring elsewhere throughout the vast estuary and entrance area. Although these studies represent an analysis of known changes, they cannot, due to the limited area covered, reveal more than an indication of the changes occurring in this vast area.

A somewhat clearer indication of the changes occurring in the lower estuary is revealed by a preliminary study of long-term scour and shoal trends made in 1961, which compared bank-to-bank hydrographic surveys of 1868 and 1958 within an 11-mile reach between north and south lines passing through Upper Sand Island and Tongue Point. This study indicated that the total net shoaling occurring within this reach of the estuary, which did not include the

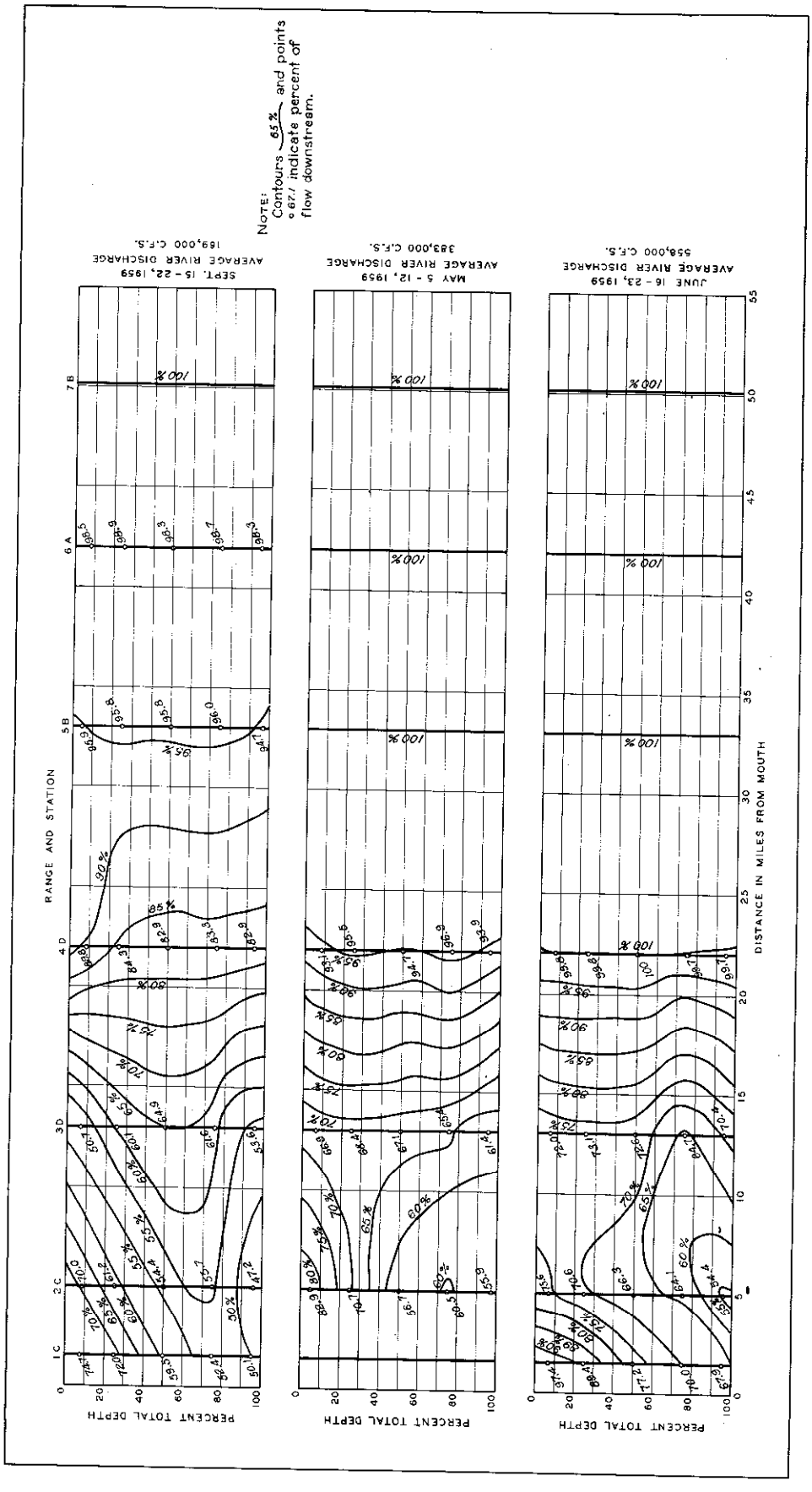



FIGURE 17. — Flow predominance profile, north channel.

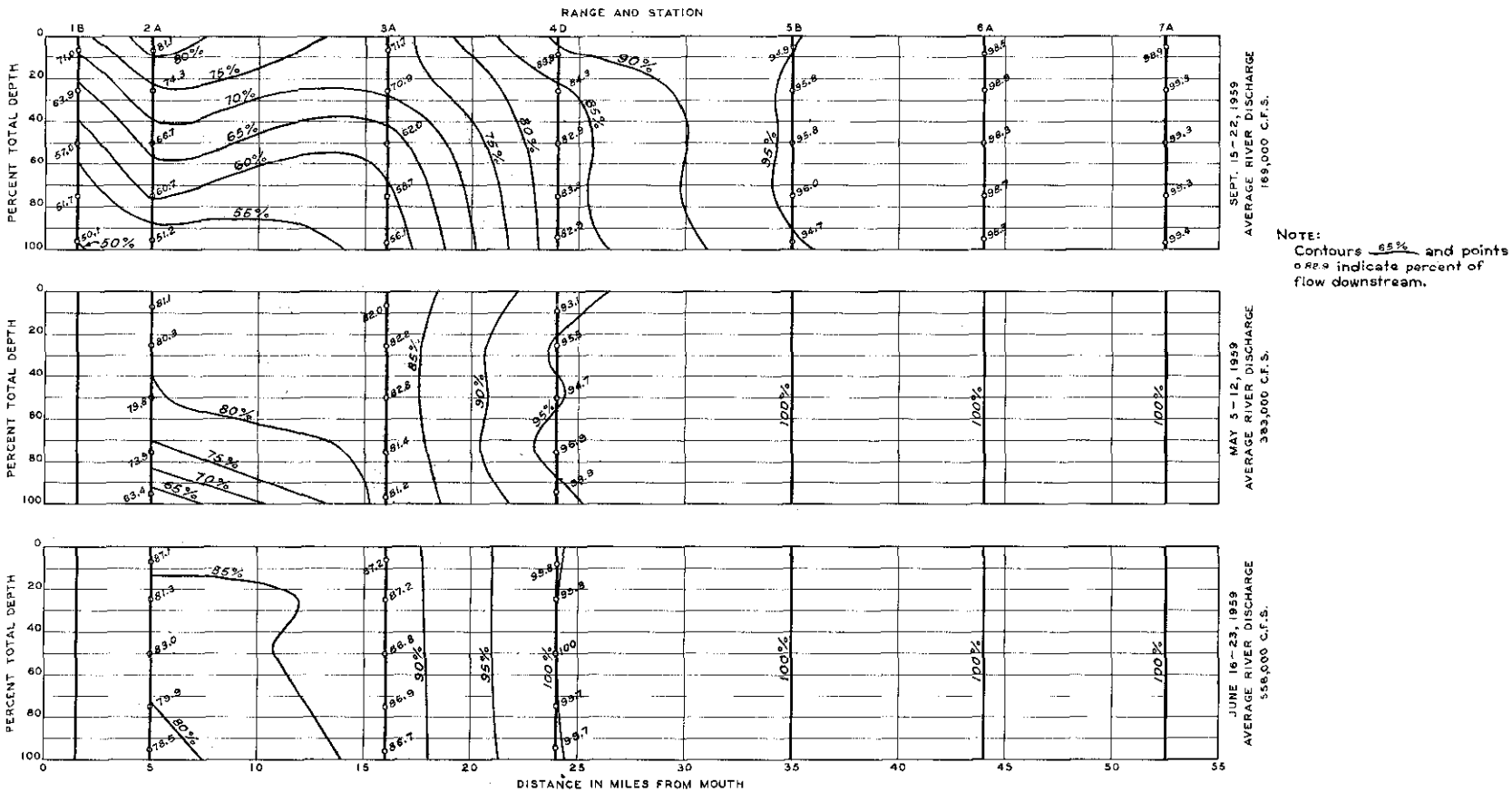



FIGURE 18. — Flow predominance profile ship, channel.

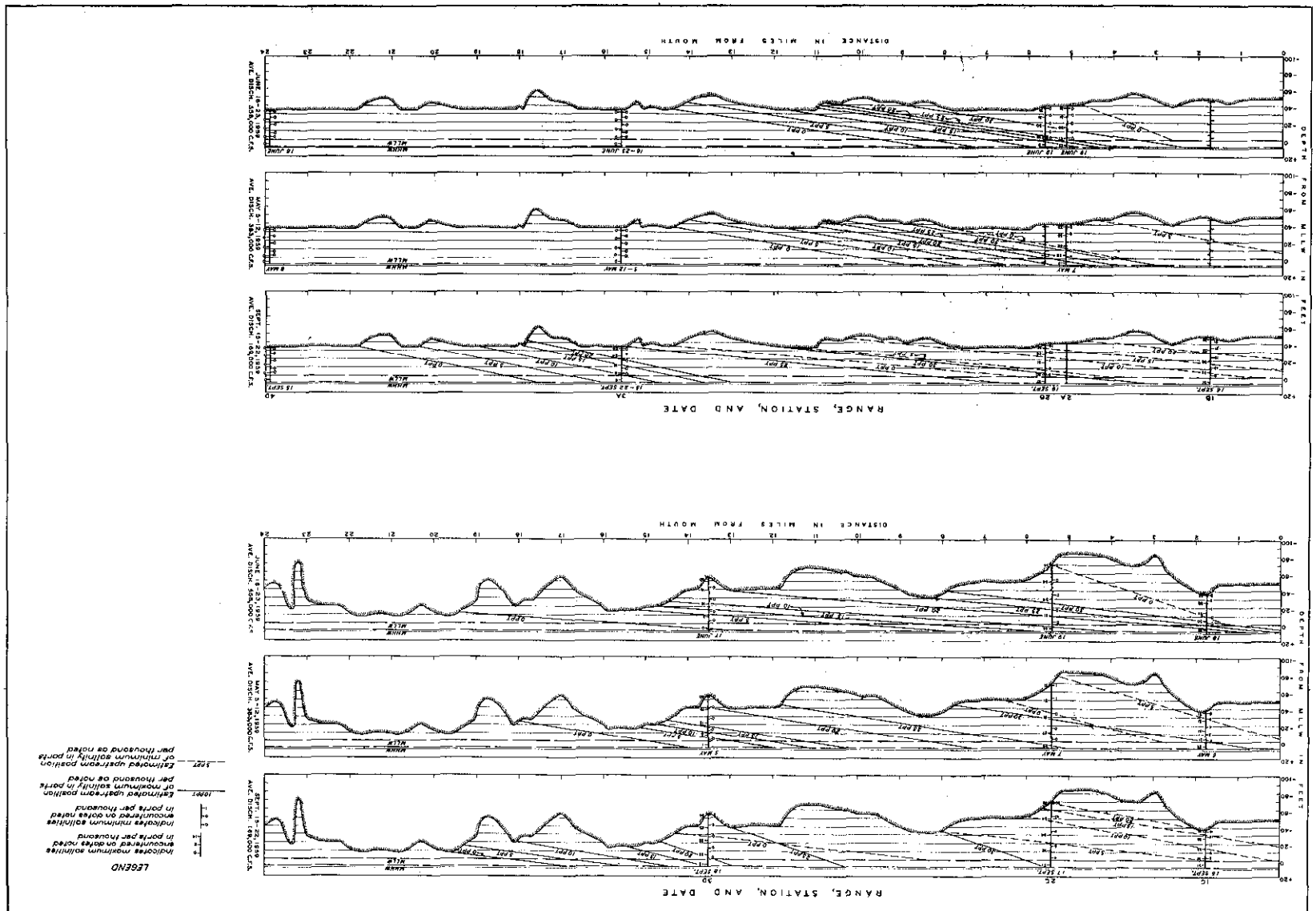



FIGURE 19.—Salinity profiles.

tremendous Clatsop Shoals, amounted to 77 million cubic yards of material over the 90-year period. Deposition of this amount of material represents an average shoaling of about 3 feet over the 40 square miles of area considered. This preliminary study tends to confirm the view that the Columbia Estuary, since the days of initial improvement, has been acting as a vast sediment trap. Of this amount of shoaling, 66 million cubic yards, or approximately 90 percent occurred in the lower 6-mile reach below Astoria, which includes the Desdemona and Flavel Shoal areas.

### *Offshore*

A study of offshore conditions landward of the 16-fathom depth line from 6 miles south to an equal distance north of the Columbia entrance shows that great changes have occurred in this area since 1877. During the 50-year period, 1877 to 1926, which corresponds roughly to the period of jetty construction, this study revealed that there occurred a net accretion of 183 million cubic yards of material north of the entrance and a net erosion of 374 million cubic yards of material south of the entrance, as shown on figure 20. This change is attributed in large part to the interception by the North Jetty of the predominately southbound littoral drift, which resulted in the accumulation of material north of the entrance and starvation of the offshore area immediately to the south. During the following 32-year period, 1926–58, in which the influence of the jetties was continued, the study revealed that the incremental net accretion to the north of the entrance amounted to about 134 million cubic yards, whereas incremental erosion to the south amounted to about 130 million cubic yards, as shown in figure 21.

### *Entrance Channel*

Studies of changes occurring within the entrance channel since 1945 have revealed a trend for increased shoaling in this channel throughout this period, notwithstanding the change in project depths from 40 to 48 feet effected in 1956, and the associated increased dredging made necessary thereby. Figure 22 shows in graphical form accumulated dredge quantities material remaining in the authorized entrance channel above 42 feet of depth, accumulated scour and shoal, and average rates of shoaling from 1945 to 1955, during which time the 40-foot project was being maintained. Figure 23 shows similar information for the period from 1956 to 1960, during which the 48-foot project was being developed. The average rates of shoaling within the entrance channel during these periods as shown on these graphs are as follows:

| *Period* | *Average rate of shoaling Cubic yards per year* |
|---|---|
| 1945–52 | 909,000 |
| 1953–55 | 1,518,000 |
| 1956-60 | 1,931,000 |

### *Desdemona Shoal*

The results of similar studies of changes occurring within the authorized navigation channel, 35 feet deep and 500 feet wide, through Desdemona Shoal during the period 1945–60 are reflected on figure 24. The average rate of shoaling prior to the change in channel alinement, effected in January 1951, amounted to about 458,000 cubic yards a year. Since that time the shoaling rate has been reduced to an average of about 270,000 cubic yards a year.

### *Flavel Shoal*

Figure 25 shows the results of studies made of changes occurring within the authorized navigation channel through Flavel Shoal since 1946. The average shoaling rate in this area during this period has been about 270,000 cubic yards a year.

## Attrition South of South Jetty

As previously mentioned, serious attrition has been taking place in adjacent offshore areas along the Oregon coastline immediately south of the South Jetty since the days of initial improvement. Perhaps the most startling visual evidence of this change is the gradual erosion during the last 25 years of the massive sand dunes which protected the arm of land connecting the midpoint of the South Jetty as completed in 1895 with Point Adams. These dunes have been breached along several hundred feet of shore immediately south of the structure and are quite thin for some distance farther south.

The driftwood line indicates that the remaining beach berm has been overtopped during recent storms and it now appears that a complete breach to the lagoon in rear of the shore may occur at any time and is inevitable within a few years unless action is taken to prevent such a catastrophe. The low, permeable enrockment along the remains of the trestle at the root of the jetty separates the lagoon from the estuary proper, as shown in figure 26, but the tide ebbs and flows freely through this structure. Because of the lag in tidal time between this part of the estuary and the open sea, about 40 minutes, a substantial head would develop along this light enrockment in the event of a breach between the sea and the lagoon.

It is quite probable that failure of the structure by scour would be rapid in that eventuality and the breach would quickly develop into the primary course of the river outlet. A program

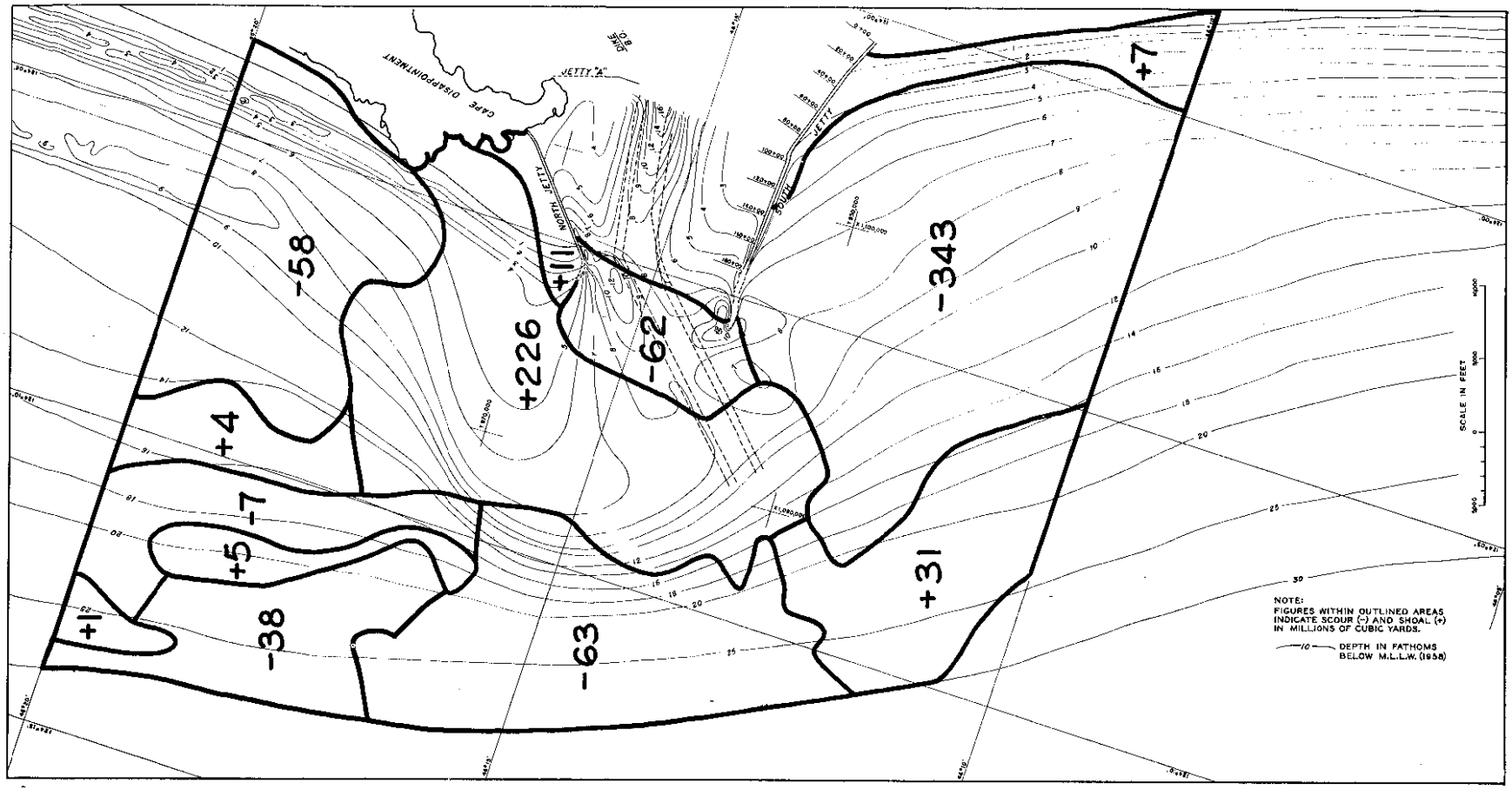



FIGURE 20. — Offshore scour and shoal volumes, 1877-1926.

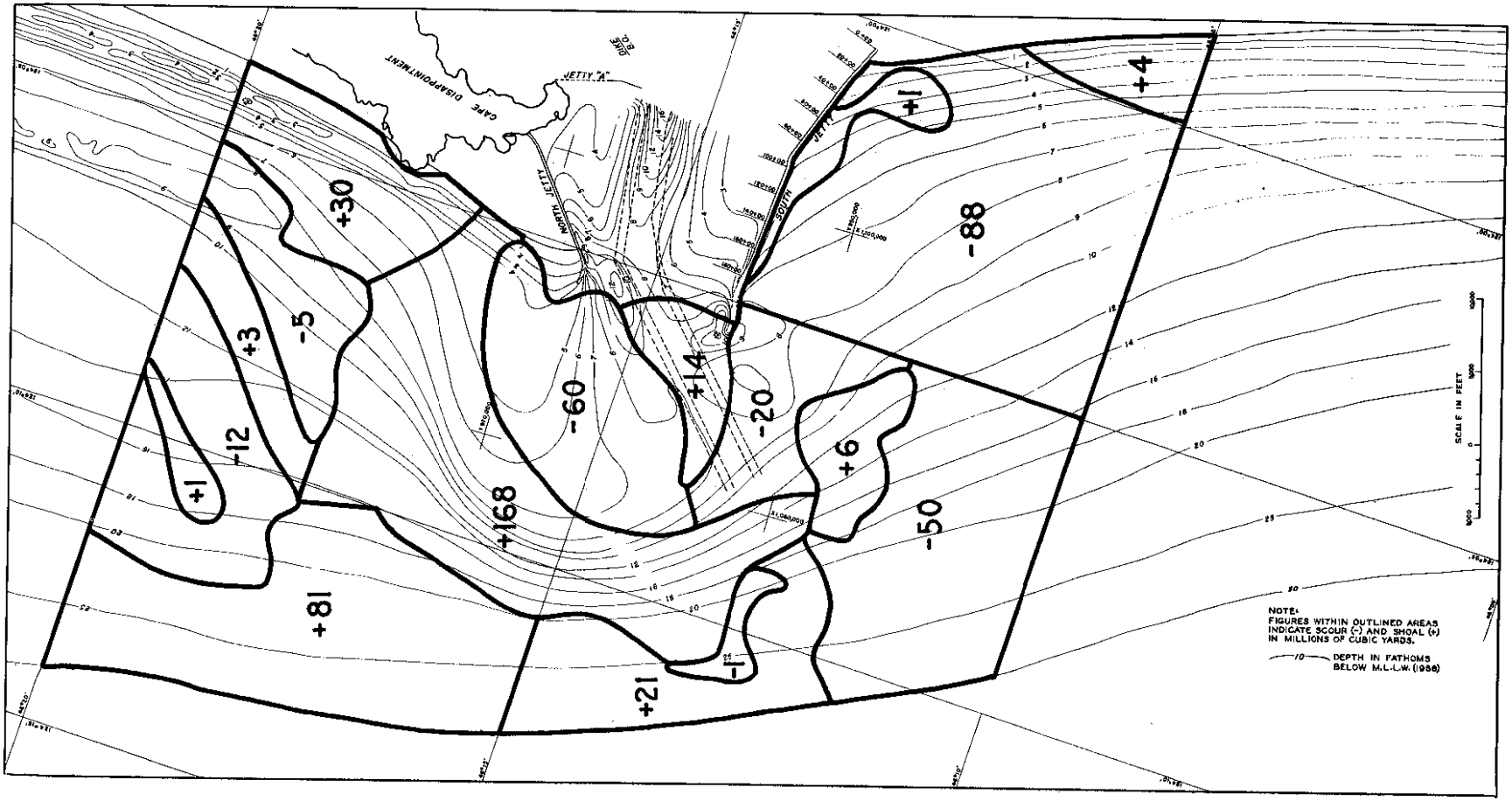



FIGURE 21. — Offshore scour and shoal volumes, 1926-58.

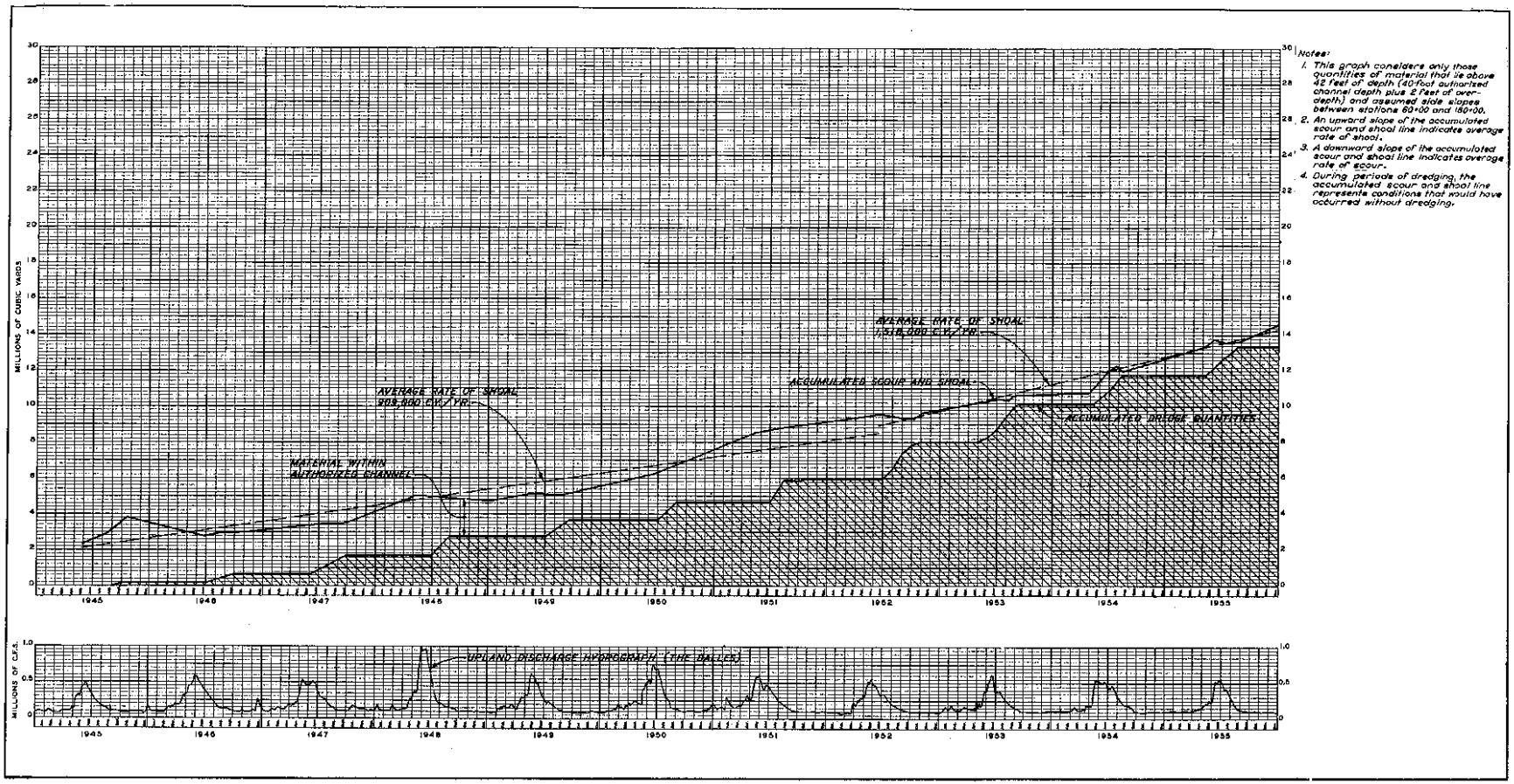
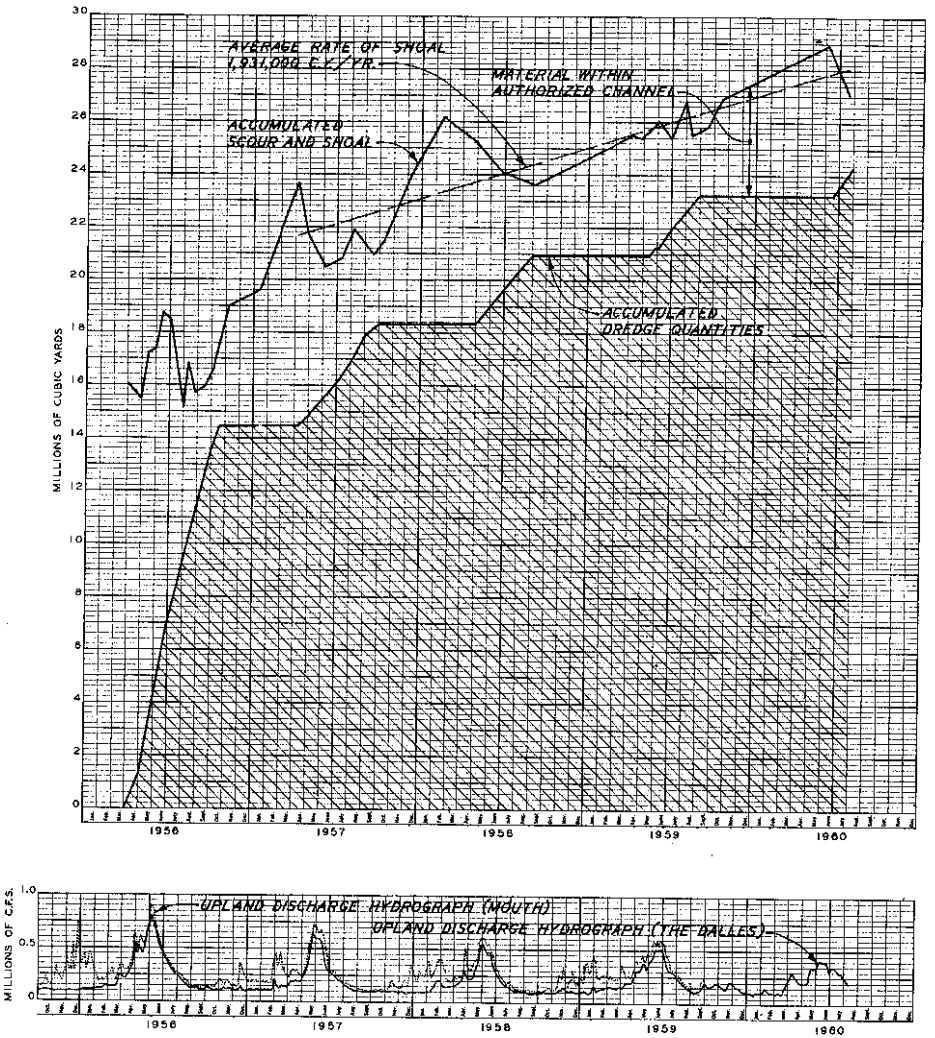



FIGURE 22.—Scour and shoal, 40-foot channel.




Notes:

1. This graph considers only those quantities of material that lie above 50 feet of depth (48-foot authorized channel depth plus 2 feet of over-depth) and assumed side slopes between stations 60+00 and 288+00.
2. An upward slope of the accumulated scour and shoal line indicates average rate of shoal.
3. A downward slope of the accumulated scour and shoal line indicates average rate of scour.
4. During periods of dredging, the accumulated scour and shoal line represents conditions that would have occurred without dredging.

FIGURE 23. — Scour and shoal, 48-foot channel.

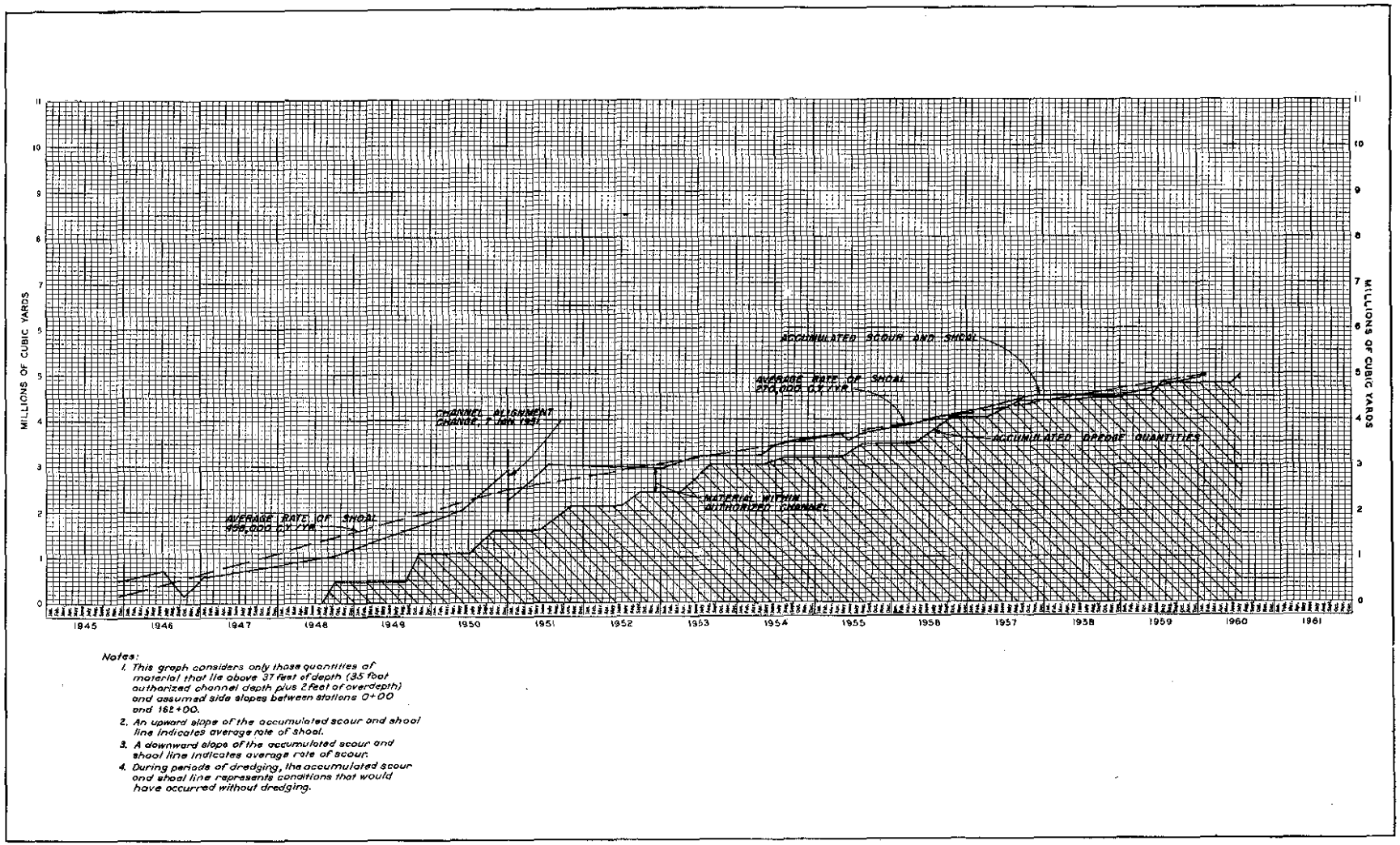
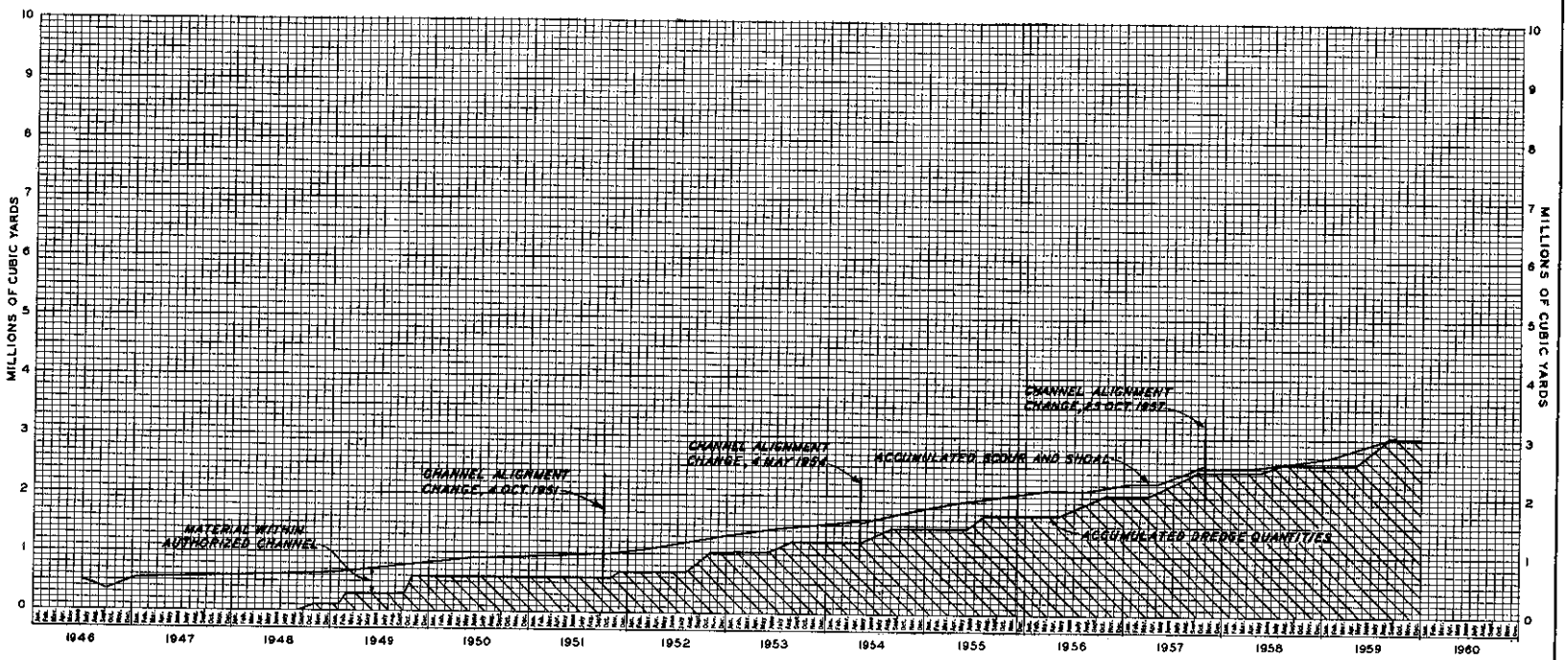



FIGURE 24. — Scour and shoal, Desdemona shoal.




Notes:

1. This graph considers only those quantities of material that lie above 37 feet of depth (35 foot authorized channel depth plus 2 feet of overdepth) and assumed side slopes between stations 50+00 and 216+00.
2. An upward slope of the accumulated scour and shoal line indicates average rate of shoal.
3. A downward slope of the accumulated scour and shoal line indicates average rate of scour.
4. During periods of dredging, the accumulated scour and shoal line represents conditions that would have occurred without dredging.

FIGURE 25. — Scour and shoal, Flavel shoal.

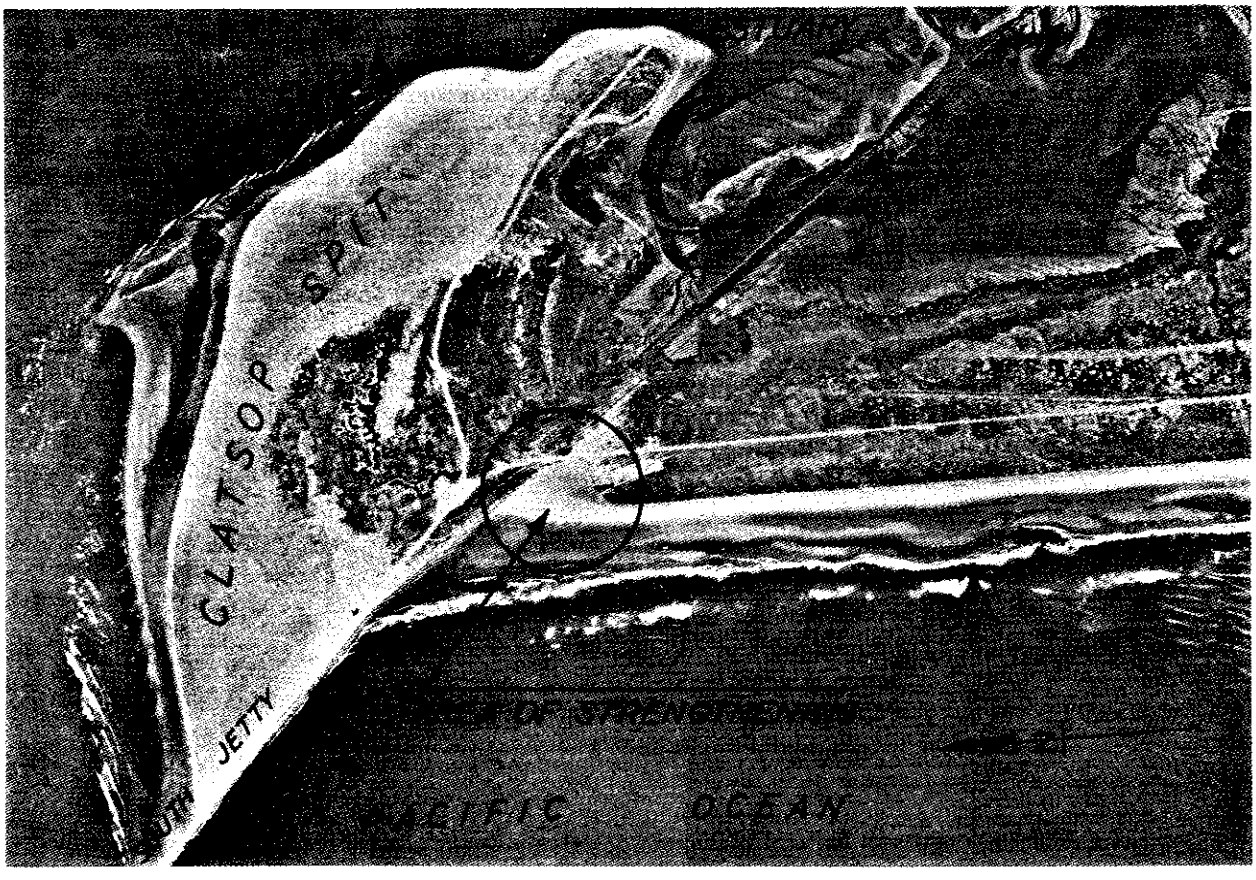



FIGURE 26. — Erosion south of South Jetty.

of study and surveillance of offshore conditions in the vicinity of the South Jetty has been launched in an effort to determine the cause or causes of this problem with the view to undertaking appropriate corrective action. In the meantime, a temporary structural strengthening of the threatened area has been made to prevent a relocation of the entrance channel, pending completion of the study and surveillance program and analysis of the problem in the hydraulic model hereinafter described in detail.

## Satistical Wave Studies

In continuation of Corps of Engineers' studies of characteristics of waves off the California coast, a contract was awarded in August 1960 to the National Marine Consultants, Inc., Santa Barbara, Calif., for accomplishment of studies, leading to the development of characteristics of waves prevailing off the coasts of Oregon and Washington. These particular studies had as their objectives: (1) the development of wave statistics for three deep-water stations off the Oregon and Washington coasts based on conditions prevailing in 1956, 1957, and 1958; (2) statistics for 12 most severe storms at these selected stations during the period from 1950 to 1960; and (3) structural design wave analyses and wave modification analyses of conditions prevailing at and near the Columbia Entrance during the period 1940–60. The results of these studies, discussed in the following paragraphs, are contained in manuscript reports prepared by National Marine Consultants, Inc.[8]

### *Hindcast Studies, 1956, 1957, and 1958*

The general area covered by these studies embraced the entire Pacific coast of Oregon and Washington as represented by three selected deep-water stations: one located off the entrance to Yaquina Bay, one off the Columbia Entrance, and one off the coast between Grays Harbor

[8] NATIONAL MARINE CONSULTANTS, INC. WAVE STATISTICS FOR TWELVE MOST SEVERE STORMS AFFECTING THREE SELECTED STATIONS OFF THE COASTS OF WASHINGTON AND OREGON, DURING THE PERIOD 1950-1960. January 1961; OCEANOGRAPHIC STUDY FOR COLUMBIA RIVER ENTRANCE. March 1961; WAVE STATISTICS FOR THREE DEEP WATER STATIONS ALONG THE OREGON-WASHINGTON COAST. May 1961. Santa Barbara, Calif.

and the Strait of Juan De Fuca. On the basis of 6-hour U.S. Weather Bureau synoptic charts covering extratropical cyclones and the Pacific high pressure cell, these studies routed the paths and decay of meteorological disturbances to develop the height, direction, and period of the resulting wave conditions at the deep-water stations. The extratropical cyclones considered in these studies included storms moving into the Gulf of Alaska, storms moving directly into the study area, storms moving onto the California coast, and western Pacific storms.

In addition to development of the characteristics of waves resulting from these meteorological conditions termed "swell," the studies also determined similar characteristics for waves caused by local wind conditions termed "sea." This latter condition often constituted a separate source of swell at an adjacent station, inasmuch as a wave in the form of "sea" at one station would show up as "swell" at an adjacent station not under the influence of the same wind condition. It was frequently found during the course of the studies that as many as six separate wave trains would sometimes occur simultaneously at the deep-water stations. As a consequence, in the annual summary shown in table 5, the time over which "swell" conditions occur is greater than the elapsed time over the period covered. This tabulation and table 6 show the significant wave height and direction frequency of waves caused by "swell" and "sea," respectively, at the deep-water station off the Columbia Entrance, located at latitude 46° 12′ North and longitude 124° 30′ West:

TABLE 5.—*"Swell": Average annual height and direction* [1] *frequency distribution* [2]

Average total hours: 10,508.45

| Height of "swell" (feet) | NNW | NW | WNW | W | WSW | SW | SSW | S | Total Σ |
|---|---|---|---|---|---|---|---|---|---|
| | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* |
| 1 to 2.9 | 0.68 | 8.20 | 12.67 | 13.69 | 6.67 | 2.31 | 2.85 | 1.90 | 48.97 |
| 3 to 4.9 | .04 | 2.16 | 12.36 | 13.61 | 4.67 | 1.83 | 1.88 | .81 | 37.36 |
| 5 to 6.9 | .04 | 1.18 | 6.14 | 5.51 | 1.37 | .98 | 1.07 | .39 | 16.68 |
| 7 to 8.9 | .04 | .45 | 2.50 | 2.67 | .54 | .74 | .55 | .23 | 7.72 |
| 9 to 10.9 | .05 | .20 | 1.11 | 1.43 | .47 | .36 | .30 | .16 | 4.08 |
| 11 to 12.9 | ..... | .09 | .66 | .63 | .30 | .09 | .11 | .13 | 2.01 |
| 13 to 14.9 | ..... | .02 | .28 | .61 | .21 | .06 | .12 | .04 | 1.34 |
| 15 to 16.9 | ..... | .02 | .13 | .21 | .09 | .04 | .13 | .10 | .72 |
| 17 to 18.9 | ..... | .07 | .06 | .20 | .06 | .02 | .02 | .02 | .45 |
| 19 to 20.9 | ..... | .04 | .04 | .07 | .02 | ..... | ..... | ..... | .17 |
| 21 to 22.9 | ..... | ..... | .02 | .09 | ..... | ..... | ..... | ..... | .11 |
| 23 to 24.9 | ..... | ..... | ..... | .04 | .07 | ..... | ..... | ..... | .11 |
| 25 to 26.9 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 27+ | ..... | ..... | ..... | [3] .04 | ..... | ..... | ..... | ..... | .04 |
| Total Σ | 0.85 | 12.43 | 35.97 | 38.80 | 14.47 | 6.43 | 7.03 | 3.78 | 119.76 |

[1] Direction from which waves approach the deep-water station.
[2] Based on 365⅓ days.
[3] 30 feet (March 1956).

TABLE 6.—*"Sea": Average annual height and direction*[1] *frequency distribution*[2]

| Height of "sea" (feet) | N | NNW | NW | WNW | W | WSW | SW | SSW | S | Total Σ |
|---|---|---|---|---|---|---|---|---|---|---|
| | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* |
| 1 to 2.9 | 2.06 | 2.25 | 7.32 | 1.44 | 2.77 | 1.00 | 2.49 | 1.57 | 3.07 | 23.97 |
| 3 to 4.9 | .84 | 1.76 | 5.25 | 1.45 | 1.51 | 1.21 | 1.61 | 1.40 | 2.43 | 17.46 |
| 5 to 6.9 | .42 | .62 | 1.71 | .54 | .79 | .73 | 1.11 | 1.44 | 1.85 | 9.21 |
| 7 to 8.9 | .16 | .28 | 1.22 | .53 | .60 | .46 | 1.14 | 1.04 | 1.06 | 6.49 |
| 9 to 10.9 | .02 | ..... | .37 | .21 | .16 | .14 | .23 | .63 | .80 | 2.56 |
| 11 to 12.9 | ..... | .04 | .43 | .13 | .09 | .11 | .35 | .45 | .95 | 2.55 |
| 13 to 14.9 | ..... | .07 | .05 | .09 | .09 | .10 | .23 | .45 | .45 | 1.53 |
| 15 to 16.9 | ..... | ..... | .05 | ..... | .09 | ..... | ..... | .12 | .25 | .51 |
| 17 to 18.9 | ..... | .02 | .07 | .02 | ..... | .02 | .05 | .19 | .35 | .72 |
| 19 to 22.9 | ..... | ..... | ..... | .02 | ..... | ..... | ..... | .12 | .06 | .20 |
| 23 to 27 | ..... | ..... | ..... | ..... | ..... | ..... | .04 | .02 | .04 | .10 |
| Total Σ | 3.50 | 5.04 | 16.47 | 4.43 | 6.10 | 3.77 | 7.25 | 7.43 | 11.31 | 100.00 |

[1] Direction from which waves approach the deep-water station.
[2] Based on 365⅓ days.
[3] Frequency distribution of "sea" offshore of 6.03 percent and of calm of 28.67 percent; calm includes waves of 0 to 0.9 foot.

### *Severe Storm Wave Characteristics, 1950-60*

Generally, extratropical cyclones giving rise to severe wind and wave conditions along the Oregon-Washington coast originate near Japan and move eastward across the Pacific toward the Gulf of Alaska. The degree to which these disturbances affect conditions off the Columbia Entrance depends not only upon the intensity of the storm but also on the disposition of the Pacific high cell. During the summer months this cell is sufficiently removed to the north so that only a few storms approach the Oregon-Washington coast and those that do are relatively weak.

During the period from October to May, however, with the southward shift of the Pacific cell, intense storms approach the coast. These storms develop in the form of rapidly moving intense frontal systems on low pressure centers and occur several times each winter. Although prefrontal southerly winds often attain velocities of over 100 miles per hour in gusts along the coast, the winds associated with these storms seldom reach hurricane force over much of the area. However, the extent of storms in terms of duration and fetch can produce hurricane magnitude waves that last for relatively long periods of time.

Principal wind fields associated with these storms produce south to southeasterly winds preceding a warm front, southwest to west-southwest winds in the warm sector, and west-southwest to northwest winds behind a cold front. Each of these wind regimes can develop high waves, but offshore topography and coastal promontories dictate shallow water effects at coastal locations. Hindcast methods reveal the characteristics of severe storm waves in deep water off the Columbia Entrance during the period 1950–60 as shown in table 7.

TABLE 7.—*Severe storm wave characteristics*

| Date | Significant height | Significant period | Direction from— |
|---|---|---|---|
| | *Feet* | *Seconds* | |
| 27 Oct. 1950 | 24 | 11 | SW |
| 21 Jan. 1951 | 27 | 14 | WSW |
| 2 Feb. 1951 | 25 | 13 | WSW |
| 18–19 Dec. 1951 | 23 | 12 | NW |
| 6–7 Dec. 1952 | 25 | 11 | SW |
| 5 Dec. 1953 | 24 | 14 | WSW |
| 12–13 Feb. 1954 | 30 | 13 | SSW |
| 19 Nov. 1954 | 23 | 12 | SW |
| 15 Jan. 1956 | 24 | 12 | SW |
| 3 Mar. 1956 | 30 | 12 | W |
| 26 Dec. 1957 | 23 | 13 | W |
| 23 Jan 1958 | 26 | 11 | SSW |
| 11 Dec. 1959 | 25 | 13 | SW |

### *Structural Criteria of Columbia Entrance*

This study was composed of two parts — one part considered the characteristics of storm waves as applied to the design of structures at the Columbia Entrance and the other part related to refraction patterns assumed by waves from different directions and of different periods as they approach a 20-mile sector of the coast centered on the Columbia Entrance. The structural design wave analyses were based on hindcast studies of a number of severe storms that approached the entrance during the last 20 years. Considering the deep-water characteristics of waves generated by the previously mentioned severe storms and the physical factors tending to modify these waves, such as exposure, refraction, shoaling, bottom friction and percolation, bottom slope and still-water depth at the entrance site, as well as the frequency distribution of wave heights, it was determined that structural criteria at the Columbia entrance should be based on the following general wave conditions:

Significant wave height

$(H_s)$ ..... $30.0^{+4}_{-2}$ feet

Significant wave period

$(T_s)$ ..... $13.0^{+1}_{-1}$ seconds

Direction of approach $(\theta)$ $235°^{+20}_{-15}$ (from)

With a tide range of 8.5 feet, storm setup of 2.5 feet, and design depth of 51 feet, the most severe condition occurring within the period of study produced a wave with a significant height of 33.5 feet, significant period of 13 seconds, and an angle of approach of 234° as shown in table 8.

TABLE 8.—*Design wave characteristics*

| Deep water | | | Modification | | Design wave at entrance | | |
|---|---|---|---|---|---|---|---|
| $H_s$ (feet) | $T_s$ (sec.) | $\Theta$ (dir.) | Refraction coefficient $K_r$ | Shoaling coefficient $K_s$ | $H_s$ (feet) | $T_s$ (sec.) | $\Theta$ (dis.) |
| 30 | 13 | 205° | 0.94 | 0.996 | 28.0 | 13 | 206° |
| 25 | 13 | 225° | 1.11 | .996 | 27.5 | 13 | 218° |
| 28 | 13 | 245° | 1.20 | .996 | 33.5 | 13 | 234° |
| 30 | 13 | 270° | .97 | .996 | 29.0 | 13 | 254° |
| 28 | 13 | 295° | .71 | .996 | 20.0 | 13 | 263° |
| 23 | 12 | 315° | 1.24 | .996 | 27.5 | 13 | 278° |

## Radioactive Analysis of Bottom Samples

In connection with the broad study being conducted by the Atomic Energy Commission of the distribution of Columbia River sediments as related to discharge of radioactive waste effluent from the Hanford complex, which is subsequently discussed, samples of bottom materials were obtained from five different points within the Columbia Entrance and from one location in the lower Estuary. In the Entrance area, one (sample A) was obtained from the channel on Range 2, two (samples B and C)

were obtained from each side of the entrance channel on Range 1, one (sample D) in the entrance channel between the ends of the jetties, and one (sample E) in the channel about 1 mile outside the jetties. In the lower estuary one sample (sample F) was obtained from the channel near Harrington Point.

These samples were analyzed by the General Electric Co. in Hanford to ascertain the relative ages of isotopes present in an endeavor to determine the source of materials forming the samples. In these samples, the radioisotope concentrations were extremely low in comparison with the natural radioactive content of the material. However, it was possible to make age estimates based on the $Zn^{65}$ (245 day half-life) and $Cr^{51}$ (26 day half-life) concentrations. A high $Cr^{51}$ concentration relative to the $Zn^{65}$ concentration would represent a "new" material whereas a low $Cr^{51}$ concentration relative to $Zn^{65}$ would represent much "older" material. The observed $Zn^{65}$ and $Cr^{51}$ concentrations in the above samples and their calculated ratio and relative ages were determined as shown in table 9. The

TABLE 9.—*Analysis of samples*

| Sample | Concentration $D/M$/gram | | Ratio $\frac{Cr^{51}}{Zn^{65}}$ | Relative age |
|---|---|---|---|---|
| | $Zn^{65}$ | $Cr^{51}$ | | |
| A | 17.1 | 11 | 0.64 | New. |
| B | 3.9 | 1 | .3 | Old. |
| C | 5.8 | 5 | .85 | New. |
| D | 3.2 | 1 | .3 | Old. |
| E | 6.3 | 5 | .79 | New. |
| F | 7.6 | 30 | 3.95 | Very new. |

newer sediments suggest the likelihood that the material has been recently formed of particles moved into the estuary from upland areas, whereas the older sediments very likely have moved into the entrance area from the ocean.

## Distribution of Columbia River Sediments

The Atomic Energy Commission has launched a broad program of study and investigation aimed (1) to explore Columbia River sediments as they bear on the fate of radioisotopes released from the Hanford production complex and (2) to determine the interactions taking place in the Columbia Estuary related to the ultimate distribution of sediments as measured by the concentration of radioactivity on these sediments. In this study, it is proposed to collect and analyze samples of bottom and suspended sediment to determine the principal chemical and physical exchange reactions involved, the sedimentation rates and accumulations, and the character of transport downstream of the sediments of sorbed or precipitated isotopes, with the view to depicting in a quantitative manner the radioactive material balance in regard to its interactions with the sediments.

In the estuarine section of the river, the study is directed toward measurement of the physical forces of tides and currents and to sample the estuarine waters, biota, and sediments to determine:

1. The dispersion patterns and rates of interchange of the dissolved radioactivity in the fresh waters with those of the saline, brackish waters;
2. The removal of radioactivity from solution by means of chemical exchange with suspended and bottom sediments in the estuary; and
3. The rate and degree of removal of radioactivity from solution into the biological food web.

### *Status of Investigation*

It is not difficult to visualize that a study of this broad scope and complexity, considering its pioneering nature, will require several years to complete. Nevertheless, considerable progress has already been made along several broad phases of the investigation. Pursuant to contracts made with the Atomic Energy Commission, the Department of Oceanography, University of Washington, has conducted field investigations to determine the characteristics of movement of Columbia River waters in the Pacific Ocean and has traced the fresh water plume of flow for many miles seaward.

Other field investigations by that Department are directed at measurements of ocean salinity, chemical composition, transparency, temperature, density, plankton, and analysis of bottom sediments along the coast, across the continental shelf, and into the abyssal plain of the ocean. In addition, studies by the Laboratory of Radiation Biology of that University of the radioactivity in biological organsms are underway.

Through another contract, the U.S. Geological Survey has activated a program to study fluvial sediment transport in the river as may be related to the sorption of radionucleides associated with such movement. Although this program will initially investigate sediment phenomena at a number of ranges across the Columbia River above Portland, it is possible that this program will be later extended to cover reaches of the river below that point. It is anticipated that the comprehensive hydraulic model of the lower Columbia Estuary and Entrance, subsequently described, will eventually be of much value in revealing the distribution of sediments in these regions.

### *Related Studies*

The Department of Oceanography, Oregon State University, for a number of years has

been engaged in the study of coastal and ocean phenomena prevailing along the coast of Oregon. In addition to its investigation of the nature of sediments in estuarine waters along this coast, the Department is engaged in preparing bathymetric charts of the offshore areas and plans shortly to map and study the physical features of the Astoria Canyon lying off the Columbia Entrance. Its study of the distribution of Columbia River water has revealed the seasonal offshore movements of ocean currents to the south during the summer and to the north during the winter. Offshore cruises have revealed large lenses of traceable river water upwards to 50 miles in diameter and 100 feet thick located hundreds of miles at sea. These lenses of water are somewhat warmer than the river water flowing from the Columbia Entrance.

## Comprehensive Hydraulic Model Investigation

### *Authorization*

Pursuant to the previously mentioned recommendations of the Committee on Tidal Hydraulics, which received the concurrence of the district engineer, Portland, and division engineer, North Pacific Division, Portland, Oreg., the chief of engineers in March 1961 authorized the undertaking of a compreshensive hydraulic model investigation of the lower Columbia Estuary and Entrance. As the result, the U.S. Army Engineer Waterways Experiment Station in Vicksburg, Miss., has constructed and will shortly place into operation a hydraulic model that will reproduce the lower 52 miles of the Columbia Estuary and a part of the Pacific Ocean extending about 8 miles north and south of the river entrance and offshore to the 20-fathom contour.

### *Design Features*

The hydraulic model has been constructed to linear scale ratios, model to prototype, of 1:500 horizontally and 1:100 vertically. Tides and tidal currents are being reproduced by a primary tide generator located in the ocean part of the model and a secondary tide generator located at the upstream limit of the model at Oak Point, Wash. The model ocean is equipped with wave generators to reproduce ocean wave effects on movement and deposition of sediments, as well as a circulating system to reproduce the effects of alongshore littoral currents on sedimentation phenomena.

The model will be operated with salt water, salinity scale 1:1, in the model ocean, and with fresh water supplied by the Columbia River and its significant tributaries, so that density effects on hydraulic and shoaling phenomena will be reproduced. The model will be of a combination fixed-bed and movable-bed type, so that critical parts of the model estuary and entrance can be converted from one type to another, thus employing the advantages of each type in conducting studies of the various problems involved. Figure 27 shows the general layout of the model as well as the limits of the fixed-bed and movable-bed parts.

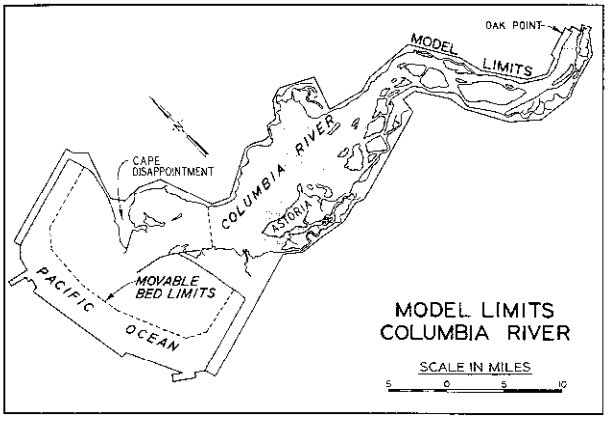



FIGURE 27. — General layout of Columbia hydraulic model.

### *Model Verification*

Hydraulic and salinity verifications of the model were based on prototype measurements of tides, current velocities, current directions, and salinities observed during the 1959 prototype measurement program. The observation stations manned during that program, as well as the range of conditions covered, were specifically planned to provide the data needed for these phases of model verification. As the model is adjusted to reproduce simultaneously all pertinent phenomena observed at the prototype stations during the 1959 measurement program, pertinent phenomena at intermediate points in the prototype are also being reproduced to scale.

Verification of bed movement in the movable-bed part of the model involves an empirical process of adjusting the forces involved, the nature of the bed material, and the duration of model operation until the model will reproduce within sufficient accuracy the significant changes in bed conditions that have occurred in the prototype during known periods of time. These significant changes include the formation of shoals in the navigation channel, accretion and erosion, or erosion in areas outside the navigation channel, and accretion or erosion attributable to structures constructed in the

prototype during the period of time used for model verification purposes.

Once the model has demonstrated its ability to reproduce all significant changes in bed conditions of the prototype, the actual model operation time required to effect such changes is used as a measure of the time scale for bed movement. Thus, if the model consistently shows that changes which occurred in the prototype between hydrographic surveys made at intervals of 1 year are being reproduced in the model during 20 hours of operation, then the time scale for bed movement would be 20 hours model to 1 year prototype, or about 1 to 438.

Although the time scale for bed movement in the Columbia model will not be known until the movable-bed verification is accomplished, the time scale for reproduction of hydraulic and salinity phenomena, derived mathematically from model linear scales, will be about 1 to 50. Thus the model, within the time scales thus determined, will reproduce tidal elevations and phases, current velocities from bank to bank and from surface to bottom, salinity concentrations and distribution throughout the saline part of the estuary, and movements of sediments within the movable-bed part of the model. In addition to reproducing these phenomena, the model will automatically reproduce the dilution, dispersion, and flushing patterns of any contaminants that might be introduced and readily mixed with the waters of the estuarine system.

### *Model Testing*

It is anticipated that during the next 25 years, continued growth and industrial development within the Columbia River basin, including construction and completion of many water development and conservation projects, will have a marked effect on the quality as well as characteristics of Columbia River flows. In order that the Columbia model may be a useful tool during this critical phase of development, the model has been constructed to have a useful life of at least 25 years. During the early part of its life the model will serve to solve problems of present-day urgency, after which it will be available to lend guidance to the solution of future estuarine problems as they develop and affect the welfare of the lower Columbia Estuary and Entrance area. Having been successfully verified, tests now proposed for the model are outlined in the following paragraphs.

#### *Entrance relocation*

The threat of a channel relocation through the shore end of the South Jetty resulting from the loss of shore material in that area poses an extremely difficult and challenging problem to those charged with maintenance of a usable and satisfactory entrance channel for navigation. When the enormous buildup of materials immediately offshore and north of the present entrance, the attrition of the shore and offshore areas immediately south thereof, and the continuing growth of the Clatsop Spit shoals are considered, it is not difficult to perceive that this threat is quite imminent, particularly when the destructive effects of storms experienced at the Columbia Entrance are directed toward such an eventuality.

Recognizing the possibility that such a relocated entrance, if developed in accordance with careful planning, could possess merit over the present entrance, insofar as channel maintenance is concerned, the model offers an opportunity for careful and complete engineering analyses of the problems associated with the development of such a new channel. In the model the factors now producing the large shoals in the present entrance can be evaluated as they would affect conditions along the relocated channel and permit engineers to investigate all phases of the potential problem before it occurs, thus saving much time, money, and effort that might otherwise be expended. If the dislocation to establish navigation that would occur by a sudden relocation of the entrance channel is considered, one of the initial model tests will be to explore the consequences of such relocation and to determine the courses of action to be taken.

#### *Jetty rehabilitation*

As previously mentioned, jetties at the Columbia Entrance have been rehabilitated on several occasions to restore parts of these structures that have been damaged or destroyed by destructive storms. Rehabilitation undertaken in the past has largely been directed toward restoration and increasing the height and sections of those parts of the structures lying above mean sea level on the assumption that the part below this level, if not in satisfactory condition, would make itself evident by complete failure of the jetty structure. This assumption has proved correct, as subsidence of the structures, particularly after their initial construction, revealed the fact that a certain amount of consolidation was required to effect a stable jetty foundation. Such a foundation has been secured over the years until now rehabilitation is confined to restoration of those parts of the structures damaged by wave action. Notwithstanding the limited nature of rehabilitation, the work involved is costly and must be conducted during the relatively calm season of the year. Its cost and the construction difficulties involved, together with experience gained at other river entrances, have raised question among engineers as to the value of such limited

rehabilitation insofar as maintenance of project depths are concerned.

Although completely rehabilitated jetties undoubtedly provide some benefit to small craft against heavy seas as long as these craft remain in the lee of these structures, these particular benefits are seldom of sufficient magnitude to justify the large rehabilitation costs involved. Further, if sea conditions beyond the jetty are such as to preclude safe operation of small craft, there is little reason to expect that the prudent operator of such a vessel would have need for any protection a rehabilitated jetty might provide. Despite the deteriorated condition of the South Jetty at the entrance to Grays Harbor, Wash., just north of the Columbia Entrance, depths in the authorized entrance channel to that harbor are more than twice those for which the jetties were designed to maintain. Although early rehabilitation of that jetty is now being considered, this work is required to preserve its structural integrity against attack by currents that threaten to undermine the jetty foundation.

Aside from work of such emergent nature, there appears to be sound basis for question as to the actual value of rehabilitation for the sole purpose of preserving the upper parts of jetties. Certain areas exist for exploration of the most economical time interval for conducting such rehabilitation operations, taking into consideration the construction problems involved and the actual benefits to be expected. The Columbia model, in its initial testing phase, will provide an opportunity to demonstrate the value of recent rehabilitation work and such further work as may be proposed for the immediate future.

*Entrance channel realinement*

As may be noted from recent condition surveys, generally as shown on figure 9, the authorized entrance channel takes a southwesterly course through the outer bar as it proceeds seaward through the Columbia jetties. Although dredging on the outer bar on this alinement is not now significant, heavy dredging is required to maintain the adopted channel through the inner bar within the jetties. Just west of the authorized channel hydrographic surveys reveal a natural parallel channel of even greater depth, which appears to offer a possibility of decreasing dredging requirements if, through some means, advantage might be taken of the greater depths afforded, without introducing an undesirable risk to navigation.

In connection with model studies of different channel alinements at the Southwest Pass entrance to the Mississippi River, it was found that the introduction of a degree of sinuosity into the channel alinement took advantage of the prevailing river and ocean forces to obtain a more satisfactory entrance channel. Such an expedient, in combination with realinement of the entrance channel, might have definite merit at the Columbia Entrance and will be tested in the comprehensive model.

*Salinity intrusion and shoaling*

The model will accurately define the extent of salinity intrusion in the Columbia Estuary under the different conditions of tide and upland discharge and will show the part such intrusion plays in the development of the shoaling regime of the estuary and entrance. The movable-bed tests will show the mechanism responsible for the formation of Clatsop Spit, the forces involved in the great imbalance of offshore materials to the north and south of the present entrance, and the role that alongshore littoral forces play in the formation of these large shoal and attrition areas. Once these forces are clearly understood and their interrelationships evaluated in the model, tests can be made of structural measures designed to take advantage of these forces in the development of an entrance channel that will be easier and less costly to maintain.

*Reduction of entrance width*

Although early engineers were guided by observations of existing bank-to-bank widths in the estuary in determining the present width between the ends of the jetties, approximately 2 miles, it has long been felt by the present-day engineers that these structures were too far apart to guide efficiently the forces at work to produce an easily maintained entrance channel. In the model it will be possible to test the value of more confining structures, such as would result from extension of the alinement of the original South Jetty past the "knuckle." Other confining works, such as groins constructed perpendicular to the South Jetty, could also be tested and evaluated.

*Jetty B*

One of the early tests in the model will evaluate the merit of jetty B as presently authorized by the 48-foot entrance project. The 1959 prototype measurement program revealed the existence of generally upstream currents at all times along the bottom of the "cul-de-sac" in which jetty B would be located. Such a structure could possibly tend to cause undermining of the existing North Jetty through reflection of the prevailing bottom upstream currents toward that structure. The merits as well as the possible adverse effects of jetty B will be explored fully in the model.

*Other tests*

The above brief discussion outlines some of the early tests proposed for the comprehensive

Columbia Estuary and Entrance model. Doubtlessly, numerous other tests will be made as new problems arise through increased growth and development of the Columbia Basin. Not the least of these will be that caused by increased pollution of Columbia River waters through domestic and industrial waste disposal. It may be expected that the Columbia model, like models of estuaries elsewhere, particularly along the heavily developed eastern seaboard, will define the flushing pattern peculiar to the Columbia Estuary and will point the way to solution of this problem as well as other problems not now envisioned by engineers of this day.

## Summary of Present Knowledge

The vastness of the lower Columbia Estuary and Entrance area is only one characteristic that belies the complexity of the problem of achieving and maintaining a reasonable degree of permanence insofar as navigation is concerned. The problem is further complicated by the dynamic forces of ocean salinity, littoral drift, river flow, and storm waves that are primarily responsible for the geographical formation of the area and the interactions taking place therein. These characteristics, together with the extremely large and variable range of diurnal tidal action prevailing along the Oregon and Washington coasts, give to the Columbia Estuary and Entrance a uniqueness beyond compare. The tremendous amounts of energy expended by these outsized forces in the entrance and estuary area exceed those expended in any similarly improved area anywhere in the world today. Past efforts to regulate these forces in the interest of navigation have led to construction of massive and long, stone training structures, the North and South Jetties, which confine the ship channel to a relatively limited area but do not assure adequate depths for navigation without continual dredging year after year.

Comparison of depths shown by hydrographic surveys undertaken since the beginning of improvement reveal that the estuary is gradually becoming shallower, being filled with bottom materials carried by the river and transported from adjacent ocean areas. Clatsop Spit shoals, which continually encroach on the present entrance channel, constitute the most striking evidence of this progress. Other evidence, although less spectacular, such as the changes occurring immediately upstream from these shoals, confirm this trend. Measurements of current direction and velocity, as well as salinity and temperature, at different depth levels throughout the estuary reveal the existence of a salinity block or salt-water wedge action along the bottom through the entrance, which not only prevents the movement of bottom materials to the sea but encourages the movement of ocean materials into the estuary. This intrusion of salinity into the estuary, an incidental byproduct of improvement, now controls the regimen of the estuary area.

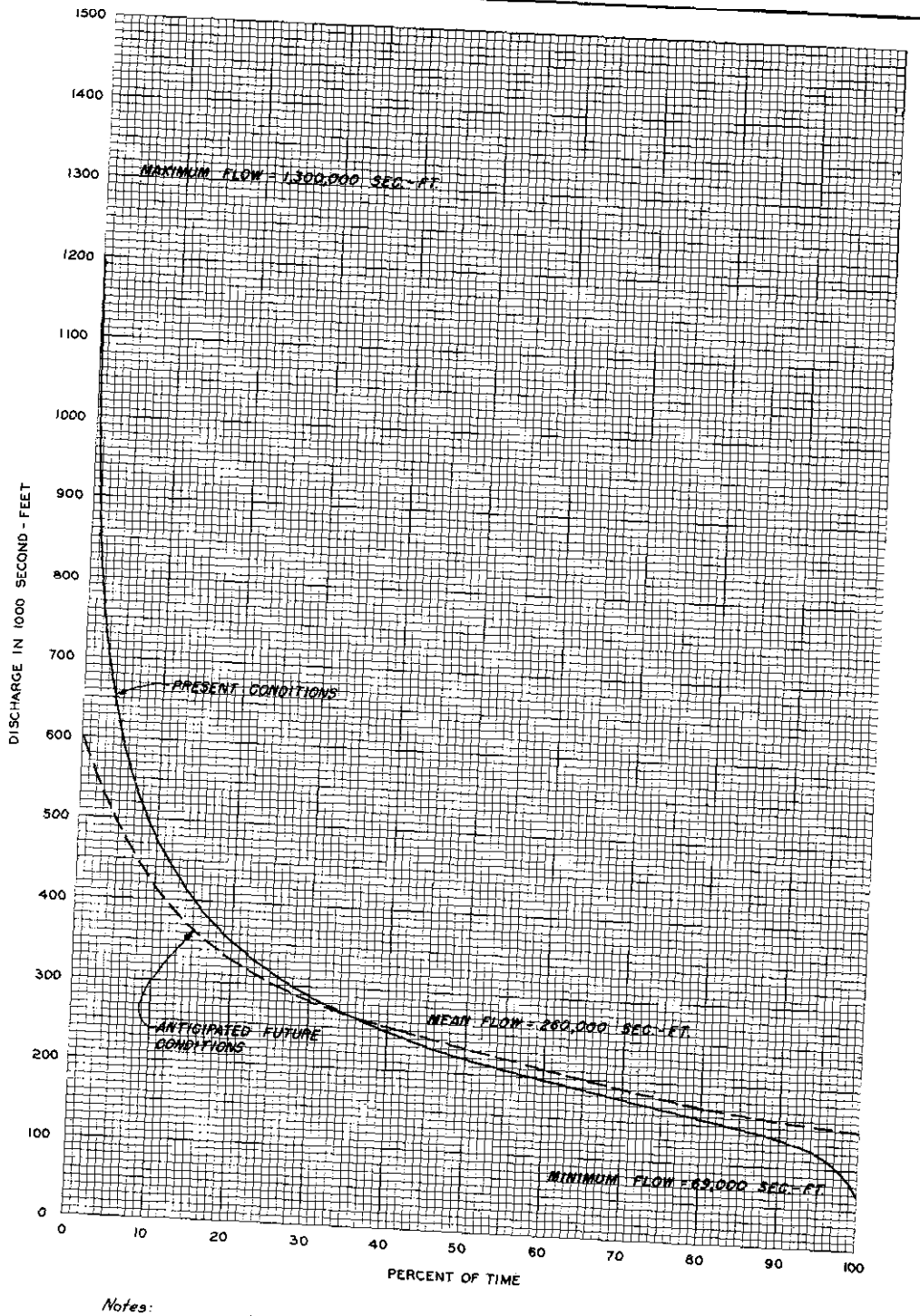
Another incidental effect of improvement, interception of the predominantly north to south alongshore littoral drift by the North Jetty, has created a severe imbalance of materials immediately offshore the entrance. This imbalance threatens not only to pinch off the present entrance channel by the accumulation of littoral material from the north but, with the assistance of storm waves, also threatens to produce a major relocation of the river outlet to the south, where attrition of the shore and offshore area has weakened the confining influence of the South Jetty.

The comprehensive hydraulic model of the Columbia Estuary and Entrance, now undergoing verification tests at the Waterways Experiment Station, offers the sole means of developing an engineering analysis and evaluation of the forces responsible for the pending cataclysm in this area and of determining the action which might be taken to prevent such an eventuality.

## Considerations for the Future

Achievement of all feasible storage in the Columbia River headwater areas in the primary interests of flood control, hydroelectric power, and irrigation will alter materially the pattern of river flows in the lower estuary. As shown on figure 28, these flows have in the past ranged from a minimum of 59,000 c.f.s. to a maximum of about 1,300,000 c.f.s. Regulation by such storage will reduce the future range of these flows by lowering extreme discharges to not more than 600,000 c.f.s. and by increasing minimum flows to about 150,000 c.f.s. This alteration of the river flow pattern may be expected to eliminate any possible flushing of bottom sediments from the estuary to the sea by extreme river discharges of the magnitude that has been experienced in the past.

On the other hand, the increase in low river flows may, to a small degree, reduce the extent of salinity intrusion in limited parts of the lower estuary. While this regulation of river flow is not expected to modify appreciably the total amount of fresh water entering the estuary, it will, by reduction of extremely high discharges, tend to maintain the permanency and quite likely increase the degree of salinity intrusion into the estuary. Also, as revealed by experience gained at the Savannah Estuary, such regulation of upland flows may tend to concentrate shoaling to a much more limited length of the Columbia Estuary.




Notes:

1. This flow-duration curve is for Columbia River at mouth.
2. Maximum, minimum and mean values are computed from observed data.
3. Drainage area is approximately 259,000 sq. mi.

FIGURE 28. — Flow-duration curve.

History of the Pacific Northwest reveals that channels of greater and greater depth are required to support the increase in ocean commerce attending the continued growth and economic development of this region. Due to the geography of the entrance area and the severe wave and weather conditions that prevail in that area during the winter months, depths of 48 feet are necessary for safe passage through the Columbia Entrance, while the main Columbia channel inland now provides only a depth of 35 feet. Recent increases in the size and draft of the world fleet of ocean carriers have revealed a need for increasing the minimum depth of the main river channel above the entrance to Portland and Vancouver from 35 to 40 feet, and plans to effect such further deepening of the main channel are now underway.

Although no corresponding deepening of the entrance channel is planned at present, due to the fact that elimination of delays caused by the 48-foot entrance channel to prospective traffic will not now justify such further deepening, it is obvious that eventually, as the region continues to grow, greater entrance depths will be required. Provision of the contemplated 40-foot channel in the river above the entrance is expected to extend the effect of salinity intrusion farther into the estuary, causing changes in shoaling patterns and increasing dredging to maintain the larger channel dimensions. Any further deepening of the present entrance channel will intensify saline conditions within the estuary and certainly modify, and quite likely aggravate, the shoaling problem.

As previously mentioned, pollution of the estuarine waters is not now a serious problem facing those charged with maintaining the quality of Columbia River water. However, with the expected growth in population and industrial development in the area tributary to this great river, experience elsewhere, particularly along the Atlantic seaboard, points to the eventual need for strict control of pollutants disposed in the Columbia River. The extent of such control will largely be governed by the flushing characteristics of the Columbia Estuary as affected by improvements designed primarily for the sole benefit of navigation. It does not appear unreasonable that the degree of pollution induced into the river may some day exceed the flushing capacity of the estuary.

All these problems are susceptible to study, analysis, and evaluation in the comprehensive hydraulic model of the lower Columbia Estuary and Entrance. There is little doubt that the model will contribute greatly to the solution of these and other problems of an unforeseeable nature at this time. In the event the model demonstrates the merit of a planned major relocation of the entrance channel through the South Jetty, it appears probable that many of the problems created by salinity intrusion into the estuary may also be solved or greatly alleviated.

Such a planned relocation would involve construction of a navigation canal in the Point Adams area of the dimensions required by ocean commerce connecting the estuary at that point with the deep ocean area just south of the present South Jetty. A lock might be required in this canal at Point Adams to eliminate or reduce the intrusion of salinity into the estuary from the canal. Also, a new but substantially shorter South Jetty may be required to fix firmly the entrance channel between this new structure and the enrockment forming the present South Jetty. With such a relocated entrance channel and salt-water lock, the present entrance would be abandoned, closed to navigation, and allowed to shoal naturally. If shoaling in the present entrance channel reduces controlling depths to the equivalent of those prevailing prior to 1885, it may be expected that the intrusion of salinity into the estuary will be correspondingly reduced until it is no longer a controlling factor in the estuarine regime.

With the reduction of depths created by shoaling it may be expected that, even with the minimum regulated flow of about 150,000 c.f.s., river flows will completely usurp the entire channel section at all times and there will no longer prevail the marked density currents that are now so characteristic of the intrusion of salinity into the estuary. At that time, except for the modification of the river flow pattern resulting from upstream storage, the restored flushing characteristics of the river would move the bottom sediments to the sea in a manner similar to that which prevailed prior to man's attempt to improve the entrance. The freshwater estuary thus obtained will be capable of more complete flushing of pollutants, and it is reasonable to expect that dredging requirements for navigation will be considerably less demanding.

Along with such achievement there would be restored the uninterrupted predominantly north to south littoral drift past the present entrance that would nourish and restore the starved shoreline immediately to the south. Although restoration of the natural littoral regime may eventually require dredging of the relocated entrance channel, this work will not be required for many years due to the depths now existing in that area. Thus, with restoration of conditions which largely prevailed prior to improvement, the strategic value of the Columbia Estuary and Entrance area may be assured throughout the foreseeable future at reasonable costs.

## Acknowledgment

The discussion and data presented herein were obtained from research conducted under the Engineer Planning Program of the U.S. Army Corps of Engineers. The permission granted by the Chief of Engineers to publish this information is appreciated.

# MEANS AND METHODS OF INDUCING SEDIMENT DEPOSITION AND REMOVAL

[Paper No. 69]

By JOHN W. HARRIS, *supervisory hydraulic engineer, U.S. Army Engineer District, Savannah, Ga.*

## Synopsis

Maintenance of Savannah Harbor has become progressively more critical as the deepening of the channels caused heavy shoaling concentrations in a highly industrialized section of the harbor where spoil disposal areas are limited. The Savannah Harbor model study developed means of inducing sediment deposition in areas where more economical means of removal could be utilized. The hydraulic changes expected to be produced by the proposed plan of improvement and their effects on shoaling, as well as their effect on the efficiency and cost of harbor maintenance, was discussed.

## Introduction

Savannah Harbor, Ga., is located in the lower 21 miles of the Savannah River. Maintenance of Savannah Harbor has long been a problem of concern to navigation interests and an item of great expense to the Government. Concurrent with the improvement of the harbor, each successive deepening has caused the shoaling to shift upstream. After the harbor was deepened to 34 ft. in 1947-48, shoaling concentrated in channels adjacent to the city of Savannah, where only limited areas are available that can be used for spoil disposal. To provide adequate maintenance of the channel without excessive increases in cost, it was evident that action to reduce or relocate the shoaling would be required to insure adequate maintenance of the channels at reasonable cost. The Savannah Harbor investigation and model study was performed for this purpose. The model studies have developed a plan to reduce shoaling in the navigation channels and to induce the deposit of most of the shoal material in a sediment trap that would be located adjacent to adequate spoil disposal areas.

## Description

Savannah Harbor (fig. 1) is located in the tidal estuary where the Savannah River empties into the Atlantic Ocean. Channel locations refer to 1,000-ft. stations with the upper limit of the harbor at station 81+499 and the lower limit at station 245+000, which is generally referred to as station 245. Similar reference is made to other 1,000-ft. stations throughout the harbor. The authorized maintenance project provides for a channel 36 ft. deep at mean low water and 500 ft. wide from deep water in the ocean across the bar to station 194 (harbor entrance); thence 34 ft. deep and generally 400 ft. wide to station 94 (Georgia Ports Authority terminals); thence, 30 ft. deep and 200 ft. wide to the upper limit (Atlantic Coastal Highway bridge). Turning basins are located in the vicinity of stations 83, 90, 95, and 104. In the vicinity of the Atlantic Coast Line Railroad terminals, the channel is 550 ft. wide. The navigation channels in the inner harbor are located in North Channel and Front River. The Atlantic Intracoastal Waterway crosses North Channel at about station 165. The center of the city of Savannah is located on the south side of Front River at about station 120 with industrial developments extending to the upper limits of the harbor.

## Shoaling

The annual shoaling rate for the inner harbor averages about 7,000,000 cu. yd. The greatest concentration occurs in Front River opposite the city of Savannah, as shown on figure 1, where over 50 percent of the shoaling takes place between station 117 and station 136. The shoaling shifted upstream to its present location after the deepening of the channels to 34 ft. in 1947-48. Figure 2 shows a graphical comparison of the shoaling before and after the deepening. The shoal material is primarily silt and clay with very little sand-size material.

## Maintenance

Pipeline dredges are used to maintain the inner harbor channels. A Government dredge is normally assigned to the district, although frequently it is necessary to obtain the services of a privately owned dredge under contract to assist the Government dredge. The present maintenance program for the inner harbor (station 194-81+499) costs over a million dollars annually and requires an average of 11 months work annually by Government plant and 2 months work each second year by contract. Dredging requirements are based on the present

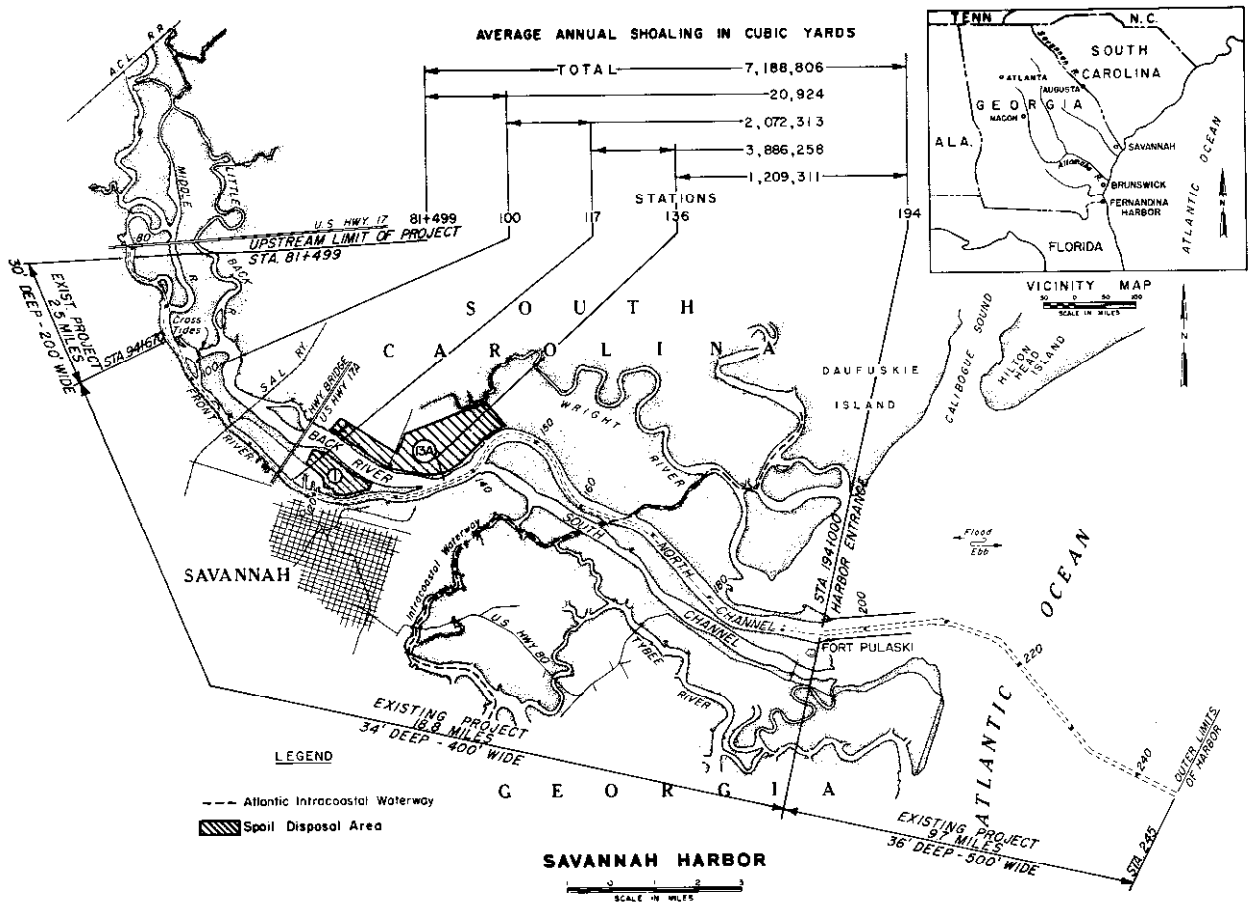



FIGURE 1. — Existing project and annual shoaling.

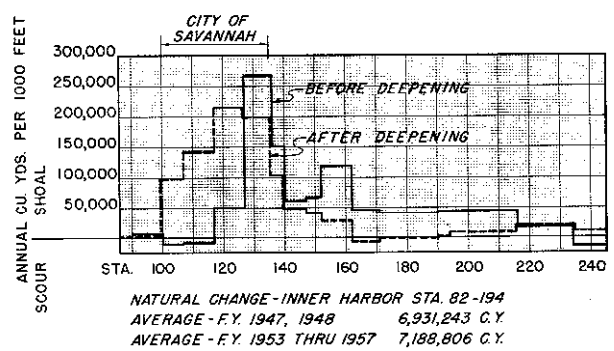



FIGURE 2. — Annual shoal and scour (natural change) before and after deepening navigation channels.

maintenance policy, which provides that dredging be performed when an area with authorized project depth of 34 ft. has shoaled to about 30 to 32 ft. Maintaining full project dimensions would require more frequent dredging than the present policy. Maintenance of the bar and jetty channels is performed with Government-owned hopper dredge. The annual shoaling on the bar (station 194-245) amounts to about 700,000 cu. yd.

## Spoil Disposal Areas

The shortage of spoil disposal areas adjacent to the rapidly shoaling channels has greatly increased the cost of channel maintenance. Spoil area No. 1 (fig. 1) has been virtually filled and area 13A has been diked and is now being used for maintenance of the major shoal sections of the channel, although pipelines in excess of 10,000 ft. are required for maintenance of some channel sections.

## Description of the Savannah Harbor Model

Approximately 413 square miles of prototype area were reproduced in the Savannah Harbor model, including the Atlantic coast from Hilton Head Island to Wassaw Sound and offshore areas well beyond the 40-ft. contour of the ocean; up the Savannah River about 50 miles to the head of tidewater at Ebenezer Landing; the extensive system of tidal tributaries, salt-water

creeks, and boundary marshes that affect tidal action throughout the harbor; and the Atlantic Intracoastal Waterway from Calibogue Sound through Skidaway River. The model was constructed on scale ratios, model to prototype, of 1:800 horizontally and 1:80 vertically. It was equipped with necessary appurtenances to reproduce and measure all pertinent phenomena such as tidal elevations, salt-water intrusion, current velocities, fresh-water inflow, pollution, and shoaling. The model was constructed to conform to the prototype conditions that existed in 1950. Hydraulic and salinity verification was based on extensive protoype measurements made in 1950 and 1951. Shoaling verification was based on the quantitative distribution for the years 1953-54, which is representative of prevailing conditions for the existing 34-ft. project depth.

## Shoaling Tests

The procedure developed and used in the general investigation for duplicating prototype shoaling involved the injection of a measured mixture of ground gilsonite and water into the model and pooling the water after a prescribed number of tidal cycles. The gilsonite that settled within the channel limits was retrieved and measured and the results expressed by a percentage distribution curve. An acceptable reproduction of prototype shoaling was attained, since exact duplication of the prototype shoaling was neither possible nor necessary. At the termination of a shoaling test all material in motion was deposited immediately in place. Dredging and spoiling techniques that are believed to have a definite effect on the prototype shoaling quantity and rate could not be reproduced in the model. However, it was not necessary to obtain exact reproduction of prototype shoaling, since the basic model shoaling distribution curve was used for evaluating the effectiveness of proposed improvement plans.

## Back River Sediment Basins

The results of the verification and preliminary tests confirmed that sediment basins might offer a possible means for reducing maintenance costs if the location, entrance, and hydraulic conditions of such basins could be designed to attain the maximum possible rate of shoaling in them and if the basins could be located adjacent to adequate spoil disposal areas. Back River is ideally located for construction of a sediment basin. It has no commercial traffic, it is located near the major shoal area, it has no industrial development on either side, and the Government has perpetual spoilage easements on extensive diked areas adjacent to the proposed location for the sediment basin. A series of tests were made to determine the most favorable conditions for a sediment basin and the effect of such a plan on the hydraulic regimen of the harbor.

## Development of a Plan

The effectiveness of each plan was judged primarily by the ability of the plan to reduce shoaling in the Front River navigation channel between stations 82 and 135 and to concentrate shoaling in the sediment basin. The model tests of a sediment basin, located in the lower part of Back River, 1,500 ft. wide and 2 miles long with an entrance channel in the mouth of Back River, reduced the shoaling in the navigation channel by about 20 percent. Since the reduction in channel shoaling occurred downstream from the entrance to Back River instead of in the critical shoaling area upstream from that point, little benefit would be realized.

Two tests were made with the entrance channel to the sediment basin located in the immediate vicinity of the shoaling concentration and with constriction dikes in the mouth of Back River, which were necessary to make the plans effective. One test was made with the entrance channel located near the middle of the major shoal area at station 125 and with a 150-ft. opening between the constriction dikes as shown on figure 3. The sediment basin retained 35 percent of the shoaling, and the navigation channel shoaling was reduced to 64 percent, but the reduction occurred downstream from the entrance to the basin with a slight increase in the upstream quantity. The other test was made with the entrance channel located near the upstream end of the major shoal area at Station 116 and with a 200-ft. opening between the constriction dikes, as shown on figure 4. The shoaling in the navigation channel was reduced to 65 percent while the sediment basin retained 38 percent. Although these plans were successful

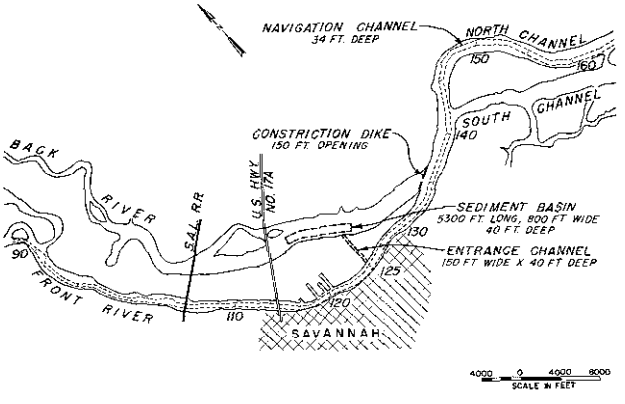



FIGURE 3.—Sediment basin with entrance channel at station 125.

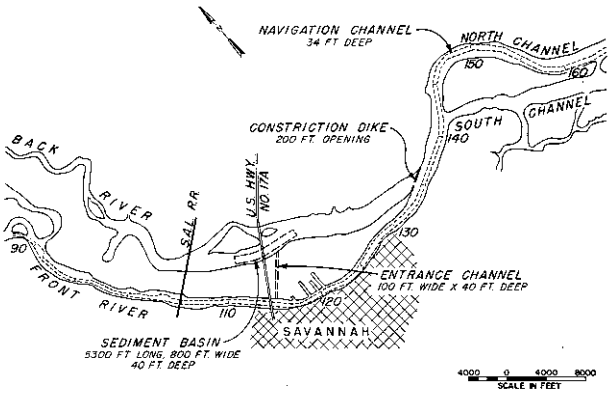



FIGURE 4. — Sediment basin with entrance channel at station 116.

to some degree, substantial shoaling would still occur in the area where spoil areas are critical.

Tests were made with a 300-ft. wide entrance channel in the mouth of Back River and a tide gate in Back River above the SAL Bridge about 1.5 miles upstream from the sediment basin, with a canal connecting the Back River tidal prism, above the tide gate, with Front River. The tide gate served to reduce the ebb flow through the sediment basin and also to increase ebb flows in Front River to prevent shoaling. This plan (fig. 5) was very effective, but the

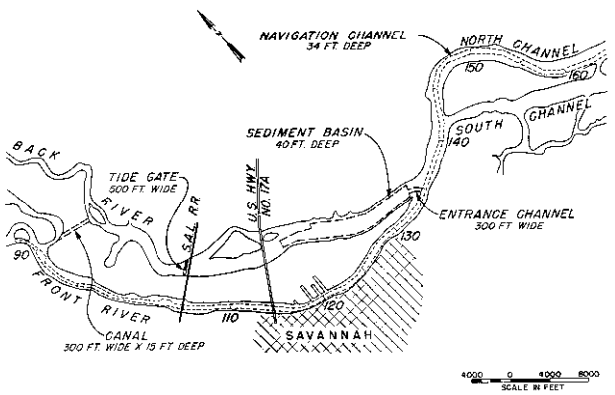



FIGURE 5. — Sediment basin with tide gate in upstream location.

sediment basin was enormous and the initial construction cost too high to produce a favorable economic ratio. It was determined by additional tests that the sediment basin could be reduced to a width of 600 ft. and still be reasonably effective. Up to this point the tests were made for the present authorized channel depth of 34 ft.

To determine the effectiveness of the sediment trap-tide gate plan with anticipated future harbor improvements, tests were run that incorporated deepening of the channels to 40 ft. These tests indicated that the ebb velocities in Front River were inadequate to prevent shoaling. The tide gate was then shifted 1.3 miles downstream to a point at the upstream end of the sediment basin to increase the volume of the Back River tidal prism, which discharged into upper Front River. This arrangement was satisfactory for the 40-ft. deep channel; however, when it was tested with the 34-ft. depth in the navigation channel, spring tide velocities in Front River were observed that would be hazardous for navigation. This condition was corrected by placing a side opening in the tide gate, which reduced the effective volume of the Back River tidal prism. These tests showed conclusively that the sediment trap-tide gate plan could be used for present project channels and also adapted to future harbor developments.

## Side Opening in the Tide Gate

The side opening in the tide gate structure, which is required to prevent excessive velocities in the existing navigation channel, demonstrates the hydraulic mechanics that reduce the shoaling in Front River. An opening of 500 ft. was successful in maintaining the ebb velocities compatible with the requirements of navigation, but the plan was not effective in reducing the channel shoaling nor for inducing shoaling in the sediment basin. An opening of 200 ft., however, was effective in reducing shoaling in upper Front River; but the sediment basin was relatively ineffective. An opening of 100 ft. was effective in reducing the shoaling in Front River, making the sediment basin effective and preventing excessive velocities in Front River.

## The Sediment Trap-Tide Gate Plan

The most effective plan was phase 8, scheme 6, test 4 (fig. 6), which requires a sediment basin in Back River 600 ft. wide, 40 ft. deep,

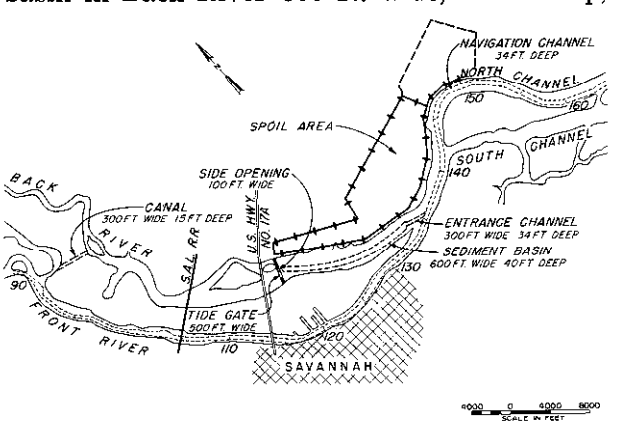



FIGURE 6. — Sediment basin with tide gate in downstream location (phase 8, scheme 6, test 4, and test 10).

and about 2 miles long; a tide gate in the vicinity of Highway 17A; and a canal connecting Back River to Front River in the upper part of the harbor. The entrance channel to the sediment basin is 300 ft. wide and 34 ft. deep. During the shoaling tests, at the end of the prescribed number of tidal cycles, a stable condition had not been reached in the navigation channels and sediment was still actively entering the sediment basin, whereas, in other tests the sediment had stabilized. Test 10 was then performed which extended the shoaling test until stability was reached. This plan reduced the shoaling between stations 82 and 135 from 77 to 5 percent and between station 135 and station 205 from 23 to 9 percent.

## The Tide Gate Structure

The tide gate structure was provided to increase the efficiency of the sediment basin by reducing ebb flows through the basin. A 500-ft. gated opening was provided in the structure that opened automatically to allow flood flows to pass through but which was closed during the ebb periods. A canal was provided to drain the Back River tidal prism above the tide gate structure into Front River, thus increasing ebb flows in Front River. During spring tide periods it would be necessary to pass a part of the water from the Back River tidal prism through the sediment basin to prevent high velocities in Front River that might be dangerous to navigation. The shoaling tests were conducted with a fixed side opening 100 ft. wide in the tide gate structure; however, the efficiency of the sediment basin probably could be increased by providing an opening only during the spring tides that produce excessively high velocities.

## Sediment Basin Maintenance

Based on the present harbor shoaling rate and the model shoaling tests, it will be necessary to dredge about 6 million cu. yd. annually from the sediment basin. To retain the effectiveness of the sediment basin, it will be necessary to dredge it annually or probably continuously, with periodic maintenance dredging in the navigation channels. The redistribution of the shoaling material is expected to produce considerable savings over the present maintenance cost because:

(1) If a conventional pipeline dredge is used, reasonably short pipelines would be required, since the spoil area is located adjacent to the basin, whereas long lines are now required to maintain navigation channels.

(2) Fixed pipelines could be utilized to eliminate much labor in handling.

(3) The dredge would be out of the navigation channel, which would eliminate loss of time permitting vessels to pass.

(4) The operation of the dredge would be confined to limits of the sediment basin, eliminating the costly movement of the dredge over many miles of navigation channel.

(5) It would be possible to let the sediment consolidate to an optimum density for efficient removal from the basin; whereas, the present specified depths in the navigation channel, which must be maintained, often necessitates dredging material that has not consolidated sufficiently for economical removal.

The savings in labor and increased efficiency of dredging the material from the sediment basin would produce appreciable reduction in the maintenance cost. Preliminary economic studies for the plan indicate a favorable benefit-cost ratio. Although no specific plans have been made, it is contemplated that some means other than a conventional pipeline dredge can be devised to maintain the sediment basin, such as a simple arrangement of pumps on barges with drags similar to the hopper dredge or dustpan-type intake, or semimobile dredge powered by electricity. Since the preliminary economic studies have been based on the maintenance of the sediment basin with a conventional pipeline dredge, it is almost certain that final studies will indicate a more favorable benefit-cost ratio for the plan.

## Improved Channel Maintenance

The present maintenance of the navigation channels requires the services of a 24-inch pipeline dredge almost continuously to dredge the 7 million cu. yd. annually at a cost of about $1.25 million. The sediment trap-tide gate plan would reduce the channel shoaling to the extent that dredging would be required only periodically and channel conditions would be greatly improved. The project channel depths, which would be almost self-maintained, is a very important benefit.

## Verification of Model Shoaling Test With Prototype Data

The shoaling tests for the sediment trap-tide gate plan are partially verified by prototype data, which indicate that Savannah River discharges of 16,000 c.f.s. or greater tend to shift the concentration of shoaling downstream and that discharges of this magnitude, if continued for an appreciable length of time, have a pronounced effect on shoaling in the harbor.

Figure 7 shows Savannah River discharges for about a 1-year period with center-line profiles in the harbor during various flow conditions. Fluff (unconsolidated sediment) is present in areas susceptible to shoaling. During a period of normal discharges up to 10,000 c.f.s. the fluff extended from station 103 to station 140. Moderately high discharges shifted the

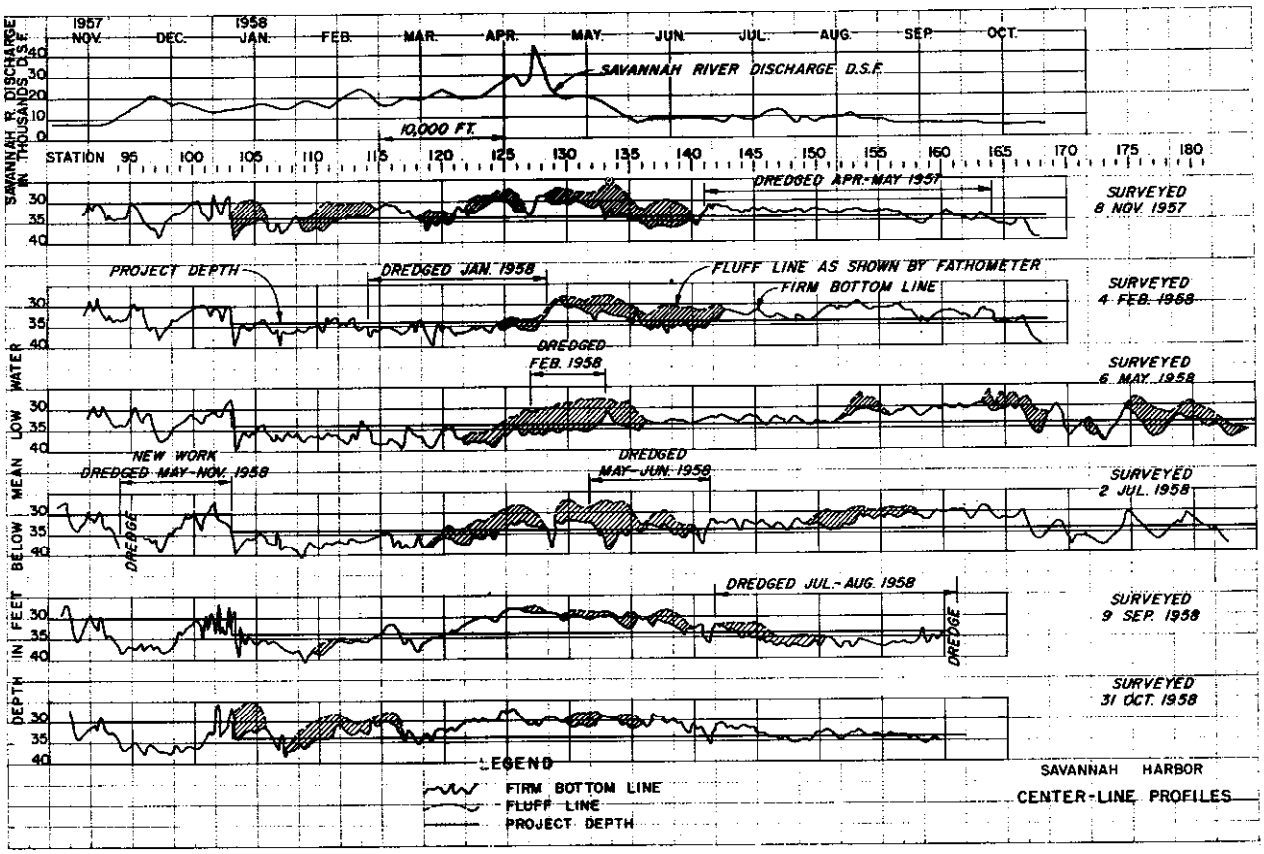



FIGURE 7. — Effect of Savannah River discharges on shoaling.

fluff downstream, and after several months of high flows the fluff extended from station 122 to station 184. Then when the discharges returned to normal the fluff moved back upstream with rapid consolidation of material occurring between station 120 and station 140. The sediment trap-tide gate plan produced hydraulic conditions in the model that eliminated shoaling in Front River similar to the effect produced by high discharges.

## Conclusions

Shoaling, which the deepening of the Savannah Harbor shifted to a section where spoil disposal areas are limited, prompted model studies which have demonstrated means of inducing shoaling in a sediment basin which is located adjacent to adequate spoil areas.

The most advantageous plan developed provides for a sediment basin and tide gate in Back River. In the model this plan was successful in increasing ebb flows in Front River, which shifted the shoaling from this critical area to the sediment basin. The model tests indicated that 89 percent of the shoaling would be induced in the sediment basin.

Considerable savings in harbor maintenance would be realized through the efficient dredging methods that would be possible in maintaining the sediment basin.

# SEDIMENTATION IN THE SAN FRANCISCO BAY SYSTEM

[Paper No. 70]

By BERNARD J. SMITH, *civil engineer, U.S. Army Engineer District, San Francisco*


## Abstract

The San Francisco Bay system is a land-locked series of embayments of 430 square miles. The annual sediment load approximates 8 million cubic yards, 85.5 percent from the Sacramento-San Joaquin River system and 14.5 percent from Bay area streams. Upon entry into the Bay system, the character of the sediment inflows change. The density of the bedload decreases from 90 to 31 lb. per cu. ft. for San Pablo Bay, 49 lb. per cu. ft. for Central Bay, and 88 lb. per cu. ft. for Suisun Bay; and the sand content is lost and the silt content increased an additional 14 percent. The clay content remains constant at about 57 percent. For fresh water inflows of around 30,000 c.f.s., the dilution factor on entering the system is about 1:54. However, no discernible drop in parts per million of sediment load has been observed. It is concluded that tidal- and wind-wave-induced turbulence erodes bottom deposits and inhibits settlement. Contribution of the supplemental supply from littoral erosion outside the Golden Gate is discounted. Historical sedimentation evidences cyclical trends and interactioning within the system. The effects of wind action on marginal and intermediate shoals are noted. Massive displacements of sediment volumes would drastically affect the biota of the estuary.


## General

The sedimentation pattern, a critical economic factor for an estuary of commercial rank, becomes of added importance in a system such as the San Francisco Bay complex in which environmental change may have far-reaching consequences on the biota of the region. The Bay system is the gateway in the spawning season for salmon runs to the headwaters of the Sacramento River system. It also is a strategic landing area for wildfowl on the Pacific flyway. These aspects are important to the regional economy and of increasing import to the State and Nation.

The complex of delta, bays, and straits making up the San Francisco Bay system is the tidal terminus of an estimated annual mean natural runoff of 34,881,000 acre-feet (*1*). This flow approximates 49 percent of the estimated total natural mean annual runoff of the State of California. Concomitantly, the system is the natural receiving basin for large sediment inflows. However, changes have been extensive, in the last 100 years particularly, so that the full natural runoff is not discharged into the system, much being diverted, and the sedimentation pattern consequently has changed. In addition, cyclic phenomena, such as droughts and floods, underlie some fluctuations in sediment inflow, deposition, and scour.

## The Source and Character of the Sediment Inflow

Figure 1 gives the soil conditions over the contributory watersheds and the historical outflow into the delta, the component bays, and the discharge through the Golden Gate for the period 1924–60, taking into account the operation of the reservoirs constructed in the period. Table 1 gives the percentage breakdown by five classes of soil cover for the tributary sediment source area.

TABLE 1.—*Soil classification of sediment source area tributary to Suisun Bay, San Francisco Bay System*[1]

| Soil type | Area | Percent of total |
|---|---|---|
| | *Square miles* | |
| Sandy loams | 23,305 | 39.5 |
| Loams | 6,490 | 11.0 |
| Clay loams | 21,004 | 35.6 |
| Clays | 3,127 | 5.3 |
| Variable | 5,074 | 8.6 |
| Total | 59,000 | 100.0 |

[1] Data obtained from reference (*2*).

The effect of diversions for municipal, industrial, and agricultural use, together with transfer of flows south through the Delta, is apparent in the 37-year average of historical outflows to the Pacific — an outflow through the Golden Gate of 21 million acre-feet as against a full natural estimated average of 35 million acre-feet. Although sediment inflow would not decline in strict proportion to declining outflow, nevertheless a substantial drop would occur.

Reservoir retention and regulation of flow, however, does not necessarily imply equivalent reduction in sediment transport to the lower bays. Floodflow releases through dams scour a portion of the trapped sediment. In the dry season the clear water releases tend to regain full competency to carry the maximum sediment load by erosion below the dams. Daines,[1] in studying sedimentation in the Colorado River system, noted that the monthly sediment loads varied considerably, with the floodflow months showing the highest loadings. Similar observa-

[1] DAINES, N. H. STUDY OF SUSPENDED SEDIMENT IN THE COLORADO RIVER. U.S. Dept. Int., Bur. Reclam. Mimeo. 16 pp. Sept. 1949.

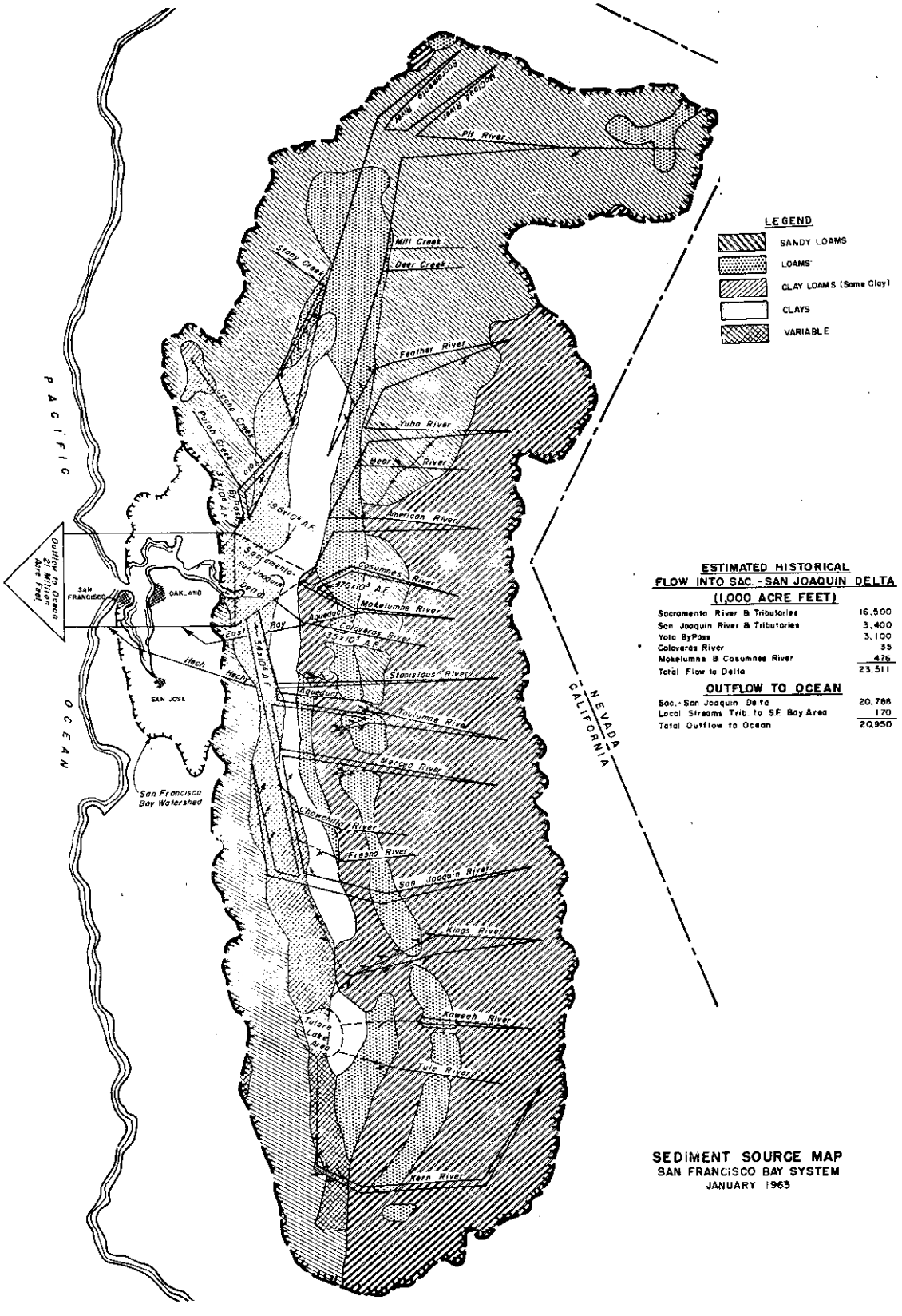



FIGURE 1. — Source of sediment of San Francisco Bay system, 1963.

tions have been recorded for Central Valley streams of California. Hence, the source of sediment can be regarded as being over the entire watershed, above and below reservoirs, with the floodflows carrying through reservoirs into downstream channels much scour material from the uplands. Figure 2 shows the major rivers and major dam sites for the Central Valley (dams constructed or authorized) and indicates the areas that normally would contribute to the sediment inflow to the Delta, i.e., areas below the lowest major dams, and the additional areas that in times of released floodflows would contribute sediment from the upland regions.

It is apparent that in years of no major flood releases, the character of the sediment inflow would reflect the soil conditions of the piedmont and alluvial fan areas in the lower elevations of Central Valley, with a high clay content and a high colloidal clay suspension (see fig. 1). In flood periods, the composition would change to reflect heavier concentrations of sand and gravel from the uplands and from scourings of channel beds. In this connection, the last upstream deposits from the hydraulic mining era should be considered. Gilbert (*7*) estimated the total debris from mining and rain erosion for the 65 years, 1850-1914, at 2,375 million cubic yards within the area tributary to Suisun Bay, distributed as shown in table 2.

TABLE 2.—*Distribution in 1914 of debris from mining and erosion from lands draining to the San Francisco bay system for the period 1850–1914*

[Estimated by G. K. Gilbert (*7*)]

| Location | Debris |
|---|---|
| | *Million cubic yards* |
| Deposits – | |
| Within the Sierra Nevada | 265 |
| In the piedmont | 520 |
| In the channels of valley rivers | 100 |
| On inundated lands, including tidal marshes | 294 |
| In the bays | 1,146 |
| In the ocean | 50 |
| Total | 2,375 |

The last three items of table 2, totaling 1,490 million cubic yards, represent mining debris and fluvial sediment that had been transported past the Delta in the bay system proper between 1850 and 1914, or a rate of movement under 23 million cubic yards per annum. The competency of the rivers were assumed to average at the same rate for 1914–60, an additional 1,054 million cubic yards would have been moved past the Delta. This would be insufficient to move the residual 885 million cubic yards remaining from the 1850-1914 period and the erosion loading that logically could be expected to increase from the Gilbert estimates of approximately 11 million yards per year when agricultural development was less intensive. However, of the 885 million, 435 million are considered by Gilbert to be permanently lodged — 100 in the mountain and 335 million in the piedmont. On this basis he concludes that within 50 years (i.e., by 1964) the mining debris load will have been cleared and erosion sediment, approximately 8 million cubic yards per annum, would constitute the sediment loading into the Delta.

This estimate by Gilbert is a remarkably close forecast of actual conditions as determined by the U.S. Geological Survey over the period 1957-59. The conclusion reached by the Geological Survey was that the annual average sediment inflow from the Central Valley and local streams would be 8 million yards, based on 8.8 million yards per annum for the 1957-59 water years (*11*).

## Estimated Future Sediment Inflow

From the preceding reports it can be inferred the mining debris from the hydraulic mining era no longer presents a problem and any contributions from the lodgments in the uplands will be negligible. Future sediment inflow will be from the normal sources. The U.S. Geological Survey investigation (*11*) determined the average daily total load for 1957-59 to be 17,000 tons and the estimated future total load as 16,000 tons. Inflow into the Delta was 14,200 tons per day, and the future inflow was estimated at 13,000 tons. The critical aspect of converting the measured and estimated sediment tonnage into volume was based on 50 lb. per cubic foot for the suspended load and 90 lb. per cubic foot for the bedload. With these conversion factors, the Delta inflows become 7.2 and 6.9 million cubic yards, respectively. For the entire system as previously mentioned, the volumes are 8.8 million cubic yards for the 1957–59 period and 8.0 million for the future.

The USGS determinations of specific weight, based on the average particle size of the material transported by the Sacramento River, were computed by methods given by Lane and Koelzer (*11*) and Hembree.[2] These methods consider the sediment as consolidated in reservoirs under varying types of operation, the initial deposition density increasing with time and with exposure to the air (for sediment exposed during drawdowns). For sand (between 0.05 and 1.0 mm.) no increase occurs with time or exposure. For clay (particles less than 5 microns) and silt (particles between 5 microns (0.005 mm.) and 0.05 mm.), the initial density

[2] HEMBREE, C. H., MELIN, K. B., BOYER, M. C., and SWENSON, H. A. 1950 PROGRESS REPORT OF THE BIGHORN RIVER DRAINAGE BASIN. U.S. Geol. Survey Water Resources Div. [1951.]

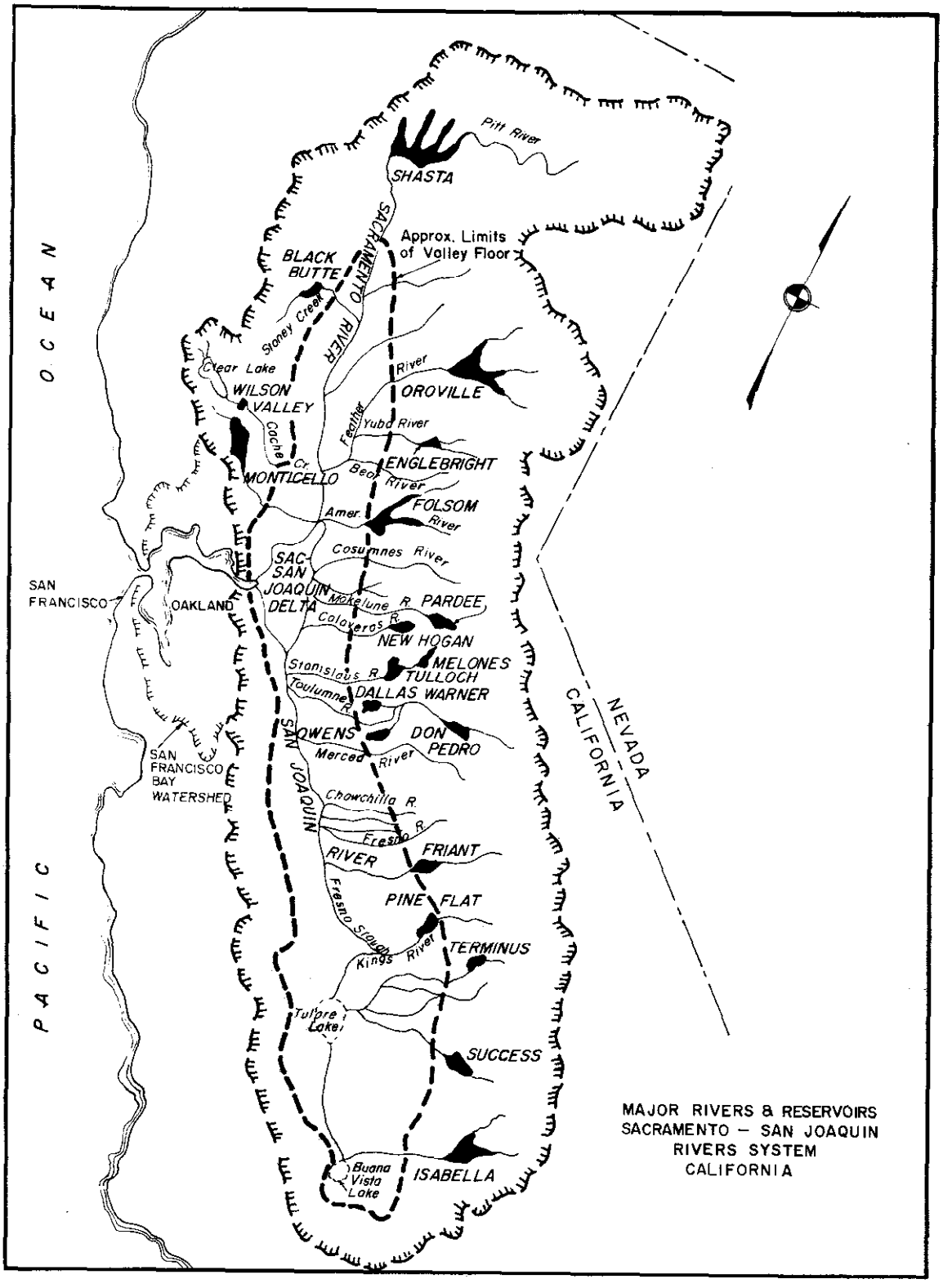



FIGURE 2. — Major rivers and reservoirs of the Sacramento and San Joaquin Rivers systems.

TABLE 3.—*Density*[1] *of sediments by material and reservoir operation*[2]

| Reservoir operation | Sand | | Silt | | Clay | |
|---|---|---|---|---|---|---|
| | *W* | *K* | *W* | *K* | *W* | *K* |
| Sediment always submerged or nearly submerged | 93 | 0 | 65 | 5.7 | 30 | 16.0 |
| Normally a moderate reservoir drawdown | 93 | 0 | 74 | 2.7 | 46 | 10.7 |
| Normally considerable reservoir drawdown | 93 | 0 | 79 | 1.0 | 60 | 6.0 |
| Reservoir normally empty | 93 | 0 | 82 | .0 | 78 | .0 |

Example: Sediment in a reservoir, useful life 100 years, with sediment 20 percent sand, 40 percent silt, 40 percent clay.

Composite density = 20 percent (93 + 0) + 40 percent (65 + 5.7 $\log_{10}$ 100) + 40 percent (30 + 16 $\log_{10}$ 100) = 93 × 20 percent + 76.4 × 40 percent + 62.0 × 40 percent = 74 lb./cu. ft.

[1] Density is indicated by symbol *W*.
[2] Values for *K* obtained from reference (*11*).

is increased by an amount equal to log time by a constant *K*. Table 3 gives values for *K* under a range of reservoir conditions (*11*).

To determine the average density of sediment inflow delivered to the perimeter of the Delta and the component bays of the system, 58 analyses of particle sizes were studied for distribution in the ranges up to 0.004 mm. for clay, 0.004 to 0.064 mm. for silt, and 0.064 to 0.500 mm. for sand. When unit weights taken from table 3 were assigned for sediment always submerged and any computation factor was omitted, the average unit weight was determined at 48.84 lb. per cubic foot. Table 4 gives the derivation of this figure. Since the California Division of Water Resources arrived at 50 lb./cu. ft. and the USGS, although arriving at a figure of 49 lb. per cu. ft., also adopted 50 lb./cu. ft. as the average density in view of the limited number of analyses concentrated in February 1959 and February 1960, 50 lb./cu. ft. was adopted in this study. Similarly, 90 lb./cu. ft. used by the USGS in this report was adopted for the weight per cubic foot of bedload.

TABLE 4.—*Sediment inflow analysis for San Francisco Bay system*[1]

| Number of samples | Month and year | Location | Percent clay | Percent silt | | Percent sand | | |
|---|---|---|---|---|---|---|---|---|
| | | | <0.004 mm. | 0.004 to 0.016 mm. | 0.016 to 0.064 mm. | 0.064 to 0.125 mm. | 0.125 to 0.250 mm. | 0.250 to 0.500 mm. |
| | | | *Pct.* | *Pct.* | *Pct.* | *Pct.* | *Pct.* | *Pct.* |
| 2 | Feb. 1959 | San Joaquin at Vernalis | 54 | 30 | 0 | 16 | ..... | ..... |
| 3 | Feb. 1960 | do | 67 | 13 | 18 | 2 | ..... | ..... |
| 7 | Feb., Mar. 1959 | Sacramento at Sacramento | 26 | 11 | 17 | 14 | 23 | 9 |
| 3 | Feb. 1960 | do | 38 | 19 | 25 | 7 | 11 | ..... |
| 2 | Feb. 1959 | Yolo Bypass at Woodland | 80 | 16 | 3 | 1 | ..... | ..... |
| 2 | Feb. 1960 | do | 74 | 19 | 3 | 2 | 1 | 1 |
| 4 | Feb. 1959 | Alameda Creek near Niles | 56 | 27 | 12 | 5 | ..... | ..... |
| 4 | Feb. 1960 | do | 72 | 17 | 9 | 2 | ..... | ..... |
| 5 | Feb. 1959 | Napa River near St. Helena | 49 | 15 | 9 | 4 | 11 | 12 |
| 8 | Jan.-Mar. 1960 | do | 38 | 20 | 18 | 8 | 16 | ..... |
| 4 | Jan., Feb. 1959 | Sonoma Creek, Boyes Hot Springs | 29 | 12 | 22 | 13 | 11 | 13 |
| 5 | Jan.-Mar. 1960 | do | 45 | 20 | 17 | 8 | 7 | 3 |
| 3 | Feb. 1959 | Walnut Creek at Walnut Creek | 62 | 10 | 19 | 8 | 1 | ..... |
| 2 | Feb. 1960 | do | 63 | 19 | 12 | 4 | 2 | ..... |
| 1 | Feb. 1960 | Guadalupe at San Jose | 92 | 7 | 1 | ..... | ..... | ..... |
| 3 | Jan., Feb. 1960 | San Francisquito Creek at Stanford | 71 | 8 | 17 | 4 | ..... | ..... |
| Totals | | | 916 | 263 | 202 | 98 | 83 | 38 |
| Percentages | | | 57.2 | 16.4 | 12.6 | 6.2 | 5.2 | 2.4 |
| Unit weight—lb. per cubic foot | | | 30 | 65 | | 93 | | |
| Weight per cubic ft. 48.84 (call 50) | | | 17.16 | 18.85 | | 12.83 | | |

[1] Data abstracted from "Suspended Sediment in California Streams," U.S. Geological Survey, Sacramento, Calif. (Unpublished records.)

These are the average densities of sediment inflow delivered at the meres of the system. Progression downbay toward the Golden Gate results in comminution of the coarser particles and in higher percentages of colloidal particles, extensive flocculation, and, hence, less compaction of sediment with consequent less average sediment densities for suspended load and bedload. If the findings of the USGS study are adopted — the most exhaustive to date — the volume of sediment delivered annually to the Bay system (including the Delta) was 8.8 million cubic yards for the 1957-59 period. Table 5 gives the definitive findings by weight of the USGS investigation.

Direct measurement of the bedload is difficult, due as much to the complexity of the phenomena of bedload movement as to the physical difficulties of measurement, with turbulence be-

TABLE 5.—*Average daily sediment inflow to San Francisco bay system*[1]

| Station or stream group | 1957–59 | | | | 1909–59 adjusted to 1957–59 conditions—total sediment |
|---|---|---|---|---|---|
| | Suspended sediment | | Unmeasured sediment | Total sediment (rounded) | |
| | measured | estimated | | | |
| | *Tons* | *Tons* | *Tons* | *Tons* | *Tons* |
| Delta system | 12,200 | 50 | 1,960 | 14,200 | 13,800 |
| Suisun Bay | 320 | 320 | 60 | 700 | 500 |
| San Pablo Bay | 218 | 580 | 80 | 900 | 1,000 |
| San Francisco Bay | 983 | 410 | 140 | 1,500 | 800 |
| Total (rounded) | 13,700 | 1,400 | 2,200 | 17,000 | 16,000 |

[1] From table 12 of reference (*12*).

ing a principal factor in the movement. Transportation of the bedload is affected by traction manifested either by rolling, sliding, or by particles making small jumps, called "saltation." The width of the stream, permitting either one or more threads of maximum velocity with accompanying areas of maximum turbulence, is also a factor. If the phenomena of formation of dunes and antidunes is also added as factors, it is apparent that, in the present knowledge, the indirect approach to estimation of the bedload is the most practical; that is, measurement of the total load and the subtraction of the suspended load. The USGS investigation determined the total load for some streams at concrete controls or where the channels were bedrock with high turbulence. For other streams with alluvial beds, the unmeasured (bed) load was estimated on the basis of mean velocities and particle size of the bed material. For the major contributors — the Sacramento and San Joaquin Rivers — total sediment discharge was computed (*3*).

TABLE 6.—*1957–59 sediment inflows by geographic areas in percentages by weight and volume*

| Geographic area | Suspended load (percent of total for area) | | Bedload (percent of total for area) | |
|---|---|---|---|---|
| | By weight | By volume[1] | By weight | By volume[1] |
| Delta | 86.9 | 92.26 | 13.1 | 7.74 |
| Suisun Bay | 91.4 | 95.02 | 8.6 | 4.98 |
| San Pablo Bay | 91.1 | 94.85 | 8.9 | 5.15 |
| San Francisco Bay | 90.7 | 94.63 | 9.3 | 5.37 |
| Total system (rounded by USGS) | 87.0 | 92.35 | 13.0 | 7.65 |

[1] Conversion based on densities of 90 lb./cu. ft. for bedload and 50 lb./cu. ft. for suspended load.

[3] GRIMM, C. I. STUDY OF TIDAL CURRENTS AND SILT MOVEMENTS IN THE SAN FRANCISCO BAY AREA, WITH PARTICULAR REFERENCE TO THE EFFECT OF A SALT WATER BARRIER UPON THEM. U.S. Corps Engin. Spec. Rpt. (App. D, House Doc. 191, 71st Cong., 3d Sess.) 1931. [Not published.]

Table 5, quoted directly from the USGS investigation (*11*), sets forth the findings from the 1957-59 series of observations and the derived bedload figures (listed as "Unmeasured sediment") by geographical distribution of the sediment inflow. Table 6 gives the percentage distribution of the total sediment load of suspended loads and bedloads by geographical areas. Table 7, derived from tables 5 and 6, gives tonnages and daily sediment inflows by geographical areas and also the rounded annual sediment inflow in cubic yards. In table 7, the 1909-59 figures adjusted to 1957-59 conditions are considered valid for the period 1960–2011.

TABLE 7.—*Sediment inflows to San Francisco Bay system, 1960–2011*

| Geographic area | Adjusted total sediment[1] | | Daily inflow | Average annual inflow | Percentage of total sediment inflow |
|---|---|---|---|---|---|
| | Tons/day | Percentage of total | | | |
| | | | *Cu. yd.* | *Cu. yd.* | |
| Delta | 13,800 | 85.7 | 19,280 | 7,040,000 | 85.5 |
| Suisun Bay | 500 | 3.1 | 720 | 260,000 | 3.2 |
| San Pablo Bay | 1,000 | 6.2 | 1,420 | 520,000 | 6.3 |
| San Francisco Bay | 800 | 5.0 | 1,140 | 415,000 | 5.0 |
| Total | 16,100 | 100.0 | 22,560 | 8,235,000 | 100.0 |

[1] 1909–59 figures adjusted to 1957–59 conditions.

### *Comparative Estimates*

The volume derived from table 5 for the 1957-59 water years agrees almost identically with the volume for the same years derived from a study of historical sedimentation that used Bureau of Reclamation figures for historical outflow of the Sacramento River at Sacramento in the long term period 1924-25 to 1959-60. The total outflow to the Delta for the combined Sacramento-San Joaquin system was determined from a rating curve (fig. 3) giving the relation of the total Sacramento-San Joaquin flow to the flow of the Sacramento River at Sacramento. The sediment-water discharge relation curve (fig. 4), used by the USGS in their 1957-59 study, was then used to obtain the suspended sediment load and this multiplied by 1.2 to obtain total sediment load. Table 8 details the results of this study. The long-term annual average sediment inflow is 6,200,000 cubic yards. For the water years 1957-59, the average is 7,280,000 cubic yards as compared to 7,200,000 given by the U.S. Geological Survey.

Gilbert (*7*) estimated 2 million cubic yards average annual sediment inflow for the period prior to 1849, 18 million for the 1849-1914 period, and 8 million for the future after clearance of the mining debris. Grimm in 1931[3] estimated 5.75 million, and the Corps of Engineers in 1954, assuming considerable reservoir regulation, determined the total at 3.36 million.

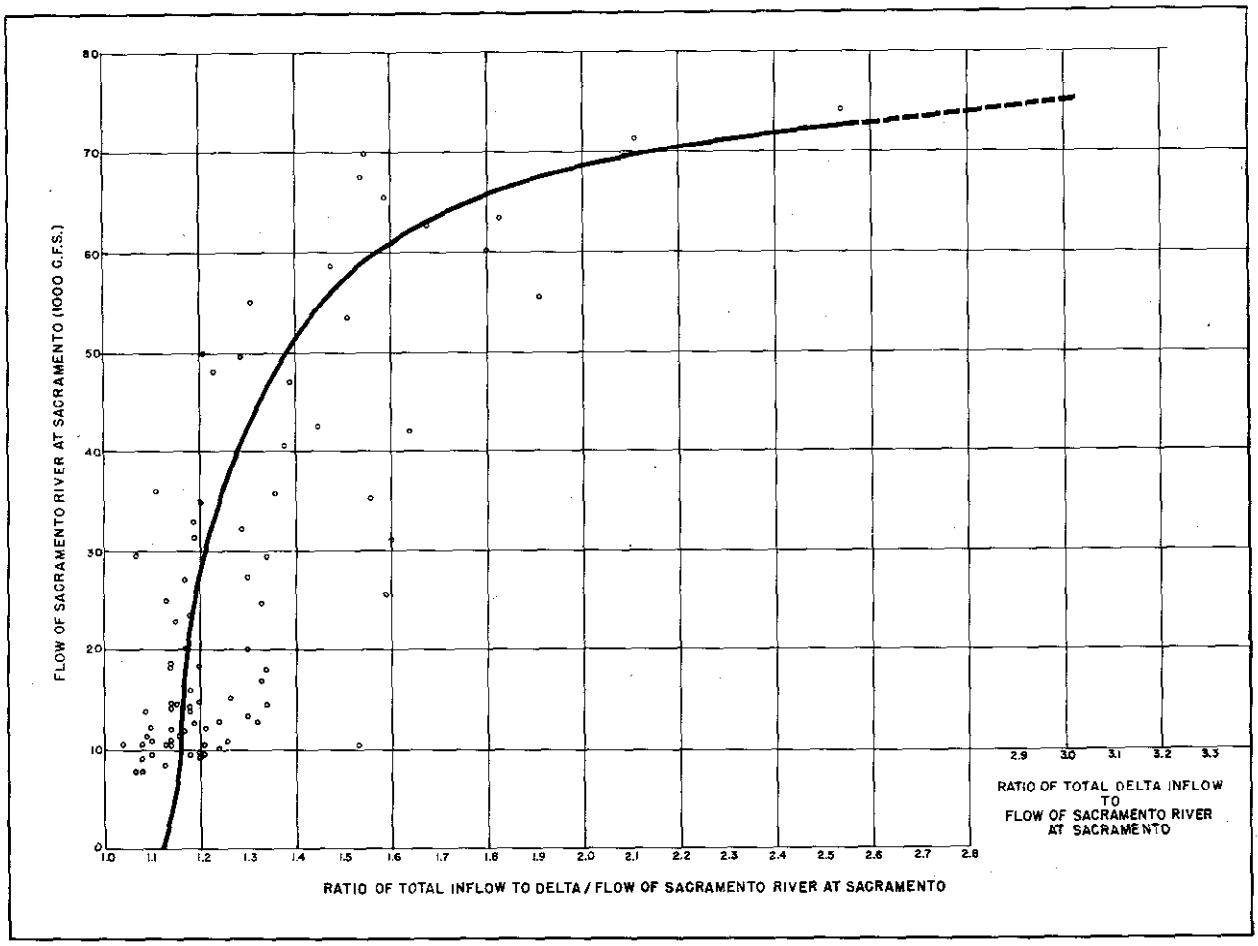



FIGURE 3. — Ratio of total delta inflow to flow of Sacramento River at Sacramento.

The State Division of Water Resources,[4] in the 1955 report on the feasibility of barriers, estimated the annual load at 4 million cubic yards. Much of the variance in these figures can be attributed to paucity of data available to the investigators, judgment being a definite factor in the estimating process. The findings of this present study, as noted, are based on records maintained continuously for 3 years.

### *Factors Affecting Future Sediment Inflow*

The estimate of 8,235,000 cubic yards per annum given in table 7 is an average based on appraisals of the major factors affecting the volume of sediment. These are land use, streamflows, and the conditions governing the formation of river channels (river morphology). Rain erosion of the land surface and channel erosion of the streambed effected by the river itself are the main determinants of waterborne sediment. Land use in turn is the major factor in rain erosion, and the competency, capacity, and existing load are the factors determining to what extent, if any, channel erosion and the shaping of new channels will take place.

If the land use aspect of the San Francisco Bay system tributary sediment source areas is considered, the change to urban use will be most marked for the local bay area watersheds and least, relatively inconsequential, for the Sacramento-San Joaquin system. The land use pattern for the local areas in 1960 was 10 percent urban, 60 percent agricultural, and 30 percent mountain, public, and wastelands. By 2020 urban use will approach 35 percent (*6*) (*15*), and agricultural use decline to 40 percent. Since the local drainage areas contribute but 14.5 percent of the present total sediment inflow, these changes will bring no noticeable variations in resulting sediment volume. Within the Sacramento-San Joaquin system the present

[4] CALIFORNIA DIVISION OF WATER RESOURCES. SEDIMENTATION. Appendix G of "Feasibility by the State of Barriers in the San Francisco Bay System." Sept. 1955.

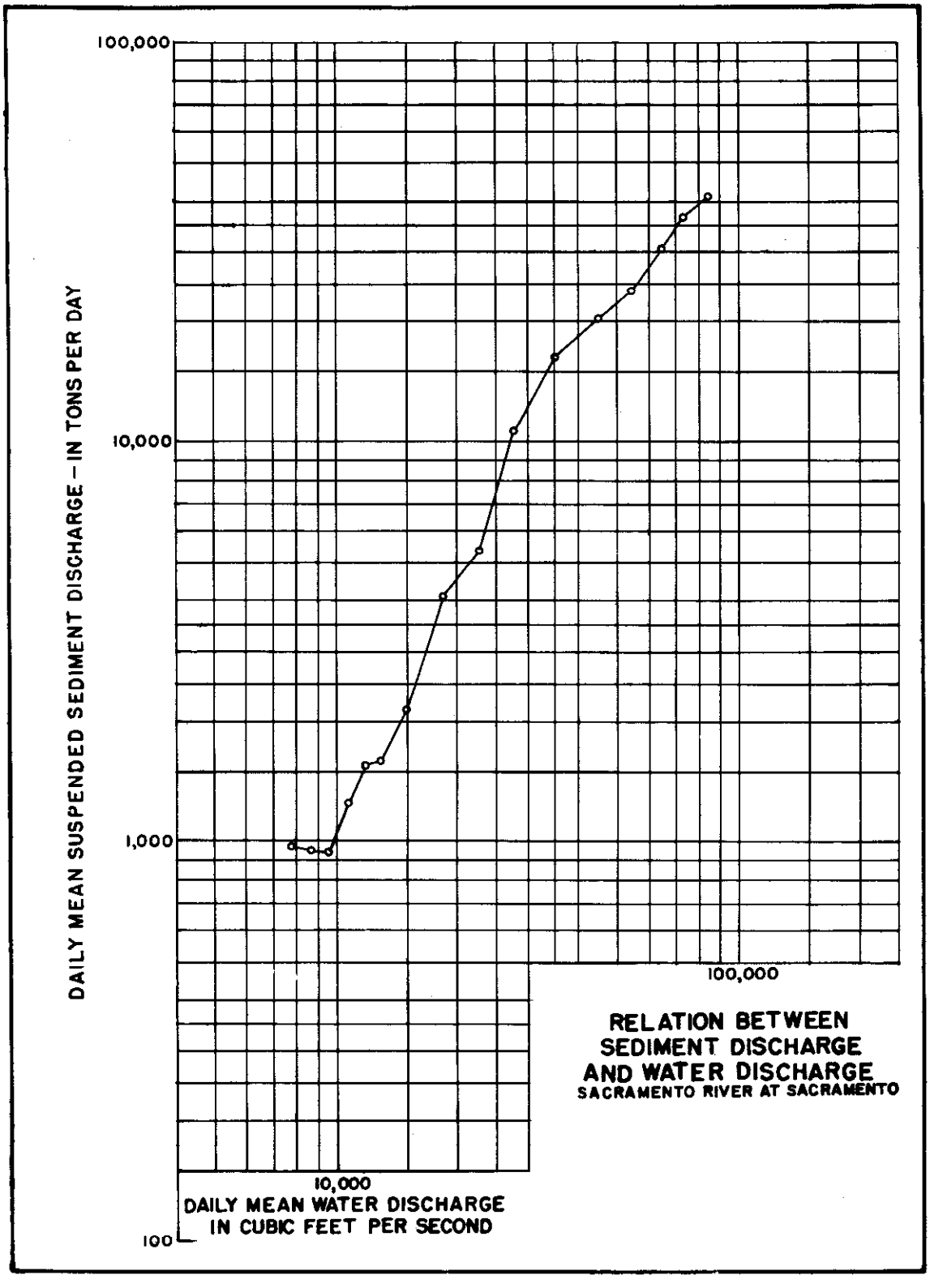



FIGURE 4. — Relation between sediment discharge and water discharge of Sacramento River at Sacramento.

TABLE 8.—*Estimated average monthly historical sediment inflow to the Sacramento-San Joaquin Delta, 1924–60*

(In thousands of cubic yards)

| Year | Oct. | Nov. | Dec. | Jan. | Feb. | Mar. | Apr. | May | June | July | Aug. | Sept. | Annual total |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1924–25 | 60 | 65 | 99 | 92 | 2,010 | 707 | 1,298 | 936 | 67 | 61 | 65 | 58 | 5,518 |
| 1925–26 | 59 | 42 | 61 | 76 | 1,384 | 353 | 1,223 | 153 | 60 | 67 | 67 | 59 | 3,604 |
| 1926–27 | 59 | 683 | 697 | 810 | 2,468 | 1,554 | 1,721 | 1,236 | 320 | 57 | 53 | 58 | 9,726 |
| 1927–28 | 59 | 260 | 190 | 301 | 720 | 2,146 | 1,313 | 368 | 57 | 62 | 65 | 60 | 5,601 |
| 1928–29 | 60 | 51 | 54 | 59 | 222 | 181 | 107 | 193 | 56 | 65 | 65 | 59 | 1,172 |
| 1929–30 | 60 | 41 | 655 | 514 | 478 | 1,022 | 548 | 289 | 55 | 65 | 64 | 58 | 3,859 |
| 1930–31 | 59 | 40 | 43 | 68 | 61 | 193 | 59 | 61 | 65 | 70 | 70 | 62 | 851 |
| 1931–32 | 62 | 41 | 217 | 568 | 668 | 727 | 679 | 1,044 | 247 | 62 | 67 | 62 | 4,444 |
| 1932–33 | 63 | 42 | 43 | 59 | 65 | 352 | 352 | 274 | 152 | 67 | 68 | 62 | 1,599 |
| 1933–34 | 61 | 41 | 83 | 502 | 234 | 478 | 196 | 68 | 64 | 68 | 68 | 62 | 1,925 |
| 1934–35 | 62 | 67 | 47 | 809 | 372 | 1,113 | 2,422 | 1,514 | 276 | 61 | 65 | 60 | 6,868 |
| 1935–36 | 59 | 40 | 41 | 1,126 | 2,001 | 128 | 1,362 | 722 | 233 | 61 | 65 | 68 | 5,906 |
| 1936–37 | 60 | 41 | 42 | 46 | 987 | 1,396 | 1,568 | 1,158 | 155 | 62 | 67 | 61 | 5,643 |
| 1937–38 | 59 | 614 | 1,760 | 779 | 2,514 | 3,125 | 2,631 | 2,497 | 1,718 | 78 | 60 | 57 | 15,982 |
| 1938–39 | 58 | 76 | 107 | 97 | 108 | 365 | 208 | 59 | 63 | 70 | 69 | 60 | 1,340 |
| 1939–40 | 61 | 62 | 60 | 1,452 | 2,103 | 1,917 | 1,855 | 785 | 88 | 63 | 64 | 60 | 8,570 |
| 1940–41 | 68 | 69 | 784 | 2,298 | 2,451 | 1,933 | 1,946 | 1,831 | 457 | 63 | 61 | 68 | 12,029 |
| 1941–42 | 68 | 58 | 1,135 | 1,525 | 2,135 | 1,024 | 1,870 | 1,579 | 866 | 58 | 62 | 58 | 10,438 |
| 1942–43 | 57 | 97 | 413 | 1,320 | 1,486 | 2,184 | 1,572 | 731 | 1,551 | 58 | 68 | 61 | 9,598 |
| 1943–44 | 59 | 56 | 61 | 83 | 470 | 559 | 211 | 349 | 57 | 62 | 62 | 60 | 2,089 |
| 1944–45 | 60 | 93 | 111 | 152 | 1,372 | 586 | 572 | 572 | 123 | 57 | 57 | 57 | 3,812 |
| 1945–46 | 59 | 161 | 1,184 | 1,677 | 365 | 622 | 844 | 617 | 91 | 59 | 59 | 56 | 5,794 |
| 1946–47 | 58 | 92 | 196 | 96 | 339 | 776 | 406 | 76 | 56 | 60 | 60 | 57 | 2,272 |
| 1947–48 | 59 | 85 | 57 | 390 | 106 | 263 | 1,590 | 1,677 | 1,006 | 57 | 58 | 68 | 5,416 |
| 1948–49 | 63 | 78 | 97 | 91 | 94 | 1,408 | 1,334 | 484 | 64 | 59 | 59 | 59 | 3,890 |
| 1949–50 | 59 | 57 | 59 | 478 | 1,251 | 848 | 1,101 | 785 | 222 | 59 | 59 | 55 | 5,033 |
| 1950–51 | 65 | 1,076 | 3,066 | 1,643 | 2,195 | 1,224 | 592 | 662 | 75 | 58 | 61 | 63 | 10,780 |
| 1951–52 | 61 | 105 | 1,031 | 2,512 | 2,664 | 2,076 | 3,084 | 3,556 | 1,232 | 232 | 71 | 83 | 16,707 |
| 1952–53 | 63 | 69 | 733 | 2,505 | 788 | 491 | 737 | 1,085 | 818 | 79 | 58 | 91 | 7,517 |
| 1953–54 | 74 | 105 | 120 | 559 | 1,575 | 1,428 | 1,539 | 463 | 77 | 59 | 59 | 79 | 6,137 |
| 1954–55 | 73 | 122 | 390 | 350 | 122 | 110 | 107 | 307 | 91 | 57 | 57 | 60 | 1,846 |
| 1955–56 | 59 | 63 | 1,221 | 5,088 | 1,731 | 1,406 | 847 | 1,317 | 492 | 96 | 91 | 68 | 12,479 |
| 1956–57 | 95 | 122 | 95 | 94 | 205 | 1,529 | 285 | 817 | 180 | 59 | 61 | 91 | 3,633 |
| 1957–58 | 235 | 245 | 361 | 1,044 | 3,907 | 2,227 | 3,886 | 1,756 | 1,014 | 122 | 118 | 153 | 15,068 |
| 1958–59 | 96 | 96 | 96 | 627 | 1,128 | 622 | 107 | 85 | 56 | 89 | 63 | 75 | 3,140 |
| 1959–60 | 58 | 57 | 59 | 81 | 1,040 | 954 | 256 | 163 | 75 | 70 | 61 | 59 | 2,933 |
| 36-year total | 2,450 | 5,112 | 15,478 | 29,971 | 41,819 | 38,117 | 40,428 | 30,269 | 12,279 | 2,552 | 2,347 | 2,387 | 223,209 |
| 36-year average | 68 | 142 | 430 | 833 | 1,161 | 1,059 | 1,123 | 841 | 341 | 71 | 65 | 66 | 6,200 |
| Percent by month | 1.1 | 2.3 | 6.9 | 13.4 | 18.7 | 17.1 | 18.1 | 13.6 | 5.5 | 1.2 | 1.0 | 1.1 | 100 |

land use pattern, dominantly agricultural and recreational, will persist with only minor change, percentagewise, in urbanized areas.

Intensified agricultural usage and an expanded highway network will tend to increase the sediment volume, but these tendencies will be offset by improved practices in soil conservation, watershed management, and highway drainage. In 1917 Gilbert (*7*) estimated premining era average sediment inflow at 2 million cubic yards when the region was largely virgin land and forecast 8 million yards for a fully developed highway system through a much expanded agricultural area. The broad conclusion, therefore is that land use changes between 1960 and 2020 will not materially affect the total sediment inflow.

When the future channel erosion and river morphology aspects are considered, the chief additional factors over the natural conditions are the mining debris deposits and the existence of numerous reservoirs. The mining debris load is disposed into the component bays, except for permanent lodgments in the mountains that are stabilized and will not contribute to the sediment inflow. Thus, the appraisal of future loadings narrows down to the effect on competencies and capacities of the rivers of the system from the construction and operation of numerous dams that now ring the tributary area at or about the alluvial fan elevations. The sediments in solution and in the colloid state that would be unaffected by velocity changes could to some extent be altered in content and volume by land usage practices, thus changing the chemistry and the flocculation tendencies.

No such changes would occur to any appreciable extent above the line of reservoirs. Below

the reservoirs the principal agents of such change would be industrial and agricultural contaminants. These in their totality could, with greatly expanded volume, present a disposal problem of large proportions requiring treatment and disposal works. Their effect is not here included, as present knowledge of this phase is inadequate to form quantitative conclusions.

If the preceding feature is omitted, future sediment loads of streams will depend for the most part on conditions below the line of dams. The physical effects of constructing a dam across a stream are pronounced, tending to change the total activity of the river — reshaping its course and affecting the competency, capacity, and load of the current.

As defined by Twenhofel (*14, p. 191*), competency is the ability of currents to transport, in terms of dimensions of particles; capacity is the ability to transport, in terms of quantity; and load is the actual quantity transported. Competency depends upon velocity and turbulence, which in turn vary with the shape of the channel, bottom irregularities, slope, depth, and discharge. There is a competent slope for each variation of channel and for each dimension of particles of a given specific gravity. Competency varies as about the fifth power of the velocity. Capacity is dependent on the same variables as competence, but additional factors are the ratio of depth to width of stream and the degree of mixing of the sediments. Thus, capacity is generally less for material of uniform dimensions than for graded materials. The stream load as a maximum equals the capacity. Capacity ranges between the 3d and

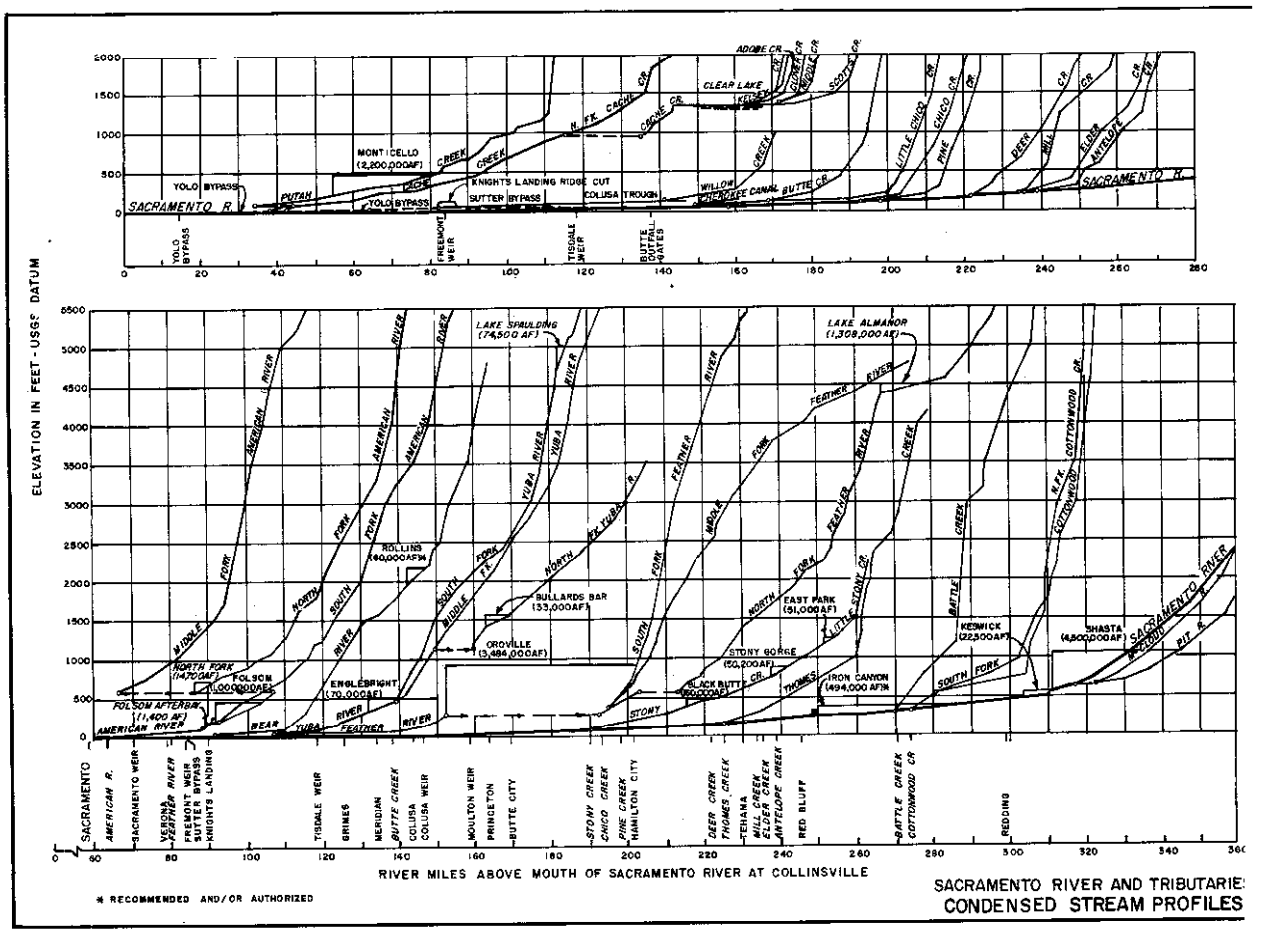



FIGURE 5. — Profiles of elevation and river miles of the Sacramento River and its tributaries.

th power of the velocity increase. If these con-
iderations to the reaches are applied immedi-
tely above and below a dam, the competency
pstream is gradually reduced by the widening
onfines of the reservoir so that a selective
rading of deposition results with a tapering of
ilt at the lower end. Aggradation of the
treambed results. With continued aggradation
iere comes about a loss of cross sectional area,
) that scour results and the deposition process
ikes place further down in the reservoir, the
ong-run result being complete siltation of the
eservoir. With loss of competency and capac-
y, releases through the reservoir are greatly
larified of suspended load. Virtually all the
edload is usually trapped in the reservoir.

Below a dam, the number and the intensity
f the threads of maximum velocity are reduced,
hen not actually eliminated, by zero flow
irough the dam or spillway. With reduced
irbulence and velocity in the immediate reach
elow the dam, only the fine silt is picked up in
ispension or rolled on the bed in traction. The
hysical effect on the streambed is loss of fine
ediments in the shoal areas and filling of the
eeper parts with part of this fine silt load.
Vith flood releases, however, the partly clarified
ream under an artificially increased head is
iaracterized by turbulence and high velocity
) that scour occurs and degradation ensues.
his is the prevalent condition for the Sierra
evada in the winter flood months.

Table 9 shows that a major part of the total
ver mileage lies below the line of lowest dams.
urther, the dams and reservoirs are in general
the breaks in gradient between the steep
oland and the flat sluggish lowland courses
here, irrespective of the existence of reservoirs
oviding stilling basin effects, there would
iturally occur heavy deposition of sediment.
he corollary follows that the competencies,
pacities, and loadings below the dams do not
ary greatly from those that prevailed on the
low-dam reaches before the reservoirs were
nstructed. Figure 5 illustrates the location at
eaks in grade of the majority of reservoirs
nstructed or authorized.

It follows, consequently, that the principal
fect of the reservoir systems of the Central
alley and local Bay area streams (where the
me situation holds) lies in the regulation of
w and thus the regulation of sediment inflow.
ie competencies and capacities of the various
reams would be affected by the velocities and
lumes of the regulated flows and, derivatively,
e sediment loadings. On analysis, this again
offset — similar to reservoirs at the break in
ade — by the concentration of sediment dis-
arge in the flood months. In the long-run

TABLE 9.—*Stream regimens, Sacramento-San Joaquin System*

| Stream in system | Elevation at— | | Length | | Average gradient | |
|---|---|---|---|---|---|---|
| | Source | Lowest dam | Source to lowest dam | Lowest dam to mouth | Upland | Below dam |
| | *Feet* | *Feet* | *Miles* | *Miles* | *Feet per 1,000* | *Feet per 1,000* |
| Sacramento River | 8,000 | 450 | 64 | 243 | 22.4 | 0.35 |
| Stony Creek | 4,000 | 380 | 48 | 24 | 14.3 | 2.28 |
| Putah Creek | 2,500 | 185 | 48 | 27 | 10.5 | 1.11 |
| Feather River | 6,000 | 180 | 104 | 64 | 10.5 | 0.53 |
| Yuba River | 7,500 | 282 | 64 | 44 | 21.4 | 1.21 |
| Bear River | 3,500 | 136 | 38 | 28 | 16.87 | 0.91 |
| American River | 8,500 | 200 | 81 | 24 | 19.4 | 1.58 |
| Mokleumne River | 8,000 | 230 | 64 | 40 | 23.0 | 1.09 |
| Calaveras River | 6,200 | 1,600 | 33 | 260 | 1.9 | 0.21 |
| San Joaquin River | 10,000 | 289 | 88 | 230 | 20.9 | 0.24 |
| Merced River | 11,000 | 405 | 72 | 48 | 39.0 | 1.35 |
| Tuolumne River | 10,000 | 331 | 86 | 48 | 21.5 | 1.19 |
| Stanislaus River | 9,000 | 537 | 66 | 48 | 30.4 | 2.42 |
| Totals | ..... | ..... | 823 | 868 | ..... | ..... |

average (see table 8), 50 percent of the annual total sediment is delivered in the flood months — January, February, and March. In flood years such as 1958, this figure rises to 70 percent. Thus flood releases would carry a major part of the annual sediment load from the lowland reaches and a certain amount of reservoir-trapped sediment through scour effect. At 7 ft./sec. velocity, all but the heaviest stones would be moved by scour. At 3½ ft./sec. (*14, p. 193*), pebbles up to walnut size would be scoured. Hence, since a flood release velocity of 3½ ft./sec. would prevail, it can be taken the sediment load below a dam would be at the maximum competence and capacity of the stream.

For the remaining months, the average conditions of flow volume and velocity would, as regards sediment inflow, about balance the total under natural conditions ranging from moderate to almost zero flow. Hence, the overall conditions for effects of reservoir at or near the alluvial fan elevation in the tributary sediment area to the Delta would seem to warrant the conclusion that no marked variation from the estimated annual average sediment figure of 7,040,000 cubic yards will occur.

The sediment inflow to the Delta will be reduced in its passage to Suisun Bay by deposition within the sloughs and by an amount carried by the transfer flows exported through the Tracy

pumping plant and the Contra Costa Canal.[5] The estimate of 7.04 million cubic yards of inflow to the Delta, based on the 1957-59 observations, covers withdrawal conditions above the Delta as of 1960. Reduction in sediment volume for future increased withdrawals and for the transfer flows to the Contra Costa Canal and through the Tracy pumping plant will closely approximate the suspended load. The bedload for the greater part will remain in the system. If the water-sediment discharge relation curve (fig. 4) is used, the reduction in suspended load for withdrawals can be obtained. Figure 4 is considered applicable to withdrawals over the entire system, including volumes from local bay area streams. Withdrawals within the Delta totaled 4,500 c.f.s. in 1960, the corresponding sediment withdrawal being 600 tons per annum, or 307,000 cubic yards per annum. Table 10 gives the range of sediment inflow for future transfers before accounting for deposition in the Delta.

TABLE 10.—*Sediment in flows to Bay system adjusted for varying water exports before deposition in Delta*
(Total sediment 1960 inflow of 8,235,000 cu. yd.)

| Water volume exported (c.f.s.) | Sediment reduction | Reduction[1] | Net sediment inflow[1] per annum |
|---|---|---|---|
| | *Tons* | *Cu. yd.* | *Cu. yd.* |
| 4,500 | 600 | 307,000 | 7,928,000 |
| 10,000 | 1,000 | 511,000 | 7,724,000 |
| 15,000 | 2,200 | 1,124,200 | 7,110,800 |
| 20,000 | 4,500 | 2,299,500 | 5,935,500 |

[1] Suspended load. Does not include bedload.

### *Sedimentation Within the System*

The estimated average annual inflow of 8,235,000 cubic yards will be delivered to the entire system and will be distributed to the component elements in the proportions given in table 7, the Delta receiving 7,040,000 cubic yards, or 85.5 percent. At present, the transfer volumes exported from the Delta average 4,500 c.f.s., representing 307,000 cubic yards of suspended sediment per annum. The sediment inflow to Suisun Bay, therefore, is 7,133,000 yards, less deposition within the Delta.

Determination of the amount of sediment deposited in the Delta — as with the other parts of the system — presents difficulties in view of the complex conditions that affect deposition, such as flocculation and tidal currents. Although research has been productive qualitatively in the field of estuarine sedimentation, a methodology derived from the mechanics of the phenomena involved that would give quantitative as well as qualitative results has not yet been perfected. Hence, an indirect approach becomes necessary to estimate present rates of deposition.

The lower reaches of the Delta are subject to cyclic flocculation as the fresh-saline water interface fluctuates upstream and downstream with the tides and all reaches are subject to the tidal impulses that tend to a storage effect on floodtide when the river inflow is met by the oncoming tidal rise, thus favoring deposition. The emptying effect of the ebbtide, particularly the long ebb, tends to cause resuspension of sediment and to scour. Similarly the advance and recedence of the saline interface brings flocculation of colloidal matter and later resuspension in the colloidal state when fresh water conditions occur. In all instances, because of the predominance of the ebb flow, a residual downstream movement of deposited material takes place. Comparison, therefore, at regular intervals over an extended period indicate changes in channel configurations throughout the Delta that would afford quantitative data on the deposition processes. In the light of qualitative research findings, reasonable projections into the future could possibly be made. Unfortunately, no regular sequential data over a long period are available for the Delta. Hence, as the best available alternate, the dredging records for the Sacramento River, the San Joaquin River, and Old River Slough, which are continuous for the 30-year period, 1930–59, are adopted.

In using these records a primary consideration is to what extent, if any, the diminishing volumes of debris from the hydraulic mining era are represented in the figures given in table 11.

TABLE 11.—*Dredging of Sacramento-San Joaquin Delta, 1930–59*[1]

| Years | Maintenance | New work | Total |
|---|---|---|---|
| | *1,000 cu. yd.* | *1,000 cu. yd.* | *1,000 cu. yd.* |
| 1930 | 1,055 | 906 | 1,961 |
| 1931–35 | 5,420 | 24,174 | 29,594 |
| 1936–40 | 12,380 | 7,469 | 19,849 |
| 1941–45 | 7,826 | ..... | 7,826 |
| 1946–50 | 6,223 | 280 | 6,503 |
| 1951–55 | 4,554 | 5,269 | 9,823 |
| 1956–59 | 2,533 | ..... | 2,533 |
| Totals | 39,991 | 38,098 | 78,089 |
| 30-year average | 1,333 | 1,270 | 2,603 |

[1] Data from files of U. S. Army Engineer District, Sacramento.

Gilbert (*7*) estimated that in 50 years from 1914 all debris except permanent lodgments in the mountains would have been cleared from

[5] At Tracy, Calif., is located the pumping plant that supplies the Delta-Mendota Canal, the supply being pumped from Old River. Since the Sacramento River furnishes the major part of the supply, there is a cross-delta transfer of flow that otherwise would follow the course of the Sacramento River into Suisun Bay.

the Bay system. This estimate has been borne out by subsequent developments. At the present time (1962), observations indicate stabilization has come about. The enormous quantities involved in this movement, estimated by Gilbert at 850 million cubic yards, of which 375 million would be sand and gravel subject to deposition at reduced velocities, preclude the possibility that the Delta could be the depository without far greater volumes of dredging than those given in table 11 — 375 million cubic yards that would be deposited as against 78 million dredged in 30 years. The conclusion must be made that the great bulk of this material has been swept through the Delta on into the Bay system. The continuous development of channel improvements for expeditious removal of floodflows lends strength to this viewpoint — that in effect all debris has been cleared through the Delta except minor quantities deposited in backwaters.

Deposits in the Delta, therefore, can be taken as the normal sedimentation for years of medium to low flows, resulting on the slackening of velocity in the Delta sloughs and residual flocculation at the western end of the Delta. This hypothesis assumes the mining debris load and heavy channel erosion load were removed by floodflows occurring at least at 5-year intervals and that the maintenance dredging average of 1,333,000 cubic yards represents dry low-flow years accumulations plus aftermath lodgments from the diminishing flood flows. If 85 percent represents the total Delta deposition (the minor sloughs not being dredged), the average annual sediment deposition in the Delta is estimated at 1.6 million cubic yards.

### *Sediment Inflow Distribution*

From the preceding discussion, the direct sediment inflow into the Bay system, excluding the Delta, i.e., deducting sediment exported with the transfer volume of 4,500 c.f.s. and 1.6 million cubic yards deposition within the Delta, is given in table 12. These sediment volumes —

TABLE 12.—*Direct sediment inflows (1960 conditions) to Bay system*

| Bay | Average annual sediment inflow |
|---|---|
| | *Cubic yards* |
| Suisun Bay (from delta) | 5,133,000 |
| Suisun Bay (from local) | 260,000 |
| San Pablo Bay | 520,000 |
| San Francisco Bay | 415,000 |
| Total | 6,328,000 |

part in the colloidal state, part in suspension, and part being moved by traction (bedload) — upon delivery into the tidal waters of the system enter an environment of higher salinity, cyclic reversals of flow and much greater wind action over large expanses of shallow water. In the changed environment the sedimentation processes are greatly changed, the end result being comminution of particles and a higher flocculation precipitation. These changes are reflected in density changes in the bottom sediments and elimination of the sand content by reduction into silt and colloids.

The changes effected in the character of the sediment load upon entrance to the tidal system and subjection to the changed environment vary over the limits of the component bays. However, a general indication is obtained by comparing the data given in table 4 with the data presented in table 13 compiled from 40 analysis of suspended sediment load taken from the Bay in September 1957 (*13*). The sand content is eliminated in the Bay suspended sediment load and the silt content increased, as shown in table 14.

Bottom sediment densities, which compare relatively with bedload densities of the inflow sediment load to the Delta, while varying by location within the Bay system, are consistently lighter than the 90 lb. per cu. ft. determined by the USGS for the Delta bedload; the differences are shown in table 15.

Other major aspects affecting sedimentation characteristics and deposition are salinity, turbidity, and wind effects. Table 16, giving salinity and turbidity (sediment p.p.m.), is to be considered in a relatively locational sense only, since wide ranges in values occur at every location due to the tidal cycles varying over the year and the seasonal fluctuation being pronounced. The data presented are averages of hourly readings made 21-22 September 1956 by the U.S. Army Engineer District, San Francisco. Within each flood and ebb cycle the sediment (parts per million) fluctuates from lows at beginning and end to maxima in the middle of the cycle, the maximum being approximately double the minimum value. Salinity shows considerably less variation and in the lower reaches none. Temperature and wind effects are discussed later in connection with historical patterns of deposition.

The environmental changes brought about as the sediment inflows from the river systems traverse the bay can be broadly grouped into (1) changes in characteristics and (2) changes in deposition patterns. From the preceding data the principal changes in characteristics are the elimination of the sand content, increase in silt content, and decreased densities of bottom sediment regarded as equivalent to the bedload of the fluvial sedimentation conditions. It is significant that no change occurs in the clay content percentage of the suspended load char-

TABLE 13.—*Bay sediment analyses (suspended load), September 1957*

| Field samples | | Clay | Silt | | Sand | | |
|---|---|---|---|---|---|---|---|
| Sample No. | Depth | Lessthan 0.004 mm. | 0.004 to 0.016 mm. | 0.016 to 0.064 mm. | 0.065 to 0.125 mm. | 0.125 to 0.250 mm. | 0.250 to 0.500 mm. |
| | *Feet* | *Pct.* | *Pct.* | *Pct.* | *Pct.* | *Pct.* | *Pct.* |
| 8T | 27.5 | 54 | 45 | 1 | ..... | ..... | ..... |
| 16T | 26.5 | 47 | 50 | 3 | ..... | ..... | ..... |
| 24 | ..... | 50 | 48 | 2 | ..... | ..... | ..... |
| 40T | 27.5 | 52 | 46 | 2 | ..... | ..... | ..... |
| 48T | ..... | 48 | 51 | 1 | ..... | ..... | ..... |
| 56T | 25.5 | 48 | 50 | 2 | ..... | ..... | ..... |
| 55T | 25.5 | 74 | 24 | 2 | ..... | ..... | ..... |
| 57T | 26.5 | 50 | 48 | 2 | ..... | ..... | ..... |
| 58T | 25.5 | 57 | 43 | ..... | ..... | ..... | ..... |
| 69T | 25.0 | 51 | 47 | 2 | ..... | ..... | ..... |
| 70T | 25.5 | 49 | 49 | 2 | ..... | ..... | ..... |
| 80 | 28.5 | 65 | 32 | 3 | ..... | ..... | ..... |
| 89 | 29.0 | 44 | 56 | ..... | ..... | ..... | ..... |
| 103 | 31.5 | 55 | 45 | ..... | ..... | ..... | ..... |
| 111 | 31.5 | 58 | 38 | 4 | ..... | ..... | ..... |
| 123 | 29.0 | 50 | 47 | 3 | ..... | ..... | ..... |
| 128 | 27.5 | 47 | 50 | 3 | ..... | ..... | ..... |
| 135 | 27.0 | 63 | 35 | 2 | ..... | ..... | ..... |
| 143 | 26.0 | 85 | 15 | ..... | ..... | ..... | ..... |
| 169 | 30.0 | 45 | 52 | 3 | ..... | ..... | ..... |
| 170 | 30.5 | 40 | 57 | 3 | ..... | ..... | ..... |
| 182 | 30.5 | 65 | 35 | ..... | ..... | ..... | ..... |
| 189 | 30.5 | 72 | 27 | 1 | ..... | ..... | ..... |
| 197 | 30.5 | 53 | 45 | 2 | ..... | ..... | ..... |
| 204 | 30.5 | 46 | 53 | 1 | ..... | ..... | ..... |
| 211 | 29.5 | 62 | 38 | ..... | ..... | ..... | ..... |
| 224 | 28.5 | 48 | 52 | ..... | ..... | ..... | ..... |
| 230 | 27.5 | 60 | 40 | ..... | ..... | ..... | ..... |
| 7 [1] | 26.5 | [1]48 | [1]42 | [1]10 | ..... | ..... | ..... |
| 5 [2] | to 30 27 to 31 | [2]79 | [2]19 | [2]2 | ..... | ..... | ..... |
| Average | ..... | 57 | 40 | 3 | ..... | ..... | ..... |
| Unit weight | ..... | 30 | 65 | | ..... | 93 | ..... |
| Weight [3] | ..... | 19.1 | 27.95 | | ..... | ..... | ..... |

[1] 7 samples; multiply percentages by 7.
[2] 5 samples; multiply percentages by 5.
[3] Weight per cubic foot is 47.05 lb., or 47.

acteristics — remaining at 57 percent. The deposition patterns are changed from fluvial types as determined by competency, capacity, and loading of the carrier streams, to fluctuating tidal forward and backward movements modified by salinity changes, wind effects, and bottom currents.

### *Increase in Volume*

Of practical consequence is the increase in volume resulting from the lowered densities of Bay bottom sediment in comparison with the bedload densities under fluvial conditions. The significant feature — the percentage of clay content in the suspended load remains constant

TABLE 14.—*Change in content of suspended loads, Delta sediment inflow and Bay sediment*

| Source | Clay | Silt | Sand |
|---|---|---|---|
| | *Pct.* | *Pct.* | *Pct.* |
| Delta inflow | 57 | 29 | 14 |
| Bay system | 57 | 43 | 0 |
| Change (to Bay system) | 0 | +14 | −14 |

under fluvial and Bay tidal conditions — would indicate that flocculation, which removes a high percentage of colloids and tends to increase the bedload, is balanced by the addition of colloids resulting from attrition of the coarser particles by tidal action and turbulence, both tending to reduce in the bedload and to increase in the suspended load. In this connection, no material change occurs in the weight per cubic foot of suspended sediment for fluvial and bay conditions — 48.84 lb. per cu. ft. for fluvial sediment and 47.05 lb. per cu. ft. for Bay suspended sediment. Hence, it is concluded there is no great

TABLE 15.—*Bottom sediment densities*[1] *San Francisco Bay system*

| General location | Bottom sediment density |
|---|---|
| | *Lb./cu. ft.* |
| Suisun Bay | 88 |
| Central (North San Francisco) Bay | 49 |
| South San Francisco Bay | 45 |
| San Pablo Bay | 31 |

[1] Analysis of 124 dredge spoil samples over period 1953–62 U.S. Army Engineers, San Francisco District Operations Division.

material change in proportions of total sediment by weight per cubic foot between suspended load and bedload.

From this reasoning increase in volume is proportional to the changes in densities of the suspended loads and bottom sediments as follows:

| | *Percent* |
|---|---|
| Suisun Bay | 6.0 |
| San Pablo Bay | 24.6 |
| San Francisco Bay | 12.6 |

The bulk of the increase is actually concentrated in the bedload to bottom sediment increase, in part accounting for the large shoaling volumes

TABLE 16.—*Average salinity and sediment throughout the San Francisco Bay system, Sept. 21–22, 1956*

| Location | Salinity [1] | | Sediment | |
|---|---|---|---|---|
| | Flood | Ebb | Flood | Ebb |
| | *P.p.t.* | *P.p.t.* | *P.p.m.* | *P.p.m.* |
| Collinsville | 0.7 | 0.7 | 89 | 71 |
| Carquinez Strait | 19.6 | 17.1 | 76 | 80 |
| San Pablo Strait | 24.3 | ..... | 12 | ..... |
| East of Treasure Island | 29.9 | 29.9 | 27 | 23 |
| Dumbarton | 29.0 | 29.0 | 97 | 48 |

[1] In parts per thousand.

occurring in the component bays. As noted previously these increases for bedload to bottom sediment are 2 percent for Suisun Bay, 249 percent for San Pablo Bay, and an average of 91 percent for San Francisco Bay. A cubic yard of bedload material, e.g., becomes 3½ cubic yards of bottom sediment in San Pablo Bay.

## Deposition of Sediment in the Tidal Bays

Upon entrance into the Bay system, the vehicle of carriage for the fluvial sediment becomes the tidal prism, which is characterized by alternate advances and recessions so that net movement downbay is very much less than under the fluvial movement in a single direction —downstream. The tidal prism is averaged at 1.575 million acre-feet, or 788,000 c.f.s., and if the average daily fresh water inflow is set at 29,000 c.f.s., or 14,500 c.f.s. for a complete 12-hour cycle, the dilution ratio is approximately 54:1. The total volume of the Bay system, however, is much greater than the tidal prism, and approximates 5.6 million acre-feet below mean sea level, excluding the Delta, which permits still greater dilution. However, there is no marked difference between the sediment concentrations under fluvial and under Bay tidal conditions, indicating that supplementary supplies are being made to the suspended load in the Bay waters either from outside the system from the ocean or from bottom mud stirred into suspension by tidal action.

Krone, in his studies on estuarial shoaling in the Bay system (*10*), states with regard to shoal material "most of the shoal material appeared to have originated from inland drainage by way of the San Joaquin-Sacramento River system, which carried principally montmorillonite, and less from coast range drainage, which carried principally illite. It was concluded that sediment is mixed throughout the Bay system, and that the composition and particle-size distribution resulted from the combination of sediment loads from the contiguous streams, as modified by local transport conditions." This statement would favor the viewpoint of the supplemental supply originating from bottom deposits, since an ocean supply would necessarily be reflected in the analysis of bottom sediment. This is supported by the general functioning of the system whereby a net daily outflow constantly occurs, entailing sediment outflow to rather sediment inflow from the ocean. It is concluded, therefore, that the additional sediment load in the tidal waters over that being delivered from the fluvial sediment input is derived from the bay bottom sediments and is a result of tidal and wind action.

The pattern of deposition, consequently, comes from the net translation and deposition resulting from reiterated reversals of competencies and capacities of the tidal flows modified by salinity variations and wind effects. The pattern is in continuous flux, a basic daily pattern being adjusted to an overriding seasonal fluctuation and a long-term flood-drought cyclical swing modifying these basic fluctuations. Geologic action also affects the deposition of sediment, such as differential settlement of land in different locations of the system. A slow rise of the Pacific Ocean is also known to be occurring. Thus as a result of the superimposed long term fluctuations over a number of decades, parts of the system would be slowly changing from a condition of accretion to one of scour or vice versa.

The normal pattern for fluvial sediment entering the tidal system is for a net daily progression downbay of the bedload increased by flocculation as the salinity concentration increases. The net downbay movement is evident on comparison of the competencies and capacities of the flood and ebbtides through a complete daily cycle for a mixed tide. The net downbay movement is not pronounced for a typical sine tide, and for the recurring case of floodtide dominance net movement would be upstream. Over the long run, and particularly for the flood months of heavy sediment inflow, net movement is downbay. If the data for a mixed tide were analyzed, an index of the overall movement downbay is obtained. For the tides of November 11-12, 1958, the average of the velocities of the two ebbtides exceeded that of the two floods by 7 percent and the corresponding average tidal prism for the ebbs exceeded that for the floods by 2.7 percent. Thus, the competency of the ebbs is 40 percent greater than the flood competency. Hence, with the greater carrying volume of the ebbtides, the resulting capacity of the ebbtides exceeded by 30 percent that of the floodtides. However, the dominance of the flood duration over the ebb duration mitigates the effect of the ebb capacity dominance, thus cutting down the net movement downbay of the suspended particles to about 0.4 mile. For sediment on the bottom, part fluvial bedload and part eroded from the bottom deposits, many variables enter into the net movement that takes place. For this aspect the movement and volume of deposits and the rates of accretion and scour recourse are best for analysis of changes in bottom configurations.

## The Historical Sedimentation Pattern

For the Bay system changes in bottom configurations can be ascertained from the soundings surveys conducted at periodic intervals from 1855 through 1956. The changes thus ob-

tained give the volumes of accretion or scour for the time intervals noted, but the changes do not give the total net deposition that has taken place throughout geologic time. This total was approximated through a series of borings from which contours of the bottom of the Bay mud lying on firm clay or sand were established. The contours for the top of the Bay mud were then obtained from the most recent soundings; by subtraction, the net volumes of Bay mud were derived, the total being slightly under 16 billion cubic yards. Table 17 details the deposi-

TABLE 17.—*Bay Mud volumes, San Francisco Bay system*

| Location | Water area | Average depth to clays and sands | Volume to clays and sands | Water volume | Bay mud volume | |
|---|---|---|---|---|---|---|
| | *Acres* | *Feet* | *1,000 acre-feet* | *1,000 acre-feet* | *1,000 acre-feet* | *Millions of cubic yards* |
| Suisun Bay: | | | | | | |
| Army Point to Chipps Island | 22,967 | 29.8 | 684 | 268 | 416 | 671 |
| Carquinez Strait: | | | | | | |
| Napa River to Army Point | 4,350 | 74.0 | 322 | 172 | 150 | 243 |
| San Pablo Bay: | | | | | | |
| Point San Pablo to Napa River | 58,361 | 57.3 | 3,344 | 631 | 2,713 | 4,379 |
| Central San Francisco Bay: | | | | | | |
| Golden Gate to south end, Yerba Buena Island to Point San Pueblo | 52,968 | 92.0 | 4,873 | 1,897 | 2,976 | 4,801 |
| San Francisco Narrows: | | | | | | |
| South end of Yerba Buena Island to Candlestick Pt. | 21,255 | 62.0 | 1,318 | 592 | 726 | 1,170 |
| South Bay: | | | | | | |
| Candlestick Point to Alviso | 83,950 | 46.0 | 3,862 | 951 | 2,911 | 4,695 |
| Total: Bay system | 243,851 | [1]59.06 | 14,403 | 4,511 | 9,892 | 15,959 |

[1] Average.

tion of the Bay mud deposits in the component bays. Deposits are lightest in Suisun Bay, heaviest in Central Bay, and roughly equal for the remaining areas, the ratio of deposition per acre being 1:3:2 for Suisun Bay, the Central Bay, and the approximately equal (by rate per acre) areas — Carquinez Strait, San Pablo Bay, and South Bay.

The total of 16 billion yards of deposits of bay mud is the net result of an indeterminate period extending back into geologic time. The present water volume of the system is slightly in excess of 10 billion cubic yards, but assumption, based on rough comparison with the 16 billion yards deposit, that a very great period would have to elapse for complete filling of all bays is not warranted. With increasing reduction of the tidal prism, the San Francisco bar would move landward with probable inflow of ocean sediment. Accelerated deposition with augmented volume of sediment would shorten the period of ultimate fill. Net deposition per annum determined from bottom configuration changes over a 100-year period is 6 million yards. If this rate would remain constant, the period for complete siltation of the system would be 1,700 years.

From the record, however, cycles of erosion and accretion occur that make unreliable any straight line projection for the pattern of deposition. A close analysis of these cyclical trends, however, is difficult, because of the overlapping and lack of continuity in the basic source data. Also some survey data have been questioned for reliability by previous investigations. For this study, data and findings of previous studies were appraised in the light of recent observations.

The source data are the surveys conducted by the USC&GS from 1855 at intervals through 1956. The investigations consulted were (1) those by Gilbert (*7*), completed in 1914; (2) the comprehensive study by the U.S. Army Corps of Engineers, known as the Grimm study completed in 1930 but not published; and (3) the Division of Water Resources, State of California, investigation released in March 1955 as part of the report "Feasibility of Construction by the State of Barriers in the San Francisco Bay System." The first comprehensive surveys by the USC&GS were carried out between 1855 and 1868. Gilbert questioned the reliability of the vertical controls used for the surveys of the northern half of San Francisco Bay and of San Pablo Bay. The USC&GS in rebuttal countered that such vertical controls were relatively unimportant, as compared to the accuracy of the depth measurements. For the southern half of San Francisco Bay (1857-58) and for Suisun Bay (1867-68), local tide stations, some tied into bench marks, supplemented the permanent station at Fort Point. The Grimm study supplemented the source data with surveys of Suisun and San Pablo Bays in 1930, but the fieldbooks and boat sheets have not been located. Changes in shorelines between surveys also made difficult the computation of volume changes on a bank to bank basis.

The methods of using the source data affect the results. Volume changes can be determined by establishing bottom contours, planimetering the areas, and computing volumes by depth intervals. An alternate method is to establish a grid over the system and average the depth of the recorded soundings within the quadrilateral; the area by depth giving the water volume when compared with the altered water volume of a subsequent survey determines the volume change in deposition. The accuracy of the grid method depends on the uniformity of spacing of the depth measurements within the quadrilateral, the shape of the bottom, and the area. The accuracy of the contour method depends on the horizontal accuracy of the contour location. Gilbert apparently used both methods, using either where most adaptable. Grimm used one minute (longitude and latitude) quadrangles, but used contours to some extent as subdivisions of the main tract. The State adopted ¼-minute quadrangles together with cross sections, to develop better the fringe tracts, especially. The present Corps of Engineers study used ⅛-minute quadrangles.

If it is assumed that water volume change equals sediment volume change, the possibility of subsidence of the Bay floors is discounted. This possibility, for lack of data to evaluate properly, was discounted in all prior investigations. In this study the possibility of subsidence was evaluated from data furnished by the U.S. Geological Survey (*8*) covering subsidence in Santa Clara County. The effect is relatively minor and is not included in the findings graphed in figures 6 through 14, which give cumulative sediment volume changes throughout the system. With this feature discounted, the sedimentation pattern is given by the major component areas.

### *Suisun Bay*

Figure 7 gives the sedimentation changes in -minute quadrangles and table 18 gives the changes by major areas for Suisun Bay. Excepting the channels that range in depth from 20 to 70 feet (including a series of navigation channels with authorized maintenance depth of 30 feet and width of 300 feet), Suisun Bay is markedly shallow, generally less than 6 feet in depth. For the period of record, the main deposition took place in the 21-year period 1867-86 and was laid down chiefly in the channel areas. By 1921 scour had reduced the 22.5 million yards to 7.3 million and to actual scour by 1942 despite a temporary accretion recorded in 1930. The area covered by the West Bay area and Suisun slough channel experienced the same general depletion trend. Similarly and to a marked degree with Honker Bay, the pattern changed from a deposit of 2.7 million yards by 1886 to a scour of 11.8 million by 1942. The Grizzly Bay area was relatively inactive through the 90-year period.

TABLE 18.—*Cumulative sediment volume changes, Suisun Bay 1866–1956*

| Area | 1866–1887 | 1921 | 1930 | 1941–42 | 1956 |
|---|---|---|---|---|---|
| | *Million cu. yd.* | *Million cu. yd.* | *Million cu. yd.* | *Million cu. yd.* | *Million cu. yd.* |
| hannel areas and middle ground | 22.5 | 7.3 | 11.5 | −0.1 | ..... |
| 'est Bay and Suisun slough channel | 10.0 | 3.2 | 5.7 | 5.5 | ..... |
| onker bay | 2.7 | −9.5 | −6.3 | −11.8 | ..... |
| rizzly Bay | 3.2 | ..... | ..... | ..... | 1.8 |
| Totals | 38.4 | 1.0 | 10.9 | −6.4 | ..... |

### *Carquinez Strait*

The accumulated sediment volume change for Carquinez Strait is given in table 19 and figure 8. The strait, in general a mile in width between steep escarpments and 8 miles long, is deepest in the middle (Dillon Point) with soundings to 100 feet and much shallower depths at either end — about 30 feet.

TABLE 19.—*Cumulative sediment volume change, Carquinez Straight, 1856–1950*

| Years | Volume |
|---|---|
| | *Million cubic yards* |
| 1856–86 | 70.0 |
| 1922 | 81.0 |
| 1950 | 98.3 |

An overall accretion of 98.3 million yards, the greater part during the years 1857-86, is shown, but this masks areas of scour and fill occurring within the same quadrangle. This particularly applies to two quadrilaterals south of Benicia bounded by 122° 8′ W. to 122° 10′ W. and 38° 2′ N. to 38° 3′ N. Detailed study of these areas showed very heavy sedimentation occurred along the north shore of the strait accompanied by compensating scour in the channel area. The mean lower low water line of Southampton Bay had advanced about 2,500 feet from deep inside the Bay to a part alined with the sweep of the strait. The mean level low water line is now located where water depths were 90 feet in 1857, most of the filling occurring in the 1857–86 period. In the vicinity of Benicia Arsenal Pier a shore line advance of 500 to 700 feet occurred during 1857 to 1886, after which the situation became stabilized until 1942. Accentuated shoaling is indicated in the vicinity of the new arsenal dock (built in 1941-42) from the data furnished by the 1949 survey.

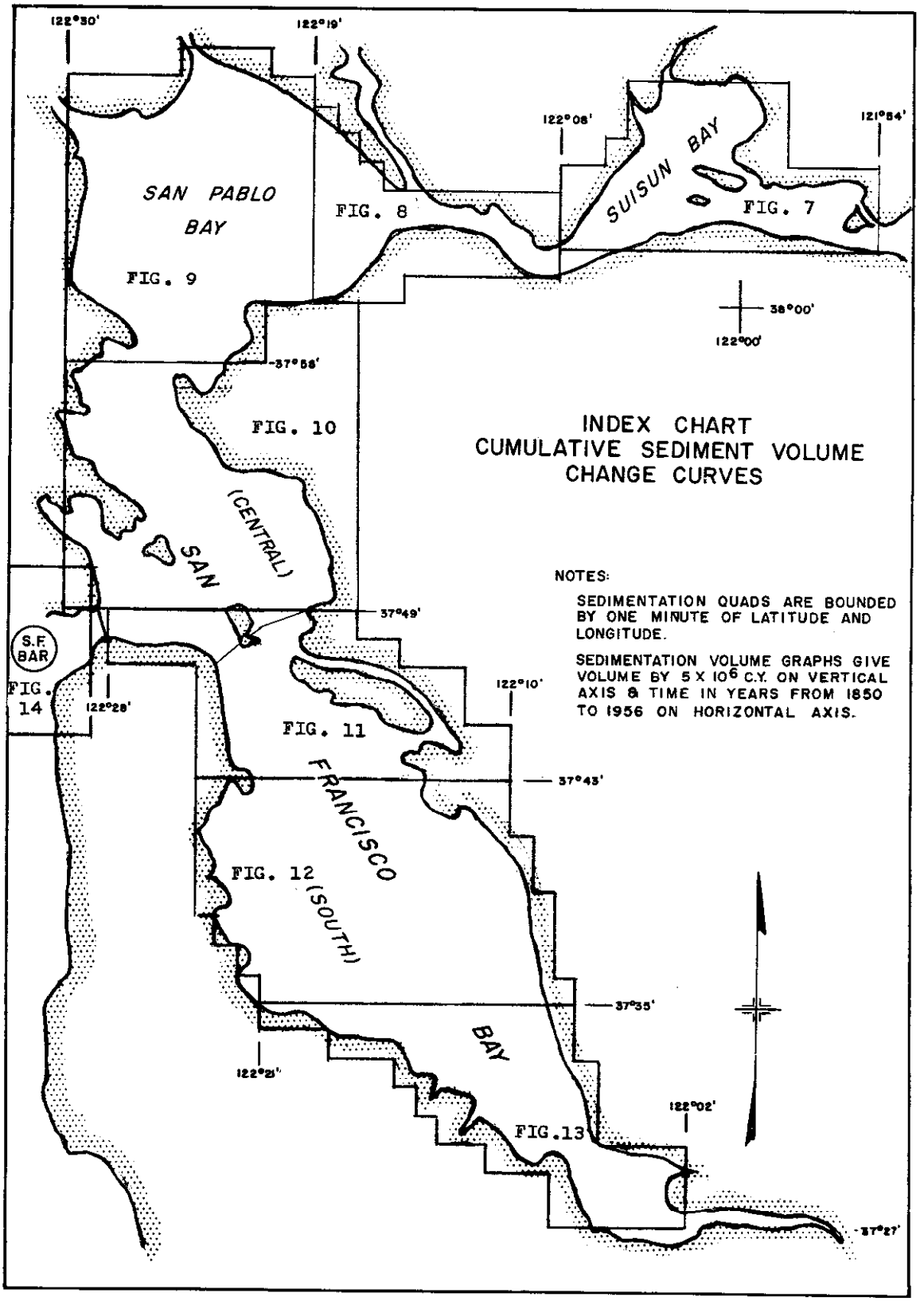



FIGURE 6. — Index chart of cumulative sediment volume change curves of San Francisco Bay. (See figs. 7 to 14.)

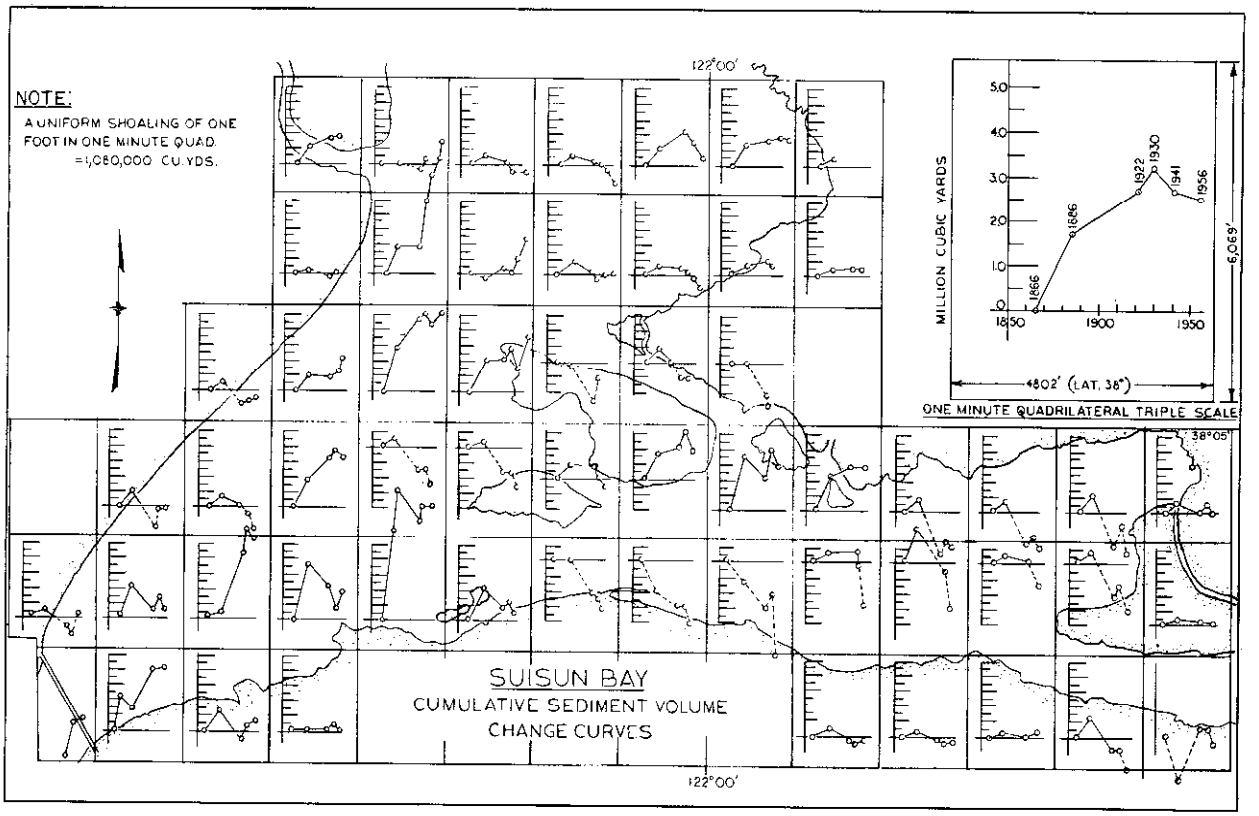



FIGURE 7. — **Cumulative sediment volume change curves of Suisan Bay.**

### *San Pablo Bay*

The sedimentation pattern of San Pablo Bay was studied by three subdivisions according to depth: (1) shoal areas of 4 feet depth or less extending over 39 quadrilaterals, (2) shoals between 4 and 20 feet in depth covering 40 quadrilaterals, and (3) channel areas over 20 feet deep covering 20½ quadrilaterals. Table 20 and figure 9 give the sediment change volume data by periods. The 4 feet and less shoal areas lie mostly in the northern and western limits of the Bay, although small areas contiguous to the southern shoreline are included. (See fig. 17). The highest deposition, relatively, has occurred in the shallow areas, with the major part occurring in 1856-87 period. The intermediate depth areas, 4 to 20 feet, lie on either side of the channel areas with the greater extent north of the channels. These areas for the total period show over twice the deposition of the marginal shoals and over 4 times that of the channel areas. The channel areas are conspicuous by consistently showing scour from the initial period of deposition in 1856-87.

### *North San Francisco Bay and Golden Gate*

North San Francisco Bay, regarded in this paper as the central bay between the northern and western arm (San Pablo and Suisun Bays) and the southern arm (below Yerba Buena) of the Bay system, is analyzed on a similar basis to San Pablo Bay by considering marginal shoals (0 to 10 feet), intermediate shoals (10 to 30 feet), and channel areas (over 30 feet). The

TABLE 20.—*Cumulative sediment and mean depth changes, San Pablo Bay, 1855–1951*

| Area | 1885–87 | 1899 | 1922 | 1930 | 1951 |
|---|---|---|---|---|---|
| Marginal shoal (0 to 4 feet) areas, million cubic yards | 74.0 | 75.8 | 101.2 | 76.3 | 105.3 |
| Mean depth of deposits, feet | 1.80 | 1.81 | 2.40 | 1.81 | 2.50 |
| Intermediate (4 to 20 feet) shoal, million cubic yards | 162.0 | 169.9 | 214.8 | 177.1 | 251.6 |
| Mean areas depth of deposits, feet | 3.74 | 3.92 | 4.95 | 4.10 | 5.80 |
| Channel (20+ feet) areas, million cubic yards | 88.4 | 83.6 | 74.4 | 47.4 | 56.3 |
| Mean depth of deposits, feet | 3.98 | 3.77 | 3.35 | 2.14 | 2.54 |
| Total volume, million cubic yards | 324.0 | 329.3 | 390.4 | 300.8 | 413.2 |
| Total average depth, feet | 3.00 | 3.04 | 3.60 | 2.78 | 3.80 |

0- to 10-foot shoals are the Berkeley mud flats and the drowned valleys of the western shore covering 22.9 quadrilaterals. The intermediate depths flank the deep water and include Southampton shoal. These areas cover 18.2 quadrilaterals. The channel areas cover 26.9 quadrilaterals within the central part of the total area and lead into Golden Gate, 6 quadrilaterals. Figures 10 and 14 and table 21 summarize the volume and depth changes from 1855 to 1948 for North San Francisco Bay and through 1956 for the Golden Gate.

The marginal shoals of the Bay eroded 0.58 foot in the initial 40-year period and then reversed to accretion in relatively small amounts that were insufficient to overcome the previous erosion: the overall effect from 1855 to 1948 being a net erosion of 0.14 foot. The intermediate shoals, 10 to 30 feet deep, by contrast continually showed accretion, with a total depth gain for the entire period of 8.16 feet. The channel areas have the same pattern as the marginal shoals — erosion to accretion — with a slight overall increase in depth of deposit, 0.29 foot. A similar general pattern was followed for Golden Gate sedimentation.

TABLE 21.—*Cumulative sediment volume change, North San Francisco Bay and Golden Gate*

| Area | 1855–95 | 1916 | 1948 |
|---|---|---|---|
| | *Million cubic yards* | *Million cubic yards* | *Million cubic yards* |
| San Francisco Bay: | | | |
| Marginal shoals (0 to 10 feet) areas | −14.6 | −5.6 | −3.4 |
| Intermediate shoals (10 to 30 feet) | 65.6 | 96.1 | 161.1 |
| Channel (deep water) areas | −53.5 | −46.2 | 8.5 |
| Totals | −2.5 | 44.3 | 166.2 |
| | 1855–73 | 1894 | 1956 |
| Golden Gate | −8.1 | −43.4 | −10.8 |

### *South San Francisco Bay*

Three general surveys, those of 1857, 1896,

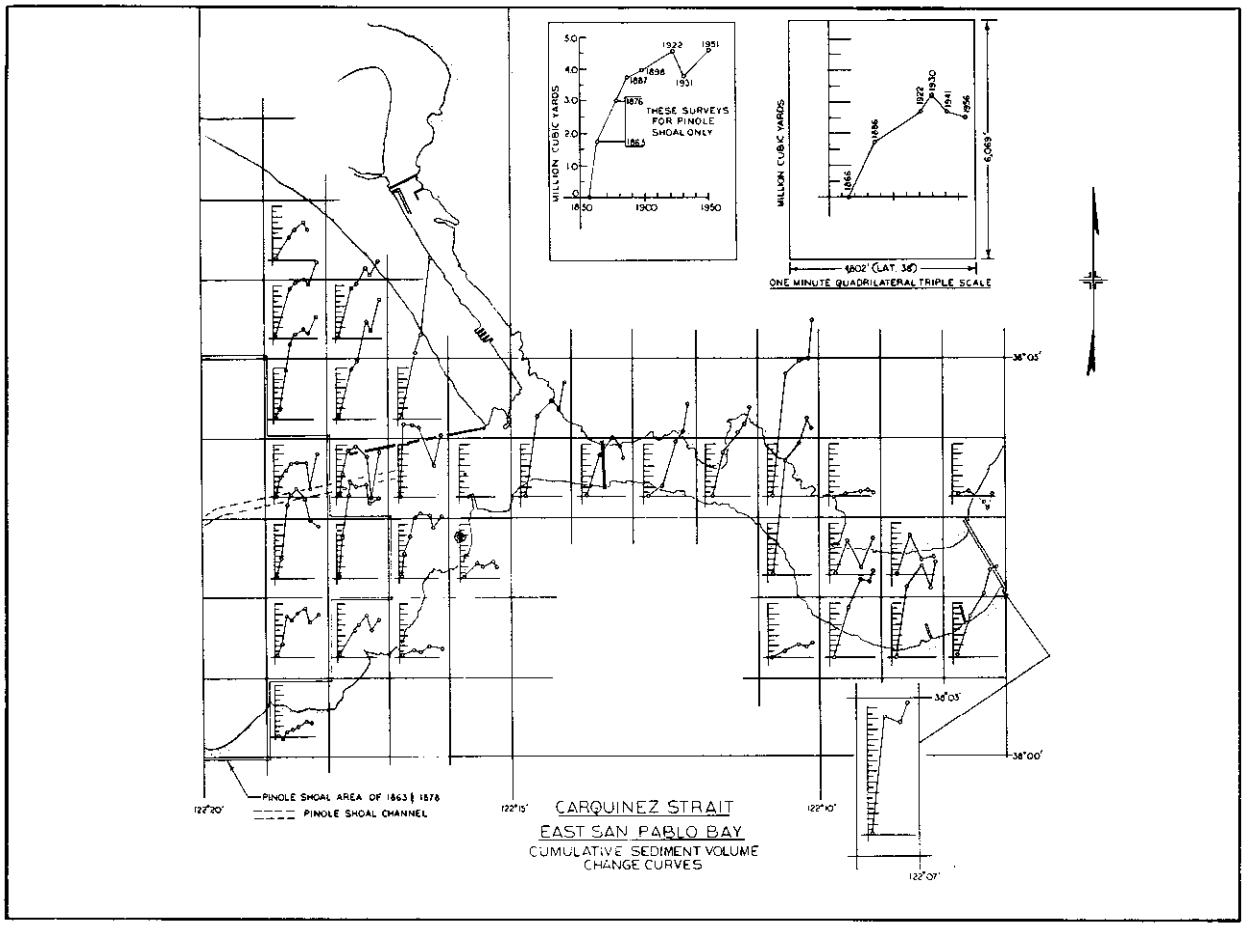



**FIGURE 8.— Cumulative sediment volume change curves of Carquinez Strait, East San Pablo Bay.**

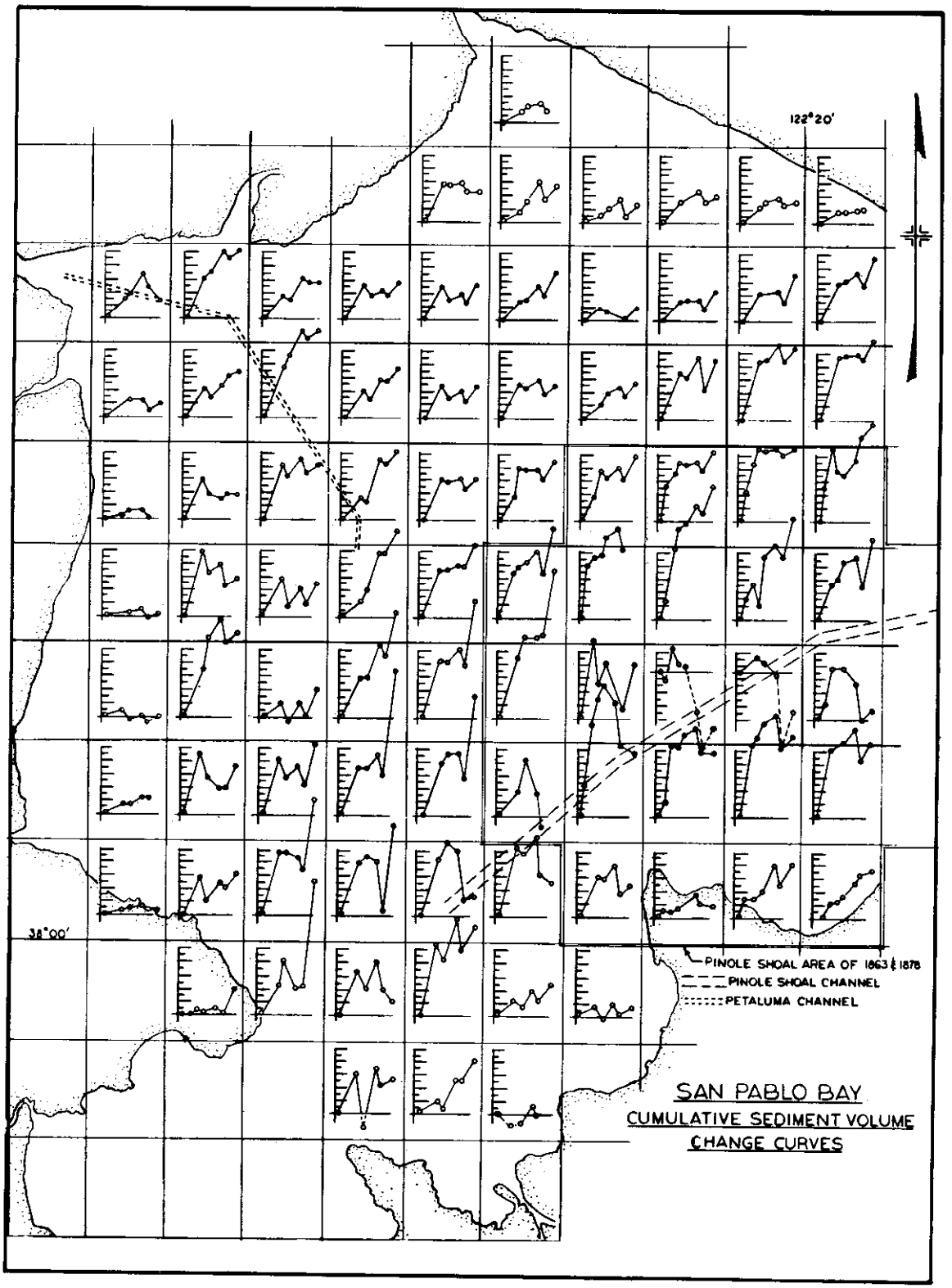



FIGURE 9. — Cumulative sediment volume change curves of San Pablo Bay.

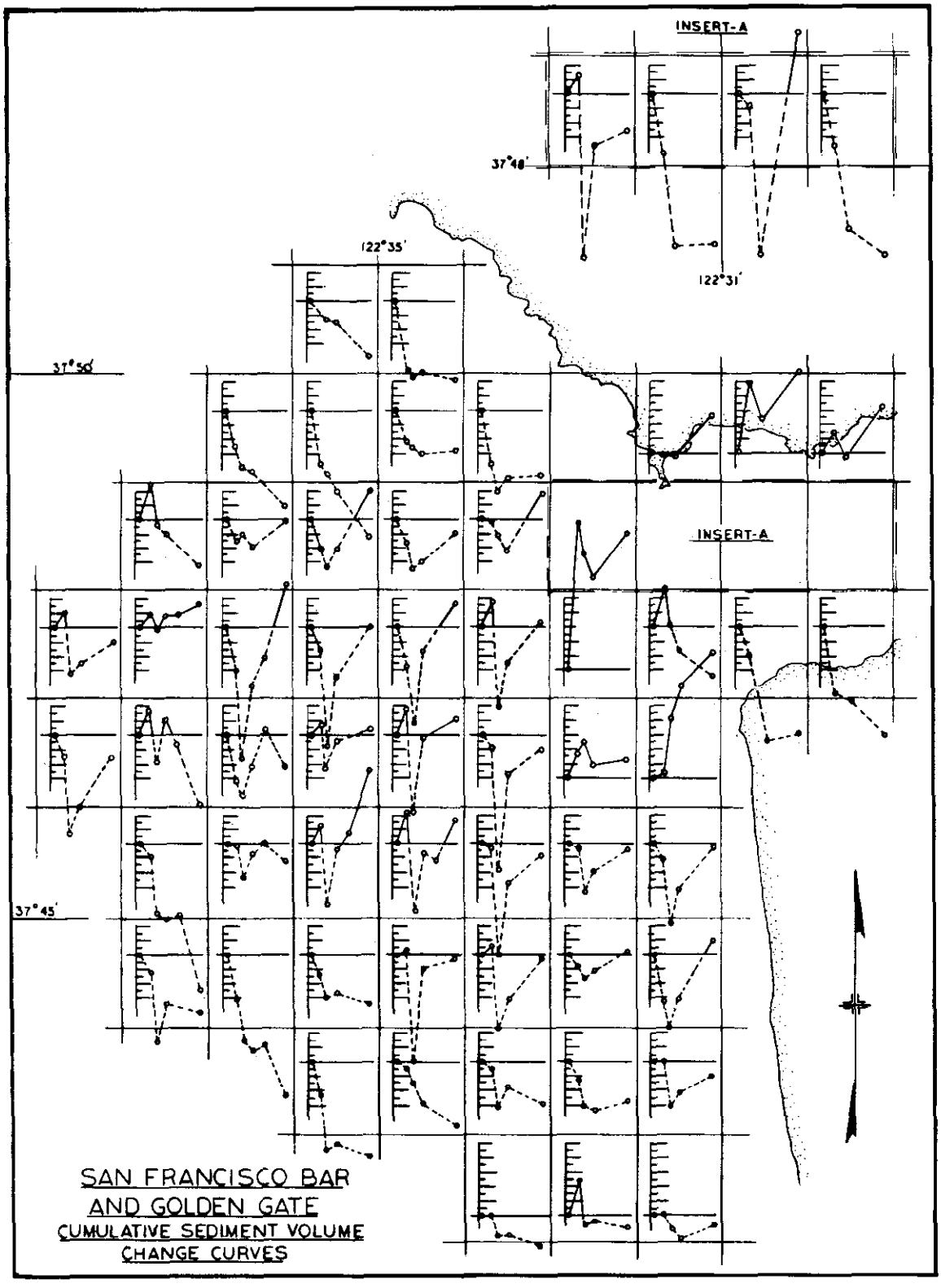



FIGURE 10. — Cumulative sediment volume change curves of San Francisco Bar and Golden Gate.

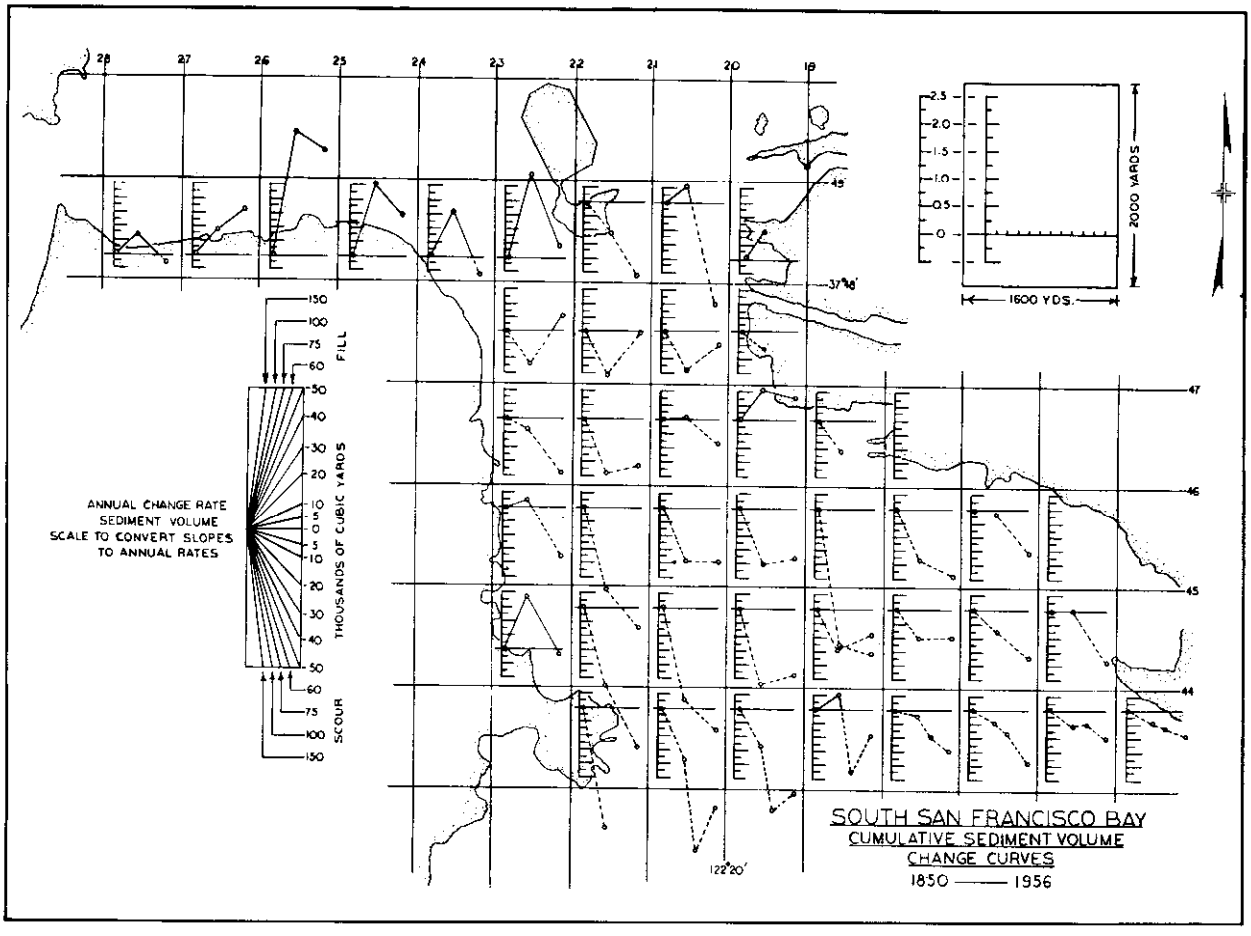



FIGURE 11. — Cumulative sediment volume change curves of South San Francisco Bay, 37° 49′ to 37° 43′ N.

and 1956, and a partial survey in 1931 are the basis for derivation of sediment volume changes in the South Bay, the largest bay in the system. The analysis is made over three zones: zone A covering 26 quadrilaterals from Treasure Island south to Hunters Point, lat. 37° 49′ to lat. 47° 44′; zone B covering 58 quadrilaterals from Hunters Point south to Point San Bruno, lat. 37° 44′ to lat. 37° 39′; and zone C covering 87 quadrilaterals from Point San Bruno south from lat. 37° 39′ to 37° 27′. Figures 11, 12, 13 and table 22 summarize the sediment volume changes.

Zone A, predominantly deep water, consistently shows scour over the past century, the change in mean depth being 1.7 feet. Scouring also occurs in zone B; light in the 40-year period 1855-96, a scour of 0.8 feet; continuing light to moderate, 0.9 foot, in the shorter period 1896-1922; and experiencing a light accretion, 0.15 foot, 1922-56. For zone C the general scour trend occurs after 1896, the period 1855-96 being one of silting (0.25 foot). The overall change for the South Bay is a scour of 173.5 million cubic yards, an erosion of 1 foot, over the period of 1855-1956.

### *San Francisco Bar*

Between 1855 and 1956 the San Francisco Bar outside the Golden Gate has lost 85 million cubic yards (fig. 14). This figure is the result of careful recomputation of all available data. Comparative data for gains and losses in the outer and inner slopes and the crest are given in table 23. The general consensus of authorities studying the bar is that it is composed of sand eroded from the coast, swept into the Golden Gate by the floodtide and swept out on the ebbtide radially. Bay mud apparently forms no part of the bar. That part of fluvial sediment and disturbed bottom sediment of the Bay system that is carried out to the ocean is presumably swept past the bar.

A complete analysis of the San Francisco Bar

is not attempted, discussion being limited to consideration of the bar relative to sedimentation within the system. Between 1855 and 1873 there appears to be a general shrinkage of volume except in the deeper inner zone. This shrinkage, relatively slight — 30 million yards out of a present total volume above the 60-ft. depth of 874 million — was accelerated to 182 million cubic yards lost for the 11-year period 1873-84. From 1884 on, a reversal to accretion occurred — 77 million between 1884 and 1900 and a further 50 million between 1900 and 1956. A study of cross sections at time intervals between 1855 and 1957 also shows no marked movement, relatively, of the bulk of the bar, although a certain amount on internal displacement between zones has occurred. It is possible that the net shrinkage of 85 million yards is but a translation of volume to depths below the 60-ft. contour on the outer slope. This aspect is under study at the present time.

### *Discussion of Historic Sedimentation Pattern*

Summarizing, the greater part (85.5 percent by weight) of the fluvial sediment load is delivered to the Delta. Direct inflows of sediment to the bays are 3.2 percent to Suisun Bay, 6.3 percent to San Pablo Bay, and 5.0 percent to San Francisco Bay. Upon entering the Bay system, the fluvial sediment loses the sand content and tends to flocculate as salinity increases. However, no simple pattern of deposition such as prevails for river regimens occurs within the bays and straits. Instead tide and wind effects and a long-term cyclic trend underlie the estuarial sedimentation processes.

### *Tidal Effects*

The principal tidal effect is a net movement of sediment downbay, the major part being disturbed bottom sediment from the bay bottom, the minor part being the fluvial bedload. This is deduced from the absence of any dilution effect in the sediment load analysis of fluvial flow

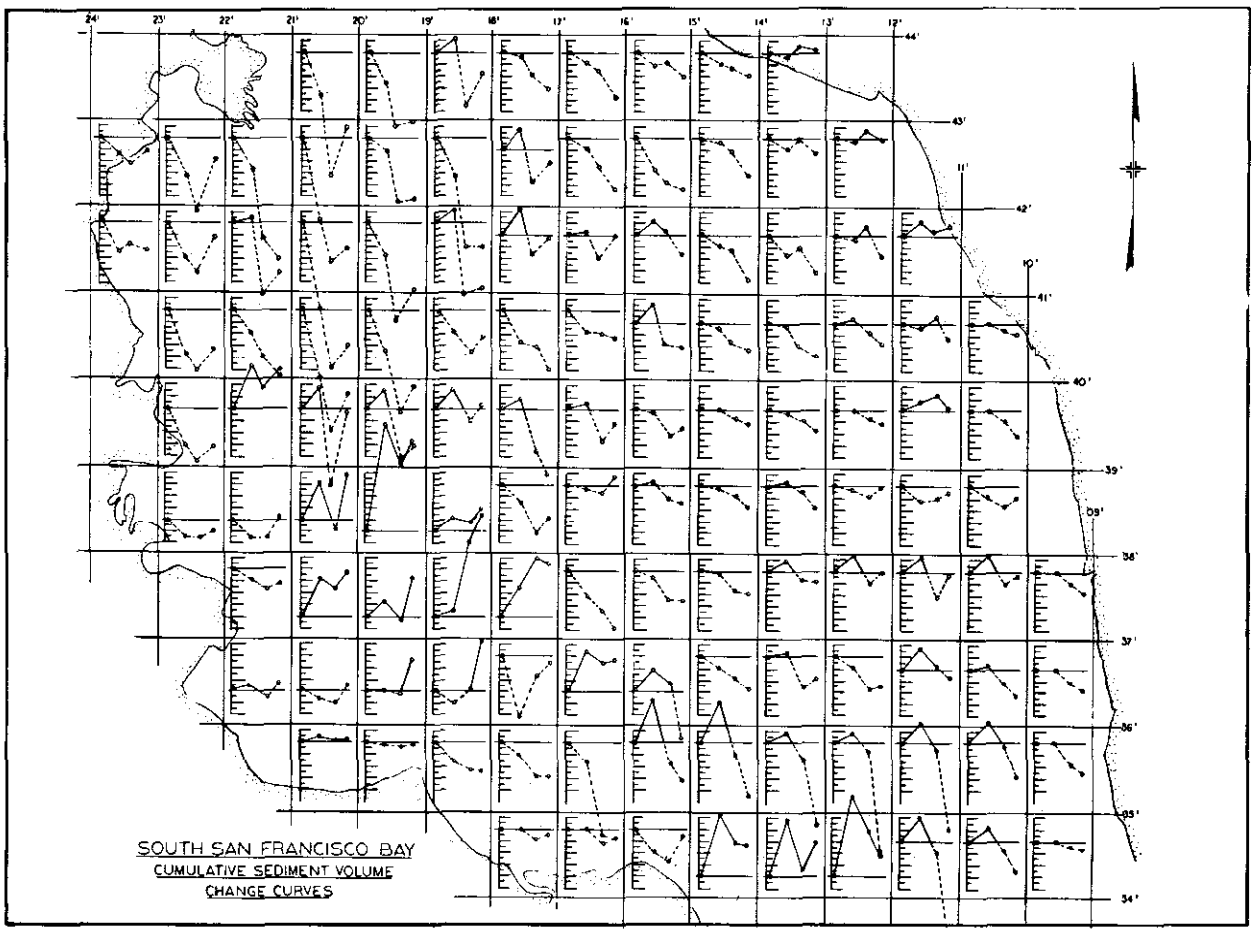



FIGURE 12. — Cumulative sediment volume change curves of South San Francisco Bay, 37° 43′ to 37° 35′ N.

sediment loadings and bay water sediment loadings. The effect of flocculation and reduction in grain-size dimensions results in lighter density bedload material. This is most marked in San Pablo Bay where flocculation is heaviest.

Movement downbay of both suspended load and bedload is the net result of the dominance of the ebb tidal competencies and capacities over those of the flood tide. This is illustrated in figure 15, which shows the high concentrations of sediment at peak flood and ebb and the dominance of the ebb over the flood.

### *Wind Effects*

The effect of wind action on sedimentation over the large expanses of the several bays can be inferred from analysis of the deposition patterns by depth ranges. For shallow depths, wind wave turbulence would be pronounced. The stirring-up effect of the wave action would be augmented by the bottom return flow, opposite in direction to the wind-induced surface flow. Progressively as the fetch increased, the turbulence and volume of sediment in suspension would increase, reducing the potential for deposition. For Suisun Bay the prevailing winds are from the northwest with Honker Bay the farthest east, Grizzly Bay next, and Suisun Slough farthest west. Hence, for the same periods, siltation over the shallow areas should be greatest in the most westerly parts and lightest in the most easterly.

This assumes not too marked a variation in salinity over the area. From table 18 the cumulative sediment volume changes show deposition occurring in descending order of volume for the shallow areas from highs in the western to lows in the eastern parts of the Bay, the least deposition occurring in Honker Bay, the farthest east. For the period 1867-86 this effect is pronounced, the ratio of deposition per unit area of shallows being approximately 4:1 from west (Suisun Slough and West Bay) to east (Honker Bay). The effect is masked in the

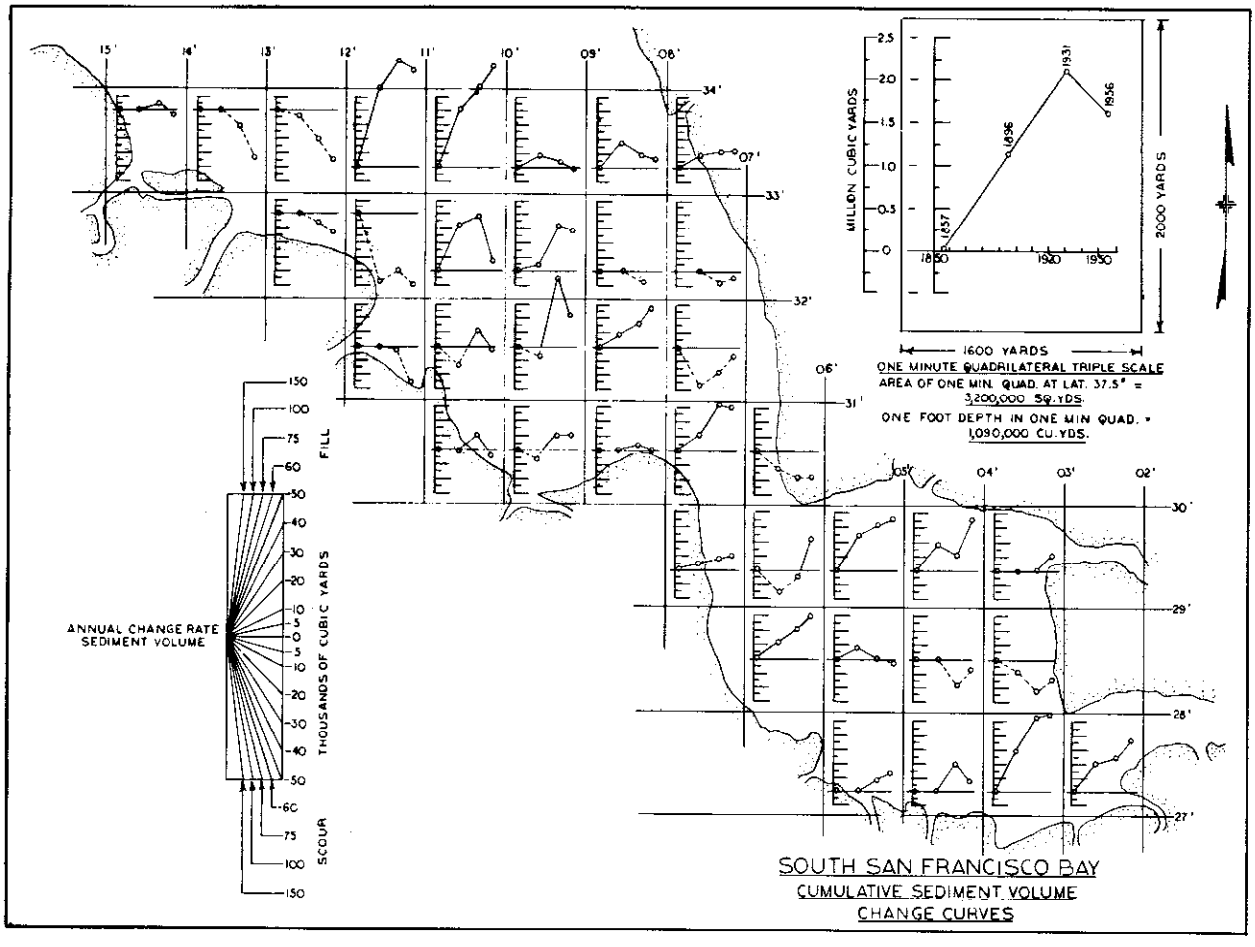



FIGURE 13.—Cumulative sediment volume change curves of South San Francisco Bay, 37° 35′ to 37° 27′ N.

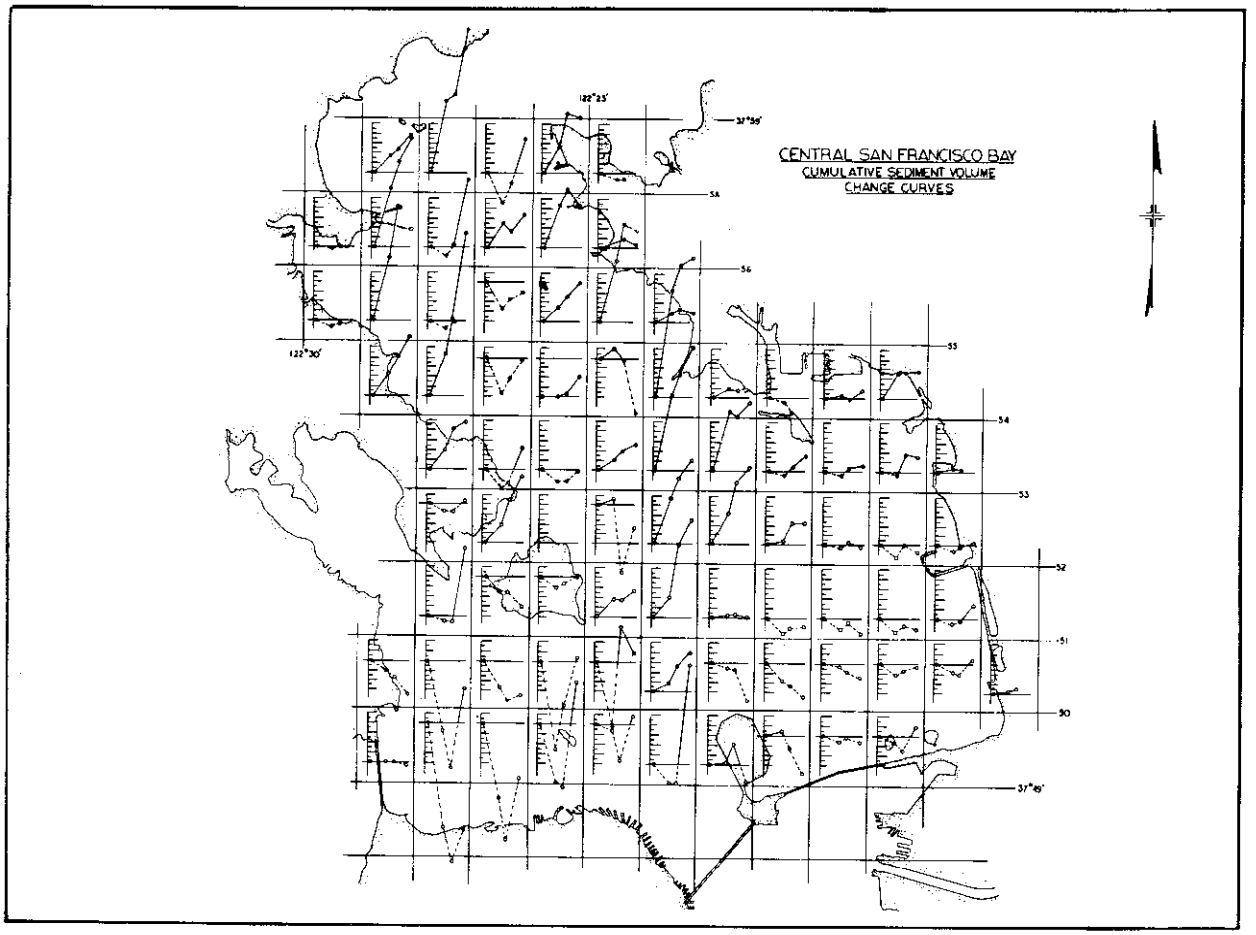



FIGURE 14. — Cumulative sediment volume change curves of Central San Francisco Bay.

drought period 1921–30 by changed flocculation conditions when the saline-fresh water interface apparently moved from San Pablo Bay into Suisun Bay.

When the shallow areas are grouped for San Pablo Bay, the rate of the deposition (see table 20) for the marginal shoals (0 to 4 feet) is 2.2 times that for the intermediate shoals (4 to 20 feet). This is not so apparent by inspection of figures 8 and 9, due to the orientation of the shallows and also the masking of the wind-induced turbulence effect by the fluctuations of salinity that occur across the Bay as the saline interface moves with seasonal fluctuations of fresh water inflows. The construction of dike 12 from the southern end of Mare Island westerly from 1,000 feet in 1908 to a total length of 12,800 feet between 1937 and 1951 materially altered the deposition pattern in the vicinity of Mare Island. Data on changes in bottom elevations along the dike are given in table 24. The accentuated accretion after 1896 can be attributed to the impounding effect of the dike.

The detailed aspects of shoaling in Mare Island Strait and the South and North Bays are presented by W. J. Homan and E. A. Schultz (*9*) and are here discussed in general terms only.

Wind wave effects on sedimentation in North San Francisco Bay, the central Bay of the system, and in South Bay are masked by the complex interactions of tidal currents and phases. The deposition rates of the marginal shoals, 0 to 10 feet deep, in the North Bay are markedly less (table 21) than the rates for the intermediate shoals, 10 to 30 feet deep. However, the Berkeley flats that constitute most of the marginal shoals are to the extreme east of the Bay where the fetch is greatest from the west. Both these flats and the Southampton Shoal are probably more affected by the tidal complex. Similarly, for the South Bay no adequate explanation is yet available for the formation of San Bruno Shoal and the silting in the vicinity of Alameda

TABLE 22.—*Cumulative volume and mean depth changes, South San Francisco Bay, 1855–1956*

| Zone volume and depth | 1855–96 | 1922 | 1931 | 1956 |
|---|---|---|---|---|
| A, Treasure Island to Hunters Pt.: | | | | |
| Volume, million cu. yds. | −29.3 | ..... | ..... | −47.7 |
| Depth, ft. | − 1.10 | ..... | ..... | − 1.70 |
| B, Hunters Pt. to Pt. San Bruno: | | | | |
| Volume, million cu. yds. | −51.5 | −106.5 | ..... | −95.9 |
| Depth, ft. | − 0.80 | − 1.70 | ..... | − 1.55 |
| C, Pt. San Bruno south: | | | | |
| Volume, million cu. yds. | +25.2 | ..... | −14.5 | −29.9 |
| Depth, ft. | + 0.25 | ..... | − 0.15 | − 0.30 |
| Total: | | | | |
| Volume, million cu. yds. | −55.6 | ..... | ..... | −173.5 |
| Depth, ft. | − 0.30 | ..... | ..... | − 1.00 |

Island. Figures 16, 17, 18, and 19, showing the shoal areas and typical cross section of the major bays and the wind roses at strategic points, indicate the extensive areas of the system subject to wind action.

### *Interactioning Within the System*

An interchange of water volumes, concentrated at the turn of the tides, occurs between the South Bay and the north bays with consequent interchange of sediment volumes. Much more coordinated prototype data coupled with model tests are needed to prove the workings of this sediment interchange. Figure 20 and table 25 present data of interest but are inadequate to form a base for firm conclusions. Over the 101-year period, the South Bay has undergone a net scour of 173.5 million cubic yards, while the North Bay has experienced a net gain of 166.1 million yards, a virtual balance.

Further indications of interactioning within the system are the relations between sedimentation in Suisun and San Pablo Bays. In this instance the coincidence of survey dates with the dry period 1922-30 is advantageous. Between 1886 and 1941 Suisun Bay was subject to scour except for the dry period 1922-30. San Pablo Bay between 1886 and 1951 experienced accretion except during 1922-30, when scour occurred the reverse to Suisun Bay. There is little doubt that normally the heavy sedimentation in San Pablo Bay reflects heavy flocculation and that during 1922-30 the migration into Suisun Bay of the saline–fresh water interface resulted in the reversed pattern for both bays.

TABLE 24.—*Historical bottom elevations (MLLW) along north side of dike 12, certain years*

| Year | Distance along dike crest from east end | | | | |
|---|---|---|---|---|---|
| | 200 feet | 2,000 feet | 7,500 feet | 8,500 | 200 ft. beyond end of dike |
| | *Feet* | *Feet* | *Feet* | *Feet* | *Feet* |
| 1856 | −7 | −33 | −27 | −27 | −19 |
| 1878 | ..... | ..... | ..... | ..... | −20 |
| 1896 | −4 | −25 | −27 | −26 | −23 |
| 1922 | ..... | ..... | ..... | −24 | −20 |
| 1951 | (1) | +2 | +2 | 0 | −8 |

[1] Shoreline.

### *Environmental Aspects*

From the biological environmental viewpoint, the most important character of the Bay system is the prevailing shallowness of the bays, with a few such exceptions as Golden Gate, Carquinez Strait, and Raccoon Strait. Depths 15 feet and less below mean sea level occupy 70 percent of the total water area, and at —20 feet the water would be but 21 percent of the total at mean sea level. This condition can be highly productive

TABLE 23.—*San Francisco Bar and adjacent offshore area sediment volume change*

| Area, volume, and depth | 1855–73 | 1873–84 | 1873–1900 | 1884–1900 | 1900–1956 | Cumulative sediment volume change |
|---|---|---|---|---|---|---|
| Inner slope above 60 ft.—11 quads.: | | | | | | |
| Volume ... million cubic yards | −4.4 | −61.5 | −17.6 | +43.9 | +29.0 | +7.0 |
| Depth ... feet | −0.36 | −5.16 | −1.47 | +3.69 | +2.44 | ..... |
| Crest inside 36-foot contour—11 quads.: | | | | | | |
| Volume ... million cubic yards | −18.2 | −18.2 | −10.3 | +7.9 | +11.0 | −17.4 |
| Depth ... feet | −1.53 | −1.53 | − .87 | +0.66 | +0.93 | ..... |
| Outer slope above 60 ft.—18 quads.: | | | | | | |
| Volume ... million cubic yards | −20.4 | −63.6 | −53.1 | +10.5 | −8.3 | −81.8 |
| Depth ... feet | −1.04 | −3.26 | −2.73 | +0.54 | −0.43 | ..... |
| Miscellaneous—below 60 ft.—10 quads.: | | | | | | |
| Volume ... million cubic yards | +13.1 | −38.5 | −23.6 | +14.9 | +17.7 | +7.2 |
| Depth ... feet | +1.21 | −3.55 | −2.18 | +1.38 | +1.63 | ..... |
| Entire 50 quads.: | | | | | | |
| Volume ... million cubic yards | −29.9 | −181.8 | −104.6 | +77.2 | +49.5 | −85.0 |
| Depth ... feet | −0.54 | −3.36 | −1.93 | +1.43 | +0.91 | −1.5 |

TABLE 25.—*Adjusted historical sedimentation, San Francisco Bay System, 1855–1956*

| Area | Cycle | | | | | | Net sedimentation, 101 years[1] | |
|---|---|---|---|---|---|---|---|---|
| | Accretion | | | Scour | | | | |
| | Years | Rate | Volume | Years | Rate | Volume | Rate | Total volume |
| | | *Feet* | *10⁶ cu. yd.* | | *Feet* | *10⁶ cu. yd.* | | |
| Suisun Bay, except Grizzly Bay | 1855– 86 (31) | + 1.85 | + 57.4 | | | | | |
| Do | | | | 1886–1922 (36) | − 0.95 | −34.2 | | |
| Do | 1922– 30 (8) | + 1.24 | + 9.9 | | | | | |
| Do | | | | 1930– 56 (26) | − 1.57 | −40.1 | | |
| Grizzly Bay | 1855– 86 (31) | + 1.68 | + 52.1 | | | | | |
| Do | | | | 1886–1956 (70) | − .02 | − 1.4 | | |
| Subtotal | | | +119.4 | | | −75.7 | +0.43 | + 43.7 |
| Carquinez Strait | 1855– 86 (31) | + 2.41 | + 74.7 | | | | | |
| Do | 1886–1922 (36) | + .30 | + 11.0 | | | | | |
| Do | 1922– 56 (34) | + .62 | 21.1 | | | | | |
| Subtotal | | | +106.8 | | | | +1.06 | +106.8 |
| San Pablo Bay | 1855– 87 (32) | +10.45 | +334.4 | | | | | |
| Do | 1887– 99 (12) | + 41 | + 4.9 | | | | | |
| Do | 1899–1922 (23) | + 2.66 | + 61.1 | | | | | |
| Do | | | | 1922– 30 (8) | −11.2 | −89.6 | | |
| Do | 1930– 56 (26) | + 5.35 | +139.1 | | | | | |
| Subtotal | | | +539.5 | | | −89.6 | +4.45 | +449.9 |
| North San Francisco Bay | | | | 1855– 95 (40) | − .063 | − 2.5 | | |
| Do | 1895–1916 (21) | + 2.23 | + 46.9 | | | | | |
| Do | 1916– 56 (40) | + 3.8 | +152.0 | | | | | |
| Subtotal | | | +198.9 | | | − 2.5 | +1.94 | +196.4 |
| Golden Gate | | | | 1855– 73 (18) | − 0.45 | − 8.1 | | |
| Do | | | | 1873– 94 (21) | − 1.68 | − 35.3 | | |
| Do | 1894–1956 (62) | + 0.53 | + 32.6 | | | | | |
| Subtotal | | | + 32.6 | | | — 43.4 | −0.11 | − 10.8 |
| South San Francisco Bay | | | | 1855– 96 (41) | − 1.39 | − 55.6 | | |
| Do | | | | 1896–1956 (60) | − 1.965 | −117.9 | | |
| Subtotal | | | | | | −173.5 | −1.72 | −173.5 |
| Total, San Francisco Bay | 1855–1956 | + 9.87 | +997.2 | 1855–1956 | − 3.81 | −384.7 | +6.06 | +612.5 |
| San Francisco Bar | | | | 1855– 73 (18) | − 1.66 | − 29.9 | | |
| Do | | | | 1873– 84 (11) | −16.53 | −181.8 | | |
| Do | 1884–1900 (16) | + 4.83 | + 77.2 | | | | | |
| Do | 1900– 56 (56) | + .88 | + 49.5 | | | | | |
| Subtotal | | | +126.7 | | | −211.7 | − .84 | − 85.0 |
| Total, Bay plus San Francisco Bar | 1855–1956 | 11.13 | +1,123.9 | 1855–1956 | − 5.91 | −596.4 | +5.22 | +527.5 |

[1] Adjustment to 101 years made by extending back to 1855 or forward to 1956 for areas lacking data from 1855 or to 1956 at the rates of the nearest cycle in time.

for aquatic life, emergent and nonemergent vegetation, and shore life. Normally estuarine conditions inhibit great population concentrations of marine life, but for the Bay system large areas of relative quiescence are conducive to bottom life such as oysters. The productive oyster bed area for the system has been estimated at 170,000 acres of bay floor with a potential yield up to 400 million pounds a year (*6*). Also the mud flats and shallows, in addition to oysters and clams, support shore bird life.

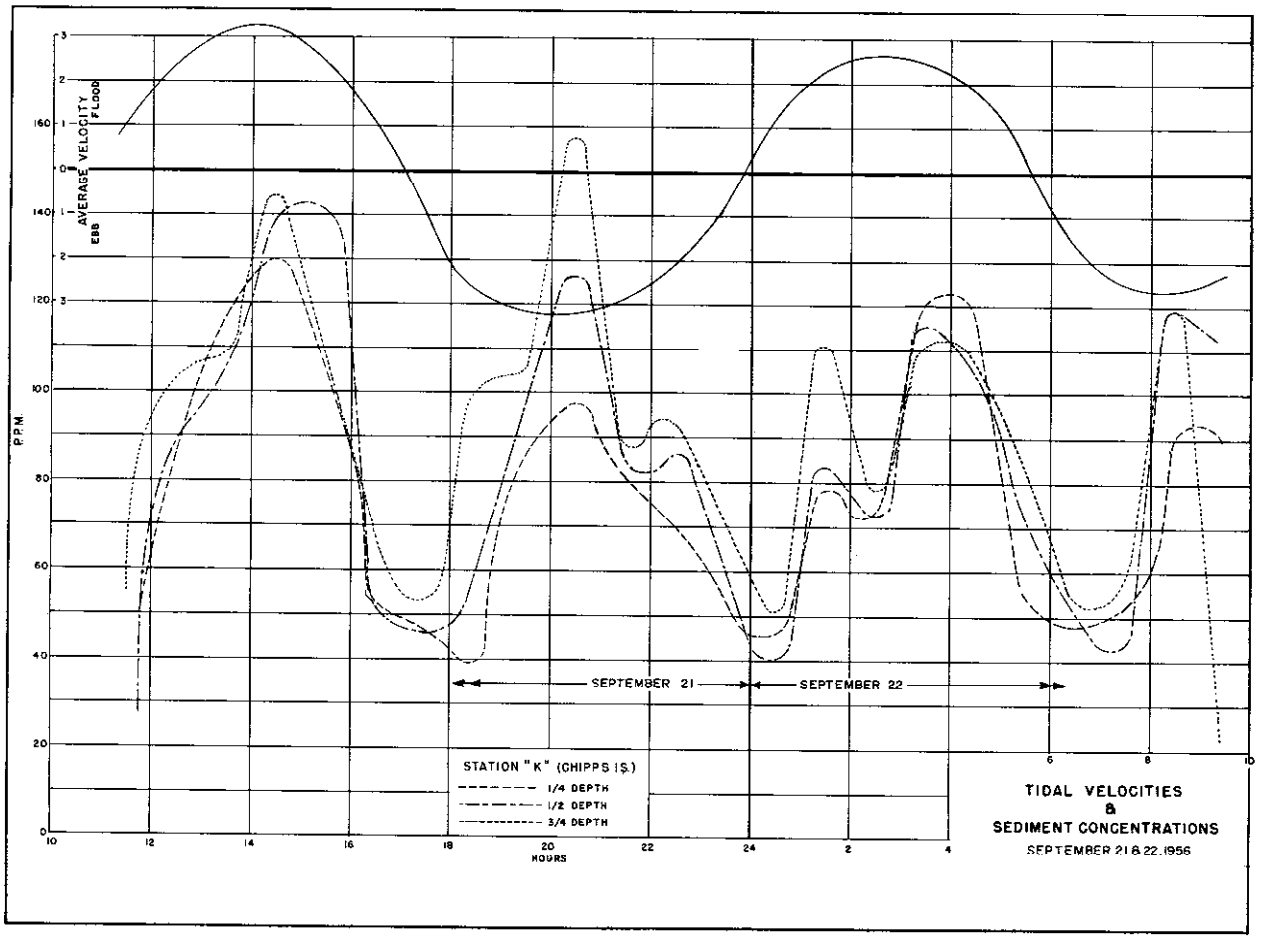



FIGURE 15.—Tidal velocities, phases, and sediment concentrations near Chipp Island, September 21 and 22, 1956.

Sedimentation by affecting turbidity and being the chief carrier of nutrient material for the estuarine biota is an important basic factor in the biotic environment. Parallel factors of equal rank are the saline and temperature gradients throughout the system (*4*, *5*).

Hence, any major displacement of sediment volumes between bays resulting from cyclic changes such as the wet-drought cycle or artificial redeployment of basic areal characteristics would have major effects on the environmental conditions sustaining the biota. Essentially such major displacement would come from altered salinity, thereby changing the flocculation pattern, altered tidal current strengths, direction and phasing affecting deposition and turbidity, altered bottom configurations affecting depths, and corresponding habitats.

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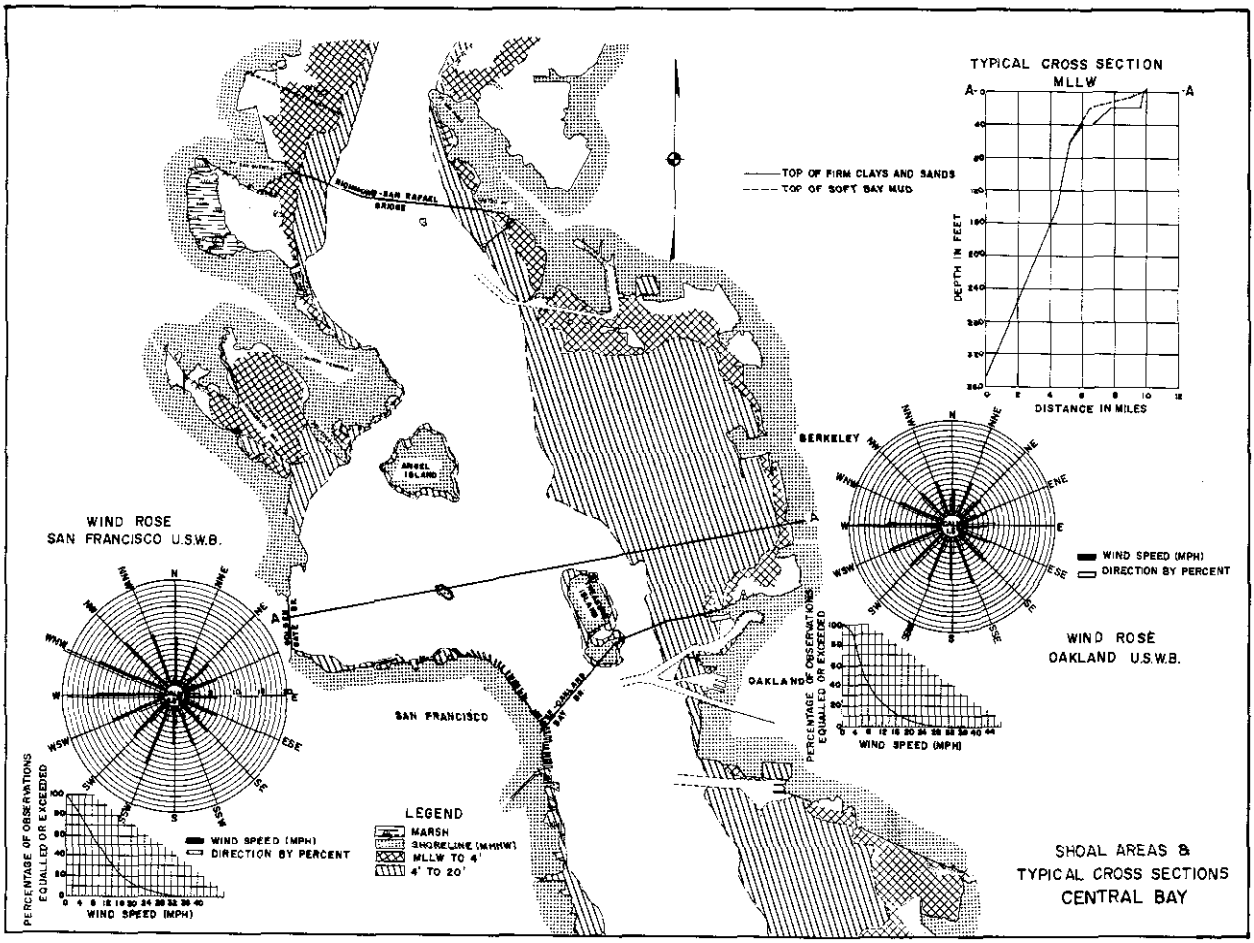


FIGURE 16. — Shoal areas, wind roses, and typical cross section of Central San Francisco Bay.


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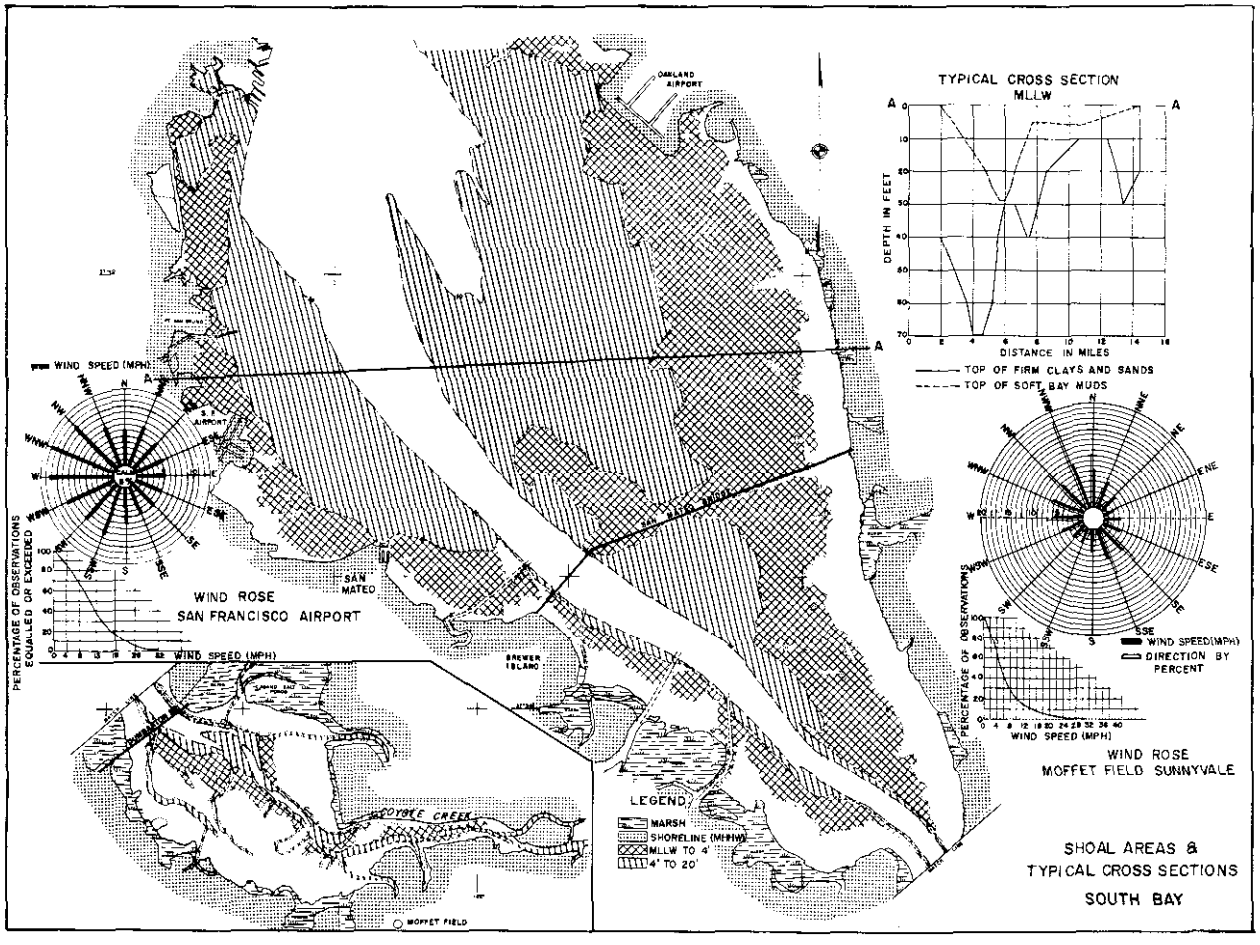


FIGURE 17. — Shoal areas, wind roses, and typical cross section of South San Francisco Bay.

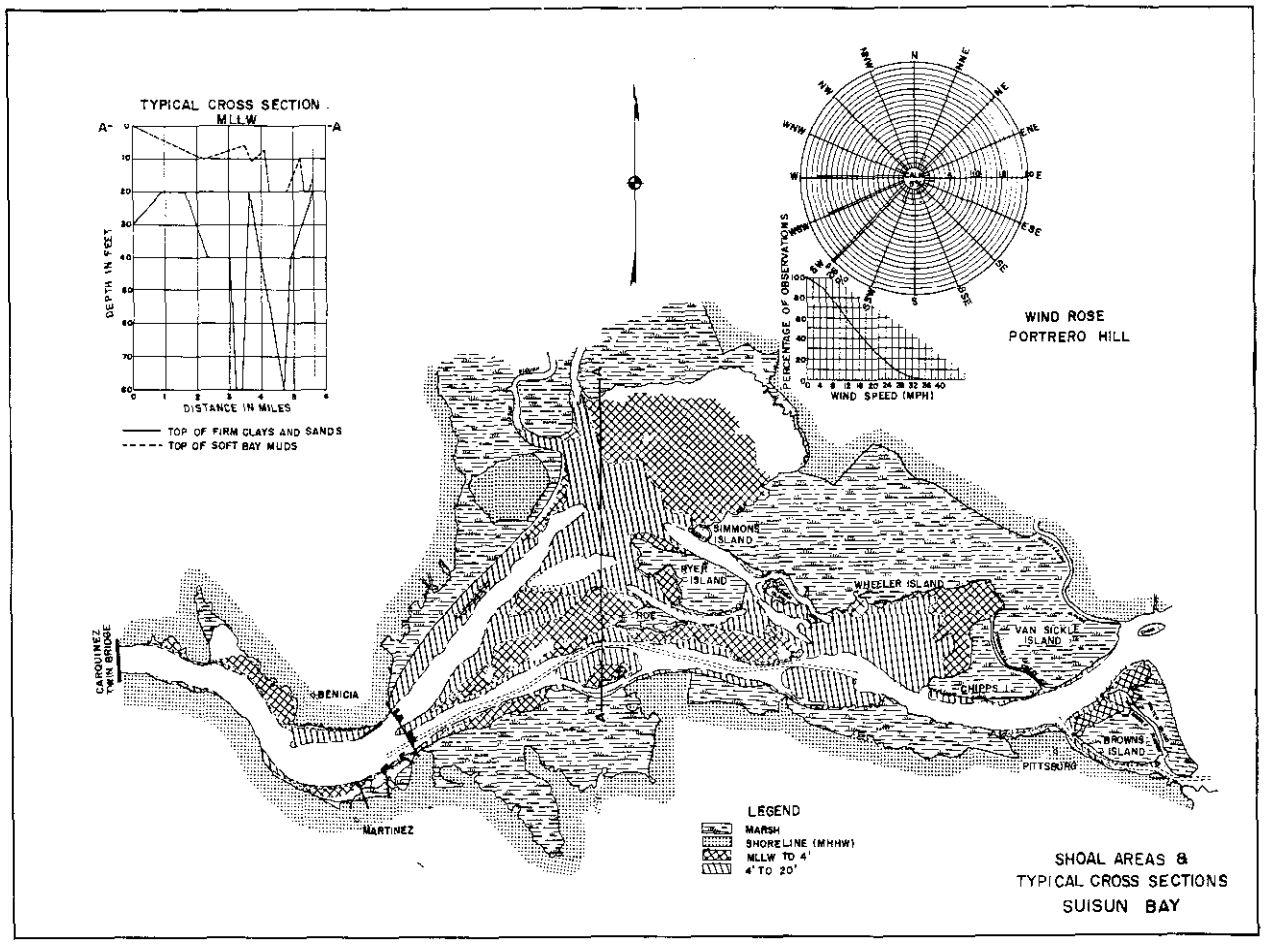


FIGURE 18. — Shoal areas, wind roses, and typical cross section of Suisan Bay.

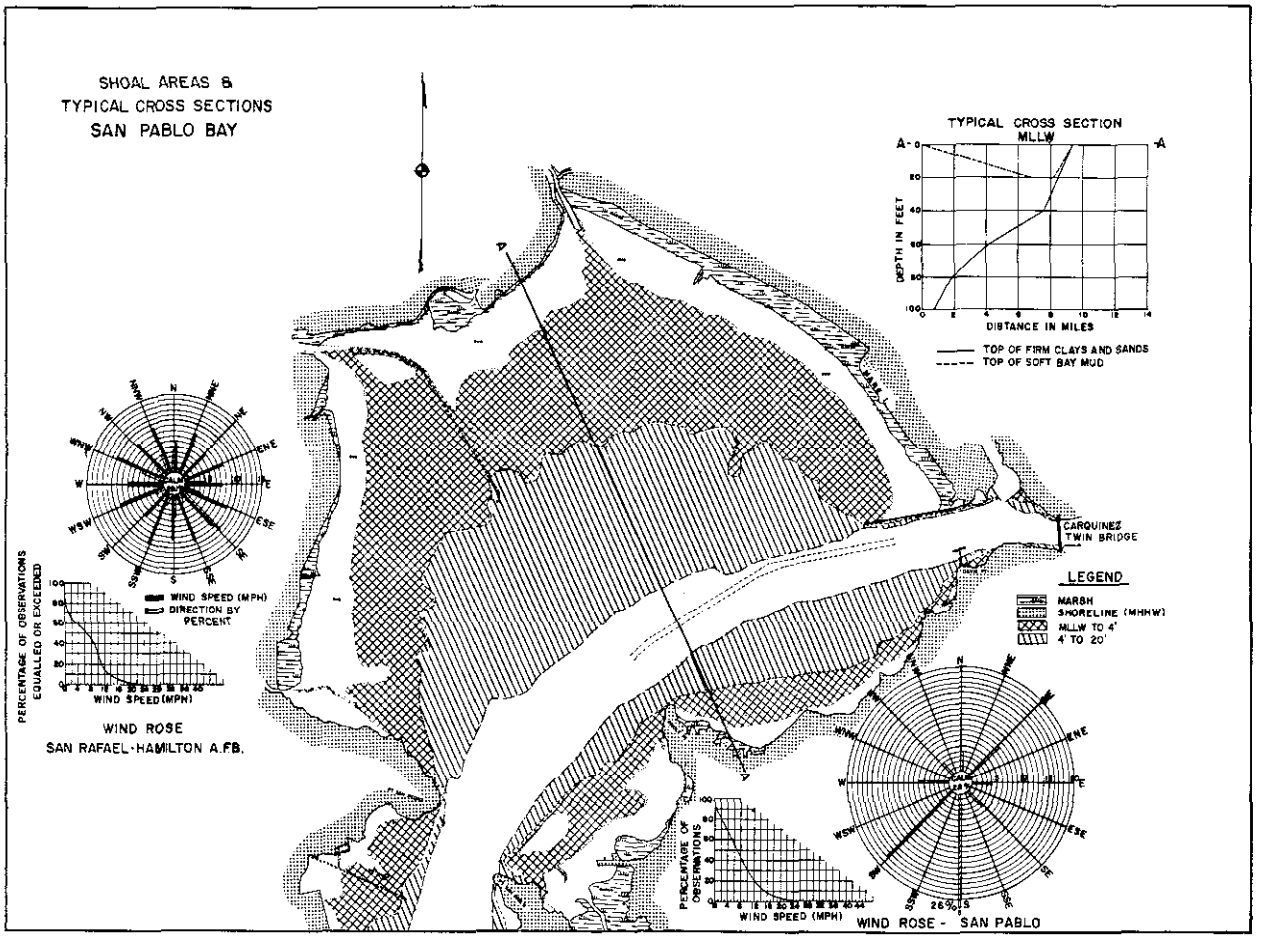


FIGURE 19. — Shoal areas, wind roses, and typical cross section of San Pablo Bay.

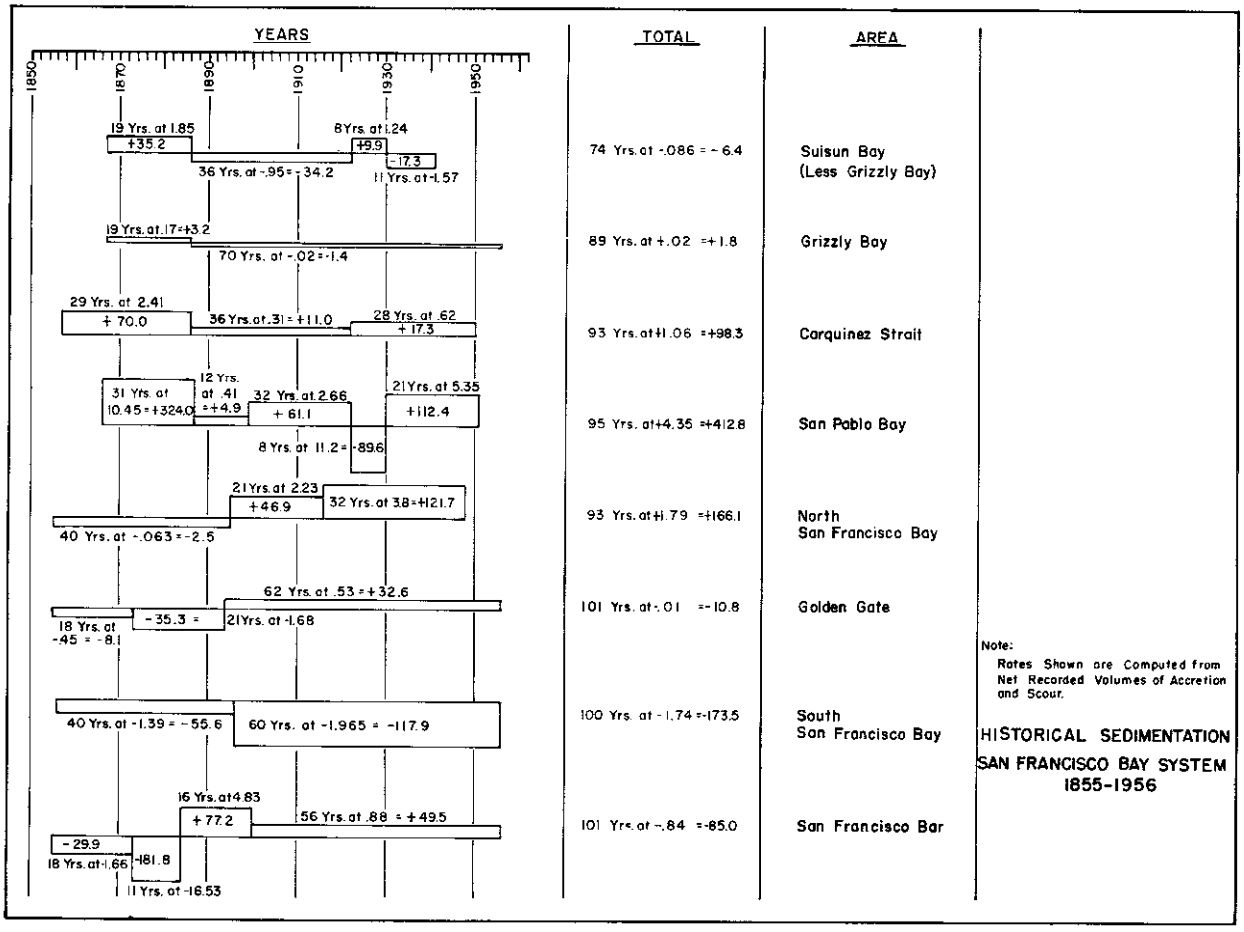


FIGURE 20. — History of sedimentation in San Francisco Bay system, 1855-1956.

# MODEL TESTS OF SHOALING AND OF DREDGE SPOIL DISPOSAL IN SAN FRANCISCO BAY

**[Paper No. 71]**

By W. J. HOMAN, *supervisory civil engineer (general)*, and E. A. SCHULTZ, *supervisory hydraulic engineer (research), U.S. Army Engineer District, San Francisco*

## Abstract

San Francisco Bay has an area of 486 square miles. The tributary area is 62,920 square miles. Much of the Bay is very shallow: 70 percent is less than 20 feet deep. The average annual silt deposition rate has been about 6,000,000 cubic yards. Estimated future sediment inflow is 5,840,000 tons a year.

There are 17 authorized navigation projects in the Bay system below Carquinez Strait. The Federal Government spends about $2,270,000 annually for maintenance dredging. Dredge spoils are dumped in deep sections of the Bay.

Model tests indicate approximately 30 percent of the material dredged from the Oakland-Alameda area is carried to sea and about 4 percent returns to navigation channels; and that dredge spoils can be dumped more than 1 mile closer to the point of origin without detrimental results. Model tests of Mare Island Strait indicate similar possibilities. Material originally deposited in shallow areas may be resuspended by wave action and returned to navigation channels.

Model tests of remedial works at the Alameda Naval Air Station indicate possible sediment reductions of as much as 69 percent.

Savings in maintenance dredging as a result of model studies are expected to far exceed the cost of such studies.

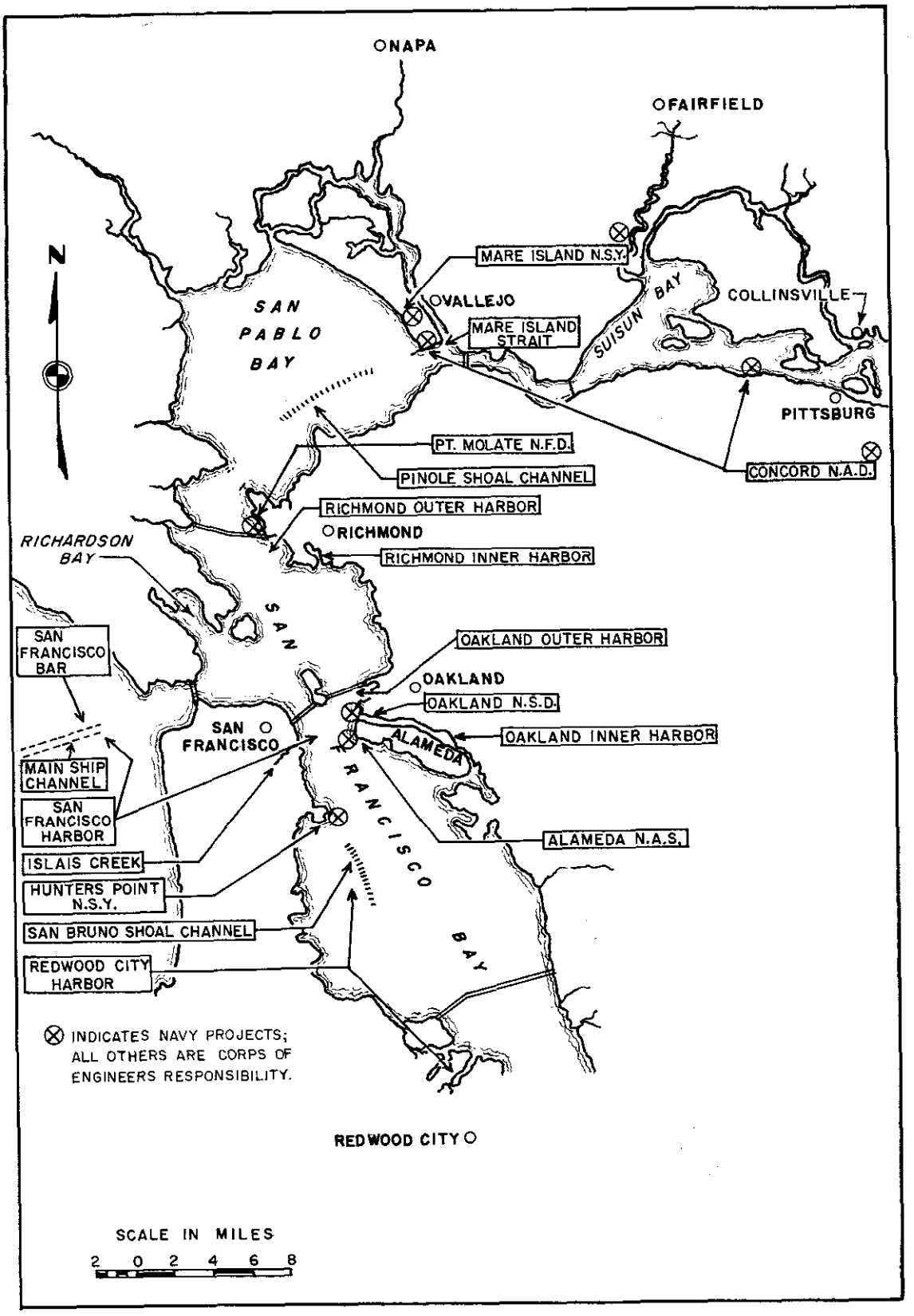


FIGURE 1. — San Francisco Bay.

## Introduction

### *The Bay*

San Francisco Bay, shown in figure 1, contains one of the largest and most important port complexes on the west coast of the United States, and is also the discharge point for the largest drainage basin in the State of California — the Sacramento-San Joaquin River system, which drains the great Central Valley of California. The active area contributing to the discharge into San Francisco Bay contains 48,883 square miles. There is an additional area of 14,037 square miles that is normally landlocked, but which, during extreme wet cycles, might contribute to the discharge into the Bay. The total tributary area is thus 62,920 square miles, or about 40 percent of the total area of the State. This area is shown in table 1.

TABLE 1.—*Area tributary to San Francisco Bay*

| Area | Drainage size |
|---|---|
| | *Square miles* |
| Sacramento River Basin | 26,322 |
| San Joaquin River Basin | 19,096 |
| Small streams discharging directly into Bay | 3,465 |
| Total | 48,883 |
| Tulare Lake Basin (south of San Joaquin Basin) | 13,625 |
| Goose Lake Basin (north of Sacramento Basin) | 412 |
| Grand total | 62,920 |

Although the channel through the Golden Gate has a maximum depth of about 385 feet, San Francisco Bay, as a whole, is relatively shallow and bordered by extensive areas of "tidelands" that are exposed at low tide. There is also a crescent-shaped bar about 5 miles seaward from the Golden Gate. The natural water depth at the deepest point over the crest of this bar is about 34 feet at mean lower low water. The area inside the Golden Gate below the junction of the Sacramento and San Joaquin Rivers at Collinsville, which excludes the Sacramento-San Joaquin Delta with an area of about 34,000 acres at mean sea level, at various elevations, and percentage of area at mean sea level at these same elevations is shown in table 2.

### *Sedimentation*

A recent estimate, by the U. S. Geological Survey, of silt expected to be discharged into the Bay system under future conditions is 5,840,000 tons a year. About 1,940,000 tons (1,600,000 cu. yd. at 90 lb. per cu. ft.) is deposited in the Sacramento-San Joaquin delta, leaving a balance of 3,900,000 tons entering the Bay proper. This is made up chiefly of fine particles in a collodial or semicolloidal suspension. Much of this material is precipitated by flocculation on contact with salt water and deposited in the Bay. Sediment originally deposited in shallow areas may be resuspended by wind and wave action and a part of material thus resuspended may be deposited in navigation channels or carried to sea through the Golden Gate. The total volume of shoals formed in the Bay is dependent on the inplace density developed by the sediment. Available data indicate average inplace densities ranging from 30 to 40 pounds per cubic foot, which is equal to a total sediment inflow between 7.2 and 9.6 million cubic yards a year.

TABLE 2.—*Area at various elevations in San Francisco Bay, 1962*

| Elevation | Area | Area | Area at mean sea level |
|---|---|---|---|
| | *Acres* | *Square miles* | *Percent* |
| Mean sea level | 278,600 | 435 | 100 |
| Mean higher high water | 294,400 | 460 | 106 |
| Mean lower low water | 253,400 | 396 | 91 |
| − 5 feet mean sea level | 199,000 | 311 | 71 |
| −10 feet mean sea level | 138,500 | 216 | 50 |
| −15 feet mean sea level | 101,700 | 159 | 37 |
| −20 feet mean sea level | 87,600 | 137 | 32 |

A study of the historical shoaling pattern of San Francisco Bay shows somewhat extreme variations. These studies also show nonuniform distribution throughout the Bay; in fact, there have been periods of degradation in almost all sections of the Bay and there appears to be a long time trend of degradation in the South Bay. Studies of the shoaling pattern and trend for the 101-year period from 1855-1956 indicate an average annual deposition of 6,060,000 cubic yards for the Bay as a whole. These studies are based on a comparison of water depths as obtained from navigation charts and surveys by the U.S. Coast and Geodetic Survey. There is some question as to the accuracy of some of this work, due in part to changes in both vertical and horizontal controls. However, findings are believed to be indicative of long time trend. Findings for the 101-year period are shown in table 3.

### *Tides*

Tides in the San Francisco Bay system are of the mixed type; there being two high tides and two low tides during each lunar day of 24 hours and 50 minutes. There is a marked and constantly changing difference in these tides. There is likewise a difference in the type of tide in the southern section of the Bay and in the central and northern section and up the Sacramento and San Joaquin River channels. Water surface elevations in the Sacramento River are affected by tides as far upstream as the vicinity of Knights Landing, some 20 miles above Sacra-

TABLE 3.—*Adjusted historical sedimentation, San Francisco Bay system, 1855–1956*

| Area | Cycle | | | | | | Net sedimentation, 101 years[1] | |
|---|---|---|---|---|---|---|---|---|
| | Accretion | | | Scour | | | | |
| | Years | Rate | Volume | Years | Rate | Volume | Rate | Total volume |
| | | *Feet* | *10⁶ cu. yd.* | | *Feet* | *10⁶ cu. yd.* | *Feet* | *10⁶ cu. yd.* |
| Suisun Bay, except Grizzly Bay | 1855– 86 (31) | + 1.85 | + 57.4 | | | | | |
| Do | | | | 1886– 22 (36) | − 0.95 | −34.2 | | |
| Do | 1922– 30 (8) | + 1.24 | + 9.9 | | | | | |
| Do | | | | 1930– 56 (26) | − 1.57 | −40.1 | | |
| Grizzly Bay | 1855– 86 (31) | + 1.68 | + 52.1 | | | | | |
| Do | | | | 1886– 56 (70) | − .02 | − 1.4 | | |
| Subtotal | | | +119.4 | | | −75.7 | 0.43 | + 43.7 |
| Carquinez Strait | 1855– 86 (31) | + 2.41 | + 74.7 | | | | | |
| Do | 1886– 22 (36) | + .30 | + 11.0 | | | | | |
| Do | 1922– 56 (34) | + .62 | 21.1 | | | | | |
| Subtotal | | | +106.8 | | | | +1.06 | +106.8 |
| San Pablo Bay | 1855– 87 (32) | +10.45 | +334.4 | | | | | |
| Do | 1887– 99 (12) | + .41 | + 4.9 | | | | | |
| Do | 1899– 22 (23) | + 2.66 | + 61.1 | | | | | |
| Do | | | | 1922– 30 (8) | −11.2 | −89.6 | | |
| Do | 1930– 56 (26) | + 5.35 | +139.1 | | | | | |
| Subtotal | | | +539.5 | | | −89.6 | +4.45 | +449.9 |
| North San Francisco Bay | | | | 1855– 95 (40) | −0.063 | − 2.5 | | |
| Do | 1895– 16 (21) | +2.23 | + 46.9 | | | | | |
| Do | 1916– 56 (40) | + 3.8 | +152.0 | | | | | |
| Subtotal | | | +198.9 | | | − 2.5 | +1.94 | +196.4 |
| Golden Gate | | | | 1855– 73 (18) | −0.45 | − 8.1 | | |
| Do | | | | 1873– 94 (21) | −1.68 | − 35.2 | | |
| Do | 1894– 56 (62) | + .53 | + 32.6 | | | | | |
| Subtotal | | | + 32.6 | | | − 43.4 | − .11 | − 10.8 |
| South San Francisco Bay | | | | 1855– 96 (41) | −1.39 | − 55.6 | | |
| Do | | | | 1896– 56 (60) | −1.965 | −117.9 | | |
| Subtotal | | | | | | −173.5 | −1.72 | −173.5 |
| Total, entire San Francisco Bay | 1855– 56 | + 9.87 | +997.2 | 1855– 56 | − 3.81 | −384.7 | +6.06 | +612.5 |
| San Francisco Bar | | | | 1855– 73 (18) | −1.66 | − 29.9 | | |
| Do | | | | 1873– 84 (11) | −16.53 | −181.8 | | |
| Do | 1884– 00 (16) | + 4.83 | + 77.2 | | | | | |
| Do | 1900– 56 (56) | + .88 | +49.5 | | | | | |
| Subtotal | | | +126.7 | | | −211.7 | − .84 | − 85.0 |
| Total, Bay plus Bar | 1855–1956 | 11.13 | +1,123.9 | 1855–1956 | −5.91 | −596.4 | +5.22 | +527.5 |

[1] Adjustment to 101 years made by extending back to 1855 or forward to 1956 for area lacking data from 1855 or to 1956 at the rates of the nearest cycle in time.

mento; and on the San Joaquin River to a few miles upstream from the Mossdale Bridge where Highway 50 crosses the river. Tides in the South Bay are predominantly of the standing wave type, which results in very poor circulation in the southern section of the South Bay. Those in the central and northern section are of the progressive wave type. Tidal heights are shown on the tidal reference plane chart figures 2 and 3. Tidal elevations referred to mean sea level at the Golden Gate and ranges at various key points are shown in table 4.

### *Maintenance Dredging*

The Federal Government spends about $2,270,000 annually for maintenance dredging in

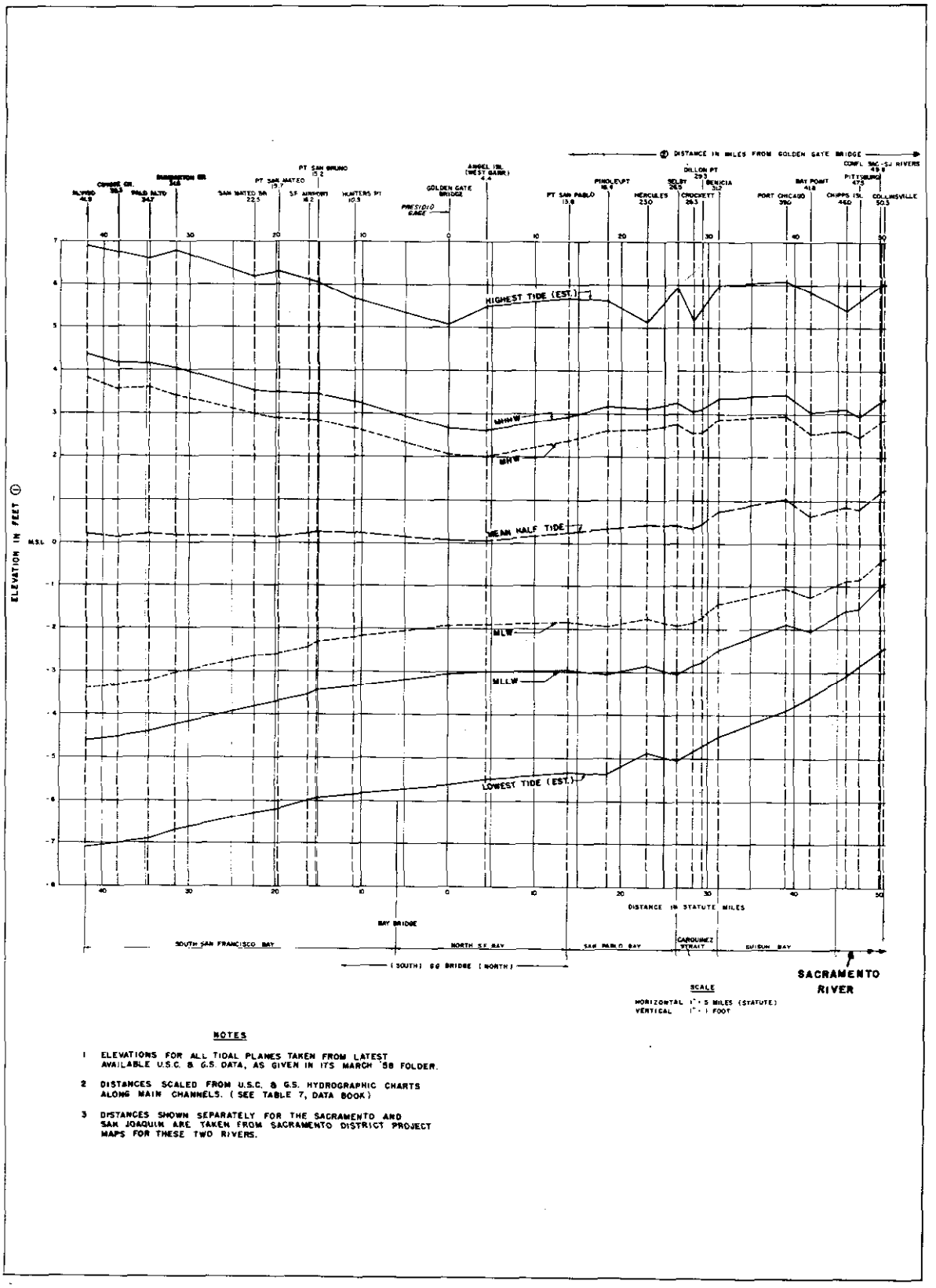


FIGURE 2. — Tidal reference plane chart, San Francisco Bay.

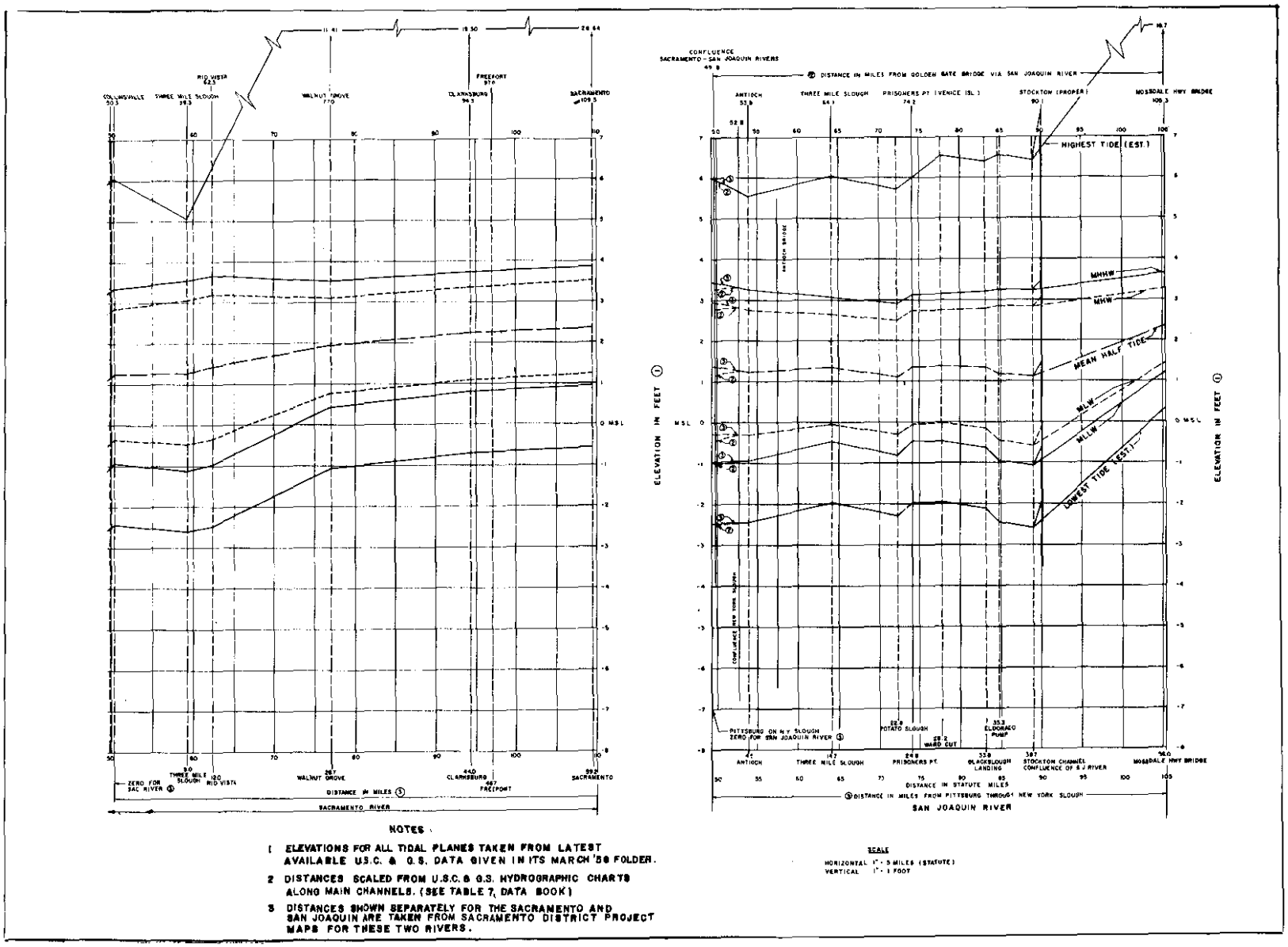


FIGURE 3.—Tidal reference plane charts, Sacramento and San Joaquin Rivers.

TABLE 4.—*Tidal elevations and ranges in San Francisco Bay*

| Location | Mean higher high water | Mean lower low water | Mean range | Estimated maximum high water | Estimated minimum low water | Maximum range |
|---|---|---|---|---|---|---|
| | *Feet* | *Feet* | *Feet* | *Feet* | *Feet* | *Feet* |
| Golden Gate | 2.6 | −3.1 | 5.7 | 5.1 | −5.6 | 10.7 |
| Point San Pablo | 2.9 | −3.0 | 5.9 | 5.6 | −5.4 | 11.0 |
| Selby | 3.2 | −3.1 | 6.3 | 5.9 | −5.1 | 11.0 |
| Benicia | 3.3 | −2.5 | 5.8 | 6.0 | −4.5 | 10.5 |
| Confluence of Sacramento and San Joaquin River | 3.3 | −1.1 | 4.4 | 6.0 | −2.5 | 8.5 |
| Hunters Point | 3.3 | −3.3 | 6.6 | 5.7 | −5.6 | 11.6 |
| San Mateo Bridge | 3.5 | −3.8 | 7.3 | 6.2 | −6.3 | 12.5 |
| Dumbarton Bridge | 4.1 | −4.3 | 8.4 | 6.8 | −6.7 | 13.5 |
| Alviso (South end of Bay) | 4.4 | −4.6 | 9.0 | 6.9 | −7.1 | 14.0 |

the San Francisco Bay system downstream from Carquinez Strait for the removal of almost 8 million cubic yards of silt from navigation channels. There are in this area a total of 17 authorized projects ranging in depth to 50 feet. The location of these authorized projects is shown on the map in figure 1. The estimated average annual shoaling rates and authorized depths of these projects is shown in table 5.

TABLE 5.—*Authorized navigation projects, project depths, and annual shoaling rates, San Francisco Bay*

| Project or facility[1] | Authorized depth | Shoaling rate |
|---|---|---|
| | *Feet* | *Cubic yards* |
| Mare Island Strait | 30 to 26 | 1,856,000 |
| Pinole Shoal Channel | 35 | 325,000 |
| Oakland Outer Harbor | 35 | 550,000 |
| Oakland Inner Harbor | 30 to 25 | 400,000 |
| Oakland Naval Supply Depot[1] | 35 | 300,000 |
| Oakland Inner Harbor Approach Channel | 30 | 80,000 |
| Alameda Naval Air Station[1] | 42 | 1,000,000 |
| Richmond Outer Harbor | 36 | 400,000 |
| Richmond Inner Harbor | 35 to 30 | 350,000 |
| Islais Creek | 35 | 50,000 |
| San Bruno Shoal | 30 | 275,000 |
| Redwood City Harbor | 30 | 250,000 |
| Main Ship Channel (over the bar) | 50 | 625,000 |
| Point Molate Naval Fuel Depot[1] | 45 to 35 | 95,000 |
| Hunters Point Naval Ship Yard[1] | 45 to 30 | 462,000 |
| Port Chicago Naval Ammunition Depot[1 2] | 32 to 15 | 72,000 |
| Mare Island Naval Ship Yard[1] | 30 to 26 | 752,000 |
| Total | | 7,842,000 |

[1] U.S. Naval facility; all others are authorized Corps of Engineers' navigation projects.
[2] Indicated as Concord N.A.D. on figure 1.

## The Model

The San Francisco Bay model, which includes most of the area outlined on the map (fig. 1), includes the Bay proper and the lower reaches of tributary streams that are affected by tides to the confluence of the Sacramento-San Joaquin River near Collinsville, and 17 miles of the Pacific Ocean beyond the Golden Gate. The ocean floor is reproduced to the 140-foot depth contour. A considerable area beyond this depth is included to provide space for the primary inflow–outflow headbay. It is one of the largest tidal models in the world. The model is built of reinforced concrete and is of the fixed-bed type. The model floor, except the ocean area, was cast in sections approximately 12 feet square with a screwjack resting on a concrete footing under each corner, to provide means of adjusting for any differential settlement. The linear scale is 1:1,000 horizontally and 1:100 vertically. Salinity in the model is maintained at the same concentration as the prototype. Other scale relationships are shown in table 6.

TABLE 6.—*Scale relationships in model of San Francisco Bay*

| Item | Model | Prototype |
|---|---|---|
| Velocity | 1 ft./sec. | 10 ft./sec. |
| Time | 0.6 min. | 1 hr. |
| Do | 1 min. | 1 hr. 40 min. |
| Do | 14.4 min. | 1 solar day (24 hr.) |
| Do | 14.9 min. | 1 lunar day (24 hr. 50 min.) |
| Do | 14.9 min. | 1 tidal cycle. |
| Do | 7.2 hr. | 30 days (1 month) |
| Discharge | 1 c.f.s. | 1,000,000 c.f.s. |
| Do | 1 g.p.m. | 1,000,000 g.p.m. |
| Volume | 1 cu. ft. | 100,000,000 cu. ft. |
| Do | 1 cu. ft. | 2,296 acre-feet (approx.) |
| Area (plan) | 1 sq. ft. | 1,000,000 sq. ft. |
| Do | 1 sq. ft. | 23 acres (approx.) |
| Do | 1 sq. ft. | 0.03 sq. mi. (approx.) |
| Area (cross section) | 1 sq. ft. | 100,000 sq. ft. |

Tides and currents are reproduced by the movement of adjustable valves to vary the inflow (delivered by pumps) and the outflow (by gravity) by which the water surface is raised or lowered at the rate required to simulate tidal action. Inflow of tributary streams is introduced and regulated by weirs and meters by which the fresh water discharge of all significant tributaries can be reproduced to scale.

## Shoaling Tests Techniques

### *Material*

Shoaling is simulated in the model by injecting finely ground gilsonite, which is a type of oil shale occurring in nature and having a specific gravity of about 1.04. This material is injected into the model as a slurry containing from 50 percent to as much as 95 percent water. Dyes and other liquid tracers may also be used to an advantage under certain conditions to determine current patterns. Before gilsonite is injected into the model, it must be thoroughly wetted to prevent excess flotation and washe

to prevent excessive discoloration of water in the model.[1]

### *Model Capabilities and Limitations*

It has been found by experience that a hydraulic model, such as that of San Francisco Bay, can be operated so as to reproduce in the model siltation and shoaling patterns similar to those that occur in the prototype. However, it must be recognized that the model cannot reproduce all of the conditions that prevail in nature. The relation between sedimentation and shoaling as it occurs in nature and that which can be reproduced in the model is qualitative only; it being impossible to reproduce and measure in the model to model scale deposition of the same quantity of material as takes place in nature. However, for many conditions the percentage distribution of deposition as it occurs in the prototype can be reproduced in the model, which is entirely satisfactory for most purposes. One reason for the inability to reproduce depositions in the model bearing a direct quantitative relationship to those that occur in nature is the volumetric scale relationship, which is 1:100,000,000. At this scale 1 cc. of material in the model would represent approximately 131 cubic yards in the prototype and if an attempt were made to maintain this relationship the amount of material deposited in navigation channels would be so small as to render reliable measurements almost impossible. Some other limiting factors are:

1. Flocculation, which takes place in nature when silt-laden fresh water comes in contact with salt water, thereby causing precipitation of finely divided suspended solids that are colloidal or semicolloidal in nature, cannot be reproduced in the model because materials used in the model to simulate silt (usually gilsonite) does not flocculate.

2. No practical method has been found to reproduce in the model the action that takes place when material that has been precipitated in shallow areas during calm weather is disturbed and comes back into circulation as a result of wind and wave action.

### *Type of Tests*

Tests conducted in the model fall in two general categories: (1) "base," or verification tests; and (2) "plan," or improvement tests. The object of "base" tests is to reproduce in the model the same distribution of shoals or sedimentation in a specific area of the model that takes place in the prototype. "Plan" tests have as their objective the determination of the effects of improvements on the sedimentation and shoaling pattern. For certain "base" tests, such as dredge spoils disposal areas, prototype data may not be available; therefore, the "base" test consists of observing and measuring the distribution of the material in the model, then modifying certain conditions such as the point at which dredge spoils are dumped; repeating the test, and noting and comparing the effect on shoaling in the model.

## Model Tests

### *Verification*

The first step of any shoaling test is to reproduce in the model the same shoaling pattern in the problem area as takes place in nature. (Hydraulic and salinity verifications are assumed to have been accomplished.) This is a trial and error procedure. Perhaps the most critical item is selection of injection points or ranges. The test material (gilsonite) is injected into the model in one or more areas near the problem area at locations that appear to be on the line of travel of sediments deposited in the problem area. Injections are continued over several tidal cycles, then operation of the model continued for several more cycles after injection has been stopped to accomplish complete distribution of the material injected. This operation is usually continuous until there is no appreciable amount of material remaining in suspension. The material is then recovered from each of the separate sections into which the problem area has been previously divided and measured and the percentage of distribution of material so recovered compared with the percentage distribution of shoaling as it occurred in the prototype.

Sometimes very close verification can be obtained. At other times or under other conditions only approximate verification is obtainable. This latter condition occurs more often in a case of deadend channels. Due to the characteristics of gilsonite, it is not transported into these deadend channels to the same extent that silt in the prototype might be so transported. Careful records must be maintained of the total quantities of gilsonite injected at each range and of the location of each injection point, since plan tests require close reproduction of the injection schedule.

### *Dredge Spoils Disposal Tests*

Model tests were conducted to check the merits of present areas used for dumping material dredged from three of the problem areas and of the results to be expected if these areas were relocated closer to areas from which the material was dredged as follows: (1) Oakland Inner and Outer Harbor; (2) Alameda Naval Air Station; (3) Mare Island Strait. Dumping

[1] A manual on "Operating Procedures for Conducting Shoaling Studies in Model of San Francisco Bay," dated August 1962, prepared by the San Francisco District, Corps of Engineers, provides detail information on the technique of conducting shoaling tests.

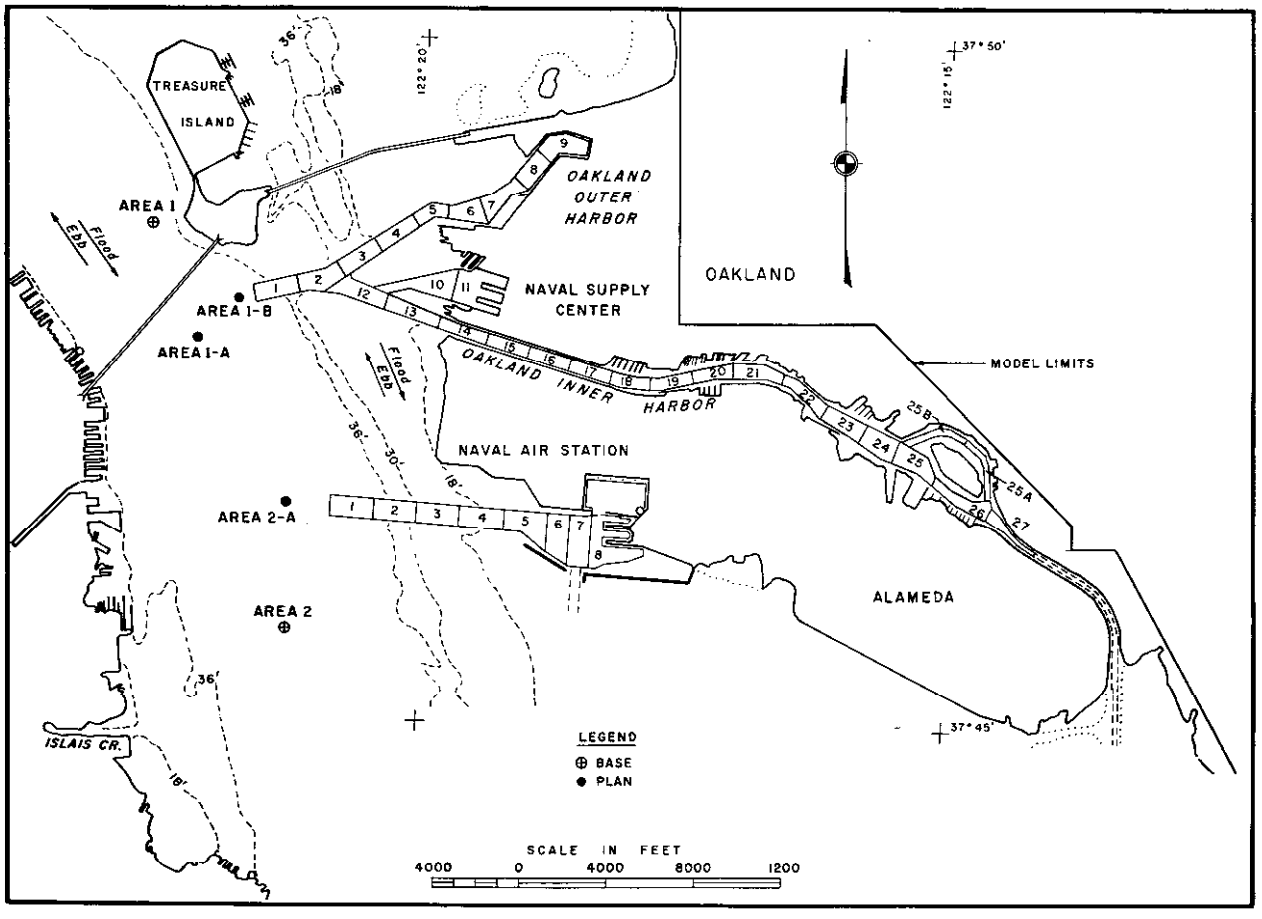


FIGURE 4. — Model of Oakland Harbor and Alameda Naval Air Station.

areas presently used and alternate areas tested in the model for Oakland Inner and Outer Harbor and for the Alameda Naval Air Station are shown in figure 4. Similar information relative to Mare Island Strait is shown in figure 5. "Base" tests consisted of injecting gilsonite at points currently used and observing and measuring its distribution through the Bay system. "Plan" tests consisted of making similar injections at alternate locations; observing and measuring the distribution and comparing the results with the "base" tests to determine the merits of the alternate locations.

### *Oakland Inner and Outer Harbor*

The presently used disposal area of material dredged from this facility is in deep water on the west side of Yerba Buena Island and about 7,000 feet northwesterly from the westerly end of the entrance channel to these facilities, designated as area 1 on the map (fig. 4). Dredge spoils are dumped at this place at any time without regard to the direction of flow of the tide. For the "base" tests a continuous injection amounting to a total of approximately 150,000 cc. of gilsonite was made at this point. In order to determine the feasibility of moving the dredge spoil disposal area nearer to the project and thereby reducing travel time and distance for the dredges, two alternate locations designated as areas 1A and 1B on the map (fig. 4), and located nearer the project area, were tested by injecting similar amounts of gilsonite in these locations. The results are shown in table 7.

Very little difference occurred in the distribution of dredge spoils dumped at the three areas considered (table 7). Tests indicate that, on an average, less than 4 percent of material dumped at any of these locations is redeposited directly in navigation channels in the Bay system, and approximately 30 percent of the dredge spoils is carried directly to sea through the Golden Gate. It also appears that approximately 65 percent

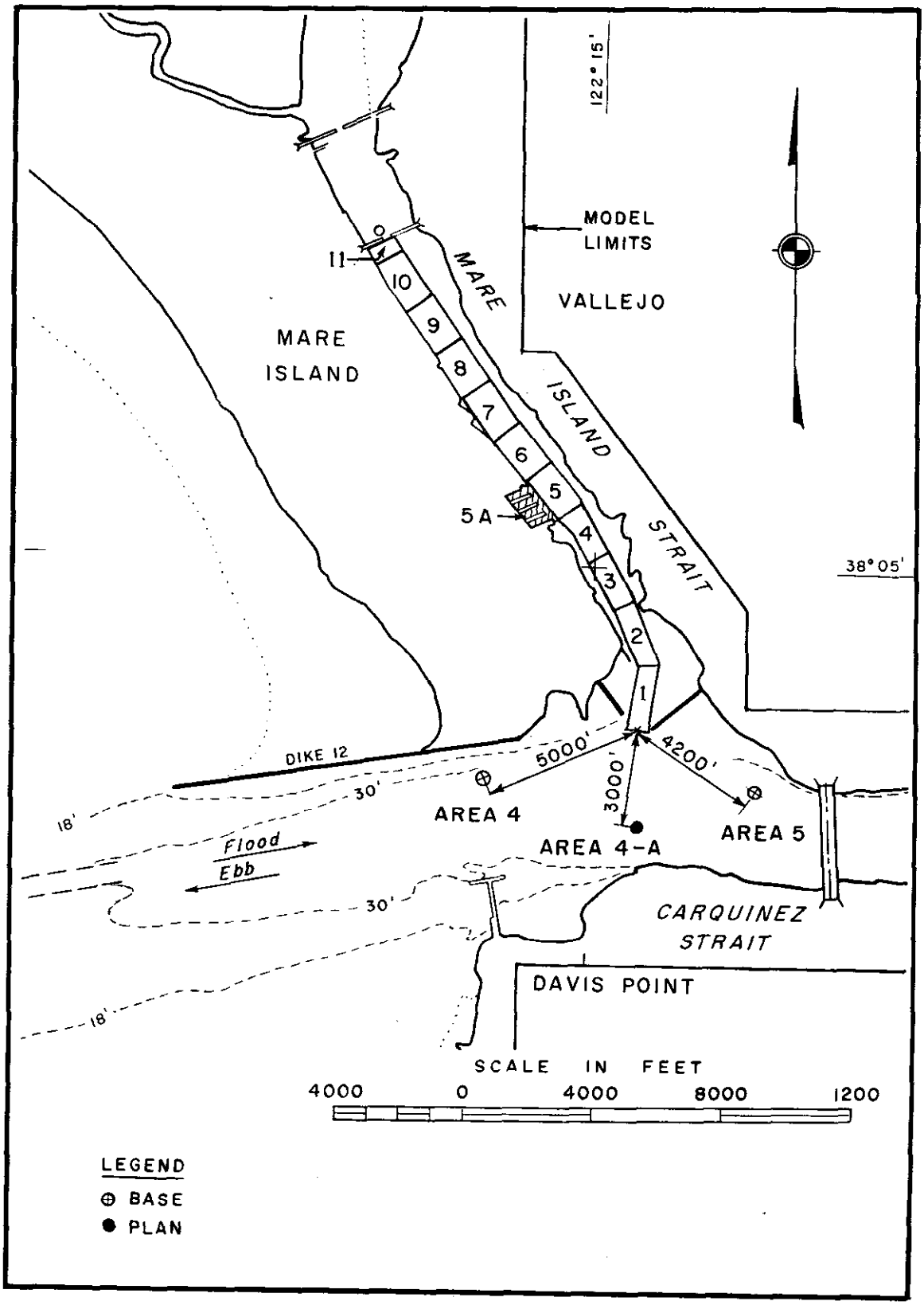


FIGURE 5. — Model of Mare Island Strait.

TABLE 7.—*Comparison of "base" and "plan" tests of dredge spoil disposal areas for the Oakland-Alameda group projects*

| Projects | Gilsonite deposits of total quantity (cc.) injected | | |
|---|---|---|---|
| | Area 1 (base) | Area 1A (plan) | Area 1B (plan) |
| | *Pct.* | *Pct.* | *Pct.* |
| Richmond Harbor Channels | 0.73 | 0.65 | 0.60 |
| Oakland-Alameda Group projects | .84 | .37 | 1.02 |
| Hunters Point and Islais Creek | .08 | .11 | .18 |
| San Bruno Shoal, South Bay | .35 | .30 | .44 |
| San Francisco Pier Slips | .67 | .70 | .76 |
| Pinole Shoal, San Pablo Bay | ..... | .10 | .12 |
| Richardson Bay Channel | .12 | .16 | .16 |
| Main Ship Channel over Bar | .86 | .60 | .60 |
| (1) Subtotal, navigation facilities | 3.65 | 2.99 | 3.88 |
| (2) Richardson Bay, outside of Channel (measured) | .45 | .64 | .62 |
| San Francisco Bar, south | 7.78 | 7.89 | 5.84 |
| San Francisco Bar, north | 3.18 | 2.45 | 2.53 |
| (3) Subtotal outside of Golden Gate minus bar channel | 34.72 | 23.18 | 29.36 |
| (4) Subtotal interior Bay, outside of navigation facilities (estimated) Minus (2) Richardson Bay) | 61.18 | 73.19 | 66.14 |
| Totals (1), (2), (3), and (4) | 100.00 | 100.00 | 100.00 |

of the dredge spoils is redeposited in parts of the Bay outside of navigation projects. It is probable that a considerable part of this material may be resuspended by wind and wave action. An indeterminate amount of the resuspended material would be deposited ultimately in navigation channels. A considerable part would also be carried to sea through the Golden Gate. Tentative conclusions derived from these tests are that alternate or "plan" areas can be used for disposal of dredge spoils without detrimental effect, and a worthwhile savings thus accomplished through reduction of haul distance.

### *Alameda Naval Air Station*

Test procedure for this facility were similar to those for the Oakland Inner and Outer Harbor. In figure 3, the presently authorized spoils disposal area designated as area 2 on the map is located some 7,000 feet southerly from the westerly end of the approach channel and the alternate or "plan" area designated as area 2A is located on the prolongation of the centerline of the approach channel. The use of this latter area would result in a decrease of approximately 2 miles in the haul distance from the project to the spoils disposal area. The results of the tests of these two areas are shown in table 8.

There is very little difference in the dispersion pattern of material dumped in either of the two areas (table 8). In fact, the tests indicate that the plan area (area 2A) has a slight advantage. The distribution pattern is very similar to that for the Oakland Inner and Outer Harbor. About 4 percent of the material is deposited in navigation channels and 25 percent to 30 percent is carried out to sea through the Golden Gate. Tentative conclusions are that the plan area (area 2A) can be adopted without detrimental results, and an appreciable savings in dredging costs accomplished through reduction of hauling distance.

TABLE 8.—*Comparison of "base" and "plan" tests for dredge spoil disposal areas for Alameda Naval Air Station*

| Location | Gilsonite deposits of total quantity (cc.) injected | |
|---|---|---|
| | Area 2 (base) | Area 2A (plan) |
| | *Pct.* | *Pct.* |
| Richmond Harbor channels | 0.45 | 0.64 |
| Oakland-Alameda group projects | .26 | .40 |
| Hunters Point and Islais Creek | .15 | .15 |
| San Bruno Shoal, South Bay | 1.63 | 1.02 |
| San Francisco Pier slips | 1.09 | .88 |
| Pinole Shoal, San Pablo Bay | .12 | .22 |
| Richardson Bay channel | .09 | .11 |
| Main Ship channel over bar | .23 | .13 |
| (1) Subtotal in navigation facilities | 4.02 | 3.55 |
| (2) Richardson Bay, outside of channel (measured) | 0.37 | 0.45 |
| San Francisco Bar, south | 5.10 | 3.63 |
| San Francisco Bar, north | 1.72 | 1.65 |
| (3) Subtotal outside of Golden Gate, minus Bar channel | 31.09 | 21.78 |
| (4) Subtotal interior, outside of navigation facilities (estimated) minus (2) Richardson Bay | 64.52 | 74.22 |
| Totals (1), (2), (3), and (4) | 100.00 | 100.00 |

### *Mare Island Strait*

Areas currently used for disposal of dredge spoil and the area tested in the model for this facility are shown in figure 5. At this facility dredge spoils are dumped during and at ebbtide at area 4, located downstream from the mouth of the Strait; and on floodtide at area 5, located upstream from the mouth of the Straits. Two separate "base" tests were conducted at this location, one during which injections were made only on the ebb at area 4, and one during which injections were made only on the floodtide at area 5. In the "base" tests approximately 50,000 cc. of gilsonite were released in each area. For the plan test at area 4A, injections were made continuously through both the ebb- and floodtide and approximately 100,000 cc. of gilsonite, or twice the quantity used for each of the individual plan tests, was injected. The distribution pattern of each of the "base" tests and of the

"plan" test and a comparison of results is given in tables 9, 10, 11, and 12.

TABLE 9.—*Spoil disposal area 4 (base), deposition within and without navigation channels and slips*

| Project | Gilsonite deposits | |
|---|---|---|
| | Cubic centimeters | Percent of total cc. injected |
| Pinole Shoal, San Pablo Bay | 5,755 | 11.5 |
| Mare Island Strait | 2,410 | 4.8 |
| Suisun Bay Channel | [1]50 | .1 |
| Total | 8,215 | 16.4 |
| Outside of Golden Gate, no evidence of deposit | ..... | ..... |
| Central Bay, very little evidence of deposit | ..... | ..... |
| Interior Bay, outside of projects | 41,785 | 83.6 |
| Totals | 50,000 | 100.0 |
| Measurements made of gilsonite, deposits in limited areas for comparison with results of subsequent tests, as follows: | | |
| (1) 2-square-mile area in Suisun Bay immediately east of Benicia, north of channel between shore and 18-ft. depth contour | 980 | 2.0 |
| (2) ½-square-mile area in Carquinez Strait from Benicia shore to 18-ft. depth contour | 500 | 1.0 |

[1] Estimated.

TABLE 10.—*Spoil disposal area 5 (base), deposition within and without navigation channels and slips*

| Project | Gilsonite deposits | |
|---|---|---|
| | Cubic centimeters | Percent of total cc. injected |
| Pinole Shoal, San Pablo Bay | 1,140 | 2.5 |
| Mare Island Strait | 2,870 | 6.2 |
| Suisun Bay Channel | 230 | .5 |
| Total | 4,240 | 9.2 |
| Outside of Golden Gate, no evidence of deposit | ..... | ..... |
| Central Bay, very little evidence of deposit | ..... | ..... |
| Interior Bay, outside of projects | 41,760 | 90.8 |
| Totals | 46,000 | 100.0 |
| Measurements made of gilsonite deposits inlimited areas for comparison with results of other gase and plan tests, as follows: | | |
| (1) 2-square-mile area in Suisun Bay immediately east of Benicia, north of channel between shore and 18-ft. depth contour | 5,200 | 11.3 |
| (2) ½-square-mile area in Carquinez Strait from Benicia shore to 18-ft. depth contour | 2,415 | 5.3 |

TABLE 11.—*Spoil disposal area 4A (plan), deposition within and without navigation channels and slips*

| Project | Gilsonite deposits | |
|---|---|---|
| | Cubic centimeters | Percent of total cc. injected |
| Pinole Shoal, San Pablo Bay | 5,075 | 5.4 |
| Mare Island Strait | 4,655 | 4.9 |
| Suisun Bay Channel | [1]300 | .3 |
| Total | 10,030 | 10.6 |
| Outside of Golden Gate, no evidence of deposit | ..... | ..... |
| Central Bay, very little evidence of deposit | ..... | ..... |
| Interior Bay, outside of projects | 84,720 | 89.4 |
| Totals | 94,750 | 100.0 |
| Davis Point (Union Oil Co. pier): | | |
| Area 1 (530,000 sq. ft.) | 110 | .1 |
| Area 2 (230,000 sq. ft.) | 230 | .2 |
| Measurements made of gilsonite deposits in limited areas for comparison with results of other base and plan tests, as follows: | | |
| (1) 2-square-mile area in Suisun Bay immediately east of Benicia, north of channel between shore and 18-ft. depth contour | 7,415 | 7.8 |
| (2) ½-square-mile area in Carquinez Strait, from Benicia shore to 18-ft. depth contour | 3,880 | 4.1 |

[1] Estimated.

TABLE 12.—*Comparison of test results for base and plan tests of dredge spoil disposal areas for the Mare Island Strait project*

| Project | Distribution | | | |
|---|---|---|---|---|
| | Area 4 base | Area 5 base | Area 4+5 base | Area 4A plan |
| | *Percent* | *Percent* | *Percent* | *Percent* |
| Mare Island Strait | 4.8 | 6.2 | 5.5 | 4.9 |
| Pinole Shoal | 11.5 | 2.5 | 7.2 | 5.4 |
| Suisun Bay Channel[1] | .1 | .5 | .3 | .3 |
| Totals | 16.4 | 9.2 | 13.0 | 10.6 |
| Outside of Golden Gate | 0 | 0 | 0 | 0 |
| Central Bay | 0 | 0 | 0 | 0 |
| Interior Bay, outside of projects | 83.6 | 90.8 | 87.0 | 89.4 |
| Totals | 100.0 | 100.0 | 100.0 | 100.0 |
| Northwest shore of Suisun Bay (limited area of 2 sq. mi.) | 2.0 | 11.3 | 6.4 | 7.8 |
| Benicia Shore (limited area of ½ sq. mi.) | 1.0 | 5.3 | 3.0 | 4.1 |
| Davis Point (Union Oil Co. pier): | | | | |
| Area 1 (530,000 sq. ft.) | ..... | ..... | ..... | .1 |
| Area 2 (230,000 sq. ft.) | ..... | ..... | ..... | .2 |

[1] Estimated.

From the foregoing tabulations practically none of the material injected in the disposal areas of Mare Island Strait appears to be car-

ried to sea and a considerable part of it actually appears to be transported up Carquinez Strait and into Suisun Bay. The test of the "plan" (area 4A) indicates slightly less redeposition in navigation channels than do either of the areas currently used. Since the annual shoaling rate and dredging rate in Mare Island Strait, including the Naval Ship Yard, is approximately 2,600,000 cubic yards, this indicates that use of the "plan" area might result in a decrease of about 16,000 cubic yards a year in the shoaling rate in Mare Island Strait and a slight reduction in Pinole Shoal.

Tentative conclusions are that the "plan" area (area 4A) can be used without detrimental results and that its use might result in some decrease in the shoaling rate in Mare Island Strait. Furthermore, a slight reduction in hauling distance could also be accomplished.

### *Model Tests of Shoaling in Navigation Projects*

Model tests were conducted to ascertain the effect of various improvements consisting chiefly of dikes and training walls on shoaling in navigation projects. The results of some of the tests of plans for the Alameda Naval Air Station, where the average shoaling rate is about 1 million cubic yards a year, are considered typical of results that might be anticipated from similar plans elsewhere. The results of tests of 11 plans are shown in table 13. Plans and channel sections are shown on figure 6.

TABLE 13.—*Shoaling test results after various improvements, Alameda Naval Air Station*

| Channel section No. | Shoaling distribution after improvements | Change in shoaling for plan— | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 8a | 9 | 10 |
| | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* |
| 1 | 0.6 | +33 | +33 | −33 | 0 | +33 | +33 | 0 | 0 | +33 | −33 | −33 |
| 2 | 0.9 | 0 | +80 | +60 | +80 | +80 | +680 | +80 | +100 | +60 | +80 | +80 |
| 3 | 0.7 | 0 | +25 | +150 | +75 | +125 | +3,375 | +75 | +25 | +50 | +50 | −20 |
| 4 | 10.1 | +38 | +113 | −24 | +55 | +18 | +85 | +40 | +18 | −11 | +16 | +25 |
| 5 | 30.9 | +6 | −19 | −36 | −14 | −31 | −69 | −20 | −76 | −50 | −32 | −44 |
| 6 | 17.6 | −10 | −41 | −44 | −31 | −23 | −65 | −9 | −76 | −54 | 0 | −20 |
| 7 | 16.4 | −30 | −52 | −53 | −39 | −26 | −67 | +21 | −83 | −64 | −15 | −10 |
| 8 | 22.8 | −55 | −69 | −66 | −58 | −47 | −79 | +7 | −93 | −76 | −23 | −33 |
| Total | 100 | −11 | −25 | −44 | −23 | −25 | −21 | +2 | −69 | −53 | −15 | −23 |

Model tests indicate that there is a direct relation between the volume of water (the tidal prism) flowing into or through a tidal basin or across an excavated navigation channel and the amount of silt deposited in such a facility. In the facility under consideration under present conditions, a considerable volume of water flows through the basin on both the flood- and ebb-tides. On the floodtide, water enters the basin from the west and a part of it passes out through the 750-foot gap in the breakwater on the south side of the docking area. On the ebbtide, the reverse condition prevails. Closing the gap, as indicated for plan 1, which reduced the tidal prism to the volume within the docking area represented by that area and the tidal range, produced a net reduction of 11 percent in siltation throughout the project, with the greatest reduction taking place in the part protected by the breakwater (sections 6, 7, and 8) but with increases in some sections of the approach channel (sections 1, 4, and 5). During the test a large eddy formed over the navigation channel seaward of the area protected by the breakwater, and a considerable volume of water that would normally flow into the sheltered area passed over sections 4 and 5, which accounts for the increase in shoaling in these sections.

Plan 2, which consisted of a closure of the 750-foot gap in the breakwater and a 2,000-foot extension of the breakwater, produced a still further decrease in net total shoaling with a large decrease in the area protected by the breakwater but with an increase in shoaling in the unprotected section of the channel (sections 1 through 4). In this test an eddy extending across the approach channel and to the shore immediately north of section 4, developed during the test. Plan 3, which includes all features of plan 2 plus a dike extending 1,300 feet southerly from the shore north of section 4, affected the pattern of this eddy and accomplished a further reduction in total shoaling but resulted in a further increase in depositions in the approach channel to the west, sections 1, 2, and 3. Plans 4, 5, and 6 are based on the same concept as plan No. 3; that is, deflection of the eddy which appeared to be responsible for depositions in that section of the channel not protected by the breakwater.

Plans 7 and 9 were tested to determine the possibility of accomplishing a worthwhile reduction without closing the 750-foot gap in the breakwater southerly from the docking area, since this opening is desired in order to provide a passage for seaplanes. The results of these plans, one of which resulted in a net increase of 2 percent in total depositions, proved conclusively that closure of this gap is a key factor.

Plan 8, which reduces the access area to the

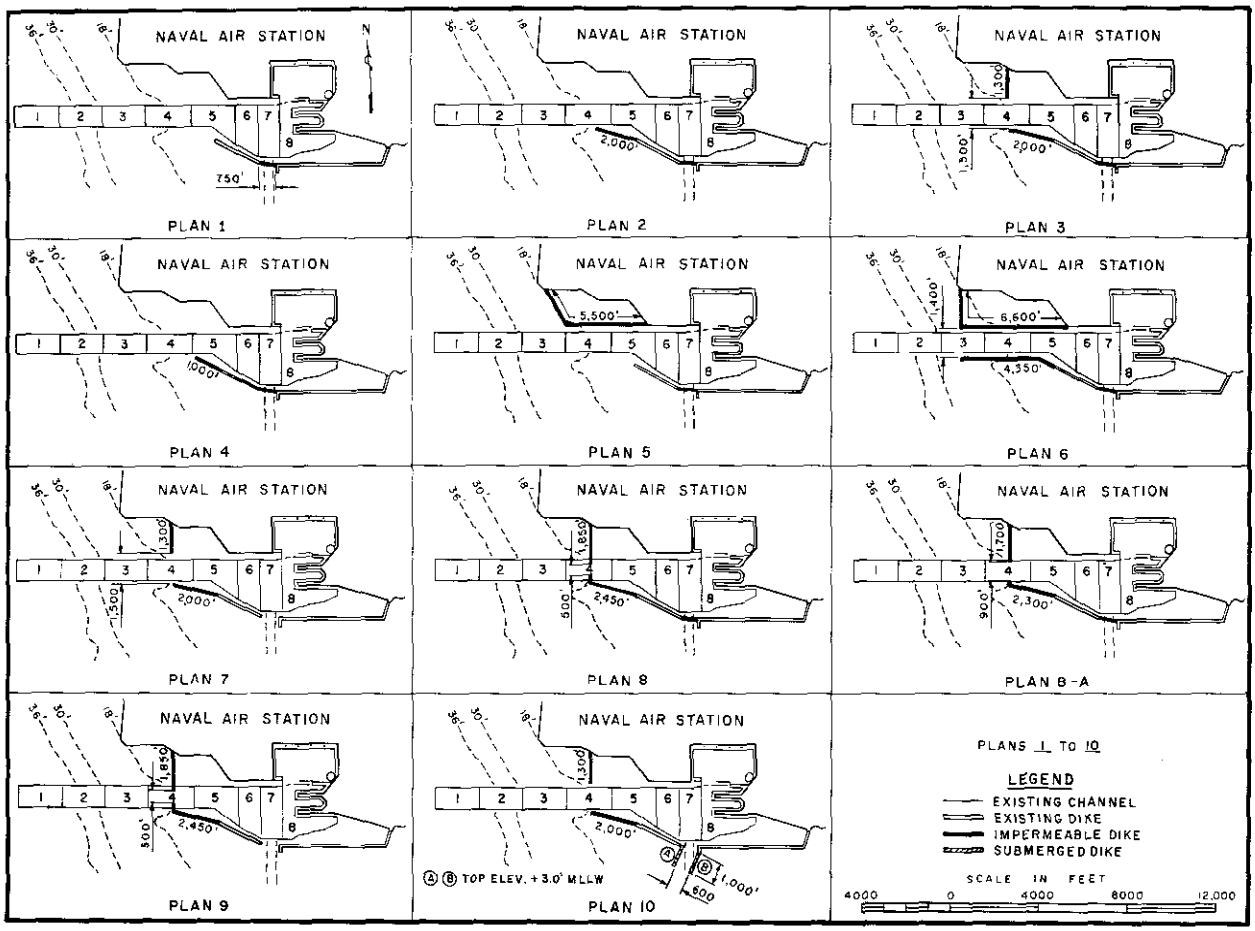


FIGURE 6. — Models of plans 1 to 10, to study shoaling in navigation projects.

eastern section of the project where about 90 percent of the shoaling takes place to a cross section 500 feet wide, proved to be the most effective in that the net reduction in shoaling was 69 percent. This plan demonstrates conclusively the theory that there is a direct relation between the amount of water flowing into or across an improved channel or docking area and siltation in the facility. This plan eliminated substantially all of the eddy currents that had resulted in great increases in deposition in sections 1 through 4, although it did show some increase in sections 2, 3, and 4. This increase is accounted for by the fact that the restriction in the approach to the sheltered area probably resulted in a slight buildup westward of the opening and also probably caused a slightly greater amount of water to pass across sections 2 and 3 and that part of section 4 westward of the entrance than would take place under existing conditions.

Plan 10 was tested to determine the effect of deflecting dikes that would divert water flowing directly through the 750-foot opening in the breakwater on an ebbtide. It was considered probable that considerable part of the silt carried into the docking area through this opening on an ebbtide traveled as a bedload movement or at least for the lower part of the stream, and that the dikes might serve to deflect the currents to the west. The results indicate some improvement; however, a comparison with plan 9 cast doubt on the effectiveness of this plan.

## Discussion

The results of the program of model tests of hopper dredge spoil disposal reported herein are adequate to evaluate the current practice of disposal of spoil from the standpoint of effects on main navigation channels, side channels, and harbors and on slips and small boat basins while dredging operations are in progress. The present practice of "free-dumping" of dredge spoil beyond the limits of the channel and the disposal areas themselves are satisfactory. However, since the amount of material that deposits

in navigation facilities is insignificant, and with very little differences resulting from the disposal areas currently being used for the newly located areas, it is concluded that "plan" areas located closer to the dredging projects should be utilized in future dredging operations. Thus, by reducing the haul distance, considerable savings can be effected in the dredging operations. Studies have not progressed far enough to determine whether a change in present practice to a more expensive practice of controlling spoil, such as the sump rehandler dredging technique (to insure that none of the dredge spoils returns to the waterway) is warranted. When consideration is given to the large amount of spoil that deposits in shallow areas of the Bay, from whence large amounts are resuspended by wave action and carried by tidal currents into the various navigation facilities, rehandling and disposal in diked areas may be warranted for some projects.

With regard to plans of improvement to reduce shoaling, additional tests and studies are required for further substantiation of results indicated and for the development of details of plans that might be recommended. Also several other problem areas appear to be susceptible of analysis and possible solution by model tests. Results to date indicate that the model tests will result in material savings in maintenance dredging costs, and in a much better understanding of shoaling problems in the San Francisco Bay system.

# CHANNEL DEPTH AS A FACTOR IN ESTUARINE SEDIMENTATION

[Paper No. 72]

By HENRY B. SIMMONS, *chief, Estuaries Section, Rivers and Harbors Branch, Hydraulics Division, U.S. Army Engineer Waterways Experiment Station, Vicksburg*

## Abstract

In most estuaries in the United States that have been improved for navigation by large oceangoing vessels, drastic changes have taken place in both annual shoaling rates and in the locations of major shoal areas as the channels were progressively deepened. Numerous problems that affect channel maintenance have arisen because of these changes, and some of these problems have now reached a critical state.

This paper presents a case history of Savannah Harbor to illustrate how shoaling rates and patterns have been changed by progressive changes in channel depth. The paper also discusses the more important reasons for such changes and suggests remedial measures that appear to offer both technical and economic solutions to certain of the problems thus created.

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When the early settlers of this country reached the Atlantic and Gulf coasts in their sailing ships, they discovered that the natural estuarine channels afforded ready access to the interior for sufficient distances to locate suitable sites for towns and farms and to provide sheltered anchorages for their ships. Cities such as Charleston, Savannah, Mobile, and many others were founded on the first terrain encountered which was sufficiently high above high tide for development. Although the natural estuarine channels were usually characterized by entrance bars and interior crossings, some having controlling depths of perhaps 6 to 12 ft. at low tide, such depths were usually adequate for safe navigation by the oceangoing ships of that era. Furthermore, after having spent weeks or even months at sea since leaving the last port of call, the ship captains were not disturbed by a delay of several hours waiting for high tide to navigate an especially troublesome bar or crossing.

Most of the natural estuaries discovered by these early settlers were apparently in a state of essential equilibrium between the rate of supply of sediments from both fluvial and oceanic sources and the ability of the currents generated by tides and freshets to remove such sediments and carry them to sea. The environment in which these natural estuaries existed was a comparatively simple one, since there were essentially no agricultural developments to accelerate erosion and no attempts had been made to alter the physical characters of the channels or the hydraulic characters of flows therein. Because of the usual semicircular entrance bars, which had crest elevations only a few feet below low tide, it is probable that density currents played only a very minor role in the sedimentation processes of these natural estuaries. Although the locations and crest elevations of bars and crossings were probably changed drastically from time to time by severe storms and major freshets, it appears likely that the controlling long-term conditions were restored fairly quickly after such events.

With the advent of steam power for ocean-going ships, the ship designers quickly discovered that increased vessel size and draft paid handsome dividends in terms of lower construction and operating costs per unit of cargo capacity; that these factors are still important today is most dramatically evidenced by the modern supertanker. As the sizes and drafts of ocean-going ships increased, the natural depths of estuaries used for navigation were no longer adequate and a chain of actions and reactions was begun that is still in progress at the present time.

The first attempts to improve navigating conditions in tidal waterways consisted largely of the removal of snags and other obstructions, followed by the closure of secondary channels to confine flows to major channels, and then by the construction of dikes and jetties to increase depths over interior crossings and entrance bars. Maximum depths sought through such improvement measures were usually 15 to 20 ft. at low tide. With a minor amount of initial and maintenance dredging, the engineers concerned were largely successful in obtaining the rather moderate depths desired. Repetitive maintenance dredging was generally not a major problem, since the environments of the waterways involved had still not been altered to a significant degree. However, as increasing vessel drafts required greater and greater depths, which were obtained and maintained largely by dredging — supplemented in some cases by channel contraction works and more extensive entrance jetty systems — it became increasingly obvious to the engineers concerned that important environmental factors had been altered to such extent that many of the improved estuaries bore little resemblance either physically or hydraulically, to those that existed in the natural state. It is therefore not surprising that principles of improvement that at first yielded highly satisfactory results became less and less effective as channel depths were progressively increased.

The various types of estuaries encountered, the hydraulic and salinity conditions characteristic of each type, and the effects of upland discharge in controlling estuary type were discussed by the author in 1955.[1] Some of the shoaling characteristics of the various types of estuaries and principles of improvement plans designed to reduce shoaling were discussed by the author and E. A. Schultz in 1957.[2] This present paper is concerned primarily with the effects of channel depth as a varying environmental factor on the amount of repetitive shoaling in estuaries, and more particularly on the locations of major shoal areas within estuaries.

At first thought it might appear that the location of the major shoal area within an estuary would not be of major importance from an economic viewpoint, provided the dredging frequency and the quantity of dredging per unit time were the same. However, because suitable areas for disposal of dredged soil are rapidly being filled, which necessitates longer discharge lines to reach remote areas and in some cases booster pumps to maintain efficiency, the unit cost of dredging may vary over a wide range, depending on the location of the shoal area being dredged. As an example, the unit cost of dredging various shoal areas within one navigation project familiar to the author ranges from as little as $0.15 to as much as $1.00 per cu. yd. This wide range depends almost entirely on whether there are spoil disposal areas immediately adjacent to the dredging site that require little or no retention dike construction or whether the spoil must be placed in remote disposal areas that require the construction of elaborate retention dikes to prevent the spoil from running back into the channels being maintained or from flowing onto and damaging adjacent properties.

The effects of channel depth on the shoaling regimen of an estuary can probably be best illustrated by an actual case history; that is, by taking a given estuary and developing a record of the rates and patterns of shoaling at various stages of its development. The author has selected Savannah Harbor for this purpose for three principal reasons. First, Savannah Harbor is characteristic of the partly mixed estuary, which is the most common type in the United States that has been improved for deep-draft navigation; second, Savannah Harbor is a relatively simple type of estuary from the standpoint of channel complexity; and third, the engineers who have been responsible for the development and maintenance of Savannah Harbor have been more painstaking than most in keeping good records, so that the determination of reasonably accurate shoaling rates and patterns for previous channel depths has been possible. Also, two rather comprehensive hydraulic model investigations of Savannah Harbor have been carried out by the Waterways Experiment Station during the past 20 years, and information gained during these investigations has contributed much to an understanding of the present and former hydraulic and shoaling regimens of the harbor.

Savannah Harbor (fig. 1) comprises the

[1] SIMMONS, H. B. SOME EFFECTS OF UPLAND DISCHARGE ON ESTUARINE HYDRAULICS. Amer. Soc. Civil Engin. Proc. Paper 792. 1955.

[2] SIMMONS, H. B., and SCHULTZ, E. A. FRESH WATER-SALT WATER DENSITY CURRENTS, A MAJOR CAUSE OF SILTATION IN ESTUARIES. U.S. Army Corps of Engineers Committee on Tidal Hydraul. Tech. Bul. 2. 1957.

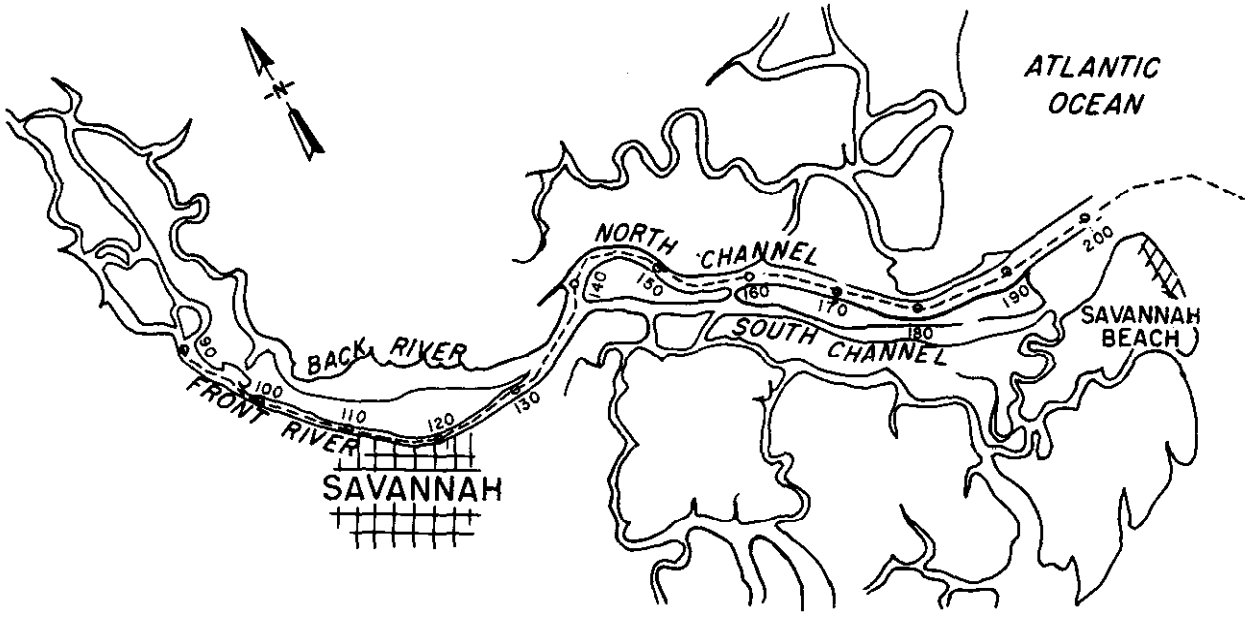


FIGURE 1. — Location map, Savannah Harbor.

lower 23.3 miles of the Savannah River, which is located on the South Atlantic coast of the United States and forms the boundary between Georgia and South Carolina, plus 7.5 miles of bar channel between deep water in the Atlantic Ocean and the entrance jetties. The city of Savannah is located 16.1 miles upstream from the entrance to the harbor, and the improved channel extends another 7.2 miles upstream from the city. The navigation channel follows North Channel from the ocean to the head of South Channel and then follows Front River to the head of navigation at channel station 82. The present navigation project for the harbor calls for a channel 500 ft. wide and 36 ft. deep at mean low water (m.l.w.) from deep water in the Atlantic Ocean to channel station 190; thence 400 ft. wide and 34 ft. deep at mean low water to channel station 94; thence 200 ft. wide and 30 ft. deep at mean low water to the head of navigation at channel station 82. The two jetties at the entrance to North Channel, which are about 2,500 ft. apart, were constructed between 1890 and 1896.

The mean range of tide at the entrance to Savannah Harbor is about 6.8 ft., that at Bull Street Gage (station 120) is about 7.4 ft., and that at the head of the navigation project is about 8.0 ft. Maximum tidal current velocities in the navigation channels are 5 ft. per sec. during spring tides. Prior to construction of Clark Hill Dam upstream from Augusta, Ga., the fresh-water discharge of the Savannah River ranged from a normal low of about 4,000 c.f.s. to a maximum of about 350,000 c.f.s. Since construction of the dam, the fresh-water discharge has been effectively regulated at about 7,000 c.f.s.; however, local heavy rains in the watershed downstream from the dam have increased the discharge at Clyo, Ga. (the head of tidal influence), to about 25,000 c.f.s. for periods of several days. For normal conditions of tide and fresh-water discharge, salt water having a salinity of about 5 parts per thousand (p.p.t.) penetrates upstream to about channel station 109 at high-water slack of the tidal current and retreats to about channel station 129 at low-water slack.

As stated previously, Savannah Harbor is characteristic of the partly mixed estuary, in that the vertical mixing of salt and fresh water is not complete over the length of salt-water intrusion. In this type of estuary, surface salinities are appreciably less than bottom salinities and a significant part of the vertical salinity difference occurs within a few feet of the depth. This transition zone may be considered to be an "interface," even though it is not well defined, because the net flow over a complete tidal cycle is downstream above the transition zone and upstream below the transition zone. The vertical distribution of flow in Savannah Harbor for conditions of mean tide and a normal fresh-water inflow of about 7,000 c.f.s. is shown in figure 2. The numerical values on this figure represent the percentage of total flow over a complete tidal cycle that was downstream or ebb at each point of measurement. A value of 50 indicates that upstream and down-

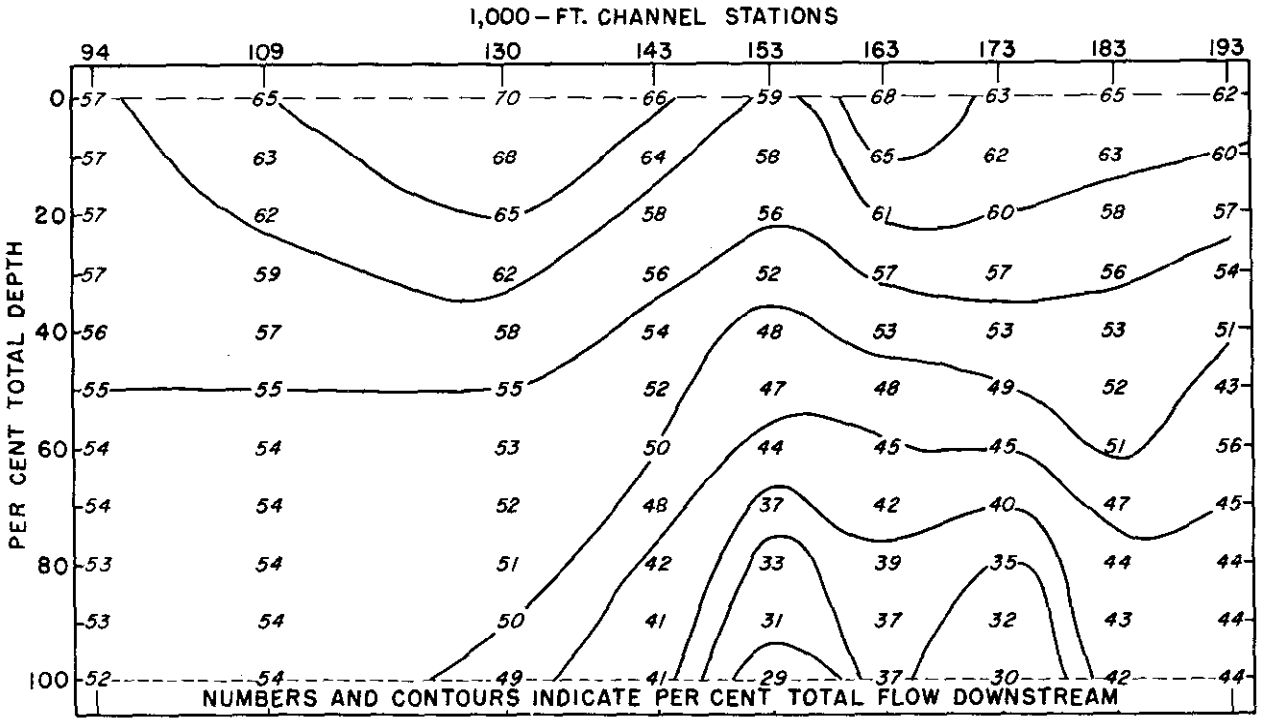


FIGURE 2. — Distribution of flow in Savannah Harbor.

stream flows were exactly in balance; values greater than 50 indicate that downstream flows predominated over upstream flows; values less than 50 indicate that upstream flows predominated over downstream flows.

Examination of figure 2 shows that the direction of net bottom flow in Savannah Harbor is upstream from the harbor entrance to about channel station 125 and is downstream from this location to the upper end of the navigation project. Bottom upstream predominance reaches a maximum in the vicinity of station 153, where the floodflow is about 71 percent of the total flow over a tidal cycle, and the ebb is only about 29 percent. The elevation of the transition zone or interface between net upstream and net downstream flow is located at about 70 percent of the channel depth at station 140, about 35 percent of the depth at station 155, and about 50 percent of the depth at station 190. The direction of net flow at the surface and at all other elevations above the interface is downstream. Upstream from the limit of salt-water intrusion, the direction of net flow is downstream at all depths, and the degree of downstream predominance at any location in this reach is an indication of the relation between the tidal discharge and the fresh-water discharge.

The sediments that deposit in Savannah Harbor to form shoals consist almost entirely of silts and flocculated clays, and it is obvious that the transportation and deposition of these lightweight sediments are greatly influenced, if not controlled completely, by the flow distribution just described. If a given sediment particle entering the upstream end of the harbor is of such nature as to deposit first in the fresh-water region, it would likely be resuspended during each strength of velocity and thus be moved progressively downstream to the vicinity of station 125, beyond which point the direction of the net flow is upstream. On the other hand, should the nature of the particle be such that it would remain in suspension until deposited through flocculation or related processes, it would likely first deposit at some location well downstream from station 125 where salinities are relatively high. Again the particle would likely be resuspended during each strength of current, but in this case it would be moved progressively upstream to the vicinity of station 125 by the net bottom upstream flow. Therefore, in both of these assumed cases, there is a net force tending to concentrate sediment in the vicinity of station 125, since the direction of net bottom flow is toward this general location from both directions.

Figure 3 shows the relation between bottom flow predominance in Savannah Harbor and the

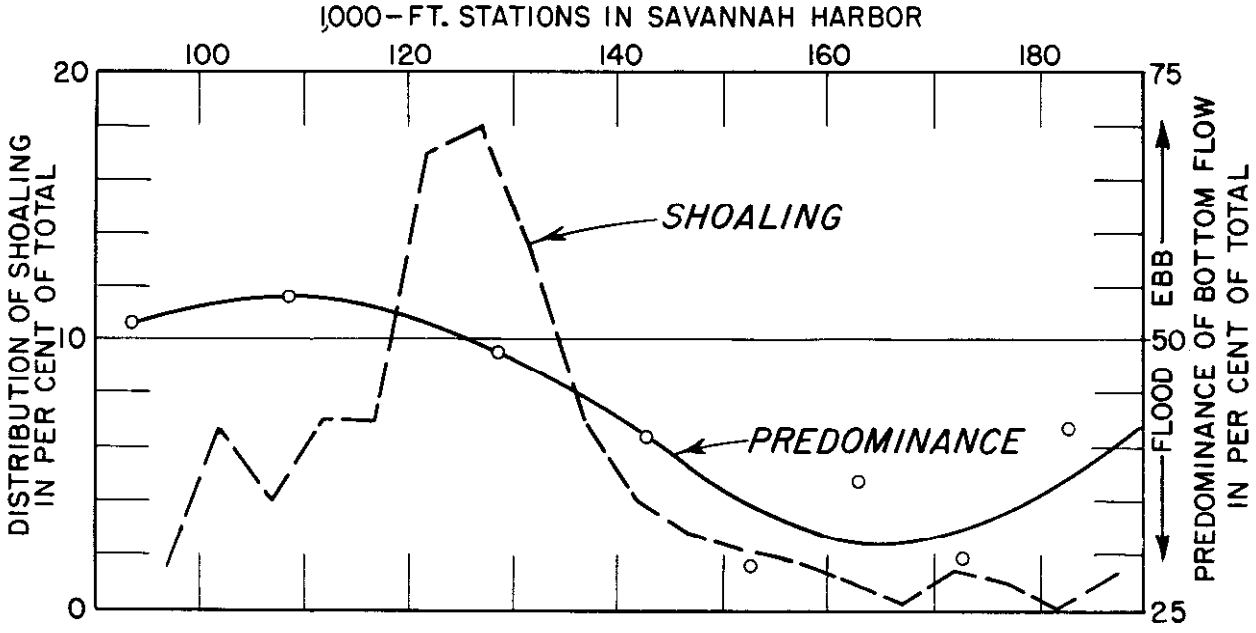


FIGURE 3. — Relation of shoaling and predominance of bottom flow.

average annual distribution of shoaling in the harbor. The solid line in figure 3 is a replot of the bottom flow predominance values from figure 2, again plotted against 1,000-ft. channel stations. A value of 50 percent again indicates that upstream and downstream flow is in balance, while values in excess of 50 percent indicate downstream predominance and values less than 50 percent indicate upstream predominance. The nodal point, or the location where upstream and downstream flow at the bottom is in balance, is at about station 125 as previously described in the preceding paragraph. The dashed line in figure 3 shows the average annual distribution of shoaling in Savannah Harbor for the years 1953 and 1954, during which period the channel depths and widths throughout the harbor were as just described.

In this plot, the harbor has been divided into 5,000-ft.-long sections, and the value plotted in the center of each section represents the percentage of total shoaling of the harbor that occurred in that section. Of the total shoaling 35 percent occurred between stations 120 and 130 or within the 2-mile-long channel reach encompassing the nodal point for bottom flow predominance. In addition, 20 percent of the total occurred in the adjacent 2-mile-long downstream reach, and 14 percent of the total occurred in adjacent 2-mile-long upstream reach. Therefore, from a total improved channel length of about 31 miles, about 69 percent of the total shoaling occurred in a 6-mile-long reach that brackets the nodal point for bottom flow predominance. Thus, more than two-thirds of the total annual shoaling occurs in less than one-fifth of the total length of the dredged channel.

Both the annual shoaling rate and the pattern of shoaling in Savannah Harbor have changed drastically as the channel depth has been progressively increased, but the shoaling pattern has changed much more drastically than the shoaling rate. From data compiled by the U.S. Army Engineer District, Savannah, and especially by Ralph F. Rhodes, special assistant to the district engineer until his retirement in 1949, it has been possible to determine the approximate shoaling patterns and the average annual shoaling rate for the following four channel conditions which are shown in figure 4: (1) 30 ft. deep at mean low water from deep water in the ocean to station 190, thence 26 ft. deep to the Seaboard Airline Railway bridge, thence 21 ft. deep to station 94; (2) 30 ft. deep at mean low water from deep water in the ocean to station 190, thence 26 ft. deep to station 94, thence 21 ft. deep to Creosote Wharf; (3) 30 ft. deep at mean low water from deep water in the ocean to the Seaboard Airline Railway Bridge, thence 26 ft. deep to Creosote Wharf; and (4) 36 ft. deep at mean low water from deep water in the ocean to station 190, thence 34 ft. deep to station 94, thence 30 ft. deep to station 82.

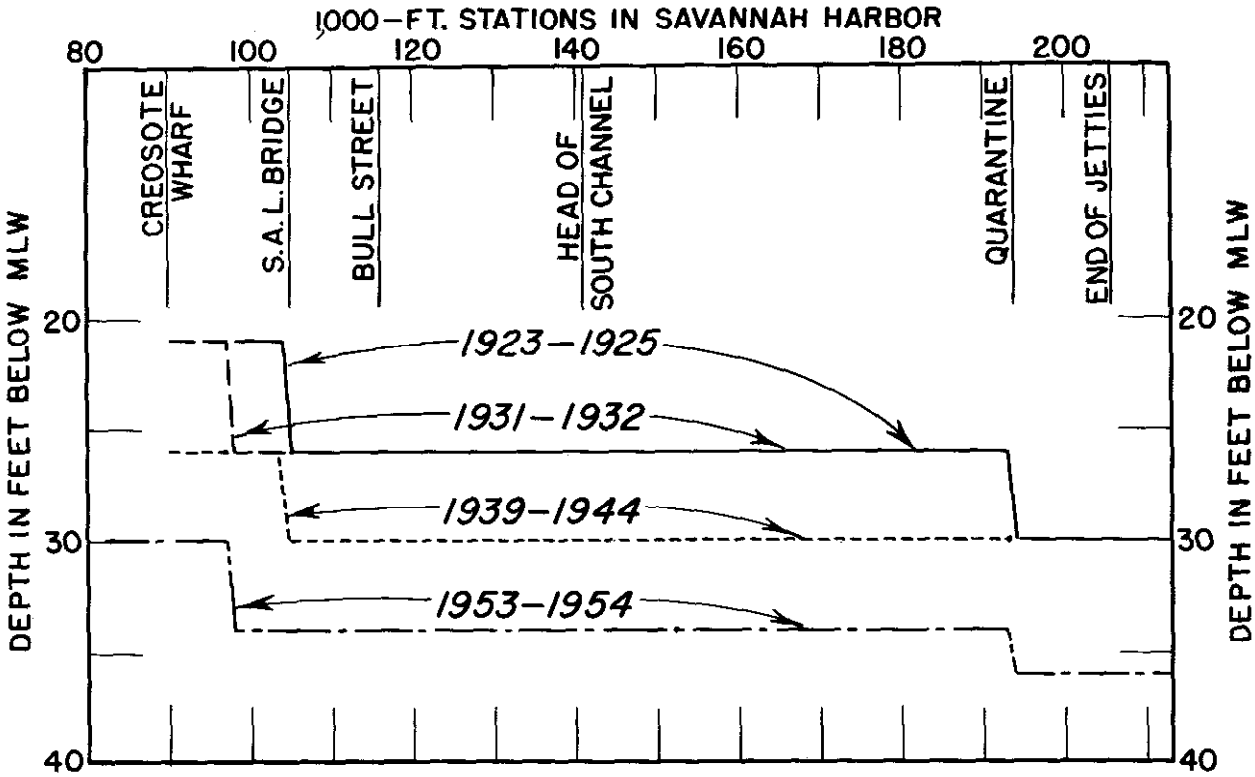


FIGURE 4.— Past and present channel depths.

For purposes of the analysis described below, data obtained for the period 1923–25 were used to represent the channel conditions described in (1) above, those for the period 1931–32 were used to represent the conditions described in (2), those for the period 1939–44 were used to represent the conditions described in (3), and those for the period 1953–54 were used to represent the conditions described in (4). These periods were selected because no changes in project channel dimensions were made immediately before or during any of these periods. The harbor was divided into the following three reaches: stations 107 to 134, representing approximately the upstream one-third of the harbor; stations 134 to 166, representing approximately the central one-third; and stations 166 to 205, representing approximately the downstream one-third. The annual average shoaling rate for each one-third of the harbor was determined for each of the four periods, as well as the annual average harborwide shoaling rate for each period.

The annual average shoaling rates for the upstream one-third (stations 107–134) of the harbor and the downstream one-third (stations 166–205) for each of the four channel conditions are plotted in figure 5. The points connected by the dashed line in figure 5 show the annual average harborwide shoaling rates, in millions of cubic yards, for each of the four periods. Examination of figure 5 indicates that the annual average shoaling rate for the lower one-third of the harbor was about 2.1 for the first channel condition, 1.5 for the second, 1.2 for the third, and 0.2 for the fourth, which shows a steady and significant *decrease* in shoaling with each deepening and/or upstream extension of the project channel. For the upstream one-third, the annual average shoaling rate was about 0.1 for the first channel condition, 0.9 for the second, 2.0 for the third, and 4.6 for the fourth, which shows a steady and significant *increase* in shoaling with each deepening and/or upstream extension of the project channel.

The annual average harborwide shoaling rate in millions of cubic yards was 2.8 for the first channel condition, 4.3 for the second, 6.2 for the third, and 7.2 for the fourth. Thus, the change from the first condition to the second increased shoaling by about 1.4 million cu. yd. per year, and the change from the second condition to the third caused a further increase of about 2 million cu. yd. per year. However, the change from the third condition to the fourth,

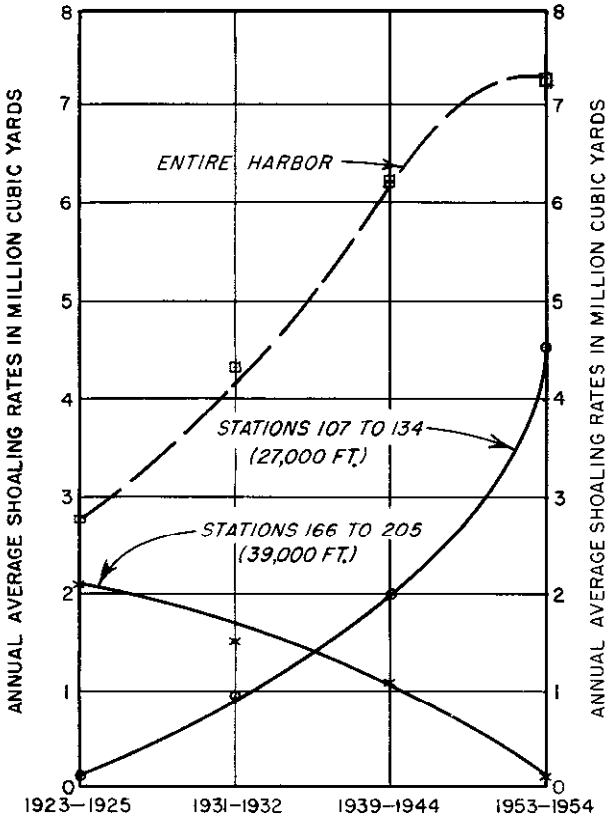


FIGURE 5. — Past and present shoaling characteristics of Savannah Harbor.

which was by far the most drastic physical change and the one that caused the maximum change in shoaling distribution, increased total shoaling by only 1 million cu. yd. per year. This trend indicates that by the time the third channel condition had been attained, density effects in the harbor had developed to the point that essentially all of the potential shoaling material from upland sources was being trapped somewhere within the system. Therefore, when the channel was further deepened and extended to the fourth condition, the changed density current regimen effected a further upstream shift in the location of the major shoal area, but the harborwide shoaling rate increased by only some 16 percent.

In the center one-third of the harbor, between channel stations 134 and 166, the annual average shoaling rate in millions of cubic yards was about 0.6 for the first channel condition, it increased to about 1.8 for the second, it increased further to about 3 for the third, and then decreased to about 2.4 for the fourth. In comparison to the downstream one-third of the harbor, the annual shoaling rate for the center one-third was much lower for the first channel condition, approximately the same for the second, about two and one-half times as great for the third, and more than 10 times as great for the fourth. In comparison to the upstream one-third of the harbor, the annual average shoaling rate for the center one-third was six times as great for the first channel condition, twice as great for the second, one and one-half times as great for the third, but only about one-half as great for the fourth.

These trends also demonstrate quite clearly the upstream migration of the major shoal area as the project channel was progressively deepened or extended, or both. For the first channel condition, the downstream one-third of the harbor experienced by far the greatest shoaling rate; for the second condition, the center one-third shoaled at a slightly greater rate than the downstream one-third and a much greater rate than the upstream one-third; for the third condition, the center one-third shoaled at a slightly greater rate than the upstream one-third and at a much greater rate than the downstream one-third; and for the fourth channel condition, the upstream one-third shoaled at about double the rate of the center one-third and at more than 20 times the rate of the downstream one-third.

Although the harborwide annual average shoaling rate increased only about 16 percent when the 30- and 26-ft. channels of condition (3) were deepened to 36 and 34 ft. as in condition (4), a very critical situation was created with respect to the disposal of dredge spoil. Downstream from about the mouth of Back River, spoil areas of almost unlimited capacity are available or can be readily obtained; however, upstream from this location the available spoil areas are being filled rapidly and there are no additional areas that can be developed economically for spoil disposal purposes. For the third channel condition discussed above, about 68 percent of the annual average shoaling rate of 6.2 million cu. yd. occurred downstream from the mouth of Back River, or where spoil disposal areas were readily available; however, for the fourth channel condition, about 64 percent of the annual average shoaling rate of 7.2 million occurred upstream from this location where spoil disposal areas were limited. The increase of about 2.6 million cu. yd. per year in Front River is rapidly filling the remaining spoil disposal areas, and the development of a critical spoil disposal problem is imminent unless corrective measures are taken to reduce shoaling in the upstream one-third of the harbor.

During the course of a recently completed hydraulic model investigation of Savannah Harbor at the Waterways Experiment Station, an improvement plan designed to reduce shoaling

in the upstream one-third of Savannah Harbor was developed and tested thoroughly. The general features of this plan are shown in figure 6 and consist of the following: (1) a deep basin,

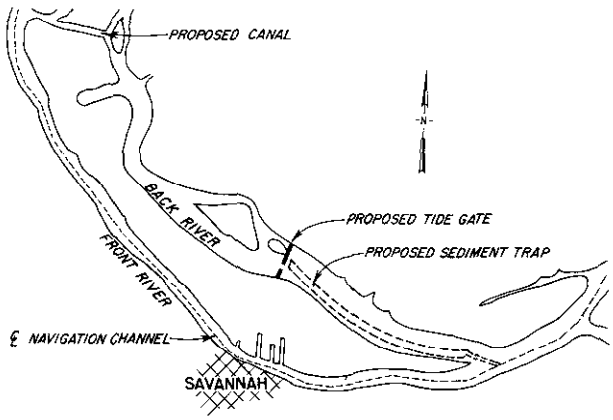


**FIGURE 6. — Elements of sediment trap plan for Savannah Harbor.**

or sediment trap, dredged in the downstream end of Back River and connected to the navigation channel by an access channel 200 ft. wide and 34 ft. deep at mean low water; (2) a dam equipped with one-way tide gates located in Back River just upstream from the sediment trap; and (3) a dredged channel, 200 ft. wide by 26 ft. deep at mean low water, connecting Back River and Middle River about 1 mile upstream from the confluence of Front and Middle Rivers.

The one-way tide gates in the Back River dam would allow the tidal prism of Back River to fill in the normal manner from the downstream end, thus permitting the inflowing currents to transport sediments from the navigation channel into the sediment trap. The gates would close when the Back River tidal prism upstream from the dam is filled, or when the water level on the two sides of the dam becomes equal. During the ebb current phase, that part of the Back River tidal prism upstream from the dam would flow through the dredge cut into Middle River and thence downstream through Front River, thus increasing the ebb current velocities in Front River to such extent that sediments would be moved downstream to the vicinity of the access channel to the sediment trap. Model tests have shown that this plan would be very effective in collecting in the sediment trap a major part of the sediments that now deposit in the upstream one-third of the harbor.

The above-described plan would not reduce the harborwide shoaling rate; in fact, model tests indicate that the harborwide shoaling rate would be increased slightly. However, because of the proximity of extensive spoil-disposal areas immediately north of the sediment trap and the fact that periodic removal of accumulated sediments from the trap to maintain its efficiency could be accomplished more economically than dredging from the channel proper, it appears that the plan would provide an acceptable and economic solution to the spoil-disposal problem. Furthermore, model tests indicate that, since most future maintenance dredging would be done from the sediment basin rather than from the channel, any given channel depth required for navigation could be maintained more effectively than by present dredging techniques.

In other words, the present practice of allowing the channel to shoal several feet above project depth and then dredging to about 2 ft. below project depth for advance maintenance would no longer be required except perhaps in isolated areas. Also, there is a strong probability that any remaining local shoal areas in Front River would merely require periodic agitation, rather than direct removal, since the net downstream flow in Front River with the plan installed should progressively move any sediments thus agitated downstream to the entrance to the sediment trap. Detailed studies are now in progress by the U.S. Army Engineer District, Savannah, to determine the technical and economic benefits of the plan in relation to the cost of its construction.

The Corps of Engineers makes detailed studies of every proposed change in navigation channel depth or extension in order to arrive at the best possible estimate of the effects of such changes on the quantity of maintenance dredging that will be required in the future. Until fairly recently, the exact location of major shoal areas was of little importance from an economic viewpoint, *since most of the recurring shoaling occurred downstream from major port areas, adjacent to lands not suitable for development,* where spoil areas were readily available. However, with increasing development of the lands adjacent to our tidal waterways for industrial and other purposes, and with the tendency for progressive upstream migration of the locations of major shoal areas because of changed density current regimens in the deeper channels, it is now of great economic importance to determine in advance the effects of a proposed deeper or longer navigation channel on the distribution of shoaling as well as on the quantity of shoaling. Even in those navigation channels where maintenance dredging operations are performed with hopper dredges rather than

pipeline dredges, every upstream migration of a major shoal area increases the haul distance to suitable dumping grounds and thus increases the unit cost of dredging.

In addition to the matter of changes in shoaling distribution discussed in this paper, the engineers of this era who are in charge of the development and maintenance of our tidal waterways are faced with many other problems that were of little or no concern to previous generations of waterway engineers. For example, the effects of a proposed change in navigation channel depth or length on the dispersion and flushing of industrial and municipal wastes must be known in advance from the standpoints of public health and water use. The effects of changed salinity and hydraulic regimens on the fish and wildlife resources of the area must be determined with maximum possible accuracy. Therefore, the modern waterway engineer should initiate his study of the effects of a proposed change in channel dimensions with a full knowledge of the effects of prior changes in the waterway involved, as well as of the effects of similar changes in other waterways of the same general type as that involved. He should be well versed in the principles of tidal hydraulics and should take advantage of every available tool, including hydraulic models when applicable and necessary, to insure that the new regimen of the waterway involved is determined with sufficient accuracy to satisfy the many and sometimes divergent interests involved. This challenge must be met if the continued development of our tidal waterways and the areas they serve is to proceed in a sound technical and economic manner.

# SEDIMENT PROBLEMS OF RIVER DEVELOPMENTS IN DELTAIC REGIMES

[Paper No. 73]

By Chester A. Peyronnin, Jr., *hydraulic engineer, U.S. Army Engineer District, New Orleans*

## Synopsis

Rivers carrying heavy sediment loads present many difficult problems for hydraulic engineers when attempts are made to develop these waterways for navigation or improve them for flood-control purposes. These problems are compounded when the river is actively building a delta system. Problems such as (1) shoaling in the navigation channels, (2) sediment control, (3) floodwater diversion, and (4) river migration must be resolved before full development of the river can be accomplished. These problems can be generalized and illustrated by typical examples of the Lower Mississippi River. This paper discusses each type, and gives some of the engineering considerations in the solution of each type of problem.

## Introduction

The comprehensive development of a river for navigation and to control floods generally requires the resolution of problems concerning flood control, navigation, water supply, and recreational activities. In the case of a river that carries a heavy silt burden in a deltaic environment, these problems become very complex. Emergency relief floodways may become inoperable after only a few years of operation; navigation channels may become completely shoaled in a few weeks; and distributaries may be forced to migrate due to sediment deposition. Considerable research and analysis activities have been devoted to the understanding of the theoretical aspects of these problems, but most solutions must be based upon engineering judgment in the application of fundamental knowledge. This paper draws upon typical problems in the Lower Mississippi River Valley to illustrate possible solutions and the actual solutions employed to solve such problems.

## General Description

The reach of the Mississippi River that will be used as an example extends from mile 130 above the Head of Passes to the Gulf of Mexico, 20 miles below the Head of Passes (fig. 1). Within this area are examples of navigational problems such as Southwest Pass, spillway and floodway siltation as typified by the Bonnet Carre spillway, migration and delta building considerations illustrated by the Cubits Gap delta, and delta destruction as shown by Isles Dernieres and the Timbalier Islands.

## Navigational Problems

The most important problems in navigation projects is the maintenance of channel dimensions. Since the heavy silt load in the rivers under discussion will tend to deposit and form shoals under certain conditions, the maintenance of the channel is very difficult. The most critical location for the shoaling is generally at the mouth of the river where the silt-laden river water meets the heavier density seawater. The fresh water flows into the sea in the form of a

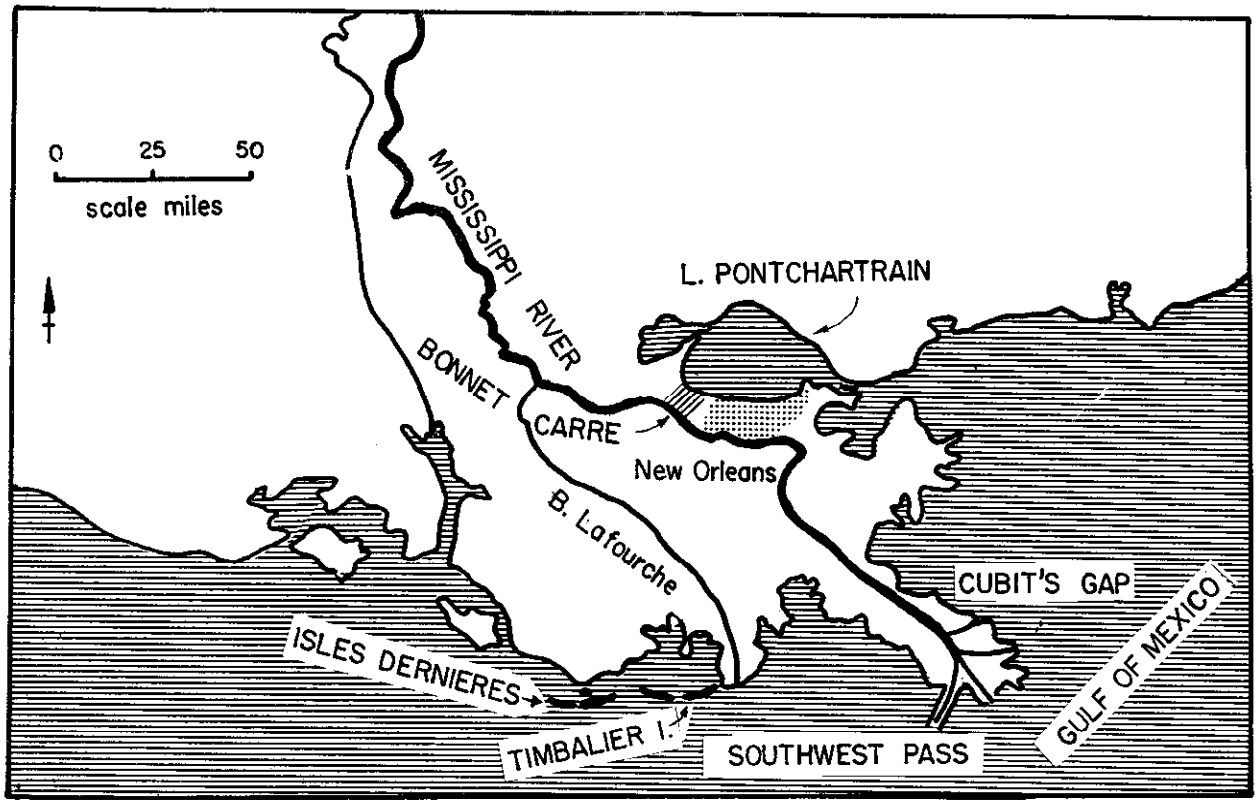


FIGURE 1. — Lower Mississippi River.

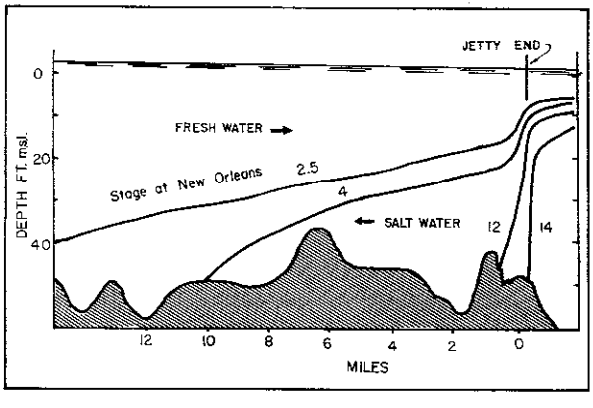


FIGURE 2. — Longitudinal profile of lower Mississippi River.

hypopycnal jet, with the lighter river water flowing over the denser seawater. An interface will be formed between the fluids, thus giving a characteristic wedge shape. This interface will be poorly defined because of mixing due to diffusion currents, but may be defined for study purposes as a line with a salinity expressed as 5,000 p.p.m. chloride ion. In rivers that have been improved for navigation, the deep channel will allow this wedge to move into the river at low stages. Salt water will flow upstream near the bottom while fresh water flows out near the surface. Any material moving near the bottom will be stopped by this wedge because of the local velocity reversal. At high rates of riverflow, this wedge will move out to the mouth, and since bed material will generally be moving at this river stage, a considerable amount of material will be stopped. The result will be a heavy shoal formation near the mouth or in the bar channel (fig. 2). Material that passes over this bar may be returned to the area by the upstream flow unless there are strong littoral currents to move it away.

The problem of heavy shoals in a bar channel may be solved in two ways. One way is to orient the main river channel so that the bedload and suspended material approach the wedge at an oblique angle and they will proceed into and over the wedge. Material carried over the wedge will be in a better position to be swept away, and material deposition at the toe of the wedge will be minimized. This method is shown in figure 3,*A*. An alternate method, which is somewhat of a departure from usual methods, is to allow the material to be deposited normally in the river channel while

navigation is carried on in an alternate channel (fig. 3,*B*). This channel must be positioned so that the salt wedge acts as a natural side to the river, thus stopping any material from moving into this pass. Some bottom flushing will be obtained by the scour action of the salt water. Model tests of both of these methods show benefits with respect to minimizing dredging in the order of 50 to 60 percent reduction over that of the previous alinement. The first method has been employed in the redesign of the Southwest Pass jetty channel and has shown good results in the actual condition. The second method showed model results equally as good as the first, but at a greater cost in this application.

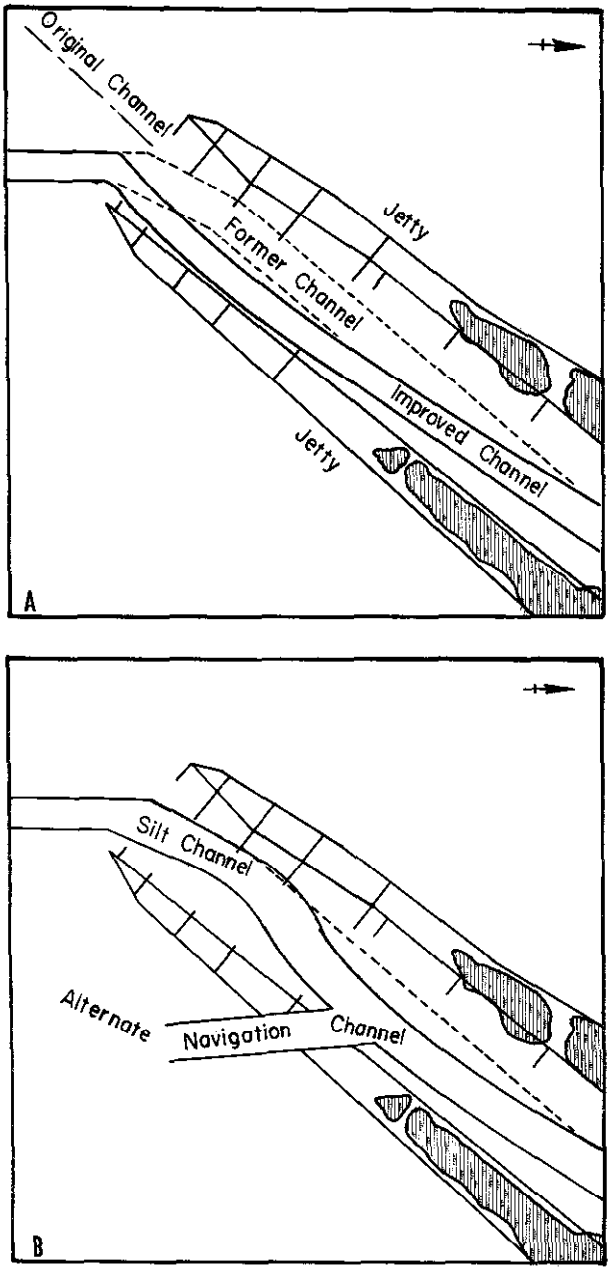


FIGURE 3. — Southwest Pass channel showing (*A*) improved alinement; and (*B*) alternate channel.

Another major problem of sedimentation is that caused by deposition under wharves or piers in the river. Material is trapped under these structures by piling bracing or by debris caught on the piles or deposited by receding high water. When the channel adjacent to the wharf is redredged to maintain the required depth, the material will slide and cause damage to the structure. In addition to slope slides, rotational failures have occurred due to the increased overburden as shown in figure 4. This problem can be solved by revising the channel maintenance program to insure that the sedimentary material is removed from under the structure. Routine maintenance can be improved to include removal of debris from under the structure at frequent intervals.

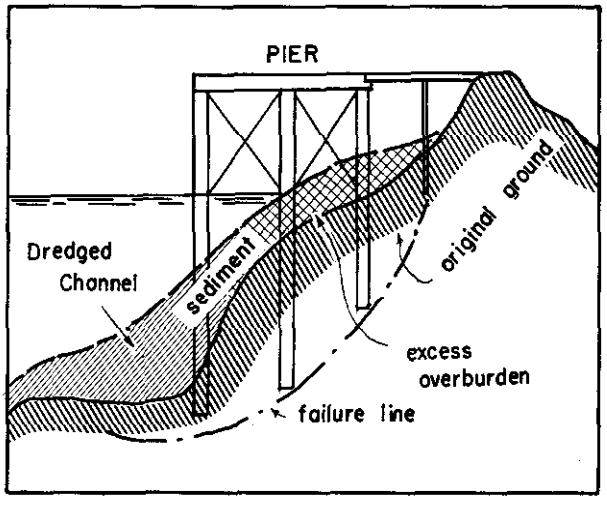


FIGURE 4. — Pier siltation.

## Flood-Control Problems

In the natural state, a river will pass a flood by enlarging its distributaries, opening new ones, or flowing into a larger flood plain. In a highly developed river system, none of these can be allowed to happen. It is therefore necessary to provide some temporary system for increasing distributary flow. Spillways with floodways to the sea are commonly used for this purpose. There are three major sedimentation problems in such a system that may cause difficulty. The first is the repeated annual flood for

which the spillway is not opened. This will allow deposits in the forebay to build up and decrease the entrance efficiency to the control structure. The second is the deposition of sediment in the floodway, which will eventually reduce capacity after several operations. The third is the sedimentation in the outlet waters, which may decrease flow efficiency but which may pose more difficult problems with respect to the ecology of the area.

Bonnet Carre Spillway illustrates these problems rather well. It was completed in 1931 and has been operated in 1937, 1945, and 1950. Pertinent data of the River are as follows:

Drainage basin: 1,245,000 square miles.
Flows at New Orleans: Low, 50,000-100,000 c.f.s.; maximum, at project conditions, 1,250,000 c.f.s.
Stages at New Orleans (Carrollton): Flood stage, 17 ft. m.s.l.; maximum at project conditions, 20 ft. m.s.l.
Sediment sizes: General maximum — 1 mm. (0 phi units) in bed; 0.1 mm. (3.33 phi units) in suspension. Median — 0.025 mm. (5.22 phi units).
Dredging required in Southwest Pass (for previous 35-ft. project depth): Approximately 2¼ million cu. yd. mean annual; 6 million annual maximum.
Salinity of river: 30 p.p.m. chloride ion.
Salinity of Gulf of Mexico: 18,000 p.p.m. chloride ion.
Project depth of Southwest Pass: 40 ft. with 2-ft. overdepth allowed.

Amounts of sediment for each operation of the spillway are shown in table 1. There are problems in the spillway forebay and floodway. There is no problem in Lake Pontchartrain, but the potential of such a problem can be illustrated by the outlet conditions.

In the forebay, the buildup of sediment between operations has caused a reduction in flow area into the structure. During the 1950 operation, it was necessary to attempt to restore capacity by remedial dredging during operation. Although this helped by opening the forebay, it hurt the overall problem by putting more material into the floodway, thereby probably silting it up more than a normal operation would have. Several years after the 1937 operation, a small levee was built to close off the forebay from the river, thus minimizing the annual deposition. This was degraded in 1944 before an expected high water and remained degraded until after the 1945 operation. It has been restored but some deterioration has taken place. Solution of this problem by using this removable levee seems to be well justified, because the levee will prevent the undesirable annual deposition and retain full forebay capacity for the time of operation (fig. 5).

TABLE 1.—*Discharge and sediment fill deposited for Bonnet Carre Spillway for the years operated*

(Design capacity, 250,000 c.f.s.)

| Operations | 28 Jan. to 16 Mar. 1937 | 23 Mar. to 18 May 1945 | 10 Feb. to 19 Mar. 1950 |
|---|---|---|---|
| Maximum discharge, c.f.s. | 211,000 | 318,000 | 223,000 |
| Average discharge, c.f.s. | 131,000 | 223,887 | 156,217 |
| Total discharge, acre-feet | 12,400,000 | 24,500,000 | 10,900,000 |
| Fill deposited, weir to Airline Highway: | | | |
| Volume, cu. yd. | 7,200,000 | 5,000,000 | 2,800,000 |
| Percent of total | 70 | 40 | 56 |
| Fill deposited, Airline Highway to Illinois Central Railroad: | | | |
| Volume, cu. yd. | 3,000,000 | 7,500,000 | 2,200,000 |
| Percent of total | 30 | 60 | 44 |
| Fill deposited in Lake Pontchartrain (approx.): | | | |
| Cu. yd. | 2,230,000 | 17,500,000 | 7,400,000 |
| Acre-feet | 2,000 | 10,800 | 4,600 |

The floodway proper of such a structure will experience severe deposition during the operation period. In addition to the normal mechanics of deposition there is the problem of deposition due to interference in the flow by trees and brush. If the spillway can be kept clear of undergrowth and large trees, the flow line can be improved and the deposition decreased somewhat. Once the floodway capacity has been reduced by filling, the only solution is to dredge a channel. There are many possibilities in this restoration process, but the most economical seems to be the use of a single channel through the area because the spoil can be placed in the remaining part of the floodway at minimum costs. It would probably be necessary to clear this channel after every operation.

In the Bonnet Carre floodway, the pattern of deposition has been influenced to a great extent by flow conditions in the floodway. In 1937, the area from the structure to the Airline Highway had not been cleared, and so a large percentage of material settled out in this area. After the operation, the land was cleared and grubbed, and in 1945 and 1950, larger percentages of material were deposited in the section between the Airline Highway and the lake. Because of this deposition in the floodway, the 1950 flow line was elevated approximately 1 ft. above the 1945 flow line near the structure, and lesser amounts near the outlet. Because of the loss of capacity, the spillway can probably pass the design flow only once more before remedial measures are necessary.

The outfall sump of a spillway must receive all of the excess sediment from the floodway. The effect of such sediment may have two aspects, hydrological and biological. The sediment is deposited by two processes; the first by decreased velocity and sediment fallout, and the second by fresh-saline water interaction. This

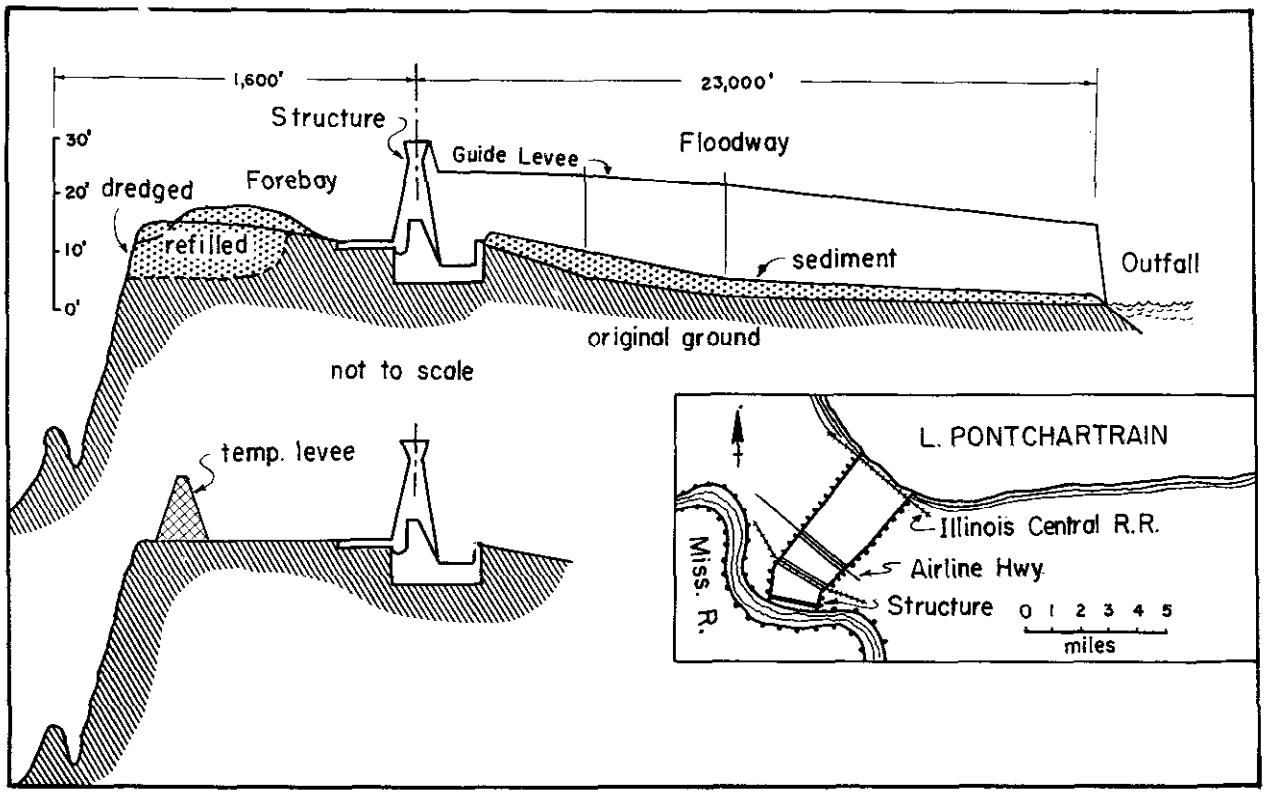


FIGURE 5. — Bonnet Carre spillway.

interaction results in flocculation which removes most of the suspended sediment. The remaining fine sediment will tend to increase the turbidity, thus giving the appearance of dirty water, but will not produce significant sedimentation because it will remain in suspension.

The quantity of sediment deposited in Lake Pontchartrain by each operation of Bonnet Carre is given in table 1, for a total of 17,400 acre-ft. All of this has been deposited within an area of 25 square miles adjacent to the floodway. At an operation frequency of approximately once in 10 years, this would amount to a 1-ft. decrease in lake depth over its 410,000 acres in a period of 825 years at present conditions. This is probably less sediment than the amount deposited by natural erosional and depositional processes active in the lake and less than the amount removed from the lake for various projects in the area that have used hydraulic fill from the lake. The effect upon the hydrology of the lake is thus relatively negligible. If this sedimentation is important in other situations, the only solution in a lake is remedial dredging. The effect upon the biological process is beneficial because of the nutritional value of the nutrient salts in the sediment. Thus, in this spillway, the outfall problem is negligible, although in others it may be important.

## Delta Growth

Deltaic coasts are those that have been built by a depositional process in which the transport capacity of some parent stream carrying material into an area exceeds that of the local area sea currents carrying it away. The material forms a delta whose shape is controlled by the regime of the area in which it is deposited. Deltas that form in shallow saline water from a freshwater stream tend to form fan-shaped fronts with braided distributaries migrating through the delta. These distributaries are continually bifurcating and migrating as they silt up or are diverted by natural phenomena. Such a formation is illustrated by the Cubits Gap formation of the Mississippi River (fig. 6). Deltas that build in deeper water tend to advance in longer, narrower formations as illustrated by the growth at Southwest Pass of the Mississippi River (fig. 7). Since the delta is essentially submerged, it is outlined by the advance of the 35-ft. contour. The fan-shaped delta builds useful marshland while the finger-style delta builds underwater shoals of little contemporary usefulness.

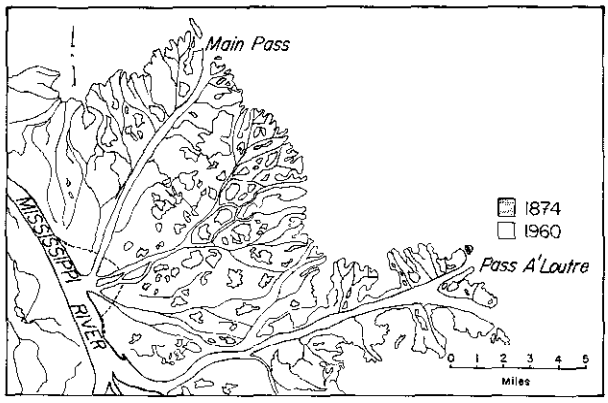


FIGURE 6. — Cubits Gap delta.

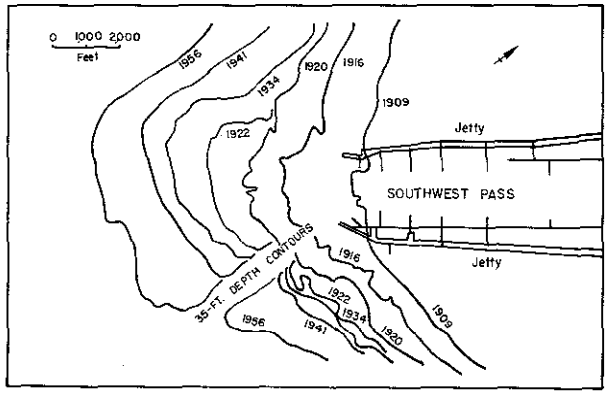


FIGURE 7. — Southwest Pass delta.

Any river work that tends to alter the natural sedimentary activity of the river will materially affect the delta. Navigation works generally require deep channels to offshore depths, which may be too deep for useful delta building. Flood improvements may require the closing of a distributary which will then rob an area of its source of supply. Power or flood-control dams may so obstruct the sediment load as to prevent an adequate amount of material from entering the delta and thus cause an imbalance in the equilibrium.

The extensive and detailed experimentation work at Southwest Pass has resulted in the improvement of flow conditions such that depositional effects of the salt wedge are minimized and the sediment load carried efficiently out to sea. Because of the great depths required for navigation, the bulk of the sediment is deposited in deep water and does not assist in the building of a useful delta (fig. 7). In order to assist in maintaining the integrity of the land along Southwest Pass, several controlled outlets to the Gulf are maintained and allowed to pass sediment to the bay area adjacent to the pass. Other than this small amount, all other sediment is wasted. This can be contrasted with the essentially natural channel forming the Cubits Gap delta.

## Delta Destruction

Once a delta is abandoned because of migration of the parent stream, the erosional and subsidence effects, which have been present, become dominant and the delta is slowly destroyed. From an economic standpoint, it is desirable to maintain these deltas because this land, while generally not inhabited or developed, is the location of important interests such as fur trapping, and wildlife conservation. If the distal fringes of these deltas can be protected, the land will continue to be useful for extended periods because vegetation growth and decay will continue to offset the subsidence to a great extent.

These fringes are subject to two types of attack. The first type of attack is that of net subsidence which sets in once the active depositional process ceases. Subsidence has been in effect all during the active delta period with new material replacing that which subsided. Once inactive, the delta front will continue to subside. The fringe beaches will not subside as rapidly, but will form offshore islands backed by shallow bays. These islands will act as protective barriers against wave attack on the mainland, but in time the islands will yield to subsidence and erosion and the attack will continue on the mainland.

The second type of attack is that which is commonly associated with sea processes; that is, wave action and littoral current movement of material. In this attack the waves will work a sorting process, moving the fine material off the beach and into the nearshore bottom where the littoral currents can move it away. If this process can continue to equilibrium, a relatively stable beach will be formed on the delta front. However, this beach will generally be attacked by higher waves during storms and the stability upset, thus beginning the process again.

### *Timbalier Islands*

The Timbalier Islands probably began as spits off of the mouth of Bayou Lafourche while it was an active distributary of the Mississippi River during both its deltaic growth in the Lafourche Delta and its later activity in the modern delta. Material was continually fed in small quantities to the mainland and islands until 1904, when Bayou Lafourche was closed off from the Mississippi River at its head for flood-control purposes. Once this source of sediment was cut off, the process of erosion became dominant. In the last 50 years, there has been a retreat of the mouth of the bayou at Belle Pass at a rate of 120 ft. per year (fig. 8).

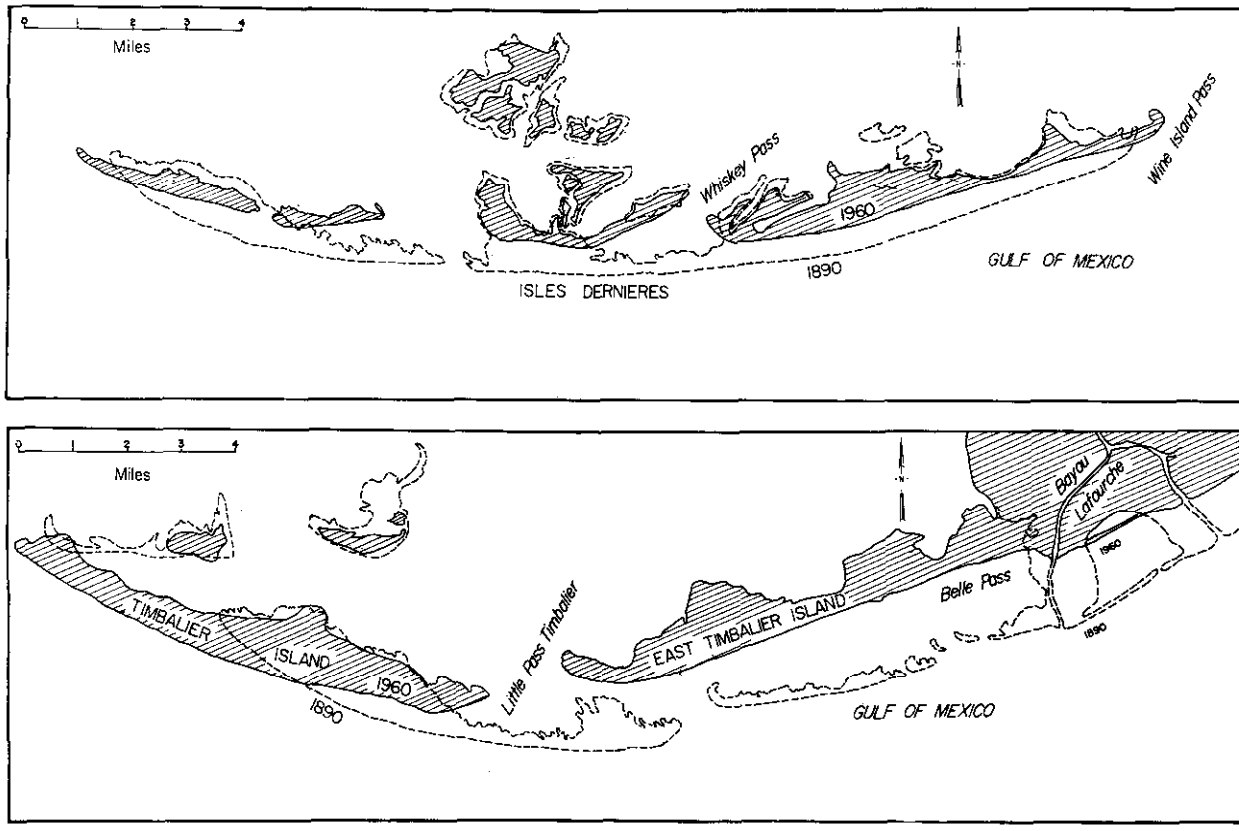


FIGURE 8. — **Timbalier Islands and Isles Dernieres.**

The littoral currents in this area are westerly and wave action from the southeast and south has a westerly longshore component, so that material which is eroded from this headland drifts westerly along the island chain. The chain is apparently building a continuous strip, but is experiencing a net loss of material in the planing action. The east end of East Timbalier Island has been retreating at a rate of approximately 100 ft. per year, and the western end at a rate of 75 ft. per year. Timbalier is retreating at a rate of only 50 ft. per year at its eastern end and is advancing on its western end as a result of the planing action. Since no new material is being added to this general mainland area, it will continue to erode and the islands will continue to erode with it. As more of the coast is attacked the rate will decrease, but eventually the islands will be pushed back to the main marsh area which, in turn, will be subject to attack.

For this regime, there is no solution for preservation. Seawalls, bulkheads, or breakwaters are too expensive for the area. Artificial nourishment is also out of the question because of a lack of available material for nourishment. Maximum sand size on the beach is 0.5 mm. and median size is 0.175 mm. No sand equal to or better than this is available in the general region for replenishment. There is some sand up in the mainland along Bayou Lafourche, but not in the quantity required, and it would be inadvisable to rob the immediate mainland that is being protected. Some fresh water is now being diverted into Bayou Lafourche from the Mississippi River by a pumping station, which began operation in 1955, but not in sufficient quantities to provide any significant amount of material.

### *Isles Dernieres*

The Isles Dernieres are remnants of the distal extremities of the old Teche Delta of the Mississippi River, which was active between 3,000 to 4,000 years ago. While the delta was active, there was a general fan-shaped alluvial delta advance into the sea. When the river migrated away, the delta front remained and was reworked into sandy beaches fringing the delta. Subsidence of the land behind these beaches caused the beaches to emerge as islands separated from the mainland by bays.

Since no new material is moving into this

area, the islands are being reworked and are succumbing to the action of wave attack (fig. 8). At the present time, the central part of these islands is receding at a rate of approximately 40 ft. per year. The eastern end is receding at a lesser rate of 24 ft. per year and the western end at the rate of 12 ft. per year. The ends are advancing slightly, due to the littoral transport of material from the center section to the ends. The littoral current off the islands generally flows from east to west with the exception of a local reversal at the eastern end of the islands. Waves from the southeast, south, and southwest refract almost perpendicular to the shore so that there is only limited lateral motion of the beach material. There is a general reduction in shore area and both shore and nearshore bottom volume. The indications are that these islands are being planed away and will gradually disappear. When the islands are gone, there will be a direct attack upon the mainland, which is predominantly a salty marsh. This mainland will again develop a frontal beach due to the removal of the fine material, but this in turn will be attacked and continuously destroyed by wave and storm attack.

The beach material on these islands is rather fine, having a maximum size of 0.5 mm. and a median size of 0.15 mm. There are no suitable sources of nourishment material available near the islands, so that artificial nourishment cannot be employed. Groins would not help, because there is very little movement along the beach. An offshore breakwater or seawall would be too expensive and would require costly maintenance because of the general subsidence of the area and would be economically unjustified because of the lack of development on the islands or adjacent mainland.

## Solutions

The standard solutions in beach-erosion control are the use of structures such as groins, seawalls, or breakwaters; the replenishment of the beach front; and the stockpiling of material to be placed by littoral processes. These are generally used singularly or in combination, depending upon the particular problem. All such solutions require the expenditure of comparatively large sums of money and, when measured against the benefits, yield a low benefit-cost ratio because of the expanse of area to be protected and the sparseness of habitation.

The best technical solution for such an area lies in the reestablishment of the inflow of sediment-laden water into it. This means that the parent stream, or branches of it, must be returned to provide natural nourishment to the area. This would seem to be an almost impossible solution if present navigation and flood-control concepts are considered. Newer thinking has presented some interesting possibilities that would make this concept worthy of consideration. For instance, with regard to the example, the Mississippi River has long been an important navigable waterway and a vast sum has been expended to develop and maintain the mouth of it as such. To the east of it, however, there is a tidewater channel being constructed from New Orleans to the Gulf of Mexico called the Mississippi River-Gulf Outlet, which will allow seagoing vessels to reach the city and enter the Mississippi River through a lock that may be constructed in the future if justifiable. This has many advantages over the river since it is not affected by flooding cycles or serious siltation problems. It thus appears feasible to construct ship channels to handle commerce to inland ports without utilizing major waterways. Below a particular point on a stream, it is possible to allow the stream to return to a modified natural meandering with many distributaries. These smaller more numerous openings would pass floods on wider flood plains and build a more useful delta.

It is not proposed to allow completely unrestricted action, since this would present many additional problems. The only practical solution to the erosion of deltas lies in the long-range preplanned control of distributaries, utilizing control structures and revetment of directional control of many of the distributaries to prevent undesirable migration. This system has been used to a limited extent in certain locations, such as along Southwest Pass of the Mississippi River, where an attempt has been made to maintain the status quo of the land adjacent to the navigable pass by controlling the several side outlets so that there is no change in the flow and sediment passing out through them but sediment is carried out so as to fill the land.

In deltaic areas that have not developed commercially or industrially, the plan would be easily implemented. Any flood-control or navigation work on rivers could be planned to include the preservation of the delta system below a certain point in the main stream.

## Conclusions

In river systems, there are many problems involving sedimentation from various sources. These are increased by river developments and, generally, the more extensive the development, the more complex are the problems. All of this can be solved or minimized by good engineering design after careful analysis of the problems.

# SEDIMENT DISCHARGE TO THE COAST AS RELATED TO SHORE PROCESSES

[Paper No. 74]

By GEORGE M. WATTS, *hydraulic engineer, Beach Erosion Board, U.S. Army Corps of Engineers, Washington, D.C.*

## Synopsis

The supply of sediments to the shore by stream and river discharge is a very important factor in the stability of the shoreline. When this supply is altered, littoral forces will accordingly try to adjust the shore zone. The development of works on upland drainage basins for flood control and other purposes generally reduces the sediment supply to the coast and the resulting effect to the shore zone will be that of erosion. Unless an appropriate plan of shore protection is developed and constructed, valuable coastal property may be lost.

This paper discusses two coastal areas — Ventura and northern Orange County, Calif. — where the supply of sand to the coast has been greatly reduced by basin development. In each case the effects on the shore zone are pointed out and the plan of improvement developed to solve the resulting shore erosion problem.

## Introduction

Engineers have long recognized the importance of sediment transport to the coastal zone by river or streamflow and the general effect on this zone if the sediment supply is altered. As the development and control of drainage basins tributary to our coast progress, it is apparent that sediments supplied to the shore zones from this source are being reduced. Quantitative survey data indicate that general erosion prevails along our coast, but these and other data are not sufficiently definitive to evaluate what part of the erosion is due to a reduction of sediments supplied to the coast by stream and river flow.

The stability of a shore or beach zone depends, for the most part, on there being a balance between supply and loss of material in the littoral zone. If a reduction in supply occurs from a reduction of sediment supply by upland streamflow, the littoral forces (mainly, waves and wave-induced currents) will tend to erode the backshore to obtain materials to balance this reduction. On the other hand, the retention of sediments by upland development would appear to provide benefits related to the problem of sedimentation within navigation channels at coastal inlets. This may be true in certain instances, but it is generally not the case. The reduction of sediment supply to the coast increases the unfavorable condition of unbalance between applied littoral forces and material movement, and if the composition of the backshore is mainly fine-sized materials, the shore may recede a considerable distance. Under these conditions it may well be that the actual quantity of littoral drift moving along the shore and into an inlet channel will be substantially greater as compared to the condition where flow from upland streams provide a supply of suitably sized sand for nourishing the shoreline for some distance along the shores adjacent to the inlet.

The U.S. Army Corps of Engineers has a direct interest in this overall aspect. The chief of engineers is responsible for making studies and carrying out beach erosion and navigation projects, in accordance with appropriate Federal Laws and Acts pertaining to the Federal role in such functions.

## Purpose

This paper presents cases where the supply of material to the coast by upland flow was greatly reduced, points out the resulting effects to the shore zone, and summarizes the plans developed to improve or stabilize the shore zone.

## Presentation of Cases

### *Ventura, Calif.*

#### *General*

The shore sector near the city of Ventura, known as Pierpoint Beach, is about 75 miles northwesterly of the city of Los Angeles. Sediments contributed to the coast in this area are from the Santa Clara and Ventura Rivers. Location of the area is shown on figure 1, the drainage basin of the rivers on figure 2. The dominant direction of littoral transport, i.e., the dominant direction of alongshore movement of littoral materials by the littoral forces, is southerly, but, due to variability in direction of wave attack, frequent transport of beach materials to the north does occur. The Santa Clara River delta advances seaward following periods of excessive flooding and tends to act as a barrier to littoral drift moving downcoast from the Ventura River mouth and the coast north thereof. The barrier effect extends to the Pierpont Beach area, and the advance or retreat of the beach in the Pierpont area has been greatly influenced by fluctuations of the Santa Clara River Delta.

#### *Sedimentation and Other Data*[1]

The mouth of the Santa Clara River is located

[1] Data from "Special Interim Report on Ventura Area," Beach Erosion Control Report, U.S. Army Engineer District, Los Angeles, Calif., August 1961; House Doc. 458, 87 Cong., 2d Sess.; House Doc. 29, 83d Cong., 1st Sess.

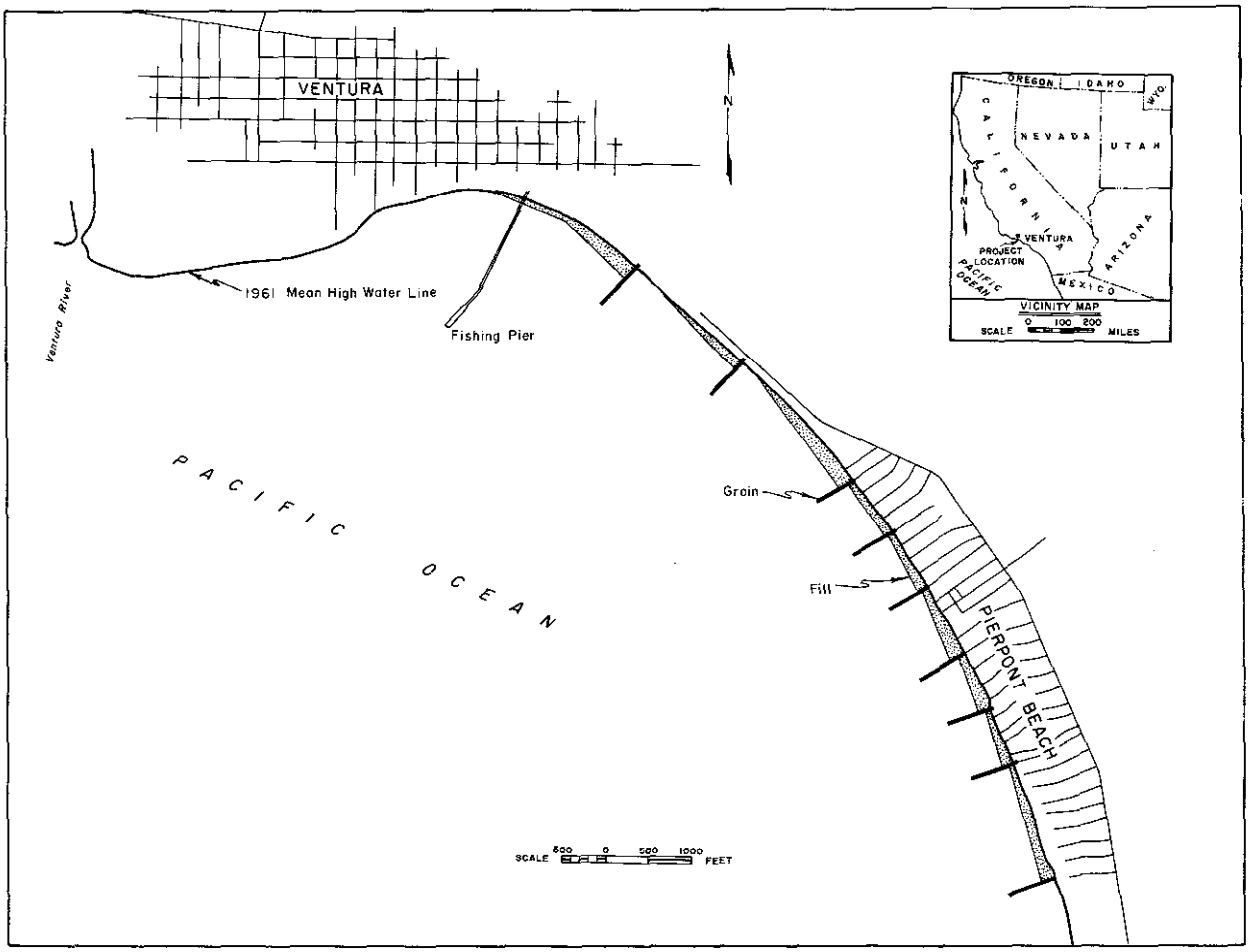


FIGURE 1. — Ventura area, California, general plan of improvement.

about 5 miles downcoast of Ventura. The drainage basin of this river comprises about 1,605 square miles, with about 90 percent of the drainage from the inland slopes of the western San Gabriel Mountains and the southwestern slopes of the eastern Santa Ynez Mountains. Slopes of the river in the mountains average about 160 feet per mile and about 15 feet per mile on the coastal plain. The basin is typical of this part of California in that it consists mostly of sand-producing sedimentary rocks. The estimated seasonal runoff for the river over a 40-year period (1893–1932) was 152,100 acre-feet. The estimated annual sedimentation rate (sediments that could reach the stream and be subjected to transportation and/or deposition therein) for the Santa Clara River is as follows:

Santa Clara River sedimentation data (1893-1932):

| | | |
|---|---|---|
| Drainage area | sq. mi... | 1,605 |
| Sand-producing part of drainage area | sq. mi... | 960 |
| Rate of sedimentation per square mile | cu. yd... | 2,900 |
| Average annual sedimentation of sand | cu. yd... | 2,800,000 |

It was estimated that the total quantity of sediment reaching the coast each year was about 50 percent of the average annual sedimentation rate, or less than 1,400,000 cubic yards. Construction of the Santa Felicia Reservoir in the Piru tributary drainage area reduced the annual sand contribution by an estimated 340,000 cubic yards. The effect of other reservoirs constructed since 1948, along with the effects of a series of extremely dry years, has further reduced the sediment contribution to the coast, and the average since 1948 is now estimated at between 633,000 and 840,000 cubic yards per year. Thus, where there was 1.4 million cubic yards per year reaching the coast southeast of Ventura prior to 1948, there is now only about 50 percent of that amount. Though the Santa Clara River sediments mainly provide

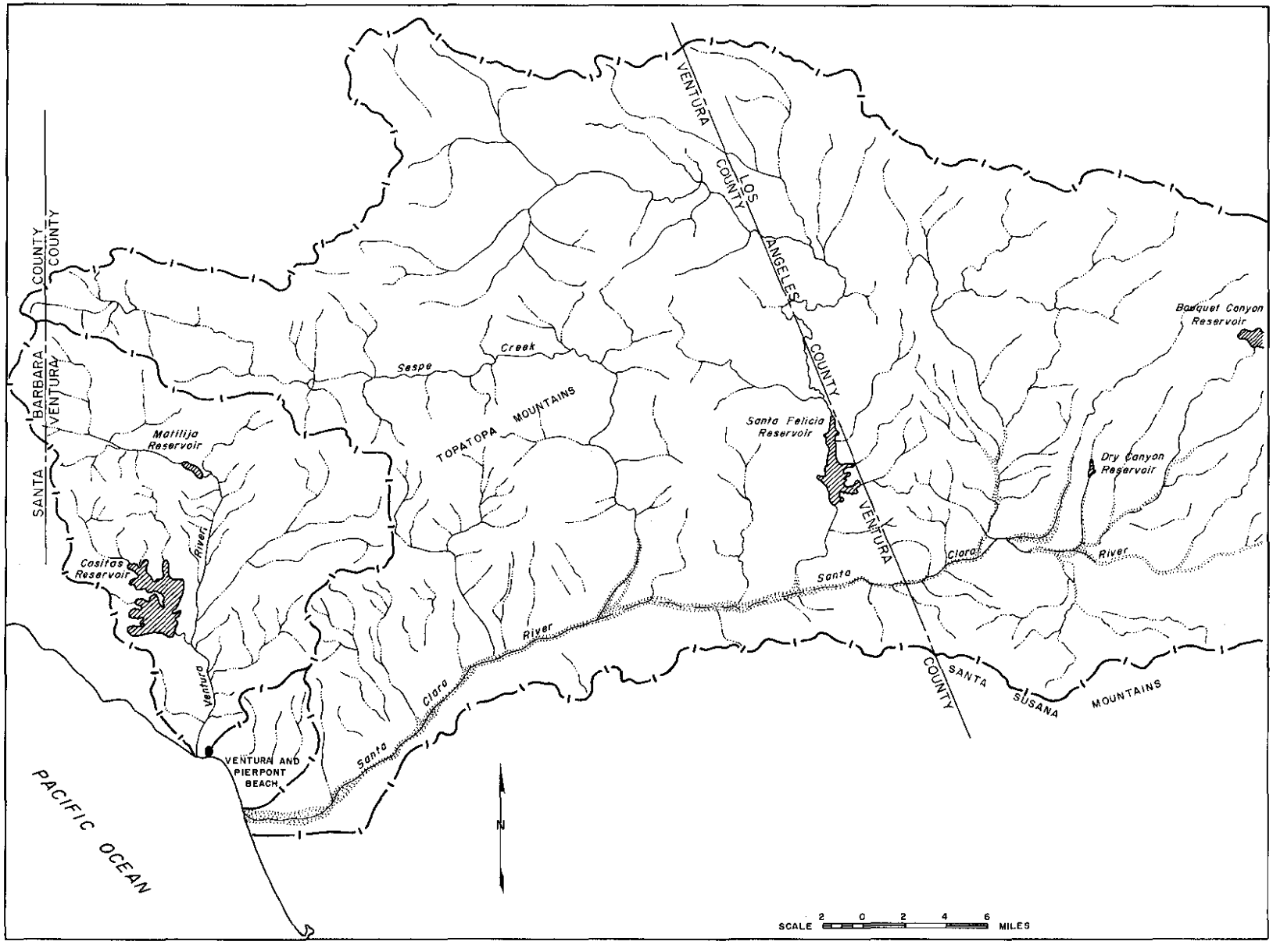


FIGURE 2. — Drainage areas, Ventura and Santa Clara Rivers, Calif.

nourishment for the shores downcoast of the river mouth, some nourishment to the upcoast shores in the Ventura area occurs by reason of periodic reversal in direction of littoral transport, i.e., to the northwest. As mentioned, the seaward position of the delta greatly affects the stability of the Pierpont Beach sector due to the groin effect of the delta.

The mouth of the Ventura River is near the northerly limit of Pierpont Beach and the city of Ventura. The drainage basin of the Ventura River is 228 square miles, and it is enclosed on the north, east, and southeast by the Santa Clara River basin and bounded on the west and southwest by the Santa Ynez River basin. Gradients are, in general, comparable with those of the Santa Clara River. The annual sedimentation of sand for the Ventura River prior to 1948 was as follows:

| Ventura River sedimentation data (before 1948): | | |
|---|---|---|
| Drainage area | sq. mi. | 228 |
| Sand-producing part of drainage area | sq. mi. | 149 |
| Rate of sedimentation per square mile | cu. yd. | 4,260 |
| Average annual sedimentation of sand | cu. yd. | 635,000 |

No data are available on the amount of sediment carried by the Ventura River to the shore, but, based on the data for the Santa Clara River, it was estimated that 50 percent reached the shore, or about 300,000 cubic yards per year. Due to construction of reservoirs on Matilijia and Coyote Creeks since 1948, it is estimated the sediment contribution to the ocean has been reduced by 50 percent.

The sediment yields to the coast for the Ventura and Santa Clara Rivers (located northwest and southeast, respectively, of Pierpont Beach) are summarized in table 1.

TABLE 1.—*Summarized sedimentation data for Santa Clara and Ventura Rivers*

| Stream | Uncontrolled drainage area | | Estimated contribution to littoral supply | |
|---|---|---|---|---|
| | Prior to 1948 | After 1948 | Prior to 1948 | After 1948 |
| | *Sq. mi.* | *Sq. mi.* | *Cu. yd.* | *Cu. yd.* |
| Ventura River | 131 | 131 | 180,000 | 105,000 |
| Matilijia Creek | 56 | ..... | 82,000 | ..... |
| Coyote Creek | 41 | ..... | 50,000 | ..... |
| Santa Clara River | 845 | 845 | 680,000 | 395,000 |
| Piru Creek | 440 | ..... | 335,000 | ..... |
| Sespe Creek | 270 | 270 | 330,000 | 190,000 |
| Santa Paula Creek | 50 | 50 | 45,000 | 26,000 |
| Total | 1,833 | 1,296 | 1,702,000 | 716,000 |

### *Discussion*

Thus with a gradual reduction of sediment supply to the coast due to upland development, there could be expected erosion of the coastal sector in the Ventura area. Reduction in sediment supply from the smaller sediment contributing river (Ventura), located northwest of Ventura, amounted to about 200,000 cubic yards per year. Since the dominant littoral transport is to the southeast, it was not surprising to find the annual erosion of the coast in the Ventura area amounted to a little less than this figure. Along the 2 miles of shore in the Pierpont Beach area the mean high water shoreline receded landward an average of 200 feet over the period 1948–61. Onshore installations, which previously had sufficient beach width fronting them to provide protection against severe wave attack, were gradually becoming vulnerable to damage by storm waves.

In evaluating plans of improvement for the problem area, it had to be assumed that the sediment contribution to the shore by the Ventura and Santa Clara Rivers would not increase in future years but, in fact, would probably be further reduced. Inland sources of suitable sand supply (by dry dryland haul) in this area are limited, thus auxiliary structures for the purpose of minimizing the annual losses of any material placed along the shore by mechanical means had to be considered. Considering all data and factors pertinent to the problem, a plan of protection was developed that involved construction of 9 groins and placement of 1,534,200 cubic yards of restoration fill along the Pierpont Beach sector. Required periodic nourishment for the restored beach was estimated to be 20,000 cubic yards per year. The recommended plan is shown on figure 1. Inland areas were located that had sand of suitable size characteristics and that contained sufficient volume to provide the initial fill for restoring the beach and future nourishment requirements.

The plan of protection is presently in the first phase of construction, which involves construction of three groins in the central and southerly sectors of the problem area and provides sand fill between the groins.

## *Northern Orange County, Calif.*

### *General*

The northwesterly limit of the Orange County shoreline is at the mouth of the San Gabriel River. The mouth of the Los Angeles River and the Los Angeles-Long Beach Harbor complex is about 4 miles upcoast from the San Gabriel River. The mouth of the Santa Ana River is located about 12 miles southeast of the San Gabriel River. Location of the area is shown on figure 3. These rivers are typical of southern California, as they transport large quantities of sediments to the coast during flood periods and relatively small quantities under normal flow. High flood conditions were experienced in 1938, which contributed large quantities of sand to the coast in the Los Angeles-northern

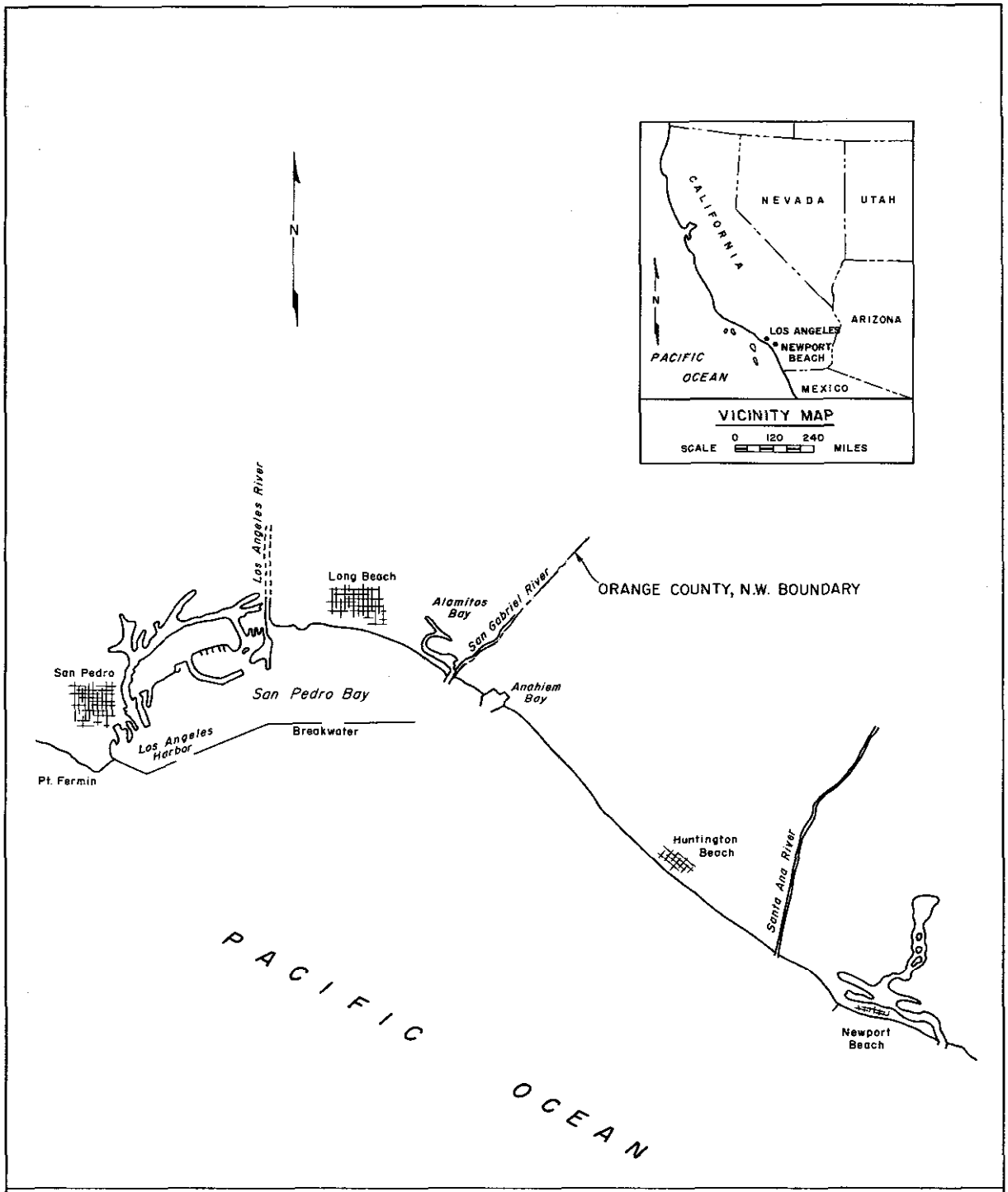


FIGURE 3
LOCALITY MAP

FIGURE 3. — Locality map, Los Angeles and Newport Beach.

Orange County areas. Since that time the upland has undergone such extensive development that should comparable flood conditions prevail, the sediment yield to the coast would undoubtedly be much less than that in 1938. This general aspect has been a factor in the shore erosion problems in the northerly reaches of Orange County, particularly between Newport Beach and Anaheim Bay. Several structures have been built along this shore sector that interrupt the littoral drift and affect shore stability; however, reduction of upland sediment supply has undoubtedly been one of the primary factors causing instability of this shore sector.

*Sedimentation and Other Data* [2 3]

The Los Angeles River outlet works, located about 4 miles north of the Orange County line, are not actually within the shore erosion problem area. However, this river has had a very significant influence on the problem and should be considered before discussing the San Gabriel and Santa Ana Rivers and the problem area. Los Angeles-Long Beach has an interesting history of development [4] as does the ocean outlet of the Los Angeles River.[5] The mouth of the river at one time was located in the area that is now the heart of the harbor complex. Prior to the extensive harbor development the river must have discharged large quantities of sand to the shore. Although waves approaching this shore sector vary in direction and are influenced by offshore islands, the resultant littoral forces produced a dominant southeasterly movement of the littoral drift. The river sand served to nourish the shore for a considerable distance to the southeast.

The origin of the Los Angeles River is in the Santa Susanna and Santa Monica Mountains. Its drainage basin is shown on figure 4. It flows along the south side of the San Fernando Valley, around the easterly terminus of the Santa Monica Mountains, thence across about 22 miles of the Coastal Plain to Long Beach. The watershed of the river makes up an area of about 818 square miles. A channel, trapezoidal in cross section, was completed in 1923 in the Long Beach area, which served to divert the lower reaches of the river and establish the ocean outlet just east of the harbor.

By 1941 the offshore San Pedro-Long Beach breakwater extended southeasterly a sufficient distance to cause dampening of the wave energy reaching the shore in the river outlet area and reduced the effectiveness of the waves to dissipate the large shoal at the mouth of the river. Survey data covering the river mouth and adjacent shore for the period of 1923–38 showed the river discharge contributed over 500,000 cubic yards of material per year. During the 1940's large quantities of material were dredged from the channel through the delta, some being placed on a 4-mile length of beach to the east of the river outlet (about 6,000,000 cu. yd.) and some placed in the harbor area to create land for improvement of harbor facilities (about 8,200,000 cu. yd.). Since 1938 this river basin has been highly developed, including extensive urban and industrial development, check dams, and many channel stabilizing measures. At present the quantity of sediment arriving at the river mouth is a very small fraction of that supplied prior to 1940. The offshore breakwater and deepened channel at the mouth of the river greatly affect natural littoral transport of the river sediments to the shores east thereof; however, these factors are of little importance to alongshore material transport if the small quantity of sand now supplied by the river discharge is considered. It is apparent that substantial adjustments, principally erosion, could be expected for the shores southeasterly of Long Beach when the large upland supply of materials to the coast through the Los Angeles River was virtually eliminated. This problem for the shore sector from the Los Angeles River to the San Gabriel River has been minimized and, in fact, the area is fairly stable, due to the southeasterly extension of the Los Angeles-Long Beach offshore breakwater system and fill placed along the shore (i.e., the 6 million cubic yards mentioned above and recent material placement from a harbor development in Alamitos Bay).

As far as is known, there are no available data on the quantity of sediment discharge to the coast from the San Gabriel River. The drainage basin of this river is shown on figure 4. It has an area of about 700 square miles. The development of works in the upper reaches of the river has virtually eliminated sand supply to the lower or coastal plain reach of the river. Urban development of areas adjacent to the lower reaches of the river and also the composition of the flat alluvial plain further reduces the potential of much sand supply to the shore. Present shoaling at the San Gabriel ocean outlet works is indicated to be from littoral processes, and there is little evidence of material

[2] Data from "Beach erosion control report on cooperative study of Orange County, California," Appendix V, Phase 2. U.S. Army Engineer District, Los Angeles, Calif., March 1962; House Doc. 602, 87 Cong., 2d Sess.

[3] HERRON, W. J., JR., and HARRIS, R. L. NEW METHODS OF CONSERVING BEACH SAND. Shore and Beach 30 (1): 34-37. 1962.

[4] MCOUAT, H. W. HISTORY OF LOS ANGELES HARBOR. Coastal Engin. Conf. Proc. 1, Council on Wave Res. Univ. of Calif. 1950.

[5] KENYON, EDGAR C., JR. HISTORY OF OCEAN OUTLETS, LOS ANGELES COUNTY FLOOD CONTROL DISTRICT. Coastal Engin. Conf. Proc. 1, Council on Wave Res., Univ. of Calif. 1950.

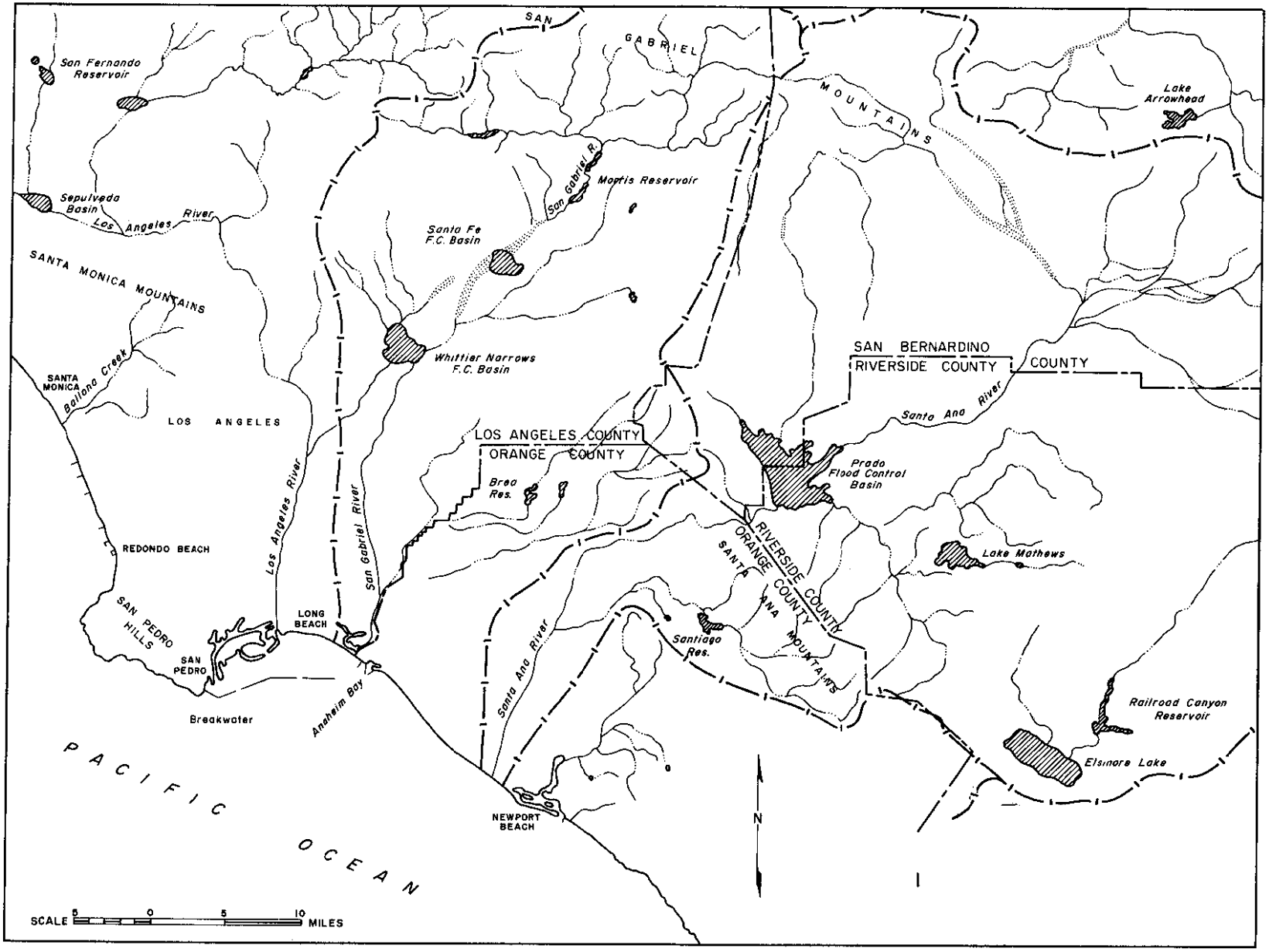


FIGURE 4. — Drainage areas of Los Angeles, San Gabriel, and Santa Ana Rivers, Calif.

supply from the river discharge. It could be reasoned if the San Gabriel River did supply material to the coast, this supply has now been virtually eliminated by the construction and development of works on the upland reaches of the river.

The Anaheim Bay entrance jetties are located about 1 mile east, or downcoast, of the San Gabriel River outlet. Sand fill was placed along the shore between the San Gabriel River and North jetty at Anaheim in 1959, and this shore sector has been fairly well stabilized. The Anaheim Bay entrance jetties were completed in 1944 and, due to their length, effectively interrupt any littoral material movement from the west. The shore immediately east of these jetties receives essentially no littoral supply from the west and as a result very serious erosion occurs. The erosion problem is progressing from the Anaheim Bay east jetty to Newport Beach, a shoreline distance of about 13 miles.

The Santa Ana River mouth is about 11 miles southeast of Anaheim Bay and within the problem area. The drainage basin is shown on figure 4. The basin area is about 2,470 square miles. The river headwaters are in the San Bernardino Mountains. There are no known data on the amount of sediments transported to the coast, but at the present time it is apparently negligible. Upland development has restricted the flow of the river in its lower reaches to the point that very little beach material reaches the shore. Except during the rainy season (November to March) the discharge is insufficient to breach the barrier formed by littoral processes at the mouth of the river.

*Discussion*

Although certain structures built along the shore between San Pedro and Newport have had varying effects on the shore, the main cause of the erosion problems is the reduction of sand supply to the shore from upland river flow. There is very little littoral drift transported around Point Fermin to the Long Beach area as far as it is known. The littoral drift moving from Anaheim Bay to Newport Beach is eventually deposited in the Newport Submarine Canyon located at the entrance to Newport Harbor. Thus, the general picture is one where no alongshore supply enters the system from the west, and the supply of material now existing within the system will be eventually lost at the eastern end. Since essentially all sand supply from upland river flow has been eliminated, shore erosion will continue unless the deficiency of alongshore material supply is made up by some other means.

One of the very important factors in considering various plans for stabilizing the shore between Anaheim Bay and Newport Beach was the limited availability, within reasonable distances, of an inland source of supply of suitable sand for initial beach restoration and periodic nourishment. In this respect, seawall and revetment along the 12½-mile reach of shore was considered but estimates of cost for either of these structures indicated them to be substantially higher as compared to other plans. Stabilizing the shoreline by a seawall or revetment would also not provide adequate widths of beach for recreational purposes. Evaluation was made for a plan involving beach fill and periodic nourishment, with and without a system of groins, and it was found that the initial fill and periodic nourishment requirements exceeded the availability of sand located in all nearby borrow areas. Another plan was considered that involved beach restoration and a series of offshore breakwaters parallel to the shore. Such a system would, in effect, compartment the shore. Each breakwater would impound the southerly moving littoral drift in its lee, and periodically the impounded material would be mechanically transferred back upcoast to a point just below the influence of the next upcoast breakwater. Although this plan would provide a long range solution to the problem, a more economical plan was developed and recommended that involved only one offshore breakwater located at the upcoast limits of Newport Beach and a feeder-type beachfill placed just below Anaheim Bay Harbor. This plan of improvement is shown on figure 5.

The fill, after being transported downcoast and impounded by the breakwater, would periodically be mechanically transferred back upcoast and deposited in the feeder beach zone just below Anaheim Bay Harbor.

Basically, this plan involves the periodic rehandling of the same sand supply. It was estimated that nearby borrow areas contained sufficient quantities of suitable materials to construct the initial feeder beach and to replace normal losses to the offshore zone that would occur between the feeder beach and the breakwater. The initial beach fill below Anaheim Bay Harbor would be about 3 million cubic yards. The downcoast littoral drift rate is about 300,000 cubic yards per year; thus, erosion of the fill zone would be about equal to this rate as well as the impoundment rate behind the offshore breakwater at Newport Beach. The offshore breakwater will provide a sheltered area from which a conventional pipeline dredge may operate. If 5-year increments are considered, this floating plant, along with a series of booster pumps located along the shore, would need to transfer 1,500,000 cubic yards in each operation. The sand-transfer operation could also be accomplished with other types of floating plant

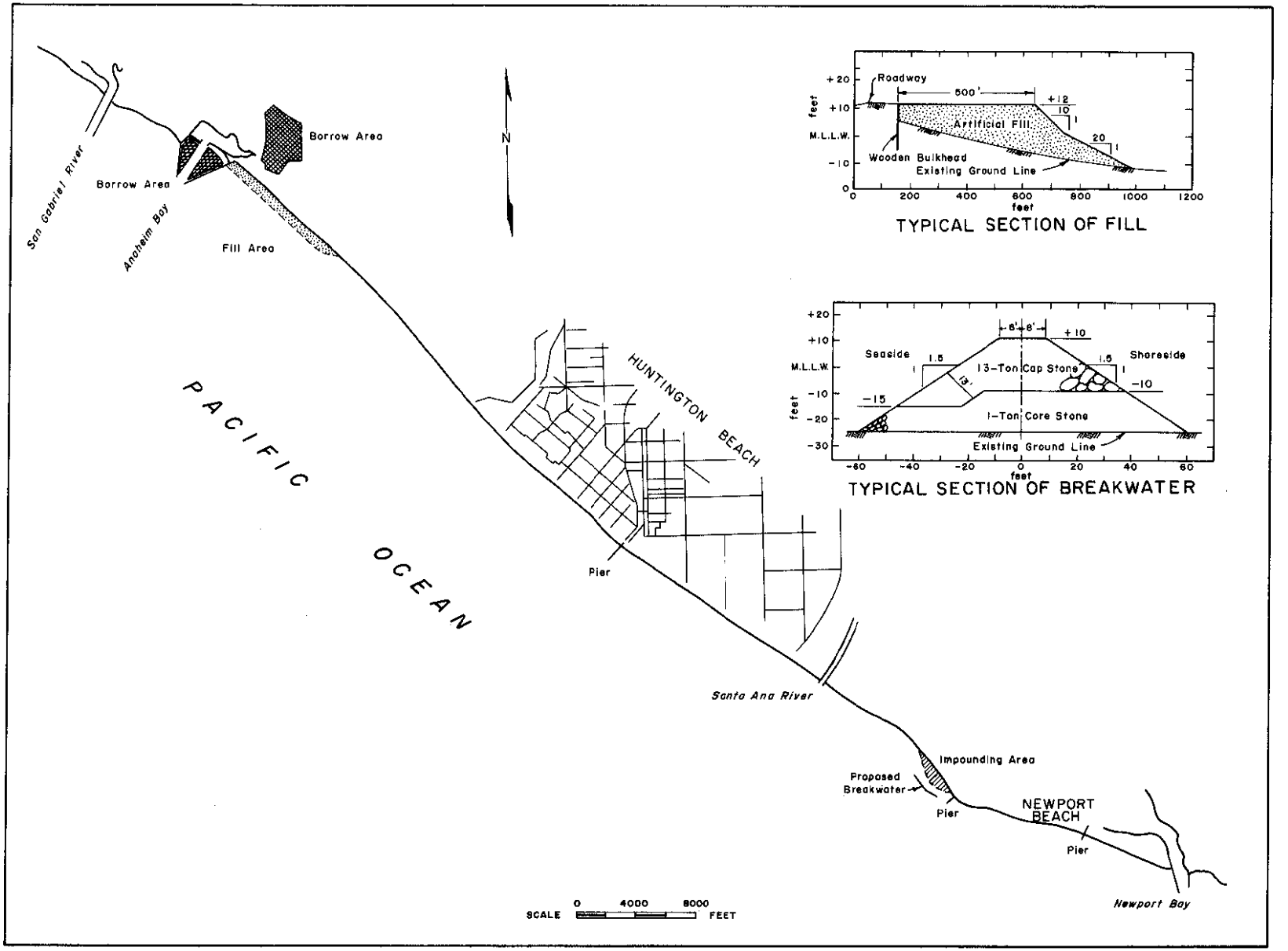


FIGURE 5. — General plan of improvement, Anaheim Bay to Newport Beach, Calif.

such as a hopper dredge with pumpout capability. It was estimated 50,000 cubic yards per year would be lost to the offshore zone out of the system between the feeder beach and the breakwater. Therefore, on a basis of 5-year intervals between pumping operations, 250,000 cubic yards would need to be obtained from other borrow sources and supplemented to the feeder beach zone to maintain complete shore stability for the problem area.

Construction of this plan of improvement has not been initiated as of this date (January 1963). The plan is unique and, when implemented, its functional operation will undoubtedly be carefully viewed by many engineers, as the project will provide valuable guidance on this approach to shore stabilization.

## Concluding Remarks

The improvement and construction of works in drainage basins tributary to a coast, by their very nature, tend to reduce the quantity of sediments reaching the coast. This is particularly true of the sand-size class of sediments which serve to nourish the beach zone. The objective of full basin development is to achieve optimum economic use of all upland resources. The advantages from predicting effects to the coastal zone during the planning stage of basin development are apparent. If detrimental effects will prevail, early corrective measures can be implemented at the coastal zone. Such early action will prevent the shore problem from reaching a critical condition and, in general, will have substantial effect on the economy of the area.

If specific sedimentation data are not available relative to the upland basin development, the engineer must deal with the factors pertinent to the coastal problem and develop his solutions accordingly. He must explore and study all information pertinent to sediment contributed to the coast by upland rivers and streams prior to, during, and after construction of stabilizing works on the upland drainage basin. Detailed criteria and up-to-date techniques for the planning and design of coastal works are presently available;[6] however, such guidance leading to ultimate structural design of coastal works has little value if realistic evaluation is not made of the past, present, and future supply of materials to the littoral zone from not only upland supply but from all sources. The economic justification for constructing certain stabilizing works along a shore are frequently based on a time increment of 50 years and in some cases on a 100-year basis. Therefore, present and projected supply of sediments from upland sources may, and generally will, be a significant factor in the functional and structural design within the plan of improvement for the coastal problem area in question.

[6] U.S. ARMY CORPS OF ENGINEERS, BEACH EROSION BOARD. SHORE PROTECTION PLANNING AND DESIGN. Tech. Rpt. 4, 243 pp. Rev. ed. 1961.

# SAND MOVEMENT ON COASTAL DUNES

[Paper No. 75]

By J. W. JOHNSON, *professor of hydraulic engineering, University of California, Berkeley*

## Introduction

The problems of the supply and loss of sediment at a shoreline are of considerable importance at many localities along the coastlines of the United States. One basic mechanism involved in this overall problem is the transportation of sand by wind action (*5*). For example, at many localities along the Washington, Oregon, and California coasts considerable amounts of sand apparently are moved inland each year by wind action (*3, 8*). A study at the major dune areas of the California coast (*8*) shows that three basic conditions are satisfied for the existence of a dune system: (1) a large supply of sand from a nearby major stream; (2) a shoreline orientation approximately parallel to the crests of the prevailing wave condition, thus creating a favorable condition for a low littoral current and consequently a location for sediment accumulation; and (3) low topography back from the beach where the prevailing onshore winds can easily move the sand inland from the region of accumulation at the beach. Figure 1 is a typical example of a major dune area on the California coast.

To obtain reliable data on the approximate magnitude of the annual supply or loss of sand from a coastline by wind action, a program of laboratory studies was undertaken in a special wind tunnel with sands of various sizes to define the relation between the rate of sand transport and the character of the wind and the sand. This laboratory program was supplemented by a limited program of field studies that was made to define the extent of the dune areas, the character of the dune sand, and the frequency of occurrence of coastal winds capable of moving sand. Thus, to provide information on the character of the sands encountered in actual dune systems and therefore to serve as a basis for the sand sizes that should be

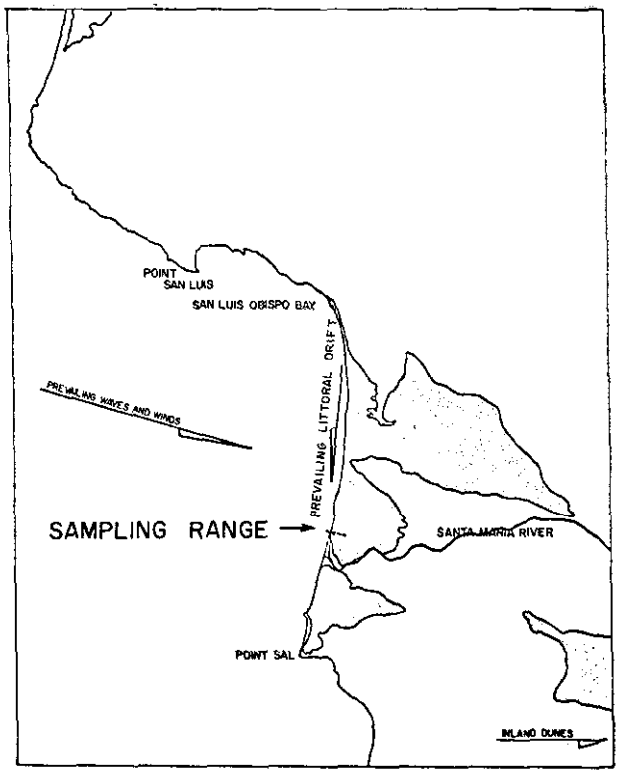


FIGURE 1.—Example of usual shoreline configuration for major dunes on the California coast.

studied in the laboratory, a sampling program was conducted at most of the important dune areas on the California coast. Surface samples were taken at the midtide level on the beach face and on a range extending inland in the direction of the prevailing onshore winds. Typical of such information is that shown in figure 2,

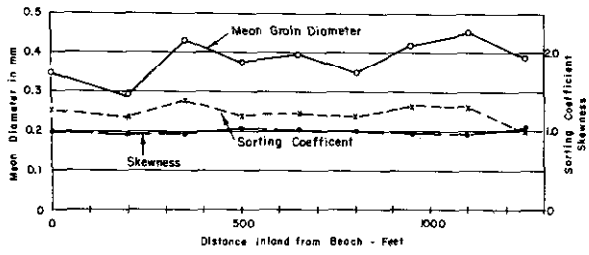


FIGURE 2.—Variation of mechanical composition of dune sand with distance from beach (near Santa Maria River mouth).

which was obtained along the range indicated in figure 1. This plot shows the mean grain size, sorting coefficient, and skewness as a function of distance from the shoreline.

Considerable work has been done on the mechanics of sand movement by wind by numerous investigators (*1, 6, 9*). Emphasis in these past studies was on the distribution of windspeed above the sand surface (with and without sand movement) and the rate of sand movement as a function of the principal variables.

The shear stress, T, produced at the sand surface by airflow, is one of the most important factors in the basic mechanics of sand movement by wind. When the shear stress exceeds a critical value, the sand particles start to move. The windspeed profile and shear velocity, $U_*$, are the primary factors involved in sustaining sand movement — the shear velocity being defined as $\sqrt{T/\rho}$ where $\rho$ is the density of the air. Bagnold (*1*), in his studies of the initiation of sand movement, derived the following expression for the threshold value of the shear velocity,

$$U_{*t} = A\sqrt{\frac{\sigma - \rho}{\rho} gd} \qquad (1)$$

where $d$ is the grain diameter, $g$ is the acceleration of gravity, and $\sigma$ and $\rho$ are the densities of sand and air, respectively. Bagnold found that the coefficient $A$ had a nearly constant value of 0.1 for a sand diameter of 0.25 mm. and greater.

Several investigators have developed expressions for the rate of sand movement as a function of certain variables. A few of these expressions are as follows:

*Bagnold formula* (*1*).—The rate of sand movement per unit width and unit time, $q$, is given by

$$q = C\sqrt{\frac{d}{D}}\,\frac{\rho}{g}\,U^3_* \qquad (2)$$

where $D$ is the grain diameter of a standard 0.25-mm. sand, $d$ is the grain diameter of the sand in question, $\rho$ is the density of the air (in c.g.s. units, $\frac{\rho}{g}$ is equal to $1.25 \times 10^{-6}$), $U_*$ is the shear velocity, and $C$ has the following values:

1.5 for a nearly uniform sand;
1.8 for a naturally graded sand;
2.8 for a sand with a very wide range of grain sizes.

*Kawamura formula* (*6*).—The rate of sand movement, $q$, is given by,

$$q = k\frac{\rho}{g}(U_* - U_{*t})\ (U_* + U_{*t})^2 \qquad (3)$$

where $\rho$ is the density of air, $U_*$ is the shear velocity, $U_{*t}$ is the threshold shear velocity, and $k$ is a constant that must be determined by experiment. For a sand of average grain size, 0.25 mm., Kawamura obtained $k$=2.78 in wind tunnel tests, with all terms being in c.g.s. units.

Experimental results obtained in wind tunnel tests by Bagnold (*1*) and Kawamura (*6*), using

a sand grain diameter of 0.25 mm., present widely differing results as shown in figure 3.

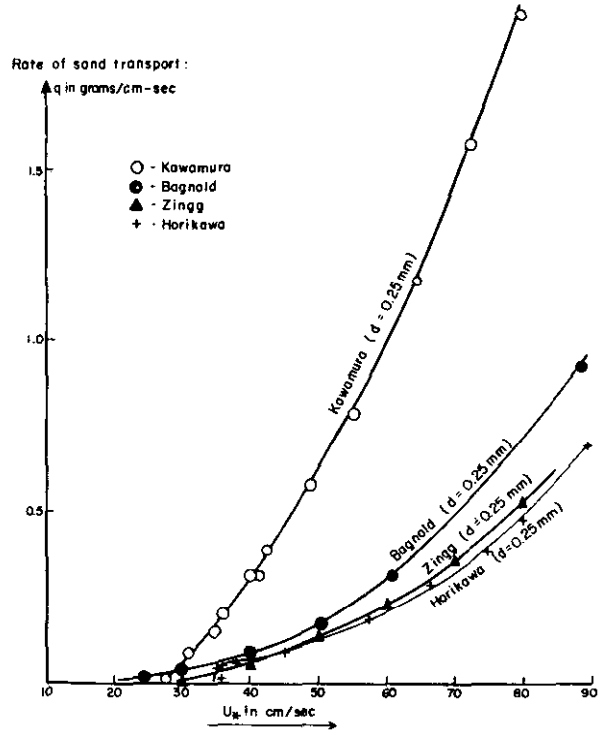


FIGURE 3.—Experimental data on sand transport determined by previous investigators.

Also plotted in this figure are the results of wind tunnel tests by Zingg (*9*) and Horikawa and Shen (*4*) for a grain diameter of 0.25 mm. From his results, Zingg (*9*) modified the Bagnold formula to

$$q = C\left(\frac{d}{D}\right)^{3/4} \frac{\rho}{g} U_*^3$$

where $C$ has the value 0.83.

In addition to the above formulas, O'Brien and Rindlaub (*7*) proposed the following formula from data derived by field tests:

$$G = 0.036\, U_5^3 \text{ (for } U_5 > 20 \text{ ft./sec.)}$$

where $G$ is the rate of transport in pounds per day per foot width, and $U_5$ is the wind velocity 5 ft. above the sand surface in feet per second.

Confirmation of the above formulas by field results is not particularly good, but since there is considerable scatter in the experimental data, these formulas are still useful in the description of a particular condition when a suitable constant is selected.

## Experimental Program

In order to reconcile some of the apparent differences in the various existing relationships for the rate of sand movement and to consider those conditions usually found in the field, a program of laboratory studies was made at the University of California under sponsorship of the Beach Erosion Board. The first tests in this program were made to develop a suitable sand trap for measuring the rate of sand transport in the field. These studies have been reported elsewhere (*4*) and are not summarized in this paper. Subsequent to these earlier tests, which were conducted in a wind tunnel 12 in. by 15 in. in cross section, a larger and more suitable tunnel 4 ft. wide by 2.5 ft. high and 101.5 ft. long (fig. 4) was constructed and used in the laboratory tests discussed below (*2*).

Wind in the tunnel was generated by a variable speed fan at the exit end. Windspeeds up to a maximum of about 40 ft./sec. could be attained. Vertical and horizontal velocity distributions during tests were made with a standard Prandtl-type pitot tube connected to a previously calibrated Magnehelic gage having a range of one-half inch of water and graduated into 0.02-inch divisions. During all tests, sand was spread over a length of about 62 ft. of the

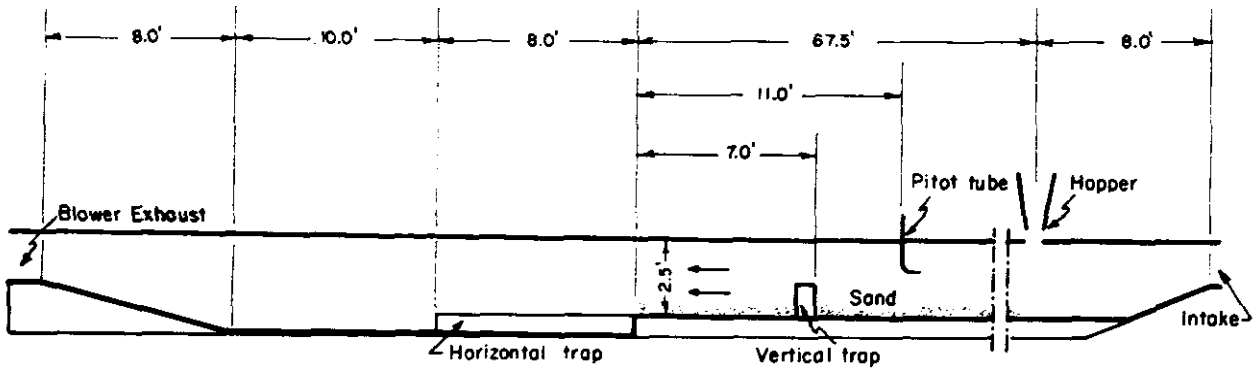


FIGURE 4.—Wind tunnel used in sand transport studies.

flume with a depth of 2 inches. A hopper to feed sand into the flume at a uniform rate was placed near the entrance to the tunnel (fig. 4). The rate of sand feed was adjusted to be equal to the rate of sand transport as measured by the sand traps, which consisted of the vertical trap developed by Horikawa and Shen (*4*) and a horizontal trap. This latter trap consisted of 18 compartments permanently fixed at the end of the sand bed (fig. 4).

In order to eliminate side-wall effects with the horizontal trap, the rate of sand transport was measured only over a width of 2 ft. in the central part of the flume. Sand was removed from the compartments in the horizontal trap at the conclusion of each run with a vacuum cleaner. Comparison of the rate of transport as measured by the vertical trap with that measured with the horizontal trap showed that the vertical trap is close to 100 percent efficient at windspeeds higher than about 27 ft./sec., but it becomes much less efficient at lower speeds (*2*). For high windspeeds, the grain-size distribution of the sand caught in the vertical trap was very close to the grain-size distribution of the bed except for the very largest grains (*2*). The relative absence of these larger grains probably is caused by the platform of the trap, which sometimes can be an obstacle to surface creep.

For windspeeds near the threshold value, the grain-size distribution of sand in the vertical trap shows a distinct lack of the larger grains. This condition cannot be attributed entirely to the inefficiency of this trap but probably indicates the manner in which sand is moving near the threshold; i.e., the larger grains are not in general movement.

The mechanical composition of the three sands used in the tests is shown in figure 5. The principal characteristics of these sands are as follows:

| Sand | Mean grain diameter (mm.) | Sorting coefficient | Skewness |
|---|---|---|---|
| A | 0.44 | 1.23 | 1.01 |
| B | .30 | 1.15 | 1.00 |
| C | .145 | 1.24 | .98 |

## Experimental Results

### *Rate of Sand Transport*

The amount of sand caught by the horizontal trap was measured for velocities varying from the threshold value to about 37 ft./sec. The feeding of sand into the wind tunnel at the upstream end of the sand bed was an important factor in sand movement for the lower wind velocities. Feeding sand into the tunnel during tests appreciably lowers the threshold velocity and, at the same time, changes the amount of sand transported for lower velocities. This effect is evident in figure 6, which shows the rate of transport as a function of wind velocity (and shear velocity) with and without sand

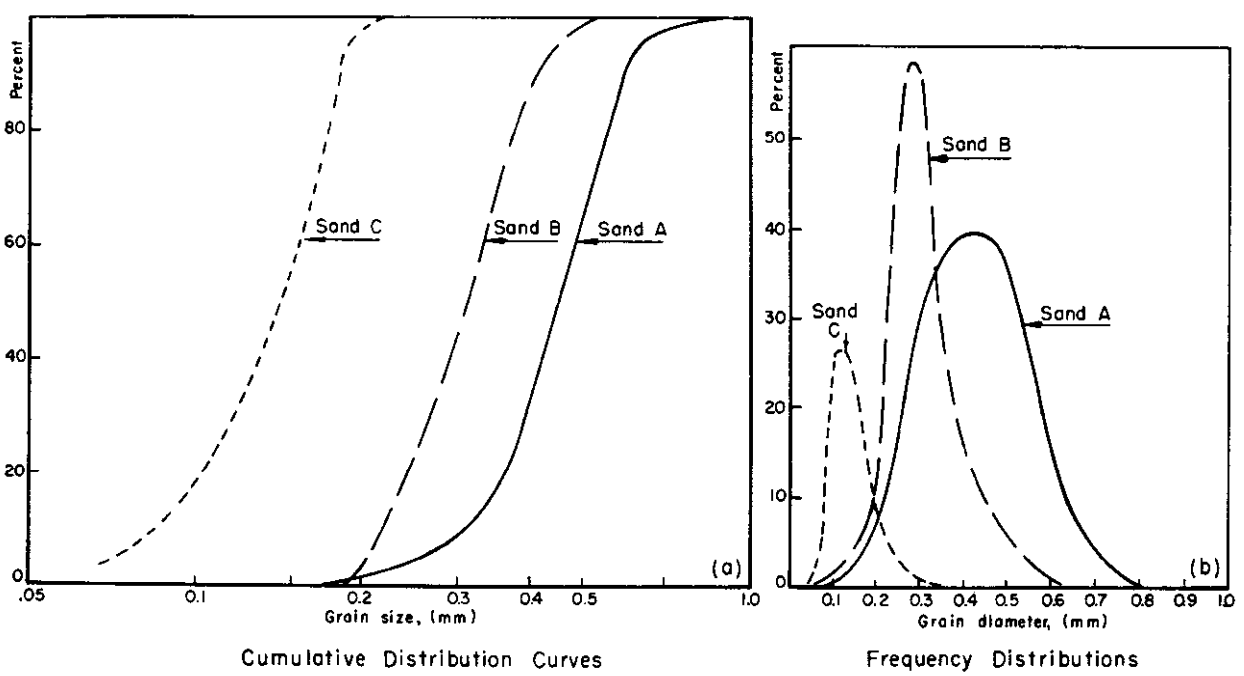


FIGURE 5. — Mechanical composition of sands tested.

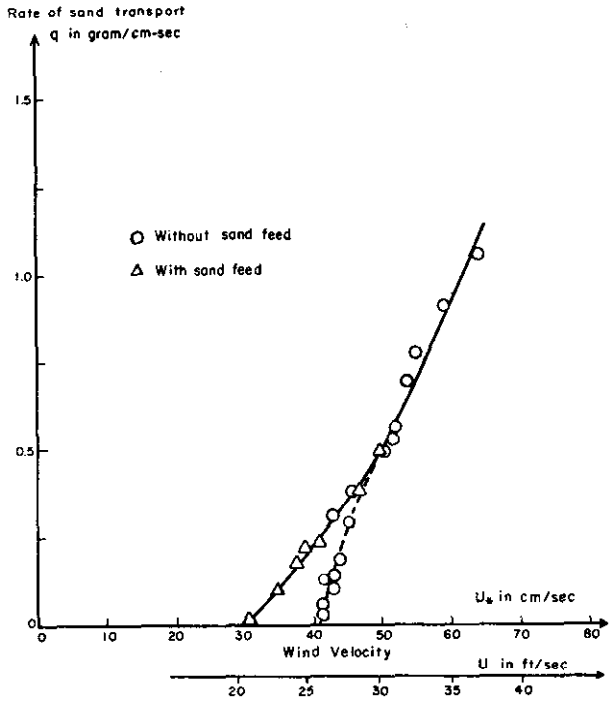


FIGURE 6. — Experimental data on the rate of transport of sand A ($d$ = 0.44 mm.) with and without a sand feed.

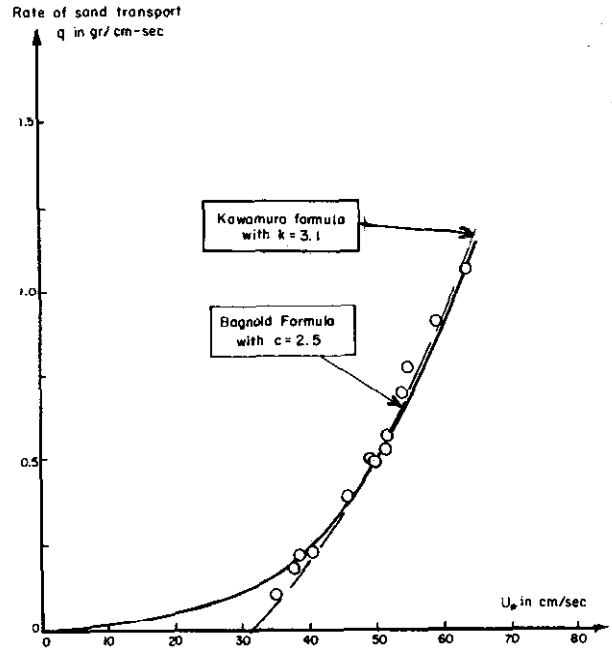


FIGURE 7. — Comparison of experimental data with the Bagnold and Kawamura formulas for sand A ($d$ = 0.44 mm.).

feed for sand A ($d$=0.44 mm.). The apparent reversal of the curve obtained without a sand feed is perhaps due to the fact that the sand used in this study had a relatively wide range of grain sizes (fig. 5). At or near the threshold condition, it is possible that the action of the smaller grains was impeded by the larger grains, thus modifying the overall values for the threshold velocity and the rate of transport. More precisely, near the threshold the sand grains move mainly by saltation. Since the surface layer remains practically immobile (no surface creep), the smaller grains are hidden by the larger ones and as a result the sand behaves as though it had a much larger mean diameter.

The experimental values for the maximum rate of transport (i.e., with a sand feed), $q$, are compared to the values predicted from the Bagnold (*1*) and Kawamura (*6*) formulas as follows.

*Sand A* ($d$=*0.44 mm.*) — The experimental value of the threshold shear velocity (with sand feed) was 30 cm./sec., which compares favorably with a value of 34 cm./sec. as calculated by the Bagnold formula (equation 1). For the rate of transport, Bagnold (equation 2) proposes a value of $C$=1.8 for normally graded sand and $C$=2.8 for a sand with a very wide range of grain size. If a value of $C$=2.5 is used for sand A, the experimental and calculated values are as shown in figure 7. From this figure, except for wind velocities near the threshold, where the Bagnold formula is not applicable, the agreement is good. In the Kawamura formula (equation 3) with a value of $U_{*t}$= 30 cm./sec. combined with a value of $k$=3.1, excellent agreement between experimental and calculated values was obtained over the entire range of shear velocity, as shown in figure 7.

*Sand B* ($d$=*0.33 mm.*). — The experimental value of the threshold shear velocity (with sand feed) was 16 cm./sec., which agrees fairly well with a value of 18 cm./sec. as calculated with Bagnold formula. With respect to rate of transport, figure 8 was prepared to show a comparison of experimental values with the Bagnold and Kawamura data on a sand of 0.25 mm. in diameter. It is evident that the curves differ considerably, although the sand in each case had almost the same mean grain diameter. The sorting coefficient of the various sands possibly was quite different. In the study with sand A ($d$=0.44 mm.) the Bagnold and Kawamura formulas could be used to describe the experimental data; however, with sand B no values for the constants $C$ and $k$ could be found to describe the rate of transport over the entire range of shear

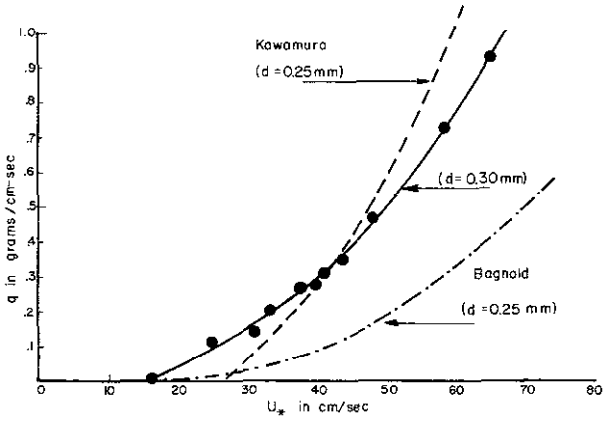


FIGURE 8. — Comparison of experimental data on sand B ($d$ = 0.30 mm.) with the experiments of Bagnold and Kawamura on a sand 0.25 mm. in diameter.

velocities. Those values that most closely described the experimental data are shown in figure 9.

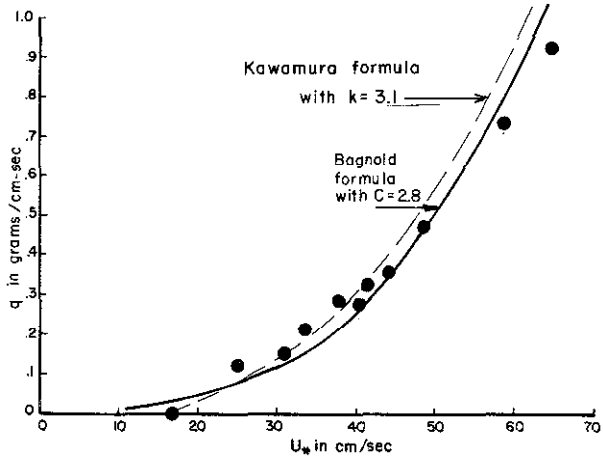


FIGURE 9. — Comparison of experimental data with Bagnold and Kawamura formulas for sand B ($d$ = 0.30 mm.).

*Sand C* ($d$=*0.145 mm.*). — The experimental value of the threshold shear velocity (with sand feed) was 22.0 cm./sec. and that calculated by the Bagnold formula was 17.8 cm./sec. This relatively large difference between the observed and calculated values of the threshold velocity is expected from the work of Bagnold (*1*), wherein it was shown that for such small grain sizes as sand $C$ the coefficient $A$ is not a constant and the shear velocity reaches a minimum. For the rate of transport, figure 10 shows the experimental values compared with the Bagnold and Kawamura formulas. In this comparison, a value of $C$=1.5 was used in the Bagnold formula. This is the lower limit of the value of $C$ recommended by Bagnold and applies to a nearly uniform grain size. For the Kawamura formula, a value of $k$=1.0 had to be selected in order that the calculated curve would pass near the experimental points.

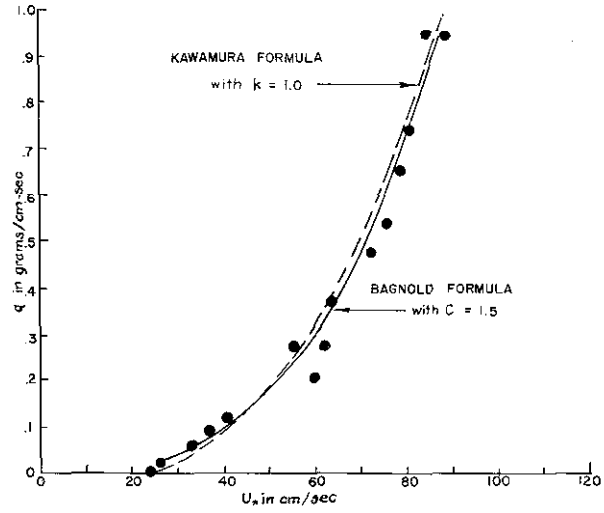


FIGURE 10. — Comparison of experimental data with Bagnold and Kawamura formulas for sand C ($d$ = 0.145 mm.).

### *Response Time of Sand Bed to Change in Windspeed*

In order to investigate the response time of the bed to a change of windspeed, some tests were made in which the wind was first allowed to blow over the sand surface (using sand A) at a velocity of 31.5 ft./sec. for a relatively long time (sufficiently long to observe a duplication of the results on the amount of sand transported, both in the vertical and horizontal traps). The windspeed then was suddenly changed to a value of 35 ft./sec. and the sand collected in the vertical trap was weighed every 2 minutes, until a new constant rate of transport was observed. The windspeed then was again adjusted to the first value, while the same measurements were made with the vertical sand trap. After a sufficiently long time, the windspeed was again adjusted to the higher value, and the same process repeated.

There was some scatter in the data, especially for the higher velocity. This dispersion probably was due to some extent to the inaccuracy of the windspeed readings, these speeds being slightly different in corresponding runs. But as the important fact is the rate of transport with respect to time, the part of the dispersion due to differences in the mean windspeed can be eliminated by considering the discrepancy between the measurements made within the first

few minutes of each run and the average of the last measurements, when the equilibrium is reached. Figure 11 shows the rate of sand transport as a function of time with this correction.

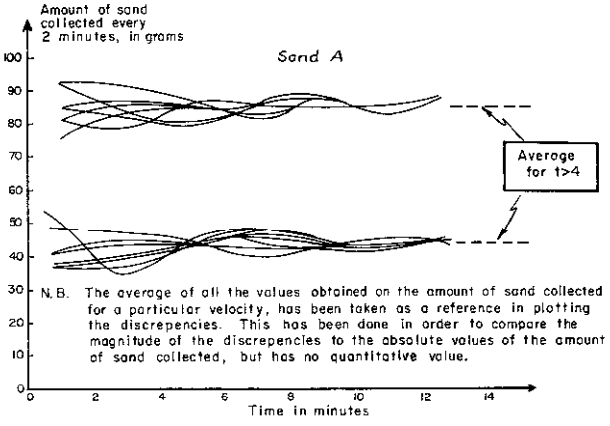


FIGURE 11. — Change in rate of transport with time following a change in wind velocity.

The dispersion was greater at the beginning than at the end of a run, but except for a slight increase during the first 4 minutes in most of the runs, no trend could be observed. In general it appears that the rate of transport adjusts itself rather quickly to a new windspeed. Consequently, under field conditions a knowledge of the actual duration of windspeeds of various magnitudes apparently should permit the calculation of the probable volume of sand moved on a given dune system in a given period of time.

### *Effect of Moisture on Sand Transport*

Generally moist sand found along coasts can be classified into two categories, depending upon the origin of the moisture:

(1) *Moist sand that has collected moisture from the atmosphere.* — Unlike very fine particles such as dust or loess, the sand does not appear to absorb moisture readily. As indicated below, however, there exists a correlation between the water content of the sand and air humidity. Since there is such a correlation and since in the field it is easier to measure the air humidity than the sand water content, the air humidity instead of the water content of the sand was taken as the variable in the studies discussed below.

(2) *Moist sand whose water comes from sources other than air humidity.* — Such sources are rain, rising of underground water, and sea water remaining in the sand by wave or tide action. When a wind not saturated with water vapor blows over such a sand, it gradually dries out the surface layer of the sand bed until an equilibrium is reached between humidity of the wind and the water content of the surface layer of the ground. The study of sand movement is relatively complex in this case, where water contents of air and sand, as well as wind duration, are factors.

To obtain information on some of the above problems, Belly (*2*) performed a series of experiments on sand A ($d$=0.44 mm.) in a closed-circuit wind tunnel where humidity could be controlled. The first series of tests were concerned with the determination of the effect on the threshold velocity where the moisture content of the sand was relatively low. In these tests, $U_*$ was determined from a series of vertical velocity distribution curves for different mean velocities and different air humidities, $h$. Figure 12 shows the relation between the shear

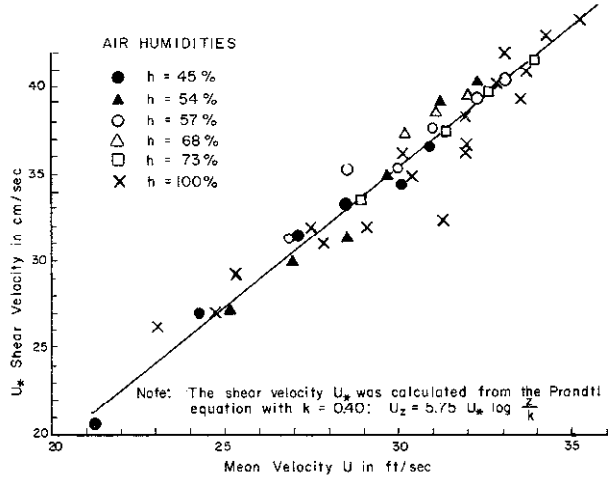


FIGURE 12. — Relation between mean velocity and shear velocity for various air humidities.

velocity, $U_*$, and the mean velocity, $U$. This relation is quite consistent and can be expressed by a straight line. Thus, it can be stated that the wind drag is practically uninfluenced by air humidity. In the later tests, only the value of the mean velocity was recorded and the value of $U_*$ was calculated from this graph.

The water content of sand obtained during the various runs is plotted against the corresponding air humidity in figure 13. The scatter of data is important, but there is a general tendency for the points to follow along a straight line (which obviously should pass through the origin).

The threshold velocity was found by investigating the lower part of the curves for the rate of sand transport. The three sample curves in figure 14 clearly indicate the change in the rate of sand transport with humidity. From such curves, the data on the variation of the threshold shear velocity distinctly show an increase

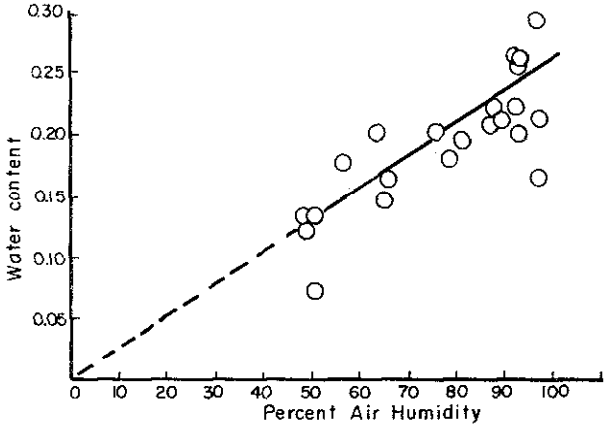


FIGURE 13. — Relation between water content of sand and air humidity.

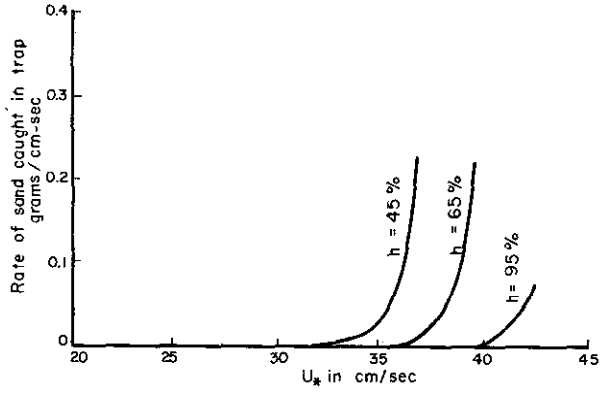


FIGURE 14. — Variation of rate of sand transport with air humidity.

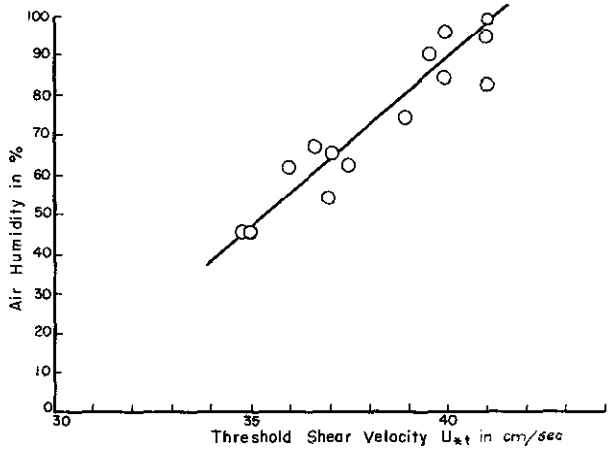


FIGURE 15. — Relation between threshold shear velocity and air humidity (sand A).

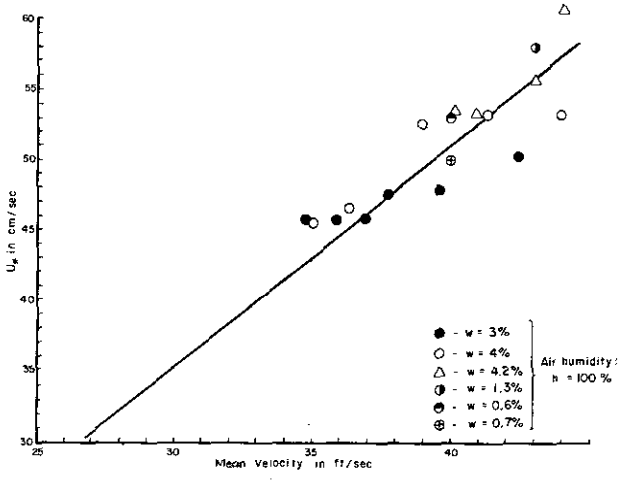


FIGURE 16.—Relation between shear velocity and mean velocity for high water contents.

with air humidity, and the relationship is nearly linear (fig. 15).

The second series of tests by Belly (*2*) on threshold velocity were concerned with sand of high water content; that is, greater than 0.25 percent. The relation between $U$ and $U_*$ under such conditions is shown in figure 16 and appears to be linear and merely an extension of the data shown in figure 12. During the tests in which these data were obtained, the air was maintained saturated, and the water content, $w$, of the sand was varied.

The tests showed that with a relatively high water content of the sand (in excess of 1 percent), the windspeed necessary to initiate sand movement becomes increasingly important. One explanation for this increase is that the sand surface becomes very smooth under wind action. The water contained in the sand fills the interstices between the grains and makes the extraction of the grains by the wind much more difficult than when dry. The experiment could not be made for a water content higher than 4 percent because the wind strength necessary to initiate the movement could not be obtained with the available equipment; however, it might be expected that the wind strength would increase very rapidly with an increase of water content. It is even probable that for a very high water content (flooded sand) the problem changes aspect and becomes closer to the problem of an interface between two fluids.

Using the relationship between water content and air humidity in figure 13, it is possible to complete the results of the study of low water content and thus find a relationship (fig. 17) between $U_{*t}$ and $w$ for the total range of water content (0 to 4 percent). As to be expected, since the air humidity in itself does not play an important role in the sand movement, there is no break at the point that joins the two sets of

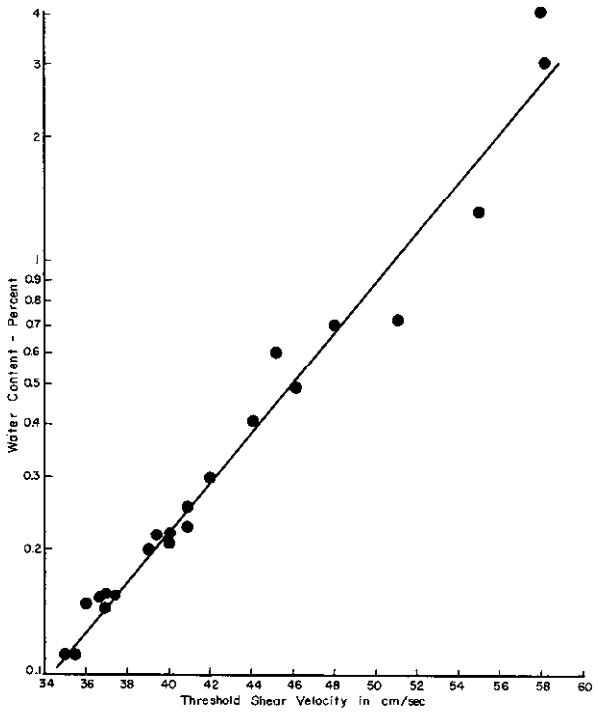


FIGURE 17.—Variation of the threshold shear velocity with moisture content (from figs. 12 and 16).

data; that is, at a threshold shear velocity of 40 cm./sec. The data follow a straight line, thus indicating that the relation between $U_{*t}$ and $w$ can be put into the form,

$$U_{*t} = a \log_{10} w + b$$

where $a$ and $b$ are two constants obtained from the graph. It was found that

$$U_{*t}=17\ (\log_{10} w+5.1)\ \text{cm./sec.}$$

or $$U_{*t}=28\ (0.6 \log_{10} w+1.8)\ \text{cm./sec.}$$

where 28 is the value of $U_{*t}$ in centimeters per second given by the Bagnold formula.

If it is assumed that moisture affects the movement of sand of different grain sizes in the same manner, the Bagnold formula for the threshold shear velocity may be modified as follows:

$$U_{*t}=A\sqrt{\frac{\sigma-\rho}{\rho} g\, d}\ (1.8+0.6 \log_{10} w)$$

where $A$ approximates 0.1 and $w$ is expressed in percent.

## Conclusions

1. The Bagnold formula for the threshold shear velocity appears adequate in giving a value for this term for use in calculating the rate of transport of sands normally existing on the coastal dunes of California.

2. The Bagnold formula for calculating the rate of transport appears superior to the Kawamura formula in that the value of the coefficient $C$ in the Bagnold formula is better defined and more limited in range than the coefficient $k$ in the Kawamura formula. That is, a value of $C$ between 2.5 and 2.8 was adequate to define the transport of sand with a wide range of grain sizes (sands $A$ and $B$) and a value of 1.5 was adequate for a sand that is nearly uniform in size (sand $C$). The Kawamura formula includes the threshold shear velocity, which introduces a further uncertainty in the calculations of transport rates, especially since this factor is influenced by the moisture content of the sand.

3. With a relatively high water content of the sand the windspeed necessary to initiate sand movement becomes increasingly important. This is especially true for the coastal dunes of California where the higher windspeeds, which usually occur during storms, are also accompanied by considerable rain.

## Acknowledgments

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# QUANTITATIVE TRACING OF LITTORAL DRIFT

[Paper No. 76]

By PER BRUUN, *engineer, Coastal Engineering Laboratory, University of Florida, Gainesville*


## Abstract

The longshore littoral drift is caused by a combination of shear stresses by wave action that break loose the material from the bottom, usually pushing it in the direction of wave propagation, and by longshore currents, which help tear loose the material and transport it longshore.

In order to gain an understanding of all the littoral drift phases, it must be split up in several parts to be investigated separately. Some of these represent a combination of elements, which have to be looked into independently. Other parts may present an integration of single elements, making possible investigations in integrated form.

The Coastal Engineering Laboratory of the University of Florida is at present carrying out a combined field and laboratory research program on friction factors for longshore currents and quantity of longshore drift. The laboratory utilizes tracing technique by luminophores in the field. Wave tank experiments are carried out simultaneously. Longshore currents are passed through waters agitated by waves propagating parallel to or perpendicular to the current action.


## Field Research

### *General*

Sediment transportation is caused by a combination of shear stresses by wave action and by current action. The wave-induced shear stresses "break loose" the material from the bottom while the longshore currents transport it longshore.

Figure 1 demonstrates a typical situation

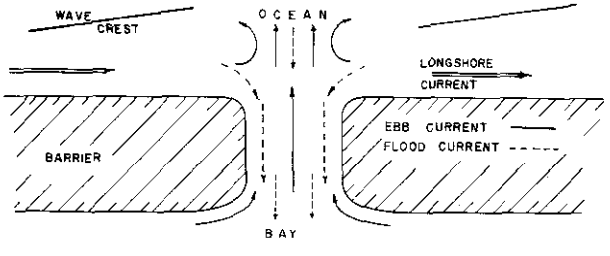


FIGURE 1. — Typical current action of barrier and inlet shore.

from a seashore like the U.S. east and gulf barrier-lined coasts. Wave action causes a longshore current that will be strongest in the breaker and uprush zones. This current will be interrupted on the unbroken barrier shore by rip currents and on the inlet broken shore by tidal inlet currents (fig. 1). This means that we are normally dealing with wave and current action perpendicular to each other. Occasionally, at inlet entrances, they run parallel to each other, either in the same or in opposite directions.

A research program on littoral drift must include the above-mentioned overall problems. The procedures described below are based on the philosophy that, although it is not possible to understand the complex problems in full unless research penetrates down in the very details (*7*), it is still possible to obtain results of scientific and practical value by considering an integrated part of the problem, including a variety of details to be looked into when the integrated problem has been investigated or analyzed to the extent this is possible.

### *Modes of Transport*

As in rivers littoral material transport occurs partly as bedload and partly as suspended load transport. It is known that bedload transport takes place in two basically different ways — partly in a fast moving sheet layer on the surface of the bottom and partly in bulk movements as slowly migrating waves on the bottom, perhaps in a multiwave system as observed in the Mississippi River where whole systems of waves advance by an overhauling procedure (*13*). Most likely the situation on the seashore is similar to the situation in rivers as mentioned in a later paragraph.

#### *Sheet-Layer Movement*

The thickness of the sheet layer depends upon the shear stresses and lift forces exerted upon the bottom and their variation with time. Under the influence of these stresses, the sand particles travel parallel to the wave crests (longshore) when the oscillating water particles are mainly in vertical movement and they migrate perpendicular to the wave crests (and to the shore) below wave crests and troughs. The thickness of the sheet layer as observed by some researchers may correspond to the height of the ripples as long as they occur on the bottom. Meanwhile, with higher water velocities the ripples are wiped out and replaced by a flat, slightly undulated surface, which carries a considerably higher bedload movement and probably increasing thickness of the "live" surface layer. Certain field and laboratory tests on sheet-layer movement are now in progress at the Coastal Engineering Laboratory of the Uni-

versity of Florida. In the field quantitative tracing is undertaken with luminescent tracers. A brief review of the manufacturing process and the proposed scanning of samples by Teleki is given in paper No. 76A. Experiments in which fluorescent tracers are used are mentioned in references (*9*, *10*, *17*, *23*, *24*, and *25*). Examples of radioactive tracing are given in references (*18* and *20*) and in papers by L. L. McDowell and by Cummins and Ingram in this Proceedings. Kamel and Johnson (*12*) mention natural tracers.

With respect to sheet-layer movement, the following procedure is used at experiments in progress at the Fernandina Beach, northeastern Florida.

Tracer material is dumped by diver or from helicopter on the bottom on the updrift side of an 800-ft. long pier extending in the ocean to a depth of 15 to 20 ft. at mean sea level (fig. 2). Suspended load samplers as well as bedload samplers are operated from the pier at various depths. The suspended load sampler is a pump with a suction pipe having four vertical intake pipes and settling basins on the pier. The bedload sampler is also supposed to function as a trap. It is a turtle-shaped steel and plastic dome about 3 ft. long, 2 ft. wide, 8 in. high, with remote-controlled doors at each end (fig. 3). To avoid excessive scour around the trap and its openings a heavy steel apron is attached to the bottom of the trap. The proper function of the trap is checked by the diver. Placed parallel to shore, the trap is operated for a few minutes, after which doors are closed and water jets circulate the water in the trap at the same time as a vertically slitted pipe in the middle sucks up the material accumulated in the middle for deposit in basin on the pier.

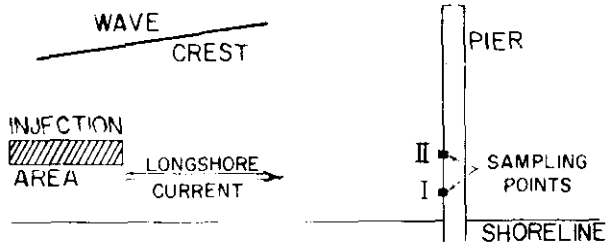


FIGURE 2.—Location of tracer material in relation to Fernandina Beach pier, northwestern Florida.

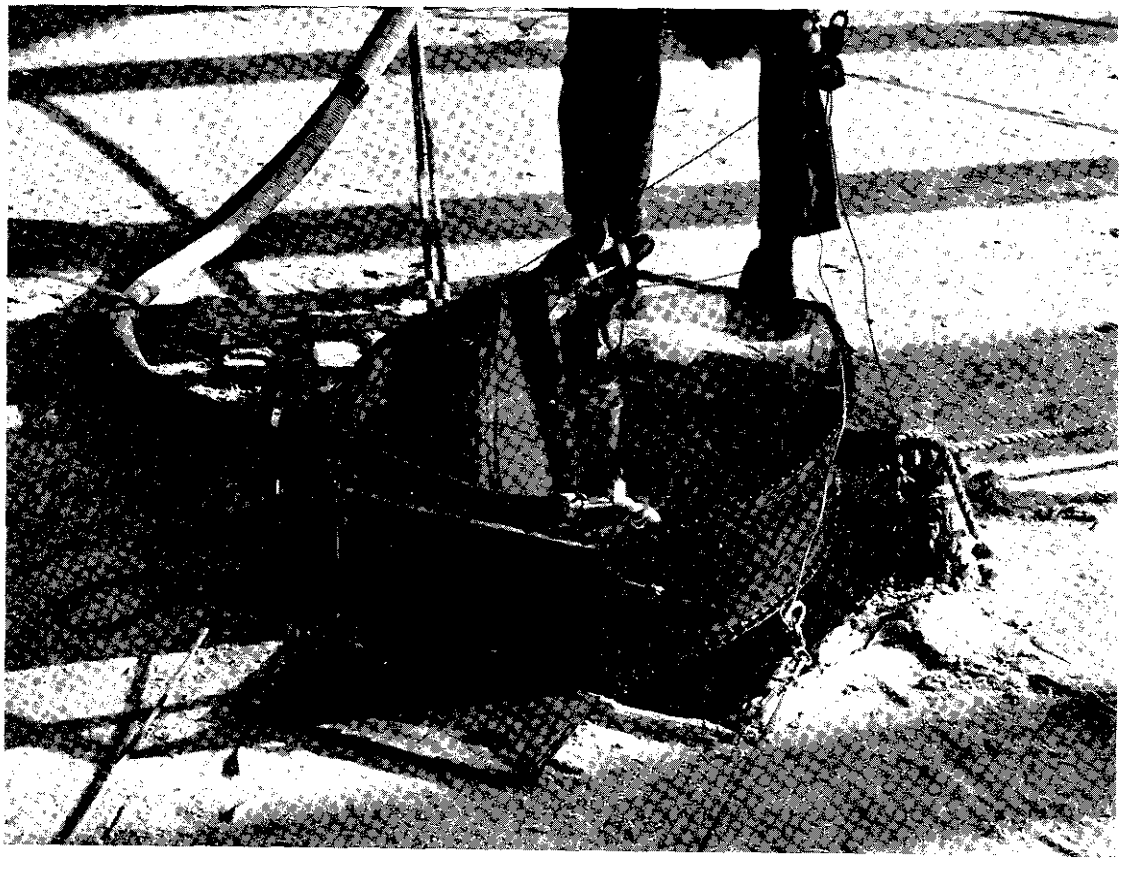

FIGURE 3.—Bedload trap.

By continuous sampling of material, the number of grains/time relationship is obtained. If $n$ tracer grains are trapped per unit time, one has $v_x$=travel velocity of grains, $b$=thickness of sheet layer, $w$=width of trap opening, and $k_1$ a material constant

$$w \cdot b \cdot v_x = n \cdot k_1$$

$$b = \frac{nk_1}{wv_x} = k_2 \frac{n}{v_x}\text{; where } k_2 = \frac{k_1}{w} \qquad (1)$$

"$b$" indicates the thickness of the imaginary layer of coated grains and is very small. The corresponding number of noncoated grains coming from the same bottom area as the coated grains is unknown. Most grains trapped in the period immediately after the dumping of tracer grains come from the area close to the pier.

If relatively heavy concentrations are assumed, coated grains will arrive at the pier in a stream that most likely will increase rather rapidly to a maximum value and then fade out slowly (fig. 4). When point A of figure 4, indicating maximum concentration, has reached the pier, discharge of material from the injection area has its maximum value. The corresponding maximum thickness of material arriving from the injection area is

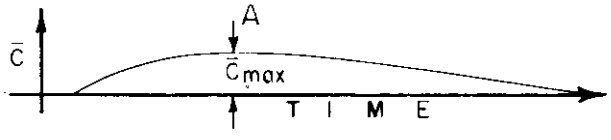


FIGURE 4. — Theoretical relationship of concentration of tracer material versus time.

$$B_{max} = \frac{N}{n_{max}} \cdot \frac{n_{max}}{Vn_{max}} \cdot k_2 \qquad (2)$$

when $N$ is the total number of grains trapped per unit time.

Next the assumption is made that the ratio $\frac{N}{n_{max}}$ can be considered as a characteristic all-time valid figure for the release of material from the injection area and is true for all the various sheet layers sliding on top of each other with different velocities.

The use of $\frac{N}{n_{max}}$ as "standard ratio" assumes that coated and noncoated grains leaving the injection area have the same fate. This means that full natural mixing has taken place, eliminating all irregularities that would tend to create fluctuations in concentrations at the sampling point. Tracer material, therefore, should be released in a certain area of the bottom parallel to shore, thereby eliminating the influence of smaller migrating sand waves. It may not be possible to eliminate the influence of the very large waves.

It still cannot be expected that maximum concentration will occur in one distinct peak of concentration. Therefore, as an example the $\frac{N}{n_{max}}$ ratio should be determined as the average of 5 maximum concentration samples:

$$\left(\frac{N}{n_{max}}\right)_{ave.} = \frac{1}{5} \sum_{1\ \max n\ \text{value}}^{5\ \max n\ \text{values}} N/n_{max}$$

The total thickness of the fast moving sheet layer up to maximum concentration may then be found by an integration:

$$B_{total} = \Sigma\, b = \sum_{n=n_1}^{n=n_{max}=n_j} N/n_{max}\ k_2 \frac{n_j}{V_j} \qquad (3)$$

The total quantity of material ($Q_{total}$) in all sheet layers may be found by direct observation of the quantity trapped. The integral below includes fast- as well as slow-moving material.

$$Q_{total} = Q_{fast} + (Q_{slow}) = \sum_{n=n_1}^{n=n_{max}=n_j} N/n_{max}\ k_2 \frac{n_j}{V_j} \cdot V_j + (Q_{slow}) \qquad (4)$$

The thickness and migration velocity of the slower and very slow moving bottom material may be determined by continued long-range observations beyond maximum concentration and may in the extreme include the migrating humps of material that has erosion slopes updrift and deposit slopes downdrift.

Figure 5 shows one of the results of introductory tests at Fernandina Beach pier, where a trap was used as shown in figure 3. When figures 4 and 5 are compared, this result is encouraging. To obtain reliable results, it is

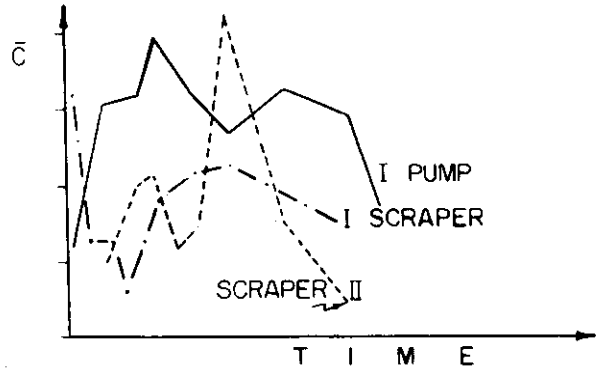


FIGURE 5. — Results of a test of concentration of tracer material versus time.

necessary to inject various color schemes in strips parallel to shore.

The $N/n$ ratio is independent of the direction of travel of material, but the trap used to pick up material and measure quantity of longshore drift must assure that the material actually is derived from the longshore movement of sheet layer corresponding to the width of the door of the trap. Placement of a steel apron around the trap prevents erosion of material that would take place at corners, which could interfere with deposits in the trap.

The results of these experiments with bed and suspended load are only valid for the conditions under which the experiments were actually run. In order to gain information useful in practice, a great many results under a variety of conditions are needed. These results must link wave and current characteristics to sediment transportation character. Concerning the hydraulic character, efforts should be intensified to determine more detailed data, such as bottom velocities, their directions, and fluctuations and turbulence in the grand as well as the small scale. The program is therefore not in the least an attempt to determine the detailed flow patterns and velocity characteristics and its relation to the sediment transport. Velocities parallel as well as perpendicular to shore therefore are measured in front of the updrift door by instruments attached to the trap.

### *Bulk Movement of Sand in Migrating Sand Waves*

The ocean floor does not have an even surface when considered on the grand scale. In addition to the longshore bars it is provided with undulations perpendicular to the shoreline. These undulations migrate "downstream" in the way that part of the material creeping up along (or eroded from) the updrift slope comes to rest on the downdrift slope thereby causing a downdrift movement of material. This in turn means that thickness of the surface sheet layer on the updrift side of the undulations is greater than on the downdrift side.

Such undulations have been observed in rivers. Lane and Eden (*13*) state that 350 million tons of sediment is transported annually in the lower Mississippi, mostly in suspension. Meanwhile surveys demonstrated that considerable quantities of bed material migrated along the bottom in large bars with varying heights of 5 to 10 ft. and lengths of 1,000 to 4,000 ft. and a travel velocity of 15 to 25 ft. per day, causing a material movement of 2,000 to 8,000 cu. yd. per day. This is further explained by Carey and Keller (*6*), with special reference to the friction elements involved. Similar phenomena are well known from flume experiments. On the seashore migrating sand waves moving toward the shore are mentioned by Thierry and van der Burgt (*21*) with reference to observations in the English Channel by Hook van Holland and Den Helder.

Bruun (*1*) mentions results of research on this phenomenon on the Danish North Sea coast where large undulations migrate along the shore in the direction of the littoral drift in the area of the beach profile near the shoreline. The "wave-length" seems to be 0.5 to 2 kilometer, the "wave height" (horizontal) 60 to 80 meters. Moreover, detailed depth soundings on the Danish North Sea coast proved the existence of deep-water, large, wave-shaped sand humps that migrated along the shore. The wavelength seemed to be 1.5 to 3 kilometers, the vertical height is 1 to 2 meters.

Similar experiments were carried out and are still in progress on the Atlantic shore at Fernandina Beach and at St. Augustine, Fla. At Fernandina Beach profiles were surveyed up to about 20 ft. depth at regular intervals. Figure 6 shows the results of such a survey made. It

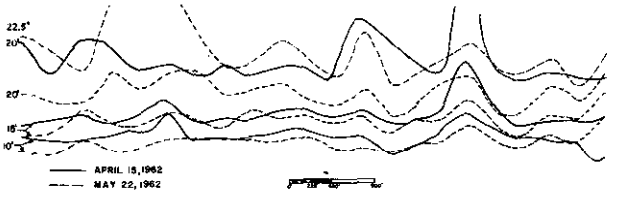


FIGURE 6. — Profiles of migrating sand waves, Fernandina Beach.

appears that the wavelength of undulations in depth contours vary from about 500 to about 1,500 ft. and that the horizontal wave height may be 1,000 ft. or more.

## Laboratory Research

### *Transport by Tidal Flow; No Wave Action*

The material transportation must follow the same physical laws as valid for unidirectional flow even if the velocity varies during the tidal half cycle. Bruun and Gerritsen (*4*) mention rational approaches in relation to tidal hydraulics. A review of "Engineering Aspects of Sediment Transport," (*2*) reports the results of research with unidirectional flow.

### *Transport by Wave Action*

Several flume experiments have been made on transportation of bed material by wave action and on the distribution of velocities, including mass transport velocities. Russell and Osorio (*17*) found by tests in a wave flume that near the sand bottom drift velocities are as predicted by Longuet-Higgins' theory (*15*) for all waves investigated in their tests. Whether the

bottom drift was on a horizontal or a sloping bottom within certain limits of steepness, it was always in the direction of propagation of wave action. So was the direction of the drift in the surface. However, in the middle of the wave flume the drift was seaward, or against the direction of propagation of wave action.

Lhermitte (*14*), in similar experiments considered the formation of ripples and the influence of viscosity in the bottom boundary layers.

It is generally known that the friction parameter depends partly upon the bottom configuration (ripples, dunes, smooth, etc.) and partly upon the movability of the undulating boundary layer. Whether the flow is unidirectional or oscillating, it is difficult to measure water velocity and particle velocity separately, unless the material is very fine (wash load), in which case the sediment and water particle velocities are identical. If the friction factor as related to the bottom configuration and the damping influence on turbulence caused by the fine sediment particles are known, it is possible to use the Prandtl-von Karman velocity distribution law, with corrected constants for unidirectional flow (*8*). But for a coarse sand bottom it is difficult or impossible to arrive at any definite "velocity profile," because of the great irregularities in velocities. Results of model experiments on scours at sluices in Holland proved that the velocity fluctuations is the important parameter, because it gives a measure for the turbulence that is all-important for the combined shear and lift forces which are responsible for material movement.

### *Transport by Tidal Flow and Wave Action Running With or Against Current Action*

Research is in progress in a 4 ft. deep, 6 ft. wide, wave tank (fig. 7), which also functions

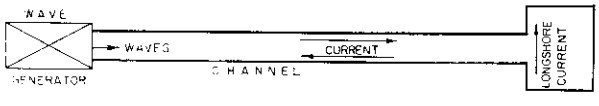


FIGURE 7.—Wave tank for littoral drift experiments.

as a flume. Before wave action and current action are introduced simultaneously in the tank, friction factors as well as sediment transportation are measured for the two independent movements of flow. For larger scale conditions of oscillating water movement an oscillation tank, as described by Bruun and Gerritsen (*4, p. 107*), may be needed.

### *Transport by Currents Perpendicular to Inlet Flow: No Wave Action*

Inman and Bowen (*11*) found, in flume experiments on combined wave and current action propagating in the same direction, that the transport rate of bed material was related to loss of wave energy for all low values of current velocity. Increases of the current velocity beyond a certain value created a complex sediment movement above the ripples, which were periodic but asymmetric; the phase-dependent phenomena then had a greater effect on the system than that of the drift velocity.

### *Transport by Currents Perpendicular to Inlet Flow: Wave Action*

Wave action may be operating under various angles with the current activity. Inasmuch as a current running parallel to shore has to turn about 90° in order to enter the inlet, angles between 0° and 90° occur. The about 90° angle is the situation that we find with normal longshore current activity.

The relative importance of the two sediment-agitating water movements is considered. As long as the longshore current movement is predominant, it may be expected that "wave friction factors" have considerable similarity to friction factors for unidirectional flow.

The longshore current theory by Bruun (*3*), using the principle of continuity, has two approaches. One, "the rip current approach," particularly refers to moderate to lower velocity longshore currents resulting from smaller angles of incidence of wave crests. Examples based on statistical wave distribution give us reason to believe that the actual friction factor may be a combination of normal bottom friction and the hydrodynamic friction factor, depending upon the wave motion.

The other approach is called "the slope approach" and is based on inflow of water from wave breaking under a certain angle, with the shoreline giving rise to longshore "hydraulic slope" of the water table.

Examples for one and multibar profiles (*3*) give reasons to believe that this approach, using corrected experience values of the Chezy factor C, may be valid for the higher velocities resulting from larger waves that break under a rather big angle with the shoreline.

With respect to the littoral drift as bedload, wave motion will tend to increase net transport by increasing the bottom shear stresses. Furthermore, wave motion will tend to create seepage forces into and out of the bed. This causes an added boundary shear. In shallow water, the seepage force is of sufficient magnitude to decrease the effective weight and, hence, the stability of the bed material as much as 20 percent.

Figure 8 shows a cross section of the wave tank depicted in figure 7. As explained above, waves are passed down the tank through the 6-ft. wide section. At the same time a longshore

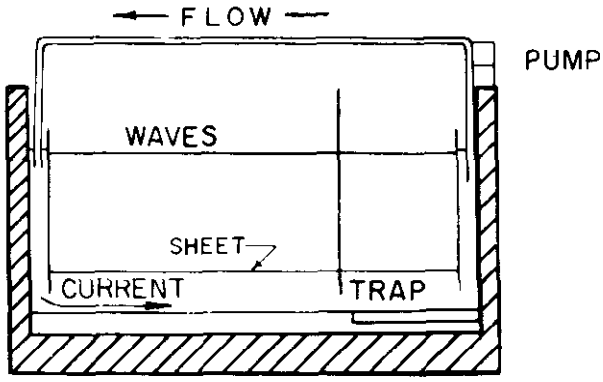


FIGURE 8. — Cross section of wave tank for littoral drift experiments.

current is run through the T-shaped end of the tank along the bottom perpendicular to the direction of wave propagation. To avoid eddy formation and dispersion of sand material from the bottom, a plastic sheet is put in about 20 centimeters above the bottom in the upper water layers of the tank. This sheet does not disturb the general motion of water particles in almost closed paths, but it hinders the free development of turbulence where it is located.

Wave and current characteristics are combined in the way that only one factor is varied while the others are kept constant. The head difference needed to keep the longshore bottom current constant varies with changes in the combined hydrodynamic and bottom roughness.

It is thereby possible to obtain data on friction as well as bedload transport parameters that range over an area from predominant unidirectional flow to predominant wave motion. In case of predominant unidirectional flow, the orbital velocities near the bottom are small compared to the longshore current velocities. For predominant wave motion, the shear stresses exerted on the bottom by the oscillating water determine the friction and transport parameters. Bedload transport is determined by bed traps installed as shown in figure 8. Its relation to wave and current conditions is recorded.

Most likely the results may indicate the following dependencies in friction as well as sediment transport parameters:

*Predominant wave motion* — *Wave and current action*

$$T_w^{n_1} \cdot f(T_w)^{n_2} \quad (T_w \cdot T_{\text{long}})^{\frac{n_1}{2}} \cdot f(T_w)^{\frac{n_2}{2}} \cdot f(T_{\text{long}})^{\frac{n_2}{2}}$$

*Predominant longshore current*

$$T_{\text{long}}^{n_1} \cdot f(T_{\text{long}})^{n_2}$$

in which $T_w$= the wave shear stresses and $T_{\text{long}}$ =the longshore current shear stresses.

## Littoral Drift at Tidal Inlets

### *General*

One of the main results of research on tidal inlets mentioned in reference (*4*) is expressed in the relation

$$A = \frac{Q_m}{C\sqrt{\frac{T_s}{\rho g}}} \tag{5}$$

in which $A$=cross sectional area
$Q_m$=maximum discharge
$T_s$ =determining shear stress for bottom stability
$C$ =Chezy's friction factor
$\rho$ =density of water
$g$ =the acceleration of gravity.

The "stability shear stress" ($T_s$) proved to be an important factor for gorge stability. Bruun and Gerritson (*4*) gave the data for the "large scale tendency" as follows:

$T_s$ with heavy sediment load=0.50 kg./m².
$T_s$ with medium sediment load=0.45 kg./m².
$T_s$ with light sediment load=0.35 kg./m².

This refers to inlets with sand bottom 0.2 to 0.4 mm. diameter.

Furthermore it was found that the $\Omega/M$ ratio ($\Omega$=tidal prism in cubic yards per half tidal cycle, $M$= net (predominant) littoral drift in cubic yards per year) seems to be useful for description of the actual "degree of stability" of the inlet.

It was concluded that ratios of $\Omega/M<100$ should be avoided and that ratios $\Omega/M>200$ are desirable for "stable inlets." The corresponding $Q_m/M$ value ($Q_m$=peak discharge per second) should be $>0.01$ to obtain "stability." If the $M/Q_m=r$ ratio was considered, it was found that this ratio is useful for the inlet's ability to bypass sand (*5*):

If $r>200$ to 300, bypassing takes place on a bar.
If $r<10$ to 20, bypassing by tidal flow takes place.

As a result of this finding, initiation of a research program, as described above, comprising investigations of friction and shear stress parameters under combined wave and current action, was suggested.

Furthermore, model studies were suggested to look into a number of pertinent tidal hydraulics and littoral drift factors and ratios between factors.

### *Research Based on Existing Field Data*

Although the relative magnitude of tidal currents and rate of littoral transport expressed by the ratios $Q_m/M$ or $\Omega/M$ plays an important role for inlet cross sectional stability, the ability of the inlet to transport solid matter by flow-induced shear stresses (rather than by the

"tidal prism" or by the "maximum discharge") must be important.

If neither scour nor accretion occurs in the inlet, the following must be valid:

(1) The littoral drift material is transferred by combined action of inlet currents, waves, and longshore currents. The total amount of sand that is handled in a certain time period is not the net littoral transport but equals the total amount of littoral transport carried to the inlet in that time, written $\Sigma|M|$, if the transport from both sides of the inlet is considered.

(2) The amount of littoral drift entering the inlet from both sides in a certain time equals the amount of net littoral drift ($V_s$) that is flushed out of the inlet channel by tidal currents during the same time period. One has:

$$V_s = p\Sigma|M| \qquad (6)$$

where $100p$ is the percentage of $\Sigma|M|$ that is handled by tidal currents and $\Sigma|M|$ is the total longshore littoral transport during one tidal cycle.

With respect to equation 6, it should be borne in mind that part of the bedload in the inlet moves back and forth as a "rolling carpet" (*4*) without being delivered to the downdrift side of the inlet during the ebbtides. From this it follows that the maximum value of $p$, occurring in case of highly predominant tidal flow bypassing, may easily exceed one. Further, with very low values of $p$, the inlet cannot survive since it is choked.

The more predominant the tidal flow bypassing is (high $p$-value), the more stable is the inlet in location and cross section.

The ratio $p = \frac{V_s}{\Sigma|M|}$ has been calculated for various inlets in order to check the above-mentioned considerations.

In most cases $M_{net}$ and not $\Sigma|M|$ is known, although only in approximation. The assumption therefore was made that the littoral transport in one direction was 1.5 times the littoral transport in the opposite direction, which means that $\Sigma|M| = 5\ M_{net}$.

$V_s$ was calculated, using Kalinske's formula for bedload transport and is indicated in table 1. Other bedload formulas valid for the same range as Kalinske's expression will give a similar result. The velocity range in question is confined to 3 to 4 ft./sec., for which reason the "proper choice" of bedload formula is not very essential.

TABLE 1.—*Some $V_s$, $M_{net}$ and p values*

| Inlet | $V_s$ per half tidal cycle | $M_{net}$ per year | $p$ (ratio) |
|---|---|---|---|
| | *Cubic yards* | *$10^6$ cu. yd.* | |
| Oregon, N.C. | 3,200 | 1.0 | 0.4 |
| Calcasieu, La. | 1,800 | .1 | 1.3 |
| Port Aransas, Tex. | 2,100 | .1 | 1.4 |
| Mission Bay, Calif. | 750 | .1 | 1.0 |
| St. Johns, Fla. | 1,610 | .3 | .8 |
| St. Augustine, Fla. | 1,000 | .3 | .5 |
| Big Pass, Fla. | 540 | .1 | .7 |
| Ponce de Leon, Fla. | 1,040 | .5 | .3 |
| Gasparilla, Fla. | 1,200 | .1 | 1.5 |

The $p$ values obtained vary between 0.3 and 1.5; and seem to describe the actual inlet stability fairly well.

A considerable number of field data (*4*) are available that furnish information of importance for the understanding of "cross-sectional stability" characterized by the $\Omega/M$ and $Q_m/M$ ratios as well as the "material transfer stability" described by the $M/Q_m$ ratio. The $p = \frac{V_s}{\Sigma|M|}$ ratio is a further development of the above mentioned ratios. The interest is now particularly concentrated in gaining more knowledge about this factor and its variations, because with this knowledge many other problems may be understood. Research in progress include:

1. Investigations on the importance of the $p$ value in general and in comparison with field data.
2. Investigations of the importance of the actual value of $p$ for inlet stability as a whole.
3. Investigations on the importance of fluctuations of $p$. $V_s$ as well as $\Sigma|M|$ varies in the field. Such variations and their reflections in "degree of inlet stability" are of utmost importance for the description of actual inlet stability as it presents itself during seasons with varying flow and sediment load. For this reason a check on the sensitivity of the model, with reference to slight changes of the relative magnitude of sediment transport by inlet and longshore currents, is important and will be handled by repeating each test for a certain value of $p = \frac{V_s}{\Sigma|M|}$ with comparatively small changes in $p$. The conclusion may be drawn after a few such tests that the observed phenomena are significant for a particular value of $p$ if, in tests run with essentially different values of $p$, significant differences are observed and if the tests with about the same value of $p$ did not show significant differences.

For such experiments reliable model scales are essential.

## Model Laws

The following section is an abstract of a paper by Bruun and Battjes presented at the International Association of Hydraulic Research in London, September 1963.

***Similarity in Bedload Transport in Uniform Flow:***

***Tidal Current By-Passing*** $(\frac{\Omega}{M}>200)$

Similarity in bedload transport requires similarity in initiation as well as in rate of transport (*10*). Shields' (*19*) investigations indicated that

$$\frac{\tau_c}{(\gamma_s-\gamma)D}=f_1\left(\frac{v_* D}{\nu}\right) \tag{7}$$

and

$$\frac{k}{D}=f_2\left(\frac{v_* D}{\nu}\right) \tag{8}$$

As to the rate of transport, most of the published formulas can be written in the dimensionless form

$$\frac{q_s}{\nu}=\frac{v_* D}{\nu}\cdot f_3\left(\frac{\tau}{\tau_c}\right) \tag{9}$$

where

$$\tau=\rho\, v_*^2=\rho g h I \tag{10}$$

From this it follows that similarity in bedload transport can be obtained if

$$\lambda\left(\frac{v_* D}{\nu}\right)=1 \tag{11}$$

and if

$$\lambda_\tau=\lambda_{\tau c}=\lambda(\gamma_s-\gamma)\cdot\lambda_D \tag{12}$$

The scale $\lambda$ is defined by

$$\lambda_n=\frac{n_{\text{prototype}}}{n_{\text{model}}} \tag{13}$$

We assume that

$$\lambda_\nu=\lambda_\rho=\lambda_g=1 \tag{14}$$

From equations 9, 11, 12, and 14 it follows that

$$\lambda_{qs}=1 \tag{15}$$

If the Froude number of the flow

$$\mathrm{Fr}=\frac{v}{\sqrt{gh}}=\frac{v}{\sqrt{ghI}}\sqrt{I}=\frac{C}{\sqrt{g}}\cdot\sqrt{I} \tag{16}$$

should be identical in model and prototype, we have

$$\lambda_I^{1/2}\,\lambda_C=1 \tag{17}$$

where $C$ is the Chezy coefficient that in fully developed turbulent flow depends on the relative roughness:

$$C=5.75\sqrt{g}\log 12\frac{h}{k}=18\log 12\frac{h}{k}\,m^{1/2}/\text{sec.} \tag{18}$$

From equations 8 and 11 it follows that

$$\lambda_k=\lambda_D \tag{19}$$

Substitution of equation 10 into equation 11 and of 18 and 19 into equation 17 yields

$$\lambda_h^{1/2}\,\lambda_I^{1/2}\,\lambda_D=1 \tag{20}$$

$$\lambda_h\,\lambda_I=\lambda(\gamma_s-\gamma)\,\lambda_D \tag{21}$$

$$\lambda_I^{1/2}=1+\frac{18}{C_p}\log\frac{\lambda_D}{\lambda_h} \tag{22}$$

Equations 20, 21, and 22 give three relationships between the four scales $\lambda_h$, $\lambda_I$, $\lambda_D$, $\lambda\,(\gamma_s-\gamma)$, of which one can be chosen freely and the others solved. The scale $\lambda_D$ cannot equal $\lambda_h$, since in that case all the scales equal unity. Therefore, $\lambda_k=\lambda_D\neq\lambda_h$ and then as follows from equations 17 and 18

$$\lambda_C=\lambda_I^{-1/2}\neq 1 \tag{23}$$

so that a distorted model is necessary.

The above theory can be extended to the case that artificial roughness elements (pegs) are needed. Obviously, equation 10 then is no more valid if $\tau$ represents the shear stress on the bed. In order to find the actual $\tau$, the characteristics of the artificial roughness elements have to be considered.

The condition of zero acceleration of a column of water of unit length and width and height $h$ in which $n$ pegs are placed with drag coefficient $c_D$ and projected area $F$ is

$$\rho g h I=\frac{\rho g}{C_o^2}v^2+n c_D F\cdot\tfrac{1}{2}\rho v^2 \tag{24}$$

where $C_o$ is the Chezy coefficient for the channel without pegs.

Inserting

$$v=C_m\sqrt{hI} \tag{25}$$

into equation 24 gives

$$C_m=C_o\sqrt{\frac{2g}{2g+C_o^2\,n c_D F}}=\frac{C_o}{\alpha} \tag{26}$$

so that equation 22 is replaced by

$$\alpha\,\lambda_I^{1/2}=1+\frac{18}{C_p}\log\frac{\lambda_D}{\lambda_h} \tag{27}$$

Rewriting equation 24 and substituting equation 26 gives

$$\frac{\rho g}{C_o^2}v^2=\tau=\frac{\rho g h I}{1+\tfrac{1}{2}C_o^2\,n c_D F/g}=\frac{1}{\alpha^2}\rho g h I \tag{28}$$

from which it follows that

$$\lambda_\nu=\lambda_{v_*^2}=\alpha^2\,\lambda_h\,\lambda_I \tag{29}$$

Substitution of equation 29 into equations 11 and 12 gives

$$\alpha\lambda_h^{1/2}\,\lambda_I^{1/2}\,\lambda_D=1 \tag{30}$$

$$\alpha^2\,\lambda_h\,\lambda_I=\lambda(\gamma_s-\gamma)\,\lambda_D \tag{31}$$

The pegs should be placed in such a way that they interfere as little as possible with the velocities near the bottom. The Karman vortices should not influence the movement of the bottom material. This may be done by putting the pegs on a thin rigid wire; the best shape and disposition of the pegs as well as their drag coefficient can be investigated in a separate flume study.

***Similarity in Bedload Transport by Waves:***

***Bar Transfer*** $(\frac{\Omega}{M}<100)$

The same criteria as above apply, viz., equations 11 and 12 are valid. For $\tau$ the expression derived from the theory of the laminar boundary layer due to wave action is used (*22*):

$$\tau_{max}=\nu^{1/2}\,\rho\,v_{max}T^{-1/2} \qquad (32)$$

where

$$v_{max}=\frac{\pi H}{T}\sinh^{-1}\frac{2\pi h}{L} \qquad (33)$$

Similarity in wave pattern is a demand also and may be obtained by making

$$\lambda_L=\lambda_h \qquad (34)$$

Substitution of equations 32, 33, and 34 into equations 11 and 12 yields

$$\lambda(\gamma_s-\gamma)\lambda_D^3=1 \qquad (35)$$

and

$$\lambda_H^4=\left(\gamma_{(\gamma_s-\gamma)}\right)^{8/3}\cdot\lambda_h^3 \qquad (36)$$

Equation 35 follows from equations 11, 12, and 14 regardless of the value of $\tau$ and is therefore equally valid for current- or wave-induced bedload transport similarity.

If similarity in the position of the breakerline is required, we get:

$$\lambda_H=\lambda_h=\left(\lambda_{(\gamma_s-\gamma)}\right)^{8/3} \qquad (37)$$

since the point of breaking in shallow water is mainly determined by the ratio $\frac{H}{h}$.

### *Similarity in Suspended Load Transport Perpendicular on the Shoreline by Waves*

A necessary although not sufficient condition is

$$\frac{\text{terminal fall velocity}}{\text{mass transportation velocity}}=\frac{\text{vertical distance}}{\text{horizontal distance}}=\lambda_I \qquad (38)$$

From equation 38 the following relationship is obtained by the use of Longuet–Higgins' theory on mass transport and equations 35 and 36:

$$\lambda_I=\lambda(\gamma_s-\gamma)^{-1} \qquad (39)$$

### *Symbols for Model Laws*

$C, C_o, C_m$ ..... Chezy coefficients ($L^{1/2}\,T^{-1}$)
$C_D$ ..... Drag coefficient
$D$ ..... Grain diameter ($L$)
$F$ ..... Area of a peg perpendicular to direction of flow ($L^2$)
$f_1, f_2, f_3$ ..... Functions
$g$ ..... Acceleration of gravity ($LT^{-2}$)
$h$ ..... Depth of flow ($L$)
$H$ ..... Wave height (crest to trough) ($L$)
$I$ ..... Slope
$k$ ..... Apparent roughness ($L$)
$L$ ..... Wave length ($L$)
$m$ ..... Subscript "model"
$M$ ..... Average net littoral transport per year ($L^3\,T^{-1}$)
$n$ ..... Number of pegs per unit area ($L^{-2}$)
$p$ ..... Subscript "prototype"
$q_s$ ..... Rate of bedload transport per unit width by volume ($L^2\,T^{-1}$)
$T$ ..... Wave period ($T$)
$v_*$ ..... Shear velocity ($LT^{-1}$)
$v$ ..... Average velocity ($LT^{-1}$)
$v_{max}$ ..... Maximum orbital velocity near the bottom ($LT^{-1}$)
$a$ ..... Coefficient
$\gamma$ ..... Specific weight of fluid ($ML^{-2}T^{-2}$)
$\gamma_s$ ..... Specific weight of sediment ($ML^{-2}T^{-2}$)
$\lambda$ ..... Scale ratio, prototype over model value
$\nu$ ..... Kinematic viscosity of fluid ($L^2T^{-1}$)
$\rho$ ..... Specific density of fluid ($ML^{-3}$)
$\tau$ ..... Shear stress on the bottom ($ML^{-1}T^{-2}$)
$\tau_c$ ..... Critical shear stress for the initiation of bedload movement ($ML^{-1}T^{-2}$)
$\Omega$ ..... Tidal prism ($L^3$)

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# A SUMMARY OF THE PRODUCTION AND SCANNING OF FLUORESCENT TRACERS

[Paper No. 76A]

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## Tracer Materials

Because of the large areas involved in tests in the field and rapid and continuous spreading of materials observed in Russian and British experiments (*1, 4, 11, 16, 20*), large quantities of tracer materials are needed and it is anticipated that the measuring unit for this material may be in terms of tons. This does not necessarily mean that several tons of tracer sand will have to be coated. According to British experience, it should be possible to dilute the coated sand 50 to 100 times with noncoated grains before release (*4*).

When manufacturing fluorescent tracers, one must take into account that the material obtained cannot differ in its physical properties from the original beach sand which it represents (*10*). Hence, it is imperative for the finished product to have the same density, hardness, and actual dimensions as the original. Shape, roundness, and sphericity values should not be altered. Requirements include that no chemical reactions would take place between tracer and seawater, that it would withstand considerable abrasion, be distinguishable and detectable in minute quantities, and its intensity of color would be subject to no change within a required period of time (*5*).

Fluorescent organic dyes, those that are water-soluble, oxidize on exposure. However, these dyes are readily available, and properly used they provide a simple means of detection. Easily dissolving in resins and set by solution-type hardeners, they increase the radius of a grain by only 300 millimicrons.

The following is a description of a procedure used by the Coastal Engineering Laboratory of the University of Florida. Quantities are approximate.

(1) 1 to 4 g. of dye is dissolved in 2.5 liters of catalyst (20 percent solution).
(2) The solution is added to 25 lb. of resin.
(3) The mixing of ingredients is carried out until color uniformity appears.
(4) The mixture is added to 200 lb. of dried sieved sand in a concrete mixer, wherein coating of the grains takes place.
(5) The wet tracer is placed in the sun on 6-mil-size polyethylene sheeting and spread thin for drying (2 to 3 hr.) or in an oven at up to 250° F. (1 hr.).
(6) Upon setting, the material consolidates into an aggregate that is quite friable. This is fed into a granulator which "delumps" it. The procedure takes about 15 min. for 200 lb. of tracer.

(7) The material is sieved back to its original grain-size distribution by using a sieving machine with large (22-in. diameter) sieves (¾ hr. for 200 lb. of sand).

Resins of both solution and suspension types may be used, depending on the dye to be incorporated. In turn, pigments, water-soluble, oil-soluble, or alcohol-soluble dyes can be used with proper matching of the incorporating plastic. Such combination of dyes, resins, and hardeners will give an almost unlimited range of colors, and with some experimentation one may achieve different values for tracer durability (e.g., 2 weeks, 3 months, 1 year). Visually, it is not possible to distinguish more than a few basic colors within a sample; however, with the utilization of sensitive electronic equipment, subtle shades of color may well be detected and recognized when based on spectrophotometric analyses.

With respect to dyes, resins, and hardeners, the following experience has been developed.

### *Dyes*

Organic dyes that fluoresce in liquid solution (No. 1) (such as $H_2O$, ethanol, MEK, Xylene, Toluene, etc.) are best in producing good effects. Those not fluorescing in solution (No. 2) and/or those not fluorescing at all (No. 3) can be utilized in conjunction with number 1.

For example:

No. 1. Rhodamine B — fluoresces in aqueous solution (red).

No. 2. Auramine 00 — nonfluorescent in solution, but fluoresces in plastic (yellow).

No. 3. Victoria Blue B — nonfluorescent dye (blue).

No. 1 + No. 2. Rhodamine B and Auramine 00 — intense fluorescence (orange).

No. 1 + No. 3. Rhodamine B and Victoria Blue B — intense fluorescence (purple).

Pigment-type coloring often causes the problem of nonuniform coating of the grains. This depends on the amount of original resin containing the pigments in pulverized form. The use of pigments most often only accentuates the problem involved in attempting to retain the original characteristic of sand grains.

### *Resins*

One of the most readily available and relatively inexpensive resin for coating is ureaformaldehyde. Melamine and epoxy resins can also be used, and methylmethacrylate will also provide good results. Whereas clearness, viscosity, and solid content factors make urea-type resins desirable for use, these plastics may rapidly decompose if not set with catalysts within a short time; therefore, they are hard to store. Deterioration can be delayed with controlled storing temperatures and humidity. The resistance to wear and to chemical decomposition of ureaformaldehydes is excellent in catalyzed form, as established by simulated abrasion tests.

### *Hardeners*

An inexpensive catalyst is available on the commercial market in the form of ammonium chloride ($NaH_4Cl$) for use with urea-type resins. However, common acids will often suffice, such as hydrochloric (HCl) or formic acid (HCOOH).

For excitation of fluorescence, both long- and short-wave ultraviolet lamps are used at the Coastal Engineering Laboratory, Gainesville. The Hanovia-type fluorescent lamps develop short-wave ultraviolet rays in the region of 2537 Å. and long-wave rays of 3650 Å. Long-wave lamps see far more use, since fluorescent tracers have proved less expensive than phosphorescent tracers, the latter requiring detection by short-wave excitation.

## Scanning of Tracers

Scanning (*6, 12*) is a method used for detection, the differentiation of size and color, and counting of particles, and it is an important procedure associated with any tracing technique of a quantitative or statistical scale.

As long as concentrations are normal (e.g., value of $10^{-4}$), a simple visual counting procedure can be used as already applied in the USSR and in England (*2, 3, 7, 10, 15, 16, 19*). The visual method does not necessarily mean that each single grain has to be counted, but rather that the intensity of a sample is compared with another "calibrated" sample of known content of grains (*11, 15, 16*). By taking subsamples out of the sample and handling the result by statistical methods, much larger actual full samples may be analyzed visually. This method will, needless to say, become inadequate as soon as bulk quantities with very high or very low concentrations are involved, making these statistical methods uncertain or impossible.

The counting method first used and still in use is visual counting by use of a luminoscope (*1, 2, 3, 8, 19*)[1 2 3] to which samples of sand are subjected under ultraviolet rays, converted from sunlight or supplied by ultraviolet lamps (*13*). This method can only handle very modest-size quantities, even if the sample in question, as mentioned, is to be interpreted as a subsample out of the actual sample taken.

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Other methods able to handle the bulk quantities needed for tracing in the open ocean are under development. Russian scientists, at this time, favor a method by which the luminophore is dissolved and the actual concentration determined by chemical analysis using a fluorometer.[4] Other methods based on photocells have been suggested and are under development (*17*). Some are based on measuring the intensity of visible light reflected from the fluorescent tracers, where correlation of intensities is made with the actual number of grains in the sample (*18*).[5] Inasmuch as 1 gram of average beach sand may include 10,000 to 80,000 single grains, it is not an easy problem to count heavy concentrations. In most practical cases one is faced with the possibility of a very low concentration, which may necessitate scanning of bulk quantities of hundreds of pounds or a number of $10^{10}$ grains.

This requires an arrangement that is capable of handling such quantities mechanically, as well as electronically, in a fast but still fully reliable way, whether a single color or a whole series of colors are to be dealt with.

After the basic ideas had been developed at the Coastal Engineering Laboratory for the requirements and general plan of an automatic scanner for dealing with fluorescent tracers, a proposal was submitted by an electronic company for building such an instrument. In essence, the scanner as designed would complete a size analysis of the sample of beach sand and would differentiate several colors at the same time. The rate of scanning would occur at 20,000 particles per second.

The instrument when constructed will consist of —

(1) A mechanical assembly consisting of two parts: a hopper to which the sample is subjected and a Vycor glass tube in which the sand falls from the hopper at a variably controllable rate utilizing free fall.

(2) An assembly of electronic circuitry. The sample falling in the tube passes in front of several optical channels enclosing photomultiplier cells (each cell in operation accounting for one of the many given colors in the sample), whereupon the photocell responds whenever a grain with the proper color crosses the light path of the optical channel. By using a method of fixed duration pulse-signal transmitting, a counter registers the signal (hence the colored grain) digitally or graphically after the signal passed through an amplifier, a pulse shaper, and a relay driver. Each photocell excludes all but one color as set by a combination of primary and secondary interference filters incorporated into the optical channels. The dimension of the output signal transmitted by the cell to the counter mechanism registers the size of the coated sand particles simultaneously with the fixation of color. Within the counter the tabulation of color, size, and quantity occurs at the rapid rate of 1.8 kg./hr.

The theory of the scanner is basically set in the following equation. The power entering the photocell is

$$P = I\,\sigma\,a\,b\,Q\frac{1}{4\pi R^2}A\,c\,d\,H_v$$

where

$I$ = Intensity of incident beam in photons
$\sigma$ = Area of sand grain
$a$ = Fraction of grain covered by fluorescent molecules
$b$ = Fraction of irradiated fluorescent molecules which absorb incident photons
$Q$ = Quantum efficiency of fluorescent process
$R$ = Distance from sand grain to lens of optical system
$A$ = Area of lens
$c$ = Filter factor
$d$ = Transmission factor for optical system
$H_v$ = Energy of an emitted photon.

As this is known in terms of fluorescence, excitation of this phenomenon occurs in the ultraviolet region of the spectrum and light transmission occurs in the visible region (*14*). Since the latter is a relatively short segment of the spectral scale, all available fluorescent colors cannot be utilized for fear of misinterpretation of the overlapping curves. For the same reason, the eye cannot distinguish but a few colors when many are mixed together. In terms of differentiation the electronic scanner, using combinations of interference filters and relying upon peak outputs of the tracer colors, is able to increase the number of employable colors for any one experiment by a factor of approximately 2.5 (10 to 12 for a scanner as compared to 4 to 5 for visual means). This alone is an important reason for using such an instrument, aside from its automatic bulk quantity counting feature.

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# DESIGN AND CONSTRUCTION OF RUBBLE-MOUND SEAWALLS AT SANTA CRUZ, CALIF.

**[Paper No. 77]**

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## Abstract

This paper presents information on the design and construction of rubble-mound seawalls along West Cliff Drive at Santa Cruz, Calif., to illustrate a means of arresting major erosion by the ocean of a soft rock cliff formation. Offshore of this location, relatively high deep-water waves produce breakers along the bluffs. Based on the existing depth of water and the anticipated scour, a 9.5-foot design wave was calculated, which required an armor stone size of approximately 5 tons and a slope of 1 vertical on 1½ horizontal. As of November 1962, 19 units of seawall ranging from 30 to 700 feet in length have been constructed. Observation of the seawalls since construction indicates that stone size is adequate but that undesirable settlement has occurred. This settlement could probably be eliminated by placing armor stone on bedrock or below the limit of beach scour in a previously excavated trench during initial construction. Based on a recent field inspection, the seawalls appear to be serving their intended purposes.

## Synopsis

This paper presents information on the design and construction of rubble-mound seawalls to illustrate a means of arresting major erosion of a soft rock cliff formation by ocean waves and currents. Prior to construction of the seawalls, considerable public and private property and about two blocks of one of the major scenic drives in the City of Santa Cruz had been progressively undermined and eroded. Further erosion of critical areas has been prevented by

construction of the seawalls, and several areas previously completely eroded have been reclaimed.

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The City of Santa Cruz is located in Santa Cruz County, in the central portion of the coast of California about 65 miles south of San Francisco. Although the 1960 population of Santa Cruz was only about 26,000, Santa Cruz serves as one of the major recreational areas for the San Francisco Bay area, which had a 1960 population of about 3.5 million. During the summer months the population of the city increases fourfold or more.

One of the major visitor attractions of the city of Santa Cruz is West Cliff Drive, which extends westerly from the main beach recreational area about 2½ miles to the city limits at Natural Bridges State Park. This drive parallels the shore and provides numerous parking areas and vantage points overlooking Monterey Bay. The shore along West Cliff Drive consists of irregular eroding cliffs of soft shale, locally known as Monterey shale, averaging about 35 feet in height. No protective beaches are found in this reach. Wave erosion has caused extensive recession of the sea cliffs facing the ocean at a rate of about 1 foot annually. Severe erosion has occurred at three separate sections of the drive. In 1961, erosion caused the closure of one 2-block section of the drive and required rerouting of traffic. In addition to these three sections, a number of separate segments of shore have been undercut and large caverns exist under the road.

Along West Cliff Drive, the only method of protection considered feasible is the construction of rubble-mound seawalls. The irregular alinement and rocky nature of this reach of coast would make placement of beach fill to obtain a stabilized protective beach economically infeasible. This section of coast contains a number of projecting points and minor headlands that serve as groins. However, their projections do not impound sufficient sand to protect the bluffs from erosion. This indicates that the use of groins in conjunction with beach fill, or alone, would not correct the problem condition.

The tides at Santa Cruz are typical of those along the central California coast, and have a diurnal inequality; the mean diurnal range being 5.3 feet. The maximum tide is about 8 feet above mean lower low water (m.l.l.w.), the datum for the Pacific Coast of the United States. Characteristic deepwater waves have significant heights ranging from 2 to more than 20 feet and periods ranging from 4 to 20 seconds. The mean height and period are 4 feet and 13 seconds, respectively. The prevailing direction of wave approach is from the northwest. Winter storms with wave heights of 10 feet or more usually approach the coast from the southwest quadrant. From the data available during the design studies, the maximum deepwater significant wave height (crest to trough) was estimated to be 25 feet for a 9-second wave from the southwest. Data computed subsequent to the studies indicate that deepwater waves with a significant wave height of 33 feet and a 13-second period from the west occurred during a storm in February 1960.

In the planning and the structural design of the seawalls, use was made of the criteria contained in Technical Report 4 entitled "Shore Protection Planning and Design," published in June 1954 by the Beach Erosion Board, Corps of Engineers, and later revised and reissued in 1961. In the design of the seawalls considered, the height of the maximum wave that would break on the structure was determined from the relation, $H_b$ (in feet) equals $d_b$ (in feet) /1.3, where $H_b$ is the breaker height and $d_b$ is the water depth at the point of breaking. For design purposes, stillwater level was estimated to be 8 feet above m.l.l.w.

In determining the size of face or capstone required, it was estimated that after settlement at the toe, the slope of the seaward face of the wall would be 1 vertical on 1½ horizontal. As the depth of scour at the toe of the walls would be limited by bedrock to about −4.5 feet m.l.l.w., the maximum depth of water at the seawall toe would be about 8.0 + 4.5, or 12.5 feet. The design wave height would be 12.5 feet/1.3, or about 9.5 feet. If this design wave and a specific gravity of stone of 2.65 and slopes of 1 on 1½ are used, the required weight of face stone was found to be about 5 tons. (The required stone weight would be about 4 tons, if the 1961 revision of Technical Report No. 4 is used.) As occasional wave overtopping would produce no adverse effect, the height of the crest of the structure was adopted as one wave height above stillwater level, resulting in a crest elevation of 9.5 + 8.0 or + 17.5 feet m.l.l.w. To reduce settlement at the toe of the seawalls, a bedding layer of stone was specified when the depth of sand on the shore exceeded 1 foot. The design section is shown in figure 1.

As of November 1962, 19 units of seawall, ranging from 30 feet to 700 feet in length, have been constructed. One of the units completed in 1961 is shown on figure 2. Observation of the seawalls since construction indicates that the stone size is adequate. However, it has been noted that settlement has occurred only near the toe of the structure, thus exposing a strip of core material that has required the placement of a relatively small amount of additional stone. If found to be economical, this additional stone

placement could probably be eliminated by placing armor stone on bedrock in a previously excavated trench during initial construction. This construction procedure was used at one location in the same project where working conditions permitted construction equipment to work directly on a narrow beach or bench. In most instances, construction equipment was found to work at the top of the cliffs.

Based on a recent field inspection, the seawalls appear to be serving their intended purposes. The 2-block section of West Cliff Drive that had been eroded has been reclaimed and is again open to traffic. It is concluded that the design and construction described is a satisfactory method of protection for the shore at West Cliff Drive.

## Acknowledgments

Material presented in this paper was obtained from House Document 179, 85th Congress, 1st Session, entitled "Santa Cruz County, California, Beach Erosion Control Study," and unpublished data on file in the Engineering Division of the U.S. Army Engineer District, San Francisco, Corps of Engineers.

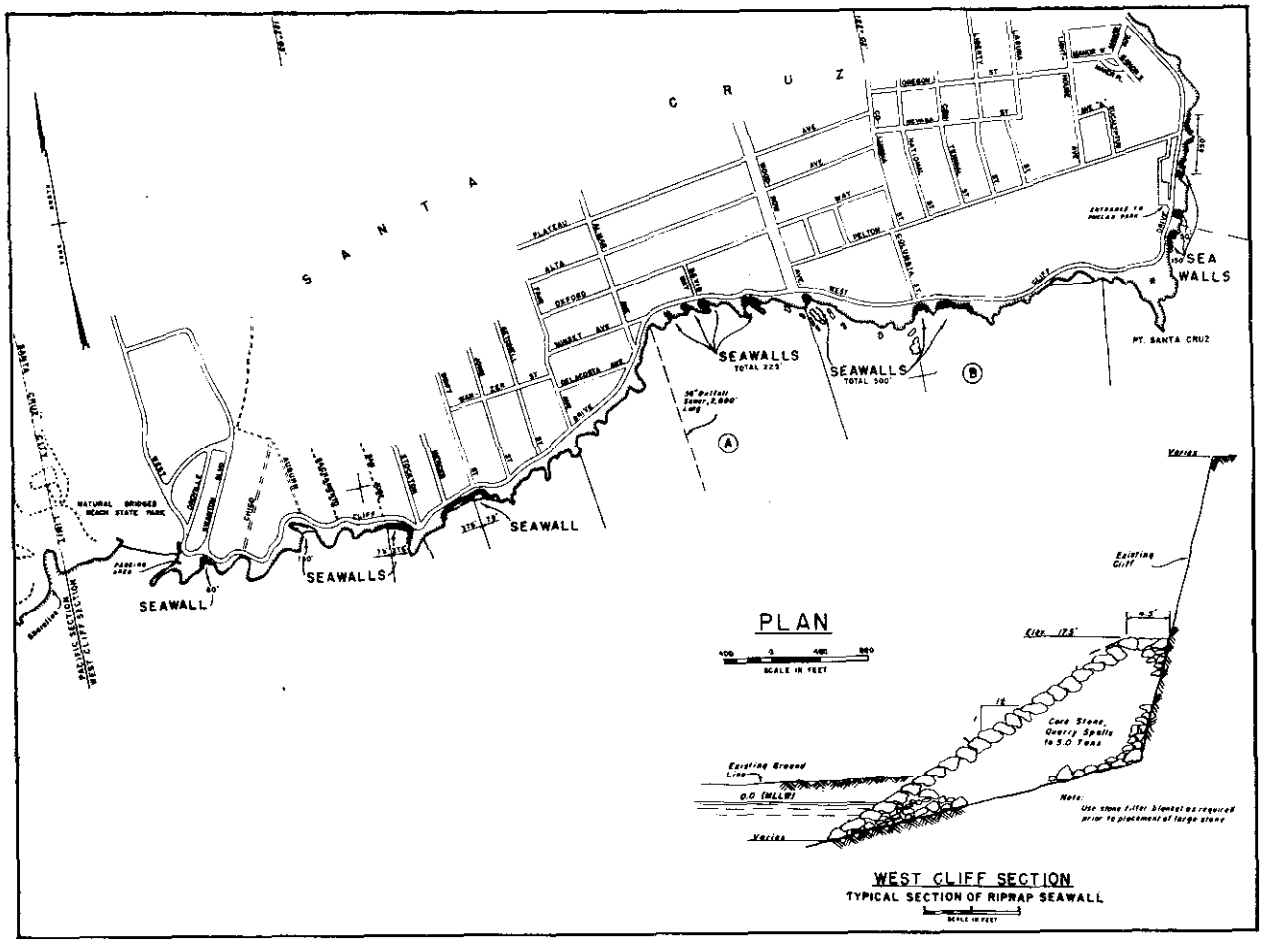


FIGURE 1. — Location of seawalls and typical seawall cross section.

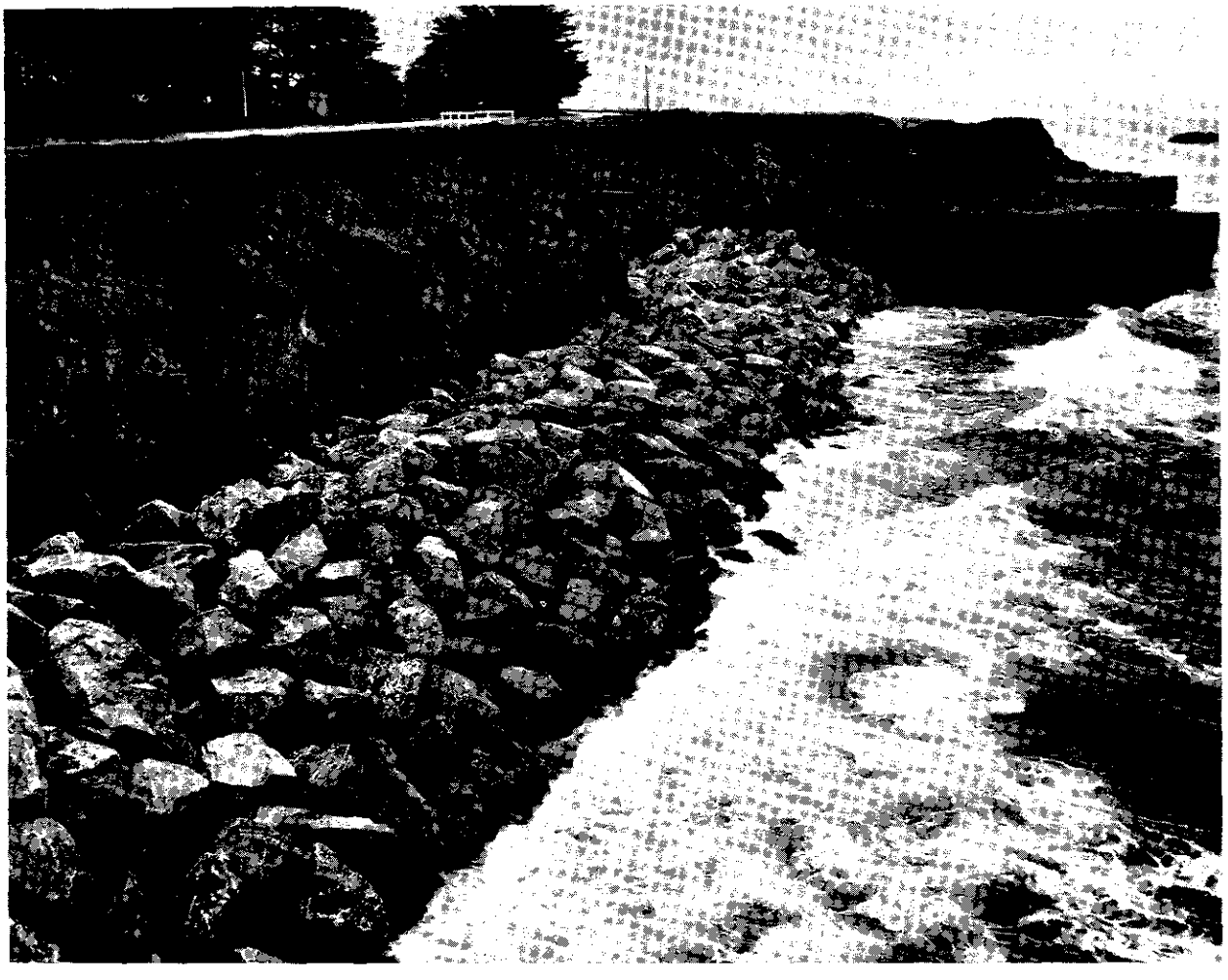

FIGURE 2. — Typical seawall unit completed in 1961.

# DISCUSSION—SYMPOSIUM 3

# Sedimentation in Estuaries, Harbors, and Coastal Areas

*Moderator:* J. W. Johnson, University of California

*Panel:* H. A. Einstein, University of California
H. B. Simmons, CE
J. M. Caldwell, Beach Erosion Board
P. Bruun, University of Florida

MR. JOHNSON:

There are a few questions that have been submitted by those in attendance. Before proceeding with the submission of these questions to our panel, however, I am asking each panel member to describe briefly certain research that they now have in progress. Such descriptions will provide those assembled here this morning with information that may not be available in published form for a year or more in the future. I am therefore asking each panel member to present his discussion in turn.

DR. EINSTEIN:

I will not give any introduction, because we have very little time left. The work I want to refer to is connected with the information needed in making any kind of calculation or model experiments pertaining to tidal flows. As you know, we're not only interested in the motion of the sediment when we do such a thing but we also have to duplicate properly the friction conditions. In choosing the proper friction factors on mud deposits you look in vain through the entire literature to find them. Nobody has

ever been able to measure decently these values as far as we could find out, and we definitely were not able to do it in our flume experiments, because the slopes are on the order of a foot to the mile and for that a standard flume is not quite long enough. So we came to the conclusion the only way to do that is to build a very long flume and make experiments in that.

The reason why the slopes are so small is that these deposits will not stand any high flow velocities. For that reason one has to operate with velocities on the order of one foot per second. The flume that we're using is a thousand feet long, and we're using this flume, as I said, just for the purpose of determining these friction factors. At the moment we're learning how to handle this monster. It takes a long time to achieve equilibrium; one must be very careful not to induce any wave action as that would make the measurements difficult. We hope to get some results in about a year. The work is supported by the National Science Foundation.

One more remark to those who are interested in dredging. Lately, we have been called in to help with the design of some dredges that were supposed to handle very large amounts of sands and to pump these up from rather large depths up to 200 feet. In that case, you need a booster pump at the end of the ladder. A jet pump was selected for the booster. You know that a jet pump has the advantage that there are no movable parts in connection with the flow. For that reason minimum wear was expected. We found, and this is the reason why I mention it, that there is an unexpected benefit due to the use of these pumps. Since a jet pump adds a considerable amount of fresh water, that means water without sediment, to the flow right at the beginning, you're absolutely sure to always have that much water available in the mixture. With proper choice of this drive discharge, the ladder can be let down into the sand, completely submerge itself, and you still do not run the risk of clogging your lines. In a 1-year operation of the prototype, it was found that as an average at least a 50 percent increase of the average concentration was possible compared to other dredges. Thank you.

MR. SIMMONS:

I think I can make mine very brief for two reasons. First, the majority of you will be in Vicksburg tomorrow, I understand, to see some of the work we have in progress, and the specific estuary model which will be on display will be the Savannah Harbor model, which was described by Mr. Harris this morning in his discussion. In addition to Savannah Harbor, we have a comprehensive model of New York Harbor, which was mentioned in the résumé of Mr. Panuzio's paper, and which has been used extensively to study plans designed to reduce sedimentation of the Hudson River navigation channels and the pier slips which abut this channel. I might emphasize the great importance of that pier slip shoaling problem, not only in New York Harbor, but in many other harbors. The maintenance of these facilities are not the responsibility of the Federal Government but the responsibility of the owners and operators of the facilities themselves. Dredging is quite an expensive proposition in these areas, because in most cases it has to be accomplished by clamshell type equipment or similar devices, with the spoil loaded on barges and hauled many miles to licensed disposal areas. As a consequence, the unit cost of dredging is much greater than has been quoted in connection with the Corps navigation projects, and in some cases runs as high as $1.50 per cubic yard.

Another semipermanent model at our installation is the Delaware River model, which reproduces that estuary from the Capes, where Delaware Bay joins the Atlantic Ocean, to Trenton, N. J., which is the head of tide in the Delaware. This model has been in existence for a great number of years, and from all indications it will be there for quite a long time to come. Incidentally, this model contributed its small bit to the decision that the sump rehandling method for disposal of dredge spoil was in fact the proper technique to be employed in the Delaware.

We have recently completed construction and adjustment of a model that reproduces the lower 52 miles of the Columbia River and pertinent offshore areas, which were discussed to a certain extent in the paper by Mr. Lockett. We are just getting into the study phase of this model, and apparently the problems involved are equally serious between the entrance, which deals primarily with rehabilitation of existing structures designed to improve flow conditions and minimize shoaling, and the upriver area, for which there has recently been authorized an increase in depth of the navigation channel. So I can foresee, in the course of this study, chasing back and forth from the entrance to the upriver areas every couple of weeks, trying to satisfy the design engineers with the information they need to carry on their work in the field.

We have recently completed construction of a deep draft navigation channel through Matagora Bay, Tex., extending from the gulf to Point Comfort, Tex. This study was rather unique in that this channel did not go through the process of evolution of most of our estuary channels, in which it was gradually deepened a few feet at a time from a natural depth to a present great depth. The channel will be dredged from a natural depth of about 12 feet to full project depth of 36 feet. I won't take the

time to mention any of the others, since those discussed represent a good cross section of the work we have in progress.

Professor Johnson handed me one question which reads as follows: Does good harbor maintenance encourage building of ships with even greater drafts, thus causing a continued compounding of the dredging problem in harbors? I think the answer to that is quite obvious, in that our experience with the commercial shipping interests indicate that they will build their ships to the maximum draft that navigation channels can accommodate, and this especially applies to the modern tankers. It is quite obvious that the unit price of transporting oil is lowest for the largest tankers, and that this factor is given major consideration in the design and construction of the modern supertankers. If navigation channels are maintained at 40 feet where the project says 40 feet rather than being allowed to shoal to lesser depth and then redredged to 40 feet, there is little doubt that vessels of greater draft will be developed to utilize these channels. Whether this is a good practice or not, I am not competent to say, but the economic benefits of that channel are thus improved if it can accommodate vessels of greater draft.

MR. CALDWELL:

I thought I would mention three things very briefly that are being done now or will be started in a short while. One is the study of inlet flow characteristics. This is part of the work of the tidal hydraulics committee of the Corps of Engineers which is sponsoring this work. We will try to define what might be called the discharge coefficient of inlet. Now we find that for navigation purposes and pollution control, the tendency is to want to open up the inlet. For the hurricane control, the tendency is to want to narrow or shut down on the inlet. Thus, we find that Fish and Wildlife Service usually would rather the inlets were not disturbed at all. So we find that we've got people working and pulling in all different directions, and we're not able to answer with definity many of the questions that arise about flow through inlets.

We're trying to do some work now to enable us to make more precise calculations of the effect of what you might call "monkeying around" with these inlets and what will happen back in the lagoons and bays fed by these inlets. Now the second piece of work I might mention ties in somewhat with Mr. Mauriello's very informative paper on dredges. We find that along the coast of the United States most of our important beaches are in an eroding condition. This is largely due to the tendency of waves to denude the beaches and much of this sand finds its way offshore or into the inlets where it's not wanted. Now we'd like to have this material back on the eroded beach, but we find that the pumping distances from the inlets where there may be a giant shoal are in many cases excessive. Right offshore in sight of these beaches there are, in some cases, tremendous quantities of sand, but so far there has not been a dredge developed that could operate out in the surf zone or just beyond the surf zone and pump this sand through the surf zone on to the beach.

One of our big projects right now is to try to develop a method of locating this sand, bringing in a suitable dredge, and pumping this sand through a pipe line through this surf zone on to the beach. Once we can perfect this method, a lot of our troubles will be over, because we will be able to find sand then (we hope at an economical price) and put it on the beach where it's most needed. The third thing I might mention is our present studies of dune rebuilding. Now this is a kind of a counterpart of what Professor Johnson was speaking of on sand movement, and here we are attempting to find the best ways to reconstitute a demolished dune line, particularly on our low barrier islands, such as the Carolina coast. There we are installing various types of sand fences, studying the resulting rate of growth of the dunes and what is the best arrangement.

Now I might mention that we have found from our recent experiments that an ordinary snow fence will fill with sand in as short a time as three or four weeks but that you gain nothing beyond that unless you install a second fence to cause this dune to build higher. Now at this low one-fence height, the dune is usually very vulnerable to any big storm that comes along. Thus, if we build it to one sand fence level, we know we should immediately go in and start building up fences to a greater height so that we can preserve and enlarge the dune. There will be periodic reports coming out, probably as technical memorandums of the Beach Erosion Board, on this last subject. Thank you very much.

MR. BRUUN:

Gentlemen, we have listened to a great number of papers informing us about the results of the latest research on sediment transport. Almost all of them dealt with river technology and represented the engineering approach emphasizing the solution of the most pressing practical problems. Progress has been made in the development of methods and in general understanding of river mechanism. Laboratory and field studies have brought to light the effect of sediment on the hydraulic characteristics of a stream and revealed the pertinent factors involved in stream geometry, the different

modes of transportation, and the great importance of bottom configuration and friction for the magnitude of sediment transport.

It is generally agreed that there is a strong need for research on bed forms and wash load transport. Special attention must be paid to the sediment problem at reservoirs and to the development of riverbeds, up- and down-stream of reservoirs and dams. The fact that good apparatus and techniques are readily available for sedimentation measurements makes it likely that results from various regions are generally comparable, and the laboratory engineer and researcher can point with a certain pride to the situation that field studies are applying laboratory results. We are, apparently, on the right track — as long as we are dealing with unidirectional river flow.

How is the situation on our seashores? Not nearly as good! Here we are faced with huge problems represented, e.g., by several million cubic yards of material to be dredged annually in order to maintain navigation entrances as the New York Harbor basins, the Columbia River, and the San Francisco Bay. On the beach erosion side of the problem: Florida is losing approximately 20 million cubic yards of sand per year, the Outer Banks in North Carolina are wearing out, and the dunes of Long Island and Cape Cod suffered severe damage during 1961–63 storms and earlier.

Progress in littoral drift technology has been hampered for several reasons. The two most important have been:

(1) Overwhelming problems.

(2) Unfortunately, the "experienced man."

Meanwhile development of coastal engineering is still progressing, also in the fields of dredging and filling and in the erosion and protection technology. What we were not able to do a few years ago is now possible. "Zulia" dredged in 1 year 57 million cubic yards of fill by boom-operations from the Maracaibo Bar. Equipment for offshore dredging for beach nourishment is on the testing stage, and we may see it in actual operation before the end of 1963.

The "experienced man," who proceeded on a trial and error basis only, making one sand trap after the other, is now retiring slowly. It has been unlikely that any inlet or harbor would be established on the Continent without adequate model experiments. For years, very few studies with models have been accomplished in this country, but the San Francisco Bay studies and the Columbia River study are now proceeding. The Savannah River and Port problems have already been subjected to intensive studies. In Florida, no model study of navigation and beach erosion problems at our 17 most important inlets and passes had been carried out until 1956. In the spring of 1963 five studies had been completed, and three are in progress. So we are making headway there, too. New physical model laws for sediment transport, as proposed by Valembois, Yarlin, and as presented by Battjes and myself at this meeting, are expected to make model calibration and tests easier and more reliable. Meanwhile, we do not yet share the benefit of the tremendous achievements in adequate instrumentation for sediment transport technology by the river engineers, although progress has been made in wave and current recording methods. The efforts made by the U.S. Council on Wave Research in coordination and information services must be highly commended, but still, progress in littoral drift science and technology has been slow. In recent years, it seems that the tracing technique developed in England, Holland, and Portugal (radioactive tracing) and in the U.S.S.R. (fluorescent tracing) is going to mean a breakthrough in this important field. So far, tracing research has been of mainly qualitative nature, but quantitative results may be incoming in the near future.

This panel discussion is supposed to air developments and new ideas. It seems, therefore, to be proper to mention briefly the new scanning instrument for fluorescent tracing, which is being built at present for the Coastal Engineering Laboratory of Florida, financed by a grant from the Department of Health, Education, and Welfare.

The main problem associated with quantitative fluorescent sand tracing is the large number of samples that have to be analyzed and recorded in an extremely short period of time. Field and statistical experiments furthermore confirm that concentration of tracers within the samples will range between $10^{-2}$ and $10^{-7}$. At either extreme, grave difficulties would be encountered if one would have to rely on manual and visual detecting, counting, and recording techniques alone, leaving too much room for repeatedly variable errors of some magnitude.

To comply with the test conditions producing samples at the approximate rate of 500 grams/minute, certain basic ideas have been developed at the Coastal Engineering Laboratory for the requirements and general plan of an automatic scanner for dealing with fluorescent tracers. On the basis of these ideas, a proposal was submitted by an electronic company for building such an instrument. In essence, the scanner as designed will complete a size analysis of samples of beach sand and differentiate within each sample several colors at the same time. The rate of scanning will occur at 20,000 particles per second.

The instrument essentially will consist of:

(1) A mechanical assembly of two parts —

a hopper for sand samples, and a Vycor glass tube in which the sand falls from the hopper at a variable controllable rate utilizing free-fall.

(2) An assembly of electronic circuitry. The sample falling in the tube passes in front of several optical channels enclosing photomultiplier cells (each cell in operation accounting for one of the many given colors in the sample), whereupon the photocell responds whenever a grain with the proper mercury-light-excited color crosses the light path of the optical channel. With a method of fixed-duration pulse-signal transmitting, a counter registers the signal (hence the colored grain) digitally or graphically after the signal passed through an emplifier, a pulse shaper, and a relay driver. Each photocell excludes all but one color as set by a combination of primary and secondary interference filters incorporated into the optical channels. The dimension of the output signal transmitted by the cell to the pulse shaper designates the size of each coated sand particle simultaneously with the fixation of its color. Within the counter, the tabulation of color, size, and quantity occurs at the rapid rate of 1.8 kg./hr.

The theory of the scanner is basically set in the following equation. The power entering the photocell is

$$P = I\sigma abQ \frac{1}{4\eta R^2} Acd \cdot H_\nu$$

where

$I$ = Intensity of incident beam in photons
$\sigma$ = Area of sand grain
$a$ = Fraction of grain covered by fluorescent molecules
$b$ = Fraction of irradiated fluorescent molecules which absorb incident photons
$Q$ = Quantum efficiency of fluorescent process
$R$ = Distance from sand grain to lens of optical system
$A$ = Area of lens
$c$ = Filter factor
$d$ = Transmission factor for optical system
$H_\nu$ = Energy of an emitted photon.

Some limitations are inherent in this system. One is that the number of colors to be scanned simultaneously is dependent upon the spacing of their respective transmission curves within the visible spectrum with a minimum of overlap. At present four colors are contemplated for use while provisions will be made that the instrument could handle an additional four colors at a future date.

Since detection is proposed at peak outputs of each fluorescent curve through the use of narrow range filter, the question of how many colors could be employed remains an optico-mechanical task.

This instrument is one of the small steps forward, which will, we hope, help us to wind up with the excellent progress that has been made by our friend, the river engineer.

MR. JOHNSON:

Before turning the chair back to the chairman I will attempt to answer two questions which relate to my paper on sand transport by wind action. The first question was submitted by Professor Vanoni. It is, "Was there a windrowing affect on the sand; that is, was the mean size of the transported material less than that or those of the materials fed into the upstream and some among the bed?" The answer in general is, "yes." When the vertical trap which was one-half inch wide and extended a foot above the bed was exposed to low wind velocities and before the duration of the run was long enough for a scouring effect to occur around the trap, a comparison between the sand that was collected in the trap and that which was fed into the upper end of the channel were identical. At the higher windspeeds, some finer material not only went over the vertical trap but also passed over the horizontal trap and on through the fan. In general the agreement between the material trapped and that in the bed was good, and I would refer you to the work by Belly for the comparison. Professor Brooks asked the question: "Did any ripples or dunes appear in the wind tunnel and how did they affect transport?" The bed was always rippled during the tests. Runs were always long enough that an equilibrium condition occurred. The bed had very regular ripples, and we would not have had sand transport unless we had the ripples. A second question by Professor Brooks is: "In the field, how would one determine the shear velocity for a dune field which can be used in the transport equation?" In a limited number of field tests on dunes near Dillon Beach, Calif., we used a multiple pitot tube arrangement and measured the pressure difference on each tube by means of a Magnehelic gage. This instrument permits pressure differences of down to 0.01 inch of water to be measured.