CHAPTER 20

GROWTH OF LONGSHORE CURRENTS DOWNSTREAM OF A SURF-ZONE BARRIER

Peter S. Eagleson
Professor of Civil Engineering
Massachusetts Institute of Technology, Cambridge, Massachusetts

ABSTRACT

Momentum flux considerations are used to formulate a differential equation governing the growth, with distance, of the mean longshore current velocity in the surf-zone on a plane, impermeable beach due to monochromatic waves. The equation is solved for the flow situation downstream of a surf-zone barrier and is shown to compare favorably with laboratory measurements.

The asymptotic (uniform flow) form of the relation is also shown to be in good agreement with the field and laboratory data of other investigators.

Conclusions are reached governing the size of laboratory models necessary to represent conditions of fully developed longshore currents.

INTRODUCTION

A thorough understanding of the mechanics of sand transportation parallel to coasts must ultimately be based upon detailed knowledge of the complicated fluid motions which occur within the surf zone. This work is devoted to the formulation and evaluation of a rational analytical model for the prediction of mean longshore current velocity as a function of beach and wave parameters.

THEORETICAL DEVELOPMENT

MOMENTUM EQUATION

The momentum equation will be written for the stationary surf-zone control volume illustrated in Figure 1. This control volume was first used in the analysis of longshore currents by Putnam, Munk and Traylor (1949), and the following development represents a refinement and extension of their pioneering study. The control volume is bounded by (1) the still-water surface ABED, (2) the plane beach surface CBFE, (3) a vertical plane ADFC parallel to the shoreline just offshore of the breaker line, and (4) parallel vertical planes ABC and DEF perpendicular to the shoreline and a distance Δx apart.

The longshore (i.e., x) component of the instantaneous momentum equation for the fluid system occupying the control volume at time t, is

\[ F_{sx} + \iiint_{c.v.} B_x \rho d\mathbf{v} = \iiint_{c.s.} V_x (\rho \mathbf{v} \cdot \mathbf{r}) dA + \frac{\partial}{\partial t} \iiint_{c.v.} V_x (\rho d\mathbf{v}) \]  

(1)
Figure 1. Control Volume for Momentum Analysis
in which:

\[ F_{sx} \] = longshore component of surface forces

\[ F_x \] = longshore component of body forces per unit mass

\[ \rho \] = mass density of fluid

\[ d\Psi \] = volume element

\[ \dot{\vec{v}} \] = fluid velocity with respect to control volume

\[ \vec{v}_x \] = longshore component of fluid velocity

\[ dA \] = area element

\[ \vec{r} \] = unit outward normal to control surface

\[ t \] = time

\( \text{c.v. signifies integration to be performed over the control volume} \)

\( \text{c.s. signifies integration to be performed over the control surface} \)

We assume that started from rest, the flow within the control volume of Figure 1, located at an arbitrary \( x \), will reach a quasi-steady state which is defined (in part) by the time average of Equation (1). This time average must be taken over an integral number of wave periods. Equation (1) will now be expanded and time averaged term by term.

In addition, it is expedient to neglect all longshore variations in the temporal mean value of the basic parameters describing the incident wave, the beach and the mean water level. Only the fluid velocities in the surf-zone will be considered dependent upon \( x \). It is worthwhile noting that this may be an approximation, even under hypothetical laboratory conditions in which the incident wave is a purely two-dimensional monochrome and where the beach is truly planar. The difficulty arises from the probable difference between (1) the longshore projection of the incident wave length and (2) the longshore wave length of the uprush-backwash process on the foreshore. This inequality causes a longshore "beat" phenomenon which is prominent in the vicinity of an impermeable surf-zone barrier and which produces spatially periodic variations in such quantities as breaker position and mean water level. The author believes this phenomenon to play an important role in the formation of beach cusps and rip currents.

Surface forces – To evaluate the surface force, \( F_{sx} \), we will make the additional assumptions:

1. The mean shear force on the vertical face ACDF is small enough to be neglected. This assumption is aided by locating this face just offshore of the breaker line where the motion is closely irrotational.
2. Wind stresses are absent from the free surface. Since the mean water level is being taken as constant everywhere, the normal forces on vertical faces ABC and DEF are equal and opposite. The only effective surface force is thus the frictional resistance on face CBEF, the time average of which is written

\[ F_{sx} = -\frac{1}{T} \int_0^T \int_{x-\Delta x/2}^{x+\Delta x/2} \int_0^b \frac{c_f}{2} \rho u_s^2(x, y, t) \sin \theta_s \sec \alpha dydxdt \]  

(2)

in which the subscript, s, is used to denote local conditions in the surf-zone and the velocity \( u_s(x, y, t) \) represents the local average, over the depth, of \( u_s(x, y, z, t) \). We will simplify Equation (2) by the further assumptions:

1. The Reynolds numbers of the surf-zone motion are high enough to insure fully developed "rough" surface behavior for all \( x, y, t \) and for all practical surface roughnesses.

2. The friction force is defined with sufficient accuracy by neglecting the longshore variability of \( u_s \) and \( \theta_s \).

Equation (2) can then be written

\[ F_{sx} = -\frac{c_f \rho}{2T} \sec \alpha \int_0^T \int_0^b u_s^2(x, y, t) \sin \theta_s dydt \]  

(3)

**Body forces** - With gravity the only body force, \( B_x \equiv 0 \) and

\[ \frac{1}{T} \int_0^T \int_\Omega \int \int B x dV = 0 \]  

(4)

**Momentum flux** - The convective momentum term of Equation (1) involves the net rate of momentum outflow across faces DEF and ABC minus the inflow rate across ADFC. Using the definitions of Figure 1, this term can be written
LONGSHORE CURRENTS

\[ \frac{1}{T} \int_{0}^{T} \int_{c.s.} V_{x}(\rho \frac{dV}{dt}) \, dA = \rho \frac{h_{b}}{T} \int_{0}^{T} \int_{0}^{b} \frac{3}{\partial x} [u_{s}(x, y, t) \sin \theta_{s}]^{2}(1 - \frac{y}{b}) \, dy \, dt \]

(5)

\[ - \frac{\rho \Delta x}{T} \int_{0}^{T} \int_{-h_{b}}^{0} u_{sb}^2(z, t) \sin \theta_{sb} \cos \theta_{sb} \, dz \, dt \]

in which the subscript, b, signifies conditions at the breaker.

Unsteady term - Since we are assuming the process to be stationary in the time average (i.e., quasi-steady), the local momentum change vanishes. That is

\[ \frac{1}{T} \int_{0}^{T} \int_{c.v.} \frac{\partial}{\partial t} V_{x}(\rho dV) = 0 \]

(6)

Summary - Equations (3), (4), (5) and (6) are now substituted into Equation (1) to obtain the longshore momentum equation:

\[ \frac{c_{f} \sec \alpha}{2T} \int_{0}^{T} \int_{0}^{b} u_{s}^2(x, y, t) \sin \theta_{s} \, dy \, dt = \]

\[ \frac{h_{b}}{T} \int_{0}^{T} \int_{0}^{b} \frac{3}{\partial x} [u_{s}(x, y, t) \sin \theta_{s}]^{2}(1 - \frac{y}{b}) \, dy \, dt \]

(7)

VELOCITY DISTRIBUTION ASSUMPTIONS

Integration of Equation (7) depends critically upon the assumed form of the functions \( u_{s}, u_{sb}, \theta_{s} \) and \( \theta_{sb} \). As a first approximation, let us assume that
1. $u_s(x, y, t) \sin \theta_s$ is uniformly distributed (i.e., constant) in the $y$ direction for any $x$ and $t$.

2. $u_s(x, t) \sin \theta_s$ varies linearly with time such that over each wave period

$$u_s(x, t) \sin \theta_s = u_{sx}(1 - \frac{t}{T})$$  \hspace{1cm} (8)

3. The mean local longshore current velocity is independent of $y$ and is given by

$$V_L(x) = \frac{1}{A_s T} \int_0^T \int_{A_s} u_s(x, t) \sin \theta_s \, dA_s \, dt$$  \hspace{1cm} (9)

Using Equation (8) in (9)

$$V_L(x) = \frac{u_{sx}(x)}{2}$$  \hspace{1cm} (10)

Then we can carry out the first momentum flux integral of Equation (7) to obtain

$$\frac{h_b}{T} \int_0^T \int_0^b \frac{3}{\theta_s} [u_s(x, y, t) \sin \theta_s]^2(1 - \frac{y}{b}) \, dy \, dt = \frac{2bh_b}{3} \frac{dV_L^2(x)}{dx}$$  \hspace{1cm} (11)

To integrate the friction term we will make the approximation

$$\frac{1}{T} \int_0^T \int_0^b \frac{[u_s(x, y, t) \sin \theta_s]^2}{\sin \theta_s} \, dy \, dt$$  \hspace{1cm} (12)
whereupon

\[
- \frac{c_f \sec \alpha}{2T} \int_0^T \int_0^b u_s^2(x, y, t) \sin \theta_s \, dy \, dt = - \frac{2bc_f \sec \alpha}{3 \sin \theta_b} V_L^2(x) \quad (13)
\]

The second momentum flux term will be integrated by assuming the instantaneous velocity distribution along face ADFC as shown in Figure 2. In this figure the local Eulerian particle velocity, \( u_b(z, t) \), due to the breaking wave is superimposed on a mean motion which consists of two parts: (1) a longshore component, \( V_L(x) \), necessary in order for the \( x \) component of the mean velocity to be continuous across the breaker line and (2) an onshore component, \( \Delta V(x) \), necessary for satisfaction of conservation of mass in the case of a non-uniform longshore current.

Applying the continuity equation to the control volume, it is easily seen that

\[
\Delta V(x) = \frac{dV_L(x)}{dx} \frac{\Delta x}{h_b \Delta x} = \frac{b}{2} \frac{dV_L(x)}{dx} \quad (14)
\]

whereupon

\[
u_{sb}(z, t) \sin \theta_{sb} = V_L(x) + u_b(z, t) \sin \theta_b \quad (15)
\]

and

\[
u_{sb}(z, t) \cos \theta_{sb} = \frac{b}{2} \frac{dV_L(x)}{dx} + u_b(z, t) \cos \theta_b \quad (16)
\]

Using Equations (15) and (16)
Figure 2. Assumed Velocity Distribution at Breaker
\[
- \frac{1}{T} \int_{-h_b}^{0} u_{sb}^2(z, t) \sin \theta_{sb} \cos \theta_{sb} \, dz \, dt =
\]

\[
- \frac{1}{2T} \int_{-h_b}^{0} u_{b}^2(z, t) \sin 2\theta_{b} \, dz \, dt - \frac{bh_b}{4} \frac{dV_L^2(x)}{dx}
\]

Summary - Substituting Equations (11), (13) and (17) into (7) yields the simplified longshore momentum equation:

\[
- \frac{2}{3} \left[ \frac{bc \sec \alpha}{\sin \theta_{b}} \right] V_L^2(x) =
\]

\[
\frac{5}{12} \left[ bh_b \right] \frac{dV_L^2(x)}{dx} = - \frac{1}{2T} \int_{-h_b}^{0} u_{b}^2(z, t) \sin 2\theta_{b} \, dz \, dt
\]

in which the integral on the right-hand side is recognized as the "wave thrust" (Lundgren, 1963). Using small amplitude wave theory, we have

\[
u_{b}(z, t) = \frac{\pi h_b \cosh k_b (h_b + z)}{T \sinh k_b h_b} \cos (ct)
\]

from which

\[
- \frac{1}{2T} \int_{-h_b}^{0} u_{b}^2(z, t) \sin 2\theta_{b} \, dz \, dt = - \frac{g h_b^2 n_b}{16} \sin 2\theta_{b}
\]

where

\[
n_b = \frac{1}{2} \left[ 1 + \frac{2k_b h_b}{\sinh 2k_b h_b} \right]
\]
Using (20) in (18) the differential equation has the solution:

\[ \frac{V_L^2(x)}{A} = 1 - \left[ 1 - \frac{V_L^2(0)}{A} \right] e^{-Bx} \]  

(22)

in which

\[ A = \frac{3}{32} \left[ \frac{gH_b^2n_b}{bc_f} \right] \cos \alpha \sin \theta_b \sin 2\theta_b \]  

(23)

\[ B = \frac{8}{5} \left[ \frac{c_f}{h_b \cos \alpha \sin \theta_b} \right] \]  

(24)

and \( V_L(0) \) is the value of \( V_L(x) \) where \( x = 0 \).

As \( x \) becomes very large, Equation (22) reduces to the relation for uniform longshore currents:

\[ V_L^2 = A \]  

(25)

Note that the width, \( b \), of the surf zone is difficult to define due to the unknown inclination of the mean water level. For practical purposes therefore we will assume a horizontal mean water level, whereupon

\[ b = h_b \cot \alpha \]  

(26)

Also for practical reasons, the unknown resistance coefficient, \( c_f \), will be expressed in terms of the Darcy–Weisbach, \( f \). Values of the latter coefficient are well known, at least for steady uniform flow, as a function of relative boundary roughness and Reynolds number. The relation between these two coefficients is, for steady uniform flow,

\[ c_f = \frac{f}{4} \]  

(27)
Using the final simplifications (26) and (27), the parameters $A$ and $B$ become

$$A = \frac{3}{8} \left[ \frac{gH_b}{h_b} \right] \frac{\sin \alpha \sin \theta_b \sin 2\theta_b}{f}$$

$$B = \frac{2}{5} \left[ \frac{f}{h_b \cos \alpha \sin \theta_b} \right]$$

Equation (28) differs slightly from that derived earlier (Eagleson, 1964) in the magnitude of the constant coefficient. The difference arises from a change in the assumed surf-zone velocity distribution [Equation (8)].

EXPERIMENTAL PROGRAM

Experiments were conducted in the Hydrodynamics Laboratory of the Department of Civil Engineering at M.I.T. The model basin used was 45 feet by 22 feet in its horizontal dimensions, and still-water depth in the constant depth portion was approximately 1 foot. The beach was constructed out of smooth cement on a 1 to 10 slope (i.e., $\tan \alpha = 0.1$) and was 30 feet long in the $x$ direction.

Monochromatic waves were generated by a plunger-type wave-maker of variable period and stroke. Training walls were installed perpendicular to the wave crests from the wave maker up onto the beach. These walls were curved to follow the calculated refraction of the wave "rays" in order to maintain a uniform energy density at breaking. The "upstream" wall completely obstructed the surf- and swash-zones while the "downstream" wall stopped just offshore of the breaker line.

The experimental technique used is reported in detail by Galvin (1965) and will only be summarized here.

Parallel-wire, resistance-type wave profile gages were used to obtain the breaking wave height, $H_b$, the mean water level at the breaking position (and hence the breaking depth, $h_b$) and the breaking wave length, $L_b$ (used in calculating $n_b$). Breaker position was located visually using an overhead sighting device developed by Galvin (1965). Major changes in the breaker angle were obtained through changing the position of the wave-maker. Minor changes of course accompanied each change in wave period.
The mean local longshore current velocity $V_L(x, y)$ was determined by using an Armstrong Miniflowmeter AWE 183/1, Issue A, which counts electronically the revolutions of a jewel-mounted propeller in a 5/8-inch-diameter housing. The probe was calibrated for each angle of the wave-maker by timing the drift of surface floats in the region of uniform currents. The probe was inserted at mid-depth at the mean $y$ position of the surface floats in the surf-zone and the propeller axis was aligned with the $x$-axis. In this way the relation between propeller counts and float velocity was obtained. This relation was assumed valid in the region of non-uniform currents as well since accurate Lagrangian velocity measurements are difficult in the presence of convective acceleration.

Absolute boundary roughness was not measured but was selected from published values (Chow, 1959) for smooth cement as

$$k_* = 1 \times 10^{-3} \text{ ft.}$$

Two sets of experiments were conducted. Test Series 1 (Galvin, 1965) consisted of 26 individual runs divided into three groups according to the offshore angle of wave incidence. This series was aimed at study of the zone of fully developed (uniform) currents and only inadvertently included non-uniform values. Test Series 2 (Eagleson, 1965) consisted of 7 individual runs at a single offshore wave angle and was specifically designed to investigate Equation (22) over as wide a range as possible with the existing equipment. Tabulations of the respective test data are given in the references cited.

PRESENTATION OF RESULTS

EVALUATION OF THE FRICTION FACTOR

It has been assumed in the derivation of Equation (22) that the instantaneous surf-zone flow is always hydraulically rough at all locations. The Darcy-Weisbach coefficient, $f$, will thus be evaluated from the Karman-Prandtl resistance equation for steady, uniform, turbulent flow in rough pipes. When written in terms of the hydraulic radius, $R$, this well-known equation becomes

$$\frac{1}{f^{1/2}} = 2 \log_{10} \frac{2R}{k_*} + 1.74$$  \hspace{1cm} (31)

For the triangular surf-zone cross section

$$R = \frac{1}{2} b \sin \alpha = \frac{1}{2} b \tan \alpha = \frac{h_b}{2}$$

\hspace{1cm} (32)
whereupon \( f \) is given by

\[
f = [2 \log_{10} \frac{h_b}{k_b} + 1.74]^{-2}
\]

in which \( k_b \) is the absolute roughness of the beach surface. Equation (33) is used to evaluate \( f \) for all the data presented in this paper. The value of \( k_b \) was chosen from published values according to the known character of the beach surface.

**IMPORTANCE OF UPSTREAM BOUNDARY CONDITION**

In the derivation of Equation (22) it was assumed that only the fluid velocities in the surf-zone varied with \( x \). This determined the origin of \( x \) to be where the surf-zone cross section first reaches its full value and was taken as the intersection of the barrier and the still-water shoreline. The mean longshore current velocity, \( V_L(0) \), at \( x = 0 \) is therefore quite sensitive to the nature of the barrier, i.e., to such factors as its height, length, angle and permeability. No attempt is made here to predict \( V_L(0) \) as a function of these variables; however, the predicted effect of \( V_L(0) \) upon the growth of the longshore current can be evaluated. This is accomplished in Figure 4 where the experiments (Eagleson, 1965) having the largest and smallest average value of the measured parameter, \( V_L(0)/A^{1/2} \), are compared with Equation (22). The agreement appears quite satisfactory for these two runs.

**CONSOLIDATION OF AVAILABLE EXPERIMENTAL DATA**

**Developing longshore current** - In Figure 5 all experiments known to the author are presented in which the distance \( x \) was reported as one of the measured variables. Although these two sets of tests were both performed in nearly the same manner and using the same experimental facilities, the data of Galvin (1965) appear in somewhat worse quantitative agreement with Equation (22). The reason for this is unknown. Two possible causes are worthy of mention, however.

1. For the series III tests of Galvin (1965) the training walls were straight rather than being curved to follow the wave refraction. This develops a local wave energy density gradient in the longshore direction.

2. Three years elapsed between the performance of the two sets of experiments. In the interim the beach surface roughness may have changed.
Figure 5. Plan view of Laboratory Basin
DEVELOPING LONGSHORE CURRENT

LEGEND FOR EXPERIMENTAL POINTS

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>( \frac{V_L(0)}{A^{1/2}} )</th>
<th>SOURCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>+</td>
<td>0.375</td>
<td>EAGLESON (1965)</td>
</tr>
<tr>
<td>Δ</td>
<td>0.092</td>
<td>EAGLESON (1965)</td>
</tr>
</tbody>
</table>

NOTE: EXPERIMENTAL POINTS ARE PLOTTED USING AVERAGE VALUES OF A AND B AND WITH AN ASSUMED ABSOLUTE ROUGHNESS, \( k_e = 1 \times 10^{-3} \) FT.

LEGEND FOR THEORETICAL CURVES

\[
\frac{V_L(x)}{A^{1/2}} = 0.092 \\
\frac{V_L(0)}{A^{1/2}} = 0.375 \\
V_L(x) = \frac{V_L(0)}{A^{1/2}} e^{-BX} \\
A = \frac{3}{8} \left[ \frac{g H_b n_b}{h_b} \right] \sin \alpha \sin \theta_b \sin 2\theta_b f \\
B = \frac{2}{5} \left[ \frac{f}{h_b \cos \alpha \sin \theta_b} \right]
\]

Figure 4. Evaluation of the Importance of \( V_L(0) \)
Discounting the series III tests of Galvin (1965) it may be concluded from Figure 5 that Equation (22) provides an adequate description of the growing current, at least at laboratory scale.

Uniform longshore current - Several experimenters have reported longshore current observations in which no mention of proximity to an upstream obstruction is made. In Figure 6 these field and laboratory measurements are compared with the theoretical uniform longshore current as given by Equation (25). Before evaluating this comparison we should note the following.

1. The breaking depth, \( h_b \), was not measured in any of the field observations. In order to apply Equation (25), the theoretical breaking height to depth ratio of McCowan (1894) was assumed applicable; i.e.,

\[
\frac{H}{h_b} = 0.78
\]  

(34)

2. The absolute roughness of the various surfaces was chosen according to the following table:

<table>
<thead>
<tr>
<th>Reference</th>
<th>Laboratory or Field</th>
<th>Description of Surface</th>
<th>Assumed Absolute Roughness, ( k_x ) (ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Putnam et al. (1949)</td>
<td>Laboratory</td>
<td>Natural Sand</td>
<td>0.0033 (1 mm.)</td>
</tr>
<tr>
<td>Putnam et al. (1949)</td>
<td>Laboratory</td>
<td>1/4-inch Pea Gravel</td>
<td>0.0208 (1/4-inch)</td>
</tr>
<tr>
<td>Putnam et al. (1949)</td>
<td>Laboratory</td>
<td>Sheet Metal or Smooth Cement</td>
<td>0.0010</td>
</tr>
<tr>
<td>Putnam et al. (1949)</td>
<td>Field</td>
<td>Natural Sand</td>
<td>0.0033 (1 mm.)</td>
</tr>
<tr>
<td>Inman and Quinn (1951)</td>
<td>Field</td>
<td>Natural Sand</td>
<td>0.0033 (1 mm.)</td>
</tr>
<tr>
<td>Galvin and Savage (1965)</td>
<td>Field</td>
<td>Natural Sand</td>
<td>0.0033 (1 mm.)</td>
</tr>
</tbody>
</table>

\(^1\)This represents a modification of work presented elsewhere (Eagleson, 1964).
DEVELOPING LONGSHORE CURRENT

Experimental points are plotted using average values of A and B and with an assumed absolute roughness, $k_a = 1 \times 10^{-3}$ ft. All data are from laboratory experiments (Uniform Longshore Current—Theory).

**Legend for experimental points**

- **Symbol**
- **Run**
- **Reference**
- **$V_L(x)/A^{1/2}$**
- **$V_L(0)/A^{1/2}$**

**Legend for theoretical curves**

- $V_L(x) = \frac{1}{A} \left( \frac{V_L(0)}{A} \right) e^{-Bx}$
- $A = \frac{3}{8} \left[ \frac{gh_b^2}{h_b} \right] \sin \alpha \sin \beta \sin \theta \sin \theta_b$ $f$
- $B = \frac{2}{5} \left[ \frac{f}{h_b \cos \alpha \sin \theta_b} \right]$

**Figure 5.** The Developing Longshore Current
3. There is great scatter in successive observations of $V_L$ reported by Inman and Quinn (1951) under the same apparent wave conditions. Only those mean values of $V_L$ which exceed the standard deviation of the given set are included here.

It is interesting to note in Figure 6 that the theory provides an upper bound for the laboratory values while the field data scatter about the theoretical curve. Perhaps this difference is due to some viscous or surface tension scale effect arising from inexact formulation of the problem, or perhaps the experimental basin was not large enough for uniform conditions to be developed. Considerable scatter in the field data is to be expected due principally to (1) actual non-uniformities in the beach and wave parameters and (2) difficulty in estimating the critical breaker parameters under field conditions.

It is concluded from Figure 6 that Equation (25) provides a good estimate of the uniform longshore current under both laboratory and field conditions.

APPLICATIONS OF THE THEORY

The rational prediction of littoral transport still awaits development of a suitable entrainment function for the surf-zone fluid motion and the "marrying" of this function to a mean flow equation such as Equation (22).

Meanwhile, however, Equation (22) can be usefully employed to estimate, among other things, the size of laboratory basin necessary in order to obtain a fully developed longshore current for given wave and beach parameters. If we require that the laboratory longshore current be at least 95% of the fully developed value, Equation (22) reduces to

$$B_x = 2.32 \quad \text{for 95% development} \quad (35)$$

This means that a model which is built to study a prototype shore process in which a fully developed longshore current plays an important role should have a beach which extends at least $2.32/B$ feet "upstream" of the area of interest. For the typical laboratory scale variables,

$$h_b = 0.2 \text{ ft.}$$
$$k_x = 1 \times 10^{-3} \text{ ft.}$$
$$\cos \alpha = 1$$
$$\theta_b = 20^\circ$$

Equation (29) gives $B = 0.14$ feet, and the added model length becomes at least $x = \frac{2.32}{B} = 16$ ft.
Figure 6. The Uniform Longshore Current
1. An expression is derived for the mean local longshore current velocity in the surf-zone on a plane, impermeable beach. This expression is in terms of readily obtained beach and wave parameters and a resistance coefficient for hydraulically rough surfaces.

2. When the resistance coefficient is calculated using an estimate of the absolute surface roughness in conjunction with the Kármán-Prandtl resistance equation for steady, uniform flow in rough pipes, the validity of the derived relationship is satisfactorily demonstrated.

3. Verification of the theory is obtained for the non-uniform developing current downstream of a surf-zone barrier through comparison with laboratory experiments. Verification for the uniform case is obtained using both laboratory and field data.

4. The derived expression appears to have present utility in the design of coastal model tests and to provide a necessary component for the future understanding of longshore sediment transport.

ACKNOWLEDGEMENTS

Financial support for this work was provided by the Coastal Engineering Research Center of the Department of the Army, Corps of Engineers under Contract No. DA-49-005-CIV-ENG-62-9. Mr. Dara Zargar assisted with the data reduction.

REFERENCES


