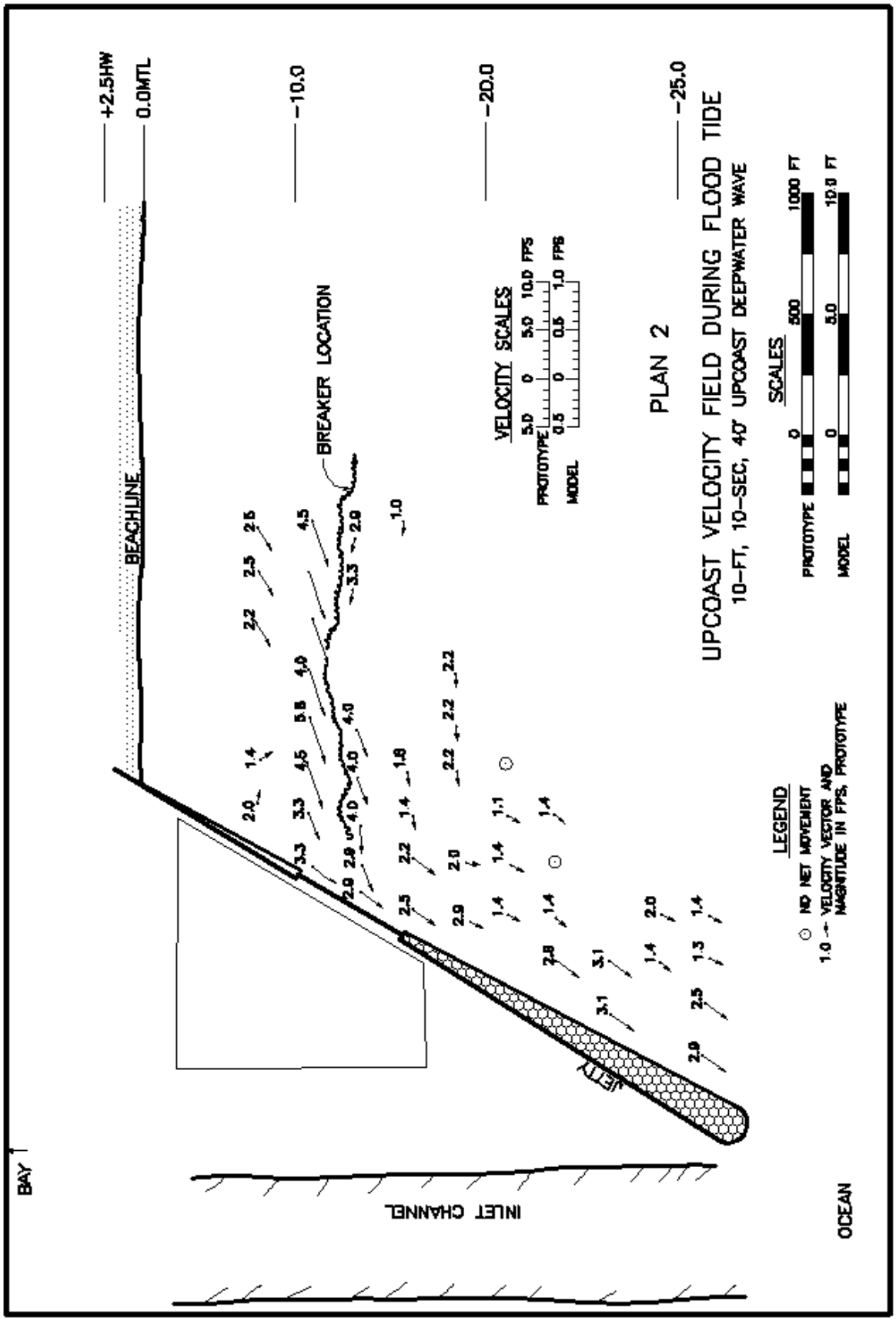


Figure II-6-13. Tidal current plus wave-generated currents approaching jettied inlet, measured in physical model study

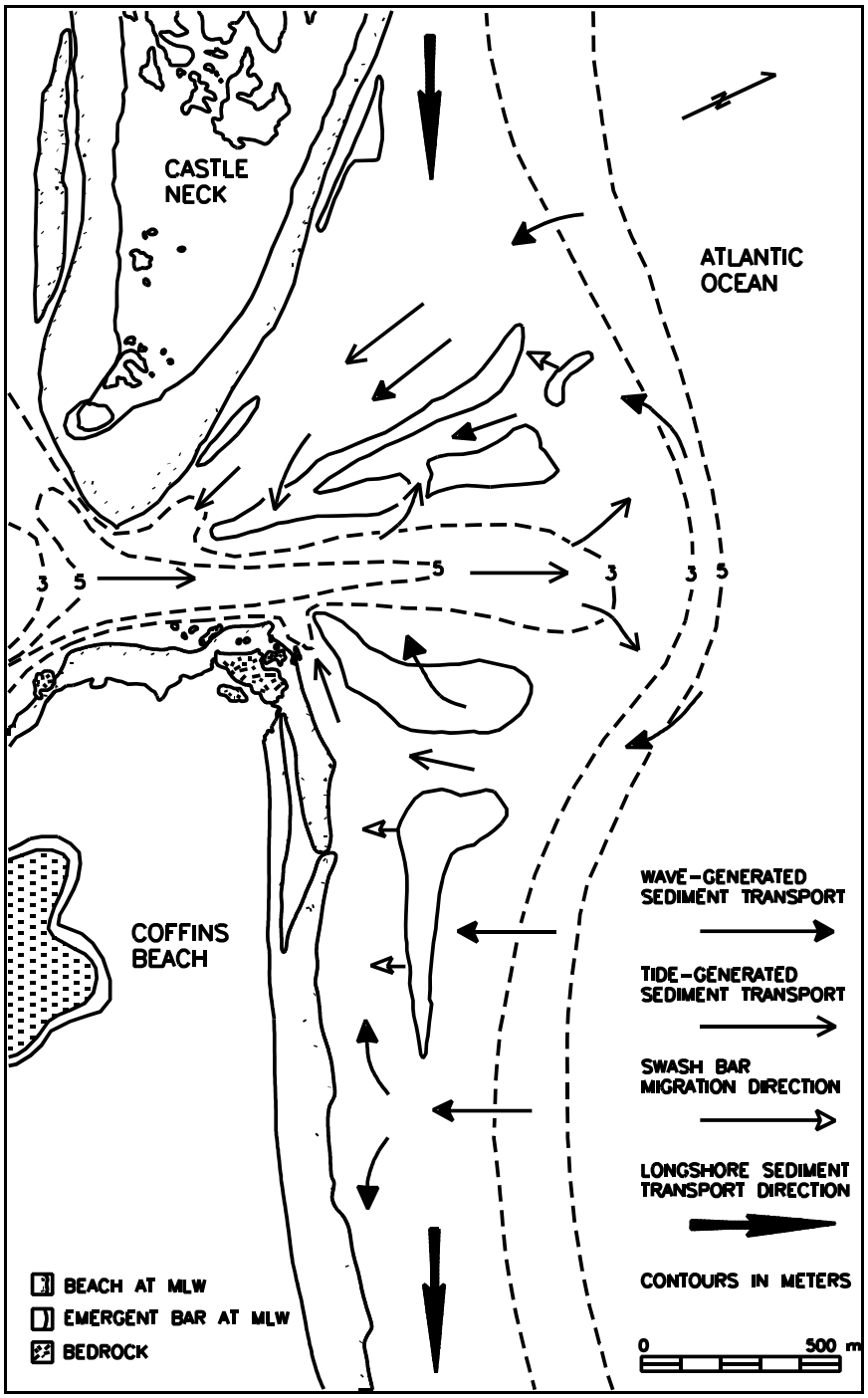


Figure II-6-14. Sediment transport gyres associated with both updrift and downdrift portions of the delta at Essex River Inlet, Massachusetts. (Note different arrow types to denote wave or tide-generated sediment transportation (Smith 1991)

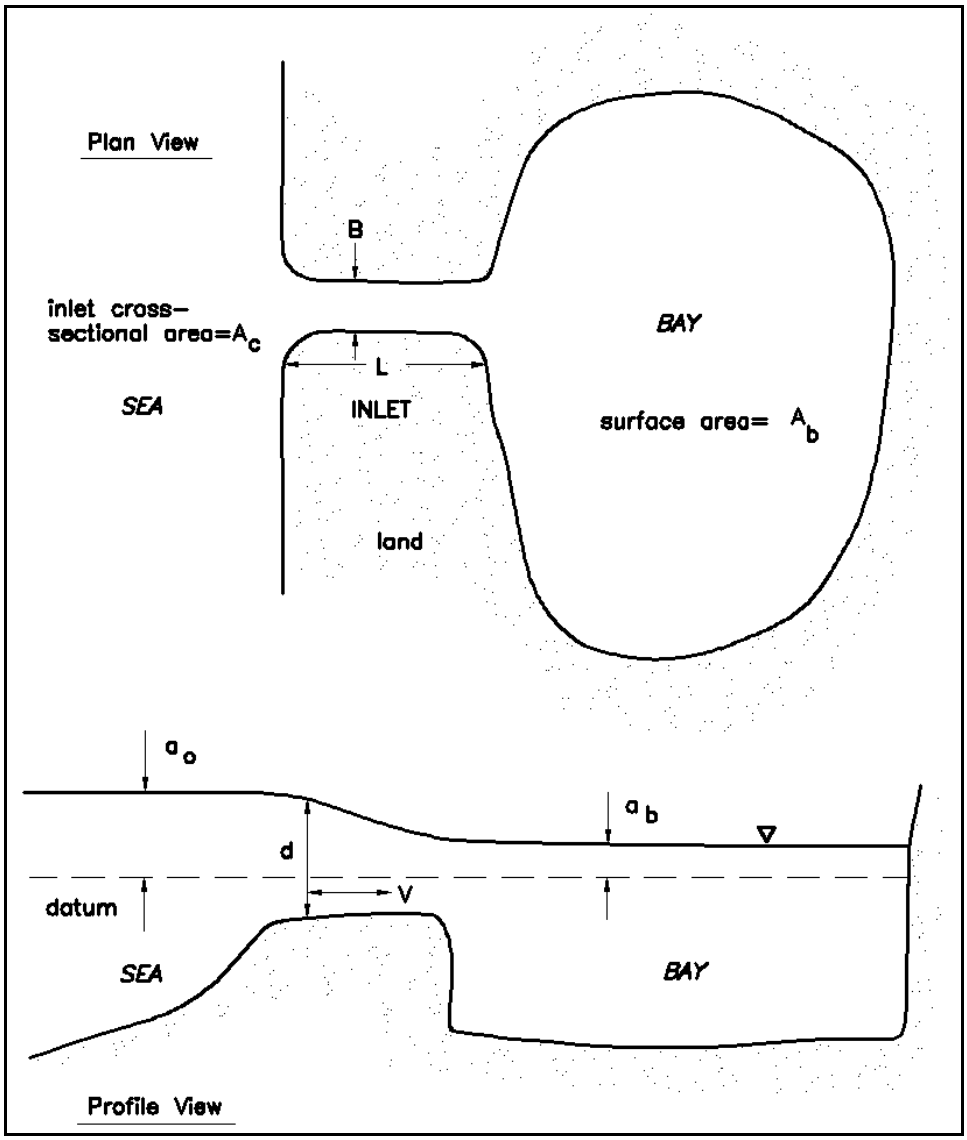


Figure II-6-15. Inlet bay system

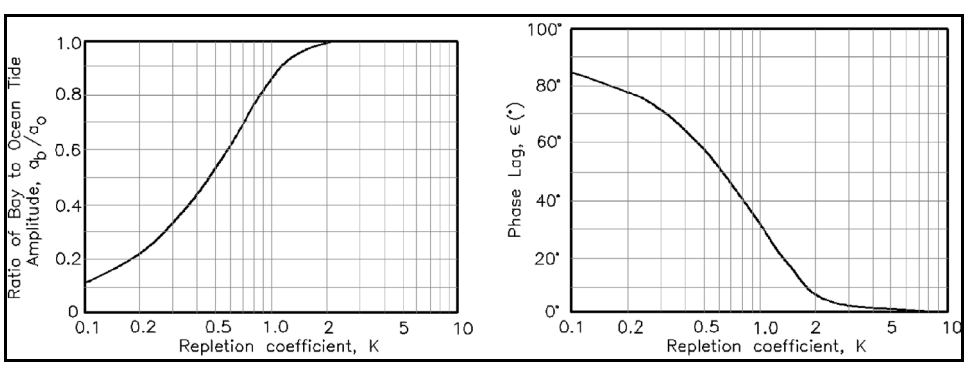


Figure II-6-16. Variation of dimensionless parameters with Keulegan's repletion coefficient, K

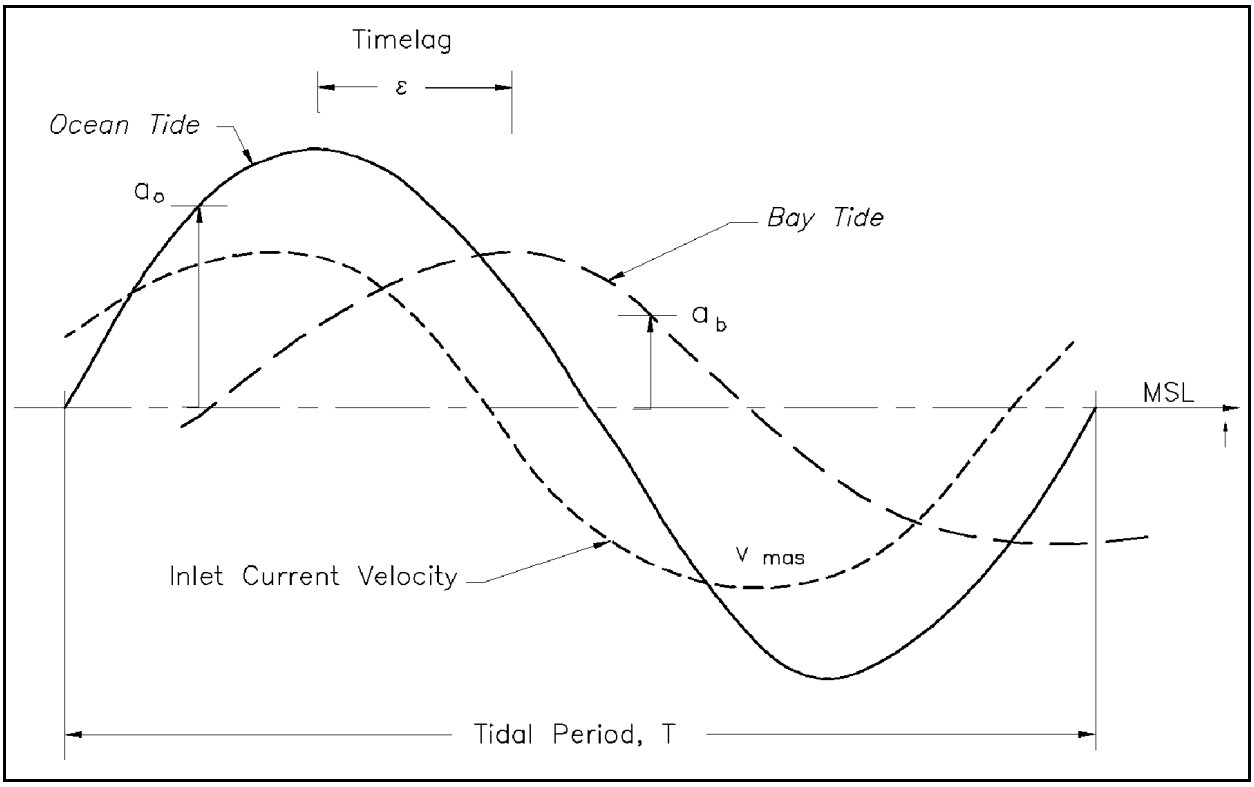


Figure II-6-17. Sample tides and currents for an ocean-inlet-bay system that satisfy the Keulegan assumptions (Keulegan 1967)

where

V_{avg} = maximum velocity averaged over entire cross section

V_{meas} = point measurement of maximum velocity

R = hydraulic radius of entire cross section

D = depth of water at current meter location

The V_{avg} value then can be used in Equation II-6-13.

d. Determining important inlet parameters. Graphs can easily be used to determine inlet hydraulics; however, evaluation of important parameters requires some effort. Mason (1975) determined appropriate techniques for defining cross-sectional area, bay area, inlet (channel) length, hydraulic radius, friction factor and exit and entrance loss coefficients. The following offers guidance in the evaluation of parameters.

(1) Cross-sectional area. Problems in defining location and magnitude of the cross-sectional area result in greatest variability in computed values of the repletion coefficient. To reduce such variability, the cross section should be determined in the following manner:

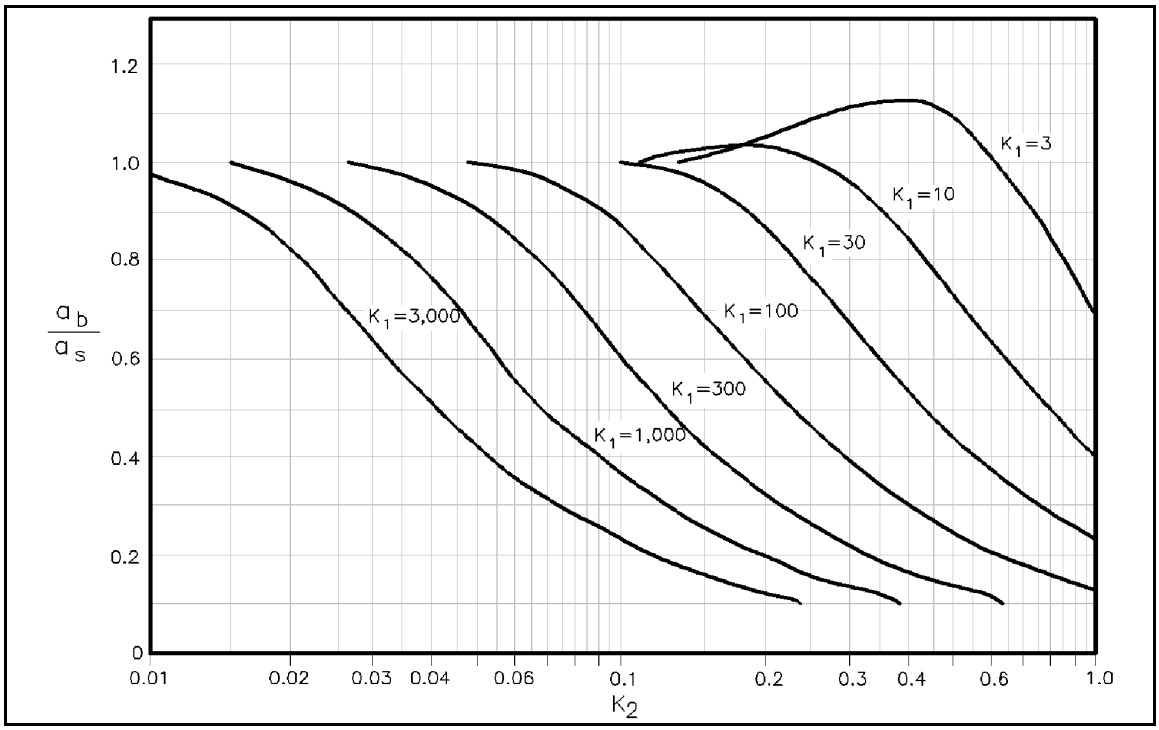


Figure II-6-18. Ratio of bay to sea tidal amplitude versus K_1 and K_2

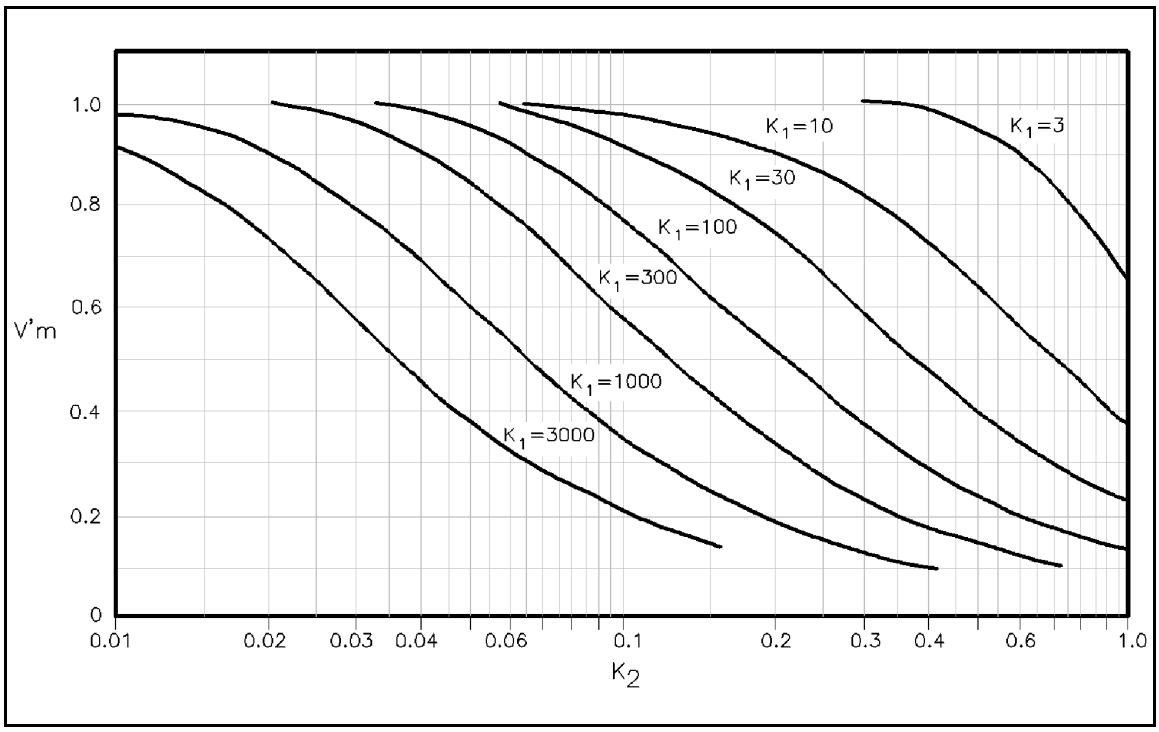


Figure II-6-19. Dimensionless maximum velocity versus K_1 and K_2

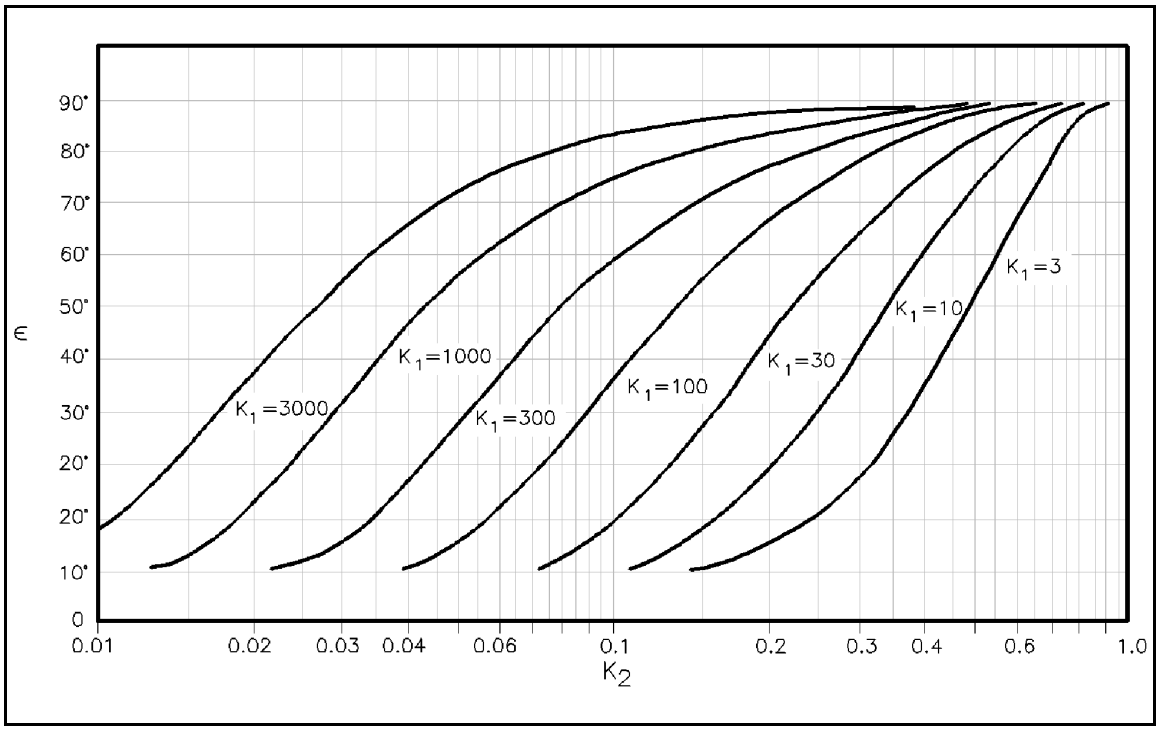


Figure II-6-20. Bay tidal phase lag versus K_1 and K_2

From a detailed bathymetric chart of the inlet, sketch lines that best approximate flood flow lines through the inlet as shown in Figure II-6-21. Establish 10 equally spaced cross-section ranges perpendicular to these flow lines, extending from MSL (mean sea level) on one side of the channel to MSL on the other (i.e. from one side of the inlet to the other). Measure cross-sectional area below MSL of each range and determine average area. Depths on most navigation charts are referenced to mean low or mean lower low water, and a correction factor should be added to these depths to determine the area below MSL.

(2) Bay area. Typically the problem is to define the extent of the inlet's influence throughout the bay. To do this, the following are recommended:

(a) Examine any existing tide records from stations located throughout the bay to define limit of the inlet's influence.

(b) Examine bathymetry of the bay to determine presence of natural sills, narrows, or shoal areas which can preclude significant tidal exchange.

(c) Locate any areas of heavy shoaling at points distant from the inlet, particularly in waterways connecting the bay of interest with more remote bodies of water, and define these sites as the effective bay limits.

(d) The surface area of the bay is usually varying to some degree, but for an initial iteration, the mean water line could be chosen (as well as it can be delineated).

(3) Friction factor.

(a) "f" is a dimensionless parameter that is a function of hydraulic radius R and Manning's roughness coefficient n:

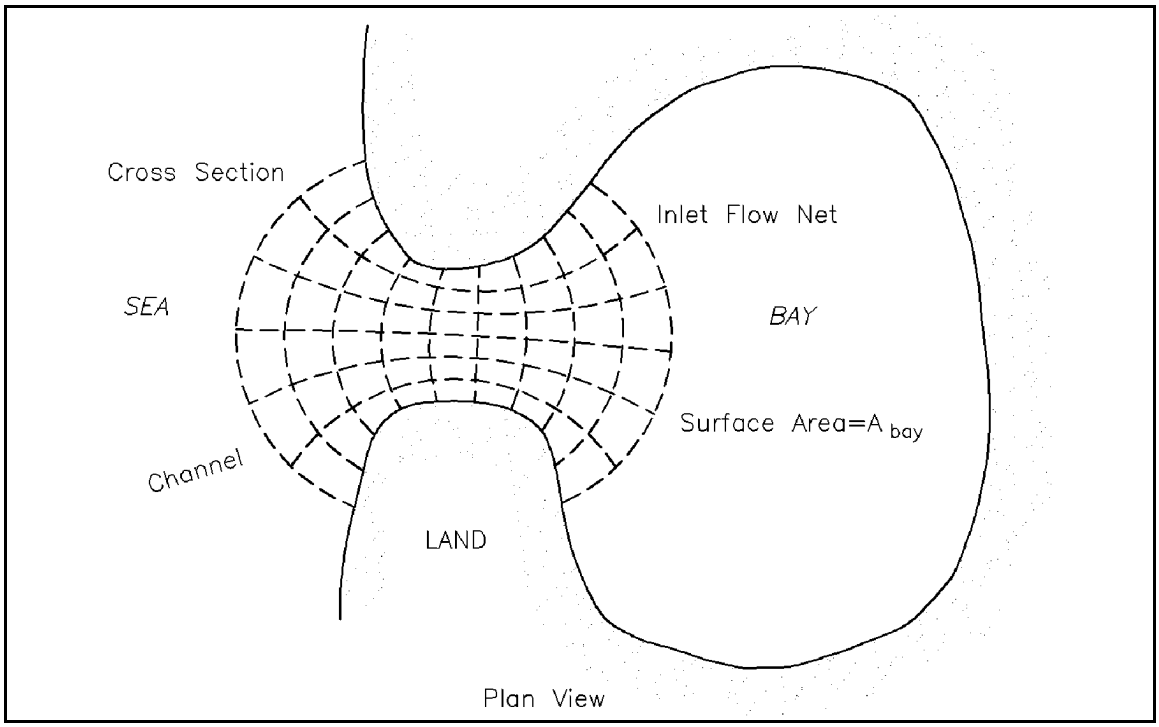


Figure II-6-21. Inlet flow net

$$f = \frac{116 n^2}{R^{1/3}} \tag{II-6-16}$$

(b) It seems reasonable to assume a constant n for an inlet in sand rather than a uniform f value. For open channels in hard-packed smooth sand, Chow (1959) recommends an n value of 0.020 (in English units, or 0.016 in SI). Since inlets often have rippled or duned bottoms, which generally increase frictional resistance, a higher n value might be expected, for instance between 0.025 and 0.030 (English units or 0.021 and 0.025, SI). Using an estimated value of n = 0.0275 (English units or 0.0225 SI), a friction factor defined by $f = 0.088/R^{1/3}$ (English units, R in feet, or $0.059/R^{1/3}$, SI, R in meters) is recommended for use in the equation for K.

Jarrett (1975) examined the friction aspect of Keulagan K (Equation II-6-4) and defined F, inlet impedance, as

$$F = k_{en} + k_{ex} + \frac{fL}{4R} \tag{II-6-17}$$

with k_{en}, k_{ex}, f, and L as defined earlier, and hydraulic radius R measured at the minimum cross section of the inlet. Figure II-6-22 plots inlet impedance against the ratio of inlet length to the 4/3 power of hydraulic radius for four ratios of minimum cross-section inlet width to hydraulic radius. These curves can be used for determining F.

(4) Inlet length. Inlet length is one of the more difficult parameters to define. The following standardized method is recommended to determine inlet length. On a detailed bathymetric chart, beginning in deep water just offshore from the inlet, mark the points of maximum depth moving across the bar, through the inlet, and into the bay. The bayward end point should be seaward of any major division of the inlet

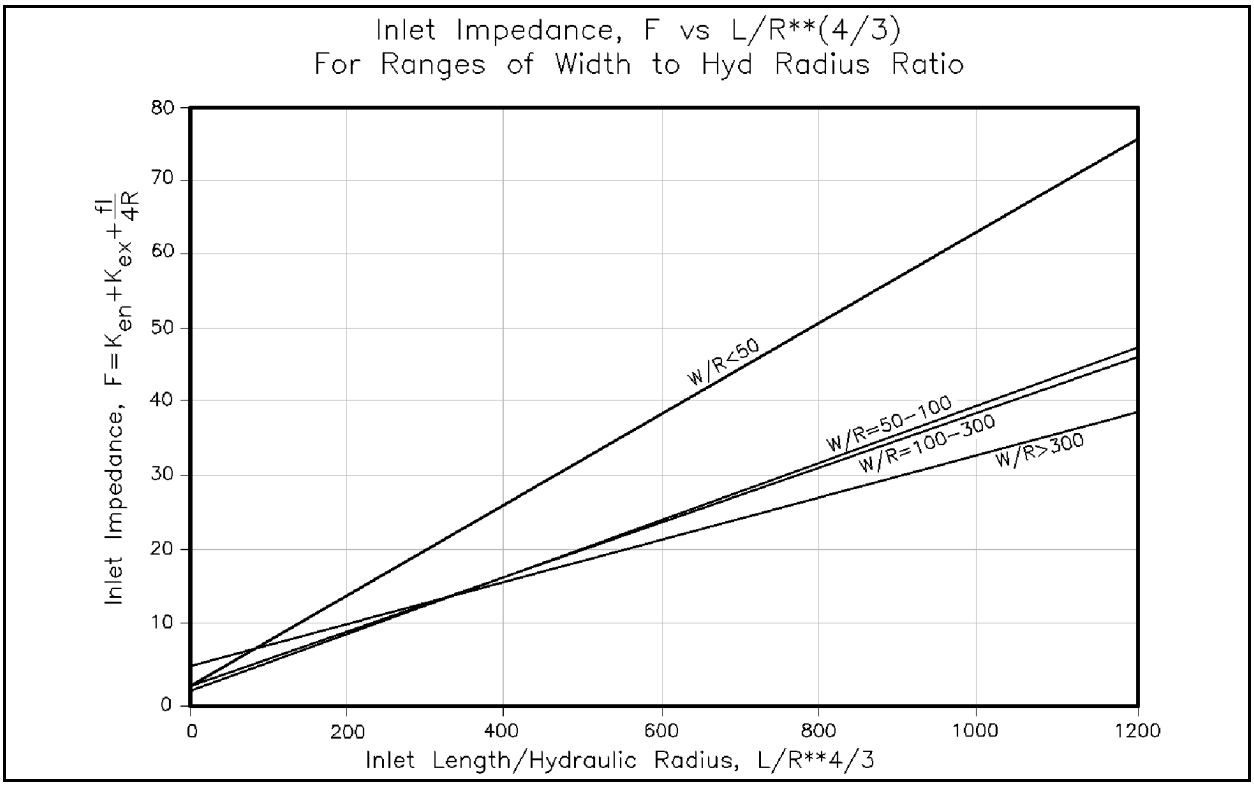


Figure II-6-22. Inlet impedance (*F*) versus the ratio of inlet length to inlet hydraulic radius to the 4/3 power ($l/R^{4/3}$) for various inlet width to inlet hydraulic radius ratios, *W/R*

channel. Connect these points of maximum depth with a series of straight lines to define the thalweg of the channel. Construct a plot of depth versus distance from the oceanward point along the thalweg. Define inlet length as the distance between the depth minimum oceanward of the deepest part of the inlet and the depth minimum bayward of the deepest part. Where the channel does not shoal near either end, the limit of the inlet must be estimated by considering the relationship of the inlet to adjacent beaches, i.e., draw a line perpendicular to the thalweg from MSL on one shoulder of the inlet to MSL on the other, and assume its intersection with the thalweg defines the channel end.

(5) Hydraulic radius. For a given cross-section area, the corresponding value of R should be computed from

$$R = \frac{A_{avg}}{Average\ \ Wetted\ \ Perimeter} \approx \frac{A_{avg}}{Average\ \ Width} \qquad \text{(II-6-18)}$$

These methods are intended to provide a simplified approach to understanding inlet hydraulics in a rapid manner. If possible, field data should be examined to correlate with existing condition calculations, adjustments made to various controlling parameters, and calculations reiterated until reasonable results are achieved. Then calculations may proceed to evaluate design changes. It must be remembered that for extreme events, when wind and wave breaking may be significant factors influencing the hydrodynamics of the inlet system, more sophisticated approaches will be needed. Even a relatively well-behaved inlet is only approximately represented by these approaches, but much insight can be gained by these techniques.

EXAMPLE PROBLEM II-6-1

Find:

If a channel has a depth below MSL of 3.7 m (12.1 ft) and a width of 180 m (590 ft), what are the maximum flow velocity, maximum discharge bay tidal range, and the volume of water flowing into and out of the bay on a tidal cycle (tidal prism) for a tide having the spring range?

Given:

A bay with a surface area of 1.9×10^7 m^2 (2.0×10^8 ft^2) and an average depth of 6.0 m (19.7 ft) is located on the Atlantic coast. The tide is semidiurnal (T = 12.42 hr), with a spring range of 1.30 m (4.3 ft), as given by the National Ocean Survey Tide Tables (U.S. Department of Commerce 1989). An inlet channel, that will be the only entrance to the bay, is to be constructed across the barrier beach that separates the bay from the ocean. The inlet is to provide a navigation passage for small vessels, dilution water to control both salinity and pollution levels, and a channel for fish migration. The channel is to have a design length of 1,100.0 m (3,609 ft) with a pair of vertical sheet-pile jetties that will extend the full length of the channel.

Solution:

Using King's method, assume k_{en} = 0.1, k_{ex} = 1.0, and f = 0.03: B = 180 m and d = 3.7 m.

$$A_c = Bd = 180\ (3.7) = 666\ \text{m}^2\ (7166\ \text{ft}^2)$$

$$R = \frac{A_c}{(B + 2d)} = \frac{666.}{(180 + 2(3.7))} = 3.55\ \text{m}\ (11.54\ \text{ft})$$

$$F = k_{en} + k_{ex} + \frac{fL}{4R} = 1.0 + 0.1 + \frac{0.03\ (1100)}{4\ (3.55)} = 3.42$$

$$K_1 = \frac{a_o\ A_b\ F}{2\ L\ A_c} = \frac{(1.30/2\ (1.9)\ (10^7)\ 3.42}{2\ (1100)\ (666)} = 28.8$$

$$K_2 = \frac{2\pi}{T}\sqrt{\frac{L\ A_b}{g\ A_c}} = \frac{2\pi}{12.4\ (60\ (60)}\sqrt{\frac{1100\ (1.9)\ 10^7}{9.8\ (666)}} = 0.25$$

From Figures II-6-18, II-6-19, and II-6-20 with the above values of K_1 and K_2

$$\frac{a_b}{a_s} = 0.78$$

$$V_m^{/} = 0.66$$

$$\varepsilon = 53^o$$

(Continued)

Example Problem II-6-1 (Concluded)

Therefore, from Equation II-6-5

$$V_m = \frac{V_m^{/}\ 2\pi\ a_s\ A_b}{A_c\ T}$$

$$V_m = \frac{0.66\ (2)\ (3.14)\ (0.65)\ (1.9)\ 10^7}{666\ (12.42)\ (3{,}600)} = 1.72\ \text{m/s} \quad (5.64\ \text{ft/s})$$

$$Q_m = V_m\ A_c = (1.72)\ (666) = 1145\ \text{m}^3\text{/s}\ (40{,}430\ \text{ft}^3\text{/s})$$

Since $a_b/a_s = 0.78$, $a_b = 0.78\ (0.65) = 0.51$ m (1.67 ft) and the bay tidal range is 0.51 (2) or 1.02 m (3.35 ft).

The tidal prism is

$$2\ a_b\ A_b = 2\ (0.51)\ (1.90)\ (10^7) = 1.94 \times (10^7)\ \text{m}^3 \qquad (6.27 \times 10^8\ \text{ft}^3)$$

If the average depth of the bay is 6.0 m and the distance to the farthest point in the bay is 6.0 km, the time t_* it will take for the tide wave to propagate to that point is

$$t_* = \frac{L_b}{\sqrt{gd_b}} = \frac{6000}{\sqrt{9.8\ (6.0)}} = 782\ \text{sec or } 0.22\ \text{hr}$$

Since this time is significantly less than 12.42 hr, the assumption that the bay surface remains horizontal is quite satisfactory.

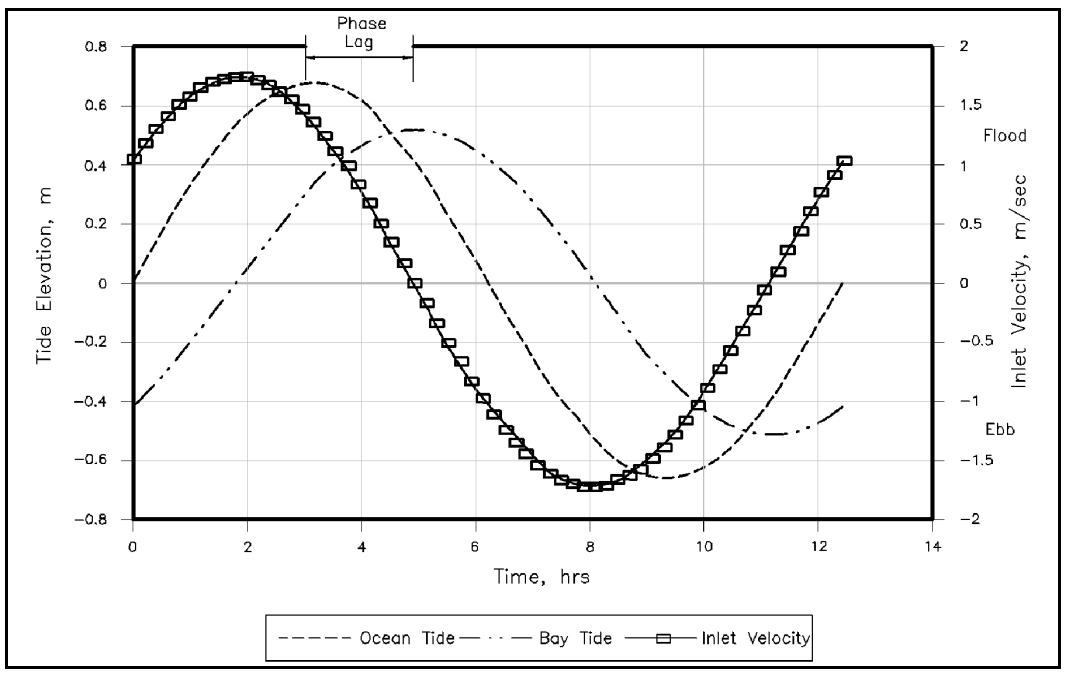


Figure II-6-23. Results from Example Problem II-6-1

e. Evaluating inlet hydraulics with Keulegan K.

(1) Introduction. The Keulegan K value can be used to understand effects of gross geometric variations on inlet hydraulics. Figure II-6-24 shows ocean and bay tide curves and velocity in the channel during a tidal cycle. Three K values are represented. The lower the K or filling coefficient, the lower the tide range in the basin and the greater the phase lag (or time difference between high water (low water) in the ocean and high water (low water) in the bay. Also noted from the velocity curve, maximum current occurs during maximum head difference between ocean and bay tides and as K decreases, maximum ebb and flood currents shift from occurrence at a mid-tide level to tidal elevation extremes. For $K = 0.2$, maximum flood currents occur near high water and maximum ebb currents occur during low water. Therefore for low K inlets, flood flow occurs when depths in the channel and over tidal deltas are greatest (near high water), and flow is more broadly distributed. During ebb flow, depths are shallower (approaching low water) and flow is likely to be more channelized. As K increases, and the bay fills more completely, peak ebb and flood flows tend to occur near the same tide level. These variations can have implications with regard to sediment transport. Mota Oliveira (1970) found that, using an analysis similar to Keulegan but with variable entrance cross-sectional area, when $0.6 < K < 0.8$, the inlet had maximum sediment flushing ability. When $K > 0.8$, there was flood dominance in sediment transport capability, meaning net bayward transport and when $K < 0.6$, there was ebb dominance of sediment transport capability. Also when a variable bay surface area was considered, ebb sediment transport was enhanced to the detriment of flood transport. Part II-6-2, paragraph *h* contains more information on flow dominance.

(2) Jarrett's classification. Jarrett (1975) performed a "hydraulic classification" of inlets based on Keulegan's K after determining tidal characteristics of a large number of U.S. inlets (see Table II-6-1). The classification is as follows:

(a) Class I: Keulegan $K < 0.3$. Phase angle (ε) equal to or greater than 70 deg. For a semidiurnal tide, the phase lag between the ocean tidal extreme and slack water is equal to or greater than 2 hr 25 min. For the most part, bays associated with inlets in this class are open and relatively shallow with only one relatively small inlet (i.e., small ratio A_c/A_b connecting the ocean with the bay. This class also could include relatively long estuaries that are actually long-wave embayments.

(b) Class II: Keulegan $K > 0.80$. Phase angle (ε) equal to or less than 40 deg. For a semidiurnal tide, the phase lag would be equal to or less than 1 hr 25 min. Bays associated with this class are either short or consist of a system of relatively deep channels (Note: by virtue of the long tidal period in the Gulf of Mexico most Gulf Coast inlets fall into this class irrespective of bay shape). The ratio A_c/A_b is relatively large for these inlets.

(c) Class III: $0.3 < K < 0.8$. Phase angle lies between 40 and 70 deg. Characteristics are intermediate to classes I and II. Analysis by Mota Oliveira (1970) indicated that the natural flushing ability of a vertical bank lagoon reaches a maximum for K values of 0.6 to 0.8.

f. Effect of freshwater inflow.

(1) If a significant amount of fresh water is introduced into the bay relative to the tidal prism of the inlet, there will be significant changes in bay tide range and velocity through the inlet. King (1974) included freshwater inflow into a Keulegan type model (which did not include the inertial term in the 1-D equation of motion). He defined a dimensionless variable

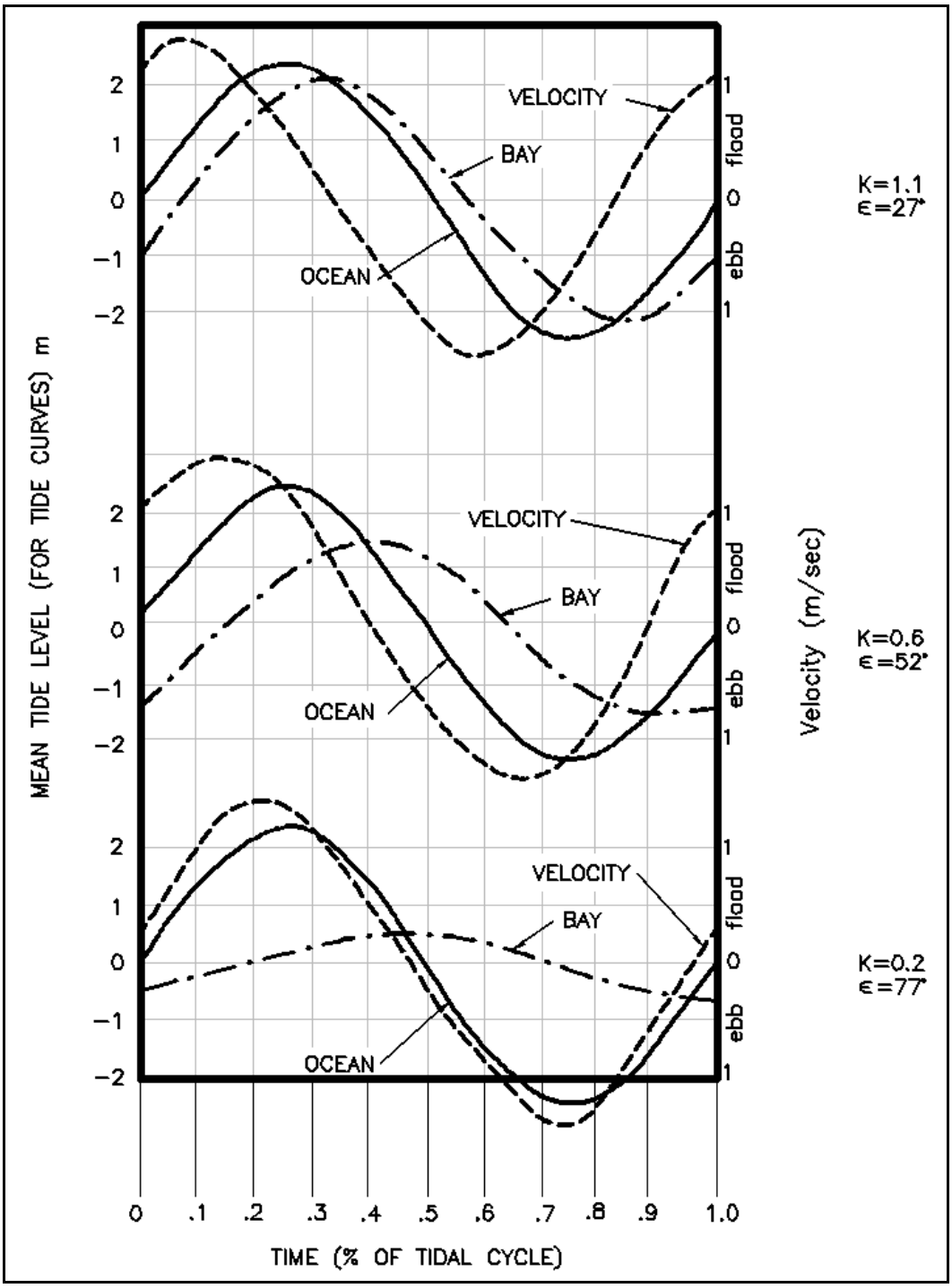


Figure II-6-24. Hydraulic response of inlet and bay tide phasing and bay tide amplitude for various Keulegan *K* values

Table II-6-1
Hydraulic Characteristics of Tidal Inlets by Cubature Method

Inlet	Mean Tidal Prism P, ft^3	Bay Surface Area A_b, ft^2	Avg Phase Range in Bay, $2a_b$	Ocean Tidal Range $2a_o$	a_b/a_o	K	ε deg
			Atlantic Coast				
Shinnecock	1.82×10^8	3.65×10^8	0.50	2.90	0.17	0.15	80
Fire Island	1.59×10^9	2.69×10^9	0.59	4.10	0.14	0.14	83
Jones	7.89×10^8	4.48×10^8	1.76	4.50	0.39	0.35	67
East Rockaway	4.03×10^8	1.06×10^8	3.80	4.50	0.84	0.95	33
Rockaway	2.24×10^9	4.64×10^8	4.83	4.70	1.03	--	--
Manasquan	1.40×10^8	4.91×10^7	2.85	4.30	0.66	0.65	49
Barnegat	4.91×10^8	1.34×10^9	0.37	4.20	0.09	0.09	85
Indian River	4.00×10^8	4.20×10^8	1.02	4.10	0.25	0.22	75
Beaufort	4.20×10^9	2.56×10^9	1.64	3.60	0.46	0.41	63
New River	1.59×10^8	5.31×10^8	0.30	3.60	0.08	0.08	86
Winyah Bay	2.47×10^9	9.14×10^8	2.70	4.60	0.59	0.56	54
Port Royal Sd	1.25×10^{10}	2.40×10^9	5.22	6.60	0.78	0.82	40
Calibogue Sd	3.05×10^9	5.28×10^8	5.78	6.60	0.88	1.07	28
Wassaw Sd	3.34×10^9	5.84×10^8	5.72	6.90	0.83	0.95	33
Ossabaw Sd	5.82×10^9	1.17×10^9	4.98	7.20	0.69	0.70	46
Sapelo Sd	6.36×10^9	9.66×10^8	6.59	6.90	0.94	1.30	20
St. Catherines Sd	5.94×10^9	1.04×10^9	5.73	7.10	0.81	0.89	36
Doboy Sd	3.43×10^9	5.19×10^9	6.62	6.80	0.97	1.50	15
Altamaha Sd	2.45×10^9	5.01×10^8	4.90	6.60	0.74	0.75	43
St. Simon Sd	5.52×10^9	8.51×10^8	6.50	6.60	0.98	1.90	0
St. Andrew Sd	8.34×10^9	1.41×10^9	5.92	6.60	0.90	1.12	26
St. Marys	4.11×10^9	7.93×10^8	5.19	5.80	0.89	1.10	27
Nassau Sd	1.87×10^9	4.40×10^8	4.25	5.70	0.75	0.76	43
St. Johns	1.50×10^9	1.22×10^9	1.23	5.20	0.24	0.21	76
Ft. Pierce	5.10×10^8	1.10×10^9	0.46	2.60	0.18	0.16	80
Lake Worth	7.00×10^8	4.00×10^8	1.75	2.60	0.67	0.66	48
			Gulf Coast				
Venice Inlet	8.50×10^7	4.43×10^7	1.92	2.60	0.74	0.76	42
Midnight Pass	2.61×10^8	1.29×10^8	2.12	2.60	0.82	0.90	35
Sarasota Bay	2.46×10^9	1.16×10^9	2.12	2.60	0.82	0.90	35
Tampa Bay	1.95×10^{10}	1.01×10^{10}	1.95	2.60	0.75	0.76	43
Pensacola Bay	5.87×10^9	4.65×10^9	1.26	1.30	0.97	1.55	14
Mobile Bay	1.56×10^{10}	1.20×10^{10}	1.30	1.30	1.00	--	0
Galveston	5.94×10^9	8.36×10^9	0.71	2.10	0.33	0.30	70

$$Q_r' = \frac{Q_r T}{2\pi a_o A_b} \tag{II-6-19}$$

with Q_r being the river or freshwater inflow to the bay, and a_0, A_b, and T as defined previously. Figures II-6-25 and II-6-26 give high (or maximum) and low (minimum) dimensionless ratio water levels for values of Q_r' and K (Equation II-6-4). Using Figures II-6-27 and II-6-28, maximum average ebb and flood currents can be found and Figure II-6-29 is used to determine ocean high water to bay high water phase lag.

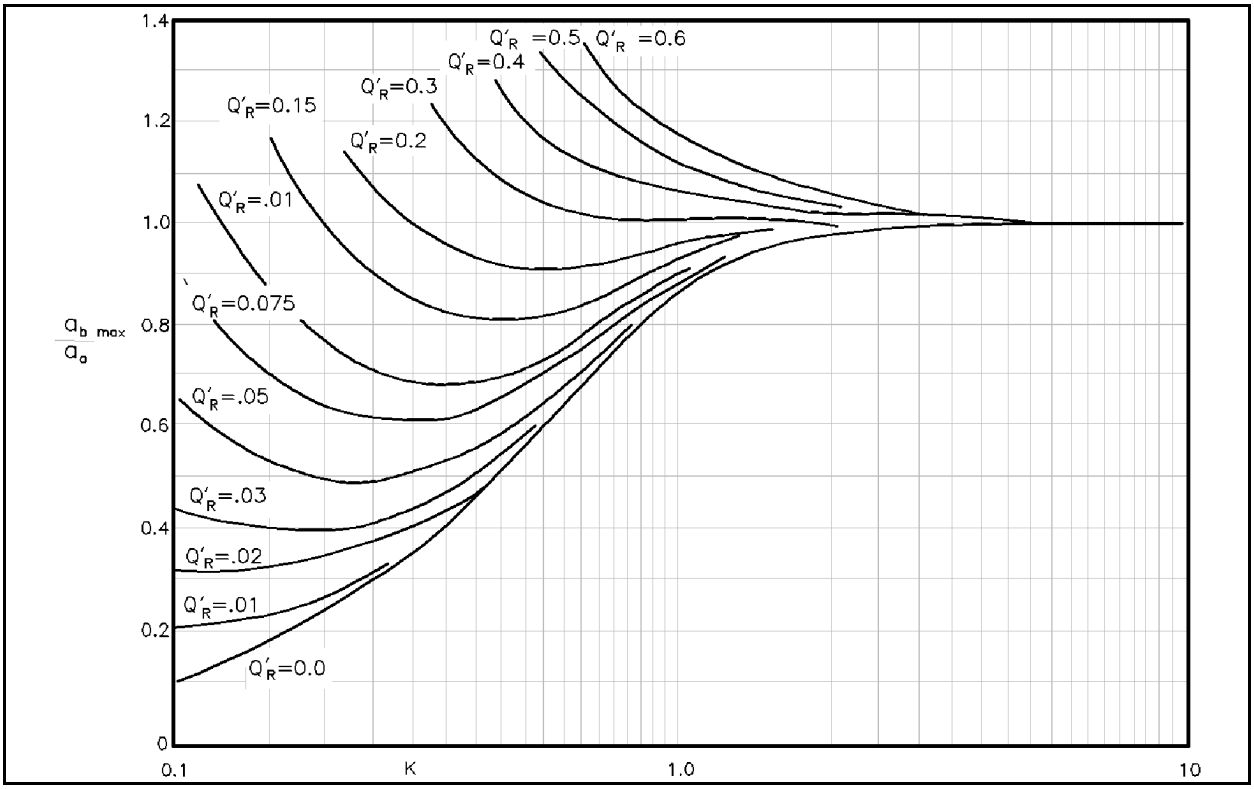


Figure II-6-25. $a_{b\,max}/a_o$ versus K for values of Q_r' (river discharge model)

(2) The above analysis assumes that waters in the inlet are well-mixed at the inlet minimum cross-section, where the salinity variation over a vertical section varies only a small fraction from the local mean salinity. As freshwater influence increases, it approaches a completely stratified flow, i.e., a flow that has a well-defined interface between fresh and saline waters, and in which flow may be simultaneously bidirectional (Wicker 1965). More sophisticated techniques should be used for such cases.

g. Bay superelevation.

(1) A superelevation of the bay means that the average level of the bay is greater than the average elevation of the ocean over a given time period. An obvious cause of this effect is the introduction of fresh water into the bay from local rivers and streams. The effect of long-term (months or years) rise in sea level causing a long-term rise in bay level is not included in this definition of bay superelevation, since both sea and bay are responding simultaneously. The bay may not rise at the same rate as the sea due to possible circulation effects if multiple inlets connect the bay to the ocean.

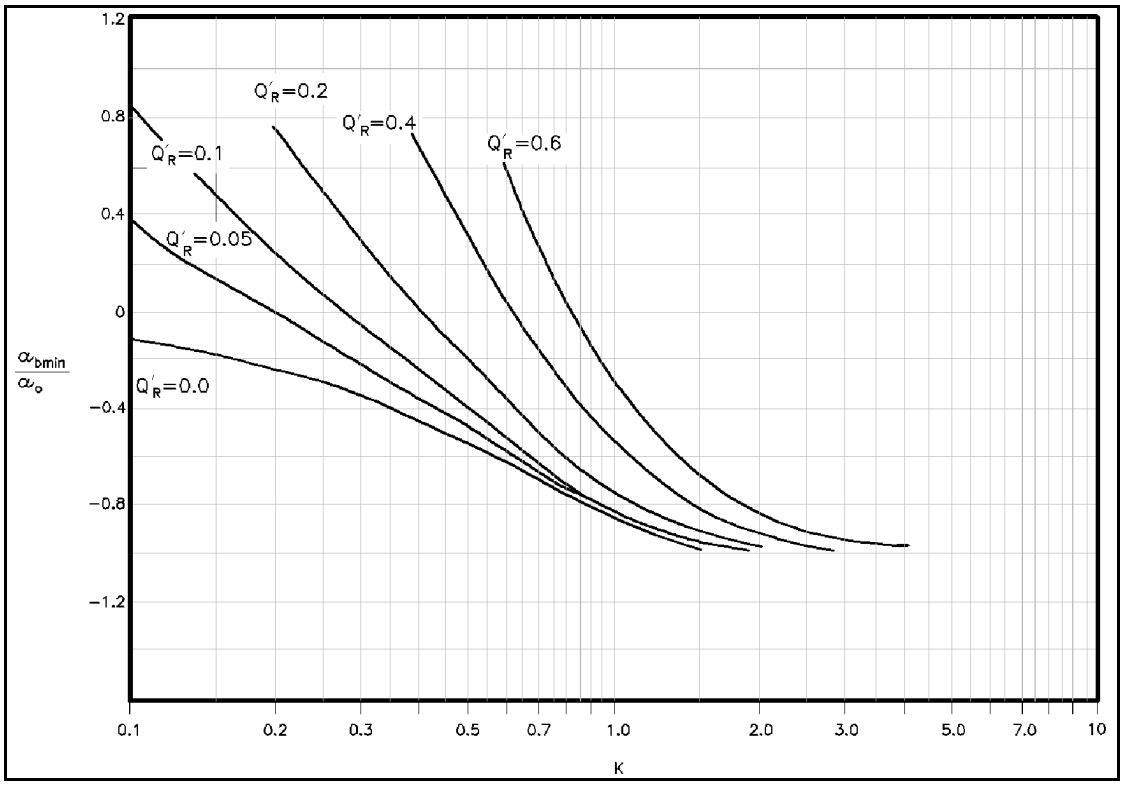


Figure II-6-26. $a_{b\,min}/a_o$ versus K for values of Q_r' (river discharge model)

(2) Laboratory and numerical studies (Mayor-Mora 1973; Mota Oliveira 1970) have indicated that there is an increasing bay superelevation as the coefficient K decreases (Figure II-6-30) and approaches nearly 20 percent of the ocean tide range. This basic cause of setup is due to increased frictional dissipation of ebb flow in comparison to flood flow as K decreases, with peak ebb flows in the channel occurring during lower water levels for inlets with low K values. This increased tractive stress for ebb flow relative to flood flow creates the setup, or increased head, necessary to preserve continuity and drive out the same tidal prism that entered the inlet.

(3) Model studies (Mayor-Mora 1973) show that as K decreases (and superelevation increases) the duration of ebb flow increases relative to flood flow in the inlet channel. Keulegan (1967) determined a relation between inflow duration and bay superelevation (due to any reason) given by

$$\frac{\Delta}{a_o} = \frac{\sin\left(\frac{2\pi t_i}{T}\right)}{1 - \cos\left(\frac{2\pi t_i}{T}\right)} \qquad \text{(II-6-20)}$$

where

Δ = superelevation

a_o = tidal amplitude in ocean

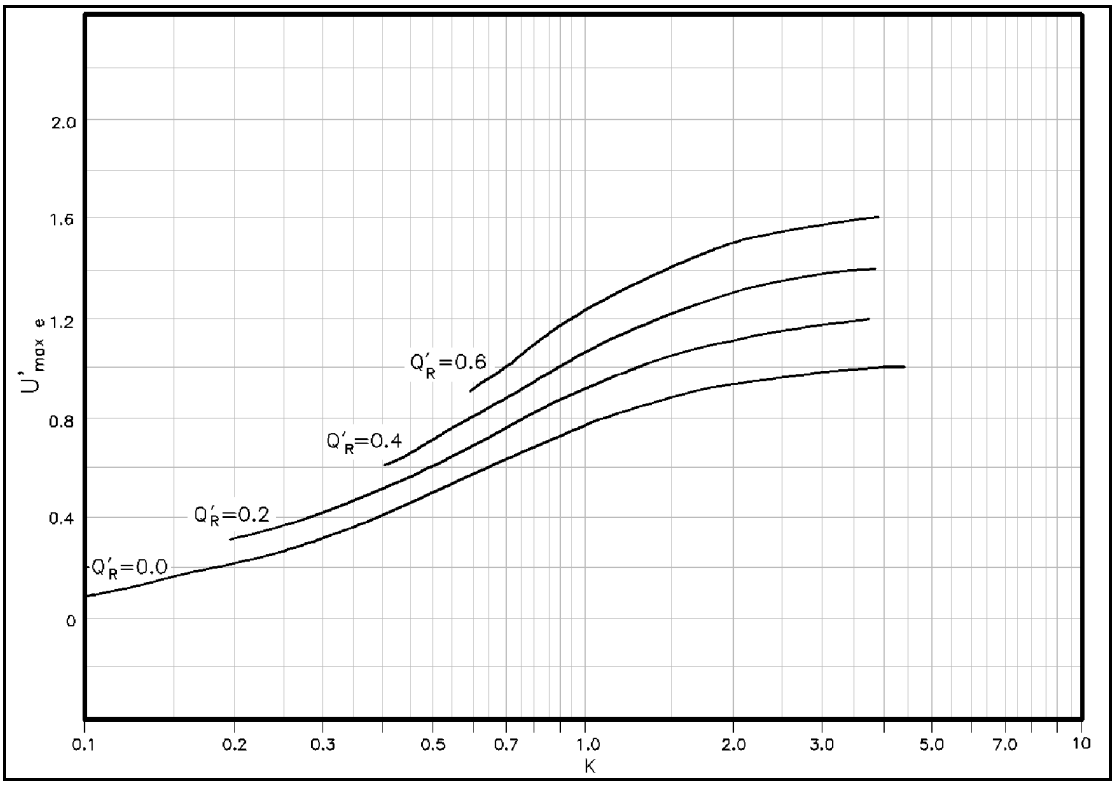


Figure II-6-27. $u'_{max\,e}$ versus K for values of Q_r' (river discharge model)

T = tidal period

t_i = duration of inflow

(4) Figure II-6-31 illustrates this. Mehta (1990) determined that superelevation due to varying entrance channel cross section (by itself only) is

$$\frac{\Delta}{a_o} = 1 - \frac{\left(\frac{a_b}{a_o}\right)^2}{4\left(\frac{d}{a_o}\right)} - \frac{a_o}{mW}\left[\frac{1}{2} - \left(\frac{a_b}{a_o}\right)\cos\varepsilon - \frac{3}{2}\left(\frac{a_b}{a_o}\right)^2 + 4\left(\frac{d}{a_o}\right)^2\right] \qquad \text{(II-6-21)}$$

where

Δ = superelevation

a_o = sea tide amplitude

a_b = bay tide amplitude

d = mean water depth in channel

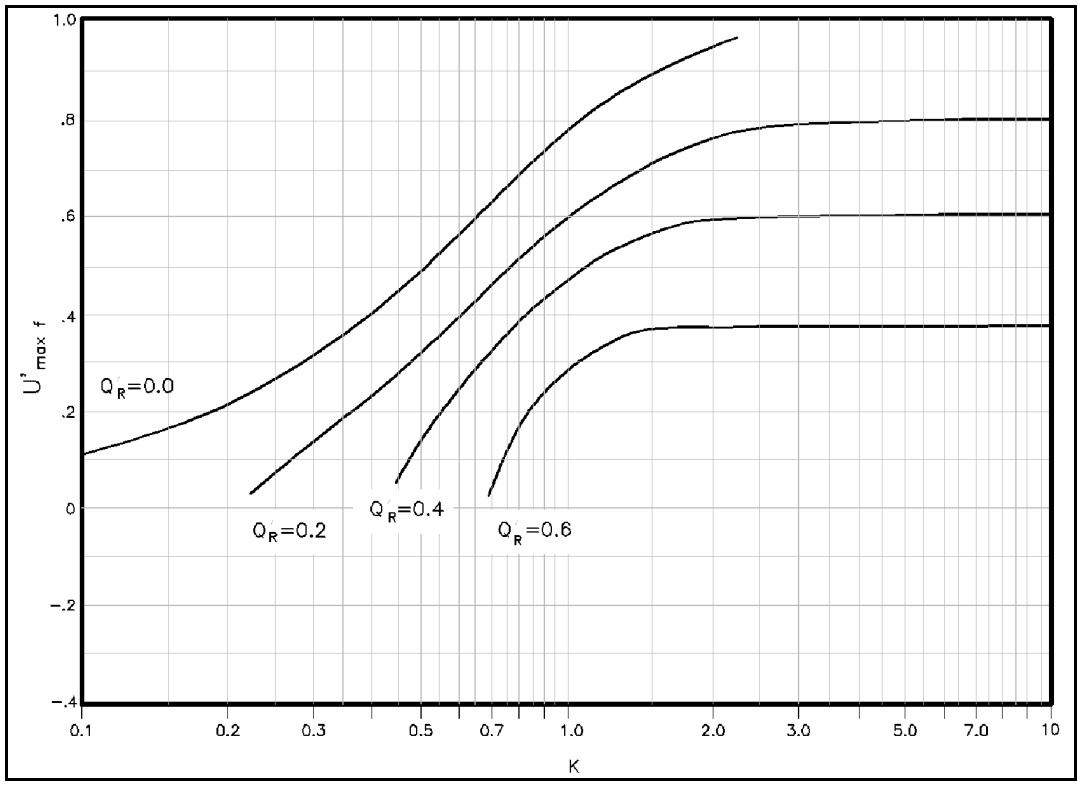


Figure II-6-28. $u'_{max\,f}$ versus K for values of Q_r' (river discharge model)

m = bank slope

W = channel width corresponding to mean water depth in channel

ε = phase lag between sea tide and bay tide

h. The inlet as a filter, flow dominance, and net effect on sedimentation processes.

(1) Introduction. As shown previously, an inlet filters some part of the ocean tide as it translates into the bay, depending on the characteristic inlet parameters defined earlier. Keulegan and King's analytical models, discussed earlier, assumed a sinusoidal ocean tide, and due to the nonlinear friction term (containing u^2), solutions of bay tide response implicitly contained higher-order harmonics (i.e., frequencies higher than that of the basic ocean tide). Other studies have shown that if a variable inlet cross section or variable bay surface area is considered, this will introduce higher-order harmonics. The forcing ocean tide, of course, is composed of many different frequency constituents (Part II-5), so there is interest in using this information to provide more accurate results. Also, investigators have shown that higher harmonics of current velocity (resulting from the forcing tides) are important in relation to sediment transport and its net movement through the inlet (King 1974; Aubrey 1986; DiLorenzo 1988). These effects are important due to the relationship of sediment transport to velocity raised to some power, e.g., V^5 (Costa and Isaacs 1977).

(2) Tidal constituents. Boon (1988) suggests that the seven tidal constituents in Table II-6-2 adequately represent shallow-water tide distortion in inlet-basin systems. Aubrey and Friedrichs (1988) and DiLorenzo (1988) used the amplitude of the ratio of M4 to M2 constituent amplitudes as a measure of nonlinear distortion. Variability in the spring-neap cycle, as well as seasonal mean water level changes, cause characteristic fluctuations in M4/M2.

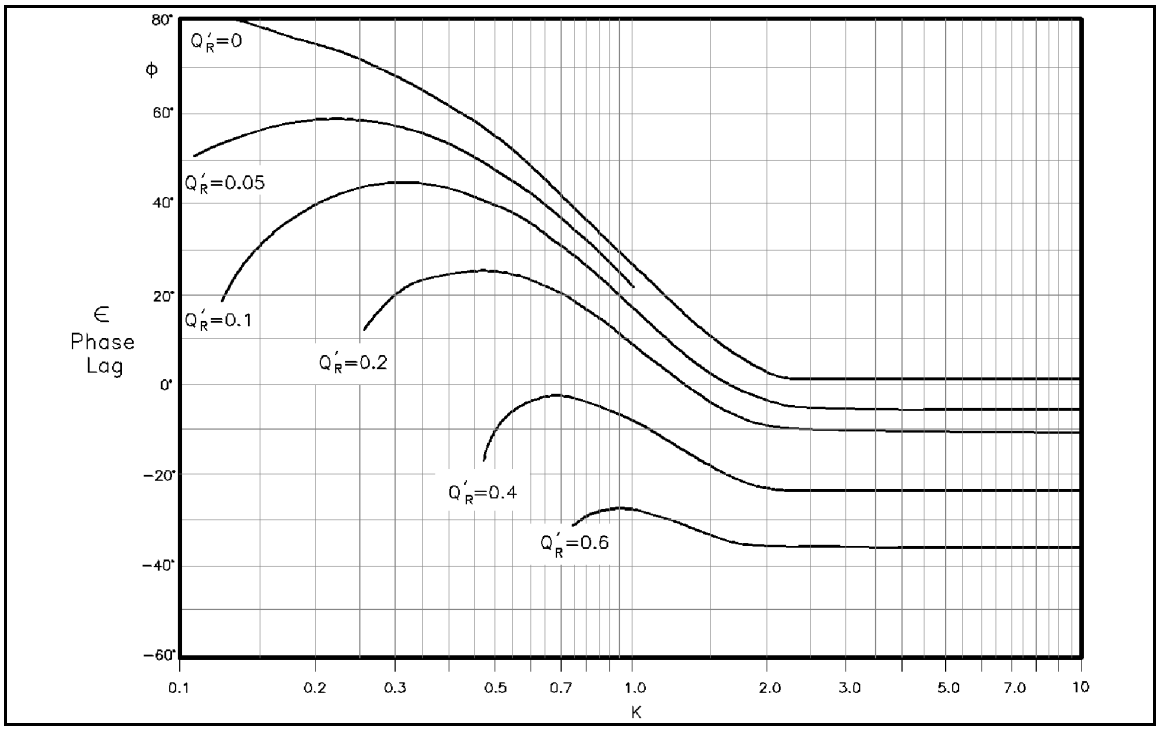


Figure II-6-29. ε versus *K* for values of Q_r' (river discharge model)

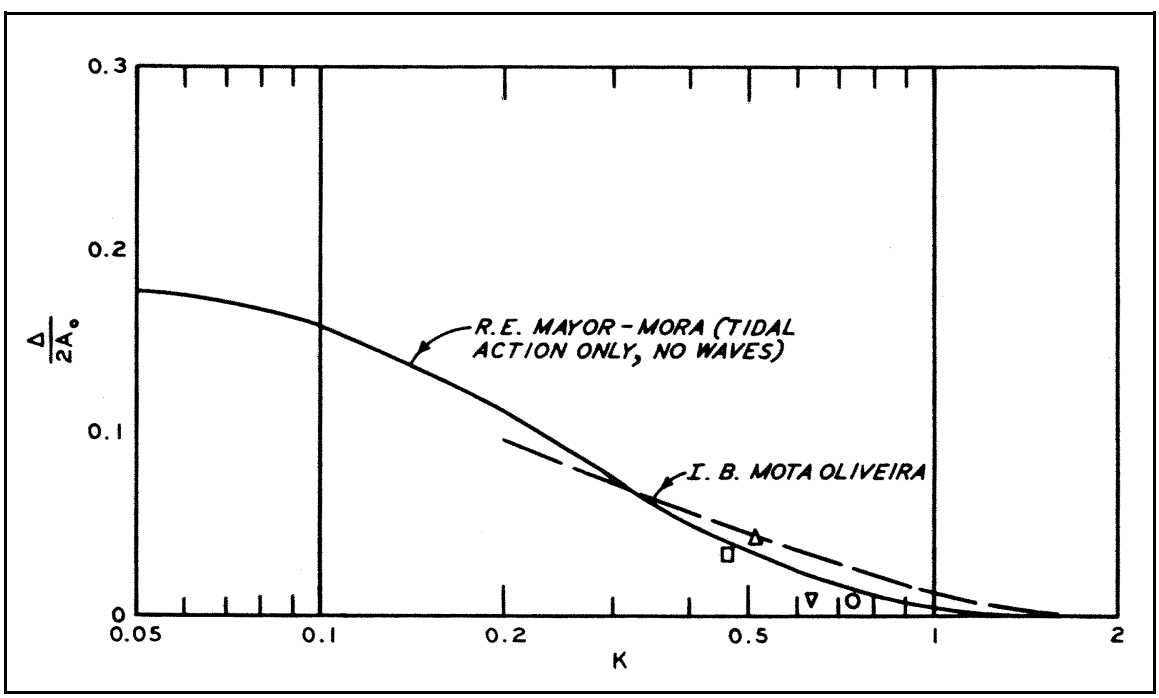


Figure II-6-30. Ratio of bay superelevation to ocean tide range as a function of the coefficient of repletion *K*

EXAMPLE PROBLEM II-6-2

Given:

Using the same inlet system discussed in Example Problem II-6-1, consider the effect of a constant river discharge of 350 m^3/s (12,360 ft^3/s).

Find:

What are the effects of river discharge on high-water and low-water elevations in the bay, bay tide range, and maximum flood and ebb velocities, the maximum flood and ebb discharges, and the bay tide phase lag.

Solution:

Using Equation II-6-19 for dimensionless variable Q_r' with a_o, A_b, and T from Example Problem 1 and Q_r given above

$$Q_r' = \frac{Q_r T}{2\pi a_o A_b}$$

$$Q_r' = \frac{350\ (44712)}{2(3.14)\ (1.30/2)\ (1.90\ x\ 10^7)} = 0.20$$

Using Equation II-6-11, find the Keulegan K, knowing K_1 and K_2, as calculated in Example Problem 1.

$$K = \frac{1}{k_2}\sqrt{\frac{1}{K_1}}$$

$$K = \frac{1}{0.25}\sqrt{\frac{1}{28.8}} = 0.75$$

(or calculate Keulegan K from Equation II-6-4).

With $K = 0.75$ and $Q_r' = 0.20$, bay tidal response is determined from Figure II-6-25

$$\frac{a_{b\ \max}}{a_o} = 0.92 \qquad a_{b\ \max} = 0.92\ (0.65) = 0.60\ \text{m}$$

and from Figure II-6-26

$$\frac{a_{b\ \min}}{a_o} = -0.56 \qquad a_{b\ \min} = -0.56\ (0.65) = -0.36\ \text{m}$$

(Continued)

Example Problem II-6-2 (Concluded)

Bay tide range is 0.96 m and is not symmetric about mean sea level, due to river flow.

Maximum channel velocities are determined from Figure II-6-27 (maximum ebb velocity)

$$U'_{maxe} = 0.80$$

Using Equation II-6-5, and substituting U'_{maxe} for V'_m and U_{maxe} for V_m, and solving for U_{maxe}

$$U_{maxe} = \frac{U'_{maxe}\ 2\pi\ a_o A_b}{A_{avg}\ T} = \frac{0.80\ (2)\ (3.14)\ (0.65)\ (1.90)\ 10^7}{666\ (12.42)\ (3600)} = 2.08\ \text{m/s}$$

From Figure II-6-28 (maximum flood velocity)

$$U'_{maxf} = 0.50$$

and in a manner similar to solving for ebb velocity, flood velocity is determined by

$$U_{maxf} = \frac{U'_{maxf}\ 2\pi\ a_o A_b}{A_{avg}\ T} = \frac{0.50\ (2)\ (3.14)\ (0.65)\ (1.90)\ 10^7}{666\ (12.42)\ (3600)} = 1.30\ \text{m/s}$$

Maximum discharges are determined from

$$Q_{maxe} = U_{maxf}\ A_c = 2.08\ (666) = 1385\ \text{m}^3/\text{s}$$

$$Q_{maxf} = U_{maxf}\ A_c = 1.30\ (666) = 866\ \text{m}^3/\text{s}$$

River flow increases maximum ebb discharge and decreases maximum flood discharge, as one would expect. The values are nearly balanced around the no-river-discharge value of 1,145 m^3/s of Example 1.

Using Figure II-6-29, the value of ε (phase lag of bay tide relative to ocean tide) is determined to be 19 deg, indicating high water occurs earlier than for the non-river-inflow condition of Example 1.

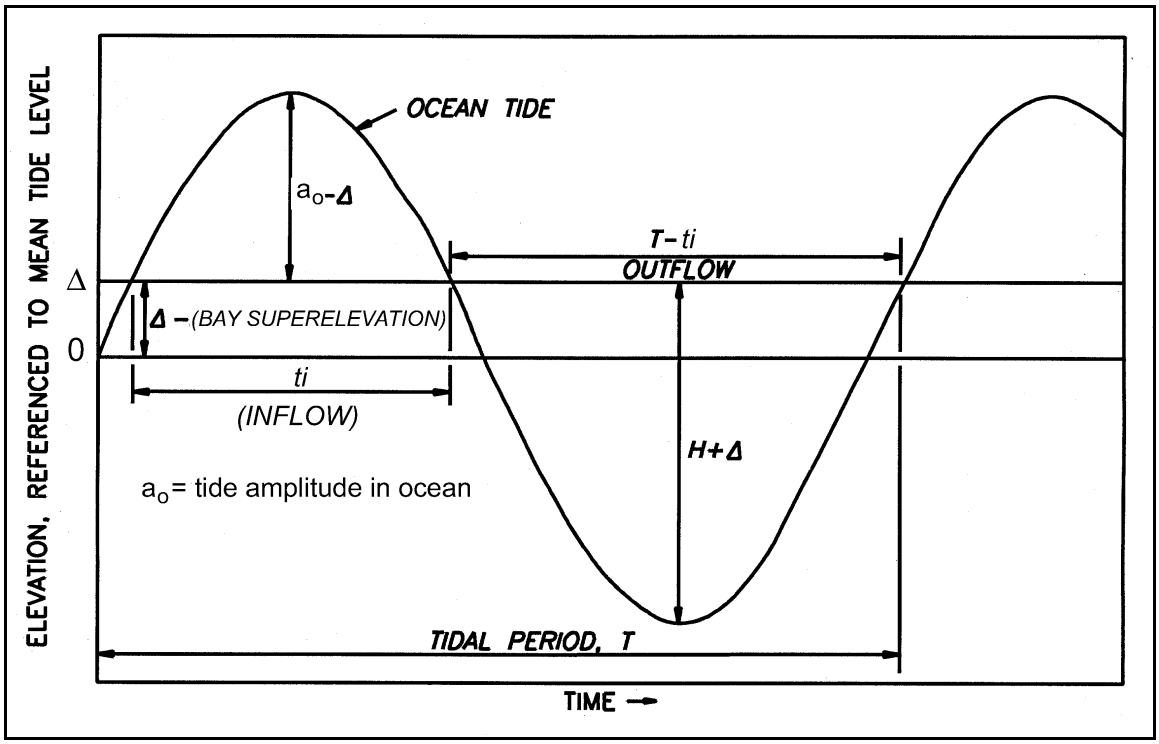


Figure II-6-31. Duration of outflow relative to bay superelevation

Table II-6-2
Semidiurnal and Shallow-Water Tidal Constituents and Harmonic Frequencies

Constituent	Frequency	Degrees/hr	Designation
M_2	a_1	28.9841	Lunar semidiurnal
S_2	a_2	30.0000	Solar semidiurnal
N_2	a_3	28.4397	Lunar elliptic
M_4	$2a_1$	57.9682	Lunar quarter-diurnal
S_4	$2a_2$	60.0000	Solar quarter-diurnal
MS_4	a_1+a_2	58.9841	Compound harmonic
MN_4	a_1+a_3	57.4238	Compound harmonic
MSf	a_2-a_1	1.0159	Lunar long-period

(3) Flow dominance. Entrance area and bay surface area variation impacts flow conditions at the inlet in terms of the continuity equation (Equation II-6-3), which indicates that

$$\frac{dh_b}{dt} \propto \frac{A_{avg}}{A_b} \tag{II-6-22}$$

The rate of change of bay elevation is directly proportional to channel area and inversely proportional to surface area of the bay. If A_{avg} is held constant while A_b increases with the rising tide, then the rate of rise decreases, while the rate of fall increases over time. This means there is a longer rise and faster fall, indicating greater ebb velocity peaks (assuming the same amount of water enters and leaves the bay), which means currents are ebb-dominant when considering only the effect of variable bay area. Similar reasoning

shows that variable entrance area increases the rate of rise of flood tide and decreases the rate fall, causing higher maximum flood velocities than ebb, creating a flood-dominant inlet. These conditions are important to help interpret the movement of sediment through inlets and the locations where shoals form.

i. Multiple inlets.

(1) Often a lagoon is connected to the ocean by more than one inlet. The basic simplified hydraulics can be determined by methods introduced previously or may be numerically modeled for more detailed effects such as residual currents and net circulation (van de Kreeke 1976, 1978). Multiple inlets with a common bay can impact inlet stability if an inlet changes or if an inlet is added (Ward 1982). Changes to a multiple inlet system may be gradual (for example, one inlet gradually lengthens as littoral transport creates shoal regions in the inlet), or change may be rapid (e.g., a hurricane cuts a channel through a barrier beach and begins to rapidly expand in size, capturing a significant portion of the tidal prism from a previously existing inlet connected to the same bay).

(2) For an existing system of multiple inlets, the K values may be added together

$$K_{inlet\ 1} + K_{inlet\ 2} + \ldots\ldots\ldots K_{inlet\ n} = K_{all\ inlets} \tag{II-6-23}$$

to determine the total K for the system, from which the bay tide amplitude and phase lag may be determined (Figures II-6-18 and II-6-20). A total maximum discharge (for the total of all inlets) from the V_{max} determined from Figure II-6-19 is

$$Q_{\max} = \frac{2\pi}{T} a_o A_b V_{\max} \tag{II-6-24}$$

(3) This total maximum discharge then may be apportioned between all inlets

$$Q_{\max} = Q_{1\max} + Q_{2\max} + Q_{3\max} + \ldots.. \tag{II-6-25}$$

with the above equation knowing that, for example, for three inlets

$$Q_{1\max} = \frac{Q_{\max}}{1 + \dfrac{K_2}{K_1} + \dfrac{K_3}{K_1}} \tag{II-6-26}$$

Q_{2max} and Q_{3max} may be determined by placement of the appropriate K values in the denominator in the same relative positions as in the above equation.

(4) For the case of an inlet connected to a bay that is segmented into separate internal bays connected by narrow openings, reference is made to Dean (1971) and Mota Oliveira (1970).

j. Tidal jets. Some analytical approaches to flow patterns at inlets treat parameters affecting the "plume" of water exiting the inlet (i.e., ebb flow). Joshi (1982) examined the effect of bottom friction, bathymetric changes, and lateral entrainment of water due to turbulent mixing. Generally, as bottom friction increases, the plume becomes wider and water velocity along the center line decreases. As bottom slope increases, the plume contracts. Özsöy (1977) and Özsöy and Ünlüata (1982) developed relationships to determine plume expansion and center-line velocity, as shown in Figures II-6-32 and II-6-33. Example

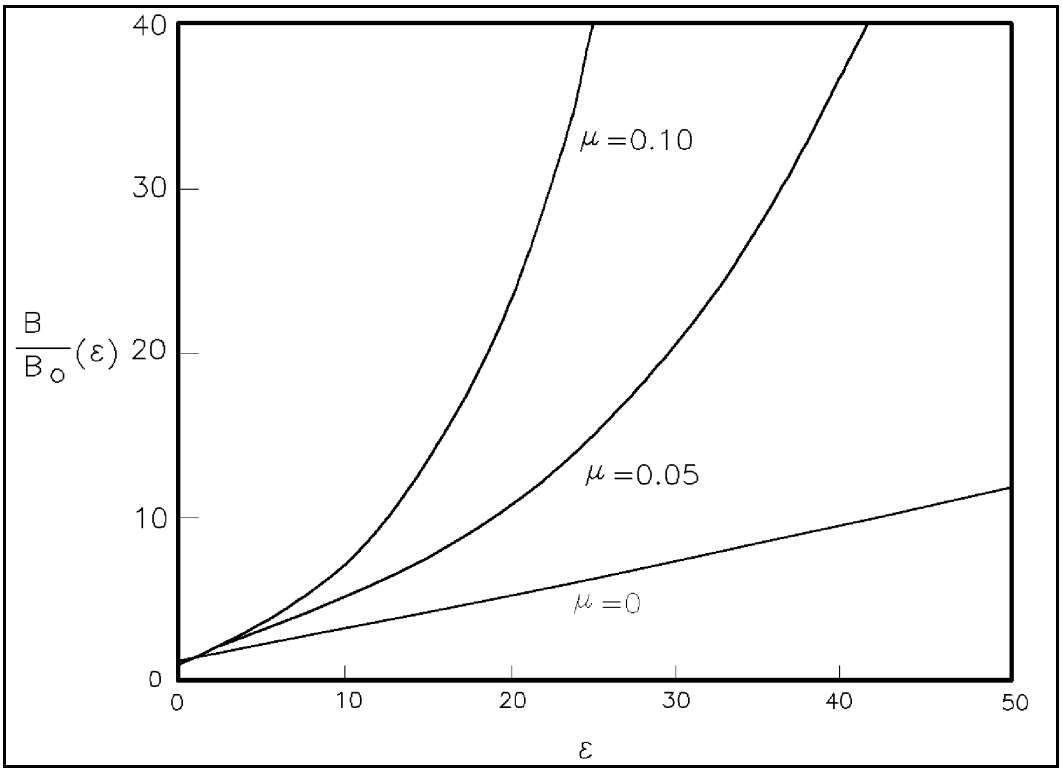


Figure II-6-32. Plume expansion

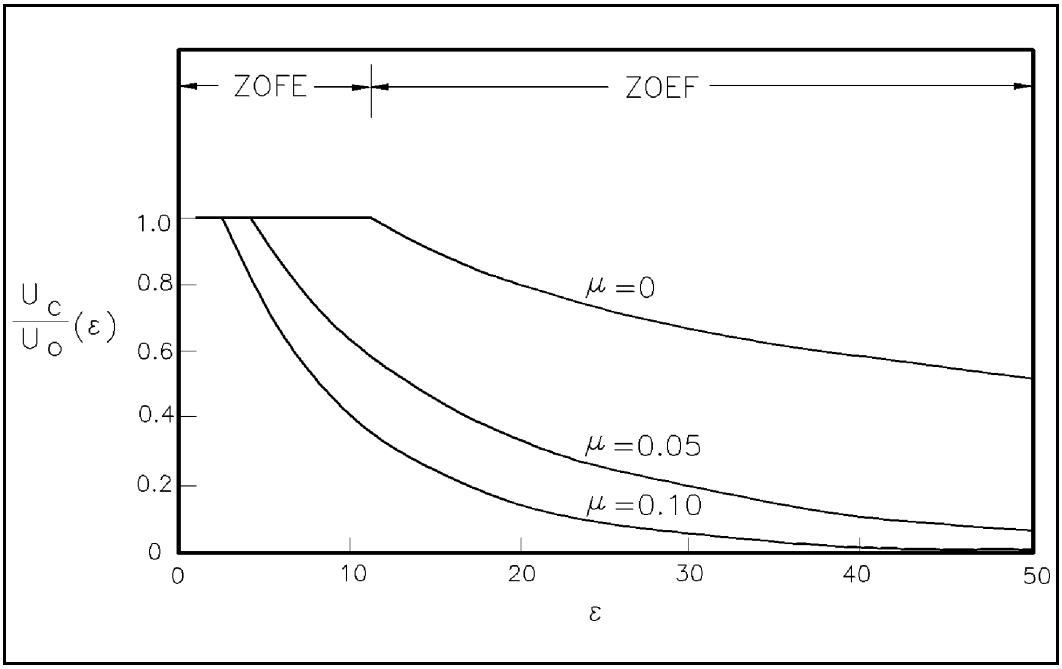


Figure II-6-33. Center-line velocity

Problem II-6-2 shows the effects of a flat bottom with friction. Özsöy and Ünlüata provide additional information for sloping bottoms and the effect of a longshore current approaching perpendicular to the ebb jet, deflecting it downcoast. Joshi and Taylor (1983) examined the alongshore circulation created by tidal jets and predicted that currents toward the inlet along the shoreline (induced by the ebb tidal jet or plume) can range from 0.01 to 0.1 m/s within 10 half-widths (of the minimum inlet width) of the natural inlet entrance. With a jetty in place, the range of induced currents along the shoreline is reduced, finally becoming zero at the junction of the shoreline and jetty. The above work is based on a simplified bathymetry and does not include wave-induced longshore currents, which can be an order of magnitude larger than jet-induced longshore currents. Ismail and Wiegel (1983) found that waves increased the rate of ebb jet spread.

k. Tidal dispersion and mixing.

(1) Flow through inlets can play an important role in the exchange processes and flushing of the bay, both of which impact water quality and bay ecology. Tidal currents, along with wind-driven currents and in some cases density-driven circulation, can drive dispersion processes in the bay where particles of water (or other substances, e.g., pollutants) are scattered or diluted. Tidal dispersion and its role in flushing of bays has been studied by many investigators including Geyer and Signell (1992), Zimmerman (1988), van der Kreeke (1983), Dyer and Taylor (1973), and Sanford, Boicourt, and Rives (1992). There is typically a broad variance in the dispersion coefficient values that can be determined for the processes causing dispersion over a large area. Therefore, analytic techniques to determine flushing of a bay are limited to small basins, and assume complete internal mixing, with detailed numerical simulations required for larger bays. Tidal dispersion is important in regions where flow separations occur; e.g., in regions of abrupt geometric change, with the inlet entrance channel a prime example. The extent of dispersive influence is limited to the tidal excursion in the bay (the distance covered by a particle entering the bay at the start of flood tide until the end of flood tide). For tidal dispersion to influence large spatial scales, spacing between major topographic features must be less than the tidal excursion distance (Geyer and Signell 1992).

(2) For small basins, where it is assumed that waters in the basin are completely mixed at all times, van de Kreeke (1983) defined residence time as the average time for a particle to enter, then leave the basin, with this average based on many particles. For small basins, and with the following relationship met

$$10 > \frac{\langle V \rangle}{\varepsilon P} > 1.5 \qquad \text{(II-6-27)}$$

where <V> is the average volume of the bay over the tidal cycle, ε is the fraction of new water entering the bay from the sea each tidal cycle, and P is the tidal prism. *A* value of $\varepsilon = 0.5$ is often used when there is no other basis for estimation of this factor. Then residence time can be calculated by

$$\frac{\tau_r}{T} = \frac{\langle V \rangle}{\varepsilon P} \qquad \text{(II-6-28)}$$

with τ_r residence time and T tidal period. The time to reduce the mass of a substance's concentration by a factor of "e" (2.3) is equal to τ_r and to reduce it a factor of 10 is $2.3\tau_r$.

l. Wave-current interaction. The region where the navigation channel intersects and/or passes through the outer ebb shoal can be a region of intense wave-current interaction. Wave-current interaction can produce difficult navigation and dredge operation conditions. Also, interaction of waves, currents, and local bathymetry/channel typically produces the most severe channel shoaling problems at inlets. These interactions will be discussed in various combinations.

(1) Wave-current interaction.

(a) Horizontal currents can modify surface gravity waves by stretching or shrinking features of a wave train. This distortion can produce a "Doppler Shift" in wave period. Wave orthogonal, crest, and ray directions are modified (see Part II-1 for definitions). Also, a current modifies surface waves by causing an exchange of energy between wave and current. The pressure field accompanying the wave also is modified. Predicting current-modified wave energy, heights, directions, and pressures is typically a complex procedure still under research; however, a graphical method presented by Herchenroder (1981) based on work by Jonsson et al. (1970), can be used to estimate the effect of current on wave height H_A, where this is wave height without current.

Using the equation:

$$H = (R_H)H_A \tag{II-6-29}$$

(b) Figure II-6-34 may be used to determine R_H, the wave height factor, which depends on:

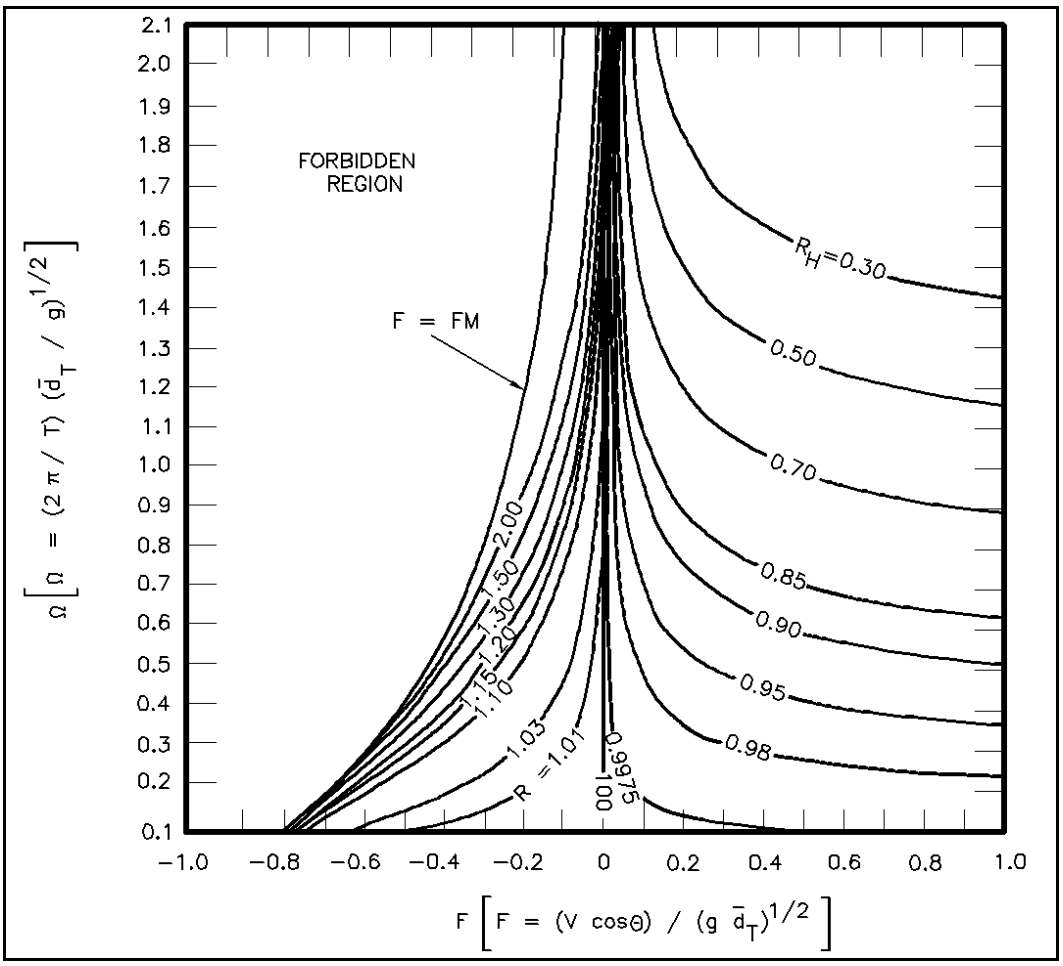


Figure II-6-34. Contours of dimensionless wave height factor R_H given by Equation II-6-29. Waves cannot propagate for values of *F* and Ω lying in the forbidden region (boundary line F = FM)

EXAMPLE PROBLEM II-6-3

FIND:

The width of the ebb jet and its maximum velocity along the centerline 2,000 m seaward from shore.

GIVEN:

The inlet channel width where the ebb jet exits the inlet is 200 m and the channel depth is 8 m. A flat bottom and Mannings n = 0.02 are assumed. The maximum average velocity in the channel is 1.5 m/sec.

SOLUTION:

The friction factor μ (from Özsöy and Ünlüata (1982)) is calculated as

$$\mu = \frac{g b_o n^2}{h_o^{0.75}} = \frac{(9.8)\ (100)\ (0.02)^2}{(8)^{0.75}} = 0.08$$

where b_o = half-width of channel = 200 m / 2 = 100 m and the dimensionless distance offshore parameter is

$$\xi = \frac{x}{b_o}$$

with x, the distance offshore, equal to 2000 m. Calculating these parameters, μ = 0.08 and ξ = 20. Using Figures II-6-32 and II-6-33, and interpolating for μ, the results are

$$\frac{b}{b_o}(\xi) = 18.5 \quad \Rightarrow \quad b = 100\ (18.5) = 1850 \text{ m}$$

and

$$\frac{u_c}{u_o}(\xi) = 0.2 \quad \Rightarrow \quad u_c = 0.2\ (1.5) = 0.3 \text{ m/s}$$

Therefore, for a flat bottom, the half-width of the jet expands from 200 m to 1,850 m and the center-line velocity of the jet decreases from 1.5 m/s at the inlet to 0.3 m/s 2,000 m from shore.

EXAMPLE PROBLEM II-6-4

Find:

Determine the wave height in an inlet channel that is 5.0 m (16.4 ft) deep.

Given:

A 5.0-sec, 1.0-m (3.28-ft) wave is entering an inlet having a 1.0 m/s (3.3 ft/s) ebb current. The angle between current and wave orthogonal is 180 deg.

Solution:

$\Omega = (2\pi/T)\ (d_T/g)^{1/2}$

$F = (V \cos \theta)/(g\bar{d}_T)^{1/2}$

T = wave period = 5 s

d_T = time-averaged water depth = 5.0 m

$g = 9.8\ m/s^2$

V = horizontal current speed = 1.0 m/s

θ = angle between horizontal velocity vector and horizontal wave vector = 180°

Therefore:

$$\Omega = \left(\frac{2\ (3.14)}{5.0}\right)\left(\frac{5}{9.8}\right)^{1/2} = 0.90$$

$$F = \frac{1.0\ (\cos\ 180)}{(9.8 \cdot (5))^{1/2}} = -0.14$$

From Figure II-6-34, $R_H = 1.35$

Wave height H_A, originally 1 m, is modified by the current to a wave height of:

$$H = (R_H)\ H_A = 1.35\ (1.0) = 1.35\ \text{m}$$

$$F = \frac{V \cos \theta}{\left(g d_T\right)^{\frac{1}{2}}} \tag{II-6-30}$$

and

$$\Omega = \left(\frac{2\pi}{T}\right)\left(\frac{d_T}{g}\right)^{\frac{1}{2}} \tag{II-6-31}$$

where

T = wave period

d_T = channel depth

θ = angle wave orthogonal makes with current

V = velocity in channel

(2) Current-channel interaction. As flow converges on an inlet entrance, the angle at which flow approaches a dredged channel can be important with regard to change in current direction and can ultimately relate to channel shoaling. The direction of current approach will depend on bottom configuration and structure(s) location. Boer (1985) developed a mathematical model to study currents in a dredged channel. He found that a current approaching obliquely to a channel is refracted within the channel and the streamlines contract within the channel causing a velocity increase (Figure II-6-35). This effect becomes relatively small for angles larger than 60° (angle between channel axis and current direction). This effect is largest near the bed and smallest near the surface. Due to continuity, depending on the relative depth of the channel to the surrounding depths, there is a decrease factor because of increased depth in the channel.

m . Other methods of inlet analysis. As mentioned in Part II-6-2, paragraph *a,* follow-up studies using techniques presented in previous sections and numerical and/or physical models, can be conducted after an initial examination of the inlet. Some available tools are discussed below.

(1) Automated Coastal Engineering System (ACES). ACES (Leenknecht et al. 1992) contains an inlet model that operates in the PC environment and estimates inlet velocities, discharges, and bay levels as a function of time. This model is designed for cases where the bay water level fluctuates uniformly. Seaward boundary conditions are specified as water level fluctuations associated with astronomical tides, storm surges, seiches, and tsunamis. Figure II-6-36 shows a flow net and Figure II-6-37 is a typical cross section indicating that the model can handle up to two inlet entrances to a single bay and that the entrance channel(s) can be divided into sub-channels to accommodate change in depth across the inlet. The model will permit up to seven sub-channels and up to sixteen cross sections of the flownet. Figures II-6-38 and II-6-39 show sample output from the program.

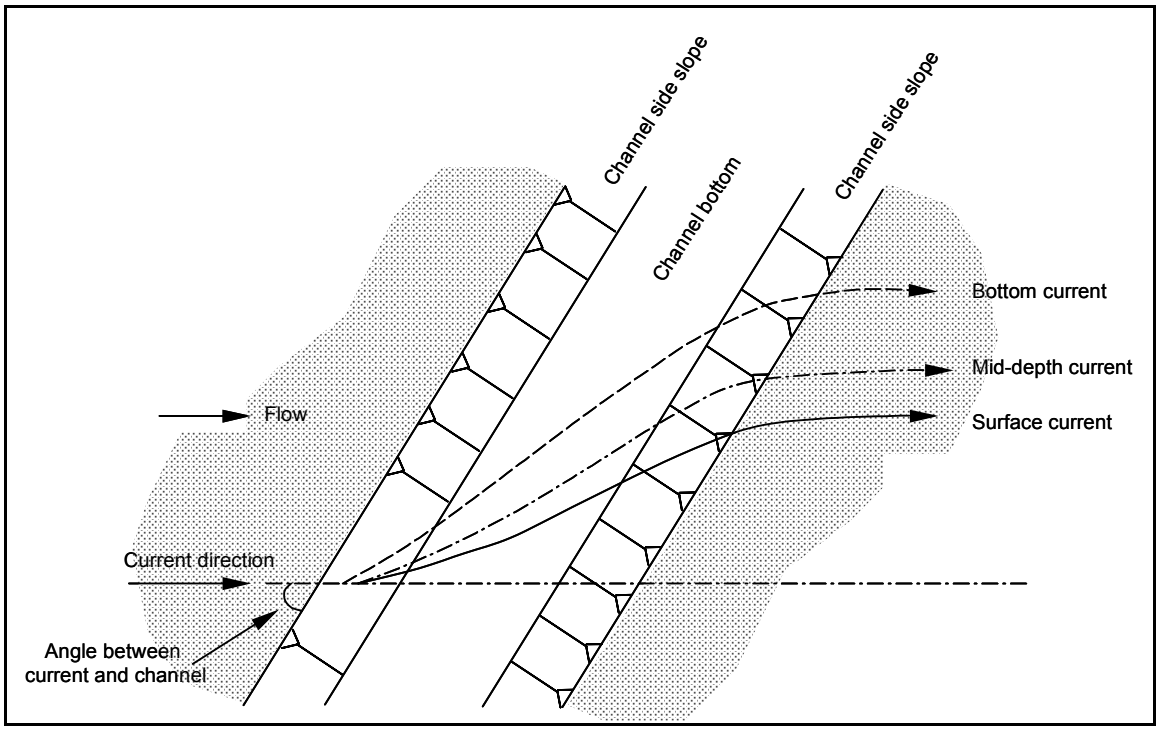


Figure II-6-35. Refraction of currents by channel

(2) DYNLET1. This model (Amein and Kraus 1991) is a one-dimensional PC model that can predict flow conditions in channels with varied geometry, and accepts varying friction factors across an inlet channel. Values of water surface elevation and average velocity are computed at locations across and along inlet channels. The inlet to be modeled may consist of a single channel connecting the sea to the bay, or it may be a system of interconnected channels, with or without bays.

(3) Coastal Modeling System (CMS). The CMS (Cialone et al. 1991) contains two hydrodynamic programs applicable to inlets. This modeling system is a supercomputer-based system of models and supporting software packages. Two models applicable to inlet studies are:

(a) WIFM. This model is a two-dimensional, time-dependent, long-wave model for solving the vertically integrated Navier-Stokes equations in a stretched Cartesian coordinate system. The model simulates shallow-water long-wave hydrodynamics such as tidal circulation, storm surges, and tsunami propagation. WIFM contains many useful features for studying these phenomena such as moving boundaries to simulate flooding/drying of low-lying areas and subgrid flow boundaries to simulate small barrier islands, jetties, dunes, or other structural features. Model output includes vertically integrated water velocities and water surface elevations.

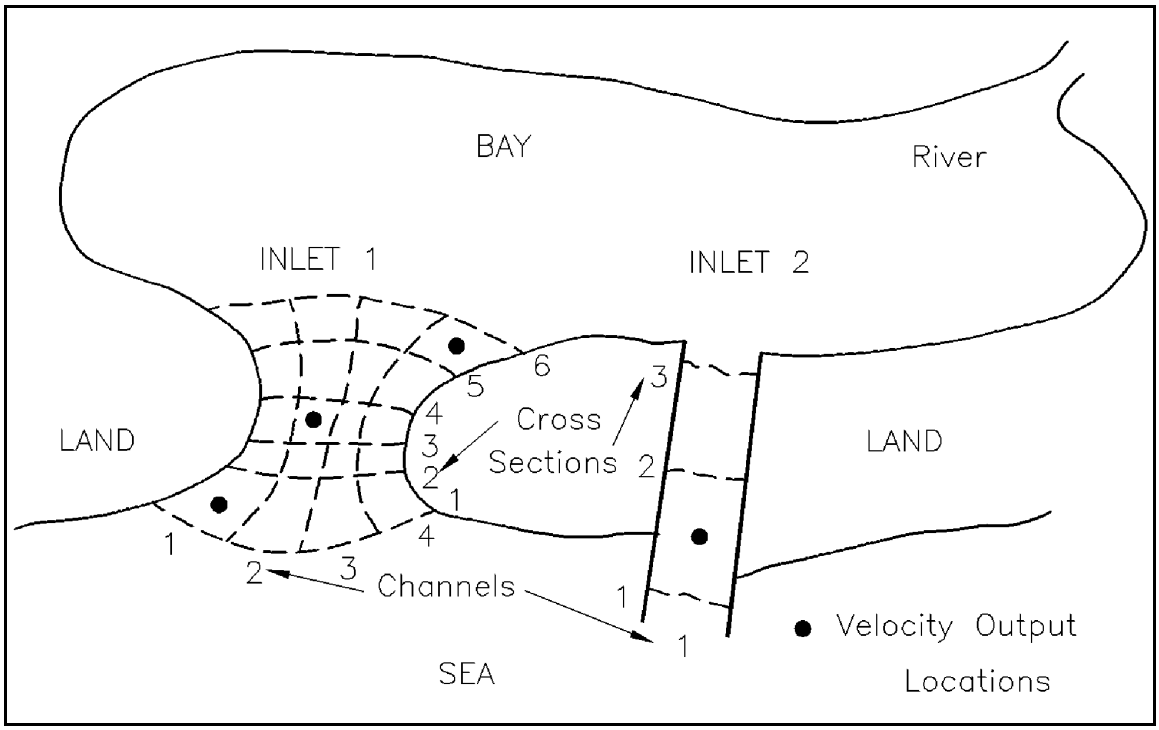


Figure II-6-36. Typical inlet flow nets

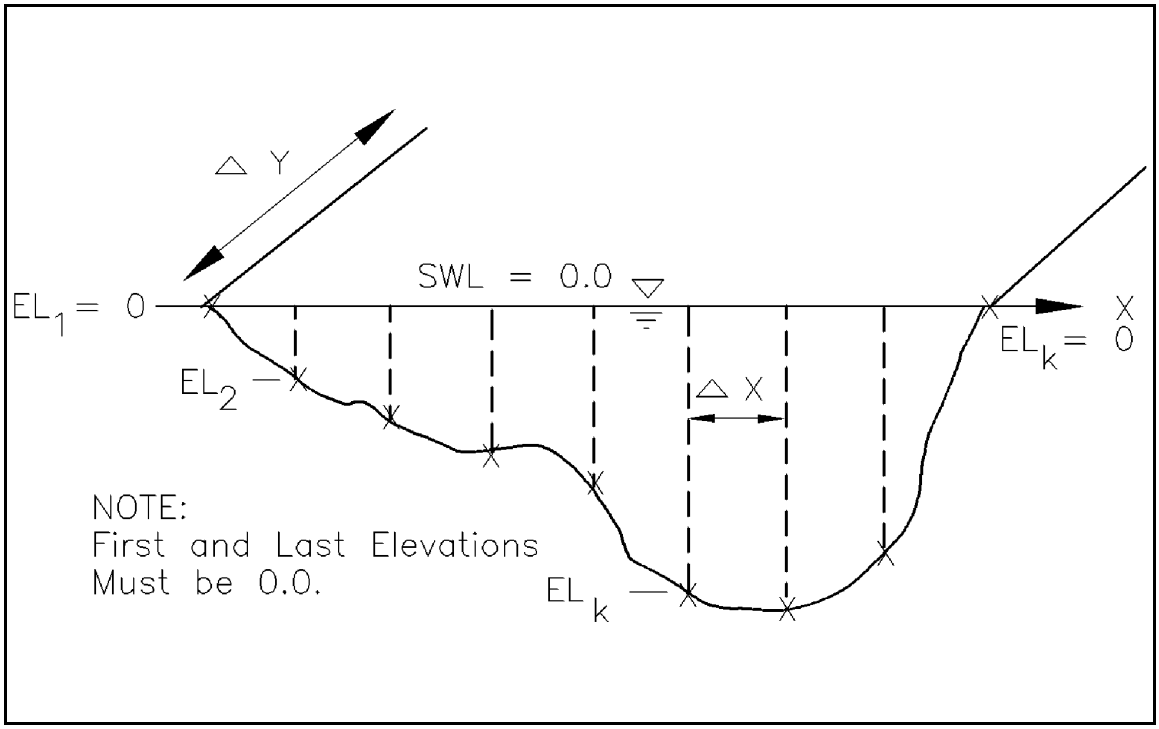


Figure II-6-37. Typical inlet cross section

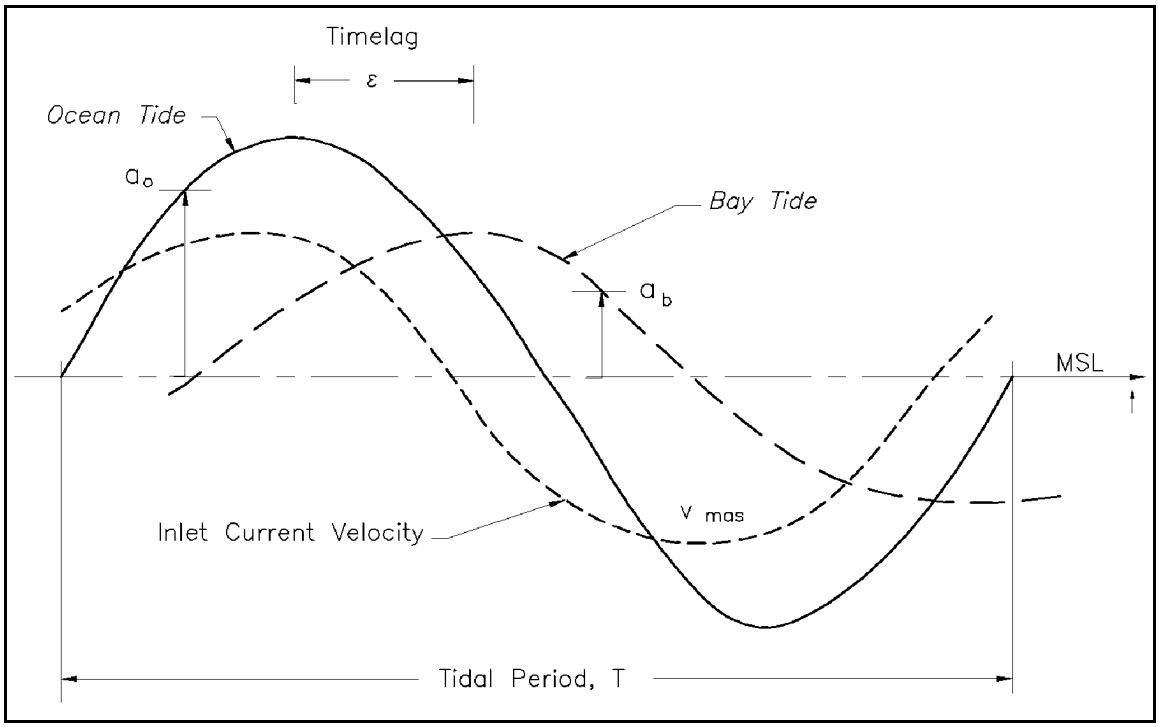


Figure II-6-38. Sea and bay water elevations at inlet 1

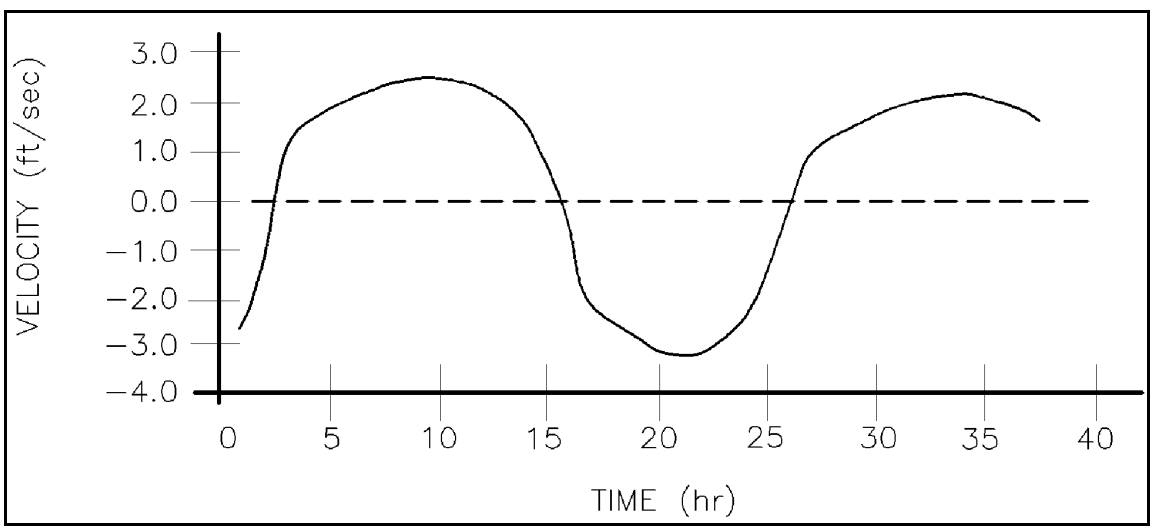


Figure II-6-39. Average velocity at minimum cross section

(b) CLHYD. This model simulates shallow-water, long-wave hydrodynamics such as tidal circulation and storm surge propagation. It can simulate flow fields induced by wind fields, river inflows/outflows, and tidal forcing. CLHYD is similar to WIFM, with the added feature of operating on a boundary-fitted (curvilinear) grid system. However, CLHYD cannot simulate flooding/drying of low-lying areas, as WIFM can. Model output includes vertically integrated water velocities and water surface elevations.

(4) Other models. Other more sophisticated models that require considerable expertise to use already exist or are being developed. The reader is advised to consult current technical literature for details. Part II-5-7, "Numerical Modeling of Long Wave Hydrodynamics," discusses two such sophisticated models, CH3D and ADCIRC.

(5) Physical models. Physical models have also been used for both the hydraulic and sedimentation aspects of inlet studies, but numerical models usually have been used in recent years due to the expense of constructing the large bay system of an inlet and the need to examine processes difficult to reproduce in a physical model, such as storm surge, which would need to include wind effects. However, physical models are appropriate and cost-effective for examining structures and detailed flow patterns in the immediate vicinity of the inlet entrance (sample references include Bottin (1978, 1982), Seabergh (1975), Seabergh and Lane (1977), and Seabergh and McCoy (1982)).

II-6-3. Hydrodynamic and Sediment Interaction at Tidal Inlets

a. Introduction. Waves, currents, and sediment interact at tidal inlets over a large physical area, with varying effects over a tidal cycle (Smith 1991). At the throat or minimum cross-sectional area of the inlet and bayward, tidal currents are the predominant forcing agent interacting with sediments. However, moving seaward and alongshore away from the inlet, the effect of waves increases. Wave energy on the edges of the inlet channel can contribute sediment from adjoining beaches, with the flood tidal currents tending to move bayward to a flood tidal shoal (or middle-ground shoal), or the ebb currents jetting sediment seaward to settle on the ebb shoal. Sediment may again be moved to adjoining beaches by the combination of waves and currents, thus "bypassing" the inlet. The interaction of tidal currents, waves, and wave-generated currents is a complex process.

b. Tidal prism - channel area relationship.

(1) Initial research focused on the relationship to flow through the inlet and the minimum cross-sectional area that could be sustained by that flow. O'Brien (1931, 1969) originally determined a relationship between minimum throat cross-sectional area of an inlet below mean tide level and the tidal prism (i.e., the volume of water entering or exiting the inlet on ebb and flood tide) at spring tide (Figure II-6-40). This relationship was predominantly for Pacific coast inlets, where a mixed tidal pattern is observed and there typically is a strong ebb flow between higher-high water and lower-low water. Jarrett (1976) continued the analysis and performed regression analysis for various coastal areas with different tidal characteristics. A regression equation is given for each of the Atlantic, Pacific, and Gulf coasts. Jarrett also recommended using O'Brien's equation for dual-jettied inlets. The equations, in metric and English units, are given in Table II-6-3.

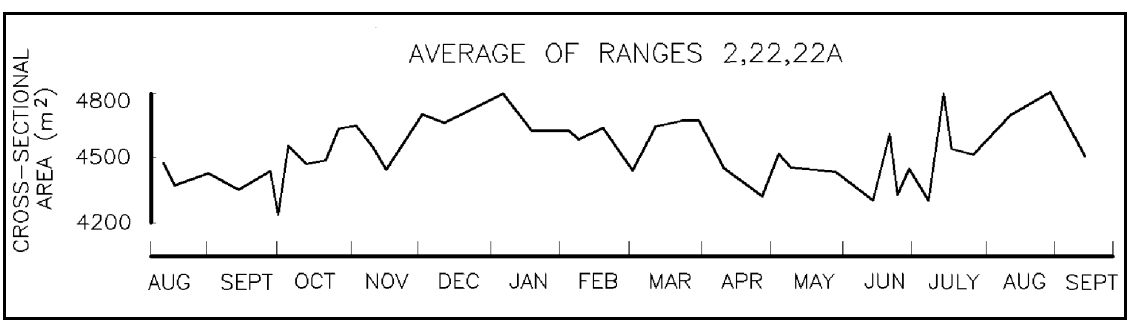


Figure II-6-40. Variations in cross-sectional area for Wachapreaque Inlet (Byrne, De Alteris, and Bullock 1974)

Table II-6-3
Tidal Prism-Minimum Channel Cross-sectional Area Relationships

Location	Metric Units	English Units
Atlantic Coast	$A_c = 3.039 \times 10^{-5}\ P^{1.05}$	$A_c = 7.75 \times 10^{-6}\ P^{1.05}$
Gulf Coast	$A_c = 9.311 \times 10^{-4}\ P^{0.84}$	$A_c = 5.02 \times 10^{-4}\ P^{0.84}$
Pacific Coast	$A_c = 2.833 \times 10^{-4}\ P^{0.91}$	$A_c = 1.19 \times 10^{-4}\ P^{0.91}$
Dual-Jettied Inlets (O'Brien)	$A_c = 7.489 \times 10^{-4}\ P^{0.86}$	$A_c = 3.76 \times 10^{-4}\ P^{0.86}$

A_c is the minimum cross-sectional area in square meters (square feet) and P is the tidal prism in cubic meters (cubic feet).

(2) Inlet tidal prisms versus minimum cross-sectional area for Jarrett's data is plotted in Figure II-6-9.

(3) Work by Byrne et al. (1980) indicated that for inlets with minimum cross sections less than 100 m^2 (1,076 ft^2), there was a departure from the relationships developed above. They studied small inlets on the Atlantic Coast in Chesapeake Bay. Their relationship for area and tidal prism is

$$A_c = 9.902 \times 10^{-3}\ p^{0.61} \text{ (metric units)} \tag{II-6-32}$$

(4) Jarrett's (1976) work pertains to equilibrium minimum cross-sectional areas at tidal channels from one survey at a given date. Byrne, DeAlteres, and Bullock (1974) have shown that inlet cross section can change on the order of ± 10 percent in very short time periods (see Figure II-6-40). This, of course, is due to the variability in tidal currents and wave energy. A storm may bring a large amount of sediment to an inlet region, since the inlet tends to act as a sink for sediment and the tidal current may require a certain time period to return the minimum cross-sectional area to some quasi-equilibrium state. Brown (1928) notes that for Absecon Inlet, New Jersey, "a single northeaster has been observed to push as much as 100,000 cubic yards of sand in a single day into the channel on the outer bar, by the elongation of the northeast shoal, resulting in a decrease in depth on the centerline of the channel by 6 to 7 feet." Sediment changes such as this will affect the hydraulics of the inlet system, which in turn will remodify channel depth and location of sediment shoaling.

(5) Adding jetty structures to natural inlets modifies the inlet's morphology. Careful engineering design can reduce the amount of induced change. For example, jetties are placed so that the minimum cross-sectional area of the inlet is maintained, leaving the volume of water exchanged over a tidal cycle (i.e., the tidal prism) unchanged from the natural state. During design of inlet training structures, the question of appropriate spacing must be answered so that excessive scour does not occur and cause settling or displacement of the structures. Using O'Brien's formula for jettied inlets, the minimum distance between jetties can be calculated. Figure II-6-41 shows lines of average depth expected for given jetty spacing and tidal prism. The data points plotted on Figure II-6-41 describe actual field conditions for 44 inlets. No attempt was made to analyze or judge whether problems existed at a particular project. For example, a very large tidal prism with very narrow jetty spacing may develop problems of erosion along one of the jetties or very high velocities might exist, producing navigational difficulties. Also, if spacing of the jetties is too wide, the channel thalweg may meander in a sinuous manner, making navigation more difficult and reducing efficiency, so as to increase shoaling and lead to possible closure. If the same minimum area is maintained between the entrance channel's jetties that existed for the natural inlet, the tidal prism will be the same, and tides will flush out the bay behind the inlet as well as they did in the natural state. Actually, a more hydraulically efficient channel usually will exist at a jettied inlet because sediment influx is reduced and there are fewer shoals.

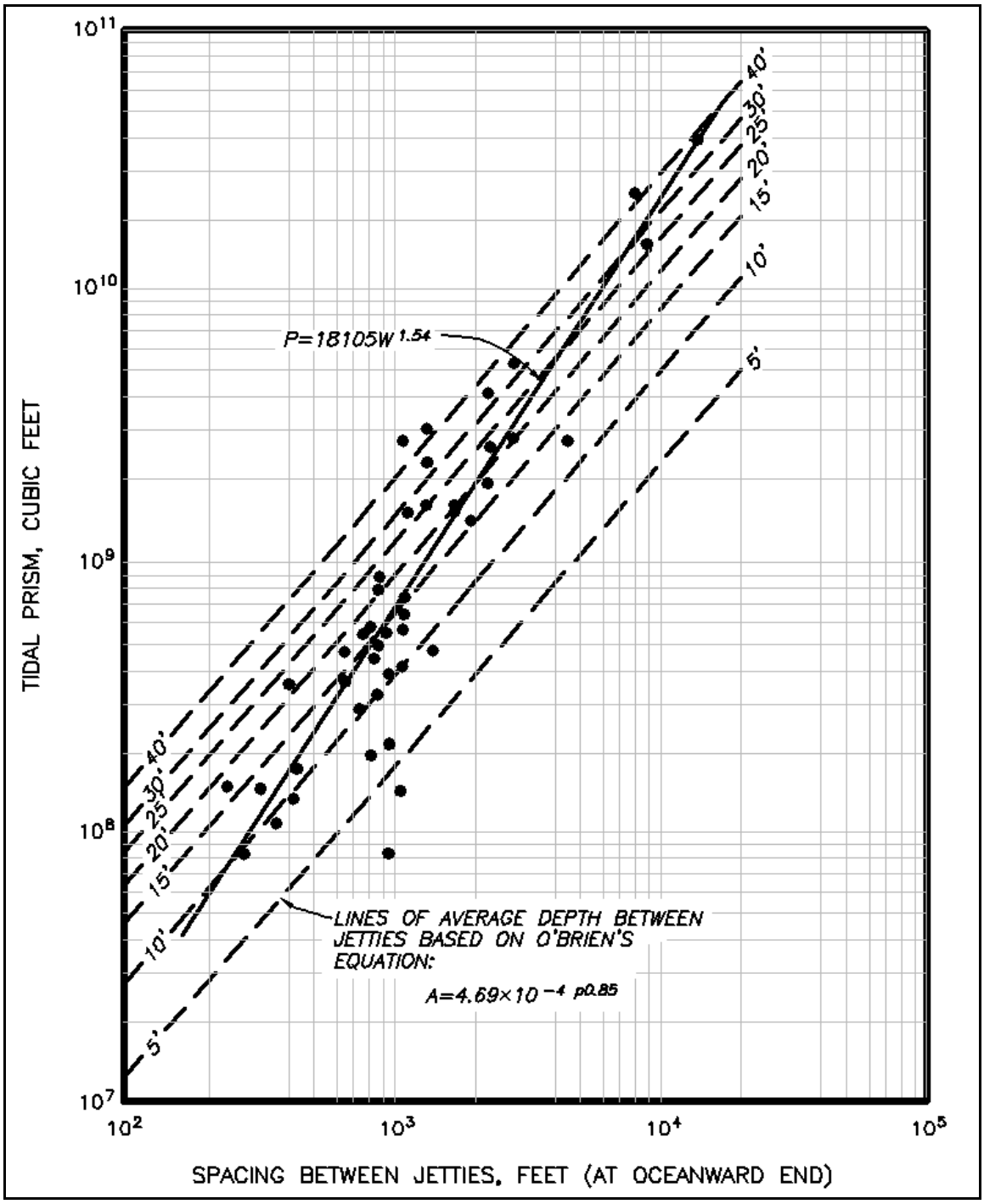


Figure II-6-41. Tidal prism (*P*) versus jetty spacing (*W*)

c. Inlet stability analysis.

(1) Dean (1971) equated the expression for tidal prism, $P = TV_m A_C/\pi$ to O'Brien's (1931) original tidal prism, inlet area relationship ($P = 5 \times 10^4 A_c$) and determined that for a tidal period of 44,640 sec, V_m for the inlet is about 1 m/sec. In other words, 1 m/sec might be interpreted as a level of velocity necessary to maintain an equilibrium flow area. Therefore, as wave action supplies sand to the inlet channel and tends to

reduce the cross-sectional area, the inlet flow will scour out any depositions that reduce the channel cross section below its equilibrium value. This concept was first developed analytically by Escoffier (1940, 1977). He proposed a diagram for inlet stability analysis in which two curves are initially plotted. The first is the velocity versus the inlet's cross-sectional flow area A_c. A single hydraulic stability curve represents changing inlet conditions when ocean tide parameters and bay and inlet plan geometry remain relatively fixed. As area approaches zero, velocity approaches zero due to increasing frictional forces, which are inversely proportional to channel area. As channel area increases, friction forces are reduced but, on the far right side of the curve, velocities decrease as the tidal prism has reached a maximum, and any area increases just decrease velocity as determined by the continuity equation. This curve can be constructed by calculating velocity V_m by varying channel area A_c. V_m can be determined by an analytical or numerical model, remembering that, if using Keulegan or King models, an average maximum velocity is determined. The continuity equation $V_{avg} A = V_m A_c$ must be used to determine maximum velocity at the minimum cross section. The other curve plotted as V_E is a stability criterion curve such as O'Brien and Jarrett's tidal prism versus cross-section area relationship. Escoffier (1940) originally proposed a constant critical velocity (e.g., 1 m/sec, which would plot as a straight horizontal line. If a P versus A_c relationship is used, the appropriate equation (Table II-6-3) can be used to relate V_m to tidal prism. The two curves are shown in Figure II-6-42. The possibilities of intersection of the two curves could possibly intersect at two locations or one location (a tangent), or there could be no intersection at all. In the first case, point b (see Figure II-6-42) is a stable root in that any deviation in area returns movement along the stability curve to its starting point. If channel area increases (move right on curve from point b) velocity will fall and more sediment can fill in the channel to bring it back to "equilibrium." If area decreases, velocity will increase scouring back to the equilibrium point. Point c is an unstable root, where if the area decreases, velocities decrease until the inlet closes. Moving to the right of point c, as velocity increases, area increases until the velocity starts falling and the stable root at point b is reached. If the stability curve falls tangent to or below the stability criterion curve, the inlet will close. Thus, if the inlet area is to the right of the unstable equilibrium point, and a storm occurs that provides a large sediment input to the inlet region, the inlet area could shift to the left of that point, and the inlet would close. van de Kreeke (1992) presents useful commentary on application of Escoffier's analysis where he notes that separation of stable and unstable inlets is not determined by the maximum in the maximum velocity curve (sometimes called the closure curve) of Figure II-6-42, but point c of that curve. Van de Kreeke emphasizes the integral use of O'Brien-type stability correlations with Esscofier's curve, rather than the use of stability equations alone.

(2) Escoffier (1977) presented work similar to the above, except the Keulegan K value was determined for the abscissa of the plot in place of the area. For a given set of parameters for a particular inlet, A_c is varied and K is determined to define the V_m curve. Note: If King's curves are used, K is related to K_1 and K_2 by Equation II-6-11. Figure II-6-43 shows an example diagram. The V_E curve then is determined by selecting an appropriate tidal-prism versus-area relationship and by determining V_{max} for various A values .

(3) A technique by O'Brien and Dean (1972) has also been used to calculate the stability of an inlet affected by deposition. The stability index ß is defined to represent the capacity of an inlet to resist closure under conditions of deposition. It incorporates a buffer storage area available in the inlet cross section, prior to deposition and includes the capability of the inlet to transport excess sand from its throat.

$$\beta = \int_{a_C}^{a_E} (V_{\max} - V_T)^3 \, dA_c \qquad \text{(II-6-33)}$$

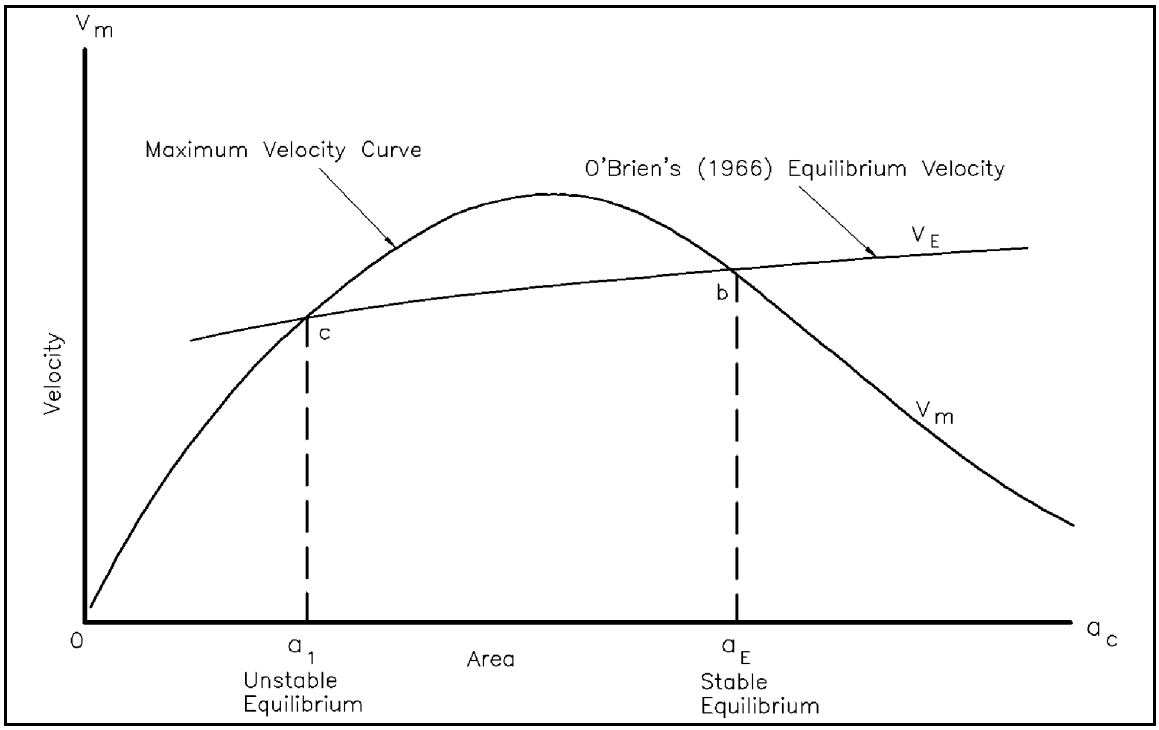


Figure II-6-42. Escoffier (1940) diagram, maximum velocity and equilibrium velocity versus inlet cross-sectional area

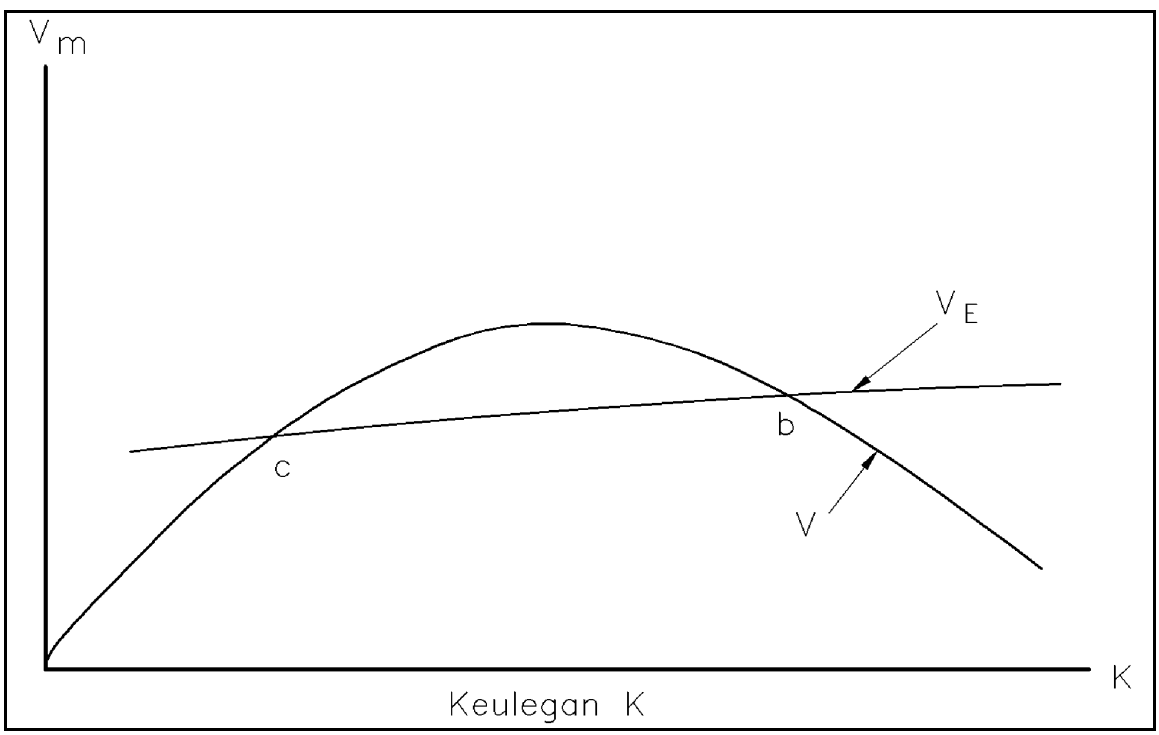


Figure II-6-43. Another version of Escoffier's (1977) diagram, maximum velocity and equilibrium velocity versus Keulegan repletion coefficient *K*

where

β = stability index (units of length^5/time^3)

a_E = equilibrium cross-sectional area of throat

V_{max} = maximum velocity in the throat

V_T = threshold velocity for sand transport

a_c = critical cross-sectional area (as from Figure II-6-42)

(4) Czerniak (1977) applied this technique successfully at Moriches Inlet, New York. Bruun and Gerritsen (1960) and Bruun, Mehta, and Jonsson (1978) developed an overall stability criteria for tidal inlets. They based their relationships on a ratio *P/M,* where *P* is the tidal prism and *M* is the total annual littoral drift (typical values for *M* and their determination are discussed in Part III). The stability of an inlet is rated good, fair, or poor as given in Table II-6-4.

Table II-6-4
Inlet Stability Ratings

$P/M_{tot} \geq 150$	Conditions are relatively good, little bar and good flushing
$100 \leq P/M_{tot} \leq 150$	Conditions become less satisfactory, and offshore bar formation becomes more pronounced
$50 \leq P/M_{tot} \leq 100$	Entrance bar may be rather large, but there is usually a channel through the bar
$20 \leq P/M_{tot} < 50$	All inlets are typical "bar-bypassers"
$P/M_{tot} \leq 20$	Descriptive of cases where the entrances become unstable "overflow channels" rather than permanent inlets

(5) Another aspect of inlet stability was discussed by Vincent, Corson, and Gingench (1991) in which inlet channel geographical location and horizontal topology were studied from aerial photography. They defined four stability indices, including minimum inlet width *W*, channel length *L*, change in geographical position *N*, and orientation variability of inlet channel *E*. Figure II-6-44 shows types of geographic channel instability. Figure II-6-45 shows the Little River Inlet, South Carolina, geographical variability. A stability limit was chosen so as to classify an inlet as stable or unstable. These limits were based on rate of change of width, length, position, and orientation. Records of 51 inlets over various time periods were examined and a change of 100 ft/month was selected as a reasonable arbitrary limit for stability in length and width of *W* and *L*. The 100-ft/month value was also selected as a divider between stable and unstable channel position. Most values examined were below the 100 ft/month rate.

d. Scour hole problems.

(1) Scour hole problems can occur at a variety of inlet locations and much remains to be understood about the mechanisms causing scour. Scour holes form as a result of the interaction of tidal currents, waves, wave-generated currents, sediment and adjacent structures (usually jetties). Scour holes have been observed at the tips of jetties, on the outside of the jetty trunk, and along the inner section of a jetty. Lillycrop and Hughes (1993) review scour problems at Corps projects.

(2) Scour at jetty tips (Figure II-6-47) can be caused by flow separation during flood flow, when turbulent eddies are generated. Ebb currents traversing through this region of scour also might contribute

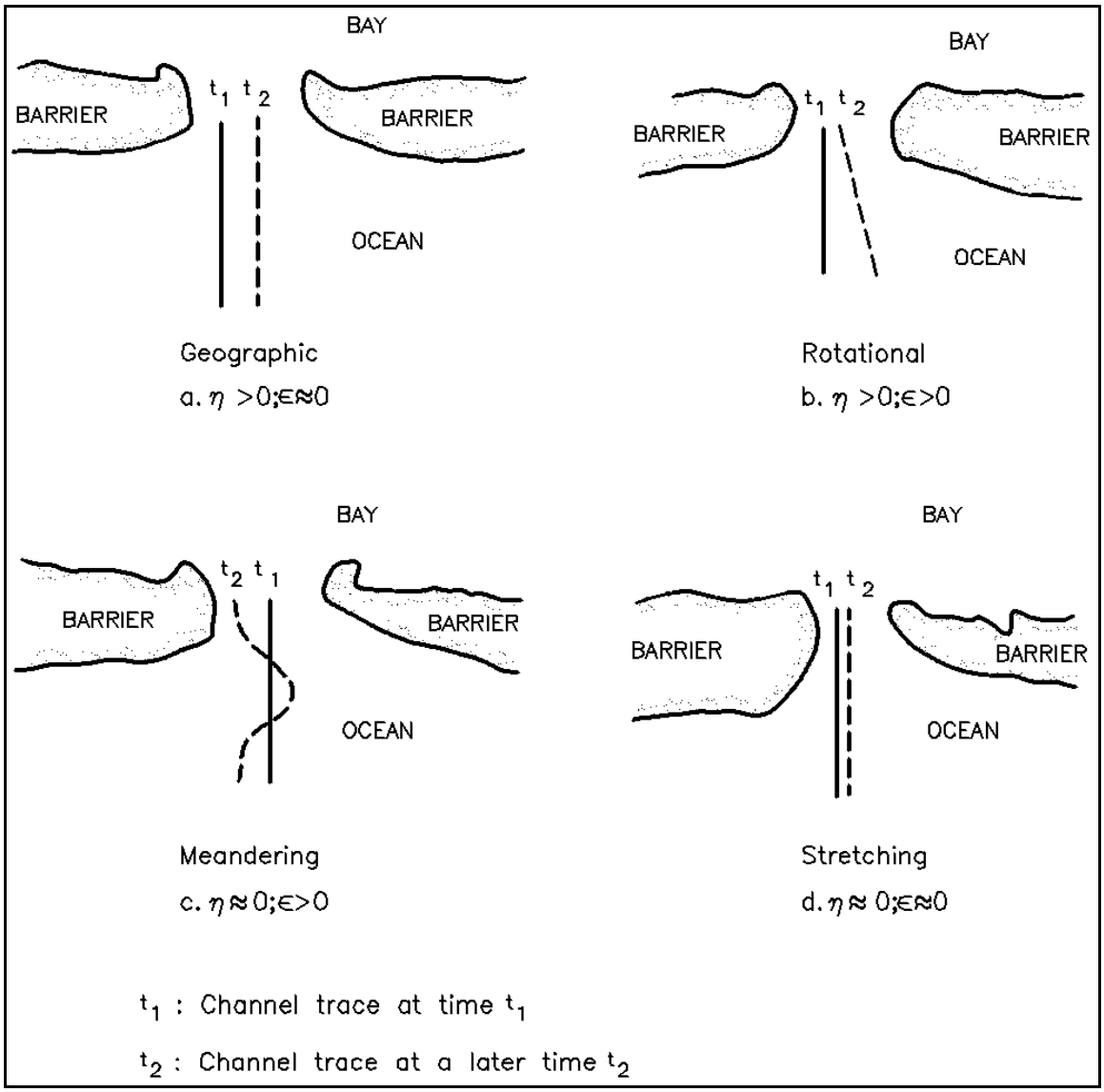


Figure II-6-44. Examples of channel geographic instability

to maintaining a scour hole if channelization in the shoreward portion of the inlet throat directs ebb currents toward the scour hole.

(3) Scour along the inside of jetties can occur as the bathymetry of a newly constructed jetty system adjusts. Little River Inlet (Chasten and Seabergh 1992) had an initial scour hole that was a remnant of the construction process. After the conclusion of construction, scour increased in this area, but as the inlet adjusted, velocities in that scour region decreased.

e. Methods to predict channel shoaling.

(1) Introduction. Entrance channel infill rates can be calculated by a variety of methods (U.S. Army Corps of Engineers 1984). One method is presented to show the parameters involved. A method to determine entrance channel infill rates is called the "Oregon Inlet Method." It is an analytic/empirical approach documented in a U.S. Army Engineer District, Wilmington (1980) report concerning the dredging

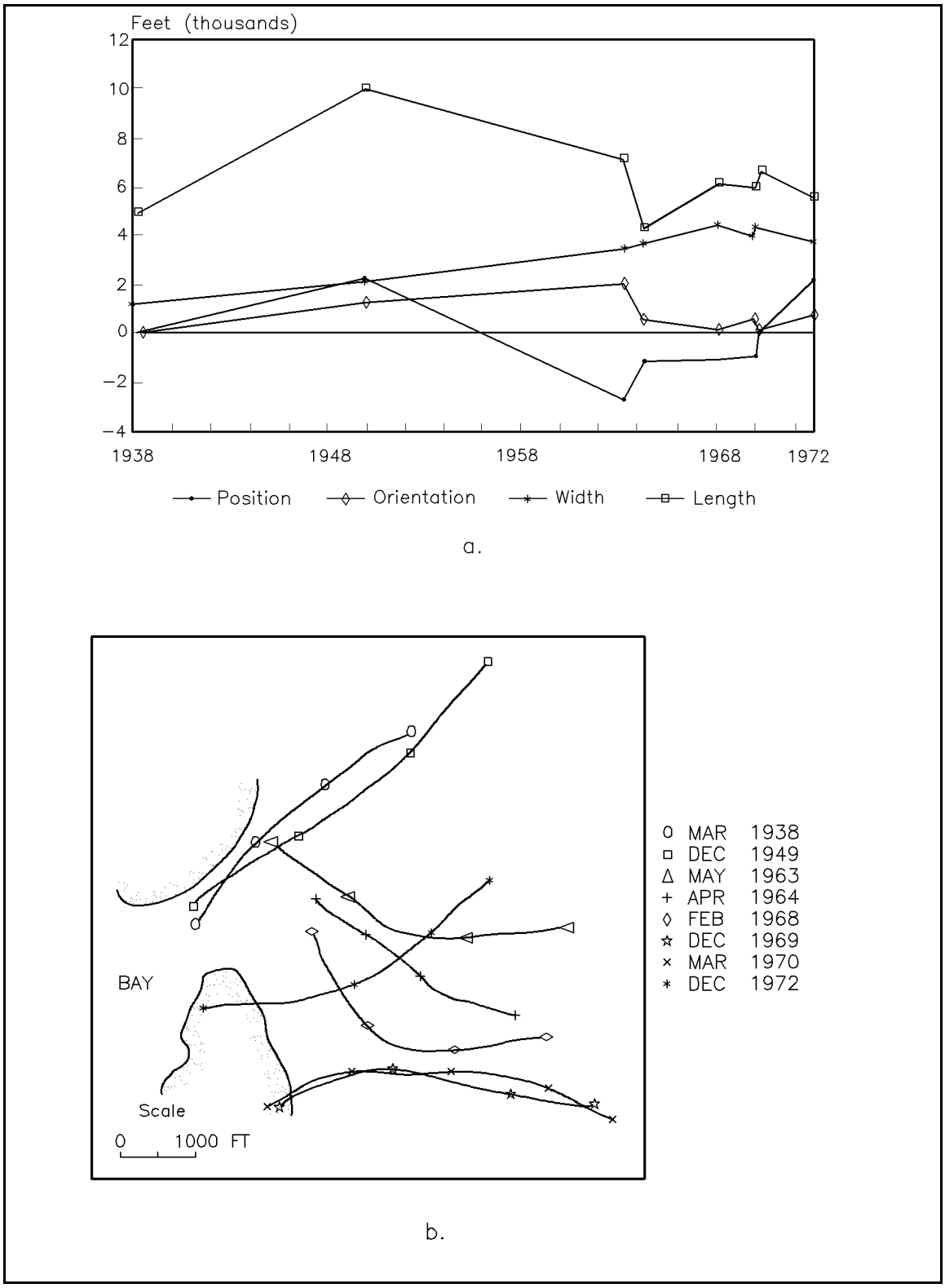


Figure II-6-45. Little River Inlet geographical variability

EXAMPLE PROBLEM II-6-5

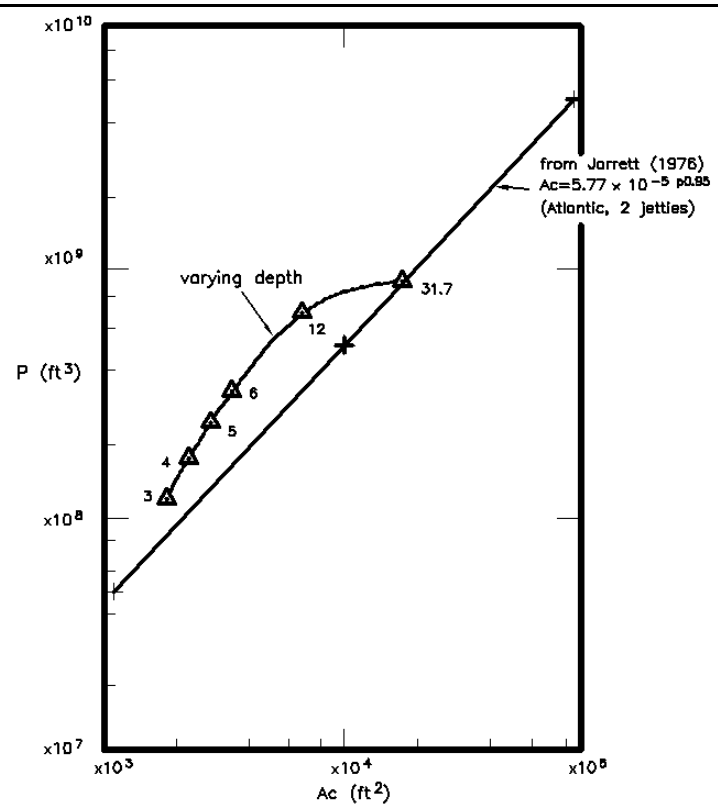
FIND:

Using the information provided in Example Problem 1, find the potential stability of the proposed channel cross section. Remember the channel has vertical sheet-pile walls, so its cross section can only change in the vertical.

SOLUTION:

By varying the cross-sectional area of the channel A_C, assuming that the channel width B remains constant and varying the channel depth d and recalculating the tidal prism as described above, the effect of channel area on the bay tidal prism can be evaluated and compared with the appropriate equation from Table II-6-3 (dual jettied inlets, $A_C = 7.489 \times 10^{-4}\ p^{0.86}$). This is done graphically in Figure II-6-46, which shows a plot of P versus A_C from the hydraulic response calculations and from the stability equation. The intersection of the two curves on the right side is the stable solution. It yields a channel cross-sectional area of 1,440 m^2 or a depth of 8 m. This shows that the 180-m by 3.7-m design channel would be unstable, with a strong tendency to erode.

Where the hydraulic response curve lies above the stability curve (as in the example) the tidal prism is too large for the inlet channel area and erosion will likely occur until a stable channel develops. If the hydraulic response curve crosses the stability curve twice, the lower point is an unstable equilibrium point from which the channel can either close or scour to the upper stability point. If the hydraulic response curve is substantially below the stability curve at all points, a stable inlet channel is unlikely to develop and the channel should eventually close.



D(ft)	3	4	5	6	8	12	16	25	---	35
K1	345	202	134	97	58	29	18	9		6
K2	.49	.43	.38	.35	.30	.25	.22	.17		.15
V'm	.12	.21	.28	.35	.50	.68	.81	.95		.98
ab/as	.12	.20	.28	.38	.59	.78	.97	1.01		1.00
Vm	4.1	5.4	5.8	6.0	6.5	5.8	5.2	3.9		2.95
P(x10^8)	1.1	1.8	2.5	3.3	5.2	6.8	8.5	8.9		8.8
Ac(ft^2)	1800	2400	3000	3600	4800	7200	9600	15000		21000

Figure II-6-46. Example Problem II-6-4. Plot of *P* versus A_C from the hydraulic response calculations and from the stability equation

The stable inlet cross-sectional area depends on other factors (e.g., wave climate, monthly tidal range variations, surface runoff) besides the spring or diurnal tidal prism. As a result, the tidal prism-inlet area equations given in Table II-6-3 only serve as an indication of the approximate stable cross-sectional area. The analysis performed in the example demonstrates that the design channel is very likely to erode to a greater depth; however, that depth, which will fluctuate with time, can vary substantially from the indicated depth of 8 m.

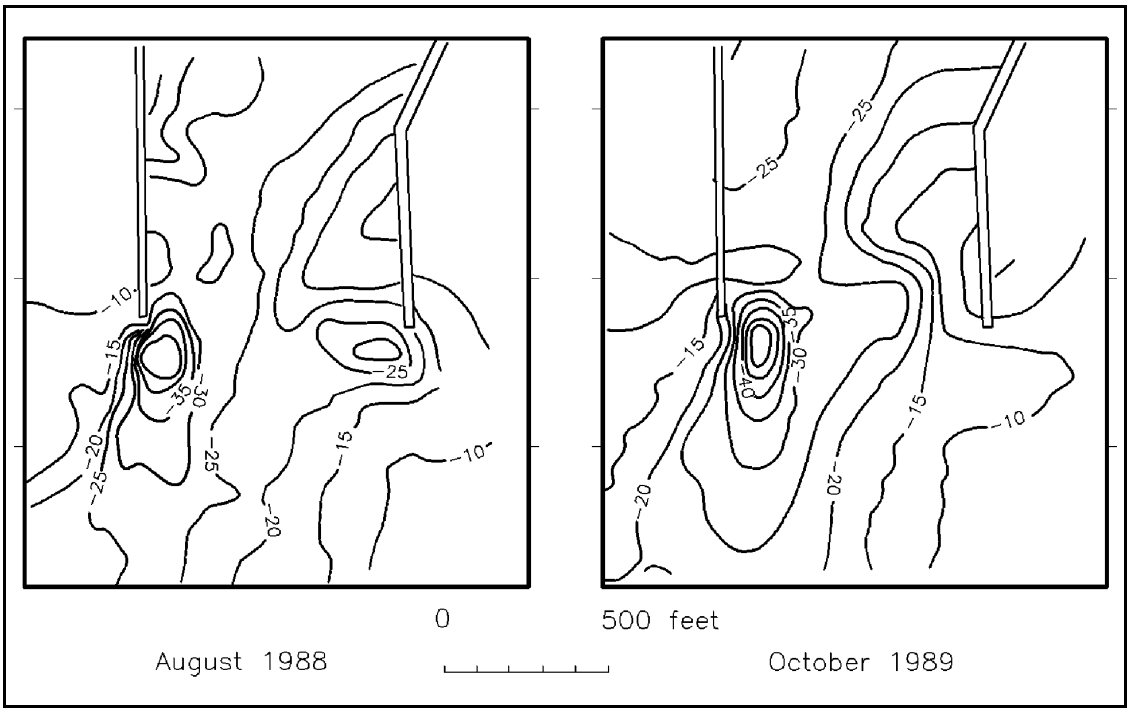


Figure II-6-47. 1988 and 1989 bathymetry of Moriches Inlet

of Oregon Inlet, North Carolina. It is based on the premise that the natural geometry of an inlet's ebb tide delta on the ocean bar results from the integrated effects of tidal currents, wave action, and associated sediment transport and deposition. Of particular importance is the natural elevation of the ocean bar, which represents both the limiting elevation of sediment accumulation resulting from the influx of littoral materials from adjacent shores and the level below which littoral sediments will collect if the ocean bar is dredged to provide a navigation channel. Sediment accumulation should increase with increasing channel depth until it becomes so deep that the channel traps all sediment crossing the inlet.

(2) Procedure. In order to develop a day-by-day simulation of siltation processes and dredging effects, a procedure consisting of three main phases was developed. The phases are (a) daily littoral materials transport volumes, (b) channel sedimentation, and (c) regression analysis.

(3) Phase I: Daily littoral materials transport volumes. It is assumed that the relative magnitude of alongshore sediment transport can be estimated simply on the basis of wave height squared in accordance with the relationship

$$Q_i = Q_g \left(\frac{H_i^2}{\sum_{i=1}^{n} H_i^2} \right) \qquad \text{(II-6-34)}$$

where

i = number of the day during the year

Q_i = littoral transport occurring on day i [L^3]

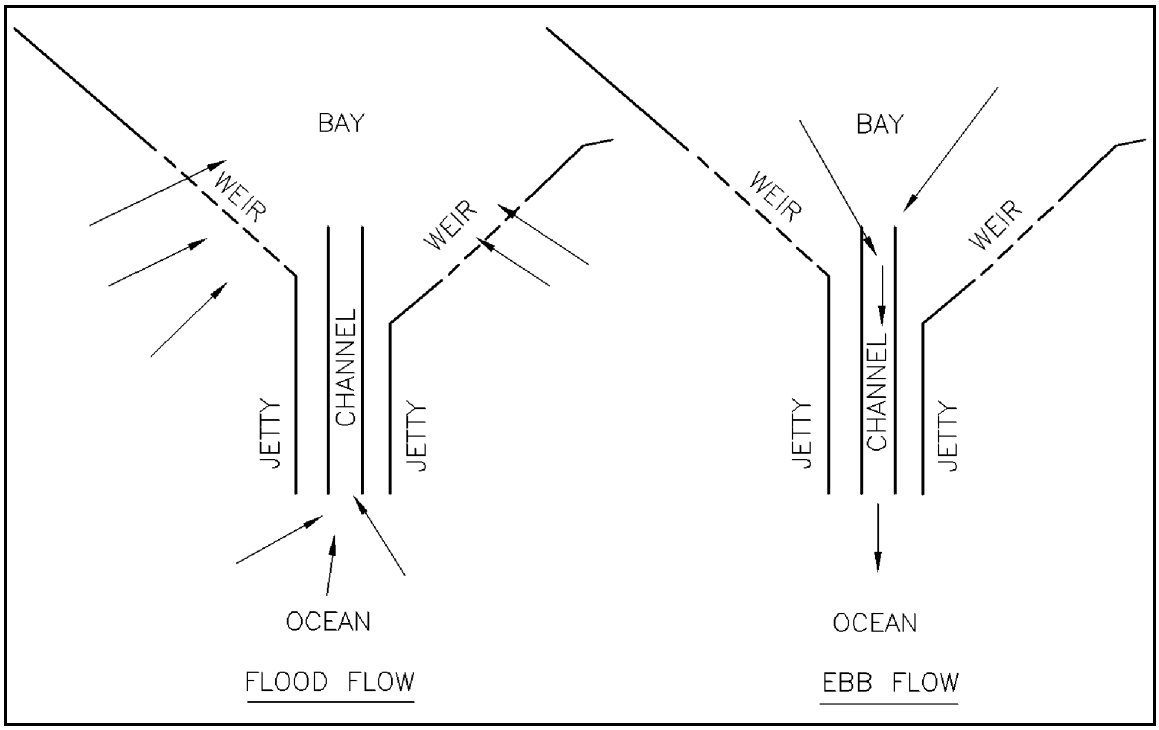


Figure II-6-48. Weir jetty flow patterns

Q_g = total volume of alongshore sediment transport to the inlet each year [L^3]

H_i = average wave height for the ith day of the year [L]

n = total number of days during the year [T]

(4) Phase II: Channel sedimentation. The first portion of the channel sedimentation procedure is

$$Transport\ ratio = \left(\frac{d_1}{d_2}\right)^{\frac{5}{2}} \tag{II-6-35}$$

where the transport ratio equals the ratio of the sediment transport potential in the dredged cut to the sediment transport on the bar before dredging and

d_1 = depth before dredging [L]

d_2 = depth after dredging [L]

On a daily basis, the amount of material that would be retained in the ocean bar channel of the inlet is given by:

$$Daily\ shoal\ volume = CQ_g \left(\frac{H_i^2}{\sum_{i=1}^{n} H_i^2}\right)\left[1 - \left(\frac{d_1}{d_2}\right)^{\frac{5}{2}}\right] \quad [L^3] \tag{II-6-36}$$

in which all terms have been previously defined except for "*C*," which is termed the "potential shoaling factor." As stated previously, the quantity of material entering and depositing in the channel increases with the depth of the cut. However, until sufficient channel depth is attained to intercept all materials moving into the inlet, some material continues to bypass the inlet via the sloping face of the ocean bar below the bottom of the channel cut. Accordingly, *C* is defined as the portion of the total alongshore transport to the inlet entering the domain of the channel. In order to determine *C*, a regression analysis was conducted to correlate the inlet bar siltation with those factors judged to be most influential in the filling and flushing of such channels.

(5) Phase III: Regression analysis. The following three factors were selected as dominant in influencing the magnitude of shoaling at a particular inlet site having a dredge-maintained ocean bar channel. These factors are described as follows:

(a) Ebb tide flow is the primary factor acting to flush intrusive littoral materials from the inlet environment. Its influence in the analysis is represented by the symbol $E_{\Delta T}$, which is the difference between the mean ebb tide flow energy flux across the ocean bar at its natural elevation and the mean ebb tide flow energy flux through the cross section of the excavated ocean bar navigation channel. It is assumed that the tidal discharge is not significantly altered from one condition to the other. A basic concept in the tidal energy flux difference is that the tidal flow velocities directed seaward over the ocean bar at its natural elevation, in combination with wave agitation, are rapid enough to prevent accumulation of sediments above the natural bar depth. If a section of the ocean bar is deepened by a navigation channel, the related average local flow velocity is diminished and sediment deposition is initiated.

(b) Wave energy reaching the littoral zones adjacent to the inlet is the primary factor controlling the quantity of littoral material moving toward the inlet and, as such, determines the shoaling characteristics of an ocean bar navigation channel. In the analysis, the unrefracted wave energy flux per unit width of wave crest offshore of the area of interest, designated E_w, is the basic measure of sediment transport toward the inlet.

(c) The depth of an ocean bar channel determines the degree to which a channel will trap the littoral sediments entering the inlet environment. In the analysis, the amount of channel entrenchment, and hence the measure of sediment entrapment potential, is taken as the ratio D_R of the depth of the channel to the depth at which the seaward slope of the ocean bar meets the sea bottom. Each of these depths is measured from the natural elevation of the ocean bar; therefore, the ratio D_R represents the extent to which the ocean bar's seaward slope has been incised by the channel.

(6) Normalized, independent filling index. The channel sedimentation potential increases or decreases as each of the three factors $E_{\Delta T}$, E_w, and D_R increases or decreases, respectively. The factors were combined as follows to establish a normalized, independent variable F_I (filling index) for the regression analysis.

$$F_I = \frac{(E_{\Delta T} \cdot E_W \cdot D_R)}{10^{14}} \quad \left[\frac{ML^2}{T^2}\right] \tag{II-6-37}$$

The normalized, dependent variable selected for the analysis was the ratio of the volume of channel infill to the computed volume of the total alongshore sediment influx to the inlet multiplied by 100. This percentage value V_R is referred to as the "volume ratio." The regression analysis was based on data from four dredged-maintained inlets within the boundaries of the Wilmington District: Oregon, Beaufort, Masonboro, and Lockwoods Folly Inlets. Information available consisted of: (a) measured tidal discharges or inlet throat cross-sectional areas, which permitted tidal discharge computations by means of tidal prism-inlet area relations, (b) site wave statistics representing one or more years of wave gauge records, (c) detailed

alongshore sediment transport analyses for the shorelines adjacent to each of the inlets, and (d) one or more sets of inlet hydrographic surveys, each set consisting of two surveys taken at different times, thereby allowing measurement of the volume of sediment filling within the navigation channels in the time periods between surveys.

(7) Ebb tidal energy flux. For each inlet selected, the parameter $E_{\Delta T}$, representing the change of ebb tidal energy flux across the ocean bar between natural and channel conditions, was computed and fixed as constant for the given inlet using the following equation:

$$E_{\Delta T} = \frac{4T}{3\pi} q_{\max}^{3} \frac{d_2^2 - d_1^2}{d_1^2 \, d_2^2} \tag{II-6-38}$$

where

T = tidal wave period [T]

q_{max} = maximum instantaneous ebb tide discharge per unit width [L^3/T/L]

d_2 = depth of navigation channel [L]

d_1 = natural depth of ocean bar [L]

The next parameter is E_w, the average daily wave energy flux per unit width of wave crest, offshore of the site of interest. This parameter represents the intensity of sediment transport from the adjacent beaches to the inlet environment. Daily wave energy flux can be expressed as

$$E_W = K \times 10^6 \, (H_0)^2 \, T \quad \left[\frac{ML^2}{T^2}\right] \tag{II-6-39}$$

where

K = constant (K = 1.769 slugs/sec in English engineering units or 0.121 Kg/sec) [M/T]

H_o = deepwater wave height equivalent to observed shallow-water wave, if unaffected by refraction and friction [L]

T = wave period [T]

The last parameter is a depth ratio D_R, which is given as

$$D_R = \frac{d_2 - d_1}{d_3 - d_1} \tag{II-6-40}$$

where

d_3 = water depth where seaward tip of ocean bar meets offshore sea bottom [L]

(8) Hoerls special function distribution. Tests of several regression equations and related curves revealed that the best fit of the data from the four inlets was attained by a "Hoerls" special function distribution, given in general form by:

$$V_R = a\ F_I^b\ e^{cF}\ I \qquad \text{(II-6-41)}$$

where a, b, and c are the best-fit coefficients. The regression equation was solved for filling index F_I values corresponding to various channel depths, which allows for the proper "C" value to be used in the "daily shoal volume" equation developed in Phase II of this method.

f. Inlet weir jetty hydraulic and sediment interaction. The subject of weir jetty design is dealt with in detail by Seabergh (1983) and Weggel (1981), who discuss several hydraulic and sedimentary aspects including flow over the weir and its elevation, location, length, jetty alignment and orientation, as well as the shoreward end of the weir. The function of the weir jetty is to ideally collect the net sediment movement behind the weir section, where it can then be handled in protected waters and bypassed. A sedimentation benefit may occur in that sediment-laden longshore currents are kept away from the channel. Also, the ebb currents may be hydraulically enhanced, as seen in Figure II-6-48, creating ebb dominance of channel currents, which can aid in flushing sediment seaward.

(1) Weir location. With respect to tidal currents within the jetty system, the farther the weir is from the navigation channel, the less likely it is to capture channel ebb currents that are directed seaward. This can depend on the location of the predominant ebb channels and their orientation. Care must be taken in evaluation of ebb flow direction because once the jetty system is constructed, the channel orientation may change due to removal of wave effects and consequent sediment movement, which, for example, may have deflected the ebb channel downcoast.

(2) Weir length. Primary transport over the weir exists at its intersection with the shoreline. If the wave climate is mild, the weir should only be as long as necessary to prevent a chance of closure, something that has not yet been noted to occur in existing weir jetty systems. The length to prevent closure would need to be evaluated based on wave conditions, beach slope, orientation of the structure, etc. (Weggel 1981). If the wave climate is highly variable, the weir should probably extend further oceanward so as to include a large percentage of the breaker zone since heavy transport over the weir will occur at the breaker location for larger wave conditions; otherwise, the sediment will move offshore along the jetty. Another factor influencing weir length will be consideration of the amount of flow that is desired in the system. If a design objective is to obtain high ebb dominance of flow in the navigation channel, then the weir should be longer, if this would not interfere with other constraints, such as placement of a portion of the weir too close to the channel. The complete hydraulic flow situation must be considered to determine whether the additional flow provided by a wider weir will substantially augment ebb flow predominance and provide additional scouring ability in the entrance channel (see Seabergh (1983)).

II-6-4. References

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II-6-5. Definitions of Symbols

β	Stability index of an inlet (Equation II-6-33) [length^5/time^3]
Δ	Bay superelevation (Equations II-6-20 and II-6-21) [length]
ε	Fraction of new water entering the bay from the sea each tidal cycle [dimensionless]
ε	Phase lag between sea tide and bay tide [deg]
θ	Angle wave orthogonal makes with current entering an inlet [deg]
μ	Friction factor
ξ	Dimensionless distance offshore parameter
τ_r	Residence time (Equation II-6-28)
a_0	Ocean tide amplitude (one-half the ocean tide range) [length]
A_{avg}	Average area over a channel length [length^2]
a_b	Bay tide amplitude (one-half the bay tide range) [length]
A_b	Surface area of a bay or basin [length^2]
a_c	Critical cross-sectional area [length^2]
A_c	Channel cross-sectional flow area of an inlet [length^2]
a_E	Equilibrium cross-sectional area of inlet throat [length^2]
A_{mw}	Cross-sectional area of an inlet at the minimum inlet width [length^2]
B	Channel width between jetties [length]
C	Potential shoaling factor (Equation II-6-36)
d	Mean water depth in channel [length]
D	Depth of water at current meter location [length]
d_1, d_2, d_3	Natural depth of ocean bar, depth of navigation channel, and water depth where seaward tip of ocean bar meets offshore sea bottom [length]
d_b	Average bay depth [length]
D_R	Ratio of depth of the channel to the depth at which the seaward slope of the ocean bar meets the sea bottom (Equation II-6-40)
d_T	Channel depth [length]
$E_{\Delta T}$	Difference between the mean ebb tide flow energy flux across the ocean bar at its natural elevation and the mean ebb tide flow energy flux through the cross section of the excavated ocean bar navigation channel (Equation II-6-38)

E_W	Average daily wave energy flux per unit width of wave crest, offshore of the site of interest (Equation II-6-39)
f	Darcy - Weisbach friction term
f	Dimensionless parameter that is a function of the hydraulic radius R and Manning's roughness coefficient n (Equation II-6-16)
F	Inlet impedance (Equation II-6-17) [dimensionless]
F_I	Filling index (Equation II-6-37)
g	Gravitational acceleration [length/time^2]
h	Inlet channel water surface elevation [length]
H_A	Wave height entering inlet [length]
H_i	Average wave height for the ith day of the year [length]
K	Keulegan repletion coefficient [dimensionless]
K	Keulegan's dimensionless coefficient of repletion or filling (Equation II-6-4)
K_1 , K_2	King's inlet friction coefficient (Equation II-6-6) and King's inlet frequency coefficient (Equation II-6-7) [dimensionless]
k_{en} , k_{ex}	Inlet entrance and exit energy loss coefficient [dimensionless]
L	Inlet length [length]
L_b	Distance from a inlet to the farthest pint in the bay [length]
L_{mw}	Oceanside inlet channel length [length]
m	Bank slope [length-rise/length-run]
M	Total annual littoral drift [length^3/year]
n	Manning's roughness coeeficient
n	Total number of days during the year
$-_O$	The subscript 0 denotes deepwater conditions
P	Tidal prism filling the bay (Equation II-6-12) [length^3]
Q	Inlet discharge [length^3/time]
Q_g	Total volume of alongshore sediment transport to the inlet each year [length^3]
Q_i	Littoral transport occurring on day i (Equation II-6-34) [length^3]
q_{max}	Maximum instantaneous ebb tide discharge per unit width [length^3/time-length]
Q_r	River or freshwater inflow to a bay [length^3/time]
Q_r'	King's dimensionless variable (Equation II-6-19)

R	Inlet hydraulic radius [length]
R_H	Inlet current wave height factor (Figure II-6-34) [dimensionless]
T	Tidal period [time]
t_i	Duration of inflow [time]
$U_{max\,f}, U_{max\,e}$	Maximum channel flood and ebb velocity [length/time]
$U'_{max\,f}, U'_{max\,e}$	Dimensionless maximum channel flood and ebb velocity
V	Inlet channel velocity [length/time]
$\langle V \rangle$	Average volume of the bay over the tidal cycle [length^3/time]
V_{avg}	Maximum velocity averaged over entire inlet cross section [length/time]
V_m	Maximum cross-sectionally averaged velocity during a tidal cycle [length/time]
V_{max}	Maximum velocity in the inlet throat [length/time]
V_{meas}	Point measurement of maximum velocity [length/time]
V_R	Hoerls special function distribution (Equation II-6-41)
V_T	Threshold velocity for sand transport [length/time]
V'_m	King's dimensionless velocity (Equation II-6-5)
W	Channel width corresponding to mean water depth in channel [length]

II-6-6. Acknowledgments

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