Wind and Wind Wave Sediment Transport in Large Lakes and Reservoirs

by

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ABSTRACT

A rather simple model has been proposed to examine the effect of wind and wind waves on shoreline erosion. The technique employed for estimating waves is that developed by the Corps of Engineers. The solutions for the combined wind and wave setup is based on radiation stress concepts, as is the generation of the longshore current. The sediment transport model is based on energetics. And finally, the shoreline evaluation model is based on conservation of sediment. The methodology and numerical results were produced for the first four tasks. Only the methodology was outlined for the fifth task.

Without the completion of the fifth task, it is difficult to quantify the effects of wind wave erosion on the shoreline. However, it is clear that the sediment transport is increased in a narrow band along the water line. Whether this sediment transport is of sufficient magnitude or duration to cause significant erosion remains unanswered.

FOREWORD

The Water Resources Research Institute, located on the Oregon State University campus, serves the State of Oregon. The Institute fosters, encourages, and facilitates water resources research and education involving all aspects of the quality and quantity of water available for beneficial use. The institute administers and coordinates statewide and regional programs of multidisciplinary research in water and related land resources. The Institute provides a necessary communications and coordination link between the agencies of local, state, and federal government, as well as the private sector, and the broad research community at universities in the state on matters of water-related research. The Institute also coordinates the interdisciplinary program of graduate education in water resources at Oregon State University.

It is Institute policy to make available the results of significant water-related research conducted in Oregon's universities and colleges. The Institute neither endorses nor rejects the findings of the authors of such research. It does recommend careful consideration of the accumulated facts by those concerned with the solution of water-related problems.

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INTRODUCTION

There are many large bodies of fresh water in the Pacific Northwest. Significant wind waves may be generated on these bodies of water which increase sediment transport along the shoreline. This increased transport may result in erosion and turbidity. Higher levels of turbidity reduce the environmental and recreational quality of the lake. Increased erosion may result in a terraced shoreline which has reduced recreational and aesthetic value and is more difficult to manage as a flood control reservoir.

A model is proposed to address this problem and involves several specific tasks. The first task, estimating wave conditions, employs existing methods. Next, an expression for the water depth along the shoreline is developed including the combined effects of wind and waves. The wind setup is due to a surface stress at the free surface while the wave setup is due to the onshore-onshore radiation stress. Longshore currents are also calculated for the combined effects of wind and waves. The forcing mechanisms are similar to those in the setup, but the flow is now resisted by a bottom drag rather than a pressure gradient. A sediment transport model is presented which is based on energetics. This model has been widely used in the marine environment and the empirical transport coefficient has been determined. The final step, determination of the shoreline response due to the longshore currents and waves, is discussed but no analytical results are presented.

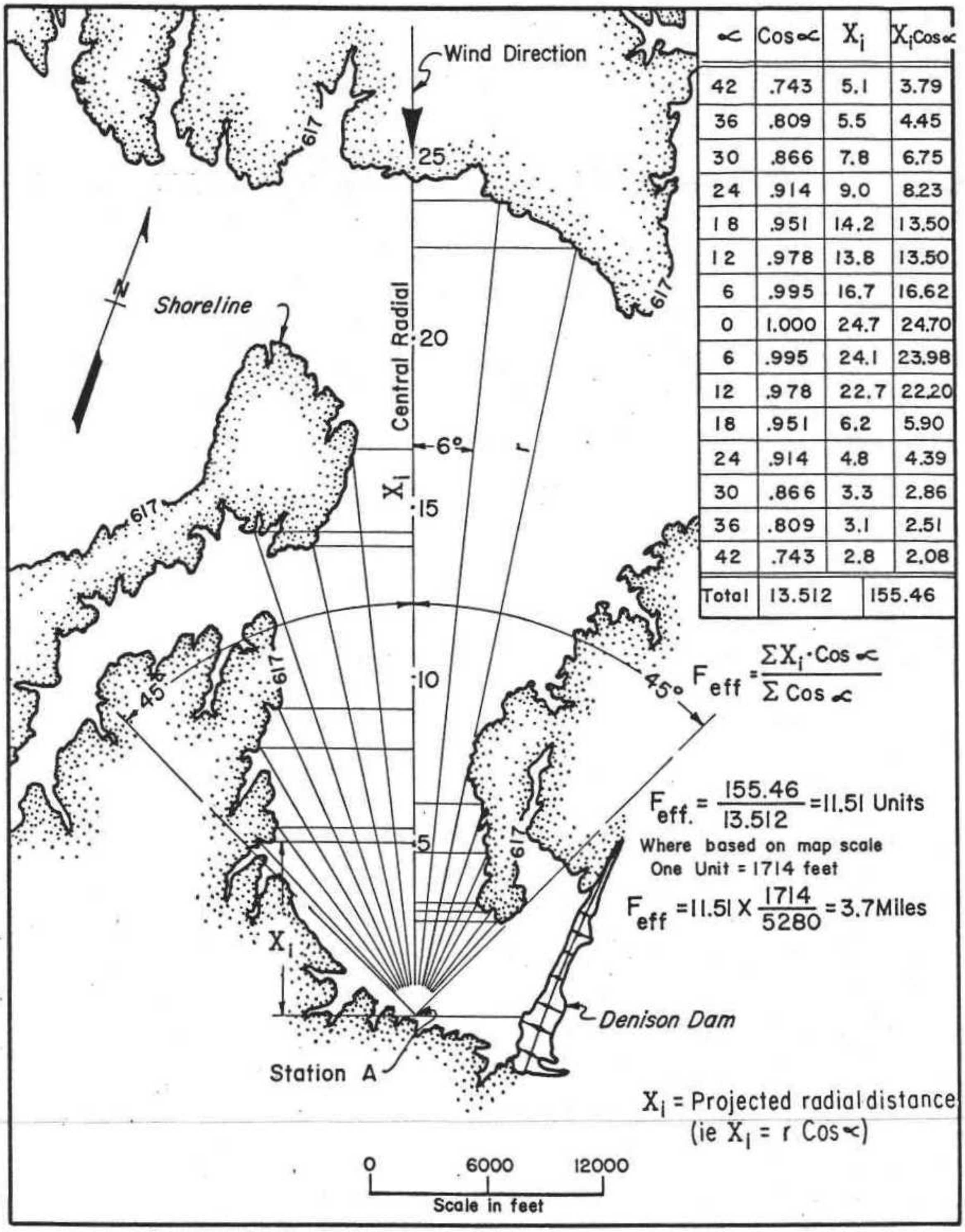
METHODOLOGY

The determination of the wind wave effects on sediment transport involves several specific tasks. These include: 1) estimating the waves from the wind speed and duration and the lake depth and length; 2) determining the influence of the wind and waves on water depth at the shoreline (setup); 3) calculating the wind and wave-induced currents; 4) estimating the sediment transport; and 5) calculating the change in shoreline configuration. Each of these tasks is discussed in greater detail in this section.

1. Wind Waves

The wind waves which are generated in the lake or reservoir are a function of the wind speed and duration, as well as the fetch length and depth of the lake or reservoir. The fetch length may be determined using the effective fetch method (U.S. Army, 1962). The application of this method is shown in Fig. 1.

The generation of wind waves for a given wind speed is either limited by the duration of the storm event or by the size of the body of water. Wave generation is, therefore, referred to as being fetch, or duration, limited. For wind speeds of interest (greater than 30 mph), a fetch length of 30 statute miles will become fully arisen in a duration of less than 4 hours. A body of water with a fetch length of 10 statute miles will require a duration of less than 2 hours. Therefore, in smaller bodies of water, it is reasonable to assume that the generation of wind waves is fetch limited. Bretschneider and Reid (1953) proposed a model considering the effects of bottom friction and percolation in



α	Cos α	X_i	X_i Cos α
42	.743	5.1	3.79
36	.809	5.5	4.45
30	.866	7.8	6.75
24	.914	9.0	8.23
18	.951	14.2	13.50
12	.978	13.8	13.50
6	.995	16.7	16.62
0	1.000	24.7	24.70
6	.995	24.1	23.98
12	.978	22.7	22.20
18	.951	6.2	5.90
24	.914	4.8	4.39
30	.866	3.3	2.86
36	.809	3.1	2.51
42	.743	2.8	2.08
Total	13.512		155.46

$$F_{eff} = \frac{\Sigma X_i \cdot \text{Cos}\,\alpha}{\Sigma\,\text{Cos}\,\alpha}$$

$$F_{eff.} = \frac{155.46}{13.512} = 11.51 \text{ Units}$$

Where based on map scale
One Unit = 1714 feet

$$F_{eff} = 11.51 \times \frac{1714}{5280} = 3.7 \text{ Miles}$$

X_i = Projected radial distance
(ie $X_i = r\ \text{Cos}\,\alpha$)

Fig. 1. Effective fetch method.

the bottom sediments. This technique was modified by Bretschneider (1965) and Ijima and Tang (1966) yielding the relationships given in Figs. 2 and 3 for wave height and period, respectively. In Figs. 2 and 3, g is the acceleration due to gravity (ft/s^2); H is the wave height (ft); U is the wind speed (ft/s); F is the fetch length (ft); and d is the water depth (ft).

The forecasting technique assumes that the bottom is flat and that the lake is of constant depth. These assumptions may be acceptable because the length scale associated with the wind waves is small with respect to the length scale of the lake. Also, if the water depth is more than twice the wave length, the waves are deep water waves and are insensitive to the depth.

2. Water Depth

In the central parts of the lake or reservoir, where the waves are generated, the waves tend to be somewhat insensitive to the bottom profile. However, as the waves approach the shoreline and ultimately break, they are very sensitive to the bottom profile and water depth. Because of this sensitivity, special care must be taken when estimating the water depth near the shoreline.

As the wind blows over the surface of the water, a shear stress is developed. This stress is balanced by a pressure gradient associated with a setup of water at the shoreline. This wind setup, sometimes referred to as a storm tide, may significantly increase the water depth at the shoreline, relative to the wave height.

There is a second mechanism, which is associated with the wind waves, that also produces a setup of shoreline water levels. This wave

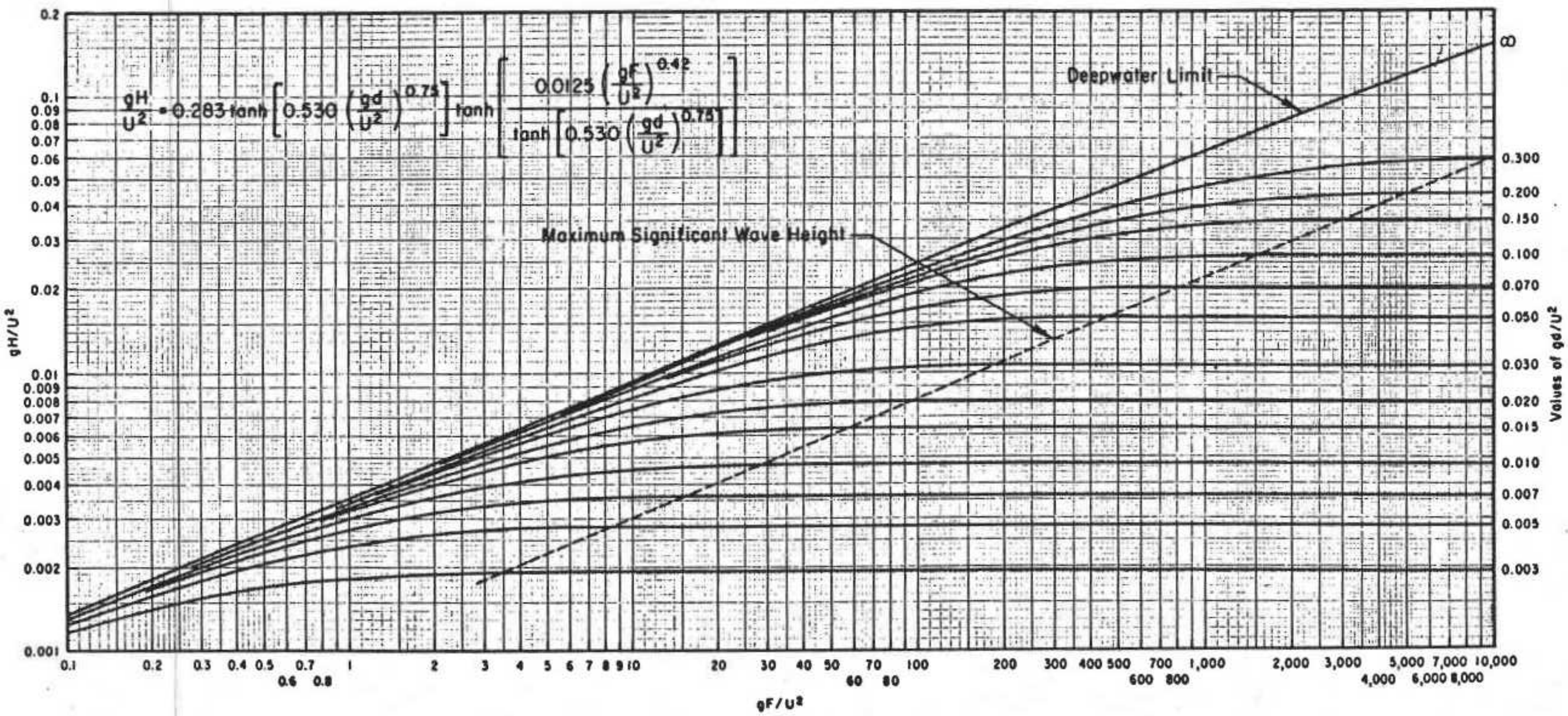


Fig. 2. Forecasting curves for wave height.

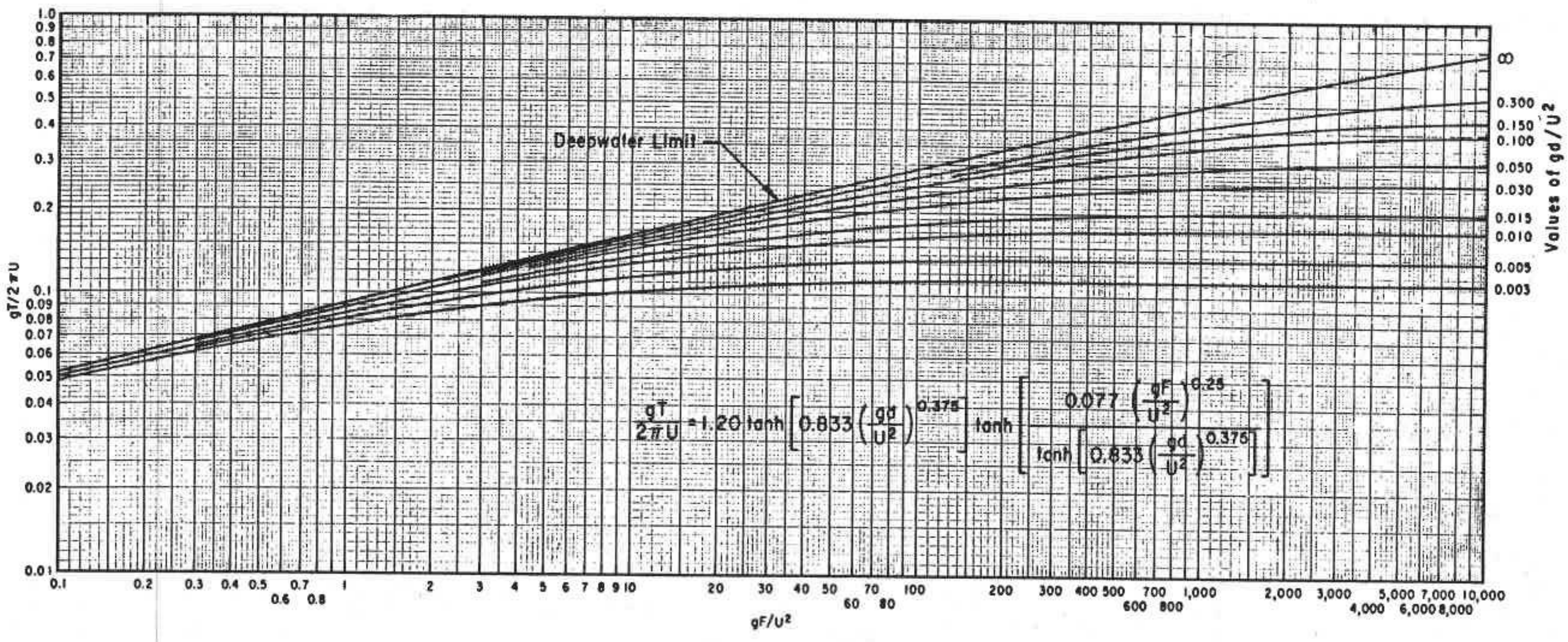


Fig. 3. Forecasting curves for wave period.

setup produces a pressure gradient which balances the change in momentum flux associated with the waves as they break approaching the shoreline. This breaking zone, or surf zone, is the region in which the effects of the waves are most important in generating turbulence, currents, and sediment transport.

Definition sketches of the nearshore region are shown in Fig. 4, in which $\langle\eta\rangle$ is the mean displacement of the free surface due to wind and waves. The brackets denote time averaging over the wave period. This is done to remove the fluctuating component of motion at the wave frequency. The MWL (mean water level) is the time-averaged free surface while the SWL (still water level) would be the location of the free surface if no wind or waves were present. The SWL is assumed to be known for the design conditions and it is necessary to determine the MWL. The still water depth is termed h and the mean water depth (total depth, actual depth) is denoted as d and is given by the sum of the still water depth plus the setup.

A general form for the depth- and time-averaged equation of motion in the surf zone is (cf. Liu and Mei, 1974)

$$\rho_w d \frac{D}{Dt} \langle u\rangle = \langle p\rangle \nabla h - \rho_w g d \, \nabla \, \langle\eta\rangle + \nabla\cdot(-\underset{\sim}{s} + \underset{\sim}{t}_r + \underset{\sim}{t}_m) + \tau_s \, |\nabla s| + \tau_b \, |\nabla b| \qquad (1)$$

in which D/Dt is the material derivative, ∇ is the two-dimensional horizontal gradient operator, $\nabla\cdot$ is the divergence operator, $\langle\cdot\rangle$ is a temporal averaging operation over one wave period, $\langle u\rangle$ is the depth- and time-averaged horizontal velocity vector, $\langle p\rangle$ is the time mean pressure, $\langle\eta\rangle$ is the time mean displacement of the free surface due to the

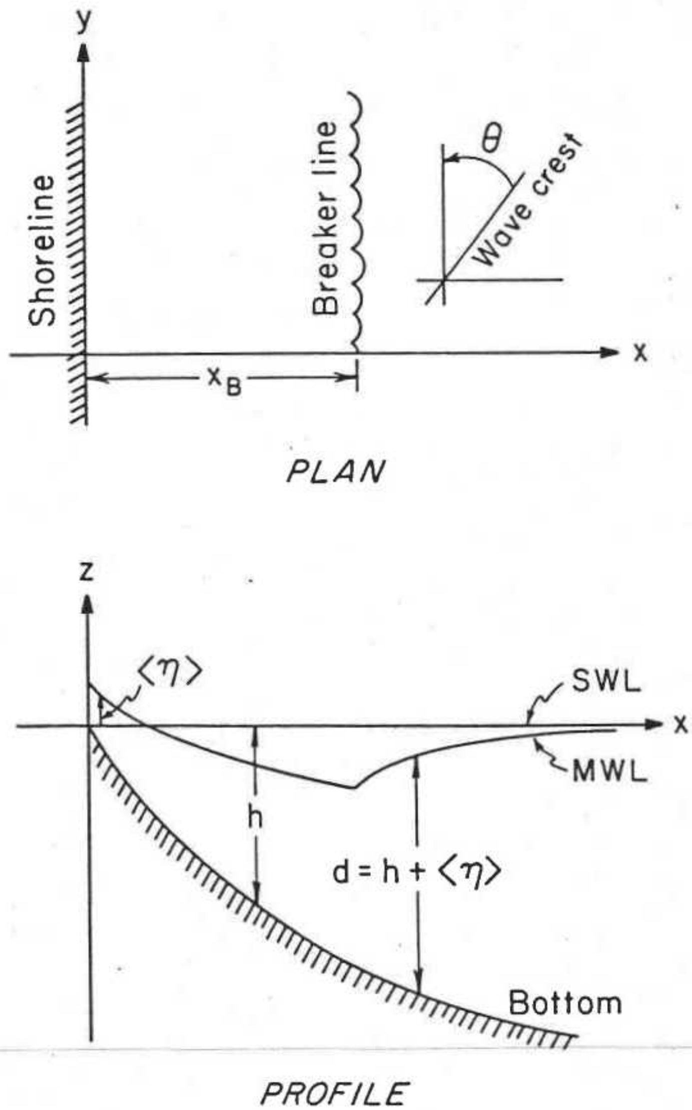


Fig. 4. Definition sketches for nearshore domain.

presence of the wind and waves, ρ_w is the water density, g is the acceleration due to gravity, d is the total depth, $\underset{\sim}{s}$ is the radiation stress tensor, $\underset{\sim}{t}_r$ is the integrated Reynolds stress tensor, $\underset{\sim}{t}_m$ is the integrated viscous stress tensor, τ_s and τ_b are the surface and bottom stresses, respectively, and s and b are the material coordinates for the surface and bottom, respectively. Seeking steady-state solutions ($\partial/\partial t = 0$) and assuming no longshore gradients ($\partial/\partial y = 0$), that the flow is sufficiently turbulent to neglect the viscous transport of momentum ($\underset{\sim}{t}_m = 0$), that the time mean pressure, bottom gradients, and mean free surface gradients vary slowly ($\langle p \rangle \nabla h = 0$, $|\nabla b| = 1$, $|\nabla s| = 1$), and that the turbulent stresses are related to the time and depth mean flows by an eddy viscosity model ($\underset{\sim}{t}_r = \mu_e d[\langle u \rangle \nabla + \nabla \langle u \rangle]$), Eq. (1) reduces to

$$- \rho_w g d \frac{d}{dx} \langle \eta \rangle - \frac{d}{dx} S_{xx} + \tau_{sx} = 0 \tag{2a}$$

$$- \frac{d}{dx} S_{xy} + \tau_{by} + \tau_{sy} + \frac{d}{dx} \left(\mu_e d \frac{d}{dx} v \right) = 0 \tag{2b}$$

in which S_{xx} is the onshore-onshore components of the radiation stress tensor, S_{xy} is the onshore-longshore component, τ_{sx} is the onshore component of surface stress, τ_{sy} is the longshore component of surface stress, τ_{by} is the longshore component of bottom stress, μ_e is an eddy viscosity, and v is the mean longshore current.

The radiation stress terms, the excess flux of momentum due to the presence of the waves, were given by Longuet-Higgins and Stewart (1960) for a coordinate system relative to the wave. Transforming to coordinates relative to the beach (see Fig. 4), these terms are given by

$$S_{xx} = E\ [(2n-1)\ \cos^2\theta + (n-1/2)\sin^2\theta] \tag{3a}$$

$$S_{xy} = -E\ [n\sin\theta\cos\theta] \tag{3b}$$

in which E is the wave energy density ($E = 1/8\ \rho g H^2$), a is the local wave height, n is the ratio of the group velocity to the local wave celerity ($n = 1/2[1+2kh/\sinh 2kh]$) provided that k, the wave number, is determined by $2\pi/T = (2k \tanh kh)^{1/2}$, T is the wave period, and θ is the wave angle as defined in Fig. 4.

Inside the breaker line ($x < x_B$), the water is assumed to be shallow and, due to refraction, the wave angle is small. It is further assumed that the breakers are of the spilling type such that the breaking wave height is related to the local water depth by $H = \kappa d$. Inside the breaker line, the radiation stress terms may then be given as

$$S_{xx} = 3/16\ \rho_w g\ \kappa^2 d^2 \tag{4a}$$

$$S_{xy} = 1/16\ \rho_w g\ \kappa^2 d^2\ \sin\theta \tag{4b}$$

To determine the total depth profile, Eq. (2a) must be integrated, employing an appropriate value for the surface stress. The only surface stress that will be considered is that due to the wind. This stress is taken to be

$$\tau_{sx} = \beta\rho_a\ c_a\ w^2\ \cos\gamma \tag{5}$$

in which ρ_a is the density of air, c_a is a wind stress coefficient, w is the wind speed, and γ is the wind direction relative to the shoreline. The wind stress coefficient was given by Van Dorn (1953) as

$$c_a = \begin{cases} 1.1 \times 10^{-6} & ; w < 16 \text{ mph} \quad (6a) \\ 1.1\text{x}10^{-6} + (2.5\text{x}10^{-6})\left(1 - \frac{16}{w}\right) & ; w > 16 \text{ mph} \quad (6b) \end{cases}$$

The β term accounts for the unsteadiness in the development of the wind current. In the simplification of the equation of motion it was assumed that only steady-state solutions would be sought. This is a reasonable assumption for the wave-generated current because it forms rapidly. However, the wind current develops more slowly. The β term accounts for this development and estimates for β may be determined from the one-dimensional equation for a wind-generated current without waves,

$$\rho_w \frac{dv}{dt} = \frac{1}{d} (\tau_s - \tau_b) \tag{7}$$

in which v is the mean fluid velocity. The wind stress is given by Eq. (4) without the β term and a simple, linear bottom stress is given by

$$\tau_b = \alpha \rho_w \, c_w v \tag{8}$$

in which α is a dimensional linearizing coefficient and c_w is the fluid stress coefficient. As a first approximation, α may be taken as 3% of the wind speed. This is based on the observation that the wind drift is about this fraction of the wind speed (Wu, 1975). Substitution of Eqs. (5) and (8) into (7) and integrating yields

$$v = \frac{\rho_a}{\rho_w} \frac{c_a}{c_w} \frac{w^2}{d} \cos\gamma \left[1 - e^{-\frac{\alpha c_w t}{d}} \right] \tag{9}$$

in which it is assumed that the water is initially at rest. The maximum current, v_∞, occurs at very large times. The ratio of the time-dependent velocity to v_∞ is

$$\frac{v}{v_\infty} = \left[1-e^{-\frac{\alpha c_w t}{d}} \right] \quad (10)$$

This ratio defines the β given in Eq. (5). Figure 5 shows this term for dimensionless time defined as $\alpha c_w t/d$.

With all of the terms in the cross-shore equation of motion specified, Eq. (2a) may be written

$$- \rho_w g \ (\langle\eta\rangle + h) \frac{d}{dx} \langle\eta\rangle - \frac{d}{dx} (3/16 \ \rho_w g \kappa^2 d^2) \quad (11)$$

$$+ \beta \ \rho_a c_a w^2 \cos\gamma = 0 \qquad ; x < x_B$$

A solution to this equation for a planar beach is given by

$$d = ax + b \left[\ln \left(\frac{d}{b} + 1 \right) - 1 \right] + c \quad (12a)$$

in which

$$a = \frac{3}{8+3\kappa^2} \ m \quad (12b)$$

$$b = \frac{\beta \rho_a c_a w^2 \cos\gamma}{a \rho_w g} \ m \quad (12c)$$

and c is an integration constant.

Seaward of the breaker line, the shallow water assumptions may not be invoked. However, seaward of the breaker line, energy is conserved

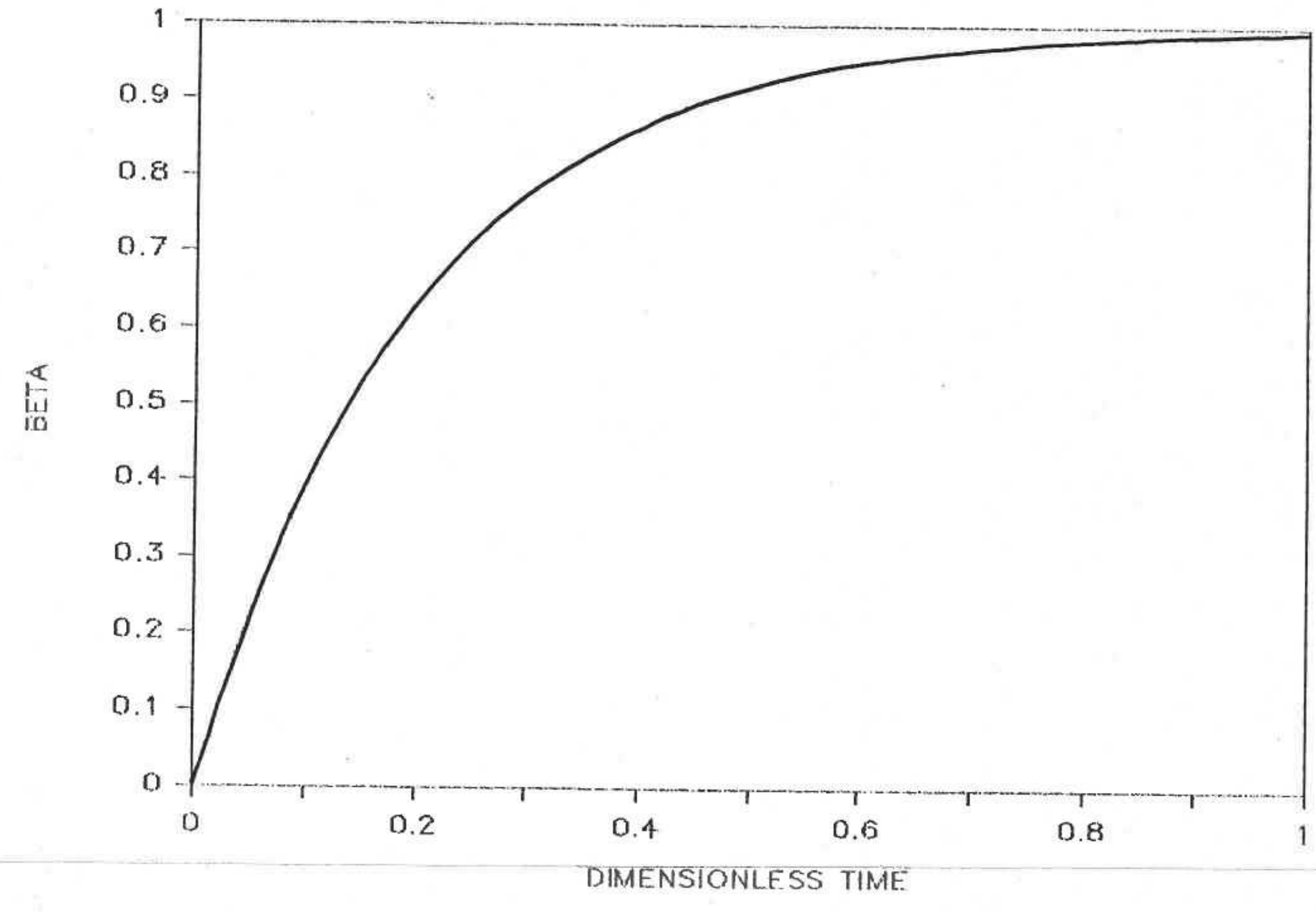


Fig. 5. Wind stress time coefficient.

(friction and percolation are small) so that an energy conservation equation may be written. The time-averaged Bernoulli equation is

$$\frac{\bar{p}}{\rho_w g} + \frac{\overline{u^2 + w^2}}{2g} + \langle\eta\rangle - \frac{\beta\rho_a c_a^2 w \cos\gamma}{\rho_w g} = \frac{\bar{p}_o}{\rho_w g} + \frac{\overline{u_o^2 + w_o^2}}{2g} + \langle\eta_o\rangle \quad ; x > x_B \tag{13}$$

in which $(\ )_o$ denotes a value in deep water. Noting that the wave-induced mean free surface displacement goes to zero in deep water, assuming that the wind setup vanishes at great distances offshore, and employing linear wave theory yields

$$\langle\eta\rangle = -\frac{H^2 k}{8\sinh 2kh} + \frac{\beta\rho_a c_a w^2 \cos\gamma}{\rho_w g} \qquad ; x > x_B \tag{14}$$

The integration constant in Eq. (12) may be determined by equating (12) and (14) at the breaker line, $x = x_B$.

$$c = \frac{\kappa^2}{16}\frac{40 - 3\kappa^2}{8 + 3\kappa^2} h_B - b\left[\ln\left(\frac{(16 - \kappa^2)h_B}{16b} + 1\right) - 1\right] \tag{15}$$

Typical setup profiles are given in Fig. 6. These setup profiles are very nearly linear. Therefore, assume

$$d = b_1 x + b_2 \tag{16a}$$

in which

$$b_1 = b \tag{16b}$$

$$b_2 = b\ln\left(\frac{1/2\ d_B + b}{d_B + b}\right) + \frac{\kappa^2}{16}\frac{40 - 3\kappa^2}{8 + 3\kappa^2} h_B \tag{16c}$$

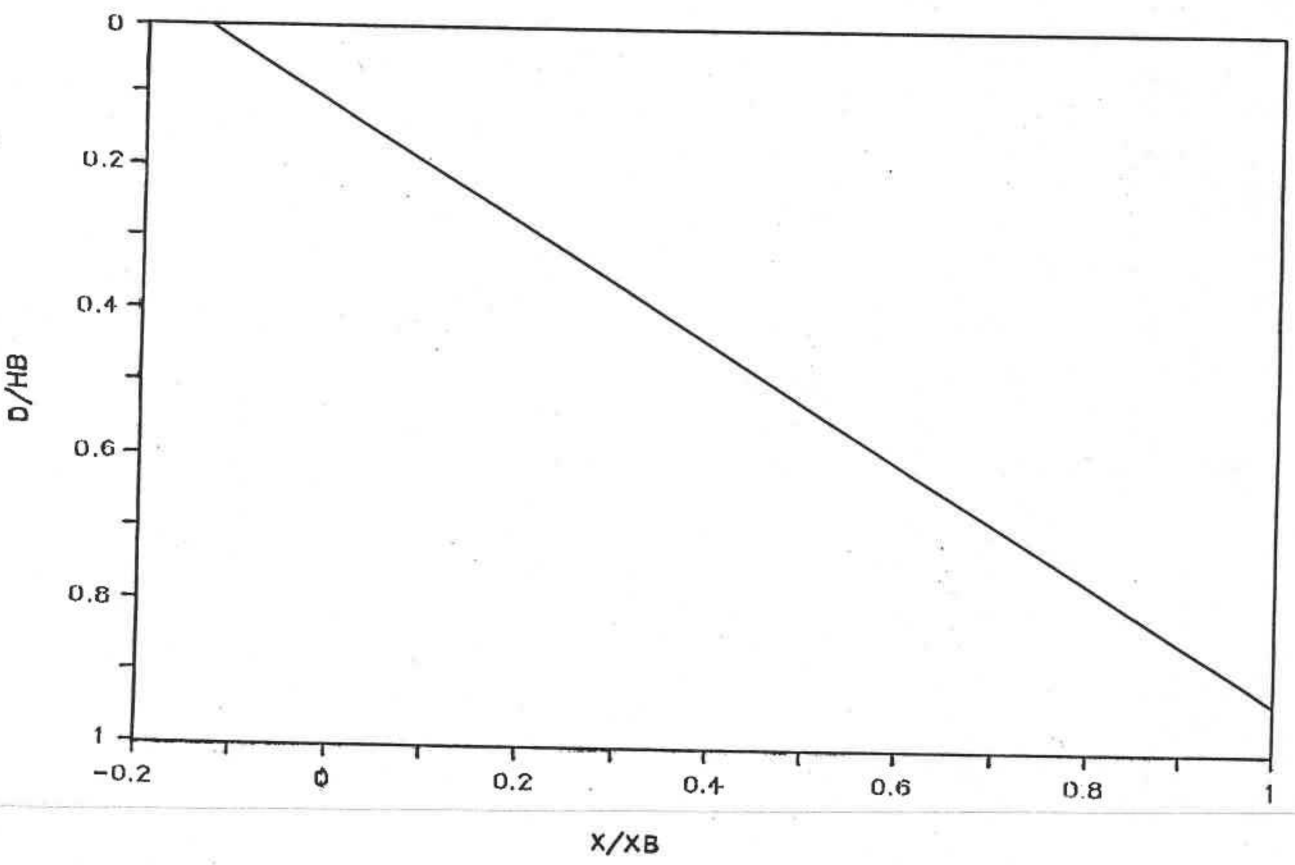


Fig. 6. Setup profile.

It is convenient to define a coordinate transformation such that the origin is located at the point on the beach where the depth goes to zero. This is given by

$$d = b_1 \hat{x} \tag{17a}$$

where

$$\hat{x} = x + b_1/b_2 \tag{17b}$$

It is also convenient to express the depth in a dimensionless form. Using upper case letters to denote dimensionless variables define

$$D = d/\hat{x}_B \tag{18a}$$

$$X = \hat{x}/\hat{x}_B \tag{18b}$$

$$B_1 = b_1 \tag{18c}$$

The dimensionless depth profile, including wind and wave setup, is given by

$$D = B_1 X \tag{19}$$

3. Longshore Current

The longshore current due to wind and waves is determined from Eq. (2b). The divergence of the radiation stress term is determined employing Snell's Law for refraction and assuming conservation of energy in the offshore.

$$\frac{d}{d\hat{x}} S_{xy} = \begin{cases} -\frac{5}{16}\rho_w g\kappa^2 \sin\theta_B \left(\frac{d}{d_B}\right)^{1/2} d \frac{d}{d\hat{x}}(d) & ; \hat{x} < \hat{x}_B \quad (20a) \\ 0 & ; \hat{x} > \hat{x}_B \quad (20b) \end{cases}$$

The wind stress term is the same as for the onshore equation of motion except that the $\sin(\gamma)$ component is taken rather than $\cos(\gamma)$ component.

The bottom stress term may be approximated using a linear shallow water, small wave angle relationship for the waves (cf. Liu and Dalrymple, 1978) and a linear relationship for the wind current (see Eq. (8)).

$$\tau_{by} = -\frac{c_w}{\pi}\kappa\, \rho_w (gd)^{1/2}\, v\, \alpha\rho_w c_w v \qquad (21)$$

The eddy viscosity is assumed to be proportional to a characteristic velocity, density, and length. The form proposed by Longuet-Higgins (1970b) is employed.

$$\mu_e = \rho_w\, N(gd)^{1/2}\, \hat{x} \qquad (22)$$

in which N is a numerical constant.

Employing the above relationship in the equation of motion yields

$$\frac{N\pi b_1}{\kappa\, c_w}\frac{d}{d\hat{x}}\left(\hat{x}^{5/2}\frac{dv}{d\hat{x}}\right) - \left(\hat{x}^{1/2} + \frac{\alpha\pi}{(gb_1)^{1/2}\kappa}\right) v =$$

$$-\frac{5}{16}\frac{\pi}{c_w}\kappa\, b_1 (g\, d_B)^{1/2} \sin\theta_B \frac{b_1}{d_B}\hat{x}^{3/2} - \frac{\beta\pi}{(gb_1)^{1/2}}\frac{\rho_a}{\rho_w}\frac{c_a}{c_w} w^2 \sin\gamma \;\; ; \hat{x} < \hat{x}_b \qquad (23a)$$

$$-\frac{\beta\pi}{(gb_1)^{1/2}}\frac{\rho_a}{\rho_w}\frac{c_a}{c_w} w^2 \sin\gamma \qquad ; \hat{x} > \hat{x}_B \qquad (23b)$$

Again, it is convenient to define dimensionless variables

$$P_1 = \frac{N\pi B_1}{\kappa \; c_w} \tag{24a}$$

$$P_2 = \frac{\pi}{\kappa} \frac{\alpha}{(gB_1 \; \hat{x}_b)^{1/2}} \tag{24b}$$

$$P_3 = \frac{B_1}{D_B} \tag{24c}$$

$$P_4 = \frac{\beta\pi}{B_1^{1/2}} \frac{\rho_a}{\rho_w} \frac{c_a}{c_w} \frac{w^2}{(g \; \hat{x}_B) v_{BL}} \sin\gamma \tag{24d}$$

$$V = v/v_{BL} = v/[\frac{5}{16} \frac{\pi}{c_w} \kappa \; b_1 (gd_B)^{1/2} \; \sin\theta_B] \tag{24e}$$

The term v_{BL} is the longshore current due to waves alone which occurs at the breaker line for the case of no turbulent lateral mixing (Longuet-Higgins, 1970a). It is common to use this for scaling longshore currents.

In dimensionless variables the equation is written as

$$P_1 \frac{d}{dX} (X^{5/2} \frac{dV}{dX}) - (X^{1/2} + P_2) V = \begin{cases} - P_3 X^{3/2} - P_4 & ; X < 1 \quad (25a) \\ - P_4 & ; X > 1 \quad (25b) \end{cases}$$

Boundary conditions are that the longshore current velocity is bounded at the shoreline and far offshore, and that the velocity and gradient of velocity match at the breaker line. Equation (25) has been integrated analytically in the course of this study. Unfortunately, the resulting solution is rather complicated and it is more convenient to just use a numerical integration. Several example profiles are shown in Fig. 7 for different wind and wave conditions.

4. Sediment Transport

Bagnold (1963) proposed a sand transport model which assumes that the sediment is mobilized by wave motion and wave power is expended to maintain the motion of the sediment. The presence of a mean current transports the sediments. The immersed-weight transport rate per unit width of surf zone is (McDougal and Hudspeth, 1983)

$$i = K' \frac{d}{dx} \left(\frac{1}{2} \rho g a^2 C_g\right) \frac{v}{u_m} \tag{26}$$

where K' is a dimensionless coefficient which depends on the degree of the transport mechanism developed, C_g is the wave group velocity. The coefficient can be treated as a constant for a particular coast from the view of long-term climate. Furthermore, it can be considered as a constant in a season or in a storm (Hou, Lee, and Lin, 1980).

Assuming linear shallow-wave conditions exist and that the wave energy flux and the bottom orbital velocity outside the breaker line can be evaluated at the breaker line, Eq. (26) becomes

$$i = \begin{cases} \frac{5}{8} K' \rho g \kappa d \frac{d}{d\hat{x}} (d)\ v & ; \hat{x} < \hat{x}_b \qquad (27a) \\ \frac{5}{8} K' \rho g \kappa \left[d \frac{d}{d\hat{x}} (d)\right]_{\hat{x}=\hat{x}_b} v & ; \hat{x} > \hat{x}_b \qquad (27b) \end{cases}$$

In terms of nondimensional variables, the immersed-weight transport rate becomes

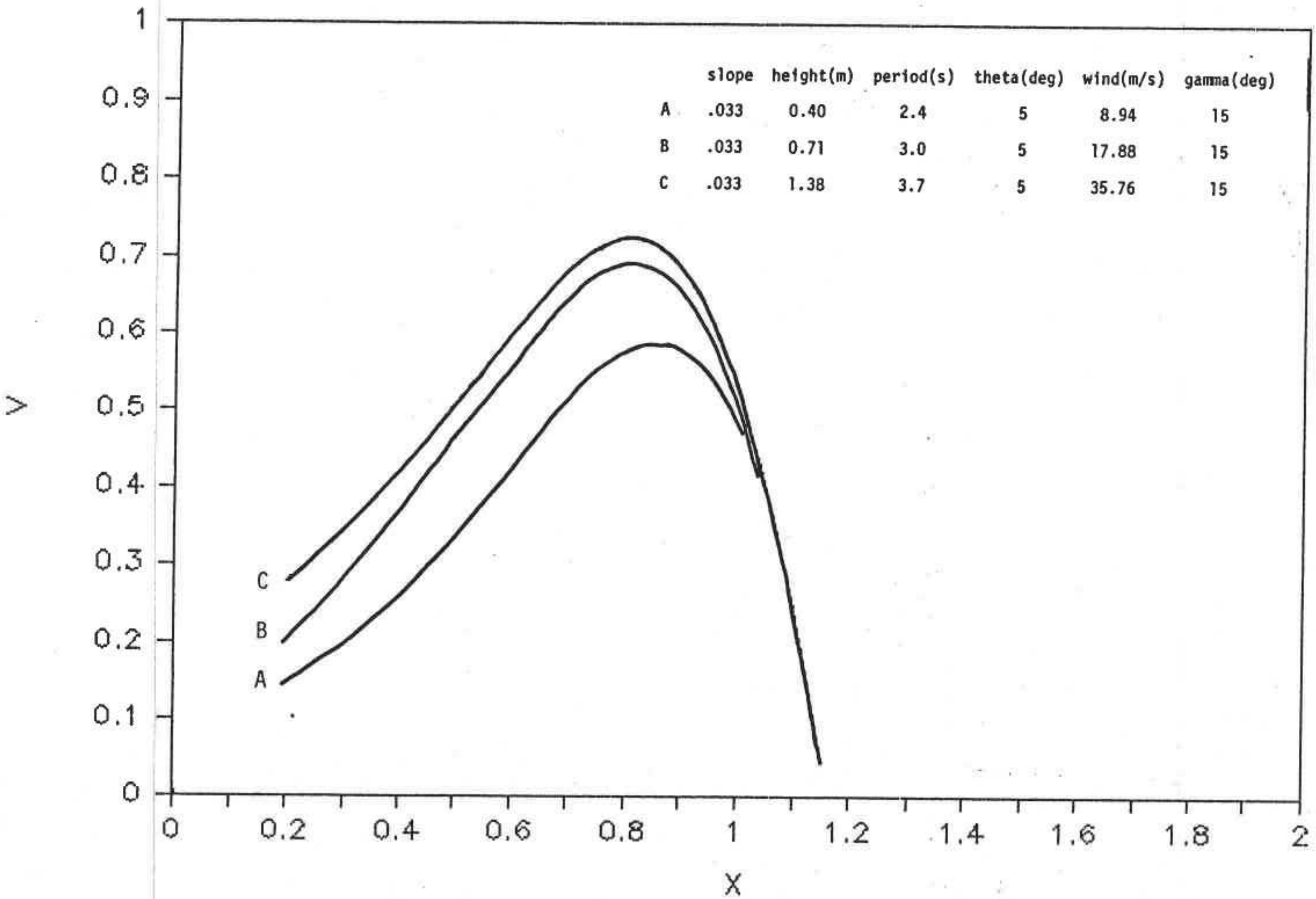


Fig. 7. Longshore current profiles.

$$I\ (X) = \begin{cases} \frac{D}{D_b} \frac{d}{dX} \left(\frac{D}{D_b}\right) V\ (x) & ;\ X < 1 \qquad (28a) \\ \\ \frac{d}{dx} \left(\frac{D}{D_b}\right)_{X=1} V\ (X) & ;\ X > 1 \qquad (28b) \end{cases}$$

where

$$I \quad = \frac{i}{i_{BL}} \tag{28c}$$

$$i_{BL} = \frac{5}{8} K' \ \rho_s g \kappa d_b S_b v_{BL} \tag{28d}$$

$$S_b \quad = \frac{d_b}{x_b} \tag{28e}$$

and ρ_s is the density of the sediment.

For the linear total depth profile given in Eq. (19), the sediment transport is

$$I\ (X) = \begin{cases} XV & ;\ X < 1 \qquad (29a) \\ \\ V & ;\ X > 1 \qquad (29b) \end{cases}$$

Dimensional transport profiles are shown in Fig. 8 for the longshore current profiles given in Fig. 7.

The total sediment transport is given by

$$I_T = \int_0^{\infty} I\ dX = \int_0^{1} XV\ dX + \int_1^{\infty} VdX \tag{30}$$

This integral is numerically evaluated.

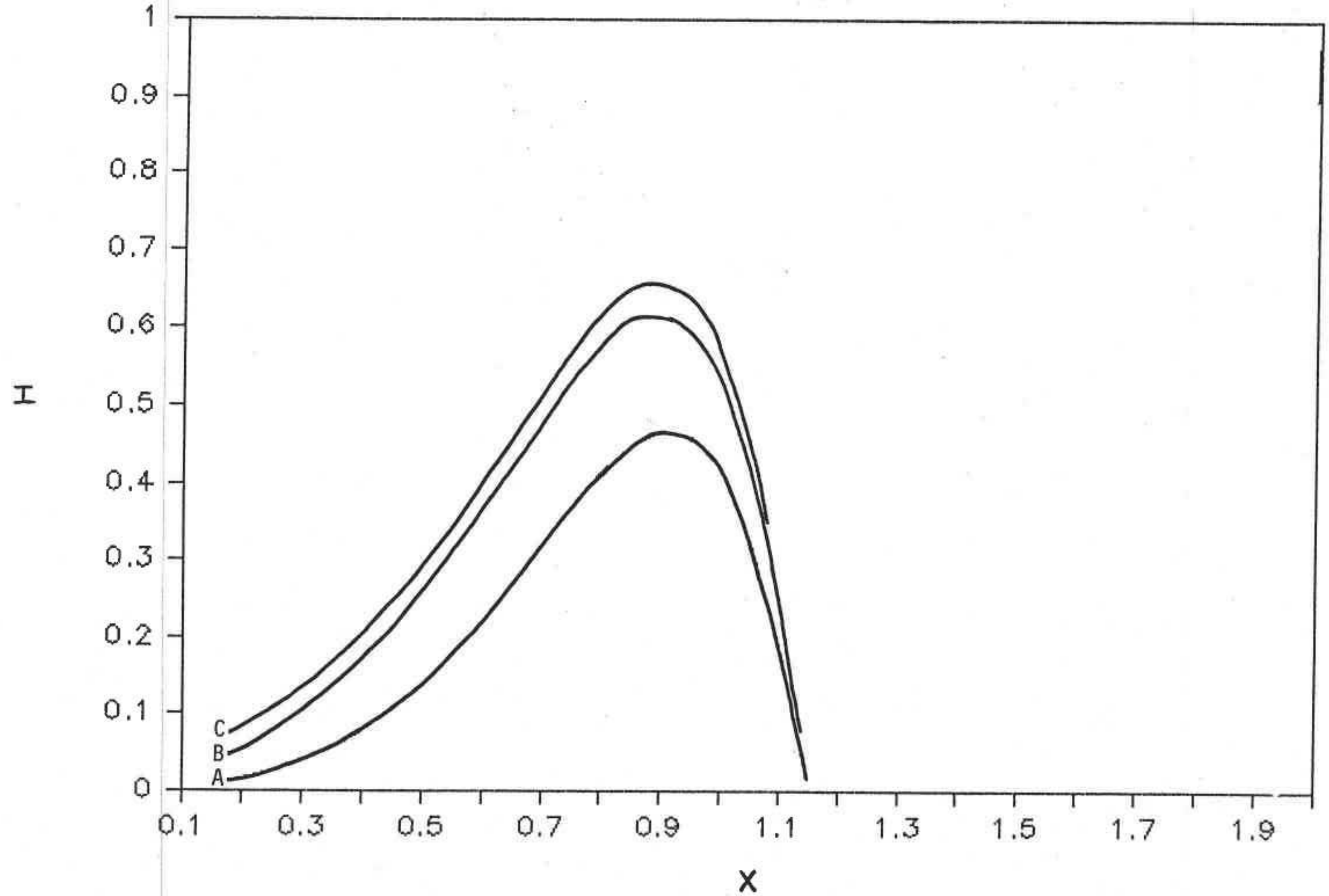


Fig. 8. Longshore sediment transport profiles.

5. Shoreline Evolution

Shoreline evolution is based on the net flux of sediment along a given beach. The shoreline is broken into a number of segments of length ΔS. The immersed weight dynamic transport given in Eq. (30) may be converted to a volume transport rate by dividing by the porosity, n, and the immersed weight density.

$$S = \frac{I}{n(\rho_s - \rho_w)g} \tag{31}$$

in which S is the volume transport and ρ_s is the density of the sediment. From conservation of the sediment mass, the change sediment in depth is given as

$$\frac{\partial \xi}{\partial t} = \frac{1}{n(\rho_s - \rho_w)g} \frac{\partial I}{\partial x} \frac{M}{\Delta s\ h_{NP}} \tag{32}$$

in which ξ is the normal to the bottom and h_{NP} is the depth to the null point. The null point is the maximum depth of wave-induced sediment transport. For lakes and reservoirs, this depth may be on the order of five feet. The change in depth also results in a change in the position of the shoreline. This change is given by

$$\frac{\partial R}{\partial t} = \left(\frac{m^2 + 1}{m^2}\right) \frac{\partial \xi}{\partial t} \tag{33}$$

in which R is the shoreline position relative to the initial condition and m is the bottom slope.

The change in shoreline may be computed using the solution obtained for total longshore transport and then solving Eq. (33) numerically. An implicit finite difference scheme may be used in space while the solution is explicitly marched in time.

It should be noted that at each change in shoreline configuration, the breaker angle must be recomputed for each segment. This may be rapidly, but rather crudely, done employing Snell's Law.

CONCLUSIONS

A rather simple model has been proposed to examine the effect of wind and wind waves on shoreline erosion. The technique employed for estimating waves is that developed by the Corps of Engineers. The solution for the combined wind and wave setup is based on radiation stress concepts, as is the generation of the longshore current. The sediment transport model is based on energetics. And, finally, the shoreline evolution model is based on conservation of sediment. The methodology and numerical results were developed for the first four tasks. Only the methodology was outlined for the fifth task.

Without the completion of the fifth task, it is difficult to quantify the effects of wind wave erosion on the shoreline. However, it is clear that the sediment transport is increased in a narrow band along the water line. Whether this sediment transport is of sufficient magnitude or duration to cause significant erosion remains unanswered.

REFERENCES

Bagnold, R.A. (1963), "Beach and Nearshore Processes, Part 1: Mechanics of Marine Sedimentation." In: M.N. Hill (Editor), The Sea, pp. 507-528.

Bretschneider, C.L. and Reid, R.O. (1953), "Change in Wave Height Due to Bottom Friction, Percolation and Refraction," 34th Annual Meeting of the AGU.

Bretschneider, C.L. (1965), "Generation of Waves by Wind; State of the Art," Report No. SN-134-6, National Engineering Science Co., Washington, D.C.

Hou, H.S, Lee, C.P., and Lin, L.H. (1980), "Relationship Between Along-shore Wave Energy and Littoral Drift in the Mid-West Coast at Taiwan," Proc. 17th Coastal Eng., Chap. 76, pp. 1255-1274.

Ijima, T. and Tang, F.L.W. (1966), "Numerical Calculation of Wind Waves in Shallow Water," Proc. 10th Conf. on Coastal Engeering, pp. 38-45.

Liu, P.L-F and Mei, C.C. (1974), "Effects of a Breakwater on Nearshore Currents Due to Breaking Waves," Ralph M. Parson Laboratory, Department of Civil Engineering, MIT, Report No. 12.

Liu, P.L-F and Dalrymple, R.A. (1978), "Bottom Frictional Stresses and Longshore Currents Due to Waves with Large Angles of Incidence," J. Mar. Res., 36:357-375.

Longuet-Higgins, M.S. and Stewart, R.W. (1960), "Changes in the Form of Short Gravity Waves on Long Waves and Tidal Currents," J. Mar. Res., 36:357-375.

Longuet-Higgins, M.S. (1970a), "Longshore Currents Generated by Obliquely Incident Sea Waves, 1," J. Geophysical Res., 75:6778-6789.

Longuet-Higgins, M.S. (1970b), "Longshore Currents Generated by Obliquely Incident Sea waves, 2," J. Geophysical Res., 75: 6790-6801.

McDougal, W.G. and Hudspeth, R.T. (1983), "Longshore Sediment Transport on Non-Planar Beaches," Coastal Eng., 7:119-131.

U.S. Army, Corps of Engineers (1962), "Waves in Inland Reservoirs, (Summary Report on Civil Works Investigation Projects CW-164 and CW-165)," TM-132, Beach Erosion Board, Washington, D.C.

Van Dorn, W.C. (1953), "Wind Stress on an Artificial Pond, " Jour. of Marine Res., Vol. 12.

Wu, Jin (1975), "Wind-Induced Drift Currents," J. Fluid Mech., 68:49-70.