

CHAPTER 91

EVALUATION OF OVERALL ENTRANCE STABILITY OF TIDAL ENTRANCES

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ABSTRACT

This paper is written in continuation of earlier published material (2, 4, 5) dealing with stability of tidal inlets on littoral drift shores. The experience available at that time was responsible for the introduction of two parameters: $V_{\text{mean max}}$, defined as the mean max. velocity in the gorge at spring tide and the Ω/M_{tot} ratio (tidal prism at spring tide divided by material transport to the entrance from the adjoining shores) as the most pertinent parameters for description of overall stability. A more detailed justification for this choice is given in this paper, based on computation of the relative sediment transport at various tidal phases. Examples of earlier date (4) and twelve new examples from India are given.

DISCUSSION ON THE JUSTIFICATION OF DESCRIBING THE RELATIVE ENTRANCE STABILITY OF TIDAL INLETS IN ALLUVIAL MATERIAL BY $V_{\text{mean max}}$ AND THE Ω/M_{tot} RATIO

$V_{\text{mean max}}$ is equal to Q_{max}/A where Q_{max} is the mean maximum discharge at spring tide and A is the cross sectional gorge area at M.S.L. The Ω/M_{tot} ratio (Ω = tidal prism at spring tide, M_{tot} = the total quantity of drift carried to the entrance per year or other time period) is related to the overall stability of the channel (2, 4, 5). There is an almost linear relationship between $V_{\text{mean max}}$ and A . According to experiences (2, 4) large Ω/M_{tot} ratios make the entrance channel flush well, while low Ω/M_{tot} ratios cause the build-up of entrance bar(s) for material which is bypassing.

The question is: Do these two criteria for the description of stability present an oversimplification of a complex problem of the same character as some other (exponential) relation between the tidal prism and gorge cross-sectional area, ignoring the obvious relation between flow and sediment transport?

While the first method of approach ($V_{\text{mean max}}$ and Ω/M_{tot}) may be expanded to give more details relating these factors to more detailed parameters, the second approach (tidal prism Ω versus gorge area A) cannot possibly give reliable results although they are convenient for a first approximation and easy to "prove" repeatedly. Another drawback is that such overall considerations must disclose odd cases which do not fit into the picture. Two entrances with the same general

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flow characteristics, but with different inputs from the sea, cannot be expected to have the same degree of stability because small sediment inputs to the channel obviously does not call for the same "effort" by inlet currents to flush the channel for these sediments as large inputs, for which more flow energy is needed for the "cleaning-action". Material in a tidal entrance is flushed out at either end of the channel and comes to rest on shoals on the bay side, and also often on the sea side or on both sides. If there were no input of littoral drift material to the channel, the channel would gradually approach a non-scouring condition (2). If the input of littoral drift material is heavy nature must administer the available flow energy to ensure that the cross-section is maintained most economically with minimum loss of flow energy.

GENERAL DISCUSSION OF STABILITY

With reference to Figure 1, the stability of an inlet at location (x) may be defined by

$$\frac{\partial S}{\partial x} = 0 \quad (1)$$

in which S is the bed material transport. This criterion may be utilized to analyse the conditions for stability.

In order to do this it is advantageous to use the total concentration of sediment load as a transport parameter and to write

$$S = Q c_T \quad (2)$$

whereby c_T is the total average sediment concentration in a given cross-section.

Applying the stability criterion $\frac{\partial S}{\partial x} = 0$ to this expression we find

$$\frac{\partial S}{\partial x} = Q \frac{\partial c_T}{\partial x} + c_T \frac{\partial Q}{\partial x} = 0 \quad (3)$$

This expression can now be further developed.

If waves have little or no influence on the magnitude of the total sediment transport, it is attractive to use Yang's unit stream power concept for the computation of the sediment transport.

In Yang's paper (12) an expression for the total sediment concentration is submitted:

$$\log c_T = \alpha + \beta \log(VS - VS_{cr}) \quad (4)$$

in which α and β are coefficients. Using V (average velocity), S (slope), and A (cross-section) as independent parameters, and assuming that VS_{cr} (critical unit stream power per sec) is small compared to VS , under maximum flow conditions, equation (4) may be reduced to

$$-\frac{1}{A} \frac{\partial A}{\partial x} + 3 c_T \beta \frac{\partial v}{\partial x} + c_T \frac{\partial v}{\partial x} = 0 \quad (5)$$

The solution of this equation requires some additional information (input) regarding the shape of the inlet ($A_x = f(x)$) and the supply of sediments from the littoral drift (c_T at $x = 0$).

Further analysis leads to a relationship of the following type

$$V_m = \frac{Q_m}{A} = f(\eta, x, S_m, (V_0 c_{T_0}),) \quad (6)$$

in which

Q_m = maximum discharge

η = shape factor (e.g. $A = A_0 e^{-\eta x}$)

x = distance from entrance

S_m = slope of water level during maximum current conditions

$V_0 c_{T_0}$ = sediment discharge at $x = 0$ per unit of surface area

Since $S_m = \frac{\tau_m}{\rho g h}$ the slope component also represents the maximum bottom shear stress in the cross-section.

Of particular interest in this relationship is the dependency of V_m (stable conditions) on $\frac{\tau_m}{h}$ and $V_0 c_{T_0}$, the first representing the shear stress per unit of volume and the second representing a measure of input of sediment into the inlet at $x = 0$ (the entrance). This is discussed later in the paper. The parameter τ_m/h indicates that the depth of the inlet channel may affect the stability shear stress as defined in earlier papers (2, 4) and proved earlier in the slight dependency of $V_{mean \max}$ upon depth (4, p 133).

In the entrance current and wave action join forces in the transport. (This mechanism, discussed earlier (4, p 114), will be mentioned in detail in a paper in progress. It has been omitted for reasons of space).

Gorges are usually protected to some extent against wave action for which reason it is the tidal (and other) currents which are the main flushing agents. This is well demonstrated at many tidal entrances, e.g. in the Thyborøn Channel on the Danish North Sea Coast (2, 4). This inlet, cut by nature in 1862, and navigable a few years later, was continuously bothered by a large offshore bar with a controlling depth of only about 10 ft. The bar was the result of heavy wave action and heavy littoral drift to the inlet entrance from both sides. Close to one million cubic meters of sand material a year is transported into the inlet and deposited on extensive bay shoals.

Fig. 2 shows the variation of gorge sections I of the inlet during the period 1887 to 1970. After 1892 important dredging operations were started on the outer bar. This is clearly reflected in the increasing cross-section. Since it became difficult to keep up with the extremely heavy littoral-drift deposits, a different strategy was adopted. Dredging was transferred to the bay shoal main channel(s). The result was that the controlling depth on the outer bar increased to 15 feet and is now at least 20 feet and mostly close to 30 feet or more. Construction in the early 1920's of a 3,000-foot-long jetty on the northern barrier of the inlet further improved this situation.

The inlet channel gradually adjusted itself to the actual current and wave situation. Table 1 shows some cross-sectional areas of varying exposure to wave action (Fig. 2). Comparing characteristics of gorge section I to similar entrances (2, 3, 4, 5) it may be noted that the gorge area is below average size. Taking into account that almost one million cubic meters of sand material is carried through each year this cross-section for depositing on inlet shoals, it seems likely that normal stability velocities may have increased a little because of the heavy material load. Since the gorge has very steep slopes, its shape factor, as compared to other inlets, has apparently improved, too.

The relative stability of cross-sectional entrance areas is as mentioned in ref. 4, a result of combined wave and current action producing shear stresses which determine the cross-sectional area and its stability under varying wave and current exposure.

TABLE 1 CROSS-SECTIONAL AREAS OF THYBORØN CHANNEL (FIG. 2)

	Approximate cross-sectional areas (m ²)		
	I	II	III
	V _{mean max} (m/sec)		
	GORGE		
m ²	5,000	5,500	8,000
Material load per year	~3/4 million cu.m	~3/4 million cu.m	~3/4 to one million cu.m
Wave action during storms	light to moderate	moderate	heavy
V _{mean max} m/sec	~1,05	~0,95	~0,65
T _s kg/m ²	~0,5 kg	~0,4 kg	~0,2 kg

The variation of cross-sectional area of improved inlet channels is dealt with in Fig. 3 (2) where, for a number of American inlets with parallel jetties, the cross-section has been plotted along the length of the inlet channel in a dimensionless diagram. Information on actual data is given in Table 2 of ref. 2. The

cross-sections (A) at different locations have been divided by the cross-section at the entrance (A_0) to obtain a dimensionless ratio, using the relative distance from the seaward end (x/L) as the second parameter.

From Fig. 3 it may 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 decreased the bottom stability because orbital velocities due to the wave action increased bottom shear stresses, resulting in greater cross-sections near the entrance of the channel and thereby a smaller "apparent" stability velocity and shear stress corresponding to the tidal flow only (ref. 4 gives details of this).

Fig. 3 reveals that Fernandina Harbor (St. Marys River entrance, North Florida) does not follow "normal practice". The jetties protecting the entrance are parallel in length for 2,500 ft, but diverge further inland. In Fig. 3 the length "L" of the jetties refers only to the parallel sections in the entrance, which differ from the other inlets in that the width is very large (3,900 ft) in comparison to the length of the parallel part. Waves, therefore, can easily enter the harbor and the energy loss towards the entrance jetties and beaches along the channel is therefore relatively small.

These examples clearly demonstrate - just as in the case of Thyborøn Channel - the importance of the bottom shear velocities for material movement. The input of material to the entrance from the adjoining ocean shores must therefore also be an important factor.

The question which hereafter arises is that velocities are defined as "mean max". It is therefore of interest to appraise the error which may result by computing bed load transport based on a criterion only considering a mean velocity occurring at one particular time when it attains its maximum value at springtide.

$\bar{V}_{\text{mean max}}$ - The situation apparently is: First we say \bar{V}_{mean} next $\bar{V}_{\text{mean max}}$ and finally we define $\bar{V}_{\text{mean max}}$ at spring tide. Regarding \bar{V}_{mean} : Consider a schematic cross-section of a tidal entrance, Fig. 4. A few isovels have been drawn and shear stresses acting on the bottom may be computed. Splitting the cross-section in these parts and assuming an average velocity of 1 m/sec over the entire cross-section shown in Fig. 4, the average velocity may approximate 0.9 m/sec in the section closest to the bank and 1.1 m/sec in the middle. As velocities along the sloping banks are smallest, this section carries relatively less bed load transport per unit width of bottom.

Assuming (according to eq. (7) mentioned later) that bed load transport $\sim V^5$, the error by using 1 m/sec instead of an integrated V^5 over the entire bottom may be evaluated by an expression like

$$\frac{V_1^5 + V_2^5 + V_3^5}{3}$$

bank middle center
section section section

which for

$$V_1 = 0.9, \quad V_2 = 1.1 \text{ and } V_3 = 1.1 \text{ m/sec}$$

is 1.07 or 7% increase compared to $V_{ave} = 1$ m/sec or about 15% increase of the bottom shear stress. If the difference between side and middle section is as much as 0.4 m (1.2 - 0.8) the error is about 27% for the shear stress. Smaller deviations from the 1 m/sec average velocity will only cause minor deviations. In all cases the increase in velocity needed to flush away the surplus material will be of the order of magnitude mentioned above, 5 to 10%, corresponding to an increase in bottom shear stress of 10 to 20%, which means that the error obtained by using the average velocity for the entire cross-section will only be small. This is true whether $V_{mean \max}$ is 0.9 m/sec, 1.0 m/sec or 1.1 m/sec. Reference is made to the following section on Ω/M_{tot} .

$V_{mean \max}$ is in this connection defined as the mean of the max current over a certain period of time (2-3 hours) at spring tide conditions and it always seems to be in the range of av. 0.9 m/sec to ab. 1.1 m/sec. The explanation of the ab. 1 m/sec is partly given in refs. (2), (4) and (5) as a result of the fact that a sand bottom for these velocities becomes "smooth" i.e. ripples disappear and low dunes develop which become increasingly lower with increasing velocity until finally antidunes develop. $V_{mean \max} = 1$ m/sec is located in that part of the "transition zone" when ripples have vanished and dunes still developing smoother with increase of velocity, appear. This situation is discussed below.

Next: Why only use the max velocity? This question obviously must be discussed in relation to the sediment transport in the channel.

The problem of start of bed load transport has been dealt with in numerous publications referring to stream flow (1). At tidal inlets in alluvials the situation is that bottom material is derived from shores and beaches on either side of the ocean entrances. Sand material of 0.1 - 0.3 mm will therefore begin to move when mean velocities close to the bottom are about 0.3 m/sec (1 ft/sec). This velocity depends to some extent upon grain size and depth (2, 10). At 0.3 m/sec bottom starts developing ripple marks.

When the velocity exceeds about 0.6 m/sec (2 ft/sec) ripples become flatter and are gradually replaced by dunes until the bottom becomes almost flat at 0.9 m/sec - 1.2 m/sec (3-4 ft/sec). During that process bed load transport increases very rapidly as the full shear stress at a still increasing extent is exerted directly upon the bottom (2, 4). For example, bed load transport

increases from approximately 100 pp m to 3,000 pp m (4, Fig. 38) when velocity increases from 0.5 m/sec to 1.0 m/sec.

The ASCE Committee on Sedimentation discusses various sediment discharge formulas (1). These formulas are not identical in detail but a feature that those which the Committee considers being the most reliable have in common (e.g. those by Engelund and Hansen, Toffaletti and Colby), is that the sediment discharge is largely a function of the bed shear stress raised to the 2.5 power, although this relationship may not be expressed directly. For example the Engelund-Hansen formula states:

$$g_s = 0.05 \gamma_s V^2 \sqrt{\frac{d_{s0}}{g(\frac{\gamma_s}{\gamma} - 1)}} \left[\frac{T_o}{(\gamma_s - \gamma) d_{s0}} \right]^{3/2} \quad (7)$$

when g_s = sediment transport per unit width per unit time

V = mean velocity

T_o = bed shear stress = $\rho u_*^2 = \frac{\rho g V^2}{C^2}$

γ_s = specific gravity of sediment

γ = specific gravity of fluid

d_{s0} = mean full diameter of bed sediment

g = acceleration of gravity

C = Chezy's friction coefficient

Engelund and Hansen, however, do not recommend their formula for cases in which the median size of the sediment is less than 0.15 mm; the geometric, standard deviation of the grain size is greater than approximately 2 and T_o is less than 0.15 kg/m². These conditions are undoubtedly fulfilled for all tidal inlets on alluvial shores when grain size usually ranges from 0.15 - 0.3 mm; sand is well graded and T_o is somewhere between 0.04 and 0.06 kg/m².

For tidal entrances subjected to flow due to simple harmonic semi-diurnal tides, and in cases when the connecting channel has a simple geometry, the velocity of the flow is approximately proportional to $\sqrt{\sin \theta}$ when θ is the tidal phase angle. The sediment discharge therefore depends approximately upon $\sin^2 \theta \sqrt{\sin \theta}$. Integrating this expression for various phase intervals it may be noted that about 80% of the sediment transport takes place between the velocity phase angles 70 and 110 degrees, when velocities vary between about 85 to 90% and 100% of the peak velocity occurring at the 90 degree velocity phase angle. This general result indicates that the mean max velocity occurring for about one hour and a quarter on either side of the peak velocity may be considered an important flow parameter in describing the equilibrium condition between flow and sediment movement, and consequently for description of bottom stability. The situation, needless to say, depends upon details of the actual tidal fluctuations and the specific time interval during which velocities are equal to or

exceed about 85 to 90% of the peak velocity. During that period the "stable" entrance channel is flushed for all or for the larger part of the material which was deposited in the entrance during the slack water period. Normally most or all of this material is derived from the littoral drift zone on one or on either side of the entrance. $V_{\text{mean max}}$, however, refers to spring tide (2, 4).

Tidal range for spring tide conditions is usually of the order of 30 to 50 per cent higher than for mean tidal range conditions, but these conditions are subjected to seasonal variations with max range at the two equinoxes. The max current velocities may then increase 10 to 15 per cent and the tidal prism perhaps a little more. This in turn means that bed load transport during the peak period may be twice as high as during mean tidal conditions. This situation lasts a few days only during the 28 days moon cycle. One may expect the gorge cross-sectional area increases slightly during this period, which to some minor extent compensates for the increase in velocity. It is probably not essential whether "normal mean high" or "spring mean high" velocities are used for description of the stability. If this were the case the effect could be expected to be relatively largest for entrances with small tidal range which should then demonstrate somewhat higher mean max velocities. Based on the presently available survey material such effect has not (yet) been noted. It may be within the range of a number of other deviations. Consequently, it must also be less important whether it is the mean Ω at spring tide or the high Ω at max spring tide that has to be considered; the important consideration is that it has to be the same general tidal conditions for all entrances to make comparisons possible.

The conclusion is apparently that $V_{\text{mean max}}$ at spring tide seems to be a useful parameter for description of the cross-sectional stability of a tidal inlet entrance channel.

Ω/M_{tot} - An evaluation of the Ω/M_{tot} ratio must of necessity include an evaluation of the overall material transport in the gorge. The general rule is that in the gorge almost all transport of sand > about 0.01 mm takes place as bed load transport while finer particles < 0.06 mm including silt and clay, if present, may be transported mainly in suspension.

Consider a tidal entrance which is subjected to input of littoral drift from the adjoining shores.

Mathematically the situation may be described as:

$$\frac{dS}{dt} = \frac{dM}{dt} \quad (8)$$

which expresses that the increase in sediment transport shall be equal to the increase in input of sediments from the seashore, M . This definition implies that the quantity of sediment transport in the inlet channel as well as the input of littoral transport to the channel (eq. 3) is known.

With regard to channel transport the situation is: When flow velocity increases beyond the limiting velocity for material movement bed load transport starts and the entire surface layer of the bottom moves forwards and backwards with the ebb and the flood current. In a stable channel currents have to carry away the surplus material which is deposited during slack water and which is attempting to choke the channel. In numerical form eq. (8) be written:

$$\Delta S(t) = \Delta M(t) \quad (9)$$

where ΔS is the increase in sediment transport per unit time which is necessary to cope with the input per unit time of sediment from the littoral drift zone of the adjoining shores. "Unit time" may for e.g. be chosen to be a tidal period. The number of those per year for semi-diurnal conditions is approximately 680. Consider an input of littoral drift material to the entrance, 350,000 m³/year, the quantity of material is a practical figure referring to conditions on the US East Coast and at many places elsewhere. This material which has to be removed during each half tidal cycle is approximately 500 m³ or

$$\Delta S \sim 500 \text{ m}^3$$

The cross-sectional area may under simplified conditions be computed as:

$$A = \frac{\pi \Omega C_2}{T} \quad (10)$$

when Ω is the tidal prism, C_2 is a coefficient varying between 0.8 and 1.0 and T is the tidal period (7).

As mentioned earlier, experience shows that the Ω/M ratio for a relatively stable gorge channel is > 150 (2, 4). With $\Omega/M = 150$, Ω would be $150 \cdot 350,000 = 5.3 \cdot 10^7$ corresponding to a $A = Q_{\text{mean max}} \text{ (mean max discharge at springtime)} = \frac{\Omega C_2 \pi}{T} = 3,400 \text{ m}^2$

assuming that the mean max ~ 1 m/sec (2, 4). No general figure can be given for the depth over width ratio. For the improved inlets in Florida it is 0.03 - 0.04. For the unimproved entrances it is of the order 0.01 - 0.02. Assuming a ratio of 0.02 for $A = 3,400 \text{ m}^2$ this corresponds to a gorge of about 500 meters wide and 8 m deep (4, Table 18). The increase in sediment transport due to the input of littoral drift material must be approximately 1 m³ (500/500) per meter per tidal cycle. 1 m³ ~ 2 tons of sand.

Using a mean max velocity of approximately 1 m/sec in the Engelund-Hansen formula, eq. (7) one has with $d_{50} \sim 0.0002 \text{ m}$, $\gamma_s = 2.65 \text{ gr/cm}^3$, $\gamma = 1 \text{ gr/cm}^3$, $g_s = 0.5 \text{ kg/sec/m}$

As mentioned above, high velocity flows (about 0.9 - 1.1 m/sec) run for about 2 - 3 hours in each half tidal cycle, which means that a total of $60 \cdot 60 \cdot 0.5 \cdot 2.5 = 4.5$ tons is transported per 2.5 hour period per meter. If this quantity is increased by another 2 tons the current velocity obviously must be increased. The increase must correspond to an increase in

transport of about 45%. As bed load transport depends upon the bottom shear stress in about the 2.5th power (1) this means that the mean bottom shear stress must increase about 20%. If this is correct one should be able to detect that kind of difference in mean max velocity which, however, only needs to be about 10% higher when comparing non-protected tidal entrances with jetty protected entrances.

It is therefore very interesting to note the situation at some hydraulically rather well defined tidal entrances mentioned in Table 2 (Table 18 of ref. 2).

TABLE 2 COMPARISON BETWEEN THE MEAN MAX VELOCITY IN JETTY PROTECTED AND IN ENTRANCES WITHOUT JETTIES (2)

Entrance	T mean max	in kg/m ²
Grays Harbor, Wash.	0.49	jetties
Port Aransas, Texas	0.46	-
Calcasieu Pass, La.	0.45	-
Thyborøn, Denmark	0.49	- Ave. 0.47
Longbout Pass, Fla.	0.55	no jetties
Big Pass, Fla.	0.55	-
East Pass, Fla.	0.54	-
Ponce De Leon, Fla.	(0.48)*	- Ave. 0.55 (0.53)

x) (large shoals - jetties built in 1970-1972)

Although the material in Table 2 is meager, there is probably an indication that the mean max bottom shear stress for stable conditions are somewhat (about 10 - 15 per cent) lower for jetty protected entrances than for non-protected. This corresponds to an increase of bed load transport of about 35 - 40 per cent against 45% mentioned above. In the case considered, about 350,000 m³ is transported by ebb and by flood currents every year. With about 40% increase this means that 140,000 m³ extra is flushed and consequently about 140,000 m³ or conversly about 40% has to be kept back by jetties or transferred mechanically if the extra flushing is to be avoided or is unobtainable.

Table 3 of ref. (4) lists a few practical examples from Florida (inf. by the U.S. Army Corps of Engineers, Jacksonville District, Florida). It may be noted that inlets with long jetties bypass 80% (by ebb currents mainly), inlets with short or medium about 40% and inlets with "very short" jetties about 20%.

These figures are interesting in that it may be seen how inlets with jetties are able to bypass by flushing relatively more material after they have been protected by jetties which on the one hand retain part of the surplus littoral drift which bothered the gorge channel and on the other hand concentrate the flow. In the latter case wave action was helpful in bypassing or otherwise leaving the material where the stream power was higher than on the shores of the channel, thereby increasing the rate of flushing.

TABLE 3 PREDOMINANT DRIFT QUANTITY AND BYPASSING DRIFT FOR SOME JETTY-PROTECTED INLETS IN FLORIDA (4)

Inlet	Jetty Length	Drift Total per year	Drift Bypassed per year	Bypassed Total max.	Bypassed or flushed by tidal flow
Jupiter	very short	225,000	150,000	60%	ab. 20%
Sebastian	short	300,000	200,000	60%	30% to 50%
South Lake Worth	medium	180,000	ab. 90,000	50%	ab. 50%
Palm Beach ^x	long	225,000	ab. 175,000	(80%)	(ab. 80%)
Ft. Pierce	very long	250,000	200,000	80%	ab. 80%

^xCondition 1964-1965

From the above-mentioned and numerous similar cases it may be concluded that generally the major part of the sediment transport in a tidal inlet is "native inlet material" which is moved forwards and backwards with the tidal currents. In addition littoral drift material which moves in "from the side" is being flushed by the inlet currents. The question still remains: Is the Ω/M_{TOT} ratio a useful parameter for describing the relative stability of the inlet gorge channel?

As explained in detail in ref. (4), the quantity of material transported as bed load is independent of depth when the mean velocity is about 1 m/sec (See Fig. 5 from ref. (4)).

This in turn means that the total sediment transport (bed load) is proportional to the width (W) of the gorge channel (cross-sections of similar geometry considered). As explained earlier, an average of 80% of the transport takes place when the max velocity is between 85% to 90% and 100% of the peak velocity. For wide inlet channels of mean depth D one has:

$$S + M \sim W \quad (11)$$

or drift of "native material" plus input of littoral drift material is proportional to the width of the channel.

$$\text{But } W \cdot D = A \left(= \frac{\pi \Omega C}{T} \right)$$

$$W \sim D \text{ for similar cross-section}$$

$$W^2 \sim \Omega$$

$$W = S + M \sim \sqrt{\Omega}$$

$$\frac{\Omega}{M} \sim \frac{(M+S)^2}{M} \quad (12)$$

From this expression it may be seen that Ω/M_{TOT} is high when $M \ll S$ and Ω/M is low when $M \gg S$. From this follows that the Ω/M ratio may be expected to be a useful parameter to describe the relative stability of the gorge cross-sectional area for

wider channels. This is also true for narrower channels when $S + M$ is rather proportional to $A \sim \Omega$ which means that $\Omega/M \sim M + S/M = 1 + S/M$, which supports the conclusion for wide channels.

The following section considers some practical $V_{\text{mean max}}$ and Ω/M_{tot} ratios.

EXAMPLES OF THE USE OF THE $V_{\text{mean max}}$ AND Ω/M_{tot}
CRITERIA IN SOME TIDAL INLETS IN INDIA'
THE UNITED STATES AND EUROPE

Table 4 lists tidal entrances in India located on the Arabian Sea as well as on the Bay of Bengal. These inlets were all studied under the Preinvestment Study of Fishing Ports, a joint project of the Indian Government and the Food and Agricultural Organization of the United Nations. Some of the information was derived from existing sources, e.g. the Poona laboratory.

In some cases e.g. at Beypore, data on tidal prism and flow were available from earlier surveys. In others, e.g. at Malpe, new surveys were undertaken. Half of the cases which are listed in Table 4, however, were not surveyed and estimates therefore were based on local information and overall inspections. The results from these cases must therefore be considered with some reservation with respect to reliability. All figures refer to the situation at the gorge channel and its immediate vicinity and not to offshore conditions when the influence of tidal currents have vanished.

It is interesting to note, however, that the mean max velocity for the inlets which have been surveyed in detail or at least to some extent varies from 0.9 to 1.2 m/sec. The average of five gorges which were surveyed in more detail is 1.02 m. In no case - whether surveyed in detail or not - was the mean max velocity lower than 0.9 m/sec or higher than about 1.2 m/sec referring to the situation(s) mentioned in Table 4. It is necessary to distinguish between monsoon and non-monsoon periods.

Consider the Ω/M_{tot} ratio it may be noted that entrance conditions may be classified in three main groups: Those (11 and 12) which are protected by rock reefs functioning as breakwaters, for which reason they are not bothered by heavy littoral drift deposits. Their Ω/M ratio is 100-150 even if tidal prism is only medium. The next category which has Ω/M ratios of 50-100 has in some cases large offshore bars (1, 3, 7, 8) but these bars can usually still be passed by shallow draft vessels, including fishing boats. These entrances have medium to large tidal prisms.

The third category (2, 4, 5, 6, 9, 10) is characterized by comprehensive bars with very shallow depth. They are "bar-bypassers". Almost all of them have relatively small tidal prisms and $\Omega/M < 50$ and mostly even ≤ 20 . Some depend upon

flushing during the monsoon, and are closed during the non-monsoon period. Others are mainly stable during the non-monsoon period when littoral drift is minimum. All cases refer to the relatively most stable condition of the entrance not the offshore channel during the seasons. Averaging figures over e.g. one year makes no sense under the climatic conditions in India. Comparison is made with the entrances listed in Table 5 (Table 13 of ref. 2) comprising a number of tidal inlets in the United States and in Europe.

In Table 5 entrances with relatively satisfactory and stable entrance conditions (1, 2, 5, 6, 7, 8, 10, 11, 12, 13, 14, 15, 16, 17, 18, 21, 22, 23) all have $\Omega/M > 150$. Those with "fair conditions" have $100 \leq \Omega/M \leq 150$ (3, 22). Entrances with conditions between "fair" and "poor" (19) have $50 \leq \Omega/M \leq 100$ and those with "poor" conditions (4, 9, 20) have $\Omega/M < 50$. Of these, Brielse Mass has been closed, Figueira Da Foz has been improved by jetties and Ponce De Leon has been improved by jetties and a submerged weir in the north jetty. There therefore seems to be good agreement between "the Indian experience" listed in Table 4 and the combined "European-American experience" listed in Table 5.

The conclusion of these investigations, therefore, is that the Ω/M_{tot} ratio seems to be a fairly reliable parameter for the description of the overall stability of a tidal entrance on an alluvial shore. Reference is still only made to the stability of the gorge channel and not to the outer part of the channel which does not carry concentrated channel flow. This problem will be dealt with in a forthcoming paper. This outer part of the channel may be subjected to deposits carried to the entrance by offshore shore-parallel currents or flushed out by ebb currents. This is typical for a number of tidal inlets in India, e.g. for Nos. 1 and 8 of Table 4. Wave action, however, will as demonstrated in the above-mentioned Thyborøen Inlet in Denmark and at inlets in Florida and elsewhere, be able to assist the tidal currents in keeping the outer part of the channel free of deposits (4). Still, there is the possibility that a moderate-size bar may form where entrance tidal currents meet sediment laden longshore currents. This is mainly characteristic for entrances with $\Omega/M_{(tot)}$ ratios of 100 to 150 and relatively larger inputs of sediments. As proven by Shemdin and Dane (9), a bar may also cause a pile up of water (changes of slope) in the lower part of the outlet. This, in turn, decreases the discharge and contributes to further deterioration of the entrance.

Design procedure - With respect to design procedure for improvement one must distinguish between preliminary and detailed design. For preliminary design one may proceed as follows (4, 5):

- (1) Secure all available information on Ω (tidal prism), $Q_{mean\ max}$ (mean max discharge under spring tide conditions). Compare Ω , $Q_{mean\ max}$ and other Q 's by computation and current

velocity measurements in the gorge channel. Evaluate bay (lagoon) tidal range based on experience from cases of similar tidal range and similar geometry of bay (or lagoon).

(2) In layout, use straight or almost straight channel boundaries to avoid scour on one side and deposits on the other side of the channel in bends.

(3) Evaluate M_{tot} as closely as possible e.g. based on experience from neighbouring shores. Check the Ω/M ratio. Observe its seasonal changes and pay particular attention to its lowest value(s). If littoral drift formulas are used, check with 2 - 3 of them.

(4) Evaluate the most likely $V_{mean\ max}$ based on experience, considering the local littoral drift capacity.

$$V_{mean\ max} = 1 \text{ m/sec} \pm \alpha \text{ m/sec} \quad \alpha = 0.1 \text{ to } 0.2 \text{ m/sec}$$

(5) Initially, use an overall relationship between tidal prism and cross-sectional area of gorge:

$$A = \frac{\Omega}{V_{max}} \frac{C_2 \pi}{T} \quad C_2 \approx 0.9$$

Compute $Q_{max} = AV_{mean\ max}$

(6) Design cross-section, horizontal bottom, slope 1 in 5 (sand bottom) or 1:X, X = slope of training wall or jetty.

(7) Check velocity distribution. Use one of the available theories (as e.g. explained in refs. 2, 4 with references).

(8) Check $V_{mean\ max}$ again. Adjust gorge area to selected $V_{mean\ max}$ in greater detail with respect to velocity distribution.

(9) Check Q_{max} and Ω by detailed computation.

(10) If Ω thereby decreases below acceptable value considering the Ω/M_{tot} ratio, try to increase Ω by increasing A and repeat computations listed above. Observe the seasonal changes in Ω/M_{tot} with special reference to low values.

(11) If Ω cannot possibly be increased, try to decrease the active M_{tot} by jetties, traps, or by an entrance geometry better suited for effective flushing, if possible. For detailed design model experiments may be advantageous or necessary to secure the most desirable velocity distribution in the inlet channel as a whole, as well as in the cross-section.

(12) In the case of improvement of an existing inlet, use tracer experiments to clarify littoral drift pattern and if necessary also the littoral drift quantity, the latter being subject to long-time experiments. Use experience values if available or energy flux considerations as mentioned in previous section.

(13) One may finally try to compute the bed load transport in the gorge channel e.g. by using Engelund and Hansens (7) procedures for bed-load transport and compare quantity with littoral drift quantity.

When it comes to detailed design, the tidal hydraulics computations may be undertaken according to Dr. Dronker's theories (4). Owing hysteresis effect (4, Fig. 35), the time period when the bottom is covered by well developed ripples is relatively short and it may be assumed that the bottom is smooth or only slightly undulated for velocities exceeding 0.5 m/sec (assuming fine and medium size sand). Friction factors of $C = \text{about } 45 \text{ m}^{1/2} \text{ sec}^{-1}$ therefore prevail for at least two thirds of the tidal cycle and $C = \text{about } 35 \text{ m}^{1/2} \text{ sec}^{-1}$ for the remaining period.

Most often the detailed design of a tidal inlet on a littoral drift shore depends upon the results of a hydraulic model study. For fixed bed flow studies adequate model laws are available. This is not yet the case for movable bed studies. One is here faced with a tedious calibration procedure which may be based on a known time-history and/or the results of tracer experiments and other results of sediment transport or semi-theoretical investigations or procedures. The general knowledge on the behaviour of tidal inlets as described above is helpful, however, in determining the reliability of the model and its reproduction of conditions in the prototype. Field studies, however, are all important. From them we learn generalizations and their variances.

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m = monsoon
n = non-monsoon
su = surveyed

e = estimated by
computation
s = Spring

TIDAL ENTRANCES AT INDIA
HYDRAULIC AND CROSS SECTIONAL CHARACTERISTICS RELATED TO OVERALL STABILITY

Name of entrance or inlet	Tidal range s spring n normal	Q 10 ⁶ m ³ /s	A m ² at MSL	Q _{max} net Drift is almost uni- directional	M 10 ⁶ m ³ per year	V _{mean} , max	gorge MLW	Depth, m MLW	G/M	Note
(1) Baysore (estuary)	15 m 5 m	1,000	300	0,2	1,0 (su)	6-7 5-6	1,5-2	- 80 m		Comprehensive bar very shallow
(2) Chandipur (estuary)	s 4	5	(e)	0,25	1,2 (e)		0,9-1,2	- 20		Comprehensive bar very shallow
(3) Honavar (estuary)	< 20 (su)	800 nm 1,000 su		0,2	1,2 su	5-7 su		50-100		Bar
(4) Kalingapatam (estuary)	1 nm (e)			0,1 (nearshore)	-			10-20		Comprehensive bar very shallow
(5) Krishnapatam (estuary)	(10) (e)	500 m 500 ± nm		0,5-0,7 (nearshore)	1,2 m	(0,5-1)	- 0,5	10-20		Comprehensive bar very shallow
(6) Machilipatam (estuary)	10 (e)su)			0,2 (nearshore)	-	- 2	- 1	20-50		Comprehensive bar very shallow
(7) Malas (estuary)	s 1,2 (su)	350 ± su	350 su 0,1	0,15	1,0 su	2-3 su	1-1,5 su	- 80		Bar
(8) Neandakara (estuary)	(9) (e)su)	(600)		0,2	0,9 (su)			- 50		Bar
(9) Nizampatam (estuary)	1-1,5 (su)	- 70 ± su		0,1 (nearshore) 0,2-0,4	1,0 su	3 su	max 1,5 su	10-20		Comprehensive bar very shallow
(10) Ponnani (estuary)	- 2 (e)	300 m 1 nm		0,2			0,5-1 m 2-2,5 nm	10-20		Comprehensive bar very shallow
(11) Sapatil (estuary)	s 4,0 - 15 (e)			0,1 to the entrance		3		100-150		Protected by rock reefs
(12) Veriova (estuary)	- 6 (e)	400 ±		0,05-0,1	0,9 (e)	3-4		100-150		Protected by shore rock

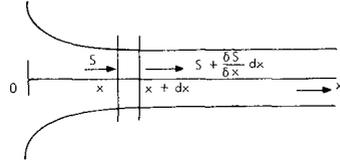


Fig. 1 Tidal Entrance Equilibrium.Schematics

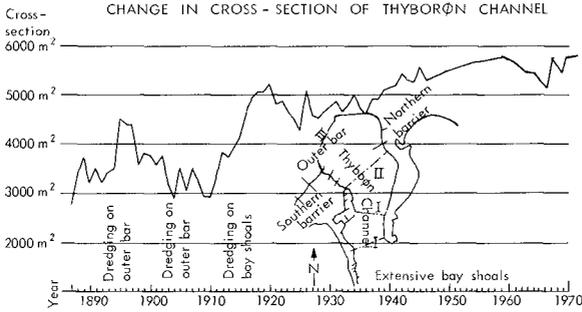


FIG. 2. DEVELOPMENT OF THE GORGE OF THYBORØN INLET ON THE DANISH NORT SEA COAST. (ref.2 ;

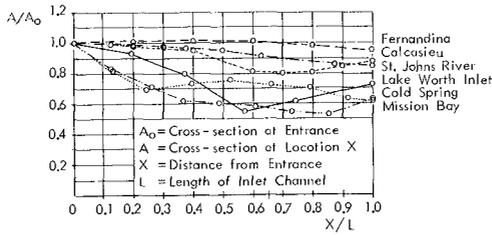


FIG. 3. CROSS-SECTION AREAS BELOW M.L.W. FOR SOME JETTYIMPROVED INLETS. (ref.2)

TABLE 5 FLOW AND LITTORAL DRIFT CHARACTERISTICS FOR SOME TIDAL INLETS IN THE UNITED STATES AND EUROPE (2)

Inlet (Kind of Improvement)	Ω^{**} Tidal Prism cu yd/half cycle	Q_{max} Maximum Discharge cu yd/sec	M^{***} Littoral Drift cu yd/year	$\frac{\Omega}{M}$
(1) Amelandse Gat, Holland (Bank stabilization on north side)	600×10^6	36,600	1.0×10^6	~600
(2) Aveiro, Portugal (jetties)	80×10^6	5,000 ¹⁾	1.3×10^6	~60
(3) Big Pass, Florida (None)	12×10^6	700	$<0.1 \times 10^6$	>120
(4) Brielse Mass, Holland before closing (Closed)	40×10^6	2,700	1.0×10^6	~ 40
(5) Brouwershaven Gat, Holland (before closing, closed)	430×10^6	30,000	1.0×10^6	~430
(6) Calcasieu Pass, La. (diurnal) (Jetties and Dredging)	110×10^6	2,600	0.1×10^6	~550 ²⁾
(7) East Pass, Florida (diurnal) (Dredging)	60×10^6	1,720	0.1×10^6	~300 ²⁾
(8) Eyerlandse Gat, Holland (None)	270×10^6	19,000	1.0×10^6	~270
(9) Figueira Da Foz, Portugal (Dredging)	20×10^6	1,200	0.5×10^6	~ 40
(10) Fort Pierce Inlet, Florida (Jetties and Dredging)	80×10^6	3,700	0.25×10^6	~320
(11) Gasparilla Pass, Florida (None)	15×10^6	900	$<0.1 \times 10^6$	>150
(12) Grays Harbor, Washington (Jetties and Dredging)	700×10^6	48,000	1.0×10^6	~700
(13) Haringvliet, Holland (before closing, closed)	350×10^6	25,000	1.0×10^6	~350
(14) Inlet of Texel, Holland (Stabilization on south side)	1400×10^6	115,000	1.0×10^6	~1400
(15) Inlet of Vlie, Holland (None)	1400×10^6	110,000	1.0×10^6	~1400
(16) Longboat Pass, Florida (None)	30×10^6	1,400	$<0.1 \times 10^6$	>300
(17) Mission Bay, California before dredging (Jetties and Dredging)	15×10^6	1,100	0.1×10^6	~150
(18) Oosterschelde, Holland (Will be closed)	1400×10^6	100,000	1.0×10^6	~1400
(19) Oregon Inlet, N.Carolina (Occasional Dredging)	80×10^6	5,100	1.0×10^6	~ 80
(20) Ponce De Leon Inlet, Fla. (before improvement)	20×10^6	1,500	0.5×10^6	~ 40
(21) Port Aransas, Texas (diurnal) (Jetties and Dredging)	60×10^6	1,900	0.1×10^6	325 ²⁾
(22) Thyborøn, Denmark (Minor Dredging)	140×10^6	7,500	1.0×10^6	~140
(23) Westerschelde, Holland (Some dredging)	1600×10^6	115,000	1.0×10^6	~1600

*** Total amount of littoral drift interfering with the inlet may deviate from this value if drift direction is too predominant or if the inlet is improved by long jetties and/or bypassing

* Spring tide ¹⁾ Increasing ²⁾ $\Omega/2M$, diurnal tide



FIG. 4. Tidal channel with a few Isolines for Velocities drawn. Schematics.

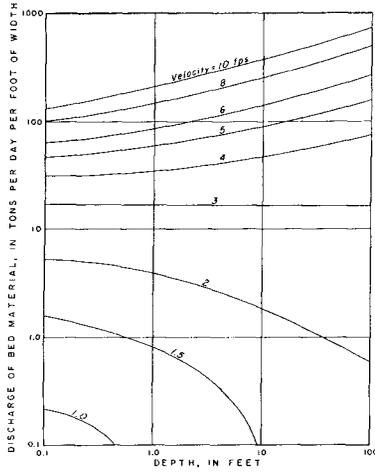


Fig. 5 Effect of Depth on the Relationship between Mean Velocity and Empirically Determined Discharges of Bed Material (0.3 mm medium diameter) at 60 degrees F. (Ref. 3, Colby, 1964)