STABILITY OF COASTAL INLETS

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ABSTRACT

This paper is a continuation of papers of earlier date (4) and (5) and is an abstract of (6). Pertinent factors involved in inlet stability are discussed briefly. Results of analysis of existing data are mentioned and a future research program is outlined.

PERTINENT FACTORS INVOLVED IN INLET STABILITY

In order to obtain a stable tidal inlet in alluvial material it appears to be an inevitable assumption that littoral drift material is being supplied continously to the inlet. Part of this material is deposited on the inlet bottom where the tidal currents will move it forward and back as a kind of "rolling carpet."

In order to obtain a relatively stable situation this carpet must not move back and forth too rapidly since it thus runs the risk of being lost at both ends (the ocean and the bay). Nor can it be allowed to move too irregularly, changing its velocity and travel time rapidly, since it may soon "get stuck" at one or at both ends in the form of excessive deposits. If -- because of insufficient littoral drift supply -- inadequate amount of material is available for building up this carpet the inlet will be constantly "shaved" and will gradually develop non-scouring open bay or perhaps estuary characteristics. Fig. 1 shows longitudinal sections through inlets of different length. In the first case the (unstable) channel is so short that the rolling carpet extends outside the inlet floor, which in turn causes material to be deposited on shoals in the sea and in the bay by the material-loaded ebb and flood-currents. In the second case the (also unstable) channel is so long that material is now deposited inside both ends of the inlet channel because it gradually became so long that currents were too slow to carry the material load out in the sea or in the bay for depositing. The third case demonstrates a stable "status quo" situation between inlet length, current velocities, and material load.

A rational approach to the material balance problem is given in (6).

To analyze the stability problem the cross-section area A of the inlet gorge is considered explicit as a function of various factors:

$$A = F (Q_m, \beta, \tau, B, \bar{c}, W_a, M, Q_o, t) \qquad (1)$$

The factors Q_m (discharge) and β (form factor of cross-section) are interrelated, and shear stress τ, bottom composition B, and the littoral drift, M, to the inlet, also have a certain direct influence on β. Factors

τ, β, $\bar{c}$ (sediment concentration), W_a (wave action), and M are also inter-related. The friction factor not included in equation (1) enters the picture through τ. The fresh water discharge Q_0 (head water from river or drainage canals) have a certain influence on flow distribution and thereby on τ and β. The time factor t represents "the time history" which is important because of the time lag between action of forces and the reaction of the elements acted upon by the forces.

The influence of the maximum flow, Q_m

Consider A = F (Q_m-----) when Q_m represents the maximum discharge per second through the inlet gorge. The relationship between A and Q_m can be expected to be fairly linear because if one inlet channel is stable with cross-sectional area A, and maximum flow Q_m joins with a neighbor channel which also has cross-section A and maximum flow Q_m, the result most likely will be a combined inlet with cross-sectional area of the order $2A$ and maximum discharge $2Q_m$. Changes in friction characteristics and other factors causing energy loss may distort this picture somewhat and the actual dimension of the inlet gorge and channel will depend upon the utilization of the cross-section for flow and the distribution and actual size of shear stresses as mentioned later in this paper.

The influence of the shape factor, β

Consider A = F (Q_m, β,-------). Studies of inlet gorges reveal a certain similitude between the cross-section of different gorges even though a considerable number of inlets are provided with gorges which do not have a simple cross-section. Some inlet channels have cross-sections split up in a "deep part" and a "shallow part". The "coefficient of utilization" for flow of these two parts are not equal. The shallow part carries comparatively little flow compared to its area while the opposite is the case with the deep part. The importance of the shape factor is thereby clear. Littoral drift, particularly with coarser material may often tend to develop steeper side slopes and therefore a more "economical" cross-section. Increased fresh water flow in certain periods may work similarly (16). With jetty-protected "improved inlets" there is usually only one channel with greater depth which means greater hydraulic radius and less loss of energy from banks, shoals and similar side effects. In other words conditions are better organized for flow; for this reason it can be expected that a comparatively smaller cross-sectional area is sufficient to carry a given amount of maximum flow.

The influence of the shear stress, τ

Consider A = F (Q_m, τ,-----) in which τ is the force exerted by the flow on a unit area of the bottom. For a cross-section with horizontal bottom of unlimited width a linear relation between A and Q_m involving a certain shear stress (more simply but not accurate replaced by"average velocity") can be expected.

Assuming steady or slowly varying conditions we have $\tau = \rho g\ RS$ in which ρ = density of water, g = acceleration of gravity, R = hydraulic radius and S = slope of energy line.

By introducing $v = C\sqrt{RS}$ and $Q = Av$ we find:

$$Q_m = AC\sqrt{\frac{\tau_s}{\rho g}} \qquad\qquad (2)$$

whereby τ_s refers to spring tide conditions. τ_s is called the deter-
mining shear stress. For an alluvial bottom tidal inlet the require-
ment of stability is either that this shear stress stays below a certain
value or is coordinated in such a way with water flow and material move-
ment that the total transport of material away from one section equals
the transport to this section from another section. According to equation
(2)

$$\tau_s = \rho g\frac{Q_m^2}{A^2 C^2}$$

The problem of inlet stability is then reduced to the determination of
the stability shear stress τ_s under a variety of boundary conditions.

In regard to the influence of channel friction reference is made to
the authors (6), which in turn refers to comprehensive literature file on
this subject. An increase in C is usually associated with an increase in
τ_s.

As mentioned in the following paragraphs various other factors will
influence τ_s. Coarse bottom material will usually result in a higher τ_s
than fine material. Sediment load injected in the tidal inlet flow from
rivers or from the longshore littoral drift will usually cause a higher
τ_s; wave action will decrease τ_s. Increase of littoral drift will raise
τ_s relatively; fresh water flow may also increase τ_s, particularly when
it is concentrated in limited periods of time and causes "at a station
changes" as observed in rivers (16).

The influence of soil condition of the inlet bottom, B

Consider $A = F (Q_m, B, -----, \tau_s)$ and $\tau_s = f (B, ------)$. A discus-
sion on the influence of soil conditions is a discussion on the influence
of soil conditions on the τ_s. Table 1 - see the following section and
reference (6) - gives certain "limiting values" for the shear stress in
canals and rivers with granular material considering clear water as well
as sediment laden flow. The actual grain size does not seem to be very
important within certain limits. Tidal inlets will because of supply of
littoral drift material to the inlet and because of its origin almost
always have alluvial material bottom and although the flow is continuously
reversing it seems reasonable to expect a certain similarity between the
behavior of rivers and tidal inlets.

The influence of suspended load, c̄

Consider $A = F (Q_m, \bar{c}, ------, \tau_s)$ and $\tau_s = f (\bar{c}, -----)$. Sediment
load may be derived from upstream sources or from the littoral drift.
According to Table 1 sediment load increases the limiting shear stress.
Increase is considerable for heavy load. It is reasonable to expect
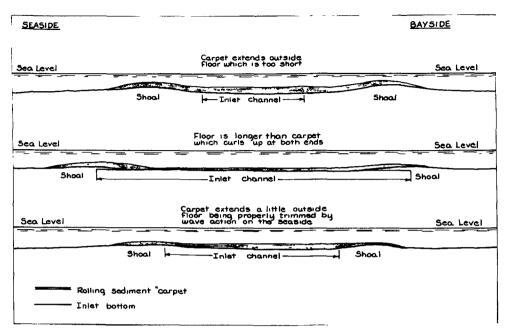similar conditions at tidal inlets as we find at rivers. According to

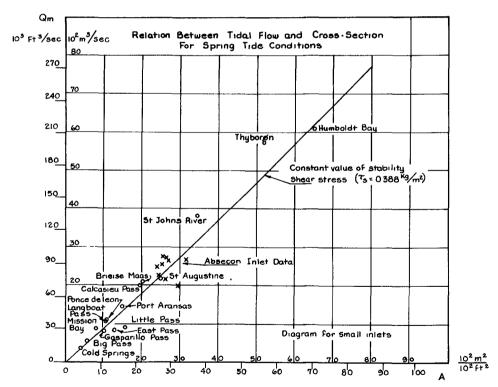Fig. 1. Material transport in inlet channels as "Rolling Carpet".

Fig. 2. Relation between maximum tidal flow and cross-section for small inlets at spring tide condition.

observations by Leopold and Maddock (16) the roughness of a channel decreases when suspended load increases. Vanoni (21) explains this effect as a result of decreased turbulence:

> "The increase in velocity or decrease in channel resistance, as suspended load is added to the flow can be explained by the effect of the sediment in reducing the turbulence. To keep the sediment from settling, work must be done on it. The energy for this work can be provided only by turbulence which is damped and reduced in intensity when it gives up energy. This means that the momentum transfer coefficient is also decreased thus allowing the velocity and velocity gradient to increase."

In regard to velocity distribution for silt laden flow the reader is referred to (8).

The influence of wave action, W_a

Consider $A = F (Q_m, W_a, \text{------}, \tau_s)$ and $\tau_s = f (W_a, \text{--------})$. The wave action makes the actual τ_s -values vary rapidly and increase material load and transport. At the present stage of our knowledge we have no specific knowledge of the influence of wave action under varying conditions including current activity. In the entrance area of an inlet flow will be more or less loaded with material stirred up by waves and currents. This will decrease the τ_s in this area but may cause an increase of the τ_s further bayward because of the material load.

The influence of littoral drift, M

Consider $A = F (Q_m, M, \text{------}, \tau_s)$ and $\tau_s = f (M, \text{----------})$. The littoral drift may influence the development of inlets directly by deposits on the side slopes of the outer part of the inlet channel thereby influencing the shape factor and indirectly by the supply of suspended material to the flow as well as extra bed material for bed load transportation and; increased thickness of the "rolling carpet." (Fig. 1) This in turn may cause an increase of the τ_s.

The influence of river discharge, Q_o

Consider $A = F (Q_m, Q_o, \text{------})$. In the estuary type inlet a river discharges through the inlet and this will change the relation between A and Q_m which as a first approximation will now have to be replaced by $Q_t + Q_o$ where Q_t is the purely tidal flow. A consequence of the fresh water discharge may be that flood and ebb currents because of density differences differ greatly in regard to current distribution in the vertical plane as described in (19) and (20). The head water run-off may result in a higher value of C and in a higher τ_s. Siltation may result because of the density currents as mentioned in (12). The density problems at estuaries and their influence on siltation and flow are mentioned in a brief report published in "Hydraulic Research, 1958, by the

STABILITY OF COASTAL INLETS

Hydraulic Research Station, Wallingford, England which distinguishes between two different types of estuaries; the"convective" type and the "salinity" type.

The time history of the inlet, t

Consider $A = F (Q_m, t, \text{-------})$. Detailed studies of inlet regimen have demonstrated that there is no single solution to the relative stability of a certain inlet but rather whole sets of solutions with different "degree of stability" depending upon how the various factors in equation (1) are put together and upon the "time history" and "age" of the inlet. The relative degree of stability

$$\text{Stab} = F \left(\frac{\Omega}{M}, \frac{Q_m}{M}, \tau_s \right)$$

mentioned later in this paper includes factors which all vary with time from the time the inlet was "born" and until it passed away because of various "diseases" or until it got a "heart attack" during one particular storm. This is elaborated further later in this paper referring to actual data.

RESULTS OF ANALYSES OF EXISTING DATA

Analyses of actual data demonstrated that the stability of the inlet gorge is usually better described by the ralation between A and Q_m than by A and Ω when Ω is the tidal prism. In his paper (17) O'Brien found the empirical relation

$$A = 1000 \left(\frac{\Omega}{640} \right)^{0.85}$$

in which the tidal prism Ω (in acre feet) is taken between mean higher high water and mean lower low water, (both typical characteristics of the U. S. West Coast) and A is in sq. ft. at mean sea level. The analysis mentioned below also uses spring tide range whether tide is diurnal or semi-diurnal. For inlets having a pronounced daily inequality, the stronger ebb currents maximum was used.

RESULTS BASED ON SHEAR STRESS ANALYSIS FROM TIDAL INLETS

The solid lines on Figs. 2 and 3 represent the relationship:

$$A = \frac{Q_m}{C\sqrt{\dfrac{\tau_s}{\rho g}}}$$

with τ_s constant 0.388 kg/m^2 or 0.080 lb/ft^2 , τ_s being the average shear stress over the cross-section of the channel.

With respect to the value of Chezy's coefficient C, it is apparent that although C primarily depends on bottom material and bottom formation,

the existence of shoals, inlet curvature, bank protection works, etc., will also influence C, but to a smaller degree. However, increasing the size of the inlets tends to increase the value of C as a result of an increasing value R/k (hydraulic radius over roughness parameter). Where a wide variety of inlet sizes is involved, as depicted in Fig. 3, variation of C has been taken into consideration. C values were determined through use of the fundamental logarithmic expressions as well as empirical knowledge of inlet characteristics. By a diagrammatic plotting of computed values for C the relation between C and A can be approximated as follows:

$$C = 30 + 5 \log A \qquad (A \text{ in } m^2 \text{ and } C \text{ in } m^{1/2} \sec^{-1})$$

or

$$C = 45 + 9 \log A \qquad (A \text{ in } ft^2 \text{ and } C \text{ in } ft^{1/2} \sec^{-1})$$

The above empirical relation gives average values. In some of the Dutch tidal rivers values for C of 68-70 $m^{1/2}$ $\sec^{-1}$ had to be introduced to bring computed values on tides in agreement with observed conditions.

The slight curvature of the solid lines on Figs. 2 and 3 is caused by variation in C, τ_s being a constant. It can be seen that considerable individual deviation is caused by variations in average stability shear stress τ_s, as well as in C. For plottings above the average line, the cross-sectional area is amaller than according to the average conditions which means higher velocities and a consequent higher value of the stabil ity shear stress. Meanwhile it can be seen that for the inlets considere the deviations in $\sqrt{\tau_s}$ are usually within the 10 per cent limit.

As mentioned earlier, the following factors will influence the stability shear stress, and thus the Q_m/A ratio:

Shape factor	β	Wave action	W_a
Soil condition of bed	B	Littoral drift	M
Sediment concentration	$\bar{c}$	Fresh-water discharge	Q_o

Because each example includes certain observation and computation deviations, it is not deemed possible to explain all individual deviation In some cases, however, a particular factor may be the main reason for the deviation. The shape factor β probably plays an important part in th actual value of the Q_m/A relation at Longboat Pass and Little Pass, Flori Gulf coast. At present Longboat Pass has a rather narrow and deep inlet and the whole cross-section is intensively used for the flow. Littoral-drift material, coming from both sides, is probably responsible for the steep slopes of the gorge which, in turn, cause a shear stress higher tha average.

Contrarily, Little Pass has a very irregular cross-section, parts of which, because of shoals, carry only a relatively small amount of flow. An uneconomical cross-section results in a lower τ_s than average.

The results of computation for the Absecon Inlet in Pennsylvania, based on detailed surveys by the U. S. Corps of Engineers, Philadelphia District, since 1880, are depicted by cross marks on Fig. 2. The shape
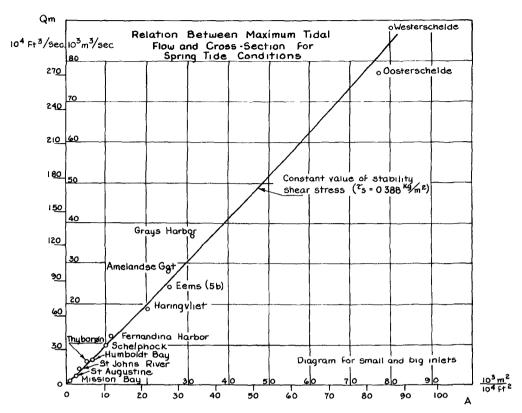
Fig. 3. Relation between maximum tidal flow and cross-section for small and big inlets at spring tide conditions.
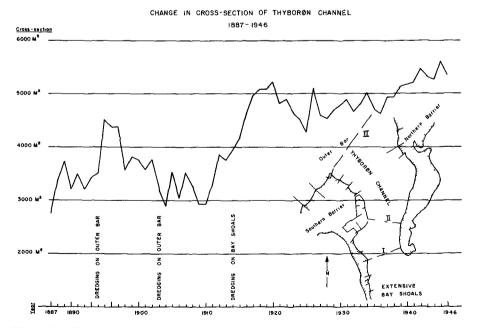

Fig. 4. Development of the gorge of Thyborøn Inlet on the Danish North Sea coast.

of the gorge is characterized by steep side slopes like Longboat Pass. This results in a higher τ_s which, for spring tide conditions seems to be approximately 0.63 kg/m2 (0.129 lb/ft^2).

A few of the tidal inlets studied are located in areas with a diurnal tide as, e.g., on the west coast of Florida and the Texas coast. The following passes belong to this group: East Pass, Florida; Port Aransas, Texas; and Calcasieu Pass, Louisiana. For those inlets Q_m has been plotted in the same way as for other inlets with a predominantly semidiurnal tide. It can be seen that the data belonging to the three inlets mentioned are scattered around the average relationships. East Pass is at the lower side of the line and the other two at the upper side. Because of the difference in tidal cycle a slight difference in the behavior of those inlets is not surprising. The period during which high velocities occur is considerably longer, but the length of the slack water period, during which depositing of material takes place, is longer also.

In regard to the influence of bottom material or soil conditions, B, many inlets run through littoral drift barriers, which means that the bed material is sand. As can be seen from Table 1, there is little difference between the limiting shear stresses for sand of 0.1 to 0.5 mm, but it must be remembered that the grain size influences the development of the bed configuration which, in turn, affects Chezy's coefficient C and, thereby, the quantity of flow. Meanwhile, the τ_s value is influenced by sediment concentration as described below.

The possible influence of wave action W_a, sediment size and concentration of suspended material $\mathcal{C}$, and littoral-drift M on the results depicted in Figs. 2 and 3 can best be discussed as a unit.

Wave action increases bed-load transportation as well as suspended load concentration and transportation. Outside the area which is directly influenced by tidal currents to and from the inlet, the bed-load transportation caused by wave action will depend on the actual mass transport of water which is rather limited, but in tidal entrances wave action may considerably increase bed-load transport by tidal currents. In this way wave action tends to decrease the stability shear stress.

The influence of wave action on suspended load transportation will often be considerable, particularly when material from the littoral drift is carried to the inlet and its tidal currents. Generally speaking, the smaller the grains and/or the heavier the wave action, the more material will be suspended in the flow.

On the United States east coast, the south shore of Long Island and the coast between Sandy Hook and Barnegat Inlet, New Jersey, have heavy wave action and a high average grain size (0.4 - 0.5 mm), while Daytona Beach, Florida, has a more moderate wave action and smaller average grain size (0.2 mm). The Gulf coast in general has light to moderate wave action and an average grain size of less than 0.2 mm, while the Pacific coast has moderate to heavy wave action with an average grain size of 0.2 - 0.3 mm.

One could, therefore, expect to find a tendency to larger inlet entrance cross-sections on the Pacific -- and perhaps on the Gulf coast -- than on said part of the Atlantic coast, but the data available give no clear indications of this.

The maximum cross-section of inlets with considerable wave action is, usually found where the suspended load transportation is at its maximum, despite the fact that sediment load to some extent tends to increase the value of the stability shear stress. The decrease of τ_s caused by the wave action in the vicinity of the entrance, seems to be much more important.

The influence of wave action directly, as well as indirectly, on τ_s seems to be visible at some of the examples, as at Thyborøn Inlet on the Danish North Sea coast.

This inlet, cut by nature in 1862 and navigable a few years later, was continuously bothered by a big offshore bar with a controlling depth of only 10 ft. The bar was the result of heavy wave action and heavy littoral drift coming to the inlet entrance from both sides. About one million cu. yd. of sand material a year is transported into the inlet and deposited on extensive bay shoals. Fig. 4 shows the variation of the gorge of the inlet during the period 1887 to 1946. After 1892 important dredging operations were started on the outer bar and this factor is clearly reflected in the diagram as increasing cross-section. Since it became difficult to keep up with the extremely heavy littoral-drift deposits, a different strategy was adopted later. Dredging was transferred to the bay shoals channels and is now done there entirely. The result has been that the controlling depth on the outer bar increased to 15 feet and is now at least 20 feet. Construction in the early 1920's of a 3,000-foot-long jetty (recently repaired and extended on the northern barrier at the inlet) further improved this situation.

The inlet channel gradually adjusted itself to the actual flow and "τ_s" situation. Fig. 4 indicates that the gorge area is below average size; and, taking into account the approximately one million cu. yd. of sand material carried each year through this cross-section for depositing on the inlet shoals, it seems likely that the stability shear stress has increased because of the heavy material load and possibly the accompanying changes in friction factor. Since the gorge has very steep slopes, the shape factor, as compared to other inlets, may also have improved.

In comparing the gorge Cross-section I, Fig. 4, with Cross-sections II and III situated closer to the entrance, some interesting tabulations will be noted in Table 2. Compare Table 2, where τ_s under medium concentration of sediment transport is 0.45 kg/m^2 , and remarks on the Eems Estuary, Holland later in this paper (Fig. 6).

The variation of cross-sectional area of improved inlet channels is dealt with in Fig. 5 where, for a number of inlets with parallel jetties, the cross-section has been plotted along the length of the inlet channel in a dimensionless diagram.

The cross-sections (A) at different locations have been divided by the cross-section at the entrance (A_O) to obtain a dimensionless ratio, using the relative distance from the seaward end (x/L) as second parameter.

From Fig. 5 it can be seen that the entrance cross-sections generally are greater than the cross-sections in other parts of the channel. The presence of wave action apparently has decreased the stability shear stres τ_s for the tidal currents because the orbital velocities of the wave action along the bottom of the channel increase the actual shear stress values, resulting in greater cross-sections near the entrance of the channel.

The importance of variation of bottom friction and its relation to sediment transportation is further elaborated in (6) which also includes some information on the influence of freshwater flow which it for space limitations is not possible to include here.

Average stability shear stresses determined from studies of existing data are given in Table 4. Limiting values for stable channels were mentioned in Table 1.

In practically all tidal inlets the median size of material as mentioned earlier is between 0.1 and 0.5 mm. According to Table 1 there is only a minor variation in the limiting shear stress for this range of grain sizes. The same will probably hold true for the stability shear stress τ_s. Taking 0.2 mm as a diameter for comparison, the limiting values for the shear stress for canals in fine noncohesive materials seem to range between 0.052 lb/ft^2 for light load and 0.078 lb/ft^2 for heavy load of sediment. The average stability shear stress for tidal inlets seems to range between 0.072 and 0.103 lb/ft^2 .

Special conditions may raise the τ_s value considerably above average. Computations for the Absecon Inlet, gave a τ_s value of 0.63 kg/m^2 (0.129 lb/ft^2) for spring tide conditions. This figure is high but littoral drift is very heavy at Absecon Inlet--probably exceeding 500,000 cubic yards per year--and bed-load as well as suspended load transport through the inlet channel is high.

This is further elaborated in (6). The movable stability of inlets as compared to the absolute stability desired at certain canals are also dealt with in (6) with reference to Bretting's (2) and Lane's theories (15).

Estuaries are inlets with a river or waterway discharging through the inlet. In case the amount of head flow passing through the estuary is so small that its influence on the vertical distribution of flow velocity is only minor the tidal hydraulic aspects of the inlet stability can be handled as with normal tidal inlets in alluvial materials. An example of an estuary is the entrance to River Eems at the border between Holland and Germany. In 1952 an extensive program of investigation was
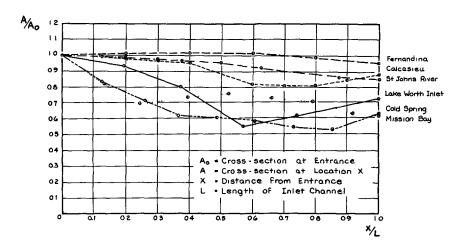
Fig. 5. Cross-sectional areas below M.L.W. for some jetty-improved inlets.
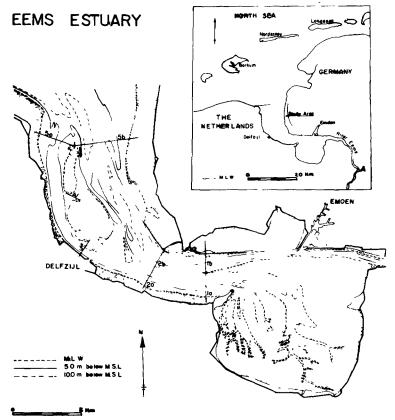

Fig. 6. The Eems Estuary, Holland.

undertaken by the Dutch "Rijkswaterstaat" to obtain information leading to the improvement of the estuary for navigation. Reference is made to (10).

The study included the following items:

a. Velocity measurements in the lines, indicated 1a, 1b,... through 5b in Fig. 6. Measurements were taken in every profile simultaneously in 5 to 8 locations. The vertical velocity distributions were determined by an Ott propeller-type current meter at 57 locations, which allowed rather precise determination of flow quantities and distribution of flow. Fig. 6 shows computed values for the maximum flood and ebb flow plotted agains the profiles surveyed at the same time.

b. Measurements of sand and silt content. Samples were taken at the surface, about 1/2 ft. from the bottom and about 1/3 of the depth from the bottom. Silt and sand concentrations were highest in line number 1. The shallow tidal bay called "Dollart", east of line 1, with extended mud flats, is responsible for this. Measured values of maximum concentrations (volume-ratio) in line 1 near the bottom are:

$$\text{Sand} \quad\quad 7.6 \times 10^{-4}$$
$$\text{Silt} \quad\quad 64.0 \times 10^{-4}$$

In lines 2 through 5 the respective concentrations were much less. For sand and silt the maximum concentration near the bottom varied between 0.6×10^{-4} and 4.4×10^{-4} in volume ratios.

c. Bottom samples were taken at the velocity measurement point. Table 5 shows values for d_{50} (diameter for 50% finer).

The information obtained seemed to be useful for examination of the stability of different bottom profiles.

Fresh-water discharge probably will play only a negligible part in the stability conditions; only in line 1b and possibly, in 1a, may it have some influence. During the period of observations river discharge amounted to 2 million m^3 during a flood or ebb period, which is a very small quantity compared with the tidal prism which amounted to 83 million m^3 during ebb tide under average conditions in line 1b. For line 5b a tidal prism of about 300 million m^3 under average flood conditions has been determined.

Re a The solid line in Fig. 7 indicated the relationship:

$$A = \frac{Q_m}{C\sqrt{\dfrac{\tau'_s}{\rho g}}}$$

τ'_s is the stability shear stress referring to mean tide conditions. As in Figs. 2 and 3 a variable C value has been introduced according to: $C = 30 + 5 \log A$ (metric system). Fair agreement obtained between
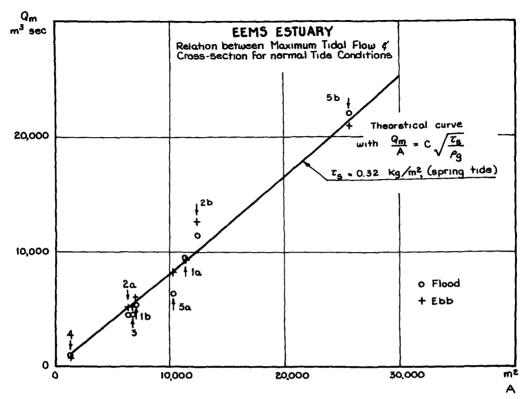
Fig. 7. Relation between maximum tidal flow and cross-section of Eems Estuary, Holland, for normal tide conditions.
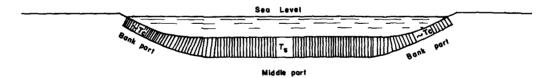

Fig. 8. Relative allowable shear stress for a tidal inlet cross-section.

observations and stability shear stress hypothesis is valid for mean range of tide conditions.

$$\tau'_s = 0.264 \text{ kg/m}^2 \quad \text{or} \quad 0.0534 \text{ lb/ft}^2$$

A constant value of τ'_s for this area seems reasonable because material load conditions are similar and the grain size of the material does not show much variation.

In order to compare the τ'_s value for the Eems with the τ_s values of Table 4 , the Eems value must be converted into spring tide conditions For this particular area of the North Sea coast, flow values during sprinj tide are about 10 per cent higher than during normal tide conditions. Because $\tau \sim v^2$, the value of τ_s for spring tide conditions will be $\tau_s = 0.264 \times (1.1)^2 \text{ kg/m}^2 = 0.320 \text{ kg/m}^2$ (or 0.0653 lb/ft^2). Because of the sheltered location of this area (no or little wave action) this figure is considered close to a minimum value ror τ_s (ref. Table 4).

In comparing the individual experimental data with the average curve it is seen that most of the individual plottings coincide rather well with this curve. Meanwhile, the profiles No. 2b and 5a show some remarkable deviations. The following may be a possible explanation for those deviations:

It is known from the investigations that profile 2b has a relatively high sediment load. This means higher stability shear stress and comparatively smaller cross-section. A favorable shape factor may also lead to a higher value of τ_s.

In many profiles there is no significant difference between ebb flow and flood flow, but the situation is different in line 5a for which the maximum flood flow is considerably lower than the maximum ebb flow. Profile 5 a is a typical (so-called) "ebb-channel" in which the ebb currents dominate the flood currents. In this case the plotting for the maximum ebb flow fits the average curve; for the maximum flood it does not. This may lead to the conclusion that where either the ebb or the flood current dominates, the predominant current will determine the size of the cross-section profile with its characteristics of silt and material load.

Section 5.6 of reference (6) gives information on some very interesting comparisons by the suthors with some results of Leopold and Maddock's studies of the geometry of stream channels and some of its physiographic implications. The relationship $V_m = \text{constant } Q^{0.1}$ or $A = \text{factor} \times Q^{0.9}$ is confirmed and compared to Bretting's theory which also has $A \sim Q^{0.9}$ (2)

THE RELATIVE STABILITY OF TIDAL INLETS

The problem of stability of an inlet can be considered in the "horizontal" as well as in the "vertical" plane. Horizontal (or location) stability is dealt with in (6). Speaking about the cross-sectional stability this problem can be said to include two kinds of stability:

the material transfer "stability" or "ability" and the cross-sectional stability described in this paper.

Material transfer stability is described by the authors in (5) and (6) which distinguishes between two different kinds of material transfer across inlets: bar-bypassing and tidal flow bypassing. If the predominant littoral drift (M_{mean}) is expressed in cubic units per year and the maximum tidal flow under spring tide conditions in the same cubic units per sec. (Q_{max}), by-passing may be described by the

$$\frac{M_{mean}}{Q_{max}} = r \quad \text{factor.}$$

Analysis demonstrated that:

with $r > 200 - 300$ we usually have bar by-passing

with $r < 10 - 20$ we usually have tidal flow by-passing

The mechanics of the by-passing and man-made influence on it is dealt with more detailed in (5) and (6).

In regard to the cross-sectional stability the relative "degree of stability" as mentioned earlier is tentatively expressed as follows:

$$\text{Stab} = F \left(\frac{\Omega}{M} , \quad \frac{Q_m}{M} , \quad \tau_s \right)$$

where the factors Ω, Q_m and τ_s are interrelated and depend on inlet and bay geometry, character of bottom soil, material load, and wave conditions. Of these factors, M may not vary much with long-range time, while Ω, Q_m and τ_s most likely will vary from the moment an inlet is "born" until it develops "full-size" and stabilizes itself before deterioration. This period may be a matter of decades or centuries and it therefore seems allowable to speak about a "number of stability solutions" which have varying degree of actual stability. It should also be remembered that the (spring tide) values of Ω and Q_m may fluctuate somewhat due to variation in tide characteristics.

Upward, the number of these stability possibilities is limited by a certain maximum tidal prism whereby for small bay areas the tidal range in the bay or lagoon equals the tidal range in the sea. The gorge cut in alluvial material will then assume its maximum size, while the actual value of τ_s will depend upon material, littoral drift and other factors as discussed earlier in this paper. Downward, the number of solutions is limited by a minimum cross-sectional area which is determined by certain minimum values of Ω/M and Q_m/M. Anyone who has worked with problems of "choking inlets" on littoral-drift coasts (5) knows that a newly opened or a natural break-through inlet is bound to close again rather soon unless the channel has attained a certain minimum cross-sectional area.

The value of τ_s may also be considered descriptive in the actual stability situation. Relatively higher values of τ_s may indicate good flushing action, and, thereby, better stability conditions. Contrarily,

401

lower values of τ_S may indicate less cleaning ability of the inlet flow and beginning or advancing deterioration of the inlet channel, perhaps associated with an uneconomical shape of cross-section (bad β- factor).

It is difficult to give specific values for the relative stability of inlets; to do so requires detailed knowledge about the time history of the inlet gorge, and such information is usually not fully available. Table 6 gives information concerning several tidal inlets for which we have some idea about the Ω/M and Q_m/M ratios from many years of deposits and dredging operations undertaken because of the importance of these inlets to navigation.

Considering first the Ω/M ratio the tidal prism Ω represents the total amount of flow passing through the inlet during one half tidal cycle. For the greater part of the time this flow is able to transport material and to clean the inlet of "surplus deposits".

In Table 6 the values of Ω/M are listed together with Q_m/M values and computed values of τ.

Keulegan's (14) expression:

$$\Omega = \frac{Q_m T}{C_2 2\pi}$$

relates the tidal prism Ω to the coefficient C_2, the maximum discharge Q_m and the length of the tidal period T.

The value of C_2 generally deviates less than 20% from unity. The tidal period T may correspond to semidiurnal or diurnal tides.

For inlets with semidiurnal characteristics the yearly average value of M is used to compute the ratio Ω/M. For similarity reasons the factor for inlets with diurnal tide characteristics should then be computed as $\sim \Omega/2M$, because the length of the tidal period for diurnal tides is approximately twice as long as for semidiurnal tides. This has been considered by computing the Ω/M values in Table 6 and the comparisons made based on Ω/M values.

Consideration of the characteristics of the inlets listed reveals tha those having a ratio Ω/M in excess of 300 have a higher degree of stabilit Inlets with Ω/M ratios <100 seem to belong to that category which have a more predominant transfer of sand on (shallow) bars or shoals across the inlet entrance and less significant tidal currents; for this reason they may be rather unstable and are usually characterized by one or more narrow, frequently shifting channels with high velocity through shoals with shallow water as described in (5).

It is not possible to say where a transition ratio of Ω/M between stable and unstable inlet channels may lie because irregularity in quantit as well as in direction of the littoral drift will likely make it impossi-ble to establish such fixed ratio. Some inlets still have a fair stabi-

lity for Ω/M ratios of 150 - 200, e.g., the Thyborøn Inlet, Denmark, which has to be dredged somewhat on its bay shoals. Compare Fig. 4 which shows the variation of the gorge of Thyborøn Inlet. Before 1910 the Ω/M ratio was < 100 and at that time dredging had to be carried out continuously on the outer bar. After 1910, when the Ω/M ratio was > 150, dredging on the outer bar became unnecessary; but as mentioned earlier some (minor) dredging operations had to be and are still being carried out in the bay channels and on the bay shoals where material carried through the inlet is deposited by the flood-currents.

In regard to the ratios Q_m/M mentioned in Table 6, the following can be said.

It is well known that only a certain (usually unknown) fraction ΔM of the longshore littoral drift M enters the inlet channel itself. The relation $\Delta M/M$ is not constant but assuming a certain similarity in inlet behavior we may expect that the relation M/Q_m (or Q_m/M) has an influence upon the stability shear stress values.

Table 6 suggests that Q_m/M ratios > 0.01 averagely present a more stable situation than ratios < 0.01.

The stability shear stress τ_s refers to the maximum tidal flow under spring tide conditions. If tide conditions were the same for all inlets considered an equal τ_s would result if all other conditions were equal. In this study inlets with quite different tide characteristics were used and this inevitably leads to local deviations in the stability shear stress.

Values for the stability shear stress were computed for various inlets according to the relation:

$$\tau_s = \rho g \frac{v_m^2}{C^2}$$

whereby v_m is the maximum value of the average current velocity during spring tide conditions, and C the Chezy coefficient.

Uncertainties in the values of v_m as well as C are introduced into the formula in the second power so that a very close determination of τ_s for most of the inlets studied was not possible. The effects of this are demonstrated in relatively strong variations of the τ_s values as indicated in Table 6. The large scale tendency of τ_s is indicated in Table 4.

Fortunately the lower limit of τ_s could be approached in the case of the estuary of the Eems as mentioned earlier in this paper.

The above should be remembered when considering some details on τ_s values included in Table 6, which is further developed in Tables 7 and 8, wherein Ft. Pierce Inlet (Florida), Averio and Figueira Da Foz Inlets (Portugal), and Gasparilla Pass (Florida), are omitted; Ft. Pierce Inlet because of some rock formation in the inlet channel; Averio Inlet

because its improvement by jetties was only completed in 1958 and experience is insufficient; Figueira Da Foz and Gasparilla Pass because they—as demonstrated by their τ_s values——— are in an unstable condition.

From Tables 7 and 8 it is apparent that large inlets, such as some of the big Dutch inlets and those like Grays Harbor, Washington, are characterized by an average τ_s value of about 0.46 kg/m^2 (0.094 lb/ft^2) with $\Omega/M \geq 600$ or $Q_{max}/M \geq 30 \cdot 10^{-3}$. Inlets with a more modest flow activity as compared to the littoral drift quantity, such as the minor Dutch inlets, Longboat Pass, Florida, and the diurnal type inlets on the Gulf of Mexico have τ_s values averaging about 0.50 kg/m (0.102 lb/ft) with $150 < \Omega/M < 600$ or $10 \cdot 10^{-3} < Q_{max}/M < 30 \cdot 10^{-3}$. The slight tendency towards increase may be explained in the way that littoral drift deposits are encroaching upon the inlet channel at these inlets partly improving its shape factor and partly increasing material movement both, in turn, increasing the τ_s.

Inlets with a still lower flow over littoral drift value, such as Big Pass and Ponce De Leon Inlet, Florida, and the now dredged Mission Bay, California, have an average τ_s value of about 0.51 kg/m^2 (0.104 lb/ft^2) with $\Omega/M \leq 150$ or $Q_m/M \leq 10 \cdot 10^{-3}$. There may be a tendency toward increased concentration of currents in the gorge caused by littoral drift deposits which the currents may have trouble keeping up with. Inlets within this group all have considerable bay and/or sea shoals.

Comparing the values of Tables 7 and 8 with the values of Table 1 it is also interesting to note that the most important difference apparently lies in the definition of "stability". While Russian standards indicate "critical" tractive forces in 0.2 - 0.5 mm material of about 0.15 kg/m^2 (0.03 lb/ft^2), Schoklitch, who recommends 0.39 kg/m^2 (0.08 lb/ft^2) for canals in fine sand, apparently counts on material movement and thereby on a unidirectional "rolling material carpet" (Fig. corresponding to the two-directional "rolling carpet" at littoral drift inlets. In the unidirectional canal flow there is no pouring in of materials from the sides, as with tidal inlets on littoral-drift shores, which may contribute to a "raise" of τ_s from about 0.39 kg/m^2 (0.08 lb/ft^2) in canals to about 0.50 kg/m^2 (0.103 lb/ft^2) at tidal inlets on littoral drift shores.

The τ_s values for tidal inlets mentioned above still refer to the maximum discharge Q_m during spring tide conditions which theoretically only occurs for a few seconds or minutes every half tidal cycle, but in practice may run for 2-3 hours. It is apparent, however, that it is the maximum velocity (and shear stress) which at tidal inlets determines the cross-sectional area which does not adjust itself to any lower value of τ_s, but leaves surplus cross-sectional area for lower velocities and quantities of flow. (Compare the Eems estuary) Because material movement almost stops at velocities lower than 1 ft/sec it would be of less

physical significance to relate the stability shear stress to an average stress over a tidal cycle.

Comparing this situation with the situation at rivers it can be said the difference lies in the time history. A river normally has ample time to adjust its cross-section to a given flow; inlets are "short of time" and tend to follow the most frequent "extremes" (peak tides).

DESIGN OF TIDAL INLETS ON A LITTORAL DRIFT SHORE

Experience has demonstrated that we are coping with a problem which involves many variables. These variables can be combined in different ways. Based on our present knowledge it is not possible to give a univalent answer to any particular problem.

However, under the assumption of noncomplex boundary conditions for a given "desirable cross-section" of simple geometrical shape, we can evaluate the size of the tidal flow necessary to keep this cross-section fairly stable, taking into consideration the actual situation of tides, bottom material, littoral material, suspended load, and different determining friction elements. If the required amount of flow is available, the desired combination of tides, bay, and inlet character-istics can be secured. Tidal hydraulics computations are necessary since they give the relationship between flow and inlet characteristics. Regime considerations determine which of the different possible combina-tions will produce the most "stable" condition.

Certain data are available for use in the "preliminary design". The design is usually based on average conditions, and nature does not always respect the "mean". Heavy storms may pour littoral-drift deposits in the inlet channel regardless of how ideal and well designed its cross-section and configuration are. Consequently, with little notice the inlet may be forced into a new situation where the tidal flow capacity will change because of the decrease in cross-sectional area. Luckily such littoral-drift deposits are almost never distributed equally over the inlet bottom but will usually accumulate at one side (often on the inside of the up-drift jetty). The result may be a concentration of flow which tends to remove (shave off) the deposit. If bed-load material from the littoral drift is furnished to the shoal at a rate which makes the inlet currents unable to wash the deposit away, these deposits will have to be removed by dredging; otherwise, the inlet may close. Model experiments may indicate that such a situation can be taken care of partly or wholly by a design which gives the entrance an "intelligent shape" for cleaning of deposits. If the inlet is of considerable size and has the right configuration in the horizontal as well as the vertical plane, the condition may develop in which depositing occurring at any flood tide is washed away at any following ebb tide because the cross-section allows the inflow of enough water to provide ample flow for adequate cleaning. However, apart from special cases such as the lagoon harbor at Abidjan on the Gold (West) Coast of Africa, such a condition may not persist because the material removed by the ebb current might be so deposited in front of the inlet that the resistance against the inlet flow will gradually increase; and the

405

inlet, left to itself, may finally deteriorate.

The important factors which have to be considered for any inlet
design are:

1. Size of the inlet gorge A, compared to the tidal
    prism Ω and maximum discharge Q_m

2. Geometrical shape of the inlet channel

3. Design shear stress

In order to secure a stable inlet a certain amount of tidal flow is
necessary, which means Ω and Q_m of proper magnitudes as compared to
the total littoral   drift   M at the inlet entrance. Q_m and Ω for a
given inlet geometry can be determined by tidal hydraulics computations
(7, 14). If a Ω/M ratio > 200 is not obtained the inlet probably will
not develop the desired stability as explained earlier.

For the actual dimension of the inlet, requirements of navigation
will establish the lower limits for the cross-sectional area. This
limit may be satisfactory if it presents a reasonable Ω/M (and/or Q_m/M)
ratio. If it does not, other problems than that of stability may be
created simultaneously in the bay or lagoon, such as floods caused by
too slow discharge of heavy rains, stagnancy, and problems of marine
biology nature. The fish-killing "Red Tide" on the Florida lower Gulf
coast has better opportunities for development in areas with insufficient
exchange of water between the area and the open sea.

If, because of problems of stagnancy for example, it is necessary
to increase Ω, no problems other than those of an economic character
may exist. From the hydraulic standpoint the inlet's stability will be
improved by increasing Ω. The situation is different if, because of
danger of flooding from the sea, it becomes necessary to decrease Ω
below the desirable value for obtaining a satisfactory Ω/M ratio for
stability. In such case provision must be made to secure the highest
possible utilization of the available Ω and to minimize or equalize M to
avoid high peaks of drift deposits which cannot be absorbed by the avail-
able sand traps and flow quantities.

A higher utilization of Ω can be obtained by securing the best
possible distribution of flow in the inlet, which means the most advanta-
geous distribution of τ over the cross-section. Proper jetties and
"canalization" of the inlet may secure the desired result. If the de-
crease in Ω (still considering a practical Ω/M ratio), cannot be obtained
in this way, it is possible, to decrease M materially by "sand traps",
possibly arranged as a bypassing sand plant permanently installed or
by a continuous dredging arrangement. Ultimately it may be necessary to
decrease the cross-sectional area of the gorge beyond the stable condition
for loose sand bottom, thereby inviting erosion by high current velocities
In order to avoid such erosion it may be necessary to provide the bottom
with a protection layer, which may be rock or specially built mattresses,

or nylon or plastic sheets loaded down or otherwise fastened to the bottom. Undesirable high-tide velocities can also be avoided by making the channel (very) long and/or providing it with friction arrangements. Often it is also necessary to protect the bottom of canals against currents caused by ships or ship screws. Velocities above 4 - 5 ft/sec are usually not desirable for reasons of navigation, and if maximum velocities are expected to exceed 5 - 6 ft/sec it may be necessary to take more radical measures, e.g., cutting off part of the tidal bay area if the configuration of the bay is such that it can be done without unreasonable expense. A cut-off dam, however, may raise a number of questions such as the establishment of sluices or other regulating works. The easiest thing, therefore, may be to build sluices in the inlet itself. This method offers various advantages such as better navigation conditions in the inlet inside the sluices, no necessity for bottom protection, no floods from the sea, and little or no sand deposit on bay shoals. Furthermore, such "closing" of an inlet may improve beach erosion conditions on the seashore (3). The disadvantages include an unstable inlet entrance subject to accretion, delays to navigation, and deposits on the seaside of the sluices which may cause difficult and costly dredging operations in improving navigation conditions and possibly the necessity for expensive longer jetties to protect the inlet from excessive littoral deposit. Economic analysis of the problem as a whole, however, may still justify such measures. The Netherlands presents a good example of such project at Ymuiden, the seaport of Amsterdam with the world's largest navigation locks.

An inlet channel should always be designed with a cross-section of simple geometrical nature, usually trapezoidal or rectangular, depending upon the structural character of the improvement itself. The question is how cross-sections should be designed when most stability is desired.

In practice one is usually faced with two situations--the construction of a new inlet, or the improvement of an existing one. Special requirements in regard to navigation necessitate certain dimensions and a certain shape and alignment of the navigation inlet. For navigation reasons, maximum peak current velocities as mentioned above should possibly be kept below 4 - 5 ft/sec; for stability reasons the shear stress at the bottom should be commensurable to the stability τ_s which depends upon the shape of the cross-section as well as on soil conditions, material load, wave action and possibly head flow.

A straight inlet channel is mostly to be preferred for reasons of navigation. Meanwhile, the entrance area may for reasons of littoral deposits have to be curved and it is moreover an experience that a curved (or slightly meandering) channel where the designer has a better chance of determining the flow pattern--instead of letting nature do it--is preferable.

Establishment of a proper design shear stress τ_s under the actual conditions corresponds to the establishment of rather the "ultimate strength" than the "allowable strength" in the field of solid mechanics in as much as we have to count on a material movement on the bottom of the inlet as a necessity in order to keep a certain cross-sectional area. The τ_s should not increase beyond this value.

The stability shear stress τ_s under the actual conditions of soil, material-load, wave action and possibly head flow, is mentioned. Values have been obtained which can be used as a guide for preliminary design. The composition of bottom material and the relative amount of littoral drift may be of considerable importance as in determining a proper τ_s. (Table 4) In inlets with only light littoral drift the smaller fractions of the bottom material may be carried away, leaving comparatively coarser particles behind. Compare the situation in canals and channels.

The finer particles will usually be deposited in bay shoals. In inlets with considerable littoral drift, bottom material under "calm weather conditions" may correspond to the material at similar depths on the seashore, while under or after more extreme current conditions the material may tend to be a little coarser because of selection by the currents. Badly sorted material on the seashore may eventually cause the inlet bottom to be composed of coarser material than the average on the seashore.

The tidal flow, released from its content of finer material on the bay shoals, will return to the sea in "purified condition" and may cause a higher shear stress at the bottom which will carry mainly finer particles seaward, away from the inlet channel. This means that the design shear stress for the inlet channel may often correspond to this ebb tide situation, which in turn may result in a slightly larger cross-section than that corresponding to flood flow.

Table 1 mentioned earlier, indicates the variation in τ_s because of grain size to be of limited importance. Supply of littoral drift to the entrance is apparently a more important factor because it brings suspended and bed material load into the tidal currents which may change friction characteristics of the flow as well as bottom roughness parameters. Heavy littoral drift is almost always connected with heavy wave action and the influence of the same amount of littoral drift on the inlet stability increases with relatively decreasing flow capacity of the inlet. In this connection it should always be remembered that it is the total quantity of material brought into the channel (from both sides) that counts, not the resultant predominant drift in one direction.

Based on the results of this study the average values shown in Table 4 for the stability shear stress seem to be useful as a first approach and guidance to the design.

As mentioned earlier in this paper, a shape factor depending upon the actual form of the cross-sectional area should always be considered in connection with a proper τ_s. In case of narrow and deep inlets the τ_s values above may be raised 10 to 20 per cent.

Summarizing the above mentioned proper practical approach to the design problem may be outlined as follows:

408

(a) A proper cross-section and the general channel alignment is determined by considering navigational requirement. The channel length is estimated by practical considerations.

(b) Preliminary computations of Q_m and Ω are then carried out based on a proper average τ_s value. Q_m/M and Ω/M ratios are considered as explained above.

(c) Isovels are then constructed, e.g., according to theories by Lane (15) or Olsen and Florey (18) for the entire cross-section including the bank part which amy be designed as nonerosive by trial and error using, e.g., Bretting's theory (2) as a guide.

In this connection careful consideration should be given to the difference between the "stability" shear stress τ_s to be used for the horizontal bottom part and the "critical" shear stress τ_c to be used for the bank part. (Fig. 8)

In regard to values of τ_c, see reference (15). Practical average τ_s values are listed in Table 4.

(d) After adjustments have been made for a "final cross-section", detailed tidal calculations including determination of actual Q_m, Ω, and τ values are carried out (7, 14). Those computations in combination with "regime-considerations" will tell at what length of the inlet channel a stable condition is obtained wherein the stability shear stress will be reached (but not exceeded) under the conditions given. The Ω/M and Q_m/M values are then checked again and if the channel length obtained in this way is too long, correction may be possible by decreasing the cross-sectional area of the inlet and/or by friction arrangements although this procedure may involve some adverse effects on navigation as well as on economy. If the calculated channel length is too short, the channel can be extended to a practical value by a corresponding increase of the cross-section. In some cases the length of the inlet channel will be given and the cross-section will then depend solely on boundary conditions such as tides in the ocean, bay geometry, and stability shear stress for the given material.

Every change in cross-section will require drawing of now isovels, corresponding adjustments according to given τ_s and τ_c values and, finally, repetition of tidal computations.

If the inlet cross-section is improved by man-made structures such as jetties, the design procedure of drawing isovels should also be followed. In some cases, and particularly when rubble mound (rugged) jetties are considered for the improvement of the inlet, some difficulties will be involved in determining the isovels in detail by theoretical methods. In such cases model experiments will be of great value. Meanwhile, it must be remembered that the determining shear stresses can be increased by proper channel bottom protection and friction arrangements. This can be investigated in detail by model experiments. Here the design cross-section should first be tested with fixed bottom in order

to compare the estimated discharge with the model flow; next, the tests should be run with proper bed material, e.g., perspex sand, baklite or gilsonite (siltation test). It can then be determined whether the flow will tend to create major irregularities in τ distributions, causing shoaling and/or erosion which require proper changes of its area and/or shape or protective measures on the bottom.

The contraction of inlet flow at the entrance as well as in the bay should also be taken into consideration. Reference is made to French's progress reports (9) dealing with the velocity distribution in tidal entrances. The part of the inlet cross-section outside the contraction zone is useless for flow and should not be included in the calculations. Through proper entrance design those deadpockets caused by the contraction can be partly or wholly eliminated.

In dealing with problems of fresh-water outflow and density currents special attention must be paid to the new conditions; depending upon the degree of changes in flow and velocity patterns, it should be determined whether a thorough model study based on detailed and accurate field surveys is essential to obtain a reliable picture of the flow conditions useful for design.

CONCLUSIONS

1. Investigations of existing data on tidal entrances in America, Denmark, Holland and Portugal have given considerable information of a general nature on the relationship between inlet characteristics.

While in earlier publications on this subject (4, 17) the tidal prism had been used to characterize flow conditions of the inlet, in this publication the maximum rate of flow per second during the tidal cycle Q_m has been used to describe the relation between flow and other inlet characteristics. Analysis demonstrated that Q_m is a better parameter than the tidal prism because the flow is directly related to the velocity and the latter to the bottom shear stress, τ, considering cross-sectional geometry.

2. The Ω/M ratio seems to be adequate for description of the actual "degree of stability". Investigations indicate that Ω/M ratios < 100 should be avoided and that ratios $\Omega/M > 200$ are preferable for inlets in sand material. The corresponding Q_m/M value should be >0.01 if possible.

3. The shape of the cross-section and the stability shear stress τ_s seem to be important factors for gorge stability considerations. Every new inlet to be dredged or any existing inlet to be improved by dredging or by jetties will probably be provided with a cross-section of simple geometrical shape, trapezoidal or rectangular. For this reason further studies, particularly those concerned with securing data useful for design, should be concentrated on the determination of τ_s which can be considered as a function of several variables. The most important

are: shape of cross-section, soil conditions of the bed, sediment load, wave action, littoral drift, and fresh-water discharge.

The suggestions for design of new inlets or improvement of existing inlets are based mainly on shear stress considerations. Shear stresses should be determined by drawing isovels for the flow. Use of the stability shear stress τ_s is recommended for the middle (horizontal) section of the channel and critical shear stress τ_c for the bank or slope part. Shear stress in the connecting part should increase from τ_c to τ_s.

Analysis of actual inlet data indicated an average value τ_s = 0.50 kg/m² = 0.103 lb/ft² . Values useful for preliminary inlet design are given in Table 4.

In comparing the "stability shear stress" with Lane's "limiting shear stress" (6, 15) it is found that the stability shear stress for tidal inlets as defined in this paper is higher than Lane's recommended values for the limiting shear stress in the design of stable channels.

4. In the future efforts should be concentrated on studies of the Ω/M and Q_{max}/M relationships to actual inlet stability under a great variety of conditions, studies of littoral drift and flow distribution in inlet channels and its relation to inlet geometry and studies of τ_s and its relation to the pertinent factors involved in inlet stability. Special hydraulic equipment as described in (6) and radioactive tracing technique may be of great value in securing such results.

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STABILITY OF COASTAL INLETS

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Table 1

LIMITING VALUES OF TRACTIVE FORCES, RIVER AND CANAL FLOW [(*)]
NONCOHESIVE MATERIAL
(lb/ft^2)

Median size of material, in millimeters	DESCRIPTION OF WATER		
	Clear water	Light load of fine sediment	Heavy load of fine sediment
0.1	0.025	0.050	0.075
0.2	0.026	0.052	0.078
0.5	0.030	0.055	0.083
1.0	0.040	0.060	0.090
2.0	0.060	0.080	0.110
5.0	0.140	0.165	0.185

(*) From: "Standards for Permissible Non-eroding Velocities",
    Bureau of the Methodology of the Hydro-Energo Plan, Moscow,
    1936.

Table 2

CROSS-SECTIONAL AREAS OF THYBORØN CHANNEL

Situation	Cross-section		
	I	II	III
m^2	5,000	5,500	8,000
yd^2	6,000	6,600	9,600
Material load per year	1 million cu. yd.	1 million cu. yd	1 million cu. yd.
Wave action	Light to moderate	Moderate	Heavy
τ_s	ab. 0.5 kg/m^2	ab. 0.4 kg/m^2	ab. 0.2 kg/m^2

413

Table 3

RELATION BETWEEN LENGTH OF INLET CHANNEL AND CROSS-SECTIONAL AREA BELOW MLW
FOR SOME JETTY-PROTECTED INLETS

Name of Inlet	Length L (ft)	Width (ft)	Cross-sect. Entrance A_o (ft²)	Cross-sect. at end of channel A_L (ft²)	$\frac{A_o}{W} = h$ (ft)	$\frac{W}{h}$	$\frac{A_L}{A_o}$
St. Johns River	4,000	1,600	62,000	52,500	38.7	41.3	0.847
Fernandina	2,500	3,900	124,500	118,800	31.9	12.2	0.954
Mission Bay before dredging	3,800	920	11,900	7,400	12.9	71.3	0.621
Cold Spring	4,100	850	21,000	13,250	24.7	34.4	0.630
Calcasieu Pass	5,000	1,000	11,750	10,300	11.75	8.5	0.876
Lake Worth	2,200	800	21,200	15,150	26.5	30.2	0.715

Table 4

STABILITY SHEAR STRESSES FOR TIDAL INLETS
BASED ON SPRING TIDE CONDITIONS

Condition	τ_s (kg/m²)	τ_s (lb/ft²)
Heavier littoral drift and sediment load	0.50	0.103
Medium conditions of littoral drift and sediment load	0.45	0.092
Lighter littoral drift and sediment load	0.35	0.072

Table 5

GRAIN SIZES OF BOTTOM MATERIAL IN EEMS ESTUARY, HOLLAND

Line	Range d_{50} (μ)	Average d_{50} (μ)
1a	80 - 365	134
1b	75 - 145	102
2a	60 - 130	86
2b	82 - 153	112
3	65 - 115	83
4	88 - 120	104
5a	79 - 97	86
5b	112 - 205	164

Table 6

FLOW AND LITTORAL DRIFT CHARACTERISTICS FOR SOME INLETS

Inlet (Kind of Improvement)	* Tidal Prism cu yd/half cycle	Q_{max} Maximum Discharge cu yd/sec	M** Predominant Littoral Drift cu yd/year	$\bar{M}$ or $\frac{-}{2M}$	$\frac{Q_{max}}{M}$ x 10^3	τ_s lb/ft^2 (kg/m^2)
Amelandse Gat, Holland (Bank stabilization on north side)	600 x 10^6	36,600	1.0 x 10^6	~ 600	37	0.103 (0 50)
Aveiro, Portugal (Jetties)	150 x 10^6	9,000[1]	0 75 x 10^6	~ 200	12	
Big Pass, Florida (None)	12 x 10^6	700	< 0 1 x 10^6	120	7	0 115 (0 56)
Brielse Mass, Holland before closing (Closed)	40 x 10^6	2,700	1 0 x 10^6	~ 40	3	0.086 (0 42)
Brouwershaven Gat, Holland (Will be closed)	430 x 10^6	30,000	1.0 x 10^6	~ 430	30	0 111 (0 54)
Calcasieu Pass, La. (diurnal) (Jetties and Dredging)	110 x 10^6	2,600	0 1 x 10^6	~ 550[2]	26	0 090 (0 44)
East Pass, Florida (diurnal) (Dredging)	60 x 10^6	1,720	0.1 x 10^6	~ 300[2]	1$^-$	0 111 (0 54)
Eyerlandse Gat, Holland (None)	270 x 10^6	19,000	1.0 x 10^6	~ 270	19	0 119 (0 58)
Figueira Da Foz, Portugal (Dredging)	20 x 10^6	1,200	0 5 x 10^6	~ 40	2	0 049 (0 24)
Fort Pierce Inlet, Florida (Jetties and Dredging)	80 x 10^6	3,700	0 25 x 10^6	~ 320	15	0.22 3) (1 07)
Gasparilla Pass, Florida (None)	15 x 10^6	900	< 0.1 x 10^6	> 150	9	0 051 (0 25)
Grays Harbor, Washington (Jetties and Dredging)	700 x 10^6	48,000	1.0 x 10^6	~ 700	48	0 105 (0 51)
Haringvliet, Holland (Being closed)	350 x 10^6	25,000	1 0 x 10^6	~ 350	25	0 070 (0 34)
Inlet of Texel, Holland (Stabilization of south side)	1400 x 10^6	115,000	1 0 x 10^6	~1400	115	0.094 (0.46)
Inlet of Vlie, Holland (None)	1400 x 10^6	110,000	1.0 x 10^6	~1400	110	0 090 (0.44)
Longboat Pass, Florida (None)	30 x 10^6	1,400	< 0 1 x 10^6	> 300	14	0 115 (0.56)
Mission Bay, California before dredging (Jetties and Dredging)	15 x 10^6	1,100	0.1 x 10^6	~ 150	11	0 127 (0 62)
Oosterschelde, Holland (Will be closed)	1400 x 10^6	100,000	1.0 x 10^6	~1400	100	0.084 (0.41)
Oregon Inlet, N. Carolina (Occasional Dredging)	80 x 10^6	5,100	1.0 x 10^6	~ 80	5	0 092 (0.45)
Ponce De Leon Inlet, Florida (None)	20 x 10^6	1,500	0.5 x 10^6	~ 40	3	0.098 (0 48)
Port Aransas, Texas (diurnal) (Jetties and Dredging)	65 x 10^6	1,900	0.1 x 10^6	~ 325[2]	19	0.098 (0.48)
Thyborøn, Denmark (Minor Dredging)	140 x 10^6	7,500	0.9 x 10^6	~ 160	9	0.10 (0.49)
Westerschelde, Holland (Some Dredging)	1600 x 10^6	115,000	1.0 x 10^6	~1600	115	0 092 (0.45)

** Total amount of littoral drift interfering with the inlet may deviate from this value if drift direction is not too predominant and/or the inlet is not improved.
* Spring tide
1) Increasing
2) ^{c}/2M
3) Rock gorge

Table 7

AVERAGE τ_s VALUES AS A FUNCTION OF DIFFERENT Ω/M VALUES

Ω/M	≥ 600	$150 < \frac{\Omega}{M} < 600$	≤ 150
τ_s kg/m^2	0.46	0.50	0.51
τ_s lb/ft^2	0.094	0.102	0.104

Table 8

AVERAGE τ_s VALUES AS A FUNCTION OF DIFFERENT Q_{max}/M VALUES

$\frac{Q_{max}}{M}$	$\geq 30 \cdot 10^{-3}$	$10 \cdot 10^{-3} < \frac{Q_{max}}{M} < 30 \cdot 10^{-3}$	$\leq 10 \cdot 10^{-3}$
τ_s kg/m^2	0.46	0.50	0.51
τ_s lb/ft^2	0.094	0.102	0.104